12.2.4. Scheme D : Inclined Intake Structure

(1) Design

For this scheme it is proposed that an inclined intake facility be constructed on the right abutment slope near the existing intake tower. The new intake will be connected to the original intake with a steel pipe. The design of the new intake structure is such that it will remain operational for any height of sediment depositions in the reservoir. The above design is shown in Figs.-12.10 and 12.11.

The abutment slope will be excavated and the inclined bed will be prepared as required to install the new intake steel structure. Three inlets at three different levels will be provided. Each inlet will be provided with a gate capable of stopping the flow at the time of inspection of the tunnels and the valves. The inlet level will be selected in accordance with the level of the sediments in the reservoir.

The steel pipe bridge, shown as a horizontal bridge in Fig.-12.11, will be used for connecting the new intake with the existing intake with a 7 m diameter steel pipe. Its structure will be similar to that of an aqueduct.

The steel pipe will be designed to withstand the external hydrostatic pressures for the case when intake gates are closed and there is no water in the pipe. The pipe will be laid on a platform supported with steel tube piles.

For connection of the pipe with the top of the original intake tower, the top of the tower will first have to be demolished and its top finished as required. After this, the steel pipe is lowered into the tower. The gap between the pipe and the structure is grouted for stability. The steel pipe bridge will be provided with two joints to allow movement.

The stability of the new intake structure is improved by placing it on the bedrock.

(2) Construction Method

The foundation work for the new intake and installation of the intake steel structure will be carried out by ordinary construction methods and controlling the reservoir water levels as required. However, the steel pipe bridge and the connecting structure to the original intake tower, will be constructed mostly underwater.

To drive the piles required for this scheme, four 300 tons barges equipped with a reversible excavator (R.R.C-15) should be provided. The excavation for the pile foundations which will be the substructure of the steel pipe bridge, will be made first and the piles will then be driven.

After cleaning the inside of the piles, and the steel reinforcement is installed, the piles will be filled with underwater concrete (tremie). The platform supporting the steel pipe will be constructed by underwater concreting. The portion above the water level will be constructed by using conventional concreting methods.

The cover on the top of the original intake tower will be removed by cutting it into small blocks with a boring machine.

The top of the tower will be prepared by pouring some concrete on it to present a smooth appearance. After this, the steel pipe will be installed and the gap between the structure and the pipe grouted.

Most of the above work will be carried out underwater by maintaining the reservoir water level at LWL. The proficiency of the divers will have a great influence on the progress of the construction. Professional divers' assistance will be required.

(3) Construction Schedule

Except for the construction of the new intake structure, construction of most other items of this scheme will be carried out in the area around the existing intake tower, on the surface of the reservoir or underwater.

During the working season, construction will be carried out by controlling the reservoir water level and the intake flow as required to facilitate construction.

For Scheme D, as was the case with Scheme B, pile driving work will be the key item of the construction schedule. As the required number of piles for Scheme D will be only 31, less than for Scheme B, construction of the pile foundations should be completed in two seasons. During the other two seasons, the new intake structure and the steel pipe bridge will be construction.

At the end of the third year, the top of the original intake tower will be removed, and part of the steel pipe bridge installed. In the fourth year, the horizontal portion of the bridge will be constructed. Details of the above construction schedule are shown in Fig.-12-12.

12.2.5. Scheme E : Vertical Intake Tower

(1) Design

Under this plan, it is proposed that a pneumatic caisson be erected at a point about 57 m downstream from the intake tower to the bedrock under which the headrace tunnel runs. The caisson will be used as a foundation for the new intake tower and the excavation of the shaft for connection with the headrace tunnel. This plan provides for a thorough improvement of the intake facilities.

As the construction of the new intake tower will be carried out within the reservoir, it is proposed that the tower be made of steel to shorten its construction period. The foundation of the tower will be constructed by sinking and stabilizing a pneumatic caisson. The caisson will also be used as watertight enclosure from which construction of the shaft will be carried out.

The 7 m diameter shaft will reach the existing headrace tunnel at a depth of 83.0 m. The original intake will be closed by stoplogs before the shaft is connected to the headrace. Construction of the plug and the connection of the shaft and the headrace will be performed as part of the final concrete work.

The new intake tower will be provided with three inlets, each at a different level. Each inlet will be provided with a gate to stop the flow in the tunnel for inspection and select the appropriate intake level as governed by the sediment depositions.

Steel lining at the elbow portion has not been considered at this stage as the flow velocity is rather slow or 1.59 m/sec. However, use of steel liners can be taken into consideration for the purpose of construction forms. It is deemed necessary to examine this subject at the step of detailed engineering.

(2) Construction Method

For construction of the intake tower foundation, it is proposed that a pneumatic caisson be installed at EL 717.0 m and sunk to EL 702.5 m. The side walls of the caisson should be raised to EL 726.0 m as required by the reservoir water level rise. All the above will be performed as open air construction.

The pneumatic caisson method is widely used for construction of intake or outlet structures within reservoir areas.

Taking about six months for construction during the first dry season, the construction could be carried out in the open by controlling the reservoir water level between EL 700.0 and 710.0 m.

The rate of progress in the vertical shaft excavation is assumed to be 0.9 m/day. Hence, the excavation of the 83 m shaft can be finished in some three months, but it will be done in two series. The first series will be finished in about a month, and this will be followed by concrete lining over about a month. The second series will start successively and will be completed in some two months, then concrete lining will follow and it will be finished in about a month.

The existing intake will be closed with stoplogs by use of divers. This will be done in the third year. The shaft will be connected to the headrace after the existing intake has been closed. Plugging of the connection and the concreting work for the shaft and the connection will be done afterwords.

All construction for the new intake tower will be carried out in the open. In the fourth season, however, the construction will be mainly carried out from the water surface with certain amount of the work required to be carried out underwater.

(3) Construction Schedule

The construction period required for this scheme is the shortest of the three schemes considered. As the construction will be carried out within the reservoir area, the working period in a year will be limited to the six months of the dry season. Therefore, the total construction period for this scheme will be four seasons, spanning over five years.

In the first year, access roads will be constructed and construction facilities will be installed. The caisson will be erected during this period by sinking.

In the second year, the shaft will be excavated to a point above the headrace tunnel and its concreting completed. The foundation of the intake tower will also be completed. During the off working season (wet season), the above above facilities will be submerged as the reservoir will rise. The construction will resume next working season after, first, unwatering the facilities.

Up to the third working season, construction will be carried out in the open without affecting the on going operation of the existing generating facilities. This is one important advantage which this scheme has over the other two schemes.

Closing of the existing intake with stoplogs and construction of the connection between the shaft and the headrace tunnel will be performed at the end of the third working season. After this, the water will start flowing into the turbines through the new intake.

In the fourth reason, the remaining work for the intake will be completed while the powerplant would be in operation. Occasional interruption of the flow for this scheme would be necessary only during the last two working seasons.

The construction schedule described above is shown in Fig.-12.15.

12.2.6. Upstream Face of the Dam

(1) Design

The slope gradient for the dam is determined on the basis of the assumed seismic conditions and the shearing strength of the rockfill materials. The shearing strength of the rock materials used in the dam construction is obtained by soil tests. Using the data from these results, the dam slope gradient is determined.

Fig.-12.16 is a plan of the dam showing the conditions of the upstream face of dam.

Fig.-12.17 and 12-18 show the dam cross sections with slope gradient 1.80:1 as required for the rehabilitation plan. Fig.-12.19 shows the location of the proposed quarry site.

Detail geological investigation should be carried out for the proposed quarry site by boring and construction of test adits. Soil tests should be performed on samples of rock materials collected at the quarry.

As the same type of diorite was used as a rock material for construction of the Ambuklao Dam, it is possible to predict the physical properties of the proposed rock materials.

In the study presented in this Report, the slope gradient of 1:1.80 was used. The rockfill volume for the embankment for this case becomes 230,000 m^3.

(2) Construction Method

Initially, a bench excavation will be made at the quarry site as shown in Fig.-12.19. Six months will be required for clearing, removal of surface soil, preparation of the disposal area, and construction of the access road. A crawler type drill will be used for the bench cut. The bench height should be about 5 - 8 m.

The volume of the material to be disposed of, is estimated at 200,000 m^3. The heavy equipment required for the operation of the quarry will consist of a 32 ton-class bulldozer, a 5 m^3-class tractor shovel for loading, a 11 ton-class dump truck for transport, and an 11ton-class bulldozer for spreading.

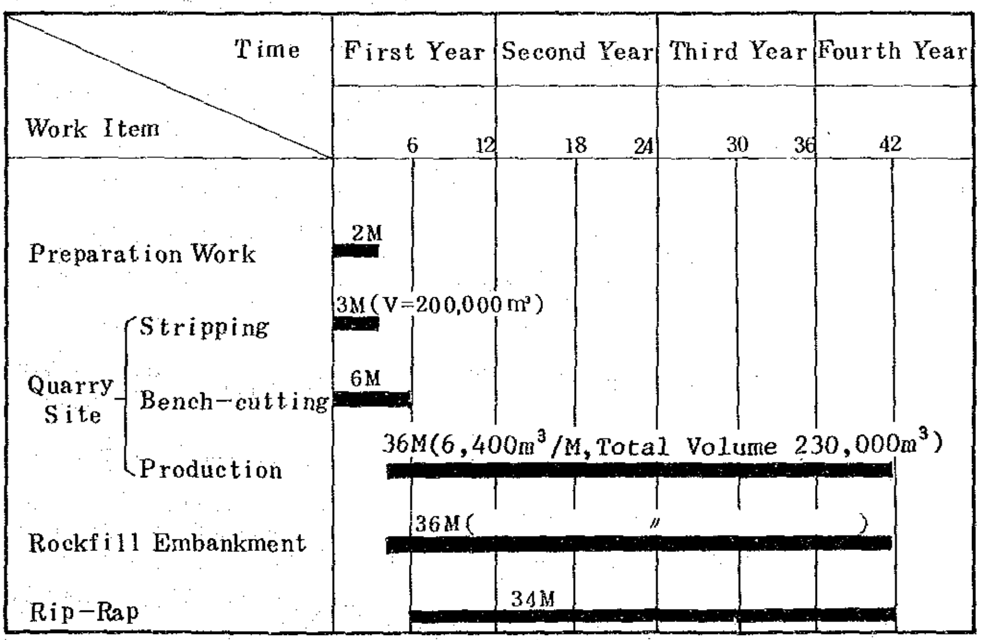
For construction of the embankment, due to the existing road conditions, an 11ton-class dump truck is proposed to be used for transportation. From the level on the upstream face at EL 680 m, the rock material will be spread with a 32ton-class bulldozer and compacted with a 13ton-class vibratory roller. The surface of the dam will be finished with a riprap backhoe.

The construction of the embankment with a volume of 230,000 m^3, is 3 years to be depended on the intake tower improvement.

(3) Construction Schedule

The construction schedule is shown in Table-12.2 below.

Table-12.2 Construction Schedule



12.2.7. Tailrace Outlet Channel (Scheme F)

(1) Design

This scheme is thoroughly described in Chapter 11. By excavating a channel along the riverbed, the water level at the tailrace outlet could be lowered by about 1.27 m. This is a temporary remedy, but it is easy to implement and effective.

This scheme consists of a channel excavated along the riverbed for a distance of about 1,500 meters. The channel has a trapezoidal section, it is 25 m wide at the bottom, and its sides are sloped 1 vertical to 2 horizontal. The channel gradient is 1 on 1,600. The channel alignment and sections are shown in Figs.-12.22 and 23.

The excavation will include bedrock and sediments. The excavated material will be disposed of along the bank in an acceptable manner. Thus, long distance hauling will be avoided. As the channel is expected to be covered with sediment deposits every wet season, removal of such deposits will have to be made every year.

By implementing the above scheme which will require first, construction of the channel, and, second, removal of sediment deposits from the channel every year, it will be possible to maintain the water level at the existing tailrace tunnel outlet at about EL 577.8 meters.

To make the above scheme effective, it will be required to construct a check dam for collection of the sediments of the Besack River. This trap dam will have to be constructed at the downstream dam of the Besack River which flows into Agno River a short distance upstream from the tailrace outlet.

(2) Construction Methods and Schedule

Underwater type bulldozer of D-7 class will be required for excavating the riverbed. The water depth in the working area will be around 1.5 m.

Using two underwater bulldozers, one ordinary D-7 class bulldozer, one backhoe, and three 12 ton dump trucks, the underwater excavation of 70,000 to 80,000 m^3 of bedrock and sediments should be completed in about two months period.

The construction should be started at the downstream end of the channel and advance upstream.

12.2.8. Tailrace Tunnel Extension (Scheme H)

(1) Design

To protect the tailrace outlet against the sediments, an 1,500 m long tailrace tunnel extension is proposed. The extension will be a free-standing reinforced concrete cut and cover conduit. By extending the tunnel in this manner, the tailrace outlet will be moved to an area of the river well protected against siltation.

The difference between the increases and losses in head due to the extension are not large. It is estimated that the total head increase will be of the order of one meter. However, the main benefit from shifting the outlet will be a very good protection of the tailrace outlet from sedimentation.

As an extension of the tailrace tunnel by a tunnel would result in excessive cost, it is proposed that the first

38 meters of the extension, next to the existing portal, be constructed as a tunnel, and the rest, as a cut and cover conduit. The conduit will be located along the right bank of the river.

The cross section of the conduit will have a diameter of 5.00 m. The cross section and the gradient of the conduit will be the same as those of the tailrace tunnels. For illustration see Figs-12.24 and 25.

(2) Construction Methods

The river flow will be diverted to the other side of the river from the area along which the conduit will be constructed. Excavation for the conduit will be done in dry. Construction will start at the downstream end and will proceed upstream. The conduit will be covered with backfill after completion.

The tunnel portion will be constructed during dry season of the last construction year. For construction of this portion, a temporary cofferdam will be required. Drilling, blasting and removal of the excavated material must be done within a short period. Connection of the conduit to the original tailrace tunnel should be done as quickly as possible, as the flow through the tailrace tunnel will have to be suspended. The period of this portion of the construction will be about four months including the concrete curing period.

A construction access road from the highway to the river-bed area along which the extension conduit will be built, should first be constructed, to secure transportation of construction materials. An area 80,000 m^2 large will be prepared immediately upstream of the tailrace outlet as required for storage.

The river is relatively wide along the proposed conduit alignment. Therefore, no problems are foreseen with regard to the diversion of the river during the dry season to enable construction of the conduit in dry. The conduit will, thus, be constructed using conventional methods.

(3) Construction Schedule

It is estimated that three months will be required for construction of the construction road, one month for riverbed preparation and four months for construction preparation. First, the conduit will be constructed starting from the downstream end, then the tunnel portion of the extension will be done. For construction of the tunnel portion of the extension, 38 meters long, the working area will have to be separated from the river by construction of a cofferdam. The excavation and concreting of the tunnel will be done in dry. The flow in the tailrace tunnel will be suspended at the time of the connection of the extension to the existing tailrace tunnel.

The construction schedule for the tunnel extension is shown in Fig.-12.26.

12.2.9. Dredging Around Intake Tower (Scheme 0)

(1) Design

As mentioned in Sections 11 and 12, sediment level around the existing intake tower will continue to rise even during the period of proposed rehabilitation work. To prevent power generating equipment from the inflow of sediment debris until the rehabilitation work is completed, it is necessary to perform dredging around the intake tower.

The range of this dredging work is planned to cover the area of 300 m radius around the intake tower as a centre, and the dredging depth is 6 m below the inlet bottom level of the intake tower. (Refer to Fig.-12.27.).

The dredging work is aimed to eliminate sediment deposits in the gross amount of approx. 250,000 m^3. In this case, as the rate of containing silt and sand is assumed to be 20%, gross quantity of dredged material will be 1,250,000 m^3.

Principal features of this scheme are as follows:

1) Dredging plan

a. Gross annual dredging volume

250,000 m^3/year

(300 m radius of the intake tower, average thickness of 1.4 m)

b. Hourly dredging volume

Annual operating hours

20 hrs/day x 25 day/month x 10 months/year

= 5,000 hours/year

Hourly dredging volume

$$\frac{250,000 \text{ m}^3}{5,000 \text{ hrs}} = 50 \text{ m}^3/\text{hr}$$

2) Dredging facilities

a. Dredger

Barge-L48m x W6.8m x D2.25m
Possible dredging depth - max. 77m
Pump size - 6' (150 mm)

b. Pump specification

Soil - 50 m^3/hr
Mud - 250 m^3/hr
Suction pipe - 95 m
Off-shore pipe line - 500 m
Upland pipe line - 1,300 m

Main pump	- Total head 51m 125 kW/4,160 V, 1 unit
Underwater pump	- Total head 20 m 50 kW/440 V, 1 unit
Booster pump	- Total head 51 m 125 kW/4,160 V, 1 unit

3) Annual power consumption

a. Equipment	Barge	360 kW
	Booster	160 kW
	Total	520 kW
b. Power load	Barge	230 kW
	Booster	100 kW
	Total	330 kW

c. Annual energy consumption
330 kW x 5,000 hrs/year = 1,650,000 kWh

d. Switching equipment 700 kVA/13.2 kV - 4.16 kV

e. Power supply (Power plant - Dam)
400 m overhead line

A stilling basin will be provided by construction of two 15 m high dams on the curved portion of the original Agno river immediately downstream of the dam. Approx. one million m^3 of sediment storage will be secured in this section between two dams.

The stilling basin will be divided into four blocks for processing discharged sediments, i.e. storage, stilling, drying and hauling.

Sand and silt dried at the stilling basin will be finally contained into spoil bank proposed around the location of tailrace outlet. (refer to Fig.-12.29) It may be necessary to conduct further study on the prevention of re-flowing out of sediment materials from the spoil bank. However, such prevention will be made possible by means of chemical treatment of sediments or physical treatment (measures for spoil bank structure).

(2) Construction method

The method is substantially same as that stated for Scheme A.

(3) Construction schedule

On the whole, rehabilitation project proposed is a combination of various works. In parallel with the dredging work, therefore, some of other works will be carried out simultaneously, and it is considered that there will be occassions of suspension or restriction during the dredging operation.

Taking into account these events, net operating duration was estimated on average 10 months per year. Remaining 2 months are set aside as provisional to such events and natural constraints like typhoon and floods.

As a matter of course, maintenance and reconditioning of mechanical equipment, interface with related works, and resumption of work operation after suspension should be well done to increase annual operating durations as much as possible. The dredging facilities are estimated to be completed in 15 months as shown in Fig.-12.28.

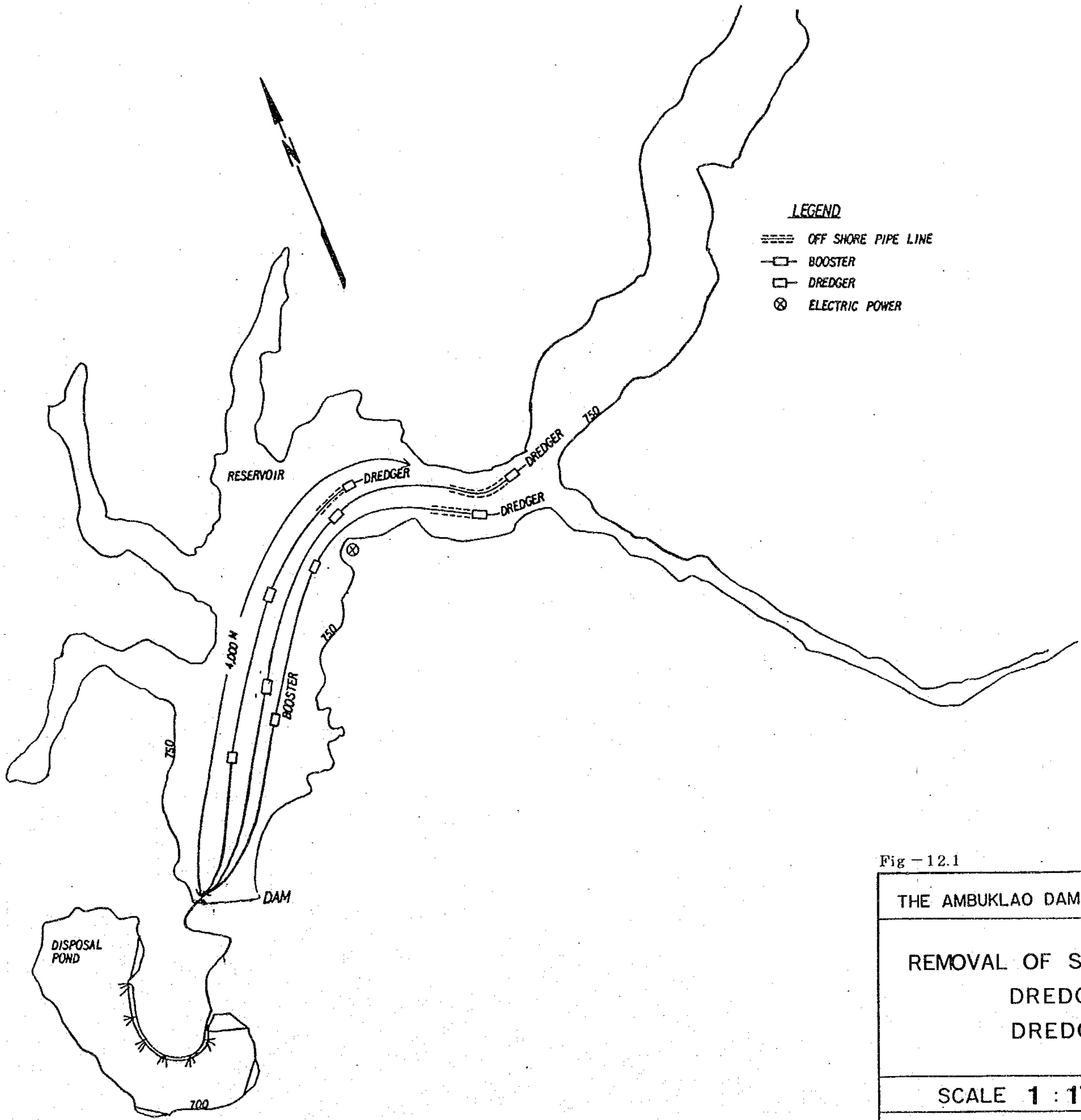
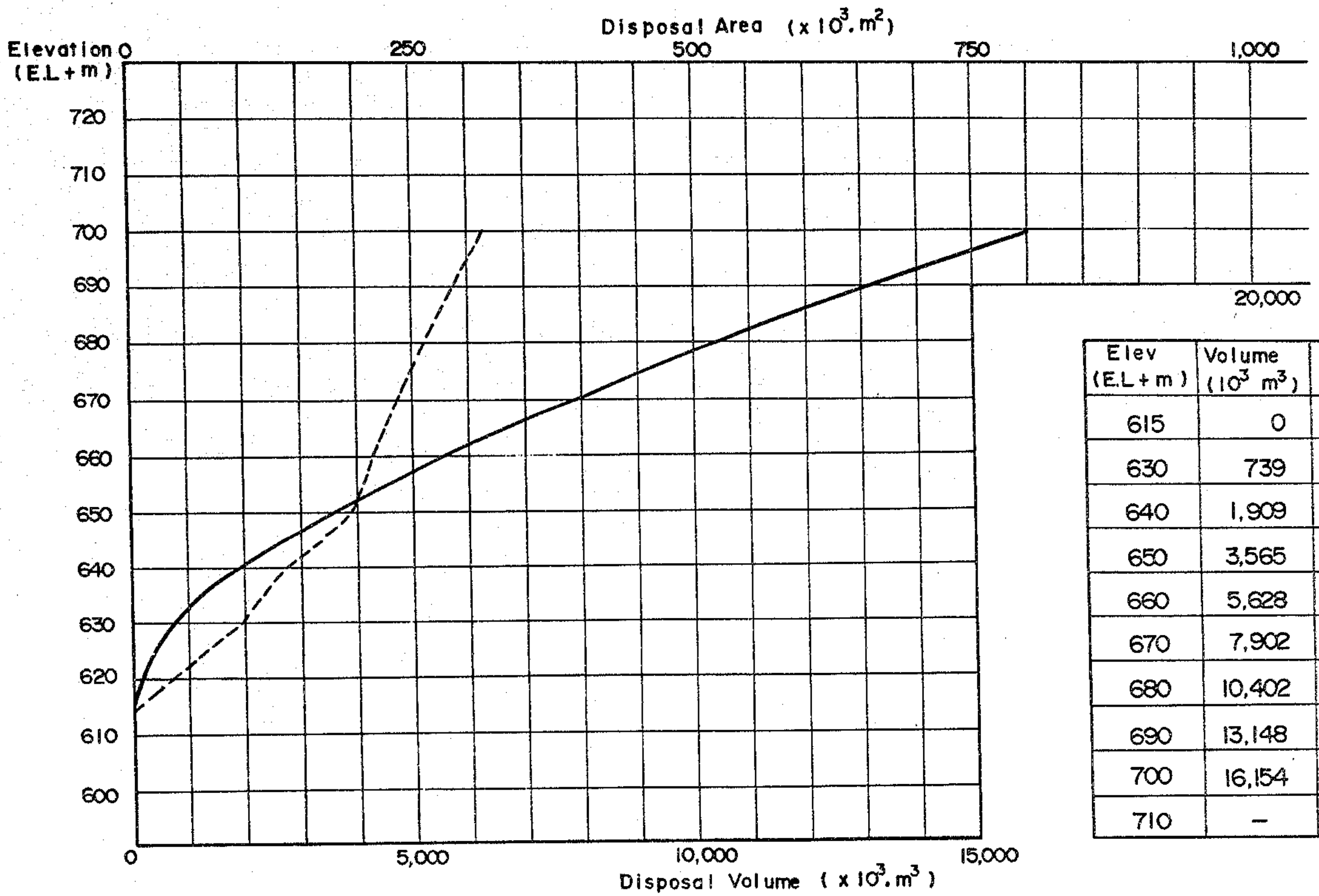


Fig －12.1

THE AMBUKLAO DAM REHABILITATION PROJECT
REMOVAL OF SEDIMENTS BY BIG DREDGING BOAT DREDGING PLAN
SCALE **1 : 17,700**
THE JAPAN INTERNATIONAL COOPERATION AGENCY



Elev (E.L+m)	Volume (10^3 m^3)	Area (10^3 m^2)
615	0	0
630	739	99
640	1,909	136
650	3,565	196
660	5,628	217
670	7,902	238
680	10,402	262
690	13,148	287
700	16,154	314
710	–	–

Fig-12.1(a) Capacity Curve of Disposal Area (Sedimentation basin)

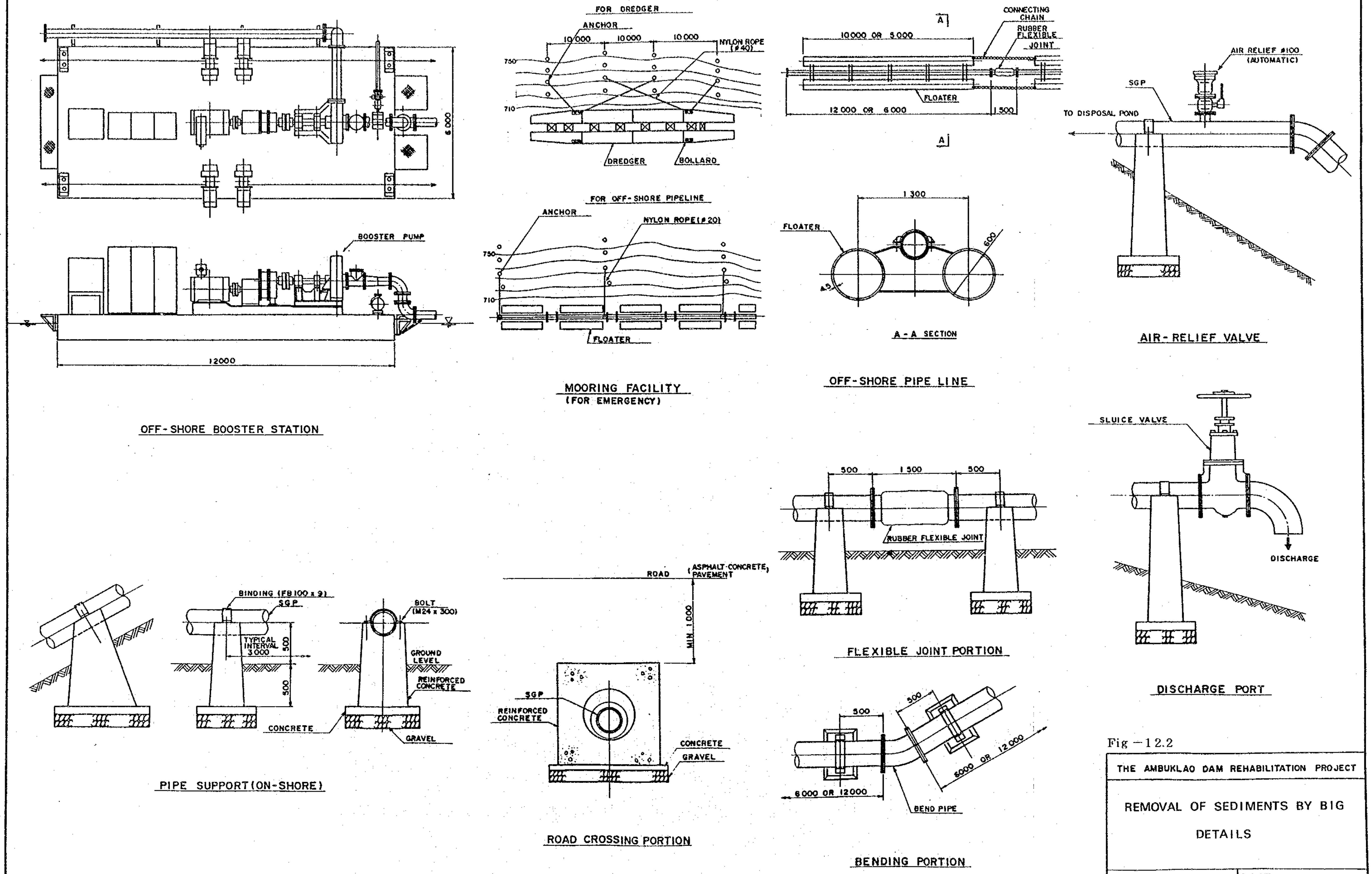


Fig -12.2

THE AMBUKLAO DAM REHABILITATION PROJECT	
REMOVAL OF SEDIMENTS BY BIG DETAILS	
SCALE FREE	DATE
THE JAPAN INTERNATIONAL COOPERATION AGENCY	

Fig – 12.3 Scheme A Removal of Sediments by Big Dredging Boat

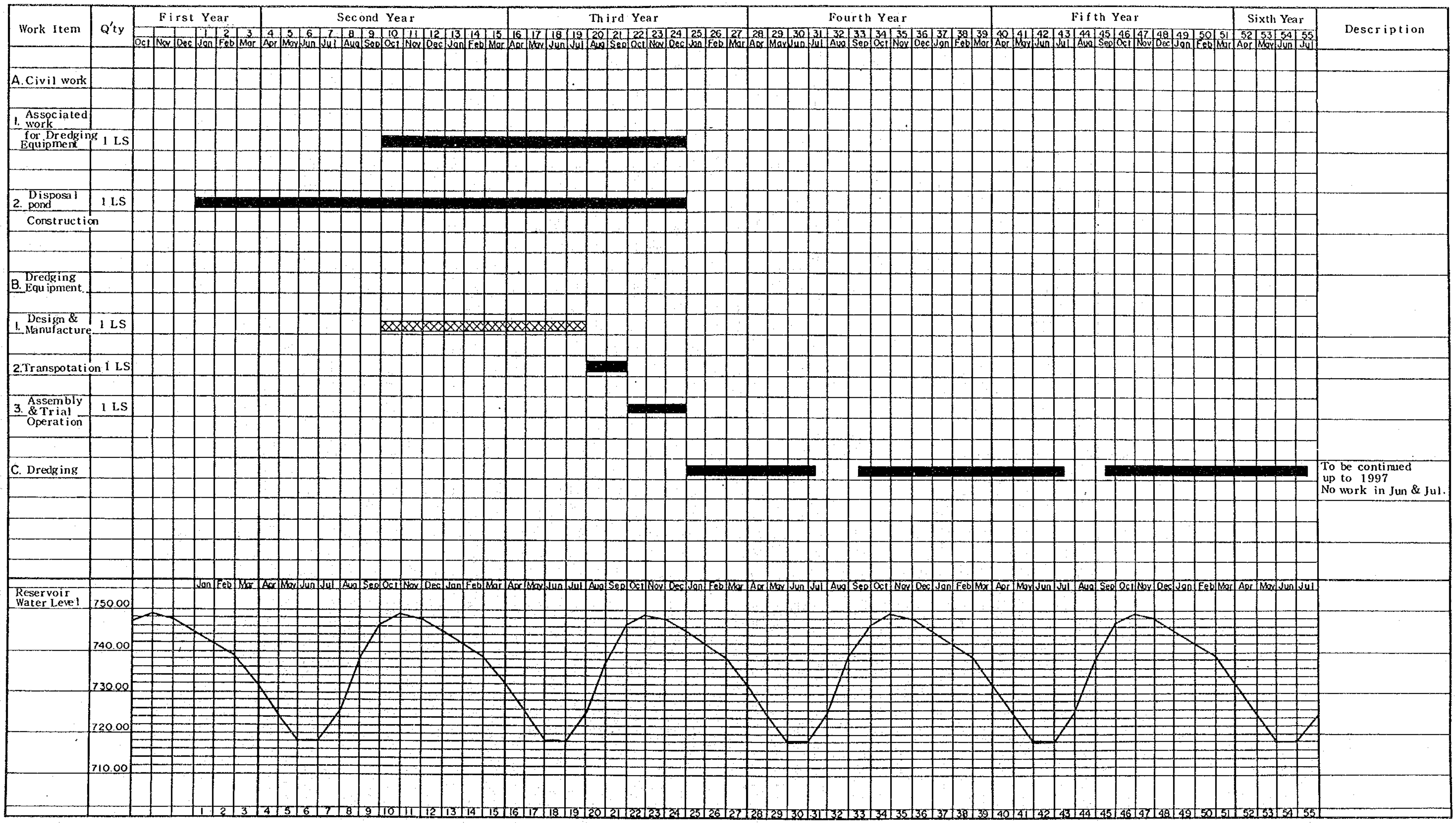


Fig – 12.3

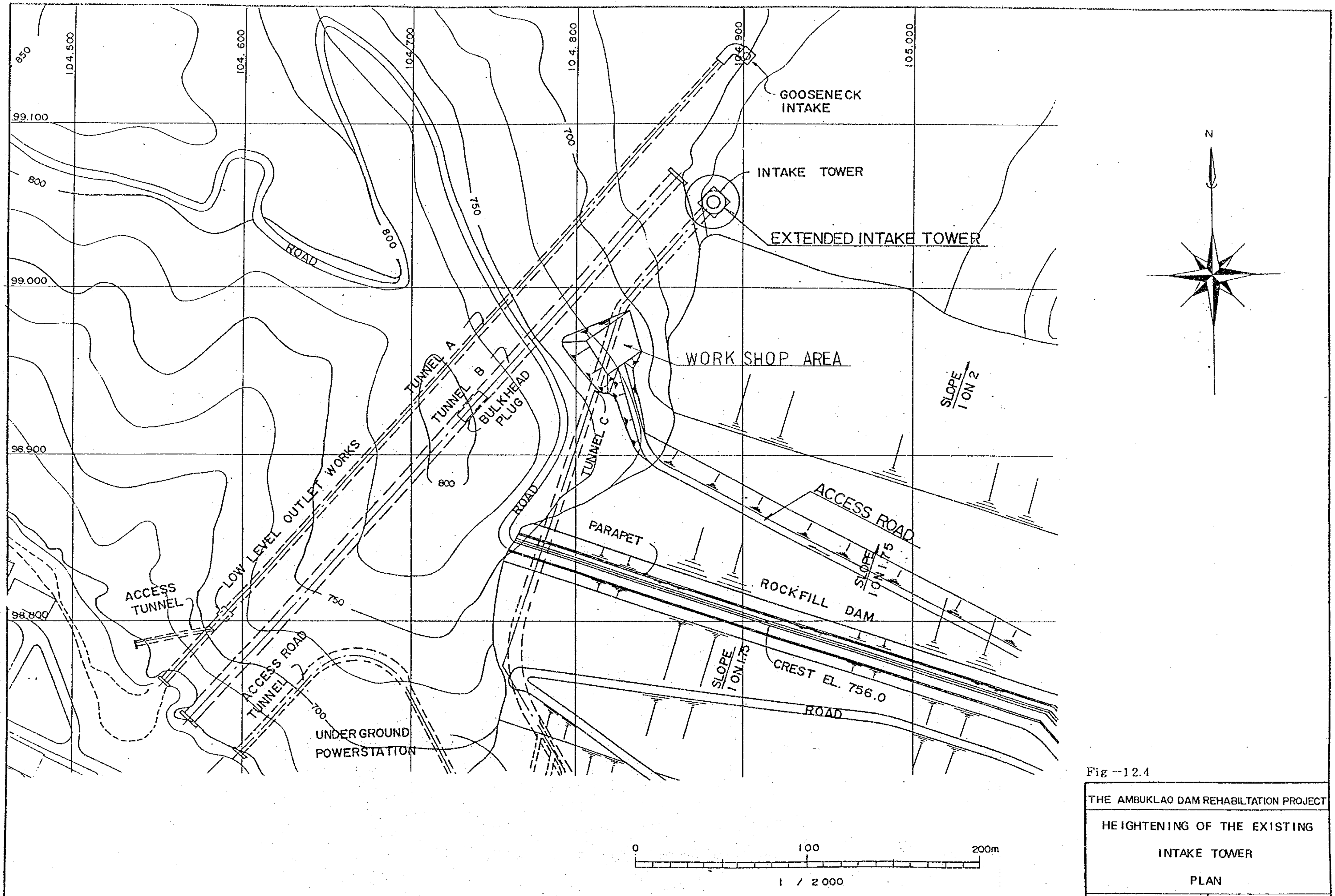
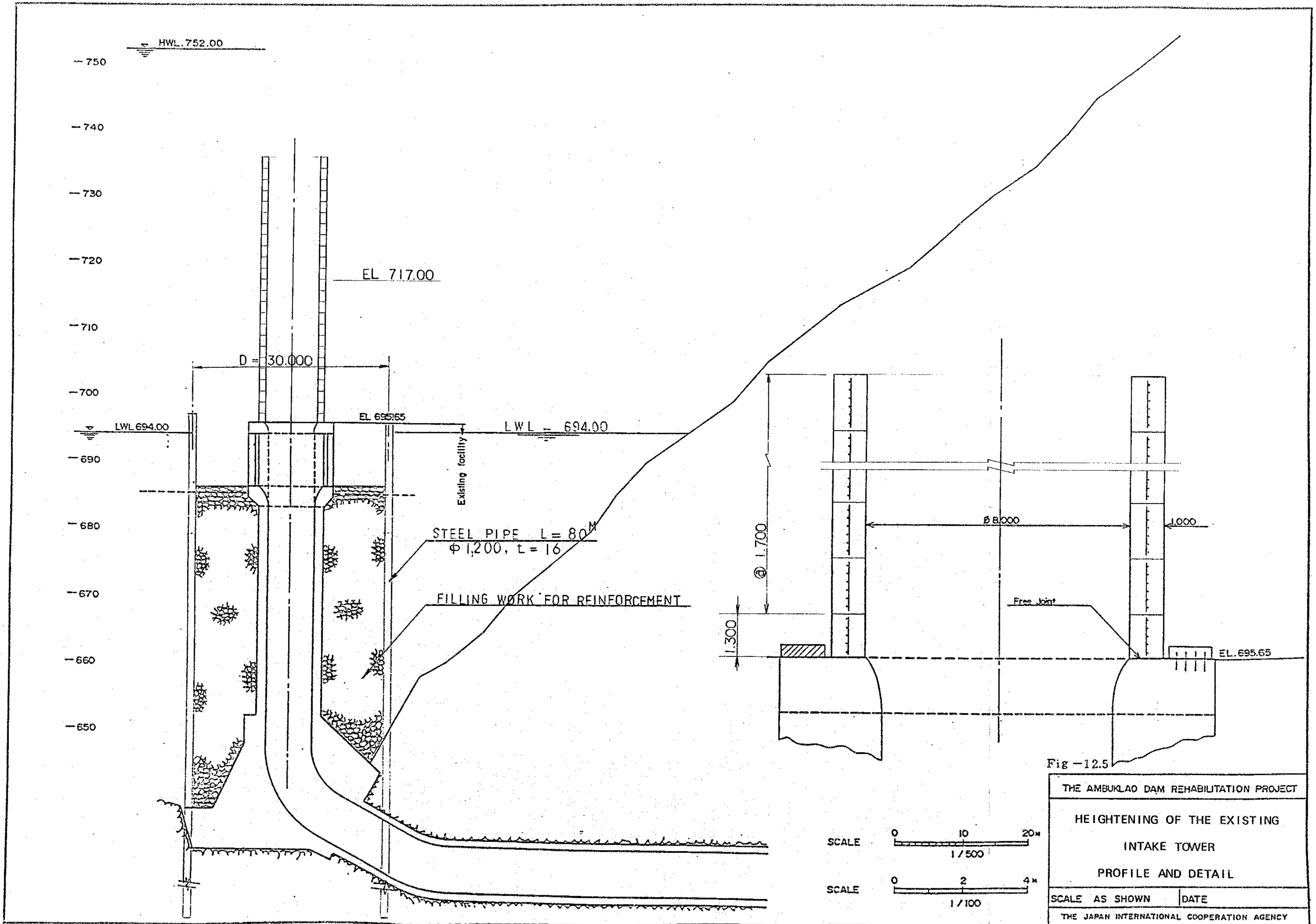


Fig —12.4

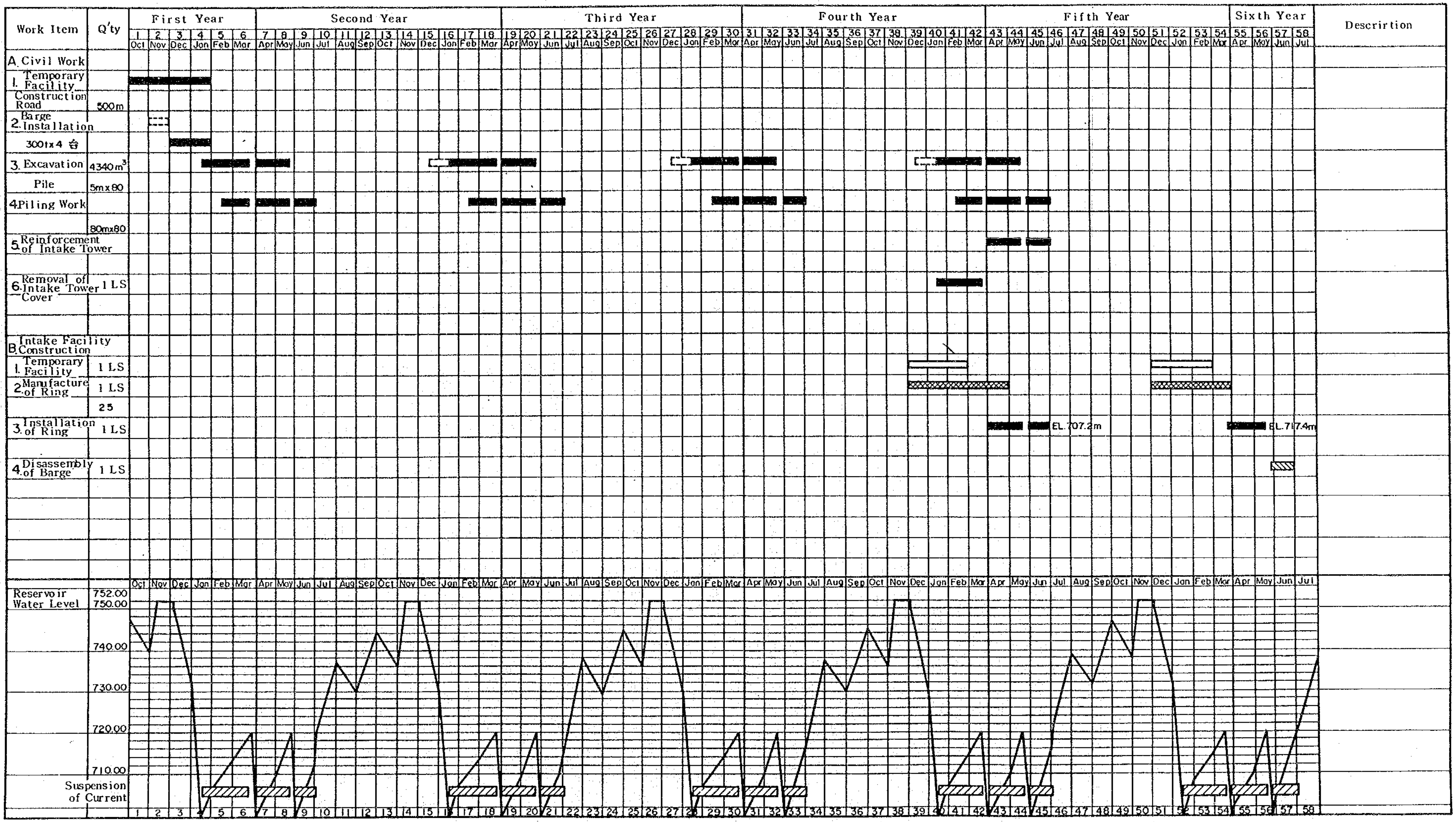
THE AMBUKLAO DAM REHABILTATION PROJECT	
HEIGHTENING OF THE EXISTING INTAKE TOWER PLAN	
SCALE 1:2000	DATE
THE JAPAN INTERNATIONAL COOPERATION AGENCY	



HWL.752.00
750
740
730
720
710
700
690
680
670
660
650
EL 717.00
D = 30.000
EL 695.165
LWL 694.00
LWL = 694.00
Existing facility
STEEL PIPE L = 80M
φ1,200, t = 16
FILLING WORK FOR REINFORCEMENT
@ 1.700
1.300
φ8.000
1.000
Free Joint
EL.695.65
SCALE
0 10 20M
1/500
SCALE
0 2 4M
1/100
THE AMBUKLAO DAM REHABILITATION PROJECT
HEIGHTENING OF THE EXISTING
INTAKE TOWER
PROFILE AND DETAIL
SCALE AS SHOWN
DATE
THE JAPAN INTERNATIONAL COOPERATION AGENCY

Fig -12.5

Fig — 12.6 Scheme B Heightening of the Existing Intake Tower



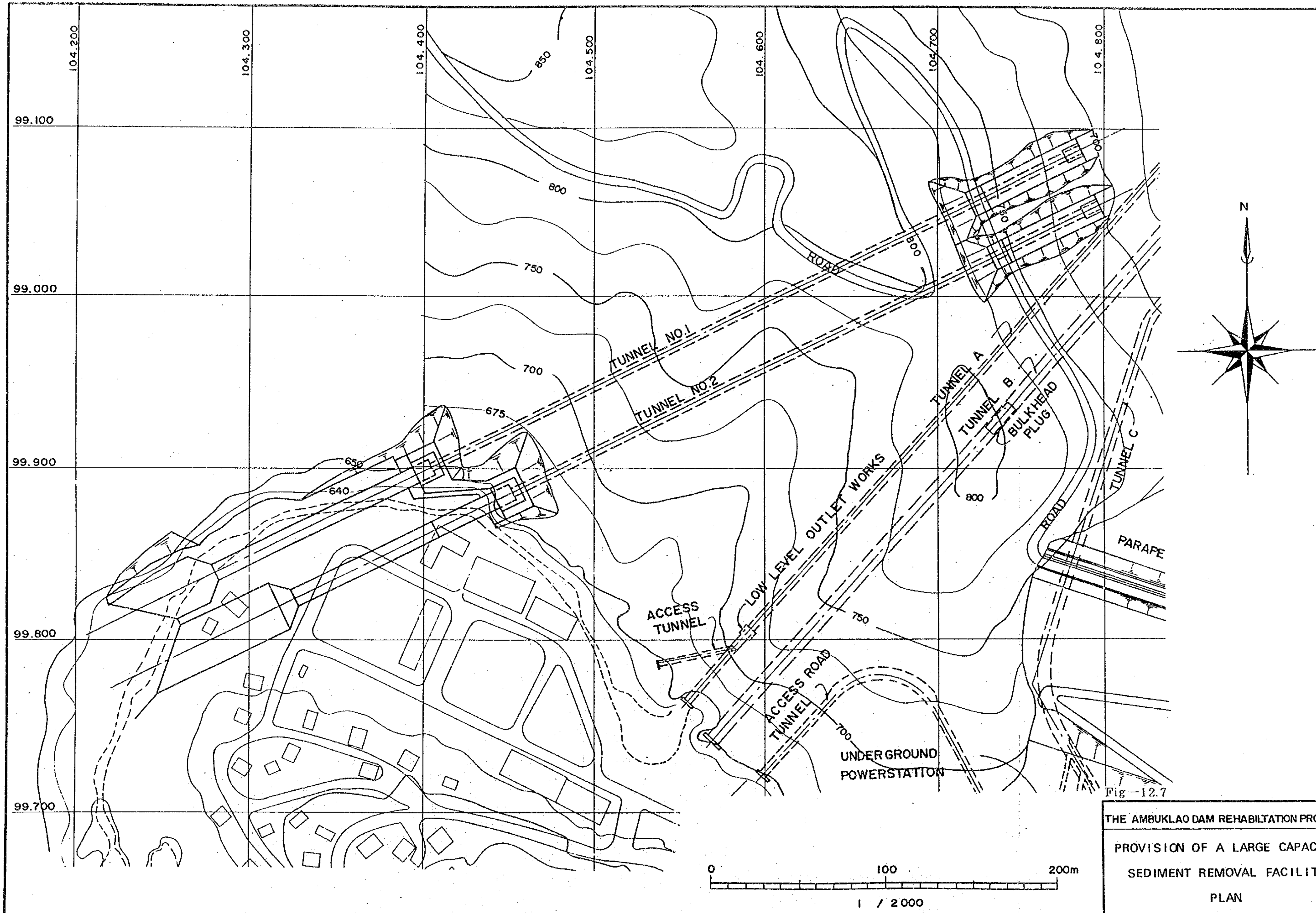


Fig −12.7

THE AMBUKLAO DAM REHABILTATION PROJECT	
PROVISION OF A LARGE CAPACITY SEDIMENT REMOVAL FACILITY PLAN	
SCALE 1:2000	DATE
THE JAPAN INTERNATIONAL COOPERATION AGENCY	

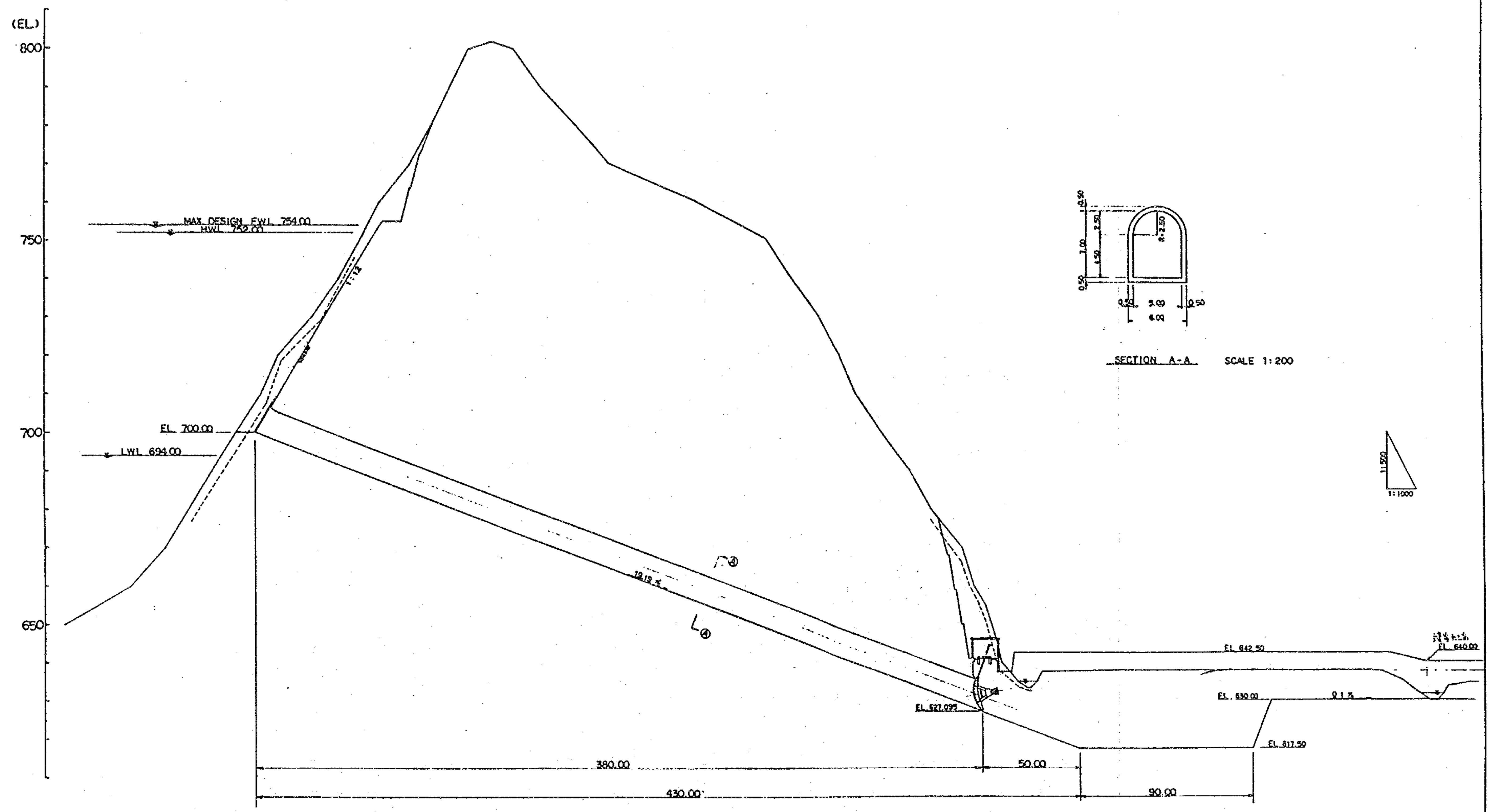


Fig −12.8

SCALE 0 10 20M 1/200

SCALE 0 50 100M 1/1000

THE AMBUKLAO DAM REHABILITATION PROJECT

PROVISION OF A LARGE CAPACITY SEDIMENT REMOVAL FACILITY PROFILE

SCALE As shown	DATE

THE JAPAN INTERNATIONAL COOPERATION AGENCY

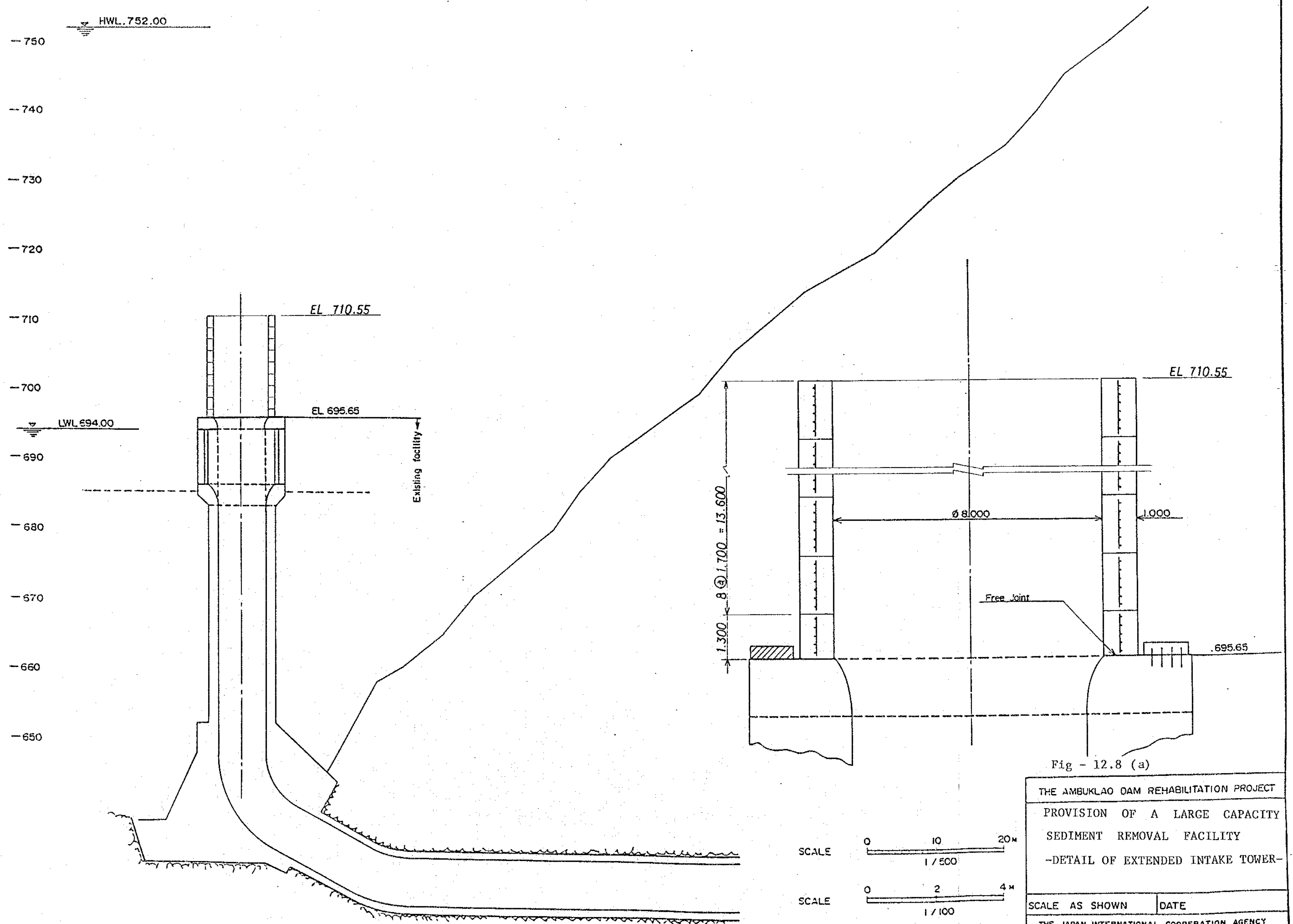


Fig - 12.8 (a)

THE AMBUKLAO DAM REHABILITATION PROJECT

PROVISION OF A LARGE CAPACITY SEDIMENT REMOVAL FACILITY
-DETAIL OF EXTENDED INTAKE TOWER-

SCALE AS SHOWN | DATE

THE JAPAN INTERNATIONAL COOPERATION AGENCY

Fig — 12.9 Scheme C Provision of a Large Capacity Sediment Removal Facility

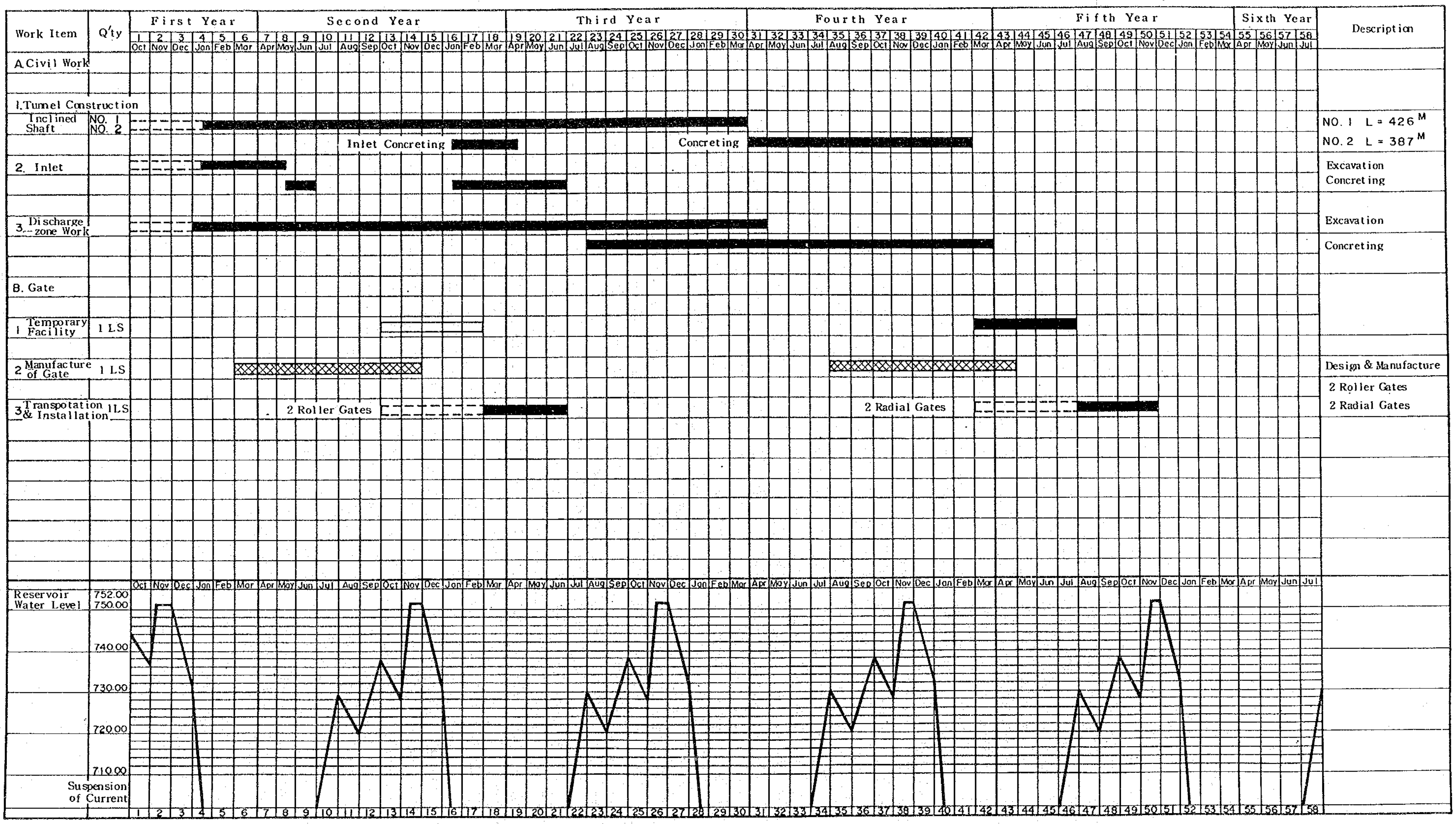


Fig —12.9

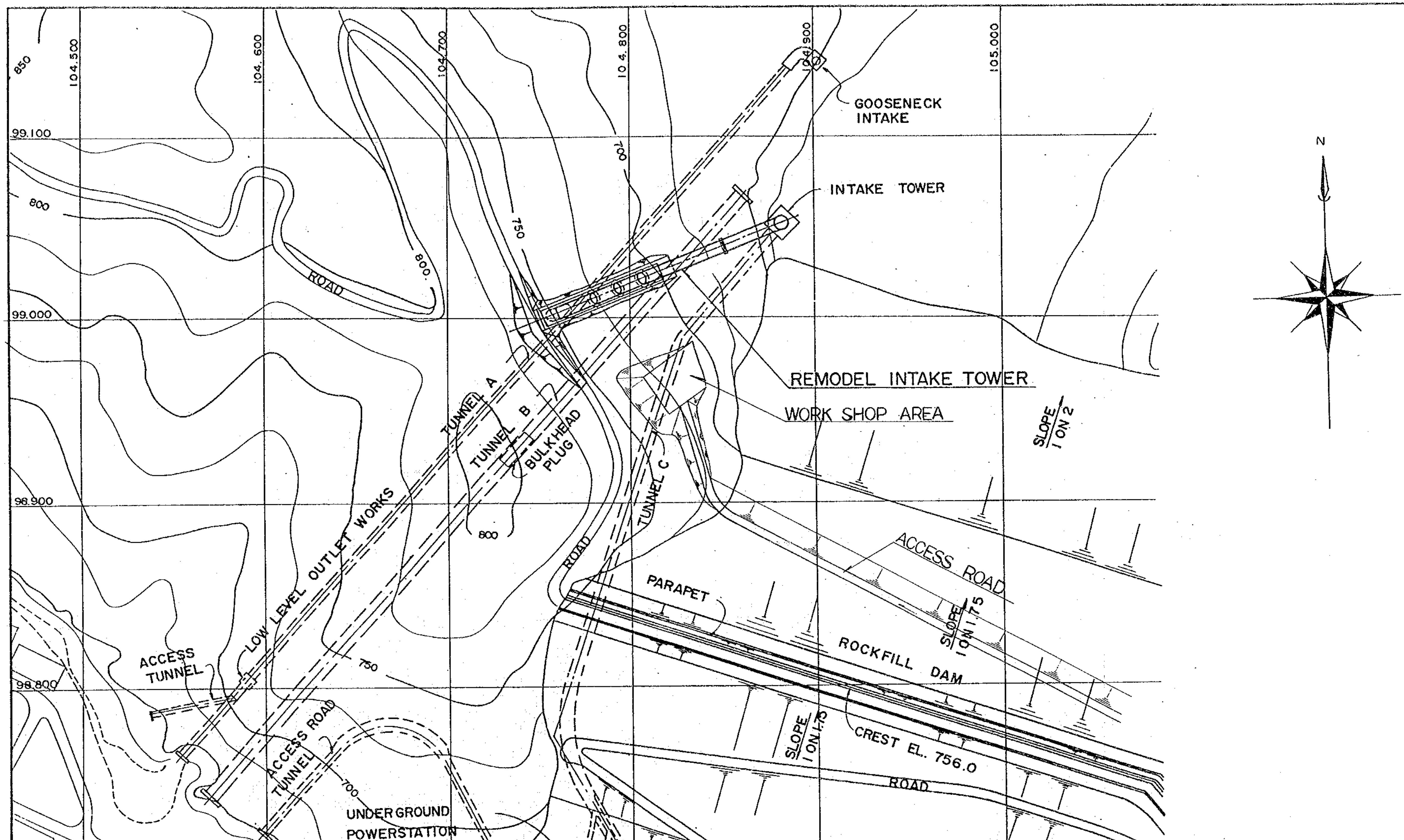


Fig – 12.10

0 100 200m

1 / 2000

THE AMBUKLAO DAM REHABILTATION PROJECT	
PROVISION OF A NEW INTAKE TOWER (SLIDE GATE) PLAN	
SCALE 1:2000	DATE
THE JAPAN INTERNATIONAL COOPERATION AGENCY	

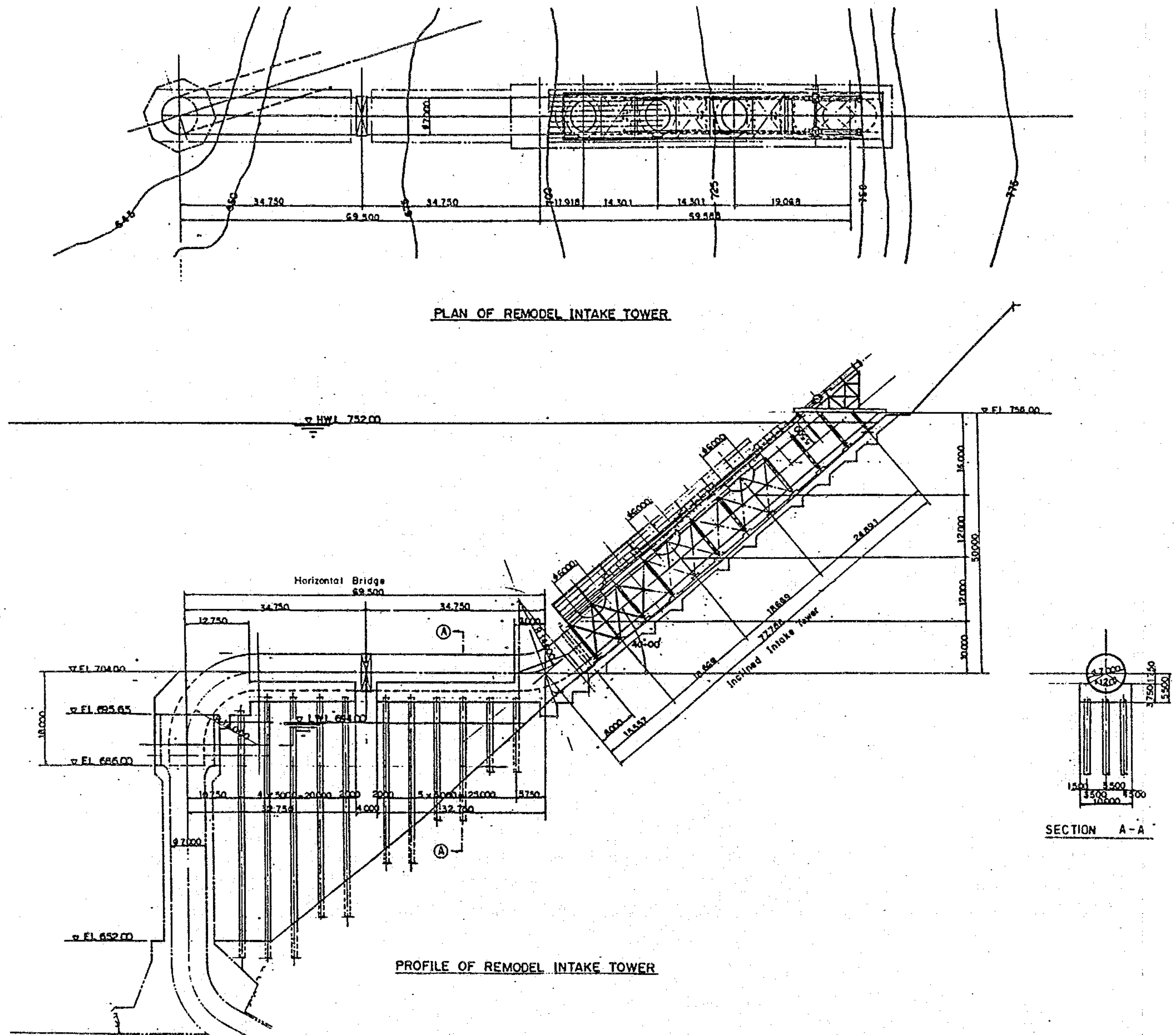
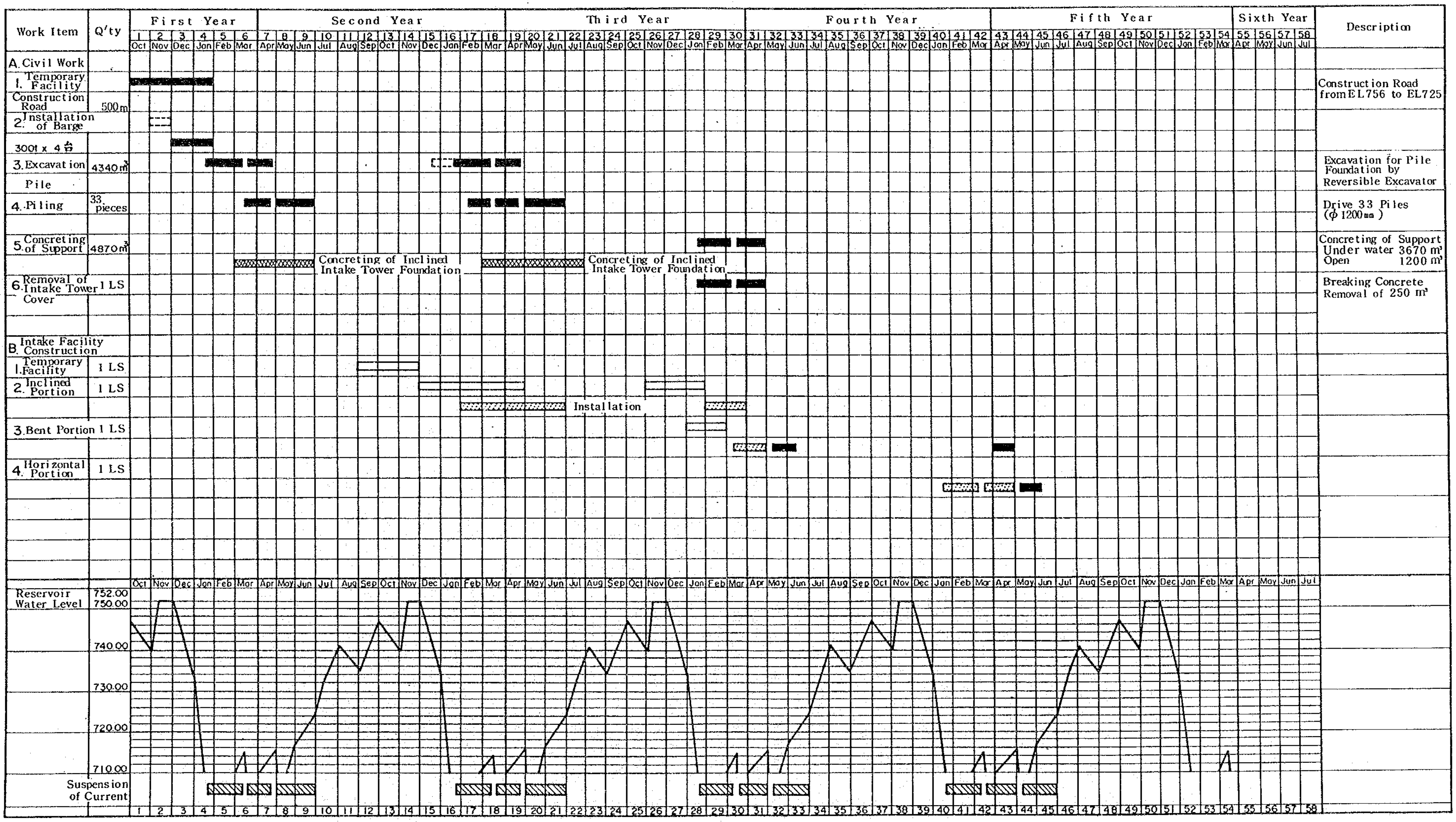


Fig — 12.11

THE AMBUKLAO DAM REHABILITATION PROJECT	
PROVISION OF A NEW INTAKE TOWER (SLIDE GATE)	
SCALE 1 : 400	DATE
THE JAPAN INTERNATIONAL COOPERATION AGENCY	

Fig－12.12 Scheme D Provision of a New Intake Tower (Slide Gate)



Fig－12.12

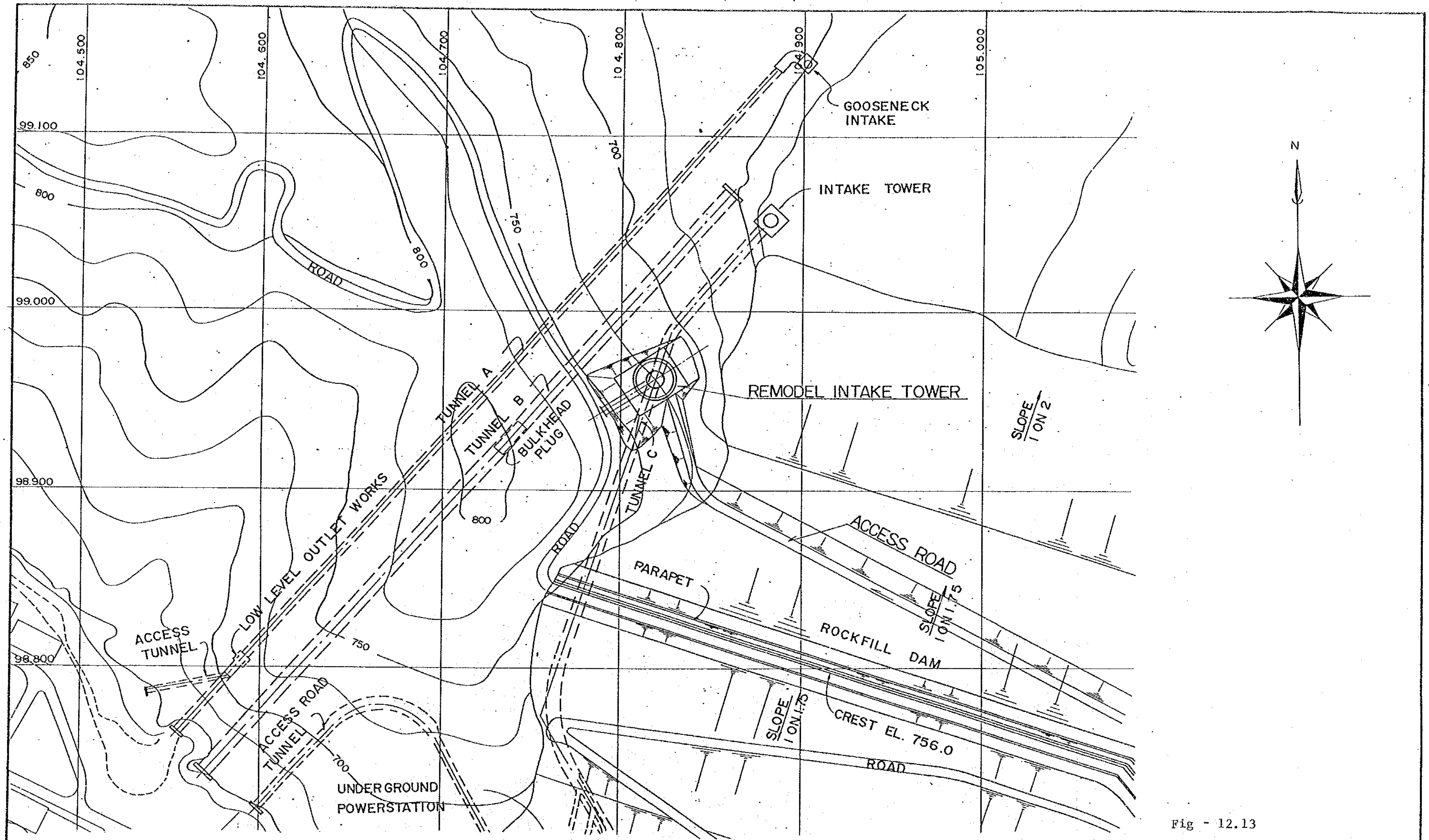


Fig - 12.13

THE AMBUKLAO DAM REHABILTATION PROJECT
VERTICAL INTAKE TOWER REMODEL PLAN (SLEEVE GATE) PLAN
SCALE 1:2000 | DATE
THE JAPAN INTERNATIONAL COOPERATION AGENCY

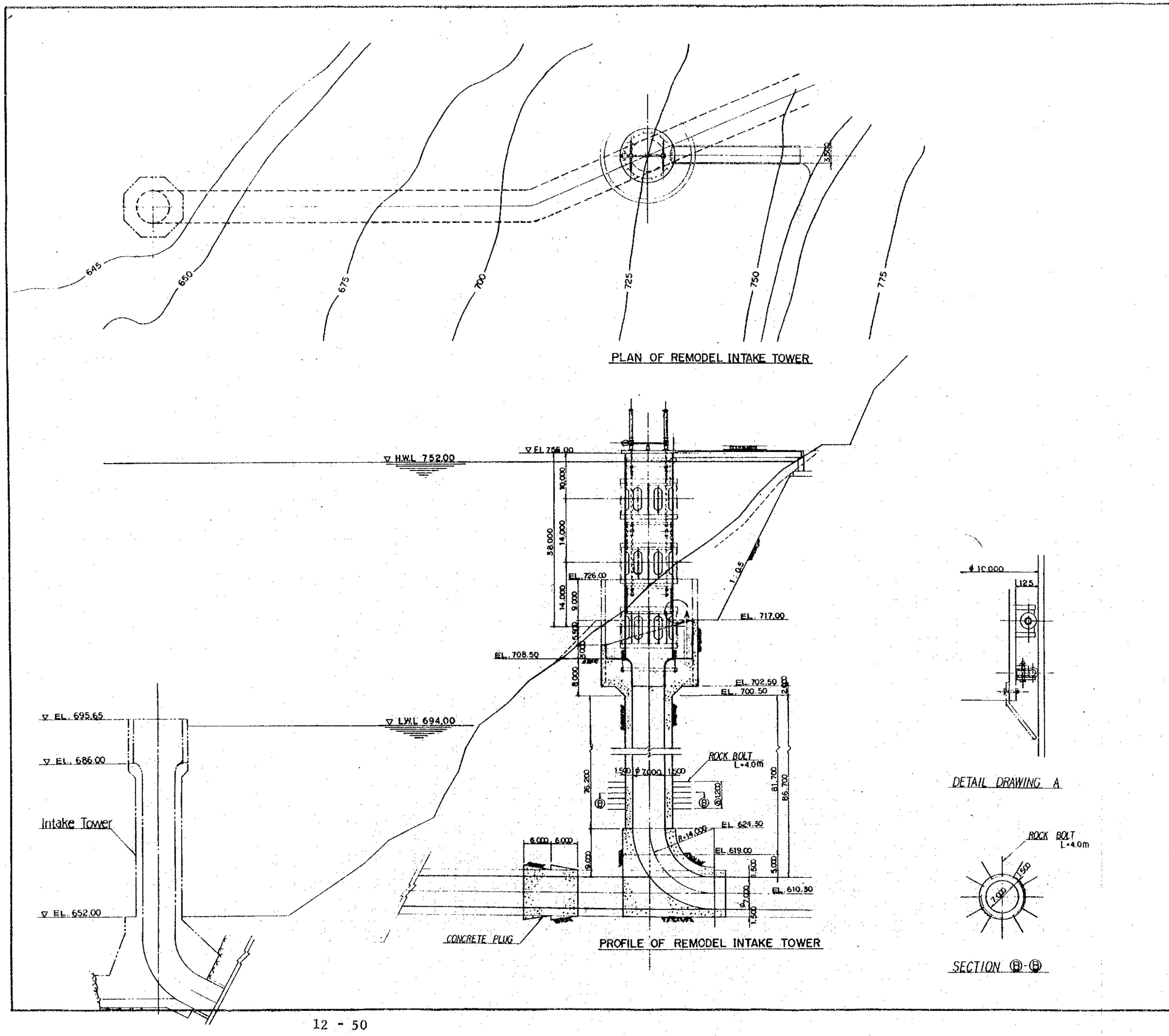
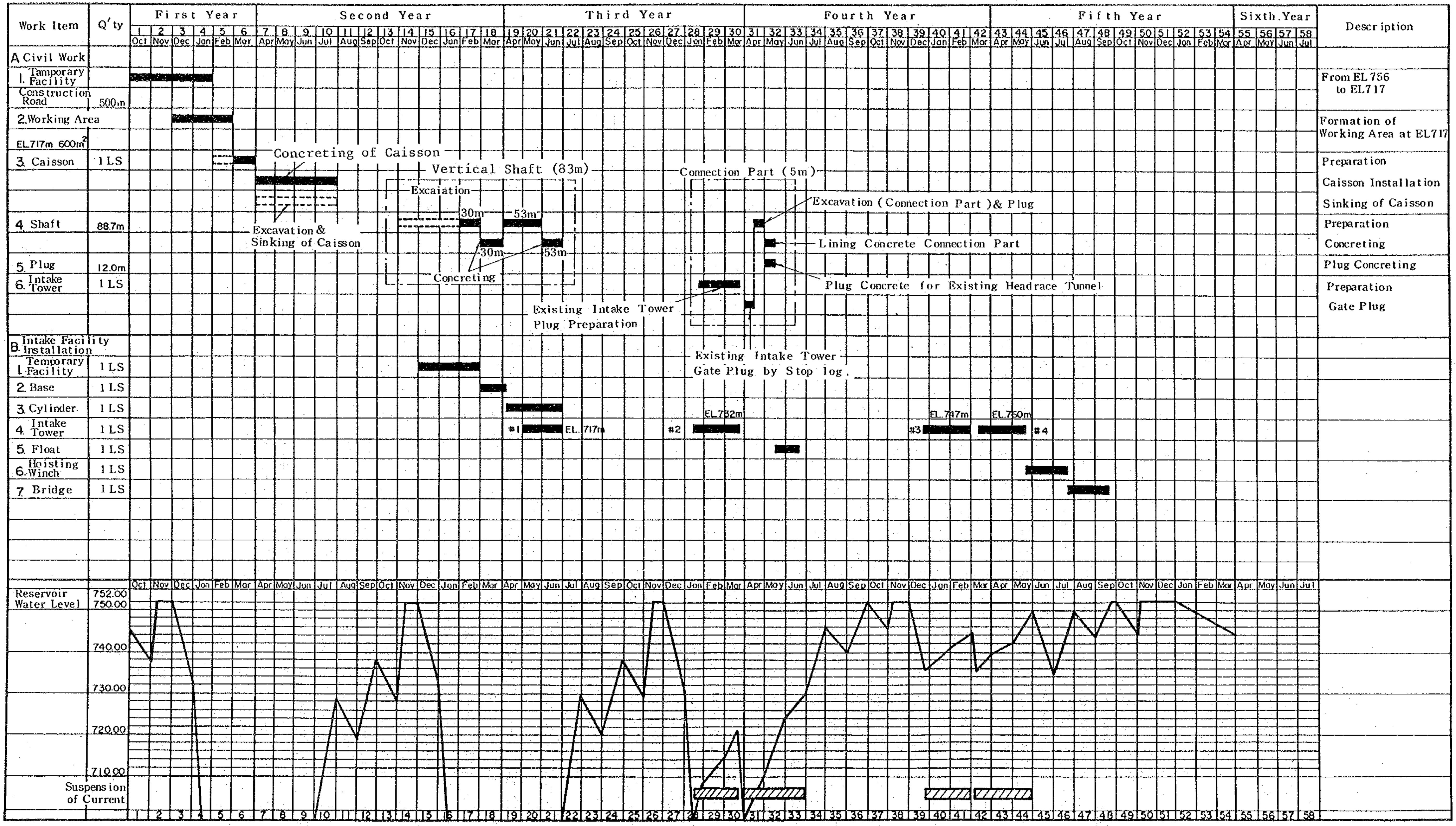


Fig - 12.14

THE AMBUKLAO DAM REHABILITATION PROJECT	
VERTICAL INTAKE TOWER REMODEL PLAN (SLEEVE GATE)	
SCALE 1:400	DATE
THE JAPAN INTERNATIONAL COOPERATION AGENCY	

Fig − 12.15　Scheme E Provision of a New Intake Tower (Sleeve Gate)



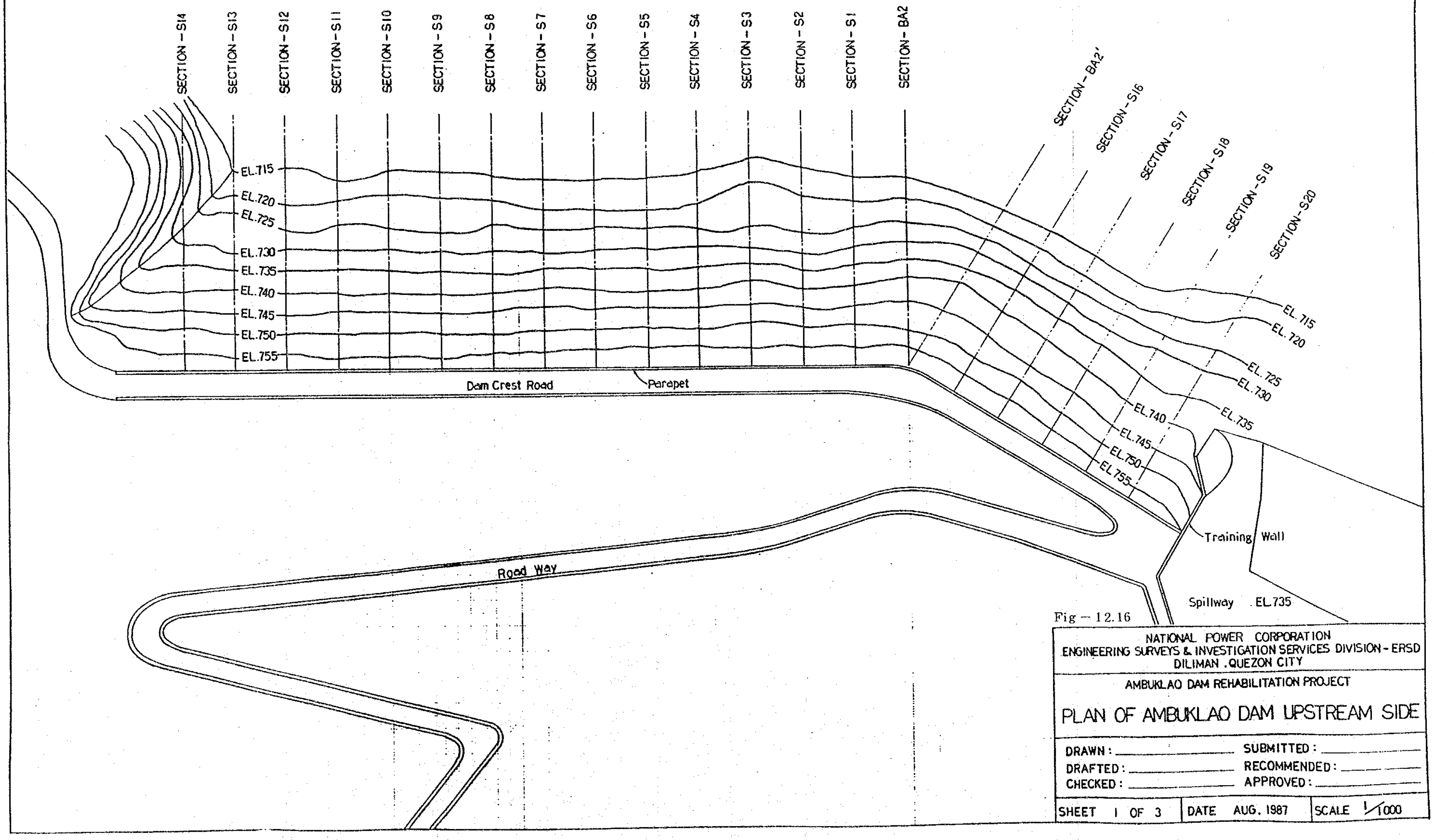
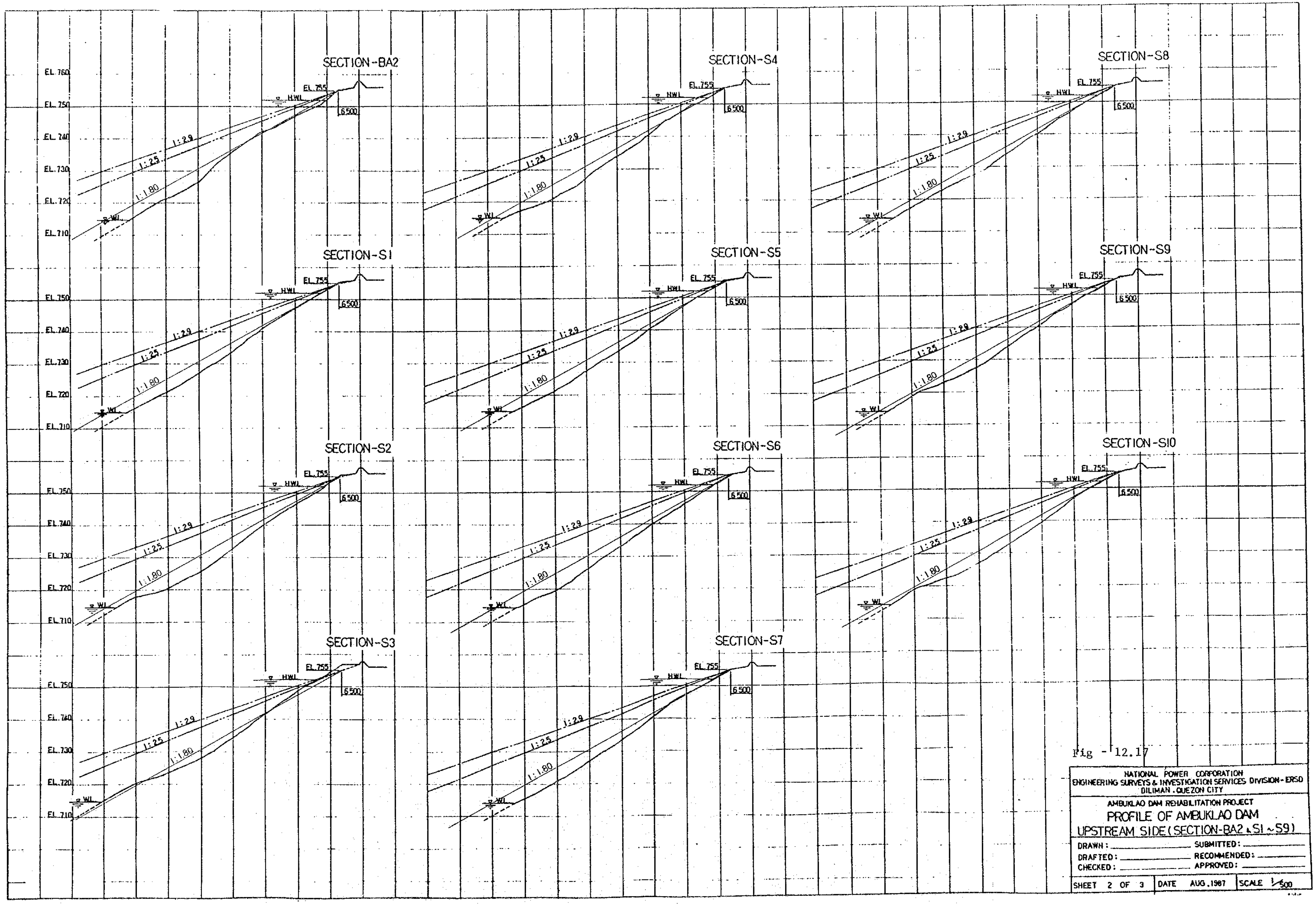
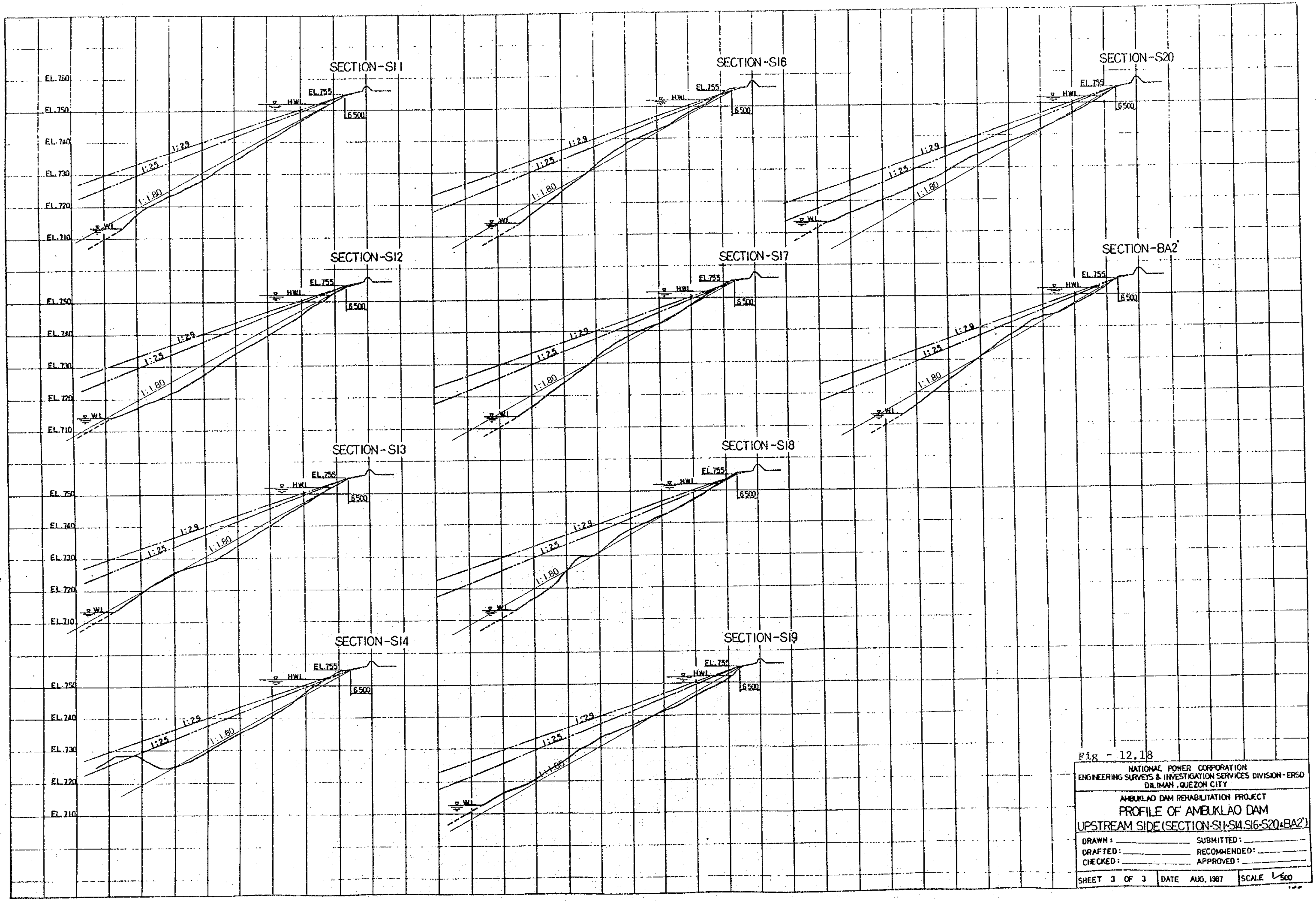


Fig — 12.16

NATIONAL POWER CORPORATION ENGINEERING SURVEYS & INVESTIGATION SERVICES DIVISION - ERSD DILIMAN, QUEZON CITY	
AMBUKLAO DAM REHABILITATION PROJECT	
PLAN OF AMBUKLAO DAM UPSTREAM SIDE	
DRAWN:	SUBMITTED:
DRAFTED:	RECOMMENDED:
CHECKED:	APPROVED:
SHEET 1 OF 3	DATE AUG. 1987
SCALE 1/1000	


SECTION-BA2
SECTION-S1
SECTION-S2
SECTION-S3
SECTION-S4
SECTION-S5
SECTION-S6
SECTION-S7
SECTION-S8
SECTION-S9
SECTION-S10
EL. 760
EL. 750
EL. 740
EL. 730
EL. 720
EL. 710
EL. 755
HWL
WL
6.500
1:2.9
1:2.5
1:1.80
NATIONAL POWER CORPORATION
ENGINEERING SURVEYS & INVESTIGATION SERVICES DIVISION-ERSD
DILIMAN, QUEZON CITY
AMBUKLAO DAM REHABILITATION PROJECT
PROFILE OF AMBUKLAO DAM
UPSTREAM SIDE (SECTION-BA2 & S1 ~ S9)
DRAWN :
DRAFTED :
CHECKED :
SUBMITTED :
RECOMMENDED :
APPROVED :
SHEET 2 OF 3
DATE AUG. 1987
SCALE 1/500
Fig - 12.17


SECTION-S11
SECTION-S12
SECTION-S13
SECTION-S14
SECTION-S16
SECTION-S17
SECTION-S18
SECTION-S19
SECTION-S20
SECTION-BA2'
EL.760
EL.750
EL.740
EL.730
EL.720
EL.710
EL.755
HWL
WL
6500
1:2.9
1:2.5
1:1.80
Fig - 12.18
NATIONAL POWER CORPORATION
ENGINEERING SURVEYS & INVESTIGATION SERVICES DIVISION-ERSD
DILIMAN, QUEZON CITY
AMBUKLAO DAM REHABILITATION PROJECT
PROFILE OF AMBUKLAO DAM
UPSTREAM SIDE (SECTION-S11-S14,S16-S20 & BA2')
DRAWN:
SUBMITTED:
DRAFTED:
RECOMMENDED:
CHECKED:
APPROVED:
SHEET 3 OF 3
DATE AUG. 1987
SCALE 1/500

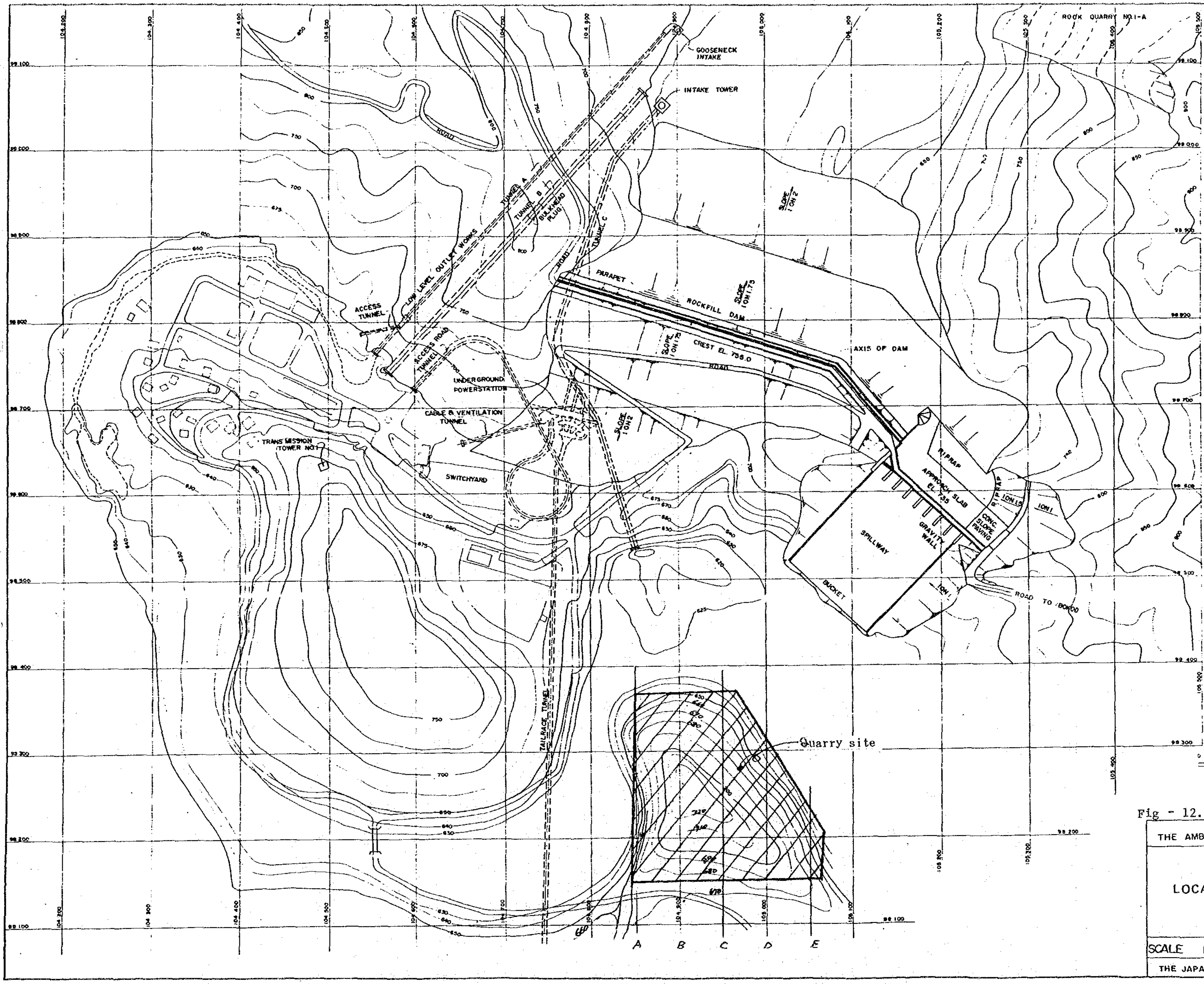


Fig - 12.19

THE AMBUKLAO DAM REHABILITATION PROJECT	
LOCATION OF QUARRY SITE	
SCALE 1:2000	DATE
THE JAPAN INTERNATIONAL COOPERATION AGENCY	

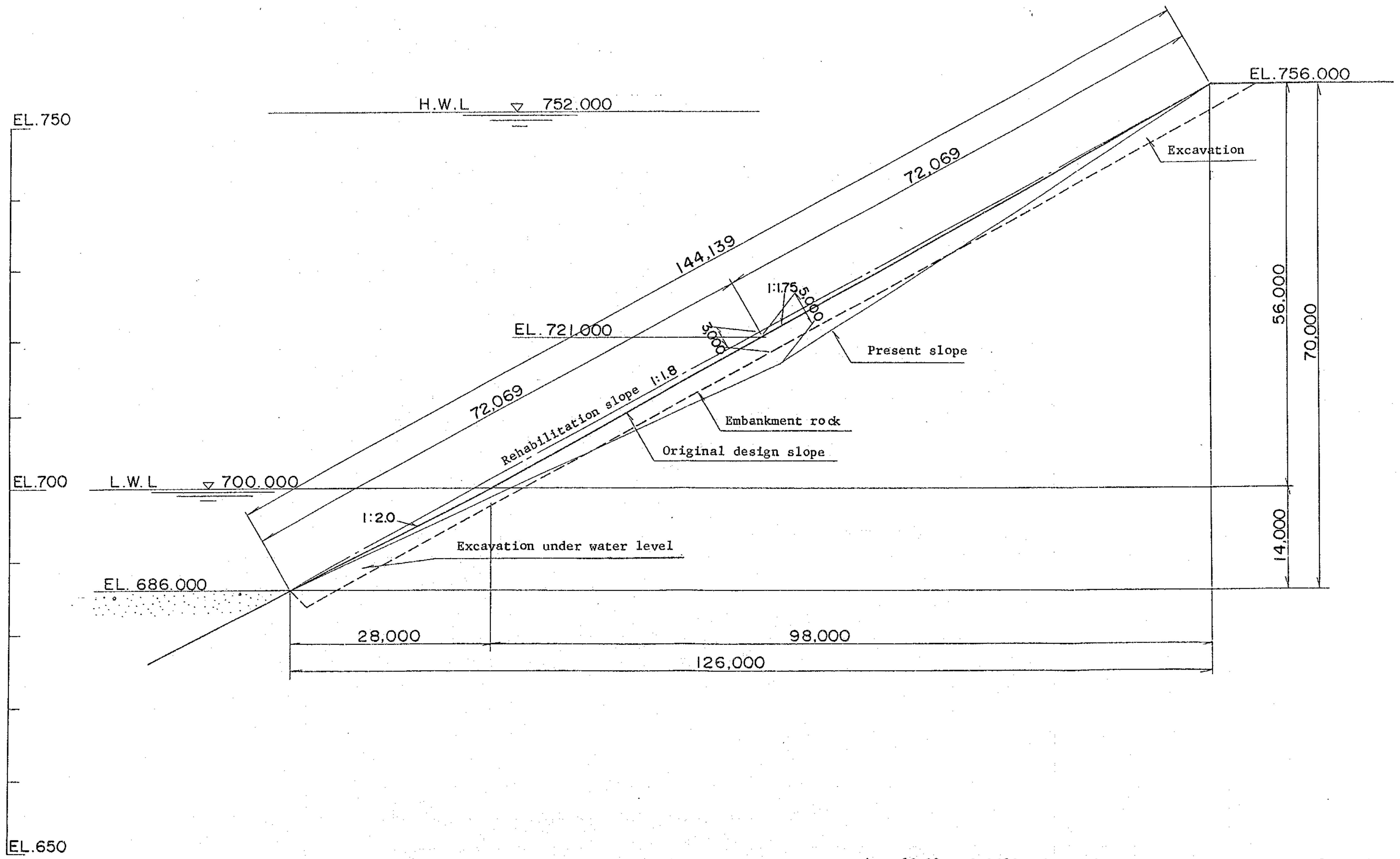


Fig - 12.20 Rehabilitation Work on Dam Upstream Face Typical Section
(Slope Gradient 1 : 1.8)

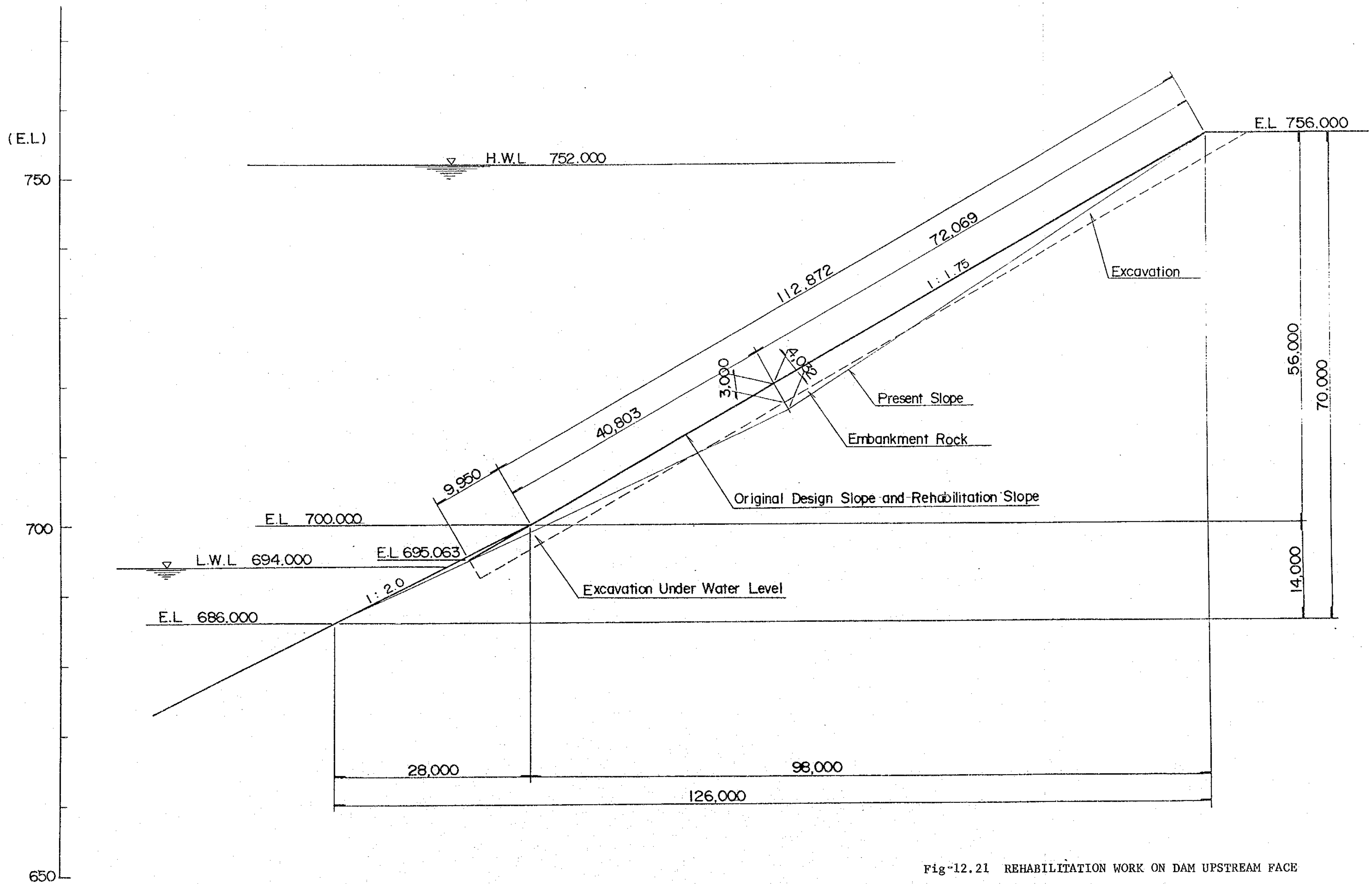
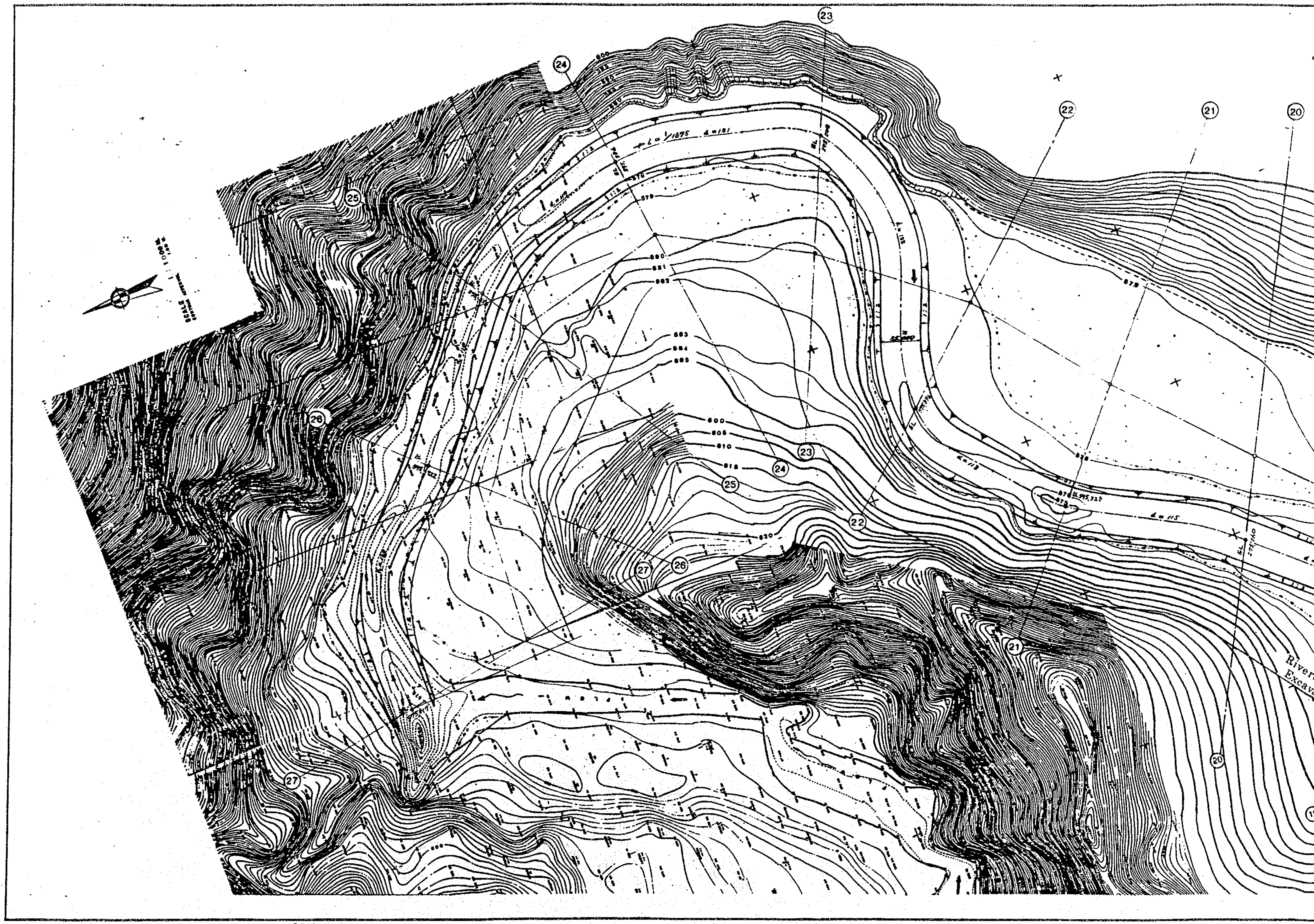
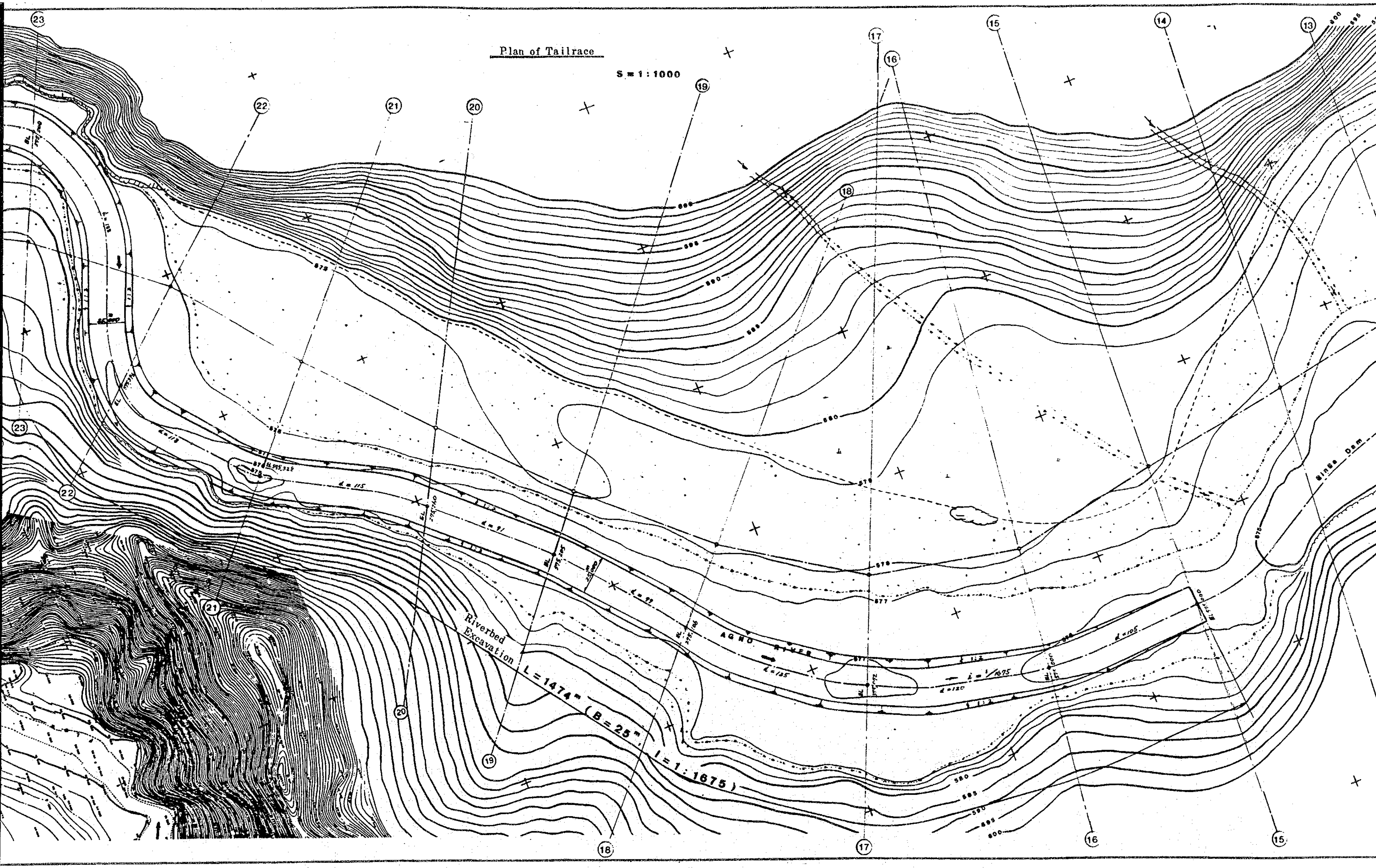


Fig-12.21 REHABILITATION WORK ON DAM UPSTREAM FACE
TYPICAL SECTION (SLOPE GRADIENT 1:1.75)



SCALE 1:1000
River
Excav


Plan of Tailrace
S = 1 : 1000
Riverbed Excavation L = 1474m (B = 25m I = 1 : 1675)
AGNO RIVER
Binga Dam

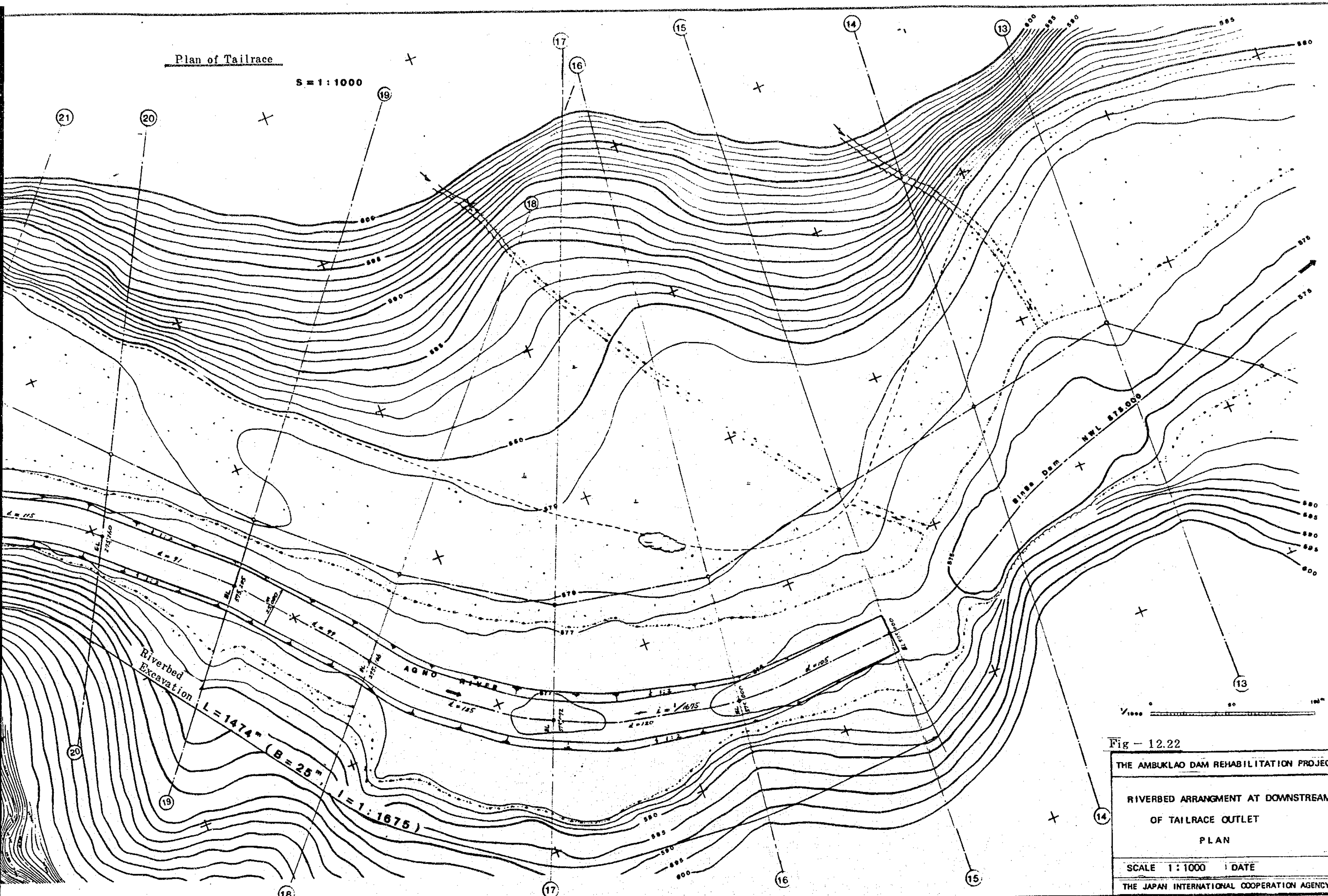


Fig − 12.22

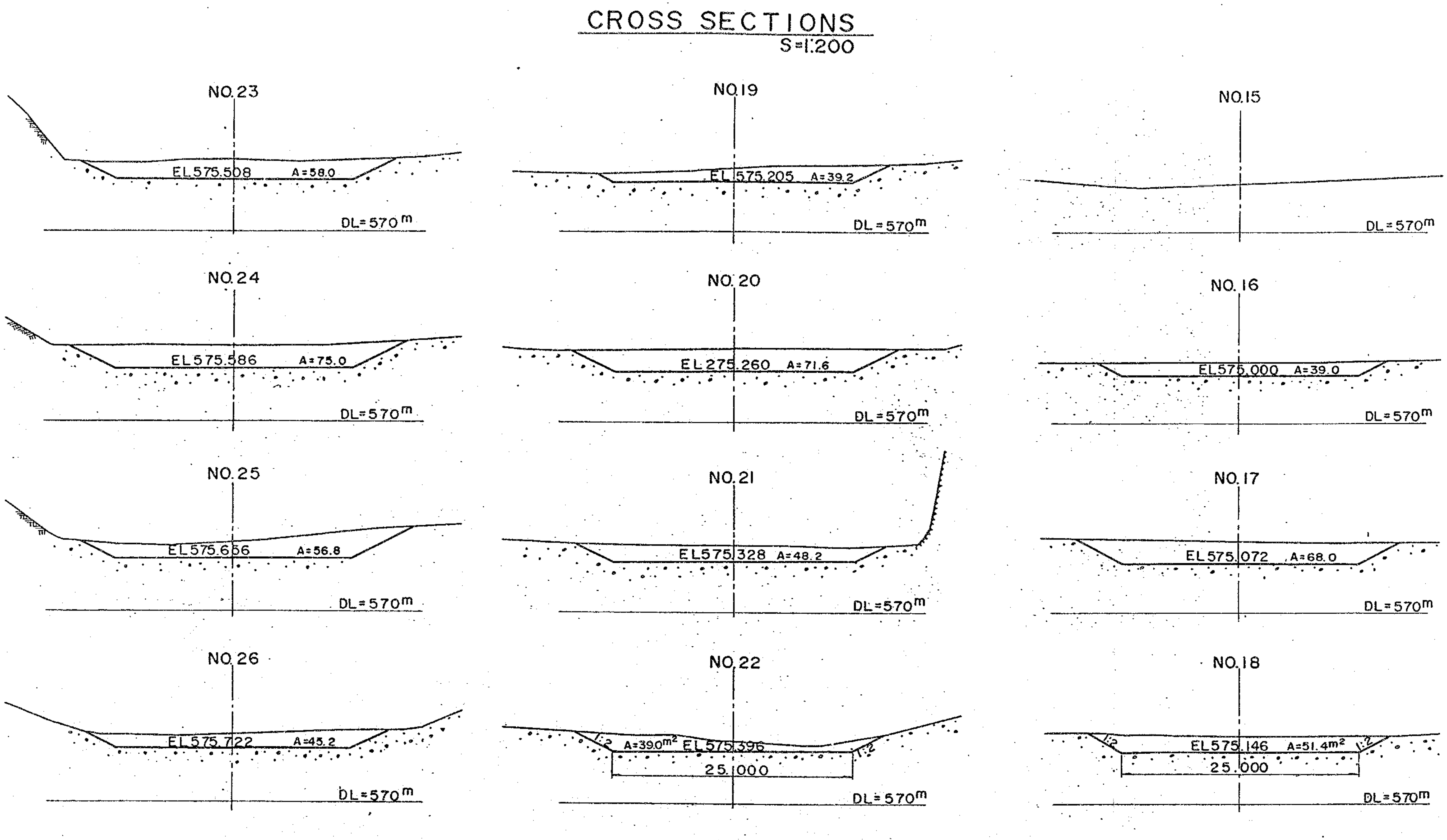
THE AMBUKLAO DAM REHABILITATION PROJECT

RIVERBED ARRANGMENT AT DOWNSTREAM OF TAILRACE OUTLET

PLAN

SCALE 1 : 1000 | DATE

THE JAPAN INTERNATIONAL COOPERATION AGENCY



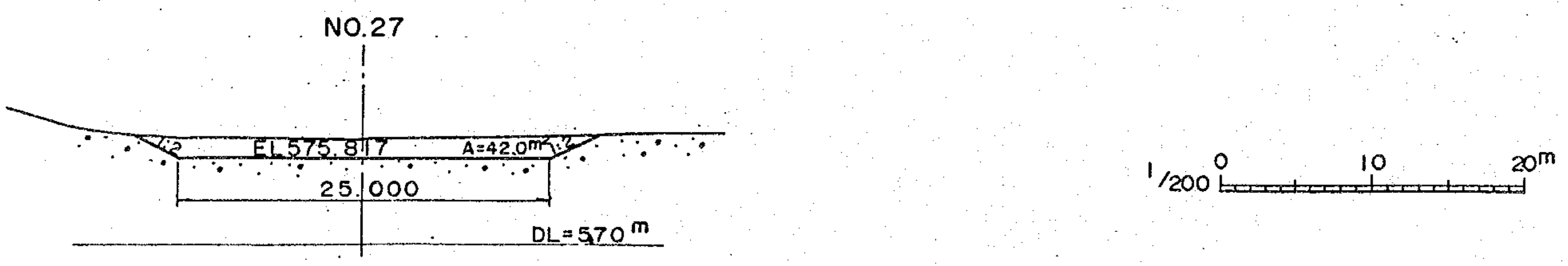
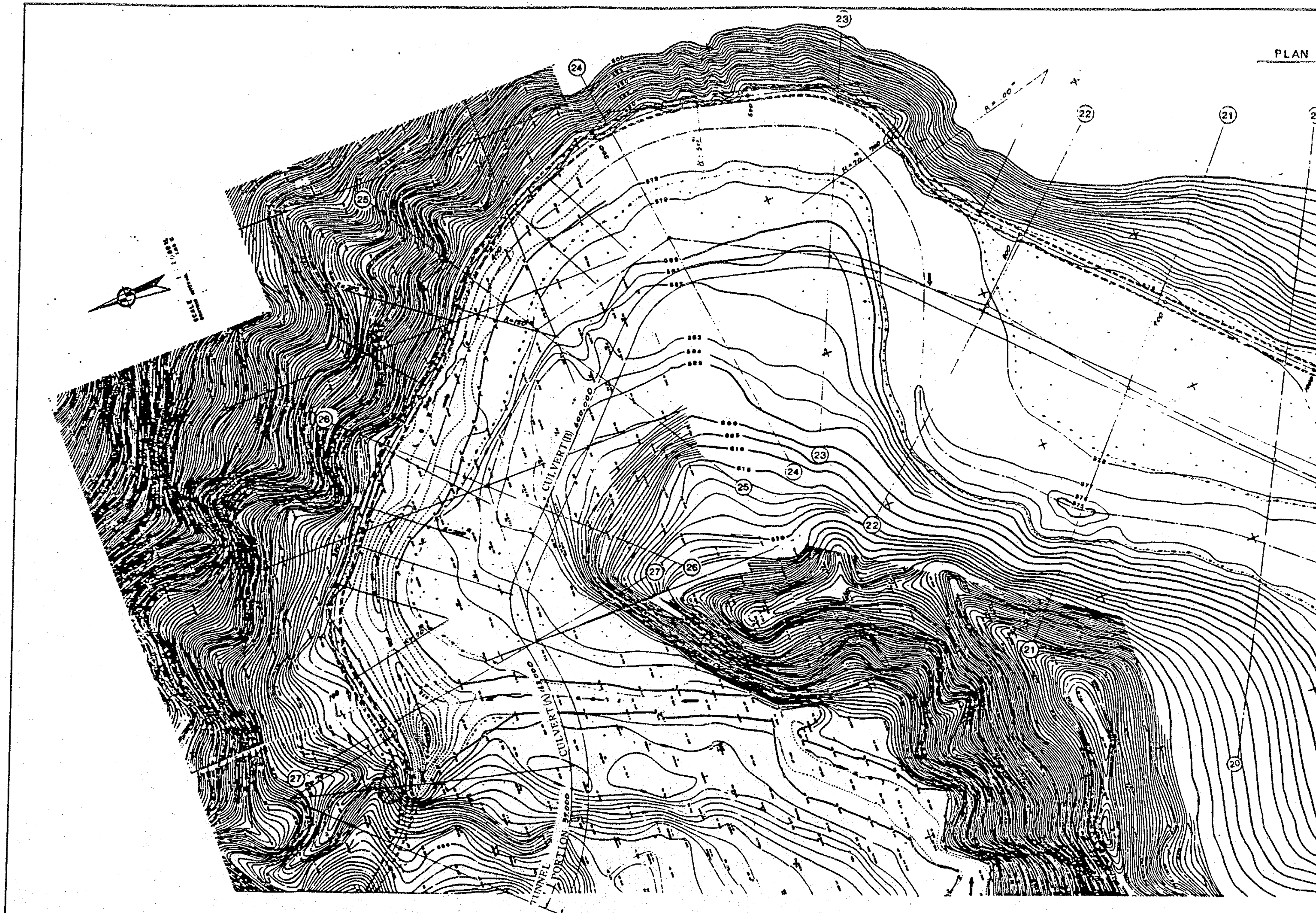


Fig − 12.23

THE AMBUKLAO DAM REHABILITATION PROJECT	
RIVER-BED EXCAVATION Cross sections	
SCALE 1 : 200	DATE
THE JAPAN INTERNATONAL COOPERATION	


PLAN
CULVERT (B)
CULVERT (A)
TUNNEL PORTION
20
21
22
23
24
25
26
27