TECHNO-ECONOMIC ASSESSMENT STUDY FOR ROGUN HYDROELECTRIC CONSTRUCTION PROJECT

PHASE II: PROJECT DEFINITION OPTIONS

Volume 3: Engineering and Design

Chapter 3: Alternatives design

Appendix 3 - Note on Flood Management during Construction

August 2014

Report No. P.002378 RP 41 rev. C

D	13/08/2014	Final – August 2014	O. Clave	L. Bouzat	L. Bouzat
C	31/03/2014	Final	O.Clave	N.Sans	N.Sans
B	15/09/2013	Comments of WB, PoE and GoT	O.Clave	A.Lara	N.Sans
A	02/04/2013	First Emission	O.Clave	A.Lara	N.Sans
Revision	**Date**	**Subject of revision**	**Drafted**	**Checked**	**Approved**

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1 OBJECTIVES AND CONTEXT

This note deals with the flood management during construction of Rogun dam.

First a series of design criteria recommended by the Consultant are presented. Then, the scheme for flood diversion during construction proposed by HPI is assessed at the light of those criteria.

And finally, the modifications proposed by the Consultant are presented and a full diversion scheme for the three alternatives is detailed.

2 DESIGN CRITERIA

2.1 Hydrological data

The probabilistic analysis of floods in the Vakhsh River at the Rogun H.P.P. (Report on Hydrology N°P.002378 RP-07/C, January 2013, TEAS Consultant) furnished the following results in terms of daily discharge and peak discharge (named “adopted” in the table) for different periods of return: see Table 2.1.

T	Results	
	Adopted	Daily
2	2 360	2 250
5	2 780	2 650
10	3 070	2 930
20	3 360	3 200
50	3 750	3 580
100	4 030	3 840
200	4 310	4 110
500	4 660	4 440
1 000	4 950	4 720
2 000	5 260	5 010
5 000	5 640	5 380
10 000	5 970	5 690

Table 2.1 : Probability of Floods

Regarding the Vakhsh river rating curves on dam site, several data are available in various documents:

- In report 1861-2-II-2 (2009), 3 curves (mean, min and max) are defined for two river sections: one close to diversion tunnels (“DT”) inlet, and another one close to diversion tunnel outlet.

- In report 1861-2-II-1 (2009), measures of water level and discharge during the year 2009 are available for 4 river cross sections: one downstream of final dam and three others on dam site (see Figure 2.1 for locations).

- In report 1861-03-001, same measurements during the year 2010 are available.

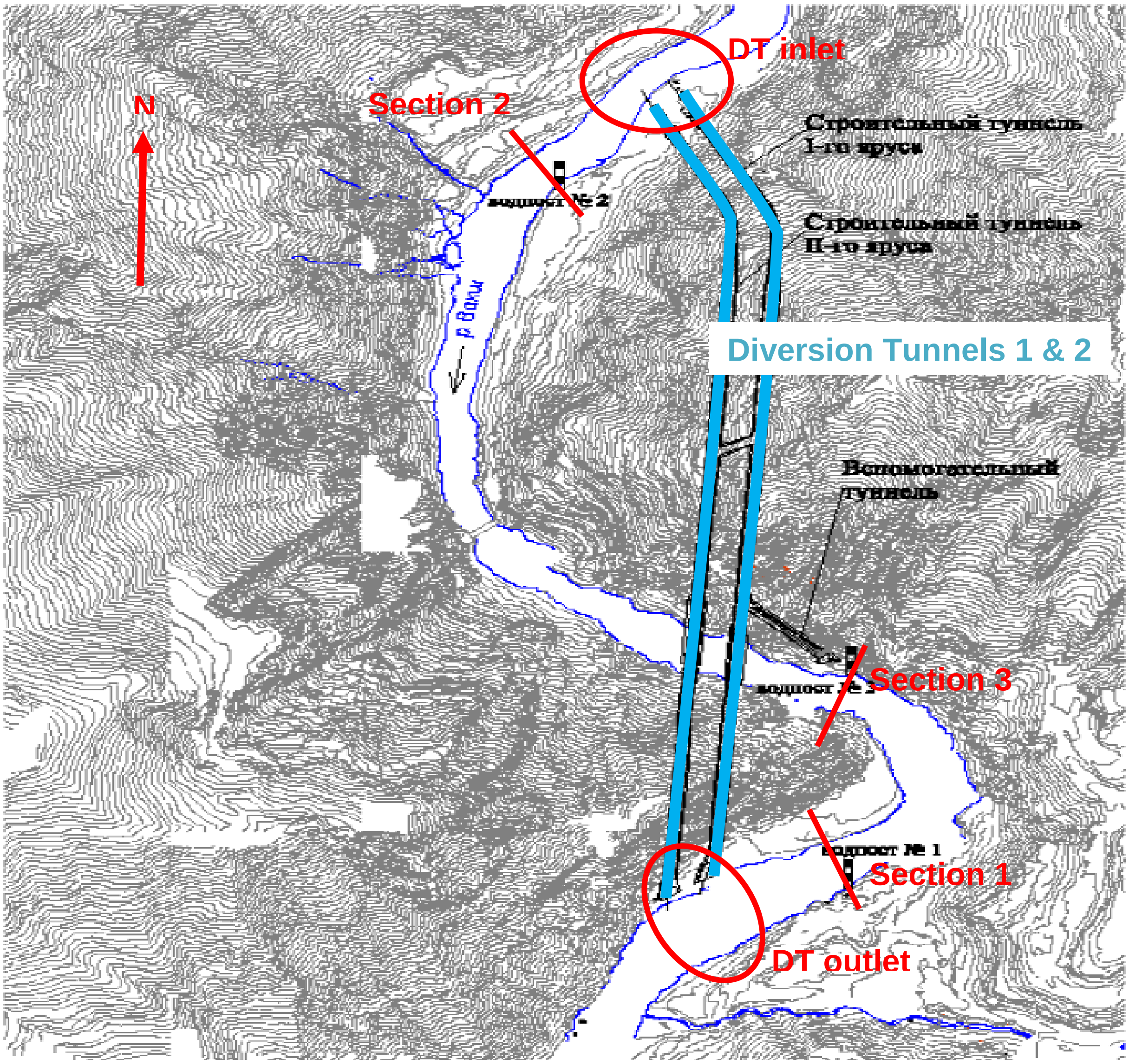


Figure 2.1 : Rating curve - Location of measurements

The following graph shows 2009 and 2010 measurements for the three sections located on the dam site. Their interpolated curves are also plotted.

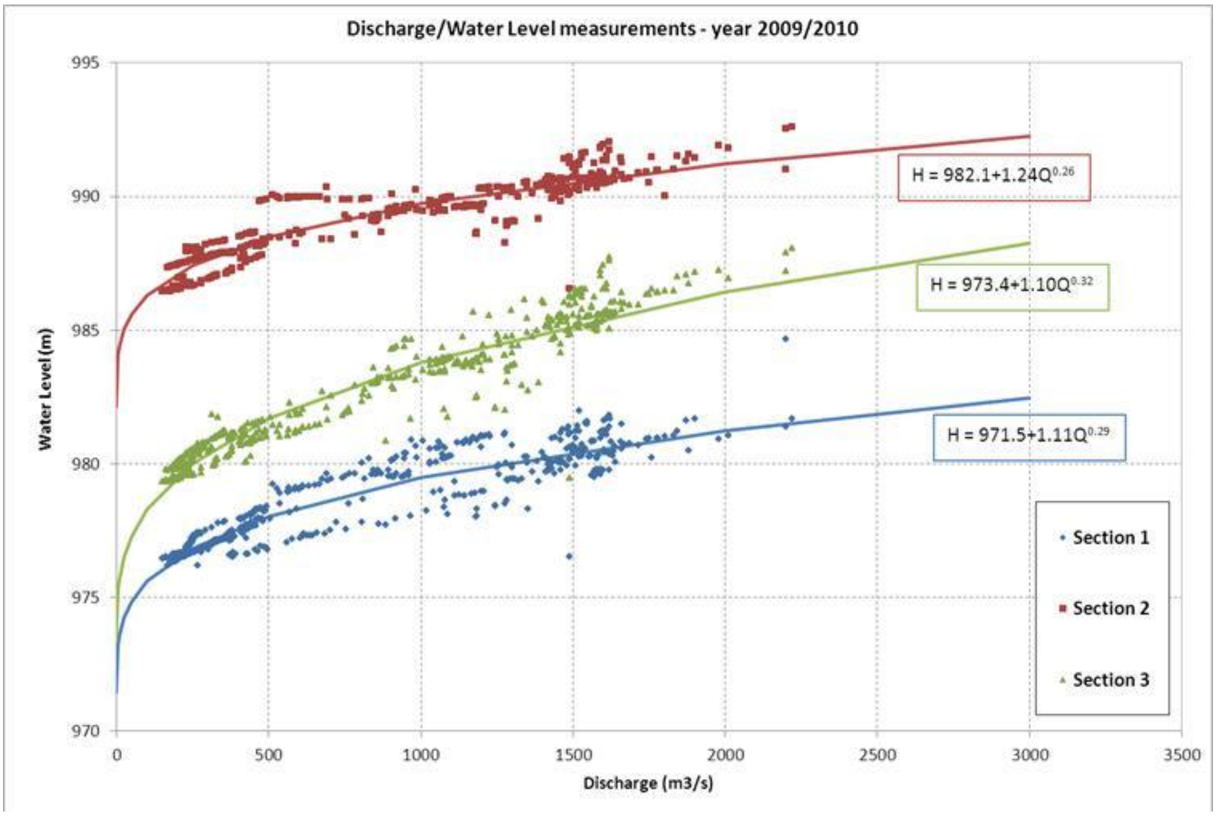


Figure 2.2: Rating curve measurements - year 2009/2010

Interpolated curves have been mathematically calculated to approximate all points at best according to the same law:

$H = H_0 + aQ^b$ where H_0 is the bottom elevation and H the water level elevation.

It appears that the section 2 rating curve is flatter than the two other for high discharges. This means that there is a particularity in the river (contraction or steepening of the bottom) that tends to modify the “regular” rating curve.

The downstream cofferdams crest elevation will be determined based on those rating curves.

2.2 Construction floods

In order to determine the design flood for each critical stage of the dam construction, the following procedure is adopted:

2.2.1 *Acceptable probability of exceedance*

A range of acceptable probabilities of exceedance of a given flood (characterized by its mean return period) within a given period of exposure of the site works is to be proposed.

A probability of exceedance is expressed, for instance, as “1/100” (read: “one in hundred” or “one case out of hundred”) or “1/50”, etc. This ratio expresses the willingness of the Owner or of the Designer to accept higher (1/50) or smaller (1/100) probability of exceedances.

That probability of exceedance can be measured as: $P_{OCC} = (1 - (1 - \frac{1}{TMR})^{TE}$, where:

"TE" represents the Time of Exposure (i.e., number of years during which the construction site may be flooded),

"TMR" represents the mean period of return of a given flood.

For small values of TE / TMR, the value of R is close to the value of this ratio.

The accepted (or tolerable) probability of exceedance should be inversely proportional to the gravity or importance of the consequences if the one case out of hundred (for instance) occurs.

It is commonly said that the implicitly accepted probability of exceedance in dam design is 1/100 if the life span of the project is of 100 years and the period of return for the design of the protection facilities is 10,000 years.

In order to launch the analysis, probabilities of exceedance ranging from 1/10 to 1/1000 will be evaluated.

2.2.2 Return period of the design floods

For a given probability of exceedance and a given period of exposure of the works, the design period of return may be deduced with the help of the equation given above.

But, as mentioned also above, for small values of the probability of exceedance $\approx \frac{TE}{TMR}$. The return period of the design flood will be then calculated as: $\approx \frac{TE}{R}$. If the time of exposure is 8 years and the accepted probability of exceedance is 1/100, the mean period of return of the design flood will be 800 years.

2.2.3 Peak of the design flood

For a given design period of return (as determined in the former paragraph) the peak of the flood is determined with the help of data given in Table 2.1

The periods of exposure of the cofferdam (CD), the Stage-1 dam (S1) and the rest of the construction up to the completion of the Main Dam (MD) are indicated in Table 2.3, Table 2.4, and Table 2.5. The matching discharges are also indicated.

Times of construction indicated in those tables correspond to the construction of a given body (for instance S1) plus the time to rise up the downstream body (for instance MD, downstream of S1) up to the same level of the crest of the former. That means that the time of exposure of the upstream body (for instance S1) lasts up to the moment when the downstream body (for instance MD) reaches the crest elevation of the former (S1) and becomes the true controller of the safety of the construction area against flooding.

The volumes of the reservoirs created at any moment are also shown in the next table in order to quantify the level of consequences if floods occur and the dam fails. It is to be noted that those volumes of water could be stored in the Nurek reservoir by just increasing the reservoir level by 2 m (CD) and 4-6 m (S1) depending on the dam alternative, but could by no means be stored for MD.

	Cofferdam	Stage 1	Main dam
FSL=1290 masl	V = 190 hm^3 ΔZ(Nurek)= 2 m	Z_{crest} = 1110 masl V = 610 hm^3 ΔZ (Nurek)= 6.3 m	V = 13 300 hm^3 ΔZ (Nurek)= out of proportion
FSL=1255 masl	V = 190 hm^3 ΔZ (Nurek)= 2 m	Z_{crest} = 1090 masl V = 480 hm^3 ΔZ (Nurek)= 5 m	V = 8 490 hm^3 ΔZ (Nurek)= out of proportion
FSL=1220 masl	V = 190 hm^3 ΔZ (Nurek)= 2 m	Z_{crest} = 1075 masl V = 360 hm^3 ΔZ (Nurek)= 3.7 m	V = 5 210 hm^3 ΔZ (Nurek)= out of proportion

Table 2.2 : Reservoir volume and consequences

These facts explain why the ranges proposed for the study (ellipses in tables) consider higher probabilities of exceedance as the dam height increases.

Design Discharge for Flood Management during Construction

as a function of Probability of Failure and Period of Exposure

Probability of exceedance	Period of exposure – FSL= 1290 masl					
	CD: 1 years		S1: 4 years		MD: 6 years	
	MPR (yr)	Daily discharge	MPR (yr)	Daily discharge	MPR (yr)	Daily discharge
1/500	500	4 500	2000	5 000	3000	5 200
1/200	200	4 100	800	4 700	1200	4 800
1/100	100	3 850	400	4 400	600	4 550
1/50	50	3 550	200	4 100	300	4 300
1/20	20	3 200	80	3 750	120	3 900
1/10	10	2 900	40	3 450	60	3 600

MPR : Mean Period of Return.
CD : cofferdam; S1 : Stage-1 dam; MD: remaining of the main dam

Table 2.3 : Design Floods. Ranges to be explored (FSL=1290 masl)

Design Discharge for Flood Management during Construction
as a function of Probability of Failure and Period of Exposure

Probability of exceedance	Period of exposure – FSL= 1255 masl					
	CD: 1 years		S1: 3 years		MD: 5 years	
	MPR (yr)	Daily discharge	MPR (yr)	Daily discharge	MPR (yr)	Daily discharge
1/500	500	4 500	1500	4 900	2500	5 100
1/200	200	4 100	600	4 550	1000	4 750
1/100	100	3 850	300	4 300	500	4 500
1/50	50	3 550	150	4 000	250	4 200
1/20	20	3 200	60	3 600	100	3 850
1/10	10	2 900	30	3 350	50	3 550

MPR : Mean Period of Return.
CD : cofferdam; S1 : Stage-1 dam; MD: remaining of the main dam

Table 2.4 : Design Floods. Ranges to be explored (FSL=1255 masl)

Design Discharge for Flood Management during Construction
as a function of Probability of Failure and Period of Exposure

Probability of exceedance	Period of exposure – FSL= 1220 masl					
	CD: 1 years		S1: 2 years		MD: 3 years	
	MPR (yr)	Daily discharge	MPR (yr)	Daily discharge	MPR (yr)	Daily discharge
1/500	500	4 500	1000	4 750	1500	4 900
1/200	200	4 100	400	4 400	600	4 550
1/100	100	3 850	200	4 100	300	4 300
1/50	50	3 550	100	3 850	150	4 000
1/20	20	3 200	40	3 450	60	3 600
1/10	10	2 900	20	3 200	30	3 350

MPR : Mean Period of Return.
CD : cofferdam; S1 : Stage-1 dam; MD: remaining of the main dam

Table 2.5 : Design Floods. Ranges to be explored (FSL=1220 masl)

In the following analysis, these ranges of protection level will be studied to assess the sensitivity of the structures design (and their costs) with respect to the protection level.

The conclusions on construction flood considered for each construction phase will be presented in §6 after having presented the complete analysis.

The hydrograph taken into account is the one established in RP07 for the PMF and 10000 years return period flood and proportionally reduced for smaller floods.

2.3 Flood management

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

For the Stage 1 phase and the final dam completion phase, the flood attenuation thanks to the reservoir routing is taken into account. The reservoir level is assumed to be 10 m lower than the dam and core crest before the flood comes; and the reservoir routing is then considered in between this 10 m minus an adequate freeboard.

2.4 Structural criteria

Flood diversion during construction is ensured by tunnels. Design criteria for those tunnels are presented in this paragraph.

2.4.1 Existing tunnels (DT1 and DT2)

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

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

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

The hydraulic tests The Consortium representatives had the occasion to witness in Moscow, for flows up to 1,600 m^3/s / tunnel, confirmed that if the downstream original elevation is restored, the water flows in supercritical conditions and no hydraulic jump occurs.

Therefore, the deposit of material proceeding from the cofferdam collapse and from the mudflow of Obi Shur creek should be removed before the river diversion.

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

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

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

As they will probably need heavy rehabilitation works, it is already considered in this analysis that they will be reduced by 30 cm along their whole perimeter as a provision for rehabilitation works.

Those tunnels shall work under a maximum head of 120 m.

2.4.2 New tunnels

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

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

The next figure presents data extracted from the book Design of Hydraulic gates written by P.Erbisti in 2004 and presenting the design head and gate area of existing high pressure gates. The red line is interpolated from the two extreme points: Tarbela (largest gates) and Beaver (higher design head). The dotted line is interpolated from all data.

It shows that a design criteria of 120-150 m is reasonable considering the size of the gates needed.

It should be noted that:

- The type of gates considered in the case of Rogun is segment gates; on the graph it can be seen that this type of gate never exceeds 140 m head.
- The gates showed in this graph are bottom outlets that are used time to time during a short period. In the case of Rogun diversion structures, the gates will be used to control the reservoir level during the whole construction period on a continuous manner.

Further justification for this criteria is provided in a Report written by P. Erbisti and presented in Appendix A.

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

Sensitivity analysis on the maximum water head criteria is made by considering also 150 m instead of 120 m head. Results of this sensitivity analysis are presented in §5.4.

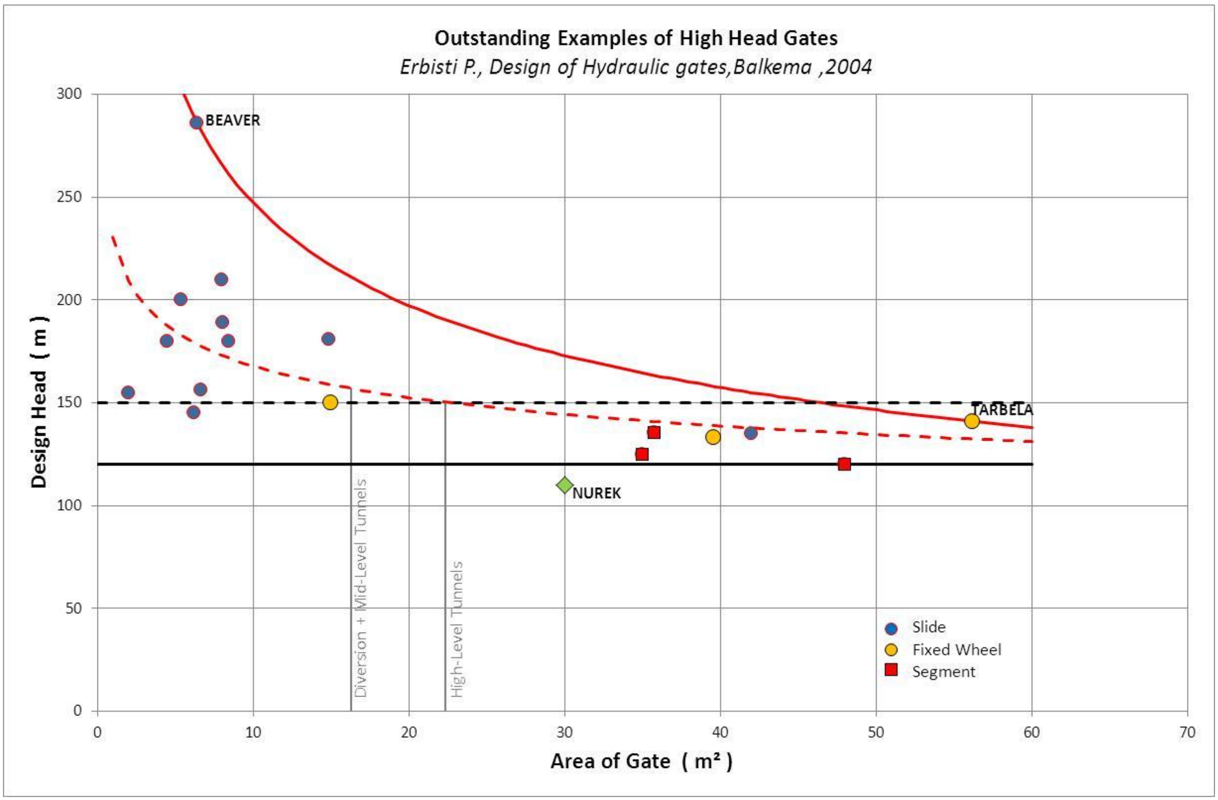


Figure 2.3 : Situation of maximum head criteria among existing exemples

2.5 Ionaksh fault

Co-seismic displacements in Ionaksh fault could be of the order of magnitude of 1 m in case of large earthquake (MCE).

No probability can be associated with this event. But the project should survive in spite of its occurrence: the protecting structure should not collapse. This shall be considered as an extreme scenario.

3 HPI DIVERSION SCHEME

3.1 Description

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

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

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

The next figure presents the location and inlet elevation of the various diversion and spillway structures. The next table presents their main characteristics.

	Type and size	Sill elevation
DT1	Pressured Tunnel, D-shaped,95.55 m²	989.60
DT2	Pressured tunnel, D-shaped,95.55 m²	1001.80
DT3	Pressured tunnel, circular Ø15 m	1035
OP3	Pressured tunnel, circular Ø15 m	1145
RS	Pressured tunnel, circular Ø11 m	1145
OSS	Gated weir, L=40 m	1288

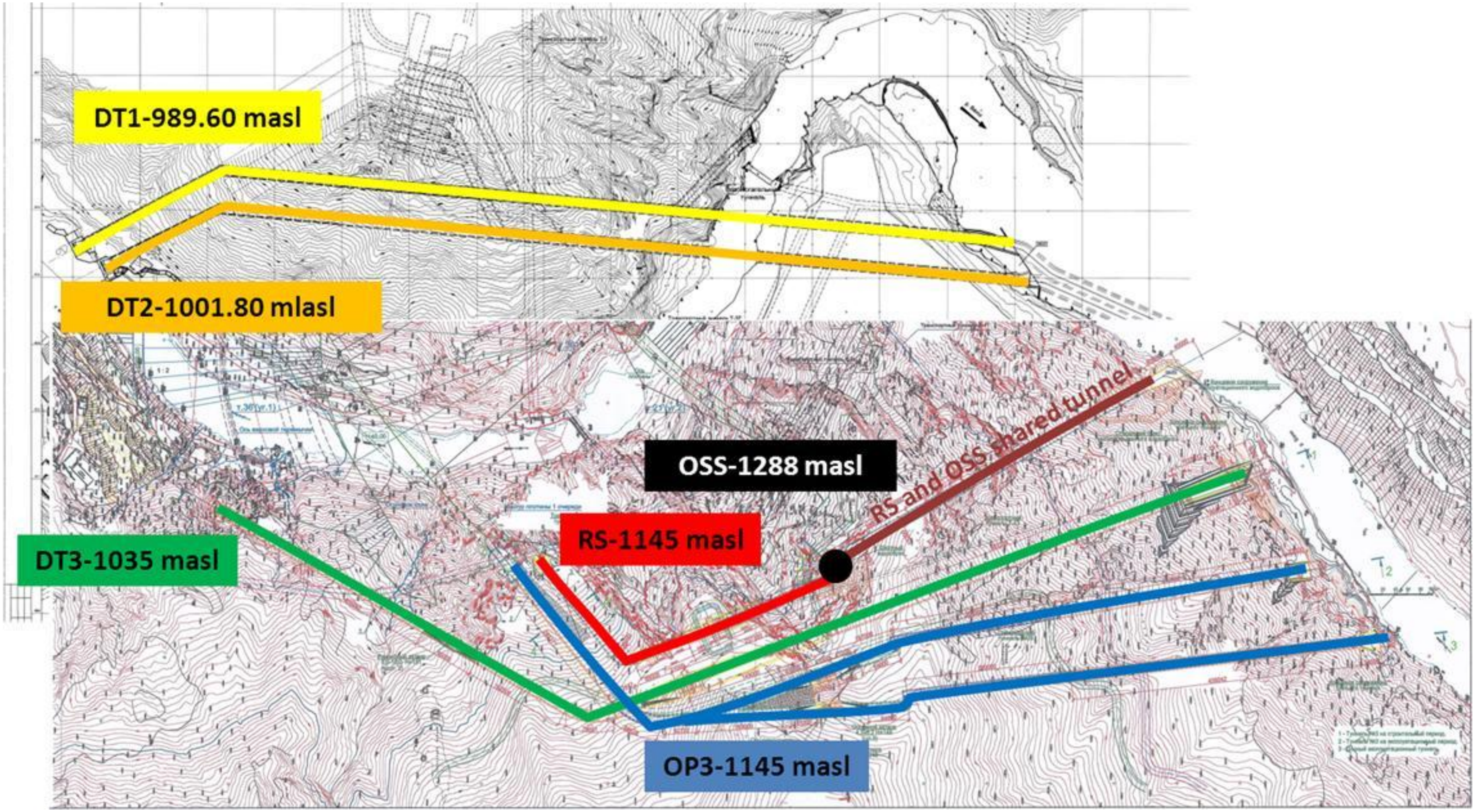


Figure 3.1 : Plan view - Diversion and spillway structures – HPI scheme

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

While the Stage 1 dam is protecting the site, DT1& DT2 and DT3 are diverting the river. At this elevation (water level at 1100 masl), the combined capacity of the three tunnels is 7100 m^3/s.

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

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

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

The Figure 3.2 presents the discharge capacity curve of all diversion tunnels and spillways.

The Figure 3.3 presents the operating range of each diversion tunnel and spillway along a water elevation axis.

The Figure 3.4 presents the HPI diversion scheme along the time axis: it presents the water level along construction period which is supposed to be 10 m below the crest, and the discharge capacity. It is also indicated which structure is operated along the construction period.

3.2 Assessment

On cofferdam phase

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

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

On Stage 1 phase

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

Between the Stage1 and elevation 1185 masl

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

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

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

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

Above elevation 1185 masl

Above 1185 masl, the final spillways ensure the river diversion. At this elevation, OP3 and RS are able to discharge 4650 m^3/s, ie a lower capacity than in the previous phase. When switching from DT2+DT3+RS to OP3+RS, the protection level reduces (see Figure 3.4).

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

Conclusion

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

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

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

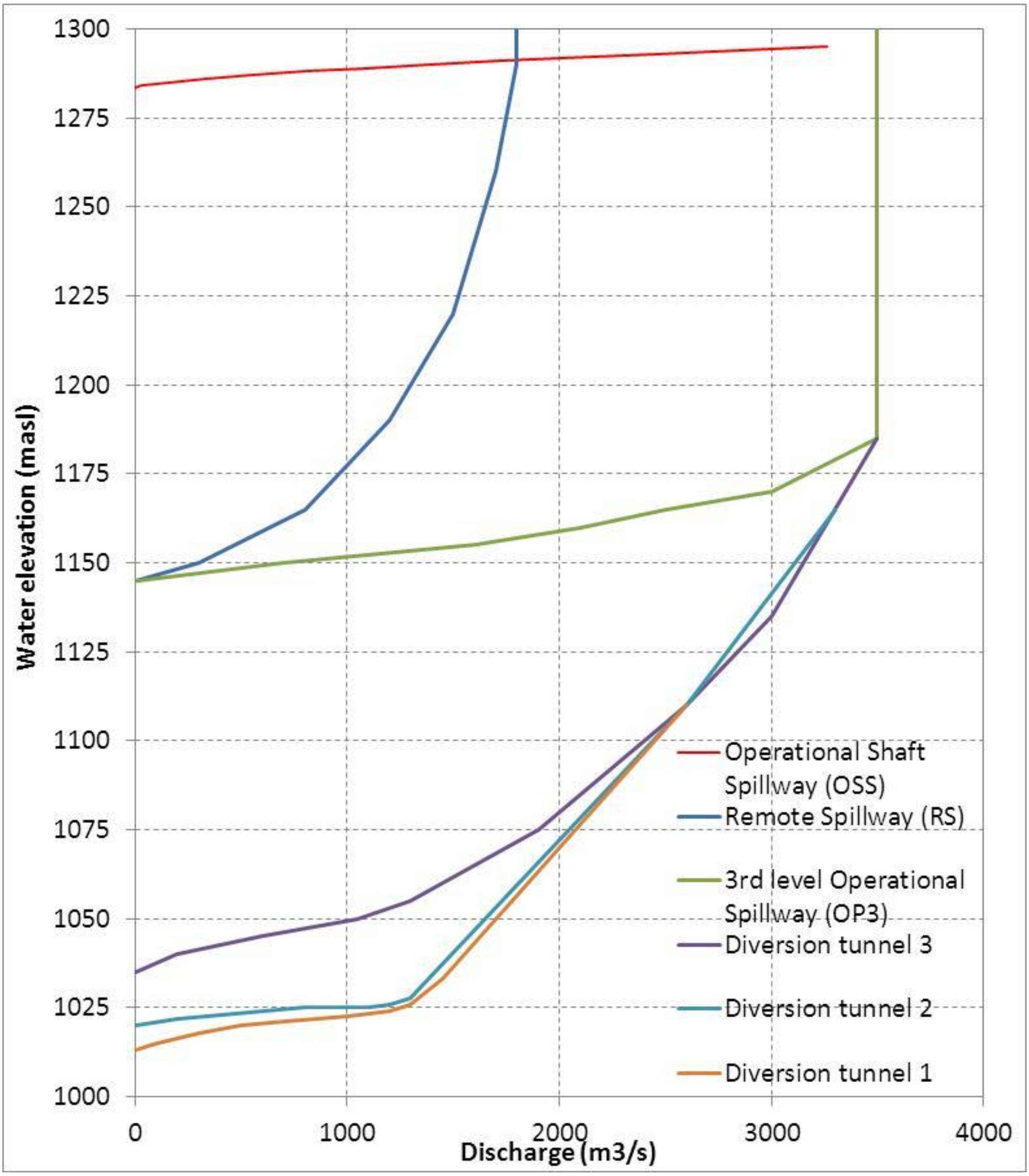


Figure 3.2 : Discharge capacity versus elevation - HPI discharge tunnels

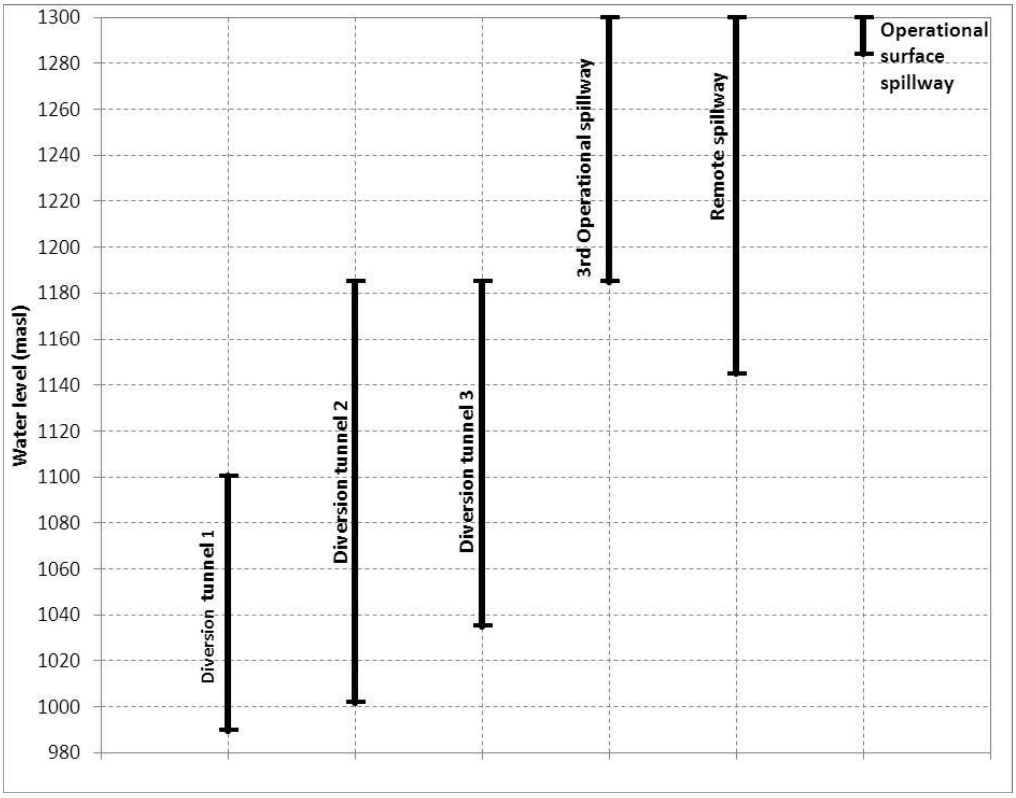


Figure 3.3 : Operation range of discharge tunnel - HPI

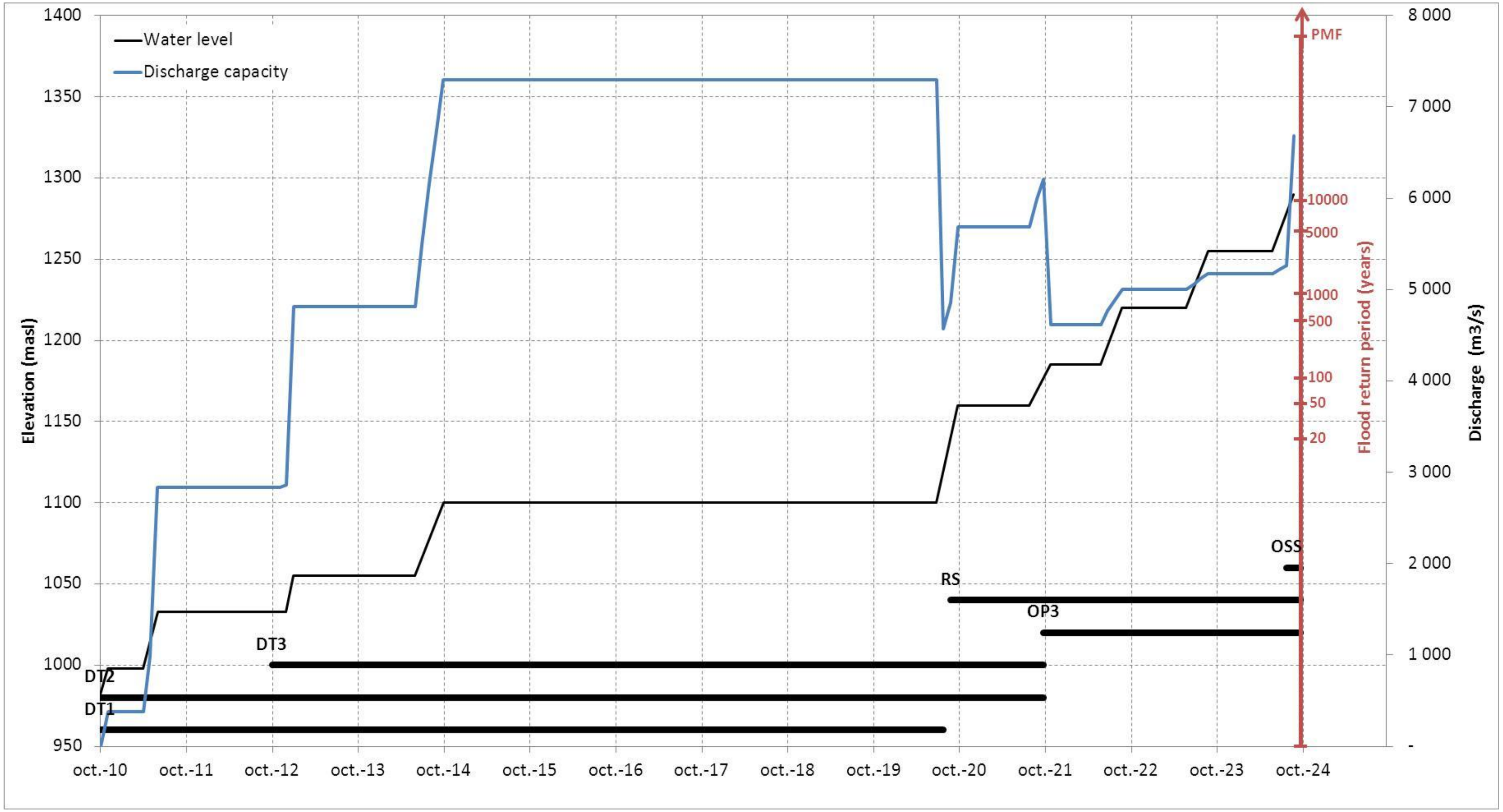


Figure 3.4 : HPI Diversion scheme

4 DESCRIPTION OF PROPOSED DIVERSION STRUCTURES

This paragraph aims at presenting the geometrical characteristics of the diversion structures and their discharge capacity curve, as well as characteristics of cofferdams proposed by the Consultant.

The various diversion structures considered are:

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

The plan view of these proposed structures is presented in the next figure.

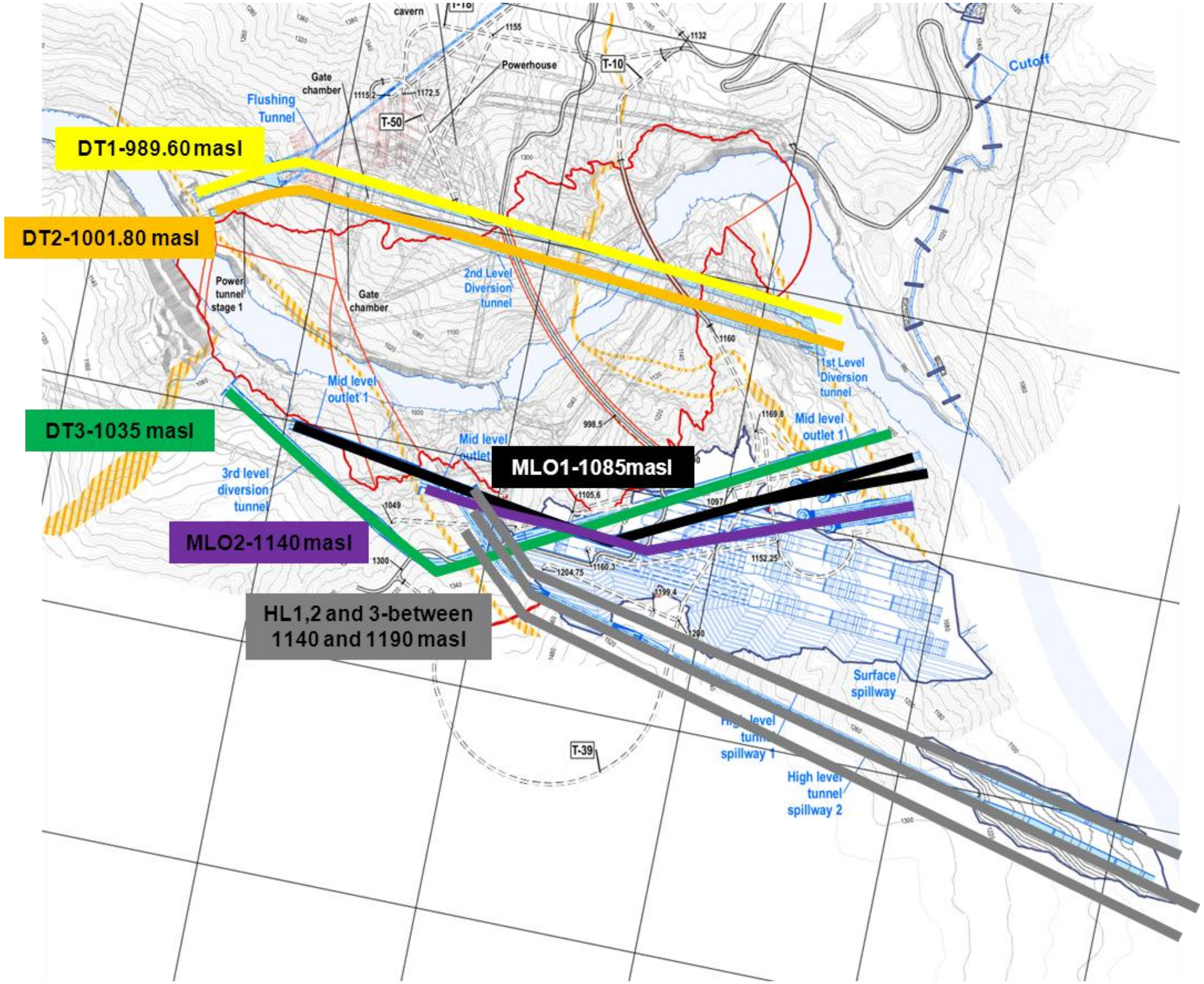


Figure 4.1 : Plan view - Diversion structures proposed

4.1 Diversion tunnel 1

This existing tunnel pressure stretch is D shaped, with a useful section of 96.55 m². This value includes the reduction of 30 cm along its whole perimeter as a provision for rehabilitation works.

The Diversion Tunnel 1 is located on the left bank at the bottom of the valley. It is the lowest tunnel used for river diversion.

Two different intakes are used in this tunnel: first a low intake then a high intake. The low intake is a classical tunnel entrance with a sill at elevation 989.60 masl. When this intake is closed with stoplogs, the water is allowed to flow over the stoplogs and enter into the tunnel. The high intake is then composed of a free weir at elevation 1020 masl that discharge into the same tunnel as the low intake. When water elevation is even higher the tunnel works as pressured tunnel.

The low intake is used at the very beginning of the works and river diversion, when the river water level is low. The high intake is planned for construction when the reservoir level rises and to avoid the sediment transport in the tunnel.

4.2 Diversion tunnel 2

This existing tunnel pressure stretch is D shaped, with a useful section of 96.55 m². This value includes the reduction of 30 cm along its whole perimeter as a provision for rehabilitation works.

The Diversion Tunnel of 2nd level is located on the left bank at the bottom of the valley.

As for the DT1, two intakes are used in this tunnel: first a low intake then a high intake. The low intake is a classical tunnel entrance with a sill at elevation 1001.80 masl. When the front of the intake is plugged with concrete, it creates a vertical wall in front of the tunnel entrance. The wall acts as a weir that the water is overflowing; then the flow is discharging in the tunnel. The high intake is then composed of a free weir at elevation 1020 masl that discharges into the same tunnel as the low intake. When water elevation is even higher the tunnel works as pressure tunnel.

As for the DT1, the low intake is used at the very beginning of the works and river diversion, when the river water level is low. The high intake is planned for construction when the water is higher and in order to avoid the sediment transport in the tunnel.

The discharge capacity of DT1+DT2 is presented in Figure 4.2.

4.3 Diversion tunnel 3

DT3 has been designed by HPI. Its construction began in the second part of 2011 and stopped in June 2012. It is therefore at the moment partly excavated.

It is circular with an inner diameter of 15 m, its sill elevation is 1035 masl.

The next graph presents the discharge capacity curve of DT3 and combined DT1 + DT2.

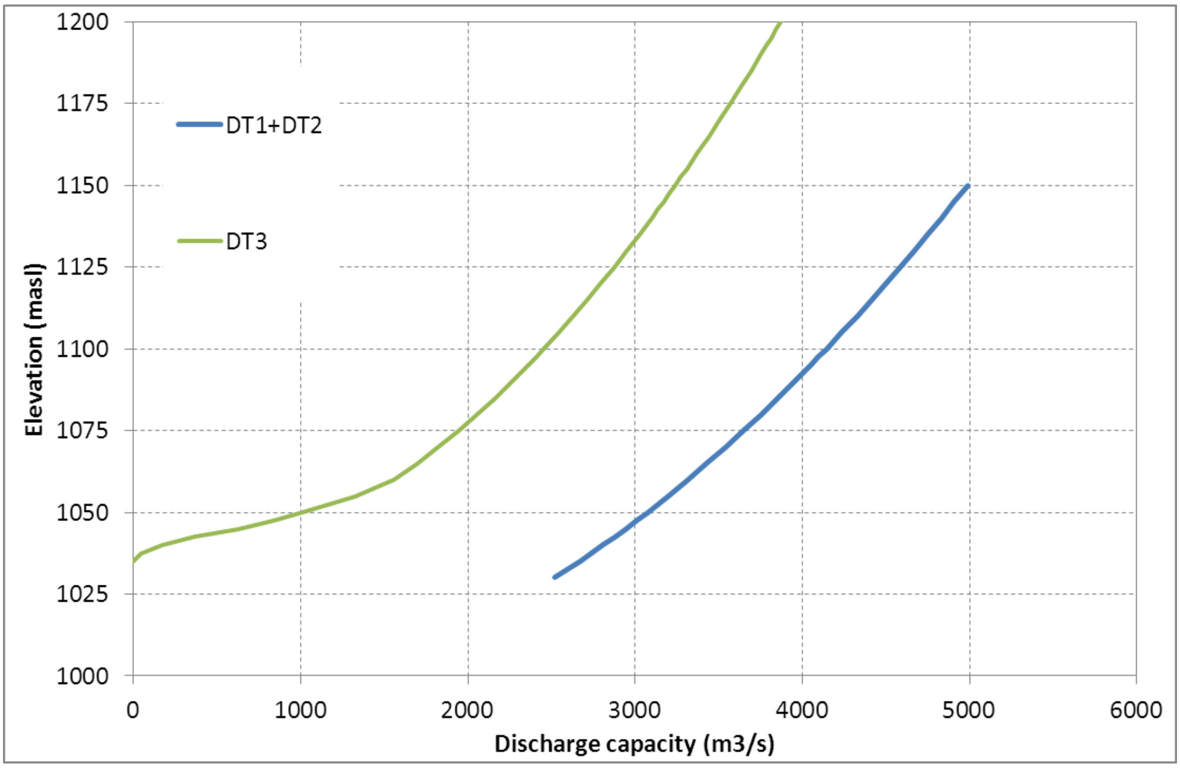


Figure 4.2 : Discharge capacity curve - DT1, DT2 and DT3

4.4 Mid-level outlets 1 and 2

Those tunnels do not exist and are not in HPI design. They are introduced here to ensure the flood diversion when the water level is between the Stage 1 and the operational spillways elevation.

For dam alternatives FSL=1290 masl, MLO1 and MLO2 are necessary, for the lower dam alternatives (FSL=1255 and 1220 masl), only MLO1 is necessary.

They are circular shaped along the pressure stretch with an inner diameter of 15 m.

MLO1 sill elevation is 1085 masl, and MLO2 is settled at 1140 masl.

The next graph presents MLO1 and MLO2 discharge capacity curve.

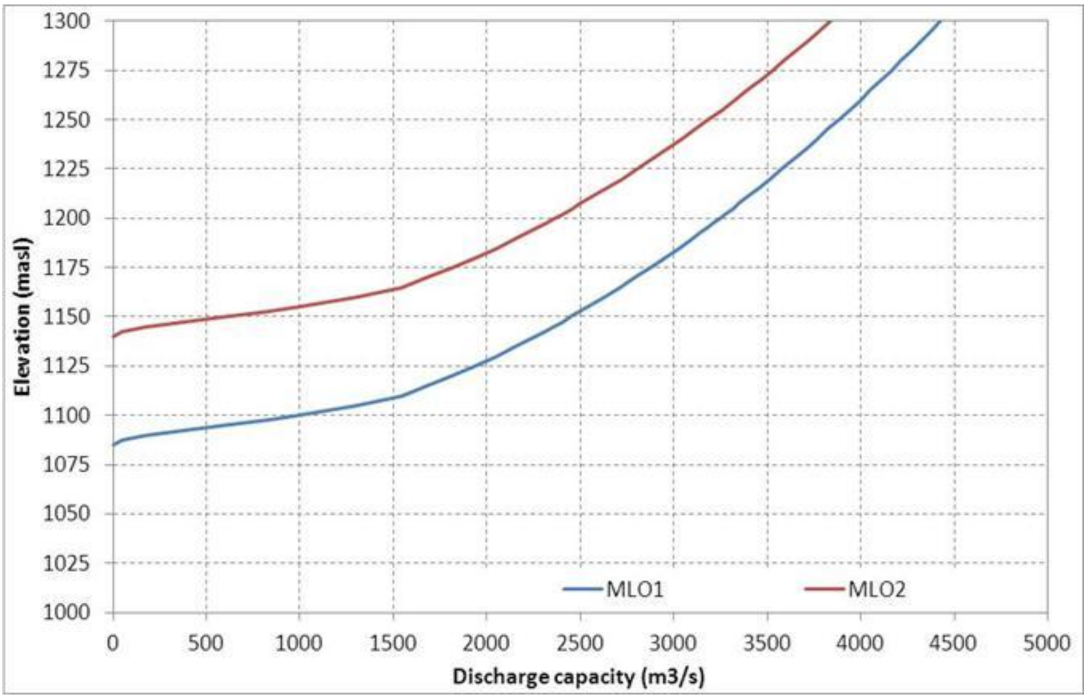


Figure 4.3 : Discharge capacity curve - MLO1 and MLO2

4.5 High level tunnels 1, 2 and 3

Those tunnels do not exist and are not in HPI design. Those tunnels are set at higher elevation and can be also used as operational spillways after the dam completion.

Those tunnels are settled at various elevations depending on the dam alternative.

They are all horse-shoe, with an inner diameter of 10 m.

The number and elevation of high level tunnels depends on the alternative. For diversion during works the necessary high spillways for the various alternatives are as followed:

- FSL =1290 masl: 2 high spillways at elevation 1190 masl;
- FSL = 1255 masl : 2 high spillways at elevation 1165 masl and 1 at elevation 1145 masl;
- FSL = 1220 masl : 1 high spillway at elevation 1140 masl.

The discharge capacity curves of all those tunnels are similar and are presented in the next graph as a unique curve: discharge versus water head.

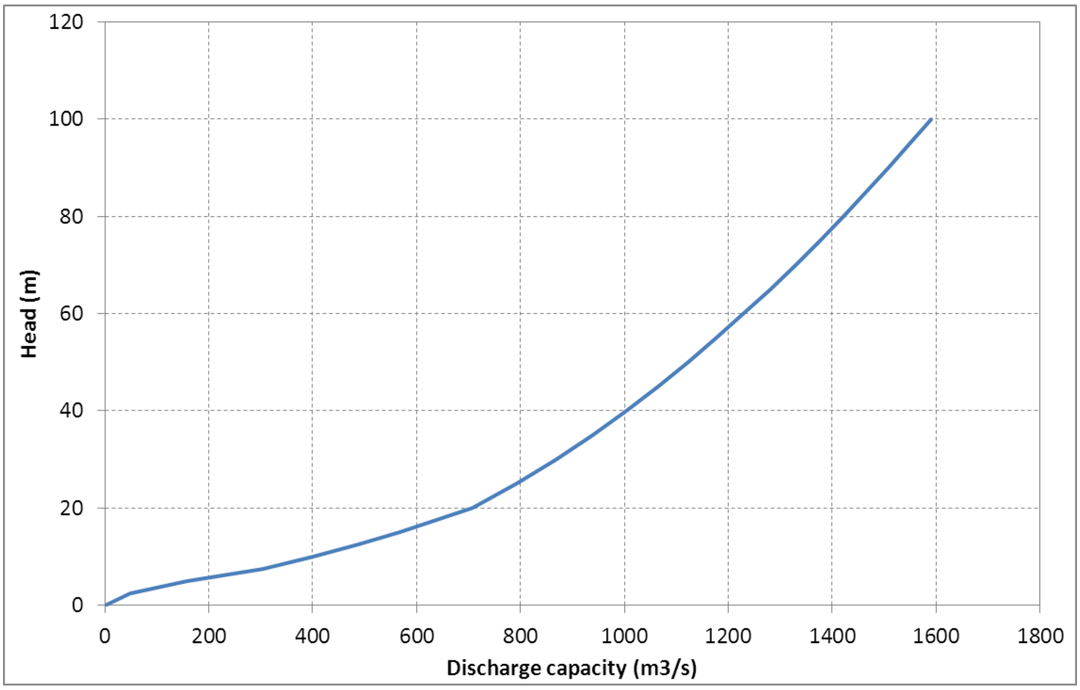


Figure 4.4 : Discharge capacity curve - High level spillways

4.6 Upstream and downstream cofferdams

The upstream cofferdam is part of the final dam. Its crest elevation is defined §6 based on hydraulic calculation results in. Once the Stage 1 dam is raised above the cofferdam, it is used as protection structure. When the final dam reaches Stage 1 crest elevation, it is used as its own protection structure.

There are two downstream cofferdams:

- The first one (DS cofferdam 1) is located just downstream of DT1 culvert (see Figure 4.5). It is used until DT1 and DT2 stretches on the right bank are completed and allows discharging water further downstream; it is used only during the “cofferdam” phase; the relevant rating curve is the one referred to as “Section 3”;
- The second one (DS cofferdam 2) is just upstream of DT1 and DT2 right bank outlet (see Figure 4.5). It is actually part of the final dam toe. It is used from the beginning of “Stage 1 phase” to the final dam completion. The relevant rating curve is the one referred to as “Section 1”

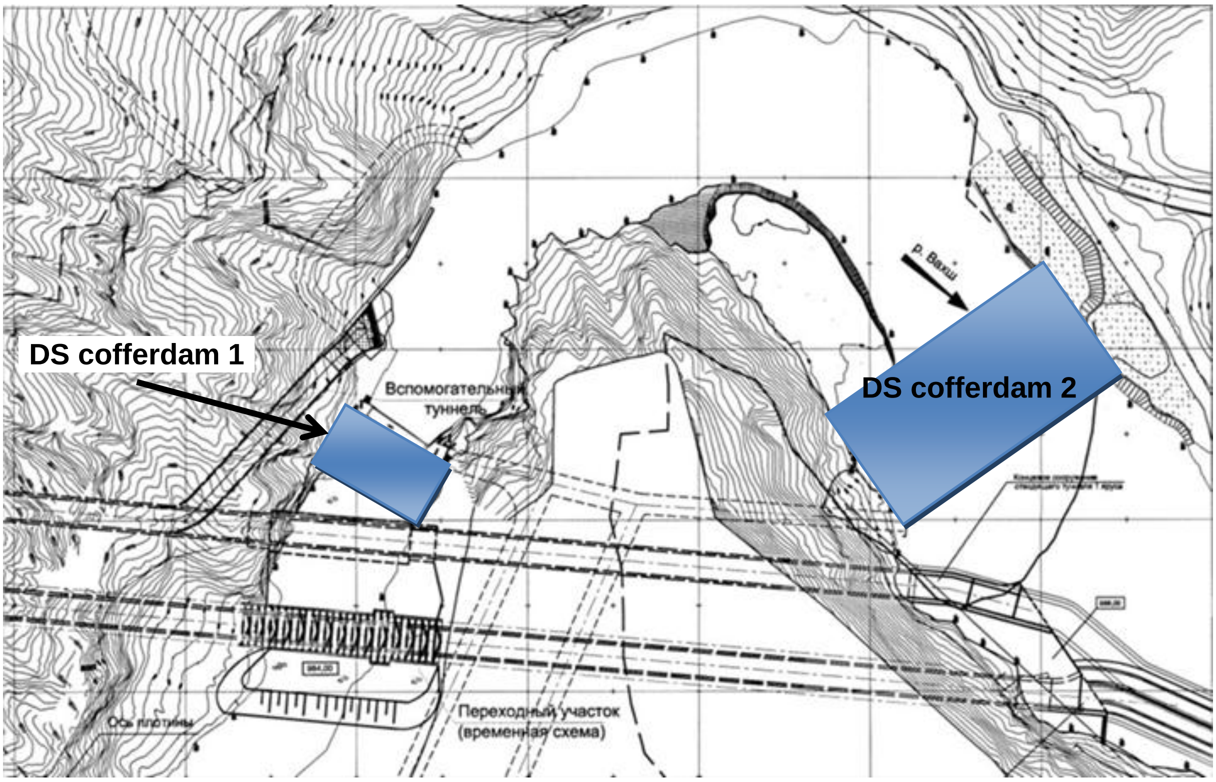


Figure 4.5 : Location of downstream cofferdams

5 HYDRAULIC CALCULATION

In this paragraph, all hydraulic calculation results are presented. It includes all phases of construction, normal and exceptional scenario and several flood level.

Those calculations aim at:

- checking that proposed structures meet the criteria fixed in §2 for all construction phases and all scenario (normal and exceptional);
- Performing a sensitivity analysis on the construction flood level within the acceptable range presented in §2.2 and determining what are the impacts on the structures design.

5.1 Cofferdam

The cofferdam is used until the Stage 1 dam watertighness reaches the cofferdam crest elevation. According to the implementation schedule this phase lasts 1 years and a half, with only one wet season.

During the cofferdam phase, DT1, DT2 and DT3 are used as diversion structures.

The next table presents the maximum water elevation reached for several scenarios: various tunnels configurations and various flood levels.

As this stage, as the reservoir storage capacity is limited, the flood attenuation due to reservoir routing is not taken into account. The maximum daily discharge is considered without any attenuation.

	Normal condition			Exceptional condition	
Flood probability of exceedance	1/50	1/100	1/20	1/50	1/50
Flood return period (years)	50	100	20	50	50
Max daily discharge (m^3/s)	3550	3850	3200	3550	3550
DT1 in operation	YES	YES	YES	NO	YES
DT2 in operation	YES	YES	YES	YES	YES
DT3 in operation	YES	YES	YES	YES	NO
Maximum water elevation (masl)	1044.9	1047.3	1042.0	1068.8	1070.4

Table 5.1 : Hydraulic calculation results – Cofferdam

It is verified that the volume made by the difference between the peak and the daily discharge can be stored in the reservoir: the difference is 200 m^3/s, ie 4.32 hm^3. The reservoir surface at 1147 masl is 3.1 km^2, the difference can be stored in 1.4 m.

In normal operation, ie when all three tunnels are available, the protection is ensured against a probability of exceedance of 1/100 (100 years return period flood) with a cofferdam crest at 1050 masl,

In normal operation, ie when all three tunnels are available, the protection is ensured against a probability of exceedance of 1/50 (50 years return period flood) with a cofferdam crest at 1047 masl,

The difference between the two acceptable flood levels is 3 m on the cofferdam crest.

Availability of the three tunnels can be ensured by adequate works schedule where the tunnels are rehabilitated and completed before the river diversion.

If the DT3 is lost because of Ionaksh fault movements, a cofferdam with a crest at 1050 masl is only protected against a peak discharge of 3030 m^3/s (for this discharge the water level raises up to the crest). This discharge matches a 10 years return period flood. If any higher flood occurs, the cofferdam will be overtopped and will collapse.

5.2 Stage 1

The Stage 1 is used until the final dam watertighness reaches the Stage1 crest elevation.

During this construction phase, DT1, DT2 and DT3 are used as diversion structures.

For each alternative, the initial water level is 10 m below the Stage 1 crest. The reservoir routing effect is taken into account within these 10 m.

The next table presents the maximum water elevation reached for several scenarios: various tunnels configuration and various flood level.

	All tunnels available		One tunnel Is out of service			
Flood probability of exceedance	1/100	1/200	1/100	1/200	1/100	1/200
Flood level (years)	400	800	400	800	400	800
Maximum daily discharge (m^3/s)	4400	4700	4400	4700	4400	4700
DT1 in operation	YES	YES	NO	NO	YES	YES
DT2 in operation	YES	YES	YES	YES	YES	YES
DT3 in operation	YES	YES	YES	YES	NO	NO
Maximum water elevation reached (masl)	1100	1100	1100	1101.7	1102.1	1106.4
Maximum discharge released downstream (m^3/s)	4400	4700	4400	4600	4220	4300

Table 5.2 : Hydraulic calculation results – Stage 1 with FSL 1100 masl

With a Stage 1 crest at elevation 1110 masl, the protection is largely ensured. Even in case of DT3 failure, the Stage 1 dam is protected against the 800 years return period flood, ie a probability of exceedance of 1/200 over the construction period.

Safety margin found is due to the fact that DT3 and Stage 1 were designed by HPI to handle the PMF.

Same calculations are run for the Stage 1 alternatives: with HWL=1065 and 1080 masl, and crest elevation at 1075 and 1090 masl respectively.

	All tunnels available		One tunnel Is out of service				
Flood probability of exceedance	1/100	1/200	1/100	1/200	1/100	1/200	1/140
Flood level (years)	300	600	300	600	300	600	400
Maximum daily discharge (m^3/s)	4300	4550	4300	4550	4300	4550	4400
DT1 in operation	YES	YES	NO	NO	YES	YES	YES
DT2 in operation	YES	YES	YES	YES	YES	YES	YES
DT3 in operation	YES	YES	YES	YES	NO	NO	NO
Maximum water elevation reached (masl)	1080	1080	1084.1	1088.6	1087.5	1093	1090
Maximum discharge released downstream (m^3/s)	4300	4550	4080	4220	3950	4051	3980

Table 5.3 : Hydraulic calculation results – Stage 1 with FSL 1080 masl

	All tunnels available		One tunnel Is out of service				
Flood probability of exceedance	1/100	1/200	1/100	1/200	1/100	1/200	1/50
Flood level (years)	200	400	200	400	200	400	120
Maximum daily discharge (m^3/s)	4100	4400	4100	4400	4100	4400	3960
DT1 in operation	YES	YES	NO	NO	YES	YES	YES
DT2 in operation	YES	YES	YES	YES	YES	YES	YES
DT3 in operation	YES	YES	YES	YES	NO	NO	NO
Maximum water elevation reached (masl)	1065	1065	1077.6	1084.1	1078.7	1086.6	1075
Maximum discharge released downstream (m^3/s)	4100	4400	3875	4080	3765	3925	3680

Table 5.4 : Hydraulic calculation results – Stage 1 with FSL 1065 masl

For lower Stage 1, the protection is still fully ensured in normal condition against a probability of exceedance of 1/200.

At this stage, structures designs are not influenced by the protection level chosen.

In case of DT3 failure, the Stage 1 with crest elevation 1090 masl is still protected against the 400 years return period flood, ie a probability of exceedance of 1/140 over the construction period.

In case of DT3 failure, the Stage 1 with crest elevation 1075 masl is still protected against the 120 years return period flood, ie a probability of exceedance of 1/50 over the construction period.

5.3 Final dam before completion

Once the final dam core highest elevation overpasses the Stage 1 dam elevation, the final dam ensures the safety against flood by itself.

For clarity purpose, this phase has been split into 3 steps:

- Step A: river diversion is ensured by DT3 and MLO1;
- Step B: DT3 is not used anymore and is replaced by high level spillways for the two lower dam alternatives (FSL =1255 and 1220 masl); and by the MOL2 for the higher dam alternatives;
- Step C: this step is necessary for the higher dam alternative only, it matches the period where MOL1 does not work anymore and is replaced by high level spillways.

For each alternatives and steps, the initial water level is 10 m below the dam crest. The reservoir routing effect is taken into account within these 10 m.

5.3.1 Step A

5.3.1.1 FSL+1290 masl

Once the final dam core overpasses the Stage 1 dam elevation, the final dam ensures the safety against flood by itself.

At elevation 1100 masl, the head in DT1 and DT2 is 120 and 100 m respectively. Those tunnels cannot operate with higher heads. Therefore, another structure has to replace them.

The next figure presents the maximum water elevation reached after the reservoir routing for several scenarios (various tunnels configuration, various flood level) with respect to the initial water level in the reservoir (10 m below the dam crest).

This type of figure will be used several times in the following paragraphs. On the horizontal axis is the initial reservoir level. Each curve presents the maximum water elevation reached by the reservoir during a specific situation (specific flood and specific tunnels configuration) depending on the initial water level.

When the reservoir level is low, the curve is flat. Indeed, in that case the initial water head is not sufficient to release the maximum flood so the reservoir level rises to reach a sufficient discharge capacity. In that case, the reservoir routing is maximum.

When the reservoir level is high, the curve tends to a 1H/1V slope. Indeed, when the initial reservoir level is high enough, the discharge capacity of the tunnel is equal or higher than the maximum flood discharge. In that case, there is no reservoir routing.

The grey line represents an elevation always 10 m higher than the initial water level. It means that when the maximum water elevation is higher than this line, the dam is overtopped (red area).

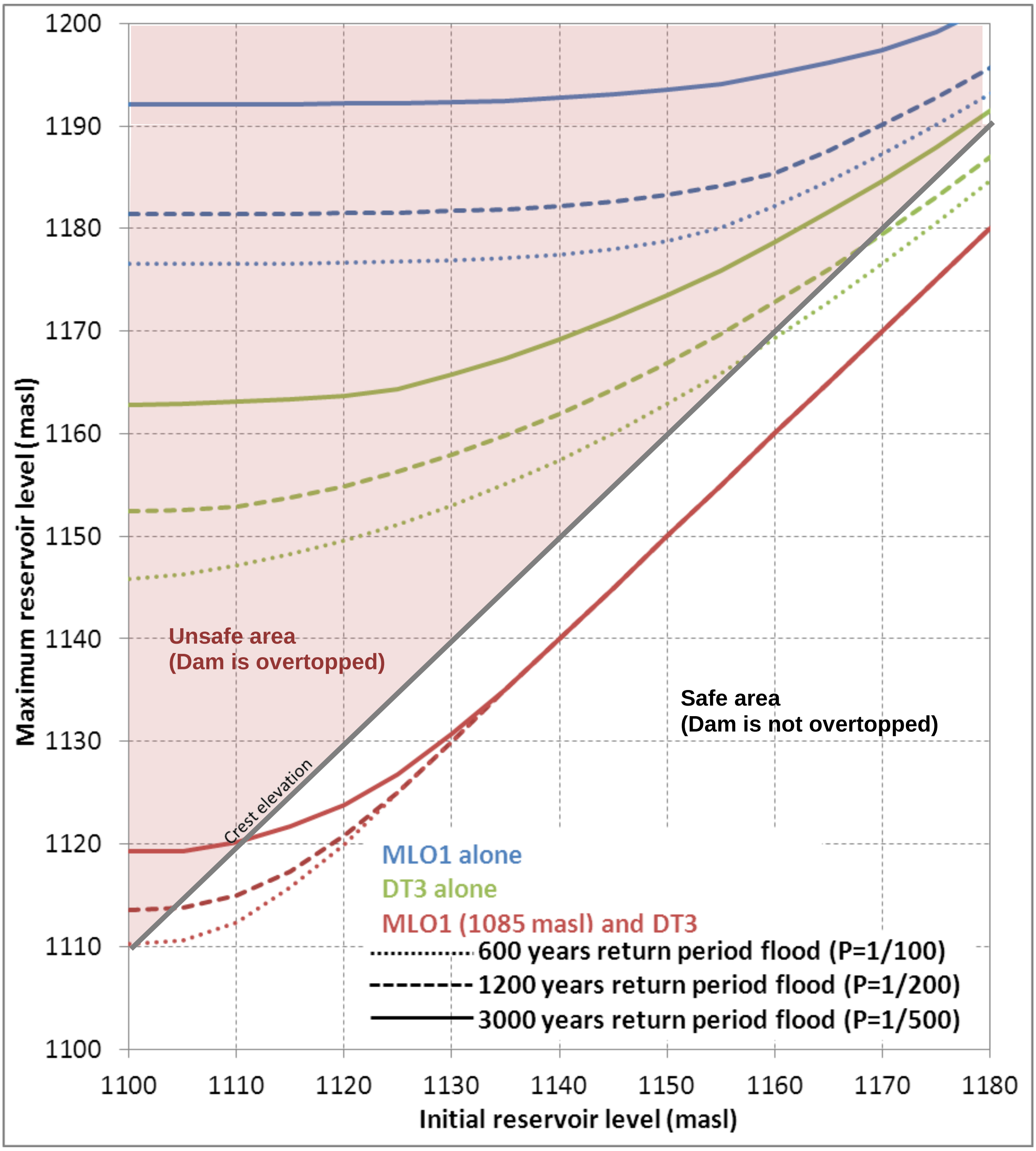


Figure 5.1 : DT3 and MLO1 reservoir routing results – FSL=1290 masl

At initial reservoir elevation 1115 masl, the water head is sufficient for DT3 and MOL1 to divert the 3000 years return period flood (probability of exceedance of 1/500) without overtopping the dam (assuming a freeboard of 10m).

At initial reservoir elevation 1110 masl, the water head is sufficient for DT3 and MOL1 to divert the 1200 years return period flood (probability of exceedance of 1/200) without overtopping the dam (assuming a freeboard of 10m).

At initial reservoir elevation 1105 masl, the water head is sufficient for DT3 and MOL1 to divert the 600 years return period flood (probability of exceedance of 1/100) without overtopping the dam (assuming a freeboard of 10m).

It means that up to reservoir elevation 1115, 1110 and 1105, masl, DT1 and DT2 have to be available in case of respectively 3000, 1200 and 600 years return period flood. In such an event, the water head in DT1 and DT2 will be 125, 120, and 115 m respectively, depending on the protection level chose. Both are acceptable as per the design criteria.

Above these elevation, in normal operation, when the two tunnels are available (MLO1 and DT3), the flood protection is ensured for 1/500, 1/200 and 1/100 probability of exceedance. Indeed, the reservoir elevation will not rise above the crest.

At this step, structures characteristics (number, size, location) are not influenced by the protection level chosen.

If one of the two tunnels is out of service, the protection is not ensured anymore: the reservoir level will rise by 30-60 meters depending on the scenario.

5.3.1.2 FSL=1255 masl

The same analysis is made for dam alternative FSL=1255 masl. The next figure presents the maximum water elevation reached after the reservoir routing for several scenarios (various tunnels configuration, various flood level) with regards to the initial water level in the reservoir.

At initial reservoir elevation 1110 masl, the water head is sufficient for DT3 and MOL1 to divert the 2500 years return period flood (probability of exceedance of 1/500) without overtopping the dam (assuming a freeboard of 10m).

At initial reservoir elevation 1105 masl, the water head is sufficient for DT3 and MOL1 to divert the 1000 years return period flood (probability of exceedance of 1/200) without overtopping the dam (assuming a freeboard of 10m).

At initial reservoir elevation 1100 masl, the water head is sufficient for DT3 and MOL1 to divert the 500 years return period flood (probability of exceedance of 1/100) without overtopping the dam (assuming a freeboard of 10m).

It means that up to reservoir elevation 1110, 1105 and 1100, masl, DT1 and DT2 have to be available in case of respectively 2500, 1000 and 500 years return period flood. In such an event, the water head in DT1 and DT2 will be 120, 115, and 110 m respectively, depending on the protection level chosen. Both are acceptable as per the design criteria.

Above these elevation, in normal operation, when the two tunnels are available (MLO1 and DT3), the flood protection is ensured for 1/500, 1/200 and 1/100 probability of exceedance. Indeed, the reservoir elevation will not rise above the crest.

At this step, structures characteristics (number, size, location) are not influenced by the protection level chosen.

If one of the two tunnels is out of service, the protection is not ensured anymore: the reservoir level will rise by 30-60 meters depending on the scenario.

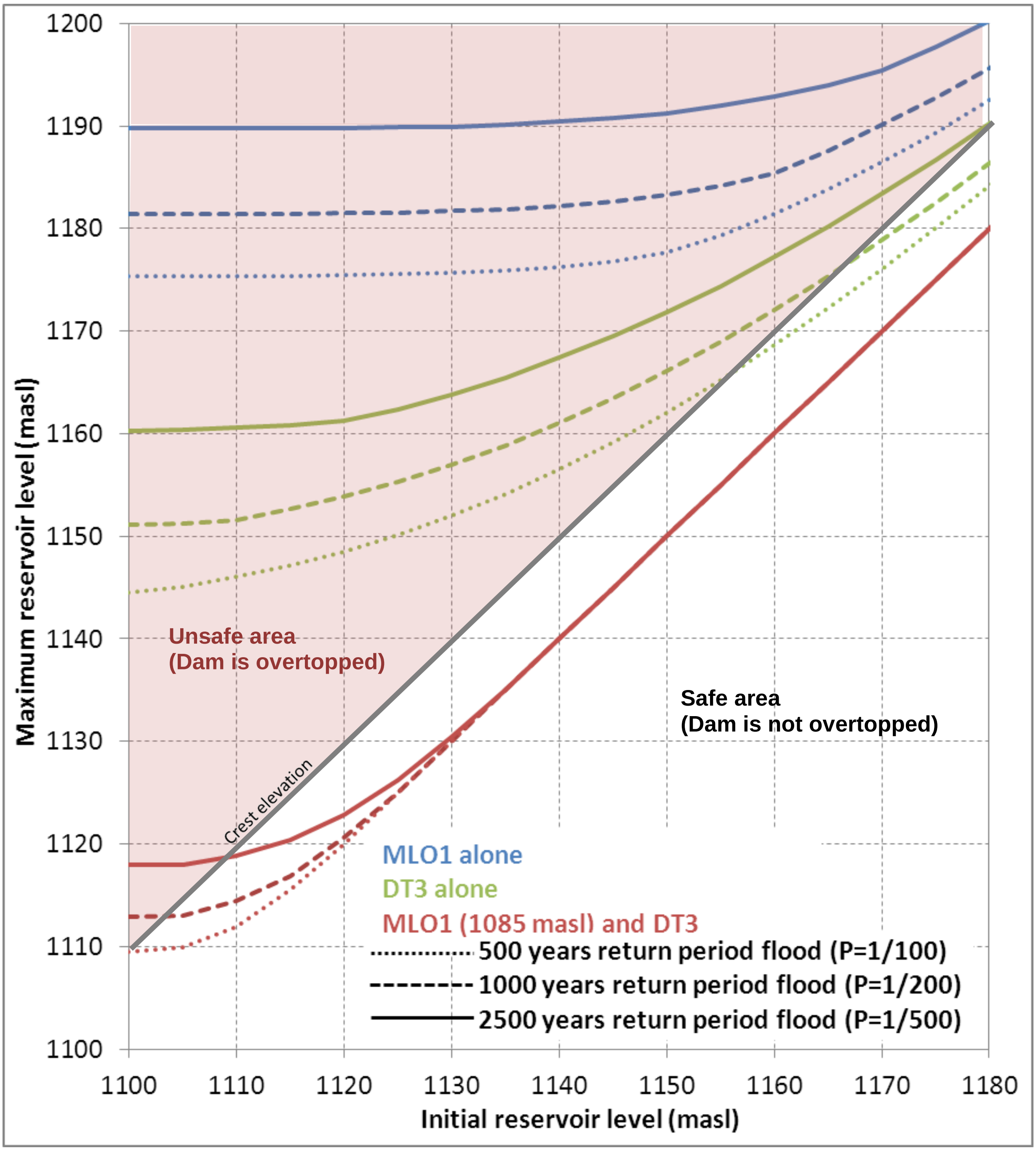


Figure 5.2 : DT3 and MLO1 reservoir routing results – FSL=1255 masl

5.3.1.3 FSL=1220 masl

The same analysis is made for dam alternative FSL=1220 masl. The next figure presents the maximum water elevation reached after the reservoir routing for several scenarios (various tunnels configuration, various flood level) with regards to the initial water level in the reservoir.

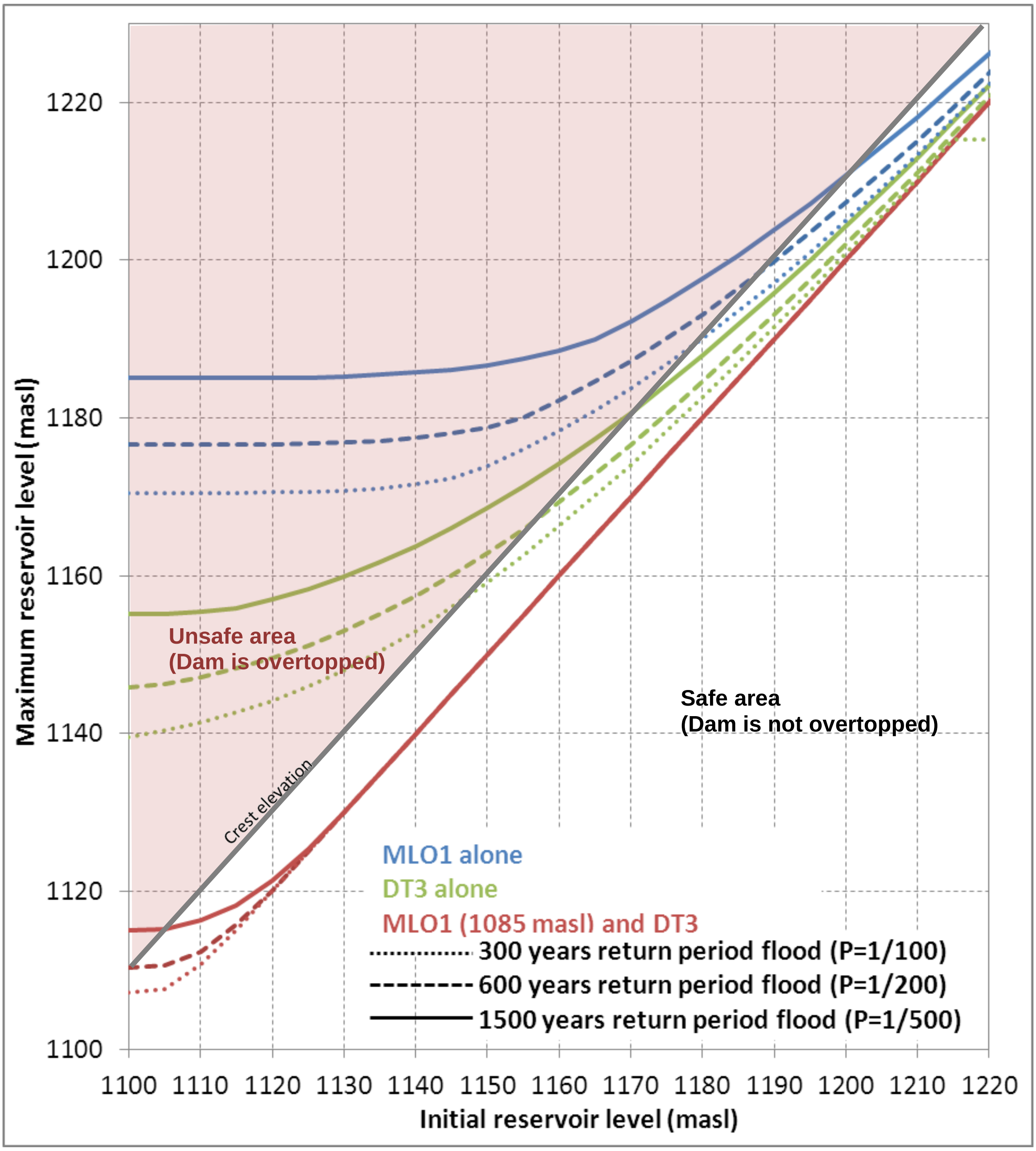


Figure 5.3 : DT3 and MLO1 reservoir routing results – FSL=1220 masl

At initial reservoir elevation 1110 masl, the water head is sufficient for DT3 and MOL1 to divert the 1500 years return period flood (probability of exceedance of 1/500) without overtopping the dam (assuming a freeboard of 10m).

At initial reservoir elevation 1105 masl, the water head is sufficient for DT3 and MOL1 to divert the 600 years return period flood (probability of exceedance of 1/200) without overtopping the dam (assuming a freeboard of 10m).

At initial reservoir elevation 1100 masl, the water head is sufficient for DT3 and MOL1 to divert the 300 years return period flood (probability of exceedance of 1/100) without overtopping the dam (assuming a freeboard of 10m).

It means that up to reservoir elevation 1110, 1105 and 1100, masl, DT1 and DT2 have to be available in case of respectively 1500, 600 and 300 years return period flood. In such an event, the water head in DT1 and DT2 will be 120, 115, and 110 m respectively, depending on the protection level chose. Both are acceptable as per the design criteria.

Above these elevation, in normal operation, when the two tunnels are available (MLO1 and DT3), the flood protection is ensured for 1/500, 1/200 and 1/100 probability of exceedance. Indeed, the reservoir elevation will not rise above the crest.

At this step, structures characteristics (number, size, location) are not influenced by the protection level chosen.

If one of the two tunnels is out of service, the protection is not ensured anymore: the reservoir level will rise by 30-60 meters depending on the scenario.

5.3.2 Step B

DT3 sill elevation is 1035 masl, as per the criteria established, it should stop operating when the water elevation is 1155 masl. Therefore, another structure should replace it around this elevation.

For the higher alternative (FSL = 1290 masl), a mid-level outlet 2 (MLO2) is necessary.

For the lowest alternative (FSL = 1220 masl), tunnels are also necessary at this elevation, but they can be final spillways and not only temporary structure. They will be around 90 meters below the FSL which is acceptable as per the design criteria.

For the intermediate alternative (FSL = 1255 masl), tunnels are also necessary at this elevation. To ease the switching between DT3 and final spillways, one is set at elevation 1145 masl and two others at elevation 1165 masl.

5.3.2.1 FSL=1290 masl

For this alternative, a mid-level outlet 2 (MLO2) is necessary.

The next figure presents the maximum water elevation reached after the reservoir routing for several scenarios (various tunnels configuration, various flood level) with respect to the initial water level in the reservoir.

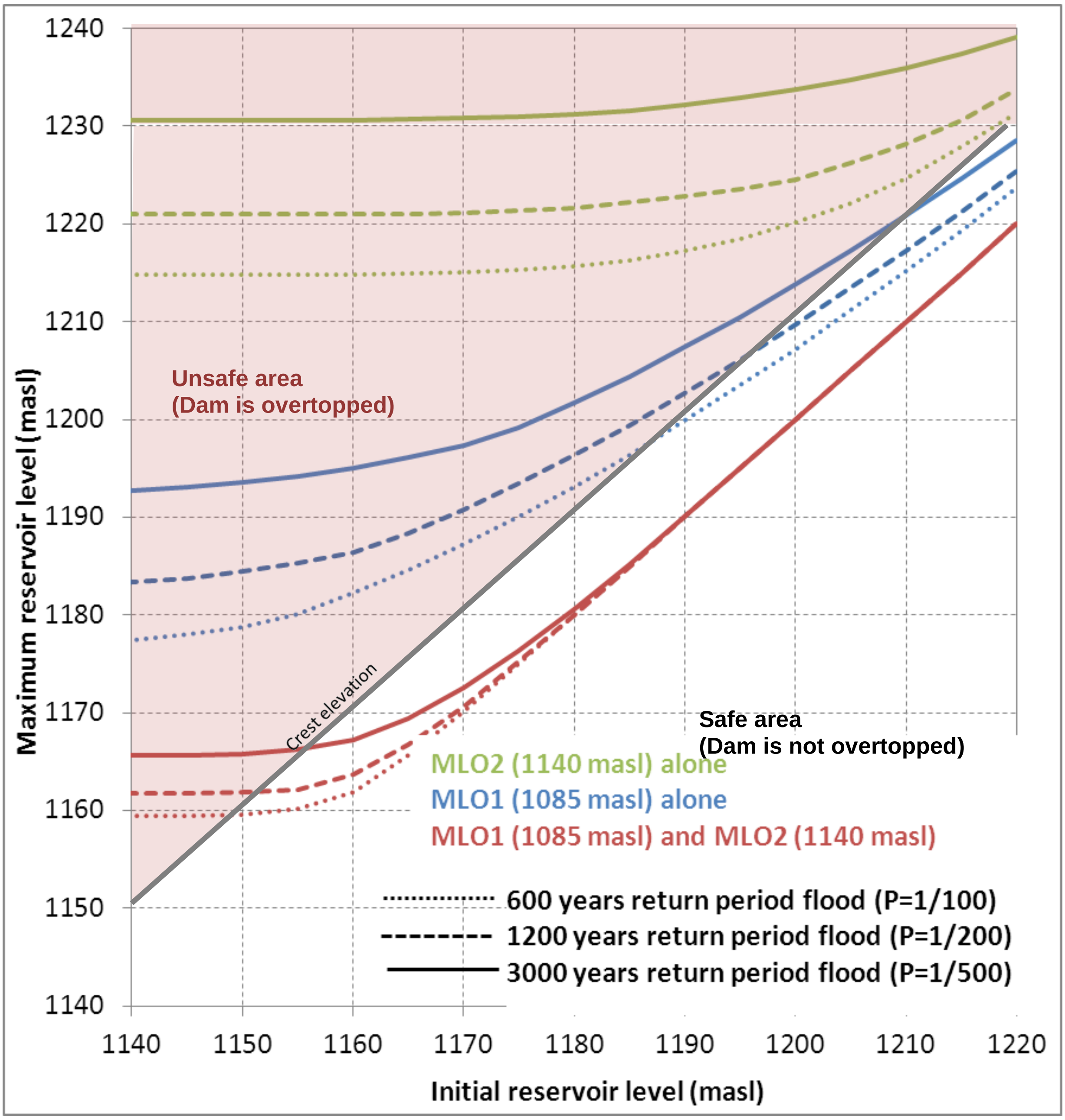


Figure 5.4 : Reservoir routing results – StepB – FSL=1290 masl

At initial reservoir elevation 1160 masl, the water head is sufficient for MOL1 and MOL2 to discharge the 3000 years return period flood (probability of exceedance of 1/500) without overtopping the dam (assuming a freeboard of 10m).

At initial reservoir elevation 1155 masl, the water head is sufficient for MOL1 and MOL2 to discharge the 1200 years return period flood (probability of exceedance of 1/200) without overtopping the dam (assuming a freeboard of 10m).

At initial reservoir elevation 1152.5 masl, the water head is sufficient for MOL1 and MOL2 to divert the 600 years return period flood (probability of exceedance of 1/100) without overtopping the dam (assuming a freeboard of 10m).

It means that up to reservoir elevation 1160, 1155 and 1152.5, masl, DT3 has to be available in case of respectively 3000, 1200 and 600, years return period flood. In such an event, the water head in DT3 will be 125, 120, and 117.5 m respectively, depending on the protection level chose. Both are acceptable as per the design criteria.

Above these elevations, in normal operation, when the two tunnels are available (MLO1 and MOL2), the flood protection is ensured for 1/500, 1/200 and 1/100 probability of exceedance from 1160 masl and above. Indeed, the reservoir level will not rise above the crest.

At this step, structures characteristics (number, size, location) are not influenced by the protection level chosen.

If one of the two tunnels is out of service, the protection is not ensured anymore: the reservoir level will rise by 15-40 meters depending on the scenario.

5.3.2.2 FSL =1255 masl

For this alternative MLO2 is not necessary. The high level outlet 1 (HL1) settled at 1145 masl ensures part of the diversion when DT3 is closed.

The next figure presents the maximum water elevation reached after the reservoir routing for several scenarios (various tunnels configuration, various flood level) with respect to the initial water level in the reservoir.

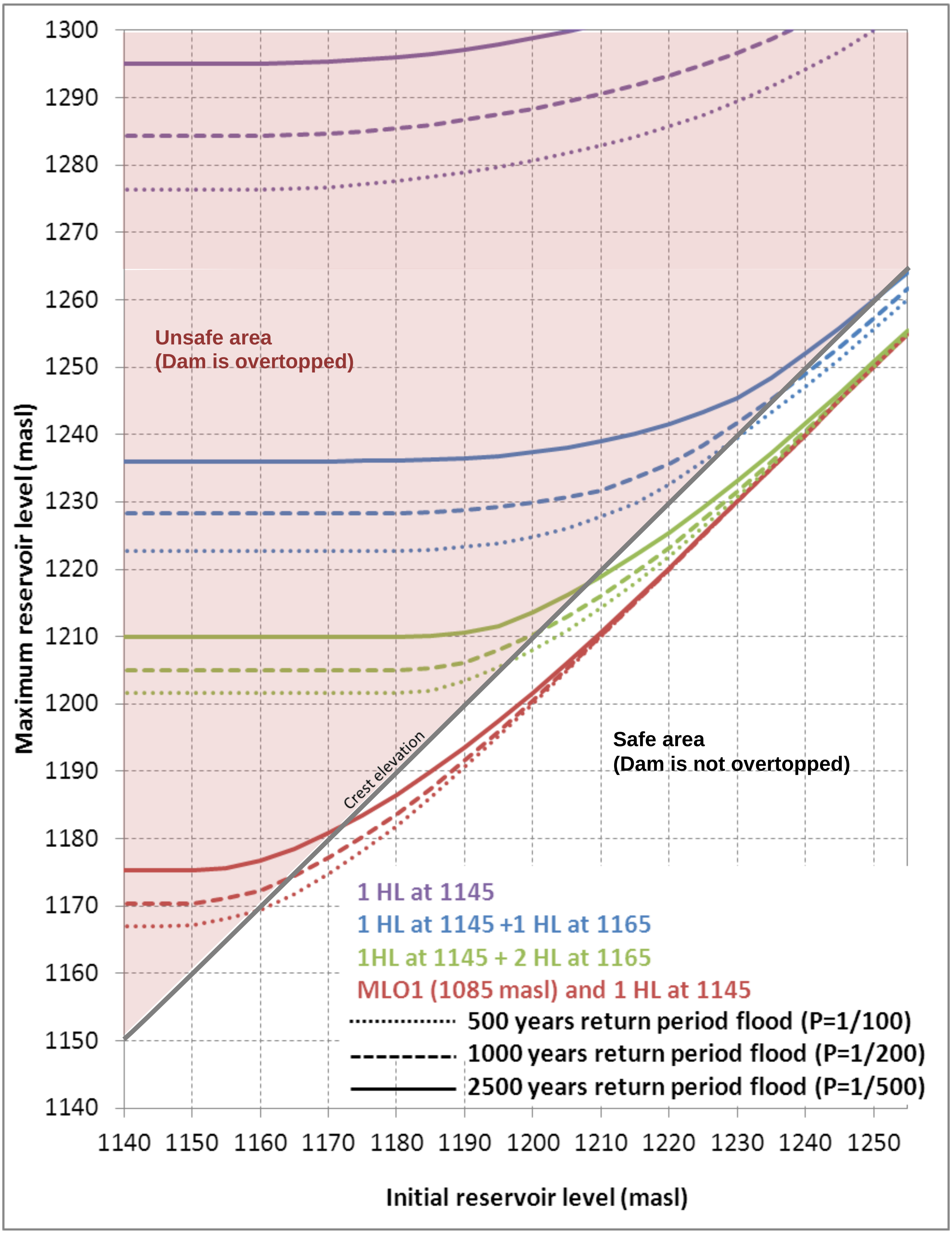


Figure 5.5 : Reservoir routing results - Step B FSL =1255 masl

At reservoir elevation 1175 masl, the water head is sufficient for MLO1 and HL1 to discharge the 2500 years return flood assuming a 10 m freeboard.

At reservoir elevation 1170 masl, the water head is sufficient for MLO1 and HL1 to discharge the 1000 years return flood assuming a 10 m freeboard.

At reservoir elevation 1165 masl, the water head is sufficient for MLO1 and HL1 to discharge the 500 years return flood assuming a 10 m freeboard.

It means that up to reservoir elevation 1175, 1170 and 1165 masl, DT3 have to be available in case of respectively 2500, 1000 and 500 years return period flood. In such an event, the water head in DT3 will be 140 m, 135 m and 130 m respectively. This is acceptable as it is a temporary and extreme condition.

Above these elevation, in normal operation, when the two tunnels are available (MLO1 and MOL2), the flood protection is ensured for 1/100, 1/200 and 1/500 probability of exceedance. Indeed, the reservoir level will not rise above the crest.

At this step, structures characteristics (size, numbers, and location) are not influenced by the protection level chosen.

In case of MLO1 failure, the reservoir level could rise by 100 m in case of 500 or 2500 years return period flood.

It should be noted that even if 2 high tunnels are available, MOL1 failure would lead to a 50 m rise in the reservoir, and 25 m rise if 3 high tunnels are available.

At reservoir elevation 1210 masl, the water head is sufficient for HL1, HL2 and HL3 to divert the 2500 years return period flood assuming a 10 m freeboard. At reservoir elevation 1205 masl, the water head is sufficient for HL1, HL2 and HL3 to divert the 1000 years return period flood assuming a 10 m freeboard. At reservoir elevation 1205 masl, the water head is sufficient for HL1, HL2 and HL3 to divert the 500 years return period flood assuming a 10 m freeboard.

It means that up to elevation 1210, 1205 and 1200 masl, MOL1 should then be available in case of respectively 2500, 1000 and 500, years return period flood. In such an event, the water head in MOL1 will be 125, respectively 120 m and 115 m. This is acceptable as per the design criteria.

Above these elevation, in normal operation, when the three tunnels are available (HL1, HL2 andHL3), the flood protection is ensured for 1/100, 1/200 and 1/500 probability of exceedance. Indeed, the reservoir level will not rise above the crest.

If only HL1 and HL2 are available, MLO1 should be available until reservoir elevation reaches 1250 masl, ie a head of 165 m. This is not acceptable as per the design criteria. Therefore, 3 high tunnels spillways are necessary to ensure a protection of 1/100, 1/200 or 1/500 at the end of construction.

At this step, structures characteristics (size, numbers, and location) are not influenced by the protection level chosen.

5.3.2.3 FSL = 1220 masl

For this alternative MLO2 is not necessary. The high level outlet 1 (HL1) settled at 1140 masl ensures part of the diversion when DT3 is closed.

The next figure presents the maximum water elevation reached after the reservoir routing for several scenarios (various tunnels configuration, various flood level) with respect to the initial water level in the reservoir.

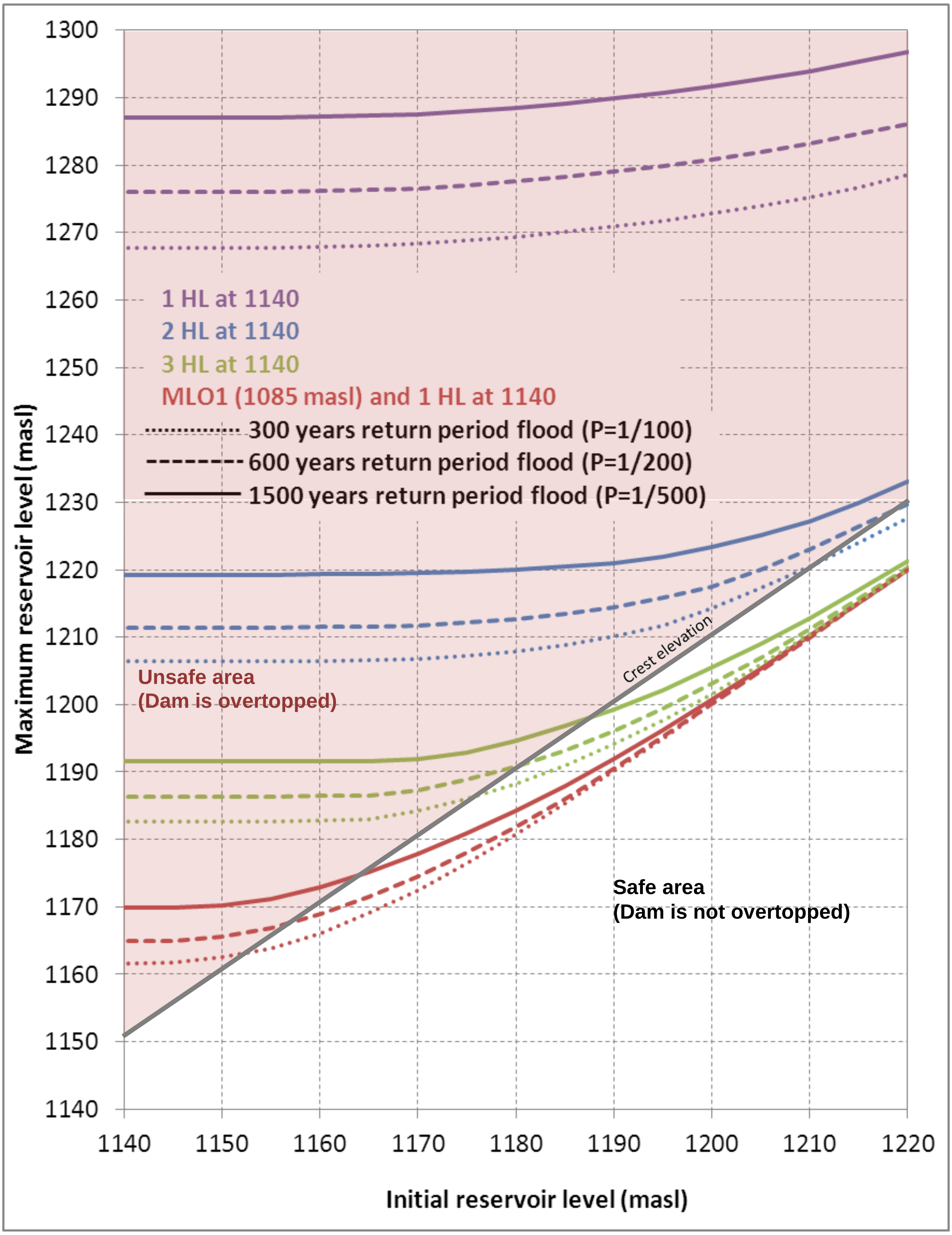


Figure 5.6 : Reservoir routing results - StepB FSL=1220 masl

At reservoir elevation 1170 masl, the water head is sufficient for MLO1 and HL1 to discharge the 1500 years return flood assuming a 10 m freeboard. At reservoir elevation 1165 masl, the water head is sufficient for MLO1 and HL1 to discharge the 600 years return flood assuming a 10 m freeboard. And at reservoir elevation 1155 masl, the water head is sufficient for MLO1 and HL1 to discharge the 300 years return flood assuming a 10 m freeboard.

It means that up to reservoir elevation 1170, masl, 1165 and 1155 masl, DT3 have to be available in case of respectively 1500, 600 and 300 years return period flood. In such an event, the water head in DT3 will be 135 m, respectively 130 m and 120 m. Both are acceptable as it is a temporary and extreme condition.

Above these elevations and until dam completion, in normal operation, MLO1 and HL1 are able to ensure a protection against a 1/500, 1/200 or 1/100 probability of exceedance event.

At this step, structures characteristics (size, numbers and location) are not influenced by the protection level chosen.

In case of MLO1 failure (shear zone), only one high level tunnel remains and the reservoir level raise by 100 m. It should be noted that even if 2 high spillways are available, the water level raises by 40 m. To fully compensate the loss of MOL1, 3 high level spillways and a reservoir level at 1195 masl are necessary.

If the risk of failure of MOL1 can be avoided, only 1 high tunnel spillways is necessary for diversion purpose during construction.

5.3.3 Step C

This step is only necessary for the highest alternative (FSL=1290 masl), as the two others are already working with their final spillways.

The high level outlet 1 and 2 (HL1 and HL2) settled at 1190 masl ensures part of the diversion when first, MLO1 is closed and then MLO2 is closed.

The next figure presents the maximum water elevation reached after the reservoir routing for several scenarios (various tunnels configuration, various flood level) with respect to the initial water level in the reservoir.

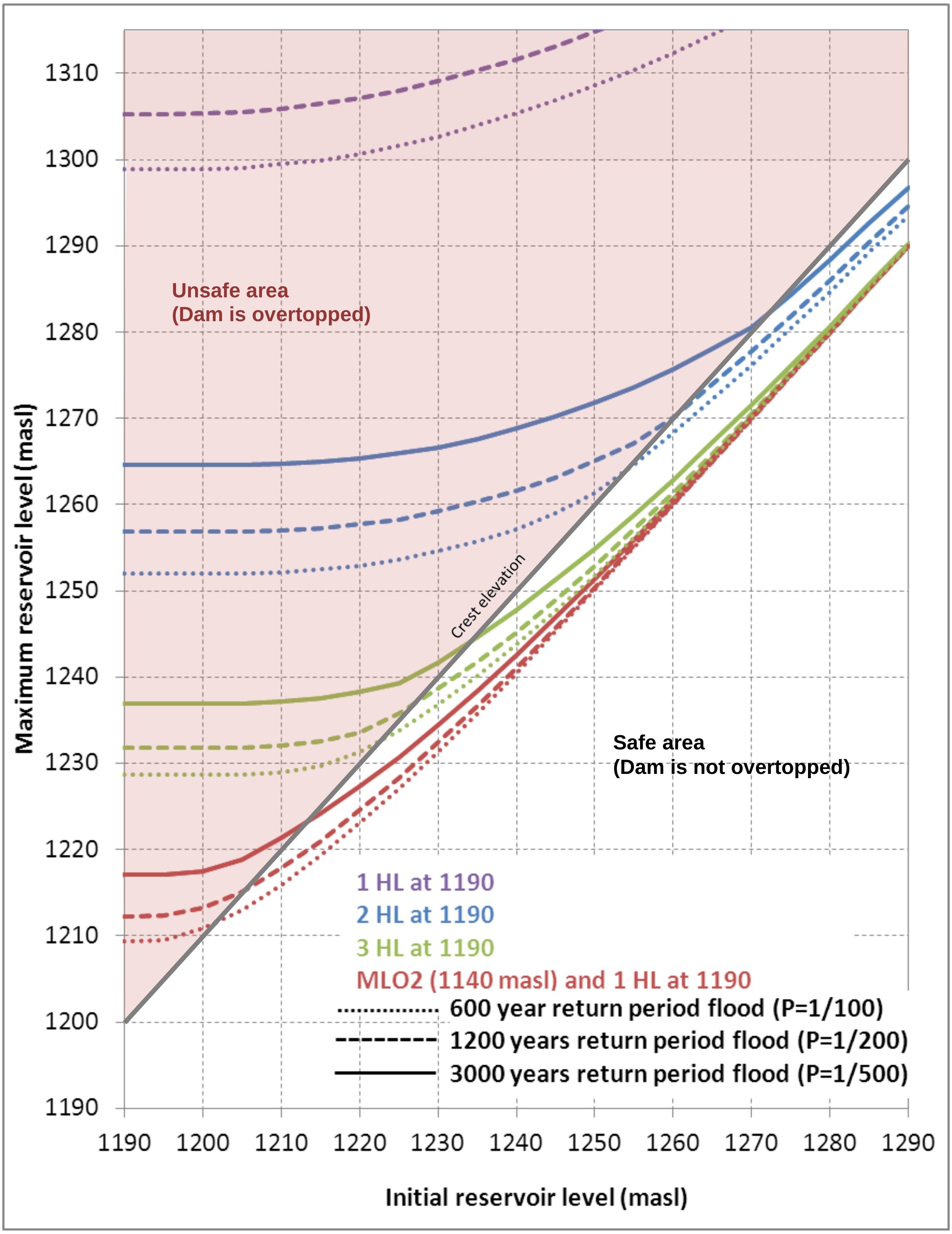


Figure 5.7 : Reservoir routing results - StepC FSL =1290 masl

At reservoir elevation 1220 masl, the water head is sufficient for MLO2 and HL1 to discharge the 3000 years return flood assuming a 10 m freeboard. At reservoir elevation 1215 masl, the water head is sufficient for MLO2 and HL1 to discharge the 1200 years return flood assuming a 10 m freeboard. At reservoir elevation 1210 masl, the water head is sufficient for MLO2 and HL1 to discharge the 600 years return flood assuming a 10 m freeboard.

It means that up to reservoir elevation 1220 masl, 1215 masl and 1210 masl, MLO1 have to be available in case of respectively 3000, 1200 and 600 years return period flood. In such an event, the water head in MLO1 will be 135 m, respectively 130 m and 125 m. Both are acceptable as it is a temporary and extreme situation.

Above these elevations, in normal operation, MLO2 and HL1 are able to ensure a protection against a probability of exceedance of 1/100, 1/200 and 1/500.

At this step, structures characteristics (size, numbers and location) are not influenced by the protection level chosen.

Then, at reservoir elevation 1260 masl, the water head is sufficient for HL1 and HL2 to divert the 600 years return flood assuming a 10 m freeboard. And at reservoir elevation 1270 masl, the water head is sufficient for HL1 and HL2 to divert the 1200 years return flood assuming a 10 m freeboard. And at reservoir elevation 1280 masl, the water head is sufficient for HL1 and HL2 to divert the 3000 years return flood assuming a 10 m freeboard.

It means that up to reservoir elevation 1260 masl, 1270 masl and 1280 masl, MLO2 have to be available in case of respectively 600, 1200 and 3000 years return period flood. In such an event, the water head in MLO2 will be 120 m, respectively 130 m and 140 m. This is acceptable as it is a temporary and extreme situation.

Above these elevations and until dam completion, in normal operation, HL1 and HL2 are able to ensure a protection against a probability of exceedance of 1/100, 1/200 and 1/500.

5.4 Sensitivity analysis on the maximum head acceptable in tunnel

The same analysis as in the previous paragraph is performed with a criteria of 150 m head as the maximum water head acceptable in the new tunnels (MLO1, MLO2 and various HLs).

MOL1 elevation is not changed as it depends on the DT1 and DT2 maximum water head, it remains at 1085 masl.

HLs elevation can be changed: for dam alternative FSL=1290 masl, they can be set at 1145 masl, for dam alternative FSL=1255 masl, they can be set at elevation 1110 masl, and for dam alternative FSL1220 masl, they can be set at elevation 1075 masl.

The diversion structure sequence is then:

- For FSL=1290 masl: up to water elevation 1115 masl, the flood diversion is ensured by DT1+DT2+DT3. Between 1115 and 1175 masl, DT3 and MOL1 are used, between 1175 masl and 1250 masl, MOL1 and 2 HLs can handle the flood diversion (at 1250 masl the exceptional head on MOL1 is 165 m) and finally above 1250 masl, 2 HLs ensured the river diversion.

- For FSL=1255 masl: up to water elevation 1115 masl, the flood diversion is ensured by DT1+DT2+DT3. Between 1115 and 1170 masl, DT3 and MOL1 are used, between 1170 masl and 1230 masl, MOL1 and one HL can handle the flood diversion and finally above 1230 masl, 2 HLs ensured the river diversion.

- For FSL=1220 masl: up to water elevation 1110 masl, the flood diversion is ensured by DT1+DT2+DT3. Between 1110 and 1205 masl, DT3 and 2 HLs are used (at 1205 masl the exceptional head on DT3 is 170 m), and finally above 1205 masl, 2 HLs ensured the river diversion.

Compared to the previous analysis:

- for the highest dam alternative, MLO2 is not necessary anymore (87 MUSD are saved);
- for the medium alternative, one HL is not necessary (60 MUSD are saved);
- for the lowest alternative, MOL1 is replaced by one HL, which is a tunnel with a smaller diameter (60 MUSD are saved).

Between 60 and 87 Millions of USD, which represents only 1.5% of the total Rogun cost can be saved by considering 150 m water head instead of 120 m.

The saving is very limited compared to the risk increase:

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

Therefore, the Consultant recommends to keep considering 120 m as a design criterion in this feasibility study. Optimization of the layout will be done in any case in further stage of the study.

6 CONCLUSIONS

Based on the calculation results detailed in the previous paragraph, conclusions on construction flood management are presented hereunder for each phase.

6.1 Cofferdam

With a cofferdam at elevation 1050 masl, and DT1, DT2, DT3 operating, the level of protection is 1/100, ie a return period of 100 years. And with a cofferdam at 1047 masl elevation, the protection is ensured against a probability of exceedance of 1/50.

These 3 meters of filling material to be added on the cofferdam crest to achieve a higher protection level would be in any case placed in the dam, as the cofferdam is embedded in the Stage 1 dam and final dam. Therefore, protecting the cofferdam against a probability of exceedance of 1/100 instead of 1/50 does not lead to any additional costs.

Moreover, these 3 meters of filling material will not change the construction period for the cofferdam.

Considering all the above, the construction flood considered for the cofferdam is the 100 years return period flood, ie a probability of exceedance of 1/100. The cofferdam crest elevation is then 1050 masl. The construction flood is discharged through DT1, DT2 and DT3.

It has been brought to the attention of the Consultant that HPI developed new capacity curves for DT1 and DT2, allowing a slightly higher discharge to pass. If this improvement is proved efficient, a slight lowering of the cofferdam crest could be foreseen in Phase 3.

Downstream cofferdam of this phase is the DS cofferdam 1, the relevant rating curve is the “Section 2”. For the construction flood considered (100 years return flood), the water elevation is 992.7 masl, and therefore **the downstream cofferdam crest shall be 994 masl**.

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

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

6.2 Stage 1

If all tunnels are available (DT1, DT2, DT3), the Stage 1 dam in its three alternative is protected against a probability of exceedance of 1/100 and 1/200.

There is no difference on the structures design if the probability of exceedance is limited to 1/100 or 1/200.

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

Downstream cofferdam of this phase is the second one; the relevant rating curve is the “Section 1”. For the construction flood considered, the water elevation is 984.2 masl, and therefore **the downstream cofferdam crest shall be 986 masl**.

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

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

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

6.3 Before final dam completion

The calculation made on the previous paragraph shows that in normal operation the proposed diversion structures ensure a protection against a probability of exceedance of 1/100, 1/200 and 1/500 for all alternatives and all construction steps.

The only difference between the three protection levels is that tunnels shall be able to handle different water heads within a maximum range of 20 m. This is not a significant design difference**. Therefore, the construction flood considered for the last construction period has a probability of exceedance of 1/200**. This matches the 600 years return period flood for the dam alternatives FSL=1220 masl, the 1000 years return period flood for the dam alternatives FSL=1255 masl and the 1200 years return period flood for the dam alternatives FSL=1290 masl.

From water elevation 1100 masl, the construction flood is discharged through to DT3 and MOL1 are discharge.

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

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

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

Downstream cofferdam of this phase is the DS cofferdam 2, the relevant rating curve is the “Section 1”. For the construction flood considered (600, 1000 or 1200 years return flood depending on the alternatives), the water elevation is 984.4 masl, 984.5 masl or 984.6 masl depending on the alternative, and therefore **the downstream cofferdam crest shall be 986 masl.**

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

MLO1 risk of failure can be avoided by designing a specific intake that does not cross the Ionaksh fault: the tunnel enters the banks downstream of the fault, and a culvert that crosses the dam drives the flow from the reservoir to the tunnel (see drawing n°40 114). The culvert inner section is D shaped, 18 m diameter, with lower chamfers and would be designed so to resist the maximum dam filling above its crown (about 35 m) and strong seismic effects. The structure is divided in segments some 25-30 m long, the first of which is lying on the Ionaksh fault. In case of fault displacements, the segment can be also displaced both with respect to the tunnel portal proper entering into the right bank and with respect to the adjoining segment upstream of it resting out of the fault, but would not collapse and the hydraulic connection between intake and tunnel proper portal would be maintained.

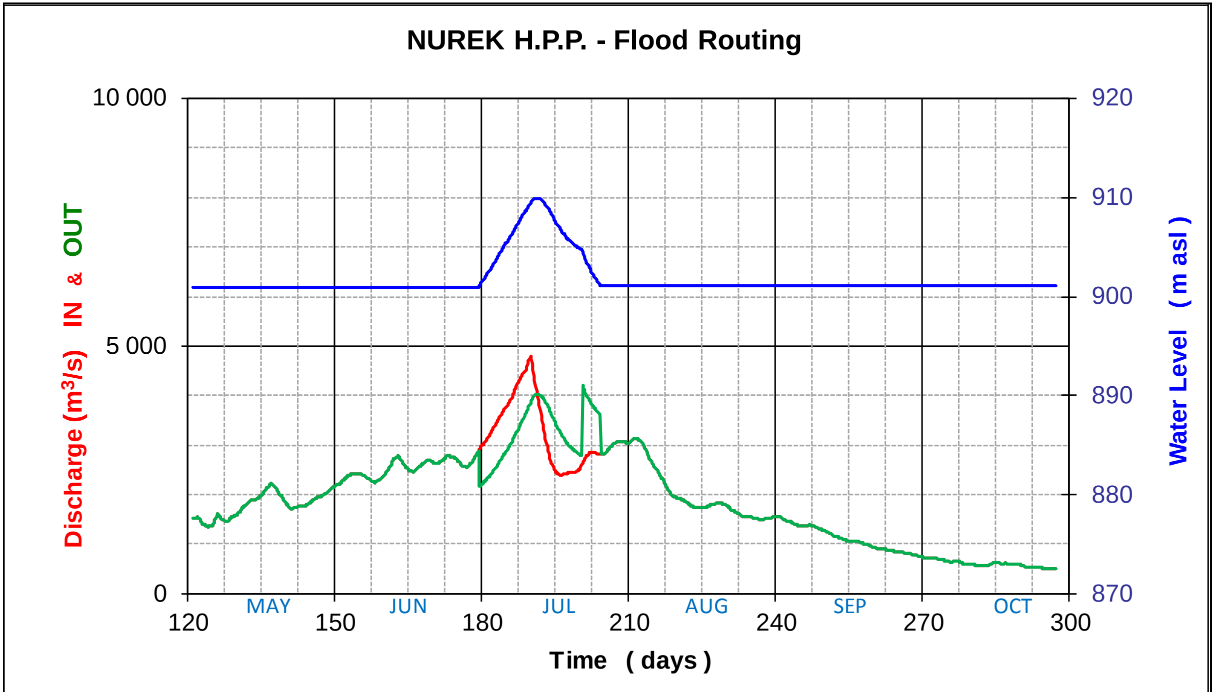
No feasible layout exists to avoid crossing the Ionaksh fault with DT3. Some mitigation measure can be put in place in the fault stretch to face at least the creeping effect and displacements of moderate entity (see report on geotechnics and report on hydraulic tunnels). The probability of having both a high flood and an important seismic event able to make DT3 collapse within the life span of DT3 is limited. This risk is found acceptable by the Consultant.

6.4 Impact of Rogun construction flood on Nurek

The maximum construction flood over the whole alternatives and construction phases has a daily maximum discharge of 4800 m^3/s.

Assuming that this flood enters in Nurek reservoir unattenuated, it can be handled by the tunnel and surface spillway without overpassing their design discharge if the water level is waiting at 901 masl, as it can be seen on the graph hereunder.

Therefore, whatever are the dam alternatives or construction phases, Nurek can handle these construction floods with the same design criteria as the one presented in the PMF management report, a.k.a “the Washington assumptions”.



Reservoir:		
V = a . (H - H°) ^b		
H°=	630	m asl
a =	4.56E-03	- -
b =	2.60	- -

Reservoir Operation Rules

FSL =	**901.00**	m asl
H° =	**901.00**	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 =	242	0	1 926	0	2 168
QMax=	2 007	0	3 388	0	5 394

Hydrograph		
Hydrograph:	**3**	
Q in-max =	**ROG**	*m3/s*
Results		
Qout-max=	4 207	*m3/s*
H max =	909.95	*m asl*
T (Hmax) =	191.20	*days*

ALA; 22.10.13

6.5 Summary

All the above is now synthetized in the two following illustrative sketches for each dam alternatives.

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

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

Refinement of the diversion discharge structures will be performed ion further design stages for the selected alternatives. Also, in line with the cautious system for flood management during construction presented in this chapter, including safety measures such as the limitation of the water head on gates and the redundancy of works, a flood forecasting and warning system shall be designed and implemented to be operational during the complete duration of Rogun construction.

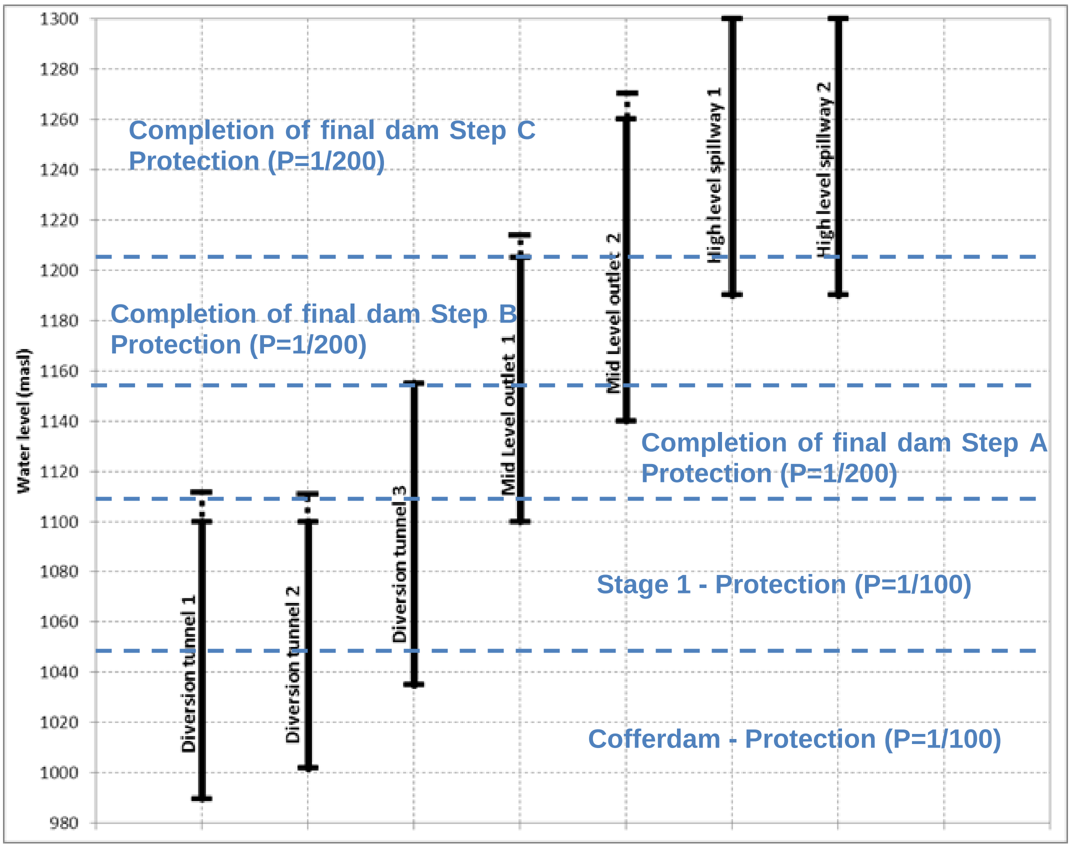


Figure 6.1 : FSL = 1290 masl - Diversion structures operating range

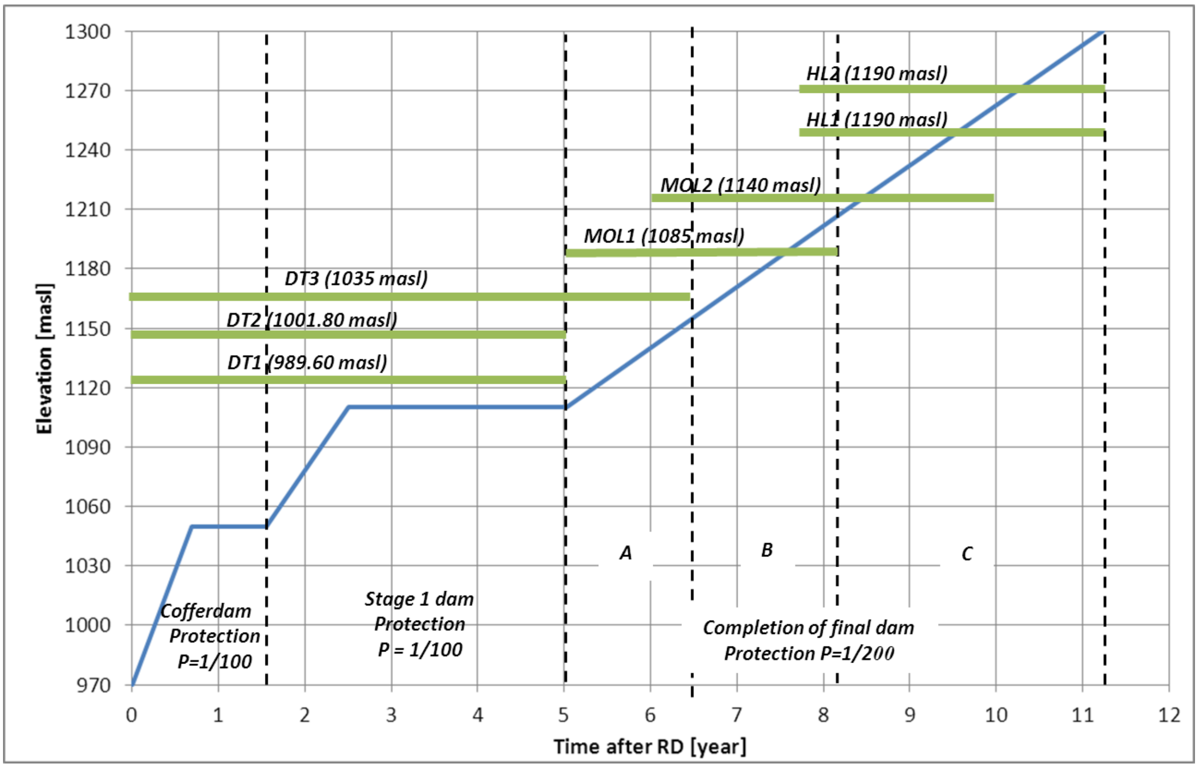


Figure 6.2 : FSL = 1290 masl - Diversion scheme along time

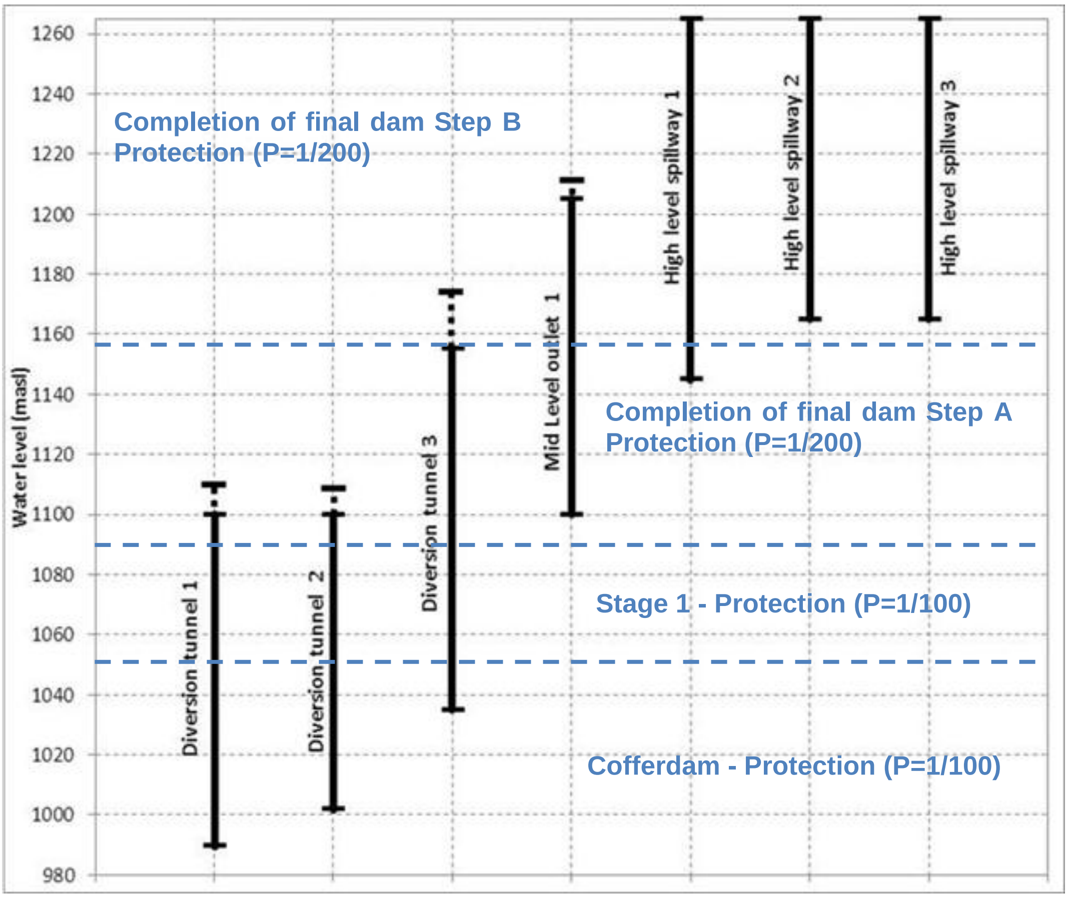


Figure 6.3 : FSL = 1255 masl - Diversion strucutres operating range

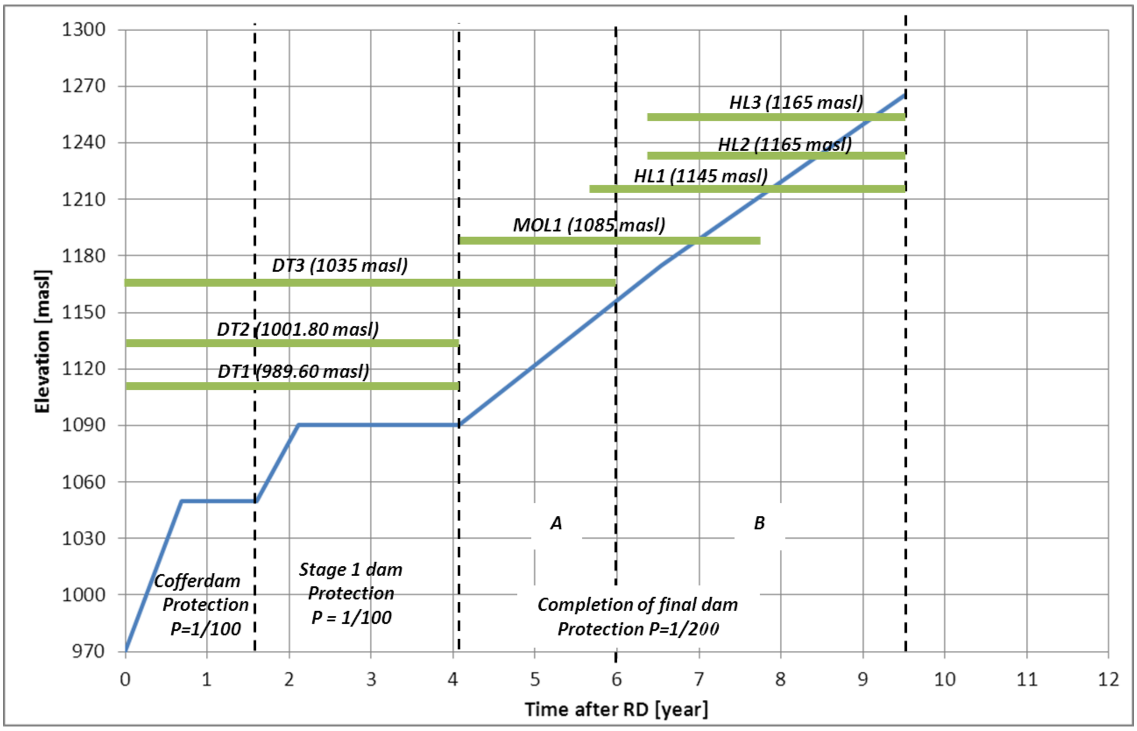


Figure 6.4 : FSL = 1255 masl - Diversion scheme along time

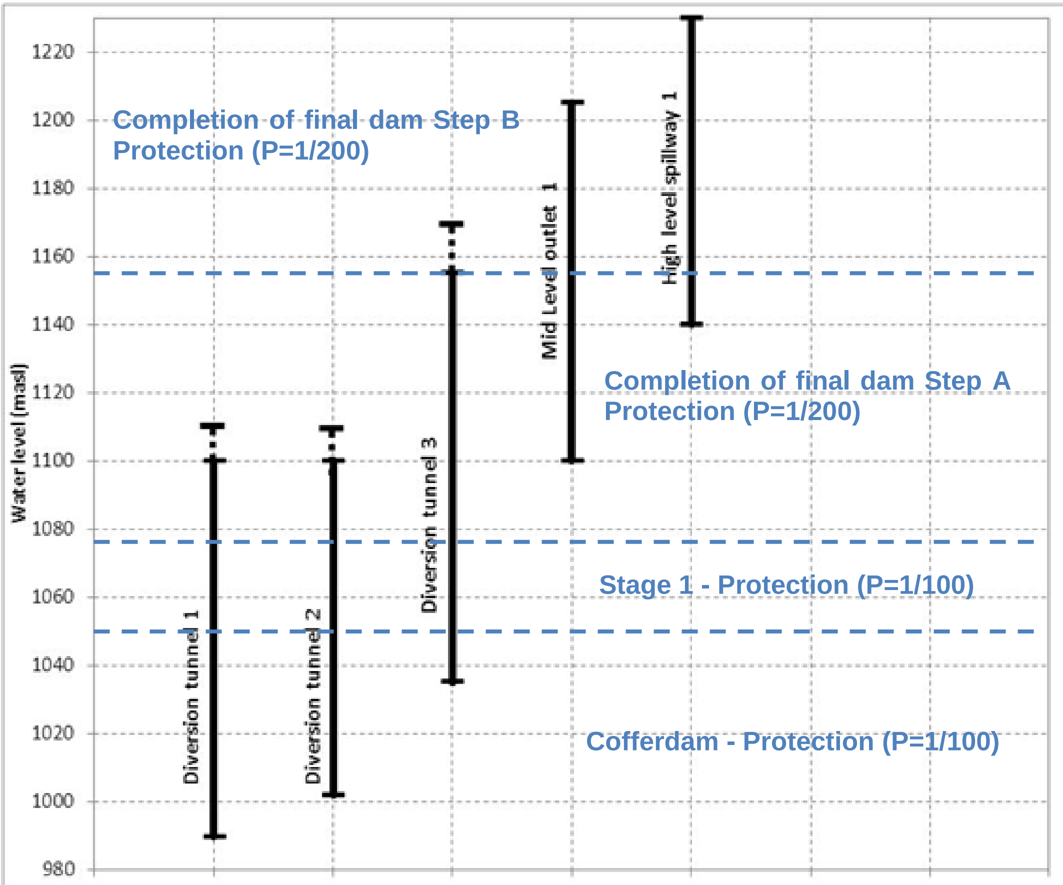


Figure 6.5 : FSL = 1220 masl - Diversion structures operating range

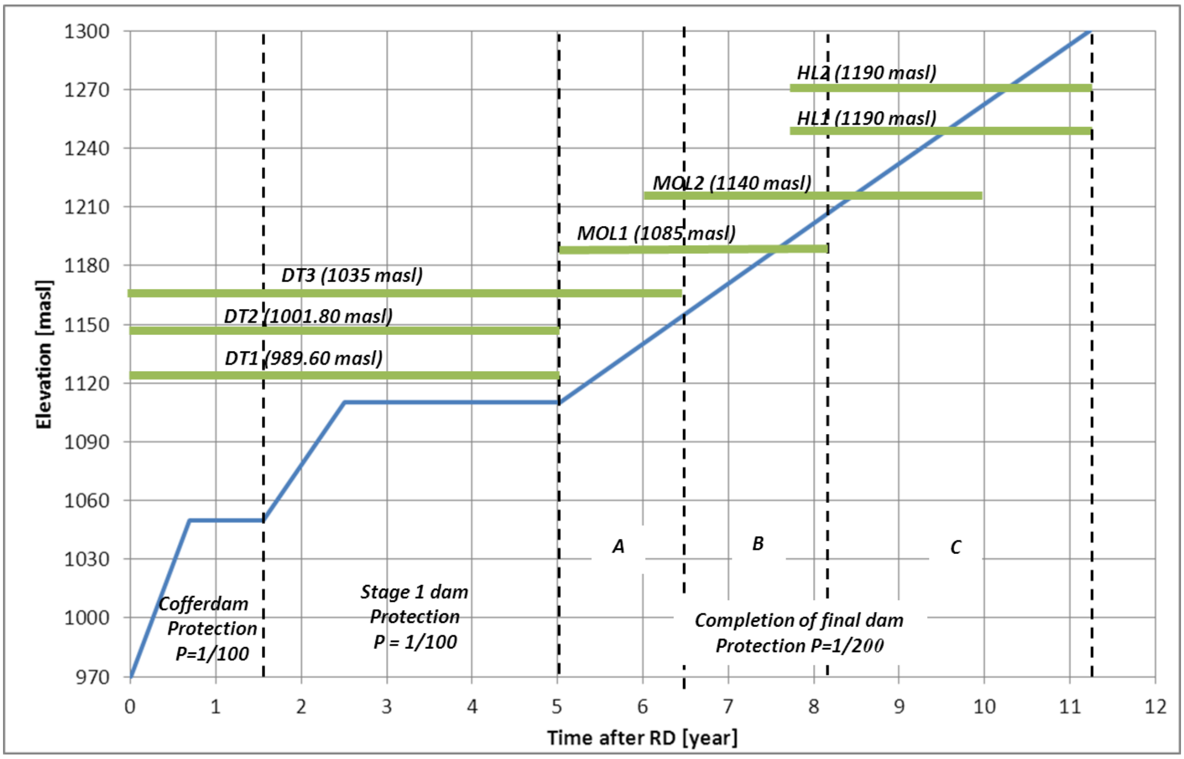


Figure 6.6 : FSL = 1220 masl - Diversion scheme along time

APPENDIX A – Tunnel Gates – Report on the Maximum Operating conditions

RISK ASSESSMENT ON THE DIVERSION TUNNEL GATES – AN OVERVIEW

There is sufficient evidence of failure and malfunction of high-head tunnel gates and hoists to be included in a risk assessment of a dam. Determination of reliability must include potential liability due to design, operation and maintenance, operator training, inspection and supervision and record keeping of incidents. Maintenance can be deficient or variable at different stations of the same authority, mainly because of lack of an adequate maintenance budget.

According to Lewin [1], there are a number of research papers on problems which have occurred at high-head tunnel gates. A comprehensive survey of the operation of high-head tunnel gates at 50 large dams was carried out in Rumania. Damage had occurred at 38 gate installations. 60% of the incidents and failures were due to vibration problems, including two structural failures, which occurred after 8 and 20 years operation. Four events of intake clogging made the high-head tunnels unavailable, and nine vibration problems were classified as serious. Incidents of inadvertent operation of gates under automatic control have also been reported in the literature.

Common cause failures, which affect the operation of a system, are the most serious risk. In electrical installations, redundancy is usually provided for transformers, mains switches and supply cables. Standby generating plant is almost invariably provided, either of the permanent or mobile type.

Little information is available about the effect on high-head tunnel gates due to earthquakes. Damage and disablement of gates following an earthquake are not the only factors to be considered; blocking of access to the installation due to a landslip or damage to roads can inhibit emergency work. Lateral movement of gates must be expected as consequence of a seismic tremor. The gates and the side embedded parts shall be designed to damp the lateral movement of the gate and to support the corresponding lateral forces caused by the earthquake.

For high-head radial gates and hoists, the main causes of faults are:

- lack or inadequacy of the aeration system
- faulty design of the radial gate top seals, which can cause leakage and unexpected uplift forces on the gate
- trunnions bearing problems (the most frequent source of faults)
- gate vibration
- hydraulic cylinder vibration
- cavitation and erosion on the steel lining and concrete exposed surfaces
- loosening of fixation bolts of gate seals and cylinder gaskets
- oil leakage in hydraulic cylinders
- limit switch malfunction
- ice problems (gate seal freezing, clogging of air-vents with ice, etc.)
- gate seal leakages
- failure of heating systems
- loss of communication links.

To these must be added:

- non-guidance of the gate in raised position

- clogging of the intake and silting of water operated gates.

Of lesser frequency are:

- control system malfunction
- uncontrolled gate descent due to hoist failure or malfunction.

From the point of view of design engineering, it is mandatory to have a careful project of all equipment (gates, winches, aeration system and tunnel steel linings), in order to avoid mistakes that may jeopardize operation. This requires:

- Effective participation of specialized engineers in the selection of equipment, definition of arrangement and details of the civil structures (gate shafts and slots, aeration systems) at all stages of the project
- Preparation of technical specifications and drawings for bidding through experienced professionals in this type of equipment
- Hiring gate manufacturers with proven experience in this type of equipment
- Complete review of the supplier’s detailed documents (calculation sheets, general and detail drawings, bills of materials) by a consulting firm with recognized expertise in this type of service
- Execution of gate model tests for knowledge of all hydraulic phenomena associated with the operation of the evacuating organs (pressure distribution, cavitation, vibration and erosion, air-vent capacity etc.)
- Monitoring all gate manufacturing phases through a company with recognized experience in this type of service and equipment
- Monitoring gate, hoists and steel lining erection through a company with recognized experience in this type of service and equipment.

Periodic inspection and maintenance of all equipment, including tunnels and steel linings, is mandatory. The Rogun evacuating organs should be designed with two gates in parallel in each tunnel in order to allow inspection and maintenance of one gate at time, in dry conditions.

A complete and adequate maintenance plan should be established for periodic inspection of the following points:

- Structure of gates and supports (warping, deformation, faulty welds, etc.)
- Corrosion protection (corrosion points, peeling, reduced thickness of paint layer etc.)
- Gate seals - replacement of aged, worn or broken parts
- Hydraulic cylinders (oil leaks, swings, abnormal noise, stability etc.)
- Gate seal bolts and nuts
- Trunnion bushings and hydraulic cylinder connections
- Cylinder gaskets and seals
- Maintenance of the cylinder power units for cleaning and replacement of filters and oil
- Electric cables, limit switches, and command and control elements.

The customer shall keep a complete record of incidents, including:

- power outages
- malfunctions
- oil leaks
- abnormal noises
- malfunction of command, control and protection devices
- malfunction of gate position indicator and limit switches.

All these problems will be exacerbated in case of operation of hydromechanical equipment for long periods of time. Also, the higher the water head is, the higher the risks are.

It becomes evident that the operation of high-head radial gates presents many risks, especially in case of extended duration as required for the Rogun's diversion tunnel gates (10-12 years).

In the technical literature there are few examples of radial gates subjected to exceptional heads. For example, the table 3.2 of the book "*Design of Hydraulic Gates"* [2] lists the largest high-head radial gates already made, and it stands only the Tarbela project in Pakistan, with four radial gates of 4.88m x 7.3m, subjected to a maximum design head of 135.6m. In the research conducted by the author, this is the only radial gate subjected to a head greater than 120m.

The aforementioned table also shows some examples of gates subjected to heads between 100m and 120m:

- Tweerivieren, 8.38m x 5.18m, head 103.48m;
- Toktogul, 5m x 6m, head 112.2m;
- Nurek, 5m x 6m, head 110m;
- Sayano-Sushenskaya, 5m x 5.5m, head 116.7m.

Thus, it is recommended:

a) to design the Rogun diversion tunnel gates for a maximum head of about 120-130m, in order to limit risks, hydraulic loads and flow velocities to acceptable values;
b) to design at least two independent tunnels, in order to allow inspection and maintenance of gates, hoists, tunnel and steel linings.

References

[1] Lewin, J., *Hazard and Reliability of Hydraulic Equipment for Dams,* The Prospect of Reservoirs in the 21st. Century, British Dam Society, edited by Paul Tedd, 1998.

[2] Erbisti, P. C. F., *Design of Hydraulic Gates*, A. A. Balkema Publishers, Lisse, The Netherlands, 2003.