10.3 Lower Kihansi Project

10.3.1 Dam and Appurtenant Structures

(1) Selection of Location

The projected dam site is located approximately 900 m upstream in straight-line distance from Kihansi Waterfalls. The topography and geology of the vicinity of the dam site are as described in Chapter 7, with the left-bank side in the form of a ridge of inclination approximately 25 deg and the right-bank side a mountainside of inclination approximately 35 deg. The river-bed portion is at an elevation of 1,115 m, with water surface width 20 to 30 m, and there are hard outcrops seen on the right bank. According to the results of drilling investigations carried out with one drillhole each at both banks, it is found that bedrock having adequate bearing force as a dam foundation exists at a depth of about 5 to 10 m from the ground surface at both banks.

As another dam location conceivable, there is the point approximately 300 m upstream from the current dam site. It is necessary for the high water level to be made around 1,150 m at this site to secure the regulating capacity of 500 x 10^3 m^3 required for daily regulating operations, but the right-bank abutment is a skinny ridge, where it is possible weathering has reached deep inside, and it is thought to be unsuitable as a dam site from the standpoints of topography and geology.

On the other hand, downstream of the dam site selected here, it is judged there is no suitable location since the river gradient is steep and it is disadvantageous from the standpoints of economics and constructability, and since effective utilization of head in planning of power generation cannot be aimed for.

(2) Selection of Dam Type

Either a concrete gravity type or a fill type is conceivable for the dam. As a result of a comparison made of the two, as shown in Table 10-6, a concrete gravity type is far more advantageous economically, and moreover in execution of work there are many advantages such as that river diversion during construction can be done with only bypassing through the dam body, and so a concrete gravity type is adopted. The concrete gravity dam and the alternative impervious center core fill-type dam are shown in Figs. 10-24 through 10-27.

(3) Shape of Dam

The dam is composed of a non-overflow section and an overflow section serving concurrently as a spillway.

The structural and hydraulic analyses for deciding on the specifications of the dam are indicated below.

i) Determination of Dam Crest Elevation

The calculation equations for a concrete gravity dam are as follows:

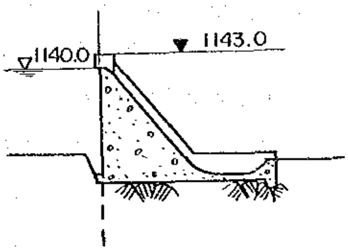
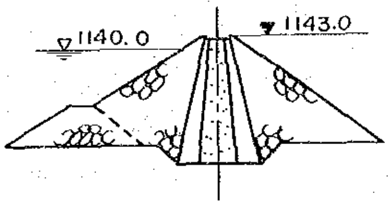
$$H_1 = H_n + h_w + h_e \quad (H_n + 2.00 \text{ when } h_w + h_e < 2)$$
$$H_2 = H_d + h_w \quad (H_d + 1.00 \text{ when } h_w < 1)$$

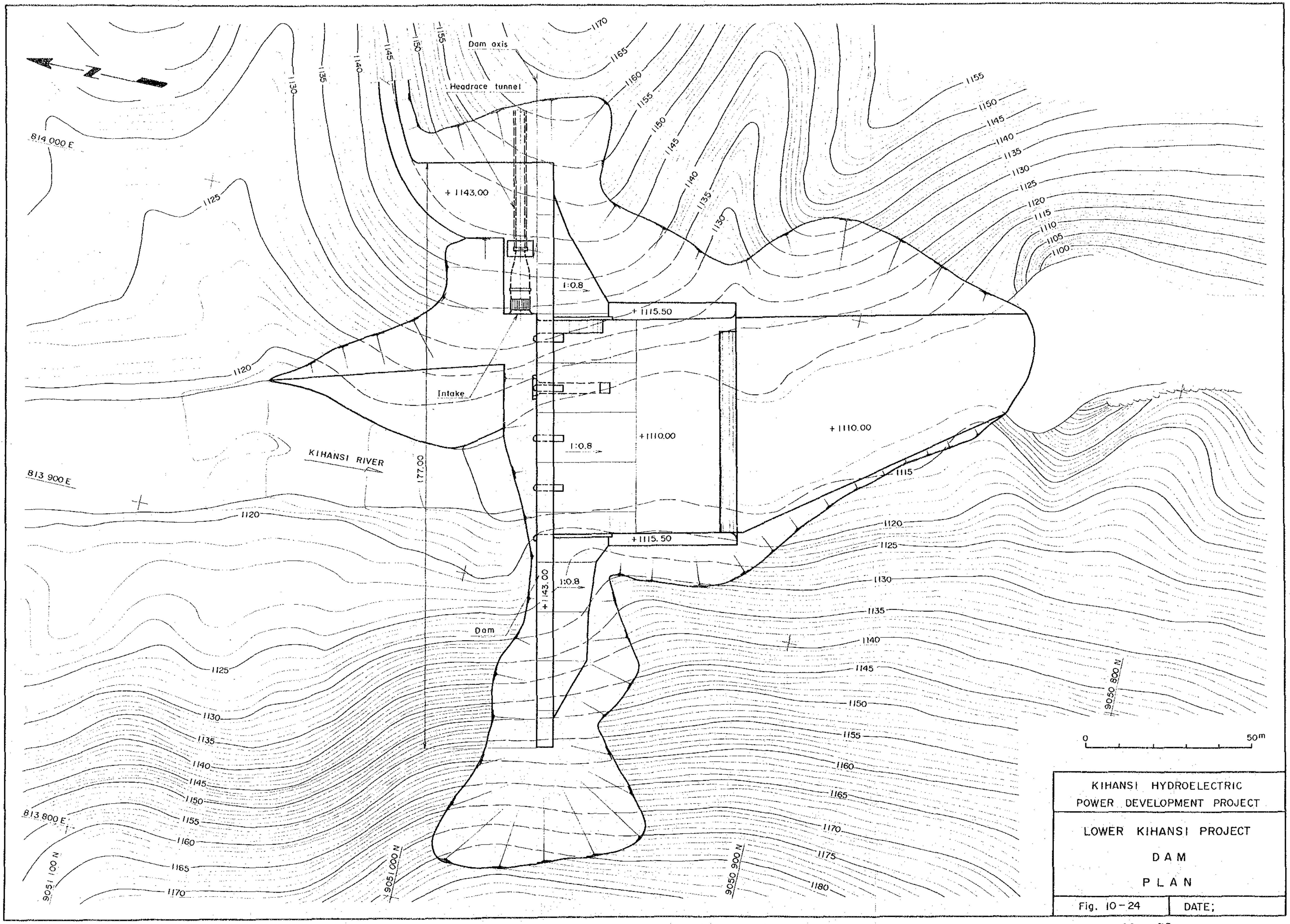
The larger of H_1 and H_2 is to be taken,

where,

H_1 : Crest elevation of non-overflow section determined from normal high water level
H_n : Normal high water level (= 1,140.00 m)
H_w : Water height due to wind
H_e : Water height due to earthquake

Table 10-6 Comparison of Dam Type

Item		Concrete Gravity Type	Rockfill Type
Typical Section			
Slope	Upstream	Vertical	1 : 2.0
	Downstream	1 : 0.8	1 : 1.8
Dam Volume (m^3)		54,000	160,000
Construction Cost		10.2×10^6 US$	16.0×10^6 US$



KIHANSI HYDROELECTRIC POWER DEVELOPMENT PROJECT

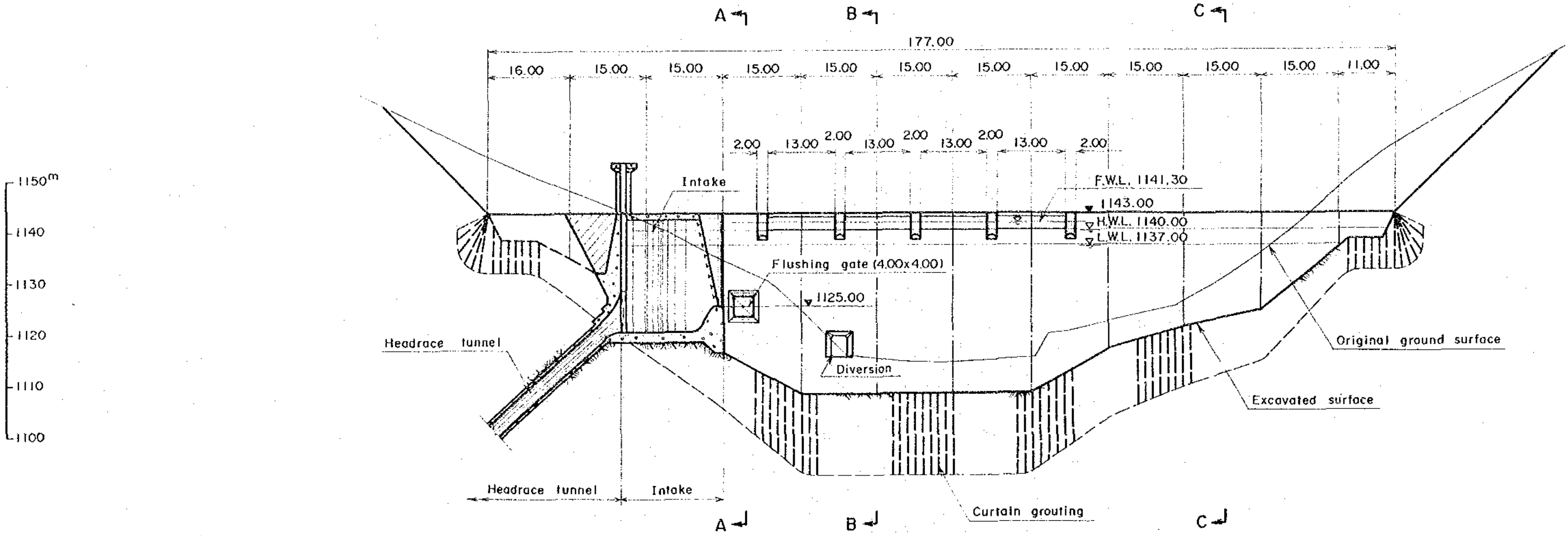
LOWER KIHANSI PROJECT

DAM

PLAN

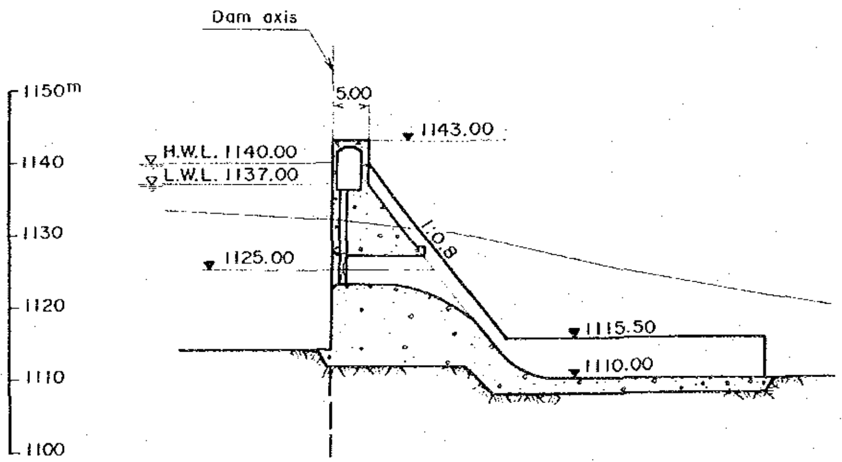
Fig. 10-24	DATE;

PROFILE OF DAM

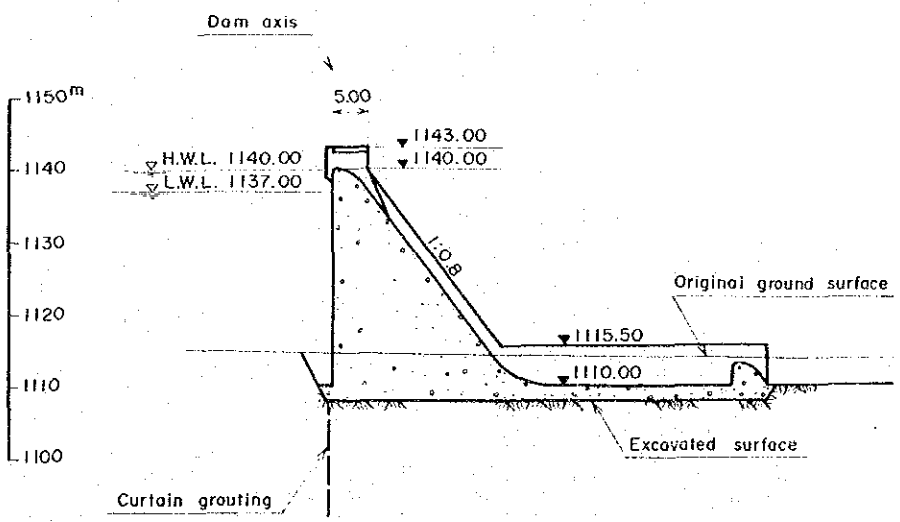


TYPICAL SECTION OF DAM

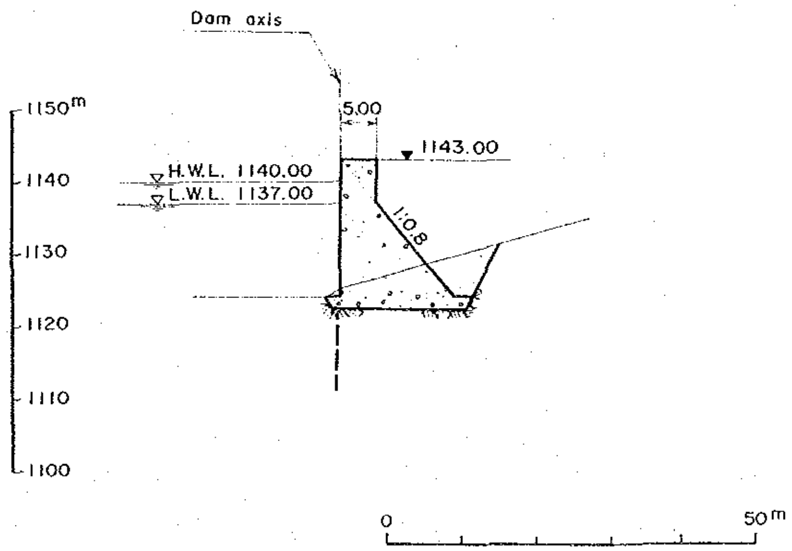
SECTION A – A



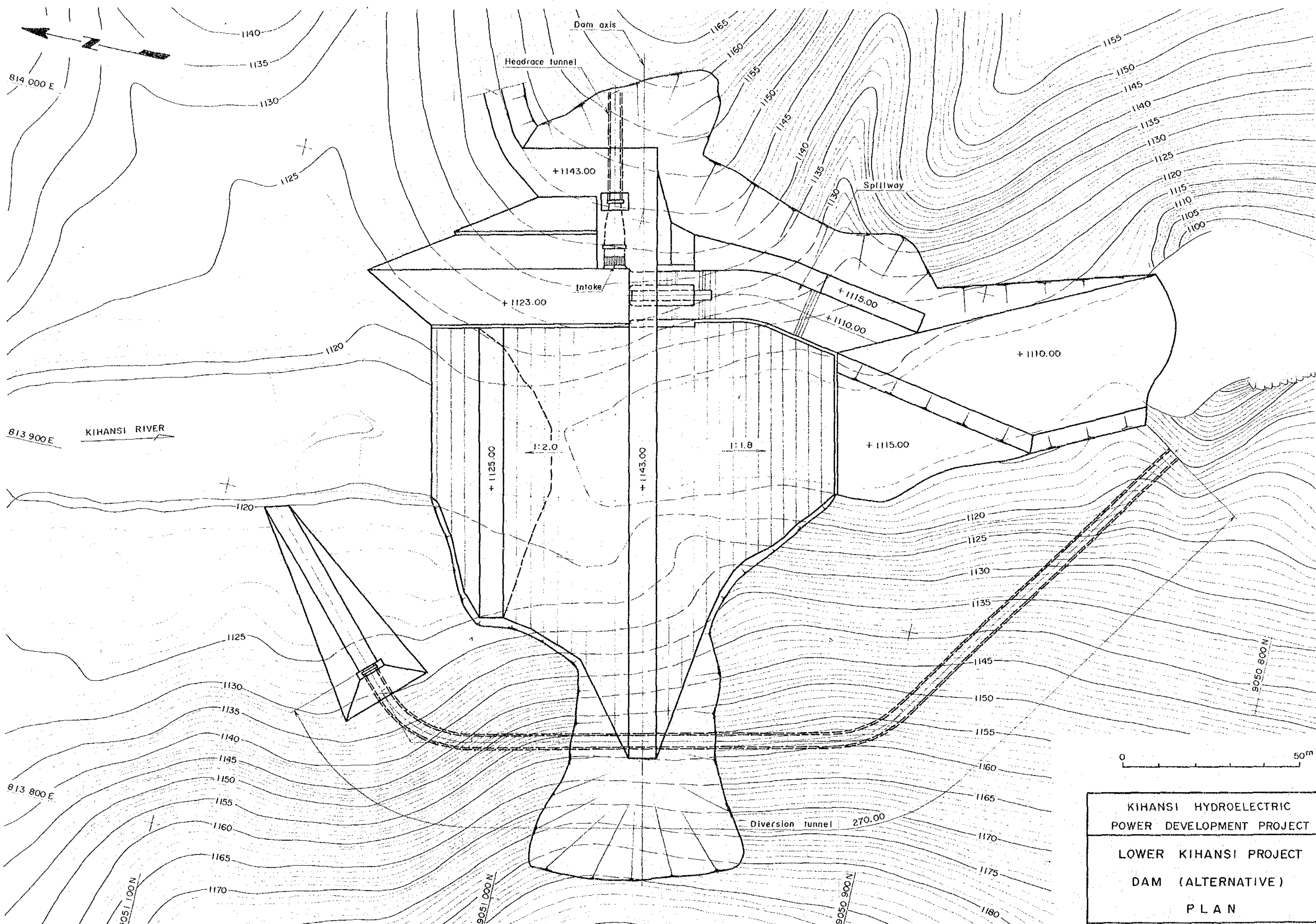
SECTION B – B



SECTION C – C

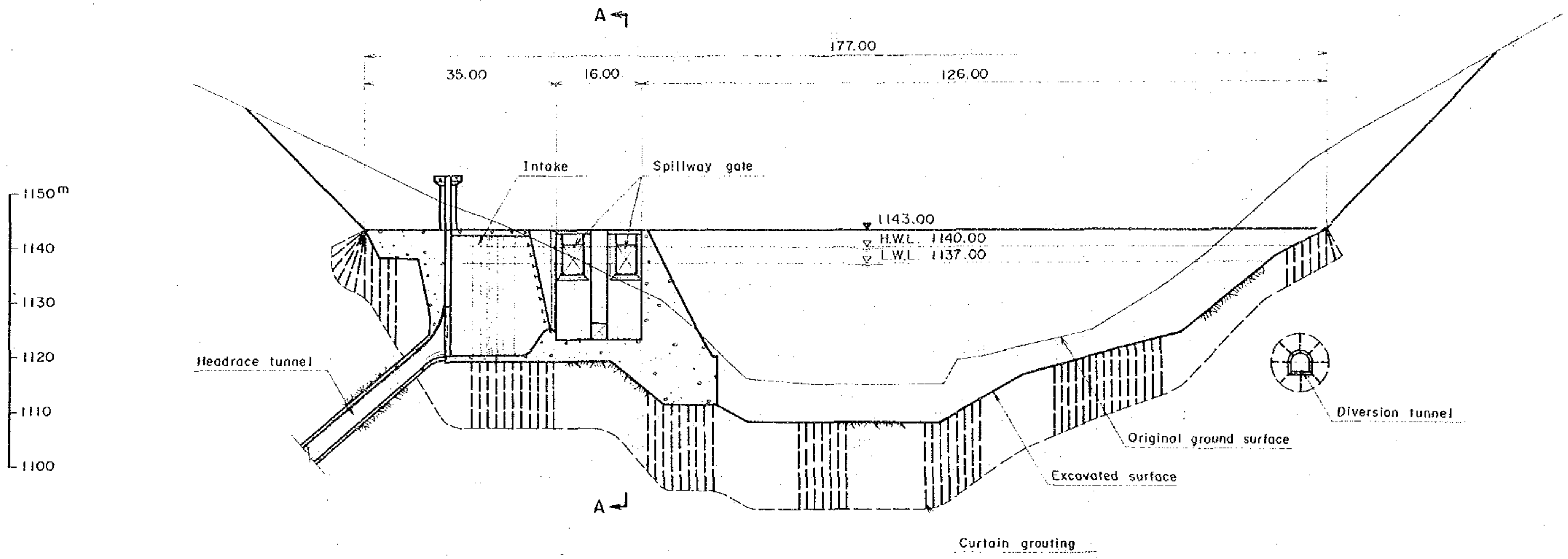


KIHANSI HYDROELECTRIC POWER DEVELOPMENT PROJECT	
LOWER KIHANSI PROJECT DAM PROFILE AND TYPICAL SECTIONS	
Fig. 10 – 25	DATE:

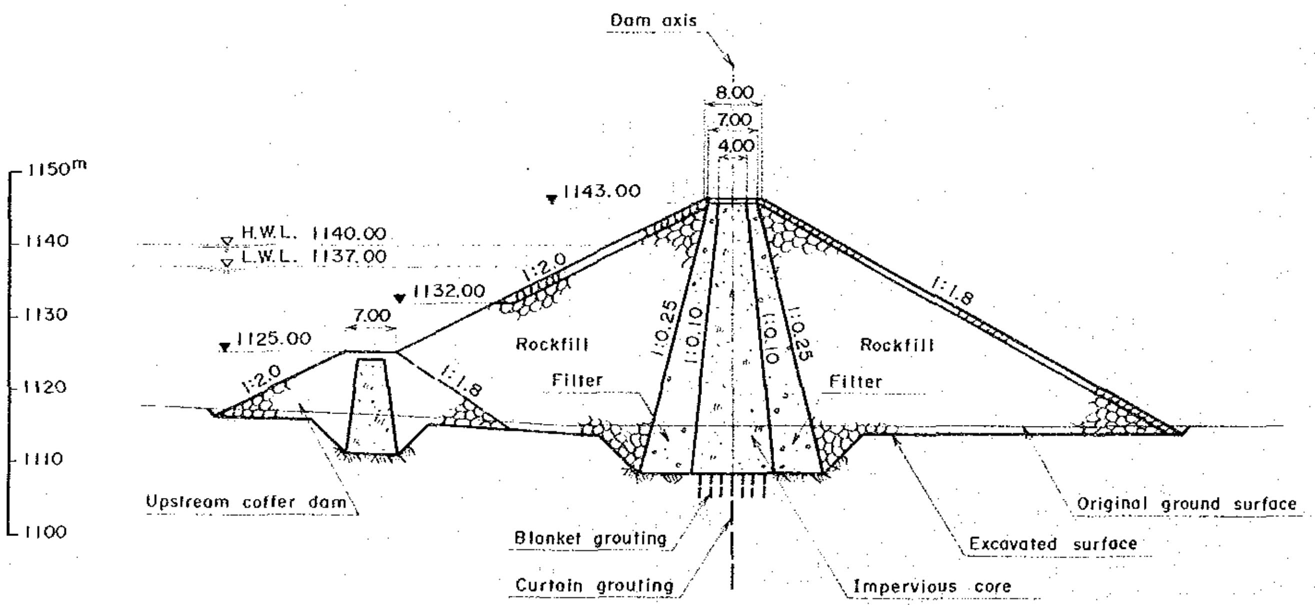


KIHANSI HYDROELECTRIC POWER DEVELOPMENT PROJECT	
LOWER KIHANSI PROJECT DAM (ALTERNATIVE) PLAN	
Fig. 10-26	DATE;

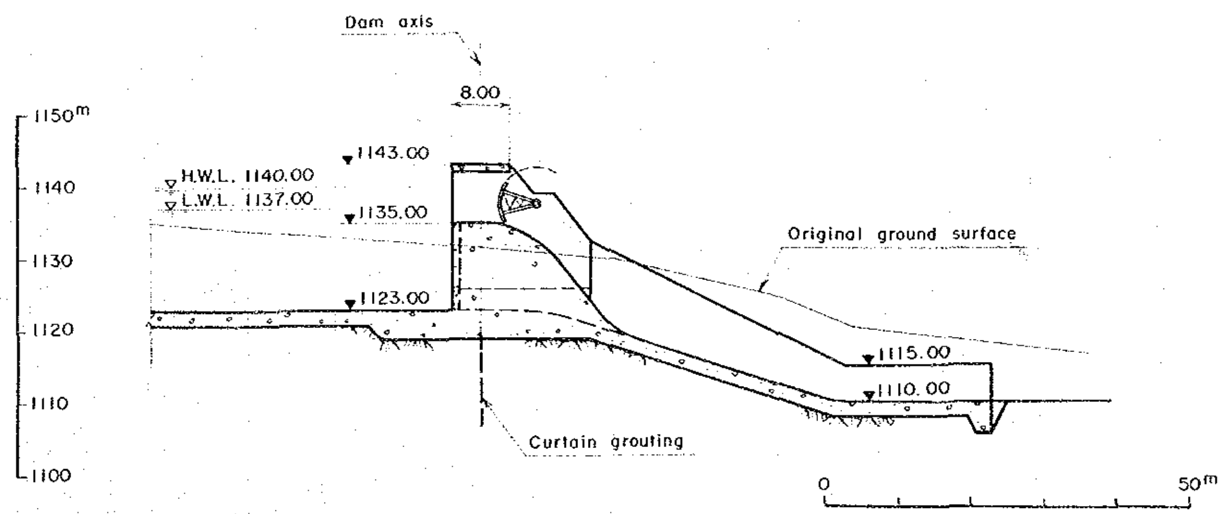
PROFILE OF DAM



TYPICAL CROSS SECTION OF DAM



SECTION A－A (SPILLWAY)



KIHANSI HYDROELECTRIC POWER DEVELOPMENT PROJECT
LOWER KIHANSI PROJECT DAM (ALTERNATIVE) PROFILE AND TYPICAL SECTIONS
Fig. 10－27 | DATE;

H_2 : Crest elevation of non-overflow section determined from design flood water level

H_d : Design flood water level (= 1,141.30 m: see ii) on determination of flood water level)

Wave Height due to Wind

Although there are various formulae available regarding the relations of wind speed and fetch with wave height, the following equation combining the S.M.B. method and Saville's method is used in this case.

$$h_w = 0.00086 \times V^{1.1} \times F^{0.45}$$

where,

hw : Wave height due to wind (m)

V : Average wind speed for 10 minutes (= 30 m/sec)

F : Maximum fetch (= 500 m)

$$h_w = 0.00086 \times 30^{1.1} \times 500^{0.45}$$
$$= 0.59 \text{ m}$$

Wave Height due to Earthquake

With regard to earthquake wave height the formula of Seiichi Sato which gives a comparatively large wave height is adopted.

$$h_e = K \bullet \tau / (2\pi) \bullet \sqrt{g \bullet H}$$

where,

h_e : Wave height due to earthquake (m)

K : Horizontal seismic coefficient (= 0.1)

τ : Earthquake cycle (= 1.0 sec)

H : Reservoir water depth from normal water level (= 25.0 m)

g : Acceleration of gravity (= 9.8 m/sec^2)

$$h_e = 0.1 \times 1.0/(2 \times \pi) \times \sqrt{9.8 \times 25.0}$$
$$= 0.25 \text{ m}$$

Results of Calculations

$$h_w + h_e = 0.59 + 0.25 = 0.84 < 2.00$$
$$h_w = 0.59 < 1.00$$
$$H_1 = H_n + 2.00 = 1,140.00 + 2.00 = 1,142.00$$
$$H_2 = H_d + 1.00 = 1,141.30 + 1.00 = 1,142.30$$

Based on the above calculations, the crest elevation is to be 1,143.00 m with an allowance added to H_2.

ii) Determination of Flood Water Level

The flood water level is to be the water level at which the design flood discharge of 400 m^3/sec flows down the overflow section of the dam when flushing gates are fully open.

Design Conditions

Design flood discharge : 400 m^3/sec
Overflow crest elevation: 1,140.00 m
Overflow crest length : 13.0 m x 4 spans = 52.00 m
Overflow crest configuration: Harold's standard overflow crest

$$y = 0.4001 \times X^{1.85}$$

Flushing gate: B x H = 4.00 m x 4.00 m

Calculation Equation

(Flushing Gate)

$$Q = C \cdot A \cdot \sqrt{2g \cdot H}$$

where,

Q : Discharge quantity (m^3/sec)
C : Runoff coefficient (= 0.85)
H : Storage head based on flushing gate center elevation (= 1,125.00 m)
A : Cross-sectional area of flushing gate (4.00 x 4.00 = 16.00 m^2)

(Overflow Section)

Fundamentally, Iwasaki's formula is used, with the influence of piers corrected using Ishii and Fujimoto's formula.

$$Q = n \cdot C' \cdot B \cdot H^{3/2}$$

$$C' = C \cdot (1 - Md \cdot (H/Hd)^{1.5})$$

For $n = 1$ or $n \geq 2$ and in addition $b/s \geq 0.8$,

$$Md = 0.0756 \cdot \left(\frac{Hd}{B}\right)^{0.5}$$

For $n \geq 2$ and in addition $b/s < 0.8$,

$$Md = 0.0756 \cdot (Hd/B)^{0.5} \cdot \{1/n + 1.465 \cdot ((n - 1)/n) \cdot (b/s)^{1.7}\}$$

$$C = 1.60 \cdot \frac{1+2a(H/Hd)}{1+a\ (H/Hd)}, \quad a = \frac{Cd-1.6}{3.2-Cd}$$

$$Cd = 2.200 - 0.0416 \cdot (Hd/W)^{0.99}$$

where,

Q :	Overflow quantity (m^3/sec)
n :	Number of spans (= 4 spans)
C′:	Runoff coefficient considering effects of piers and abutments
B :	Overflow width of a single span (= 13.00 m)
H :	Overflow head (m)
Md:	Adjustment factor for piers and abutments in relation to two-dimensional overflow coefficient
C :	Two-dimensional overflow coefficient for no piers and abutments
Hd:	Design head (m)
a :	Constant
Cd:	Overflow coefficient at H = Hd
W :	Dam height (= 30.0 m)
b :	Pier width (= 2.00 m)
s :	Horizontal distance from dam crest to pier end (= 2.40 m)

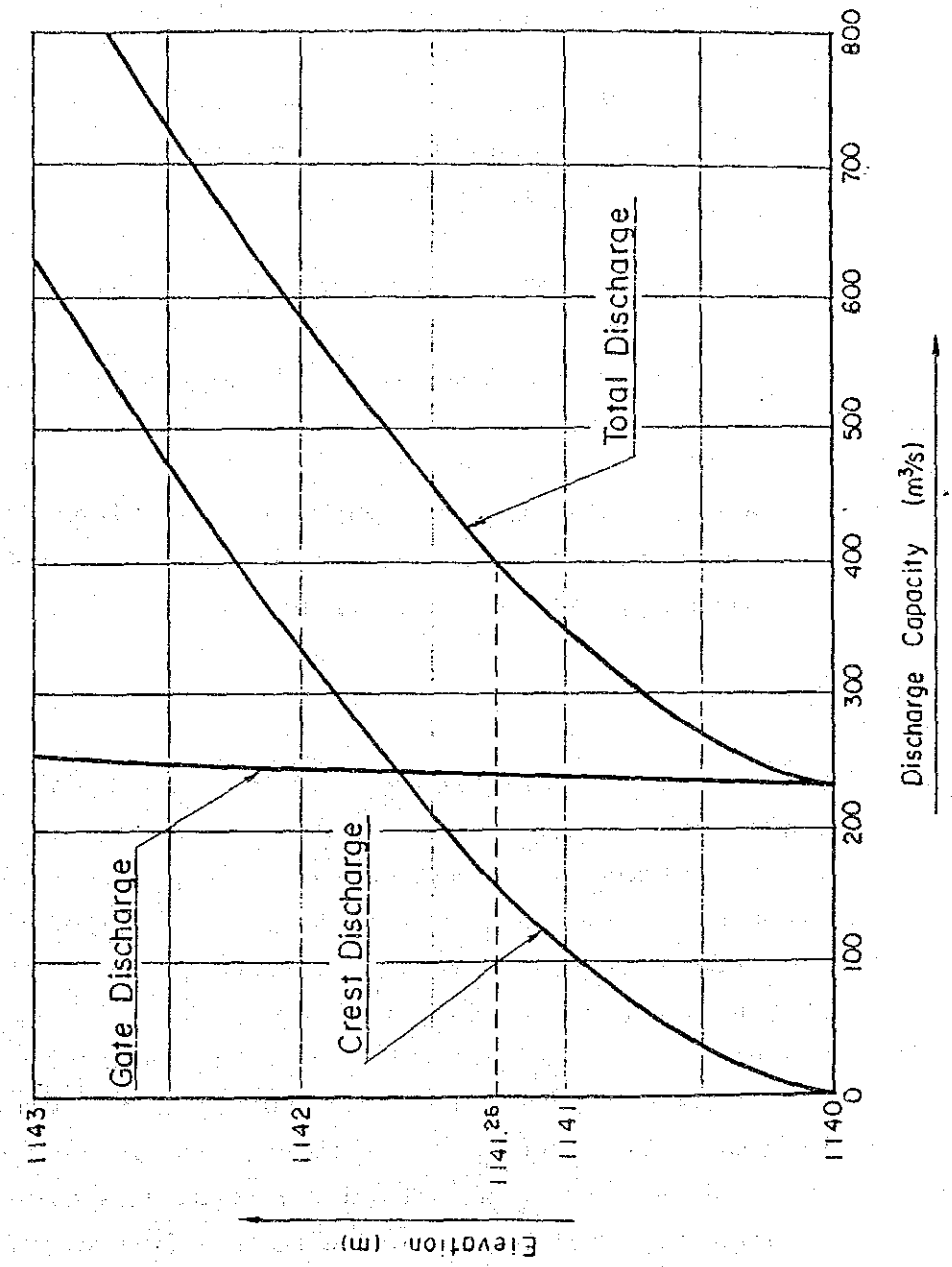
Calculation Results

The water level when the design flood discharge is flowing down is calculated as 1,141.26 m, and the flood water level is set at 1,141.30 m adding some allowances. The results of calculations are shown in Fig. 10-28.

iii) Stability Calculations of Dam

As a consequence of the foregoing study, the dam height is estimated a maximum of 35 m from the foundation rock at EL. 1,108.00 m to the crest at EL. 1,143.00 m and the crest length 177 m. Next, the stability calculations are made to determine the cross-sectional shape of the dam.

Fig. 10-28 Discharge Capacity of Spillway



The calculations are made for two cases of design flood and earthquake at normal high water level assuming the cross-sectional shape of the dam to have a downstream face with slope of 1:0.8 and a vertical upstream face.

The design conditions are as follows:

- That tensile stresses within the vertical plane perpendicular to the upstream face of the dam don't occur at the upstream end.

- That there is safety against shear.

- That the allowable compressive stress is not exceeded.

According to the results of calculations, the design conditions are satisfied with regard to overturning, sliding, and bearing strength.

(4) Spillway

The structure of the spillway, as given as design conditions under the subsection of (3), ii) on determination of flood water level, is to be a free overflow type, and floods are to be coped with by combined utilization of this spillway and flushing gates (4.0 m x 4.0 m). From the facts that the design flood discharge at the dam site is comparatively small at 400 m^3/sec even at PMF, while further, it is only about 50 m^3/sec for the past maximum, it is judged to be advantageous from the standpoints of the economics and maintenance and inspection to have a free overflow type.

As the energy dissipation system, forced jumping is to be induced by providing an end sill approximately 60 m downstream of the dam axis. Furthermore, from the facts