CHAPTER 4 SITE CONDITION

4.1 General

The field investigations of site conditions, inclusive of topography, meteo-hydrology, geology, construction materials and environment, have been carried out in Stage I and II. All findings are provided in this Chapter. Fig.-4.1.1 illustrates the major investigations carried out.

4.2 Topography

4.2.1 General Damsite Topography

The Kali Gandaki and the Trisulganga which flow in Central Nepal, meet each other at the foothill of the Mahabharat Range in the southern part of Central Nepal. After the confluence, the river is named as the Sapt Gandaki River, which flows in an about 2.5 km long reach through a gorge confined with low cliffs in both banks, and then, finally enters into the Inner Terai Plain.

The watershed of the Sapt Gandaki faces the Karnali and West Rapti basins in the west and to the Sun Koshi and Bagmati basin in the east. The catchment area of the Sapt Gandaki is 31,100 km^2 measured at the damsite located at about 1 km downstream from the confluence of the Kali Gandaki and the Trisulganga. The elevation of the basin ranges from EL.8,000 m at the peak of the Great Himalayan Range to EL.180 m at the river bed of the damsite. The mean slope of the river channel is around 0.0015.

The damsite is situated in the gorge confined with low cliffs on both banks just before entering into the Inner Terai Plain. The river channel is about 200 m in width and the elevation of the river bed is at EL.180 m. Both banks form a tableland covered with forest.

4.2.2 Available Topographic Data

Existing Topographic Data: The following existing topographic data were gathered through data collection during the field investigation.

(i) Aerial photographs covering the project area.

(ii) Topographic map of 1 to 10,000 in scale covering the Sapt Gandaki River basin.

(iii) Topographic map of 1 to 63,360 in scale covering the whole Nepal.

(iv) Data of the Indian Survey Triangulation Stations near the project area.

(v) Data of the existing bench marks which are used in the Chitwan Valley Development Project and the Road Department, HMG.

Additional Topographic Data: In order to carry out a detailed study of the project, preparation of more accurate topographic maps of the project area in a larger scale, including establishment of bench marks and control points, and checking of the existing maps or topographic data, was considered necessary. Thus, various topographic surveys were carried out in the field investigation, and the following topographic data of the project area were added to the existing ones.

(i) Topographic map of 1 to 500 in scale covering the area of 169 ha around the damsite:

The above was prepared through the direct ground survey by control point survey and plane table survey and is usable for the design of various structures of the project. The area covered by the above map is shown in Fig.-4.2.1.

(ii) Topographic map of 1 to 2,000 in scale covering the project area of 4 km^2:

This topographic map was prepared by aero-photo mapping, using the existing aerial photographs. This is usable for overall project planning including the temporary construction facilities, access roads and accommodations, etc. and also for the geological analysis. The area covered by the above topographic map are indicated in Fig.-4.2.2.

(iii) Topographic map and longitudinal section along the Mugling Road:

These were prepared through the direct ground survey by traverse survey, levelling and plane table survey to estimate the length of the Mugling Road to be submerged by the reservoir and its necessary relocation cost. The topographic map and longitudinal section along the Mugling Road cover up to about 15 km upstream from the damsite.

(iv) River cross sections from the damsite to about 4 km downstream:

The river cross sections are taken at 250 m interval from the damsite to about 4 km downstream, to establish the rating curve by investigating the exact topography of the river bed, to measure the future possible river bed change and to make necessary adjustment of the rating curve.

The river cross sections taken are shown in Fig.-4.2.3.

(v) New bench marks and control points:

Eight new bench marks were installed in the project area by direct levelling from the existing bench mark. The positions of these new bench marks are as shown in Fig.-4.2.4 together with the control points of the topographic survey carried out.

The coordinate and elevation of each point are as given in Table-4.2.1.

All the topographic data which have become available through the topographic survey in the field investigation are presented in ANNEX (A) separately.

4.3 Meteorology and Hydrology

4.3.1 General

The field investigation for meteo-hydrology were carried out in Stage I and II of the feasibility study. The results of the field investigation and analysis are detailed in ANNEX (C) in a separate volume. The objectives of meteo-hydrologic investigations are summarized as follows.

- To collect and review meteo-hydrologic data in and around the Sapt Gandaki Basin.
- To carry out the check measurements of the streamflow at the Gaging Station 450 (Narayangark) for the purpose of examining the accuracy of the existing rating curve and constructing a new rating curve if necessary.
- To determine the long-term discharges at the proposed damsite using the rating curves employed and to generate the runoff data for the interrupted period of water level observation.
- To estimate the frequency and magnitude of the floods for the design of the dam and appurtenant structures of the project.
- To estimate the rate of sediment transported into the reservoir and to clarify the water quality of the Sapt Gandaki River.

In Stage I, the field investigation was focused on the discharge measurement of low flow, while that of high flow were conducted in Stage II. It covered the period from the early of August to the end of November so that the existing rating curve could be checked in a wide range.

The Department of Irrigation, Hydrology and Meteorology (hereinafter called DIHM) is responsible for collecting and analyzing meteorological and hydrologic data in Nepal. During the field survey period, the meteo-hydrologic data relevant to the Sapt Gandaki basin were collected from the DIHM Office in Kathmandu as much as possible.

4.3.2 Meteorology

(1) General features of the basin climate

The climate of Nepal is strongly affected by the southeast monsoon during the wet season and the northwest monsoon during the dry season. The wet season lasts for four months from June to September, while the dry season is from November to April. May and October are the transition period of these seasons. In general, it is humid and hot in the wet season or summer, while it is dry and cold in the dry season or winter. 60 to 80% of the yearly rainfall occurs in the wet season due to the influence of the southeast monsoon.

The elevation of the basin varies widely from over 8,000 m at the peak of the Great Himalayan Range to EL.180 m at the damsite. The Sapt Gandaki River basin can be climatologically divided into four zones in accordance with the elevation encountered, namely Himalayan, Tibetan, Middle and Inner Terai zones.

The Himalayan zone is characterized by the eternal glacier and snow cover which spread over the area exceeding 6,000 m in elevation and which contribute to yielding of streamflow in dry season. In the Tibetan zone which ranges between 3,000 and 6,000 m in elevation, the desertic climate is prevailing and a large quantity of sediment is transported from this zone to the project area in the wet season due to the extremely poor vegetation.

The Middle zone accounts for the major part of the basin. Its elevation varies in a range from 600 to 3,000 m. This zone is relatively populated than other zones. It is responsible for the sediment yielding at the damsite. This is due to its steep topography, deforestation resulting from the development of new cultivated land in the low land area and even in the high mountainous area. The climate in the middle zone is generally temperate.

The Inner Terai zone is located in the south of the Mahabharat Range. Its climate is subtropical. The annual mean air temperature and relative humidity recorded at the Rampur Station are 24.0°C and 74% respectively.

(2) Evaporation

In Nepal, the rate of evaporation in the period from March to October is high due to high temperature, while that in the period from November to February is relatively low due to the northeast wind with low temperature. The evaporation is being observed at sixteen climatologic stations of Nepal by using Class A pan. The annual mean pan A evaporations at these climatologic stations range from 949 mm at the

Jiri station to 2,519 mm at the Dum Kauli station. Their arithmetic mean is calculated at 1,453 mm.

The actual evaporation rate from the Sapt Gandaki reservoir surface is determined by taking 70% of the Pan A evaporation records at Pokhara as shown below.

Monthly Evaporation from the Sapt Gandaki Reservoir (in mm)

Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
49.6	68.6	110.7	123.9	121.5	111.3	112.8	117.2	100.8	86.8	58.8	47.7	1,110.0

(3) Rainfall

In Nepal, the rainfall concentratedly occurs during the wet season from June to September owing to the influence of the southeast monsoon. In the dry season, from November to April, the rainfall amount is extremely small compared with that in the wet season.

There are sixty seven rainfall stations in and around the Sapt Gandaki River basin, among which several stations have been abolished up to now. The annual mean rainfalls at these stations are tabulated in Table-4.3.1. As shown in Table, the annual mean rainfalls distribute in a wide range from 257 mm at Jomsom to 5,149 mm at Lumle. On the basis of these annual mean rainfalls, the isohyetal map for the Sapt Gandaki River basin is established as shown in Fig.-4.3.1. In preparing the map, the annual rainfall in the high mountainous area of the Himarayan and Tibetan zone is determined taking into account the yearly isohyetal maps which are suggested by the DIHM in the "CLIMATOROGICAL RECORDS OF NEPAL" because no rainfall data for the region is available.

The annual monthly mean rainfall at Pokhara, Jomsom and Rampur are shown in the following.

Annual Monthly Mean Rainfall

Month	Station (Sta. No.)		
	Jomsom (601) (1958 - 1980)	Pokhara (803) (1956 - 1975)	Rampur (902) (1967 - 1980)
Jan.	17.1	27.1	21.4
Feb.	13.8	31.2	14.5
Mar.	24.7	54.5	16.9
Apr.	18.6	87.5	41.1
May	9.6	246.9	109.2
Jun.	19.7	649.5	386.2
Jul.	39.6	886.0	526.6
Aug.	46.0	824.3	404.6
Sept.	31.8	575.7	330.7
Oct.	30.9	193.9	85.2
Nov.	6.2	19.3	8.0
Dec.	2.7	8.5	10.0
Annual	260.7	3,640.4	1,954.4

The above table indicates that around 90% of the annual rainfall at the Rampur station occurs in the wet season from June to September. The Rampur station which is located in the Inner Terai zone represents the project area. In this zone, the subtropical climate is prevailing.

The annual rainfall amount at the Jomsom station is very little, compared with those at the Pokhara and Rampur stations. This reveals that the Jomsom station which is located at the southern foothill of the Himalayan range on the Kali Gandaki river is not so strongly influenced by the southeast monsoon as the other two stations. As seen in the Fig.-4.3.1, the annual mean rainfall in the Himalayan and Tibetan zones ranges between 200 and 1,000 mm.

The Pokhara station is situated in the midland zone which forms the major part of the Sapt Gandaki River basin. The seasonal rainfall pattern in this zone is relatively close to that in the Inner Terai zone. However, the annual mean rainfall in this region distributes in a wide range between 1,000 and 6,000 mm due to its variable topography.

4.3.3 Hydrology

(1) Runoff

Water levels of the Sapt Gandaki River has been measured at the regular Stream Gaging Station 450 which is located around 0.5 km downstream from the damsite. The type of observation at the Station is staff gage reading. The observation of water level is usually carried out twice a day in the dry season and more than twice a day in the wet season. The Station was set up in 1963, and the discharge records are available for 18 years from 1963 to 1980 with some interruption in between.

The water levels have been converted into discharges for the period from 1963 to 1968 by using the stage height-discharge rating curve which was established by the DIHM. For the present feasibility study, the JICA team hydrologist conducted discharge measurements at the Gaging Station 450 and set up the new rating curve as shown in Fig.-4.3.2. For the purpose of estimate of streamflow at the damsite, it is assumed that the change in these rating curves occurred at the middle of the period of May 1967 to July 1972 when no discharge measurement was carried out. The monthly mean discharges so obtained are shown in Table-4.3.2 and below.

Unit: m^3/sec

Month	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
Mean	376	303	283	373	572	1,548	3,576	4,231	2,967	1,578	795	520

The annual mean discharges of the Sapt Gandaki River at the damsite is calculated to be 1,436 m^3/sec. As seen in the above Table, around 85% of the annual runoff occurs in the wet and transition seasons from May to October.

The reliability of runoff records at the Sapt Gandaki damsite is roughly checked by comparing the specific runoff with that at the Gaging Station 420 on the Kali Gandaki River. The Gaging Station 420 is located in the Sapt Gandaki River basin, being around 5 km west from the damsite. It covers a catchment area of 11,400 km^2 which accounts for around 37% of the project catchment area of 31,100 km^2. The specific runoff at the Gaging Station 420 for the period from 1964 to 1977 is estimated to be 4.30 $m^3/sec/100\ km^2$, while that at the Gaging Station 450 is derived to be 4.52 $m^3/sec/100\ km^2$ from Table-4.3.2. The specific runoff at the Gaging Station 420 is slightly less than that at the Gaging Station 450. As shown in Fig.-4.3.1, this appears to result from the extremely small rainfall amount in the upper reach of the Kali Gandaki river. Thus, the runoff records at the Sapt Gandaki damsite are judged to be reasonable.

Since the project is planned to be of run-of-river scheme, the frequency of low flow discharges has a significant effect on the dependability of the power plants. As shown in Table-4.3.2, the daily discharge records at the damsite are incomplete for the nine years of 1963, 1971 to 1972, 1974 to 1978 and 1980. For the purpose of constructing the discharge duration curve at the damsite, daily discharges in the wet season are simply generated by applying discharge records at the Gaging Station 420 and a catchment ratio of Gaging Station 450 with the Gaging Station 420. The discharge records for the period of 14 years exclusive of those in 1971, 1972 and 1976 became available for the purpose of examining the frequency of the low flow at the damsite. The discharge duration curve at the damsite is illustrated as shown in Fig.-4.3.3 and the daily discharges for selected exceedance probability are as follows.

	Percentage of time when daily discharge is exceeded									
	10	20	30	40	50	60	70	80	90	100
Discharge (m^3/sec.)	3,890	2,730	1,620	982	684	523	424	343	290	150

The firm discharge for power generation is taken as 290 m^3/sec which is warranted with 90% firmness of duration.

(2) Flood

The flood analysis was made based on the annual maximum water levels recorded at the Gaging Station 450 because the project catchment is too vast to make a unitgraph study and the data to analyse the relationship between rainfall and runoff at the damsite are insufficient for the time being. The flood frequency was made using the log-normal, Gumbel's and Iwai's method as shown in the following table.

Estimated Peak Discharge

(Unit: m^3/sec)

Recurrence Interval in Year	Log Normal	Gumbel's	Iwai's
2	9,100	9,000	8,700
5	10,900	11,300	10,400
10	12,100	12,900	11,500
20	13,100	14,400	12,400
50	14,300	16,300	13,500
100	15,200	17,800	14,400
200	16,100	19,200	15,200
1,000	18,000	22,600	16,900
10,000	20,700	27,400	19,400

As shown in the above table, the peak discharges estimated by the Gumbel's method are larger than those by the other methods for recurrence intervals of more than 5 years. Taking into consideration uncertainties involved in the extreme huge catchment area of 31,100 km^2, it is recommended to adopt the results by the Gumbel's method for the project plan and design.

In order to check the reliability of the above result of the flood frequency analysis, the estimated discharges for the Gaging Station 450 are compared with those for the Gaging Station 420 on the Kali Gandaki River and unusual floods in the whole Nepal by using the following Creager's formula.

$$Q = 46CA^{b}$$

$$b = 0.894A^{-0.048} - 1$$

where, Q: flood peak in second-feet
A: catchment area in square miles
C: Coefficient

The frequency analysis for the Gaging Station 420 was made for the annual maximum water levels for the period from 1964 to 1977 using the Gumbel's method. The above Creager's C corresponding to the estimated peak discharges for the Gaging Station 420 and 450 are tabulated below.

Recurrence Interval in Year	Creager's C	
	Gaging Station 420 (C.A. = 11,400 km^2)	Gaging Station 450 (C.A. = 31,100 km^2
200	62	71
10000	94	100

As shown in the above table, the values of Creager's C for the 200 year and 10000 year flood for the Gaging Station 450 are slightly larger than those for the Gaging Station 420. The value of C is considered to depend on the characteristics of the drainage basin such as rainfall intensity, topography and so on. Therefore, taking into account the similarity of both basins, the results of the frequency analysis for the Gaging Station 450 are judged to be reasonable.

In Fig.-4.3.4, the recorded maximum peak discharges at the respective Gaging Stations in Nepal are plotted against their catchment areas using the said C value. The highest C value, as shown in the Figure, is derived to be 66. Accordingly, the 10,000 year flood of 27,400 m^3/sec at the damsite which corresponds to C = 100 has a considerable allowance for the recorded unusual floods in Nepal. Furthermore, it is generally accepted that the value of C = 100 gives the largest peak flood at any part of the world.

From the above hydrological viewpoints, it is recommended that the spillway has a capacity of discharging the estimated 10,000 year flood at the damsite with an allowance to some extent.

(3) Sediment

The sediment volume transported into the reservoir was estimated based on the results of suspended load measurements at the damsite. Fig.-4.3.5 shows the results of suspended load measurements conducted by the DIHM and the JICA team for the period from October 1975 to June 1980. For the purpose of estimating the long-term sediment inflow into the reservoir, the following sediment rating formula was established based on the said results of suspended load measurements.

$$Qs = 7.53408 \times 10^{-4} \times Q^{2.54268}$$

where, Q and Qs are daily mean discharges in m^3/sec and suspended load in ton/day, respectively.

To estimate the long-term sediment inflow into the reservoir, the followings are assumed.

i) The density of sediment materials is 1.4 in ton/m^3.

ii) The rate of bed load transport is equivalent to 15% of that of the suspended transport.

The annual monthly mean sediment transports which are estimated using the daily mean runoff data at the Gaging Station 450 are shown in the following.

Annual Monthly Sediment Yield

(Unit: 10^3 m^3)

Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
76	39	36	78	244	3,646	26,356	35,791	15,990	4,877	498	168	87,799

The above table reveals that around 93% of the annual sediment transport occurs in the wet season of June to September and that the specific sediment transport of the Sapt Gandaki basin expressed in $m^3/km^2/year$ is around 2,800.

Since the ratio of the gross reservoir storage capacity of the proposed reservoir to the annual inflow discharge at the project site is very low, the trap efficiency of the Sapt Gandaki reservoir is estimated to be very low. Further, the flood discharge in the wet season during which large sediment transport occurs is planned to be mostly discharged downstream through the spillway gate. Therefore, the low value of 30% (According to the past records in U.S.A., its values range between 20% and 50%) is taken as the trap efficiency for this project. With this trap efficiency, the annual sedimentation in the reservoir will be about 26 x $10^6 m^3$ and the reservoir below the spillway crest, of which capacity is about 130 x $10^6 m^3$, will be filled up with the sediment in several years.

4.4 Geology

4.4.1 Regional Geology

<u>General</u>: The Trisulganga River and the Kali Gandaki River as shown in the location map head in the Tibetan Marginal Mountains, then pass through an area of granitic rocks and ancient metamorphic rocks gathering many tributaries and finally meet at 1 km upstream of the Sapt Gandaki Damsite, after which it is called the Sapt Gandaki River. The damsite area is underlain by the Tertiary sedimentary rocks of the Siwalik Group which bounds on the above mentioned metamorphic rocks along the east-west trending Main Boundary Thrust running about 5 km north of the damsite.

<u>Stratigraphy</u>: As shown in Fig.-4.4.1 "GEOLOGICAL MAP OF RESERVOIR AREA", the oldest stratigraphic unit within the reservoir area consists of a variety of Paleozoic metamorphic rocks which are exposed in the northern part of the Main Boundary Thrust. On the other hand in the southern side of the Main Boundary Thrust, the Tertiary sedimentary rocks of the Siwalik Group widely spread up to the flat alluvial plain of the Chitwan Valley.

The Siwalik Group of this area consists of (1) banded sandstones and slate, (2) banded sandstones and mudstones, (3) massive and pebbly sandstones and (4) conglomerates from lower to upper horizons, which are named as Bsl Formation, Bst Formation, Sst Formation and Cgl Formation respectively in this report. Bsl Formation occupy the strip bounded by the Main Boundary Thrust and the inner Siwalik fault. The upper part of Bsl Formation, and the lower part of Bst Formation are petrographically similar.

The Sapt Gandaki damsite is located on the gradual transition zone between Bst Formation and Sst Formation. The other unit of geological facies is alluvial terrace deposits which develop on various elevations and are classified into four terraces as shown in Fig.-4.4.1. It is remarkable that the terrace IV is thickly developed in front of the Main Boundary Thrust and the inner Siwalik fault and includes thick cemented conglomerates and breccias. The layers of cemented conglomerates and/or breccias are observed not only in the terrace IV deposits, but also in other terrace deposits except the terrace III which widely extends on the left bank of the damsite.

Folding and Faulting: The most striking structural feature is the Main Boundary Thrust which is located in the northern part of the reservoir area and bounds the Paleozoic Midland Group and the Tertiary Siwalik Group. There are two other prominent faults, the inner Siwalik fault and the foothill fault, which trend east to west as the Main Boundary Fault does. An anticline extends in the northern side of a foothill fault plunging to the east. Strikes of bedding, an axis of folding and directions of faulting are consistently oriented in the east to west direction.

4.4.2 Geological Investigation

Field investigation on the geology was carried out in the period of February and March 1981 (Stage I) and October 1981 to April 1982 (Stage II), which contained the following survey items and quantities.

Survey Item	Quantity
i. Geological mapping:	
Reservoir area (1:10,000 in scale)	100 km^2
Damsite (1:2,000 in scale)	1.8 km^2
ii. Core drilling in the damsite	21 holes, 941.5 linear meter in total
iii. Permeability test in the boreholes	20 holes, 116 stages in total
iv. Seismic refraction prospecting	
Damsite	17 profiles, 9.75 km in total
Borrow area	4 profiles, 2.6 km in total
v. Test adit excavation	2 adits, 100 linear meters in total
vi. In-site rock tests	
Plate loading test	3 points
Block shear test	15 blocks
vii. Test grouting in one site	Drilled 7 holes 170 linear meters in total, grouted in 21 stages and Lugeon tested in 26 stages.
viii. Test trench for geological observation	4 trenches, 146 lienar meters in total
ix. Rock test in laboratory	34 pcs.

The location of the investigation is shown in Fig.-4.1.1, "LOCATION MAP OF INVESTIGATION". Details of the geological investigation and the obtained data are presented in ANNEX (E). Interpretation and assessment of the Project geology is explained in the following sections.

4.4.3 Damsite Geology

General: The damsite is located in the straight channel of the Sapt Gandaki River which traverses the Siwalik Hills and flows into the Terai Plain. At the damsite, the bedrock is exposed on both banks and the river is 200 m wide at the flow level. Three alternative dam-axes are proposed within 1 km of this stretch and named A-, B- and C-site from upstream to downstream.

The bedrock consists of the banded sandstone and mudstone formation (Bst Formation) and the massive pebbly sandstone formation (Sst Formation) of the tertiary Siwalik group as shown in Fig.-4.4.2 "GEOLOGICAL MAP AND SECTION OF DAMSITE". Bst Formation is composed of orderly repetitions of thick sedimentary cycles, each of which consists of graded sandstones, laminated siltstones, intraformational breccias and greenish mudstones, in order from lower to upper layers. Thickness of the component layers varies for each cycle. In this report, the zones of thick sandstone and thin other banded layers are denoted by the name of massive sandstone members (Ms members), and the zones of thin sandstone and rather thick banded layers by the name of alternation of sandstone, silstone and mudstone members (At members).

In the damsite, Bst Formation contains eight Ms members and the same number of At members which are alternated and numbered Ms-1 to Ms-8 and At-0 to At-7 from lower to upper zone respectively.

On the left bank are three stages of terrace at the elevations 260 m, 230 m and 200 m which are called the higher terrace, middle terrace and lower terrace respectively hereinafter. Among them, the middle terrace has substantial significance for the dam construction due to its wide and thick deposits lying within the range of dam foundation.

Alternative Damsite-A: A geological profile along the dam-axis of the alternative damsite-A is shown in Fig.-4.4.2 as "Geological Section A-A'".

On the left bank the middle terrace widely develops with a thick deposit which was penetrated by the core drilling B81-10 confirming the bedrock at 37.6 m in depth. Although the core drilling B81-1 was drilled also on the middle terrace deep to 30 m, the terrace deposits of silt, sand and gravel still continued. This wide and thick overburden renders the alternative damsite-A little attractive compared with other alternatives. The bedrock of the left bank underlying the middle terrace consists of Sst Formaiton.

The boreholes B80-1 and B80-2 in the river bed, each of which was drilled on the dam axis at about a quarter of the width of the riverbed from either bank, reached the bedrock at 16.2 m and 17.8 m of depths respectively. In the central part of the river channel, however, the bedrock appears to lower down to about 30 m of depth, according to the interpretation of seismic refraction prospecting in which the boundary between velocity layers of 2.2 km/sec and 2.8 km/sec is deemed correlative to the contact of the river deposit and the underlying bedrock.

In the area from the river channel to the right bank, the bedrock consists of At-4 to Ms-8 Members of Bst Formation out of which the massive sandstones of Ms-6 and Ms-7 Members seem some weak and the rock of At-4 Member rather sound.

<u>Alternative Damsite-B</u>: A geological profile along the dam-axis of the alternative damsite-B is shown in Fig.-4.4.2 as "Geological Section B-B'".

A geotechnically favourable feature of this site is that the middle terrace on the left bank appears to decrease in width compared with the damsite-A. On the other hand, the river gravel deposit may be thicker than in the damsite-A, as indicated by the bottom of the 2.2 km/sec velocity layer of seismic prospecting. While the bore hole B81-11 revealed the thickness of the riverbed deposit being 30.5 m at the center of the river on the dam axis, the interpretation of seismic refraction profile SL-10 (along the dam axis) shows the thickest point to be thirty plus a few meters. It seems that the thickness of the river gravel deposits tends to increase downstream as suggested by the bore hole B81-6 located about 140 m downstream which did not reach the bedrock even drilled deep to 36 m.

The bedrock of the damsite-B consists of the At members and Ms members of Bst Formation, except the left bank rim of massive pebbly sandstones (Sst Formation). At-4 and Ms-4 Members overlain by the riverbed deposits appear hard and sound in the core drilling B81-13 and B81-15. Some fractured zones of 0.5 to 2.0 m in thickness were observed in the core drilling B81-3, which was drilled on the right bank into the rocks of At-3 Member.

Alternative Dmaiste-C: A geological profile along the dam axis of the alternative damsite-C is shown in Fig.-4.4.2 as "Geological Section C-C'".

The lower and middle terraces on the left bank cover wider area in this site than in the alternative damsite-B; that is, 283 m as against 234 m in the site-B in the width along the dam axis. The contact of middle terrace deposit and underlying bedrock appears to be at EL.200 m to 210 m as inferred by the boundary between the 0.8 km/sec and 1.8 km/sec velocity layers in the interpretation of seismic refraction profile SL-11.

It seems that the depth of the riverbed deposit in the damsite-C is not shallower than the depth in the site-B, because the damsite-C is located 400 m downstream from the damsite-B.

The bedrock consists of the strata At-1 Member to At-5 Members as shown in Fig.-4.4.2. On the right bank, intercalations of thin slip fault clay are commonly observed along bedding planes.

Although the damsite-C is similar to the damsite-B in the features of bedrocks, terraces and riverbed deposits, it seems not to be better than the damsite-B in all of those features from the engineering geological viewpoint.

4.4.4 Engineering Geology

Physical properties and strength of bedrock: The results of laboratory rock tests are shown in Table-4.4.1, which is self-explanatory about the physical properties and strengths of rocks in solid test pieces without apparent cracks or fissures.

Strength of the bedrocks as a mass was measured by means of the in-situ plate loading test and the block shear test in the test adits TA-1 and TA-2 in the damsite-A, which were excavated at the dam axis on the left bank and 220 m upstream from the dam axis on the right bank respectively. The tests were made for (1) massive pebbly sandstone of Sst Formation, (2) laminated medium sandstone of Ms-7 Member, (3) massive medium sandstone of Ms-7 Member and (4) mudstone of At-7 Member.

From the above tests, moduli of elasticity (Es), moduli of deformation (D), cohesions (τ_0) and angles of internal friction (ϕ) were determined as tabulated below.

Rock	Modulus of elasticity Es (kg/cm^2)	Modulus of deformation D (kg/cm^2)	Cohesion (kg/cm^2)	Angle of int. frict. ϕ (degree)
(1) Pebbly sandstone	1.6×10^4	1.1×10^4	9.0	55
(2) Laminated sandstone	2.5×10^4	1.1×10^4	9.0	50
(3) Medium sandstone	1.1×10^4	7.4×10^3	8.0	40
(4) Mudstone	--- Not tested	---	6.5	37

The bedrock on the dam axis A consists of eight strata, Ms-5, At-5, Ms-6, At-6, Ms-7, At-7 and Ms-8 Member of Bst Formation, arranged from the right bank to the left bank in order, and the massive Sst Formation on the most left side as shown in Fig.-4.4.2.

It appears that the strength of Sst Formation can be represented by the strength determined on the pebbly sandstone tested. Hence,

Shear strength $\tau = 9.0 + \sigma \tan 55°$ (kg/cm^2) for Sst Formation.

Where, σ denotes the vertical stress in kg/cm^2.

The rocks in Ms-5, Ms-6, Ms-7 and Ms-8 Members can be assumed to be equivalent to the tested medium sandstone or laminated sandstone. Accordingly,

Shear strength $\tau = 9.0 + \sigma \tan 50°$ (kg/cm^2)

or $\tau = 8.0 + \sigma \tan 40°$ (kg/cm^2)

for the Ms members of Bst Formation.

The members At-5, At-6 and At-7 comprise various kinds of rocks, such as calcareous breccias, sandstones, siltstones, mudstones and thin intermediates. The At members are very often characterized by frequent beddings and laminations along which shear planes may easily develop, while they comprise fairly large portions of hard calcareous rocks of which compressive strengths are 1.3 to 5 times higher than the other rocks according to the laboratory tests. It is resulted from this condition that the resistance against shear in the At members depends largely on the shearing strength in bedding planes or weakest strata such as mudstones. As for shearing along bedding planes, the situation is deemed similar to that of the laminated sandstone, of which test gave 9.0 kg/cm^2 of cohesion and 50° of internal friction angle. Test in mudstone resulted in 6.5 kg/cm^2 of cohesion and 37° of internal friction angle, which could be some lower values than actual, considering the shearing plane in the test developed partly out of rock onto the bottom of the concrete test block which appeared to have been bound insufficiently to the underlying rock.

Accordingly, it is deemed that the shear strength of the At member varies in the range between:-

Shear strength $\tau = 9.0 + \sigma \tan 50° \ (kg/cm^2)$

and $\tau = 6.5 + \sigma \tan 37° \ (kg/cm^2)$

From the above results, it will be appropriate to take 8.0 kg/cm^2 for cohesion (τ_o) and 40° for angle of internal friction (ϕ) for the evaluation of safety against sliding of concrete gravity dam.

The result is applicable to the alternative damsites-B and -C as well, because those damsites are situated on the same formations.

Permeability: Determination of permeability of the bedrocks was done by the "Lugeon test" method with pressured water pumped into test sections confined by a rubber-packer. Permeability in the terrace deposits and riverbed deposits was observed by the "pumping-in" test and the "open-end pipe" test methods under static water head built up in the hole by pouring water to maintain a constant water level, of which details are explained in the ANNEX (E).

Permeability test in the terrace deposits was done in two bore-holes of B81-1 and B81-10 showing the permeabilities of 1.41×10^{-4} cm/sec and 4.25×10^{-3} cm/sec respectively. In consideration with the geological and testing conditions, it appears that the permeability of terrace deposits in ordinary condition which consists of gravels and sand with silty matrices could be lower than the above results, whereas the permeability of aquifer which contains sand and gravels may be higher such as the range of 5×10^{-3} to 5×10^{-4} cm/sec.

Lugeon tests for rock masses were done in 112 sections of 17 holes of which test results are shown in the Table-4.4.2. The bedrock of the damsite is generally massive and low in permeability as is indicated by the fact that 67% of sections tested shows less than 4 Lugeon unit of leakage. Scattered sections of more than 10 Lugeon unit cover only 17% of the tested zone.

Grout take: A set of test grouting with five grouting holes aligned in a line and two check holes was performed on the left bank of the alter-native damsite-B in which the bedrock consists of fine sandstones and mudstones of the At-5 member. It was considered that the At members could give higher permeability and grout takes than the massive Ms members.

The average grout take of the primary holes allocated in 4 m apart was 51.6 kg of cement per linear meter and the average of the secondary and tertiary holes drilled in split-spacing between two primary holes is 1.54 kg of cement per linear meter. The results of Lugeon test show the marked decline of permeabilities such as from 90.1 Lugeon in average of the primary holes up to 0.5 Lugeon in average of the check holes, through 2.58 Lugeon in average of the secondary holes and tertiary holes.

The test grouting shows that the cement grouting in the holes arranged at two meters is able to reduce the permeability of the bedrock effectively and obtain the sufficient water-tightness of less than 4 Lugeon. The average grout take of the whole grouting was 25.4 kg of cement weight per bore-hole linear meter.

4.5 Construction Materials

4.5.1 Material Requirement

The possible types of dam in this project are of concrete gravity type or rockfill type. In the case of the concrete gravity dam, total concrete volume including those for appurtenant structures would be about 1,300,000 m^3 while in the case of the rockfill dam, the total concrete volume would be about 810,000 m^3. The concrete requires coarse aggregates of 150 mm to 5 mm grain size and fine aggregates of grain size less than 5 mm. Taking into consideration the above concrete requirement, the quantity required for coarse aggregates and fine aggregates is estimated as follow;

Required Concrete Aggregates

Concrete Aggregates	Concrete Gravity Dam (ton)	Rockfill Dam (ton)
Coarse aggregates	2,400,000	1,500,000
Fine aggregates	1,000,000	650,000

Further, in the case of the rockfill dam, it requires the following dam materials.

Major Construction Materials	Required Volume
Rockfill material	1,500,000 m^3
Earth core material	346,000 m^3
Filter material	161,000 m^3

4.5.2 Construction Material Investigation

Field investigation of available quantity and quality for the said major construction materials was carried out by means of reconnaissance, test pits and laboratory tests. Details of the investigation and its results can be referred to ANNEX (B) in which all details are presented in a separate volume.

Fig.-4.4.1 indicates locations of the construction material investigation by test pitting together with other geological investigations.

Table-4.5.1 shows the laboratory test items carried out for quality investigation of each material.

4.5.3 Engineering Assessment

The engineering assessment on the available quantities and qualities of the construction materials is carried out based on the results of the field investigations and laboratory tests.

The engineering assessment carried out are separately detailed in ANNEX (B) for respective construction material of the concrete coarse and fine aggregates, core material, rockfill material and filter material, and the followings are conclusions obtained through the above engineering assessment.

(1) Concrete Coarse Aggregates

The concrete coarse aggregates will be available from the gravel deposits in up and downstream of the damsite. The available quantity is sufficient in each grain size required for the standard dam concrete. The quality is also satisfactory.

(2) Concrete Fine Aggregates

The sand in such large rivers as the Sapt Gandaki, Trisulganga and Kali Gandaki is too fine, siliceous and mixed with fine fragments of slates and other sedimentary rocks, to use for the concrete fine aggregates. The sand bars in the Khageri Khola are much better source than others in view of the acceptable hauling distance of 7 to 8 km, relatively preferable grain size distribution and a large volume of the sand deposit.

The sand in the Khageri Khola is not of very good quality for the concrete fine aggregates. Its quality, however, is within the acceptable range.

Thus, the sand bars in the Khageri Khola are recommended as the source of the concrete fine aggregates.

(3) Core Materials

The clayey earth materials covering the tableland in the left bank near the damsite are the best one in view of its easy accessibility and a short distance for material haulage.

The materials contain too much fine grain size to use it as the core materials and are required to be mixed with some coarser materials. There is no problem in the available quantity.

(4) Rockfill Materials

Gravel deposits scattered in the rivers and a quarry site in 7 to 10 km north of the damsite are possible sources of the rockfill materials of the dam.

However, the extraction of the gravels in the rivers will require screening facilities and several temporary bridges and access roads, resulting in a very costly work. Therefore, the rock quarry located 7 to 10 km north of the damsite is the only recommendable source for the rockfill materials of the dam.

Since the quarry rock is observed to be considerably cracky, the rockfill materials to be extracted from this quarry is anticipated to result in a rather small grain size gradation, and it is judged that the properties necessary for the dam rockfill materials can be secured in consideration of the degree of the cracks.

The available quantity is sufficient.

(5) Filter Materials

The filter materials of the dam will be produced by mixing river sand and gravel or quarry rock dust, since materials with a proper grain size distribution to satisfy the requirement for the filter are not available nearby.

The investigations carried out for the concrete coarse and fine aggregates and the quarry rock indicated that there is no problem with respect to available amount and quality to produce the filter materials by mixing them together.

4.6 Land Use around Project Area

The project site is situated in a relatively developed flat area just before entering into the Terai Plain. The downstream area of the project site is densely populated with a high ratio of land use. However, around the damsite both banks of the river are covered with forests. This is the national forest and there is no agricultural land and residential house near the damsite.

Productive agricultural lands and residential houses are scattered on the flat table lands in both banks along the Trisulganga and on the Terai Plain which spread out from about 4 km downstream of the damsite.

The flat tablelands along the Trisulganga are mostly cultivated as paddy and upland fields. Paddy fields occupy about one-tenth of the total agricultural land. They are located in the low elevation of the tableland and are used in the production of rice. The upland fields occupy the remaining nine-tenth and are alternately producing wheat, potato, corn, millet or mustard, etc. The residential houses are mainly made of wood and scattered on the tablelands. The major tablelands, however, are limited within about 10 km upstream from the confluence of the Kali Gandaki and the Trisulganga. Further upstream of the Trisulganga (up to Mugling) and the area along the Kali Gandaki become mountainous and form relatively deep gorge. Thus, no major land use is found there.

In the downstream area, which is the entrance to the Inner Terai Plain, there is a town called Narayangar with about 20,000 inhabitants and crowded with residential houses. A large agricultural land consisting mostly of paddy fields spreads out there. The irrigation project, the Chitwan Valley Development Project is now in progress in this downstream area.

4.7 Water Use around Project Area

Present municipal water is distributed by gravity flow to the towns or villages from a water tank installed at high elevation in the mountainous area and collects water from several minor streams. In addition, the municipality has sunk a well to supplement for shortage of water in the dry season.

Irrigation water is being collected from nearby small streams.

There is not any water use from the large rivers such as the Kali Gandaki, the Trisulganga or the Sapt Gandaki, etc. at present. However, a pumping station is now under construction at about 4 km downstream of the damsite in the Chitwan Valley Development Project and in the near future, irrigation water of about 20 m^3/sec will be pumped up from the Sapt Gandaki river.

4.8 Access to Site

The damsite is located in a relatively developed area near the Terai Plain. The town, Narayangar, near the project site is one of the key stations in the road network of Nepal. Access to the site is very convenient, compared with other mountainous areas.

With the recent completion of a highway between Mugling and Narayangar (Mugling Road), the car traffic from Kathmandu to Narayangar through Mugling became possible. The roadway is mostly flat and paved except for a section of several kilometers near Kathmandu which is now being paved.

Narayangar is also linked with the East-West Highway. Transportation of heavy construction equipment or materials can be made by using the Calcutta - (railway) - Birganj - (paved road) - Hetauda - (East-West Highway) - Narayangar - the project site route. This route is approximately 170 km long from Birganj.

From Birganj to Hetauda, there are nine bridges out of which three bridges are weak and will not sustain the load of 35 tons. From Hetauda to Narayangar a new road is almost completed. There is no restriction which will prevent the haulage of occasional heavy loads of 35 tons, since the bridges in the new road are designed for AASHO H-20 S-16 loadings. From Narayangar to the project site, the existing unpaved road of 5 to 7 m in width is provided along the left bank of the Sapt Gandaki up to Devighat in the upstream of the project site, making the haulage of 35 tons load possible.

The heaviest load for the Sapt Gandaki Project works is estimated at 35 tons in the haulage of the roter by a trailer. As mentioned, three bridges in the road between Birganj and Hetauda are impossible to haul the heaviest loading for the project work. However, a negotiation is being held between World Bank and Department of Roads, HMG to strengthen the weak bridges. If these bridges are renewed before the middle of 1984 there are no problems in the transportation work for the project. If not renewed in time, it is possible to ford the rivers in the dry season, providing a bypass.

Table-4.2.1: COORDINATE AND ELEVATION OF BENCH MARKS AND CONTROL POINTS

Station No.	Coordinate Northing (X) in M	Coordinate Easting (Y) in M	Elevation in M	Remarks
BM35	-	-	193,586	Existing bench mark
BM1	3,068,493.54	542,209.74	207,954	New bench mark
BM2	3,068,303.61	541,968.17	209,174	"
BM3	3,067,568.23	542,368.10	194,548	"
BM4	3,067,502.03	542,176.35	193,832	"
BM5	3,065,649.49	543,002.87	194,119	"
BM6	3,072,423.24	546,657.38	254,560	"
BM101	3,065,222.54	542,390.49	189,401	"
BM102	3,064,580.21	541,923.06	190,769	"
No.1	3,067,729.70	540,525.50	568,200	Indian triangulation point
No.2	3,075,919.50	542,438.10	1,184,200	"
No.36	3,063,600.80	529,932.70	-	"
T1	3,070,126.70	545,160.20	240.41	Main traverse station
T2	3,070,773.55	544,040.93	225.36	"
T3	3,069,559.47	543,601.83	201.63	"
T4	3,069,110.10	541,685.31	251.89	"
T5	3,066,527.49	541,539.91	234.45	"
T6	3,065,861.20	538,609.60	191.23	"
T7	3,066,631.85	537,284.34	212.58	"
T8	3,065,126.74	535,994.56	182.20	"
T9	3,064,181.47	531,658.97	182.54	"
T10	3,063,069.69	530,808.96	174.62	"
T11	3,063,719.28	530,251.29	187.00	"
T5-1	3,065,212.86	540,704.49	196.07	"
T5-2	3,067,728.86	540,531.94	572.53	"
GC1	3,069,718.94	542,447.82	223.85	Ground control point
GC2	3,069,296.88	541,511.26	254.51	"
GC3	3,065,687.72	543,223.25	212.12	"
GC4	3,066,336.66	541,736.93	208.73	"

Table—4.3.1: ANNUAL RAINFALL IN AND AROUND THE BASIN

No.	1	2	3	4	5	6	7	8	9	10	11	12	13
Name of sta.	Rukumkot	Shera Gaun	Libang Gaun	Bijuwar Tar	Musikot	Jomsom	Thakmarpha	Baglung	Tatopani	Lete	Mukutinath	Beni Bazar	Kushma
Sta. No.	0501	0502	0504	0505	0514	0601	0604	0605	0606	0607	0608	0609	0614
Elevation (El.m)	1.560	2,152	1.270	832	2.100	2.744	2.566	984	1,243	2,384	3.609	835	
Year 1957												1,332.7	
1958	1,933.3	1,124.0	1,132.5			259.6						1,401.5	
1959	1,839.8	1,103.6	1,317.3			252.0						1,331.8	
1960	1,634.9	1,358.5	974.2			327.0						1,316.4	
1961	3,395.3	1,539.1	2,319.7			317.3						1,634.6	
1962	3,268.9	1,533.6	1,294.7			244.1						1,322.9	
1963	4,748.3	1,604.0	1,295.4			346.7						1,532.7	
1964	3,171.0	1,381.4	892.7			284.7						1,567.8	
1965	1,645.3	1,171.3	755.1			284.8						1,164.7	
1966	1,956.4	1,302.4	725.2			157.3						1,440.8	
1967	2,384.6	981.5	942.3			141.3						1,044.2	
1968	3,504.6	1,275.6	1,058.9			347.0						1,406.2	
1969	-	1,386.1	1,064.8			324.2						1,314.0	
1970	4,729.8	1,444.5	-	1,292.7	-	189.3	499.4	2,354.8	1,321.7	821.8	-	1,448.2	1,920.5
1971	3,119.3	-	-	1,216.5	-	-	370.0	1,695.8	1,445.8	1,053.0	177.8	-	1,524.8
1972	6,922.0	-	-	892.7	-	-	398.4	1,033.2	1,199.5	1,202.0	432.8	-	1,790.5
1973	-	1,457.1	-	1,753.8	-	450.7	392.0	-	1,445.5	1,005.8	450.8	1,796.5	2,202.0
1974	-	1,293.6	1,515.3	897.9	1,339.4	301.4	264.8	1,901.6	-	1,062.4	412.7	1,306.5	2,420.0
1975	-	1,345.8	1,704.2	1,479.6	1,899.2	259.9	329.2	1,885.3	-	1,080.0	372.0	1,766.2	2,658.8
1976	-	1,334.4	-	836.8	1,668.0	214.7	253.2	1,689.0	1,605.5	1,015.7	341.5	1,432.8	-
1977	-	1,351.8	-	912.2	1,331.5	91.2	361.5	1,678.5	1,907.2	1,060.1	327.9	1,436.0	-
1978	1,346.5	1,725.6	1,925.9	1,531.1	1,489.0	64.0	-	1,463.2	1,143.1	1,084.1	936.2	1,252.8	-
1979	-	1,357.8	2,635.2	1,373.5	-	-	-	1,650.7	1,336.3	961.6	303.7	1,212.4	-
1980	-	1,580.0	1,852.7	1,415.3	2,538.2	-	475.3	2,148.2	1,626.2	1,167.9	472.6	1,430.2	-
Mean	3,040.0	1,364.4	1,376.8	1,236.6	1,710.9	256.7	371.8	1,750.0	1,447.9	1,046.8	422.8	1,404.5	2,086.1

- To be continued -

No.	14	15	16	17	18	19	20	21	22	23	24	25	26	27
Name of sta.	Ridi Bazar	Tansen	Butwal	Beluwa	Dumkali	RAMBAS	Kirtipur Churi	Musikot	Jagat Setibas	Khudi Bazar	Pokhara Hosp.	Pokhara Airp.	Syanja	Kuncha
Sta. No.	0701	0702	0703	0704	0706	0713	0714	0722	0801	0802	0803	0804	0805	0807
Elevation (El.m)	442	1,067	205	150	154		842	1,280	1,334	823	918	845	860	855
Year 1957	1,003.8	1,415.0	2,678.4					1,658.4			3,215.9			1,805.6
1958	1,112.3	1,487.0	2,333.8	1,856.3				1,989.0	1,475.5	3,325.1	3,920.7			2,113.6
1959	1,568.7	2,219.7	2,328.7	2,362.5				2,050.5	1,519.5	3,398.1	3,268.8			2,758.8
1960	1,458.5	1,661.6	2,391.7	1,532.0				2,010.8	1,401.5	2,787.0	3,131.1			2,757.8
1961	1,727.5	2,009.5	1,966.3	2,352.3				551.5	1,518.3	3,579.8	3,936.3			2,398.6
1962	1,512.9	1,916.8	2,791.8	3,168.1				1,391.7	1,373.8	3,256.9	4,004.4			3,143.3
1963	1,144.4	1,611.8	2,239.0	2,924.4				1,815.7	1,328.3	3,382.3	3,233.5			2,089.8
1964	1,143.2	1,211.0	2,733.7	2,612.9				577.7	1,013.2	3,064.1	3,515.4			2,077.4
1965	1,019.4	1,400.8	2,374.5	2,151.9				1,440.2	1,287.9	3,237.4	3,312.9			2,349.5
1966	1,021.2	1,004.2	2,963.1	2,686.2				1,428.8	835.4	3,201.8	3,245.9			1,941.9
1967	1,046.3	1,007.0	2,140.4	2,079.8				1,312.9	1,003.1	3,392.2	3,332.6			2,098.3
1968	1,302.8	1,479.2	2,176.0	2,453.8				1,818.1	1,002.0	2,936.1	3,609.1			2,081.9
1969	1,251.0	1,042.2	1,895.8	2,052.4				1,544.1	1,284.6	2,484.7	2,752.8			1,821.6
1970	1,516.6	1,951.8	3,693.1	2,703.2	-	2,817.9	-	1,742.3	1,354.8	3,510.9	3,877.3	3,731.6	-	2,057.8
1971	1,547.6	951.7	3,245.0	2,595.5	2,604.2	1,952.1	3,811.2	1,666.2	-	3,441.2	4,063.0	3,275.6	-	2,629.4
1972	1,021.2	-	2,104.0	1,753.7	-	-	2,929.1	1,398.7	1,559.5	3,146.7	3,270.7	3,208.5	-	2,871.5
1973	2,271.0	-	3,128.7	2,781.1	-	-	-	2,819.4	1,649.6	3,738.2	4,072.6	4,096.4	3,357.7	3,648.1
1974	1,036.6	-	2,807.5	3,212.6	-	-	-	1,861.7	1,288.8	3,560.7	4,641.6	4,605.0	3,458.3	3,294.9
1975	1,895.6	-	3,240.8	2,903.3	2,264.3	-	-	1,952.5	1,909.3	3,453.9	4,388.5	4,666.0	3,828.4	2,922.0
1976	980.8	-	-	2,178.8	1,927.0	-	-	2,106.9	1,453.6	3,145.7	-	4,249.9	3,102.3	2,674.3
1977	1,241.2	993.0	-	1,815.6	1,715.8	-	-	2,060.8	1,629.2	3,381.4	-	3,810.0	-	2,433.0
1978	1,870.1	1,523.1	-	2,420.1	2,555.3	-	-	2,657.5	1,605.6	2,829.8	-	3,960.0	3,103.4	2,973.1
1979	1,167.5	1,167.6	-	1,869.5	1,929.5	-	-	1,786.1	1,331.3	3,343.2	-	4,010.7	2,936.9	3,095.6
1980	1,583.6	1,809.3	-	2,271.8	1,893.3	-	-	1,966.6	1,239.9	3,059.7	-	3,843.0	2,880.5	2,303.2
Mean	1,351.8	1,466.4	2,591.2	2,379.9	2,127.1	2,385.0	3,370.2	1,733.7	1,366.6	3,245.9	3,620.7	3,950.6	3,238.2	2,514.2

- To be continued -

No.	28	29	30	31	32	33	34	35	36	37	38	39	40
Name of sta.	Bandipur	Gorkha	Chapkot	Malepatan	Lumle	Khairani	Chame	Rampur	Jhawani	Chisapani Gadhi	Daman	Hetauda(N.F.I.)	Amlekhganj
Sta. No.	0808	0809	0810	0811	0814	0815	0816	0902	0903	0904	0905	0906	0907
Elevation (El.m)	965	1,097	400	856	1,642	500	2,680	256	270	1,706	2,314	474	359
Year 1956													2,399.1
1957	1,312.4	1,731.0											1,485.2
1958	1,390.2	1,585.7	1,439.0						1,283.0	1,449.8			2,324.3
1959	1,511.0	1,720.0	1,828.3						1,478.3	1,878.3			2,161.0
1960	1,924.2	-	2,494.0						1,830.6	1,873.4			1,910.5
1961	2,341.8	-	3,267.2						1,978.8	2,545.9			2,060.4
1962	2,514.4	-	3,271.0						2,458.7	2,340.9			2,555.8
1963	1,838.7	-	1,167.6						1,300.0	1,815.1			2,099.9
1964	1,866.8	1,606.5	1,271.0						1,854.0	1,382.3			2,057.4
1965	1,852.9	1,972.9	1,474.1						1,838.9	2,640.0			2,412.1
1966	1,765.0	1,494.2	1,237.3						1,916.9	2,457.0			2,543.4
1967	1,601.0	1,537.5	1,187.1	3,364.2	-	-	-	-	1,247.7	2,095.9	-	2,575.0	2,298.0
1968	1,927.2	1,594.7	1,651.2	-	-	-	-	1,953.6	1,616.4	2,187.8	1,251.8	2,275.1	-
1969	1,219.6	1,362.1	1,098.7	3,770.2	-	-	-	1,376.3	1,656.3	1,560.9	1,427.5	-	1,892.3
1970	2,385.3	1,467.3	1,462.2	3,960.1	5,610.6	-	-	1,943.6	1,512.3	2,514.3	1,259.2	-	1,581.6
1971	1,942.0	1,969.6	1,405.6	3,106.8	5,963.5	-	-	2,202.5	2,771.5	2,267.6	1,716.7	-	3,275.0
1972	2,139.0	1,940.5	1,632.8	-	5,368.4	2,516.5	-	1,648.3	1,445.6	2,510.3	2,084.0	1,601.8	1,584.7
1973	1,963.7	-	2,401.2	4,258.4	5,757.0	1,965.0	-	2,341.0	1,865.5	2,719.2	2,426.6	2,709.3	2,445.1
1974	2,170.4	1,939.1	1,832.6	-	-	2,202.9	-	2,532.4	2,297.9	2,889.8	2,308.1	2,397.4	2,440.4
1975	1,794.8	1,794.3	2,644.8	-	5,355.6	-	751.3	2,314.0	2,023.4	2,676.2	2,159.9	2,525.9	3,017.5
1976	1,373.1	2,312.0	-	-	4,713.1	-	665.4	1,824.8	1,560.3	2,173.1	1,929.4	2,359.9	1,972.5
1977	1,599.5	1,333.7	1,578.7	3,832.4	4,863.1	2,433.6	862.2	1,725.9	1,571.1	1,815.1	1,528.5	1,947.6	1,583.0
1978	1,908.8	2,288.4	1,894.7	3,499.1	4,951.9	2,622.5	667.7	2,287.8	2,058.5	2,544.4	2,311.2	2,431.8	1,985.2
1979	1,766.4	1,826.8	1,480.6	3,619.9	3,946.1	2,405.9	-	1,825.3	2,323.1	1,806.3	1,854.8	1,788.5	2,117.7
1980	1,794.4	1,522.8	1,981.3	3,501.3	4,959.7	1,965.9	1,461.2	1,584.0	1,740.1	1,560.5	1,111.6	-	1,757.2
Mean	1,829.3	1,736.8	1,804.6	3,656.9	5,148.9	2,301.8	881.6	1,966.1	1,810.0	2,161.0	1,797.6	2,261.2	2,165.0

- To be contineud -

No.	41	42	43	44	45	46	47	48	49	50	51	52	53
Name of sta.	Parwanipur	Bhartpur	Chitlang	Hetauda	Birganj	Makawampur	Beluwa	Timule	Aru Ghat	Nuwakot	Dhading	Kakani	Nawalpur
Sta. No.	0911	0914	0915	0917	0918	0919	0920	1001	1002	1004	1005	1007	1008
Elevation (El.m)	115	223	1,530	466	91	1,030	274	1,900	518	1,003	1,420	2,064	1,592
Year 1957										1,759.8			
1958								1,017.6	2,331.4	-	1,303.4		
1959								961.4	2,286.4	1,839.0	2,018.4		
1960								968.1	1,328.2	1,650.0	2,846.8		2,220.1
1961								1,091.6	3,019.5	-	4,013.6		2,307.1
1962								1,089.0	3,042.2	2,116.1	4,182.9		2,720.6
1963								1,001.5	2,767.0	-	3,605.3		2,256.4
1964								1,007.5	1,916.1	1,727.0	1,615.0		2,639.2
1965								915.3	2,615.7	-	1,999.4		2,115.6
1966								956.6	2,008.4	-	2,043.0		1,946.6
1967	865.2							991.6	1,653.5	-	1,534.6		2,389.7
1968	881.9							1,069.1	1,791.9	1,187.8	1,526.0		2,475.0
1969	1,060.3							1,045.9	2,056.2	1,431.0	1,690.4		2,011.7
1970	1,760.4							879.7	2,759.3	1,952.4	2,021.1		2,242.9
1971	1,352.3	2,993.5						-	2,921.0	1,873.4	2,194.0		2,942.5
1972	893.4	-	1,473.7					758.8	3,133.5	1,702.6	1,930.4		1,950.9
1973	1,851.3	-	1,865.0					885.7	3,942.5	2,015.2	2,507.9		2,585.6
1974	1,939.5	-	1,829.0	2,591.0				660.4	3,252.9	2,065.0	2,013.2		2,424.4
1975	1,716.8	-	1,865.1	2,673.3	988.0	3,178.6	2,687.2	890.7	2,626.6	2,076.2	2,044.7		2,682.2
1976	1,026.8	-	1,706.4	1,963.7	907.4	2,689.0	2,264.2	256.5	2,987.2	1,741.8	2,269.5	2,652.4	2,857.5
1977	1,146.6	-	1,370.3	2,097.5	934.0	2,220.4	2,133.6	986.4	3,048.8	2,033.5	2,098.2	2,393.2	2,392.0
1978	1,809.2	-	2,960.3	2,613.9	1,276.9	2,970.4	2,737.7	1,122.6	3,060.5	2,541.6	2,301.4	3,239.6	3,008.7
1979	1,549.8	-	1,079.0	2,004.9	1,388.2	1,624.9	2,189.3	760.9	2,366.8	1,603.2	1,555.6	1,734.1	2,114.3
1980	1,161.0	-	1,095.8	1,569.2	1,036.8	919.7	2,440.0	1,345.8	2,557.2	1,732.7	1,915.3	2,842.1	2,357.6
Mean	1,358.2	2,993.5	1,693.8	2,216.2	1,088.6	2,268.7	2,408.7	939.2	2,585.8	1,836.0	2,227.4	2,572.3	2,411.5

- To be continued -

	54	55	56	57	58	59	60	61	62	63	64	65	66	67
Name of sta.	Chautra	Sundarijar Powerhouse	Sundarijar Reservoir	Kathmandu	Thankot	Sarmathang	Baunepati	Ranipauwa	Godavari	Kathmandu Airport	Thamachit	Dhunche	Tarke Ghyang	Changu Narayan
Sta. No.	1009	1012	1013	1014	1015	1016	1018	1019	1022	1030	1054	1055	1058	1059
Elevation (El.m)	1,660	1,364	1,576	1,324	1,630	2,625	845	1,828	1,400	1,336	1,847	1,982	2,480	1,543
Year 1941			2,525.1											
1942			2,808.9											
1943			3,010.0	1,009.6										
1944			2,425.1	1,185.7										
1945			2,909.3	1,585.6										
1946			3,259.9	1,395.6										
1947			-	-										
1948	2,154.1	2,755.0	3,125.6	1,793.4										
1949	2,151.5	2,445.9	2,478.2	1,368.9										
1950	2,131.9	2,406.6	1,439.4	1,536.1										
1951	1,847.9	2,037.6	1,692.3	1,224.4										
1952	-	1,967.5	2,302.4	1,280.4										
1953	-	2,007.1	2,224.9	1,363.6					1,764.4					
1954	-	2,525.4	2,553.4	1,593.7					2,083.6					
1955	1,918.8	-	-	1,130.6					2,063.9					
1956	3,061.9	2,455.8	2,309.0	1,776.4					2,393.9					
1957	1,540.7	1,641.3	1,512.4	1,000.7					1,397.9					
1958	1,664.9	-	-	1,134.3					1,455.7					
1959	2,023.1	2,062.7	1,994.5	1,195.3					1,633.0					
1960	980.0	1,393.0	1,691.7	1,201.5					1,766.2					
1961	2,048.4	1,674.5	1,947.4	1,704.7					-					
1962	2,330.4	2,482.8	2,366.7	1,261.5					2,337.2					
1963	1,853.7	2,111.8	2,263.1	1,313.5					-					
1964	2,143.1	1,762.2	1,904.1	1,384.8					-					
1965	1,648.6	1,450.2	1,649.3	1,333.2					-					
1966	-	1,655.8	1,728.8	1,223.8					-					
1967	-	1,992.0	2,061.7	1,348.6					-					
1968	2,049.5	-	2,445.1	1,539.2				2,748.0	-					
1969	-	-	1,929.0	1,131.2	1,189.0			2,624.0	-					
1970	-	-	2,267.8	1,439.9	1,589.6				-	1,362.3				
1971	-	-	2,382.5	1,601.0	1,811.0		2,066.6	-	1,992.8	1,511.3				
1972	2,994.4	-	1,779.2	1,509.5	1,851.6		1,604.9	-	1,927.2	1,261.4	1,196.0	1,681.5		
1973	2,994.4	-	-	1,969.2	2,632.6	2,353.3	1,920.2	-	2,430.4	1,799.8	1,497.6	1,820.0		
1974	1,491.1	-	2,007.1	1,135.5	2,206.6	3,470.3	1,360.9	-	-	1,225.1	1,498.0	1,712.6	2,434.7	
1975	1,519.9	-	2,540.8	1,526.7	2,091.8	3,856.6	1,691.2	-	-	1,424.5	1,668.3	2,090.6	772.1	1,644.5
1976	1,810.6	-	2,333.0	-	2,643.2	4,799.2	1,642.7	-	-	1,490.6	-	1,910.2	1,118.6	1,795.2
1977	1,958.0	-	-	-	2,287.6	3,535.5	1,480.8	-	-	1,297.3	2,524.0	1,661.5	584.8	-
1978	3,150.4	-	-	-	2,911.1	4,976.0	2,253.7	-	-	1,556.0	1,919.0	2,215.2	2,667.3	2,043.6
1979	1,723.2	-	-	-	2,576.8	4,012.4	1,483.2	-	-	1,356.4	913.0	2,432.7	849.6	1,144.4
1980	2,242.1	-	-	-	2,216.3	4,422.0	1,582.0	-	-	1,340.9	1,924.0	2,636.5	-	1,259.7
Mean	2,018.3	2,108.9	2,245.9	1,381.2	2,167.3	3,928.2	1,708.6	2,686.3	1,937.2	1,398.8	1,642.5	2,107.9	1,404.5	1,577.5

Table-4.3.2: MONTHLY MEAN RUNOFF AT GAGING STATION 450 (NARAYANGAHR)

(Unit: m^3/sec)

Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Year
1963	-	-	-	-	-	1,656	3,781	5,165	-	-	-	-	-
1964	452	377	310	412	444	1,180	4,061	4,451	3,510	1,764	987	638	1,561
1965	472	323	268	470	642	1,524	3,133	4,606	2,816	1,312	953	592	1,435
1966	387	313	245	285	615	1,328	3,490	5,325	3,379	1,322	852	583	1,521
1967	431	328	309	422	578	1,402	3,795	4,059	2,985	1,385	806	541	1,429
1968	407	308	320	366	637	1,920	4,471	4,176	3,147	2,822	1,043	670	1,699
1969	495	380	368	399	591	1,307	3,117	4,083	3,721	1,513	851	542	1,455
1970	378	278	236	372	588	1,828	4,382	4,412	2,175	1,213	606	371	1,414
1971	269	214	204	319	489	-	3,551	4,046	2,309	1,546	-	-	-
1972	-	-	233	239	542	1,165	-	3,408	2,388	1,086	656	-	-
1973	303	244	259	360	607	1,816	2,512	3,919	3,372	2,843	939	553	1,486
1974	416	327	288	379	502	1,020	3,388	-	-	-	781	540	-
1975	454	398	338	421	619	2,518	-	3,662	4,072	1,629	871	575	-
1976	-	-	374	396	347	1,891	2,465	3,298	2,625	-	-	-	-
1977	228	261	317	505	664	1,312	3,754	-	-	-	675	446	-
1978	374	304	313	431	976	2,110	4,742	4,690	2,527	1,426	702	435	1,598
1979	310	266	217	289	441	902	3,011	4,168	2,085	1,060	577	405	1,153
1980	278	225	223	286	446	1,452	-	-	3,340	1,172	629	390	-
Mean	376	303	283	373	572	1,548	3,576	4,231	2,967	1,578	795	520	1,436

Table-4.4.1: <u>SUMMARY OF ROCK TEST</u>

		1-1 to 1-5			2 - 2			3-1 and 3-2			4 - 1			5 - 1		
SAMPLE NO.		1-1 to 1-5			2 - 2			3-1 and 3-2			4 - 1			5 - 1		
LOCATION NO. & DEPTH		B81-14 (6.2 to 7, 10 to 11, 21 to 21.4 m) B81-5 (10.4 to 10.7, 16 to 16.6, 25 to 25.6 m)			TG-4 (9.0 to 10.0 m)			B81-5 (21.5 to 22.0 m) TG-4 (7.6 to 8.0 m)			B81-5 (31.4 to 31.9 m)			B81-14 (13.0 to 13.5 m)		
ROCK NAME IN LITHOLOGY		Medium sandstone			Sandy mudstone			Siltstone			Breccia			Fine sandstone		
OBSERVATION								Calcareous			Calcareous			Calcareous		
CONDITION OF SPECIMEN IN ROCK TEST	MOISTURE	NAT /1	SAT /2		NAT	SAT		NAT			NAT			NAT		
	ANISOTROPY															
APPARENT SPECIFIC GRAVITY G		2.407			2.561			2.658			2.690			2.666		
DENSITY γ (g/cm³)																
NATURAL WATER CONTENT Wn (%)																
WATER ABSORPTION Wsat (%)		5.24			4.00			1.69			0.70			0.85		
APPARENT POROSITY n' (%)		12.07			9.92			4.36			1.88			2.25		
P & S WAVES VELOCITIES	P WAVE Vp (km/sec)	1.63	2.07		3.03	2.98		3.27			5.02			4.38		
	S WAVE Vs (km/sec)	0.91	0.97		1.65	1.58		1.81			2.99			2.50		
	DYNAMIC MODULUS OF ELASTICITY ED (kg/cm²)	5.11x10⁴	6.34x10⁴		1.90x10⁵	1.72x10⁵		2.23x10⁵			6.02x10⁵			4.39x10⁵		
	DYNAMIC POISON'S RATIO μD	0.271	0.359		0.287	0.303		0.279			0.226			0.262		
UNCONFINED COMPRESSION	UNCONFINED COMPRESSIVE STRENGTH σc (kg/cm²)	128.1	100.0		211.0	75.2		169.1		-	688.8			686.1		
	STATIC MODULUS OF ELASTICITY Es (kg/cm²)	2.03x10⁴	1.13x10⁴		2.06x10⁴	4.5x10³		3.17x10⁴			3.53x10⁵			3.14x10⁵		
	STATIC POISON'S RATIO μs	0.329	0.308		0.335	0.355										
TENSION	BRAZILIAN TENSILE STRENGTH σt (kg/cm²)	6.02			7.94											
TRIAXIAL COMPRESSION	CONFINING PRESSURE σ_3 (kg/cm²)	20 10														
	STRESS DIFFERENCE $\sigma_1 - \sigma_3$ (kg/cm²)	340.3 285.7														
MOHR ENVELOPE	COHESION τ_0 (kg/cm²)	20			26											
	ANGLE OF INTERNAL FRICTION φ (°)	54			53											
WEATHERING TEST	DRY & SAT. PASSING (times)	5			2											
	PERCENT LOSS (%)	0			100											

REMARKS /1 NAT : Tested under the natural moistured condition.
/2 SAT : Tested under the saturated condition.

Table-4.4.2: LUGEON TEST RESULTS

(Unit in Lugeon)

Depth (m)	Bore-holes on Left Bank								in River Channel				on Right Bank					Stage Average
	B80-3	B81-9	B81-12	B81-2	B81-5	B81-14	B81-7	B81-18	B80-1	B80-2	B60-15	B80-13	B81-4	B81-8	B81-3	B81-17	B81-16	
0 – 5						0.6												0.6
5 – 10						0.1							159.3	1.0	0.71			40.3
10 – 15		7.4			0.4	2.8		3.2					10.4 14.9	1.1	2.33	2.4		4.03
15 – 20	1.75	4.7	3.4	9.0	1.04	3.6		3.68			17.7		2.8	3.3	2.45	1.2	4.91	4.58
20 – 25	1.59	5.9	2.2	13.08	0.64	22.2	13.48	2.54		3.13		119.5	16.1	7.2	1.28	0.8	10.2	14.6
25 – 30	1.49	5.8	9.9	0.69	1.69	6.5	0.63		6.64	5.6			2.3	7.6	1.83	1.14	6.88	4.19
30 – 35	82.9	8.2	11.0	0.53	10.78	0.2	6.8	3.6	3.15	3.49	0.34	0.05	2.3	20.4	2.3	1.08	12.5	9.98
35 – 40		8.1	12.2	0.31	0.89	0.6	1.78	0.98	6.57	4.97	0.39	0	10.4	2.22	2.07	0.62	2.46	3.41
40 – 45			2.1	1.1	1.14	0.02	0.9				0.32	0.05			4.66	1.88	6.44	1.86
45 – 50			1.1	1.18	1.18	0	7.28	1.88			0.55	0.15			1.62	2.04	16.75	3.07
50 – 55			1.3															1.3
55 – 60			1.3															1.3
Average	21.9	6.68	4.94	3.70	2.22	3.66	5.14	2.65	5.45	4.30	3.86	23.9	27.3	6.12	2.14	1.39	8.59	7.35

Table-4.5.1: LABORATORY TEST ITEMS AND TESTED SAMPLES FOR CONSTRUCTION MATERIALS

Construction Material	Laboratory Test Items	Tested Samples (In Test Pit No.)
Concrete coarse aggregates	- Sieving analysis	CTP-1 to CTP-12
	- Specific gravity and absorption	"
	- Washing test	"
	- Abrasion test	CTP-1, CTP-2, CTP-5, CTP-6, CTP-9 to CTP-11
	- Soundness test	CTP-1, CTP-2, CTP-5, CTP-9 to CTP-11
	- Weight of unit volume	CTP-1, CTP-2, CTP-5 CTP-6, CTP-9 to CTP-11
Concrete fine aggregates	- Sieving analysis	FTP-1 to FTP-5
	- Specific gravity and absorption	"
	- Washing test	"
	- Soundness test	FTP-1, FTP-3
	- Weight of unit volume	FTP-1 to FTP-5
Earth core material	- Sieving analysis	TP-1 to TP-7
	- Field moisture	"
	- Specific gravity	"
	- Liquid limit test	"
	- Plastic limit test	"
	- Compaction test	TP-1, TP-3, TP-4, TP-7
	- Triaxial test	TP-3, TP-4
	- Permeability test	"
Rock material	- Specific gravity and absorption	R-1 to R-4
	- Abrasion test	"
	- Soundness test	"
Filter material	Covered with the laboratory tests for concrete coarse and fine aggregates	

Table-4.5.2: SIEVING ANALYSIS RESULT FOR COARSE AGGREGATE

Sieve (mm)	Passing (Remaining) Rate of Each Sieve in % (Excluding size larger than 150 mm dia.) Test Pit No.												Remark
	CTP-1	CTP-2	CTP-3	CTP-4	CTP-5	CTP-6	CTP-7	CTP-8	CTP-9	CTP-10	CTP-11	CTP-12	
80	92.5(7.5)	91.5(8.5)	89(11)	95(5)	86(14)	95(6)	80(20)	100(0)	75(25)	90(10)	83(17)	95(5)	
40	58(42)	58(42)	65(35)	60(40)	55(45)	75(25)	60(40)	81(19)	36(64)	69(31)	52(48)	76.5(23.5)	
20	36(65)	42(58)	41.5(58.5)	38(62)	33(67)	47.5(52.5)	43(57)	54(46)	25.5(74.5)	50(50)	37(63)	52(48)	
10	24(76)	33(67)	31(69)	28(72)	23(77)	31.5(68.5)	35(65)	36(64)	20.5(79.5)	34(66)	30(70)	35(65)	
5	19(81)	27(73)	23(77)	21(79)	18(82)	23(77)	30.5(69.5)	30(70)	18(82)	28(72)	25(75)	29(71)	

Sieve (mm)	Remaining Rate of Each Sieve in % (Excluding size larger than 150 mm and smaller than 5 mm) Test Pit No.												Remark
	CTP-1	CTP-2	CTP-3	CTP-4	CTP-5	CTP-6	CTP-7	CTP-8	CTP-9	CTP-10	CTP-11	CTP-12	
80	9.26	11.64	14.29	6.33	17.07	6.49	28.78	0	30.49	13.89	22.67	7.04	
40	51.85	57.53	45.45	50.63	54.88	32.47	57.55	27.14	78.05	43.06	64.00	33.10	
20	80.25	79.45	75.97	78.48	81.71	68.18	82.01	94.29	90.85	69.44	84.00	67.61	
10	93.83	91.78	89.61	91.14	93.90	88.96	93.53	65.71	96.50	91.67	93.33	91.55	
5	100	100	100	100	100	100	100	100	100	100	100	100	

Table-4.5.3: QUALITY TEST RESULT FOR COARSE AGGREGATE

Test Pit No.	Specific Gravity and Absorption		Washing Test	Abrasion Test	Soundness Test	Weight of Unit Volume
	Specific Gravity	Absorption (%)	Rate Passing 0.088 mm Sieve (%)	Weight Reduction (%)	Weight Reduction (%)	(t/m^3)
CTP - 1	2.72	0.77	0.39	27.4	0.43	1.920
- 2	2.70	0.77	0.55	24.4	1.90	1.892
- 3	2.69	0.69	0.46	-	-	-
- 4	2.70	0.70	0.30	-	-	-
- 5	2.69	0.71	0.06	16.43	2.40	1.890
- 6	2.76	0.33	0.06	16.43	-	1.850
- 7	2.75	0.51	0.06	-	-	-
- 8	2.71	0.30	0.30	-	-	-
- 9	2.73	0.60	0.20	11.10	0.88	1,861
- 10	2.70	0.74	0.46	18.77	1.24	1.846
- 11	2.71	0.73	0.35	16.53	2.35	1.870
- 12	2.69	0.65	0.43	-	-	-
- 13	-	-	-	-	-	-
- 14	-	-	-	-	-	-

Note: Grain size in Test Pit No.13 and No.14 was too fine, and judged not applicable for coarse aggregate. Thus, no laboratory test was carried out for the above.

Table-4.5.4 SIEVING ANALYSIS RESULT FOR FINE AGGREGATE

Sieve (mm)	Passing (Remaining) Rate of Each Sieve in %						Remarks
	Test Pit No.					Average	
	FTP-1	FTP-2	FTP-3	FTP-4	FTP-5		
5	85(15)	38(62)	87(13)	63(37)	44.5(55.5)	63.5(36.5)	
2.5	80(20)	31.5(68.5)	84(16)	57(43)	38(62)	58.1(41.9)	
1.2	76(24)	26.5(73.5)	82.5(17.5)	53.5(46.5)	33(67)	54.3(45.7)	
0.6	53.5(46.5)	13(87)	77.5(22.5)	38.5(61.5)	20(80)	40.5(59.5)	
0.3	15.5(84.5)	3.5(96.5)	14(86)	10(90)	4(96)	9.4(90.6)	
0.15	2(98)	1.0(99)	2(98)	2(98)	2(98)	1.8(98.2)	

Sieve (mm)	Remaining Rate in Each Sieve in % (Excl. size larger than 5 mm)						Remarks
	Test Pit No.					Average	
	FTP-1	FTP-2	FTP-3	FTP-4	FTP-5		
5	0	0	0	0	0	0	
2.5	5.88	17.11	3.45	9.52	14.61	10.11	
1.2	10.59	30.26	5.17	15.08	25.84	17.39	
0.6	37.06	65.79	10.92	38.89	55.06	41.54	
0.3	81.76	90.79	83.91	84.13	91.01	86.32	
0.15	97.65	97.37	97.70	96.83	95.51	97.01	
F.M.	2.33	3.01	2.01	2.44	2.82	2.52	

Table-4.5.5: QUALITY TEST RESULT FOR FINE AGGREGATE

Test Pit No.	Specific Gravity and Absorption		Washing Test	Abrasion Test	Soundness Test	Weight of Unit Volume
	Specific Gravity	Absorption (%)	Rate of Passing 0.088 mm Sieve (%)	Weight Reduction (%)	Weight Reduction (%)	(t/m^3)
FTP - 1	2.67	1.93	10.09	-	9.58	1.283
- 2	2.65	1.94	1.93	-	-	1.891
- 3	2.68	2.63	2.20	-	12.03	1.295
- 4	2.63	1.84	1.07	-	-	1.615
- 5	2.54	2.28	6.14	-	-	1.800

Table-4.5.6: QUALITY TEST RESULT FOR CORE MATERIAL (1)

Samples					Consistency			Gradation							
								Passed Percentage (%)					Particle Size (mm)		
Test Pit No.	Depth (m)	Soil Classifi-cation	Field Moisture Content Wf, (%)	Specific Gravity, Gs(g/cm^3)	L.L. (%)	P.L. (%)	P.I. (%)	mm 4.76	mm 2.00	mm 0.42	mm 0.074	mm 0.005	D60	D30	D10
TP-1	2	CL	17.8	2.656	35.2	25.3	9.9	100	99.1	97.5	86.1	51.0	0.01	-	-
TP-2	2	CH	26.9	2.659	52.9	36.4	16.5	100	100	99.1	94.2	53.0	0.008	-	-
	4	CL	12.6	2.710	41.5	20.1	21.4	100	98.9	97.5	82.0	28.0	0.035	0.0063	-
TP-3	2	CL	20.2	2.688	44.8	18.7	26.1	100	100	96.9	85.0	48.0	0.013	-	-
	4	CL	10.1	2.699	33.5	23.6	9.9	96.9	95.6	87.6	67.8	22.0	0.065	0.012	-
TP-4	2	CL	20.5	2.677	40.5	24.3	16.2	100	99.8	96.8	85.0	44.0	0.017	0.0035	-
TP-5	2	CL	23.6	2.663	37.9	20.3	17.6	100	100	97.9	88.8	46.0	0.024	0.001	-
TP-6	2	CL	23.4	2.663	46.7	27.2	19.5	100	100	96.9	85.8	50.0	0.007	0.0015	-
TP-7	2	CL	24.0	2.659	41.6	28.7	12.9	100	100	98.4	91.2	65.0	0.0042	-	-

Table-4.5.7: QUALITY TEST RESULT FOR CORE MATERIAL (2)

Samples	Condition of Test	Dry Density, rd (g/cm3)	Mositure Content, W (%)	Wet Density, rt (g/cm3)	Void Ratio, e	Degree of Saturation, Sr (%)	Triaxial Compression Test U-U Cohesion, C (kg/cm2)	Triaxial Compression Test U-U Friction Angle, ϕ (°)	Triaxial Compression Test C-U Cohesion, C (kg/cm2)	Triaxial Compression Test C-U Friction Angle, ϕ (°)	Permeability Test Permeability Coefficient, k (cm/s)
TP-3 (at 2 m depth)	Max. dry density and optimum moisutre content	1.560	22.6	1.913	0.723	84.0	1.9 (-)	15°00' (-)	- (-)	- (-)	2.78×10^{-7}
	95% dry density and 80% of degree of saturation	1.482	24.2	1.841	0.814	80.0	1.2 (-)	12°00' (-)	0.3 (0.08)	15°30' (35°30')	3.63×10^{-6}
TP-4 (at 2 m depth)	Max. dry density and optimum moisture content	1.485	26.2	1.868	0.803	87.3	1.4 (-)	15°00' (-)	- (-)	- (-)	4.87×10^{-7}
	95% dry density and 80% of degree of saturation	1.411	26.8	1.789	0,897	80.0	1.3 (-)	13°30' (-)	0.42 (0.15)	11°30' (33°30')	2.29×10^{-6}

Samples	Compaction Test Compaction Energy, Ec (%)	Max. Dry Density, rd Max. (g/cm3)	Optimum Moisture Content Wopt (%)	Wf - Wopt (%)	Allowable Maximum Grain Size (mm)	Diameter of Mold (mm)	Weight of Rammer (kg)
TP-1 (at 2 m depth)	100 (5.625 m.kg/m3)	1.555	24.0	-6.2	4.76	100	2.5
TP-3 (")	"	1.560	22.6	-2.4	"	"	"
TP-4 (")	"	1.485	26.2	-5.7	"	"	"
TP-7 (")	"	1.510	25.8	-1.8	"	"	"

Notes: (i) WF = Moisture content at field.
(ii) ( ) shows the effective stress analysis.

Table-4.5.8: QUALITY TEST RESULT FOR QUARRY ROCK

Sample No.	Specific Gravity and Absorption		Abrasion	Soundness
	Specific Gravity	Absorption (%)	Weight Reduction (%)	Weight Reduction (%)
R-1	2.934	0.24	29.48	0
R-2	2.880	0.52	23.88	0
R-3	2.907	0.41	16.97	0
R-4	2.845	0.97	32.50	0

CHAPTER 5 PLAN FORMULATION

5.1 Optimization Study

5.1.1 General

It was found through the field investigation that the sand and gravel terrace deposit in the left bank of the damsite proposed in the Prefeasibility Report (Damsite-A) is much thicker and wider than that in the alternative damsites in the downstream. The condition of the foundation rock at the damsite A was also found to be same as or less favourable than that at the alternative damsites. It was judged that the damsite A would not necessarily be the most advantageous site and that the detailed technical and economic comparative study is required for selection of the damsite.

Two dam types, concrete gravity type and fill type, are considered as possible dam types in the project site. To handle the big magnitude of flood in the Sapt Gandaki during the construction period, the concrete gravity dam is favored as the fill type dam will require a huge diversion scheme. However, the investigation disclosed that the strength of the foundation rock is marginally low for constructing a concrete gravity dam with a height of more than 80 m presently planned.

Under such condition, the concrete gravity dam requires a large concrete mat at its base for its safety against sliding. Further, in consideration of the thick river gravel deposit, 30 m to 35 m in depth, in the river channel, the construction of the concrete gravity dam, requiring large excavation of the river gravel deposit and its replacement with a concrete base mat, is anticipated to be very costly. Such being the case, a detailed comparative study is considered necessary for the selection of the dam type.

Determinatiin of the optimum development scale and the optimum installed capacity to yield the most effective and advantageous net benefit to the project also requires further examination. Then, the optimization study was carried out to determine the optimum damsite, dam type, development scale and installed capacity. The detailed descriptions of the result of the study are provided hereinunder.

5.1.2 Selected Cases for Comparative Study

Damsite: There are three conceivable damsites in the Sapt Gandaki Project (Damsites A, B and C shown in Fig.-4.1.1) within about 1 km river length from 1 km downstream of the confluence of the Kali Gandaki and the Trisulganga. No suitable damsites are found in further upstream or downstream due to the topographic condition. In order to choose the most advantageous damsite for the Sapt Gandaki Project, geological investigations by borings, seismic explorations, test adittings and in-situ rock test, etc. were carried out at Stage-II for these three damsites. Characteristics of each damsite found through the geological investigations are as outlined below:

(1) Damsite-A

Outstanding characteristics of this site is the extensive terrace deposit which covers some 400 m wide area between the hill on the left bank and the river channel. In the center of the deposit, their thickness is deemed to be 40 m or more. All the excavation of this terrace deposit down to the bed rock will be required in the case of the concrete gravity dam construction in this area. In the case of the fill type dam construction, impervious core trench or cut-off wall should be sunk to the bed rock through the terrace deposit.

On the abutment slopes on the both banks, other than the above area, solid rock will be reached by approximately 10 m deep excavation.

The fresh rock under the river bed shows for the most part the relatively low seismic wave velocity of 2.8 km/sec. The bed rock surface under the river deposit appears to form two stages of rather flat planes. The higher planes exist in the both sides of the river channel which are 16 m to 18 m deep from the surface of the river gravels. The lower plane in the middle part of the river channel is deemed to be around 28 m deep from the result of seismic exploration. The depth of this part had first appeared to be around 20 m in depth, but the profile was modified in view of the finding in the downstream site that seismic exploration gives shallower depth than the actual condition for this lower plane in the narrow trench under the river bed.

The bed rock is alternating massive mica-bearing coarse sandstones, massive medium sandstones, laminated fine to medium sandstones, pebbly conglomerate layers, mudstones and laminated siltstones, soft to moderately hard and some calcareous. Its strength is measured at 8 kg/cm^2 in cohesion and 40° in internal friction angle which is deemed marginally low for constructing a concrete gravity dam with a height of more than 80 m.

The grout test carried out at Stage-II reveals that permeability of the base rock can be decreased as required for dam construction by the usual cement grouting.

(2) Damsite-B

The terrace deposit on the left bank at this site is found to be 150 m or less in width, and 35 m in thickness in the thickest portion. This means much reduction of the size of terrace deposit in comparison with that in the damsite-A. In this aspect, this site seems more favourable than the damsite-A.

The river gravel deposit is approximately 30 m in thickness in the middle part of the river channel. About 130 m downstream from the dam axis it increases to a thickness of over 35 m.

On the left abutment of the site, the depth of the foundation excavation for the concrete gravity dam or the impervious core trench of the fill type dam will be approximately 6 m, except for the zone of the terrace deposit. On the right abutment, it will be from 10 m to 20 m.

The properties of the foundation base rock at the site are generally the same as those at the damsite-A.

(3) Damsite-C

The condition of the terrace deposit on the left bank at this site is nearly similar to that at the damsite-B. The river gravel deposit is deemed as slightly deeper than that at the damsite-B. However, it appears not much different so far as the result of seismic exploration is concerned. Weathering is generally deeper than the other two damsites.

Comparing the conditions at the damsite-A and damsite-B, if the construction work can be done in accordance with the usual dam construction procedure in which diversion tunnels are provided in a bank or both banks, the damsite-B will be the more advantageous damsite because the terrace deposit in the left bank is much reduced at this damsite. As a result, the required dam becomes smaller in scale, although the river deposit is slightly deeper. However, in the case of the Sapt Gandaki Project, a tunnel large enough for the river diversion purpose is difficult to construct in the banks due to insufficient rock covering depth above the tunnel and the open channels will have to be provided for the purpose of river diversion. In such a case, the conditions at the damsite-B, which lead to the cheaper project cost in the usual case, will not necessarily be advantageous due to the increase of the excavation volume of the diversion open channels.

Accordingly, a detailed comparative study of the damsite-A and the damsite-B was considered necessary. As for the damsite-C, its conditions of both banks are nearly the same as those in the damsite-B, while the river deposit is slightly deeper. Since no merit can be expected in comparison with the damsite-B, the damsite-C was dropped.

Dam Type: Outstanding characteristics of concrete gravity dam or fill type dam to be constructed at the Sapt Gandaki Project site are as follows;

(1) Concrete Gravity Dam

Since the cohesive strength of the foundation rock is only 8 kg/cm^2, construction of the concrete gravity dam with the usual section is impossible. The concrete gravity dam must be provided with a thick concrete mat at its base to secure its safety against sliding. Large amount of excavation of the thick river deposit will also be inevitable to place the concrete mat on the foundation rock, which is considered costly and burdensome.

However, to handle big floods in the Sapt Gandaki during the construction period, the concrete gravity dam is preferred to the fill type dam. Since the concrete dam under construction is allowed to be overtopped by big floods and the required diversion scheme will be of small scale and less expensive.

(2) Fill Type Dam

The fill type dam does not allow overtopping by flood flow during construction. Therefore, a very large capacity of river diversion for handling the recorded maximum flood of 16,350 m^3/s is needed. This leads to the provision of two large diversion open channels (one in each bank), with a large quantity of excavation. In the fill type dam, however, the treatment required for the river deposit and the terrace deposit will be of relatively small scale, i.e. the core trench, the cut-off wall or the grouting for water stoppage.

Each dam type has both merit and demerit as mentioned above, and the determination of the dam type required an economic comparative study, each provided with sufficient technical safety in the design.

Development Scale and Installed Capacity: The optimum project scheme is defined as one to produce the maximum net benefit. The larger development scale and the installed capacity of power plant will produce the larger benefit. It, however, requires a larger amount of investment. Thus, the net benefit which is defined as the produced benefit less the necessary investment will not always be larger in the larger development scale and the installed capacity of the power plant. To find the optimum development scale and the installed capacity of the power plant, various development scales and installed capacities were assumed and the optimum one was tried to be selected through the assessment of the net benefits in all the cases.

The assumed development scales indicated by the reservoir full supply level (FSL) and installed capacities for the comparative study are as follows in which the topographic limit of EL.240 is taken into consideration.

Assumed Development Scales (in FSL) and Installed Capacity (in MW) for Comparative Study

Assumed Development Scale in FSL		1 EL.210	2 EL.220	3 EL.230	4 EL.240
Assumed Installed Capacity in MW for Each Development Scale	1	75.0 MW	112.5 MW	150.0 MW	187.5 MW
	2	112.5 MW	150.0 MW	187.5 MW	225.0 MW
	3	150.0 MW	187.5 MW	225.0 MW	262.5 MW
	4	187.5 MW	225.0 MW	262.5 MW	300.0 MW

Note: The installed capacity in the above table is expressed by the capacity of the first 3 units, excluding an additional unit planned to be installed in a future when the upstream storage type project is realized.

Summary of Selected Cases for Comparative Study: The cases selected for the comparative study based on the considerations mentioned above, are summarized below;

Summary of Selected Cases for the Comparative Study

(1) Dam site : Damsite-A and damsite-B

(2) Dam type : Concrete gravity dam and fill type dam

(3) Development scale : 4 cases of FSL 210, 220, 230 and 240

(4) Installed capacity :

- For FSL.210 : 4 cases of 75 MW, 112.5 MW, 150 MW and 187.5 MW
- " 220 : 4 cases of 112.5 MW, 150 MW, 187.5 MW and 225 MW
- " 230 : 4 cases of 150 MW, 187.5 MW, 225 MW and 262.5 MW
- " 240 : 4 cases of 187.5 MW, 225 MW, 262.5 MW and 300 MW

5.1.3 Preliminary Design for Comparative Study

The optimization study of the project required the cost estimate of various cases assumed for the comparative study. For the above cost estimate, preliminary design was carried out for the assumed various cases. The major considerations in the preliminary design for the cost estimate in the optimization study were as follows. The details of the project design are given in Chapter 6 "PROJECT DESIGN".

<u>River Diversion:</u> Coffering works by driving sheet piles are judged to be difficult due to the existence of much gravels in the river deposit. Therefore, the diversion method consisting of the construction of a diversion open channel in the bank and of upstream and downstream cofferdam is adopted.

For the concrete gravity dam scheme, the flood magnitude of 2,000 m^3/s which corresponds to about a 25 years recurrence flood in the dry season (November to May) was adopted as the diversion design flood in consideration of a large seasonal variation of the flood magnitude. This is intended to allow overtopping the dam under construction for floods exceeding 2,000 m^3/s during the rainy season. Based on the above diversion design flood, one diversion open channel with a 55 m bottom width is provided on the left bank. Provided with the upstream cofferdam with its crest level at EL.190.5, the above diversion channel has the capacity to discharge the diversion design flood of 2,000 m^3/s.

For the fill type dam scheme, as overtopping over the dam under construction is not allowed, the diversion system should have the capacity to discharge big floods during the rainy season. Then, two diversion open channels (one on each of the left and right banks) were planned so that the recorded maximum flood of 16,350 m^3/s, which corresponds to about a 50 years recurrence flood magnitude, can be handled, provided with the upstream cofferdam with the crest level at EL.196.0.

Concrete Base Mat of Concrete Gravity Dam: The concrete gravity dam was intended to be designed to satisfy the design standard in Japan, i.e. the safety factor for sliding more than 4.0 and no occurrence of tensile stress in any portion, based on the foundation strength investigated at cohesive strength of 8.0 kg/cm^2 and friction angle of 40°. The provision of a wide concrete base mat was needed due to the above design condition, and the determination of the dimensions of the concrete base mat was made as follows;

(1) In the concrete gravity dam, the spillway structure will be provided on the dam body. With the spillway on the dam body in the river channel portion, the concrete base mat in the immediate downstream portion of the spillway will work as the stilling basin. Therefore, the upper surface level and the minimum required length of the base mat are hydraulically determined so as to work as the stilling basin against a 100 years recurrence spillway outflow of 17,800 m^3/s.

(2) In the case that the necessary safety of the dam cannot be assured by the above minimum required dimension of the base mat, the base length was extended toward upstream by providing the upstream fillet of dam.

(3) In the event that the necessary safety of the dam is not obtained by the above extension of the base mat to upstream due to the increase of dam height, the base mat was extended to downstream.

(4) The required thickness of the base mat was examined by analyzing the stress caused in the foundation rock, using the computer program of the Finite Element Method. The thickness which favourably distributed the stress in the foundation and controlled the stress within the allowable stress was determined as the minimum required thickness of the base mat.

The minimum required thickness of the base mat determined for each dam height is as follows;

Dam Crest Level	Minimum Required Thickness of Base Mat (in M)
EL.215.0	10
EL.225.0	11
EL.235.0	12
EL.245.0	13

Intake: Sedimentation of the Sapt Gandaki is of a huge amount of 2,800 m^3/km^2/year or 88 x $10^6 m^3$/year at the damsite. Assuming the trap efficiency of 30%, the reservoir will be filled up with the sediment up to the spillway overflow crest level in several years. The power intake is provided with the intake wall to protect the inflow of sediment adjacent to the spillway so that the sediment near the power intake can be discharged out together with the spillway overflow discharge. Further, the crest level of the intake wall is raised up by 10 meters above the spillway overflow crest level and the overflow portion of the intake wall is provided in parallel to the spillway discharge flow direction so as to facilitate sand flushing in front of the intake wall. The non-overflow section of the intake wall was raised up to the flood water level since the effective sand flushing in front of this portion is not expected.

Desitling Basin: River water in the Sapt Gandaki contains much sand that would harm and damage the turbine blades by abrasion. Thus, the provision of a desilting basin was studied to counteract these effect. The study which came to preclude the idea of the desilting basin is described in detail as follows;

Fig.-5.1.8 shows an examined design of the desilting basin by which particles larger than 0.5 mm diameter can be settled. The incremental cost to build this desilting basin is estimated at about U.S.$70 x 10^6, resulting in a high project cost. The particle size of the sediments (suspended load) is very fine as seen in the result of grain size analysis shown in Table-5.1.7. If it is intended to effectively settle these fine particles, the required area of the desilting basin is more than twice the design shown in Fig.-5.1.8. It is understood that it is technically difficult and not practical to settle such very fine particles of the sediments which would not be so harmful to the turbine blade. Then, it is recommendable to adopt the following cheap and practical countermeasure;

(i) To provide an intake wall in front of the power intake as mentioned in the previous paragraph and take only surface water in which most of the large size particles are already settled.

(ii) To use the 13 Chromium Hi-Nickel Steel as the material of the turbine blades. This material is durable against abrasion and the life of the turbine blades is expected to extend up to around 10 years.

(iii) To repair the damaged turbine blades due to abrasion by the build-up-welding.

In the event that the damage of turbine blades by abrasion will occur, it will be possible to repair the damaged blades by the build up welding at low cost. It is recommendable to keep one spare unit of turbine blade unit for making repair without any serious interruption of power generation. In the event of the occurrence of blade damage, it can be replaced with the spare unit and the damaged one will be kept as the spare unit after repairing.

The cost necessary for the above repairing and the spare unit of turbine blade, etc. will be less than 1.0% in annual expenditure compared with the provision of the huge desilting basin.

For reference, the estimated costs for the repairing and the spare unit of turbine blade are shown below;

- Repairing (Once ten years) :

Man-month:	\$10,000 x 6 months x 3 persons	= \$180,000
Welding rod:	\$6,500/ton x 1 ton	= \$ 6,500
Tool, etc.		= \$ 4,500
		\$191,000

- Spare unit of turbine blade : \$450,000

Spillway: In accordance with the design standard in Japan, the spillway for the concrete gravity dam is designed to have the capacity to pass the 200 years recurrence peak flood discharge of 19,200 m^3/s at the flood water level. For the fill type dam, it is designed to have the capacity to pass 1.2 times of the 200 years recurrence peak flood discharge at the flood water level. Any effect of regulation by the reservoir is not taken into account for the determination of the spillway capacity in compliance with the design standard.

The above spillway was also confirmed to have the capacity to handle the 10,000 years recurrence flood with peak discharge of 27,400 m^3/s (considered generally equivalent to the probable maximum flood) with a freeboard of 3 m.

5.1.4 Power Generation

5.1.4.1 Reservoir Operation

General: The project has to be designed as a run-off river type due to much sedimentation. The optimization study of the project requires to estimate the benefit to be produced in each assumed scheme in order to compare the advantageousness in terms of the net benefit. The benefit from the project is the power benefit. Thus, the dependable power and energy to be produced in each assumed scheme were calculated by carrying out the simulation of the reservoir operation on the daily basis for the period of 18 years. The reservoir operations were made by the use of comptuer as follows.

Inflow Stream Data: The stage height observation at the Gauging Station No.450 which is located at the immediate downstream of the damsite, was commenced from 1963 by DIHM, and the observation has been continued on daily basis. Thus, the daily average stream flow data converted from the observed stage height by using the rating curve are available from 1963 to 1980 (for 18 years). The daily stream flow data are given in ANNEX (C). This daily stream flow was assumed as the inflow into the reservoir.

Reservoir Capacity Curve and Tailwater Rating Curve: In principle, water to be used for the power generation in the run-of-river type is the inflow discharge, keeping the reservoir water level at the full supply level. However, in a small inflow discharge, the effective storage down to the low water level is utilized for the daily power operation. Thus, the reservoir water level will fluctuate daily between the full supply level and the minimum operation level. The tailwater level will also fluctuate in accordance with the change of the outflow discharge.

To obtain the water head for the power generation in accordance with the fluctuation of the water levels in the reservoir and the tailrace, the reservoir capacity curve after sedimentation (relationship between the reservoir storage volume and the reservoir water level) and the tailwater rating curve (relationship between the outflow discharge and the tailwater level) are required. The effective storage capacity after sedimentation was decided to be 8.5×10^6 m^3, assuming that the sedimentation would occur with same slope as the river bed (0.0015) from the spillway crest elevation. The tailwater rating curve was obtained as a result of the discharge measurement at the Stage-II field investigation. It is shown in Fig.-5.1.1.

The computer carried out all the readings of these curves given as input data and the calculation of water head for the power generation.

Operation Water Levels of Reservoir: The reservoir operation was carried out based on the full supply level and the minimum operation level.

The optimum project development scale was sought through a comparative study from several assumed development scales which are expressed by the reservoir full supply level. As stated in Section 5.1.2, four full supply levels of EL.210, 220, 230 and 240 m are selected for the comparative study.

The minimum operation level (M.O.L.) was determined at 4 m below the full supply level (F.S.L.), which means the effective storage volume of 8,500,000 m^3, based on the following considerations;

For the daily operation, the reservoir water level should be recovered up to the full supply level while the operation is stopped (during 12 hours). The inflow volume during 12 hours at 90% firm river discharge, 290 m^3/s, is about 12.5×10^6 m^3, and then, the reservoir should not be emptied beyond this volume of 12.5×10^6 m^3 which corresponds to 6 m drawdown below F.S.L.

It is found through an examination that the larger drawdown will yield the more economic advantage owing to an increase of the dependable firm power. Therefore, so far as the economic advantage is concerned, it is desirable to have the maximum drawdown of 6 m. In this case, the top elevation of the intake wall to prevent the inflow of sediment has to be set at 10 m below F.S.L. because minimum 4 m is further required as the approach flow depth above the top of the wall to limit the flow velocity to less than 1 m/sec. However, in view of importance of effective sediment prevention, the drawdown was fixed as 4 m (2 m less) at a sacrifice of some economic merit. This means to raise the top of the intake wall by 2 m. Anyway, the setting of M.O.L. should be examined in detail through a model test at the detailed design stage.

<u>Rated Head</u>: The determination of the smaller rated head for the generating equipment leads to the increase of the power output to be generated, and as a result, the increase of the power benefit. However, it will also result in the cost increase of the generating equipment due to the necessity of a large discharge capacity of the turbine.

The most benefitial rated head should be found out by a trial calculation as one which will produce the maximum net benefit. However, the optimum rated water level is known to be usually at around the center of the operation water levels. Considering that the determination of the rated water level at the center of the operation water levels will not give any effect on the result of the comparative study, the rated head was determined as follows;

(1) The rated water level in the reservoir was determined as the middle of the full supply level (F.S.L.) and the minimum operation level (M.O.L.), i.e. worked out as (F.S.L. + M.O.L.)/2.

(2) The rated tailwater level was found out by the trial calculation, utilizing the following relation which must be satisfied among the peak output power (P), the efficiency (η), the rated turbine discharge (Qr), the rated reservoir water level (H_1), the rated tailwater level (H_2) and the rated loss of head (H_ℓ);

$$P = 9.8 \eta Qr (H_1 - H_2 - H_\ell)$$

or

$$H_2 = H_1 - \frac{P}{9.8 \eta Qr} - H_\ell$$

In the above equation, H_ℓ and H_2 are functions of Qr. Since H_ℓ is very small compared with the gross head ($H_1 - H_2$) in the case of the Project, H_ℓ is assumed to be 2.5% of the gross head. H_2 is defined by the tailwater rating curve as given in Fig.-5.1.1. Therefore, Qr is the only one unknown parameter to be obtained by solving the above equation.

The above equation is solved by trial calculation. Firstly, an estimate of H_2 is calculated by substituting the assumed initial value for Qr of the above equation. A trial value of Qr is read on the tailwater rating curve as corresponding to the estimated H_2. If both the values of Qr are nearly equal to each other, then the latter is adopted as Qr. If not, the above procedure is repeated with a new assumed value of Qr being the latest trial one.

(3) The rated head of the generating equipment was determined as the rated reservoir water level (H_1) less the sum of the rated tailwater level (H_2) and the rated loss of head (H_ℓ).

Water Use and Power Operation: The reservoir has no capacity to regulate the large seasonal variation of the river run-off, i.e. the project is designed as the run-of-river type due to much sedimentation in the Sapt Gandaki as stated. Therefore, the inflow discharge without regulation is the available discharge for the power generation. The inflow discharge which causes a large seasonal fluctuation was handled as follows in consideration of its maximum use for the power generation. In the power generation, it is assumed that the Sapt Gandaki Project will share supply for the power demand of around 12 hours duration. Thus, the power plant was intended to be operated at 50% plant factor in principle.

The followings are the detail of the water use for power generation in the Sapt Gandaki Project (Definition Sketch is shown in Fig.-5.1.3).

CASE 1 : During the period of abundant inflow discharge to such an extent that the turbine cannot accommodate all of the inflow discharge, the full power operation is continued for 24 hours per day, keeping the reservoir water level at the full supply level. The extra inflow discharge exceeding the turbine discharge is spilled out through the spillway.

CASE 2 : During the period of the inflow discharge less than the turbine discharge, the full power operation is conducted as much as possible by utilizing the reservoir storage capacity for daily operation as illustrated in CASE 2 of Fig.-5.1.3.

CASE 3 : In the case of extremely small inflow discharge to such an extent that the full power operation can not be continued at 50% plant factor, the output power is lowered and the operation made at 50% plant factor as illustrated in CASE 3 of Fig.-5.1.3.

5.1.4.2 Dependable Power and Energy Output

Dependable Power Output: The dependable power output is defined as the power which can be guaranteed at 90% firmness. As stated, it is assumed that the Sapt Gandaki Project will take charge of the supply for the power demand of about 12 hours duration (50% plant factor). Therefore, the power to be supplied with 90% firmness at 50% plant factor was determined as the dependable power output, and applied for the capacity benefit. The dependable power output in each assumed case for the comparative study, which was calculated by the reservoir operation based on the stream flow data, is as seen in Table-5.1.4.

Energy Output: The energy produced in the reservoir operation is also shown in Table-5.1.4. The energy output is classified into two kinds, i.e. the primary energy and the secondary energy in accordance with the firmness of the energy to be supplied. The primary energy is defined as the valuable energy to be supplied with the firmness of 90% and the plant factor of 50%, i.e. the primary energy is counted as the energy to be generated at 50% plant factor by the dependable power. And this primary energy is assumed to be able to be absorbed in the power system of Nepal with the limitation of the power demand.

The surplus energy other than the above primary energy, which is generated beyond 50% plant factor in the high stream flow season, is separately classified into the secondary energy. The secondary energy is assumed to be a surplus energy in the power system of Nepal in consideration of the followings;

The secondary energy is generated in the high stream flow season during which a group of other hydropower plants in Nepal will also generate much energy and adequately satisfy the power demand in Nepal. Such being the case, it is assumed that there is very little room for effective accommodation of the secondary energy from the Sapt Gandaki Project and that the secondary energy can be supplied to India who requires any energy supply.

5.1.5 Project Benefit

The project benefit is the power benefit which is considered as the cost of the least cost alternative. The least cost alternative in Nepal is the coal-fired thermal plant which is the cheapest and most likely power supply source to be selected for addition to the Nepal power supply system in the absence of the project.

Thus, the power benefit consisting of the capacity benefit and the energy benefit was assessed from the cost necessary for the alternative coal-fired thermal plant. Further details of the concept of the benefits attributable to the project are explained in Chapter 9.

5.1.6 Project Cost

The project optimization study was made by means of comparison of the net benefit for various assumed cases of the project schemes. Then, it required the estimation of the project costs for the above various assumed project schemes to be compared.

To estimate the project costs, the preliminary designs were made for all the assumed cases of the project schemes in accordance with the criteria of the preliminary design which are technically scrutinized and detailed in Section 5.1.3, and the project costs were estimated based on the measured work quantities and the assumed unit prices.

The unit price of each work item was determined at 1982 July price level, based on the investigated price for labour, construction materials and construction equipment as given in Table-8.4 and Table-8.5, the construction planning carried out, and reference to the contract prices of the similar projects. Details of the above are provided in Chapter 8. List of the determined unit prices are as shown in Table-8.2.

The project cost includes the cost for the preparatory works, the land acquisition, the engineering and government administration, and the physical contingency, which are not estimated on the basis of the unit price, and also the electrical and mechanical works. Details of these estimate are also provided in Chapter 8.

The estimated project costs are given in Table-5.1.1 and 5.1.2 for all the assumed cases.

5.1.7 Result of Optimization Study

The results of the optimization study are given in Table-5.1.5 and Table-5.1.6, and graphically shown in Fig.-5.1.4 to Fig.-5.1.7. Following is an outline of the results of the optimization study.

Scrutinizing the graphs shown in Fig.-5.1.4 to Fig.-5.1.7, it is found that the larger project development scale (FSL 230 and 240) makes the selection of the damsite-B more advantageous in both cases of the concrete gravity dam and the fill type dam. The main reason for the above is that the concrete structures in the left bank of the damsite-A become prominently bigger due to its deep foundation rock line. Further, the project development scale with FSL higher than EL.220 makes the selection of the fill type dam scheme more advantageous by the reason that the necessary dam volume of the concrete gravity dam increases in the higher dam scheme.

Comparing the larger project development scale with the smaller one, the larger one is evidently much more advantageous in terms of the net benefit to be obtained, as seen in the graphs. Thus, the optimization study revealed that the selection of the larger project development scale, the damsite-B and the fill type dam scheme would be beneficial.

A comparison was made between FSL.230 and FSL.240 of the larger project development scale (in the case of the damsite-B and the fill type dam). The maximum net benefit in FSL.230 which appeared at the installed capacity of 225 MW resulted in a slightly larger net benefit, compared with the maximum net benefit in FSL.240 which appeared at the installed capacity of 262.5 MW. This is due to a rather constant increase of the benefit against a prominent increase of the investment cost in FSL.240. Then, it is considered that there is no merit in increasing the development scale beyond FSL.230.

Consequently, the selection of the following project scheme for the Sapt Gandaki Project is considered as the most beneficial.

Most Beneficial Scheme

Damsite	:	Damsite-B
Dam type	:	Fill type dam
Development	:	F.S.L. 230
Installed capacity	:	225 MW

5.2 Problem in the Optimum Project Scheme

5.2.1 General

The most beneficial project scheme is obtained through the optimization study as mentioned in Section 5.1.7 above. However, it is considered necessary to make further examination for the problem of the head loss to arise in the Kali Gandaki No.2 project which is located at about 20 km upstream from the Sapt Gandaki Project site, in relation with determining the above project scheme as the optimum one.

5.2.2 Head Loss in the Kali Gandaki No.2 Project

There is a project site at about 20 km upstream of the Kali Gandaki from the Sapt Gandaki Project named as the Kali Gandaki No.2 Project. Although the above Kali Gandaki No.2 Project will be realized in a far future, there is a possibility of its realization in accordance with the increase of the power demand. The tailwater level at this upstream project is investigated at EL.220. On the other hand, the full supply level of the most beneficial project scheme in the Sapt Gandaki Project is worked out at EL.230. If the full supply level of the Sapt Gandaki Project is set at EL.230, the head loss of about 10 m will occur in this upstream project, resulting in some benefit loss in the upstream project in future. Then, the project scheme of the Sapt Gandaki Project should be determined in due consideration of the relationship with the upstream project.

In order to evaluate this head loss in the upstream project, an economic evaluation was carried out assuming that the upstream project would be realized in 20 years after the completion of the Sapt Gandaki Project.

The economic evaluation was made by comparing the economic merit combined with the upstream project between the following two cases;

CASE (1) : The Sapt Gandaki Project will be built based on the project scheme of FSL.230 and 225 MW installed capacity by disregarding the relation with the upstream project.

CASE (2) : The Sapt Gandaki Project will be limited to the project scheme of FSL.220 and 225 MW installed capacity intending to avoid the future head loss at the upstream project where the tailwater level is EL.220.

In the CASE (1), compared with the CASE (2), the power benefit corresponding to the water head of 10 m will be obtained at the earlier stage immediately after the completion of the Sapt Gandaki Project. However, similar power benefit will be lost at the upstream project after its realization. Further, the necessary investment is also increased due to its higher project scheme compared with the CASE (2). In the calculation, the cost and benefit obtained for the most beneficial scheme in the optimization study are provided as those in the Sapt Gandaki Project. For the Kali Gandaki No.2 Project, it is assumed that the difference of the benefit between CASE (1) and CASE (2) will be lost from the annual benefit to be obtained at the Kali Gandaki No.2 Project.

In the CASE (2), the power benefit corresponding to the water head of 10 m will not be obtained in the Sapt Gandaki Project. However, the future benefit loss at the Kali Gandaki No.2 Project will be avoided. Besides, the investment for the Sapt Gandaki Project will become smaller than that in the CASE (1). In the calculation, the cost and benefit obtained for the scheme of FSL.220 and 225 MW installed capacity in the optimization study are provided as those in the Sapt Gandaki Project, and for the Kali Gandaki No.2 Project, it is assumed that there is not any benefit loss such as the CASE (1).

The comparison of the obtained economic merit in terms of the present worth at 1989/90 between the CASE (1) and the CASE (2) is as indicated in following page.

CASE (1):

Combined Benefit, $B = 641,711 + 2.13156\ B_k - 10,667$

" Cost, $C = 493,062 + 2.13156\ C_k$

" Net Benefit, $NB_1 = B - C$

$= 137,982 + 2.13156\ (B_k - C_k)$

where, B_k: Annual benefit in the Kali Gandaki No.2 Project

C_k: Annual cost in the Kali Gandaki No.2 Project

CASE (2):

Combined Benefit, $B = 538,812 + 2.13156\ B_k$

" Cost, $C = 471,537 + 2.13156\ C_k$

" Net Benefit, $NB_2 = B - C$

$= 67,275 + 2.13156\ (B_k - C_k)$

Difference of Economic Merit:

$NB_1 - NB_2 = +\ 70,707$ (x 10^3 US$)

As shown above, the CASE (1), which is of the most beneficial project scheme of the Sapt Gandaki Project and acquires the power benefit in the earlier stage, is found out to be much more advantageous. In the above calculation, it is assumed that the difference of the benefits between EL.230 and EL.220 scheme of the Sapt Gandaki Project will be lost at the Kali Gandaki No.2 in the future. Considering that the river discharge at the Kali Gandaki No.2 site is actually smaller than that at the Sapt Gandaki Project site, the future benefit loss at the Kali Gandaki No.2 will be less than the assumed one, resulting in more economic merit of the CASE (1) than the calculated one. Then, it was concluded that any adjustment of the Sapt Gandaki Project scheme due to the relation with the upstream project would not be necessary and that the Sapt Gandaki Project should be constructed as the most beneficial scheme of the project itself, as far as the relation with the upstream project is concerned.

Table-5.1.1: PROJECT COST OF ASSUMED PROJECT SCHEMES

(A) Damsite - A

A.1 Fill Type Dam Scheme

A.1.1 F.S.L.240.0 M

Items	Amount 300 MW	(*1000$) 262.5 MW	225 MW	187.5 MW
1. Preparatory Works	4,660	4,579	4,507	4,439
2. Civil Works				
C-1 Diversion	46,900	46,900	46,900	46,900
C-2 Dam & spillway	142,663	142,663	142,663	142,663
C-3 Intake	82,030	80,597	79,465	78,225
C-4 Powerhouse & tailrace	39,084	35,094	31,412	28,169
3. Mechanical Works	26,560	25,630	24,740	23,840
4. Electrical Works	74,369	63,899	53,988	46,108
5. Land Acquisition	9,500	9,500	9,500	9,500
Sub-total	425,768	408,862	393,176	379,845
6. Engineering & Govern. Adomo	31,933	30,665	29,488	28,488
7. Physical Contingency	61,499	59,473	57,536	55,867
Total	519,200	499,000	480,200	464,200

A.1.2 F.S.L.230.0 M

Items	Amount 262.5 MW	(*1000$) 225 MW	187.5 MW	150 MW
1. Preparatory Works	3,389	3,265	3,173	3,072
2. Civil Works				
C-1 Diversion	41,930	41,930	41,930	41,930
C-2 Dam & spillway	94,272	94,272	94,272	94,272
C-3 Intake	50,744	47,403	44,473	42,068
C-4 Powerhouse & tailrace	38,989	34,083	30,847	26,515
3. Mechanical Works	25,980	25,190	24,260	23,190
4. Electrical Works	69,741	58,784	51,367	41,391
5. Land Acquisition	9,000	9,000	9,000	9,000
Sub-total	334,045	313,927	299,322	281,437
6. Engineering & Govern. Adomi.	25,053	23,545	22,449	21,108
7. Physical Contingency	47,402	44,828	42,929	40,655
Total	406,500	382,300	364,700	343,200

A.1.3 F.S.L.220.0 M

Items	225 MW	187.5 MW	150 MW	112.5 MW
	Amount (*1000$)			
1. Preparatory Works	2,500	2,353	2,250	2,127
2. Civil Works				
C-1 Diversion	37,000	37,000	37,000	37,000
C-2 Dam & spillway	64,176	64,176	64,176	64,176
C-3 Intake	26,688	22,329	19,261	16,253
C-4 Powerhouse & tailrace	38,814	33,350	29,566	24,350
3. Mechanical Works	26,060	24,830	23,820	22,440
4. Electrical Works	65,616	55,195	45,848	36,751
5. Land Acquisition	5,500	5,500	5,500	5,500
Sub-total	266,353	244,732	227,421	208,597
6. Engineering & Govern. Adomi.	19,976	18,355	17,057	15,645
7. Physical Contingency	37,071	34,313	32,022	29,658
Total	323,400	297,400	276,500	253,900

A.1.4 F.S.L.210.0 M

Items	187.5 MW	150 MW	112.5 MW	75 MW
	Amount (*1000$)			
1. Preparatory Works	2,334	2,103	1,923	1,731
2. Civil Works				
C-1 Diversion	33,331	33,331	33,331	33,331
C-2 Dam & spillway	44,275	44,275	44,275	44,275
C-3 Intake	34,639	26,578	19,818	13,883
C-4 Powerhouse & tailrace	43,337	35,985	30,746	23,915
3. Mechanical Works	26,560	25,100	23,540	22,160
4. Electrical Works	65,269	52,986	41,637	31,207
5. Land Acquisition	2,000	2,000	2,000	2,000
Sub-total	251,745	222,357	197,269	172,501
6. Engineering & Govern. Adomi.	18,881	16,677	14,795	12,938
7. Physical Contingency	34,974	31,066	27,736	24,361
Total	305,600	270,100	239,800	209,800

A.2 Concrete Gravity Dam Scheme

A.2.1 F.S.L.240.0 M

Items	300 MW	Amount 262.5 MW	(*1000$) 225 MW	187.5 MW
1. Preparatory Works	4,667	4,559	4,475	4,395
2. Civil Works				
C-1 Diversion	21,914	21,914	21,914	21,914
C-2 Dam & spillway	206,687	206,687	206,687	206,687
C-3 Intake	56,504	52,624	49,123	46,314
C-4 Powerhouse & tailrace	26,003	22,721	20,625	18,116
3. Mechanical Works	26,560	25,630	24,740	23,840
4. Electrical Works	74,369	63,899	53,988	23,840
5. Land Acquisition	9,500	9,500	9,500	9,500
Sub-total	426,204	407,533	391,053	376,874
6. Engineering & Govern. Adomi.	31,965	30,565	29,329	28,266
7. Physical Contingency	61,631	59,202	57,218	55,360
Total	519,800	497,300	477,600	460,500

A.2.2 F.S.L.230.0 M

Items	262.5 MW	Amount 225 MW	(*1000$) 187.5 MW	150 MW
1. Preparatory Works	3,448	3,324	3,250	3,158
2. Civil Works				
C-1 Diversion	20,580	20,580	20,580	20,580
C-2 Dam & spillway	143,280	143,280	143,280	143,280
C-3 Intake	37,698	34,006	31,681	29,193
C-4 Powerhouse & tailrace	28,340	23,714	21,109	17,484
3. Mechanical Works	25,980	25,190	24,260	23,190
4. Electrical Works	69,741	58,784	51,367	41,391
5. Land Acquisition	9,000	9,000	9,000	9,000
Sub-total	338,068	317,878	304,527	287,277
6. Engineering & Govern. Adomi.	25,355	23,841	22,840	21,546
7. Physical Contingency	47,977	45,381	43,733	41,577
Total	411,400	387,100	371,100	350,400

A.2.3 F.S.L.220.0 M

Items	225 MW	Amount 187.5 MW	(*1000$) 150 MW	112.5 MW
1. Preparatory Works	2,678	2,530	2,436	2,319
2. Civil Works				
C-1 Diversion	19,511	19,511	19,511	19,511
C-2 Dam & spillway	98,012	98,012	98,012	98,012
C-3 Intake	39,159	25,470	22,440	19,063
C-4 Powerhouse & tailrace	30,835	25,688	22,456	17,991
3. Mechanical Works	26,060	24,830	23,820	22,440
4. Electrical Works	65,616	55,195	45,848	36,751
5. Land Acquisition	5,500	5,500	5,500	5,500
Sub-total	278,370	256,736	240,022	221,586
6. Engineering & Govern. Adomi.	20,878	19,255	18,002	16,619
7. Physical Contingency	38,952	36,209	34,076	31,695
Total	338,200	312,200	292,100	269,900

A.2.4 F.S.L.210.0 M

Items	187.5 MW	Amount 150 MW	(*1000$) 112.5 MW	75 MW
1. Preparatory Works	2,368	2,169	2,013	1,835
2. Civil Works				
C-1 Diversion	18,560	18,560	18,560	18,560
C-2 Dam & spillway	70,712	70,712	70,712	70,712
C-3 Intake	34,149	27,543	21,868	16,089
C-4 Powerhouse & tailrace	34,432	27,752	23,069	16,964
3. Mechanical Works	26,560	25,100	23,540	22,160
4. Electrical Works	65,269	52,986	41,367	31,207
5. Land Acquisition	2,000	2,000	2,000	2,000
Sub-total	254,050	226,822	203,399	179,527
6. Engineering & Govern. Adomi.	19,054	17,012	15,255	13,465
7. Physical Contingency	35,296	31,766	28,646	25,508
Total	308,400	275,600	247,300	218,500

(B) Damsite - B

B.1 Fill Type Dam Scheme

B.1.1 F.S.L.240.0 M

Items	300 MW	Amount 262.5 MW	(*1000$) 225 MW	187.5 MW
1. Preparatory Works	4,011	3,902	3,802	3,668
2. Civil Works				
C-1 Diversion	45,750	45,750	45,750	45,750
C-2 Dam & spillway	149,183	149,183	149,183	149,183
C-3 Intake	30,139	26,405	23,305	18,920
C-4 Powerhouse & tailrace	42,330	38,771	35,220	30,686
3. Mechanical Works	26,560	25,630	24,740	23,840
4. Electrical Works	74,369	63,899	53,988	46,108
5. Land Acquisition	9,500	9,500	9,500	9,500
Sub-total	381,843	363,039	345,488	327,655
6. Engineering & Govern. Adomi.	28,638	27,228	25,912	24,574
7. Physical Contingency	54,619	52,233	50,000	47,671
Total	465,100	442,500	421,400	399,900

B.1.2 F.S.L.230.0 M

Items	262.5 MW	Amount 225 MW	(*1000$) 187.5 MW	150 MW
1. Preparatory Works	3,109	2,935	2,883	2,744
2. Civil Works				
C-1 Diversion	41,160	41,160	41,160	41,160
C-2 Dam & spillway	96,722	96,722	96,722	96,722
C-3 Intake	27,470	21,596	19,882	15,498
C-4 Powerhouse & tailrace	41,941	36,205	34,456	29,585
3. Mechanical Works	25,980	25,190	24,260	23,190
4. Electrical Works	69,741	58,784	51,367	41,391
5. Land Acquisition	9,000	9,000	9,000	9,000
Sub-total	315,124	291,593	279,731	259,291
6. Engineering & Govern. Adomi.	23,634	21,869	20,980	19,447
7. Physical Contingency	44,442	41,238	39,789	37,162
Total	383,200	354,700	340,500	315,900

B.1.3 F.S.L.220.0 M

Items	225 MW	Amount 187.5 MW	(*1000$) 150 MW	112.5 MW
1. Preparatory Works	2,695	2,498	2,362	2,175
2. Civil Works				
C-1 Diversion	36,855	36,855	36,855	36,855
C-2 Dam & spillway	66,109	66,109	66,109	66,109
C-3 Intake	33,493	26,419	21,223	14,991
C-4 Powerhouse & tailrace	43,228	37,141	33,288	27,048
3. Mechanical Works	26,060	24,830	23,820	22,440
4. Electrical Works	65,616	55,195	45,848	36,751
5. Land Acquisition	5,500	5,500	5,500	5,500
Sub-total	279,556	254,546	235,005	211,869
6. Engineering & Govern. Adomi.	20,967	19,091	17,625	15,890
7. Physical Contingency	39,177	35,863	33,270	30,141
Total	339,700	309,500	285,900	257,900

B.1.4 F.S.L.210.0 M

Items	187.5 MW	Amount 150 MW	(*1000$) 112.5 MW	75 MW
1. Preparatory Works	2,566	2,266	2,037	1,790
2. Civil Works				
C-1 Diversion	33,108	33,108	33,108	33,108
C-2 Dam & spillway	45,490	45,490	45,490	45,490
C-3 Intake	45,508	33,534	23,985	15,015
C-4 Powerhouse & tailrace	46,938	38,926	33,214	25,716
3. Mechanical Works	26,560	25,100	23,540	22,160
4. Electrical Works	65,269	52,986	41,637	31,207
5. Land Acquisition	2,000	2,000	2,000	2,000
Sub-total	267,439	233,410	205,011	176,486
6. Engineering & Govern. Adomi.	20,058	17,506	15,376	13,236
7. Physical Contingency	37,403	32,784	28,913	25,078
Total	324,900	283,700	249,300	214,800

B.2 Concrete Gravity Dam Scheme

B.2.1 F.S.L.240.0 M

Items	Amount (*1000$) 300 MW	262.5 MW	225 MW	187.5 MW
1. Preparatory Works	4,423	4,335	4,225	4,146
2. Civil Works				
C-1 Diversion	18,026	18,026	18,026	18,026
C-2 Dam & spillway	195,772	195,772	195,772	195,772
C-3 Intake	53,197	49,969	46,882	44,187
C-4 Powerhouse & tailrace	27,901	25,262	20,967	18,417
3. Mechanical Works	25,890	24,960	24,070	23,170
4. Electrical Works	74,369	63,899	53,988	46,108
5. Land Acquisition	9,500	9,500	9,500	9,500
Sub-total	409,078	391,724	373,430	359,326
6. Engineering & Govern. Adomi.	30,681	29,379	28,007	26,949
7. Physical Contingency	58,941	56,797	54,463	52,625
Total	498,700	477,900	455,900	438,900

B.2.2 F.S.L.230.0 M

Itesm	Amount (*1000$) 262.5 MW	225 MW	187.5 MW	150 MW
1. Preparatory Works	3,261	3,149	3,075	2,983
2. Civil Works				
C-1 Diversion	17,037	17,037	17,037	17,037
C-2 Dam & spillway	135,274	135,274	135,274	135,274
C-3 Intake	37,348	33,511	31,225	28,762
C-4 Powerhouse & tailrace	27,741	24,123	21,468	17,786
3. Mechanical Works	25,310	24,520	23,590	22,520
4. Electrical Works	69,741	58,784	51,367	41,391
5. Land Acquisition	9,000	9,000	9,000	9,000
Sub-total	324,712	305,398	292,037	274,753
6. Engineering & Govern. Adomi.	24,353	22,905	21,903	20,606
7. Physical Contingency	45,935	43,497	41,760	39,641
Total	395,000	371,800	355,700	335,000

B.2.3 F.S.L.220.0 M

Items	Amount (*1000$) 225 MW	187.5 MW	150 MW	112.5 MW
1. Preparatory Works	2,522	2,420	2,323	2,182
2. Civil Works				
C-1 Diversion	16,177	16,177	16,177	16,177
C-2 Dam & spillway	92,184	92,184	92,184	92,184
C-3 Intake	28,522	26,036	22,942	18,726
C-4 Powerhouse & tailrace	31,236	26,950	23,565	18,347
3. Mechanical Works	25,390	24,160	23,150	21,770
4. Electrical Works	65,616	55,195	45,848	36,751
5. Land Acquisition	5,500	5,500	5,500	5,500
Sub-total	267,147	248,622	231,690	211,638
6. Engineering & Govern. Adomi.	20,036	18,647	17,377	15,873
7. Physical Contingency	37,217	34,931	32,733	30,089
Total	324,400	302,200	281,800	257,600

B.2.4 F.S.L.210.0 M

Items	Amount (*1000$) 187.5 MW	150 MW	112.5 MW	75 MW
1. Preparatory Works	2,260	2,061	1,907	1,729
2. Civil Works				
C-1 Diversion	15,414	15,414	15,414	15,414
C-2 Dam & spillway	66,838	66,838	66,838	66,838
C-3 Intake	33,378	26,911	21,371	15,734
C-4 Powerhouse & tailrace	35,055	28,262	23,499	17,278
3. Mechanical Works	25,890	24,430	22,870	21,490
4. Electrical Works	65,269	52,986	41,637	31,207
5. Land Acquisition	2,000	2,000	2,000	2,000
Sub-total	246,104	218,903	195,536	171,690
6. Engineering & Govern. Adomi.	18,458	16,418	14,665	12,877
7. Physical Contingency	34,138	30,479	27,499	24,333
Total	298,700	265,800	237,700	208,900

Table-5.1.2: PROJECT COST SUMMARY OF ASSUMED PROJECT SCHEMES

(10^6 US$)

No.	Type of Dam	FSL (m)	300 MW	262.5 MW	225 MW	187.5 MW	150 MW	112.5 MW	75 MW
A.1.1	Fill	240	519.2	499.0	480.2	464.2			
A.1.2	"	230		406.5	382.3	364.7	343.2		
A.1.3	"	220			323.4	297.4	276.5	253.9	
A.1.4	"	210				305.6	270.1	239.8	209.8
A.2.1	C.G./1	240	519.8	497.3	477.6	460.5			
A.2.2	"	230		411.4	387.1	371.1	350.4		
A.2.3	"	220			338.2	312.2	292.1	269.9	
A.2.4	"	210				308.4	275.6	247.3	218.5
B.1.1	Fill	240	465.1	442.5	421.4	399.9			
B.1.2	"	230		383.2	354.7	340.5	315.9		
B.1.3	"	220			339.7	309.5	285.9	257.9	
B.1.4	"	210				324.9	283.7	249.3	214.8
B.2.1	C.G.	240	498.7	477.9	455.9	438.9			
B.2.2	"	230		395.0	371.8	355.7	335.0		
B.2.3	"	220			324.4	302.2	281.8	257.6	
B.2.4	"	210				298.7	265.8	237.7	208.9

Note: /1; C.G. .... Concrete Gravity

Table-5.1.3: PROJECT COST FOR DESILTING BASIN SCHEME

Work Items	Amount (10^3 US$)
1. Preparatory works	3,706
2. Civil works:	
Diversion	41,160
Dam & spillway	95,625
Intake	80,595
Powerstation & T/L	29,689
3. Mechanical works	31,090
4. Electrical works	58,784
5. Land acquisition	9,000
Sub-total:	349,649
6. Engineering and government administration	26,224
7. Physical contingency	50,127
Total:	426,000

Table-5.1.4: POWER CAPACITY AND ENERGY APPLIED FOR POWER BENEFIT

Fiscal Year	1989/90	1990/91	1991/92	1992/93	1993/94
Power Requirement (MW)	76	104	142	182	224
Energy Requirement (GWh)	332	456	621	796	982

Case No.	F.S.L.	Instal-led Capacity	Depend-able Peak	Primary Energy	Second-ary Energy	1989/90 Capa-city	1989/90 Energy Prima-ry	1989/90 Energy Second-ary	1990/91 Capa-city	1990/91 Energy Prima-ry	1990/91 Energy Second-ary	1991/92 Capa-city	1991/92 Energy Prima-ry	1991/92 Energy Second-ary	1992/93 Capa-city	1992/93 Energy Prima-ry	1992/93 Energy Second-ary	1993/94 Capa-city	1993/94 Energy Prima-ry	1993/94 Energy Second-ary	1994/95
	Power and Energy to be Generated					Capacity and Energy Applied for Benefit															
	(EL.m)	(MW)	(MW)	(GWh)	(GWh)	(MW)	(GWh)	(GWh)	(MW)	(GWh)	(GWh)	(MW)	(GWh)	(GWh)	(MW)	(GWh)	(GWh)	(MW)	(GWh)	(GWh)	
1	240	300	215	933	1,153	76 (100)	332	317	100 (100)	431	258	142 (200)	621	575	182 (200)	796	470	215 (300)	933	692	Onward
2	"	262.5	214	929	983	76 (87.5)	332	253	104 (175)	456	581	142 (175)	621	482	182 (262.5)	796	670	214 (262.5)	929	590	"
3	"	225	212	923	798	75 (75)	323	194	104 (150)	456	482	142 (150)	621	383	182 (225)	796	555	212 (225)	923	479	"
4	"	187.5	187	810	694	76 (125)	332	442	104 (125)	456	368	142 (187.5)	621	530	182 (187.5)	796	425	187 (187.5)	810	416	"
5	230	262.5	175	760	1,013	76 (87.5)	332	251	104 (175)	456	539	142 (175)	621	440	175 (262.5)	760	608	Onward			
6	"	225	174	757	852	75 (75)	322	193	104 (150)	456	450	142 (150)	621	351	174 (225)	757	511	"			
7	"	187.5	173	252	672	76 (125)	332	427	104 (125)	456	352	142 (187.5)	621	482	173 (187.5)	752	403	"			
8	"	150	150	647	559	76 (100)	332	313	100 (100)	456	238	142 (150)	621	351	150 (150)	647	335	"			
9	220	225	135	587	870	75 (75)	321	190	104 (150)	456	401	135 (150)	580	326	135 (225)	587	522	"			
10	"	187.5	135	585	717	76 (125)	332	392	104 (125)	456	317	135 (187.5)	585	430	Onward						
11	"	150	134	580	544	76 (100)	332	297	100 (100)	429	239	134 (150)	580	326	"						
12	"	112.5	112	483	425	75 (75)	321	190	104 (112.5)	456	271	112 (112.5)	483	255	"						
13	210	187.5	95	413	718	76 (125)	332	330	95 (125)	410	283	95 (187.5)	413	431	"						
14	"	150	95	412	577	76 (100)	332	256	95 (100)	403	214	95 (150)	412	346	"						
15	"	112.5	94	409	414	76 (75)	319	174	94 (112.5)	409	248	Onward									
16	"	75	75	319	290	75 (75)	319	174	Onward												

Note: Figures in parentheses show the power installation in the Sapt Gandaki Project.

Table-5.1.5: SUMMARY OF OPTIMIZATION STUDY (1)

DAMSITE - A

(Unit: 10^3 US$)

Rockfill Dam Scheme							Concrete Gravity Dam Scheme						
F.S.L. (EL.m)	Installed Capacity (MW)	Cost (C)	Benefit (B)	Net Benefit (B - C)	B/C	E.I.R.R.	F.S.L. (FL.m)	Installed Capacity (MW)	Cost (C)	Benefit (B)	Net Benefit (B - C)	B/C	E.I.R.R.
240	300	640,662	670,646	29,984	1.05	12.7	240	300	629,568	670,646	41,078	1.07	12.8
"	262.5	622,847	656,305	33,458	1.05	12.8	"	262.5	608,622	656,305	47,684	1.08	13.1
"	225	601,405	639,307	37,903	1.06	12.9	"	225	585,810	639,307	53,497	1.09	13.3
"	187.5	587,308	585,788	-1,520	1.00	12.0	"	187.5	570,144	585,788	15,644	1.03	12.4
230	262.5	505,324	585,569	80,246	1.16	14.4	230	262.5	502,352	585,569	83,217	1.17	14.5
"	225	477,211	565,042	87,831	1.18	14.7	"	225	474,079	565,042	90,963	1.19	14.8
"	187.5	461,490	553,404	91,914	1.20	15.0	"	187.5	460,333	553,404	93,071	1.20	15.0
"	150	435,452	484,779	49,327	1.11	13.7	"	150	435,269	484,779	49,509	1.11	13.7
220	225	400,391	477,470	77,080	1.19	15.0	220	225	412,957	477,470	64,513	1.16	14.5
"	187.5	375,178	469,250	94,072	1.25	16.0	"	187.5	387,667	469,250	81,583	1.21	15.4
"	150	349,771	445,160	95,389	1.27	16.3	"	150	363,245	445,160	81,915	1.23	15.6
"	112.5	322,236	378,840	56,605	1.18	14.8	"	112.5	336,167	378,840	42,673	1.13	14.1
210	187.5	384,899	367,941	-16,958	0.96	11.3	210	187.5	383,276	367,941	-15,335	0.96	11.3
"	150	341,191	350,079	8,888	1.03	12.4	"	150	343,038	350,079	7,041	1.02	12.3
"	112.5	305,494	330,979	25,485	1.08	13.4	"	112.5	309,903	330,979	21,076	1.07	13.2
"	75	269,802	117,845	-151,957	0.44	1.0	"	75	275,673	117,845	-157,829	0.43	1.0

Note: Cost and benefit in the above Table are indicated in the present worth as of 1988/89.

Table-5.1.6: SUMMARY OF OPTIMIZATION STUDY (2)

DAMSITE - B

(Unit: 10^3 US$)

Rockfill Dam Scheme							Concrete Gravity Dam Scheme						
F.S.L. (EL.m)	Installed Capacity (MW)	Cost (C)	Benefit (B)	Net Benefit (B - C)	B/C	E.I.R.R.	F.S.L. (EL.m)	Installed Capacity (MW)	Cost (C)	Benefit (B)	Net Benefit (B - C)	B/C	E.I.R.R.
240	300	568,064	670,646	102,584	1.18	14.5	240	300	602,609	670,646	68,037	1.11	13.6
"	262.5	547,065	656,305	109,240	1.20	14.8	"	262.5	583,822	656,305	72,484	1.12	13.7
"	225	522,587	639,307	116,720	1.22	15.1	"	225	558,241	639,302	81,067	1.15	14.1
"	187.5	501,412	585,788	84,376	1.17	14.4	"	187.5	542,711	585,788	43,076	1.08	13.2
230	262.5	473,896	585,569	111,673	1.24	15.5	230	262.5	481,480	585,569	104,089	1.22	15.2
"	225	440,237	565,042	124,805	1.28	16.2	"	225	454,537	565,042	110,505	1.24	15.6
"	187.5	428,835	553,404	124,569	1.29	16.3	"	187.5	440,681	553,404	112,723	1.26	15.9
"	150	398,773	484,779	86,005	1.22	15.2	"	150	415,617	484,779	69,162	1.17	14.6
220	225	421,014	477,470	56,456	1.13	14.1	220	225	395,114	477,470	82,357	1.21	15.4
"	187.5	390,374	469,250	78,876	1.20	15.2	"	187.5	374,733	469,250	94,517	1.25	16.1
"	150	361,419	445,160	83,740	1.23	15.7	"	150	349,938	445,160	95,222	1.27	16.4
"	112.5	326,908	378,840	51,933	1.16	14.6	"	112.5	320,312	378,840	58,528	1.18	15.0
210	187.5	409,675	367,941	-41,734	0.90	10.3	210	187.5	370,536	367,941	-2,595	0.99	11.9
"	150	358,543	350,079	-8,464	0.98	11.6	"	150	330,200	350,079	19,879	1.06	13.0
"	112.5	317,516	330,979	13,463	1.04	12.7	"	112.5	297,317	330,979	33,662	1.11	14.0
"	75	275,967	117,845	-158,123	0.43	1.0	"	75	263,109	117,845	-145,264	0.45	1.0

Note: Cost and benefit in the above Table are indicated in the present worth as of 1988/89.

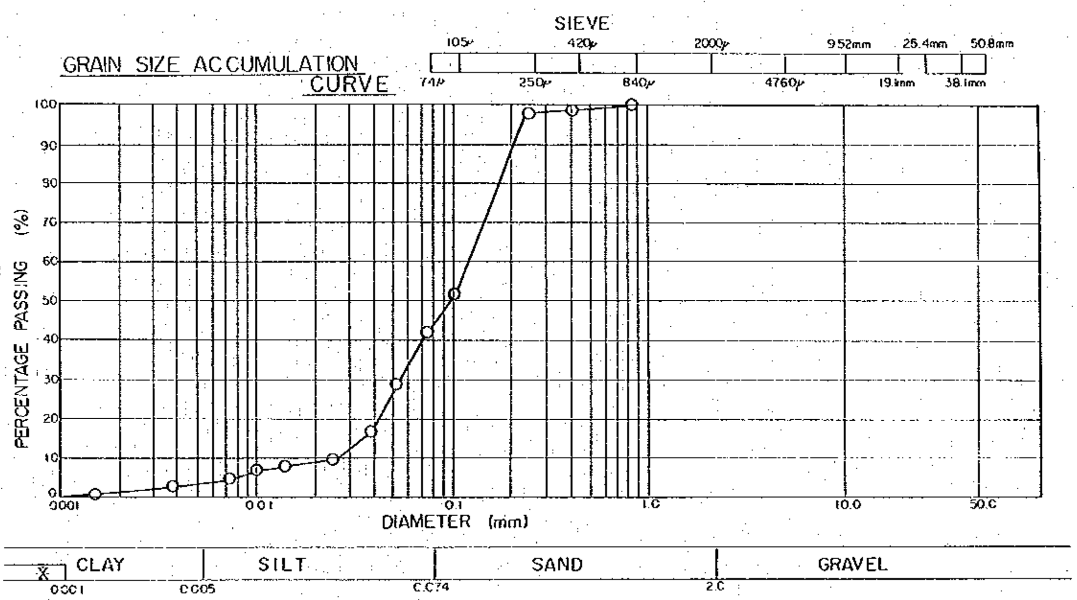
Table-5.1.7: RESULT OF GRAIN SIZE ANALYSIS FOR SEDIMENTS (SUSPENDED LOAD)

PARTICLE SIZE & WEIGHT PERCENTAGE OF PARTICLES UNDER THE SIZE

SPECIFIC GRAVITY Gs 2.732

SIEVE	GRAIN SIZE (mm)	50.8	38.1	25.4	19.1	9.52	4.76	2.00	0.84	0.42	0.25	0.105	0.074
SIEVE	TOTAL PASSING(%)								100	99.8	98.3	51.9	42.0
HYDROMETER	GRAIN SIZE (mm)	0.0527	0.0386	0.0249	0.0144	0.0102	0.0073	0.0037	0.0015				
HYDROMETER	TOTAL PASSING(%)	29.1	17.3	10.2	8.2	7.4	5.4	3.1	0.7				

GRAIN SIZE ACCUMULATION CURVE



※ COLLOID

PROPORTION			
4.76mm <	0 %	MAXMUM DIAMETER	0.84 mm
4.76~2.00mm	0 %	60% DIAMETER	0.12 mm
2.00~0.42mm	0 %	30% DIAMETER	0.055 mm
0.42~0.074mm	58 %	10% DIAMETER	0.024 mm
0.074~0.005mm	38 %	COEFFICIENT OF UNIFORMITY	5.0
0.005mm >	4 %	COEFFICIENT OF CURVATURE	1.1