TECHNO-ECONOMIC ASSESSMENT STUDY FOR ROGUN HYDROELECTRIC CONSTRUCTION PROJECT

PHASE II: PROJECT DEFINITIONS OPTIONS

Volume 3: Engineering and Design

Chapter 3: Design of alternatives

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1 INTRODUCTION

This chapter includes the description and justification of the design proposed by the Consultant for the nine Rogun project alternatives based mainly on the assessment of the HPI 2009 Rogun project.

Several annexes are appended to this report to elaborate in details the design approach followed by TEAS consultant in its own definition of alternatives.

2 SITE SPECIFIC FEATURES

2.1 Location within regional geological frame

The Rogun dam site is located in the central mountainous region of Tajikistan, where the dominant trends are high mountain ranges and intense folding of the sedimentary series under high tectonic stresses. The area is tectonically active, with major active faults at the very proximity of the site (regional Gissar-Kokshal Fault and Illiak-Vakhsh Fault, then the comparatively minor Ionakhsh and Gulizindan faults).

The dam site itself can roughly be divided into three parts:

- The sector located north of Ionakhsh Fault, where thrust movement along this major tectonic feature generated an asymmetrical syncline (Kirbich syncline)
- The thrust block limited to the north by the Ionakhsh Fault and Fault 35 to the south, where the main structures of the hydropower plant are to be located; bedding of the rocks has a similar attitude as Ionakhsh Fault, dipping steeply towards SSE,
- The area located downstream of Fault 35

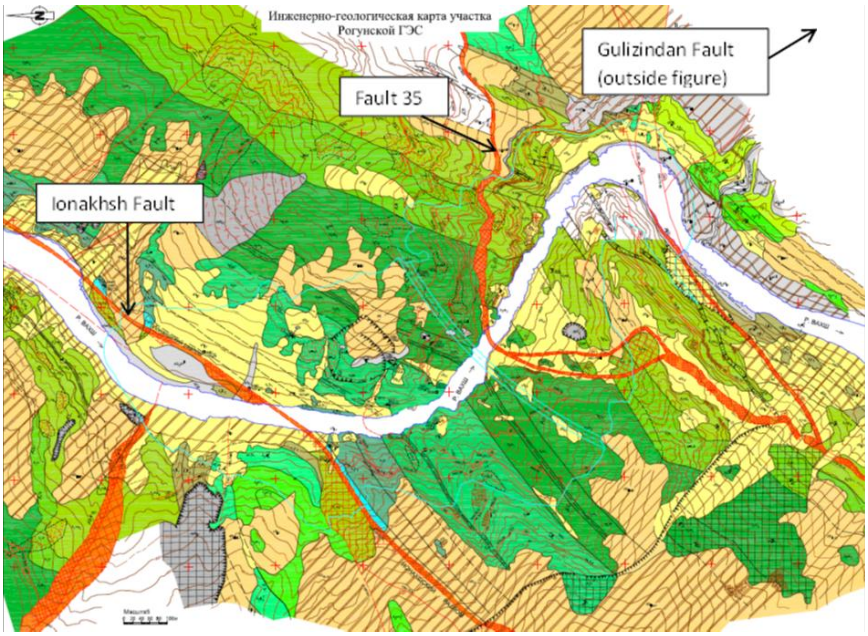


Figure 2-1: Identification of major tectonic features on the dam site.

2.2 Nature of rocks of the dam foundation

With regard to the nature of the rocks of the foundation, the sedimentary series of the dam site are essentially made of an alternance of less resistant claystones and siltstones, against more resistant sandstones, with diversely represented gypsum. Younger formations such as Upper Cretaceous and Paleogene additionally present strata of limestone, shales or chalk.

Distribution of the rock masses is as follows:

- Geological formations from Jurassic (Gaurdak) to Mingbatman make the foundation of the projected dam, on the south-east side of Ionakhsh Fault,

- Mingbatman Formation and younger one are present on the north-eastern side of Ionakhsh Fault, as well as further in the south-east, beyond the dam site, and within the “disturbed zone”.

2.3 Salt rock of Ionakhsh Fault

Salt rock pertaining to the Jurassic Gaurdak Formation is present along the two main thrust faults (Ionakhsh and Gulizindan), as well as diapirs along the Illiak-Vakhsh Fault, upstream of the dam site.

Given that Ionakhsh Fault is to be located below the Stage 1 dam (elevation 1140), and therefore under the upstream shell of the main dam, many investigations have been dedicated to the knowledge of the geometry of the salt wedge,

Geotechnical issues related to the presence of the salt wedge within the Ionakhsh Fault and the assessment and the impact of a possible dissolution are the object of the Phase 0 Report.

2.4 Geomorphological features of the dam site

2.4.1 General aspect

At the dam site, the Vakhsh River makes a sharp bend from a NE-SW direction parallel to the regional lithology, then turns to NW-SE around the downstream toe of the forecasted stage 1 dam, where it flows perpendicular to the bedding of the embedding rock formations. It finally turns back to the NE-SW direction at the downstream toe of the main Rogun dam.

The gorge at the dam site is V-shaped, with steep flanks of inclination from 40 to 60 degree with locally steep cliffs along the river stream, especially in the sandstones formations. Figure 2-2 gives an good idea of the topographic features of the site.

Figure 2-2: View of the gorge of Vakhsh River on the dam site.

With regard to geodynamics phenomena, rockfalls, collapses and landslides are intensively developed on and around the dam site.

Rockfalls are common, especially when raining, due to the steepness of slopes and differential erosion between the interlayered siltstones and sandstones.

2.4.2 "Disturbed zone" of right bank

The right bank of the Vakhsh River is characterized by a peculiar morphologic feature, whereas it presents, on the top, a large and relatively flat plateau at elevation 1700-1750. This very particular feature of the site is visible on Figure 2-3.

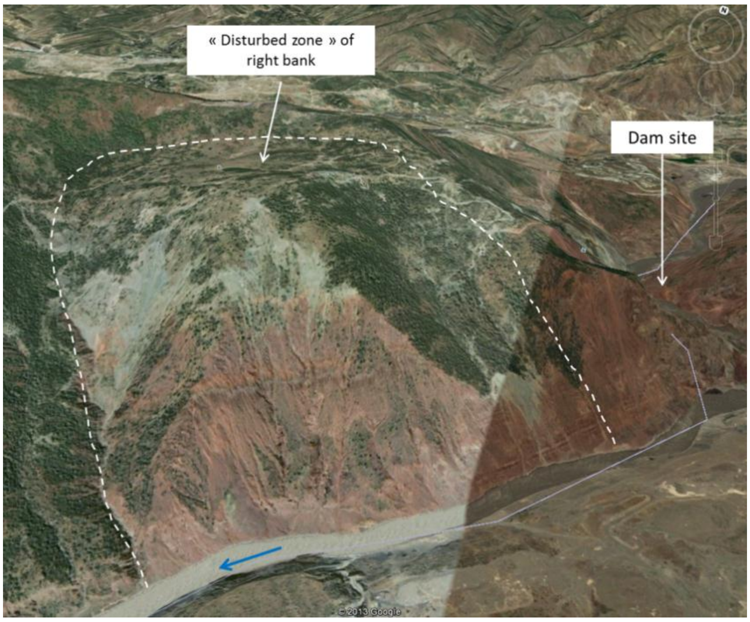


Figure 2-3: 3D view of the dam site (river gorge on the right hand of the figure), and the "disturbed zone" of the right bank with its approximate limits (from Google freeware)

This feature is clearly described in chapter 2 (Geology of the present report; Volume 2 Basic data).

It is however to mention the presence of large potentially unstable masses in the front part of this structure, at the foot of which large amount of debris produced by the scouring of the slope accumulate. The river bed of the Vakhsh River was moved off the right bank of 70 to 90 m between 1978 and 2005 due to this reason.

2.4.3 Mudflows

Due to the high tectonic activity and rapid rising of the relief along the tectonic structures, and also sometimes due to the presence of soluble rocks (gypsum, salt), slope instabilities are common in the catchment areas of the diverse tributaries of the Vakhsh River.

Especially, mudflows flowing down the Obi-Shur River, which joins with the Vakhsh River on left bank immediately downstream of the dam site, have already proved to present a subsequent risk for the power plant. Mudflows from the Obi-Shur River - occurring subsequently to the breach of the upstream cofferdam in the night of 8 to 9 May 1993 – temporarily dammed the Vakhsh River, resulting in the flooding of the major underground structures, machine and transformer cavern (the work had been already suspended during the previous year).

3 DAM

3.1 HPI dam design

3.1.1 Description

3.1.1.1 Design

The Rogun dam as designed by HPI (2009) consists of a zoned fill embankment 335 m high over foundation level. The total volume of the fill amounts to 71.7 Mm^3, 7.2 Mm^3 of which represent the impervious central core. The excavations sum up to a total volume of 4.6 Mm^3.

A general plan view of the dam is shown hereunder.

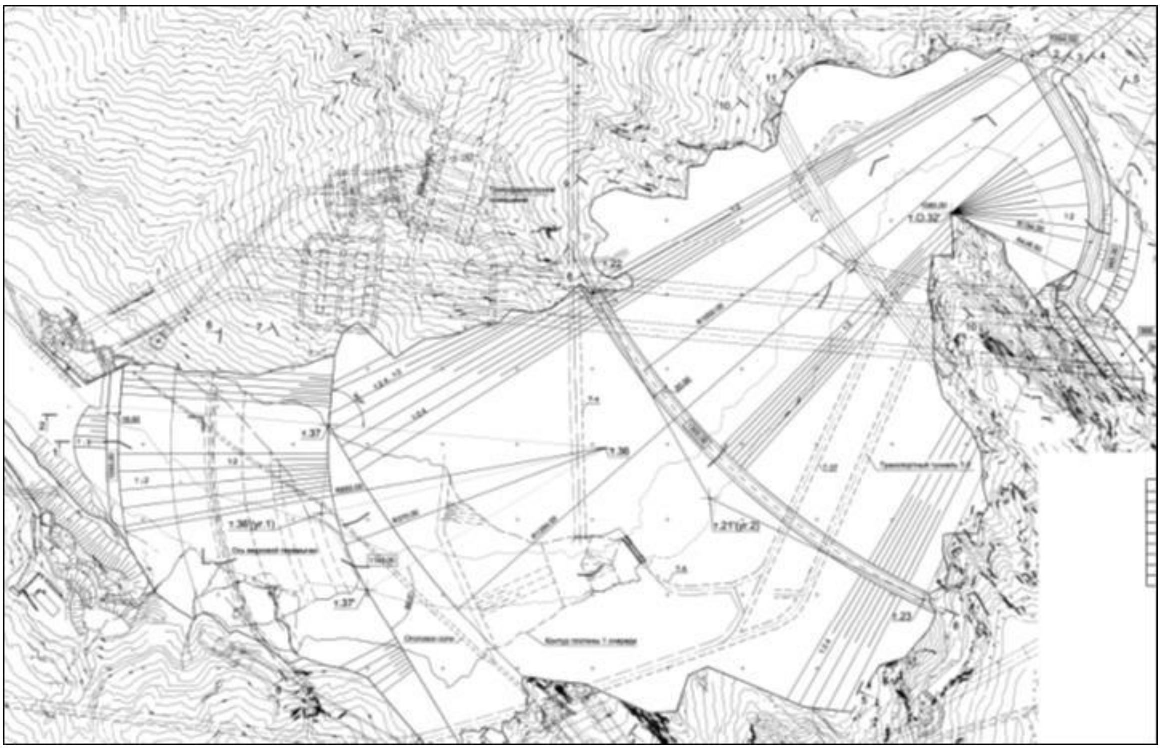


Figure 3-1 : Dam layout – source HPI

The dam axis is shaped as 1000 m radius circle. The crest level is 1300 masl, while the full supply level is 1290 masl. The crest width is 20 m and its length is 625 m.

Slope of the downstream face is 2H/1V. Above 1140 masl, the upstream slope is 2.4H/1V and under 1140 masl, the upstream slope is 2H/1V. In the upstream shell, there is a large (between 0 m on the left bank and 120 m on the right bank) risberm at elevation 1140 masl.

The material zoning inside the dam differs from one drawing to another, the two typical cross section found by the Consultant are reproduced hereunder.

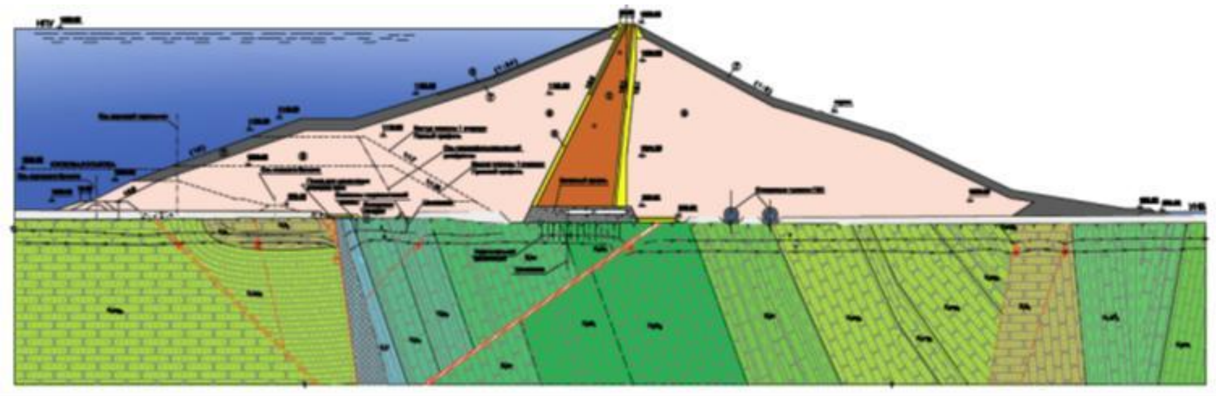


Figure 3-2 : Typical cross section of the dam - HPI 2010

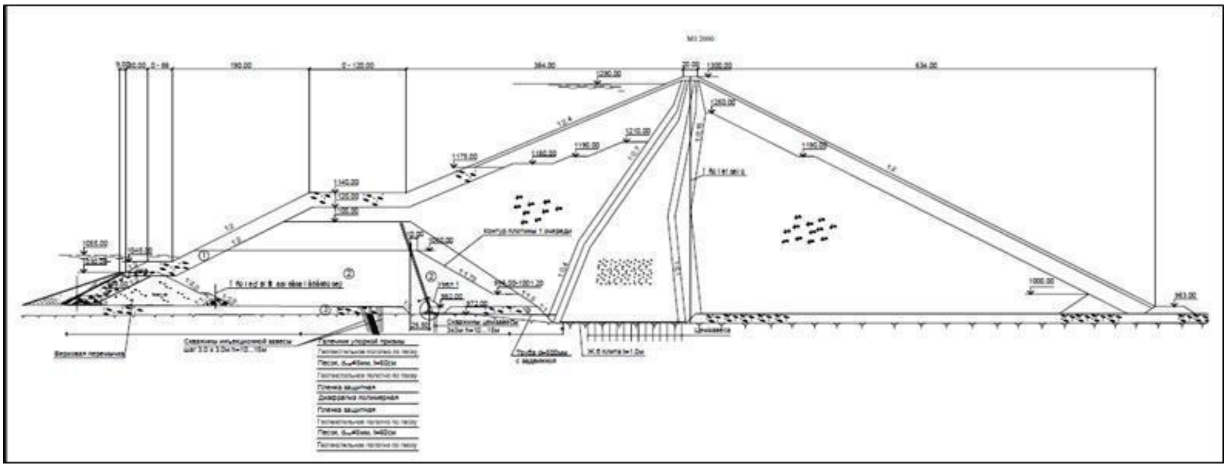


Figure 3-3: Typical cross section (extract from "Dam stability 3D modelling, Hydroproject, 2009)

On Figure 3-2, the thickness of the impervious core varies from 8 m at crest level to about 140 m at foundation level. The hydraulic gradient through the core is lower than 2.5 throughout. On Figure 3-3, the upstream part of the core is thin: under 60 m water head the core thickness is only 10 m.

The core setting out has been tailored so as to avoid the fault n°35 emerging in the valley bottom about 30 m downstream of the core toe.

Two galleries are set in the core at two different elevations from one bank to another.

The core is set on a thick concrete slab that fills the valley bottom and creates a large platform.

Fine and coarse filter zones border both faces of the core. On Figure 3-2, upstream both the fine and coarse filters are 4 m wide. Downstream the fine layer is also 4 m wide and the coarse layer is 10 wide. On Figure 3-3, it appears that all filter layers are 10 m wide.

A filter blanket is set downstream of the core to cover the fault 35 emerging on the ground.

The embankment shoulder and shell are made of random coarse alluvium, which is found in large quantity within the submerged reservoir area. The overall volume of random fill is estimated at 57.4 Mm^3.

Both upstream and downstream shells are protected by a 20 m thick rockfill layer. On Figure 3-3, this rockfill layer is getting thicker with elevation, and above elevation 1210 masl, the upstream shell is only made of rockfill.

A grout curtain around 100 m deep is set up in the foundation prolonging the impervious core alignment within the banks. The grouting works would be carried out from galleries excavated within the banks at regularly spaced levels over the whole dam height (6 different levels).

The stage 1 dam is embedded in the main dam upstream shell. Its upstream slope is shared with the main dam upstream slope. The Stage 1 downstream slope is 1.7H:1V. Its watertight component is a membrane.

3.1.1.2 Dam stability calculation

The Appendix 1 presents the complete analysis on the dam stability. The first paragraph presents the HPI documents made available to the Consultant on that topic and their assessment. Main elements are synthetized here.

The documents available contain: slope stability of the dam, 2D static and dynamic finite element model calculation, and 3D static and dynamic finite element model calculation.

The slope stability concludes that the dam respects the safety factor required. It shows also that the downstream slope of the stage 1 dam should not be smaller than 1.68H/1V to fulfill stability criteria. Pseudo static calculations have also been performed.

The 2D finite element (FE) analyses were developed in order to assess an order of magnitude of displacements and stresses in the structure during construction and operation conditions and to assess the dam dynamic behavior.

The construction of the dam was modeled by activation of horizontal layers. The maximum settlement was estimated to 5.6 m in the core of the dam between levels 1050 and 1070 masl. The maximum vertical stress reaches 1117 t/m² near the upstream toe of the core.

The 2D dynamic finite element model estimated the natural period of the typical cross section: 3s. The dynamic analysis consisted on 10 time history analysis, i.e., the acceleration at the base of the model is imposed by a signal. All the acceleration records had a peak acceleration of 5.4 m/s², ie 0.55 g. The residual displacements generated by the 10 cases were all below 60 cm in both vertical and horizontal direction.

The 3D finite element model has been developed following the same procedure than for the 2D case. The construction of the dam was modeled by activation of horizontal layers. The maximum settlement was estimated to 4.4m, against 5.6m for the 2D model. The maximum vertical stress reaches 978 t/m², against 1117 t/m² for the 2D model.

For 3D model the natural period of oscillation was estimated to 2s (against 3s for the 2D analysis). Here, only the most critical earthquake was applied to the model. As in the previous 2D case the peak acceleration was of 5.4 m/s, ie 0.55 g. Peak acceleration at crest induced by the earthquake was estimated to 8.2 m/s², ie 0.84 g.

3.1.1.3 Dam material

The quantities defined by HPI, 2010 are detailed in the table below:

Table 3-1: Materials quantities. HPI, Design 2010

Dam part	Characteristic size	Quantity
[-]	*[mm]*	*[m³]*
Core	-	7 247 000
1st transition layer	0 – 10	4 893 000
2nd transition layer	0 – 40	
Upstream lower transition	0 - 80	
Alluvium shoulder	≤700	39 567 000
Rock shell	≤700	17 753 000
Rip rap	300 – 1000	1 497 000
Concrete slab	-	481 000
TOTAL		**71 438 000**

The following table summarizes the available quantities of material to be placed in the dam.

Source (Quarry/Borrow area)	Total initial quantities	Extracted quantities	Stockpiled quantities	Remaining quantities	Suitable quantities for dam (alluvium shoulders, core, rock shell, rip rap)	Needed quantities for dam	Suitable quantities for concrete aggregates
	[Mm³]	[Mm³]	[Mm³]	[Mm³]	[Mm³]	[Mm³]	[Mm³]
Borrow area 15	75.6	26.6	22.1	49	64.7	Alluvium shoulders : 43,0	Suitable for concrete aggregates after processing
Lyabidora	6.6	4.0	4.0	1.0	5.0	Transitions : 5,6	-
Borrow area 17	17	2.5	2.5	14.5	17.0	Core : 6,9	-
Quarry 26 A and 26B	5.2 18	0.8	0.8	22.4	23.2	Rock shell : 17,3 / Rip rap : 0,6	-

Figure 3-4: Source of materials. Definition of quantities.

3.1.1.4 Foundation treatment

The foundation treatment foreseen by HPI is described here under for each of the construction phase: pre-cofferdam, cofferdam, stage 1 and main dam.

For the pre-cofferdam, no specific foundation treatment is foreseen.

For the cofferdam, only bank instabilities and over hanging rock will be removed to secure the working area. The riverbed alluviums are let in place but a jet grouting is performed to ensure its watertighness.

For the Stage 1 dam, the membrane foreseen by HPI is anchored in a concrete slab. Under the slab, a grout curtain of 30 m depth reaches rock zone III, and a consolidation grouting of 6 m depth is foreseen. The base of the concrete slab (10 m width) is cleaned, cavities and cracks filled with concrete and rock instabilities are removed, but no rock excavation is foreseen. Under the Stage 1 dam shells, the river bed alluviums are let in place.

For the final dam core, the riverbed alluviums are removed and as well as the poor rock layer (zone I) and replaced by concrete. The consultant had no information on what excavations are foreseen by HPI in the core abutment: a volume of 1.6 Mm3 of rock excavation is indicated in the construction schedule but no drawing has been found to describe the location and geometry of such volume.

The final dam grout curtain and consolidation is foreseen below the dam core. The grout curtain is meant to reach the rock Zone IV (intact rock), ie a depth of 60-100 m.

In the 1978 project a drainage curtain was also foreseen under the dam core.

3.1.2 Comments

3.1.2.1 Design

As a general comment, the design appears safe and safety-conscious for a project which sets a world record.

However it shall be noted that:

- no information or drawing have been found by the Consultant on the excavation to be carried out for the core foundation. The excavations entailing the adjustment of the local topography are quite substantial (see Figure 3-5). Such fine-tuning excavations are compulsory so as to limit differential settlement of the fill and internal stress distribution, and avoiding any risks of hydrofracturation through the core. This was incorporated in TEAS design.

Figure 3-5 : Central core area excavations

- Two inspection galleries crossing the impervious core from bank to bank and extending into the dam foundation are designed at El. 1120 & 1240. Such galleries are not standard practice. The existence of rigid elements (concrete galleries) within a plastic medium (core material) subject to deformations may lead to a redistribution of stresses within the core with risks of hydrofracturation. Internal deformations of the core may be significant either during the construction stage (calculated settlement are of the order of 3 m) or during operation under seismic load (calculated displacement are of the order of 0.5 m). Such deformations are not coherent with the existence of rigid structures within the core.

- The Stage 1 watertight membrane is not acceptable. To our knowledge the arrangement is a "première" in the world for a structure of this height. Impervious membranes are usually used for repair works of existing structures, particularly for concrete ones. In such cases membranes are applied to the upstream face of the concrete works in order to replace the original watertightness which has failed.

The use of such membranes in fill dams, which are deformable, subject to significant settlements during construction, is not standard practice. Some existing examples refer to cofferdams, i.e. temporary structures under full pressure conditions limited in time to flood duration. No reference is in our records of an embankment higher than 100m including a membrane as internal watertight mean.

The use of an impervious membrane entails moreover delicate contact problems with the foundation. The arrangement adopted for Rogun consists in a concrete slab anchored to the rock: the membrane is fixed to the concrete with metal profiles: such system creates to a rigid link between the membrane and the slab (see the Figure hereunder).

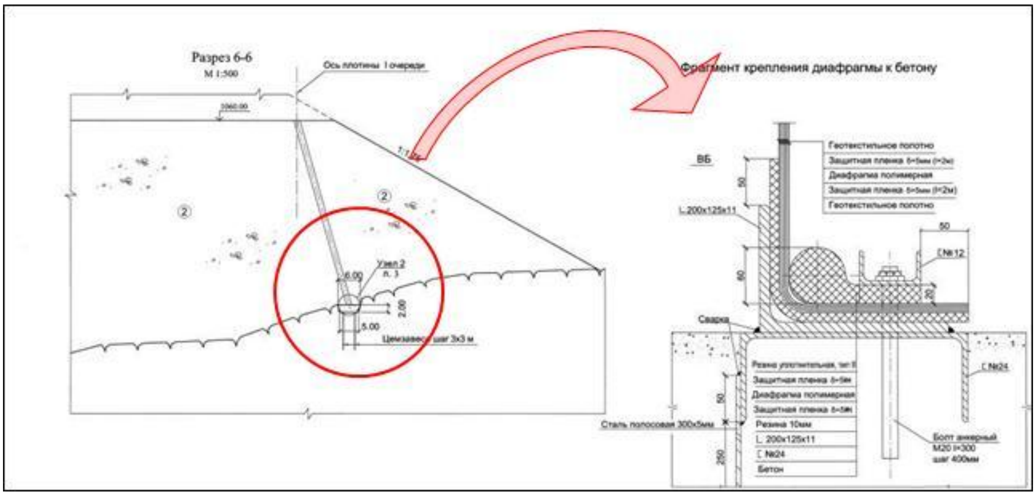


Figure 6.7.2 – Internal watertight membrane - Details

During construction the embankment shell and shoulder undergo settlement by the load of the placed fill. Due to the peculiar topography of the site and of the steep slope of the natural ground, such settlements will be centered close to the banks and the differential settlements will be significant at the contact with the concrete slab (see the sketch hereunder). The phenomenon would involve a major risk of rupture of the membrane along the bank abutments, with potential leakage through the shell and shoulder. The direct consequence would be the risk of internal erosion.

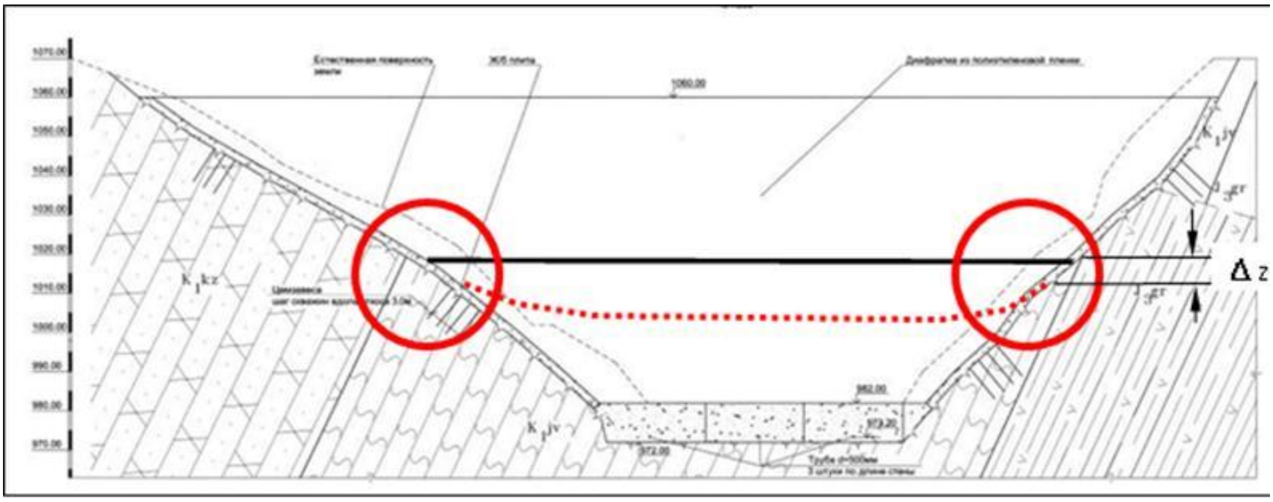


Figure 3-6 : Settlements

- No information has been given to the Consultant about the monitoring system to implement on the final dam. The monitoring system (number, location and type of instruments) is one of the most important features of the dam in operation; it is the only way to appreciate the dam behavior after completion and is therefore mandatory. This was incorporated in the TEAS proposed alternatives.

3.1.2.2 Dam stability calculation

In general, not enough information is provided to make a real assessment of the available stability studies.

Nevertheless, it can be noted that the permanent dam displacements found in the 2D finite element analysis appear quite small (below 1 m) and should be further refined.

3.1.2.3 Dam material

The following table summarizes the corrective processes to be implemented in order to let those materials satisfy the technical specifications as laid down by HPI. A list of subjects requiring further studies is also given.

Source (Quarry/Borrow area)	Processing / Treatment to bring to specifications
Borrow area 15	Remove materials > 700 mm which represents about 2-3% (for alluvium shoulders)
Lyabidora	Remove boulders > 100 mm which represents about 13 - 16 % (for transitions)
Borrow area 17	Reduce moisture content to 10-12 %. Remove materials > 200mm. Increase fine content (*)
Q 26 A and B	Physical and mechanical properties are to be tested and defined precisely.

3.2 Proposed design

The main dam design proposed by the TEAS Consultant is presented on drawings n°40-101, 102, 201, 202, 301, 302. It is based on the HPI design and adapted according to the Consultant assessment.

The Rogun site is very tight and a lot of constrains impose the lay out of the dam:

- The existing diversion tunnels intakes: the upstream dam toe has to be set downstream of those intakes.
- The Ionaksh fault: the main dam or Stage 1 dam watertight component should not cross the Ionaksh fault, and should be set downstream of it. This aims at limiting the pore pressure gradients across the Ionaksh fault which could increase salt dissolution.
- The fault 35: the main dam core should not cross this fault to avoid differential movement and shearing within the core.

Therefore, the Consultant has not changed the axis of the dam and kept it as planned by HPI for the three dam alternatives. The slightly curved axis is also kept as designed by HPI.

For each alternative, the dam crest is set 10 m above the FSL, while the core crest is set 3.75 m below the crest. Those 3.75 m are the result of a geometrical construction that aims at ensuring:

- the core protection against weather conditions (snow and rain infiltrations);
- sufficient layer thickness to ease the construction and material placement;
- symmetry of the dam crest to ease the camber design and construction.

The freeboard requirements for crest and core elevation includes considerations about high floods, GLOFs, waves, and earthquakes. The phenomena combinations considered are detailed in the following table.

	High floods	GLOFs	Waves due to wind	Earthquake
1.	PMF	no	no	no
2.	10 000 years flood	no	yes	no
3.	no	no	no	MCE
4.	no	yes	no	no

Table 3-2 : Freeboard combinations

The appendix 6 presents the detailed evaluation of the freeboard against waves. The next table summarizes the verification of core crest and dam crest elevation with respect to freeboard consideration. It shows that with the proposed design, all requirements are met.

Regarding alternative 1290 and 1255, the dam crest is set 10 m above the FSL, while the core crest is set 3.75 m below the crest.

Regarding 1220, the dam crest is set 11.5 m (=10 + 1.5 m according to the conclusions of PMF) above the FSL, while the core crest is set 3.75 m below the crest.

According to the case FSL 1255 masl - 2, the water elevation is higher than the criteria defined (1265 – 3.75 = 1261.25 m masl). However, this situation (75 cm above the crest, 1262 m asl– 1261.25 m asl = 75 cm) can be managed temporary.

According to the case FSL 1220 masl - 1, the water elevation is higher than the criteria defined (1231.5 – 3.75 = 1227.75 m masl). However, this situation (14 cm above the crest, 1227.9 m asl– 1227.75 m asl = 14 cm) can be managed temporary.

Dam Alternatives	Combinations	High floods (max water elevation, masl)	GLOFs (freeboard, m)	Minimum core crest (m)	Waves due to wind (freeboard, m)	Earthquake (provision for dam settlement during to earthquake, m)	Minimum dam crest (masl)
FSL=1290 masl	**1.**	1291.9	0	**1291.9**	0	0	**1291.9**
	2.	1278.9	0	**1278.9**	1.88	0	**1280.78**
	3.	0	0	**1290**	0	8	**1298**
	4.	0	2.8	**1292.8**	0	0	**1292.8**
FSL=1255 masl	**1.**	1262.0	0	**1262.0**	0	0	**1262.0**
	2.	1232.5	0	**1232.5**	1.87	0	**1234.37**
	3.	0	0	**1255**	0	8	**1263**
	4.	0	4	**1259**	0	0	**1259**
FSL=1220 masl	**1.**	1226.4	0	**1226.4**	0	0	**1226.4**
	2.	1227.9	0	**1227.9**	1.85	0	**1229.75**
	3.	0	0	**1220**	0	8	**1228**
	4.	0	6	**1226**	0	0	**1226**

Table 3-3 : Freeboard verification

The upstream and downstream slopes of the main dam are kept as designed by HPI for the three dam alternatives (2H/1V downstream and 2.4H/1V upstream above 1140 masl and 2H/1V upstream under 1140 masl). The stability analysis performed showed that those slopes were sufficient to ensure the stability of the dam.

The next figures present the typical dam cross section and layout for the highest dam alternative (FSL=1290 masl).

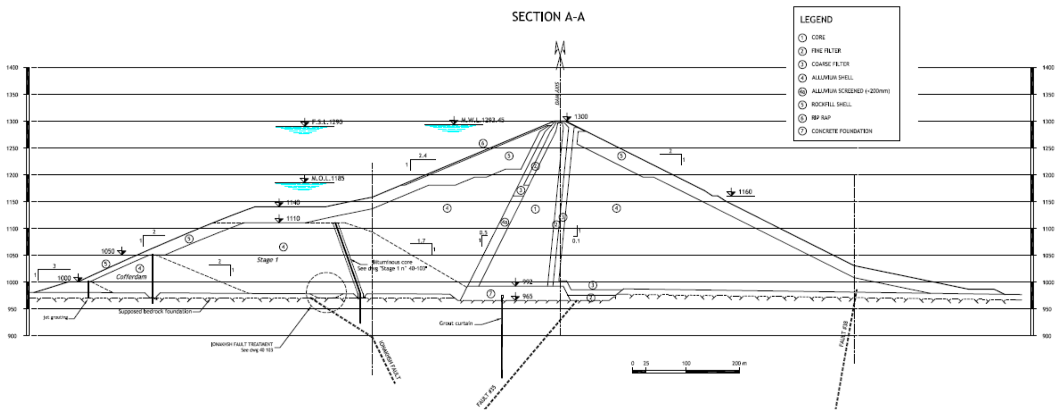


Figure 3-7: Dam cross section. Alternative FSL=1290 m.a.s.l.

The material zoning recommended by the Consultant includes, from upstream to downstream:

- A rip rap on the upstream slope above the minimum operating level;
- A rockfill layer of minimum 20 m thick;
- The random alluvium material;
- The coarse filter 10 m thick;
- The fine filter 10 m thick;
- The core

More information is given on the various material characteristics in Appendix 1.

The rockfill layer is minimum 20 m thick; in the upper part of the dam this layer thickness increases to cover the complete upstream and downstream shell. It has been shown in the stability analysis (Appendix 2) that the highest 50 m of the dam are the most critical area during an important earthquake. Therefore, the Consultant prefers setting a material with a higher friction angle such as rockfill in this area.

The filters and transition thickness are chosen on the basis of the stability analysis and horizontal permanent displacements estimated based on the stability calculation results (Appendix 2) as well as on the experience. 10 m thick filters ensure keeping the material continuity even in case of large earthquake and horizontal shearing.

All along the downstream face of the core and along the upstream face between the crest and the Minimum Operating Level, the 2 layers of filters are set: fine filters against the core and coarse filter against the alluvium. The filter layers aim at avoiding fine transportation from the core to the shells.

Under the minimum operating level, the two filter layers are not necessary and the layer (4a) is there only to ensure a transition between the core and the alluvium.

The filters and transition layers are also set on the foundation in order to cover the fault #35 and avoid fine material transportation along the fault.

A layer of alluvium is necessary between the coarse filter and the rockfill to ensure the filters and transition function.

The alluvium layers are considered as pervious. Therefore, a drain layer in the valley bottom is not necessary: the potential core leakage will be driven downstream of the dam through the alluvium shell.

The core is inclined downstream, minimum 8 m thick on the top. The upstream core face is inclined downstream with a slope of 0.5H/1V. The downstream core face is inclined downstream with a slope of 0.1H/1V. The slope sum is then 0.4H/1V.

3.3 Dam stages

Rogun dam construction is phased in several construction steps: pre-cofferdam, cofferdam, Stage 1 dam and Main Dam.

3.3.1 Pre-cofferdam and cofferdam

The pre-cofferdam is used to start the river diversion, and is made of large blocks thrown in the river from the banks and then completed by random filling material which ensures the water tightness of the structure.

The cofferdam is made of the same alluvium and rockfill as the main dam. Its crest elevation is 1050 masl, ie 15 m higher than HPI design. This is justified in §4.3.1. Slopes are 2H/1V both upstream and downstream which are sufficient given the material used (that has an internal friction angle between 39° and 42°) and given the temporary life span of this structure. The fill material will be compacted.

The cofferdam shall be completed before the month of high river discharge (June and July). The cofferdam volume being 2.27 Mm^3, with a material placement rate of 0.3 Mm^3/month, this condition is fulfilled.

Nevertheless, to fit with this schedule, the watertight component placement should not slow down the cofferdam construction. Vertical central bituminous core: it is set as a central vertical core, founded on concrete plinth on each bank and in the river bed. With this solution the river bed alluviums have to be removed in the core vicinity to anchor the plinth on rock. Bitumen is a plastic material than will adapt to the differential settlement between the cofferdam core and the banks.

The bituminous core solution is the safest but it required to remove all river alluvium to be anchored on the rock. Therefore, in the TEAS design the bituminous solution is adopted.

3.3.2 Stage 1

3.3.2.1 Introduction

The Rogun H.P.P. was studied by the Hydro-Project Institute (HPI) of Moscow, with a full supply level (FSL) set at elevation 1290 m asl. According to the Terms of Reference of the TEAS two other dam height alternatives are to be assessed together with the HPI's design.

The two other full supply levels to be evaluated have been established at elevations 1255 m asl and 1220 m asl.

The Stage-1 dam constitutes an intermediate stage of the main dam, allowing for early energy generation while the construction of the dam and other project components continues to its end.

The stage 1 dam design is presented on drawing n° 40 103, 40 203 and 40 303. It is made of the same alluvium and rockfill as the main dam. The Consultant kept the same slope as HPI design, ie 2H/1V upstream and 1.7H/1V downstream. As shown in Appendix 2, those slopes have found to be sufficient to ensure the Stage 1 stability during its limited life span.

The axis of the Stage 1 dams are defined by several constrains listed below:

- The stage 1 core have to be set always downstream of the Ionaksh fault to avoid pore pressure gradients across the fault which could lead to a salt dissolution increase;
- The downstream slope cannot be steeper than 1.7H/1V for stability reasons as shown by HPI in their stability calculations (see Appendix 2);
- The Stage 1 downstream shell should not overlap the final dam core footprint;

Because of the above listed constraints, the bituminous core is slightly inclined toward upstream with a slope of 0.2H/1V. This slope is necessary to limit the overlapping of the Stage 1 downstream toe and final dam core footprint.

The Consultant introduced a slight change of Stage 1 axis for the highest alternative, because of the Ionakhsh fault and Stage 1 core crossing each other. As shown in the next figure, the new axis is such that it never crosses the fault at any elevation.

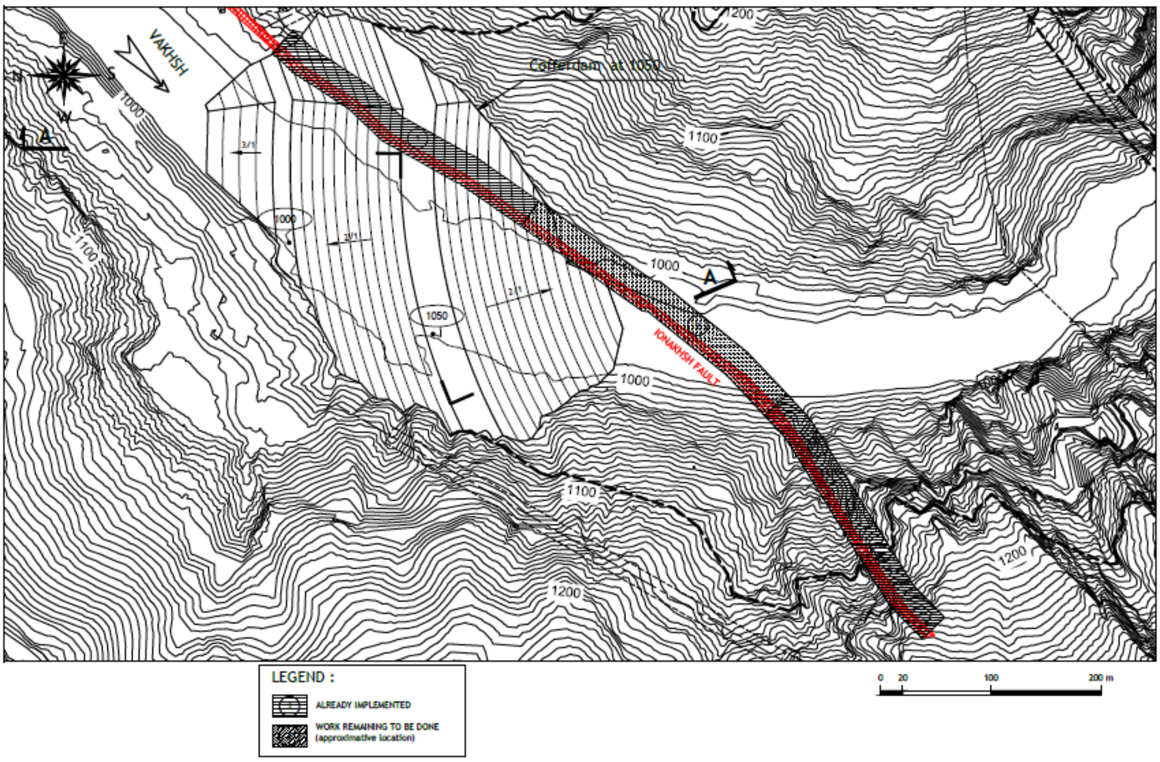


Figure 3-8: Ionakhsh fault and Stage 1 dam.

It is to be emphasized that, for the highest dam alternative, the Stage 1 dam layout is very constrained and does not include any degree of freedom whereas the topography and the Ionaksh fault cannot be located with a lot of precision.

For the lowest dam alternatives, the final dam core footprint is smaller; therefore, the constraints are not as restricting as for the highest alternative. A minimum distance of 10 m has even been considered between the Stage 1 dam toe and final dam core excavation area.

The Consultant chose to replace the membrane by a bituminous core because of all the reasons explained in §3.1.2.1. The bituminous core is 80 cm thick, anchored on a concrete plinth itself anchored on the rock. Two filters layers are set on each side of the core.

In the following, the concepts and the results of the preliminary evaluation of the stage-1 dams corresponding to the two other main dam height alternatives to be considered are introduced.

3.3.2.2 Stage 1 alternatives description

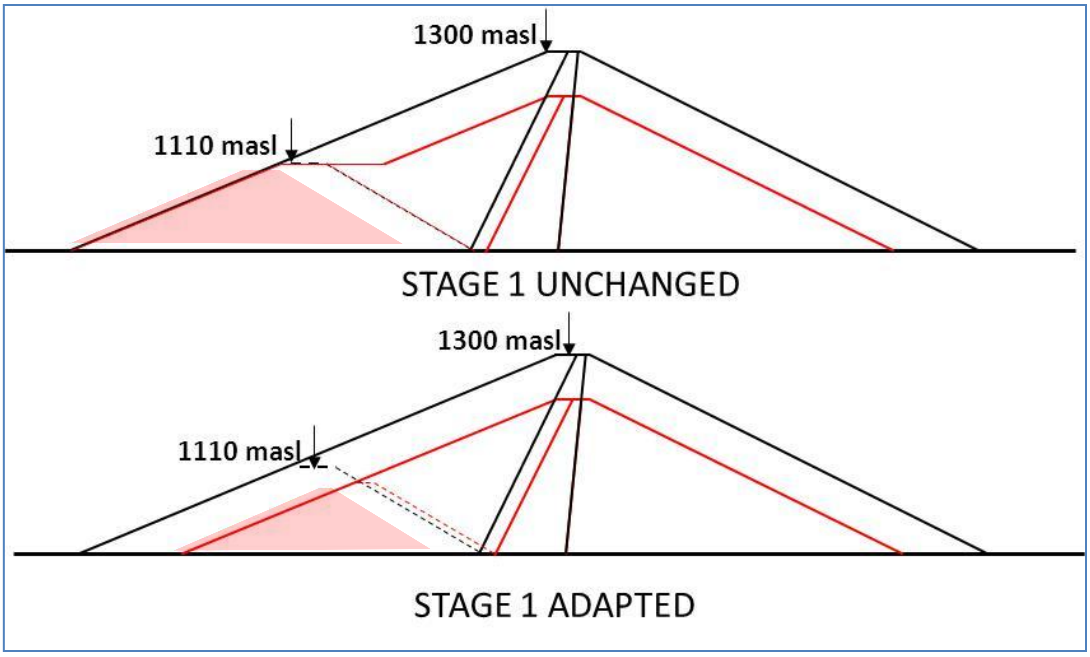
General

Two main design concepts have been studied for the alternative stage-1 dams:

- The “unchanged Stage-1 dam”: the height and slopes of the Stage-1 dam as well as its location remain the same as the Stage-1 dam designed by HPI for the dam with final FSL at El. 1290: the crest of the stage-1 dam remains at elevation 1110 masl, and lies at the same location as in HPI’s.

- The “adapted Stage-1 dam”: the concept is to design the highest dam that fits into the upstream shell of the final dam. It leads to a lower crest level and consequently to a smaller final dam volume.

The two main concepts are illustrated in the next figure. Note that the upstream and downstream slopes of the dam are kept unchanged.



For the final dam at FSL = 1220 m asl

For the final dam at FSL 1220 masl, the layout constrains of the site such as the location of the main dam core, the location of the Ionakhsh fault and the location of existing diversion tunnels intakes allows for unchanged or an adapted Stage1.

The next figure shows the 3D model of both alternatives: the red area represents the dam part common to the two alternatives, the green area represents the unchanged Stage1, and the purple one represents the adapted Stage1.

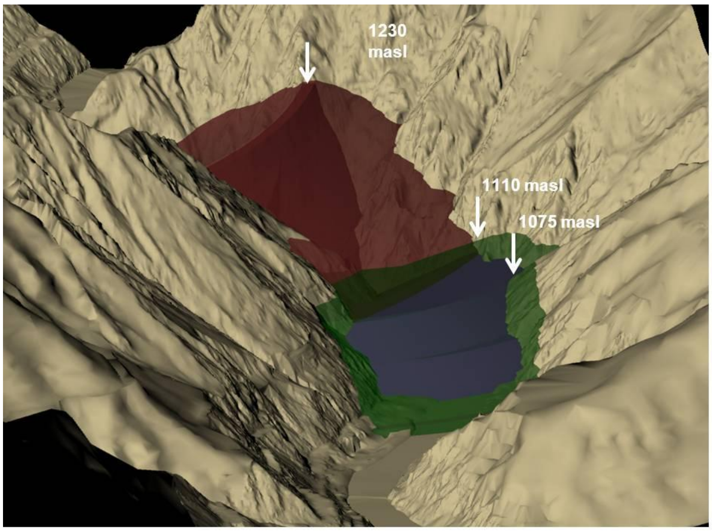


Figure 3-9: 3D representation of both alternatives - FSL = 1220 masl (downstream view)

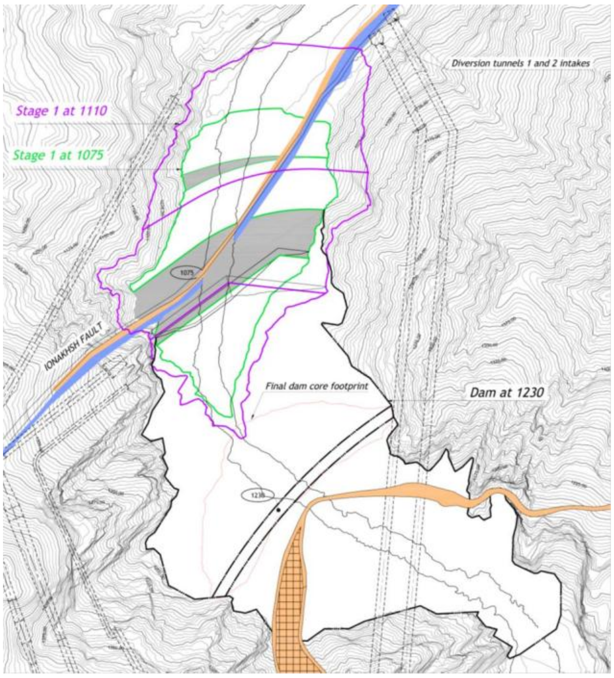


Figure 3-10: Plan view of both alternatives - FSL = 1220 masl

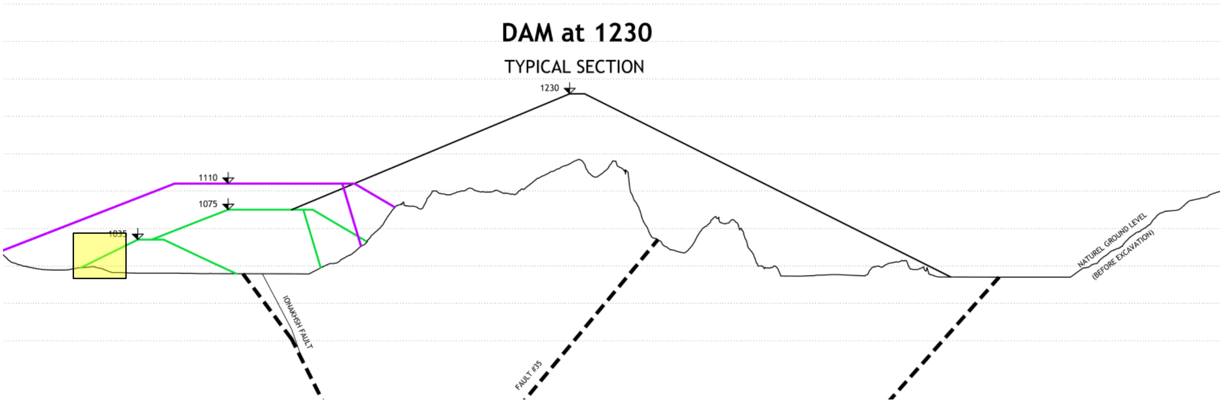


Figure 3-11 : Section of both alternatives - FSL = 1220 masl

The « unchanged Stage 1 » leads to a final dam presenting a very large platform at 1110 masl : its length varies from 200 m to 225 m . As for the 1290 masl dam, the upstream toe of the dam is located just downstream of the diversion tunnels intakes.

The “adapted Stage 1” has a crest at 1075 masl, i.e. 35 m lower than the unchanged Stage1. The upstream toe of the dam is located 150 m downstream of the diversion tunnels intakes and the platform at 1075 masl is limited to 100 m.

The “adapted Stage 1” downstream toe is located 20-25 m upstream of the final dam core footprint, while the “unchanged Stage 1” downstream toe is slightly overlapping the final dam core.

The total volume of both solutions has been calculated and is presented in the following table.

Final dam volume with an unchanged stage 1	**41 Mm^3**
Final dam volume with an adapted stage 1	**35 Mm^3**
Difference	**-15%**

Table 3.4 : Final dam volume with a FSL at 1220 masl

<u>For the final dam at FSL = 1255 masl</u>

For the final dam at elevation 1255 masl, the same layout constraints (location of the main dam core, of the Ionakhsh fault and of the existing diversion tunnels intakes) of the site lead to the possibility of adopting a layout of the dam with both an unchanged and an adapted Stage1 as followed.

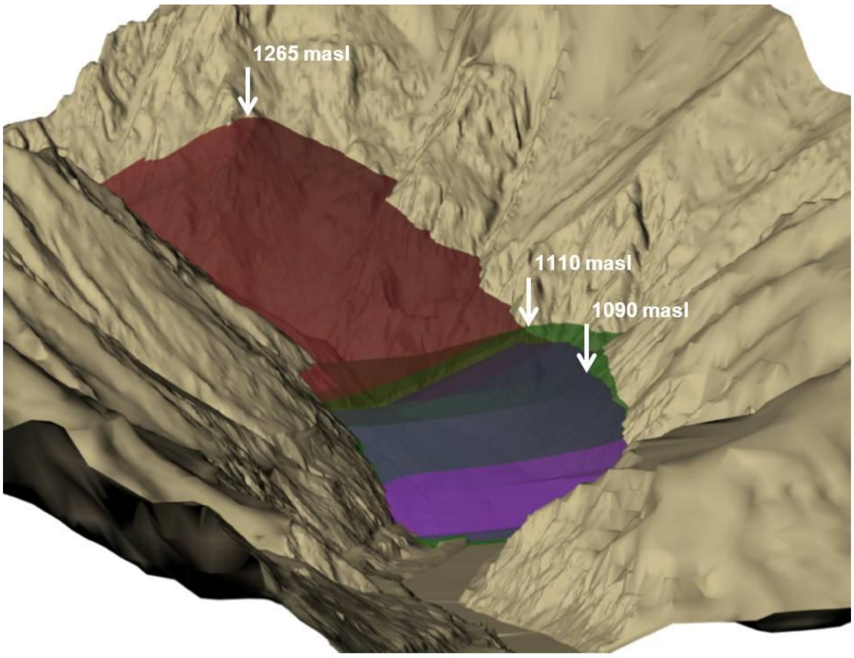


Figure 3-12 : 3D representation of both alternatives - FSL = 1255 masl (downstream view)

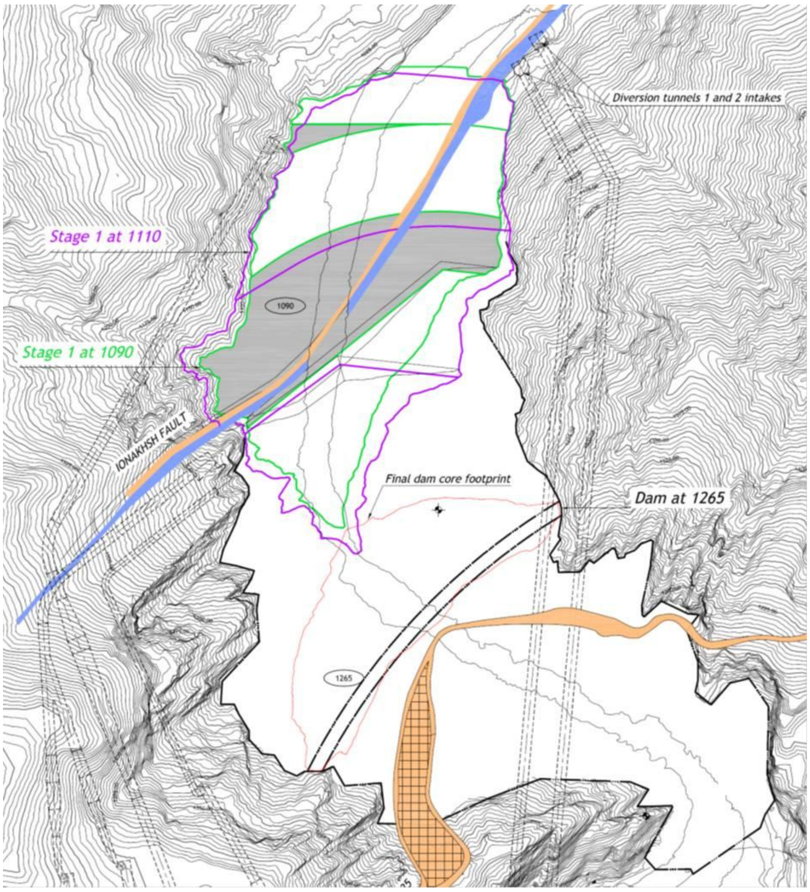


Figure 3-13 : Plan view of both alternatives - FSL = 1255 masl

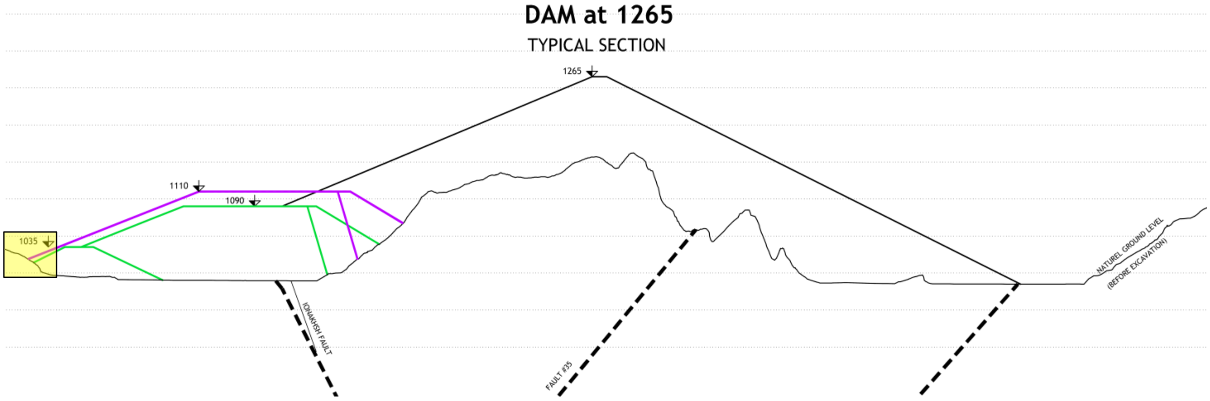


Figure 3-14 : Section of both alternatives - FSL = 1255 masl

The « unchanged Stage 1 » and the “adapted Stage 1” are very similar: as for the 1290 masl dam, the upstream toe of the dam is located just downstream of the diversion tunnels intakes for both alternatives.

The “adapted Stage 1” has a crest at 1090 masl, i.e. 20 m lower than the unchanged Stage1.

The only interesting difference between the two alternatives is the fact that with the “adapted Stage1”, the downstream toe of the Stage 1 is not crossing the final dam core.

The total volume of both solutions has been calculated and presented in the following table.

Final dam volume with an unchanged stage 1	55.6 Mm^3
Final dam volume with an adapted stage 1	54.5 Mm^3
Difference	-2%

Table 3.5 : Final dam volume with a FSL at 1255 masl

3.3.2.3 Implementation schedule

Based on the difference in volume between the unchanged stage 1 dam and adapted stage 1 dam for both FSL 1220 masl and 1255 masl alternatives, there will be a reduction in placement time necessary to reach the level required for early generation. This reduction will be significant for the 1220 masl alternative.

3.3.2.4 Energy production

The most important issue concerning the energy production during construction is the winter energy.

The winter energy is significantly increased as soon as Rogun can be used as a regulation device: at Rogun itself because the discharge has been increased, and also at Nurek because it can remains at its full supply level during all winter increasing the head and so the energy.

The regulation at Rogun can be started as soon as the reservoir level reaches the minimum level allowing final turbines to operate, ie 1130 masl and 1120 masl for alternatives FSL 1255 masl and 1220 masl respectively.

These elevations are reached sooner with the “adapted Stage 1” in all cases due to the reduction in overall volume of the intermediate dam.

3.3.2.5 Conclusions

The present note showed the advantages and drawbacks in terms of investment (volume of material to place) and energy production to adapt the Stage 1 design for the dam alternatives (FSL = 1220 and 1255 masl).

	FSL = 1255 masl	**FSL = 1220 masl**
Reference Stage 1 unchanged	Stage 1 adapted	Stage 1 adapted
Investment (material placement)	- 2 %	- 16 %
Energy production	negligeable	significant

For the final dam with FSL 1255 masl, The saving in construction is low as well as the gain in energy production. However, the impact being positive it is recommended to adopt the adapted stage 1 solution for the final dam with a FSL of 1255 masl, i.e. crest level at 1090 masl.

For the final dam with FSL 1220 masl, it is interesting to adapt the design in order to save time during construction and produce more energy in the construction period. The Consultant therefore recommends adopting the adapted stage 1 solution for a final dam with a FSL of 1220 masl i.e. stage 1 crest level at 1075 masl.

3.4 Dam material

Source of materials

In the beginning of the project, several quarries and borrow areas were preselected to provide the materials needed for dam construction. Since that time, and after more analysis of the materials, some of these quarries have been considered not suitable by previous designers.

Currently, four quarries/borrow areas are considered suitable and adapted with respect to specifications and constraints of the project:

- Borrow area 15 mainly for alluvium shoulders and transition and filter material,
- Stockpiles from Lyabidora borrow area to be used for transition and filters,
- Borrow area 17 for the dam core,
- Quarry 26 for rock shell and rip rap.
- Concrete aggregates are proposed to be processed from materials of borrow area 15.

Assessment of material quantities

The required materials quantities are detailed in Table 3-6 for each alternative.

Table 3-6: Material quantities in situ [m^3], TEAS.

Material / Alternative		Alt. 1290	Alt. 1255	Alt. 1220
1	Alluvium shell	43,063,864	33,182,921	18,924,605
2	Rockfill shell	17,365,059	12,475,052	9,352,361
3	Bituminous core	23,704	20,148	17,778
4	Core	6,992,490	5,104,518	3,714,728
5	Fine filter	2,466,655	1,350,195	747,638
6	Coarse filter	3,154,955	2,033,519	2,000,334
7	Rip rap	554,675	368,629	302,589
TOTAL		**73,621,402**	**54,534,982**	**35,060,033**

Conclusion and Recommendations

The volumes needed for the dam are available in quarries / borrow areas and associated storages. The filters materials, in priority, are to be used from stockpiles already available after extraction from borrow area of Lyabidora. The volumes are however not sufficient in this stockpile, and the missing volumes are to be extracted from borrow area 15, and processed in order to meet the specifications for filters. Specific care shall be given to timely extraction of material from BA 15 as this borrow area is bound to be flooded at the early stages of the construction.

The necessary quantities of concrete aggregates are covered by the exceeding materials from borrow area 15, which presents a large grading, adapted for concrete aggregate purpose, and by considering a specific treatment and selection of suitable materials.

Concerning core materials, a comprehensive analysis on the impact of fine content on watertightness is awaited in order to fix the required fine content, and adapt the processes needed to meet these specifications. Based on its experience, the TEAS Consortium considered that a conservative approach is to be adopted for this Feasibility Study. , Therefore, it has been considered in the cost estimate that a mixing of borrow area 17 materials with fine materials was done in order to increase fine content, and this for the whole material of the dam core. Fine materials have been identified in sufficient quantities from different sources.

Moisture content of borrow area 17 is also a point of concern, and the moisture control has been taken into account in cost estimate by considering special storage conditions.

The studies of construction materials and associated studies revealed the need of a comprehensive campaign of testing of all materials in both laboratory and in situ conditions at the next stage of design. It was also understood through the meetings with the Client that studies about this topic had been initiated. The best moment to carry out these tests is before tendering for the reasons cited here above. The cost of such a campaign remains low compared to the project total cost, and may represents a very positive input for the further steps of the Rogun project.

3.5 Foundation treatment

The dam foundation condition has been assessed in Geotechnical Report (Phase 2 report, Vol1, Ch3). The following is highlighted:

- An extensive and comprehensive campaign of scaling or supporting of the rock masses over the work site is to be engaged;
- Alluvium deposits in the river bed have not been observed, therefore the final decision to let in place or remove it, is to be made after river diversion. It will mostly depend upon their degree of compacity and proportion of fine elements (which should be comparable as dam shoulder material); It will anyway have to be removed from the core foundation (concrete slab);
- Excavation for the main dam core foundation is to be performed to reach down to the sound rock foundation (Zone III). Considering the sensitivity to weathering of the siltstones, it is recommended to shotcrete the excavations to avoid scouring and weathering. Where possible, leaving a layer of 1 m thick over the designed foundation level to be removed just before placing the dam body material would be advisable;
- a grout curtain is necessary to limit water seepage flows through the foundation;
- Contact and consolidation grouting is to be performed below the dam core in order to restore the properties of the rock foundation which will be altered by the blasting works;

The consultant intended design for the foundation treatment are explained here under for each of the dam stages: pre-cofferdam, cofferdam, Stage 1 and main dam.

For the pre-cofferdam, which is a small (10 m height) and temporary structure, the river bed alluviums are not removed. This structure is used to start the river diversion and dry out the river bed. Therefore, infiltrations through the river alluvium made of sand, gravel, pebbles and boulders, should be controlled. A jet grouting wall through the river bed alluvium is proposed to reduce those infiltrations.

For the cofferdam, the bituminous core is anchored on a concrete plinth; this plinth shall be founded on the bedrock. Therefore, the following works have to be performed:

- Riverbed alluvium removal in the plinth footprint vicinity;
- Cleaning of the bed rock and banks, removal of unsteady rocks, filling of cracks and holes with concrete;
- Consolidation grouting under the bituminous core plinth.

For the Stage 1 dam, the bituminous core is anchored on a concrete plinth. This plinth shall be funded on the bedrock. Therefore, the following works have to be performed.

- Riverbed alluvium removal in the plinth footprint vicinity;
- Cleaning of the bed rock and banks, removal of unsteady rocks, filling of cracks and holes with concrete;

- Consolidation grouting under the bituminous core plinth.

- A grout curtain is also recommended to ensure an efficient watertightness of the Stage 1 dam, it should be 90 m deep and should not cross the Ionaksh fault.

Main dam

In addition to the geotechnical requirement of reaching zone III rock stated above, the topography of the dam core footprint shows important irregularities, brutal ground slope variation and large overhanging masses…

Therefore, the excavation of the core foundation has been studied by the Consultant which aims at:

- excavating the dam core footprint by 5 m depth to remove Zone I and II weathered rock;

- smoothing the irregular topography to avoid stress concentration and ensure the dam core integrity;

- making sure to have a horizontal (or inclined toward upstream) contact between the core and the foundation.

Drawings of the core excavation are presented: n°40-103,203 and 303. The amount of rock excavations is presented in the next table for the three dam alternatives. An optimization of the excavation layout is possible and necessary in the next phase of the study.

FSL=1290 masl	FSL=1255 masl	FSL=1220 masl
2.34 Mm^3	1.73 Mm^3	1.64 Mm^3

Table 3-7 : Rock excavation volume for core

The lower Obigarm mudstones, on which the dam core is set, present a low permeability. However, to prevent any seepage through the foundation, some grouting works are necessary to:

- seal the fissures in the core foundation that could result to core erosion;

- prevent the suffusion phenomena due to the presence of gypsum in the Obigarm layer.

The fault #35 is considered as impervious and is a natural watertight barrier. Therefore, the grouting curtain is proposed to be maximum 2/3 of the dam height to reach the intact rock of zone II or IV but should not cross the fault #35. It a vertical curtain made of primary holes spaced by 12 m.

The grout curtain works will be performed from galleries: in the concrete slab in the river bottom and in each bank. Details of galleries network will be designed in further phases of the study.

The consolidation works have also to be performed under the core foundation to repair any superficial damages that could have been caused by the excavation works.

3.6 Salt wedge treatment and impact on dam

This is further detailed in Phase 0 report.

3.6.1 Description

The present location of the dam has been selected such as to be positioned in the gorge made by a bend towards south of the Vakhsh River, between Ionakhsh Fault to the North, and Fault 35 to the South.

The area is tectonically very active, and geodetic measurements carried out before 1978 demonstrated that both Ionakhsh Fault and Fault 35 were creeping at a rate of about 1.5 to 2 mm per year.

Therefore, the dam location was selected such as the dam axis, as well the core of the dam, is to be located on the block between those two faults, where no movement were assumed to occur.

When it comes to evaporites, and especially salt, leaching is a phenomenon that may be very rapid, and generates dramatic consequences.

3.6.2 Mitigation measures

The proposed mitigation technique is the combination of hydraulic curtain and grouting of the wedge cap. Grouting needs to be optimal, and thus checked with Lugeon tests and if needed (everywhere values higher than 1 LU are observed) re-implemented until the control water test shows everywhere values lower than 1 LU.

Both efficient grouting and efficient hydraulic barrier are by far necessary to prevent salt leaching, or to reduce it to the acceptable rate of 25 cm/year.

Moreover, the results evidence the fact that even if efficient hydraulic barrier only, as well as efficient grouting only is also acceptable, it is clear that at least one of these two mitigation measures shall be maintained operational throughout the lifetime of the scheme.

We would recommend that intervention for restoring efficiency of both mitigation measures to be allowed.

In order to follow the efficiency of the design mitigation measures, an adequate monitoring is required, so that in-time reaction and repair works can be carried out as soon as possible. Suggestions for this monitoring are given here below.

With the implementation of the hydraulic and grouting barriers, the related monitoring system, and the design of remediation works in case of the barriers failure, the thorough analysis of the scenarios shows that the leaching issue at the Ionakhsh Fault does not affect the project feasibility.

3.7 Dam instrumentation

Main dam

The monitoring system is the only way to follow the dam behavior once constructed, and checking that it is in agreement with its expected behavior. It is therefore of a prime importance to plan it at early study stages.

The dam movements are monitored thanks to:

- Topographical reflectors set on the dam crest and slopes;
- Settlements cells set inside the dam body;
- Accelerometers set on the dam crest and foundation.

Instruments using a vertical tube and a probe such as vertical settlemeters and inclinometers cannot be used on the Rogun dam given its height.

Settlement cells are actually made of two components set on the same horizontal layer: one cell is set into the dam body and the other is set on the dam surface. The settlement cells allow measuring the relative settlement of the dam body with respect to the reference cell set on the dam surface. To make this relative displacement absolute, a topographic reflector should be set next to each reference cell. It can also be checked that the cumulative settlement measured by all cells set in the same vertical axis is consistent with the crest vertical displacement measured by topographical means.

In addition to the settlement (vertical displacement), topographic reflectors measure the planimetrical movements (radial and tangential to the dam axis) of the dam crest, downstream slope and the upstream slope when the reservoir level is low.

Accelerometers at the dam foundation and at the crest are used to follow the dam behavior during earthquake and perform efficient back analysis on the dam model. Two additional accelerometers are also set on both banks. This should be put in service as soon as possible (one year before the river diversion for instance) to record the seismic background of the site and be able to compare it with the seismic signal during reservoir impounding. This should help detecting triggered seismicity and adapt the reservoir filling rate if necessary.

The hydraulic behavior of the dam can be monitored thanks to pore pressure cells set in the core, in the upstream and downstream dam shells, as well as in the dam foundation. The objective is to make sure that the hydraulic behavior of the dam is correct: watertightness of the core and of the foundation, and pervious shells.

The seepage discharge through the dam body cannot be measured on Rogun: the river level is higher than the downstream dam toe; therefore the seepage will follow the valley bottom and reach the river under the water level. However the leaks can be estimated thanks to the piezometric measurements and a hydraulic calculation.

In addition, optical fibres can be used as a seepage detector: several fibres set along the core axis just downstream of the coarse filters layer, and between 50 m and 100 m above the foundation, can detect if the seepage through the core is increasing. If the water current lines in intersects the optical fibres, it will detect it. The advantage is to detect the approximate location of the leak along the dam axis whereas piezometers are only set on the selected profiles.

The drawings n°40 106 and n°40 107 in Volume 3 – Chapter 4 drawings show a concept for the dam monitoring system. The various instruments are set in 6 radial profiles. This will have to be further studied in the next project phases, to solve some practical issues such as the wiring of all the measurements cells.

As it is inevitable to loose part of the measurements cells during construction, the number of cells set during construction is important.

The dam monitoring system should also include:

- a general topographic network with at least 6 landmarks that can be considered as constant (no movements).
- series of piezometers and topographical reflectors on both banks.

Salt wedge

For the salt wedge surveillance, the recommendations made in Phase 0 report are reported here below:

- Topographical survey of the crest and slopes of the Stage 1 dam during final dam completion;
- Regular sonar inspection of the dam upstream face to detect any abnormal deformation after reservoir impounding;
- Micro gravity campaign during all Stage 1 phase from its crest ;
- Conductivity by 12 inclined boreholes 6 profiles of two boreholes, one inclined at 60°, 60 m long, and the other at 45° and 70 m long. Both with 4 conductivity cells at equal distance. Permanent monitoring is required.

4 RIVER DIVERSION

The Rogun river diversion is a complex issue that is influenced and constrained by several points:

- the river hydrology and the construction schedule;
- the site topography;
- the existing structure;
- the early impounding of the reservoir and early energy generation.

4.1 Specific design criteria

4.1.1 Construction flood

The construction flood to be considered in the river diversion is discussed in details in Appendix 3 – Flood management during construction. The main elements are synthesized here.

Considering the length of the construction period and its phasing, three construction steps have been differentiated: the cofferdam phase, the Stage 1 dam phase and the main dam phase. For each of these a different design flood is chosen. The Table 4-1 presents the occurrence probability chosen to be the protection level during the construction and the matching return period for each dam alternative.

		Cofferdam	Stage 1	Completion of main dam
Probability of occurrence		**1/50**	**1/100**	**1/200**
Return period (year)	FSL= 1290 masl	100	450	2500
	FSl = 1255 masl	100	450	1000
	FSL = 1220 masl	100	300	700

Table 4-1 : Construction flood - probability of occurrence and return period

The protection level is increasing as the dam is rising, indeed the higher the dam is the greater are the consequence of a failure. The volumes of the reservoirs created at any moment are also shown in the Table 4-2 in order to quantify the level of consequences if floods occur and the dam fails. Notice that those volumes of water could be stored in the Nurek reservoir by just increasing the reservoir level by 2 m (CD) and 6 m (S1), but could by no means be stored for MD.

A sensitivity analysis has been performed on the protection level chosen to assess the impact on the structures number and size. It showed that it has minor impact on the diversion structures. This analysis is presented in details in Appendix 5.

	Cofferdam	Stage 1	Main dam
FSL=1290 masl	V = 190 hm^3 dZ(Nurek)= 2 m	V = 610 hm^3 dZ(Nurek)= 6.3 m	V = 13 300 hm^3 dZ(Nurek)= !!! m
FSL=1255 masl	V = 190 hm^3 dZ(Nurek)= 2 m	V = 480 hm^3 dZ(Nurek)= 5 m	V = 8 490 hm^3 dZ(Nurek)= !!! m
FSl=1220 masl	V = 190 hm^3 dZ(Nurek)= 2 m	V = 360 hm^3 dZ(Nurek)= 3.7 m	V = 5 210 hm^3 dZ(Nurek)= !!! m

Table 4-2 : Reservoir volume and consequences

4.1.2 Structural criteria

Existing structures

The general idea is to reuse as far as possible the existing works. Here the limitations of the existing structures are presented.

As already discussed in the Phase 1 report, the hydraulic behavior of the two diversion tunnels is not totally satisfactory, because a hydraulic jump occurs inside the downstream stretch of the tunnels, which should always work in free-flow conditions.

This is due to the fact that the tailwater level is by now higher than originally foreseen by several meters, due to the deposit of material resulting from the cofferdam collapse and from the mudflow of Obi Shur creek.

The hydraulic tests the consultant had the occasion to witness in Moscow, for flows up to 1,600 m^3/s / tunnel, confirmed that if the downstream original elevation is restored, the water flows in supercritical conditions and no hydraulic jump occurs. It is thus considered necessary, and has been taken into account in both the cost estimate and the implementation schedule, that the stretch of river downstream from the diversion tunnel outlets is dredged so to avoid the formation of the hydraulic jump inside said tunnels.

Another drawback is constituted by the rise of pressure inside the tunnels in the stretch close to the junction with the powerhouse collectors.

It is also matter of concern the structure of the tunnels, which was analyzed in the preliminary report of October 2011 and which was not found in line with the presently internationally recognized design criteria for such kind of structures.

For all the above reasons, we deem that the use of the two diversion tunnels as spillways should be limited both in respect to the time and to the water head.

According to the results of the structural assessment carried out in Phase I Report, they will need heavy rehabilitation works. Therefore, it is already considered in this analysis that their diameter will be reduced by 60 cm as a provision for rehabilitation works.

Those tunnels shall work under a maximum head of 120 m, and their maximum discharge should preferably not exceed 1600 m^3/s/tunnel.

New tunnels

The maximum head tolerated in diversion tunnels (temporary structures) is 120 m. This value can be overpassed by 30 m, i.e. 150 m, in extreme conditions such as high flood or seismic event.

This limit is set in order to keep the maximum water speed through the gates openings within the limits proposed as design criteria, so to avoid cavitation, excessive air entrainment and flow instability phenomena.

The next figure presents data extracted from the book Design of Hydraulic gates written by P. Erbisti in 2004 and presenting the design head and gate area of existing high pressure gates. The red line is interpolated from the two extreme points: Tarbela (largest gates) and Beaver (higher design head). The dotted line is interpolated from all data. It shows that a design criterion of 120-150 m is reasonable considering the size of the gates needed.

At any time of the construction, at least two tunnels shall be operational.

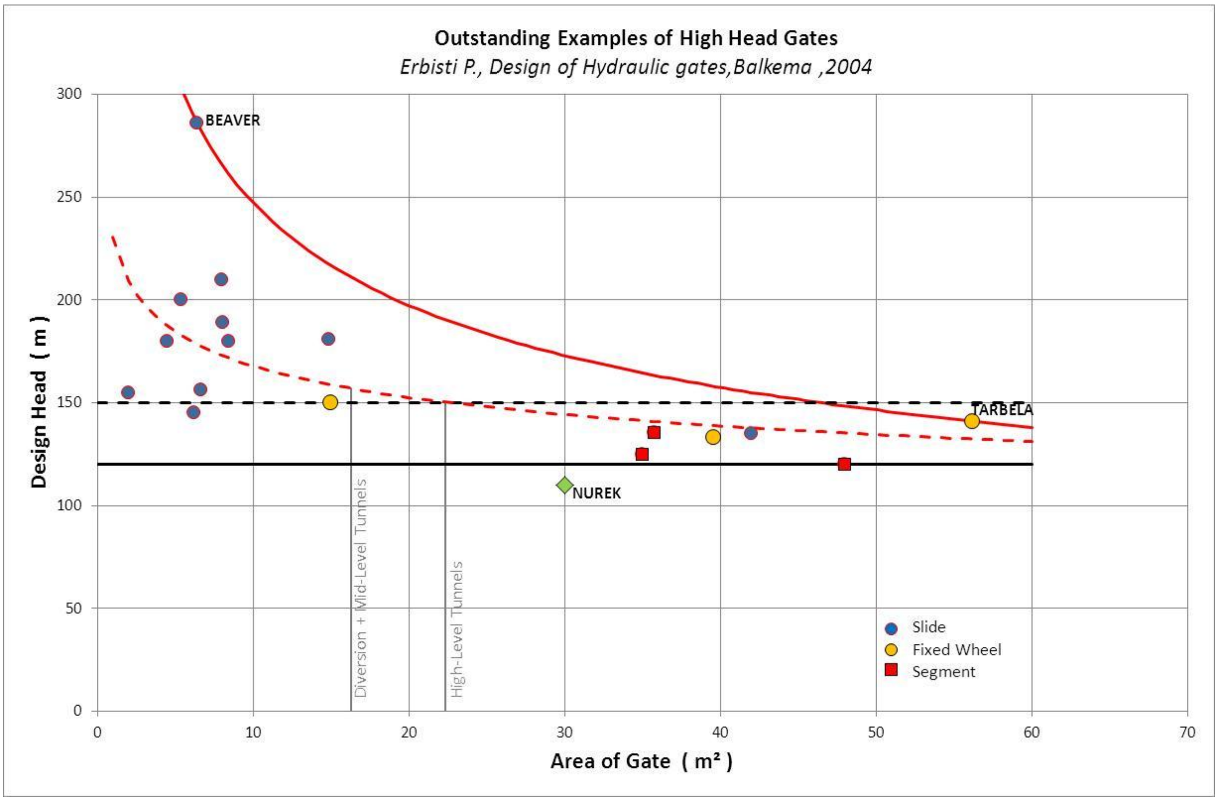


Figure 4-1 : Situation of maximum head criteria among existing examples

A sensitivity analysis to the maximum water head criteria has been performed by considering also a maximum water head of 150 m instead of 120 m. In that case, it has been found that:

- for the highest dam alternative, MLO2 would not be necessary;
- for the medium alternative, one HL would not be necessary;

- for the lowest alternative, MOL1 could be replaced by one HL, which is a tunnel with a smaller diameter.

About 1.5% of the total Rogun cost can be saved by considering 150 m water head instead of 120 m.

The saving is very limited compared to the risk increase:

- the operating complication due to the head in the final spillways (HL);
- the increased reliance on DT3 which crosses Ionakhsh fault.

Therefore, the Consultant recommends and has considered 120 m maximum water head as a design criterion in this TEAS. Optimization of the layout might be considered in any case in further stage of the study.

4.1.3 Others

Turbines discharge capacities are not taken into account into the flood discharge system. Indeed, during high floods, the powerhouse might be out of service; access to the powerhouse can be interrupted, etc. The turbines are not ensured to be working during high floods, so they are not considered as a flood control structure.

Reservoir routing is taken into account except for the cofferdam phase which has a very limited reservoir capacity. The hydrograph taken into account is the one established in Volume 2-Chapter 5, for the PMF and 10000 years return period flood and proportionally reduced for smaller floods.

Discharges considered are under or in the range of Nurek discharge capacity. A “special” operation of Nurek during Rogun construction is therefore not necessary.

Co-seismic displacements in Ionaksh fault are metric. No totally safe remedial solution exists to cope with this range of displacements in high pressure tunnels. No probability can be associated with this event. But the project should survive in spite of its occurrence: the fault-crossing structures should not collapse. This shall be considered as an extreme scenario.

4.2 HPI river diversion scheme

4.2.1 Description

According to HPI scheme, from the river diversion to the completion of the final dam, 6 different structures are used to divert and discharge the floods:

- Diversion tunnel of 1st level (DT1)
- Diversion tunnel of 2nd level (DT2)
- Diversion tunnel of 3rd level (DT3)
- Operational tunnel of 3rd level (OP3)
- Remote spillway (RS)

- Operational shaft spillway (OSS). The remote spillway and the operational shaft spillway share the same downstream tunnel and outlet.

The next figure presents the location and inlet elevation of the various diversion and spillway structures.

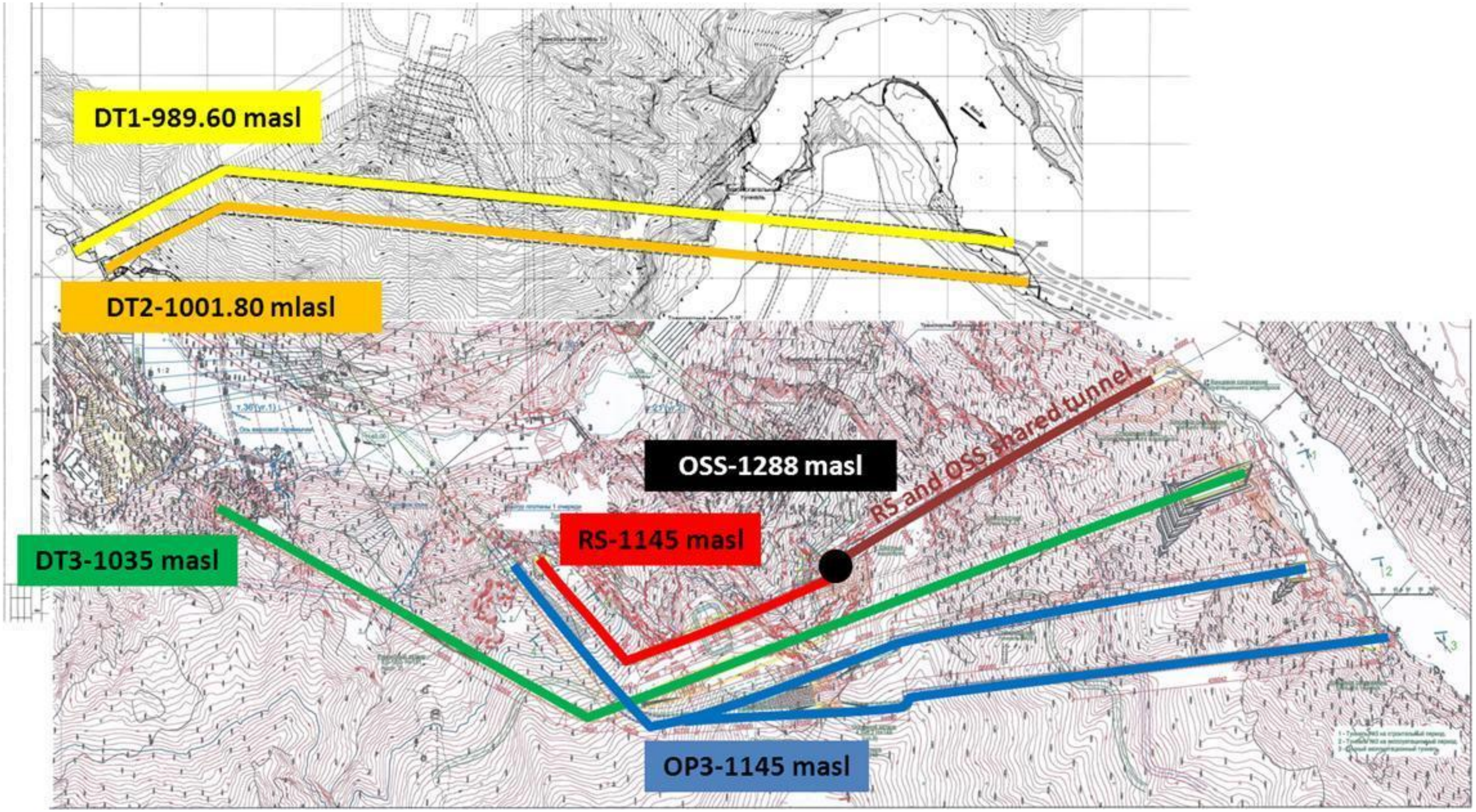


Figure 4-2 : Plan view - Diversion and spillway structures – HPI scheme

At the beginning of the works, the site is protected by a cofferdam with a crest at 1035 masl. Only DT1 and DT2 are diverting the river flow. The capacity of both tunnels is shown to be 2900 m^3/s with water elevation at 1033 masl on the 2010 HPI drawing.

While the Stage 1 dam is protecting the site, DT1& DT2 and DT3 are diverting the river. At this elevation (water level at 1110 masl), the combined capacity of the three tunnels is 7100 m^3/s.

Between 1110 masl and 1145 masl, DT2 and DT3 are ensuring the river diversion. DT2 discharge is limited to 1800 m^3/s by closing some of its gates.

Between 1145 and 1185 masl, DT2, DT3 and the remote spillway are ensuring the river diversion.

Above 1185 masl, the river diversion and flood discharge is ensured by final spillways: OP3; RS and OSS. They are able to discharge 7100 m^3/s at water elevation 1290 masl.

4.2.2 Assessment

A thorough analysis of the HPI scheme that is presented in Appendix 5 leads to the following comments:

On cofferdam phase

As per HPI design, the discharge capacity during this phase is 2900 m^3/s, i.e. a return period flood of 7 years.

Moreover, considering the state of diversion tunnel 1 and 2, and the provision for rehabilitation works, the level of protection reduces to 2650 m^3/s with water at 1035 masl, i.e. a return period lower than 5 years. Considering the cofferdam life span of two years, it gives a risk of 1/2.5. This protection level is not acceptable for a cofferdam.

On Stage 1 phase

As per HPI design, the Stage 1 is protected against the PMF. This level of protection was relevant when the Stage 1 was considered as a stand-alone project. Now that this possibility has been discarded, the level of protection of the Stage 1 can be reduced.

Between the Stage1 and elevation 1185 masl

When the reservoir level rises up to 1110 masl, DT2 and DT3 are able to discharge 4400 m^3/s if DT2 discharge is limited as announced and 5200 m^3/s if it is fully open.

When reservoir level is 1185 masl, the water head supported by DT2 intake is 183 m, the water head supported by the gates sill is 199 m, and the water head supported by DT3 is 150 m. Those value are much more than the one set as limit value for normal operation in temporary structure by the Consultant.

At 1185 masl, DT2, DT3 and remote spillway combined discharge capacity is 6400 m^3/s, and half of this discharge is actually passing through DT3.

From the Stage 1 to reservoir elevation 1185 masl, the safety depends largely on DT3 which crosses the Ionaksh fault.

Above elevation 1185 masl

Above 1185 masl, the final spillways ensure the river diversion. At this elevation, OP3 and RS are able to discharge 4650 m^3/s, i.e. a lower capacity then in the previous phase.

After the dam completion, OP3 and RS would handle in normal operation a head of 145 m which is higher than the limit set by the Consultant for this type of structure.

Conclusion

According to the Consultant criteria, several items appear not to be fully safe:

- The level of protection of the cofferdam is not sufficient;
- The water head that all structures (temporary or final) have to support is too high;
- The Ionaksh fault particularity is not mentioned and no remedial measures are proposed to cope with its displacements whereas there is a significant construction period of high dependence on DT3.

Therefore, the Consultant proposes another flood management scheme that is detailed in the following paragraphs.

4.3 River Diversion Management

The various diversion structures considered by TEAS are:

- Diversion tunnel 1 and 2 (DT1 and DT2);
- Diversion tunnel 3 (DT3);
- Mid-level outlet 1 and 2 (MLO1 and MLO2);
- High level tunnels 1, 2 and 3 (HL1, HL2 and HL3).

4.3.1 Cofferdam

The construction flood considered for the cofferdam has a probability of occurrence of 1/100, i.e. a 100 years return period. The cofferdam crest elevation is 1050 masl. The construction flood is discharged thanks to DT1, DT2 and DT3.

The cofferdam elevation might be decreased whenever a higher discharge capacity of the existing DT1 and DT2, which has been indicated by the designer, would be proved through model studies; the existing construction situation shall be checked anyway.

Downstream cofferdam of this phase is the cofferdam set just downstream of the DT's culvert across the river. For the construction flood considered (100 years return flood), the water elevation is 992.7 masl, and therefore the downstream cofferdam crest shall be 994 masl.

If DT3 is out of service because of the Ionaksh fault shearing, the cofferdam will be protected only against a 10 years return period flood (probability of exceedance of 1/10). In case of higher flood, the cofferdam will be overflowed. Therefore, failure of DT3 will most likely lead to the cofferdam failure. This failure will be dumped in Nurek as the cofferdam reservoir volume is limited (approximately 55 hm3) and represents 50 cm of Nurek reservoir.

The event of a co-seismic displacement within the two years of cofferdam life span in addition to a flood higher than the 10 years return flood is unlikely. And the consequence will be to destroy the cofferdam and all on-going works behind but will not have any consequence downstream of Nurek. The Consultant considers this as an acceptable risk.

4.3.2 Stage 1

The construction flood considered for the Stage 1 dam is the flood with a probability of occurrence of 1/100. It matches the 400 years return period flood for stage 1 at 1110 masl and 1090 masl, and the 300 years return period flood for stage 1 at 1090 masl, and 200 years return period flood for stage 1 at 1075 masl. Those construction floods are discharged with DT1, DT2 and DT3.

Downstream cofferdam of this phase is the one set downstream of the final dam toe. For the construction flood considered, the water elevation is 984.2 masl, and therefore the downstream cofferdam crest shall be 986 masl.

If DT1, DT2 or DT3 is out of service, the protection is still ensured for the highest Stage 1. For the two lowest, a massive overflow would be expected in the event of the 1/100 probability of occurrence flood.

Nevertheless, in case of DT3 failure, the Stage 1 with crest at 1090 masl is still protected against the 400 years return period flood (risk of 1/140), which is acceptable as this is an exceptional situation: a combination of two rare events.

For the Stage 1 with crest at 1075 masl and in case of DT3 failure, the dam is still protected against the 120 years return period flood (risk of 1/50).

The Consultant considers this as an acceptable risk.

4.3.3 Main dam completion

From water elevation 1100 masl, the construction flood is discharged thanks to DT3 and MOL1 are discharge. The construction flood considered for the Main dam completion phase is the flood with a probability of occurrence of 1/200.

For the higher dam alternative (1290 masl), DT3 is switched off at water elevation 1160 masl. From 1160 masl to 1215 masl, the construction flood is discharged thanks to MOL1 and MOL2. From 1215 masl to 1270 masl, the construction flood is discharged thanks to MOL2 and HL1. Above 1270 masl and until dam completion, the construction flood is discharged thanks to HL1 and HL2.

For the medium dam alternative (1255 masl), DT3 is switched off at water elevation 1170 masl. From 1170 masl to 1210 masl, the construction flood is discharged thanks to MOL1 and HL1. Above 1210 masl and until dam completion, the construction flood is discharged thanks to HL1, HL2 and HL3.

For the lower dam alternative (1220 masl), DT3 is switched off at water elevation 1165 masl. From 1165 masl to dam completion, the construction flood is discharged thanks to MOL1 and HL1.

The Downstream cofferdam is embedded in the dam downstream toe. For the construction flood considered (700, 1000 or 1600 years return flood depending on the alternatives), the water elevation is 984.4 masl, 984.5 masl or 984.6 masl depending on the alternative, and therefore the downstream cofferdam crest shall be 986 masl.

In case of Ionaksh fault co-seismic movement, DT3 and MOL1 tunnels could be put out of service.

MLO1 risk of failure can be avoided by designing a specific intake that does not cross the Ionaksh fault: the tunnel enters the banks downstream of the fault, and a culvert that crosses the dam drives the flow from the reservoir to the tunnel (see drawing n°40 115). The culvert inner section is D shaped, 18 m diameter, with lower chamfers and would be designed so to resist the maximum dam filling above its crown (about 35 m) and strong seismic effects. The structure is divided into segments some 25-30 m long, the first of which is lying on the Ionaksh fault. In case of fault displacements, the segment can be also displaced both with respect to the tunnel portal proper entering into the right bank and with respect to the adjoining segment upstream of it resting out of the fault, but it would not collapse and the hydraulic connection between the intake and the tunnel proper portal would be maintained.

No feasible solution exists to avoid crossing the Ionaksh fault with DT3. Some mitigation measures can be put in place in the fault stretch to face at least the creeping effect and displacements of moderate entity. This provision has been described at paragraph 2.2 - Main Features and Hydraulics of the Diversion Tunnel of the Report on Hydraulics of the Project Components. The probability of having both a high flood and an important seismic event able to make DT3 collapse within the life span of DT3 is limited. This risk is accepted by the Consultant.

Synthesis

All the above is now synthesized thanks to 2 illustrative sketches for each dam alternatives and one graph of discharge capacity.

The first one presents the operating range of each structure along a water level axis. The black lines show the normal operating range and the dotted lines represent the addition exceptional operating range. It indicates the water elevation for which each tunnels should be switched on (low line extremity) and off (high line extremity).

The second one presents the protection level, operating structures, and maximum water level all along the construction period.

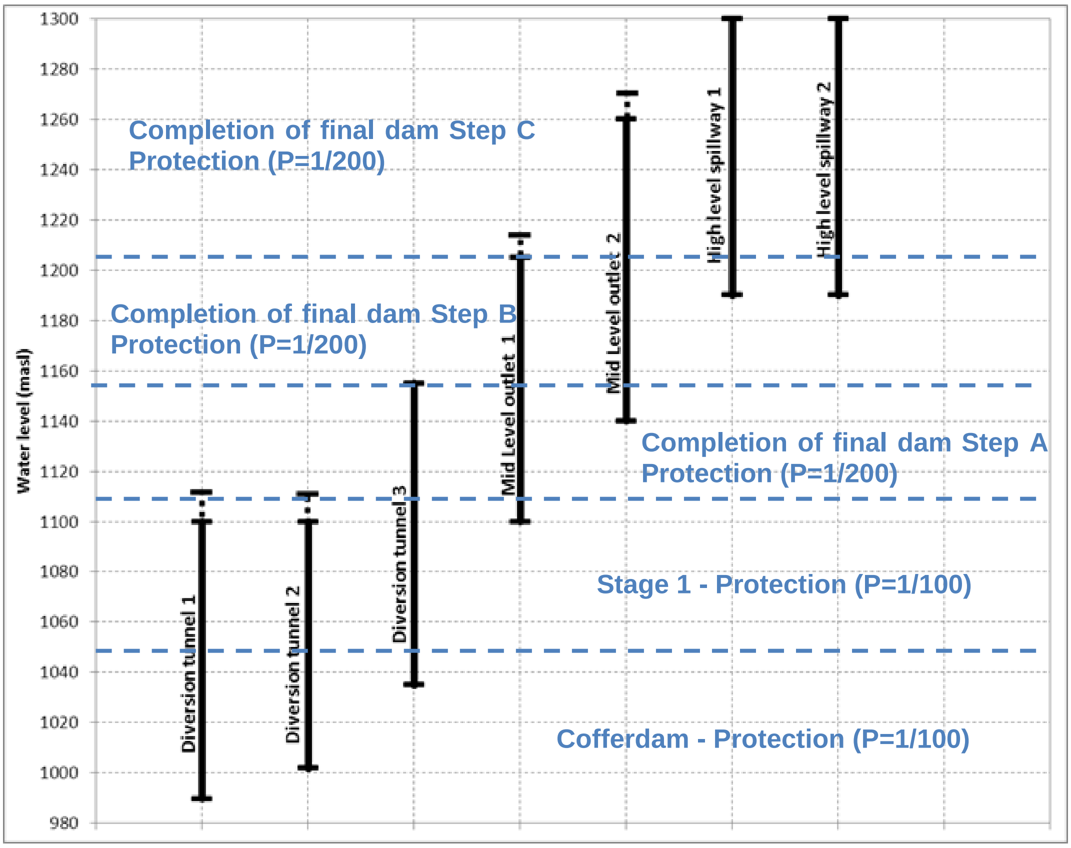


Figure 4-3 : FSL = 1290 masl - Diversion structures operating range

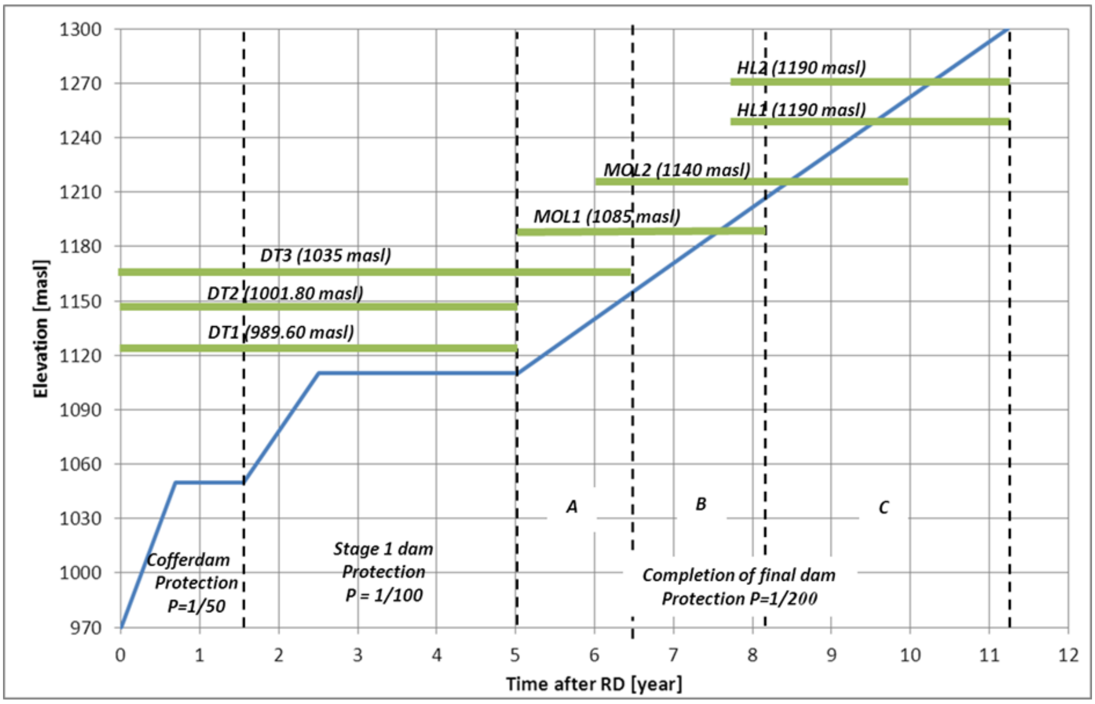


Figure 4-4 : FSL = 1290 masl - Diversion scheme along time

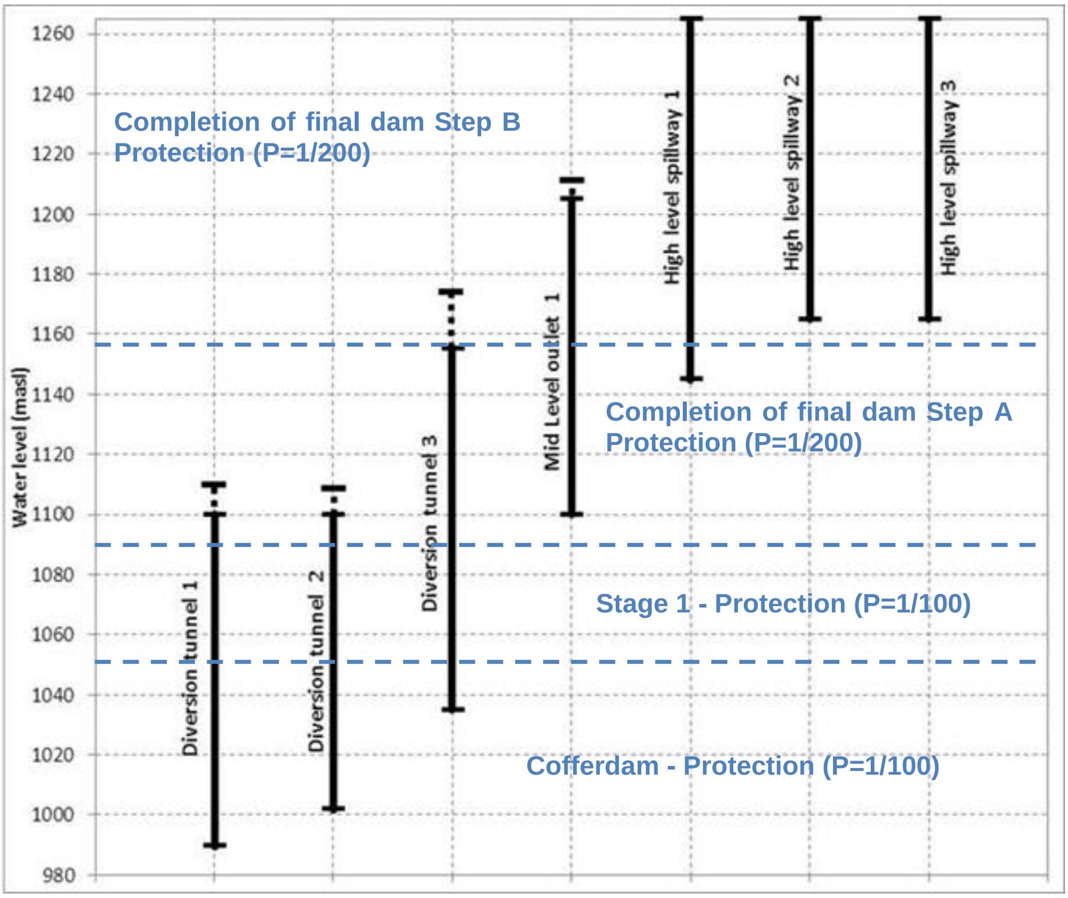


Figure 4-5 : FSL = 1255 masl - Diversion structures operating range

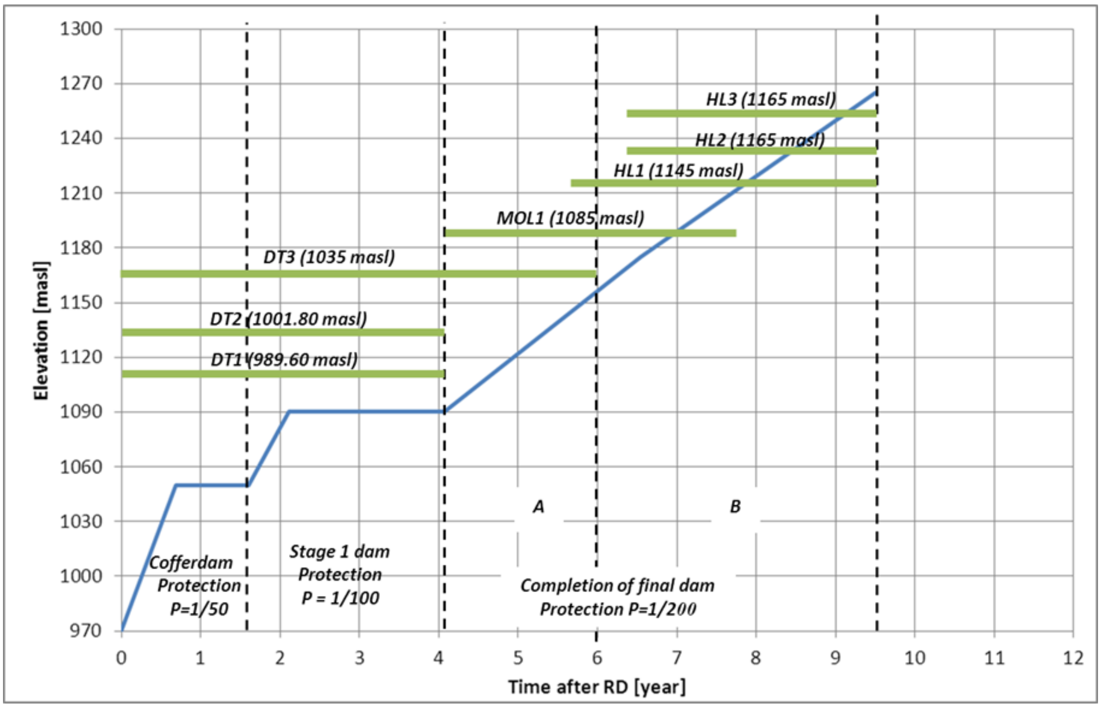


Figure 4-6 : FSL = 1255 masl - Diversion scheme along time

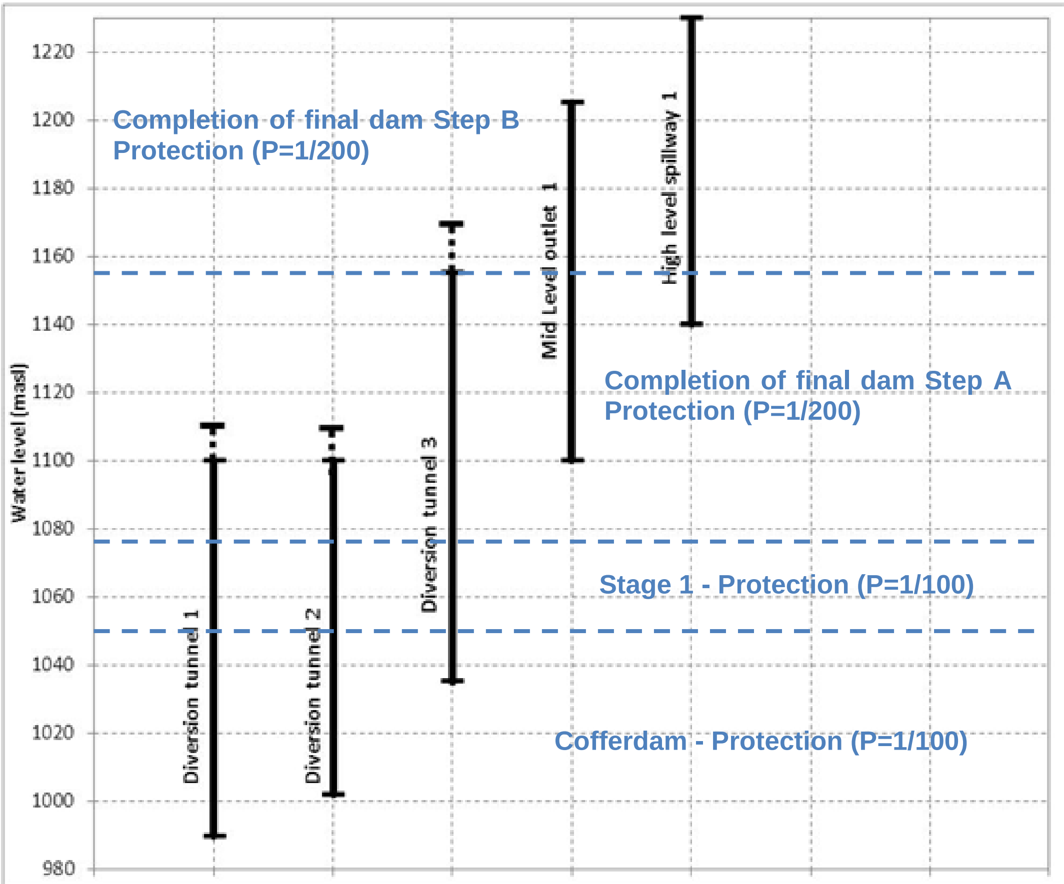


Figure 4-7 : FSL = 1220 masl - Diversion structures operating range

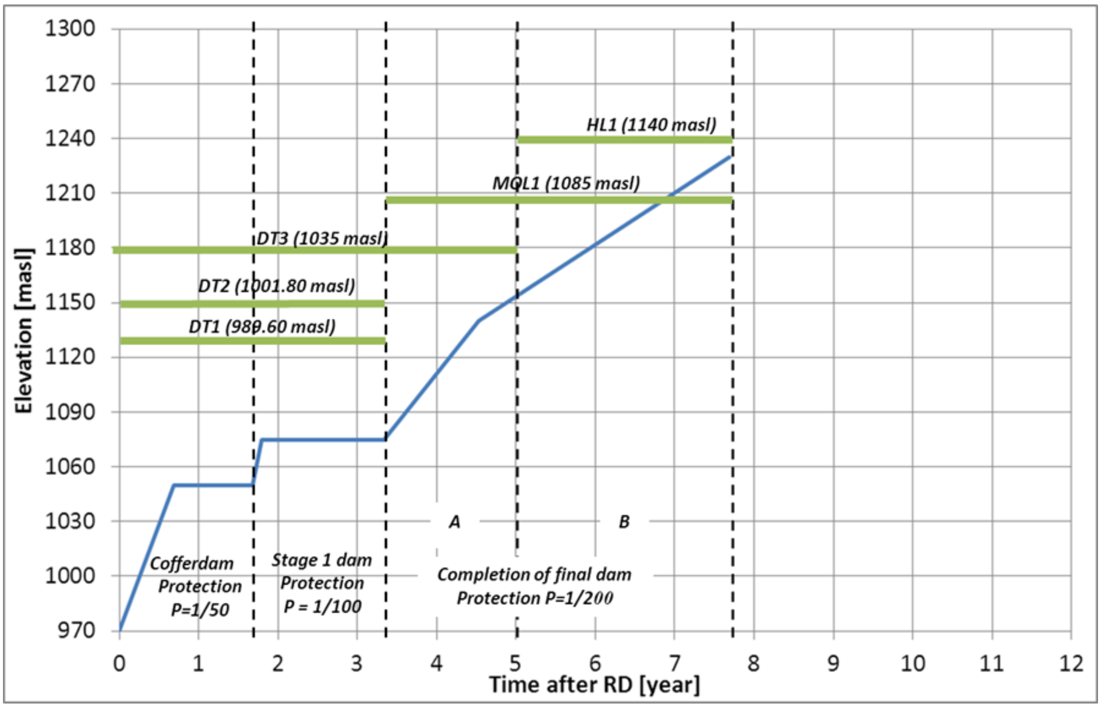


Figure 4-8 : FSL = 1220 masl - Diversion scheme along time

4.4 DT1 and DT2

The existing 1st and 2nd level diversion tunnels develop parallel at a distance between axes of some 45 to 58 m, their intakes are placed shortly upstream the pre-cofferdam, close to the intake of the Stage 1 headrace tunnel, and their floor elevations are set at 989.6 m a.s.l. for tunnel n° 1 and 998.8 for tunnel n° 2.

In order to avoid entrance of bed load sediment, it is foreseen that after a short initial period of operation, the intakes front openings will be closed and water will enter through a trapezoidal opening located on the intakes structures roof at el 1,020 m a.s.l. The downstream portion of the tunnels will be used as free-flow tailrace tunnels for discharging the flow used for energy generation. They will be plugged just upstream from the junctions with the two collector tunnels, each connected with the draft tubes of three units.

Since the tunnels technical assessment contained in Phase 1 report concluded that the tunnels are not adequate in the present conditions for the purpose they have been designed for, measures adequate to improve their structural stability are necessary.

Recommended interventions include the implementation of a patterns of dowels, a drainage system and an additional reinforced concrete lining, 30 or 40 cm min thickness in vault, 50 cm min on invert, with a horseshoe shape.

The latter provision produces a reduction of the tunnels internal cross sectional area, which has as a consequence the increase in the headlosses along the tunnels pressure stretches from intake up to the gates control section, thus impacting to some extent on the tunnels discharge capacity. The revised discharge curve calculated by the consultant shows that after the new lining is constructed, each tunnel can discharge a flow of about 1,325 m^3/s at el 1,035 m a.s.l. and 1,525 m^3/s at el 1,050 m a.s.l.

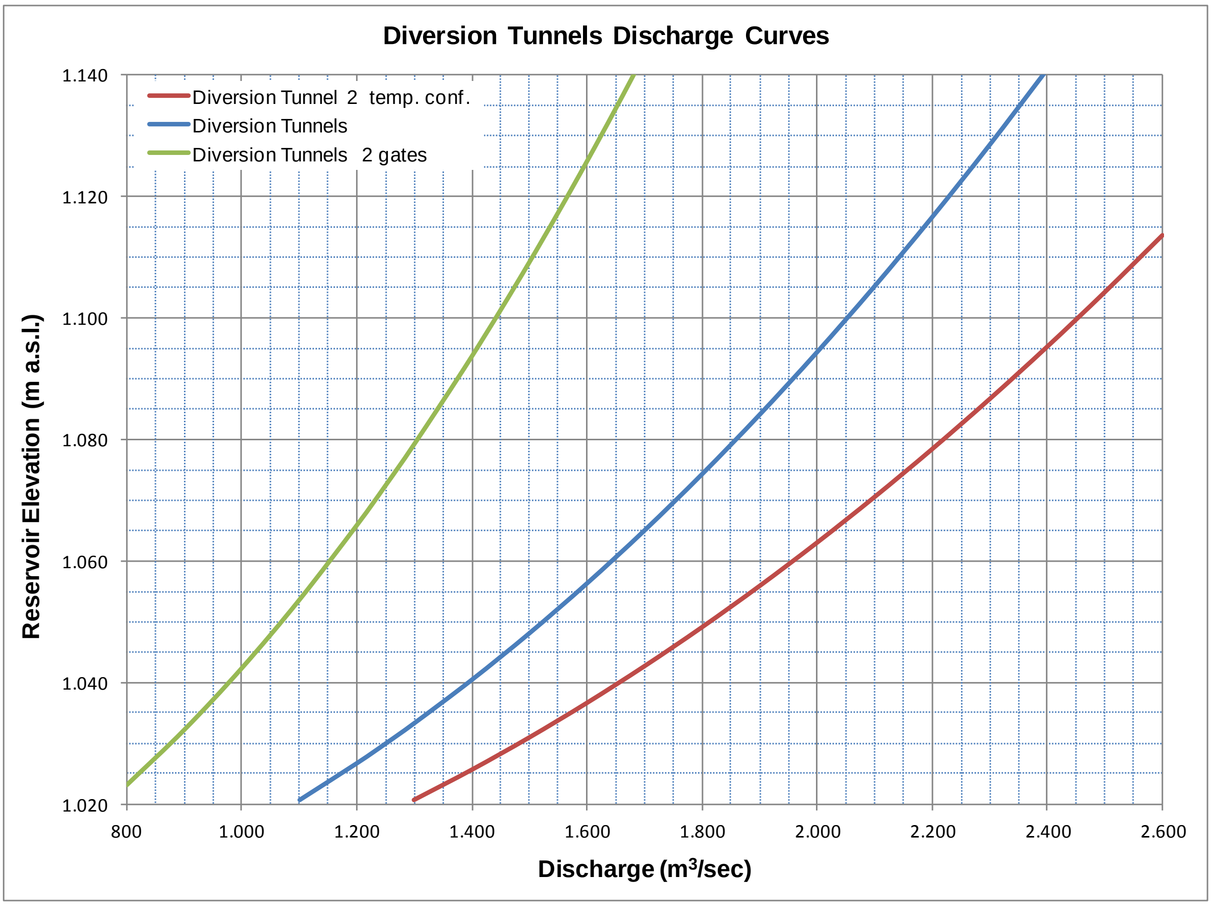
Thus the aggregate total capacity of both tunnels would become 2,650 and 3,050 m^3/s at 1,035 and 1,050 m a.s.l. respectively.

It is to be noted that the above values have been calculated based on the tunnels final configurations, i.e. with the front openings closed and all electromechanical equipment installed.

If the situation in which in diversion tunnel n° 2 the sector gates are not installed yet and the opening is larger than the final one (total cross section area of the three openings about 79 m^2 instead than 61 m^2), those figures become 1,575 m^3/s at el 1,035 m a.s.l. and 1,810 m^3/s at el 1,050 m a.s.l., thus the aggregate discharge capacity for both tunnels at the same elevations is brought to 2,900 and 3,335 m^3/s respectively.

This situation is that taken into account by HPI during the first year after the river diversion.

The discharge curves are shown in the graph here below.



On the other hand, the analysis of the hydraulic behaviour of those tunnels, reported in the Stage 1 dam conceptual design of July 2011, led to recommend limiting their discharge capacity, in order to avoid drawbacks due to the occurrence of the hydraulic jump inside the tunnels downstream stretch. In that occasion, we noted that for flows in the order of 1,500 m^3/s a hydraulic jump occurs at some 500 m from the tunnel outlet, being the tunnel filling ratio downstream of the same jump approximately 86 %. Thus, we consider that, in order to remain within a range of safe and reliable operation of the tunnel, the released flow should not be much higher than the above figure.

From the hydraulic tests carried out at the Hydraulic Laboratory in Moscow, results of which are described in a specific report, for the highest flows analyzed (about 1,600 m^3/s) a rise of water pressure inside the tunnels in the stretch close to the junction with the powerhouse collectors was recorded. This would have as a consequence that the water-air foam would rise into the aeration shafts up to the elevation of the above T-8 permanent tunnel.

According to the hydraulic laboratory specialists, it would be feasible to keep this shaft plugged until the tunnels are used as spillways, since aeration is anyway provided just downstream the sector gates. The second aeration shafts have been designed mainly for the final stage, when the diversion tunnels will be plugged upstream of the junction with the draft tubes collectors and used only as tailrace tunnels.

It is to clarify that this situation is mainly caused by the fact that the material deposited in the river downstream from the diversion tunnels outlets, following the collapse of the cofferdam and the mud flow proceeding from Obu Shur creek, has modified the rating curve in that area, so that at present higher water elevations than expected in the original design occur.

Therefore, the following are the operation ranges and conditions for operating the two diversion tunnels:

- Up to el. 1,035, the tunnels can operate with all gates open, being the maximum discharge flow equal to 2,650 m^3/s for final configuration (equipment totally installed) and 2,900 m^3/s in case DT2 operates in provisional configuration, w/o sector gates and openings area of about 79 m^2;
- From 1,035 up to 1,050, DT2 also should operate according to the final configuration (sector gates erected, openings area about 61 m^2; the maximum global discharge is 3,050 m^3/s;
- Above el. 1,050, under the above configuration the flows would be such that the formation of hydraulic jump would occur. Therefore, it is necessary to restore the original riverbed elevations, at least to the extent that the flow downstream from the gates remains supercritical all along the tunnel. Whenever necessary, it is also possible to limit the flows by keeping one out of the three gates closed. However, the flow discharged in there conditions may be insufficient for floods control purpose.

According to the floods management studies carried out, the need for availing of discharge facilities at various elevations arose as a consequence of the criteria proposed for the safety of the works under construction and of the limitations in operating the same facilities.

In fact, according to the construction time and the period during which the works are exposed to the risk of floods, the needed discharge capacities at different elevations have been established in the above mentioned studies.

Following the above analyses, the construction of the third level diversion tunnel already proposed by HPI was confirmed as necessary.

4.5 Diversion Tunnel 3

According to the revised layout of HPI design, a further diversion tunnel was foreseen in the right bank of the river (third level diversion tunnel), which intake is located some 300 m downstream from the existing diversion tunnels portal. DT3 was designed in 2011 by HPI and the temporary support was designed by Tana Energy, Iran. At present, along its upstream stretch some 400 m of excavation works have been already carried out in correspondence with the crown, reaching the beginning of the upstream transition of the proposed maintenance/emergency gates chamber. The construction of DT3 was put on hold by the World Bank in July 2012, when only safety maintenance works were allowed to be carried out. The Client agreed to implement the improvements to the design proposed by the TEAS Consultant while the construction progresses. It is deemed that the status of the works performed so far is such that any change can still be incorporated.

This tunnel is needed to complement the discharge capacity of the existing diversion tunnels, allowing meeting the criteria set to protect the dam against floods during the construction.

The concept of this tunnel is shared by the consultant, and its location in the right bank has been substantially confirmed, with some adjustments in the route downstream portion required to accommodate the various hydraulic facilities described in the report on hydraulics.

During the studies, the possibility to find an alternative route in the left bank for the diversion tunnel N. 3 was examined, with the aim to avoid the crossing of Ionakhsh fault.

Due to the presence of existing tunnels and facilities, the intake should have been placed upstream from the existing diversion tunnel 1 & 2 inlets and the route surrounded the existing underground structures, reaching the riverbed downstream from Obi Shur creek junction. In fact, due to the interference with various structures and to the unfavorable morphology and geology, there is no option to place the outlet upstream from Obi Shur creek.

Anyway, even if the crossing of Ionakhsh fault is avoided, the tunnel passes through several other faults, among which fault 35, which in any case implies to implement measures to face the possible differential displacements. Also the crossing with Obi Shur creek is critical, due to the elevation of the riverbed, which would require the adoption of particular construction techniques. An alternative route much upstream along the riverbed would improve the crossing conditions but would imply a much longer route, with consequent higher costs and construction time.

The diversion tunnel n° 3 upstream stretch was located on the same alignment proposed by HPI and the same intake elevation was adopted, i.e. 1,035 m a.s.l., while the downstream portion has been rerouted, as already above mentioned.

The tunnel is approximately 1,550 m long and reaches the right bank of the river downstream from the dam toe at 200 m distance from the diversion tunnel n 2 outlet, thus possible scour in the riverbed would not impact on the structures of such facility.

The pressure operation tunnel stretch, with circular cross-section of 15.0 m diameter, is about 810 m long up to the sector and emergency gates chamber. Downstream from this chamber, a horseshoe cross-section 14.5 m wide and 9.75 m to the springline with a circular arch roof reaching a maximum height of 17.0 m has been adopted.

It is to be noted that the tunnel will cross both the Ionakhsh fault and fault n° 35, at some 700 m from its intake and at some 100 m before the outlet portal, respectively.

Therefore, provisions have been made in correspondence of both faults in order to face the effects of creeping and/or possible large displacements in occasion of seismic events.

The provisions consist basically in implementing a very thick highly reinforced concrete lining divided into short stretches (rings) along the reach of the shear zone and some meters in excess of it at both sides: in case of differential movements, the rings may displace but, due to their robust structure, the tunnel cavity will remain available. To avoid an immediate repercussion on the tunnel internal surface, the “rings” are larger than the current section by some 4.0 m in diameter, and a second inner concrete lining matching with the tunnel current section is foreseen. The space between the two linings is filled with cellular concrete, which provides support to the internal lining and can absorb part of the displacements, since the spaces present in its mass allow for a compression of its volume.

As a further provision, the first gates chamber, located at a distance of about 460 m from the intake, is proposed to be equipped with four wheel gates similar to those installed in the emergency gates chamber: these gates, with an area of about 30 m^2 each, can be operated under flow with the maximum head. This allows, in case large that displacements at fault section provoke damages to the inner lining and block the downstream gates, to put the tunnel out of service and carry out repair works. It is noted that the above gates arrangement could be modified to an equivalent one with the same discharge capacity, provided that the gates are capable to close under flow and with the maximum head.

As for the tunnel current sections, a conservative approach has been adopted in defining the lining thickness, which is normally in the order of 1/10 of the tunnel internal diameter or thicker.

The tunnel must operate during several years with the discharge associated to the reservoir water level for all the dam alternatives with a maximum exceptional head of 150 m. The maximum velocity of the flow in the reach operating under pressure conditions was kept in the order of 20 m/s, and the free board in the reach operating in free flow conditions is higher that 25% of the total height of the tunnel.

Since the water velocity through the gates openings is in the order of 41 m/s, the stretches corresponding to the gates structures (rectangular conduits) and a portion of the transitions (5 m upstream and 10 m downstream) have been considered to be steel lined.

Also, in order to control the risks due to cavitation, immediately downstream from the gates aeration has been provided both in the crown of the chamber and all around the gates, by enlarging the section laterally and designing a step in the floor with allows bringing air below the jet.

Also, transitions from the gates sections to the tunnel current cross section are quite gradual, with a deviation angle in the order of 4-5°.

As for the water release at the tunnel outlet, in consideration of the low difference in elevation between the outlet portal (1023.45) and the riverbed, a simple short chute some 90 m long with terminal flip bucket located a few meters above the river water level (1004.3 m a.s.l.) has been designed for restitution of the water downstream from the dam.

The considerations relevant the energy dissipation and need for plunge pool, as well as the possible scour effects are contained in the detailed computations performed in the report on hydraulics (Vol. 3 – Ch. 3 – Appendix 4).

According to the hydraulic calculation performed by the consultant, such tunnel would be capable to evacuate some 1,325 m^3/s with reservoir water level at 1,055 m a.s.l. and 2,450 m^3/s when the water level reaches 1,100 m a.s.l.

All features of Diversion Tunnel N° 3 remain unchanged for all dam alternatives.

The tunnel will remain in use until the sediments will have reached the intake entrance, which might happen after 7/10 years; then the tunnel would be put out of service and permanently plugged.

4.6 MLO1 and MLO2

4.6.1 Middle Level Outlet 1

According to the results of the studies presented in Vol. 3 – Ch. 3 – Appendix 3 "Flood Management during Construction", the Middle Level Outlet 1 is required for all dam height alternatives, for dam protection during the construction starting from water elevation 1,100.0 m a.s.l. This same elevation is considered as the limit of normal operation of Diversion Tunnels N° 1 and 2, being the head above these tunnel 120 m in this situation. However, they will remain available in case an emergency (for instance DT3 out of service due to shearing at Ionakhsh fault section) obliges to make us of them for discharging floods.

The definition of the discharge capacity of the hydraulic facilities that must assure protection against floods during the dam construction is based on two main criteria:

- The maximum head acceptable, which in normal conditions should be not higher than 120 m and in exceptional conditions is accepted to go up to 150 m;
- The fact that at least two tunnels must be available at any elevation.

The above led to design the hydraulic facilities for managing flood during construction with similar characteristic, so that when one is put out of service another with the same discharge capacity is replacing it. The global discharge capacity of the outlets was checked in the report on floods management accounting for the criteria established for protecting the structures depending upon the time of exposure.

The tunnel general features of the MLO1 in the pressure portion which controls the discharge capacity of the tunnel are the same as those of Diversion tunnel N° 3, being the tunnel current cross section circular with 15.0 m diameter. The elevation of the tunnel invert in correspondence with the portal is 1085.0 m a.s.l. The tunnel extends for about 760 m up to the sector and emergency gates chamber. A maintenance gates chamber is located at a distance of about 360 m from the end of the culvert.

The intake arrangement foresees a concrete culvert some 300 m long from the dam upstream embankment surface up to the underground stretch portal, so that the tunnel proper is starting shortly downstream from Ionakhsh fault crossing. Such a culvert has internal cross section D-shaped, 18 m wide and 18 m high, so that possible displacements at Ionakhsh fault section can occur without interrupting the hydraulic route to the tunnel. The culvert is constituted by short stretches with walls thickness of some 3.5 m; this robust structure would accept displacements and relative movements without collapsing, thus maintaining the tunnel operative.

The outlet area elevation is around 1075 m a.s.l., which is about 100 m higher than the riverbed: thus the problem of restituting the flow, which is about 3700 m^3/s, required proper consideration, to avoid undesirable scouring effects which might also trigger bank instability.

The most obvious solution, that is constituted by a chute with terminal flip bucket, was analyzed, but discarded due to risks of cavitation phenomena. In fact it has to be considered that the flow velocity at the sector gates section is about 40 m/s and that along the short downstream stretch of tunnel to the outlet it remains anyway very high. Therefore the flow reaches the chute top with a high energy, which is further increased in the sloped chute, with the consequence that water velocities above 50 m/s are reached in proximity of the flip bucket. This figure, together with a specific discharge in the order of 105 $m^3/s/m$ (considering a flip bucket width of 35 m) implies cavitation risks and important scouring effects.

Subsequently, a solution with vortex chambers and drop shafts was analyzed, which allows dissipating a large percentage of energy while keeping the water velocity within the ranges acceptable for hydraulic structures.

Due to the large flow involved, it was split into two streams, which allow bringing the maximum individual discharge of each shaft to about 1850 m^3/s.

It shall be noted that this way the specific flow at the outlets is much lower and thus a better control of the scouring effects can be achieved.

In addition to the solution with vortex shafts, other possible alternatives have been analyzed, with the aim to reduce the number of outlets and therefore of the points of impact to the Vakhsh River. In particular, the possibility to make use of the cascade system envisaged at the outlet of the surface spillway, constituted by a sequence of chute and stilling basins, was evaluated.

This was found to be feasible for MLO1, being the elevation at the tunnel outlet some meters higher than the surface spillway bucket. The latter was somewhat modified, providing an additional stretch about 50 m long between the toe of the upstream chute and the final curve. This way the flow discharged by MLO1 can expand from the tunnel 12 m span to the whole canal width, and the specific flow becomes about 56 $m^3/s/m$.

Since the water speed for the design flow of about 1,840 m^3/s is substantially the same occurring when the surface spillway is operating, i.e. around 40 m/s, the hydraulic behavior is compatible with the analysis carried out for the latter structure, being the impact in the riverbed even more favorable due to the lower specific discharge.

The tunnel pressure stretch, with inner section of 15 m diameter, is branching into two circular tunnels with 10.8 m inner diameter, each provided with an emergency and sector gates chamber, running along the same axes of the surface spillway and connected with two corresponding channels.

This solution was thus definitively adopted for MLO1, in view of the following advantages:

- Reduced number of outlets in the Vakhsh River left bank;
- Lower specific flow at the outlet and correspondingly reduced scour in the riverbed;
- The crossing with faults 35 and other shearing zone is avoided;
- The downstream structures spans (tunnels and gates chambers) are lower than those corresponding to a single tunnel, with consequent positive impact on the construction activities and on the structural behavior.

As for the tunnel current lining sections, a conservative approach has been adopted in defining the lining thickness, which is normally in the order of 1/10 of the tunnel internal diameter or thicker.

All features of MLO1 remain unchanged for all dam alternatives.

In consideration of the fact that the outlets will operate during several years during the dam construction, measures were adopted to prevent cavitation, providing steel lining in the gates structure reach and providing adequate aeration downstream from the same control gates. Also, transitions from the gates sections to the tunnel current cross section are quite gradual, with a deviation angle in the order of 4°.

The middle level outlets must operate in safety condition with the discharge associated to the reservoir water level for all the dam alternatives with a maximum exceptional head of 150 m. The maximum velocity of the flow in the reach operating under pressure conditions was kept in the order of 20 m/s, and the free board in the reach operating in free flow conditions is higher that 25% of the total height of the tunnel.

The tunnel would be maintained operative as long as possible, so that in case of need, it can be used for performing the reservoir draw-down.

In the case of MLO1, a lifespan in the order of 12/15 to 25/30 years can be envisaged, depending upon the FSL alternative, before the sediments will have reached the intake level. At this point the tunnel would be put out of service and permanently plugged.

4.6.2 Middle Level Outlet 2

The possibility to discharge the flow into the surface spillway was also analyzed for this hydraulic facility. This would have implied to split the tunnel into two branches, as for MLO1, bringing one of them to the third surface spillway channel, while the remaining would have needed an additional dedicated cascade discharge. However, being in this case the tunnel outlet elevation about 1,130 m a.s.l., the connection would have been possible only with the lower stilling basin. Taking into account the need for allowing the flow to expand and the conditions required for the hydraulic jump formation, it was found that the space available was insufficient.

Therefore, the solution with vortex shafts was adopted for MLO2.

The intake of the Middle Level Outlet n. 2 is set at El. 1,140.0, and the pressure tunnel exhibits circular cross-section of 15.0 m diameter. The tunnel extends for about 750 m between the intake and the sector and emergency gates chamber. The maintenance gates chamber is located at a distance of about 400 m from the intake. Downstream from the sector and emergency gates chamber, a rectangular cross-section 15.8 m wide and 9.1 m high to the springline with a circular arch roof reaching a maximum height of 17.0 m has been adopted. The section is divided in two halves by a 1.80 m thick wall, each half flowing into a vortex shaft.

The proposed solution is constituted by an upper vortex chamber in which the twirling effect is imparted to the flow, followed by a drop shaft which discharges to the river through a free flow tailrace tunnel, provided with terminal chute and flip bucket.

It is recognized that by adopting the vortex shaft the risks of erosion due to the sediment transported by the flow would increase, due the twirling effect of the flow along the drop shafts and the outlet tunnels. However this risk was considered acceptable when the problem of dissipating the energy at the outlet and the possible consequences are analyzed.

Applications of vortex shafts are becoming more and more frequent, and no drawbacks have been reported in respect to their behaviour, even if it has to be recognized that so far the existing prototypes apparently have not been working under flows as high as that proposed for Rogun. The Consultant deems that with a proper investigation on model, this solution can be adopted for the Project.

It is ought to mention that provisions have been made at the crossing of tailrace tunnels with fault 35 and other shearing zone to allow for creeping effects or moderate displacements, as already done for the case of DT3.

Also in this case a structure constituted by “rings” of very thick, heavily reinforced concrete lining have been foreseen along a stretch somewhat longer then the fault development. To avoid an immediate repercussion on the tunnel internal surface, the “rings” are larger than the inner current section by some 3.0 m in diameter, and a second concrete lining matching with the tunnel current section is foreseen. The space between the two linings is filled with cellular concrete, which provides support to the internal lining.

For MLO2, the water discharge reaches 3,710 m³/s for the maximum exceptional head of 150 m, being the individual flow of each discharge equal to 1855 m³/s.

The geometrical features of the upstream stretch, operation modes and provisions adopted for MLO2 to face erosion / cavitation problems are conceptually the same implemented for MLO1.

The indicative lifespan of MLO2 before the sediment deposit reaches its intake is about 50/55 years. It shall be maintained operative as long as possible, being required for floods control in the long term during the plant operation of alternative with FSL 1290 m a.s.l. Only once the sediments will prevent its use, it will be permanently plugged. At this point, an additional module of the surface spillway shall be constructed, so to replace the discharge capacity of MLO2.

5 SPILLWAYS

5.1 Specific design criteria

The floods considered for the protection of Rogun dam, whatever is the selected alternative, are the PMF and the 10 000 years return period flood, as stated in the Design criteria.

The probabilistic analysis of floods in the Vakhsh River at the Rogun H.P.P. (Vol. 2 – Chapter 6 - Hydrology) gives the following results in terms of daily maximum and peak discharge for the two design floods.

Return period	Peak m3/s	Daily m3/s
10 000 years	5970	5690
PMF	8260	7770

Table 5-1 : Peak and daily maximum discharge

Assuming N orifice spillways and n gates for the surface spillway (for n=0, the surface spillway is a free-overflow spillway):

- for the 10 000 years flood, either with N-1 orifice spillways or with the n-1 gates of the surface spillway(s) (n-2 if the number of gates is more than 6), the maximum water level should be not higher than Reference Level (RL) less the total dry freeboard. Note that it is the tunnel with the largest expected discharge which should be considered as not being in operation.

- For the PMF, with the N orifice spillways and the n gates of the surface spillway, the maximum water level should be not higher than RL less the total dry freeboard.

In addition several safety principles are taken into account:

- The PMF is an exceptional extreme event during which the access to the power plant could be dangerous or unavailable. Therefore, the turbines cannot be considered as a spilling facility in the overall spillage capacity of Rogun. Only dedicated spillage facilities will be considered for the evacuation of the PMF. This approach is also generally applied on other projects designed by the Consultant.

- The consultant's practice and recommendation is not to rely on tunnel spillways only: they are subject to operational and maintenance issues and they are not flexible with respect to any variation above the design discharge. They are indeed hardly able to evacuate an additional flow when the water level rises more than expected. This gives an important constrain to the spillage system, allowing little uncertainty around the design flows adopted and little possibility of adapting the system to future trends in design floods (climate change…).

- The maximum normal water head in tunnel shall be limited to 120 m as already stated in 4.1.2.

- All discharge facilities shall be independent: One incident on one of them shall not impact any of the other devices.

- Tunnels crossing faults might be damaged because of fault movements during large earthquakes or due to accumulation of creeping. This can result in the unavailability of the said tunnel. Therefore, it is recommended to avoid fault crossing as much as possible and to adopt a special design when it is not possible to avoid such crossing to handle part of displacements and maintain the integrity of the structure.

5.2 HPI spillways

5.2.1 Description

This is further detailed in Appendix 5 – PMF Management.

In the solution proposed by HPI in the 2009/2010 study, the spillways for the dam at elevation 1300 masl (FSL 1290 masl) are the following:

- 3rd operation spillway (OP3)

- Remote spillway (RS)

- Operation shaft spillway

The total spillage capacity is 7 100 m^3/s, that is to say the value PMF as estimated in HPI study.

5.2.2 Assessment

It is shown in the present analysis that the solution designed so far suffers from several drawbacks:

- Maximum head on the tunnels beyond the value recommended by the Consultant.
- Necessity to re-design the distribution of flow between spillways in order to meet the N-1 criterion.
- Two devices sharing the same outlet.
- Lack of experience for such high capacity of vortex facilities for permanent flood evacuation facilities.

The Consultant considers that the safety conditions are not met. Therefore, other options are studied in the rest of the present appendix in order to select the appropriate solution for each dam height.

5.3 Flood management

5.3.1 Spillways available at the end of construction

As discussed in the Volume 3, Chapter 3, Appendix 3 “Flood Management During Construction”, the following tunnel spillways remain available at the end of construction as high level spillway for each dam height alternative:

Dam alternative	**Number of tunnels available**	**Number of gates per tunnel**	**Intake elevation of tunnels**
FSL = 1220 masl	1	3	1140 masl
FSL = 1255 masl	3	3	1 at 1145 masl and 2 at 1165 masl
FSL = 1290 masl	2	3	1190 masl

These tunnels can remain in operation until the reservoir is filled with sediments as the operating head does not exceed the criterion fixed by the Consultant (120 m).

5.3.2 Possible types of spillway

As described in the Volume 3, Chapter 3, Appendix 4 "Hydraulics of the Project Components", technical feasible solutions have been found for both surface spillway and pressured tunnel type spillway.

Dam alternative	Number gates per waterway	Width of the gates	Sill Elevation
FSL = 1220 masl	4	8m	1214 masl
FSL = 1255 masl	4	8m	1249 masl
FSL = 1290 masl	4	8m	1284 masl

5.3.3 Analysis of various flood management options

For each alternative, several combinations of spillway facilities (numbers and types) have been studied with respect to the design criteria and safety principles, as well as their sensitivity to the parameters described in 4.3 of Appendix 5 – Note PMF Management.

5.3.4 Conclusion and recommendation

The conclusions and recommendations for each alternative are the following:

Alternative FSL = 1220 masl
It is recommended to implement at Rogun the option with 2 surface spillways and 1 tunnel. It is recommended to add, at least, a surface spillway at Nurek to ensure protection of the dam against the PMF. The value of these works has been taken into consideration as the minimal value for the works to be implemented on the Cascade if this alternative is chosen.

Alternative FSL = 1255 masl
It is recommended to implement at Rogun the option with 3 tunnels and 1 surface spillway.

Alternative FSL = 1290 masl
It is recommended to implement at Rogun the option with 1 surface spillway and 2 tunnels.

5.4 Tunnels spillways

The high level spillways are constituted by a set of tunnels working under a maximum head of about 80-110 m with individual maximum discharge capacity close to 1,500 m^3/s. According to the design criteria for this kind of structures, when the dam will be completed they must allow to manage a flood with 10,000 years return time even if only N-1 devices (tunnels) are available and the PMF being all of them fully operative. Combined flood routing in Rogun and Nurek showed that with this arrangement it is actually possible to manage the 10,000 years return time and PMF floods.

The high level spillways shown in the layouts (Chapter 4 – Drawings) are located close to the dam footprint, at some 500 m downstream from the 3rd level diversion tunnel intake. They run initially in south-west direction and after 150-200 m starts bending to south-south east, reaching the right bank downstream from the dam after a further stretch of about 1,250 m.

Given the intakes elevations indicated above, the maximum head on the gates is about 100 m, to which a maximum water speed through the openings of the gates of about 33 m/s corresponds.

It shall be noted that they are placed some 30 m higher than the power intakes, so to assure that they will remain operative as long as possible, even if sediments would reach the headrace tunnels.

As for the general layout, the more upstream of the tunnels has been located some 15-20 m downstream from the Ionakhsh fault, so to avoid as much as possible that creeping or displacements impact on the structure stability. For alternatives requiring two or only one tunnel, those farther from the fault have been selected, so to increase the safety of the structures. In any case, before proceeding with the implementation of the project, specific investigations shall be carried out to decide the most appropriate alignment.

It is envisaged to provide each tunnel with two maintenance gates and sets of three emergency and sector gates, following the concept adopted in other tunnels. As for the hydraulic behaviour, the stretch downstream from the sector gates is working under free-flow condition, so that the discharge capacity is controlled by the headlosses in the pressure stretch and by the orifice of the sector gates.

If the highest dam option is considered, the high level tunnel spillways outlet would be at about el. 1,180 m a.s.l., about 200 m higher than the riverbed.

There are two possible ways to release the flow to the river:

- by providing the tunnels with an open air outlet structure constituted by a chute with terminal flip bucket, bringing down the water as much as possible and then allowing it to jump into the river;

- by adopting some device for dissipating energy and releasing the water as low as possible with respect to the riverbed.

With respect to the above alternatives, it has to be considered the following:

The first of them is adopted in several plants, when the head is not too high and the impact area is such that the scour effect can be effectively controlled, due to the natural conditions of the riverbed of by implementing adequate protective measures. In the case of Rogun project, the head is considerable and the natural conditions of the riverbed are matter of concern, due to the possibility of triggering lateral slope instability phenomena.

If a long chute is designed, which would allow protecting the right bank from erosion phenomena, high water speeds are reached at the end of the same, and the risks of cavitation are not negligible, even if adequate measures are envisaged. Also, even if the area of influence of the jump and the maximum scour depth can be evaluated, the consequences on the banks stability are not fully predictable.

The solution constituted by the vortex spillways in this case was not taken into consideration, due to the height of the shaft required. Another option would have been that to adopt a large size vertical shaft having the effect of dissipating the energy by impact like a vertical stilling basin. No information about the application of the latter solution to other large hydro plant has been found.

The goal to control cavitation risks and to dissipate as much energy as possible was achieved by combining in cascade two commonly adopted hydraulic provisions, i.e. chutes with such an head that the water speed remains in the range of values acceptable for hydraulic structures and stilling basins.

In fact, with limiting the maximum gross head to some 75 m and considering the headlosses along the stretch, the water speed is not higher than 40 m/s at the toe of the chute and the residual energy is dissipated into the stilling basin. Aeration shafts have been foreseen along the chute for a better control of the cavitation phenomena.

Computation carried out showed that the cavitation index remains higher than 0.1, which implies that the design is acceptable with adequate provisions.

It has been also shown in the hydraulic report that most of the energy is dissipated along the cascade system of chute and stilling basins, thus reducing substantially the problem of scour in the river.

In fact only the energy corresponding to the last chute head, reduced by the headlosses, remains at the end of the cascade system, where a terminal flip bucket is placed.

The above described solution has been adopted for all HLTSs proposed, just adapting to the morphological conditions and difference in total elevation the slopes and height of the chutes.

As in the case of other tunnels, the HLTSs will remain operative until the sediments will reach their intakes, finally preventing the flows discharge. Their lifespan would range from 25/30 years for alt. FSL 1220 up to 70/75 years for alt. FSL 1290.

At this point the HLTSs will be plugged and the surface spillway shall be completed so to avail of its full discharge capacity.

5.5 Surface spillway

5.5.1 Design criteria

The surface spillway, as an ultimate stage flood evacuation organ, should replace in the long term the flood evacuation organs planned for the beginning of the useful life of the project (first stage evacuation organs).

Its discharge capacity must be, accordingly, equal to the peak discharge of the Probable Maximum Flood (PMF).

It is supposed to be partially or fully operational when the sediment load in the reservoir affects the discharge capacity of tunnels having low level intakes.

Because of the sediment load, the surface spillway is to be designed and built in such a way that erosion damage caused by sediments running along it can be easily repaired by isolating part of the spillway.

5.5.2 Description

The spillway structure consists of three independent conduits having the following components:

- an approach bay,
- a control sill with four gated bays (the total number of gated bays is 3x4=12),
- a sub-horizontal free surface flow channel along tunnels excavated through the high hill on the right bank,
- an open air stepped chute channel with intermediate energy dissipation. Each step is 70 m high and consists of a steep chute (0.8H:1V) followed by a sub-horizontal reach where a stilling basin dissipates part of the energy. There are three steps in the two higher dam alternatives and just two steps in the smallest one.
- a ski-jump end structure in the last step of the chute,
- a plunge pool in the river bed.

All these components have been designed and dimensioned on the basis of the state of the art and have proved to be feasible. Model tests are recommended anyhow.

The proposed layout implies large excavation works, to which quite high slopes correspond. Stabilization measures of the same have been envisaged, consisting in a pattern of long rock anchors and protection of the excavation surface with reinforced shotcrete layers.

According to the conclusions of Volume 3 – Chapter 3 – Appendix 5 “PMF Management”, only one “module” will be required in the initial stage of the project operation for alternatives with FSL 1290 and 1255, whilst two modules are adopted for the alternative with FSL 1220. For the two highest alternatives, this is a consequence of the need of keeping the flow discharged from Rogun within values which assure that the safety of Nurek is not impaired.

The final complete configuration, constituted by three modules, will be needed in the long term, once the reservoir sedimentation will prevent the discharge of the floods though other hydraulic facilities, in particular the high level spillway tunnels.

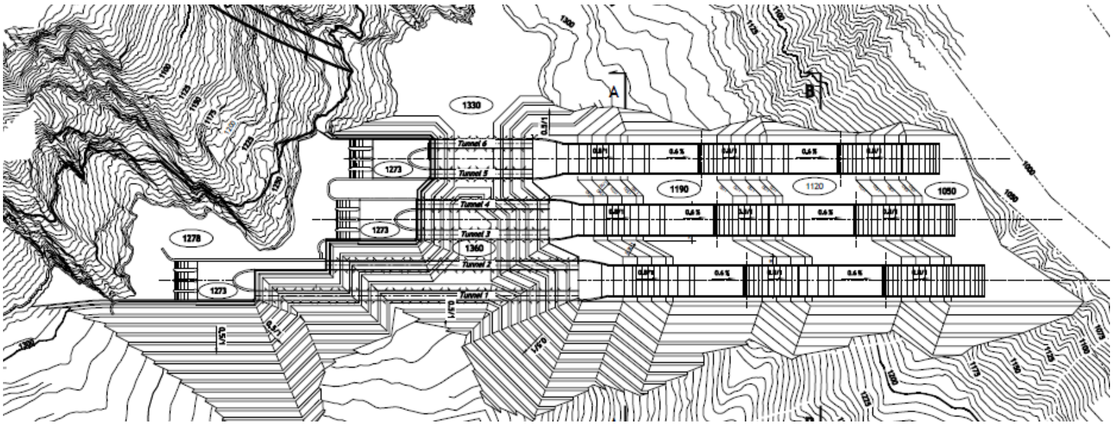


Figure 5-1: Plan view of the Surface Spillway

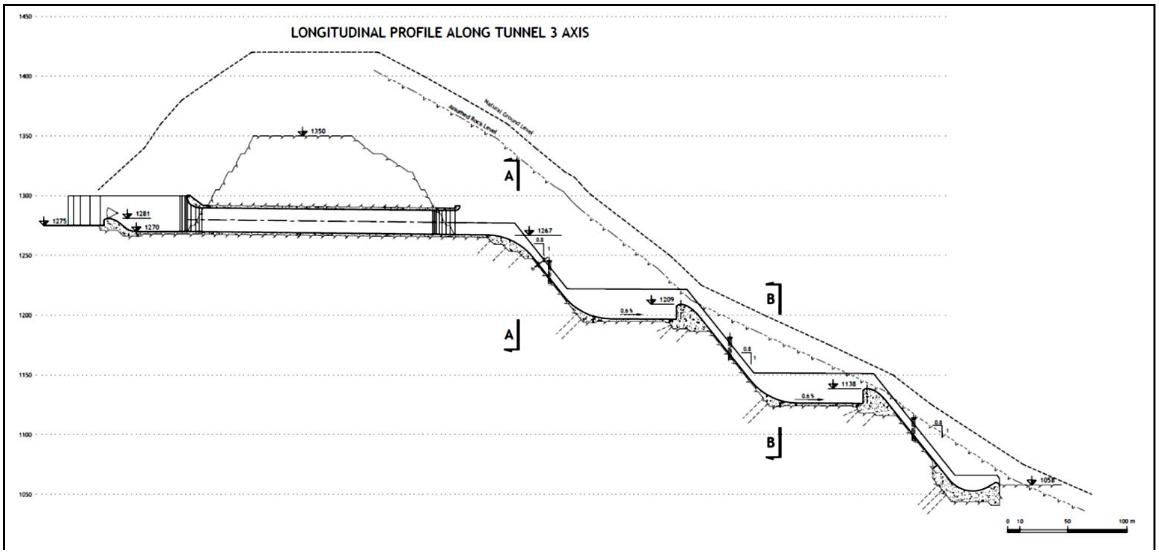


Figure 5-2: Longitudinal section of the stepped spillway

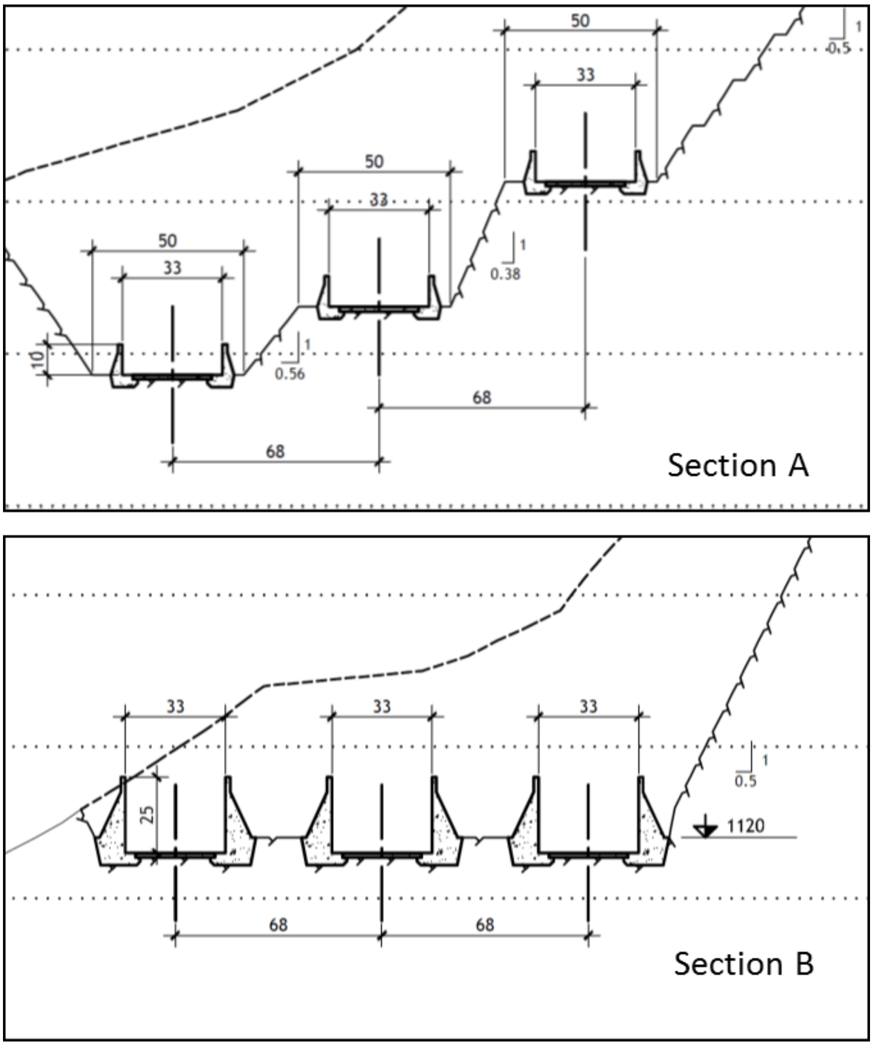


Figure 5-3: Transverse section of the stepped spillway

5.6 Consistency with the downstream cascade

In addition to the various options for flood management presented in §5.3, the Consultant raises an additional alternative: the "Downstream cascade protection".

Because Nurek and other downstream structures have been designed in the 70's, their design flood obeys to the Soviet standard and is defined as the 10 000 years return period flood. The Consultant developed a PMF management that envisages the possibility of storing part of the PMF in Rogun and release a flow downstream of Rogun acceptable to the spillage capacities of Nurek and other downstream structures. It makes use of:

- Tunnels spillway able to control the outflow;
- The Rogun reservoir operation: the reservoir lowering in winter creates a large storage volume used to dump the PMF.

Details of this study are presented in Appendix 5. It shows that:

- to ensure the safety of Nurek and the downstream Vakhsh cascade, the flow has to be controlled and limit to 4500 m^3/s at the outlet of Rogun. This can be ensured by 3 tunnels spillways of 1500 m^3/s capacity each.
- in April, at the beginning of the PMF, the reservoir should not be higher than 1270 masl for the dam alternatives FSL=1290 masl, 1210 masl for the dam alternatives FSL=1255 masl, 1140 masl for the dam alternatives FSL=1220 masl.
- the "normal" reservoir level in April is lower than the one necessary to damp the PMF; therefore the energy production is not impacted by the PMF management;
- for the dam alternatives FSL=1220 masl, the reservoir volume is not sufficient to store the PMF, the dam shall either be re-profiled completely to be heightened or additional discharge capacity shall be provided to the different plants downstream.

The 3 tunnels are not sufficient to ensure to full safety of Rogun: the criteria of n-1 tunnels for the 10 000 years flood is not respected, to rely only on tunnel spillways is not recommended and the maximum spillway capacity is 4500 m^3/s which means that the dam safety relies on the reservoir operation, i.e. on human decisions. Therefore, those 3 tunnels are considered not to ensure a full proof dam safety and the Consultant recommends implementing a surface spillway to complete the discharge capacity and ensure the full safety of Rogun dam.

As for the solution "combination surface/tunnel spillways" presented in earlier, the surface spillway will need to be extended in the future to be able to discharge the PMF daily discharge of 7770 m^3/s. At short term, it is possible to construct only the channel necessary to complete the discharge capacity provided by the tunnels.

	Configuration at short term	**Maximum discharge (m^3/s)**
FSL=1290 masl	1 Tunnel + 1 module of surface spillway	8400
FSL=1255 masl	3 Tunnels + 1 module of surface spillway	8400
FSL=1220 masl	1 Tunnel + 2 modules of surface spillway + strengthening of the cascade	8400

Table 5-2 : Spillways configuration - "Downstream cascade protection" alternative

It has to be noted that in any case the "Downstream cascade protection" function is efficient only during the first period of the Rogun dam life span. At long term (several decades), the protection will not be ensured anymore.

5.7 Reservoir Draw-down

Reservoir draw-down operation may be performed by making use of the available hydraulic facilities, namely Mid-level Outlets 1 & 2 and High Level Spillway Tunnels.

As mentioned in Appendix 4 – Hydraulics of the Project Components, the above facilities will be maintained operative until the level of the sediments deposit will have reached the corresponding intakes. It is fact considered that the need for lowering the reservoir may arise at some point of the plant useful life, for facing any emergency situation relevant to the permanent structures of the plant or in connection with the reservoir slopes stability. Some extraordinary maintenance operation at levels normally remaining under the water level could also be required.

The configuration of the facilities is different from an alternative to the other, as follows:

FSL 1,290 m a.s.l.: 2 High Level Spillway Tunnels at el. 1,190 m a.s.l.
Mid-Level Outlet 2 at el. 1,140 m a.s.l.
Mid-Level Outlet 1 at el. 1,085 m a.s.l.

FSL 1,255 m a.s.l.: 2 High Level Spillway Tunnels at el. 1,165 m a.s.l.
1 High Level Spillway Tunnels at el. 1,145 m a.s.l.
Mid-Level Outlet 1 at el. 1,085 m a.s.l.

FSL 1,220 m a.s.l.: 1 High Level Spillway Tunnel at el. 1,140 m a.s.l.
Mid-Level Outlet 1 at el. 1,085 m a.s.l.

As above recalled, these discharge facilities will be progressively put out of service, following the reservoir siltation level.

Therefore, the possibility to perform the draw-down will vary with the time, according to the discharges facilities configuration that will be available in the specific moment, both in terms of minimum reservoir level achievable and in term of time required to evacuate the corresponding volumes.

An evaluation of some possible scenarios was performed, initially considering the availability of the highest discharge tunnels only, and progressively adding the lower ones. The time for lowering the reservoir was calculated taking into account the global discharge capacity of the various tunnels at various levels as well as the Vakhsh River incoming flows. As a reference, the average flows of the wet season (April/September) and of the dry season (October/March) were considered, 1,041 and 233 m^3/s respectively.

As further constrains, it was considered to limit the global discharge to 4,040 m^3/s, following the criteria set in order to protect the cascade, and to put out of operation a certain discharge facility when the relevant hydraulic head reaches 120 m.

The results obtained for the different alternatives are reported in the following tables.

The results of the above exercise are to be considered indicative only, being provided to show the capability of the available discharge facilities to lower the reservoir in different situations.

It should be noted that no constrain has been considered as far as the reservoir lowering rate is concerned. Therefore the above durations are to be considered the maximum achievable for the different configurations and the corresponding incoming flows, with the only constrains above indicated in respect to the maximum discharged flow and maximum hydraulic head.

ALTERNATIVE FSL = 1220

Available hydraulic facilities

1 HLTS el.	1,140.00	m a.s.l.	1 HLTS el.	1,140.00	m a.s.l.
			1 MLO1 el.	1,085.00	m a.s.l.
Incoming Flows m^3/s	wet s. 1041.00	dry s. 233.00	Incoming Flows m^3/s	wet s. 1041.00	dry s. 233.00
Draw-down time (days)	169.53	62.96	Draw-down time (days)	45.51	20.15
Res. Bottom Elev. m a.s.l.	1,185.00	1,147.50	Res. Bottom Elev. m a.s.l.	1,185.00	1,147.50
Res. Volume (hm^3)	3,058.17	1,537.75	Res. Volume (hm^3)	3,058.17	1,537.75
			Draw-down time (days)	68.87	38.82
			Res. Bottom Elev. m a.s.l.	1,100.00	1,090.00
			Res. Volume (hm^3)	607.33	479.00

ALTERNATIVE FSL = 1255

Available hydraulic facilities

2 HLTS el.	1,165.00	m a.s.l.	2 HLTS el.	1,165.00	m a.s.l.	2 HLTS el.	1,165.00	m a.s.l.
			1 HLTS el.	1,145.00	m a.s.l.	1 HLTS el.	1,145.00	m a.s.l.
						1 MLO1 el.	1,085.00	m a.s.l.
Incoming Flows m^3/s	wet s. 1041.00	dry s. 233.00	Incoming Flows m^3/s	wet s. 1041.00	dry s. 233.00	Incoming Flows m^3/s	wet s. 1041.00	dry s. 233.00
Draw-down time (days)	65.04	43.10	Draw-down time (days)	28.14	23.98	Draw-down time (days)	24.47	20.09
Res. Bottom Elev. m a.s.l.	1,180.00	1,170.00	Res. Bottom Elev. m a.s.l.	1,180.00	1,170.00	Res. Bottom Elev. m a.s.l.	1,180.00	1,170.00
Res. Volume (hm^3)	2,784.00	2,389.00	Res. Volume (hm^3)	2,784.00	2,389.00	Res. Volume (hm^3)	2,784.00	2,389.00
			Draw-down time (days)	37.16	50.81	Draw-down time (days)	26.11	22.69
			Res. Bottom Elev. m a.s.l.	1,170.00	1,152.50	Res. Bottom Elev. m a.s.l.	1,170.00	1,152.50
			Res. Volume (hm^3)	2,389.00	1,697.75	Res. Volume (hm^3)	2,389.00	1,697.75
						Draw-down time (days)	47.07	42.17
						Res. Bottom Elev. m a.s.l.	1,100.00	1,090.00
						Res. Volume (hm^3)	607.33	479.00

ALTERNATIVE FSL = 1290								
Available hydraulic facilities								
2 HLTS el. 1,190.00 m a.s.l.			2 HLTS el. 1,190.00 m a.s.l.			2 HLTS el. 1,190.00 m a.s.l.		
			1 MLO2 el. 1,140.00 m a.s.l.			1 MLO2 el. 1,140.00 m a.s.l.		
						1 MLO1 el. 1,085.00 m a.s.l.		
Incoming Flows m^3/s	wet s. 1041.00	dry s. 233.00	Incoming Flows m^3/s	wet s. 1041.00	dry s. 233.00	Incoming Flows m^3/s	wet s. 1041.00	dry s. 233.00
Draw-down time (days)	124.53	64.62	Draw-down time (days)	45.63	35.27	Draw-down time (days)	45.47	34.73
Res. Bottom Elev. m a.s.l.	1,202.50	1,195.00	Res. Bottom Elev. m a.s.l.	1,202.50	1,195.00	Res. Bottom Elev. m a.s.l.	1,202.50	1,195.00
Res. Volume (hm^3)	4,017.75	3,606.50	Res. Volume (hm^3)	4,017.75	3,606.50	Res. Volume (hm^3)	4,017.75	3,606.50
			Draw-down time (days)	91.26	74.06	Draw-down time (days)	54.10	41.53
			Res. Bottom Elev. m a.s.l.	1,155.00	1,145.00	Res. Bottom Elev. m a.s.l.	1,155.00	1,145.00
			Res. Volume (hm^3)	1,796.50	1,476.50	Res. Volume (hm^3)	1,796.50	1,476.50
						Draw-down time (days)	71.59	59.90
						Bottom elevation m a.s.l.	1,100.00	1,090.00
						Res. Volume (hm^3)	607.33	479.00

6 MULTI LEVEL INTAKES

In Volume 2 – Chapter 6 “Sedimentation” the total yearly solid run-off of the Vakhsh River is estimated to range between 87 and 140 millions of tonnes per year, or between 62 and 100 Million m^3 per year.

This huge amount of sediments represents a serious drawback for the plant, since it has a considerable impact on the useful lifespan of the project and on the energy generation.

Therefore, the possible alternatives for mitigating such negative impacts have been analyzed, to find out the most effective one that can be implemented within a reasonable range of cost.

The Consultant firstly proposed to implement facilities for performing at least the sediment flushing in the areas more sensitive to the problem of the silting, i.e. the area of the permanent power intakes, through tunnels located in the left bank, just below the power intakes. Even though this solution did not represent the optimum, it was considered that it could at least provide protection to the equipment and to the power waterways during some time, allowing operating the plant for a longer period.

However, this solution was deemed not properly behaving, due to the fact that it should have worked under high head. Some other drawbacks were noted, linked also to the nature of the Obi-Shur creek, which is the only possible point of discharge for a tunnel starting from the power intakes area.

Therefore, the possibility to implement multi-level intakes was analysed. Such provision would allow to continue operating the plant even when the silt deposit will be higher than the power waterways inlets, offering a solution for mitigating the sedimentation effects with a relatively low cost. The possibility to draw turbidity currents into the power inlets was also taken into consideration. It shall be noted that with this solution the discharged water is anyway passed through the turbines, generating energy. However, before deciding whether this operation is feasible, further studies have to be performed, aimed at assessing the eventual negative impact of the suspended sediments on the hydro-mechanical equipment, as well as at verifying the adequacy of the solution proposed.

The multi-level intakes solution proposed for Rogun consists of an inclined concrete culvert, resting on the bank slope in correspondence with the power waterways inlets, provided with openings at various levels, from the power inlets up to the dam crest elevation. Whenever it would be decided to drag the turbidity currents into the power waterways, the culvert would start from elevation 1,090 m a.s.l.

The preliminary design of the intakes considers a concrete culvert some 16.0 m wide and 12.0 m high, so that the water velocity inside the duct for a 270 m^3/s flow is about 1.73 m/s, which provides negligible headlosses. Water velocity through the intakes openings (preliminarily 6.50 m wide by 8.25 m high each, two openings each intake) is in the order of 2.52 m/s, which allow to keep low headlosses while providing a velocity sufficient to drag the turbidity currents.

The power waterways inlets proper would be provided with removable trashracks (2 openings 7.5 m wide by 42 m high, maximum water velocity on gross area 0.43 m/s). Also the culvert multi-level intakes would be provided with removable trashracks featuring widely spaced bars, the only purpose of which is that to avoid that large floating bodies can enter to the culvert. The intakes would be provided with steel elements for closure in order to exclude those that will be progressively submerged by the sediments deposit. Those elements are not due to assure watertightness and will be operated under water pressure balanced conditions only.

More considerations and details can be found in paragraph 1.1.5 of Appendix 4 – Hydraulics of the project components under Volume 3 – Chapter 3 - Alternatives Design.

7 POWER HOUSE

The Powerhouse Cavern, located in a sedimentary complex constituted by sandstone and siltstone, is approximately 21 m wide, 69 m high and 220 m long.

The access / erection area level is set at el. 974.6 m a.s.l., the turbines axes at el .959.0 m a.s.l. and the draft tubes floor at 933.4 m a.s.l.

Large amounts of excavation works have been already carried out, in particular in the area of Units 5 and 6, where the elevation of the spiral case was reached, and considerable quantities of excavation supports were installed in the past.

The complex constituted by the caverns of the powerhouse and transformers hall is protected against the external water pressure by a system of drainage galleries surrounding all the area, located at various elevations, from which grouting curtains and pattern of large drillholes are performed. This way the water present in rock mass is drained and, according to the results of the hydrogeological model implemented by HPI, there is a strong reduction of the pressure in correspondence with the structures.

The present status of the powerhouse has been already discussed in detail in Phase I assessment, in which the existing problems were also underlined and the progress of the analyses carried out reported.

As there indicated, in order to carry out an independent assessment, the consultant has prepared and analyzed the results of a 2D model, implementing reliable constitutive laws suitably representing both the elastic-plastic and the time-dependent behaviours of the rock mass, as a the tool for the powerhouse complex assessment required by the TEAS studies.

The conclusions drawn from the modelling work performed by the Consultant confirm that the present status of the cavern excavation is critical and that under the design provisions proposed so far its stability cannot be achieved. Therefore, a different set of stabilization measures is proposed, as follows:

1. Installation of rock anchors on both the downstream and upstream sidewalls, between the rock dowels already in place above the present excavation level in the Units 5 and 6 zone, with the same characteristics as those already installed in the MH cavern, although their length is estimated to be 35 m approximately.

2. Stabilization of the highly de-stressed rock mass in the pillar between the MH and TH caverns, to be achieved by installing steel piles (micro-piles) with properly spaced valves ("tube à manchette" - sleeved pipe), to allow for consolidation grouting at predetermined pressure levels. Contrary to what is commonly done in soil grouting with the usual pipe sleeve, the suggestion here is use the Multiple Packer Sleeved Pipe (MPSP) system developed in the early 1990 by Rodio (Bruce and Gallavresi 1988, Barla and Jarre 1986) and successfully applied in a number of projects.

Other possible stabilization measures have been outlined in paragraph 4.10 of Phase I report.

The provision of a suitably distributed monitoring system before proceeding with the next excavation stages is considered mandatory. The records of the convergence increments shall regard a selected set of significant points along the sidewalls, such to exhaustively represent the deformations of the same, with the excavation progress, according to a continuous procedure. This method implies that monitoring plays a very active role in both the design and construction, allowing planned modifications to be carried out within an agreed contractual framework that involves all the main parties.

It is to be remarked that until the immediately needed stabilization measures are completely installed and a fully functioning monitoring system has been provided, the excavation activities in the Cavern shall not be resumed.

With the implementation of those provisions, the powerhouse can be brought to the required safety conditions and can be used for the scope for which has been constructed.

It is here recalled that the maximum installed capacity to be considered in the studies of the alternatives is 3,600 MW, as indicated in the TEAS methodology, i.e. the capacity for which the present structure has been designed. For the above, based on the Consultant's evaluations, the powerhouse can accommodate the generation equipment corresponding to the various alternatives proposed in the studies, which foresee the same number of units as the original scheme, without need for major modifications.

8 ELECTRO-MECHANICAL EQUIPMENT

According to the design carried out by HPI for dam at FSL = 1,290, the powerhouse is equipped with six Francis units, each rated 600 MW and working under a barycentric head of 268 m. The project foresees a preliminary arrangement for the start up of generation when the dam is under construction.

The characteristics of the generating units, in the final stage, are listed herein under.

Final Arrangement Features

Number of units	6
Maximum exceptional reservoir level (m a.s.l.) (a)	1291.5
Maximum normal reservoir level (m a.s.l.) (b)	1290.0
Minimum reservoir level (m a.s.l.) (c)	1185.0
Estimated maximum gross head (a-d -1)	323.4
Estimated minimum gross head (c-e)	205.2

Turbine

Design net head (m)	245
Baricentric head (m)	268
Minimum normal net head (m)	185
Maximum net head (m)	320
Power at minimum head (MW)	360
Maximum rated power (MW)	600
Maximum power at overload conditions (MW)	810
Rotational speed (rpm)	166.7
Runway speed (rpm)	310
Expected max. efficiency (%)	96.2
Maximum flow (m^3/s)	271
Required submergence measured from spiral case axis (m)	-13.6
Turbine centerline elevation (m a.s.l.)	959.0
Bottom of tailrace tunnel (m a.s.l.) (d)	967.1
Water el. in tailrace channel with three units at 811.5 m^3/s (m a.s.l.) (e)	979.8
Available submergence with three units at 811.5 m^3/s (m a.s.l.)	-20.8

Generator

Rated output (MVA)	666
Rated voltage (kV)	15.75
Rated power factor (p.u.)	0.9
Rated frequency (Hz)	50
Number of poles	36
Generator pit external diameter (m)	15.3
Insulation (information to be confirmed)	class F

To the above features, connected with the reference case with dam at FSL 1290, a plant factor of 0.44 is associated.

As indicated in other sections of the report, the following alternatives have been analyzed:

	FSL = 1220 masl	FSL = 1255 masl	FSL = 1290 masl
High installed capacity	2 800 MW	3 200 MW	3 600 MW
Medium installed capacity	2 400 MW	2 800 MW	3 200 MW
Low installed capacity	2 000 MW	2 400 MW	2800 MW

Preliminarily for each alternative the number of units, capacity and other main features have been defined, as reported in the table of the section "Key data for each alternative".

The generating units have been defined considering units of capacity not higher than that of the original design, which is in the order of 600 MW.

The same number of units has been adopted for all options, in consideration of the present progress of the works, so that the flow can be evenly distributed in the two existing tailrace tunnels.

In fact, it was considered that the number of units to be installed shall be pair because it would be unjustified to have maximum water velocities and maximum head losses in one tunnel higher than in the other.

By maintaining the number of units equal to that foreseen in the plant layout proposed by HPI, it is possible to vary the installed capacity, reducing it, without major modifications of the layout and of the works already made.

The only possible modification of the number of units which can be made in order to reduce their total cost would be that to reduce them from six to four. Such change would have a very marginal impact on the total cost of the plant, whilst would allow reducing the unit capacity only in the case of 2,000 MW installed, from 600 to 500 MW (see below table):

Total installed (MW)	Unit capacity with 4 units (MW)	Unit capacity with 6 units (MW)
2000	500	333.3
2400	600	400.0
2800	700	466.7
3200	800	533.3
3600	900	600.0

The feasibility of units with very high capacities, above those presently foreseen, should be carefully investigated. In addition, it should be recalled that, as already commented, one of the major electromechanical problem is the transportation of very large unit transformers (three phase transformers are required; otherwise the layout should be substantially changed with modification of the works already done). This was possible at the time of Soviet Union, but it is no more viable nowadays due to the present relationships between the Central Asia countries. The assembly of the transformers at site, which apparently is the only possible solution with a unit capacity of 600 MW, was proposed by a potential supplier, but it was never applied in the world and may be cause of possible future problems in case the maintenance of a transformer is required. We remark that, according to international statistics, the problems on transformers are among the more frequent ones in the electromechanical equipment of hydro plants.

By reducing the unit capacity from 600 MW to lower values, we would have higher probabilities to mitigate the transformers transportation difficulties.

As for the hydro-mechanical equipment, no substantial changes have been deemed necessary with respect to what foreseen in the original design.

Therefore, the headrace tunnels are provided with a gate shaft where a servomotor operated wheel mounted gate and an upstream maintenance gate are installed. The intake works are located about 100-150 m upstream from the gates shaft. At the intakes mouth, trashracks are installed, but no racking machine is foreseen, due to the high head between the FSL and the intakes elevation. A gantry crane for removing and cleaning the trashracks is instead foreseen, stored at dam crest elevation.

The portion of tunnel after the gates is steel lined, as well as the penstocks.

As for the various hydraulic facilities, they are essentially spillway tunnels, for which three sets of gates are foreseen, i.e. the maintenance gates located at a few hundred meters after the intake, emergency gates and sector gates, being the latter normally operated to control the flow. The emergency gates can be operated under flow to close the tunnels in case the sector gates are out of order.

9 KEY DATA OF EACH ALTERNATIVE

In the following table, the main features of the Rogun project for the three alternatives are presented.

9.1 Dam

	FSL = 1290 masl	FSL = 1255 masl	FSL = 1220 masl
Dam crest	1300 masl	1265 masl	1230 masl
Foundation level	965 masl	965 masl	965 masl
Dam height	335 m	300 m	265 m
Crest length	660 m	565 m	500 m
Crest width	20 m	20 m	20 m
Core crest level	1296.25 masl	1261.25 masl	1226.25 masl
Maximum water level	1293.45 masl	1257.25 masl	1220.25 masl
Minimum operational level	1185 masl	1161 masl	1137 masl
Reservoir active storage	10 300 hm^3	6 454 hm^3	3 927 hm^3
Total reservoir capacity	13 300 hm^3	8 550 hm^3	5 220 hm^3
Average yearly inflows	20 100 hm^3	20 100 hm^3	20 100 hm^3
Dam slopes	US 2.4 H/1V DS 2 H/1V	US 2.4 H/1V DS 2 H/1V	US 2.4 H/1V DS 2 H/1V
Stage 1 elevation	1110 masl	1090 masl	1075 masl
Core crest thickness	8 m	8 m	8 m
Core slopes	US: 0.5 H/1V DS -0.1 h/1V	US: 0.5 H/1V DS -0.1 h/1V	US: 0.5 H/1V DS -0.1 h/1V
Filters thickness	US: 2 layers of 10 m each above the minimum operation level and one layer of 10 m below DS : 2 layers of 10 m each	US: 2 layers of 10 m each above the minimum operation level and one layer of 10 m below DS : 2 layers of 10 m each	US: 2 layers of 10 m each above the minimum operation level and one layer of 10 m below DS : 2 layers of 10 m each

9.2 River diversion structures

	FSL = 1290 masl	FSL = 1255 masl	FSL = 1220 masl
Diversion tunnel 1			
Total tunnel length	1439.5 m	1439.5 m	1439.5 m
Pressure Stretch Section (D-Shape)	96.55 m²	96.55 m²	96.55 m²
Low intake level	989.60 masl	989.60 masl	989.60 masl
High intake level	1020 masl	1020 masl	1020 masl
Design head	120 m	120 m	120 m
Minimum operational level	989.60 masl	989.60 masl	989.60 masl
Maximum operational level	1110 masl	1110 masl	1110 masl
Design discharge	2490 m3/s	2490 m3/s	2490 m3/s
Diversion tunnel 2			
Total tunnel length	1420.7 m	1420.7 m	1420.7 m
Pressure Stretch Section (D-Shape)	96.55 m²	96.55 m²	96.55 m²
Low intake level	1001.80 masl	1001.80 masl	1001.80 masl
High level intake	1020 masl	1020 masl	1020 masl
Design head	120 m	120 m	120 m
Minimum operational level	1001.80 masl	1001.80 masl	1001.80 masl
Maximum operational level	1110 masl	1110 masl	1110 masl
Design discharge	2490 m3/s	2490 m3/s	2490 m3/s
Diversion tunnel 3			
Total tunnel length	1560 m	1560 m	1560 m
Diameter of Pressure Stretch	15 m	15 m	15 m
Intake level	1035.0 masl	1035.0 masl	1035.0 masl
Outlet portal level	1023.45 masl	1023.45 masl	1023.45 masl
Design head	150 m	150 m	150 m
Minimum operational level	1035 masl	1035 masl	1035 masl
Maximum operational level	1160 masl	1170 masl	1165 masl
Design discharge	3694 m3/s	3694 m3/s	3694 m3/s

The data refer to the condition of maximum exceptional head.

9.3 Spillways

9.3.1 Middle level outlet

	FSL = 1290 masl	FSL = 1255 masl	FSL = 1220 masl
Middle level outlet 1			
Total tunnel length	1464.0 m	1464.0 m	1464.0 m
Diameter of Pressure Stretch (Circular)	15 m	15 m	15 m
Intake level	1083.50 masl	1083.50 masl	1083.50 masl
Outlet tunnel level	1077.60 masl	1077.60 masl	1077.60 masl
Design head	150 m	150 m	140 m
Minimum operational level	1100.0 masl	1100.0 masl	1100.0 masl
Maximum operational level	1215 masl	1210 masl	1210 masl
Design discharge	3686 m3/s	3564 m3/s	3564 m3/s
Middle level outlet 2			
Total tunnel length	1117.0 m		
Diameter of Pressure Stretch (Circular)	15 m		
Intake level	1140 masl		
Outlet tunnel level	1026.80 masl		
Design head	150 m		
Minimum operational level	1160 masl		
Maximum operational level	1270 masl		
Design discharge	3710 m3/s		

The data refer to the condition of maximum exceptional head.

9.3.2 High level tunnels

	FSL = 1290 masl	FSL = 1255 masl	FSL = 1220 masl
High level tunnel 1			
Total tunnel length	1264.1 m	1385.7 m	1416.8 m
Diameter of Pressure Stretch (Horse-shoe)	10 m	10 m	10 m
Intake level	1190 masl	1145 masl	1140 masl
Outlet tunnel level	1177.70 masl	1131.74 masl	1126.30 masl
Outlet Structure level	1000.00 masl	1000 masl	1000 masl
Outlet Spillway length	440.3 m	376.6 m	367.3 m
Design head	100 m	110 m	80 m
Minimum operational level	1190 masl	1145 masl	1140 masl
Maximum operational level	1290 masl	1255 masl	1220 masl
Design discharge	1570 m3/s	1640 m3/s	1410 m3/s
High level tunnel 2			
Total tunnel length	1410.1 m	1501.6	
Diameter of Pressure Stretch (Horse –shoe)	10 m	10 m	
Intake level	1190 masl	1165 masl	
Outlet tunnel level	1176.57 masl	1151.66 masl	
Outlet Structure level	1000 masl	1000 masl	
Outlet Spillway length	415.9 m	385.6 m	
Design head	100 m	90 m	
Minimum operational level	1190 masl	1165 masl	
Maximum operational level	1290 masl	1255 masl	
Design discharge	1570 m3/s	1490 m3/s	
High level tunnel 3			
Total tunnel length		1585.1 m	
Diameter of Pressure Stretch (Horse –shoe)		10 m	
Intake level		1165 masl	
Outlet tunnel level		1149.85 masl	
Outlet Structure level		1000 masl	
Outlet Spillway length		371.2 m	
Design head		90 m	
Minimum operational level		1165 masl	
Maximum operational level		1255 masl	
Design discharge		1490 m3/s	

The data refer to the condition of maximum exceptional head.

9.3.3 Multi-level Intakes

	FSL = 1290 masl	FSL = 1255 masl	FSL = 1220 masl
Intakes culverts developed length	312.5 m	259.1 m	205.8 m
Culverts Inner Dimensions	16 x 12 m	16 x 12 m	16 x 12 m
Upper Power Intakes level (Units 1, 2, 5, 6)	1167 masl	1140 masl	1115 masl
Lower Power Intakes level (Units 3, 4)	1152 masl	1130 masl	1150 masl
Number of Intakes active inlets	4	3	2
Higher Intakes active inlets level	1179.3 masl	1154.3 masl	1129.3 masl
Lower Intakes active inlets level	1104.3 masl	1104.3 masl	1104.3 masl
Power Intakes Gates Design Head	140 m	130 m	115 m

9.3.4 Surface spillway

	FSL = 1290 masl	FSL = 1255 masl	FSL = 1220 masl
First Stage			
Number of modules	1	1	2
Number of tunnels	2	2	4
Final Stage			
Number of modules	3	3	3
Number of tunnels	6	6	6
Tunnel width (D-shape)	9.40 m	9.40 m	9.40 m
Tunnel height (D-shape)	15 m	15 m	15 m
Fall height	224 m	189 m	154 m
Number intermediate spillways	2	2	1
Width of intermediate spillways	33 m	33 m	33 m
Design discharge (PMF)	7800 m3/s	7800 m3/s	7800 m3/s
Sill level	1284 masl	1249 masl	1214 masl
Flip bucket exit level	1060 masl	1060 masl	1060 masl
Minimum operational level	1284 masl	1249 masl	1214 masl
Maximum operational level	1296 masl	1261 masl	1226 masl

9.4 Power house and EM Equipment

Final Dam Elevation 1290 m.a.s.l.

Total Capacity Installed (MW)	3600	3200	2800
Number of units	6	6	6
Number of units reused (*)	2	2	2
Pmax (MW)	600	533.3	466.7
Pmin (MW)	360	270	245
Hmax (m)	320	320	320
Hmin (m)	185	185	185
Hrated (m)	285	285	285
rpm	166.7	166.7	166.7

Final Dam Elevation 1255 m.a.s.l.

Total Capacity Installed (MW)	3200	2800	2400
Number of units	6	6	6
Number of units reused (*)	2	2	2
Pmax (MW)	533.3	466.7	400
Pmin (MW)	260	225	200
Hmax (m)	285	285	285
Hmin (m)	131	131	131
Hrated (m)	210	210	210
rpm	125	125	125

Final Dam Elevation 1220 m.a.s.l.

Total Capacity Installed (MW)	2800	2400	2000
Number of units	6	6	6
Number of units reused (*)	2	2	2
Pmax (MW)	466.7	400	333
Pmin (MW)	190	170	145
Hmax (m)	250	250	250
Hmin (m)	107	107	107
Hrated (m)	190	190	190
rpm	125	125	150

(*) Adopting final runners since the commissioning

LIST OF APPENDICES