3. DESIGN OF FACILITIES AND STRUCTURE OF THE LOWER TEKAI

3. Design of Facilities and Structure of the Lower Tekai

3.1 Installed Capacity and Generated Energy

3.1.1 Head and Tail-Water Level

The head water level was set at EL 73.50, which is lower than the Lower Tekai Reservoir's H.W.L. of 75.00 by 1/3 of the effective depth of 4.5 m. The tail-water level was set at EL 55.60 from the discharge capacity of the river. The relationship between the water level and discharge according to actual river survey is as illustrated in Fig. 3-1.

3.1.2 Head Loss

(1) Head loss at intake

a) Head loss by inflow (ha)

$$ha = fe \cdot \frac{V_2^2}{2g}$$

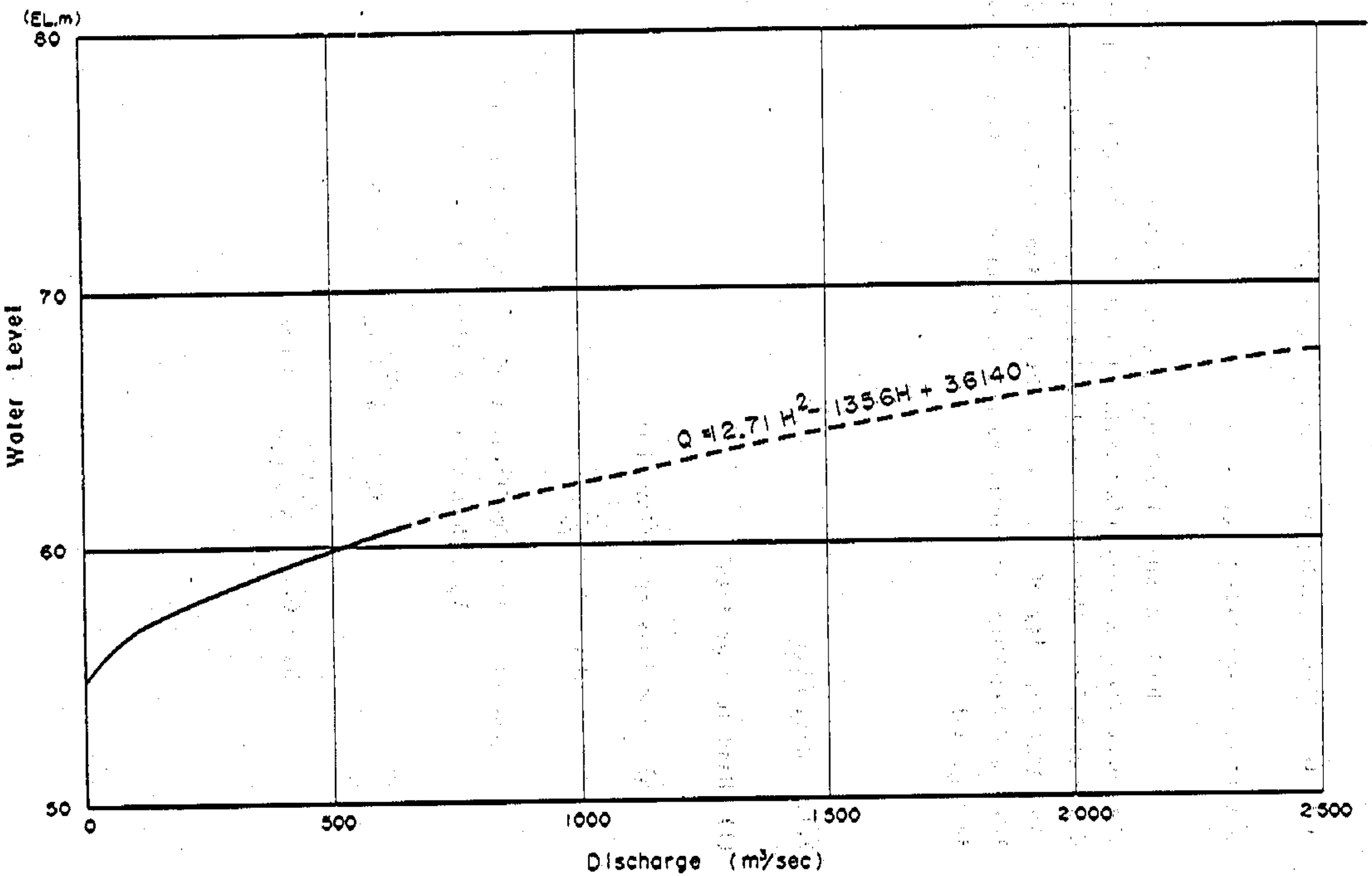
where; fe : Coefficient of loss by inflow = 0.2

V_2 : Mean velocity after inflow (m/s)

$$V_2 = \frac{40.0}{9.0 \times 9.0 \times 2} = 0.494 \text{ m/s}$$

$$ha = 0.2 \times \frac{0.494^2}{2 \times 9.8} = 0.002 \text{ m}$$

Fig 3-1 Water Level - Discharge Curve of Lower Tekain Dam Site



b) Head loss by screen (hb)

$$hb = fr \cdot \frac{V_1^{\,2}}{2g}$$

$$fr = \beta \cdot \sin\theta \cdot (\frac{t}{b})^{4/3}$$

where; β : Coefficient determined by the sectional form of the screen bar = 1.60

θ : Tilting angle of screen = 63°26'06"

t : Thickness of screen bar = 0.016 m

b : Mesh size of the screen bar = 0.15 m

V_1 : Mean velocity in upstream of screen (m/s) = 0.494 m/s

$$fr = 1.60 \times \sin(63.435°) \times (0.016/0.15)^{4/3} = 0.072$$

$$hb = 0.072 \times \frac{0.494^2}{2 \times 9.8} = 0.0009\ m$$

c) Head loss due to gradual contraction of section (hc)

$$hc = fgc \times \frac{V_2^2}{2g}$$

fgc : Coefficient of loss by gradual contraction

V_2 : Mean velocity after gradual contraction

$\theta = 24°47'26'$

$A_1 = 9.0 + 9.0 = 81.0\ m^2$

$A_2 = 5.0 + 5.0 = 25.0\ m^2$

$A_2/A_1 = 0.309$

$fgc = 0.023$

$$V_2 = \frac{40.0}{5.0 \times 5.0} = 1.600\ m/s$$

$$hd = 0.023 \times \frac{1.600^2}{2 \times 9.8} = 0.003 m$$

d) Head loss at intake (h_1)

$h_1 = ha + hb + hc$

$= 0.002 + 0.001 + 0.003$

$= 0.006\ m$

(2) Head loss in penstock

a) Head loss by friction (ha)

$$ha = f \cdot L \cdot \frac{V^2}{2g}$$

f : Coefficient of friction loss

$$f = \frac{124.5\ n^2}{D^{4/3}}$$

L : Extended length of iron pipeline (m)

D : Diameter of iron pipe (m)

V : Mean velocity in pipe (m/s)

n : Coefficient of roughness = 0.012

No.	D (m)	f	V (m/s)	$\frac{V^2}{2g}$	L (m)	h_a (m)
1	5.0	0.0016	2.037	0.212	11.690	0.004
2	5.0∿3.8 (4.4)	0.0019	2.631	0.353	12.055	0.008
3	3.8	0.0022	3.527	0.635	3.032	0.004
4	3.8∿2.6 (3.2)	0.0026	4.974	1.262	12.055	0.040
5	2.6	0.0031	7.534	2.896	10.950	0.098
Total						0.154

b) Head loss due to gradual contraction of section (hb)

$$hb = fgb \cdot \frac{V_2^2}{2g}$$

fgb : Coefficient of loss due to gradual contraction

V_2 : Mean velocity after gradual contraction (m/s)

No.	θ	D_1(m)	D_2(m)	A_1(m^2)	A_2(m^2)	A_2/A_1	fgb	V_2(m/s)	h_b (m)
1	5°42'38"	5.0	3.8	19.635	11.341	0.578	0.001	3.527	0.001
2	5°42'38"	3.8	2.6	11.341	5.309	0.468	0.001	7.534	0.003
Total									0.004

c) Head loss by curvature (hc)

$$hc = f_{b1} \times f_{b2} \times \frac{V^2}{2g}$$

f_{b1} : Coefficient of loss determined by the ratio between the radius of curvature P and pipe diameter D (P/D)

f_{b2} : Ratio of losses between each central angle of curvature θ and the central angle 90°

V : Mean velocity in pipe

No.	ρ	Dm	θ	f_{b1}	f_{b2}	V	$\frac{V^2}{2g}$	hc
1	13.000	4.4	53°07'48"	0.139	0.768	2.631	0.353	0.038
2	13.000	3.2	53°07'48"	0.134	0.768	4.974	1.262	0.130
Total								0.168

d) Head loss in penstock (h_2)

$$h_2 = ha + hb + hc$$
$$= 0.154 + 0.004 + 0.168$$
$$= 0.326$$

(3) Head loss at outlet (h_3)

a) Abandonment head loss of reaction turbine (ha)

$$ha = fse \cdot \frac{V_1^2}{2g}$$

$$fse = \{1 - (\frac{A_1}{A_2})\}^2$$

fse : Coefficient of loss by quick expansion

V_1 : Mean velocity before quick expansion (m/s)

= 2.000 m/s

A_1, A_2 : Cross sectional area of flow before and after quick expansion

Since $A_1 \ll A_2$ fse = 1

$$ha = \frac{2.000^2}{2 \times 9.8} = 0.204 \text{ m}$$

b) Head loss at outlet (h_3)

$$h_3 \doteq ha = 0.204 \text{ m}$$

(4) Total head loss

Head loss at intake (m)	0.006
Head loss at penstock (m)	0.326
Head loss at outlet (m)	0.204
Other head losses (m)	0.164
Total (m)	0.70

3.1.3 Power Generating Capacity

(1) Installed capacity

The installed capacity is calculated as follows:

$$P = 9.8 \times Q \times H \times \eta$$

where;

Q : Maximum discharge (= 40 m^3/s)

H : Effective head (= 73.50 - 55.60 - 0.70 = 17.2 m)

η : Comprehensive efficiency of turbine and generator (= 0.87)

$$P = 9.8 \times 40 \times 17.2 \times 0.87 = 5{,}865 \doteqdot 5{,}800 \text{ KW}$$

(2) Generated energy

The generated energy was calculated using daily discharge data by the mass curve and reservoir water level from 1961 to 1981. The average annual generated energy of 20 years was 40.3 GWH. Monthly variation was rather small, the average maximum and minimum outputs having been 3.8 GWH in January and 3.0 GWH in September, respectively.

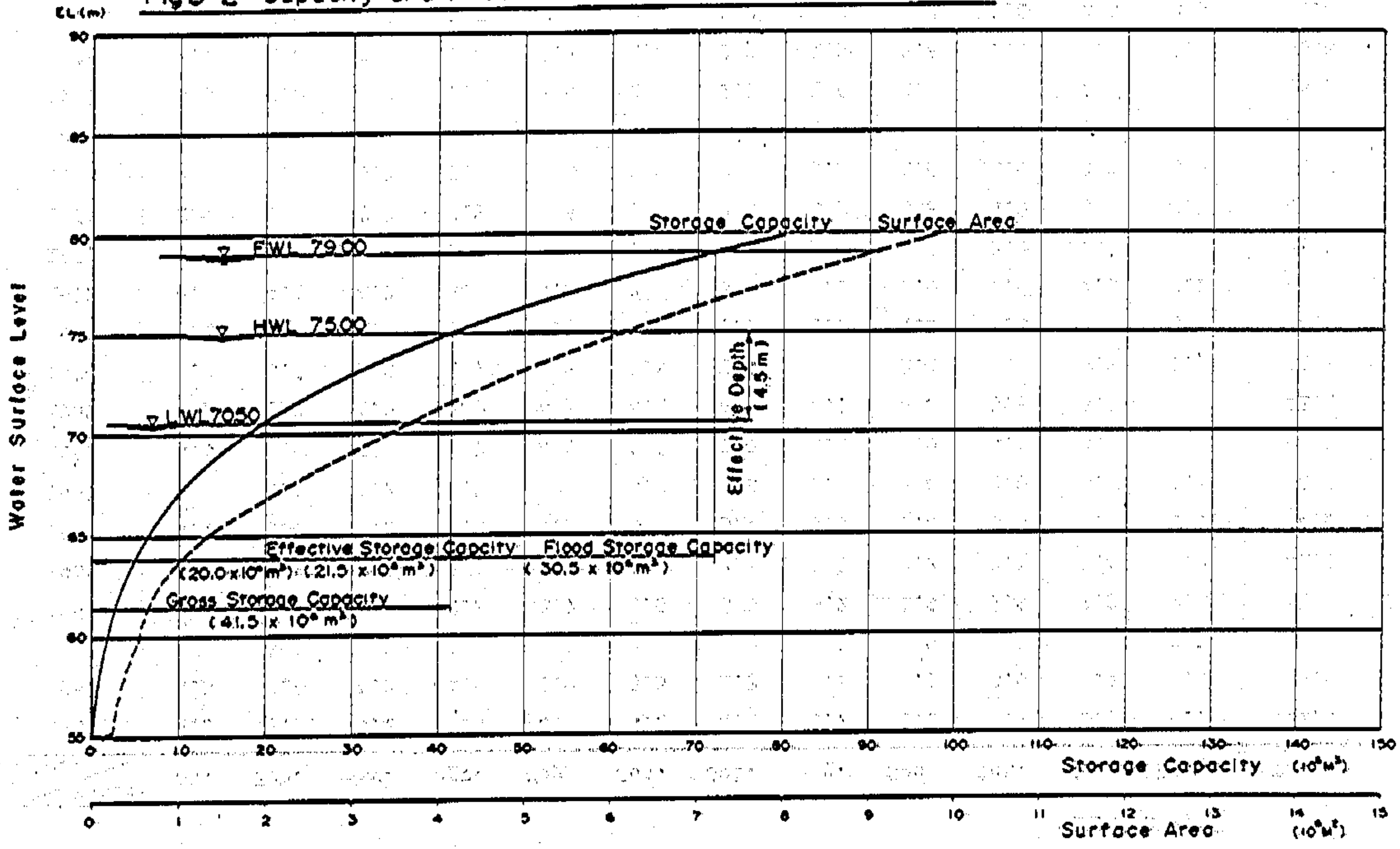
The monthly generated energy is shown in Table 3-1.

Table 3-1 Monthly Generated Energy of the Lower Tekai Power Station

(Unit: GWH)

Month / Year	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Total
1961	4.3	3.9	4.2	3.9	4.3	4.2	4.1	3.7	3.1	3.4	3.8	4.1	47.0
1962	4.3	3.8	4.2	4.0	3.9	3.5	3.3	3.1	3.5	3.6	3.4	4.0	44.6
1963	4.2	3.8	4.0	3.7	3.6	3.3	3.1	3.1	2.9	3.1	3.0	3.7	41.5
1964	4.2	3.9	4.0	3.8	3.7	3.0	3.3	3.2	2.9	2.9	3.2	3.8	41.9
1965	4.1	3.5	3.6	3.1	3.4	4.0	3.8	3.5	3.2	3.1	3.4	3.8	42.5
1966	4.1	3.8	4.2	4.0	4.1	3.6	3.4	3.3	3.0	3.3	3.5	3.9	44.2
1967	4.1	3.4	4.2	4.1	4.3	4.0	3.8	3.5	3.0	3.0	3.0	4.1	44.5
1968	4.3	3.9	3.9	3.7	4.0	3.8	3.9	4.0	3.9	4.1	3.9	3.9	47.3
1969	4.2	3.8	4.0	3.9	4.1	4.1	4.2	4.1	4.0	3.8	3.8	4.1	48.1
1970	4.2	3.7	3.8	3.7	3.8	3.6	3.4	3.1	2.8	3.1	3.2	2.9	41.3
1971	4.1	3.5	4.0	3.9	3.8	3.7	3.7	3.7	4.0	3.8	3.2	3.4	44.8
1972	4.3	4.1	4.2	3.8	4.0	3.7	3.5	3.0	2.6	2.6	2.5	3.2	41.5
1973	3.4	3.5	3.8	3.5	3.6	3.4	3.3	3.0	2.6	2.3	2.3	3.5	38.2
1974	2.9	2.6	2.9	2.9	3.6	3.7	3.9	3.9	3.8	4.0	3.7	3.7	41.6
1975	3.8	3.2	3.2	3.3	3.8	3.9	3.9	3.8	3.6	3.9	3.3	3.5	43.2
1976	3.5	2.0	2.1	2.0	2.2	2.2	2.4	2.4	2.7	3.2	3.3	3.2	31.2
1977	3.3	3.0	3.3	3.1	3.0	2.8	2.7	2.6	2.4	2.7	2.9	3.1	34.9
1978	3.1	2.8	3.0	2.8	2.8	2.8	2.9	2.9	2.6	2.5	2.5	3.1	33.8
1979	3.3	2.9	3.0	2.7	2.7	2.4	2.2	1.8	1.7	2.0	2.4	3.5	30.6
1980	1.9	1.7	1.8	1.7	1.9	1.9	1.8	1.8	1.8	2.2	2.6	2.9	24.0
Total	75.6	66.8	71.4	67.6	70.6	67.6	66.6	63.5	60.1	62.6	62.9	71.4	806.7
Average	3.8	3.3	3.0	3.4	3.5	3.4	3.3	3.2	3.0	3.1	3.1	3.5	40.3

Fig 3-2 Capacity and Area Curve of Lower Tekai Reservoir



3.2 Design Flood

The design flood discharge was determined so as to cope with the highest probable discharge which occurs only once every 1,000 years. The discharge volume was obtained by a simulation study which takes into account surcharge of the Upper Tekai Reservoir in addition to the probable flood discharge from the remaining catchment which occurs only once every 1,000 years (Refer to Report on Hydrology).

Flood discharge factors are as follows:

Peak flood discharge : 1100 m^3/s
Maximum water level : EL 79.00

3.3 Design Sedimentation

Refer to 2.3.

3.4 Design Seismic Intensity

Refer to 2.4.

3.5 Dam Stability Analysis

(1) Basic data on dam

Type of dam		Concrete gravity dam
Dam crest elevation	Overflow section	EL 75.000 m
	Non-overflow section	EL 81.000 m
Normal high water level		EL 75.000 m
Low water level		EL 70.500 m
Design flood water level		EL 79.000 m
Design sedimentation level		EL 63.500 m
Downstream water level	Normal stage	EL 56.500 m
	Flood stage	EL 62.700 m

(2) Design water level

Studies will be made for the following two cases.

(Under normal conditions)
Normal high water level + wave-height by wind + wave - height by earthquake

(In the case of flood)
Design flood water level + wave-height by wind + wave - height

i) Wave-height by wind (hw)

$$hw = 0.00086V^{1.1}F^{0.45} \quad \text{(S.M.B. method)}$$

where; hw : Wave-height (m)
V : Wind velocity = 30 m/s
F : Fetch (m) = 800 m

$$\therefore hw = 0.00086 \times 30^{1.1} \times 800^{0.45}$$
$$= 0.73 \text{ m}$$

ii) Wave-height by earthquake (he)

$$he = \frac{1}{2} \cdot \frac{K \cdot \tau}{\pi} \sqrt{g \cdot Ho}$$

where; he : Height of wave by earthquake from the water level of reservoir (m)

K : Design seismic intensity = 0.10

τ : Cycle of seismic tremor (sec) = 1.0 sec

Ho : Water depth of reservoir of normal high water level = 30,000 m

$$\therefore he = \frac{1}{2} \frac{0.1 \times 1.0}{\pi} \times \sqrt{9.8 \times 30.0}$$
$$= 0.27 \text{ m}$$

iii) Design water level

Design water levels resulting from the above calculation are:

(Under normal conditions)

EL 75.000 m + 0.73 + 0.27 m = EL 76.000 m

(In the case of earthquake)

EL 79.000 m + 0.73 m + 0.27 m = EL 80.000 m

(3) Various numerical values used in the above calculation:

Wc : Unit weight by volume of dam concrete

Plain concrete	2.30 t/m^3
Reinforced concrete	2.40 t/m^3
Wo : Unit weight of reservoir water	1.00 t/m^3

γ : Unit weight by volume of silted deposition in water

Unit weight by volume of silted deposition in the air;

$$\gamma o = 1.80\ t/m^3$$

Porosity;

$$n = 0.4$$

$$\therefore \gamma = \gamma o - Wo\ (1 - n) = 1.20\ t/m^3$$

Ce : Coefficient of earth-pressure (in water)

$$Ce = \frac{1 - \sin 20°}{1 + \sin 20°} = 0.49 \doteqdot 0.5$$

U : Uplift

k : Horizontal seismic intensity

Water-filled state	0.10
Empty state	0.05

Pd : Dynamic water-pressure by earthquake

According to the formula by Westergaard;

$$Pd = \frac{7}{12} Wo \cdot k \cdot Ho^{\frac{1}{2}} \cdot h^{\frac{2}{3}}, \quad yd = 0.40h$$

where; Ho : Water depth from the surface of dam foundation to the high water level (m)

h : Water depth from the high water level to each section (m)

k : Horizontal seismic intensity

yd : Height from each section to the point of action by the resultant force of dynamic water pressure (m)

σ : Allowable unit stress of concrete

Allowable unit compressive stress	σca = 70 kg/cm^2
Allowable unit tensile stress	σta = 3 kg/cm^2
Allowable unit shearing stress	τca = 25 kg/cm^2

fc : Coefficient of internal friction for concrete = 0.80

τ : Shearing strength of rock bed

General part	20 kg/cm^2
Weathered part	10 kg/cm^2

fr : Coefficient of internal friction for rock bed = 0.70

(4) Symbol

Symbols used in this calculation are defined as follows:

W_1 : Weight of concrete dam body

W_2 : Weight of pier in overflow section

F : Seismic force of dam body acting toward downstream in water filled state

F' : Seismic force of dam body acting toward upstream in empty state

F_1 : Seismic force of pier in the overflow section acting toward downstream in water-filled state

F_1' : Seismic force of pier in the overflow section acting toward upstream in empty state

P_1 : Hydrostatic pressure acting in the upstream

V_1 : Vertical load by hydrostatic pressure of upstream

V_2 : Vertical load by sedimentation in the upstream

E : Horizontal load by sedimentation in the upstream

U : Uplift

Pd : Dynamic water pressure by earthquake

xo : Arm length from dam axis (Y axis)

yo : Arm length from each section

M : Moment by external force

ΣH : Horizontal resultant force

ΣV : Vertical resultant force

ΣH : Resultant moment

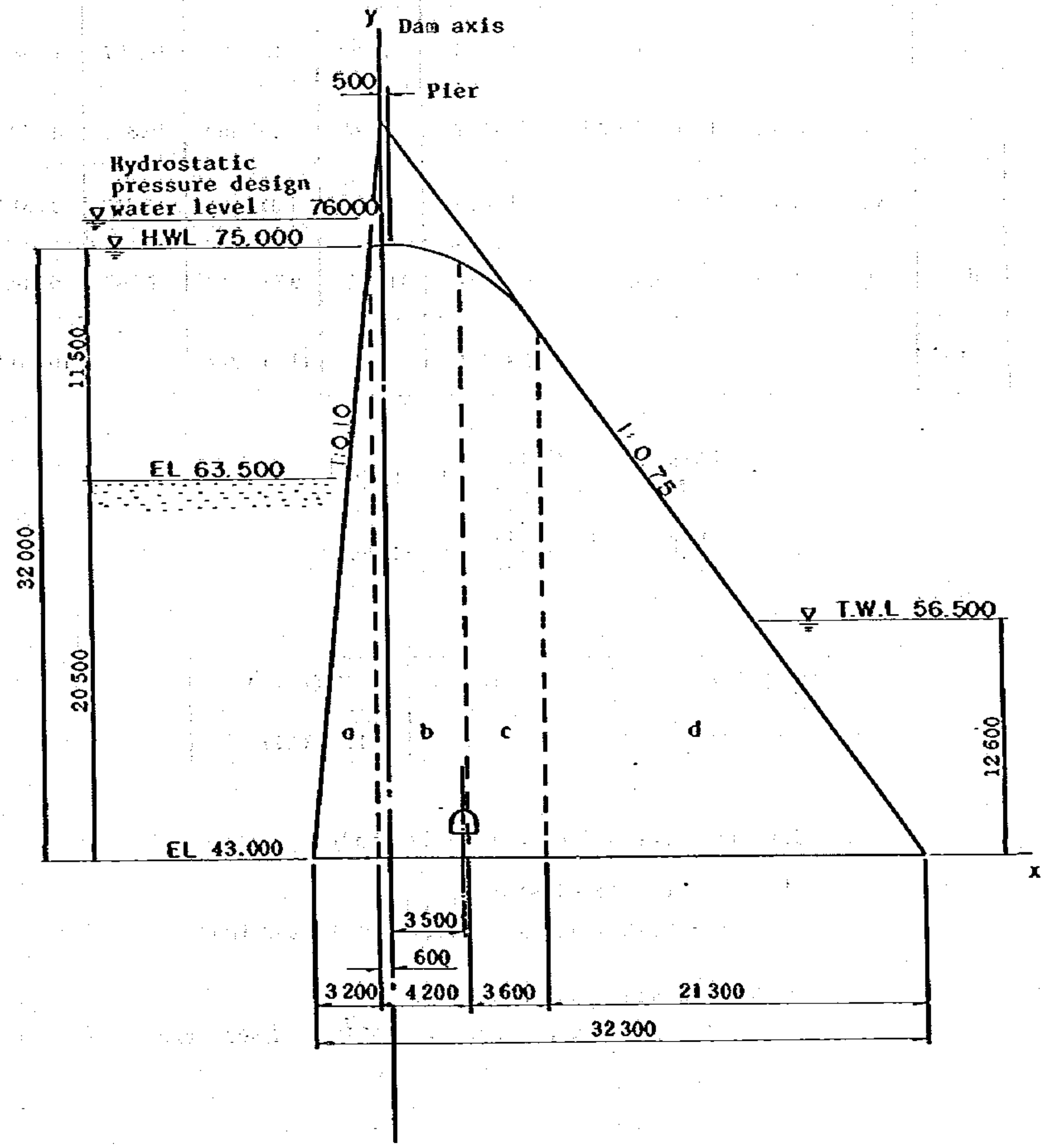
(5) Stability calculation

Calculation of the stability of dam body must be carried out so as to satisfy the following conditions against external force and its dead weight on horizontal section and the contact area of the dam body and the foundation rock bed.

- To be safe against sliding.
- Vertical tensile stress is not generated at the end of upstream.
- Comprehensive stress and tensile stress should not be exceeded the allowable stresses.

Calculation was carried out on the cross-section of the overflow section for the three cases of normal high water level, design flood water level and the empty state.

Fig. 3-3 Stability Calculation of Overflow Section during Normal High Water Level



i) External force and dead weight

a) Dead weight and moment of dam body

	W (t/m)	x	W·x	y	W·y
a	$3.20 \times 32.00 \times \frac{1}{2} \times 2.30 \doteq 117.760$	−1.667	−196.306	10.667	1 256.146
b	$4.20 \times (32.00+31.40) \times \frac{1}{2} \times 2.30 = 306.222$	1.493	457.189	15.850	4 853.619
c	$3.60 \times (31.40+27.80) \times \frac{1}{2} \times 230 = 245.088$	5.364	1 314.652	14.818	3 631.714
d	$21.30 \times 27.80 \times \frac{1}{2} \times 2.30 = 680.961$	14.300	9 737.742	9.267	6 310.466
Total	1 350.031		11 313.277		16 051.945

$$x = \frac{\Sigma W \cdot x}{\Sigma W} = \frac{11\,313.277}{1\,350.031} = 8.380 \text{ m}$$

$$y = \frac{\Sigma W \cdot y}{\Sigma W} = \frac{16\,051.945}{1\,350.031} = 11.890 \text{ m}$$

W_1 (t/m)	x (m)	M (tm/m)
1 350.031	8.380	11 313.277

b) Seismic force and moment on dam body

* High water level state

$$F = W_1 \cdot k = 1\,350.031 \times 0.10 = 135.003 \text{ t/m}$$

* Empty state

$$F' = W_1 \cdot \frac{k}{2} = 1\,350.031 \times \frac{0.10}{2} = 67.502 \text{ t/m}$$

High water level state			Empty state		
F (t/m)	y (m)	M (tm/m)	F' (t/m)	y (m)	M (tm/m)
135.003	11.890	1 605.186	— 67.502	11.890	— 802.599

c) Horizontal force and moment of hydrostatic pressure

$$P_1 = \frac{1}{2} \cdot Wo \cdot (h + hw + hw)^2$$

$$= \frac{1}{2} \times 1.0 \times (32.00 + 0.73 + 0.27)^2 = 544.500$$

P_1 (t/m)	y (m)	M (tm/m)
544.500	11.000	5 989.500

d) Vertical force and moment of hydrostatic pressure

$$V_1 = \frac{1}{2} \times 3.20 \times 32.00 \times 1.00 = 51.200 \quad t/m$$

V_1 (t/m)	x (m)	M (tm/m)
51.200	— 2.733	— 139.930

e) Horizontal force and moment of sedimentation

$$E = \frac{1}{2} (hs \times Ce \times \gamma) \times hs$$

$$= \frac{1}{2} (20.5 \times 0.5 \times 1.20) \times 20.5$$

$$= 126.075 \quad t/m$$

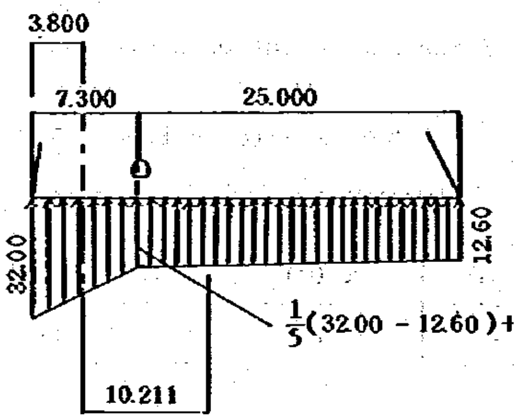
E (t/m)	y (m)	M (tm/m)
126.075	13.667	1723.067

f) Vertical force and moment of sedimentation

$$V_2 = \frac{1}{2} \times 2.05 \times 20.5 \times 1.20 = 25.215 \quad t/m$$

V_2 (t/m)	x (m)	M (tm/m)
25.215	− 3.117	− 78.595

g) Uplift and moment



$$\frac{1}{5}(32.00 - 12.60) + 12.60 = 16.480 \quad t$$

$$V = \frac{1}{2} \times (32.00 + 16.48) \times 7.30 = 176.952$$

$$\frac{1}{2} \times (16.48 + 12.60) \times 25.00 = 363.500$$

$$= 540.452 \quad t/m$$

U (t/m)	x (m)	M (tm/m)
− 540.452	10.211	− 5 518.555

h) Dynamic water pressure and moment by earthquake

$$Pd = \frac{7}{12} \cdot Wo \cdot k \cdot Ho^{1/2} \cdot h^{3/2}$$

$$= \frac{7}{12} \times 1.0 \times 0.10 \times 32.00^{1/2} \times 32.00^{3/2}$$

$$= 59.733$$

Pd (t/m)	y (m)	M (tm/m)
59.733	12.800	764.582

i) Weight and moment of pier

W_2 (t/m)	x (m)	M (tm/m)
12.000	0.500	6.000

j) Seismic force and moment on pier

			Empty state		
F_2 (t/m)	y (m)	M (tm/m)	F_2' (t/m)	y (m)	M (tm/m)
1.200	38.000	45.600	− 0.600	38.000	− 22.800

k) Total of external force and dead weight

∘ High water level state

(a)+(d)+(f)+(g)+(i)		(b)+(c)+(e)+(h)+(j)		Resultant moment (tm/m)
ΣV (t/m)	ΣM_V (tm/m)	ΣH (t/m)	ΣM_H (tm/m)	$\Sigma M=\Sigma M_V+\Sigma M_H$
897.994	5 582.197	866.511	10 127.935	15 710.132

∘ Empty state

(a) + (i)		(b) + (j)		Resultant moment (tm/m)
ΣV (t/m)	ΣM_V (tm/m)	ΣH (t/m)	ΣM_H (tm/m)	$\Sigma M=\Sigma M_V+\Sigma M_H$
1 362.031	11 319.277	68.102	825.399	12 144.676

ii) Stability calculation

a) Stability against sliding

$$\frac{f \cdot \Sigma V + \tau \cdot b}{\Sigma H} = n > 4.0$$

where; f : Coefficient of internal friction = 0.70
τ : Shearing strength = 100 t/m^2
b : Length of section (m)
n : Safety factor against shearing friction

∘ High water level state

f·ΣV (t/m)	τ·b (t/m)	f·ΣV+τ·b (t/m)	ΣH (t/m)	n
628.596	3 230.000	3 858.596	866.511	4.5

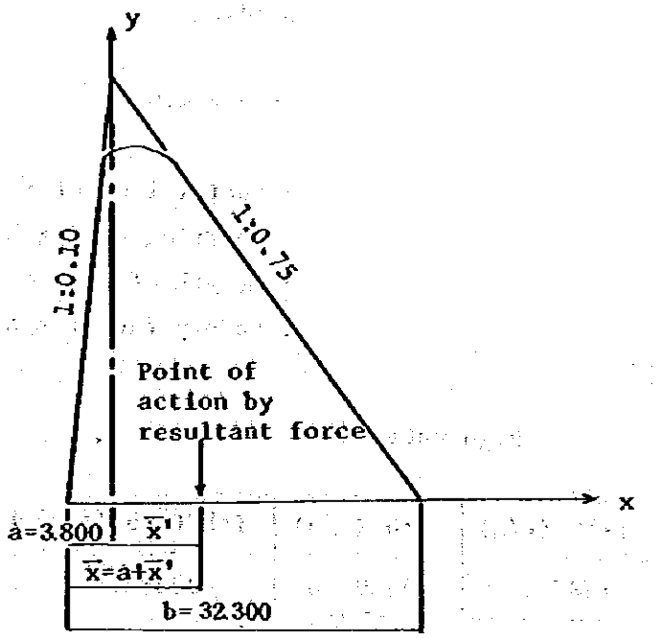
∘ Empty state

f·ΣV (t/m)	τ·b (t/m)	f·ΣV+τ·b (t/m)	ΣH (t/m)	n
953.422	3 230.000	4 183.422	68.102	61.4

As above, n > 4.0 so that the stability against sliding is assured both in the high water level state and the empty state.

b) Stability against tumbling

$$\bar{x}' = \frac{\Sigma M}{\Sigma V}$$



∘ High water level state

ΣM (tm/m)	ΣV (t/m)	$\bar{x}'$ (m)	$\bar{x}$ (m)	$\frac{1}{3}b$ (m)	$\frac{2}{3}b$ (m)
15 710.132	897.994	17.495	21.295	10.767	21.533

∘ Empty state

ΣM (tm/m)	ΣV (t/m)	$\bar{x}'$ (m)	$\bar{x}$ (m)	$\frac{1}{3}b$ (m)	$\frac{2}{3}b$ (m)
12 144.676	1 362.031	8.917	12.717	10.767	21.533

The point of action by resultant force is within the middle thirds so that stability against tumbling is assured.

c) Stability against compressive stress, etc.

$$\left.\begin{array}{l} Pd \\ \\ Pu \end{array}\right\} = \frac{\Sigma V}{b}\left(1 \pm \frac{6\cdot e}{b}\right)$$

where;

e : Distance from the center of section to the point acted resultant force (m)

Pd : Unit compressive stress at the downstream end (t/m^2)

Pu : Unit compressive stress at the upstream end (t/m^2)

∘ High water level state

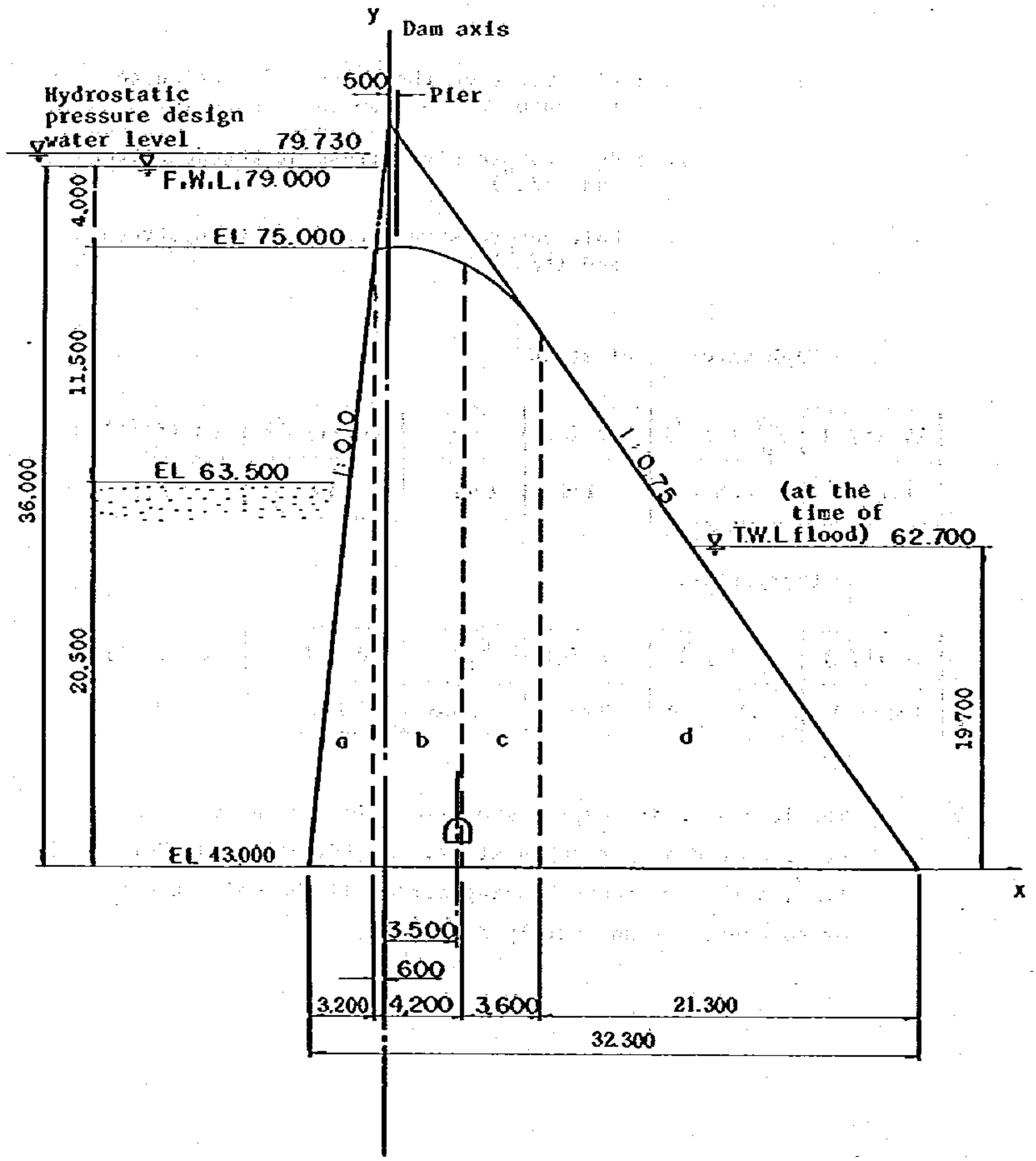
ΣV (t/m)	$\frac{\Sigma V}{b}$ (t/m^2)	e (m)	$\frac{6\cdot e}{b}$	Pd (t/m^2)	Pu (t/m^2)
897.994	27.802	5.145	0.956	54.4	1.2

∘ Empty state

ΣV (t/m)	$\frac{\Sigma V}{b}$ (t/m)	e (m)	$\frac{6\cdot e}{b}$	Pd (t/m^2)	Pu (t/m^2)
1 362.031	42.168	−3.433	−0.638	15.3	69.1

The largest unit compressive stress is generated at the upstream end in the empty state. As its value is 69.1 t/m^2, both the ground bearing force and the unit stress of concrete are adequately assured.

Fig. 3-4 Stability Calculation during Flood



1) External force and dead weight

a) Dead weight and moment of dam body

	W (t/m)	x	W·x	y	W·y
a	$3.20 \times 32.00 \times \frac{1}{2} \times 2.30 = 117.760$	−1.667	−196.306	10.667	1 256.146
b	$4.20 \times (32.00+31.40) \times \frac{1}{2} \times 2.30 = 306.222$	1.493	457.189	15.850	4 853.619
c	$3.60 \times (31.40+27.80) \times \frac{1}{2} \times 2.30 = 245.088$	5.364	1 314.652	14.818	3 631.714
d	$21.30 \times 27.80 \times \frac{1}{2} \times 2.30 = 680.961$	14.300	9 737.742	9.267	6 310.466
Total	1 350.031		11 313.277		16 051.945

$$x = \frac{\Sigma W \cdot x}{\Sigma W} = \frac{11\,313.277}{1\,350.031} = 8.380 \text{ m}$$

$$y = \frac{\Sigma W \cdot y}{\Sigma W} = \frac{16\,051.945}{1\,350.031} = 11.890 \text{ m}$$

W_1 (t/m)	x (m)	M (tm/m)
1 350.031	8.380	11 313.277

b) Seismic force and moment on dam body

Not considered.

c) Horizontal force and moment of hydrostatic pressure

$$P_1 = \frac{1}{2} \cdot Wo \cdot (h + hw + hw)^2$$

$$= \frac{1}{2} \times 1.0 \times (36.00 + 0.73)^2 = 674.546 \text{ t/m}$$

P_1 (t/m)	y (m)	M (tm/m)
674.546	12.243	8 258.467

d) Vertical force and moment of hydrostatic pressure

$$V_1 = \frac{1}{2} \times 3.60 \times 36.00 \times 1.00 = 64.800 \text{ t/m}$$

V_1 (t/m)	x (m)	M (tm/m)
64.800	− 2.600	− 168.480

e) Horizontal force and moment of sedimentation

$$E = \frac{1}{2} (hs \times Ce \times \gamma) \times hs$$

$$= \frac{1}{2} (20.5 \times 0.5 \times 1.20) \times 20.5$$

$$= 126.075 \text{ t/m}$$

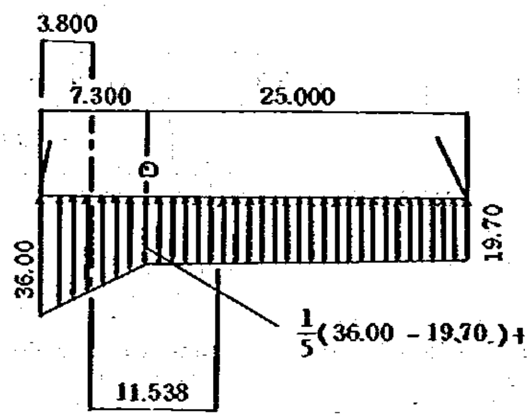
E (t/m)	y (m)	M (tm/m)
126.075	13.667	1723.067

f) Vertical force and moment of sedimentation

$$V_2 = \frac{1}{2} \times 1.85 \times 18.5 \times 1.20 = 20.535 \text{ t/m}$$

V_2 (t/m)	x (m)	M (tm/m)
25.215	− 3.117	− 78.595

g) Uplift and moment



$\frac{1}{5}(36.00 - 19.70) + 19.70 = 22.96$ t

$$V = \frac{1}{2} \times (36.00 + 22.96) \times 7.30 = 215.201$$

$$\frac{1}{2} \times (22.96 + 19.70) \times 25.00 = 533.25$$

748.451 t/m

U (t/m)	x (m)	M (tm/m)
— 748.454	11.538	— 8 635.662

h) Dynamic water pressure and moment by earthquake

Not considered.

i)) Weight and moment of pier

W_2 (t/m)	x (m)	M (tm/m)
12.000	0.500	6.000

j)) Seismic force and moment on pier

Not considered for the state of the design flood water level.

k) Total of external force and dead weight

(a)+(d)+(f)+(g)+(i)		(b)+(c)+(e)+(h)+(j)		Resultant moment (tm/m)
ΣV (t/m)	ΣM_V (tm/m)	ΣH (t/m)	ΣM_H (tm/m)	$\Sigma M=\Sigma M_V+\Sigma M_H$
703.592	2436.540	800.621	9981.534	12418.074

ii) Stability calculation

a) Stability against sliding

$$\frac{f \cdot \Sigma V + \tau \cdot b}{\Sigma H} = n > 4.0$$

where; f : Coefficient of internal friction = 0.70

τ : Shearing strength = 100 t/m^2

b : Length of section (m)

n : Safety factor against shearing friction

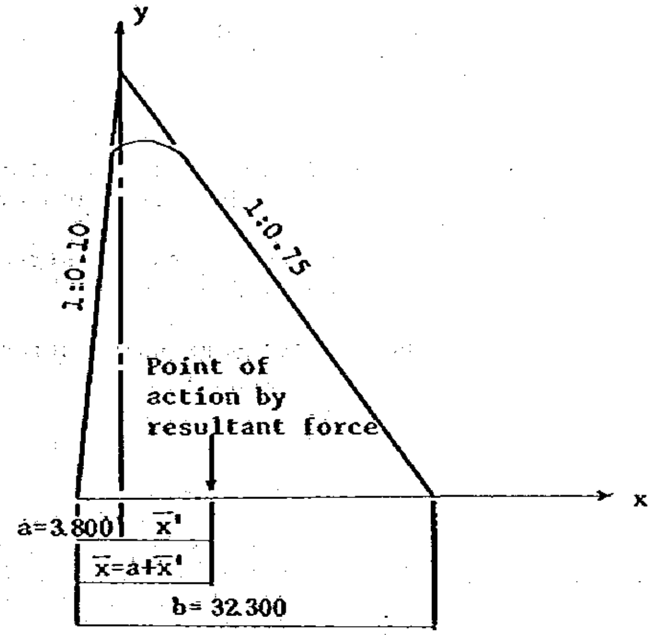
- Design flood water level state

$f \cdot \Sigma V$ (t/m)	$\tau \cdot b$ (t/m)	$f \cdot \Sigma V + \tau \cdot b$ (t/m)	ΣH (t/m)	n
492.514	3230.000	3722.514	800.621	4.6

As above, n > 4.0 so that the stability against sliding is assured both in the high water level state, and the empty state.

b) Stability against tumbling

$$\bar{x}' = \frac{\Sigma M}{\Sigma V}$$



ΣM (tm/m)	ΣV (t/m)	$\bar{x}'$ (m)	$\bar{x}$ (m)	$\frac{1}{3}$b (m)	$\frac{2}{3}$b (m)
12 418.074	703.592	17.650	21.450	10.767	21.533

The point of action by resultant force is within the middle thirds so that stability against tumbling is assured.

c) Stability against compressive stress, etc.

$$\left.\begin{matrix} Pd \\ \\ Pu \end{matrix}\right\} = \frac{\Sigma V}{b} \left(1 \pm \frac{b \cdot e}{b}\right)$$

where; e : Distance from the center of section to the point of action by resultant force (m)

Pd : Unit compressive stress at the downstream end (t/m^2)

Pu : Unit compressive stress at the upstream end (t/m^2)

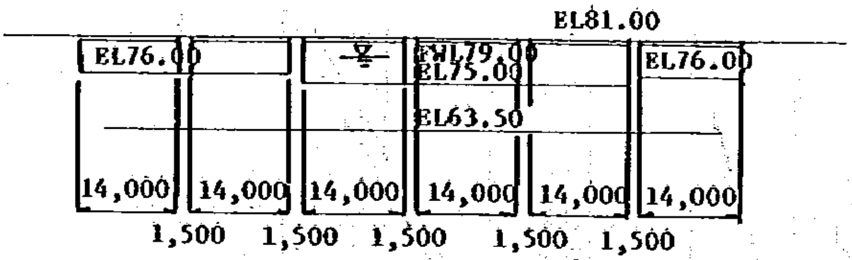
ΣV (t/m)	$\frac{\Sigma V}{b}$ (t/m^2)	e (m)	$\frac{6 \cdot e}{b}$	Pd (t/m^2)	Pu (t/m^2)
703.592	21,783	5.300	0.985	43.2	0.3

The largest unit compressive stress is generated at the upstream end. As its value is 43.2 t/m^2, the ground bearing force and the unit stress of concrete are adequately assured.

3.6 Spillway

3.6.1 Spillway Discharge

The spillway should allow flow down of the design flood discharge of 1,100 m^3/s at the reservoir water level of EL 79.00.



Overflow discharge

$$Q = Cd\ (B - K.N.H) \cdot H^{3/2}$$

$$Cd = 2.200 - 0.0416\ (Hd/w)^{0.990}$$

- Central part of spillway

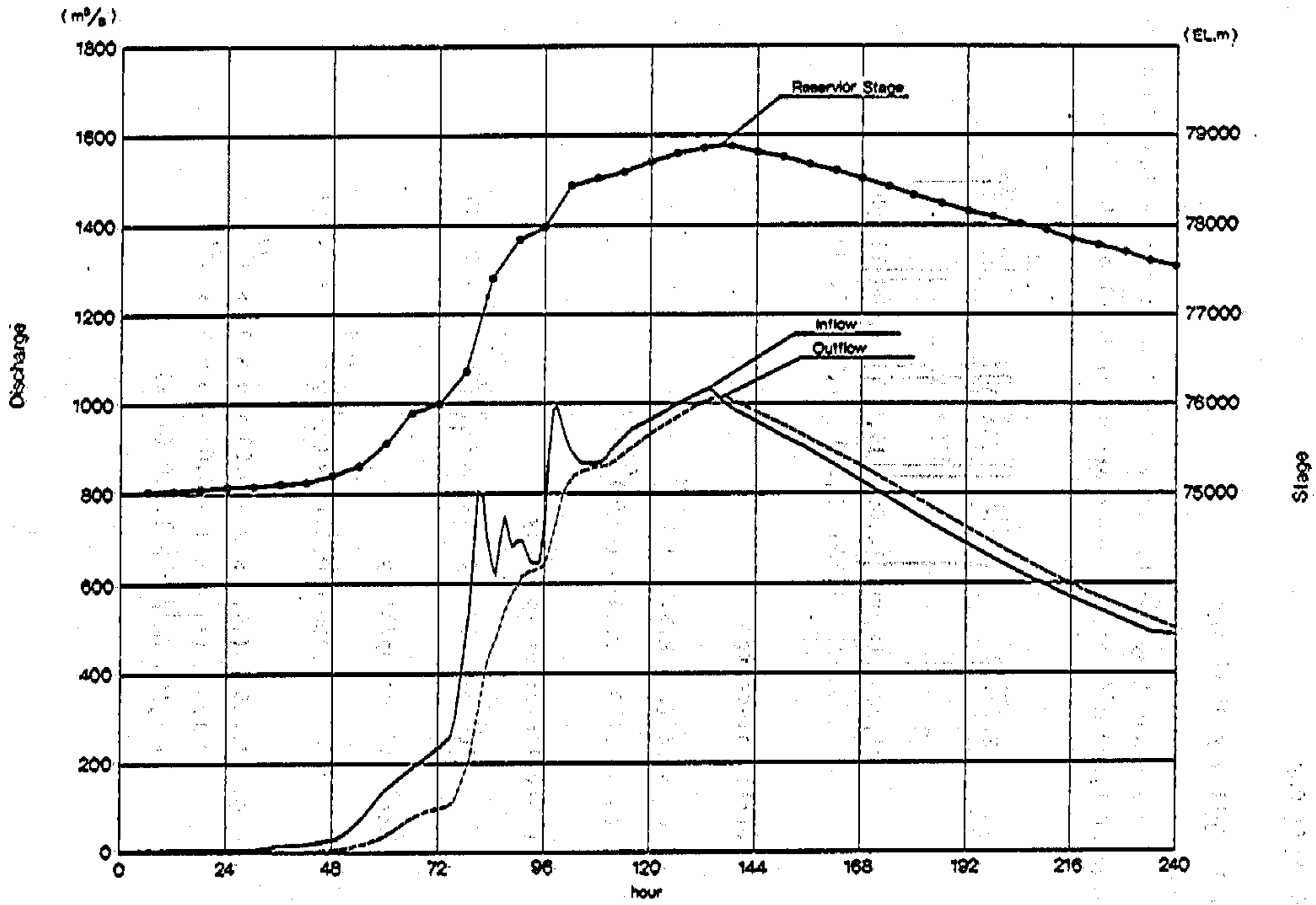
$$Cd = 2.200 - 0.0416 \times \left(\frac{4.0}{11.5}\right)^{0.99} = 2.185$$

- Each side of spillway

$$Cd = 2.200 - 0.0416 \times \left(\frac{3.0}{12.5}\right)^{0.99} = 2.190$$

$$Q = 2.185 \times (14.0 \times 3 - 0.03 \times 6 \times 4) \times 4^{3/2} +$$
$$2.190 \times (14.0 \times 3 - 0.02 \times 6 \times 3) \times 3^{3/2}$$
$$= 1,195\ (m^3/s) > 1,100\ (m^3/s)$$

Fig. 3-5 Reservoir Inflow, Stage, Outflow Hydrograph



3.6.2 Stilling Basin

Hydrographic calculation is carried out for dissipator in the event that $Q = 1,100\ m^3/s$ flows down. The spillway of this dam is 91.5 m wide at overflow portion and 50.0 m wide at the stilling basin. As it is impossible to obtain the precise hydrographic volume at the stilling basin, approximate calculation is carried out by assuming a channel width of 50 m considering safety side of design. Detailed review is left to hydrographic experiment in the Engineering Stage.

The water depth h_1 at the inflow portion of the stilling basin can be obtained by the following formula.

$$h_1 = \frac{q}{\sqrt{2g\ (W + H - h_1 - h_f)}}$$

where;
- q : Discharge per unit width
- W : Height from the surface of dam apron to the crest of spillway
- H : Water head of overflow
- h_j : Friction head loss at the downstream of dam which can be calculated according to the following formula:

$$\frac{h_f}{H} = 0.02\ \frac{W}{H}\ (\frac{W}{H} + 1)$$

Approximate value of H is calculated by assuming a channel width of 50 m as follows:

$$H = (Q/CB)^{2/3}$$

$$= (\frac{1100}{2 \times 50})^{2/3}$$

$$= 5.0$$

$$q = (1100/50) = 22\ m^3/s/m$$

$$W = 75.0 - 47.0 = 28.0\ m$$

$$h_f = H \times 0.02\ \frac{W}{H}\ (\frac{W}{H} + 1) = 5.0 \times 0.74 = 3.7\ m$$

$$h_1 = \frac{22}{\sqrt{19.6 \times (28 + 5 - 3.7 - h_1)}}$$

$$h_1 = 0.93 \text{ m}$$

The mean velocity of flow at this time $v_1 = q/h_1 = 24$ m/s

Froude number $F_1 = 7.9$

Accordingly, the water depth h_2 after hydraulic jump is:

$$h_2 = \frac{h_1}{2} \left(\sqrt{8F_1^2 + 1} - 1 \right) = 9.9 \text{ m}$$

The water level at the terminating end of hydraulic jump on the assumption that free hydraulic jump takes place is:

EL 47.0 + 9.9 = 56.9 m

Meanwhile, the downstream water level when $Q = 1{,}100$ m^3/s flows down is EL 62.7 m according to the stage-discharge curve.

Accordingly, the water level at the terminating end of hydraulic jump when a free hydraulic jump is assumed to take place at the time $Q = 1{,}100$ m^3/s flows down is lower by 5.8 m from the water level of the river in the downstream of the stilling basin. Thus, free hydraulic jump will not take place but submerged jump is anticipated instead. Dissipation, however, is also possible by submerged jump.

As for the length of the stilling basin,

$$L_j = 4.5\, h_2 = 4.5 \times 9.9 = 45 \text{ m (approximate)}$$

is planned for the initial design by applying the experimental formula for the length of free hydraulic jump. However, this will be reviewed later by hydrographic experiment.

3.7 Intake

In designing the inlet, its form was determined by conducting hydraulic model test. The hydraulic experiment and inlet form are discussed in 2.7.

3.8 Penstock

3.8.1 Type and Layout

The penstock goes through the concrete gravity dam and is led to the power generating plant located immediately below. As for the number of water turbine and generator installed capacity is 5.8 MW which can be operated 24-hours and loss of energy during the annual maintenance is not large. Therefore, one unit was adopted.

3.8.2 Economical Diameter of Penstock

(1) Contents studied

The internal diameter of the iron penstock was studied from the two economic factors of costs, consisting of the construction costs, and benefits, consisting of reduction in head losses.

(2) Study results

From Fig. 3-6, the optimum internal diameter of iron pipe is 5.0 m. Table 3-3 indicates the decreases and increases in benefits and costs for each internal diameter.

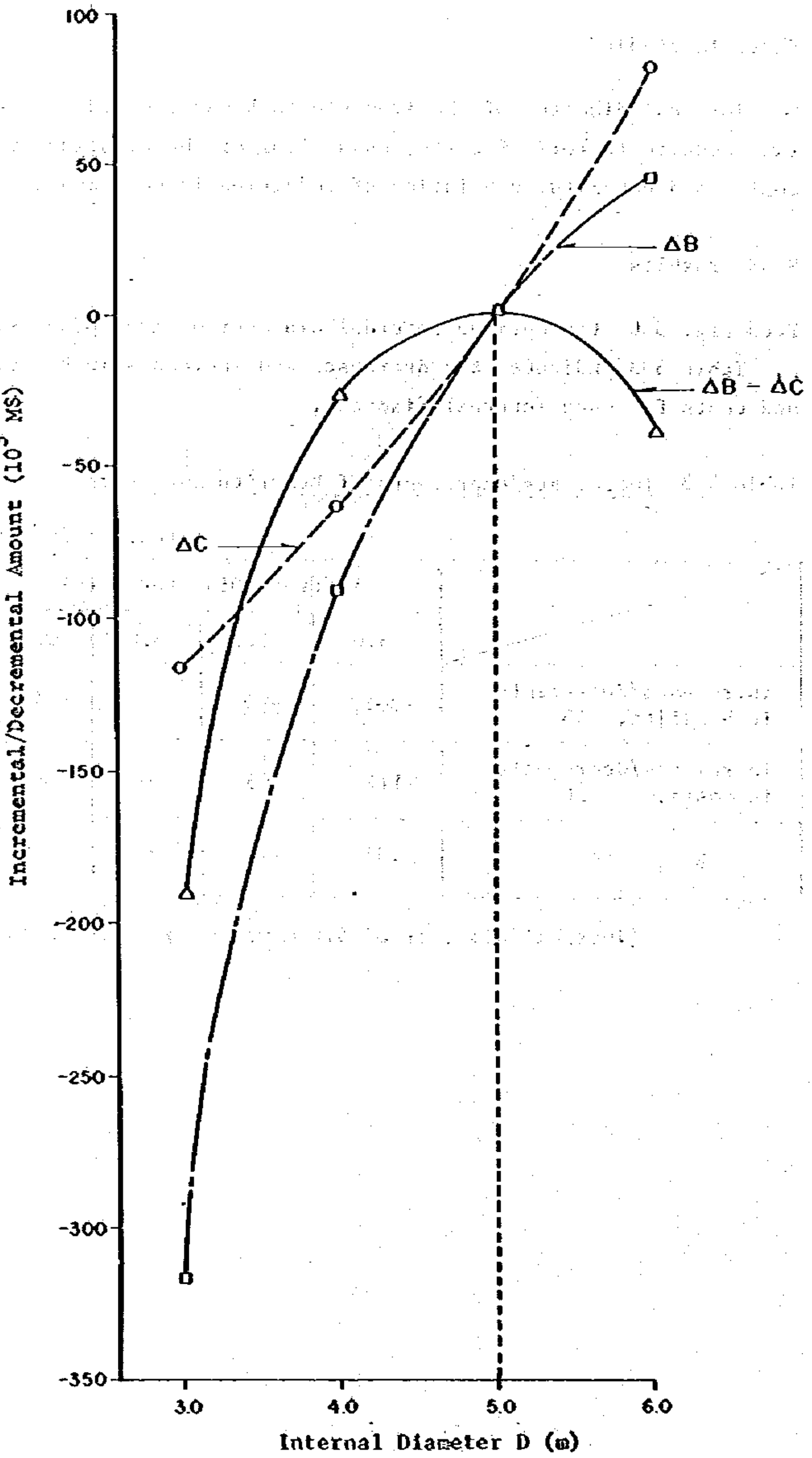
Table 3-3 Increments/Decrements of Benefits and Costs

(Unit : 10^3 H$)

	Internal Diameter D (m)			
	3.0	4.0	5.0	6.0
Increments/decrements in benefits, ΔB	-308	-89	0	43
Increments/decrements in costs, ΔC	-117	-65	0	81
$\Delta B - \Delta C$	-191	-24	0	-38

(Internal diameter of 5.0 m was used as the standard)

Fig. 3-6 Relationship between Internal Diameter of Pipe and ΔB, ΔC, and ΔB - ΔC



(3) Methods of study

When the increase and decrease in benefits and annual costs by changing the internal diameter of an iron pipe of a certain reference internal diameter are defined as ΔB and ΔC, the internal diameter which makes the value of $\Delta B - \Delta C$ the largest is the most economical internal diameter.

ΔB is obtained from variations in head losses and ΔC is obtained from variations in iron pipeline construction costs according to the respective formula shown below.

$$\begin{aligned}\Delta B &= (\Delta KWh) \times (\text{Unit price of benefits per KWh}) \\ &\quad + (\Delta KW) \times (\text{Unit price of benefits per KWh}) \\ &= \Delta KWh \times 0.190 \text{ M\$/KWh} + \Delta KW \times 142.70 \text{ M\$/KW}\end{aligned}$$

where;

$$\Delta KWh = 9.8 \times \eta_G \times \Sigma Q \times 24 \times [\text{Variation in head losses}]$$

$$\Delta KW = 9.8 \times \eta_G \times Q_{max} \times [\quad " \quad]$$

η_G : Combined efficiency = 0.87

$$\begin{aligned}\Delta C &= [\text{Increment/decrement in iron pipeline construction cost}] \\ &\quad \times (1 + \text{interest during construction}) \times (\text{Annual cost rate}) \\ &= [\text{Increment/decrement in iron pipeline construction cost}] \\ &\quad \times 1.2 \times 0.11586\end{aligned}$$

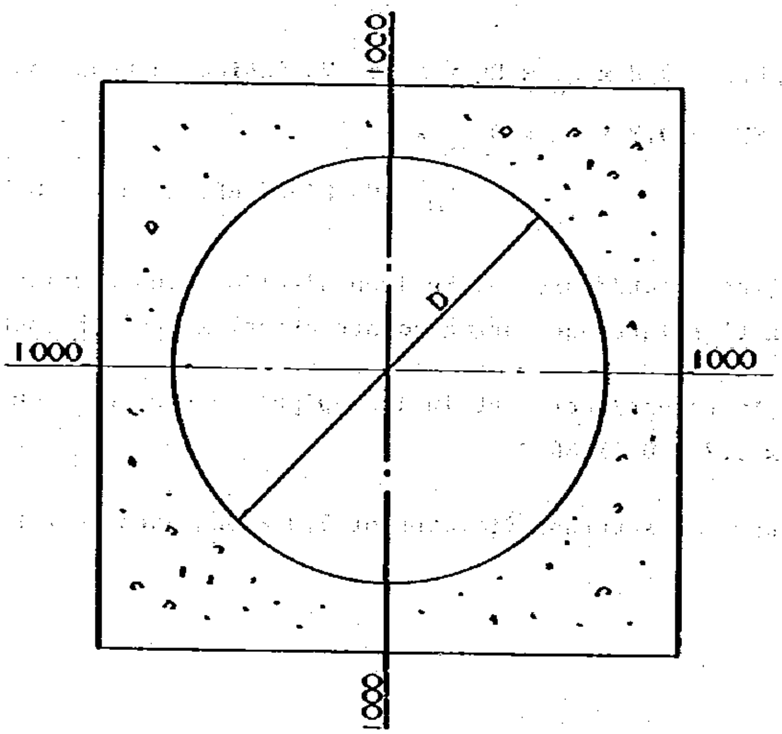
In this study, an internal diameter of 5.0 m was used as reference.

(4) Basic data assumed for study

- Extended length of iron pipeline to be 49.782 m.
- For the cross section, the cross section illustrated in Fig. 3-7 was used as reference, and calculations were made for the four cases of 3 m, 4 m, 5 m, and 6 m in internal diameter.
- The design water heads are 21.5 m for hydrostatic pressure and 30% of hydrostatic pressure (6.45 m) for water-hammer pressure.
- Data on other items will be specified individually.

Fig. 3-7 Cross Section Studied

(D = 3, 4, 5, and 6 m)



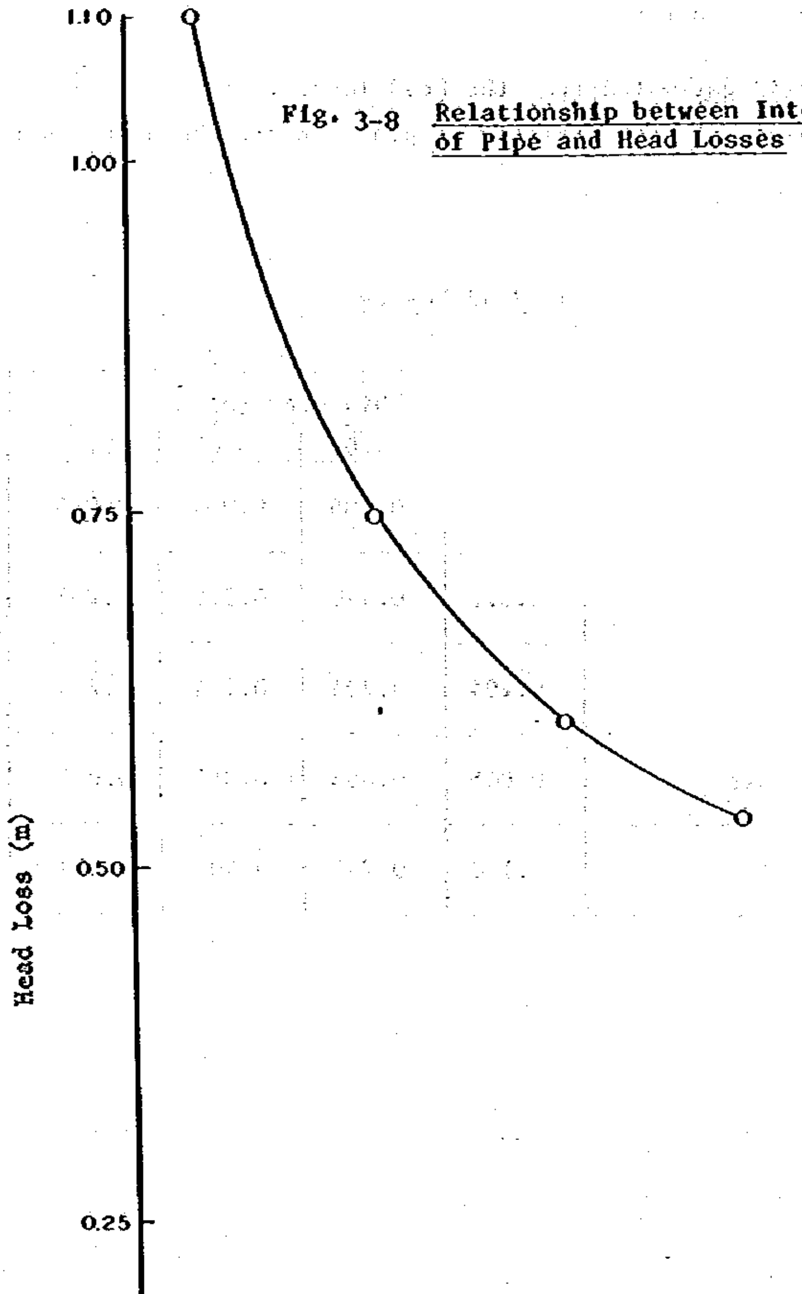
(5) Calculation of benefits

The benefit is estimated from the head losses.
The head losses of each diameter of penstock are shown in Table 3-4.

Table 3-4 Head Losses

Case / Loss	Diameter (m) 3.0	4.0	5.0	6.0
Head loss at intake	0.005	0.005	0.005	0.005
Head loss at penstock	0.826	0.470	0.326	0.255
Head loss at outlet	0.204	0.204	0.204	0.204
Other head loss	0.065	0.065	0.065	0.065
Total	1.100	0.744	0.60	0.529

Fig. 3-8 Relationship between Internal Diameter of Pipe and Head Losses



ii) Benefits

Variations in head losses ΔH when the internal diameter of 5.0 m is used as standard are shown in Table 3-5.

Table 3-5 <u>Variations in Head Losses</u>

Internal Diameter (m)	Head Loss (m)	ΔH (m)
3.0	1.100	-0.500
4.0	0.744	-0.144
5.0	0.600	0
6.0	0.529	0.071

Increase/decrease in benefits ΔB calculated by the formula shown in 3 above are shown in Table 3-6.

Table 3-6 <u>Increase/Decrease in Benefits</u>

	Unit	Internal Diameter D (m)			
		3.0	4.0	5.0	6.0
ΔH	m	-0.500	-0.144	0	0.071
ΔKWh	$10^3 KWh$	-1494	-430	0	212
ΔKW	KW	-171	-49	0	24
$\Delta KWh \times 0.190$	$10^3 M\$$	-284	-82	0	40
$\Delta KW \times 190.20$	$10^3 M\$$	-24	-7	0	3
ΔB	$10^3 M\$$	-308	-89	0	43

(6) Calculation of annual costs

The quantity of iron pipe and the volume of civil works for each different internal diameter were estimated and the construction costs were calculated. Thus, the increase and decrease in annual costs were obtained.

i) Weight of iron pipe

a) Study on pipe thickness

∘ Design internal pressure

The design internal pressure shall be the sum of the hydrostatic pressure and the water-hammer pressure. Here, the water-hammer pressure is assumed to be 30% of the hydrostatic pressure.

Hydrostatic pressure : $Ho = 75.00 - 52.000 = 23.00$ m

Water-hammer pressure : $Zo = 23.00 \times 0.3 = 6.900$ m

Maximum design internal pressure : $Ho + Zo = 29.90$ m

The design internal pressure for each point can be obtained by the following formula.

$$H = H_x + Z_x$$

H : Design internal pressure for each point
H_x : Hydrostatic pressure at each point
Z_x : Water-hammer pressure at each point

Here, Z_x is assumed to be largest at the center of value and zero at the starting point of iron pipe, and to change linearly in proportion to the pipe length in-between.

- Pipe thickness

As the material for penstock, steel material for welded construction (SM41) is to be used.

The following formula is used in calculating the pipe thickness.

$$to = \frac{H(Do + \varepsilon)}{2\sigma_a} + \varepsilon$$

where;

to : Pipe thickness (cm)
H : Design internal pressure (kg/cm^2)
Do : Internal diameter of iron pipe (cm)
ε : Extra thickness against corrosion and abrasion = 0.15 cm
σ_a : Allowable tensile stress = 1300 kg/cm^2

However, this is provided that the value shall not be less than the minimum pipe thickness indicated by the following formula.

$$t = \frac{Do + 800}{400} \quad (mm)$$

Calculations are made for the cases of pipe diameter (Do) being 4 m, 5 m, 6 m and 7 m.

∘ Weight of iron pipe

The following formula is used in calculation.

$$W' = \{(Do + 2t)^2 - Do^2\} \cdot \gamma s \frac{\pi}{4}$$

$$W = W' \cdot L \cdot \alpha$$

where;

W' : Weight of iron pipe per 1 m (t/m)

W : Total weight (t)

Do : Internal diameter (m)

t : Pipe thickness (m)

γ_s : Unit weight by volume of steel material 7.80 t/m^3

L : Extended length of pipeline

α : Premium rate of iron pipe volume due to accessories = 1.14

c)) Calculation results

The pipe thickness and iron pipe weight obtained by the above formula are shown in Table 3-7.

Table 3-7 Pipe Thickness and Iron Pipe Weight

Section (m)		0 ~11.690	11.690 ~23.745	23.745 ~26.777	26.777 ~38.832	38.832 ~49.782	
Sectional Distance (m)		11.690	12.055	3.032	12.055	10.950	
D=3.0	dia. (m)	3.00	3.0~2.6	2.6	2.6	2.6	
	t (mm)	10	10	10	10	9	
	W' (t/m)	0.738	0.725	0.713	0.701	0.619	
	W (t)	9.835	9.963	2.464	9.634	7.727	ΣW=40
D=4.0	dia. (m)	4.00	4.0~3.4	3.40	3.4~2.6	2.6	
	t (mm)	12	12	11	11	9	
	W' (t/m)	1.180	1.092	0.919	0.839	0.619	
	W (t)	15.725	15.007	3.177	11.530	7.727	ΣW=53
D=5.0	dia. (m)	5.0	5.6~4.4	4.40	4.4~2.6	2.60	
	t (mm)	15	15	12	12	9	
	W' (t/m)	1.843	1.641	1.150	0.989	0.619	
	W (t)	24.561	22.552	3.975	13.592	7.727	ΣW=72
D=6.0	dia. (m)	6.00	6.0~4.4	4.4	4.4~2.6	2.80	
	t (mm)	17	17	13	13	9	
	W' (t/m)	2.507	2.173	1.406	1.151	0.619	
	W (t)	33.410	29.863	4.860	15.818	7.727	ΣW=92

ii) Volume of civil work for iron pipeline

In the design section illustrated below, the required volume of concrete for the cases of D being 4 m, 5 m, 6 m and 7 m are shown in Table 3-8, allowing however, 3% extra concrete.

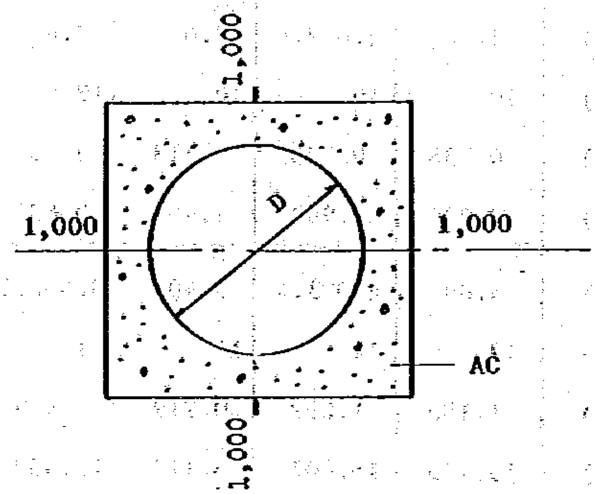


Table 3-8 Volume of Concrete

Internal Diameter (m)	Ac (m^2)	Volume of Concrete (m^3)
3.0	17.931	702
4.0	23.434	917
5.0	29.365	1149
6.0	35.726	1399

iii) Iron pipeline construction cost

The iron pipeline construction cost is shown in Table 3-9.

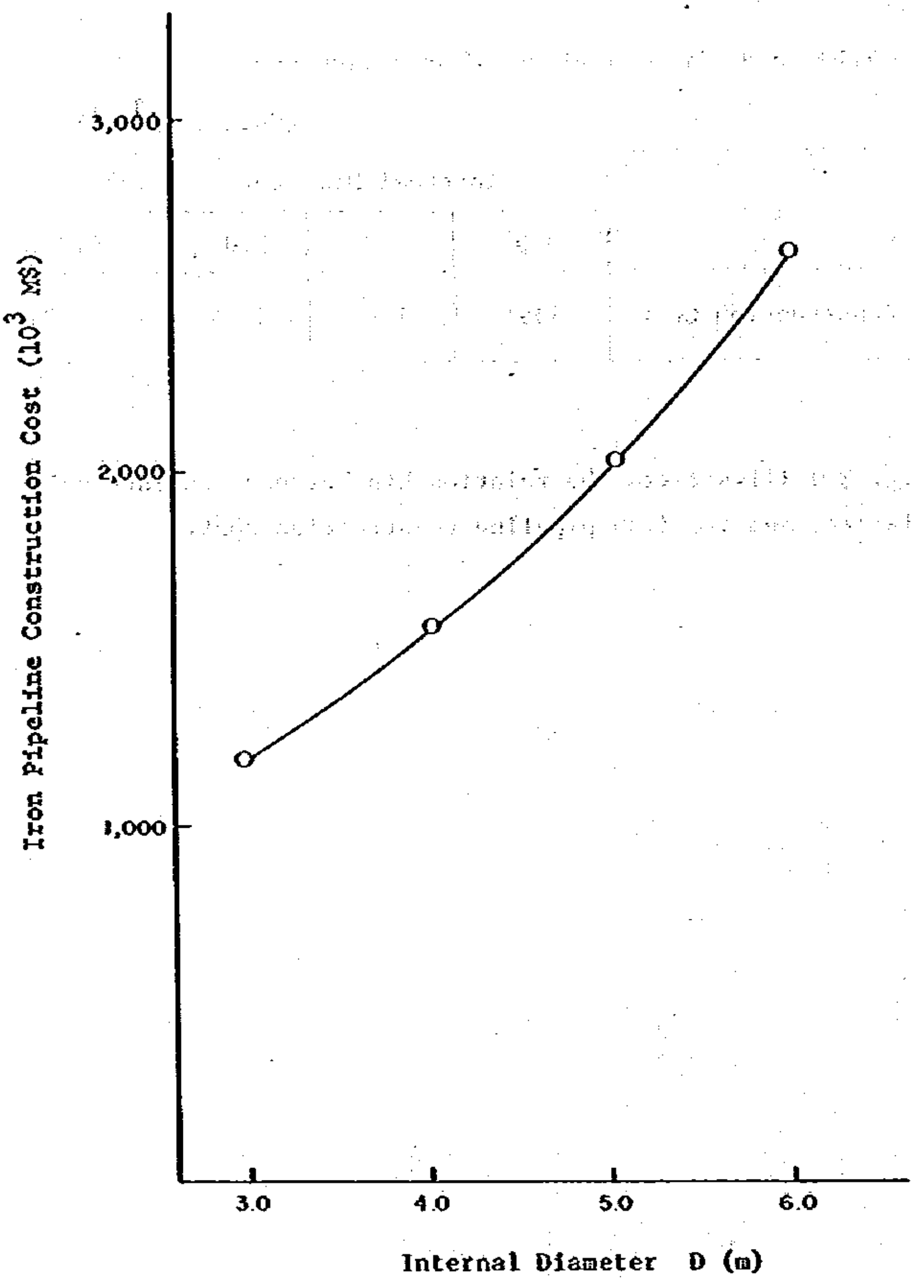
Table 3-9 Iron Pipeline Construction Cost

(Unit : 10^3 M$)

	Internal Diameter D (m)			
	3.0	4.0	5.0	6.0
Construction Cost	1194	1563	2034	2617

Fig. 3-9 illustrates the relationship between the internal diameter and the iron pipeline construction cost.

Fig. 3-9 Relationship between Internal Diameter of Pipe and Construction Cost



iv) Annual costs

Table 3-10 shows the increase and decrease in the iron pipeline construction costs using the internal diameter of 5.0 m as the standard.

Table 3-10 Increase/Decrease in Construction Cost

Internal Diameter (m)	Iron Pipeline Construction Costs	Increase/Decrease in Iron Pipeline Construction Costs
3.0	1,194	-840
4.0	1,563	-471
5.0	2,034	0
6.0	2,617	583

Table 3-11 shows the increase/decrease in annual costs ΔC obtained by the formula described in (3) above.

Table 3-11 Increase/Decrease in Annual Costs

Internal Diameter (m)	ΔC (10^3 M$)
3.0	-117
4.0	-65
5.0	0
6.0	81

3.9 Diversion Work

In designing diversion work, the design flood discharge is assumed to be the probable flood discharge which occur once every 10 years since it is the concrete gravity dam.

For flood run-off from the upstream dam catchment basin, 675 m^3/s was assumed in consideration of the reservoir surcharge, and flood run-off from the remaining catchment basin was assumed to be 200 m^3/s. Accordingly, in executing the construction of the lower dam, the flood control function of the upper dam will be utilized so that the discharge to be covered by the diversion work will be 200 m^3/s, which is the probable flood discharge of once every 10 years.

For the diversion work, a comparative study was made on two plans; a temporary diversion tunnel and a sluice diversion methods, and the sluice diversion method was adopted.

Table 3-12. Comparison of Diversion Tunnel and Sluice Diversion Methods

Item	Diversion Tunnel Method	Sluice Diversion Method	Remarks
Features	Dia. 6 m x Length 295 m	Sheet Piling & Diversion Outlet	
Construction Time	7 months	2 months + 3 months	Two stage Construction
Cost (M$)	7,153,000	5,078,000	

(1) The structure of cofferdam

Generally, the temporary cofferdam for sluice diversion will be as follows;

a) Coffering with earth

b) Coffering with concrete structure

c) Coffering with sheet piling work (when silted deposition is present)

d) Other special coffering methods (when silted deposition is present). The appropriate method is selected after comparative study of the conditions of the river.

Table 3-13 Comparison of Coffering Method According to the Conditions of River

Condition / Method	Coffering Scope	Cut-Off	Difficulty of Work	Construction Time	Cost
Coffering with earth	As river-bed width is small, improper. Acceptable only for downstream secondary coffering X	Can be expected O	None O	Short construction period ◎	Not expensive, but damage by flood heavy. O
Coffering with concrete	Possible, though very limited allowance when width of water way is considered. Δ	Excellent ◎	Utmost care is needed to demolish concrete placed in advance on the right bank. Δ	Relatively long as demolition of concrete takes time. X	High cost Δ
Coffering with sheet pile	Conditions favorable than for concrete. ◎	Inside filling earth, etc. must be considered ◎	Depth of silted deposition, grain size, etc. are considered. Δ	Short construction period possible. ◎	High cost Δ

The comparison table indicates that coffering with sheet pile is advisable since the construction period of the lower dam is limited as it must be built by utilizing the flood control function of the upper dam.

The construction method for coffering with sheet piling work (double wall steel sheet piling method, with inner binding work between sheet piles) is studied below.

(2) Design discharge

For reasons of safety, the probable flood discharge which occurs once every 10 years was adopted for designing. The remaining catchment basin for the lower dam (C.A. = 180 km^2) after filling the upper dam with water is converted into discharge as:

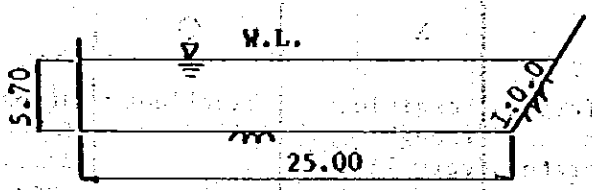
$$\frac{1,400 \times 180}{1,380} = 182 \doteqdot 200 \ m^3/s$$

(3) Calculation of discharge through the temporary diversion conduit

$Q = 200 \ m^3/sec$

$I = 1/1000$

$n = 0.04$



On discharge calculation,

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

Table 3-14 Calculation of Discharge Volume

Water Depth h (m)	P (m)	Sectional Area A (m^2)	Wetted Perimeter S (m)	Hydraulic Radius R (m)	Hydraulic Gradient I	Flow Velocity V m/sec	Discharge Volume Q m^3/sec
1	25.6	25.3	27.2	0.93	1/1000	0.75	19.1
2	26.2	51.2	29.3	1.75	"	1.15	58.8
3	26.8	77.7	31.5	2.47	"	1.44	112.2
3.5	27.1	91.2	32.0	2.80	"	1.57	143.2
4	27.4	104.8	33.7	3.11	"	1.68	176.5
4.5	27.7	118.6	34.8	3.41	"	1.79	212.4
5	28.0	132.5	35.8	3.70	"	1.89	250.6

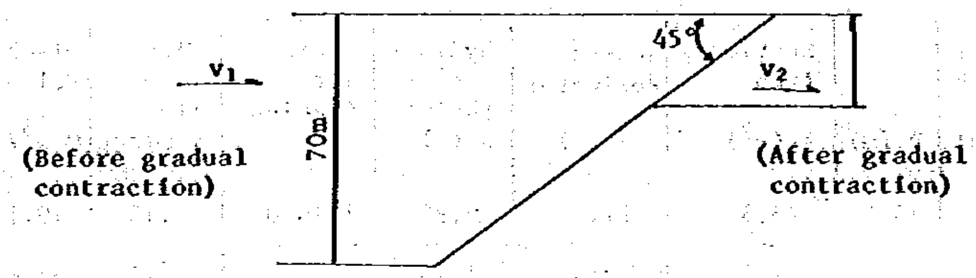
(4) Determination of the section of the waterway

From the assumed section used in the foregoing discharge volume calculation, the section was calculated to have a bottom width of 25 m, a water depth of 4.5 m and a face of slope gradient of 0.6 (cut face).

As for the height of the cofferdam, the planned water depth at the training work and discharging work was assumed to be 4.5 m, and 0.5 m extra height was allowed.

The portion to compensate for the head loss accompanying gradual contraction of the water way width in the upstream was obtained according to the following formula:

$$\Delta h_e = f_e \cdot \frac{V_2^2}{2g} + \left(\frac{V_2^2}{2g} - \frac{V_1^2}{2g}\right)$$



Normally, it is deemed that $v_1 \doteq 0$, so that from

$$h_e = (1 + f_e) \cdot \frac{V_2^2}{2g}$$

and assuming $f_e = 0.5$,

$$h_e = (1 + 0.5) \times \frac{(1.40)^2}{2 \times 9.8} = 0.15 \text{ (m)}$$

Therefore, the height of cofferdam is:

$$4.5 \text{ m} + 0.5 \text{ m} + 0.15 \text{ m} = 5.15 \text{ m}$$

Accordingly, EL 55.0 + 5.15 = 60.15.
However, in view of study results on the bypass within day of the secondary cofferdam which will be described later, H.W.L. was made EL 60.7 m.

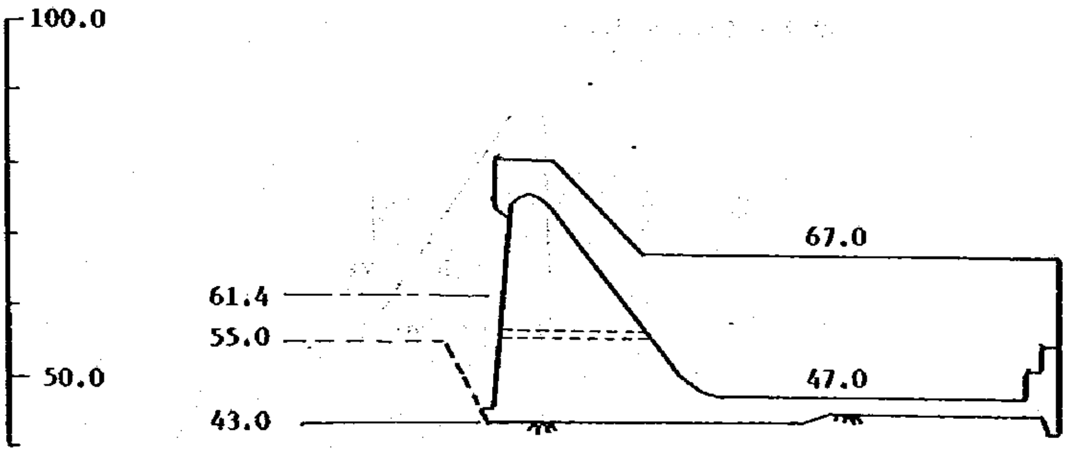
(5) Review of the temporary diversion conduit within dam

It is desirable to make the water head larger and the section smaller in deciding the section of the temporary diversion conduit within the dam. Also, when a plural number of temporary diversion conduits within the dam are to be built, it is desirable to provide a difference of elevation in the height of their beds to facilitate closure.

As for its section, the conventional semi-circular form for the upper part and the rectangular form for the lower part are satisfactory.

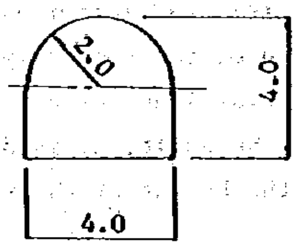
Its section and the position to be installed are reviewed in the following.

The overflow section is as illustrated below.



As the elevation of the river bed is approximately 55.0 m at present, the height of bed of temporary diversion conduits within the dam is made EL. 55.5 m so as to prevent entry of sediment. The channel gradient is made level to facilitate work.

The section is as illustrated below.



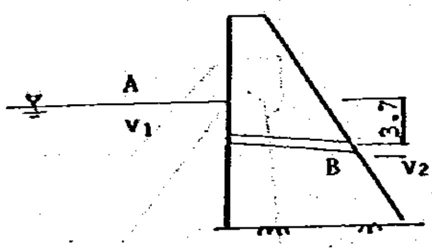
A=14.28 m^2

When the levee crown of the secondary cofferdam is aligned to the levee crown of the primary cofferdam, the maximum water level will be:

$$55 + 5.7 = 60.7 \text{ m}$$

As the center height at the outlet of temporary diversion conduit is 57.0 m, the difference of water head will be:

$$60.7 - 57.0 = 3.7 \text{ m}$$



Bernoulli's theorem is applied between Point A and Point B.

$$\alpha_1 \cdot \frac{V_1^2}{2g} + \frac{P_1}{W} + 3.7 = \alpha_2 \cdot \frac{V_2^2}{2g} + \frac{P_2}{W} + \text{loss}$$

Since it may be deemed as $V_1 = 0$, and also assuming that $P_1 = P_2$,

$$3.7 = 1.1 \times \frac{V_2^2}{2g} + \text{loss}$$

If we consider that loss portion consists of inflow loss and friction loss,

Inflow loss $\quad h_e = f_e \cdot \frac{V_2^2}{2g}$

$$= 0.2 \cdot \frac{V_2^2}{2g}$$

Friction loss $\quad h_f = f \cdot \frac{V_2^2}{2g}$

$$f = \frac{2g\,n^2}{R^{1/3}} = \frac{2 \times 9.8 \times (0.013)^2}{1^{1/3}}$$

$$= 0.0033$$

$$\therefore \; h_f = 0.0033 \, \frac{V_2^2}{2g}$$

From the above

$$3.7 = 1.1 \times \frac{V_2^2}{2g} + (0.2 + 0.0033) \, \frac{V_2^2}{2g}$$

$$V_2 = 7.5 \text{ m}$$

$$\therefore \; Q = 14.28 \times 7.5 = 107.1 \text{ m}^3$$

Accordingly, it is necessary to provide another temporary diversion conduit within the dam.

When this conduit is installed at an elevation 0.8 m higher than the other, as in the foregoing,

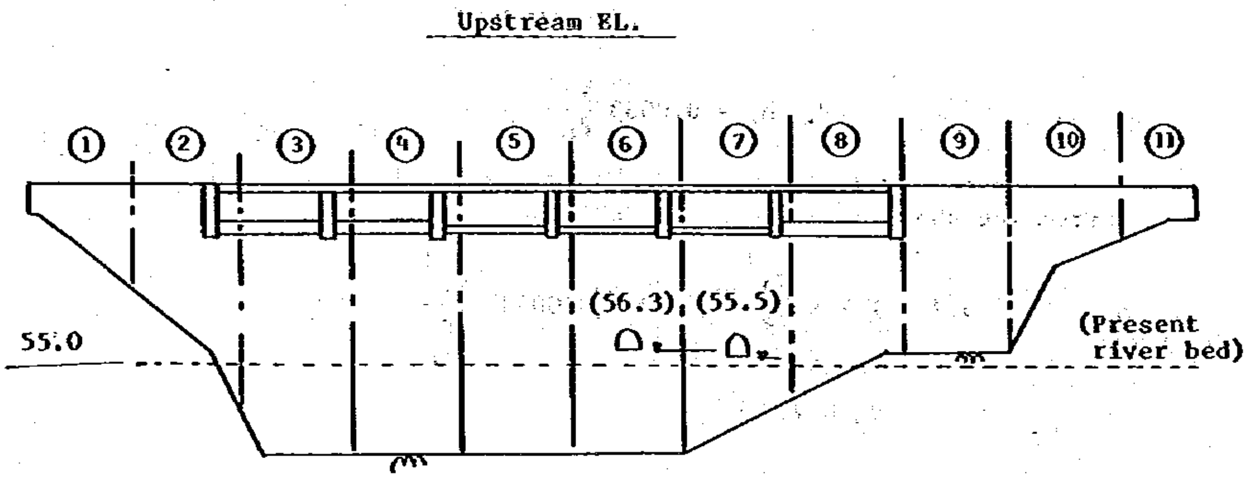
$$2.9 = 1.1 \times \frac{V_2^2}{2g} + (0.2 + 0.0033)\frac{V_2^2}{2g}$$

$$V_2' = 6.6 \text{ m/s}$$

$$Q' = 14.28 \times 6.6 = 94.28$$

$$Q + Q' = 107.1 + 94.28 = 201.38 > 200 \ (m^3/s)$$

Based on the above, temporary diversion conduits within the dam will be arranged as illustrated below.



4. CONSTRUCTION PLAN

4. Construction Plan

4.1 Construction Planning

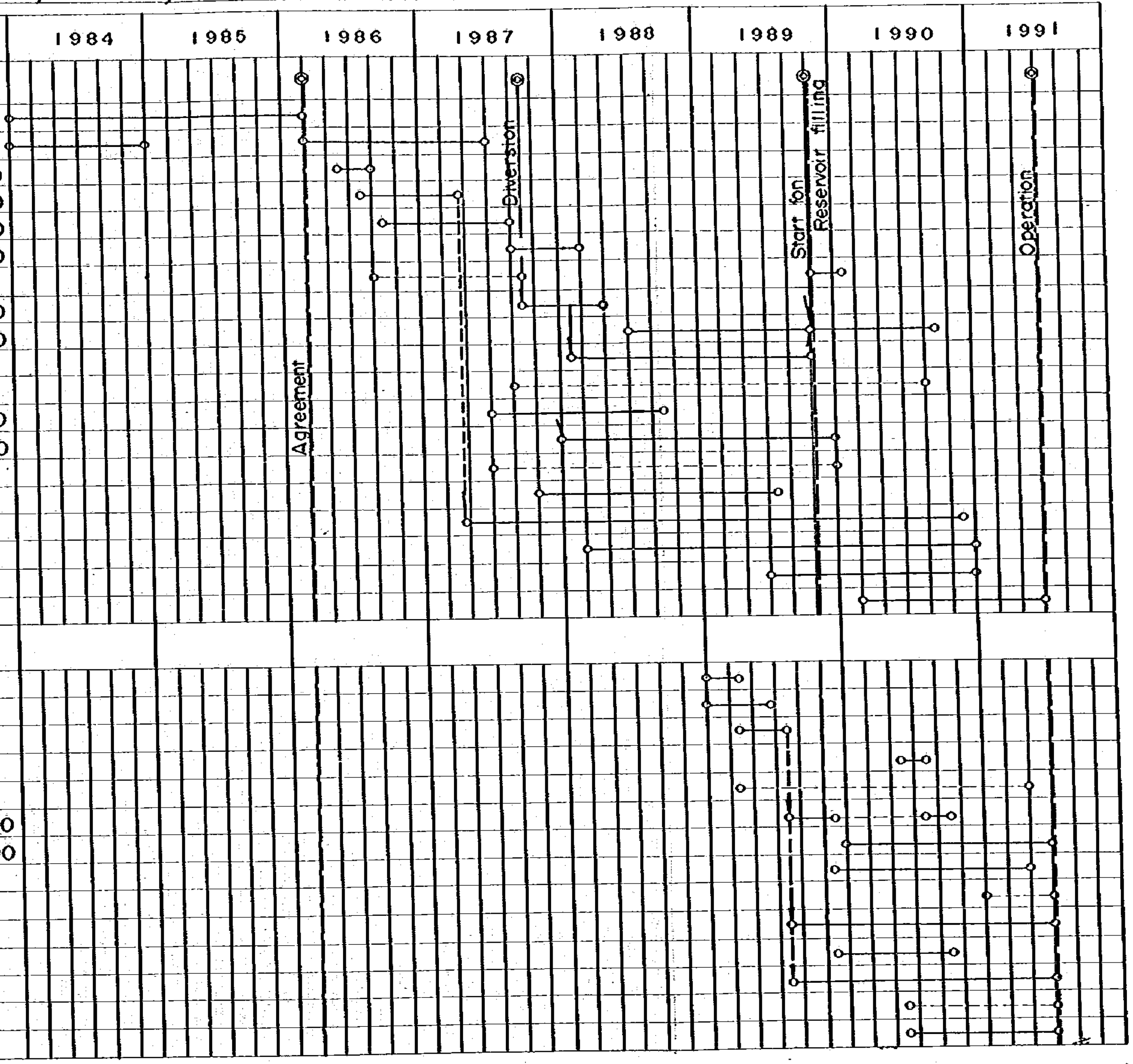
Construction shedule is shown in Fig. 4-1. Work quantities of Upper and Lower Tekai Sites are shown in Fig. 4-2 and Fig. 4-10.

Table 4-1 Description of Projects

Item	Unit	Upper Tekai	Lower Tekai
Type	-	Rockfill dam with imperious centre core	Concrete gravity dam
Dam Height	m	101	38
Crest Length	m	350	160
Dam Volume	m^3	3,125,000	56,900
Effective Depth	m	10	4.5
Installed Capacity	MW	150	5.8

Fig. 4-1 Construction Schedule, Tekai Hydro-electric Power Development Project

Description of Works		Unit	Quantity
(Upper Tekai)			
Preparation	Access Road	L.S	1
	Temporary-Facilities	L.S	1
Diversion	Excavation	m³	57,700
	Tunnel Excavation	m³	56,000
	Lining Concrete	m³	16,500
	Coffer Dam	m³	357,900
	Others	L.S	1
Dam	Excavation	m³	442,000
	Embankment	m³	2,795,000
	Grouting	L.S	1
	Others	L.S	1
Spillway	Excavation	m³	283,000
	Concrete	m³	48,450
	Others	L.S	1
Other Structures	Intake	L.S	1
	Penstock	L.S	1
	Power House, Switch Yard	L.S	1
	Others	L.S	1
Generating Equipment		L.S	1
(Lower Tekai)			
Preparation	Access Road	L.S	1
	Temporary Facilities	L.S	1
Sluice Diversion	(1st)	L.S	1
	(2nd)	L.S	1
	Others	L.S	1
Dam	Excavation	m³	42,300
	Concrete	m³	56,900
	Grouting	L.S	1
	Others	L.S	1
Other Structures	Intake	L.S	1
	Penstock	L.S	1
	Power House, Switch Yard	L.S	1
	Others	L.S	1
Generating Equipment		L.S	1



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CONSTRUCTION SCHEDULE

FIGURE 4-1

4.2 Upper Tekai Site

4.2.1 Construction Schedule

Construction Schedule of Upper Tekai is shown in Fig. 4-2.

Fig.

Description of Works		Unit	Quantity
Preparation	Temporary - Road	L. S	1
	Temporary - Facilities	L. S	1
Diversion	Open Excavation	m^3	57.700
	Tunnel Excavation	m^3	56.000
	Linning Concrete	m^3	16.500
	Outlet Structuve	m	29
	Access Tunnel	m	246
	Others	L. S	1
	Coffer Dam (1st)	m^3	27.500
	Coffer Dam (2nd)	m^3	330.400
Dam Body	Excavation	m^3	442.000
	Embankment	m^3	2.795.000
	Curtain Grouting	t	3.300
	Others	L. S	1
Spillway	Excavation	m^3	283.000
	Concrete	m^3	48.450
	Others	L. S	1
Intake	Excavation	m^3	33.000
	Concrete	m^3	9.045
	Others	L. S	1
Penstock	Access Tunnel	m	150
	Tunnel Excavation	m^3	39.300
	Penstock	t	3.000
	Concrete	m^3	14.300
	Grouting	L. S	1
	Others	L. S	1
Power House	Excavation	m^3	83.500
	Concrete	m^3	30.000
	Super Structutuve	L. S	1
	Generating Equipment	L. S	1
	Switch Yard	L. S	1
	Others	L. S	1

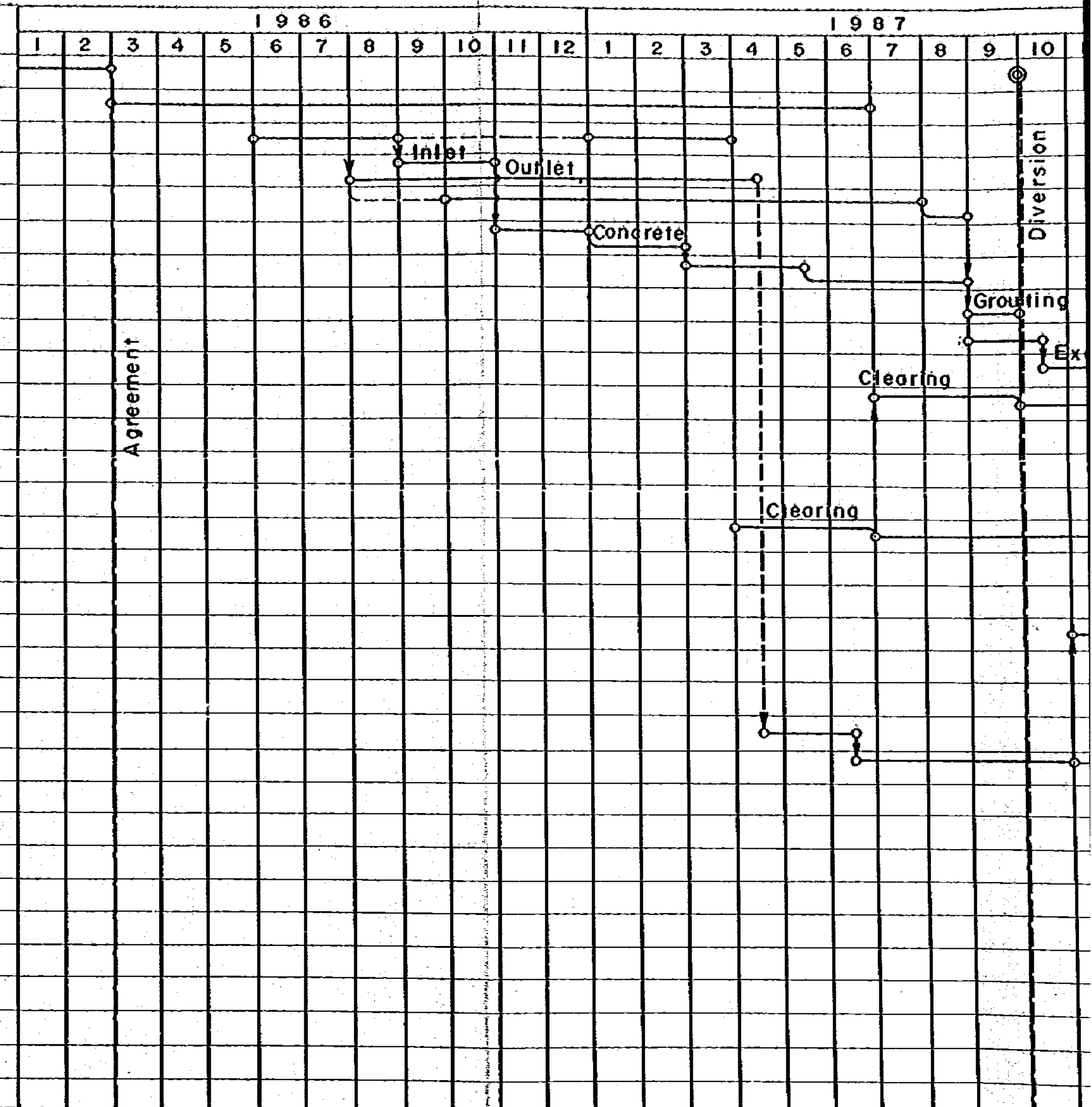
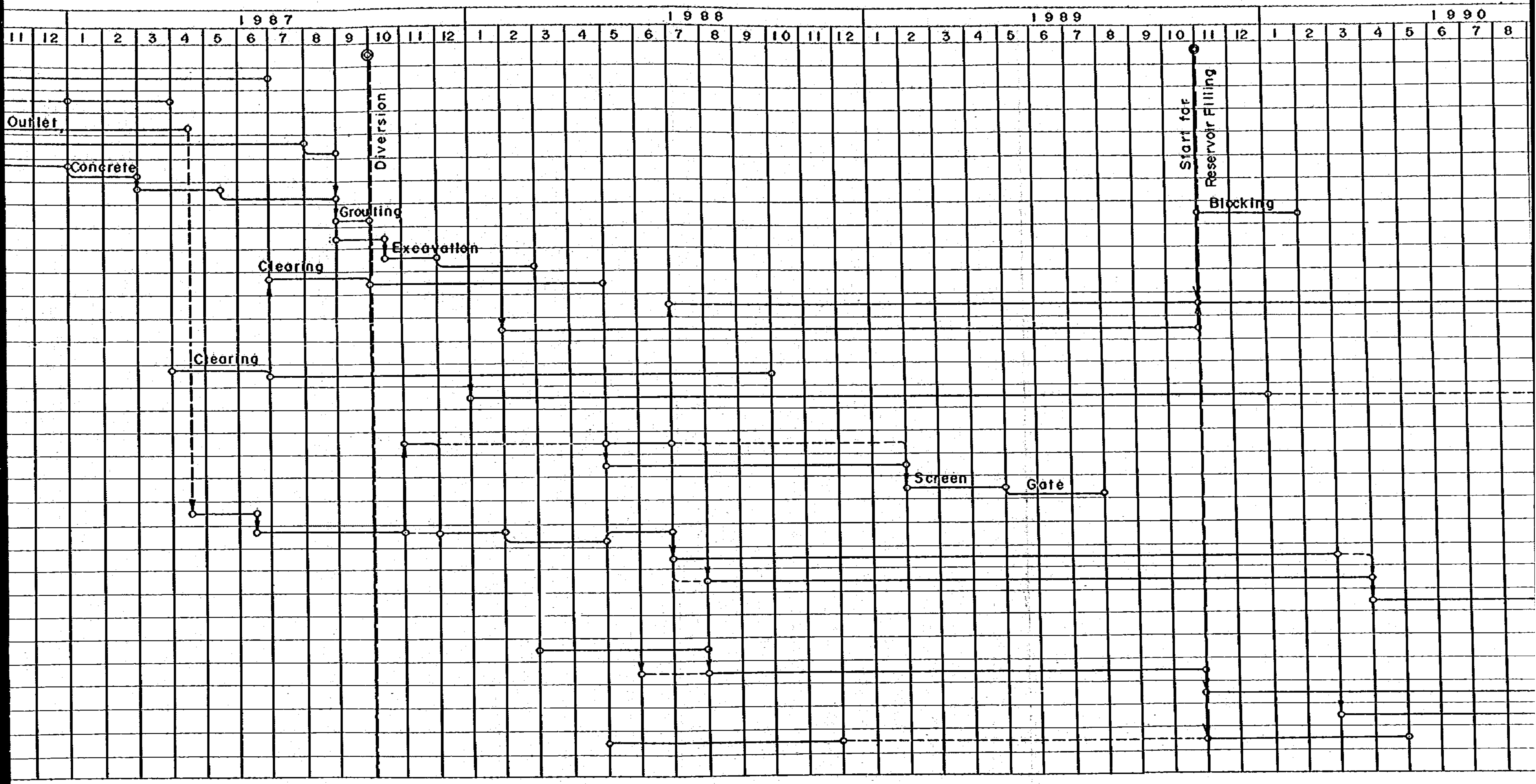
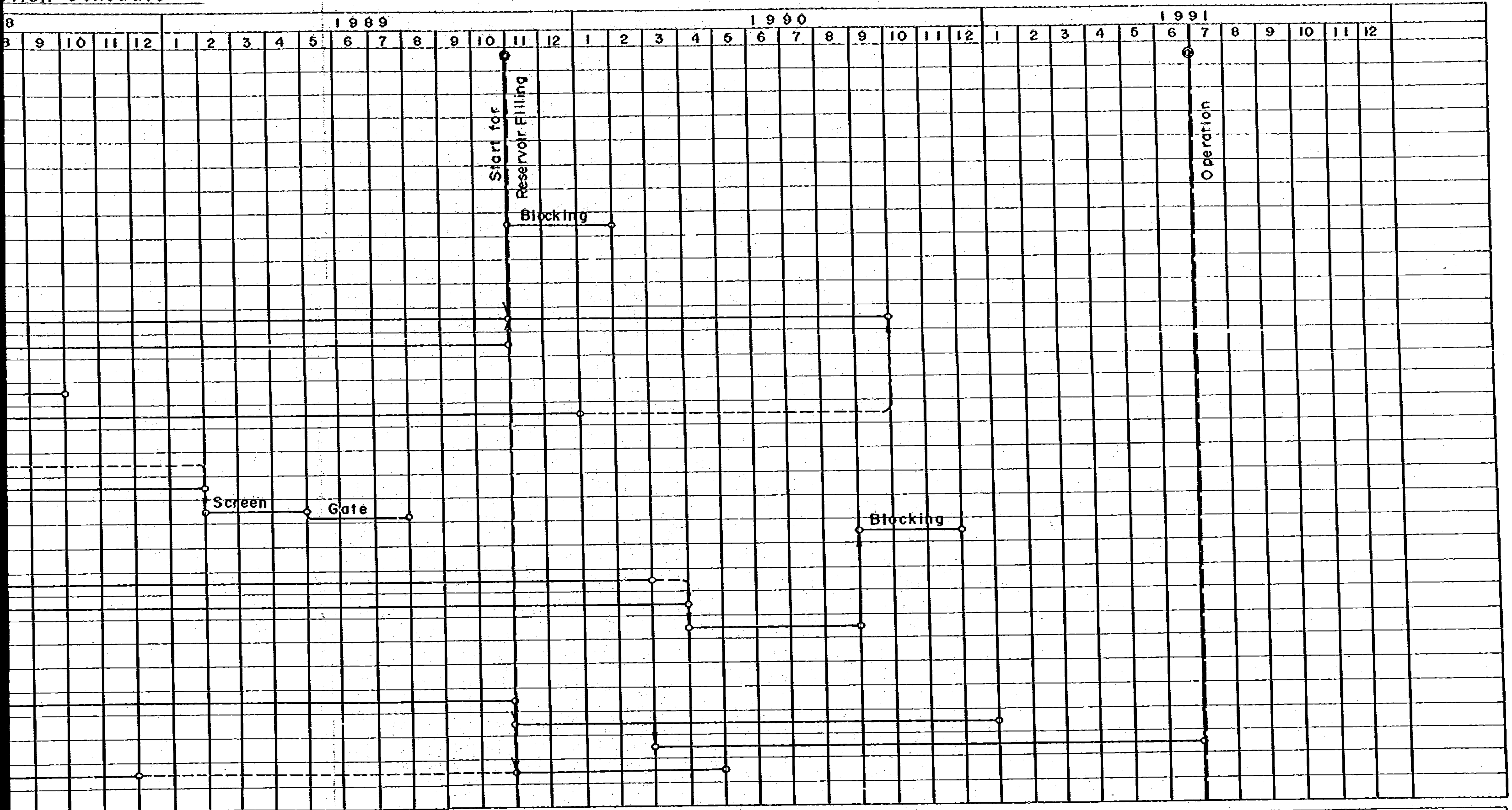


Fig. 4-2 Upper Tekai Costruction Schedule



ction Schedule



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FEASIBILITY STUDY OF TEKAI HYDRO-ELECTRIC POWER DEVELOPMENT PROJECT

UPPER TEKAI
CONSTRUCTION SCHEDULE

FIGURE 4-2

4.2.2 Diversión Works

(1) Description of Excavation

Excavation will be made according to the upper-half-facing cutting type by short bench.

Lining concrete will be placed parallelly for a distance of 150m backward of the facing. (Steel form: L = 6.00m).

At the plug portion, the careful lining will be made at the time of excavation and plug works will be minimized.

(2) Construction Method

Excavation will advance from both sides (inlet and outlet sides) and then will proceed to the shaft after construction of 100m section from the inlet.

i) Type of tunnel excavation machine

o Inlet side	Pneumatic 2-boom jumbo Tractor shovel (1.7 m^3)
o Outlet side	Hydraulic 2-boom jumbo Wheel loader (2.1 m^3) Dump truck (11t), 3 units

ii) Concrete lining

Steel forms of 6m in length will be used to place lining concrete sequentially from the outlet site.

Type of principal equipment:

o Steel form	L = 6.0m, 1 unit
o Vehicle-mounted concrete pump	60 m^3/h (Boom vehicle -- Used also for open placement)
o Truck mixer	6 m^3 x 3

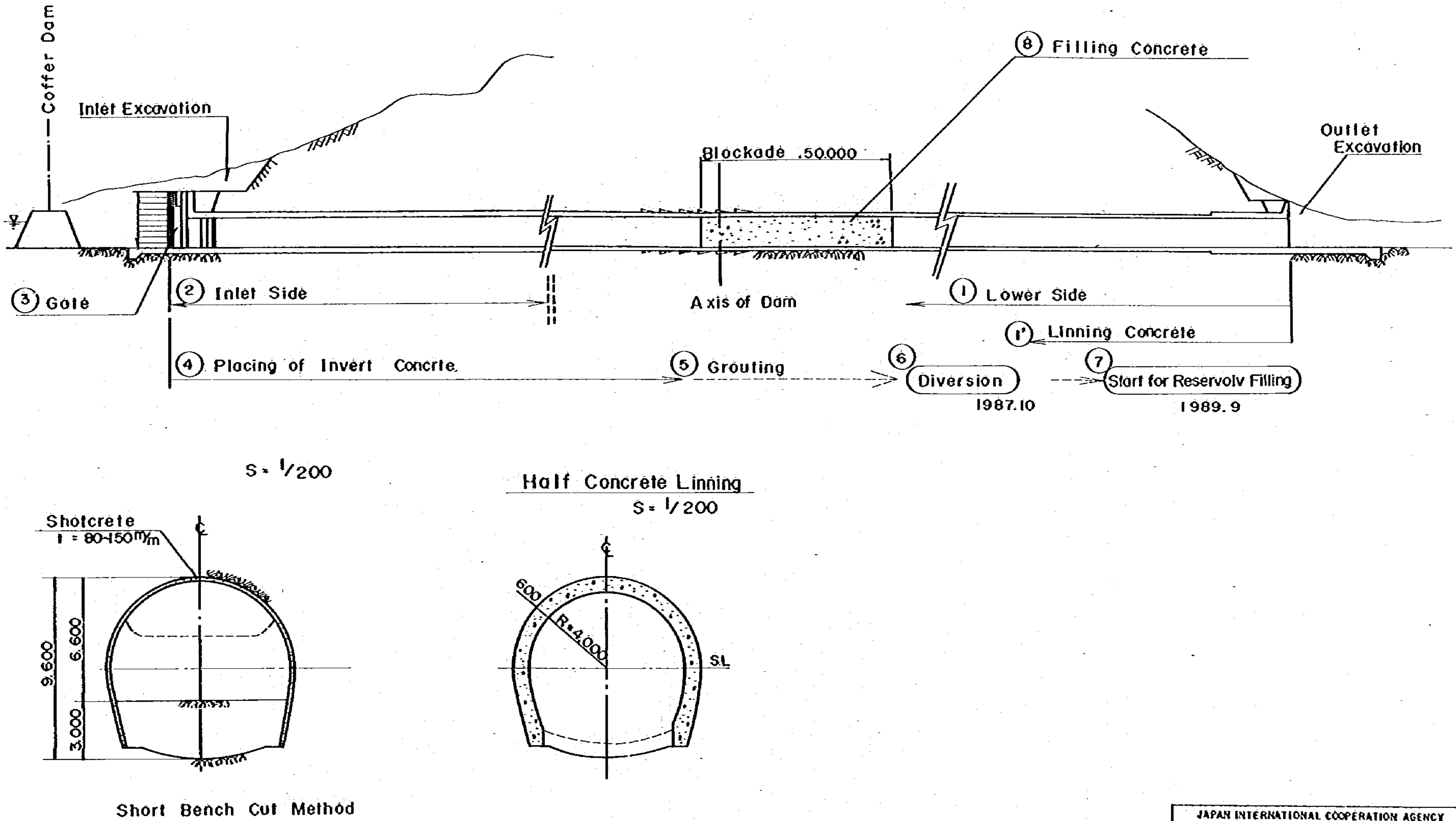
(3) Supporting

Principal supporting will be shotcrete in thickness of 8.0 - 15.0m. Depending on the geological conditions, supporting of rock bolts (length = 2.0 - 3.0m), wire net, steel timbering (H = 100), etc. will be added.

(4) Construction Flowchart

The construction flowchart is shown in Fig. 4-3.

Fig 4.3 Diversion tunnel Construction Planning



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FEASIBILITY STUDY OF TEKAI HYDRO-ELECTRIC POWER DEVELOPMENT PROJECT

UPPER TEKAI
Diversion Tunnel Construction Planning

FIGURE 4-3

4.2.3 Dam

(1) Embankment Material

a. Description of material

i) Core material

Core material will be taken from two sites on the right bank about 1 km downstream of the dam site. Since cohesive soil appropriate to core material is expected to be obtained at the borrow site, no particular core screening plant is planned.

A temporary core stock area will be provided near the upper borrow site (Site B-1).

ii) Filter material

Weathered rocks under subsurface will be used as the filter. Fine and coarse materials will be divided.

iii) Rock material

Rock material will be taken from the mountain side of the right bank (Site B-1) downstream of the dam site. Material will be taken by bench cut.

b. Quantity of Dam

Table 4-2 Quantity of Dam

	Embankment quantity	Quarry quantity	Change rate	Yield rate
Core	534,000	897,000	0.85	70%
Filter	255,000	228,000	1.40	80%
Rock	1,927,000	1,720,000	1.40	80%
Riprap	79,000	71,000	1.40	80%
TOTAL	2,795,000	2,916,000		

Table 4-3 Quantity of Coffer Dam

	Embankment quantity	Quarry quantity	Change rate	Yield rate
Core	70,300	119,000	0.85	70%
Filter	25,300	23,000	1.40	80%
Riprap	234,800	210,000	1.40	80%
TOTAL	330,400	352,000		

Table 4-4 Total Quantity

	Embankment quantity	Quarry quantity	Change rate	Yield rate
Core	604,300	1,016,000	0.85	70%
Filter	280,300	251,000	1.40	80%
Rock	2,161,800	1,930,000	1.40	80%
Riprap	79,000	71,000	1.40	80%
TOTAL	3,125,400	3,268,000		

c) Quantity of Borrow and Quarry

i) Core

Core material will be taken from Site B-1 on the mountain side of the right bank downstream of the Upper Tekai and from Site B-2 on the river side of the right bank downstream of the Upper Tekai.

B - 1	455,000 m^3	
B - 2	567,000 m^3	
Total	1,022,000 m^3	$>$ 1,016,000 m^3

ii) Filter and Rock material

o Filter material

Filter material (251,000 m^3) taken from the core site. An approximate ten-day quantity of material will be stored temporarily and the remaining material will be transported directly from the plant according to the embankment speed.

o Rock material

Rock material, riprap material and concrete aggregate will be taken from Site B-1.

Quantity: 2,200,000 m^3.

d) Concrete Aggregate

Table 4-5 Quantity of Concrete Aggregate

Item	Concrete Volume	Change rate	Yield rate	Quarry quantity	Remarks
Upper Tekai	131,100	1.65	60%	133,000	
Lower Tekai	89,900	1.65	60%	90,800	
TOTAL	221,000 m^3			223,800 m^3	

(2) Transportation Plan

The following roads are planned for transportation of rock, filter and core materials.

Table 4-6 Transportation Road

Name of road	Width (m)	Total length (m)	Remarks
Access to quarry site	15.0	(1,150) 400	
Access to upper core quarry	15.0	340	
Access to spoil area and right bank EL = 75	15.0	(200)	
Access to right bank EL = 105	15.0	700	
Access to right bank crest EL = 135	8.0	630	
Access to right bank dam crest	8.0	400	

Name of road	Width (m)	Total length (m)	Remarks
Access to intake	6.0	470	
Spillway crest on left bank	6.0	1,240	
Top of diversion tunnel EL = 90	6.0	740	
Access to left bank	6.0 8.0	(250) (190)	
Access to lower core quarry	15.0	(650)	
Access to bottom of diversion tunnel	8.0	700	
Access to second coffering	8.0	500	
Access to dam downstream level	8.0	1,200	
TOTAL		(2,440m) 7,320m	

* (   )! Traffic possible by March, 1986

(3) Embankment Plan

a. Basic Schedule

Construction schedule was made based on the following conditions.

o Dam excavation after diversion

o Reservoir filling

o Actual operation days a year

Fig. 4-4

Basic schedule is shown as follows;

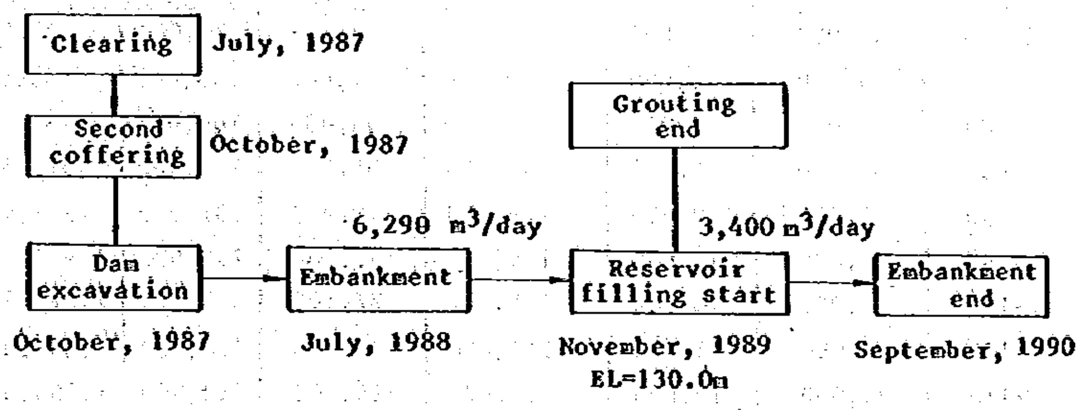


Table 4-7 Embankment volume at each stage

Elevation EL	Core m^3	Filter m^3	Rock, Riprap m^3	2nd Coffering m^3	Total m^3	Grand Total m^3
65-70	16,000	4,000	-	-	20,000	20,000
-75	25,000	6,000	47,000	71,000	149,000	169,000
-80	28,000	7,000	67,000	64,000	166,000	335,000
-85	31,000	8,000	90,000	51,500	180,500	515,500
-90	31,000	9,000	109,000	42,900	191,900	707,400
-95	34,000	10,000	129,000	36,600	209,600	917,000
-100	34,000	10,000	138,000	27,500	209,500	1,126,500
-105	34,000	11,000	149,000	20,800	214,800	1,341,300
-110	34,000	12,000	153,000	14,300	213,300	1,554,600
-115	34,000	12,000	155,000	1,800 (EL.111.5)	202,800	1,757,400
-120	33,000	13,000	153,000	-	199,000	1,956,400
-125	31,000	14,000	146,000	-	191,000	2,147,400
-130	29,000	15,000	135,000	-	179,000	2,326,400
-135	27,000	15,000	124,000	-	166,000	2,492,400
-140	25,000	16,000	108,000	-	149,000	2,641,400
-145	23,000	18,000	80,000	-	121,000	2,762,400
-150	22,000	19,000	59,000	-	100,000	2,862,400
-155	19,000	22,000	41,000	-	82,000	2,944,400
-160	16,000	15,000	31,000	-	62,000	3,006,400
-165	6,000	16,000	12,000	-	34.000	3,040,400
-166.2	2,000	3,000	1,000		6,000	3,046,400
Total	534,000	255,000	1,927,000	330,400	---	3,046,400

Fig. 4-5 Upper Tekai (Dam Body) Embankment Schedule

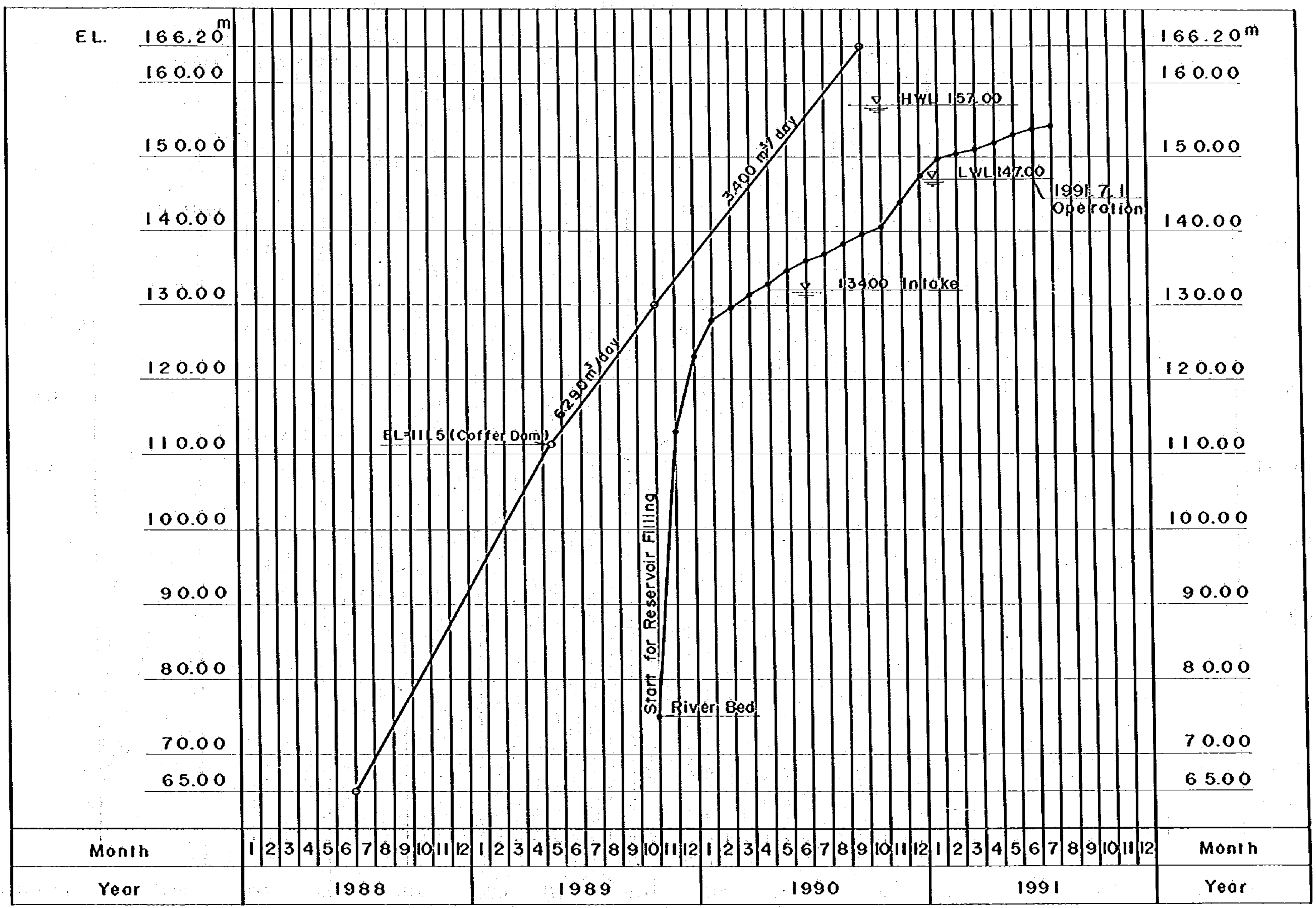


Fig.4-6 Upper Tekai (Dam Body) Cumulation Curve of Embankment Quantity

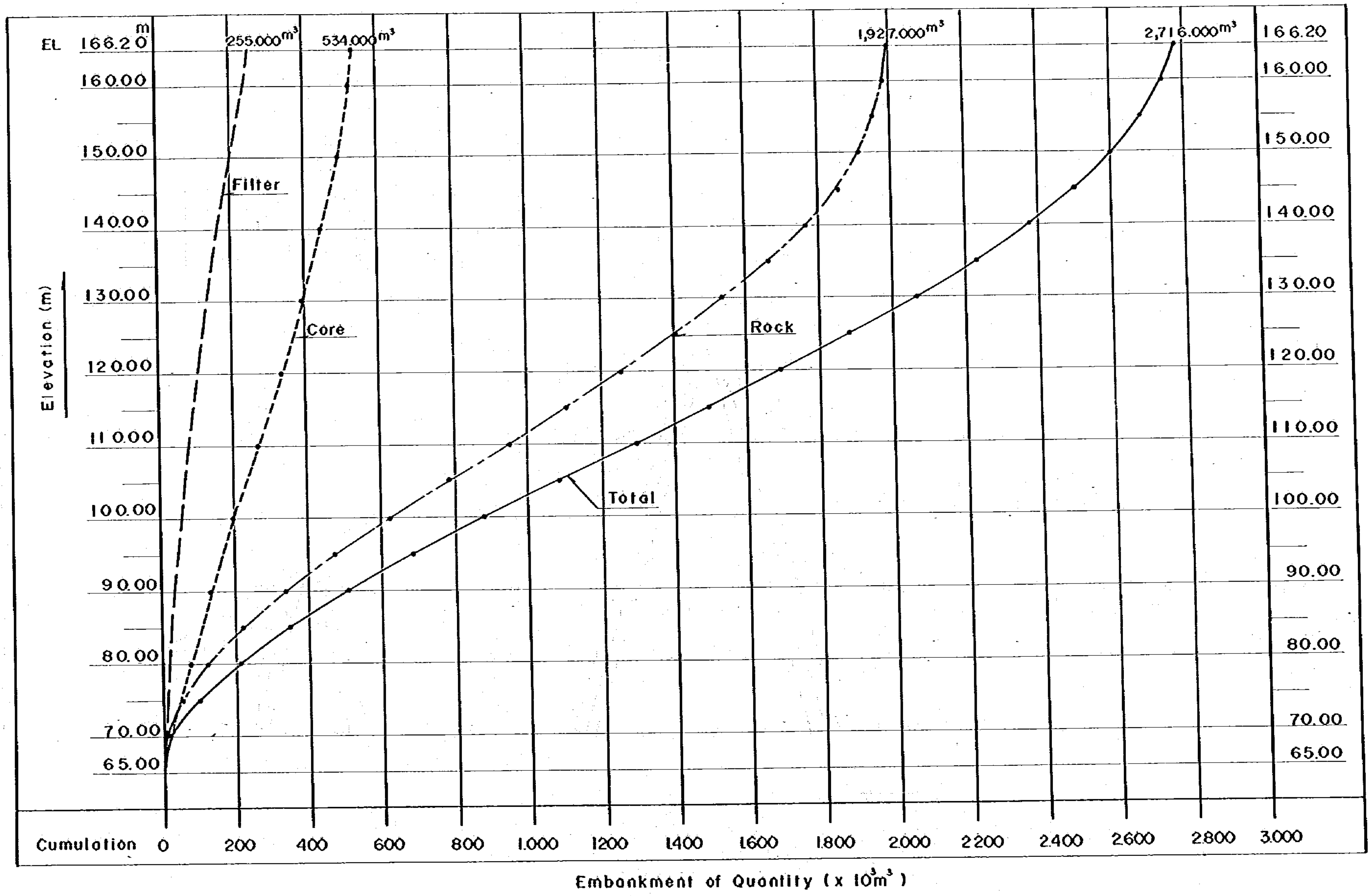
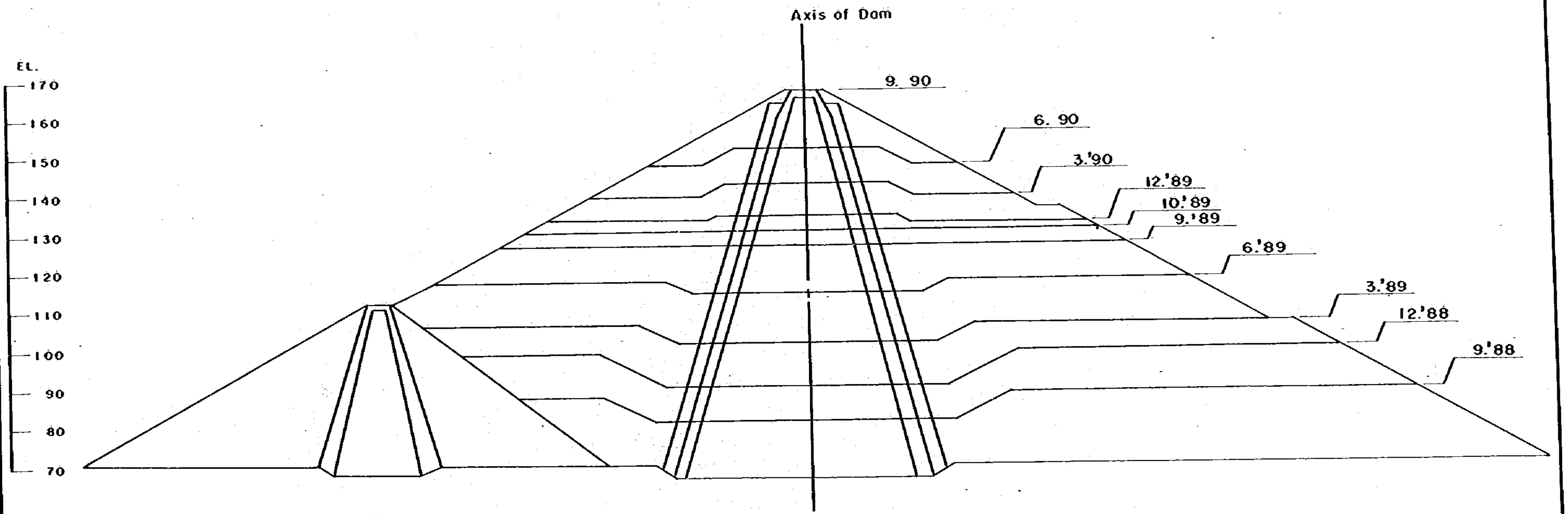


Fig. 4-7 Embankment Schedule

Quantity of Daily Embankment

(Unit m³)

	EL. 70~130	EL. 130~166.2
Core	1.620	820
Filter	540	730
Rock	4.120	1.850
Total	6.290	3.460



b. Construction Days

In estimation of the construction days, the following conditions were taken into consideration.

Since the construction work will continue even in the wet season, no down period will be set.

- o For maintenance — One day/month
- o Day-off — Two days/month
- o Work cessation due to rainfall
  - Core embankment -- Work cessation when rainfall exceeds 2.0 mm/day
  - Shell embankment -- Work cessation when rainfall exceeds 15.0 mm/day
  - Concrete placement -- Work cessation when rainfall exceeds 20 mm/day
- o Special day-off
  - Seven days during Chinese New Year in February
  - Seven days during Hari Raya Puasa in September

The rainfall data listed below is calculated from the correlation coefficient with the Uru Tekai Station, and was determined by using the rainfall data of Kangsar, which are listed in the Table 4-8.

Table 4-8 Work days

(19 years, 1961 - 1979)

		1	2	3	4	5	6	7	8	9	10	11	12
Total number of days		31	28	31	30	31	30	31	31	30	31	30	31
Limit due to rainfall	Core	10	9	9	11	11	8	7	10	11	15	14	16
	Shell	2	2	3	3	4	3	3	4	4	6	6	6
Maintenance period		1	1	1	1	1	1	1	1	1	1	1	1
Holiday		2	2	2	2	2	2	2	2	2	2	2	2
Special off-day		-	7	-	-	-	-	-	-	7	-	-	-
Total of non-working days	Core	13	19	12	14	14	11	10	13	21	18	17	19
	Shell	5	12	6	6	7	6	6	7	14	9	9	9
Total of working days	Core	18	9	19	16	17	19	21	18	9	12	13	12
	Shell	26	16	25	24	24	24	25	24	16	22	21	22

Working days per year

i)) Core ——→ 183 days (Monthly mean 15.3 day/month)

ii)) Shell ——→ 269 days (Monthly mean 22.4 day/month)

** Working days of the earth work is assumed to be equal to working days of the shell embankment.

Table 4-9 Down days of construction for core embankment

Year \ Month	1	2	3	4	5	6	7	8	9	10	11	12
1961	13	6	15	17	11	8	4	7	12	11	15	19
1962	10	8	9	6	8	10	6	14	11	16	19	16
1963	9	12	9	3	10	5	10	4	10	14	15	16
1964	13	20	11	15	11	11	11	6	8	17	15	18
1965	3	8	5	14	12	4	7	12	10	12	17	16
1966	20	12	14	8	7	7	6	13	11	18	10	15
1967	11	11	8	12	13	3	7	7	12	11	22	18
1968	7	0	9	8	7	8	7	10	7	17	6	19
1969	10	6	4	10	14	9	8	15	5	14	8	11
1970	12	6	9	12	11	7	4	6	13	17	14	20
1971	10	6	8	6	12	7	7	17	10	17	10	24
1972	10	9	5	14	10	9	3	11	15	15	11	15
1973	11	6	8	16	13	13	7	15	10	16	13	16
1974	3	12	7	18	11	7	11	10	16	15	18	11
1975	14	10	12	12	12	8	8	9	12	15	19	15
1976	5	3	8	14	9	8	7	12	15	15	13	19
1977	2	15	3	7	10	12	3	12	9	13	16	19
1978	13	8	12	11	13	8	12	5	8	14	14	14
1979	11	5	6	10	7	12	10	7	14	18	17	8
Total	187	163	162	213	201	156	138	192	208	285	272	309
Average	10	9	9	11	11	8	7	10	11	15	14	16

Table 4-10 Down days of construction rock embankment

Year \ Month	1	2	3	4	5	6	7	8	9	10	11	12
1961	4	3	2	7	3	3	3	3	6	8	2	7
1962	6	3	4	2	4	5	1	6	4	2	10	6
1963	0	1	1	1	3	3	2	0	5	4	10	11
1964	3	4	2	2	2	5	3	2	3	7	6	3
1965	0	1	0	4	8	2	2	5	3	8	6	6
1966	3	1	4	4	3	2	3	3	4	7	4	4
1967	5	5	3	3	6	1	2	1	6	7	11	8
1968	0	0	3	4	4	2	2	4	2	3	2	5
1969	2	0	1	1	5	5	4	8	2	7	6	6
1970	4	0	4	5	5	5	2	2	5	9	5	12
1971	3	4	4	1	7	1	3	6	3	4	3	10
1972	1	4	2	3	3	3	0	7	9	8	2	8
1973	2	1	4	9	3	3	3	8	4	6	6	6
1974	0	5	2	4	3	2	4	4	3	4	7	3
1975	2	3	4	2	6	6	5	3	7	4	11	4
1976	1	0	1	6	1	4	0	3	3	5	4	8
1977	1	6	1	2	1	5	1	6	3	5	7	4
1978	5	1	5	2	5	2	4	2	5	3	5	7
1979	2	2	1	2	2	4	6	3	3	10	8	0
Total	44	44	48	64	74	63	50	76	80	111	115	118
Average	2	2	3	3	4	3	3	4	4	6	6	6

c. Construction plan

i) Second coffer dam

The works will be executed at the same time as dam excavation after diversion. Excavation will be completed in 1.5 months and the embankment in two months (December, 1987 - January, 1988). Rock embankment will be made first on the upstream side. And daily works will be as follows;

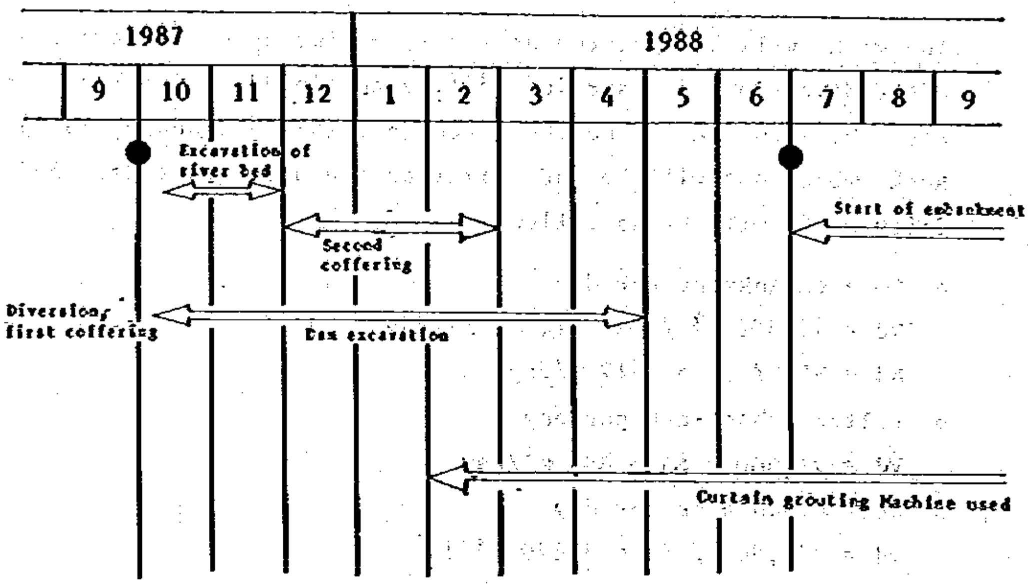
- o Core embankment per day
  $Vd = 70,300\ m^3 / 39 = 1,800\ m^3/day$
  $hd = 35m / 30 = 1.17\ m/day$
- o Filter embankment per day
  $Vd = 25,300 / 64 = 395\ m^3/day$
- o Rock embankment per day
  $Vd = 234,800 / 64 = 3,670\ m^3/day$
- o Total embankment per day
  $Vd = 330,400 / 64 = 5,160\ m^3/day$

ii) Core Embankment of the Main Dam

The embankment work days for core and filter are calculated shown in Table 4-8.

- o Construction period
  The work period for the dam body embankment is planned from the beginning of July 1988 to the end of September 1990 based on the reservoir filling. Start of embankment work is as scheduled in Table 4.11

Table 4-11



o Daily embankment

When embankment reaches EL ≒ 130.00m during the peirod from July 1988 to the end of October 1989;

EL 70.00 - EL 130.00 (July 1988 - Oct. 1989)

$394,000\ m^3/243\ days = 1,620\ m^3/day$

EL 130.00 - EL 166.20 (Nov. 1989 - Sep. 1990)

$140,000\ m^3/171\ days = 820\ m^3/day$

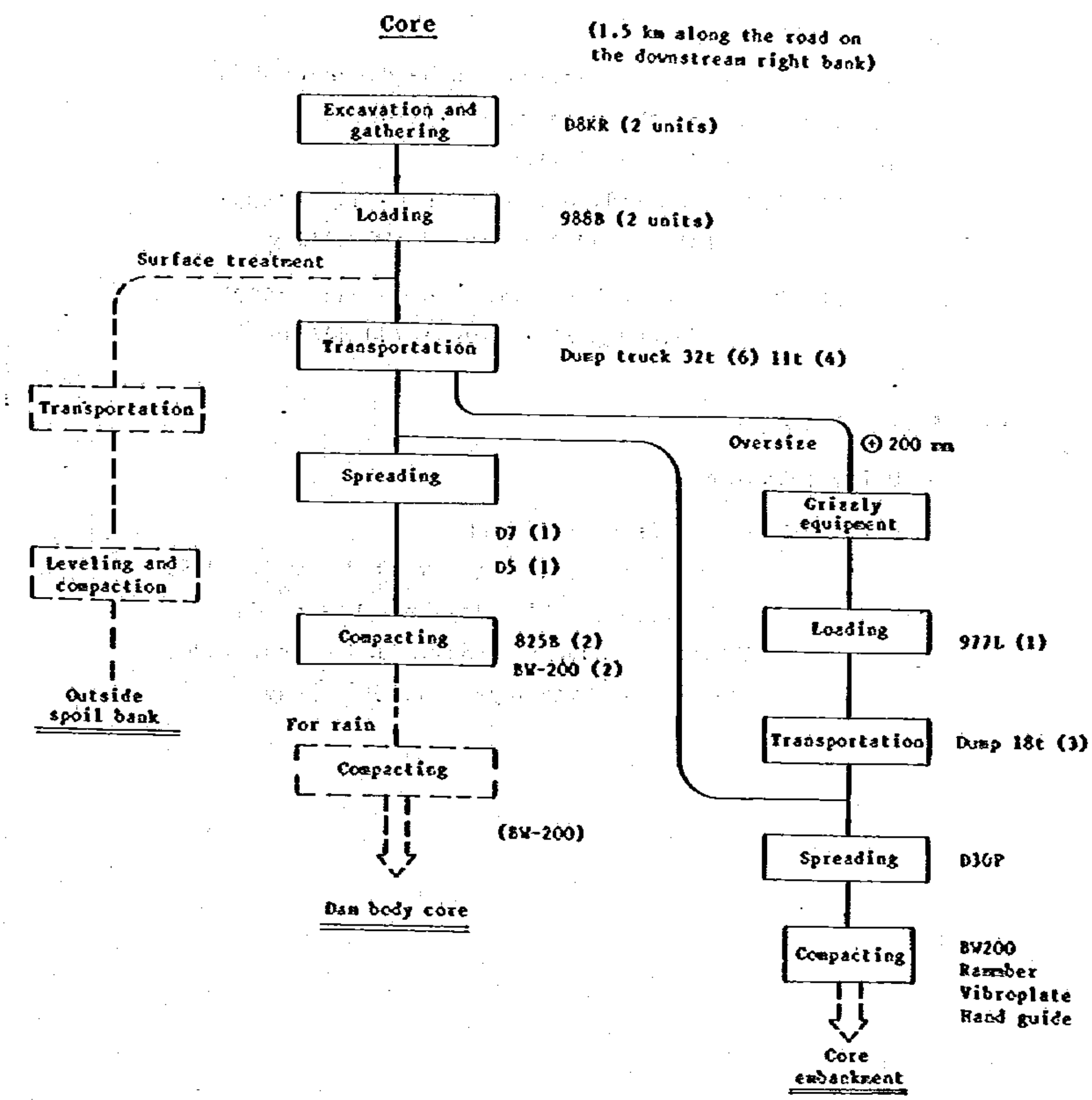


Fig. 4-8 Flow chart of core embankment and machine work

iii) Filter embankment

o Daily embankment

Speed of filter construction can be determined in consideration of the reservoir filling.

The daily speed is as follows;

EL 70.00 - EL 130.00 (July 1988 - Oct. 1989)

131,000 m^3/243 days = 540 m^3/day

EL 130.00 - EL 166.20 (Nov. 1989 - Sep. 1990)

124,000 m^3/171 days = 730 m^3/day

iv) Rock embankment

o Daily embankment

Daily embankment is as follows;

EL = 70.00 - EL 130.00 (July 1988 - Oct. 1989)

1,471,000 m^3/356 days = 4,130 m^3/day

EL = 130.00 - EL 166.20 (Nov. 1989 - Sep. 1990)

456,000 m^3/247 days = 1,850 m^3/day

4.2.4 Foundation Treatment

Blanket grouting will not be carried out.
Curtain grouting will be carried out before the dam embankment.

The quantity is shown in below.

Curtain grouting: 22,100 m

4.2.5 Spillway

(1) Excavation

Excavation quantity is shown in the Table 4-12.

Table 4-12

Work classification	Elevation	Excavation quantity	Remarks
Inlet	EL 151.00	m^3 26,300	Soil: 25% ($70,000\ m^3$) Rock: 75% ($213,000\ m^3$)
Overflow	150.00		
Ramp	85.00	135,300	
Energy dissipator	68.50	121,400	
Total		283,000	

i) Inlet

o Common excavation
This work will begin from the surface soil at the inlet. Then, assuming surface soil thickness to be about 5m, the soil will be pushed toward the downstream side and the river bed by bulldozer.

o Rock excavation
Rock will be ripped by bulldozer and transported to the downstream spoil area.

ii) Inclined

Surface soil and rock will be gathered to the stilling basin, loaded at EL = 85.00m level, and transported to the spoil area.

iii) Stilling basin

Excavated soil from the basin will be entirely disposed (121,400 m^3).

iv) Finish excavation

Excavation of hard rocks and finish excavation will be sequentially carried out downward from the top.
Blasting will be according to the smooth balsting method so as to minimize manual finish excavation work.

(2) Excavation schedule of spillway

Table 4-13 Excavation Schedule

Work classification	Excavation	1987						1988									Quantity
		7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	m^3
Inlet Ramp, Overflow	Soil Soft rock Hard rock																26,300
Chute	Soil Soft rock Hard rock																135,300
Stilling basin	Soil Soft rock Hard rock																121,400

* Excavation of stilling basin will be completed by the end of September 1988. Concrete placement will start in January 1988.

(3) Concrete work of spillway

Tale 4-14 Concrete Quantity

Classi- fication	Total length	Base		Wall	
		Number of placements	Concrete quantity	Number of placements	Concrete quantity
Inlet	Average 30.0	9	1,580	Both sides 50	660
Overflow	30.0	29	6,430	60	1,940
Ramp	170.0	51	5,960	187	5,310
Enegey dissipator	105.0	48	12,470	220	13,320
Pier concrete				6	780
Total	335.0	137	26,440	523	22,010

Spillway body 48,450 m^3

(4) Concrete placement schedule

Table 4-15 Schedule of Base

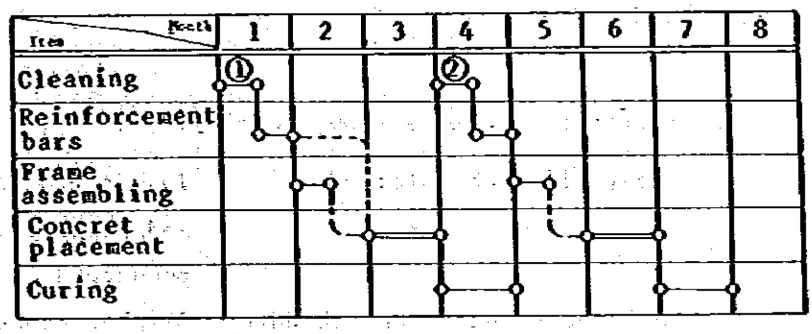
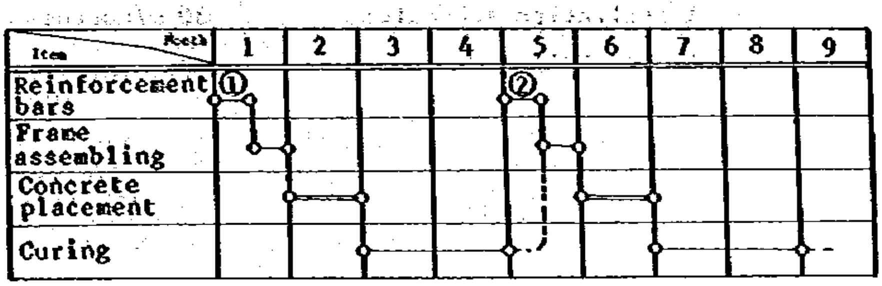


Table 4-16 Schedule of Wall



4.2.6 Penstock

Excavation of pressure tunnel will start after completion of excavation of diversion tunnel. Since the water rises to the intake elevation (EL 130.00) in three months after start of filling, an access tunnel (total length 150m) will be constructed to cope with the limited work period and to prevent interference with power generator equipment.

Basically the work includes (1) Excavation of access tunnel, (2) Excavation of the lower horizontal tunnel (3) Excavation of intake, (4) Excavation of the upper horizontal tunnel, (5) Excavation of inclined tunnel, and (6) Excavation of branch in this order.

After completion of these excavations, the penstock setting and plug concrete placement will be carried out.

Fig. 4-9 shows the flowchart of the work plan.

(1) Excavation

Construction machines are the same as for excavation of the diversion tunnel, and the short bench work method will be employed.

The excavation cycle is shown hereinafter.

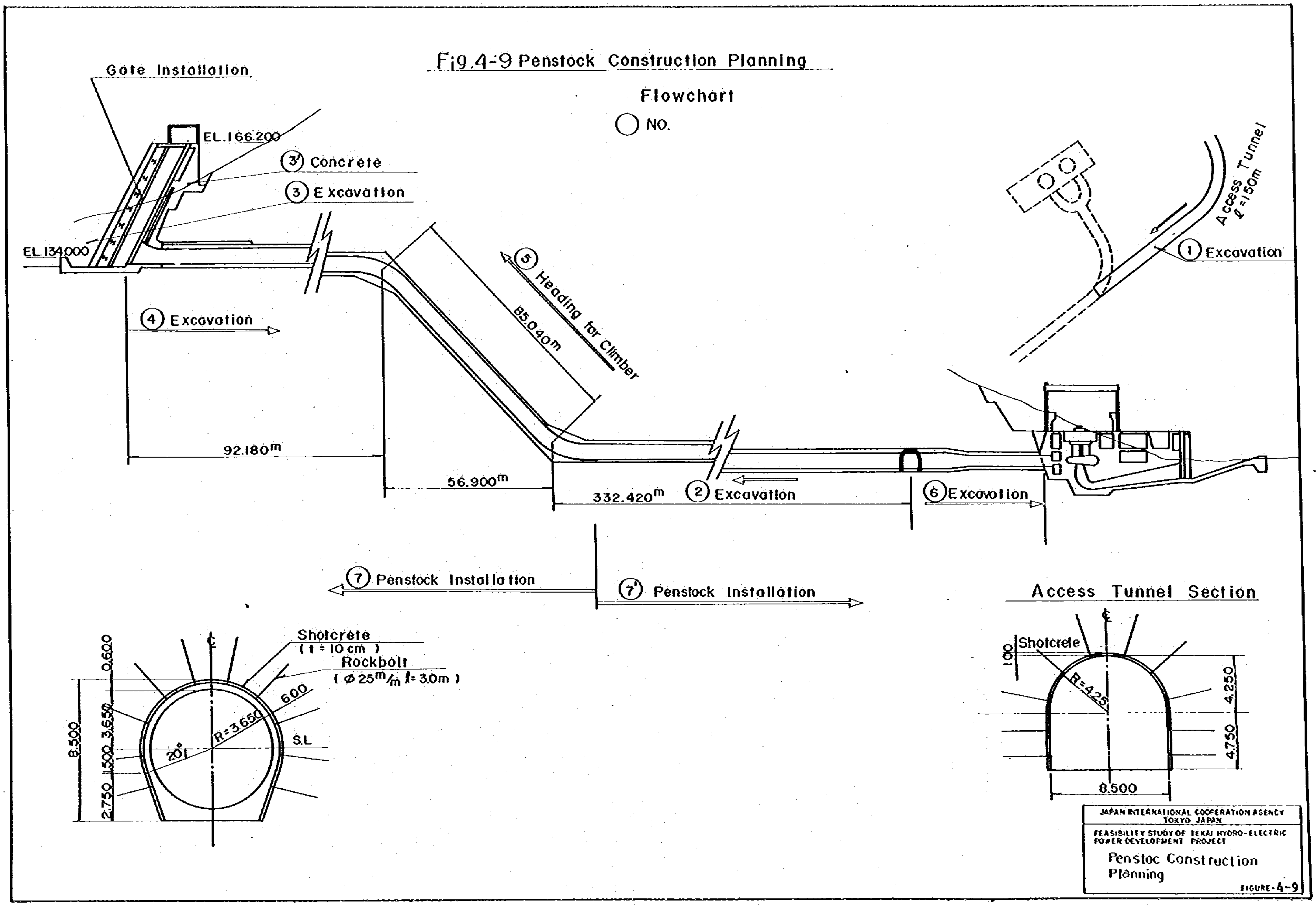
Excavation schedule:	75 m/month (Access tunnel and horizontal run of penstock)
Inclined tunnel work schedule:	Climber tunnel - 60 m/months
	Expansion - 60 m/months
Installation schedule: of penstock	30 m/months

Table 4-17 Cycle Time

Item	Unit	Q'ty	Specification	Time	Remarks
Cross-section of excavation	m^2	64			
Progress length per blast	m	1.50			
Muckper blast Muck per blast	m^3	163			Change rate 1.70
Shotcrete	m^3	6.6	t = 10 cm		22m x 1.5m x 0.1 x 2
Rock bolt	pc	10			Part of arch
Drilling preparation	min.			10	
Drilling	min.	2	Drilling rate 0.70 m/min	90	145 hole 2 booms
Gunpowder preparation	min.			30	Incl. muck
Gunpowder	min.			70	143 hole 4 x 2 min/hole
Evacuation and blasting	min.			30	Incl. ventilation for 15 min.
Removal of muck	min.		2.1 m^3	135	27 units x 5 min. = 130 min.
Chopping	min.			15	
Shotcrete	min.			70	6.6 x 0.1 m^3/min
Rock bolt	min.			100	10 pcs x 10 min/pc
Others	min.			50	x 10%
Total cycle time	min.			600	
Daily progress	m		1,200 min	3.0	1.5 x 2
Monthly progress	m		25 days	75.0	

i) When working 20 hours a day
ii) 25 days a month

Fig.4-9 Penstock Construction Planning



4.3 Lower Tekai Site

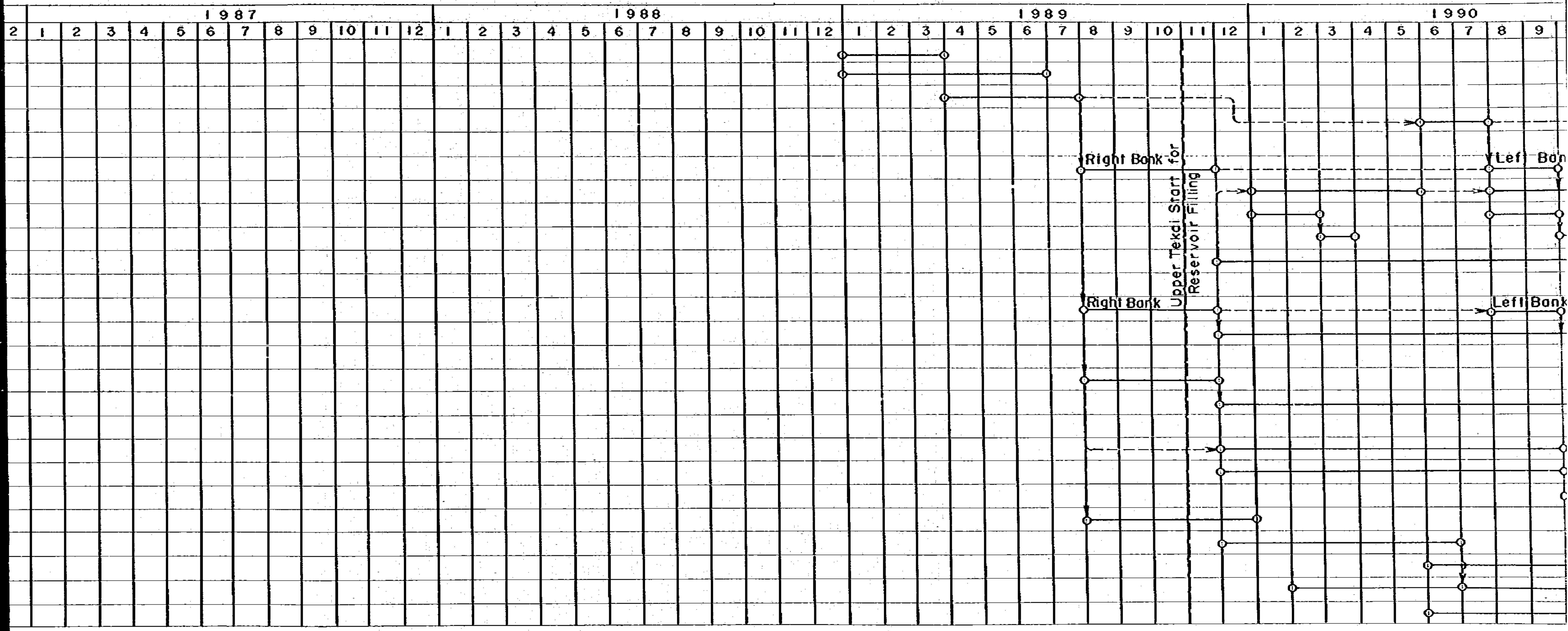
4.3.1 Construction Schedule

Construction Schedule of Lower Tekai is shown in Fig. 4-10.

Fig. 4-10 Lower Tekai

Description of Works		Unit	Quantity	1986												1987												
				1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12	1
Preparation	Temporary - Road	L. S	1																									
	Temporary - Facilities	L. S	1																									
Sluice Diversion	1st	L. S	1																									
	2nd	L. S	1																									
	Removing	L. S	1																									
Dam Body	Excavation	m^3	42.300																									
	Concrete	m^3	56.900																									
	Excavation (Tunnel)	m^3	810																									
	Linning Corcrete	m^3	330																									
	Grouting	t	1.425																									
	Others	L. S	1																									
Spillway	Excavation	m^3	32.900																									
	Concrete	m^3	14.900																									
	Others	L. S	1																									
Intake	Excavation	m^3	4.200																									
	Concrete	m^3	2.450																									
	Others	L. S	1																									
Penstock	Concrete	m^3	1.400																									
	Penstock	t	111																									
	Others	L. S	1																									
Power Honse	Excavation	m^3	25.500																									
	Concrete	m^3	11.500																									
	Generating Equipment	L. S	1																									
	Super Structure	L. S	1																									
	Others	L. S	1																									

Fig. 4-10 Lower Tekai Construction Schedule



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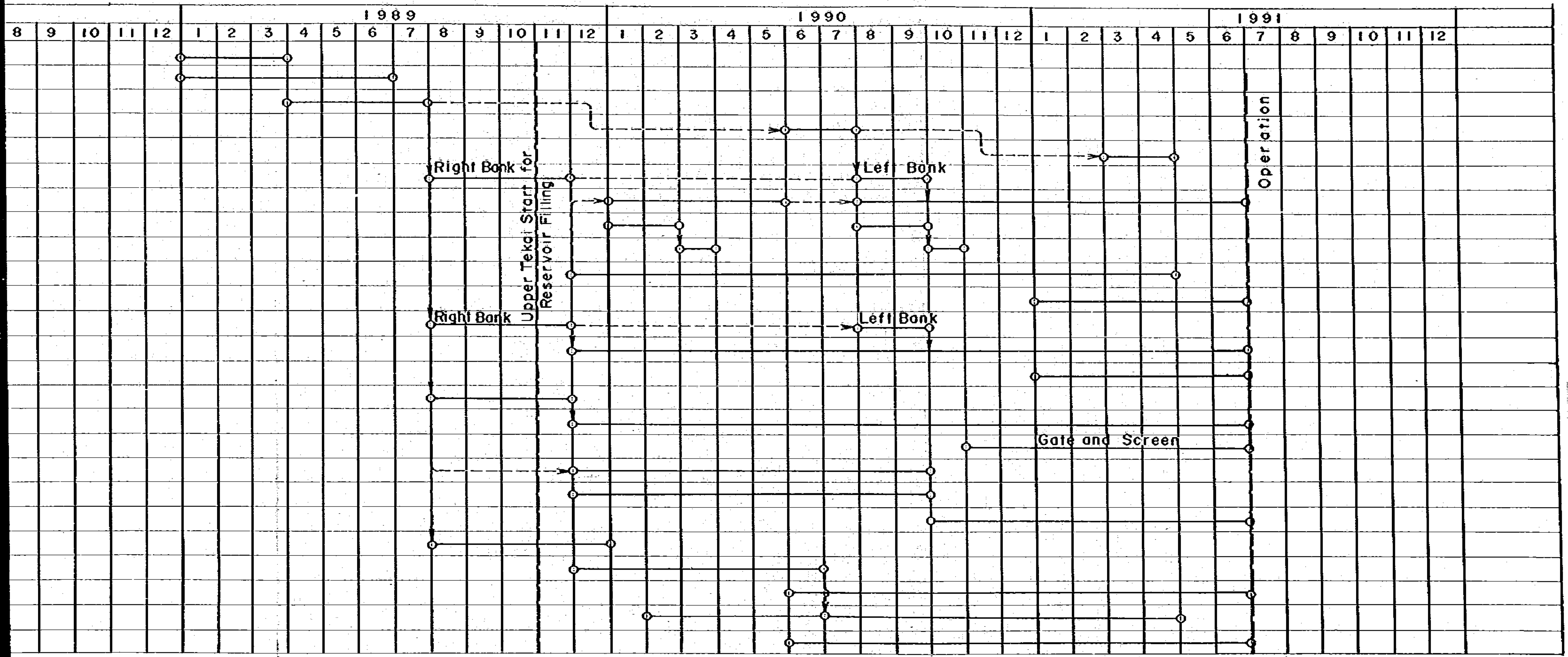

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FEASIBILITY STUDY OF TEKAI HYDRO-ELECTRIC POWER DEVELOPMENT PROJECT

LOWER TEKAI
CONSTRUCTION SCHEDULE

FIGURE 4-10

4.3.2 Diversion Works

Refer to 3.9

Construction of River Treatment

i) construction sequence of the first coffering

- o Driving of sheet pile (width 6, in two rows)
- o Filling work (in sheet pile)
- o Excavation of dam body on right bank
- o Foundation treatment
- o Concrete placement of dam body (construction of diversion in dam body)

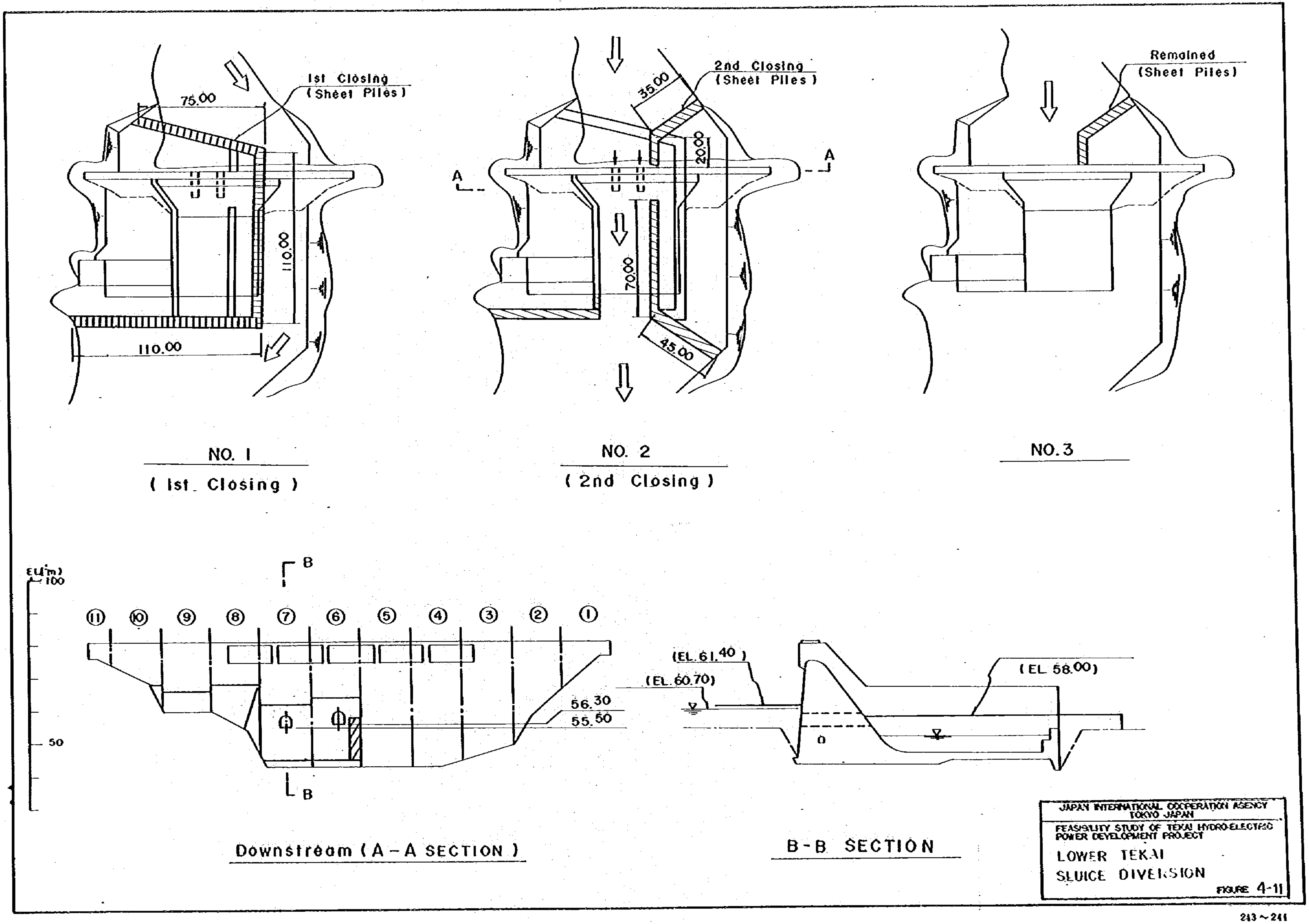
ii) Construction sequence of the second coffering

- o Construction inside the first coffering (energy dissipator and upstream)
- o Coffering at upstream and downstream location
- o Excavation on left bank
- o Foundation treatment
- o Placement of dam body concrete

iii) Temporary coffering removal sequence

- o Coffering of diversion tunnel inside dam body
- o Coffering at downstream
- o Discharge (pump up) of water inside energy dissipator
- o Removal of temporary coffering (sheet pile) inside energy dissipator
- o Removal of the second coffering (downstream)
- o Second coffering (upstream) cannot be removed.

The construction flow diagram is shown in Fig. 4-11.



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FEASIBILITY STUDY OF TEKAI HYDRO-ELECTRIC POWER DEVELOPMENT PROJECT

LOWER TEKAI SLUICE DIVERSION

FIGURE 4-11

4.3.3 Dam

(1) Construction Days

The following conditions are taken into consideration when calculating the days for the work:

i) Season
No down time will be planned as concrete placement will be carried out even during the wet season.

ii) Maintenance
One day/month

iii) Days off
2 days/month

iv) Rainfall
Concrete placement will not be carried out when the rainfall exceeds 20 mm/day. No special limit will be established for earth work.

v) Temperature
This will not be taken into consideration as a pre-cooling system will be provided.

vi) National holiday
7 days during Chinese New Year in February
7 days during Hari Raya Puasa in September

Table 4-18 Concrete work days

Item \ Month	1	2	3	4	5	6	7	8	9	10	11	12
Days month	31	28	31	30	31	30	31	31	30	31	30	31
Rainfall	1.8	1.5	1.6	2.5	2.8	2.3	2.2	2.8	2.7	3.7	4.5	4.2
Maintenance	1	1	1	1	1	1	1	1	1	1	1	1
Days off	2	-	2	2	2	2	2	2	-	2	2	2
National holiday	-	7	-	-	-	-	-	-	7	-	-	-
Total down days	4.8	8.5	3.6	5.5	5.8	5.3	5.2	5.8	10.7	6.7	7.5	7.2
Total work days	26.2	19.5	27.4	24.5	25.2	24.7	25.8	25.2	19.3	24.3	22.5	23.8 Av. 24.0

Table 4-19 Down days due to rainfall

Year \ Month	1	2	3	4	5	6	7	8	9	10	11	12
1961	4	2	1	5	3	3	2	2	3	5	2	3
1962	3	3	1	1	3	2	1	4	2	1	8	5
1963	0	1	0	0	1	2	2	0	4	1	10	7
1964	2	2	1	1	2	4	2	2	3	4	3	2
1965	0	0	0	4	7	1	1	2	2	6	5	6
1966	2	0	3	4	2	1	3	0	3	4	4	2
1967	5	4	5	3	4	1	2	1	4	4	6	4
1968	0	0	3	3	2	1	2	3	0	3	1	2
1969	2	0	1	1	4	4	3	7	1	6	3	6
1970	3	0	2	4	3	4	2	2	3	6	4	10
1971	3	2	4	1	6	1	2	6	0	2	2	9
1972	1	2	2	1	1	3	0	6	7	4	3	4
1973	1	2	0	8	3	3	3	5	3	5	5	2
1974	0	4	1	2	1	1	4	3	2	3	6	3
1975	1	3	3	2	3	3	4	2	5	3	6	4
1976	1	0	0	3	1	3	0	2	1	3	3	4
1977	1	4	1	2	1	3	1	6	2	2	5	2
1978	5	0	2	1	4	1	4	1	4	2	3	4
1979	0	0	1	2	2	3	4	0	3	7	6	0
Total	34	29	31	48	53	44	42	54	52	71	85	79
Average	1.8	1.5	1.6	2.5	2.8	2.3	2.2	2.8	2.7	3.7	4.5	4.2

The above data is calculated by obtaining the correlation coefficient of the Uru. Tekai Station from the rainfall data of Kangsar.

(2) Excavation Plan

i) Excavation

Excavation of the Lower Tekai will start on August 1989 when the first coffering of sluice diversion is completed. Excavation will be simultaneously carried out for the dam body, spillway, intake, and power plant.

The excavation quantity in the first coffering is as listed below.

Dam body	31,000 m^3	(Excavation in four months)
Spillway	18,000 m^3	(")
Intake	16,000 m^3	(")
Power plant	25,500 m^3	(Excavated in five months)
Total	90,500 m^3	

Thus, the mean excavation quantity per month is 21,000 m^3.

The excavation quantity after the second coffering is as listed below.

Dam body	11,000 m^3	(Excavation in two months)
Spillway	15,000 m^3	(")
Total	26,000 m^3	

Thus, the mean excavation quantity per month is 13,000 m^3.

Before start of excavation, the 6 ton one-end travelling type cable crane for the concrete placement will be installed for transportation of materials and equipment.

Excavation will be made according to the bench cut method (H=5m). Excavated soil and rock will be deposited onto the river bed and, from there, will be loaded onto dump trucks to be transported to the spoil area. About 20 cm of mortar will be sprayed over the excavated surface to protect it from weathering.

Prior to concrete placement, manual finish excavation will be carried out.

ii) Soil disposal

The total soil disposal quantity is about 120,000 m^3. soil and rocks produced here will not be re-used for other purpsoes, but completely disposed of in the spoil area. This spoil area (200,000 m^3 capacity) will be located at about 3 km upstream of the Lower Tekai.

(3) Dam Concrete Placement Plan

i) Placement Process

The dam concrete placement equipment is described in 4.4. The 6 ton one-end travel type cable crane will be used. The bucket capacity is 2.0 m^3 and the batcher plant capacity is 30 m^3/h.
The dam body will be divided into 11 blocks with one block running 15m in the axial direction. The layer system will be employed in the upstream and downstream direction.
Concrete from the batcher plant will be lifted by the crane and transported to the required block. Concrete lowered onto the block will be compacted to the required layer by means of a vibration.

ii) Placement schedule

The lift schedule of dam concrete is shown in Fig. 4-12. To prevent temperature rise, concrete will be placed in 1m lift up to EL = 60.0m (height 17.0m) above the river bed and 1.5m lift from EL = 60m to the dam crest.
The maximum layer placement quantity will be
34.0 (length) x 15.0 (width) x 1.0 (lift) = 510 m^3
By care of 30 m^3/h, placement period will be
510 m^3 / 30 m^3/h = 17hours

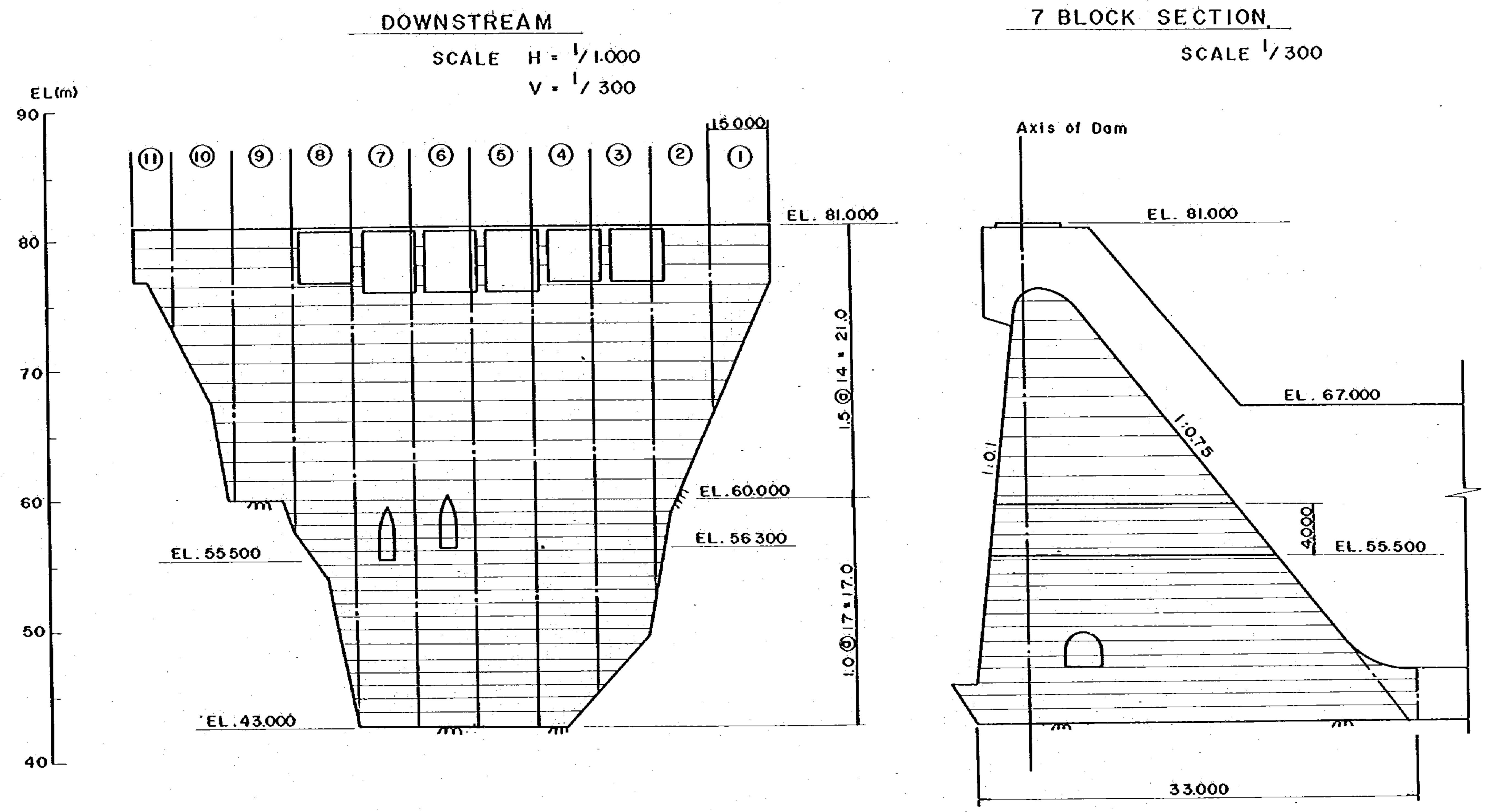
Concrete will be continuously placed day and night.
The total concrete quantity is 56,900 m^3. Placemnt work will start in January 1990 and be completed in 16 months (excluding June and July 1990 for the diversion work) until July 1991.

During the five-month period after the first coffering, concrete placement of about 20,000 m^3 will be made to raise block ⑥ through ⑪ from EL = 62 to EL = 69 m on the right bank. Then concrete will be placed for block ① through ⑤ on the left bank after the second coffering. When block ① through ⑤ reach the same level as block ⑥ through ⑪, concrete placement will be made alternatively for block ① through ⑪.

Concrete placement will be made once every five days.

Table 4-20 Cycle time of concrete placement

Kind of work \ Total days	1	2	3	4	5	6	
Placement							
Curing							
Cleaning (Creen cut)							
Form setting							



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LOWER TEKAI (DAM BODY) CONCRETE BLOCK AND LIFT SCHEDULE

FIGURE 4-12

4.4 Temporary Facilities

4.4.1 Outline

Temporary facilities for Tekai Project were studied in accordance with the following conditions.

(1) Contract package for construction shall be devided into Access Road, Upper Tekai, Lower Tekai and Electrical Works.

(2) Since the power requirement will be about 5,000 KVA in total, the power facilities can be concentrated in one place to facilitate operation and maintenance.

(3) The aggregate plant is planned separately for Access Road, Upper Tekai and Lower Tekai. As a quarry site of aggregate was found in D site for Access Road and Lower Tekai, in B site for Upper Tekai.

Location map of a quarry site is shown in Fig. 4-13.

Principal facilities of Tekai Project are shown in Table 4-21. The concrete quantities of Upper Tekai and Lower Tekai are shown in Table 4-22.

Table 4-21 Principal faicltities

	Name	Specification	Remarks
Access Road	Asphalt plant	30 t/h	
	Aggregate plant	70 t/h	
Upper Tekai	Batcher plant	60 m^3/h	Stocked for 5 days
	Cement silo	250 t	
	Aggregate plant	50 t/h	
Lower Tekai	Batcher plant	30 m^3/h	
	Cement silo	300 t	Stocked for 5 days
	Cable crane	6.0 t	One-end travel type
	Aggregate plant	80 t/h	

Table 4-22 Design concrete quantity

(Unit: m^3)

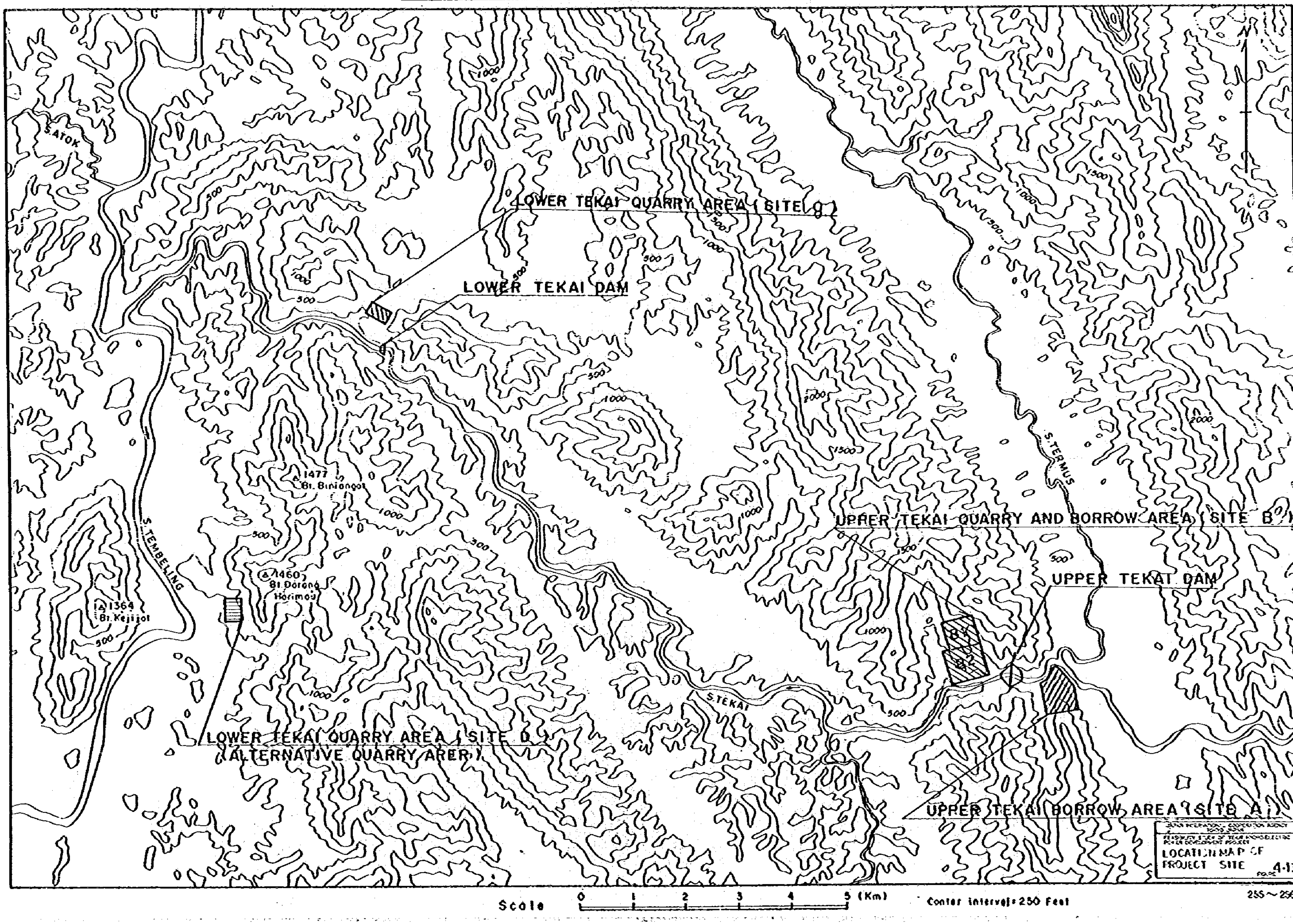
	Dam	Open	Tunnel	Total
Upper Tekai	-	111,180	19,920	131,100
Lower Tekai	56,900	32,800	200	89,900

Concrete placement period

Upper dam: August 1986 - November 1990 (52 months)

Lower dam: January 1990 - July 1991 (16 months)

Fig. 4-13 LOCATION MAP OF PROJECT SITE



4.4.2 Facilities

- o Batcher plant
- o Cement silo
- o Aggregate plant

These facilities are described below in the order given.

(1) Batcher plant

(a) Upper Tekai

o Total placement quantity:	131,100 m^3
o Operation period:	50 months
o Mean placement quantity per month:	2,600 m^3
o Mean placement quantity per day: (22 days)	120 m^3
o Max. placement quantity per month:	5,200 m^3
o Max. placement time per day: (22 days)	240 m^3

The batcher plant to be used will be a 1 m^3 forced mixer (60 m^3/h)

Transport capacity from batcher plant: 40 m^3/h

Batcher operation time (for max. placement quantity per day):

$240\ m^3/day\ /\ 40\ m^3/h = 6\,h/day$

(b) Lower Tekai

o Total placement quantity (except diversion) :	89,900 m^3
o Operation period:	16 months
o Mean placement quantity per month:	5,600 m^3
o Mean placement quantity per day: 22 days	250 m^3
o Max. placement quantity per month:	8,800 m^3
o Max. placement time per day: 22 days	400 m^3

Batcher plant and placement equipment to be used will have the following performance

For standard placement quantity: 30 m^3/h

Mean placement time per day: 250 m^3 / 30 m^3/h = 8.4h

Max. placement time per day: 400 m^3 / 30 m^3/h = 13.4h

Machines will be selected based upon the following standards for a multi-purpose dam

Table 4-23

	Principal placement equipment	Bucket capacity	Mixer	Standard placement
①	4.5T	1.5 m^3	0.756 m^3	20 m^3/h
②	6.0T	2.0 m^3	0.756 m^3	30 m^3/h
③	9.0T	3.0 m^3	1.512 m^3	40 m^3/h

From the above table, 2 is selected.

* Batcher plant: 0.756 m^3 x 3
* Crane : 6.0 ton One-end traveling type

(2) Cement Silo

o Capacity of cement silo of the Upper Tekai

For 5 day stock;

With mean placement quantity per day at 120 m^3,

120 m^3/day x 5 days x 0.3 t/m^3 = 180t 250t

o Capacity of cement silo of the Lower Tekai

With the mean placement quantity per day at 250 m^3

250 m^3/day x 5 days x 0.2 t/m^3 = 250t 300t

(3) Aggregate plant

i) Products of the aggregate plant

Products of this plant are as follows;

(1) Concrete aggregate for the Upper Tekai

(2) Concrete aggregate for the Lower Tekai

(3) Shotcrete aggregate

(4) Subbase course material for access road

(5) Subbase course material for temporary road

(6) Filter material for embankment (considered for the primary plant only)

ii) Aggregate consumption

(1) Concrete aggregate for the Upper Tekai

$131{,}100\ m^3 \times 2.1\ t/m^3 = 275{,}300t$

(2) Concrete aggregate for the Lower Tekai

$89{,}900\ m^3 \times 2.1\ t/m^3 = 188{,}800t$

(3) Shotcrete aggregate

$7{,}700\ m^3 \times 2.1\ t/m^3 = 13{,}900t$

(4) Subbase coarse material for access road

$110{,}000\ m^3 \times 2.1\ t/m^3 = 231{,}000t$

$620\ m^3 \times 2.1\ t/m^3 = 1{,}300t$

iii) Aggregate consumption schedule

The aggregate consumption schedule is shown in Table 4-24.

Table 4-24 Process requiring aggregate

(Unit: ton)

	Quantity	1984	1985	1986	1987	1988	1989	1990	1991
Aggregate for the Upper Tekai	275,300					6,000	t/month	(46)	
Aggregate for the Lower Tekai	188,800							11,800	(16)
Shotcrete aggregate	13,900				300	t/month		(42)	
For access road	232,300	9,700	t/month	(24)					

iv) Capacity of aggregate plant

$$A = \frac{V}{D \cdot H \cdot E}$$

where,

A : Production per unit time (t/h)

V : Mean aggregate consumption per month (t/month)

D : Aggregate plant operation days per month (25 days)

H : Working hours a day (8 hours)

E : Plant operation factor.

° Access Road

$$A = \frac{9,700}{25 \times 8 \times 0.8} = 61 \text{ (t/h)} \fallingdotseq 70 \text{ (t/h)}$$

° Upper Tekai

$$A = \frac{6,000}{25 \times 8 \times 0.8} \fallingdotseq 38 \text{ (t/h)} \fallingdotseq 50 \text{ (t/h)}$$

° Lower Tekai

$$A = \frac{11,800}{25 \times 8 \times 0.8} = 74 \text{ (t/h)} \fallingdotseq 80 \text{ (t/h)}$$

Table 4-25 Short-term schedule for temporary facilities (at the work start)

Item		1986											1987					
Item	Date	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6
Temporary building	Office, quarters, others																	
Aggregate plant				Excavation, leveling 150,000 m3			Installation											
Batcher plant																		
Diversion tunnel	Outside temporary facilities																	
Access road	Divesion tunnel inlet																	
— // —	Quarry site (Site B-1)																	
— // —	Second coffering on the right bank																	
— // —	Spillway crest on the left bank																	
— // —	Access way to the right bank																	
— // —	Intake																	
Penstock	Outside facilities																	