4.4 Building Work

4.4.1 General Description

4.4.1.1 General

The major design criteria applied in calculations are standerd requirements conforming to "Architectural Institute of Japan Standard for Structural Caiculation of Rainforced Concrete Structures and Commentary" and "Architecteral Institute of Japan Standard for Structural Calcuration of Steel Structures".

4.4.1.2 Design Criteria

Loading Condition

In this structural caicurations, the loads and external forces that act on the structure are the following.

(A) Dead load
(B) Live load

Table 1 Combination of Loads

Condition of Stresses		Combination of Stress
Permanent stress	Normal time	G + P

where ;

G ; stress due to dead load
P ; stress due to live load

The dead and live loads of each part of building are applied in accordance with the Japanese Building Standard Law Enforcement Order.

4.4.1.3 Structural Analysis

(1) Structural analysys

Stress analysys of reinforced concrete frames are carried out through a computer , NEC PC-9801.

Structural analysys for the vertical and holizontal load is obtained through the stiffnsee matrix method considering the axial , flexual and shering deformations based on elastic theory.

(2) Moduius of Elasticity

Module of elasticity are as follows ;

Concrete ; Ec = 215.2 t/cm2
Steel ; Es = 2100 t/cm2

Shear modulus of elasticity as follows ;

Concrete ; Gc = 92.2 t/cm2
Steel ; Gs = 810 t/cm2

4.4.1.4 Design of Members

The design of reinforced concrete structure shall be based on "AIJ Standerd for structural Calculation of Reinforced Concrete Structure".

Compressive strength of concrete at 28 days shall be 210 Kg/cm2 and more.

Reinforcement bar materials shall comply with deformed bar, "SD295". (JIS G 3112)

Weight of reinforced concrete shall be calculated as 2.4 t/m3 and the "Young Ratio" of reinforcement bar to concrete shall be "n = 15".

4.4.1.5 Allowable Design Stress of Materials

(1) Concrete and Reinforcing Bar

Allowable design stress of concrete and reinforcing bar will be summarized as follows ;

	Tension	Compression	Shear
Concrete (Fc=210kg/cm2	-------	Fc/3 = 70	4.25
Rain.-bar (JIS G 3112)	1800	1800	1000

Allowable bond stress per unit surface of reinforcing bar shall be shown as follows ;

	Top bars	Other bars
Deformed bar	Fc/15 14.0	Fc/10 21.0

Top bar , in reference to load, shall be holizontal bar so placed that more than 30cm of concrete is casted in the member below the bar.

(2) Allowable Bearing Capacity of Soil

The bearing capacity of soil is 30 t/m2 for permanent load.

4.4.2 Dam Control House

§ .1 ASSUMED LOAD

FLOOR LOAD TABLE

TITLE	MATERIAL	(t/m3)	TICK. (cm)	WEIGHT (kg/m2)		DL (kg/m2)	LL (kg/m2)	TL (kg/m2)	NOTE
ROOF	WATER PROOFING			10	TO FLOOR	360	180	540	
	CEMENT MORTAR	2.00	2.0	40	TO BEAM		180	540	
	SLAB	2.40	12.0	288	TO FRAME		130	490	
	CEILING			20					
FLOOR	FINISHED	2.00	3.0	60	TO FLOOR	400	300	700	
	SLAB	2.40	13.0	312	TO BEAM		300	700	
	CEILING			20	TO FRAME		180	580	
CANOPY	WATER PROOFING			10	TO FLOOR	430	180	610	
	CEMENT MORTAR	2.00	2.0	40	TO BEAM		180	610	
	SLAB	2.40	15.0	360	TO FRAME		130	560	
	CEILING			20					
BALCONY	CEMENT MORTAR	2.00	2.0	40	TO FLOOR	460	180	640	
	SLAB	2.40	16.5	396	TO BEAM		180	640	
	CEILING			20	TO FRAME		130	590	

DEAD LOAD OF GIRDER, COLUMN, WALL

① GIRDER, BEAM

NO	B	D	CONCRETE	FINISHED	WEIGHT
	25.0	60.0	288	63	360
	25.0	80.0	408	83	500
	35.0	60.0	504	0	510

② COLUMN

NO	B	D	CONCRETE	FINISHED	WEIGHT
	30.0	30.0	216	65	290

③ WALL

NO	t	CONCRETE	FINISHED	WEIGHT
CB20	20.0	310	100	410

5.2 PREPARTORY CALCULATION

2-1 CALCULATION OF AXIAL FORCE OF COLUMNS

NO	FLOOR	TITLE	CALCULATION		W (t)	ΣW (t)
C - 1	2	ROOF	0.49 x 2.6 x 3.0	3.8		
		G	0.36 x (2.3 + 2.7)	1.8		
		C	0.29 x 3.6 / 2	0.6	6.1	
	1	FL	0.59 x 2.6 x 3.0	4.6		
		G		1.8		
		C	0.29 x 3.6	1.0	7.3	13.4
	F	Fd	0.44 x 5.0	2.2		
		CB20	0.41 x 5.0 x 3.0	6.2		
		C		0.6	8.9	22.3
B - 1	2	ROOF	0.49 x 2.6 x 4.0	5.1		
		G	0.36 x (2.3 + 3.7)	2.2		
		C		0.6	7.9	
	1	FL	0.58 x 2.6 x 4.0	6.0		
		G		2.2		
		C		1.0		
		CB20	0.41 x 3.7 x 3.0	4.6	13.8	21.6
	F	Fd	0.44 x 3.7	1.6		
		C		0.6		
		CB20		4.6	6.7	28.3
C - 2	2	ROOF	0.49 x 4.0 x 3.0	5.9		
		G	0.36 x (3.7 + 2.7)	2.3		
		C		0.6	8.7	
	1	FL	0.58 x 4.0 x 3.0	7.0		
		G		2.3		
		C		1.0		
		CB20	0.41 x 3.7 x 3.0	4.6	14.9	23.6
	F	Fd	0.44 x 3.7	1.6		
		C		0.6		
		CB20		4.6	6.7	30.3

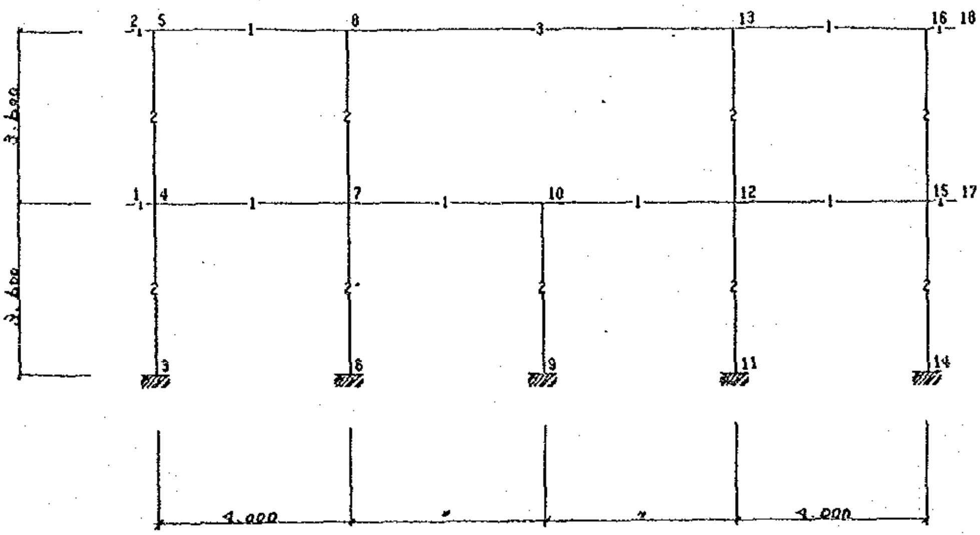
A 2 3 [illegible] (52 x 36)

NO	FLOOR	TITLE	CALCULATION		W (t)	ΣW (t)
B-2	2	Roof	0.49 × 4.0 × 6.0	11.8		
		W	0.36 × (11.85 + 2.7 + 2.75/2)	2.7		
			0.50 × 2.85	1.9		
		C		0.5	16.9	
	1	FL	0.58 × 4.0 × 4.0	9.3		
		W	0.36 × 7.4	2.7		
		C		1.0	13.0	26.9
	F	C		0.5	0.5	27.4

§3. CALCULATION OF STRESS.

3.1 INPUT DATA

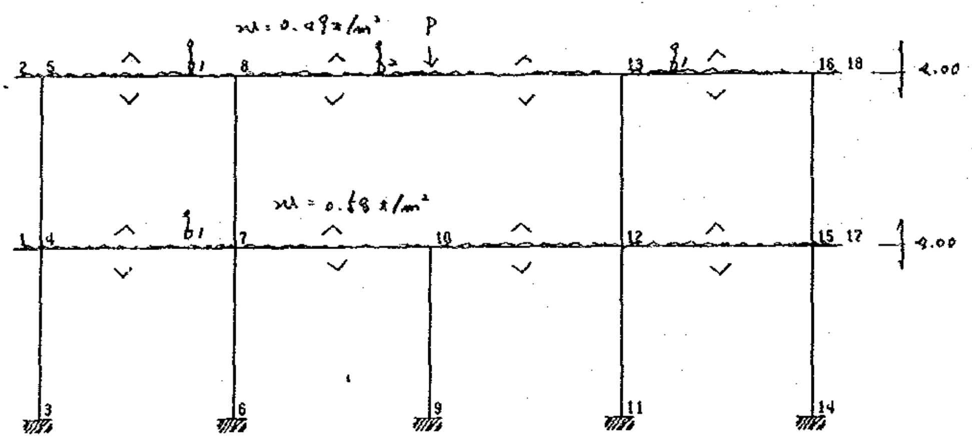
Ⓑ FRAME



① 250 × 600

② 300 × 300

③ 250 × 800

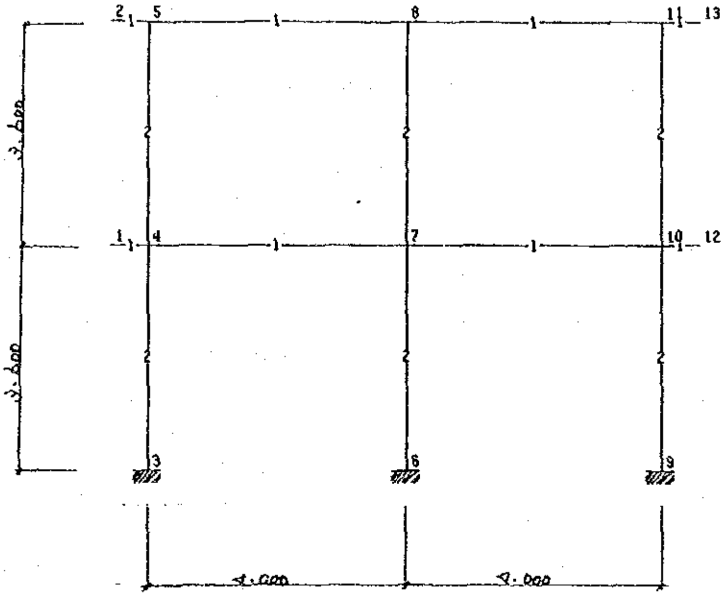


w_1 = 0.36 t/m

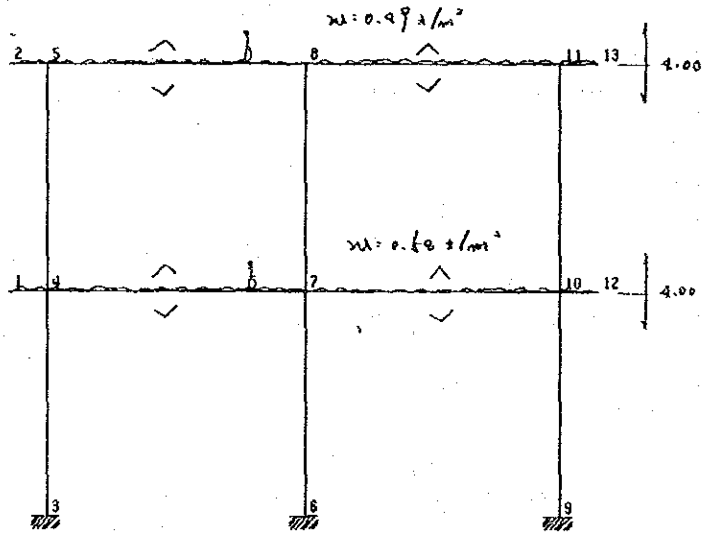
P = 0.36 × 3.75 = 1.35 t

w_2 = 0.60 t/m

② FRAME



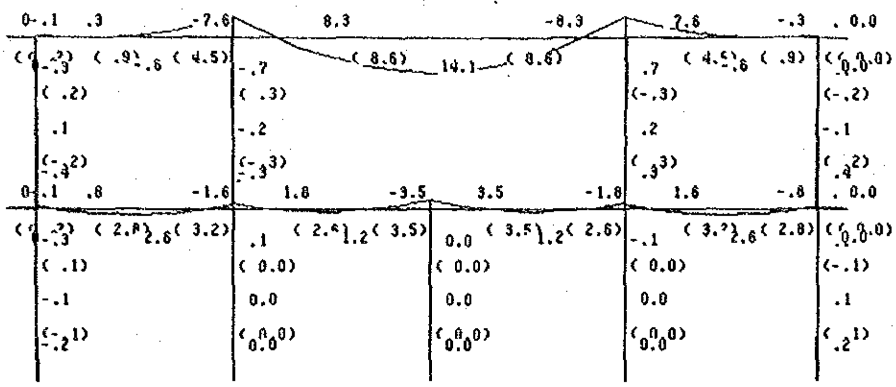
① 250 × 600
② 300 × 300



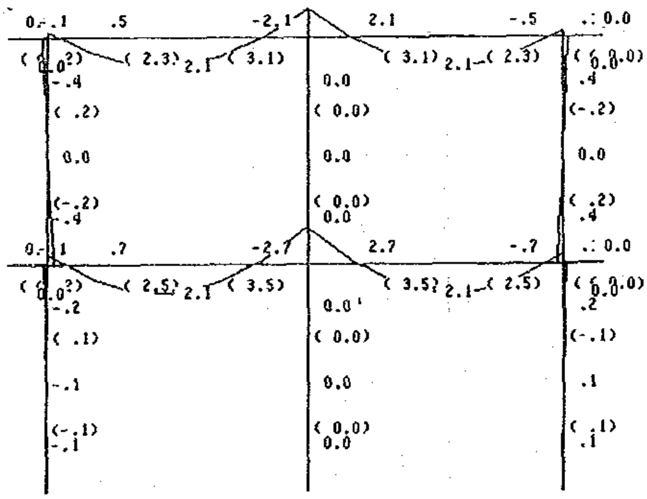
g : 0.36 t/m

3.2 STRESS DIAGRAM

Ⓑ FRAME



② FRAME



3.4 DESIGN OF GIRDER AND COLUMN

4.1 GIRDER

NO (/')	PLACE	SECTION b (cm) / bd (10³cm²)	D (cm) / bd² (10³cm³)	d (cm) / j (cm)	M (tm) L.E / L.F	E / E'	TE / TF	Q (t) L / Qa	E / ...	T / Qa	MAIN BAR C (kg/cm²) / I	pt (%) / at (cm²)	ψE (cm) / ψY (cm)	STIRRUP τ (kg/cm²) / a	pw	TOP BAR / BOTTOM BAR
G		25	80	74	8.3			8.6				7.22		5.31	0.20	3 - D19
RB1 A 2~4	E													1.34		3
																3
	C				19.1							12.10		D10@	200	6
( )	E															
G		25	60	54	7.6			9.5				4.94		3.81	0.20	4 - D19
RB2 B 1~2	E															3
																3
	C				2.6							2.21		D10@	200	3
( )	E															
G		25	60	54	2.1			2.1				2.47		2.62	0.20	3 - D19
RB3 2 B~C	E															3
																3
	C				2.1							2.47		D10@	200	3
( )	E															
G		25	60													3 - D19
RB4	E															3
																3
	C													D10@	200	3
( )	E															
G		25	60	54	3.5			3.5				2.12		2.96	0.20	3 - D19
1B1 B 2~3	E															3
																3
	C				1.2							1.41		D10@	200	3
( )	E															
G		25	60													3 - D19
1B2	E															3
																3
	C													D10@	200	3
( )	E															
G		25	60													4 - D19
1B3	E															3
																3
	C													D10@	200	3
( )	E															
FG1		30	70													3 - D19
	E															3
																3
	C													D10@	200	3

4.2 COLUMN

NO		SECTION b(cm)	D(cm)	bD (10^3cm^2)	bD^2 (10^3cm^3)	STRESS	P (t)	M_1 (tm)	Q (t)	MAIN BAR P/bD (kg/cm^2)	M/bD^2 (kg/cm^2)	P_t (%)	a_t (cm^2)	HOOP τ (kg/cm^2)	nφ @ (cm)	Y / X ΣN φ
C_1 / B-2	X					L	16.9	1.7	0.3			0				
						E										
						T_1									D10	
		30	30			T_2							0	0.98	@100	
	Y					L										
						E										
						T_1										4-D19
						T_2										
C	X					L										
						E										
						T_1										
						T_2										
	Y					L										
						E										
						T_1										
						T_2										
C	X					L										
						E										
						T_1										
						T_2										
	Y					L										
						E										
						T_1										
						T_2										
C	X					L										
						E										
						T_1										
						T_2										
	Y					L										
						E										
						T_1										
						T_2										
C	X					L										
						E										
						T_1										
						T_2										
	Y					L										
						E										
						T_1										
						T_2										
C	X					L										
						E										
						T_1										
						T_2										
	Y					L										
						E										
						T_1										
						T_2										

5.5 DESIGN OF SLAB

	NAME	Wo	LL	Lx	Ly	reqt	W	NO	t	PLACE		M	At		
1	ROOF	70	180	400	400	9.90	539	S1	12.0	MAIN BAR X	(U)	0.36	2.85	D10	@200
											(D)	0.24	1.90	D10	@200
										MAIN BAR Y	(U)	0.36	3.25	D10	@200
											(D)	0.24	2.17	D10	@200
2	FL	80	300	400	400	10.68	692	S2	13.0	MAIN BAR X	(U)	0.46	3.25	D13	@200
											(D)	0.31	2.17	D13	@200
										MAIN BAR Y	(U)	0.46	3.66	D13	@200
											(D)	0.31	2.44	D13	@200

S2

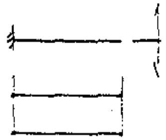
W = W =
q = 0.61 t/m
p =

t = 15 cm (d = 11 cm)

(B.E.) M = 0.31 tm
Q = 0.61 t

at = 1.79 cm² (1.5 at = 2.68 cm²) [D10 @200 / D10 @200
τ = 0.64 kg/cm² < fs

(T.E.) [



W = W =
q =
p =

= (d =)

(B.E.) M =
Q =

at = (at =) [
τ =

(T.E.) [

S4

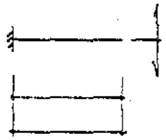
W = W =
q = 0.70 t/m
p = 0.34×1.1 = 0.37 t

t = 15 cm (d = 11 cm)

(B.E.) M = 0.35 + 0.37 = 0.72 tm
Q = 0.70 + 0.37 = 1.07 t

at = 4.16 cm² (1.5 at = 6.23 cm²) [D13 @150 / D10 @150
τ = 1.11 kg/cm² < fs

(T.E.) [



W = W =
q =
p =

= (d =)

(B.E.) M =
Q =

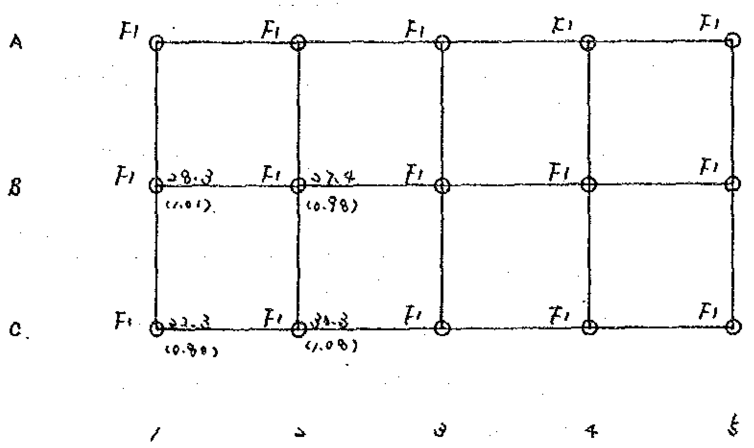
at = (at =) [
τ =

(T.E.) [

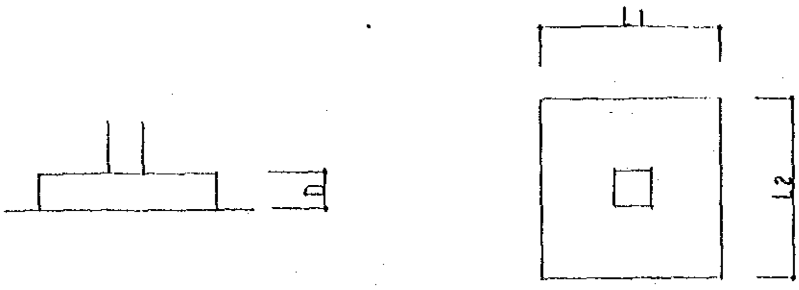
3.6 DESIGN OF FOUNDATION

$fa = 30.0\ t/m^2$

$fa' = 30.0 - 2.0 \times 1.0 \times 1.0^2 = 28.0\ t/m^2$ ( ) : req A



F1 : 1100 × 1100 ($A = 1.21\ m^2$)



F 1	L1 x L2 = 1,100 x 1100	D = 45 cm	(d = 35 cm)

M = 28.0 × 1.10 × 0.40² / 2 = 2.5 tm
Q = 〃 × 0.40 = 12.32

at = 2.54 cm²   6 - D16
ø = 19.3 cm
τ = 3.65 kg/cm² < fs

F	L1 x L2 =	D =	(d =)

M =
Q =

at =
ø =
τ =

F	L1 x L2 =	D =	(d =)

M =
Q =

at =
ø =
τ =

F	L1 x L2 =	D =	(d =)

M =
Q =

at =
ø =
τ =

4.4.3 Gate Control House

§ .1 ASSUMED LOAD

FLOOR LOAD TABLE

TITLE	MATERIAL	(t/m3)	TICK. (cm)	WEIGHT (kg/m2)		DL (kg/m2)	LL (kg/m2)	TL (kg/m2)	NOTE
ROOF	WATER PROOFING			10	TO FLOOR		180	540	
	CEMENT MORTAR	2.00	2.0	40					
	SLAB	2.40	12.0	288	TO BEAM	360	180	540	
	CEILING			20					
					TO FRAME		130	490	

DEAD LOAD OF GIRDER,COLUMN,WALL

① GIRDER,BEAM

NO	B	D	CONCRETE	FINISHED	WEIGHT
	25.0	60.0	288	63	360

② COLUMN

NO	B	D	CONCRETE	FINISHED	WEIGHT
	25.0	40.0	240	70	310

③ WALL

NO	t	CONCRETE	FINISHED	WEIGHT
W20	20.0	480	100	580

コクヨ コヒ−15D (52×36)

5.2 PREPARTORY CALCULATION

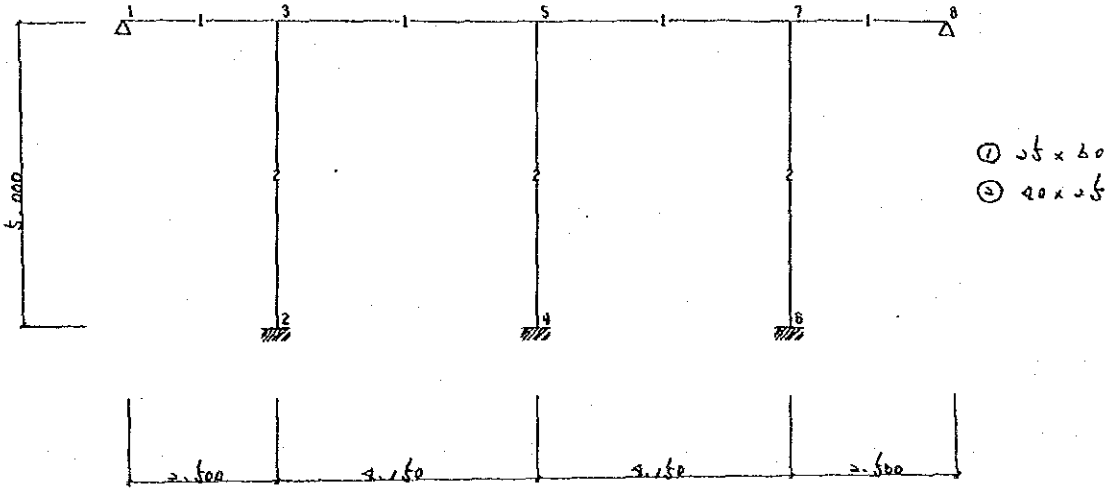
2.1 CALCULATION OF AXIAL FORCE OF COLUMNS

NO	FLOOR	TITLE	CALCULATION		W (t)	ΣW (t)
A-1	1	ROOF	0.49 × 3.325 × 3.07	5.0		
		B	0.36 × (3.075 + 2.77)	2.1		
		C	0.31 × 5.0 / 2	0.8		
		W20	0.59 × 3.075 × 5.0 / 2	4.5	12.4	
	F	C		0.8		
		W20		4.5	5.3	17.7
A-2	1	ROOF	0.49 × 4.15 × 3.07	6.2		
		B	0.36 × (3.90 + 2.77)	2.4		
		C		0.8		
		W20	0.58 × 3.90 × 5.0 / 2	5.7	15.1	
	F	C		0.8		
		W20		5.7	6.5	21.6

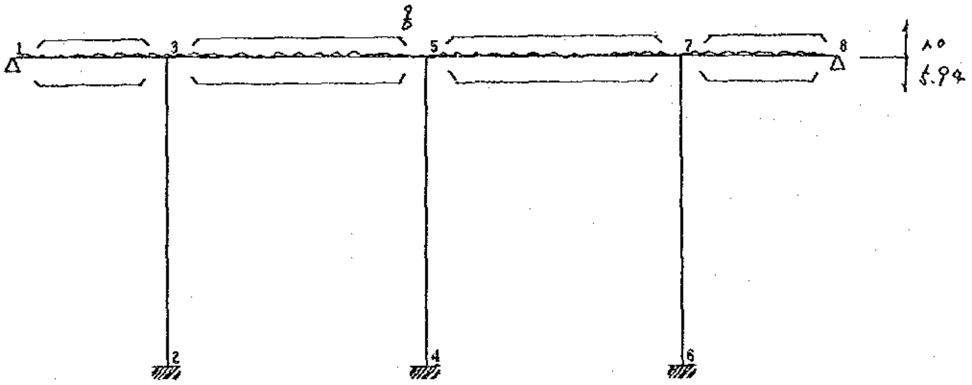
§.3 CALCULATION OF STRESS

3.1 INPUT DATA

Ⓐ FRAME

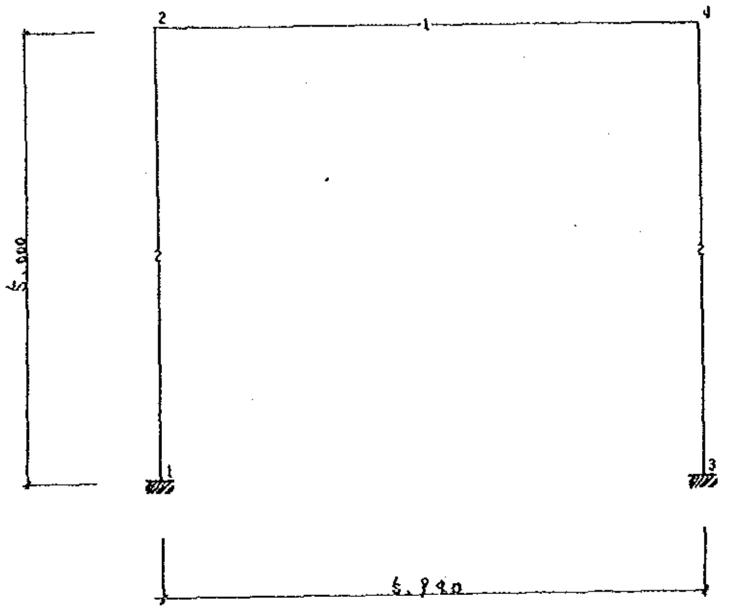


w = 0.49 t/m²

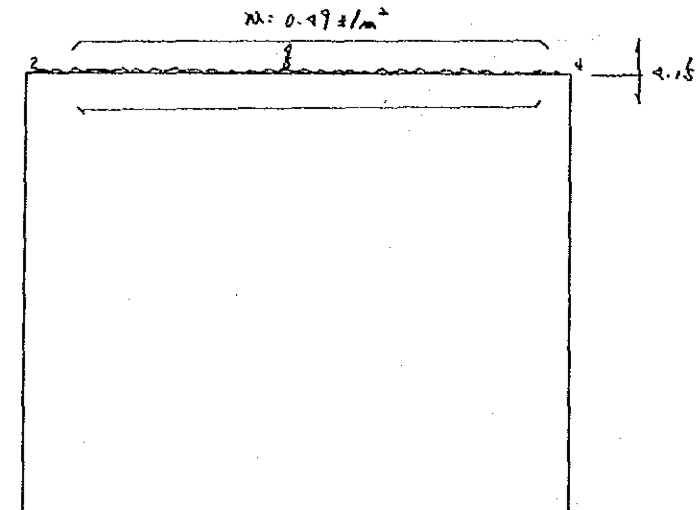


g/b : 0.36 t/m.

② FRAME



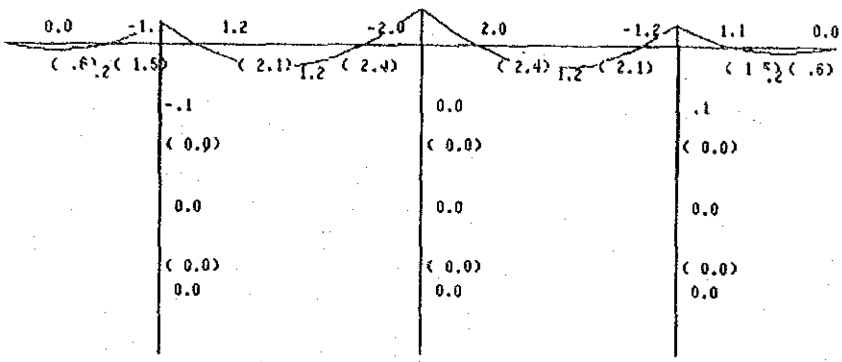
① 25 × 60
② 25 × 40



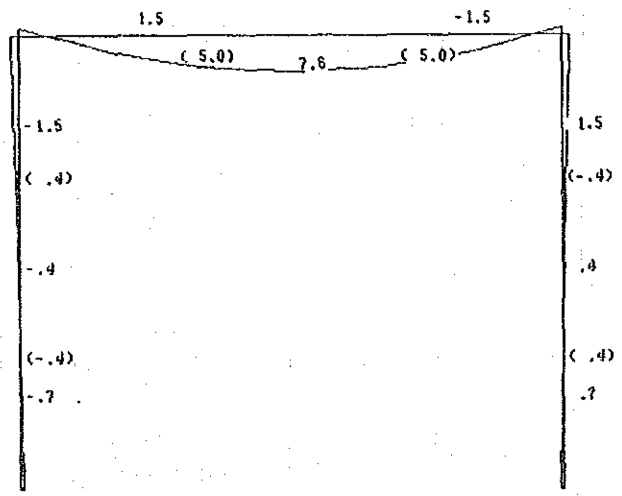
q = 1.36 t/m.

3.2 STRESS DIAGRAM

Ⓐ FRAME



③ FRAME



4.1 GIRDER

QD – QL

NO (′)	PLACE	SECTION b (cm) / bd (10²cm²)	D (cm) / bd² (10³cm³)	d (cm) / j (cm)	M (tm) L.E / L.F	E / E′	T.E / T.F	Q (t) L / QL	E	T / QL	MAIN BAR C (kg/cm²) / t	pt (%) / at (cm²)	φt (cm) / φF (cm)	STIRRUP r (kg/cm²) / a	pw	TOP BAR / BOTTOM BAR
GB1		25	60	54	1.5			5.0				0.76		4.23	0.20	3 – D16
	E															3
																3
	C				2.6							0.94		D10@	200	5
( )	E															
GB2		25	60	54	2.0			2.4				0.35		2.03	0.20	3 – D16
	E															3
																3
	C				1.2							0.91		D10@	200	3
( )	E															
G																– D
	E															
	C															
( )	E															
G																– D
	E															
	C															
( )	E															
G																– D
	E															
	C															
( )	E															
G																– D
	E															
	C															
( )	E															
G																– D
	E															
	C															
( )	E															

2.2 COLUMN

NO		SECTION b (cm)	D (cm)	bD ($10^3 cm^2$)	bD^2 ($10^3 cm^3$)		STRESS P (t)	M (tm)	Q (t)	MAIN BAR P/bD (kg/cm²)	M/bD² (kg/cm²)	P_t (%)	at (cm²)	HOOP τ (kg/cm²)	nφ @ (cm)	Y → X ΣN φ
C1 A-2	X					L										
						E										
						T_1										
						T_2										
	Y					L	15.1	15	0.4			0				
						E										
						T_1										
		25	40			T_2							0	0.54	D10 @100	8 - D16
C	X					L										
						E										
						T_1										
						T_2										
	Y					L										
						E										
						T_1										
						T_2										
C	X					L										
						E										
						T_1										
						T_2										
	Y					L										
						E										
						T_1										
						T_2										
C	X					L										
						E										
						T_1										
						T_2										
	Y					L										
						E										
						T_1										
						T_2										
C	X					L										
						E										
						T_1										
						T_2										
	Y					L										
						E										
						T_1										
						T_2										
C	X					L										
						E										
						T_1										
						T_2										
	Y					L										
						E										
						T_1										
						T_2										

8.5 DESIGN OF SLAB

	NAME	Wo	LL	Lx	Ly	reqt	W	NO	t	PLACE		M	At		
1	ROOF	50	180	390	594	11.27	518	S1	12.0	MAIN BAR X	(U)	0.55	4.39	D13	@200
											(D)	0.37	2.93	D13	@200
										MAIN BAR Y	(U)	0.33	2.98	D10	@200
											(D)	0.22	1.99	D10	@200
2	ROOF	50	180	100	665	2.62	518	S2	12.0	MAIN BAR X	(U)	0.04	0.34	D10	@200
											(D)	0.03	0.23	D10	@200
										MAIN BAR Y	(U)	0.02	0.20	D10	@250
											(D)	0.01	0.13	D10	@250

8.6 DESIGN OF FOUNDATION

$f_a = 30.0\ t/m^2$

$f_a' = 30.0 - 2.0 \times 1.0 \times 1.0^2 = 28.0\ t/m^2$

(A-1)

$N = 17.7\ t$

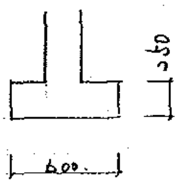
$req\ W = 17.7 / (28.0 \times 3.325) = 0.19 \rightarrow 600$

(A-2)

$N = 21.6\ t$

$req\ W = 21.6 / (28.0 \times 4.15) = 0.19 \rightarrow 600$

FOOTING



$\sigma = 21.6 / (4.15 \times 0.60) = 8.7\ t/m^2$

$M = 8.7 \times 0.30^2/2 = 0.4\ tm$

$Q = 8.7 \times 0.30 = 2.6\ t$

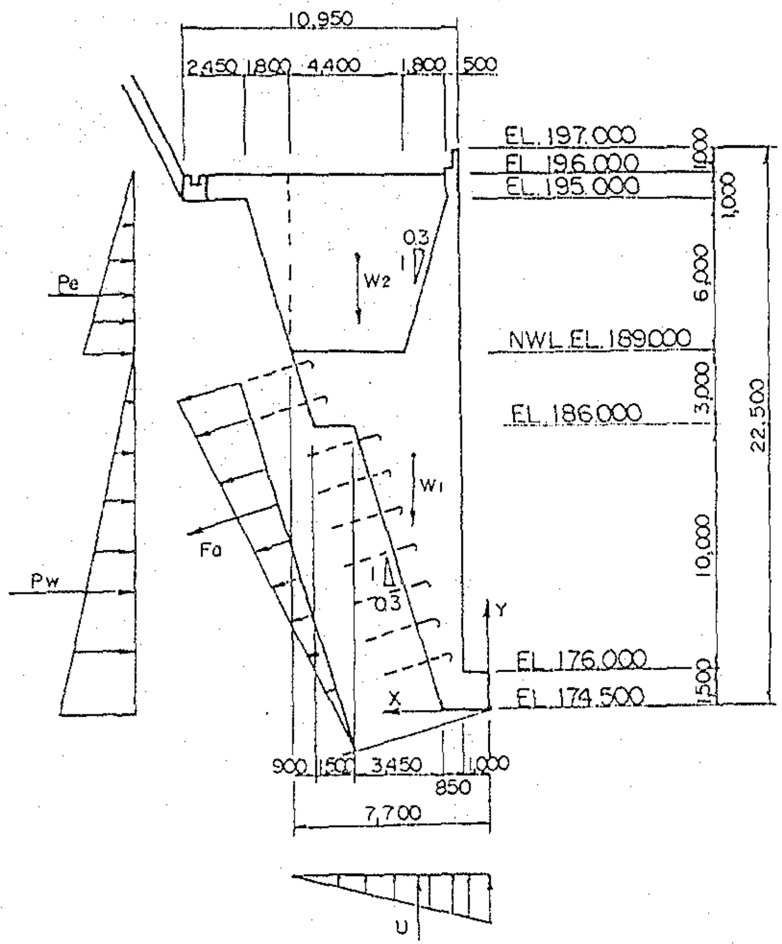
$D = 25\ cm \quad (d = 15\ cm)$

$a_t = 1.69\ cm^2$ D13 @ 200

$\tau = 1.98\ kg/cm^2 < f_s$

$\psi = 9.43\ cm$

Table 4.1.1: 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)

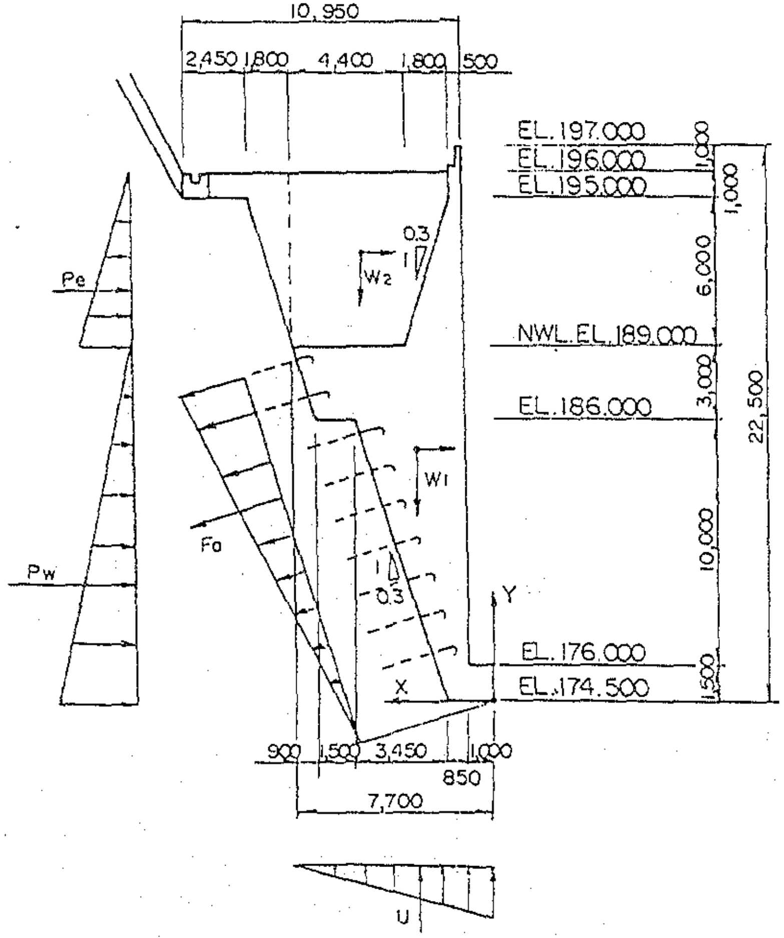
Safety factor for sliding: $Fs = \frac{f \cdot \Sigma V + \tau \cdot A + Ha}{\Sigma H} = \frac{0.55 \times 158.545 + 20 \times 3.35 + 63.306}{116.532} = 1.87 > 1.5$

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

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 4.1.2: 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)

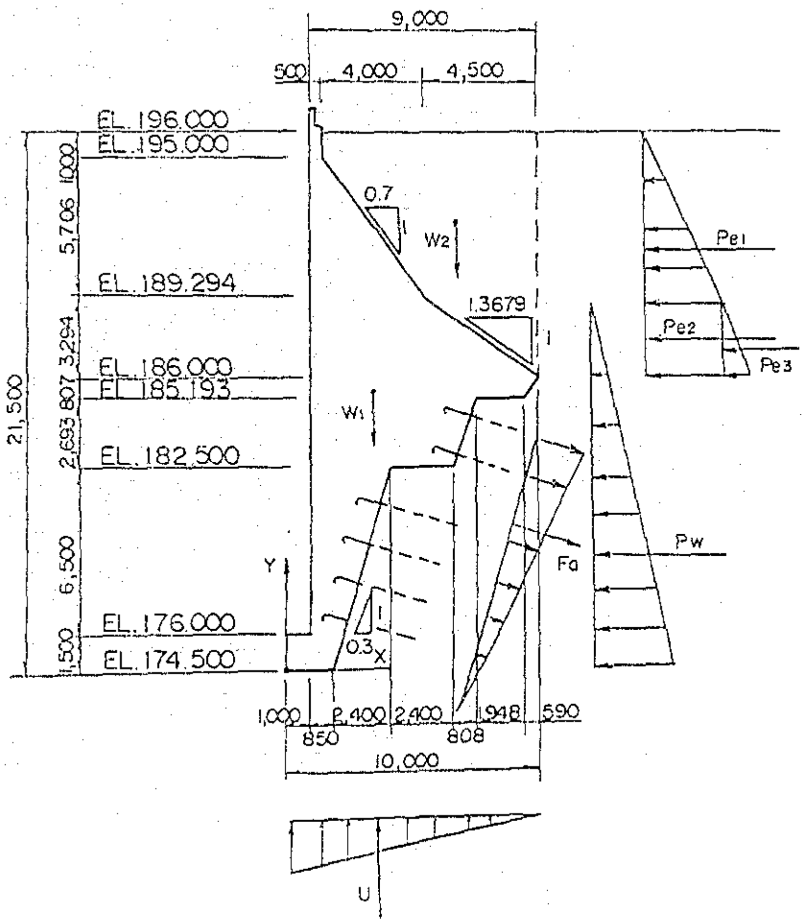
Safety factor for sliding: $Fs = \frac{f \cdot \Sigma V + \tau \cdot A + Ha}{\Sigma H} = \frac{0.55 \times 158.545 + 20 \times 3.35 + 63.306}{128.312} = 1.7 > 1.2$

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 4.1.3 : 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)

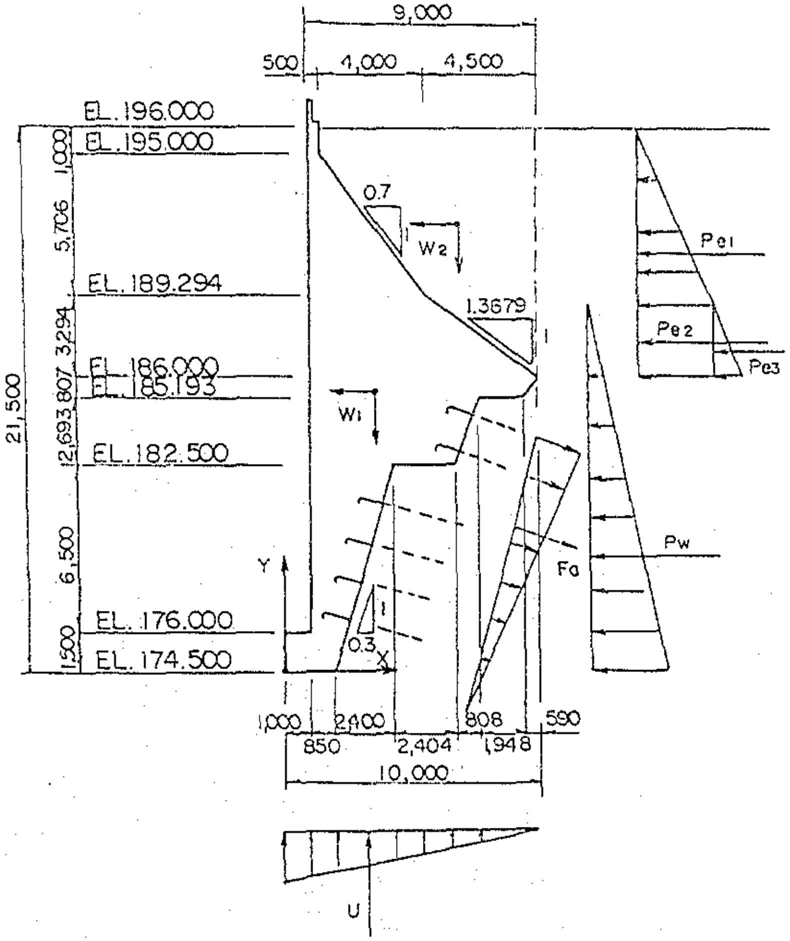
Safety factor for sliding: $Fs = \frac{f \cdot \Sigma V + \tau \cdot A + Ha}{\Sigma H} = \frac{0.55 \times 199.615 + 20 \times 6.202 + 68.564}{121.214} = 2.5 > 1.5$

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 4.1.4 : 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)

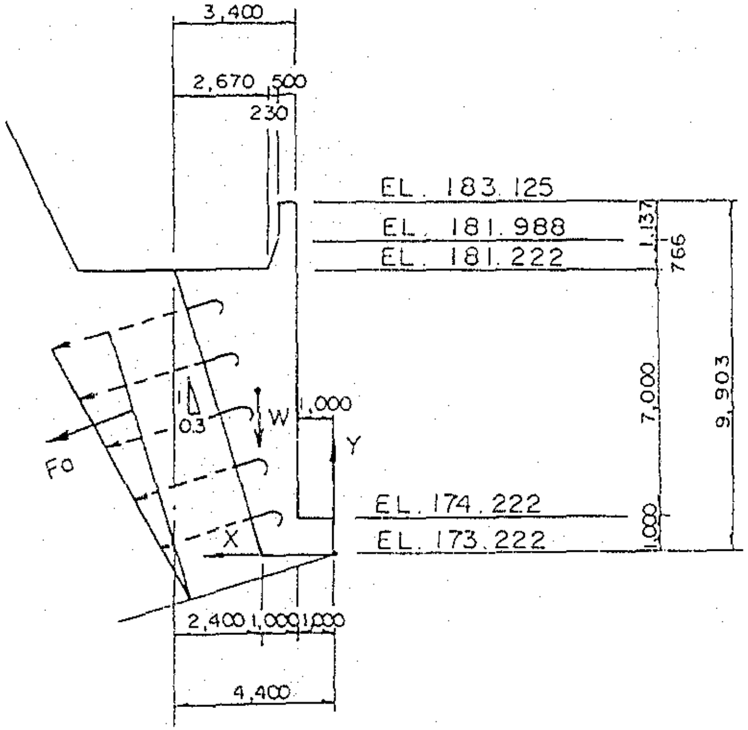
Safety factor for sliding: $Fs = \frac{f \cdot \Sigma V + \tau \cdot A + Ha}{\Sigma H} = \frac{0.55 \times 199.615 + 20 \times 6.202 + 68.564}{136.750} = 2.2 > 1.2$

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 4.1.5: 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)

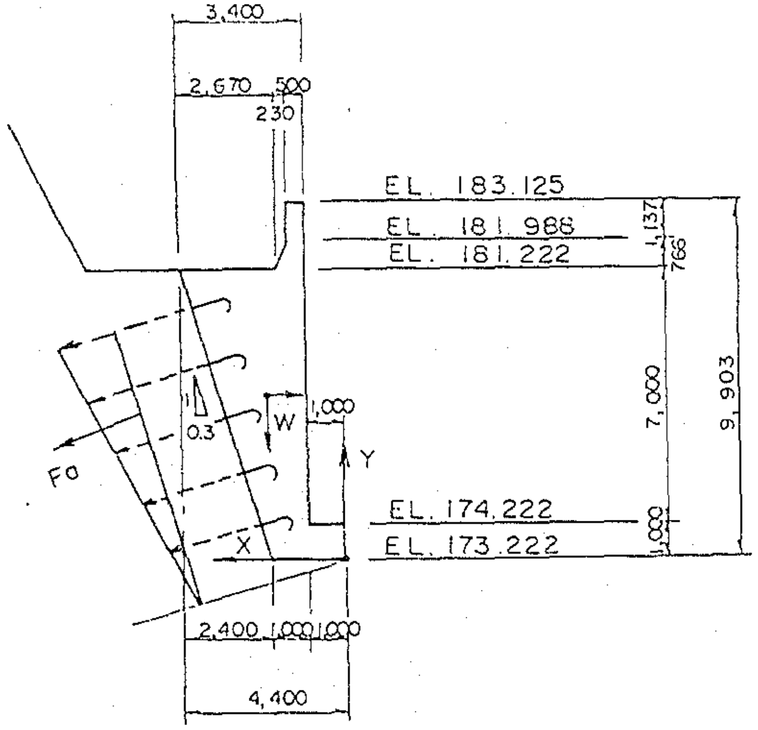
Safety factor for sliding: $Fs = \frac{f \cdot \Sigma V + \tau \cdot A + Ha}{\Sigma H} = \infty > 1.5$

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 4.1.6: 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)	Mt (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)

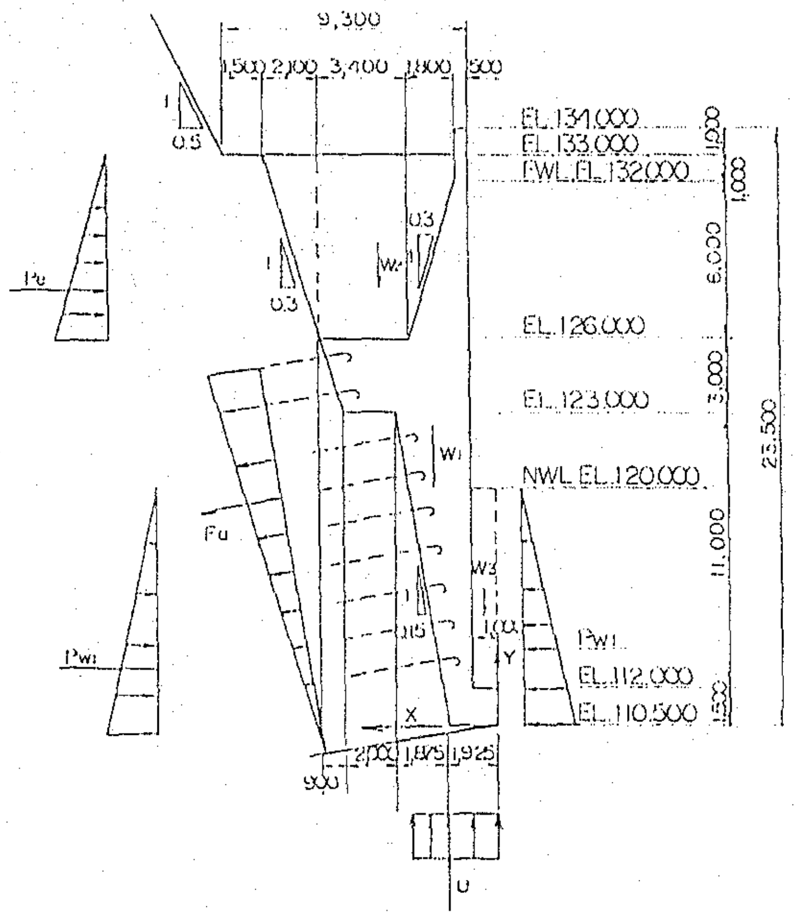
Safety factor for sliding: $$Fs = \frac{f \cdot \Sigma V + \tau \cdot A + Ha}{\Sigma H} = \frac{0.55 \times 47.135 + 20 \times 2.0 + 35.16}{2.357} = 42.9 > 1.2$$

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

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

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

Table 4.1.7: STABILITY ANALYSIS OF SPILLWAY STRUCTURE
(Stillway 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: 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)

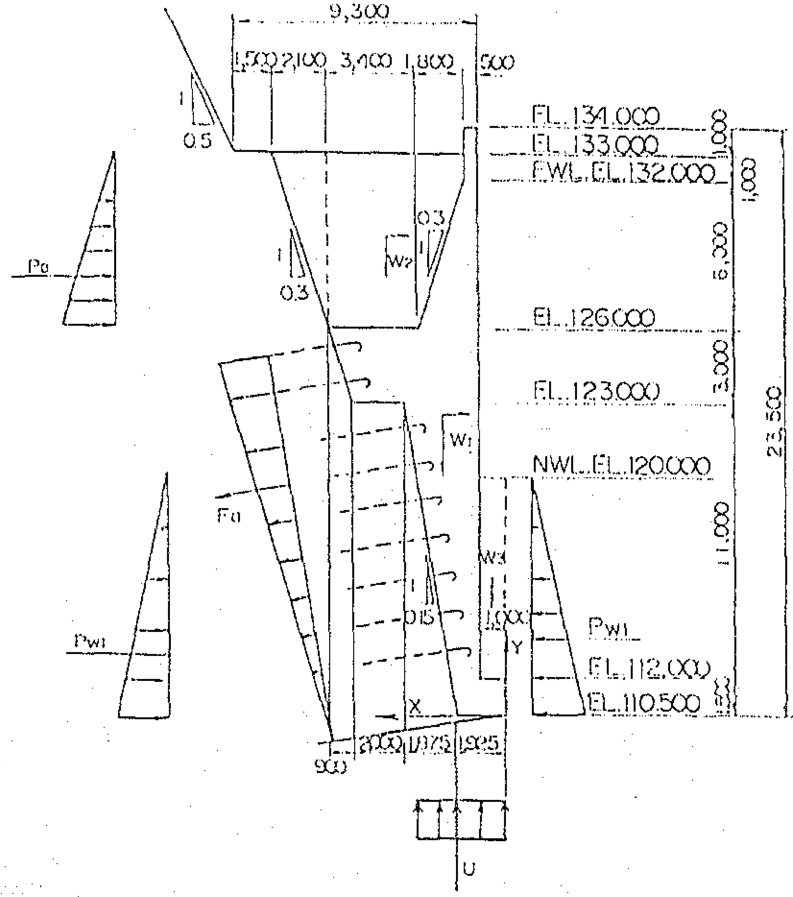
Safety factor for sliding: $Fs = \frac{f \cdot \Sigma V + \tau \cdot A + Ha}{\Sigma H} = \frac{0.55 \times 140.840 + 20 \times 3.925 + 67.802}{11.407} = 19.6 > 1.5$

Safety factor for overturning: $Fs = \frac{\Sigma Mr}{\Sigma Mt} = \frac{468.240 + 773.896}{203.431} = 6.1 > 1.5$

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 4.1.8 : STABILITY ANALYSIS OF SPILLWAY STRUCTURE
(Stillway 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)

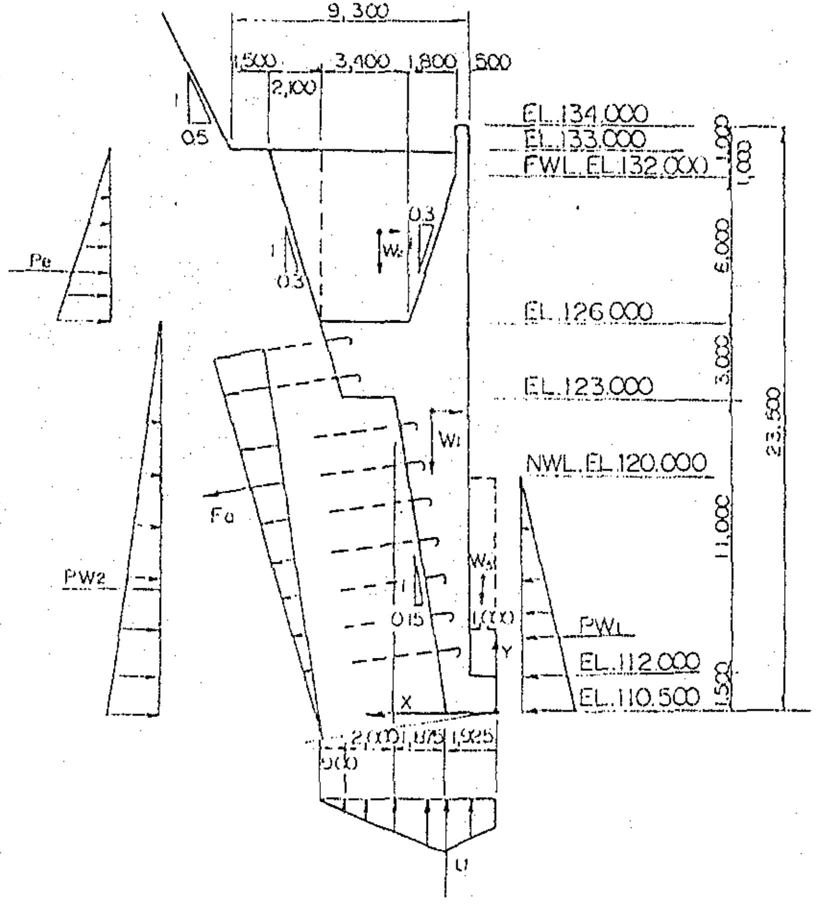
Safety factor for sliding: $Fs = \frac{f \cdot \Sigma V + \tau \cdot A + Ha}{\Sigma H} = \frac{0.55 \times 140.840 + 20 \times 3.925 + 67.802}{20.702} = 10.81 > 1.2$

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

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 4.1.9: STABILITY ANALYSIS OF SPILLWAY STRUCTURE
(Stillway 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: $Fs = \frac{f \cdot \Sigma V + \tau \cdot A + Ha}{\Sigma H} = \frac{0.55 \times 111.596 + 20 \times 3.925 + 67.802}{86.407} = 2.4 > 1.2$

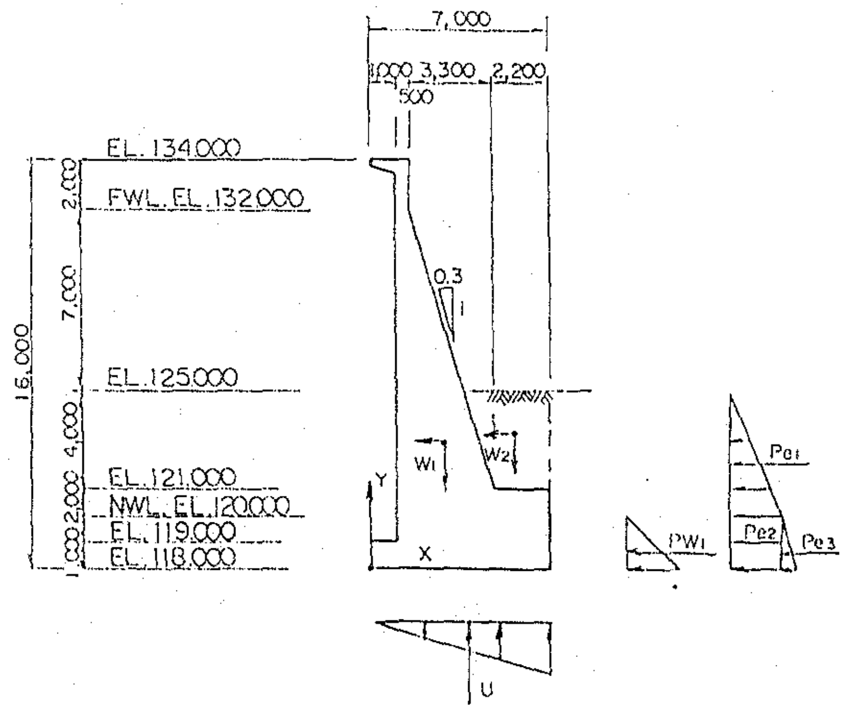
Safety factor for overturning: $Fs = \frac{\Sigma Mr}{\Sigma Mt} = \frac{366.395 + 773.896}{921.431} = 1.24 > 1.2$

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

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



SECTION AND LOADING CONDITION

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

Max. Resisting Moment of Anchor Bar: $Ma = 0$

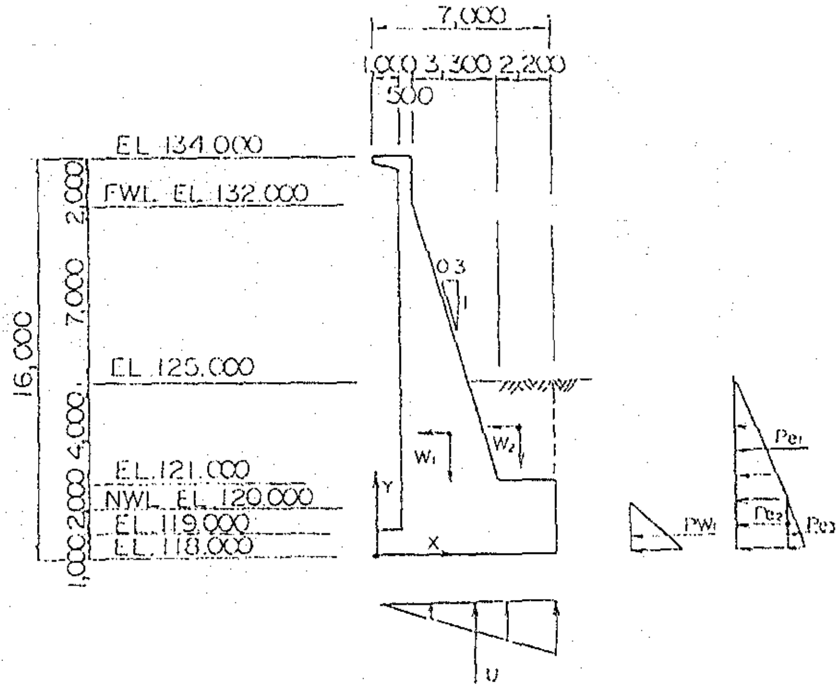
Safety factor for sliding: $Fs = \frac{f \cdot \Sigma V + \tau \cdot A + Ha}{\Sigma H} = \frac{0.55 \times 120.722 + 20 \times 7.0}{9.897} = 20.9 > 1.5$

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 4.1.11: 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

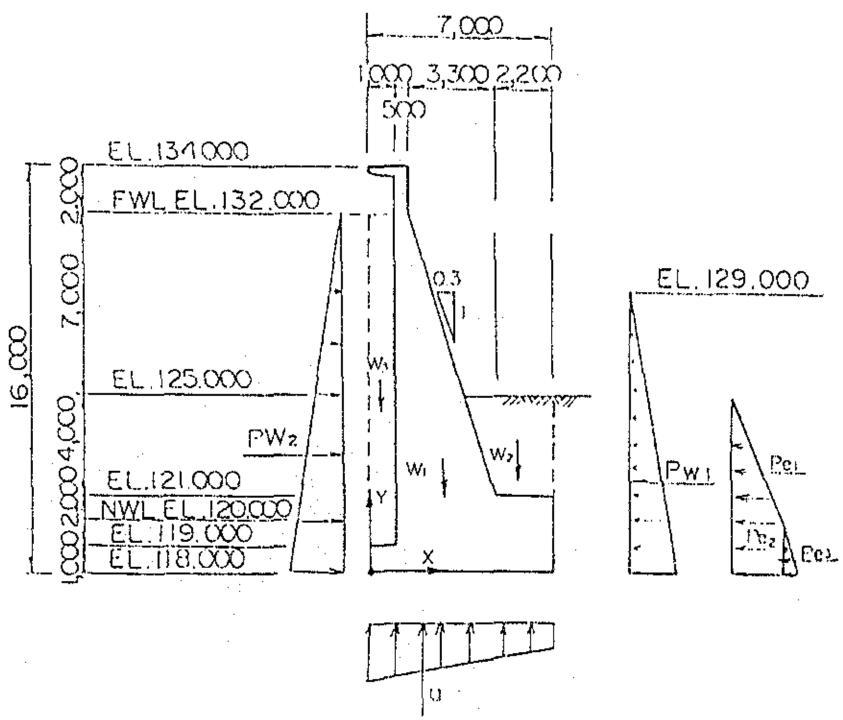
Safety factor for sliding: $Fs = \frac{f \cdot \Sigma V + \tau \cdot A + Ha}{\Sigma H} = \frac{0.55 \times 120.722 + 20 \times 7.0}{17.018} = 12.1 > 1.2$

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 4.1.12: STABILITY ANALYSIS OF SPILLWAY STRUCTURE
(Stillway 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

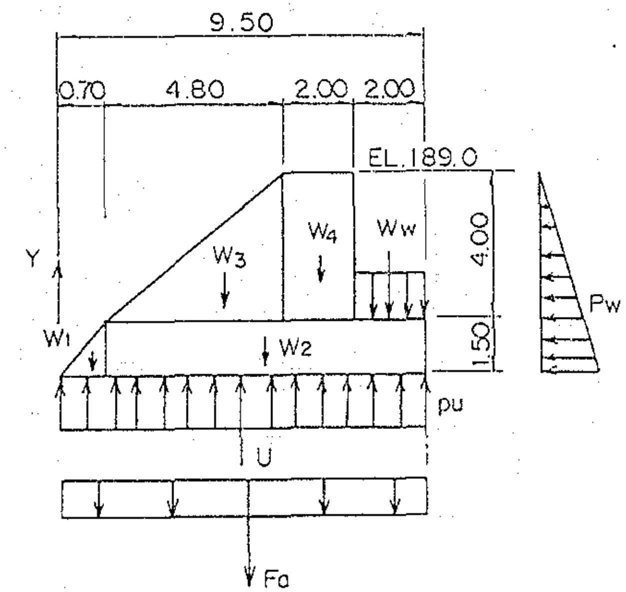
Safety factor for sliding: $Fs = \frac{f \cdot \Sigma V + \tau \cdot A + Ha}{\Sigma H} = \frac{0.55 \times 53.222 + 20 \times 7.0}{78.575} = 4.0 > 1.2$

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 4.1.13 : 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)

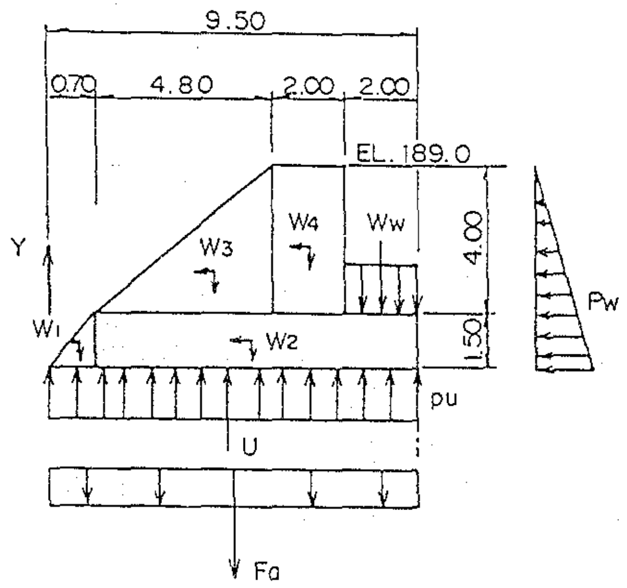
Safety factor for sliding: $Fs = \frac{f \cdot \Sigma V + \tau \cdot A + Ha}{\Sigma H} = \frac{0.55 \times 27.8 + 20 \times 9.5}{15.13} = 13.6 > 1.5$

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 4.1.14: 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)

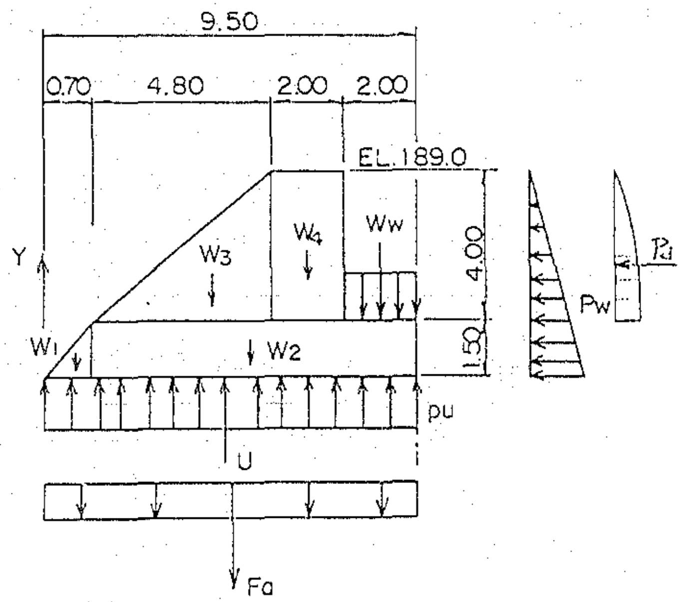
Safety factor for sliding: $Fs = \frac{f \cdot \Sigma V + \tau \cdot A + Ha}{\Sigma H} = \frac{0.55 \times 27.8 + 20 \times 9.5}{19.20} = 10.7 > 1.5$

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 4.1.15 : 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: $Fs = \frac{\tau \cdot A}{\Sigma H} = \frac{20 \times 9.5}{15.13} = 12.6 > 1.2$

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

Table 4.1.16 : **STRESS ANALYSIS OF REINFORCED CONCRETE**
(Spillway Side 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 = 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			0.010	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 4.1.17 : **STRESS ANALYSIS OF REINFORCED CONCRETE**
(Spillway Side 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 4.1.18 : **STRESS ANALYSIS OF REINFORCED CONCRETE**
(Spillway 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 4.1.19 : **STRESS ANALYSIS OF REINFORCED CONCRETE**
(Spillway Side Wall, Section I - I (1))

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.	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 = 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			0.006	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 4.1.20 : **STRESS ANALYSIS OF REINFORCED CONCRETE**

(Spillway Side Wall, Section I - I (2))

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.	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 4.1.21 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 4.1.22 SUMMARY OF SHARE 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 : Share due to dead load
L.L : Share due to live load

Table 4.1.23 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 4.1.24 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 4.1.25 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 4.1.26 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 4.1.27 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 4.1.28 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 (cm^2)
• 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 (cm^2)	Moment of Inertia of Area (cm^4)
• 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 cm^3
- Distance and section modulus (See Fig. 4.4.7):

Distance (cm)		Section Modulus (cm^3)	
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/cm^2):

	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 4.1.29 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 (cm²)
• 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 (cm²)	Moment of Inertia of Area (cm⁴)
• 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 cm³
- Distance and section modulus (See Fig. 4.4.7):

Distance (cm)		Section Modulus (cm³)	
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/cm²):

	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 4.1.30 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 4.1.31 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 4.1.32 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

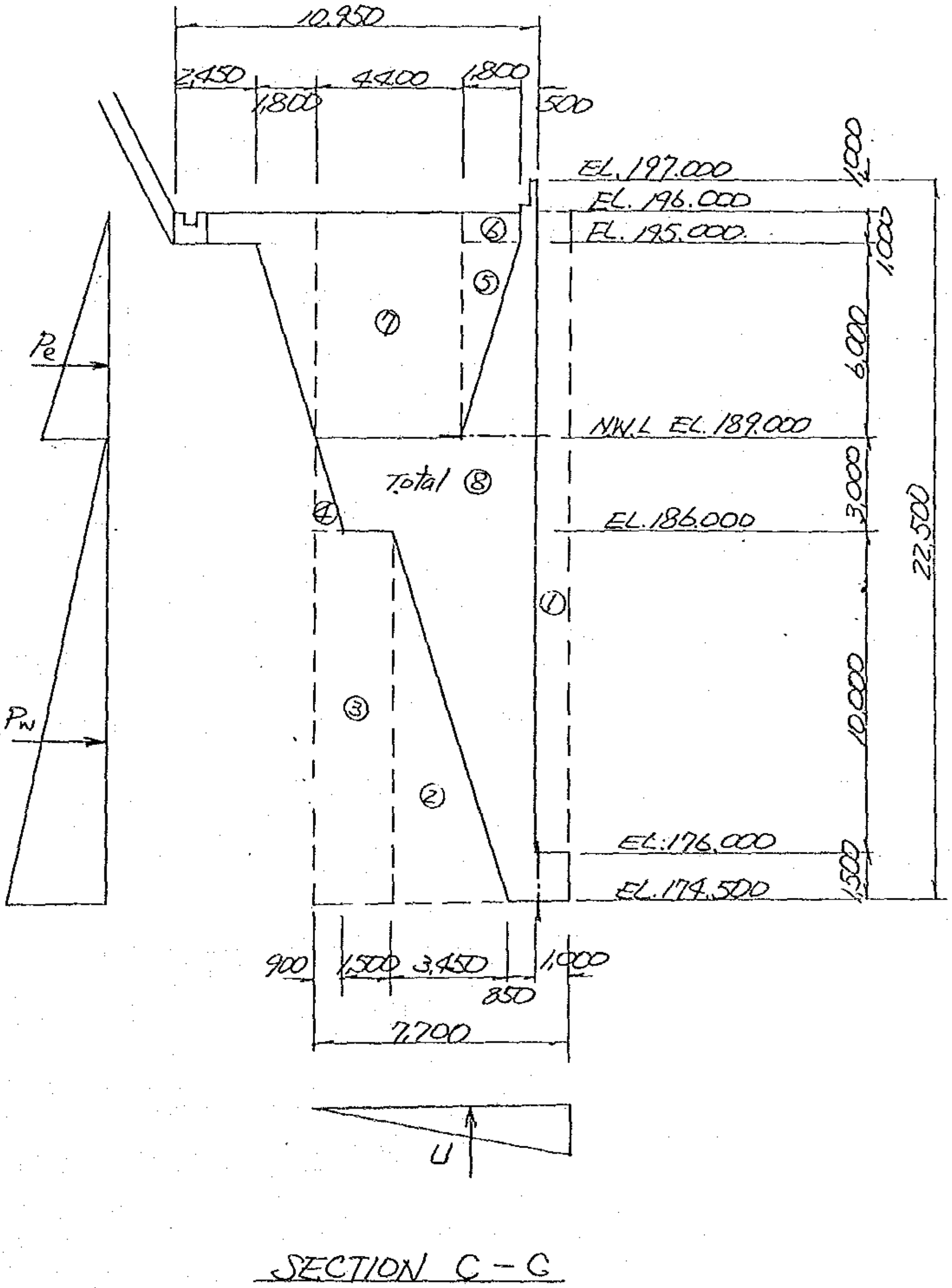


Fig.4.1.1 Side channel wall, Section C-C

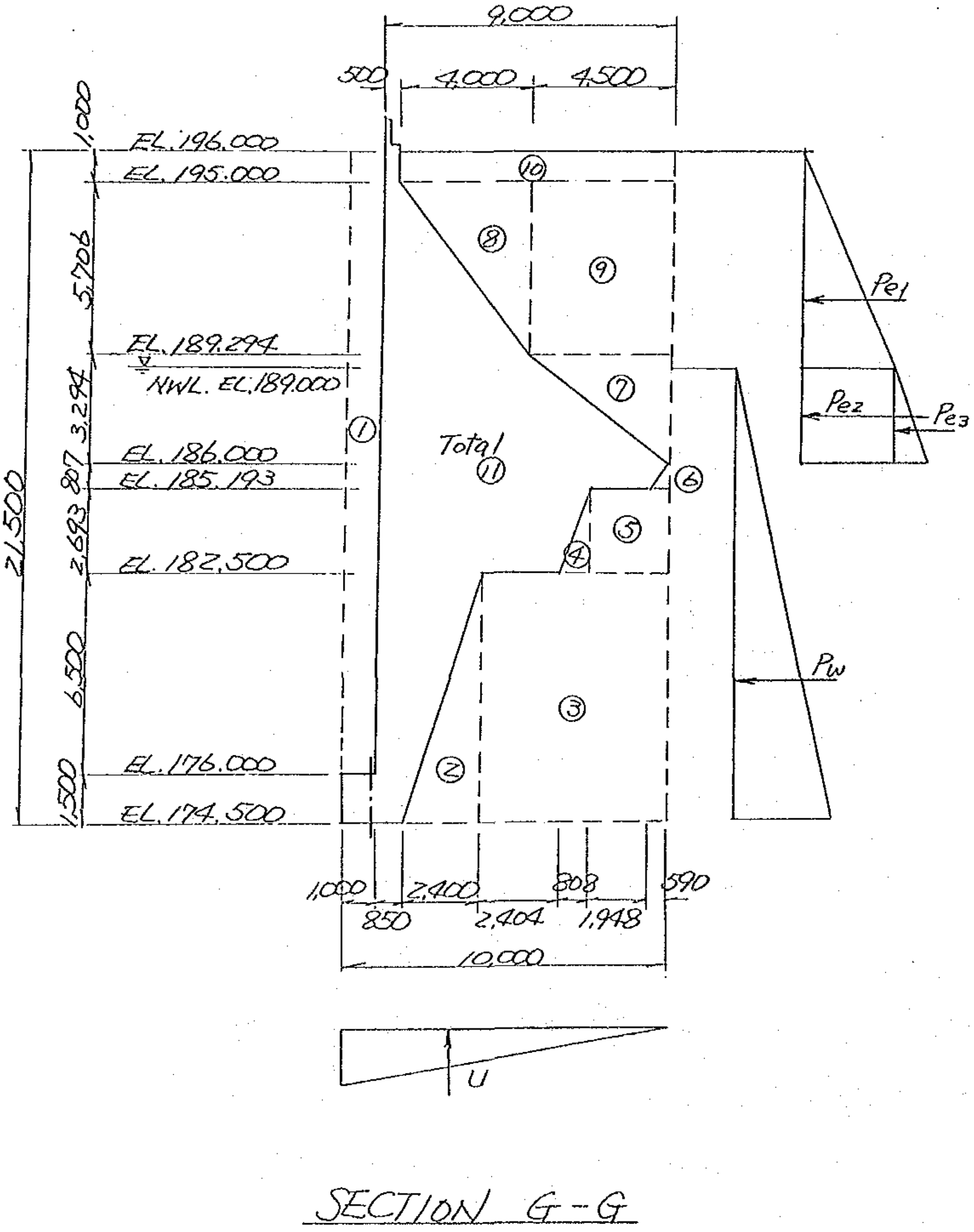


Fig.4.1.2 Transition wall, Section G-G

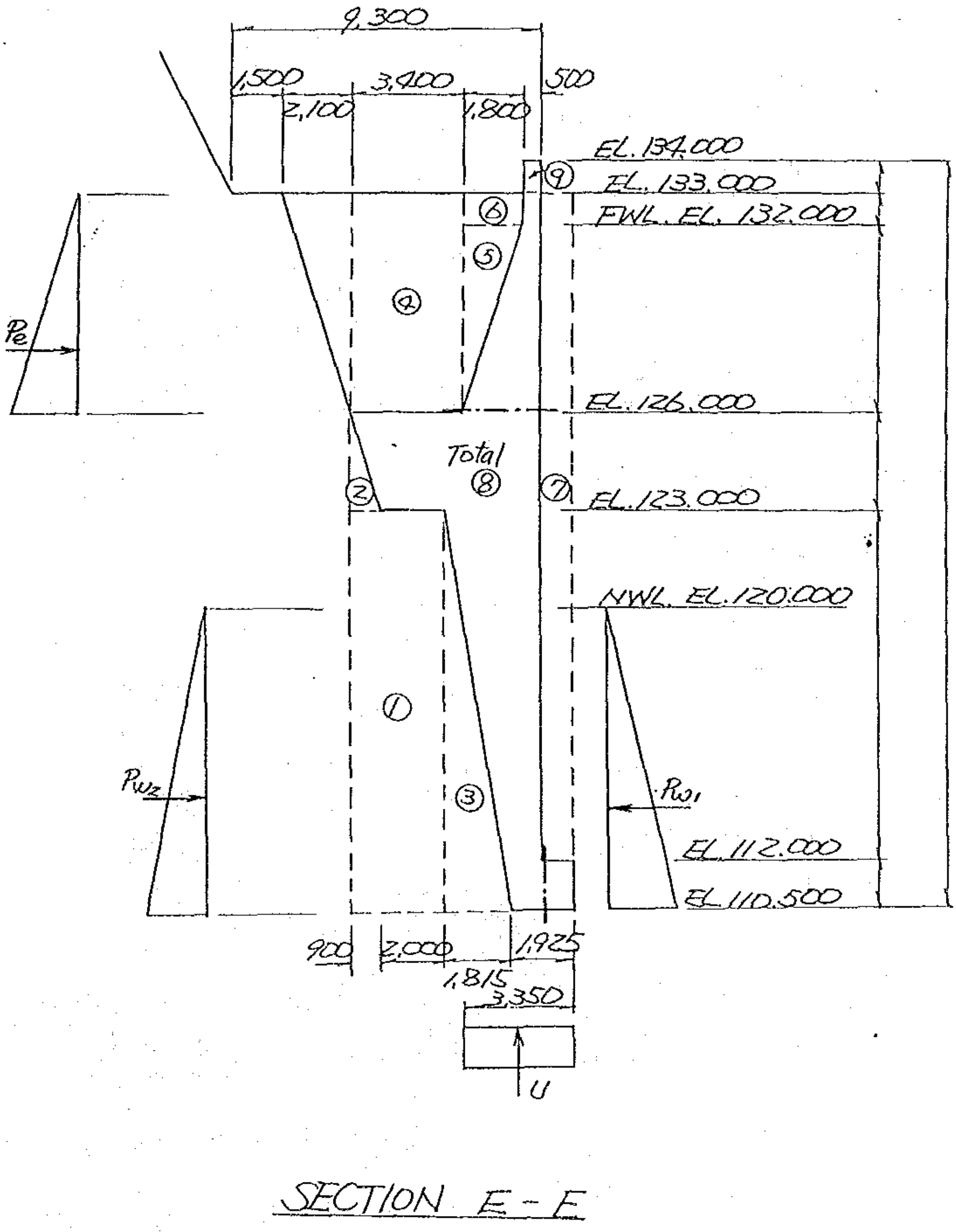


Fig.4.1.3 Stilling basin side wall, Section E-E

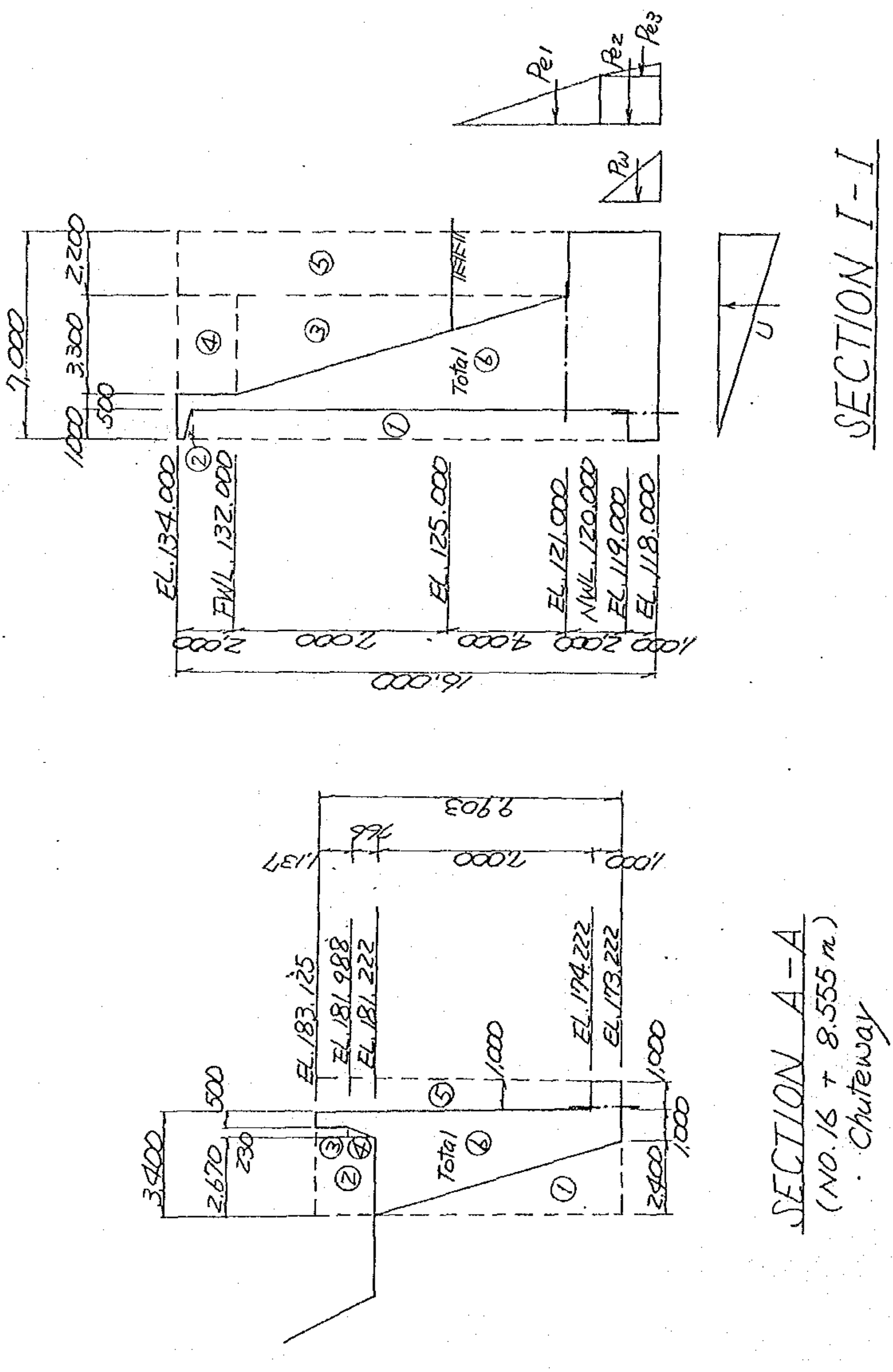
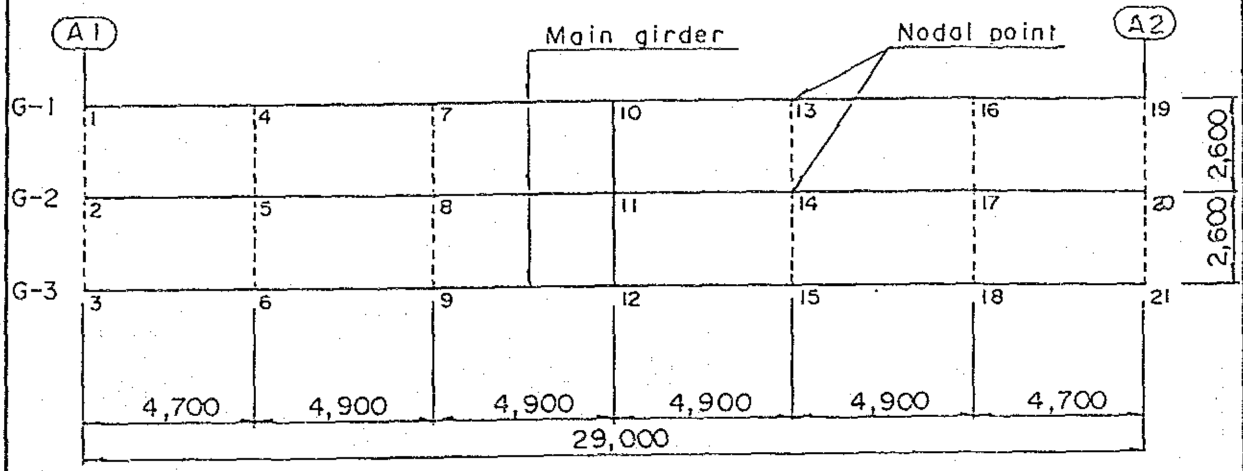
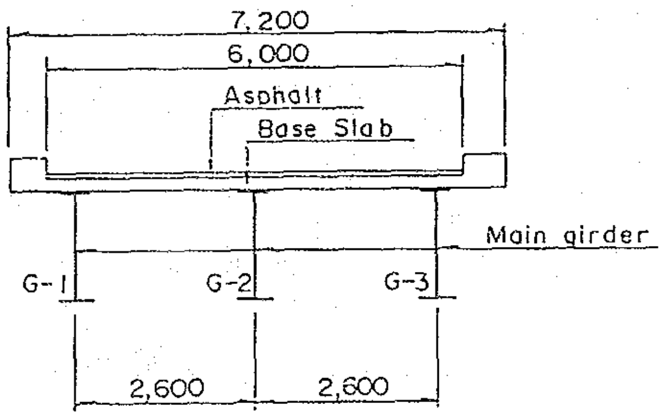


Fig.4.1.4 Stilling basin side wall, Section I-I
and chuteway side wall, Section A-A

Fig. 4.1.5



FRAME PLAN OF COMPOSITE GIRDER

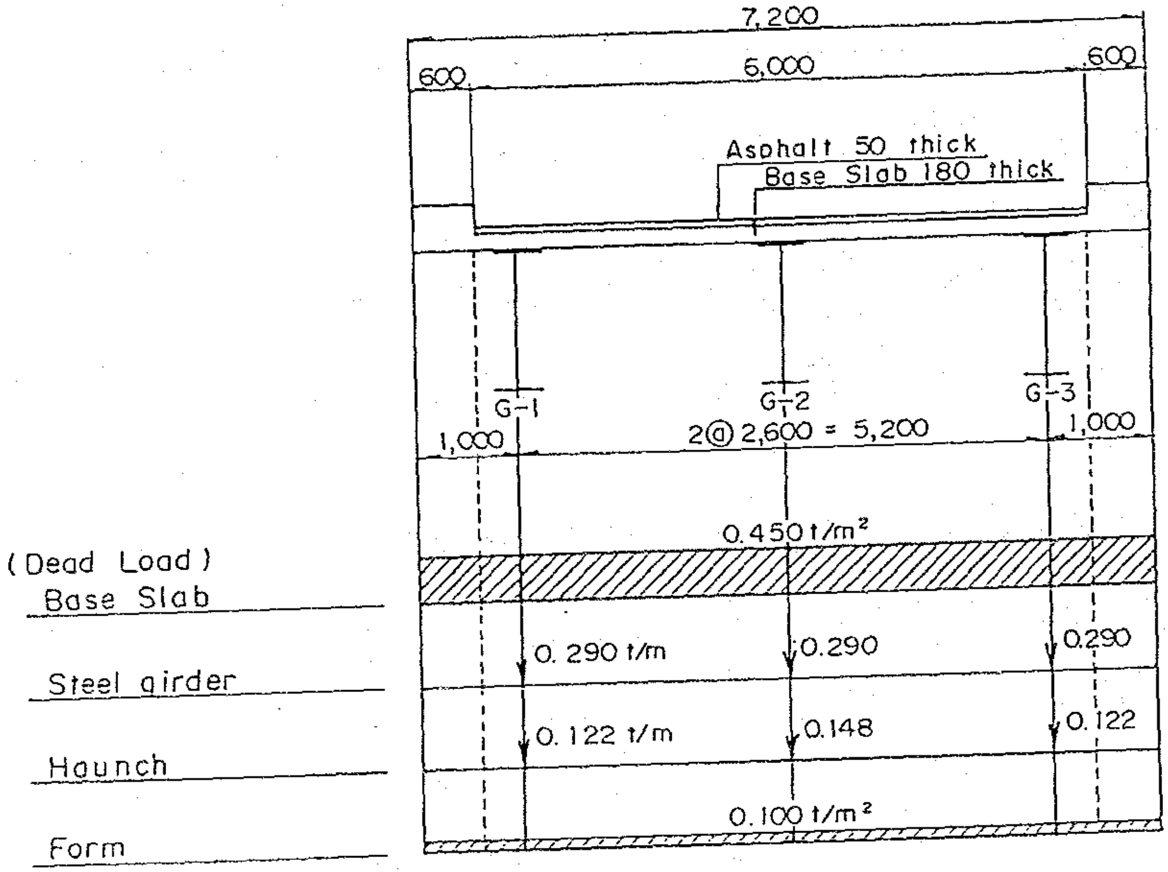


SECTION OF COMPOSITE GIRDER

FRAME PLAN AND SECTION OF COMPOSITE GIRDER

GOVERNMENT OF MAURITIUS
PORT LOUIS WATER SUPPLY PROJECT

JAPAN INTERNATIONAL COOPERATION AGENCY



LOADING ON COMPOSITE GIRDER
(BEFORE COMPOUNDING)

GOVERNMENT OF MAURITIUS
PORT LOUIS WATER SUPPLY PROJECT
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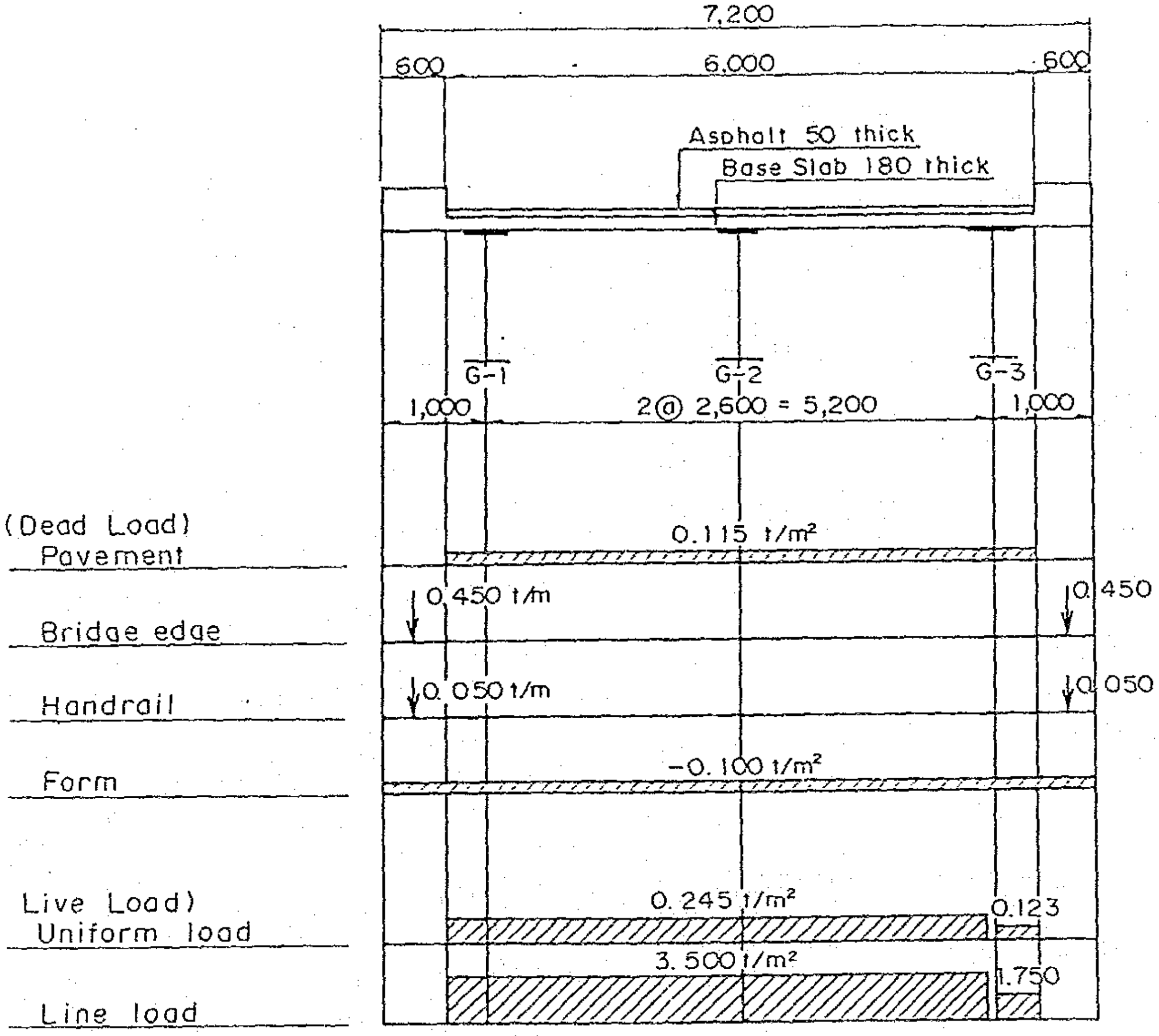
Fig. 4.1.7

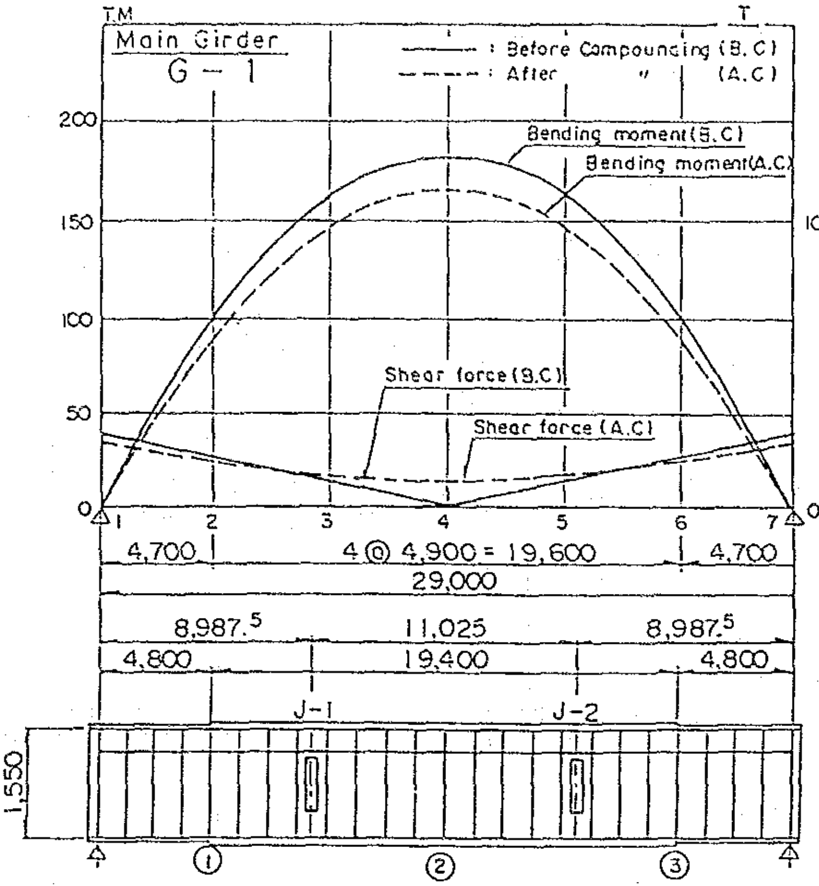


LOADING ON COMPOSITE GIRDER
(AFTER COMPOUNDING)

GOVERNMENT OF MAURITIUS
PORT LOUIS WATER SUPPLY PROJECT

JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. 4.1.8



		Section Nos.		
		①	②	③
UFLG	B	230	280	230
	T	11 (3)	14 (3)	11 (3)
WEB	H	1550	1550	1550
	T	9 (3)	9 (3)	9 (3)
LFLG	B	280	440	280
	T	11 (3)	19 (3)	11 (3)
SU		-1327	-1702	-1328
SUA		-1412	-1765	-1412
SUA-SU		85	63	84
SL		2068	2055	2068
SLA		2100	2100	2100
SLA-SL		32	45	32
TU		239	63	239
TUA		1200	1200	1200

UFLG : Upper flange
WEB : Web
LFLG : Lower flange
B : Width of flange (mm)
H : Height of web (mm)
T : Thickness (mm)
( ) : Steel material (1): SS41, (2): SM50, (3): SM50Y, (4): SM58

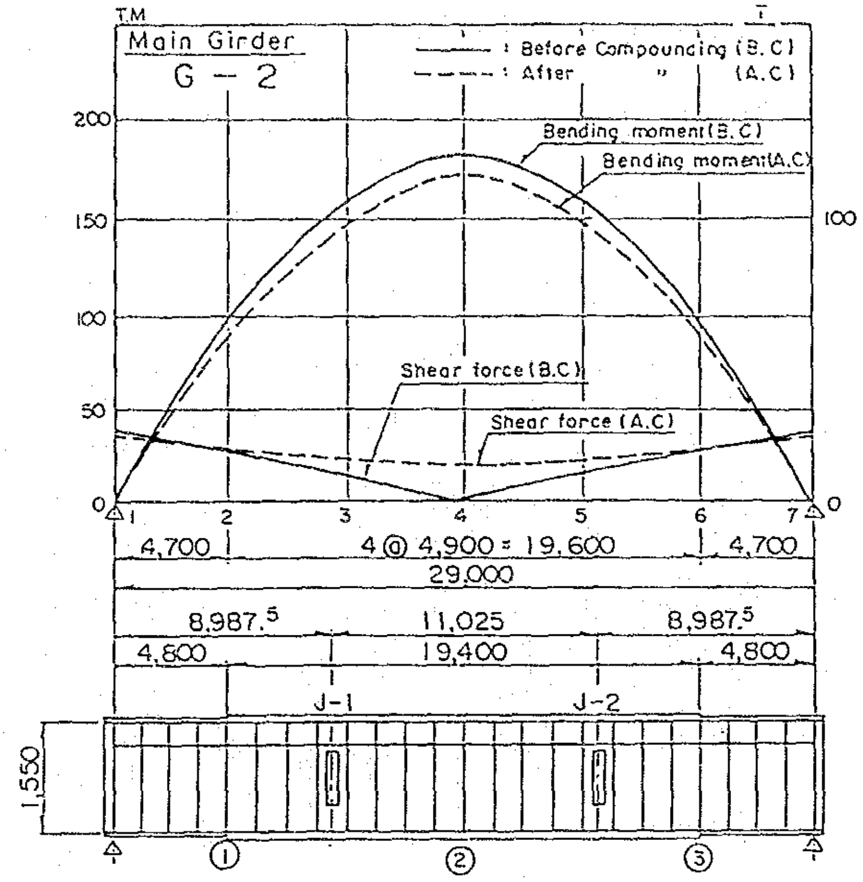
SU : Stress in upper flange (kg/cm^2)
SUA : Allowable stress for upper flange (kg/cm^2)
SL : Stress in lower flange (kg/cm^2)
SLA : Allowable stress for lower flange (kg/cm^2)
TU Shearing stress (kg/cm^2)
TUA Allowable shearing stress (kg/cm^2)

RESULT OF ANALYSIS ON COMPOSITE GIRDER
(No. 1 Main Girder (G-1))

GOVERNMENT OF MAURITIUS
PORT LOUIS WATER SUPPLY PROJECT

JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. 4.1.9



		Section Nos.		
		①	②	③
UFLG	B	230	280	230
	T	10 (3)	14 (3)	10 (3)
WEB	H	1550	1550	1550
	T	9 (3)	9 (3)	9 (3)
LFLG	B	280	440	280
	T	11 (3)	19 (3)	11 (3)
SU		-1346	-1720	-1346
SUA		-1379	-1765	-1379
SUA-SU		33	45	33
SL		2033	2066	2033
SLA		2100	2100	2100
SLA-SL		67	34	67
TU		252	92	252
TUA		1200	1200	1200

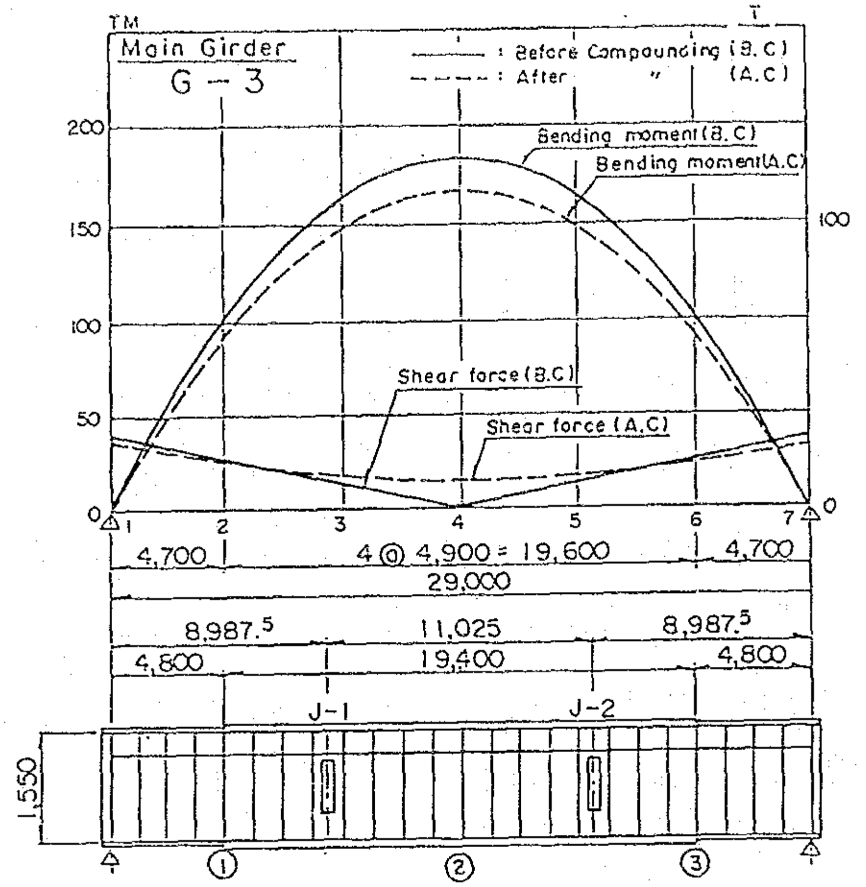
UFLG : Upper flange
WEB : Web
LFLG : Lower flange
B : Width of flange (mm)
H : Height of web (mm)
T : Thickness (mm)
( ) : Steel material
(1): SS41, (2): SM50,
(3): SM50Y, (4): SM58

SU : Stress in upper flange (kg/cm^2)
SUA : Allowable stress for upper flange (kg/cm^2)
SL : Stress in lower flange (kg/cm^2)
SLA : Allowable stress for lower flange (kg/cm^2)
TU Shearing stress (kg/cm^2)
TUA Allowable shearing stress (kg/cm^2)

RESULT OF ANALYSIS ON COMPOSITE GIRDER
(No. 2 Main Girder (G-2))

GOVERNMENT OF MAURITIUS
PORT LOUIS WATER SUPPLY PROJECT

JAPAN INTERNATIONAL COOPERATION AGENCY



		Section Nos.		
		①	②	③
UFLG	B	230	280	230
	T	11 (3)	14 (3)	11 (3)
WEB	H	1550	1550	1550
	T	9 (3)	9 (3)	9 (3)
LFLG	B	280	440	280
	T	11 (3)	19 (3)	11 (3)
SU		-1327	-1702	-1328
SUA		-1412	-1765	-1412
SUA-SU		85	63	84
SL		2068	2055	2068
SLA		2100	2100	2100
SLA-SL		32	45	32
TU		239	66	239
TUA		1200	1200	1200

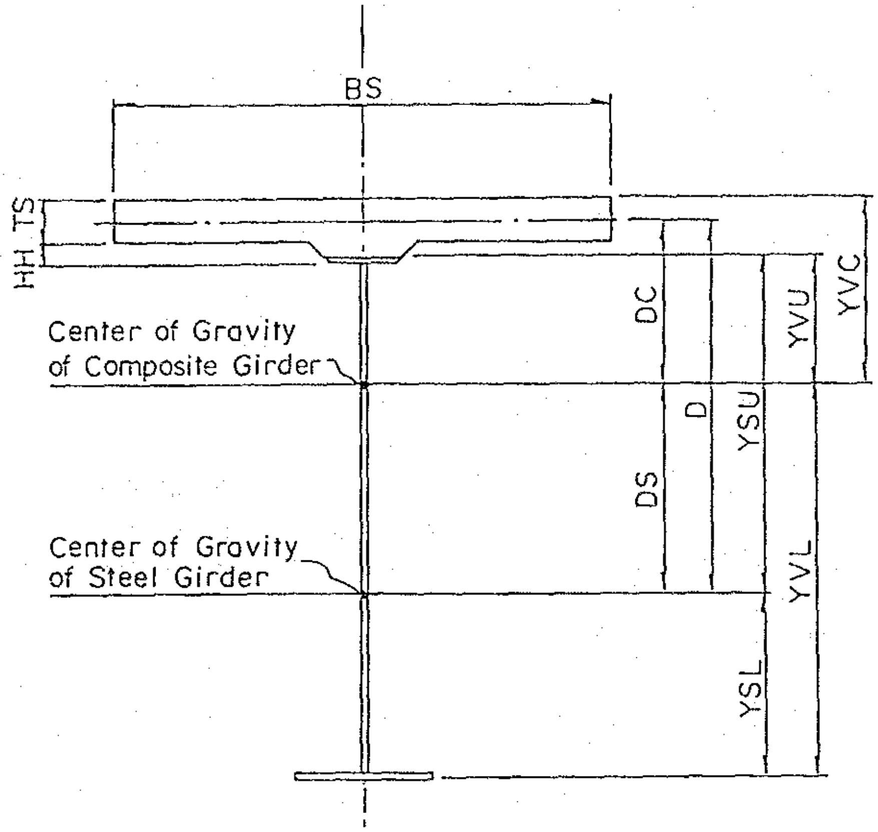
UFLG : Upper flange
WEB : Web
LFLG : Lower flange
B : Width of flange (mm)
H : Height of web (mm)
T : Thickness (mm)
( ) : Steel material
(1): SS41, (2): SM50,
(3): SM50Y, (4): SM58

SU : Stress in upper flange (kg/cm^2)
SUA : Allowable stress for upper flange : (kg/cm^2)
SL : Stress in lower flange (kg/cm^2)
SLA : Allowable stress for lower flange : (kg/cm^2)
TU Shearing stress (kg/cm^2)
TUA Allowable shearing stress (kg/cm^2)

RESULT OF ANALYSIS ON COMPOSITE GIRDER
(No. 3 Main Girder (G-3))

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

Fig. 4.1.11



TS : Base slab thickness

BS : Effective base slab width

HH : Haunch height

WSU : Section modulus at upper flange edge before compounding

WSL : Section modulus at lower flange edge before compounding

WVU : Section modulus at upper flange edge after compounding

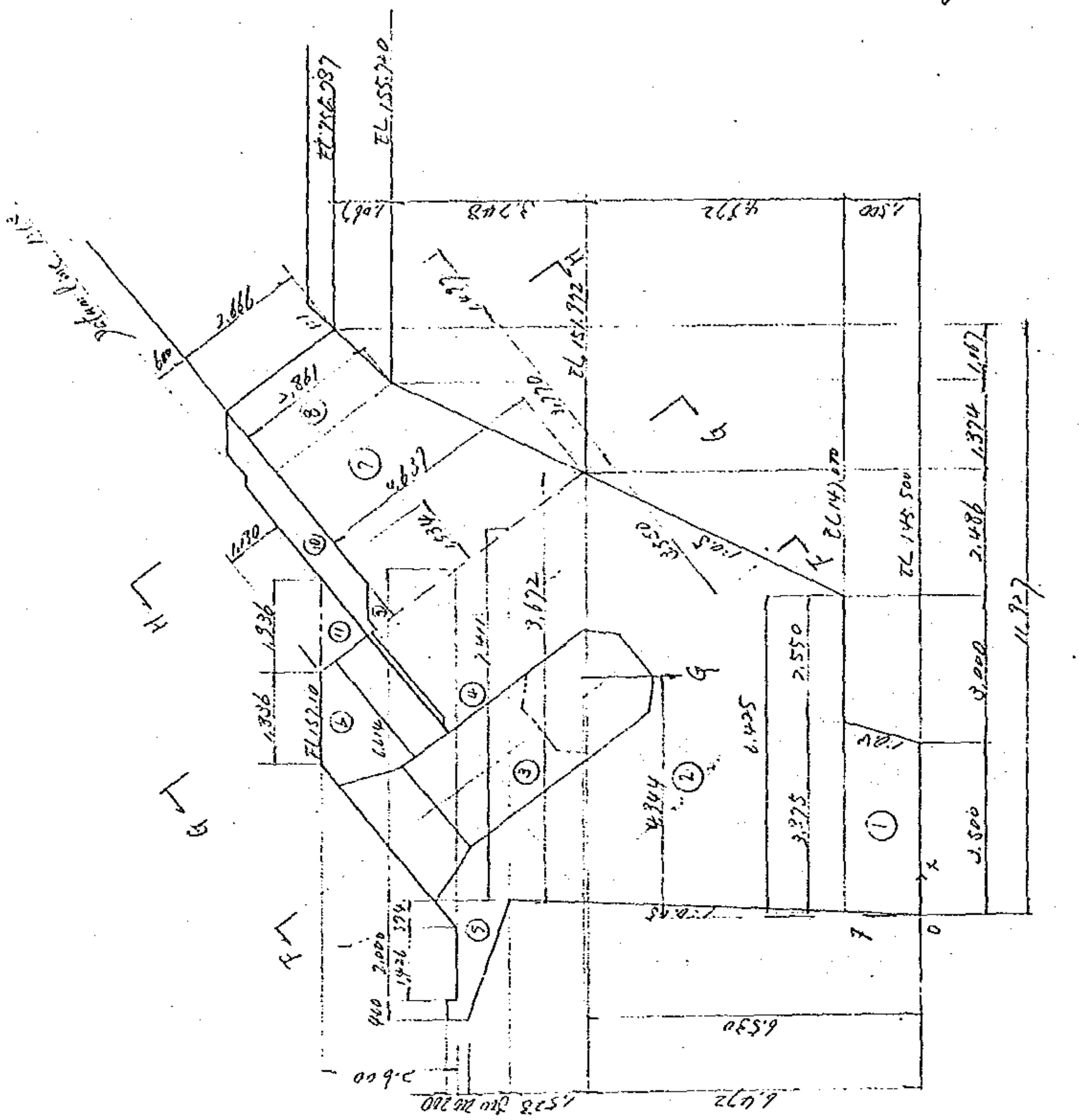
WVL : Section modulus at lower flange edge after compounding

SYMBOLS OF COMPOSITE GIRDER

GOVERNMENT OF MAURITIUS
PORT LOUIS WATER SUPPLY PROJECT

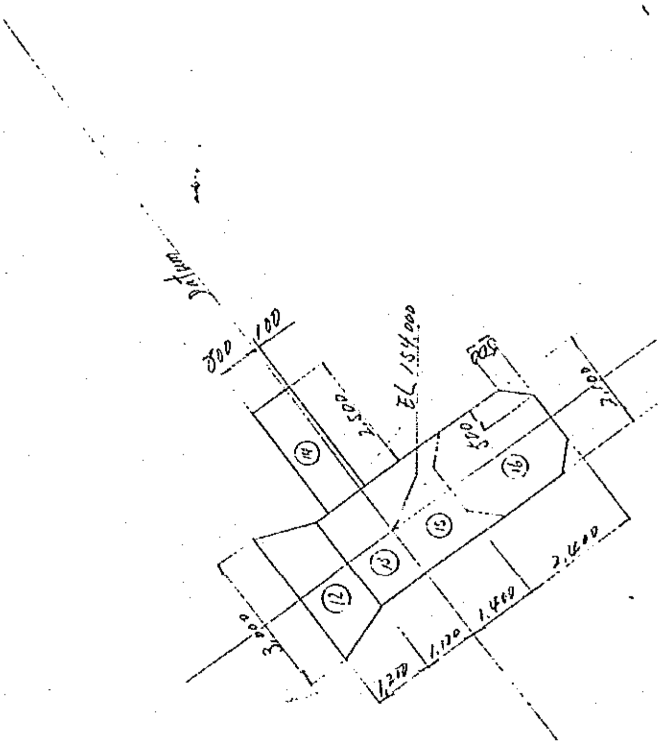
JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. 4.3.1

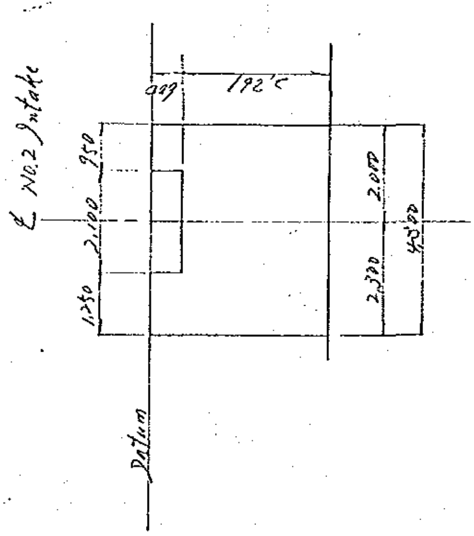


PROFILE OF NO.2 INTAKE

Fig. 4.3.2

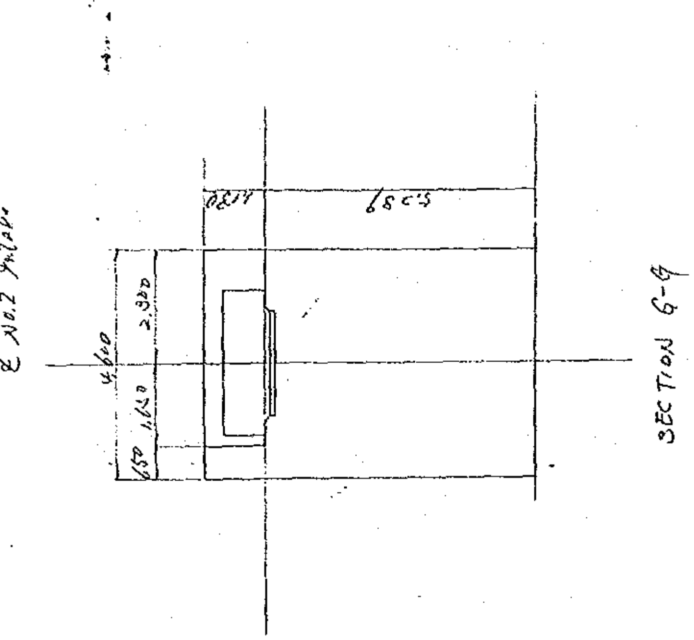


VOID PORTION

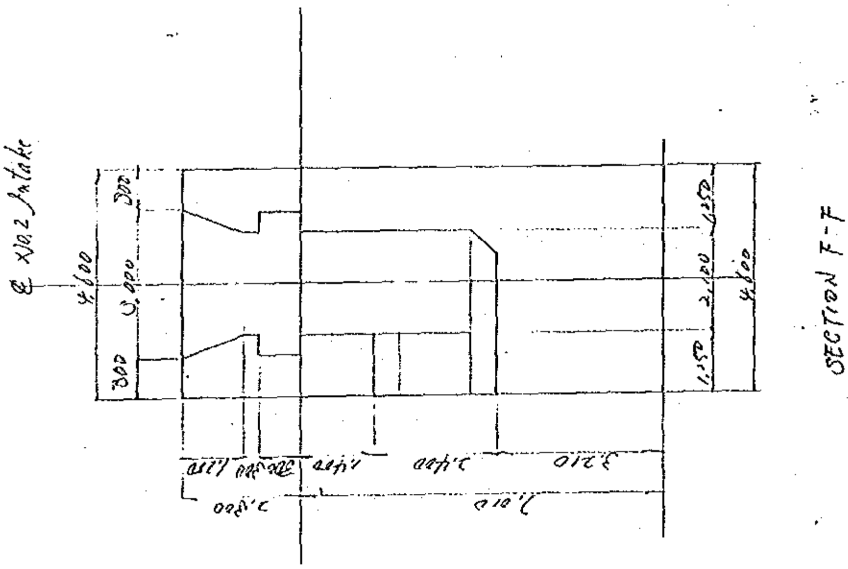


SECTION H-H

Fig. 4.3.3.

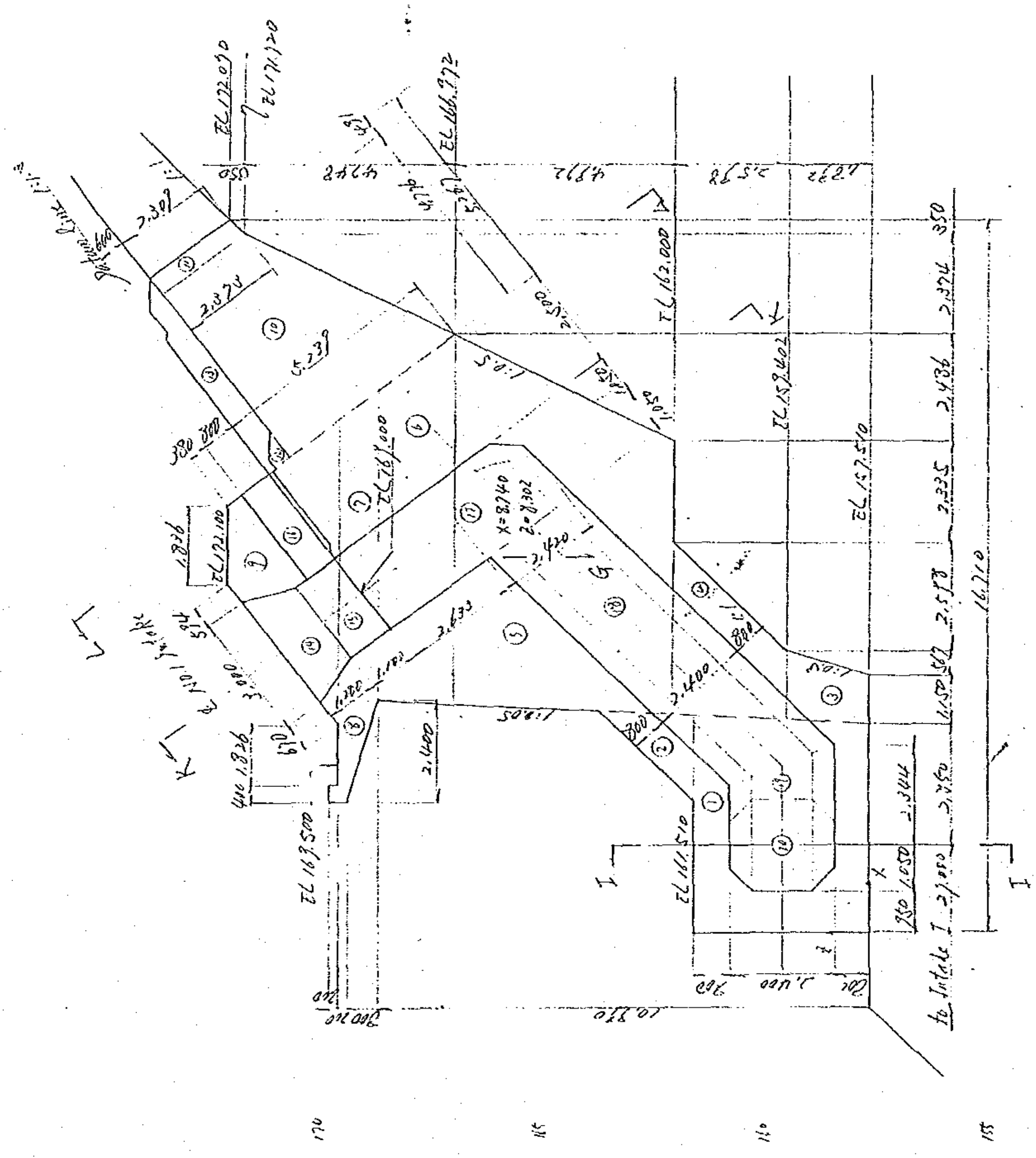


℄ No.2 Intake
4,600
2,300
1,150
SECTION G-G

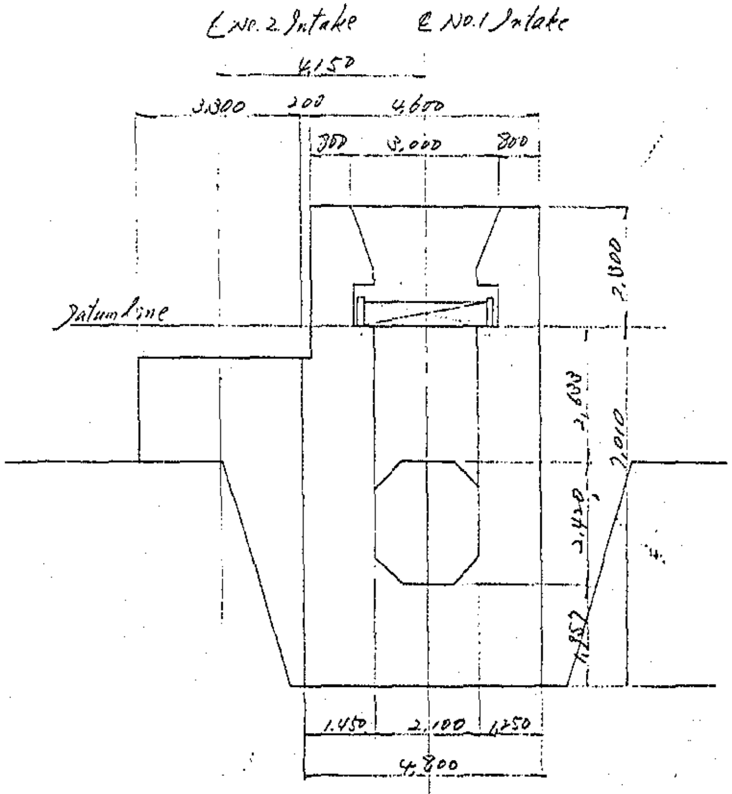


℄ No.2 Intake
4,600
3,000
300
2,100
1,250
SECTION F-F

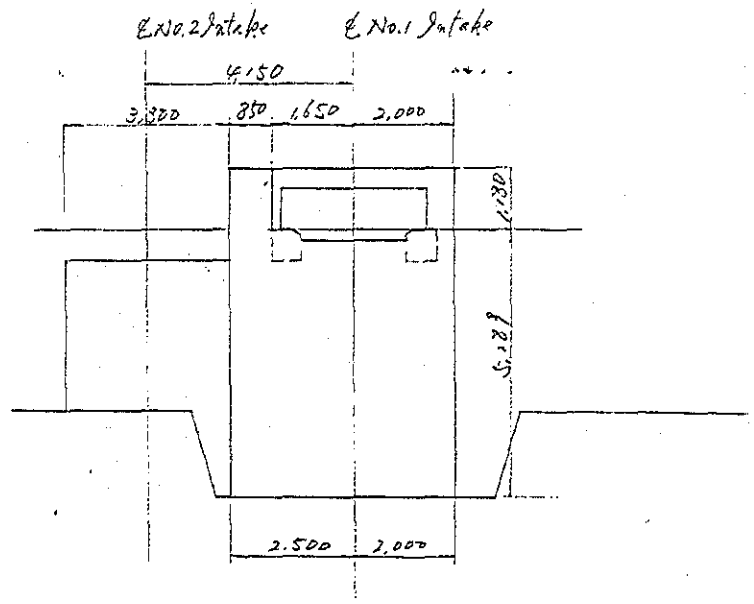
Fig. 4.3.4



PROFILE OF NO.1 INTAKE



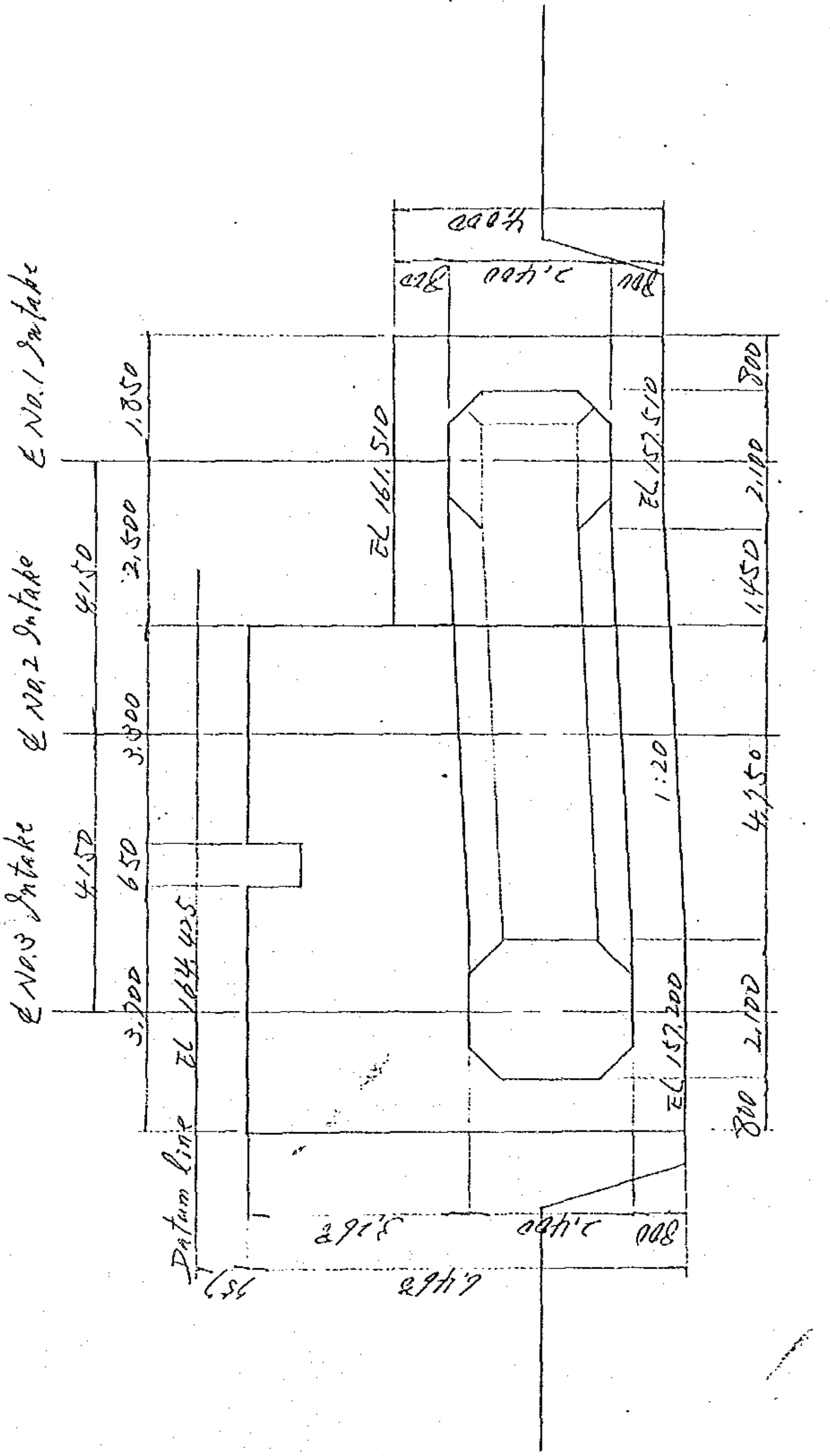
SECTION K-K



SECTION L-L

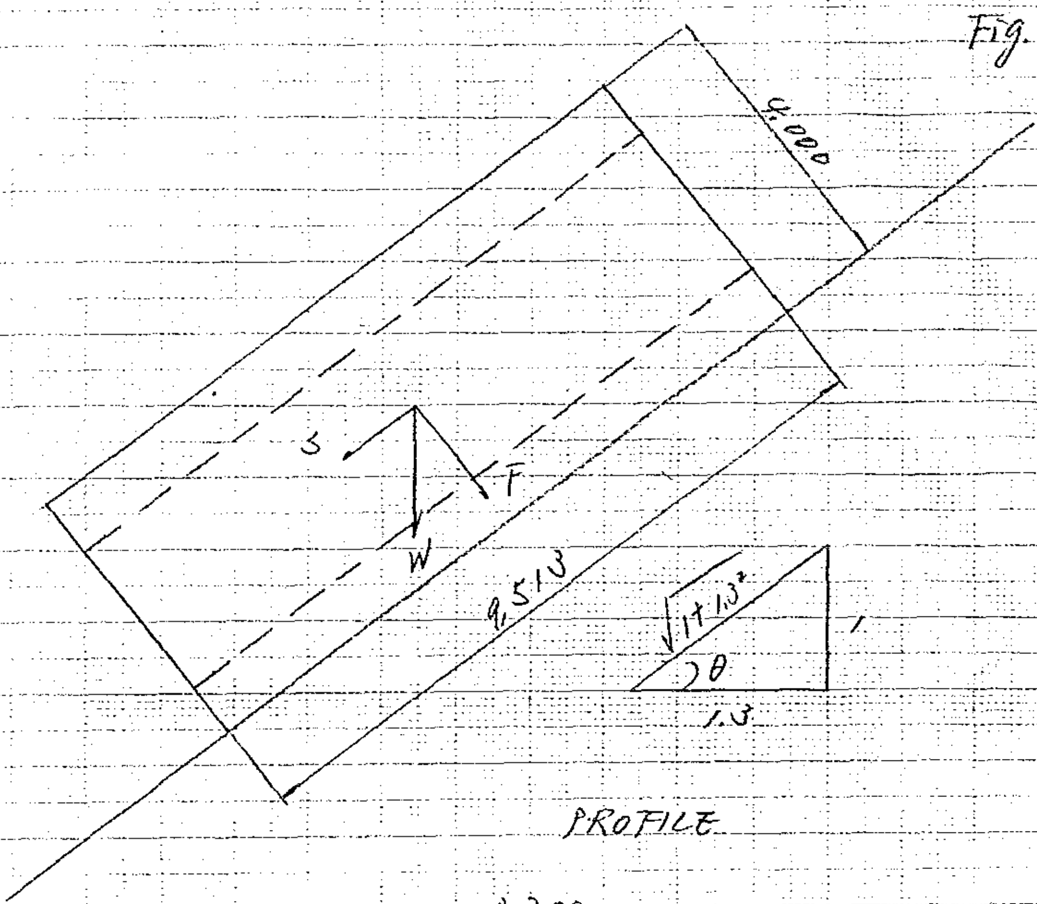
Fig.4.3.5

Fig. 4.3.6

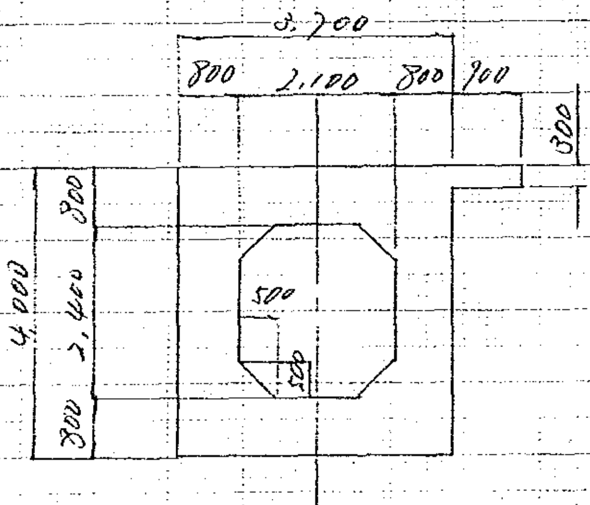


SECTION I-I (Vertical)

Fig. 4.3.7

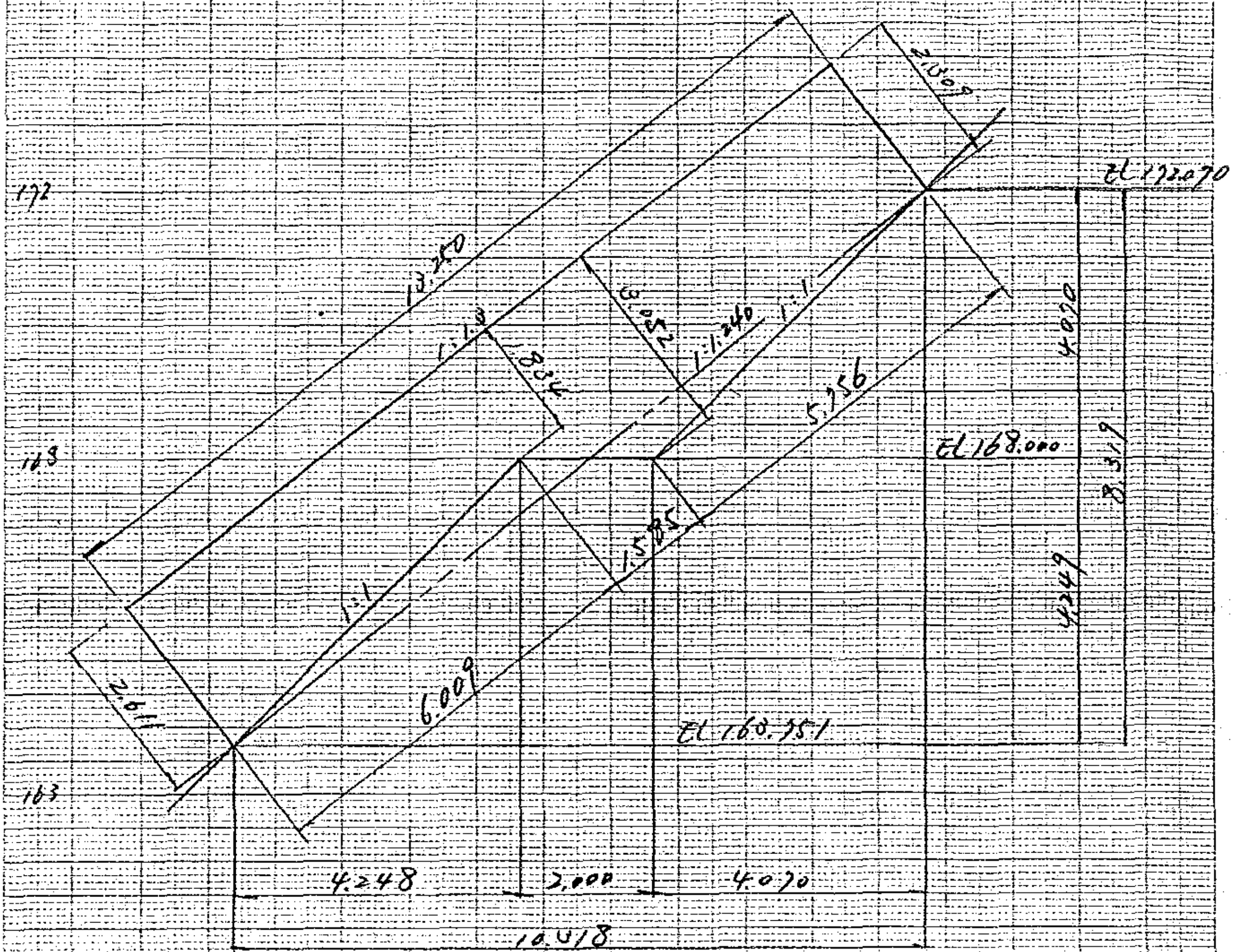


PROFILE

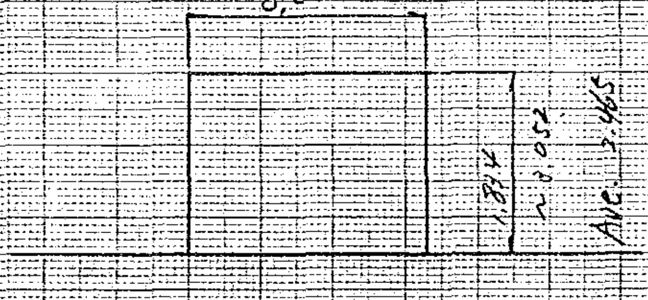


TYPICAL SECTION

Fig. 4.3.8



PROFILE



SECTION

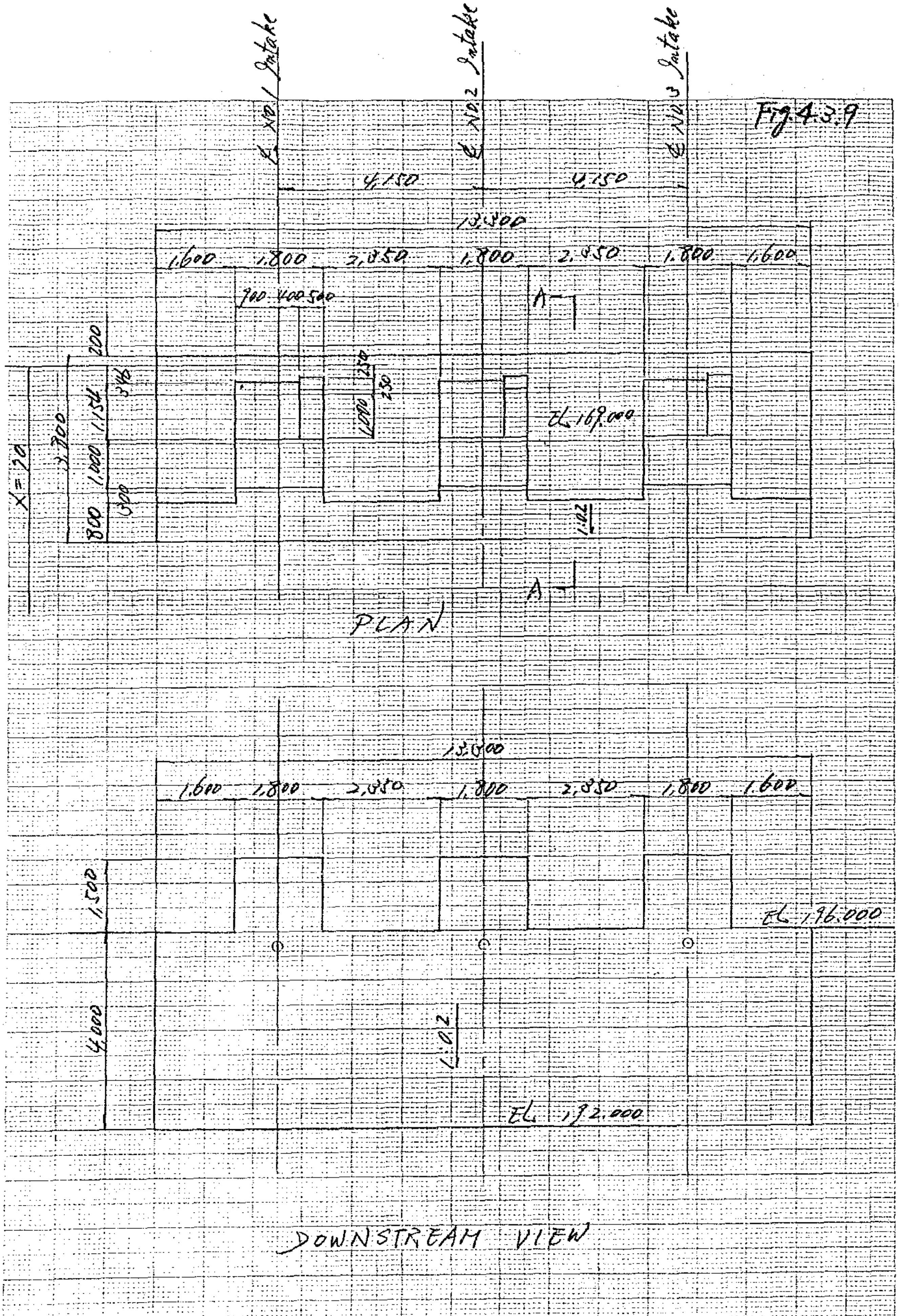
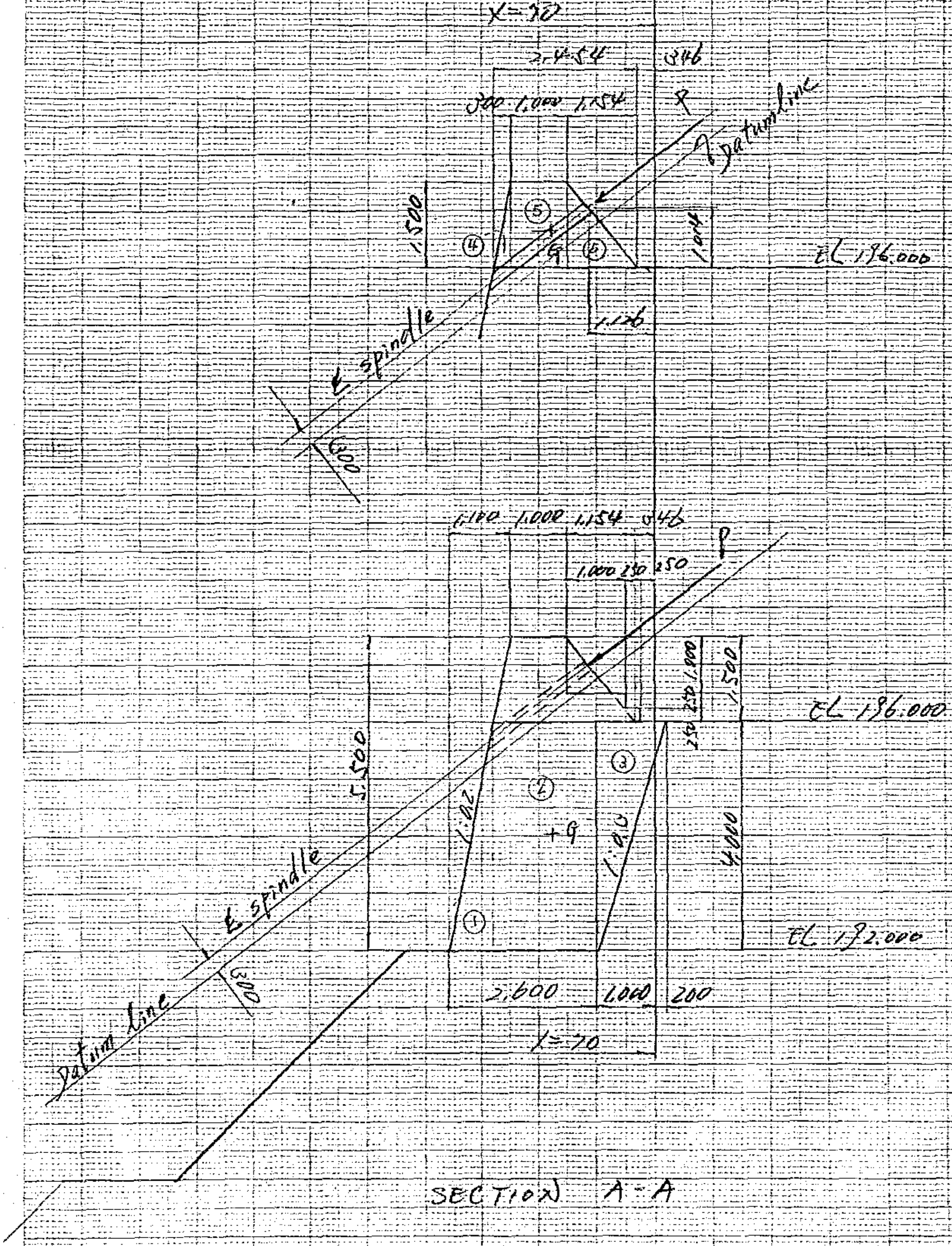


Fig.4.3.9
℄ No.1 Intake
℄ No.2 Intake
℄ No.3 Intake
4,150
4,150
13,800
1,600
1,800
2,850
1,800
2,850
1,800
1,600
700 400 500
A
EL 169.000
PLAN
13,800
1,600
1,800
2,850
1,800
2,850
1,800
1,600
1,500
4,000
EL 196.000
EL 192.000
DOWNSTREAM VIEW

Fig. 4.3.10



SECTION A-A

PART V STRESS CALCULATION

PART V. STRESS CALCULATION

5.1 Main Dam Gallery

5.1.1 Design Sections

Dimensions of the design section of the gallery are as shown in Fig. 5.1.1.

5.1.2 Design Values

(1) Unit weight

Reinforced concrete	$r_c = 2.40$ t/m^3
Water	$r_w = 1.0$ t/m^3
Core material (embankment)	$r_{wet} = 1.80$ t/m^3 (wet condition)
	$r_{sat} = 1.72$ t/m^3 (saturated con.)

(2) Elastic modulus and Poisson's ratio

	Elastic modulus (t/m^2)	Poisson's ratio
Concrete	2.55×10^6	0.2
Rock	5.5×10^5	0.3

(3) Allowable stress

	Compression	Tension	Shearing
Concrete	70 kg/cm^2	0 kg/cm^2	8 kg/cm^2
Steel bar	-	1,800 kg/cm^2	1,800 kg/cm^2

5.1.3 Loading Conditions

It is schemed that the height of the dam will be risen when water demand increase in future. Accordingly structure analyses are carried out in the condition that the dam is filled up to the final crest elevation of EL. 215.0 m.

Under the above fill, the following two loading cases are adopted as design load conditions. (See Fig. 5.1.2)

Load case 1: Reservoir water is at FWL (=212.5 m) without uplift

Load case 2: Reservoir water is at FWL (=212.5 m) with full uplift

The loads are calculated as follows:

Load case 1

$p1 = (212.5 - 112) \times 1.80\ t/m^3 = 180.9\ t/m^2$

$p2 = (215 - 212.5) \times 1.72\ t/m^3 = 4.3\ t/m^2$

Load case 2

$p1 = (212.5 - 112) \times 1.80\ t/m^3 = 180.9\ t/m^2$

$p2 = (215 - 212.5) \times 1.72\ t/m^3 = 4.3\ t/m^2$

$p3 = (212.5 - 112) \times 1.0\ t/m^3 = 100.5\ t/m^2$

$p4 = \{212.5 - (112 - 4.4)\} \times 1.0\ t/m^3 = 104.9\ t/m^2$

5.1.4 Stress Analysis

Stress analyses on the design section are carried out by a finite element method. A computer FEM programme C-143 registered in Nippon Koei is used.

A mesh model and boundary conditions of the model are as shown in Fig. 5.1.3.

5.1.5 Results of FEM Analysis

Compressive/tensile stress in horizontal and vertical directions, shear stress and principle stress in every element in the model are given in Table 5.1.1 and 5.1.2. The compressive/tensile stress and shear stress which are required for design of reinforcement bar arrangement are extracted from these tables and put down in the relevant elements in the design section, as shown in Fig. 5.1.4 and 5.1.5.

From these tables and figures, the followings are noticeable.

(1) A maximum compressive stress in the design section is low enough for the allowable compressive stress of concrete (=70 kg/cm^2). The maximum value in each loading conditions is as listed below.

(Unit: kg/cm^2)

	X-direction	Y-direction
Load case 1	23.0 Element 279	53.0 Element 164
Load case 2	38.8 Element 146	54.3 Element 164

(2) There are some elements where tensile stress occurs in both loading conditions. The tensile stress in load case 1 is larger than load case 2, in comparison with both conditions. Reinforcement bars against these tensile stress are required consequently.

(3) Shear stress in the members in the design section are checked at Line ④, ⑤, ⑥, ⑦, ⑧, ⑨ and ⑩ as shown in the figures. An average value at each line is given in Table 5.1.3

From this table, it is noticed that

- load case 2 is severer loading condition in shear stress, and

- the average shear stress at Line ⑤, ⑧, and ⑨ are larger than the allowable shear stress of concrete without stirrups (=8.5 kg/cm^2) but below the allowable shear stress with stirrups (=19 kg/cm^2) according to Standard Specification for Design and Construction of Concrete Structures 1986 Part 1, JSCE Clause 14.3.

Consequently, stirrups are required for these shear stresses.

5.1.6 Reinforcement Bars

(1) Re-bars for tensile stress

A required area at re-bars against tensile stress is calculated with the following formula.

$$As = \frac{\text{Total tensile strength in 1m depth (kg)}}{\text{Allowable tensile stress in re-bar (kg/cm}^2\text{)}}$$

i) At the center of the upper slab (Line ① - ① in Fig. 5.1.4)

$$As = \frac{\frac{1}{2} \times 6 \text{ kg/cm}^2 \times 35 \text{ cm} \times 100 \text{ cm}}{1{,}800 \text{ kg/cm}^2}$$

$$= 5.8 \text{ cm}^2$$

ii) At the Line ② - ② in Fig. 5.1.5, where the largest tensile stress occur in the upper part of the slab

$$As = \frac{\frac{1}{2} \times 14 \text{ kg/cm}^2 \times 90 \text{ cm} \times 100 \text{ cm}}{1{,}800 \text{ kg/cm}^2}$$

$= 3.5 \text{ cm}^2 \rightarrow$ D29 @150 = 42.9 cm^2

iii) At the center of the invert slab

$$As = \frac{\frac{1}{2} \times 6 \text{ kg/cm}^2 \times 30 \text{ cm} \times 100 \text{ cm}}{1{,}800 \text{ kg/cm}^2}$$

$= 4.9 \text{ cm}^2$

(2) Min. reinforcement bars

The above standard (by JSCE), Clause 6.2.3, specifies a minimum area of tensile reinforcement bars of 0.2% for the concrete area.

According to this, a minimum area of re-bar at Line ① and ③ is calculated as follows:

i) Line ① - ①

$As \geq 130 \text{ cm} \times 100 \text{ cm} \times 0.2/100$
$= 26 \text{ cm}^2 \rightarrow$ D25 @150 = 33.8 cm^2

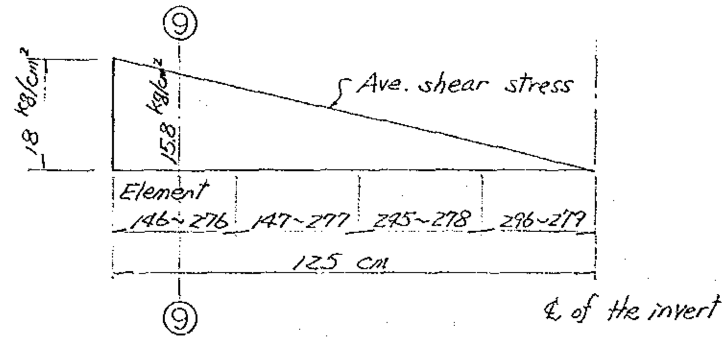
ii) Line ③ - ③

$As \geq 150 \text{ cm} \times 100 \text{ cm} \times 0.2/100$
$= 30 \text{ cm}^2 \rightarrow$ D25 @150 = 33.8 cm^2

(3) Stirrups

i) Line ⑨ - ⑨

- Shear stress distribution is as shown below.



A total shear stress is:

S = 0.5 x 18 kg/cm^2 x 100 cm x 125 cm

= 1,125 kg/cm^2

- Required area of stirrups

As = (S x t)/(1,800 kg/cm^2 x jd)

= 1,125 x 30/(1,800 x 150 x 2/3)

= 0.19 cm^2 → ø13 (=1.327 cm^2)

where; t : spacing at stirrups (cm)

jd : distance from compression resultant to centroid of tension reinforcement (m)

Pitch of the stirrups ≥ D25 x 15 = 375 cm or

ø13 x 48 = 624 cm or

130/2 = 65 cm

→ 350 cm

ii) Line ⑤ and ⑧

The shear stress at Line is smaller than Line . Accordingly the same stirrup arrangement in the invert slab as given above is made.

(4) Surface crack prevention re-bars

For prevention of surface cracks, D22 @300 is arranged.

(5) From the above (1) ~ (4), a reinforcement bar arrangement is made as shown in Fig. 5.1.6

5.2 Spillway

5.2.1 Wall Section C-C

(1) Case I

1) EL. 189.000 m

Shearing force

- earth P.	$Qe = 1.93 \times 7^2 \times \frac{1}{2} \times 0.241$	=	11.40 t

Bending moment

- earth P.	Me = 11.40 x 7/3	=	26.60 t.m

2) Toe

Shearing force

- reaction	Qr = 23.06 x 1.0		=	23.06 t
- concrete	Qc = -1 x 1.5 x 2.4		=	-3.60 t
- uplift	Qu = 14.5 x 1.0		=	14.50 t
		Q	=	33.96 t

Bending moment

- reaction	Mr = 23.06 x 1.0/2		=	11.53 t.m
- concrete	Mc = -3.6 x 1.0 x 1/2		=	-1.80 t.m
- uplift	Mu = 14.50 x 1.0 x 1/2		=	7.25 t.m
		M	=	16.98 t.m

(2) Case II

1) EL. 189.000 m

Shearing force

- earth P.	Qe = 11.40 x 1.05	=	11.97 t
	Qe = 1.8 x 1.0 x 1.93 x 0.05	=	0.17 t
	Qe = 1.8 x 6 x 1/2 x 1.93 x 0.05	=	0.52 t
- concrete	Qc_1 = 0.48 x 0.05	=	0.02 t
	Qc_2 = 1.2 x 0.05	=	0.06 t
	Qc_3 = 17.2 x 0.05	=	0.36 t

	$Qc_4 = 12.96 \times 0.05$		=	0.65 t
		Q	=	13.75 t

Bending moment

- earth P.	Me = 11.97 x 7/3		=	27.93 t.m
	Me = 0.17 x (6.0 + 0.5)		=	1.01 t.m
	Me = 0.52 x 6 x 2/3		=	2.08 t.m
- concrete	$Mc_1 = 0.02 \times (7 + 0.5)$		=	0.15 t.m
	$Mc_2 = 0.06 \times (6 + 0.5)$		=	0.39 t.m
	$Mc_3 = 0.36 \times 3$		=	1.08 t.m
	$Mc_4 = 0.65 \times 6/3$		=	1.30 t.m
		M	=	33.94 t.m

2) Toe

Shearing force

- reaction	Qr = 23.06 x 1.0		=	23.06 t
- concrete	Qc = -1 x 1.5 x 2.4		=	-3.60 t
- uplift	Qu = 14.5 x 1.0		=	14.50 t
		Q	=	33.96 t

Bending moment

- reaction	Mr = 23.06 x 1.0/2		=	11.53 t.m
- concrete	Mc = -3.6 x 1.0/2		=	-1.80 t.m
- uplift	Mu = 14.50 x 1.0/2		=	7.25 t.m
		M	=	16.98 t.m

5.2.2 Wall Section G-G

(1) Case I and II

1) Toe

Shearing force

- reaction	Qr = 22.0 x 1.0		=	22.00 t
- concrete	Qc = -1.5 x 1.0 x 2.4		=	-3.60 t
- uplift	Qu = 14.5 x 1.0		=	14.50 t
		Q	=	32.90 t

Bending moment

- reaction	$Mr = 22.00 \times 1.0/2$		=	11.00 t.m
- concrete	$Mc = -3.6 \times 1.0/2$		=	-1.80 t.m
- uplift	$Mu = 14.50 \times 1.0/2$		=	7.25 t.m
		M	=	16.45 t.m

5.2.3 Wall Section A-A

(1) Case II

1) Toe

Shearing force

- reaction	$Qr = 13.1 \times 1.0$		=	13.10 t
- concrete	$Qc = 1.0^2 \times -2.4$		=	-2.40 t
		Q	=	10.70 t

Bending moment

- reaction	$Mr = 13.10 \times 1.0/2$		=	6.55 t.m
- concrete	$Mc = -2.40 \times 1.0/2$		=	-1.20 t.m
		M	=	5.35 t.m

5.2.4 Wall Section E-E

(1) Case I

1) EL. 126.000 m

Shearing force

- earth P. $Qe = 1.93 \times 7^2 \times \frac{1}{2} \times 0.241$ = 11.40 t

Bending moment

- earth P. $Me = 11.40 \times 7 \times \frac{1}{3}$ = 26.60 t.m

2) Toe

Shearing force

- reaction	Qr = 24.10 x 1.0		=	24.10 t
- concrete	Qc = -1.0 x 1.5 x 2.4		=	-3.60 t
- water	Qw = -1.0 x 8.0 x 1.0		=	-8.00 t
- uplift	Qu = 12.5 x 1.0		=	12.50 t
		Q	=	25.00 t

Bending moment

- reaction	Mr = 24.10 x 1.0/2		=	12.05 t.m
- concrete	Mc = -3.6 x 1.0/2		=	-1.80 t.m
- water	Mw = -8.0 x 1.0/2		=	-4.0 t.m
- uplift	Mu = 12.50 x 1.0/2		=	6.25 t.m
		M	=	12.50 t.m

(2) Case II

1) EL. 126.000 m

Shearing force

- earth P.	Qe = 11.40 x 1.05		=	11.97 t
	Qe = 1.8 x 1.0 x 1.93 x 0.05		=	0.17 t
	Qe = 1.8 x 6 x 1/2 x 1.93 x 0.05		=	0.52 t
- concrete	Qc = 22.56 x 0.05		=	1.128 t
		Q	=	13.79 t

Bending moment

- earth P.	Me = 11.97 x 7x 1/3		=	27.93 t.m
	Me = 0.17 x (0.5 + 6.0)		=	1.11 t.m
	Me = 0.52 x 6 x 2/3		=	2.08 t.m
- concrete	Mc = 2.4 x 0.05 x (1 + 6.0)		=	0.84 t.m
	Mc = 7.2 x 0.05 x 6/2		=	1.08 t.m
	Mc = 12.96 x 0.05 x 6/3		=	1.30 t.m
		M	=	34.34 t.m

2) Toe

Shearing force

- reaction	Qr = 24.10 x 1.0		=	24.10 t
- concrete	Qc = -1.0 x 1.5 x 2.4		=	-3.60 t
- water	Qw = -1.0 x 8.0 x 1.0		=	-8.00 t
- uplift	Qu = 12.5 x 1.0		=	12.50 t
		Q	=	25.00 t

Bending moment

- reaction	Mr = 24.10 x 1.0/2		=	12.05 t.m
- concrete	Mc = -3.6 x 1.0/2		=	-1.80 t.m
- water	Mw = -8.0 x 1.0/2		=	-4.0 t.m
- uplift	Mu = 12.50 x 1.0/2		=	6.25 t.m
		M	=	12.50 t.m

(3) Case III

1) EL. 126.000 m

Shearing force

- earth P. $Qe = 1.93 \times 7^2 \times \frac{1}{2} \times 0.241$ = 11.40 t

Bending moment

- earth P. $Me = 11.40 \times 7 \times \frac{1}{3}$ = 26.60 t.m

2) Toe

Shearing force

- reaction	Qr = 19.7 x 1.0		=	19.70 t
- concrete	Qc = -1.0 x 1.5 x 2.4		=	-3.60 t
- water	Qw = -1.0 x 8.0 x 1.0		=	-8.00 t
- uplift	Qu = 15.5 x 1.0		=	15.50 t
		Q	=	23.60 t

Bending moment

- reaction	Mr = 19.7 x 1.0/2	=	9.85 t.m
- concrete	Mc = -3.6 x 1.0/2	=	-1.8 t.m
- water	Mw = -8.0 x 1.0/2	=	-4.0 t.m

- uplift	Mu = 15.50 x 1.0/2	=	7.75 t.m
	M	=	11.80 t.m

5.2.5 Wall Section I-I

(1) Case I

1) EL. 121.000 m

Shearing force

- earth P.	$Qe = 0.241 \times 1.93 \times 4.0^2 \times \frac{1}{2}$	=	3.72 t

Bending moment

- earth P.	$Me = 3.72 \times \frac{4}{3}$	=	4.96 t.m

2) Toe

Shearing force

- reaction	Qr = (22.40 + 20.93)/2 x 1.0	=	21.67 t
- concrete	Qc = -1.0 x 1.0 x 2.4	=	-2.40 t
- uplift	Qu = 0.29 x 1.0/2	=	0.15 t
	Q	=	19.42 t

Bending moment

- reaction	Mr = 20.93 x 1.0 x 1.0/2 + 1.47 x 1.0/2 x 1.0 x 2/3	=	10.96 t.m
- concrete	Mc = -2.4 x 1.0/2	=	-1.20 t.m
- uplift	Mu = 0.15 x 1.0/3	=	0.05 t.m
	M	=	9.81 t.m

(2) Case II

1) EL. 121.000 m

Shearing force

- earth P.	Qe = 3.72 x 1.05	=	3.91 t
	Qe = 4.632 x 0.05	=	0.23 t

- concrete	Qc = 66.48 x 0.05		=	3.32 t
		Q	=	7.46 t

Bending moment

- earth P.	Me = 3.91 x 4 x 1/3		=	5.21 t.m
	Me = 0.23 x 4 x 2/3		=	0.61 t.m
- concrete	Mc = 7.2 x 0.05 x (0.1 + 12.8)		=	4.64 t.m
	Mc = 0.36 x 0.05 x (0.2 + 12.5)		=	0.23 t.m
	Mc = 0.36 x 0.05 x (0.15 + 12.5)		=	0.23 t.m
	Mc = 1.80 x 0.05 x $(1.5 \times \frac{1}{2} + 11.0)$		=	1.06 t.m
	Mc = 3.2 x 0.05 x 11 x $\frac{1}{2}$		=	3.63 t.m
	Mc = 43.56 x 0.05 x 11 x $\frac{1}{3}$		=	7.99 t.m
		M	=	23.60 t.m

2) Toe

Shearing force

- reaction	Qr = (26.50 + 23.86)/2 x 1.0		=	25.18 t
- concrete	Qc = -1.0 x 1.0 x 2.4		=	-2.40 t
- uplift	Qu = 0.29 x 1.0/2		=	0.15 t
		Q	=	22.93 t

Bending moment

- reaction	Mr = 23.86 x 1.0 x 1.0/2 + 2.64 x 1.0/2 x 1.0 x 2/3		=	12.81 t.m
- concrete	Mc = -2.4 x 0.5		=	-1.20 t.m
- uplift	Mu = 0.15 x 1.0/3		=	0.05 t.m
		M	=	11.66 t.m

(3) Case III

1) EL. 121.000 m

Shearing force

- earth	Qc = 0.241 x (1.93 - 1.00) x $4.0^2/2$		=	1.79 t
- water	Qw = $11.0^2/2$		=	-60.50 t
	Qw = $8.0^2/2$		=	32.00 t
		Q	=	-26.71 t

Bending moment

- earth	Mc = 1.79 x 4.0/3		=	2.39 t.m
- water	Mw = -60.50 x 11.0/3		=	-221.83 t.m
	Mw = 32.00 x 8.0/3		=	85.33 t.m
		M	=	-134.11 t.m

2) Toe

Shearing force

- reaction	Qr = 0.04 x 0.15/2		=	0.01 t
- concrete	Qc = -1.0 x 1.0 x 2.40		=	-2.40 t
- water	Qw = -13.0 x 1.0 x 1.0		=	-13.00 t
- uplift	Qu = (14.00 + 13.57)/2 x 1.0		=	13.79 t
		Q	=	-1.60 t

Bending moment

- reaction	Mr = 0.01 x 0.15/3		=	0.01 t.m
- concrete	Mc = -2.4 x 1.0/2		=	-1.20 t.m
- water	Mw = -13.0 x 1.0/2		=	-6.50 t.m
- uplift	Mu = 13.57 x 1.0/2 + 0.43 x 1.0/2 x 1.0 x 2/3		=	6.93 t.m
		M	=	-0.76 t.m