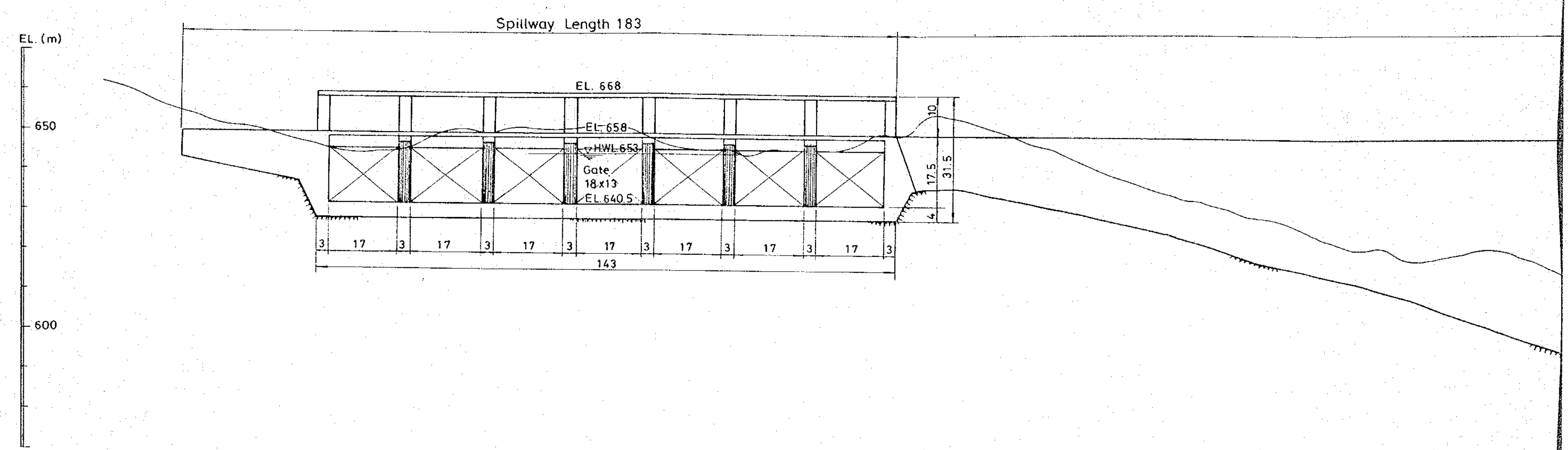
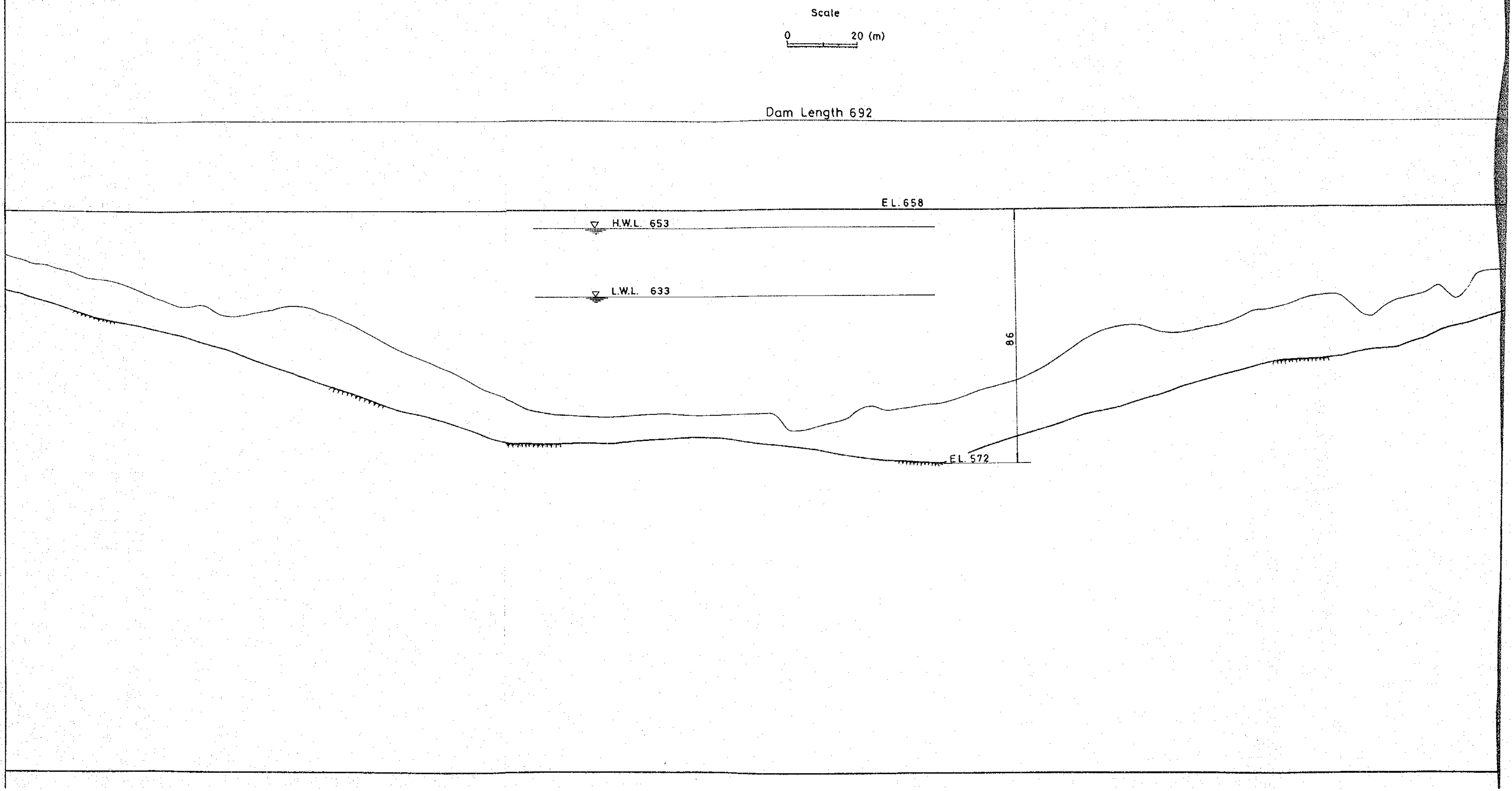
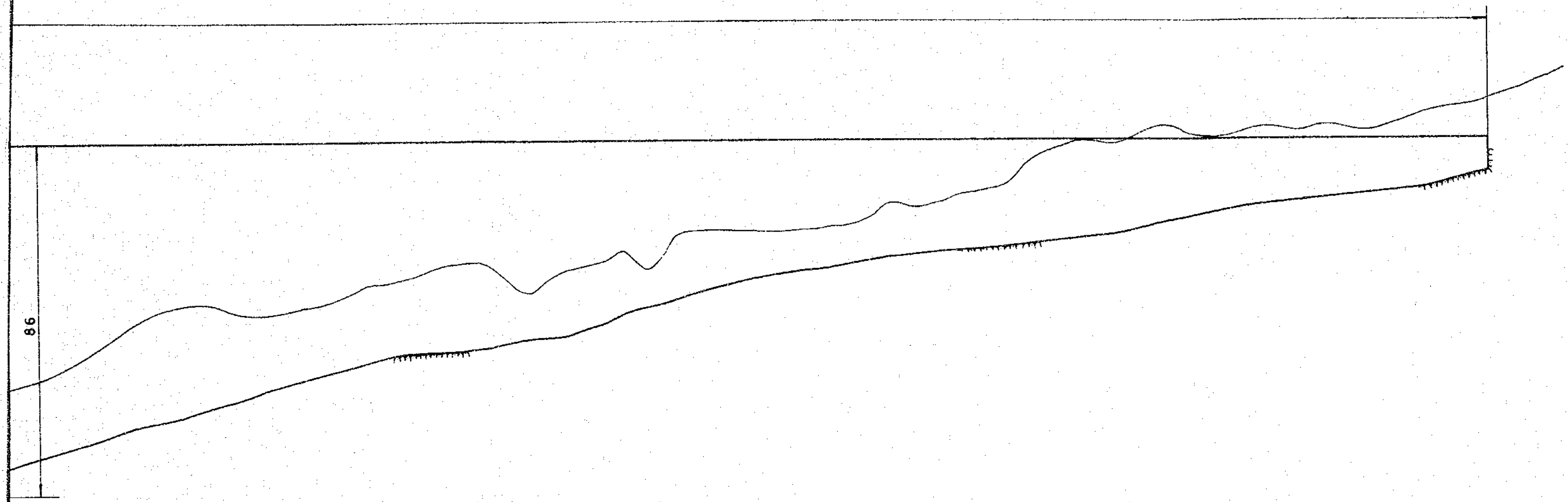

Spillway Length 183
EL. (m)
650
600
EL. 668
EL. 658
HWL 653
Gate
18x13
EL. 640.5
10
17.5
31.5
4
3
17
143

Longitudinal Section of Reservoir Dam and Spillway (No.2 Damsite)



ay (No.2 Damsite)



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Longitudinal Section of Reservoir Dam and Spilway (No. 2 Damsite)

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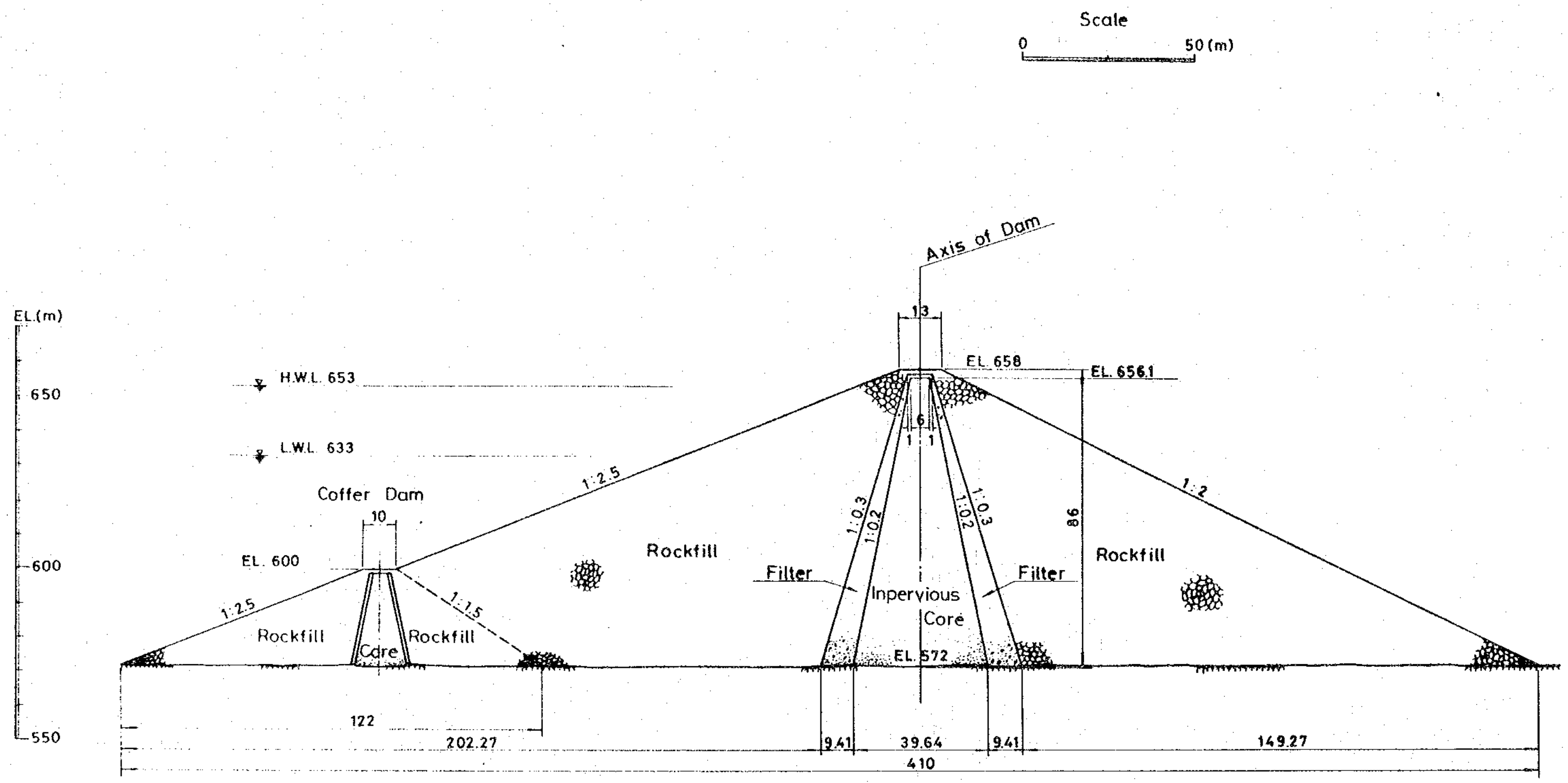
Cross-Section of Reservoir Dam (No.3 Damsite)



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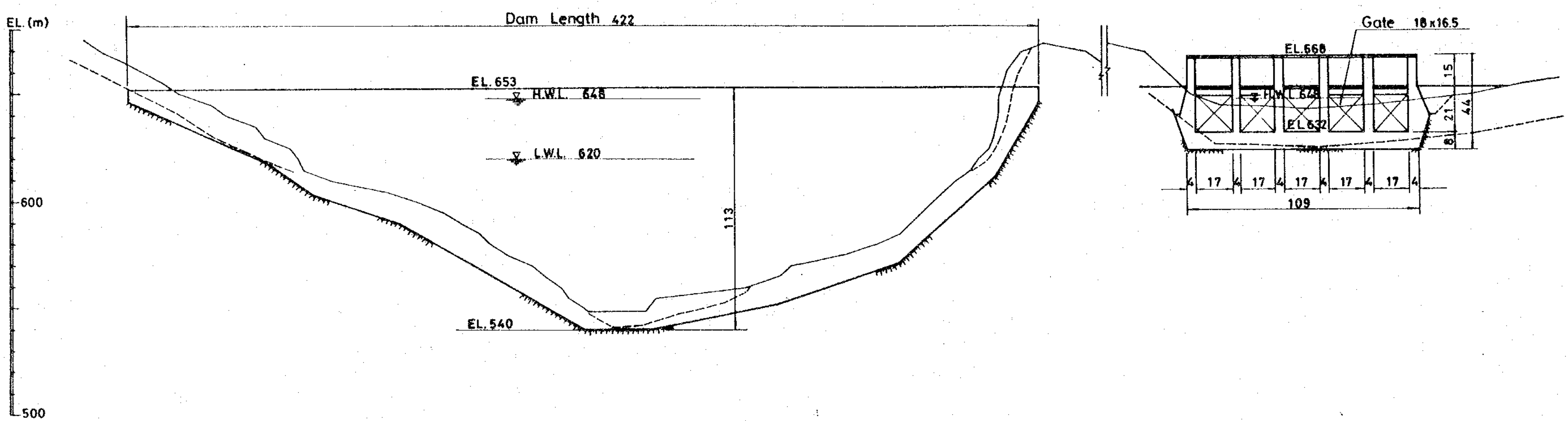
Cross-Section of Reservoir Dam
(No.3 Damsite, Fill Dam)

October 1980 Fig. 3-1-9

Longitudinal Section of Reservoir Dam and Spillway (No.3 Damsite)



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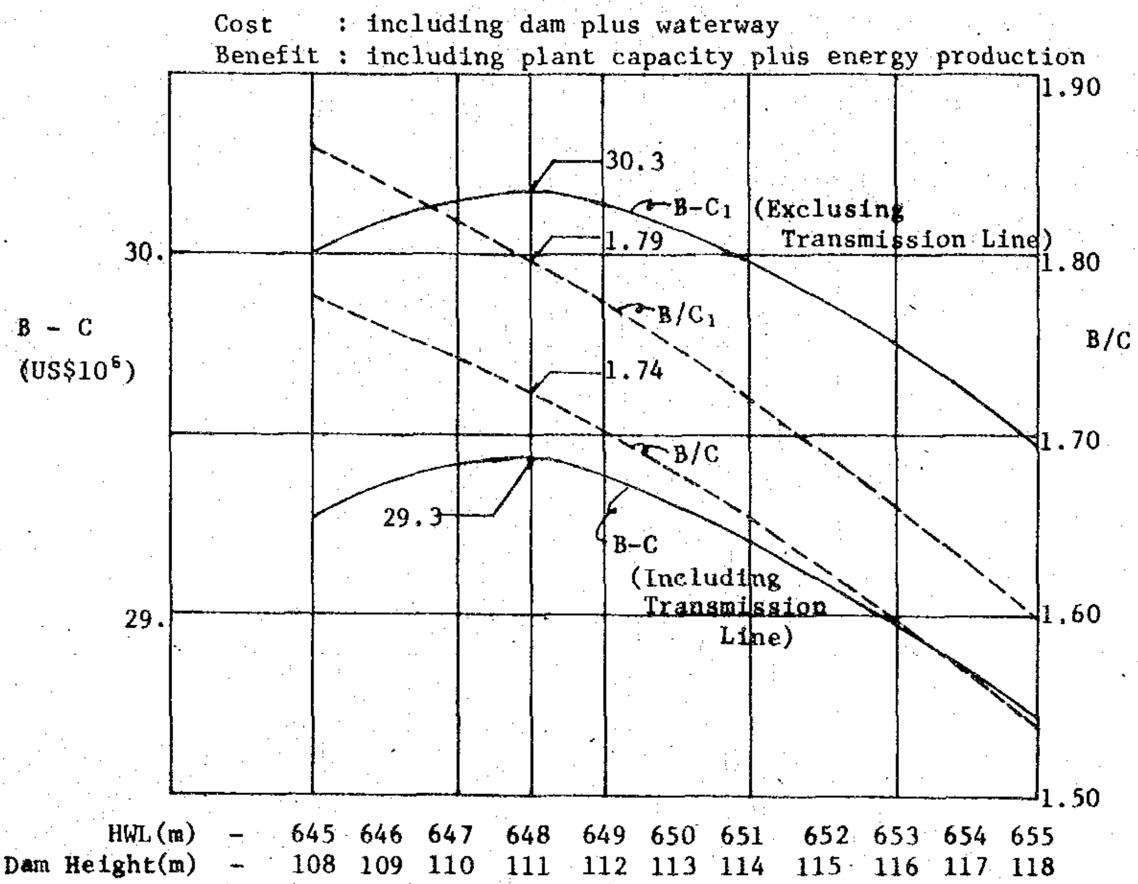
Longitudinal Section
of Reservoir Dam & Spillway
(No.3 Damsite, Fill Dam)

October	1980	Fig 3-1-10

Comparative Economic Analysis of Optimum Dam Height

No. 3 Damsite



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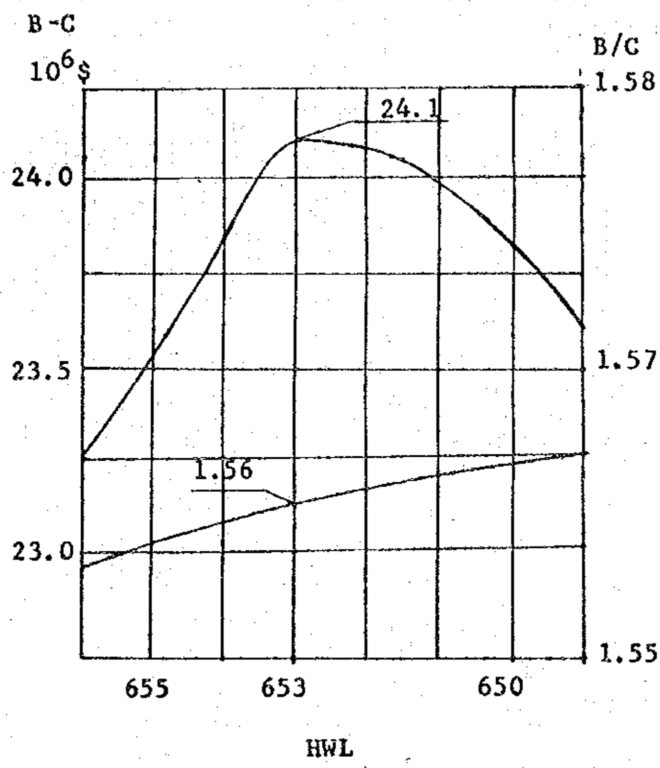
Comparative Economic Analysis of Optimum Dam Height (No. 3 Damsite)	
October 1980	Fig. 3-1-11

No.2 Damsite



for Total Construction Cost

Diduyon Hydroelectric Project Upper Cagayan River Republic of the Philippines	
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Comparative Economic Analysis of Optimum Dam Height (No.2 Damsite)	
October 1980	Fig. 3-1-12

3.2. Design of Principal Components of the Project

3.2.1. Calculation of storage reservoir operation and generated energy

The operation of the storage reservoir and the generated energy are calculated as follows:

(1) Turbine discharge

Symbol:

Qfirm:	Firm turbine discharge
Qmax :	Maximum turbine discharge
Qp(i):	Power generation turbine discharge
Qi(i):	Inflow to the storage dam
Qo(i):	Overflow discharge from storage reservoir
QD(i):	Discharge supplied from storage reservoir
VO(i):	Storage reservoir volume

a) Assuming that the storage reservoir is full at the moment i:

i) **When power is generated with the storage reservoir at the full water level we have:**

① When $Qi(i + 1) \geq Qmax$

$Qp(i + 1) = Qmax$

$Qo(i + 1) = Qi(i + 1) - Qmax$

$QD(i + 1) = 0$

$V(i + 1) = 0$

② When $Qfirm < Qi(i + 1) < Qmax$

$Qp(i + 1) = Qi(i + 1)$

$Qo(i + 1) = 0$

$QD(i + 1) = 0$

$V(i + 1) = 0$

ii) When power is generated with supply from the storage reservoir we have:

When $Qi(i + 1) \leqq Qfirm$

$QP(i + 1) = Qfirm$

$Qo(i + 1) = 0$

$QD(i + 1) = Qfirm - Qi(i + 1)$

$V(i + 1) = QD(i + 1) \times \Delta t$

b) **When the storage reservoir is not full at the moment i,** calculation is carried out assuming that inflow to the river $Qi(i + 1)$ is temporarily stored in the storage reservoir.

i) When the storage reservoir becomes full.

(1) When $Qi(i + 1) \times \Delta t - v(i) \geqq Qmax \times \Delta t$

$Qp(i + 1) = Qmax$

$Qo(i + 1) \times \Delta t = [Qi(i+1) \times \Delta t - V(i)] - Qmax \times \Delta t$

$V(i + 1) = 0$

$QD(i + 1) = 0$

(2) When $Qfirm \times \Delta t < Qi(i+1) \times \Delta t - V(i) < Qmax \times \Delta t$

$QP(i+1) \times \Delta t = Qi(i+1) \times \Delta t - V(i)$

$QD(i+1) = 0$

$V(i+1) = 0$

ii) When power is generated with supply from the storage reservoir.

When $Qi(i+1) \times \Delta t - V(i) < Qfirm \times \Delta t$

$QP(i+1) = Qfirm$

$Qo(i+1) = 0$

$QD(i+1) \times \Delta t = [Qfirm - Qi(i+1)] \times \Delta t + V(i)$

$V(i+1) = QD(i+1) \times \Delta t$

Taking into consideration discharge from the upper dam, the inflow to the lower dam is assumed to be Discharge Qo(i) from the upper storage reservoir, the turbine discharge Qp(i) of the upper dam, and the inflow Qio(i) from the remnant river basin. Consequently, the inflow Qi(i) to the lower dam can be given by the following expression:

$$Qi(i) = Qio(i) + Qo(i) + QP(i)$$

(2) **Calculation of output and generated energy**

Output: P

$$P = 9.8 \times \eta_g \times He \times Qp$$

Where: η_g : Composite efficiency of turbine-generator

He : Effective head (storage reservoir water level - discharge water level - head loss)

QP : Generator turbine discharge

Generated energy: E

$$E = 9.8 \times \eta_g \times H \times Qp \times T$$

Where:

T: time

The tabulation of calculated monthly and annual energy generation is given in Table 3-2-1, with an average of 956.8 GWh for 16 years of records.

3.2.2. Design of Dam Structure

(1) Determination of dam center line

The dam center line shall be determined by taking into consideration two aspects, i.e., the situation of the dam foundation rockbed and flood handling. However, at the present damsite, the hydraulic conditions are very severe, because the river flow is bent to the left with a steep angle immediately downstream of the dam site, and in addition the river width immediately downstream of the dam is very narrow.

The possibility of adoption of a horizontal apron was studied but this alternative is not recommendable from the economic point of view because the volume of the civil engineering and concrete work of the stilling pool could be huge. Thus, a combination of backwatering by means of downstream subsidiary dam and underwater buckets is adopted in order to cope with energy dissipation needs.

It is assumed that if the downstream subsidiary dam is adequately located, it will be possible to cope with both energy dissipation and change of the river flow direction at the same time. A large scale three dimensional hydraulic experiment will be required in order to verify and confirm the appropriateness of this alternative. Thus, it will be necessary to carry out hydraulic experiments prior to defining the overall design. In addition, since the design flood discharge is very large compared with the river width, the river width will be insufficient if the overall discharge is handled with overflow only, being consequently required to cut out a considerable volume of the natural bedrock. However, that will mean a considerable increase in the construction cost and difficulty in working conditions. Thus, it is assumed that part of the flood discharge will be handled by converting diversion tunnels into supplementary tunnel spillways.

(2) Design of the saddle dam at the right bank ridge

The results of geological studies indicate that the rockbed height of the right bank ridge is located under the full water level, and in addition the rockbed is of D (inferior) grade, being therefore inadequate as the foundation of a high, rigid structure. Consequently, the suddle dam to be constructed here will be a fill dam of low height, and the foundation rockbed will be reinforced for strength and impermeability by means of grouting.

As for banking, it is desirable to use materials available in the neighbourhood of the damsite, but the results of the tests carried out with samples collected nearby indicate that materials widely distributed over the neighbouring areas present many problems for construction of high dams, e.g., high natural humidity, low value of maximum dry density, high plasticity and so forth. However, the construction of a saddle dam of reduced height is assumed to be possible using weathered materials such as diorite.

(3) As a result of the considerations above, a design flood discharge of 8,900m^3/sec will be adopted for the concrete gravity dam spillway at the No.3 damsite.

As for the design of the capacity of diversion tunnels during the construction of reservoir dams, a flood discharge of the 2-year return period, i.e., 1,900 m^3/sec, will be adopted for a concrete gravity dam at the No.3 damsite, while those of the 20-year return period are adopted for dams of fill type, i.e., 5,500 m^3/sec. at the No.2 damsite and 5,600 m^3/sec. respectively.

(4) Design and division of flood spillways

At the damsite No.3, the river width of 40m is very narrow compared with the design flood discharge of $8{,}900m^3$/sec. If the whole flood discharge is designed to overflow the normal dam crest, the necessary width has to be of the order of 100m, and this will require the cutting of an ample volume of the bedrock located downstream of the dam, resulting in huge increase of construction cost and difficult working conditions. In view of the considerations above, part of the flood volume is designed to be handled utilizing temporary diversion tunnels at the final stage of the construction period in addition to the overflow from the dam crest gates.

Consequently, the flood spillway will be composed of the two parts mentioned below.

(i) Chute type flood spillway from the crest gates of 5,100 m^3/sec. capacity.

(ii) Tunnel type flood spillway of 3,800 m^3/sec. capacity, equipped with gates and shafts to guide the discharge into the diversion tunnels.

Capacity of these spillways is checked as given below.

- Overflow capacity from the chute type spillway:

$$Q_1 = C \cdot B \cdot H^{\frac{3}{2}}$$

where, $C = 1.974$
$B = 15 \text{ m} \times 3 = 45 \text{ m}$
$H = 15 \text{ m}$

$$= 1.974 \times 45 \text{ c } (15)^{\frac{3}{2}}$$

$$= 5{,}160 \text{ m}^3/\text{sec.} \;>\; 5{,}100 \text{ m}^3/\text{sec.}$$

- Capacity of the tunnel spillways:

$$Q = A\sqrt{2gH}$$

where, $A = \frac{\pi r^2}{2} + 3.86 \text{ m} \times 7.72 \text{ m} = 53.2 \text{ m}^2$
$H = \text{EL } 633 \text{ m} - \text{EL } 541 \text{ m} = 92 \text{ m}$

$$= 53.3\sqrt{2 \times 9.8 \times 92}$$

$$= 2{,}259 \text{ m}^3/\text{sec.}$$

$$Q_2 = 2Q = 4{,}518 \text{ m}^3/\text{sec.} \;>\; 3{,}800 \text{ m}^3/\text{sec.}$$

Hence, $Q_1 > 5{,}100$ m /sec.
$Q_2 > 3{,}800$ m /sec.
$Q_1 + Q_2 > 8{,}900 \text{ m}^3/\text{sec.}$

The designed two types of spillway can safely pass the design flood discharge of 8,900 m /sec.

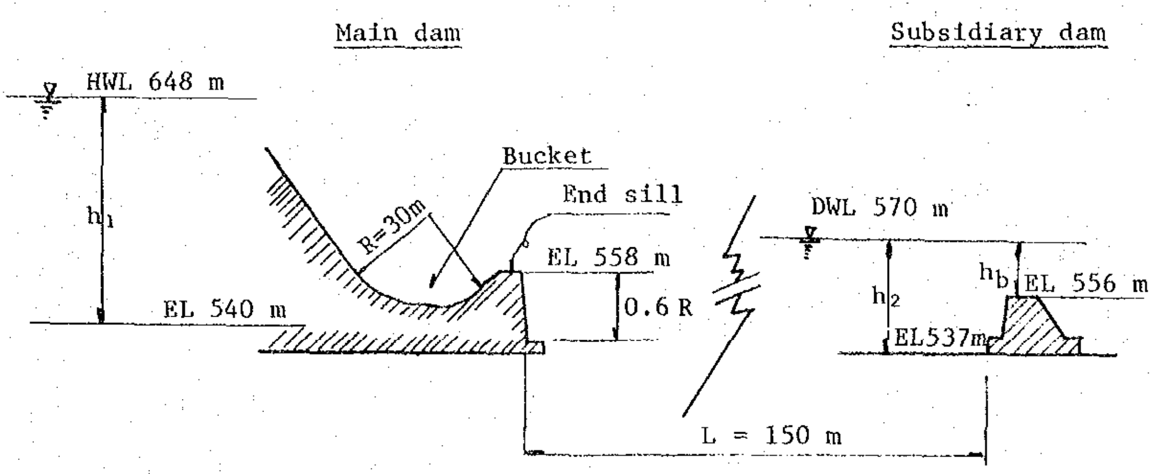
Then, the downstream dissipators are designed in the following manner.

Water jump at the hydraulic bucket will be given by McPerson-Karr's condition as follows.

$$q/\sqrt{g} \cdot h_1^{\frac{3}{2}}$$

where, q : design flood discharge per 1 m (m^3/sec.)
h_1: Upstream head (m)

The bucket and end sill as well as the downstream subsidiary dam are designed as represented in the following sketch.



The design flood discharge from the dam crest will be passed through 3 gates @15 m x 15.5 m. Then, we obtain

$$q = 5{,}100\ m^3/sec./45\ m \fallingdotseq 113.3\ m^3/sec./m$$

$$h_1^1 = WL\ 648\ m - EL\ 540\ m = 108\ m$$

$$*\quad q/\sqrt{g}\cdot h_1^{\frac{3}{2}} = 113.3/\sqrt{9.8}\cdot 108^{\frac{3}{2}} = 0.0322$$

A subsidiary dam will be built at 105 m downstream of the end of the main dam, for the two purposes of smoothly changing the discharged flow after dissipation and securing an adequate depth h in front of the subsidiary dam. From the chart given in "Hydraulic Formulae" compiled by the J.S.C.E., we can obtain the following relation, using the value 0.0322 as calculated above:

$$h_2/h_1 \fallingdotseq 0.3,\quad \frac{h_b}{h_1} = 0.12,\quad h_b = 0.4\ h_2,\quad \frac{h_1}{R} = 3.6$$

Hence, $h_2 = 108\ m \times 0.3 = 32.4\ m$

$R = h_1/3.6 = 30\ m$

$h_b = 32.4 \times 0.4 = 12.96\ m$

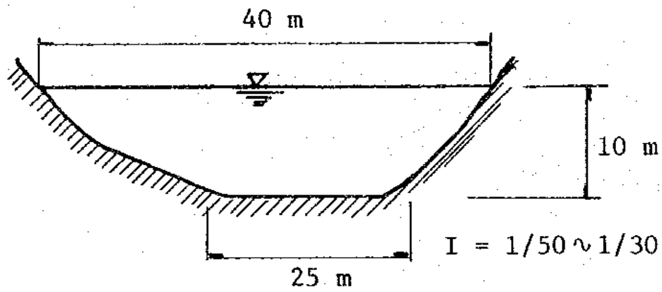
Thus, the water level of the dissipating basin will be:

$$537 \text{ m} + 32.4 \text{ m} = 569.4 \text{ m} \fallingdotseq \text{EL } 570 \text{ m}$$

and the overflow depth at the downstream subsidiary dam becomes 14 m, and its crest will be at EL 556 m.

The overflow velocity from the subsidiary dam shall be chosen at a value which will be seen in the natural river at the flood time.

The configuration of the river is as shown below:



Assuming an average gradient of the river course at 1/40, we obtain

$$A = (40 + 20)/2 \times 10 = 325 \text{ m}^2$$

$$v = \frac{1}{n} \cdot R^{\frac{2}{3}} \cdot I^{\frac{1}{2}}$$

$$= \frac{1}{0.045} \times (6.5)^{\frac{2}{3}} \times \left(\frac{1}{40}\right)^{\frac{1}{2}}$$

$$= 12.2 \text{ m/sec.}$$

This velocity is considered normal as compared with those seen at the flood time, i.e. 10∿13 m/sec.

With the subsidiary dam with 60 m crest width, hydraulic requirement will be checked.

For $h = 14$ m, $B = 60$ m, $hc = 9.3$ m, $Vc = 9.5$ m/sec. and $Q = 5{,}100$ m^3/sec., we obtain

$$Q = 9.3 \times 9.5 \times 60 = 5,300 \text{ m}^3/\text{sec.} > 5,100 \text{ m}^3/\text{sec.}$$

Thus, the design proves to be satisfactory.

It is noted that the hydraulics in the dissipating basin shall be ascertained by means of an adequate hydraulic model test.

(5) Stability of the dam

Details on the stability of the dam will be presented in 3.4.

3.2.3. Intake

An inclined type lateral intake will be constructed at the left bank of the river.

In order to prevent air entering the pressure tunnel, the intake will be located at an elevation of EL 605 m, which corresponds to a position 15 m below the low water level of the storage reservoir.

A set of screens shall be provided at the front of the intake. The gate shall be a roller gate type.

Grooves for insertion of stop logs shall be provided, in order to cope with situations such as gate trouble or repair. Air holes shall also be provided at the rear of the gate, in order to prevent the occurrence of vacuum in the pressure tunnel in case of emergency shutting of the gate (Fig. 3-2-1).

3.2.4. Headrace Tunnel

Design principles of selecting the headrace tunnel route, natural ground coverage and structure of the tunnel section are presented in Vol. 1.1, 3.4.4.

The following is the approximate calculation method referring to economic criterion for optimization of the pressure tunnel cross section.

In the design of the waterway facilities, the layout and scale are determined based mainly upon economic criteria. The approximate calculation referring to economic criteria is as follows:

Power : $P = \alpha \times Q \times H$

Electric energy : $W = \beta \times \Sigma Q \times H \times T$

Construction cost : $c = ei$

Change of head : Hi

Change of construction cost : ΔCi

where :

Q = turbine discharge (m^3/sec)

H : effective head (m)

The relationship between the change of head and construction cost and the economy due to changes in the waterway length, waterway diameter, steel pipe diameter and so forth is given by the following expression:

$$B \pm \Delta B = (P \pm \Delta P)\xi + (W \pm \Delta W)\eta$$
$$= \xi Q(H \pm \Delta H) + \beta \Sigma Q(H \pm \Delta H)T$$

$$C \pm \Delta C = C(1 \pm \frac{\Delta C}{C})$$

$$\frac{B \pm \Delta B}{C \pm \Delta C} = \frac{B(1 \pm \frac{\Delta B}{B})}{C(1 \pm \frac{\Delta C}{C})} = \frac{\xi \alpha QH(1 \pm \frac{\Delta H}{H}) + \beta \Sigma QH(1 \pm \frac{\Delta H}{H})T}{C(1 \pm \frac{\Delta C}{C})}$$

The following relation will be valid if the change of ΔH and ΔC are contained within a relatively limited range.

$$B \pm \Delta B \; / \; C \pm \Delta C = f(1 \pm \frac{\Delta H}{H} + \frac{\Delta C}{C})$$

Thus, it is possible to have obtain an idea of economy by calculating the difference of H/H and C/C.

Next are studied the waterway center line and the diameter. The influence of the waterway diameter and length on economy is approximately calculated as follows:

(i) The friction loss in the waterway is given as follows:

$$he = \frac{f}{R}\frac{V^2}{2g} L$$

where: he = friction loss (m)
f = coefficient of loss due to friction
V = velocity (m/sec)
L = length of pressure tunnel (m)
R = hydraulic mean depth (m)

Since $v = \frac{Q}{A}$ and putting $n = 0.013$ and $Q = 85.2m^3/s$ we have:

$$he = \frac{124.5 \times n^2 \times Q^2}{2g \times (\frac{\pi}{4})^2} \cdot \frac{L}{D^{16/3}}$$

$$= 12.633 \cdot \frac{L}{D^{16/3}}$$

Diameter (m)	Friction loss (m)
6.1	8.186×10^{-4} L
5.9	9.779×10^{-4} L
5.7	1.175×10^{-3} L
5.5	1.422×10^{-3} L

(ii) Determination of the waterway center line and waterway length.

The waterway center line can be selected between the distances of 12.450 m to 10,530 m, depending upon the method of crossing the Didipio creek. There are two alternatives for the center line selection, i.e., the shortest one assuming the construction of an aqueduct, and the other assuming the possibility of a tunnel design with construction of reinforcing steel pipe inside the tunnel lining at the limited section crossing the creek. The following is a comparison of the two alternatives. Assuming minimum covering at the creek crossing section and more than 20% of the internal pressure (670 m - 580 m = 90 m), the tunnel will be reinforced with steel pipes and grouting.

Alt.	Waterway length (m)	Diameter (m)	Friction loss (m)	Difference of loss (m)	Construction cost difference ($\times 10^6$¥)
1.	12,450	5.9	12.175	–	Base plan
2.	11,700	5.9	11.441	+0.734	Increment due to brace pipe portion of 150 m + 100 m = 250 m : plus 440 Decrease due to shorter tunnel extension of 750 m : minus 850 Total : minus 410
3.	10,530	5.9	10.297	+1.878	Increment due to 100 m long aqueduct : plus 1,200 Increment due to 250 m long brace pipe portion : plus 880 Decrease due to shorter tunnel extension of 1,920 m : minus 2,190 Total : minus 90

Comparison of alternatives 1, 2 and 3 indicates that with regard to 1,

Alternative 2 is $1 + 0,734/451 + \frac{4.1}{1,170} = 10.0016+0.0035=1.0051$

Alternative 3 is $1 + 1,878/451 + \frac{0.9}{1,170} =1+0.004+0.0008=1.0048$

As for B/C, we have:
Alternative 2 is approximately 0.51% larger, and
Alternative 3 is approximately 0.48% larger.

As a result of the considerations above, alternative 2 with the highest B/C is adopted for the waterway center line.

Carrying out the same study for the diameter of the waterway, we obtain the following results for L = 11.7km.

Diameter (m)	Loss Coefficient	Friction Coefficient	Difference of Loss (m)	Difference of Construction ($\times 10^6$¥)	B/C
6.1	$8.186\times10^{-4}L$	9.578			1
5.9	$9.779\times10^{-4}L$	11.441	1.863	Δ444	1.0001
5.7	$1.175\times10^{-3}L$	13.748	4.170	Δ889	0.998
5.5	$1.422\times10^{-3}L$	16.637	7.059	Δ1,345	0.996

From the considerations above, we conclude that 5.9 m is the most adequate value for tunnel diameter.

Consequently, the tunnel will be designed with 5.9 m diameter and 11.7 km length.

The average flow velocity in the interior of the pressure tunnel will be 3.116 m/sec. The coefficient of roughness of the concrete lining surface is assumed to be 0.013 (Fig. 3-2-2).

3.2.5. Design of Surge Tank

(1) Surge tank planning conditions

Waterway tunnel extension : 11,700 m

of which Concrete lining section : 11,450 m

Brace pipe section : 250 m with a

sectional area of $fA = \frac{\pi}{4} \cdot D^2 = 27.34\ m^2$

Maximum turbine discharge : $Q = 85.2\ m^3/s$

Maximum velocity : $v = \frac{Q}{fA} = 3.116$ m/sec.

Coefficient of roughness

n = 0.115	In case of upsurge
n = 0.0145	In case of downstream
n = 0.013	In case of operation

Waterway loss coefficient

Friction loss

$$he = f/R \times L \times \frac{V^2}{2g} = \frac{124.5\ m^2}{D^{4/3}}\ L\ \frac{(3.116)^2}{2g}$$

In case of n = 0.013

$$he = \frac{124.5 \times 1.69 \times 10^{-4} \times 9.709}{10.661 \times 196} \times L = 0.768 \times L \times 10^{-3}$$

In case of n = 0.015

$$he = 0.765 \times L \times 10^{-3}$$

In case of $n = 0.0145$

$$h_e = 1.216 \times L \times 10^{-3}$$

Total loss of the waterway is calculated by adding the intake loss and caused by the concrete lining of the waterway.

Intake loss	0.500		
Waterway friction loss	n = 0.0115	0.0145	0.013
(L = 11,450 m)	8.759	13.923	11.193
Brace pipe loss	n = 0.011	0.0130	0.0120
(L = 250 m)	0.423	0.591	0.504
Other loss	0.318	0.436	0.298
Total (Zo)	10.0	15.5	12.5
Loss coefficient C	1.030	1.596	1.237

(2) Calculation in case of shut-off

$$F = \frac{\pi D2}{4} = 50.265\ m^2 \quad (\text{Diameter} = 8\ m)$$

$$Z^* = \nu_0 \sqrt{Lf/gF} = 3.116 \times \sqrt{\frac{11,700 \times 27.34}{9.8 \times 50.265}} = 79.40$$

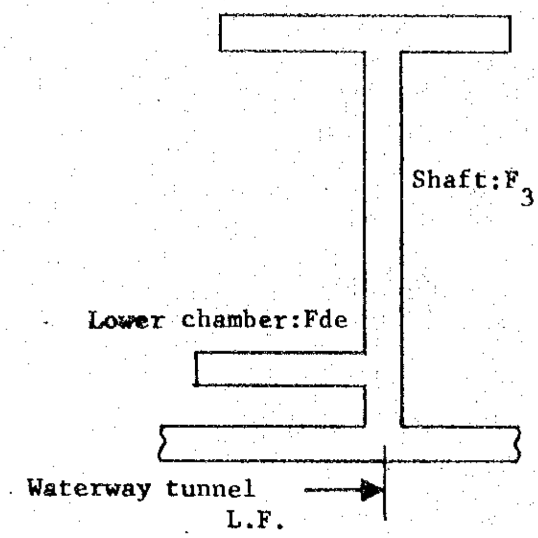
The condition for vibration stability is given as follows.

$$F > (1 + 0.482 \frac{Z^*}{Ho}) \frac{L \cdot f}{2CgHo}$$

$$\text{or } F > (1 + 0.0482 \frac{79.40}{456}) \frac{11,700 \times 27.34}{2 \times 1,030 \times 9.8 \times 456}$$

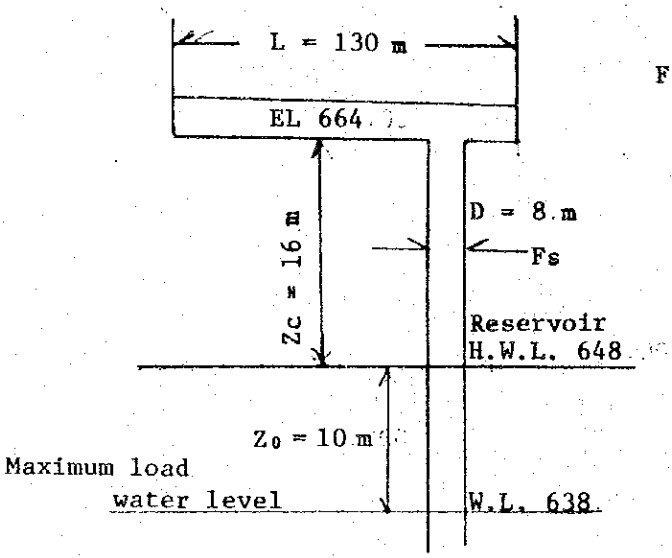
$$\text{or } F > (1 + 0.08) \times 34.75 = 37.5\ m^2$$

Stability of vibration is attained with $F = 50.265\ m^2$ as designed with the shaft of 8 m diameter.



The capacity and position of the upper chamber in case of instantaneous shut-off of the load is approximately calculated by subtraction, using the Frank-Schulter formula as follows.

$$A/Ec\cdot\chi m = \frac{Z}{Ec}\chi c - \log e\ [\{1 - \exp\frac{Z}{Ec}(\chi c - 1)\frac{Fs}{Fc}\}/(1 + \frac{Z}{Ec}\chi m)$$



$Fc = 8 \times 125 = 1{,}000\ m^2$

Assuming $F_s = \frac{\pi}{4} \times 8^2 = 50.265\ m^2$, and $F_c = 8 \times 125 = 1{,}000\ m^2$,

$$\frac{Fs}{Fc} = 0.0503 \qquad X_c = \frac{Zc}{Zo} = \frac{-16}{10} = -1.6$$

$$\varepsilon c = \frac{LfVo^2}{gFeZo^2} = \frac{11{,}700 \times 27.34 \times (3.116)^2}{9.8 \times 1{,}000 \times 10^2} = 3.169$$

$$\frac{1 - Xc}{\varepsilon c} \times \frac{Fs}{Fc} = \frac{1 - (-1.6)}{3.169} \times 0.0503 = 0.0413$$

$$\frac{Xc}{\varepsilon c} = \frac{-1.6}{3.169} = -0.505$$

Assuming $Xm = -2.207$, $\quad \frac{2Xm}{\varepsilon c} = \frac{2 \times (-2.207)}{3.169} = 1.392$

$$\frac{2}{\varepsilon c}\, Xc = 2 \times (-0.505) = -1.010$$

$$\log e \left[\left\{1 - \frac{Fc}{Fs}\left(1 - \exp\frac{2}{\varepsilon c}\,(Xc - 1)\,\frac{Fs}{Fc}\right)\right\} \Big/ \left(1 + \frac{2}{\varepsilon c}\,Xm\right)\right]$$

$$\log e \left[\left\{1 - 19.895\left(1 - \exp\frac{2}{\varepsilon c}\,(Xc - 1)\frac{Fs}{Fc}\right)\right\} \Big/ \left(1 \times \frac{2}{\varepsilon c}\,Xm\right)\right]$$

$$= \log e\, [\{1 - 19.895\ (1 - 0.9208)\}/(1 - 1.3929)]$$

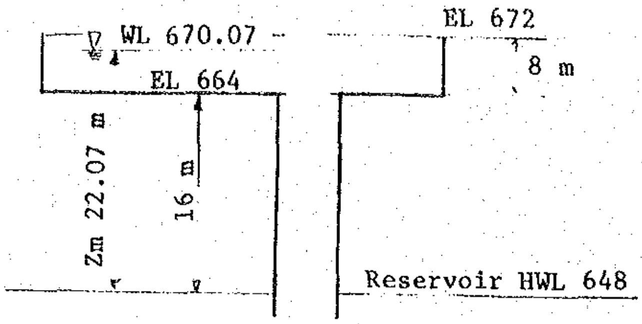
$$= \log e\, [(1 - 1.5758)/-0.3929] = \log e\, 1.4655 = 0.382$$

$$\frac{2}{\varepsilon c}\, Xc - \text{Log}\, e \left[\left\{1 - \frac{Fc}{Fs}C - \exp\frac{2}{\varepsilon c}\,(Xc - 1)\frac{Fs}{Fc}\right\} \Big/ 1 + \frac{2}{\varepsilon c}Xm)\right]$$

$$= -1.010 - 0.382 = 1.392$$

Hence we obtain $Xm = -2.207$.

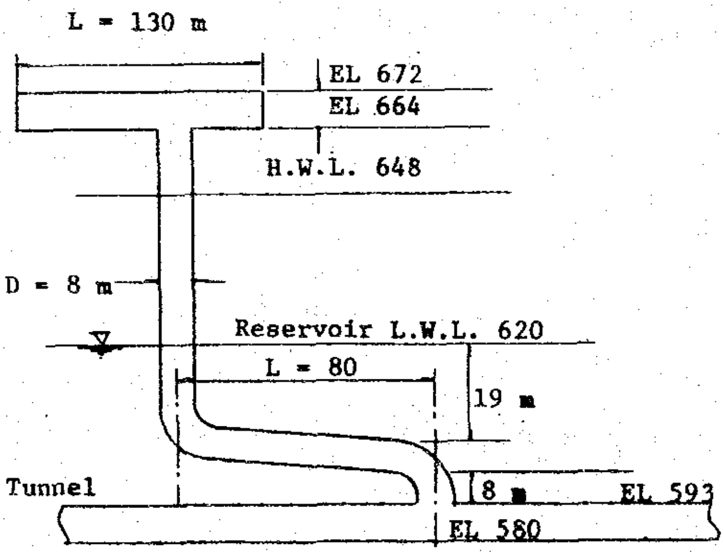
$$Zm = Zo \times Xm = 1.0 \times (2.207) = -22.070$$



Since the horizontal tunnel will be constructed in accordance with the topography, it is necessary to determine conveniently the chamber length according to the topography or to ensure a minimum area of 1,000 m^2.

(3) Capacity of the lower water chamber

For the lower chamber, the configuration shown in the figure below will be adopted by taking into consideration tophgraphical and geological aspects.



C = 15.5/9.709 = 1.596

The length Lc of the lower water chamber is calculated by means of the following formula :

$$Lc = \frac{\varepsilon s}{\varepsilon_1} Fs \frac{(Xm - \alpha^2)^{\varepsilon_1/\varepsilon_2}(1 - \varepsilon_2/\varepsilon s)}{\sum_{Xc}^{Xm} (x - \alpha)^{2(\varepsilon_1/\varepsilon_2 - 1)} BC\Delta x}$$

$$\varepsilon_1 = 2(Xm - \alpha^2) / \{ \log e (\frac{Xm - 1}{Xm - \alpha^2}) + \frac{1}{\sqrt{Xm}} \log e (\frac{\sqrt{Xm} + 1}{\sqrt{Xm} - 1} \cdot \frac{\sqrt{Xm} - \alpha}{\sqrt{Xm} + \alpha}) \}$$

$$\varepsilon_2 = \frac{1}{(1 - \alpha)^2} \{ (Xm - \alpha^2 - \frac{\phi}{2})^2 - (\frac{\phi}{2})^2 \}$$

$$\phi = (1 - \alpha) \{ \frac{\pi}{4} (3 + \alpha) - 2 \}$$

In case of sudden change from half load to full load.

$Zm = 27m$ $Zo = 15.5m$ $Zc = 19m$

$$Xm = \frac{Zm}{Zo} = \frac{27}{15.5} = 1.742$$

Assuming $\alpha = 0.5$ we obtain

$$\phi = (1 - 0.5) \{ 0.785 \times 3.5 - 2 \} = 0.374$$

$$\varepsilon_1 = 2(Xm - \alpha)^2 / \log e (\frac{Xm - 1}{Xm - \alpha_2}) + \frac{1}{\sqrt{Xm}} \log e (\frac{\sqrt{Xm} + 1}{\sqrt{Xm} - 1} \cdot \frac{\sqrt{Xm} - \alpha}{\sqrt{Xm} + \alpha})$$

$$= 2(1.742 - 0.25) / \log e (\frac{1.742 - 1}{1.742 - 0.25}) + \frac{1}{\sqrt{1.742}} \log e (\frac{1.742 + 1}{1.742 - 1} \times \frac{1.742 - 0.5}{1.742 + 0.5})$$

a denominator

$$= \log e (\frac{0.742}{1.492}) + \frac{1}{1.320} \log e (\frac{2.320}{0.320} \times \frac{0.820}{1.820})$$

$$= \log e\ 0.497 + \frac{1}{1.320} \log e\ 3.226 = -0.699 + 0.897 = 0.198$$

$$\varepsilon_1 = \frac{2 \times 1.492}{0.198} = 15.071$$

$$\varepsilon_2 = \frac{1}{(1 - \alpha)^2} \{ (Xm - \alpha^2 - \frac{\phi}{2})^2 - (\frac{\phi}{2})^2 \}$$

$$= \frac{1}{(0.5)^2} \{ (1.742 - 0.25 - \frac{0.374}{2})^2 - (\frac{0.374}{2})^2 \} = \frac{1}{0.25} \{ 1.703 - 0.035 \} = 6.672$$

$$\varepsilon s = \frac{L \times f \times v^2}{9.8 \times Fs \times Zo} = \frac{11.700 \times 27.34 \times 3.116^2}{9.8 \times 50.265 \times 15.5} = 26.244$$

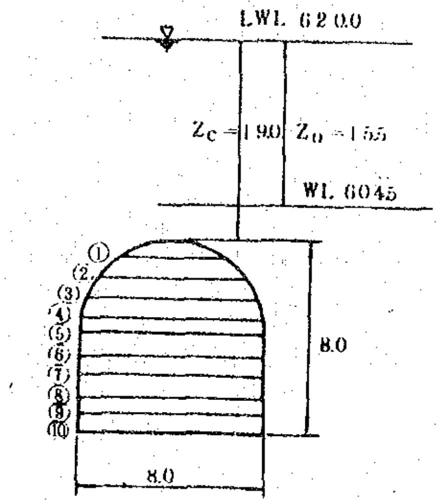
$$\frac{\varepsilon_1}{\varepsilon_2} = \frac{15.071}{6.672} = 2.259$$

$$\frac{\varepsilon_2}{\varepsilon s} = \frac{6.672}{26.244} = 0.254$$

$$\frac{\varepsilon s}{\varepsilon_1} = \frac{26.244}{15.071} = 1.741$$

$$\Delta x = \frac{\Delta Z}{Zo} = \frac{0.800}{15.5} = 0.0516 = 0.052$$

$$Lc = \frac{\varepsilon_2}{\varepsilon_1} Fs \frac{(Xm - \alpha^2)^{\frac{\varepsilon_1}{\varepsilon_2}} (1 - \varepsilon_2/\varepsilon s)}{\sum_{Xc}^{Xm} (x - \alpha)^{2(\frac{\varepsilon_1}{\varepsilon_2} - 1)} BC\Delta x}$$



№	B	Z	x	$x-a^2$	$(x-a^2)^{\frac{\varepsilon_1}{\varepsilon_2}-1}$	$(x-a^2)^{\frac{\varepsilon_1}{\varepsilon_2}-1} Bc\triangle 2$	摘要
1	3.487	19.4	1.252	1.002	1.003	0.182	
2	5.713	20.2	1.303	1.053	1.067	0.317	
3	6.928	21.0	1.355	1.105	1.134	0.409	
4	7.631	21.8	1.406	1.156	1.200	0.476	
5	7.960	22.6	1.458	1.208	1.269	0.525	
6	8.000	23.4	1.510	1.260	1.338	0.557	
7	8.000	24.2	1.561	1.311	1.406	0.585	
8	8.000	25.0	1.613	1.363	1.477	0.614	
9	8.000	25.8	1.665	1.415	1.548	0.644	
10	8.000	26.6	1.716	1.466	1.619	0.674	
計						4.983	

$$Lc = \frac{\varepsilon s}{\varepsilon_1} Fs \frac{(Xm-a^2)^{\frac{\varepsilon_1}{\varepsilon_2}}\left(1-\frac{\varepsilon_2}{\varepsilon s}\right)}{\sum_{Xc}^{Xm}(x-a)^{\frac{\varepsilon_1}{\varepsilon_2}-1} Bc\,\triangle x}$$

$$= \frac{1.741 \times 50.265 \times (1.742-0.25)^{2.259}(1-0.254)}{4.983} = 32.4\,m$$

Consequently, the lower chamber will have an extension of 80 m and a gradient of approximately 1/100, and will be arranged in such a way as to be located in the core of the mountain with an included angle of 30°.

The surge tank designed in accordance with the procedure described above will be as follows:

Vertical tunnel : Height 77.1 m from T.S.W.L. to B.S.W.L., with a circiular cross section of 8 m diameter

Water chamber :

- Upper chamber : 130 m long tunnel with 57 m^2 cross sectional area

- Lower chamber : 80 m long tunnel with 57 m^2 cross sectional area

(see Fig. 3-2-3)

3.2.6. Penstock

In case of penstock constructed on the ground, the steel pipe length will be approximately 2,000 m, and the increase of the water hammer will be considerably high. Thus, the water pressure increase will be restricted by extending somewhat the valve shut-off time. As for the diameter of the pipes, the results of approximate calculations indicate that of the two possible alternatives, i.e. 4.7 m - 3.7 m - 2.7 m and 5.0 m - 3.8 m - 2.7 m, the second one with varied diameters of 5.0 m - 3.8 m - 2.7 m is slightly advantageous from the economic point of view, and consequently desin will be carried out by adopting this alternative.

The design procedure of the penstock is given as follows.

	5.0	4.7	4.4	4.1	3.8	Bifur-cator	2.7	
Ai	19.625	17.341	15.198	13.196	11.335		5.723	
Vi	4.341	4.913	5.606	6.457	7.517		7.444	
Li	377	403	390	390	391		59	including gradual decrease of diameter
		(6)	(6)	(6)	(6)			
h_f	0.933	1.386	1.908	2.781	4.179		0.978	$h_f = \frac{13.211 \times Li}{D^{16/3}}$
$D^{16/3}$	5341	3840	2701	1853	1236		199.7	$\Sigma h_f = 12.165$
$\frac{v^2}{2g}$		1.232	1.603	2.127	2.883	2.162		for gradual decrease of diameter $0.001 \frac{v^2}{2g}$ 0.010 m loss due to bending

From the above table, we obtain

$$
\begin{aligned}
Am &= \Sigma Li/\Sigma(Li/Ai) \\
&= 2{,}069/(19.210 + 23.240 + 25.661 + 29.554 + 34.495 + 20.619) \\
&= 2{,}069/152.779 \\
&= 13.542
\end{aligned}
$$

Vom = Qo/Am = 6.292 m/sec.

$$\theta = \frac{8}{4{,}020/1{,}070} = 2.129$$

$$\rho = \frac{1{,}070 \times 6.292}{19.6 \times 525} = 0.654$$

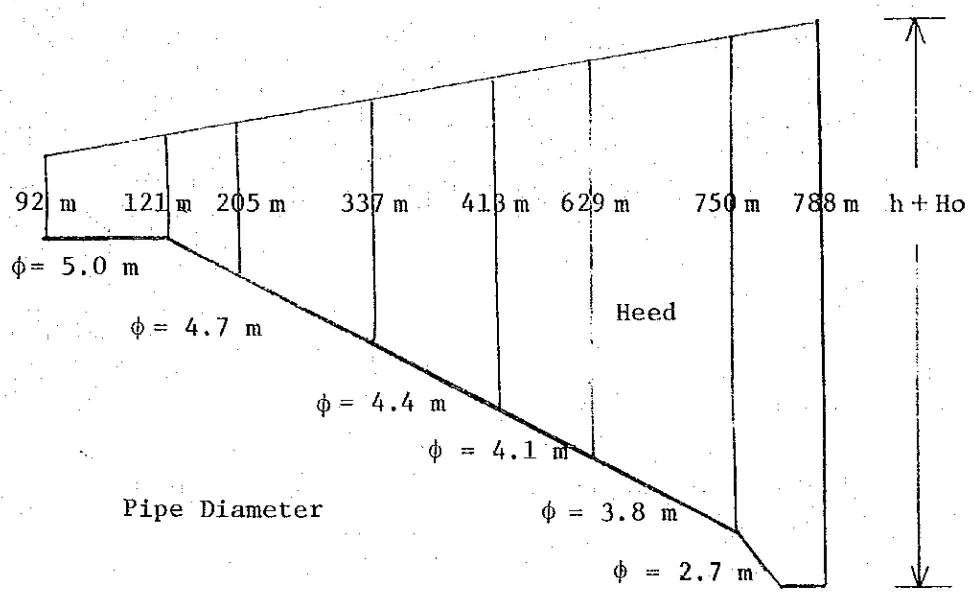
$$n = - = \frac{0.654}{2.129} = 0.307$$

The pressure rise will be as follows:

$$\frac{h}{H_0} = \frac{2 \cdot n}{1 + n \cdot (\theta - 1)} = \frac{2 \times 0.307}{1 + 0.307 \cdot (2.129 - 1)} = \frac{0.614}{1.347} = 0.456$$

Consequently, assuming water pressure rise of 0.50 at the guide vane position, we have :

$h + H_0 = 525\,m \times 1.5 = 788\ m$



Pipe wall thickness is given by the following formula:

$$t = \frac{P \times D}{2 \times \sigma} = \frac{P \times D}{5,760} \text{ (cm)}$$

D (m) \ H (kg/cm^2)	9.2	20.5	33.7	41.3	62.9	75.0	78.8	Remarks
5.0	4,600 (14+2)	10,250 (18 + 2)						P x D in t·mm.
4.7		9,635 (17 + 2)	15,839 (28 + 2)					Figure in ( ) plate thickness with allowance in mm
4.4			14,828 (26 + 2)	18,172 (32 + 2)				
4.1				16,933 (30 + 2)	25,789 (45 + 2)			
3.8					23,902 (42 + 2)	28,500 (50 + 2)		
2.7						29,250 (36 + 2)	21,276 (37 + 2)	

Assuming the use of high tensile strength steel with σ = 80 kg/mm^2 (the yield point is assumed to be 70 kg/mm^2 and the safety factor is assumed to be 2.18 instead of the conventional 1.8).

70 kg/mm^2 $\div$ 2.18 = 32 kg/mm^2

The following values are assumed for the various values of plate thickness:

for thickness up to 50 mm 32 kg/mm^2

for thickness larger than 50 mm 31 kg/mm^2

Assuming a welding efficiency of 90%, we have:

$\sigma = 3,200 \text{ kg/cm}^2 \times 0.9 = 2,880 \text{ kg/cm}^2$

(The wall thickness for D = 5 m will be given as follows, by the minimum thickness formula : $t = (D + 800)/400 = 16$ mm.)

Steel pipe weight is given as follows.

D (m)	t (mm)	W (t/m)	Weight (t)
5	16 20	1,979 2,476	847
4.7	19 30	2,211 3,500	1,151
4.4	28 34	3,058 3,718	1,322
4.1	32 47	3,261 4,807	1,573
3.8	44 52	4,172 4,940	1,768
2.7	38 39	2,566 2,635	323
Bifur-cator			140 t
Total			7,124 t

Assuming addition of 15% under the pretext of accessory structures and some margin, the total weight of the steel pipe will be 7,124 t x 1.15 = 8,200 t.

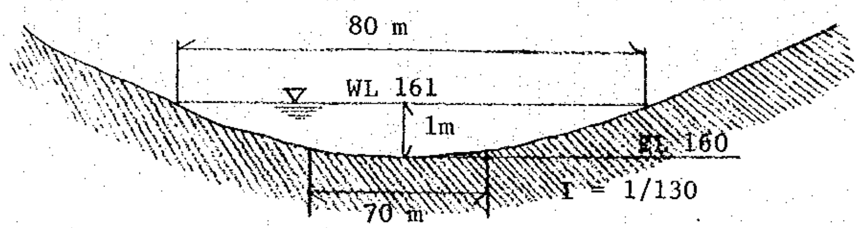
Summing up the calculations above, the penstock will have inner diameter ranging from 5.0 m to 2.7 m, and a total extension of 2,013 m .

The flow speed in the interior of the penstock will have variable values ranging from 4.341 m/sec to 7.444 m/sec, according to the penstock diameter. In case of variation of the penstock diameter, the optimum cross section is obtained by comparing the head loss caused by the diameter variation and the required cost. The penstock wall thickness is gradually changed from 16 mm to 52 mm, depending upon the water pressure. The total weight of the penstock will be 8,200 t. In view of the large extension of the steel pipe and the high hydro-dynamic pressure, the adoption of high tensile strength steel has two remarkable positive effects, i.e., reduction of thickness of the steel plate used in the penstock wall construction, and reduction of the required quantity of steel. (see Fig. 3-2-4)

3.2.7. Tailrace

(1) Determination of tailrace water levels

According to the 1/1,000 scale map, the river bed elevation above sea level at the neighbourhood of the outlet is of the order of 159 - 160 m, and the gradient of the river bed around the tailrace outlet is assumed to be of the order of 1/130.



The riverflow discharge at the tailrace outlet is given as follows:

Maximum plant discharge : 85.2 m^3/sec.
Riverflow from the residual catchment area:
75 m^3/s (1-month riverflow at the damsite)

$$\times \frac{610 - 477 \text{ km}^2 \text{ (residual C.A. between the damsite and the outlet)}}{477 \text{ km}^2 \text{ (catchment area of the damsite)}}$$

= 16 m^3/sec.
Total : 101 m^3/sec.

Assuming that the river cross section has the configuration shown in the figure above, the riverflow discharge at depth of 1 m will be as follows:

$$Q = AV \quad \text{and}$$

$$V \fallingdotseq \frac{1}{n} R^{\frac{2}{3}} I^{\frac{1}{2}}$$

where, $A \fallingdotseq 75\ m^2$

$R \fallingdotseq 0.94$

$I^{\frac{1}{2}} \fallingdotseq \frac{1}{11.4}$

$\frac{1}{n} \fallingdotseq \frac{1}{0.04}$

$$V \fallingdotseq 25 \times 0.96 \times \frac{1}{11.4} \fallingdotseq 2.1\ m/sec$$

Thus, we obtain

$$Q = AV$$

$$= 157\ m^3/sec \quad \text{at W.L. } 161\ m$$

$$> 101\ m^3/sec$$

Consequently, the river water stage EL 161 m is sufficient to pass the riverflow discharge of 97 m^3/sec., but with design safety against sedimentation at the tailrace outlet into consideration the design tailrace water stage at the outlet is chosen at EL 162 m.

Then, we proceed to calculate the water stage in the tailrace afterbay.

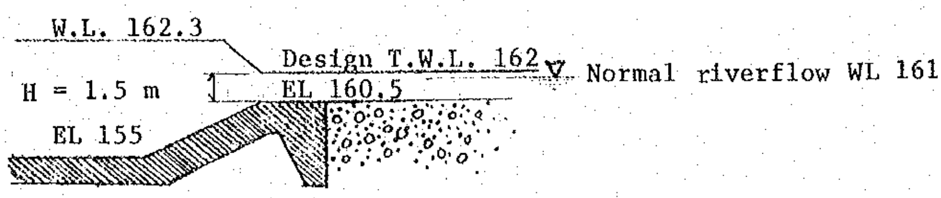
Overflow velocity at the outlet submerged weir will be:

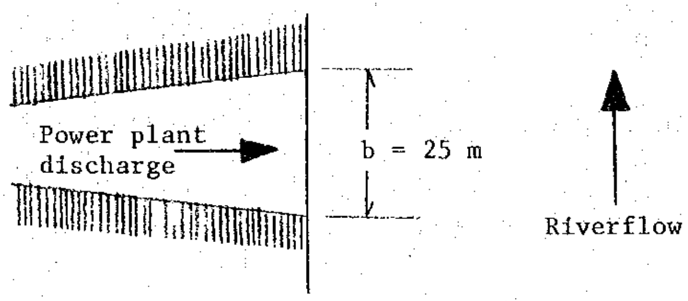
$$V = \frac{Q}{A} = \frac{85.2}{1.5 \times 25} = 2,272\ m/sec.$$

Velocity head will be:

$$be = \frac{V^2}{2g} = 0.26 \fallingdotseq 0.30\ m,$$

W.L. before the outlet = 162 m + 0.3 m = 162.3 m





The loss head in case of the tailrace being in a state of pressure tunnel and miscellaneous other losses in the 110 m tailrace are estimated at 0.2 m + 0.6 m = 0.8 m.

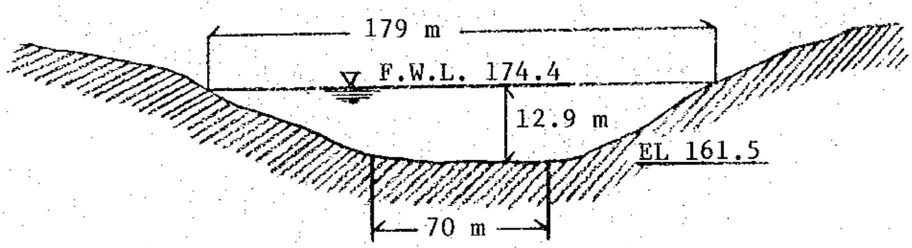
From the considerations above, the afterbay water level is assumed to be 163.1 m.

(2) Design ground level around the powerhouse

Not taking into consideration the flood mitigating effect in the reservoir, the 200-year flood discharge around the powerhouse site will be:

$$8{,}900 \text{ m /s} \times \frac{610 \text{ km}^2}{477 \text{ km}^2} = 11{,}400 \text{ m}^3\text{/s}$$

The flood level in the vicinity of the powerhouse will be calculated as follows :



$$A = \frac{1}{2}(179 + 70) \times 12.9 \doteqdot 1,606 \ m^2$$

$$P = 232 \ m$$

$$R = 6.922$$

$$R^{2/3} = 3.632 \qquad \left\{ \begin{array}{l} n = 0.04 \\ I = \frac{1}{130} \end{array} \right.$$

$$v = \frac{1}{n} \cdot R^{2/3} \cdot I^{1/2}$$

$$= \frac{1}{0.04} \times 3.632 \times (\frac{1}{130})^{1/2} \doteqdot 8 \ m/sec$$

$$\therefore \ Q = Av$$

$$= 1,606 \ m^2 \times 8 \ m/sec = 12,848 \ m^3/sec \quad \text{at WL } 174.4 \ m$$

$$> 11,400 \ m^3/sec$$

Since the flood water is safely discharged at a level of 174.4 m, the powerhouse will be perfectly safe even in case of flood of the Diduyon River if the ground surface level around the powerhouse structure is assumed to have an elevation of 176.3 m.

(3) The center elevation of the turbines is determined as follows:

The characteristic speed of the Francis turbines is calculated as follows.

$$n_s = \frac{1,650}{(451)^{1/2}} = \frac{1,650}{21.237} = 77.7$$

$$N = n_s \cdot H^{5/4} / P^{1/2}$$

$$= 77.7 \times 451^{5/4} / (172,500)^{1/2} = 388$$

N will be chosen at 360 r.p.m. for 60 Hz

The draft head of the turbines is derived from the following formulae.

$$H_s = 10 - \sigma H, \text{ and}$$

$$\sigma = 0.053\left(\frac{ns}{100}\right)^{4/3} \qquad \text{where; } ns = 77.7 \text{ as given above}$$

$$= 0.053\left(\frac{77.7}{100}\right)^{4/3} \doteqdot 0.038$$

$$\therefore\ H_s = 10 - 0.038 \times 451 = -7.1$$

Since the water stage at the afterbay is 163.1 m as calculated above, the center elevation of the turbines will be 163.1 m - 7.1 m = 156 m.

(4) Determination of tailrace outlet location

Topographically the available space for the powerhouse is limited to the terrace at the left bank of the Diduyon River. The location of the tailrace outlet is also practically restricted up to the point approximately 250 m downstream of the powerhouse site owing to the confluence of a creek with the Diduyon River. There is possibility of occurrence of backwatering in this area in case of flooding. Thus, in order to utilize the available head in this portion and also to prevent the occurrence of overhead flooding innudation and other trouble in the powerhouse during the construction work and its operating period, it is assumed better to locate the powerhouse site at a position distant from the Diduyon River, and connect the powerhouse with tailrace outlet by means of a short waterway. In view of considerations above, the conduction of water is done by means of a tailrace tunnel (culvert) approximately 200 m long up to the confluence of a creek with the Diduyon River.

3.2.8. Powerhouse

(1) Design of the powerhouse

The design of an open powerhouse type is adopted in this project. The dimensions of the building are 29.6 m width, 63 m length and 37 m height.

The outline of powerhouse design is presented in Fig.3-2-5.

1) Location and size of the powerhouse building

As explained earlier, the powerhouse will be located on the terrace at the left bank of the Diduyon River. The outdoor switchyard will be constructed at the upstream side of the powerhouse, and the access road for transportation of the equipment will be located at the downstream side, taking into consideration factors such as the access road for transportation of equipment, location of the outdoor switchyard and topographical conditions.

The powerhouse building will be of the semi-underground type, in view of the restrictions imposed by the maximum flood level and the machinery layout. 2 turbines, 2 generators, 2 inlet valves, auxiliary equipment, control equipment, repair shop, storage, etc. will be installed in this building.

The following are the approximate dimensions of this building:

	Main building	Auxiliary building	Overall dimensions of the building
Width	20.3 m	9.3 m	29.6 m
Length	63 m	63 m	63 m
Height	37 m	26.8 m	37 m
(Open portion	15.9 m	7.7 m	
Under-ground portion)	21.1 m	19.1 m	

The plan and the section of the powerhouse building are shown in Fig.3-2-6 - Fig.3-2-12.

The dimensions of the building presented above are determined based upon the following values:

Inlet diameter of the turbine runner	Approx. 4.0 m
Outlet diameter of the turbine runner	Approx. 2.44 m
Height of the turbine runner	Approx. 1.52 m
Maximum diameter of spiral case	Approx. 2.27 m
Distance between centers of 2 turbines	18 m
Draft tube depth (distance between the turbine distributor center and the lowest point of the draft tube)	Approx. 11.5 m
Inner diameter of generator housing (inscribed circle of octagon)	Approx. 10 m
Crane rail span	Approx. 18.2 m
Distance from floor of 1F to crane runways level	7.7 m

2) Layout of equipment in the interior of the powerhouse building

The powerhouse building is composed of a total of 6 floors, i.e., 2 floors above the ground and 4 underground floors. The equipment of the these floors are arranged as follows :

a) 4th basement (El 155.4 m and EL 157.4 m)

On this floor are installed the inlet valves and the turbine spiral cases. This floor is located at the lowest part of the building and contains the cooling water equipment such as the strainers for cooling of the turbine generator and so forth, draft dewatering pumps, drainage pumps of the building, sump tanks of governor oil pumping sets, lubrication oil storage tanks, and so forth.

The inlet valve side (mountain side) of this floor is located 2.0 m lower compared with the auxiliary building side (river side) in order to make possible construction of the concrete pedestals of the inlet valves.

b) 3rd basement (EL 161.4 m)

This floor is called the turbine room and contains the equipment necessary for operation and control of the turbines.

The main equipment on this floor are the speed governors,the oil pumping sets of the speed governors and inlet value, the air compressors and the motor control centers.

c) 2nd basement (EL 166.0 m)

This floor is generally called the generator room, and contains the 2 generators. The isolated phase buses and circuit breakers connecting the generators and main transformers, excitation transformers potential transformer cubicles, surge diverter and disconnect cubicles for No.1 unit, station service transformer for No.1 unit, automatic voltage regulator cubicles, 4.16 kV and 480 V power distribution switchboards and storage are also installed on this floor.

d) 1st basement (EL 172 m)

This floor contains the top deck of the generator housings, the room for disassembly and assembly of the turbine generator, rectifier and inverter room and repair shop. The auxiliary building on this floor contains the No. 2 unit isolated phase bus coming from the 2nd basement, surge divertor and disconnect switch cubicles for No.2 unit, station service transformer for No.2 unit.

e) Ground floor (EL 176.5 m)

The platform to carry-in the main equipment is installed on this floor. The battery room, cable gallery, dining room and air conditioner room are also arranged on this floor.

f) 2nd floor (EL 179.5m)

This floor is the operating center of the power station, which accommodates control room, telephone and relay room, superintendent room, meeting room, rest room and air conditioning machine room.

3.2.9. Main Electrical Equipment

Design of main equipment

The output of the turbine generator with regard to the water level and turbine discharge of this powerhouse is presented in the table below.

	Turbine generator output at various head values		
	Maximum head	Standard head	Minimum head
Turbine discharge (m^3/sec)	89.4	87.7	86.8
Intake level (EL)	648.	638.7	620
Tailrace water level(EL)	162	162	162
Total head (m)	486	467.7	458
Head loss (m)	38.5	37.0	36.2
Effective head (m)	447.5	430.7	421.8
Turbine efficiency	0.898	0.9	0.896
Generator efficiency	0.98	0.98	0.975
Turbine output (kW)	352,000	333,200	321,500
Generator output (kW)	345,000	326,500	313,400

The number of units of turbines and generators is determined as a result of studies referring to the following aspects: fluctuation of the turbine discharge through the year, weight of the transportation of equipment, conditions of the access road up to the powerhouse main building, % occupied by unit scale within the whole system, storage reservoir operation plan, economic and technical studies referring to division into the various units, economy of civil engineering facilities, ease of operation, etc. It was decided to adopt the 2 unit system in this project.

(1) Turbine

In case of adopting 2 turbines, the maximum output of each turbine will be:

352,000 kW/2 = 176,000 kW (239,300 metric HP).

In this case, it is possible to adopt either the Francis turbine, or the Pelton turbine, for the available effective head of 447.5 m and the output of 176,000 kW. However, this powerhouse has a small power fluctuation, and it can be operated close to the maximum output almost 100% of the time. Thus, it is impossible to imagine operation below 50 - 60% of the turbine output. Under such conditions, the most important advantage of the Pelton turbine, i.e., better efficiency at lower power outputs compared with the Francis turbine, cannot be fully utilized. In addition, since the Pelton turbine is more expensive and has a more complicated construction, resulting into more difficult operation and maintenance compared with the Francis turbine, it was decided to adopt the Francis turbine for this project.

The following are specifications of the turbine.

Type		Vertical shaft, single runner, single flow, spiral type Francis turbine
Hydraulic power	Maximum	176,000 kW (239,300 metric HP)
	Standard	166,600 kW (226,500 metric HP)
	Minimum	160,800 kW (218,600 metric HP)
Rated speed		360 r.p.m.
Number of units		2

(2) Inlet valve

Since the maximum static head at the powerhouse is 486 m, the inlet valve to be installed will be a rotary one.

(3) Generator

Two generators will be connected to the 230 kV transmission line with the main step-up power transformers, in order to transmit power to the Santiago substation. Since the line charging capacity of 230 kV transmission line is very large which causes reactive power, the power factor of the generator should be 90%.

In view of the generator speed (360 r.p.m.) and the output of the generator, it should be conventional vertical type turbine generator, i.e., with the thrust bearing above the rotor and two guide bearings above and below.

The specifications of the generator are as follows;

Type	Vertical shaft, totally enclosed cooling system, air cooled, 3 phase synchronous generator
Output	Rated output : 167,700 kVA
	115% of rated output : 191,700 kVA
Power factor	90% lag
Voltage	13.8 kV
Frequency	60 Hz
Speed	360 r.p.m.
Number of units	2

A conceptual configuration of the turbine and generator is presented in Figure 3-2-13.

(4) Main transformer

The unit system with one main transformer connected to each generator is adopted in this project in order to improve maintenance reliability.

The total weight of each transformer is approximately 190 tons, and a special 3 phase transformer making possible the division into each phase is adopted, in view of the restriction of a maximum transportation weight of 40 tons. The output of this transformer is 178,000 kVA in order to cope with 115% of rated output of the generator.

The following are specifications of this transformer.

Type	Special 3-phase transformer, oil-immersed, forced-oil, forced-air-cooled, suitable for outdoor use
Output	191,700 kVA
Primary voltage	13,200 V
Secondary voltage	230,000 V
Frequency	60 Hz
Number of units	2

As for the connection of the generator main lead terminals and the low voltage side of the main transformer, it is impossible to use high voltage cables because the current of this circuit reaches a maximum value of 8,300 A. Consequently, an isolated phase bus is used to make this connection. On the other hand, the high voltage side of the main transformer is connected to the 230 kV bus bars by means of overhead lines.

(5) Crane

The maximum weight which will be hoisted by the crane is the generator rotor. The rotor is expected to have a weight of approximately 170 tons. Generally speaking, 1 or 2 cranes are installed, taking into consideration the magnitude of the weight to be hoisted, the number of units of generators to be installed and shortening of the installation work. However, since the weight to be hoisted in this powerhouse is relatively small, and only 2 generators are installed, the quantity of cranes will be 1 unit.

The specifications of the crane are as follows:

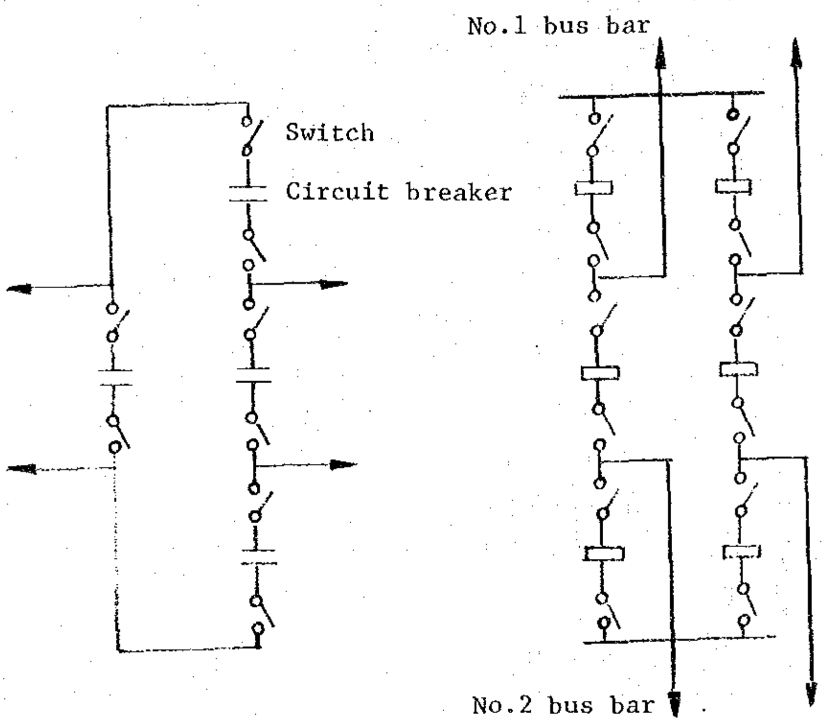
Type	Low speed type overhead travelling crane for indoor installation	
Rating	Main crane	170 tons
	Auxiliary crane	40 tons
	Hoist	10 tons
Rail span	18.2 m	

(6) Switchyard

The Diduyon power plant and the Santiago substation will be interconnected by menas of a 230 kV two-circuit transmission line. Consequently, 2 banks of main transformers and the 230 kV transmission line will be connected to this power plant.

The 230 kV switchyard will be constructed at the upstream side of the powerhouse building.

As for the method of connection to the main bus of the 230 kV switchyard, the breaker and a half scheme (so-called $1\frac{1}{2}$ breaker) and the ring bus scheme (circuit breakers connected in a ring and the various circuits taken out from between these circuit breakers) can be taken into consideration in the studies as possible alternatives.



Ring bus scheme $1\frac{1}{2}$ CB scheme

Generally speaking, the ring bus scheme is economical compared with the $1\frac{1}{2}$ CB scheme, and former one is adopted when the number of outgoing circuits is less than 5. The scheme will be constructed in accordance with the ring bus scheme when the number of circuits to be used at the beginning is less than 5, and the design may take into consideration the possibility of change to the $1\frac{1}{2}$ CB scheme in order to cope with future increase in the number of circuits if there is such a possibility. However, since this power station will operate with two circuits of 230 kV transmission line, and 2 units of transformer, without possibility of outgoing circuits in the future, the ring bus scheme will be adopted, and the possibility of conversion to the $1\frac{1}{2}$ CB is not taken into consideration.

The area of the switchyard will be approximately 5,124 m^2; approximately 61 m in width and 84 m in length. (see Fig.3-2-14, Fig.3-2-15)

(7) Communication equipment

The communication system will be power line carrier equipment utilizing two 230 kV transmission lines between the power plant and the Santiago substation, and will comprise the following facilities:

(i) Power line carrier telephone
(ii) Carrier relay (between the power plant and the Santiago substation)
(iii) Remote control (between the power plant and dispatching center)

Table 3-2-1 Monthly and Annual Energy Generated

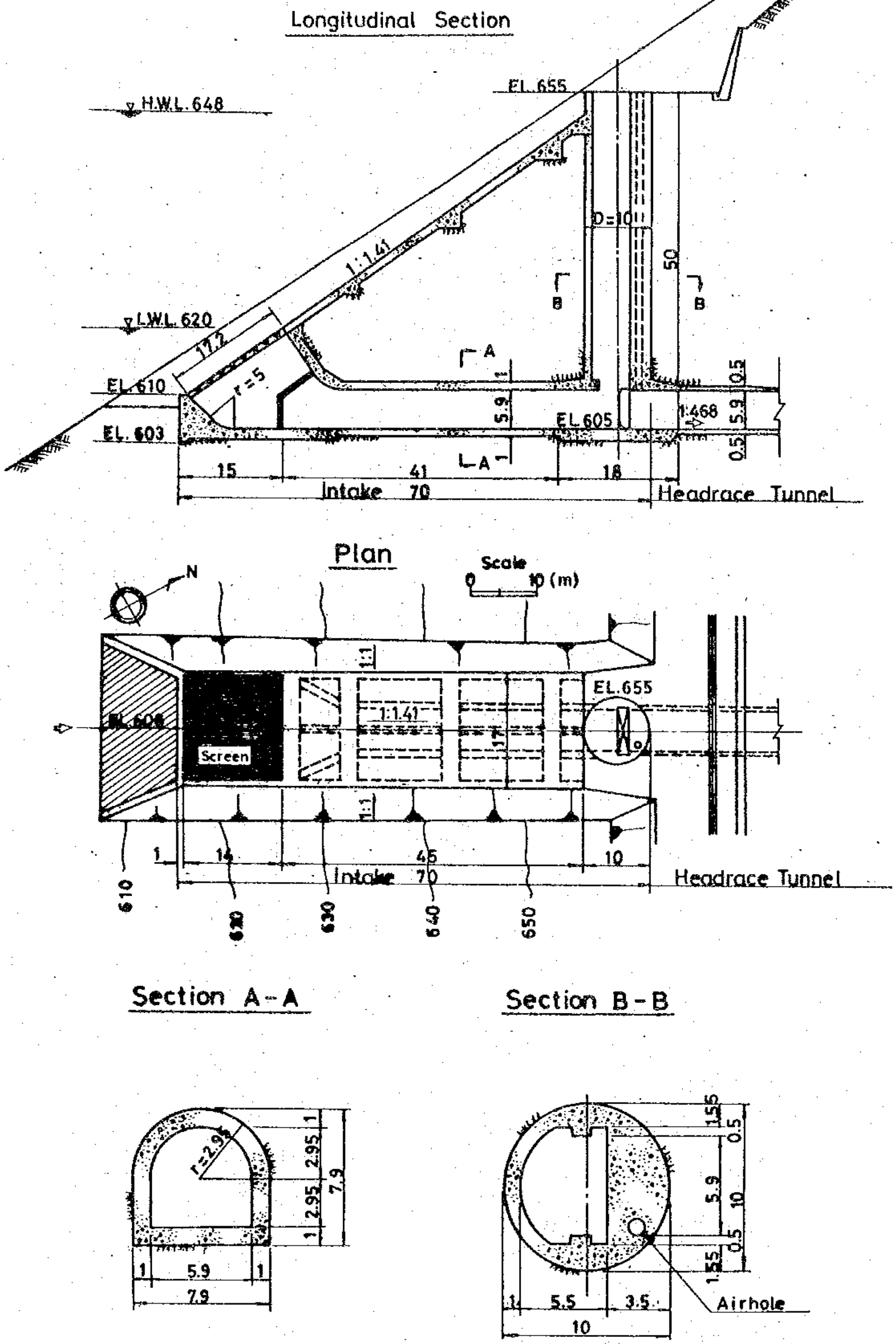
(Unit: 10^6 kWh)

Year / Month	1960	1961	1962	1963	1964	1965	1966	1967	1968	1969	1970	1971	1972	1973	1974	1977
Jan.	102.1	76.1	77.6	76.6	77.2	127.6	68.2	173.2	94.1	60.0	59.5	114.7	99.1	59.8	59.8	60.5
Feb.	126.3	55.4	57.3	68.1	62.3	90.7	54.7	59.1	56.6	54.1	53.9	67.9	56.6	54.0	54.0	54.6
Mar.	72.7	65.9	60.6	60.6	60.6	60.6	60.4	60.5	60.4	59.7	59.6	115.4	60.3	59.6	59.6	60.3
Apr.	58.6	58.6	58.6	58.6	58.6	58.6	58.3	58.5	58.3	57.5	57.5	74.0	58.1	57.3	57.3	58.2
May	60.5	60.5	60.5	60.4	60.5	60.4	60.3	60.3	60.1	59.1	59.2	114.0	59.8	58.9	58.8	60.1
June	58.6	58.5	58.5	58.5	58.5	58.3	58.5	58.2	58.1	56.8	57.2	94.5	57.7	56.6	56.9	58.1
July	62.9	60.5	60.5	60.5	61.7	60.2	60.4	59.9	60.1	58.2	58.9	123.4	59.6	58.0	58.9	60.1
Aug.	100.1	63.3	89.9	133.2	82.6	60.3	60.4	59.8	60.0	57.7	58.7	60.6	59.7	57.4	58.8	60.1
Sep.	80.8	70.4	82.9	92.5	88.8	58.4	58.5	58.0	58.1	55.5	56.9	108.8	57.7	55.1	56.8	58.3
Oct.	157.3	97.4	90.5	72.6	215.4	74.8	60.4	60.3	60.1	57.3	59.6	242.3	59.5	56.9	58.9	60.4
Nov.	78.9	126.9	150.4	59.6	234.5	105.4	234.2	233.8	58.2	55.9	233.9	234.5	57.5	56.2	57.8	79.1
Dec.	87.2	117.4	109.7	130.6	200.6	117.6	242.3	119.5	60.1	58.8	202.3	242.3	59.7	59.4	241.9	103.3
Total	1,046.0	910.9	957.0	931.8	1,261.3	932.9	1,076.6	1,061.1	744.2	690.6	1,017.2	1,592.4	745.3	689.2	879.5	773.1

Total Energy : 15,309.1 x 10^6 kWh (for 16 years)

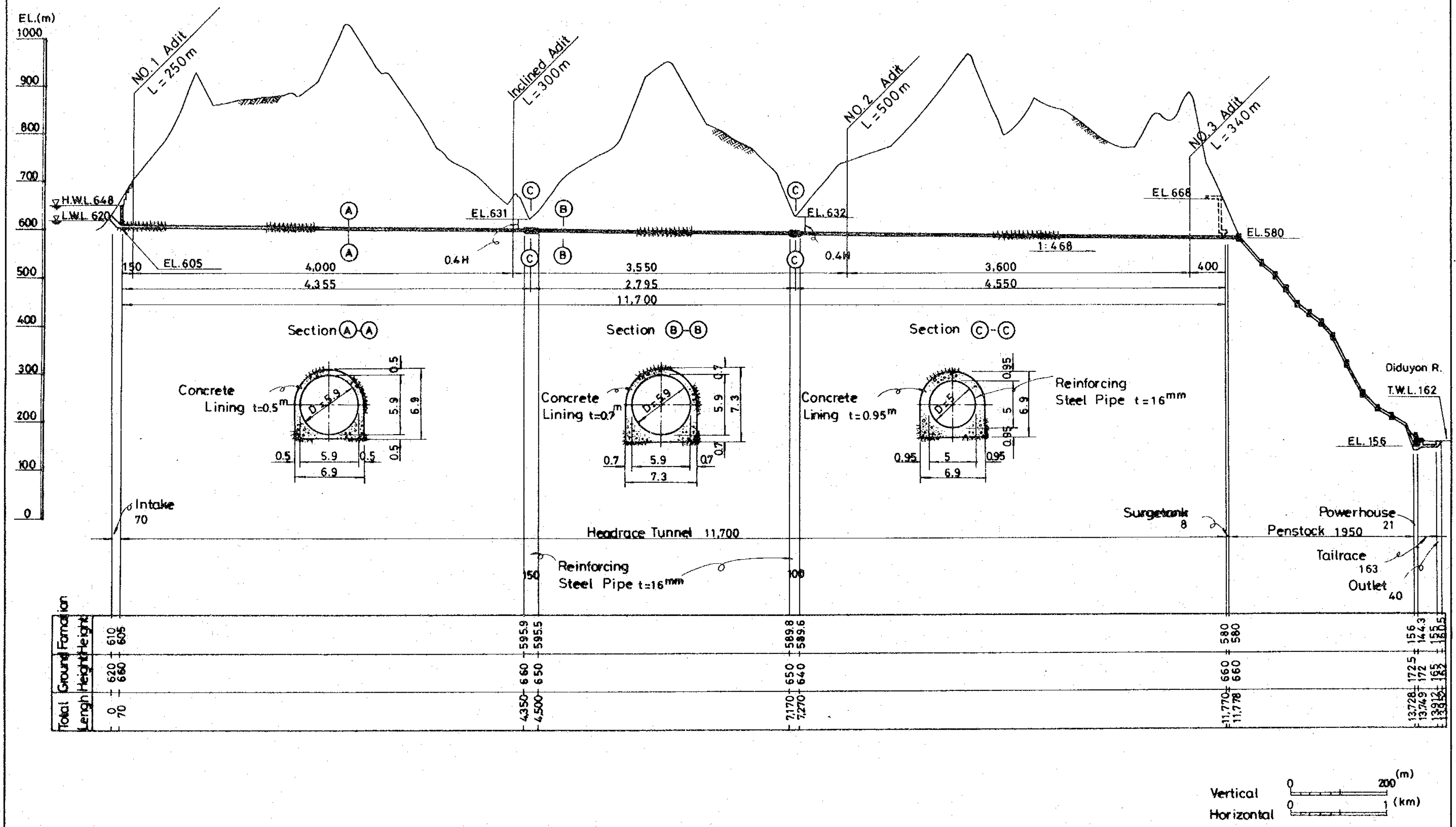
Average Annual Energy : 956.8 x 10^6 kWh

Plan and Section of Intake Structures



Diduyon Hydroelectric Project Upper Cagayan River Republic of the Philippines
Japan International Cooperation Agency
Plan and Section of Intake Structures
October 1980 Fig. 3-2-1

Profile and Section of Waterway, Headrace and Tailrace

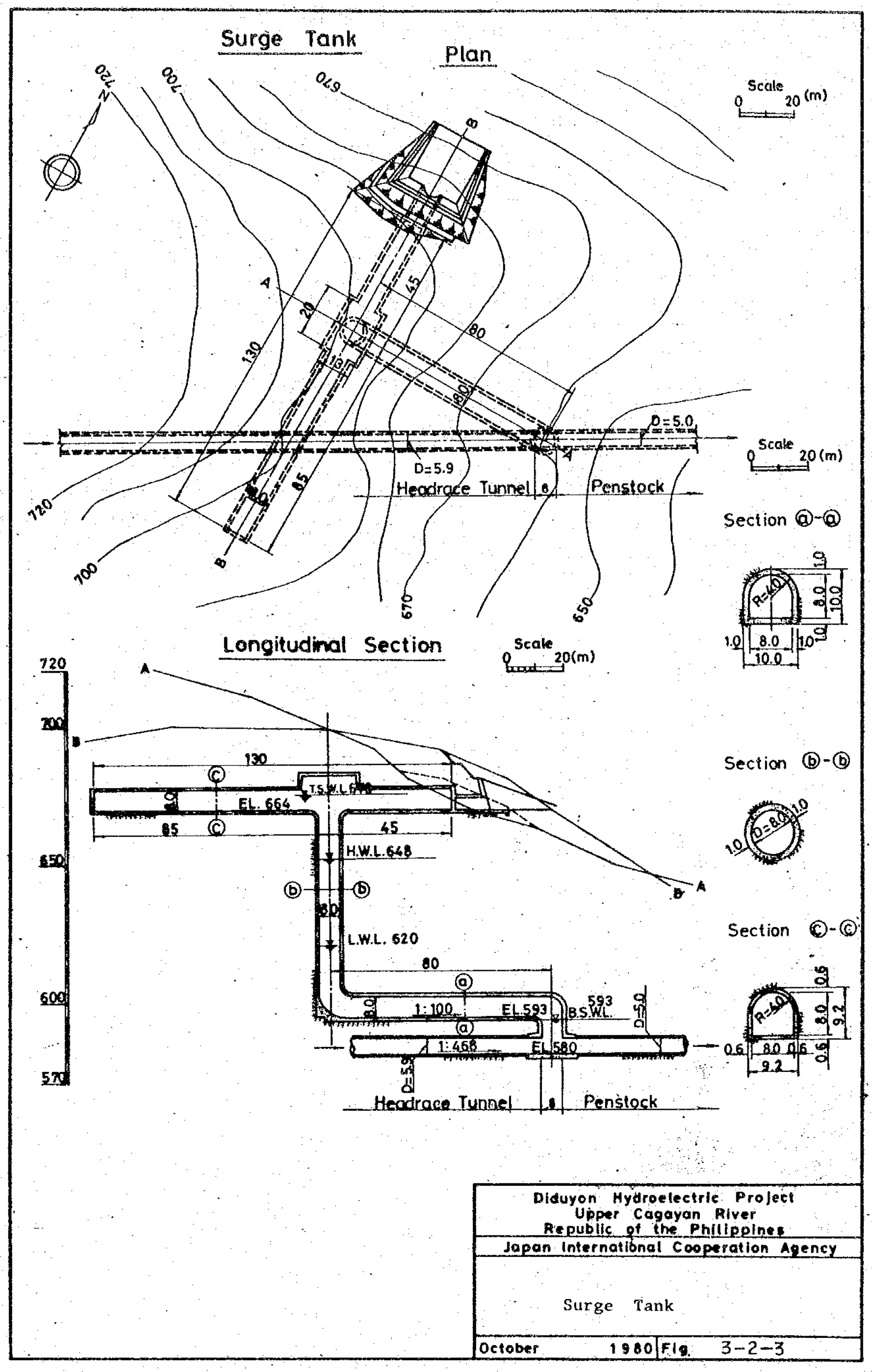


Diduyon Hydroelectric Project
Upper Cagayan River
Republic of the Philippines

Japan International Cooperation Agency

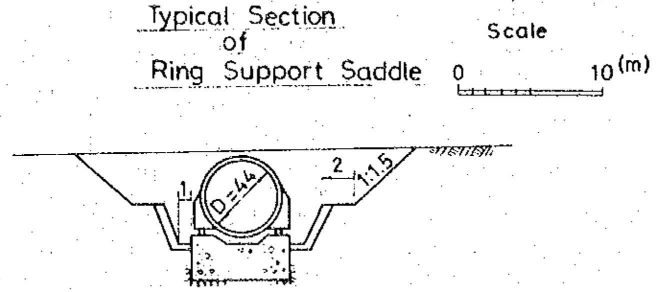
Profile and Section of Waterway,
Headrace & Tailrace

October 1980 Fig. 3-2-2

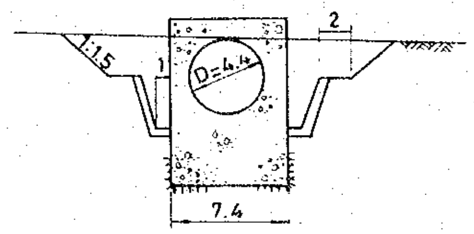

Surge Tank
Plan
Scale 0 20 (m)
720
700
670
650
N
A
B
130
20
45
80
13
180
85
D=5.0
D=5.9
Headrace Tunnel
Penstock
Section ⓐ-ⓐ
R=4.0
1.0
8.0
10.0
Longitudinal Section
Scale 0 20(m)
600
570
T.S.W.L 690
EL. 664
H.W.L. 648
L.W.L. 620
1:100
EL.593
593
B.S.W.L.
1:468
EL.580
D=5.9
Section ⓑ-ⓑ
D=8.0
Section ⓒ-ⓒ
0.6
9.2
Diduyon Hydroelectric Project
Upper Cagayan River
Republic of the Philippines
Japan International Cooperation Agency
Surge Tank
October 1980
Fig. 3-2-3

Penstock

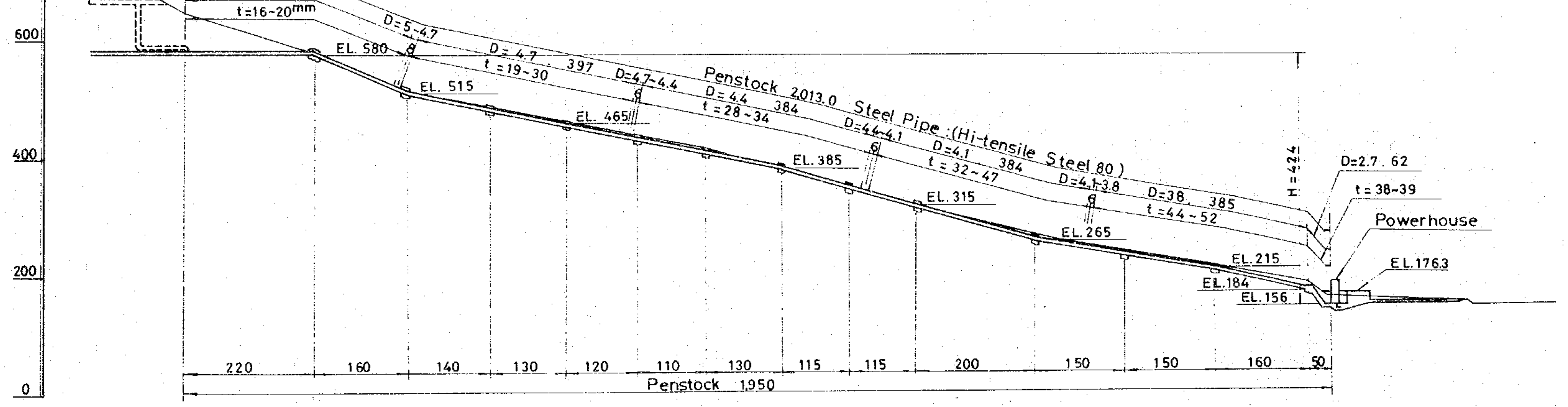
Typical Section of Ring Support Saddle



Anchor Block



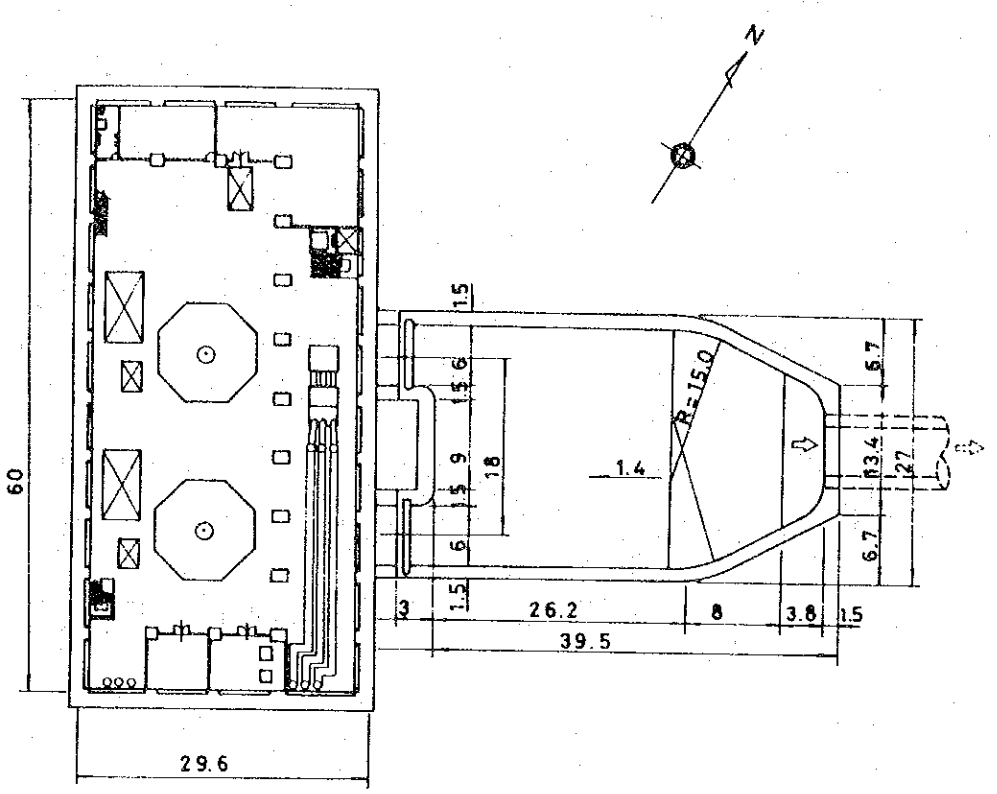
Longitudinal Profile



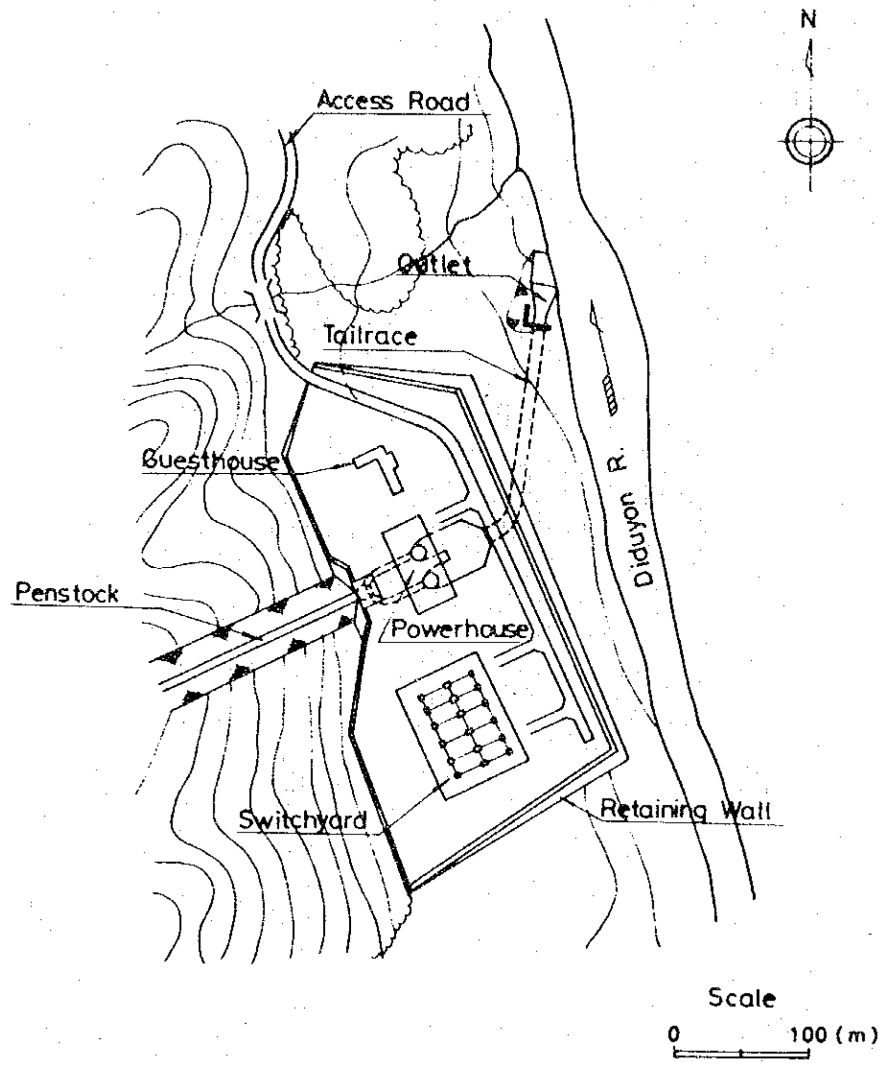
Diduyon Hydroelectric Project Upper Cagayan River Republic of the Philippines
Japan International Cooperation Agency
Penstock
October 1980 Fig. 3-2-4

3-79, 3-80

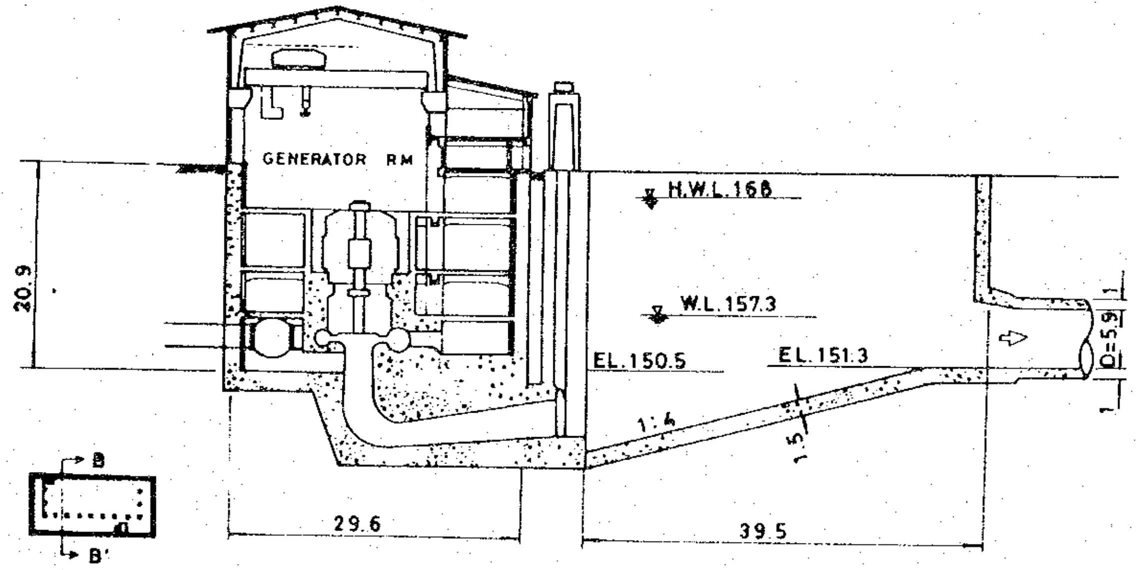
Plan of Powerhouse



Plan Around Powerhouse



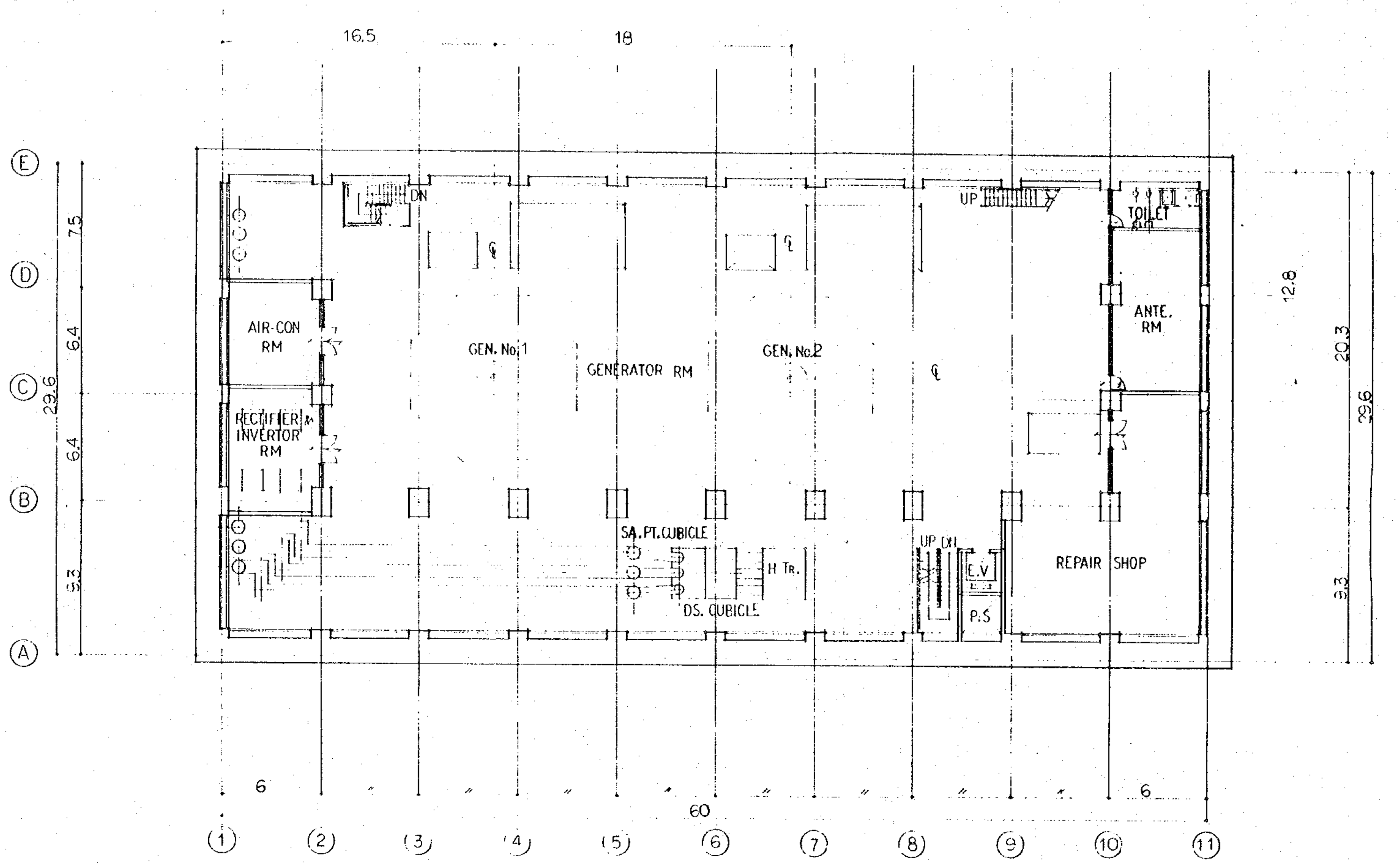
Longitudinal Section



Diduyon Hydroelectric Project Upper Cagayan River Republic of the Philippines	
Japan International Cooperation Agency	
Powerhouse and Tailrace	
October 1980	Fig. 3-2-5

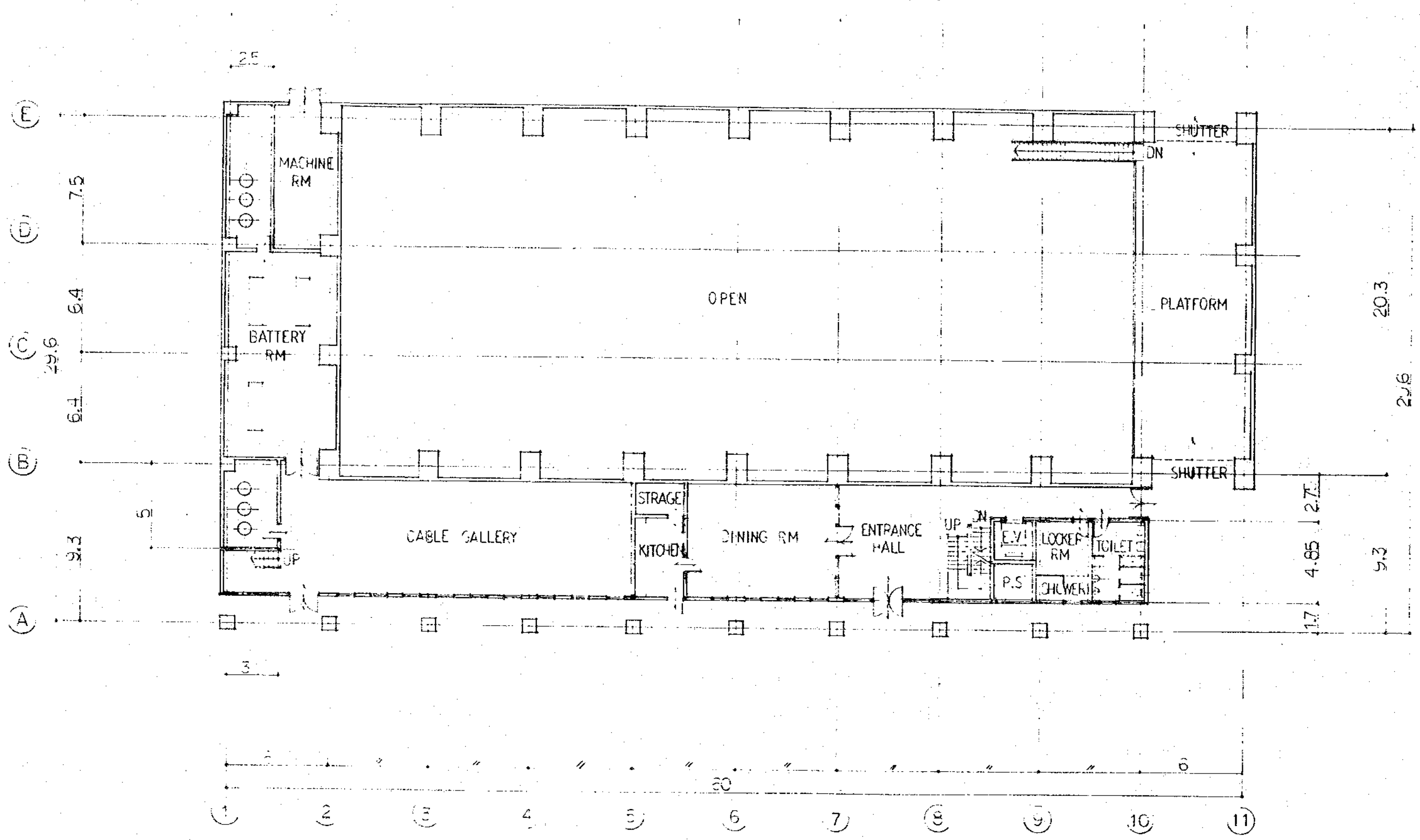
3-81, 3-82

Archtectural Plan of Powerhouse M.F. (EL.17 2m)



Diduyon Hydroelectric Project Upper Cagayan River Republic of the Philippines	
Japan International Cooperation Agency	
Architectural Plan of Power-house M.F. (EL. 172)	
October 1980	Fig. 3-2-6

Guest House, 1F. Plan (EL. +176.5)

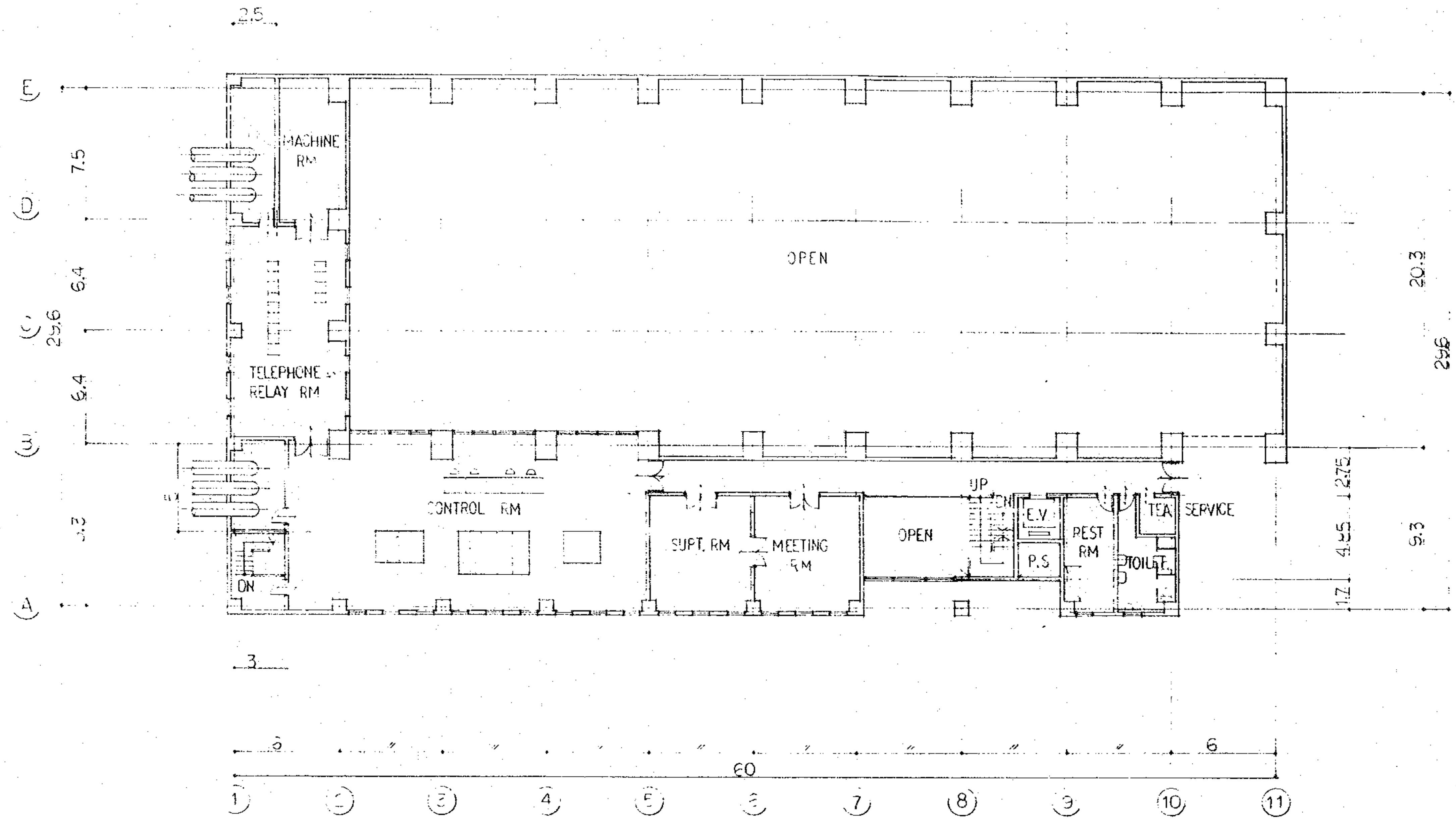


Diduyon Hydroelectric Project
Upper Cagayan River
Republic of the Philippines
Japan International Cooperation Agency

Guest House, 1F.
Plan (EL. +176.5)

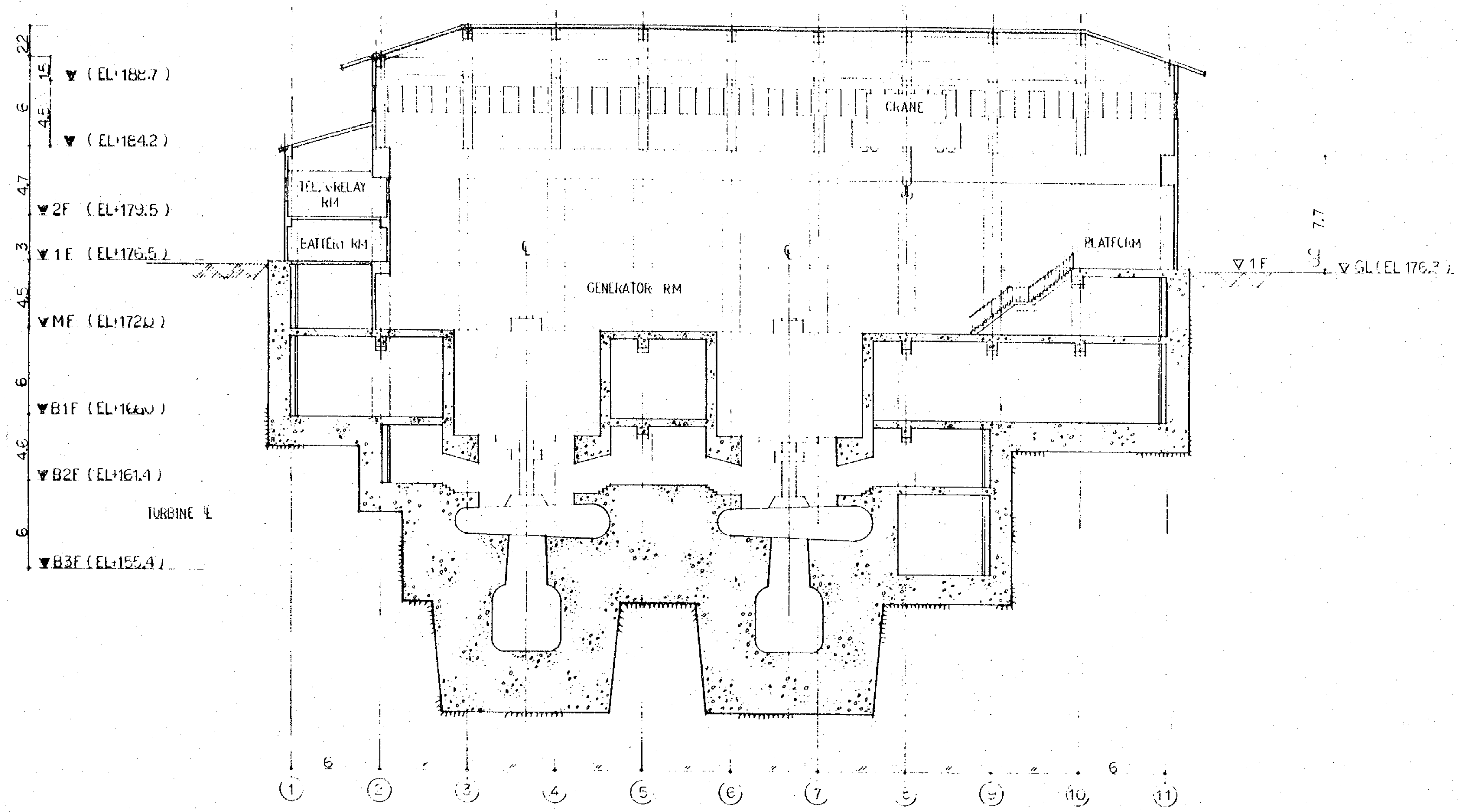
October	1980	Fig. 3-2-7

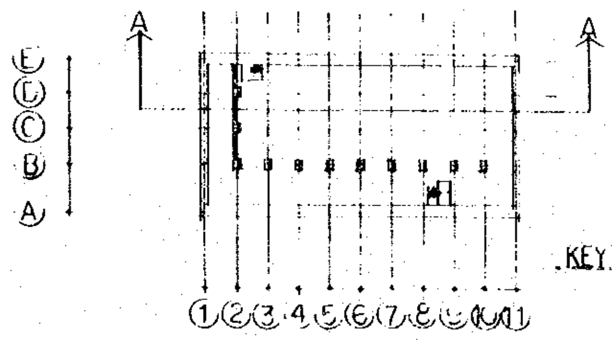
Architectural Plan of Powerhouse 2F. (EL.+179.5)



Diduyon Hydroelectric Project Upper Cagayan River Republic of the Philippines
Japan International Cooperation Agency
Architectural Plan of Power-house 2F (EL. +179.5)
October 1980 Fig. 3-2-8

Typical Logitudinal Section of Powerhouse

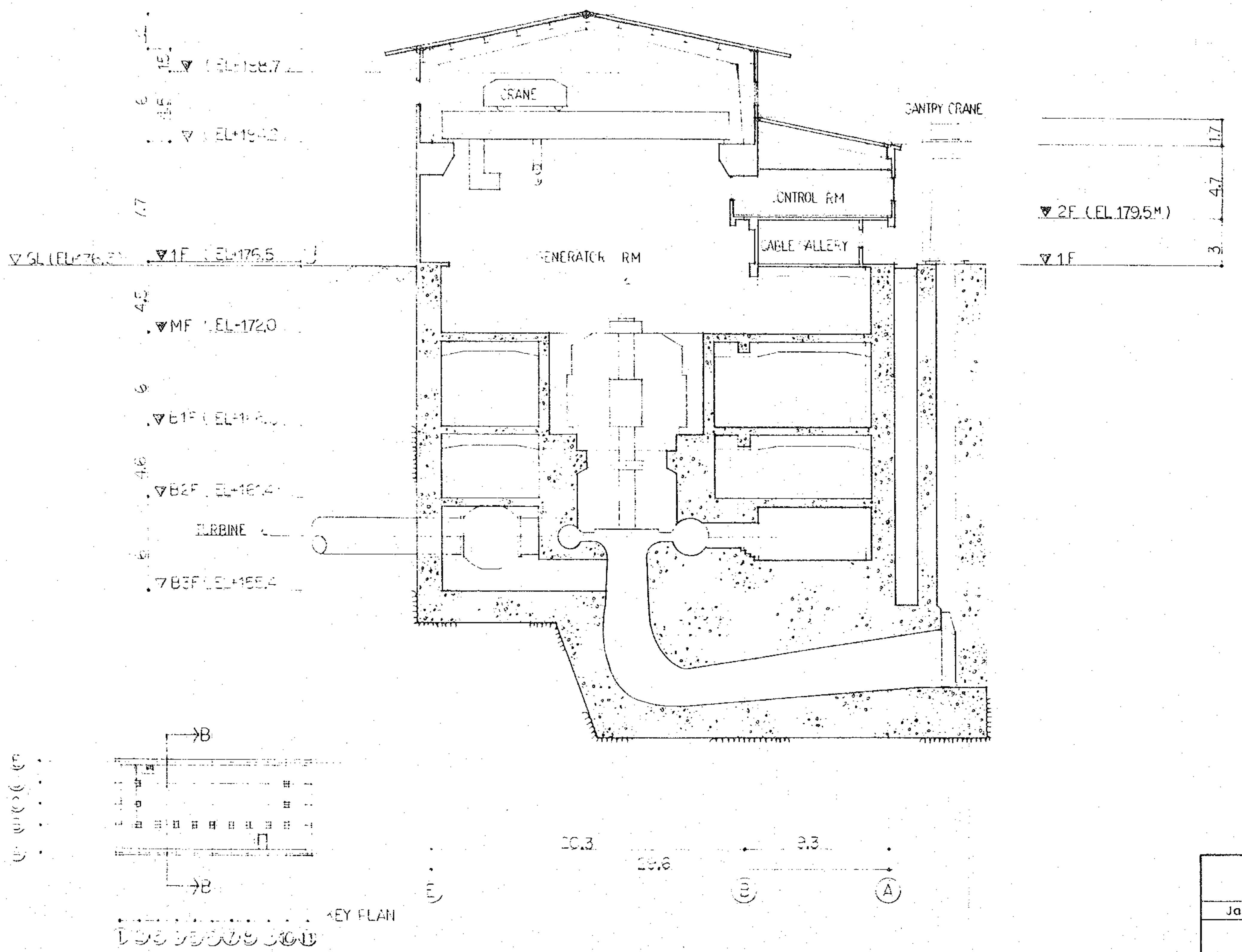




KEY PLAN

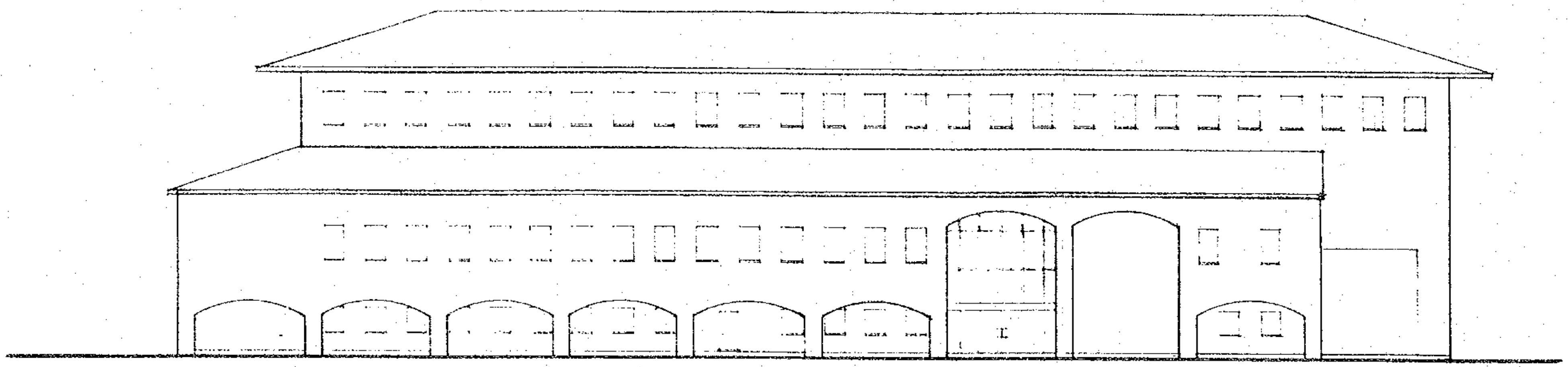
Diduyon Hydroelectric Project Upper Cagayan River Republic of the Philippines	
Japan International Cooperation Agency	
Typical Logitudinal Section of Powerhouse	
October 1980	Fig. 3-2-9

Typical Cross-Section of Powerhouse

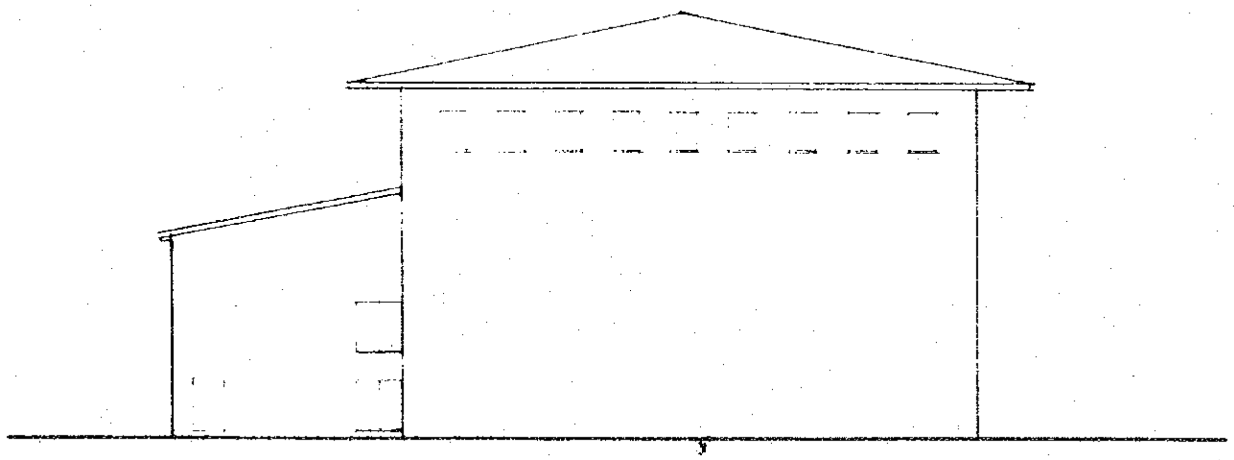


Diduyon Hydroelectric Project Upper Cagayan River Republic of the Philippines
Japan International Cooperation Agency
Typical Cross-Section at Powerhouse
October 1980 | Fig. 3-2-10

Facade of Powerhouse



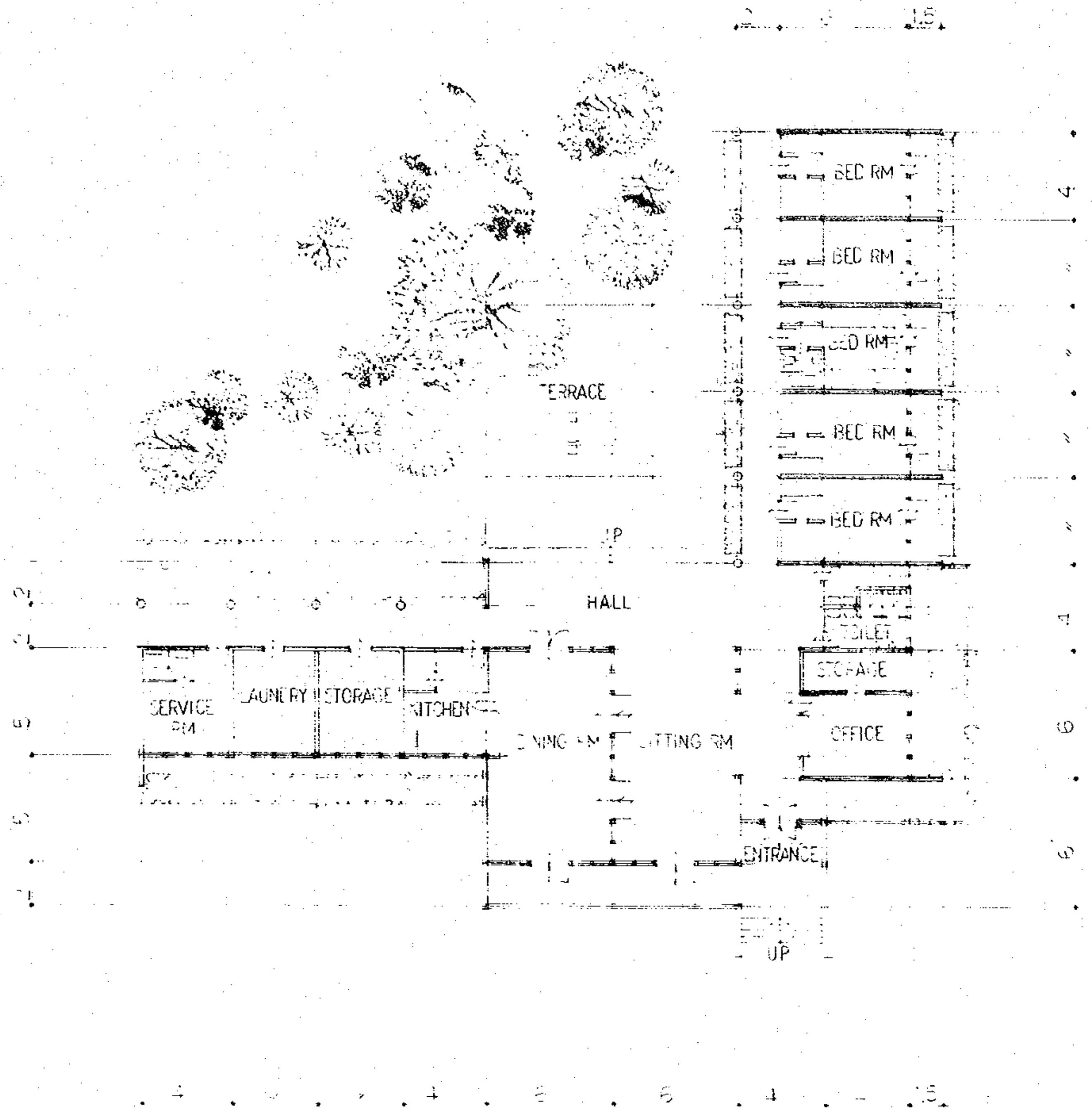
FRONT ELEVATION



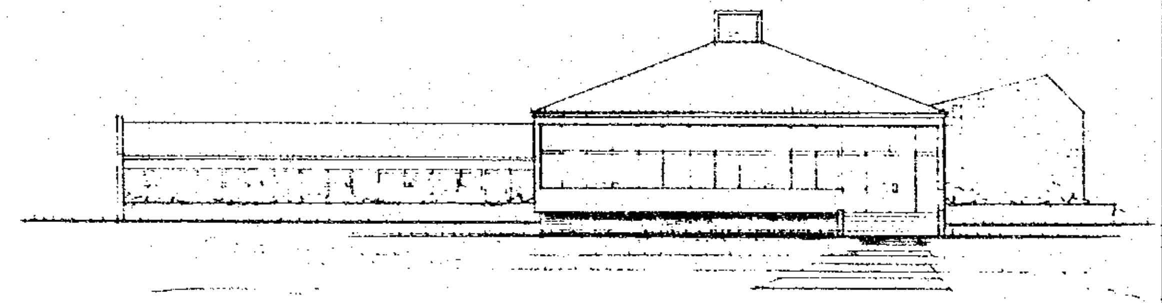
[illegible] ELEVATION

Diduyon Hydroelectric Project Upper Cagayan River Republic of the Philippines
Japan International Cooperation Agency
Facade of Powerhouse
October 1980 Fig. 3-2-11

Guest House, Facade and Floor Plan



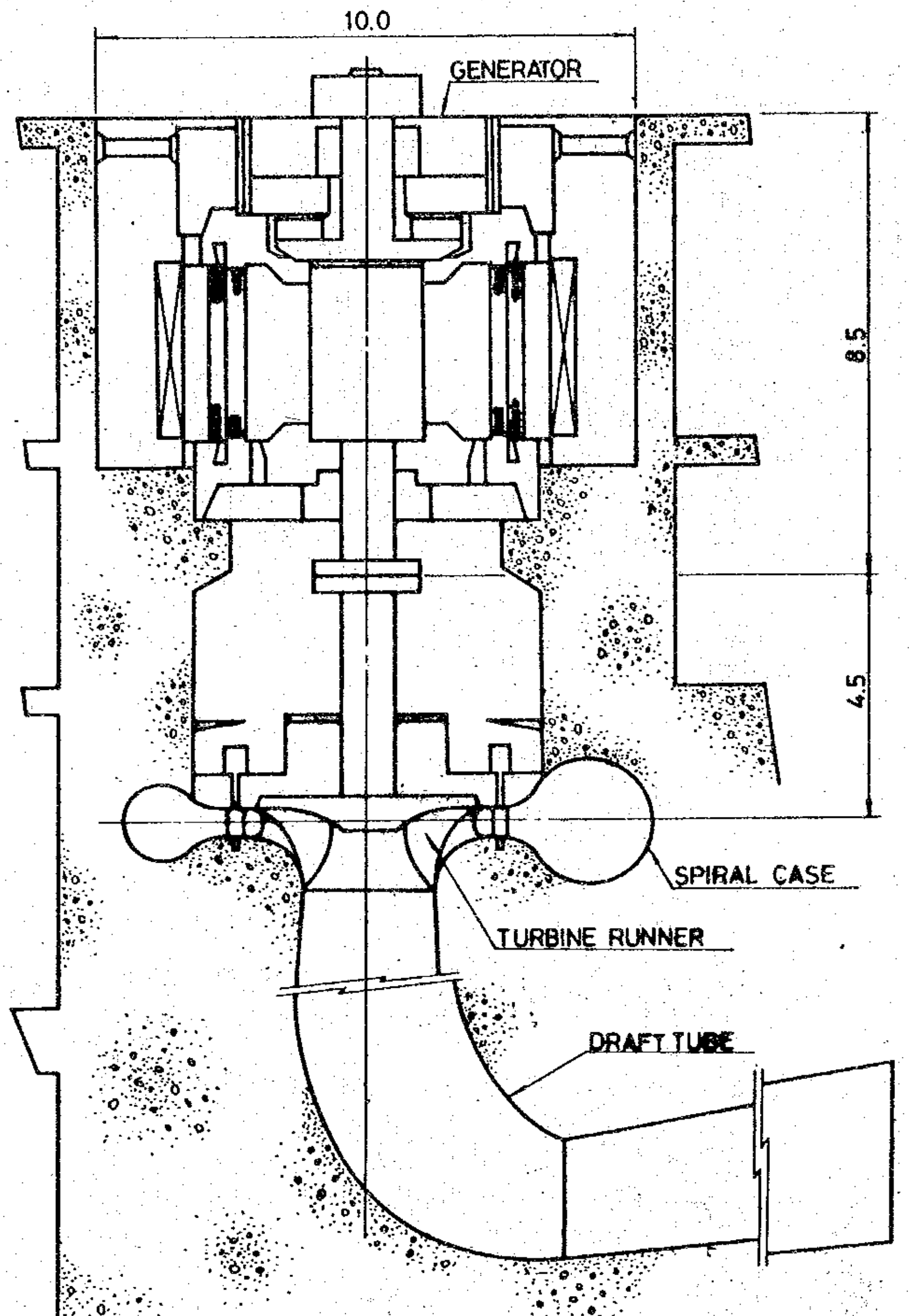
FLOOR PLAN



FRONT ELEVATION

Diduyon Hydroelectric Project Upper Cagayan River Republic of the Philippines
Japan International Cooperation Agency
Guest House, Facade and Floor Plan
October 1980 Fig. 3-2-12

Cross-Section of Turbine and Generator



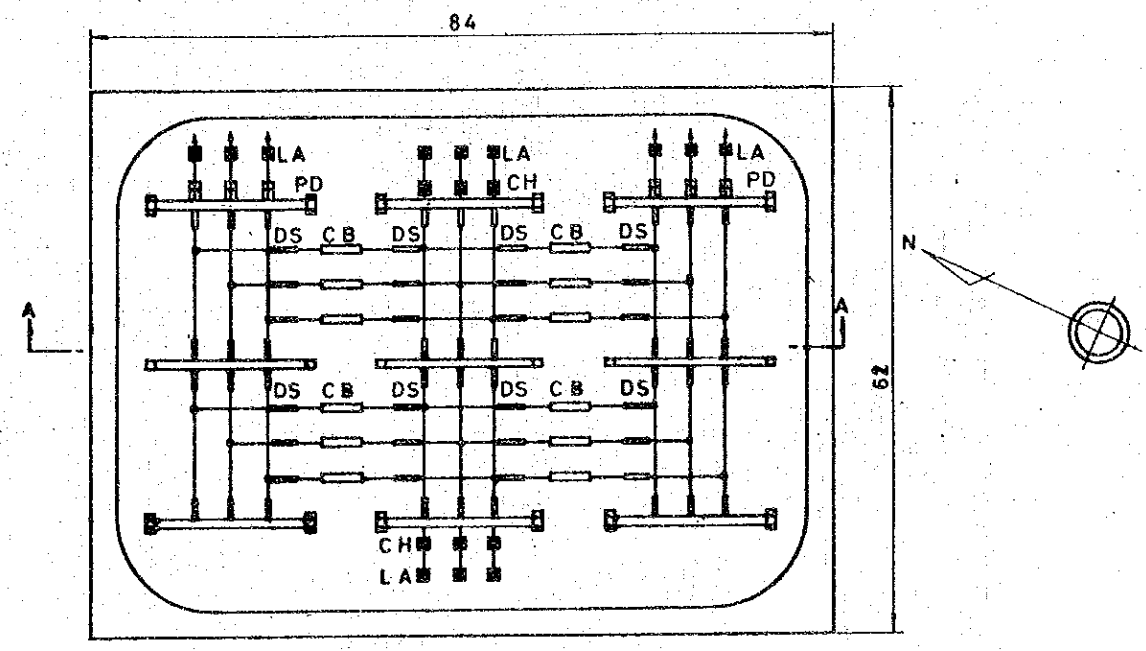
Diduyon Hydroelectric Project Upper Cagayan River Republic of the Philippines
Japan International Cooperation Agency
Cross-Section of Turbine and Generator
October 1980 Fig. 3-2-13

Plan and Section of Switchyard Equipment

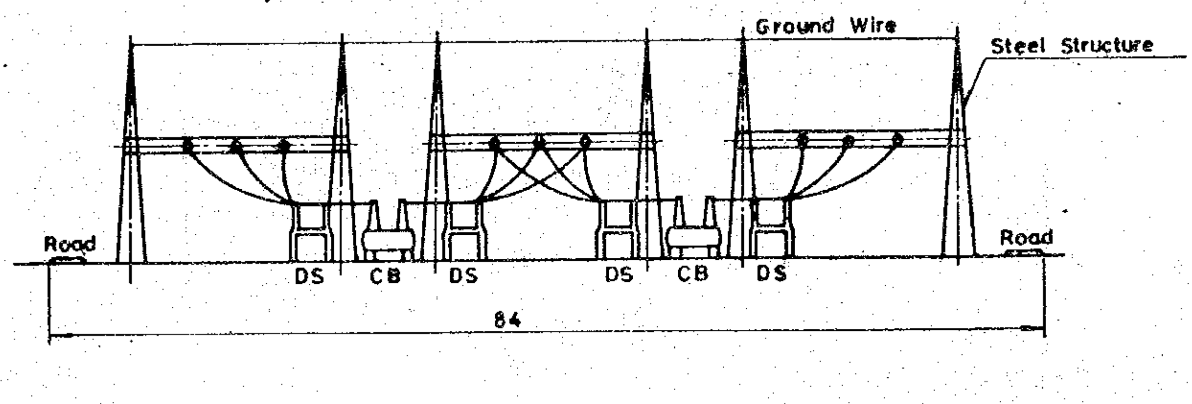
Legend

L A : Lightning Arrestor
P D : Potentiol Device
D S : Disconnecting Switch
C B : Circuit Breaker
C H : Cable Head

Plan



A - A Section

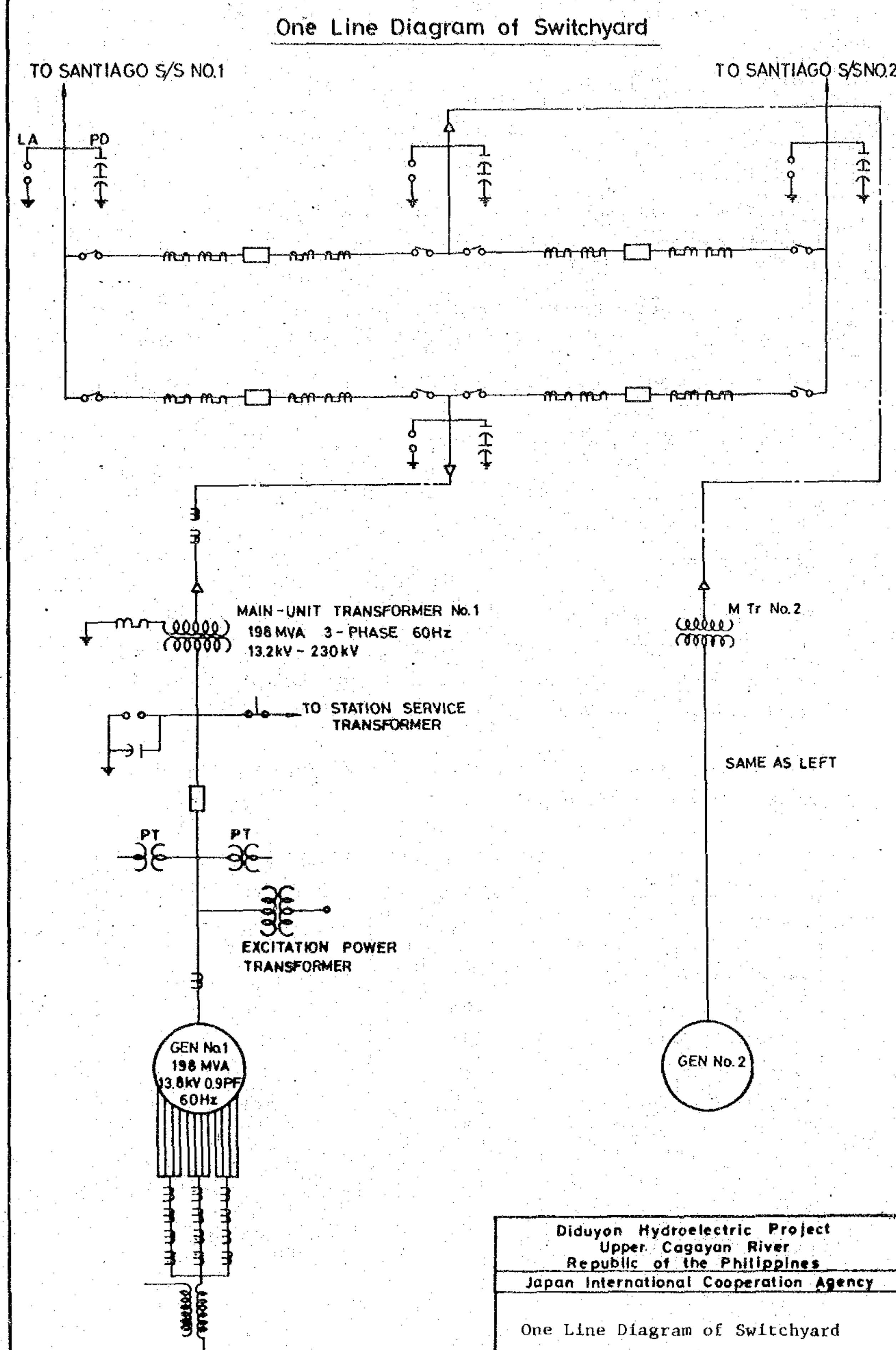


Diduyon Hydroelectric Project
Upper Cagayan River
Republic of the Philippines

Japan International Cooperation Agency

Plan and Section of
Switchyard Equipment

October 1980 Fig. 3-2-14



Diduyon Hydroelectric Project Upper Cagayan River Republic of the Philippines	
Japan International Cooperation Agency	
One Line Diagram of Switchyard	
October 1980	Fig. 3-2-15

3.3. Determination of Plant Capacity

3.3.1. Basic conditions

The studies will be carried out based upon the following conditions:

- Intake level	Maximum water level, HWL	648 m
	Minimum water level, LWL	620 m
- Tail water level		162 m
- Turbine discharge		85.2 m^2/sec.

3.3.2. Effective head

The loss head and effective head corresponding to the maximum turbine discharge are presented on Table 3-3-1. The computation of all loss head in various components of the project is made in accordance with the "Hydraulic Formulae, J.S.C.E., 1971".

The effective head for the maximum and normal peaking capacity respectively is given as follows.

Item	For maximum capacity	For normal peaking capacity
Turbine discharge (m^3/sec)	85.2	85.2
Intake level (m)	648	620
Tail water level (m)	162	162
Gross fall (m)	486	452
Loss head (m)	35	35
Effective head (m)	451	423

3.3.3. Calculation of generator output

(1) Theoretical hydraulic power

$$Pe = 9.8 \times He \times Qp$$

where,

Pe : Theoretical hydraulic power (kW)

He : Effective head for normal peak load = 423 m

Qp : Turbine discharge (m^3/sec) = 85.2 m^3/sec

$$Pe = 9.8 \times 423 \times 85.2 = 353,188 \text{ (kW)}$$

(2) Generating output

$$E = \eta \times Pe$$

Where,

E : Generating power

Pe : Theoretical hydraulic power

η : Resultant efficiency of the power plant : 0.8736 (= turbine efficienty : 0.896 x generator efficiency : 0.975)

$$E = 0.8736 \times 353,188 = 308,545 \text{ (kW)}$$

The electric equipment is calculated according to the procedures indicated in 3-2-8, based upon the above-mentioned figures.

The generating output is especially used in economic analysis of the Diduyon power plant.

Table 3-3-1

Calculation of Loss Head with Formulae Adopted and Results for $Q = 85.2\ m^3/s$

Structural component	Loss due to	Formula adopted	Constant adopted, or calculating Procedure	Loss head (m) Obtained results	Other loss	Total	Subtotal	Grand Total
	1.1. Inlet	$hi = (1 + fe) \cdot \frac{v_2^2}{2g}$	fe = 0.2	0.062	0.038	0.100		
	1.2. Separating pier	$h_2 = \frac{Q^2}{2g} \{ \frac{1}{c^2 b_2^2 (H_1 - \Delta hp)} - \frac{1}{b_1^2 \cdot H_1^2} \}$	Repetition method	0.001	-	0.001		
1. Intake	1.3. Screen	$h_3 = \beta \sin\theta (\frac{t}{b})^{4/3} \cdot \frac{v_1^2}{2g}$		0.001	-	0.001	0.330	
	1.4. Friction in inlet tunnel	$h_4 = f' \cdot \frac{L}{R} \cdot \frac{v^2}{2g}$	n = 0.013	0.119	0.031	0.150		
	1.5. Allowance			-	-	0.078		
	2.1. Inlet	$h_1 = fe \cdot \frac{v^2}{2g}$	fe = 0.2	0.099	0.021	0.120		
	2.2. Friction	$h_2 = f \cdot \frac{L}{D} \cdot \frac{v^2}{2g}$	n = 0.012, n = 0.013	11.841	0.069	11.910		
2. Headrace tunnel	2.3. Gradual reduction of section	$h_3 = fgc \cdot \frac{v_2^2}{2g}$	fgc = 0.0075	0.014	0.006	0.020	15.990	
	2.4. Gradual enlargement of section	$h_4 = fgc \cdot fse \cdot \frac{v_1^2}{2g}$	fgc = 0.65, fse = 0.125	0.156	0.004	0.160		
	2.5. Allowance			-	-	3.780		
	3.1. Friction	$h_1 = f \cdot \frac{L}{D} \cdot \frac{v^2}{2g}$	n = 0.012	10.704	0.196	10.90		35 m
	3.2. Bifurcation	$h_2 = fd \cdot \frac{v_1^2}{2g}$	fd = 0.75	2.160	0.000	2.160		
3. Penstock	3.3. Bend	$h_3 = fb_1 \cdot fb_2 \cdot \frac{v^2}{2g}$		0.478	0.022	0.500	17.760	
	3.4. Allowance			-	-	4.200		
	4.1. Change of cross-section at afterbay	$h_1 = he + (\frac{v_2^2}{2g} - \frac{v_1^2}{2g})$, $he = fgc \cdot \frac{v_2^2}{2g}$	fgc = 0.485	0.057	-	0.057		
	4.2. Entrance	$h_2 = fsc \cdot \frac{v_2^2}{2g}$	fsc = 0.5	0.248	-	0.248		
4. Tainrace	4.3. Friction in tunnel	$h_3 = f \cdot \frac{L}{D} \cdot \frac{v^2}{2g}$	n = 0.013	0.107	-	0.107	0.920	
	4.4. Submerged outlet weir	$h_4 = h_1' - h_2'$, $Q = \mu' \cdot B \cdot h_2 \sqrt{2g(h_1' - h_2')}$	Repetition method	0.265	-	0.265		
	4.5. Allowance			-	-	0.243		

Computation was all made on the basis of "Hydraulic formulae, JSCE, 1971".

3.4. Stability Analysis of Dam

The basic design of the Diduyon Reservoir Dam as well as its stability analysis are all made in accordance with the "Design Criteria for Dams, Japanese National Committee on Large Dams, April, 1976".

Calculation is made assuming occurrence of earthquake with the dam at the high water level of the rivervoir.

3.4.1. Determination of Basic Dam Form and Elevation

(1) Elevations of various parts (See Fig. 3-4-1 - 3-4-4)

Dam crest	EL 651 m
Basic triangle crest	EL 654 m
High water level of reservoir	EL 648 m
Overflow portion crest	EL 633 m
Non-overflow portion crest	EL 651 m
Design elevation of silt deposit	EL 608 m
Foundation bedrock in standard cross section	
Overflow portion	EL 540 m
Non-overflow portion	EL 547 m

(2) Design water level and dam non-overflow portion crest elevation

The design water level is calculated by adding the various rises of water level as described hereunder to the high water level EL 648 m of the reservoir.

1) Increase of water level due to wind and waves

$$hw = 0.00086\ V^{1.1}\ F^{0.45}$$

where;

V : 10-minute average wind speed 30 m/sec.

F : Fetch distance 3,000 m

$$hw = 0.00086 \times (30)^{1.1} \times (3{,}000)^{0.45} \fallingdotseq 1.33 \text{ m}$$

2) Rise of water level due to earthquakes

$$he = \frac{1}{2}\, k \cdot \tau / \pi \sqrt{g \cdot H_0}$$

where;

k : Horizontal seismicity 0.12

τ : Period of seismic waves in second

he : Depth of reservoir water = 648 m - 540 m = 108 m

$$he = \frac{1}{2} \times 0.12 \times \frac{1}{\pi} \times \sqrt{9.8 \times 108} = 0.62 \text{ m}$$

3) Rise of water level due to unexpected accidents in operating spillway gates

ha = 0.50 m

4) Additional height to freeboard according to type of dam

hi = 0, because the dam is of concrete gravity type.

5) Determination of basic dam form and elevations

The dam will be quite safe against any kind of water level rises if the elevation of the non-overflowing dam part is chosen at EL 651 m, namely 3 m or approximately 3% of the given depth of reservoir water (108 m) higher than the normal high water level of the reservoir (EL 648 m).

As for the form of the basic triangle of the dam, a triangle with the top elevation at EL 654 m and slope gradients of 1 : 0.1 at the upstream side and 1 : 0.8 at the downstream side is adopted, which will be checked and partially modified to the safer side by the following stability analysis of dam.

3.4.2. Stability Analysis of Dam

Since the Diduyon Dam is of concrete gravity type, its stability is analysed in accordance with the well-established conventional method.

(1) Basic Requirements for Analysis

1) Various numerical data for design and stability analysis of dam

Unit volume weight of dam body concrete	$r_0 = 2.35\ t/m^3$
Unit weight of pier concrete	$r_1 = 2.40\ t/m^3$
Unit weight of water	$w_0 = 1.00\ t/m^3$
Unit weight of silt deposit (apparent)	$w = 1.70\ t/m^3$
Unit weight of silt deposit (measured in water)	$w_1 = 1.05\ t/m^3$

where:

$w_1 = w - (1 - \gamma)w_0 = 1.70 - (1.0 - 0.35) \times 1.00 = 1.05\ t/m^3$

r = porosity = 35%

Coefficient of silt pressure	$Ce = 0.5$
Horizontal seismicity	$K = 0.12$

2) Allowable stress

Allowable compressive stress of concrete $Psa = 40\ kg/cm^2$

Allowable tensile strength of reinforcement bars (SD35)

$Pca = 2{,}000\ kg/cm^2$

Allowable bearing force of bedrock $300\ t/m^2$

3) Safety conditions

i) Sliding safety

Displacement formula

$$N = \frac{f_1 \Sigma v + \tau \ell}{\Sigma H} \geqq 4$$

where;

- N : Safety factor for shear friction
- f : Coefficient of internal friction 0.65
- v : Normal force (t)
- τ : Minimum value of shear strength = $250\ t/m^2$
- H : Horizontal force (t)
- ℓ : Length for shear resistance (m)

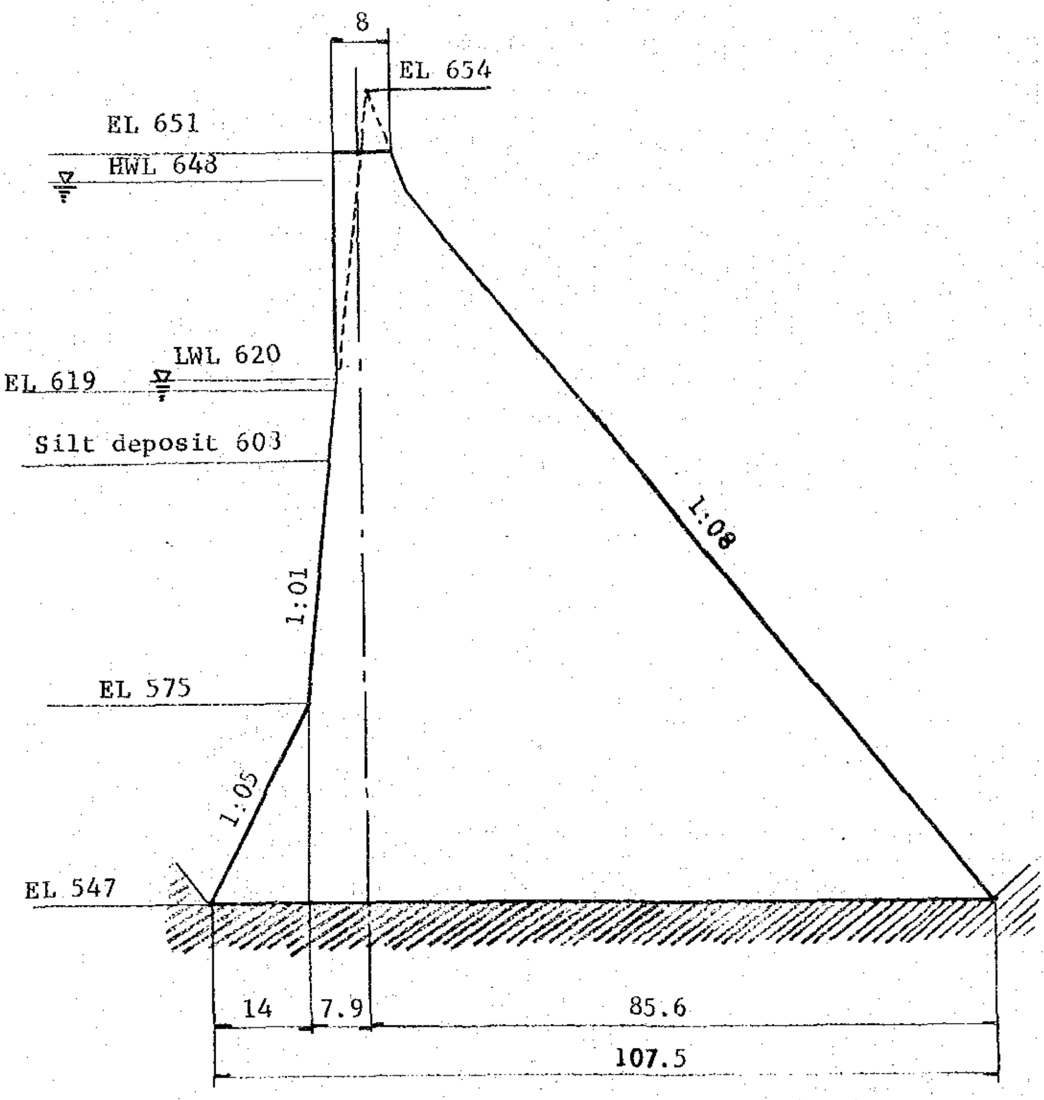
ii) *Compressive strength*

$$\sigma = \frac{\Sigma V}{e} \left(1 \pm \frac{\sigma \cdot e}{\ell}\right) \leqq \sigma a$$

where;

- ℓ : Width of foundation surface (m)
- V : Normal force (t)
- e : Offset (m)
- σa : Allowable stress of bedrock (kg/cm^2)

(2) Stability Analysis of Non-overflowing Section



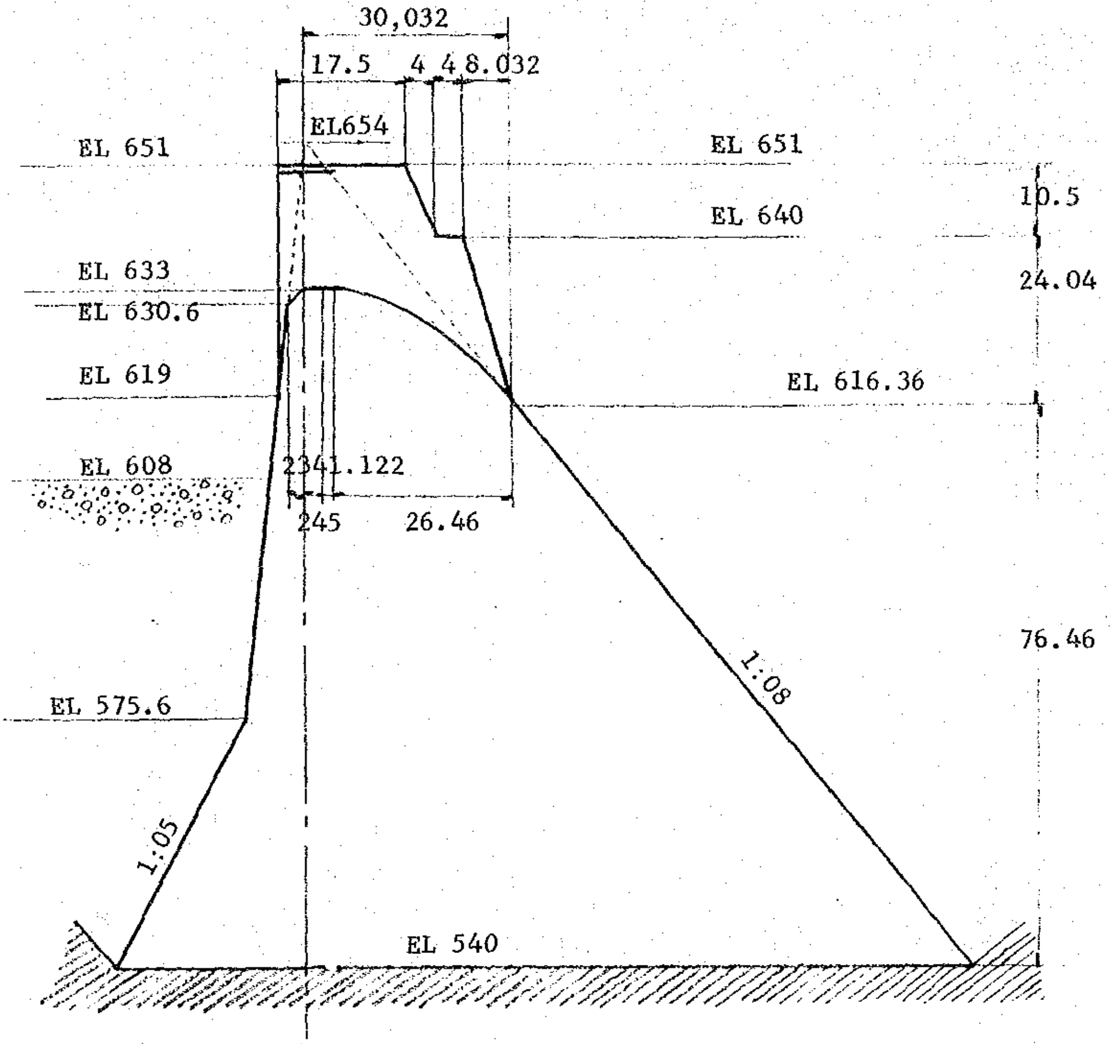
Item	External force V	External force H	Arm (m)	Moment (tm)
Weight of dam body	12,598.938		23,578	297,057.764
Water weight on upstream slope	1,487.480		-13,801	-20,528.711
Weight of silt deposit	748.073		-14,923	-11,163.493
Hydrostatic pressure		5,350.951	34,483	184,516.843
Hydraulic pressure		714.07	40,400	28,848.428
Silt pressure		781.41	20,383	15,888.410
Uplift	-2,444.200		2,985	-7,295.937
Earthquake force		1,511.873	35,460	53,611.017
Total	12,390.291	8,358.304		540,934.317

ΣV	ΣH	ΣM	ℓ	$\Sigma V \cdot f$ ($f=0.65$)	$\tau \cdot \ell$ ($\tau=250$ t/m^2)	$\Sigma V \cdot f + \tau \cdot \ell$	$N = \frac{\Sigma V \cdot f + \tau \cdot \ell}{\Sigma H}$
12,390.291	8,358.304	540,934.317	107.500	8,053.689	26,875	34,928.689	4.18
							> 4

$\frac{\Sigma M}{\Sigma V}$	e	$\frac{\sigma e}{\ell}$	$\frac{1-\sigma e}{\ell}$	$\frac{1+\sigma e}{\ell}$	$\frac{\Sigma V}{\ell}$	ℓu	ℓd
43.658	11.808	0.659	0.341	1.659	115.259	39.303	191.215
						< 300	

It is concluded from the results above that the planned crossection is safe.

(3) Stability Analysis of Overflowing Section



Analysis is made assuming occurrence of earthquake with the overflowing dam section at the high water level of the reservoir.

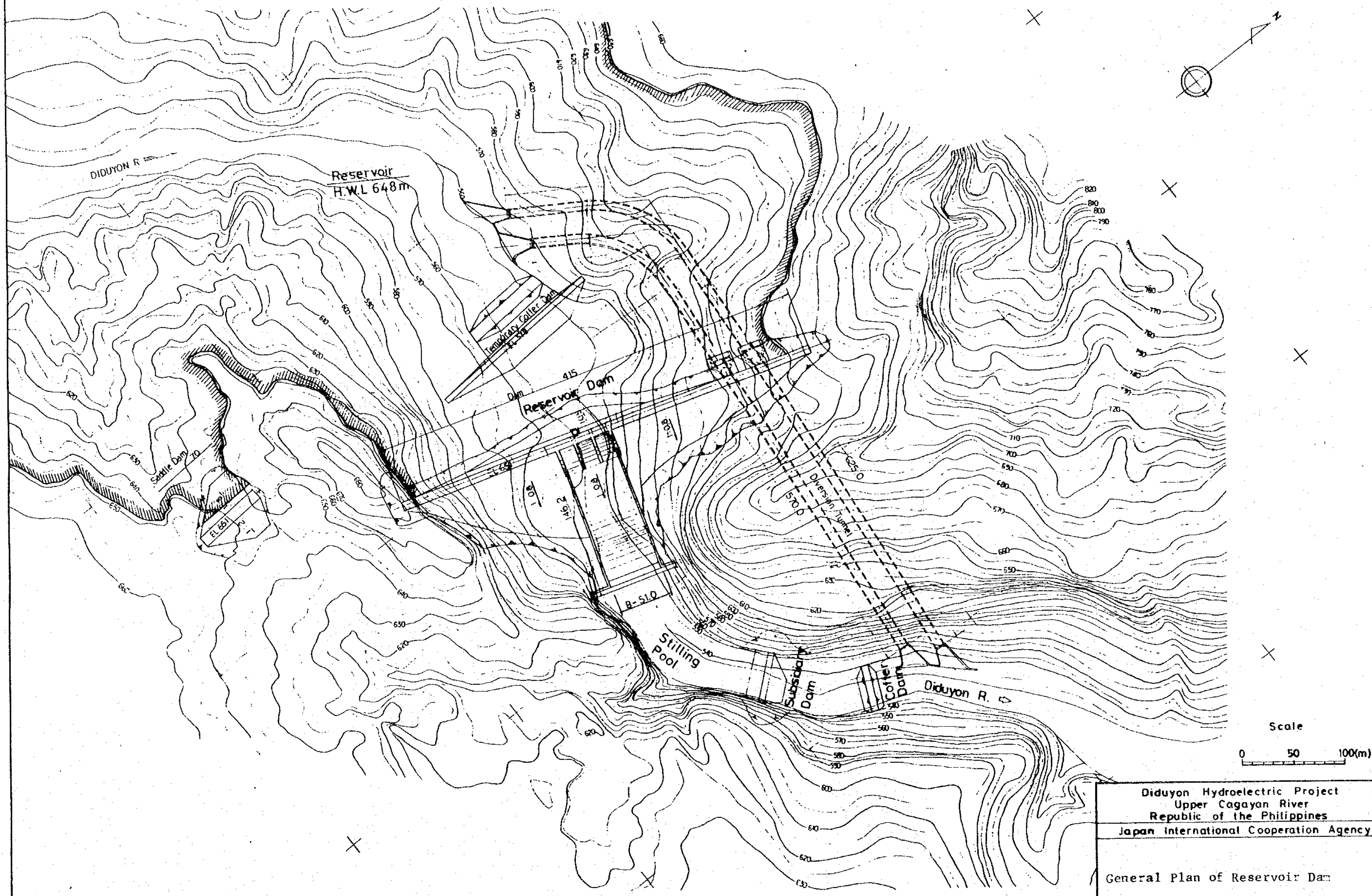
Item		External force (f) V	External force (f) H	Arm (m)	Moment (t-m)
Weight of accessories		3,980.831		9.950	39,609.268
Weight of dam body		247,663.6		25.676	6,359,010.593
Water weight on upstream slope	overflow part	1,879.95		- 0.495	- 930.575
	dam body	33,512.490		-15.785	- 528,994.655
Water weight on downstream slope		8,820.000		81.867	722,066.940
Hydrostatic pressure on upstream side			109,792.825	36.846	4,045,426.429
Hydrostatic pressure on downstream side			-11,025.000	11.667	- 128,628.675
Silt pressure			17,478.720	22.667	396,190.146
Weight of silt deposit		17,731.980		-17.031	- 301,993.351
Hydrodynamic pressure			14,696.641	43.20	634,394.891
Uplift		-108,752.090		22.744	-2,473,456.377
Earthquake force	on accessories		477.70	98.62	47,110.774
	on dam body		29,719.632	34.444	1,023,663.604
Total		204,836.810	161,140.510		9,833,968.388

ΣV	ΣH	ΣM	ℓ	$\Sigma V \cdot f$ (f=0.65)	$\tau \cdot \ell \cdot B$ (τ=250t)	$\Sigma V \cdot f + \tau \cdot \ell \cdot B$	$N = \frac{\Sigma V \cdot f + \tau \cdot \ell \cdot B}{\Sigma H}$
204,836.810	161,140.510	9,833,968.388	166.600	133,143.927	524.700	657,843.927	4.08
							> 4

$\frac{\Sigma M}{\Sigma V}$	e	$\frac{\sigma e}{\ell}$	$1-\frac{\sigma e}{\ell}$	$1+\frac{\sigma e}{\ell}$	$\frac{\Sigma V}{\ell \cdot B}$	σu	σd
48,009	15.109	0.777	0.223	1.777	97.597	21.764	173.430
						< 300	

It is concluded from the results above that the planned crosssection of the dam is also safe.

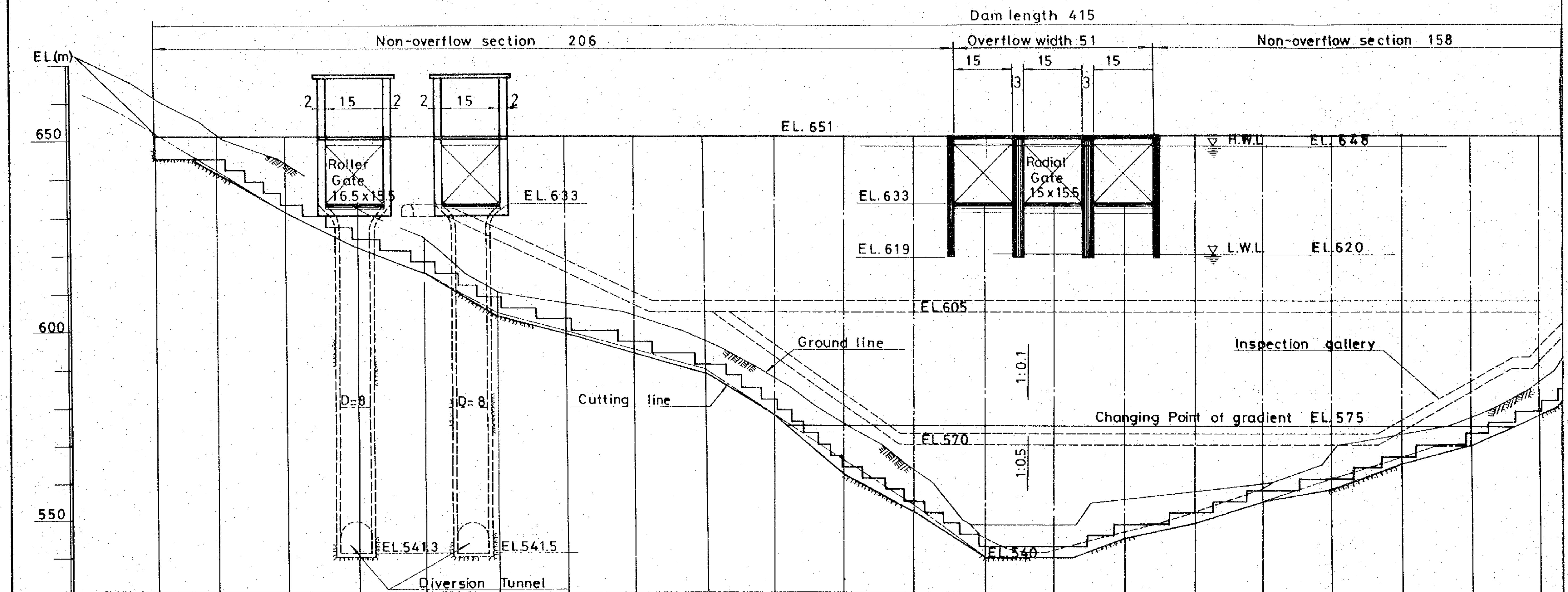
General Plan of Reservoir Dam



Diduyon Hydroelectric Project Upper Cagayan River Republic of the Philippines	
Japan International Cooperation Agency	
General Plan of Reservoir Dam	
October 1980	Fig. 3-4-1

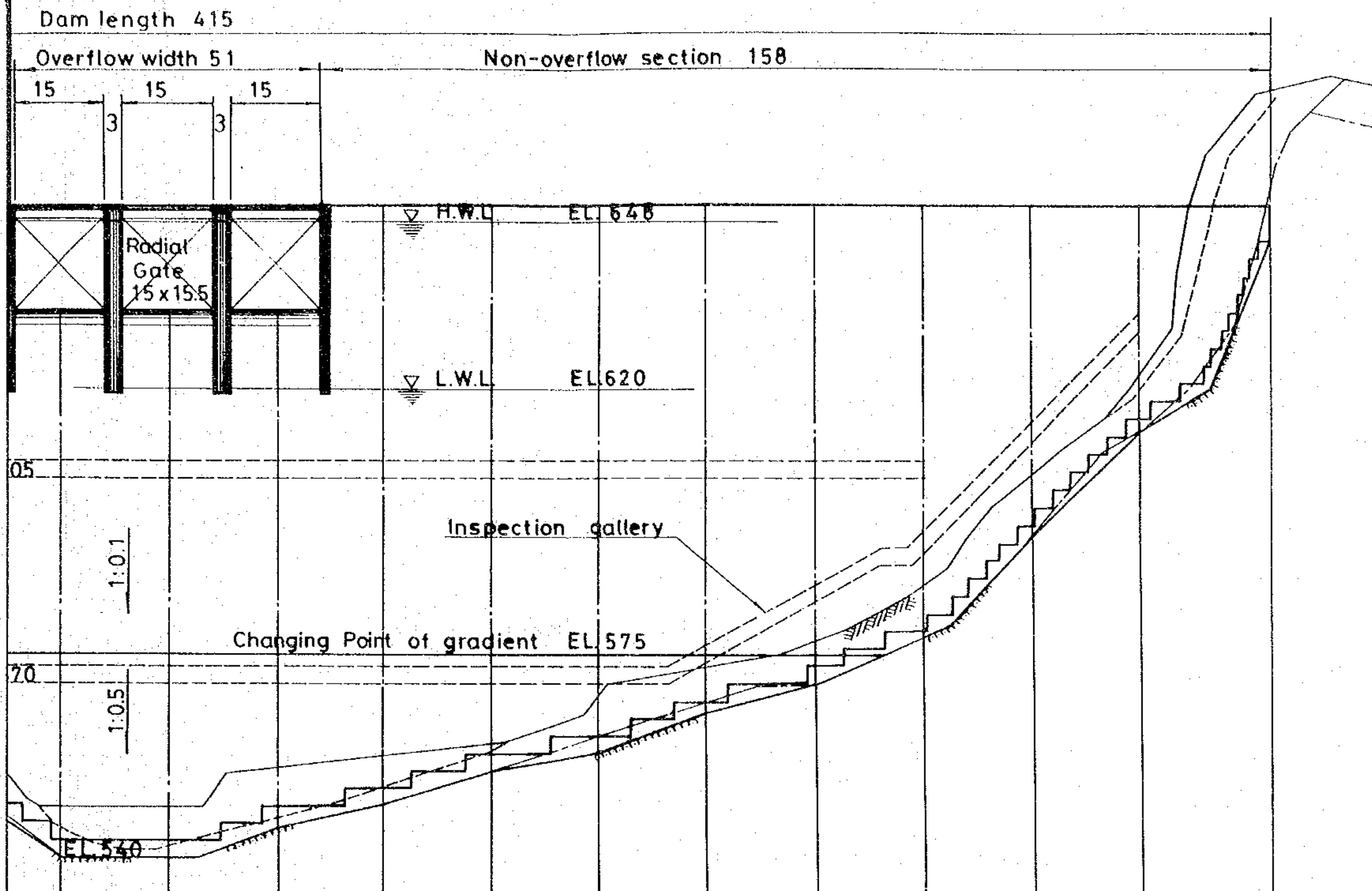
3-113, 3-111

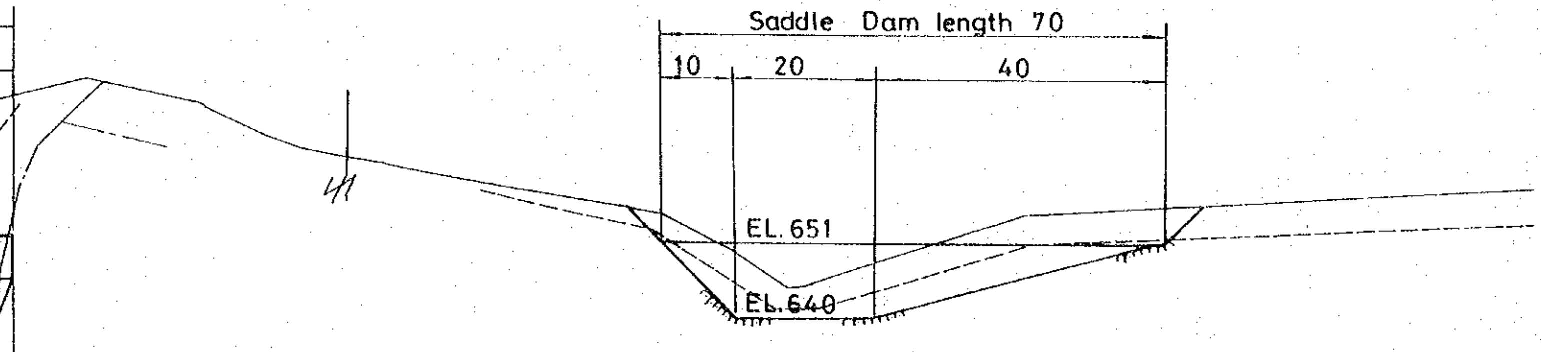
Upstream Elevation of Reservoir Dam and Saddle Dam



FH (m)	665	642	631	622	615	604	599	594	589	578	562	552	540	540	545	549	555	558	565	570	578
GH (m)	661	651	644	632	624	610.5	607	604	597.5	587.5	575.5	563.5	549	549	555.5	557.5	560	568	573	577.5	587
TD (m)	0	15.5	33.5	51.5	69.5	87.5	105.5	123.5	141.5	159.5	177.5	195.5	213.5	231.5	249.5	267.5	285.5	303.5	321.5	339.5	357.5
PD (m)	0	15.5	18	〃	〃	〃	〃	〃	〃	〃	〃	〃	〃	〃	〃	〃	〃	〃	〃	〃	〃
P	0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20

Upstream Elevation of Reservoir Dam and Saddle Dam

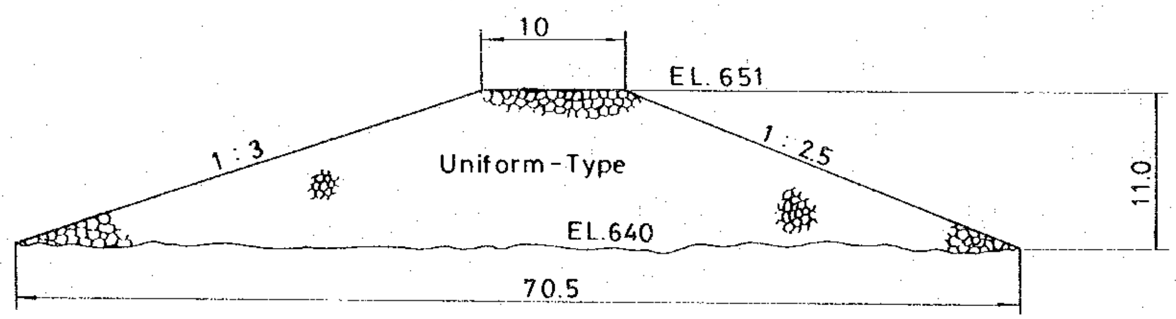




540	540	545	549	555	558	565	570	578	595	613	645
549	549	555.5	557.5	560	568	573	577.5	587	606	623	671
213.5	231.5	249.5	267.5	285.5	303.5	321.5	339.5	357.5	375.5	393.5	415
//	//	//	//	//	//	//	//	//	//	//	21.5
12	13	14	15	16	17	18	19	20	21	22	23

Typical Cross Section of Saddle Dam
(Enlarged)

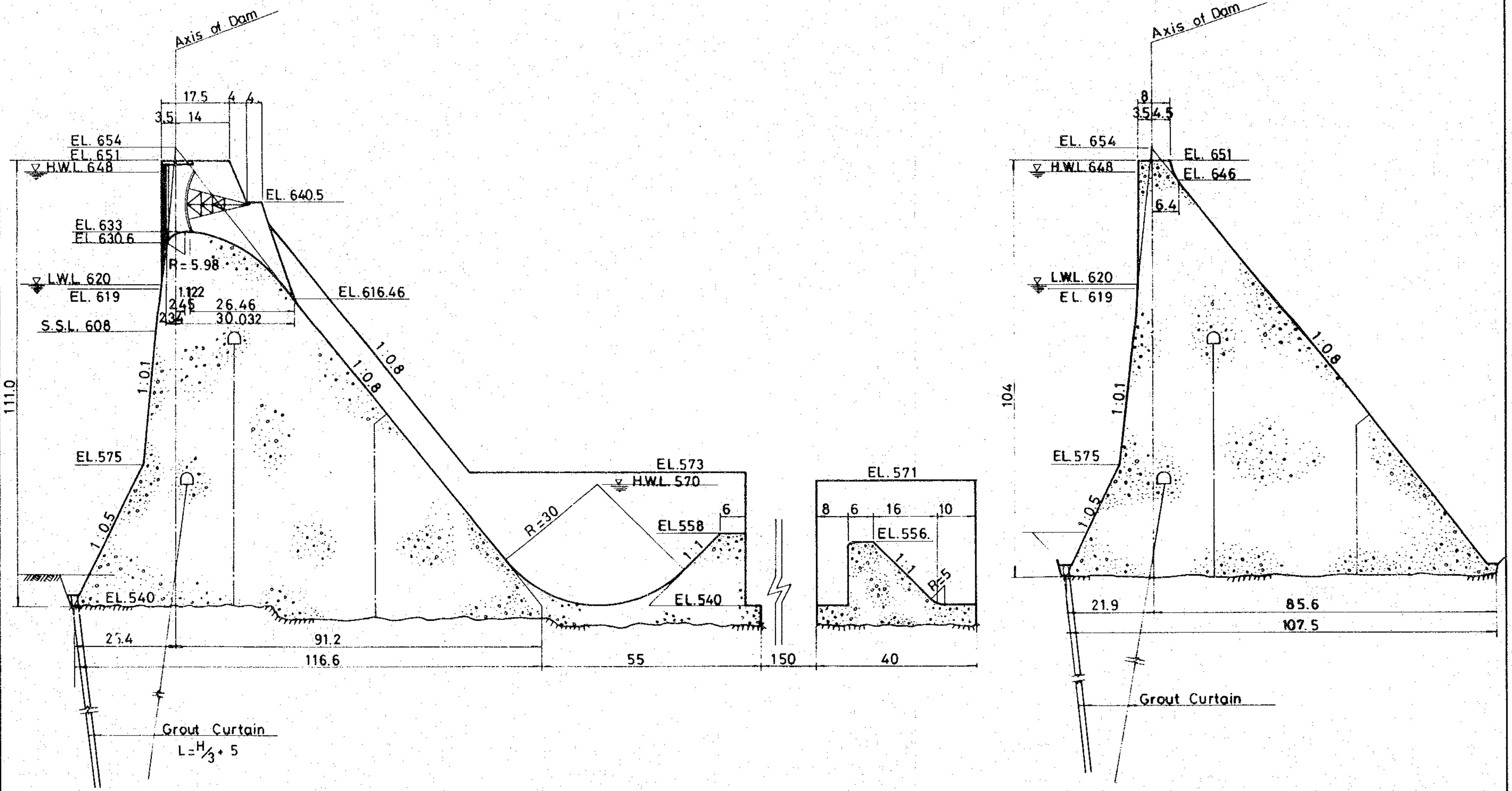
Scale
0 10



Scale
0 50(m)

Diduyon Hydroelectric Project Upper Cagayan River Republic of the Philippines
Japan International Cooperation Agency
Upstream Elevation of Reservoir Dam and Saddle Dam
October 1980 Fig. 3-4-2

Cross-Section of Reservoir Dam



Diduyon Hydroelectric Project
Upper Cagayan River
Republic of the Philippines

Japan International Cooperation Agency

Cross-Section of Reservoir Dam

October 1980 Fig. 3-4-3

3.5. Considerations on Preparatory Works, Construction Road and Materials

The basic transportation plan, the construction road to be constructed anew and the procurement plan for construction materials and equipment will be studied here based upon the results of studies carried out in the project area.

3.5.1. Basic Plan

(1) Port facilities (Manila port)

It will be necessary to secure an area for installation of the following temporary facilities for unloading of machinery and equipment.

- Shed	250 m^2
- Warehouse	650 m^2
- Temporary storage area (Yard)	3,600 m^2
- Total	4,500 m^2

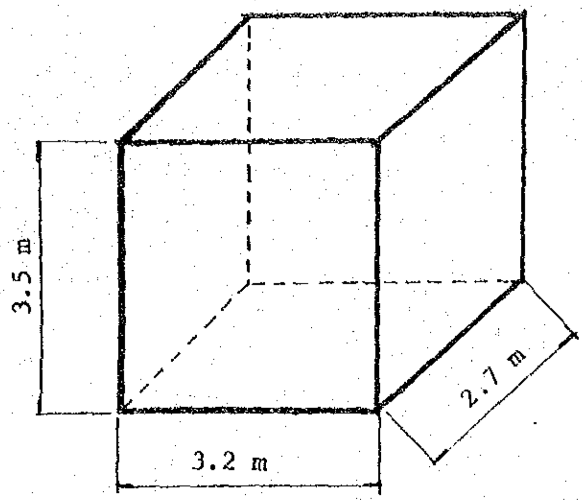
(2) Additional reinforcement to the existing bridges or construction of temporary bridges

The additional reinforcement to the existing bridges and/or the construction of temporary bridges is required at approximately 10 places on highways or local roads between Manila and Bambang.

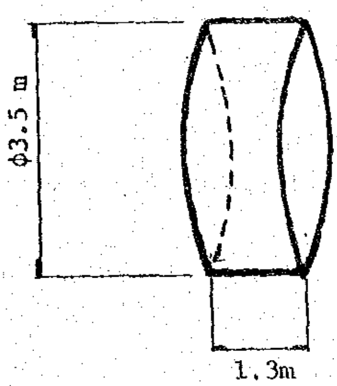
a) Basic conditions

i) Approximate dimensions and weight of the cargo to be transported are as follows:

3-phase transformer —— Weight: 37t



Turbine runner —— Weight: 14t



ii) Transportation vehicles

Trailers of 40t capacity will be used because the maximum weight of a single cargo will be approximately 37t. The total weight of loaded trailer will be 62t as given below.

- Tractor tire 10t
- Trailer tire 12t
- Maximum weight to be transported 40t (with margin)
- Total 62t

Refer to Figure 3-5-1 for details on trailer such as dimensions and weight per axle.

(3) Plan for additional reinforcement to the existing bridges and construction of temporary bridges

i) Reinforcement plan

The reinforcement system to the existing bridges will be decided upon studying aspects such as the span, load withstanding capacity and ease of construction of reinforcement to existing bridges, water level of the river, and so forth. As for the reinforcement method, the overbridge system and the support column system shown in Figure 3-5-2 are considered, and the most adequate method for each bridge will be adopted.

ii) Plan for construction of temporary bridges

If the reinforcement of existing bridges is impossible, the problem can be solved either by constructing a temporary bridge or constructing a temporary road on the riverbed. However, the road will be inundated at the time of high water in case of the latter solution. As shown in Figure 3-5-3, temporary bridges are composed mainly of assembly structure (truss type panels) and H shaped steel, and are assembled at the erection site. Temporary bridges should be constructed at the vicinity of the existing ones, except in case of special restrictions. In this case, the bridge length should be adjusted to the conditions prevailing at the construction site, but the maximum span should be of the order of 25 meters. The design load for construction of temporary bridges should be determined by taking as a reference the 1st class bridges (automobile load: TL-205). In case of running of 50t trailer load, increase in the allowable stress should be taken into consideration, disregarding secondary loads such as group loads, shock loads and so forth.

(4) Access roads and communication roads

A total extension of approximately 105km of access road and communication road should be constructed anew. The basic conditions taking into consideration should be as follows. (Fig. 3-5-4)

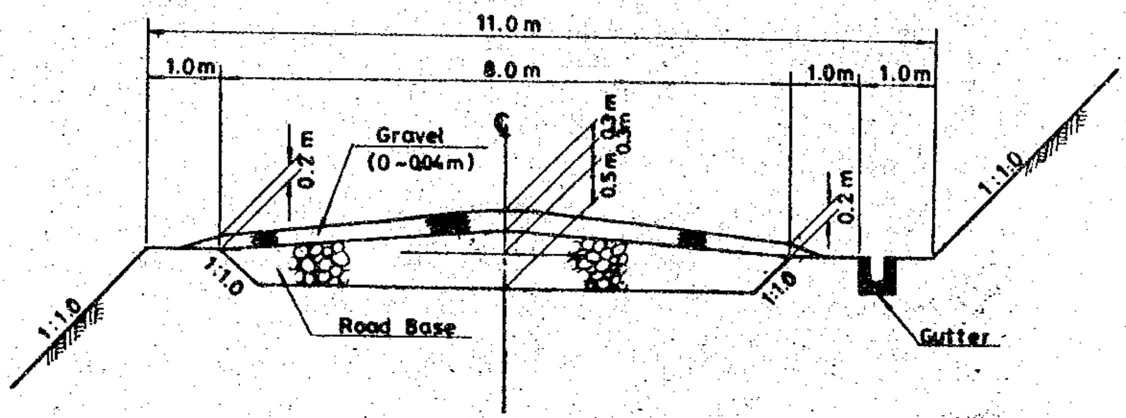
a) Use

These roads will be used for construction of the powerhouse and other facilities, i.e., for transportation of the materials and equipment required for construction, transportation of heavy equipment such as generators, transformers, etc., and after finishing the construction of the power plant, these roads should be used for maintenance and operation purposes.

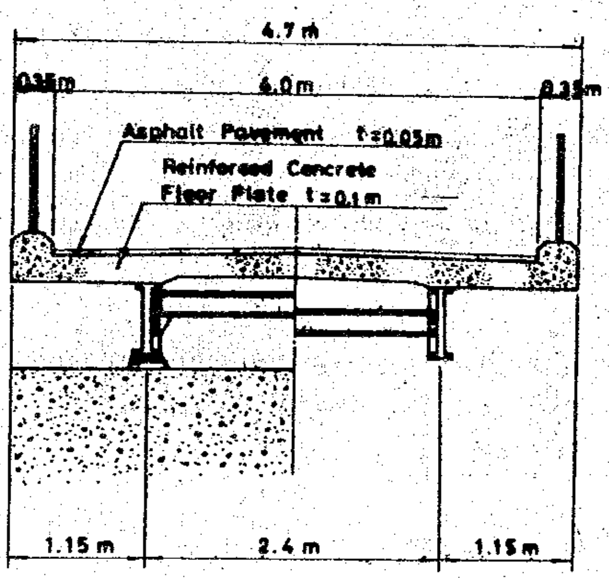
b) Number of traffic lanes

The access and communication roads should have at lease 2 traffic lanes, in view of the maximum quantity of cargo transported per day, the type of trucks used and other related factors.

Standard Section of Road



Standard Section of Bridge



c) Longitudinal gradient

The maximum slope of the roads in mountainous sections should be under 8%.

d) Minimum radius of curvature

The minimum radius of curvature of the roads should be 50m, taking into consideration traffic of 40 ton trailers.

e) Design load

The automobile load of TL205 will be adopted in the planning of bridges to be constructed anew. In case of traffic of 40ton trailers, they should cross the bridges individually and at low speed.

f) Standard section of construction road and bridge

Typical sections of road and bridge are as shown below.

(5) Aggregate transportation roads

Aggregate is the material used in largest quantity in dam construction work. Consequently, in case of transporting aggregate by means of trucks, data related to the correcponding road such as width, slope, curvature, etc., should be carefully studied in order to ensure stable transportation of the required quantity.

i) The longitudinal slope of the aggregate transportation roads should be of the order of 8%, but for general slopes an order on 10% may be adopted.

ii) Radius of curvature

The radius of curvature should generally be of the order of 100 m, and in unavoidable cases it may be of the order of 50m.

iii) Maximum speed

The maximum traffic speed should be of the order of 30km/h.

iv) Width

In general, dump trucks of the 15-45 ton class are used for transportation of aggregate. Table 3-5-1 presents the standard dimensions of typical dump trucks, and the required road width in case of 1 traffic lane and 2 traffic lanes. Roads with 2 traffic lanes and total width of approximately 13-15.5m are required in order to make possible the traffic of dump trucks with total weights of the order to 20-32 ton.

(6) Road safety facilities

The construction roads should be perfectly equipped with safety facilities. The safety facilities mentioned below should be taken into due consideration at an early stage of planning.

i) Installation of traffic signs of various types and illumination facilities at the crossings and branching points.

ii) Construction of guard fences, guardrails or delineators in order to prevent the fall of vehicles at the valley side shoulders of the construction roads.

iii) Securing of good visibility at curves, or installation of curve mirrors.

iv) Installation of rock nets, spraying of mortar, construction of protection fences against falling rocks on mountain side slopes of the construction road shoulder.

(7) Maintenance and management of the roads

Maintenance and administration to keep the road permanently in satisfactory condition is very important in order to ensure transportation efficiency, and the following items should be carefully studied.

i) Maintenance of the roadbed in satisfactory conditions using river deposit pebbles and/or crushed gravels supplied from quarry.

ii) Periodic repair of the roads using motorgraders.

iii) Spraying of water in order to prevent dust clouds.

iv) Perfect draining of water with lateral culverts and other means at places where the road crosses streams, creeks, etc.

v) Construction of longitudinal gutters, troughs, entrance well and so forth along the mountain side.

vi) Rock fall during transportation of aggregate and from steep slopes at the excavation site causes not only wear and tear of the tires of dump trucks, but also damage to the springs. In addition, they hinder the traffic of dump trucks. Thus, it is necessary to have workers and tire dozers for removal of falling rock.

(8) Plan for procurement of materials and equipment for construction.

Most of the hardware, structural steel and other materials and equipment required for construction should be imported from overseas, with the exception of the lumber, aggregate, cement, concrete reinforcement bars, dynamite and other minor materials.

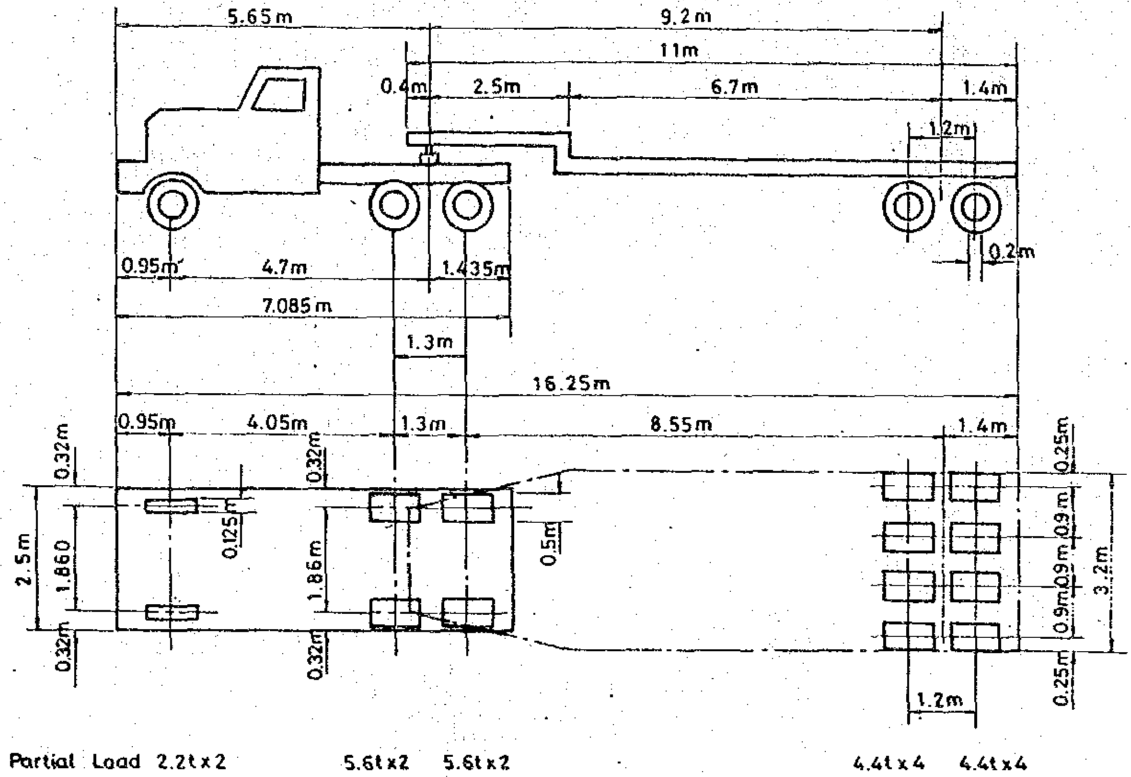
As for the construction machinery, everything which can not be supplied in the Philippines should also be procured from abroad.

Table 3-5-1 Specifications of Dump Trucks and Minimum Width of Road

Model	Max Loading Capacity (t)	Dimensions of Trucks (mm)			Structure and Width of Road (m)				
		Length	Width	Height	Lane	Central Safety Zone	Shoulder	One lane	Two lanes
Hino ZG 1500	15	6,595	3,000	3,275	4.00	1.50	1.00	6.00	11.50
Komatsu HD180	18	7,295	3,000	3,300	4.00	2.00	1.00	6.00	12.00
" HD200	20	7,450	3,360	3,450	4.35	2.00	1.00	6.35	12.70
" HD320	32	7,800	3,670	3,950	4.65	3.00	1.50	7.65	15.30
" HD460	46	8,900	4,050	4,260	5.05	3.50	2.00	9.05	17.60

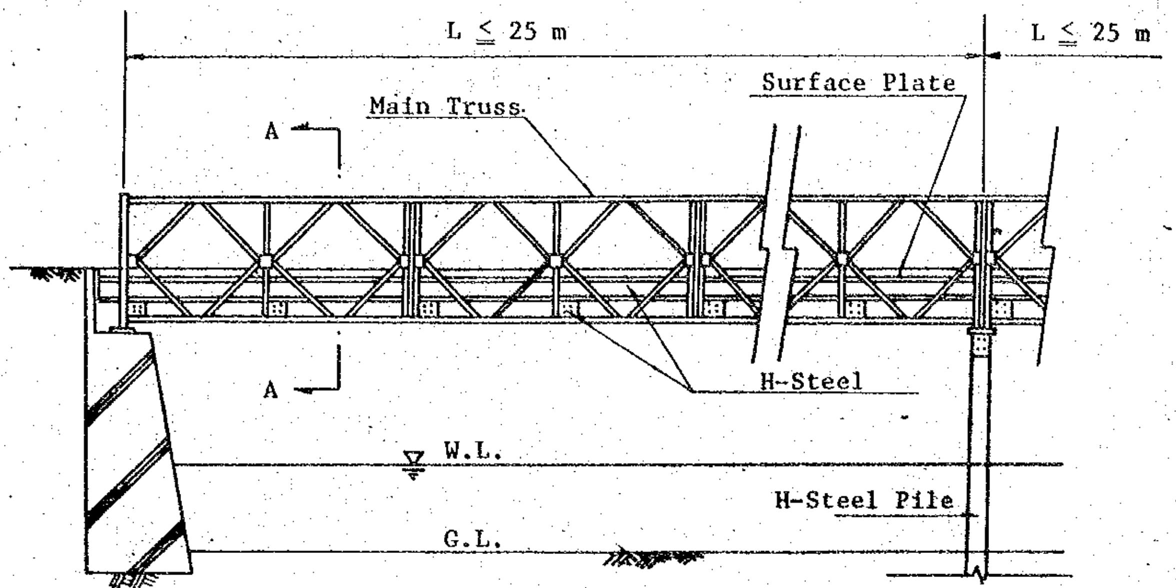
Note: The space for side-gutters, guard rails and so on shall be considered when needed.

Dimension of 40-t Trailer

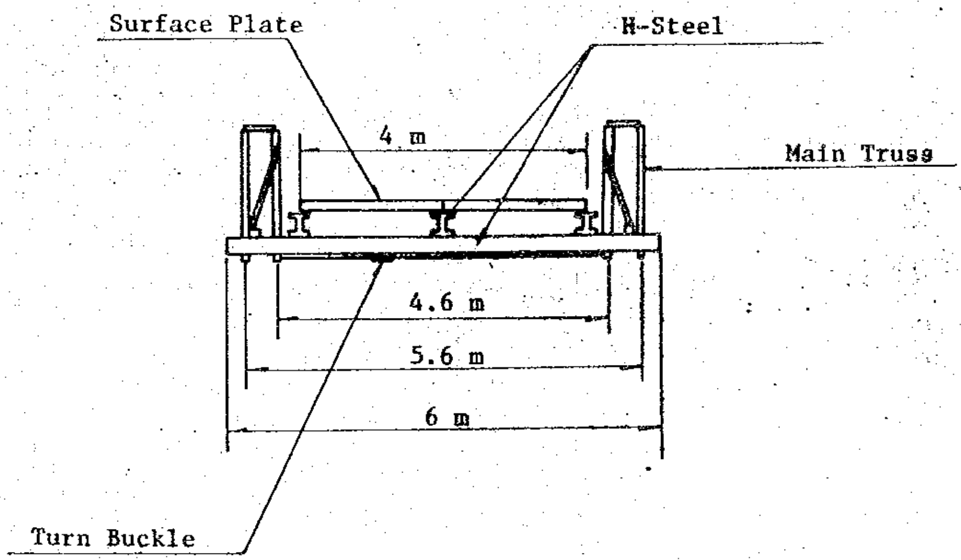


Diduyon Hydroelectric Project Upper Cagayan River Republic of the Philippines
Japan International Cooperation Agency
Dimension of 40-t Trailer
October 1980 Fig. 3-5-1

General Structure of Temporary Bridge



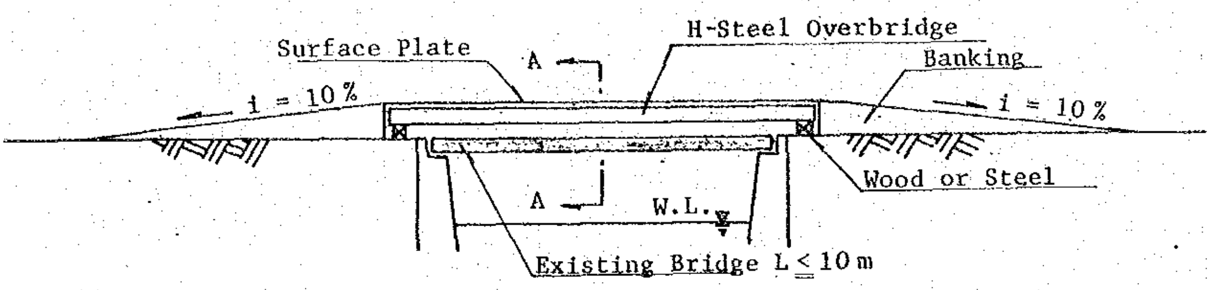
A - A Section



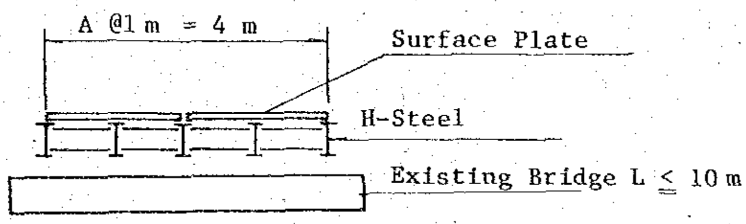
Diduyon Hydroelectric Project Upper Cagayan River Republic of the Philippines	
Japan International Cooperation Agency	
General Structure of Temporary Bridge	
October 1980	Fig. 3-5-2

Reinforcement of Existing Bridges

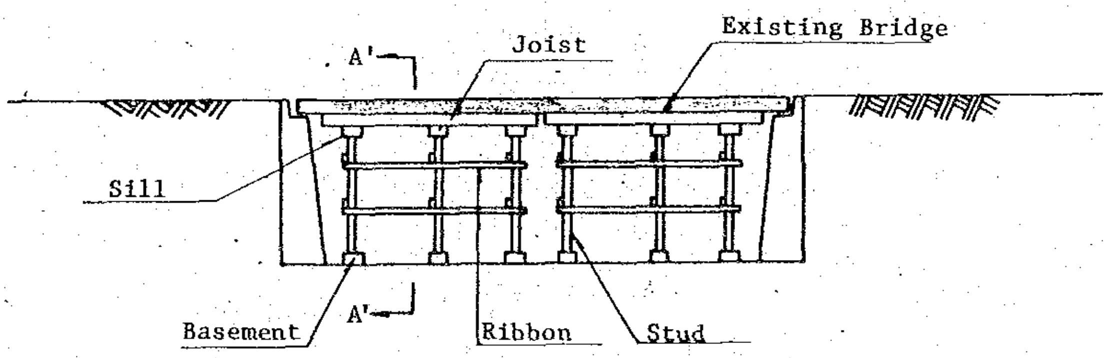
(1) Overbridge



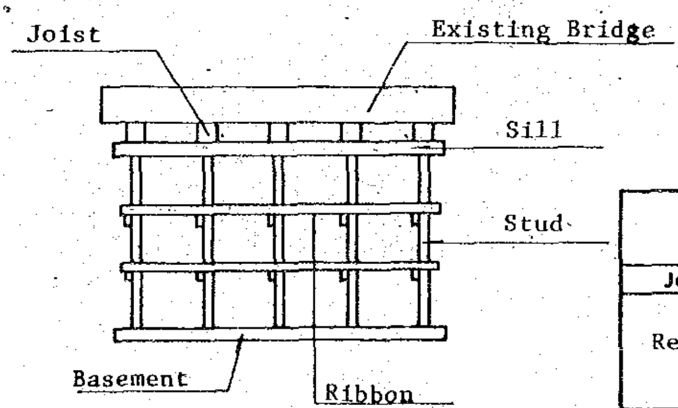
A ∿ A Section



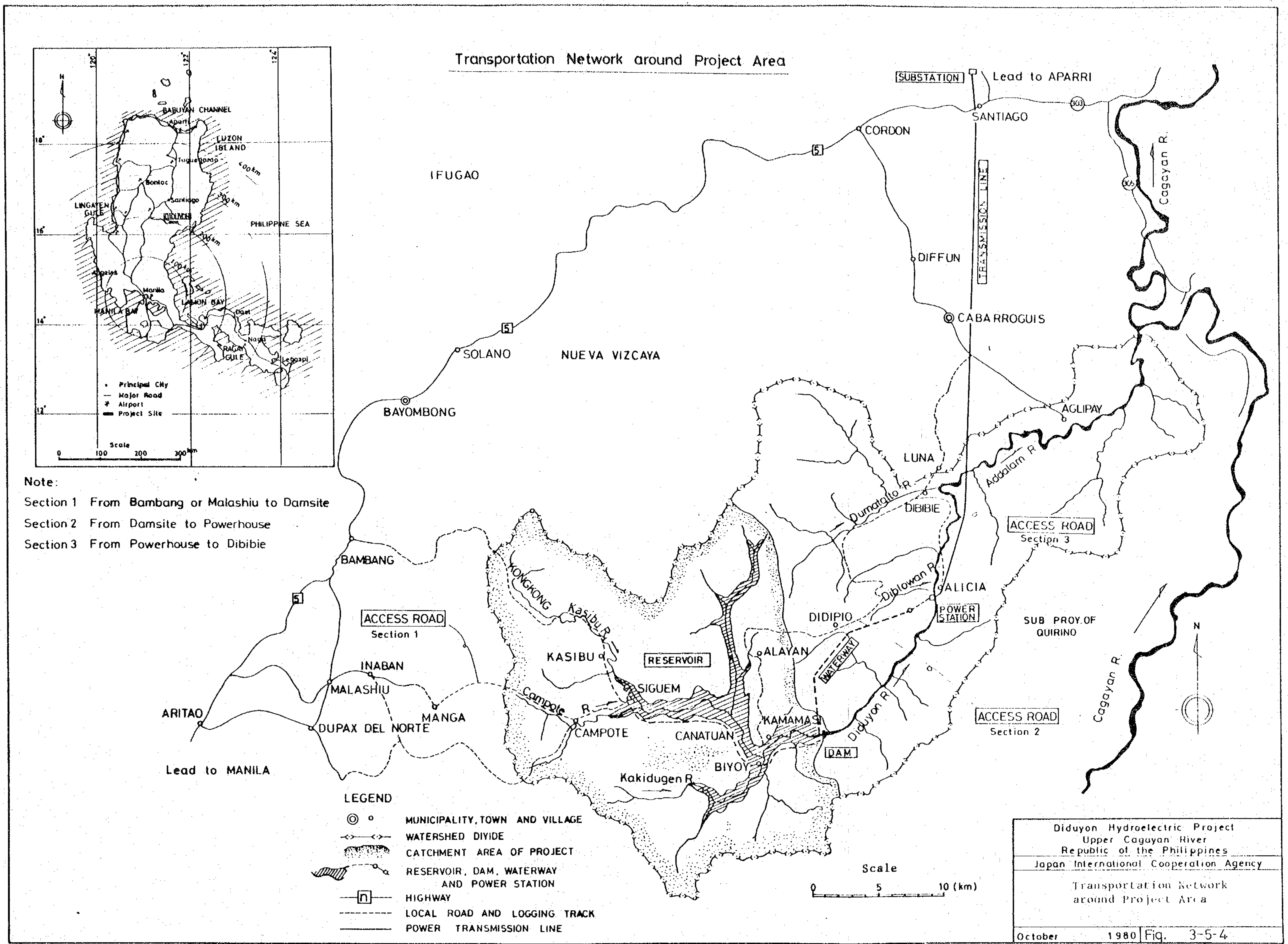
(2) Steel or Wood Stud Type



A' ∿ A' Section



Diduyon Hydroelectric Project Upper Cagayan River Republic of the Philippines	
Japan International Cooperation Agency	
Reinforcement of Existing Bridges	
October 1980	Fig. 3-5-3


Transportation Network around Project Area
SUBSTATION
Lead to APARRI
SANTIAGO
CORDON
5
IFUGAO
TRANSMISSION LINE
Cagayan R.
DIFFUN
CABARROGUIS
SOLANO
NUEVA VIZCAYA
BAYOMBONG
AGLIPAY
LUNA
Addalam R.
Dumatalto R.
DIBIBIE
ACCESS ROAD
Section 3
BAMBANG
KONGKONG
Kasibu R.
ALICIA
Diblowan R.
POWER STATION
DIDIPIO
SUB PROY. OF QUIRINO
ACCESS ROAD
Section 1
KASIBU
RESERVOIR
ALAYAN
WATERWAY
INABAN
MALASHIU
SIGUEM
Campote R.
MANGA
ARITAO
DUPAX DEL NORTE
CAMPOTE
CANATUAN
KAMAMAS
Diduyon R.
DAM
ACCESS ROAD
Section 2
Cagayan R.
N
Lead to MANILA
BIYOY
Kakidugen R.
BABUYAN CHANNEL
Aparri
LUZON ISLAND
Tuguegarao
Bontoc
Santiago
LINGAYEN GULF
PHILIPPINE SEA
Angeles
Manila
MANILA BAY
LAMON BAY
Daet
Naga
RAGAY GULF
Legaspi
400 km
300 km
200 km
100 km
18°
16°
14°
12°
120°
122°
124°
Principal City
Major Road
Airport
Project Site
Scale
0 100 200 300 km
Note:
Section 1 From Bambang or Malashiu to Damsite
Section 2 From Damsite to Powerhouse
Section 3 From Powerhouse to Dibibie
LEGEND
MUNICIPALITY, TOWN AND VILLAGE
WATERSHED DIVIDE
CATCHMENT AREA OF PROJECT
RESERVOIR, DAM, WATERWAY AND POWER STATION
n HIGHWAY
LOCAL ROAD AND LOGGING TRACK
POWER TRANSMISSION LINE
Scale
0 5 10 (km)
Diduyon Hydroelectric Project
Upper Cagayan River
Republic of the Philippines
Japan International Cooperation Agency
Transportation Network around Project Area
October 1980
Fig. 3-5-4

3.6. Environmental Considerations

3.6.1. Influence upon the environment and assessment

The influence of this project on the environment can be broadly classified into direct and indirect influences.

i) Direct influence - Influences which occur necessarily as a consequence of the physical changes taking place in the environment, due to the construction work of the project and to the appearance of newly built structures such as dam, powerhouse, transmission line and so on.

ii) Indirect influences - effects which occur as a chain reaction triggered by the direct influence mentioned above, or secondary effects resulting from the operation of the storage reservoir dam, and the power plant.

A wide variety of factors contribute to the composition of the environment and these factors have intimate mutual relationship. Depending upon the element which actuates as a cause, the influence of the development project has different extents of influence, both in space and in time. Bearing in mind the considerations above, the environmental elements assumed to be influenced by this project have been picked out and listed in Table 3-6-1. Next, in Table 3-6-2 is summarized the relationship between the most important items of the various environmental elements and the impact of the project.

3.6.2. Influence Upon the Physical Environment

(1) Noise and vibration

Noise and vibration can be classified into that caused by the machinery used for construction of the dam and that caused by the heavy vehicles used for transportation of materials and equipment. Noise and vibration caused by the construction machinery are generated at the dam construction site and at other job sites. Since the dam construction site is located far from any village or hamlet, this will not have any significant influence upon human beings, but it is assumed that it will have a temporary influence upon wild animals living in the area. However, it is assumed that these wild animals will move to places outside the immediate area where living and food collecting activities will not be influenced.

The transportation occur along the construction roads. The actually existing roads are located close to villages and hamlets at very few places, and in addition they are more than 50m from residences in most cases. Thus, it is assumed that they will have negligible influence on these villages and hamlets. As for the transportation roads to be constructed anew, routes have been planned under similar conditions.

(2) Quality of water

Changes in the quality of the river water and its turbidity will occur during the construction work and after the appearance of the storage reservoir. During the construction work, the excavation of river deposits, excavation of the riverbed, construction of roads, washing of aggregates, etc., the resulting gravel, mud, rejected cement milk, etc., will cause contamination and turbidity of water and will sediment to the riverbed gravel. These sedimented materials will have a long lasting influence upon the fauna growing in the river, with the possibility of influence also upon river fish.

In addition, the use of a large number of workers in construction of the dam will cause the generation of large quantities of living waste water (in some cases beyond the self-cleaning capacity of the river) resulting in nutritional enrichment of the river water.

As a result of the appearance of the storage reservoir, there will be decomposition of the flora of the ponding area lasting for a long time. As a result, there will be nutritional enrichment of the water, in addition to the risk of change in the water quality due to mixing of fertilizers and agricultural chemicals resulting from the farming in the areas located upstream of the storage reservoir, besides the mixing of living waste water. However, the occurrence of such problems can be prevented by adopting restrictions and adequate countermeasures during the construction and after completion.

(3) Water temperature

Changes will occur in the water temperature in the storage reservoir, as a consequence of the difference of form of the storage reservoir and the quantity of the water flowing into and from it.

The water temperature at the bottom of the storage reservoir will be lower than the temperature prevailing at its surface and the temperature of river water in general.

The ratio α between the quantity of water flowing into the storage reservoir and the quantity of water stored in the reservoir can be used as a criterion for judging if water temperature layers are formed or not. The ratio α is represented in the following form.

$$\alpha = \frac{\text{Quantity of water flowing annually into the reservoir}}{\text{Total quantity of stored water}}$$

In general, it is said that when the ratio is smaller than 10 the temperature layers will form and when α is larger than 20 the layers are mixed. Since in the Diduyon storage reservoir we have:

$$\alpha = \frac{980 \times 10^6 m^3}{580 \times 10^6 m^3} = 1.7$$

it is assumed that there will be an inevitable formation of water layers of different temperature. As for the temperature difference, it cannot be clearly foreseen, because it is influenced by factors such as the configuration of the storage reservoir, intensity of wind, etc.

Generally speaking, in case of man-made lakes with formation of water layers of different temperature as described above, if the power generation water taken from low water levels is utilized for agricultural irrigation purposes, the crops neighbouring the water outlet may suffer the influence of low water temperature if the irrigated field is located a short distance from the reservoir.

In the Diduyon project the water discharged from the power plant will flow a fairly long distance through open waterways (river) before reaching rice paddies located downstream, being warmed by passive solar energy meanwhile. In addition, since the river has many tributaries joining it, and there are no agricultural water intake facilities located along the river downstream of the dam, it is assumed that this kind of question due to the presence of low temperature layers formed in the reservoir will not occur as an urgent problem.

(4) Changes in the quantity of water and water level

Changes in the quantity of water and water level will have different characteristics at points located upstream of the dam, between the dam and the power plant, and at points located downstream of the power plant.

At places located upstream of the dam, the water level will be raised from the actually prevailing water level to the planned high water level of the storage reservoir. Consequently, fields and houses located in the reservoir area will be inundated, and a large lake, 16km in length, 5km maximum width and $27km^2$ in area, will appear. The water level of this storage reservoir will present variations according to the operation of the power plant and the quantity of water fed from the river, but the fluctuation range will be limited within a maximum value of 28m. In order to make possible efficient operation of the power plant, the storage reservoir will generally be kept at a water level several meters below the high water level. It may be drawn down to the low water level (28m below the high water level) only in drought years (approximately once or twice every 10 years). Since the water discharge to be used for generating electricity is relatively small compared with the effective volume of the storage reservoir, the daily, weekly and monthly water level fluctuation will be small in normal years and rainy seasons as well.

In the section between the dam and the power plant the flow of water will be drastically reduced, because the whole quantity of water fed by the river located upstream of the dam will be stored in the storage reservoir, and in the section between the dam and the outlet of the power plant the only water sources will be the small tributaries. The water level fluctuation in this section will be small compared with that occurring before, because flood will be controlled by the storage reservoir.

At places located downstream of the power plant the water level will be high during peak operation periods because the Diduyon power plant will carry out peak operation, and the water level will be low out of the peak operation period (operation stop period), because only the water of the remnent basin will flow. The difference in water level in and out of the peak operation period will be approximately 0.5 m in the neighbourhood of the outlet. This water level difference will be gradually reduced towards the downstream side, due to the confluence of the river water coming from the remnant basin such as the Dumatalto River and others and to dispersion of water flow along the riverbed.

(5) Landslides, erosion of mountain side and topographical changes

As a result of the appearance of the storage reservoir, the actually prevailing water level of the river will be raised up to the high water level of the reservoir, the shore of the lake will become very long, and in addition the reservoir water level will present fluctuations throughout the year. Consequently, the storage reservoir water level will present an annual fluctuation ranging from the high water level (including the influence of waves caused by wind) to the low water level. The available depth of 28 m plus the rise of water level due to wind may cause land-slides and errosion of mountain side depending upon the geology and inclination of the slope and seismical conditions.

In addition, the neighbourhood of the outlet may suffer similar erosions.

Since the surge tank and the penstock are constructed on a slope, they will be accompanied by some changes in the topography. Here the excavated soil and rock will be stored in spoil banks built on slopes, which may result in a risk of the occurrence of failure if the spoil banks are not adequately constructed. In addition, between the damsite and the powerhouse, long roads will be constructed in mountainous areas. The resulting cuttings, embankment and spoil banks are subject to slope erosion and landslides if prevention measures are not adequate.

As a result of these landslides and slope errosion, the quantity of gravel accumulating in the riverbed will increase, causing modification of the riverbed configuration. There might be a possibility of mud contamination influencing the flora and fauna.

The following countermeasures will be required in order to cope with the problems mentioned above.

i) It is necessary to study geologically and morphologically the periphery of the storage reservoir, especially the part comprehended between the high water level and the low water level, inspecting potential sites of landslide and slope errosion. Protective measures should be taken at places where there is risk of such trouble.

ii) Adequate design and investigation should be carried out during the main construction work, preparation work and road construction work, in order to eliminate the risk of the trouble mentioned above.