International Panel of Experts

Safety of Nenskra Hydropower Project - Georgia

STAGE II REPORT - Part 2 & Final

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Signed: IPOE Chair ……..……..

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Executive Summary

An International Panel of Experts (IPOE) in the fields of hydropower and dams has been tasked with assessing the Nenskra hydropower project against "Good International Practice" relating to all matters of dam safety and the safe design and construction and efficient operation and maintenance of the project components. The review over the past 12 months has been extensive and has delved independently into all the critical issues associated with the project to be satisfied that good practice has been utilised.

The IPOE has reviewed several iterations of the Engineering, Procurement and Construction (EPC) Contractor's Basic Design proposals with a focus on all the Dam and Project Safety aspects. Particular contribution has been made to the embankment; asphaltic concrete face; foundation seepage treatment; spillway; tunnels and natural hazards risk assessment.

The EPC Contractor completed its final Basic Design submission in December 2016 and this report contains the IPOE's final views on that design.

The IPOE supports the choice of Dam location; principles of the Asphalt Faced Rockfill Dam (AFRD) type design, including design features to ensure safety against extreme floods and extreme earthquakes; and valley floor foundation treatment with an 85m deep cut-off wall to limit seepage.

The proposed Tunnel Spillway approach is supported in preference to a surface Spillway. The IPOE recommends further consideration be given to the alignment of the Spillway tunnel to establish further separation between the downstream sections of the Spillway and Bottom Outlet tunnels. Such separation increases the independence of these two critical safety structures. In addition, the log boom requires further detailed design consideration to ensure spillway blockage risk is safely managed.

The Natural Hazard risk posed by a suspected landslide zone on the right bank above the reservoir has received particular attention from the EPC team. The IPOE accepts the analysis that this is not a major landslide risk and agrees that this zone does not pose a safety risk to the project. The IPOE recognises that design measures are proposed to adequately deal with the risks posed by avalanches and debris flows.

Some key Dam Safety issues remain to be addressed by the EPC team in the detailed design stage. They involve:

- further consideration of ground treatment for the soft lacustrine deposits encountered in the foundations at the upstream toe of the embankment where the cut-off is located. Such treatment must ensure safety against Dam instability and excessive deformation;
- necessary trial grouting in the abutments above the valley floor to demonstrate that the material is groutable and that the target low permeabilities can be achieved to limit seepage; if this is not the case, the foundation cut-off wall is likely to be extended into the abutments as well;
- further improvements to the Asphalt Face design and inspection gallery arrangements based on recommendations from the IPOE.

The revised Nakra Weir layout, which includes gates to control the flow through the Transfer Tunnel, provides safe control of floods and an appropriate arrangement to manage sediment, environmental flows and fish passage.

From an operating perspective, the IPOE has also stressed the importance of Emergency Preparedness Planning and Bottom Outlet operating rules to ensure public safety is assured.

The IPOE has a social specialist on the panel and the IPOE supports public disclosure of the ESIA package subject to addressing key IPOE recommendations including:

- JSC Nenskra and ESIA Consultants to include “open houses” in public engagement measures;
- JSC Nenskra and ESIA Consultants to include community safety amongst top subjects on the consultation agenda;
- EBRD to ensure consistency between compensation measures in the Nenskra LALRP and those in the Nenskra – Khudoni transmission line currently being considered by EBRD, which is an Associated Facility to the Nenskra project;
- JSC Nenskra to support local culture within the framework of the Community Investment Plan that is currently under preparation.

In conclusion, the IPOE considers that the final Basic Design submitted by the EPC Contractor in December 2016 meets international good practice leading into the detailed design phase of the project into which the IPOE has contributed a number of recommendations.

1. Introduction

JSC Nenskra Hydro, the company developing the Nenskra Hydropower Project (HPP) in Georgia, has established an International Panel of Experts (IPOE) to:

- **Review the documentation for the development of the project against "Good International Practice" relating to all matters of Dam safety and the safe design and construction and efficient operation and maintenance of the project components.**

A first report was prepared by the IPOE dated 21 May 2016.

For the first stage of the review the IPOE comprised the following experts:

Roger Gill (Chair)
Norihisa Matsumoto
Georg Schaeren

Unfortunately Mr Matsumoto was not available to continue with the IPOE after June 2016 and Mrs Ljiljana Spasic-Gril joined the Panel as a general dam and seismic specialist in September 2016. Subsequently the Panel's dam expertise has been enhanced with the inclusion in January 2017 of Mr Tomoyuki Tsukada who has specific Asphalt Faced Rockfill Dam (AFRD) expertise.

To link the technical work of the Panel with the Project's environment and social assessments the Panel's expertise was broadened with the inclusion in November 2016 of a social specialist, Mr Frederic Giovannetti.

1.1. Scope of Stage II

There are three tasks being addressed by the IPOE in Stage II:

Task 1: Document review of the Basic Design - Dam structural and Seismology

The EPC Contractor and Designer submitted the initial Basic Design documents in July 2016. An alternative solution, to consider the matters raised by the review of the Owner's Engineer (OE) and the IPOE's May 2016 recommendations, was prepared by the Designer and submitted in the middle of September 2016 and further updated in December 2016 in the final Basic Design. The IPOE is tasked with commenting on the final Basic Design.

Task 2: Update of the previous IPOE recommendations

The IPOE issued its first Report in May 2016. This report included IPOE recommendations for the safe design and implementation of the Nenskra Project. These recommendations have been summarized in a list of actions and have been responded to by the EPC Contractor/Designer, the Client and the Owner's Engineer. The task of the IPOE is to review this list and final Basic Design and provide opinions on the adequacy of the EPC Contractor's response to address the IPOE recommendations regarding safety, design and construction risks and efficient operation and maintenance of the Project. The IPOE is requested to update its recommendations related to the Basic Design stage and, as appropriate, provide recommendations for the Detailed Design stage.

Task 3: Review of the Alternative Design and Natural Hazard Assessment

Under Task 3, the IPOE will in particular – but not limited to - review and comment on the relevance and appropriateness with regards to the Project safety and risk of the:

(i) assessments carried out by the EPC Contractor to date or planned to be carried out,

(ii) proposed risk mitigation measures,

(iii) Natural Hazard Assessment including suspected deep-seated landslide and rock mass collapse,

(iv) the risk of internal erosion of the dam foundation together with an optimal seepage value for the dam safety,

(v) the safety of the dam, and

(vi) any other matter in the following fields:

- Geology and Tunnelling:
- Operational Safety:
- Dam structural and Seismology:
- Floods and Public Safety.

1.2. Process

The IPOE has reviewed relevant documents prepared by the EPC team and Owner's Engineer subsequent to the IPOE May 2016 reporting process. In addition Mrs Spasic-Gril visited the Nenskra and Nakra sites on 22/23 September 2016.

Mr Gill and Mrs Spasic-Gril participated in a technical workshop in Tbilisi in 25th and 26th September 2016.

The IPOE received feedback from a design review workshop held in Lausanne in November 2016 that included the EPC team, Lenders and Lenders advisors, Client and Owner's Engineer. Outcomes are listed at Section 3.1.1.

The IPOE prepared a short status update in December 2016 pending the completion by the EPC design team of the final Basic Design documentation.

Mr Tsukada together with Mrs Spasic-Gril participated in a briefing by the EPC Designer in Milan on 25th January 2017.

Part 1 Report

The IPOE's Findings from its **Stage II - Part 1** report, dated 6 October 2016, are summarised in Section 3.1 of this report.

Part 2 Report

The IPOE's Findings related to Tasks 1, 2 & 3 are updated in this **Stage II - Part 2 & Final** report based on a review of the final Basic Design documents submitted by the EPC Contractor in late December 2016 and further clarifications obtained during the Milan meeting on 25th January 2017. The findings noted in this report represent the latest position of the IPOE and therefore supersede previous positions of the IPOE.

Documents Reviewed for this Stage II - Part 2 Report

Updated material was made available to the IPOE in December 2016 and over the period of the review the best possible use was made of the available information. In general this included:

- Updated Drawings of the Nenskra HPP final Basic Design, Salini/Lombardi;
- Slide Presentation by Lombardi, Lausanne, September 2016;
- Updated Lombardi Technical Reports submitted for the Final Basic Design (December 2016);
- Updated Owner's Engineer Reports submitted for the Basic Design.
- Slide Presentation by Lombardi, Milan, January 2017;

Specific reports are referenced as necessary in this final IPOE report.

1.3. Status of IPOE's May 2016 Recommendations

The IPOE made extensive recommendations regarding the Safety and Operation of the Nenskra HPP in its first report in May 2016. The EPC team's response to the IPOE's recommendations and further assessment by the IPOE of the EPC final Basic Design have resulted in an updated stance by the IPOE on the matters of Safety and Project Operations. These matters are discussed in detail in Section 3 of this report and new recommendations are listed at Section 5. The recommendations in this report update the earlier views of the IPOE.

2. Summary of Findings

1. The IPOE has reviewed the EPC team's final Basic Design proposal of December 2016 for the development of the Nenskra HPP and reviewed all the Dam Safety aspects. The IPOE has already endorsed many of the elements of the engineering design during the design development process over the past 12 months and contributed comments in particular to the embankment; asphaltic concrete face; foundation seepage treatment; spillway; tunnels and natural hazard risk assessment.

2. The IPOE supports the:

    a. choice of Dam location;

    b. principles of AFRD type design, including design features to ensure safety against extreme floods and extreme earthquakes;

    c. valley floor foundation treatment with an 85m deep cut-off wall to limit seepage.

3. The proposed Tunnel Spillway approach is supported in preference to a Surface Spillway. The IPOE recommends further consideration be given to the:

    a. alignment of the Spillway tunnel to establish further separation between the downstream sections of the Spillway and Bottom Outlet tunnels. Such separation increases the independence of these two critical safety structures;

    b. log boom detailed design to ensure spillway blockage risk is safely contained.

4. The Natural Hazard risk posed by a suspected major landslide zone on the right bank above the reservoir has received particular attention from the EPC team. The IPOE accepts the analysis that this is not a major landslide risk and agrees that this zone does not pose a safety risk to the Project.

5. The IPOE recognises that design measures are proposed to adequately deal with the risks posed by avalanches and rock debris flows.

6. Some key Dam Safety issues remain in the process of being addressed by the EPC team in the detailed design stage. They involve:

    a. further consideration of ground treatment for the soft lacustrine deposits encountered in the foundations at the upstream toe of the embankment where the cut-off is located. Such treatment must ensure safety against Dam instability and excessive deformation;

    b. necessary trial grouting in the abutments above the valley floor to demonstrate that the material is groutable and that the targeted low permeabilities can be achieved to limit seepage; if this is not the case, the foundation cut-off wall is likely to be extended into the abutments as well;

    c. further improvements to the Asphalt Face design and inspection gallery arrangements based on detailed recommendations from the IPOE.

7. The revised Nakra Weir layout, which includes gates to control the flow through the Transfer Tunnel, provides safe control of floods and an appropriate arrangement to manage sediment, environmental flows and fish passage.

8. From an operating perspective the IPOE has also commented on the importance of Emergency Preparedness Planning and Bottom Outlet operating rules to ensure public safety is assured.

9. The IPOE supports public disclosure of the ESIA package subject to addressing recommendations by the IPOE that include measures related to public engagement as noted at summary finding 43 below.

10. In conclusion, the IPOE considers that the final Basic Design submitted by the EPC Contractor in December 2016 meets international good practice leading into the detailed design phase of the project into which the IPOE has contributed a number of recommendations.

The following comments summarise the IPOE findings more specifically:

NATURAL HAZARDS

11. The EPC team have undertaken complementary detailed site assessments of the natural hazard risks in the Nenskra and Nakra valleys. This includes review of rock avalanches, potential landslides, debris flows and snow avalanche zones, instabilities of colluvial/alluvial fans within the reservoir and glacial lake burst risks. The IPOE accepts the analysis that the zone on the right bank above the reservoir is not a major landslide risk and agrees that this zone does not pose a safety risk to the project.

12. A risk register has been prepared to identify where preventative design measures will be required to mitigate potential natural hazard impacts on the Nenskra HPP structures. The IPOE endorses the need for such design measures and recognises that these will be developed in the project's detailed design phase.

13. Once all natural hazard risk mitigation actions are developed a Residual Risk register should be produced to go into the Emergency Preparedness Plan (EPP) and Operations and Maintenance (O&M) Plan.

GEOLOGICAL ASSESSMENT

14. The IPOE considers that sufficient geological investigation work has been carried out to enable sound conclusions to be made for the development of the final Basic Design. However, further investigation will be necessary to enable the Detailed Design to be completed. The IPOE has provided comment on the need in some cases for such additional investigation.

FLOOD ASSESSEMENT

15. The IPOE endorses the Nenskra Probable Maximum Flood (PMF) value set at 1,101m^3/s and notes this is a significant increase from the earlier Nenskra PMF value of 456m^3/s.

16. The IPOE note that the relationship between the Nenskra PMF and Nenskra 1:10,000 year flood is a factor of 3.67, which seems unusually high and might indicate that the floods for lower return periods are underestimated. The IPOE had recommended in its Part 1 report that further reviews be undertaken of the peak discharges for the lower return period floods. The 1 in 25 year flood is particularly important as it sets the parameters for diversion flood management and flood management during the early generation phase. In the Basic Design documents there has been no change to the statistically obtained flood peak discharges. As a result, the IPOE recommends that the EPC team undertakes a sensitivity analysis on the level of flood protection provided during diversion and early generation.

17. The possible climate change impacts on the Nenskra HPP have been suitably clarified by the EPC team. The IPOE notes that a conservative design PMF value, with a freeboard on the associated maximum reservoir level, helps to ensure the Project's resilience to cope with maximum hydrological events.

SEISMIC ASSESSMENT

18. Safety of the Dam in seismic conditions has been checked for an Operating Basis Earthquake (OBE), with a return period of 1 in 145 years and 1 in 475 years, and a Maximum Credible Earthquake (MCE), with a return period of 1 in 10,000 years. Selection of the design earthquakes is in line with recommended practice.

    Performance of the Dam to the design earthquakes has been checked using a pseudo-static and 2D and 3D dynamic modelling. Seven horizontal and vertical time histories have been applied in the dynamic analyses and is found to be satisfactory.

NENSKRA DAM SAFETY

19. The Dam axis of the AFRD is now settled in the EPC team's final Basic Design arrangement and is accepted by the IPOE. The IPOE reiterates its comment that the proposed Dam is (1) a very high AFRD and (2) has very deep alluvial, fluvio-glacial and glacial deposits in the river floor on an international scale. These key aspects of the Project have been at the forefront of the IPOE's considerations.

Foundation Seepage and Erosion Risk

20. The ground investigation confirmed that the maximum thickness of the soil deposits over the bedrock in the valley floor is up to 160m. In the Stage II - Part 1 report the IPOE recommended that the EPC Contractor drill complementary investigation boreholes in the foundations of the right abutment to confirm a conservative geological model has been used in the analysis. We understand that a borehole (BH-R-150-2) is being drilled in the right abutment to confirm the depth to the bedrock.

21. The Dam design includes a diaphragm cut-off wall below the upstream toe of the main Dam body in the valley floor and a grout curtain in the abutments to prevent excessive foundation seepage and the risk of internal foundation erosion. Based on the IPOE's recommendations, the EPC Designer has undertaken a seepage sensitivity analysis. As a result the diaphragm wall has been extended from an initial 60m depth to now become 85m deep, reaching the elevation of 1225masl and going a minimum 5m into the glacial deposits. The deepened cut-off wall limits the seepages to <200l/s, which complies with the Project requirements and minimises the risk of progressive suffusion.

22. Based on the IPOE's recommendations, the final Basic Design now includes a drainage layer over the footprint of the embankment; the drainage layer is 5m thick in the valley floor and 0.8m thick in the abutments; this drainage layer will ensure the assumption about the "dry embankment fill" can be supported;

23. The IPOE recommends undertaking a trial grouting in the abutments above the valley floor to demonstrate that the material is groutable and that the targeted low permeabilities can be achieved to limit seepage; if this is not the case, the foundation cut-off wall is likely to be extended into the abutments as well;

Soft Lacustrine deposits in the foundations below Zone 3A of the upstream embankment shoulder

24. Soft lacustrine deposits, up to 10m thick, have been encountered in the valley floor below Zone 3A of the upstream embankment shoulder. The artesian ground water table encountered in the deposits is +0.5m to –1m below the ground level. These soft deposits were originally envisaged to be either excavated and replaced or treated in-situ. Since the ground water level is high, it is most likely that the deposits will not be excavated but treated in-situ. Final design of the ground treatment is yet to be developed to ensure that the treated soft lacustrine deposits have similar stiffness and strength properties as the surrounding alluvial deposits.

Embankment

25. The proposed Nenskra Dam will be the highest AFRD developed to date. Careful attention to the details of the design and construction of the asphalt face, as well as the connected structures and the foundation, will be critical to ensure the safety of the structure over its operating life. The IPOE is comfortable that a suitable asphalt face design can be developed and implemented at Nenskra. The IPOE has provided detailed recommendations to guide the face design as the Project moves from the Basic Design stage into the Detailed Design phase.

26. The IPOE previously expressed its preference for an upstream slope of 1:1.7 to facilitate the construction of a high quality asphalt facing to increase confidence of the long-term effective performance of the Dam. The final Basic Design incorporates a slope of 1:1.6. The IPOE emphasises the importance of the use of highly specialized equipment and skilled and experienced resources to produce a high quality face and accepts the 1:1.6 slope only on this basis. As well, to facilitate any remedial works on the face over the life of the project the IPOE has previously recommended that the crest should not include a large upstream crest wall, which would inhibit ready access to the face. The crest arrangement proposed in the EPC team's final Basic Design with a 1m high removable upstream crest wall is endorsed subject to detailed design considerations noted in this report.

27. The final Basic Design now includes a 6m high wall constructed at the downstream side of the crest. Stability analysis of this wall has been presented in the final Basic Design report, as a part of the 2D and 3D stability analysis of the Dam under seismic loading (see point 29 below) and is found to be satisfactory.

28. **Safety against extreme floods -** the IPOE noted in its Part 1 report that the Dam's downstream slope stability should be checked for the Design Flood at 1433masl. It was recommended that it also be checked for the Probable Maximum Flood (PMF) at 1435masl. This has now been done and factors of safety obtained are satisfactory.

    The IPOE reviewed the Dam freeboard requirements and recommended that a minimum freeboard of 0.9m be allowed for in the case of the PMF. A 1m high parapet wall has now been incorporated at the upstream slope of the Dam crest. The road level at the crest can remain at 1435masl.

29. **Safety against earthquakes** – assessment was undertaken for OBE and MCE earthquakes, as defined in point 18 above.

    During an OBE earthquake, with 1 in 145 year return period (PGA of 0.10g), a factor of safety against sliding greater than unity has been obtained, which is satisfactory.

    3D dynamic analyses performed for the MCE, with a PGA of 0.65g, generated maximum horizontal and vertical displacements of the crest of approximately 1m and 0.44m respectively. It is considered that these displacements are acceptable in case on an MCE earthquake, when the water level in the reservoir is expected to be at least 5m below the Dam crest. Nevertheless, the displacements obtained in the 2D and 3D dynamic analyses indicate a strong effect of the narrow

valley shape on the seismic behaviour of the Dam.

Spillway

30. The IPOE supports the Tunnel Spillway concept, but suggests further consideration of the alignment of the tunnel to maintain independence of the Spillway from the Bottom Outlet at the downstream zone.

31. The design of the log boom must address the risk of passing semi-submerged log debris. Furthermore, the IPOE suggests consideration be given to installing a second, back-up log boom as a contingency measure.

32. Log debris retrieval and removal capability must be provided for long-term operations.

NAKRA WEIR

33. The IPOE endorses the EPC team's improved arrangements for the Nakra Weir to enhance its functionality regarding stilling apron maintainability, sediment management, fish passage and environmental flow control.

34. In particular, the IPOE notes that provision has been made for Transfer Tunnel flow control to assist in reducing inflows to the Nenskra valley in scenarios where the Nenskra reservoir is spilling.

TUNNELLING

35. The Transfer Tunnel now discharges into the northern end of the Nenskra Reservoir. The IPOE's recommendations have been taken into account concerning the alignment of the Transfer Tunnel between the northwards shifted Nakra intake and Nenskra outlet in terms of risks linked to the tectonized Alibeck-fault zone and mountain overburden. The final alignment allows for almost unchanged overburden conditions compared to the initial alignment.

36. The Headrace tunnel passes orthogonally through complex geological conditions. The IPOE reiterates its previous recommendation that preliminary hydrogeological observation and eventually monitoring (including natural springs) is undertaken. Borehole investigation being still outstanding, the IPOE recommends paying great attention to the section close to the Frontal Thrust where overburden and distance to the slope are minimal.

PENSTOCK AND POWER HOUSE

37. The IPOE notes that the Power House has been moved downstream from its initial location to avoid the risk of debris flow from the large catchment area above. It is also recognised that where the Penstock crosses from the ridge to the Power House it will be underground and not exposed to debris flow impact risk.

OPERATIONAL SAFETY

38. The IPOE endorses the proposal from the EPC Contractor that an Emergency Preparedness Plan (EPP) will be in place at least 1 year prior to impoundment for early generation.

39. The IPOE again notes the importance of undertaking a dam break analysis that must feed into the EPP. The IPOE recommends that the dam break analysis takes into account any impact on Enguri Dam as well as considering potential impact on the dams downstream of Enguri.

40. The IPOE notes that a project risk framework is being developed by the Contractor to assess the Project's residual risks once all the mitigation actions have been put in place. The IPOE supports

this approach and again reiterates the importance of the Project Owner reviewing the completed risk assessment closely during the detailed design and construction stages and prior to commissioning to ensure full compliance with the mitigation actions has been achieved.

41. The IPOE recommends that particular attention be paid to establishing Bottom Outlet operating rules and security arrangements to ensure that the potential for very high discharges does not impact on the safety of downstream settlements and infrastructures. A response for inadvertent Bottom Outlet operation should be included in the Emergency Preparedness Plan.

42. Monitoring of the Dam is essential and is part of the EPP and Operations and Maintenance (O&M) plan. An Instrumentation Plan should be prepared as a part of the Detailed Design and should provide proposed instrumentation layouts, sections, details and specifications. The plan should also provide frequency of reading and trigger values and should link to the EPP and O&M plan.

SOCIAL ASPECTS REVIEW

43. The IPOE supports public disclosure of the ESIA package subject to addressing some comments that have been communicated directly to the ESIA consultants. Key IPOE recommendations include:

    a. JSC Nenskra and ESIA Consultants to include "open houses" in public engagement measures to be conducted shortly on the ESIA, as these are more conducive, in the Georgian cultural context, to meaningful consultation;

    b. JSC Nenskra and ESIA Consultants to include community safety amongst top subjects on the consultation agenda as this has been a repeated community concern;

    c. EBRD to ensure consistency between compensation measures in the Nenskra LALRP and those in the Nenskra – Khudoni transmission line currently being considered by EBRD, which is an Associated Facility to the Nenskra project;

    D. JSC Nenskra to support local culture within the framework of the Community Investment Plan that is currently under preparation.

DETAILED RECOMMENDATIONS

44. The detailed recommendations from this Stage II – Part 2 report are listed at Section 5. The actions and changes in the Basic Design that have resulted from the IPOE's recommendations in its May 2106 report have now been accepted by the IPOE or new recommendations have been made in this Stage II Part 2 report.

3. General Discussion

3.1. Previous IPOE Findings and Updated Design Considerations

In the Stage II Part 1 report the IPOE recognised good progress on many of the matters raised in its first May 2016 report. Many issues were accepted and closed out including significantly that the:

- Proposed Dam alignment has been endorsed;
- Upstream slope of 1:1.6 is agreed with a 6m high wall on the downstream side of the crest;
- Updated PMF of 1,101 m^3/s is endorsed.

However, several key Dam Safety issues were recognised as needing further consideration, including:

a) further analysis of the risk of progressive suffusion around and downstream of the cut-off wall that could lead to high pore pressures at the downstream toe of the Dam;

b) seepage sensitivity assessment of the range of values for a foundation seepage envelope;

c) protection against overtopping of the embankment during a PMF by provision of 1m high parapet wall on the upstream side of the crest;

d) checking of displacements of the crest for earthquake conditions;

e) review of the Nenskra spillway options.

The IPOE also noted it was waiting to review the EPC team's updated Natural Hazards report following further detailed site inspection work carried out by the EPC team. These matters and others are discussed in the following sections.

3.1.1. Updated Design Considerations

On 10 & 11 November 2016 a design review meeting was held between the EPC Contractor and Designer, the Owner's Engineer, and the Lender's advisors together with JSC Nenskra staff. The IPOE was not at the meeting to retain its independence from the design decision-making process.

Key outcomes from the meeting included:

a) Alignment of the Transfer Tunnel (TT) is to be optimised in order to keep it as far as possible from the Alibeck fault. It will be excavated using a double shield TBM;

b) Alignment and construction of the Head Race Tunnel (HRT) is to be subject to further risk assessment by the EPC Contractor;

c) Additional work was proposed to more accurately determine the instability risk of a potential landslide area on the right bank above the reservoir. The key concern being the generation of waves that could overtop the Dam;

d) Further improvements were noted on the Nakra Weir design;

e) EPC Contractor agreed that an Emergency Preparedness Plan will be ready one year before the first impounding;

f) Agreement that Nenskra Dam cut-off wall will reach elevation 1225masl and grouting of both banks will reach bedrock.

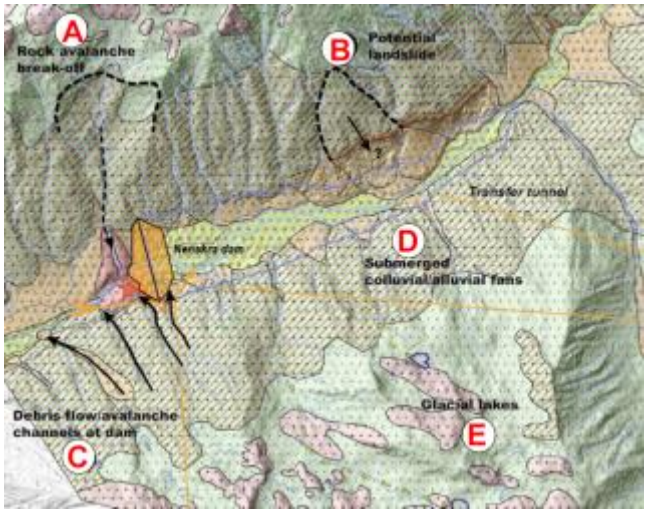
g) Adoption of a tunnel spillway, with the EPC Contractor to assess the log debris blockage risk for the tunnel spillway and develop appropriate mitigation.

3.2. Natural Hazards

The comments of this section are based on the Natural Hazards Risk Assessment Report dated 16 December 2016 (EPC Report L-6768-B-GL-GE-GE-TR-005_003). They also take into account the former versions of this report (version 000-18.07.16, revisions 001-04.10.16, 002-30.11.16) as well as the presentation for the 11.11.16 Workshop in Lausanne and the discussion held on 11.11.16 between the EPC and IPOE geologists.

It is noted that important investigation work has been undertaken since the IPOE workshop in April 2016. This included a helicopter survey of the upper parts of the slopes, detailed analysis and reinterpretation of field observation, analysis and interpretation of Radar Interferometry Data, especially in correlation with the Right Bank Potential Landslide (RBPL) - a major potential issue for the project that will be discussed further below. While the potential RBPL threat has been temporarily considered as a very relevant concern, the complementary information gathered since November 2016 and the re-interpretation of the local geology turn out to be favourable and the RBPL is no longer considered a high risk to the project.

After several updates of the technical report it appears that the various discussed natural hazards have been thoroughly addressed (avalanches, debris flows, rockfall, landslides, glacial lake outbursts). According to the whole documentation established by EPC and analysed by the IPOE there is no high risk identified, and furthermore the ones qualified as moderate can be reduced by design measures.



IPOE notes that attention is drawn to 5 zones/types of natural hazards within the extension from the Dam to the upper end of the reservoir, namely a "channelized rock avalanche" (**A**) which points to the alluvial fan immediately downstream of the Dam for which it is most likely responsible, the already mentioned Right Bank "Potential Landslide" (**B**) in the upper half of the reservoir, "Debris flow/avalanche channels" (**C**), "Submerged colluvial-alluvial fans" (**D**) and "Glacial Lakes" (**E**).

Concerning "Rock Avalanche (**A**)", the IPOE agrees with the EPC team's conclusion that it is low risk.

The latest update of the report, based on helicopter survey, re-interpretation of geological data and the Radar Interferometry Data, provides a detailed analysis of the Right Bank "Potential Landslide" (**B**). As a result, this potential landslide is no longer considered as a high risk for the Project. The IPOE notes that EPC's arguments are convincing and meet the IPOE's preliminary view on this subject: no pre-existing unfavourable structure exists, hypothetical unfavourable jointing

discontinuous and steeper than slope (Fig. 23 of technical report dated 16.12.16) and favourable geomechanical characteristics.

The IPOE also notes zone **C** where mention is made of "periodically downhill transported mixed moraine and slope debris". Considering the morphology of these materials the question of rock glaciers is raised, with consideration of the consequences of climate change with upslope migration of the permafrost limit the eventual increase of debris flow frequency can be postulated. The infrastructure protection measures proposed by the EPC team, particularly for the Spillway intake zone, are therefore critically important.

Concerning "submerged colluvial and alluvial fans" (**D**) the draw-down instability risk appears to be limited by the high permeability of this material that should easily support draw-down velocities up to say 10 m/day.

With respect to "Glacial Lakes" (**E**) and the connected potential GLOF (Glacial Lakes Outburst Floods), the IPOE draws attention to the fact that such floods would probably be accompanied by material transport (debris flows). The potential risk, however, is not higher than for the debris flows discussed earlier.

Recommendation Summary

a. The Natural Hazard risk posed by a suspected major landslide zone on the right bank above the reservoir has received particular attention from the EPC team. The IPOE accepts the analysis that this is not a major landslide risk and agrees that this zone does not pose a safety risk to the project.

b. The IPOE considers that the various discussed natural hazards have been thoroughly addressed (avalanches, debris flows, rockfall, landslides, glacial lake outbursts) and there is no high risk identified, and furthermore the ones qualified as moderate can be reduced by design measures.

3.3. Flood Assessment

The IPOE has reviewed the summary assessment of the Project's flood projections as described in "Hydrological Study – Technical Report" (EPC Report L-6768-B-HY-GE-GE-TR-001_003 dated 15.12.2016)

3.3.1. PMF as Design Flood

As per the IPOE's earlier recommendation, a review has been undertaken of the PMF assessment; the Nenskra PMF has been increased to 1,101 m^3/s (from previously estimated 456 m^3/s). This is in line with the expectations of the IPOE. The IPOE maintains that the PMF should be used as the design flood for the project, namely the spillway should be designed to evacuate the PMF and the embankment should have a minimum required freeboard against the PMF.

The IPOE notes that the EPC Designer has provided an upstream crest wall to ensure there is sufficient freeboard for the PMF, which is accepted by the IPOE.

1,000yr and 10,000yr floods

The EPC Designer has assessed the other statistically obtained values for floods at Nenskra as listed in Table 1 below;

	Peak discharge [m^3/s]		
Tr [years]	Dam Site	Nakra Intake	PH
2.33	99	87	122
5	121	106	148
10	138	121	170
25	160	140	197
100	193	169	237
200	209	183	257
500	230	202	283
1000	246	216	303
10'000	300	263	369

Table 1. Flood Peak Discharges

The IPOE notes that the relationship between the Nenskra PMF and Nenskra 1:10,000 year flood is a factor of 3.67, which seems unusually high and might indicate that the floods for lower return periods are underestimated. The IPOE had recommended in its Part 1 report that the Owner's Engineer further reviews the other flood peak discharges. The 1 in 25 year flood is particularly important as it sets the parameters for diversion flood management and flood management during the early generation phase. In the final Basic Design documents there has been no change to the statistically obtained floods. As a result, the IPOE recommends that the EPC team undertakes a sensitivity analysis on the level of flood protection provided during diversion and early generation taking into consideration the as planned progress of Dam construction.

3.3.2. Impact of Climate Change

The EPC team have included a commentary on the possible impact of climate change on the hydrology and flood management for the Project. The findings are summarised in the Hydrological Study. While there are large uncertainties the assessment suggests that during the period 2012-2050 a "very slight increase of total runoff of approximately +0.5%" is foreseen. While during the second half of the 21st century the situation could progressively head towards a reduction in available annual runoff of -9% by the year 2100. While there is increasing annual precipitation postulated for the period 2021-2050 this does not translate necessarily into a greater intensity of single storm events.

Since the IPOE is tasked with addressing project safety, it is noted that a conservative design PMF value, with a freeboard on the associated maximum reservoir level, helps to ensure the Project's resilience to cope with maximum hydrological events.

Recommendations

a. The IPOE recommends that the EPC team undertakes a sensitivity analysis on the level of flood protection provided during diversion and early generation taking into consideration the as planned progress of Dam construction.

b. The climate change impacts on the Nenskra HPP have been suitably clarified by the EPC team. The IPOE notes that a conservative design PMF value, with a freeboard on the associated maximum reservoir level, helps to ensure the Project's resilience to cope with maximum hydrological events.

3.4. Seismic Assessment

Safety of the Dam in seismic conditions has been checked for an Operating Basis Earthquake (OBE), with a return period of 1 in 145 years and 1 in 475 years, and a Maximum Credible Earthquake (MCE), with a return period of 1 in 10,000 years. Selection of the design earthquakes is in line with recommended practice stated in ICOLD bulletin 148.

Performance of the Dam to the design earthquakes has been checked using a pseudo-static and 2D and 3D dynamic modelling. Seven horizontal and vertical time histories have been applied in the dynamic analyses. Results are discussed in Section 3.6 below.

3.5. Asphalt Faced Rockfill Dam

3.5.1. Dam Axis

The upstream Dam axis has now settled in the EPC team's final Basic Design arrangement and the IPOE agrees with the recommendation bearing in mind geological conditions at the right abutment.

3.5.2. Foundation: Seepage and Erosion Risk

The Dam design includes a cut-off wall below the main Dam body in the valley floor and a grout curtain in the abutments to address foundation seepage and the risk of internal foundation erosion. Section 3.5.2.1 below addresses comments on the seepage modelling and the cut-off wall design, while Section 3.5.2.2 comments on the grout curtain proposed for the abutments.

3.5.2.1 Valley Floor

Geological model for the valley floor seepage analysis

A geological model adopted by the EPC Contractor for the seepage analysis in the valley floor is shown in Figure 1. below.

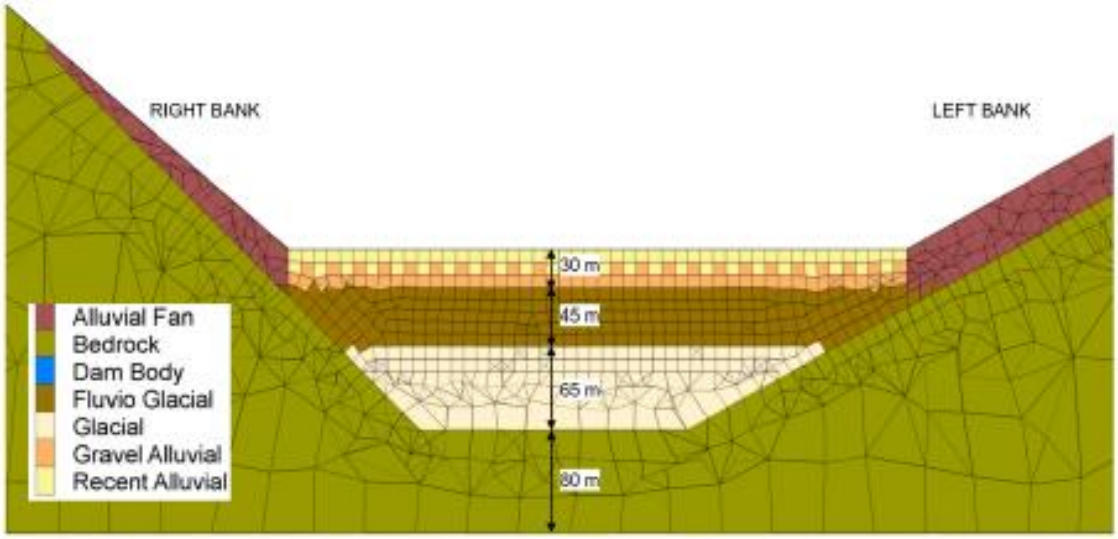


Figure 1 Soil Strata - Cross section at the valley floor

In the Stage II - Part 1 report the IPOE recommended that the EPC Contractor drill complementary investigation boreholes in the foundations of the right abutment to confirm a conservative geological model has been used in the analysis. We understand that BH-R-150-2 is being drilled in the right abutment; it has reached about 80m depth and is yet to confirm the depth to the bedrock.

A model of the embankment in the valley section, used in the seepage analysis by the EPC team, is shown on Figure 2. below.

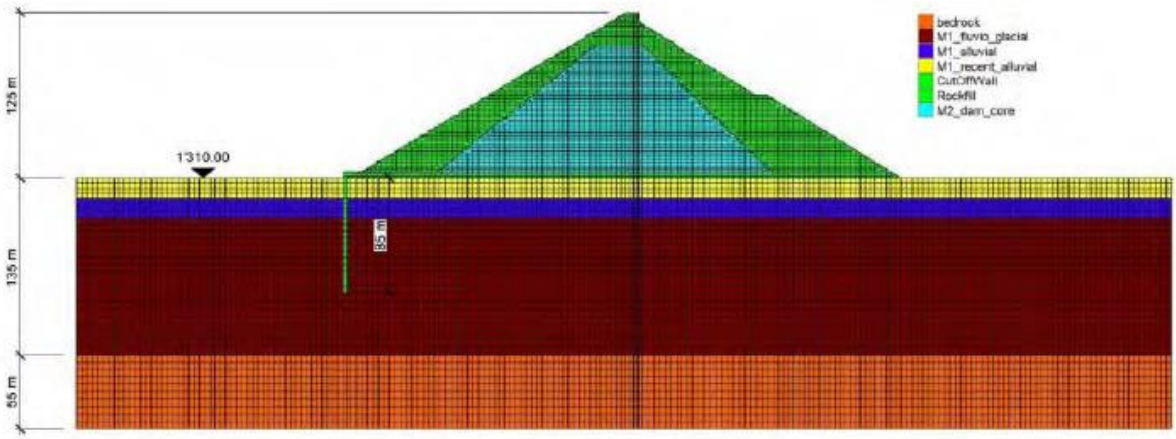


Figure 2 Dam Section

The Dam design includes a diaphragm cut-off wall below the upstream toe of the main Dam body in the valley floor and a grout curtain in the abutments to prevent excessive foundation seepage and the risk of internal foundation erosion. Based on the IPOE's recommendations, the EPC Designer has undertaken a seepage sensitivity analysis that resulted in an extension of the cut-off wall from its initial depth of 60m down to 85m reaching the elevation of 1225masl and going a minimum of 5m into the glacial deposits.

The embankment fill has been modelled as dry, which will be achieved by provision of a 5m thick drainage layer in the valley floor.

Permeabilities adopted in the model

Permeabilities adopted are as shown in Table 2 below.

Location	Material	Thickness (m)	K_H (m/s)	K_V (m/s)
Valley floor	Alluvial / Recent alluvial	30	$8*10^{-5}$	$K_H/5$
	Fluvioglacial	45	$1*10^{-4}$	$K_H/5$
	Glacial	60	$5*10^{-6}$	$K_H/5$
	Weathered bedrock	20	$2*10^{-6}$	$2*10^{-6}$
	Cut-off wall	60	$1*10^{-9}$	$1*10^{-9}$
Abutments	Alluvial fan / colluvium	40	$1*10^{-4}$	$K_H/5$
	Weathered bedrock	20	$2*10^{-6}$	$2*10^{-6}$
	Grout curtain	-	$1*10^{-7}$	$1*10^{-7}$

Table 2 Permeabilities

The above permeabilities have been adopted by the EPC Designer based on the following, measured data (Figure 3):

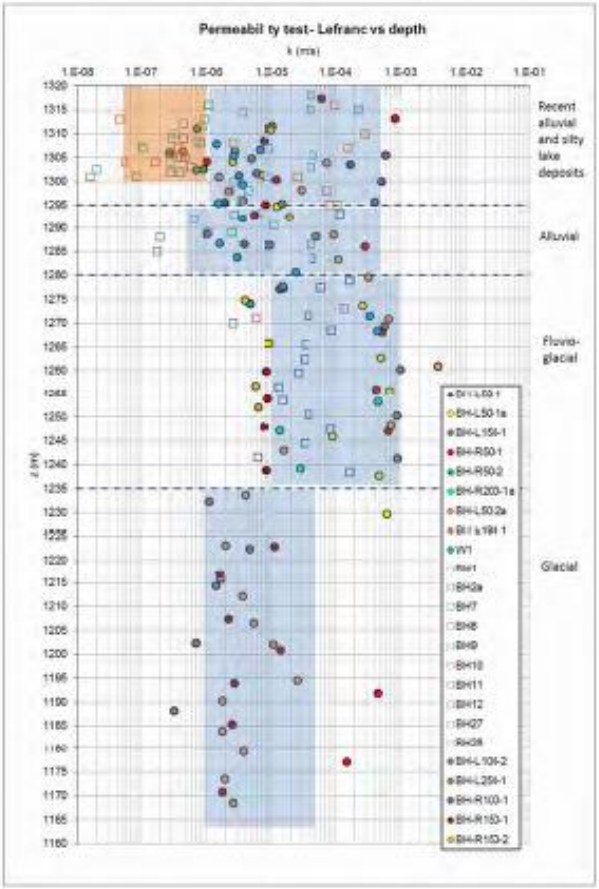


Figure 3 Permeability Data

Due to high variability in permeability in the alluvial and fluvio-glacial deposits, the IPOE recommended in its October 2016 report that the EPC Contractor undertake a sensitivity seepage analysis, i.e. vary the cut-off wall depth, permeability and ratio of K_h/K_v in the alluvial and fluvio-glacial deposits and their interface, in order to produce a seepage envelope that shows likely seepages vs cut-off wall depth for various scenarios.

This sensitivity analysis has been undertaken in the final Basic Design for the following scenarios:

Parameter		Reference values	Sensitivity range
Cut-off wall depth from average ground level elev. of 1'310 m asl		60 m (1310÷1250 m slm)	50, 60; 70; 80 and 85m
Permeability coefficient of the foundation layers	Recent alluvial/Alluvial	8.0E-5 m/s	6.0E-5 and 1.0E-5 m/s
	Fluvio-Glacial/Colluvium	1.0E-4 m/s	6.0E-5, 4.0E-4 and 8.0E-4 m/s
	Glacial	5.0E-6 m/s	1.0E-5 and 2.0E-6 m/s
Elevation of the interface between the Fluvio-Glacial and Glacial strata		1235 m asl	1225 m asl and 1245 m asl
kh/kv ratio		5	2 and 10

Results of the seepage sensitivity analysis are presented in Figure 4 below. It can be seen that for an 85m deep cut-off wall a seepage of 170 l/s is expected, which is acceptable and is within the specified requirements.

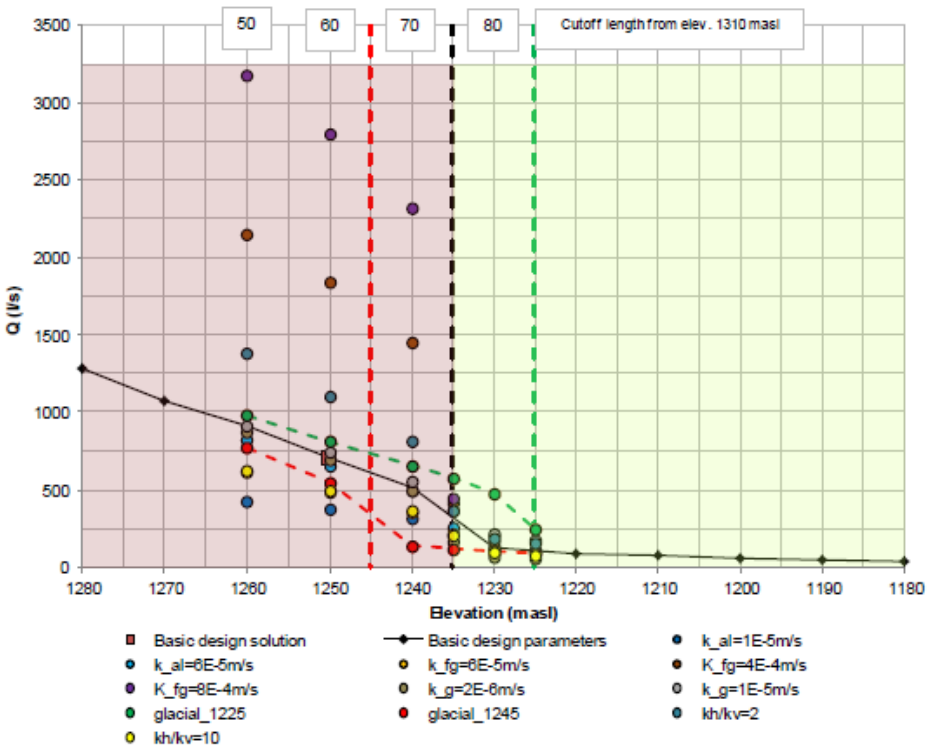


Figure 4 Seepage Sensitivity Results

The analysis has also shown that the maximum water table rise in the embankment would be 4m, which justifies a 5m thick drainage layer within the footprint of the embankment in the valley sections.

The IPOE recommends that the EPC Contractor demonstrate that the seepage gradients across the cut-off wall are acceptable; this should be included in the detailed design stage.

Internal Erosion

The EPC Contractor examined four possible types of internal erosion, as recommended by ICOLD Bulletin 164 on "Internal Erosion":

1) Concentrated leak, which could lead to development of a pipe;
2) Backward erosion, which could also lead to a pipe;
3) Contact erosion of finer soils into the coarser soils, which may develop a pipe;
4) Suffusion, where some finer fraction is eroded leaving the coarse matrix of soil. Typically, no pipe is formed, but the permeability of the soil may increase.

Upon the IPOE recommendation in its Stage II - Part 1 report, the EPC Contractor has undertaken an analysis to check the risk of progressive suffusion around and downstream of the cut-off wall that could lead to high pore pressures at the downstream toe of the Dam.

The analysis has shown that the high gradients identified at the bottom of the cut-off wall are unlikely to lead to suffusion, due to confinement of the particles. Some local migration of particles might occur, but a presence of a thick filter layer, that would be placed between the foundation soil and the 3A embankment fill, over 80m length, should mitigate the risk that might be caused from the upwards movement of soil particles. The IPOE agrees with the analysis and conclusions.

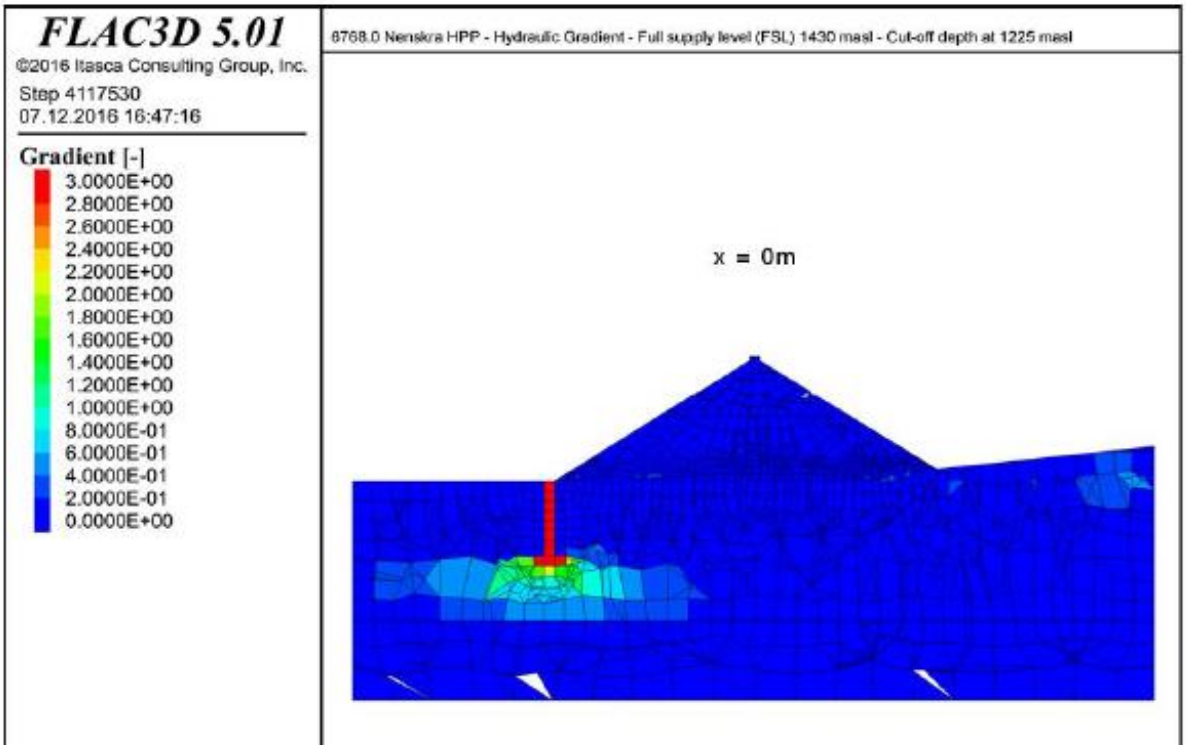


Figure 5 Hydraulic Gradient at the valley floor (cut-off wall depth at 1225masl)

3.5.2.2 Abutments

Geological model for the abutment seepage analysis

A geological model adopted for the seepage analysis in the abutments is shown in Figure 6 below.

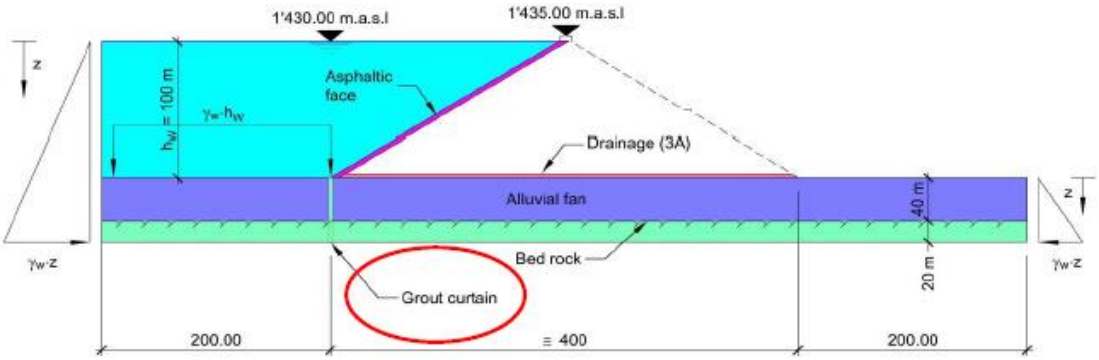


Figure 6 Seepage Model

Permeability in the alluvial fan layer of 10^{-4} m/s has been adopted in the abutment seepage analysis; a 40m deep grout curtain has been envisaged which will have a permeability of 10^{-7} m/s.

The IPOE recommends the EPC Contractor undertake a trial grouting in the abutments to demonstrate that the foundation material is groutable, and the targeted permeabilities can be achieved. If this is not the case, the cut-off wall is likely to extend into the abutments as well.

Recommendation Summary

a. The IPOE understands that drilling of borehole BH-R150-2, located on the alignment of the cut-off wall and which is still in progress, is planned to be driven into the bedrock, thus meeting the IPOE's recommendation from its Stage II – Part 1 report.

b. With regards to the depth of the diaphragm cut-off wall, the seepage gradients and any potential for progressive suffusion: the EPC Consultant has undertaken a seepage sensitivity analysis and based on that extended the diaphragm wall to 85m, reaching the elevation of 1225masl. The deepened cut-off wall would be in the glacial deposits for a few meters; this will limit the seepages to <200 l/s and minimize the risk of progressive suffusion. The IPOE is in agreement with the proposed deeper cut-off wall.

c. The EPC Consultant has, in its final Basic Design documents of December 2016, proposed a 3A drainage layer over the footprint of the embankment; the drainage layer is 5m thick in the valley floor and 0.8m thick in the abutments. The drainage layer will ensure that any water table rise is contained within the drain and the embankment fill remains dry. This is in line with the IPOE's previous recommendations.

d. The EPC Contractor must undertake a trial grouting in the abutments to demonstrate that the foundation material in the abutments is groutable and the targeted permeabilities can be achieved. If this is not the case, the cut-off wall is likely to extend into the abutments as well.

3.5.3. Soft Lacustrine Deposits in the Foundations

The design documents submitted in December 2016 show that soft lacustrine deposits, up to 10m thick, have recently been encountered in the valley floor, below the Zone 3A of the upstream embankment shoulder (see Figure 7 below). The artesian ground water table encountered in the deposits is +0.5m to –1m below the ground level.

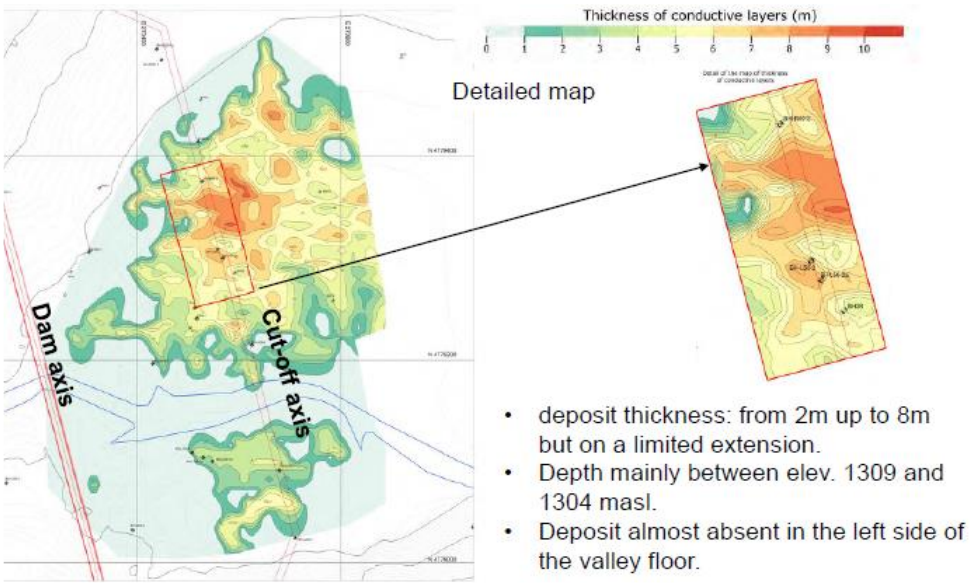


Figure 7 Location of Lacustrine Deposits

These soft deposits were originally envisaged to be either excavated and replaced or treated in-situ. Since the ground water level is high, it is most likely that the deposits will not be excavated, but treated in-situ.

The IPOE notes that the final design of the ground treatment is yet to be developed to ensure that the treated soft lacustrine deposit zone has similar stiffness and strength properties to the surrounding alluvial deposits.

3.5.4. Embankment

Upstream slope and crest arrangement

The IPOE previously expressed its preference for an upstream slope of 1:1.7 to facilitate the construction of a high quality asphalt facing to increase confidence in the long-term performance of the Dam. The final Basic Design incorporates a slope of 1:1.6. The IPOE emphasises the importance of the use of highly specialised equipment and skilled and experienced resources to produce a high quality face and accepts the 1:1.6 slope only on this basis. As well, to facilitate any remedial works on the face over the life of the project the IPOE requires that the crest should not include a large upstream crest wall, which would inhibit ready access to the face. The crest arrangement proposed in the EPC final Basic Design shown in the Figure below is endorsed given its adjustment to the freeboard and seismic assessment matters noted below.

As shown on Figure 8 below, the final Basic Design now includes a 6m high wall constructed at the downstream side of the crest and a 1m high upstream parapet wall.

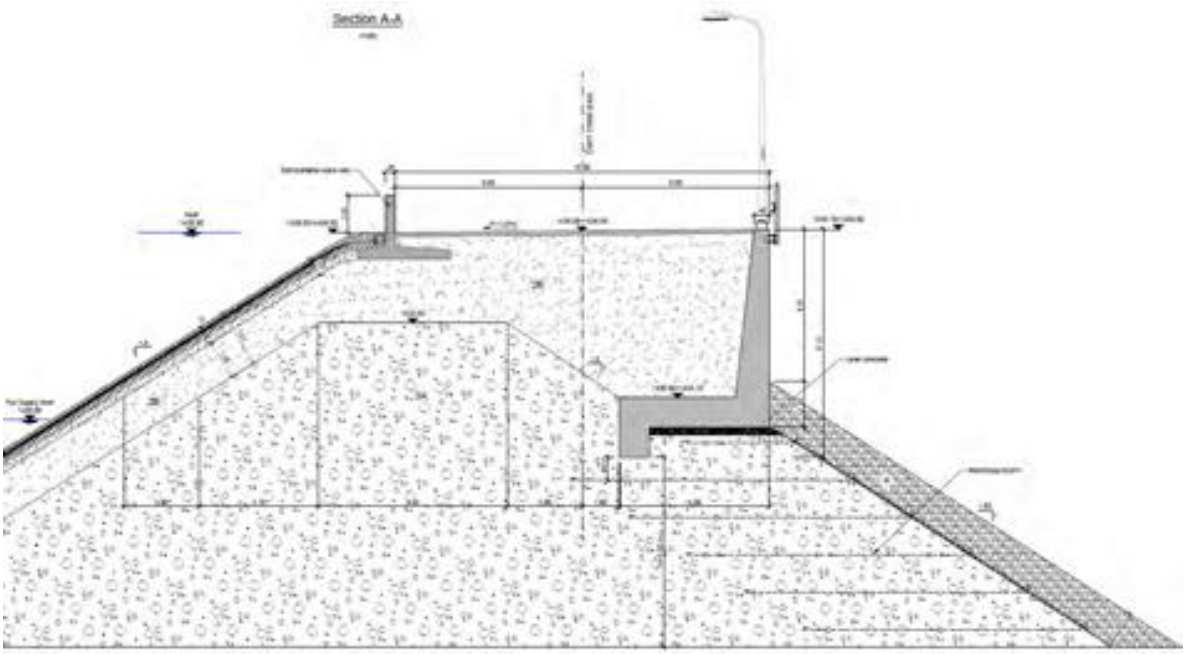


Figure 8 Crest Detail

Slope Stability Static (aseismic) Analysis

Stability of the upstream and downstream slopes in static (aseismic) conditions has been checked in accordance with the USBR Guidelines for dams, namely:

- Usual Condition, the reservoir water level at FSL and no seismic loading,
- Unusual Condition, rapid draw down from FSL to Minimum Water Level and no seismic loading, or increased pore pressures in the foundations and no seismic loading;
- Extreme Condition, maximum water level with no seismic loading.

The IPOE noted in its Part 1 report that, for the Extreme Condition, the slope stability was checked for the maximum water level at 1433masl. It was recommended that the stability of the downstream slope be also checked for the Probable Maximum Flood (PMF) at 1435masl. This has now been done and factors of safety obtained are satisfactory.

Seismic Analysis

This was undertaken for OBE and MCE earthquakes, as defined in Section 3.4 above. As per the USBR guidelines and ICOLD bulletin 148, the seismic condition is an Extreme Loading Condition when the seismic loading is combined with a reservoir water level at FSL; it is required that:

- for an OBE there should be no or insignificant damage to the Dam and the appurtenant structures;
- for an MCE damage can be accepted, but there will be no uncontrolled release of water from the reservoir.

During an OBE earthquake, with 1 in 145 year return period (PGA of 0.10g), a factor of safety against sliding greater than unity has been obtained, which is satisfactory and meets the safety requirements.

2D and 3D dynamic analyses were performed for the MCE, with a PGA of 0.65g. Laboratory tests for reconstituted specimens were made at ISMGEO and centrifuge tests were also performed.

The 2D analysis gave permanent deformation of 0.45m and 0.25m horizontally and vertically, respectively. The 3D analysis generated maximum horizontal and vertical displacements of the crest of approximately 1m and 0.44m, respectively. It is considered that these displacements are acceptable for an MCE earthquake when the water level in the reservoir is expected to be at least 5m below the Dam crest. Nevertheless, the displacements obtained in the 2D and 3D dynamic analyses indicate a strong effect of the narrow valley shape on the seismic behaviour of the Dam.

It is also noted that the input acceleration response spectrum (0.1-0.2 sec) for the seismic analysis is different from the predominant period (0.7-0.9 sec) of the Nenskra Dam. The IPOE recommends a study of the acceleration response spectra of earthquake records around the Dam site to confirm the validity of the period characteristics of the input acceleration response spectrum used for the analysis. This could be performed during Detailed Design stage, as the currently generated displacements are considered to be on the conservative side.

Freeboard allowance

The IPOE reviewed the Dam freeboard requirements and recommended that a minimum freeboard of 0.9m be allowed for in the case of the PMF. A 1m high parapet wall has now been incorporated at the upstream slope of the Dam crest. The wall could be removable in case repairs to the face are necessary; the need for it to be removed could be decided depending on the equipment and the accessibility needed at that time. The road level at the crest can remain at 1435masl.

With the parapet wall added to the Dam crest, the freeboard added to the FSL is 6m and to the design flood at 1433masl is 3m. The freeboard will be sufficient to accommodate combined flood inflows and wind wave action as well as potential waves triggered by debris flows.

3.5.5. Asphalt Facing

The proposed Nenskra Dam will be the highest AFRD developed to date. Careful attention to the details of the design and construction of the asphalt face, as well as the connected structures and the foundation, will be critical to ensure the safety of the structure over its operating life. The IPOE is comfortable that a suitable asphalt face design can be developed and implemented at Nenskra. The following comments are provided to guide the face design as the project moves from the completion of the Basic Design stage into the Detailed Design phase.

Thickness of the asphalt face

As previously noted by the IPOE, Nenskra Dam is a very high AFRD, it will be subjected to large hydrostatic pressure and further consideration needs to be given to the appropriate thickness of the face. From past records of dam construction it can be noted that the thickness of the asphalt face increased as the height of the dam as well as the maximum water pressure. However, the thickness of the proposed Basic Design is uniformly 31 cm. Figure 9 below shows the thickness of the asphalt face vs the height of the current AFRDs.

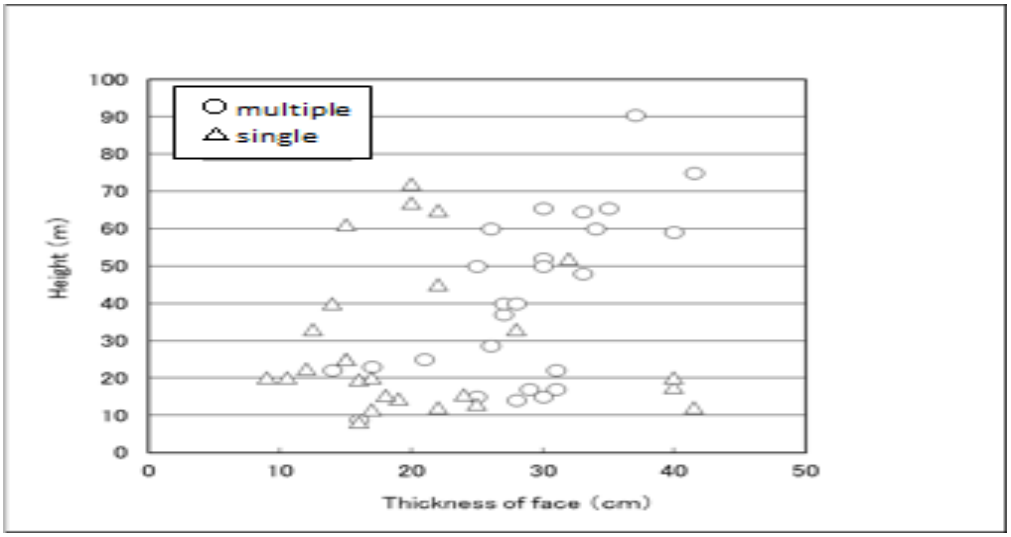


Figure 9 Height of AFRD and Thickness of Asphalt Face

The IPOE recommends that further consideration be given to the appropriate thickness of the asphalt face, which shall be determined by taking into consideration properties obtained from laboratory and field tests and the required performance.

As for the required thickness of the drainage layer, for example, it can be estimated as follows: Under the assumption of a permeability coefficient of 1×10^{-10} m/sec for the impermeable layer and 1×10^{-4} m/sec for the drainage layer, about 30cm of thickness of the intermediate drainage layer is required at the bottom of the asphalt face in order to secure sufficient drainage capacity to safely drain all water seepage. If the asphalt face is designed as currently proposed, the permeability coefficient of the intermediate drainage layer should be designed and constructed to be about 3.8×10^{-4} m/sec or more.

(current design)

$q=ki*h/ti*\triangle L$ (m^3/sec/m/m)

$\triangle L=dh*\sqrt{(1^2+(1.6)^2)}$

$Q=1.89*\int q\cdot dh$ (m^3/sec/m)

$=1.89*1/2*ki*h^2/ti$ (h=0～125m)

1.84E-05 m^3/sec

0.018 l/sec

velocity in the drainage layer

$vd=kd*i$ (i=1/1.6)

6.25E-05 m/sec

neccesary thickness of drainage layer

td'=Q/vd

0.29 m > td=0.08m

q: flow per unit length per unit depth length

Q: flow per unit length

ki: permeable coefficient of impermeable layer= 1.0E-10 m/sec

ti: thickness ofupper impermeable layer =8cm

$\sqrt{(1^2+(1.6)^2)}=$ 1.89

L: slope length(m)

h: water depth from base of the gallery

kd:permeable coefficient of drainage layer= 1.0E-04 m/sec

td: thickness of drainage layer =8cm

i: hydraulic gradient=gradient of slope 1:1.6

Where there is a concern about cracking of the upper impermeable layer due to earthquakes, leakage water flows from potential cracks should also be taken into account.

Seismic performance

The amount of water leakage due to face cracking under earthquake loading should be estimated and the adequacy of the permeability and thickness of the intermediate drainage layer should be checked.

As for zones with large water depth, such as the inspection gallery and particularly its block joints, it will be necessary to carefully evaluate the analytical value of the strain.
The safety of the cut-off wall in case of earthquake has not been checked at this stage. In case of a breakdown of the cut-off wall, leakage may rapidly increase and cause hydro fracturing and large strain of the asphalt face at the connecting part with the inspection gallery due to large displacement of its foundation. It is anticipated that the EPC Designer will carry out such a safety assessment during the Detailed Design stage. After deciding the composition of the material for the cut-off wall, it is necessary to capture its physical properties, such as the elastic modulus and strength of the material, and re-analyse to confirm its safety.

In the current analysis conducted by the EPC Designer, the hydrodynamic effect caused by an earthquake is not considered. There is, however, a probability of larger strain of the asphalt face under the water affected by the hydrodynamic pressure. Thus, the IPOE recommends checking this effect by using added mass as the hydrodynamic pressure, if possible. The added mass can be calculated, for example, by Zanger‘s formula, as shown in Figure 10 below.

$$p_d = C\, w_o\, \alpha\, h \qquad (1)$$

$$C = \frac{C_m}{2}\left[\frac{y}{h}\left(2-\frac{y}{h}\right)+\sqrt{\frac{y}{h}\left(2-\frac{y}{h}\right)}\right] \qquad (2)$$

p_d : hydrodynamic pressure (tf/m²)
α : pseudod-static coefficient
h : water depth(m)
y : distance from water surface (m)
C : pressure coefficient
C_m : maximum value of C (see right Figure)
w_o : unit volume weight of water (tf/m³)

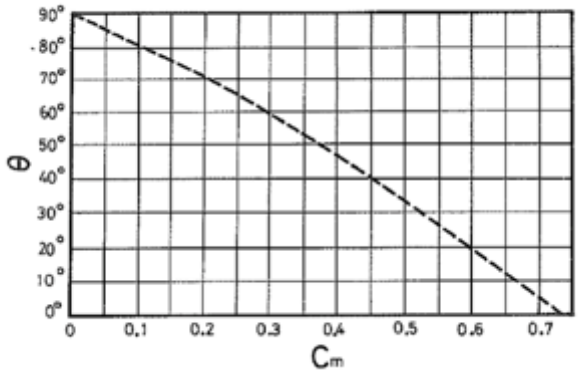


Figure 10 Zanger’s Formula

Maximum allowable strain of asphalt concrete

When designing asphalt concrete, it is necessary to consider conditions of temperature and strain rate, since the mechanical characteristics of the asphalt mixture vary in accordance with temperature and strain rate. Also, the failure strain of the material of the impermeable layer, which is made with fine-grained asphalt concrete, should be checked under each condition. The lower the temperature decreases and the higher the strain rate becomes, the lower the failure strain of asphalt concrete becomes.

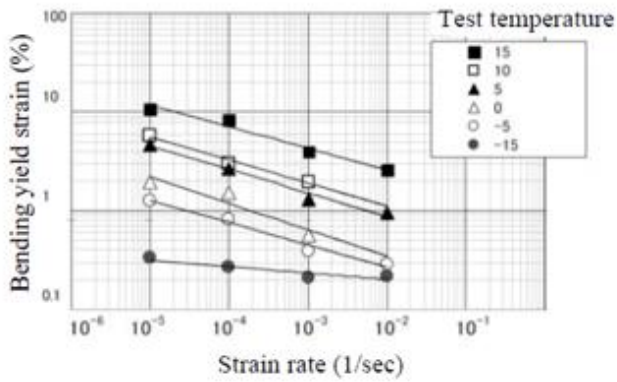


Figure 11 Relationship between Bending Yield Strain and Strain Rate of Yashio Dam

The EPC Designer conducted an evaluation of the safety of the asphalt face of Nenskra Dam. In the evaluation, maximum allowable tensile strain was set as 2 % for FSL at the condition of 0 degree Celsius (℃), and 1% for minimum operating level (MOL) at the condition of -5 ℃, during the earthquake analysis.

On the other hand, maximum allowable tensile strains for similar dams in Japan were set based on bending tests and/or indirect tests as follows:

- Yashio Dam (1994): 0.2% at -15 ℃, 1.0% at 5 ℃
- Ooseuchi Dam (2007): 0.033 to 0.042 % at -10 ℃
- Kyogoku Upper Reservoir (2014): 0.037% at -20 ℃, 0.09% at 0 ℃

All cases were under the condition of strain rate of 10^{-2} 1/sec. In comparison with similar dams in Japan, the current allowable maximum tensile strain of Nenskra Dam face seems too large.

Failure tensile strain of the Kyogoku upper reservoir and Ooseuchi Dam is smaller than the one of Yashio Dam. The asphalt content of fine-grained asphalt concrete of Kyogoku Upper Reservoir and Ooseuchi Dam were 7.4 %, and 7.7 %, those were smaller than 8.5% of Yashio Dam as shown in Table 3 below.

	maximum aggregate size(mm)	composition of fine grained asphalt concrete (kg/ton)						
		asphalt	Aggregate		Crushed sand	Fine sand	Filler	
			13-5mm	5-2.5mm	2.5-0mm	2.5-0mm	stone powder	additive
Yashio	13	85	166	267	276	83	115	8
Ooseuchi	13	77	842				79	2
Kyogoku		74	792				132	2

Table 3 Asphalt Content of Japanese AFRDs

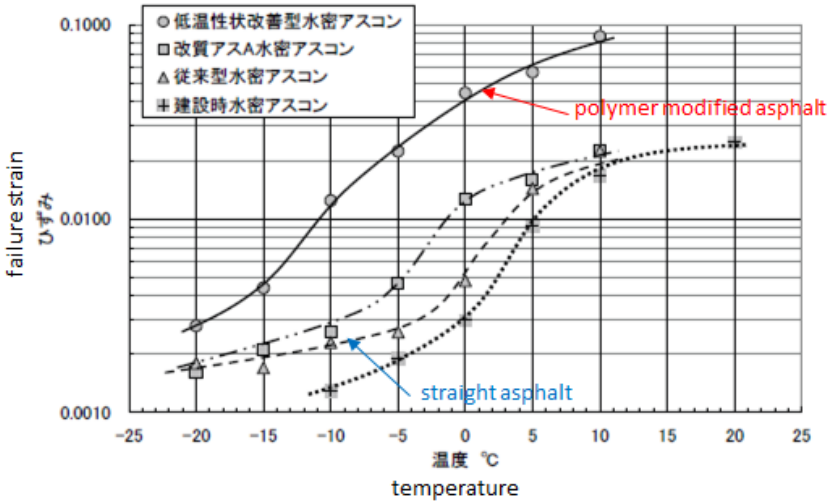
Since the slope gradient of the upstream face of Nenskra Dam is relatively steep at 1:1.6, it is conceivable to reduce the asphalt content for the fine-grained asphalt concrete for the impermeable layer in order to suppress asphalt flowing on the slope. As a result the failure strain value may decrease.

According to the EPC Designer's presentation in Milan on 25 January 2017, the mixture design of fine grained asphalt concrete of Nenskra Dam was tentatively set as 7.3% of asphalt content, which is a smaller asphalt content than for similar dams in Japan.

In consideration of the conditions mentioned above, it seems difficult to ensure the allowable maximum strain of 1% at -5 ℃ and 2% at 0 ℃ while using the same material as these dams. Therefore, it may be necessary to use special material such as polymer modified asphalt that was developed for the purpose of improving deformation performance under low temperatures. This material also has sufficient resistance against flow under high temperatures. The slope flow value of fine-grained asphalt concrete using this material was about one third of straight asphalt[1]. Also, this material had

[1] Nakamura,Y., Ohne,Y., Narita,K., Okumura, T., Nomura, K., Shimazaki, M. and Mizuno, T., Earthquake damages and remedial works for an earth dam with asphalt facing, ICOLD 75th. Annual meeting symposium, 2008

about three times larger failure strain than that of fine grained asphalt concrete using straight asphalt as shown in Figure 12 below. Follow-up surveys were conducted around five years after the repair works of an asphalt faced earth dam damaged by cracking in East-Yamanashi earthquake (M5.8) in Japan. The results revealed that repaired asphalt concrete had remained in a satisfactory condition without any sign of deterioration by ageing.[2]



Shimazaki, M., Tsunoo, T., Kasahara, A., Application of low temperature properties improvement asphalt to repair work of rock fill dam with asphalt facing, Journal of Japan Society of Civil Engineers, Ser. E1 (Pavement Engineering), Vol. 67 (2011) No. 1 P, (in Japanese)

Figure 12 Polymer modified asphalt

Durability of Asphalt Face

An investigation of the asphalt face of Yashio Dam was carried out in 2011. The results of boring and sampling in the investigation show that there is no deterioration in any face layers even in the surface impermeable layer. It indicates that deterioration of the asphalt face by aging may not occur even 20 years after construction as long as the protection layer is healthy. In fact, the protection layer of Yashio Dam has not been re-painted.

It is also important to take into account the resistance of the asphalt face against fatigue failure. The IPOE recommends the EPC Designer confirm safety against fatigue failure from earthquake loading through cyclic loading tests.

Concentration of strain at the joints between the asphalt face and concrete structures

Yashio Dam was damaged by the extreme Tohoku Earthquake in Japan in 2011. Strain concentration at the crest concrete block joints was observed. Cracks occurred along the block joints on the asphalt face in a direction at right angles to the dam axis. Thus alleviation of the strain concentration at the joints should be taken into account in the Detailed Design stage at Nenskra.

[2] Mizuno, T. & Shimazaki, M., Nakamura,Y., Ohne,Y., Narita,K., Okumura, T., and, Performance of Highly Ductile Modified Asphalt for Use in Impervious Facing Zone, ICOLD 80th. Annual meeting symposium, 2012

Inspection galleries of not only Yashio Dam, but also of most other existing AFRDs in Japan have been installed on the bedrock. Therefore, gallery block displacements will have been relatively small and no significant strain can be assumed at the block joints of the inspection gallery concrete.

On the other hand, the inspection gallery of Nenskra Dam will be installed on an alluvial deposit. Thus, the IPOE recommends the EPC Designer evaluate the strain at the block joints of the inspection gallery concrete. For estimation of the strain concentration, it is normally assumed that the gallery concrete is a rigid body, and axial displacement of a concrete block is interpreted as a joint's displacement.

The strain concentration in the crest concrete is estimated in the same manner. Once earthquake induced cracks occur in the crest concrete along the direction of the block joints the cracks may extend downward along the slope of the face. It may cause a leakage and result in repair work that is more difficult than for leaks caused by cracks in a dam axial direction.

In the case that analysis results show the strain exceeds the failure strain (maximum allowable strain of the face material), countermeasures should be taken. It is necessary to make the structure of the joint of the upstream crest concrete and the asphalt face less strain concentrated. It should be assessed in the detailed design stage how large a strain is acceptable. In the case of Yashio Dam, a detailed study was conducted on reinforcement work for the asphalt face[3]. As a result of the study, polymer modified asphalt, developed to improve deformation performance under low temperatures[1], was used for the reinforcement work.

The design concept of reinforcement work for the Yashio Dam is as follows: The reinforcement work was designed by using material that has an excellent elongation so that the strain would not be transferred from the joint opening to the asphalt facing. The asphalt mastic used was 10cm in width for overall facing thickness. The property of the asphalt mastic was confirmed by bending tests. The failure tensile strain of the asphalt mastic is more than 50%. Finite element analysis was conducted which confirmed the tensile strain of the asphalt facing due to the assumed joint opening is small in comparison with the failure strain. A copper plate was set beneath the asphalt mastic not to transfer the stress and the strain from the concrete block joint. Furthermore, the facing in the surrounding areas near concrete block joints were re-paved with the asphalt concrete whose composition was modified to have larger elasticity using polymer modified asphalt.

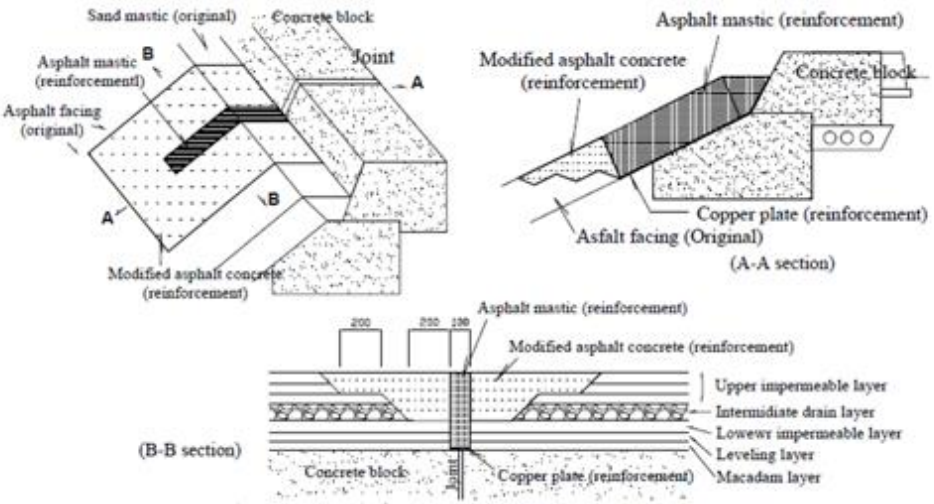


Figure 13 Reinforcement at the dam crest area of Yashio Dam

[3] Tsukada, T., Yamamoto, H., Shimada, Y., Uchita, Y. and Takasawa, K., Study on behavior of AFRD during earthquake and conducted reinforcement, Proceedings ICOLD 2013 International Symposium, 2013

Method for the asphalt face construction

Differential settlement/displacement of the asphalt face is a crucial issue for AFRDs. The IPOE recommends that the base for the foundation of the asphalt face be well compacted horizontally during embankment construction and compacted in the slope direction with a roller pulled from the crest just before paving in order to avoid differential settlement. The foundation of the inspection gallery should also be consolidated to avoid any deformation which causes large strain exceeding the allowable maximum strain of the asphalt face.

According to the reports and drawings of Nenskra Dam, a tack coat/bonding layer is applied between layers. According to Japanese Civil Engineering Society, however, such bonding layer or tack coat is not required because close connection can be attained without it. When the upper layer is paved, the lower existing layer is automatically heated by the upper paving layer. In case the amount of heat is insufficient, a gas burner or other devices for heating can be used. Such additional heat can attain the necessary close connection between the layers.

On the other hand, it is a concern that the tack coat may cause weakness between layers, such as slips or sliding, and also cause blistering which is a phenomenon of swelling by steam pressure of trapped gases. Thus, the IPOE recommends that the use of a tack coat be re-assessed by testing the shear strength of the contact between the layers with and without a tack coat.

If the upper impermeable layer is to be paved with a thickness of 8 cm, a powerful asphalt finisher should be used. In Japan, a thick pavement layer was adopted in the construction of Ooseuchi Dam and Kyogoku upper reservoir, however, it was limited to the flat bottom area in each construction. The thickness was 10cm for Ooseuchi Dam and 8cm for Kyogoku upper reservoir. In addition, the thick layer may lead to increasing risk of asphalt flowing on the slope, so the impermeable layer must have both large flexibility and small flowability.

The EPC Contractor has designed a curved shape for the connection part of the asphalt face between the inspection gallery and the asphalt face. According to the EPC Designer, that design is necessary to construct the cut-off wall work and embankment work in parallel so that construction schedule can be shortened.

Even though the IPOE understands the EPC Designer's intention, the IPOE has a concern that it may be difficult to construct the paving of the curved asphalt face as designed using an asphalt finisher. Thus, the IPOE recommends that the EPC Designer study the possibility of application of the shape of the connection part as shown in Figure 15 below.

Regarding design of the joint part of the asphalt face and inspection gallery concrete, it seems difficult to pave the layers with asphalt finishers, since the thickness of each layer of the asphalt face is changing from place to place as shown in Figure 14. Therefore, the IPOE recommends that the connection part of the asphalt face with the gallery be designed as shown in Figure 15. The top of the gallery concrete should be a stepped shape, like stairs, so that the thickness of each layer can be uniform and straight. Paving work will be easier with asphalt finishers and achieve a higher quality result. The red lines in Figure 14 indicate an example of the modified shape of the top of the inspection gallery concrete and each layer of asphalt face.

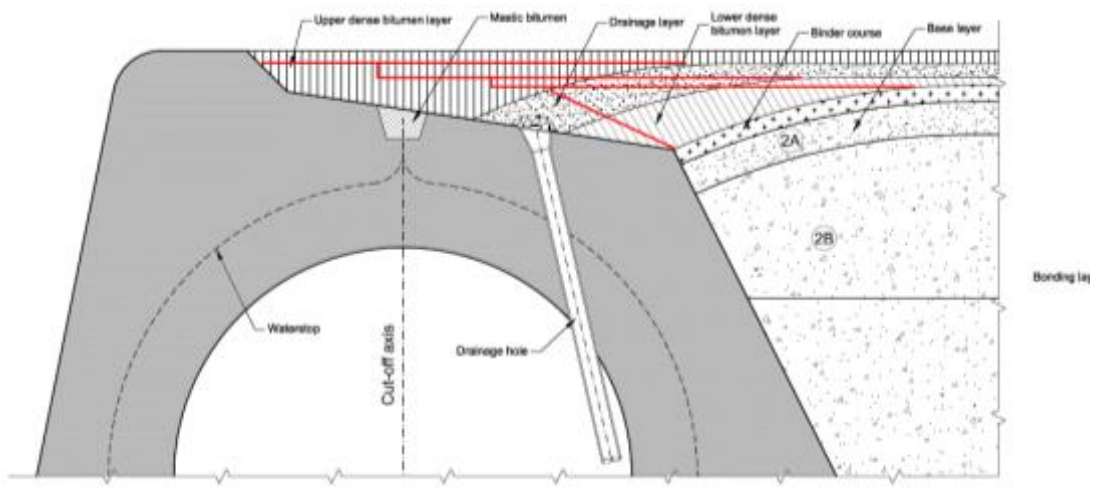


Figure 14 Basic Design - Gallery – Asphalt Face Connection Detail

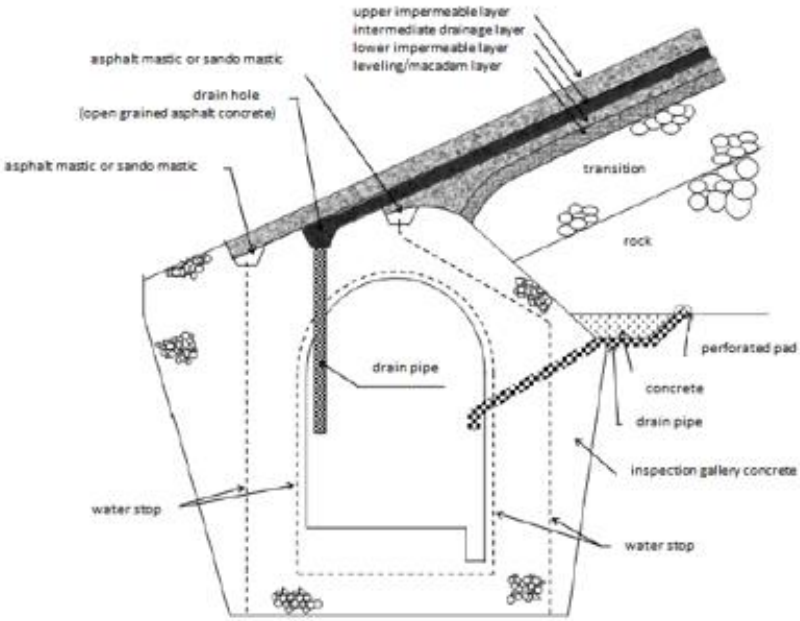


Figure 15 Alternative Gallery – Asphalt Face Connection Detail

Face Structure at the Dam crest

It seems difficult to pave the intermediate drainage layer near the crest, since it is gradually thinning as shown in Figure 16 below. The IPOE recommends that the shape of this part be modified in consideration of the construction stage.

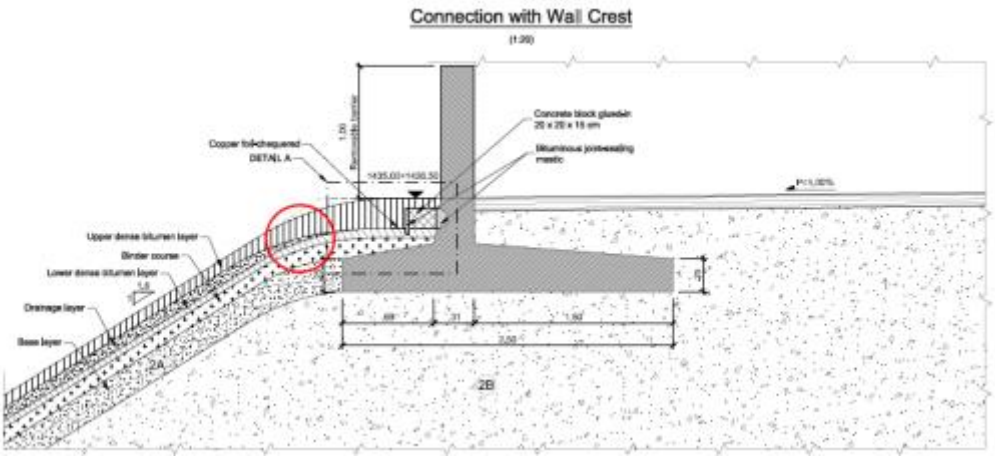


Figure 16 Asphalt Face at the Dam Crest

While an example of the face structure construction is shown below, it should be considered very carefully in order to alleviate the concentration of strain. The red lines indicate a potential modified shape of each layer of asphalt face and filler.

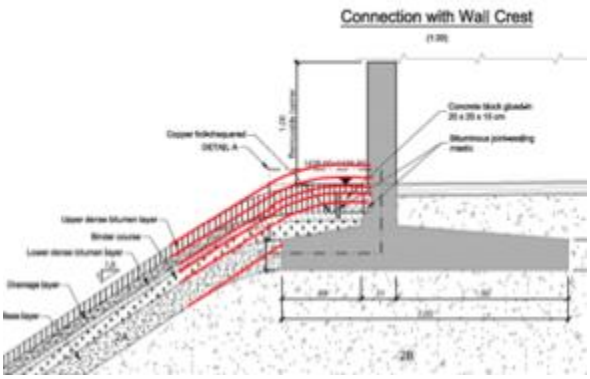


Figure 17 **(Step1) Paving**: Each layer of asphalt face is paved with a shape that rounds the crest shoulders.

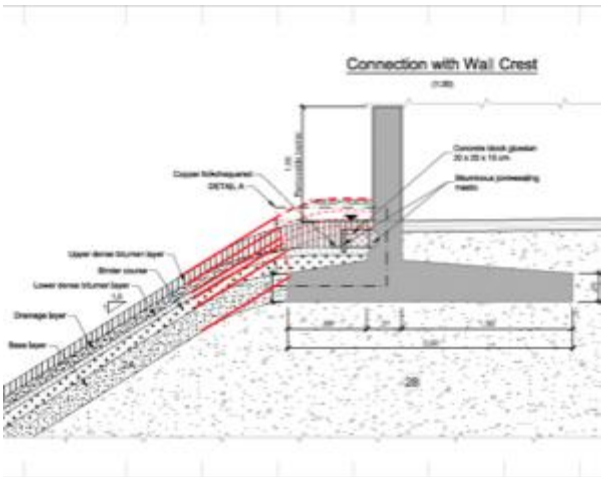


Figure 18 **(Step 2) Removing:** The part surrounded by the broken line is cut and removed.

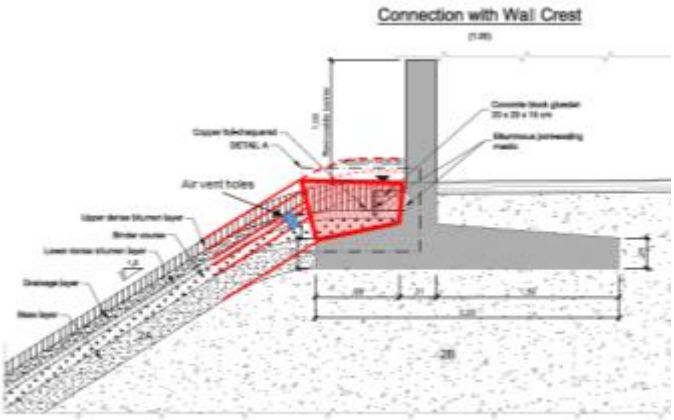


Figure 19 **(Step 3) Re-filling:** Filler, such as asphalt mastic, is put into the removed part

The IPOE therefore recommends modifying the design of the face structure near the crest concrete in order to provide a larger flexibility against the displacement of the crest concrete during an earthquake.

Air vent holes near the crest

Air vent pipes should be installed above FSL at regular intervals over the full length of the dam crest for smooth drainage of leakage water. The position and direction of the air vent pipes is recommended as shown in Figure 19.

Drain hole on the downstream side of inspection gallery

In addition, leakage may occur from the asphalt face, cracks and joints of the gallery concrete as well as penetration water from the foundation. In order to avoid back pressure on the asphalt face, the IPOE recommends that drainage holes be installed at the downstream side of the inspection gallery as shown in Figure 15. In case there is a problem at the boundary between the gallery and the cut-off wall, leakage water could significantly increase and lead to high pressure on the back of the asphalt face. If there is no drainage hole at the inspection gallery, all water pressure may act on the back of the asphalt face. When the reservoir level is lowered for repair work, the asphalt face may be destroyed by the back pressure. For this reason, a drain on the downstream side of the inspection gallery is necessary.

A concern may be that having drains from the formation into the gallery could lead to an increased risk of internal erosion by locally establishing very high hydraulic gradients in the event of damage to the top of the cutoff wall. As a countermeasure to the risk of such internal erosion, installation of a valve for each drainage hole is one of the solutions. When necessary, water can be drained through the valves observing turbidity of water.

To assess whether or not back pressure acts on the asphalt face, observation of pore water pressure in the dam body is useful. In order to observe pore water pressure, pressure gauges and meters should be installed at the valves of the drainage holes, and at several places in the drainage layer on the footprint of the dam from inspection gallery to the downstream toe of the dam. In case of emergency, it is then possible to safely drawdown the reservoir water level while observing and confirming the water level in the dam body with the pore pressure gauges and meters.

The IPOE also recommends that leakage from the asphalt face, cracks or joints and penetration water from the foundation be measured separately.

In the case of Yashio Dam, leakage water has been measured separately as follows:

1) Facilities for leakage water measurement include:

a. Drainage channels on both sides of upstream and downstream in the gallery are installed as shown in Figure 20.
b. Drainage pipes are installed at the upstream side from the intermediate layer to the gallery. The pipes are connected to the center part of the upstream drainage channels.
c. Drainage pipes are also installed at the downstream side from the Dam to the downstream drainage channels in the gallery.
d. Triangular-notch weirs are installed for automatic water measurement at both sides of left and right banks in the upstream drainage channels.
e. Triangular-notch weirs are also installed in the downstream drainage channels.

2) Measurement of water from the impermeable asphalt face:

a. The water from the impermeable asphalt face is lead from the intermediate layer to the gallery.
b. The water from each drainage pipe is collected in the center part of the upstream drainage channel with the connected pipe.
c. The water from both left and right banks is separately measured at the triangular-notch weirs installed in the upstream drainage channel.

3) Measurement of water from cracks and joints of the gallery concrete as well as the foundation:

a. The water from cracks and joints of the gallery concrete upstream is collected and lead to the downstream drainage channels through the upstream drainage channels, the separate wall and cross channels.
b. The water from cracks and joints of the gallery concrete downstream is also collected through downstream drainage channels.
c. The water from the foundation is collected through drainage pipes installed downstream of the gallery and lead to the downstream drainage channels.
d. Leakage water from cracks and joints together with water from the foundation is measured at the triangular-notch weirs installed in the downstream drainage channels, separately for from left and right banks.

4) Measurement of water for each drainage pipe of upstream and downstream can be done manually. At Yashio Dam, 23 upstream drainage pipes in total were installed at about 10m regular intervals in the inspection gallery. This enabled easy identification of cracking positions of the asphalt face when the Tohoku Earthquake happened in 2011. Actually, the asphalt face of the Yashio Dam was cracked by the earthquake. Increase of leakage water was confirmed at three drainage pipes which were located just below the cracks of the asphalt face on both right and left banks

5) All collected water is drained to the downstream toe of the Dam through the drain duct.

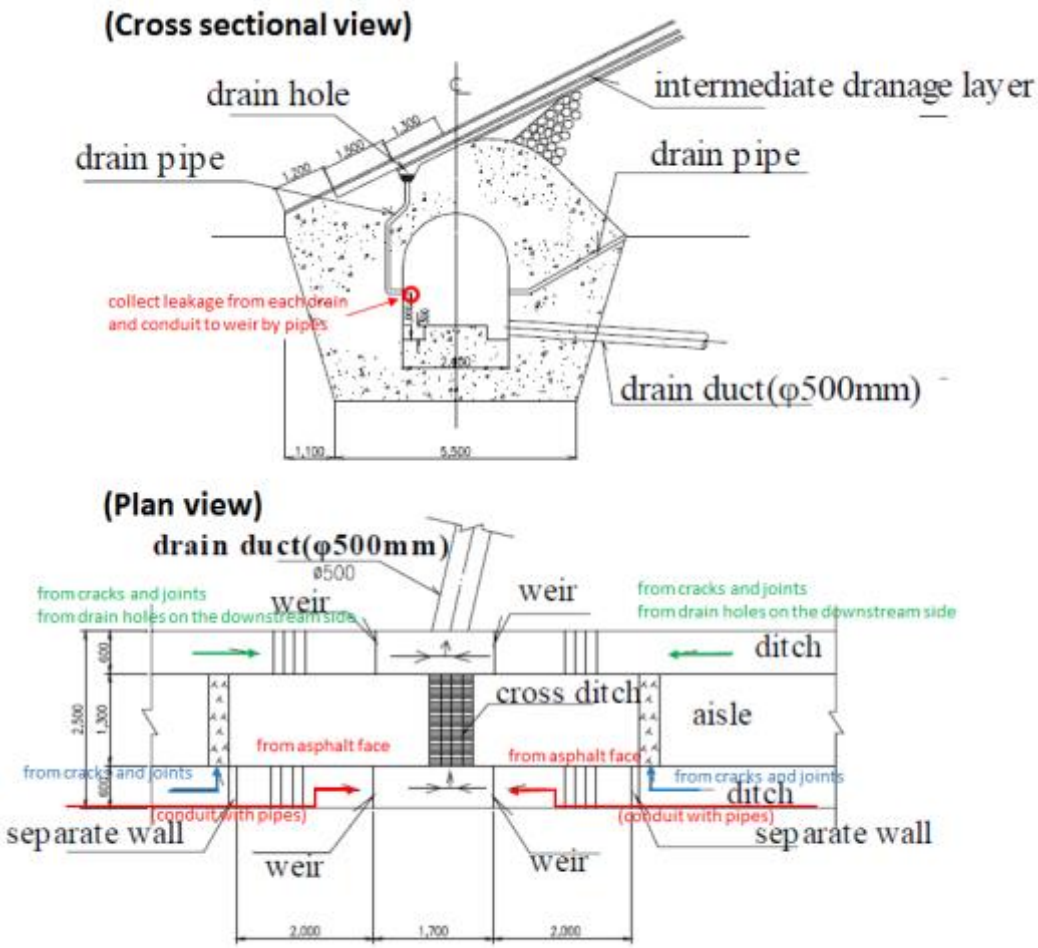


Figure. 20 Leakage measurement equipment in the inspection gallery

Asphalt Face Recommendation Summary

The following recommendations are provided to guide face design in the Detailed Design stage:

a. To determine the appropriate thickness of the asphalt face taking into consideration properties obtained from laboratory and field tests and the required performance.

b. To estimate the required thickness of the drainage layer from a view point of drainage capacity taking into account the permeability of each layer of the asphalt face.

c. The IPOE noted that the input acceleration response spectrum (0.1-0.2 sec) used for the seismic analysis is different from the predominant period of the Nenskra Dam (0.7-0.9 sec). The IPOE notes that this should be reviewed in the Detailed Design stage; however, the present analysis is believed to give conservative deformations values.

d. To carefully evaluate the strain at the block joints of the inspection gallery concrete, including the effect of earthquake loading. In the case that analysis results show the strain exceeds the failure strain (maximum allowable strain of the face material), countermeasures should be taken.

e. To conduct a safety assessment of the cut-off wall during the Detailed Design stage. After deciding the composition of the material for the cut-off wall, it is necessary to assess the physical properties, such as the elastic modulus and the strength of the material, and re-analyse to confirm its safety.

f. To check the effect of the hydrodynamic pressure on the seismic analysis by using added mass, if possible.

g. To examine temperature, strain rate and failure strain of asphalt concrete, since it seems difficult to ensure the allowable maximum strain as proposed by the EPC Designer. Therefore, it may be necessary to use special material such as polymer modified asphalt.

h. To confirm safety against fatigue failure of the asphalt face during an earthquake through cyclic loading tests.

i. To adequately compact the base layer or the foundation of the asphalt face horizontally during embankment construction and compact in the slope direction with a roller pulled from the crest just before paving in order to avoid differential settlements. The foundation of the inspection gallery should be consolidated to avoid excessive deformation.

j. To re-assess the necessity of a face layer tack coat by testing the shear strength of the contact layers with and without a tack coat.

k. To ensure that the impermeable layer of the asphalt face has both large flexibility and small flowability of fine grained asphalt concrete, since a thick layer may lead to increasing risk of asphalt flowing on the slope.

l. To study the possibility of using a simpler shape for the connection part of the asphalt face to the gallery allowing easier construction with a resultant increase in face quality in that zone.

m. To modify the shape of the intermediate drainage layer near the Dam crest in consideration of the construction stage.

n. To modify the design of the face structure at the connection part with the crest wall.

o. To install air vent pipes above FSL at regular intervals over the full length of the dam crest for smooth drainage of leakage water. The position and direction of the air vent pipes is recommended as shown in Figure 19.

p. To install drainage holes at the downstream side of the inspection gallery in order to avoid back pressure on the asphalt face. To attach valves with these drainage holes and install pore pressure gauges at the drainage holes and pore pressure meters on the footprint of the dam to enable monitoring of water levels in the dam body.

q. To separately measure leakage water from asphalt face, cracks or joints of the gallery concrete and penetration water from the foundation.

3.5.6. Spillway

The EPC Contractor has assessed the comparative risks of two spillway alternatives (EPC Report L-6768-B-SA-SP-GE-RA-001_000 Spillway options risk assessment):

a) a surface spillway on the left bank with an ungated overflow weir and stepped chute;

b) an underground spillway including an inclined shaft and a mildly sloping tunnel ending with a ski jump adjacent to the bottom outlet.

The risk assessment concluded that: "*Both alternatives are considered technically feasible and might be adopted for the present project.*"

At the Lausanne design meeting in November 2016 it was decided to adopt the tunnel spillway alternative. However, it was recognised that issues of intake clogging by log debris and construction of the outlet section of the tunnel in loose material will require careful assessment and design.

The IPOE has now reviewed the Spillway Basic Design elaborated in:

- "Hydraulic Structures – Technical Report" (EPC Report L-6768-B-HY-GE-GE-TR-002_004 dated December 2016);
- "Risk Assessment for Spillway blocking – Technical Report" (EPC Report L-6768-B-SA-SP-WE-RA-001_000 dated December 2016);
- Drawings of the construction support arrangement at the downstream section of the Spillway Tunnel and Bottom Outlet;
- Geological profile along the tunnel alignment.

The IPOE supports the use of a Tunnel Spillway subject to the comments below.

<u>Log Debris Protection</u>

The EPC team has recognised the importance of providing adequate measures to prevent clogging at the intake of the tunnel spillway with the use of a log boom device. While the log boom is a matter for detailed design it warrants comment from the IPOE since in this case it is a critical dam safety protection device. Log debris may arrive at the spillway in both floating and semi submerged condition. At the design flood level (1433 masl) there will be a 3m water surcharge above the spillway crest (1430masl). A floating log boom must therefore be capable of preventing semi- submerged logs from passing over the spillway. Log boom examples exist to deal with such semi-submerged material, particularly in the Amazon region of South America where use is made of steel frames hanging vertically and suspended by floats. The critical role of the log boom warrants consideration of providing two parallel booms where one acts as a contingency measure.

There will also need to be provision for floating log debris retrieval and removal for the long-term safe operation of the project.

The IPOE notes the assessment by the EPC team of a scenario where 50% spillway capacity reduction takes place due to clogging. The importance of an independent Bottom Outlet is recognised as in that scenario the Bottom Outlet can provide an emergency discharge capability to assist in passing extreme floods.

Downstream Tunnel Section

As noted above the value of having a Bottom Outlet tunnel independent from the Spillway tunnel is a critical safety measure at Nenskra. It is important therefore for the two tunnels to be as independent from one another as practicable. The IPOE is concerned that the current arrangement of the two tunnels coming in close proximity at the downstream sections (Figure 21) limits this independence. Since the downstream sections are to be constructed in loose material should local movement arise due to earthquake or settlement then both tunnels are likely to be simultaneously affected. If the outlets of the two tunnels are separated this risk is reduced. Therefore the IPOE suggests further consideration of the alignment of the Spillway tunnel. This is discussed further at section 3.6.3.

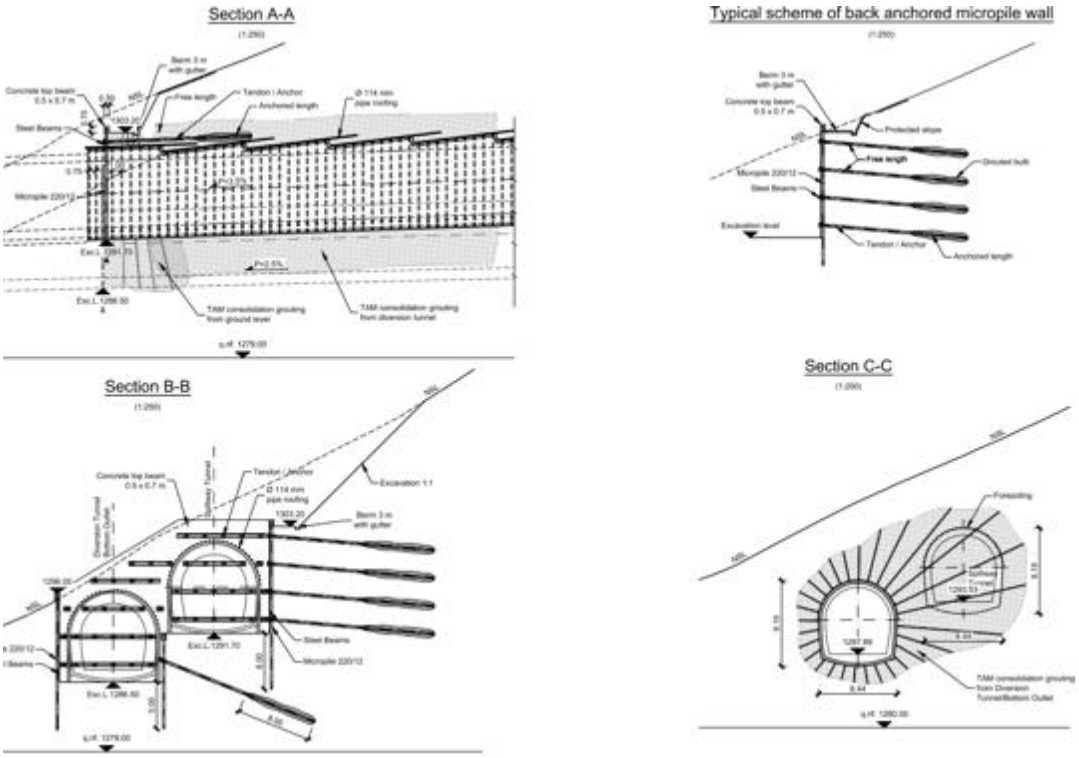


Figure 21 Downstream zone of the Bottom Outlet and Spillway Tunnel

Recommendations

a. The IPOE supports the Tunnel Spillway concept, but suggests further consideration of the alignment of the tunnel to maintain independence between the Spillway and the Bottom Outlet tunnels at the downstream zone.

b. The design of the log boom must address the risk of passing semi-submerged log debris. Furthermore, the IPOE suggests consideration be given to installing a second, back up log boom as a contingency measure.

c. Log debris retrieval and removal capability must be provided for long-term operations.

3.6. Nakra Weir

The EPC team has reviewed the Nakra Weir arrangement to improve its functionality regarding stilling apron maintainability, sediment management, fish passage and flow control, The IPOE has reviewed the "Nakra Weir – Description and operation" report (Ref xx L-6768-B-EN-NH-GE-MN-001_000 Description and Operation)

From a project safety and operational flexibility perspective the addition of two planar gates to stop the diversion of water towards the Nenskra reservoir "in case of necessity, when it is already full and spilling through the spillway" is an important improvement.

As well sediment flushing has been more effectively considered.

The provision of a local diesel generator and a UPS for the operation of hydro-mechanical equipment plus wireless communication system significantly enhances the operability of the facility.

3.7. Tunnels

3.7.1. Transfer Tunnel

Alignment

The alignment issues have been discussed in the first IPOE report of May 2016. The IPOE recommended shifting the tunnel to increase the distance to the probably highly tectonized Alibeck Fault which adversely runs more or less parallel to the tunnel, dipping NE. The major Alibeck Fault thus runs closer to the tunnel (or even crossing it) than it appears on the map, where it is shown on the surface.

Considering the general layout on the geological map (L-6768-B-GS-TT-GE-DW-001_000 dated 15.12.2016 and Figure 22) the IPOE notes that both Nakra intake and Nenskra outlet have been shifted towards upstream, while the central vertex has been shifted North, both to increase the distance to the fault and to minimize the maximum overburden.

From the geological point of view, the present alignment may be qualified as optimal, keeping in mind that the sub-parallelism of the alignment and the main geological pattern means that deficient rock mass zones, if touched by the tunnel, may still affect it over long distances. But this inconvenience cannot be avoided.

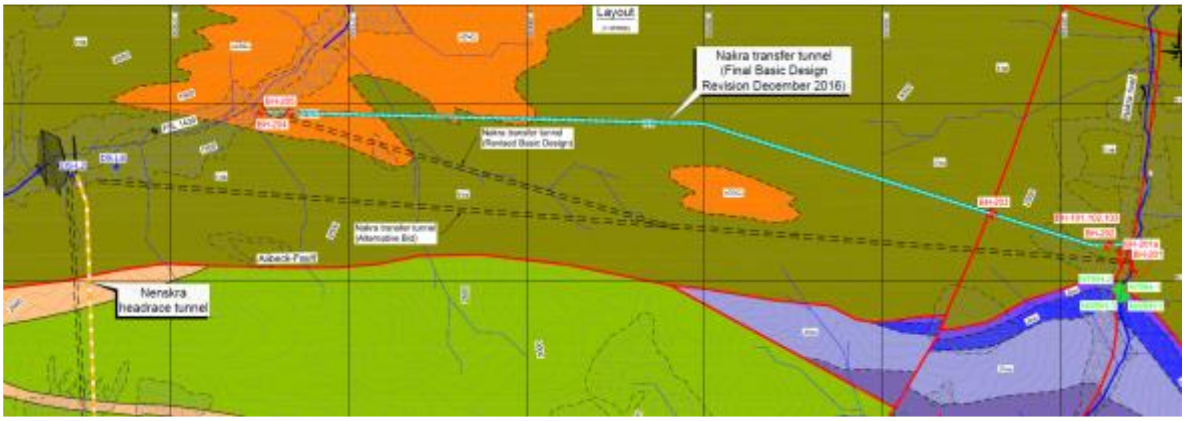


Figure 22 Alignment of TT on geological map (L-6768-B-GS-TT-GE-DW-001_000, dated 15.12.16)

Another comment concerns borehole BH-203 (Figure 23) planned to investigate a fault crossing the tunnel on the side of the Nakra Portal. The opinion of the IPOE is that taking account of the depth of the tunnel such a borehole - as it is drawn on the profile - may well give some useful information on the fault zone. However, in-depth extrapolation of this information over several hundreds of meters is extremely challenging in terms of position, as well as fabric and width, not to talk about hydrogeology. Since it will hardly be possible to drill it to the depth of the tunnel, and even less to be sure to touch the fault close to tunnel elevation. Such type of boreholes will not alleviate the need for investigations during excavation by means of drilling ahead of the TBM when approaching an expected or supposed fault zone.

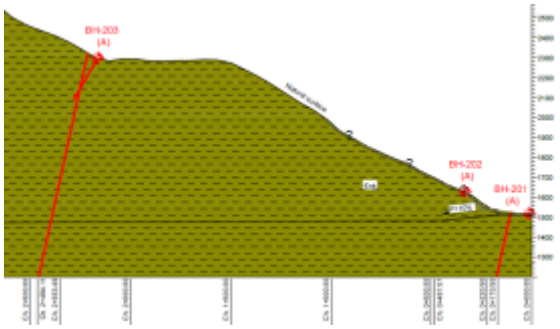


Figure 23 Section of the geological profile on former Transfer Tunnel alignment

Construction Issues

The IPOE notes that after having changed from double-shield TBM with segmental lining to open TBM, the project switched back to double-shield TBM. The IPOE supports the choice of double-shield TBM with segmental lining as it provides increased tunnel security over the longer term. Since now both tunnels (Transfer and Headrace) will be excavated the same way it is suggested to refer to the section 3.7.2 dedicated to the Headrace Tunnel for further comments.

The main constructive recommendation, even if less critical than for an open TBM, will be, for both tunnels, that the TBM, by means of a pre-installed drill rig, allows for sufficiently long borehole investigation ahead of the machine (minimum could be a one day-shift progress). This will prevent unexpected penetration into a geologically strongly disturbed zone (fault zone, strongly aquifer zone, etc), and provide the means of preventively and adequately treating such a zone (grouting, drainage, etc).

3.7.2. Headrace Tunnel

The Risk Assessment report concerning the Headrace Tunnel has been updated and is dated 14.12.16.

Before going into detailed discussion it is noted that except at the intake area, in depth investigation (boreholes) have not yet been made to date along the alignment of this tunnel. The comments will thus remain on a general level, however noting that the planned investigations are not on the critical path.

A general remark, especially by comparison with the Transfer Tunnel, is that the Headrace Tunnel will cross a much more changing geology and that its alignment is practically perpendicular to the main structures. Given the challenging geology, this is a basically favourable situation since it means that deficient zones, which are generally parallel to the main structure, will be crossed over the shortest possible distance.

As for any tunnel, one significant risk is to penetrate unexpectedly into a geologically or hydrogeologically disturbed zone. As for the Transfer Tunnel, or even more because of the changing geology and the crossing of several fault zones, it is important for the Headrace Tunnel to undertake pilot-boreholes ahead of the TBM, especially when tectonized zones are expected to be approached. And, again as for the Transfer Tunnel, the TBM will have to allow for ground improvement measures ahead of the TBM, especially drainage and grouting. Such boreholes are also to be planned when longer stand stills of the TBM are necessary (maintenance) in order to avoid those stand stills to be located in particularly unfavourable zones.

By-passing the TBM is mentioned by the EPC team as an ultimate measure, for example in case the machine is jammed by an unexpected squeezing zone. Such a by-pass, however, is a very time-consuming, technically difficult, and thus expensive option (especially for double-shield TBMs with segmental lining). The by-pass itself needs heavy constructive measures to be applied close to the jammed TBM head. Avoiding the necessity of such a by-pass is one more reason to proceed to borehole investigation ahead of the machine, quasi systematically in critical zones.

Concerning water losses in pressure tunnels, one "rule of thumb" (e.g. Talobre Rule in France) is that no special measures will have to be taken as long as the overburden is equal to the internal pressure expressed in meters of water head, and that the lateral distance to the slope is greater than 2.5 times this pressure. On the major part of the Headrace Tunnel alignment, these conditions are comfortably respected. The IPOE however draws attention to the section around chainage 09+500 Figure 24) where the overburden, as well as the lateral distance to the slope, are both close to that "limit". Furthermore that section is close to a potentially permeable structure called "Frontal Thrust" on the geological map. It is strongly suggested to consider this zone for future borehole investigations. The results of such investigation could lead to a slight re-alignment, either by shifting the point where the tunnel changes direction further downstream or, if it then gets too close to the fault zone, by slightly changing the azimuth of the tunnel upstream of that point.

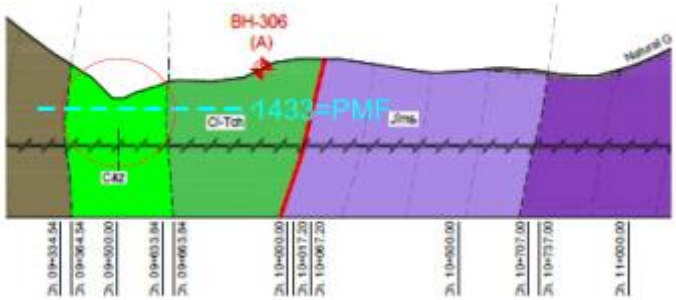


Figure 24 Section of the geological profile (L-6768-B-GL-HR-GE-DW-002_002, 15.12.16)

Regarding the future borehole investigations, as for the Transfer Tunnel mentioned above, the IPOE draws attention to the very limited information that can be obtained from "short" boreholes like those drawn on the geological section (even if they are 200-300 m deep), crossing a fault zone hundreds of meters above the tunnel. As already mentioned, in-depth extrapolation of that type of information is extremely challenging, especially where an expected fault zone is supposed to subdivide at depth (Figure 25).

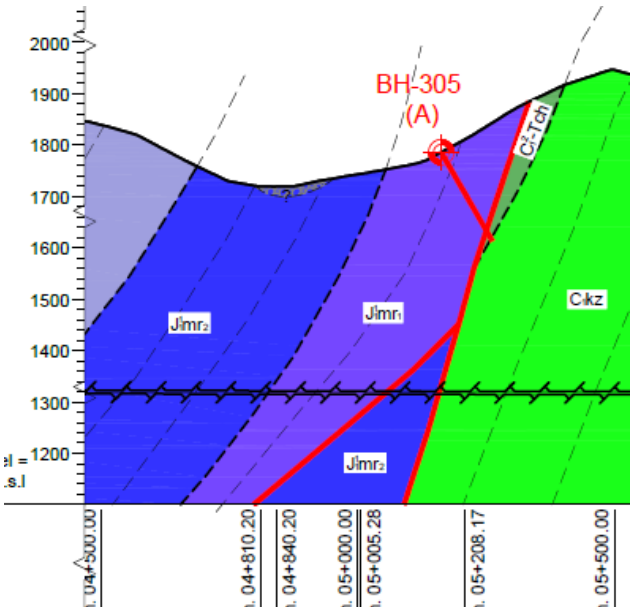


Figure 25 Section of the geological profile (L-6768-B-GL-HR-GE-DW-002_002, 15.12.16)

Furthermore, such "superficial" investigation will hardly give any useful hydrogeological information. It is thus recommended to take account of that aspect for the location, direction and depth of the planned boreholes along the alignment of the Headrace Tunnel.

The IPOE also notes that baseline monitoring of the hydrogeology is accepted by the EPC team and we reiterate that monitoring of natural springs should begin at least 1 year before excavation.

Recommendations

a. The IPOE supports the choice of double-shield TBM with segmental lining as it provides increased tunnel security during construction and over the longer term.

b. The IPOE recommends that investigation and construction installations (for grouting and/or drainage) are provided on the TBM equipment.

c. The IPOE notes the limited useful information that may be gathered from boreholes drilled from surface when the results have to be extrapolated over great depths. Such boreholes will neither exempt the necessity of investigation ahead of the TBM, nor allow to precisely predict when the latter will have to be done.

3.7.3. Bottom Outlet and Tunnel Spillway

Being very close to each other, these two tunnels will be discussed together from the geological point of view. The comments will concern the outlet zone of both tunnels. This does not mean that input portals or excavation in bedrock do not raise some constraints or difficulties, but the latter are judged as not being exceptional.

Tunnel Alignment

The alignment of both tunnels, in their downstream part, is very oblique to the slope. While this facilitates the restitution of the flows safely to the river via ski jump structures, the consequences from a construction point of view will be that:

- both tunnels will have to be excavated in soft cover material, possibly including blocks (according to present knowledge of the geological conditions);
- excavation will be in soft ground over a considerable length: 250m according to the provided profiles, but it could be longer as the geological profiles are not yet supported by detailed investigations;
- penetration from soft ground into bedrock will presumably be very oblique, in both the vertical and the horizontal planes, with the consequence that there will probably be long sections (possibly 50m) of the tunnels to be excavated in both soft ground on one part (excavator) and hard rock on the other (mixed face), not to talk about construction measures like jet grouting which will also have to adapt to very changing conditions.

The results of boreholes may well be extrapolated in terms of geological structures, but such extrapolations become hazardous when the bedrock surface is concerned. The shape of the bedrock surface is driven by erosion, and may be subject to strong and unexpected variations over short distances. As the IPOE understands, some geophysical investigations have been performed. Such investigations are certainly wise, but need to be "calibrated" by borehole investigations, since geophysics is unable to precisely detect abrupt changes.

In addition, the "outlet zones" of both tunnels are located in the foot zone of two channels that seem prone to debris flows.

As noted previously it is important that these two tunnels are independent of each other with the Bottom Outlet acting as a back-up for the Spillway under extreme circumstances. These elements, considered from the geological and safety perspective, suggest that the alignment of these two tunnels should be further optimized, based on complementary investigations. It is recognised that for ideal hydraulic performance of the Spillway tunnel it should be straight in plan. The IPOE suggests that

the Spillway tunnel alignment be reviewed to obtain a practical independence of the two tunnels as shown on Figure 26 below.

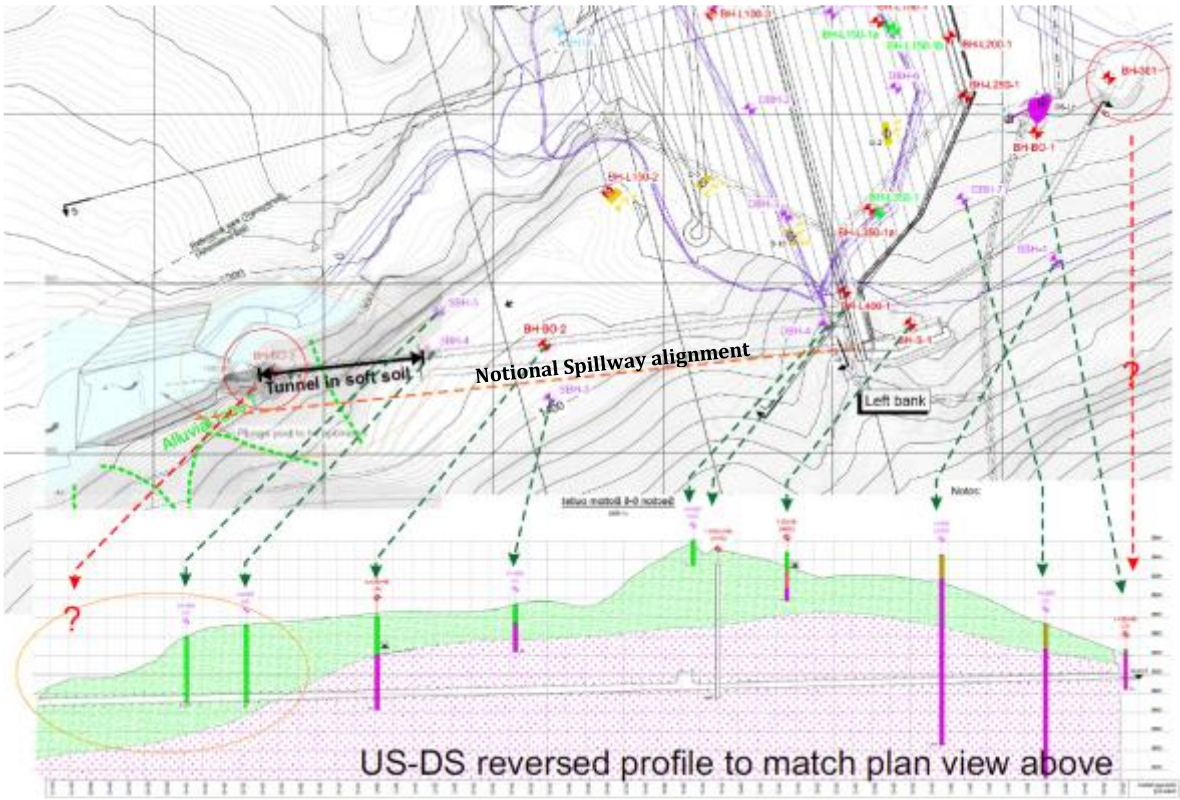


Figure 26 Possible alternative for the Spillway tunnel
(Profile: Bottom Outlet only)

Construction Methods

Concerning construction methods one consideration is that jet-grouting is an efficient but heavy method, which is well adapted to that type of soft soil with poor cohesion. It is however very time consuming and expensive.

The IPOE further notes that no special treatment is foreseen underneath the tunnel floor, also founded on poor material, which could eventually need vertical jet-columns at least on both sides of the floor.

Based on these considerations, the IPOE asks if vertical jet-grouting of the tunnel zone from the ground surface has been considered as an alternative that could be entirely realized before excavation.

Recommendations

a. The IPOE suggests reconsideration of the alignment of the spillway tunnel in conjunction with further and detailed geological investigation.

3.8. Penstock and Powerhouse

Natural Hazard Risk Exposure

In its first report (May 2016) the IPOE noted the potential debris flow Natural Hazard risk above the powerhouse and at the base of the penstock. The EPC team has appropriately moved the powerhouse downstream to avoid this risk. At the September workshop the IPOE was further informed that the Penstock crossing between the ridge and the powerhouse would occur underground and thus also be protected from the natural hazard risk.

Concerning the penstock, limited geological information is available so far due to access restrictions. Considering the contour lines on the map (EPC Report L-6768-B-GS-PN-GE-DW-001_000), the alignment on the ridge seems well adapted to the topography. For the construction however, and along with the planned boreholes, a detailed geological map will be necessary, notably showing rocky outcrops and eventually localized or superficial potentially unstable or rockfall zones.

3.9. Project Risk Assessment

During the Basic Design development process the EPC team outlined a proposed framework to complete a major safety risk assessment of the Nenskra hydropower project overall. It is a classic risk methodology where Risk = Likelihood x Damage and damage is assessed based on human life at risk. The IPOE understands that the EPC Contractor is still populating the risk table and the proposed mitigating actions and will assess the residual risk once the mitigation actions are implemented. The IPOE had recommended that this document be finalised and submitted as part of the design package provided to the Owner at the time of submission of the final Basic Design. However, since it is not available in the December final Basic Design reports, the IPOE recommends that the Project Risk Assessment report be completed prior to commencing the Detailed Design phase so that the Owner can track the implementation of the proposed risk mitigations

The IPOE strongly supports maintaining a risk management oversight of the project from detailed design through construction and into the operations phase. The IPOE recommends the project Owner review the completed risk assessment closely prior to commissioning to ensure full compliance with the mitigation actions has been achieved. Following on from this it is recommended that the risk register be kept up to date over the life of the project.

Recommendation

a. The IPOE recommends that the Project Risk Assessment report be completed prior to commencing the Detailed Design phase so that the Owner can track the implementation of the proposed risk mitigations.

b. The IPOE reiterates its recommendation that the project Owner review the completed risk assessment closely prior to commissioning to ensure that full compliance with the mitigation actions has been achieved. Following on from this it is recommended that the risk register be kept up to date over the life of the project

3.10. Emergency Preparedness Plan

The IPOE endorses the proposal from the EPC Contractor that an Emergency Preparedness Plan (EPP) will be in place at least 1 year prior to impoundment for early generation.

The IPOE recommends that the EPP is prepared to cover the following stages:

- River diversion;
- Operation during first impoundment;
- Long term operation.

The IPOE notes the importance of undertaking a dam break analysis that must feed into the EPP. The dam break analysis shall take into account impact on the bridges and other infrastructure downstream of Nenskra Dam, Enguri Dam and as well consider potential impact on the dams downstream of Enguri.

Also, the IPOE recommends that particular attention be paid to establishing Bottom Outlet operating rules and security arrangements to ensure that the potential for very high discharges do not impact on the safety of downstream settlements and infrastructures. However, from an emergency scenario perspective, a response to inadvertent Bottom Outlet operation should be included in the EPP.

Monitoring of the Dam is essential and is part of the EPP and O&M. An Instrumentation Plan should be prepared as a part of Detailed Design and should provide proposed instruments layout, sections, details and specifications. The plan should also provide frequency of reading and trigger values and should link to the EPP and O&M.

Recommendation

a. The IPOE endorses the proposal from the EPC Contractor that an Emergency Preparedness Plan (EPP) will be in place at least 1 year prior to impoundment for early generation.

b. The dam break analysis shall take into account impact on the bridges and other infrastructure downstream of Nenskra Dam, Enguri Dam and as well consider potential impact on the dams downstream of Enguri.

c. IPOE recommends that particular attention be paid to establishing Bottom Outlet operating rules and security arrangements to ensure that the potential for very high discharges do not impact on the safety of downstream settlements and infrastructures. Nevertheless, a response to inadvertent Bottom Outlet operation should be included in the EPP.

d. An Instrumentation Plan should be prepared as a part of Detailed Design and should provide proposed instruments layout, sections, details and specifications. The plan should also provide frequency of reading and trigger values and should link to the EPP and O&M.

4. Social Review

The social review was conducted by IPOE member Frederic Giovannetti (social and resettlement specialist). It is structured along the following key themes:

- ESIA process and documentation;
- Labour;
- Community safety and security;
- Land acquisition and resettlement;
- Potential applicability of Indigenous Peoples policies;
- Cultural Heritage.

The review is based on the package prepared by ESIA Consultants SLR (draft versions of the ten volumes disclosed to the IPOE in December 2016 and January 2017). The ten volumes of the ESIA are the following:

1. Non-Technical Summary (NTS);
2. Project definition;
3. Social Impact Assessment (SIA);
4. Biodiversity;
5. Water;
6. Natural hazards and Dam safety;
7. Stakeholder Engagement Plan (SEP);
8. Environmental and Social Management Plan (ESMP);
9. Land Acquisition & Livelihood Restoration Plan (LALRP);
10. Cumulative Impact Assessment (CIA).

4.1. ESIA Process and Documentation

The IPOE review was subsequent to the lenders review and the version of the package reviewed by the IPOE had already received numerous comments from the lenders and their independent engineer. As a result, IPOE comments are limited. The ESIA package is comprehensive and robust. It addresses all potential environmental and social issues, risks and impacts. It has been directly suggested by the IPOE to the ESIA Consultants that a clearer definition of the Project as opposed to Associated Facilities was warranted (Volume 2 and NTS). The NTS itself is clear, reasonably well illustrated, and appears fit for the purpose of meaningful disclosure and subsequent consultation, subject to a number of clarifications of low criticality requested directly of the ESIA Consultants by the IPOE. The ESMP is particularly well structured and well presented.

Stakeholder engagement on the ESIA is now critical as the community's and more generally the stakeholders' reactions to the proposed mitigation are yet to be assessed. The SEP includes provisions related to further consultation, these will have to be implemented as of the disclosure of the ESIA package, particularly with regards to the SIA, the LALRP, the ESMP, and the natural hazards volume.

In this regard, whilst the IPOE recognises that public meetings in the main communities that are potentially affected are probably necessary, the IPOE is of the view that an "open houses" format of consultation will be more conducive to proper engagement with communities. An open house in each of the affected communities would allow residents of affected communities to:

- Obtain in-depth information from project personnel in face-to-face or small group interaction;
- Get acquainted with documents through access to the full package as well as through simplified, user friendly versions (the NTS, a project leaflet, a brief "Guide to Land Acquisition and Compensation"), and posters and/or presentations running on available computers;
- Lodge queries and grievances to Project personnel as needed.

In both public meetings and open houses, Project personnel will have to document all feedback from stakeholders, including by not limited to affected community members. This feedback will have to be included (in summary) in an update to the SEP and/or communicated directly to lenders and the IPOE and publicly disclosed.

4.2. Labour

The Georgia Labour Code was revised in 2013 and many of the gaps of previous labour regulations that had been repeatedly flagged by international trade union organisations are reportedly bridged by the 2013 Labour Code. While the Project SIA (Vol. 3) does not present a systematic review or gap analysis of these new labour regulations against ILO conventions and lender policies, the SIA and ESMP contain robust measures to meet these conventions and policies. The key challenge will be (1) to ensure that these measures are implemented by the EPC Contractor and sub-contractors, and (2) to put in place an operational enforcement mechanism. The ESMP structure, that clarifies respective responsibilities of the different parties, is a first positive step towards such enforcement.

4.3. Community Safety and Security

Community Safety

Volume 6 of the ESIA package was still being finalised at the time of this review. Since its inception, the IPOE has provided extensive inputs into risks and incident scenarios considered in Volume 6, with these dam safety technical aspects reported in other sections of this report. From a community perspective, and particularly since concerns were expressed by communities downstream in regards of dam safety, it will be important to convey key dam safety messages relevant to both construction and operations during the disclosure and consultation period to start shortly. In this regard, publicly disclosing a Georgian translation of this IPOE report should be considered by JSC Nenskra, as the IPOE brings an independent review of proposed dam safety measures.

Security

JSC Nenskra is committed to the Voluntary Principles for Security and Human Rights. The SIA contains a number of measures that are typical for projects of this type in regards of use of private security, management of low level incidents (such as blockages or local demonstrations) without resorting to disproportionate force, and training of private security. JSC Nenskra will have to engage the local police (locally – in Khaishi – and in Zugdidi) as of the ESIA disclosure process.

4.4. Land Acquisition and Resettlement

The LALRP (Volume 9) is robust, with impacts adequately identified and addressed in line with lenders standards. The IPOE noted two gaps that should be addressed by JSC Nenskra and their consultants prior to the disclosure period (this has been discussed directly with the consultants and it is understood it is in the process of being addressed):

- The document mentions land impact thresholds of 10% and 20% that make related households eligible to livelihood restoration package ("significantly" and "severely" affected households, respectively). While the IPOE concurs with this general approach, the rationale for the thresholds should be strengthened;
- The overhead transmission line from the Nenskra switchyard to the Khudoni sub-station is yet to be addressed (a route was not available at the time the LALRP was prepared): while the planning and construction of this associated facility is under Georgian State Electrosystem (GSE) responsibility, the IPOE understands that the EBRD recently launched on behalf of GSE a call for tenders in regards of an ESIA and Resettlement Framework that covers this facility. It is critical that land impacts for this facility are addressed in conformance with compensation policies developed for the Nenskra project (and more generally with lenders policies). JSC Nenskra may have limited leverage to ensure this is the case. However, the IPOE recommends that EBRD, which is understood to seek involvement in the transmission line component, should ensure consistency (and compliance with its own resettlement and land acquisition standard).

Subject to the two comments above being addressed, the IPOE is of the view that the LALRP can be publicly disclosed.

At this point, and although some consultation did take place (focus groups and face-to-face interaction), there is yet limited evidence that proposed consultation and livelihood restoration are agreeable to affected communities, as measures as they stand in the LALRP are yet to be consistently consulted upon. This will take place during the disclosure period. Similar to recommendations above, this will have to be documented and related evidence will have to be included and publicly disclosed (as part of an updated version of either the SEP or the LALRP).

4.5. Potential Applicability of Indigenous Peoples Policy

The IPOE has reviewed sections of the SIA relevant to the possible applicability of lenders' Indigenous Groups policies. The IPOE concurs with the view taken in the ESIA package that these policies do not apply to the Svan group, in spite of certain criteria in lenders policies being partially applicable. Discussions took place with the Project ESIA consultant to strengthen the related discussion in the SIA and IPOE comments are in the process of being included in a further iteration of the SIA.

4.6. Cultural Heritage

The cultural heritage assessment (baseline and mitigations) in the ESIA package is robust. The IPOE has no comments on tangible cultural heritage. Insofar as intangible cultural heritage is concerned, the IPOE is of the view that, while anticipated Project impacts on intangible cultural heritage are unlikely to be of any significance, it would be good practice for JSC Nenskra to support the conservation and enhancement of a number of intangible features specific to the area and Svaneti. This should be considered as part of the Community Investment Plan (CIP) currently being prepared by JSC Nenskra and could include measures meant to support local groups and cultural festivals in the areas of traditional dancing, songs, tales, etc… Supporting cultural initiatives in local schools that target the younger generation should be given priority.

5. List of Detailed Recommendations

5.1. Natural Hazards

a. The Natural Hazard risk posed by a suspected major landslide zone on the right bank above the reservoir has received particular attention from the EPC team. The IPOE accepts the analysis that this is not a major landslide risk and agrees that this zone does not pose a safety risk to the project.

b. The IPOE considers that the various discussed natural hazards have been thoroughly addressed (avalanches, debris flows, rockfall, landslides, glacial lake outbursts) and there is no high risk identified, and furthermore the ones qualified as moderate can be reduced by design measures.

5.2. Flood Assessment

a. The IPOE recommends that the EPC team undertakes a sensitivity analysis on the level of flood protection provided during diversion and early generation taking into consideration the as planned progress of Dam construction.

b. The climate change impacts on the Nenskra HPP have been suitably clarified by the EPC team. The IPOE notes that a conservative design PMF value, with a freeboard on the associated maximum reservoir level, helps to ensure the Project's resilience to cope with maximum hydrological events.

5.3. Dam Foundation Seepage

a. The IPOE understands that drilling of borehole BH-R150-2, located on the alignment of the cut-off wall and which is still in progress, is planned to be driven into the bedrock, thus meeting the IPOE's recommendation from its Stage II – Part 1 report.

b. With regards to the depth of the diaphragm cut-off wall, the seepage gradients and any potential for progressive suffusion: the EPC Consultant has undertaken a seepage sensitivity analysis and based on that extended the diaphragm wall to 85m, reaching the elevation of 1225masl. The deepened cut-off wall would be in the glacial deposits for a few meters; this will limit the seepages to <200 l/s and minimize the risk of progressive suffusion. The IPOE is in agreement with the proposed deeper cut-off wall.

c. The EPC Consultant has, in its final Basic Design documents of December 2016, proposed a 3A drainage layer over the footprint of the embankment; the drainage layer is 5m thick in the valley floor and 0.8m thick in the abutments. The drainage layer will ensure that any water table rise is contained within the drain and the embankment fill remains dry. This is in line with the IPOE's previous recommendations.

d. The EPC Contractor must undertake a trial grouting in the abutments to demonstrate that the foundation material in the abutments is groutable and the targeted permeabilities can be achieved. If this is not the case, the cut-off wall is likely to extend into the abutments as well

5.4. Asphalt Face

The following recommendations are provided to guide face design in the Detailed Design stage:

a. To determine the appropriate thickness of the asphalt face taking into consideration properties obtained from laboratory and field tests and the required performance.

b. To estimate the required thickness of the drainage layer from a view point of drainage capacity taking into account the permeability of each layer of the asphalt face.

c. The IPOE noted that the input acceleration response spectrum (0.1-0.2 sec) used for the seismic analysis is different from the predominant period of the Nenskra Dam (0.7-0.9 sec). The IPOE notes that this should be reviewed in the Detailed Design stage; however, the present analysis is believed to give conservative deformations values.

d. To carefully evaluate the strain at the block joints of the inspection gallery concrete, including the effect of earthquake loading. In the case that analysis results show the strain exceeds the failure strain (maximum allowable strain of the face material), countermeasures should be taken.

e. To conduct a safety assessment of the cut-off wall during the Detailed Design stage. After deciding the composition of the material for the cut-off wall, it is necessary to assess the physical properties, such as the elastic modulus and the strength of the material, and re-analyse to confirm its safety.

f. To check the effect of the hydrodynamic pressure on the seismic analysis by using added mass, if possible.

g. To examine temperature, strain rate and failure strain of asphalt concrete, since it seems difficult to ensure the allowable maximum strain as proposed by the EPC Designer. Therefore, it may be necessary to use special material such as polymer modified asphalt.

h. To confirm safety against fatigue failure of the asphalt face during an earthquake through cyclic loading tests.

i. To adequately compact the base layer or the foundation of the asphalt face horizontally during embankment construction and compact in the slope direction with a roller pulled from the crest just before paving in order to avoid differential settlements. The foundation of the inspection gallery should be consolidated to avoid excessive deformation.

j. To re-assess the necessity of a face layer tack coat by testing the shear strength of the contact layers with and without a tack coat.

k. To ensure that the impermeable layer of the asphalt face has both large flexibility and small flowability of fine grained asphalt concrete, since a thick layer may lead to increasing risk of asphalt flowing on the slope.

l. To study the possibility of using a simpler shape for the connection part of the asphalt face to the gallery allowing easier construction with a resultant increase in face quality in that zone.

m. To modify the shape of the intermediate drainage layer near the Dam crest in consideration of the construction stage.

n. To modify the design of the face structure at the connection part with the crest wall.

o. To install air vent pipes above the FSL at regular intervals over the full length of the dam crest for smooth drainage of leakage water. The position and direction of the air vent pipes is recommended as shown in Figure 19.

p. To install drainage holes at the downstream side of the inspection gallery in order to avoid back pressure on the asphalt face. To attach valves with these drainage holes and install pore pressure gauges at the drainage holes and pore pressure meters on the footprint of the dam to enable monitoring of water levels in the dam body.

q. To separately measure leakage water from asphalt face, cracks or joints of the gallery concrete and penetration water from the foundation

5.5. Spillway

a. The IPOE supports the Tunnel Spillway concept, but suggests further consideration of the alignment of the tunnel to maintain independence between the Spillway and the Bottom Outlet tunnels at the downstream zone.

b. The design of the log boom must address the risk of passing semi-submerged log debris. Furthermore, the IPOE suggests consideration be given to installing a second, back up log boom as a contingency measure.

c. Log debris retrieval and removal capability must be provided for long-term operations.

5.6. Tunnels

a. The IPOE supports the choice of double-shield TBM with segmental lining as it provides increased tunnel security during construction and over the longer term.

b. The IPOE recommends that investigation and construction installations (for grouting and/or drainage) are provided on the TBM equipment.

c. The IPOE notes the limited useful information that may be gathered from boreholes drilled from surface when the results have to be extrapolated over great depths. Such boreholes will neither exempt the necessity of investigation ahead of the TBM, nor allow to precisely predict when the latter will have to be done.

d. The IPOE suggests reconsideration of the alignment of the spillway tunnel in conjunction with further and detailed geological investigation.

5.7. General

a. The IPOE recommends that the Project Risk Assessment report be completed prior to commencing the detailed design phase so that the Owner can track the implementation of the proposed risk mitigations.

b. The IPOE reiterates its recommendation that the project Owner review the completed risk assessment closely prior to commissioning to ensure that full compliance with the mitigation actions has been achieved. Following on from this it is recommended that the risk register be kept up to date over the life of the project.

c. The IPOE endorses the proposal from the EPC Contractor that an Emergency Preparedness Plan (EPP) will be in place at least 1 year prior to impoundment for early generation.

d. The dam break analysis shall take into account impact on the bridges and other infrastructure downstream of Nenskra Dam, Enguri Dam and as well consider potential impact on the dams downstream of Enguri

e. IPOE recommends that particular attention be paid to establishing Bottom Outlet operating rules and security arrangements to ensure that the potential for very high discharges do not impact on the safety of downstream settlements and infrastructures. Nevertheless, a response to inadvertent Bottom Outlet operation should be included in the EPP.

f. An Instrumentation Plan should be prepared as a part of Detailed Design and should provide proposed instruments layout, sections, details and specifications. The plan should also provide frequency of reading and trigger values and should link to the EPP and O&M.

5.8. Social Aspects

a. The IPOE supports public disclosure of the ESIA package subject to addressing some comments that have been communicated directly to the ESIA consultants. IPOE recommendations include:

   i. JSC Nenskra and ESIA Consultants to include "open houses" in public engagement measures to be conducted shortly on the ESIA, as these are more conducive, in the Georgian cultural context, to meaningful consultation;

   ii. JSC Nenskra and ESIA Consultants to include community safety amongst top subjects on the consultation agenda as this has been a repeated community concern;

   iii. EBRD to ensure consistency between compensation measures in the Nenskra LALRP and those in the Nenskra – Khudoni transmission line currently being considered by EBRD, which is an Associated Facility to the Nenskra project;

   iv. JSC Nenskra to support local culture within the framework of the Community Investment Plan that is currently under preparation.

b. Publicly disclosing a Georgian translation of this IPOE report should be considered by JSC Nenskra, as the IPOE brings an independent review of proposed dam safety measures.

Appendix A List of Abbreviations

AFRD – Asphalt Faced Rockfill Dam
CFRD – Concrete Faced Rockfill Dam
CIA – Cumulative Impact Assessment
CN – Curve Number
CPI – Community Investment Plan
EPC – Engineering, Procurement, and Construction
EPP – Emergency Preparedness Plan
ESIA – Environmental and Social Impact Assessment
ESMP – Environmental and Social Management Plan
FLAC – Fast Lagrangian Analysis of Continua
FSL – Full Supply Level
GLOF – Glacial Lake Outburst Flood
GSI – Geological Strength Index
HPP – Hydropower Project
ICOLD – International Commission on Large Dams
IPOE – International Panel of Experts
ISMGEO – Istituto Sperimentale Modelli Geotecnici
JSC – Joint Stock Company
LALRP – Land Acquisition & Livelihood Restoration Plan
masl – meters above sea level
MCE – Maximum Credible Earthquake
MOL – Minimum Operating Level
MSK – Medvedev-Sponheuer-Karnik
NTS – Non-Technical Summary
O&M – Operations and Maintenance
OBE – Operating Basic Earthquake
PH – Powerhouse
PGA – Peak Ground Acceleration
PMF – Probable Maximum Flood
PMP – Probable Maximum Precipitation
PSHA – Probabilistic Seismic Hazard Assessment
Q10 – 10 year return period flood
Ref. – Reference
RMR/GSI – Rock Mass Rating/Geological Strength Index
RTS – Reservoir Triggered Seismicity
SIA – Social Impact Assessment
SCS – Soil Conservation Service
SEE – Safety Evaluation Earthquake
SEP – Stakeholder Engagement Plan
SLR – SLR Consulting Ltd, France
TAM – Tube A Manchette
TBM – Tunnel Boring Machine
ToR – Terms of Reference
USBR – United States Bureau of Reclamation