

Paper No. 681

DIAMER BASHA DAM PROJECT

Dr. Izhar ul Haq, Syed Tanveer Abbas

DIAMER BASHA DAM PROJECT

Dr. Izhar ul Haq¹ & Syed Tanveer Abbas²

ABSTRACT

Diamer Basha Dam Project would be 272m high, roller compacted concrete (RCC) dam, with storage capacity of 9.05 Bcm of water for irrigation and power generation capacity of 4500 mw. The project prefeasibility was carried out in 1984, Feasibility in 2004 and the Detailed Engineering Design was completed in 2008.

An optimized project layout was selected before detailed engineering design and its economic indicators were established. For preparing detailed engineering design the following tasks were required:

Additional geotechnical investigations were carried out to complement the existing information for detailed design. These investigations included drilling, exploratory adits, geophysical surveys, field and laboratory tests. About 50 meters deep river alluvium will have to be removed from the base of the concrete dam in the river valley.

Special attention was given to the dam foundation and abutments, powerhouse caverns, tunnels and underground works, cut slopes and other structure locations.

The safety of main dam was checked for the various combinations of load, as per international design practices. Special studies were carried out for thermal analysis of RCC dam. Detailed mix design tests were carried out to achieve the required strength and reduce the heat of hydration by reducing cement content and adding pozzolans.

In the design of project underground works state-of-the art techniques were used including Finite Element Method and rock wedge stability. During design due consideration was given to in situ stresses in the rock mass. Similarly the seismic analysis was carried out to check the stability of these structures under dynamic loading.

The powerhouse cavern, being the largest underground works, requires special attention. Its dimensions were optimized as a function of the electro-mechanical equipment, control system and auxiliary equipment to be housed.

Details of foundation treatment, grout curtain, drainage, instrumentation, construction method and construction planning were worked out. Structural integrity of the proposed dam was confirmed for the worst combination of loads.

¹ General Manager, WAPDA.

² Deputy Director, WAPDA.

The detailed hydraulic and structural design of the spillway, and energy dissipation system was completed keeping in view the result of hydraulic model studies carried out.

Resettlement Action plan and Environment Management Plan have been prepared according to the guidelines of International Financial Institutions.

GENERAL

Diamer Basha Dam Project is proposed to be located on Indus River 315 km upstream of Tarbela Dam, and is accessible via Karakoram Highway. Location Map is in Figure 1. It is designed to form a large reservoir on Indus River with an active storage of 7.89 BCM (6.39 MAF). The second purpose of the project is to generate hydropower with average annual generation of 18097 GWH, as well as an additional generation of 1111 GWH/annum at Tarbela Dam. This will be accomplished through two powerhouses, one under each bank, with a total installed capacity of 4500 MW.

Pre feasibility was carried out by Montreal Engineering consultants (MONENCO) from 1982-84. Feasibility was carrier out by NEAC consultants from 2002 – 2004. In 2005, Diamer Basha Consultants (DBC) were engaged by WAPDA for conducting 'Review of Feasibility, Detailed Engineering Design and Tender Drawings/Documents of Diamer Basha Dam Project'. DBC completed tender design for various works of the project in June, 2008.

Project will have some adverse impacts during construction activities (about 10 years) and subsequent impounding of reservoir. Main impacts will be dislocation of 28650 people and acquisition of 15150 ha (37419 acres) of land. These have been thoroughly evaluated and the relevant mitigation / management measures proposed in Resettlement Action Plan (RAP). Detailed Environmental Impact Assessment and Cultural Impact Assessment have been done. Resettlement Action Plan and Environmental Management plans have been prepared.

1. PROJECT FEATURES

The project comprises of the following major components:-

- | | | |
|------|----------------------------|--|
| i. | Dam | 272 m high roller compacted concrete (RCC) Gravity |
| ii. | Spillway | 14 bays, each 11.5 m wide and 16.24 m high |
| iii. | Low Level Outlets | 2 No. 7.2 m dia. |
| iv. | Reservoir Flushing Outlets | 5 No. 9.0 m dia. |
| v. | Power Houses | 2 No. 2250 MW each, underground |
| vi. | Power Units | 12 No. 375 MW each |
| vii. | Power Waterways | 4 No. concrete headrace tunnels (15.3 m dia. each)
12 No. steel penstocks (7.2 m dia. each)
4 No. tailrace tunnels, shotcreted
4 No. surge chambers |

- | | | |
|-------|-------------------------------|---------------------------------|
| viii. | Power Intake Flushing Tunnels | 2 No. 1 on each bank |
| ix. | Temporary Works | 1 No. diversion canal |
| | | 2 No. diversion tunnels |
| | | 2 No. coffer dams (u/s and d/s) |

2. PROJECT LAYOUT

Layout of the project is shown in Figure – 2. The dam will be slightly curved gravity structure of roller compacted concrete (RCC) with a crest length of over 1000 m and maximum height of 272 m above the rock foundation. A spillway will be located over the dam, while five Reservoir Flushing Outlets (RFOs) and two Low Level Outlets (LLOs) will be embedded through the dam body at lower level. The spillway will be used for controlling the reservoir water level in the event of floods. LLOs will be operated to replenish the downstream supplies in case of any shortfall through power outflows. RFOs have been designed for the purpose of flushing sediment deposited in the reservoir after a period of about 40 years or so of the project operation.

Two underground powerhouses will be constructed, one on each bank with a capacity of 2250 MW (total 4500 MW). Total number of turbo-generator units will be 12, each of 375 MW. Each power scheme will consist of two headrace tunnels, two surge tanks, six penstocks, six power units and two tailrace tunnels. The design discharge for each unit will be 247 m³/s.

The access to the left bank works is available via KKH, however access to the right bank works will have to be provided. A suspension bridge has been designed for permanent access to the right bank. This bridge will be located about 1100 m downstream of the dam axis. The bridge will span right across the river, about 200 m between the abutments. The clear road width will be 7 m with 1.65 m walkways on either side. The bridge has been designed according to AASHTO specifications, for a live load of 150 tons.

The dam and spillway will require some 16.7 million m³ of RCC together with some 2 million m³ of reinforced conventional concrete. An estimated 27.51 million m³ of soil and rock will be excavated from both dam abutments and from the diversion canal. All of the excavation will have to be performed prior to the start of dam construction.

3. ROLLER COMPACTED CONCRETE (RCC) DAM

Diamer Basha dam will be a concrete gravity dam with a crest level of 1170 m asl and maximum height of 272 m above the rock foundation. Total length of dam along crest will be 1169 m.

The dam will be a Roller Compacted Concrete (RCC) gravity structure. In plan, the dam body is curved to activate and exploit arching forces and thereby better withstand seismic loads. The dam comprises 36 separate blocks inclusive of the abutment blocks at both ends of the curved crest. On the left upper bank, the topography requires a small wing dam, also made of RCC, connected to the

abutment block. The joints between blocks will not be sealed with grout. In the central river section the blocks are each 35 m wide, while on the flanks they are all 25m wide.

The bulk of the dam will be constructed with roller compacted concrete (RCC) with water-tight upstream facing of reinforced concrete having double metal water-stops at vertical block joints. The dam body will have adequate system of drainage galleries with connecting pipe systems, and foundation drains drilled into the bed rock. The foundation improvement measures will include consolidation grouting under the dam base, and a deep grout curtain under the heel of the dam. The following project components will be incorporated within the body of the dam: -

- i. Spillway
- ii. Low level outlets
- iii. Reservoir flushing outlets

The basic parameters for the dam are as follows:

• Lowest Foundation level	898 m asl
• Elevation of dam crest	1170 m asl
• Maximum height	272 m
• Crest thickness	13 m
• Overall crest length	1168.75 m
• Block width (central sections)	35m
• Maximum operating level:	1160.0 m asl
• Minimum operating level:	1060.0 m asl
• Volume of RCC	16,752,500 m ³

3.1 Design Philosophy

The main component of the Diamer Basha Dam Project is the construction of a 272 m high gravity dam using roller-compacted concrete (RCC). The dam section will have upstream - downstream width of about 220 m at foundation level. Contraction joints formed in the transverse direction will divide the dam into separate slice-shaped blocks, each with a width of normally 35 m, extending over the full height of the dam and over the whole upstream to downstream length of the dam section. Figure – 3 presents the longitudinal section of the dam comprising of 35 blocks and the spillway located in the central seven blocks. Figure – 4 gives the cross section of the dam through the spillway.

In RCC, horizontal construction joints are inevitable because of its lift method of construction, whereby each lift comprises a number of ‘fresh to fresh’ compacted layers. Due to the differential temperatures developing during the cooling process of the dam body, tensile stresses will occur in zones near the dam

faces. Where these tensile stresses exceed the tensile strength of the RCC, particularly at the lift joints, de-bonding effects may occur.

De-bonded joints, in combination with thermal cracks, which can hardly be eliminated in case of large concrete gravity dams, constitute potential flow paths of seepage water in the dam body. Seepage would be associated with pore water pressures inside joints and possible cracks, which would be causing uplift, thus affecting the stability of the dam.

3.2 RCC Placement

For RCC placement the dam was divided into twenty-four zones. For the construction of the first three zones, designated as Zones 1, 2 and 3, truck delivery is assumed to have an available sustainable placement capacity of 190m³/hr. For the remaining 21 zones of the dam belt delivery of sustained capacity 900m³/hr is anticipated. The construction is anticipated in three (3) meter high lifts. Each lift is constructed in layers of 0.3m layers. The layers will have a slope of about 1 to 10. The sloped layer placement method proved to permit better temperature control than RCC placement in horizontal layers. To efficiently apply this method, several blocks will be combined into RCC placing zones. RCC placing shall take place parallel to the river flow direction but may also be carried out in the transversal (cross valley) direction if a suitable method to combine with the facing is developed by the contractor.

Due to the sloped layer method, the surface area of the fresh RCC is limited, a faster bond to the next layer is possible, the risk of cold joints decreases and the temperature stress in the RCC can be better controlled. Another benefit is the limitation of the number of working zones. This eases the construction process and reduces breaks needed for equipment moving.

- **Placement Rates**

Scheduling is based on a sustained RCC delivered rate of 190 m³/hr for truck placement (zones 1, 2 and 3) and 900 m³/hr for belt placement - consisting of 2 belts each of sustained 450 m³/hr capacity.

- **Placement Zones**

The placement schedule is divided into placement zones, each zone covering one or more blocks. Thus when placement is specified as being in an upstream-downstream direction, then all indicated blocks are placed together, i.e. with a placement width given by the combined width of all blocks specified for the zone at a given elevation. Note that this width can change because the number of blocks in the zone at a given elevation can change.

- **Placement around galleries and shafts**

Special placement is required around galleries and vertical shafts. These are mainly located adjacent to the upstream face, the extent of this special placement zone being assumed to extend to 10m downstream from the

downstream edge of the gallery. Within this zone, placement rates were reduced for galleries and for shafts.

- **Upstream face**

The 1.5m wide upstream face to be constructed of reinforced conventional mass concrete (CMC) is to be placed simultaneously with the adjacent RCC layer. It is assumed that this placement, the required reinforcement steel, as well as the 0.5m wide grout enriched roller compacted concrete (GE_RCC), will not interrupt construction of the adjacent RCC, i.e. special concrete delivery facilities will have to be provided, i.e. cable crane, additional belt, etc. for placing the CMC and GE_RCC.

- **Spillway zones**

Zones 15 and 22 indicate the spillway bays. These will be constructed of reinforced CMC.

- **Placement Temperature**

Special temperature control is to be maintained for RCC placement where the minimum dimension is larger than 1 meter and the volume is more than 30 cubic meters. This RCC shall be normally placed within the range of 4 to 26 °C.

Temperature will be measured 20 minutes after placement. Heating of the mixing water of aggregate will not be permitted until the temperature of the concrete decreases to 6 °C. The material shall be heated in such a manner that it will be free from ice, snow and frozen lumps before entering the mixer.

- **Aggregate Cooling Temperature**

Aggregate temperature at the mixing plant shall be atleast 3°C less than the specified for placement of RCC.

3.3 Leakage / Seepage control through the Dam

To prevent leakage, the dam will have an upstream facing of reinforced concrete raised concurrently with the dam blocks. The concrete reinforcement in the face will limit the width of any cracks that may develop. At the block joints, the reinforced concrete facing will have vertical joints with double seals and a drainage control system.

In order to prevent seepage and associated uplift in case of the Diamer Basha Dam, an integral concept including three (3) lines of defence against seepage will be applied, which are as follows:

- Upstream Impervious Facing (First Line of Defence)
- Drainage System (Second Line of Defence)
- RCC Dam Body (Third Line of Defence)

3.4 Design Parameters

The structural design of the dam was based on the selection of the most suitable cross-section which would obviate the tensile stresses and initiation of crack on upstream. The selection included study of RCC mixes, properties of the RCC and its placement schedules. The stability of the dam has been checked with standard design load cases including usual, unusual and extreme conditions, inclusive of all static and dynamic situations. The parameters for the dam foundation as well as dam material have been obtained from extensive geotechnical investigations comprising of in-situ as well as laboratory tests. Earthquakes peak horizontal accelerations and response spectra have been developed by experts on seismology and selected design values of horizontal acceleration are as follows:-

- i. Operating Basis Earthquake (OBE) = 0.22 g
- ii. Maximum Design Earthquake (MDE) = 0.37 g
- iii. Maximum Credible Earthquake (MCE) = 0.46 g

Vertical accelerations have been assumed at two-third of respective horizontal values.

3.5 Load Cases

The following load cases have been used for stability analysis of the dam: -

i. Usual

U 1 – Normal minimum reservoir level, i.e. 1060 m asl

U 2 – Normal maximum reservoir level, i.e. 1160 m asl

ii. Unusual

Unusual Static – Safety check flood, i.e. 1167.7 m asl

Unusual Dynamic – Normal maximum reservoir level + OBE

iii. Extreme

E 2 – Normal maximum reservoir level + MDE

E 3 – Normal maximum reservoir level + MCE

PS 1 – Post MCE static

PD 1 – Post MCE dynamic

3.6 Stability Analyses

The following stability analyses have been carried out for the specified load cases.

- i. The stability of the dam has been checked for sliding.
 - a. Within planes of weakness in the foundation
 - b. At the interface of dam and rock
 - c. At horizontal joints within the dam body.

- ii. Appropriate uplift pressures have been taken into account
- iii. In addition, various planes in the dam body have been checked for stresses under specified loading conditions.
- iv. Stress development by thermal effects has also been considered.
- v. Slope stability has been checked by wedge method for the dam abutments.
- vi. Dam-spillway interface stability has been checked.

The stability analyses have been carried out by computer model EFESYS.

3.7 Factors of Safety

In view of a reasonable certainty of strength values obtained for the project through detailed investigations, the following minimum safety factors have been adopted for stability within the dam body: -

- Usual static loading 2.0
- Unusual static loading 1.5
- OBE seismic loading 1.1
- MDE & MCE loading sliding displacement limited to 10 cm

3.8 Thermal Studies

The methodology uses both a one-dimensional (construction-line) analysis method for simulating the precise placement and thermal conditions as the block is under construction, as well as a coupled analysis using two dimensional finites in order to obtain more accurate mass-gradient cracking potential, i.e. to estimate more accurately the onset of cracking. Both mass-gradient and surface cracking data are obtained by this method. Surface cracking is analysed for both the upstream/downstream and the cross-valley (possible exposed block faces during construction), i.e. by considering the construction sequences also for the two adjacent blocks. The analysis of near-surface thermal gradients and interface conditions between the adjacent blocks (times of lift joining, thermal conditions before lift joining, etc.) as well as for upstream facing concrete is also included in the analysis, thus giving a three-dimensional picture of block thermal and crack potential issues. Age dependant material properties are tracked as the analysis proceeds, and creep effects are included. Material data is also passed from the construction-line analysis to the finite element pre- and post-processing routines for accurate analysis and interpretation of the dam safety, including extensometers, etc

3.9 Instrumentations

The dam contains a large number of instrumentation which is designed to suit the principle purposes of measuring and monitoring temperatures, stresses, deformations, hydraulic phenomena and seismic accelerations. Following instruments would be used

- a) Distributed temperature measurements by fibre optics (DFOT) and additional temperature transducers to monitor the temperature performance in the RCC.
- b) Three-dimensional displacement vector probes (TRIVEC) to measure horizontal and vertical deformations in the dam foundation rock
- c) The joint metering at the plinth joint
- d) Crack detection measurement (extensometer bars)
- e) Leakage detection (DFOT) between waterstops of the block joints
- f) Pore water pressure in lift joints of the RCC

From the 3 strong motion accelerographs to be installed, one at the dam and two will be located in the grouting galleries in the rock abutments. These will be installed at the time when the galleries are excavated and the dam has reached a level which covers these galleries. The installation shall take place before completion of the dam and first reservoir filling, to be able to measure seismic events which may be triggered from reservoir filling. The third equipment will be installed directly on the dam, in the uppermost drainage gallery, when the dam is completed.

4. SPILLWAY

4.1 Spillway Design

The design of the spillway is based on a gated overflow ogee-shaped crest arranged in the central part of the RCC dam with a chute and flip bucket at the downstream face of the dam. The spillway structure is readily aligned in plan in accordance with the curvature of the dam. Outflows are controlled by radial crest gates.

The spillway, with a discharge capacity of 18,128 m³/s at FSL, is placed approximately in the centre of the dam. Fourteen (14) steel radial gates, each 16.25m (H) x 11.5m (W), are arranged in pairs on seven of the central dam blocks. Floods will be discharged via a chute and flip bucket constructed on the downstream dam face and it is expected that a plunge pool will form in the downstream river bed. The unit discharge released over the spillway crest is 120 m³/s per meter crest length for the basic design flood but may reach up to 218 m³/s per meter, during passage of the safety check flood from a Glacial Lake Outburst Flood (GLOF) or catastrophic moraine dam break occurring somewhere on the river network upstream of the reservoir. Following are the spillway features:

Dam crest level	1170.0 m asl
Full supply level	1160.0 masl
Spillway crest level	1145.5 m asl
Number of bays/gates	14
Width of bay at crest	11.5 m each

Design head for crest	17.5 m
Slope of chute	62° with horizontal
Radius of flip bucket	40 m
Lip level of flip bucket	1030.0 m asl

5. LOW LEVEL OUTLETS (LLOs)

Seven steel lined low level outlets will pass through the lower section of each of the seven spillway blocks. Two of these low level outlets – those at the two outer ends – will have steel lined diameters of 7.2m and will release water, in excess of that released through the two power houses, during the season of high irrigation demand. The five (5) remaining outlets will have steel lined diameters of 9.0 m and are designed to flush sediments from the reservoir at reservoir water levels below minimum operating level (MOL). These larger low level outlets will also serve as standby for the two smaller, but more heavily used, outlets. Under normal operation, all of the steel liners will be subject to an internal water pressure corresponding to the reservoir level.

6. FLOOD ROUTING STUDIES

Adequacy of the spillway as a specific device in case of high flows has been verified by flood routing of Basic Design Floods as well as Safety Check Floods. The flood routing has been performed with various alternative scenarios and results are given hereunder in Table 1:-

Table 1: Summary of Flood Routings Through Diamer Basha Dam Reservoir

Flood Event	Peak Inflow	Starting Reservoir Level	14 Spillway Gates (11.5 × 17.5 m) Operative		One Spillway Gate Inoperative		14 Spillway Gates along with 5 RFO's & 2 LLO's Operative	
			Peak Outflow	Maximum Reservoir Level	Peak Outflow	Maximum Reservoir Level	Peak Outflow	Maximum Reservoir Level
			(m ³ /s)	(m asl)	(m ³ /s)	(m asl)	(m ³ /s)	(m asl)
Basic Design Flood-1 (1 in 10,000 years)	20,170	1160.00	18,859.56	1160.36 970.73 *	18,412.19	1160.85 970.40 *	-	-
Basic Design Flood-2 (SHYOK GLOF)	23,710	1160.00	19,957.62	1160.90 971.43 *	19,144.59	1161.23 970.92 *	-	-
Safety Check Flood-1 (SHYOK GLOF + 1 in 100 years)	32,690	1160.00	25,293.60	1163.37 974.60 *	24,640.79	1163.94 974.20 *	29590.8	1160.55 976.90 *
Safety Check Flood-3 (BIAFO GLOF / PMF)	49,410	1160.00	35,689.61	1167.71 979.80 *	34,920.64	1168.49 979.50 *	37710.4	1164.23 980.80 *

* Tailwater Level

It can be seen from this Table that the dam is not overtopped even in extreme case of SCF-3 (PMF).

7. POWER HOUSES

Two power houses one on left bank and the other on right bank are structurally independent from the dam. Both are developed with power waterways as short as possible within the restrictions of geological features of the site, e.g. of faults, rock cover on the structures, main joint system and estimated rock surface below ground level. Due to the different features encountered on each bank, there are some differences in the design of the schemes, the major of these differences constituting the length of the head race tunnels.

Each power house involves 2 concrete lined headrace tunnels of 15.30 m diameter with intake structures well below minimum operating level of 1060 m asl – each tunnel feeding 3 generating units -, 2 surge shafts of 38 m and 45 m diameter, and 6 single steel lined unit pressure shafts leading to the power houses. There are two underground power house and transformer & switchgear caverns. For the underground power generation facilities large size openings such as power house caverns (length-227 / width-30 / height-57.8 m) and transformer caverns (198 / 21 / 35.75 m) will be required. The cables would lead from the transformer cavern through a shaft to switchyard . The tailrace tunnel system combining the outflow of each 3 units in one tunnel, with outlet structures at the river. There are extensive Adit systems for access, ventilation and drainage. The drainage system from the dam dewater into sumps in the left bank powerhouse.

The operating range of the power houses lies between the maximum and minimum reservoir operating levels of 1160 m asl and 1060 m asl respectively. The invert level of power intakes has been determined as 1030 m asl on consideration of required minimum submergence below the minimum operating level of 1060 m asl. The rated discharge for each unit has been determined as 246.8 m³/s with a rated head of 170.5 m corresponding to reservoir level of 1130 m asl. Hydraulic head losses have been determined for each waterway profile, from intake to outlet, including friction losses and local form losses.

Downstream of the powerhouses, 3 single draft tube manifold tunnels join one tailrace tunnel with dimensions 18.8 x 18.8 m (D-Section) to the tailrace outlets.

7.1 Power Intakes

The power intakes are of bell-mouth shape section and are equipped with a fixed course rack at the inlet section. The inclination of the intake section with 80° against the horizontal will prevent clogging, e.g. trash if any will sink down below the tunnel invert. For that purpose, the base platforms in front of the intakes are set some 15 m below the tunnel invert which is at 1030.0 m asl. It is expected that the reservoir level will be drawn down to MOL=1060.0 m asl almost every year towards the end of the dry season. Then, the platform on top of the intakes at 1085.0 m asl will be accessible, and trash if any accumulated in front below the intake racks can be removed by mobile equipment.

The headrace tunnels between their intakes and the ends of their manifolds (with the start of the steel liners in the gate chamber) are lined by reinforced

concrete. Lining is considered continuous (without permanent joints) and crack width controlled by the reinforcement. Seepage from the tunnels will hence be avoided. Within the range between the power intakes and the grout curtain underneath the dam, internal and external water pressure acting on the lining will be balanced, with the exception that for inspection / maintenance a tunnel may be emptied. Hence, tightness of the tunnel lining will not be critical in this range, and lining is carried out for hydraulic reasons and to avoid any rockfall as could occur without lining. Downstream of the grout curtain extending from the dam foundation, lining shall achieve water tightness against the internal pressure. Seepage from the dam is unlikely to load the exterior of the tunnels.

7.2 Sediment Flushing Tunnels

At Diamer Basha Dam one flushing tunnel each shall be provided in the vicinity of power intakes on the left and right banks of the river. These tunnels have been proposed for the following objectives.

- (i) At the advanced stage of construction of project when the diversion canal and diversion tunnels have been closed, the flushing tunnels may be temporarily utilized to supplement the capacity of available outlets for disposal of river flows.
- (ii) The main purpose of these flushing tunnels shall be to flush out the sediments from the power intake areas on both banks so that there is no accumulation which may enter the power waterways and damage the turbines.
- (iii) The Power Intake Flushing Tunnels will be operated under a range of reservoir levels from 1060.00 to 1160.00 m asl.
- (iv) The minimum velocity through the Power Intake Flushing Tunnels should be equal to or more than 4 m/s, so that the sediments are not allowed to settle, while the settled sediments are eroded.
- (v) The maximum velocity through the tunnels should not exceed 10 m/s, so that the concrete and unlined surfaces will not be damaged by abrasion and cavitation.

8. GEOLOGY AND GEOTECHNICAL INVESTIGATIONS

The proposed project site is located in the northern mountainous area of the Kohistan region. The Kohistan terrain was formed in the course of the tectonic collision of the Indian and the Eurasian plates, accumulating a 40 km thick sequence of mafic, ultramafic, plutonic, volcanic, metamorphic and sedimentary rocks. This Kohistan Island Arc is bounded by the Main Karakoram Thrust (MKT) in the north and west and by the Main Mantle Thrust (MMT) to the south and east.

Diamer Basha dam site is underlain by bedrock of the Chilas Complex, comprising gabbro-norite and ultramafic rock intersected by doleritic dikes and pegmatite veins. Overburden consists of river and nullah deposits in form of

terraces and alluvial fans, locally extensive moraine sediments, and slope debris from rock toppling and sliding.

The area of the project was mapped by MONENCO in 1982, field geologist of WAPDA (2001 – 2002), NEAC (2004) and DBC (2005-07). Resultantly, the following types of rock masses are distinguished.

The rock outcrops cover about 26% of reservoir area and mainly comprise norite (a basic igneous rock belonging to the gabbro clan) with subordinate outcrops of hornblendite, amphibolite, diorite and gneiss along with minor intrusion and veins of ultra basic mafic and ultramafic igneous rocks.

Figure 5 presents the Geological map of the site.

8.1 Drilling and Geological Investigation

The geological investigations conducted by MONENCO in 1984 included geological mapping of the dam site and the reservoir, drilling and Water Pressure Tests (WPT) at the dam site and borrow areas. Mapping was done on a 1:2000 scale for the dam site. The topography was derived from plane table survey. The resulting geological map was very basic. Monenco investigated sites in a river length of about 10 km and finally selected site C. Figure 6 gives the location of alternate sites studied. A total of 63 boreholes were drilled with a length of 4451 m. Out of that, 37 bores with 3753 m length were for assessment of the dam site geology. In 9 of those holes a total of 94 WPT had been carried out in bedrock. 19 Standard Penetration Tests (SPT) in overburden had been carried out in 5 boreholes accompanied by 24 falling head permeability tests in river alluvium.

In 2001 – 2002, wapda used its own resources and expertise to carry out further investigations. These included new topographic surveys, geological mapping and additional drilling. The topographic surveys had been done manually with plane table equipment and produced 110 topographic sheets covering the dam site and appurtenant structures on 1:500 scale with one meter contours up to an elevation of 1300 m asl. Fifteen boreholes were drilled along the C2-axis, at location of downstream/ upstream cofferdams and along the waterways and portals. The total amount of drilling was 1073 m. 15 boreholes were drilled in the active river bed from pontoons. It was found that depth of alluvium at the selected site was of the order of 50 m and it comprised of boulders, gravels, sand and silt.

The investigations and studies carried out by NEAC Consultants in 2002 – 2004 consisted of upgrading of the geological maps for the regional geology, the dam site and the reservoir area, rock discontinuity (joint) surveys, additional drilling and geotechnical testing in boreholes, first at dam site C and then later at dam site D, geophysical survey at the dam site and in the reservoir area, seismic hazard studies, studies of reservoir slopes in glacial deposits, evaluation of liquefaction potential of alluvial deposits, assessment of safe rock slopes, and laboratory testing. NEAC also initiated a programme for the excavation of two adits, one on each flank at dam site D as well as a programme of in-situ rock testing in the adits. Drilling at the dam sites C and D consisted of a total of 1830 m in 23

boreholes, 5 of which were drilled in the river. Three of these were drilled at dam site D and two midway between dam sites C and D. Location of alternate dam sites is shown in Figure-6.

The drilling program accomplished by DBC Consultants in 2005 – 2007 during tender design phase comprised of 65 boreholes aggregating to 9918 m. This included 21 holes for the dam; 35 holes for left / right bank powerhouses; 7 for diversion works; and 2 for installation of inclinators for monitoring of movement of steep abutment slopes.

Most of these bore-holes were logged with bore-hole scanner system ETIBS. The data recorded in the bore-holes using this system have been taken into account for stability analysis.

The detail of the drilling and investigation works is given in Table – 2.

Table 2: Details of Geotechnical Investigations Conducted at Site

I. Drilling

Sr. No.	Organization	No. of Boreholes	Total Depth (m)	Period
1	MONENCO	63	4451	1981-84
2	WAPDA	15	1073	2001-02
3	NEAC	23	1830	2002-04
4	DBC	65	9918	2006-08
	TOTAL	166	17272	

II. Aditing

Adit No.	Organization	Location	Length (m)	Remarks
1	NEAC	Right Bank (No. 2)	216	At the dam Axis D proposed in NEAC's Feasibility Report (2004)
2	NEAC	Left Bank (No. 1)	400	
3	DBC	Left Bank (No. 3)	256	Including Main (129.7 m) and side heading (126.1 m) at axis C2 (2006).
4	DBC	Left Bank (No. 4)	532	Including Main (422 m) and side heading / branch of 110 m at axis C2 d/s (2007).
5	DBC	Right Bank (No. 5)	650	Including Main (450 m) and two perpendicular branches of 100 m each at axis C2 d/s (2007).
		TOTAL	2054	

8.2 Exploratory Adits

Five exploration adits were excavated and geotechnically mapped. Adit one and two were carried out in 2004 at Axis-A during feasibility studies. Sheared ultramorphic rocks in Adit-2 in left abutment were the deciding factor to revert back to site – C. Three adits (No. 3, 4 and 5) aggregating to 1438 m were excavated for the tender design during 2006 / 2007. The results from adits 4 and 5 were considered decisive for the planning of tunnels and power caverns. It was recommended by Panel of Experts (POE) to carryout in-situ rock mechanics tests in these adits through over-coring down to draft tube level of planned powerhouses as well as plate load flat jack tests at the crown levels. Rock over coring tests have been carried out.

8.3 Laboratory Testing

A large number of tests on rock samples were carried out to determine the unconfined compressive strength (UCS) and the deformability of the intact rock. Also point load strength index tests (PLSIT) and shear box tests were conducted. Test results are given in the following Table 3.

Table-3: Summary of Laboratory Results Vs Standard Values

Sr. No.	Description of Value	Unconfined Compression Strength (MPa)	Point Load Strength Index (IS 50) (MPa)	Young's Modulus of Deformation (GPa)	Poisson Ratio
1.	Standard Average	140	7.38	59	0.148
2.	Actual				
	2.1 Average	97	7.39	77.2	0.206
	2.2 Standard Deviation	40	3.17	62.3	0.213
	2.3 Minimum	14	1.29	3.7	0.015
	2.4 Maximum	203	14.00	340	0.952

8.4 Rock Mechanics Parameters

According to borehole data and the results of geotechnical mappings, it has been assumed that approx. 70 % of the rock formation along the waterways (intake, headrace tunnel up to outlet, tailrace) can be allocated class 2, about 25 % to class 3 and some 5 % to class 4.

Geomechanics classification system of Bieniawski frequently referred to as Rock Mass rating (RMR), for various rock classes are: -

- Class 1 or “very good”: RMR = 81 – 100
- Class 2 or “good”: RMR = 61 – 80
- Class 3 or “fair”: RMR = 41 – 60
- Class 4 or “poor”: RMR = 21 – 40
- Class 5 or “very poor”: RMR \leq 20

9. SEISMIC HAZARD EVALUATION

A deterministic and stochastic seismic hazard analysis has been carried out for the proposed dam site in accordance with the guidelines of ICOLD and international engineering practices in order to evaluate the possible impact of seismic events on the planned structures and to make allowance in the related designs. From these studies, values of horizontal peak ground accelerations (PGA) have been derived as design criteria: -

Operating Basis Earthquake (OBE)	=	0.22 g
Maximum Design Earthquake (MDE)	=	0.37 g
Maximum Credible Earthquake (MCE)	=	0.46 g

For design purposes, vertical acceleration components have been assumed as two-third of the respective horizontal PGA values.

9.1 Micro Seismic Monitoring System

A micro seismic monitoring program was initiated as a part of investigations for Seismic Hazard Evaluation of Diemer Basha Dam Project.

10 micro seismic stations have been installed all around the dam and its reservoir area, upstream and downstream of the dam site. These proposed locations were surveyed for their access and ambient noise level. Final selection was carried out on the bases of noise survey results and accessibility.

The station containing equipment consists of a fibre glass, thermally insulated, water proof vault and a protective steel fence around the vault. This vault is constructed in two parts, lower part is constructed by reinforced concrete and upper part is of fabricated fibre glass sheets.

The micro seismic instruments consist of following parts

1. Micro seismic sensor LE-3Dlite
2. Quanterra Q330
3. Quanterra PB14F Baler
4. Battery

10. HYDRAULIC MODEL STUDIES

Physical hydraulic model of Diamer Basha Dam Project was started at IRI, Nandipur in April 2007 and concluded in October 2008. Following two models were prepared and tested:

- 1– Comprehensive model of the project
- 2– Sectional model of spillway

10.1 Comprehensive Model (1:80 Scale)

Objective of 1:80 scale comprehensive model was to confirm the hydraulic adequacy of the designed structures and to ensure smooth functioning of various diversion works and safe passage of the river discharge during different stages of the river diversion. Measurements / observations on the model included discharge capacities of the diversion works, water levels and flow velocities. The model simulation and testing was based on Froude's Law. The functioning of flushing tunnels under the intakes was also checked.

The (1:80 scale) comprehensive model covered Indus River reach of 7.5 km with dam located at site C-d/s having an upstream length of 5 km. Besides the main dam, it incorporated the major project components as listed below and shown in Figure 7.

- Diversion Canal
- Diversion Tunnels
- Upstream and Downstream Cofferdams
- Spillway with 14 Bays and Gates
- Five Reservoir Flushing Outlets
- Two Low Level Outlets
- Left Bank Power Waterways including Surge Tank
- Right Bank Power Waterways including Surge Tank
- Left Bank Power Intake Flushing Tunnel
- Right Bank Power Intake Flushing Tunnel

10.2 Sectional Model of Spillway (1:50 scale)

Sectional model of spillway comprised of four left bays of the spillway with ogee crest; piers; radial gates; chute; aerator; flip bucket; and plunge pool. Inflow source for this sectional model comprised a large tank located just upstream of the spillway. One of the basic objectives of this sectional model was to understand hydraulic behavior of the spillway with greater accuracy. This sectional model was also helpful in determining numerous hydraulic parameters of spillway including discharge rating, depth and velocity profiles and pressure fluctuations along the profile with better precision due to its larger scale than the comprehensive model.

10.3 Scope of Hydraulic Model Studies

Physical model studies were divided and carried out according to the following four stages:

- Stage – I Diversion works / arrangements: Testing and documentation including related modification of the existing 1:80 scale comprehensive model.
- Stage – II (a) Adjustment / reconstruction of comprehensive model (1:80 scale) incorporating the tender design features of: dam; spillway; reservoir flushing outlets; low level outlets; power waterways including surge tanks; and power intake flushing tunnels
- II (b) Construction of sectional model of spillway (1:50 Scale)
- Stage – III (a) Hydraulic testing on comprehensive and sectional models.
- III (b) Sedimentation simulation studies on comprehensive model
- Stage – IV Documentation, interpretation and reporting of model testing carried out under Stages III (a) and III- (b) above.

11. ROCK CUT SLOPE STABILITY ANALYSES

Stability analyses for the open-cut rock slopes in the area of the intake and outlet structures of the waterways and for the construction pit of the dam have been dealt with. The stability analyses are carried out for planar sliding surfaces which are formed by one or two discontinuities.

The stability analyses for potential, kinematically unstable sliding wedges at the open-cut rock slopes were carried out with the aid of the program code GGU-STABILITY (version 8) which is based on the block sliding method. This code was developed by GGU and has been successfully applied in the past for a large number of rock engineering problems.

11.1 Diversion Tunnel and Headrace Tunnel Intakes (Right Bank)

The open-cut rock slopes are located in gabbro-norite. These slopes are designed in steps of normally 15 m height with 5 m wide berms. The steps are sloping at 1H : 5V. The stability of slopes have been checked with block sliding method and found to be stable, with adequate safety factors for static and dynamic loadings.

11.2 Tailrace Tunnel Outlets and Right Bank Diversion Tunnel Outlets

The rock cut slopes on left and right banks power outlets are located in gabbro-norite, with heights of 106 m and 150 m respectively. For right bank diversion tunnel outlet, the open rock cuts have a height from 85 m to 91 m. The designed slopes will be 1H : 5V, stepped with at least 5 m wide berms. These slopes are stable with OBE and MDE earthquakes. Conservatively, the angle of friction on discontinuities is taken as $\Phi = 35^\circ$, and cohesion as $c = 1000$ kPa. The selected slopes are safe with ample safety margins.

11.3 Excavation Pit for Dam Foundation

Stability analyses were carried out for five cross-sections of the excavation pit, assumed to be located in gabbro-norite. The maximum slope height is about 166 m. The typical design of cuts comprise 15 m high steps (at 1H : 5V slope) and 5 m wide berms.

The left and right abutment rock slopes will be about 240 m high on both sides, with angles of approximately 40 degrees. The stability analyses for all the slopes for static and dynamic load cases prove the designed configuration to be stable.

Further recommendations for protective measures can be determined when rock slopes are exposed during construction. For protection against rock fall and local block mass wedges, safety measures like wire netting and rock bolts should be foreseen. During construction, excavation works have to be accompanied by intensive geotechnical mapping. Also in the open-cut rock slopes a monitoring program for measuring the displacements due to excavation have to be set up,

12. RIVER DIVERSION WORKS

According to the selected tender design the diversion works consists of the following components

- | | | | |
|--------------------------|--------------------------|----------|----------------|
| a.) Diversion Tunnels | 2 | Lengths: | 887 m & 1016 m |
| b.) Diversion Canal | 1 | Length: | 1092 m |
| c.) Upstream Cofferdam | crest level – 977.5 masl | | Rockfill |
| d.) Downstream Cofferdam | crest level – 973.0 masl | | Rockfill |

The dimensions of the diversion tunnels are selected to satisfy the requirements of discharging safely the dry season design flood. The design is governed further by the objective to convert one of the diversion tunnels to a flushing tunnel which will maintain the right bank power intakes free of sediments during advanced levels of reservoir sedimentation.

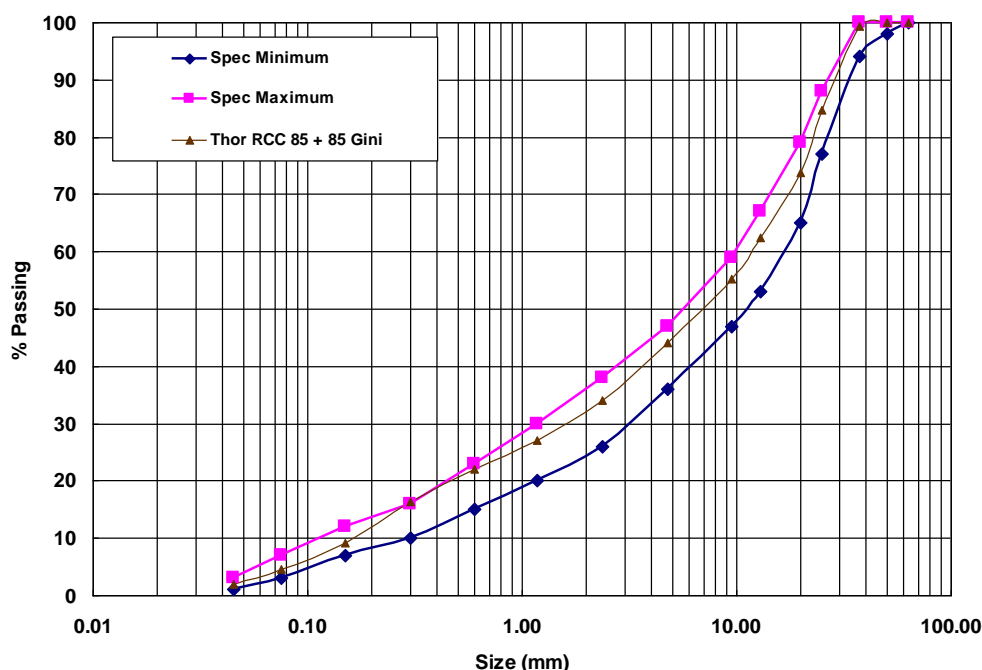
The height of upstream and downstream cofferdam is selected based on the maximum water levels resulting from discharging the diversion design flood through the diversion canal as well as the dry season diversion design flood through the diversion tunnels. With progress in dam construction the diversion canal will be closed and sluicing outlets as well as low level outlets will serve for river diversion in addition to the diversion tunnels.

Under such condition the main dam will act as upstream cofferdam. The concept of operating the combined flushing and low level outlets for river diversion results in high flow velocities at the diversion outlet works. Therefore, the area downstream of the outlet works and the diversion canal will be protected by concrete lining as required to cope with the hydraulic impact of high velocity currents and impinging jets downstream of the dam etc.

13. RCC MIX DESIGN

To start with the RCC Mix Design programme, selection of irrefutable sources of ingredients for mix design was done including aggregate site having sufficient area for the substantial processing equipment with huge stock piles and no risk of delay or interference with other aspects of the project. The selected materials were then processed by series of different types of crushers to get the particle size as per design envelope of the gradation given below and to keep the particle shape within the limits. This processed material was sent to central Material testing Laboratory of Wapda for preparation of trial mixes and its subsequent testing.

Gradation Envelope for aggregates



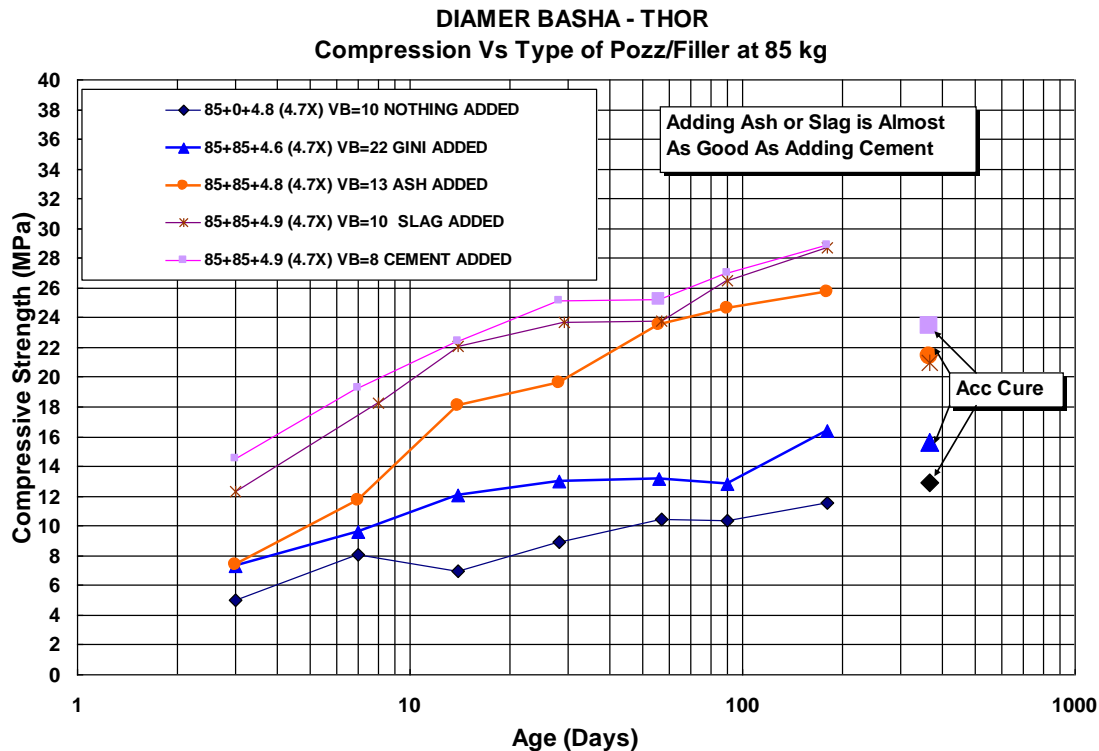
The work on Roller Compacted Concrete Mix Design was done for the first time in Pakistan. Therefore, considerable up-gradation of CMTL was carried out. New equipment was inducted and necessary infrastructure was built to meet the requirements.

In all 36 number regular RCC mixes were prepared using one brand of cement (Maple Leaf), Two types of aggregates, one from rock quarry (Norite) and the other from river gravels (Thor), and three types of added materials (ash, slag and natural pozzolan Gini). In addition to this 18 Nos. of Lean RCC mixes were also prepared and tested accordingly. All the mixes were tested for 8 ages (3,7,14,28 56,90,180 and 360 days) along with observing stress-strain relation and calculating the shear modulus. The results obtained were excellent and close to the prediction / requirement, depicting that the selection of the materials was satisfactory.

Summary of the results showing comparison of predicted and actual values are given in Table-4.

Table No 4 - Result Summary

Material	Mix (Cement + added material)	Strength (Mpa)	Age
Cement	85 + 00	15	1 Year
Cement	85 + 00	12	180 days
Cement+ Ash	95 + 95	26	1 year
Cement+ Ash	85 + 85	26	180 days
Cement+ Gini	85 + 85	16	180 days
Cement + Slag	85 + 85	40	180 days



The results of Mix design were very conclusive as followings:

- Norite and Thor aggregates can be used and mixed in any proportion
- Thor aggregate has technical advantage being slightly heavier and lower modulus
- Thor site being down hill gives energy and fuel saving. It will be independent of dam excavation and schedule. Will have no contamination and can be cooled on long down hill conveyer to Dam

- Thor has cost advantage as it does not need to crush sand, 18 million ton screening vs crushing will save on the order of US\$300,000,000
- Natural pozzolan Gini works but not as good as expected
- Ash works as good as cement at 35% to %50
- Slag works as good or better than cement. At greater than 50% Strengths are better than cement, Ash and Slag mixes
- Tensile strengths are excellent, 14% of compressive strength.
- Modulus is stiffer than desired

14. ENGINEERING CHALLENGES

There are a number of features of the project which constitute engineering challenges and require extraordinary technical and scientific approach e.g.:

- The RCC dam of unprecedented height and volume, which requires a unique analytical approach to development of hydration heat and stresses in the dam under static and seismic load condition. The cooling period of the dam will be of the order of 20 years. The analytical approach to stresses, deformations, and crack limitation under severe seismic conditions was necessary to demonstrate the technical viability of the dam.
- The location of the high RCC gravity dam in a seismic active area in which high earthquake shaking could be generated.
- The high RCC placement rates which have to continue during the hot seasons and thereby involve the risk of heating up the RCC when placed, in addition to its own hydration heat. This requires extraordinary low cement contents of the RCC, extensive pre-cooling of aggregates, the utilization of retarders, the application of sloped placement of RCC and extensive surface protection and curing of the placed RCC.
- The innovative method of providing integrated reinforced concrete facing with the RCC on the upstream face of the dam. The facing is designed to be crack width controlled and placed in fresh to fresh conditions between RCC and conventional concrete.
- The adequate planning of all of these above measures has an overruling effect on stress development in the dam body. Zonation of RCC strength in the dam body aims to avoid the development of mass gradient cracks and failure of lift joints during seismic shaking.
- The river diversion during construction including the preparations up to placement of the cofferdams for dam construction and the period of flood handling thereafter requires a series of unconventional approaches. The design floods during construction are of such magnitude that they can only be handled by a combination of diversion

tunnels, a large open rock canal through the dam, breaches in the dam, and a number of low level outlets. The pattern of utilization of these different discharge evacuation facilities varies from year to year and from season to season and needs to be linked to the RCC placement schedule, which finally effects the heat and stress development in the dam.

- The analytical approach for controlling heat development, stresses and avoidance of mass gradient cracking, dependent on the construction progress and limitations by river diversion requirements, forms a sound basis for continuous RCC placement control during project execution.
- Underground Works for the two power stations are of extraordinary size, and their location and orientation had to be selected with careful consideration of rock mass jointing and the larger discontinuities as induced by the seismo-tectonic environment in the area. The distribution of probably significant tectonic stresses in the project area – especially in the area of the large underground power houses have been explored by in-situ testing programme.

REFERENCES

- i. American Concrete Institute Committee 207 (1999): Roller-Compacted Mass Concrete. Report ACI 207.5R-99.
- ii. Diامر Basha Consultants, Detailed Engineering Design Reports (2008). (Fourteen Volume)
- iii. Forbes, B. A.: Using Sloped Layers to Improve RCC Dam Construction, HRW, July 2003.
- iv. MONENCO Feasibility Study Reports (1984)
- v. NEAC Consultants Feasibility Reports (2004)
- vi. Selected Drawings of the Tender Documents
- vii. US Army Corps of Engineers: Engineer Manual EM 1110-2-2006 Roller-Compacted Concrete.
- viii. Wittke, W.: Stability analysis for tunnels – Fundamentals. Geotechnical Engineering in Research and Practice, WBI-PRINT 4, VGE, 2000
- ix. Diامر Basha Dam Project Panel of Experts Reports No. 1 – 6. (From 2003 to 2007)
- x. Statistical evaluation of rock laboratory data and standard values for geotechnical design; *Diامر Basha Consultants*; Geological Report for Tender Design – Part 2 “Geology & Engineering Geology

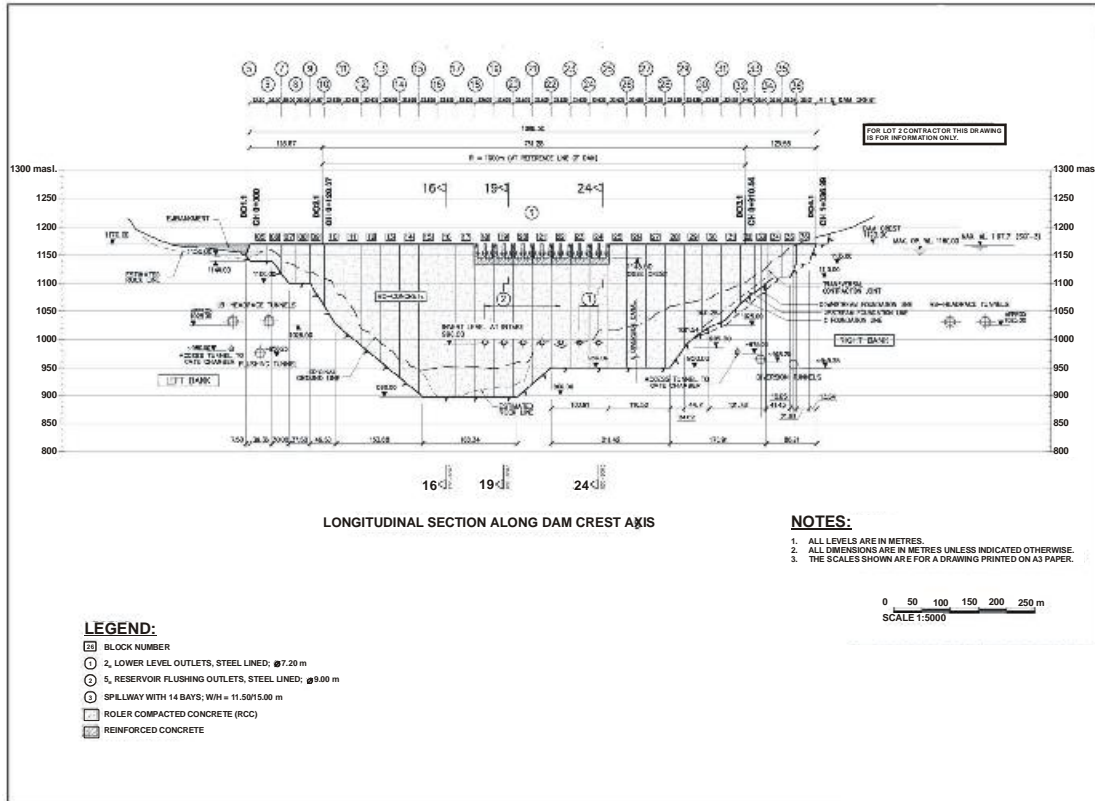


Figure 3: Longitudinal Section Along Dam Crest Axis

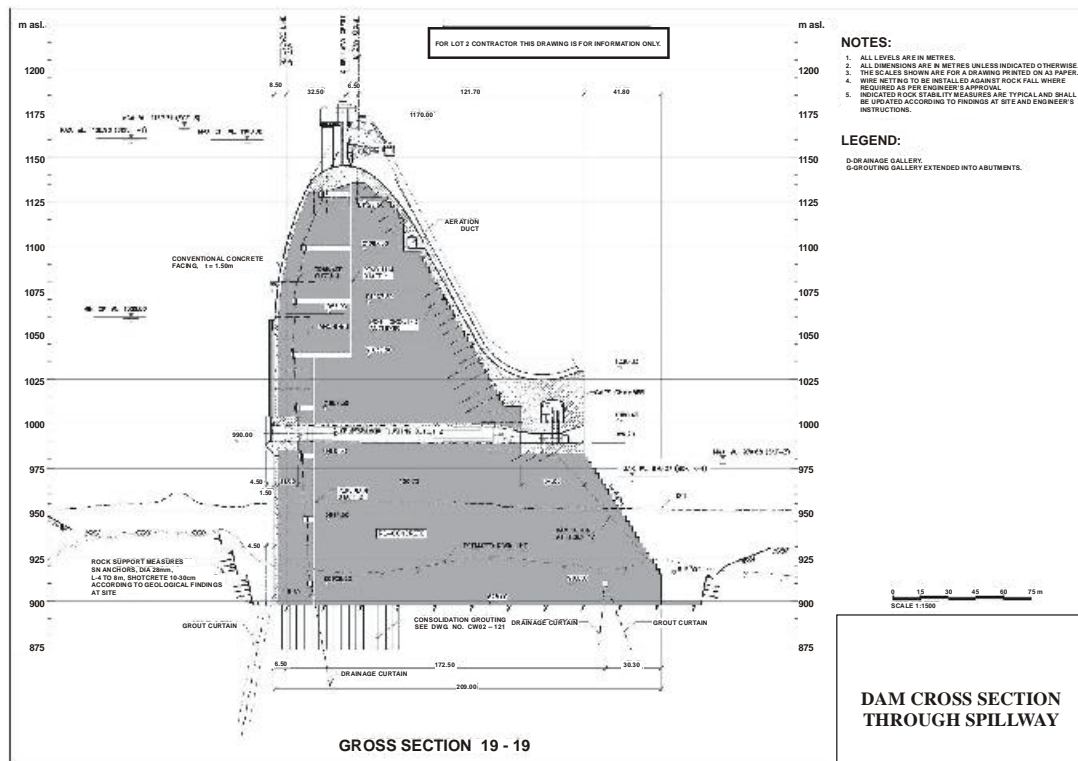


Figure 4: Dam Cross Section

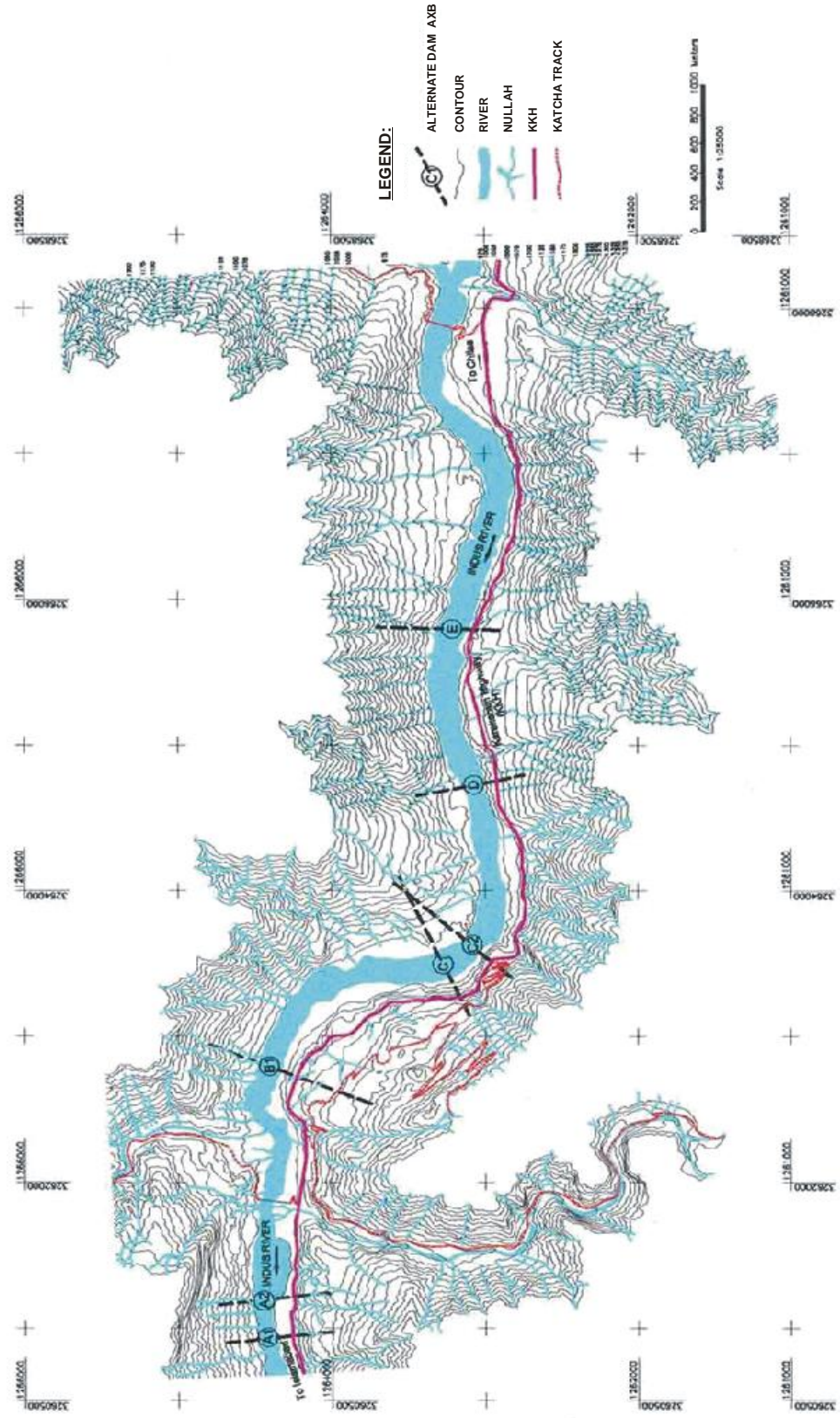


Figure 6: Location Map of Alternative Dam Axis

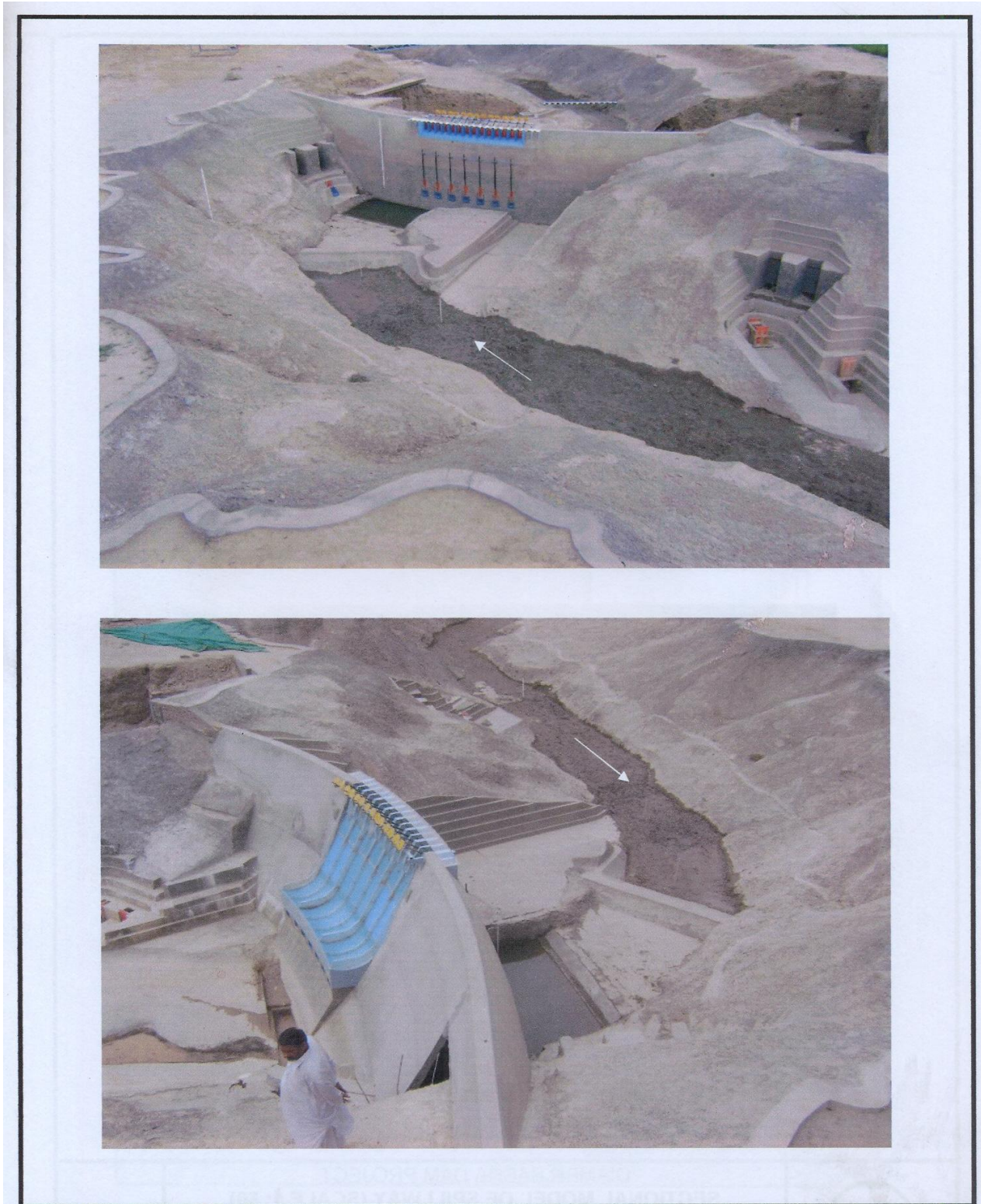


Figure 7: Comprehensive Model of the Project