Appendix G
Dam Development Study

FEASIBILITY STUDY
ON
THE DEVELOPMENT OF MUNDA DAM MULTIPURPOSE PROJECT
IN
ISLAMIC REPUBLIC OF PAKISTAN

FINAL REPORT
VOLUME III
SUPPORTING REPORT

Appendix G : Dam Development Study

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APPENDIX G DAM DEVELOPMENT STUDY

G1 Plan Formulation

G1.1 Concept of Plan Formulation

The development concept of the Project is of multi-purpose, combining power generation, irrigation, and flood control. Investigation results on the present situation in these sectors and studies of the development plans are discussed in Chapters 4, 5 and 6 of the Main Report and Appendices D, E and F, respectively. The expected development plans presented in these Chapters and Appendices for the respective sectors are briefed as follows:

1) In the government's power policy, the priority is given to hydropower development to utilize indigenous sources of energy. The Munda power station is planned with a commissioning date of early 2010s to cope with the growing demands according to the GOP development program. Peak power generation is expected for the Munda power station so that a merit of the hydropower generation responding immediately to the demand is fully attained.
2) The new irrigation scheme extends in both sides of the Swat River. The planned CCA is 4,066ha for the left bank area and 2,043ha for the right bank area, total 6,109ha. A tunnel system was selected as the feeder system at the left bank, while a lifting system with pumps was planned at the right bank from the economical viewpoint. Some deficits of irrigation water supply in the LSC were found, which are to be supplemented by the Munda Dam. The water supply of 8.49 m^3/s (equivalent to 300cusec) to the civil canal located downstream of the Munda Headworks is also required.
3) An optimum flood control space was decided to be 100 million m^3 through comparative study of the space alternatives and their benefits. The space is allocated above FSL.

Taking into account thereof, the optimum development scale was formulated through the following three phases of optimization process:

Phase 1 - development of alternatives conceived based on magnitude of inflow and storage capacity, water requirements for power generation and irrigation, required flood control space, and requirements of re-regulation weir including options of improvement of the existing Munda Headworks;

Phase 2 - comparison of alternatives and selection of optimum scale through simulation made by reservoir operation simulation model and optimization; and

Phase 3 - scheduling of the optimum power development timing in consideration of power demand forecast and other power development programs in hand within the national power system.

G1.2 Development Site

The Pre-F/S report published in 1992 proposed the Munda Dam site at around 5km upstream of the existing Munda Headworks. In the "Preliminary Feasibility Report for Lower Swat Gorge Development" published by WAPDA in August 1969, five alternative dam sites had been identified in the lower Swat gorge on the map as shown in Table G1.1 and Figures G1.1 and G1.2, but for reason of security, four dam sites excluding the Munda had not been reconnoitered. Of these, two dam sites, Bazargai and Ambahar were located at a river-distance of 6.4 km and 18.5 km from the Munda, sufficiently far away from the Munda dam site. These were, however, neither visited nor studied in the Pre-F/S. In view of maximum exploitation of the hydro-potentials, it was considered necessary to examine these two sites further.

In the dam development plans at Bazargai and Ambahar described in the above preliminary feasibility report, it is proposed to create a huge reservoir, some 9.2 billion m3 (7.5 MAF) of storage volume, by constructing a 280-m-high dam which would submerge the upper Swat plain, now densely populated. It is obvious that the development plan with wide submergence of the settled area should never be acceptable in all respects. Thus, in examination of upstream dam site alternatives, the possible maximum reservoir water level was tentatively taken as an elevation of 580 m from environmental viewpoint on the basis of the 1:50,000 scale map.

The upstream dam sites were examined looking at narrow gorges which are topographically higher than El. 580 m in the map of 1:50,000. The examination identified three dam sites at river-distances of 3 km, 5 km, and 11.5 km from the Munda as shown in Figure G1.3, and the table below.

Features at the Alternative Dam Sites

Dam Sites	Munda Site	3 Km Site	5 Km Site (Bazargai Site)	11.5 Km Site (Ambahar Site)
Max reservoir level	EL. 580 m	EL. 580 m	EL. 580 m	EL. 580 m
Dam crest	EL. 583 m	EL. 583 m	EL. 583 m	EL. 583 m
Dam height	226 m	219 m	211 m	190 m
Installed capacity	700 MW	700 MW	700 MW	700 MW

In order to compare the alternative dam sites on the same basis, the following assumptions are applied:

1) the same maximum reservoir water level, say, 580 m with general layout based on concrete face rockfill dam as shown in Figure G1.4,
2) the same planning values as appeared in Pre-F/S report, i.e. flood storage volume, water release for power and irrigation, and reservoir operation model,
3) cost parameters based on the collected data, and

4) benefit derived from power generation alone as other benefits are marginal and almost common to all the alternatives.

The comparison was made based on capitalized energy cost, i.e. present worth of base cost including O & M over the project life divided by present worth of energy generated over the project life at a discount rate of 12%. The result revealed that the Munda Dam site alternative has the least capitalized energy cost among the others as tabulated below:

Capitalized Energy Costs

Dam Sites	Munda Site	3 Km Site	5 Km Site (Bazargai Site)	11.5 Km Site (Ambahar Site)
Base cost * (Million US$)	1,464	1,688	1,362	1,242
Annual energy production (GWh)	2,293 (2,215)	2,159	2,055	1,761
Capitalized energy cost (US$/kWh)	0.038 (0.040)	0.048	0.041	0.043

* :Base cost excluding taxes and duties
( ) :Figures in parentheses are values under peak operation condition.

The table shows that the Munda site is economically most preferable among the alternatives. The field reconnaissance made in October 1998 confirmed the appropriateness of the Munda Dam site especially in terms of topography and geology.

G1.3 Development Scheme Optimization

G1.3.1 Development Scale Alternatives

The optimum development scale is generally the one which produces maximum net benefit in the economic indices after estimation of overall benefits of power generation, irrigation and flood control through the reservoir operation simulation study and cost/benefit calculation. Further consideration is also required to decide the optimum scale in view of dam site topography and geology, and sedimentation deposited at the upstream end of the reservoir. The development scale optimization study was carried out for the proposed Munda Dam site.

The possible maximum reservoir full supply level (FSL) was determined to be EL. 580 m from environmental aspects, which does not cause wide submergence of the settled area in the upstream end of the reservoir area based on the 1:10,000 scale map, as well as topographic limit at the Munda Dam site where a saddle dam may be required for the higher elevations.

Sediment level in the reservoir was assumed for each development scale alternative based on the estimated volume of accumulated sediment up to 100 years after impounding. The detail of the sediment volume estimates is presented in Section 3.3 Hydrology of the Main Report.

Lowest minimum operating level (MOL) of the reservoir was set taking into account the sediment level and enough water depth above the power intake sill, which is equivalent to about twice the power tunnel diameter, to prevent intake of water from air entrapment through turbulence and vortex.

In selecting development scale alternatives, all the issues mentioned above were taken into account. The following is a list of the development scale alternatives selected for comparison:

Development Scale Alternatives

No.	Case	FSL	MOL	Sediment Level	Dam Crest	Dam Height	Effective Storage
(1)	505-487	EL. 505 m	EL. 487 m	EL. 470 m	EL. 514 m	164 m	213 mil m^3
	505-490	- do -	EL. 490 m	- do -	- do -	- do -	171 mil m^3
	505-495	- do -	EL. 495 m	- do -	- do -	- do -	120 mil m^3
(2)	510-488	EL. 510 m	EL. 488 m	EL. 470 m	EL. 519 m	169 m	260 mil m^3
	510-495	- do -	EL. 495 m	- do -	- do -	- do -	188 mil m^3
	510-500	- do -	EL. 500 m	- do -	- do -	- do -	137 mil m^3
(3)	515-490	EL. 515 m	EL. 490 m	EL. 471 m	EL.524 m	174 m	308 mil m^3
	515-495	- do -	EL. 495 m	- do -	- do -	- do -	257 mil m^3
	515-505	- do -	EL. 505 m	- do -	- do -	- do -	137 mil m^3
(4)	520-491	EL. 520 m	EL. 491 m	EL. 471 m	EL. 529 m	179 m	366 mil m^3
	520-500	- do -	EL. 500 m	- do -	- do -	- do -	273 mil m^3
	520-510	- do -	EL. 510 m	- do -	- do -	- do -	137 mil m^3
(5)	525-493	EL. 525 m	EL. 493 m	EL. 472 m	EL. 534 m	184 m	433 mil m^3
	525-505	- do -	EL. 505 m	- do -	- do -	- do -	293 mil m^3
	525-515	- do -	EL. 515 m	- do -	- do -	- do -	156 mil m^3
(6)	530-494	EL. 530 m	EL. 494 m	EL. 473 m	EL. 539 m	189 m	521 mil m^3
	530-505	- do -	EL. 505 m	- do -	- do -	- do -	381 mil m^3
	530-520	- do -	EL. 520 m	- do -	- do -	- do -	176 mil m^3
(7)	535-495	EL. 535 m	EL. 495 m	EL. 473 m	EL. 544 m	194 m	589 mil m^3
	535-510	- do -	EL. 510 m	- do -	- do -	- do -	401 mil m^3
	535-525	- do -	EL. 525 m	- do -	- do -	- do -	176 mil m^3
(8)	540-496	EL. 540 m	EL. 496 m	EL. 473 m	EL. 549 m	199 m	667 mil m^3
	540-500	- do -	EL. 500 m	- do -	- do -	- do -	625 mil m^3
	540-515	- do -	EL. 515 m	- do -	- do -	- do -	420 mil m^3
	540-530	- do -	EL. 530 m	- do -	- do -	- do -	176 mil m^3
(9)	545-497	EL. 545 m	EL. 497 m	EL. 474 m	EL. 553 m	203 m	772 mil m^3
	545-505	- do -	EL. 505 m	- do -	- do -	- do -	672 mil m^3
	545-520	- do -	EL. 520 m	- do -	- do -	- do -	467 mil m^3
	545-535	- do -	EL. 535 m	- do -	- do -	- do -	203 mil m^3
(10)	550-499	EL. 550 m	EL. 499 m	EL. 474 m	EL. 558 m	208 m	866 mil m^3
	550-510	- do -	EL. 510 m	- do -	- do -	- do -	719 mil m^3
	550-525	- do -	EL. 525 m	- do -	- do -	- do -	495 mil m^3
	550-540	- do -	EL. 540 m	- do -	- do -	- do -	231 mil m^3
(11)	555-493	EL. 555 m	EL. 493 m	EL. 474 m	EL. 563 m	213 m	1,043 mil m^3
	555-510	- do -	EL. 510 m	- do -	- do -	- do -	834 mil m^3
	555-515	- do -	EL. 515 m	- do -	- do -	- do -	766 mil m^3
	555-530	- do -	EL. 530 m	- do -	- do -	- do -	522 mil m^3
	555-545	- do -	EL. 545 m	- do -	- do -	- do -	231 mil m^3

No.	Case	FSL	MOL	Sediment Level	Dam Crest	Dam Height	Effective Storage
(12)	560-494	EL. 560 m	EL. 494 m	EL. 475 m	EL. 568 m	218 m	1,148 mil m³
	560-510	- do -	EL. 510 m	- do -	- do -	- do -	950 mil m³
	560-530	- do -	EL. 530 m	- do -	- do -	- do -	637 mil m³
	560-550	- do -	EL. 550 m	- do -	- do -	- do -	231 mil m³
(13)	565-494	EL. 565 m	EL. 494 m	EL. 475 m	EL. 573 m	223 m	1,296 mil m³
	565-510	- do -	EL. 510 m	- do -	- do -	- do -	1,108 mil m³
	565-530	- do -	EL. 530 m	- do -	- do -	- do -	795 mil m³
	565-550	- do -	EL. 550 m	- do -	- do -	- do -	389 mil m³
(14)	570-495	EL. 570 m	EL. 495 m	EL. 475 m	EL. 577 m	227 m	1,455 mil m³
	570-515	- do -	EL. 515 m	- do -	- do -	- do -	1,198 mil m³
	570-535	- do -	EL. 535 m	- do -	- do -	- do -	866 mil m³
	570-555	- do -	EL. 555 m	- do -	- do -	- do -	432 mil m³
(15)	575-497	EL. 575 m	EL. 497 m	EL. 475 m	EL. 582 m	232 m	1,592 mil m³
	575-515	- do -	EL. 515 m	- do -	- do -	- do -	1,356 mil m³
	575-535	- do -	EL. 535 m	- do -	- do -	- do -	1,024 mil m³
	575-555	- do -	EL. 555 m	- do -	- do -	- do -	590 mil m³
(16)	580-497	EL. 580 m	EL. 497 m	EL. 475 m	EL 587 m	237 m	1,751 mil m³
	580-520	- do -	EL. 520 m	- do -	- do -	- do -	1,446 mil m³
	580-540	- do -	EL. 540 m	- do -	- do -	- do -	1,094 mil m³
	580-560	- do -	EL. 560 m	- do -	- do -	- do -	633 mil m³

FSL: Full supply level
MOL: Minimum operating level

In the alternatives, 16 dam heights, where FSL ranges from EL. 505 m to EL. 580 m with an interval of 5 m, were considered. Each alternative is represented by a figure of FSL, e.g. the alternative 505 or a combination of FSL and MOL, e.g. the alternative 505-487. Table G1.2 shows detailed features of the respective development scale alternatives.

The flood control space of 100 million m³ was considered to be provided above FSL as was discussed in Chapter 6 of the Main Report. This provision allows the power and irrigation sectors to utilize the entire space of the reservoir between FSL and MOL.

G1.3.2 Reservoir Operation Simulation

In order to select the most optimum development scale among the alternatives presented in the preceding subsection, simulation studies of the reservoir operation were carried out by developing a simulation model.

Conditions for the simulation are itemized as follows:

1) Data to be input into the model such as inflow, evaporation and water required for power generation and irrigation are on a monthly basis. In the simulation model, all the input data and calculation results such as stored volume and spill-out discharge are represented by monthly volume.
2) Installed capacity was determined on the basis of the rated head and maximum plant discharge. The rated head was taken as 96% of gross head derived by

subtracting tail water level at the maximum plant discharge from a reservoir water surface level between FSL and MOL, i.e. two-thirds of drawdown between FSL and MOL. The maximum plant discharge is the discharge available for a minimum 4 hours per day throughout a year at a dependability of 95% under the selected FSL and MOL conditions.

3) Minimum plant discharge is 50% of the maximum plant discharge for one unit of turbine.

4) Plant discharge to be used for power generation is set as follows:

Setting of Plant Discharge

	Total Plant Discharge, Vout (million m^3)		Spill-out (million m^3)
	Discharge for Peak	Discharge for Off-peak	
Vin > (Vmax + Vmin)	Vmax	Vin–Vevap– Virr-Vmax	(S1+Vin-Vevap-Vout)-Smax
Vin < (Vmax + Vmin)	Vmax or S1+Vin-Vevap-Smin	0	0

Where,

Vin : Monthly inflow volume (million m^3)

Vmax : Monthly discharge volume for peak power generation (million m^3)

Vmin : Monthly discharge volume for off-peak generation (million m^3)

Vout : Total monthly discharge volume for generation (million m^3)

Vevap : Monthly evaporation volume (million m^3)

Virr : Monthly irrigation water volume (million m^3)

S1 : Reservoir storage volume at the beginning of month (million m^3)

Smax : Maximum reservoir storage volume at FSL (million m^3)

Smin : Minimum reservoir storage volume at MOL (million m^3)

This discharge release rule was decided to achieve possible maximum power generation under the conditions to meet the downstream requirements for irrigation and other water supply and to minimize the spill-out volume.

5) An average of reservoir water levels at the beginning and end of the month is applied for estimating the head for power generation. While the higher value between tailwater level and two thirds of maximum depth of the re-regulation pond is used for estimation of the head.

Outline of the developed simulation model and flow chart of the simulation are illustrated in Figure G1.5.

G1.3.3 Data Input to Reservoir Operation Simulation Model

Data to be input to the reservoir operation simulation model are inflow discharges at the Munda Dam site, evaporation, water level–reservoir surface area–storage volume relationships of the Munda reservoir and re-regulation weir, irrigation and other water supply requirement, peak operation hour, combined efficiency of generator and turbine and tailwater rating curve.

(1) Inflow at Munda Dam Site

The inflow was estimated through hydrological analysis incorporating water intake volume planned to be tapped upstream of the dam site in the future. The estimated monthly inflow covers 42 years from 1956 to 1997. The estimated annual average inflow is 206 m^3/s, ranging from 30 m^3/s in January to 623 m^3/s in July. The detail of the inflow estimates is given in Section 3.3 of the Main Report.

Water year was introduced to the simulations, that is, the simulation starts from the beginning of October, which is the start of the dry season.

(2) Evaporation

The reservoir evaporation rate used for the simulations was assumed by multiplying the monthly pan evaporation records observed at Peshawar from 1966 to 1997 by 0.7. The factor of 0.7 is known as the pan coefficient developed by research in the United States and commonly used to derive the reservoir evaporation rate from evaporation records of class A pan. The estimated annual average of reservoir evaporation rate is 1,172 mm. The detail of the pan evaporation data at Peshawar is presented in Appendix C.

(3) Water Level–Reservoir Surface Area–Storage Volume Relationship

The water level - reservoir surface area - storage volume curve of the Munda reservoir was developed as shown in Figure G1.6 by using the 1:10,000 scale maps. The curves for the alternative re-regulation weirs, which are located at 3.5 km, 4.3 km and 5.0 km (existing Munda Headworks site) downstream from the Munda Dam axis respectively, were also obtained based on the same maps as above as illustrated in Figure G1.7.

(4) Irrigation and Other Water Supply Requirement

Downstream irrigation and other water requirements consist of a) supply for the new irrigation scheme of the left and right banks of the Swat River, b) supply for the Palai scheme, c) supply for the existing Lower Swat Canal (LSC) and Doaba Canal, and d) supply for the civil canals including an allowance. Detail of the requirements is given in Chapter 5 of the Main Report and Appendix E.

(5) Peak Operation Hour

A 4 hour period of peak operation was decided on the basis of the recent daily load curves of WAPDA and KESC system and assumed future trend as discussed in Chapter 4 of the Main Report and Appendix D.

The daily load curves of the Pakistan power system are of typical evening-peak pattern and the duration of the peak load is around 4 hours throughout the year.

According to records of more developed countries, the shifting from the evening peak with the duration of 4 hours to daytime peak with the duration of 8 hours occurs when the per capita GDP reached US$ 1,500 to US$ 2,000. The present per capita GDP of Pakistan is around US$ 500 and future growth rate of the per capita GDP will be 3 to 4% annually. Therefore, it is forecasted that the per capita GDP would not reach US$ 1,500 to US$ 2,000 within the foreseeable future and hence the change of peak load duration from 4 hours to 8 hours would not happen in the near future.

The detail of the peak operation hour is discussed in Appendix D.

(6) Combined Efficiency

Curves of the combined efficiency of generator and turbine were prepared for the simulation study purpose on the basis of the current experiences.

(7) Tailwater Rating Curve

The tailwater rating curve was prepared based on the results of non-uniform flow computations for the stretch between the Munda Dam site and the existing Munda Headworks as represented in Figure G1.8. The river cross sections surveyed in this Feasibility Study were used for the non-uniform flow computations. Details of the river cross section survey are presented in Appendix A.

G1.3.4 Simulation Results of Reservoir Operation

The reservoir operations were simulated using the developed simulation model. All the above input data were incorporated in the simulation. Fifty eight development scale alternative cases in total were simulated and as a result, annual energy production was estimated as shown in Table G1.2.

G1.3.5 Dam Type

From the topographic and geological constraints, the dam types considered suitable for the Munda site were concrete gravity and fill type dams. These dam types were examined in detail, including materials surveys, preliminary layouts and cost estimates. The dam types examined were (1) a concrete face rockfill dam (CFRD), (2) an earth core rockfill dam (ECRD), and (3) a roller compacted concrete (RCC) dam. Of 16 dam height alternatives from 164 m to 237 m discussed in the preceding subsection, the RCC dam was examined for only the lowest dam height (164 m).

Dam type comparison for the Munda site was made for the Alternative 555 (dam height of 213 m) between ECRD and CFRD where layout of ECRD is as shown in Figure G1.9.

ECRD considered here is almost identical to CFRD in layout but its upstream slope is 1:2.2 and downstream slope is 1:2.0, so the diversion tunnels and power tunnel are longer than those of CFRD. It was found that CFRD is superior over ECRD for the following reasons:

1) CFRD was cheaper by 12% than in ECRD, mainly owing to shorter diversion tunnels.
2) CFRD's construction period is at least one year less than ECRD's, owing to smaller embankment volume.
3) River diversion costs and risks are less for CFRD than for ECRD.
4) Leakage emerging downstream of CFRD has a basically different significance than leaks through dams with earth cores, because there is no possibility of earth core erosion and no potential threat to the dam safety.
5) Since the entire CFRD embankment is dry, earthquakes cannot cause pore pressure in the rockfill voids.

A possibility to construct RCC type dam for the lowest dam case of 164 m in height was examined as seen in Figure G1.10, and the cost was found to be 27% higher than that for CFRD. This is mainly due to the high cost incurred for dam; more than 3 million m^3 of concrete were required, caused by the foundation geology where applicable maximum design parameters of foundation rock were as low as 2.5 MPa of shearing strength with internal friction angle of 40 degrees.

Based on these results, CFRD was selected as the most appropriate dam type.

G1.3.6 Optimum Development Scheme

All the costs were estimated for the basic design developed for the respective alternatives and converted to economic costs, applying a standard conversion factor of 0.89 to the local currency portion. The construction period of 7 years is assumed for the development scale alternative 505, 8 years for the alternatives 510 to 555, and 9 years for the alternatives 560 to 580. Figure G1.11 shows layouts for the alternatives 505 (164 m high dam), 530 (189 m high dam), 555 (213 m high dam), and 580 (237 m high dam).

From the annual energy production estimated for the respective alternatives, the economic power benefit was calculated by utilizing long run marginal cost (LRMC) updated to September 1999 level. The capacity cost of LRMC is US$ 583/kW and annualized one is US$ 70.29/kW. The energy cost of LRMC is US¢ 3.92/kWh for peak and US¢ 3.36/kWh for off-peak. Details of the power benefit calculation are presented in Chapter 12 of the Main Report.

In addition to the power benefits estimated above, the economic agricultural benefit of US$ 5.8 million/year and economic flood control benefit of US$ 0.9 million/year were taken into account for totaling overall economic benefits. For details of the agricultural and flood control benefits, please refer to Chapter 12 of the Main Report.

The development scale was optimized by comparing economic indices of the development alternatives derived from cash flow analyses. The study results were expressed in net present value (NPV), benefit cost ratio (B/C), and economic internal rate of return (EIRR) for the respective alternatives. The relationships of NPV/EIRR and MOL for Alternatives 505 to 580 are presented in Table G1.2 and Figure G1.12.

Based on this comparative study, the most economical combination of FSL and MOL was selected for the respective dam heights. The following table shows the economic indices for the most economical combinations for given FSLs:

Economic Indices for Alternatives

Alternative	NPV (US$ million)	B/C	EIRR (%)
505-487	37.2	1.12	11.0
510-488	55.7	1.17	11.5
515-490	75.9	1.23	11.9
520-491	86.7	1.25	12.1
525-493	104.5	1.29	12.4
530-494	113.7	1.30	12.4
535-495	125.4	1.31	12.5
540-496	142.4	1.35	12.8
545-497	149.0	1.35	12.8
550-499	163.4	1.35	12.8
555-510	175.5	1.36	12.9
560-510	178.9	1.35	12.8
565-510	183.7	1.34	12.7
570-515	181.5	1.32	12.6
575-535	167.6	1.29	12.3
580-540	138.6	1.23	11.8

These economic indices are also illustrated in Figure G1.13. According to the table and the figure, the values of EIRR are almost identical for FSLs of 540 to 565, while those of NPV vary from US$ 142 to 184 million with the highest NPV for Alternative 565-510.

However, further consideration was given to the realistic crest level of the Munda Dam from the viewpoints of topography and geology of the dam site and sedimentation expected at the upstream end of the reservoir. The following are points of the consideration:

1) The left abutment of the dam site shows 200 m long flat configuration above EL.565 to 570 m. Although it is still possible to construct a dam with a crest level of EL. 587 m corresponding to FSL 580 m, realistic maximum crest level is judged to be around EL 565 m topographically.

2) A series of water pressure tests on the right bank of the dam site suggest relatively intensive slacking of the rock in the parts of the slope higher than

around EL. 560 m. Considering several meters of foundation excavation, the level of around EL. 565 m is deemed to be a limit of the dam crest.

3) A preliminary estimate of sediment depth to be deposited at the upstream end of the reservoir gives a value of around 20 m above the FSL. The river bed level at the upstream end is around EL. 580 m. Adding some allowance to the estimated sediment depth, FSL 555 m is considered to be the maximum. The method of sediment depth estimate was the same as that for the Mangla reservoir studied in 1973 (see reference 3).

These considerations led to a conclusion that FSL 555 m with MOL 510 m, which corresponds to the dam crest level of EL. 563 m, is the practical and most economical alternative.

The effective storage volume of the alternative 555-510 is 834 million m^3 between FSL and MOL. Through the Study, the installed capacity is 740 MW, of which the plant factor is 37%. The annual energy production is 2,407 GWh, in which the firm energy (peak generation) is 847 GWh and secondary energy (off-peak generation) is 1,560 GWh. The installed capacity derived from this simulation study is further verified in the following subsection.

Table G1.3 and Figure G1.14 show the simulation results of the reservoir operation for the selected alternative 555-510. According to the result, deficit of the irrigation water supply occurs twice in February and once in March during the simulation years from 1956 to 1997. Frequency of the deficit is so small that sufficient supply of irrigation water required can be attained.

G1.3.7 Installed Capacity

The optimum installed capacity was studied comparing economic indices of the capacity alternatives such as NPV, B/C, and EIRR. FSL and MOL are EL. 555 m and 510 m as determined in the preceding subsection.

The result of the economic comparison is summarized as follows.

Comparative Study of Optimum Installed Capacity

Installed capacity	690 MW	740 MW	800 MW	880 MW
Maximum discharge (Dependability)	470 m^3/s (98%)	505 m^3/s (95%)	545 m^3/s (90%)	600 m^3/s (85%)
Dependable peak output	570 MW	590 MW	430 MW	250 MW
Annual energy, Total	2,360 GWh	2,407 GWh	2,455 GWh	2,506 GWh
Firm energy	835 GWh	847 GWh	627 GWh	368 GWh
Secondary energy	1,525 GWh	1,560 GWh	1,828 GWh	2,137 GWh
Plant factor	39%	37%	35%	33%
Economic indices				
NPV	164.6 mil US$	175.5 mil US$	171.4 mil US$	154.6 mil US$
B/C	1.35	1.36	1.33	1.28
EIRR	12.8%	12.9%	12.7%	12.3%

According to this table, the installed capacity of 740 MW shows the most economical indices. Therefore, the optimum installed capacity is determined to be 740 MW.

A possibility of base power operation at the Munda Dam was examined. Two groups of installed capacity alternatives, which are categorized by the plant factor of 87-98% and 61-66%, were compared by net present values. The result of the comparison is presented below:

Base Power Plant Alternatives

	Case A1	Case A2	Case A3	Case A4	Case B1	Case B2	Case B3	Case B4
Installed capacity (MW)	110	130	140	150	230	250	260	290
Plant factor (%)	98	93	89	87	66	64	63	61
Annual energy, total	941	1,054	1,096	1,148	1,348	1,421	1,455	1,552
Firm (GWh)	824	853	680	377	832	852	680	376
Secondary (GWh)	117	202	416	771	516	569	775	1,176
NPV (US$ Mil)	-102.4	-84.1	-79.0	-78.8	-30.4	-17.2	-15.9	-6.0

The comparison lead to negative net present values for all the alternatives. Therefore, the base power operation is not feasible at the Munda Dam.

G1.3.8 Unit Capacity

The number of the turbines and generators and their unit capacity were examined for the optimum installed capacity of 740 MW from the economical viewpoint and selected comparing the following alternatives:

Comparative Study of Optimum Unit Capacity

Number of units	3 Units	4 Units	5 Units	6 Units
Unit capacity	247 MW	185 MW	148 MW	123 MW
Annual energy, Total	2,399 GWh	2,407 GWh	2,411 GWh	2,413 GWh
Firm energy	846 GWh	847 GWh	847 GWh	847 GWh
Secondary energy	1,553 GWh	1,560 GWh	1,564 GWh	1,566 GWh
Plant factor	37%	37%	37%	37%
Economic indices				
NPV	173.3 mil US$	175.5 mil US$	174.9 mil US$	165.7 mil US$
B/C	1.36	1.36	1.36	1.33
EIRR	12.9%	12.9%	12.8%	12.7%

This table shows the maximum NPV for the case of 4 units with the unit capacity of 185 MW. The optimum unit capacity was therefore decided to be 185 MW.

G1.4 Development Layout Optimization

G1.4.1 Dam Axis

At the 2-km long stretch of the Munda Dam site, various dam axes including those examined in the Pre-F/S were reconnoitered in the field and a dam axis that suits the topography was selected. The dam axis selected, close to that recommended in the Pre-F/S, runs on the ridge at both banks, almost perpendicular to the river course and lies geologically on the pelitic schist, green schist and siliceous schist, which have no serious foundation problem according to the geological assessment, so nothing constrains the layout of the appurtenant structures such as diversion tunnels, spillway, and powerhouse.

G1.4.2 Dam Type

As stated in the previous Section, the CFRD was selected as the type of Munda Dam.

The existing world highest CFRD is Aguamilpa Dam of 187 m high, which was completed in 1993 in Mexico. Recently, many CFRDs are under planning or design in the world, among which the highest is Shuibuya Dam in China. The Shuibuya Dam is 232 m in height and embanked with limestone.

G1.4.3 Dam Height and Spillway Capacity

The dam crest level was examined adding the freeboard to the reservoir water level under the condition of the selected FSL 555 m in combination with the spillway capacity. The following is a summary of spillway alternatives and the required dam crest level, which are discussed in detail in Chapter G2:

Spillway Alternatives and Required Dam Crest Level

	Case 1	Case 2	Case 3	Case 4
Spillway, Gate portion				
Number of gate	2 nos	3 nos	4 nos	5 nos
Gate width	15.0 m	15.5 m	15.5 m	16.0 m
Gate height	25.1 m	21.9 m	18.4 m	15.8 m
Spillway, Non gate part				
Crest length	500 m	180 m	80 m	0 m
Crest elevation	EL.555.0 m	EL.555.0 m	EL.555.0 m	-
Full supply level, FSL	EL.555.0 m	EL.555.0 m	EL.555.0 m	EL.555.0 m
Flood water level, FWL(PMF)	EL.559.7 m	EL.560.8 m	EL.561.8 m	EL.562.8 m
Freeboard				
Normal above FSL	3.0 m	3.0 m	3.0 m	3.0 m
Minimum above FWL	1.0 m	1.0 m	1.0 m	1.0 m
Dam crest level	EL.561.0 m	EL.562.0 m	EL.563.0 m	EL.564.0 m
Economic total construction cost	US$748 mil	US$745 mil	US$741 mil	US$750 mil

According to this summary table, Case 3 is the most economical alternative. Therefore, the dam crest level was determined to be EL. 563 m.

The required freeboard shown in this table was estimated conforming to the USBR procedure. The effective fetch was computed based on Figure G1.15.

G1.4.4 Development Layout

Dam layouts for the development scale alternatives were prepared for the purpose of cost comparison and optimization of the alternatives. Dam heights range from 164 m for Alternative 505 to 237 m for Alternative 580. The newly prepared topographic map of scale 1:1,000 was used for the dam layout study. The layouts developed are shown in Figures G1.11 for the Alternatives 505, 530, 555 and 580.

The layout for all the alternatives is composed mainly of two lanes river diversion tunnels, a main concrete face rockfill dam with an integrated upstream cofferdam, a combined type spillway with gated and non-gated overflow portion, a power intake and waterway, a surface type powerhouse, a river outlet tunnel, and so forth.

The CFRD was selected for the layout study as the most appropriate dam type considering topography and geology of the dam site, availability of construction material, and technical and cost advantages of this type. The possibility to construct a RCC type dam for the lowest dam case of 164 m in height was examined. The cost comparison of the CFRD and RCC dam revealed the cost advantage of CFRD with a cost increase of 27% for the RCC dam. Therefore, even for the lowest dam alternative, CFRD is applied for development scale optimization study.

At least two river diversion tunnels are required so the river outlet facilities can be installed in the one of the tunnels while the other tunnel continues to divert the river flow.

The combined type spillway was laid out on the left abutment to suit the topography. The non-gate weir portion will release excess reservoir water automatically even if the spillway gates malfunction. The emergency spillway of fuse dike type was not designed because of huge amount of excavation work which leads to higher cost and environmental issues.

A power intake, an intake gate shaft, a headrace tunnel, a surge tank, penstock tunnels, open penstocks, a surface type powerhouse, and an open switchyard were laid out on the right abutment.

The river outlet facilities constructed in the one of the diversion tunnels will be utilized to release river water downstream for meeting irrigation and other water requirement during reservoir impounding as well as to release reservoir water to draw down the water level in an emergency case.

Re-regulation facilities are required when the Project is developed as a peak power station. Through comparison of three alternative sites of the facilities, which are located 3.5 km, 4.3 km, and 5.0 km (existing Munda Headworks site) downstream from the proposed Munda Dam axis, the 3.5 km site was selected to be the most appropriate for the reasons of economical and environmental advantages. The 4.3 km site will submerge the village of Pati Banda located on the right bank. The remodeling of the existing Munda Headworks will lead to modification of not only the Headworks itself but also intake structures for the Lower Swat Canal and Doaba Canal.

The reservoir area and general layout of the selected development scheme are shown in DWG. C01 and C02, respectively.

G1.5 Installation Timing

G1.5.1 General

In the long-term Generation Program prepared by the National Power Planning group of WAPDA, the Munda Dam Project is planned to be commissioned in 2011 in case of the Normal Growth Scenario and in 2013 for the Low Growth Scenario. Reference is made to Tables D6.1 and D6.2 in Appendix D. According to the programs, a number of relatively small hydro power plants are planned after the 9th 5-Year Plan period (2003 to 2005).

As mentioned in Chapter 9, the earliest possible completion time of the Project will be 2009.

The optimum installation timing of the Munda Project is defined as the year when the Project is to be put in the least cost sequence of the long-term installation program of the power plants in the national grid. A comprehensive and sophisticated power system planning program, the Electric Generation Expansion Analysis System (EGEAS) was used for finding the optimum installation timing of the Munda Project.

G1.5.2 Conditions and Assumptions

Examination was made by use of EGEAS under the following conditions and assumptions:

1) Applying the collected system load data consisting of the low growth scenario of the peak load and annual energy demands, which is similar to World Bank one, and load duration data.
2) Inputting the fixed system including the Chashma nuclear plant, Chashma low head hydro plant, Ghazi Barotha hydro plant, and 15 private sector plants, which are scheduled during the 9th five-year plan, as well as the existing/operating plants.
3) Inputting expansion hydro candidates consisting of Neelum Jhelum, Golen Gol, Jinnah, Kohala and Taunsa, of which the proposed install capacity is close to or more than 100 MW, as well as Munda.

4) Inputting expansion thermal candidates such as a coal fired plant, a combined cycle plant and a gas turbine plant, of which the capacity is variable and decided by the EGEAS computation.
5) Setting the base year of 1999, minimum reserve capacity of 20% and discount rate of 12%.

G1.5.3 Installation Timing

The result of the EGEAS computation revealed that the Munda Project has an optimum installation timing of year 2010 as tabulated in Table G1.4 and illustrated in Figure G1.16.

G2 Feasibility Design

G2.1 General

The feasibility design for the main civil structures was carried out conforming to the internationally applied and accepted design criteria and standards. The development scale of the Munda Dam was decided as described in Chapter 7 of the Main Report and Chapter G1 of this Appendix, in which FSL is 555 m with the installed capacity of power generation being 740 MW. General plan is shown in DWG. C03. This chapter presents optimization of the structures for the selected development scale and feasibility design for the optimal structures.

G2.2 Main Civil Structures

G2.2.1 River Diversion

The river diversion is required to divert the river flow during construction of the main dam, spillway plunge pool, powerhouse, and other structures located on and beside the Swat River. Only the tunnel type in combination with the cofferdam is conceivable for the river diversion at the Munda Dam site from the topographical viewpoint.

At least two tunnels are required for installing the river outlet facilities in the one of the tunnels while the other tunnel continues to divert the river flow. Taking account of the longer construction period of 6 years and non-availability of the reliable flood hydrograph records at the dam site, it was decided to apply 3,630 m^3/s of peak discharge, a 25-year probable flood as a diversion design flood and to size the tunnels as inflow peak being equivalent to the outflow.

Both of the diversion tunnels are laid out on the left bank. One of the advantages of the left diversion tunnels is shorter tunnel length compared with that of the right diversion tunnels. The another merit is that energy of water released from the river outlet can be dissipated by utilizing the plunge pool of the spillway after completion of the dam. The selected left bank diversion tunnels are 975 and 964 m long.

As explained in the subsequent section, intake of one of the diversion tunnels used as the river outlet facilities is to be aligned at a higher elevation than the normal high water level of the re-regulation pond. The designed inlet and outlet sill levels of the tunnel are EL. 379 m and EL. 370 m, respectively. The another diversion tunnel is aligned with the inlet sill level of EL. 363.5 m and outlet one of EL. 362.5 m as close as the riverbed level so that the initial river diversion can be made easily.

In view of the existence of a deep gully on the left bank just upstream of the main dam site, by which construction of independent main cofferdam is made more costly, a cofferdam, which is later integrated with the main dam, was laid out. The height of the cofferdam would not affect the total construction cost much but is limited to an extent where the cofferdam can be constructed within one dry season after the river is diverted to the tunnels. Thus, a 60-m-high cofferdam with crest elevation of 410 m was decided.

Discharge capacity of the diversion tunnels was computed for different tunnel diameters under the conditions of both free and pressure flows, of which the result is shown in Figure G2.1. As a result of the computation, 12.0 m diameter was found appropriate for both tunnels.

The designed river diversion is shown in DWG. C04 and C05.

G2.2.2 Main Dam

(1) Freeboard

The freeboard consists of a normal freeboard above FSL and a minimum freeboard above the flood water level for PMF.

The normal freeboard was estimated to be 3.0 m summing the wind setup and wave runup of 1.6 m with the effective fetch of 1.6 km, earthquake generated wave height of 1.1 m, and allowance of 0.3 m for malfunction of spillway gate.

The minimum freeboard was estimated to be 1.0 m summing the wind setup and wave runup of 0.7 m, and the allowance for spillway gate malfunction of 0.3 m.

Adding the minimum freeboard of 1.0m to the flood water level of EL. 561.8 m for PMF, the dam crest elevation was determined to be EL. 563.0 m as discussed in Chapter 7 of the Main Report and Chapter G1 of this Appendix. The dam crest EL. 563.0 m satisfies the normal freeboard requirements, in which the FSL is EL. 555m and the normal freeboard is 3.0m.

Furthermore, the intermediate freeboard was also computed for confirmation of the dam crest elevation. The estimated intermediate freeboard is 2.5 m subtracting half of the earthquake generated wave height from the normal freeboard. The surcharge water level is EL. 559.4m, hence the dam crest of EL. 563m satisfies the intermediate freeboard requirement.

(2) Dam Embankment

General:

Main features of CFRD are as follows:

Crest elevation:	EL. 563 m
Parapet wall top elevation:	EL. 564 m
Dam base:	EL. 356 m
Plinth base:	EL. 350 m
Maximum dam height above plinth:	213 m
Dam slopes:	1: 1.4 for upstream,
	1: 1.5 for downstream
Dam crest length:	760 m
Dam crest width:	12 m
Width of concrete face slab:	15 m
Area of concrete face slab:	191,300 m^2
Length of plinth:	920 m
Dam embankment volume:	16,500,000 m^3

The plan, front elevation, typical dam cross section and details are shown in DWG C06, C07, and C08. The cofferdam is integrated later with the main dam.

Rockfill Materials:

Rock materials to be used for the dam embankment are limestone, quartzite, and siliceous schist. The limestone is available at Sappare quarry site located approximately 3 km northeast of the dam site with exploitable volume of around 6 million m^3. The quartzite and siliceous schist can be quarried from Todobo Banda quarry situated approximately 1 km upstream of the dam site in an order of 15 million m^3. Excavated rock from the spillway or other structure sites, mainly consists of schist, is planned to be used as part of the rockfill material.

Worldwide engineering experience in CFRD construction have proved appropriateness of limestone, quartzite, and schist as rockfill material for CFRD as seen in Table G2.1. Details of the rockfill materials are discussed in Section 3.2 of the Main Report and Appendix B.

Dam Slopes:

The main dam was designed with the upstream and downstream slopes of 1:1.4 and 1:1.5, respectively, based on the experience of the Study Team and in consideration of slope data of the constructed or designed CFRD as given in Table G2.1. In dam zoning, limestone is placed on the upstream zone and quartzite and schist are located on the downstream zone. According to Table G2.1, the slope of 1:1.4 is dominant for the CFRD constructed with limestone, while 1:1.5 is an average for the CFRD constructed with shale and schist.

According to ICOLD bulletin "Rockfill Dams with Concrete Facing", downstream slope of 1:1.4 is suggested for the area such as Munda site with the possible earthquake magnitude of 7 and peak acceleration of 0.15g. Therefore, the adopted downstream slope of 1:1.5 is acceptable.

Dam Zoning:

The embankment dam composes the following zones:

Zone 1A:	Impervious earthfill over plinth
Zone 1B:	Random fill over plinth
Zone 2A:	Fine filter
Zone 2B:	Crusher run
Zone 3A:	Selected small rock
Zone 3B:	Rockfill, limestone
Zone 3C:	Rockfill, quartzite and siliceous schist
Zone 3D:	Rockfill, excavated rock
Zone 3E:	Selected large rock

Siliceous schist for Zone 3C is known in some cases to produce a non-free draining rockfill. The limestone zone (3B) will, therefore, be placed at the bottom of the downstream zone in order to draw seepage water downstream.

The excavated rock (3D) is placed in the downstream in order to use the excavated material efficiently and in order to minimize deformation of the dam body which affects the concrete face slab.

Concrete Face Slab, Plinth and Joints:

The concrete face slab will be placed in 15 m widths with water stops along the vertical joints and perimetric joints at face slab and plinth. Thickness of the reinforced concrete face slab was decided applying the equation: $T = 0.3 + 0.003h$, where T is thickness and h is vertical height below dam crest. The slab concrete will be placed with use of a slip form on the fine transition zone. The impervious earthfill zone is essential, acting as a joint or crack healer.

The three types of plinth were designed with the width and thickness of 6 m and 0.6 m, 8 m and 0.8 m, and 10 m and 1.0 m depending on the magnitude of static water pressure incorporating the experiences of CFRD. The maximum hydraulic gradient is estimated at around 20.

The perimetric joints are composed of copper waterstop and stainless steel waterstop covered with cohesionless fines. The vertical joints are designed with copper waterstop and hypalon waterstop. The tension vertical joint is covered with cohesionless fines. These ideas are based on the experience of CFRD.

Parapet Wall:

A vertical parapet wall will be provided at the dam crest in continuation of the upstream concrete face slab. The wall is 5.6 m high. The parapet wall is used to reduce the rockfill embankment volume and to provide a sufficient space to accommodate the face slipform equipment.

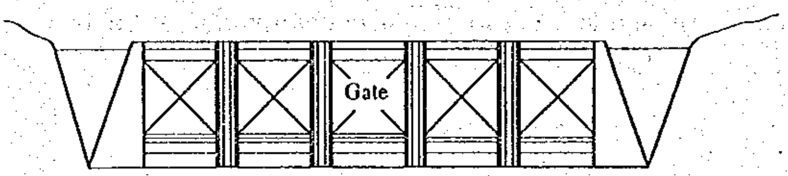
G2.2.3 Spillway

(1) Spillway Configuration Optimization

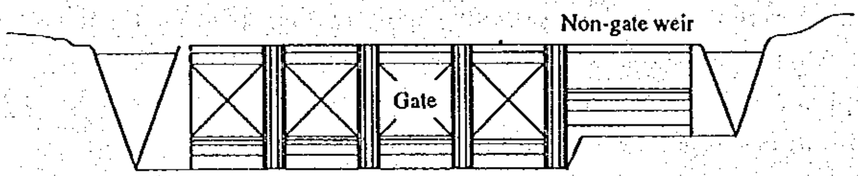
In order to regulate inflow floods into the Munda reservoir and ensure safety of the dam against extraordinary floods, the spillway configuration was optimized.

Because of existence of numbers of deep gullies running on the right bank side, the alternative to locate the spillway structures on the right bank was not considered and thus was discarded in the beginning. Similarly, a non-gated weir alone was not considered due to relatively large PMF. Taken up for optimization examination are (1) gated spillway alone, (2) a combination of gated spillway with non-gated weir and (3) gated spillway with fuse plug which may be considered for the higher FSL as the reservoir rim may allow for construction of the fuse plug topographically. The following three spillway types were compared to select the optimum type:

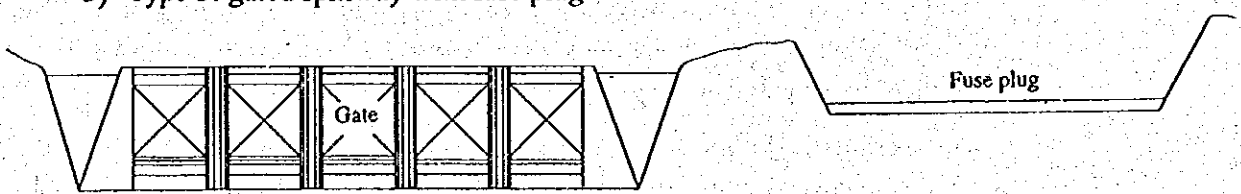
1) Type 1: gated spillway alone



2) Type 2: a combination of gated spillway with non-gated weir



3) Type 3: gated spillway with fuse plug



The spillway Type 1 has 5 sets of spillway gates, 16 m wide and 15.8 m high. The spillway has sufficient capability for flood control with constant ratio-constant outflow discharging rule and sufficient discharge capacity for PMF. However, this type of spillway has no flexibility to discharge floods in case of malfunction of the gates. The total construction cost was estimated at US$ 750 million.

The spillway Type 2 has 4 sets of gates, 15.5 m wide and 18.4 m high, and with a 80 m long non-gated overflow weir with the crest level of EL.555 m, which is the same elevation as FSL. This type of spillway has an advantage to release excess flood water automatically through the non-gated weir even in case of malfunction of the gates. The spillway Type 2 has both sufficient flood control ability and discharge capacity for PMF. The total construction cost for this type was estimated to be US$ 741 million.

The spillway Type 3 has 5 sets of gates, 16 m wide and 14.9 m high, with 130 m wide fuse plug type emergency spillway. This type of spillway also provides sufficient flood control ability and discharge capacity for PMF. However, huge amount of excavation volume for the emergency spillway causes environmental issues as well as high project costs. The estimated total construction cost is US$ 820 million.

As a result of the comparison among those three types, the spillway Type 2 was selected in view of the lowest construction cost and more flexible operation resulting in securing the dam's safety.

An optimization study was further made for the selected spillway Type 2 in terms of the number and dimensions of the spillway gates and length of the non-gated overflow weir as follows:

Comparison of gate number and dimension, and non-gated overflow weir length

	Case 2A	Case 2B	Case 2C
Dam crest level	EL. 561.0 m	EL. 562.0 m	EL. 563.0 m
Full supply level	EL. 555.0 m	EL. 555.0 m	EL. 555.0 m
Surcharge water level	EL. 559.4 m	EL. 559.4 m	EL. 559.4 m
Flood water level	EL. 559.7 m	EL. 560.8 m	EL. 561.8 m
<u>Gate portion</u>			
Number of gate	2 nos.	3 nos.	4 nos.
Dimension of gate (width x height)	15.0 m x 25.1 m	15.5 m x 21.9 m	15.5 m x 18.4 m
Crest length	30.0 m	46.5 m	62.0 m
Crest elevation	EL. 534.8 m	EL. 538.0 m	EL. 541.5 m
Discharge	8,000 m^3/s	11,100 m^3/s	12,300 m^3/s
<u>Non-gated overflow weir</u>			
Crest length	500.0 m	180.0 m	80.0 m
Crest elevation	EL. 555.0 m	EL. 555.0 m	EL. 555.0 m
Discharge	11,000 m^3/s	5,500 m^3/s	3,100 m^3/s
Total economic cost	US$ 748 million	US$ 745 million	US$ 741 million

According to this table, the Case 2C was selected as it is the cheapest. The selected spillway has 4 sets of gates each 15.5 m wide and 18.4 m high, with a 80 m long non-gated overflow weir.

(2) Spillway Components

General:

Based on the spillway optimization study mentioned above, the feasibility design was performed applying the following parameters:

Dam crest:	EL. 563 m
Design discharge for chute:	3,800 m^3/s (1,000 years probable flood outflow)
Design discharge for plunge pool:	1,900 m^3/s (100 years probable flood outflow)
Gated weir:	4 nos., 15.5 m wide x 18.4 m high gates, ogee crest EL.541.5 m
Non-gated weir:	80 m long with crest EL. 555 m
Chute:	60 m wide and 4.7 to 5.2 m high wall
Plunge pool:	175 m long and bed level EL.354 m

Figure G2.2 shows the discharge capacity curves of the designed spillway. DWG. C09 gives plan, profile and sections of the spillway.

The following describes specific considerations related to the spillway components, (i.e. forebay, headworks, chute and plunge pool) but it must be noted that design of the spillway is subject to hydraulic model studies which will be conducted in the next phase of the project activities.

Forebay:

The forebay to be provided in front of the spillway headworks functions to lead flood discharge smoothly to the overflow weir of the spillway. The bed excavation level was set at El. 531.5 m, 10m lower than the gated ogee crest level, by which the approach velocity is less than 0.4 m/s.

Headworks:

The headworks, consisting of gated and non-gated portion, work to control flood discharges and ensure dam safety to release PMF at maximum. The gate portion is EL. 541.5 m at crest and equipped with 4 sets of spillway gates of 15.5 m wide and 18.4 m high. The upstream face of the weir at the gated portion is inclined at a slope of 1 (vertical): 2/3 (horizontal) to maximize the overflow discharge coefficient. The non-gated portion has 80 m long overflow weir with the crest of EL. 555 m.

At PMF with the maximum reservoir water level of EL. 561.8 m, discharge capacity is 12,300 m^3/s for the gated portion and 3,100 m^3/s for the non-gated portion as shown in Figure G2.2. The ratio of the discharge capacity of the non-gated portion is 1:0.25. The flood control operation will be made by use of both the gated portion and non gated portion.

Chute:

The spillway chute conveys the discharge released from the reservoir downstream smoothly. The chute width was decided to be 60 m considering a width of the plunge pool without disturbing the surrounding river banks. The chute is mainly divided into upper part and lower part, the former is 1:6.5 in bed slope and 4.7 m in wall height and the latter is 1:1.7 in bed slope and 5.2 m in wall height. The design discharge is for a 1,000 year probable flood outflow of 3,800 m^3/s. PMF outflow of 15,400 m^3/s is also to be conveyed without overtopping from the chute wall.

In the chute, aerators are to be provided for preventing erosion of the chute surface due to cavitation appearing on the chute floor.

Plunge pool:

The design discharge is for a 100 year probable flood outflow of 1,900 m^3/s. The water jet jumped from the flip bucket reaches around 310 m from the flip bucket when the design discharge is released. The length of the plunge pool is set at 175 m and bed level is excavated to EL. 354 m.

G2.2.4 River Outlet

River outlet was designed for the purposes of:

a) releasing water required for irrigation and other water supply needed downstream of the Munda Dam when power generation halts for some reasons,
b) releasing water required for irrigation and other water supply during impounding of the Munda reservoir after closing the last diversion tunnel, and
c) lowering the reservoir water level below the ogee crest of the gated spillway in case of emergency.

As mentioned earlier, the river outlet facilities were planned to be provided in and by remodeling the left diversion tunnel. In order to meet the above requirements, the river outlet should have an independent intake from the diversion tunnel at a higher elevation than the assumed sediment level in the reservoir. The intake is to be connected to the tunnel by a vertical shaft.

Diameter of the vertical shaft was decided to be 4.5 m taking into account a discharge velocity of 5.0 m/s in the shaft and the expected maximum downstream water requirement of 80.6 m^3/s in June. The velocity of 5.0 m/s is considered a maximum allowable velocity in the concrete conduit in normal case. The velocity reaches around 23 m/s in the shaft at the maximum discharge of 358 m^3/s to be released in case of emergency.

The river outlet facilities are accommodated in a gate chamber arranged in the left diversion tunnel and at a higher elevation than the maximum water level of the re-regulation pond so that the relevant structures are always kept dry. On the other hand, the outlet invert of the diversion tunnel cum. river outlet will be set at an elevation close to that of the right diversion tunnel so that structural configuration of the spillway flip bucket and these outlet structures can be made easily.

Access to the gate chamber is via a shaft constructed just downstream of the dam axis. Because of high head of 170 m, two sets of slide gates were designed, each being housed by steel boxes with upstream bellmouthed conduits and downstream waterway being steel-lined by 100m in length for protecting concrete surface of the tunnel from the erosion due to high velocity jet flow.

Reservoir impounding is scheduled to commence in December and the reservoir water level be raised at a rate of 1 m per day. The required flow area of the river outlet facilities during the impounding till the reservoir water level reaches the river outlet intake sill was calculated for each month by applying the average reservoir water level and water supply volume required for downstream irrigation. Consequently, the required flow area of the river outlet gates was estimated to be 9.0 m^2.

The following summarizes the general configuration of the river outlet facilities:

Intake:	Morning glory type
Crest level of intake:	EL. 480 m
Shaft:	4.5 m in diameter and 100 m deep
Gate chamber:	7 m high and 10 m long
Gates:	2 sets of guard and service gates,
Upstream steel liner:	Two lanes of steel conduits with a bellmouth inlet
Downstream steel liner:	100 m long
Gate bottom:	EL. 383 m
Invert level at outlet:	EL. 370 m

DWG. C04 shows the plan and profile of the river outlet.

G2.2.5 Power Waterway

(1) Waterway Alignment

There are several alternatives conceivable for layout of power waterway; one is aligned at the left bank and the other at the right bank. If the waterway is short, surge tank may be omitted and hence, with or without surge tank will be other alternatives. In the Pre-F/S report, an alternative to having waterway at the right bank with surge tank and the use of diversion tunnel was selected. After scrutiny of all the other alternatives, the following four alternatives were kept for economical comparison:

Case 1 : waterway with a surge tank on the right bank,

Case 2 : waterway with a surge tank on the right bank utilizing a diversion tunnel (when one of the river diversion tunnels is to be aligned on the right bank),

Case 3 : waterway without a surge tank on the right bank, and

Case 4 : waterway without a surge tank on the left bank (when one of the river diversion tunnels is to be aligned on the right bank and there is an enough space for the powerhouse on the left bank).

The plan and profile for each case are shown in Figure G2.3. The features and related construction cost of the waterway alternatives are tabulated below.

Features and Construction Cost of Waterway Alternatives

	Case 1	Case 2	Case 3	Case 4
Headrace tunnel	490m long x 12.0m in dia	490m long x 12.0m in dia	190m long x 12.0m in dia	250m long x 12.0m in dia
Surge tank	68m high x 15-25m in dia	68m high x 15-25 in dia	None	None
Penstock liner	2 lanes x 520m long x 7.4m in dia	2 lanes x 610m long x 7.4m in dia	2 lanes x 840m long x 7.4m in dia	2 lanes x 360m long x 7.4m in dia
Pressure rising	68m (35%)	82m (43%)	148m (77%)	79m (40%)
Use of diversion tunnel	None	350m long	None	None
Related construction cost	US$ 41.5 million	US$ 74.4 million	US$ 105.8 million	US$ 49.4 million

As shown in this table, Case 1 is the most economical one among the alternatives. Therefore, the alternative Case 1, the waterway with a surge tank aligned on the right bank was selected.

Alternative study on the right bank waterway was further executed in terms of the powerhouse type, which is either an open-air type or an underground type. Figure G2.4 represents these alternatives. The waterway with the underground powerhouse has an advantage of shorter penstock, that is a major cost component, compared with the open-air powerhouse type waterway. However, the comparative study shows that cost of the waterway with the underground powerhouse is 1.3 times higher than that with the open-air powerhouse. Therefore, the waterway with the open-air type powerhouse was chosen.

(2) Waterway Component

Power Intake

The power intake was designed under the inflow velocity condition of 1.0 m/s to avoid intrusion of air and trash into the intake and to minimize the intake head loss.

Headrace Tunnel

Optimum diameter of the headrace tunnel was examined through comparison of the sum of annualized construction cost, maintenance cost and energy loss for the alternative diameters ranging from 10.4 m (velocity of 6 m/s) to 17.9 m (velocity of 2 m/s) for one lane option and from 7.4 m to 12.7 m for two lane option. Figure G2.5 illustrates results of the comparison, which shows the lowest cost for the diameter of 12 m of one lane. Thus, the diameter of the headrace tunnel was decided to be 12 m as the optimum for one lane. The velocity in the headrace tunnel of 12 m in diameter is calculated to be 4.5 m/s.

Surge Tank

The surge tank was designed applying the restricted orifice type, which is generally the most economical type. The following are main features of the surge tank designed based on the surging calculation results given in Figure G2.6:

Orifice diameter:	7.1 m
Diameter of lower chamber:	15.0 m
Diameter of upper chamber:	25.0 m
Up-surge level:	EL. 565.0 m
Down-surge level:	EL. 503.2 m

Penstock

Optimum diameter of the penstock was studied in the same manner as that for the headrace tunnel as shown in Figure G2.7. Two lane penstock was designed in order to reduce the risk of damage on the penstock. The diameter alternatives range between 6.8 m (velocity of 7.0 m/s) and 9.0 m (velocity of 4.0 m/s). The optimization study led to the optimum diameter of 7.4 m with the velocity of 6.0 m/s.

DWG. C10 shows plan and profile of the waterway.

G2.2.6 Power Station

The riverbed gradient of the Swat between the Munda Dam site and the existing Munda Headworks is approximately 1/2,500 according to the cross section survey results and water surface is around 1/1,000 according to the non-uniform flow calculation result. Under these circumstances, extension of waterway downstream is not considered realistic to attain more water head for hydropower generation. Location of the power station was, therefore, selected just downstream of the main dam.

It is inappropriate to locate the spillway at the right bank because of the existence of deep gullies. It is also difficult to locate the spillway and power station at the left bank as the areas are so congested. For this reason, the location of power station was considered preferable to be at the right bank.

According to the development scale optimization study, the installed capacity was 740 MW in total, consisting of four 185 MW units.

Ground level of the powerhouse was decided to be EL. 383 m considering the flood water level of 10,000 years probable flood and maximum water level of the re-regulation pond. DWG. C11 shows detail of the powerhouse.

Outdoor open switchyard and GIS were compared in terms of economy. The outdoor open switchyard is cheaper by 20% than GIS. Therefore, the outdoor open switchyard was selected.

G2.2.7 Re-regulation Weir

The function of the re-regulation weir is to store water released from the power station during the peak generation for 4 hours a day, and to regulate the river flow releasing the stored water constantly downstream the weir.

A possibility to use the existing Munda Headworks as the re-regulation weir was examined through the field reconnaissance and review of the drawings collected. It was concluded that an alternative to use the Munda Headworks is technically, economically and environmentally not feasible and was discarded for further scrutiny. The main reasons were:

a) Foundation condition on which the Munda weir is found is not known and no detailed drawings are available. However, according to rough assessment, the weir would not be technically bearable for the heightening of 10 m, i.e. cannot be economically heightened.
b) During re-modeling of the weir together with new intakes of LSC and Doaba canal, it cannot be guaranteed to secure the constant supply of irrigation water because of its complexity.
c) Quite a number of people settled and unsettled will be subject to submergence, hence, to be resettled to the other locations.

The best location of re-regulation weir is discussed beside the Munda Headworks as follows.

The storage volume required for the re-regulation weir is calculated to be approximately 7 million m^3 based on 4 hours of peak operation in a day. The following two weir sites were identified beside the existing Munda Headworks through the field reconnaissance:

Site A: 3.5 km downstream from the proposed Munda Dam axis.

Site B: 4.3 km downstream from the proposed Munda Dam axis.

For these two sites the comparative study was carried out. These sites are shown in Figure G2.8. The location of the existing Munda Headworks is also represented in the figure as Site C for reference. According to the comparative study, the respective sites have the following features (site C for reference):

Features of Re-regulation Weir at Sites A, B and C

	Site A	Site B	Site C
Maximum water level (El.m)	381.8	374.4	371.7
Maximum reservoir area (km^2)	0.70	0.96	1.13
Crest length (m)	310	400	970
Weir height (m)	22.5	15.0	12.5

Site B will submerge the existing villages and will induce serious environmental issues. Therefore, the Site A was selected as the re-regulation weir site.

The re-regulation weir will function to regulate the peak power discharge during the normal condition and release floods safely at the flood condition. The design discharge for the re-regulation weir was decided as follows:

1) Released discharge : 81 m^3/s (= discharge/sec of 4-hour peak discharge)
2) Flood discharge : 2,420 m^3/s (= 200 years probable flood outflow)

The maximum water level in the normal condition and flood water level were estimated to be EL. 381.8 m and EL. 372.2 m, respectively.

The re-regulation weir was considered to be equipped with a number of gates. During the normal condition, several gates will be opened to release the stored water constantly downstream. When floods occur, the other gates will also be operated depending on the magnitude of the floods.

Two alternatives of the re-regulation weir were compared, one was a curtain wall type with radial gates and the another was a full radial gate type as presented in Figure G2.9. Cost comparison revealed that the full gate type is around 20% more expensive in construction cost over the curtain wall type. Therefore, the curtain wall type was selected.

Figure G2.10 shows result of the optimization study for the number of gates for the re-regulation weir. The figure concluded the optimum gate number of 7. The width and height of the gate are 8.0 m and 11.7 m, respectively. DWG. C12 gives the detail of the re-regulation weir.

Since water level of the re-regulation weir fluctuates 17 m every day, a particular caution for dissemination of the people and/or their evacuation should be taken.

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