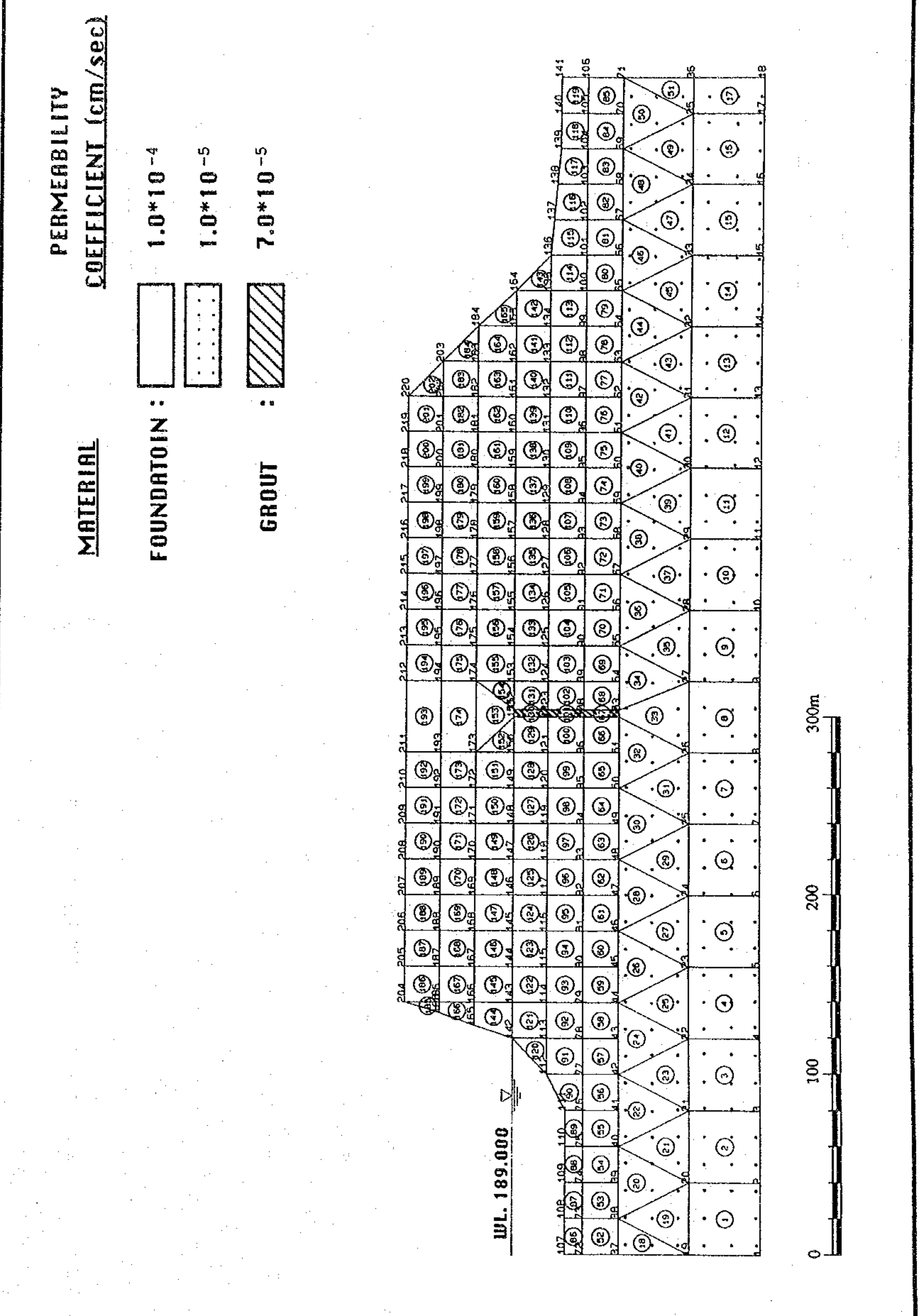
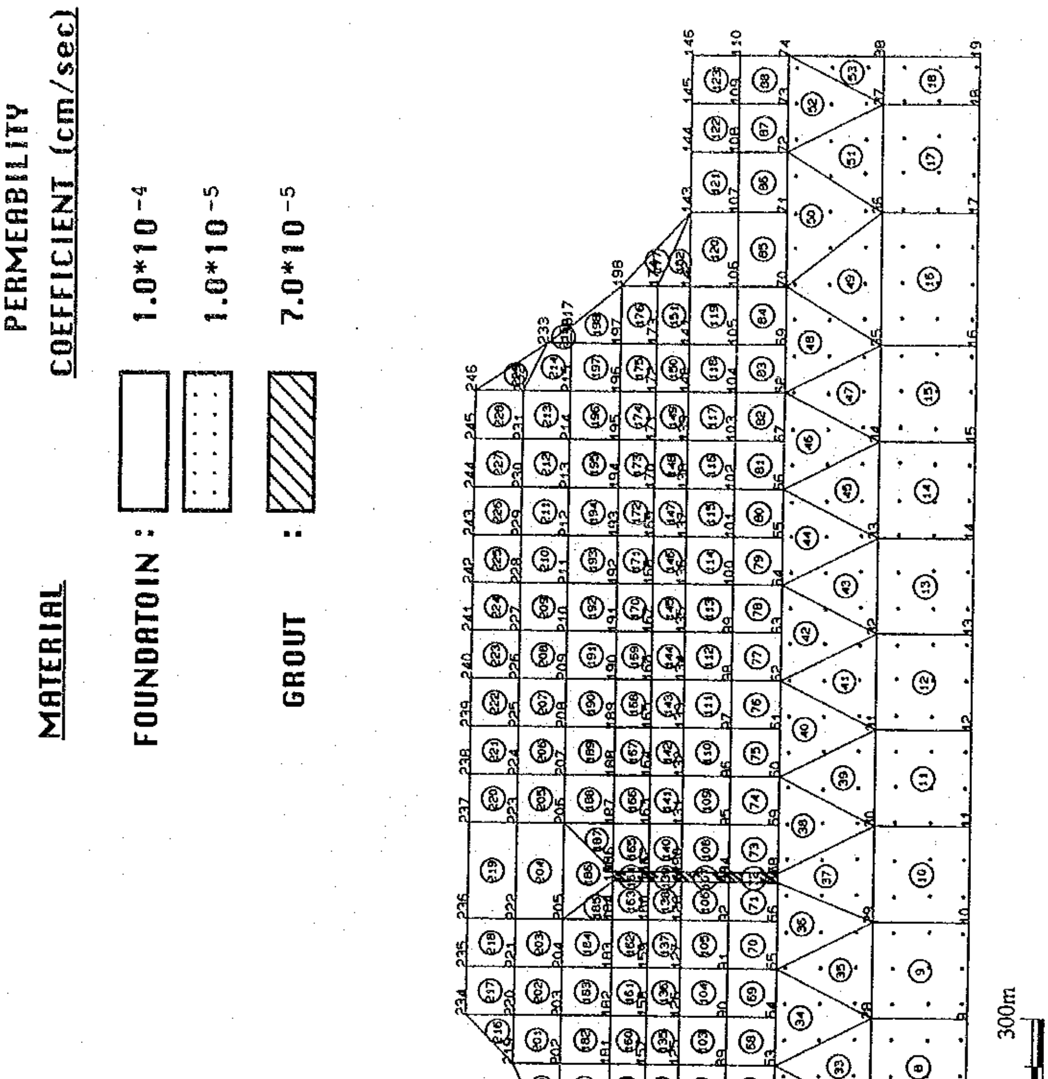
Fig. 3.9.4



MODEL FOR F.E.M. SEEPAGE ANALYSIS (LEFT BANK, SECTION A–A)	GOVERNMENT OF MAURITIUS PORT LOUIS WATER SUPPLY PROJECT
	JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. 3.9.5



MODEL FOR F.E.M. SEEPAGE ANALYSIS
(LEFT BANK, SECTION B–B)

GOVERNMENT OF MAURITIUS
PORT LOUIS WATER SUPPLY PROJECT

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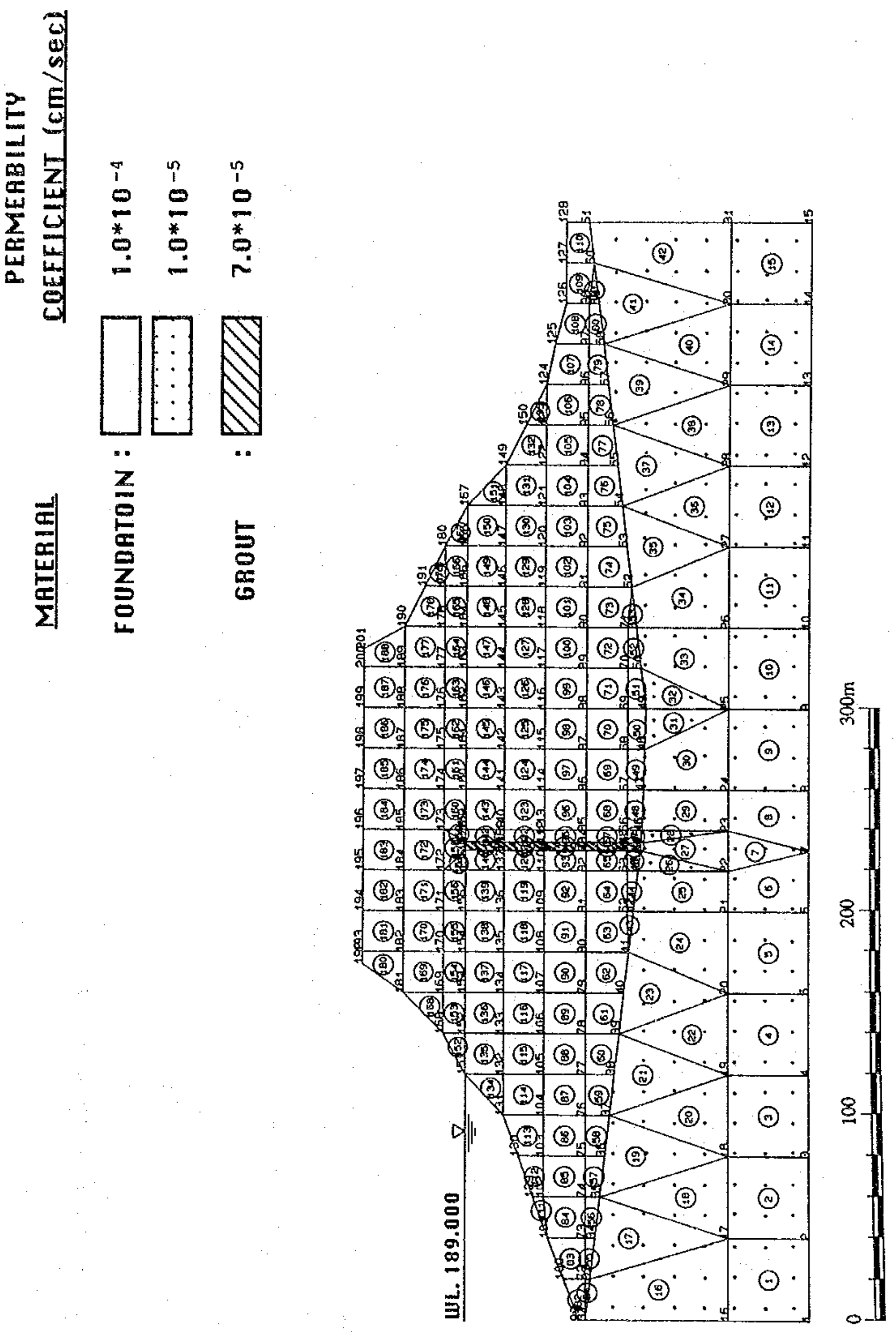
Fig. 3.9.6



MODEL FOR F.E.M. SEEPAGE ANALYSIS
(LEFT BANK, SECTION C–C)

GOVERNMENT OF MAURITIUS
PORT LOUIS WATER SUPPLY PROJECT

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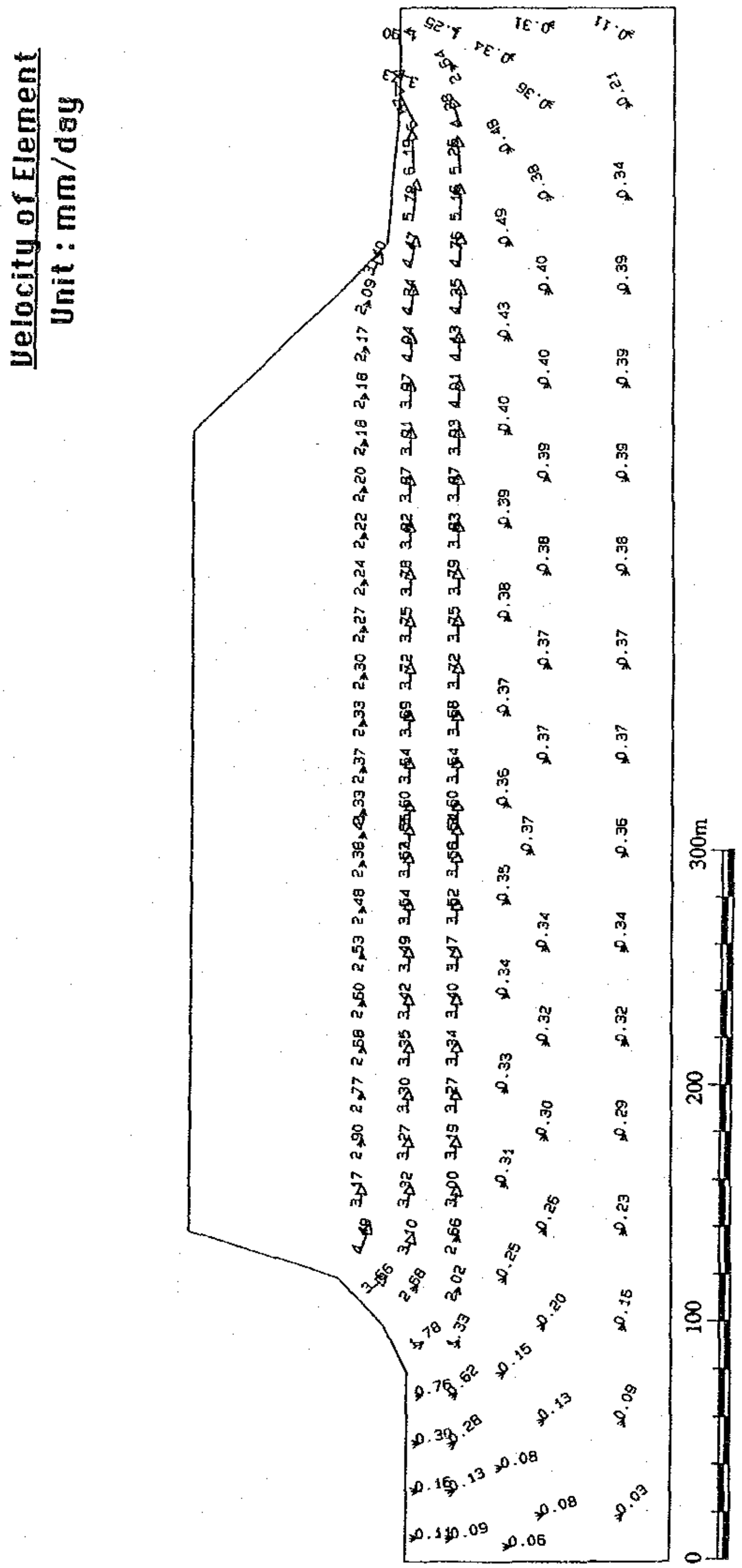
Fig. 3.9.7



VELOCITY OF ELEMENT
(LEFT BANK, SECTION A-A)

GOVERNMENT OF MAURITIUS
PORT LOUIS WATER SUPPLY PROJECT

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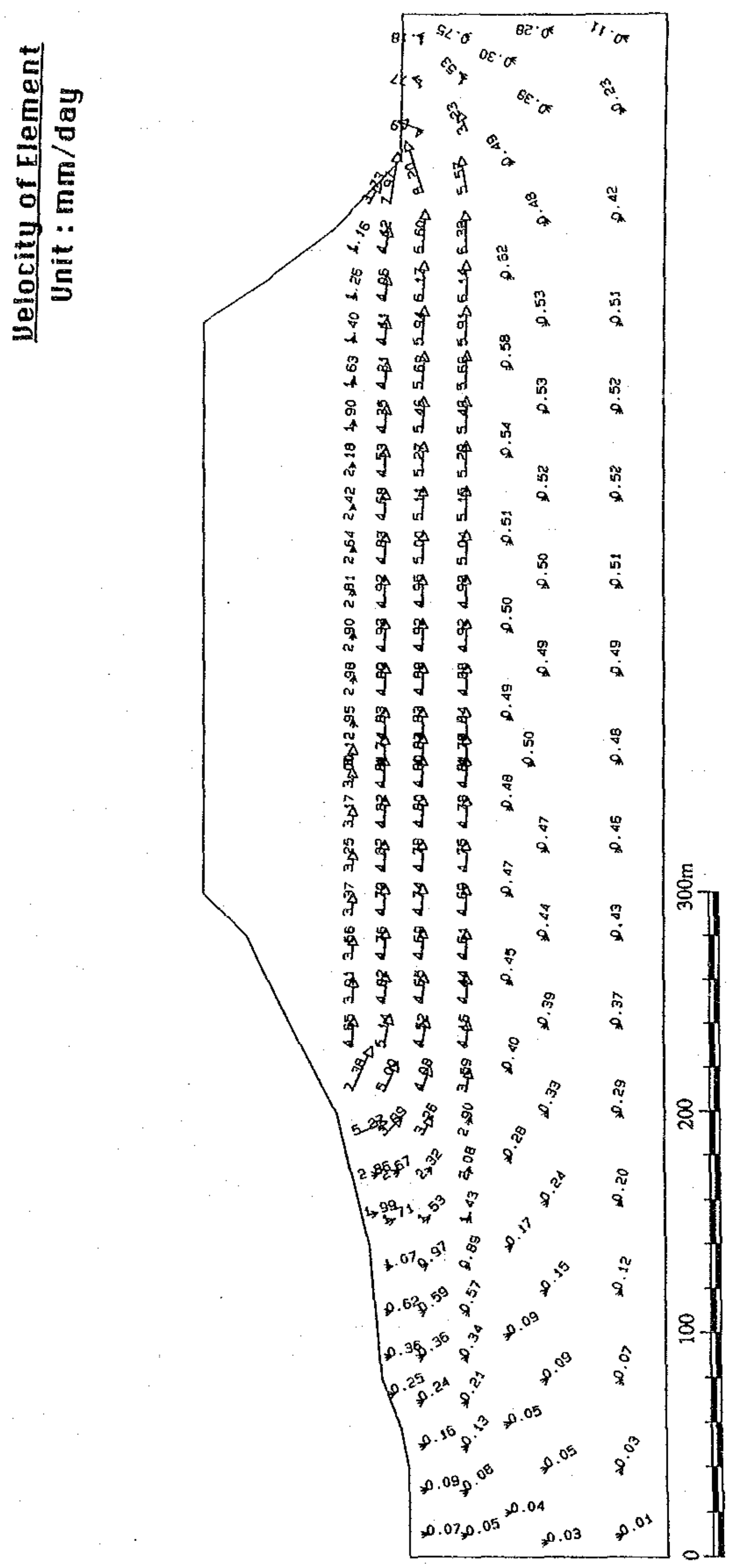
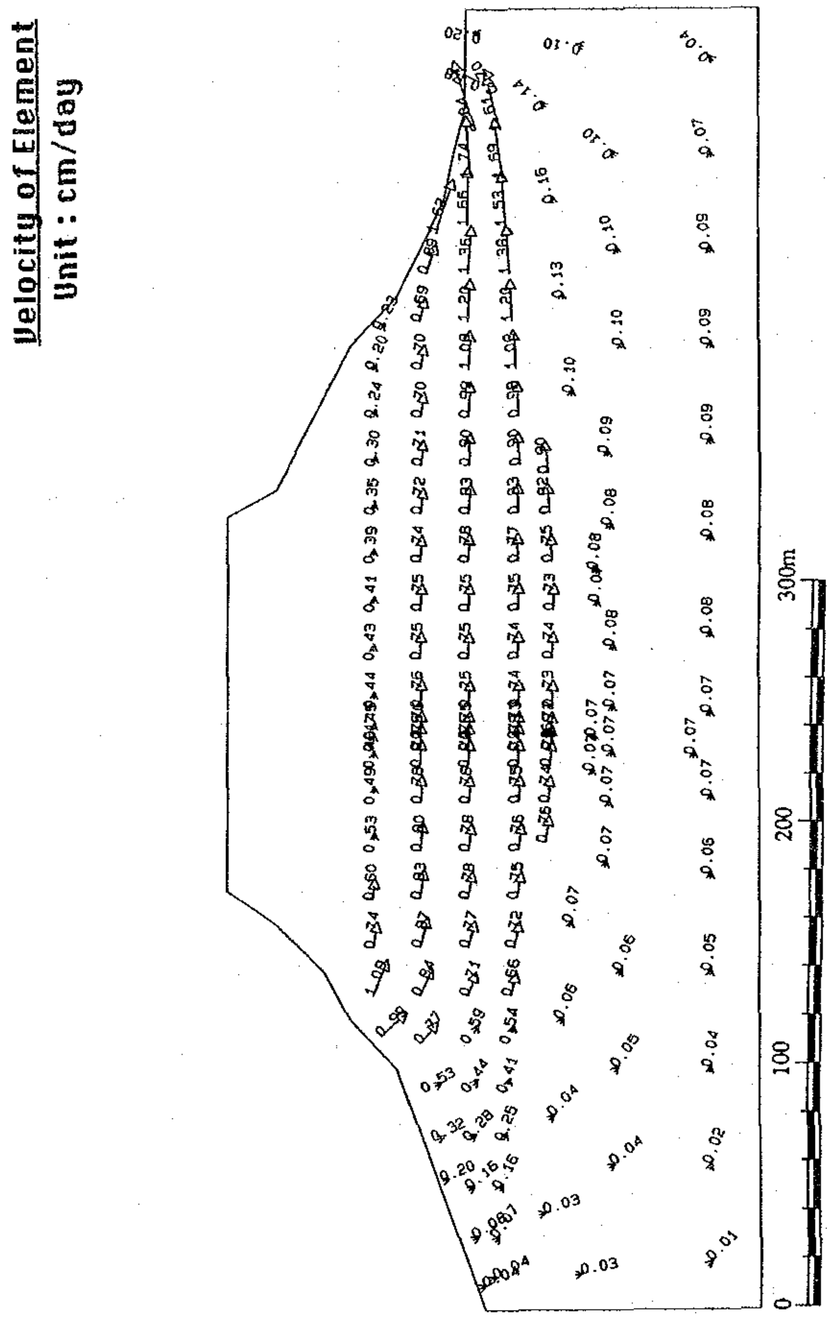
Fig. 3.9.8

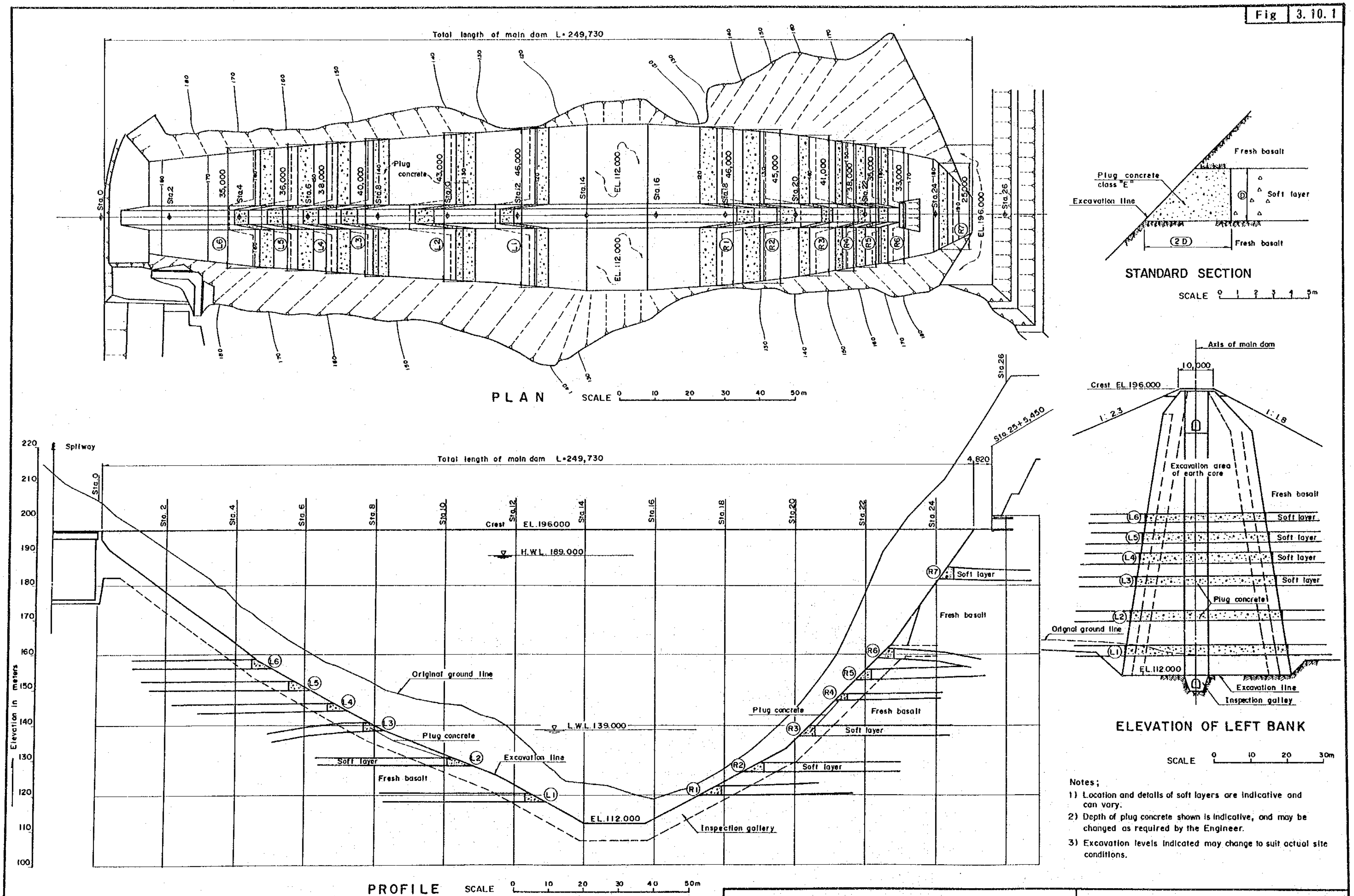


VELOCITY OF ELEMENT (LEFT BANK, SECTION B-B)	GOVERNMENT OF MAURITIUS PORT LOUIS WATER SUPPLY PROJECT JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. 3.9.9



VELOCITY OF ELEMENT (LEFT BANK, SECTION C-C)	GOVERNMENT OF MAURITIUS PORT LOUIS WATER SUPPLY PROJECT JAPAN INTERNATIONAL COOPERATION AGENCY



Notes;

1) Location and details of soft layers are indicative and can vary.
2) Depth of plug concrete shown is indicative, and may be changed as required by the Engineer.
3) Excavation levels indicated may change to suit actual site conditions.

MAIN DAM
PLUG CONCRETE FOR CORE FOUNDATION

GOVERNMENT OF MAURITIUS
PORT LOUIS WATER SUPPLY PROJECT
JAPAN INTERNATIONAL COOPERATION AGENCY

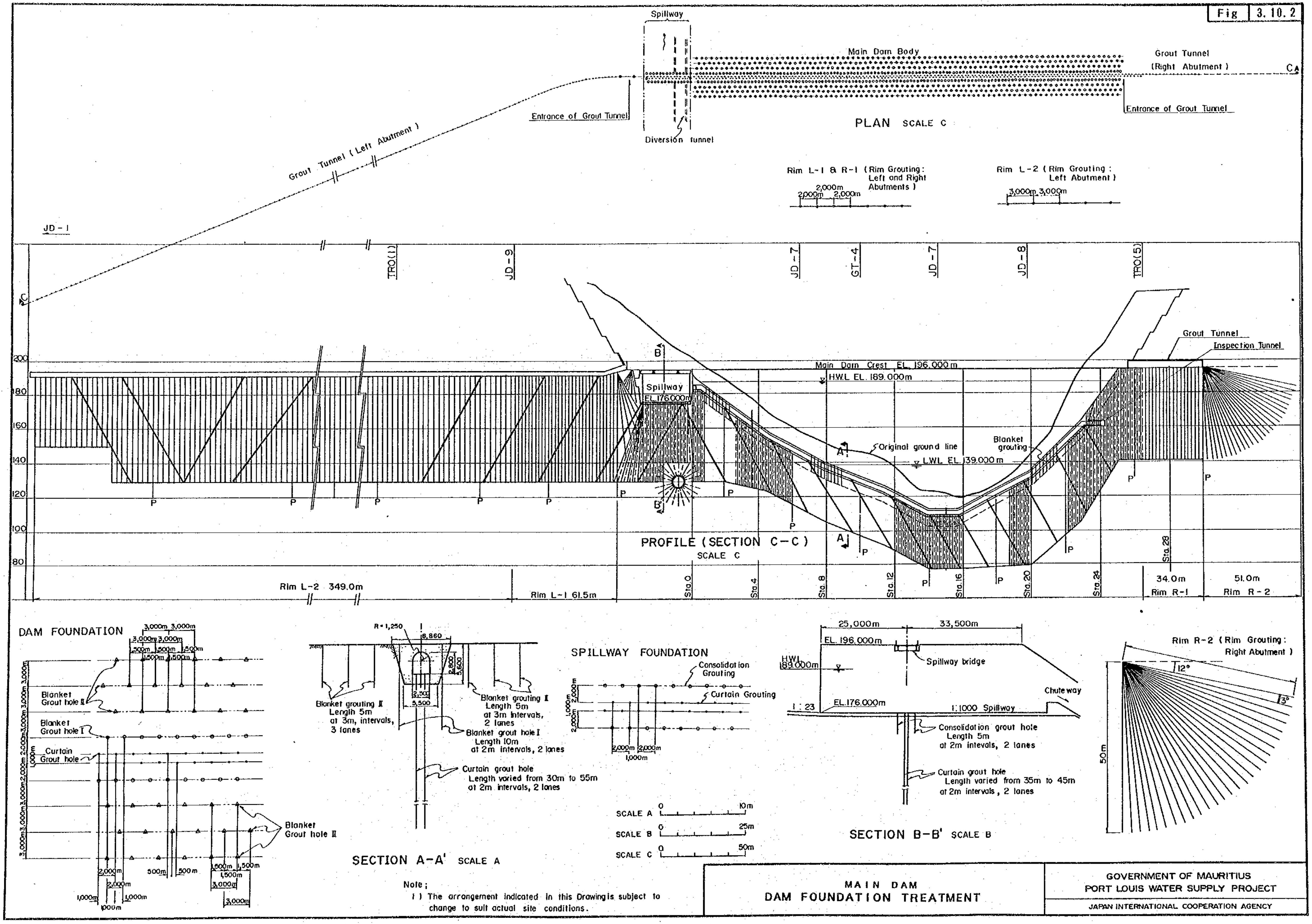
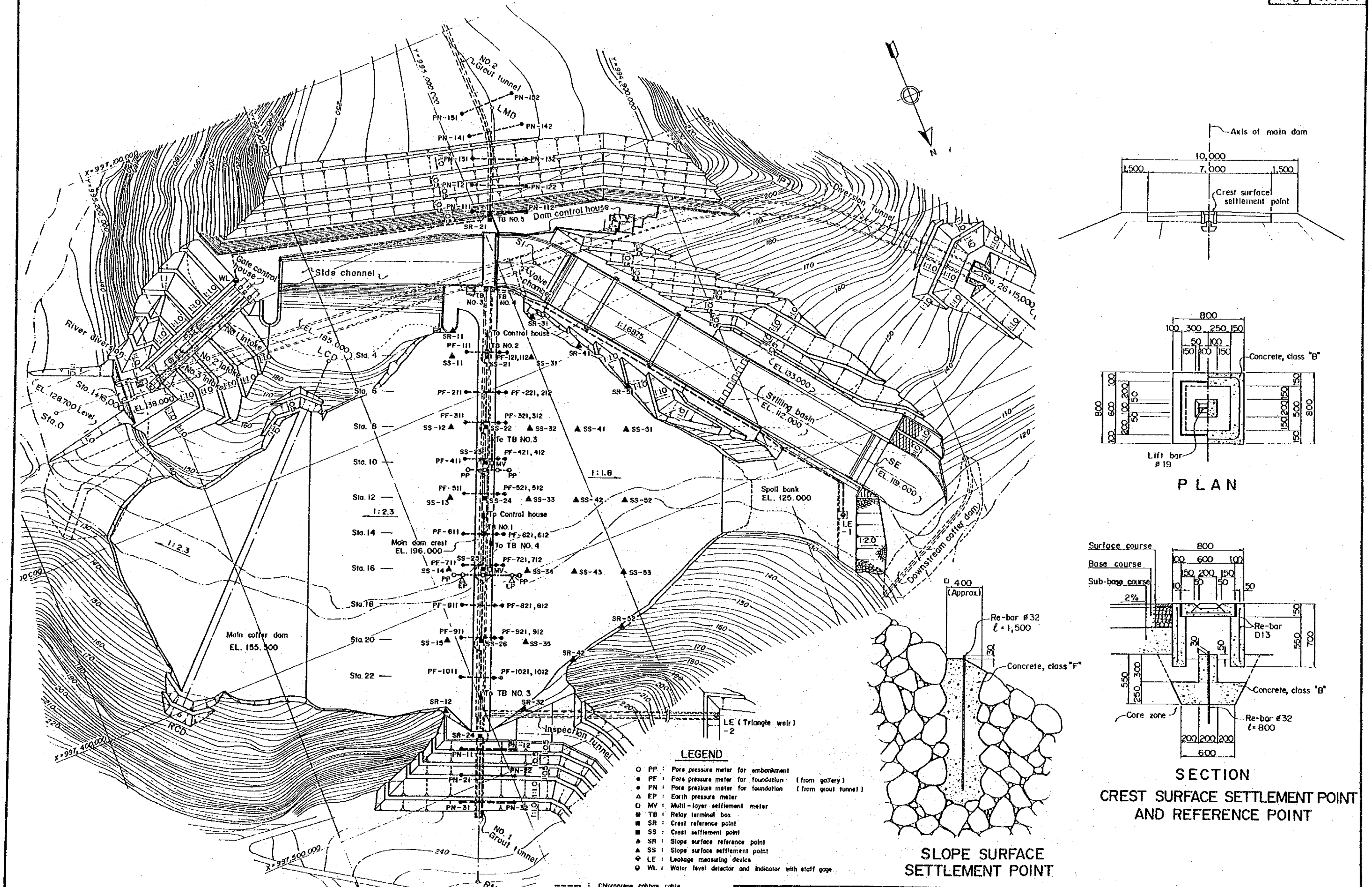


Fig 3.10.2
Spillway
Main Dam Body
Grout Tunnel (Right Abutment)
Entrance of Grout Tunnel
Diversion tunnel
PLAN SCALE C
Grout Tunnel (Left Abutment)
Rim L-1 & R-1 (Rim Grouting: Left and Right Abutments)
Rim L-2 (Rim Grouting: Left Abutment)
Main Dam Crest EL. 196.000m
H.W.L EL. 189.000m
Spillway EL.176.000m
Original ground line
LWL EL. 139.000m
Blanket grouting
Grout Tunnel
Inspection Tunnel
PROFILE (SECTION C-C) SCALE C
Rim L-2 349.0m
Rim L-1 61.5m
34.0m Rim R-1
51.0m Rim R-2
DAM FOUNDATION
Blanket Grout hole II
Blanket Grout hole I
Curtain Grout hole
Blanket Grout hole II
SECTION A-A' SCALE A
Blanket grouting II Length 5m at 3m, intervals, 3 lanes
Blanket grouting II Length 5m at 3m intervals, 2 lanes
Blanket grout hole I Length 10m at 2m intervals, 2 lanes
Curtain grout hole Length varied from 30m to 55m at 2m intervals, 2 lanes
SPILLWAY FOUNDATION
Consolidation Grouting
Curtain Grouting
SCALE A 0 10m
SCALE B 0 25m
SCALE C 0 50m
EL. 196.000m
HWL 189.000m
Spillway bridge
Chute way
EL.176.000m
1:1000 Spillway
Consolidation grout hole Length 5m at 2m intervals, 2 lanes
Curtain grout hole Length varied from 35m to 45m at 2m intervals, 2 lanes
SECTION B-B' SCALE B
Rim R-2 (Rim Grouting: Right Abutment)
Note;
1) The arrangement indicated in this Drawing is subject to change to suit actual site conditions.
MAIN DAM
DAM FOUNDATION TREATMENT
GOVERNMENT OF MAURITIUS
PORT LOUIS WATER SUPPLY PROJECT
JAPAN INTERNATIONAL COOPERATION AGENCY

Fig 3.11.1

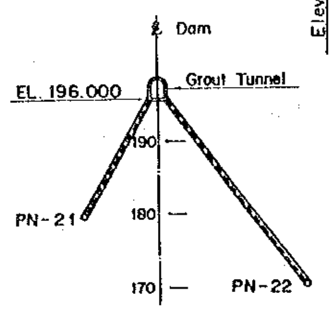


MAIN DAM
MEASURING FACILITIES,
INSTRUMENTATION (1)

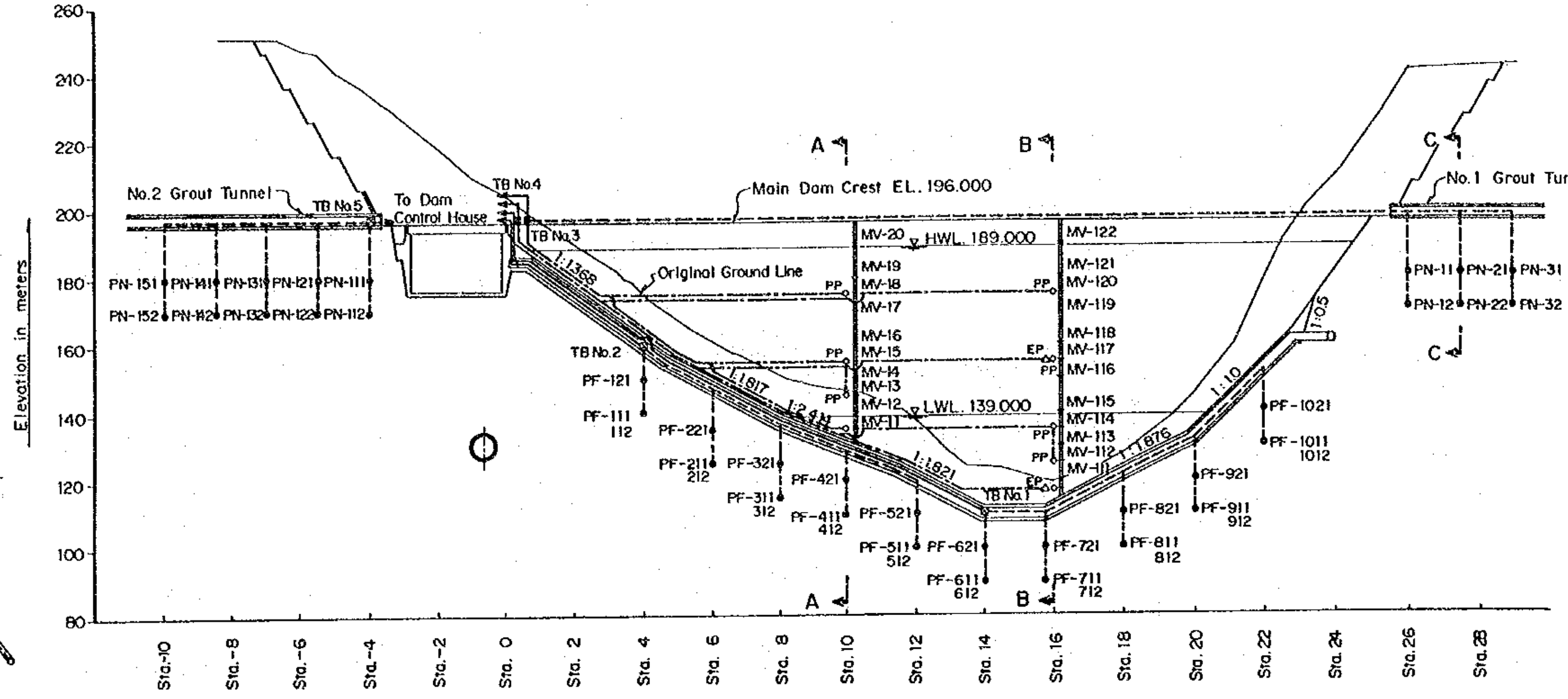
GOVERNMENT OF MAURITIUS
PORT LOUIS WATER SUPPLY PROJECT

JAPAN INTERNATIONAL COOPERATION AGENCY

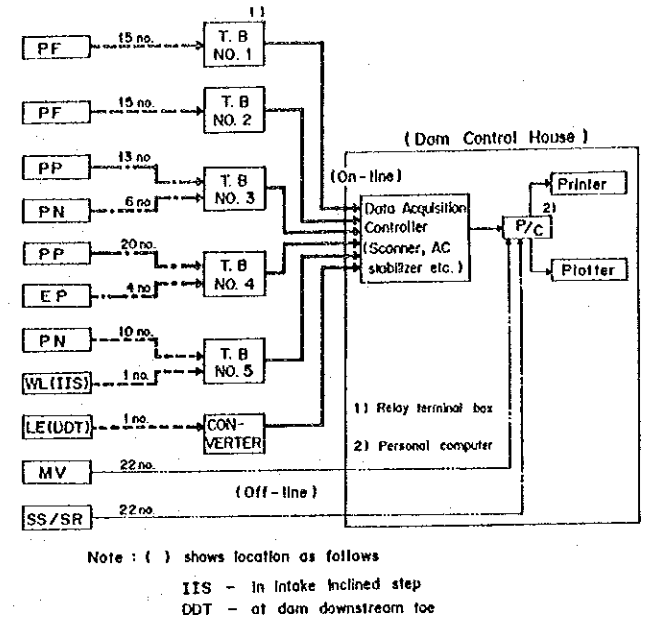
Fig 3.11.2



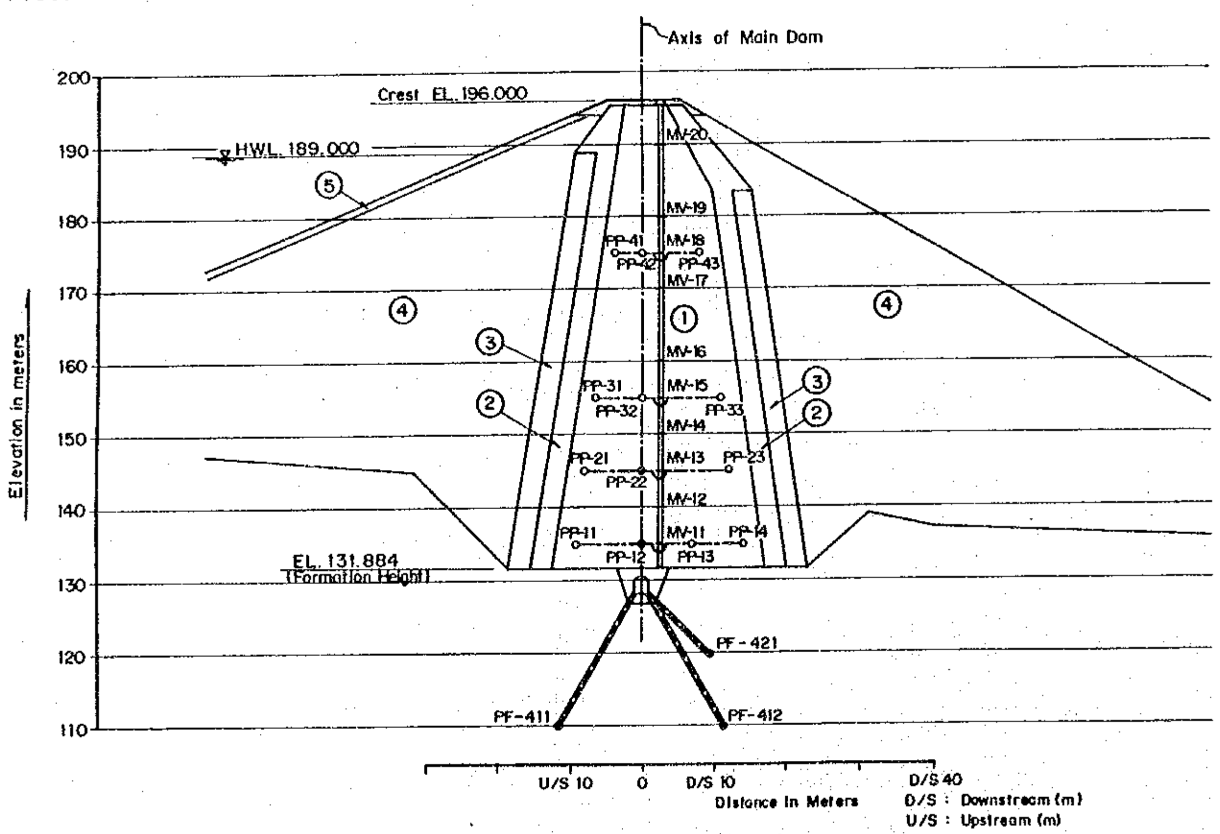
SECTION C-C SCALE B



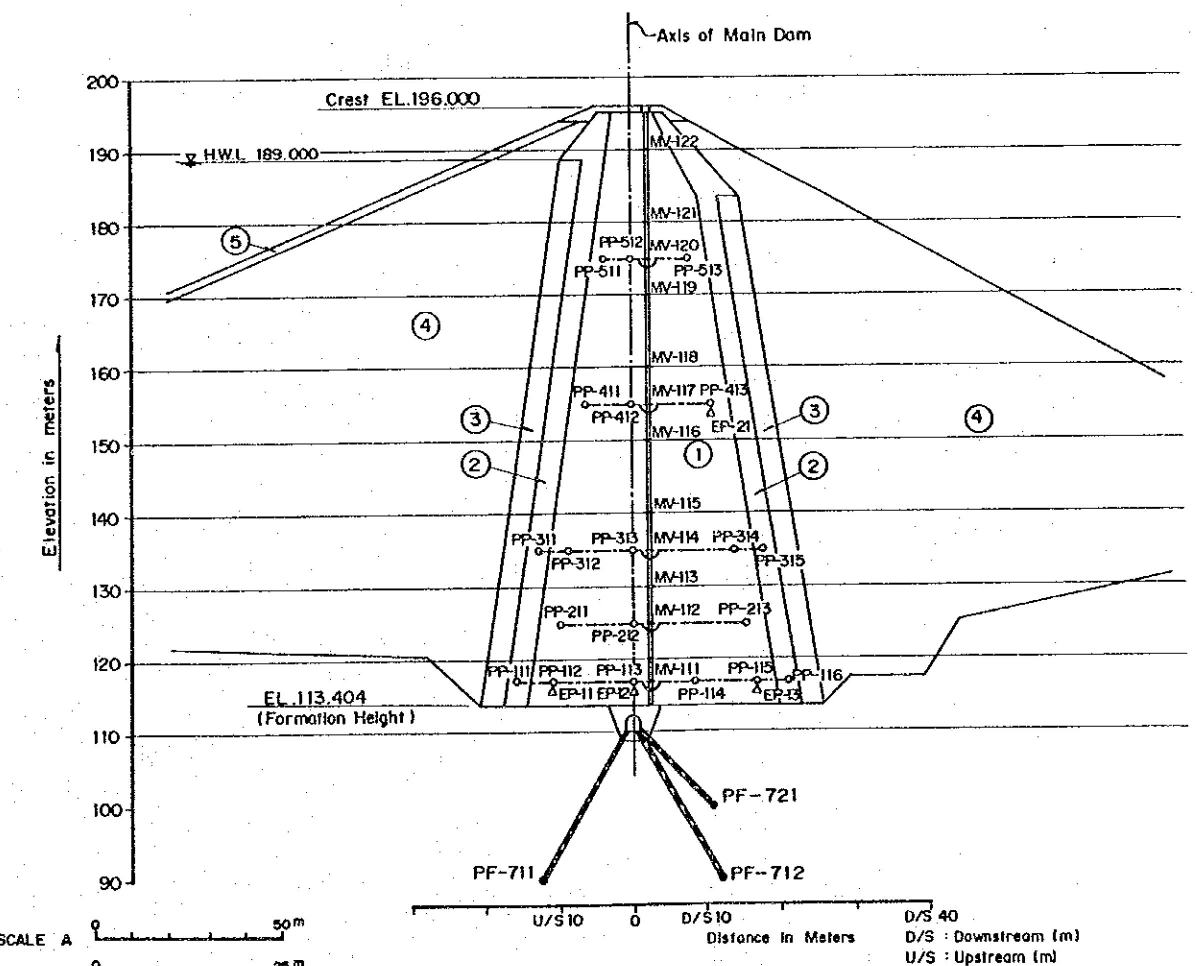
PROFILE SCALE A



OVERALL BLOCK DIAGRAM OF INSTRUMENTATION FOR MEASURING APPARATUS



SECTION A-A (Sta. 10) SCALE B



SECTION B-B (Sta. 16) SCALE B

SCALE A 0 — 50m
SCALE B 0 — 25m

LEGEND

- ○ PP : Pore Pressure Meter for Embankment
- ● PF : Pore Pressure Meter for Foundation (from Gallery)
- ● PN : Pore Pressure Meter for Foundation (from Grout Tunnel)
- △ EP : Earth Pressure Meter
- ╫ MV : Multi - Layer Settlement Meter
- ⊠ TB : Relay Terminal Box
- ---- : Chloroprene Cabtyre Cable
- -·-·- : Chloroprene Cabtyre Cable (Heavy Duty)
- —— : Multi Core Cable
- ◆ LE : Leakage measuring device
- ● WL : Water level detector and indicator with staff gage

Symbol	Zone
1	Impervious Core
2	Fine Filter
3	Coarse Filter
4	Rock
5	Riprap

MAIN DAM
MEASURING FACILITIES, INSTRUMENTATION (2)

GOVERNMENT OF MAURITIUS
PORT LOUIS WATER SUPPLY PROJECT

JAPAN INTERNATIONAL COOPERATION AGENCY

CHAPTER IV. DESIGN OF SPILLWAY

4.1 General

The type and location of the spillway are determined through comparative studies on conceivable alternatives as follows:

- As for the discharge carrier of the spillway, an open chuteway type and a tunnel type to utilize the diversion tunnel are conceivable. Comparative studies on the above two types come to the following conclusion: that is, the safety for handling floods will seriously be lessened in the case of the tunnel type. Besides that, any cost advantage or other merits are not expected in the tunnel type in the case of this project. Hence, the open chuteway type is selected.

- Alternative locations for the spillway are considered on the right and left abutments. A comparative study between both locations between both locations reveals that the spillway to be located on the left abutment will give a cost advantage mainly due to the topographic conditions, coming to a conclusion that the spillway should be located on the left abutment.

- Three (3) types of the spillway control structure are conceivable: (i) gate control type, (ii) uncontrolled type (non-gated) and (iii) combined type of (i) and (ii) above. Out of the three (3) types, the uncontrolled type (non-gated type) is adopted for the project, especially regarding the safety as important. The proposed type of spillway control structure will ensure a high safety by avoiding floods due to malfunction or misoperation of spillway gates.

- As for the type of uncontrolled weir, a side channel type which reduces the excavation volume is evidently most suitable from the economical point of view.

With regard to the energy dissipator, a horizontal apron type energy dissipator with an end sill (a stilling basin type of energy dissipator) is selected through comparative studies on other three (3) types of the hydraulic jump type, inclined apron type and roller bucket type as follows:

(i) Three types of the hydraulic jump type, inclined apron type and roller bucket type require an excessive excavation for the bottom of energy dissipator because their hydraulic mechanism requires to lower the bottom elevation sufficiently below the downstream water level.

The above does not make these types justifiable economically.

(ii) The ski-jump type requires a large plunge pool for energy dissipation of the energetic jet. An examination indicates the plunge pool of 100 m length, 30 m width and 13 m depth would be required. This type indicates a slight cost advantage (93% of the cost for the stilling basin type). However, the type has a high possibility to cause a remarkable scouring of the river bed or banks due to an very energetic jet. It is considered desirable to avoid the occurrence of such environmental and other unexpected troubles.

4.2 Hydraulic Design of Spillway

Probable flood peak discharges are analyzed as follows:

Return Year (Year)	Peak Discharge (m^3/sec)	Specific Discharge (m3/sec/km^2)	Creager's C
10	440	8	17
20	520	9	19
100	1,040	18	37
200	1,200	22	46
P.M.F.	1,890	35	72

P.M.F.: Probable Maximum Flood

Fig. 4.2.1 shows the hydrographs of the above probable floods.

P.M.F. of 1,890 m^3/s is taken as the spillway design flood which is required to safely be handled with the necessary freeboard by the spillway. On the one hand, 100-year flood is taken as the design flood of the energy dissipator, meaning that the dissipator will completely dissipate the flood discharge energy up to 100-year probable flood magnitude.

The above is based on the consideration that damages of energy dissipator if any or incomplete energy dissipation will not result in any fatal failure of the project and that about 100-year flood magnitude should be taken as its design flood from an economic aspect.

However, side walls of the energy dissipator should not be overtopped by P.M.F. since the overtopping may damage the main dam.

The Basic Design prepared the design for the spillway based on hydraulic analyses, which is shown in Fig. 4.2.2 as the original design of the spillway. In view that the spillway is a very important structure, and therefore, confirmations of its hydraulic details and adjustments where necessary are essential, a hydraulic model test of the spillway was carried out.

Details of results of the hydraulic model test are presented in the Spillway Hydraulic Model Test Report in detail.

Major items pointed out in the hydraulic model test of the spillway are as follows:

(1) Improvement of abutments of the overflow weir

Contracted flows at both abutments of the overflow weir would obstruct the discharge capacity of the overflow weir. Training walls with 12 m length should be provided towards the reservoir at the abutments on both banks, and the shape of upstream end of the wall should be circular with a radius of 4.5 m.

With the above improvement, the relationship between discharge coefficient (C) and overflow depth (H) is expressed as follows:

$$C = -0.0301 \cdot H^2 + 0.2645 \cdot H + 1.5827$$

At F.W.L. EL. 193.5 m (H=4.5 m), C is given as 2.16.

(2) Weir crest length

It is impossible to satisfy completely the necessary discharge capacity of the weir even after improvement of the abutments. As such, the length of overflow weir crest should be extended. If the length is extended by 2 m beyond the original length of 90 m, the discharge at F.W.L. EL. 193.5 m will be 1,897 m^3/sec as calculated below, satisfying the necessary capacity.

$$Q = C \cdot B \cdot H^{3/2}$$
$$= 2.16 \times 92.0 \times 4.5^{3/2} = 1{,}897 \text{ m}^3/\text{sec}$$

(3) Improvement of the transition channel

From the aspect of the Froude number which is desirable to be less than 0.5, the required height of the sill should be around 4.0 m. However, the sill height of 3.0 m is recommended to maintain a perfect overflow on the weir.

In order to enhance the effect of stilling of the sill, the sill should be provided downstream from the bend. A contracted flow occurs along the side wall of the bend on the right bank due to insufficient distance between the bend of the transition channel and the sill. It makes the overflow on the sill and flow regime in the chuteway undesirable. Thus, the length of the transition channel should be extended.

Further, the alignment of the transition channel should be of more smooth curve.

As a result of the tests, the proposed shape of the transition channel is given in Fig. 4.2.3 as the revised design. In this design, shock waves in the chuteway and eccentric hydraulic jump in the stilling basin disappear and adequate stilling effect is obtained.

(4) Modification of bottom slope of side channel

The tests confirmed that the flow regime would be satisfactory, when the elevation at upstream end of the side channel is about 1.0 m higher than the elevation of the sill crest. Since the sill crest is raised by 2.0 m, the elevation at the upstream end of the side channel is also raised by 2.0 m, resulting in the elevation of EL. 180.0 m at the upstream end of the side channel and the bottom slope of the side channel of 1 to 23 which is considered a reasonable value.

(5) Depth of stilling basin

For the discharge of 1,040 m^3/sec (100-year flood), the hydraulic jump in depth 9 m is apt to become submerged flow, and the flow regime in depth 7 m, is critical for the hydraulic jump. In the case of depth 8 m, the most desirable hydraulic jump was observed in the examination.

For the discharge of 1,890 m^3/sec (P.M.F), the hydraulic jump is not perfect and in turbulence for any depth. The hydraulic jump for depth 9.0 m and 8.0 m are fairly stable, while for depth 7.0 m super-critical flow appears in the stilling basin, and the hydraulic jump is considerably unstable.

As a result of the examination, it is reasonable that the depth of stilling basin be 8.0 m. Under such condition, the flow is in strong jump, and the water will overflow sometimes with intermittent fluctuation of the water surface due to turbulence. Hence, the side wall should be made more high to prevent overflow.

(6) Provision of chute block

By providing chute blocks at the entrance of the stilling basin, the hydraulic jump would be more stable. For the case of six chute blocks with height 2.0 m, width 2.5 m, intermittent fluctuation of water surface hardly appears and it is confirmed that the hydraulic jump is quite stable. Hence, the height of side wall could be minimized by adopting the chute blocks.

(7) Chuteway

The height of side walls in the chuteway should be 7.0 m so as to safely discharge P.M.F of 1,890 m^3/sec with a freeboard over 1.5 m.

It is found on the basis of the results of the model test that the design of the spillway should be as shown in Fig. 4.2.3. Hydraulic conditions for this final design are given in Figs. 4.2.4 to Fig. 4.2.9.

4.3 Structural Design of Spillway

4.3.1 Stability Analysis

4.3.1.1 General

The spillway is a side channel type with an open chuteway, having dimensions of about 30 m in width, about 364 m in length and 75 m in height.

The spillway is composed of the following components:

(i) Side channel
(ii) Chuteway
(iii) Stilling basin (Energy dissipator)

These structures are the reinforced concrete structure with anchor bars.

4.3.1.2 Design Value

The design values for examining the stability of the structures are determined on the basin of the field test results and/or the design standards and are summarized as follows:

(1) Unit weight

Material		Unit Weight (t/m^3)
Concrete		2.40
Water		1.00
Backfill material	(wet)	1.94
	(submerged)	1.23
Impervious core for dam embankment	(wet)	1.72
	(submerged)	0.80
Rock and riprap for dam embankment	(wet)	2.14
	(submerged)	1.37

(2) Internal friction angle (ϕ) and cohesion (C)

Material	ϕ (deg)	C (t/m^2)
Backfill material (Free Draining)	36	0
Foundation rock : Highly weathered	35	20
Slightly weathered	40	190
Impervious core for dam embankment	30	0
Rock and riprap for dam embankment	40	0

(3) Friction coefficient with concrete

Material	Coefficient
Concrete to concrete	0.65
Concrete to rock	0.55

(4) Modulus of elasticity (E) and Poisson's ratio (P)

Material		E (kg/cm^2)	P
Reinforced concrete	(SIG28 = 180 kg/cm^2)	2.4×10^5	0.2
	(SIG28 = 210 kg/cm^2)	2.55×10^5	0.2
Steel	(reinforcement bar)	2.1×10^6	0.3

Note: 1) SIG28 means compressive strength at the age of 28 days.
2) Reinforced concrete of SIG28 = 210 kg/cm^2 is applied only to spillway bridge.

(5) Seismic coefficient

Horizontal component	Kh	0.05
Vertical component	Kv	0

(6) Allowable stress

(A) Reinforced concrete

SIG28 (kg/cm^2)	Allowable stress (kg/cm^2)				
	Compression	Tension	Shearing*	Bond	Bearing
180	60	-	4, 8	14	54
210	70	-	4.25, 8.5	16	63

* The first value is for beam and second for slab.

(B) Steel (reinforcement bar)

Tensile stress (ultimate) 3,000 kg/cm^2

(C) Foundation rock

Bearing stress 100 t/m^2

4.3.1.3 Design Criteria

Design criteria for stability analyses of the spillway structure follow the standards for retaining walls and are summarized below:

(1) Body force

Dead load is a self weight of structure including weight of earth, water and others are as formulated below.

$$W = U_w \cdot V \text{ (or } Uw \cdot A)$$

where, W : dead load (t)

U_w : unit weight (t)

V : volume (in case of 3-dimensional calculation)

A : area (in case of 2-dimensional calculation)

Seismic force originated in self weight above is calculated by the following formula.

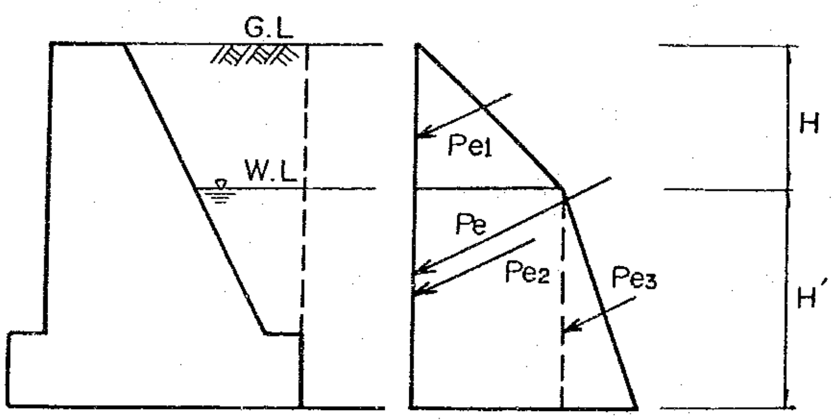
$$Ws = Kh \cdot W$$

where, Ws : seismic force (t)

Kh : seismic coefficient

(2) Earth pressure

Earth pressure acting on the structures is given as follows:



$$\begin{aligned} Pe &= Pe_1 + Pe_2 + Pe_3 \\ &= \frac{1}{2} K\gamma_{wet} H^2 + K\gamma_{sub} HH' + \frac{1}{2} K\gamma_{sub} H'^2 \end{aligned}$$

where, Pe : earth pressure (t)

K : coefficient of earth pressure

γ_{wet} : unit weight of earth in wet condition (t/m^3)

γ_{sub} : unit weight of earth in submerged condition (t/m^3)

H : height from ground level to water level (m)

H' : height below water level (m)

Coefficient of Coulomb's active earth pressure shown below is applied for the load of structural calculation.

- under normal condition

$$ka = \frac{\cos^2(\phi - \theta)}{\cos^2\theta \cdot \cos(\theta + \delta) \cdot \left(1 + \sqrt{\dfrac{\sin(\phi + \delta) \cdot \sin(\phi - \alpha)}{\cos(\theta + \delta) \cdot \cos(\theta - \alpha)}}\right)^2}$$

where, θ : inclination of the back with the vertical

ϕ : internal friction angle of backfill material

δ : angle of wall friction between the back and backfill material

α : surface slope of back fill with the horizontal

- under seismic condition

$$kae = \frac{\cos^2(\phi - \theta o - \theta)}{\cos\theta o \cdot \cos^2\theta \cdot \cos(\delta + \theta + \theta o) \cdot \left(1 + \sqrt{\dfrac{\sin(\phi + \delta) \cdot \sin(\phi - \alpha - \theta o)}{\cos(\delta + \theta + \theta o) \cdot \cos(\theta - \alpha)}}\right)^2}$$

where, θo : combined angle = $\tan^{-1} Kh/(1-Kv)$

Kh : horizontal component of seismic coefficient

Kv : vertical component of seismic coefficient

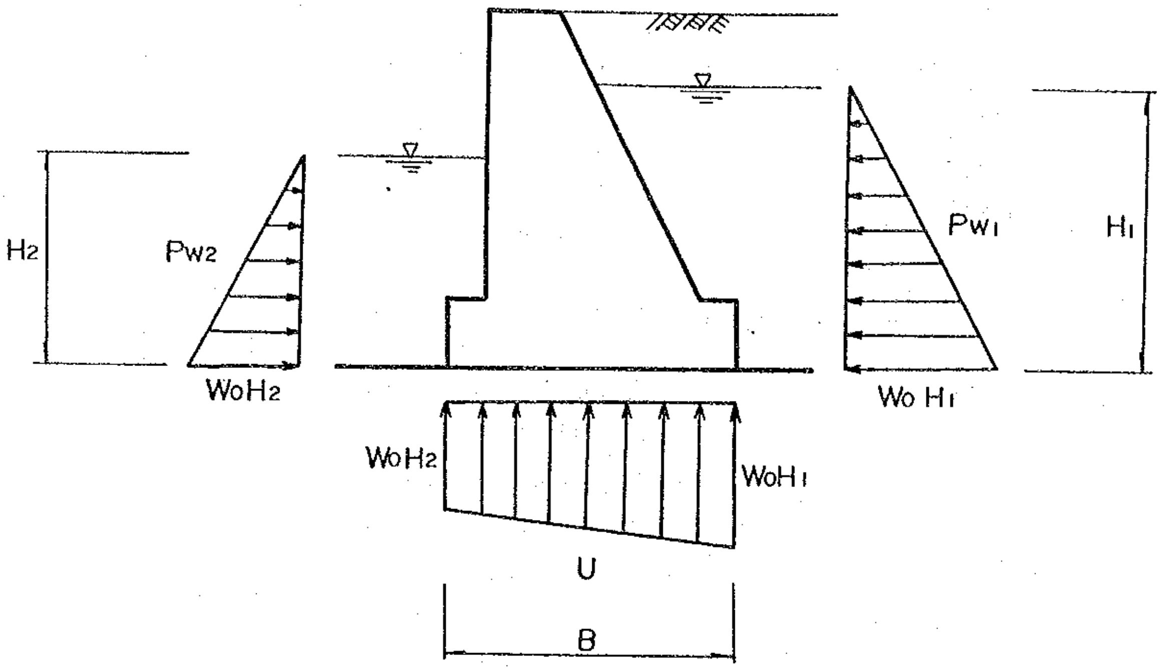
θ, ϕ, δ and α: above-mentioned

The following values are adopted for wall friction angle (d) in accordance with conditions:

Condition	δ
Stability calculation (earth to earth), normal	ϕ
Stability calculation (earth to earth), seismic	$\phi/2$
Stress calculation (earth to concrete), normal	$\phi/3$
Stress calculation (earth to concrete), seismic	0

(3) Hydraulic pressure

Hydraulic pressures act on the structure are as follows:



Static water pressure $P_{w1} = \frac{1}{2} Wo\, H_1{}^2$

$P_{w2} = \frac{1}{2} Wo\, H_2{}^2$

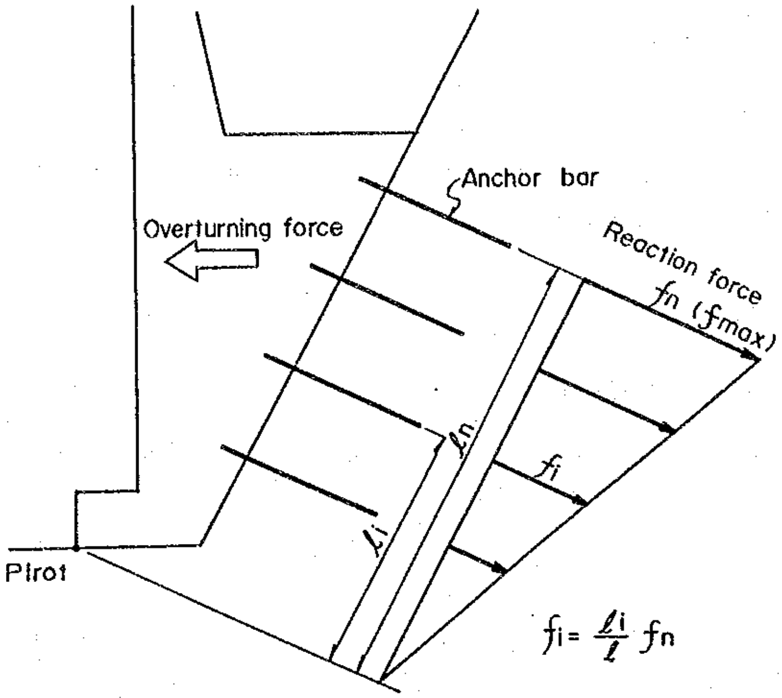
where, Pw1, Pw2 : Static water pressure (t)

U : uplift pressure (t)

Wo : unit weight of water (t/m^3)

(4) Resisting force of anchor bar

As a concrete structure is assumed to be rigid, the reaction force to be caused in a series of anchor bars is proportional to the distance from overturning pivot as follows:



The expective maximum resisting force in the uppermost anchor bar is

$$f_{max} = \sigma_{su} \cdot As$$

where, f_{max} : maximum resisting force (kg)

σ_{su} : tensile stress (ultimate) of steel bar (kg/cm^2)

As : sectional area of steel bar (cm^2)

Anchor bar	σ_{su} (kg/cm^2)	As (cm^2)	fmax (t)
D25	3,000	5.067	15.201
D29	3,000	6.424	19.272
D32	3,000	7.942	23.826

Total resisting moment is calculated by the following equation.

$$M_{ra} = \sum_{i=1}^{n} fi \cdot li = \sum_{i=1}^{n} \frac{fn}{ln} \cdot li^2$$

where, M_{ra} : total resisting moment of anchor bars (t.m)

fi : resisting force of each anchor bar (t)

li : distance from overturning pivot to anchor bar (m)

fn : resisting force of uppermost anchor bar (t)

ln : distance from overturning pivot to uppermost anchor bar (m)

n : numbers of anchor bars

(5) Stability calculation

(A) Stability against sliding

Stability against sliding is judged by the safety factor calculated below.

$$Fs = \frac{f\Sigma V + \tau A + Ha}{\Sigma H}$$

where, Fs : safety factor for sliding

ΣV : sum of vertical forces (t)

ΣH : sum of horizontal forces (t)

τ : shearing strength (= 20 t/m2 from result of soil test)

A : area of horizontal base (m2)

Ha : Resisting force of anchor bar (t) (Horizontal component)

(B) Stability against overturning

Stability against overturning is examined by the following equation.

$$Fs = \frac{\Sigma Mr}{\Sigma Mt}$$

where, Fs : safety factor for overturning

ΣMr : sum of resisting moment (t.m)

ΣMt : sum of overturning moment (t.m)

(C) Stability against bearing capacity of foundation

Bearing stress of foundation is calculated as below:

$$q = \frac{\Sigma V}{B} < qa$$

where, q : maximum bearing stress of foundation (t/m^2)

qa : allowable bearing stress of foundation (t/m^2)

ΣV : sum of vertical force (t)

B : projected base (m)

(D) Safety factor requirement

Condition of stability	Loading condition	
	Normal	Extreme*
Safety factor for sliding	1.5	1.2
Safety factor for overturning (for retaining wall of reinforced type)	1.5	1.2

* Flood or seismic conditions

4.3.1.4 Analysis

(1) Analysis for Side Walls

Stability analyses for the side walls of the side channel, transition portion, chuteway and stilling basin are made in Table 4.3.1 to Table 4.3.12.

Cases and loading conditions of the stability analyses are summarized as follows (As for section name, reference is made to figures of spillway structure design):

Section of Wall	Loading Case No.	Conditions of Loading
(a) Side Channel: (Section C-C)	Case I	- Normal condition - It is assumed that water from the reservoir comes to the back side of the wall under the condition of side channel empty. - Water pressure above HWL of the reservoir is considered to be released through drain system.
	Case II	- Seismic condition - This case considers that under the condition of Case I above.

(b)	Transition portion (Section G-G)	Case I	-	Normal condition
			-	This case also assumes that water pressure from the reservoir acts to the back side of the wall.
		Case II	-	Seismic condition
			-	The case considers the seismic force acts to the wall under the condition of Case I above.
(c)	Chuteway: (Section A-A)	Case I	-	Normal condition
			-	The case does not consider any water pressure from back side of the wall since water will be drained by the drain system provided in the back side of the wall.
		Case II	-	Seismic condition
			-	Seismic force is loaded under the above Case I.
(d)	Stilling basin: (Section E-E)	Case I	-	Normal condition
			-	Water levels are considered to be balanced at EL. 120.0 m between the stilling basin side and back side of the wall.
		Case II	-	Seismic condition
			-	Seismic force is loaded under the above Case I.

Case III - Flood condition

- Water level in the stilling basin side is set at EL. 120.0 m. (Hydraulic model test indicates the water level in the stilling basin does not lower below EL. 120.0 m at flooding time)

- Water level in back side of the wall is set at EL. 126.0 m which is the uppermost level in the back side of the wall at flooding time. Water above EL. 126.0 m will be drained through weep holes.

(e) Stilling basin (Section I-I)

Case I - Normal condition

- No water pressure in the stilling basin side is assumed with earth and water pressure in back side of the wall.

Case II - Seismic condition

- Seismic force is loaded under the above Case I.

Case III - Flood condition

- FWL EL. 132.0 m up to which water level will rise is assumed in the stilling basin side.

(2) Examination on Length of Anchor Bar in Side Walls

Anchor bars are required to be examined on the shearing strength around bore hole and on the bond strength between anchor bar and mortar.
Then, necessary length to withstand the acting force has to be provided for the anchor bar.

Assuming the bore hole diameter of 64 mm and reinforcement bar diameter of D29 mm, the necessary length of anchor bar is examined as follows :

$F_1 = \pi \cdot D_1 \cdot \tau_1 \cdot l = 3.14 \times 6.4 \times 2 \times 100 = 4{,}019$ kg/m
$F_2 = \pi \cdot D_2 \cdot \tau_2 \cdot l = 3.14 \times 2.9 \times 14 \times 100 = 12{,}748$ kg/m

where, F_1 : shearing strength around bore hole for anchor bar per meter (kg/m)

F_2 : bond strength between anchor bar and mortar for anchor bar per meter (kg/m)

D_1 : diameter of bore hole (cm)

D_2 : diameter of anchor bar (cm)

τ_1 : shearing strength around bore hole (2 kg/cm^2 for highly weathered rock)

τ_2 : bond strength between anchor bar and mortar (14 kg/cm^2), and

l : length of anchor bar (100 cm)

As seen above, the shearing strength around bore hole is much less than the bond strength between anchor bar and mortar. Thus, the necessary length of anchor bar is determined on the basis of the shearing strength around bore hole.

The necessary length of anchor bar is calculated by the following equation :

$$l = l_0 + F_{max} / (\pi \cdot D_1 \cdot \tau_1)$$

where, l : necessary length of anchor bar (m),

F : maximum tensile force of anchor bar (kg),

D_1 : diameter of bore hole (64×10^{-3} m),

τ_1 : shearing strength around bore hole (2×10^4 kg/m^2), and

l_0 : loose depth of rock due to excavation work (0.5m)

The necessary length of anchor bar will be as follows :

For D29 anchor bar :

$$l = 0.5 + 19.272 \times 10^3 / (3.14 \times 64 \times 10^{-3} \times 2 \times 10^4)$$
$$= 5.295 \text{ (m)}$$

For D25 anchor bar :

$$l = 0.5 + 15.201 \times 10^3 / (3.14 \times 64 \times 10^{-3} \times 2 \times 10^4)$$
$$= 4.282 \text{ (m)}$$

(3) Examination on Anchor Bar in Side Channel Slab

The side channel slab will be subject to uplift pressure under the condition of side channel empty, requiring anchor bars to withstand the uplift pressure. The anchor bars of D29 will be provided under the slab at 1.0 m interval.

Tensile force acting to anchor bar, tensile stress of anchor bar and necessary length of anchor bar after the flood (P.M.F) are examined as follows :

Tensile force acting to one anchor bar :

Body force : $W = 1.0m \times 1.0m \times 1.5m \times 2.4t/m^3 = 3.6\ t$

Uplift : $U = (193.5 - 176.0)t/m^2 \times 1.0m \times 1.0m = 17.5\ t$

Acting Force : $F = U - W = 17.5 - 3.6 = 13.9\ t$

Tensile stress of anchor bar :

$\sigma_s = F/As = 13{,}900\ kg/ 6.602\ cm^2 = 2{,}105\ kg/cm^2$

$F_s = 3{,}000 / 2{,}105 = 1.43 > 1.2$

Necessary length of anchor bar :

The shearing strength around bore hole determines the necessary length of anchor bar as follows :

$$l = l_0 + F / (\pi \cdot D_1 \cdot \tau_1)$$
$$= 0.5 + 13{,}900 / (3.14 \times 64 \times 10^{-3} \times 2 \times 10^4)$$
$$= 3.96\ m$$

(4) Examination on Anchor Bar in Stilling Basin Slab

The stilling basin slab will be subject to the remaining uplift pressure after floods. Hence, anchor bars are provided to withstand the above uplift pressure. D32 anchor bars will be provided at 1.5m interval. Tensile force acting to the anchor bar, its tensile stress and necessary length of the anchor bar are examined below :

Tensile force acting to one anchor bar :

The loading condition considers the following case that is, the water level in the stilling basin will rise up to EL.132.0m during the flood (P.M.F). The water level will lower down to EL.120.0m (the top elevation of end sill) after the flood. However, the uplift pressure of EL. 132.0m will remain under the stilling basin slab.

Thus,

Body force : $W = (2.4 \times 1.5 + 1.0 \times 8.0) \times 2.25 = 26.10\ t$

Uplift : $U = (132 - 112) \times 2.25 = 45.0\ t$

Acting Force : $F = U - W = 45.0 - 26.10 = 18.9\ t$

Tensile stress of anchor bar :

$\sigma_s = F / As = 18{,}900kg / 8.038\ cm^2 = 2{,}351\ kg/cm^2$

$F_s = 3{,}000 / 2{,}351 = 1.28 > 1.2$

<u>Necessary length of anchor bar</u> :

The shearing strength around bore hole is the factor to determine the necessary length of anchor bar which is calculated as follows :

$$l = l_0 + F / (\pi \cdot D_1 \cdot \tau_1)$$

$$= 0.5 + 18{,}900 / (3.14 \times 64 \times 10^{-3} \times 2 \times 10^4)$$

$$= 4.7 \text{ m}$$

(5) Analysis for Overflow Weir

The stability analysis for the spillway overflow weir is made in Table 4.3.13 to Table 4.3.15.

The analysis is made for the following three (3) loading condition :

(i) Normal condition :
This case assumes that the reservoir water level is at H.W.L EL.189.0 m and that the base of the weir is subject to the full uplift pressure of 5.5 m (EL.189.0 - EL. 183.5).

(ii) Seismic condition :
In this case, the seismic force acts to the weir under the above normal condition.

(iii) After the flood (P.M.F) :
This case assumes that the base of the weir is subject to the remaining uplift pressure of 10.0 m (EL.193.5 - EL.183.5) after the flood (P.M.F).

As seen in the analyses, anchor bars will be required to withstand the loading condition after the flood (P.M.F). D29 anchor bars will be provided at 2.0 m interval. Necessary length of anchor bar which is determined by the shearing strength around bore hole is $l = 5.295$ m (= 5.5m).

4.3.2 Stress Analysis of Reinforced Concrete

Reinforced concrete stress analyses for the spillway side walls are made in Table 4.3.16 to Table 4.3.20, based on bending moments, shear forces and axial forces acting to the walls, of which calculations are given in the Data Book.

In the analyses, the allowable stresses are increased by 50 % for such tentative and rare cases as the seismic and flood conditions in accordance with the standard.

As seen in the analyses, the reinforcement bar arrangement of D19 @ 200 which is considered minimum requirement of the reinforcement bars for the structure will withstand the acting forces.

4.4 Spillway Bridge

4.4.1 General

The spillway structure is located on the left abutment of the damsite. The spillway is a side channel type with an open chuteway.

A bridge is required to cross this open channel at the dam crest level of EL. 196.0 m. The total length of the bridge is 29.8 m. Its span is 29.0 m in terms of the length between two supports. From an economical point of view, the bridge is provided with an effective width of 6.0 m which is the minimum width for traffic to pass each other.

The bridge is designed as the second-order bridge for which the design car load is specified to be 14.0 ton with the following consideration: that is, although the dam design is based on a consideration that the possibility of utilization of dam crest as a traffic road is not ruled out, its road will not be a trunk road but a branch road.

The bridge is designed as a composite girder in which the acting loads are borne by both the girder and floor slab. This section presents the main parts of the design analyses consisting of those on the composite girder. Reference is made to the Data Book with regard to analyses on other detailed parts.

4.4.2 Design Condition

The design conditions of the spillway bridge which is based on the Specification for Road Bridge by Japan Society of Road are summarized below:

- Total length .... 29.8 m
- Span .... 29.0 m
- Class of bridge .... Second-order
- Bridge type .... Composite girder
- Total width .... 7.2 m
- Effective width .... 6.0 m

- Pavement thickness .... 50 mm
- Floor slab thickness .... 180 mm
- Allowable stress:
  - Compressive strength of concrete .... 77.1 $kg/cm^{2(*)}$
  - Tensile stress of reinforcement steel bar (SS41) .... 1,400 $kg/cm^{2(**)}$
  - Tensile and compressive stress of shape steel (SM50Y) .... 2,100 $kg/cm^{2(**)}$
  - Shearing stress of reinforcement bar (SS41).................... 800 $kg/cm^{2(**)}$
  - Shearing stress of shape steel (SM50Y)......................... 1,200 $kg/cm^{2(**)}$

(*) It is specified that the concrete strength of base slab should not be less than 270 kg/cm^2 and that its allowable stress be 1/3.5 times the above concrete strength.

(**) These allowable stresses also follow the Specification for Road Bridge by Japan Society of Road.

- Increase of allowable stress for concrete:
  The allowable compressive strength of concrete is allowed to increase by 15% in the case that the temperature difference between base slab and steel girder is taken into consideration.

- Increase of allowable stress for shape steel:
  The Specification allows to increase the allowable stress of shape steel as follows:

	Loading Condition		Increase of Allowable Stress (%)
(1)	Main loading except the effect due to creep and drying shrinkage		0
(2)	Main loading including the effect due to creep and drying shrinkage	Compressive side	15
		Tension side	0
(3)	(2) + Effect due to temperature difference between base slab and steel girder	Compressive side	30
		Tension side	15
(4)	Construction stage	Compressive side	25
		Tension side	25

The design allows the above increase of the allowable stress.

- Temperature difference between base slab and steel girder .... 10°C
- Seismic coefficient 0.05
- Wind speed .... 40 m/sec.

4.4.3 Design Calculation

(1) Analysis of composite girder

Fig. 4.4.1 shows the frame plan, section and dimensions of the composite girder. As seen, the composite girder of bridge consists of three (3) main girders (G-1, G-2, G-3) and seven (7) crossing frames.

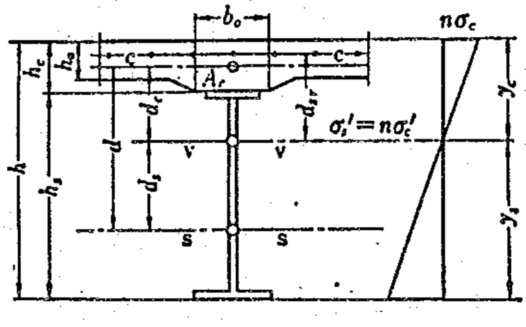
Fig. 4.4.2 and 4.4.3 show the loading conditions. Fig. 4.4.2 shows the loading condition before compounding the base slab and steel girders, and Fig. 4.4.3 shows that after compounding. As mentioned, the bridge is designed as the second-order bridge for which the car load of 14 ton is taken into consideration as the live load. The Specification mentions for this condition that the design should consider the live load and uniform load of 3.5 ton/m and 0.245 ton/m^2 respectively with a width of 5 m (A half of the above is imposed on the remaining width). Thus, the loading condition considers the above live load in addition to the dead load as seen in Figures.

The analysis of the composite girder is made in Table 4.4.1 to 4.4.12. Table 4.4.1 to 4.4.3 presents the bending moments and shearing forces to occur in the frames due to the loadings. Table 4.4.4 to 4.4.12 analyze the stresses to arise in the composite girder. Fig. 4.4.4 to 4.4.6 summarize all results of analyses on the composite girder. As seen in Fig. 4.4.4 to 4.4.6, all the stresses to be caused by the loadings are within the allowable stresses, and the composite girder will safely withstand the loadings.

(2) Major formula used for stress analysis

Major formula used for the stress analysis are as follows:

<u>Stress due to the bending moment:</u>



where,

v - v : Neutral axis of composite girder

c - c : Center of gravity of concrete section

s - s : Center of gravity of steel girder

Ac : Sectional area of concrete section

As : Sectional area of steel girder section

n = Es/Ec = Young modulus ratio between steel and concrete (n = 7.0)

The stress are calculated as follows:

$$\sigma_c = M \cdot y_c/I_v, \qquad \sigma_c' = M \cdot y_c'/I_v$$

$$\sigma_s = n \cdot M \cdot y_s/I_v, \qquad \sigma_s' = n \cdot \sigma_c'$$

where,

σ_c : Stress in upper edge of concrete

σ_c' : Stress in lower edge of concrete

σ_s : Stress in lower edge of steel

σ_s' : Stress in upper edge of steel

I_v : Moment of inertia of area for neutral axis of composite girder (v-v)

$$I_v = I_s + \frac{1}{n} \cdot I_c + A_s \cdot d_s^2 + \frac{1}{n} \cdot A_c \cdot d_c^2$$

I_s : Moment of inertia of area for axis of center of gravity of steel girder (s-s)

I_c : Moment of inertia of area for axis of center of gravity of concrete (c-c)

M : Bending moment

Stress due to creep:

The stress due to creep is obtained by the following formula:

$$\sigma_{cu} = \frac{N_c}{A_c} - \frac{M_c}{I_c} \cdot Y_{cu}$$

$$\sigma_{cl} = \frac{N_c}{A_c} - \frac{M_c}{I_c} \cdot Y_{cl}$$

$$\sigma_{su} = \frac{N_s}{A_s} - \frac{M_s}{I_s} \cdot Y_{su}$$

$$\sigma_{sl} = \frac{N_s}{A_s} - \frac{M_s}{I_s} \cdot Y_{sl}$$

$$N_c = -N_s = -N_{co}(1 - e^{-F\varphi_1})$$

$$N_{co} = -N_{so} = -\frac{d_c A_c}{nI_v} \cdot M_o$$

$$M_s = d \cdot N_c$$

$$M_c = \frac{H}{1 - F}(e^{-F\varphi_1} - e^{-\varphi_1})$$

where,

σ_{cu} : Stress due to creep at upper edge of concrete

σ_{cl} : Stress due to creep at lower edge of concrete

σ_{su} : Stress due to creep at upper edge of steel girder

σ_{sl} : Stress due to creep at lower edge of steel girder

N_c and N_s: Incremental axial force due to creep acting to center of gravity of concrete slab and steel girder, respectively

M_c and M_s: Incremental bending moment due to creep acting to center of gravity of concrete slab and steel girder, respectively

N_{co} and N_{so}: Axial force at time T=0 acting to center of gravity of concrete slab and steel girder, respectively

M_{co} and M_{so}: Bending moment at time T=0 acting to center of gravity of concrete slab and steel girder, respectively

y_{cu} : Distance from neutral axis of concrete slab to its upper edge

y_{cl} : Distance from neutral axis of concrete slab to its lower edge

y_{su} : Distance from neutral axis of steel girder to its upper edge

y_{sl} : Distance from neutral axis of steel girder to its lower edge

φ_1 : Creep coefficient from time T=0 to T=∞ ($\varphi_1 = 2.0$ is specified to be used in the case of the composite girder)

M_o : Bending moment acting to composite girder

$$F = \frac{1}{1 + \frac{A_c}{nA_s} + \frac{A_c d^2}{I_c + nI_s}}$$

$$H = \frac{I_c}{nI_s} d F N_{co}$$

<u>Stress due to temperature difference (between concrete slab and steel girder)</u>:

The stress due to temperature difference between concrete slab and steel girder is obtained by the following formula:

$$\pm\sigma_{cu} = \frac{N_c}{A_c} - \frac{M_c}{I_c} \cdot Y_{cu}$$

$$\pm\sigma_{cl} = \frac{N_c}{A_c} + \frac{M_c}{I_c} \bullet Y_{cl}$$

$$\pm\sigma_{su} = \frac{N_s}{A_s} - \frac{M_s}{I_s} \bullet Y_{su}$$

$$\pm\sigma_{sl} = \frac{N_s}{A_s} + \frac{M_s}{I_s} \bullet Y_{sl}$$

$$N_c = -N_s = \frac{\alpha t E_s}{\frac{n}{A_c} + \frac{1}{A_s} + \frac{nd^2}{I_c + nI_s}}$$

$$Mc = N_c d \frac{I_c}{I_c + nI_s}$$

$$M_s \bullet N_c d \frac{nI_c}{I_s + nI_s}$$

where,

σ	:	Incremental stress due to temperature difference
α	:	(1.2×10^{-5})
t	:	Temperature difference between concrete slab and steel girder (10°C is specified to be used as standard)
N_c	:	Incremental axial force due to temperature difference acting to center of gravity of concrete slab
M_c	:	Incremental bending moment due to temperature difference acting to center of gravity of concrete slab
M_s	:	Incremental bending moment due to temperature difference acting to center of gravity

Stress due to shrinkage of concrete slab:

The stress due to shrinkage of concrete slab is obtained by the following equations:

$$\sigma_{su} = \frac{N_s}{A_s} - \frac{M_c}{I_c} \bullet Y_{su}$$

$$\sigma_{sl} = \frac{N_s}{A_s} + \frac{M_s}{I_s} \bullet Y_{sl}$$

$$\sigma_{cu} = \frac{N_c}{A_c} - \frac{M_c}{I_c} \bullet Y_{cu}$$

$$\sigma_{cl} = \frac{N_c}{A_c} + \frac{M_c}{I_c} \bullet Y_{cl}$$

$$N_s = N_c = \frac{\varepsilon_s \cdot E_s}{\dfrac{n_\varphi}{A_c} + \dfrac{1}{A_s} + \dfrac{n_\varphi d^2}{I_c + n_\varphi I_s}}$$

$$Ms = \frac{n_\varphi I_s}{I_c + n_\varphi I_s} N_c d$$

$$Mc = \frac{I_c}{I_c + n_\varphi I_s} N_c d$$

$$n_\varphi = n\left(1 + \frac{\varphi_2}{2}\right) \qquad (\varphi_2 = 4.0)$$

where,

σ : Incremental stress due to shrinkage of concrete slab

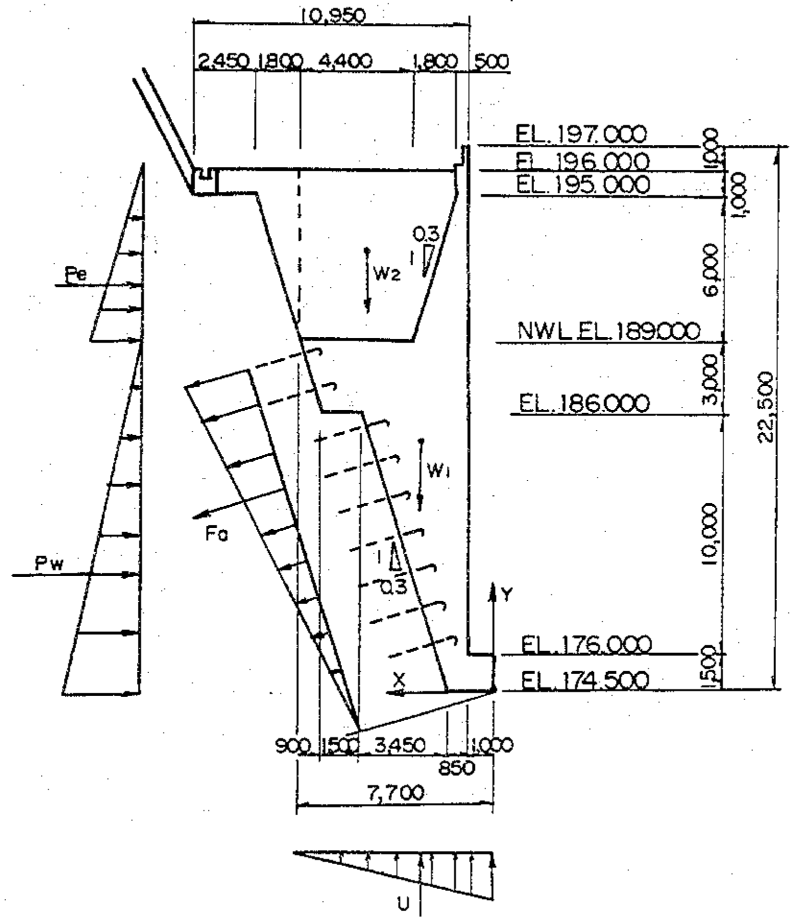
ε_s : Final rate of shrinkage (18×10^{-5})

N_c and M_c: Incremental axial force and bending moment due to shrinkage acting to center of gravity of concrete slab

N_s and M_s: Incremental axial force and bending moment due to shrinkage acting to center of gravity of steel girder

TABLES

Table 4.3.1: STABILITY ANALYSIS OF SPILLWAY STRUCTURE
(Side Channel Wall, Section C-C, Loading Case I)



SECTION AND LOADING CONDITION

Load		V (t)	H (t)	X (m)	Y (m)	Mv (t.m)	MH (t.m)
Body force	W1	141.030		2.848		401.645	
Earth force	W2	73.340		4.955		363.419	
Water pressure	Pw		105.125		4.833		508.104
Earth pressure	Pe		11.407		16.833		192.023
Uplift	U	-55.825		2.567		-143.284	
Total		158.545	116.532			621.779	700.128

Max. Resisting Force of Anchor Bar: Fa = 66.093 (t) (1.5m pitch)
- Vertical component: Va = 18.992 (t)
- Horizontal component: Ha = 63.306 (t)

Max. Resisting Moment of Anchor Bar: Ma = 713.008 (t.m)

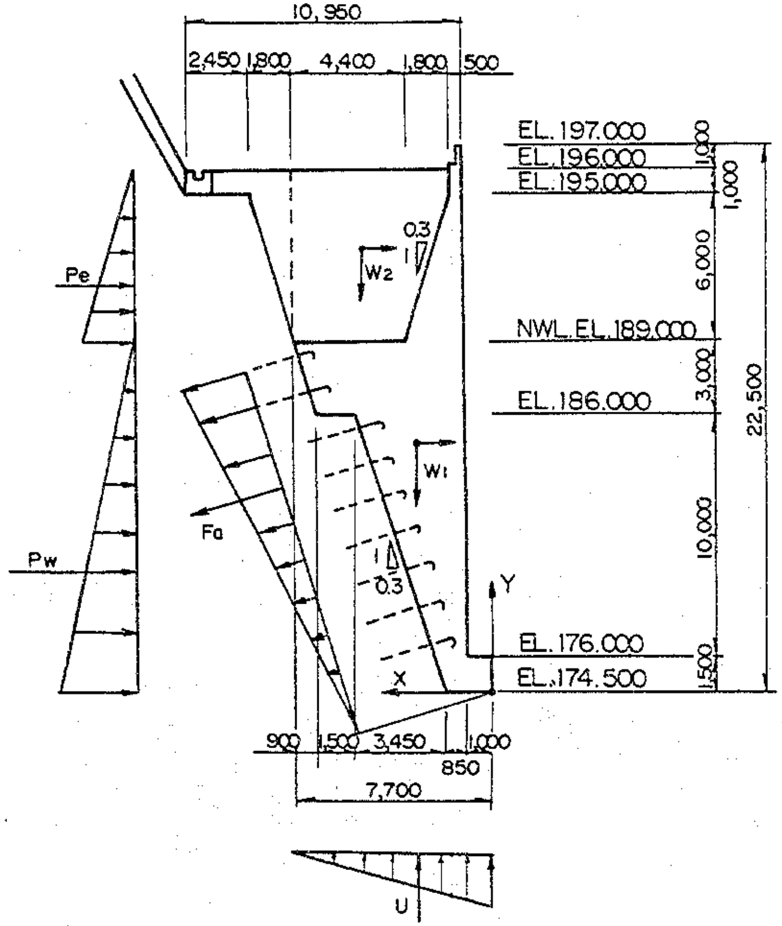
Safety factor for sliding: $Fs = \dfrac{f \cdot \Sigma V + \tau \cdot A + Ha}{\Sigma H} = \dfrac{0.55 \times 158.545 + 20 \times 3.35 + 63.306}{116.532} = 1.87 > 1.5$

Safety factor for overturning: $Fs = \dfrac{\Sigma Mr}{\Sigma Mt} = \dfrac{621.779 + 713.008}{700.128} = 1.906 > 1.5$

Safety for bearing: $q = \dfrac{\Sigma V}{B} = \dfrac{158.545 + 18.992}{7.7} = 23.06\ t/m^2 < 100\ t/m^2$

Note: As for Section name, see Figures of spillway structure design.

Table 4.3.2: STABILITY ANALYSIS OF SPILLWAY STRUCTURE
(Side Channel Wall, Section C-C, Loading Case II)



SECTION AND LOADING CONDITION

Load		V (t)	H (t)	X (m)	Y (m)	Mv (t.m)	MH (t.m)
Body force	W1	141.030	7.052	2.848	10.312	401.645	72.713
Earth force	W2	73.340	3.667	4.955	18.213	363.419	66.788
Water pressure	Pw		105.125		4.833		508.104
Earth pressure	Pe		12.469		16.833		209.889
Uplift	U	-55.825		2.567		-143.284	
Total		158.545	128.312			621.779	857.494

Max. Resisting Force of Anchor Bar: Fa = 66.093 (t) (1.5m pitch)
- Vertical component: Va = 18.992 (t)
- Horizontal component: Ha = 63.306 (t)

Max. Resisting Moment of Anchor Bar: Ma = 713.008 (t.m)

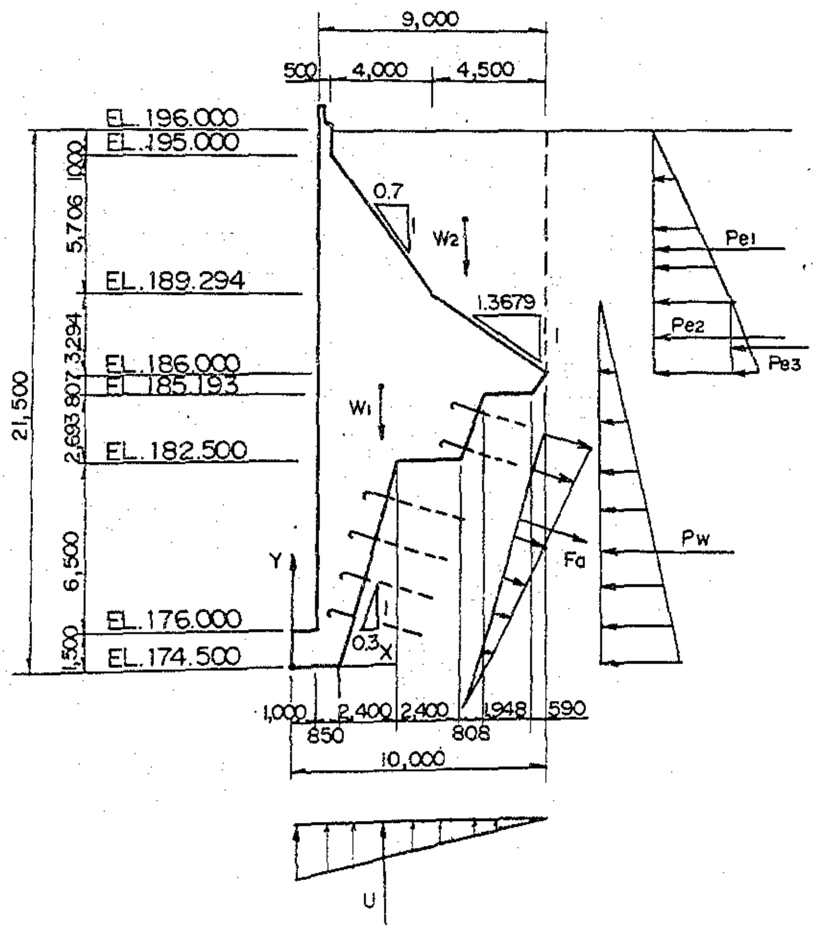
Safety factor for sliding: $Fs = \frac{f \cdot \Sigma V + \tau \cdot A + Ha}{\Sigma H} = \frac{0.55 \times 158.545 + 20 \times 3.35 + 63.306}{128.312} = 1.7 > 1.2$

Safety factor for overturning: $Fs = \frac{\Sigma Mr}{\Sigma Mt} = \frac{621.779 + 713.008}{857.494} = 1.56 > 1.2$

Safety for bearing: $q = \frac{\Sigma V}{B} = \frac{158.545 + 18.992}{7.7} = 23.06\ t/m^2 < 100\ t/m^2$

Note: As for Section name, see Figures of spillway structure design.

Table 4.3.3: STABILITY ANALYSIS OF SPILLWAY STRUCTURE
(Transition Wall, Section G-G, Loading Case I)



SECTION AND LOADING CONDITION

Load		V (t)	H (t)	X (m)	Y (m)	Mv (t.m)	MH (t.m)
Body force	W1	187.773		3.526		662.103	
Earth force	W2	84.342		6.763		570.370	
Water pressure	Pw		105.125		4.833		508.104
Earth pressure	Pe1		12.523		16.833		210.802
Earth pressure	Pe2		2.496		13.000		32.451
Earth pressure	Pe3		1.070		12.500		13.373
Uplift	U	-72.500		3.333		-241.667	
Total		199.615	121.214			990.807	764.730

Max. Resisting Force of Anchor Bar: Fa = 71.583 (t) (1.0m pitch)
- Vertical component: Va = 20.569 (t)
- Horizontal component: Ha = 68.564 (t)

Max. Resisting Moment of Anchor Bar: Ma = 535.917 (t.m)

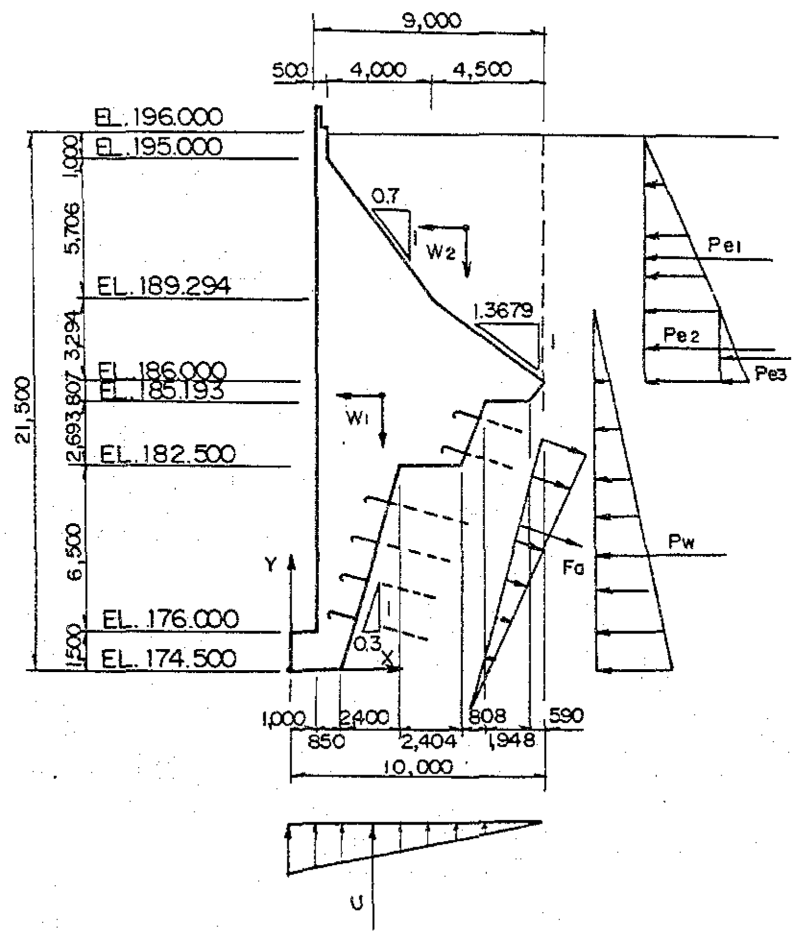
Safety factor for sliding: $Fs = \frac{f \cdot \Sigma V + \tau \cdot A + Ha}{\Sigma H} = \frac{0.55 \times 199.615 + 20 \times 6.202 + 68.564}{121.214} = 2.5 > 1.5$

Safety factor for overturning: $Fs = \frac{\Sigma Mr}{\Sigma Mt} = \frac{990.807 + 535.917}{764.730} = 2.00 > 1.5$

Safety for bearing: $q = \frac{\Sigma V}{B} = \frac{199.615 + 20.569}{10.0} = 22.0\ t/m^2 < 100\ t/m^2$

Note: As for Section name, see Figures of spillway structure design.

Table 4.3.4: STABILITY ANALYSIS OF SPILLWAY STRUCTURE
(Transition Wall, Section G-G, Loading Case II)



SECTION AND LOADING CONDITION

Load		V (t)	H (t)	X (m)	Y (m)	Mv (t.m)	MH (t.m)
Body force	W1	187.773	9.389	3.526	10.866	662.103	102.013
Earth force	W2	84.342	4.217	6.763	17.837	570.370	75.221
Water pressure	Pw		105.125		4.833		508.104
Earth pressure	Pe1		14.025		16.833		236.091
Earth pressure	Pe2		2.796		13.000		36.344
Earth pressure	Pe3		1.198		12.500		14.977
Uplift	U	-72.500		3.333		-241.667	
Total		199.615	137.091			990.807	972.750

Max. Resisting Force of Anchor Bar: Fa = 71.583 (t) (1.0m pitch)
- Vertical component: Va = 20.569 (t)
- Horizontal component: Ha = 68.564 (t)

Max. Resisting Moment of Anchor Bar: Ma = 535.917 (t.m)

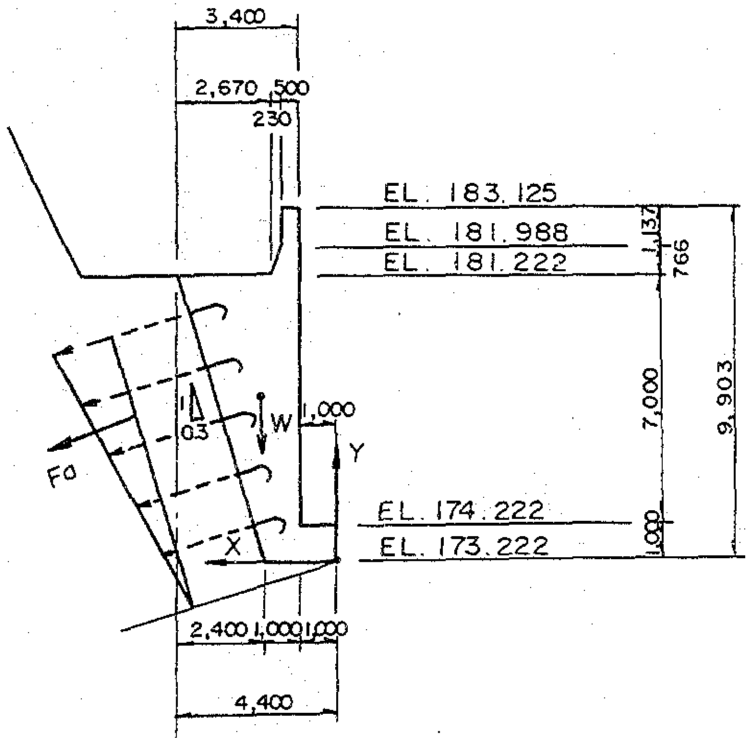
Safety factor for sliding: $Fs = \frac{f \cdot \Sigma V + \tau \cdot A + Ha}{\Sigma H} = \frac{0.55 \times 199.615 + 20 \times 6.202 + 68.564}{136.750} = 2.2 > 1.2$

Safety factor for overturning: $Fs = \frac{\Sigma Mr}{\Sigma Mt} = \frac{990.807 + 535.917}{972.750} = 1.57 > 1.2$

Safety for bearing: $q = \frac{\Sigma V}{B} = \frac{199.615 + 20.569}{10.0} = 22.0\ t/m^2 < 100\ t/m^2$

Note: As for Section name, see Figures of spillway structure design.

Table 4.3.5: STABILITY ANALYSIS OF SPILLWAY STRUCTURE
(Chuteway Side Wall, Section A-A, Loading Case I)



SECTION AND LOADING CONDITION

Load		V (t)	H (t)	X (m)	Y (m)	Mv (t.m)	MH (t.m)
Body force	W1	47.135		2.073		97.700	
Earth force	W2	0				0	
Water pressure	Pw	·	0				0
Earth pressure	Pe		0				0
Uplift	U	0				0	
Total		47.135	0			97.700	0

Max. Resisting Force of Anchor Bar: Fa = 36.709 (t) (1.5m pitch)
- Vertical component: Va = 10.548 (t)
- Horizontal component: Ha = 35.160 (t)

Max. Resisting Moment of Anchor Bar: Ma = 196.415 (t.m)

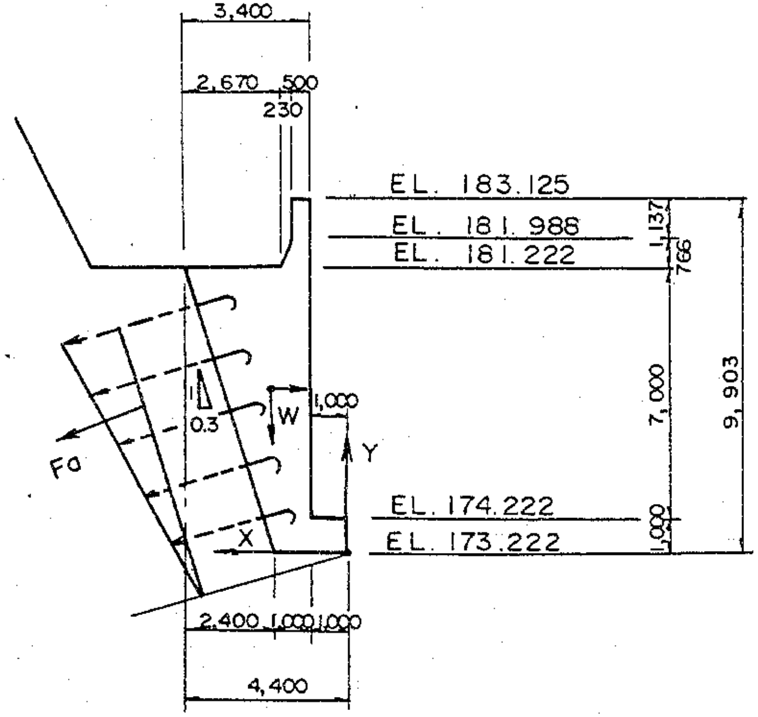
Safety factor for sliding: $Fs = \frac{f \cdot \Sigma V + \tau \cdot A + Ha}{\Sigma H} = \infty > 1.5$

Safety factor for overturning: $Fs = \frac{\Sigma Mr}{\Sigma Mt} = \infty > 1.5$

Safety for bearing: $q = \frac{\Sigma V}{B} = \frac{47.135 + 10.548}{4.4} = 13.1\ t/m^2 < 100\ t/m^2$

Note: As for Section name, see Figures of spillway structure design.

Table 4.3.6: STABILITY ANALYSIS OF SPILLWAY STRUCTURE
(Chuteway Side Wall, Section A-A, Loading Case II)



SECTION AND LOADING CONDITION

Load		V (t)	H (t)	X (m)	Y (m)	Mv (t.m)	MH (t.m)
Body force	W1	47.135	2.357	2.073	4.733	97.700	11.153
Earth force	W2	0				0	
Water pressure	Pw		0				0
Earth pressure	Pe		0				0
Uplift	U	0				0	
Total		47.135	2.357			97.700	11.153

Max. Resisting Force of Anchor Bar: Fa = 36.709 (t) (1.5m pitch)
- Vertical component: Va = 10.548 (t)
- Horizontal component: Ha = 35.160 (t)

Max. Resisting Moment of Anchor Bar: Ma = 196.415 (t.m)

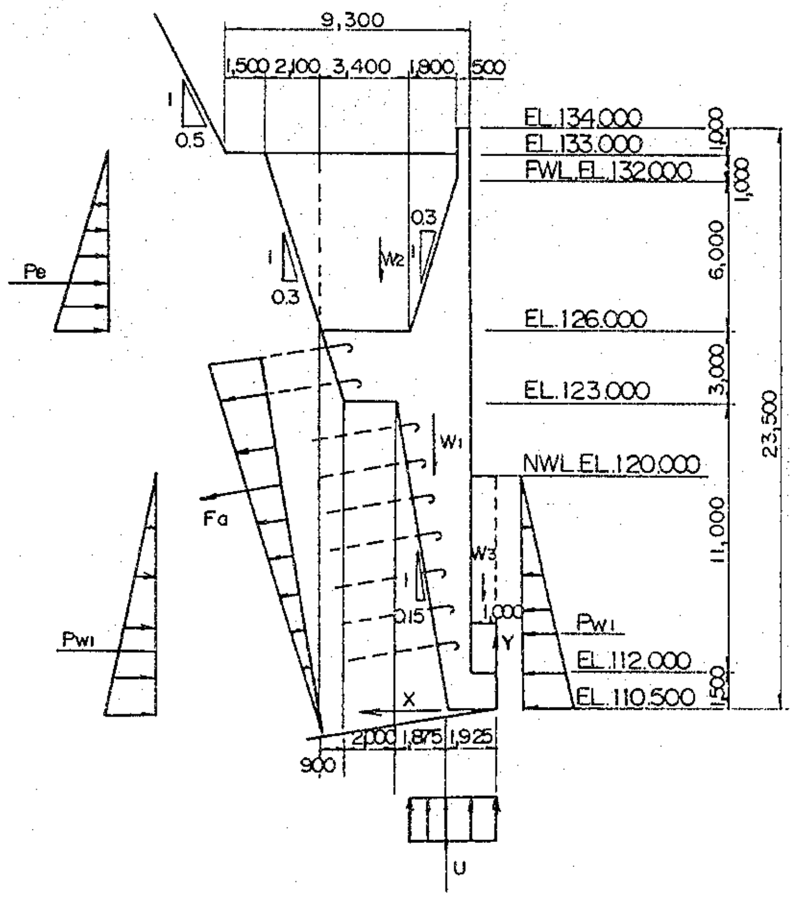
Safety factor for sliding: $Fs = \frac{f \cdot \Sigma V + \tau \cdot A + Ha}{\Sigma H} = \frac{0.55 \times 47.135 + 20 \times 2.0 + 35.16}{2.357} = 42.9 > 1.2$

Safety factor for overturning: $Fs = \frac{\Sigma Mr}{\Sigma Mt} = \frac{97.700 + 196.415}{11.153} = 26.37 > 1.2$

Safety for bearing: $q = \frac{\Sigma V}{B} = \frac{47.135 + 10.548}{4.4} = 13.11\ t/m^2 < 100\ t/m^2$

Note: As for Section name, see Figures of spillway structure design.

Table 4.3.7: STABILITY ANALYSIS OF SPILLWAY STRUCTURE
(Stilling Basin Side Wall, Section E-E, Loading Case I)



SECTION AND LOADING CONDITION

Load		V (t)	H (t)	X (m)	Y (m)	Mv (t.m)	MH (t.m)
Body force	W1	104.835		2.398		251.400	
Earth force	W2	59.830		4.448		266.147	
Water force	W3	8.000		0.500		4.000	
Earth pressure	Pe		11.407		17.833		203.431
Uplift	U	-31.825		1.675		--53.307	
Total		140.840	11.407			468.240	203.431

Max. Resisting Force of Anchor Bar: $F_a = 70.787$ (t) (1.5m pitch)
- Vertical component: $V_a = 20.341$ (t)
- Horizontal component: $H_a = 67.802$ (t)

Max. Resisting Moment of Anchor Bar: $M_a = 773.896$ (t.m)

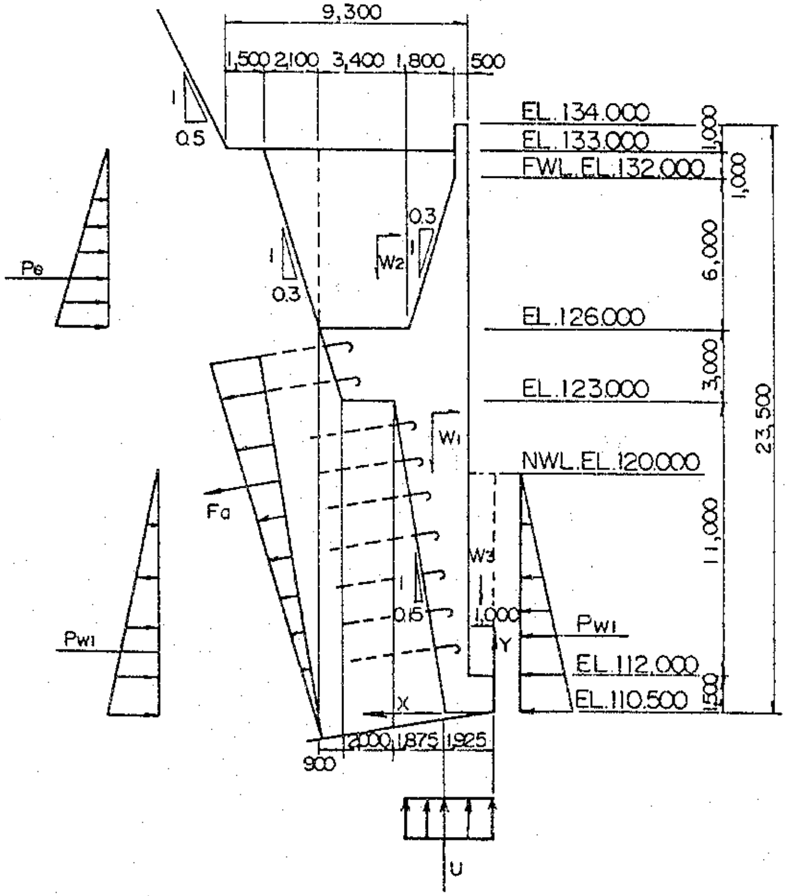
Safety factor for sliding: $$F_s = \frac{f \cdot \Sigma V + \tau \cdot A + H_a}{\Sigma H} = \frac{0.55 \times 140.840 + 20 \times 3.925 + 67.802}{11.407} = 19.6 > 1.5$$

Safety factor for overturning: $$F_s = \frac{\Sigma M_r}{\Sigma M_t} = \frac{468.240 + 773.896}{203.431} = 6.1 > 1.5$$

Safety for bearing: $$q = \frac{\Sigma V}{B} = \frac{140.840 + 20.341}{6.7} = 24.1\ t/m^2 < 100\ t/m^2$$

Note: As for Section name, see Figures of spillway structure design.

Table 4.3.8: STABILITY ANALYSIS OF SPILLWAY STRUCTURE
(Stilling Basin Side Wall, Section E-E, Loading Case II)



SECTION AND LOADING CONDITION

Load		V (t)	H (t)	X (m)	Y (m)	Mv (t.m)	MH (t.m)
Body force	W1	104.835	5.242	2.398	12.034	251.400	63.079
Earth force	W2	59.830	2.992	4.448	19.261	266.147	57.620
Water force	W3	8.000		0.500		4.000	
Earth pressure	Pe		12.469		17.833		222.357
Uplift	U	-31.825		1.675		-53.307	
Total		140.840	20.702			468.240	343.057

Max. Resisting Force of Anchor Bar: Fa = 70.787 (t) (1.5m pitch)
- Vertical component: Va = 20.341 (t)
- Horizontal component: Ha = 67.802 (t)

Max. Resisting Moment of Anchor Bar: Ma = 773.896 (t.m)

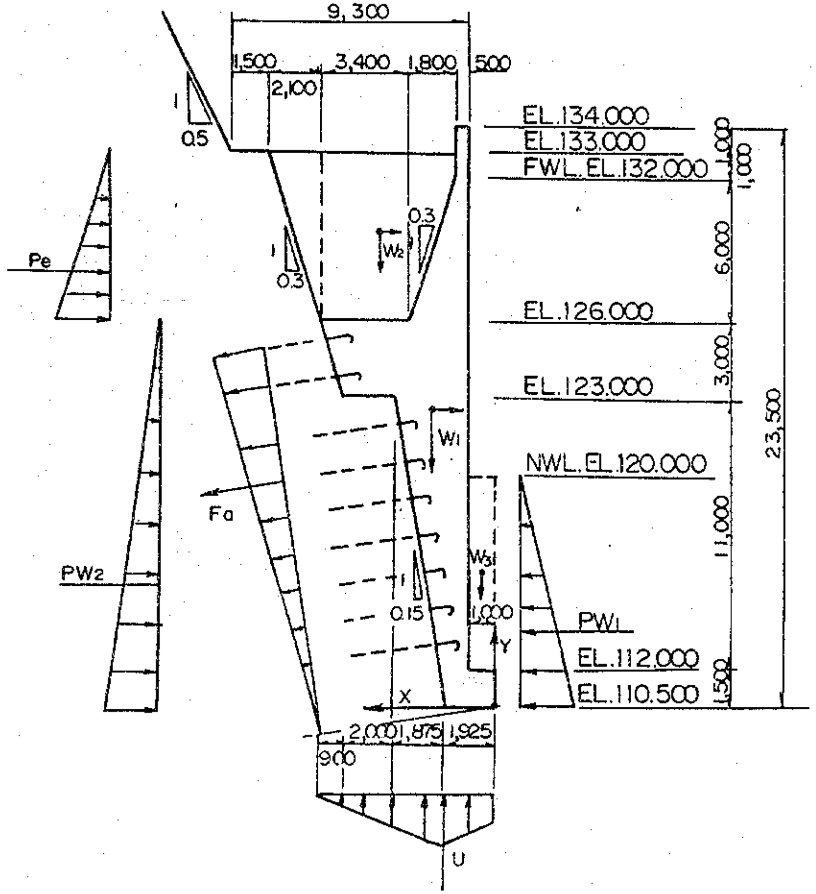
Safety factor for sliding: $Fs = \dfrac{f \cdot \Sigma V + \tau \cdot A + Ha}{\Sigma H} = \dfrac{0.55 \times 140.840 + 20 \times 3.925 + 67.802}{20.702} = 10.81 > 1.2$

Safety factor for overturning: $Fs = \dfrac{\Sigma Mr}{\Sigma Mt} = \dfrac{468.240 + 773.896}{343.057} = 3.62 > 1.2$

Safety for bearing: $q = \dfrac{\Sigma V}{B} = \dfrac{140.840 + 20.341}{6.7} = 24.1\ t/m^2 < 100\ t/m^2$

Note: As for Section name, see Figures of spillway structure design.

Table 4.3.9: STABILITY ANALYSIS OF SPILLWAY STRUCTURE
(Stilling Basin Side Wall, Section E-E, Loading Case III)



SECTION AND LOADING CONDITION

Load		V (t)	H (t)	X (m)	Y (m)	Mv (t.m)	MH (t.m)
Body force	W1	104.835		2.398		251.400	
Earth force	W2	59.830		4.448		266.147	
Water force	W3	8.000		0.500		4.000	
Earth pressure	Pe		11.407		17.833		203.431
Water pressure	Pw1		-45.125		3.167		-142.896
Water pressure	Pw2		120.125		7.167		860.896
Uplift	U1	-18.288		0.963		-17.602	
Uplift	U2	-37.006		3.517		-130.139	
Uplift	U3	-5.775		1.283		-7.411	
Total		111.596	86.407			366.395	921.431

Max. Resisting Force of Anchor Bar: Fa = 70.787 (t) (1.5m pitch)
- Vertical component: Va = 20.341 (t)
- Horizontal component: Ha = 67.802 (t)

Max. Resisting Moment of Anchor Bar: Ma = 773.896 (t.m)

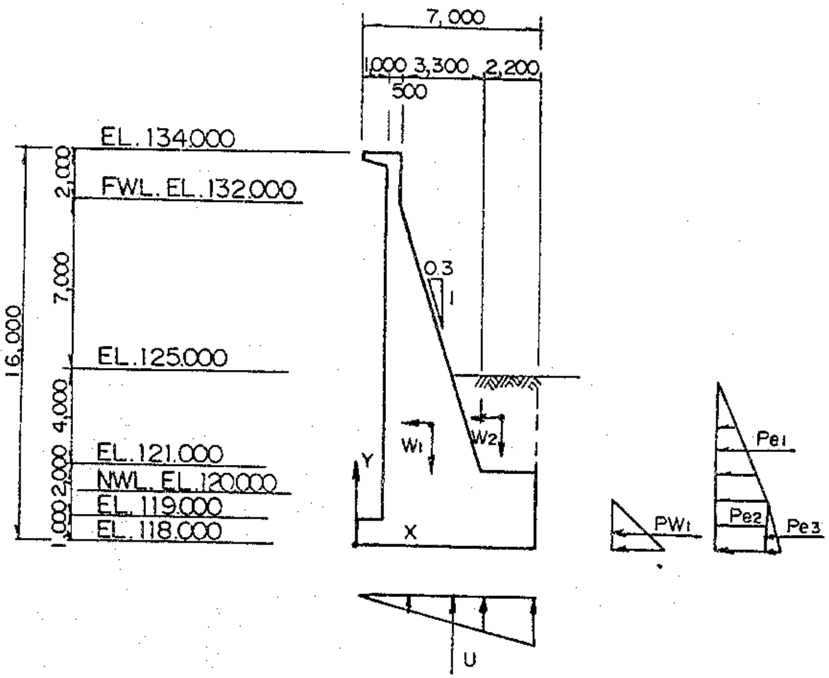
Safety factor for sliding: $Fs = \frac{f \cdot \Sigma V + \tau \cdot A + Ha}{\Sigma H} = \frac{0.55 \times 111.596 + 20 \times 3.925 + 67.802}{86.407} = 2.4 > 1.2$

Safety factor for overturning: $Fs = \frac{\Sigma Mr}{\Sigma Mt} = \frac{366.395 + 773.896}{921.431} = 1.24 > 1.2$

Safety for bearing: $q = \frac{\Sigma V}{B} = \frac{111.596 + 20.341}{6.7} = 19.7\ t/m^2 < 100\ t/m^2$

Note: As for Section name, see Figures of spillway structure design.

Table 4.3.10: STABILITY ANALYSIS OF SPILLWAY STRUCTURE
(Stilling Basin Side Wall, Section I-I, Loading Case I)



SECTION AND LOADING CONDITION

Load		V (t)	H (t)	X (m)	Y (m)	Mv (t.m)	MH (t.m)
Body force	W1	105.720		2.907		307.276	
Earth force	W2	22.002		5.558		122.285	
Water pressure	Pw		2.000		0.667		1.333
Earth pressure	Pe1		5.820		3.667		21.340
Earth pressure	Pe2		1.484		1.000		1.484
Earth pressure	Pe3		0.593		0.667		0.396
Uplift	U	-7.000		3.500		-24.500	
Total		120.722	9.897			405.061	24.553

Max. Resisting Force of Anchor Bar: Fa = 0
- Vertical component: Va = 0
- Horizontal component: Ha = 0

Max. Resisting Moment of Anchor Bar: Ma = 0

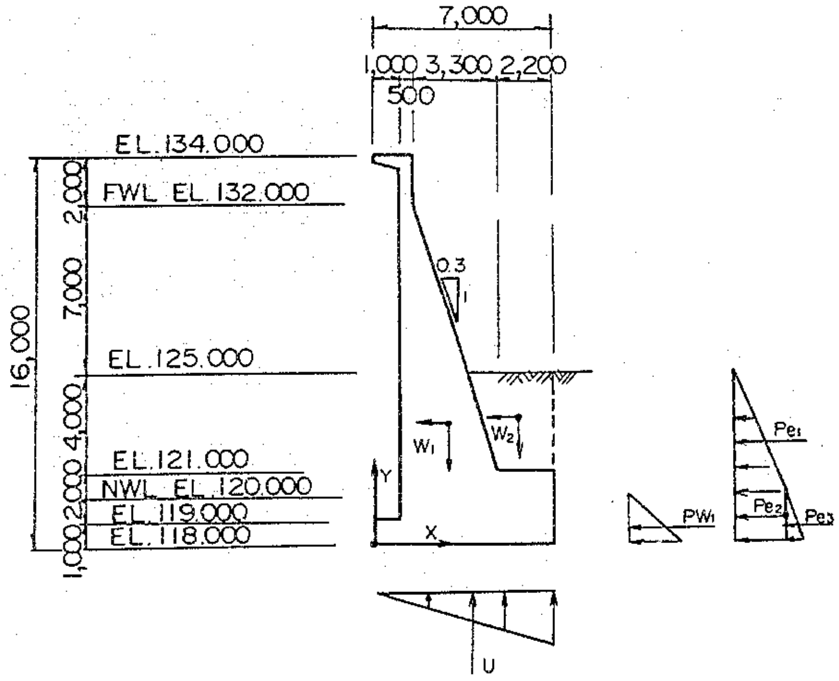
Safety factor for sliding: $Fs = \frac{f \cdot \Sigma V + \tau \cdot A + Ha}{\Sigma H} = \frac{0.55 \times 120.722 + 20 \times 7.0}{9.897} = 20.9 > 1.5$

Safety factor for overturning: $Fs = \frac{\Sigma Mr}{\Sigma Mt} = \frac{405.061}{24.553} = 16.5 > 1.5$

Safety for bearing: $q = \frac{\Sigma}{B}\left(1 \pm \frac{6e}{B}\right) = \frac{120.722}{7.00}\left(1 \pm \frac{6 \times 0.348}{7.00}\right) = 22.4,\ 12.1\ t/m^2$

Note: As for Section name, see Figures of spillway structure design.

Table 4.3.11: STABILITY ANALYSIS OF SPILLWAY STRUCTURE
(Stilling Basin Side Wall, Section I-I, Loading Case II)



SECTION AND LOADING CONDITION

Load		V (t)	H (t)	X (m)	Y (m)	Mv (t.m)	MH (t.m)
Body force	W1	105.720	5.286	2.907	4.916	307.276	25.988
Earth force	W2	22.002	1.100	5.558	5.152	122.285	5.668
Water pressure	Pw		2.000		0.667		1.333
Earth pressure	Pe1		6.362		3.667		23.326
Earth pressure	Pe2		1.622		1.000		1.622
Earth pressure	Pe3		0.649		0.667		0.432
Uplift	U	-7.000		3.500		-24.500	
Total		120.722	17.018			405.061	58.369

Max. Resisting Force of Anchor Bar: Fa = 0
- Vertical component: Va = 0
- Horizontal component: Ha = 0

Max. Resisting Moment of Anchor Bar: Ma = 0

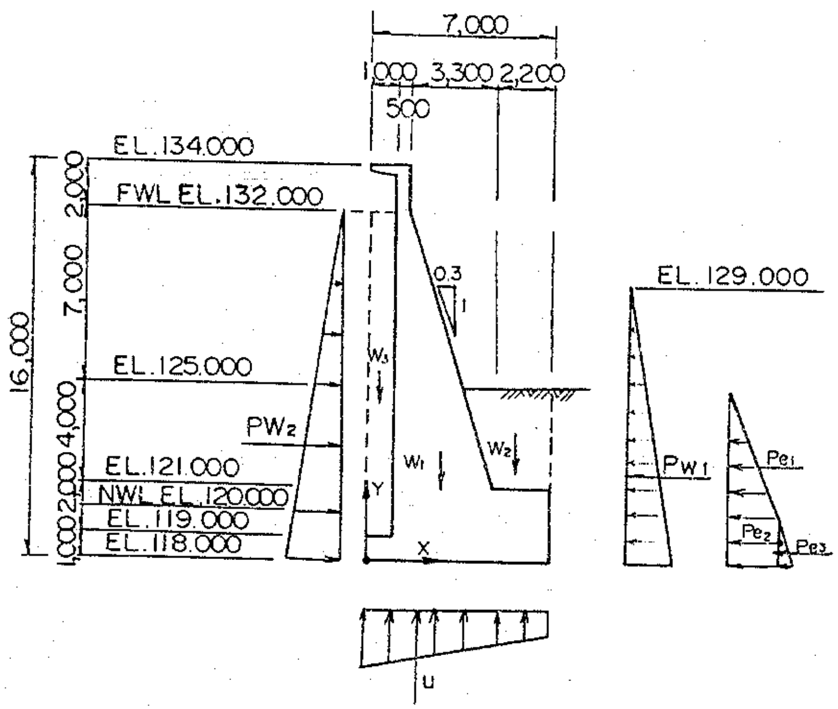
Safety factor for sliding: $Fs = \frac{f \cdot \Sigma V + \tau \cdot A + Ha}{\Sigma H} = \frac{0.55 \times 120.722 + 20 \times 7.0}{17.018} = 12.1 > 1.2$

Safety factor for overturning: $Fs = \frac{\Sigma Mr}{\Sigma Mt} = \frac{405.061}{58.369} = 6.94 > 1.2$

Safety for bearing: $q = \frac{\Sigma}{B}\left(1 \pm \frac{6e}{B}\right) = \frac{120.722}{7.00}\left(1 \pm \frac{6 \times 0.628}{7.00}\right) = 26.5,\ 8.0\ t/m^2$

Note: As for Section name, see Figures of spillway structure design.

Table 4.3.12: STABILITY ANALYSIS OF SPILLWAY STRUCTURE
(Stilling Basin Side Wall, Section I-I, Loading Case III)



SECTION AND LOADING CONDITION

Load		V (t)	H (t)	X (m)	Y (m)	Mv (t.m)	MH (t.m)
Body force	W1	105.720		2.907		307.276	
Earth force	W2	22.002		5.558		122.285	
Water force	W3	13.000		0.500		6.500	
Water pressure	Pw1		60.500		3.667		221.833
Water pressure	Pw2		-98.000		4.667		-457.333
Earth pressure	Pe1		116.075		2.333		270.842
Earth pressure	Pe2						
Earth pressure	Pe3						
Uplift	U1	-77.000		3.500		-269.500	
Uplift	U2	-10.500		2.333		-24.500	
Total		53.222	78.575			142.061	35.342

Max. Resisting Force of Anchor Bar: Fa = 0
- Vertical component: Va = 0
- Horizontal component: Ha = 0

Max. Resisting Moment of Anchor Bar: Ma = 0

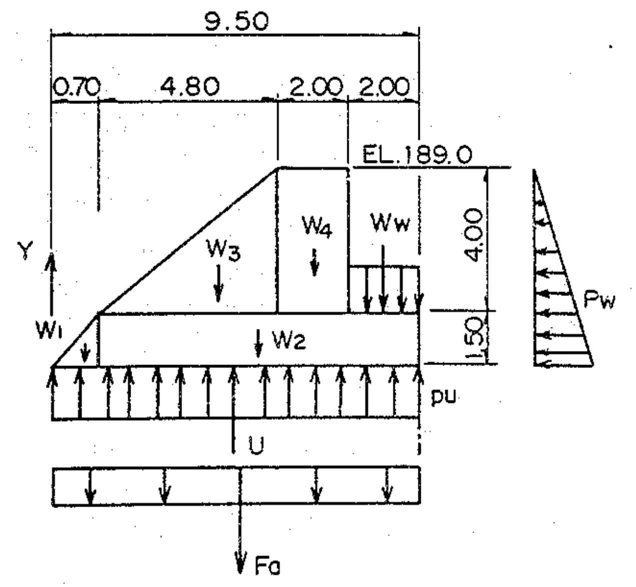
Safety factor for sliding: $Fs = \frac{f \cdot \Sigma V + \tau \cdot A + Ha}{\Sigma H} = \frac{0.55 \times 53.222 + 20 \times 7.0}{78.575} = 4.0 > 1.2$

Safety factor for overturning: $Fs = \frac{\Sigma Mr}{\Sigma Mt} = \frac{142.061}{35.342} = 3.4 > 1.2$

Safety for bearing: $q = \frac{2\Sigma V}{B} = \frac{2 \times 53.222}{6.015} = 17.7\ t/m^2 < 100\ t/m^2$

Note: As for Section name, see Figures of spillway structure design.

Table 4.3.13: STABILITY ANALYSIS OF OVERFLOW WEIR
(Normal Condition)



SECTION AND LOADING CONDITION

Load		V (t)	H (t)	X (m)	Y (m)	Mv (t.m)	MH (t.m)
Body force	W1	1.21		0.47		0.57	
	W2	30.36		5.10		154.84	
	W3	22.08		3.90	4.833	86.11	508.104
	W4	18.40		6.50	16.833	119.60	192.023
Water force	Ww	8.00		8.50		68.00	
Water pressure	Pw		-15.13		1.38		-27.63
Uplift	U	-52.25			4.50		-235.13
Total		27.80	-15.13			429.12	-262.76

Max. Resisting Force of Anchor Bar: Fa = 48.18 (t)(1.5mpitch)
Max. Resisting Moment of Anchor Bar: Ma = 216.81 (t.m)

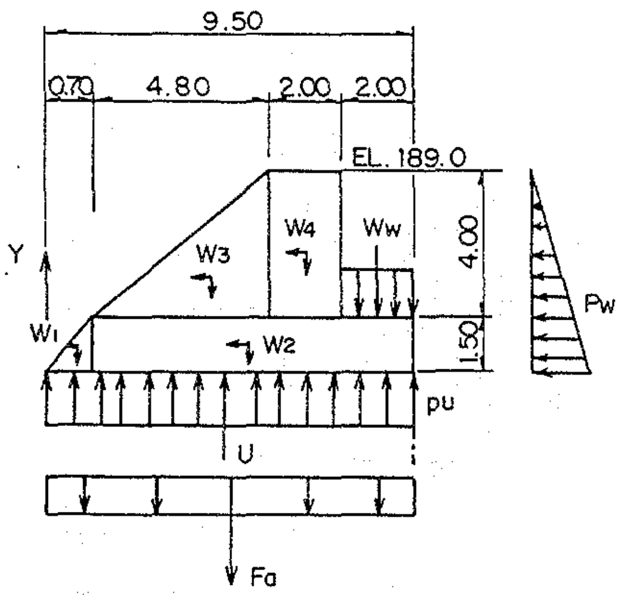
Safety factor for sliding: $Fs = \frac{f \cdot \Sigma V + \tau \cdot A + Ha}{\Sigma H} = \frac{0.55 \times 27.8 + 20 \times 9.5}{15.13} = 13.6 > 1.5$

Safety factor for overturning: $Fs = \frac{\Sigma Mr}{\Sigma Mt} = \frac{429.12 + 216.81}{262.76} = 2.46 > 1.5$

Safety for bearing: $q = \frac{\Sigma V}{B} = \frac{27.80 + 48.18}{9.5} = 8.0 t/m2 < 100 t/m2$

Note: As for Section name, see Figures of spillway structure design.

Table 4.3.14: STABILITY ANALYSIS OF OVERFLOW WEIR
(Seismic Condition)



SECTION AND LOADING CONDITION

Load		V (t)	H (t)	X (m)	Y (m)	Mv (t.m)	MH (t.m)
Body force	W1	1.21	-0.06	0.47	0.50	0.57	-0.03
	W2	30.36	-1.52	5.10	0.75	154.84	-1.14
	W3	22.08	-1.10	3.90	2.83	86.11	-3.11
	W4	18.40	-0.92	6.50	3.50	119.60	-3.22
Water force	Ww	8.00		8.50		68.00	
Water pressure	Pw		-15.13		1.83		-27.63
	Pd		-0.467		3.10		-1.45
Uplift	U	-52.25			4.50		-235.13
Total		27.80	-19.20			429.12	-271.71

Max. Resisting Force of Anchor Bar: Fa = 48.18 (t)(1.5mpitch)
Max. Resisting Moment of Anchor Bar: Ma = 216.81 (t.m)

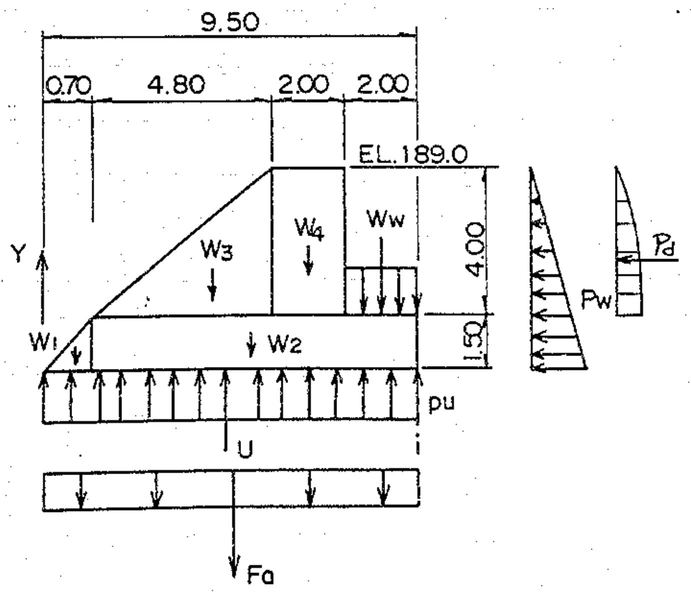
Safety factor for sliding: $Fs = \frac{f \cdot \Sigma V + \tau \cdot A + Ha}{\Sigma H} = \frac{0.55 \times 27.8 + 20 \times 9.5}{19.20} = 10.7 > 1.5$

Safety factor for overturning: $Fs = \frac{\Sigma Mr}{\Sigma Mt} = \frac{429.12 + 216.81}{271.71} = 2.38 > 1.5$

Safety for bearing: $q = \frac{\Sigma V}{B} = \frac{27.80 + 48.18}{9.5} = 8.0 t/m2 < 100 t/m2$

Note: As for Section name, see Figures of spillway structure design.

Table 4.3.15: STABILITY ANALYSIS OF OVERFLOW WEIR
(After the Flood (P.M.F))



SECTION AND LOADING CONDITION

Load		V (t)	H (t)	X (m)	Y (m)	Mv (t.m)	MH (t.m)
Body force	W1	1.21		0.47		0.57	
	W2	30.36		5.10		154.84	
	W3	22.08		3.90	4.833	86.11	
	W4	18.40		6.50	16.833	119.60	
Water force	Ww	8.00		8.50		68.00	
Water pressure	Pw		-15.13		1.83		-27.63
Uplift	U	-95.00			4.50		-427.50
Total		-14.95	-15.13			429.12	-455.13

Max. Resisting Force of Anchor Bar: Fa = 48.18 (t)(1.5mpitch)
Max. Resisting Moment of Anchor Bar: Ma = 216.81 (t.m)

Safety factor for sliding: $Fs = \dfrac{\tau \cdot A}{\Sigma H} = \dfrac{20 \times 9.5}{15.13} = 12.6 > 1.2$

Safety factor for overturning: $Fs = \dfrac{\Sigma Mr}{\Sigma Mt} = \dfrac{429.12 + 216.81}{455.13} = 1.42 > 1.2$

Table 4.3.16 : **STRESS ANALYSIS OF REINFORCED CONCRETE**

(Side Channel Wall, Section C - C)

Load Cond-tion	Member	Spot	Internal force				Sectional dimension					d'/d	M'=M+Nu	M'/bd^2	Q/bd
			Direc-tion	M (t.m)	Q (t)	N (t)	b (cm)	h (cm)	u (cm)	d (cm)	d' (cm)		(t.m)	(kg/cm2)	(kg/cm2)
Nor.	EL.189			26.60	11.40	-	100	230		220			26.6	0.55	0.518
Seis.	EL.189			33.94	13.75	-	100	230		220			33.94	0.701	0.625
Nor.	Toe			16.98	33.96	-	100	150		140			16.98	0.866	2.426
Seis.	Toe			16.98	33.96	-	100	150		140			16.98	0.866	2.426

f=M/N+u	f/d	Sectional area of reinforcing bar		As'/As	np =	Coeff. from Nomogram			Stress (kg/cm2)		
		As (cm2)	As' (cm2)		n.As/bd	C	S	Z	SIGc= CM'/bd^2	SIGs= nSM'/bd^2	Tau= ZQ/bd
		D19@200 = 14.33			0.010	15.9	105	1.05	8.7	866	0.5
		D19@200 = 14.33			0.010	15.9	105	1.05	11.1	1104	0.7
		D19@200 = 14.33			0.015	13.2	70.3	1.06	11.4	914	2.6
		D19@200 = 14.33			0.015	13.2	70.3	1.06	11.4	914	2.6

n=Es/Ec=15, Allowable stress : SIGca=60 & 90(*) kg/cm2, SIGsa = 1,800 & 2,700(*) kg/cm2, TAUa = 8 &12(*) kg/cm2

* : Allowable stresses marked with (*) are applied for the sesmic and flood conditions.

Table 4.3.17 : **STRESS ANALYSIS OF REINFORCED CONCRETE**

(Transition Wall, Section G - G & A - A)

Load Cond-tion	Member	Spot	Internal force Direc-tion	M (t.m)	Q (t)	N (t)	Sectional dimension b (cm)	h (cm)	u (cm)	d (cm)	d' (cm)	d'/d	M'=M+Nu (t.m)	M'/bd^2 (kg/cm2)	Q/bd (kg/cm2)
Nor.															
& Seis	Toe	G-G		16.45	32.90	-	100	150		140			16.45	0.839	0.015
Seis.	Toe	A-A		5.35	10.70	-	100	100		90			5.35	0.660	1.189

f=M/N+u	f/d	Sectional area of reinforcing bar As (cm2)	As' (cm2)	As'/As	np = n.As/bd	Coeff. from Nomogram C	S	Z	Stress (kg/cm2) SIGc= CM'/bd^2	SIGs= nSM'/bd^2	Tau= ZQ/bd
		D19@200=14.33			0.015	13.2	70.3	1.06	11.0	885	2.5
		D19@200=14.33			0.024	10.9	44.6	1.07	7.2	442	1.3

n=Es/Ec=15, Allowable stress : SIGca=60 & 90(*) kg/cm2, SIGsa = 1,800 & 2,700(*) kg/cm2, TAUa = 8 &12(*) kg/cm2

* : Allowable stresses marked with (*) are applied for the sesmic and flood conditions.

Table 4.3.18 : **STRESS ANALYSIS OF REINFORCED CONCRETE**

(Stilling Basin Side Wall, Section E -E)

Load Cond-tion	Member	Spot	Internal force				Sectional dimension					d'/d	M'=M+Nu (t.m)	M'/bd^2 (kg/cm2)	Q/bd (kg/cm2)
			Direc-tion	M (t.m)	Q (t)	N (t)	b (cm)	h (cm)	u (cm)	d (cm)	d' (cm)				
Nor.&															
Flood	EL.126.00			26.60	11.40	-	100	230		220			26.6	0.55	0.518
Seis.	EL.126.00			34.34	13.79	-	100	230		220			34.34	0.71	0.627
Nor.&															
Seis.	Toe			12.50	25.00	-	100	150		140			12.5	0.638	1.786
Flood	Toe			11.80	23.60	-	100	150		140			11.8	0.602	1.686

f=M/N+u	f/d	Sectional area of reinforcing bar		As'/As	np = n.As/bd	Coeff. from Nomogram			Stress (kg/cm2)		
		As (cm2)	As' (cm2)			C	S	Z	SIGc= CM'/bd^2	SIGs= nSM'/bd^2	Tau= ZQ/bd
		D19@200=14.33			0.01	15.9	105	1.05	8.7	866	0.5
		D19@200=14.33			0.01	15.9	105	1.05	11.3	1117	0.7
		D19@200=14.33			0.015	13.2	70.3	1.06	8.4	673	1.9
		D19@200=14.33			0.015	13.2	70.3	1.06	7.9	634	1.8

n=Es/Ec=15, Allowable stress : SIGca=60 & 90(*) kg/cm2, SIGsa = 1,800 & 2,700(*) kg/cm2, TAUa = 8 &12(*) kg/cm2

* : Allowable stresses marked with (*) are applied for the sesmic and flood conditions.

Table 4.3.19 : **STRESS ANALYSIS OF REINFORCED CONCRETE**

(Stilling Basin Side Wall, Section I - I (1))

Load Cond-tion	Member	Spot	Internal force				Sectional dimension					d'/d	M'=M+Nu	M'/bd^2	Q/bd
			Direc-tion	M (t.m)	Q (t)	N (t)	b (cm)	h (cm)	u (cm)	d (cm)	d' (cm)		(t.m)	(kg/cm2)	(kg/cm2)
Nor.	EL.121.0	Back		4.96	3.72	-	100	380		370			4.96	0.036	0.101
Seis.	EL.121.0	Back		23.60	7.46	-	100	380		370			23.6	0.172	0.202
Flood	EL.121.0	Front		134.11	26.71	-	100	380		370			134.11	0.98	0.722

f=M/N+u	f/d	Sectional area of reinforcing bar		As'/As	np =	Coeff. from Nomogram			Stress (kg/cm2)		
		As (cm2)	As' (cm2)		n.As/bd	C	S	Z	SIGc= CM'/bd^2	SIGs= nSM'/bd^2	Tau= ZQ/bd
		D19@200=14.33			0.006	20	173	1.03	0.7	94	0.1
		D19@200=14.33			0.006	20	173	1.03	3.4	447	0.2
		D19@200=14.33			0.006	20	173	1.03	19.6	2542	0.7

n=Es/Ec=15, Allowable stress : SIGca=60 & 90(*) kg/cm2, SIGsa = 1,800 & 2,700(*) kg/cm2, TAUa = 8 &12(*) kg/cm2

* : Allowable stresses marked with (*) are applied for the sesmic and flood conditions.

Table 4.3.20 : **STRESS ANALYSIS OF REINFORCED CONCRETE**

(Stilling Basin Side Wall, Section I - I (2))

Load Cond-tion	Member	Spot	Internal force				Sectional dimension					d'/d	M'=M+Nu (t.m)	M'/bd^2 (kg/cm2)	Q/bd (kg/cm2)
			Direc-tion	M (t.m)	Q (t)	N (t)	b (cm)	h (cm)	u (cm)	d (cm)	d' (cm)				
Nor.	Toe	Low		9.81	19.42	-	100	100		90			9.81	1.211	2.158
Seis.	Toe	Low		11.66	22.93	-	100	100		90			11.66	1.44	2.548
Flood	Toe	Upper		0.76	1.6	-	100	100		90			0.76	0.094	0.178

f=M/N+u	f / d	Sectional area of reinforcing bar		As'/As	np = n.As/bd	Coeff. from Nomogram			Stress (kg/cm2)		
		As (cm2)	As' (cm2)			C	S	Z	SIGc= CM'/bd^2	SIGs= nSM'/bd^2	Tau= ZQ/bd
		D19@200=14.33			0.024	10.9	44.6	1.07	13.2	810	2.3
		D19@200=14.33			0.024	10.9	44.6	1.07	15.7	963	2.7
		D19@200=14.33			0.024	10.9	44.6	1.07	1.0	63	0.2

n=Es/Ec=15, Allowable stress : SIGca=60 & 90(*) kg/cm2, SIGsa = 1,800 & 2,700(*) kg/cm2, TAUa = 8 &12(*) kg/cm2

* : Allowable stresses marked with (*) are applied for the sesmic and flood conditions.

Table 4.4.1: SUMMARY OF BENDING MOMENT IN COMPOSITE GIRDER (Main Girder)

Unit : t•m

Nodal Point	D.L (B.C)	D.L (A.C)	L.L Max. (A.C)	L.L Min. (A.C)	Max. (A.C)	Min. (A.C)
No. 1 Main Girder (G-1):						
1	- 0.0	- 0.0	0.0	- 0.0	0.0	- 0.0
4	100.1	22.5	66.8	- 3.7	89.3	17.7
	100.1	22.5	66.8	- 3.7	89.3	17.7
7	162.5	32.1	114.6	- 7.6	146.7	22.3
	162.5	32.1	114.6	- 7.6	146.7	22.3
10	182.2	27.8	137.3	- 11.4	165.1	12.9
	182.2	27.8	137.3	- 11.4	165.1	12.9
13	162.5	32.1	114.6	- 7.6	146.7	22.3
	162.5	32.1	114.6	- 7.6	146.7	22.3
16	100.1	22.5	66.8	- 3.7	89.3	17.7
	100.1	22.5	66.8	- 3.7	89.3	17.7
19	- 0.0	- 0.0	0.0	- 0.0	0.0	- 0.0
No. 2 Main Girder (G-2):						
2	0.0	0.0	0.0	- 0.0	0.0	- 0.0
5	98.0	10.5	80.1	- 0.0	90.6	10.4
	98.0	10.5	80.1	- 0.0	90.6	10.4
8	161.2	26.1	122.7	- 0.0	148.8	26.1
	161.2	26.1	122.7	- 0.0	148.8	26.1
11	184.6	46.5	128.2	- 0.0	174.7	46.5
	184.6	46.5	128.2	- 0.0	174.7	46.5
14	161.2	26.1	122.7	- 0.0	148.8	26.1
	161.2	26.1	122.7	- 0.0	148.8	26.1
17	98.0	10.5	80.1	- 0.0	90.6	10.4
	98.0	10.5	80.1	- 0.0	90.6	10.4
20	0.0	0.0	0.0	- 0.0	0.0	- 0.0
No. 3 Main Girder (G-3):						
3	- 0.0	- 0.0	0.0	- 0.0	0.0	- 0.0
6	100.1	22.5	66.8	- 3.7	89.3	17.7
	100.1	22.5	66.8	- 3.7	89.3	17.7
9	162.5	32.1	114.6	- 7.6	146.7	22.3
	162.5	32.1	114.6	- 7.6	146.7	22.3
12	182.2	27.8	137.3	- 11.4	165.1	12.9
	182.2	27.8	137.3	- 11.4	165.1	12.9
15	162.5	32.1	114.6	- 7.6	146.7	22.3
	162.5	32.1	114.6	- 7.6	146.7	22.3
18	100.1	22.5	66.8	- 3.7	89.3	17.7
	100.1	22.5	66.8	- 3.7	89.3	17.7
21	- 0.0	- 0.0	0.0	- 0.0	0.0	- 0.0

Note : B.C : Before compounding
A.C : After compounding
D.L : Moment due to dead load
L.L : Moment due to live load

Table 4.4.2: SUMMARY OF SHEAR FORCE IN COMPOSITE GIRDER (Main Girder)

Unit : ton

Nodal Point	D.L (B.C)	D.L (A.C)	L.L Max. (A.C)	L.L Min. (A.C)	Max. (A.C)	Min. (A.C)
No. 1 Main Girder (G-1):						
1	25.5	6.2	16.3	- 0.8	22.5	5.1
4	17.1	3.4	13.2	- 2.0	16.6	0.7
	17.1	3.4	13.2	- 2.0	16.6	0.8
7	8.4	0.5	10.2	- 4.1	10.8	- 4.1
	8.4	0.5	10.2	- 4.1	10.8	- 4.1
10	- 0.4	- 2.3	7.5	- 6.5	7.5	- 8.8
	0.4	2.3	6.5	- 7.5	8.8	- 7.5
13	- 8.4	- 0.5	4.1	- 10.2	4.1	- 10.8
	- 8.4	- 0.5	4.1	- 10.2	4.1	- 10.8
16	- 17.1	- 3.4	2.0	- 13.2	- 0.8	- 16.6
	- 17.1	- 3.4	2.0	- 13.2	- 0.7	-16.6
19	- 25.5	- 6.2	0.8	- 16.3	- 5.1	- 22.5
No. 2 Main Girder (G-2):						
2	24.7	1.8	21.5	- 0.3	23.2	1.3
5	17.0	2.7	15.7	- 3.3	18.4	- 1.6
	17.0	2.7	15.7	- 3.3	18.4	- 1.6
8	8.8	3.7	11.0	- 7.4	14.7	- 5.9
	8.8	3.7	11.0	- 7.4	14.7	- 5.9
11	0.7	4.7	7.4	- 11.8	12.1	-10.7
	- 0.7	- 4.7	11.8	- 7.4	10.7	- 12.1
14	- 8.8	- 3.7	7.4	- 11.0	5.9	- 14.7
	- 8.8	- 3.7	7.4	- 11.0	5.9	- 14.7
17	- 17.0	- 2.7	3.3	- 15.7	1.6	- 18.4
	- 17.0	- 2.7	3.3	- 15.7	1.6	- 18.4
20	- 24.7	- 1.8	0.3	- 21.5	- 1.3	- 23.2
No. 3 Main Girder (G-3):						
3	25.5	6.2	16.3	- 0.8	22.5	5.1
6	17.1	3.4	13.2	- 2.0	16.6	0.7
	17.1	3.4	13.2	- 2.0	16.6	0.8
9	8.4	0.5	10.2	- 4.1	10.8	- 4.1
	8.4	0.5	10.2	- 4.1	10.8	- 4.1
12	- 0.4	- 2.3	7.5	- 6.5	7.5	- 8.8
	0.4	2.3	6.5	- 7.5	8.8	- 7.5
15	- 8.4	- 0.5	4.1	- 10.2	4.1	- 10.8
	- 8.4	- 0.5	4.1	- 10.2	4.1	- 10.8
18	- 17.1	- 3.4	2.0	- 13.2	- 0.8	- 16.6
	- 17.1	- 3.4	2.0	- 13.2	- 0.7	- 16.6
21	- 25.5	- 6.2	0.8	- 16.3	- 5.1	- 22.5

Note : B.C : Before compounding
A.C : After compounding
D.L : Shear due to dead load
L.L : Shear due to live load

Table 4.4.3: SUMMARY OF REACTION FORCE AT SUPPORTS

Unit : ton

Nodal Point	D.L (B.C)	D.L (A.C)	L.L Max. (A.C)	Total Reaction Force
No. 1 Main Girder (G-1):				
1	25.5	6.2	16.3	48.0
19	25.5	6.2	16.3	48.0
No. 2 Main Girder (G-2):				
2	24.7	1.8	21.5	47.9
20	24.7	1.8	21.5	47.9
No. 3 Main Girder (G-3):				
3	25.5	6.2	16.3	48.0
21	25.5	6.2	16.3	48.0

Note : B.C : Before compounding
A.C : After compounding
D.L : Reaction due to dead load
L.L : Reaction due to live load

Table 4.4.4: *STRESS ANALYSIS RESULT OF COMPOSITE GIRDER (NO. 1 MAIN GIRDER, NO. 1 SECTION)*

- Bending moment before compounding........................MS = 101.88 t·m
- Bending moment after compounding.........................MV = 90.90 t·m
- Bending moment by dead load after compounding ....MVD = 22.84 t·m
- Base slab thickness ....TS = 18.0 cm
- Haunch...HH = 6.0 cm
- Effective base slab width....................................BS = 228.1 cm
- Distance between fixed points of flange......................P = 490.0 cm
- Section and sectional area of steel girder:

	Section (mm)	Sectional Area (cm^2)
• Upper flange :	230 x 11	25.3 (SM50Y)
• Web :	1,550 x 9	139.5 (SM50Y)
• Lower flange :	280 x 11	30.8 (SM50Y)
TOTAL		195.6

- Sectional area and moment of inertia of area:

	Sectional Area (cm^2)	Moment of Inertia of Area (cm^4)
• Concrete section	: AC = 4,105	IC = 110,832
• Steel girder section	: AS = 195.6	IS = 620,099
• Composite section	: AV = 782	IV = 1,951,186

- Geometrical moment of area of concrete (AC x DC)........QC = 97,226 cm^3
- Distance and section modulus (See Fig.4.4.7):

Distance (cm)		Section Modulus (cm^3)	
D =	94.7	WSU =	7,675
DS =	71.0	WSL =	8,116
DC =	23.7	WVU =	199,399
YSU =	80.8	WVL =	13,236
YSL =	76.4		
YVU =	9.8		
YVL =	147.4		
YVC =	32.7		

- Axial force
  - • Due to drying schrinkage ....NSH = 17.1 ton
  - • Due to creep ....NCR = 2.5 ton
  - • Due to temperature change...............................NTM = 12.0 ton
- Stress (kg/cm^2):

	Concrete Base Slab	Upper Flange	Lower Flange
(1) Stress before compounding	-	- 1,327	1,255
(2) Stress after compounding	- 21.8	- 46	687
(3) Stress due to drying schrinkage	3.1	- 297	110
(4) Stress due to creep	1.9	- 42	16
(5) Stress due to temperature difference	- 0.6	- 207	75
(6) = (1)	-	- 1,327	1,255
Allowable stress	-	- 1,412	2,625
(7) = (1) + (2)	- 21.8	- 1,373	1,942
Allowable stress	- 77.1	- 2,100	2,100
(8) = (1) + (2) + (3) + (4)	- 16.8	- 1,712	2,068
Allowable stress	- 77.1	- 2,415	2,100
(9) = (1) + (2) + (3) + (4) + (5)	- 17.5	- 1,918	2,143
Allowable stress	- 88.7	- 2,730	2,415

Table 4.4.5: STRESS ANALYSIS RESULT OF COMPOSITE GIRDER (NO. 1 MAIN GIRDER, NO. 2 SECTION)

- Bending moment before compounding........................MS = 182.18 t·m
- Bending moment after compounding.........................MV = 165.06 t·m
- Bending moment by dead load after compounding ....MVD = 27.76 t·m
- Base slab thickness ....TS = 18.0 cm
- Haunch..HH = 6.0 cm
- Effective base slab width......................................BS = 228.1 cm
- Distance between fixed points of flange......................P = 490.0 cm
- Section and sectional area of steel girder:

		Section (mm)	Sectional Area (cm^2)
• Upper flange	:	280 x 14	39.2 (SM50Y)
• Web	:	1,550 x 9	139.5 (SM50Y)
• Lower flange	:	440 x 19	83.6 (SM50Y)
TOTAL			262.3

- Sectional area and moment of inertia of area:

		Sectional Area (cm^2)		Moment of Inertia of Area (cm^4)
• Concrete section	: AC =	4,105	IC =	110,832
• Steel girder section	: AS =	262.3	IS =	987,001
• Composite section	: AV =	849	IV =	3,032,152

- Geometrical moment of area of concrete (AC x DC)........QC = 134,244 cm^3
- Distance and section modulus (See Fig. 4.4.7):

Distance (cm)		Section Modulus (cm^3)	
D =	105.8	WSU =	10,703
DS =	73.1	WSL =	14,936
DC =	32.7	WVU =	158,724
YSU =	92.2	WVL =	21,783
YSL =	66.1		
YVU =	19.1		
YVL =	139.2		
YVC =	41.7		

- Axial force
  - • Due to drying schrinkage ....NSH = 20.8 ton
  - • Due to creep ....NCR = 3.3 ton
  - • Due to temperature change..................................NTM = 15.1 ton
- Stress (kg/cm^2):

	Concrete Base Slab	Upper Flange	Lower Flange
(1) Stress before compounding	-	- 1,702	1,220
(2) Stress after compounding	- 32.4	- 104	758
(3) Stress due to drying schrinkage	4.1	- 284	67
(4) Stress due to creep	1.7	- 44	10
(5) Stress due to temperature difference	- 1.6	- 205	48
(6) = (1)	-	- 1,702	1,220
Allowable stress	-	- 1,765	2,625
(7) = (1) + (2)	- 32.4	- 1,806	1,977
Allowable stress	- 77.1	- 2,100	2,100
(8) = (1) + (2) + (3) + (4)	- 26.6	- 2,134	2,055
Allowable stress	- 77.1	- 2,415	2,100
(9) = (1) + (2) + (3) + (4) + (5)	- 28.2	- 2,338	2,103
Allowable stress	- 88.7	- 2,730	2,415

Table 4.4.6: STRESS ANALYSIS RESULT OF COMPOSITE GIRDER (NO. 1 MAIN GIRDER, NO. 3 SECTION)

- Bending moment before compounding……………………MS = 101.89 t•m
- Bending moment after compounding……………………MV = 90.91 t•m
- Bending moment by dead load after compounding………MVD = 22.85 t•m
- Base slab thickness……………………………………TS = 18.0 cm
- Haunch……………………………………………………HH = 6.0 cm
- Effective base slab width………………………………BS = 228.1 cm
- Distance between fixed points of flange…………………P = 490.1 cm
- Section and sectional area of steel girder:

	Section (mm)	Sectional Area (cm^2)
• Upper flange :	230 x 11	25.3 (SM50Y)
• Web :	1,550 x 9	139.5 (SM50Y)
• Lower flange :	280 x 11	30.8 (SM50Y)
TOTAL		195.6

- Sectional area and moment of inertia of area:

	Sectional Area (cm^2)	Moment of Inertia of Area (cm^4)
• Concrete section	: AC = 4,105	IC = 110,832
• Steel girder section	: AS = 195.6	IS = 620,099
• Composite section	: AV = 782	IV = 1,951,186

- Geometrical moment of area of concrete (AC x DC)……..QC = 97,226 cm^3
- Distance and section modulus (See Fig. 4.4.7):

Distance (cm)		Section Modulus (cm^3)	
D =	94.7	WSU =	7,675
DS =	71.0	WSL =	8,116
DC =	23.7	WVU =	199,399
YSU =	80.8	WVL =	13,236
YSL =	76.4		
YVU =	9.8		
YVL =	147.4		
YVC =	32.7		

- Axial force
  - • Due to drying schrinkage……………………NSH = 17.1 ton
  - • Due to creep…………………………………NCR = 2.5 ton
  - • Due to temperature change……………………NTM = 12.0 ton
- Stress (kg/cm^2):

	Concrete Base Slab	Upper Flange	Lower Flange
(1) Stress before compounding	-	- 1,328	1,255
(2) Stress after compounding	- 21.8	- 46	687
(3) Stress due to drying schrinkage	3.1	- 297	110
(4) Stress due to creep	1.9	- 42	16
(5) Stress due to temperature difference	- 0.6	- 207	75
(6) = (1)	-	- 1,328	1,255
Allowable stress	-	- 1,412	2,625
(7) = (1) + (2)	- 21.8	- 1,373	1,942
Allowable stress	- 77.1	- 2,100	2,100
(8) = (1) + (2) + (3) + (4)	- 16.8	- 1,712	2,068
Allowable stress	- 77.1	- 2,415	2,100
(9) = (1) + (2) + (3) + (4) + (5)	- 17.5	- 1,918	2,144
Allowable stress	- 88.7	- 2,730	2,415

Table 4.4.7: STRESS ANALYSIS RESULT OF COMPOSITE GIRDER (NO. 2 MAIN GIRDER, NO. 1 SECTION)

- Bending moment before compounding........................MS = 99.73 t•m
- Bending moment after compounding..........................MV = 92.14 t•m
- Bending moment by dead load after compounding ....MVD = 10.68 t•m
- Base slab thickness ....TS = 18.0 cm
- Haunch...HH = 9.9 cm
- Effective base slab width..BS = 263.9 cm
- Distance between fixed points of flange.......................P = 490.0 cm
- Section and sectional area of steel girder:

	Section (mm)	Sectional Area (cm^2)
• Upper flange :	230 x 10	23.0 (SM50Y)
• Web :	1,550 x 9	139.5 (SM50Y)
• Lower flange :	280 x 11	30.8 (SM50Y)
TOTAL		193.3

- Sectional area and moment of inertia of area:

	Sectional Area (cm^2)	Moment of Inertia of Area (cm^4)
• Concrete section	AC = 4,750	IC = 128,255
• Steel girder section	AS = 193.3	IS = 604,926
• Composite section	AV = 872	IV = 2,114,354

- Geometrical moment of area of concrete (AC x DC)........QC = 104,844 cm^3
- Distance and section modulus (See Fig. 4.4.7):

Distance (cm)		Section Modulus (cm^3)	
D =	99.6	WSU =	7,408
DS =	77.5	WSL =	8,018
DC =	22.1	WVU =	506,868
YSU =	81.7	WVL =	13,826
YSL =	75.4		
YVU =	4.2		
YVL =	152.9		
YVC =	31.1		

- Axial force
  - • Due to drying schrinkage ....NSH = 16.3 ton
  - • Due to creep ....NCR = 1.0 ton
  - • Due to temperature change..................................NTM = 11.2 ton
- Stress (kg/cm^2):

	Concrete Base Slab	Upper Flange	Lower Flange
(1) Stress before compounding	-	- 1,346	1,244
(2) Stress after compounding	- 19.3	- 18	666
(3) Stress due to drying schrinkage	2.3	- 301	116
(4) Stress due to creep	0.7	- 17	6
(5) Stress due to temperature difference	- 0.1	- 204	77
(6) = (1)	-	- 1,346	1,244
Allowable stress	-	- 1,379	2,625
(7) = (1) + (2)	- 19.3	- 1,364	1,910
Allowable stress	- 77.1	- 2,100	2,100
(8) = (1) + (2) + (3) + (4)	- 16.3	- 1,682	2,033
Allowable stress	- 77.1	- 2,415	2,100
(9) = (1) + (2) + (3) + (4) + (5)	- 16.4	- 1,886	2,109
Allowable stress	- 88.7	- 2,730	2,415

Table 4.4.8: STRESS ANALYSIS RESULT OF COMPOSITE GIRDER (NO. 2 MAIN GIRDER, NO. 2 SECTION)

- Bending moment before compounding.......................MS = 184.64 t·m
- Bending moment after compounding.........................MV = 174.66 t·m
- Bending moment by dead load after compounding ....MVD = 46.45 t·m
- Base slab thickness ....TS = 18.0 cm
- Haunch...HH = 9.9 cm
- Effective base slab width.......................................BS = 263.9 cm
- Distance between fixed points of flange......................P = 490.0 cm
- Section and sectional area of steel girder:

	Section (mm)	Sectional Area (cm²)	
• Upper flange :	280 x 14	39.2	(SM50Y)
• Web :	1,550 x 9	139.5	(SM50Y)
• Lower flange :	450 x 19	85.5	(SM50Y)
TOTAL		264.2	

- Sectional area and moment of inertia of area:

	Sectional Area (cm²)	Moment of Inertia of Area (cm⁴)
• Concrete section :	AC = 4,750	IC = 128,255
• Steel girder section :	AS = 264.2	IS = 995,003
• Composite section :	AV = 943	IV = 3,322,056

- Geometrical moment of area of concrete (AC x DC)........QC = 146,672 cm³
- Distance and section modulus (See Fig. 4.4.7):

Distance (cm)		Section Modulus (cm³)	
D =	110.2	WSU =	10,735
DS =	79.3	WSL =	15,164
DC =	30.9	WVU =	248,339
YSU =	92.7	WVL =	22,923
YSL =	65.6		
YVU =	13.4		
YVL =	144.9		
YVC =	39.9		

- Axial force
  - • Due to drying schrinkage ....NSH = 20.7 ton
  - • Due to creep ....NCR = 4.7 ton
  - • Due to temperature change................................NTM = 14.6 ton
- Stress (kg/cm²):

	Concrete Base Slab	Upper Flange	Lower Flange
(1) Stress before compounding	-	- 1,720	1,218
(2) Stress after compounding	- 30.0	- 70	762
(3) Stress due to drying schrinkage	3.4	- 289	71
(4) Stress due to creep	2.4	- 64	16
(5) Stress due to temperature difference	- 1.0	- 203	49
(6) = (1)	-	- 1,720	1,218
Allowable stress	-	- 1,765	2,625
(7) = (1) + (2)	- 30.0	- 1,790	1,980
Allowable stress	- 77.1	- 2,100	2,100
(8) = (1) + (2) + (3) + (4)	- 24.1	- 2,143	2,066
Allowable stress	- 77.1	- 2,415	2,100
(9) = (1) + (2) + (3) + (4) + (5)	- 25.2	- 2,346	2,115
Allowable stress	- 88.7	- 2,730	2,415

Table 4.4.9: STRESS ANALYSIS RESULT OF COMPOSITE GIRDER
(NO. 2 MAIN GIRDER, NO. 3 SECTION)

- Bending moment before compounding......................MS = 99.75 t•m
- Bending moment after compounding........................MV = 92.16 t•m
- Bending moment by dead load after compounding ....MVD = 10.68 t•m
- Base slab thickness ....TS = 18.0 cm
- Haunch...HH = 9.9 cm
- Effective base slab width.......................................BS = 263.9 cm
- Distance between fixed points of flange.....................P = 490.0 cm
- Section and sectional area of steel girder:

	Section (mm)	Sectional Area (cm^2)
• Upper flange :	230 x 10	23.0 (SM50Y)
• Web :	1,550 x 9	139.5 (SM50Y)
• Lower flange :	280 x 11	30.8 (SM50Y)
TOTAL		193.3

- Sectional area and moment of inertia of area:

	Sectional Area (cm^2)	Moment of Inertia of Area (cm^4)
• Concrete section	: AC = 4,750	IC = 128,255
• Steel girder section	: AS = 193.3	IS = 604,926
• Composite section	: AV = 872	IV = 2,114,354

- Geometrical moment of area of concrete (AC x DC)........QC = 104,844 cm^3
- Distance and section modulus (See Fig. 4.4.7):

Distance (cm)		Section Modulus (cm^3)	
D =	99.6	WSU =	7,408
DS =	77.5	WSL =	8,018
DC =	22.1	WVU =	506,868
YSU =	81.7	WVL =	13,826
YSL =	75.4		
YVU =	4.2		
YVL =	152.9		
YVC =	31.1		

- Axial force
  - Due to drying schrinkage ....NSH = 16.3 ton
  - Due to creep ....NCR = 1.0 ton
  - Due to temperature change..............................NTM = 11.2 ton
- Stress (kg/cm^2):

	Concrete Base Slab	Upper Flange	Lower Flange
(1) Stress before compounding	-	- 1,346	1,244
(2) Stress after compounding	- 19.3	- 18	667
(3) Stress due to drying schrinkage	2.3	- 301	116
(4) Stress due to creep	0.7	- 17	6
(5) Stress due to temperature difference	- 0.1	- 204	77
(6) = (1)	-	- 1,346	1,244
Allowable stress	-	- 1,379	2,625
(7) = (1) + (2)	- 19.3	- 1,365	1,911
Allowable stress	- 77.1	- 2,100	2,100
(8) = (1) + (2) + (3) + (4)	- 16.3	- 1,682	2,033
Allowable stress	- 77.1	- 2,415	2,100
(9) = (1) + (2) + (3) + (4) + (5)	- 16.4	- 1,886	2,110
Allowable stress	- 88.7	- 2,730	2,415

Table 4.4.10: STRESS ANALYSIS RESULT OF COMPOSITE GIRDER
(NO. 3 MAIN GIRDER, NO. 1 SECTION)

- Bending moment before compounding........................MS = 101.88 t·m
- Bending moment after compounding.........................MV = 90.90 t·m
- Bending moment by dead load after compounding.........MVD = 22.84 t·m
- Base slab thickness..TS = 18.0 cm
- Haunch..HH = 6.0 cm
- Effective base slab width.....................................BS = 228.1 cm
- Distance between fixed points of flange......................P = 490.0 cm
- Section and sectional area of steel girder:

		Section (mm)	Sectional Area (cm^2)
• Upper flange	:	230 x 11	25.3 (SM50Y)
• Web	:	1,550 x 9	139.5 (SM50Y)
• Lower flange	:	280 x 11	30.8 (SM50Y)
TOTAL			195.6

- Sectional area and moment of inertia of area:

	Sectional Area (cm^2)	Moment of Inertia of Area (cm^4)
• Concrete section	AC = 4,105	IC = 110,832
• Steel girder section	AS = 195.6	IS = 620,099
• Composite section	AV = 782	IV = 1,951,186

- Geometrical moment of area of concrete (AC x DC)........QC = 97,226 cm^3
- Distance and section modulus (See Fig. 4.4.7):

Distance (cm)		Section Modulus (cm^3)	
D =	94.7	WSU =	7,675
DS =	71.0	WSL =	8,116
DC =	23.7	WVU =	199,399
YSU =	80.8	WVL =	13,236
YSL =	76.4		
YVU =	9.8		
YVL =	147.4		
YVC =	32.7		

- Axial force
  - Due to drying schrinkage.................................NSH = 17.1 ton
  - Due to creep...NCR = 2.5 ton
  - Due to temperature change...............................NTM = 12.0 ton
- Stress (kg/cm^2):

	Concrete Base Slab	Upper Flange	Lower Flange
(1) Stress before compounding	-	- 1,327	1,255
(2) Stress after compounding	- 21.8	- 46	687
(3) Stress due to drying schrinkage	3.1	- 297	110
(4) Stress due to creep	1.9	- 42	16
(5) Stress due to temperature difference	- 0.6	- 207	75
(6) = (1)	-	- 1,327	1,255
Allowable stress	-	- 1,412	2,625
(7) = (1) + (2)	- 21.8	- 1,373	1,942
Allowable stress	- 77.1	- 2,100	2,100
(8) = (1) + (2) + (3) + (4)	- 16.8	- 1,712	2,068
Allowable stress	- 77.1	- 2,415	2,100
(9) = (1) + (2) + (3) + (4) + (5)	- 17.5	- 1,918	2,143
Allowable stress	- 88.7	- 2,730	2,415

Table 4.4.11: STRESS ANALYSIS RESULT OF COMPOSITE GIRDER (NO. 3 MAIN GIRDER, NO. 2 SECTION)

- Bending moment before compounding........................MS = 182.18 t·m
- Bending moment after compounding..........................MV = 165.06 t·m
- Bending moment by dead load after compounding ....MVD = 27.76 t·m
- Base slab thickness ....TS = 18.0 cm
- Haunch...HH = 6.0 cm
- Effective base slab width..BS = 228.1 cm
- Distance between fixed points of flange.......................P = 490.0 cm
- Section and sectional area of steel girder:

	Section (mm)	Sectional Area (cm^2)
• Upper flange :	280 x 14	39.2 (SM50Y)
• Web :	1,550 x 9	139.5 (SM50Y)
• Lower flange :	440 x 19	83.6 (SM50Y)
TOTAL		262.3

- Sectional area and moment of inertia of area:

	Sectional Area (cm^2)	Moment of Inertia of Area (cm^4)
• Concrete section	: AC = 4,105	IC = 110,832
• Steel girder section	: AS = 262.3	IS = 987,001
• Composite section	: AV = 849	IV = 3,032,152

- Geometrical moment of area of concrete (AC x DC)........QC = 134,244 cm^3
- Distance and section modulus (See Fig. 4.4.7):

Distance (cm)		Section Modulus (cm^3)	
D =	105.8	WSU =	10,703
DS =	73.1	WSL =	14,936
DC =	32.7	WVU =	158,724
YSU =	92.2	WVL =	21,783
YSL =	66.1		
YVU =	19.1		
YVL =	139.2		
YVC =	41.7		

- Axial force
  - • Due to drying schrinkage ....NSH = 20.8 ton
  - • Due to creep ....NCR = 3.3 ton
  - • Due to temperature change................................NTM = 15.1 ton
- Stress (kg/cm^2):

	Concrete Base Slab	Upper Flange	Lower Flange
(1) Stress before compounding	-	- 1,702	1,220
(2) Stress after compounding	- 32.4	- 104	758
(3) Stress due to drying schrinkage	4.1	- 284	67
(4) Stress due to creep	1.7	- 44	10
(5) Stress due to temperature difference	- 1.6	- 205	48
(6) = (1)	-	- 1,702	1,220
Allowable stress	-	- 1,765	2,625
(7) = (1) + (2)	- 32.4	- 1,806	1,977
Allowable stress	- 77.1	- 2,100	2,100
(8) = (1) + (2) + (3) + (4)	- 26.6	- 2,134	2,055
Allowable stress	- 77.1	- 2,415	2,100
(9) = (1) + (2) + (3) + (4) + (5)	- 28.2	- 2,338	2,103
Allowable stress	- 88.7	- 2,730	2,415

Table 4.4.12: STRESS ANALYSIS RESULT OF COMPOSITE GIRDER
(NO. 3 MAIN GIRDER, NO. 3 SECTION)

- Bending moment before compounding........................MS = 101.89 t•m
- Bending moment after compounding..........................MV = 90.91 t•m
- Bending moment by dead load after compounding ....MVD = 22.85 t•m
- Base slab thickness ....TS = 18.0 cm
- Haunch...HH = 6.0 cm
- Effective base slab width.......................................BS = 228.1 cm
- Distance between fixed points of flange......................P = 490.1 cm
- Section and sectional area of steel girder:

		Section (mm)	Sectional Area (cm^2)
• Upper flange	:	230 x 11	25.3 (SM50Y)
• Web	:	1,550 x 9	139.5 (SM50Y)
• Lower flange	:	280 x 11	30.8 (SM50Y)
TOTAL			195.6

- Sectional area and moment of inertia of area:

	Sectional Area (cm^2)	Moment of Inertia of Area (cm^4)
• Concrete section	: AC = 4,105	IC = 110,832
• Steel girder section	: AS = 195.6	IS = 620,099
• Composite section	: AV = 782	IV = 1,951,186

- Geometrical moment of area of concrete (AC x DC)........QC = 97,226 cm^3
- Distance and section modulus (See Fig. 4.4.7):

Distance (cm)		Section Modulus (cm^3)	
D =	94.7	WSU =	7,675
DS =	71.0	WSL =	8,116
DC =	23.7	WVU =	199,399
YSU =	80.8	WVL =	13,236
YSL =	76.4		
YVU =	9.8		
YVL =	147.4		
YVC =	32.7		

- Axial force
  - • Due to drying schrinkage ....NSH = 17.1 ton
  - • Due to creep ....NCR = 2.5 ton
  - • Due to temperature change.................................NTM = 12.0 ton
- Stress (kg/cm^2):

	Concrete Base Slab	Upper Flange	Lower Flange
(1) Stress before compounding	-	- 1,328	1,255
(2) Stress after compounding	- 21.8	- 46	687
(3) Stress due to drying schrinkage	3.1	- 297	110
(4) Stress due to creep	1.9	- 42	16
(5) Stress due to temperature difference	- 0.6	- 207	75
(6) = (1)	-	- 1,328	1,255
Allowable stress	-	- 1,412	2,625
(7) = (1) + (2)	- 21.8	- 1,373	1,942
Allowable stress	- 77.1	- 2,100	2,100
(8) = (1) + (2) + (3) + (4)	- 16.8	- 1,712	2,068
Allowable stress	- 77.1	- 2,415	2,100
(9) = (1) + (2) + (3) + (4) + (5)	- 17.5	- 1,918	2,144
Allowable stress	- 88.7	- 2,730	2,415

FIGURES

COORDINATE (Unit; m)

POINTS	X	Y
SU	997,184.581	995,111.825
SD	997,230.345	995,004.147
SI (IP. 1)	997,240.123	994,981.138
SE	997,417.480	994,847.614
IP. 2	997,439.205	994,831.259
SC	997,252.393	994,994.187

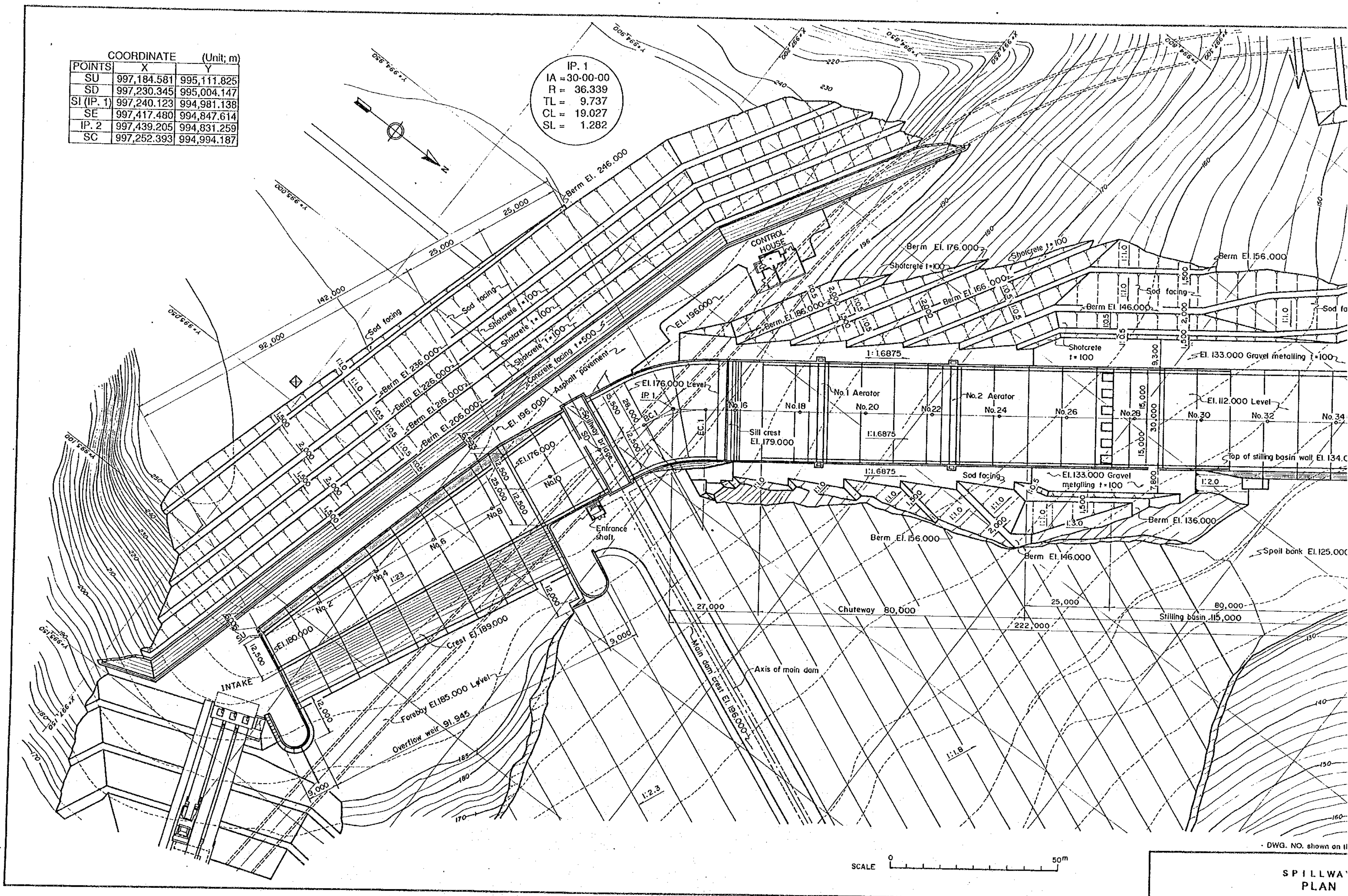
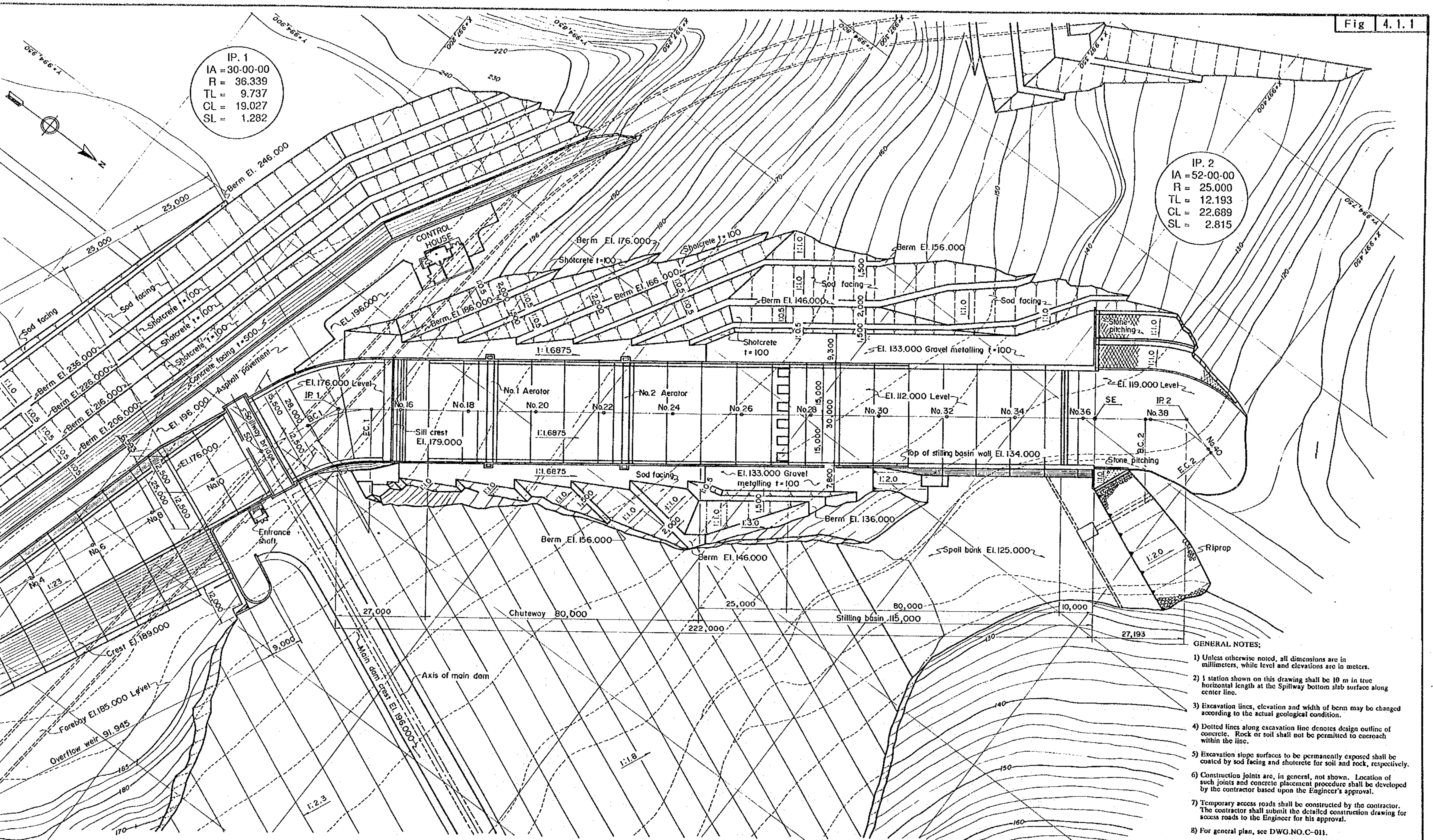


Fig 4.1.1



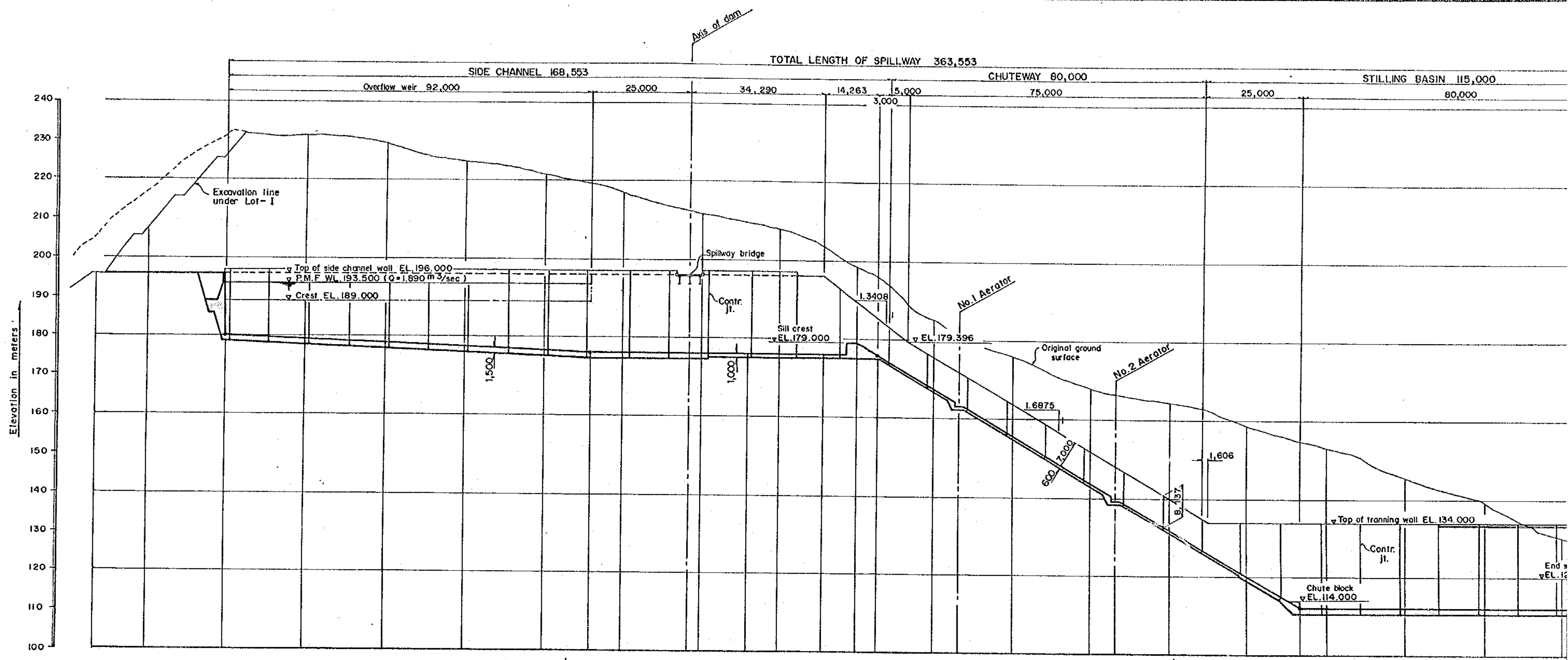
- DWG. NO. shown on this Figure indicates the Tender Drawing No..

SCALE 0 50m

GENERAL NOTES;

1) Unless otherwise noted, all dimensions are in millimeters, while level and elevations are in meters.
2) 1 station shown on this drawing shall be 10 m in true horizontal length at the Spillway bottom slab surface along center line.
3) Excavation lines, elevation and width of berm may be changed according to the actual geological condition.
4) Dotted lines along excavation line denotes design outline of concrete. Rock or soil shall not be permitted to encroach within the line.
5) Excavation slope surfaces to be permanently exposed shall be coated by sod facing and shotcrete for soil and rock, respectively.
6) Construction joints are, in general, not shown. Location of such joints and concrete placement procedure shall be developed by the contractor based upon the Engineer's approval.
7) Temporary access roads shall be constructed by the contractor. The contractor shall submit the detailed construction drawing for access roads to the Engineer for his approval.
8) For general plan, see DWG.NO.C-011.
9) Drain ditch type "A" , refer to DWG.NO.S-001, shall be provided at every 2 m wide berm, but not shown on the drawing.

SPILLWAY PLAN	GOVERNMENT OF MAURITIUS PORT LOUIS WATER SUPPLY PROJECT JAPAN INTERNATIONAL COOPERATION AGENCY



STATION NUMBER	GRADIENT	FORMATION HEIGHT	ORIGINAL GROUND HEIGHT	ACCUMULATED DISTANCE	DISTANCE
No.-3 -4.000		196.000		-34.000	-14.000
No.-2		196.000		-20.000	-20.000
No. 0	180.000	180.000	231.20	0.000	0.000
No. 2		179.130	231.20	20.000	20.000
No. 4	I=1:23(4,348%) L=92,000	178.261	229.30	40.000	20.000
No. 6		177.391	224.80	60.000	20.000
No. 8		176.552	221.50	80.000	20.000
No. 9 +2.000	176.000	176.000		92.000	12.000
No. 10		176.000	216.90	100.000	8.000
No. 11 +7.000		176.000		117.000	17.000
No. 12		176.000	211.60	120.000	3.000
BC. 1		176.000		132.263	12.263
No. 14		176.000	207.70	140.000	7.737
EC. 1		176.000		151.290	11.290
No. 16		176.000	198.50	160.000	8.710
+ 5.553	176.000	176.000		165.553	5.553
No.18		167.439	185.20	180.000	14.447
+6.553		162.963		186.553	6.553
No. 20		155.587	175.80	200.000	13.447
No.22	I=1:1.6875 (59.259%) L=108,000	143.735	167.90	220.000	20.000
+ 6.553		139.239		226.553	6.553
No.24		131.883	164.30	240.000	13.447
No.26		120.031	158.50	260.000	20.000
No. 27 +3.553	112.000	112.000		273.553	13.553
No. 28		112.000	153.00	280.000	6.447
No.30		112.000	145.40	300.000	20.000
No.32	Level L=80,000	112.000	139.20	320.000	20.000
No.34		112.000	130.20	340.000	20.000

SCALE 0 50m

PROFILE

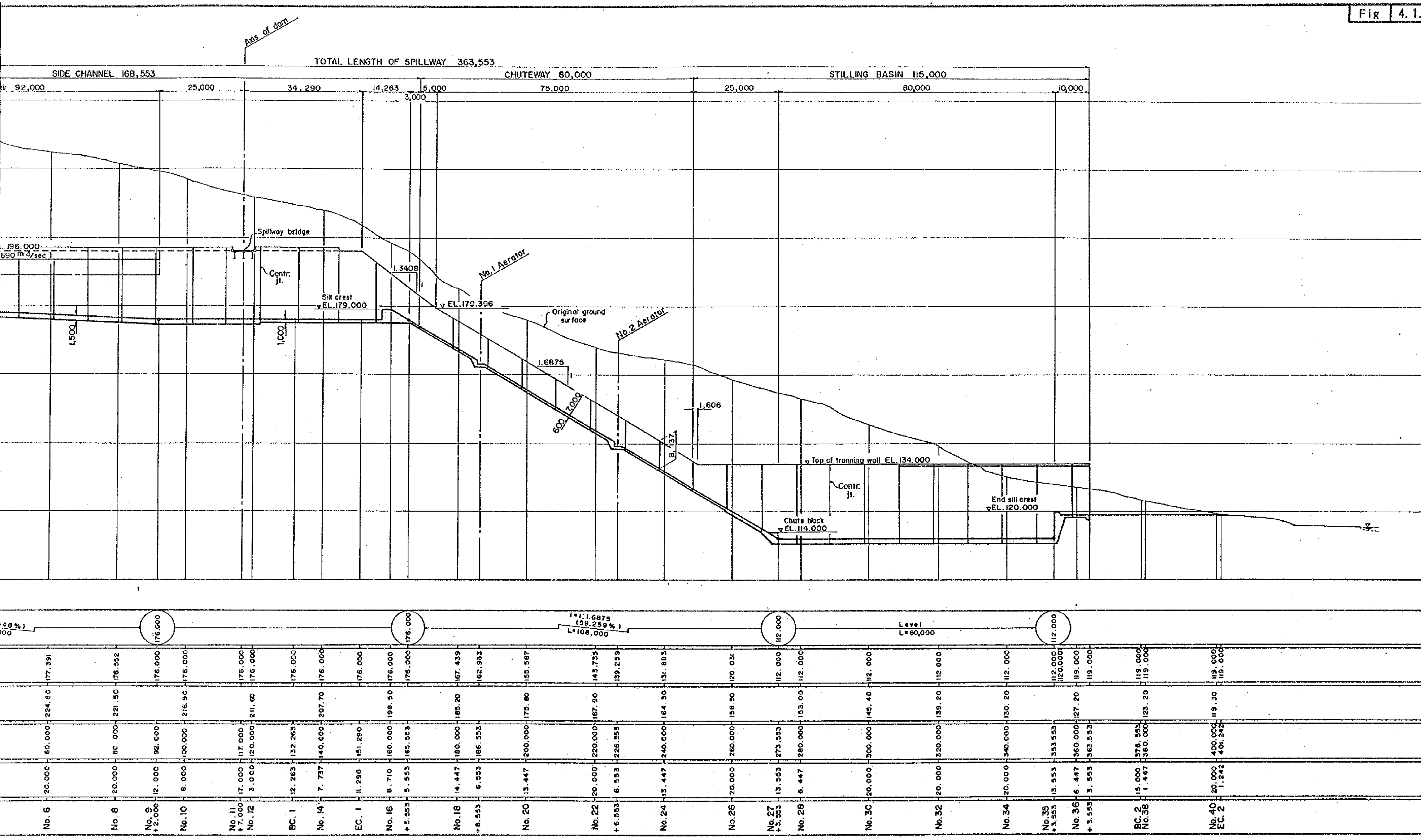
NOTE;

For notes, see DWG.NO.C-051.

- DWG. NO. shown on this Figure indicates the Tender Drawing No..

SPIL
PROF

Fig 4.1.2



PROFILE

NOTE;

For notes, see DWG.NO.C-051.

- DWG. NO. shown on this Figure indicates the Tender Drawing No..

SPILLWAY
PROFILE

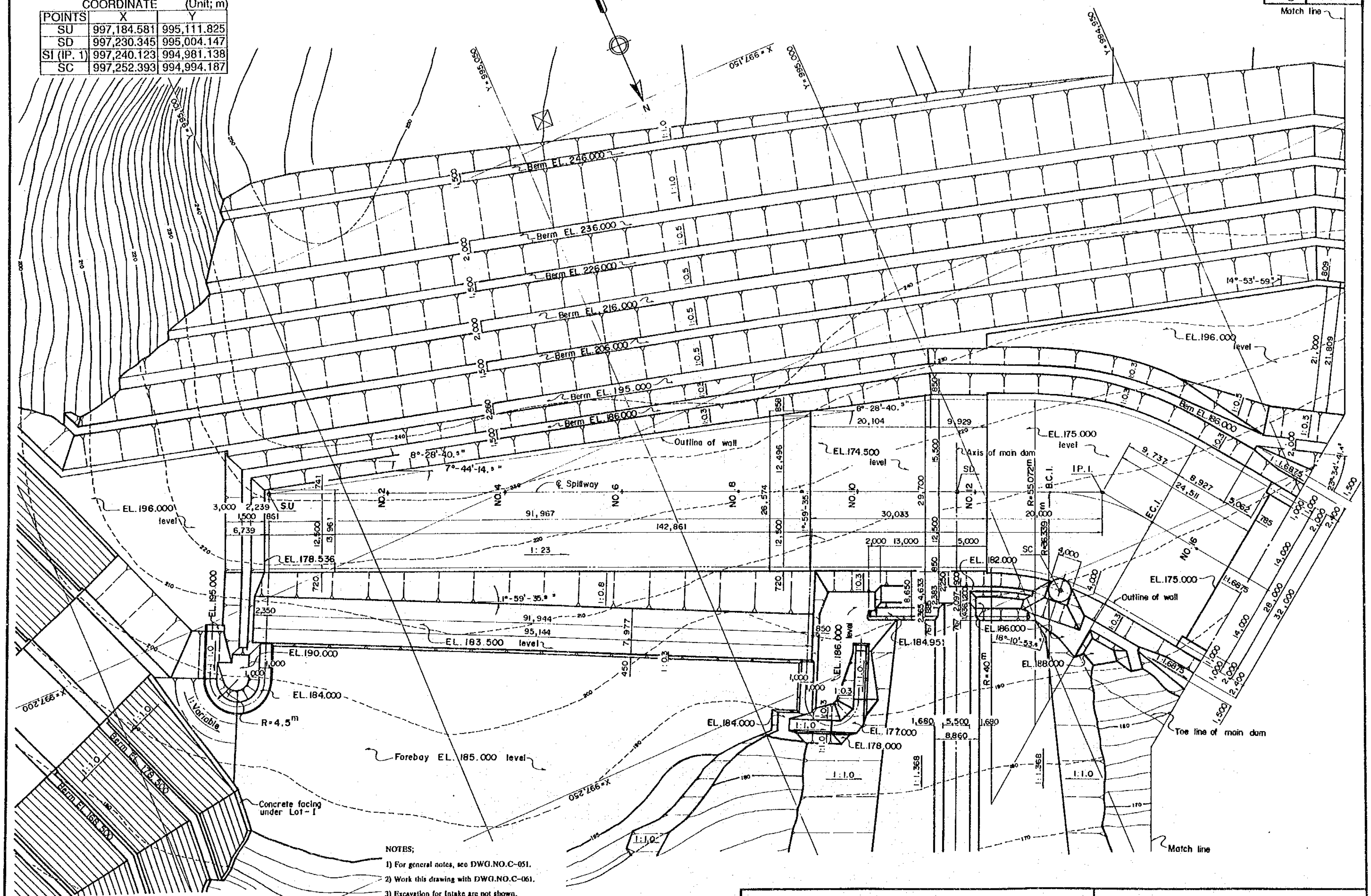
GOVERNMENT OF MAURITIUS
PORT LOUIS WATER SUPPLY PROJECT

JAPAN INTERNATIONAL COOPERATION AGENCY

Fig 4.1.3

COORDINATE (Unit; m)

POINTS	X	Y
SU	997,184.581	995,111.825
SD	997,230.345	995,004.147
SI (IP. 1)	997,240.123	994,981.138
SC	997,252.393	994,994.187



NOTES;

1) For general notes, see DWG.NO.C-051.

2) Work this drawing with DWG.NO.C-061.

3) Excavation for Intake are not shown, see DWG.NO.C-104 and C-105.

4) Trench excavation for drain ditch are not shown on the drawing.

・DWG. NO. shown on this Figure indicates the Tendor Drawing No..

SPILLWAY
EXCAVATION PLAN (1)

GOVERNMENT OF MAURITIUS
PORT LOUIS WATER SUPPLY PROJECT

JAPAN INTERNATIONAL COOPERATION AGENCY