TECHNO-ECONOMIC ASSESSMENT STUDY FOR ROGUN HYDROELECTRIC CONSTRUCTION PROJECT

PHASE II: PROJECT DEFINITION OPTIONS

Volume 3: Engineering and design

Chapter 2: Selection of project site, powerhouse location, dam type and alternatives to be studied

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1 PREAMBLE

1.1 Brief history of Rogun Project

Many studies have been carried out over the years dealing with the Rogun Hydro Power Plant (HPP) Project. These studies considered various reservoir elevations and various types of dam and appurtenant works. The experience gained in these previous studies is very valuable, and helps in the selection of the best options.

The studies on Rogun HPP project started at the time of the Soviet Union. They were part of the vast project for development of the hydroelectric potential of the Vakhsh River which was initiated in 1963. These initial studies on Rogun were carried out in parallel with the construction of Nurek dam, completed in 1978, revised in 1981, at the time when Nurek has been put into operation. In the initial project, the reservoir full supply level (FSL) was 1290 meters above sea level (m a.s.l), and the dam height was 335 m.

The construction of the Rogun project was initiated upon the completion of Nurek. It was suspended in 1990, due to some questioning on the dam height. Indeed, several organizations within the USSR had requested some additional studies, where lower reservoir and dam crest elevations had to be considered, and taking into account some concerns about the seismicity of the site, about the reliability of the project, and about some environmental issues. These additional studies were carried out by Hydroproject Tashkent in 1992-1993. They explored a range of the reservoir elevations: 1100, 1185, 1240 and 1260 masl.

After the independence of Tajikistan in 1991, the Government of Tajikistan expressed the wish to resume construction of the dam and HPP. In year 2000, Barki Tojik commissioned Hydroproject Institute of Moscow (HPI) to determine the optimum way. It was found that the optimal FSL for Stage 1 would be 1180 masl.

In the years 2004-2006, several studies were carried out for RUSAL, a major Russian aluminium producer, which made plans for the project of a new aluminum smelter in Tajikistan. These studies were carried out by Hydrospetsproject (HSP) in Moscow and subsequently by Lahmeyer (Germany).

In 2009, Rogun HPP company appointed the Hydroproject Institute of Moscow (HPI) for studying the completion of Rogun HPP. In these studies, the design is very similar to the 1980 design. The main changes are brought in the spillage capacity and arrangement.

In 2011, Barki Tojik appointed the Consortium Coyne et Bellier/Electroconsult/IPA to perform a Technico-Economic Assessment Study (TEAS) of the Project as currently laid down in existing studies. This report is an overall review of the existing information leading to the definition of the alternatives proposed to be studied under the present assessment.

1.2 Key features of HPI 2009-2010 project

The project studied by HPI Moscow in 2009-2010 is the latest version available of Rogun HPP Project. This version is the reference for the assessment of the project and the definition of the alternatives.

1.2.1 Layout

When completed, the Rogun project would consist of a zoned fill embankment 335 m high over foundation level, supplying water to an underground powerhouse, located on the Left Bank, of 3,600 MW of installed capacity.

A general plan view of the dam is shown hereunder on Figure 1.2.1.

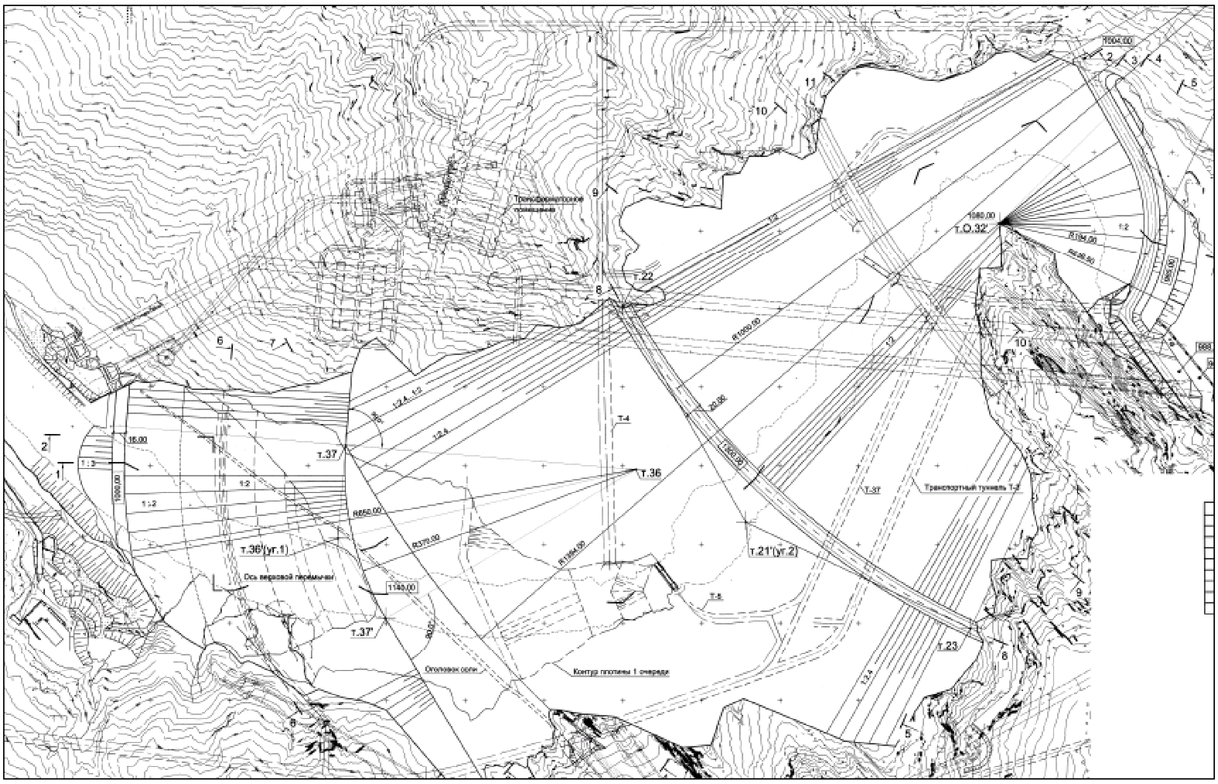

Figure 1.2.1 - General layout

The initial reservoir capacity at Full Supply Level (FSL – 1290 masl) would be around 13,300 Mm^3, corresponding to about 67% of the mean annual flow of the Vaksh River.

1.2.2 Final Dam and reservoir

According to HPI design, Rogun dam at final stage would consist of a zoned fill embankment, 335 m high over foundation. The total volume of the fill amounts at 71.4 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.

Figure 1.2.2 hereunder shows a typical section of the dam.

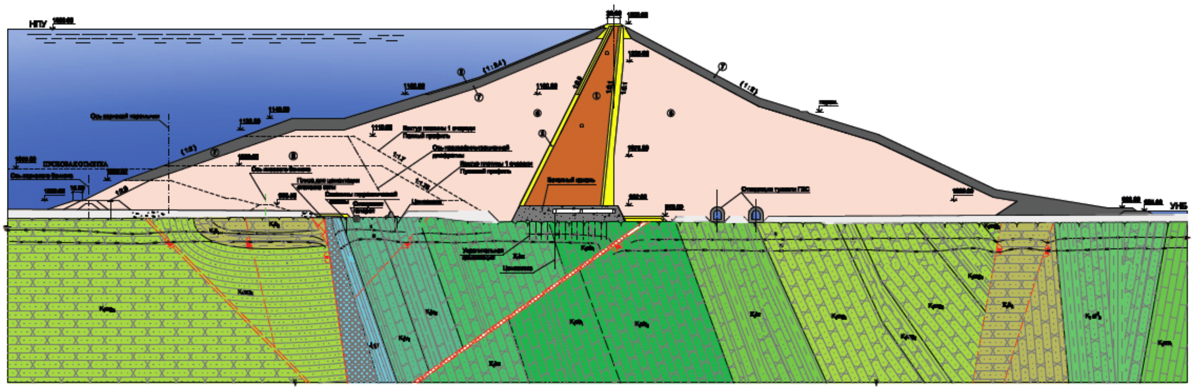

Figure 1.2.2 – Typical cross section of the dam

The inclined central impervious core is made of alluvial material of wide particle size distribution with a fine element fraction (<80µm) between 15% and 30% and a Dmax = 200 mm.

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.

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.

Fine and coarse filter zones border both faces of the core.

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

Average slopes of upstream and downstream face are 2.9H/1V and 2.6H/1V respectively.

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).

1.2.3 Final Power house

The underground powerhouse is set within the Left Bank: it comprises six identical generating sets of 600 MW, housed in a cavern 69 m high, 21 m wide and 220 m long. A section through the powerhouse is shown hereunder on Figure 1.2.3.

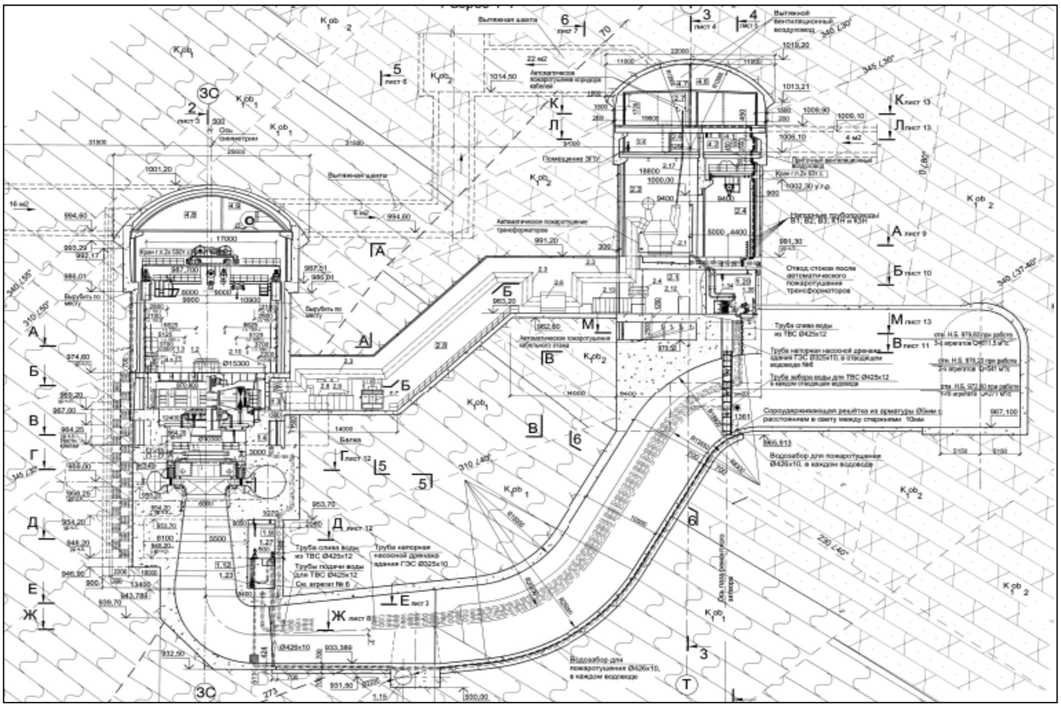

Figure 1.2.3 – Powerhouse – Typical cross section

Six independent tunnels, 7.5 m inside diameter, carry the flow from the reservoir to the powerhouse location; the intake works, set up at El. 1152-1172 on the Left Bank, provide for the inlet of the flow from the reservoir as down as the Minimum Operating Level (MOL - El. 1185), which corresponds to a dead volume of 5,000 Mm^3.

Flood control works comprise tunnels and shafts excavated in the Right Bank hillside: the whole is dimensioned for passing the Possible Maximum Flood (PMF) of 7,100 m^3/s under the Maximum Flood Level (MFL) set at El. 1292.

Construction of a specific structure of protection against mudflow started in the downstream zone of the Left Bank tributary Obishur, close to the powerhouse waters outflow. The structure is a 85 m high concrete gravity dam.

The overall programme of construction works spans over 15 years from the date of diverting the river through the two Left Bank diversion tunnels.

1.2.4 Spillways at Completion

1.2.4.1 Original 2009 design

In the 2009 design of Hydroproject Institute (HPI), the facilities for discharging floods during the construction of the stage 1 dam up to el. 1,060 m a.s.l. are constituted by the two Diversion/Tailrace tunnels with capacity of evacuating 3,290 m^3/s at el. 1,033 m a.s.l. (called also construction & operation tunnels of 1st and 2nd level).

The first discharge tunnel on the right bank (called 3rd level diversion tunnel) was originally foreseen at elevation 1,060.0 m a.s.l., which is the same elevation of the dam crest above mentioned, being therefore usable only for the subsequent stage of the project, i.e. during the operation of the plant with preliminary arrangement of the two units n° 5 and 6 and dam at el. 1,110 m a.s.l.

Its total length was about 1,700.0 m, internal circular section of 15 m diameter in the upstream stretch some 1,000 m long, which works under pressure flow conditions. The tunnel then divides into two branches with the same above mentioned section, and after further 100 m the sector gates are placed. Downstream from the gates, the tunnels section is D-shaped, 12.0 m high and 10.0 m wide, and steel lining is provided on sidewalls and floor. Then two tunnels discharge into the river, being the distance between the outlets some 200 m. Flip buckets are foreseen at the outlets of both tunnels and protection works are indicated along the bank slope down to the river.

The tunnel was equipped with one set of maintenance wheel gates at some 600.0 m from its intake; one further set of emergency wheel gates and one set of sector gates, all operated by servomotors, was provided on each tunnel after the branch, reflecting a quite conservative approach. All gates are operated from a gate chamber located directly above them.

In a second stage, another tunnel (diversion and operation 3rd level tunnel) was foreseen in the right bank at higher elevation, with intake at 1,165.0 m a.s.l. and same geometrical features of that above described; at a distance of 500 m from its intake, this tunnel was connected to the previous one with a 110 m high vertical shaft. With the construction of this tunnel, the lower one was to be plugged just upstream of the connection, probably because its use would be prevented both by the high water speed under head progressively increasing above 100 m and subsequently by the sediment deposit.

This tunnel was provided with a maintenance wheel gate at some 195.0 m from its intake, following the same arrangement of other tunnels.

Further to this outlets system, an additional spillways system was foreseen, constituted by:

- A spillway tunnel with intake at el. 1,145.0 m a.s.l., some 600.0 m long, connected to a vertical shaft, which develops from el 1,283.5 m a.s.l. down to 992.6 m a.s.l., with 16.0 m diameter in the portion below the tunnel connection;

- A vertical shaft spillway, with an entrance equipped with three radial gates 14.0 m wide and 5.5 m high, elevation of sill at 1,283.5 m a.s.l., 12.0 m diameter down to the above tunnel. Shortly above the connection with the tunnel, the normal section of the shaft is reduced to a throttled section with a diameter of 9.2 m, which acts as control point for larger discharges ("throat control"), i.e. energy dissipation would take place in the shaft above the throttle.

At the bottom elevation of the shaft, there was a discharge tunnel 585 m long with geometrical features apparently similar to the circular cross section of the third level diversion tunnel above described.

The 2009 design does not include "bottom outlets". The two low-level tunnels for river diversion during construction will be plugged after their use and will be converted into free flow tailrace tunnels.

1.2.4.1.1 Changes in 2010 design

The above scheme was modified in 2010, when Hydroproject Institute issued a study updating the project features, mainly in respect to the discharge facilities.

In this document the previous 3rd level diversion tunnel was changed to a tunnel totally independent from the other ones, with inlet at elevation 1,035.0 m a.s.l.

This tunnel, which develops on the right bank of the valley, is approximately 1,700.0 m long, with 15.00 m diameter circular section in the upstream stretch operating under pressure; the following current free-flow sections are D shaped, 17.0/16.5 m high and 14.0/13.0 m wide.

Its intake is located very close to the right abutment of the first stage dam and runs towards south-south west for some 700 m, turning then to south-south east for further 1,000 m.

At its outlet, an open air chute some 140 m long with terminal sky jump located a few meters above the river water level is provided.

Also this tunnel is equipped with two sets of wheel gates and one set of sector gates, all operated by servomotors, which reflects the usual approach.

The diversion and operation 3rd level tunnel originally located at el. 1,165 m a.s.l. was lowered to el. 1,145 m a.s.l., maintaining the concept of branching into two tunnels after about 420 m.

What was instead substantially modified is the concept of the discharge tunnels after the sector gates, since a solution with vertical shaft of 13.0 m diameter in which a twirling effect is obtained was adopted for both of them.

The discharge tunnels (13 m diameter circular section) are located at el. 990.0 m a.s.l. approximately, i.e. the shafts bottom elevation, and are provided with terminal flip bucket.

The upper spillways system (tunnel + shaft spillway) already above described was maintained unchanged.

1.2.5 Early generation

From the beginning of Rogun studies in the 70's, a staged construction have been planned in order to generate energy before the dam completion.

A smaller dam embedded in the main one allows raising the reservoir level before the main dam completion. This is the Stage 1 dam which has a crest elevation of 1110 m a.s.l.

A temporary power intake and two temporary units have also been designed to make this early generation possible.

The main steps of the early generation are as followed according to HPI 2009 design:

- October of year 1: river diversion
- December of year 3: the reservoir is raised up to 1055 masl, the temporary units 5 and 6 are commissioned;
- September of year 5: units 5 and 6 runners are replaced by final ones, the reservoir is at elevation 1100 masl (Stage 1), it will remains at this level during several years;

- September of year 11: units 5 and 6 rotational speed is changed to the final one, units 3 and 4 are also commissioned, reservoir level is 1165 msl.

- September of year 12: commissioning of units 1 and 2, reservoir level is 1185 masl.

1.3 Existing structures

Presently, several structures are already constructed, or partly constructed. The Phase I report details those structures as well as their present condition.

The underground works performed at Rogun project sum up a total length of about 28 km and can be subdivided into different groups according to their role in the project implementation and their specific function.

The most important Permanent Structures are the partly excavated Powerhouse and Transformer Hall Caverns (including bus duct galleries, collectors of draft tubes and the drainage and grouting galleries), the Cables Tunnels, the Diversion Tunnels, the right bank Dam Grouting Galleries, the Dam Curtain-grouting galleries with steel lining in both banks, the Transportation Tunnels T-2, T-3', T-4, T-6, T-8, T-18.

Among the Stage 1 Configuration Structures, it is ought to mention the Stage 1 Power Waterways (including Gates and Erection Chambers), the Upstream Gates Chambers of the Diversion Tunnels and relevant Drainage and Grouting Galleries.

Finally, there are several Temporary Structures which are used during the construction of the project and that will be abandoned at some moment, according to the progress of the construction. They include the Transportation Tunnels T-3, T-37, T-37A, T-37', T-7, T-7A, T-22, Access Adits, Connections and Other minor structures.

The present progress of the most important structures is here below shortly commented.

The powerhouse is located underground in the left bank of the Vakhsh River, just upstream from the dam core projection, with the main longitudinal axis inclined by some 15 degrees clockwise with respect to the west-east direction.

The main dimensions of the powerhouse are approximately 220.0 m in length, 21.0 m in width and the cavern maximum height is almost 69.0 m from the top of the roof to the draft tube level.

The excavation works have reached the elevation of 966.5 m a.s.l. all along the area of units 1 to 4, while in the area of Units 5 and 6 presently the excavation elevation is around 958.20, i.e. approximately the spiral case axis.

Overhead travelling crane concrete beams, crown concrete arch and lateral walls have been already casted. Recent activities carried out at site include the substitution of the original anchoring system (passive anchor bolts and tendons) installed during the excavation phase, the construction of reinforced concrete columns supporting the overhead travelling crane beams and some additional supporting structures (transversal struts).

Beside the main structure, several appurtenant works (basically those required for access and operation of the plant in the Stage 1 configuration) have been also constructed, i.e. access tunnels, bus ducts for units 5 & 6, draft tubes and collector of the same units, cables galleries etc. and most of the corresponding electromechanical equipment of Stage 1 is already procured. Recently also the bus duct and draft tube for unit n. 4 have been also constructed.

The transformers cavern has been almost totally excavated, supports were installed and concrete lining was casted in the crown and at the sidewalls.

As for the tunnels, the most important are certainly the diversion tunnels 1 and 2, which are used to provide protection against floods to the cofferdam and to the Stage 1 configuration dam.

The two tunnels have been totally excavated and almost completely lined, being missed only the river crossing culverts.

The construction of the tunnels started in the 1980s and major collapses occurred during the first period of operation in both of them.

According to the information provided on this topic, the collapses would have been caused by different reasons:

Tunnel N° 1 excavation and stabilizations works had not been completed yet by the time when it was used for discharging river floods, in fact downstream from ch. 760 a steep ramp some 8.5 m high was rising to the upper heading and the invert profile continued with this elevation to the end of the tunnel. According to what was reported, the flow was impacting on the lining, causing its collapse and allowing for the water eroding and destabilizing crown and sidewalls. The collapse extended up to 20 m above the tunnel crown along the centerline, rising up to some 50 m and 30 m far from the left sidewall.

Tunnel N° 2 collapse was reported to be caused by the large erosion that took place mainly in the invert at right side; the subsequent process of erosion and undermining of the right sidewall provoked its collapse, involving also the crown up to the centerline. Also in this case the collapse area showed a considerable extension, reaching some 30 m above the tunnel crowns and extending up to 40 m above and 20 m from the right sidewall.

Extensive remedial works have been carried out, stabilizing the collapses and constructing a new heavily reinforced concrete lining 1.8 m thick, while steel lining has been installed in the reaches more subject to erosion by sediments.

At present there are no evidences of local or global instability problems and only minor finishing defects have been detected.

In the reach of the tunnels where Fault 35 is crossed, a reinforced concrete lining 1.8 m thick divided into rings 3 m long was adopted: this permits relative displacements between these elements in case creeping would occur in correspondence with the shear zone. Site staff reported that the joints between the rings were sealed with elastic filler, bonded to both surfaces of adjoining elements, which was not shown on the drawings provided to the consultant.

Following the assessment conducted during Phase I studies, the Consultant proposed strengthening measures, consisting in additional rock anchors, drainage drillholes and an additional inner reinforced concrete lining. Drainage galleries have been also suggested in some stretches in order to avoid that the water pressure rises above the values to adopt in designing the new lining.

Out of the remaining permanent structures, it is to note that tunnels T-2 and T-8 are excavated and the supports placed during the excavation include a sort of first stage concrete lining, but final lining, that would include provisions for fault 35 crossing at T-2, is still to be casted. As for T-18, that would provide access to the penstocks upper chamber, some works at the initial stretch have been carried out but it is still almost totally to be excavated and lined.

Also the third level diversion tunnel DT3 was partially constructed, being the inlet stretch excavated and some relevant stabilization measures implemented. The most updated information about the progress of DT3 was reported in paragraph 9.5 of Chapter 1 - Implementation Schedule and Construction Methods of Volume 4.

Out of the main Stage 1 Configuration Structures, some remaining works have to be done on the Stage 1 Power Waterways, i.e. the inlet stretch of the power tunnel and intake structures, as well as the terminal stretch of the penstock at the connection with the powerhouse (including a portion of steel lining).

In respect to the temporary structures, the most important still to finalize is tunnel T-22, that will provide access to the crest of stage 1 configuration dam, at 1,060 m a.s.l.

As indicated in the Phase I assessment report, there are several structures, besides the powerhouse, that require that remedial/strengthening measures be implemented for being considered in line with the internationally recognized criteria of safety and serviceability.

For the powerhouse and transformers caverns complex the installation of a new set of dowels and tendons is envisaged, together with the adoption of Multiple Packer Sleeved Pipes (MPSP). Possible alternatives to the proposed set of stabilization measures can be investigated and evaluated in detail at a later design stage. The installation of struts can also be analyzed.

As for the remaining structures, the interventions, which refer to several tunnels, generally consist on a pattern of additional rock bolts, drainage holes and additional reinforced concrete lining; in the case of some galleries the latter is substituted by a reinforced shotcrete layer.

Special measures shall be implemented in the tunnels stretches crossing fault 35, consisting in casting a new thick lining heavily reinforced, provided with transversal joints, which allows for relative movements between the “rings” in case creeping effect occurs at fault section. This kind of measures could be applied also in other situations where the tunnels cross main active faults.

Further, additional grouting is envisaged along the stretches of the two existing Diversion Tunnels and in Stage 1 Headrace Tunnel, where high rock mass permeability was detected.

Finally, repairs of the areas of local damages and incomplete concrete lining with new reinforced concrete or shotcrete are recommended for transportation tunnels T-3, T-37 and T-37’; along their development, a safety steel mesh bolted to the existing concrete is to be placed. Whenever necessary, lattice girders would be adopted to assure the tunnel section stability, embedded in a new concrete lining or shotcrete.

The general layout of the underground structures is mainly linked to the location of the powerhouse and appurtenant structures. Given the fact that, as explained in a following section, the presently constructed powerhouse will be maintained unchanged for all the options under study, also all the numerous existing works can be used for being permanently incorporated into the final project layout, or for fulfilling the role for which they have been designed.

2 SELECTION OF THE DAM SITE, TYPE AND AXIS

2.1 Dam site

The Rogun dam site has been selected by Hydroprojekt Tashkent at Soviet time and has not been re-considered ever since 1981.

The Consultant has not found any documents presenting a study of site alternatives or justifying the present Rogun site even though it has been understood that three sites were originally considered.

Nevertheless, it can be understood why this site have been chosen:

- the particular topography shows a very narrow valley compared to the rest of the river that allows constructing a high dam with rather limited quantity of material.
- Upstream of Rogun site, the Ionaksh fault runs along the river in the same axis. Therefore, the dam core would have been set across an active fault, which is not acceptable.

Nevertheless, as explained in the geological assessment, the particular Rogun site is a “fault knot” that is a very particular feature for a dam site. However, accounting for the important number of existing structures to be included in the overall layout, dam site was found acceptable.

2.2 Possible dam types

Considering the topographical conditions, the narrowness of the site only, several types of dam could be envisaged:

a. Impervious Core Rockfill dam,

b. Concrete arch dam,

c. Gravity RCC dam,

d. Concrete Face Rockfill dam,

Other types of dam could also be considered; they are combinations of the types listed above, as:

e. Concrete Arch gravity dam,

f. Impervious Core Rockfill dam which upstream heel is cut with a RCC block,

g. Impervious Core Rockfill dam which downstream toe is cut with a RCC block

All types of dam mentioned above were envisaged in the past, but not always for FSL 1290.

2.3 Comparison of dam alternatives

2.3.1 General

The advantages and disadvantages of every alternative are compared on the basis of the following considerations on:

- References of dams of this type higher than 200 m,
- Then the sensitivity of the dam alternative to:
  - seismic events,
  - movements along faults,
  - argillite/siltstone characteristics,
  - differential settlements,
  - dissolution of salt in Ionaksh fault,
  - flood underestimate or inefficient spillways,
  - unexpected large events as upstream landslides and “Glofs”.

Table 2.3.1 presents in a synthetic manner the comparison. The following comments have to be added:

On the references of dams higher than 200 m (see Table 2.3.2),

The reason for introducing consideration on the references is to check that there is enough experience in the construction of high dams for each type envisaged.

- There are enough references of constructed arch dams and impervious core rockfill dams. The maximum heights of constructed dams in each category are respectively 292 and 300 m. A 305 m high arch dam is under construction in China.
- There is no sufficient experience in the construction of high RCC gravity dams, as the maximum height reached is 217 m (in China). A 249 m high dam is under construction in Ethiopia.
- For Concrete Face Rockfill dam, the maximum height of a constructed dam of this type is 233 m.

In brief, without any other considerations on hydrology or geology, the technical aspects of design and construction of very high arch dams or rockfill dams with internal “impervious” core are fully mastered. For the two other main types of dam, the maximum height reached is, for the time being, slightly above 200 m; the step up to the height of Rogun is about 100 m, which is a considerable technology step.

Concerning the three composite alternatives, there is no reference of dam of these types higher than 200 m. Clearly, it would not be reasonable to envisage a RCC arch or even arch-gravity dam alternative for such high dam. On the contrary, again without considerations on hydrology and geology, there is no reason to reject combinations of RCC blocks, not more than 150 m high, with a rockfill dam with internal “impervious” core.

2.3.2 Sensitivity of dam body to seismic events

Only the response of a given dam to an earthquake is considered here; the sensitivity to movements along faults is the subject of the following section.

In fact, if well designed and well-constructed, the sensitivity of dam body to seismic events is moderate whatever the dam type is. In fact the occurrence of significant damages in dams submitted to earthquake is rare.

For concrete dams, no case of dam break was recorded, except for Shi-Kang in Taiwan; but in the latter case, it was due to an active fault crossing the dam axis. For the other concrete dams, the damages were little, in general cracks, larger or smaller; in the case of Rapel (arch dam in Chili), damages occurred to the intake tower and the spillway. Such damages can be repaired and do not endanger the dam.

Dam type	References of dams higher than 200 m	Sensitivity of dam body to seismic events	Sensitivity to movements along faults (with part of dam founded on faults)			Risk related to salt fill of Lonakhsh fault	Sensitivity to argilites /siltstones characteristics	Sensitivity to differential settlements	Sensitivity to flood underestimate or inefficient spillways	Sensitivity to unexpected large events as uspstream landslides and "Glof"
			Lonakhsh	N70	N35					
Impervious Core Rockfill dam	9		Upstream shoulder	Shoulders and core	Downstream shoulder					
Concrete Arch dam	23			In the upper part of the banks						
Gravity RCC dam	5			In the banks						
Concrete Face Rockfill dam	10		Upstream shoulder	In the upper part of the banks	Central part and downstream shoulder					
RCC Arch Gravity or Arch dam	Highest dam 140 m (china)			In the banks						
Rockfill dam with earth core dam with upstream RCC block cuting the heel	-		Upstream shoulder and block	Shoulders and core	Downstream shoulder					
Rockfill dam with earth core dam with downstream RCC block cuting the toe	-		Upstream shoulder	Shoulders and core	Downstream shoulder and block					
Comments		The risk is moderate if well designed (adjustment of the slopes, ect)	No reliable mitigations measures - The risk of movement can not be avoiced			Risk can be mitigated by adequate design, monitoring and maintenance	The higher is the sensitivity and the deeper are the excavations		Risk can be removed by adequate studies and design	Risk can be removed by adequate studies and design

Sensitivity level	
No sensitivity	
Moderate	
High	
Very High	

Table 2.3.1 - Sensitivity of dam alternatives to various factors of risks

Type of dam	Name (country)	Height	Complet. Date
Gravity RCC dam	Basha Diamer (Pakistan)	270	UD
	Gibe III (Ethiopia)	249	expect. 2015
	Longtan (China)	217	2009
	Huangdeng (China)	202	expect. 2016
	Guangzhao (China)	201	2009
Concrete Face Rockfill dam	Shuibuya (China)	233	2008
	Houziyan (China)	224	expect. 2017
	Nam Ou (Laos)	224	planned
	Nam Ngum 3 (Laos)	220	UC
	Jiangpinghe (China)	219	UC
	Agvaliysk (Russian Fed.)	215	UD
	Munda (Pakistan)	213	expect.2021
	Bakun (Malaysia)	205	2010
	Campos Novos (Brazil)	200	2007
	Xe Kaman 2 (Laos)	200	planned
Clay Core Rockfill dam	Nurek (Tadjikistan)	300	1980
	Chicoasen (Mexico)	261	1980
	Tehri (India)	261	1997
	Guavio (Colombia)	246	1989
	Mica (Canada)	246	1973
	Chivor (Colombia)	237	1975
	Oroville (USA)	230	1968
	San Roque (Phippines)	210	2001
	Keban (Turkey)	207	1974
Concrete Arch dam	Bakthiari (Iran)	315	UD
	Jingping (China)	305	UC
	Xiaowan (China)	292	2010
	Xiluodu (China)	286	UC
	Inguri (Georgia)	272	1980
	Vajont (Italy)	262	1961
	Mauvoisin (Switzerland)	251	1957
	Laxiwa (China)	250	2009
	Deriner (Turkey)	247	2012
	Ertan(China)	240	2000
	Ermenek (Turkey)	235	2009
	El Cajon (Honduras)	234	1985
	Chirkey (Rus. Fed.)	233	1978
	Goupitan (China)	232,5	2011
	Karun IV	230	2011
	Mrantinje (Montenegro)	220	1976
	Contra (Switzerland)	220	1965
	Glen Canyon (USA)	216	1966
	Daganshan (China)	210	UC
	Berke (Turkey)	210	2002
	Luzzone (Switzerland)	208	1963
	Dez (Iran)	207	1962
	Villarino	202	1970

UD Under design
UC Under construction

Table 2.3.2 - References of dams higher than 200 m

As far as the embankment dams are concerned, the experience shows that failures occurred only for very small dams or for dams founded on liquefiable soils or for hydraulic fill dams. The type of damages recorded are generally cracks (transversal and longitudinal), or settlements.

Recently (in 2008), the CFRD dam of Zipingpu in China was subject to an earthquake of magnitude 8 for the epicentral distance of 17 km. The concrete face has undergone very significant damages which could have possibly endangered the dam if the reservoir had been full.

Moreover, CFRD in narrow valley have been recently subject to difficulties and short term settlements with damage to concrete face.

A reasonable conclusion is that amongst all alternatives envisaged for such a high dam, there is some doubt about the CFRD. In addition, to repair the concrete slab needs the full emptying of the reservoir, which would be extremely problematic (to fill again the reservoir would last around 10 to 15 years).

2.3.3 *Possible mitigation measures*

If, according to the calculations, the behaviour of the dam is not satisfactory, then mitigation measures would consist in adjusting the shape of the dam or in adopting measures specific to the type of dam considered.

<u>Sensitivity to movements along faults</u>

There are three main faults which may concern the various types of dam:

- the most upstream is the Ionakhsh fault, dipping downstream; it has a vertical slip rate of 0.5-1.8 mm/year. The order of magnitude of a movement during a major earthquake is estimated at 1m.
- the most downstream is the fault 35, dipping upstream; it has a vertical displacement; faults 35 and Ionakhsh delimit a wedge moving upward. Comparison between the evolvement of the two faults, lead to an estimate of the possible movement of fault 35 during major earthquake of few centimeters.
- fault 70 is parallel to fault 35 and is located between the two previous faults, in the vicinity of dam axis; apparently, its slip rate is very low. Nevertheless, due to a phenomenon of sudden stress release, slips may occur during earthquake events, which could reach several centimeters.

In addition, there is, in the wedge mentioned above, a set of faults parallel to fault 35, which may move during earthquakes in a manner similar to fault 70. Therefore any concrete structure being the component ensuring water tightness to the dam should be avoided in this area; the risk is very high to have cracks occurring, due to movements of faults. From this point of view, the CFRD, RCC gravity and arch dam solutions appear very risky.

On the contrary, the rockfill dam with impervious core may accept movements of faults, provided the possible displacements are not too large and may be accommodated by adequate measures, as increase of the filters or transitions. It is therefore clear that the core of the dam should not be located on Ionakhsh fault; preferably, a direct placement of the core on fault 35 will be also avoided.

The solution, deriving from the previous one, consisting in cutting the upstream shell of the dam with a RCC block, should be avoided, as it would be impossible to avoid the concrete block to be partly founded on the Ionakhsh fault.

On the contrary, the other solution, consisting in cutting the downstream shell with a RCC block is feasible, provided the block is not founded on fault 35.

Possible mitigation measures

It is impossible to block even locally the movement of active faults, therefore these movement have to be accommodated by the dam body.

2.3.4 Risk related to the dissolution of salt in Ionakhsh fault

The risk related to the dissolution of the salt in the Ionaksh fault, as well as the corresponding mitigation measures are detailed in phase 0 report.

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.

The most sensitive dam from this risk perspective is the Concrete Face Rockfill Dam. Sensitivity of the rockfill dam with internal core is moderate if the expected depth of dissolution remains rather small, as shown in the Phase 0 report; indeed the only component of the dam concerned is the upstream shell, which can easily accommodate local displacements.

The sensitivity of concrete dams, which foundation is far from the upper part of the Ionakhsh fault, to salt leaching remains moderate.

Mitigation measures

With adequate design, monitoring and maintenance, this risk could be greatly mitigated. Further details are enclosed in Phase 0 report.

2.3.5 Sensitivity to argillites/siltstones/mudstones

The argillite/siltstone/mudstone (designated as siltstone in this section) is present in several formations which are found in the foundation of the dam.

According to the investigations done, the laboratory tests, the deformation measurements (the walls of the powerhouse excavation), the deformability of the sound argillite/siltstone is relatively high (around 4 to 5 GPa); in addition, as soon as this material is submitted to loading (dam) or unloading (excavation), creeping induces additional deformations, therefore long term modulus of this material is lower than the values given above.

Sound rock corresponds to the category IV in the Russian classification, then:

- cat III is the zone of decompressed rock; measured perpendicularly from the surface, the depth of the base of cat III depends on the formation, but is around 100 m (the value being less below elevation 1100).
- cat II is the zone of decompressed and slightly weathered rock; the depth of the base of cat II is around 70 m

- cat I correspond to the weathered rock; the depth of the base of cat I is around 35 m.

A high concrete gravity dam needs a foundation of high elasticity modulus in the long term; the base of cat II rock seems to be a minimum; the minimum corresponding depth would be around 70 m above 1100 and around 35m below. Due to extremely high level of stresses involved by a 330 m high arch dam, it is not certain that siltstone is an acceptable foundation; this type of dam should be founded only on sound sandstone.

The level of stresses created by rockfill dams (either with internal impervious core or with a concrete face) on the foundation is less; in addition they can adapt to reasonable deformations (less for the CFRD than for the dam with internal impervious core). Therefore they can be founded directly at the surface of cat. I rock (modulus slightly more than 1 GPa).

For solutions integrating blocks of RCC within a rockfill dam, to found these blocks on siltstone could be acceptable, provided:

- the height of RCC block is limited to 150 m or so,
- the watertightness of the RCC block is not sought; it is not a barrier for the water (this condition give the possibility of founding the RCC block in the upper part of cat I siltstone).

Mitigation measures

If the long term settlements resulting from the calculations are too high, the solution would be to deepen the foundation of the dam.

2.3.6 Sensitivity to differential settlements

Most of the formations present in the foundation of the dam are made of subvertical layers of sandstone and siltstone alternating irregularly. The sandstone has lower deformability than the siltstone. Therefore, such high dam may be subject to significant differential settlements.

It is clear that the sensitivity of concrete dams to such differential settlements is much higher than for rockfill dams. The level of sensibility is in fact similar to that described in the previous section.

Mitigation measures

As the stratification is subvertical, there are few mitigation measures; may be to deepen the foundation of the dam may help in reducing the differential settlements to acceptable values.

2.3.7 Sensitivity to flood underestimate or inefficient spillway

A flood underestimate or inefficient spillway could lead to an overtopping of the dam. Obviously, the risk of failure is much higher for an earthfill-rockfill dam than for a concrete dam, and in the case of Rogun, the consequences would be disastrous and amplified by the potential subsequent failure of Nurek.

Mitigation measures

This risk can be reduced by using rather conservative design criteria and an extremely careful assessment of the flood. In particular, the use of the Probable Maximum Flood is compulsory for this project. Also, the adoption of a surface spillway, which is more reliable than a tunnel spillway, contributes to mitigate the risks of dam overtopping.

Note that the concrete dam solution is not free of any risk: in case of overtopping the water falling from the crest could erode some parts of the foundation.

2.4 Other factors considered for the comparison of dam alternatives

2.4.1 Construction in stages

For 2009 Rogun project, mention is made of two stages:

- The first stage corresponds to a dam crest elevation 1110, a reservoir FSL 1100, with two units installed and operated as soon as the water in the reservoir reaches elevation 1060.
- The second and final stage is for the dam crest at elevation 1300.

In fact, the term “stage” has not the usual meaning for dams, i.e. a stage of construction and operation which remains for a long time.”Stage” 1 corresponds here to the installation and operation of powerplant’s units 5 and 6; in the case of Rogun, construction of the dam is assumed continuous up to the final crest elevation 1300.

Table 2.4.1 presents for each alternative:

- First, the possibility of construction in stages,
- Then the possibility of regular reservoir rising within each stage.

Dam alternative	Possibility of construction in stages	Possibility of regular reservoir rising for each stage
Impervious core rockfill	Easy	Yes
Concrete or RCC Arch	Difficult or impossible	Difficult or impossible
Gravity RCC	Yes	Yes
Concrete Face Rockfill	Yes	No
RCC Arch Gravity	Yes	Difficult or impossible
Rockfill dam with earth core dam with upstream concrete block cutting the heel	Yes	Yes
Rockfill dam with earth core dam with downstream concrete block cutting the toe	Yes	Yes

Table 2.4.1 - Possibility of stages in the construction of the dam

There is no problem for the rockfill dam with internal impervious core or the two alternatives which derives from it.

The concrete face rockfill dam can be constructed and operated in stages. But, for each stage, the watertight upstream concrete face can only be placed when the dam reaches the crest of the stage. Therefore, the water in the reservoir cannot follow the rising of the dam within each stage.

As far as concrete or RCC dam are concerned, it is possible to construct the dam in stages, with the reservoir following the dam rising. It is uneasy or, for some shapes, impossible to build an arch dam in stages. For arch gravity dams, construction in stages is possible, but to have the reservoir following the dam rising for each stage may be difficult.

2.4.2 Construction schedule

According to the study carried by HPI, the total duration of Rogun dam construction for FSL 1290 is around 14 years. The total volume of the dam being 70 Mm3, the average placement rate, all materials included, is around 420 000 m3/month.

As the number of types of materials is much less for a concrete face rockfill dam than for a rockfill dam with internal impervious core, it is easier to construct. In addition, as there is no core, it would be less sensitive to the winter conditions. Therefore, we can assume that the average placement rate of the concrete faced rockfill could be higher. However, the actual saving of time would not be that important, as sufficient time should be added for the placement of the concrete slab (in three stages of 110 m). Finally, the total duration of construction of a project with a CFRD solution would be similar to the one for expected for the rockfill with impervious core.

As discussed above, at Rogun, a RCC dam would need foundations deeper than a rockfill dam. An acceptable foundation could be found at the base of rock of category II; the corresponding dam volume is estimated at 16.5 million m3. The duration of the construction of this RCC dam would be 14 years, based on usual placement rates. In order to obtain the total duration of the works, we should add the time for the deep excavations: from 70 m above elevation 1100 down to 35 m below; the volume would be more than 5 million m3. In brief, it would be difficult to implement a project with a RCC dam for duration less than 15 years.

If we assume an arch dam founded also at the base of a rock of category II, which is rather optimistic, the volume is estimated at 10 million m3 by using the Lombardi's slenderness coefficient. The highest completed arch dam in the World is Xiaowan in China. This dam is 292 m high, and its volume is around 7,5 millions m3; the duration of the dam works were 10 years. In the case of Rogun site conditions, the duration of the works would be about 13 years. Taking into account the excavations works required, the total duration of the works could not be less than 14 years.

The above analysis shows that there is no significant difference between the duration of the construction works for the different alternatives. Nevertheless an obvious advantage of the embankment dams is that several construction tunnels have been already excavated and a significant part of the preparatory works are completed.

As far as the other alternatives are concerned, being either not usual (arch RCC dam) or more complicated (RCC blocks cutting the toes of the rockfill dam), they should result in a construction schedule longer than for the other alternatives.

2.4.3 Other components of Rogun project

Several components of the project do not depend on the type of dam, as:

- The diversion system,
- The powerhouse,
- The transformer cavern,

Indeed, a significant quantum of work has been already implemented, and these components should be used to the maximum extent possible. t.

On the other hand part of the spillways and the low level outlets could be located in the concrete dams. The repartition between the spillways in the concrete dam and in the banks would depend on the spilling discharge which could be accepted at a short distance from dam toe in the narrow valley, without provoking an erosion of the foundation in the vicinity of the dam.

The spillway arrangement for the CFRD would be exactly the same as for the rockfill dam with internal core.

As discussed in the report on sediment management, in view of the large amount of material that would deposit in the reservoir, we deem necessary to implement mitigation measures in order to extend as much as possible the lifetime of the powerplant. After an initial attempt to implement a flushing tunnel, due to difficulty in its operation and some drawbacks related with the difficult conditions at its discharge into Obishur creek, it was decided to propose multi-level intakes, which allow to continue operating the plant even when the silt deposit will be higher than the power intake proper. The possibility to pass turbidity currents through the turbines was also envisaged; however a decision in this respect can be taken only after further studies will have been performed, in particular in respect to the possible negative impact of the suspended material transported by the flow on the hydro-mechanical equipment.

This solution makes use of the power waterways system associated to the various dam solutions and do not require specific separate analysis.

2.5 Selection of a dam type

The synthesis of the elements of comparison of the various types of dam considered, detailed above, is given in

TABLE 2.3.1.

For each item of comparison and each type of dam, a degree of control by adequate action results from the analysis:

- Problematic – and no appropriate measure can be envisaged – For instance, it is not sound to envisage installing a concrete dam on an active fault, which movement cannot be mastered.
- Problematic but can be solved by appropriate measures – For instance, with a conservative freeboard and an adequate monitoring in the catchment area (for the risks of GLOF or reservoir bank instabilities), the risk of overtopping can be mitigated.
- Element to be studied carefully – For instance to found a very high RCC dam on siltstone need careful investigations in order to make sure that the foundation can afford to resist to the high stresses imposed by the structure, but, in principle, it is not problematic.
- Not problematic.

The following comments result from the analysis of this table.

- The concrete face rockfill dam suffers from several handicaps:
  - There is no reference of a dam of this size,
  - The concrete face is very sensitive to movement of active faults,
  - The risks related to salt fill in the Ionakhsh fault are higher than for the rockfill dam with internal impervious core,
  - This dam is also very sensitive to overtopping,
  - To have the reservoir following the rise of the dam is not possible.
- The rockfill dam with internal impervious core which upstream shell is cut with a RCC block is also not feasible, because it would be impossible to avoid the RCC block to be founded on the Ionakhsh fault.
- Then the arch dams (conventional concrete and RCC) cumulate two main handicaps:
  - They are incompatible with active faults of the family of fault 35 (as fault 70)

- They don't allow a continuous filling of the reservoir during construction.

- The RCC gravity dam also is not compatible with active faults.

- Unlike the previous types of dam, the impervious core rockfill dam is not exposed to risks which cannot be controlled. In particular it remains safe when settled on active faults provided the movements remain reasonable and provided adequate design criteria are used.

  This conclusion is in line with the statements of ICOLD bulletin 112 "Neotectonics and dams" (1998):

  "There is considerable confidence that a suitably designed embankment dam can accept without failure large movements along faults intersecting the dam body" (section 7.2 of the bulletin).

  "In some cases, it is impossible or quite unfavorable to discard a dam site with potential for contemporaneous fault activity in the foundations. In such cases a conservative embankment dam design is the answer, applying large transition zones of non-cohesive materials" (section 8.4 of the bulletin).

  The risks for this type of dam which need to be mitigated by appropriate measures are:

  - The overtopping – The measures were already described above,

  - The effect of dissolution of the salt of the Ionakhsh fault – The mitigation measures are addressed in the Phase 0 report.

- The comments for the rockfill dam with internal impervious core which downstream shell is cut with a RCC block are similar to the comments made for the previous one. Nevertheless careful studies of the foundation of the RCC blocks should be carried out similarly to those described for the RCC dam solution.

To conclude, the solution of the rockfill dam with internal impervious core, which is proposed and designed for Rogun project, is the most adequate type of dam. The design criteria for this dam should be adapted to particular conditions of this site, which are:

- Active faults,

- Presence of salt in the Ionakhsh fault,

- High seismicity,

- Risks of GLOF and of reservoir bank instabilities.

Dam type	References of dams higher than 200 m	Sensitivity to movements along faults	Risk related to salt fill of Ionakhsh fault	Sensitivity to rock quality	Sentivity to overtopping	Regular reservoir rising during construction
Clay core Rockfill dam						
Concrete Arch dam						
Gravity RCC dam						
Concrete Face Rockfill dam						
RCC Arch Gravity of Arch dam						
Rockfill dam with earth core dam with upstream concrete block cutting the heel						
Rockfill dam with earth core dam with downstream concrete block cutting the toe						

	Problematic
	Problematic but van be solved by appropriate measures
	Element to be studied carefuly
	Not problematic

Table 2.5.1- Comparison of dam types - Synthesis

2.6 Location of Dam axis

Due to the topographic conditions, there is little possibility to move significantly the axis of a rockfill dam with internal impervious core.

Indeed, the dam is located in a narrow gorge, not long enough to contain the entire dam; as a result, the shells fall turning in the upstream and downstream meanders.

Upstream, the location of the toe of the dam is restrained by the intakes of the diversion tunnels installed in the left bank.

Downstream:

- on one hand, the toe of the dam must not block the confluence with Obishur, left bank tributary,
- on the other hand, enough space should remain for the restitutions of the various spillways.

Therefore only very limited adjustment to the location of dam axis could be envisaged in the next stages of the project.

3 SELECTION OF POWERHOUSE SITE AND TYPE

As above indicated, the powerhouse cavern has been already partly constructed, as well as several facilities strictly linked with it.

However, during the inspections carried out by the consultant, certain problems were noticed, that required interpretation and analysis in view of assessing the suitability of the structure to be incorporated into the project. Details can be found in Phase I Report and relevant Annexes.

In fact, significant walls deformations have been recorded within Units 5 & 6, amounting to almost 600 mm in the siltstone portion up to August 2008 and further 140 mm up to August 2012, the latter being due to the deepening of the excavations in units 5 & 6 area.

Cracks and damages are affecting the lining concrete structures at various locations on the cavern sidewalls and neighboring structures as a consequence of this convergence process, as well as other less significant problems.

Models for analyzing the complex of powerhouse and transformers caverns stability were implemented by the designer HPI and by the Consultant, and the outcomes are commented in Phase I report.

The conclusions drawn from the modeling work performed by the Consultant confirm that the present status of the cavern excavation is critical and that under the design provisions previously proposed its stability cannot be achieved. Therefore, a different set of stabilization measures has been proposed by the Consultant, which includes active anchors and dowels, but also the adoption of Multiple Packer Sleeved Pipes (MPSP) that will reinforce the de-stressed rock mass of the “pillar area” between the two caverns, and can also be used for performing consolidation grouting. This set of stabilization measures, which are described in detail in Phase I report, is intended to be extended along the longitudinal axis of the cavern starting from the western end of the same (Unit 6 side) for a length of about 115 m. 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. Possible alternatives to the Multiple Packer Sleeved Pipe system can be investigated. For instance, an intensive consolidation grouting campaign can be carried out and tendons with two heads crossing the whole pillar in between the two caverns installed. This latter solution and any other possible envisaged need to be evaluated in detail at a later design stage.

Based on the above conclusions, and given the present progress of the construction, the Consultant deems that there is no reason for looking for an alternative to the powerhouse structure, which is already excavated and on which the Client made a substantial investment.

It is to be noted that a possible alternative, which could have been considered in case the results of the stability analyses recently carried out had been negative, would have had a very heavy impact on the overall project arrangement, implying not only the construction of a new powerhouse, but also of a large number of other structures linked to the same, such as the transformers hall, the drainage galleries, the stage 1 waterways, the access tunnel, the draft tubes and collectors, the cables galleries, etc.

As for the powerhouse itself, it is to recall 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.

It should be mentioned that in an earlier stage of the studies, when the powerhouse stability had not been assessed yet, alternative solutions had been evaluated.

Among them, the possibility for excavating a new powerhouse cavern in the same area of the existing one, but on the opposite side with respect to the access tunnel. Access could have been provided through a stretch of tunnel branching from the same T-4, and a connection tunnel to the existing cables galleries, passing above the same T-4, could allow routing the cables through the same galleries.

It shall be noted that the new powerhouse would have been designed to house two units only, since only the portion of the existing cavern in siltstone was found not suitable to be incorporated into the project. It is in fact unlikely that the entire existing cavern was not usable, since sandstone has shown much better geomechanical properties than siltstone rock mass and thus at least the portion corresponding to units from 4 to 1 would have resulted stable and usable.

Consideration was also given to an outdoor powerhouse, for the cases of dam with footprints shorter than the rockfill or the CFRD ones.

Whenever a suitable location is found, the outdoor powerhouse in general has some advantages with respect to the underground solution, such as:

- Construction works are generally faster and cheaper than the underground option.
- Accessibility is better and it is easier to deliver materials and to carry out the equipment erection; it is also easier to find solutions in case modifications are required to face problems or needs arisen during the commissioning phase.
- Accessibility is usually better also during the operation period.

On the other hand, in the particular case of Rogun, it is to be considered that the difficult weather conditions during the winter would negatively impact on the construction schedule, whilst this does not occur with the present underground solution: thus one of the advantages of selecting on outdoor location would be lost or strongly reduced.

Also, there is an important risk associated with possible flooding, which may occur if a landslide or a mudflow event similar to that already recorded some 20 years ago happens again.

Finally, the morphology in the gorge stretch is not very favorable to this kind of solution, due to the very steep banks, that would imply large excavation and stabilization works if the building is located on a side of the river.

Downstream from the bend of the river, on the left bank the morphological and geological conditions are also not favorable, both due to the characteristics of the foundation and to the difficulty of finding a suitable route for the power waterways. The several outlets of the hydraulic facilities present on the opposite bank would strongly interfere with the power waterways and with other related facilities of the powerhouse, while its operation may be impaired by the vicinity of water discharges.

Also, consideration could have been given to a powerhouse in the right bank of the river, developing a completely new power waterways system and relevant accesses. Also in this case, this option would have led to discard all the works already done, except the diversion tunnels DT1 and DT2: this decision could have been justified only if the existing powerhouse cavern was to be totally abandoned. It is highlighted that the waterways system could be envisaged to extend by about 2 km, in order to reach the right bank in an area relatively free from other hydraulic facilities, with a consequent quite high cost.

It shall be further considered that the tailraces of the powerhouse would have been located in proximity of the outlets of various hydraulic facilities, being thus highly disturbed during the floods discharge by the turbulence occurring in the river due to the impact of the jets. Also possible drawbacks from the possible scouring effects at the plunge pools location can be expected.

Therefore, in any case, in consideration of the present progress of the works, the existing solution is deemed the most convenient from the implementation schedule and from the economic point of view.

4 SELECTION OF ALTERNATIVES TO STUDY

As stated in the TEAS Terms of Reference, the Consultant study 3 full supply level alternatives and 3 installed capacity per full supply level, which gives 9 alternatives studied.

4.1 Full Supply Level

The maximum FSL is the one considered by HPI: 1290 masl, ie a 335 m high dam. This is the maximum FSL foreseen in the 1978 design. It is not considered recommendable to exceed this height on safety and environmental grounds.

This configuration is going to operate in a full sediment trapping mode, with an expected life of 150 to 200 years because no sediment management strategies are expected to be feasible due to dam height.

Dam modification/ decommissioning in the long terms should be factored in the economic analysis.

The minimum FSL is the minimum level for a storage project with an expected reservoir life of at least 50 years. Any configuration with a FSL level below 1220 is expected to be a run of river scheme like Stage 1. Sediment management strategies, aimed at project sustainability, may be possible at this elevation. The third one is the median in between, ie 1255 masl.

4.2 Installed capacity

As stated in the TOR's the maximum installed capacity to considered in the study is 3600 MW, the one chosen by HPI.

For each FSL alternative, 3 installed capacities have been studied:

- The maximum one: it is the installed capacity having the same plant factor as P=3600 MW for FSL = 1290 masl (HPI design).
- The minimum one: it is the minimum installed capacity that still allows 4hrs peaking of the 95% probability of exceedance of the natural Vakhsh run off. This objective here is to cover the largest range of installed capacity physically possible on Rogun site whatever is the type of operation (peak, base…).
- The intermediate one: it is the median value between the two others.

A preliminary Rogun reservoir operation calculation gives a plant factor of 45% for FSL = 1290 masl and Pinst= 3600 MW (HPI design).

Therefore, adopting the same plant factor for the two other dam heights, maximum installed capacity are 3200 MW and 2800 MW for FSLs 1255 and 1220 masl respectively.

The next graph shows the discharge duration curve of the Vakhsh river run-off at Rogun:

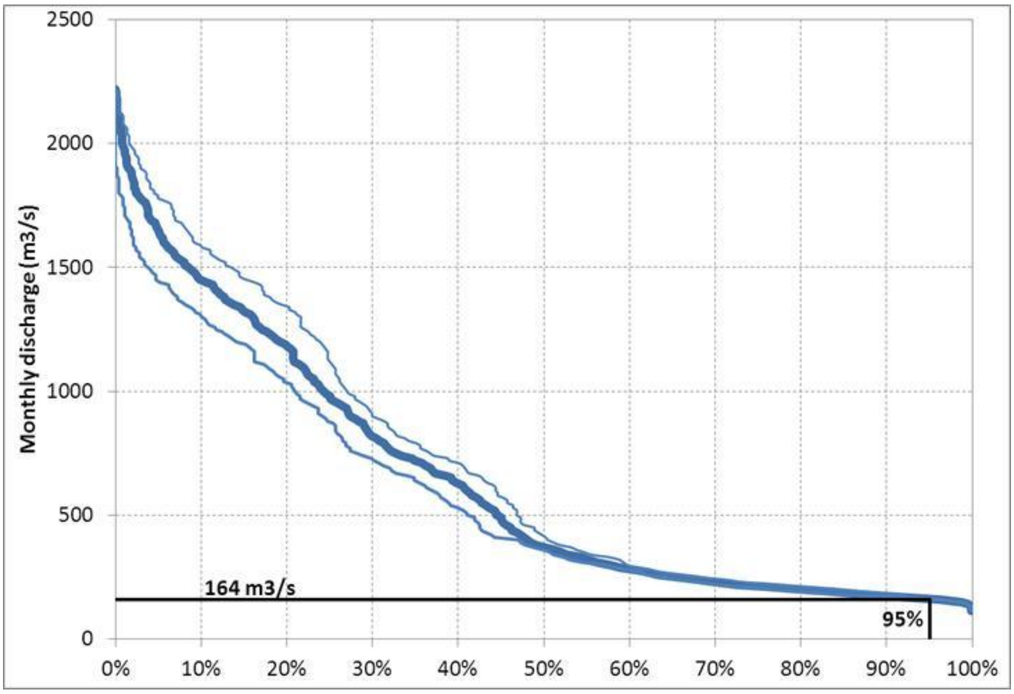

Figure 4.1 : Vakhsh discharge duration curve at Rogun site (1932-2008)

The discharge of 95% probability of exceedance is 164 m^3/s. Assuming that the power plant is not working full time but only 4hrs per day, the volume conservation gives a peak turbined discharge of:

$$Q_{PEAK} = \frac{Q_{95\%}.24hrs}{4hrs} = 984\ m^3/s$$

The needed capacity is then calculated for the three FSL alternatives:

$$P_{PEAK} = 0.9.g.\Delta H.Q_{PEAK}$$

Where ΔH is the maximum head

Finally, the calculated installed capacities are rounded up, and the intermediate one is calculated from the two others. It gives the following installed capacities for each FSL alternatives.

	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

Table 2 : Installed capacities selected

5 CONCLUSIONS

The Rogun HPP project has been studied since many years and in various alternatives of type and height.

The Consultant has considered the same dam site and axis as HPI 2009.

The Consultant, based on its assessment of the HPI 2009 project, has confirmed the dam type: a rockfill dam with internal impervious core. It has found to be the most suitable for the Rogun site mainly because of the important seismic and fault activity.

Three dam full supply levels have been studied by the Consultant : 1290 masl which is the one chosen by HPI 2009, 1220 masl which was found to be in the optimum by previous studies, and a median one 1255 masl.

The powerhouse, as it has been already partially excavated is also kept in its present alignment. The safety of the last part of the powerhouse cavern excavated in the siltstone is assessed in the Phase I report. The modeling work performed by the Consultant shows that with implementing an adequate set of stabilization measures in the Units 5 & 6 area (active anchors, dowels and Multiple Packer Sleeved Pipes, that will reinforce the de-stressed rock mass between the two caverns), the powerhouse can be brought to the required safety conditions and can be used for the scope for which has been constructed. The powerhouse will remain the same for all alternatives under study; in fact, being designed for an installed capacity of 3,600 MW, it is suitable to house the units of all options below indicated.

Three installed capacity per FSL alternatives have been studied by the Consultant. This choice considers as the highest installed capacity studied the one selected by HPI, the lowest one is based on peak energy production consideration and the last one is the median value between the maximum and the minimum.

	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

Consequently, the assessment and selection of best alternative is based on a range of 9 alternatives (3 dam heights, and 3 installed capacities for each one).