4

Civil Engineering Design

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4. Civil Engineering Design

4.1 General

The feasibility layout and design for the Madian Hydropower Project comprises the following main components:

- Concrete weir structure with gated spillway and flushing structure
- Power intake on left bank adjacent to weir structure with raking machine
- Desanding facilities
- Power waterways consisting of headrace tunnel, pressure shaft, pressure tunnel, manifold, tailrace and power outlet
- Powerhouse with switchyard / transformer cavern
- Diversion works consisting of upstream and downstream cofferdam and diversion tunnel
- Access roads, permanent and temporary camps
- Dumping sites for deposition of surplus excavation material

4.2 Design Criteria

For the feasibility design the following hydraulic and civil design criteria have been established in co-ordination with the Project Sponsor:

4.2.1 General Design Criteria

The properties of water in the Swat River were determined by in the period 2006 to 2007 at Kedam gauging station by the Consultant as follows:

Variation	Temperature ° C	Density kg/m³	Kinematic Viscosity 10^{-6} m^2 / s
Minimum	0	999.84	1.79
Average	10	999.70	1.31
Maximum	14	999.24	1.17

Table 4.1: Properties of Water at Project Site

According to the elevation of the project area between 1495 m at the weir structure and 1340 m at the power outlet the gravity acceleration is applied as follows:

at mean sea level	9.810 m/s²
at weir site (1495)	9.795 m/s²
at powerhouse site (1340)	9.796 m/s²

Accordingly a gravitational acceleration of 9.8 m/s² is applied to the feasibility design.

4.2.2 Design Floods

Design floods are defined for the following purposes and structures:

Weir – Spillway	Design Flood	(according to ICOLD)
	Safety Check Flood	(according to ICOLD)
Powerhouse	Design Flood	
	Design Discharge at Rated Operation Conditions	
Diversion Floods	Design Flood	

4.2.2.1 Spillway Design Floods

In view of the size of the weir structure and consequences of potential failure the following design floods are considered adequate as a conservative approach in accordance with the recommendations of ICOLD-Bulletin 82: "*Selection of Design Flood – Current Methods*".

For details regarding the methodology applied to the calculation of the design floods reference is given to Chapter 3 – Hydrology:

Design Flood:	$HQ_{1,000}$ = 1450 m³/s	with one gate malfunctioning and normal freeboard (1.5 m)
Safety Check Flood	$HQ_{10,000}$ = 2002 m³/s	all gates open and minimum freeboard (1.0 m)

In the event of the design flood the reservoir is assumed to be at normal operation water level.

The selection of the spillway crest elevation shall be made in a way that efficient evacuation of bed load is ensured. The ratio of maximum head to spillway design head shall not exceed 1.3 to limit negative pressure on the ogee according to ASCE Design Guidelines.

4.2.2.2 Powerhouse Operation Design Flood

The powerhouse shall be operational up to the powerhouse design flood which is defined as the flood with a return period of 1000 years. In the event of higher floods the gates at the power intake shall be closed. The duration of periods with high flows in Swat River is rather short (approximately one day, see Figure 4.1). It is considered recommendable and economically justified to shutdown the turbine units in the event of extraordinary floods with return periods exceeding 100 years in view of the expected high sediment concentrations.

Design Flood:	HQ1,000 = 1,785 m³/s
Recommended Max.Operation Flood	HQ 100 = 1,095 m³/s

In the event of the Maximum Operation Flood it is recommended to draw down the reservoir level to reduce sedimentation of the head pond and enable a certain reservoir flushing.

4.2.2.3 Summary of Operation Design Floods

The powerhouse shall be operational up to the powerhouse design flood which is defined as the flood with a return period of 1000 years. For the spillway the selected design flood has a return period of 1000 years and the Safety Check Flood of 10,000 years (see Table 4.2).

Site	Maximum Flood (m^3/s)		
	100-y	**1,000-y**	**10,000-y**
Weir	860	1,450	2,002
Power House	1,095	1,785	2,405

Table 4.2: Estimated Maximum Rainfall Generated Floods

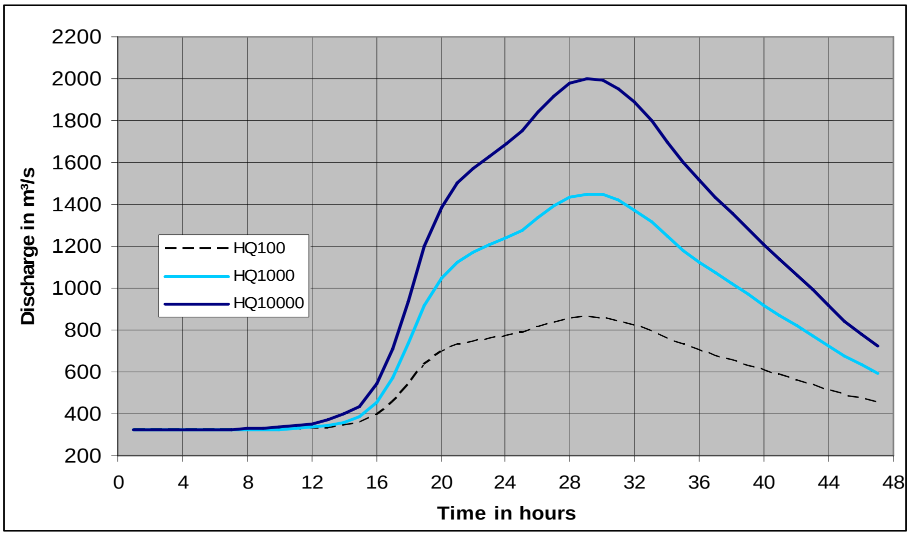

Figure 4.1: Hydrograph for Design Floods (at weir site) with Return Periods of 100; 1,000; and 10,000 years

4.2.2.4 River Diversion Design Flood

The selection of the design flood is to be made in conjunction with the principle of river diversion applicable to the prevailing topographic, hydrological and geological and other related conditions. The magnitude of the diversion design flood shall consider adequately potential risks and damages for life and goods which may result in the event of a flood larger than the design flood.

At the weir site the Swat valley is rather narrow and little space is available for excavating the construction pit to the foundation depth of the weir structure. This circumstance governs the selection of the principle for river diversion during weir construction .

Therefore, a conventional river diversion concept is applied with a left bank diversion tunnel, upstream and downstream cofferdam instead of a concept with staged river diversion.

The estimated construction period for the weir including stilling basin and power intake is 3 years. In accordance with common design practice a flood with a return period of 20 years is selected as diversion design flood for the weir and the powerhouse construction pit.

Diversion Design Flood	Weir	HQ_{20} = 656 m³/s
Diversion Design Flood	Powerhouse	HQ_{20} = 731 m³/s

Optimization of the diversion works shall be performed with the constraint that the upstream cofferdam shall not exceed the elevation of the Kedam – Kalam road which represents the only access to the weir site and need to be maintained open for public transport during the entire construction period. In the event of the diversion design flood the road shall not be overtopped.

In view of the height of the cofferdams above riverbed of approx. 18 m, a freeboard of 1.5 m is considered adequate in the event of the design flood.

4.2.3 Spillway Design

For the design of the spillway the design floods apply as given in chapter 4.2.2 Design Floods as follows:

Design Flood: $HQ_{1,000}$ = 1,450 m^3/s with one gate malfunctioning and a freeboard of 1.5 m which corresponds to a reservoir water level of 1494.5 m asl (SoP)

Safety Check Flood: HQ_{10000} = 2,002 m^3/s with all gates open and a minimum freeboard of 1.0 m, which corresponds to a maximum reservoir water level of 1495.0 m asl (SoP)

The spillway shall be a gated structure to be able to control the reservoir at normal operation water level of 1494 m asl (SoP) and to enable reservoir flushing when required.

4.2.3.1 Design of Spillway Structure

According to ASCE design guide lines the shape of the spillway ogee can be defined for a design head less than the maximum head H_0. Selection of the design head shall account for a possible additional head of up to 30 % in case of the Safety Check Flood, however, sub-atmospheric pressure on the ogee shall not fall below -4 m.

The design of the spillway downstream of the crest shall comply with international standards such as e.g. ASCE (Design of Small Dams). The corresponding design parameters are given in Annex A-4.1. The thickness of piers shall be selected to safely transfer forces in the main dam body.

The pier shape shall be selected in a way to avoid separation of flow and to guarantee minimum head losses.

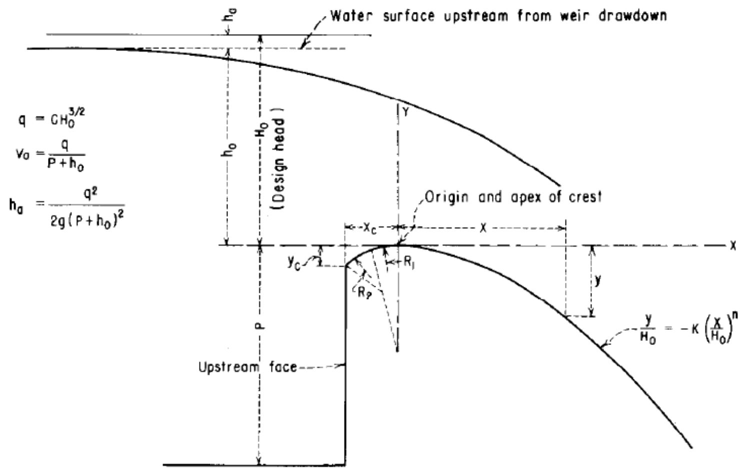

Figure 4.2 Design Chart for Spillway Ogee

The ogee crest structure is designed applying WES standard profile as defined by the Hydraulic Design Charts by USACE for the equation downstream of the crest axis.

$$\frac{Y}{H_d} = K\left(\frac{X}{H_d}\right)^n \qquad (1)$$

Where,

X	=	horizontal distance in downstream direction
Y	=	vertical distance from crest level
H_d	=	Spillway Design Head
K,n	=	Factors defining the nappe-shape of crest
K	=	Variable depending upon upstream slope, 0.5
n	=	Variable depending upon upstream slope, 1.835 in this case

By putting the variables in above equation; $X^{1.835} = 2.0 \times H_d^{0.835} \times Y$

The shape of the spillway pier upstream and downstream faces is to be selected to guarantee a high discharge capacity and limitation of the height of downstream rooster (shock) waves. For the design of the spillway crest structure and calculation of the discharge capacity the following effect shall be taken adequately into account:

- effect of head on overfall coefficient and hydraulic effective width
- effect of abutment and pier shape on hydraulic effective width

The spillway discharge capacity is calculated applying the following standard formula:

$$Q = CB'\sqrt{2gH_e{}^3} \qquad (2)$$

Where,

C	=	Discharge coefficient
B'	=	Effective width
H_e	=	Head over the crest

The effect of piers and abutments on the hydraulically effective crest width and thus on the spillway discharge capacity is estimated using the following relationship:

$$B' = B - 2(n \times k_p + k_a) \times H_e \qquad (3)$$

Where,	B'	=	Effective width
	B	=	Clear waterway width
	n	=	Number of piers
	k_p	=	Pier contraction coefficient
	k_a	=	Abutment contraction coefficient
	H_e	=	Head above crest level

The piers and abutments cause side contraction of the overflow, thus reducing the effective width of the spillway bay, also depending on the head over the crest. The contraction coefficients K_p and K_a are affected by the shape of the pier nose and abutment shape respectively. For Madian HPP spillway K_p=0.01 and K_a=0.0667 apply.

Provisions shall be made to enable flushing of debris that may accumulate at the power intake by arranging a flap gate on top of the tainter gate.

At the weir structure a certain minimum flow needs to be maintained in the Swat River for ecological reasons (see Section 03 Hydrology). At low river flows these required releases will be largely discharged through a small Francis turbine unit arranged at the weir structure. With increasing river flow and when the turbine unit is not operational, the flap gate will be used for the fine regulation of releases at the weir site. The turbine unit will generate electricity for the station’s own use and feed into the existing 11 kV transmission line to supply excess energy to the nearby villages.

Provisions shall be made that reservoir flushing can be conducted to remove large depositions of silt and sand fractions during the high flow season. Reservoir flushing shall be conducted as joint operation of the flushing outlets and the spillway gates.

4.2.3.2 Design of Stilling Basin

The riverbed of Swat River consists of large scale boulders and at selected locations rock is outcropping. From the geotechnical field investigations it is known that the thickness of alluvial material may exceed 20 m in the riverbed. In view of the fact that the right river bank consists largely of erodible moraine deposits where the Kedam – Kalam road and various houses are located downstream of the weir site, a stilling basin shall be arranged. The stilling basin is to be designed to dissipate largely the hydraulic energy generated by the drop at the spillway and thus avoid excessive riverbed and bank erosion downstream of the weir structure.

To ensure that the hydraulic jump is maintained within the stilling basin for the entire range of river discharges the elevation of the end sill is selected with an additional safety factor of 1.05.

The length of the stilling basin shall be designed according to common design approaches such as that reported by BLIND “Wasserbauwerke aus Beton”. Accordingly the minimum length of the stilling basin is 5 times the difference of the conjugated depth:

$$L_{SB} = 5 \text{ x } (h_{2req} – h_1) \quad (4)$$

Where,

L_{SB}	=	Length of stilling basin m
h_1	=	Conjugated depth at begin of hydraulic jump
h_2	=	Conjugated depth at end of hydraulic jump

4.2.4 Vortex Prevention at Power Intake

In order to avoid vortex formation at intakes the submergence must be sufficient. Gordon [Water Power, April 1970] has assembled comprehensive field data and prepared an empirical formula according to the definition given in Figure 4.3 for the required minimum submergence.

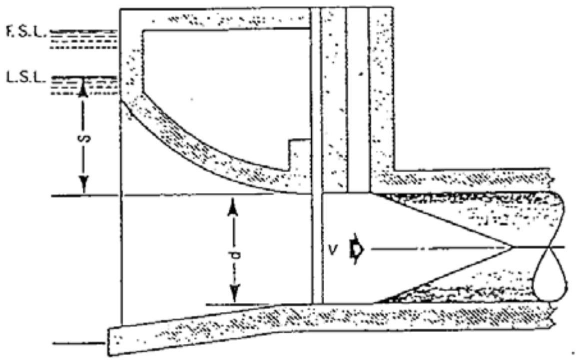

Figure 4.3 General Configuration of Power Intake

Gordon’s formula defines: $s = 0.72 \, . \, v\sqrt{d}$ (5)

Where:

s	=	submergence, (m)
d	=	diameter of tunnel, (m)
v	=	tunnel flow velocity, (m/s)

4.2.5 Head Losses in Waterways

Along the waterways the flow passes conduits of different size and shape and stream lines are deflected by bends, dividing or combining flow, such as at trash racks, branches, bends, enlargements and contractions. These local changes of the stream line direction cause an addition head loss to that resulting from frictional resistance. All head losses involved in each conduit system are individually evaluated according to methods and formulas described subsequently.

4.2.5.1 Friction Caused Head Losses

The equation used for the calculation of friction losses in the conduit is the Darcy-Weisbach formula:

$$Hf = f \cdot \frac{L}{D} \cdot \frac{v^2}{2g} \qquad (6)$$

Where:

H_f	=	Head loss due to friction, (m)
f	=	Friction factor
L	=	Length of conduit or section (m)
D	=	Diameter of conduit (m)
v	=	Velocity of flow, (m/s)
g	=	Gravity acceleration constant, (m/s^2)

The head losses due to friction are determined by means of the Prandtl-Colebrook formula

$$\frac{1}{\sqrt{f}} = -2\log\left(\frac{2.51}{Re\sqrt{f}} + \frac{e/D}{3.71}\right) \qquad (7)$$

Where: Re	= $\frac{v.D}{\nu}$	Reynolds number
e	=	Equivalent sand roughness , (m)
ν	=	Kinematic viscosity, (m^2/s), see Table 4.1

The equivalent wall roughness varies from one type of tunnel lining to the other within the limits given in the table in Annex A-4.2. For design purposes the value shall be applied that results (within the given range) in the more critical condition. Energy calculation shall be based on mean roughness coefficients.

4.2.5.2 Local Hydraulic Head Losses

Intake Loss

The entrance loss for a pipe or tunnel is defined

$$H_L = K_E \cdot \frac{v^2}{2g} \qquad (8)$$

Where:

K_E	=	entrance loss coefficient
v	=	velocity in tunnel or pipe, (m/s)

For the level of feasibility design a single head loss coefficients is applied to the entire intake structure taking into account the combined effect of entrance, trashrack, gate slots, gradual constriction and expansion of flow as given by the design charts according to ASCE.

Expansion Loss

The sudden expansion loss is described by Borda's Formula

$$H_L = \left(1 - \frac{A_2}{A_1}\right)^2 \cdot \frac{v_2^2}{2g} \qquad (9)$$

Where

A_1	=	area of cross section flow incoming from, (m^2)
A_2	=	area of cross section flow is going to, (m^2)
v_2	=	velocity in cross section 2, (m/s)

Bend Losses

The head loss produced by a bend with circular cross section is

$$H_L = K_b \frac{v^2}{2g} \qquad (10)$$

The head loss coefficient K_b shall be determined based on well established references such as the figure given in Annex A-4.2. In case of subsequent bends, an adequate reduction of head loss coefficients shall be made to account for bend-bend interaction.

4.2.6 Hydraulic Surge Tank Design

Surge tanks are required in order to facilitate governing and fast start up of turbines fed by tunnels or penstocks. A first general criterion to conclude the necessity of a surge tank is:

$$T_w = \frac{L \cdot v}{g \cdot H} > 3.0s \qquad (11)$$

Where:

T_w	=	starting time (if grid stabilization is required 2.5 s), (s)
v	=	flow velocity (m/s)
g	=	gravity acceleration constant

For stability of governing a minimum cross section area (THOMA-Criterion) of the surge tank is required

$$A_{s\,min} = \frac{L_H \cdot A_H}{H_{L\,Min}\,(H - H_{L\,min})} \cdot \frac{v_H^2}{2g} \qquad (12)$$

Where

L_H	=	length of headrace tunnel, (m)
A_H	=	area of headrace tunnel, (m²)
$H_{L\,min}$	=	minimum head loss in tunnel, (m)
H	=	minimum gross head, (m)
v_H	=	velocity in headrace tunnel, (m/s)

In order to sufficiently dampen possible oscillations a safety factor n is applied to the minimum surge tank area varying in the range n = 1.3 … 1.6: The surge tank may be provided with an orifice at its bottom. The most common type is a cylindrical surge tank. Lower or upper expansion chambers may be applied. In the event that an orifice is applied its area shall not be less 50 per cent of the pressure shaft or penstock in order to facilitate total pressure wave reflection.

4.2.6.1 Load Cases

The load cases producing maximum up and down-surges are described in the following for three turbine units for a surge tank at the end of a headrace tunnel. In the event of a different number of turbines they have to be modified accordingly.

Load Case I: Maximum Downsurge
Full load acceptance of one turbine is followed by the other turbines after a time interval to be defined. For this load case the minimum reservoir level and the maximum head losses have to be considered. The minimum downsurge occurs approximately at a time interval equal to T/2 where T is the period of the surge oscillation.

$$T = 2\,\pi \,.\sqrt{\frac{L\,.\,A_s}{g\,.\,A_H}} \qquad (13)$$

Simulation of various time intervals is required in order to detect the most unfavourable case.

Load Case II: Maximum Downsurge
The maximum downsurge may alternatively occur for a load reduction by 50 per cent and subsequent complete load acceptance. For this load case the minimum reservoir level and the maximum head losses have to be considered, too.

Load Case III: Maximum Upsurge
The most unfavourable load case with regard to the upsurge is a total load rejection subsequent to Load Case 1 or Load Case 2. The load case producing the higher discharge in the tunnel prior to rejection is the relevant one. For load case 3 the maximum water level in the reservoir and the minimum head losses have to be considered.

4.2.6.2 Analysis of Surge Oscillation

The mass oscillation in the system comprising the tunnel and the surge tank are described by the equation of motion; equation (14b) considers the continuity

$$\frac{dQ_T}{dt} \pm g\,.\frac{A_T}{L_T}z + \frac{1+K_T}{2\,L_T\,A_T}\,Q_T\,.ABS\,(Q_T) \pm \frac{dz}{dt}\,.ABS\left(\frac{dz}{dt}\right).K_0\,.\frac{A_s^2}{A_0^2}\,.\frac{g\,.\,A_T}{L_T} = 0 \qquad (14a)$$

$$\pm\left(Q_T - Q_p\right) - A_s\,.\frac{dz}{dt} = 0 \qquad (14b)$$

Where:

Q_t	=	discharge in tunnel, (m³/s)
t	=	time, (s)
A_T	=	cross section area of tunnel, (m²)
L_T	=	length of tunnel, (m)
z	=	water level in surge tank referring to reservoir level, (m)
K_T	=	head loss coefficient for tunnel
K_0	=	head loss coefficient of orifice at surge tank
A_0	=	cross section area of orifice, (m²)
A_s	=	cross section area of surge tank, (m²)
Q_p	=	Turbine discharge, (m³/s)

Since all above mentioned load cases are combined load cases, the most unfavourable instants for load acceptance or rejection have to be evaluated by trial and error procedure. As a justified simplification the steel lining of the pressure shaft and pressure tunnel is assumed to adsorb the full internal pressure (including water hammer) and no supporting effect of the surrounding rock is considered. If the results of rock testing show adequate rock parameters, the thickness of steel lining may be slightly reduced in the detailed design.

4.2.7 Hydraulic Design of Desanding Facilities

Settling basins are required if the river flow contains high concentrations of suspended sediment which may cause severe damage to the turbine runners.

For the design of a desander the following criteria have to be considered.

<u>Design Grain Diameter:</u>	Critical Sediment Grain Size, grain size to be removed to 95 per cent or more			
	-	Head	20 - 50 m	D = 0.30 mm
	-	Head	50 - 100 m	D = 0.25 mm
	-	Head	100 - 300 m	D = 0.20 mm

<u>Settling Basin Cross Section</u>
The removal rate of fine sediments depends largely on the settling velocity of the particles, the velocity of flow and the depth of the basin. The width to depth ratio of a basin shall be below unit; recommended: W:D = 1:1.2 to 1.5

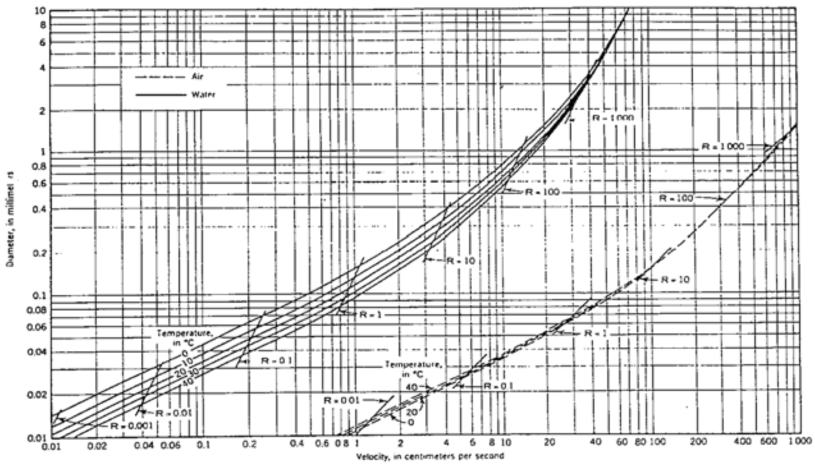

Figure 4.4: Settling Velocity of Grain Particles

The cross section required to prevent re-suspension of sediments area can be determined applying the following empirical relationship:

$$A = \frac{Q}{(0.44\sqrt{d})} \qquad (15)$$

Where:

Q = design discharge of basin, (m^3/s)
d = grain size, (mm)

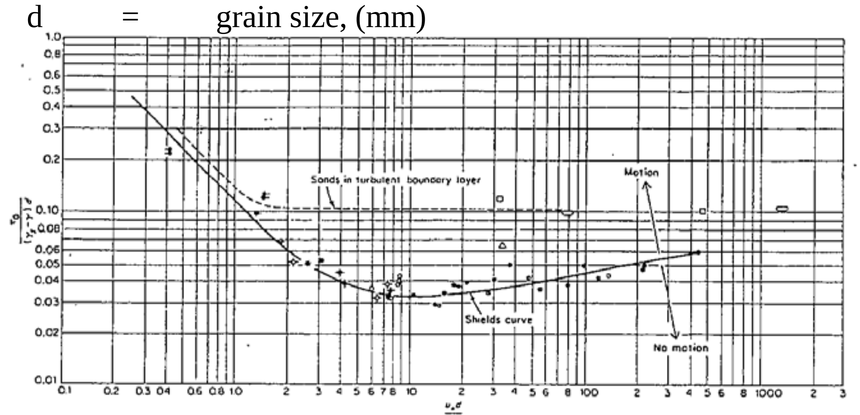

Figure 4.5: Mobilization/Resuspension Criterion after SHIELDS

4.2.7.1 Desander Cavern Design Guidelines

Transition – Entrance of Flow
The flow should enter the basin smoothly, abrupt transitions from the intake channel to the basin shall be largely avoided. Transitions in width and depth shall be gradual not exceeding 10 degrees.

Deposition of Sediment
The sediment removed from the flow is deposited on the bottom of the basin. For this purpose a corresponding storage capacity has to be provided. In order to facilitate adequate flushing or sluicing of deposits the walls shall be steeply sloped with an angle of not less than 40 degrees (H:V = 1.2 : 1.0)

Removal Rates
The required length of a settling basin depends on the fall velocity of the sediment particles, the velocity and the depth of flow. In order to avoid extremely long settling basins the ratio of width to depth shall be smaller than unity. For evaluating of the removal rate Camp's method will be used.

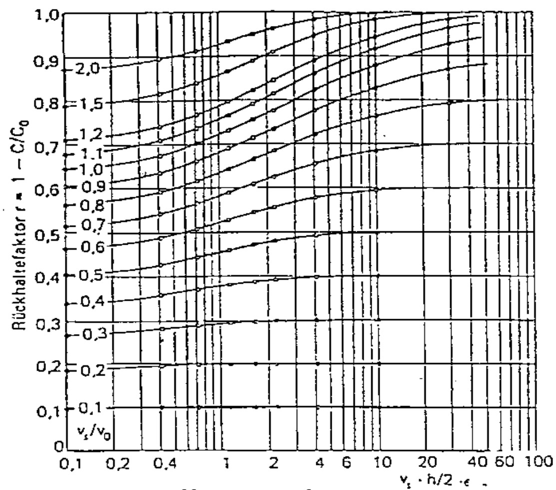

Figure 4.6: Trapping efficiency after Camp

Flushing or Sluicing of Deposits
For removal of the deposited sediments generally different methods are used. Only the periodical flushing has practically proven to be sufficiently reliable. The disadvantages of the continuous sluicing are the very complex waterways and water-sediment mixture is quite non-uniform and tends to produce clogging in pipes.

Generally physical model test are required for verification and optimization of the desander and sluicing arrangements. These model tests shall be specified and performed during the tender/detailed design stage.

Reservoir Sedimentation
At rivers with sediment inflow reservoirs created by dams or weir structures are subject to a certain degree to sedimentation due to deposition of suspended sediment and bed load. The extent of sedimentation depends on the type and quantities of the sediment and the type of the reservoir, e.g. suspended sediment consisting to a high percentage of silt and clay will

cause a moderate sedimentation in a small and narrow reservoir. However, bed load will be largely trapped even in a small reservoir unless the sediment may pass the spillway crest. For evaluation of the volume of sediment trapped and assessment of the corresponding loss of storage in large reservoirs, empirical methods or numerical modelling techniques may be applied. For small reservoirs with a correspondingly small trapping efficiency, reservoir sedimentation is mainly due to bed load material.

The elevation of the intake's invert has to be arranged safely above the top of the deposition at the weir / dam. According to experience from prototype hydropower plant operation (Marsyangdi, Nepal) for small and narrow reservoirs with steep river bed gradients (> 2 %), sediments can be largely removed by reservoir flushing. For this purpose flushing gates have shall be arranged at a possible low elevation.

4.2.8 Stage - Discharge Relationships

For the design of weir structure, stilling basin and power outlet, stage-discharge relationships are required to provide the relevant water levels for a safe design and adequate operation of the hydraulic structures.

For this purpose two backwater models were setup applying the HEC-RAS software. For development of the numerical models the Consultant conducted a comprehensive topographic survey of river cross-sections.
(for reference see Report on Topographic Survey, Volume V and Section 3.3 of this main report).

Kedam gauging station on Swat River provides comprehensive data on discharge measurements (see Volume IV of this Feasibility Report) and the corresponding stage-discharge relationship. Due to its location downstream of the proposed weir site and upstream of the power outlet, it permits the calibration of riverbed roughness for a range of river discharges from approximately 20 to 500 m³/s. Conducting a number of HEC-RAS simulation runs for various river discharges the corresponding riverbed roughness coefficient was calibrated reproducing the recorded water levels. It can be reasonably assumed that the roughness characteristics of Swat River at Kedam gauging station is similar to that at the weir and power outlet site. Accordingly the calibrated roughness coefficients were used to setup stage-discharge relationships for these locations along Swat River.

4.2.8.1 Powerhouse Tailwater Rating Curve

The selected site of the power outlet is located on the left bank of Swat River at the end of the U-shaped valley some 1.2 km upstream of the Bridge over Swat River at the northern outskirt of Madian town.

The Consultant elaborated a backwater model using the a HEC-RAS software and altogether 26 river cross-sections which cover a river reach of 1.53 km length. The cross-sections were surveyed in February 2007 and connected to SOP system elevations in September 2007.

As the results of various model runs and applying the set of calibrated riverbed roughness coefficients, the stage discharge relationship (16) as shown in Figure 4.7 was established.

$$
\begin{aligned}
H_{tw} = & - 1.4063E\text{-}18 \cdot Q^{6} \\
& + 10.686E\text{-}15 \cdot Q^{5} \\
& - 3.1659\,E\text{-}11 \cdot Q^{4} \\
& + 4.6128E\text{-}8 \cdot Q^{3} \\
& - 3.4568E\text{-}5 \cdot Q^{2} \\
& + 1.5517E\text{-}2 \cdot Q \\
& + 1339.6
\end{aligned}
$$

Stage-Discharge Relationship at power outlet

(16)

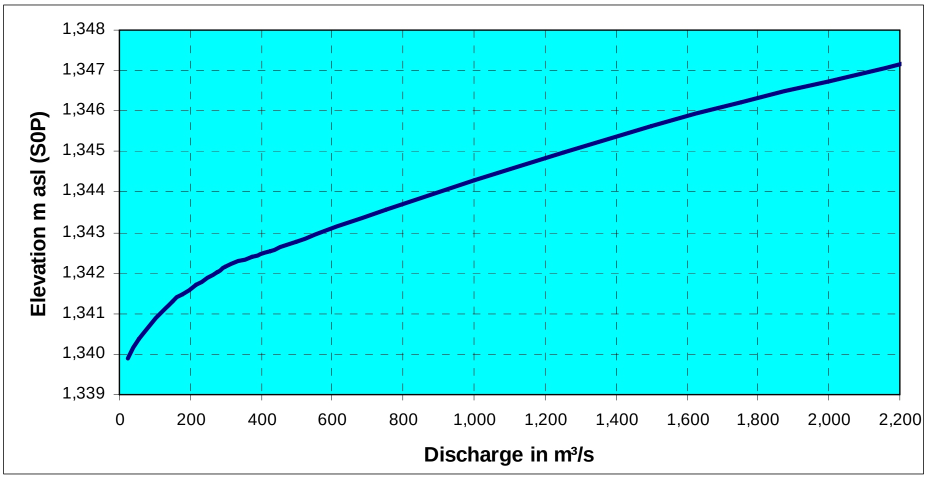

Figure 4.7: Tailwater Rating curve for Selected Powerhouse Site

4.2.8.2 Weir – Spillway Rating Curve

The selected site of the weir axis is located 90 m upstream of the confluence of Swat River and Kedam Nullah. Detailed knowledge on the hydraulic conditions downstream of the stilling basin and flushing outlet is essential for the design of the weir structure.

The Consultant elaborated a backwater model using the a HEC-RAS model comprising altogether 13 river cross-sections which cover a river reach of 1.2 km length. The cross-sections were surveyed in March 2007 and January 2008 and adjusted to SOP system of elevations. As the results of various model runs and applying the set of calibrated riverbed roughness coefficients, the stage discharge relationship (17) as shown in Figure 4.8 was established for the location of the end sill of the stilling basin.

$$
\begin{aligned}
H_{tw} = & - 1.313E\text{-}18 \cdot Q^{6} \\
& + 9.8435E\text{-}14 \cdot Q^{5} \\
& - 2.8892\,E\text{-}11 \cdot Q^{4} \\
& + 4.2453E\text{-}8 \cdot Q^{3} \\
& - 3.4045E\text{-}5 \cdot Q^{2} \\
& + 1.85795E\text{-}2 \cdot Q \\
& + 1476.7
\end{aligned}
$$

Stage-Discharge Relationship at stilling basin end sill

(17)

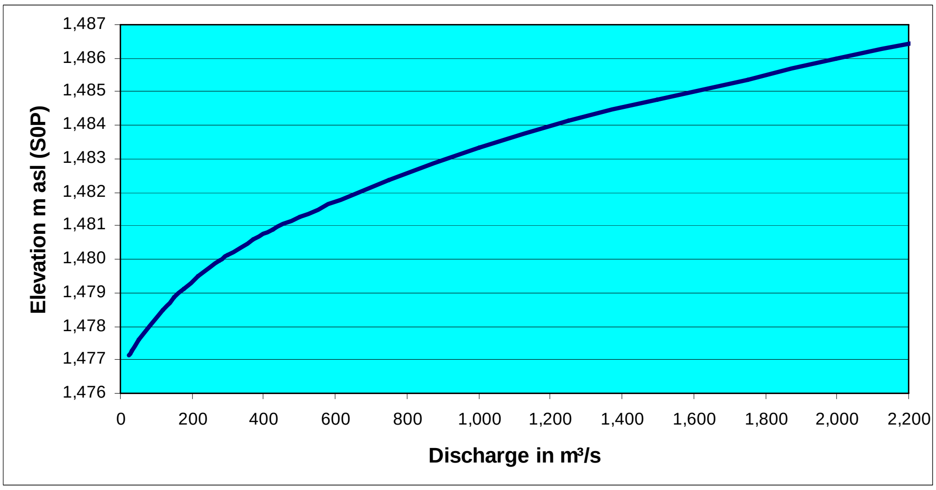

Figure 4.8: Tailwater Rating curve for Weir Site Stilling Basin

4.3 Design of the Weir Structure

4.3.1 General Design Concept

The weir axis was selected according to the prevailing geological, topographic and design boundary conditions. It is located approximately 90 m upstream of the confluence of Kedam Nullah and Swat river at Kedam Village, approximately 14.2 km upstream of the town of Madian.

The normal operation water level of 1494 m asl is based on the definition of PPIB to ensure the coordinated development of the Madian HPP and the upstream located Asrit-Kedam HPP on Swat River.

Accordingly the weir structure has a height of 19 m above river bed and will create a reservoir with a length of approximately 1.46 km and a total volume of 0.48 million m³ as shown in Figure 4.9.

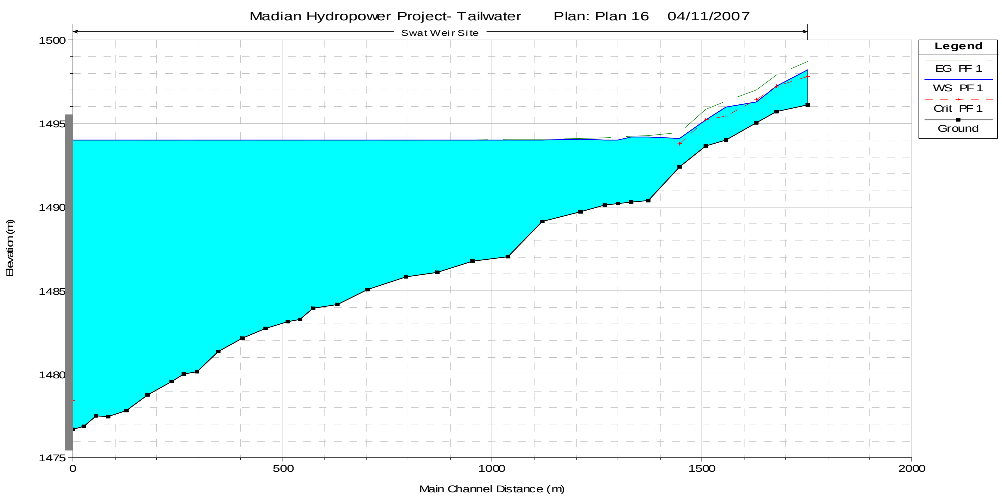

Figure 4.9 Profile through the Madian HPP Reservoir

At the weir part of the river flow is diverted to the power waterway system at the left bank power intake. Some 12 km further downstream the power house is located where the water is returned to Swat River. In order to maintain the power intake free of sediments, in particular of bed load that may accumulate upstream of the weir structure, a flushing structure is arranged in the left part of the weir structure close to the power intake.

The concrete weir structure across the Swat River has a crest length of approximately 65 m. The presently assumed foundation level varies between 1468 and 1470 m asl. On top of the weir structure a road is planned to provide access to the intake structure and the control structures.
The overflow spillway of the weir is arranged at the location of the original riverbed which has a width of approximately 25 m at the weir axis. The spillway is equipped with three hydraulically operated tainter gates. One set of stop logs is provided for maintenance and erection.

The most left bank tainter gate is equipped with a fish belly flap on top for fine regulation of the flow and for flushing of floating debris. The inclined weir ogee is followed by a stilling basin at its end. The overflow spillway is designed to pass a safety check flood of 2002 m^3/s. Each spillway bay can be closed with stop logs for repair and maintenance works.

For the anticipated run-off river operation the reservoir level remains constant at elevation 1494 m asl (SoP) throughout the year. At periods of extremely low river flow pondage operation might be permitted and limited to a maximum draw down of 2.0 m from elevation 1494 to 1492 m asl.

From the total storage capacity of 0.48 million m^3 a volume of 126,000 m^3 would be available for such pondage operation. This storage capacity has been selected to improve the conditions for turbine operation (and turbine efficiency) at times of extremely low river flow.

Max. water level	1,494	m asl (SoP)
Total storage volume	480,000	m^3
Length of the reservoir	1.46	km

Considerable sediment concentrations may occur during the high flow season. For this reason desanding facilities are required to protect the electro-mechanical equipment, in particular the guide vanes and the turbine runner from excessive wear and tear. At the weir site area no space is available for arranging desanding facilities in open air. Therefore, under-ground desanding caverns are selected for the Madian HPP. For the required dimensions of the desander caverns the head is not available at the weir site to achieve evacuation of the sediment – water mixture be gravity. The optimum location of the desander caverns was found some 2.1 km further downstream upstream of Ashkon Nullah.

Removal of gravel and sand that may deposit in front of the power intake, is required and flushing facilities are, therefore, required between power intake and the gated weir section.

The flushing (or sluicing) gates discharge into a chute separated from the stilling basin to allow for its maintenance and repair while the stilling basin is in operation.

During the high flow snowmelt season (May to September) the spillway gates will be operated partially open and significant amounts of suspended sediments and bed load may be discharged at the weir site. A certain reservoir sedimentation can, however, not be avoided. For reservoir flushing the spillway gates may be opened and in combined operation with the flushing/sluicing outlet and the reservoir level is gradually drawn down according to their combined discharge capacity and the prevailing river flow. However, reservoir flushing shall be dealt with care to avoid destabilization of reservoir slopes.

4.3.2 Hydraulic Design of Spillway and Stilling Basin

4.3.2.1 Design of Spillway Ogee Structure

In accordance with the design criteria the design head was selected 30 % smaller than the head in the event of the Safety Check Flood.

Maximum reservoir level:	1494.5 m asl
Spillway crest elevation:	1482.5 m asl
Maximum head	12.0 m
Design head 12.0 / 1.3 =	9.2 m

The ogee crest structure is designed applying WES standard profile as defined by the Hydraulic Design Charts by USACE (see Annex A-4.1) for the equation downstream of the crest axis with the design head H_d = 9.2 m. The thickness of piers was selected to be 3.0 m to safely transfer forces in the main dam body. A rounded pier shape is applied to avoid separation of flow and high discharge capacity (see Plate 13, Volume VII).

The dimensions of the spillway gates were selected as follows:

Number of Gates	3
Width x Height	7.6 x 12.0 m

4.3.2.2 Spillway Discharge Capacity

The spillway structure was designed having three identical bays with a crest elevation of 1482.5 m asl and a head of 12.0 m. The crest elevation was selected to be 0.5 m below the invert of the power intake to avoid intrusion of bed load into the power intakes during the high flow period (when bed load is passing the spillway crest). The spillway discharge capacity is calculated applying the standard formula (2) given in the Hydraulic Design Criteria for two scenarios, all gates open and n-1 gate open with or without the flushing outlet operational.

The effect of piers and abutments on the effective crest width as well as the variation of the discharge coefficient with head are considered in the calculation of the spillway discharge capacity.

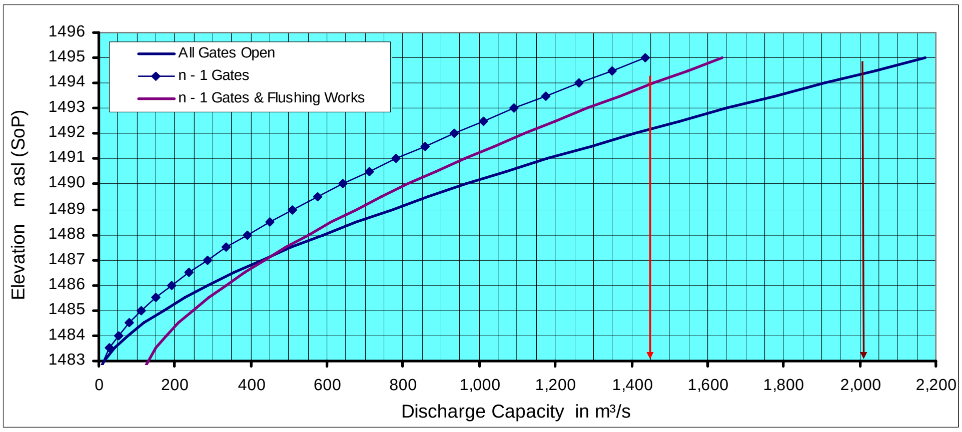

Figure 4.10: Discharge Capacity of the spillway of the Madian HPP Weir

For the relevant design floods the calculation of the discharge capacity was performed and the corresponding results are given in Figure 4.10. In the event of the Safety Check Flood $HQ_{10,000}$ = 2002 m³/s at the required minimum freeboard of 1.0 m is the discharge capacity is sufficient for all 3 spillway gates being fully open as demonstrated in Table 4.3.

Spillway Discharge 2069 m³/s > 2002 m³/s

All Gates Fully Open										
Reservoir Level	Approach Channel Loss	Head H	Design Head Ho	Relative Head H / Ho	No. of Bays	Width of Bay	Total Width	Hydraulic Effective Width	Discharge Coefficient	Discharge Capacity
m	m	m	m		-	m	m	m	-	m³/s
1482.60	0.00	0.10	9.20	0.01	3.00	7.60	22.80	22.78	1.742	1.3
1483.00	0.00	0.50	9.20	0.05	3.00	7.60	22.80	22.71	1.770	14.2
1483.50	0.00	1.00	9.20	0.11	3.00	7.60	22.80	22.63	1.804	40.8
1484.00	0.00	1.50	9.20	0.16	3.00	7.60	22.80	22.54	1.836	76.0
1484.50	0.00	2.00	9.20	0.22	3.00	7.60	22.80	22.45	1.867	118.6
1485.00	0.00	2.50	9.20	0.27	3.00	7.60	22.80	22.37	1.897	167.7
1485.50	0.00	3.00	9.20	0.33	3.00	7.60	22.80	22.28	1.925	222.9
1486.00	0.00	3.50	9.20	0.38	3.00	7.60	22.80	22.19	1.952	283.6
1486.50	0.00	4.00	9.20	0.43	3.00	7.60	22.80	22.11	1.977	349.7
1487.00	0.00	4.50	9.20	0.49	3.00	7.60	22.80	22.02	2.002	420.8
1487.50	0.00	5.00	9.20	0.54	3.00	7.60	22.80	21.93	2.025	496.6
1488.00	0.00	5.50	9.20	0.60	3.00	7.60	22.80	21.85	2.047	576.9
1488.50	0.00	6.00	9.20	0.65	3.00	7.60	22.80	21.76	2.068	661.5
1489.00	0.00	6.50	9.20	0.71	3.00	7.60	22.80	21.67	2.089	750.2
1489.50	0.00	7.00	9.20	0.76	3.00	7.60	22.80	21.59	2.108	842.7
1490.00	0.00	7.50	9.20	0.82	3.00	7.60	22.80	21.50	2.126	939.0
1490.50	0.00	8.00	9.20	0.87	3.00	7.60	22.80	21.41	2.144	1,038.7
1491.00	0.00	8.50	9.20	0.92	3.00	7.60	22.80	21.33	2.160	1,141.9
1491.50	0.00	9.00	9.20	0.98	3.00	7.60	22.80	21.24	2.176	1,248.2
1492.00	0.00	9.50	9.20	1.03	3.00	7.60	22.80	21.15	2.192	1,357.6
1492.50	0.00	10.00	9.20	1.09	3.00	7.60	22.80	21.07	2.206	1,469.8
1493.00	0.00	10.50	9.20	1.14	3.00	7.60	22.80	20.98	2.220	1,584.8
1493.50	0.00	11.00	9.20	1.20	3.00	7.60	22.80	20.89	2.233	1,702.4
1494.00	0.00	11.50	9.20	1.25	3.00	7.60	22.80	20.81	2.246	1,822.5
1494.50	0.00	12.00	9.20	1.30	3.00	7.60	22.80	20.72	2.258	1,944.9
1495.00	0.00	12.50	9.20	1.36	3.00	7.60	22.80	20.64	2.269	2,069.5

Table 4.3: Discharge Capacity of Madian HPP Spillway – all gates open

In the event of the spillway design flood $HQ_{1,000}$ = 1450 m³/s the operation of two spillway gates (one gate closed) and the flushing outlet is assumed

Spillway Discharge 1254 m³/s
Flushing outlet 197 m³/s
Total Capacity 1451 m³/s > 1450 m³/s

n - 1 Gates Fully Open										
Reservoir Level	Approach Channel Loss	Head H	Design Head Ho	Relative Head H / Ho	No. of Bays	Width of Bay	Total Width	Hydraulic Effective Width	Discharge Coefficient	Discharge Capacity
m	m	m	m		-	m	m	m	-	m³/s
1482.60	0.00	0.10	9.20	0.01	2.00	7.60	15.20	15.18	1.742	0.8
1483.50	0.00	1.00	9.20	0.11	2.00	7.60	15.20	15.05	1.804	27.1
1484.00	0.00	1.50	9.20	0.16	2.00	7.60	15.20	14.97	1.836	50.5
1484.50	0.00	2.00	9.20	0.22	2.00	7.60	15.20	14.89	1.867	78.7
1485.00	0.00	2.50	9.20	0.27	2.00	7.60	15.20	14.82	1.897	111.1
1485.50	0.00	3.00	9.20	0.33	2.00	7.60	15.20	14.74	1.925	147.4
1486.00	0.00	3.50	9.20	0.38	2.00	7.60	15.20	14.66	1.952	187.4
1486.50	0.00	4.00	9.20	0.43	2.00	7.60	15.20	14.59	1.977	230.7
1487.00	0.00	4.50	9.20	0.49	2.00	7.60	15.20	14.51	2.002	277.3
1487.50	0.00	5.00	9.20	0.54	2.00	7.60	15.20	14.43	2.025	326.8
1488.00	0.00	5.50	9.20	0.60	2.00	7.60	15.20	14.36	2.047	379.1
1488.50	0.00	6.00	9.20	0.65	2.00	7.60	15.20	14.28	2.068	434.1
1489.00	0.00	6.50	9.20	0.71	2.00	7.60	15.20	14.20	2.089	491.6
1489.50	0.00	7.00	9.20	0.76	2.00	7.60	15.20	14.13	2.108	551.5
1490.00	0.00	7.50	9.20	0.82	2.00	7.60	15.20	14.05	2.126	613.6
1490.50	0.00	8.00	9.20	0.87	2.00	7.60	15.20	13.97	2.144	677.9
1491.00	0.00	8.50	9.20	0.92	2.00	7.60	15.20	13.90	2.160	744.1
1491.50	0.00	9.00	9.20	0.98	2.00	7.60	15.20	13.82	2.176	812.2
1492.00	0.00	9.50	9.20	1.03	2.00	7.60	15.20	13.74	2.192	882.0
1492.50	0.00	10.00	9.20	1.09	2.00	7.60	15.20	13.67	2.206	953.5
1493.00	0.00	10.50	9.20	1.14	2.00	7.60	15.20	13.59	2.220	1,026.6
1493.50	0.00	11.00	9.20	1.20	2.00	7.60	15.20	13.51	2.233	1,101.1
1494.00	0.00	11.50	9.20	1.25	2.00	7.60	15.20	13.44	2.246	1,177.0
1494.50	0.00	12.00	9.20	1.30	2.00	7.60	15.20	13.36	2.258	1,254.1
1495.00	0.00	12.50	9.20	1.36	2.00	7.60	15.20	13.29	2.269	1,332.4

Table 4.4: Discharge Capacity of Madian HPP Spillway – n-1 gates open; powerhouse not in operation

The discharge capacity is sufficient to maintain the defined freeboard of 1.5 m as shown in Table 4.4 and stipulated by the design criteria.

4.3.2.3 Design of Spillway Stilling Basin

The riverbed of Swat River consists of large scale boulders and at selected locations rock is outcropping. At the site of the stilling basin, the geotechnical investigations indicate a thickness of alluvial material exceeding 25 m. In view of the alluvial material of considerable thickness and the fact that the right river bank consists largely of erodible moraine deposits where the Kedam – Kalam road and various houses are located downstream of the weir site, a stilling basin must be arranged.

With reference to the design floods and the corresponding design criteria, the stilling basin is designed for the design flood with a return period of 1000 years (HQ1,000 = 1450 m³/s). However, proper function of the stilling basin is reconfirmed in addition by verifying the stilling basin performance in addition for smaller floods covering return periods from 2 to 100 years:

HQ2 = 445 m³/s
HQ20 = 656 m³/s
HQ50 = 712 m³/s
HQ100 = 860 m³/s

The hydraulic analysis of the flow conditions in Swat River by means of the HEC-RAS backwater model and visual observations of flow during the high flow season reveal that Swat River changes frequently the mode of flow from sub-critical to super-critical and vice versa. This applies in particular to the river reach downstream of the stilling basin. For medium and high river flows critical flow establishes at the end sill of the stilling basin.

The hydraulic stilling basin was performed as follows:

1. Determine the energy head at the spillway crest for the selected spillway design at normal operation level of 1494 m asl (velocity head estimated based on a cross section area of 841 m²);
2. Determine the hydraulic conditions along the ogee, in particular at the transition from ogee to stilling basin by means of the Consultant's water surface profile program CHUTEFLOW;
3. Calculate the conjugated depth h_2 of the hydraulic jump and the required length of the stilling basin L_{STB} according to BLIND;
4. Determine the relevant tailwater level and calculate the required elevation of the stilling basin floor.

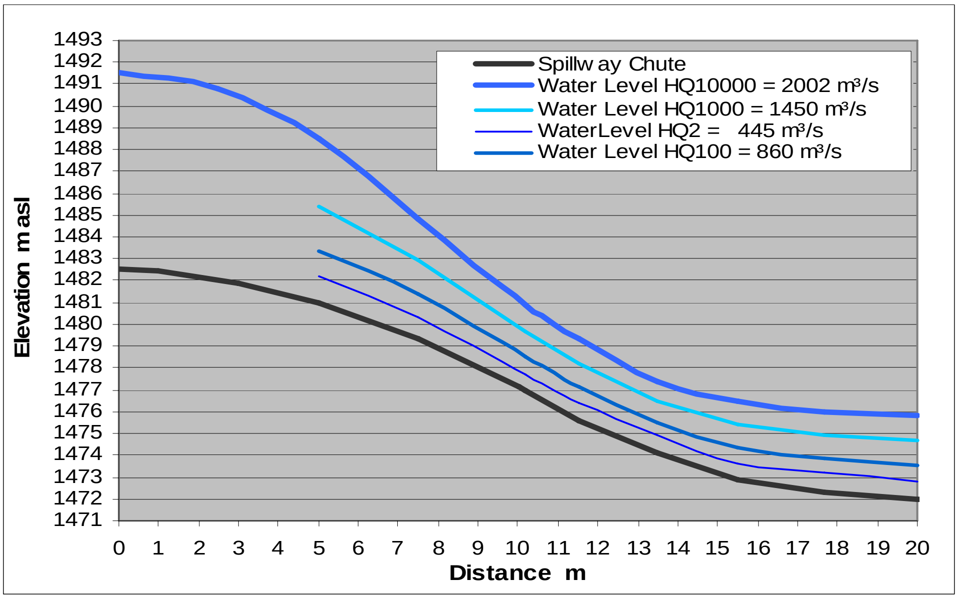

Figure 4.11: Flow Profiles along Spillway Ogee for Floods from HQ_2 to $HQ_{10,000}$

For the required length of the stilling basin different approaches are given in hydraulic design manuals of the type $L_{STB} = K \times (h_2 - h_1)$ with K being in the range from 4.5 to 6. The Consultant considers the coefficient K = 5.0 proposed by BLIND ("Wasserbauwerke aus Beton") given in the Hydraulic Design Criteria. The validity of this coefficient shall be reconfirmed by testing of the structure in a physical model in the next planning stages.

The hydraulic conditions from spillway crest to the stilling basin were determined for discharges between HQ_2 and $HQ_{10,000}$, i.e. 445 to 2002 m^3/s. As the result the stilling basin with the following dimensions was selected:

Invert of stilling basin	1472.0 m asl
Width of stilling basin	28.8 m
Elevation of river bed d/s	1476.1 m asl
Length of stilling basin	54.0 m

For the entire range of discharges considered in the present analysis, the criterion that the downstream water level exceeds the conjugated depth h_2 is maintained including a safety margin of 5 %.

Discharge m³/s		Energy Head m	Depth h1 m	Flow velocity v1 m/s	Froude number Fr1	Conjugated Depth h2 m	Tailwater level m asl	Stilling basin floor m asl	Basin length m
2002	HQ10,000		3.78	18.37	3.018	14.35	1487.98	1472.91	52.87
1450	HQ1,000	1494.15	2.66	18.91	3.704	12.67	1485.95	1472.65	50.03
860	HQ100	1494.05	1.54	19.35	4.981	10.11	1483.42	1472.81	42.83
656	HQ20	1494.03	1.17	19.43	5.738	8.93	1482.40	1473.02	38.79
445	HQ2	1494.01	0.8	19.41	6.932	7.45	1481.21	1473.38	33.27

Table 4.5: Hydraulic Design Parameters for the Spillway Stilling Basin

A profile along the spillway centre line is given in Drawing Plate 14 in Volume VII.

4.3.2.4 Flushing Structures

The weir is equipped on the left side with a flushing structure consisting of two short steel lined ducts (see Plate 15, Volume VII). The flushing outlets are equipped with hydraulically operated sliding gates. For erection, maintenance and repair the flushing (bottom) outlets can be closed upstream and downstream with stop logs.

The flushing outlets serve for the following purposes:

- Additional optional release facility for the Safety Check Flood;
- To fulfil the n-1 rule for the 1,000 years flood, if one of the overflow spillway gates can not be opened for whatever reason;
- Control device for maintenance and draw down of the reservoir;
- Sediment flushing facility to keep the power intake sediment free;
- For reservoir flushing in combination with a reservoir draw down; (To be verified by physical model tests and consider slope stability)

No continuous flushing is foreseen at the weir structure. With progressing reservoir sedimentation, deposition of bed material may occur also in front of the power intake. The flushing structure is designed to evacuate sediment which deposits in front of the power intake thereby avoiding the entrainment of coarse sediment fractions (gravel and cobble) into the power intake at times of high river flow at normal operation level 1494 m. Flushing shall be, therefore, possible at a depth of water of approx. 18 m at the weir site.

The invert of the flushing ducts is arranged at riverbed level. The sliding gates are arranged at the upstream part of the rather short ducts. Rather high flow velocities may occur and will prevent the bed load from depositing within the gate or the adjacent chute section. The design discharge capacity was selected to be 200 m³/s at full reservoir supply level of 1494 m asl.

For the flushing outlet the following dimensions were selected:

Flushing Ducts	No. 2			
Length	10.2 m	(pressurized section)		
Width	at intake	2.0 m	at gate section	2.0 m
Height	at intake	6.0 m	at gate section	3.0 m

During the high flow season some 130 m³/s will be diverted through the power intake and the remaining river flow may be released either over the spillway or through the flushing outlet. The flushing outlet is designed to be capable of removing fine and medium fractions of sediments (sand and gravel) that must not accumulate excessively in front of the power intake.

A stilling basin was not arranged to avoid that pebble, cobbles erode the concrete in the stilling basin within short time. Along the left bank of the Swat River rock is outcropping at the weir site and along the stilling basin so that no major damage can be expected from a high velocity current leaving the Madian weir structure occasionally.

Head Pond Leve m asl (SoP)	Flushing Discharg m³/s
1495.00	203.39
1494.00	197.66
1493.00	191.76
1492.00	185.67
1491.00	179.37
1490.00	172.85
1489.00	166.07
1488.00	159.00
1487.00	151.60
1486.00	143.82
1485.00	135.59
1484.00	126.84
1483.00	117.43
1482.00	107.20

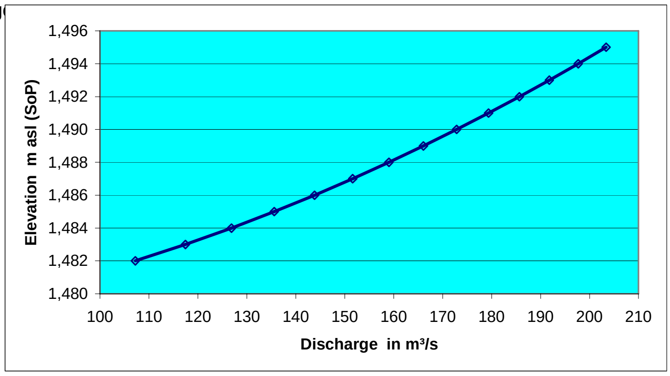

Figure 4.12: Discharge Capacity of the Flushing Outlets Madian HPP Weir

The flushing duct and the corresponding downstream chute are arranged separated from the spillway stilling basin to permit inspection, maintenance and repair when required and independent from spillway operation.

When necessary a considerable reservoir draw down can be achieved by opening the spillway gates and the flushing outlet gradually. However, care shall be taken and a rapid reservoir draw down shall be avoided not to provoke destabilization of the reservoir valley slopes, e.g. as the result of activating pore pressure.

In order to stabilize the right reservoir bank where the Kedam – Kalam road is located, an approximately 2 to 3 m thick mineral filter and erosion protection of riprap will be placed along the right reservoir bank using selected tunnel excavation material. The material will serve in addition as protection against current and wave action in the reservoir. Reservoir draw down shall be limited to 3 m per day.

It is recommended to reconfirm the hydraulic efficient performance of the spillway, power intake and flushing outlet by hydraulic model tests in the tender design stage.

4.3.3 Stability Calculation for the Weir Structure

The following chapter describes general structural design considerations of the weir. To achieve a safe and optimized structural layout at low costs as regards in particular quantities of concrete and reinforcement, structural analyses computations have been carried out for the following considerations:

- Overturning of the Weir body
- Sliding of the Weir Structure
- Uplift of Weir Structure and Stilling Basin
- Soil Loads
- Water Pressure
- Seismic Loads
- Dimensioning of Bore Piles

The relevant standards such as Engineering Manuals of the US Army Corps of Engineers (USACE), EM 1100-2-2200 - have been taken into account as well as good engineering practice.

The structural system in general is the weir body being supported by

1. a bore pile row under the weir crest being also the sealing wall against seepage
2. bedrock at the left river bank
3. a bore pile wall at the right river side serving as construction pit retaining wall during construction
4. a bore pile row under the weir at the first d/s expansion joint.

Accordingly the stilling basin will be supported by items 2 and 3 as above and have a permeable bore pile row below the d/s sill. The bore pile heads will be constructed to provide elastic vertical support in order to use the bearing capacity of the soil for structural support even after some settlement and to avoid gaps between the foundation concrete and the river sediments.

4.3.3.1 Load Assumptions

Horizontal loads are applied to the structure by water pressure of the reservoir, sediment load on the weir and seismic loads.

The following values were used:

Water: $\gamma = 10$ kN/m³

Sediments: $\gamma = 16$ kN/m³

Concrete: $\gamma = 24$ kN/m³

Seismic: OBE $k_h = 0.26$

For the sediment load the equivalent hydrostatic pressure is used. Reservoir sedimentation is assumed conservatively to have reached the condition that the entire reservoir is filled up to the crest level of the weir structure.

The seismic hydrodynamic effects of the reservoir water are computed according to Westergaard's formula:

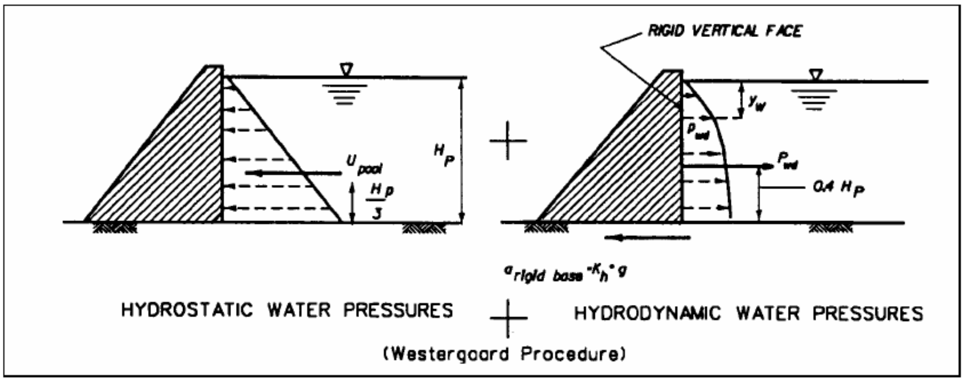

Figure 4.13: Hydrostatic and hydrodynamic loads resulting from horizontal ground acceleration according to WESTERGAARD

4.3.3.2 Weir Stability

The horizontal loads from water pressure and sediments are resisted by bedrock, bore piles and friction between the weir bottom and river sediments. This combination provides sufficient safety against sliding.

Sample computations are shown below:

Overturning Analysis of the Weir **1. Normal Condition maximum Operation Level**

	structure # load #	width [m]	length [m]	height [m]	volume [m³]	weight / force [kN]	y center of gravity / lever	retaining moment	overturning moment
right wall	1	2.5	28.5	27.0	1,924	46,170	12.3	565,583	
weir body *)	2	7.6		221.2	5,043	121,041	13.7	1,658,257	
grouting gallery *)	2a	7.6		5.7	-130	-3,119	19.2	-59,886	
guide walls	3	3.0	23.0	27.0	1,863	44,712	10.5	469,476	
left wall	4	4.5	28.5	27.0	3,463	83,106	12.3	1,018,049	
access shaft to gr. gallery	5	5.0	10.0	27.0	1,350	32,400	21.5	696,600	
access shaft in 6	6	3.0	4.0	21.0	-252	-6,048	18.0	-108,864	
water pressure u/s	7					130,670	9.0		1,176,030
water pr. bottom	8					29,000	19.5		565,500
sediment load	9					68,800	5.2		357,760
bottom / tailwater	10					25,780	11.0		283,580
						total moments:		**4,239,214**	**2,382,870**
						safety against overturning:			**1.78**
									> 1.5

*) area from AutoCad computation, "height" is sectional area

overall vertical loads:	292,482	kN		
overturning moment:	2,382,870	kNm		
excentricity:	**8.15**	**m < 25/3=**	**8.33**	**m**

Table 4.6: Overturning Analysis for Normal Operational Conditions

Due to the concept of constructing a sealing wall against seepage across the valley, i.e. bore pile wall with additional grouting curtain, the d/s groundwater level will be in the range of the tailwater level. As an additional safety measure drain pipes will be laid underneath the stilling basin so that the maximum ground water level will be tailwater level.

Overturning Analysis of the Weir **2. Unsusual Condition: maximun Operation Level and OBE**

OBE: 0.26

	structure # load #	width [m]	length [m]	height [m]	volume [m³]	weight / force [kN]	y center of gravity / lever	retaining moment	overturning moment
right wall	1	2.5	28.5	27.0	1,924	46,170	12.3	565,583	
weir body *)	2	7.6		221.2	5,043	121,041	13.7	1,658,257	
grouting gallery *)	2a	7.6		5.7	-130	-3,119	19.2	-59,886	
guide walls	3	3.0	23.0	27.0	1,863	44,712	10.5	469,476	
left wall	4	4.5	28.5	27.0	3,463	83,106	12.3	1,018,049	
access shaft to gr. gallery	5	5.0	10.0	27.0	1,350	32,400	21.5	696,600	
access shaft in 6	6	3.0	4.0	21.0	-252	-6,048	18.0	-108,864	
water pressure u/s	7					130,670	9.0		1,176,030
water pr. bottom	8					29,000	19.5		565,500
sediment load	9					25,811	5.2		134,217
bottom / tailwater	10					25,780	11.0		283,580
OBE water u/s	11					39,600	10.8		427,680
OBE sediment	12					68,800	5.2		357,760
OBE weir body	2					31,471	5.9		185,676
OBE walls and shaft	1, 3, 4					52,088	13.5		703,193
						total moments:		**4,239,214**	**3,833,637**
						safety against overturning:			**1.11**
									> 1.10

*) area from AutoCad computation, "height" is sectional area

Westergaard: $P_E = 7/12 \times k_h \times \gamma_w \times h^2$ = 1,106 kN/m => 39,600 kN

Table 4.7: Overturning Analysis for Unusual Operational Conditions (OBE-1)

The critical load condition will be empty stilling basin and maximum groundwater level. To achieve most economic results the wing walls of the stilling basin have been taken into account for the retaining forces, required reinforcement in the slab has been checked.

An in depth analysis against overturning has been performed (see Tables 4.6 and 4.7), since there are relatively high gates compared to the overall height of the structure, thus reducing concrete masses and consequently the retaining vertical forces. The critical load conditions appeared to be maximum operation level with minimum tailwater level under normal as well as under OBE conditions.

Considering the d/s bore pile row near the expansion joint between weir and stilling basin is the most heavily loaded one under operational conditions, a design analysis has been performed for these piles. The results are shown in Table 4.8 below.

Soil Pressure and d/s Bore Pile Wall Analysis

overall vertical loads: 292,482 kN
overturning moment: 2,382,870 kNm
excentricity: 8.15 m < 25/3= 8.33 m

soil pressure neglecting pile support:

$\sum V =$ 292,482 kN => 8,170 kN/m

max p = 668.29 kN/m² ==> bore piles to be taken into account

Load on d/s bore pile row: 5,719 kN/m

assumed min. bore pile diameter is 80 cm, concrete min. compressive strength 25 N/mm²

bearing capacity of one pile is 7,177 kN

==> bore piles dia 80 cm spaced c/c 1.25 m

Table 4.8: Results of Static Analysis of Bore Piles

4.4 Design of Diversion Works

The selection of the principle for river diversion during construction of the weir structure is largely governed by the following two aspects :

a) Limitation of available space within the construction pit
b) Magnitude of diversion design flood

At the weir site the riverbed is only some 20 to 25 m wide. Because of the estimated thickness of the alluvium of up to 30 m and the steep slopes at the right bank of the weir site, it is technically very difficult and costly to excavate down to the foundation level on moderately weathered rock. For this reason the concept of a staged development of river diversion with construction of part of the weir structure subsequently on the left and then on the right river bank is not feasible.

Accordingly the Consultant designed conventional river diversion works with the following components:

<u>Weir Structure</u>

1. Upstream rock fill cofferdam with sealing
2. Downstream cofferdam constructed on bore pile wall
3. Diversion tunnel on left river bank

<u>Powerhouse / Power Outlet</u>

1. Gabion cofferdam with sealing (PVC sealing)

In accordance with common design practice and the hydraulic design criteria a design flood for river diversion during construction with a return period of 20 years is selected resulting in the following discharge values:

Diversion Design Flood	Weir	HQ20 = 656 m^3/s
Diversion Design Flood	Powerhouse	HQ20 = 731 m^3/s

For the diversion works at the weir structure the height of the upstream cofferdam is limited by the Kedam - Kalam road which shall not be overtopped in the event of the diversion design flood. Therefore, the crest elevation of the upstream cofferdam is limited to elevation 1496.0 m asl.

The Consultant conducted the corresponding analysis of the hydraulic conditions along the diversion tunnel inlet structure, within the tunnel and at the tunnel outlet structure. For low to medium river discharges up to the flood with a return period of 10 years, free flow conditions prevail along the diversion tunnel. With further increasing diversion discharge the transition to pressurized flow establishes from downstream to upstream.

The hydraulic calculations for free flow conditions were conducted applying the program HEC-RAS. For pressurized flow conditions, the calculation is based on the approach of Darcy-Weissbach and Prandtl-Colebrook (see hydraulic design criteria). The hydraulic conditions at transition from free flow to pressurized flow were interpolated as it is common practice.

The discharge capacity of the diversion works is presented in Figure 4.14. The hydraulic calculations are based on the stage-discharge relationship (17) established for the Swat River reach downstream of the weir site (see Figure 4.8). Based on the hydraulic analysis the following dimensions of the diversion works were defined:

Diversion Tunnel	D-shaped	Width =	8.0 m
		Height =	9.2 m
		Length =	275.0 m
		Tunnel inlet invert	1479.5 m asl
		Tunnel outlet invert	1476.3 m asl
Upstream rock fill cofferdam		Crest Elevation	1496.0 m
		Crest Width	6.0 m
Downstream bore pile cofferdam		Crest Elevation	1481.0 m
With riprap embankment		Crest Width	6.0 m

For the upstream cofferdam a freeboard of 1.5 m is maintained.

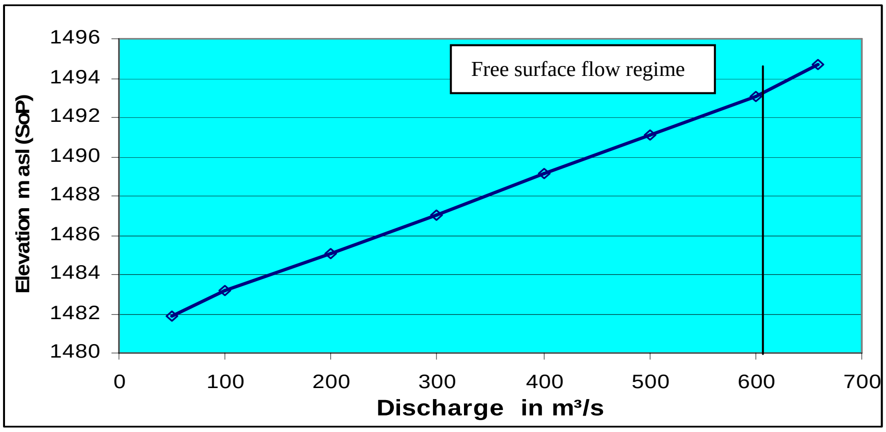

Figure 4.14: Discharge Capacity of the Left Bank Diversion Tunnel

4.5 Conceptual Design of Power Waterways

Based on the studies conducted on pre-feasibility level, the project concept was further developed in the course of the feasibility study, however considering a slightly modified location for weir and powerhouse sites. Otherwise the overall project concept was maintained and consists of the following major components:

a) Power intake on left bank of Swat River
b) Desander basins, No. 3
c) Headrace tunnel, 11.8 km long
d) Surge tank
e) Vertical pressure shaft
f) Horizontal pressure tunnel
g) Manifold
h) Powerhouse
i) Tailrace and Power outlet

4.5.1 General Project Parameter and Dimensions

The hydraulic design of the power waterway system is based on the following basic parameters and dimensions:

Rated Turbine Discharge:	3 x 43	m³/s	(see chapter 4.5.5)
Invert of Power Intake	1483.0	m asl	
Full Supply Level	1494.0	m asl	
Maximum Operation Level	1494.5	m asl	
Minimum Operation Level	1492.0	m asl	
Minimum Tailwater Level	1339.6	m asl	
Maximum Tailwater Level	1346.0	m asl	

4.5.2 Alternative Project Concepts

As part of the feasibility design the arrangement of the power waterway system was made based on the alternative concepts of a free surface and an underground powerhouse each equipped with 3 vertical Francis turbine units. The power waterway system is located on the left bank of Swat River and consists of the components as summarized in Table 4.9.

Power Waterway Components	Free Surface PH	Underground PH
Intake structure, No. of inlets	1,3	1,3
Desander basin	3	3
Headrace tunnel, concrete lined	1	1
Pressure shaft, concrete/steel lined	1	1
Pressure tunnel, steel lined	1	1
Manifold, steel lined	3	3
Francis Turbine-generator units	3	3
Tailrace tunnel	-	1
Power Outlet, No. of Outlets	1,3	1,1

Table 4.9: Components of Power Waterway System

In the pre-feasibility study excavation of the headrace tunnel was assumed to be carried out using a tunnel boring machine (TBM). A corresponding straight alignment was defined aiming on the shortest possible distance between power intake and surge tank. For the selected alignment an approximate tunnel length of 11.6 km was defined.

In accordance with the terms of reference (ToR) the Consultant investigated alternative tunnel alignments for the application of a TBM and alternatively by conventional drill and blast method.

4.5.2.1 Headrace Tunnel Alignment – TBM - Excavation

Detailed information on the technical characteristics, transport requirements, performance and costs of tunnel construction, was inquired at leading TBM manufacturers. The present analysis is based on information provided by HERRENKNECHT AG (September 2007) on a gripper-type TBM with excavation diameter of approximately 7.0 m and information from ROBBINS (October 2007) for a similar unit presently in operation in Island for the Karanhjukar HPP.

Considering the requirements to setup a TBM at the downstream tunnel portal, the tunnel alignment selected in the pre-feasibility study was subject to minor adjustments. A modified starting point was selected to achieve the required space for setting up the TBM at elevation approximately 1450 m.

The advantage of TBM technology is the high progress of work provided geological conditions are rather homogeneous without excessive waiting time, e.g. for treatment (grouting) of fault/shear zones. For the prevailing conditions at the Madian HPP progress of work could be expected to reach 15 to 20 m per day.

4.5.2.2 Headrace Tunnel Alignment–Conventional Drill & Blast Excavation

Alternatively the Consultant developed a tunnel alignment for conventional drill and blast excavation method in a such a way that the rock cover shall not be less than approximately 50 m, in particular in the area of nullahs (depressions where perennial streams form in the rainy season). Some of these depressions represent supposed faults or shear zones such as observed at Gornai Nullah located some 400 m downstream of the weir site.

In view of the length of the headrace tunnel of almost 12 km, conventional tunnel construction would need to proceed in parallel in several tunnel stretches. Aiming on an economic feasible construction period, a total number of 4 tunnel reaches with a maximum length of 3.6 km was defined.
The first and last tunnel reach were selected slightly shorter because of the interference with works at other project components such as the intake structure upstream and surge tank as well as pressure shaft downstream.
Alternative alignments for the headrace tunnel were compared aiming on a short headrace tunnel length on one hand and an acceptable length and convenient access to the adits on the other. The location of the construction adits was defined taking into account the following criteria:

1) Three adits are necessary to divide the headrace tunnel into four stretches of similar length;
2) Adit portals shall be close to existing bridges over the Swat river but not interfering with densely populated areas
3) Adits shall be short and accessible by heavy equipment

Headrace Tunnel	TBM Feasibility m	D&B Feasibility m
Reach 1		2,474
Reach 2		2,680
Reach 3		3,802
Reach 4		2,934
Total HR-Tunnel Length	11,893	11,890
Adit at Surge Tank	201	150
Constr. Adit No. 1		280
Constr. Adit No. 2		380
Constr. Adit No. 3		250
Total Adit Length	201	1,060

Table 4.10 Length of Headrace tunnel for TBM and Conventional Excavation

Based on the above criteria the headrace tunnel alignment shown in Plate 3; Plate 12; Plate 13; (Volume VII of this Feasibility Report) and summarized in Table 4.10 was selected at the result of an iterative optimization process. The headrace tunnel alignment was eventually defined as the result of a trade-off between additional costs resulting from extra headrace tunnel length and the cost of extra length of the construction adits. The assessment of the conditions for the access to project site, revealed however, that transport of TBM equipment is technically not feasible as long as three existing road bridges will not be replaced or at least rehabilitated (see Section 3.5). For this reason and in view of the higher cost per meter of tunnel construction, the Consultant in co-ordination with the Project Sponsor decided to proceed with the feasibility design of the Madian HPP based on the concept of excavating the headrace tunnel by conventional drill and blast method.

4.5.3 Selection of Powerhouse Type

As specified by the ToR the Consultant conducted a technical and economic assessment of design concepts based on an open air and alternatively an underground powerhouse in the feasibility design stage. Technical and Economic aspects are addressed as well as environmental impacts and potential risks during construction and operation of the hydropower plant.
This assessment aimed on the recommendation of the preferred powerhouse type to be finally designed on feasibility level. The results of the geo-technical field investigation and its evaluation / interpretation were taken into account in the assessment.

4.5.3.1 Powerhouse Location and Type

The selection of potential locations for the powerhouse/power outlet is restricted by the following aspects:

Figure 4.15: Location of Powerhouse

a) The original powerhouse site proposed by GTZ had to be abandoned. Several multi-storey hotels were built on the left river bank at the north of Madian town at this favourable powerhouse location.

b) Further upstream of the GTZ-location a wide plain has formed which is partly populated by houses and which is believed to have considerable thickness of slope wash and overburden.

c) At the pre-feasibility powerhouse site the rock slopes form a U-shaped valley without a significant river terrace.

The space available for construction of an open air powerhouse and its outlet bay is limited and a deep cut into the rock slope would be required.

The Madian HPP shall be of the run-of river type. For the prevailing topographic, hydrological and geological conditions the following powerhouse types are feasible in principal:

1. Free surface powerhouse
2. Shaft powerhouse
3. Underground (cavern) powerhouse

The Consultant studied and optimized the installed capacity, number and individual size of turbine units. As the result a powerhouse with three 3 identical Francis turbine units with vertical axis was obtained. Therefore, for the comparison of the powerhouse types under consideration, 3 Francis units of identical size were considered. The turbine setting might be slightly deeper in case of the underground powerhouse which would permit a minor cost saving for the electro-mechanical equipment.

<u>Free Surface Powerhouse</u>
A standard open air powerhouse may result as the most economic solution provided sufficient space is available for construction and flood water levels and availability of sound rock at reasonable depth do not result in excessive quantities of concrete in the substructure. Of particular importance in case of free surface powerhouses in Pakistan are the aspect of slope stability in the powerhouse area and the extent of slope stabilization measures in view of frequent earthquake activities as well as security aspects in general.

<u>Underground Powerhouse</u>
The feasibility of a cavern type powerhouse depends largely on the prevailing geological conditions. Cost of civil works (and equipment) can be expected to be somewhat higher compared to a free surface powerhouse. Based on the Consultant’s geological mapping, geophysical survey and core drilling at the powerhouse site, the geological conditions were assessed to be reasonable for an underground powerhouse with rock class B to C (see Report on Geology, Section 3.4). An underground powerhouse requires construction of access, cable tunnels, ventilation shafts and a transformer cavern. At the selected powerhouse site, these tunnels can be kept short.
An underground powerhouse provides the highest level of security (against earthquake and vandalism/terror).

<u>Shaft Powerhouse</u>
A shaft powerhouse combines the advantages of an open air and underground powerhouse. Vertical shafts are excavated and concrete lined to accommodate the E&M electro-mechanical and electrical equipment.
For erection and maintenance access is maintained from the surface.
One single power house shaft may accommodate one or two units.
The spacing between three powerhouse shafts would result in long upstream steel lined waterways and result in high costs together with the large dimensions of the superstructure. A shaft powerhouse with 3 shafts is evidently economically less attractive than a free surface powerhouse.

4.5.3.2 Open Air versus Underground Powerhouse Type

As an alternative to the project layout proposed in the pre-feasibility study a layout with underground powerhouse was elaborated which is identical from the power intake to the end of the vertical pressure shaft. The estimated quantities and costs for each powerhouse type are given in Table 4.11 based on the preliminary dimensions and quantities (on pre-feasibility level).
For both powerhouse types a provision is made for miscellaneous items.

Project MADI001, FREE SURFACE POWERHOUSE

No	Item	Quantity	Unit	Unit Cost in US$	Total Cost in US$
8	excavation in rock open	71300.9	m3	15.0	1,069,514
15	backfilling	21390.6	m3	5.5	117,648
16	concrete	18264.4	m3	123.0	2,246,521
17	reinforcement	1892	to	1361.0	2,575,012
18	formwork	38338.3	m2	16.4	626,831
19	Miscellaneous items	27.5	%		1,652,391
	TOTAL	in US$			**8,287,917**

Project MADI001, UNDERGROUND POWERHOUSE

No	Item	Quantity	Unit	Unit Cost in US$	Total Cost in US$
8	excavat.cavern cl.1-2-3, 120m2	38184	m3	61.4	2,343,734
15	rock bolt	9282	m	30.3	281,245
16	wire mesh	53.32	to	1830.0	97,576
17	shortcrete lining	1487	m3	163.5	243,125
18	concrete to cavern	14961	m3	139.0	2,079,579
19	reinforcement to cavern	1496.1	to	1361.0	2,036,192
20	formwork to cavern lining	18923	m2	17.6	332,666
	Miscellaneous items	27.5%			2,038,882
	TOTAL	in US$			**9,452,998**

Table 4.11: Cost Estimate for Free Surface and Underground Powerhouse

The comparison of costs given in Table 4.12 indicates that the design concept with underground powerhouses requires slightly higher investment costs compared to the concept with an open air powerhouse, however, the difference is marginal. The additional costs for the underground powerhouse and the required access and cable tunnel are almost compensated by the savings in the steel lining for the horizontal high pressure tunnel.

Design Aspect	Unit	Powerhouse Type		Surplus Cost of
		Free Surface	Underground	Free Surface PH
Length of Vertical Pressure Shaft	m	126	130	-4
	million USD			-0.025
Length of Horizontal Pressure Tunnel	m	168	16	152
	million USD			0.874
Tailrace Tunnel	m	0	110	-110
	million USD			-0.541
Pressure Tunnel Steel Lining	m	215.3	67.3	148
	million USD			2.711
Access Tunnel	m	0	200	-200
	million USD			-1.750
Cable Tunnel	m	0	125	-125
	million USD			-0.467
Powerhouse Length x Width x Height	m			
	million USD	**8.288**	**9.453**	-1.165
Transformer & Switchyard cavern / yard	million USD	**0.478**	**1.272**	-0.794
Slope Stabilizing retaining wall	million USD	**1.213**	**0.325**	0.888
Total	**US$**			**-0.268**

Table 4.12: Effect of Powerhouse Type Selection on Overall Project Cost

An advantage of the layout with underground powerhouse compared to an open air powerhouse is the circumstance that the length of the high pressure tunnel can be kept short and instead a more economic (concrete lined) tailrace tunnel can be arranged.

The difference in cost between the two powerhouse alternatives is minor so that from the economical point of view both alternatives can be considered equivalent. Preference to an alternative can be made taking into account the following monetarily not yet evaluated aspects such as:

- risks during construction and operation
  (vandalism, terrorism, extraordinary floods, earth slides etc.);
- costs during operation (maintenance, access etc.);
- environmental and socio-economic impact.

The extent of underground works for the Madian HPP is considerable comprising desander caverns, tunnel works and a surge tank. The Contractor will have at its disposal adequate and efficient equipment and staff for underground works and will achieve the necessary experience of the site specific conditions. Based on the available knowledge of the project area, the Consultant gives preference to the underground powerhouse option in view of potential risk in the Swat area as mentioned above.

4.5.4 Optimization of Installed Capacity

4.5.4.1 Methodology for Optimization of Installed Capacity

Optimization of the Madian HPP means to determine the waterway design discharge and respective installed capacity for which development of the project results in the most favourable configuration according to the economic optimization criterion applied. For optimization of hydropower projects commonly the following optimization criteria are applied:

a) Maximum Internal Rate of Return on investments (IRR);
   or minimum specific cost of generation in US c /kWh
b) Maximum Net Benefit.

From the prospective of a private project developer the preferred optimization criterion is that which provides the maximum rate of return on investment. Since Madian HPP is developed by a private sponsor, maximizing the rate of return is considered as the relevant optimization criterion. In addition and for comparison purposes, the optimum powerhouse design discharge was determined on least specific generation cost basis and for the maximum present value (benefit-cost). Furthermore, a sensitivity analysis was undertaken to assess the effect of the variation of relevant parameters on the resulting optimum design discharge.

In accordance with common practice all relevant project related costs and benefits are expressed in terms of their present value referring to the same date to be comparable. For ease of comparison the year of project implementation is selected as the reference date. For the selected project layout the dimensions and costs of some project components such as powerhouse and power waterway system vary with the installed plant capacity and others do not such as the weir structure, cofferdams, diversion tunnel etc.

By means of the Consultant's hydropower optimization program HPC (Hydropower Costing) the design of the project components and the corresponding elaboration of the bill of quantities, costing and simulation of annual energy generation was performed. This procedure was applied to powerhouse design discharges in the range from 100 to 180 m^3/s with 10 m^3/s increments, i.e. from 84 % to 1.52 times the mean annual flow.
Following this approach, for a total of 19 hydropower project alternatives the corresponding design was elaborated, the BoQ established, operation of the power plant and annual annual energy generation simulated, total costs and benefits as well as the corresponding economic parameters calculated.
For each alternative annual energy generation was based on 10 daily river flow series recorded in the 46 year long period from 1961 to 2006.

4.5.4.2 Basic Parameters for Optimization

Project Costs
For each alternative project the design was adjusted to the corresponding powerhouse design discharge and a bill of quantities was established to estimate the total costs of each alternative. Estimation of costs was performed for the given set of site specific conditions such as topography, hydrology and geological design relevant aspects as well as the main design parameters such as e.g. the head pond full supply level. The major structural components required for the hydropower project were considered.

These components are:

- General data such as location, alternative, design discharge
- Water levels (headwater and tailwater)
- Discharges (headwater and tailwater)
- Access to the power plant components (surge tank, reservoir, etc)
- Reservoir (land acquisition)
- Weir (dam, intake)
- Desander Cavern (cavern type)
- Headrace tunnel
- Surge tank
- Pressure shaft and pressure tunnel
- Powerhouse, cavern type
- Tail race

For all above components estimated data from field visits and/or from desk studies, for example depth of foundation of the weir or inclination of left/right slope were used. If certain input data were not known, the program uses adequate default values.

The program also calculates the optimum tunnel and pressure shaft diameter for the selected design discharges including the corresponding power and energy output. It has to be mentioned however that all quantities, cost, power and energy data obtained from the HPC software are for comparison purpose only. The final feasibility design will determine actual quantities and costs. The total cost of the alternative layouts (powerhouse discharge or installed capacity) was achieved by adding up within the program HPC the so called indirect cost as a percentage of direct cost.

4.5.4.3 Construction Costs of Project Alternatives

The direct cost was achieved by multiplying the quantities with the unit rates for the major construction activities for each component of the project.
The unit rates for civil works applied to the calculation of cost were derived from tender documents or feasibility studies of similar projects in Pakistan during the last 4 years (see Section 9). The corresponding rates were analyzed and escalated to the cost reference date of the project July 1st 2007. These rates are considered primarily and serve for comparison of the different layout alternatives. A more detailed elaboration will be carried out for estimation of cost of the selected feasibility design. The cost for hydro-mechanical equipment was calculated on basis of weights and confirmed by an international manufacturer same as the cost for the electrical equipment. The cost for the powerhouse switchyard was added as a lump sum.
Besides the direct cost of the project, indirect costs were added to the direct cost of the alternatives. A rate of 10 % was used as interest during construction. Four years were assumed for construction of the project.

4.5.4.4 Annual Energy Generation

A flat rate tariff of 0.07 US $ / kWh was applied to the assessment of energy generation related benefits in co-ordination with the Project Sponsor for determination of the annual benefits. A loss of revenues of 0.5 % was considered to account for reduced revenues related to outages of the project.
The simulation of reservoir operation and powerhouse operation was based on series of 10-daily river discharges at Kalam gauging station transformed to the Madian HPP weir site. The flow series were transferred to the selected weir site considering the increase of river flow according to the intermediate catchment (see Section 3.1: Hydrology).

<u>Normal maintenance</u> is scheduled for the low flow season between November and March, preferably in the month of February. During most of this period a single turbine unit is operated and the other turbine unit(s) may be inspected and maintained without causing losses to energy generation.

4.5.4.5 The Optimized Installed Capacity

For the range of powerhouse design discharges under consideration (from 100 to 180 m³/s), the headrace tunnel diameter, turbine dimensions and technical key parameters (e.g. runner speed) etc. are subject to variation.

In practice and similarly in the Consultant' HPC program, these variations occur stepwise and not continuously. Starting from a certain threshold value certain equipment parameters remain constant unless the subsequent threshold value is reached and the parameter increases. This is the case, e.g. for the turbine runner speed and thus the turbine dimensions and costs.
In particular the threshold value for changes in the turbine runner speed may vary from one turbine manufacturer to the other.

For the purpose of optimizing the installed capacity (powerhouse design discharge) instead of these stepped cost characteristics, functions of cost and annual energy generation where established by best fit regression analysis. This approach avoids the affect of such to a certain extent arbitrary definition of threshold values on the resulting optimum installed capacity.

Optimization of Design Discharge - Base Case

Power Revenues	0,07	US$ / kWh	**Cost Fact :**	**1**	-	
O&M Cost	1,5	%	Net Head :	133	m	
Interest Rate	10,00	%	Forced Outage :	1	d	(Full Operation)
Life Time	60	Years	Increm. Discharge	10	m^3/s	
Construc. Period	4	Years	CRF	0,10033	-	

Design Discharge	m^3/s	100	110	120	130	140	150	160	170	180
Original Cost	US$	246,001	261,45	275,334	291,087	302,922	317,756	335,263	345,929	364,996
Orig. Ann. Energy	GWh	644,00	687,45	724,48	762,76	799,11	832,92	860,00	889,20	911,09
Installed Capacity	MW	120	133	145	157	168	180	192	203	215
Proj. Cost (Funct.)	Mill US$	249,057	261,389	274,331	287,915	302,171	317,133	332,836	349,316	366,613
Energy Reduction	GWh	2,850	3,134	3,419	3,704	3,989	4,274	4,559	4,844	5,129
Ann. En. (incl.Red.)	GWh	641,150	684,316	721,061	759,056	795,121	828,646	855,441	884,356	905,961
Ann. En. (Function)	GWh	640,765	683,193	723,012	760,221	794,821	826,811	856,192	882,964	907,127
Accum. Factor	-	1,16	1,16	1,16	1,16	1,16	1,16	1,16	1,16	1,16
Present Value Cost	Mill US$	288,968	303,276	318,293	334,053	350,594	367,954	386,173	405,294	425,363
Annual Benefits	Mill US$	44,854	47,824	50,611	53,215	55,637	57,877	59,933	61,807	63,499
Annual O&M Cost	Mill US$	3,736	3,921	4,115	4,319	4,533	4,757	4,993	5,240	5,499
Annual (B-O&M)	Mill US$	41,118	43,903	46,496	48,897	51,105	53,120	54,941	56,568	58,000
Present Value Bene	Mill US$	409,827	437,585	463,431	487,361	509,370	529,453	547,605	563,820	578,092
PV(B-C)	Mill US$	120,859	134,309	145,138	153,308	158,776	161,500	161,432	158,525	152,729
Cost / kWh	US$ / kWh	0,0511	0,0503	0,0499	0,0498	0,0500	0,0504	0,0511	0,0520	0,0531
Cost / kW	US$ / kW	2408	2280	2195	2128	2087	2044	2011	1997	1978
CRF	-	0,142	0,145	0,146	0,146	0,146	0,144	0,142	0,140	0,136
1/CRF	-	7,028	6,908	6,846	6,832	6,860	6,927	7,029	7,165	7,334
C/B (auxilliary Val.)	-	7,028	6,908	6,846	6,832	6,860	6,927	7,029	7,165	7,334
IRR	%	14,224	14,472	14,604	14,633	14,573	14,432	14,222	13,952	13,629

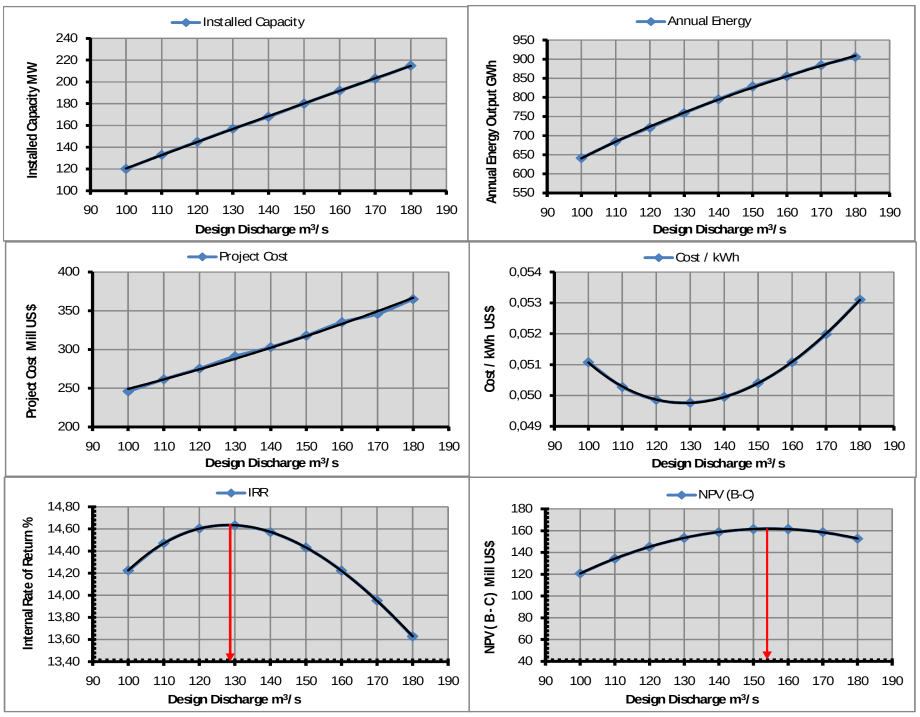

Table 4.13: Optimization of the Installed Capacity for Madian HPP

The corresponding trend lines for cost and energy output were established by an exponential function and a polynomial of second degree respectively. These functions were used for determination of the economic parameters such as IRR and Present Value.

Optimization was performed for the following Base Case parameters

Base Case:

Tariff	0,070	US $ /kWh
Interest Rate	10	%
Cost Overrun	0	%
Forced Outage	1	day
Construction Period	4	years
Project life time	50	years

Based on the above described approach and using the given set of input data an optimum power waterway design discharge of 129 m^3/s was obtained for the highest rate of return (refer to Table 4.13). For the assumed turbine characteristics this discharge corresponds to an optimum available capacity (ex transformer) of 3 x 52.43 = 157.3 MW for the Madian HPP.

4.5.4.6 Sensitivity Analysis

As already mentioned before, a sensitivity analysis was carried out to assess the potential effect of a variation of relevant input parameters on the optimum powerhouse design discharge as follows:

Additional Cases:

Case 1.1, 1.2	Tariff	0.0589	0.08	US $ /kWh
Case 2.1, 2.2	Interest Rate	8	12	%
Case 3.1, 3.2	Cost Overrun	20	33	%
Case 4.1, 4.2	Forced Outage	1 %	2.5	% of annual revenues
Case 5.1, 5.2	Construction Period	5	6	years

The results of the sensitivity analysis are summarized in as follows:

- Applying IRR as optimization criterion for all scenarios, the optimum design powerhouse discharge is 129 m^3/s. However, the magnitude of IRR varies between 10.68 % and 16.92 % for the range of parameters considered in the sensitivity analysis.

- Similarly the optimum powerhouse design discharge remains 129 m^3/s for all scenarios if applying the minimum specific generation cost/kWh as optimization criterion.

- In case the maximum net benefit is applied as the optimization criterion (max B-C), the optimum powerhouse discharge is found to be 153 m^3/s and resulting in an available capacity (ex transformer) of 183 MW.

The sensitivity analysis confirms the selection of the optimum power house and waterway design discharge of 129 m³/s, i.e. 3 x 43 m³/s. According to these preliminary results, the corresponding available capacity (at maximum turbine design discharge) is 3 x 52.43 MW = 157.3 MW. This preliminary value will be reconfirmed in the detailed feasibility design of civil works and electro-mechanical equipment described in continuation.

4.5.5 Optimization of Number and Size of Turbine Units

Based on the analysis of the hydrological data base and common design practice, the Consultant established alternative combinations of the number and capacity of turbine units taking into account the following criteria:

- For selection of reasonable combinations of Francis turbine units the minimum turbine discharge is approximately 40 % of the maximum turbine design discharge;

- The following combinations of turbine units are considered feasible
  - a) 1 large + 1 small ruled out, too low flexibility in operation
  - b) 3 large + 0 small
  - c) 2 large + 1 small
  - d) 2 large + 2 small
  - e) 3 large + 1 small ruled out, no economic advantage over d)

Based on the above considerations, the combinations b) to d) were selected for detailed assessment. For the concepts c) and d) various combinations of the capacity of the large and small turbine units can be established.

The Consultant optimized the combination of rated turbine discharges of the units in a way to maximize the annual energy generation simulating run-of-river operation based on daily river flow data. The corresponding optimum alternative combinations of number and rated discharge of turbine units are:

ALT 1:	3 units of identical size:	3 x 43 m³/s
ALT 2:	2 large units and 1 small unit	2x50.5 + 1 x 28.0 m³/s
ALT 3:	2 large units and 2 small units	2x41.0 + 2 x 23.5 m³/s

As the next step the Consultant elaborated a project design for the three alternative concepts applying the design and hydropower project assessment tool HPC. For each alternative the corresponding design, bill of quantities and cost estimates was generated including the particular powerhouse dimensions depending on the number and capacity of the turbine units.

For the assessment of benefits, simulation of hydropower plant operation (run-of river) was carried out based on 46 years of daily river flow data (historical period from 1961-2006). The annual benefits from power generation were reduced by the operation and maintenance costs which were estimated as a percentage of total investment cost. O&M costs increase with the number of units and in case of installing units of different capacity.

The highest annual energy generation can be achieved by 2 large and 2 small units (ALT 3) which permit operation at high turbine efficiency during most of the time. The advantage of ALT 3 over ALT 2 is minor as regards annual energy generation. Taking the annual energy output of three identical units (ALT 1) as 100 %, a 2.67 % higher annual generation is achieved for 2+1 units (ALT 2) and a 2.71 % higher for 2+2 units (ALT 3).

Most of the difference in annual energy generation between ALT 1 and ALT 2 results from the fact that the turbine units of ALT 1 are not operational when the river flow falls below 17.2 m³/s (during 0.8 % of time as the long term mean).

The result of the analysis of costs and benefits of the three alternative combinations of number and capacity of turbine units is given in Table 4.15. The costs shown in Table 4.14 are indicative only and do not include all components of the project. Final project costs are given in Section 9.

Number of Units		ALT 1 3 + 0	ALT 2 2 + 1	ALT 3 2 + 2
Direct Civil Costs	US $	172,982,165	173,546,546	173,701,383
Total Civil Costs	US $	224,011,904	224,742,777	224,943,291
Direct E&M Equipment Cost	US $	19,205,650	21,266,650	21,735,650
Total E&M Equipment Cost	US $	22,470,611	24,881,981	25,430,711
Direct Elect. Equipment Cost	US $	30,153,600	30,577,300	33,654,350
Total Elect. Equipment Cost	US $	35,279,712	35,775,441	39,375,590
Additional Cost (Switchyard	US $	3,014,000	3,014,000	3,014,000
Project Cost	Mill US $	**284.776**	**288.414**	**292.764**
surplus in relation to ALT 1	Mill US $		3.638	7.987
Present Value Cost	Mill US $	**330.412**	**334.633**	**339.679**
Annual energy generation	GWh	767.680	788.208	788.482
Annual benefits	Mill US $	**53.738**	**55.175**	**55.194**
Annual O&M cost	Mill US $	4.272	5.191	5.489
Annual (B-O&M)	Mill US $	49.466	49.983	49.704
Present Value Benefits	Mill US $	**493.035**	**498.189**	**495.412**
Cost / kWh	US $/kWh	0.0487	0.0492	0.0502
CRF	-	0.1497	0.1494	0.1463
C/B (aux. Val. Solver)	-	6.6796	6.6949	6.8340
EIRR	%	**14.967**	**14.933**	**14.629**

Table 4.14: Economic Assessment of different Number and Size of Turbine Units

As expected, the annual energy generation for the alternative with three identical Francis turbines (ALT 1) is less compared to other combinations. Furthermore, the alternative with 3 identical units represents the least cost solution compared to any other alternative. The alternative with two large and two smaller units (ALT 3) offers the highest flexibility in plant operation and requires the highest investment.

The alternatives with 3 turbine units (ALT 1 and ALT 2) are equivalent as regards their economic key parameters with a minor advantage for the concept with turbine units of identical size. The concept with two large and one small unit (ALT 2) has higher costs, but compensates the increase in cost by a higher power output compared to ALT 1.

In co-ordination with the Project Sponsor, the Consultant proposes the installation of 3 Francis units of identical size, i.e. ALT1.

This recommendation can be considered conservative. The present analysis is based on the simulation of run-of-river operation. At times of extremely low river flow, the available flow (with consideration of ecological releases) might me not sufficient to operate a Francis unit safely on continuous basis. With consideration of pondage operation, daily power generation can be maintained. Pondage operation may increase the annual energy generation by 1.5 to 2.0 % in a dry year. It makes the alternative with three identical turbine units even more attractive (approximately additional annual power generation of 12 GWh).

The Consultant recommends installation of three identical turbine units with the installed capacity of 3 x 60.8 MW (ex turbine) for the following reasons:

- The concept with three identical units results in the highest EIRR and the lowest specific generation costs;
- The concept with three identical units requires the minimum investment costs and least time for erection;
- The concept with three identical units requires the minimum operation and maintenance costs;
- Applying the concept of pondage operation during days with extremely low river flow may increase the economic key parameters further.

In view of the merits of the optional application of pondage operation, the Consultant makes the corresponding provisions in the feasibility design of the weir and intake structures to enable pondage operation between elevation 1494 and 1492 asl.

4.5.6 Optimization of Power Waterway Dimensions

The alingment of the power waterways was selected aiming on the most economic overall project layout taking into account the prevailing hydrological, topographic, geotechnical and econocmic boundary conditions. The alignment is based on the only feasible construction of conventional drill and blast method and the anticipated time of construction as discussed in the previous sections.

As part of the overall project optimization, the dimensions of the power waterway conduit system are optimized applying the relevant economic parameters based on the present value of cost of construction and energy and capacity forgone. The methodology and the corresponding results are presented in this chapter.

For the purpose of optimizing the conduit diameter, unit rates from similar hydropower projects (Duber Khwar HPP and Khan Khwar HPP, Patrind HPP) are applied after a critical review with the corresponding rates and escalation to account for the time of bidding and the reference date of the present Feasibility Study for the Madian HPP.

Economic Key Parameters

The effect of head losses on the revenues from energy generation and available capacity is assessed based on the duration curve of flow in Swat River at the power intake for a characteristic year for run-off river operation.

Based on the “Policy for Power Generation Projects – Year 2002” issued by the Government of Pakistan the tariff structure applicable to Madian HPP consists of an Energy Purchase Price (EPP) and a Capacity Purchase Price (CPP), the latter being 60% to 66 % of the total tariff.

The following assumptions are made by the Consultant:

Economic life time of the Project:	60 years	(for civil works)
Interest Rate	10 %	
Cost of power	0.070 US c /kWh	*
EPP	0.0251 US c /kWh	
CPP	0.0465 US c /kWh	

* defined in coordination with the Project Sponsor

Design and Hydraulic Key Parameters

The key design parameters for optimization of the tunnel diameters are:

Power Conduit	Q =	129.0 m^3/s
Max / Min Reservoir level	1494.0 – 1492.0 m asl	
Maximum Gross Head	H_{max} =	154.4 m
Minimum Gross Head	H_{min} =	146.0 m

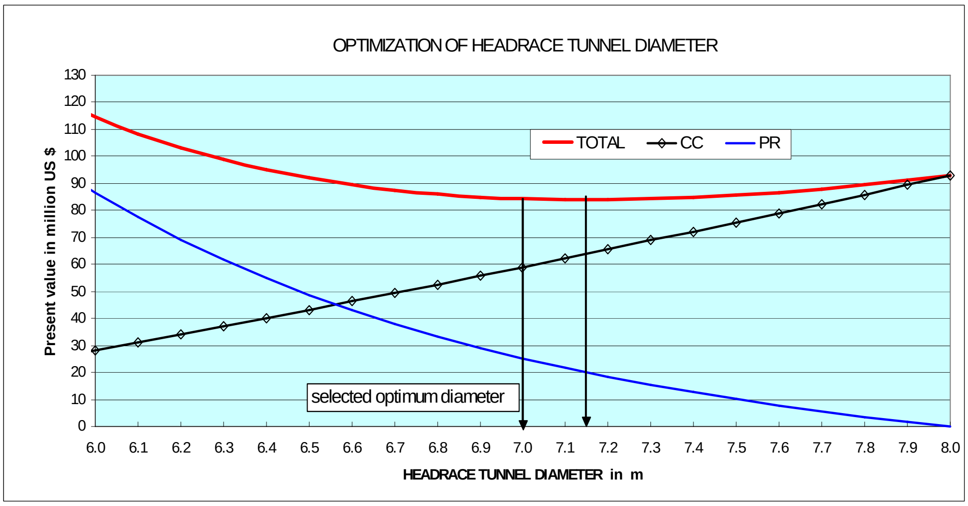

Figure 4.16: Optimization of Headrace / Tailrace Tunnel Diameter – Base Case

As indicated in Figure 4.16 the optimum headrace tunnel diameter is 7.15m. There exists a range of diameters from 6.95 to 7.40 m without significant variation of the optimization criterion. With the aim to minimize investment cost for an optimum project layout, a headrace tunnel diameter of 7.0 m is selected.

4.5.6.1 Cost-Benefit Optimization of Headrace Tunnel Diameter

As part of the headrace tunnel diameter optimization, sensitivity studies were made to analyze the effect of variation of selected parameters on the optimum diameter as follows:

a)	Cost overrun	-10 %	0%	+10%	+20 %	variation of unit rates
b)	Power Revenues	6.0	6.5	7.0	7.5	UScent/kWh tariff
c)	Interest rate	8 %	9 %	10 %	12 %	
d)	Construction Time	3.5	4.0	4.5	5.0	in years for tunnel only

Headrace Tunnel

a) Cost Overrun

Scenario		Optimum Diameter	Optimum Range (1.0 %)
Base Case	0 %	7.1 m	6.9 – 7.4 m
Cost overrun +	10 %	7.0 m	6.9 – 7.2 m
Cost overrun +	20 %	6.9 m	6.7 – 7.2 m
Cost overrun -	10 %	7.3 m	7.1 – 7.5 m

b) Power Revenues

Scenario		Optimum Diameter	Optimum Range (1.0 %)
Base Case	7.0 USc/kWh	7.1 m	6.9 – 7.4 m
Tariff	6.0 USc/kWh	7.0 m	6.8 – 7.2 m
Tariff	6.5 USc/kWh	7.1 m	6.9 – 7.3 m
Tariff	7.5 USc/kWh	7.2 m	7.0 – 7.4 m

c) Interest Rate

Scenario		Optimum Diameter	Optimum Range (1.0 %)
Base Case	10.0 %	7.1 m	6.9 – 7.4 m
Rate	8.0 %	7.4 m	7.2 – 7.6 m
Rat	9.0 %	7.3 m	7.1 – 7.5 m
Rate	12.0 %	6.9 m	6.7 – 7.1 m

d) Tunnel Construction Period

Scenario		Optimum Diameter	Optimum Range (1.0 %)
Base Case	4.0 years	7.1 m	6.9 – 7.4 m
Construction period	3.5 years	7.2 m	7.0 – 7.4 m
Construction period	4.5 years	7.1 m	6.9 – 7.3 m
Construction period	5.0 years	7.1 m	6.9 – 7.3 m

e) Extreme Assumptions

Scenario	Optimum Diameter	Optimum Range (1.0 %)
Base Case	7.1 m	7.0 – 7.3 m
Least Favourable Combination*	6.5 m	6.4 – 6.7 m
Most Favourable Combination **	7.5 m	7.4 – 7.8 m

* Cost overrun 20%; tariff 6.0 USc/kWh; interest rate 12 %; Construction period 5.0 years
** Cost overrun -10%; tariff 7.5 USc/kWh; interest rate 8 %; Construction period 3.5 years

The sensitivity study shows that the optimum headrace tunnel diameter is not extremely sensitive to the variations of the main parameters governing cost and revenues. For all cases the tunnel diameter is in the range from 6.5 to 7.5 m which reconfirms the selected diameter of 7.0 m.

4.5.6.2 Empirical Method for Optimization of Power Waterway Diameter

For optimization of diameters of the short pressure shaft / tunnel an empirical approach is applied. Based on the analysis of a large number of existing hydropower plants a strong correlation was found between optimum conduit diameter, type of lining, design head and discharge [Ref. FAHLBUSCH]. This correlation is applied for the waterways of the Madian HPP for the diameter of pressure shaft and pressure tunnel. The conduit diameter is calculated for a design discharge of 129 m³/s as follows:

Pressure Shaft – vertical, concrete lined:	$D = 0.56 \times Q^{0.48} = 5.77$ m
Selected D = 5.8 m	v = 4.88 m/s
Pressure Tunnel, steel lined:	$D = 1.12 \times Q^{0.45} \times H^{-0.12} = 5.45$ m
Selected D = 5.4 m	v = 5.63 m/s

The optimization of the diameter of the concrete lined part of the vertical pressure shaft results in a conduit diameter of 5.8 m. In its lower third the shaft is assumed steel lined. Starting from the steel lined section the conduit diameter reduces to 5.4 m. The corresponding design flow velocities coincide well with prototype data of a number of similar hydropower plants.

4.6 Hydraulic Design of Power Waterway System

4.6.1 Power Intake

For the selected design with three underground desanding caverns, air entrainment at the power intake would not cause a major problem for turbine operation, since any air entrained can be extracted and evacuated from the desander caverns. Nevertheless, the power intakes are arranged with sufficient submergence at minimum operation water level to prevent vortex formation. Such vortex formation shall be avoided to prevent unfavourable dynamic loads on the trash rack.

As defined by the hydraulic design criteria the required submergence is calculated applying Gordon's formula for a design discharge of 129.0 m³/s:

$$S/D = C \times Fr = C \times v / (g \times D)^{1/2}$$
$$S/D = 2.2 \times 3.36 / (9.80 \times 4.04)^{1/2} = 1.175 \qquad v = 3.36 \text{ m/s}$$
$$D = 4.04 \text{ m} \qquad S = 4.75 \text{ m}$$

For the equivalent tunnel diameter of 4.04 meter (of each intake barrel) and the coefficient C = 2.2 for laterally approaching flow, the required surcharge is as follows:

Minimum Operation Water Level	1492.00 m
Required submergence	4.75 m
Power Tunnel Diameter	4.04 m
Selected Invert Level	1483.00 m
Additional safety	0.21 m

Submergence is sufficient in case of pondage operation and a reservoir draw down of 2.0 m. At normal operation water level of 1494 m asl, adequate safety against vortex formation is provided with a safety margin of 2.21 m. Even in case of a slightly increased tunnel discharge for flushing the desander caverns, the safety margin is sufficient to prevent critical vortex formation. The Consultants recommends hydraulic model tests to reconfirm the prevention of vortex formation in the tender design stage.

The gross cross-sectional area of the trashrack is selected for a flow velocity not exceeding v_{tr} = 1.0 m/s for the design discharge of 43 m³/s.

Minimum requirement:	$A_{required}$ > Q / 1.0 m/s A > 43.0 m²
Selected Dimensions	A =W x H = 7.5 x 5.9 = 44.25 m² (v=0.97m/s)

When the auxiliary turbine unit is operated in addition to ensure the required ecological release to Swat River, the discharge may increase by up to 3.6 m³/s in the right inlet bay. The increase of the flow velocity at the trash rack will remain acceptable (v_{max} = 1.05 m/s).

4.6.2 Headrace Tunnel and Surge Tank

The first section of the headrace tunnel with an internal diameter of 7.0 m is arranged from the power intake to the desander caverns (situated some 2.1 km downstream of the power intake). The second section starts downstream of the desander caverns and proceeds to the surge tank.

Downstream of the surge tank a rock trap is arranged in the tunnel bottom for the following two reasons. First of all certain tunnel sections may be constructed unlined (only shotcrete support) and some rock material may fall from the tunnel soffit. Second it can not be excluded that solid particles fall or are thrown into the surge tank. The rock trap provides safety to prevent the turbine units from damage resulting from entrainment of large solid particles.

At the surge tank a maintenance gate is arranged to close the headrace tunnel during times of maintenance and inspection of the pressure shaft and manifold system without the need to empty and re-fill the entire headrace tunnel (0.36 million m³ water). Downstream of the maintenance gate the transition to the pressure shaft is arranged. The pressure shaft / tunnel commences with a vertical 90 degree bend and an internal diameter of 5.8 m in the concrete lined section and continues in the steel lined section with an internal diameter of 5.4 m. At the lower end of the pressure shaft a vertical 90 degree bend is arranged as transition to the steel lined pressure tunnel.
In flow direction, the headrace tunnel has a slope of 0.45 % (4.5 m/km) from the power intake to the desander cavern and 0.25 % (2.5 m/km) from the desander caverns to the surge tank. At the surge tank the headrace tunnel axis is at elevation 1452.4 m asl, which is sufficient to accommodate the maximum down surge. The design of the rock support and corresponding tunnel lining depending on the classification of rock along the tunnel alignment is discussed in detail in Section 3.4 of this Feasibility Report.

4.6.2.1 Hydraulic Starting Time of Waterways Need for Surge Tank

The length of the hydraulic waterways which is effective for hydraulic transient operation is measured from the free surface water level upstream of the turbine units to the nearest free water surface downstream.

For a project layout without surge tank the effective length is from the power intake to the power outlet whereas for a layout with upstream surge tank the distance is measured from the surge tank to the power outlet. Since the Madian HPP is characterized by a very long headrace tunnel, an upstream surge tank is mandatory as demonstrated below.

<u>Waterway System w/o ST</u>

$t_s = (L_i * v_i) / (g * H)$

t_s = 40735/ (g * 138)

t_s = 30.1 seconds

<u>Waterway System with ST</u>

t_s = 1735 / (g * 138)

t_s = 1.28 seconds

Reach No.	Length m	v m/s	L x v	L x v / g x H
Intake	68.00	3.42	232.68	0.17
Headrace	1997.00	3.35	6693.94	4.94
Desander inlet	33.50	3.42	114.63	0.08
Desander	256.00	0.25	63.62	0.05
Desander Outlet	112.00	3.42	383.25	0.28
Headrace	9401.00	3.35	31512.13	23.28
Pressure Shaft	111.47	4.88	544.25	0.40
High Pressure Tunnel	66.65	5.63	375.42	0.28
Manifold	55.80	6.08	339.45	0.25
Turbine inlet	6.60	7.51	49.57	0.04
Draft tube extension	53.80	3.10	166.98	0.12
Tailrace Tunnel	84.02	3.08	258.96	0.19

Table 4.15: Characteristics of Power Waterway System w/o Surge Tank

The hydraulic starting time of the waterways without surge tank would be an unacceptably high period of 30.1 seconds, however, with arrangement it attains a value of 1.28 seconds which is safely below the criterion of 2.5 seconds. The calculation confirms the need for arranging an upstream surge tank. It confirms that a downstream surge tank is not required and full flexibility in power plant operation, e.g. for load following is guaranteed.

4.6.2.2 Hydraulic Surge Tank Design

The following two alternative principal surge tank designs were considered:

a) Throttled cylindrical surge tank
b) Throttled cylindrical surge shaft with upper/lower chamber.

At the intersection between headrace tunnel and surge tank a throttle with a cross-section area of not less than 50 % of the pressure shaft is arranged in accordance with common design practice to damp pressure fluctuations.
In view of the expected high flow velocities at the throttle, the same shall be steel lined.

In accordance with common design practice and the hydraulic design criteria, the cross sectional area of the cylindrical surge tank is selected 70 % larger than the THOMA-Criterion (actual safety factor 1.7) to ensure adequate stability of plant operation. Based on the characteristics of the upstream waterways and the design water levels, the minimum surge tank dimensions were determined as shown in Table 4.16.

MADIAN HYDRO POWERH PROJECT
LAYOUT WITH FREE SURFACE POWERHOUSE ON LEFT BANK

Minimum cross section for surge tank at headrace tunnel

Total headrace tunnel discharge		Q_T =	129.00	m^3/s
Length of tunnel		L_T =	11800.00	m
Diameter of Tunnel		D_T =	7.00	m
Cross section area		A_T =	38.48	m^2
Minimum head loss		$h_{L,min}$ =	8.78	m
Velocity in tunnel		v_T =	3.35	m/s
Velocity head		$v_T^2/2g$ =	0.57	m
Minimum reservoir level		WL_{Rmin} =	1492.00	m
Maximum tailwater level		WL_{Tmax} =	1346.00	m
Minimum head difference		$H_{o,min}$ =	146.00	m
Minimum net head		$H_{n,min}$ =	137.22	m
A_{Smin} =	202.639	1.5 A_{Smin} =	303.96	m^2
D_{Smin} =	16.063	$D_{Srequired}$ =	19.67	m
		$D_{Sselected}$ =	21.00	m

$$A_{S_{\min}} = \frac{L_T \cdot A_T}{(H_{o_{\min}} - H_{L_{\min}}) \cdot (H_{L_{\min}} + 1.0 v^2 / 2g)} \cdot \frac{v^2}{2g}$$

Table 4.16: Dimensions of Surge Tanks: Headrace Tunnel

For load acceptance of the turbine units and subsequent full load rejection the following scenarios are assumed

LC-UP1) Load acceptance from partial load, not exceeding 50 % total load increase; subsequent full load rejection;

LC-UP2) Load acceptance of two units after synchronization; subsequent full load rejection;

LCDP1) Full load acceptance of one turbine followed by another turbine after a certain time interval; this interval is to be adjusted to detect the most unfavourable moment;

LCDP2) Load reduction by 50 per cent and subsequent complete load acceptance.

Load Case Maximum Upsurge

The load case is defined assuming that full load rejection occurs in the most unfavourable moment, i.e. when flow in the headrace tunnel is at its maximum (flow velocity) as the result of preceding partial load acceptance.

For the maximum upsurge analysis the following assumptions are made:

Turbine load acceptance after synchronization:	8.0 seconds (10 to 100 %)
Wicket gate closure at turbine load rejection:	8.0 seconds (100 to 0 %)
Reservoir water level:	1494 m
Maximum turbine discharge	129 m^3/s
Minimum tunnel roughness	0.1 mm

Maximum Upsurge was calculated for the following scenarios:

LC-UP1:

Q_T [m^3/s]	129	129	64.5	64.5	129	129	0	0
T [sec]	0	60	68	384	392	734	742	1500

LC-UP2:

Q_T [m³/s]	0	25.8	86	86	0	0
T [sec]	0	60	68	376	389	1500

Load Case Maximum Downsurge

The relevant load cases are defined assuming partial load acceptance following load rejection in the most unfavourable moment, i.e. when the flow in the headrace tunnel is at its maximum (reverse flow velocity) after full load rejection. For sake of conservativeness in the present tender design, full load acceptance of two units connected to the same headrace tunnel will be assumed as extraordinary load case for which the surge tank design shall guarantee the specified minimum freeboard margins. For the minimum downsurge analysis the following assumptions are made:

Turbine load acceptance after synchronization:	8.0 seconds (10 to 100 %)
Wicket gate closure at turbine load rejection:	8.0 seconds (100 to 0 %)
Reservoir water level:	1492.0 m
Maximum turbine discharge	129 m³/s
Maximum tunnel roughness	2.0 mm

Minimum Downsurge was calculated for the following scenarios:

LC-DP1:

Q_T [m³/s]	12.9	12.9	43	43	55.9	86	86	98.9	129	129
T [sec]	0	60	68	715	775	783	1364	1424	1432	2500

LC-DP2:

Q_T [m³/s]	129	129	64.5	64.5	129	129
T [sec]	0	60	68	434	440	900

4.6.2.3 Results of Surge Tank Simulation

For the most critical load cases surge tank water level, flow into and out of surge tank and turbine discharge with time are shown below in Figure 4.17a. For the load case resulting in the maximum downsurge the corresponding variation of surge tank water level, is shown in Figure 4.17b. The extreme surge tank water levels are given in Table 4.17 for all relevant load cases.

	Load Case	Water Level
Upsurge	LC-UP 1	1527.32
	LC-UP 2	1522.01
Down surge	LC-DP 1	1469.08
	LC-DP 2	1470.37

Table 4.17: Extreme Water Levels Surge Tank

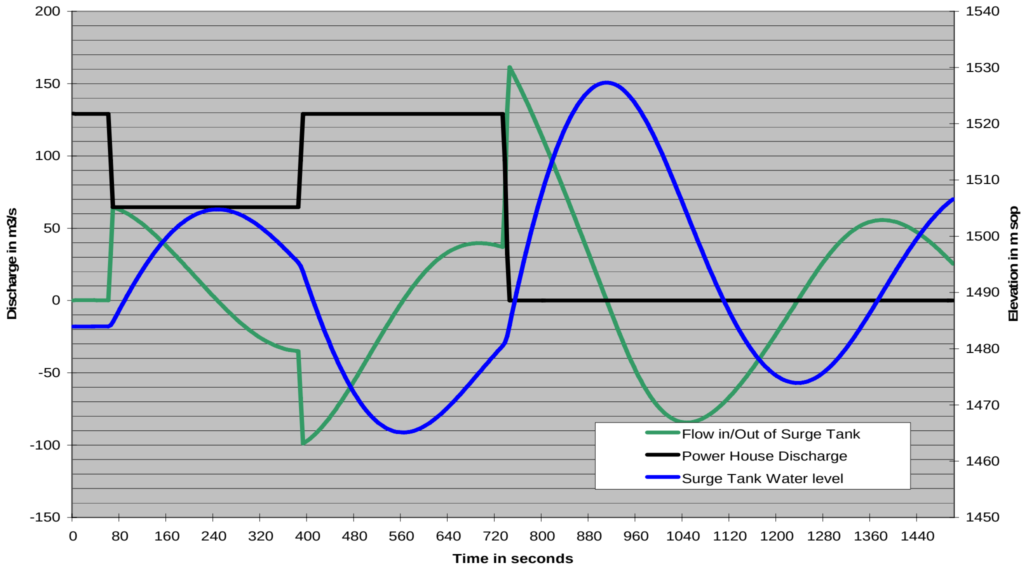

Figure 4.17a: Surge Tank: Combined Load Case–Maximum Upsurge

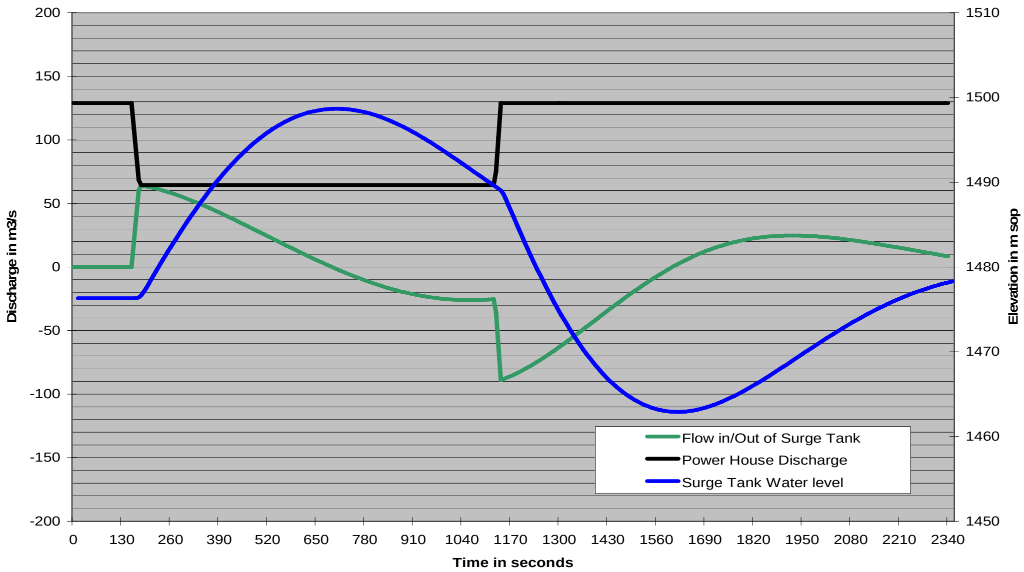

Figure 4.17b: Surge Tank: Combined Load Case–Maximum Down surge

4.6.2.4 Maintenance Gate

Within the surge tank a gate shaft is arranged which accommodates a maintenance sliding (bulkhead) gate. Such an arrangement permits maintenance of the pressure shaft, pressure tunnel and manifolds without the need to empty the 9.5 km long headrace tunnel downstream of the desander caverns. The gates will have the following approximate dimensions:

Gate dimensions: height x width = 5.8 x 4.5 m

The hydraulically effective area at the gate section is selected to be that of the downstream located pressure shaft with an internal diameter of 5.8 m.

4.6.2.5 Rock Trap

Immediately downstream of the gate shaft a rock trap is arranged in the tunnel bottom with the following dimensions to trap any large rock blocks or trash that may enter the headrace tunnel either at the intake, fall from the tunnel wall or soffit or that may fall into the surge tank.

Length: 8.4 m   Width : 5.8 m   Depth : 4.0 m

Access is possible either through the plug arranged in the adit to the headrace tunnel and through the gate shaft at the surge tank.

4.6.3 Pressure Shaft, Pressure Tunnel and Manifold

For ease of construction by means of the raise boring method the 5.8 m diameter pressure shaft is designed vertical. In view of the expected internal tunnel pressure and the rock mass characteristics in the pressure shaft area, concrete lining is required. In view of the internal pressure (transient analysis) steel lining is required in the lower third of the pressure shaft only. The two vertical bends of 90 degrees are arranged applying a radius of 17.4 m (R = 3.0 x D) thus representing a good compromise between economic design and low head losses. In the lower part the pressure shaft is steel lined and has an internal diameter of 5.4 m. The lining thickness increases from 20 to 28 mm towards the powerhouse cavern. The 10 m long horizontal steel lined pressure tunnel connects the pressure shaft with the manifold system. The internal diameter of the steel lined pressure tunnel is 5.4 m.

At the end of the pressure tunnel consecutively three manifolds branch off the main tunnel at an angle of 55 degrees. Each manifold has the internal diameter of 3.0 m including the confusor arranged as transition to the safety butterfly flap of 2.5 m nominal diameter. A straight alignment is provided towards the turbines over a length of at least 10 times the conduit diameter.

4.7 Head Loss Characteristics of the Power Waterway System

As shown in Table 4.18 the head losses of the waterways are in the order of 14.7 m for operation under rated conditions.

Underground Powerhouse , Section through Unit No. 2

Reach No.	Length [m]	Area [m²]	Perimeter [m]	Diameter [m]	Roughness [mm]	local head loss coefficient	Description of local head loss	Flow velocity	Head loss [m]
Intake	68.00	12.57	12.57	4.00	0.60	0.330	inlet loss, trahrack etc.	3.42	0.330
Headrace	1997.00	38.48	21.99	7.00	0.60	0.111	various bends	3.35	1.984
Desander inlet	33.50	12.57	12.57	4.00	0.60	0.000		3.42	0.065
Desander	256.00	173.04	46.63	14.84	0.60	0.000	Dividing flow & bend 55°	0.25	0.001
Desander Outlet	112.00	12.57	12.57	4.00	0.60	0.860	R/D=3	3.42	0.732
Headrace	9401.00	38.48	21.99	7.00	0.60	0.035	surge tank	3.35	9.062
Pressure Shaft	111.47	26.42	18.22	5.80	0.60	0.369	2 x bend 90°, R/D = 3	4.88	0.732
High Pressure Tunne	66.65	22.90	16.96	5.40	0.10	0.000		5.63	0.184
Manifold	55.80	7.07	9.42	3.00	0.10	0.040	dividing flow 55 °	6.08	0.431
Turbine inlet	6.60	5.73	8.48	2.70	0.10	0.050	confusor, butterfly valve	7.51	0.216
Draft tube extension	53.80	13.85	13.19	4.20	0.60	0.570	combining flow 55°	3.10	0.361
Tailrace Tunnel	84.02	41.85	22.93	7.30	0.60	1.100	outlet, gate slots	3.08	0.598
							Head loss hl =		14.697
							hl = N x 10^{-6} x Q²		883.158

* draft tube loss is included in turbine efficiency

Table 4.18: Head Loss Characteristics of Waterways, Underground Powerhouse

4.8 Water Hammer Analysis

For verification of pressure conditions along the power waterways, a transient analysis was carried out applying the Engineer's software tool WATHAMMER based on the method of characteristics and the following basic parameters:

Reservoir Levels	max 1494.0 m asl	min 1492.0 m asl
Tailwater Levels	max 1346.0 m asl	min 1339.0 m asl
Operation Mode	Load Rejection of 3 units within 8 seconds	
	Q_T =3 x 43 = 129 m³/s	

The load cases and corresponding input parameters considered in this transient analysis are similar as applied to the hydraulic design of the surge tank (see Section 4.6.2).

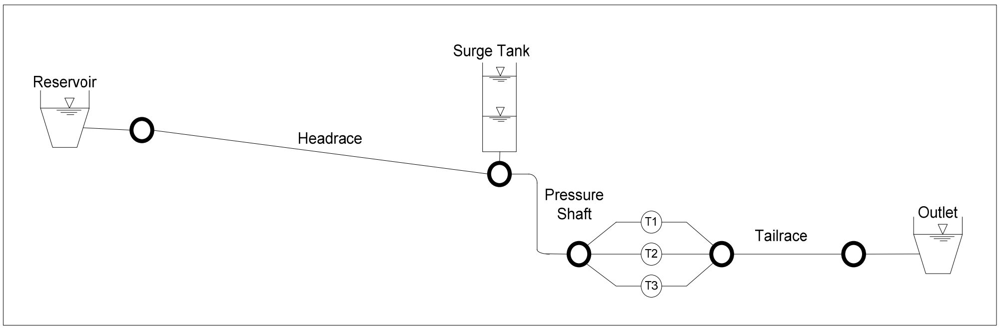

Figure 4.18: Schema of the Numerical Model for Water Hammer Analysis of the Madian Hydropower Project for Transient Analysis

Occurrence of maximum and minimum pressures along the power waterway system is different upstream or downstream of the surge tank. In the headrace tunnel highest heads result largely from the gradual water level oscillation in the surge tank which are superimposed by the pressure fluctuation induced by water hammer. However, as the result of the length of the headrace tunnel of 11.8 km, the water hammer induced pressure fluctuations are largely damped when maximum and minimum water levels establish in the surge tank (see Figure 4.19). Between surge tank and turbines as well as between turbines and tailrace, the extreme pressure conditions are largely related to water hammer as can be seen in Figure 4.19

Four load cases were defined as combinations of maximum and minimum reservoir and tailwater levels in combination with the anticipated maximum and minimum roughness conditions of the power waterway system.
The resulting minimum and maximum head along the power waterway system is given in Table 4.19 for the assumed boundary conditions. The variation of head with time along the power waterway system is presented in Figure 4.19 for a load case with reservoir level 1494 m and tailwater level of 1339 m for load rejection of all three units following partial load acceptance at the most unfavourable moment.

Load Case		Headrace Tunnel at ST		End of Pressure Shaft		U/S Turbine		D/S Turbine		Start Tailrace	
Res. Level	Tailwater	minimum	maximum	minimum	maximum	minimum	maximum	minimum	maximum	minimum	maximum
m asl	m asl	m Head	m Head	m Head	m Head	m Head	m Head	m Head	m Head	m Head	m Head
1492*	1339*	1475,1	1523,1	1469,5	1523,6	1464,4	1523,7	1332,8	1344,9	1334,2	1343,8
1494**	1339*	**1471,5**	1518,9	**1462,9**	1519,2	**1457,2**	1519,4	**1332,8**	1345,0	**1334,2**	1343,8
1494*	1339*	1477,0	**1525,1**	1472,4	**1525,6**	1467,3	**1525,8**	1431,3	1344,9	1334,2	1343,8
1492**	1339**	1477,3	1524,9	1471,2	1525,2	1465,9	1525,4	1437,9	**1345,0**	1334,2	**1343,8**
	maximum		**1525,14**		**1525,59**		**1525,76**		**1345,00**		**1343,80**
	minimum	**1471,52**		**1462,85**		**1457,22**		**1332,80**		**1334,20**	

Table 4.19: Minimum and Maximum Internal Head along the Power Waterways
* minimum roughness condition ** maximum roughness condition

The hydrodynamic pressure rise as the consequence of transient phenomena is limited to 23 % of static head in the headrace tunnel.

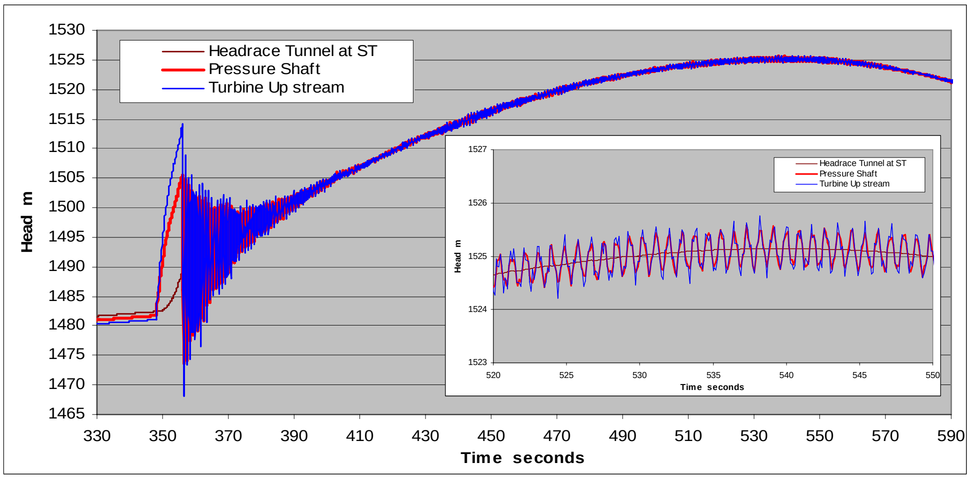

Figure 4.19: Fluctuation of Head in the Upstream Waterway System after Full Load Rejection of 3 units

In this case a maximum head of 1530 m applies to the headrace tunnel design equivalent to 8 bar of internal water pressure as the design parameter. The following design heads are recommended for each power waterway component:

Headrace Tunnel:	Maximum	1530 m	max 8 bar
Pressure Shaft	Maximum	1530 m	max 20 bar
Manifold	Maximum	1530 m	max 20 bar

The throttled surge tank dampens sufficiently pressure fluctuations and results in an economic design of the headrace tunnel.

4.9 Hydraulic Design of the Desander System

4.9.1 General Aspects of the Design of Desanding Facilities

It shall be born in mind that sediments in suspension will unavoidably result in a certain wear and tear, in particular at the turbine runner. The extent of the abrasion depends largely on the concentration, size and mineralogical characteristics of the sediment particles on one hand and the turbine type, materials used and runner speed on the other. This abrasion and the resulting need for overhaul and replacement of runners cannot be avoided when water with high sediment content is diverted for power generation. Desanding facilities are arranged to control or better say to reduce the frequency of the required change and overhaul of turbine runners.

During the high flow season the Swat River has the potential to transport large quantities of sediments in suspension as well as bed load. Sediment concentrations of up to 10,000 ppm have been recorded in Swat River (see Figure 3.9 in Section 3.1). From the petrographic analysis of rock and sand samples it can be assumed that quartz minerals may make up to 10 % of the suspended sediments.

The proposed design of the weir structure and the flushing outlets ensures that coarse sediments (sand, gravel and cobble) can be prevented from entering the power intake and will be flushed through the sediment sluice. Sand and silt fraction will remain largely in suspension in the small reservoir of the Madian HPP and unavoidably enter the power intake. Desanding facilities are, therefore, required to remove most of the sediment particles larger than the design particle diameter of 0.2 mm (see Hydraulic Design Criteria).

Desanding facilities are preferably arranged close to the weir structure at the free surface. The topographical and geological conditions make the arrangement of open air desanding basins impossible in the case of the Madian HPP due to the narrow valley and steep valley slopes. For this reason, underground desanding facilities are arranged for the Madian HPP.

For the design discharge of 129 m^3/s, the minimum number of desander caverns (basins) is three to avoid excessive large caverns which otherwise may even exceed the powerhouse cavern in cross-section area.
For a selection of three or four caverns the resulting height of the desander cavern (including the flushing ducts) exceeds the depth of flow at the weir structure and evacuation of the sediment water mixture from the desander caverns by gravity is not possible at the weir site. Therefore, the desander caverns were arranged further downstream. A suitable site was identified some 2.1 km downstream of the weir.

4.9.2 Alternative Concepts of Desanding Facilities

The Consultant studied possible alternative types of desanding facilities that may be applied to the feasibility design. The publication of ORTMANN (2006) can be considered of the state of engineering in desander design with particular view on design practice and operation experience of more than 100 desander facilities that have been implemented during the last 40 years.
Among the different types of desanding facilities the following are known and relevant for practical application for hydropower projects:

> Long basin-type desander, with subdivision made for the type of flushing system used, with intermittent or continuous flushing (System Büchi, Bieri, Serpent Sediment Sluicing System (“4-S”))

As discussed in more detail in Section 7.5.3, the Consultant analysed the existing desander types and flushing systems and elaborated a modified Bieri – desander flushing system. In the proposed system “rubber-hoses” substitute the mechanical valves which controls the evacuation of sediments from the desander into the flushing duct by variation of its internal pressure.
Under normal operation conditions the desander caverns are operated under pressure. Intermitted flushing of the chamber sections is possible by operating the proposed pneumatic rubber seals.

The seals open automatically when a certain quantity of sand accumulates and exerts a certain pressure on the seal or they may be operated at pre-defined intervals. The individual sections of a desander basin (cavern) are sealed by maintaining a defined pressure in the rubber hose by a pump, e.g. 5 bar. For flushing of a desander basin section, the internal pressure in the (rubber-hose) seal is reduced gradually so that a narrow gap opens between sealing and concrete wall and the sediment may be flushed out through the flushing duct.

The discharge in the flushing ducts is controlled by the flushing gates situated at the junction to the central flushing tunnel. From there the water-sediment flow is returned to Swat River by free surface flow at the confluence with Ashkon Nullah. The flushing tunnels shall be lined with concrete.

For information Table 4.20 shows key parameters of Hydropower Projects with desanding facilities of similar dimensions as to the Madian HPP.

Project	Installed Capacity	Total Powerhouse Discharge	Net Head	Design Grain Diameter excluded > 90%	Maximum Sediment Concentration
	MW	m³/s	m	mm	mg/l
Lower Marsyangdi, Nepal	3 x 23	100..0	83	0.1	14000
Arun 3,Nepal	6 x 26.67	160.0	286	0.22	10000
Upper Arun	6 x 85	114.4	506	0.20	8000

Table 4.20: Basic data of desanding facilities of other hydropower plants

4.9.3 Design of Desander Facilities for the Madian HPP

The Consultant determined the required dimensions for the long basin desander applying his program DESANDER which is based on the theoretical approaches of CAMP and SARIKAYA. The results of the design and thus the key parameters of the desander caverns are given in Table 4.21.

Desander:		
Design discharge	129	m^3/s
Number of settling chambers	3	
Effective length of chamber	206	m (without transition)
Width of chamber	13.7	m
Average depth of chamber	16	m
Mean velocity	0.2	m/s
Grain size to be excluded	0.20	mm

Table 4.21: Technical Key Parameters of the Desander Works

For verification of the selected design the below given recommendations published by GIESEKE & MOSONYI (1998) are checked:

Criterion	Proposed Design
B < L / 8	13.7 < 206 m / 8 (= 25.75)
H : B = 1.25 : 1.0	16.85 : 13.7 = 1.23 : 1.0

The desanding works comprise three 206 m long desander caverns. Manifold systems branch off the headrace tunnel upstream and return the flow downstream to the headrace tunnel again. Maintenance gates are arranged upstream and downstream of each desander cavern to enable inspection and maintenance of one cavern while the others are in operation.

For design details reference is given to the drawings given in Volume VII of this Feasibility study Report. The efficiency of the desander works was assessed applying the approach of SARIKAYA and SCHRIMPF to the selected design. With increasing diameter the rate of removal increases as shown in Table 4.22 below.

Particle Diameter mm	Settling velocity mm/s	Rate of Removal %
0.40	58.0	100.0%
0.20	22.0	98.0%
0.15	15.0	82.0%
0.10	9.0	56.0%
0.06	3.5	22.0%
0.02	0.4	0.0%

Table 4.22: Rates of Removal of Suspended Sediments at the Desander Caverns

Approximately 98 % of the sediments are removed from the water sediment mix at the desander works of the sediment fraction of the design particle diameter of 0.2 mm. For fractions with larger particle size the removal rate approaches 100 % and for particles of 0.1 mm diameter the removal rate is still above 50 %. Table 4.22 demonstrates that the selected desander design is adequate. Based on the sediment removal rates given in Table 4.22, the efficiency of the operation of the desander was simulated for days with "normal" and "extraordinary" flow and sediment concentration. As given in the Report on Hydrology and Meteorology (Feasibility Study, Volume IV) the composition of the suspended sediments in Swat River is as follows:

Clay 19 % (D < 0.0055 mm)
Silt 53 % (0.0055 mm < D < 0.0625 mm)
Sand 28 % (D > 0.0625 mm)

The results of this analysis in terms of rates of removal and sediment concentrations at the turbine units are given in Table 4.23. The amount of clay particles removed at the desander is marginal, silt fractions are removed to a certain extent and nearly all fine sand particles.

Description of Event	River Discharge m^3/s	Susp. Sediment Load in Swat River t/day	Sediment Concentration u/s desander mg/l (ppm)	Desander Removal Rate* %	Sediment Removal at desander t/day	Sediment Concentration d/s desander mg/l
average	25	89	41	37.1%	33	25.9
extreme	25	343	159	37.1%	127	99.7
average	50	232	54	37.1%	86	33.8
extreme	50	893	207	37.1%	332	130.0
average	129	862	77	37.1%	320	48.6
extreme	129	3,313	297	37.1%	1,230	186.8
average	250	2,151	100	37.1%	412	62.6
extreme	250	8,269	383	37.1%	1,585	240.6
average	500	5,608	130	37.1%	537	81.6
extreme	500	21,560	499	37.1%	2,066	313.7
average	800	10,740	155	37.1%	643	97.7
extreme	800	41,293	597	37.1%	2,473	375.5

Table 4.23: Rates of Removal of Suspended Sediments at the Desander Caverns

4.10 Powerhouse and Tailrace

Based on the foregoing analyses a concept with underground powerhouse was selected. The turbine draft tube extensions form a manifold system that joins into a single tailrace tunnel of 93 m length. At its end a power outlet structure at the left bank of the Swat River is arranged.

4.10.1 Underground Powerhouse

The proposed underground powerhouse is a conventional cavern structure for three identical Francis units with vertical axis of 60.8 MW installed turbine capacity and a runner diameter of 2.22 m. Within the powerhouse the main inlet butterfly valve of nominal diameter of D = 2.5 m is arranged immediately upstream of each turbine unit. After passing through the turbines, the water is discharged via the draft tube extension into the common tailrace tunnel and from there to the outlet bay.

Each draft tube can be closed by a draft tube flap gate for maintenance or repair of a turbine unit. The distance between the turbine unit centre lines is 15.15 m. Alternative arrangements may be studied in the tender design and will be submitted by the EPC Contractors in their technical proposals.
The tentative (excavated) dimensions of the powerhouse cavern are as follows:

Width	20.0 m
Length	73.5 m above generator floor
	48.8 m below generator floor
Height	35.0 m at draft tube
	31.0 m at valve floor
	20.6 m at service bay

The turbine setting is defined according to the requirements to prevent cavitation at the turbine units at elevation 1336.0 m asl based on the minimum tailwater level of 1339 m asl for the selected turbine and the prevailing hydraulic conditions.

On both lateral walls of the cavern crane beams of reinforced concrete are arranged anchored to the rock for the overhead travelling crane. A single service and erection bay is provided in the northern part of the cavern at elevation 1345.45 m. For access to the powerhouse cavern and further to the transformers cavern a common access tunnel is provided with a width and height of 5.5 m. Between powerhouse and transformer cavern the access tunnel is horizontal and 6.3 % inclined between tunnel portal and powerhouse cavern.

The No. 10 single phase transformers are arranged in a small cavern which is arranged at 30 m distance from the powerhouse cavern. Aiming on a high reliability it was decided to consider a SF6 gas insulated switchyard arranged underground adjacent to the transformer cavern.

The transformer cavern is approximately 9.0 m wide, 7.4 m high and 64 m long (excavated dimensions) whereas the switchyard cavern is larger in cross section with 13.7 m width and 10.5 m height. The inclined cable and ventilation shaft starts from the switchyard cavern and leads to the terminal structure at its portal for interconnection to the 220 kV high voltage transmission line.

The powerhouse cavern contains the machine hall, control and monitoring room, accessory hydro-mechanical and electrical equipment as well as store and workshop facilities. The dimensions of the powerhouse given in this report are preliminary and governed by the sizes of the turbine-generator-units and the space requirements for the electrical equipment, such as high and low voltage equipment, battery room, standby generator, auxiliary transformer etc. These may subject to variation according to the design of the particular suppliers of the electro-mechanical equipment. Detailed design drawings are provided in Volume VII of this Feasibility Report which demonstrate in detail the selected feasibility design. The rock support for the rock mass characterized as good to fair rock is discussed in detail in Section 3.4 of this report and in Volume III of this Feasibility Report.

4.10.2 Tailrace System and Outlet Structure

A tailrace tunnel is arranged to convey the turbine discharge to the power outlet structure on the left bank of the Swat River. The dimensions of the draft tube and its rectangular exit cross-section are determined based on general turbine design principles for vertical Francis turbine units (see Section 8). The following dimensions of the draft tube exit were selected:

Draft Tube outlet: W x H = 4.20 x 3.40 m exit velocity 3.1 m/s

At the end of each of the three draft tubes an offset of 10 cm is arranged. The three draft tube extensions join at the starting point of the tailrace tunnel. The concrete lined draft tube extension tunnels have an equivalent diameter of 4.2 m to maintain the flow velocity at the draft tube exit. Similarly the headrace tunnel diameter is selected for the design flow velocity of 3.1 m at rated conditions resulting in a tailrace tunnel diameter of 7.3 m. From its starting point to the portal of the outlet structure the length of the concrete lined headrace tunnel is 93.5 m.

According to the hydraulic design criteria the maximum water level in the event of the design flood at the power outlet structure is:

Powerhouse Design Flood: $HQ_{1,000}$ = 1,785 m³/s
Design Water Level 1346.2 m asl (SoP)

The elevation of the invert at the outlet structure is selected at elevation 1336.0 m asl. The working and access platform are arranged at elevation 1355.0 m asl safely above the above the maximum flood water levels of the Swat River at the powerhouse location. Slots for setting stop logs are arranged at the power outlet bay for maintenance or inspection of the tailrace tunnel.

Annex A-4.1: Design Criteria for Weir Structure
Source: ASCE and USBR, Design of Small Dams

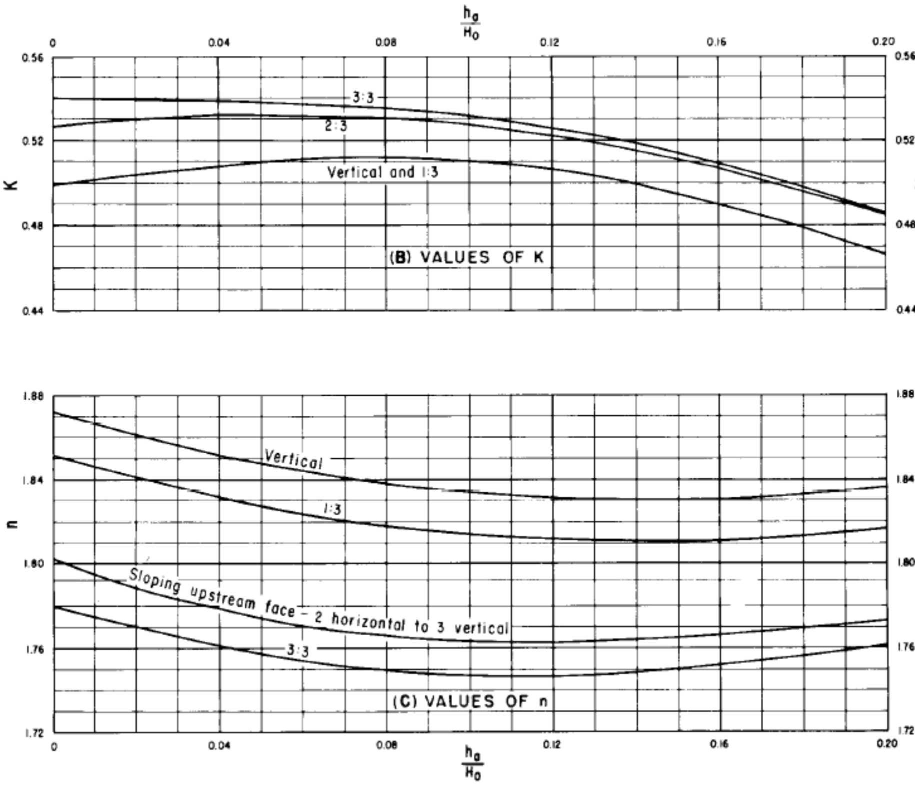

Figure A-4.1: Design of Small Dams: Design Parameters for the Spillway Ogee

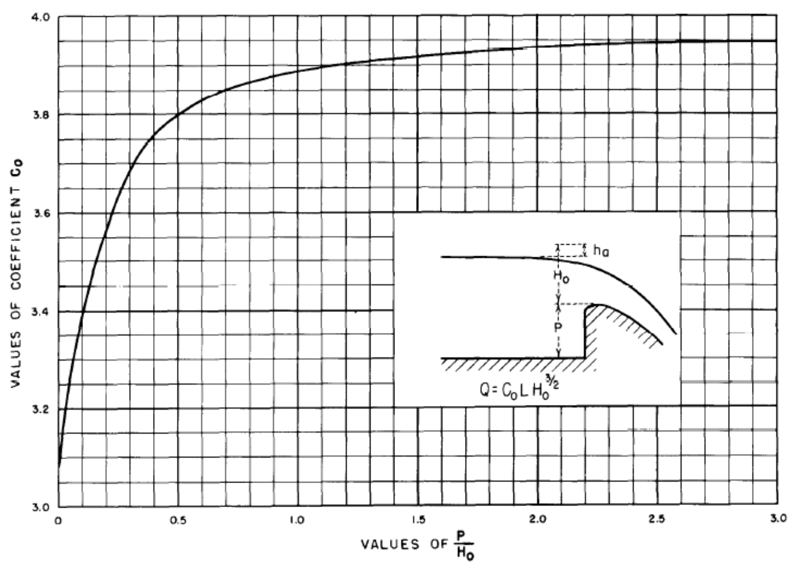

Figure A-4.2 Discharge Coefficient for Spillway with Standard Ogee

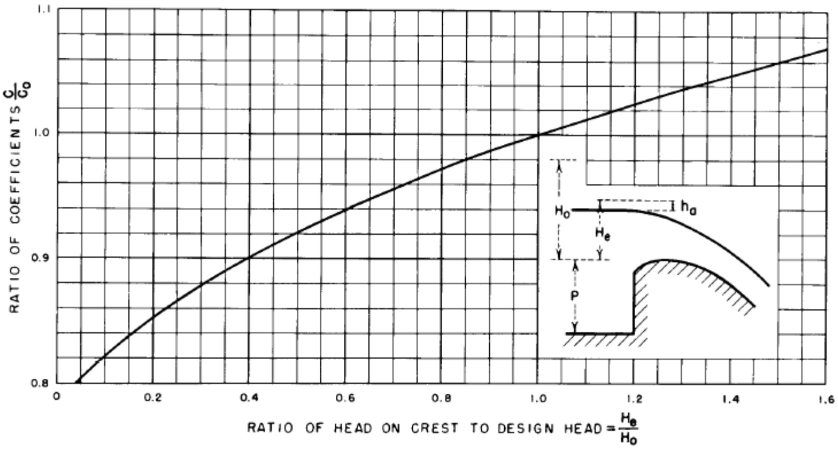

Figure A-4.3 Variation of Discharge Coefficients with Head Conditions

The contraction coefficients K_p and K_a are affected by the shape of the pier nose and abutment shape respectively.

Values of K_p:

- 0.02 Square-nosed piers with corners rounded on a radius equal to about 0.1% of the pier thickness
- 0.01 Round-nosed piers
- 0.00 Pointed-nosed piers

Values of K_a:

- 0.20 Square abutments with headwall at 90° to direction of flow
- 0.10 Rounded abutments with headwall at 90° to direction of flow, with rounded corners having a radius of 0.15 - 0.5 x H_0
- 0.00 Rounded abutments with headwall placed at not more than 45° to the direction of flow and radius > 0. 5 x H_0

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Annex A-4.2: Design Criteria for Calculation of Head Losses in Power Conduit Systems

Lining Type	**Minimum** k_s [mm]	**Mean** k_s [mm]	**Maximum** k_s [mm]
Concrete cast–in–situ steel forms	0.10	0.60	2.00
Concrete segmental lining/wooden forms	1.00	1.50	3.00
Steel lining – coated	0.05	0.10	0.30
Bored Tunnels not shotcreted	3.00	4.00	6.00
Bored Tunnels shotcrete lined	6.00	8.00	10.0
Rock Drill & Blast : normal blasting, well trimmed	100.0	150.0	300.0
Rock Drill & Blast : well- trimmed and shotcreted	50.00	70.0	100.0

Table A-4.1: Equivalent Sand Roughness of Waterways

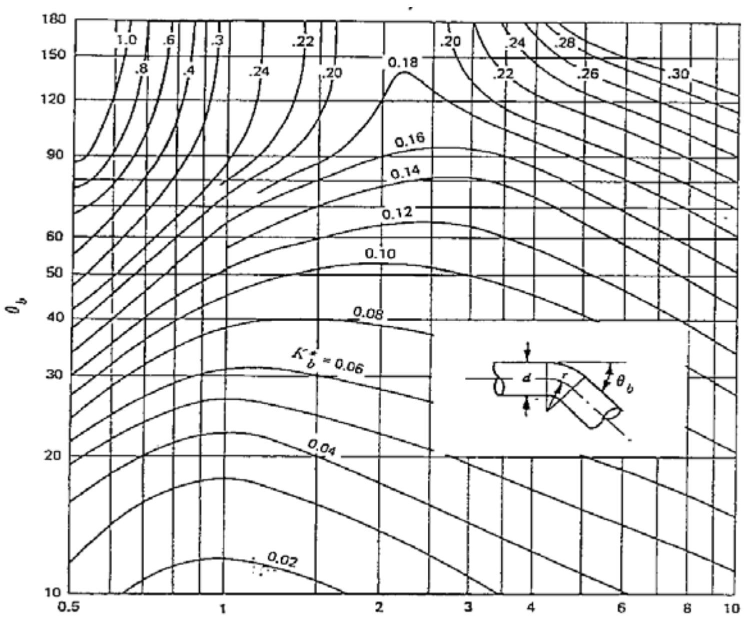

Figure A-4.4: Head Loss Coefficient for Circular Bends in Conduits