The leakage conditions are not much related with the sort of rock, except for some parts of calcareous rocks as on the left bank of the sub-dam No.1 where the developed solution cavities and the accompanying decomposition by weathering are obviously the cause of very high leakage. Groundwater tables measured in the drilling holes are all higher than the water level of the Amban Ganga and rising toward both banks. This is a negative evidence to possibility of such solution cavities as to cause serious water leakage and deformation of groundwater regime in the damsite.

I.2.3.3 Excavation Line

(1) Foundation of concrete gravity dam and impervious core zone of fill type dam

Excavation lines are determined from the strength of rock and the permeability or its treatability. In the rock classification in Table I.1.1, the intensively weathered zone and the moderately weathered zone are not applicable for foundation of concrete dam and impervious core because of their insufficient stability. Permeability in those zones are not only so high as 50 Lugeon unit but also difficult to improve by grouting because of intensive deterioration of the rocks. The slightly weathered zone and the fresh rock zone are acceptable in the aspect of strength, as clarified in Sub-section 2.3.1. In view of leakage, the upper part of the slightly weathered zone is highly permeable with more than 50 Lugeon unit. However, considering that this leakage is obviously through open cracks in hard rock, it can be rather easily improved by grouting. In consequence, the foundation excavation shall be to the surface of the slightly weathered zone.

The rocks surrounding the solution cavities will have to be excavated deeper than the other parts, because they are intensively weathered to a considerable depth along with the development of the cavities and because it is deemed rather difficult and uneconomical to treat them by grouting or concrete replacement, as explained in Sub-section 2.3.4.

The proposed excavation lines for the damsites are shown in Geological Profile of the Damsite.

(2) Foundation of shell zone of fill type dam

Organic top soil and residual soil in 1 to 2 m of thickness should be removed for the foundation of shell zone. In the other words, the excavation line it shall rest on the surface of the intensively weathered zone.

I.2.3.4 Foundation Treatment

If the intensively weathered zone and the moderately weathered zone are excavated, no difficulties are seen in treatment of the slightly weathered zone by grouting for consolidation and leakage cut-off in the reach of high permeability. Treatment of the solution cavities is the only problem.

Test grouting was performed on the left abutment of the sub-dam No.1 through the calcareous rocks to examine effectiveness of grouting for the cavities and the surroundings.

By the grout hole GH3 which was drilled first to 38 m of depth, it was confirmed that cavities and high leakage zone were located within 26 m of depth. On the other hand, the zone to 14 m is so intensively weathered that it is, no doubt, the subject to excavation. Consequently, interest is concentrated to the section between 14 m and 26 m.

Depth (m)	Geological Condition	Leakage (Lug.unit)
0-15	Intensively weathered zone with cavity (Decomposed rock)	More than 100
14-26	Slightly weathered zone and fresh rock zone, occasionally with cavities	35 - 62
26-38	Fresh rock zone	less than 1

Cavities were encountered in the depth from 10 m to 20 m, and some open fissures were also found at 1 to 2 m intervals in the depth from 10 m to 26 m. These open fissures seem to be connected with the cavities. The fresh rock zone between 26 m and 38 m is solid, massive and watertight.

Grouting was carried out by injecting neat cement-water mixture and partly fluid mortar into five grout holes, in order, by stage as follows:-

The 1st stage	4.85 - 14.0 m
The 2nd stage	14.0 - 18.5
The 3rd stage	18.5 - 23.0
The 4th stage	23.0 - 27.5

The depths of the stages were subject to modification depending on the actual rock condition encountered in the grout holes. The tested spacing of the grout holes were 2.15 m and 1.23 m. (See Table I.2.7 and Fig. I.2-4).

Grout takes in each of the five grout holes were as follows:-

Hole No.	Grouting Section (m)	Grout Take (kg/m)
GH3	14.85-28.90	803.7
GH1	13.80-27.45	189.9
GH2	13.60-27.45	133.3
GH4	12.65-27.45	217.7
GH5	12.95-27.70	351.7
	Average grout take	339.3 kg/m

The effect of grouting was examined by comparing the leakage in check hole GH6 after grouting with that in grout holes prior to grouting. In this comparison, leakage rate in Lugeon unit decreased from 61.5 to 44.3 in the 2nd stage, from 38.8 to 17.9 in the 3rd stage and from 35.4 to 25.5 in the 4th stage. (Test in the 1st stage was renounced because this stage was within the intensively weathered zone that was inevitably to be excavated.) Improvement of permeability is clearly seen but not sufficient.

As mentioned in Section 2.2, the cavities observed in the adit No.8 were narrow openings of crevice type at widths from 20 cm to 50 cm, which were sub-vertical or inclined and filled with loose sandy loam. It seems that the injected grout was likely to be more or less isolated into small pockets in this sandy loam, which was neither penetrated nor washed out.

All of the above knowledges lead to the following strategy for treatment of the cavities:

- The fact that improvement of permeability was not sufficient with 1.23 m of grout hole spacing would not always mean that the hole spacing was still too large. On the contrary, it seems very probably that any shorter hole spacing could not result in remarkably better effect. Thorough washing-out of the filling material in the cavities is essential.

- The narrow crevice-shape of the cavities as observed in the adit No.8 renders it very difficult to make any manual work inside, such as for washing or concrete placing. In view of conceivable difficulties in washing for grouting, construction of deep concrete cut-off wall by excavation of trench or adits is a method worth consideration.

Principle for general foundation treatment for the area without cavities will be as follows:-

- The foundation treatment shall comprise curtain grouting, consolidation grouting for concrete gravity dam and blanket grouting for fill type dam. Drainage holes shall be drilled for up-lift pressure relief.

- Grout curtain to decrease leakage should be deep enough to reach the fresh and watertight zone with permeability lower than 5 Lugeon unit, that is, 20 to 30 m deep from the excavation line.

- Depths of consolidation and blanket groutings for the purpose of tightening the surfacial zone of the foundations for concrete gravity dam and impervious core zone of fill dam shall be 5 or 10 m. These works can be concentrated only to the parts of frequent cracks.

- In view of high solidity of the rocks, with very sparse joints and high shear strength, fairly high grouting pressure, if necessary, can be applied with little risk of damaging the foundation.

I.2.4 Foundation of Diversion Weir Site

The contemplated diversion weir site is located about 200 m downstream from the damsite. The axes of the weir are across the Amban Ganga and the western channel of the sub-dam No.1.

The bed rock is gneiss with garnet and foundation conditions are very similar to those in the damsite. Acceptable firm rock of the slightly weathered zone (2.0-2.2 km/sec in P-wave velocity) is reached at 10 to 20 m of depth, through the intensively and moderabely weathered zones overlying it. Unconsolidated residual soil covering the slopes is only 1 or 2 m thick. The bottoms of both the Amban Ganga and the western channel are filled with river deposit, composed of sand or sandy silt, at 7 to 10 m thickness.

No low velocity zones were detected in seismic exploration along the dam axis, suggesting no serious fracturing or weak zone in the foundation.

I.2.5 Geology of the Reservoir Area

The reservoir area covers approximately 40 km^2 (depending on the high water level) within about 20 km upstream from the damsite. In this area the Amban Ganga flows meandering from west to east, with tributaries Ma Oya and Kalugalatta Oya from north, and Kambaraua Ganga and Galboda Oya from south. These tributaries trend nearly north-south, reflecting the geological structures. Ridges at 1,000 to 1,500 m in altitude, around and across the reservoir area, also show north-south trends. Along the Amban Ganga and the tributaries develop narrow cultivated lands, and the rest parts are covered by jungle.

Topographically, this area shows characteristics of the valley in late mature stage. By developed side erosion the valleys are wide open, though slopes on the hillsides are fairly steep.

Geologically, the bed rocks in the reservoir area are gneisses of various mineral composition, charnockite, quartzite, calc gneiss and crystalline limestone, all of the pre-Cambrian Highland Series (See Fig. I.2-5). Quaternary unconsolidated deposits, such as river sand and silt, residual soil from weathering of rock and talus deposits, are rather thin.

The rockbeds strike N-S to N40^{o}E and dip horizontal to 40^o westward in general, except in foldings.

Very gentle warping folds are found to the north and to the south of the damsite, with axis roughly stretching north-southward. Faults with NE-SW trend and the others with NW-SE trend are assumed geomorphologically in the area of Kambaraua Ganga, Galboda Oya, Mt. Moragahakanda and the damsite, that is, in the eastern area of the reservoir.

Due to the mentioned thinness of unconsolidated Quaternary deposits and the high solidity of bed rocks, no possibility is conceivable for land sliding. As for possibility of water leakage from the reservoir, the solution cavities in calcareous rocks should be drawn to attention. Those calcareous rocks, i.e. crystalline limestone and calc gneiss, are ordinarily solid, scarcely cracked and watertight, if they do not bear those solution cavities as observed in drilling and aditting on the left bank of the sub-dam No.1. The cavities, formed by solution of calcium carbonates into water during a long geological span of time, develop so irregularly that it is not easy to trace their stretchings and it is extremely difficult to obtain a comprehensive picture of their exact development and distribution in a certain area.

What can be said about this problem in this reservoir area is the followings.

(1) Development of calcareous rock beds is limited in certain localities in the reservoir area, that is, two strata at about 300 m of thickness in the area upstream from the confluence of Ma Oya and one through the sub-damsite to the western slope of the right bank ridge along the Kambaraua Ganga, all of which beds stretch in the north-south direction. Thickness of the watershed in the north and south where these calcareous rock beds pass out of the reservoir area is more than 500 m at high water level. Only in the sub-damsite and on the eastern slope of Mt. Moragahakanda, a calcareous rock bed runs out from the reservoir in a short distance. The calcareous bed in the sub-damsite will be treated as described in Sub-section 2.3.4.

(2) Development of cavities is rather limited in certain localities in the calcareous rock beds, and not prevalent in those beds. In geological investigations carried out so far, cavities were found rather rarely; only on the left bank of the sub-damsite No.1 and on the eastern slope of Mt. Moragahakanda. The former were encountered by drilling and aditting. The latter are exposed on the ground surface; one is thin crevice-shaped openings with 20 to 30 cm of width around EL. 190 m and another is a group of developed narrow cavities at the zone far higher than the planned dam crest. Vertical development of the cavities is within 35 m, in so far as known about those in the sub-dam No.1.

(3) In drillings at the damsite, no extraordinary disturbance or depression of groundwater table was observed in spite of those cavities as mentioned above. On the eastern slope of Mt. Moragahakanda where some cavities are found, drilling DM39 from EL. 352 m for research of quarry revealed that the groundwater table

was about 100 m higher than the contemplated high water level. All of the above suggest that those cavities are not so influential as to cause serious water leakage.

(4) While some cavities are encountered as mentioned above, no kharstic topography worth noting is observed in the reservoir area. Neither noticeable water springs are seen.

The above situations releave fairly the fear of water loss from reservoir through the solution cavities. Foundation treatment in the damsite will be the only work required for the problem of cavities.

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I.3 CONSTRUCTION MATERIALS

I.3.1 Outline of Investigations

Topographic characteristics of the Moragahakanda damsite necessitate two subdams besides the main dam. As the construction materials, the proposed Concrete Gravity Type Dam will need fine and coarse aggregates while the proposed Rockfill Type Dam will require impervious core material, filter material and rock material, whereas earth material will be necessary for the proposed Earthfill Type Dam.

The material investigations were conducted in order to collect the available data of previous investigations conducted on these construction materials since 1961 and to clarify the available quantity and the characteristics of each of these construction materials based on the results of field investigation and laboratory tests and analysis of the results of previous survey.

All the field tests and laboratory tests in investigations were conducted by the staff of the Material Testing Laboratory of the Irrigation Department of Sri Lanka.

Material investigations for Moragahakanda Dam were conducted at four different stages including the initial survey made by UNDP/FAO in 1961 and the investigation conducted by JICA in 1978.

Therefore, the study consisted of the collection and analysis of the available data of the previous investigations and the additional field investigation and laboratory tests to confirm and supplement the previous investigations.

All the available data collected in 1978 are as listed in Table I.3.1.

The field investigations added 4 test pits and 28 auger holes for soil materials, 6 pits for the extraction of samples for filter materials and for the fine concrete aggregates, 4 holes of core drillings (DM29, DM39, DM40 and DM41) and blasting at 2 points for the extraction of samples for rock material and coarse concrete aggregates.

The laboratory tests conducted on the samples extracted in this way are shown in Tables I.3.2, I.3.3, and I.3.4.

Large Scale Shearing Tests on rock materials using an adit in the damsite were also carried out in 1978.

I.3.2 Soil Materials

I.3.2.1 Available Quantity of Soil Materials and the Selection of Borrow Areas

Borrow areas of impervious core materials for the rockfill dam and earth materials for the earthfill dam were investigated in the upstream and downstream areas within reach of about 6 km from damsite. In the first investigation conducted in 1961, an area in the upstream within a radius of about 3 km from damsite was investigated with 49 test pits allocated at intervals of 400 m. Although the detailed data of investigations could not be collected during the investigations in 1978, the Summary Report, shows that the upstream borrow areas cover 5,600 m^2 on the left bank and 2,400,000 m^2 on the right bank and the effective thickness of the layer of earth material overlying the decomposed rock stratum is about 1.5-2.5 m. The soils are composed mainly of SC material (by Unified Soil Classification) and a small quantity of GC material and CL material. Except for a very small amount of CL material with a high clay content, all these materials are suitable as impervious core material and earth material. The quantity of soil materials available in the upstream borrow area was estimated to be about 2,500,000 m^3.

The second investigations were conducted in 1965 under the advice of UN Engineering Geologist Dr. J.B. Auden. In this investigation, importance was given to the downstream borrow area where 9 test pits and 97 auger holes were distributed. At the same time 4 test pittings and 29 augerings were also made in the upstream borrow area. Although detailed results of laboratory tests are not available, the downstream borrow area covers more than 1,600,000 m^2 and the average thickness of soils was estimated to be 2.4 m according to the Summary Report. Almost all of the soil samples were classified as SC materials and the available quantity of earth materials was estimated to be about 3,800,000 m^3.

The third investigations were started in May 1978 by the Material Testing Laboratory of the Irrigation Department. This investigation was conducted in the downstream borrow area to confirm and supplement the results of the second investigation made in 1965, 45 test pittings and 25 augerings had already been made while gradation tests of 31 samples, several tests for the determination of the specific gravity of soil particles and several compaction tests were completed.

According to their results, the downstream borrow area consists of mainly SC materials though some CL materials and GC materials could also be seen. All these materials are suitable for impervious core material and earth material. The thickness of soil layer is about 1.5-2.5 m and the soil types agreed well with the results of the second investigation.

In the investigations, field survey was limited to 4 test pits and 28 auger holes to confirm and supplement the investigations conducted so far, while detailed tests were conducted with emphasis laid on soil test.

The soil types agreed well with the results of the previous three investigations with the majority covered by brown or reddish brown SC material, and small quantities of reddish brown GC material (Quartz gravel mixed) and light brown CH material.

Since the borrow areas proposed in the four investigations cover a wide extent in both downstream and upstream of the damsite, hereinafter they are referred to as follows. Upstream borrow area on the left bank is divided into Borrow Area A and B, while that on the right bank is divided into Borrow Areas C_1 and C_2. Downstream borrow area on the right bank is denoted as Borrow Area D while those on the left bank are divided into Borrow Areas E, F_1, F_2, F_3 and G.

The details of each borrow area are given in Table I.3.5.

According to Dr. J.B. Auden (1965), with regards to the Borrow Areas A, B, C_1 and C_2, since the base rock is limestone, the selection of borrow area in the upstream and the removal of natural blanket in the reservoir basin can lead to future leakage of reservoir. Although this is a problem of possibility, it is not necessary to persist on the upstream borrow areas when the same type of materials can be obtained under the same conditions in both upstream and downstream borrow areas.

The downstream Borrow Area D is located on the right bank and the hauling distance can become extremely large depending on the site selected for the bridge over Amban Ganga. Moreover, since it is very close to the river and is relatively a low area at EL. 145 m - 150 m, this area cannot be considered suitable as a borrow area when considering the problems such as drainage during the rainy season.

Borrow Area G is too far from the damsite and partly has a low lying area where drainage will be a problem. Considering these factors, Borrow Areas E and F are considered the most preferable. Out of these, specially Borrow Area F_1 is located in a short distance (about 1.4 km) to the damsite and has a gentle slope throughout its area with favourable drainage condition. Therefore, Borrow Area F_1 can be considered as the most suitable borrow area.

The next favourable borrow area in a reasonable distance to the damsite is Borrow Area E although outcrops of limestone is seen here and there. If this area is further extended to Borrow Area F_2, it is possible to collect 3,500,000 m^2 of soil materials.

However, problem is that clearing of the thick jungle and removal of topsoil will largely influence the unit price of excavation since the layer of soil materials is very thin. But, as better areas are not found within a radius of 5 km from the damsite, Borrow Areas F_1, F_2 and E can be the most suitable borrow areas in the region.

The typical geological profile of these borrow areas is shown in the DRAWINGS.

I.3.2.2 Physical and Mechanical Properties

The laboratory test results in previous investigations and the present ones (Figs. I.3-1, I.3-2, I.3-3 and I.3-4 and Tables I.3.6, I.3.7 and I.3.8) clearly show that the grain size and physical properties of each soil type are almost the same in Borrow Areas A, B, C, D, E and F. Therefore, the soil material in the borrow areas within a radius of 6 km, both in the upstream and downstream of the damsite, could be represented at large by three soil types; SC materials, GC materials and CH, CL materials.

SC Material

SC material is widely distributed throughout this region in large quantities. It is generally brown or reddish brown in colour and has a grain size distribution with 20-30% clay content, 10-20% Silt content and 50-70% Sand and Gravel content. The specific gravity of soil particles is about 2.66. It is very plastic owing to the high clay content and has a field moisture content of 12-20% and a plastic index of 12-27%.

The mechanical properties of SC material are represented by MK-51 (in Table I.3.9 and Fig. I.3-5(3)). These Tables and Figures show the results of tests conducted to verify the variation of mechanical properties of soil with the change of moisture content, within a range of about ±4% from the Optimum Moisture Content.

The results of tests conducted in the investigation in 1978 show strength characteristics somewhat differ from those in the previous investigations (see Table I.3.7). However, this is due to the fact that new triaxial compression tests were conducted in Saturated Consolidated Undrained condition while the previous tests were conducted in Non-saturated Unconsolidated Undrained condition.

As clear from Fig. I.3-5(3/3), the compaction of SC material is very effective showing a dry density of 1.78-1.87 t/m^3 within a range of ±3% of the optimum moisture content, that is 14% and is slightly on the dry side of the field moisture content.

The compression tests show an angle of internal friction ϕ' of over 30° and a cohesion C' of over 2 t/m^2. The coefficient of permeability is not very much affected by the density of compaction due to the high clay content and is very low at 10^{-8} cm/sec. showing that this soil is a very impervious material suitable for core material.

GC Material

GC material is distributed only in some parts of the upstream and downstream borrow areas, at depths comparatively close to the topsoil. It is generally reddish brown in colour and is composed of 15-25% Clay content, 5-10% Silt content, 15-45% Sand content and 15-50% Gravel content. The specific gravity of soil particles is about 2.66. The soil is plastic with a field moisture content of 8-12% and a Plasticity Index of 12-33%.

The mechanical properties of GC material are represented by MK 50 (in Table I.3.9 and Fig. I.3-5(2/3)).

As shown in Fig. I.3-5(2/3), compaction of GC material is extremely good with a dry density of 1.81-1.92 t/m^3 within a range of 3% of the optimum moisture content. The optimum moisture content is 13% and is about the same as the field moisture content.

This material has an angle of internal friction ϕ' of over 30° and a cohesion of over 3 t/m². The coefficient of permeability is very low at 10^{-8} m/sec. owing to the high content of clay. This material is the most suitable for core material.

CH. CL Material

This material is widely destributed in the upstream and downstream borrow areas at relatively shallow depths in moderate quantity. It is generally brown or light brown in colour and has a grain size distribution of 25-55% Clay content, 10-30% Silt content and 15-40% Sand and Gravel content. The specific gravity of soil particles is about 2.68. The field moisture content has a wide range of 17-31%, but the soil is plastic with a plasticity index of 11-33%.

The mechanical properties of CH. CL material are represented MK 49 in Table I.3.9 and Fig. I.3-5(1/3).

The dry density in the compaction test within a range of ±3% of the optimum moisture content is comparatively low at 1.61-1.71 t/m³. The optimum moisture content is 18% and is on the dry side of the field moisture content.

Within the same range of moisture content, the angle of internal friction ϕ' is over 17° and the cohesion C' is over 2 t/m². The coefficient of permeability is not affected by compaction due to high content of clay and is 10^{-8} cm/sec. This very impervious material can be used as core material, but its strength is comparatively low.

I.3.2.3 Suitability as Impervious Material

Judging from the physical and mechanical characteristics of each soil type, GC material is rated as the best suited soil material to be used as impervious core material in the rockfill dam or as earth material in the earthfill dam, followed by SC material and CH. CL material in that order.

Out of the above three soil types, even the inferior CH. CL material is not necessarily judged as not suitable for the impervious core material or earth material. Its impermeability is excellent and moreover, the field moisture content is very close to the optimum moisture content. In the compaction test, dry density show an increase when the compaction energy is raised to 300% of the standard compaction energy, showing no problems due to overcompaction (see Figs. I.3-6 and I.3-7). With an average plasticity index of 25, this material has sufficient resistance against piping. The strength is somewhat low, nevertheless, this material could be rated as sufficiently suitable for a dam with height less than 70 m.

Among the three soil types, the available quantity of SC material is the largest while that of GC material is the smallest. Moreover, since GC material is distributed very close to the ground surface, it is difficult to depend on this quantity. Therefore, when Borrow Areas E and F are selected, SC material becomes the main material accompanied by some

CH. CL material. Anyway, since these soils are distributed only within a small depth making it necessary to collect the soil material from a wide area, it is virtually impossible to differenciate the soils of those three categories in the site.

Therefore, the various design criteria for the basic design of the dam should be determined with respect to the inferior CH. CL materials.

Considering that the moisture content is controlled within a range of ±3% of the optimum moisture content, the minimum values in this range are taken as the design values of the CH. CL material. However, since all the new triaxial compression tests were conducted in saturated condition, the strength in the unsaturated condition was estimated at C = 4.0 t/m^2, or twice the value in saturated condition, after thorough consideration of the results of previous triaxial compression tests conducted in unsaturated condition. The design values are summarised below (see also Fig. I.3-5(1/3)).

Moisture Content	=	21%
Specific Gravity of Soil Particles	=	2.66
Dry Density	=	1.61 t/m^3
Bulk Density	=	1.95 t/m^3
Cohesion (when Saturated) C'sat	=	2.0 t/m^2
Angle of Internal Friction ϕ'	=	17^o
Coefficient of Permeability k	=	10^{-7} cm/sec

3.2.4 Matters of Special Attention

The major part of soil materials selected for impervious material will be SC material. However, CH. CL material will also be mixed in large quantities. Therefore, development of high pore water pressures, which is detrimental to the stability of the dam, could be expected during compaction. However, it is difficult to observe the actual behaviour of pore water pressure in the trixial compression tests conducted in the present investigation since the pore water pressures are not measured completely.

In order to overcome the possible development of high pore water pressures during construction, it may be necessary in the design, to take suitable measures such as the limiting of the thickness of impervious core in the case of rockfill dam and the installation of Chimney Drains, etc., in the case of earthfill dam.

Moreover, prior to the detailed design, it is also necessary to determine the appropriate type of rolling equipment, the number of turns of rolling, the thickness of spreading and the working moisture content, etc., by conducting field compaction tests.

Since the impervious material required has to be collected from a comparatively thin layer of soil extending to a wide area, the use of power shovels in excavation can not be considered due to extremely low efficiency. Moreover, when considering the severe irregularities on the surface of base rock, bulldozers are considered more suitable than the scrapers for excavation and collection of soil material. Therefore, a combination of bulldozers with wheel loaders, etc., for loading and dump trucks for the transport of material is considered appropriate for this site.

In the proposed borrow areas, the exploitable soil material is covered by a comparatively thick layer of topsoil. Therefore, excavation and removal of a tremendous volume of topsoil becomes inevitable. It is an efficient and economical procedure to refill an already excavated area with disposed topsoil from the other area which is excavated later.

The stagnation of rain water, etc., in the borrow area can largely affect the quality of soil material. Since the proposed borrow areas are specially wide, it is necessary to pay special attention on the drainage in borrow areas during construction.

I.3.3 Filter Material and Fine Aggregates for Concrete

I.3.3.1 Availability and Exploitable Quantity

With respect to filter material for the fill type dams and fine aggregates for the concrete dam, investigations were carried so far on the riverbed deposits of Amban Ganga.

The first investigation of the riverbed deposit was conducted in 1966 along with the investigation of borrow areas for earth material. In this investigation, the thickness of sand at the riverbed was measured by penetrating a metal probe into the riverbed deposit. According to this survey conducted in a range of 3 km upstream and 1.3 km downstream, the average thickness of riverbed deposit was about 3 m.

Assuming the average thickness of riverbed deposit at 3 m and the average width of the river at 40 m, about 520,000 m^3 of riverbed deposit could be expected in the 4.3 km stretch of river investigated above.

The second investigation was conducted from July to September 1978 on the riverbed deposit of Amban Ganga in the upstream of dam site. According to the results of this investigation (see DRAWINGS), about 100,000 m^3 - 150,000 m^3 of sand is deposited for every kilometer length of the river in the investigated section.

As above, investigations had been conducted twice in the past on the riverbed deposit and the volume of deposit does not differ very much in them. Therefore, in the present investigation, laying emphasis on the physical and mechanical properties of this material, tests for specific gravity, gradation and relative density and moreover, triaxial compression tests (at $D_r = 65\%$) were conducted.

Riverbed materials are deposited also in the low-lying areas other than the riverbed, for example, in the Borrow Areas C_1 and D. However, the results of reconnaissance and auger borings conducted in these areas show that the content of silt and finer material is relatively high with about 10%-30% and therefore, these materials could not be considered suitable. On the other hand, the riverbed material deposited under the water which is naturally washed and cleaned through the long years could be considered the most desirable material.

The sand suitable for filter material in the fill type dams and for fine aggregates in the concrete dam is available in Amban Ganga as riverbed material deposited to an average depth of 3 m. It is reasonable to consider a volume of about 100,000 m^3 per kilometer of river length.

Possibility of alternative fine materials to be obtained by crushing the rocks available in the damsite was examined by laboratory tests, similar to those conducted on river sand, on the samples from the adit in the damsite.

I.3.3.2 Physical and Mechanical Properties

In accordance with the results of the tests conducted in 1966 on riverbed deposit of Amban Ganga,while those of the tests conducted in the present investigation area shown in Table I.3.9.

River sand collected from Amban Ganga is very clean, and as clear from the grain size distribution curves shown in Fig. I.3-8, is a fairly coarse sand with an average Uniformity Coefficient of 2.64. This sand has a Fineness Modulus of 3.40 on the average.

Although there is a slight variation in the specific gravity of sand in the previous investigation, this variation is very small in the present investigation. In the dry basis, the average value of specific gravity is 2.55 and in the saturated surface dry basis, this value is 2.62.

Also in the absorption test, the results in the two investigations differ largely. However, in the present investigation, the average absorption is 1.20%.

From the above results, specially in the present investigation, the riverbed deposit of Amban Ganga is suitable both as a filter material for the fill type dams and as the fine aggregate in the concrete dam.

Further, in order to grasp the mechanical properties of riverbed deposit, triaxial compression tests are being conducted using sand with a relative density of 65%.

The grain size distribution curve of rock blastings obtained from the adit is shown in Fig. I.3-8 (curve MKF-28). Since the content of silt and finer particles is over 30%, this material is not considered suitable for the filter material or for the fine aggregate.

I.3.3.3 Suitability as Filter Material

The river sand of Amban Ganga is the only material available in the close proximity of the damsite and suitable as a filter material for filltype dams and as a fine aggregate for the concrete dam. The test conducted on rock blastings obtained form an adit in the damsite prove that rock blastings cannot substitute the river sand due to the very high content of silt and finer grains, which exceeds 30%, in the former.

The river sand is found to be clean and homogeneous throughout the area investigated. Its relationship as a filter material with respect to the impervious material is excellent satisfying the conditions F_{15}, $5B_{15}$ and F_{15} $5B_{85}$ (see Fig. I.3-9). Moreover, with practically on particles passing No. 200 Standard Sieve, this sand satisfies all the conditions of a filter material, including the condition; F_5 0.0075 mm. The only shortcoming of this river sand is that its grain size distribution curve is not parallel to the grain size distribution curves of impervious material but rather steep. Even this would not be a problem if the width of the filter is taken larger than 3 m.

The design values of the filter material are established as follows based on Table I.3.9.

Moisture content w	=	6%
Specific Gravity of Soil Particles	=	2.62
Dry Density γ_d	=	1.80 t/m^3
Bulk Density γ_t	=	1.91 t/m^3
Cohesion C	=	0
Angle of Internal Friction ϕ	=	30^o

Since the results of the tests for the determination of density and the results of triaxial compression tests conducted at 65% relative density are yet to be obtained, the generally used values are adopted here tentatively. The angle of internal friction of sand is generally in the range of 30^o-35^o and the lowest value of 30^o is taken here as the design value.

I.3.3.4 Suitability as a Fine Aggregate for Concrete

The fine aggregates for dam concrete should be clean, excellent in hardness and durability, of suitable particle size and should be free form harmful amounts of dirt, mud and organic material, etc. In this respect, riverbed material of Amban Ganga is very clean and practically does not contain any harmful amounts of organic material, etc.

The most appropriate grain size distribution of fine aggregates for dam concrete is said to be within the range shown in Table I.3.10, which is established with long practical experience. Particles finer than 0.15 mm are very important both in reducing the bleeding of concrete and improving the workability. However, the riverbed material tested in this investigation, practically has no particles finer than 0.15 mm and on the other hand is a relatively coarse sand with an average fineness modulus of 3.40.

Therefore, this riverbed material alone could not play the role of fine aggregate satisfactorily. However, it can be used satisfactorily as a fine aggregate for dam concete by adjusting its grain size distribution using fine sand available in the sands of the river or using the fine material obtained by crushing the rocks available in the damsite.

Next, with regards to the use of quarry rocks for a fine aggregate for dam concrete, the tests conducted in this investigation prove that this material is not desirable for this purpose due to its high content (38%) of particles finer than 0.15 mm. Moreover, considering the shape of crushed particles which has a large influence over the strength of concrete and the large costs necessary for crushing plants, etc., river sand is remarkably superior to the rock blastings in many ways.

I.3.3.5 Matters of Special Attention

In collecting the material, since it is necessary to dredge the riverbed material from a relatively long stretch of the river, the use of pump dredgers are considered desirable rather than drag lines, etc., from the view points of movability and efficiency. However, in this case, considerable turbity of river can be expected due to turbulance at the riverbed and it may be necessary to take suitable precautions.

I.3.4 Rock Material and Coarse Aggregates for Concrete

I.3.4.1 Available Quantity and the Selection of Quarries

In the investigations conducted in 1965, after reconnaissance of the vicinity of damsite, 4 spots were considered as the possible quarries to collect rock material for the rockfill dam and coarse aggregates for the concrete dam. According to the report on this investigation, Quarry No.1 is located 3.5 km north of the damsite and consists mainly of crystalline limestone. Quarry No.2 is located on the left bank of the river about 2.0 km upstream of the dam site. This too, has crystalline limestone as the main rock type. Quarry No.3 is a small hill located on the left bank near Elahera Anicut in the downstream and consists mainly of charnockite. Quarry No.4 is located on the right bank about 6 km downstream and consists of charnockite.

In the present investigation, reconnaissance of these proposed quarries was made once again and it was found that Quarries No.1 and No.4 are not preferable for long distance of haulage, while Quarries No.2 and No.3 are both small hills and not sufficient for a massive requirement.

On the other hand, the hill with its peak at about EL. 355 m, proposed by the Japanese Preliminary Survey Team as a prospective quarry, is located on the right bank about 1.2 km upstream of the damsite.

Geologically, this area consists of gneiss, charnockits, crystalline limestone, calc gneiss, etc., of the Pre-Cambrian Highland Series. All of these rocks are in fresh condition hard and homogeneous with very few cracks, qualifying this site as an excellent quarry for rock materials and coarse aggregates. Results of Boring DM-29 show that rocks above EL. 225 m are gneiss and charnockite while those below this elevation are mainly crystalline limestone and calc gneiss.

The strike of the rocks varies from N-S to N 40°E. Dip is about 10°-30° to west. Fault structures donot exist in this area.

Quaternary deposit covering these baserocks, is only top soil not exceeding 1 m - 2 m in thickness. Talus deposits are rather rare. The flat areas close to Amban Ganga are covered with river deposits.

In Boring DM-29, the thickness of weathered zone is about 15 m. this weathered zone is in a condition where relatively fresh blocks remain among decomposed rock. Blocks of fresh gneiss are very hard and suitable as aggregates for concrete.

By selecting the area above EL. 200 m of this hill for quarry, it will be easy to construct a road for the transport of materials to the right bank of the damsite. This proposed quarry, which is referred to as Quarry Q-I, has a haulage distance of only about 1.5 km. By the adoption of Bench Cut Method, it is possible to obtain sufficient cut face. This proposed quarry meets in every way requirement of a massive production of material. Sufficient material is available here only by cutting a small part of this hill (see Fig. I.3-10).

With regards to the rock material for the rockfill dam, Moragahakanda Hill which makes the left abutment for the second sub dam is considered suitable for a quarry as proposed earlier by the Japanese Preliminary Survey Team. This area hereinafter referred to as Quarry Q-II is geologically very much similar to Quarry Q-I. In Boring DM-39, rocks above EL. 280 are seen to be gneiss and charnockite while those below this elevation are calc gneiss, crystalline limestone, etc. Both drillings DM-37 and DM-40 encountered crystalline limestone or calc gneiss. As in the case of Quarry Q-I, all the fresh rocks are hard and homogeneous with very few cracks.

The strike of the rocks is almost in the N-S direction and the dip ranges from 15° to the horizontal.

Quaternary deposit distributed over the baserock is only topsoil of thickness not more than 1 m - 2 m, while very thin layers of talus deposit are observed rarely.

Thickness of the weathered zone is about 4 m in DM-37, 9 m in DM-39 and 2 m in DM-40. The thickness in DM-39 is deemed common on the slope, whereas thin weathered zone as is seen in DM-37 and DM-40 is exceptionally characteristic for the terrace which had undergone intensive erosion in the past.

Whethered condition is very much similar to that in Quarry Q-I, with relatively fresh blocks contained in decomposed rock.

Considering the eastern side of this Moragahakanda Hill becomes the abutment to the second sub dam, the quarry should be located on a relatively steep slope above EL. 200 m on the southeast face of the hill, for advantages as follows:

- By selecting the quarry at an elevation above the highest water level in the reservoir, future problems of leakage through exposed rock could be avoided,

- The soil covering is thin,

- To make it possible to have cut faces as long as possible, and

- To bring down the average gradient of the road for transport of materials to about 5%.

As a result, the average distance of haulage from the Quarry Q-II to the damsite is about 1.5 km. Considering the scale of this hill and the quality of material available here, there is no problem with respect to the quantity of exploitable rock matereial (see Fig. I.3-11).

I.3.4.2 Physical and Mechanical Properties

Quarries Q-I and Q-II consist of various rock types such as gneiss, limestone, calc gneiss, charnockite, etc., either in fresh condition or in weathered condition. Several samples picked up from drill cores in various degrees of weathering were tested for the physical and mechanical properties such as specific gravity, absorption and compressive strength. The results of these tests are summarized in Table I.3.11.

In this table, samples denoted with DM-29 represent Quarry Q-I, while those with DM-37 represent Quarry Q-II. The remaining samples are obtained from the drillings made along the dam axis.

It is clear from this table that fresh gneiss has a specific gravity in the range of 2.91 to a rarely high value of 3.35, and a mean value of 3.04. Specific gravity of weathered gneiss is about 2.82 while that of calc gneiss is in the range of 2.83-2.91. Fresh limestone shows values in the range of 2.88-2.92.

Except for weathered gneiss which shows a relatively high degree of absorption in the range of 0.37%-0.57%, all the rock types show values generally less than 0.25%. The average absorption for the fresh rocks of all types is 0.17%.

The compressive strength of fresh gneiss is relatively high with values varying from 1,111 kg/cm^2 to 1,832 kg/cm^2 and with an average value of 1,542 kg/cm^2. Calc gneiss shows a compressive strength in a wide range of 661 kg/cm^2 - 1,124 kg/cm^2. Although only two samples were tested, fresh limestone has a compressive strength in the range of 668 kg/cm^2 - 842 kg/cm^2. It was impossible to determine the strength of weathered gneiss because of cracks in the boring core specimens.

As clear from the forgoing, the rocks available in Quarries Q-I and Q-II have a compressive strength in a wide range of about 650 kg/cm^2 - 1,800 kg/cm^2.

Furthermore, tests were also conducted with blasted rock samples obtained from outcrops in the vicinity of Quarries Q-I and Q-II. The results of these tests for the various properties such as absorption, abrasion, soundness, density and specific gravity are presented in Table I.3.12. The rock types used in these tests are basic charnockite in the case of Quarry Q-I and limestone in the case of Quarry Q-II. These rock material obtained from outcrops are generally weathered during the long period of their exposure to the atmosphere.

The specific gravity of charnockite in Quarry Q-I is about 3.02 and that of limestone in Quarry Q-II is about 2.91.

In the soundness test by the Sodium Sulphate Method as per Designation 19 of U.S.B.R. Concrete Mannual, the percentage loss is less than 4.7% in the case of charnockite, but as large as 15.8% for the particles passing 3/8" sieve and retained in US-4 sieve in the case of weathered limestone. However, the average percentage loss of all coarse aggregates passing 3" sieve and retained in US-4sieve is only 2.2% in the case of charnockite and 7.9% in the case of weathered limestone.

Abrasion of charnockite is 7.4% after 100 revolutions and 36.1% after 500 revolutions as against the corresponding values of 22.0% and 68.4% of weathered limestone.

The average absorption of coarse aggregates is 0.27% for charnockite and 0.40% for weathered limestone.

I.3.4.3 Suitability as Rock Material

As mentioned earlier in 3.4.1, Quarry Q-II is meant for rock material for the rockfill dam. The laboratory test of the rock samples gives compression strength in the range of 650 kg/cm^2 - 1,800 kg/cm^2. The percentage loss in soundness test gradually decreases with the increase in size for particles above 5 mm. Taking the above into account, this material can be satisfactory for rock material for the rockfill type dam.

The design values of rock material are tentatively established as follows:

Moisture Content w	=	2%
Specific Gravity Gs	=	2.70
Dry Density γd	=	1.80 t/m^3
Bulk Density γt	=	1.84 t/m^3
Angle of Internal Friction ϕ	=	40^o (assumed conservatively)

Similar quality can be expected for calcareous rocks in the sub-damsite No.1. In case that diversion channel is excavated through this site, the excavated material can be utilized for rock fill material.

I.3.4.4 Suitability as Coarse Aggregates for Concrete

Quarry Q-I is proposed as the quarry for coarse aggregates for dam concrete. Results of the tests (see Table I.3.12) show that rocks obtained from this quarry sufficiently satisfy the following conditions for coarse aggregates as specified in the Concrete Manual;

Average absorption	=	0.22% - 1.0%
Soundness (% loss)	=	2.2% - 10%
Abrasion after 100 rev.	=	7.4% - 10%

Abrasion after 500 rev.	=	36.1% - 40%
Specific gravity	=	3.02 - 2.60

By selecting rocks of charnockite and gneiss in Quarry Q-I above EL. 200, sufficient quantity of material suitable as coarse aggregates for the concrete dam could be obtained.

I.3.4.5 Matters of Special Attention

The Large Scale Direct Shear Tests for which the preparations are being made at the field should be conducted without fail prior to the detailed design of the dams. The results of these tests should be taken into account in determining the design values.

At the stage of detailed design, it may be required to consider the necessity of excavating adits and conducting blasting tests in order to confirm the thickness of weathered zone and the variation of rock types in Quarries Q-I and Q-II.

TABLES

Table I.1.1 STRATIGRAPHIC COLUM OF THE PROJECT AREA

Era	Period	Formation	Main rock types and lithological description	
Ceno -zoic	Quat -ernary	Alluvial Deposit	River bed and flood plane deposit. Composed mainly of clay, silt, sand, with gravels	
		Reddish brown earth	Clayey sand, silt and loam. Residual deposit and talus deposit.	
Paraeo -zoic	Cambrian	Vijayan series	Gneiss, gneissose granite, granitic gneiss, granite, Augen gneiss, migmatite	: Composed of quartz, microcline (potassium feldspar), plagioclase, biotite and hornblende.
Pre-cambrian		Highland series	(Khondalite group)	
			Garnet - sillmanite schist and gneiss or Khondalite.	: Metamorphosed clays or shales (alumina-rich sediment). Characterized by alumina-rich minerals as sillimanite and garnet (almandin). Garnets are very large. Containing small amounts of graphite. .
			Quartzite and Quartz schist	: Metamorphosed sandstones. Composed of shapless crystalline quartz, with small amounts of sillimanite, garnet and magnetite. Quartz schist contains much feldspar. Frequently jointed and very permeable.
			Quartz-feldspar granulite and garnetiferous gneiss.	: Metamorphosed sandy clays or clayey sands. Light coloured rocks. Composed mainly of quartz and feldspar, with varying amounts of mica and garnet. Sometimes containing sillimanite and graphite.
			Crystalline limestone or Marble.	: Metamorphosed sedimentary limestone. Generally white coloured rocks. Composed mainly of calcite and dolomite, with varying amounts of silicate minerals. Partly accompanied by solution caves.
			Calc granulite and calc gneiss	: Metamorphosed calcareous muds or marls and calcareous sands. Dark greenish to blackish green coloured rocks. Composed mainly of diopsite, scapolite and hornblende, with abundant sulphide minerals. Sometimes containing much mica.
			Graphitiferous schist	: Metamorphosed muds with much organic matter. Characterized by many graphite and sulphide minerals.
			(Charnockites) Charnockites have several various rock types, range from fine to coarse-grained in the size of minerals, from acidic to basic in mineral composition, and from equigranular to gneissic in texture.	: Greenish-grey or bluish-grey coloured rock. Characterized by greenish or greyish coloured crystals of quartz and feldspar, and dark pyroxene (hypersthene). Composed mainly of quartz and feldspar or plagioclase or pyroxene, with mica, hornblende and garnet. Mechamorphosed sediments or volcanic rocks.
			(Kadugannawa gneisses) Amphibolites (hornblende-plagioclase schist)	: Composed mainly of hornblende, plagioclase, feldspar, mica and some pyroxene. Metamorphosed impure calcareous rocks or volcanic rocks.

Table I.2.1 RESULT OF CORE DRILLING

Hole No.	Depth (m)	Inclination of hole	Water press. test (times)	Location and remarks
DM-29	46.00	Vertical	-	Quarry site "Q-I"
DM-30	52.70	60 degree from horizontal	14	Main damsite
DM-31	52.40	60 degree from horizontal	14	Main damsite
DM-32	12.00	Vertical	-	Main damsite
DM-33	13.90	Vertical	-	Main damsite
DM-34	15.60	Vertical	-	Main damsite
DM-35	60.95	Vertical	18	First sub damsite
DM-36	61.25	Vertical	16	Second sub damsite
DM-37	61.60	Vertical	18	Second sub damsite
DM-38	58.35	Vertical	14	Second sub damsite
DM-39	91.45	Vertical	-	Quarry site "Q-II"
DM-40	91.75	Vertical	-	Quarry site "Q-II"
DM-41	45.70	Vertical	-	Quarry site "Q-II"
DW-1	30.00	Vertical	-	Diversion weir site
DW-2	19.50	Vertical	-	Diversion weir site
14 holes	713.15		94	

Table I.2.2 RESULT OF SEISMIC EXPLORATION

Exploration line	Length (m)	Location	Remarks
CA	800	Main damsite	Parallel with dam axis
CB	385	Main damsite	Parallel with dam axis
CC	305	Main damsite	Vertical with dam axis
CD	305	Main damsite	Vertical with dam axis
CE	205	Main damsite	Vertical with dam axis
RA	800	First sub damsite	Parallel with dam axis
RB	300	First sub damsite	Vertical with dam axis
EA	605	First sub damsite	Vertical with dam axis
EB	405	First sub damsite	Vertical with dam axis
EC	505	Second sub damsite	Parallel with dam axis
NCP-A	355	Diversion weir site	Vertical with river
NCP-B	335	Diversion weir site	-
12 lines	5,305		

Table 1.2.3 RESULT OF TEST ADIT

No.	Length: Open cut (m)	Tunnel (m)	Total (m)	Location
1	7.20	25.80	33.00	Right bank of main damsite
2	9.00	26.50	35.50	"
3	9.75	48.75	58.50	"
4	7.00	29.00	36.00	Left bank of main damsite
5	4.50	40.50	45.00	"
6	15.00	30.00	45.00	Right bank of first sub damsite
7	65.00	α	65.00 +α	Left bank of first sub damsite
8	16.00	21.00	37.00	"
	133.45	221.55 + α	355.00 + α	

Note: No.7 is not completed.

Table I.2.4 RESULT OF TEST GROUTING

Hole No.	Drilling depth (m)	Depth of grouting section (m)	Remarks
GH-1	27.45	11.05 - 27.45	Grouting hole
GH-2	27.45	13.60 - 27.45	"
GH-3	38.00	4.85 - 38.00	"
GH-4	27.45	12.67 - 27.45	Check and grouting hole
GH-5	27.70	12.95 - 27.70	Grouting hole
GH-6	27.45		Check hole
6 holes	175.50		

Location: Left bank of first sub damsite

Table I.2.5 ROCK CLASSIFICATION OF THE DAMSITE AREA

Rock classification	Velocity (Primary wave) km/sec	Geological condition
Intensively weathered zone	0.6 - 1.0	Mostly decomposed by weathering, looks like a sediment of sandy silt. Containing big fresh boulder (sometimes 2 m to 3 m in diameter)
Moderately weathered zone	1.2 - 1.5 (partly 1.5 - 1.7)	Partly decomposed by weathering, having wide open cracks (sometimes 1 m in width) filled by weathered sandy silt and clay.
Slightly weathered zone	2.0 - 2.7 (Partly 1.7 - 2.0)	Mostly fresh rock, with rare cracks. Cracks are stained by weathering, sometimes containing weathered clay (1 cm to 10 cm in width)
Fresh rock	5.5 - 6.0 (partly 4.4)	Massive and solid rock, with rare cracks. Cracks are closed tightly.

Table I.2.6 RESULTS OF IN-SITU ROCK TEST

Shear test

Block No.	BS-1 Pn = 25 ton		BS-2 Pn = 1 ton		Bs-3 Pn = 35 ton		BS-4 Pn = 10 ton	
Upheaval point	Pi=130t	σ = 17.50 τ = 34.53	Pi=170t	σ = 14.08 τ = 45.16	Pi=200t	σ = 25.97 τ = 53.13	Pi= 80t	σ = 9.27 τ = 21.25
Principal yield point	Pi=300t	σ = 31.31 τ = 79.69	-	- -	Pi=270t	σ = 31.65 τ = 71.72	-	- -
Failure point	Pi=320t	σ = 32.93 τ = 85.00	Pi=250t	σ = 20.58 τ = 66.41	Pi=280t	σ = 32.46 τ = 74.38	Pi=235t	σ = 21.86 τ = 62.43
Re-test Failure point	Pi=105t	σ = 15.47 τ = 27.89	Pi= 30t	σ = 2.71 τ = 7.97	Pi=110t	σ = 18.66 τ = 29.22	Pi=145t	σ = 14.55 τ = 38.52

Plate loading test

Test No.	Tangential elasticity Et kg/cm^2	Secantial elasticity Es kg/cm^2	Modulus of deformation D kg/cm^2	Creep ratio Cf %
PL-1	-	-	-	-
PL-2		250,000	86,000	63
PL-2'	-	1,247,000	-	20
PL-3	90,000	84,000	52,000	-

Note: Displacement was generally very little under the applied *maximum load of* 40 to 60 tons in the plate loading test. Also creep was sometimes very little or none.

Pn : Constant normal load
Pi : Inclined load
σ : Normal stress (kg/cm^2)
τ : Shear stress (kg/cm^2)

Upheaval point : an inclined load where the test block begins upward displacement.
Principal yield point : the load where shearing displacement rate start to increase.
Failure point : the load where the load does not rise any more.

Table I.2.7 RESULTS OF GROUTING TEST

Hole No.	Stage No.	Depth (m)	Water pressure test			Grouting							
			Max. press. in the test section kg/cm^2	Coefficient of permeability cm/sec	Lugeon unit	Max. pumping press. kg/cm^2	Max. density of grout S:C:W	Grouting time min	Grout Injected volume litre	Cement Injected quantity kg	Cement Injected quantity kg/m	Sand Injected quantity kg	Sand Injected quantity kg/m
GH3	1	7.29 - 12.04	2.80	6.59×10^{-4}	51.5								
	"	4.83 - 12.04	0.9	5.28×10^{-4}	158.5	0	0:1:1	67	3,443	2,208.3	306.3	-	-
	"	9.42 - 14.20	1.3	2.04×10^{-3}	169.7	0	0.97:1:1.3	173.4	10,333	5,100	1066.9	5,253	1099.0
	2	14.83 - 19.61	7.7	7.63×10^{-4}	61.5	1.05	0:1:1	74.6	4,800	2,205	461.3	-	-
	"	14.88 - 18.19	3.5	9.76×10^{-4}	95.7	3.15	0:1:1	63	877	392	118.4	-	-
	3	19.53 - 24.31	8.1	4.66×10^{-4}	38.8	0	0:1:1	94.3	4,200	1,785	373.4	-	-
	"	19.71 - 24.54	2.2	1.51×10^{-3}	117.8	6.0	0.97:1:1.3	114.6	3,020	1,666	625.3	703	145.5
	4	24.05 - 28.88	6.8	4.62×10^{-4}	35.4	0	0:1:1	90.6	3,400	1,554	321.7	-	-
	"	24.05 - 28.88	6.8	5.81×10^{-4}	44.6	8.0	0.97:1:1.3	131.2	4,105	1,805	373.7	348	72.0
	"	24.05 - 28.88	10.8	1.41×10^{-4}	9.1	8.0	0:1:1	125.1	4,650	1,885	390.2	-	-
	5	28.63 - 33.45	12.8	9.05×10^{-6}	0.49								
	6	33.20 - 38.02	12.8	5.45×10^{-6}	0.38								
Total									38,828	18,600.3		6,304	
GH1	1	11.07 - 14.91	7.9	4.15×10^{-4}	44.2	0	0.97:1:1.3	142.2	6,333	2,954	769.3	1,056	275.0
	2	13.82 - 19.61	2.8	6.40×10^{-4}	55.8	4.0	0:1:3	129.0	2,560	638	110.2	-	-
	3	16.28 - 22.86	5.1	5.08×10^{-4}	40.4	8.0	0:1:1	145.6	4,068	1,625	247.0	-	-
	4	22.66 - 27.43	3.6	6.67×10^{-4}	60.8	10.0	0:1:3	74.1	1,444	329	69.0	-	-
Total									14,405	5,546		1,056	
GH2	2	13.59 - 18.69	4.7	7.36×10^{-4}	63.6	6.0	0:1:3	75.9	2,240	578	113.3	-	-
	3	14.12 - 22.86	7.0	3.78×10^{-4}	30.9	8.0	0:1:3	65.1	1,300	284	32.5	-	-
	4	22.86 - 27.43	5.3	8.81×10^{-4}	167.5	10.0	0:1:1	271.4	3,110	984	215.3	-	-
Total									6,650	1,846			
GH4	2	12.67 - 18.64	7.6	5.53×10^{-4}	38.4	3.0	0.97:1:1.3	188.6	6,210	2,735	563.9	596	122.9
	3	20.50 - 23.16	10.0	4.80×10^{-4}	26.7	3.0	0:1:5	64.5	668	140	28.9	-	-
	4	22.70 - 27.45	12.0	3.37×10^{-4}	17.6	4.0	0:1:3	85.6	1,500	347	73.1	-	-
Total									8,378	3,222		596	
GH5	2	12.95 - 18.30	7.7	5.72×10^{-4}	35.8	3.0	0.97:1:1.3	142.2	5,150	2,208	412.7	371	69.3
	3	18.05 - 22.85	8.2	5.93×10^{-4}	39.9	4.0	0.97:1:1.3	168.4	4,550	2,050	427.1	668	139.2
	4	22.90 - 27.70	8.3	5.16×10^{-4}	32.1	5.0	0:1:1	117.5	2,636	929	193.5	-	-
Total									12,336			1,039	
GH6	2	13.65 - 18.50	4.7	6.70×10^{-4}	44.3								
	3	20.15 - 25.00	10.2	2.79×10^{-4}	17.9								
	4	24.15 - 27.45	12.2	4.62×10^{-4}	25.5								

Table I.3.1 LIST OF DATA COLLECTED ON PREVIOUS SOIL SURVEY

1. Map of Dam Site and Vicinity with Location of Soil Survey, Scale: 1/10,000.

2. Report on the Preliminary Soil Investigation for Moragahakanda Reservoir Project, Oct. 1960.

    2-1. Location Map of Auger Holes.
    2-2. Field Notes and Summarized Data on Logs of Auger Holes; 31 holes.
    2-3. Summary of Laboratory Test Results.

3. Report on the Investigation of Earth Materials for the Moragahakanda Reservoir Scheme, 1961.

    3-1. Location Map of Test Pits; 49 pits.
    3-2. Summary of Laboratory Test Results.

4. Report on the Subsoil Investigation of Moragahakanda Reservoir, 1966.

    4-1. Location Map of Auger Holes and Test Pits.
    4-2. Logs of Upstream Test Pits; 4 pits.
    4-3. Logs of Upstream Auger Holes; 29 holes.
    4-4. Logs of Downstream Test Pits; 9 pits.
    4-5. Logs of Downstream Auger Holes; 97 holes.

5. Moragahakanda Dam Axis Soil Investigation, May 1978.

    5-1. Logs of Auger Holes; 25 holes.

6. Moragahakanda Borrow Area Soil Investigation, Jul.-Nov. 1978.

    6-1. Sketch of the Location of Test Pits
    6-2. Logs of Test Pits; 44 pits.
    6-3. Results of Laboratory Tests.
    6-4. Summary of Laboratory Test Results.
    6-5. Results of the Investigation of Riverbed Material.

LABORATORY TESTS CONDUCTED FOR INVESTIGATIONS BY JICA (1978/79)

Table I.3.2 NUMBER OF LABORATORY TESTS CONDUCTED ON IMPERVIOUS MATERIAL

TEST ITEM \ BORROW AREA		A		B		C1		C2		D		E		F		Total	
		Auger Holes	Test Pits	Auger Holes	Test Pits	Auger Holes	Test Pits	Auger Holes	Test Pits	Auger Holes	Test Pits	Auger Holes	Test Pits	Auger Holes	Test Pits	Auger Holes	Test Pits
Specific Gravity		3		3		3		3		3		3		10	3	28	3
Field Moisture Content		3		3		3		3		3		3		10	3	28	3
Gradation		3		3		3		3		3		3		10	3	28	3
Atterberg Limits		3		3		3		3		3		3		10	3	28	3
Compaction	Ec=100%														3		3
	Ec=300%														3		3
Permeability	at O.M.C.														3		3
	Dry at O.M.C.-4%														3		3
	Wet at O.M.C.+4%														3		3
Triaxial Compression (SCU)	at O.M.C.														3		3
	Dry at O.M.C.-4%														3		3
	Wet at O.M.C.+4%														3		3
Consolidation	at O.M.C.														3		3
	Dry at O.M.C.-4%														3		3
	Wet at O.M.C.+4%														3		3

Table I.3.3 NUMBER OF LABORATORY TESTS CONDUCTED ON FILTER MATERIAL AND FINE AGGREGATES

TEST ITEM \ LOCATION	Upstream Riverbed	
Specific Gravity	7	
Gradation	7	
Density (Maximum, Minimum and Relative)	2	
Triaxial (SCU) at 65% Relative Density	2	

Table I.3.4 NUMBER OF LABORATORY TESTS CONDUCTED ON COARSE AGGREGATES AND ROCK-FILL MATERIAL

TEST ITEM \ LOCATION	Q-I	Q-III	DAM AXIS
Specific Gravity	Boring Core 10 Blastings 2	Boring Core 2 Blastings 2	Boring Core 9
Absoption	Boring Core 10	Boring Core 2	Boring Core 9
Compression	Boring Core 10	Boring Core 2	Boring Core 9
Sodium Sulphate Soundness	Boring Core 2	Boring Core 2	
Los Angeles Abrasion	Boring Core 2	Boring Core 2	
Unit Weight	Boring Core 2	Boring Core 2	

* In addition to these, Large Scale Direct Shear Tests are conduoted for Rock Materials.

Table I.3.5 AVAILABILITY OF IMPERVIOUS MATERIAL IN EACH BORROW AREA

Borrow Area		A	B	C1	C2	D	E	F1	F2	F3	G
~~Location~~	Left Bank or Right Bank	L.B.	L.B.	R.B.	R.B.	R.B.	L.B.	L.B.	L.B.	L.B.	L.B.
	Upstream or Downstream	U/S	U/S	U/S	U/S	D/S	D/S	D/S	D/S	D/S	D/S
Aerial Distance to the Dam Site from the Center of Borrow Area (km)		2.0	2.4	1.3	2.8	1.3	1.4	1.4	2.2	2.2	2.8
Elevation (m)	from	150	150	145	150	145	150	175	175	150	140
	to	200	175	155	165	160	175	200	200	175	155
Area (x 1,000 m^2)		1,200	560	450	650	420	850	1,000	520	520	1,100
Possible Depth of Excavation (m)	from	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	to	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
Thickness of Top Soil (m)		0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
Minimum Volume of Impervious Material Available (x 1,000 m^3)		1,800	840	675	975	630	1,300	1,500	780	780	1,650

Note: See II-29, 30 for the location of Borrow Areas

Table I.3.6 SUMMARY OF LABORATORY TEST RESULTS FOR SOILS IN UPSTREAM BORROW AREAS (1961 ID)

Borrow Area	Borrow Area I on Left Bank												Borrow Area II on Right Bank							
Soil Classification	SC Material				GC Material				CH,CL Material				SC Material				GC Material			
	Mean	Max	Min	N	Mean	Max	Min	N	Mean	Max	Min	N	Mean	Max	Min	N	Mean	Max	Min	N
Gradation				20				4				5				9				4
Clay (smaller than 2μ) %	31	46	12		14	17	12		48	54	40		32	42	22		15	18	12	
Silt (5μ - 60 μ) %	14	22	9		9	12	6		25	44	14		18	28	13		7	11	4	
Sand Fine (60μ-200μ) %	17	32	4		7	11	3		13	18	8		24	40	11		5	10	2	
Sand Medium (200μ-600μ) %	18	30	6		10	15	4		9	13	5		18	23	11		7	12	3	
Sand Coarse (600μ-2.0mm) %	10	24	1		12	16	8		4	8	1		6	14	0		11	14	8	
Gravel (2.0mm -19.0mm) %	7	31	1		32	41	23		1	1	0		2	7	0		43	48	33	
Cobbles (larger than 19 mm) %	3	20	0		16	21	13		0	0	0		0	0	0		11	21	1	
Specific Gravity	2.67	2.73	2.60	20	2.65	2.69	2.55	4	2.72	2.76	2.68	5	2.66	2.71	2.58	9	2.68	2.72	2.62	4
Liquid Limit %	39	56	27		36	40	32		46	52	43		35	43	30		40	47	31	
Plastic Limit %	20	33	9	20	20	23	18	4	24	32	21	5	19	22	16	9	22	24	18	4
Plastic Index	19	25	15		16	21	11		22	27	20		16	21	13		18	23	13	
Triaxial Compression Test				6				1				2				3				
Cohesion kg/cm^2	0.70	0.91	0.56		0.56				0.77	1.12	0.42		0.70	1.01	0.56					
Angle of Internal Friction Deg	22	26	17		23				11	12	9		22	26	18					
Compaction				20				3				5				9				3
Optimum Moisture Content %	16.1	21.0	13.5		14.4	15.7	13.6		23.7	29.0	22.0		16.5	18.8	14.7		13.6	13.1	14.0	
Maximum Dry Density g/cm^3	1.80	1.89	1.63		1.84	1.87	1.81		1.57	1.63	1.38		1.78	1.86	1.62		1.92	1.87	1.97	
Penetration Resistance Kg/cm^2	41.8	70.3	28.1		28.1	31.6	19.3		29.5	35.1	24.6		49.2	91.4	31.6		51.3	56.2	49.7	
Permeability x 10^{-8}cm/sec	5.8	9.6	1.0	20	19.3	21.3	8.7	3	2.9	4.8	1.0	4	8.7	19.3	1.9	7	22.2	44.3	8.7	3
Consolidation %	4.8	6.8	3.2	20	4.9	6.0	4.3	3	3.8	4.8	2.8	4	5.4	7.1	4.5	7	3.5	4.3	2.9	3

Note: Figures shown in column N show the number of tests conducted.

Table I.3.7 COMPARISON OF LABORATORY TEST RESULTS OF SOILS IN PREVIOUS AND RECENT INVESTIGATIONS

BORROW AREA	UPSTREAM BORROW AREAS I AND II (1961, IB)									BORROW AREAS A,B,C,D,E AND F (1978/9,JICA)								
Soil Classification	SC Material			GC Material			CH,CL Material			SC Material			GC Material			CH,CL Material		
	Mean	Max.	Min.	Mean	Max.	Min.	Mean	Max.	Min.	Mean	Max.	Min.	Mean	Max.	Min.	Mean	Max.	Min.
Gradation																		
Clay (smaller than 2μ) %	31	46	12	15	18	12	48	54	40	22	34	17	26	35	18	43	61	23
Silt (5μ–60μ) %	16	28	9	8	12	4	25	44	14	15	26	9	4	7	1	18	34	6
Sand Fine (60μ–200μ) %	17	40	4	6	11	2	13	18	8	16	35	9	3	8	0	13	24	6
Sand Medium (200μ–600μ) %	17	30	6	8	15	3	9	13	5	19	28	6	5	13	0	14	20	10
Sand Coarse (600μ–2.0mm) %	9	24	0	11	16	8	4	8	1	18	32	1	22	29	15	10	21	3
Gravel (2.0mm–19.0mm) %	5	31	0	38	48	23	1	1	0	8	20	0	34	55	14	2	10	0
Cobbles (larger than 19mm) %	2	20	0	14	21	1	0	0	0	0	0	0	0	0	0	0	0	0
Specific Gravity	2.67	2.73	2.58	2.66	2.69	2.55	2.72	2.76	2.68	2.65	2.85	2.60	2.64	2.67	2.61	2.67	2.70	2.61
Field Moisture Content %										15.8	20.7	12.6	11.6	12.5	8.1	23.0	31.1	17.1
Liquid Limit %	38	56	27	38	47	31	46	52	43	34.8	51	27	44.3	56	31	48.9	61	28
Plastic Limit %	20	33	9	21	24	18	24	32	21	17.4	24	12	21.9	24	19	23.8	27	17
Plasticity Index	18	25	13	17	21	1	22	27	20	17.4	27	12	22.4	33	12	25.1	33	11
No. of Test Samples	29			8			5			15			6			12		

Table I.3.8 SUMMARY OF LABORATORY TEST RESULTS (JICA-1978)

Sample No.	Borrow Area	Auger Hole/Test Pit No.	Depth Below Ground Level (m)	Soil Classification	Field Moisture Content %	Consistency: Liquid Limit %	Consistency: Plastic Limit %	Consistency: Plasticity Index	Gradation: % Clay (0.002 mm)	Gradation: % Silt (0.002 - 0.06 mm)	Gradation: % Fine Sand (0.06 - 0.2 mm)	Gradation: % Medium Sand (0.2 - 0.6 mm)	Gradation: % Coarse Sand (0.6 - 2.0 mm)	Gradation: % Gravel (2.0 - 19.0 mm)	Gradation: % Cobbles (19.0 mm)	Gradation: % Passing 19.0mm Mesh	Gradation: % Passing 200 Mesh	Specific Gravity of Soil Particles
MK 49	F	T1	1.0-2.0	CH	17.1	50.8	24.2	26.6	42	7	16	20	13	2	0	100	49	2.66
MK 50	F	T2	0.5-1.0	GC	12.5	46.9	24.2	22.7	19	2	1	0	23	55	0	85	21	2.70
MK 51	F	T3	1.0-2.0	SC	20.4	27.4	12.9	14.5	21	10	21	28	11	9	0	94	32	2.66
MK 52	F	I	1.0-2.0	CH	23.4	48.0	24.6	23.4	44	13	11	16	14	2	0	100	58	2.62
MK 53	F	4	0 -1.0	GC	12.3	31.2	18.9	12.3	18	7	8	13	22	32	0	100	25	2.66
MK 54	F	5	1.0-1.5	SC	16.6	36.0	16.9	19.1	17	12	11	18	32	10	0	100	30	2.85
MK 55	F	6	1.0-2.0	CH	31.0	61.0	32.3	28.7	52	25	8	10	4	1	0	100	78	2.66
MK 56	F	10	1.0-2.0	CL	28.3				38	30	12	14	6	0	0	100	70	2.66
MK 57	F	13	1.0-2.0	SC	17.5	29.1	16.2	12.9	18	20	22	25	13	2	0	100	38	2.61
MK 58	F	17	1.0-2.0	SC	12.4	42.9	21.9	21.0	25	6	5	10	29	25	0	100	32	2.75
MK 59	F	22	0 -1.0	SC	17.8	37.1	21.6	15.5	22	26	12	19	17	4	0	100	49	2.60
MK 60	F	24	0 -1.0	SC	20.6	31.7	15.2	16.5	18	18	19	19	16	10	0	100	38	2.64
MK 61	F	25	0 -1.0	SC	27.7	46.6	24.2	22.4	30	34	19	12	4	1	0	100	71	2.61
MK 62	F	26	1.0-2.0	CH	23.8	59.8	26.8	33.0	53	12	9	15	10	1	0	100	65	2.61
MK 63	A	1	1.0-2.0	SC	16.5	29.8	14.9	14.9	25	20	35	19	1	0	0	100	47	2.63
MK 64	A	2	0 -1.0	CL	21.1	44.8	21.9	22.9	35	22	17	13	10	3	0	100	63	2.63
MK 65	E	1	1.0-2.0	SC	15.8	35.2	17.4	17.8	27	18	17	24	12	2	0	100	46	2.64
MK 66	E	2	1.0-2.0	SC	10.9	27.2	14.8	12.4	19	10	19	26	19	7	0	100	30	2.64
MK 67	E	3	1.0-2.0	SC	12.5	27.2	12.3	14.9	20	11	2[illegible]	28	18	2	0	100	31	2.65
MK 68	A	3	0 -2.0	CH	18.9				47	17	18	14	3	1	0	100	65	2.64
MK 69	B	1	0 -2.0	CH	21.9	49.7	22.6	27.1	61	6	7	12	12	2	0	100	67	2.62
MK70A	B	2	0-0.75	SC	18.3	40.5	20.1	20.4	29	19	12	14	15	11	0	100	50	2.64
MK70B	B	2	0.75-2.0	SC	18.8	28.1	17.3	10.8	23	29	24	17	6	1	0	100	61	2.67
MK 71	B	3	0-1.25	SC	14.2	37.9	19.3	18.6	25	11	10	14	24	16	0	100	37	2.71
MK 72	C1	1	0-0.75	SC	14.2				35	5	3	6	15	36	0	100	40	2.61
MK 73	C1	2	0 -2.0	SC	15.3	34.0	17.1	16.9	22	11	13	14	24	16	0	100	34	2.68
MK 74	C1	3	0 -1.0	SC	13.9	50.7	24.1	26.6	34	9	1	6	30	20	0	100	40	2.62
MK 75	D	1	0 -1.5	SC	10.2	44.0	21.6	22.4	29	5	2	0	20	44	0	100	35	2.67
MK 76	D	2	0 -1.5	SC	14.5	39.2	16.5	22.7	31	17	13	21	17	7	0	100	43	2.60
MK 77	D	3	0 -0.6	SC	12.8	38.8	22.2	16.6	32	14	9	16	18	11	0	100	46	2.61
MK 78	C2	1	0 -1.5	CL	20.0	50.6	21.7	28.9	42	8	10	14	16	10	0	100	50	2.66
MK 79	C2	2	0 -1.5	GC	8.1	56.4	23.1	33.3	30	1	0	1	23	45	0	100	31	2.61
MK 80	C2	3	0 -1.5	CL	23.0	50.0	22.6	27.4	47	7	6	14	21	5	0	100	54	2.63

Notes: 1) Samples MK 49 - 51 are extracted from test pits and all remaining samples are extracted from auger holes.
2) See also Table 2.5. for the results of mechanical tests conducted with samples MK 49 - 51.

Table I.3.9 SUMMARY OF MECHANICAL TESTS OF IMPERVOUS MATERIAL

Sample No.		MK 49 (CH.CL)					MK 50 (GC)					MK 51 (SC)				
Compaction Std.	Moisture Content %	16.55	17.65	18.18	19.39	20.40	12.17	12.36	14.47	15.24	15.80	11.79	12.90	13.71	14.80	16.64
	Dry Density (g/cm^3)	1.695	1.707	1.712	1.678	1.640	1.894	1.903	1.872	1.839	1.809	1.818	1.847	1.858	1.853	1.780
Compaction 300%	Moisture Content %	13.70	14.70	15.50	16.51	17.50	9.30	10.46	11.10	13.00	15.00	9.70	10.90	11.50	12.10	13.30
	Dry Density (g/cm^3)	1.813	1.826	1.852	1.810	1.792	1.970	1.997	2.020	1.948	1.776	1.953	1.967	1.981	1.967	1.921

Sample No.		MK 49 (CH.CL)			MK 50 (GC)			MK 51 (SC)		
Triaxial SCU Test	Moisture Content %	13.82	18.40	22.03	9.12	12.70	16.84	9.64	14.33	17.67
	Cohesion (kg/cm^2)	0.18	0.42	0.11	0.21	0.70	0.14	0.18	0.35	0.17
	Angle of Internal Friction (deg)	34°18'	22°23'	15°10'	33°6'	30°35'	43°50'	30°56'	36°1'	27°45'
(1)	Moisture Content %	15.20	18.30	22.50	9.70	13.10	17.00	9.90	14.90	18.40
	Rate of Percolation $\times 10^{-8}$ (cm/sec)	1.02	1.03	-	7.15	0.65	0.62	1.01	0.93	0.74
	Consolidation %	3.00	3.55	8.00	2.60	3.62	7.53	4.05	6.68	6.22
(2)	Moisture Content %	14.23	17.71	21.11	9.33	12.41	16.73	9.70	13.90	18.50
	Coef. of Permeability $\times 10^{-6}$ (cm/sec)	1.30	1.15	1.20	1.90	0.38	0.91	0.76	0.61	0.71
Max Dry Density (g/cm^3)		1.712 (at O.M.C.= 18.26%)			1.922 (at O.M.C.= 13.00%)			1.874 (at O.M.C.= 14.00%)		

Remarks: I. Compaction tests are conducted for Eo=100% and 300% (Hammer weight: 5 1/2 lb, Hammer Drop: 12" Number of Layers: 3, Number of Blows per Layer: 25 for Eo=100% and 75 for Eo=300%)

II. Triaxial tests are conducted under Saturated Consolidated Undrained Condition (SCU)

III. (1) Results of Consolidation and Percolation Tests (Consolidated Load: 1.397kg/cm^2)

IV. (2) Results of Consolidation Tests (Consolidated Load : 1.40 kg.cm^2)

Table I.3.10 SUMMARY OF LABORATORY TESTS ON RIVERBED MATERIAL AND ROCK BLASTINGS

Sample No.		MKF 28	MKF 23	MKF 24	MKF 25	MKF 26	MKF 27	Mean
Location of Sampling (Distance from Dam Axis)		Q-I	1 km U/S	2 km U/S	3 km U/S	4 km U/S	5 km U/S	Value
Type of Material		Blastings	Riverbed Material					
Clay (smaller than 2 μ)	%	13	0	0	0	0	0	0
Silt (5μ – 60 μ)	%	17	0	0	0	0	0	0
Sand Fine (60μ–200μ)	%	14	1	2	1	1	1	1.2
Sand Medium (200μ – 600μ)	%	19	8	11	27	17	10	14.6
Sand Coarse (600μ – 2000μ)	%	27	71	67	69	75	63	69.0
Gravel (larger than 2000μ)	%	10	20	20	3	7	25	15.0
Percentage Passing 1/4" mesh		100	100	97	100	99	93	97.8
Persentage Passing 200 mesh		33	0	0	0	0	0	0
Uniformity Coefficient U_o			2.55	3.13	2.32	2.62	2.56	2.64
Specific Gravity Dry Basis			2.58	2.61	2.55	2.60	2.59	2.55
Specific Gravity Saturated Surface Dry Basis			2.62	2.64	2.58	2.63	2.61	2.62
Absorption	%		1.33	1.28	1.33	1.16	0.92	1.20
Fineness Modulus		1.77	3.52	3.58	2.89	3.28	3.73	3.40

Table I.3.11 COMPARISON OF THE STANDARD RANGE OF GRAIN SIZE DISTRIBUTION RECOMMENDED FOR FINE AGGREGATES IN DAM CONCRETE WITH THE ACTUAL GRADATION OF RIVER SAND AND ROCK BLASTINGS

Particle Size (mm)	Recommended Standard Range by Weight	River Sand (Amban Ganga)		Rock Blastings	
		Actual Range by Weight (%)	Remarks	Actual Range by Weight (%)	Remarks
10.0 - 5.0	0 - 8	0 - 10	Satisfactory	1	Satisfactory
5.0 - 2.5	5 - 25	1 - 22	Satisfactory	5	Satisfactory
2.5 - 1.2	10 - 25	10 - 28	Satisfactory	13	Satisfactory
1.2 - 0.6	10 - 30	27 - 38	Realtively high	16	Satisfactory
0.6 - 0.3	15 - 30	9 - 26	Satisfactory	15	Relatively Low
0.3 - 0.15	12 - 20	1 - 12	Relatively Low	12	Relatively Low
0.15 - 0	3 - 10	0 - 1	Relatively Low	38	Excessive

Table I.3.12 LABORATORY TEST RESULTS OF ROCK SAMPLES FROM BORING CORE

SAMPLE No.	LOCATION OF EXTRACTION		ROCK TYPE	SPECIFIC GRAVITY		ABSORPTION	COMPRESSIVE STRENGTH
	SITE	DEPTH (M)		S.S.D	DRY	(%)	(Kg/cm2)
DM 29-83	Quarry I	14.5	Weathered	2.817	2.807	0.374	- *
DM 29-85	"	14.5	Gneiss	2.832	2.816	0.570	- *
DM 29-127	"	19.0	Fresh Gneiss	-	-	-	1832
DM 29-128	"	19.2	"	2.914	2.912	0.064	1492
DM 29-135	"	20.8	"	-	-	-	1542
DM 29-139	"	20.9	"	2.997	2.986	0.377	1200
DM 29-155	"	23.3	"	3.164	3.162	0.080	1813
DM 29-162	"	24.5	"	2.803	2.797	0.202	1536
DM 29-162	"	24.5	"	3.353	3.349	0.134	1536
DM 29-176	"	26.1	"	-	-	-	1818
DM 29-177	"	26.2	"	-	-	-	1111
DM 29-183	"	28.8	Calc Gneiss	2.834	2.827	0.251	1124
DM 29-220	"	37.7	"	2.847	2.842	0.185	661
DM 29-254	"	45.3	Fresh Limestone	2.880	2.874	0.233	668
DM 37-16	Quarry II	2.6	Calc Gneiss	2.914	2.913	0.038	773
DM 37-37	"	6.8	Fresh Limestone	2.922	2.919	0.107	842
DM 6-2	Dam Axis	11.7	Fresh Gneiss	3.055	3.054	0.037	1996
DM 7-18	"	11.5	"	3.063	3.061	0.021	1847
DM 16-77	"	15.7	"	2.917	2.916	0.045	1268
DM 16-86	"	18.2	"	2.927	2.926	0.043	1792
DM 19-28	"	13.1	Crystalline Limestone	2.896	2.891	0.149	473
DM 20-23	"	7.0	Gneiss	3.180	3.106	0.052	1889
DM 21-25	"	15.8	Calc Gneiss	2.915	2.911	0.163	1132

Note: * Unable to determine the compressive strength due to cracks in the rock samples.

Table I.3.13 SUMMARY OF THE TEST RESULTS OF QUARRY MATERIALS - (ROCK BLASTINGS)

Quarry No.																
Laboratory No.			MSQA-1					MSQB-1					MSQB-2			
Rock Classification			Basic Charnockite					Weathered Limestone					Weathered Limestone			
Abrasion Test	After 100 rev		7.40 %					21.43%					22.53%			
	After 500 rev		36.07 %					67.80%					69.15%			
Aggregate Size	Sieve Size Passing	Retained	Soundness % Loss	Bulk Density (t/m³)	Absorption %	Specific Gravity SSD	Specific Gravity DRY	Soundness % Loss	Bulk Density (t/m³)	Absorption %	Specific Gravity SSD	Specific Gravity DRY	Soundness % Loss	Bulk Density (t/m³)	Absorption %	Specific Gravity SSD	Specific Gravity DRY
Coarse Aggregates	3"	1 1/2"	0.43	1.584	0.284	3.004	3.069	1.33	1.469	0.247	2.900	2.899	1.83	1.505	0.239	2.919	2.912
	1 1/2"	3/4"	1.12	1.607	0.378	2.995	3.058	8.97	1.531	0.356	2.930	2.920	6.99	1.536	0.530	2.890	2.870
	3/4"	3/8"	2.56	1.531	0.102	3.052	3.049	3.70	1.502	0.20	2.820	2.765	11.24	1.565	0.620	2.907	2.890
	3/8"	US-4	4.70	1.439	0.315	3.043	3.034	15.80	1.568	0.51	2.911	2.896	13.90	1.567	0.504	2.918	2.903
Fine Aggregates	US-4	US-8	3.2					6.5					14.9				
	US-8	US-16	6.5					7.2					11.0				
	US-16	US-30	4.5					4.5					6.3				
	US-30	US-50	4.3					6.2					6.7				
	US-4	US-200		1.823	1.797	3.106	3.053		1.717	0.49	2.944	2.929		1.693	2.04	2.810	2.760

Remarks:

1) Los Angles Abrasion Test was conducted as per Designation 21 of U.S.B.R. Concrete Manual using Grading A.

2) Soundness Test was conducted as per Designation 19 of U.S.B.R. Concrete Manual by the Sodium Sulphate Method.

FIGURES

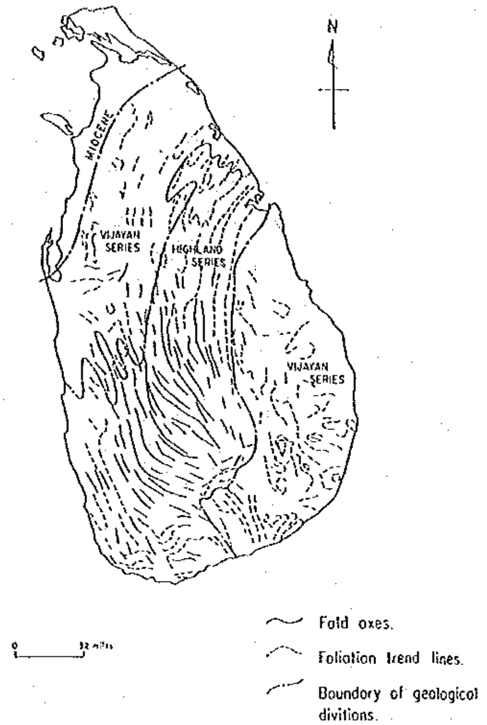

Fig I.1-1 The Main Georogical Divisions and Simplified Geological Structures

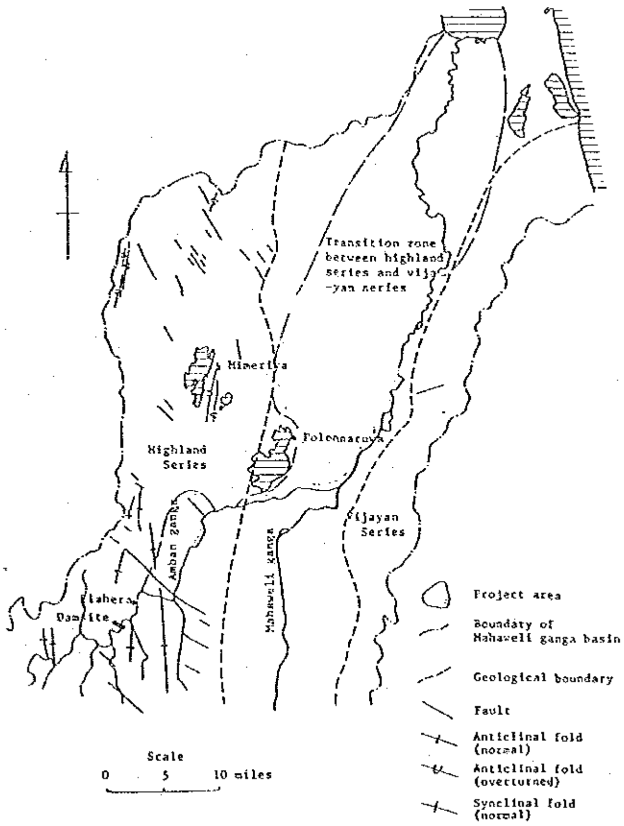

Fig I.1-2 General Geology of the Project Area

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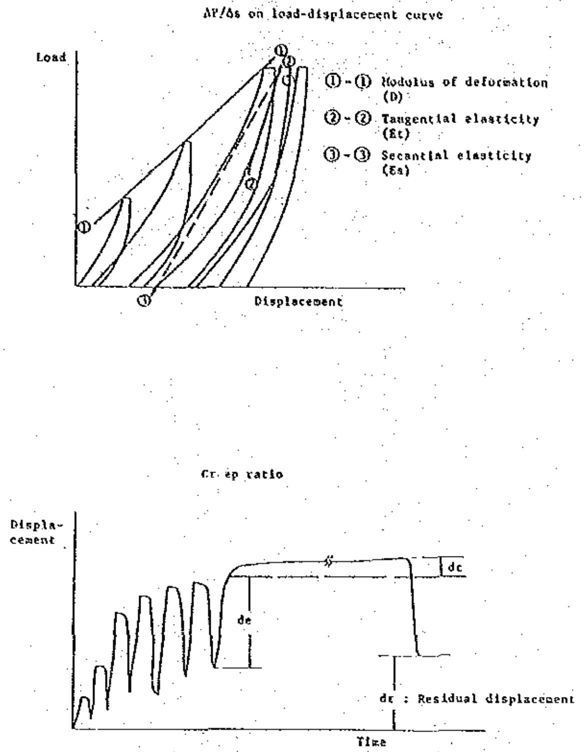

Fig I.2-1 ⊿P/⊿s on Load-displacement Curve and Creep Ratio

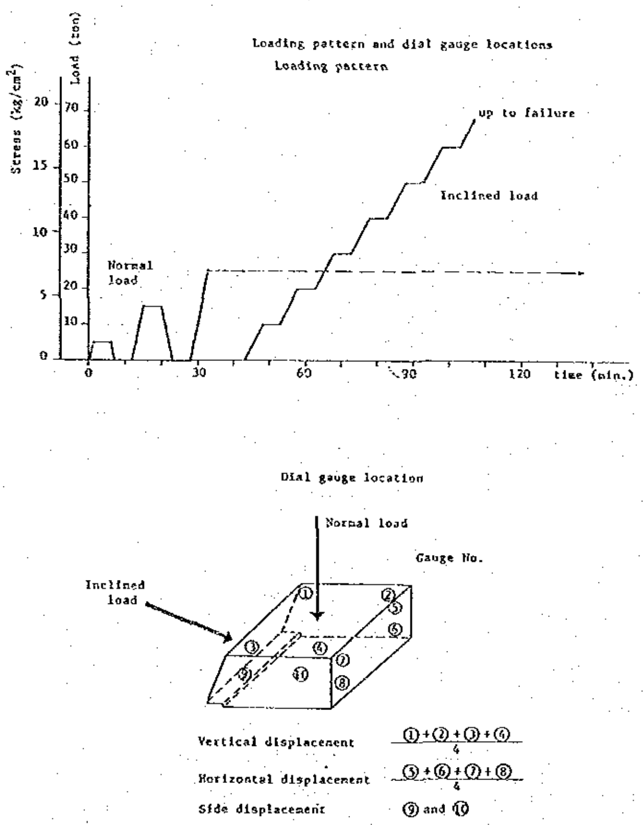

Fig I.2-2 Loading Pattern and Dial Gauge Locations

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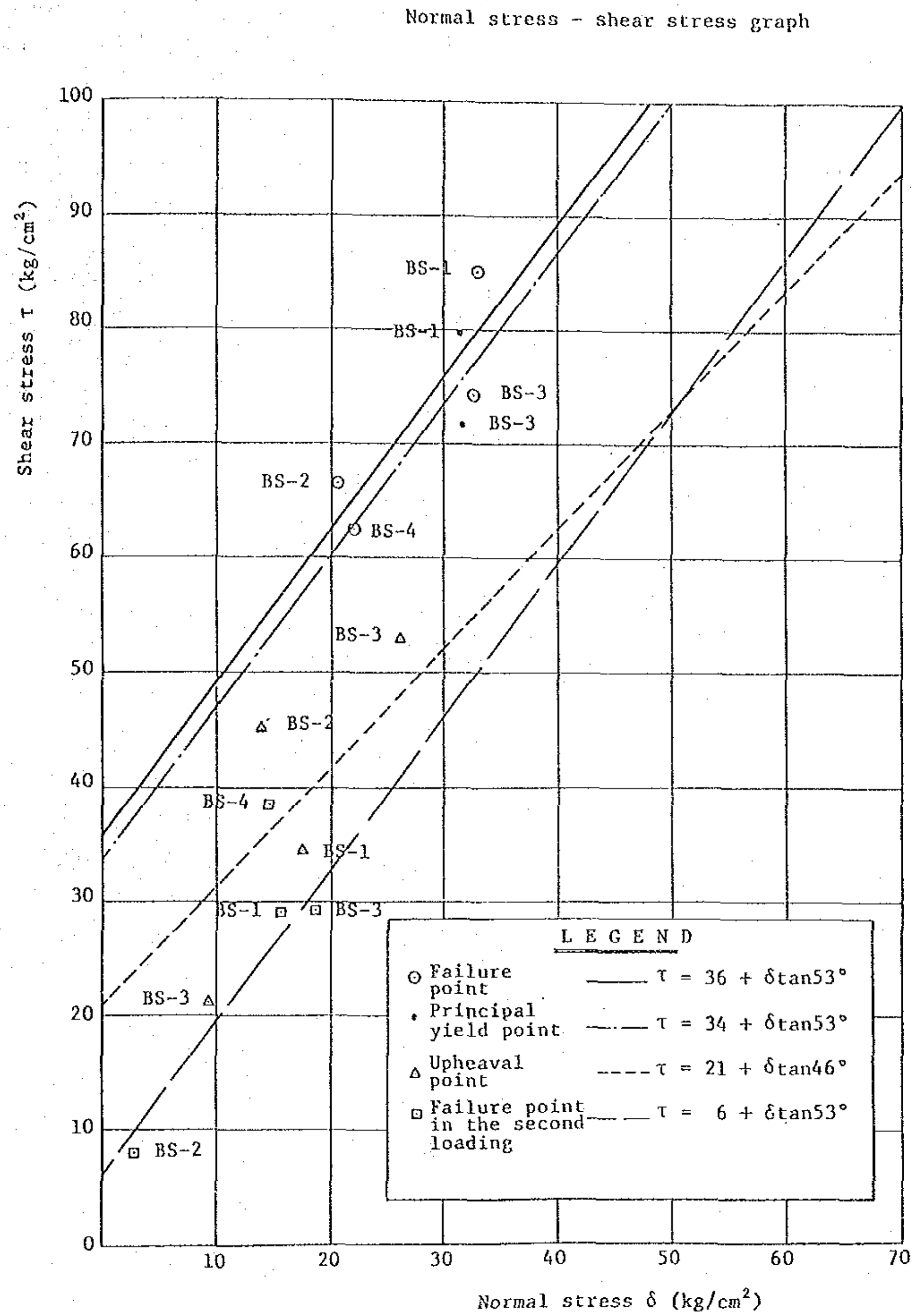

Fig I.2-3 Shear Strength

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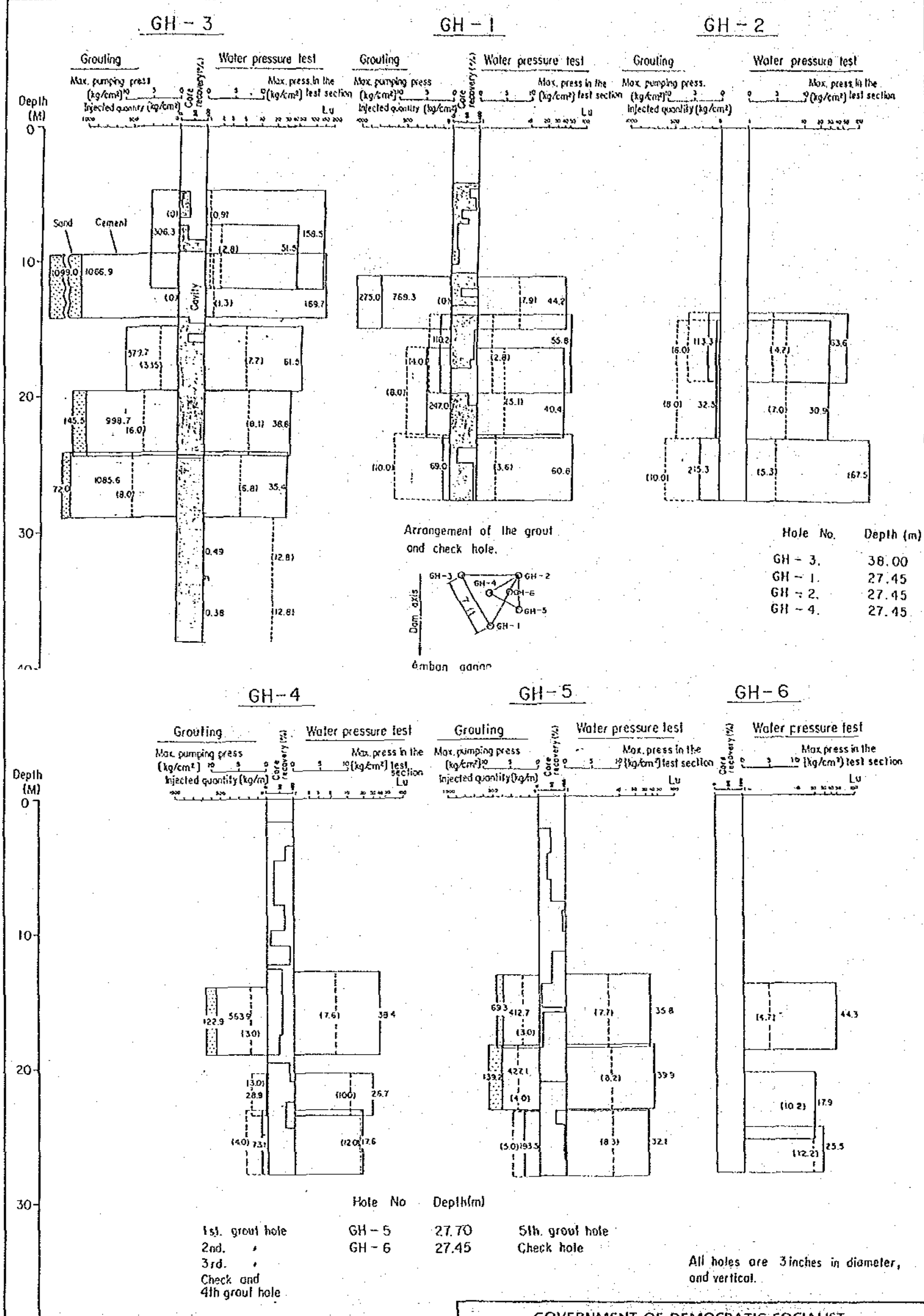

Fig I.2-4 Results of the Grouting Tests

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Fig I.2-5 Geological Map of the Reservoir Area

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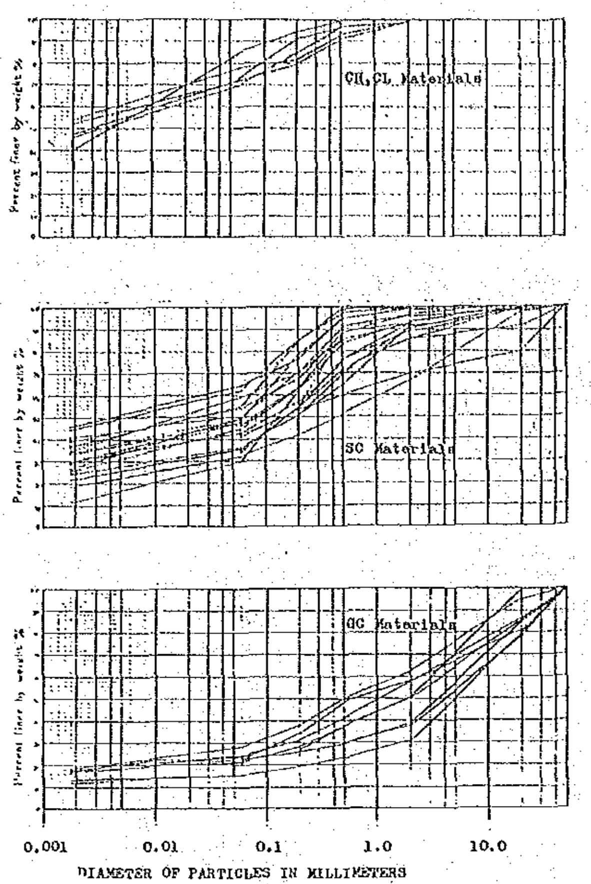

Fig I.3-1 Grain Size Distribution Curves of Soilsin Upstream Borow Areas

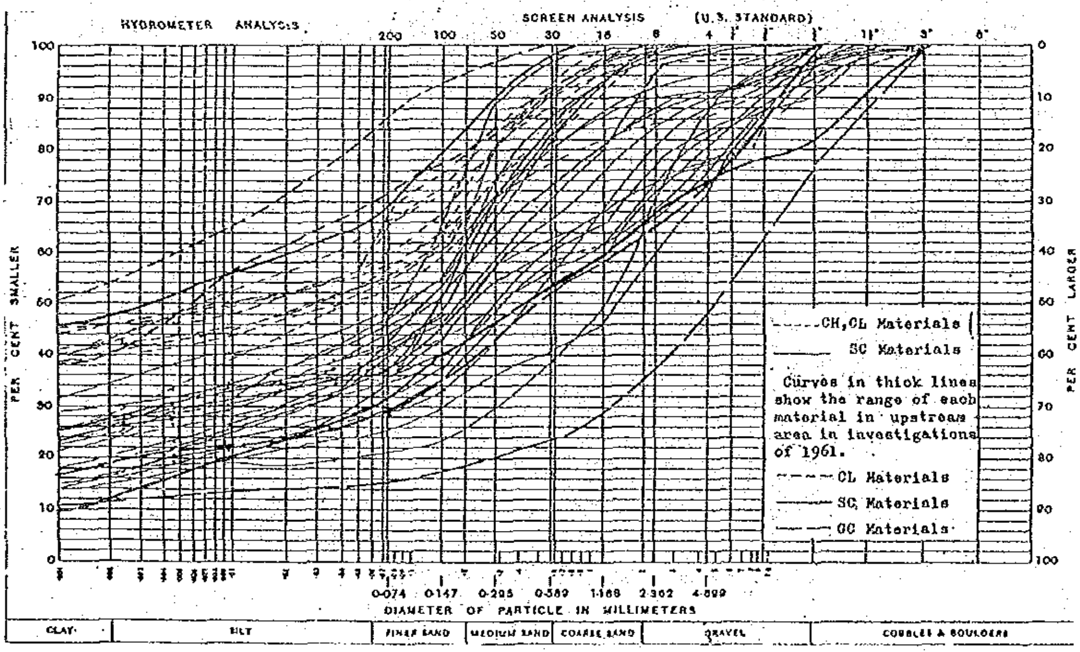

Fig I.3-2 Grain Size Distribution Curves of Soilsin Borow Area E

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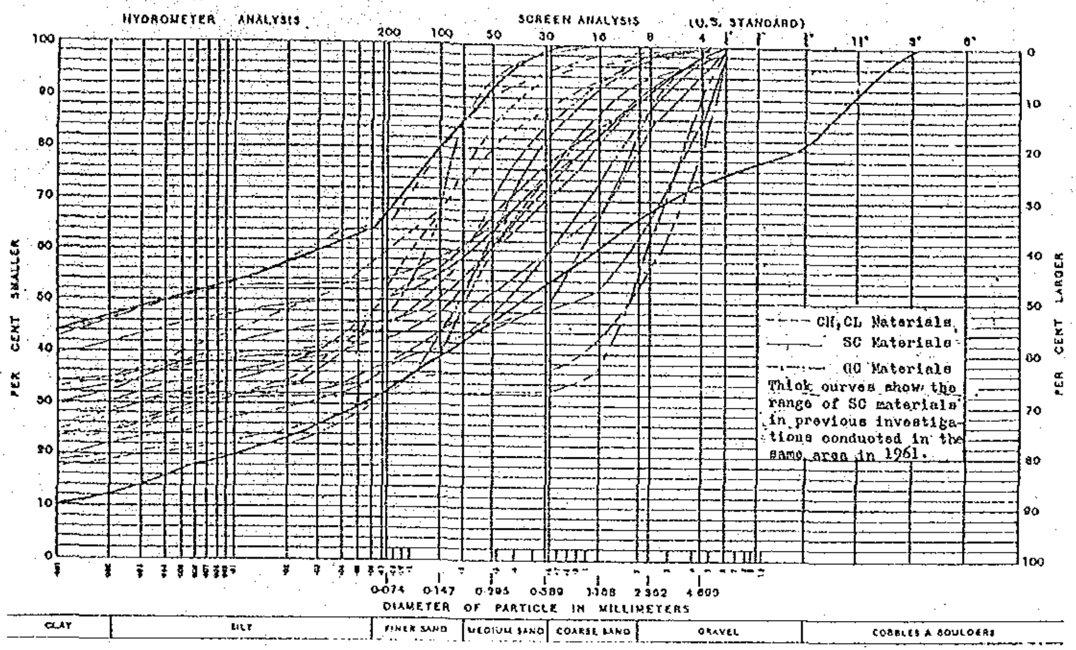

Fig I.3-3 Grain Size Distribution Curves of Soils in Borow Area A,B,C1,C2,D and E

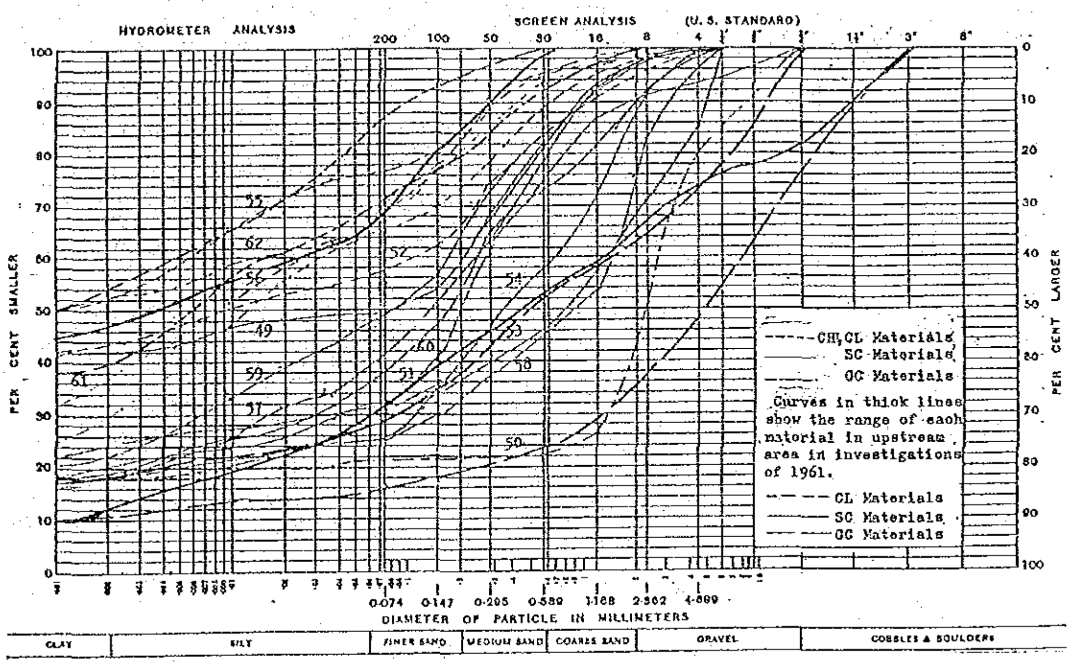

Fig I.3-4 Grain Size Distribution Curves of Soils in Borow Area F

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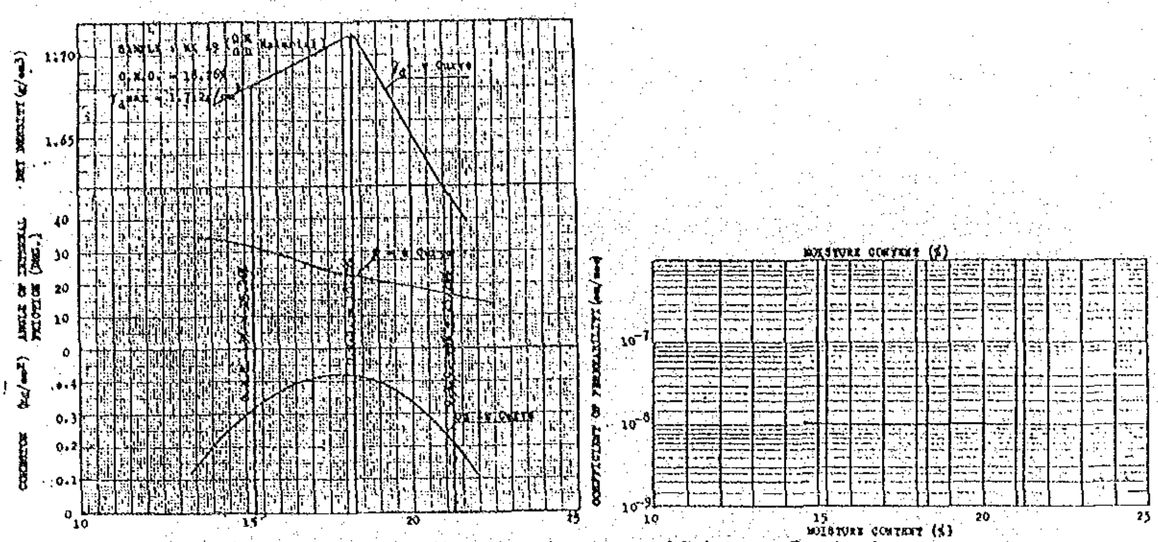
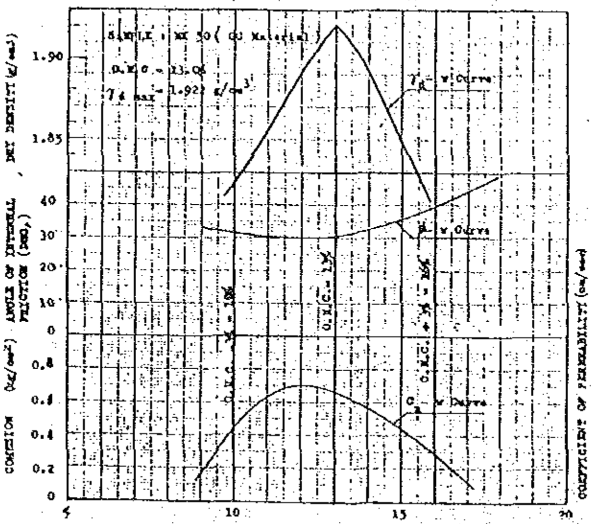


Fig I.3-5(1/3) Reration between Moisture Content and Dry Density,Cohesion Angle of Intrenal Friction and Coefficient of Permeability



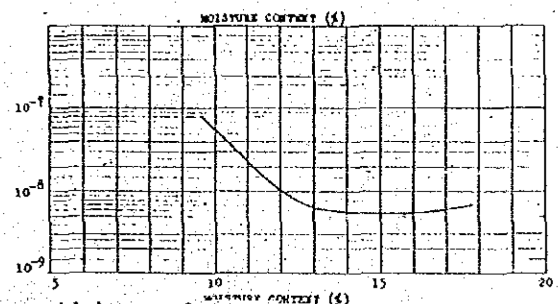


Fig I.3-5(2/3) Reration between Moisture Content and Dry Density,Cohesion Angle of Intrenal Friction and Coefficient of Permeability

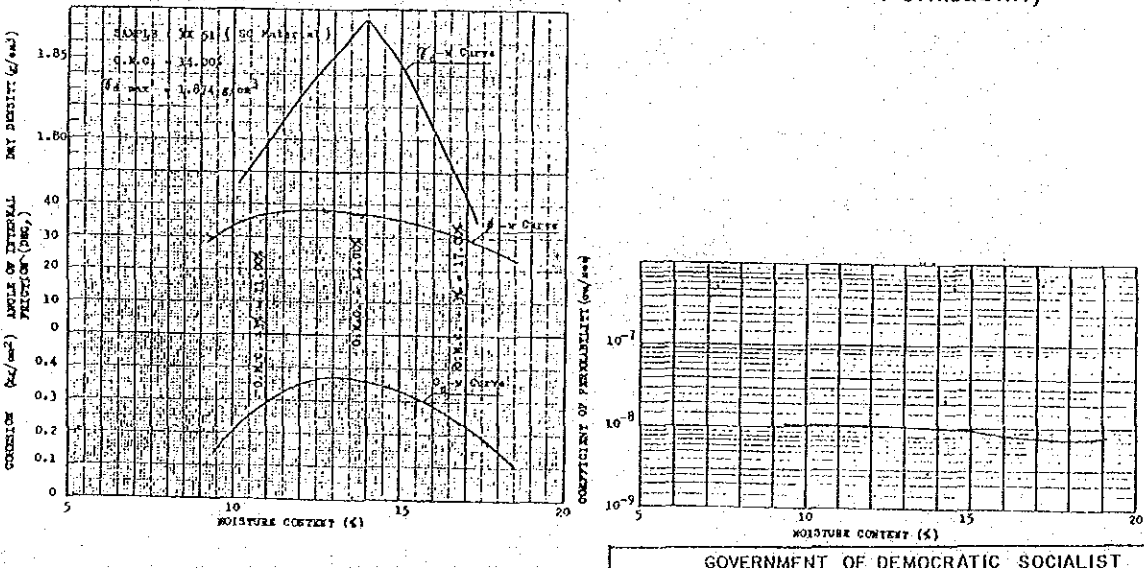


Fig I.3-5(3/3) Reration between Moisture Content and Dry Density,Cohesion Angle of Intrenal Friction and Coefficient of Permeability

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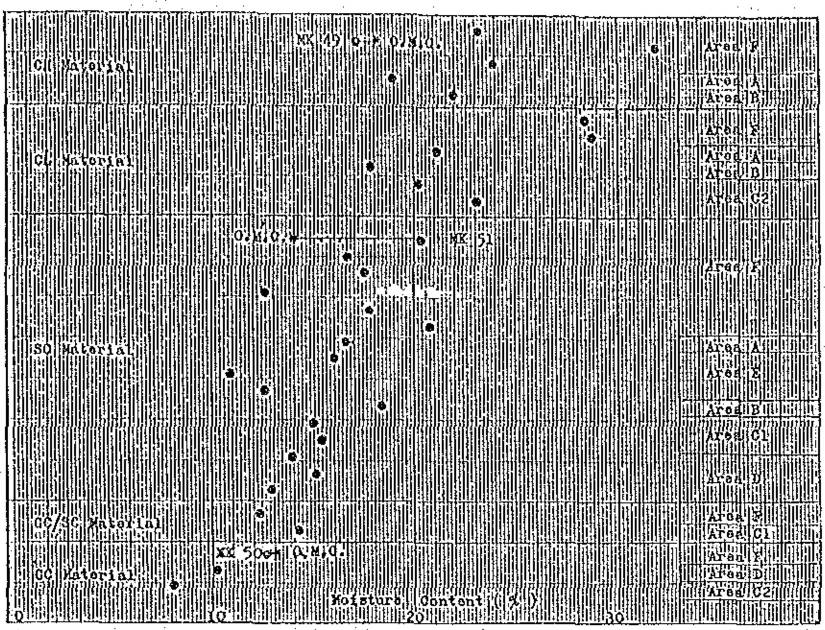

Fig I.3-6 Reration between Field Moisture Content and Optimum Moisture Content

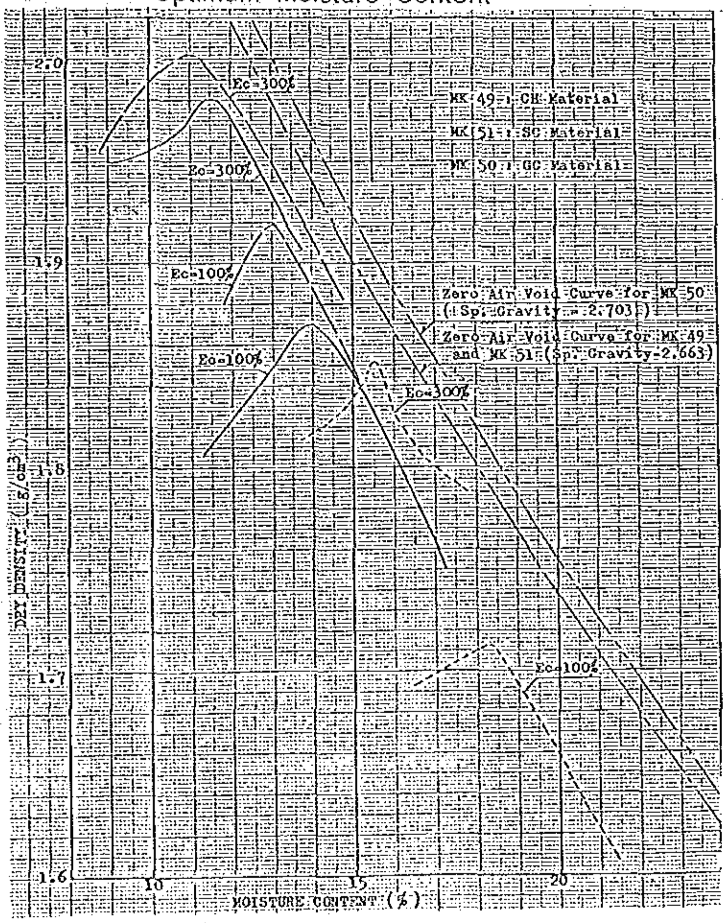

Fig I.3-7 Results of Compaction Test

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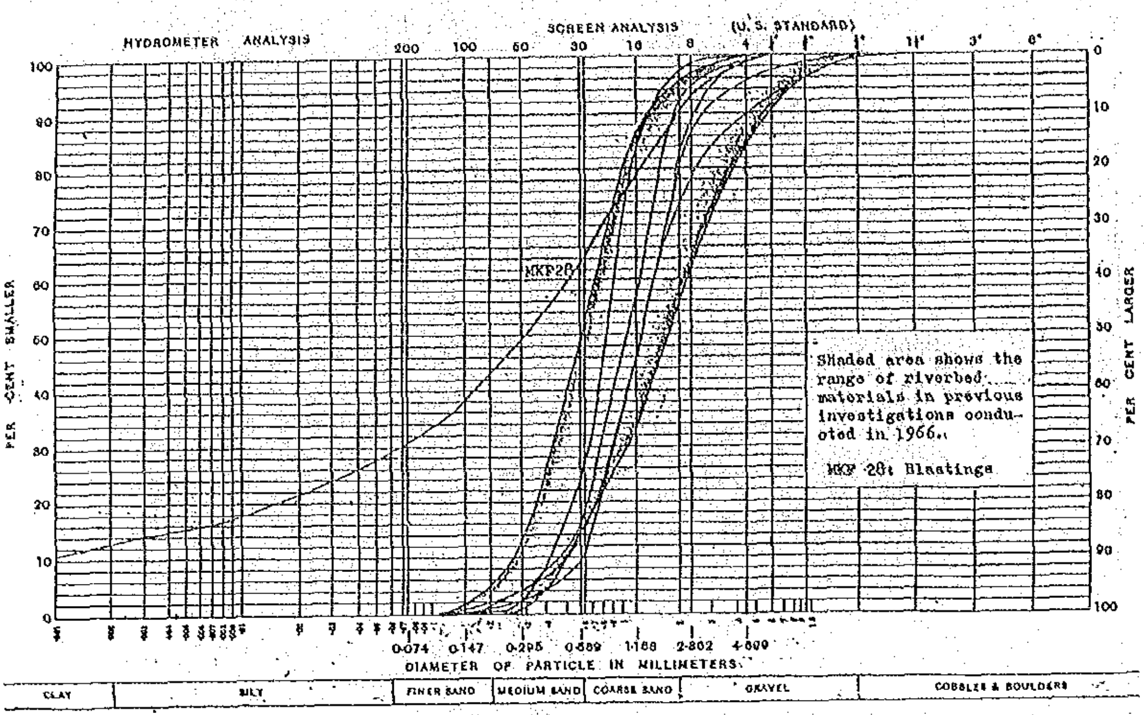

Fig I.3-8 Grain Size Distribution Curves of Riverbed Materials and Rock Blasting

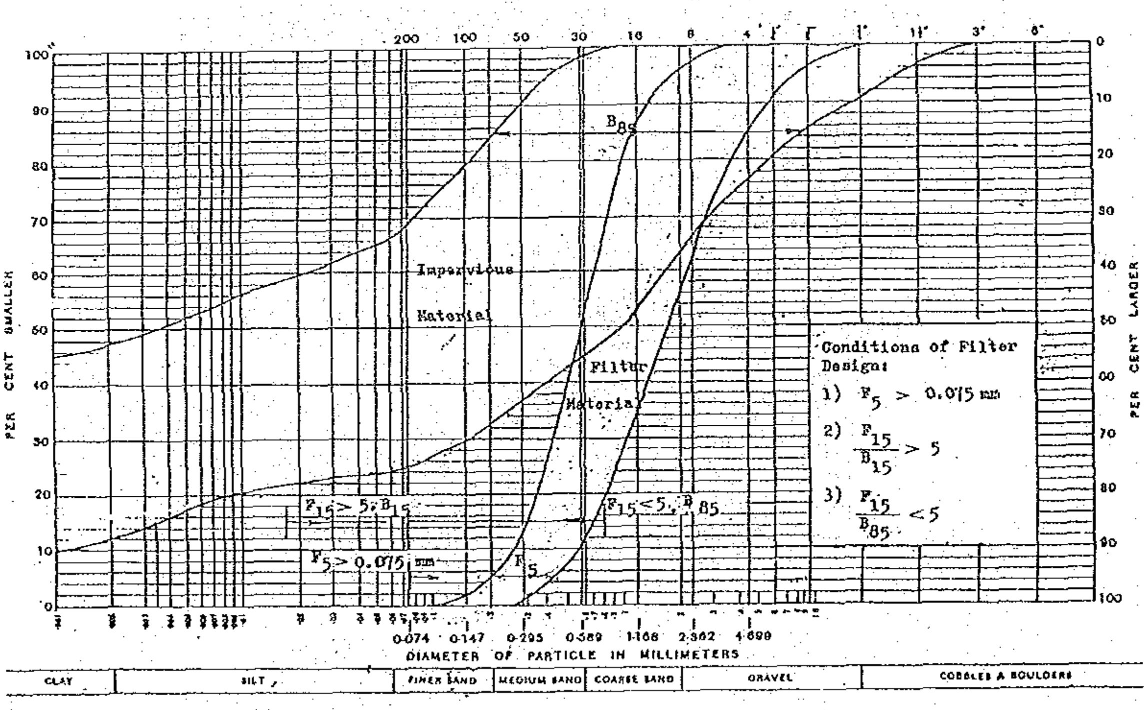

Fig I.3-9 Conditions for the Design of Filter

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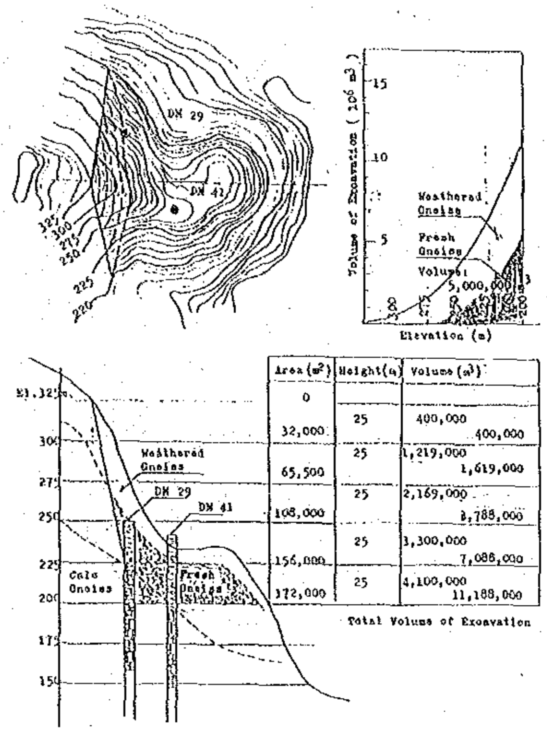
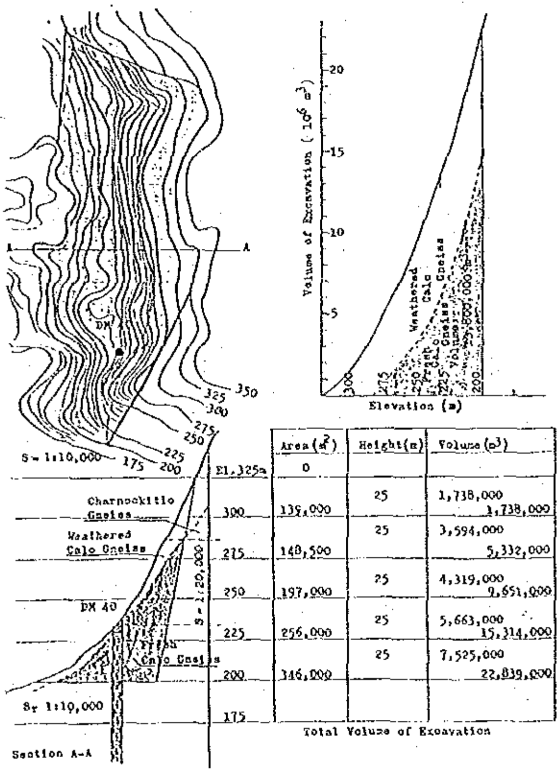


Area (m^2)	Height (m)	Volume (m^3)	
0			
32,000	25	400,000	400,000
65,500	25	1,219,000	1,619,000
108,000	25	2,169,000	3,788,000
156,000	25	3,300,000	7,088,000
172,000	25	4,100,000	11,188,000

Fig I.3-10 Availability of Rock Materials (Quarry I)



Area (m^2)	Height (m)	Volume (m^3)	
0			
139,000	25	1,738,000	1,738,000
148,500	25	3,594,000	5,332,000
197,000	25	4,319,000	9,651,000
256,000	25	5,663,000	15,314,000
346,000	25	7,525,000	22,839,000

Fig I.3-11 Availability of Rock Materials (Quarry II)

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ANNEX - J

OPTIMUM SCALE OF DAM AND POWER STATION

ANNEX - J

OPTIMUM SCALE OF DAM AND POWER STATION

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ANNEX-J OPTIMUM SCALE OF DAM AND POWER STATION

J.1 OPTIMIZATION CRITERIA

With the water released for irrigation water requirement and head created by the dam, certain power is generated. Further if more effective reservoir capacity is provided by a higher dam, more power will be produced due to higher head and more discharge available. In such case, more cost will be borne naturally. A study aims to find out the optimum scale of dam incorporating power generation.

The optimization procedure is mentioned as follows:

(1) The optimization of development scale of dam and power station were principally based on the net benefit (B-C) maximization criteria under 10 per cent discount rate.

(2) In order to simplify the study, the net incremental benefit is estimated for comparison. The incremental cost, which consists of incremental dam cost and cost of power facilities including indirect cost, and the incremental benefit borne by the power generation.

Net benefit (B-C) = Incremental benefit at present worth
- Economic incremental cost at present worth

(3) The estimation of the construction cost for the comparative study was made applying the unit prices as mentioned in ANNEX-L and the work quantities which were calculated based on the preliminery design. They are expressed in February 1988 price leval.

(4) The economical incremental cost at present worth was calculated, assuming economic life of the Project for 50 years, discount rate at 10 per cent per annum and operation and maintenance cost at 0.5 per cent of the total construction cost for civil work and hydromechanical works of dam and 2.5 per cent of generating equipment and transmission line. The economic cost was assumed to be 85 per cent of the financial cost.

(5) The estimation of the incremental benefit at present worth was made based on the unit power benefit as mentioned in Sub-section G.2.2, and the dependable peak power and firm and secondary energy, which were calculated based on the result of reservoir operation study as stated in Sub-section G.2.3.

J.2 SELECTION OF ALTANATIVE SCALE OF DEVELOPMENT

There are the following two alternatives for selection of development scale: 1) Reservoir capacity alternatives and 2) Installed capacity alternatives of powerplant.

As for the reservoir capacity alternatives, the following three (3) cases were selected:

Case	H.W.L (EL.-m)	Operat.W.L (EL.-m)	L.W.L (EL.-m)
1	188.0	154.0	154.0
2	195.0	174.4	170.0
3	200.0	182.8	175.0

Prior to the selection of installed capacity alternatives, the firm power potential was worked out for each scale of the reservoir according to the operation rules as mentioned in Sub-section G.2.3. The results are as follows:

Case	H.W.L (EL.m)	Firm power Potential (kW)
1	188.0	0
2	195.0	6,000
3	200.0	10,000

The selection of installed capacity alternatives was made according to the following concepts :

(1) One concept is to limit the maximum discharge capacity of turbine at 56.6 m^3/s, which is the maximum intake capacity of the Elahera diversion weir, in order to eliminate the construction cost of afterbay.

(2) The other concept is to install a larger capacity of powerplant than the above, with afterbay construction to meet the daily peak demand. The peaking hours in the CEB system is around 3 to 4 hours and the plant factor ranges between 0.125 and 0.167.

In addition to the above concepts, the following design conditions and criteria of powerplant were applied to obtain the installed capacity :

(1) In case of the alternatives based on the former concept, rated water level of the reservoir is set at one-third level of the total drawdown from the high water level. In this case, turbine can discharge water amounting to 56.6 m^3/s at the low water level. One unit installation of powerplant is taken.

(2) In case of the alternatives based on the latter concept, rated water level of the reservoir is also placed at one- third level of the total drawdown from the high water level. In this case, two units of powerplant are installed.

Based on the above concepts and design criteia, the following five cases of the alternative are obtained :

Case	H.W.L (EL.-m)	Installed Capacity (MW)	Max. Plant Discharge (m^3/s)
1-1	188.0	0	0
1-2	188.0	23	56.6
2-1	195.0	26	56.6
2-2	195.0	42	91.7
3-1	200.0	28	56.6
3-2	200.0	69	138.1

The principal features of the above alternatives are shown in Table J.1.1 and illustrated in Fig. J.2-1 for Cases 2-1 and 2-2, and Fig. J.2-2 for case 1-1 and 1-2.

J.3 ELAHERA REGULATION DAM

As stated in the previous section, the Cases 2-2 and 3-2 require construction of an afterbay to regulate the discharge released from the power station during the peaking time to 56.6 m^3/s corresponding to the maximum capacity of the existing Elahera - Minneriya Canal. The afterbay weir will be constructed immediately downstream from the existing Elahera diversion weir.

The outlines of the afterbay weir are as follows :

	Case 2-2	Case 3-3
Regulating Pond:		
F.W.L.(EL.-m)	146.7	147.9
H.W.L.(EL.-m)	142.4	143.6
L.W.L.(EL.-m)	138.7	138.7
Net storage capacity (10^6 m^3)	1.87	2.89
Weir:		
Crest EL.(m)	150.2	151.4
Crest length (m)	1,100	1,250
Max. height (m)	20.1	21.3
Type of spillway	Non-gated,free overflow type	
Design flood (m^3/s)	3,400	
Irrigation intake	Overflow type with 1-flap gate, 20 m wide x 3 m high	
Construction cost (US$$10^6$)	10.81	13.5

A sketch of the afterbay weir is presented in Fig.J.3-1.

J.4 COMPARATIVE STUDY

Based on the above mentioned conditions and criteria, the economic costs and benefits were estimated and their relevant outputs are shown in Tables J.4.1 to J.4.3.

As seen in the Table J.4.1, it was resulted that the optimum development scale of dam and power station was given to the Case 2-1, in which the highest net incremental benefit would be obtained.

The major features are shown below:

Reservoir				
H.W.L		EL.195.0m		
L.W.L		EL.170.0m		
Operation W.L		EL.174.4m		
Dam & Spillway		Main	1st Saddle	2nd Saddle
Crest EL.		199.0m	197.5m	199.0m
Crest length		490.0m	396.0m	490.0m
Max.height		72.0m	62.5m	42.0m
Volume		$2,430x10^3m^3$	$376x10^3m^3$	$430x10^3$
Spillway		Overflow type and stilling basin		
Design flood	inflow	4,650 m^3/s		
	outflow	3,400 m^3/s		
Penstock				
Diameter		3,900 - 3,200 mm		
Length		87 m		
Power Station				
Installed capacity		26 MW		
Turbine		1 unit, vertical sheft, Francis		
Generator		1 unit, 30.5 MVA		
Rated head		54.8 m		
Max.discharge		56.6 m^3/s		
Dependable peak power		16.1 MW		
Annual energy		145.3 GWh		
Firm energy		66.4 GWh		
Secondary energy		78.9 GWh		
Transmission Line				
Voltage		132 kV, single circuit		
Distance		16 km		

TABLES

Table J.1.1 COMPARATIVE STUDY OF PROJECT DEVELOMENT SCALE

Item	Case 1-1	Case 1-2	Case 2-1	Case 2-2	Case 3-1	Case 3-2
1. Reservoir						
H.W.L. (EL.-m)	180.0	188.0	195.0	195.0	200.0	200.0
L.W.L. (EL.-m)	154.0	154.0	170.0	170.0	175.0	175.0
Operation W.L. (EL.-m)	154.0	154.0	174.4	174.4	182.8	182.8
Net storage capacity (MCM)	606	606	686	686	686	802
2. Dam						
Crest EL. (m)						
Main & 2nd saddle dam	192.2	199.0	199.0	199.0	204.0	204.0
1st saddle dam	190.7	190.7	197.5	197.5	202.5	202.5
Crest length (m)						
Main & 2nd saddle dam	950.0	950.0	980.0	980.0	1,020.0	1,020.0
1st saddle dam	371.0	371.0	396.0	396.0	421.0	421.0
3. Power Generation						
Installed capacity (MW)	-	23.0	26.0	42.0	28.0	69.0
Dependable peak power (MW)	-	0.0	16.1	22.8	16.5	40.2
Firm energy (GWh)	-	0.0	66.4	66.4	91.6	91.6
Secondary energy (GWh)	-	104.4	78.9	78.9	85.1	85.1
Annual energy (GWh)	-	104.4	145.3	145.3	176.7	176.7
4. Construction cost (US$ million)	117.1	147.6	163.6	186.8	184.1	219.2
5. Economic Evaluation *(US$ million) in discount rate of 10%*						
Economic incremental cost	-	27.1	40.8	60.8	59.3	88.6
Economic incremental benefit	-	30.8	81.9	88.1	92.1	114.0
Net incremental benefit	-	3.1	41.1	27.3	33.8	25.4
Benefit-Cost Ratio	-	1.14	2.00	1.45	1.58	1.29

Table J.4.1 ECONOMIC INCREMENTAL COST AND BENEFIT

Item	Case 1-1	Case 2-1	Case 1-2	Case 2-2	Case 1-3	Case 2-3
1. Economic Incremental Cost						
Construction cost (US$ 1,000)	-	147,610	163,650	186,850	184,090	219,250
Civil works	-	116,360	130,850	146,700	149,620	170,900
Mech & elect. works	-	31,250	32,800	40,150	34,470	48,350
Economic cost (US$ 1,000)						
Capital cost	-	25,940	39,580	59,300	56,950	86,840
O & M cost						
Civil works	-	490	560	620	640	730
Mech & elect. works	-	660	700	850	730	1,030
Total		27,090	40,840	60,770	58,320	88,600
2. Economic Incremental Benefit						
Power & energy						
Dependable peak power (MW)	-	0	16.1	22.8	16.5	40.2
Firm energy (GWh)	-	0	66.4	66.4	79.5	79.5
Secondary energy (GWh)	-	104.4	78.9	78.9	83.2	83.2
Annual energy (GWh)	-	104.4	145.3	145.3	162.7	162.7
Economic benefit (US$ 1,000)						
kW benefit	-	0	14,850	21,030	15,220	37,080
kWh benefit (firm)	-	0	43,710	43,710	52,340	52,340
kWh benefit (secondary)	-	30,850	23,310	23,310	24,580	24,580
Total		30,850	81,870	88,050	92,140	114,000
3. Net Incremental Benefit (US$ 1,000)		3,760	41,030	27,280	33,820	25,400
4. Benefit-Cost Ratio		1.14	2.00	1.45	1.58	1.29

Table J.4.2 CONSTRUCTION COST AND ECONOMIC COST

Item	Case 1-1	Case 2-1	Case 1-2	Case 2-2	Case 1-3	Case 2-3
1. Construction Cost						
1) General items	6,380	6,380	6,380	6,380	6,380	6,380
2) Diversion canal	3,795	3,795	3,795	3,795	3,795	3,795
3) Main dam	22,200	22,200	26,394	26,394	30,860	30,860
4) First saddle dam	40,150	40,150	45,197	45,197	52,720	52,720
5) Spillway & stilling basin	4,450	4,450	4,897	4,897	5,640	5,640
6) Power intake	-	310	313	540	330	640
7) Second saddle dam	6,390	6,390	7,455	7,455	8,960	8,960
8) Powerhouse	-	5,040	5,072	6,450	5,072	7,930
9) Hydromechanical works	5,884	7,391	7,391	8,314	7,391	9,584
10) Generating equipment	-	15,870	17,086	21,930	18,400	27,080
11) Transmission line	-	1,215	1,215	1,215	1,215	1,215
12) Afterbay weir	-	-	-	10,810	-	13,500
13) Compensation	2,490	2,490	3,000	3,000	3,450	3,450
Sub-total, 1) to 13)	91,739	115,681	128,195	146,377	144,213	171,754
14) Government administration	4,581	5,782	6,410	7,319	7,211	8,586
15) Engineering service	5,500	5,936	7,695	8,784	8,656	10,310
Sub-total, 1) to 15)	101,820	127,399	142,300	162,480	160,080	190,650
16) Physical contingency	15,270	19,250	21,350	24,370	24,010	28,600
Grand total	117,090	146,649	163,650	186,850	184,090	219,250
2. Incremental Construction Cost	-	30,520	46,560	69,760	67,000	102,160
3. Economic Incremental Cost	-	25,940	39,580	59,300	56,950	86,840

Table J.4.3 COST ESTIMATE OF ALTERNATIVE CASES (1/2)

Work Item	Unit Price (US$)	Case 1-1		Case 1-2		Case 2-1		Case 2-2		Case 3-1		Case 3-2	
		Qty	Amount	Qty	Amount	Qty	Amount	Qty	Amount	Qty	Amount	Qty	Amount
1. General Items		L.S.	6,380	L.S.	6,380	L.S.	6,380	L.S.	6,380	L.S	6,380	L.S.	6,380
2. Diversion Canal													
Excavation	4.7/m3	603,800	2,856	603,800	2,856	603,800	2,856	603,800	2,856	603,800	2,856	603,800	2,856
Coffering		L.S.	758	L.S.	758	L.S.	758	L.S.	758	L.S.	758	L.S.	758
Miscellaneous		L.S.	181	L.S.	181	L.S.	181	L.S.	181	L.S.	181	L.S.	181
Sub-total			3,795		3,795		3,795		3,795		3,795		3,795
3. Main Dam													
Excavation	4.9/m3	560,500	2,746	560,500	2,746	578,100	2,860	578,100	2,860	601,700	2,948	601,700	2,948
Embankment	7.8/m3	1,980,000	15,444	1,980,000	15,444	2,429,100	18,893	2,429,100	18,893	2,895,000	22,581	2,895,000	22,581
Grouting		L.S.	2,560	L.S.	2,560	L.S.	2,955	L.S.	2,955	L.S.	3,310	L.S.	3,310
Miscellaneous		L.S.	1,450	L.S.	1,450	L.S.	1,686	L.S.	1,686	L.S.	2,021	L.S.	2,021
Sub-total			22,200		22,200		26,394		26,394		30,860		30,860
4. First Saddle Dam													
Excavation	6.6/m3	235,000	1,551	235,000	1,551	248,300	1,631	248,300	1,631	265,000	1,749	265,000	1,749
Concrete	103.8/m3	334,000	34,669	334,000	34,669	375,910	38,992	375,910	38,992	440,000	45,672	440,000	45,672
Grouting		L.S.	2,015	L.S.	2,015	L.S.	2,421	L.S.	2,421	L.S.	2,780	L.S.	2,780
Miscellaneous		L.S.	1,915	L.S.	1,915	L.S.	2,153	L.S.	2,153	L.S.	2,519	L.S.	2,519
Sub-total			40,150		40,150		45,197		45,197		52,720		52,720
5. Sopillway & Stilling Basin													
Excavation	7.0/m3	55,000	389	55,000	389	58,500	406	58,500	406	64,500	452	64,500	452
Concrete	166.6/m3	23,100	3,849	23,100	3,849	25,570	4,258	25,570	4,258	29,500	4,915	29,500	4,915
Miscellaneous		L.S.	212	L.S.	212	L.S.	233	L.S.	233	L.S.	273	L.S.	273
Sub-total			4,450		4,450		4,897		4,897		5,640		5,640
6. Power Intake													
Concrete	116.8/m3	-	-	2,490	291	2,560	299	4,350	508	2,650	310	5,150	602
Miscellaneous		-	-	L.S.	19	L.S.	14	L.S.	32	L.S.	20	L.S.	38
Sub-total					310		313		540		330		640
7. Second Saddle													
Excavation	4.9/m3	175,000	858	175,000	858	177,100	874	177,100	874	185,000	907	185,000	907
Embankment	7.3/m3	359,000	2,621	359,000	2,621	430,600	3,101	430,600	3,101	565,000	4,125	565,000	4,125
Grouting		L.S.	2,610	L.S.	2,610	L.S.	3,126	L.S.	3,126	L.S.	3,500	L.S.	3,500
Miscellaneous		L.S.	301	L.S.	301	L.S.	354	L.S.	354	L.S.	428	L.S.	428
Sub-total			6,390		6,390		7,455		7,455		8,960		8,960

Table J.4.3 COST ESTIMATE OF ALTERNATIVE CASES (2/2)

Work Item	Unit Price (US$)	Case 1-1		Case 1-2		Case 2-1		Case 2-2		Case 3-1		Case 3-2	
		Qty	Amount	Qty	Amount	Qty	Amount	Qty	Amount	Qty	Amount	Qty	Amount
8. Powerhouse													
Excavation	7.9/m3	-	-	118,800	939	118,800	940	151,200	1,194	118,800	940	186,600	1,474
Concrete	139.0/m3	-	-	11,500	1,599	11,710	1,628	14,900	2,071	11,710	1,628	18,300	2,544
Superstructure		-	-	L.S.	2,261	L.S.	2,261	L.S.	2,878	L.S.	2,261	L.S.	3,532
Miscellaneous		-	-	L.S.	241	L.S.	243	L.S.	307	L.S.	243	L.S.	380
Sub-total		-	-		5,040		5,072		6,450		5,072		7,930
9. Hydromechanical Work													
Spillway Gate	7,000/t	560	3,920	560	3,920	560	3,920	560	3,920	560	3,920	560	3,920
Diversion	6000/t	104	624	104	624	104	624	104	624	104	624	104	624
Conduit Gates													
River Outlet	5,000/t	268	1,340	268	1,340	268	1,340	268	1,340	268	1,340	268	1,340
Intake Gate	6,500/t	-	-	66	429	66	429	107	700	66	429	165	1,080
Steel Penstock	3,500/t	-	-	255	893	255	893	415	1,460	255	893	625	2,190
Tailrace Gate	4,500/t	-	-	37	185	37	185	60	270	37	185	95	430
Sub-total			5,884		7,391		7,391		8,314		7,391		9,584
10. Generating Equipment			-	L.S.		L.S.		L.S.		L.S.		L.S.	
11. Transmission Line (132 kV)			-	L.S.	1,215	L.S.	1,215	L.S.	1,215	L.S.	1,215	L.S.	1,215
12. Afterbay Weir													
Excavation	4.9/m3	-	-	-	-	-	-	110,000	544	-	-	133,000	652
Embankment	7.3/m3	-	-	-	-	-	-	618,000	4,511	-	-	810,000	5,913
Concrete	103.8/m3	-	-	-	-	-	-	15,500	1,609	-	-	18,000	1,868
Grouting		-	-	-	-	-	-	-	3,158	-	-	-	3,840
Miscellaneous		-	-	-	-	-	-	-	988	-	-	-	1,227
Sub-total									10,810				13,500

FIGURES

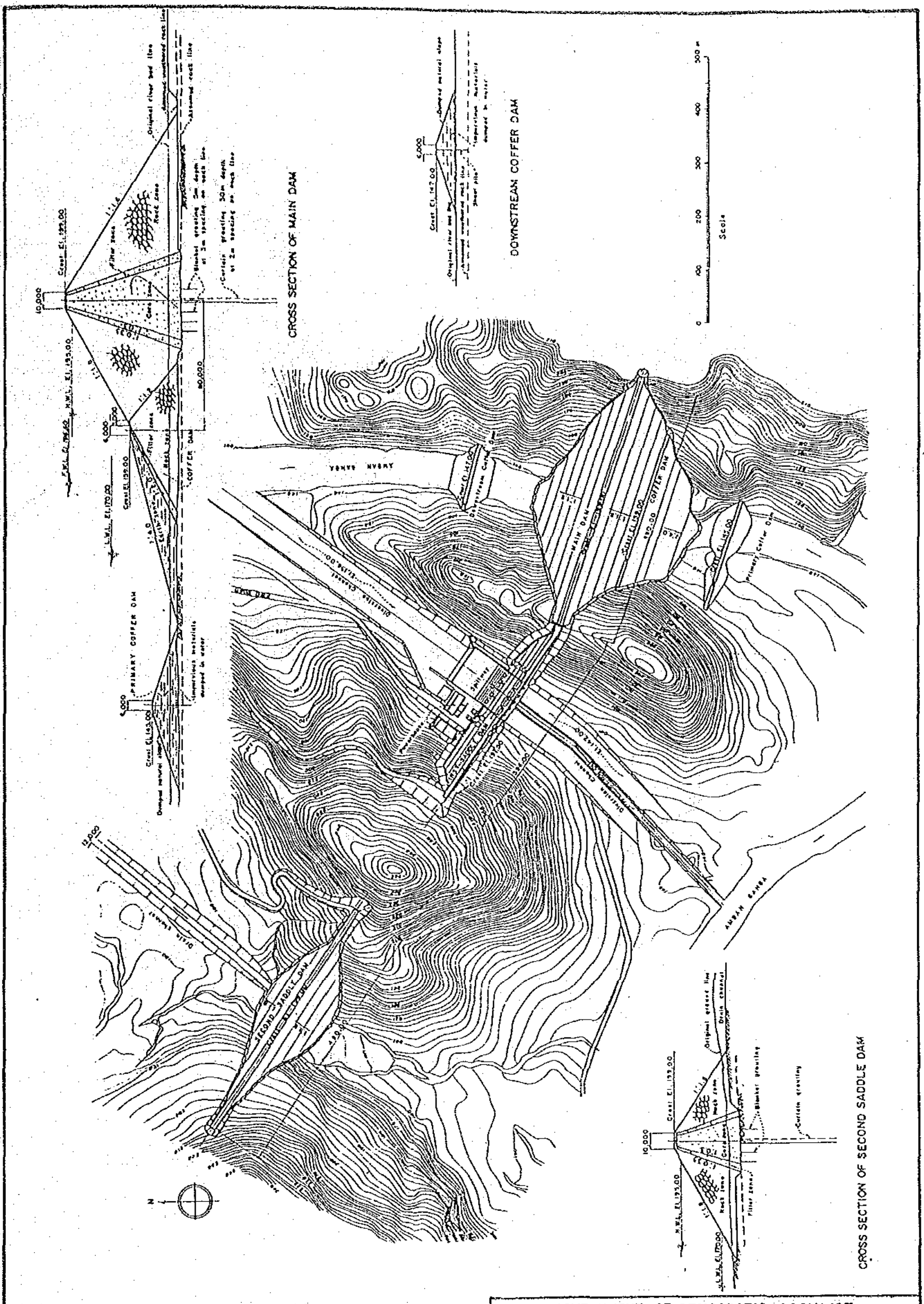

Fig. J.2-1 General Plan of Optimal Case of Moragahakanda Dam

GOVERNMENT OF DEMOCRATIC SOCIALIST REPUBLIC OF SRI LANKA
MINISTRY OF MAHAWELI DEVELOPMENT
THE STUDY ON EXTENSION OF THE MORAGAHAKANDA AGRICULTURAL DEVELOPMENT PROJECT
JAPAN INTERNATIONAL COOPERATION AGENCY

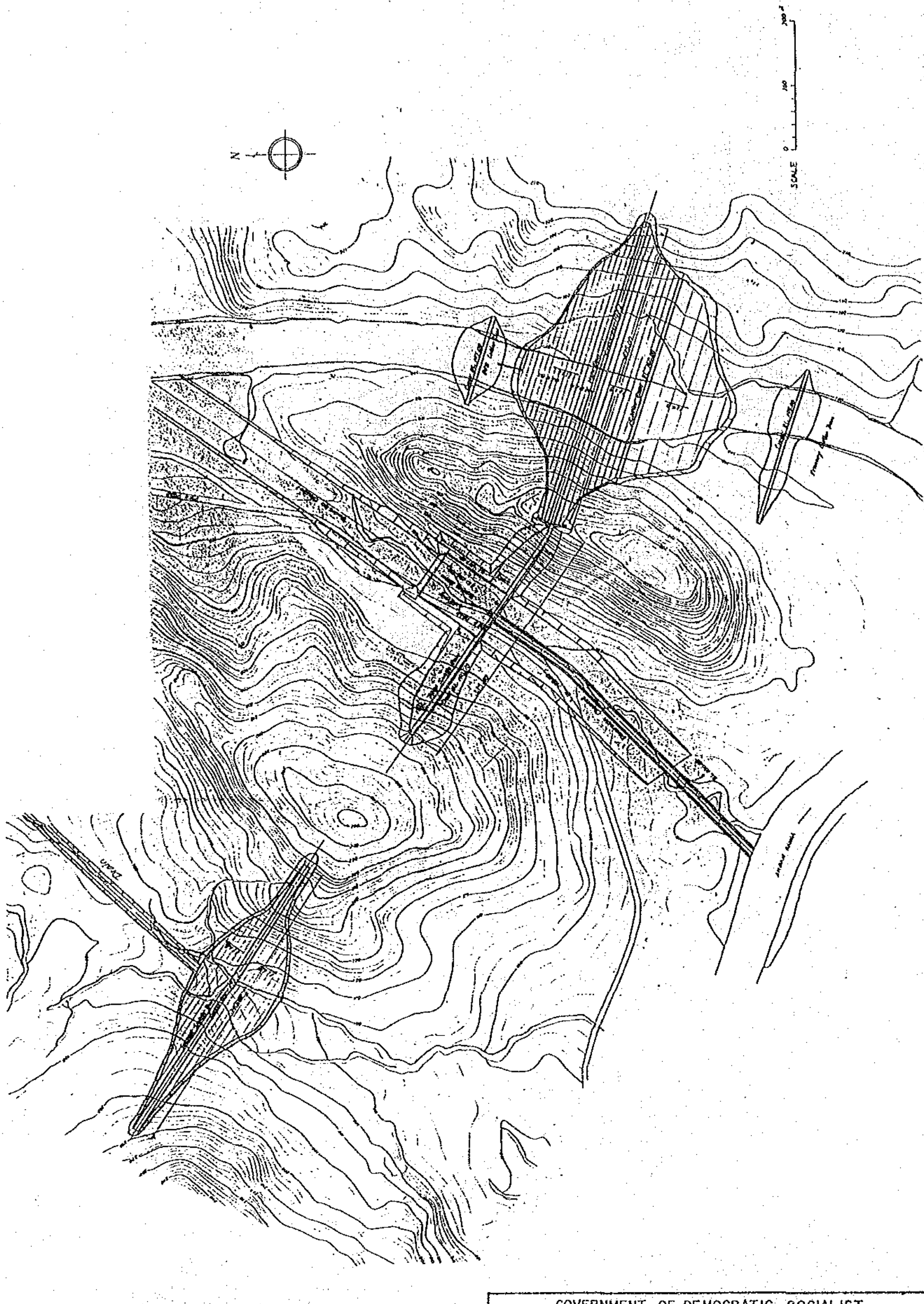

Fig. J.2-2 General Plan for Moragahakanda Dam for Irrigation Purpose

GOVERNMENT OF DEMOCRATIC SOCIALIST REPUBLIC OF SRI LANKA
MINISTRY OF MAHAWELI DEVELOPMENT
THE STUDY ON EXTENSION OF THE MORAGAHAKANDA AGRICULTURAL DEVELOPMENT PROJECT
JAPAN INTERNATIONAL COOPERATION AGENCY

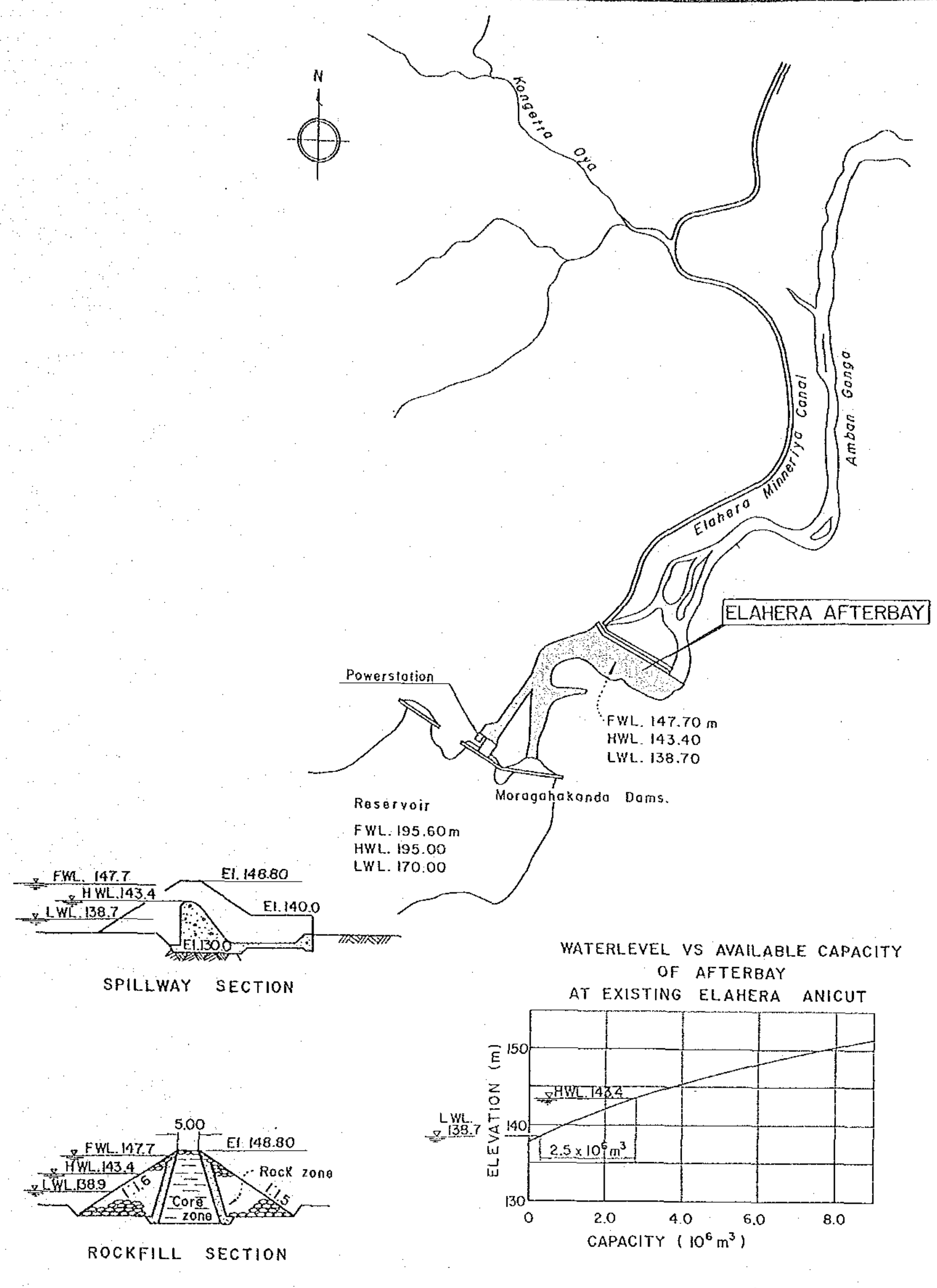

Fig. J.3-1 Plan for Elahara Afterbay

GOVERNMENT OF DEMOCRATIC SOCIALIST
REPUBLIC OF SRI LANKA
MINISTRY OF MAHAWELI DEVELOPMENT
THE STUDY ON EXTENSION OF
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