CHAPTER 6 FLOOD CONTROL STUDY

6.1 Methodology of Flood Benefit Estimate

The Swat River, a tributary of the Kabul River in the Indus basin, after the Munda Headworks, bifurcates and flows down as the Abezai and Khiali Rivers. These branches rejoin at the confluence point near Charsadda about 20 km upstream of Nowshera and fall into the Kabul River. The Kabul River then joins the Indus at Attock. A number of tributaries join the Indus River downstream Attock before it empties into the Arabian Sea.

The assessment of the flood benefits focussed on the reach from the dam site, virtually the Munda Headworks in the Khiali River to Nowshera in the Kabul River. Additional benefits may be expected in the reach downstream of Attock in the Indus River but quantification of such benefits is extremely uncertain due to the cumulative impacts of flows from other tributaries which join the Indus River. Thus, the estimates of flood benefits derived at this time are considered on the conservative side.

Assessment of flood benefits was carried out as shown in a flow chart below:

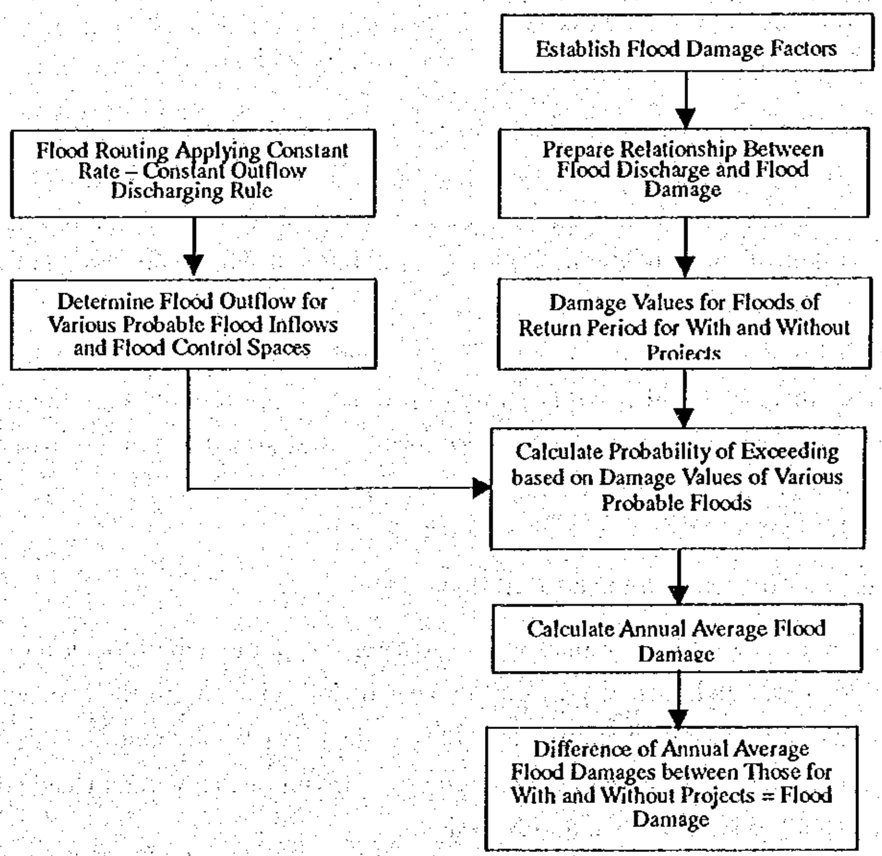


Figure 6.1.1 Methodology of Flood Benefit Estimate

6.2 Historical Flood Damages

6.2.1 Flood Damage Investigation

The field investigation including interviews with local people about flood damage was carried out at 60 points over the Swat/Kabul River flood plain area downstream of the Munda Headworks and between Warsak Dam and Nowshera.

According to the field investigation results, the lower Swat River reach, downstream of the Munda Headworks is flooded causing damage twice to four times every year during the rainy season from June to September. The inundation area covers not only the areas flooded by the Swat River itself but also those flooded from civil canals which are running parallel with and affected by the Swat, Kabul Rivers, and its tributaries. According to the old residents, the inundation caused by the 1929 flood was the largest ever experienced in about the past 100 years in terms of flood magnitude, inundation area, and duration.

6.2.2 Historical Floods

Recorded maximum flood peak is 4,500 m^3/s (159,000 cusec) in 1929 at the Munda Headworks as observed by Irrigation Department (ID) of NWFP. The second maximum peak discharge is 2,413 m^3/s (85,280 cusec) on July 25, 1995 as recorded by ID. The third one is 2,158 m^3/s (76,250 cusec) on July 15, 1988.

River runoff observation was made at the Munda Headworks by reading staff gauges, and the measured water stages were converted to discharges applying stage-discharge relationship. As described in Section 3.3, the discharge data recorded at the Headworks during low flow season were concluded to be unreliable. However, in view that the gates would be fully opened during the high flood, the records at high floods could be more reliable to some extent. Since there is no other data or way to estimate more reliably than the records at the Munda Headworks, the flood control studies stated below are based on the discharge records at the Munda Headworks.

6.2.3 Inundation Area, Depth and Duration

A flood inundation area map was drawn for the floods which occurred in 1929, 1988, 1995, and normal year on the basis of interviews with the residents, flood marks survey, and available 1:50,000 scale topographic maps, and shown in Figure 6.2.1 which covers the stretch downstream of the Munda Headworks and between the Warsak Dam and Nowshera.

The area of the inundation for the floods of 1929, 1988 and 1995 as well as that for the normal year was estimated on the basis of Figure 6.2.1 and is as follows:

Estimated Inundation Area (km^2)

River	Stretch	Historical flood (1929/8/28)	Medium class flood (ex.1995/7/25)	Low class (Normal year)
Swat	From Munda H/W to Swat-Kabul confluence	188.75	95.75	57.50
Swat/ Kabul	Total inundation area	697.75	448.25	244.25

Figure 6.2.2 is the discharge – inundation area curve showing the relationships between the flood peak discharge and the direct flood inundation area of the Swat River alone excluding the areas affected by the floods of tributaries.

According to the results of field investigation, the agricultural land along the Swat and Kabul Rivers, where the main crop is sugarcane, is affected by floods occurring twice to four times a year and with an inundation depth of 0.3 to 1.0 m and a duration of 1 to 3 days. However, such floods only affected a small number of houses.

At the time of the 1988 and 1995 floods, the inundation depth was 0.5 to 2.0 m with a duration of 4 days. A lot of villagers' houses made of mud walls collapsed. The inundation duration of the 1929 flood was about one week.

6.2.4 Bank Erosion and Meandering

Topography downstream of the existing Munda Headworks forms an alluvial fan, where the riverbed slope abruptly becomes gentler. Thereby, sand bars are formed and bank erosions occur at bends in the river course downstream of the Headworks.

ID of NWFP is in a position to take actions such as bank protection and spur dike construction in order to stabilize the river course and protect the banks from erosion. Figure 6.2.3 indicates locations of bank protection works planned and completed under the Flood Protection Sector Projects.

6.2.5 Governmental Compensation for Flood Damage

No reliable record nor data with regard to the flood damage in the past were obtained. Among the data collected, the only indicative data are district-wise details of losses and damages due to floods and heavy rains obtained from Provincial Relief Commissioner. Summaries for Charsadda and Nowshera districts which are relevant to the Khiali (Swat) and Kabul Rivers are as follows:

Total Compensation Amount (Rs.)

Year	Charsadda District	Nowshera District
1995	4,551,700	861,000
1996	1,760,500	-
1997	592,000	1,508,000

6.3 Effect of Swat River Flood on Kabul River

In order to know the area of flood control effect by the Munda Dam, the flood hydrographs measured at Nowshera, Chakdara, Munda Headworks and Warsak were categorized into the following three patterns and frequency of occurrence of these patterns was examined:

Type-A: Flood Events in the Swat River measured at Chakdara and Munda H/W and in the Kabul River measured at Warsak, both of which occurred at the same time

Type-B: Flood Events in the Kabul River measured at Warsak which occurred only in the Kabul River (No floods measured at Chakdara and Munda)

Type-C: Flood Events in the Swat River measured at Chakdara and Munda H/W which occurred only in the Swat River (No floods measured at Warsak)

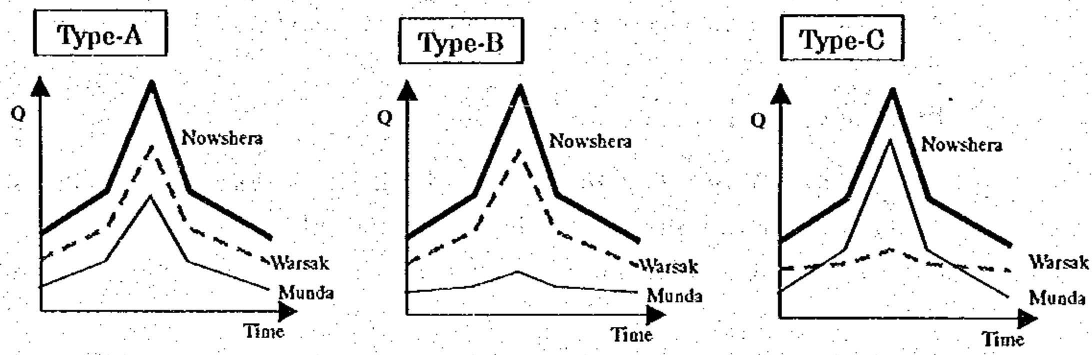


Figure 6.3.1 Flood Patterns

The examination was made based on the daily runoff hydrographs between 1964 and 1995 at four gauges and all major peak floods over 3,200 m^3/s measured at Nowshera, which is equivalent to a 2-year return flood. During the period examined, 28 floods occurred exceeding 3,200 m^3/s in total. Of these, three floods were categorized as Type-A, which happened at the Swat River and affected the Kabul River, corresponding to 11% of the total. Twenty floods (70% of the total) were categorized as Type-B, where the floods occurred in the Kabul River only. On the other hand, Type-C (floods in the Swat River alone) accounted for five floods, corresponding to 18%.

Thus, 29 - 30% of the major floods which occurred at Nowshera, are somehow relevant to the Swat River.

6.4 Flood Damage Estimation Area

The flood damage estimation area was delineated to include the area inundated due to Swat floods in the Swat River, that affected by backwater of the Swat floods in the Kabul River, and that between the confluence of the Swat and the Kabul Rivers and Nowshera. In order to know the backwater effect of Swat floods in the Kabul River it is desirable to conduct flood plain analysis by developing a mathematical simulation model of inundation. However, there are no input data such as detailed topographic maps and river sections except 1:50,000 scale topographic maps for the inundation areas, and hence, it requires many assumptions on the offered data. For these reasons, it was decided not to carry out the flood plain analysis. In the present Study, the flood damage estimation area was decided on the basis of 1:50,000 scale topographic maps and field investigation results.

The inundation areas experienced in 1929, 1995, and normal year were divided into the four areas in terms of influence of the Swat and Kabul floods, i.e. Area A from Munda H/W to Swat-Kabul confluence in the Swat River, Areas B1 from Warsak Dam to backwater end of Swat flood, B2 from backwater end of Swat flood to Swat-Kabul confluence and C from Swat-Kabul confluence to Nowshera in the Kabul River. The result of the division is shown in Figure 6 4.1.

In this flood control study, Area A, Area B2, and Area C were chosen as the flood damage estimation area. This covers the areas of flood control effect of the Munda Dam. Area B1 was not included in the flood damage estimation area because inundation of this area results from the Kabul floods only. The flood damage in Areas A, B2, and C was estimated under the following conditions:

1) 100% of flood damage is computed in Area A, since the inundation in Area A is caused by the Swat floods only.
2) Areas B2 and C are inundated by either of or both of the floods of Swat and Kabul Rivers. Thirty percent of the Swat floods were known to contribute to the floods at Nowshera as described in Section 6.3. An examination of the floods at the Swat River between 1988 and 1995 in connection with those at the Warsak Dam revealed that the Swat floods would contribute half of floods in volume that happened at Nowshera. Consequently, 15%(=30% x 50%) of

the flood damage in Areas B2 and C was taken as the base of the flood control effect of Munda Dam.

3) At Nowshera in Area C, the majority of river banks is of flood plain owned by the government and flood damage was considered marginal.

6.5 Flood Damage Factors

Flood damage factors are unit damage cost in Rupees per square kilometers flooded for each classified land use of the damage caused by a flood. Flood damages principally comprise damage to agricultural crops, housing, infrastructure, and other facilities. The flood damage factors were estimated based on agricultural crops damage factors, private housing damage factors, road damage factors, other direct damage factors, and indirect damage factors as explained in Appendix F.

The estimated flood damage factors per square kilometer for each land use categories for crops, private houses, roads, other direct and indirect damages are summarized in Table 6.5.1.

6.6 Flood Damages

The total potential losses can be estimated by applying the flood damage factors for each classified land use to the area inundated under "with" and "without" the construction of the Munda Dam.

For estimation of flood damages, the recent land use classification data were plotted on a 1:50,000 scale map and represented as 500 m x 500 m mesh data as shown in Figure 6.6.1.

The inundated areas of the "1929 flood", "1995 flood", and "normal year flood" were plotted in 500 m x 500 m mesh map. Inundation areas due to respective Swat floods and Kabul floods as well as the area affected by both floods are illustrated in Figure 6.4.1.

The inundation areas for each classified land use are calculated on the basis of the 500 m square mesh data for the respective Areas A, B and C. The total flood damage costs for Areas A, B1, B2 and C are summarized in Table 6.6.1.

The flood damages associated with the specific flood peak discharges were plotted on Figure 6.6.2, to obtain relationship between flood peak discharge and damage.

6.7 Flood Routing

6.7.1 Rule of Flood Regulation

Reservoir operation study and flood routing in the Munda reservoir were made by applying a constant rate-constant outflow discharging rule against the flood inflow hydrographs as shown in the following figure. The rule is to first select an inflow (Qa) for starting gate operation to release the discharge at a constant rate until it reaches a peak discharge (Qp), and then to release a constant discharge (Qp') thereafter.

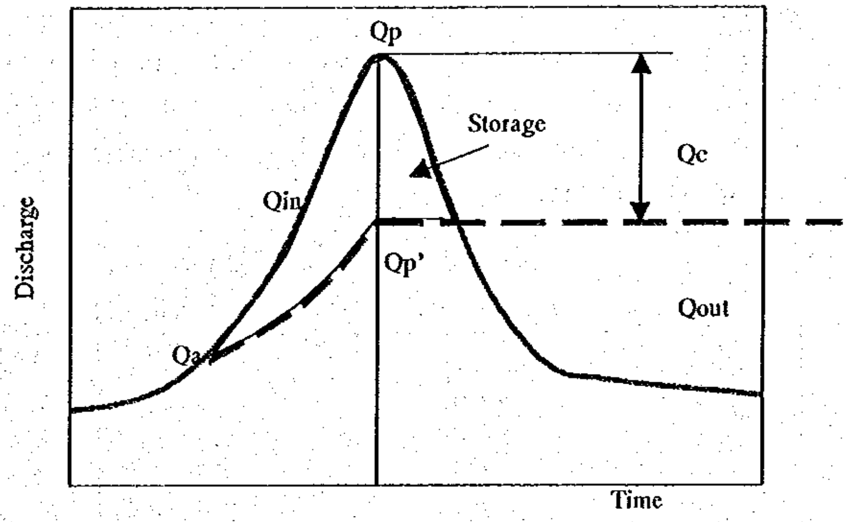


Figure 6.7.1 Constant Rate – Constant Outflow Rule

The constant rate-constant outflow rule is known to be effective even for medium to low floods and floods where no river improvement works are implemented at downstream reach.

For more effective flood regulation, the introduction of a flood forecasting system is to be made. The flood forecasting system will measure rainfall and discharge with use of telemetering system set up in the upstream catchment area and predict flood inflow into the reservoir. Federal Flood Commission has some experience of introducing the telemetering system in Pakistan with the financial assistance of ADB. It is recommended that the possibility of using a telemetering system in the Swat basin be studied in future.

6.7.2 Flood Frequency Curve

On the basis of the results of probable flood analysis for selected return periods of 1.5, 2, 5, 10, 20, 25, 50, 100, 200, 500, 1,000, and 10,000 years, the flood frequency curve for the Munda Dam site was developed as shown in Figure 6.7.2.

6.7.3 Flood Control Space

In order to find the optimum flood control space in the reservoir, eleven alternative flood control spaces varying from 0 to 300 million m^3 were compared.

The flood outflows after flood routing were then calculated by applying the constant rate-constant outflow rule against respective probable flood inflows and eleven alternative flood control spaces. According to the results, the flood peak inflows are reduced or mitigated depending on the flood control spaces. For example, mitigation effects against a 200-year probable flood of 5,720 m^3/sec are summarized below:

200-year Probable Flood and Peak Outflow

Flood Control Spaces (10^6 m^3)	Peak Outflow (m^3/sec)	Reduced Discharge from Peak Inflow (m^3/sec)	Remarks (Equivalent Probable Flood)
0	5,720	0	No Effect
1	5,650	70	180 year return flood
10	5,170	550	120 year return flood
20	4,780	940	70 year return flood
50	3,760	1,960	27 year return flood
75	3,050	2,670	14 year return flood
100	2,420	3,300	7 year return flood
150	1,470	4,250	3.5 year return flood
200	930	4,790	1.9 year return flood
250	670	5,050	
300	470	5,250	

From the above, it is seen that with a flood control space of 100 million m^3, the 200-year probable flood is reduced by 3,300 m^3/sec and mitigated up to a level of a 7-year return flood, almost equivalent to the 1995 flood.

Flood control benefits will be derived from the reduction of inflow discharge toward downstream reach.

6.7.4 Average Annual Flood Damage

Using the linear relationship between flood damage and flood peak inflows as shown in Figure 6.6.2, the annual average flood damage costs for different flood control spaces and probable flood peak inflows of different return periods were calculated. Tables 6.7.1 and 6.7.2 show, as the example, calculation results for

"Without Project" and those for a flood control space of 1 million m^3 "With Project".

6.8 Flood Control Benefits

The flood control benefits are attained as the reduction of average annual flood damages expressed as difference of annual average damages between with and without the Project. Figure 6.8.1 shows the relationship between the flood control benefits and control spaces.

According to Figure 6.8.1, optimum flood control space may fall within a range between 75 million and 100 million m^3. Increment of the benefit is marginal even if the control space is set at more than 100 million m^3 and hence it was concluded that a flood control space of 100 million m^3 be taken.

The larger flood control space may provide with more flexible operation for the flood control. In case 100 million m^3 of space is provided below FSL. it would result in the reduction of 0.4% of annual energy production being equal to US$0.6 million annual reduction while it causing the heightening of the spillway gate by some 4.4 m of which the annual cost is US$0.2 million in case that the flood control space is to be provided above FSL. Since the dam is designed on the basis of the normal reservoir water level (in this case, FSL.) against the Probable Maximum Flood as explained in Chapter 3, it is considered appropriate to allocate the flood control space above FSL.

CHAPTER 7 PLAN FORMULATION OF MULTIPURPOSE DAM

7.1 Concept of Plan Formulation

The development concept of the Project is of multipurpose, 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 the preceding Chapters 4, 5 and 6, respectively. The development plans presented in these Chapters for the respective sectors are 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 the 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 riverbank sides of the Swat River. The planned CCA is 4,066 ha for the left bank area and 2,043 ha for the right bank area. A tunnel system was selected as the feeder system at the left bank, while a lifting system with pumps at the right bank. Some deficits of irrigation water supply in LSC were found, which are to be supplemented by the Munda Dam. The water supply of 8.49 m^3/s to the civil canal 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.

The following are detailed descriptions of the study and results on the development scheme optimization conducted in this feasibility study stage.

7.2 Development Scheme Optimization

7.2.1 Development Scale Alternatives

The optimum development scale alternative is 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.

Dam site alternatives were studied during the first home office work stage in July 1998 and the results are described in Appendix G. The energy costs of three upstream dam sites as well as the proposed Munda Dam site were examined and compared, and it was concluded that the Munda site is economically most preferable among the alternatives. In the subsequent second field investigation stage in October 1998, field reconnaissance was made for the upstream dam sites, which confirmed the appropriateness of the Munda Dam site in terms of topography and geology.

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 details of the sediment volume estimates is presented in Section 3.3 Hydrology.

Lowest minimum operating level (MOL) of the reservoir was set taking into account the sediment level and an 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
(12)	560-494	EL. 560 m	EL. 494 m	EL. 475 m	EL. 568 m	218 m	1,148 mil m^3
	560-510	- do -	EL. 510 m	- do -	- do -	- do -	950 mil m^3
	560-530	- do -	EL. 530 m	- do -	- do -	- do -	637 mil m^3
	560-550	- do -	EL. 550 m	- do -	- do -	- do -	231 mil m^3
(13)	565-494	EL. 565 m	EL. 494 m	EL. 475 m	EL. 573 m	223 m	1,296 mil m^3
	565-510	- do -	EL. 510 m	- do -	- do -	- do -	1,108 mil m^3
	565-530	- do -	EL. 530 m	- do -	- do -	- do -	795 mil m^3
	565-550	- do -	EL. 550 m	- do -	- do -	- do -	389 mil m^3

No.	Case	FSL	MOL	Sediment Level	Dam Crest	Dam Height	Effective Storage
(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 7.2.1 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. This provision allows the power and irrigation sectors to utilize the entire space of the reservoir between FSL and MOL.

7.2.2 Reservoir Operation Simulation

In order to select the optimum development scale among the alternatives presented in the preceding section, a simulation model was developed and the reservoir operation of each alternative was simulated.

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 7.2.1.

7.2.3 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 at Upper Swat, upstream reaches 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.

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 by 0.7 the monthly pan evaporation records observed at Peshawar from 1966 to 1997. 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 1,172mm. The detail of the pan evaporation data at Peshawar is presented in Appendix C: Hydrology.

(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 7.2.2 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.

(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.

(5) Peak Operation Hour

The period of peak operation was decided to be 4 hours on the basis of the recent daily load curves of WAPDA and KESC system and assumed future trend.

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 advanced developing 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. The river cross sections surveyed in this feasibility study stage were used for the non-uniform flow computations.

7.2.4 Simulation Results of Reservoir Operation

The reservoir operations were simulated using the developed simulation model. All the above input data were incorporated in the simulations. 58 development scale alternative cases in total were simulated and as a result, annual energy production was estimated as shown in Table 7.2.1.

7.2.5 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 section, 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 7.2.3.

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 can be constructed in at least one year less than in ECRD in terms of construction period, owing to smaller embankment volume.
2) River diversion risks hence costs are less for CFRD than for ECRD.
3) 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.
4) Since the entire CFRD embankment is dry, earthquakes cannot cause pore pressure in the rockfill voids.
5) CFRD was cheaper by 12 % than in ECRD, mainly owing to shorter diversion tunnels.

A possibility to construct RCC type dam for the lowest dam case of 164 m in height was examined as seen in Figure 7.2.4, 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 of internal friction angle of 40 degrees.

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

7.2.6 Optimum Development Scheme

All the costs were estimated for the basic design developed for the respective alternatives and converted to the economic costs applying the 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 7.2.5 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.

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.

The development scale was optimized by comparing economic indices of the development alternatives derived from cash flow analyses. The 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 7.2.1 and Figure 7.2.6.

Based on this comparison, 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 7.2.7. 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 142 to 184 million US$ 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 with 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 some 200 m long flat topographic configuration above EL.565 to 570 m, to which dam crest level should be topographically limited.
2) As is described in Section 3.2, a series of water pressure tests done 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.
3) Sediment at the upstream end of the reservoir was preliminarily estimated to be deposited by some 20 to 25 m above the FSL. Since the river bed level at the upstream end of the reservoir appears to be around EL. 580 m, FSL 565 m is not considered appropriate.

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 the above simulation study is further verified in the following subsection.

Figure 7.2.8 shows 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.

7.2.7 Installed Capacity

For the FSL 555 and MOL 510 selected as above, an economic comparison to find the optimum installed capacity was made.

The result showing economic indices of the capacity alternatives such as NPV, B/C and EIRR is summarized below.

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. Four alternatives of the installed capacity were compared by net present values, which are base power plants of 110, 130, 140 and 150 MW. The comparison lead to negative net present values for all the alternatives. Therefore, it is judged that the base power operation is not feasible at the Munda Dam. The detailed discussion is given in Appendix G.

7.2.8 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	173.3mil 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 %

The above 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.

7.3 Development Layout Optimization

7.3.1 Dam Layout

(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.

(2) Dam Type

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

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

(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 8:

Spillway Alternatives and Required Dam Crest Level

	Case 1	Case 2	Case 3	Case 4
Spillway, Gate portion Number of gate Gate width Gate height	 2 15.0 m 25.1 m	 3 15.5 m 21.9 m	 4 15.5 m 18.4 m	 5 16.0 m 15.8 m
Spillway, Non gate part Crest length Crest elevation	 500 m EL.555.0 m	 180 m EL.555.0 m	 80 m EL.555.0 m	 0 m -
FSL Flood water level, FWL (PMF)	EL.555.0 m EL.559.7 m	EL.555.0 m EL.560.8 m	EL.555.0 m EL.561.8 m	EL.555.0 m EL.562.8 m
Freeboard Normal above FSL Minimum above FWL	 3.0 m 1.0 m	 3.0 m 1.0 m	 3.0 m 1.0 m	 3.0 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 the above 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 the above table was estimated conforming to the USBR procedure.

(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 7.2.5 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 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 in case of malfunction of the spillway gates. 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.

7.3.2 Generating and Transmission Facilities

(1) Generating Facilities

As the result of technical and economic studies on the dam and power station taking into account the topographic conditions, the powerhouse was planned as a normal surface type structure on the right bank of Swat river facing to the river. An access road suitable for access to the power station and transportation of equipment and materials including heavy items needs to be constructed along the right bank of the river starting from the main road. The machine erection bay will be arranged at the entrance of powerhouse on the same elevation as the access road and the upper level of main generators. The heavy equipment will be unloaded from trailers using the overhead travelling cranes of the powerhouse.

The main step-up transformers will be located at the back of the powerhouse on the same level as the access road for easy connection with the outdoor switchyard. For easiness in assembling and repairing transformers, a rail line will be lead from the transformer bay to the erection bay in powerhouse. The 220 kV open outdoor switchyard to connect with the outgoing transmission circuits will be located on a hill to the back of powerhouse. The main transformers and the outdoor switchyard will be connected with overhead lines. The power station side of the lines will be anchored to the top of the powerhouse structure.

(2) Transmission Facilities

As mentioned in Subsection 8.4.2, the most appropriate substation to receive the Munda power will be the 220 kV New Shahibagh substation. A 220 kV transmission line will be constructed between the Munda power station and New Shahibagh substation. For a certain length from the power station where site topography is steep, the line route will be aligned along the access road for easiness in access and transportation and then shortest route to the substation in flat cultivated fields.

The 220 kV bus of the New Shahibagh substation needs to be extended by two bays to connect two transmission line circuits from the Munda power station. The land space for such bus extension needs to be secured.

7.3.3 Irrigation Facilities

(1) General

There are two types of facilities conceivable for offtaking and feeding water to the new command area. One is a tunnel to directly tap water from the Munda reservoir to the entrance of the command area as proposed in the pre-F/S and another, a pump station to lift water after its use for power generation from the re-regulating pond located downstream the power station.

As the feeder system is a pivot of the irrigation system and governs relevant facilities in irrigation development, it is necessary to select an optimum layout of the feeder facility being followed by main canal system. A comparative study was carried out for two alternatives of tunnel and pump station as feeder system and main canals taking account of the following features:

Feeder System Alternatives

	Alternative I	Alternative II
Type	Tunnel	Pump Station
Specific Items considered in comparison	(1) intake facilities, (2) tunnel, (3) outlet facilities, (4) main canal and distributors etc. (5) mechanical works such as high pressure intake gates, outlet gates, etc.	(1) pump house (2) pipelines, (3) main canals and distributors (4) mechanical and electrical works such as intake gates, pumps, auxiliaries, etc.
O & M cost	Specific to tunnel feeder system	Specific to pumping station including electricity for pump operation
Energy Loss as Cost	Considered as water is tapped without harnessing for power generation	Nil

(2) Economic Comparison of Respective Alternatives of Feeder Facility and Main Canal

1) Facility Planning

a) Alternative I (Tunnel)

Irrigation water will be off-taken at either bank of the Munda dam. Through the tunnel, water offtaken will be fed and released to the main canal at the high-elevated portion on the left bank area but at the river-side on the left bank area. Main canal shall be by open channel at the either banks. Canal alignment will be winding due to irregularities of topographic condition at the left bank.

The tunnel is designed with circular shaped cross section as non-pressure tunnel with high pressure intake facilities. Alignment of the tunnel will be such that the shortest distance can be attained. In laying out the tunnel, consideration will be given on the topography, particularly deep nullahs crossing in between. Soil coverage of three diameters of tunnel is considered with radius of tunnel curvature being more than 30 m.

Shape of the canal system of the new command area is subject to the type of feeder facility. In case tunnel is selected as the feeder system, the canal system should be of an open channel network composed of lined main canal running in higher mountainous portion and distributaries flowing into lower command areas as proposed in the pre-F/S.

b) Alternative II (Pumping Station)

A pump station will be laid out by the proposed re-regulating pond with required lifting function. A part of released water from the power station will be lifted to the high-elevated portion at the left bank area but the riverside at the right bank area. Water level in suction sump of the proposed pump station varies more than 10 m, corresponding to the planned drawdown of the re-regulating pond. Therefore, it is difficult to select a suitable pump type. Vertical shaft double suction multistage centrifugal pump is the only pump type applicable for the proposed left bank pump station because of high head of more than 100 m required under the condition that plus-back inflow system in the sanction sump is adopted. Since the pump station at the right bank requires total head less than 20 m, a vertical shaft mixed flow pump is applicable.

In case lifting system with pump station is selected, the canal system should be a conduit network in which pressured pipe line will be applied in some sections of main canal. Conduit is advantageous in alignment selection where the terrain is rugged and steep slope, and water can not be fed by gravity.

Based on the above concepts in mind, the alternatives were developed for cost comparison. Salient feature of the alternatives developed for feeder facility and main canal are explained in following table:

Salient Feature of the Alternatives

Alternative I (by Tunnel)		Alternative II (by Pumping Station)	
Left Bank Area	Right Bank Area	Left Bank Area	Right Bank Area
(Feeder system) • Circular shaped cross section non-pressure tunnel • Excavated Dia : 2.80 m • Finished Dia : 2.20 m • Length : 5,000 m • Slope : 1/2,000 • Discharge : 4.39 m^3/s • Water Depth : 1.84 m • Outlet level : EL 470 m • Intake level : EL 515 m	(Feeder system) • Circular shaped cross section non-pressure tunnel • Excavated Dia : 2.80 m • Finished Dia : 2.20 m • Length : 2,600 m • Slope : 1/2,000 • Discharge : 2.20 m^3/s • Water Depth : 1.11m • Outlet Level : EL 385 m • Intake level : EL 515 m	(Feeder system) • Vertical Shaft Double Suction Multistage Centrifugal Pump • Actual Head : 94.0 m • Total Head : 117.7 m • Discharge : 3.92 m^3/s • Suction Level : EL 366 m • Pump number : 4 nos. • Bore : 700 mm • Pump Output : 1,500 kW • Power source : WAPDA grid	(Feeder system) • Vertical Shaft Mixed Flow Pump • Actual Head : 14.0 m • Total Head : 18.9 m • Discharge : 2.20 m^3/s • Suction Level : EL 366 m • Pump number : 4 nos. • Bore : 500 mm • Pump Output : 200 kW • Power source : WAPDA grid
(Open Channel) • Discharge : 4.39 m^3/s • Length : 14,200 m • Slope : 1/4,000	(Open Channel) • Discharge : 2.20 m^3/s • Length : 12,900 m • Slope : 1/4,000	(Open Channel) • Discharge : 2.230 m^3/s • Length : 11,200 m • Slope : 1/4,000	(Open Channel) • Discharge : 2.20 m^3/s • Length : 12,900 m • Slope : 1/4,000
(Conduit) Not applicable	(Conduit) Not applicable	(Conduit) Discharge : 3.92 m^3/s Length : 4,900 m Dia : 1800 mm	(Conduit) Not applicable

Irrigation facilities except feeder system and main canal are not presented because of no difference for any alternatives.

2) Construction Cost

Construction costs of alternatives for feeder facility and main canals are tabulated as follows:

Construction costs of alternatives for feeder facility and main canals

	Alternative I (by Tunnel)	Alternative II (by Pumping Station)
Left Bank	977,839,500 Rs (19,556,790$)	1,569,167,500 Rs (31,383,350$)
Right Bank	720,461,750 Rs (14,409,235$)	687,105,100 Rs (13,742,102$)
Total	1,048,301,250 Rs (20,966,025$)	2,256,272,600 Rs (45,125,452$)

3) Operation & Maintenance Cost

Operation and maintenance cost for the Alternative I of feeder facility composes expenditures of offtaking operation, repairing of tunnel facility, and maintenance of main canal. That for the Alternative II comprises of expenditures pump house maintenance, pump operation, and maintenance of main canal. Cost of the pump operation which is a major expenditure for the Alternative II, is estimated in consideration of the power station being of peak station and the required quantity of irrigation water with an assumption that operation of the pump station is dependent on electricity supply through main power supply grid. Annual electric power consumption for pump operation is as follows:

Annual Electric Power Consumption for Proposed Pump Operation

	Left Bank Area			Right Bank Area			Total
	Lifted water (10^3m^3)	Operating hour	MWh	Lifted water (10^3m^3)	Operating hour	MWh	MWh
Jan	2,307.3	163.7	982.0	1,295.7	163.3	130.6	1,112.6
Feb	2,615.2	185.5	1,113.0	1,468.6	185.1	148.1	1,261.1
Mar	2,487.1	176.4	1,058.5	1,396.7	176.0	140.8	1,199.4
Apr	5,170.6	366.8	2,200.6	2,903.7	366.0	292.8	2,493.4
May	7,534.9	534.5	3,206.9	4,231.4	533.3	426.6	3,633.5
Jun	9,757.1	692.1	4,152.7	5,479.3	690.6	552.5	4,705.1
Jul	5,555.7	394.1	2,364.5	3,119.9	393.2	314.6	2,679.1
Aug	4,818.7	341.8	2,050.9	2,706.1	341.1	272.8	2,323.7
Sep	5,390.8	382.4	2,294.4	3,027.3	381.5	305.2	2,599.6
Oct	3,097.3	219.7	1,318.2	1,739.4	219.2	175.4	1,493.6
Nov	2,130.2	151.1	906.6	1,196.3	150.8	120.6	1,027.2
Dec	1,741.4	123.5	741.2	977.9	123.3	98.6	839.8
			22,389.5			2,978.6	25,368.2

Buying rate of electricity is applied at 72 Rs./kW/Month of basic rate, and 2.24 Rs./kWh of additional rates.

Annual Operation and Maintenance Cost by Alternatives of Feeder Facilities

	Alternative I (by Tunnel)	Alternative II (by Pumping Station)
Left Bank	• Maintenance Cost 432,000 Rs • Operation Cost – • Repairing Cost 400,000 Rs	• Maintenance Cost 576,000 Rs • Operation Cost 55,336,480 Rs • Repairing Cost 400,000 Rs
Right Bank	• Maintenance Cost 288,000 Rs • Operation Cost – • Repairing Cost 300,000 Rs	• Maintenance Cost 432,000 Rs • Operation Cost 7,363,260 Rs • Repairing Cost 1,000,000 Rs
Total	1,420,000 Rs (28,400 $)	68,707,740 Rs (1,374,155 $)

(3) Energy Loss as Cost

In case of Alternative II, the water is lifted after it was utilized for power generation while no water can be used for power generation for the Alternative I as it is tapped directly from the reservoir. For fair comparison between two alternatives, the energy which can not be generated with use of water for irrigation should be evaluated. According to the reservoir operation study, if the water used for irrigation for the command area is utilized for power generation, the annual energy will be counted at 19,374 MWh and 10,880 MWh at the left bank and the right bank, respectively, being equivalent to annual revenues of

72,846,240 Rs. and 40,908,800 Rs., applying selling rate of 3.76 Rs./kWh, average tariff in April 1999. Those are deemed as energy losses, thus cost adherent to Alternative I.

(4) Overall Economic Comparison between Alternatives

Optimum plan for feeder facility can be determined through economic comparison of respective cost items. Here net present values (NPV) of selected items of the alternatives are calculated on economic cost basis so as to evaluate the alternatives comprehensively throughout the project life where the project lasts 50 years with discount rate of 10 %. Construction works are assumed to be completed within the initial five years.

Overall Economic Comparison between Alternatives for Feeder Facility

(1,000 Rs.)

Items	Alternative I (by Tunnel)		Alternative II (by Pumping Station)	
	Left Bank	Right Bank	Left Bank	Right Bank
Feeder Facility	286,854	143,747	399,238	118,154
Main Canal	389,345	363,204	662,453	363,204
O&M	5,605	3,961	403,596	59,249
Energy Loss as Cost	490,724	275,579		
Total	1,172,528	786,491	1,465,287	540,607

Based on the result of the comparison between alternatives, it was concluded that feeder system should be a tunnel system for the left bank area, and a pump station for the right bank area.

(5) Possible Alternative Power Source for Pump Station

In the above comparison, the energy cost/loss was calculated assuming the power for pump station is fed extending low voltage line from WAPDA grid as the Munda power is available only for 4 hours daily. As the head in the tunnel feeder system is substantial and the timing of water use is the same at both banks, it will be conceivable to utilize the head and irrigation water for the left bank by constructing a mini hydropower plant at the end of the feeder tunnel. If such idea can be materialized, the power generated there may be used for the pump station at the right bank. This idea was not studied in the present feasibility study but may be carried out at the time of detailed design.

7.4 Installation Timing

(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.

(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 numbers 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 %.

(3) Installation Timing

The result of the EGEAS computation revealed that the Munda project has an optimum installation timing of year 2010 as illustrated in Figure 7.4.1.

CHAPTER 8 FEASIBILITY DESIGN

8.1 General

The feasibility design for the main civil structures, hydromechanical works, electrical works and irrigation facilities was carried out conforming to the design criteria and standards internationally applied and accepted. The development scale of the Munda dam was decided as described in Chapter 7, in which FSL is 555 m with the installed capacity being 740 MW. General plan is shown in DGW. C03. This chapter presents optimization of the structures for the selected development scale and feasibility design for the optimal structures.

8.2 Main Civil Structures

8.2.1 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 because of the topography.

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 8.2.1. The required discharge capacity of one tunnel is 1,850 m^3/s. 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.

8.2.2 Main Dam

(1) Freeboard

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

The normal freeboard is 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 is 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.

Based on 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. The dam crest EL. 563.0 m satisfies the normal freeboard requirements.

(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 volume:	16,500,000 m^3

The plan, profile and typical dam cross section 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 experiences of CFRD construction have proved appropriateness of limestone, quartzite and schist as the rockfill material for CFRD as seen in Table 8.2.1. Details of the rockfill materials are discussed in Section 3.2.

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 8.2.1. In dam zoning, limestone is placed on the upstream zone and quartzite and schist are located on the downstream zone. According to Table 8.2.1, 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", application of downstream slope of 1:1.4 is suggested for the area such as Munda with the earthquake magnitude of 7 and peak acceleration of 0.15g. Therefore, the adopted downstream slope of 1:1.5 is considered 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 on the face slab is essential particularly at the lower elevation, 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 designs 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.

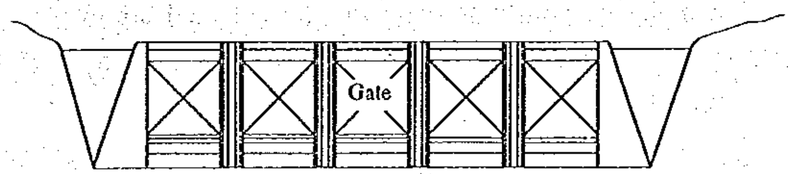
8.2.3 Spillway

(1) Spillway Configuration Optimization

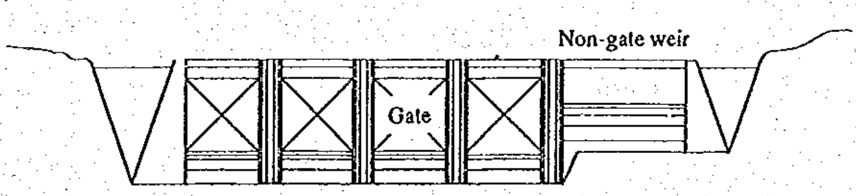
In order to regulate inflow floods into the Munda reservoir and ensure safety of the dam against extraordinary floods, the spillway configuration was optimized.

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. Further, since there exist neither proven criteria to accommodate device for sluicing the sediment for such high dam with wide valley nor the example where the device was successful, no consideration on sluicing the sediment through the spillway structure was made. 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 the spillway types were compared to select the optimum type:

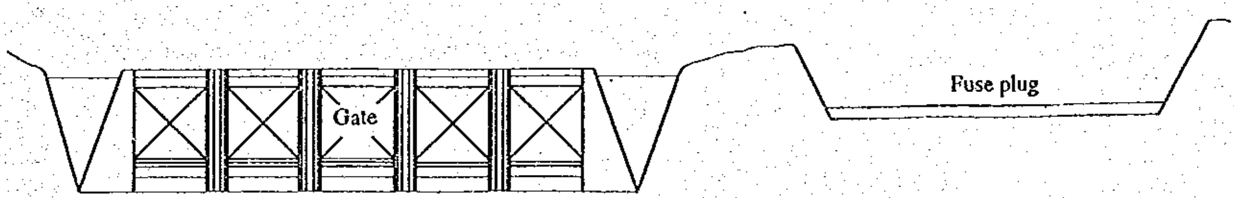
1) Type 1 : gated spillway alone



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



3) Type 3 : fuse plug with gated spillway



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, 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 case 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 cases, spillway Type 2 was selected in views 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
Gate portion			
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
Non-gated overflow weir			
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 cheapest, Case 2C, was selected. 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 overflow for chute:	3,800 m^3/s (1,000 years probable flood outflow)
Design overflow 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 at 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

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, 10 m lower than the gated ogee crest EL. by which the approach velocity is less than 0.4 m/s.

Headworks:

The headworks, consisting of gated and non-gated portions, 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, 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 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 illustrated in Figure 8.2.2. The ratio of the discharge capacity of 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 the 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 a 1,000-year probable flood outflow of 3,800 m^3/s. PMF outflow of 15,400 m^3/s is also able to be conveyed without overtopping from the chute wall. In the chute, aerators are to be provided for preventing from erosion due to cavitation on the chute floor.

Plunge pool:

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