Appendix B :
Geological Investigation

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Appendix B Geological Investigation

B1 Introduction

The Project Site is located about 30 km (aerial) South west of Kathmandu Valley and is located in Makwanpur District. The project site is situated nearby the Tribhuvan Highway in Bhainse and Kamalmatta.

The geological and geotechnical investigations include the core drilling, standard penetration tests, water pressure tests, seismic refraction as well as electric resistivity prospecting, in-situ rock testing such as plate loading test and block shear test, and laboratory tests. The laboratory tests include the measurement o the uniaxial compressive strength of core samples, bulk density, water absorption, ultra sonic wave velocity and tensile strength.

Total quantity of the geological as well as geotechnical investigations are as follows.

(1) Core Drilling	26 holes	1,272	m
(2) Standard Penetration Test	14 holes	101	times
(3) Water Pressure Test	11holes	68	stages
(4) Seismic Refraction Prospecting	5 lines	1,245	m
(5) Electrical Resistivity Prospecting	3 lines	1,750	m
(6) In-situ Rock Test			
Plate Loading Test		3	spots
Block Shear Test		3	spots
(7) Laboratory Test	20 holes	523	pcs

B2 Regional Geology

B2.1 General

B2.1.1 Topography

Nepal is situated in an active tectonic zone developed in the southern foothills of the Himalayan ranges where the Indian plate submerges northward under the Eurasian plate. Therefore, the geological setting of Nepal is characterized by discernible features of geomorphology. Himalayan ranges exist in the north and lowland plains occur near the southern border with India.

Nepal is composed of four topographical divisions bounded by major WNW-ESE trending thrust faults. Higher Himalayan mountains of more than 8,000 m in elevation lie in the north, Lower and Sub Himalayan ranges are located adjacent to its southern boundary in the central, and Lowland plains lie in the southernmost part of Nepal. These mountain ranges and plains are bounded by major WNW-ESE trending thrust faults.

B2.1.2 Geology

The Higher Himalayas are bounded by the Main Central Thrust (MCT) from the Lower Himalayas, and the Lower Himalayas are separated by the Main Boundary Thrust (MBT) from the Sub Himalayas. Himalayan Front Thrust (HFT) lies in the boundary between these Himalayas in the north and Lowland Plains in the south.

Geology of the zone in between the thrust faults is, in general, younger strata in the north and older in the south . Higher Himalayas consist mainly of metamorphic rocks of Pre-Cambrian to Paleozoic age. Lower Himalayas consists chiefly of unfossiliferous meta-sedimentary rocks of Pre-Cambrian to Paleozoic age with remnants of the Higher Himalayas. Sub Himalayas of Tertiary Siwalik sandstone extends in the south of MBT and form the southern-most belt of the Himalayas. (See Figure B2.3.1)

B2.1.3 Tectonic Setting

Nepal is located in an active tectonic zone as mentioned above, and the geology of Nepal is chiefly bounded by these major thrust faults, such as Main Central Thrust (MCT), Mahabarat Thrust (MT), Main Boundary Thrust (MBT), and Himalayan Front Thrust (HFT) from north to south. Some sections of the Mahabarat Thrust lie on the Main Central Thrust (MCT). Distribution of these thrust faults is not always linear, but circular due to folding in some localities.

B2.2 Geology of the Project Site

B2.2.1 Topography

The project area is located at around 30km southwest of Kathmandu in the zone of Lower and Sub Himalayas. The project area is situated in the Rapti River basin, which is adjacent to south of Kulekhani river basin. The Rapti river originates in a ridge on the Mahabarat ranges around 4 km east of Bhimphedi, and flows southwest to southward. The riverbed elevation of the Rapti River is 1,100m at Bhimphedi, 600m at Bhainsedobhan and 500m at Hetauda. Mountain ranges with a highest peak of 1,373m are confined in the direction of WNW to ESE reflecting geology of the area. The Rapti River has N-S trend and flows down to south dissecting the said mountain ranges in the area.

B2.2.2 Geology

The project area consists of Redua Formation, Bhainsedobhan Marble, Kalitar Formation of Bhimpedi Group, Benighat Slate, Malekhu Limestone, Robang Formation of Upper Nawarkot Group and Siwalik Group overlain by Quaternary deposits of limestone breccias, terrace deposits, riverbed deposits and scree deposits.

Strike of formations is generally WNW-ESE. The boundary between Upper Nawakot Group and Bhimphedi Group is called the Mahabharat Thrust, and Upper Nawakot Group and Siwalik Group are bounded by the Main Boundary Thrust.

Geological Map in and around the project area is shown in Figure B2.2.2, and Figure B.2.2.3. Geological profile along waterway is shown in Figure B2.2.4.

Stratigraphy of the project area is described as follows.

<u>Raduwa Formation</u> consists of coarse-crystalline, highly garnetiferous mica schist, gneissic schist. Some quartzites are also seen in this formation.

<u>Bhaise Dobhan Formation</u> comprises coarse crystalline marble, limestone with intercalation of thin schist. Marble and limestone are massive and well bedded.

<u>Kalitar Formation</u> is composed of dark green to gray colored two mica and biotite schist with intercalation of strongly micaceous quartzite and amphibole containing some garnets.

<u>Benighat Formation</u> comprises dark gray slates and phyllites together with black carbonaceous slate. Intercalation of Malukhu limestone are often seen in this formation.

<u>Malekhu Formation</u> consists mainly of light-to-dark and greenish gray siliceous dolomites. Thin bands of fine crystalline limestone and calc-phyllites intercalate in dolomites.

<u>Robang Formation</u> comprises Lower Robang and Upper Robang phyllites, and Dunga Quartzite. Robang phyllite consists of blue green phyllites and phyllites are chloritic in general. Dunga Quartzite is mainly composed of quartzite. Intercalation of thin phyllite beds are seen in this quartzite at some localities.

<u>Siwalik Group</u> consists of sandstone, mudstone, and small portion of conglomerates.

Recent Deposits is composed of riverbed deposits, talus deposits and terrace deposits. Decomposed limestone beds are also seen sporadically in higher elevation forming small terraces bounded by steep escarpment. These deposits are distributed over the said bedrocks.

River deposits **:** River deposits are mainly composed of non-consolidated sands and gravels.

Talus **:** Talus and terrace deposits consist loosely of well consolidated sands and gravels. Decomposed limestone beds comprises chiefly of cobble to boulder sized limestone. The decomposed limestone has a silt or sandy matrix and calcareous at places. (See Figure B2.2.5)

Stratigraphy of Project Area

AGE	GROUP	FORMATION	SYMBOL	ROCK TYPE	GEOLOGY
Cenozoic	Recent Deposits		Rd	Riverbed deposits	Sand and gravels with bolders
			Ta	Talus and/or Terrace	Talus deposits and terrace deposits.
		(Uncomformity)			
	Siwalik Group		Sw	Conglomerate, Sandstone, Mudstone	Sandstone, mudstone, and small portions of conglomerates. Relatively soft and fractured near MBT.
		(Main Boundary Thrust)			
Paleozoic	Upper Nawakot Group	Robang Formation	Phy (2)	Phyllite (2)	Blue green slatic phyllites, generally chloritic. Intercalation of calcalious beds. Relatively compact in general.
			Qz	Quartzite	Quartzite. Intercalation of thin phyllite at some localities. Massive and compact in general.
			Phy (1)	Phyllite (1)	Blue green phyllites, generally chloritic. Relatively compact in general.
		Malekhu Formation	Dl	Siliceous Dolomite	Light-to-dark and greenish gray siliceous dolomites. Intercalation of thin crystalline limestone and calc-phyllites. Massive and relatively well bedded.
		Berighat Formation	Sl	Slate(Phyllitic)	Dark gray slates and phyllites together with black carbonaceous slate. Fractured and weathered near MBT.
		(Mahabharat Thrust)			
Pre-Cambrian	Bhimphedi Group	Kalitar Formation	Sq	Schist, Quarzite	Dark green to gray colored two mica and biotite schist with intercalation of quartzite and garnets. Strongly folded and fractured at places.
		Bhaise Dobhan Formation	Mb	Limestone	Coarse crystalline marble, limestone with intercalation of thin schist. Marble and limestone are massive and well bedded.
		Raduwa Formation	Sch	Schist	Coarse-crystalline, highly garnetiferous mica schist, gneissic schist. Some quartzites are also seen in this formation.

(3) Geological Structure

The project area consists of Precambrian rock of Bhimphedi group of Kathmandu Nappe and Paleozoic rocks of Upper Nawarcot group as well as Cenozoic Siwalik group. The Mahabharat Thrust (MBT), which is said to be the basement thrust of Kathmandu Nappe, and the Main Boudary Thrust (MBT) are distributed in the area. (See Figure B2.2.6)

1) Main Boundary Thrust

The Main Boundary Thrust froms the boundary between Lower Himalaya and Sub Himalaya. Siwarik sandstone of folded and faulted the Tertiary sedimentary rock have been overthrusted in the south of the Main Boudary Thrust (MBT).

2) Mahabharat Thrust

The Mahabharat Thrust can be considered as an extension of the Main Central Thrust (MCT). The Main Central Thrust (MCT) is formed due to collision of Indidan plate during Cenozoic. Thrusting of Main Central Thrust (MCT) appears to begin at 50 million years ago and continue today. The rate of northward movement is considered to be 5cm/year in recent years.

3) Discontinuity

Discontinuities of the rock mass are highly developed in the project area. In general, surface of the continuities of quartzite, dolomite and sandstone are fresh, rough and open, however those of phyllite and slate are weathered, relatively smooth, closed or partly filled with fine materials. Discontinuities along major thrusts of the Mahabharat Thrust and the Main Boundary Thrust are slickensided and filled with clayey materials in general. Some discontinuities of major thrusts are recrystallised with calcareous or siliceous materials.

B3 Geological Investigation

B3.1 General

The geological and geotechnical investigation works have been executed from March 11th to September 15th 2002, in accordance with the technical specifications for this study. The methodology of the work and the results of the investigations are presented in this section of the supporting report.

B3.2 Core Drilling

B3.2.1 Methodology

The core drilling, with standard penetration tests and water pressure tests, are performed to obtain geo-technical data about the sub-surface conditions of the sites of the headworks, regulating dam, connection tunnels, underground powerhouse, tailrace tunnel and bridges, etc.. The core drilling of 1,272 m in total length was performed at 26 locations. The location of the drilling holes and number of tests performed for the project are shown in Figure B3.1.1 and the simplified drill logs are shown in Figure B3.1.2.

The work has been performed in accordance with the plan and the specifications for this project. Core drillings are made for the bedrock, soil, gravel deposits, colluvial deposits and talus deposits those may contain boulders.

B3.2.2 Equipment and Material

The core drillings are performed by use of hydraulic driven rotary machine, at the locations, in the directions and up to the depth as specified. The equipment used for core drilling are summarized as follows :

Equipment Used in Core Drilling

S.No.	Hole No.	Location	Drilling Machine	Remarks
1.	BI – 2	Khani Khola Intake	U-D5 TONE	
2.	BCT – 1	Connection Tunnel alignment	U-D5 TONE	
3.	BCT – 2	Connection Tunnel alignment	U-D5 TONE	
4.	BO – 1	Outlet of Connection Tunnel		
5.	BMT – 1	Connection Tunnel alignment	U-D5 TONE	
6.	BD – 4	Yangrang Khola	KOKEN	
7.	BD – 5	Yangrang Khola	KOKEN	
8.	BD – 6	Yangrang Khola	KOKEN	
9.	BD – 7	Yangrang Khola	KOKEN	
10.	BD – 8	Yangrang Khola	KOKEN	
11.	BD – 9	Yangrang Khola	KOKEN	
12.	BD – 10	Yangrang Khola	KOKEN	
13.	LS – 1	Yangrang Khola	KOKEN	
14.	LS – 2	Yangrang Khola	KOKEN	
15.	BS – 1	Salumtar Village	JOY	
16.	BPV – 1	Top of Power House	U-D5 TONE	
17.	BPV – 2	Top of Power House	VOLT – 35	
18.	BPV – 3	Top of Power House	ACKER	
19.	BPH	Power House	U-D5 TONE	Horizontal
20.	DHT – 4	Keshadi Khola	VOLT – 35	
21.	DHT – 6	Keshadi Khola	VOLT – 35	
22.	BTO – 1	Nakauli Khola	KOKEN	
23.	BA – 1	Left Bank of Rapti River	U-D5 TONE	
24.	BA – 2	Right Bank of Rapti River	U-D5 TONE	
25.	BP – 1	Rapti River	VOLT – 35	

The recovered core samples are placed in order in the core boxes and have been stored in the NEA office of Nibuwatar. Each core box consists of five grooves, each groove with adequate dimensions for containing one metre of the core section. Accordingly, every core box contains core samples of 5 m section.

The core samples are placed in order, in the same length of grooves of the core box as the length, which has been drilled to obtain those core samples. Parts of no core recovery are left vacant in the grooves. Marks are put regularly to the grooves to indicate depths of sampling. Every core box is marked with the borehole number and depth of the section of which the core samples are put in it.

Water level in boreholes are measured and recorded every morning before commencement of the day's drilling work. Such measurement has been continued during the period when the hole is being drilled. Daily water level was measured and described in the drill logs.

B3.2.3 Installation of Screen pipes

A line of PVC screen pipes is installed in the boreholes as listed below for a long term monitoring of the groundwater table. The inside diameter of the pipe is 40mm . For filtering effect, granule sand are filled in the space between the hole and the pipe, or the pipe are wrapped with pervious fabrics or thin jute cloth.
The top of the pipe has been protruded to the air and is kept higher than the ground surface at least by 60 cm. The pipe on the ground surface are fixed with concrete and locked in a cover for protection. The sign are indicated to the concrete to show the hole number and depth. The details of screen pipe installed are summarized as below:

B3.2.4 Result

The results of core drillings were utilized for preperation of the geological figurees and summarized in the form of drill log sheets attached in the Attachment to this Appendix B.

Summary of Stand Pipe Peizometer Installation

S.No.	Hole No.	Length of Screen Section	Length of Stand Pipe (meter)	Remarks
1.	BCT – 1	4.0	40.2	
2.	BCT – 2	6.0	40.6	
3.	BO – 1	14.0	21.0	
4.	BMT – 1	4.0	80.2	
5.	BD – 4	47.0	51.0	
6.	BD – 5	45.00	51.0	
7.	BD – 6	31.0	41.0	
8.	BD – 7	28.0	31.0	
9.	BD – 8	22.0	31.0	
10.	BD – 9	27.0	31.0	
11.	BD – 10	25.0	31.0	
12.	LS – 1	6.0	31.0	
13.	LS – 2	16.0	31.0	
14.	BS – 1	75.0	111.0	
15.	BPV – 1	113.5	116.0	
16.	BPV – 2B	76.0	78.4	
17.	BPV – 3	36.0	100.3	
18.	DHT – 6	58.0	61.0	
19.	BTO – 1	13.0	21.0	
20.	BA – 1	10.0	30.3	
21.	BA-2	10.0	30.3	
22.	BP – 1	25.0	41.0	
23.	BPH – 1	59.0	61.0	
24.	BI – 2	4.0	30.2	

B3.3 Standard Penetration Test

B3.3.1 Methodology

The standard penetration tests, in accordance with the USBR specification (Earth Manual) or the equivalent, are carried out every 1.5 metres of depth or is indicated in the sections of the boreholes which are located within soils or un-cemented deposits or intensively weathered rocks, in order to evaluate the mechanical strength of those materials.

The tandard penetration tests are performed in the sections of the hole that are drilled through un-consolidated deposits or intensively weathered rock. The tests are done every 1.5 metre intervals in depth till the borehole encounters bedrock which is hard enough to require more than 50 times' blow of the test hammer for 30 cm penetration of a split-tube sampler. The SPT value hence N value is noted by dropping a 63.5 kg hammer freely from a height of 75 cm to penetrate last 30 cm soil. The number of blows required to drive this distance is recorded and bearing capacity and/or the relative density of the soil mass can be assessed.

The results of the tests are recorded in the daily report of drilling in number of blows for every 10 cm of penetration of the 30 cm long test drive. The soil samples collected in the split-tube during the penetration tests are put in plastic bags and into the core box at the corresponding depth.

B3.3.2 Result

Results of tests are summarized in Table B3.3.1 and the drill logs compiled in the Attachment of this Appendix B.

B3.4 Water Pressure Test (Lugeon Test)

B3.4.1 Methodology

The water pressure tests were carried out, in accordance with the following the "Lugeon test" procedure of the descending stage method, for every five metre section, in the parts of boreholes through bedrock in order to evaluate the seepage potential of the foundation rocks.

The water pressure test in the bedrock, (the Lugeon test), were performed in the sections of the borehole passing through bedrock by 5m long stage in descending order, sealed by the packer. The test was conducted as follows:

When a borehole had been drilled to the depth of bottom of a section to be tested in the bedrock, it was washed inside by flushing water through the drill rod inserted to the bottom of the hole. When the returning water becomes clean, a packer was installed at the top of the 5 m long test section and water was pumped into the section through the injection pipe. Under a certain water pressure with regulated constant head, the water injection rate was observed for 10 minutes. Through this 10 minutes' observation period, the injected quantity of water was observed and recorded every minute. This procedure was repeated under varied pressures, 1 kgf/cm^2 (0.1 MPa), 4 kgf/cm^2 (0.4 MPa), 7 kgf/cm^2 (0.7 MPa), 10 kgf/cm^2 (1.0 MPa), 7 kgf/cm^2, 4 kgf/cm^2 and 1 kgf/cm^2, in general.

The water level in the borehole and the relative height of the pressure gauge from the neck of the hole was recorded in each test.

Upon the completion of the above observation the drilling are resumed for another 5 m. The new 5 m section is again tested by the same procedure as above.

In case that the pressure did not rise up to the designated maximum at an injection rate of 100 liters per minute because of high leakage potential in the test section, the test was made only for the attainable pressures.

Water to be injected for the test was sufficiently clean.

The results are presented in coefficient of permeability and Lugeon unit, as calculated by the following formulae:

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

$k = (q \times 10^3 \times \sinh^{-1}(L/2r)) / (2LH \times 60)$ when $10r > L > r$

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

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) $H = A + B + C - H_f$

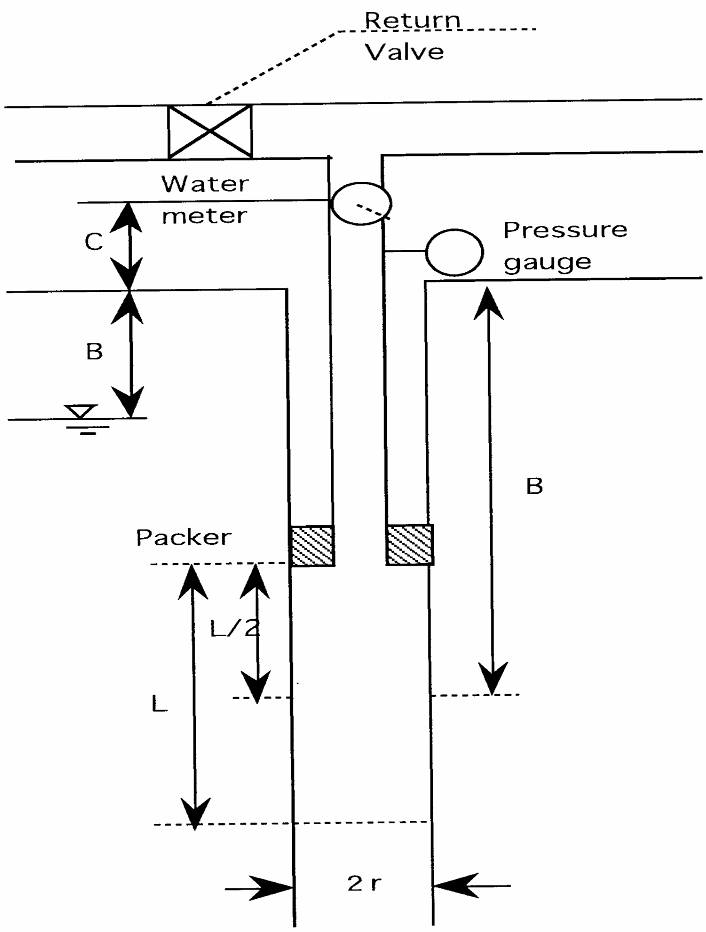
A : Pumping pressure head (cm)

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).

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

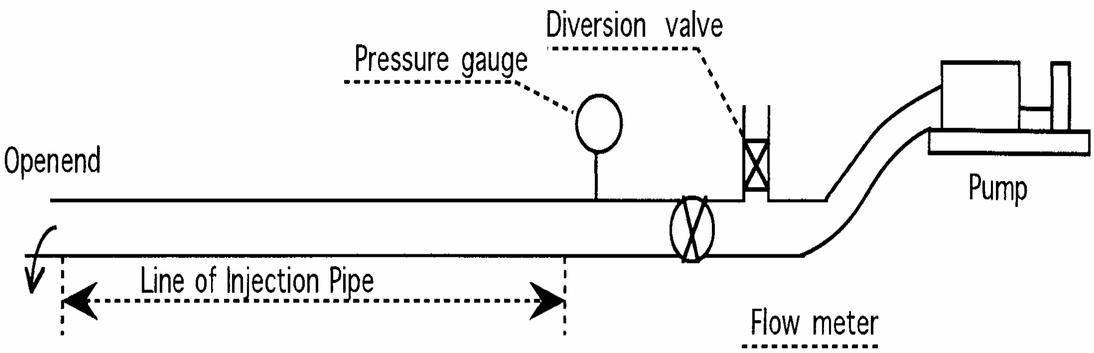
Q : Water injection rate (litre/min.)



B3.4.2 Measurement of Friction Energy Loss

The measurement of the friction energy loss was done in the following manners. When a large quantity of water is injected through the injection pipe, energy of flow will be lost due to friction with the wall of the pipe. Thus the water pressure indicated by a pressure gauge at the neck of the hole will not represent the real pressure built in the test section, near the bottom of the hole. Correction of the water pressure test results will be necessary. To estimate the decreased head (Hf) due to the friction loss, the following test are done, using injection pipes similar to those actually used for the water pressure test in the site.

The injection pipes are connected to form a line of about 80 m length. One end of the line shall be left open and the other end, on which a pressure gauge will be fixed, shall be connected to the hose coming from a pump. A water flow meter is attached between the pressure gauge and the pump. To control the pressure, a diversion valve is attached between the flow meter and the pump. The line of injection pipes is laid on the ground, horizontally.



At first, water is pumped through the line in full capacity with the diversion valve closed. The water flow rate and the pressure at the pressure gauge are recorded. Then the water flow is decreased and the pressure lowered, by opening the diversion valve. The pressure and the flow rate are recorded again. In the same manner, by opening the diversion valve to various extents, the flow rates (l/min) and the corresponding pressure (kgf/cm^2 or MPa) are recorded and plotted on a graph. The operations are repeated 5 times, with measurement of the flow rate every 5 minutes.

Similar observations are done for varying length of the injection pipeline: e.g., 60 m, 40 m and 20 m, approximately.

The flow rate vs. pressure graph obtained gives the decreased head (Hf) for a certain injection rate and a certain length of the injection pipe line.

B3.4.3 Result

Results of water pressure tests are summarized in Table B3.4.1 and figures (P~Q curves) have been presented in the attachment of this Appendix - B.

B3.5 Seismic Refraction Prospecting

The geophysical explorations were carried out to determine the sub-surface geotechnical condition and the location of weak zones etc.

The seismic refraction prospecting were performed with four (4) traverse lines and for 1,245 meters of the total plan length, as indicated in Figure B3.5.1.

B3.5.1 Methodology

The seismic refraction prospecting were carried out in the connection tunnel and tailrace tunnel of the project in order to obtain geological and geotechnical information by classifying the sub-surface condition on the bas is of the difference in velocity of the seismic wave propagation.

B3.5.2 Equipment and Material

The equipment used are listed below:

Equipment and Accessories for Seismic Refraction Prospecting.

S.No.	Equipment Name	Quantity	Remarks
1.	Exploration, Seismograph – EG & G Geometric ES 1225	1 pcs.	
2.	No. of Geo-phone	12 pcs.	
3.	Car Battery	1 pcs.	
4.	Spread Cable – 130 m long	1 spool.	
5.	Hammer Switch	3 pcs.	
6.	Hammering Cable – 130m long	1 spool.	
7.	Ordinary Electric Cable	1 spool.	
8.	Hammering Pad	1 pcs.	
9.	Sledge Hammer – 5 Kg.	1 pcs.	
10.	Measuring Tape 50 m	1 pcs.	
11.	Telescopic Compass with Tripod	1 pcs.	
12.	Staff	1 pcs.	
13.	Tool Box	1 pcs.	

B3.5.3 Field Operation

The field operation was done in the following procedure.

(1) Setting of Prospecting Traverse

As per the selected profile line, the geophones are placed at the 5m spacing along the ground surface and connected with takeout cable and seismograph.

At each spread, there are five to nine sets of shooting: two to four forward shooting in various offset distances, mid-shooting and two to four reversed shooting in various offset distances. Each shooting gets a set of waveform records in hard copy.

(2) Profile Survey

Each geophones and shooting points are surveyed from the known benchmarks with the help of telescopic compass. Ground height of every detector point is surveyed accurately by leveling to draw a topographic profile of every prospecting traverse line to the scale of 1/1000.

(3) Shooting

For the seismic sources, explosives are blasted off. The signals are enhanced by means of stacking method. Shooting is made effectively and safely with subsurface explosion in hand-dug pits or augur holes, by use of dynamite and instantaneous electric detonators. Prior to blasting, adequate warning are given to all people, whether of the project or the public, staying within a distance of 50 metres from the blasting point.

(4) Detecting

Detectors or geophones are allocated at a regular interval of 5 metres in each spread on the prospecting traverse line. For next spread of the same profile line, the geophones are again placed at 5m spacing. The work process is repeated from 2 to 5. The works are continued up to desired profile length of investigation.

(5) Recording

Recording of every shooting are reviewed at the site by plotting tentative time-distance graphs. When any record is not clear or questionable, the shooting and recording shall be made again. Ends of every spread are overlap with ends of the adjoining spreads for continuity of the records over a prospecting traverse line.

B3.5.4 Data Interpretation

From each wave from the record, the first arrival times are picked up for each geophone. These arrival times are plotted in Travel Time – Distance Graph. The scale of graph should be proper matching. Normally one cm is 10 millisecond at time scale and 5m. at distance scale.

Travel time distance graph is plotted as per seismic profile line, which might consists of one spread or several spreads.

Travel time distance graph will clearly show the forward shootings, mid-shooting and reversed shooting.

At the travel time distance graph, the best-fit lines are drawn. Each line represents a layer. The slope of the line is reciprocal of the apparent velocity of that particular layer. The depth and thickness of every layer are determined by the intercept time method and ABC methods, which are quite commonly used.

Depending upon the density, water content and porosity, the material can have the different compressional seismic velocity. The velocity of different material is given in Table below:

Compressional Seismic Velocity and Its Corresponding Materials

Velocity m/s	Material Description
340	Air
1470	Water
3200	Ice
3100	Concrete
5900	Steel
200 – 400	Soft unconsolidated surface deposit
400 – 1500	Unconsolidated clays and silts, unsaturated sands and gravels
1500 – 2000	Saturated sands and gravels, compacted clays and silts, completely weathered rocks
2000 – 2500	Partially consolidated sediments, probably water saturated; highly weathered / fractured metamorphic and igneous rock; weathered and / or jointed sandstones and shales
2500 – 3700	Partially weathered to fresh shales and sandstones; weathered and / or sheared metamorphic, igneous or limestone rocks
3700 – 4500	Slightly weathered and / or fractured metamorphic or igneous rocks or limestone; some very hard or indurate sandstones and shales
4500 – 6000	Un-weathered metamorphic and igneous rocks; some limestone and dolomites

B3.5.5 Result

The results including the time – distance curves and the velocity layer profiles are summarized in the Figure B3.5.2 to B3.5.5.

a. Connection Tunnel

<u>BCT-1 to BCT-2 Section</u>

Other geological data obtained through the site reconnaissance as well as the core drilling are considered when the velocity layers are analyzed. The velocity layers consist of four different velocity layer along connection tunnel.

Maximum velocity zone is 5.5 km/sec and lower velocity layer are analized to be as follows.

First velocity layer	0.3 km/s
Second velocity layer	0.6 km/s to 0.8 km/s
Third velocity layer	1.2 km/s to 1.4 km/s
Fourth velocity layer	2.2 km/s to 2.4 km/s
Fifth velocity layer	5.5 km/s

Low velocity zones which are the segments of remarkably low speed such as 1.5 km/sec to 2.2 km/sec zone were observed in the highest velocity zone of 5.5 km/s. The low velocity zone may represent defect of the bedrock such as a fractured zone and other anomalies. In the case of the connection tunnel, several low velocity zones are distributed and they are regarded as fractured zone of Mahabharat Thrust and adjacent fractured zones.

As the result of the seismic exploration, the top soil or overburden appears to be thin of less than 10m and the baserock along the connection tunnel route appears to be hard and compact along the tunnel route. However, several fractured zones might be encountered at lithological boundaries including the Mahabharat Thrust during the excavation work.

<u>Outlet of Connection Tunnel</u>

The other geological data of geological reconnaissance and drill logs are considered to analyze velocity. Three different velocity layers are confirmed near the outlet of the connection tunnel.

Maximum velocity zone is in the range from 3.0 km/sec to 3.3 km/sec and lower velocity layer are analised to be as follows:

First velocity layer	0.3 km/s	to	0.4 km/s
Second velocity layer	1.5 km/s	to	1.7 m/s
Third velocity layer	3.0 km/s	to	3.2 m/s
Fourth velocity layer	5.8 km/s	to	6.0 km/s

As the result of the seismic exploration, the top soil or overburden appears to be thin of less than 5 m, however the baserock at the outlet of the connection tunnel is assumed to be relaxed up to 15 m in depth from the ground surface. The rock below this relaxed zone appears to be hard and compact like other section of the connection tunnel.

b. Tailrace Tunnel

<u>Outlet of Tailrace Tunnel</u>

Velocity layers consist of two along connection tunnel. Maximum velocity zone is 2.8 km/sec and lower velocity layer are analized to be as follows.

First velocity layer	0.4 km/s	to	0.5 km/s
Second velocity layer	1.2 km/s	to	1.4 km/s
Third velocity layer	2.7 km/s	to	2.8 km/s

As the result of the seismic exploration, the top soil or overburden appears to be thin of less than 5 m and the baserock along the tailrace tunnel is assumed to be less compact than that of the connection tunnel.

B3.6 Electrical Resistivity Prospecting

B3.6.1 Methodology

The horizontal two dimensional (2D) electrical resistivity prospecting were carried out in the tailrace tunnel, and underground power house structures in order to obtain geological and foundation engineering information by classifying the sub-surface ground on the basis of difference in electrical resistivity. It provides overall picture of the subsurface soil stratifications, ground water table and degree of saturation, any fluid contents and contamination, porosity, permeability and coefficient of filtration, resistivity value of soil strata, mineralized zone, condition of foundation and detect depth of sound rock, locations of weak zones, faults, etc.

The horizontal 2D electrical resistivity prospecting consists of data collection by conventional electrical resistivity prospecting and sophisticated analysis of data obtained. The analysis for the horizontal 2D electrical resistivity prospecting is composed of modeling geological structure and mathematical inversion analysis on the basis of the model made. Computerized analysis is common in general.

The electrical resistivity prospecting is performed with one traverse line of 350 m in length along the tailrace tunnel. The underground powerhouse structures was investigated with two traverse lines of 700 m in length. The total length of survey line is 1750 m.

B3.6.2 Equipment and Material

The equipment used are indicated below.

Table B3.6.1 Equipment for Electrical Resistivity Prospecting

S. No.	Equipment Name	Quantity	Remarks
1.	TERRAMETER SAS 300C	1 Set.	
2.	BOOSTER 2000	1 Set.	
3.	Rechargeable batteries (4 AH)	4 Pcs.	
4.	Battery Charger	1 Pcs.	
5.	Steel Electrodes (1m long)	40 Pcs.	
6.	Connection Clips	30 Pcs.	
7.	Computer	1 Pcs.	
8.	Floppy Disks	10 Pcs.	
9.	Electric Cables	4 Rolls.	
10.	Cables of different Length	15 Pcs.	
11.	DC Cable	20 Rolls.	

B3.6.3 Field Operation

The field operation was done in the following manners

(1) Setting of Prospecting Traverse

The ground surface profile of every traverse line is surveyed, and all arrangement lines are marked with distance from an end of each traverse line. The field prospecting work is made spread by spread until all the length of each traverse line is covered.

(2) Profile Survey

Ground height along the line is surveyed accurately by leveling to draw a topographic profile of every prospecting traverse line to the scale of 1/1000. The wooden pegs are place at 5m spacing throughout the whole line of investigation. Each pegs are surveyed from the reference points with the help of EDM.

(3) Recording

Record of every line are reviewed at the site. When any record is not clear or questionable, the recording are made again. Ends of every spread need overlap with ends of the adjoining spreads for continuity of the records over a prospecting traverse line. For the resistivity measurement, any one array as mentioned above can be used. Pole-Pole array gives maximum points at depth. So pole-pole array will be adopted. First set of resistivity measurement is carried out at the 5m electrode spacing at every pegs along the profile line. Next set of readings is at various electrode spacing:

10m, 15m, 20m, 25m 30m, so on.

B3.6.4 Data Interpretation

Resistivity data are processed in two stages for 2D resistivity imaging – electrical resistivity topography: least squares inversion model and robust inversion model. Three different variations of the least squares method are used: a very fast quasi-Newton method, slower but more accurate Gauss-Newton method, and a moderately fast hybrid technique with combination of quasi-Newton and Gauss-Newton method. For these analysis, a computer software is required as it involves rigorous mathematical calculations and alternation of calculated apparent resistivity values that agrees with the measured values from the field survey. It seems that there are several type of computer software developed by different companies. As for example: RES2DINV, has been used.

The record are plotted on the graphs, and then interpreted into profiles of electrical resistivity layers. The apparent resistivity pseudosection is plotted in graph paper in order to evaluate the field data.

The electrical resistivity layers are shown in profiles, using the ground surface profile prepared by the profile survey.

The electrical resistivity layers distinguished are geologically and geo-technically interpreted in correlation with the findings in the surface geological mapping, the core drilling, the test pitting, etc.

Depending upon the mineral composition, porosity, water content and fluid content, the material can have wide range of resistivity value. The resistivity of general rocks, minerals and chemicals is given in Table.

Resistivity of some Common Rocks, Mineral and Chemicals

Material	Resistivity (Ω.m)
Igneous and Metamorphic Rocks	
Granite	5,000 to 1,000,000
Basalt	1,000 to 1,000,000
Slate	600 to 40,000,000
Marble	100 to 250,000,000
Quatzite	100 to 200,000,000
Sedimentary Rocks	
Sandstones	8 to 4,000
Shale	20 to 2,000
Limestones	0 to 400
Soils and Waters	
Clay	1 to 100
Alluvium	10 to 800
Fresh Ground Water	10 to 100
Sea water	0.2
Chemicals	
Iron	9.07×10^{-8}
0.01 M Potassium Chloride	0.708
0.01 M Sodium Chloride	0.843
0.01 M Acetic Acid	6.13
Xylene	6.998×10^{16}

B3.6.5 Result

Using the geological concept and knowledge of resistivity of the different formations, the following ranges of the resistivity values are given for the materials found in the study area.

Geological Descriptions	**Resistivity Ohm.m**
Graphite, Graphite slate and Phyllites	1 – 100
Disturbed and Crushed Siwalik Formation	100 – 200
Intact Siwalik Rock	> 300
Dolomite (Lesser Himalayan Rock)	> 2000
Unsaturated granular material, near Surface	> 300

The interpretation of each section is summarized in the related figures. The location is shown in Figure B3.6.1. The results of the prospecting were shown in Figure B3.6.2 to B3.6.4.

A short description of each section is provided in the text below.

Tailrace Tunnel (ERP-1)

The profile was conducted over the contact between Lesser Himalaya and Siwalik. The section shows highly conductive graphitic slates in one side of the profile and highly disturbed Siwalik Formation on the other side. The end of the profile indicates intact bedrock of the Siwalik Formation. Unsaturated granular material (mostly coarse-grained) near the surface is indicated by high electrical resistivity. The section is presented as Figures.

Powerhouse Area (ERP-2 & ERP-3)

Powerhouse Area was investigated by two profiles, which crosses each other near the ridge. Dolomite beds are mapped by virtue of its high electrical resistivity in comparison to graphitic slates and phyllites. The results of each section are summarized in the related tomograms. The electrical resistivity modeled in profile ERP-2 is higher than in ERP-3. This may be due to the effect of the anisotropy in the resistivity. The resistivity measured across the foliation is higher than measured parallel to the foliation. From the observation near the foothills it is expected that the Profile ERP-2 may run almost across the foliation plane whereas ERP-3 may run to some degree parallel to the foliation plane. Furthermore, there could be minor 3D effect of the ridge in the section ERP-2. However, the resistivity contrast between different formations is high enough to map different zones to a good degree.

B3.7 In-situ Rock Test

B3.7.1 General

The In-situ rock test were performed in the branch adit of the exploratory adit for an underground power station in order to obtain rock mechanics data for design of the underground cavern. The in-situ rock test provides the moduli of the deformation and the elasticity (Young's modulus) by the plate loading, and on the shear strength by the applying vertical confining load and the tangential shearing load at the same time on each of the concrete test blocks placed on the rock.

The shear test were carried out on three test blocks and the plate-loading test are at three location. The location of the in-situ rock testing executed are indicated in Figure B3.7.1 and the condition of the branch adit are shown in Figure B3.7.2.

B3.7.2 Plate Loading Test

(1) Methodology

The plate bearing test is one of the most common methods to determine the deformability of rock mass in-situ. In this method a load is applied to a specially prepared flat surface by means of a rigid or semi rigid finite plate and measuring the deformation at any convenient point within the rock mass. Thus rock modulus can be calculated using the relationship developed depending on the shape of the loading plate and the nature of the rock mass.

The plate loading tests were conducted in accordance with the technical requirement, and results are summarized as below :

Plate loading test was carried out at three spots of the branch adit around 10 m from the excavation face.

Both the modulus of the elasticity and the modulus of the deformation can be calculated with the same formula as follows.

$$E \text{ or } D = (1-\mu^2)\ X\ \ d F / d S\ X\ 0.5\ a$$

Where,

E: Modulus of elasticity	(Kgf/cm^2)
D: Modulus of deformation	(Kgf/cm^2)
a: Radius of steel loading plate	(cm)

μ = Poisson's ratio (0.2 for hard rock and 0.25 to 0.3 for soft rock)
d F = increased load in a section of load-displacement curve (Kgf = ton/1000)
dS = increased displacement for the same section as above (cm)

If a gradient of a tangential line of stress-displacement curve for the peak stress is placed in the place of dF /dS, the formula will give a modulus of elasticity. If a gradient of a line enveloping the stress-displacement curves of the initial stresses is used for dF /dS, it will give a deformation modulus.

The plate loading tests are conducted is accordance with the technical requirement and results are summarized as below :

(2) Equipment and Material

Apparatus and material for preparation and test are as follows:

- Hydraulic jack of the maximum loading capacity of 100 tons, with hand pump and load gauge with the minimum reading of 1 ton. Well calibrated before transportation to the site. 1 set

- Hydraulic jack of the maximum loading capacity of 100 tons with hand pump and load gauge with the minimum reading of 1 ton. Well calibrated before transportation to the site. 2 sets

  or

  Hydraulic jack with the maximum capacity of 200 tons with similar specifications for other aspects 1 set

- Steel loading plates, steel column supports and a spherical load adjuster, 1 lot

- Quantity depends on diameter of the adit. It is convenient for good progress of the work to prepare the supports and plates a little more than the minimum requirement, for instant:

  Steel plates: Disk type:

  Thickness 3 cm, ϕ50 cm 2 nos.

  Thickness 3 cm, ϕ70 cm 2 nos.

  Rectangular:

  Thickness 3 cm, 50 cm x 70 cm 2 nos.

  Thickness 3 cm, 70 cm x 70 cm 2 nos.

  Steel column support ϕ45 cm: Height 50 cm 3 nos.

  Height 25 cm 2 nos

Height 10 cm 2 nos.

Height 5 cm 1 no.

Spherical load adjuster: 1 no.

- Gas pipes to support dial gauges 15 m
- Dial gauges: Stroke not less than 30mm, minimum calibration 0.01mm, 12 nos.

- Extensometer system (trancuder module, mechanical anchor, extension tube centralisers and setting tool and rods) 2 lot

- Electric lighting facilities with generator. 1 set

- A concrete mixer with facilities to weigh cement and aggregates 1 set

- Reinforcement bars of ϕ10 mm
- Portland cement. Sand. Gravel. 1 lot

- Wooden form for concrete placement for shear test blocks and counterweight blocks 1 lot

- Quick setting agent for concrete 3 litres

- Tools for excavation of tunnel muck on the adit floor, rock chisels and hammers to make rock surface even, flash light to help reading the dial gauges, etc.
  1 lot

(2) Results

Results of the test are shown in Figures B3.7.3 and B3.7.4.

LOCATION	MODULUS OF DEFORMATION (MPa)	MODULUS OF ELASTICITY (MPa)
PL-1	3,183.9	14,650.3
PL-2	9,366.9	25,340.5
PL-3	1,869.5	8,392.9

B3.7.3 Block Shear Test

(1) Methodology

The sliding stability of any structure is governed by the shear strength of the interface between the concrete and the rock. Thus, the evaluation of the shear strength is considered to be most important step to ensure the stability of any structure against the sliding or shearing failure. The best way to evaluate the shear strength of the interface of concrete and rock face is to conduct in-situ test of shear test of concrete block over the rock face. Contouring of the test pad was done before and after the test.

The block shear test is performed at 3 spots. The concrete block of 70 x 70 x 30 cm was placed covering trimmed base rock in each testing spot.
The procedure of conducting the in-situ shear tests is described in ISRM (1974) and IS 7746-(1975). The shear strength of rock mass depends upon number of factors such as direction of shear force relative to direction of joints, foliations, strength of rock, saturation, rate of loading, rate of shearing etc.
This test measures peak and residual direct shear strength as a function of stress to the sheared plane.
Shear and normal stress are computed as follows:

Shear stress = $\tau = Ps/A = (Psa \cos\theta) /A$ (1)

Normal stress $\sigma n = Pn/a = (Pna + \mathrm{Sin}\,\theta) /A$ (2)

Where, Ps = total shear force
Pna = total normal force
Psa = applied shear force
P = applied normal force
θ = inclination of the applied shear force to the shear plane
A = area of the shear surface overlap

The test measures the peak and the residual direct shear strength as a function of the stress normal to the sheared plane. The test has been conducted in accordance with the technical requirement and the results are summarized below :

(2) Equipment and Material

The same equipments and materials utilized for the plate loading test were applied the block shear test.

(3) Result

The test has been conducted in accordance with technical requirement and the results are summarized below :

The plots between shear stress and horizontal displacements of 3 blocks are shown in the figures below. The normal stress has been varied as 5 kg/cm^2 , 7.5 kg/ cm^2 and 10 kg/ cm^2. The plot between the normal stress and the shear stress is also shown in Figures of B3.7.5, B3.7.6, B3.7.7.

As a result of testing, following values are obtained.

τ =0.3 to 0.5 MPa, φ = 40.0° to 50°

However, this figure is discordant with the results of plate loading tests. The results of the plate loading test indicates hard and compact state of rock. This result of shearing test, where both (palte loading and rock shear testing) tests are performed in adjacent location, indicates the relatively soft and the poor rock condition.

Considering the results of the rock shear test in comparing the other test results, such as the plate loading test and the laboratory test, these figures shall be utilized as a reference data only, in this stage of the study.

B3.8 Laboratory Test

B3.8.1 General

Representative rock specimens selected from the drilling core samples of cylindrical form were sent to the laboratory for the purpose of confirming the basic physico-mechanical characteristics of the rocks in geotechnical aspects.

B3.8.2 Methodology

The sample selected was sent to the laboratory and the laboratory test was executed in the listed manners. Geotechnical interpretation and evaluation of the testing results were done.

The items and the envisaged quantities of the laboratory test are as listed below.

- Water absorption (ASTM C 127-1/2) : 80 samples
- Bulk specific gravity (ASTM C 127-2/2) : 86 samples
- Unconfined compression test of rock core specimen (ASTM D2938) : 81 samples
- Ultra-sonic wave velocity test (ASTM D2845) : 100 samples
- Splitting tensile strength test, or Brazilian test (ASTM D3967) : 98 samples

B3.8.3 Results

The result of the sample are summarized with the stratigraphy of the project area as shown in Table B3.8.1 and the results of the test are listed in the Table B3.8.2.

B3.9 Exploratory Adit

Excavation of the exploratory adit had been completed as of March.08, 2001. The exploratory adit was planned to drive to check actual condition of siliceous dololmite at excavation face. The sectional area forms 3.0m wide and 3.0 high in the section from the portal to the distance of 237m. The sectional area of the adit downstream from 237m is reduced to 1.7m wide and 2.0m high.

Geological condition along the exploratory adit are described as follows:

The boundary between slate and sericeous dolomite was observed at 380m from tunnel portal. As a result of observation of the adit, rock condition in the section around 70 m tob 90 m from the adit portal is poor to relatively poor, however, moderately fair rock condition is confirmed in rest of the section.

The groundwater ingress of approximately 150 to 200 l/min was observed near the end of the adit of 320 m in total length on September 28, 2001. (approximately 310 to 315 m from the adit portal). Nearly same amount of water flow was observed in the adit portal at that time. The groundwater inflow of approximately less than 50 l/min was observed at same location at the beginning of July, 2002. The observation result of the exploratory adit is described in detail in the Figure B3.9.1.

Branch adit of 2.5m x 2.0m x 10.0m in size was excavated for in-situ rock test. Siliceous dolomite in hard and compact state is predominantly distributed on excavation face of the adit. (See Figure B3.9.1)

B4 Engineering Geology

B4.1 Rock Classification

B4.1.1 Discussion on Test Results

In-situ rock tests were carried out in the branch adit excavated around excavation face of the exploratory adit. Results of the test were carefully examined and some discordances were found in comparison with results of laboratory test and site observation. As a result of the rock shear test, shear strength of 0.3 to 0.5 MPa and friction of 45 degrees were obtained. As discussed in the former chapter, these figures are judged to be utilized as reference data considering results of other tests.

As a result of laboratory test of rock samples obtained of BPV-1 and PPH-1 drill holes near the in-situ rock testing site, the uniaxial compression strength of dolomite appears to be more than 50 MPa.

According to the study on the relationship between the uniaxial compression strength and the shear strength of similar rock of dolomite on the basis of numerous data of rock samples obtained in Japan (Journal of the Japan Society of Engineering Geology, 1983), the following correlation is proposed.

Unixial comression strength	Shear strength
50 MPa	2 to 3 MPa

The modulus of deformation is assumed to be 2,000 to 3,500 MPa when the uniaxial compression strength of 50 MPa on the basis of the said study of correlation between the uniaxial compression strength and the modulus of deformation.

Unixial comression strength	Modulus of Deformation
50 MPa	2,000 to 3,500 MPa

As the result of the plate loading test, 1,869.5 to 9,366.9 MPa were obtained. These figures are correlated to the modulus of deformation based on the uniaxial compression strength. Therefore, the shear strength is judged to be 2 to 3 MPa, not 0.3 to 0.5 MPa.

The modulus of deformation obtained as a result of plate loading test are carefully studied. Test result of PL-3 location, which shows 1,869.5 MPa , is judged to be too low figure in comparison with that of the average rock mass in the test adit. Discotinuities like joints and foliation were highly developed when the testing location was carefully observed after testing.

Therefore, the modulus of deformation obtained as results of testing at location of PL-1 (3,1838.9 MPa) and PL-3 (9,366.9MPa) are considered to be representative figures of the rock in the testing area.

Modulus of deformation of rock at testing location is judged to be more than 3,000 MPa .

Considering the result of in-situ rock tests and loboratory tests, following figures of physical properties of rock at the location of in-situ rock testing are assumed.

Shear strength 2 to 3 MPa

Friction angle 45 to 50 degrees

Modulus of Deformatoin 3,000 to 5,000 MPa

B4.1.2 Classification by Q Values

Q value rock classification have been used for the Updated Feasibility Study done by NEA and this classification is applied on the basis of Q classification system developed by Barton, Lien and Lunde (1974). Q system is primary developed for tunnel design work. The main advantage to the Q classification system is sensitiveness to minor variations in rock properties. One disadvantage of the Q system is difficulty for inexperienced users to apply.

Q value is calculated on the basis of a function of 6 independent parameters by using following formula.

$$Q = RQD/Jn \times Jr/Ja \times Jw/SRF$$

where,

Jn : Joints Set Number

Jr: Joints Roughness Number

Ja: Joints Alterraion Number

Jw: Joints Water Reduction Number

Q system parameters are shown in Table B4.1.1.

B4.1.3 Classification by CRIEPI

Central Research Institute of Electric Power Industry (CRIRPI) of Japan has developed rock classificatoin system on the basis of abundant research data for long period of activities. Rock classification of regulating dam is judged on the basis of this CRIEPI rock classification system. (See Table B4.1.2)

The correlation of the Q system and the CRIEPI system rock classification are shown in Fig. B4.1.1.

B4.2 Geology and Physical Properties

Rock properties of the different grade of rock are proposed on the basis of results of in-situ rock tests as well as laboratory tests performed for this study and the existing data of similar geological condition of other projects.

Physical properties of rock at the location of in-situ rock testing are assumed to be 2 to 3 MPa of shear strength and more than 3,000 MPa of modulus of deformation.

According to the rock classification of Q system and CRIEPI, the rock at the location of in-situ rock testing is judged to be around Q2 and CH on the basis of results of geological and geotechnical investigation and observation of rock at the site. Following physical properties are proposed on the basis of this judgment.

Rock Grade	Modulus of Deformation (MPa)	Shear Strength (MPa)	Friction Degree (degree)	Q value by Q system	CRIEPI classification system
Q1	> 3,000	> 2.5	> 50	> 40	B
Q2	3,000	2.5	50	10 to 40	CH
Q3	1,000	1.2	45	4 to 10	CM
Q4	500	0.6	40	1 to 4	CL
Q5	250	0.1	35	1 >	D

Q classification system is developed mainly for tunneling work and not applied for other engineering field. CRIEPI classification system was firstly developed for foundation of large structures. Therefore, attention shall be paid on mentioned correlation of two classification systems when this correlation is applied for design work.

B5 Geology

Geological and geotechnical condition of the sites are summarized on the basis of the investigation results.

B5.1 Underground Structures

The underground or semi-underground type of powerhouse was planned to be built. The geological investigation of BS-1 drill hole has been carried out for this study in semi-underground type powerhouse site on the right bank of the Rapti River near the confluence with the Kesadi River. According to results of the geological investigations including existing investigation results, foundation of this site was weathered slate and it was covered by unconsolidated deposits in thickness of 33.5m (416.9m in elevation).

Five holes of core drilling for underground powerhouse were executed. Two holes were carried out from the exploratory adit. Unexpected longer duration of drilling works appeared to be required due to limited working space for drilling works in the adit, therefore three drill holes were executed on the ground surface.

Enlargement of the exploratory adit was performed for execution of drilling work in the adit. Results of drilling carried out from exploratory adit identified sound rock condition in and above the underground cavern. Thickness of the dolomite layer is judged to be around 150m as the results of other executed drillings for underground powerhouse.

The underground structures including the powerhouse cavern are planned to be placed in siliceous dolomite layer of around 150 m in thickness, which was confirmed by drill holes of BPV-2,3, and 4 in the right bank of Rapti river. General trend and dips of the siliceous dolomite are N60° W and 60° N respectively. The siliceous dolomite is hard and has no solution cavity below the connection tunnel level of around 580m down to the powerhouse section of around 460m in elevation as a result of drill hole of BPV-1. Rock mass of the siliceous dolomite is not homogeneous, but platy joints of 10 to 30 cm interval are frequently observed in the drill hole of BPV-1. The sufficient water-tightness having less than 5 lugeon was generally confirmed in the drill hole of BPV-1 and BPH-1. Characteristics of dolomite was carefully studied on the basis of rock test and utilized such information for design work.

As a result of the geotechnical investigation, the modulus of deformation of more than 3,000 MPa, cohesion strength of 2 to 3 MPa and friction angle of 45 to 50 degree are judged to be adequate in the rock mass around powerhouse
Poor geological condition between the siliceous delomite and slate has been confirmed during the excavation of the exploratory adit. Attention shall be paid on the geological condition of the boundary between the dolomite and slate, even if the underground powerhouse is to be built more than 100 m below the exploratory adit level. Attention shall be paid on the groundwater inflow toward the

underground cavern considering the elevation of the powerhouse, which is below the Rapti and Kesadi Rivers. (See Figure B5.1.1 and B5.1.2)

B5.2 Regulating Dam

The proposed dam axis and their pondage area were thoroughly surveyed in terms of geological and geo-technical viewpoints.

The bedrock of the proposed dam site consists of phyllite. The phyllite was fresh and hard below 3 to 5 m from rock surface, and it could have sufficient strength of bearing capacity for the planned dam.

According to core drilling carried out along the upstream dam axis near the proposed axis in 2002, the surface of the bedrock is moderately weathered in thickness of 1 to 5 m, and fresh and compact bedrock is distributed underneath. Permeability of the phyllite appears to be relatively low, which varies from 7.6 x 10^{-6} to 4.8 x 10^{-4} cm/sec.

Sufficient bearing capacity for the proposed dam could be obtained in the river-bed section, however weathrered and relaxed portions will be expected at the abutments. Permeability of phyllite bedrocks is low of less than 5 lugeon in general. Lugeon value of 5 to 10 are confirmed in upper parts of the bedrock. (See Figure B5.2.1, B5.2.2, B5.2.3, B5.2.4)

B5.3 Regulating Pond

A land of gentle slope at 15 degrees and less from horizontal is developed in a 150 m wide and 150 m long area in the pondage of regulating dam. It is located on the right bank of the the Yongrin River near the upstream-end of the pondage. Potential landslide (Landslide R-1) was suspected in previous study and drill holes of LS-1 and LS-2 were allocated.

Top soil of around 1 m thick are distributed covering the whole gentle slope area. Relaxed rock mass and debris of 23m and 2m in thickness are found at LS-1 and LS-2 points respectively. Stable rock mass are confirmed below these debris zones. The foot of this potential landslide area is submerged in high water-level. Daily fluctuation of water level of the pondage is expected to be 10 m. Careful attention shall be paid on the distribution of this potential landslide area.

The landslide block of relatively large scale is located in the upstream (Right bank) of the reservoir. Capacity of the reservoir likely decrease in some extent when the landslide occurred. The geological investigation shall be performed to clarify the landslide block and the countermeasures shall be proposed.

Analysis of the slope stability and countermeasures were described in Section 3.4 of "Landslide and Sedimentation in Project Area". (See Figure B5.2.1)

B5.4 Connection Tunnel

Proposed alignment of the 3.5 km connection tunnel is planned on the right bank of the Rapti River. The tunnel drives through marble, schist, slate, quartzite, and phyllite. Drill hole of BI-2 was placed at the inlet portal, BCT-1, BMT-1, and BCT-2 were along the tunnel route, and BO-1 was at the outlet of connection tunnel. As a result of drilling, relatively fair rock condition was confirmed. However, length of drill holes along the tunnel was insufficient to reach the tunnel level. Geological condition along the connection tunnel is confirmed to be generally good considering the results of the drilling and site reconnaissance. Marble (0.7km in length), schist (0.3km), phyllite(2) (0.7km), Quarzite (1.0km) and phyllite(1) (0.8km) are supposed to be encountered during the excavation period.

No serious problem is envisaged on stability of tunnel faces. However, careful attention shall be paid on the lithological boundaries including the Mahabharat Thrust, where they might be fractured and altered to clay in some extent. Groundwater ingress also appears to be encountered in such boundary sections with high groundwater table. (See Fig. B2.2.3, B5.4.1)

B5.5 Headrace Tunnel

Proposed route of the headrace tunnel of 0.4km in length is planned on the right bank of Rapti river. Phyllite and siliceous dolomite are supposed to encounter along tunnel route. Lithological boundary of these two strata might be fractured judging from the excavation face of the exploratory adit and outcrops observed along the river. Careful attention will be paid on excavation works in this section. The groundwater inflow is supposed to be small and no serious problem might be envisaged during the excavation work. (See Figure B2.2.3)

B5.6 Tailrace Tunnel

Tailrace tunnel of 2.1 km in length is planned in the right bank of the Rapti River crossing the Main Boundary Thrust. Slate of Paleozoic, Tertiary Siwarik sandstone of Cenozoic is supposed to encounter in the course of excavation work. Drill holes of DHT-4, DHT-6 were located along the tunnel route, and BOT-1 at portal.

According to the results of drilling, bedrock of Siwarik sandstone is confirmed through the whole of three drill holes. Sandstone distributed in tunnel elevation was confirmed to be poor geological condition according to a result of drill hole of DHT-4. The geological condition of sandstone is relatively fair in downstream part of the tunnel according to results of core drilling of DHT-6 and BTO-1. The slate is confirmed at depth of 33.5m of drill hole BS-1.

The slate and sandstone are fractured and in poor condition in and around the Main Boundary Thrust. The fractured zone of the MBT appears to around 200m in width.

Open excavation for culvert in the section of the MBT can be considered as an alternative. Large amount of groundwater might be encountered in the river section. Therefore, drainage work for open excavation shall be carefully prepared. (See Fig. B2.2.3, B5.6.1)

B5.7 Head Works

Headworks site is located around the tailrace of the KL-II powerhouse site in the Khani river. Present river deposits of sand and gravels are distributed underlain by the bedrock of limestone.

Drill hole of BI-2 was executed at planned intake site for connection tunnel. Relatively massive limestone is confirmed to be distributed below unconsolidated river-bed deposits. Sound foundation can be expected in the limestone.

B5.8 Access Bridge

Construction of bridge is planned for crossing the Rapti River immediately upstream of the confluence with the Kesede River. The foundation of the bridge was investigated by three drill holes of BA-1, BP-1, and BA-2. Two holes are located at both right and left banks and one hole at river section. The foundation of the bridge was weathered bedrock of slate which is confirmed by whole three drill holes. Bedrock is judged to be sound enough for foundation of piers of the bridge. Besides, some part of unconsolidated layer might have sufficient bearing capacity. However, careful attention shall be paid on the slope stability of foundation of abutment at both side of the bridge.

B5.9 Geotechnical Condition of Waterway Tunnels and Underground Structures

Geological profile and rock classification along the waterway including the connection, headrace, underground powerhouse, tailrace tunnels are shown Figure B5.9.1. The length of the rock mass of different rock grade is summarized in the following table.

The geology and rock grades of the strata that are expected to be encountered along the tunnels and underground powerhouse structures are estimated and presented below. The rock grades that was used for the Upgrading Feasibility Study done by NEA is applied for this report. Rock grade of Q1 is expected in some sections of the connection tunnel, but may be intercalated with Q3 and/or Q4. The rock grade of Q2 is applied for such sections. The distribution of rock grades is shown in the following table.

Rock Grade for Structure Sites (Unit: m)

Structure Site	Total Distance	Q1	Q2	Q3	Q4	Q5
Connection Tunnel	3,475	-	1,685	1,495	220	75
Headrace Tunnel	389	-	54	315	20	-
Underground P/H Structures	132	-	132	-	-	-
Tailrace Tunnel	2,149	-	79	1,245	520	305
Total	6,145	-	1,950	3,055	760	380

Fair to good rock condition (Q2 to Q3 in rock grade) is expected along more than 90% of the toal length of the connection tunnel as well as the headrace tunnel. The underground powerhouse is planned to be built in the dolomite of Q2 in rock grade. Around 40% of the tailrace tunnel is in the rock grade of Q3, and remaining 60% are Q4 and Q5 in rock grade. Relatively poor rock condition is supposed to be encountered during the excavation work of the tailrace tunnel.

B6 Seismicity and Seismic Risk

Numbers of earthquakes have been occurred and the seimicity has been recorded in Nepal. Limited numbers of the reliable seismic records are available in prior to the development of seismographs in the late 19th centuries. The seismograph network have been gradually well facilitated and large numbers of earthquakes have been recorded since then.

The seismicity in Nepal including northern part of India have been studied in various fields. The relationship between the occurrence of earthquakes and the faulting have also been studied. As the results of the study, the major earthquakes occurred in Nepal are said to be related to the inferred subsurface faulting, not to the surface faulting.

Therefore, seismicity records collected through one of the most well facilitated seismograph network of United States Geological Survey (USGS) are utilized for seismicity anlysis.

For evaluation of the seismic risk of the project area, all earthquakes data over 1.0 of Magnitude in the Richter scale with epicenters within 300 km of distance from the project site were collected through United State Geological Survey (USGS). Total number of 153 data were obtained after screening the said conditions of seismicity in the period between 1973 and 1991. (See Figure B6.1)

The seismic risk was mainly evaluated by using formulae according to Cornell. The other formula according to Kawasumi was applied and those calculated figures were used as reference data. The formula proposed by Kawasumi is developed on the basis of the seismic data of earthquakes occurred in Japan, where the earthquake occurred so frequently and seismic risk is so high in comparison with other countries.

The formula according to Cornell (Cornell, C.A., 1968, Engineering seismic analysis, Bull. Seism. Soc. Am.Vol.58, pp.1583-1606) and the formula according to Kwasumi is described as follows.

<u>The formula according to Cornell</u>

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

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, pp 469 - 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)

The number of earthquake events were 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 obtain the number of events per year exceeding the given intensity (Nc).

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

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

where, p and q are constants. The values of I and Nc were plotted on a graph , 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) (See Figure B6.2). The result is as follows:

	Maximum Intensity in MM Scale	Maximum Intensity in JMA Scale	Maximum Peak Acceleration
According to Cornell	7.3	-	160 gal
According to Kawasumi	-	3.3	20.1 ga1

Kawasumi's formula is used especially in high seismic risk areas like Japan, therefore seismic factors of maximum probable intensity obtained by Kawasumi's formula could be different from other calculation results ,when seismic risk is relatively low.

On the other hand, it is assumed that Maximum Credible Earthquake is generated at the distance of 10 km on the Main Mantle Thrust (This thrust fault is distributed in between the earth crust and mantle in the globe, and located deeper than any other thrust faults like the MCT and MBT etc..) and has Magnitude of 7.0 and focal depth of 40 km. Intensity and peak acceleration are estimated at 9 and 500 gal.

The peak acceleration works only for a fraction of a second and is virtually unable to create any damage upon dam structures. Substantially durable acceleration is far lower than the peak value, for example a third of 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.

Seismic risk study was done in a 1988 feasibility study by NEA. According to calculation using total earthquake events of 281 from 1913 to 1987, probable maximum intensity of 8.4 and probable maximum acceleration of 0.21 g in 100 years return period were obtained, and design seismic coefficient of 0.15 to 0.20 is recommended.

With all the above results of evaluation in mind, the appropriate design earthquake acceleration is considered at 0.15g.

TABLES

Appendix B

Standard Penetration Test

Table B3.3.1 Results of Standard Penetration Tests

Hole No.	Test Depth		N-Value	Knocking Times/15cm			Remarks
				15	15	15	
BI - 2	1.00 - 1.45			48	50	49	DCPT
	2.00 - 2.50		-	39	43	45	DCPT
	4.00 - 5.00		-	47	46	49	DCPT
BO – 1	0.0 – 1.0		28	9	9	8	
	1.0 – 2.0		50	14	23	13	
	2.0 –3.0			80	-	-	
	3.0 –4.0			65	15	-	
	4.0 – 5.0			55	25	-	
	5..0 – 6.0			73	7	-	
LS – 1	0.0 –1.0		24	6	8	10	
	1.0 – 2.0		25	6	8	11	
	2.0 – 3.0		50	12	38	-	
	3.0 – 4.0		50	50	-	-	
	4.0 – 5.0		50	80	-	-	
	5.0 –6.0		50	18	26	32	
	6.0 –7.0		50	80	-	-	
	7.0 – 8.0		50	50	-	-	
	8.0 – 9.0		50	22	28	-	
LS – 2	0 – 1.0		10	2	3	5	
	1.0 – 2.0		25	6	9	10	
	2.0 – 3.0		50	32	18	-	
	3.0 – 4.0		19	4	7	8	
	4.0 – 5.0		20	6	7	7	
	5.0 – 6.0		28	9	10	9	
	6.0 – 7.0		32	14	7	11	
	7.0 – 8.0		31	9	10	12	
	8.0 – 9.0		27	6	8	13	
	9.0 – 10.0		50	24	35	21	
	10.0 – 11.0		35	8	13	14	
	11.0 – 12.0		50	18	27	35	
	12.0 – 13.0		50	36	44	-	
BA – 1	1.00-2.00		50	30	30	32	DCPT
	3.50-4.00		50	31	28	35	DCPT
	5.50-7.00		50	36	33	37	DCPT
	9.00-10.00		50	41	45	44	DCPT
	14.00-16.00		50	48	47	49	DCPT
	17.50-19.00		50	46	40	37	DCPT
BA – 2	2.00-3.00		50	30	30	31	
	4.50-6.00		50	34	35	36	
	7.50-8.00		50	44	43	33	
	10.00-11.00		50	33	38	38	
	12.00-12.50		50	26	26	28	
	14.00-15.00		50	31	30	31	

Hole No.	Test Depth		N-Value	Knocking Times/15cm			Remarks
				15	15	15	
BI - 2	1.00 - 1.45		49.50	48	50	49	
	2.00 - 2.50		44.00	39	43	45	
	4.00 - 5.00		47.50	47	46	49	
BO – 1	2.0 – 3.0		50	80	15	80	
	3.0 – 4.0		50	80	19	80	
	4.0 – 5.0		50	80	21	80	
	5.0 – 6.0		50	80	16	80	
BS – 1	0 – 1.0		12	3	4	5	
	1.5 – 3.0		34	9	10	14	
	3.0 – 4.0		50	9	11	30	
	4.0 – 5.0		32	10	13	9	
	5.0 – 6.0		23	9	8	6	
	6.0 – 7.0		19	6	6	7	
	7.0 – 8.0		20	6	6	8	
	8.0 – 9.0		32	9	10	13	
	9.0 – 10.0		27	6	9	12	
	10.0 – 11.0		45	11	16	18	
	11.0 – 12.0		50	13	26	37	
	12.0 – 13.0		42	11	11	20	
	13.0 – 14.0		36	8	12	16	
	14.0 – 15.0		50	12	16	18	
	15.0 – 16.0		50	25	55	-	
	16.0 – 17.0		50	47	33	-	
	17.0 – 18.0		50	35	45	-	
	18.0 – 19.0		50	53	27	-	
	19.0 – 20.0		50	50	30	-	
	20.0 – 21.0		50	49	31	-	
	21.0 – 22.0		50	56	24	-	
	22.0 – 23.0		50	52	28	-	
	23.0 – 24.0		50	55	25	-	
	24.0 – 25.0		50	27	53	1	
	25.0 – 26.0		50	35	39	6	
	26.0 – 27.0		50	39	41	-	
	27.0 – 28.0		50	30	50	-	
	28.0 – 29.0		50	60	20	-	
	29.0 – 30.0		50	51	29	-	

Hole No.	Test Depth		N-Value	Knocking Times/15cm			Remarks
DHT – 4	1.50 – 2.0		50	37	43	-	
	2.0 – 3.0		50	50	30	-	
	3.0 – 4.0		50	45	35	-	
	4.0 – 5.0		47	11	20	16	
	5.0 – 6.50		50	11	20	42	
	6.50 – 7.0		50	73	7	-	
	7.0 – 8.0		50	20	15	15	
	8.0 – 9.0		50	32	24	22	
	9.0 – 10.0		50	53	27	-	
	10.0 – 11.0		50	45	35	-	
	11.0 – 12.0		50	80	-	-	
DHT – 6	0.0 – 1.0		50	60	20	-	
BTO – 1	0.0 – 1.0		50	80	-	-	
	1.0 – 2.0		50	80	-	-	
	2.0 – 3.0		50	80	-	-	
	3.0 – 4.0		34	7	7	20	
	4.0 – 5.0		45	10	10	25	
	5.0 – 6.0		50	80	-	-	
BP – 1	0.0 – 2.0		50	36	44	-	
	1.0 – 2.0		50	80	-	-	
	2.0 – 3.0		50	19	61	-	
	3.0 – 4.0		50	28	25	18	
	4.0 – 5.0		50	20	31	29	
	5.0 – 6.0		50	15	15	19	
	6.0 – 7.0		50	19	21	23	
	7.0 – 8.0		50	80	-	-	
	8.0 – 9.0		50	50	30	-	
	9.0 – 10.0		50	15	23	14	
	10.0 – 11.0		33	17	11	5	
	12.0 – 13.0		42	8	4	30	
	13.0 – 14.0		39	13	15	11	
	14.0 – 15.0		50	80	-	-	
BA – 1	1.0-2.0		31.00	30	30	32	
	3.50-4.0		31.50	31	28	35	
	5.50-7.0		35.00	36	33	37	
	9.00-10.0		44.50	41	45	44	
	14.0-16.0		48.00	48	47	49	
	17.50-19.0		38.50	46	40	37	

Hole No.	Test Depth		N-Value	Knocking Times/15 cm			Remarks
				15	15	15	
BPV-2A	0.0 - 1.0 m		50	50	-	-	
	1.0 - 2.0 m		80	80	-	-	
BPV-2B	0.00-1.0 m		29	4	9	16	

Water Pressure Test

Table B3.4.1 Results of Water Pressure Test Results.

Hole No.	Test Section, m	Total Pressure (Kg/Cm^2)	Unit Take Q (lit/min/m)	Lu Value
BD – 4	3.0 – 8.0	- 0.36	0.9	-
		2.64	5.9	22.50
		0.36	1.8	-
BD – 4	8.0 – 13.0	-0.15	0.5	-
		2.74	1.8	-
		5.74	2.4	-
		8.74	4.2	4.81
		5.74	3.5	-
		2.74	2.6	-
		-0.27	0.7	-
BD – 4	13.0 – 18.0	0.49	0.2	-
		4.21	2.0	-
		7.21	3.6	-
		10.21	4.4	4.33
		7.21	2.6	-
		4.21	2.3	-
		1.21	0.3	-
BD – 4	18.0 – 23.0	0.54	1.1	-
		3.54	3.0	-
		6.54	3.1	-
		9.54	7.1	7.40
		6.54	4.5	-
		3.54	3.1	-
		0.54	0.8	-
BD – 4	23.0 – 28.0	0.54	2.6	-
		3.54	6.0	-
		6.54	7.8	11.87
		3.54	4.7	-
		0.54	2.5	-
BD – 4	33.38	0.46	1.0	-
		3.46	2.80	-
		6.46	3.40	-
		9.46	3.80	4.02
		6.46	2.82	-
		3.46	2.28	-
		0.46	0.40	-
BD – 4	37.0 – 42.0	0.54	7.6	-
		3.54	2.9	-
		6.54	2.6	-
		9.54	3.0	3.15
		6.54	2.2	-
		3.54	2.6	-
		0.54	1.1	-

Hole No.	Test Section, m	Total Pressure Kg/Cm^2	Unit Infiltration Volume Q (lit/min/m)	Lu Value
BD – 4	41.0 – 46.0	0.48	7.6	-
		3.48	3.0	-
		6.48	3.4	-
		9.48	3.7	3.95
		6.48	3.0	-
		3.48	2.7	-
		0.48	1.1	-
BD – 4	45.0 – 50.0	0.52	2.2	-
		3.52	3.3	-
		6.52	2.9	-
		9.52	3.4	3.57
		6.52	3.0	-
		3.52	3.3	-
		0.52	7.6	-
BD – 5	15.50 – 18.50	1.05	3.0	-
		4.05	4.4	10.86
		1.05	2.8	-
BD – 5	18.0 – 23.0	7.05	0.9	-
		4.05	7.2	-
		7.05	7.7	2.44
		4.05	7.3	-
		7.05	0.9	-
BD – 5	23.0 – 28.0	7.21	2.4	-
		4.21	3.6	-
		7.21	4.8	-
		10.21	6.0	5.92
		7.21	4.0	-
		4.21	3.1	-
		1.21	2.1	-
BD – 5	28.0 – 33.0	1.20	0.6	-
		4.20	1.8	-
		7.20	2.8	-
		10.20	5.2	5.06
		7.20	4.1	-
		4.20	2.1	-
		1.20	0.5	-
BD – 5	33.0 – 38.0	1.05	1.2	-
		4.05	2.0	-
		7.05	3.6	-
		10.05	5.2	5.21
		7.05	2.6	-
		4.05	1.6	-
		1.05	0.6	-

Hole No.	Test Section, m	Total Pressure Kg/Cm^2	Unit Infiltration Volume Q (lit/min/m)	Lu Value
		1.05	1.06	-
		4.05	2.22	-
		7.05	3.12	-
BD – 5	38.0 – 43.0	10.05	4.00	3.98
		7.05	2.80	-
		4.05	2.22	-
		1.05	1.00	-
		0.43	1.0	-
		3.43	1.5	-
		6.43	2.5	-
BD – 6	14.0 – 19.0	9.43	3.3	3.46
		6.43	2.3	-
		3.43	1.5	-
		0.34	0.8	-
		0.39	1.5	-
		3.34	2.7	-
		6.34	3.2	-
BD – 6	19.0 – 24.0	9.34	4.9	5.23
		6.34	2.4	-
		3.34	2.1	-
		0.34	1.4	-
		0.42	2.3	-
		3.42	3.4	-
		6.42	4.7	-
BD – 6	24.0 – 29.0	9.42	5.7	6.05
		6.42	4.1	-
		3.42	3.0	-
		0.42	2.0	-
		0.45	1.2	-
		3.45	3.2	-
		6.45	3.0	-
BD – 6	29.0 – 34.0	9.45	4.6	4.83
		6.45	3.2	-
		3.45	3.4	-
		0.45	1.5	-
		0.42	3.2	-
		3.42	4.8	-
		6.42	3.8	-
BD – 6	35.0 – 40.0	9.42	5.4	5.69
		6.42	4.1	-
		3.42	3.3	-
		0.42	3.1	-

Hole No.	Test Section, m	Total Pressure Kg/Cm^2	Unit Infiltration Volume Q (lit/min/m)	Lu Value
BD – 7	4.0 – 9.0	1.12	0.3	-
		4.12	0.4	-
		7.12	1.7	-
		10.12	4.9	4.86
		7.12	1.1	-
		4.12	0.5	-
		1.2	0.2	-
BD – 8	8.0 – 13.0	0.53	5.6	-
		3.53	9.2	26.18
		0.53	4.5	-
BD – 8	12 – 17.0	0.70	1.4	-
		3.70	6.7	-
		6.70	9.6	14.36
		3.70	6.0	-
		0.70	1.6	-
BD – 8	16.0 – 21.0	0.65	1.4	-
		3.65	1.5	-
		6.65	2.0	-
		9.65	2.9	2.96
		0.65	1.2	-
BD – 8	21.0 – 26.0	0.67	1.8	-
		3.67	2.2	-
		6.67	2.5	-
		9.67	3.5	3.62
		6.67	2.8	-
		3.67	1.8	-
		0.67	1.1	-
BD – 8	25.0 – 30.0	0.67	1.8	-
		3.67	1.3	-
		6.67	1.5	-
		9.67	2.0	2.03
		6.67	1.2	-
		3.67	1.0	-
		0.67	1.9	-
BD – 9	11.0 – 16.0	0.92	0.44	-
		3.92	2.00	-
		6.92	4.80	-
		9.92	4.96	5.0
		6.92	4.04	-
		3.92	2.88	-
		0.92	0.72	-

Hole No.	Test Section, m	Total Pressure Kg/Cm^2	Unit Infiltration Volume Q (lit/min/m)	Lu Value
BD – 9	16.0 – 21.0	0.92	1.6	-
		3.92	3.0	-
		6.92	3.7	-
		9.92	5.6	5.67
		6.92	4.9	-
		3.92	2.5	-
		0.92	1.2	-
BD – 9	21.0 – 26.0	0.92	0.2	-
		3.65	3.3	-
		6.65	3.8	-
		9.65	4.4	4.56
		6.65	3.4	-
		3.65	1.0	-
		0.65	0.2	-
BD – 9	25.0 – 30.0	0.92	0.9	-
		3.92	4.6	-
		6.92	4.0	-
		9.92	4.7	4.76
		6.92	2.6	-
		3.92	3.6	-
		0.92	0.2	-
BD – 10	10.0 – 15.0	-0.02	0.3	-
		2.98	0.58	-
		5.98	1.12	-
		8.98	1.64	1.83
		5.98	1.12	-
		2.98	0.58	-
		-0.02	0.26	-
BD – 10	15.0 – 20.0	-0.75	0.4	-
		3.92	0.8	-
		6.92	1.3	-
		9.92	1.9	1.92
		6.92	1.2	-
		3.92	0.6	-
		0.92	0.2	-
BD – 10	25.0 – 30.0	0.34	0.4	-
		3.34	1.0	-
		6.34	1.8	-
		9.34	2.4	2.61
		6.34	1.7	-
		3.34	1.3	-
		0.34	0.7	-

Hole No.	Test Section, m	Total Pressure Kg/Cm^2	Unit Infiltration Volume Q (lit/min/m)	Lu Value
DHT – 4	35.0 – 40.0	1.16	0.8	-
		4.16	1.40	-
		7.16	2.00	2.80
		4.16	1.36	-
		1.16	0.56	-
DHT – 4	55.0 – 60	1.14	0.6	-
		4.17	1.4	-
		7.17	3.0	-
		10.17	3.7	3.68
		7.17	2.9	-
		4.17	1.9	-
		1.17	0.8	-

Laboratory Test

Table B3.8.1 Results of Laboratory Test with Stratigraphy

AGE	GROUP	FORMATION	SYMBOL	ROCK TYPE	RESULTS OF LABORATORY TESTS: Unit Weight (g/cm3)				Absorption (%)				Specific Gravity (g/cm3)				Uniaxial Comp Strength (MPa)				Ultra Sonic Velocity (m/sec)				Tensile Strength (MPa)			
					Max	Min	Ave	Data Nos	Max	Min	Ave	Data Nos	Max	Min	Ave	Data Nos	Max	Min	Ave	Data Nos	Max	Min	Ave	Data Nos	Max	Min	Ave	Data Nos
Cenozoic	Recent Deposits		Rd	Riverbed deposits																								
Cenozoic	Recent Deposits		Ta	Talus and/or Terrace																								
Cenozoic	Siwalik Group	Unconformity	Sw	Conglomerate, Sandstone, Mudstone	2.56	2.00	2.31	13	4.65	0.99	2.87	5	2.77	2.66	2.72	5	26.3	4.2	13.7	12	3,833	1,715	2,419	7	4.27	0.94	2.06	8
		Mahabharat Thrust																										
Paleozoic	Upper Nawakot Group	Robang Formation	Phy (2)	Phyllite (2)																								
Paleozoic	Upper Nawakot Group	Robang Formation	Qz	Quartzite																								
Paleozoic	Upper Nawakot Group	Robang Formation	Phy (1)	Phyllite (1)	3.04	2.65	2.83	13	1.49	0.46	0.98	14	2.75	2.65	2.70	14	32.1	5.9	18.4	9	6,936	2,328	4,927	18	12.6	2.51	7.09	17
Paleozoic	Upper Nawakot Group	Malekhu Limestone	Dl	Siliceous Dolomite	2.89	2.47	2.78	42	2.00	0.20	0.55	44	2.78	2.64	2.73	43	106	5.35	45.6	43	6,731	4,010	5,638	56	18.6	3.4	9.83	56
Paleozoic	Upper Nawakot Group	Berighat Slate	Sl	Slate(Phyllitic)	2.64	2.58	2.61	2	0.28	0.28	0.28	1	2.75	2.75	2.75	1	48.5	20.5	34.5	2	4,902	4,429	4,665	2	-	-	-	-
		Main Boundary Thrust																										
Pre-Cambrian	Bhimphedi Group	Kalitar Formation	Sq	Schist, Quarzite																								
Pre-Cambrian	Bhimphedi Group	Bhaise Dobhan Marble	Mb	Limestone	2.72	2.62	2.67	15	0.47	0.16	0.31	13	2.79	2.65	2.72	13	51.4	12.7	30.3	15	6,703	2,988	5,663	18	14.6	1.28	7.04	17
Pre-Cambrian	Bhimphedi Group	Raduwa Formation	Sch	Schist																								

Table B3.8.2 Summary of Laboratoty Test								
Drill Hole	Depth	Rock	Unit weight	Absoption	Specific Gravity	Uniaxial Compressive Strength	P-wave velocity	Tensile strength
BD-4	4.40 - 4.65	Phyllite	2.93	-	-	262.41	-	-
	6.65 - 6.85	Phyllite	2.92	-	-	134.95	-	-
	7.50 - 7.65	Phyllite	-	1.30	2.74	-	-	-
	8.02 - 8.22	Phyllite	2.90	-		59.98	-	-
	8.15 - 8.25	Phyllite	2.81	0.74	2.75	104.96	-	-
	9.00 - 9.25	Phyllite	3.04	1.12	2.71	-	-	-
	9.60 - 9.82	Phyllite	2.75	1.49	2.67	-	-	-
BD-5	11.80 - 11.92	Phyllite	-	-	-	-	6262	11.30
	15.40 - 15.56	Phyllite	-	-	-	-	5976	8.38
	15.40 - 15.56	Phyllite	-	-	-	-	5647	6.86
	21.40 - 21.56	Phyllite	-	-	-	-	6310	6.35
	21.40 - 21.56	Phyllite	-	-	-	-	5196	5.97
	21.40 - 21.56	Phyllite	-	-	-	-	5004	2.75
	12.20 - 12.40	Phyllite	2.84	-	-	299.42	-	-
	22.65 - 22.80	Phyllite	-	0.86	2.66	-	-	-
	35.12 - 35.30	Phyllite	-	-	-	-	2332	2.94
	35.12 - 35.30	Phyllite	-	-	-	-	2328	2.51
	35.12 - 35.30	Phyllite	-	-	-	-	2358	4.25
	36.64 - 36.84	Phyllite	2.82	-	-	320.81	-	-
	49.25 - 49.40	Phyllite	-	0.80	2.70	-	-	-
BD-6	12.14 - 12.34	Phyllite	-	-	-	-	6072	7.81
	12.14 - 12.34	Phyllite	-	-	-	-	6148	4.97
	15.25 - 15.40	Phyllite	-	-	-	-	4921	12.64
	15.25 - 15.40	Phyllite	-	-	-	-	4657	12.36
	16.16 - 16.28	Phyllite	-	-	-	-	3630	8.84
	16.16 - 16.28	Phyllite	-	-	-	-	3595	7.31
	27.20 - 27.32	Phyllite	-	0.84	2.70	-	-	-
	40.50 - 40.70	Phyllite	2.70	-	-	-	-	-
BD-7	4.20 - 4.32	Dolomite	-	-	-	-	6310	14.85
	4.20 - 4.32	Dolomite	-	-	-	-	6308	8.14
	4.20 - 4.32	Dolomite	-	-	-	-	6308	6.51
	5.48 - 5.68	Dolomite	2.84	-	-	74.86	-	-
	6.30 - 6.43	Dolomite	-	-	-	-	6384	3.96
	6.30 - 6.43	Dolomite	-	-	-	-	6731	3.40
	6.30 - 6.43	Dolomite	-	-	-	-	5807	3.59
	12.12 - 12.25	Dolomite	2.47	1.12	-	53.47	-	-
	24.66 - 24.91	Dolomite	2.88	2.00	2.74	534.69	-	-
	33.42 - 33.58	Dolomite	-	1.10	2.76	419.86	-	-
	38.15 - 38.30	Dolomite	-	-	-	-	6005	11.09
BD-10	6.50 - 6.65	Dolomite	-	0.79	2.72	-	-	-
	7.60 - 7.82	Dolomite	-	-	-	-	5811	7.27
	7.60 - 7.82	Dolomite	-	-	-	-	6451	6.86
	7.60 - 7.82	Dolomite	-	-	-	-	6358	8.62
	8.10 - 8.35	Dolomite	2.80	0.50	2.70	329.89	-	-
	8.50 - 8.70	Dolomite	-	1.15	2.72	-	-	-
	9.00 - 9.25	Dolomite	2.65	1.44	2.66	-	-	-
	10.25 - 10.44	Dolomite	-	1.77	2.76	-	-	-
	12.40 - 12.60	Dolomite	-	-	-	-	6103	6.84
	12.40 - 12.60	Dolomite	-	-	-	-	6298	11.58
	12.60 - 12.82	Dolomite	-	1.57	2.69	-	-	-
LS-1	27.82 - 27.94	Phyllite	-	1.42	2.72	-	-	-
LS-2	16.52 - 16.80	Phyllite	2.87	1.46	2.71	179.94	-	-
	23.28 - 23.40	Phyllite	-	-	-	-	6936	6.10
	23.28 - 23.40	Phyllite	-	-	-	-	6382	9.25

Drill Hole	Depth			Rock	Unit weight	Absoption	Specific Gravity	Uniaxial Compressive Strength	P-wave velocity	Tensile strength
	24.20	-	24.40	Phyllite	2.78	0.70	2.71	128.72	-	-
	24.70	-	24.90	Phyllite	-	0.93	2.67	-	-	-
BO-1	9.00	-	9.20	Phyllite	-	0.67	2.68	-	-	-
	10.25	-	10.45	Phyllite	2.72	0.94	2.65	164.94	-	-
	18.30	-	18.55	Phyllite	2.65	0.46	2.66	-	-	-
BTO-1	10.00	-	10.14	Sandstone	2.36	-	-	42.77	-	-
	11.82	-	12.00	Sandstone	-	-	-	-	3833	3.34
	11.82	-	12.00	Sandstone	-	-	-	-	3731	4.27
	17.50	-	17.67	Sandstone	2.53	1.58	2.66	171.10	-	-
DHT-4	16.74	-	16.87	Sandstone	-	0.99	2.77	262.61	-	-
	23.46	-	23.63	Sandstone	2.33	-	-	149.71	-	-
	24.88		25.00	Sandstone	-	-	-	-	1717	1.29
	29.61	-	29.79	Sandstone	2.00	-	-	53.47	-	-
	36.00	-	36.16	Sandstone	2.39	-	-	262.61	-	-
	50.70	-	50.81	Sandstone	-	-	-	-	1854	1.44
	50.70	-	50.81	Sandstone	-	-	-	-	1715	1.38
	54.88	-	55.00	Sandstone	2.56	-	-	58.36	-	-
	72.50	-	72.66	Sandstone	2.12	-	-	-	-	-
DHT-6	8.09	-	8.25	Sandstone	-	4.41	2.72	-	-	25
	14.63	-	14.79	Sandstone	2.46	-	-	-	-	
	22.73	-	22.86	Sandstone	-	-	-	-	1989	0.94
	22.73	-	22.86	Sandstone	-	-	-	-	2100	1.3
	26.86	-	27.00	Sandstone	2.09	-	-	58.50	-	-
	28.24	-	28.35	Sandstone		2.73	2.75		-	-
	29.84	-	30.00	Sandstone	2.31	-	-	128.32	-	-
	33.37	-	33.51	Sandstone	2.30	-	-	117.63	-	-
	38.41	-	38.59	Sandstone	2.29	-	-	235.26	-	-
	57.80	-	57.94	Sandstone		4.65	2.72		-	-
	58.00	-	58.12	b	2.31	-	-	107.00	-	-
BPV-1	4.40		4.53	Dolomite	2.60	-	-	176.20	-	-
	5.60		5.73	Dolomite	-	-	-	-	4010	18.59
	9.45		9.57	Dolomite	-	0.50	2.75	-	-	-
	9.85		9.96	Dolomite	-	-	-	-	4159	8
	14.40		14.54	Dolomite	-	0.46	2.78	-	-	-
	17.45		14.54	Dolomite	2.84			295.30	-	-
	17.61		17.35	Dolomite	-	0.27	2.74	-	-	-
	17.85		18.00	Dolomite	-	-	-	-	6138	11.92
	20.82		20.92	Dolomite	-	-	-	-	6043	7.43
	22.15		22.27	Dolomite	-	0.36	2.69	-	-	-
	22.30		22.40	Dolomite	-	0.29	2.70	-	-	-
	22.80		22.90	Dolomite	-	0.40	2.77	-	-	-
	22.90		23.00	Dolomite	-	0.35	2.68	-	-	-
	24.80		24.95	Dolomite	-	-	-	-	5818	6.96
	24.80		24.95	Dolomite	-	-	-	-	5795	8.98
	24.80		24.95	Dolomite	-	-	-	-	5595	7.82
	25.60		25.77	Dolomite	2.82	-	-	383.54	-	-
	27.60		27.74	Dolomite	-	0.33	2.73	-	-	-
	29.10		29.23	Dolomite	2.81	-	-	410.85	-	-
	31.20		31.40	Dolomite	-	0.35	2.77	-	-	-
	35.30		35.47	Dolomite	-	0.38	2.76	-	-	-
	35.55		35.70	Dolomite	-	-	-	-	5305	5.78
	36.10		36.25	Dolomite	-	0.39	2.66	-	-	-
	38.90		39.00	Dolomite	-	-	-	-	5252	8.46
	43.00		43.17	Dolomite	2.82	-	-	385.17	-	-
	44.53		44.69	Dolomite	2.76	-	-	407.37	-	-

Drill Hole	Depth		Rock	Unit weight	Absoption	Specific Gravity	Uniaxial Compressive Strength	P-wave velocity	Tensile strength
	46.50	46.65	Dolomite	2.75	–	–	331.41	–	–
BPV-1	48.42	48.58	Dolomite	2.88	–	–	529.43	–	–
	49.60	49.74	Dolomite	–	0.34	2.64	–	–	–
	51.60	51.75	Dolomite	–	0.41	2.76	–	–	–
	57.55	57.69	Dolomite	–	0.31	2.78	–	–	–
	57.70	57.82	Dolomite	–	–	–	–	4922	12.05
	58.00	58.13	Dolomite	–	–	–	–	5946	12.53
	59.00	59.15	Dolomite	–	–	–	–	6289	11.93
	59.00	59.15	Dolomite	–	–	–	–	6289	10
	61.76	61.90	Dolomite	2.82	–	–	715.94		
	65.00	65.14	Dolomite	–	–	–	–	5573	12.96
	65.00	65.14	Dolomite	–	–	–	–	5621	9.03
	66.80	66.93	Dolomite	–	0.39	2.76	–	–	–
	68.78	68.92	Dolomite	2.83	–	–	921.66	–	–
	69.86	70.00	Dolomite	–	–	–	–	5910	12.7
	70.40	70.60	Dolomite	–	–	–	–	5826	8.77
	74.80	74.90	Dolomite	–	–	–	–	5871	5.93
	74.80	74.90	Dolomite	–	–	–	–	5667	4.71
	75.10	75.24	Dolomite	2.81	–	–	462.19	–	–
	79.50	79.69	Dolomite	2.80	–	–	924.38	–	–
	84.80	94.98	Dolomite	–	0.30	2.77	–	–	–
	86.00	86.13	Dolomite	–	–	–	–	4625	10.55
	86.30	86.44	Dolomite	2.84	–	–	667.61	–	–
	86.84	87.00	Dolomite	–	–	–	–	6029	4.82
	90.50	90.65	Dolomite	–	–	–	–	6443	10.45
	92.10	92.26	Dolomite	–	–	–	–	5557	13.43
	92.10	92.26	Dolomite	–	–	–	–	5767	8.97
	92.50	92.68	Dolomite	2.81	–	–	873.03		–
	93.40	93.54	Dolomite	2.83	–	–	308.13	–	–
	93.55	93.69	Dolomite	2.85	–	–	231.10	–	–
	95.15	95.36	Dolomite	2.82	–	–	1063.13	–	–
	96.50	96.70	Dolomite	–	–	–	–	5924	12.37
	96.50	96.70	Dolomite	–	–	–	–	5189	11.73
	96.50	96.70	Dolomite	–	–	–	–	5276	8.98
	98.20	98.33	Dolomite	–	–	–	–	6041	9.71
	98.20	98.33	Dolomite	–	–	–	–	6040	6.71
	99.60	99.74	Dolomite	–	0.42	2.65	–	–	–
	100.75	100.98	Dolomite	–	–	–	–	5798	7.52
	100.75	100.98	Dolomite	–	–	–	–	5674	10.01
	102.10	102.27	Dolomite	–	0.34	2.75	–	–	–
	108.75	108.91	Dolomite	2.69	–	–	139.22	–	–
	112.10	112.25	Dolomite	–	0.39	2.66	–	–	–
BPH	3.40	3.53	Dolomite	2.74	–	–	506.25	–	–
	5.60	5.73	Dolomite	2.74	–	–	138.86	–	–
	7.70	7.87	Dolomite	2.81	–	–	555.42	–	–
	10.40	10.55	Dolomite	–	0.26	2.70	–	–	–
	11.30	11.43	Dolomite	–	0.34	2.73	–	–	–
	14.00	14.14	Dolomite	2.83	–	–	484.44	–	–
	16.12	16.23	Dolomite	–	0.55	2.70	–	–	–
	18.85	19.00	Dolomite	–	–	–	–	4755	16.35
	19.75	19.90	Dolomite	–	–	–	–	4311	13.38
	19.85	20.00	Dolomite	–	0.40	2.73	–	–	–
	20.75	20.90	Dolomite	–	–	–	–	4957	16.24
	20.75	20.90	Dolomite	–	–	–	–	4375	18.5
	22.20	22.36	Dolomite	–	–	–	–	4901	9.4

Drill Hole	Depth		Rock	Unit weight	Absoption	Specific Gravity	Uniaxial Compressive Strength	P-wave velocity	Tensile strength
	22.20	22.36	Dolomite	−	−	−	−	4079	13.17
BPH	23.20	23.69	Dolomite	2.79	−	−	921.22	−	−
	29.42	29.60	Dolomite	2.83	−	−	513.56	−	−
	29.80	30.00	Dolomite	2.76	−	−	855.97	−	−
	31.05	31.20	Dolomite	−	0.26	2.74	−	−	−
	33.60	33.71	Dolomite	−	0.64	2.77	−	−	−
	36.00	36.13	Dolomite	−	0.24	2.74	−	−	−
	41.50	41.66	Dolomite	2.75	−	−	141.23	−	−
	44.20	44.35	Dolomite	−	−	−	−	5094	10.79
	44.20	44.35	Dolomite	−	−	−	−	5985	10.95
	44.20	44.35	Dolomite	−	−	−	−	5515	11.09
	44.80	44.92	Dolomite	−	0.36	2.67	−	−	−
	45.00	45.13	Dolomite	2.78	−	−	282.10	−	−
	44.20	44.31	Dolomite	2.76	−	−	177.97	−	−
BPV-2A	13.20	13.40	Dolomite	2.89	0.22	2.71	773.70	−	−
	14.90	15.00	Dolomite	2.73	0.39	2.78	−	−	−
	19.00	19.10	Dolomite	2.69	0.34	2.77	343.00	−	−
BPV-2B	31.00	31.15	Dolomite	−	−	−	−	5162	4.32
	31.35	31.48	Dolomite	−	−	−	−	6072	12.08
	31.35	31.48	Dolomite	−	−	−	−	6172	12.27
	31.50	31.63	Dolomite	−	−	−	−	5121	9.4
	32.50	32.60	Dolomite	2.80	0.46	2.75	401.24	−	−
	34.40	34.70	Dolomite	2.79	0.20	2.78	308.34	−	−
	57.35	57.53	Dolomite	2.87	0.47	2.74	503.13	−	−
	70.70	70.82	Dolomite	−	0.36	2.74	−	−	−
BPV-3	2.50	2.60	Dolomite	2.87	0.20	2.73	355.13	−	−
	2.65	2.75	Dolomite	2.66	−	−	405.29	−	−
	16.50	16.50	Dolomite	2.65	−	−	404.15	−	−
BCT-1	4.80	5.00	Limestone	2.69	0.23	2.71	177.06	−	−
	5.60	5.70	Limestone	−	0.24	2.71	−	−	−
	10.30	10.50	Limestone	2.62	0.45	2.75	244.07	−	−
	11.90	12.00	Limestone	−	−	−	−	6308	4.77
	12.40	12.52	Limestone	2.64	0.26	2.65	306.99	−	−
	12.75	12.90	Limestone	2.70	0.47	2.75	513.82	−	−
	13.25	13.50	Limestone	2.64	0.16	2.75	503.06	−	−
	14.00	14.12	Limestone	−	−	−	−	6366	5.33
	14.00	14.12	Limestone	−	−	−	−	6703	4.66
	14.25	14.50	Limestone	2.66	0.45	2.74	391.52	−	−
	14.52	14.65	Limestone	−	−	−	−	6355	3.49
	15.00	15.20	Limestone	2.63	0.19	2.73	227.01	−	−
	15.80	15.94	Limestone	2.68	−	−	281.01	−	−
	19.00	19.10	Limestone	−	0.23	2.70	−	−	−
	20.13	20.30	Limestone	−	−	−	−	6327	12.08
	20.13	20.30	Limestone	−	−	−	−	6110	9.71
	22.70	22.90	Limestone	−	−	−	−	5731	9.67
	22.70	22.90	Limestone	−	−	−	−	6133	10.04
	29.25	29.38	Limestone	−	−	−	−	6469	10.25
	30.25	30.38	Limestone	−	−	−	−	6411	10.84
	30.25	30.38	Limestone	−	−	−	−	6132	5.29
	30.65	30.90	Limestone	2.68	0.47	2.74	305.25	−	−
	32.30	32.42	Limestone	2.68	0.24	2.69	332.57	−	−
	34.60	34.75	Limestone	−	−	−	−	6360	7
	34.60	34.75	Limestone	−	−	−	−	6637	6.8
	39.00	39.25	Limestone	2.72	0.39	2.71	127.37	−	−
BCT-2	20.00	20.22	Slate	2.64	−	−	485.37	−	−

Drill Hole	Depth		Rock	Unit weight	Absoption	Specific Gravity	Uniaxial Compressive Strength	P-wave velocity	Tensile strength
BCT-2	23.62	23.74	Slate	-	-	-	-	-	-
BCT-2	36.50	36.60	Slate	2.58	0.28	2.75	204.95	-	-
BMT-1	bbb	12.70	Schist	-	0.55	2.70	-	-	-
	16.20	16.34	Schist	-	0.29	2.63	-	-	-
	16.31	16.45	Schist	-	-	-	-	4902	-
	67.30	67.46	Schist	-	-	-	-	4429	-
	76.25	76.40	Schist	-	0.33	2.72	-	-	-
	76.68	76.80	Schist	2.78	-	-	1077.50	-	-
BI-2	3.59	3.84	Limestone	2.72	0.31	2.79	411.06	-	-
	6.45	6.55	Limestone	2.66	-	-	128.27	-	-
	8.00	8.15	Limestone	2.63	-	-	281.40	-	-
	8.45	8.57	Limestone	2.66	-	-	308.29	-	-
	27.70	27.85	Limestone	-	-	-	-	3481	-
	29.00	29.14	Limestone	-	-	-	-	2988	14.59
	29.40	29.56	Limestone	-	-	-	-	4039	1.28
	29.40	29.56	Limestone	-	-	-	-	4591	1.74
	29.40	29.56	Limestone	-	-	-	-	4796	2.06

1. Rock Quality Designation		RQD
A	Very poor	0 - 25
B	Poor	25 - 50
C	Fair	50 - 75
D	Good	75 - 90
E	Excellent	90 - 100

Note: (i) Where RQD is reported or measured as ≤ 10(including 0), a nominal value of 10 is used to evaluate Q.
(ii) RQD intervals of 5, i.e. 100, 95, 90 etc. are sufficiently accurate.

2. Joint Set Number		Jn
A	Massive, no or few joints	0.5 - 1.0
B	One joint set	2
C	One joint set plus randam joints	3
D	Two joint sets	4
E	Two joint sets plus randam joints	6
F	Three joint sets	9
G	Three joint sets plus randam joints	12
H	Four or more joint sets, random, heavily jointed,"sugar cube", etc	15
J	Crushed rock, earthlike	20

Note: (i) For instance, use (3.0 x Jn)
(ii) For portals, use (2.0 x Jn)

3. Joint Roundness Number		Jr
a) Rock wall contact, and b) Rock wall contact before 10 cm shear		
A	Discontinuous joints	4
B	Rough or irregular, undulating	3
C	Smooth, undulating	2
D	Slickensided, undulating	1.5
E	Rough or irregular, planar	1.5
F	Smooth, planar	1.0
G	Slickensided, planar	0.5
c) No rock-wall contact when sheared		
Note: (i) Descriptions refer to small scale features and intermediate scale features, in that order.		
H	Zone containing clay minerals thick enough to prevent rock wall contact	1.0
J	Sandy, gravelly or crushed zone thick enough to prevent rock wall contact	1.0

Note: (i) Add 1.0 if the mean spacing of the relevant joint set is greater than 3m.
(ii) Jr = 0.5 can be used for planar slickensided joints having linearions, provided the lineations are orientated for minimum strength.

4. Joint Alteration Number		Φr (approx.)	Ja
(a) Rock wall contact (no mineral fillings, only coatings)			
A	Tightly healed, hard, non-softening, Impermeable filling, I.e. quartz or epidote		0.75
B	Unaltered joint walls, surface staining only	25 to 35°	1.0
C	Slightly altered joint walls. Non-softening mineral coatings, sandy particles, clay-free disintegrated rock etc.	25 to 30°	2.0
D	Silty-, or sandy-clay coatings, small clay fraction (non-softening)	20 to 25°	3.0
E	Softening or low friction clay mineral coatings, i.e. kaolinite, mica. Also chlorite, talc, gypsum and graphite etc., and graphite etc., and small quantities of swelling clays. (Discontinuous coatings, 1-2 mm or less in thickness)	8 to 16°	4.0
(b) Rock wall contact before 10 cm shear (thin mineral fillings)			
F	Sandy particles, clay-free disintegrated rock, etc.	25 to 30°	4.0
G	Strangely over-consolidated, non-softening caly mineral fillings (continuous, but <5mm in thickness)	16 to 24°	6.0
H	Medium or low over-consolidation, softening, clay mineral fillings (continuous, but <5mm in thickness)	12 to 16°	8.0
J	Swelling-clay filling, i.e., montmorillonite (continuous, but <5mm in thickness). Value of Ja depends on percent of swelling clay-size particles, and access to water etc.	6 to 12°	8.0 to 12.0
(c) No rock wall contact when sheared (thickness mineral fillings)			
KL M	Zones or bands of disintergarated or crushed rock and clay (see G.H. J for description of clay condition)	6 to 24°	6.0, 8.0 or 8.0 to 12.0
N	Zones or bands of silty or sandy clay, small clay fraction (non-softening)	-	5.0
OP R	Thick, continuous zones or bands of clay (see G,H,J for description of clay condition)	8 to 24°	10.0,13.0 or 13.0 to 20.0

Note (i) Values of (Φ)r are intended as an approximate guide to the mineralogical properties of the alteration products, if present.

5. Joint Water Reduction Factor		Approx. Water pressure(kg/cm2)	Jw
A	Dry excavation or minor inflow, i.e. 5l/min locally	<1.0	1.0
B	Medium inflow or pressure, occasional outwash of joint fillings	1.0 to 2.5	0.66
C	Large inflow or high pressure in competent rock with unfilled joints	2.5 to 10.0	0.5
D	Large inflow or high pressure, considerable outwash of joint fillings	2.5 to 10.0	0.33
E	Exceptionally high inflow or water pressure at blasting, decaying with time	>10.0	0.2 to 0.1
F	Exceptionally high inflow or water pressure continuing without noticeable decay	>10.0	0.1 to 0.05

Note: (i) Factors C to F are crude estimates. Increase Jw if drainngs measures are installed
(ii) Special problems caused by ice formation are not considered.

6. Stress Reduction Factor		SRF
a) Weakness zones intersecting excavation, which may causes loosening of rock mas when tunnel is excavated.		
A	Multiple occurrences of weakness zones containing clay or chemically distintigrated rock, very loose surrounding rock. (any depth)	10.0
B	Single weakness zones containing clay or chemically disintegrated rock (depth of excavation ≤ 50m))	5.0
C	Single weakness zones containing clay or chemically disintegrated rock (depth of excavation > 50m))	2.5
D	Multiple shear zones in competent rock (clay-free) loose surrounding rock (any depth)	7.5
E	Single shear zones in competent rock (clay-free), (depth of excavation ≤ 50m))	5.0
F	Single shear zones in competent rock (clay-free), (depth of excavation > 50m)	2.5
G	Loose, open joints, heavily jointed or "sugar cube", etc. (any depth)	5.0

Note: (i) Reduce these values of SRF by 25-50% if the relevant shear zones only influence but do not intersect the excavation.

b) Competent rock, rock stress problems		σ_c/σ_1	σ_t/σ_1	SRF
H	Low stress, near surface, open joints	>200	>13	2.5
J	Medium stress, favorable stress condition	200 to 10	13 to 0.66	1.0
K	High stress, very tight structure. Usually favourable to stability, may be unfavourable for wall stability.	10 to 5	0.66 to 0.33	0.5 to 2.0
L	Moderately stabbing after > 1 hour in massive rock	5 to 2.5	0.33 to 0.16	5.0 to 10.0
M	Heavy rock burst (strain-burst) and immediate dynamic deformation in massive rock	<2.5	<0.16	10.0 to 20.0

Note: (i) For strongly anisotropic virgin stress field (if measured): when $5 \leq \sigma_1/\sigma_3 \leq 10$ reduce σ_c to $0.8\sigma_c$ and σ_t to $0.8\sigma_t$. When $\sigma_1/\sigma_3 > 10$ reduce σ_c, σ_t to $0.6\sigma_c$, $0.6\sigma_t$ respectively. Where: σ_c = unconfined compression strength, σ_t = tensile strength (point load), σ_1 and σ_3 =major and minor principal stress (estimated from elastic theory).
(ii) Few case records available where depth of crown below surface is less than span width. Suggest SRF increase from 2.5 to 5 for such cases (see H).

c) Squeezing rock : plastic flow of incompletent rock under the influence of high rock pressure		SRF
N	Mild squeezing rock pressure	5.0 to 10.0
O	Heavy squeezing rock pressure	10.0 to 20.0

Note: (iv) Cases of squeezing rock may occur for depth $H > 350.0^{1/3}$ (Singh et al., 1992). Rock mass compression strength can be estimated from $Q = 0.7\gamma Q^{1/3}$(MPa) where γ = rock density in kN/m^2 (Singh, 1993)

d) Swelling rock: chemical selling activity depending on presence of water		
P	Mild swelling rock pressure	5.0 to 10.0
R	Heavy swelling rock pressure	10.0 to 15.0

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}$$

Table B4.1.1. Rating for the six Q system Parameters

Table B4.1.2 Rock Classification by CRIEPI (1/2)

Rock Class	Subdivision	Observation in the Test Adit Condition of Rock
A	A, I, a	Fresh and hard, no deterioration in the rock-forming minerals. Crack spacing lager than 50 cm. Cracks are closely adhered, no deterioration nor discoloration.
B	A, II-III, b	Hard Rock color is light brown. Crack spacing about 15-50cm. Limonite adhered along cracks.
CH	B, III-IV, b-c	Relatively hard Biotite and plagioclase are somewhat deteriorated. Crack spacing about 5-30 cm. Very thin clay is sandwiched along the opening.
CM	C, IV-V、c	Breaks when struck by hammer. Deterioration of plagioclase developed. Crack spacing smaller than 15 cm. Clay is sandwiched along the opening face.
CL	C-D, III, a-b ; C, IV-V, d	Biotite turns golden color, but quartz particles are hard.Plagioclase is deteriorated. When struck by hammer breaksinto pieces. Crack spacing smaller than 5 cm.
DH	D, II-III, b ; D, III, a-b	Can be broken by hand. It is easy to break by hammer. Biotite turning to golden color, and brown in the periphery. Particles are hard, forming small, sand-like pieces. Apparent spacing of cracks becomes wider.
DM	E1, I-II, b-c ; E1, II, b	Breaking by hand, it becomes sand-like remaining crystal of quartz and potassium feldspar. Mica loses its crystal form and plagioclase is mostly deteriorated. Apparent spacing of cracks becomes even wider.
DL	E2, I, c	Breaking by hand, mostly becomes powder, expect for party sand form. Most feldspar is deteriorated and becomes clayish soil. Original joint planes become indistinguishable.

Table B4.1.2 Rock Classification by CRIEPI (2/2)

Class	Criteria for Judgement
A	When struck by hammer, rock piece cannot be broken easily, with metallic sound. Fresh, no deterioration of rock-forming minerals.
B	When struck by hammer, makes metallic sound-resonant sound. Joint are adhered, fresh.
C	Rock becomes broken when struck lightly by hammer, making resonant sound. (Smashing by finger-pressure for more than 20 times, rock piece keeps almost intact)
D	Crushing by finger-pressure barely being possible, each piece is hard with feldspar remained in the periphery of the quartz. (fragmental-sandy) (Rock pieces become broken by 7-10 times finger crushing with more than 70% medium-small pieces)
E1	Crushed when squeezed with finger, remaining particles of quartz and potassium feldspar. (Pieces become broken by 3-5 times finger crushing with 30-50% in powder form, 50-90% in small pieces)
E2	Generally in powder form when crushed by finger-pressure in the palm partly sand form. (Pieces become broken by 1-3 times finger crushing with more than 50-70% in powder form)

Class	Judgement Criteria
I	Larger than 50 cm
II	50 - 30cm
III	30 - 15 cm
IV	15 .- 5 cm
V	Smaller than 5 cm

Class	Judgement Criteria
a	Closely adhered, no deterioration or dis-colouring
b	Adhesion of limonite along adhered cracks orvery thin clay (brown in color) is sandwiched
c	Deterioration along crack, about 1-2 cm clay(white-greyish white) is sandwiched
d	Opening