8.2.4 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 reason,
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, which would happen in June. The velocity of 5.0 m/s is considered a maximum allowable velocity in the concrete conduit in normal case.

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 100 m in length for protecting concrete surface of the tunnel from the erosion due to high velocity jet flow.

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 detail of the river outlet.

8.2.5 Power Waterway

There are several alternatives conceivable as to the 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 the 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 8.2.3. 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, a waterway with a surge tank aligned on the right bank, was selected.

Alternative study on the powerhouse type was further made concentrating on its alignment at the right bank, which is either an open-air type or an underground type. Figure 8.2.4 represents these alternatives. In case of the underground powerhouse type, the waterway has an advantage of shorter penstock, that is a major cost component, compared with that for the open-air powerhouse type waterway. However, the comparative study shows that cost of the waterway with underground powerhouse is 1.3 times higher than that of the open-air powerhouse. Therefore, the waterway with the open-air type powerhouse was chosen.

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.

Optimum Diameter of 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 8.2.5 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 8.2.6:

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

Optimum Diameter of Penstock

Optimum diameter of the penstock was studied in the same manner as that for the headrace tunnel as shown in Figure 8.2.7. Two lane penstock was designed in order to reduce the risk of damage on the penstock. The diameter alternatives range from 6.8 m (velocity of 7.0 m/s) to 9.0 m (velocity of 4.0 m/s). The optimization study led to an optimum diameter of 7.4 m with the velocity of 6.0 m/s.

DWG. C10 shows the waterway.

8.2.6 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 towards 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.

As stated in subsection 8.2.3, 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 these reasons, the 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 4 units of 185 MW/unit capacity.

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

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

8.2.7 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 the 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 sure 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 the re-regulation weir in addition to the Munda Headworks is discussed in this subsection.

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 a comparative study was carried out. These sites are shown in Figure 8.2.8, where the location of the existing Munda Headworks is also represented as Site C for reference. According to the comparative study, the respective sites have the following features:

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, thus being probable to induce serious environmental issues. Therefore, 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 : $81m^3/s$ (= discharge/sec of 4-hour peak discharge)
2) Flood discharge : $2,420m^3/s$ (= 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, being subject to its optimization. Operation rule will be such that during the normal condition, several gates will be opened to release the stored water constantly; 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 8.2.9. Cost comparison revealed that the full gate type is by around 20% expensive in construction cost over the curtain wall type. Therefore, the curtain wall type was selected.

Figure 8.2.10 illustrates result of the optimization study for the number of gates for the re-regulation weir. The optimum number of gates is 7. The width and height of the gate are 8.0 m and 11 7 m, respectively. The details of the re-regulation weir are shown in DWG. C12.

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.

8.3 Hydromechanical Works

The following hydromechanical works are required for the Munda Dam and reservoir and for hydropower facilities:

Diversion tunnel	Diversion gate:	Slide gate, span 6.0 m, height 12.0 m, 2 sets,
Spillway	Spillway gate:	Radial gate, span 15.5 m, height 18.4 m, 4 sets,
	Spillway stoplog:	Span 15.5 m, 4 sets, gantry crane
Waterway	Trashrack:	Span 8.0 m, height 23.4 m, 3 sets,
	Intake gate:	Fixed wheel gate, span 6.0 m, height 12.0 m, 2 sets,

	Penstock steel liner:	7.4 m dia., 520 m long, 2 lanes
	Draft tube gate:	Slide gate, span 5.6 m, height 5.4 m, 2 sets, gantry crane,
River outlet	Trashrack:	Span 6.0 m, height 12.0 m, 2 sets
	Diversion gates:	Slide gate, span 6.0 m, height 12.0 m, 2 sets,
	Trashrack:	Span 5.8 m, height 2.4 m, 6 sets,
	Outlet gates:	High pressure slide gate, span 3.0 m, height 3.1 m, 4 sets,
	Downstream steel liner:	Span 6.0 m, height 3.1 m, length 100 m,
Re-regulation weir	Re-regulation gate	Radial gate, span 8.0 m, height 11.7 m, 7 sets,
	Re-regulation stoplog	Span 8.0 m, 7 sets, gantry

8.4 Electrical Works

8.4.1 Generating Equipment

As mentioned in Subsection 7.2.3, the Munda power station will be operated as a peak power station with normal peaking duration of 4 hours per day. Therefore, the turbine generators are to be designed for this operation mode of power station.

The basic hydraulic conditions for the generating equipment are summarized below:

- Full supply water level of intake reservoir : El 555 m
- Lowest operating water level : El 510 m
- Tail water level at plant discharge : El 369 m
- Tail water level at one unit operation : El 367 m
- Tail water level at no flow : El 364 m
- Maximum head : 186.0 m
- Minimum head : 141.0 m
- Rated effective head : 162.5 m
- Maximum plant discharge : 505 m^3/sec

The total output of the power station is determined at 740 MW (four units of 185 MW) as mentioned in Subsection 7.2.6.

(1) Number of Units and Unit Capacity

Number of units and unit capacity of turbine-generators of the power station are to be determined taking into account the following factors:

- The total output of power station is 740 MW and the power station will be operated as a peaking station with daily peak duration of 4 hours.
- The overall construction cost of a hydro power station of same output capacity decreases with decrease in number of units; from this viewpoint the number of units can be reduced up to two.
- Due to limited availability of river flow, the power station needs to be operated under partial output in dry season.
- The limit of transportation weight on the access road will be around 90 tons according to past examples. The largest unit size in the Generation Development Plan is not larger than 200 MW in the surrounding area.
- Operation flexibility and influence to the power system in case of the separation of one unit. The normal lowest limit of Francis turbine operation is 40 to 50% of the rated output depending on the duration of operation.

Taking into account these factors as well as the result of comparative study made in Chapter 7, it was determined that the power station is to be provided with 4 units of 185 MW water turbine generators.

The rated output of water turbine will be 189 MW taking into account the generator efficiency of around 98% at full output.

(2) Water Turbines

Type: Considering the working head and unit capacity, the water turbines of this power station will be of vertical-shaft Francis type.

Overload capacity: The water turbine will be suitable for operation at over load larger than the rated output when the working head is higher than the rated head. Of the WAPDA power stations, the overload capacity of the Tarbela turbines is 110% and that of the Mangla turbines is 115%. Referring also to examples in various other countries, the overload capacity of the Munda turbines is selected at 110%.

Rotating speed: Referring to the upper limit of turbine specific speed of the selected water turbine in various standards, the speed of turbine generator is selected at 187.5 rpm in the speed class of 50 Hz generator, which corresponds to turbine specific speed of 140.4 m-kW.

Speed governors: An electronic PID governor system will be provided for speed control of each turbine. The governing system will be connected to and be fully compatible with the power station control and monitoring equipment.

Inlet valves: A butterfly valve will be provided to each turbine to open and close the water flow into turbine.

(3) Generators

Type: The generators will be vertical-shaft, AC 3-phase synchronous generators of revolving field and semi-umbrella type to be operated directly coupled with the above water turbines.

Unit output capacity: The unit capacity of generator will be 220 MVA based on unit power output of 185 MW and power factor of 0.85. that is suitable to generate 110% overload power under power factor of around 95%.

Rated voltage: The rated voltage of the generator is tentatively selected at 13.2 kV, which is widely applied for this output class of generator.

Rated speed: According to the speed limit of water turbine, the generator speed is selected at 187.5 rpm of the 50 Hz system. The number of generator magnetic poles corresponding to this rotating speed is 32.

Excitation system: The static excitation system to provide generator excitation with a set of thyristor rectifiers will be applied to the generators. An automatic voltage regulator (AVR) will be provided to regulate the generator terminal voltage and as well as the 220 kV bus voltage.

(4) Step-up Transformers

The main transformers to step-up the generator voltage of 13.2 kV to the transmission voltage of 220 kV will have 220 MVA capacity, same as the generators.

The 3-phase transformer will consist of 3 single-phase units to reduce the transportation weight of 3-phase transformer to the site.

(5) Control System

A computer-aided Distributed Control System (DCS) will be applied for supervision, control and protection of the power station. All these functions will be allocated to computer managed local control centers for each unit of water turbine generator, auxiliary power system, outdoor switchyard, spillways, etc. These centers will be interconnected with each other and with the station control computer and central system by means of data communication link. The connection with the load dispatching center will also be maintained.

Normal power station supervision and control is performed in the central control room. For equipment adjustment and test, local control is performed at the local control centers. The control of important equipment will be possible also from the load dispatching center.

(6) 220 kV Outdoor Switchyard

The 220 kV outdoor switchyard to connect the generator circuits with the transmission circuits will be located on a hill top to the backside of the powerhouse.

The switchyard will be of open outdoor construction with double bus bar. The switchyard will have 7 circuits, 4 circuits for the generator circuits, 2 for outgoing transmission line, and one for the bustie circuit.

The control of switchyard equipment should normally be performed from the power station main control room, but local control from the switchyard control room will also possible as mentioned in Paragraph (4) above.

(7) Overhead Travelling Crane

The maximum lifting weight of overhead travelling crane will be 450 ton, the weight of generator rotor. The heavy items of the power station will be lifted with two cranes of equal capacity, 2 x 225 tons, using a lifting beam. Thus, two erection works with relatively small lifting weight will be able to be performed by parallel operation of two cranes.

(8) Overall System Connection

The overall connection on the power station is presented in Figure 8.4.1.

8.4.2 Transmission Line and Substation

(1) System Connection

For sending out the power station maximum output of 740 MW over 30 km distance, 220 kV will be as the most appropriate transmission voltage as referred to many examples in southeast Asian countries. The 500 kV transmission is suitable to long distance transmission of large power, for instance transmission of the Tarbela power to Lahore and Gatti. Adoption of the 500 kV transmission to the Munda project will result in very high cost as explained in Appendix D10.

There are several alternative plans for the 500 kV system, but its construction cost is at least 2.4 times of the proposed 220 kV plan to connect to the New Shahibagh substation.

In the surrounding area of the Munda power station, there are 5 220 kV substations including existing and planned ones; the existing Mardan and Peshawar 500 kV and the planned Shahibagh, Nowshera and Charsadda with relative locations shown below:

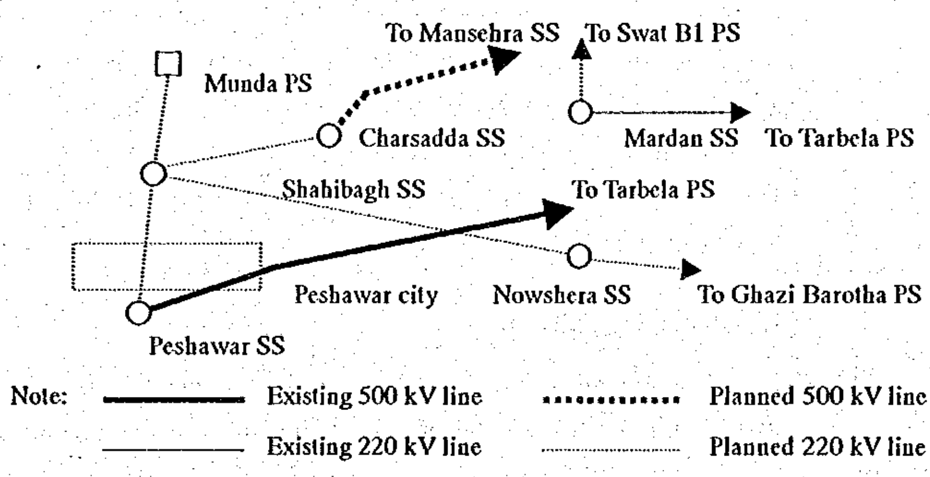

Figure 8.4.2 Relative Location of Munda PS and Substations

The New Shahibagh and Charsada substations are nearer to the Munda power station compared with the other three substations. The site of the Charsadda substation has not yet been selected, but distance from the Munda power station to these 2 substations is almost equal, 25 to 30 km. While, the distances to the existing two substations and to the Nowshera substation are much longer, around two times. To receive all the Munda power at one of the existing substations, bus modification will be required to increase the power transfer capacity. While, new substations can be designed taking into account transfer of the large Munda power.

Peshawar city is a large power demand center and the Munda power will be consumed mostly in the Peshawar area. The New Shahibagh Substation at the northern suburb of Peshawar and nearest to Munda is considered as the most appropriate 220 kV substation to connect with the Munda power station. Available land for construction of 220 kV lines to the New Shahibagh Substation is limited, and it is necessary to allocate line routes for connecting transmission lines taking into account the line from the Munda Power Station.

The northern 500/220 kV system connection in 2010 is shown in Figures D9.1 for the 500 kV system and D9.2 for the 220 kV system of Appendix D9, Transmission System Analysis.

(2) Transmission Line

The 220 kV transmission line between the Munda power station and the New Shahibagh substation will be a double circuit line with 2-bundled Rail conductors with particulars mentioned in Appendix D10. The safe transmitting power of this line under single circuit operation is around 700 MVA, and satisfies the 'N-1' criteria of system reliability. The influence to the national network due to loss of one circuit of transmission line is not larger than the loss of one generator or transformer of the Munda power station, and the chance of one circuit separation will be rare as explained in Appendix D10.

(3) Receiving Substation

The receiving substation of the Munda power will be the New Shahibagh substation, to be constructed in the north suburb of Peshawar.

(4) Line Protection

The current differential protection will be applied to line protection as the most reliable practice currently employed world-widely for important transmission lines. Duplicate system (two systems of main relays) will be employed for the planned important lines. The optical system is required for data transmission of this protection.

In case that the optical communication system is not provided on the transmission line, the normal carrier-aided distance protection practice will be applied instead of the above current differential protection.

The distance protection system with the fault locating function will be provided for standby protection.

(5) Transmission System Analysis

It was intended to determine the method of transmission line connection according to results of transmission system analysis. Therefore, transmission characteristics were compared for connections to the New Shahibagh substation and to the Charsadda substation, using the PSS/E software of PTI Inc. of USA for power flow analysis. The details of analysis are included in Appendix D9.

1) Studied transmission system

The transmission system behavior was analyzed for the 500/220 kV transmission system in 2010 just after the Munda project will be commissioned. According to the generation development plan prepared in NPP in Appendix D6, the system conditions for analysis were decided as follows:

- All the hydroelectric projects in the northern area in the Generation Program for the Normal Growth Scenario, which are planned to be completed prior to the Munda project, were taken into account. Transmission lines to connect the national grid with all major power stations were also taken into account.
- The northern transmission system that covers up to the Gatti and Lahore substations was analyzed. Power flows in and out from these two substations were substituted with equivalent loads.

2) Contents of analysis

The power flow analysis will be carried out for the three conceived alternatives mentioned below for the cases with and without Munda output were analyzed.

Case 1: Connection to the 220 kV bus of Shahibagh substation
Case 2: Connection to the 220 kV bus of Charsadda substation
Case 3: Connection to the 500 kV bus of Charsadda substation

For the Charsadda connection cases, the Charsadda substation was assumed to be existing for Munda connection.

Particulars of analysis are shown in Appendix D9.

3) Results of analysis

No problems due to connection of the Munda power station were found in the transmission system for the selected three alternatives. At all the points of transmission system, both power flow and system voltage are within normal ranges. However, the following are noted from the analysis results:

- The generated power of the Munda power station will be consumed in the Peshawar area, at the New Shahibagh and Nowshera substations. In case that the Munda power station is connected to the Charsadda substation, the power flows to Shahibagh through Charsadda.

- The power flow on the Mansehra-Charsadda section is too little to justify construction of a 500 kV system by 2010.

The required rupturing capacity of 220 kV circuit breakers in the area is not much as large power stations will be connected through long transmission lines. The present standard rupturing capacity of 31.5 kA will be sufficient to all 220 kV circuit breakers related to the Munda project.

(6) Conclusion for Transmission System Selection

The connection to the New Shahibagh substation with a double-circuit transmission line with 2-bundled Rail conductors is judged as the least cost and most appropriate plan satisfying technical requirements as detailed in Appendix D10, Transmission System Study.

(7) Communication System

The Munda power station is an important power station for the power system operation. Therefore, a high-speed and reliable communication system is required for power station operation and line protection. All major power stations need to have stable communication paths with the national load dispatching center.

There is a proposal to provide the optical communication system on the 220 kV line in the section of Ghazi Barotha–Nowshera–Shahibagh–Peshawar as mentioned in Subsection 4.5.7. In addition, the 500 kV Manthera–Charsadda system will also be provided with the optical communication system. These optical systems are to be extended to the Munda power station through OPGW on the 220 kV line or other means.

The PLC system will be provided on the transmission line for standby duty of the optical system. The frequency allocation for the PLC system in the Peshawar area is very congested.

The radio system for emergency and maintenance communication will be provided with UHF duplex or VHF simplex communication system.

8.5 Irrigation Facilities

8.5.1 Basic Concept for the Feasibility Designing

Irrigation facilities for the New Irrigation Scheme of the Study is to be designed as an newly established system. The Irrigation system should be designed as being functional to apply demand base irrigation, on the assumption that the proposed irrigation development plan can be realized. Layout of the irrigation system will take into consideration the deployment of beneficial farm lands and location of rivers and water courses so as to apply proposed on-farm development system.

8.5.2 Feeder System

As studied and concluded in the Development Layout Optimization of Irrigation Facilities in Chapter 7, a tunnel system in left bank area and a lifting system with pump in right bank area are selected for the feeder system. Layout of the proposed feeder system are shown in DWG. I01, I02 and I03. Dimensions of proposed feeder systems are as follows:

Dimensions of proposed feeder systems

Left Bank Irrigation System		Right Bank Irrigation System	
Tunnel Type:	Circular shaped cross section non-pressure tunnel	Pump Type:	Vertical Shaft Mixed Flow Pump
Excavated Dia.:	2.40 m (8.0 feet)	Actual Head:	14.0 m (46.7 feet)
Finished Dia.:	2.20 m (7.33 feet)	Total Head:	18.88 m
Length :	4,950 m (16,500 feet)	Low Sanction Level:	EL 366.0 m
Longit. Slope:	1/2,000	Pump Number:	4 nos.
Discharge:	4.391m^3/sec (155.0 Cs.)	Discharge:	2.204 m^3/sec (77.9 Cs.)
Design Water Depth:	1.836 m	Bore:	500 mm
Discharge Level:	EL 466.0 m	Pump output:	200 kw
Intake Level:	EL 470.0 m		

8.5.3 Main Canal and Distributaries System

(1) Main Canal System

Main canal is applied as an open channel with concrete lining. Alignment was made optimum in construction cost covering all beneficial farm lands by gravity. Many related structures of main canal such as river crossing bridge turnouts are required to be installed. Salient features of main canals of the Project including numbers of several related structures are as follows:

Salient features of main canals

Left Bank Irrigation System		Right Bank Irrigation System	
Maximum discharge:	4.391 m^3/sec	Maximum discharge:	2.204 m^3/sec
Length:	13,950 m	Length:	12,900 m
Bed slope:	1/4,000	Bed slope:	1/4,000
Side slope:	1: 1.5	Side slope:	1: 1.5
(Related structures)		(Related structures)	
Super passage:	4 nos.	Super passage:	3 nos.
Nala culvert:	34 nos.	Nala culvert:	36 nos.
Canal escape	3 nos.	Canal escape	4 nos.
Bridge:	26 nos.	Bridge:	25 nos.
Offtake:	4 nos.	Offtake:	5 nos.
Mogha:	2 nos.	Mogha:	0 no.

Structures of main canal and those related facilities are shown in Appendix E.

(2) Distributaries System

Canal water shall offtake to distributaries at the required points. Layout of distributaries will be taken in consideration with deployment of beneficial farm plots and location of existing water courses so as to supply irrigation water by gravity. Features of distributaries of the Project are as follows:

Features of Distributaries

Name	Command Area (ha)	Discharge (m^3/s)	Length (m)*
(Left Bank Area)			
D1	1,750	1.890	8,400
D2	525	0.354	4,500
D3	850	1.140	6,800
D4	450	0.486	2,900
(Right Bank Area)			
D1	355	0.382	1,450
D2	253	0.273	1,400
D3	215	0.232	600
D4	815	0.879	2,150
D5	405	0.437	1,950

*: This includes a length of minor canal.

Structures of distributary and those related facilities are shown in Appendix E.

8.5.4 On-Farm System

General layout of on-farm development is shown in Appendix E. Diverted irrigation water at the *mogha* flows into main watercource, and then is introduced to each field at *pucca nacca*. Command area of the *mogha* is subject to the capacity of the *mogha*. Capacity of main watercourse should be smaller than 2.5 - 3.0 cusecs so that farmers can easily do water management by themselves. The Command area of *mogha* at the CCA base is, therefore, decided at 280 acres (113 ha) at best, applying specific peak discharge for irrigation of 11.15 cusecs/10^3acres.

Along the main watercourse, irrigation water diverts to several watercourses. Command area of *mogha* is divided into modal farm units which be commanded by *pucca nacca*. The *pucca nacca* locates along the watercourse. As applying turning irrigation, number's of the modal farm units is recommended to be multiples of six, modal farm unit is laid down around 12 acres (4.7 ha) obtained dividing 280 by 24.

Typical layouts should be divided into two groups, one is the case that *mogha* is equipped along distributary directly (Distributary to Watercourse [DTM] system), and the other is the case that the same locates in minor distributary (Minor canal to Watercourse [MTW] system).

CHAPTER 9 CONSTRUCTION PLAN AND COST ESTIMATE

9.1 Construction Plan and Schedule

9.1.1 Conditions and Assumptions

(1) Access to the Site

The project site is located about 37 km north of Peshawar in NWFP. The proposed dam axis is located on the Swat River about 5 km upstream of the existing Munda Headworks at Abazai near the town of Shabqadar Deri. The paved highway is extended from Peshawar to the Munda Headworks. The Munda Headworks can be also reached from Nowshera by road via way of Charsadda and Tangi with road distance of about 60 km. This route will be used for transportation of heavy and bulky cargoes to the left bank of the dam site, since the concrete bridge provided alongside the Headworks is not suitable for passage of heavy truck trailers.

Imported goods including construction machinery, generating equipment, and steel materials for metal works will be landed at the port of Karachi and transported to the site on road without difficulties. The road distance from Karachi to Peshawar (via Jamshoro, Mianwali, Tarnal, Taxila, Attock, and Nowshera) is 1,700 km approximately.

Between Munda Headworks and proposed dam site, a gravel road constructed by WAPDA is available on the right bank of the Swat River. 4-WD vehicle can also travel along the left bank trail up to half way to the dam site. These roads need upgrading for heavy vehicles before construction work.

(2) Meteorological Conditions

(a) Rainfall data

The most reliable data source for rainfall analysis in construction planning is Abazai rain gauging station located about 10 km north of the project site. The average monthly rainfall and rainy days recorded at Abazai from 1961 to 1997 are summarized in the tables below.

Monthly Rainfall Record at Abazai

(mm)

Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec	Total
22	44	91	44	11	8	100	102	30	16	20	28	516

Monthly Rainy Days at Abazai

Range (mm)	Jan	Feb	Mar	Apr	May	June	July	Aug	Sep	Oct	Nov	Dec	Total
0	28.2	23.6	24.8	25.9	29.6	29.1	26.0	25.6	27.9	29.6	28.6	28.6	327.3
0 - 5	0.9	1.6	1.6	1.6	0.6	0.3	0.9	1.4	0.5	0.5	0.2	0.6	11.0
5 - 10	1.0	1.2	1.4	0.9	0.4	0.3	1.1	0.8	0.4	0.3	0.4	0.8	8.8
10 - 20	0.6	0.9	1.5	0.9	0.2	0.2	1.3	1.1	0.8	0.5	0.4	0.5	8.9
20 - 40	0.2	0.6	1.4	0.6	0.1	0.1	1.1	1.5	0.3	0.1	0.3	0.4	6.8
40<	0.0	0.1	0.3	0.1	0.0	0.0	0.7	0.6	0.1	0.0	0.1	0.1	2.3

As shown in these tables, the annual rainfall is about 520 mm and rain is concentrated in March, July, and August. The construction works will not be hampered much by rainfall even in these three rainy months, since major open-air activities are excavation work, rock embankment work, and concrete work.

(b) Ambient temperature

The maximum and minimum temperatures recorded at Peshawar from 1961 to 1990 are averaged as shown below:

Ambient Temperature Record

(C°)

	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec
Max.	18.3	19.5	23.7	30.0	35.9	40.4	37.7	35.7	35.0	31.2	25.6	20.1
Min.	4.0	6.3	11.2	16.4	21.3	25.7	26.6	25.7	22.7	16.1	9.6	4.9

Temperature control of mixed concrete by means of chilled water and/or ice flake is essential especially in hot-weather season from May to September.

(c) Hydrological conditions

The average discharge at the proposed dam site is estimated as shown below based on the hydrological data obtained from the river gauging stations in the Swat river basin.

Estimated Discharge at Dam Site

(m^3/s)

Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec	Average
29.7	34.8	100.9	250.2	337.3	530.3	623.2	372.4	88.0	35.8	36.1	34.1	206.1

Fluctuation of natural discharge is very large through a year, and the period between June and August can be identified as high flow or flood season. Based on these data, the most adequate timing of river diversion and reservoir impounding will be as follows:

i) River diversion : in October (when non-flood season starts)

ii) Reservoir impounding : between September and May (to avoid rapid rise of reservoir water level)

The probable flood in certain return period is calculated as below:

Probable Flood at the proposed Dam Site

Return period (year)	Discharge (m^3/s)
2	1,052
5	2,053
10	2,743
20	3,406
25	3,630
50	4,373
100	5,013

The temporary river diversion facilities including diversion tunnels and cofferdams are designed for a 25-year probable flood.

(3) Source of Construction Materials

The sources of natural materials for construction work are envisaged as below:

Source of Construction Materials

Materials	Source	Hauling distance
1. Dam embankment		
Zone 1A	Borrow area in west Sadar Garhi	7 km
Zone 1B	River deposit downstream of Munda Headworks	8 km
Zone 2A	River deposit downstream of Munda Headworks with plant process	8 km
Zone 2B	River deposit downstream of Munda Headworks with plant process	8 km
Zone 3A	Sappare quarried rock (selected small rock)	6 km
Zone 3B	Sappare quarried rock	6 km
Zone 3C	Todobo Banda quarried	2 km
Zone 3D	Fresh rock from mandatory excavation	1.5 km
Zone 3E	Sappare quarried rock (selected large rock)	6 km
2. Concrete aggregate		
Coarse aggregate	River deposit downstream of Munda Headworks with plant process	8 km
Fine aggregate	River deposit downstream of Munda Headworks with plant process and sand production	8 km

Major construction materials such as cement, reinforcement bars, and explosives are readily available from local industries and market. Shortage of such construction materials that would hamper construction of the project has not occurred in recent years, thus stable supply of these materials is expected.

(4) Spoil Bank

Total excavation volume of earthwork is estimated at about 5,930,000 m^3, of which about 4,340,000 m^3 is required to be disposed in the site area.

Disposal Volume of Earth Materials

(Unit: 1000 m^3)

Description	Common	Rock	Total
Excavation	840	5,090	5,930
Re-use for rockfill 3D zone	-	1,590	1,590
Disposal	840	3,500	4,340

The disposal materials are planned to dumped off in a spoil bank located on a left bank terrace about 1 km upstream of the proposed dam site shown in Figure 9.1.1. The two alternative spoil banks in Sappare and left bank gully near the proposed re-regulation weir (about 1 km downstream of the weir) are not appropriate for the following reasons:

Sappare: There exists wide flat land in Sappare. However, higher hauling cost for dumping work is unavoidable due to long hauling distance of 6 km and elevation difference of some 200 m.

The left bank gully: The main access road to the dam site will pass in this gully, so the traffic of project vehicles will be disturbed, if the spoil bank is located in this area.

(5) Construction Power Supply

The electric power necessary for the construction work is estimated at about 5,000 kVA.

The 132/66 kV substation nearest to the Munda dam site is 132 kV Tangi substation about 15 km from the dam site. The 132/66 kV transmission system in these area is generally heavily loaded. According to the Distribution Company, at the present stage, 5000 kVA power to the Munda site can be supplied from the Tangi substation. However, the actual situation must be reviewed again prior to commencing power receiving.

A 11 kV distribution line does not have enough transfer capacity to send 5000 kVA power over a distance of 15 km. The construction cost of 132 kV system extension is high, so it will be more appropriate to transfer power at 33 kV. Temporary power stepping down facilities are required at the Tangi substation, and 33 kV power needs to be stepped down again to 11 kV to feed power to each work site. The 33 kV line can be used for emergency power supply to the power station after commissioning.

For receiving construction power (Temporary Supply) from the Distribution Company, an application should be submitted at least one year earlier than commencement of use with a certain amount of deposit, and particulars of power receiving should be finalized.

(6) Construction Water Supply

Water supply will be required for construction camp, aggregate plant, concrete plant, boring work, dam embankment work, and concrete curing work during construction. The water will be supplied from the Swat river by pumping equipment and necessary piping.

(7) Contract Packaging

The construction works will be carried out by various contractors having different expertise. The preparatory works will be executed by local contractors prior to main construction work. The main construction work will be undertaken by international contractors selected through international competitive bid. The contract packages contemplated at this moment are as follows:

(a) Local contract package

Lot L-1 Access road	: Construction of permanent access road between the Munda Headworks and dam site. (right bank route : 6 km, left bank route : 5 km)
Lot L-2 WAPDA camp	: Construction of permanent offices and residential buildings with 15 ha land area in Shabqadar Deri. The proposed location is shown in Figure 9.1.1
Lot L-3 Power supply system	: Construction of 33 kV transmission line from Tangi S/S to the project site for construction power supply.

(b) International contract package

Lot I-1 Diversion tunnel	: Construction of diversion tunnels

Lot I-2	Main civil works	: Construction of all the main civil work structures including concrete faced rockfill dam, spillway, power intake, headrace tunnel, surgetank, penstock, powerhouse, switchyard, bottom outlet and re-regulation weir.
Lot I-3	Gate and penstock	: Supply and erection of metal works including spillway gates, intake gates, penstock steel liners, river outlet facilities and re-regulation gates.
Lot I-4	Turbines and auxiliaries	: Supply and erection of turbines, inlet valves, O/H travelling cranes and control equipment,
Lot I-5	Generator and auxiliaries	: Supply and erection of generators and control equipment
Lot I-6	Switchgear equipment	: Supply and erection of 220 kV switchgear and transformers
Lot I-7	Transmission line & S/S	: Supply and erection of 220 kV transmission line and substation
Lot I-8	Irrigation facilities	: Construction of irrigation facilities in both right and left banks

In order to squeeze total construction period by fast-tracking procedure, diversion tunnel work (Lot I-1) is separated from the main civil work for the river to be diverted earlier. The procurement of Lot I-1 work will be made 11 months ahead of the Lot I-2 main civil work, thus the overall time schedule can be reduced by one year.

9.1.2 Construction Method

Construction method which would be applied to the major construction works is described hereinafter.

(1) River Diversion Work

Temporary river diversion during construction period is planned to be achieved by the diversion tunnel method. To cope with a 25-year probable flood of 3,630 m^3/s, the following two diversion tunnels will be provided in the left bank of the dam site.

Diversion Tunnels

No.	Diameter	Length	Inlet sill level	Outlet sill level	Section
No. 1	12.0 m	940 m	EL. 363	EL. 362	Circular
No. 2	12.0 m	950 m	EL. 379	EL. 370	Circular

Diversion tunnel No.1 is a main diversion tunnel which has a enough discharge capacity during non-flood season by itself alone with a cofferdam having crest elevation of 410 m. While, diversion tunnel No. 2 is an auxiliary diversion tunnel which will be used only during the flood season and reservoir impounding period. The diversion tunnel No. 2 will be located about 16 m higher than the diversion tunnel No.1 and river outlet facilities will be provided in the tunnel.

Both tunnels will be driven by top heading and bottom bench-cut method. Employing such construction equipment as 3-boom hydraulic drill jumbo, 3 m^3 side dump loader, and 20 ton articulated dump trucks, a driving rate of 50 m/month and 70 m/month is expected to be attained for top heading and bottom bench cut operation, respectively. Since construction of diversion tunnel is on the critical path, tunnel will be excavated from both inlet and outlet portals. Eleven months will be required for breakthrough of the diversion tunnel with these arrangements. Excavation of diversion tunnel No. 2 will be commenced with the same equipment after excavation work in diversion tunnel No. 1 is completed.

Concrete lining work will follow the tunnel driving work in the diversion tunnel No. 1. The tunnel wall and crown concrete will be placed at first by 9 m-long circular travelling form to allow concurrent work with excavation work. Invert concrete will be placed separately afterward.

After completion of the diversion tunnel No. 1, upstream and downstream primary cofferdams will be embanked to divert the river flow into the tunnel. The river will be diverted in the beginning of October 2004 when the high flow season ends. The dimensions of primary cofferdams which will endure 2-year probable flood are as follows:

Primary Cofferdam

Cofferdam	Crest Elevation	Height	Volume
Upstream	El. 380	18 m	88,000 m^3
Downstream	El. 376	15 m	57,000 m^3

Integrated cofferdam scheme (by partial facing of slab concrete of the main dam) is adopted in this study without construction of an independent cofferdam between main dam and upstream primary cofferdam for the following reasons:

i) Stage construction method has been widely applied in construction of CFRD. The partial placing of slab concrete up to a certain level will provide a function of cofferdam to the main dam. With this arrangement, main cofferdam is required for only the first flood-season, but not for the whole construction period.

ii) The height and embankment volume of the independent cofferdam will be 45 m and 700,000 m^3 approximately for the designated flood discharge of 3,630 m^3/s. Furthermore, alignment of the diversion inlet channel will be changed and this results in some 10,000 m^3 of additional rock excavation. The additional cost for these works is estimated at about 5.5 million US$.

The primary cofferdams are required to be constructed twice in 2004 and 2005, since they may be flushed out during the flood season in 2005. Overall procedure of river diversion is summarized below:

Procedure of River Diversion Work

Period	Activity
Sept/2004 to May/2005	1) River diversion by the primary cofferdams 2) Dam foundation excavation in riverbed (down to EL. 349) 3) Concrete work for plinth under EL. 360 4) Dam embankment up to EL. 360 5) Backfill to original riverbed level, EL. 362
June/2005 to Aug/2005	1) River flow will be diverted to the original river course. 2) Dam work suspended except for plinth above EL. 380
Sept/2005 to May/2006	1) River diversion by primary cofferdams 2) Removal of backfill material (down to EL. 349) 3) Concrete work for plinth below EL. 410 4) Curtain grouting beneath plinth concrete 5) Dam embankment up to EL. 410 (u/s portion only) 6) Concrete facing below EL. 410
After June/2006	1) Dam embankment continues without interruption

Dewatering arrangement should be carefully planned during 1st and 2nd non-flood seasons, since seepage flow through the existing riverbed sediment of about 8 m thickness might disturb excavation and concrete works downstream of the primary cofferdam. The dewatering and seepage control will be achieved by submergible pumps with pits, well point facilities and/or jet grouting in the riverbed sediment layer.

(2) Embankment Work for CFRD

The concrete face rockfill dam has a total embankment volume of about 16,500,000 m^3, and its embankment volume curve is as shown in Figure 9.1.2. The dam body comprises the following embankment zones:

Embankment Volume of CFRD

Zone	Materials	Volume (1,000m³)	Proportion
1A	Impervious soil	92	0.6%
1B	Random fill	286	1.7%
2A	Fine filter	8	0.1%
2B	Crushed rock	273	1.7%
3A	Selected small rock	512	3.1%
3B	Blasted rock	5,900	35.8%
3C	Blasted rock	7,175	43.6%
3D	Blasted rock	2,066	12.5%
3E	Selected large rock	154	0.9%
Total		16,466	100.0%

The embankment materials will be obtained from the borrow pit and quarry sites described in Chapter 9.1.1 (3). Monthly embankment volume of 500,000 m^3 will be attained by arrangement of the following construction equipment for 3B, 3C, and 3D rock zones with day and night shifts work.

Bulldozer, 65 ton	5 units
Wheel loader, 11 m^3	5 units
Dump trucks, 45 ton	45 units

The rock materials hauled by heavy dump trucks will be spread at the dam site by bulldozers and compaction work will be made by the following equipment.

Compaction of Embankment Materials

Zone	Compaction Equipment	Layer thickness (m)	Nos. of Roller Pass
1A	Tamping roller	0.25	8
1B	Vibratory roller	0.5	6
2 & 3A	Plate compactor Vibratory roller	0.4	6
3B	Vibratory roller	0.8	6
3C & 3D	Vibratory roller	1.5	6

(3) Concrete Work for CFRD

The face slab concrete will be placed with slip form of about 15 m wide. The mixed concrete delivered to the dam site by agitator trucks will be further conveyed to the placing area through concrete chute. The slip form will be moved by winch with travelling speed of some 1.5 m/hr and the concrete will be continuously placed without making horizontal joint.

The face slab construction work is planned to be carried out in 3 stages as detailed below:

Stage Construction of Face Slab Concrete Work

Zone	1st stage	2nd stage	3rd stage
Top elevation	410 m	500 m	563 m
Bottom elevation	354 m	405 m	500 m
Top width	220 m	540 m	800 m
Slab area	22,000 m^2	66,000 m^2	76,000 m^2
Block (layer) width	15 m	15 m	15 m
Nos. of block (layer)	15	36	54
Nos. of slip form	2	3	3
Work period	4 months Jan/2006-Apr./2006	6 months Jul/2007-Dec./2007	7 months Sep/2008-Mar./2009

The stage construction method described above was planned in consideration of the following advantages for the project:

1) To provide cofferdam function to the main dam with 1st stage slab concrete in early stage of the construction.
2) To enable concurrent work of embankment and concrete works, thus shorten total dam construction period.
3) To avoid segregation of concrete mix during conveyance of concrete in long chute.

The sequence of dam construction work is illustrated in Figure 9.1.3.

(4) Spillway

The work quantities of spillway work are summarized below:

Work Quantity of Spillway

Area	Excavation	Concrete
Headworks	1,029,000 m^3	103,500 m^3
Chuteway		125,000 m^3
Plunge pool	1,050,000 m^3	-
Total	2,079,000 m^3	228,500 m^3

The excavation work will be commenced from the headwork area with bench cut blasting method and continued in a downward direction along the chuteway. In the plunge pool area, slope excavation of the left bank extended about 160 m high will be required. Since the bottom elevation of the plunge pool is about 5 m lower than the existing riverbed level, excavation work will be carried out only during the non-flood season with coffer dike arrangement surrounding the work area.

The concrete work for the headwork and chuteway will be done by various concrete placing equipment including concrete pumps, truck mounted belt conveyer placers, and tower cranes. The equipment will be selected according to the following criteria in general:

Working Performance of Concrete Equipment

Equipment	Concrete pump truck	Concrete pump, stationary	Conveyer placers	Tower cranes
Max. aggregate size	40 mm	40 mm	80 mm	150 mm
Slump of concrete	12 cm or more	12 cm or more	-	-
Horizontal coverage	20 m	300 m	30 m	75 m
Vertical coverage	30 m	100 m	15 m	100 m

With average concreting volume of 10,000 m^3/month, about two years will be required for the concrete work.

The installation of the radial gates will be started by Lot I-3 contractor following the concrete work in the headworks.

(5) Power Intake

The work quantities of power intake work are summarized below:

Work Quantity of Power Intake

Work	Quantity
Excavation, open	150,000 m^3
Excavation, gate shaft	10,200 m^3
Concrete, intake structure	8,500 m^3
Concrete, gate shaft	5,500 m^3

The excavation work in the inlet area will be done with ordinary blasting operation. This work is required in the early stage of construction program for earlier commencement of the headrace excavation work.

The gate shaft will be excavated by sinking method with shotcrete and rock bolt supporting. If actual geological condition does not allow this, initial concrete lining will be provided after each round of excavation work.

The construction of gate shaft and intake structure will be made after breakthrough of headrace tunnel to avoid possible work interference.

(6) Waterway

The power waterway, about 1 km long, comprises headrace tunnel, surge tank and penstock sections.

Work Quantity of Waterway

Work	Headrace tunnel	Surge Tank	Penstock
Excavation, open	-	7,000 m^3	125,000 m^3
Excavation, tunnel	490 m	-	190 m
Excavation, shaft	-	61 m	-
Concrete	28,900 m^3	8,400 m^3	12,500 m^3

The headrace tunnel having finished diameter of 12.0 m will be driven from its inlet by top heading and bottom bench-cut method, employing such construction equipment as 3-boom hydraulic drill jumbo, 3 m^3 side dump loader and 20 ton articulated dump trucks. The average excavation rate of 50 m/month is expected to be attained with these arrangements.

The surge tank shaft will be excavated either by sinking method or upward pilot hole drilling & downward enlargement method, and this will be left to the contractor's option.

The penstock tunnel will be excavated from its outlet with same method and equipment employed for headrace tunnel work after completion of open excavation in its portal area.

Installation of the penstock steel pipe will be carried out by Lot I-3 contractor following the penstock civil work.

(7) Powerhouse

The construction of powerhouse will be started with excavation work in the right bank slope. Since the downstream cofferdam is located downstream of the powerhouse site, the excavation work will be carried out easily even in the area below the existing riverbed level. Following the excavation work, concrete work for powerhouse substructure will be done with tower crane and concrete pump. Draft tubes will be installed by Lot I-4 contractor during the concrete work.

Installation of overhead travelling crane in the powerhouse superstructure is a milestone event for the succeeding installation work of hydropower plants. In the construction program, 18 months are allocated for installation work thereof.

(8) River Outlet Work and Final River Closure

The river outlet facilities provided in the diversion tunnel No. 2 comprise high pressure gates, maintenance gates, gate chamber, connecting tunnel to the chamber, and steel liner in the downstream section of the gates.

Installation work of river outlet facilities will be commenced from October 2007 when non-flood season will start, discharging river flow through diversion tunnel No.1. Installation of the river outlet gates will be completed during this non-flood season, however, steel liner installation and corresponding concrete work will be continued in the flood season in 2008. If a 25 year probable flood occurs during flood season in 2008 when only diversion tunnel No.1 can release river water, upstream water level is estimated to reach at EL. 496 m. This means that the following works are required to be completed before June 2008.

1) Facing slab concrete below EL. 500 m
2) Installation of intake gates (Intake sill level is EL. 485 m.)

After the river outlet facilities will be installed, impounding of reservoir can be commenced. The reservoir impounding plan is needed to be established taking the following requirements into account:

1) The reservoir impounding should be executed only during non-flood season in order to avoid rapid rise of reservoir water level which may adversely effect the dam body and inside slope in the reservoir area.
2) Even in the impounding period, river water should be discharged downstream for irrigation use as follows:

Irrigation Water Requirement

(m^3/s)

Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec	Mean
13.9	21.0	34.8	56.9	73.3	73.3	45.3	56.1	60.0	50.7	40.5	24.6	45.9

Based on these requirements, the final river closure is planned to be achieved in the following manner:

Procedure of Final River Closure

	Date	Activity
1.	Beginning of December 2008	1) Diversion tunnel No.1 is closed by stoplogs. 2) Plug concrete work starts in diversion tunnel No. 1. 3) Inlet gates of diversion tunnel No. 2 is to be left open. 4) Water level rises to sill level of diversion tunnel No. 2 within three days.
2	December 2008 to March 2009	1) Water is discharged through river outlet gates by partial/intermittent gate control. 2) Excess inflow is filled in the reservoir. 3) Water level rises at a rate of 25 m/month and reaches at upper bell mouth of bottom outlet at the end of March 2009. 4) When water level reaches at the bell mouth (El. 470 m), inlet gates of diversion tunnel No. 2 are closed by hydraulic remote control system. The hydraulic piping should be double circuits to avoid malfunction of remote control system.
3	April 2009	1) Water level reaches at Minimum Operation Level (El. 510) at the end of April. 2) Water rising rate is 50 m/month in this period.
4	May 2009 to June 2009	1) Wet test of generating units can be started from the beginning of May. 2) Water level reaches Full Supply Level (El. 555) in the middle of June.

(9) Re-regulation Weir

The construction of re-regulation weir will be achieved by provision of diversion canal in the left bank terrace, taking advantage of topography in the proposed site.

After diverting the river flow into the diversion canal by cofferdams, foundation excavation and concrete work will be started. The mass concrete will be placed by tower cranes, while concrete pump will be used for structural concrete placement.

(10) Irrigation facilities

The left bank irrigation facilities include a feeder tunnel of 4,900 m long and finished diameter of 2.2 m. The tunnel will be driven by blasting method with rail mucking system from both upstream and downstream portals. The tunnel will be temporarily supported with shotcrete and rock bolting. Concrete lining work will be carried out by needle-beam type concrete form.

The lengths of irrigation canals are 14,200 m and 12,900 m for left bank and right bank, respectively. The canal will be constructed by cut and fill method employing 0.7 m^3 backhoes, 11 ton dump trucks and vibratory rollers.

9.1.3 Construction Schedule

The implementation schedule of the project after this feasibility study, which will be completed in March 2000, is shown in bar chart form in Figure 9.1.4. The civil work will be completed in the first quarter of 2009 and all the generating units will be put into commercial operation at the end of December 2009 after 8 years construction period.

The detailed construction program is shown in Figure 9.1.5. The program was prepared in consideration of required duration of each activity, appropriate activity sequence, hydrological constraints, and leveling of construction resources.

The following activities constitute the critical path in the overall time schedule.

1) Preparation of PC-I (Implementation Program)
2) Financial arrangement
3) Selection of consultant
4) Detailed design and bid document preparation
5) Procurement procedures
6) Access road construction
7) Diversion tunnel construction
8) River diversion (end September 2004)
9) Dam construction
10) Reservoir impounding (from December 2008)
11) Wet test of generating equipment
12) Commissioning of power plant No.1 (end June 2009)
13) Commissioning of all the power plant (end December 2009)

9.2 Cost Estimate

9.2.1 Conditions and Assumptions

The project cost was estimated for the basic design with the following conditions and assumptions:

(1) The estimate is made in US dollars (US$) for both foreign and local currency components.

(2) The local currency component covers cost of locally available materials including cement, reinforcing bars, fuel and explosives and local labors. The costs of imported machinery for mechanical and electrical works and

depreciation of construction equipment are allocated in the foreign currency component.

(3) The price level of the estimate is September 1999 when site investigation work was carried out. The exchange rate used in the estimate is US$ 1.0 = Rs.50 which prevailed in the same period.

(4) The construction work is assumed to be undertaken by competent contractors selected through international competitive bidding (ICB) and local competitive bidding (LCB).

(5) The unit prices of the works were determined in reference to prevailing unit cost of labour, construction materials and equipment in principle. The recent bidding data of similar projects in Pakistan and other Asian countries were also reflected in the estimate. The price data for labour, materials and equipment used in the estimate are tabulated in Table 9.2.1, 9.2.2 and 9.2.3 respectively.

9.2.2 Cost Estimate Method

The following estimate method was applied for the respective cost categories.

(1) Preparatory Works (Lot L-1 and L-2)

The construction cost of the permanent access road (Lot L-1) was estimated with required road length (11 km) and unit cost per linear meter (300 US$/m). The cost for WAPDA base camp was estimated as a lump sum in reference to the data from similar projects in Pakistan.

(2) Civil Works (Lot I-1, I-2 and I-8)

The construction cost for diversion tunnels, main civil works and irrigation facilities was estimated with unit price estimating method. The work quantities and corresponding unit prices are shown in Tables 9.2.5, 9.2.6, 9.2.9, and 9.2.10.

(3) Gate and Penstock (Lot I-3)

The unit price estimating method was applied. The weight of steel structures was calculated in view of its dimension and design hydraulic pressure. The recent bidding data of similar works was referenced to determine unit price per weight of respective type of structure.

(4) Electrical Works (Lot I-4, I-5, I-6 and I-7)

The lump sum estimating method was applied. The price was estimated based on various factors including design head, discharge, rated capacity, dimensions, unit numbers and recent bidding data of similar works.

(5) Engineering Service

The cost for engineering service for detailed design, procurement of works and site supervision was estimated at 6.5% of the direct construction cost. This rate was determined in reference to the ongoing hydropower project in Pakistan.

(6) Administration Expense

Administration expense of the Project owner (WAPDA) is estimated at 2.5% of the direct construction cost. This rate was determined in reference to the on-going hydropower project in Pakistan.

(7) Land Compensation and Environmental Mitigation

Land compensation and resettlement cost is estimated by unit price estimate method, taking results of the latest environmental assessment into account. The cost items and relevant cost data are shown in Tables 9.2.11 and 9.2.12.

(8) Tax

The local taxes imposed on the contract works are customs duties, sales tax and corporate tax. The following tax rates were considered in the cost estimate:

Tax Rates

Tax and Duties	Rate
Customs duty	
- Steel products	35%
- Electrical equipment	10% to 35%
- Construction equipment	25% to 35%
Sales tax	15%
Corporate tax	6%

(9) Contingencies

The contingencies required for the project budgeting comprise i) price contingency to compensate future price escalation and ii) physical contingency to cover changes of physical conditions unforeseeable at this stage.

Price contingency is estimated with assumed price escalation rate of 2.4% per annum for both foreign and local currency components. The rate of 2.4% is derived from the latest projection of MUV (Manufacturing Unit Value in G-5 countries) during 1999 - 2010 indicated by World Bank.

The rates of physical contingencies applied in the estimate are as below:

Physical Contingency

Works	Rate
Preparatory works (L-1 to L-3)	12%
Diversion tunnel (I-1)	12%
Main civil work (I-2)	12%
Electrical and mechanical works (I-3 to I-7)	7%
Irrigation facilities (I-8)	12%
Engineering Service	10%
Administration	10%
Land compensation	10%
Environmental mitigation	10%

9.2.3 Total Project Cost

The total project cost is estimated at US\$ 1,148.9 x 10^6 comprising foreign currency of US\$ 611.8 x 10^6 and local currency component of US\$ 537.1 x 10^6 as summarized below and detailed in Table 9.2.4.

Estimate of Total Project Cost

(Unit: million US\$)

Description	F.C.	L.C.	Total
I. Base Cost	474.0	414.0	888.0
Construction Cost	440.0	257.7	697.7
Engineering Service	34.0	11.3	45.3
Administration		17.4	17.4
Land compensation		2.5	2.5
Environmental mitigation		5.0	5.0
Tax		120.1	120.1
II. Contingency	137.8	123.1	260.9
Price contingency	91.5	77.6	169.1
Physical contingency	46.3	45.5	91.8
Total Project Cost	611.8	537.1	1,148.9

Breakdown of cost estimate is shown in the following tables:

1) Diversion tunnels : Table 9.2.5
2) Main civil works : Table 9.2.6
3) Gate and penstock : Table 9.2.7
4) Electrical work : Table 9.2.8
5) Irrigation facilities : Table 9.2.9 and 9.2.10
6) Land compensation : Table 9.2.11
7) Environmental mitigation : Table 9.2.12

9.2.4 Annual Disbursement Schedule

The annual disbursement is estimated in accordance with the estimated project cost and the construction time schedule as detailed in Table 9.2.13 and summarized below:

Summary of Annual Disbursement Schedule

(Unit: million US$)

Year	F.C.	L.C.	Total
2001	5.9	3.9	9.8
2002	14.9	20.4	35.3
2003	27.6	38.8	66.4
2004	43.7	49.1	92.8
2005	65.5	53.4	118.9
2006	117.0	90.0	207.0
2007	136.0	103.9	239.9
2008	136.0	118.2	254.2
2009	65.1	59.4	124.5