APPENDIX B GEOLOGY

B1 Regional Geology

The contemplated Munda dam and reservoir on the Swat river is situated in a tectonic zone developed in the southern foothills of the Himalayan range where the Indian plate submerges northward under the Eurasian plate. The Swat river, a tributary to the Kabul river in the Indus basin, rises north in Kohistan zone of Himalaya and flows southward through a winding valley.

According to the Tectonic Map of Pakistan by Kazmi and Rana (1982), the Project area is located in Himalayan Crystalline Schuppen Zone between a couple of east-west trending major thrust faults, that is, Main Mantle Thrust (MMT) on the north and Main Boundary Thrust (MBT) on the south (Ref. Figure B1.1). The bedrock is largely composed of crystalline schists and the terrain is repeatedly cut by north-dipping thrusts to form a shingle block structure.

The thrust faults of a simple east-west trend, however, turn northward in the east to form syntaxis or sharp curves convex to north near Muzaffarabad and east of Gilgit. These northward protrusions of the land south of the thrusts are interpreted to show local outstanding intrusion of the Indian plate that resulted in applying strong stress to both northwest and northeast. (Ref.: A. H. Kazmi and M. Qasim Jan, 1997, Geology and Tectonics of Pakistan, Graphic Publishers, Karachi, Pakistan)

Bedrocks between those two thrust faults are considerably distorted and disturbed, with bedding planes generally striking north to south but with many and varied local deviations. Folding and fracturing of diverse sizes are common as indicated by the frequent and irregular changes in strike and dip of strata, as ranging from N70°E/80°SE at Malakand Pass to N60°W/40°NE at the Munda dam site. The tectonic movement is reflected also in development of Mesozoic melange zones, a mixture of volcanic rocks, ultrabasic rocks and other oceanic sediments.

The prevailing rock type is crystalline schist of Palaeozoic to Mesozoic, that includes graphite schist, chlorite schist, mica schist with garnet. Calcareous schist and crystalline limestone are also important local members. The schists are intruded at places by basic rocks and granitic rocks, of which the latter is metamorphosed to gneiss in marginal zones. (Ref. Figure B4.1)

B2 Geological Investigation

B2.1 Previous Geological Investigation

In advance to the current Feasibility Study, a series of geological investigation was conducted for the Munda dam scheme by WAPDA in periods from 1964 to 1966 and from 1990 to 1991. A comprehensive geological and geotechnical study was carried out in this periods including geological mapping and the core drilling of more than 900 m in total length with borehole water pressure tests as listed below:

Geological Investigation Conducted from 1964 to1991

Borehole No.	Location	Angle	Depth
MV-1	Right bank, low level	30° from V	300 ft (91m)
MV-2	- do -	30° from V	200 ft (61m)
MR-3	Right bank, high level	20° from V	220 ft (67m)
MR-4	Right bank, middle height	Vertical	450 ft (137m)
MP-5	Right bank, downstream	Vertical	100 ft (30m)
ML-6	Left bank, middle height	Vertical	450 ft (137m)
MR-7	Right bank, high level	Vertical	520 ft (158m)
ML-8	Left bank, high level	Vertical	150 ft (46m)
ML-9	Left bank, downstream	Vertical	200 ft (61m)
MV-10	Left bank, on the terrace	Vertical	400 ft (122m)

The core drilling already included three long boreholes over 130 m of each depth that must satisfied the criteria for deep drilling to cope with the height of the contemplated dam.

B2.2 Geological Investigation for the Feasibility Study

The following geological investigations were performed in the years of 1998 and 1999 for this feasibility study:

a) Geological mapping of the dam site and the quarry sites at Sappare and Todobo Banda.

b) Preparation of the geological map of the reservoir area with the data from Geological Survey of Pakistan, Peshawar Office, and through geological reconnaissance.

c) Core drilling with water pressure test:

Dam – powerhouse area	20 locations,	1,300 m in total length
Quarry sites (by WAPDA)	7 locations,	450 m in total length

d) Exploratory adit:

Dam site	4 locations,	210 m in total length

e) Seismic refraction prospecting (by WAPDA)

Dam site	8 lines	6,000 m in total length
Quarry sites	3 lines	2,450 m in total length

f) Test pit (by WAPDA)

Sand and gravel borrow site	7 pits
Earth borrow site	8 pits

g) Laboratory test

g-1) Earth material (Soil mechanics test)	Samples
Particle size analysis by sieve and hydrometer (ASTM D422)	18
Liquid limit, plastic limit, plastic index (ASTM D431)	18
Specific gravity of soil (ASTM D854)	15
Natural water content of soil (ASTM D4959)	18
Proctor compaction test (ASTM D698)	15
Permeability test of compacted soil (USBR E-13)	5
Triaxial compression, UU (ASTM D2850)	5
Triaxial compression, CU with pore pressure observation (ASTM D4767)	5
Dispersive characteristics (ASTM D4221)	3
g-2) Sand and gravel (Concrete aggregate test)	Samples
Sieve analysis of aggregates (ASTM C136)	5
Specific gravity and water absorption of coarse aggregate (ASTM C128)	5
Specific gravity and water absorption of fine aggregate (ASTM C128)	5
Clay lumps and friable particles in aggregate (ASTM C142)	5
Soundness test by sodium sulfate (ASTM C88)	3
Abrasion test of coarse aggregate by Los Angeles machine (ASTM C535)	3
Chemical (alkali) reactivity test (ASTM C289)	5
g-3) Rock core specimen (Rock test)	Samples
Water absorption and bulk specific gravity test (ASTM C127)	25
Unconfined compression test of rock core specimen (ASTM D2938)	25
Triaxial compression test of rock core specimen (ASTM D2664)	5

Locations are as shown in Figures B2.1, B3.2, B5.1, B5.2-1 and B5.2-2.

Detailed quantities of the drilling, the aditting, the seismic prospecting and the pitting are shown in Tables B1 to B4.

B2.3 Method of Geological Investigation

The core drilling of the dam – powerhouse area was performed by High Technology Engineering Consultants, Islamabad, under the supervision of the JICA Study Team, using four hydraulic-driven rotary drilling machines of Acker, Kano and Russia. The maximum depth of the boreholes was 180 m. A 30 m long vertical hole was drilled at the river bed from a raft on the water.

The core drilling of the quarry sites were conducted by WAPDA by use of a Koken OP-1, the drill rig of the similar system to the above. Double core barrels were used for coring with diamond bits.

Drill logs are presented in the Data Book.

Rock grade classification was made mainly after the criteria of Central Research Institute of Electric Power Industry (CREPI), Japan. The criteria are as shown in Tables B8.1 through B8.3 (Ref. K. Kikuchi, K. Saito, K. Kusunoki (1982): Geotechnically Integrated Evaluation on the Stability, Q.53, R.4, Fourteenth International Congress on Large Dams).

Water pressure tests were carried out at 5 m stages in all boreholes in the dam site and the Sappare quarry site (See Water Pressure Test Data in the Data Book). The water pressure tests were for the most part conducted by means of the Lugeon test, except for four tests in holes M98-11 and M98-20 that were made by the open-end constant-head method. For water leakage to the ground surface, the Lugeon test was not performed within the depth of 2.0 m.

In the Lugeon test, water loss was recorded for 10 minutes under each of seven different pressures, rising from 0.1 MPa to 1.0 MPa and then lowering back to 0.1. The value of Lugeon unit that represents the seepage potential of the bedrock was calculated by use of the following formula:

$$k = (q \times 10^3 \times \ln(L/r)) / (2\pi LH \times 60) \qquad \text{when} \quad L > 10r$$

$$k = (q \times 10^3 \times \mathrm{Sin\,h}^{-1}(L/2r)) / (2\pi LH \times 60) \qquad \text{when} \quad 10r > L > r$$

$$Lu = q \times 10^6 / LH$$

Where

k:	Coefficient of permeability (cm/sec)	Lu:	Lugeon unit
L:	Length of test section (cm)	r:	Radius of hole (cm)

H: Water pressure in head (cm) $H = A + B + C - H$

A: Pumping pressure head (cm)

B: Static water head from the middle part of the test section up to the top of the hole. If the groundwater is higher than the middle part of the test section, this shall be the head from the water level to the top of the hole (cm).

C: Height of the water pressure gauge from the top of the hole (cm).

Hf: Friction loss of energy in the injection pipe (cm).

q : Average water injection rate (litre/min.)

In the open-end constant-head test for very soft rock or deposit, water was poured into a casing pipe installed to the bottom of the hole to maintain a constant water table in it. The pouring rate was recorded as representing the seepage rate from the open end at the bottom of the casing. The coefficient of permeability was calculated by the following formula:

$$K = Q/(60 \times 5.5rH)$$

Where.

k: Coefficient of permeability (cm/sec.)
r: Radius of casing (cm)
H: Water head from the bottom of the hole up to the constant water level.
If groundwater table is higher than the bottom of the hole, H is the height of the constant water level from the groundwater table. (cm)
Q: Rate of water supply (cm^3/min.)

The exploratory adit was excavated by High Technology Engineering Consultants, Islamabad, under the supervision of the JICA Study Team. Three air compressors were mobilized for this work. For solidity of the bedrock, the adit support was rarely required.

The seismic refraction prospecting was conducted by WAPDA, using the amplifier-oscillograph-recorder system of Oyo McSEIS-SX with 24 channels with geophones at 5 m intervals. The records of the seismic refraction prospecting was interpreted by WAPDA following the method of Generalised Reciprocal Method (GRM), and then reviewed through the Hagiwara Method by the JICA Study Team, taking into consideration the outcomes of the core drilling.

The test pits for earth borrow were dug by WAPDA to the depth of 5 m or till rock or groundwater was encountered within that depth. The test pits for sand and gravel deposits were dug to 1.5 or 2.0 m. The samples were taken in the pits and carried to the material testing laboratory for the tests as listed above. The tests were performed to meet the specifications according to ASTM as shown in the list.

B3 Site Geology

B3.1 Geology of Dam Site

The Munda dam site is located approximately 5 km northwest of the Munda Headworks on the Swat river, where the river flows east-northeast for a section of about 2 km in length while its general course is oriented southeast.

The bedrock is composed of crystalline schists of Permian Duma Formation that strikes at N30° to 70°W across the river nearly at right angle and dips more than 40° northeast or downstream. Strong joints of one group are nearly parallel with the schistocity or the bedding plane, and those of the other group strike at ENE-WSW in the direction similar to the river course dipping either southeast or northwest. In the Pre-Feasibility Study the schist was classified into several groups, that is, chlorite-mica schist, quartz-mica schist with talcosic bands, chlorite-mica schist with limestone bands and carbonatious graphitic schist, in order from the upstream outcrops to the downstream, or the lower strata to the upper horizon. Other than the schist, limestone beds of several meters to several tens of meters thickness are intercalated in the schist downstream and signs of local occurrence of doleritic rock in the dam site are reported.

Considering the complexity in mineral composition of the schist, a different approach of classification referring to their original rock type is made in this report, as follows:

- coarse pelitic schist or psammitic schist,
- fine pelitic schist,
- calcareous pelitic schist,
- green schist (coarse and fine),
- siliceous schist,
- limestone or marble.

The classification of pelitic schist is for the metamorphosed rock originating in muddy sedimentary rock, which includes a major part of the chlorite-mica schist in the Pre-feasibility Report. The green schist covers the doleritic rock that is more or less metamorphosed and schistose, and schists that are formed by alteration of tuff or other rocks of volcanic origin. The siliceous schist, composed largely of quartz, is nearly correlative with the quartz-mica schist of the Pre-Feasibility Report.

The dam will found mainly on the hard siliceous to psammitic schist on the left bank and the green schist on the right bank. Spillway weir and chute will be put on the siliceous schist and the green schist. The plunge pool will be situated in the calcareous pelitic schist. The pelitic schist with limestone band will be foundation of power house. (Ref. Figure B3.1)

At any classification, the bedrocks in fresh and intact condition are hard or moderately hard. On outcrops, the rock appears more or less weaker on the surface, weathered and slacked by open joints and foliation planes. In the meantime, the weathering does not appear so deeply developed, and sometimes becomes slight or virtually ineffective at the depth of 5 m from the ground surface, especially near the valley floor. In the adit No.1 at EL. 380 m on the left bank, the rock was found slacked only in the superficial 2 m section and sufficiently solid in the other part. (Ref. Figures B3.3-1, B3.5-1 and B3.5-2)

Lugeon tests in the hole M98-3 at the lower part of the abutment on the right bank and in the hole M98-2 at the middle height showed Lugeon values less than 10 at depth more than 10 m or 15 m (Ref. Figures B3.4-1 to b3.4-4). The hole M98-1 at the highest level of EL. 603 m near the contemplated dam crest, however, showed 128 to 42 Lugeon unit to the depth of 20 m and 10 to 20 Lugeon unit even in the deeper section to the depth of 65 m. This is deemed to reflect relatively intensive slacking of the rock in the parts of the slope higher than the level around EL. 560 m. This level is to be considered as a sort of boundary in foundation engineering condition. Higher than this level, the bedrock is visibly more slacked and unstable as is represented in thin and winding ridges. In the borehole M98-13 at EL. 564 m on the right bank saddle, the zone deeper than 20 m indicates only a little Lugeon value lower than 3.0.

On the left bank, the borehole M98-14 on the terrace, 20 m higher than the river, showed high water loss of 53 Lugeon unit only between the depths of 2 m and 7 m, but lower seepage potential less than 2.0 Lugeon unit in all sections deeper than 7 m. Other boreholes on the left abutment, however, tend to give rather high Lugeon values till relatively deeper reaches. For instance, a value of 20 Lugeon unit was given at the depth of 25 to 30 m in M98-15 at the middle height and a 28 Lugeon unit was recorded at the depth of 35 m in M98-16 at 170 m' height from the river surface. The borehole M98-17 drilled at a higher EL. 572 m for the spillway weir site has Lugeon value over 10 till the depth of 25 m. It is till the depth of 15 m in M98-18 at the similar height.

It should be noted that the permeability distribution as mentioned above does not mean outstandingly poor condition. Very high Lugeon values over 25 are not observed in the zone deeper than 15 m, and the bedrock deeper than 30 m or 35 m shows Lugeon values lower than 5, the condition that will neither take cement grout nor require grouting. This appears rather favorable, even compared with other examples.

In the seismic refraction prospecting, as shown in Figures B3.6-1 through B3.6-7, the bedrock under the dam axis is classified into four velocity layers as shown below in order descending from the ground surface:

Layer 1 the velocity 0.6 – 2.0 km/sec. for surface soil and zones of completely to highly weathered rock (Grade D and CL) or slacked rock, Layer 2 the velocity 2.1 -- 2.9 km/sec. for moderately weathered rock and moderately slacked, cracky and/or highly pervious rock zone (Grade CM),

Layer 3 the velocity 3.0 – 3.9 km/sec. for slightly weathered with open cracks for noticeable permeability (poorer part of Grade CH),

Layer 4 the velocity 4.0 – 5.4 km/sec. for slightly weathered or fresh rock zone, practically watertight with closed cracks.

The surface zone of very high Lugeon value corresponds with the Layers 1 and 2 mentioned above, of which bottom reaches to depth of 15 m at places. The Layer 3 represents the zone of 5 to 25 Lugeon unit and the Layer 4 meet the deep sound rock zone with Lugeon value less than 5.

River gravel deposit has thickness of 7.9 m, or approximately 8 m, according to the drilling M98-5 at the middle part of the river bed on the dam axis. It shows 1.5 km/sec of wave velocity in the seismic prospecting. Soft and closely foliated schist underlies the river deposit. Sound rock was reached at the depth of 13.55 m from the bottom of the river. The borehole M98-4, drilled at an angle of 60° from horizontal across the bedrock under the river bed, met neither fault nor fractured zone of any substantial size.

In the vicinity of the upstream edge of the dam, or around the location of a plinth in case of the concrete facing rockfill dam, four boreholes were drilled. Three holes on the left bank are M98-7, inclined from the river bank toward the opposite bank, M98-6 about 30 m higher than the river and M98-19 at the middle height, while on the right bank M98-8 was drilled at a spot 50 m higher than the river.

The hole M98-7 frequently met thin fractured zones and sheared zones before it reached 35 m of depth. While each of those weak zones has thickness of only 0.5 to 1.0 m, their aggregation makes the bedrock poor as a whole. The strength of the rock, even with those fractures, can be sufficient for foundation of the dam and the seepage potential shows less than 5 Lugeon unit or even none in sections deeper than 15 m. Considering that the water pressure tests indicate more than 60 Lugeon in depths less than 15 m, the figure of 15 m is to be regarded as the duly conservative requirement for the foundation excavation (Ref. B3.3-5).

The hole M98-19 at EL. 489 m on the left bank encountered sections of extraordinarily high water loss more than 100 Lugeon unit at the depths of 20 – 25 m and 30 – 35 m, while the sections deeper than 45 m are water-tight. The high permeability sections are deemed due to local fractured zones or cavities in the calcareous psammitic schist. Even if it is due to water loss through local calcareous cavities, they will not be of such major structure as to cause serious water loss from the reservoir. The bedding plane or schistocity strikes across the river, not developing parallel with the river channel, and karstic solution cavities, if any, is rather limited within a surface zone as explained in the later paragraph on the reservoir water-tightness.

In the meantime, the foundation conditions are far better in M98-6 and M98-8 high on the banks. Solid rock is reached within the depth of 5 m, even though some minor soft rock zones are intercalated in parts. The bedrock deeper than 14 m is almost completely free from harmful deterioration.

The limestone beds are located approximately 2 km downstream of the dam site, far enough to have no effect on under-seepage through the dam foundation. Thin limestone layers intercalated in the schist nearer to and downstream of the dam axis show signs of solution and cavity formation at places, but will cause no under-seepage problem because of their direction right angle to the river channel.

B3.2 Geology of Powerhouse Site

The powerhouse on the right bank will be placed on the pelitic schist, of which surface zone is highly weathered into the rock class CL (See Tables B8.1 to B8.3) to the depth around 3 m. The weathered rock, however, is strong enough to support the powerhouse, which has no severe foundation engineering requirement.

B3.3 Geology of Diversion/Power Tunnel

A power tunnel is planned on the right bank, and two diversion tunnels are on the left bank. Most of the tunnel routes consist of intact rocks, and no serious problem is envisaged on stability of tunnel faces and spring water during the excavation (Ref. B3.3-4). Outlets of the three tunnels are located on the stable rock bed with dip-slope. The existing balance of stability should not be disturbed by careless uncontrolled excavation.

For the outlet of the power tunnel on the right bank, the borehole M98-11 shows that the rock is solid below EL. 373 m, or the depth of 8 m from the bottom of the gully near the powerhouse site.

B3.4 Geology of Spillway Site

The spillway chute on the left bank is underlain by siliceous schist, green schist, and pelitic schist (Ref. Figure B3.3-3). At the higher portion of the chute, the siliceous schist is flaky and slippery, containing much mica. Occurrences of many local rockslides are seen on the east-facing dip-slopes.

For the plunge pool, a substantial quantity of excavation will be required at an existing steep slope of the pelitic schist that will in turn create a new steep slope on its left side. The new slope will have joint planes dipping toward riverside among joints of other orientations. Rock sliding is possible to occur along the joint planes, if the slope is high and steep. It is therefore advisable that layouts causing high cut-slopes should be avoided. High and steep slopes should be minimised.

As revealed by the drilling at M98-17, the foundation of the spillway weir will be on the siliceous schist which shows high Lugeon values over 20 till the depth of 25 m, or EL. 547 m. If the foundation of the spillway weir is lower than this elevation, it will be in sound rock that may take little grout. At the lower end of the spillway chute, the moderately weathered pelitic schist lies under colluvial deposit with thickness of 2 m, and a good foundation of sound rock is reached at the depth of 5 m, according to the drilling M98-12.

B4 Geology of Reservoir

The reservoir will develop in the long narrow gorge of the Swat river, where the bedrock is widely exposed and the overburden is limited and thin (Ref. Figure B4.1). Large land slide is not likely to occur. A main question lies upon possibility of water leakage from the reservoir, especially through limestone beds that might have been karstified. Crystalline limestone beds occur in the calcareous schist of the Lower Triassic Kashala Formation and are also a member of the Permian Duma Formation.

The Kashala Formation is met by the reservoir at a few kilometers upstream of the dam site and in two parts farther upstream. Solution cavities are seen on the ground surface in some area, e.g. on the Ambahar river, a right bank tributary. These cavities, however, appear to develop only in the surface zone of the bedrock within an extent of rain water infiltration and drainage. Thickness of ridges separating the reservoir from adjacent basins is as thick as several kilometers to 10 km. The Kashala Formation nearest to the dam site is contained inside the barrier of other formations of Saidu and Duma to stop direct water leakage.

The dam site is on the Duma Formation that is composed mainly of schists. Limestone beds are intercalated at places as mentioned above but make minor components in the formation. Signs of limestone solution are seen at places but seemingly within the surface zone where rain water circulates.

Bore hole water pressure test was performed in the limestone beds at Sappare quarry site, 3 km east-northeast of the dam site, to examine seepage potential of limestone. In a borehole Qs-1 drilled at southwestern part of Sappare and in an upstream part of a gully through the deserted village, rather high water losses of 30 Lugeon unit or more were observed to the depth of 45 m with only one exceptional test section that showed 2.6 Lugeon. A 5 m long test section below the depth of 45 m indicated only 7.1 Lugeon. It, however, is not known whether the low seepage potential continues deeper than the bottom of the hole at the depth of 50 m. In the meantime, the other hole Qs-2, a 100 m deep hole drilled at northeastern part and downstream, showed low Lugeon values less than 5 in test sections deeper than 10 m. All these conditions are deemed to prove that development of the limestone solution is limited within several tens of meters in depth where the infiltrated surface water circulates underground and drains again to the ground surface.

The limit of localised water seepage paths is also indicated by quick disappearance of water springs at Sappare every dry season. The aerial photograph also shows discontinuity of the limestone bed between the reservoir area and Sappare.

The limestone beds, therefore, is deemed to provide no serious passage for water leakage from the reservoir. The reservoir is deemed practically watertight.

B5 Construction Material

Construction material required for the Munda dam consists of (1) earth material for the impervious core zone of the rockfill dam, (2) pervious material for the filter zone of the rockfill dam, (3) rock material for the rockfill dam and (4) coarse and fine concrete aggregates. A considerable amount of investigation and laboratory tests have been done in the previous Pre-Feasibility Study. The present circumstances for prospect of material sources are as follows:

B5.1 Earth Core Material

In the Pre-Feasibility Study, sources of the earth core material were contemplated with clayey deposits on both sides of the Swat river within 6 km downstream of the existing Munda headwork, that is, "Abazai clayey silt source" on the left bank and "Sadar Gari sandy silt source" on the right bank. These borrow pits area, however, would compel to sacrifice extensive and fertile farm lands.

In the stage of the Feasibility Study, the investigation was extended to other potential borrow areas (Ref. Figure B5.1), that is;

- Kas Koruna, on low hills on the left bank or the north bank of the Lower Swat Canal that develops from the left bank of the Munda head work to east-southeast. The haul distance to the dam site will be around 11 km. This site around Kas Koruna is for the most part not utilized for cultivation. Four (4) test pits, P-8 to P-11, were dug in this site.
- Tangi, on flat land and low isolated hills approximately 2 km east of Kas Koruna. The haul distance to the dam site is 13 km. Three (3) test pits, P-12 to P-14, were dug.
- West Saddar Garhi, in the plain west of the previously contemplated Saddar Garhi borrow area across the motor road. The haul distance to the dam site is approximately 10 km. As a merit, this area appears now not being used for any purpose, while there are remnants of many houses allegedly deserted by Afghan immigrants who have already evacuated. One (1) test pit, P-15, was dug in this area.

All these pits were dug to the depth of 5 m except the pit P-14 that was 2.5 m deep (Ref. Table B3.1 and Log of Pit in the Data Book).

Fifteen samples were taken from the pits P-8 through P-15 and tested on the characteristics of the earth material for impervious core zone of the rockfill dam. The test items covered the grain size analysis, Atterberg limits, compaction, and then, triaxial compressive strength and permeability of the compacted material. The results are summarised in Table B5.1.

Dispersion test was made for samples of P-10 at Kas Koruna, P-13 at Tangi and P-15 at West Sadar Garhi, proving that the soils are not dispersive.

The materials often contain large proportion of silt, while the clay contents are limited to around 12% or lower. An exceptionally high value is 20% at the pit P-11 at Kas Koruna which, however, lacks component coarser than silt. It appears that the materials of the Kas Koruna and Tangi borrow area considerably vary its quality for the changes in the sedimentary environment.

a) Earth Material of Kas Koruna

The material in the pit P-8 in the northern part of the Kas Koruna borrow is divided into two layers; the upper layer and the lower layer with a boundary at the depth of 3.5 m. The upper layer containing much gravel falls under the category of GP or GW in the Unified Soil Classification of USBR. The lower layer is classified to SW or SP with the 92% sand contents and a little fine component. While these materials are good for compaction, the compacted material will have still high permeability and are not suitable for the earth core of dam. The low permeability reported from the laboratory is deemed due to the test on the sample of reduced contents of gravel over 20 mm.

The material at the depth from 0.5 m to 5.0 m of the pit P-9 is for the most part clayey sand, classified into SM close to ML, and usable for the impervious core zone even if not very stable from mechanical point of view. The material in the surface zone not deeper than 0.5 m, which is fine and plastic, is suitable for the material for layer of contact with the rock surface.

Major part of the material in the pit P-10 deeper than 1.2 m is composed of sandy silt classified into CL. The laboratory test indicated no serious drawback for the low clay contents of 6%. The consolidated-undrained triaxial test of a compacted sample has given cohesion of 18 kPa (0.18kgf/cm^2) and internal angle of friction at 28°.

Material in the pit P-11 near the Lower Swat Canal has relatively high clay contents of 13 - 20% and almost no sand and gravel contents. The cohesion and the internal angle of friction by the triaxial test of the compacted sample are 20 kPa and 31°, respectively. For the high plasticity this material is suitable for the contact layer with the foundation rock.

Material of the Kas Koruna borrow site will be usable for the earth core zone of rockfill dam, except for that in the northern part represented by the pit P-8. The diversity in its quality, as represented by the varied grain size distribution, may cause some difficulty in quality control of the embankment. The optimum moisture contents varies between 10 and 20%, that is generally higher than natural moisture contents

b) Earth Material of Tangi

Material of the pit P-12 in the western part of this area is classified to GW with gravel contents of 60 to 80%, and not suitable for the earth core zone.

Material of the pits P-13 and P-14 falls under CL of the Unified Soil Classification and can be used for the earth core, except the upper layer not deeper than 2 m of P-13 that is gravelly and rather similar to the sample from P-12. Only in the material of P-14, the optimum moisture contents are lower than the natural moisture contents. The strength of the compacted P-13 material is similar to that of P-11 of Kas Koruna, the pit nearest to Tangi.

This borrow area has drawback in the longer haul distance and the possible interference with cultivated land and housing areas.

c) Earth Material of West Saddar Garhi

West Saddar Garhi has only one test pit and accordingly is not relevant to discuss about diversity in the material. This area, however, is topographically characterised by an extensive plain or a single sedimentary environment between the Swat river and the hills to the west, not divided by low hills as other borrow areas. The sample from the pit P-15 falls under the category of SM in the Unified Soil Classification of USBR, suitable for embankment material and easy to compact. The hauling distance can be shortest of all the borrow areas. This borrow site in a deserted land also has a merit of less environmental problem, or less interference to lands cultivated or inhabited.

B5.2 Concrete Aggregates and Filter Material

For concrete aggregates and filter material, five test pits, P-1 to P-5, were dug up to the depth of 1.5 m in the sand-gravel bar on the Swat river bed downstream of the Munda Headworks. Other two pits, P-6 and P-7, were also dug to the depth of 5 m in the terrace gravel beds on the hills on the left bank and to the east of the Munda Headworks. (Ref. Figure B5.1)

Six samples from the pits P-1, P-2. P-3, P-5 and P-7 were tested in the laboratory for the items of sieve analysis, water absorption, specific gravity, friable particles, soundness by sodium sulfate, abrasion by Los Angeles machine and chemical reactivity. The results are presented in Table B6.1.

The tests indicated almost commonly that the sand contents in those deposits are rather low, with 27.4% as the highest value obtained and 15% as the lowest. Gradation curve shows especially low percentages for the medium and coarse sand. Sand for fine concrete aggregate will have to be artificially produced by crushing the gravel.

Gravel of the Swat river bed can be acceptable for coarse aggregate of concrete, with specific gravity more than 2.70, absorption less than 1.0%, weight loss also less than 1.0% in the sodium sulfate soundness test, not more than 6% loss in the Los Angeles abrasion test and less than 0.3% of contaminating substances with soft particles and clay lumps. All the samples fell under the area of innocuous aggregate in the chemical alkali reactivity test.

Gravel deposit on the left bank hill is inferior for concrete aggregates to the Swat river gravel, in its high water absorption and contamination. It, however, has possibility for a filter material if boulders of size over 200 mm are removed.

B5.3 Rock Material

Solid and massive rock material was sought for dam embankment. The previous investigation found two potential limestone quarry sites. One is located around an old and deserted village of Sappare on a 200 m high hill on the left bank of the Swat river and approximately 3 km northeast of the dam site. The other is at an isolated hill near the village of Tangi at 16 km of haul distance.

The Tangi quarry site with very limited quantity and long haul distance has been discarded. The Sappare quarry with two limestone beds with apparent thickness of 40 m and 200 m appeared to give sufficient quantity of rock material. More detailed geological survey, however, revealed that the second layer is only locally thick due to a tight folding and is thinner in most parts. The expectable reserve has been thereby reduced to approximately 5 million cubic meters.

As a result of further geological reconnaissance, another potential quarry site at Todobo Banda was introduced for the study. This site is located on the right bank of the Swat river approximately one kilometer upstream of the dam site and will produce quartzite and siliceous schist. In consequence, the core drilling and the seismic refraction prospecting were carried out in Sappare and Todobo Banda (Ref. B5.4 and B5.5-1 to B5.5-3). The locations and quantity allocation are shown in Figure B2.1, Figure B5.2-1 and Figure B5.2-2. The results of the laboratory rock tests are seen in Table B7.1.

In the Sappare quarry site with limestone, two bore holes were drilled, i.e., Qs-1 with the depth of 50 m, and Qs-2, with the depth of 100 m. The seismic refraction prospecting was carried out on a single line of 500 m in length through those two drilling locations. Rock zones of the velocity 3.5 to 4.0 km/sec, under a few meters thick overburden, will produce sufficiently solid rock material as indicated by the core drilling. Five test pieces were selected from the core samples and sent to the laboratory for tests of unconfined compressive strength, water absorption and bulk density. The compressive strength of fresh limestone ranged from 31.35 MPa to 39.18 Mpa, or approximately 30 to 40 Mpa. The outstanding merit of the Sappare quarry is the availability of boulders or big rock blocks, as indicated by the drilling core samples of unit length over 1.0 m, while quantity of the hard limestone is limited. The Sappare quarry will be viable if the schist associating the limestone is utilised for parts of the dam body.

In the Todobo Banda quarry with quartzite and quartz mica schist, five bore holes, Qt-1 through Qt-5, were drilled to explore quality and quantity of this quarry site; with the hole Qt-2 to the depth of 100 m and others respectively to 50 m. The seismic refraction prospecting was performed on two crossing lines with the lengths of 1.0 km and 0.5 km. Ten test pieces were collected from the core samples of the holes Qt-2 and Qt-3 and tested in the laboratory for the unconfined compressive strength, water absorption and bulk density. Results of the tests showed some variation. The unconfined compressive strength was high for seven samples, ranging from 58 Mpa to 167 Mpa, while three other samples fell under a range between 8.5 Mpa and 22.3 Mpa. The rock in Qt-1 was mainly quartzite, while in all other holes quartz mica schist dominated. The rock of Todobo Banda is characteristically tends to break into small fragments of 5 to 15 cm of diameter by schistose foliations. Quantity is sufficient.

B6 Seismicity

The project area is located in a highly active tectonic zone with thrust faults in the Himalayan foot-hills, and in a region of high seismicity.

Earthquake risk of the dam was evaluated based on earthquake records obtained through internet from US Geological Survey. Those are data of:

i) 26 events of significant earthquakes over 5.5 of Magnitude in the Richter scale with epicenters within 300 km of distance from the Munda Dam site (34.35°N/71.33°E) that were recorded in the period between 1896 and 1992, and

ii) 2,259 events or all earthquakes over 1.0 of Magnitude in the Richter scale with epicenters within 300 km of distance from the Munda Dam site that were recorded in the period of 25.5 years from 1973 to 1998.

The latter data of 2,259 earthquakes were mainly used for evaluation of seismicity. For each earthquake estimated was intensity that could have been felt at the Munda Dam site by use of formulae of attenuation relationship; one from Cornell and the other by Kawasumi.

The formulae of attenuation relationship used are as follows:

- Formula according to Cornell

(Cornell, C.A., 1968, Engineering seismic analysis, Bull. Seism. Soc. Am. Vol.58, pp.1583-1606)

$I = 8.0 + 1.5\ M - 2.5\ Ln\ r$

where,

I: Earthquake Intensity in Modified Mercalli Scale felt at the dam site

M: Magnitude in Richter Scale

r: Focal distance in kilometer $r = (d^2 + h^2 + 400)^{0.5}$

d: Epicentral distance (km)

h: Focal depth (km)

$\log A = 0.014 + 0.30\ I$ *

where,

A: Peak horizontal acceleration (cm/sec^2 or gal)

(*Trifunac, M.D. and Brady, A.G., 1975, On the correlation of seismic intensity scales with the peak of recorded strong ground motion, Bull. Seism. Soc. Am. Vol.65, pp.139-162)

- Formula according to Kawasumi

(Kawasumi, H., 1951, Measures of earthquake danger and expectancy of maximum intensity throughout Japan as inferred from the seismic activity in historical times, Bull. Earthq. Res. Inst., 21, pp469-482.)

$Ij = 2\ M - 4.6052 \log d - 0.00183\ d - 0.307$ (when d is not less than 100 km)

$Ij = 2\ (M - \log r) - 0.01668\ r - 3.9916$ (when d is less than 100 km)

$A = 0.45 \times 10^{(Ij/2)}$ (when Ij is not more than 5.5)

$A = 20 \times 10^{(Ij/5)}$ (when $5.5 < Ij < 7.0$)

where,

Ij: Earthquake intensity in Japan Meteorological Agency Scale (JMA)

M: Magnitude in Richter Scale

d: Epicentral distance (km)

r: Focal distance (km)

A: Peak ground acceleration (cm/sec^2 or gal)

Number of earthquake events was counted for each intensity step, i.e., Intensity 1 (0.5 to 1.4), Intensity 2 (1.5 to 2.4), Intensity 3 (2.5 to 3.4), etc., and then accumulated to obtain the number of events in 25.5 years exceeding the given intensity for each of the same intensity steps. Each number for each step was divided by 25.5 (years) to get the number of events per one year exceeding the given intensity (Ne).

According to Gutenberg, the earthquake intensity (I) has a linear relationship with the logarithm of the number of earthquakes exceeding that intensity, that is,

$$\log Nc = p + q.I,$$

where p and q are constants. The values of I and Nc were plotted on a graph (Ref. Figure B6.1), and the point where the I - log Nc line intersects the horizontal line for 0.01 of Nc gives the probable maximum earthquake intensity for the return period of 100 years (and 0.005 of Nc for 200 years). The result is as follows:

Maximum Intensity and Peak Acceleration

	Maximum Intensity in MM Scale	Maximum Intensity in JMA Scale	Maximum Peak Acceleration
According to Cornell	7.3	-	154 gal
According to Kawasumi	-	4.5	80 gal

On the other hand, it is assumed that Maximum Credible Earthquake is generated at the distance of 10 km due to the Main Mantle Thrust and has Magnitude of 7.0 and focal depth of 40 km. The distance of 10 km is used conservatively to cover some obscurity in the exact location of the major fault and also possible subordinate faults nearer to the dam site. Magnitude 7 is deemed to be the possible maximum from the existing records of earthquakes generated at depths within 50 km and in the south of the Main Mantle Thrust. The recorded Magnitude there do not exceed the range of 5 to 7 (Ref. A. H. Kazmi, 1979, Preliminary Seismotectonic Map of Pakistan, Geological Survey of Pakistan). Intensity and peak acceleration are estimated at 9 and 500 gal by Cornell's attenuation formula shown above.

The Maximum Credible Earthquake acceleration of 0.5g estimated above is a little higher than those in the Pre-Feasibility that ranges from 0.15g (according to Bolt and Abrahamson, 1982) to 0.35g (according to Krinitzky Chang & Nuttili, 1988). There, however, seems to be no essential theoretical discrepancy up to this point.

The peak acceleration works only for a fraction of a second and virtually unable to effect any damage upon dam structures. Substantially durable acceleration is far lower than the peak value, for example a third the peak acceleration. From this viewpoint, the practically damaging acceleration level from the maximum credible earthquake can be 170 gal. The maximum credible earthquake, however, is the conceivable strongest earthquake of which probability of occurrence is very low. It is defined that some damages upon structures by

Maximum Credible Earthquake should be accepted if those damages do not lead to serious hindrance of their function.

With all the above results of evaluation in mind, the appropriate earthquake acceleration is proposed at 0.15g for the pseudo-static design against earthquake risk. This value is deemed reasonable compared with other examples in such highly tectonic zones as Indonesia, Nepal and Japanese Archipelago as well as some examples in Pakistan.

Situated in the tectonic zone, the possibility of reservoir-induced earthquake cannot be eliminated. However, there is no major fault in the reservoir area that might generate earthquakes of high magnitude. The assumption for the estimate of the Maximum Credible Earthquake mentioned above, i.e., an earthquake of Magnitude 7 at the distance of 10 km and the depth of 40 km, will well cover a possible earthquake that may occur in the reservoir. The magnitudes of reservoir-induced earthquakes are not high in general, as in the examples of Cirata Dam, Indonesia, not more than 3.8. The maximum peak acceleration less than 500 gal or 0.5g that works only in an instant will be coped with by the proposed design acceleration of 0.15g.

B7 Future Investigation

The detailed geological investigation will further be required for the detailed design in the future. The main purpose is the confirmation of the foundation condition at exact locations of the structures to meet the finally determined layout, the confirmation of the rock mechanical characteristics, the detailed confirmation of the quarry sites, especially about Todobo Banda quarry, and the detailed confirmation of the borrow area at West Sadar Garhi. The investigation will include but not be limited to the following items:

(1) Core drilling

Dam axis, from the level of dam crest on both banks	200 m x 2 holes
Plinth foundation for a concrete facing rockfill dam	50 m x 6 holes
Spillway chute	40 m x 2 holes
River bed for plinth and coffer dams	30 m x 4 holes
Quarry sites, Todobo Banda	50 m x 8 holes
Sappare	50 m x 4 holes
Re-regulation weir site	50 m x 6 holes
Irrigation system	200 m
	(Total 2,000 m)

(2) Exploratory adit

Todobo Banda quarry site — 100 m x 2 adits

(3) In-situ rock test in the existing adit at the dam site

- Deformation modulus — 12 spots
- Shear strength each with three test blocks — 4 times

(4) Test pitting at West Saddar Garhi — 12 locations

(5) Laboratory tests

- Rock material, drilling core sample — 30 samples
- Earth embankment material — 20 samples

TABLES

Table B1.1 List of Drill Holes

No.	Depth (m)	Inclination (degree)	Location	Coordinate		Elevation (m)
				Northing	Easting	
Dam site						
M98-1	70.00	90	Dam axis; right bank	1,124,626.040	3,067,969.091	602.613
M98-2	100.00	90	Dam axis; right bank	1,124,807.933	3,067,915.867	508.620
M98-3	100.00	90	Dam axis, right bank	1,125,022.213	3,067,851.069	417.211
M98-4	70.00	60	Dam axis; riverbed	1,125,138.763	3,067,817.933	379.708
M98-5	30.00	90	Dam axis; riverbed	1,125,094.000	3,067,825.000	358.500
M98-6	70.00	90	Dam u/s end; left bank	1,125,140.165	3,067,562.789	395.089
M98-7	100.00	60	Dam u/s end; riverbed	1,125,018.645	3,067,584.851	370.389
M98-8	70.00	90	Dam u/s; right bank	1,124,881.850	3,067,610.461	417.032
M98-9	30.00	90	Tunnel intake	1,124,779.195	3,067,458.727	388.542
M98-10	30.00	90	Power station	1,125,230.008	3,068,226.471	384.106
M98-11	30.00	90	Tunnel outlet, right b.	1,125,236.704	3,068,380.158	375.300
M98-12	30.00	90	Spillway chute	1,125.479.286	3,068,221.870	401.918
M98-13	30.00	90	Right bank saddle	1,124,213.465	3,067,664.089	563.674
M98-14	180.00	90	Dam axis, left bank	1,125,167.569	3,067,834.841	382.011
M98-15	100.00	90	Dam axis, left bank	1,125,237.329	3,067,787.588	451.253
M98-16	70.00	90	Dam axis, left bank	1,125,411.839	3,067,732.004	546.269
M98-17	60.00	90	Dam axis, left bank	1,125,567.789	3,067,740.659	571.860
M98-18	30.00	90	Dam axis, left bank	1,125,811.380	3,067,632.113	560.337
M98-19	70.00	90	Dam u/s end, left bank	1,125,285.282	3,067,590.818	488.720
M98-20	30.00	90	Tunnel intake, left b.	1,125,104.856	3,067,375.848	418.234
Total	1,300.00					
Quarry site						
Qt-1	50.00	90	Todobo Banda	1,125,326.226	3,065,899.435	662.321
Qt-2	100.00	90	Todobo Banda	1,125,364.729	3,066,101.804	601.605
Qt-3	50.00	90	Todobo Banda	1,125,425.783	3,066,422.702	456.615
Qt-4	50.00	90	Todobo Banda	1,125,905.256	3,066,280.909	501.710
Qt-5	50.00	90	Todobo Banda	1,124,946.310	3,066,564.495	489.247
Qs-1	50.00	90	Sappare	1,126,172.176	3,070,357.495	528.236
Qs-2	100.00	90	Sappare	1,126,371.357	3,070,560.418	513.373
Total	450.00					

Table B2.1 List of Exploratory Adits

Adit No.	Location	Coordinates of Portal		Elevation (m)	Direction of Adit	Length (m)
		Northing	Easting			
1	Dam axis, left bank	1,125,198.642	3,067,877.682	383.244	N43°W	60
2	Dam axis, right bank	1,125,040.924	3,067,828.403	401.020	S12°E	50
3	Dam axis, left bank	1,125,446.022	3,067,758.288	538.467	N09°E	50
4	Dam axis, right bank	1,124,764.137	3,067,941.393	540.237	S16°E	50
Total						210

Table B3.1 List of Pits

No.	Depth (m)	Location
Earth core material		
P-8	5.00	Kas Koruna
P-9	5.00	Kas Koruna
P-10	5.00	Kas Koruna
P-11	3.00	Kas Koruna
P-12	5.00	Tangi
P-13	5.00	Tangi
P-14	2.50	Tangi
P-15	5.00	Sadar Garhi
Sand and gravel		
P-1	1.50	Swat river bed d/s Munda Headwork
P-2	1.50	Swat river bed d/s Munda Headwork
P-3	1.50	Swat river bed d/s Munda Headwork
P-4	1.50	Swat river bed d/s Munda Headwork
P-5	2.00	Swat river bed d/s Munda Headwork
P-6	5.00	Left bank hill near Munda Headwork
P-7	5.00	Left bank hill near Munda Headwork

Table B4.1 List of Seismic Prospecting Lines

Line No.	Location	Length (m)	Start			End		
			Northing	Easting	Eelvation (m)	Northing	Easting	Elevation (m)
Damsite								
S-1	Dam axis (The right bank to the left bank)	1,000	1,125,513.551	3,067,709.196	556.648	1,124,556.127	3,067,988.054	594.393
S-2	Dam axis (Middle portion of the left bank)	300	1,125,669.652	3,067,728.887	581.053	1,125,403.266	3,067,724.460	540.205
S-3	Dam axis (Higher portion of the left bank)	500	1,126,077.998	3,067,549.626	596.631	1,125,616.831	3,067,744.122	575.607
S-4	Spillway route	600	1,125,497.075	3,068,251.527	426.021	1,125,508.167	3,067,576.152	480.654
S-5	Left bank terrace	900	1,125,335.881	3,068,228.871	380.677	1,124,975.383	3,067,374.608	393.785
S-6	Upstream part of the diversion/ power tunnel	700	1,124,597.000	3,068,092.000	569.594	1,124,752.947	3,067,411.045	369.168
S-7	Downstream part of the diversion/ power tunnel	1,200	1,125,436.384	3,068,524.853	368.018	1,124,440.793	3,067,731.573	559.115
S-8	Left bank saddle	800	1,124,661.758	3,067,997.923	584.112	1,124,040.320	3,067,495.192	592.220
Total		6,000						
Quarry site								
Lt-1	Todobo Banda	1,080	1,124,850.416	3,066,592.854	528.468	1,126,001.150	3,066,252.550	505.327
Lt-2	Todobo Banda	850	1,125,443.539	3,066,516.027	421.611	1,125,284.667	3,065,681.008	772.829
Ls-1	Sappare	520	1,126,376.011	3,070,656.544	506.326	1,126,162.144	3,070,258.041	535.016
Total		1,450						

Table B5.1 Summary of Laboratory Test Results on Earth Material

No.	Pit No.	Depth (m)	Grain size Coble (%)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	Natural Water Contents (%)	Atterberg Limit LL (%)	PL (%)	PI	Compaction test MMD (Mg/m^3)	OMC (%)	Sp.Gr. (Mg/m^3)	Triaxial Test CU C(kPa)	CU ø(°)	UU C(kPa)	UU ø(°)	Permeability Coefficient K (cm/sec)
C01	P-8	0.15 - 3.50	5	72	11	12	0	2	25	18	7	2.11	8.5	2.67	-	-	-	-	1.65×10^{-6}
C02	P-8	3.50 - 5.00	0	0	92	8	0	2	----	NP	----	1.69	13.5	2.76	-	-	-	-	-
C03	P-9	0.10 - 0.40	0	8	15	60	17	14	33	23	10	1.68	19.0	2.66	-	-	-	-	-
C04	P-9	0.50 - 5.00	0	23	35	32	10	11	26	21	5	1.89	13.0	2.68	5	32	96	0	1.00×10^{-5}
C05	P-10	0.15 - 1.00	0	1	59	29	11	4	-----	NP	----	1.64	19.0	2.67	-	-	-	-	-
C06	P-10	1.20 - 5.00	0	0	36	58	6	5	-	-	-	1.67	16.0	2.68	18	28	125	0	2.39×10^{-5}
C07	P-10	1.20 - 5.00	0	0	28	72	0	5	-	-	-	-	-	2.68	-	-	-	-	-
C08	P-11	0.15 - 1.70	0	0	2	85	13	15	43	30	13	1.43	25.0	2.69	-	-	-	-	-
C09	P-11	1.70 - 3.00	0	0	1	79	20	10	40	28	12	1.69	19.0	2.69	20	31	50	0	-
C10	P-12	0.15 - 1.50	0	80	7	7	6	5	30	23	7	2.07	11.6	2.67	-	-	-	-	-
C11	P-12	1.60 - 5.00	0	75	20	3	2	1	----	NP	----	2.03	10.7	2.75	-	-	-	-	-
C12	P-13	0.15 - 2.00	0	62	20	13	5	4	-	-	-	2.03	10.2	2.69	-	-	-	-	-
C13	P-13	2.10 - 5.00	0	2	24	74	0	10	26	22	4	1.69	17.7	2.69	19	30	62	0	6.63×10^{-6}
C14	P-13	2.10 - 5.00	0	2	14	70	14	10	26	22	4	-	-	2.69	-	-	-	-	-
C15	P-14	0.15 - 1.60	0	1	30	55	14	18	27	22	5	1.81	13.5	2.68	-	-	-	-	-
C16	P-14	1.70 - 2.50	0	3	61	33	3	19	-----	NP	----	1.95	11.0	2.70	-	-	-	-	-
C17	P-15	0.15 - 3.00	0	4	46	50	0	6	30	24	6	1.83	14.3	2.67	24	29	44	0	3.43×10^{-6}
C18	P-15	0.15 - 3.00	0	4	42	40	14	6	30	24	6	-	-	2.67	-	-	-	-	-

Dispersion Test: P-10 (1.20 - 5.00m) Non dispersive, P-13 (2.10 - 5.00m) Non dispersive, P-15 (0.15 - 3.00m) Non dispersive

Note. NP = Non Plastic

Table B6.1 Summary of Laboratory Test Results on Sand and Gravel

No.	Pit No.	Depth (m)	Grain size Cobble (%)	Gravel (%)	Sand (%)	Specific gravity C. Agg. (Mg/m3)	F. Agg. (Mg/m3)	Absorption C. Agg. (Mg/m3)	F. Agg. (Mg/m3)	Sodium sulphate soundness (%)	Clay lump & friable particles (%)	Los Angeles abrasion (% of water)	(Uniformity of water)	Alkali-silica reactivity (m.moles/lit.) Sc	Rc
S01	P-1	0.15 - 1.50	20.6	62.5	16.9	2.89	2.90	0.31	1.68	0.19	0.21	6.0	0.19	21.15	80.0
S02	P-2	0.15 - 1.50	-	-	-	2.78	2.90	0.46	1.49	-	0.20	-	-	24.31	90.0
S03	P-3	0.85 - 1.50	28.4	44.2	27.4	2.77	2.89	0.30	1.55	0.31	0.19	5.0	0.19	12.32	40.0
S04	P-5	0.15 - 1.25	19.1	65.9	15.0	2.77	2.88	0.52	1.21	0.77	0.19	5.0	0.15	17.65	72.5
S05	P-5	1.25 - 1.50	-	-	30.0	-	2.88	-	1.21	-	-	-	-	-	-
S06	P-7	0.15 - 5.00	31.5	44.5	24.0	2.68	2.53	2.34	4.93	-	6.61	-	-	18.15	105.0

Table B7.1 Summary of Laboratory Test Results on Rock Material

No.	Hole No.	Depth (m)	Bulk density (Mg/m^3)	Water absorption (%)	Unconfined compressive strength (Mpa)	Internal friction angle (°)	Cohesion (MPa)
R01	M98-1	8.63 - 8.90	2.633	2.34	11.52	-	-
R02	M98-2	44.48 - 44.91	2.778	0.37	11.52	-	-
R03	M98-3	16.00 - 16.27	2.844	0.46	13.40	-	-
R04	M98-4	14.67 - 14.82	2.943	0.13	10.37	-	-
R05	M98-6	9.46 - 9.73	2.794	0.19	12.10	-	-
R06	M98-7	3.36 - 3.69	2.989	0.41	14.40	-	-
R07	M98-8	15.00 - 15.67	2.977	0.10	50.72	-	-
R08	M98-9	9.40 - 9.66	2.935	0.16	19.02	-	-
R09	M98-10	5.50 - 5.70	2.833	0.47	40.34	-	-
R10	M98-11	8.38 - 8.62	2.895	0.39	19.02	-	-
R11	QS-1	4.40 - 4.65	2.664	0.06	39.18	-	-
R12	QS-1	10.32 - 10.73	2.667	0.03	32.33	-	-
R13	QS-1	20.23 - 20.52	2.678	0.21	31.35	-	-
R14	QS-1	27.72 - 28.00	2.678	0.07	32.13	-	-
R15	QS-1	37.15 - 37.43	2.687	0.00	35.07	-	-
R16	QT-3	3.70 - 3.85	2.751	0.42	8.50	-	-
R17	QT-3	8.35 - 8.52	2.711	0.00	66.81	-	-
R18	QT-3	13.82 - 14.00	2.634	1.50	13.78	-	-
R19	QT-3	20.60 - 20.75	2.614	0.88	22.27	-	-
R20	QT-3	47.55 - 47.70	2.701	0.08	58.33	-	-
R21	QT-2	4.86 - 5.00	2.674	0.53	69.20	-	-
R22	QT-2	19.44 - 19.60	2.720	0.10	68.93	-	-
R23	QT-2	26.40 - 26.52	2.709	0.23	167.00	-	-
R24	QT-2	44.45 - 44.58	2.711	0.17	153.78	-	-
R25	QT-2	55.70 - 55.88	2.758	0.17	84.84	-	-
R26	M98-1	8.18 - 8.48	-	-	-	32.6	3.3
R27	M98-2	47.00 - 47.48	-	-	-	25.4	11.5
R28	M98-3	16.30 - 16.93	-	-	-	33.7	5.9
R29	M98-4	14.18 - 14.51	-	-	-	35.0	3.6
R30	M98-5	15.00 - 15.67	-	-	-	41.8	10.5

Table B8.1 Factors of Rock Classification

Grade	Stiffness	Assumed unconfined compression strength (kgf/cm^2)
A	Very hard	$600 \leqq$ Qu (hardly broken with clear sound by hammer
B	Hard	$300 \leqq$ Qu $<$ 600 (difficult to break with clear sound)
CH	Fairly hard	$100 \leqq$ Qu $<$ 300 (broken with dim sound by hammer)
CM	Moderate	$50 \leqq$ Qu $<$ 100 (easy to break with dull sound)
CL	Fairly soft	$30 \leqq$ Qu $<$ 50 (easy to break into small pieces)
D	Soft	Qu $<$ 30 (easy to break without sound by

Grade	Joint Interval (cm)
1	$30 \leqq I$
2	$15 \leqq I < 30$
3	$5 \leqq I < 15$
4	$2 \leqq I < 5$
5	$I < 2$

Grade	Joint Condition
a	Fresh, closely adhered
b	Stained and slightly deteriorated along joints
c	Stained and deteriorated, intercalated with soft materials, or slightly open
d	Open or un-distinguishable (Fragmented, sandy, silty, or clayey)

Table B8.2 Combination of Factors for Rock Classification

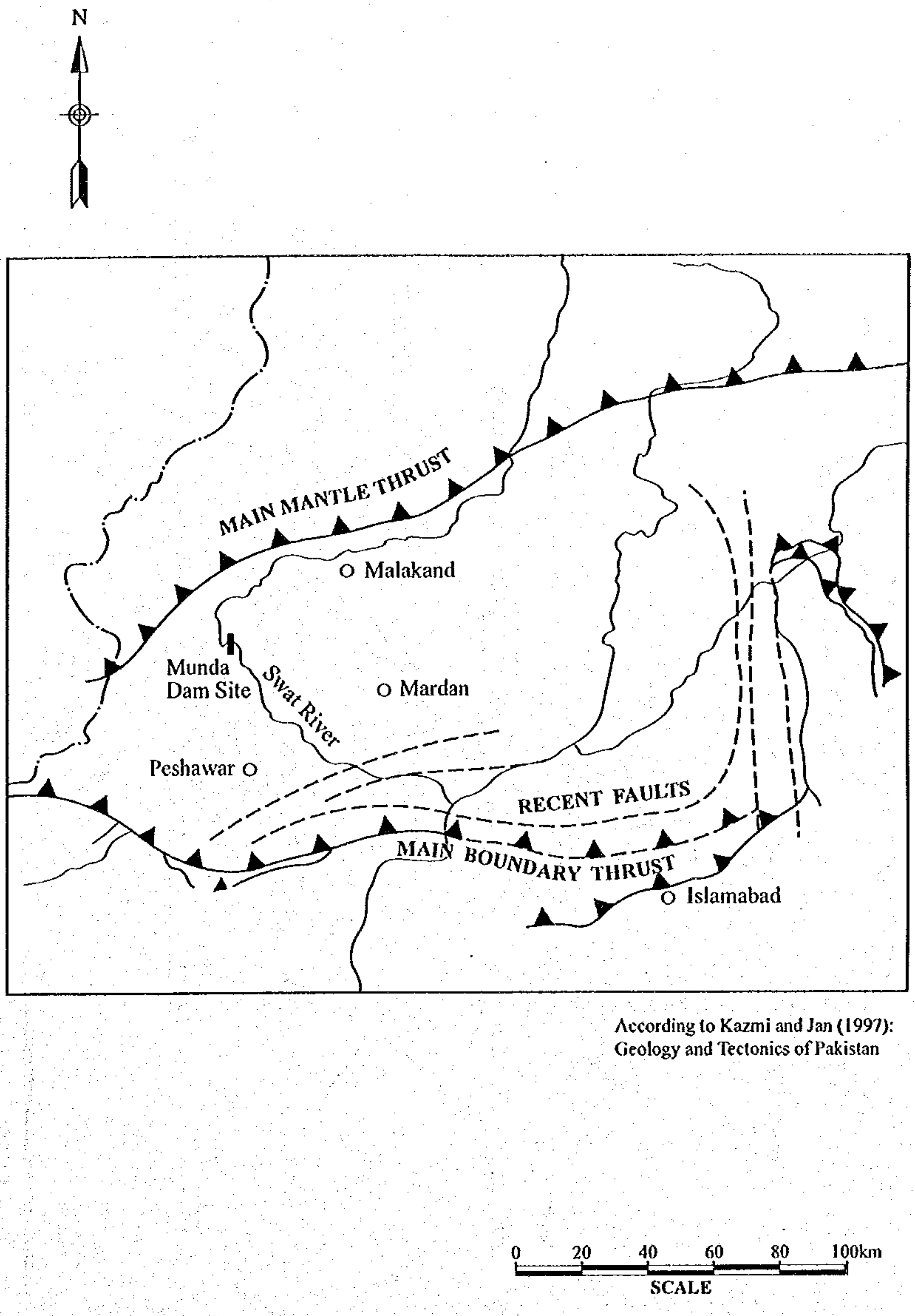
	A					B					C				
	1	2	3	4	5	1	2	3	4	5	1	2	3	4	5
a	B	B	CH	CH	CM	B	CH	CH	CM	CM	CH	CH	CH	CM	CL
b	B	CH	CH	CM	CL	CH	CH	CM	CM	CL	CH	CH	CM	CL	CL
c	CH	CH	CM	CL	CL	CH	CM	CM	CL	D	CH	CM	CL	CL	D
d	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

	D					E					F				
	1	2	3	4	5	1	2	3	4	5	1	2	3	4	5
a	CH	CH	CM	CL	D	CM	CM	CL	CL	D	CM	CL	CL	D	D
b	CH	CM	CM	CL	D	CM	CL	CL	CL	D	CL	CL	D	D	D
c	CM	CL	CL	D	D	CL	CL	CL	D	D	D	D	D	D	D
d	-	-	-	-	-	D	D	D	D	D	D	D	D	D	D

Table B8.3 Expectable Physico-mechanical Properties

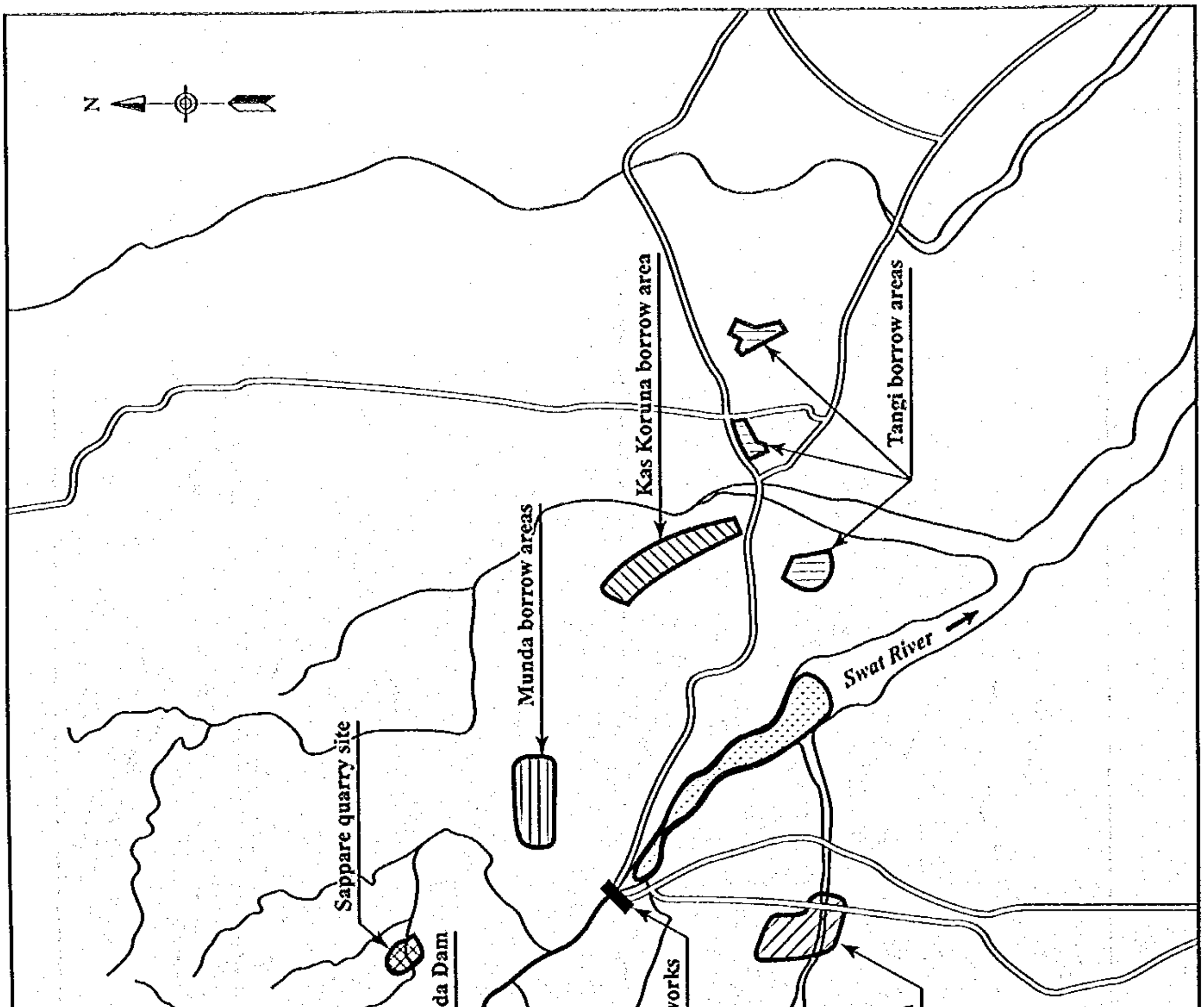
Rock classification	Modulus of deformation (kgf/cm^2)	Modulus of elasticity (kgf/cm^2)	Shear strength: Cohesion (kgf/cm^2)	Shear strength: Int. friction angle (°)	Seismic velocity (km/sec)	Repulsiveness through rock hammer test
A, B	50,000 -	80,000-	40-	55-65	3.7-	36-
CH	20,000-50,000	80,000-40,000	20-40	40-55	3.0-3.7	27-36
CM	5,000-20,000	15,000-40,000	10-20	30-45	1.5-3.0	15-27
CL, D	-5,000	-15,000	-10	15-38	-1.5	-15

FIGURES



According to Kazmi and Jan (1997):
Geology and Tectonics of Pakistan

FEASIBILITY STUDY ON THE DEVELOPMENT OF MUNDA DAM MULTIPURPOSE PROJECT IN ISLAMIC REPUBLIC OF PAKISTAN	Figure B1.1
JAPAN INTERNATIONAL COOPERATION AGENCY	Tectonic Sketchmap of Project Site



FEASIBILITY STUDY ON THE DEVELOPMENT OF MUNDA DAM MULTIPURPOSE PROJECT IN ISLAMIC REPUBLIC OF PAKISTAN	**Figure B2.1** **Location Map of Project Area**
JAPAN INTERNATIONAL COOPERATION AGENCY	