TECHNO-ECONOMIC ASSESSMENT STUDY FOR ROGUN HYDROELECTRIC CONSTRUCTION PROJECT

PHASE II: PROJECT DEFINITION OPTIONS

Volume 3: Engineering and Design

Chapter 3: Alternatives design

Appendix 5: PMF Management

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Revision	**Date**	**Subject of revision**	**Drafted**	**Checked**	**Approved**

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1 INTRODUCTION

The present Appendix 5 to the Chapter 3 “Alternatives Design” is an evaluation of the possible options for the management of the Probable Maximum Flood (PMF) for the different dam height alternatives.

As outlined in the design criteria of the project, the primary objective is that the dam must be self-protected against the PMF (no overtopping of Rogun Rockfill dam can be envisaged even in case of extreme floods).

Therefore, the first step of this appendix 5 will be to evaluate the protection provided to the dam in the project elaborated by HPI, and, if this protection is considered not satisfactory, to propose alternative solutions.

Then, given the storage capacity of Rogun reservoir and its flood attenuation capacity, this appendix 5 will evaluate the possibility of releasing from Rogun flows acceptable for Nurek dam.

A special attention has been paid in this report to the spillway arrangement necessary during the operation life of the power plant and on the long term in order to provide a full protection of the dam against the PMF. It appears that, whatever the solution implemented and due to the sediments that will be accumulated in the reservoir, it is mandatory that, on the long term, a surface spillway is made available to ensure the safe evacuation of the PMF without overtopping of Rogun dam.

The scope of the present appendix is to identify suitable solutions to, first, protect Rogun against high flood, and then, protect the cascade, for the three dam height alternatives, with Full Supply Levels at elevations 1290 masl, 1255 masl and 1220 masl,.

2 BASIC CONSTRAINTS

2.1 Design Criteria

As stated in the Chapter 1 of Volume 3“Design Criteria”, the following conditions have to be met for the 10 000 year return flood and the Probable Maximum Flood (PMF):

These floods are to be discharged under the elevation of the top of the dam core. Dam core top elevation is placed 3.75 m below the dam crest elevation for all the three alternatives of full supply level.

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 year 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 the top of the dam core. Note that it is the tunnel with the largest expected discharge which should be considered as being not 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 the top of the dam core.

2.2 Floods considered

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

The analysis of floods in the Vakhsh River at the Rogun H.P.P. (Volume 2, Chapter 5 "Meteorology, Hydrology and Climate Change) gives the following results in terms of daily maximum and instantaneous peak discharge for the two design floods.

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

Table 2-1 : Peak and daily maximum discharge

In the present note, we will round up the discharges considered to the 100 m3/s higher (5 700 m3/s for the 10 000 years return flood and 7 800 m3/s for the PMF).

The hydrographs of these floods are forecasted as shown in Figure 2-1 below. The flood is expected to start in June with a peak in mid-July.

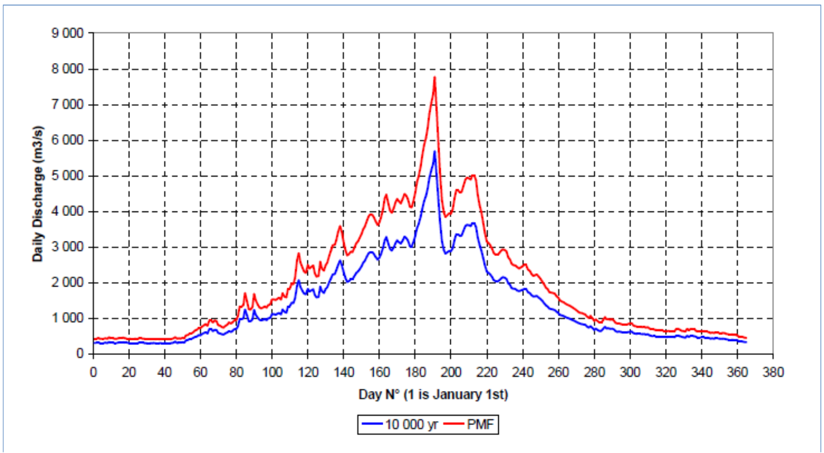


Figure 2-1: PMF and 10000 hydrographs

2.3 Safety principles on flood management

In addition to the design criteria stated above, the following safety principles are taken into account:

- **Turbine operation:** The PMF is an exceptional extreme event during which the normal operation of 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 during the peak period of the PMF. During such an extreme event, only dedicated facilities will be considered for the evacuation of the PMF. This approach is also generally applied on other projects designed by the Consultant.

  The peak period defined for this stage of the studies starts on day 180 and has a duration of 3 weeks.

  For the next stages of the project, when refining the study of the flood evacuation system, it shall be considered that before day 180 the flow already reaches values as high as 4 500 m^3/s corresponding to the peak flow value of the 500 years flood and under which condition the normal operation of the powerhouse is uncertain. Two main risks shall be considered: access availability and electricity transmission system availability. Indeed during extreme floods, snow would be melting around the site and landslides could occur, possibly blocking accesses or damaging transmission lines. It should therefore be analyzed whether these risks can be mitigated (for example using an early flood detection system) or if they have to be compensated for by considering a lower limit for spillage through the turbines (for example until a flow value corresponding to the 30 years flood i.e. 3 500 m^3/s).

- **Flood forecasting:** as Rogun is under a snow and ice melt regime, it is possible to forecast high flood by monitoring the amount of snow accumulated during the previous winter. The analysis performed here considers that it is possible to forecast the flood and take appropriate measures (lower reservoir level for instance) before occurrence of a high flood event.

- **Type of spillways:** 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. For a given increase in head, the incremental discharge capacity is much lower for a tunnel spillway than for a surface spillway. This gives an important constraint to the flood evacuating system. This allows little uncertainty around the design discharge adopted and little possibility of adapting the system to future trends in design floods (climate change…).

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

- **Tunnels and fault crossing:** 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 or, when impossible to avoid such crossing, to adopt a special design to sustain the displacements and maintain the integrity of the structure.

2.4 Nurek design feature

In the original design of HPI, the turbines of Nurek Powerplant are considered in the total flood evacuation capacity of the project. The spillage capacity considered by original designers at Nurek was 4 040 m3/s (spillways) + 1 420 m3/s (turbines). As stated above, the Consultant does not consider the powerplant as a spillage device during the peak period of the PMF. The discharge capacity considered by the consultant at Nurek is 4 040 m3/s during the peak period of the PMF and 5 400 m3/s out of the peak period of the PMF. The spillage capacity is provided by two structures: one bottom spillway set at elevation 857 masl and one surface spillway set at elevation 897 masl. Each spillway has a capacity of 2020 m3/s when the reservoir level is at elevation 910 masl.

3 ANALYSIS OF THE SOLUTION PROPOSED BY HPI

3.1 Description of HPI solution

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

- 3rd operation spillway (OP3): it is a tunnel spillway with intake at elevation 1145 masl, some 400 and 600 m long, it splits into 2 branches. Each branch includes a vertical vortex shaft ended with a vortex device that dissipates a large part of the energy. The final tunnel stretch is horizontal and ended with a flip bucket set up a few meters above the river. (see Figure 3-1)
- Remote spillway (RS): it is a tunnel spillway with intake at elevation 1145 masl, some 400 and 600 m long. The final tunnel stretch is horizontal and ended with a flip bucket set up a few meters above the river. (see Figure 3-2)
- Operation shaft spillway: it is an overflow spillway, with a circular entrance equipped with 3 radial gates 14 m wide, the elevation of sill is at 1283.5 masl, this entrance is connected to a shaft 12m diameter which is connected to the same shaft as the remote spillway. Shortly above the connection with the remote spillway, 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. (see Figure 3-2)

Figure 3-1 : Extract of HPI study - 3rd operation spillway - longitudinal section

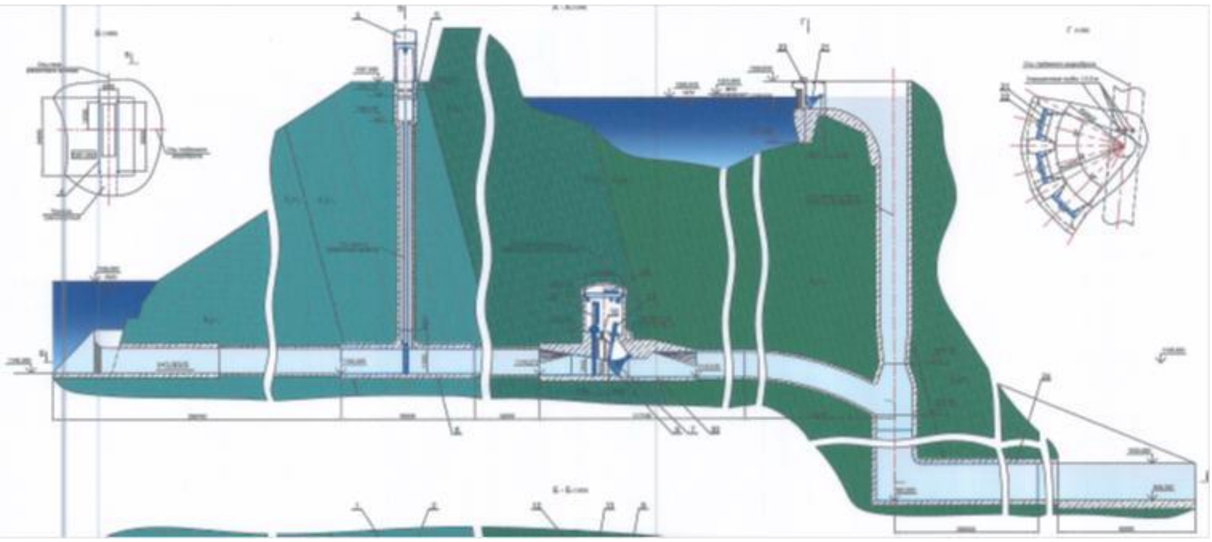


Figure 3-2 : Extract of HPI study - Remote spillway and Operation shaft spillway - longitudinal section

The total spillage capacity is 7 100 m3/s, that is to say the value of PMF as estimated in HPI study.

The next graph presents the discharge capacity of the three operation spillways designed by HPI.

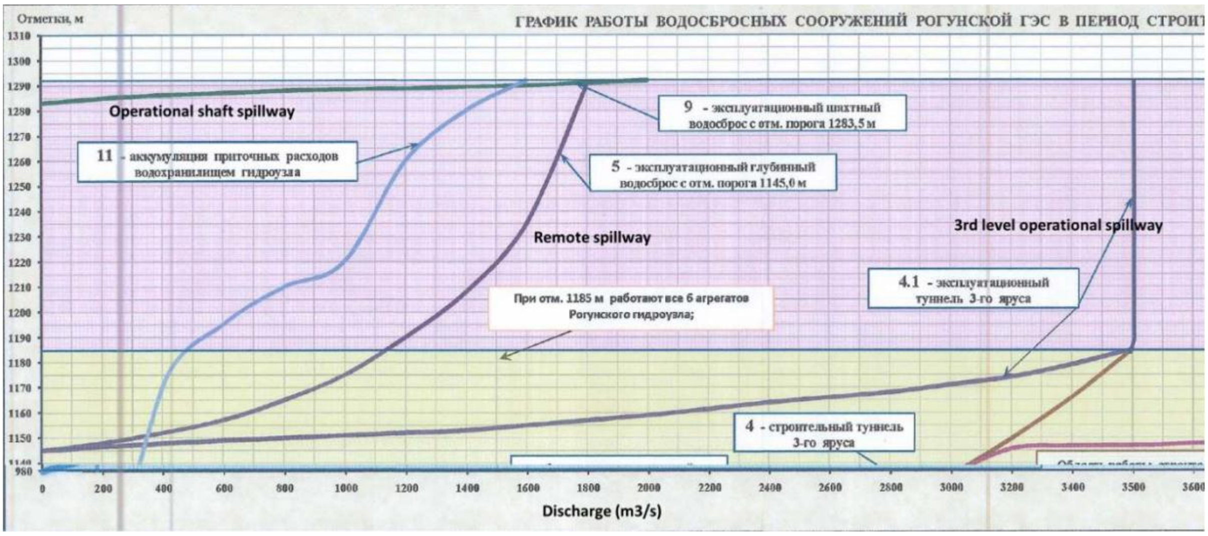


Figure 3-3: Extract of HPI study - Spillways discharge capacity versus reservoir elevation

3.2 Analysis of the solution with respect to the basic constraints

The solution proposed by HPI provides a protection against the PMF and should be able to pass the new value of the PMF, that is to say 7 800 m3/s. However, when the spillway is used during the flood season, the head on the tunnels is 145 m. This value is higher than the maximum head considered acceptable by the TEAS Consultant as presented in the Appendix 3 “Flood Management during Construction” in Chapter 3 “Alternatives Design” that is to say 120 m.

The N-1 criterion for the 10 000 years return flood can be met with three spillways. However, with the current design proposed by HPI, without the 3rd level Operational spillway, the 10 000 years return flood cannot be handled in Rogun: the operational shaft spillway can discharge up to 2000 m3/s and the remote spillway 1800 m3/s.

In the analysis of HPI design performed by the Consultant, only the dedicated spillage facilities are considered (turbines are excluded) for the evacuation of the PMF what is consistent with the basic constraints mentioned above.

3.3 Additional comments on HPI solution

It is to be noted that:

- Two of the three spillways are sharing the same outlet tunnel. A failure of the upstream stretch will impact two of the three tunnels, i.e. 50% of the total discharge capacity.
- On the long term, as indicated in the Chapter 6 “Sedimentation”, the reservoir will be filled with sediments. Therefore, on the long term, the tunnel spillways will not be operational anymore, as pressure tunnels cannot be operated with high sediment load, and because their intake will be plugged at long term.
- Although the solution with tunnel spillways with a vortex device for energy dissipation is a good option for the energy dissipation and has been implemented on some dams across the world, there is no reference for such a large design flood. The only comparable devices have been implemented in Tehri (India) with capacities of 1900 m^3/s and 1950 m3/s as described in the Volume 3, Chapter 3, Appendix 4 “Hydraulics of the project components”. Since commissioning it has operated only once, for a discharge of 480 m^3/s as it is not the main spillway facility. In Tehri, there is also a classical surface spillway with a chute channel, flip bucket and stilling basin, that ensures 40% of the total discharge capacity. Relying on vortex devices only for permanent spillway facilities with such a large capacity is not considered as a safe concept by the TEAS Consultant.

3.4 Conclusion on HPI solution analysis

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.
- Long term flood discharge facilities are not provided in HPI design.

The Consultant considers that safety requirements 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.

4 ROGUN PROTECTION AGAINST HIGH FLOODS

4.1 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

Table 4-1 high level spillways available at the end of construction

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).

It is to be noted that for the alternative FSL = 1290 masl, the Mid Level Outlet 2 (MLO2) should not be affected by sediments for about 50 to 60 years. Therefore, it could be used as an additional outlet in case of PMF.

4.2 Possible types of spillway

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

Tunnel spillways would have the same features as the tunnels available at the end of construction described in 4.1 above.

It is to be noted that the surface spillway is the kind of facility necessary on the long term when the reservoir will be filled with sediments to ensure the long term sustainability of the project. The surface spillway, as presented in the Volume 3, Chapter 3, Appendix 4 "Hydraulics of the Project Components", is made of three independent waterways. This modular design allows building only part of the spillway during the dam construction and completing it when required. One module of surface spillway would have the following features:

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

Table 4-2 Surface spillway main featuresTable 2-1

4.3 Parameters impacting the performance of the flood management

Flood attenuation capacity: the flood attenuation capacity will get reduced with time due to the sediments settled in the reservoir. The volume available for flood attenuation is shown in the following graphs. Each curve represents the volume of the reservoir according to the water level. The volume is calculated at the beginning of construction, and every 20 years. This volume will impact the flow to be evacuated from the reservoir. Therefore, a reduction of volume for flood attenuation in time means an increase of flow to be discharged.

The analysis will be carried out at 40 years after the stage 1, which is consistent with the period considered in the economic analysis. However, it is to be noted that for the alternative FSL = 1220 masl, 40 years after the river diversion, the intake of the tunnel spillways is below the level of sediments. Therefore, for the alternative FSL = 1220 masl, results are given at 30 years after river diversion which corresponds to the end of operation of the tunnel spillways.

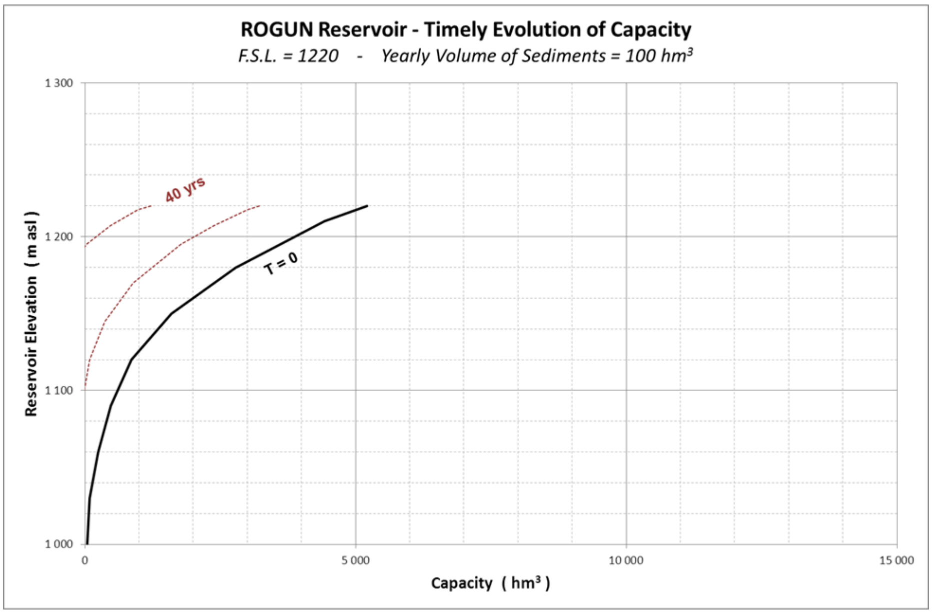


Figure 4-1 H/V curves for alternative FSL 1220 masl and with 100 hm3 of annual sediments load

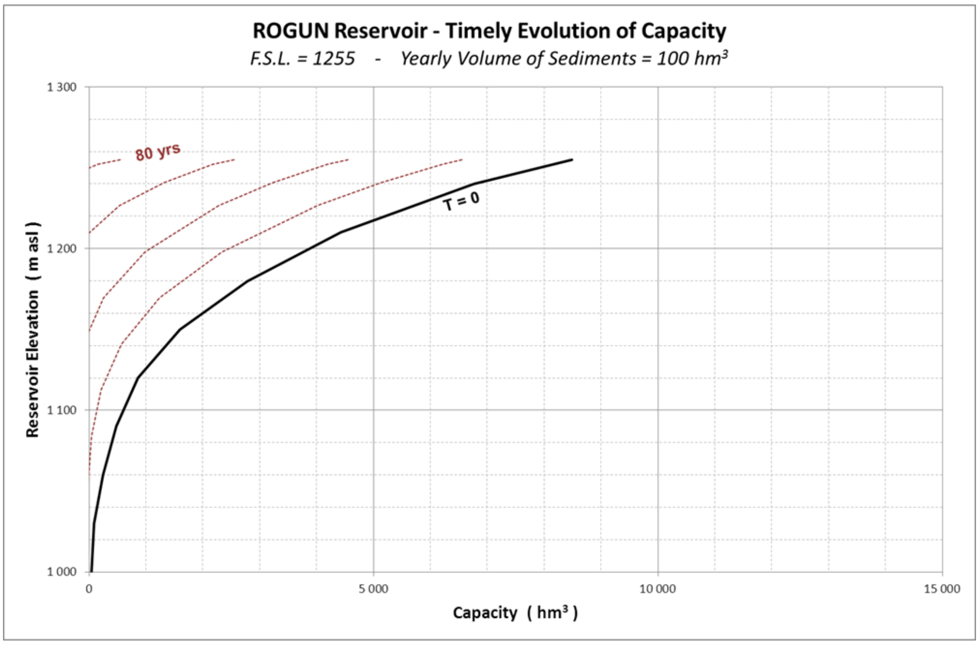


Figure 4-2 H/V curves for alternative FSL 1255 masl and with 100 hm3 of annual sediments load

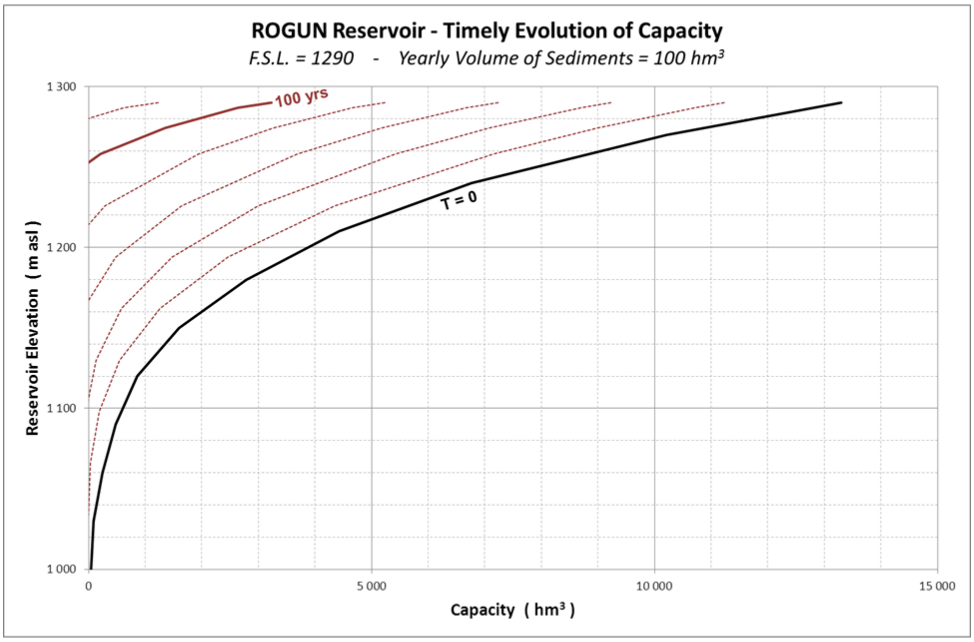


Figure 2-1Figure 4-3 H/V curves for alternative FSL 1290 masl and with 100 hm3 of annual sediments load

Elevation in the reservoir at the beginning of the flood: the following graph shows historical data on the date of occurrence of flood peaks for the Vakhsh River itself as well as for its affluents. It shows that the peak of the annual flood can happen in July as assumed in the hydrograph of flood shown in 2.2, but also that a delay in occurrence of this peak can be observed. The result of the delay is that the flood is detected later and consequently the reservoir elevation at the beginning of the flood is higher than expected. This is equivalent to a reduction of the volume of the reservoir available for flood attenuation and will also result in an increase of flow to be evacuated during the PMF. However, it is to be noted that, in case of important stock of snow accumulated in winter, it is recommended to deviate from the normal operation rule curve of the reservoir in order to keep a sufficient storage capacity for the attenuation of an extremely high flood, reducing the impact of the delay in occurrence of the peak.

The high flood forecast could be based on the mechanism used by ICWC for estimating sharing of water among basin countries.

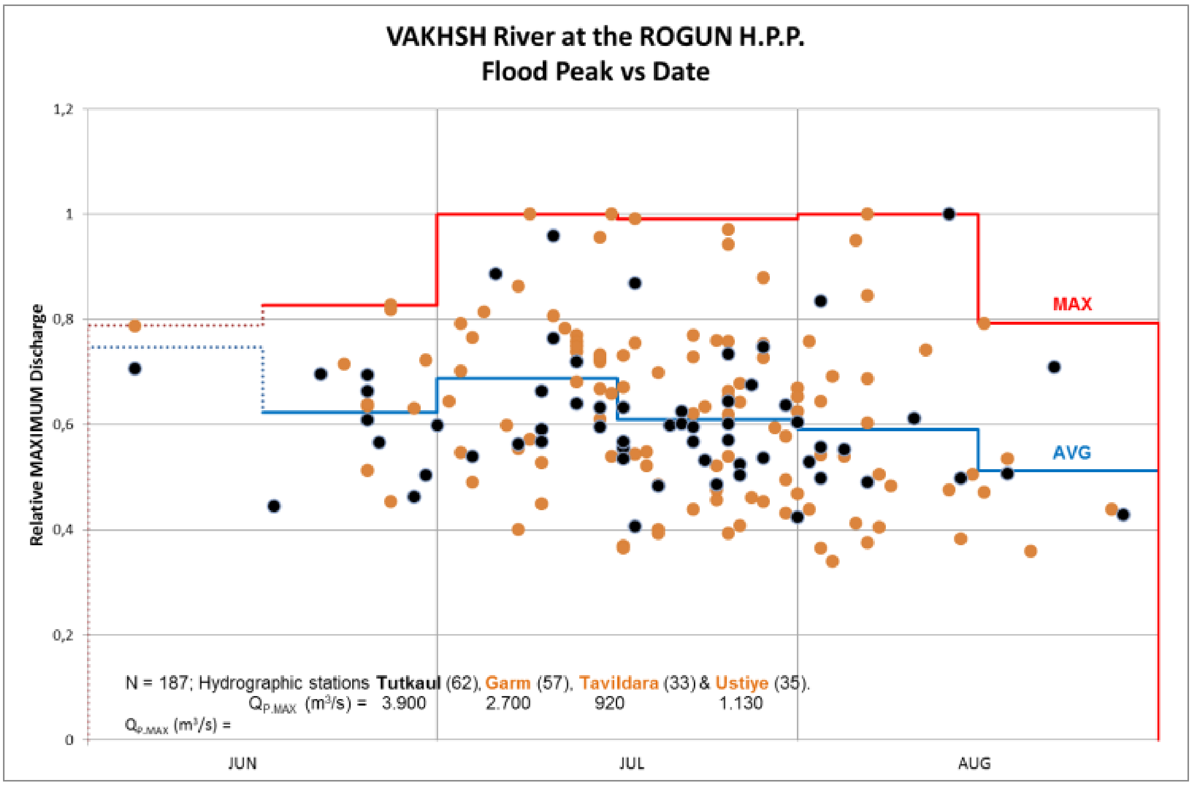


Figure 4-4 Historical distribution of flood peaks of vahksh river and affluent

Number of gates of tunnels available: As mentioned in 2.3, tunnels can be subject to maintenance and operation difficulties. It is therefore important to evaluate the impact of non-availability of gates on the discharge capacity during floods. This potential loss of discharge capacity can be balanced by either a raise of the dam crest or a surface spillway made available. In case of additional surface spillway, the number of surface spillway gates to be opened needs to be evaluated. This is a sensitivity analysis.

4.4 Analysis of Rogun protection 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 that is to say, number of tunnel gates unavailable (balanced by surface spillway gates opening or crest rising), reduction of flood attenuation capacity with time and elevation in the reservoir at the beginning of the flood.

For each combination, it is assumed, as stated in 2.4, that the turbines will operate until the peak of the PMF starts, that is to say the day 180.

The annex 1 gives the results of all flood attenuation calculations performed with the different alternatives, spillways combinations, sedimentation status, elevation in the reservoir at the beginning of the flood etc.

The following conclusions can be drawn from these calculations for each alternative:

Alternative FSL = 1220 masl

One tunnel spillway built for construction is available. Combinations including 1 to 3 tunnels and 0 to 2 modules of surface spillway have been studied.

The calculations presented in Annex 1 show that:

- As the water level in April is lower than the intake of the tunnel spillways, there is no difference between a scenario with water level in April at 1125 masl and a scenario with water level in April at 1135 masl.

- In a solution with 3 tunnel spillways only and a dam crest level at 1230 masl the maximum water level is higher than the core crest level (see next table) in Rogun which is 1226,25 masl.

Time after stage 1		
0 year	20 years	30 years
1225	1230,0	1231,6

Table 4-3 Maximum water level in the reservoir during the PMF , 3 tunnels spillways

This solution does not meet the requirements of the design criteria and does not provide a satisfactory protection to the dam. And it is reminded here that, as stated in 2.3, the consultant does not recommend solutions relying on tunnels only.

- The solutions that provide an acceptable protection of Rogun dam against the PMF and the 10 000 years flood are:

A solution with 3 tunnels and 1 module of surface spillway

Or a solution with 2 tunnels, 1 module of surface spillway and a dam crest rise by 4 m

Or a solution with 1 tunnel, 2 modules of surface spillway and a dam crest rise by 1.5 m

Or a solution with 1 tunnel and 3 modules of surface spillway.

The maximum water level in the reservoir during the PMF is shown in the following table:

Solution to be implemented	Time after stage 1		
	0 year	20 years	30 years
3 tunnels and 1 module of surface spillway	1219.4	1223.1	1223.9
2 tunnels and 1 module of surface spillway + dam crest rise 4 m (core crest is 1230.25 masl)	1228.1	1229.1	1229.2
1 tunnel and 2 modules of surface spillway + dam crest rise 1,5 m (core crest is 1227.75 masl)	1226.3	1226.4	1226.4
1 tunnel and 3 modules of surface spillway (core crest is 1226.25 masl)	1223.73	1223.80	1223.80

Table 4-4 Maximum water level in the reservoir during the PMF

Regarding the protection against the 10 000 years flood, calculations give the following results:

Δt (years) =	0				20				30			
Dam crest	1230	1234	1231.5	1230	1230	1234	1231.5	1230	1230	1234	1231.5	1230
Initial tunnels	3	2	1	1	3	2	1	1	3	2	1	1
Tunnels available	2	1	0	0	2	1	0	0	2	1	0	0
Initial Surface gates	4	4	8	12	4	4	8	12	4	4	8	12
surface gates available	3	3	6	8	3	3	6	8	3	3	6	8
Hmax	1219.6	1228.2	1227.8	1225.66	1220.2	1230.2	1227.9	1225.74	1221.8	1230.4	1227.9	1225.74
Qmax	3492	4339	5346	5407	3596	5022	5423	5467	3962	5102	5427	5471

Table 4-5 Maximum water level and flow at Rogun during the 10 000 years flood turbine operating up to day 180

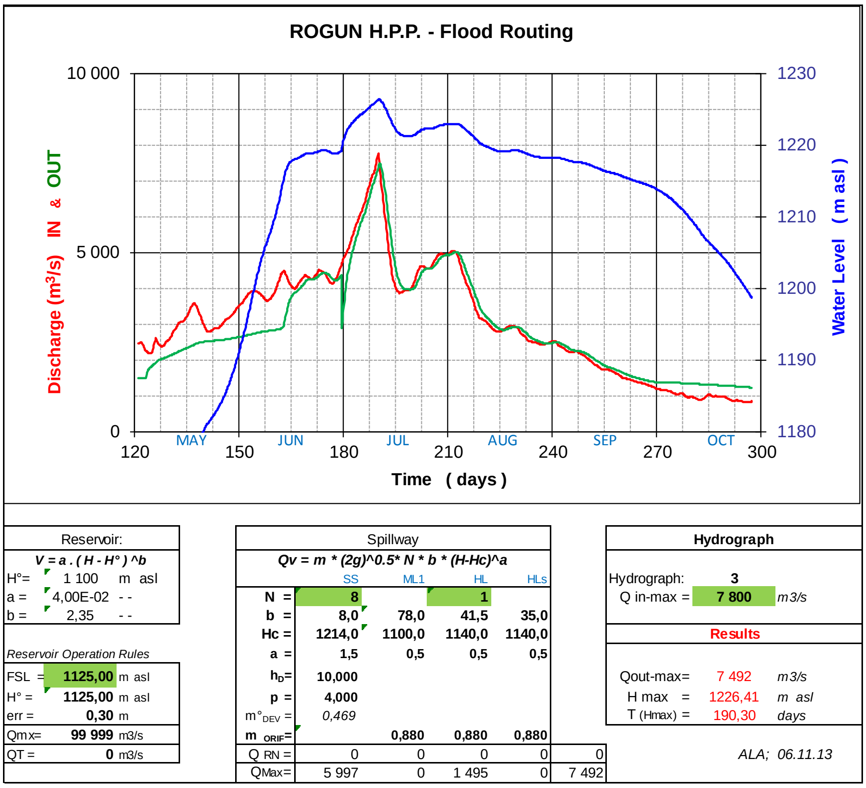


Figure 4-5 Alternative FSL = 1220 masl – PMF attenuation with 2 modules of the surface spillway and 1 tunnel after 20 years of sedimentation and with turbines operating up to day 180

Comparing the cost of the different possible solutions, the one with 2 module of surface spillway, 1 tunnel and a dam crest rise by 1,5 m appears to be the least expensive (see next table).

The 1.5 m crest rise shall also include a rise of the dam core crest elevation by 1.5 m.

Spillway arrangement	3 tunnels + 1 module of surface spillway	2 tunnels + 1 module of surface spillway + dam crest rise (4m)	1 tunnel + 2 modules of surface spillway + dam crest rise (1,5 m)	1 tunnel + 3 modules of surface spillway
Cost of tunnels	303 MUSD	202 MUSD	101 MUSD	101 MUSD
Cost of surface spillway	136 MUSD	136 MUSD	190 MUSD	275 MUSD
Total	439 MUSD	338 MUSD	291 MUSD	376 MUSD

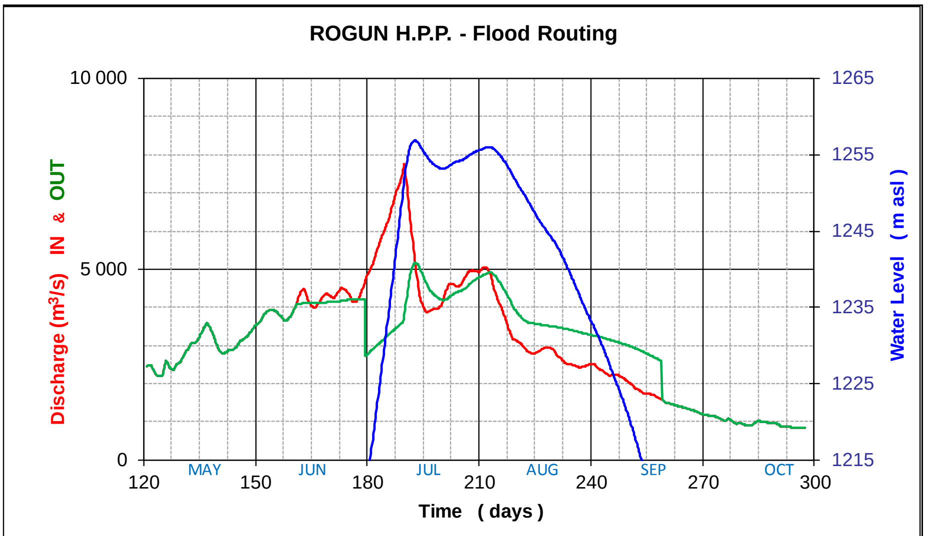
Table 4-6 Cost of spillways according to Volume 4 Chapter 2 “cost estimate”

Alternative FSL = 1255 masl:

In this alternative, the calculations show that with the 3 tunnels available at the end of construction and all gates opened, the PMF can be handled in Rogun reservoir. This solution that relies on tunnel spillways only is however not recommended.

The solution combining the 3 tunnels available at the end of construction and 1 module of surface spillway provides a satisfactory protection of the dam against the PMF before the reservoir is silted.

In this solution, the protection against the 10 000 years flood is also ensured: with the loss of 1 tunnel and 1 surface gate not available, the maximum water level in the reservoir is 1 252.6 masl.



Reservoir:		
V = a . (H - H°) ^b		
H°=	1 145	m asl
a =	2,45E-01	- -
b =	2,08	- -
Reservoir Operation Rules		
FSL =	**1205,00**	m asl
H° =	**1205,00**	m asl
err =	**0,30**	m
Qmx=	**99 999**	m3/s
QT =	**0**	m3/s

Spillway					
*Qv = m * (2g)^0.5* N * b * (H-Hc)^a*					
	SS	ML1	HL	HLs	
N =	**4**	**0**	**1**	**1,33**	
b =	**8,0**	**78,0**	**41,5**	**41,5**	
Hc =	**1249,0**	**1085,0**	**1145,0**	**1165,0**	
a =	**1,5**	**0,5**	**0,5**	**0,5**	
h_D=	**10,000**				
p =	**4,000**				
$m°_{DEV}$ =	*0,469*				
m_{ORIF}=		**0,880**	**0,880**	**0,880**	
Q RN =	0	0	1 242	1 343	2 586
QMax=	1 413	0	1 703	2 050	5 166

Hydrograph		
Hydrograph:	**3**	
Q in-max =	**7 800**	*m3/s*
Results		
Qout-max=	5 166	*m3/s*
H max =	1256,84	*m asl*
T (Hmax) =	192,82	*days*

ALA; 21.10.13

Figure 4-6 Alternative FSL = 1255 masl – PMF attenuation with 1 surface spillways and 3 tunnel, 2 gates of the tunnels closed after 40 years and turbines working up to day 180.

Alternative FSL = 1290 masl:

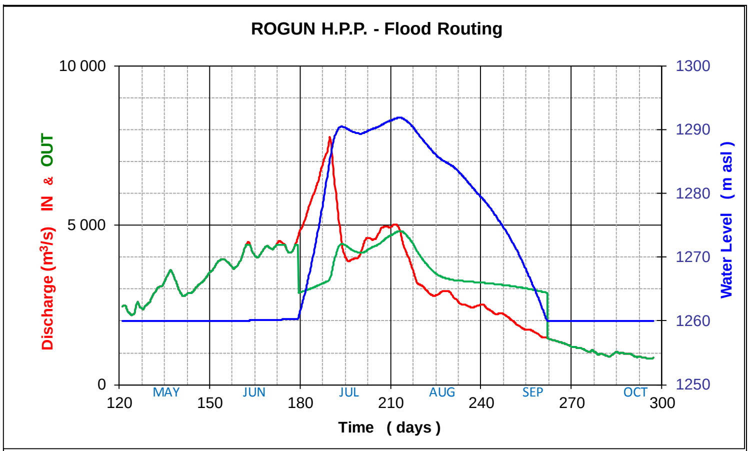
In this alternative, 2 tunnels are available at the end of construction that can be complemented with 1 additional tunnel or 1 module of surface spillway or both.

It is to be noted that, as high floods are exceptional events, the Mid Level outlet 2 built for the construction period can be used as high flood management device during operation: The maximum head on the tunnel is 150 m at FSL and 120 m at the minimum level in the reservoir. In addition, the discharge capacity of this tunnel being higher than the one of the 2 high level tunnels, using this tunnel provides a large flexibility for the management of high floods.

The calculations show that, with 2 high level tunnels with all gates opened and the Mid Level Outlet with 1 gate opened, the PMF can be handled in Rogun reservoir. However, this solution that relies on tunnel spillways only is not recommended.

The solution combining these three tunnels and a module of surface spillway provides a satisfactory protection to Rogun dam before the reservoir is silted.

With this solution, the protection against the 10000 years return flood is also ensured. The maximum water level in the reservoir is 1289.1 masl.



Reservoir:		
V = a . (H - H°) ^b		
H°=	1 100	m asl
a =	5,64E-03	- -
b =	2,72	- -

Reservoir Operation Rules

FSL =	1260,00	m asl
H° =	1260,00	m asl
err =	0,25	m
Qmx=	99 999	m3/s
QT =	0	m3/s

Spillway					
Qv = m * (2g)^0.5* N * b * (H-Hc)^a					
	SS	ML1	ML2	HLs	
N =	4	0	0,33	1,33	
b =	8,0	78,0	78,0	41,5	
Hc =	1284,0	1085,0	1140,0	1190,0	
a =	1,5	0,5	0,5	0,5	
h_D=	10,000				
p =	4,000				
$m°_{DEV}$ =	0,469				
m_{ORIF}=		0,880	0,880	0,880	
Q RN =	0	0	1 094	1 787	2 881
QMax=	1 434	0	1 233	2 161	4 828

Hydrograph		
Hydrograph:	3	
Q in-max =	7 800	m3/s
Results		
Qout-max=	4 828	m3/s
H max =	1291,91	m asl
T (Hmax) =	213,16	days

ALA; 21.10.13

Figure 4-7 Alternative FSL = 1290 masl – PMF attenuation with 1 surface spillway, 2 high Level tunnels with 2 gates closed and 1 gate of the Mid Level Outlet tunnel opened after 40 years and turbines operating up to day 180

4.5 Conclusion on the protection of Rogun against the PMF

For the alternative FSL=1220 masl, four solutions are technically suitable for the protection of Rogun dam. Among them, the solution with 1 tunnel, 2 modules of surface spillway and a dam crest rise by 1,5m is the least expensive solution.

For the alternative FSL=1255 masl, the protection against the PMF is satisfactory with the 3 tunnels necessary for the construction and 1 module of surface spillway.

For the alternative FSL=1290 masl, the protection against the PMF is satisfactory with the 2 high level tunnels and the mid level outlet necessary for the construction and 1 module of surface spillway.

5 VAKHSH PROTECTION AGAINST HIGH FLOODS

As stated in the introduction: Rogun shall be self-protected against high floods during its whole life span and this is the base design studied in the previous paragraphs. However the possibility of protecting Nurek against high floods in specific conditions could be an additional service provided by Rogun. This opportunity is studied in the following section for the three dam alternatives.

5.1 Spillways available at Nurek

2 spillways are available at Nurek:

- 1 tunnel spillway with intake at about 810 masl and a capacity of 2020 m3/s for a water level in the reservoir at 910 masl.

- 1 gated surface spillway with 2 gates, sill at 897.3 masl and a width of 12 m each. This surface spillway is followed by a tunnel 10 m wide and 11 m high. The capacity of this spillway is given as 2020 m3/s for a water level in the reservoir at 910 masl.

As for Rogun, the Consultant recommends not to consider the turbines capacity in the spillage capacity during the peak period of the flood. Therefore, the maximum evacuation capacity without turbines is given as 4040 m3/s. The discharge capacity through the turbines is 158 m3/s x 9 turbines. Therefore, with turbines, the discharge capacity at FSL in Nurek is 5 462 m3/s.

5.2 Cascade protection requirements

Nurek and the downstream cascade are designed to handle a flood lower than the PMF. Rogun project implementation can be the opportunity to protect the cascade against PMF by limiting the discharge downstream of Rogun to a level acceptable for the downstream structures.

As said above Nurek spillway design discharge is 4040 m3/s and 5462 m3/s including the turbines.

The Consultant noted during the course of the study that the water velocity in the Nurek surface spillway is high: 55 m/s. The matching cavitation index is at 0.08 at the end of the first stretch of the tunnel (change of slope). Curves presenting velocities, water depth and cavitation index are shown in annex 3 for 3 different flows: 2 000 m^3/s, 2 400 m^3/s and 2 800 m^3/s.

A cavitation index below 0.1 is considered not acceptable in international standards. The Consultant understands that mitigation measures have been implemented to bring this cavitation index up to acceptable values. However, these are not known by the Consultant, so the adequacy of these measures for a flow increase is not known.

Therefore, the restriction considered in the present chapter is that discharge through the Nurek surface spillway should not overpass its design value 2020 m^3/s. Consequently, the Nurek reservoir level should not be higher than 910 masl.

The Consultant assumes that the hydropower plant located downstream of Nurek are designed to handle at least the same discharge as Nurek. Therefore, the limitation in Nurek discharge is a criteria that ensures the protection of the entire Vakhsh cascade.

5.3 Flood attenuation capacity at Nurek

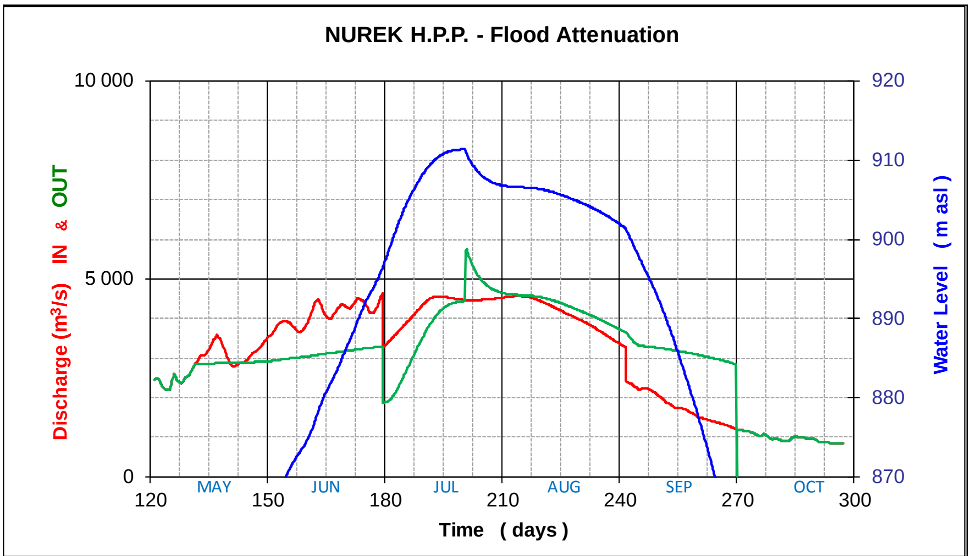
In normal operation, the water level in Nurek will be around the FSL, that is to say 910 masl.

As most of the sediments will be captured in Rogun during the observation period of 40 years, the volume available for flood attenuation will not change during this period.

Given the volume of Nurek reservoir, a low water level at the beginning of the flood could have a significant impact on Nurek flood attenuation efficiency. Therefore, it could be of interest to drawdown the level in Nurek when a flood is detected, in order to take advantage of its large volume to attenuate the flow entering in the reservoir.

The following graph illustrates the scenario of level drawdown in Nurek for the alternative FSL = 1255 masl at Rogun and with 3 tunnels and 1 surface spillway in operation. It is considered that the water level in Nurek has been successfully drawn down early June. At this time, the inlet flow in Nurek is around 4 000 m3/s as the turbines are still in operation at Rogun. The available flow evacuation devices at Nurek are the tunnel spillway and the turbines. Due to its limited head on the tunnel spillway, the discharge capacity, including turbines is around 3 000 m3/s in June. At the same time, the inlet flow in Nurek is more than 4 000 m3/s. This difference represents a volume of 2.6 billion cubic meters for one month, that is to say most of Nurek storage capacity. This means that, when the peak of the flood starts and the turbines are not available, Nurek is almost full (water level at 900 masl).

After 3 weeks, the turbines can restart. In the graph below, all the turbines are restarted, bringing the flow discharged from Nurek at 5 750 m3/s. The number of turbines to be restarted at Nurek can be adjusted in order to remain around the design flow of Nurek, that is to say 5 400 m3/s.



Reservoir:		
V = a . (H - H°) ^b		
H°=	630	m asl
a =	4,56E-03	- -
b =	2,60	- -

Reservoir Operation Rules

FSL =	**860,00**	m asl
H° =	**860,00**	m asl
err =	**0,15**	m
Qmx=	**99 999**	m3/s
QT =	**0**	m3/s

Spillway					
Qv = m * (2g)^0.5* N * b * (H-Hc)^a					
	SS	ML1	HL	HLs	
N =	**1**	**0**	**1**	**0**	
b =	**21,9**	**78,0**	**52**	**172**	
Hc =	**897,7**	**1100,0**	**810,0**	**810,0**	
a =	**1,5**	**0,5**	**0,5**	**0,5**	
h_D=	**10,000**				
p =	**4,000**				
$m°_{DEV}$ =	*0,469*				
m $_{ORIF}$=		**0,880**	**0,880**	**0,880**	
Q RN =	0	0	1 421	0	1 421
QMax=	2 409	0	3 452	0	5 861

Hydrograph		
Hydrograph:	**3**	
Q in-max =	**ROG**	*m3/s*
Results		
Qout-max=	5 750	*m3/s*
H max =	911,38	*m asl*
T (Hmax) =	200,38	*days*

ALA; 21.10.13

Figure 5-1 Alternative FSL = 1255 masl PMF management with 3 tunnels and 1 surface spillway at Rogun after 40 years of sedimentation. Flood attenuation at Nurek with initial level at 860 masl.

5.4 Flood attenuation cases studied

The cases studied are the possible options defined for the protection of Rogun and upgraded if necessary to fulfill the protection of the cascade function.

Alternative FSL = 1220 masl:

The possible options identified for the protection of Rogun dam for the alternative FSL = 1220 masl are:

- 3 modules of surface spillway of 4 gates each and 1 tunnel with 3 gates, dam crest level at 1230 masl

- 2 modules of surface spillway of 4 gates each and 1 tunnel with 3 gates each, dam crest level at 1231.5 masl

- 1 module of surface spillway of 4 gates and 2 tunnels with 3 gates each, dam crest level at 1234.5 masl.

- 1 surface spillway of 4 gates and 3 tunnels with 3 gates each; dam crest level is 1230 masl;

For each spillways combination, the calculations are done for a PMF coming after 30 years of sedimentation, and a water level in Rogun reservoir at the beginning of the flood at 1155 masl

Another calculation is performed for a 10 000 years return period flood coming after 30 years of sedimentation, a water level in Rogun reservoir at the beginning of the flood at 1155 masl, N-1 tunnels in operation and the surface spillway with 3 gates opened for 1 surface spillway and 6 gates opened for 2 or 3 modules of surface spillway.

Alternative FSL = 1255 masl:

The efficient option identified for the protection of Rogun dam for the alternative FSL = 1255 masl is 1 surface spillway with 4 gates, 3 tunnels with 3 gates each and a dam crest at 1265 masl

The calculations are done for a PMF coming after 40 years of sedimentation, a water level in Rogun reservoir at the beginning of the flood at 1205 masl.

Another calculation is performed for a 10 000 years return period flood coming after 40 years of sedimentation, a water level in Rogun reservoir at the beginning of the flood at 1205 masl and 2 tunnels in operation and the surface spillway with 3 gates

Alternative FSL = 1290 masl:

The efficient option identified for the protection of Rogun dam for the alternative FSL = 1290 masl is 1 surface spillway with 4 gates, 2 high level tunnels with 3 gates each, 1 mid level outlet with 3 gates and a dam crest at 1300 masl

The calculations are done for a flood coming after 40 years of sedimentation, a water level in Rogun reservoir at the beginning of the flood at 1260 masl.

Another calculation is performed for a 10 000 years return period flood coming after 40 years of sedimentation, a water level in Rogun reservoir at the beginning of the flood at 1260 masl and 2 tunnels in operation and the surface spillway with 3 gates opened.

5.5 Analysis of the results of calculations

Alternative FSL = 1220 masl

The results of calculations performed according to the cases described in 5.3 are as follows:

Rogun : 3 surface spillways, 1 tunnel, dam crest at 1230 masl

Δt (years) = 30 years	PMF		10000 years	
Starting water level at Nurek	900	860	900	860
tunnel gates at Rogun	3	3	0	0
surface gates at Rogun	12	12	10	10
Hmax at Nurek	918	916,7	911,6	900,4
Qmax outlet of Nurek	6775	6283	4580	3511

Rogun : 2 surface spillways, 1 tunnel, dam crest at 1232 masl

Δt (years) = 40 years	PMF		10000 years	
Starting water level at Nurek	900	860	900	860
tunnel gates at Rogun	3	3	0	0
surface gates at Rogun	8	8	6	6
Hmax at Nurek	917,9	916,6	911,4	900,1
Qmax Nurek Surface spillway	6733	6223	4607	3485

Rogun : 1 surface spillways, 2 tunnels, dam crest at 1234 masl

Δt (years) = 30 years	PMF		10000 years	
Starting water level at Nurek	900	860	900	860
tunnel gates at Rogun	6	6	3	3
surface gates at Rogun	4	4	3	3
Hmax at Nurek	916,7	915,0	907,3	895,3
Qmax outlet of Nurek	6255	5759	4533	3286

Rogun : 1 surface spillways, 3 tunnels, dam crest at 1230 masl

Δt (years) = 30 years	PMF		10000 years	
Starting water level at Nurek	900	860	900	860
tunnel gates at Rogun	9	9	6	6
surface gates at Rogun	4	4	3	3
Hmax at Nurek	914,7	914,2	908,8	902,8
Qmax outlet of Nurek	5885	5862	4985	3840

Red cells highlight values higher than the dam core crest level.

Orange cells highlight values between the FSL and dam core crest level (discharge through Nurek surface spillway exceeding its design discharge). .

These results show that:

- in all cases, the water level in Nurek exceeds the maximum water level accepted during the PMF. This means that acceptable options for Rogun need to be adapted in order to meet the requirement of protection of the structures downstream of Rogun.

- In all cases, the maximum flow discharged from Nurek is higher than the design value of 5462 m3/s.

- In all cases, for the PMF management, drawing down Nurek when a flood is detected does not bring a sufficient difference in the maximum water level at Nurek and in the flow at the outlet of Nurek to make the solutions acceptable.

The way to reduce the maximum water level in Nurek and the maximum flow at the outlet of Nurek is to reduce the flow at the outlet of Rogun and, as a consequence, increase the storage capacity in Rogun, and consequently the Rogun dam crest.

Calculations show that no acceptable solution exists for options with one tunnel only: with one tunnel, the only way to bring the maximum water level in Nurek to an acceptable value is to increase the dam crest at Rogun at about 1280 masl.

With two tunnels, similar calculations show that the dam crest must be raised up to 1258 at Rogun and that only one gate of the surface spillway should be partially opened. In this case, the maximum water level in Nurek is 910 masl with a starting water level in April at 860 masl and the peak flow in the tunnel spillway is 2 021 m3/s.

With 3 tunnels in order to come to a situation where the maximum water level in Nurek is around 910 and the maximum flow in the spillway is around 2020 m3/s, it is necessary to use 3 tunnels with a partial opening of the gates (two tunnels fully opened and the third one opened 60%), and to rise the dam crest at Rogun up to 1251 masl. The results in Nurek are as follows:

Rogun : 2 tunnels opened, 1 tunnel 60% opened, dam crest at 1251 masl

Δt (years) = 30 years	PMF		10000 years	
Starting water level at Nurek	900	860	900	860
tunnel gates at Rogun	7,8	7,8	6	6
surface gates at Rogun	0	0	0	0
Hmax at Nurek	910,3	910,1	907,6	899,2
Qmax Nurek Surface spillway	2105	2043	1423	72

Therefore, there is no feasible solution to protect Nurek and the cascade against the PMF with a dam alternative FSL =1220 masl without significantly increasing the available freeboard, hence changing drastically the dam layout. The dam crest should be at least at elevation 1251 masl.

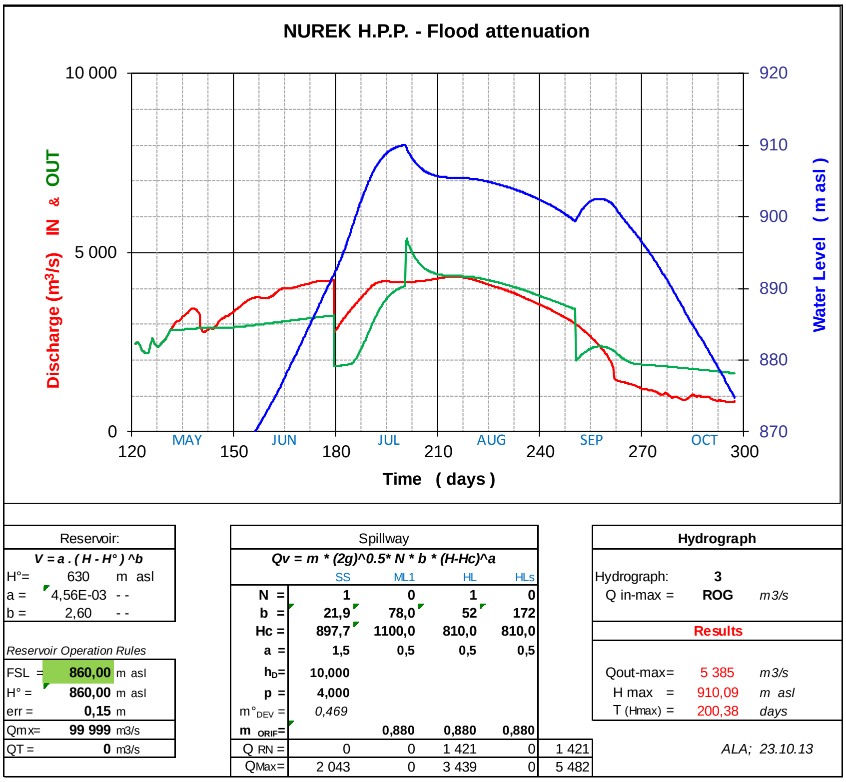


Figure 5-2 Alternative FSL = 1220 masl PMF management with 3 tunnels 1 tunnel opened 60% at Rogun after 30 years of sedimentation. Flood attenuation at Nurek with initial level at 860 masl.

Alternative FSL = 1255 masl

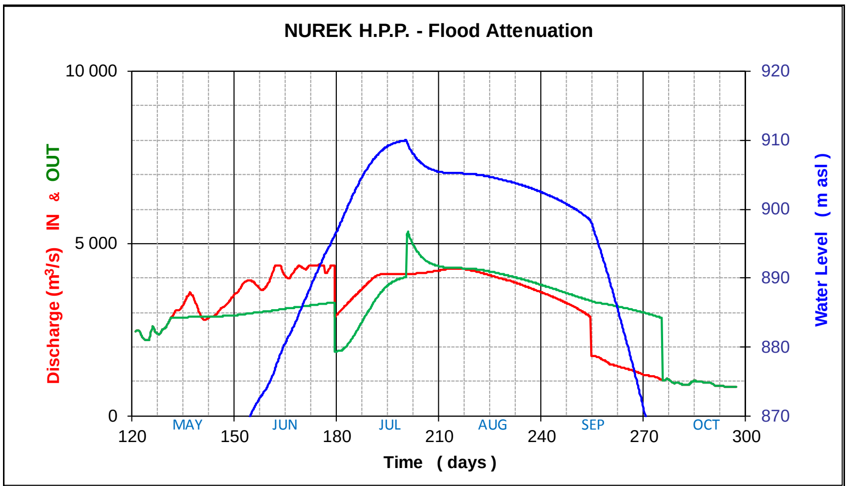
The results of calculations performed according to the cases described in 5.3 are as follows:

Rogun : 1 surface spillways, 3 tunnels, dam crest at 1265 masl								
Δt (years) = 40 years	PMF				10000 years			
Water level in April	900	860	900	860	900	860	900	860
tunnel gates	9	9	9	9	9	9	9	9
surface gates	0	0	4	4	0	0	4	4
Hmax	911,5	911,4	911,5	911,4	909,1	904,8	909,1	904,8
Qmax	5414	5391	5414	5391	4694	4127	4694	4127

They show that:

- Drawing down Nurek when a flood is detected brings an additional safety for floods of the order of magnitude of the 10 000 years flood. However, for the most extreme ones, it does not bring a significant difference in the maximum water level at Nurek and in the flow at the outlet of Nurek.

- The maximum water level in Nurek reservoir and the maximum flow in Nurek surface spillway are similar to the current design features of Nurek provided that in Rogun, 2 tunnels are fully in operation and 1 is opened at 60% at Rogun and the surface spillway is closed.



Reservoir:		
V = a . (H - H°) ^b		
H°=	630	m asl
a =	4,56E-03	- -
b =	2,60	- -
Reservoir Operation Rules		
FSL =	860,00	m asl
H° =	860,00	m asl
err =	0,15	m
Qmx=	99 999	m3/s
QT =	0	m3/s

Spillway					
Qv = m * (2g)^0.5* N * b * (H-Hc)^a					
	SS	ML1	HL	HLs	
N =	1	0	1	0	
b =	21,9	78,0	52	172	
Hc =	897,7	1100,0	810,0	810,0	
a =	1,5	0,5	0,5	0,5	
h_0=	10,000				
p =	4,000				
m°$_{DEV}$ =	0,469				
m $_{ORIF}$=		0,880	0,880	0,880	
Q RN =	0	0	1 421	0	1 421
QMax=	2 012	0	3 438	0	5 450

Hydrograph		
Hydrograph:	3	
Q in-max =	ROG	m3/s
Results		
Qout-max=	5 351	m3/s
H max =	909,97	m asl
T (Hmax) =	200,38	days

ALA; 23.10.13

Figure 5-3 Alternative FSL = 1255 masl PMF management with 3 tunnels 1 tunnel opened 60% at Rogun after 40 years of sedimentation. Flood attenuation at Nurek with initial level at 860 masl. Turbines operating up to the day 180.

Alternative FSL = 1290 masl

The results of calculations performed according to the cases described in 5.3 are as follows:

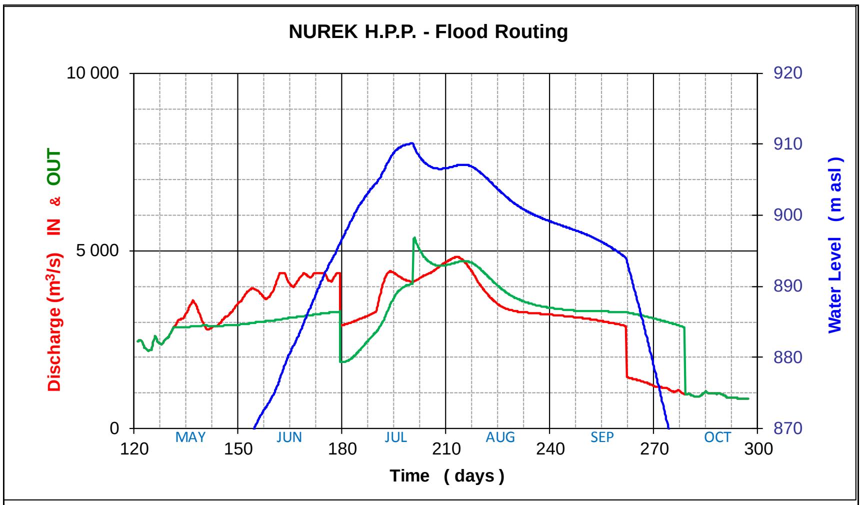
Rogun : 1 surface spillway, 2 high level tunnel, 1 mid level outlet, dam crest at 1300 masl

Δt (years) = 40 years	PMF		10000 years	
Starting water level at Nurek (may)	900	860	900	860
high level tunnel gates at Rogun	4	4	4	4
mid level outlet gates at Rogun	1	1	1	1
surface gates at Rogun	4	4	4	4
Hmax at Nurek	910,3	910,1	909,4	906,4
Qmax Surface spillway Nurek	2090	2050	1127	0
Qmax outlet of Nurek	5425	5389	4444	3292

They show that:

- Drawing down Nurek when a flood is detected brings an additional safety for floods of the order of magnitude of the 10 000 years flood. However, for the most extreme ones, it does not bring a significant difference in the maximum water level at Nurek and in the flow at the outlet of Nurek

- The maximum water level in Nurek reservoir and the maximum flow in Nurek surface spillway are similar to the current design features of Nurek provided that in Rogun two gates of the high level tunnels and two gates of the mid level outlet are closed.



Reservoir:		
V = a . (H - H°) ^b		
H°=	630	m asl
a =	4,56E-03	- -
b =	2,60	- -

Reservoir Operation Rules

FSL =	860,00	m asl
H° =	860,00	m asl
err =	0,15	m
Qmx=	99 999	m3/s
QT =	0	m3/s

Spillway					
*Qv = m * (2g)^0.5* N * b * (H-Hc)^a*					
	SS	ML1	HL	HLs	
N =	1	0	1	0	
b =	21,9	78,0	52	172	
Hc =	897,7	1100,0	810,0	810,0	
a =	1,5	0,5	0,5	0,5	
h_D=	10,000				
p =	4,000				
$m°_{DEV}$ =	*0,469*				
m_{ORIF}=		0,880	0,880	0,880	
Q RN =	0	0	1 421	0	1 421
QMax=	2 050	0	3 439	0	5 489

Hydrograph		
Hydrograph:	3	
Q in-max =	ROG	*m3/s*
Results		
Qout-max=	5 389	*m3/s*
H max =	910,11	*m asl*
T (Hmax) =	200,38	*days*

ALA; 23.10.13

Figure 5-4 Alternative FSL = 1290 masl PMF management with 2 high level tunnels opened 67%, 1 mid level outlet opened 33% and the surface spillway in operation at Rogun after 40 years of sedimentation. Turbines operating up to day 180. Flood attenuation at Nurek with initial level at 860 masl.

5.6 Conclusion on the management of high floods

Alternative FSL = 1220 masl

For this alternative, 3 options meet the criteria defined in paragraph 2 “Basic Constraints”. Out of these options, none is able to provide a protection to Nurek with a flow released through Nurek surface spillway in the range of its design value.

The closest solution for Nurek protection is to have 3 tunnels and a surface spillway at Rogun and a dam crest raised to at least at 1251 masl.

Alternative FSL = 1255 masl

For this alternative, the solution chosen at Rogun attenuates the flow enough so the maximum water level and the maximum flow in the surface spillway meet the values required.

Alternative FSL = 1290 masl

For this alternative, the solution chosen at Rogun attenuates the flow enough so that the maximum water level and the maximum flow in the surface spillway meet the values required.

6 CONCLUSION AND RECOMMENDATION

Alternative FSL = 1220 masl

Three options have been found to provide an acceptable protection to Rogun. The least expensive option, and the recommended one, is to have 2 modules of the surface spillway, 1 tunnel and a crest rise of 1.5 m.

Out of these three options, none can be reasonably adapted to also protect Nurek and the cascade. The closest solution could be to have the dam crest raised to at least at 1251 masl.

Functions	**Rogun protection only**	**Cascade protection**
Structures and requirements	2 module of surface spillway, 1 tunnels + 1.5 m crest rise	1 module of surface spillway, 3 tunnels and a dam 21 m higher
Costs	291 M€	603 M€

The Consultant recommends reviewing in detail the design of Nurek surface spillway during phase 3 studies in order to make sure that no damage will occur even at its design flow.

The maximum levels in the reservoirs and maximum flows are the following:

t=30 years	Maximum levels	Maximum Outflows
Rogun during the PMF	1226.4 masl	7496 m^3/s
Rogun during the 10 000 years flood	1227.9 masl	5427 m^3/s
Nurek during the PMF	916.6 masl	6223 m^3/s
Nurek during the 10 000 years flood	900.1 masl	3485 m^3/s

Alternative FSL = 1255 masl

The solution with the 3 tunnels necessary for the construction complemented by 1 surface spillway provides an acceptable protection of Rogun against the PMF and the 10 000 years return period flood.

Closing partially one tunnel when a potential high flood is detected provides a protection to Nurek and the downstream facilities.

The surface spillway at Rogun is compulsory in order to face the unavailability of gates in the tunnels.

In addition, the Consultant recommends reviewing in detail the design of Nurek existing surface spillway during phase 3 studies in order to make sure that no damage will occur even at its design flow.

The maximum levels in the reservoirs and maximum outflows are the following:

t=40 years	Maximum levels	Maximum outflows
Rogun during the PMF	1262,0 masl	4380 m3/s
Rogun during the 10 000 years flood	1232,5 masl	3614 m3/s
Nurek during the PMF	910,0 masl	5351 m3/s
Nurek during the 10 000 years flood	901 masl	3587 m3/s

Alternative FSL = 1290 masl

The solution with the 2 high level tunnels and the mid-level outlet necessary for the construction complemented by 1 surface spillway provides an acceptable protection of Rogun against the PMF and the 10 000 years return period flood.

Closing partially one high level tunnel and opening only 1 gate of the mid level outlet when a potential high flood is detected provides a protection to Nurek and the downstream facilities

The surface spillway is compulsory in order to face the unavailability of gates in the tunnels.

In addition, the Consultant recommends reviewing in detail the design of Nurek surface spillway during phase 3 studies in order to make sure that no damage will occur even at its design flow.

The maximum levels in the reservoirs are the following:

t=40 years	Maximum levels	Maximum Outflows
Rogun during the PMF	1291,9 masl	4828 m3/s
Rogun during the 10 000 years flood	1278,9 masl	3394 m3/s
Nurek during the PMF	910,1 masl	5389 m3/s
Nurek during the 10 000 years flood	895,9 masl	3292 m3/s

Conclusion

It has been decided, during the course of the study, to make Rogun a multi-purpose dam and design it to protect the whole cascade against the PMF. This additional advantage is provided by the two highest alternatives studied (FSL= 1255 masl and FSL= 1290 masl). Since these benefits are inherent in the system costs for these designs, for a proper comparison, it was necessary to include the costs of providing similar flood protection benefits in the economic analysis for the No Rogun case and any of the Rogun design options which do not confer this benefit.

To quantify this, we considered the avoided costs which would have to be incurred for an alternative method, namely constructing additional spillways at the Nurek HPP. The cost of these spillways was estimated at USD 318 million, although estimates to protect the full cascade could be as much as USD 945 million.

In the next stages of the project, it is recommended to study more in detail the protection system of the whole Vakhsh cascade, focusing in particular on the Nurek spillway capacity, on the flow limit for spillage though the turbines at Rogun and on the possible mitigation measures to implement such as an early flood detection system.

Annex 1- Calculation Results

Red cells indicate situations where the reservoir level is higher than the crest. Cases relating to red cells are not acceptable.

Orange cells indicate situations where the reservoir level is higher than the core crest. These cases, when exceptional for a solution are considered acceptable.

FSL = 1220 masl

Δt (years) =	0																
Water level in April	1125																
tunnel gates	9	6	7	8	9	6	7	8	9	6	7	8	9	6	7	8	9
surface gates	0	1	1	1	1	2	2	2	2	3	3	3	3	4	4	4	4
Hmax	1225,0	1238,8	1234,5	1229,9	1224,1	1235,6	1231,4	1227,4	1223,5	1233,6	1230,1	1226	1223	1232,1	1229,2	125,5	1222,7
Qmax	4447	4841	4735	4682	4601	5586	5074	4804	4698	6047	5438	4862	4763	6366	5710	5014	4811

Δt (years) =	0																
Water level in April	1135																
tunnel gates	9	6	7	8	9	6	7	8	9	6	7	8	9	6	7	8	9
surface gates	0	1	1	1	1	2	2	2	2	3	3	3	3	4	4	4	4
Hmax	1225,0	1238,8	1234,5	1229,9	1224,1	1235,6	1231,4	1227,4	1223,5	1233,6	1230,1	1226	1223	1232,1	1229,2	1225,5	1222,7
Qmax	4447	4842	4735	4682	4694	5587	5074	4804	4698	6048	5438	4862	4763	6367	5710	5014	4811

Δt (years) =	20																
Water level in April	1125																
tunnel gates	9	6	7	8	9	6	7	8	9	6	7	8	9	6	7	8	9
surface gates	0	1	1	1	1	2	2	2	2	3	3	3	3	4	4	4	4
Hmax	1230,0	1243,8	1238,8	1233,5	1228,2	1238,7	1235,4	1231,5	1227,3	1235,7	1233,3	1230,2	1226,6	1233,7	1231,8	1228,2	1226,1
Qmax	4577	5675	5371	5128	4987	6447	6052	5631	5279	6842	6456	5969	5499	7071	6731	6219	5672

Δt (years) =	20																
Water level in April	1135																
tunnel gates	9	6	7	8	9	6	7	8	9	6	7	8	9	6	7	8	9
surface gates	0	1	1	1	1	2	2	2	2	3	3	3	3	4	4	4	4
Hmax	1230	1243,8	1238,8	1233,5	1228,2	1238,7	1235,4	1231,5	1227,7	1235,7	1233,3	1230,2	1226,6	1233,7	1231,8	1229,2	1226,1
Qmax	4577	5675	5371	5128	4987	6447	6052	5631	5279	6842	6456	5969	5499	7071	6371	6219	5672

Δt (years) =	30																
Water level in April	1125																
tunnel gates	9	6	7	8	9	6	7	8	9	6	7	8	9	6	7	8	9
surface gates	0	1	1	1	1	2	2	2	2	3	3	3	3	4	4	4	4
Hmax	Sediments above starting water level in april																
Qmax																	

Δt (years) =	30																
Water level in April	1155																
tunnel gates	9	6	7	8	9	6	7	8	9	6	7	8	9	6	7	8	9
surface gates	0	1	1	1	1	2	2	2	2	3	3	3	3	4	4	4	4
Hmax	1231,6	1244,4	1239,7	1234,7	1229,7	1239	1236	1232,4	1228,5	1235,9	1233,7	1230,9	1227,7	1233,8	1232,1	1229,7	1227
Qmax	4619	5789	5507	5286	5162	6548	6199	5826	5515	6920	6598	6180	5772	7127	6861	6435	5970

FSL = 1255 masl:

Δt (years) =	0																		
Water level in April	1195																		
tunnel gates	7	8	9	6	7	8	9	6	7	8	9	6	7	8	9	6	7	8	9
surface gates	0	0	0	1	1	1	1	2	2	2	2	3	3	3	3	4	4	4	4
Hmax	1261,4	1250,2	1240,5	1264,5	1258,8	1250,2	1240,5	1260,9	1257,3	1250,2	1240,5	1258,8	1256,2	1250,1	1240,5	1257,5	1255,5	1250,1	1240,5
Qmax	4060	4213	4392	4447	4297	4213	4392	4702	4532	4213	4392	4808	4672	4213	4392	4864	4761	4213	4392

Δt (years) =	0																		
Water level in April	1205																		
tunnel gates	7	8	9	6	7	8	9	6	7	8	9	6	7	8	9	6	7	8	9
surface gates	0	0	0	1	1	1	1	2	2	2	2	3	3	3	3	4	4	4	4
Hmax	1262,7	1251,9	1242,7	1264,8	1259,4	1251,8	1242,7	1261	1257,6	1251,6	1242,7	1258,9	1256,5	1251,5	1242,7	1257,5	1255,6	1251,4	1242,7
Qmax	4182	4436	4644	4486	4363	4436	4644	4724	4594	4436	4644	4819	4721	4436	4644	4869	4798	4436	4644

Δt (years) =	0																		
Water level in April	1215																		
tunnel gates	7	8	9	6	7	8	9	6	7	8	9	6	7	8	9	6	7	8	9
surface gates	0	0	0	1	1	1	1	2	2	2	2	3	3	3	3	4	4	4	4
Hmax	1264,9	1254,7	1245,5	1265,2	1260,3	1254,1	1245,5	1261,1	1258	1253,7	1245,5	1258,9	1256,7	1253,3	1245,5	1257,5	1255,7	1253	1245,5
Qmax	4365	4644	4644	4543	4458	4644	4644	4751	4666	4644	4644	4832	4769	4644	4644	4875	4829	4670	4644

Δt (years) =	40																		
Water level in April	1195																		
tunnel gates	7	8	9	6	7	8	9	6	7	8	9	6	7	8	9	6	7	8	9
surface gates	0	0	0	1	1	1	1	2	2	2	2	3	3	3	3	4	4	4	4
Hmax	1271,9	1258,6	1247,0	1266,6	1262,2	1256,4	1247,0	1262,7	1258,8	1255,2	1247,0	1261,2	1257,1	1254,4	1247,0	1260,1	1256,4	1253,8	1247,0
Qmax	4137	4316	4537	4740	4681	4594	4537	5102	4816	4727	4537	5504	4875	4804	4537	5800	5022	4853	4537

Δt (years) =	40																		
Water level in April	1205																		
tunnel gates	7	8	9	6	7	8	9	6	7	8	9	6	7	8	9	6	7	8	9
surface gates	0	0	0	1	1	1	1	2	2	2	2	3	3	3	3	4	4	4	4
Hmax	1272,5	1259,6	1248,3	1266,6	1262,3	1256,8	1248,3	1263,1	1258,9	1255,4	1248,3	1261,5	1257,4	1254,5	1248,3	1260,4	1256,8	1253,8	1248,3
Qmax	4214	4440	4644	4746	4692	4625	4644	5196	4820	4748	4644	5608	4957	4817	4644	5494	5166	4861	4644

Δt (years) =	40																		
Water level in April	1215																		
tunnel gates	7	8	9	6	7	8	9	6	7	8	9	6	7	8	9	6	7	8	9
surface gates	0	0	0	1	1	1	1	2	2	2	2	3	3	3	3	4	4	4	4
Hmax	1273,9	1261,5	1251,3	1266,7	1262,5	1257,3	1251,25	1263,9	1259,5	1255,6	1251,2	1262,2	1258,6	1255,1	1251,1	1260,9	1257,8	1254,8	1251,1
Qmax	4366	4644	4648	4758	4715	4678	4695	5380	4942	4779	4738	5808	5248	4953	4778	6115	5485	5099	4815

Δt (years) =	60																		
Water level in April	1195																		
tunnel gates	7	8	9	6	7	8	9	6	7	8	9	6	7	8	9	6	7	8	9
surface gates	0	0	0	1	1	1	1	2	2	2	2	3	3	3	3	4	4	4	4
Hmax	Sediments above 1195 masl																		
Qmax																			

Δt (years) =	60																		
Water level in April	1205																		
tunnel gates	7	8	9	6	7	8	9	6	7	8	9	6	7	8	9	6	7	8	9
surface gates	0	0	0	1	1	1	1	2	2	2	2	3	3	3	3	4	4	4	4
Hmax	1272,4	1261,8	1253,5	1266,4	1262,3	1257,4	1253,3	1263,8	1260,2	1256,7	1253,3	1262,2	1259,2	1256,2	1252,9	1261,0	1258,4	1255,7	1252,8
Qmax	4304	4460	4703	4714	4691	4681	4830	5360	5080	4958	4940	5810	5423	5178	5036	6133	5686	5358	5121

Δt (years) =	60																		
Water level in April	1215																		
tunnel gates	7	8	9	6	7	8	9	6	7	8	9	6	7	8	9	6	7	8	9
surface gates	0	0	0	1	1	1	1	2	2	2	2	3	3	3	3	4	4	4	4
Hmax	1272,6	1262,2	1254,2	1266,4	1262,3	1257,9	1253,9	1263,9	1260,4	1257,2	1253,7	1262,3	1259,4	1256,6	1253,5	1261	1258,6	1256,1	1253,3
Qmax	4383	4644	4722	4717	4695	4740	4882	5387	5122	5044	5016	5838	5471	5283	5132	6162	5739	5476	5233

FSL = 1290 masl

Δt (years) =	0														
Water level in April	1260														
Mid Level tunnel gates	1	1	1	2	2	2	2	1	1	1	2	1	1	1	2
High Level tunnel gates	4	5	6	3	4	5	6	4	5	6	3	4	5	6	3
surface gates	0	0	0	0	0	0	0	1	1	1	1	2	2	2	2
Hmax	1300,9	1291,8	1283,4	1288,5	1279,9	1274,4	1270,8	1296,4	1290,8	1283,4	1288,1	1294,1	1290,0	1283,4	1287,9
Qmax	4386	4644	4644	4644	4644	4819	5210	4386	4644	4644	4644	4495	4644	4644	4644

Δt (years) =	0														
Water level in April	1270														
Mid Level tunnel gates	1	1	1	2	2	2	2	1	1	1	2	1	1	1	2
High Level tunnel gates	4	5	6	3	4	5	6	4	5	6	3	4	5	6	3
surface gates	0	0	0	0	0	0	0	1	1	1	1	2	2	2	2
Hmax	1306,0	1297,0	1288,6	1293,9	1285,4	1281,5	1278,2	1298,3	1293,5	1288,0	1291,7	1295,0	1291,7	1287,6	1290,4
Qmax	4552	4644	4644	4644	4644	4985	5407	4552	4644	4644	4644	4660	4644	4644	4644

Δt (years) =	0														
Water level in April	1280														
Mid Level tunnel gates	1	1	1	2	2	2	2	1	1	1	2	1	1	1	2
High Level tunnel gates	4	5	6	3	4	5	6	4	5	6	3	4	5	6	3
surface gates	0	0	0	0	0	0	0	1	1	1	1	2	2	2	2
Hmax	1311,6	1302,7	1294,3	1299,9	1292,8	1289,1	1286,1	1299,8	1296,2	1293,2	1295,1	1297,3	1294,9	1292,5	1294,1
Qmax	4644	4644	4644	4644	4680	5155	5609	4644	4746	4967	4806	5153	5218	5297	5229

Δt (years) =	40														
Water level in April	1260														
Mid Level tunnel gates	1	1	1	2	2	2	2	1	1	1	2	1	1	1	2
High Level tunnel gates	4	5	6	3	4	5	6	4	5	6	3	4	5	6	3
surface gates	0	0	0	0	0	0	0	1	1	1	1	2	2	2	2
Hmax	1306,0	1295,8	1286,3	1292,1	1282,5	1276,8	1272,7	1298,3	1293,0	1286,2	1290,9	1295,0	1291,5	1286,1	1289,9
Qmax	4387	4644	4644	4644	4644	4877	5261	4430	4644	4644	4644	4669	4644	4644	4644

Δt (years) =	40														
Water level in April	1270														
Mid Level tunnel gates	1	1	1	2	2	2	2	1	1	1	2	1	1	1	2
High Level tunnel gates	4	5	6	3	4	5	6	4	5	6	3	4	5	6	3
surface gates	0	0	0	0	0	0	0	1	1	1	1	2	2	2	2
Hmax	1310,5	1300,4	1290,9	1297,1	1287,8	1283,0	1279,6	1299,4	1294,7	1289,3	1293,0	1295,4	1292,3	1289,0	1291,1
Qmax	4552	4644	4644	4644	4644	5028	5443	4578	4644	4644	4644	4750	4729	4767	4713

Δt (years) =	40														
Water level in April	1280														
Mid Level tunnel gates	1	1	1	2	2	2	2	1	1	1	2	1	1	1	2
High Level tunnel gates	4	5	6	3	4	5	6	4	5	6	3	4	5	6	3
surface gates	0	0	0	0	0	0	0	1	1	1	1	2	2	2	2
Hmax	1315,6	1305,6	1296,4	1302,5	1294,9	1290,6	1287,2	1301,2	1298,1	1294,9	1297,0	1298,8	1296,4	1293,8	1295,5
Qmax	4644	4644	4644	4644	4719	5188	5635	4814	4974	5146	5015	5493	5514	5543	5509

Δt (years) =	100														
Water level in April	1260														
Mid Level tunnel gates	1	1	1	2	2	2	2	1	1	1	2	1	1	1	2
High Level tunnel gates	4	5	6	3	4	5	6	4	5	6	3	4	5	6	3
surface gates	0	0	0	0	0	0	0	1	1	1	1	2	2	2	2
Hmax	1307,5	1298,1	1289,0	1294,8	1285,4	1280,7	1276,4	1298,8	1294,0	1288,3	1292,1	1295,2	1292,0	1287,8	1290;7
Qmax	4389	4644	4644	4644	4644	4966	5359	4495	4644	4644	4644	4708	4671	4644	4647

Δt (years) =	100														
Water level in April	1270														
Mid Level tunnel gates	1	1	1	2	2	2	2	1	1	1	2	1	1	1	2
High Level tunnel gates	4	5	6	3	4	5	6	4	5	6	3	4	5	6	3
surface gates	0	0	0	0	0	0	0	1	1	1	1	2	2	2	2
Hmax	1310,2	1300,9	1291,9	1297,8	1289,3	1284,9	1280,1	1299,5	1295,0	1290,5	1293,3	1295,9	1293,1	1290,1	1292,0
Qmax	4552	4644	4644	4644	4644	5062	5479	4583	4644	4713	4644	5096	4876	4918	4865

Δt (years) =	100														
Water level in April	1280														
Mid Level tunnel gates	1	1	1	2	2	2	2	1	1	1	2	1	1	1	2
High Level tunnel gates	4	5	6	3	4	5	6	4	5	6	3	4	5	6	3
surface gates	0	0	0	0	0	0	0	1	1	1	1	2	2	2	2
Hmax	1314,1	1305,0	1296,4	1302,1	1295,0	1290,9	1287,5	1300,9	1297,9	1294,9	1296,9	1298,6	1296,3	1293,9	1295,5
Qmax	4644	4644	4644	4644	4721	5193	5642	4775	4959	5307	5008	5456	5502	5553	5505

Annex 2: description of Flood attenuation calculations

ROGUN Hydro Power Project

Reservoir Routing

Description of the Spread-sheet used for the

Calculation of Flood Propagation along Reservoirs

1. BASIC EQUATIONS

Spread-sheet calculations have been performed to simulate the flood propagation along reservoirs (Rogun and / or Nurek) in order to estimate maximum discharges and water levels.

The unsteady flow calculations are based on the continuity equation only, as inertia and resistance forces do not play a significant role(1). The governing equation becomes therefore a balance between INFLOW, OUTFLOW and the reservoir STORAGED VOLUME in the unit time.

$$Q_{IN}(t) - Q_{OUT}(H) = \frac{dV}{dt} \qquad \text{(Eq. 1)}$$

Where:

Q_{IN} : INFLOW: discharge produced by the river and/or by the upstream reservoir

Q_{OUT} : OUTFLOW: discharge resulting from the reservoir routing

$\frac{dV}{dt}$: Variation of the reservoir volume in the unit time

H; t : “H” and “t” indicate the reservoir level and time, respectively

Equation (1) is solved step by step making the approximation $dt \approx \Delta t$.

At each time step the calculation proceeds in the following way:

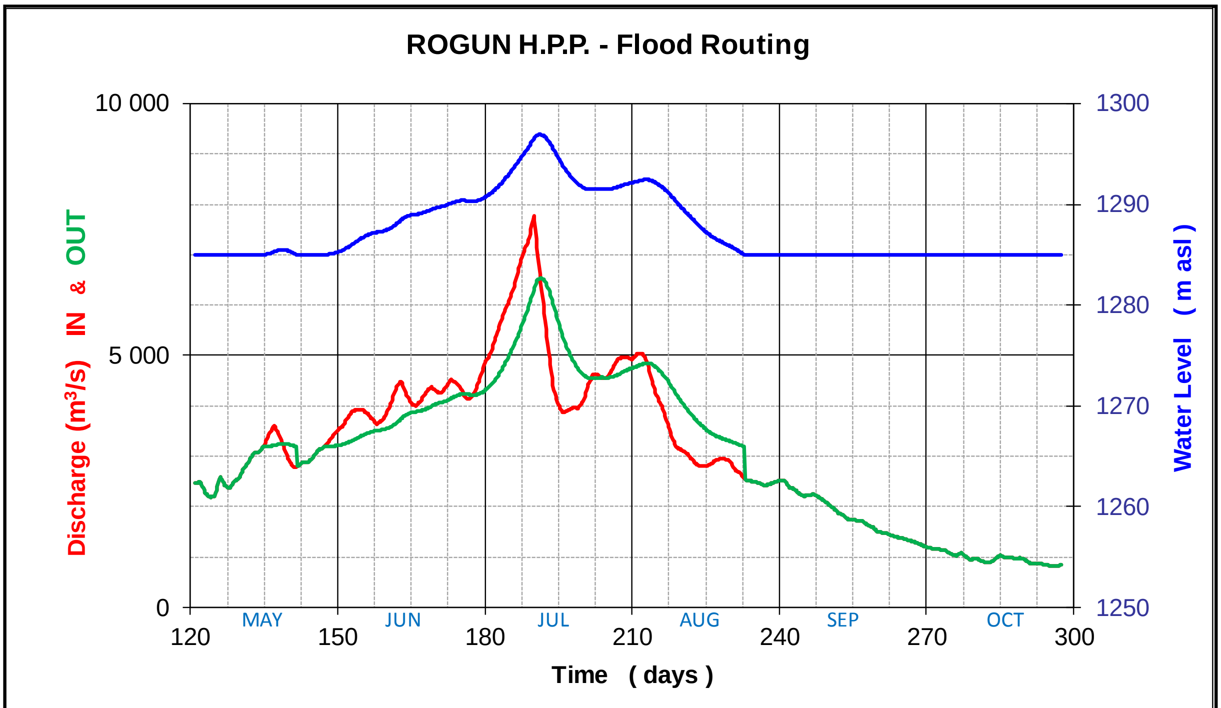
- a new value of the time dependent inflow is entered, and the corresponding volume “$Q_{IN} \times \Delta t$” is evaluated. Time series may be hourly, daily, etc.
- for the same time instant, the outflow is evaluated as a function of the reservoir elevation. The corresponding volume “$Q_{OUT} \times \Delta t$” is also evaluated.
- the differential volume “$(Q_{IN} - Q_{OUT}) \times \Delta t$” is calculated and added to the volume of water in the reservoir estimated in the previous time step. A positive (negative) difference generates an increase (decrease) of the volume of water stored in the reservoir and consequently also an increase (decrease) of the reservoir level.
- calculations are repeated passed the peak of the outflow wave.

(1) « Open Channel Flow », F.M. Henderson. MacMillan Series in Civil Engineering.1966.

2. SPREAD-SHEET DESCRIPTION

2.1 Typical Output

Figure 1 shows the typical presentation of data and results of the spread-sheet calculation.



Reservoir:		
V = a . (H - H°) ^b		
H°=	960	m asl
a =	3,60E-06	- -
b =	3,80	- -
Δt =	0	years
Reservoir Operation Rules		
FSL =	**1285,00**	m asl
H° =	**1285,00**	m asl
err =	**0,02**	m
Qmx=	**99 999**	m3/s
QT =	**0**	m3/s

Spillway					
*Qv = m * (2g)^0.5* N * b * (H-Hc)^a*					
	SS	ML1	ML2	HLs	
N =	**4**	**0**	**0**	**2**	
b =	**8,0**	**78,0**	**78,0**	**41,5**	
Hc =	**1284,0**	**1085**	**1140**	**1190,0**	
a =	**1,5**	**0,5**	**0,5**	**0,5**	
h_D=	**10,000**				
p =	**4,000**				
$m°_{DEV}$ =	*0,469*				
m_{ORIF}=		**0,880**	**0,880**	**0,880**	
Q_{FSL} =	55	0	0	3 136	3 192
QMax=	3 206	0	0	3 329	6 535

Hydrograph		
Hydrograph:	**3**	
Q in-max =	**7 800**	*m3/s*
Results		
Qout-max=	6 535	*m3/s*
H max =	1296,92	*m asl*
T (Hmax) =	191,20	*days*

ALA; 31.08.13

Figure 1: typical output of the spread-sheet reservoir routing calculation

The different fields of the calculation output furnish information about the reservoir capacity, the reservoir operation rules, the discharge capacity of spillways (surface spillways as well as tunnels) and also identify by a code number the hydrograph of the inflowing flood as well as its peak discharge. Finally, results are given graphically (input and output hydrographs and evolution of reservoir surface water level) and numerically (maximum discharge and water level).

The following paragraphs describe in detail that information.

2.2 Inflow

Different inflow hydrograph forms may be used. They are stored in the calculation sheet and identified by a code number (sequential natural number). The stored hydrographs do have a maximum discharge equal to unity.

The operator needs just to declare the shape number and the peak discharge. The program scales up the hydrograph to the given peak discharge.

Hydrographs are defined by sets of (t; Q_{in}) points. The program interpolates data between two consecutive points in order to rewrite the inflow hydrograph with constant time steps.

Time steps may be made as small as desired, looking for precision in the integration process.

2.3 Reservoir characteristics

For the reservoir routing the only reservoir information required is the H-V-S curve (Height-Volume-Surface); i.e. the reservoir capacity as a function of the water surface level.

That information is given in the form of a power function: $V = a \cdot (H – H°)^b$. Origin "H°", coefficient "a" and power "b" result from a power function adjustment to the existing set of data " H ; V ". Figure 2 indicate the function adjustment for the reservoir in its original conditions.

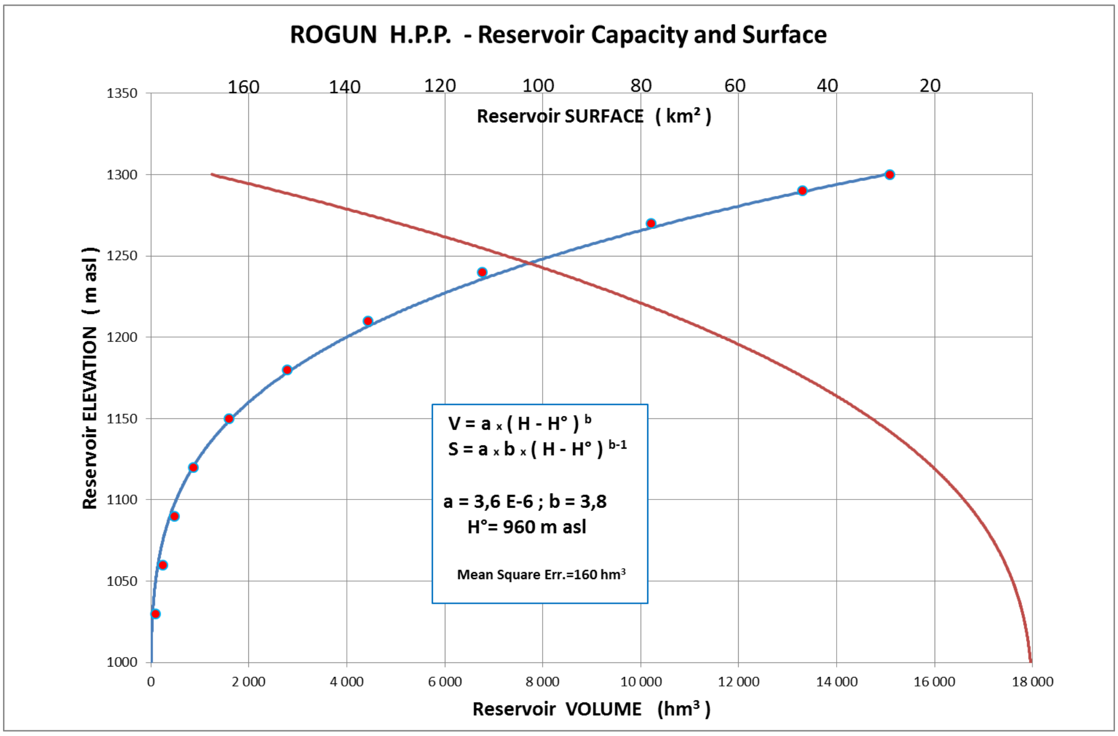


Figure 2: H-V-S curve of the ROGUN Reservoir (original conditions)

The VAKHSH River transports important amounts of sediments. In Report RP-43 on Sedimentation it is reported that the annual volume of sediments entering in the reservoir is about 60 – 100 hm^3/yr.

Taking this information into account series of H-V-S curves were calculated for the three dam height alternatives (full supply levels 1290; 1255 and 1220) and for those two annual rates of solid inflow. These calculations were performed using BUREC's methodology established in "Revision of the Procedure to Compute Sediment Distribution in Large Reservoirs. The results are shown in figure 3, where H-V-S curves are given for time intervals of 20 years.

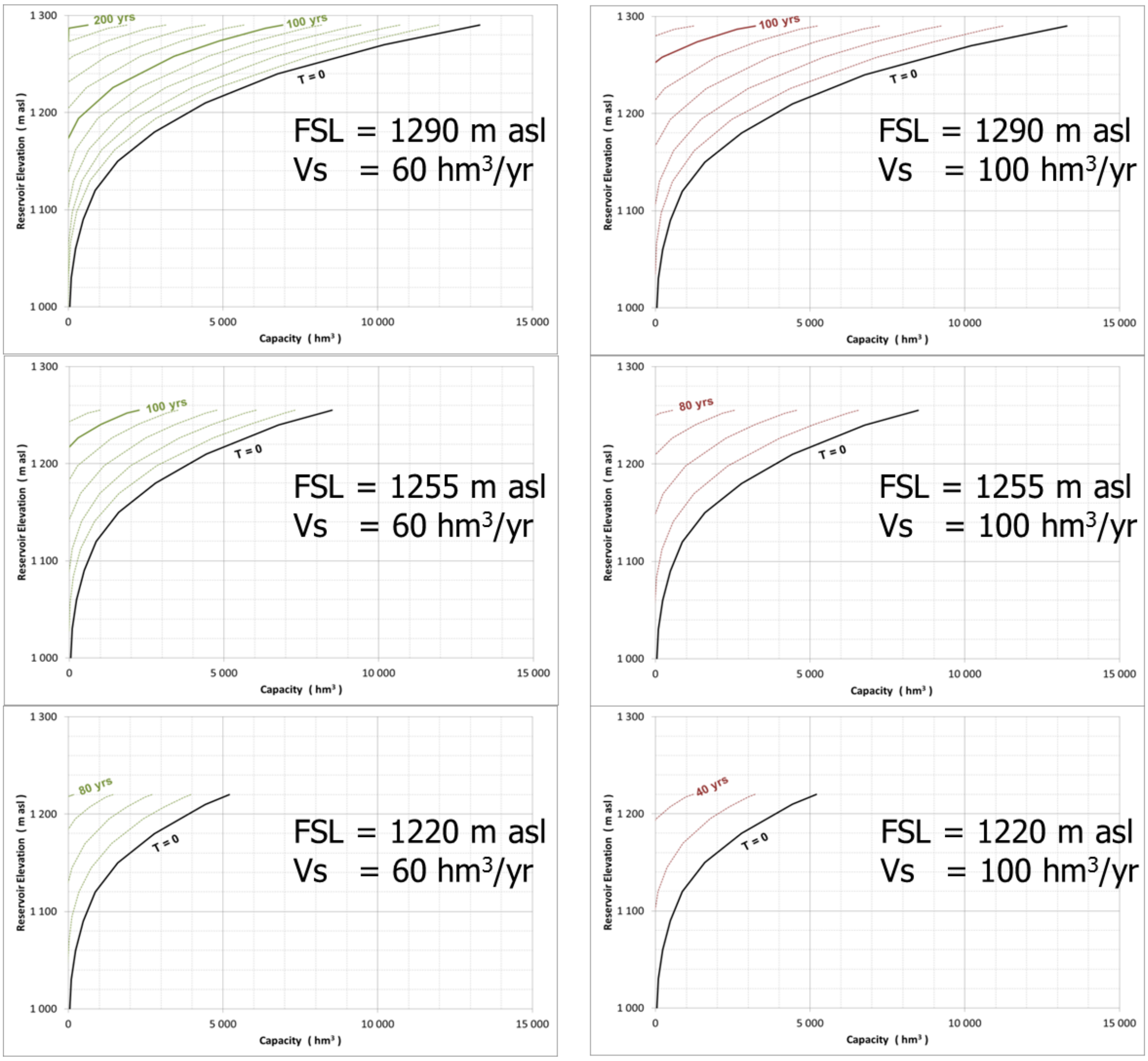


Figure 3: Timely evolution of H-V-S curves

for three dam height alternatives and two sediment rates

A power function was adjusted to each one of those curves and an extra routine was created in the reservoir routing spread-sheet in order to allow the operator to select a timely horizon for the flood routing calculations.

Power Function Adjustement to H-V-S Curves

FSL	sa	Δt	H°	a	b
m asl	hm3/yr	yrs	m asl	div.	- - -
1290	60	0	960	3,60E-06	3,80
1290	60	20	1020	3,89E-04	3,07
1290	60	40	1040	2,25E-04	3,20
1290	60	60	1100	3,19E-02	2,39
1290	60	80	1135	1,09E-01	2,22
1290	60	100	1170	3,80E-01	2,04
1290	60	120	1200	1,59E+00	1,81
1290	60	140	1230	5,08E+00	1,64
1290	60	160	1240	3,82E-01	2,30
1290	60	180	1260	6,21E-04	4,40
1290	60	200	1270	3,99E+00	2,05
1290	100	0	960	3,60E-06	3,80
1290	100	20	1035	4,52E-04	3,07
1290	100	40	1100	5,64E-03	2,72
1290	100	60	1165	3,58E-01	2,05
1290	100	80	1205	5,48E-01	2,06
1290	100	100	1250	6,38E+00	1,68
1290	100	120	1280	2,10E+01	1,77
1255	60	0	960	3,60E-06	3,80
1255	60	20	1030	5,99E-04	3,01
1255	60	40	1085	9,77E-03	2,59
1255	60	60	1145	4,69E-01	1,96
1255	60	80	1175	4,75E-01	2,03
1255	60	100	1200	2,59E-02	2,84
1255	60	120	1245	3,17E+01	1,50
1255	60	140	1250	1,00E+02	0,00
1255	60	160			
1255	60	180			
1255	60	200			
1255	100	0	960	3,60E-06	3,80
1255	100	20	1060	2,75E-03	2,78
1255	100	40	1145	2,45E-01	2,08
1255	100	60	1200	5,44E-01	2,10
1255	100	80	1245	9,51E-02	3,76
1255	100	100	1200	2,59E-02	2,84
1255	100	120	1245	3,17E+01	1,50
1220	60	0	960	3,60E-06	3,80
1220	60	20	1050	2,38E-03	2,78
1220	60	40	1125	6,99E-02	2,32
1220	60	60	1175	2,64E-01	2,26
1220	60	80	1215	1,00E+02	0,00
1220	60	100			
1220	60	120			
1220	60	140			
1220	60	160			
1220	60	180			
1220	60	200			
1220	100	0	960	3,60E-06	3,80
1220	100	20	1100	4,00E-02	2,35
1220	100	40	1175	2,69E-02	2,81
1220	100	60			
1220	100	80			
1220	100	100			
1220	100	120			

Table 1: Parameters of the H-V-S curves used in the calculations

2.4 Reservoir Operation Rules

A few simple rules define the reservoir operation in the Reservoir Routing spread-sheet, as shown in the lowest, left area of the calculation outputs (figure 1).

FSL: it stands for Full Supply Level. It is the normal water surface level to be kept in the reservoir except if the discharge capacity at that level is not enough to evacuate the entering flood. If the reservoir level is lower than FSL the program “stores” water to reach it. If the reservoir level is higher than FSL, as soon as the outflow capacity is larger than the inflow, the program will bring the reservoir level back to FSL and will remain at that level.

Ho: it is the initial water surface elevation. It may differ from FSL. If it is lower than FSL the program will “store” water to raise the reservoir level up to FSL. It is also called “waiting level” as it may be planned to be reached prior to the flood season, in order to maximize storage capacity increasing the protection against flooding of areas downstream of the reservoir.

err: it represents the tolerance in precision of the calculation. When rapid changes occur (for instance gate opening or closure) calculation instabilities (uncontrolled discharge or level fluctuations) may occur. This parameter gives the operator the possibility of “erasing” those fluctuations.

Q_{MAX}: it allows stopping water releases at a given discharge. The goal of this rule is to avoid excessive flooding downstream areas. As a counter part of this measure the reservoir levels will continue increasing. This possibility is not used in reservoir routing calculations for ROGUN H.P.P.

Q_T: it adds a constant water discharge as could be the releases from turbines. This possibility is not used in reservoir routing calculations for ROGUN H.P.P.

2.5 Spillways

Flood evacuation organs are characterized by the following equation:

$$Q = m\,.\sqrt{2g}\,.N\,.b\,.(H - H_0)^a$$ Eq. (2)

Where:

Q : spillway discharge capacity

m : discharge coefficient

N : number of bays of surface spillways or number of tunnels

b : width of bays or area of the governing section in tunnels

$H;\ H_0$: reservoir level; spillway crest elevation (or sill of the governing section in tunnels)

a : power of the discharge equation: a = 1.5 in surface spillway and a = 0.5 in tunnels.

The discharge coefficient of surface spillways “$m_{0\text{-}DEV}$” is to be given for the design head.

The design head “h_D” is to be declared as the discharge capacity is sensitive to the difference in head with respect to the design head. The discharge coefficient for heads over the spillway crest other than the design head are corrected with the equation:

$$r1 = 0.78 + 0.22\ (h / h_D)^{0.63} \qquad \text{Eq. (3)}$$

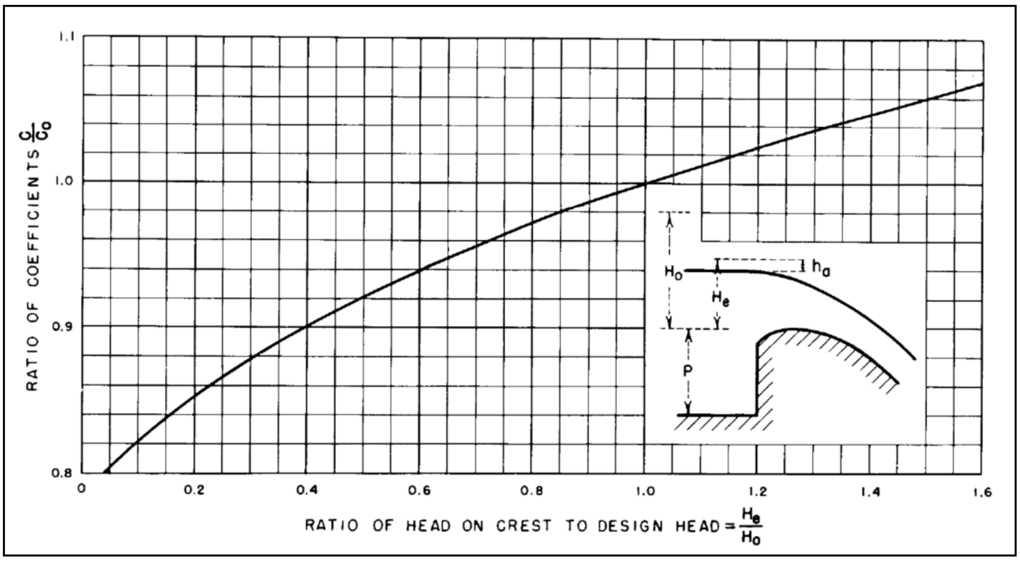


Figure 4: Adjustment of the discharge coefficient for spillway operation

with heads other than the design head (USBRs Design of Small Dams)

When the discharge curve of a given organ (spillway or tunnel) is known, the discharge coefficient “$m_{0\text{-}DEV}$” or “m_{ORIF}” may be obtained (after having introduced the geometrical parameters) by adjusting the coefficient until the right discharge (as in the discharge curve) is obtained in the line “Q_{FSL}” of the calculation output (figure 1).

2.6 Results

Results of the reservoir routing calculation are shown graphically (timely evolution of inflow, outflow and water surface level). The maximum discharge and water surface level are given in the lower, right area of the printout.

* * *

Annex 3: Nurek surface spillway results with flow increase

In the present annex, we present the results of calculation of velocities, water depth and cavitation index for the tunnel after the surface spillway at Nurek.

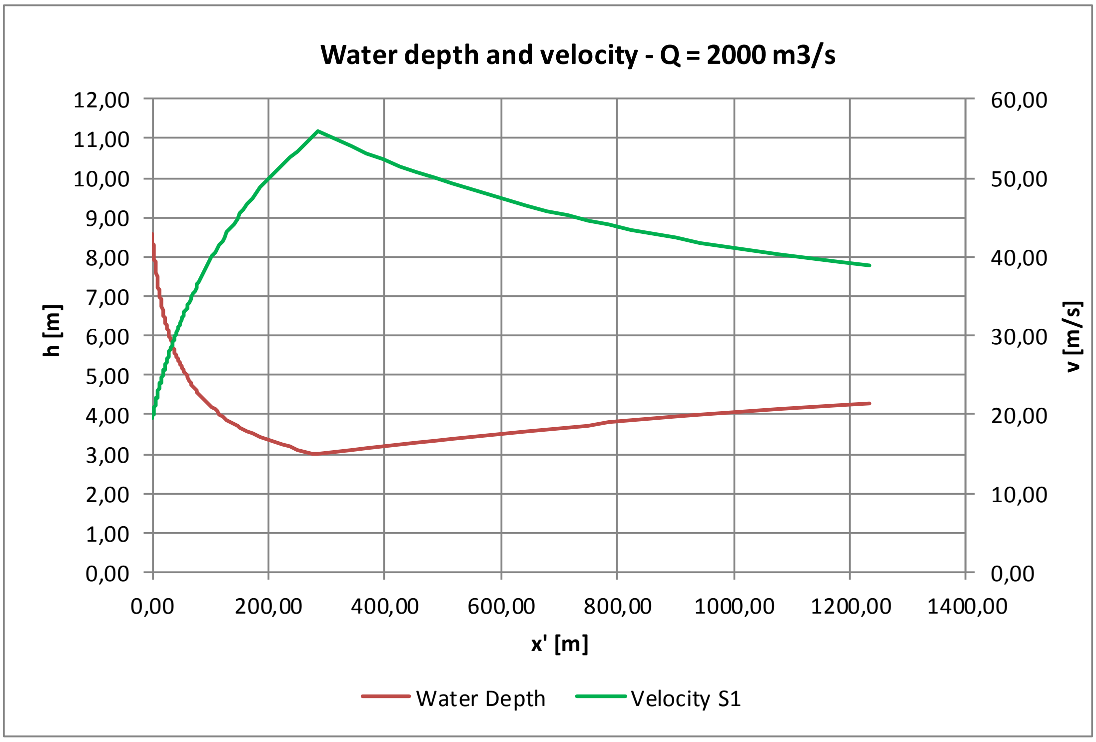
The calculations were done based on drawings of the tunnel after surface spillway of Nurek sent by Barki Tojik to the Consultant on August 20th 2013.

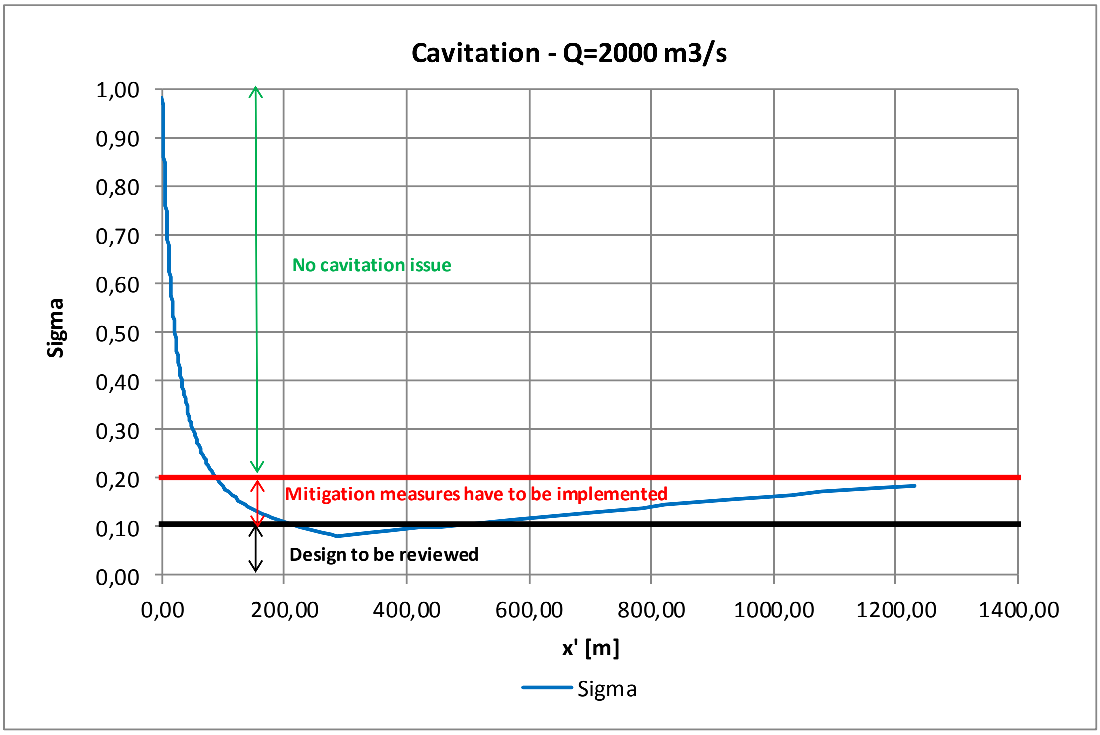
The calculations have been performed for 3 different flows: 2 000 m3/s, 2 400 m3/s; 2 800 m3/s.

For each flow, one graph presents the velocity and the water depth all along the tunnel, and one graph presents the cavitation index.

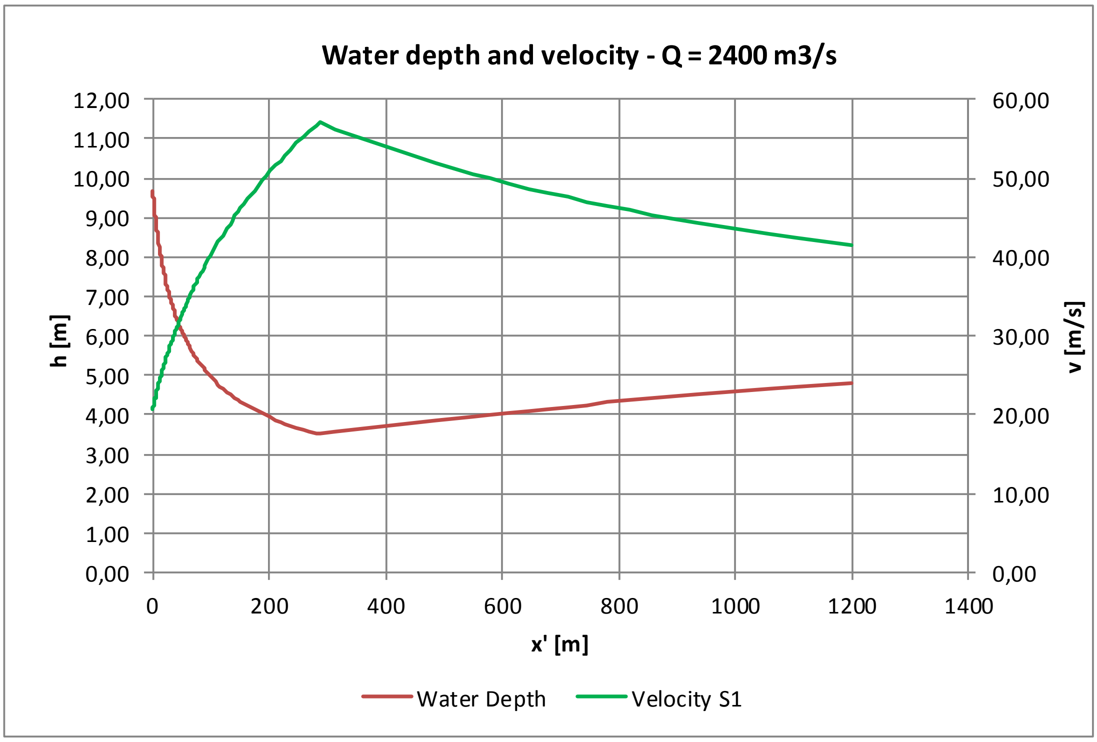
For the cavitation index, it is considered that, for values higher than 0,2, there is no risk of cavitation, for a value between 0,1 and 0,2, mitigation measures must be implement in order to avoid damages linked to cavitation. For values smaller than 0.1, it is considered that the design must be reviewed.

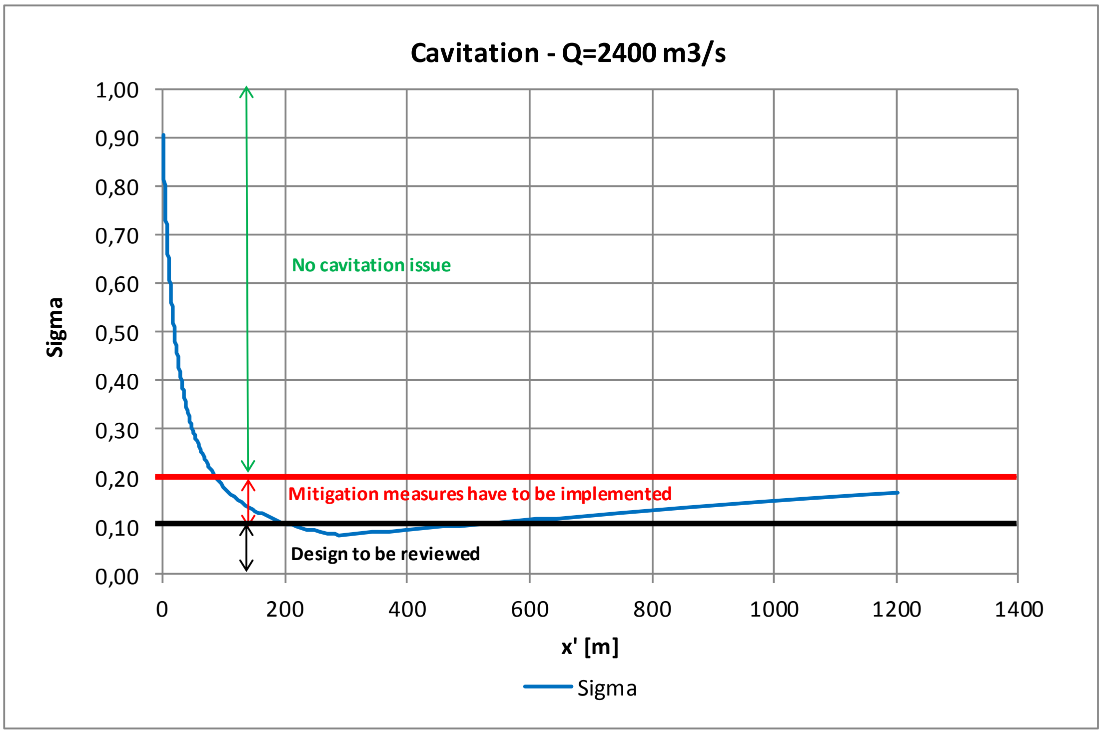
Q= 2000 m3/s


Water depth and velocity - Q = 2000 m3/s
h [m]
v [m/s]
x' [m]
Water Depth
Velocity S1

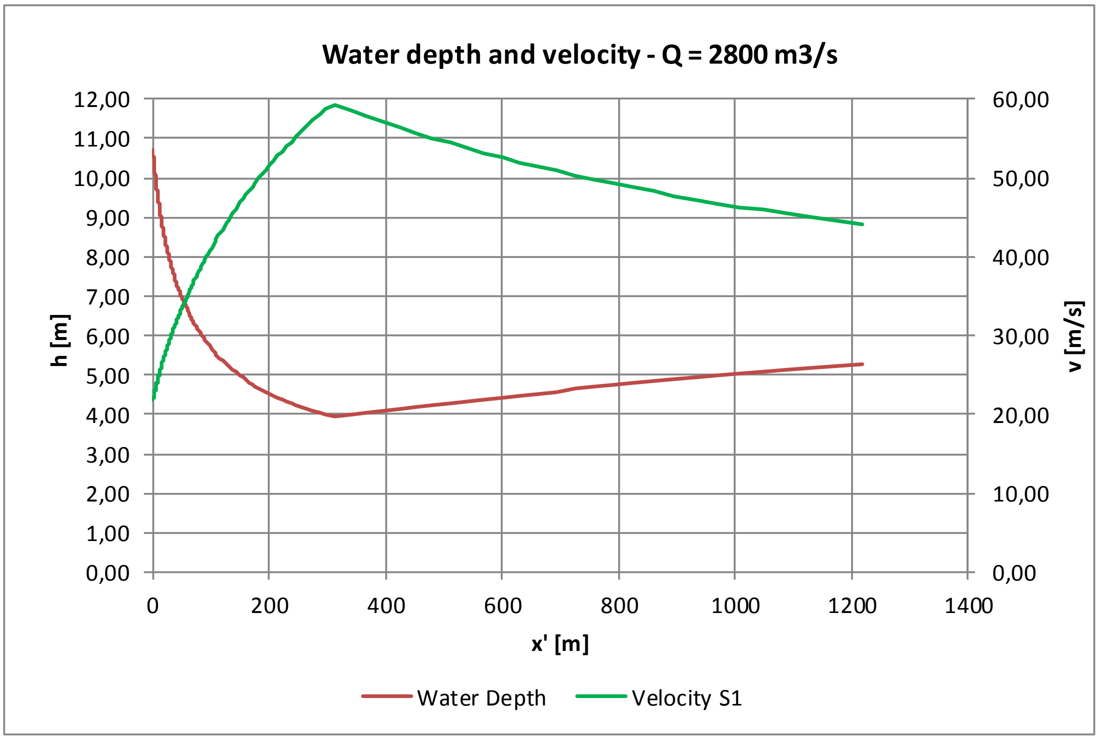

Cavitation - Q=2000 m3/s
Sigma
No cavitation issue
Mitigation measures have to be implemented
Design to be reviewed
x' [m]
Sigma

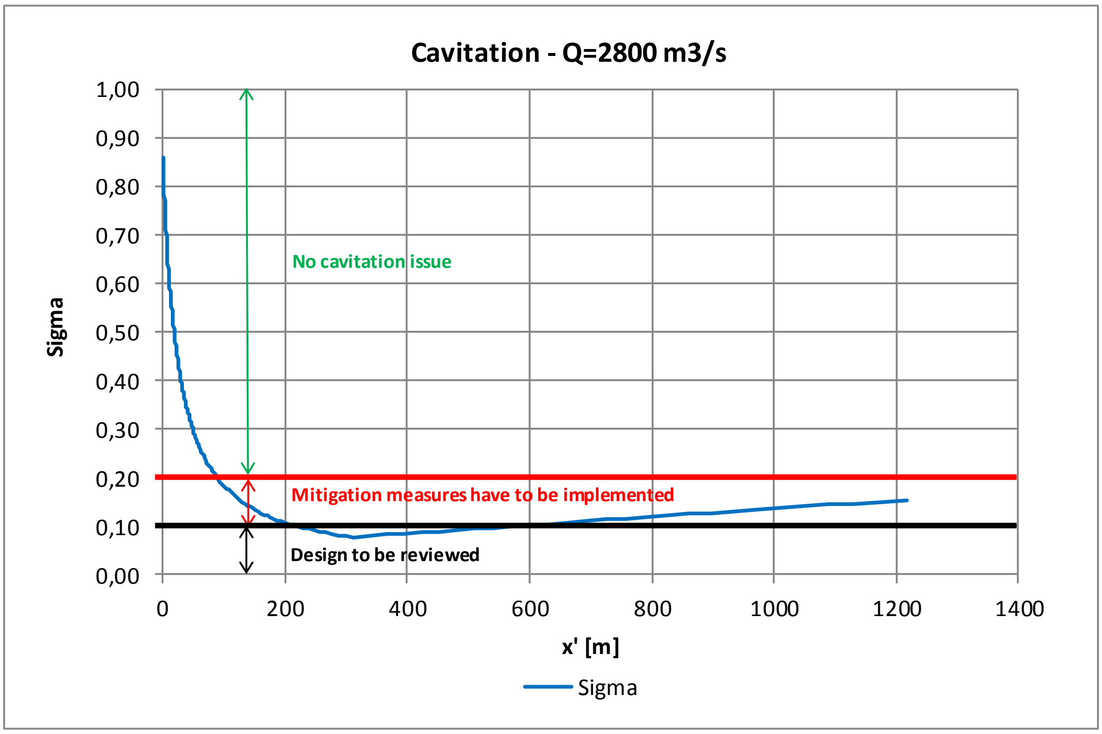
Q= 2400 m3/s


Water depth and velocity - Q = 2400 m3/s
h [m]
v [m/s]
x' [m]
Water Depth
Velocity S1


Cavitation - Q=2400 m3/s
Sigma
No cavitation issue
Mitigation measures have to be implemented
Design to be reviewed
x' [m]
Sigma

Q= 2800 m3/s


Water depth and velocity - Q = 2800 m3/s
h [m]
v [m/s]
x' [m]
Water Depth
Velocity S1


Cavitation - Q=2800 m3/s
Sigma
No cavitation issue
Mitigation measures have to be implemented
Design to be reviewed
x' [m]
Sigma