3.7.2. Structural Analysis

The structural analyses were conducted for components of the powerhouse structure under El. 98.00 m. Each component such as the walls, floors, tailrace, draft pit and foot of the spillway was assumed to be a flat slab, beam, and the combination of them under the several load conditions, which are shown in Fig. 3.7.4.

3.7.2.1. Computation Cases and Structural Models

(1) Computation Cases

The following loading conditions were considered for the design of the powerhouse. The PMF condition is added to the load condition for stability analysis of the powerhouse for the design of the outer wall on the spillway side.

a. Normal condition : This is a normal condition after completion of the powerhouse.

b. Flood condition : This is added to the normal condition in which the water level in the stilling basin is subject to the flood of the 100 year return period (Q = 340 m^3/s).

c. Seismic condition : This is a seismic condition, which occurs when an earthquake hits the project during operation.

d. PMF condition : This is the unusual condition in which the water level in the stilling basin is subject to PMF (Q = 1,310 m^3/s).

(2) Structural Models

The structural models and load combination are shown in Table 3.7.9

3.7.2.3. Loads

Parameters and the formulae of the main loads are shown in the previous section 1.7.1. "Stability of Powerhouse" and "the Design Criteria Report, March 1999".

The following loads are considered for the structural design of the powerhouse.

① Dead Load	:	The dead load of concrete slabs and beams is considered. In seismic condition, inertia of dead load is also considered.
② Soil Pressure	:	The soil pressure is considered as pressure of backfill under EL. 97.00 under normal condition and seismic condition.
③ Ground water pressure	:	The pressure is assumed to exert on the outer walls under EL.84.90.
④ Water Pressure	:	The water pressure is considered to exert on the spillway side walls below EL. 92.08 and EL. 98.00 under flood condition and PMF condition, respectively.
⑤ Dynamic Water Pressure	:	The water pressure in seismic condition is considered.
⑥ Floor Load	:	Uniform load of 1.0 t/m^2 is considered for the design of floors at El. 97.5 and EL. 93.00. For the assembly bay at EL. 97.50, uniform load of 5.0 t/ m^2 is considered.
⑦ Traffic Load	:	Uniform load is assumed as traffic load on the ground at El. 97.00. It is also assumed that soil pressure due to traffic load would exert on the side walls of the powerhouse.
⑧ Ground Reaction	:	Ground reaction calculated in the previous section 1.7.1 "Stability of Powerhouse" is applied for the design of bottom slabs.

3.7.2.4. Allowable Stress

The allowable stresses are tabulated in the following table.

Load Condition		Normal Condition	Flood Condition	Seismic Condition	PMF Condition
Concrete	Type	K-225			
	28 day Compressive Strength	225 kgf/cm^2			
	Allowable Compressive Stress	75 kgf/cm^2		112.5 kgf/cm^2	
	Allowable Shearing Stress	8 kgf/cm^2 (without shear reinforcement)		12 kgf/cm^2 (without shear reinforcement)	
Reinforcing Bars	Material	SD30			
	Allowable Tensile Stress	1,800 kgf/cm^2		2,700 kgf/cm^2	
	Allowable Compressive Stress	1,800 kgf/cm^2		2,700 kgf/cm^2	

3.7.2.5. Sectional Forces

The sectional forces of structural members of the powerhouse were calculated as follow.

(1) Flat slabs

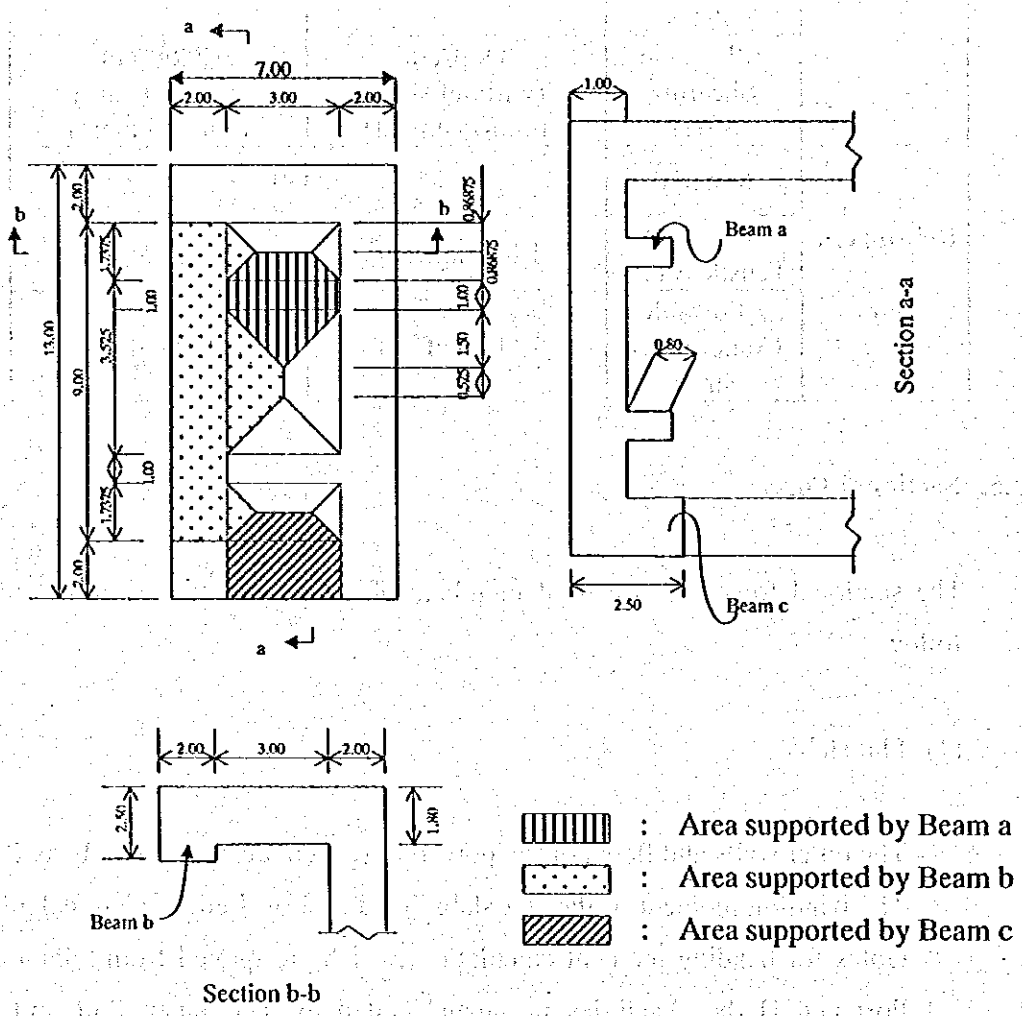
The outer walls and floors of the powerhouse were designed mainly as flat slabs. The bending moment of the flat slabs fixed at 3 or 4 edges were calculated by tables for bending moment calculation, which are quoted from "Standards for Port and Harbor Facilities in Japan" edited by The Japan Port and Harbor Association. Calculation sheets are shown in Fig. 3.7.5.

For design of bottom slab C (BS-1) and outer wall on the spillway side (WS-1), computer program MSC/NASTRAN was employed. The results of the analyses are shown in Fig. 3.7.6 and 3.7.7.

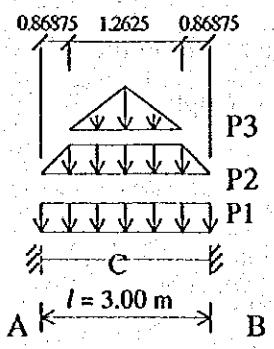
(2) Slab with beams

The assembly bay at EL. 97.50 was designed as a slab supported by beams. The sectional forces were calculated as follows;

It was assumed that three beams support floor load of 5.0 t/m^2 in the area as shown in the below figure.



a) Beam a



Load per 1 m width

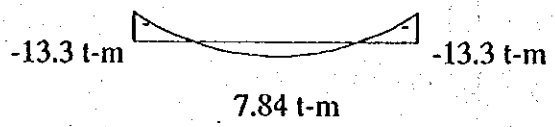
P1 = 0.8 × 2.5 = 2.0 t/m²

P2 = (1.0 + 0.86875 × 2) × (2.5 × 1.0 + 5.0) / 1.0 = 20.5 t/m²

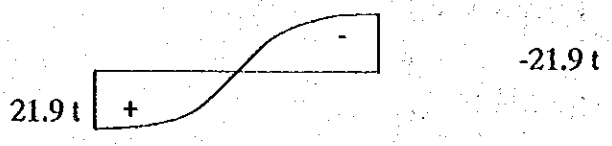
P3 = (1.5 - 0.86875) × (2.5 × 1.0 + 5.0) / 1.0 = 4.73 t/m²

Sectional Forces

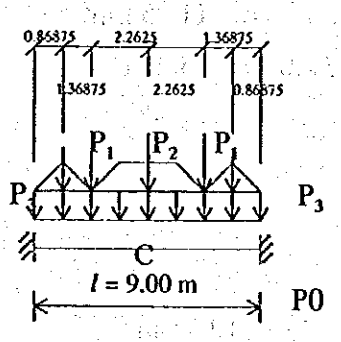
Moment



Sharing Force



b) Beam b



Load per 2 m width

P0 = 2.5 × 2.5 + 5.0 = 11.25 t/m²

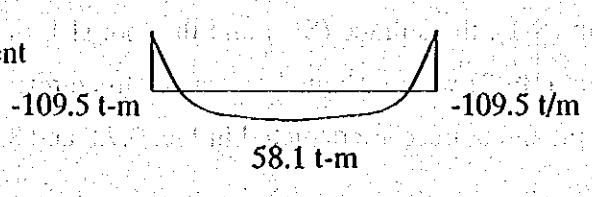
P1 = RA (Reaction of Beam a) = 21.9 t/m²

P2 = $\frac{\ }{2}$ × (3.525 +0.525) × 1.5 × (2.5 × 1.0 + 5.0) = 22.8 t/m²

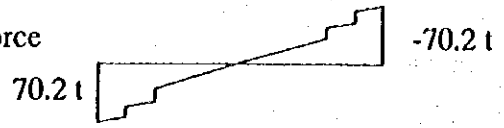
P3 = $\frac{\ }{2}$ × 1.7375 × 0.86875 × (2.5 × 1.0 + 5.0) = 5.66 t/m²

Sectional Forces per 1 m width

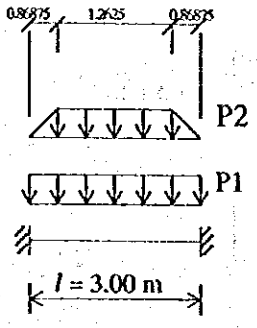
Moment



Sharing Force



c) Load of Beam c per 2 m width



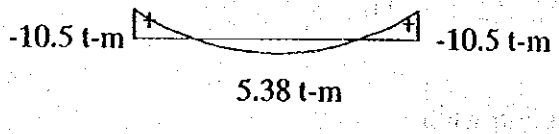
<u>Load per 1 m width</u>

P1 = 2.5 × 2.5 + 5.0 = 11.25 t/m²

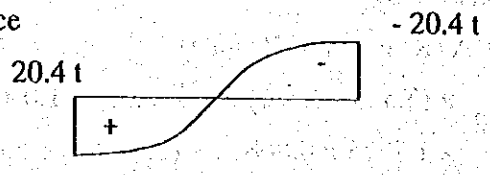
P2 = 0.86875 × (2.5 × 1.0 + 5.0) / 2.0 = 3.26 t/m²

<u>Sectional Forces</u>

Moment



Sharing Force



(3) Frame structure

The draft pit (S-1), the tailrace (S-2) and the foot slab of the outer wall on the spillway side (BS-2) were designed as frame structures. The sectional forces of the draft pit and tailrace are depicted in Fig. 3.7.8 and 3.7.9.

3.7.2.6. Arrangement of Reinforcing Bars

Bar arrangement and stress calculation of the reinforcing bars are shown in Table 3.7.10.

Table 3.7.9(1/4) Structural Models and Load Combination

Case No.	Structure	Structural Model	Loads and Load Combination			
			a. Normal Condition	b. Flood Condition	c. Seismic Condition	d. PMF Condition
W-1	Outer Wall on the Mountain Side	*Flat Slab* EL.97.50 12.60 m EL.84.90 Fixed 15.00 m t = 1.00 ~ 2.00 m	⑦Traffic load ②Soil pressure EL.97.00 12.10 m 0.251 t/m² 5.77 t/m²	None	①Inertia of dead load ②Soil pressure 0.403 t/m EL.97.00 0.800 t/m² 8.44 t/m²	None
W-2	Outer Wall on the Spillway Side	*Flat Slab* EL.98.00 13.10 m EL.84.90 15.00 m t = 2.00 m	None	④Water pressure EL.92.08 7.18 m 7.18 t/m	①Inertia of dead load ⑤Dynamic water pressure EL.98.00 13.10 m 0.800 t/m² EL.92.08 7.18 m 1.01 t/m²	④Water pressure EL.98.00 13.10 m 13.1 t/m²
W-3	Outer Wall at the Downstream End	*Flat Slab* EL.96.80 3.80 m EL.93.00 9.00 m t = 2.00 m	⑦Traffic load ②Soil pressure 3.80 m 0.251 t/m² 1.91 t/m²	None	①Inertia of dead load ②Soil pressure 0.15 0.800 t/m² 2.80 t/m²	None
W-4	Outer Wall at the Downstream End	*Flat Slab* EL.92.00 7.10 m EL.84.90 9.00 m t = 2.00 m	⑦Traffic load ②Soil pressure 2.38 7.10 m 0.251 t/m² 5.77 t/m²	None	①Inertia of dead load ②Soil pressure 3.48 0.800 t/m² 8.44 t/m²	None
W-5	Outer Wall at the Upstream End	*Flat Slab* EL.97.50 12.60 Fixed t = 2.00 m EL.84.90 9.00	⑥Floor load ②Soil pressure 1.26 t/m² 5.77 t/m²	None	①Inertia of dead load ②Soil pressure 0.800 t/m² 8.44 t/m²	None

Table 3.7.9(2/4) Structural Models and Load Combination

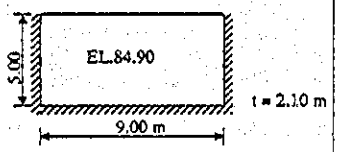
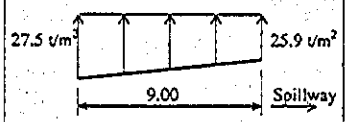
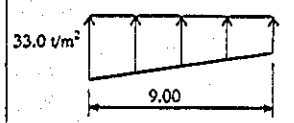
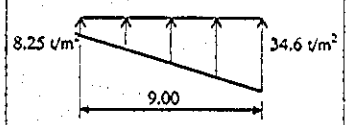
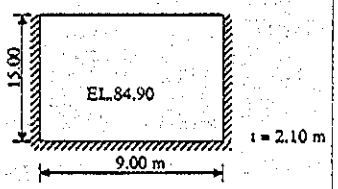
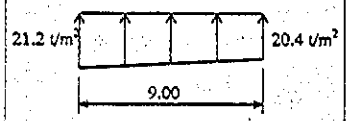
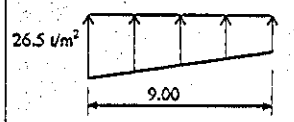
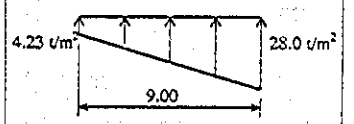
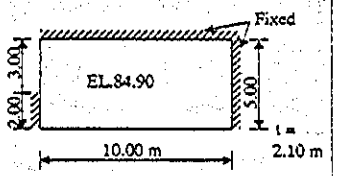
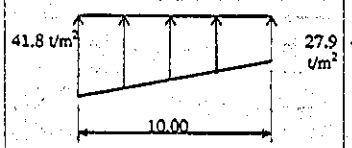
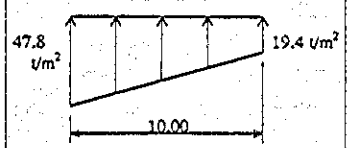
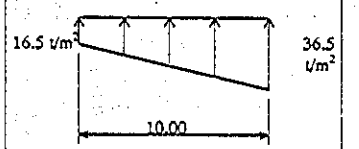
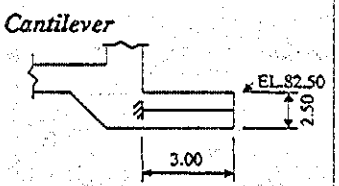
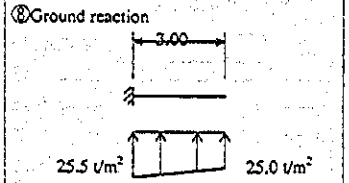
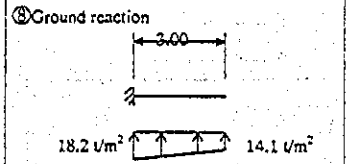
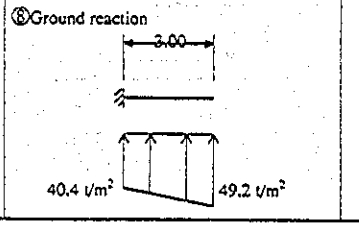
Case No.	Structure	Structural Model	Loads and Load Combination			
			a. Normal Condition	b. Flood Condition	c. Seismic Condition	d. PMF Condition
B-1	Bottom Slab A at EL.84.9	*Flat Slab* 	Ⓢ Ground reaction 	Ⓢ Ground reaction 	Ⓢ Ground reaction 	None
B-2	Bottom Slab B at EL.84.9	*Flat Slab* 	Ⓢ Ground reaction 	Ⓢ Ground reaction 	Ⓢ Ground reaction 	None
BS-1	Bottom Slab C at EL.84.9	*Flat Slab* 	Ⓢ Ground reaction 	Ⓢ Ground reaction 	Ⓢ Ground reaction 	None
BS-2	Foot Slab	*Cantilever* 	Ⓢ Ground reaction 	Ⓢ Ground reaction 	Ⓢ Ground reaction 	None

Table 3.7.9(3/4) Structural Models and Load Combination

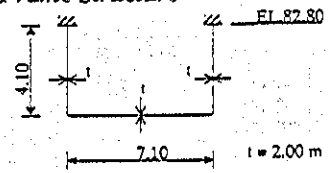
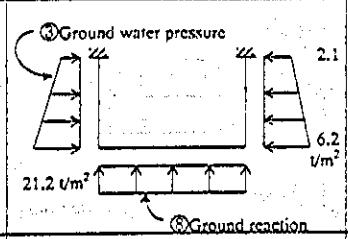
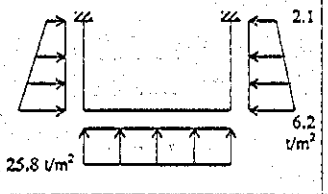
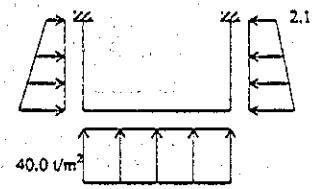
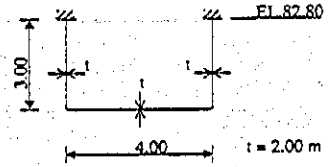
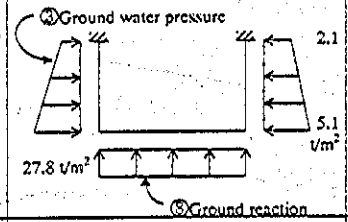
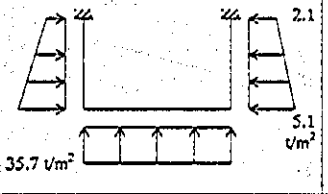
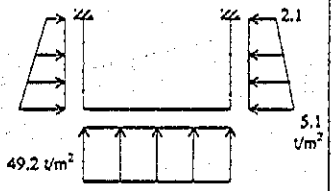
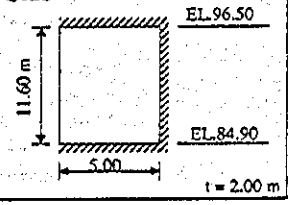
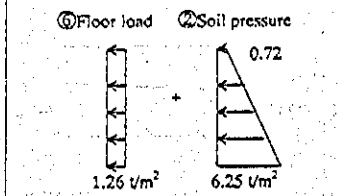
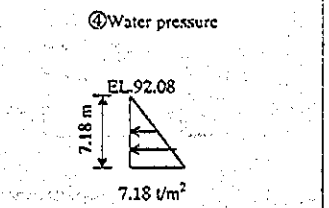
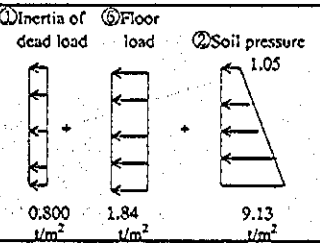
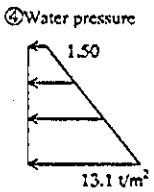
Case No.	Structure	Structural Model	Loads and Load Combination			
			a. Normal Condition	b. Flood Condition	c. Seismic Condition	d. PMF Condition
S-1	Draft Pit	*Frame Structure* 				None
S-2	Tailrace	*Frame Structure* 				None
WS-1	Outer Wall on the Spillway Side	*Flat Slab* 				

Table 3.7.9(4/4) Structural Models and Load Combination

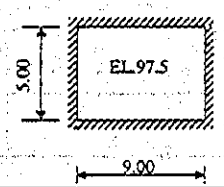
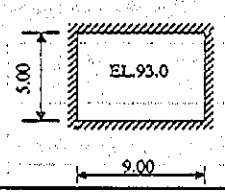
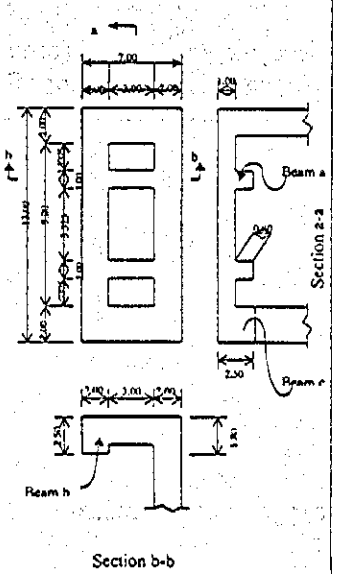
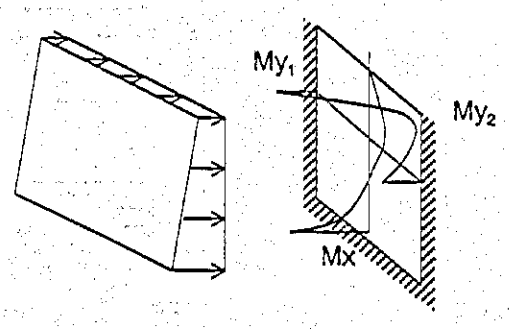
Case No.	Structure	Structural Model	Loads and Load Combination			
			a. Normal Condition	b. Flood Condition	c. Seismic Condition	d. PMF Condition
F-1	Floor of Control Room at EL.97.5	*Flat Slab*  t = 0.70 m	①Dead load 1.75 t/m² + ⑥Floor load 1.00 t/m² 9.00	None	None	None
F-2	Floor of Cable Room at EL.93.00	*Flat Slab*  t = 1.00 m	①Dead load 2.5 t/m² + ⑥Floor load 1.00 t/m² 9.00	None	None	None
F-3	Floor of Assembly Bay at EL.97.5	*Slab with Beams*  	i) Beam a ⑥Floor load 25.2 t/m² ①Dead load 2.0 t/m² 5.00 ii) Bream b Reaction of Beam a 21.9 t ⑥Floor load 5.00 ①Dead load 11.25 t/m² 9.00 iii) Beam C ⑥Floor load 3.26 t/m² ①Dead load 11.3 t/m² 3.00	None	None	None

Table 3.7.10 (1/3) Arrangement of Reinforcing Bars for Slabs Fixed at 3 edges



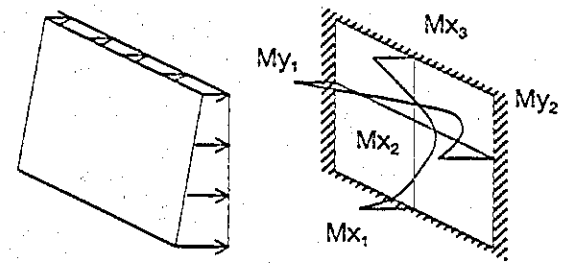
Note; 1) Load Condition N: Normal Condition, F: Flood Condition, S: Seismic Condition, P: PMF Condition

Case No.	Bending Moment (t-m/m)		Load Condition	Design of Reinforcing Bars						Remarks
				Thick-ness	Bar Arrangement	Calculated Stress (kgf/cm²)		Allowable Stress (kgf/cm²)		
						Reinforcing Bars	Concrete	Reinf. Bars	Concrete	
W-1	Mx	-45.3	N	200	D25@250 D19@250	1,247	16	1,800	75	
	My1	-35.5	N	100	D25@125 D19@250	1,081	30	1,800	75	
	My2	16.1	N	100	D19@125 D25@250	848	17	1,800	75	
W-2	Mx	-97.9	P	200	(D25+D19)@250 D19@250	1,735	28	2,700	112.5	
	My1	-78.2	P	200	D22@125 D19@250	1,419	23	2,700	112.5	
	My2	33.5	P	200	D19@250 D22@250	1,605	15	2,700	112.5	
W-5	Mx	-24.9	N	200	D19@250 D19@250	1,193	11	1,800	75	
	My1	-25.5	N	200	D19@250 D19@250	1,222	11	1,800	75	
	My2	12.0	N	200	D19@250 D19@250	575	5	1,800	75	

Table 3.7.10(1/3) Continued

Case No.	Bending Moment (t-m/m)		Load Condition	Design of Reinforcing Bars						Remarks
				Thick-ness	Bar Arrangement	Calculated Stress (kgf/cm²)		Allowable Stress (kgf/cm²)		
						Reinforcing Bars	Concrete	Reinf. Bars	Concrete	
WS-1	Mx	-28.7	N	200	D19@250 D19@250	1,375	13	1,800	75	
	My1	-39.8	N	200	D19@125 D19@125	966	12	1,800	75	
	My2	14.1	N	200	D19@250 D19@250	675	6	1,800	75	
BS-1	Mx	-116.2	N	210	D25@125 D19@250	1,581	29	1,800	75	
	My1	-171.3	N	243 *	D29@125 D29@250	1,571	29	1,800	75	* Thickness includes haunch.
	My2	79.6	N	210	D29@250 D29@250	1,645	22	1,800	75	

Table 3.7.10 (2/3) Arrangement of Reinforcing Bars for Slabs Fixed at 4 edges



Note; 1) Load Condition N: Normal Condition, F: Flood Condition, S: Seismic Condition, P: PMF Condition

Case No.	Bending Moment (t-m/m)		Load Condition	Design of Reinforcing Bars						Remarks
				Thick-ness	Bar Arrangement	Calculated Stress (kgf/cm²)		Allowable Stress (kgf/cm²)		
						Reinforcing Bars	Concrete	Reinf. Bars	Concrete	
W-3	Mx1	-3.07	S	100	D19@250 D19@250	317	4	2,700	112.5	
	My1	-1.87	S	100	D19@250 D19@250	193	3	2,700	112.5	
	My2	0.28	S	100	D19@250 D19@250	29	1	2,700	112.5	
W-4	Mx1	-24.9	S	100	D19@250 D19@250	2,571	35	2,700	112.5	
	Mx2	10.2	S	100	D19@250 D19@250	1,053	15	2,700	112.5	
	My1	-19.0	S	100	D19@250 D19@250	1,961	27	2,700	112.5	
	My2	6.3	S	100	D19@250 D19@250	651	9	2,700	112.5	
B-1	Mx1	-43.3	F	210	D19@125 D19@250	1,025	14	1,800	75	
	My1	-54.6	F	210	D19@125 D19@250	1,293	17	1,800	75	
	My2	26.4	F	210	D19@250 D19@125	1,231	11	1,800	75	

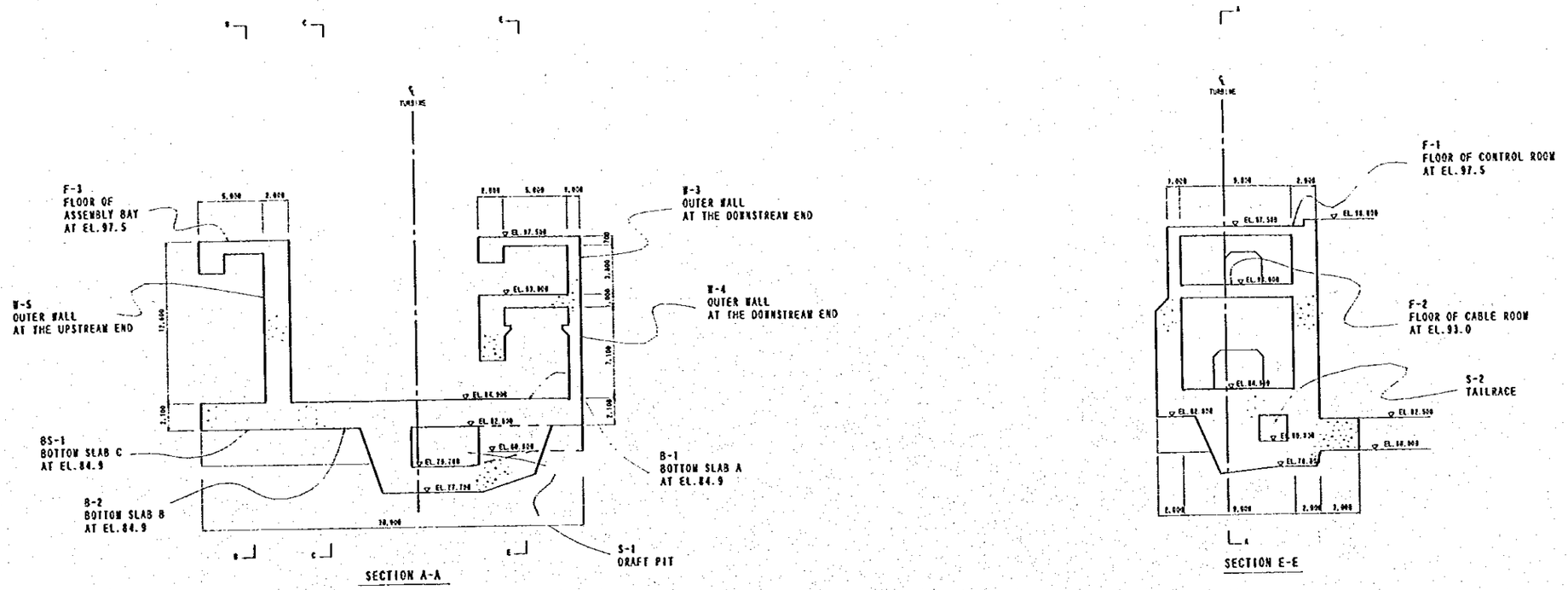
Table 3.7.10(2/3) Continued

Case No.	Bending Moment (t-m/m)		Load Condition	Design of Reinforcing Bars						Remarks
				Thick-ness	Bar Arrangement	Calculated Stress (kgf/cm²)		Allowable Stress (kgf/cm²)		
						Reinforcing Bars	Concrete	Reinf. Bars	Concrete	
B-2	Mx1	-147.0	F	210	D29@125 D29@250	1,584	32	1,800	75	
	Mx2	67.7	F	210	D29@250 D29@250	1,430	19	1,800	75	
	My1	-103.5	F	210	D25@125 D19@250	1,408	26	1,800	75	
	My2	24.5	F	210	D19@250 D25@250	1,142	23	1,800	75	
F-1	Mx1	-5.5	N	70	D19@250 D19@250	864	15	1,800	75	
	Mx2	2.66	N	70	D19@250 D19@250	418	7	1,800	75	
	My1	-3.91	N	70	D19@250 D19@250	614	11	1,800	75	
F-2	Mx1	-7.13	N	100	D19@250 D19@250	736	10	1,800	75	
	Mx2	3.48	N	100	D19@250 D19@250	359	5	1,800	75	
	My1	-4.98	N	100	D19@250 D19@250	514	7	1,800	75	

Table 3.7.10 (3/3) Arrangement of Reinforcing Bars for Beams and Frame Structures

Note; 1) Load Condition N: Normal Condition, F: Flood Condition, S: Seismic Condition, P: PMF Condition

Case No.	Bending Moment (t-m/m)		Sharing Force (t)	1) Load Condition	Design of Reinforcing Bars								Remarks
					Thick-ness	Bar Arrangement	Calculated Stress (kgf/cm²)			Allowable Stress (kgf/cm²)			
							Tension σ_t	Compr-ession σ_c	Sharing τ	Tension σ_t	Compr-ession σ_c	Sharing τ	
BS-2	Mmax	-208.2	134.4	S	250	D25@125 D19@250	2,337	39	6	2,700	112.5	12	
F-3 Beam a	Mmax	-13.3	21.9	N	180	D19@250 D19@250	714	7	1	1,800	75	8	
Beam b	Mmax	-109.5	70.2	N	250	D22@125 D19@250	1,562	22	3	1,800	75	8	
Beam c	Mmax	-10.5	20.4	N	250	D19@250 D19@250	396	3	1	1,800	75	8	
S-1	Mmax	81.9	91.5	F	200	D22@125 D22@125	1,480	35	5	1,800	75	8	
S-2	Mmax	38.9	71.4	F	200	D19@125 D19@125	944	12	4	1,800	75	8	



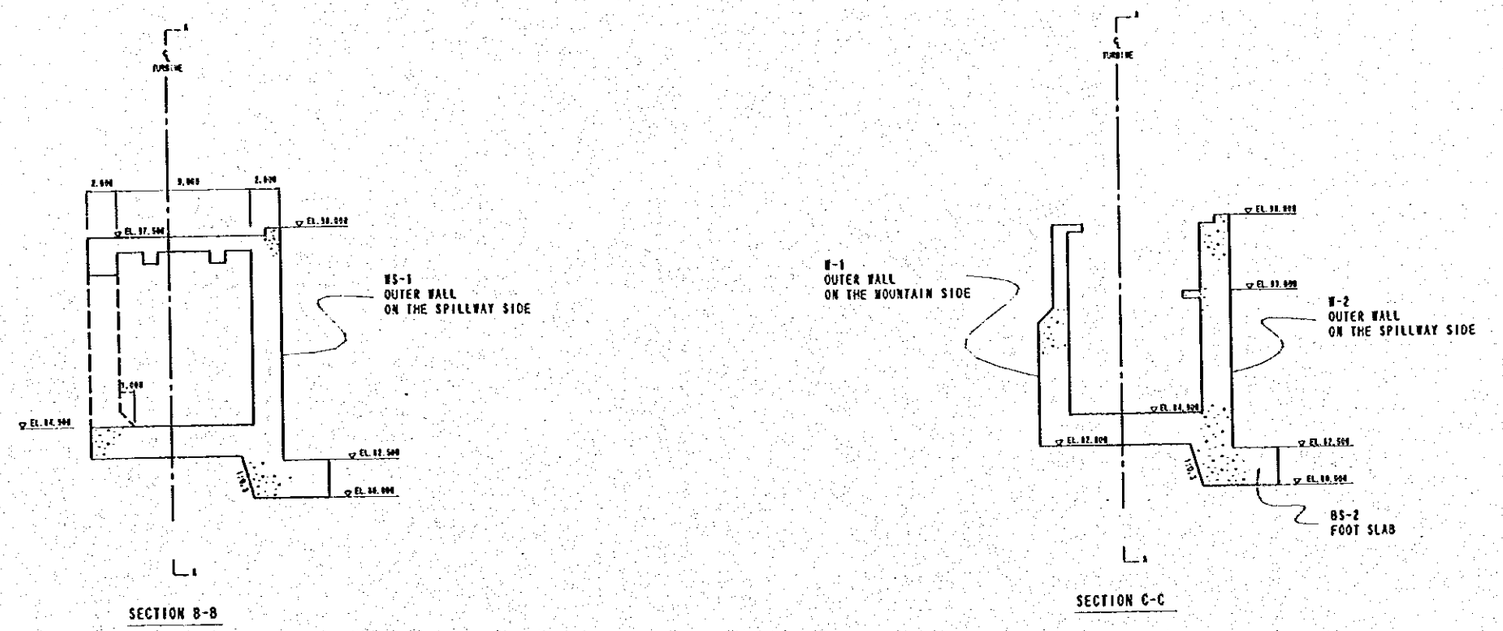


Fig. 3.7.4
POWERHOUSE GENERAL SECTIONS

Fig. 3.7.5(1/9) Bending Moment of Flat Slabs

W-1 : Powerhouse Outer Wall (12.6 x 15.00) : Normal Condition

1) Moment Coefficient

lx =	12.60 m	Uniform Load	w =	0.251 t/m2
ly =	15.00 m	Trangulation Load	q =	5.77 t/m2
=	0.84			

Load Type		No.	1	2	3	4	5	6	7
Uniform Load	Kx	I	-0.0837	-0.0188	0.0089	0.0174	0.0166	0.0109	0.0000
		II	-0.0509	-0.0105	0.0052	0.0089	0.0072	0.0038	0.0000
		III	0.0000	-0.0034	-0.0093	-0.0140	-0.0171	-0.0195	0.0000
	Ky	I	-0.0139	0.0042	0.0224	0.0369	0.0471	0.0539	0.0594
		II	-0.0085	0.0066	0.0087	0.0127	0.0149	0.0160	0.0171
		III	0.0000	-0.0204	-0.0560	-0.0840	-0.1025	-0.1167	-0.1247
Trangulation Load	Kx	I	-0.0458	-0.0043	0.0105	0.0120	0.0078	0.0029	0.0000
		II	-0.0305	-0.0017	0.0069	0.0068	0.0039	0.0009	0.0000
		III	0.0000	-0.0162	-0.0050	-0.0060	-0.0057	-0.0049	0.0000
	Ky	I	-0.0077	0.0025	0.0104	0.0146	0.0155	0.0150	0.0146
		II	-0.0051	0.0018	0.0055	0.0062	0.0053	0.0039	0.0028
		III	0.0000	-0.0141	-0.0300	-0.0360	-0.0343	-0.0293	-0.0205

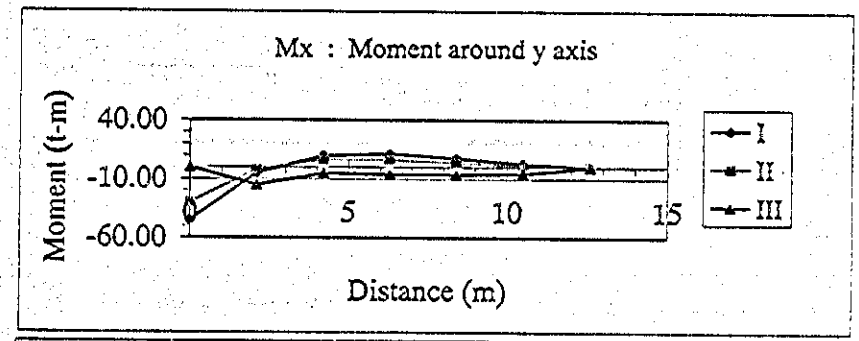
Note; The above coefficients were calculated in proportion to those tabulated in "The Technical Standards for Port and Harbor Facilities" by The Japanese Port and Harbor Association.

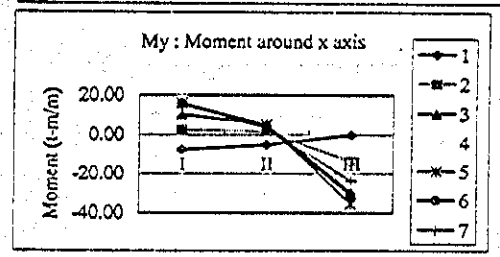
2) Moment

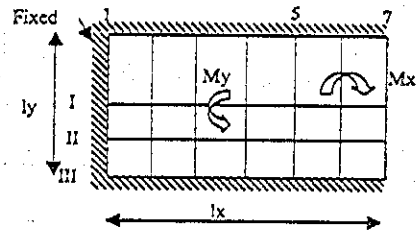
< 1 : Mx = Kx·q·lx^2 (around y axis)

: My = Ky·q·lx^2 (around x axis)

Load Type		No.	1	2	3	4	5	6	7
Uniform Load	Mx	I	-3.3353	-0.7484	0.3561	0.6932	0.6628	0.4356	0.0000
		II	-2.0273	-0.4186	0.2082	0.3545	0.2875	0.1511	0.0000
		III	0.0000	-0.1352	-0.3723	-0.5571	-0.6795	-0.7755	0.0000
	My	I	-0.5557	0.1662	0.8931	1.4715	1.8764	2.1496	2.3686
		II	-0.3368	0.2625	0.3448	0.5078	0.5918	0.6369	0.6821
		III	0.0000	-0.8128	-2.2312	-3.3487	-4.0835	-4.6502	-4.9690
Trangulation Load	Mx	I	-41.9695	-3.9610	9.6478	10.9669	7.1855	2.6565	0.0000
		II	-27.9540	-1.5609	6.3097	6.2401	3.5396	0.8281	0.0000
		III	0.0000	-14.8289	-4.5912	-5.5183	-5.2618	-4.4886	0.0000
	My	I	-7.0132	2.3268	9.5525	13.3413	14.2317	13.6967	13.3596
		II	-4.6535	1.6306	5.0749	5.7198	4.8294	3.5469	2.5979
		III	0.0000	-12.9016	-27.4887	-32.9923	-31.4130	-26.8071	-18.8046







Load Type		No.	1	2	3	4	5	6	7
Uniform + Traiangulation Loads	Mx	I	-45.30	-4.71	10.00	11.66	7.85	3.09	0.00
		II	-29.98	-1.98	6.52	6.59	3.83	0.98	0.00
		III	0.00	-14.96	-4.96	-6.08	-5.94	-5.26	0.00
	My	I	-7.57	2.49	10.45	14.81	16.11	15.85	15.73
		II	-4.99	1.89	5.42	6.23	5.42	4.18	3.28
		III	0.00	-13.71	-29.72	-36.34	-35.50	-31.46	-23.77

Required Re-bar a = 1800 kg/cm2 Re-bar covering depth 10 cm

			1	2	3	4	5	6	7
Distance(m)			0	2.1	4.2	6.3	8.4	10.5	12.6
Thickness (cm)			200	200	200	200	100	100	100
Required Re-bar for	Mx	I	-15.1	-1.6	3.3	3.9	5.5	2.2	0.0
		II	-10.0	-0.7	2.2	2.2	2.7	0.7	0.0
		III	0.0	-5.0	-1.7	-2.0	-4.2	-3.7	0.0
	My	I	-2.5	0.8	3.5	4.9	11.4	11.2	11.1
		II	-1.7	0.6	1.8	2.1	3.8	3.0	2.3
		III	0.0	-4.6	-9.9	-12.1	-25.0	-22.2	-16.8

Fig. 3.7.5 (2/9) Bending Moment of Flat Slabs

W-2 : Powerhouse Wall facing Spillway (13.1 x 15.00) : PMF Condition

1) Moment Coefficient

lx = 13.10 m Uniform Load w = 0 t/m2
ly = 15.00 m Trangulation Load q = 13.1 t/m2
= 0.873

Load Type		No.	1	2	3	4	5	6	7
Uniform Load	Kx	I	-0.0780	-0.0162	0.0093	0.0165	0.0155	0.0101	0.0000
		II	-0.0474	-0.0090	0.0053	0.0084	0.0066	0.0034	0.0000
		III	0.0000	-0.0034	-0.0091	-0.0134	-0.0161	-0.0183	0.0000
	Ky	I	-0.0130	0.0045	0.0220	0.0356	0.0449	0.0510	0.0560
		II	-0.0079	0.0057	0.0084	0.0121	0.0139	0.0149	0.0159
		III	0.0000	-0.0202	-0.0543	-0.0804	-0.0970	-0.1095	-0.1162
Trangulation Load	Kx	I	-0.0436	-0.0034	0.0104	0.0113	0.0072	0.0026	0.0000
		II	-0.0289	-0.0012	0.0067	0.0064	0.0035	0.0008	0.0000
		III	0.0000	-0.0133	-0.0049	-0.0058	-0.0055	-0.0046	0.0000
	Ky	I	-0.0073	0.0027	0.0104	0.0142	0.0149	0.0140	0.0135
		II	-0.0048	0.0018	0.0054	0.0060	0.0050	0.0036	0.0025
		III	0.0000	-0.0140	-0.0294	-0.0348	-0.0326	-0.0273	-0.0185

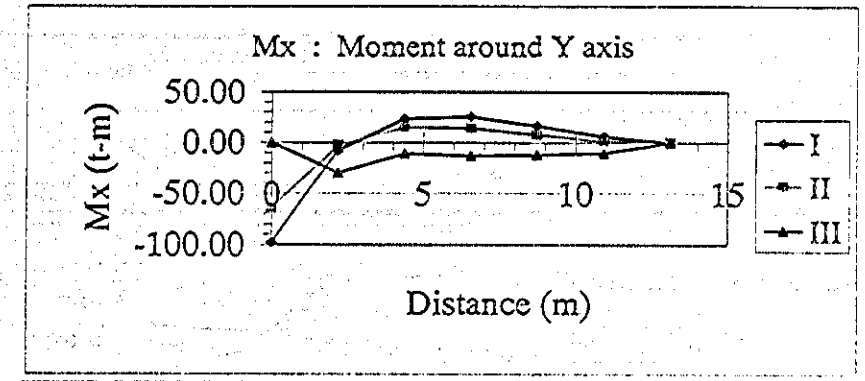
Note: The above coefficients were calculated in proportion to those tabulated in "The Technical Standards for Port and Harbor Facilities" by The Japanese Port and Harbor Association.

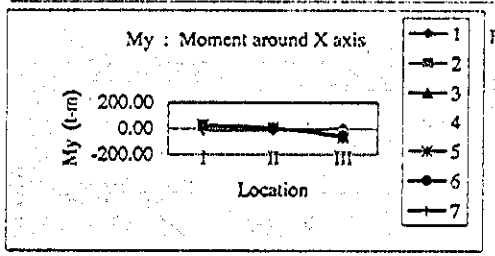
2) Moment

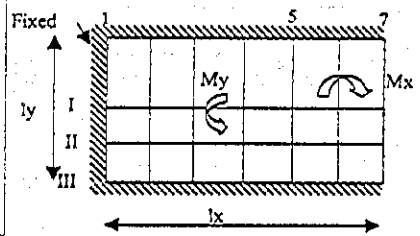
< 1 : Mx = Kx·q·lx^2 (around y axis)

: My = Ky·q·lx^2 (around x axis)

Load Type		No.	1	2	3	4	5	6	7
Uniform Load	Mx	I	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
		II	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
		III	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	My	I	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
		II	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
		III	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Trangulation Load	Mx	I	-97.9328	-7.7424	23.2872	25.4754	16.2552	5.7701	0.0000
		II	-65.0358	-2.7517	14.9753	14.3248	7.9073	1.7026	0.0000
		III	0.0000	-29.8876	-11.0276	-13.0629	-12.2836	-10.2663	0.0000
	My	I	-16.3421	6.1598	23.3831	31.9619	33.4576	31.5752	30.3582
		II	-10.8208	4.1515	12.1547	13.4076	11.1625	8.0152	5.6562
		III	0.0000	-31.4823	-66.0519	-78.1496	-73.2548	-61.2620	-41.5927







Load Type		No.	1	2	3	4	5	6	7
Uniform + Traiangulation Loads	Mx	I	-97.93	-7.74	23.29	25.48	16.26	5.77	0.00
		II	-65.04	-2.75	14.98	14.32	7.91	1.70	0.00
		III	0.00	-29.89	-11.03	-13.06	-12.28	-10.27	0.00
	My	I	-16.34	6.16	23.38	31.96	33.46	31.58	30.36
		II	-10.82	4.15	12.15	13.41	11.16	8.02	5.66
		III	0.00	-31.48	-66.05	-78.15	-73.25	-61.26	-41.59

Required Re-bar a = 2700 kg/cm2 Re-bar covering depth 10 cm

			1	2	3	4	5	6	7
Distance(m)			0	2.18	4.37	6.55	8.73	10.92	13.10
Thickness (cm)			200	200	200	200	200	200	200
Required Re-bar for	Mx	I	-21.8	-1.7	5.2	5.7	3.6	1.3	0.0
		II	-14.5	-0.6	3.3	3.2	1.8	0.4	0.0
		III	0.0	-6.7	-2.5	-2.9	-2.7	-2.3	0.0
	My	I	-3.6	1.4	5.2	7.1	7.5	7.0	6.8
		II	-2.4	0.9	2.7	3.0	2.5	1.8	1.3
		III	0.0	-7.0	-14.7	-17.4	-16.3	-13.6	-9.3

Fig. 3.7.5(3/9) Bending Moment of Flat Slabs

W-3 : Powerhouse Downstream Side Wall 2 of 2 (3.80 x 9.00) : Seismic Condition

1) Moment Coefficient

lx = 3.80 m Uniform Load w = 0.95 t/m2
ly = 9.00 m Trangulation Load q = 2.65 t/m2
= 0.422

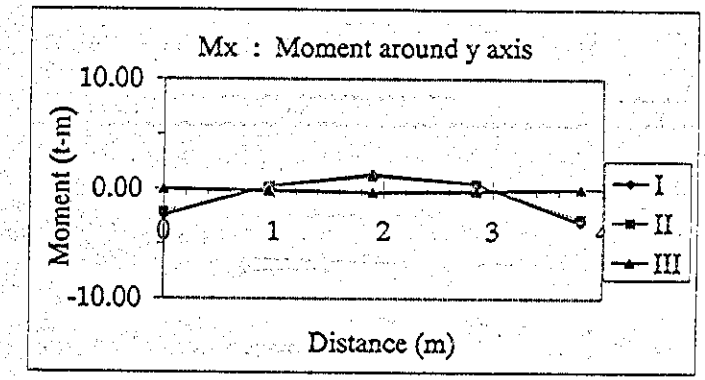
Load Type		No.	1	2	3	4	5
Uniform Load	Kx	I	-0.0837	0.0108	0.0416	0.0108	-0.0837
		II	-0.0731	0.0105	0.0345	0.0105	-0.0731
		III	0.0000	-0.0058	-0.0095	-0.0058	0.0000
	Ky	I	-0.0140	0.0026	0.0086	0.0026	-0.0140
		II	-0.0122	0.0059	0.0131	0.0059	-0.0122
		III	0.0000	-0.0345	-0.0569	-0.0345	0.0000
Trangulation Load	Kx	I	-0.0335	0.0017	0.0208	0.0090	-0.0502
		II	-0.0284	0.0017	0.0173	0.0088	-0.0448
		III	0.0000	-0.0021	-0.0048	-0.0036	0.0000
	Ky	I	-0.0056	0.0007	0.0043	0.0019	-0.0084
		II	-0.0047	0.0022	0.0066	0.0037	-0.0074
		III	0.0000	-0.0127	-0.0284	-0.0218	0.0000

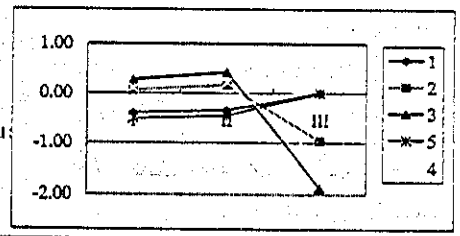
Note; The above coefficients were calculated in proportion to those tabulated in "The Technical ... for Port and Harbor Facilities" by The Japanese Port and Harbor Association.

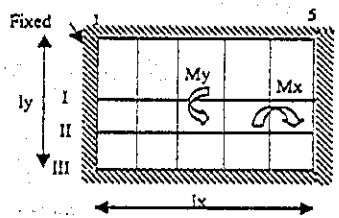
2) Moment

< 1 : Mx = Kx·q·lx^2 (around y axis)
: My = Ky·q·lx^2 (around x axis)

Load Type		No.	1	2	3	4	5
Uniform Load	Mx	I	-1.1476	0.1477	0.5701	0.1477	-1.1476
		II	-1.0031	0.1436	0.4737	0.1436	-1.0031
		III	0.0000	-0.0796	-0.1303	-0.0796	0.0000
	My	I	-0.1914	0.0361	0.1174	0.0361	-0.1914
		II	-0.1675	0.0815	0.1800	0.0815	-0.1675
		III	0.0000	-0.4733	-0.7809	-0.4733	0.0000
Trangulation Load	Mx	I	-1.2815	0.0668	0.7947	0.3461	-1.9197
		II	-1.0851	0.0634	0.6607	0.3363	-1.7130
		III	0.0000	-0.0804	-0.1837	-0.1386	0.0000
	My	I	-0.2134	0.0259	0.1633	0.0719	-0.3206
		II	-0.1816	0.0838	0.2530	0.1429	-0.2849
		III	0.0000	-0.4860	-1.0876	-0.8350	0.0000







Load Type		No.	1	2	3	4	5
Uniform + Trainangulation	Mx	I	-2.43	0.21	1.36	0.49	-3.07
		II	-2.09	0.21	1.13	0.48	-2.72
		III	0.00	-0.16	-0.31	-0.22	0.00
	My	I	-0.40	0.06	0.28	0.11	-0.51
		II	-0.35	0.17	0.43	0.22	-0.45
		III	0.00	-0.96	-1.87	-1.31	0.00

Required Re-bar a = 1800 kg/cm2 Re-bar covering depth 10 cm

			1	2	3	4	5
Distance(m)			0	0.95	1.9	2.85	3.8
Thickness (cm)			100	100	100	100	100
Required Re-bar for	Mx	I	-1.7	0.2	1.0	0.3	-2.2
		II	-1.5	0.1	0.8	0.3	-1.9
		III	0.0	-0.1	-0.2	-0.2	0.0
	My	I	-0.3	0.0	0.2	0.1	-0.4
		II	-0.2	0.1	0.3	0.2	-0.3
		III	0.0	-0.7	-1.3	-0.9	0.0

Fig. 3.7.5(4/9) Bending Moment of Flat Slabs

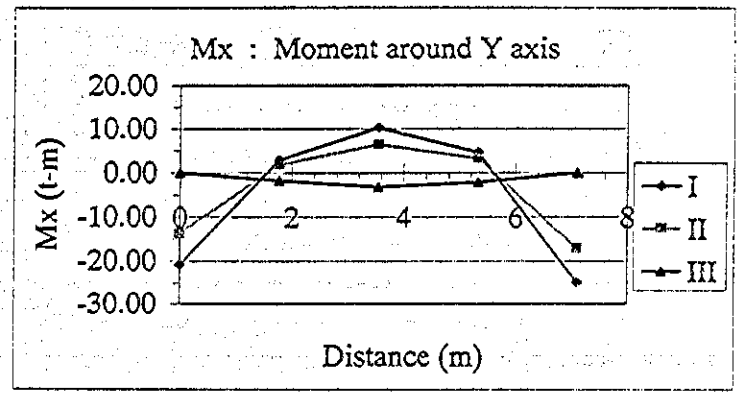
W-4 : Powerhouse Downstream Side Wall 1of 2 (7.10 x 9.00) : Seismic Condition

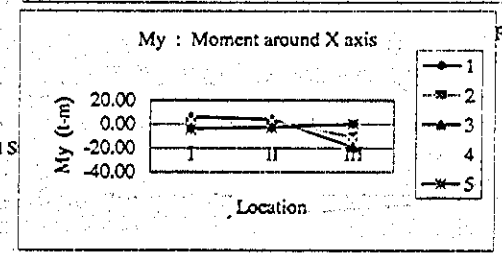
1) Moment Coefficient

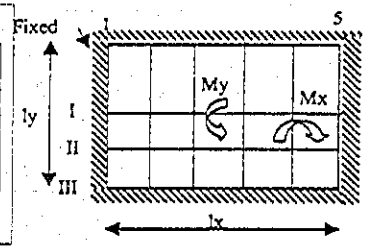
lx =	7.10 m	Uniform Load	w =	4.28 t/m2
ly =	9.00 m	Trangulation Load	q =	4.96 t/m2
=	0.789			

Load Type		No.	1	2	3	4	5
Uniform Load	Kx	I	-0.0672	0.0108	0.0301	0.0108	-0.0672
		II	-0.0453	0.0075	0.0187	0.0075	-0.0453
		III	0.0000	-0.0057	-0.0093	-0.0057	0.0000
	Ky	I	-0.0112	0.0093	0.0183	0.0093	-0.0112
		II	-0.0076	0.0067	0.0120	0.0067	-0.0076
		III	0.0000	-0.0342	-0.0557	-0.0342	0.0000
Trangulation Load	Kx	I	-0.0254	0.0019	0.0150	0.0089	-0.0418
		II	-0.0157	0.0009	0.0093	0.0066	-0.0296
		III	0.0000	-0.0021	-0.0046	-0.0036	0.0000
	Ky	I	-0.0043	0.0039	0.0092	0.0055	-0.0070
		II	-0.0025	0.0023	0.0060	0.0044	-0.0050
		III	0.0000	-0.0124	-0.0279	-0.0216	0.0000

Note: The above coefficients were calculated in proportion to those tabulated in "The Technical S
for Port and Harbor Facilities" by The Japanese Port and Harbor Association.







2) Moment

< 1 : Mx = Kx·q·lx^2 (around y axis)

: My = Ky·q·lx^2 (around x axis)

Load Type		No.	1	2	3	4	5
Uniform Load	Mx	I	-14.4934	2.3263	6.4851	2.3263	-14.4934
		II	-9.7780	1.6191	4.0332	1.6191	-9.7780
		III	0.0000	-1.2380	-2.0012	-1.2380	0.0000
	My	I	-2.4203	2.0108	3.9526	2.0108	-2.4203
		II	-1.6388	1.4551	2.5996	1.4551	-1.6388
		III	0.0000	-7.3731	-12.0156	-7.3731	0.0000
Trangulation Load	Mx	I	-6.3547	0.4806	3.7577	2.2325	-10.4414
		II	-3.9189	0.2345	2.3370	1.6380	-7.4127
		III	0.0000	-0.5173	-1.1596	-0.9001	0.0000
	My	I	-1.0668	0.9640	2.3009	1.3663	-1.7380
		II	-0.6151	0.5767	1.4958	1.1057	-1.2418
		III	0.0000	-3.1115	-6.9748	-5.4118	0.0000

Load Type		No.	1	2	3	4	5
Uniform + Traiangulation Loads	Mx	I	-20.85	2.81	10.24	4.56	-24.93
		II	-13.70	1.85	6.37	3.26	-17.19
		III	0.00	-1.76	-3.16	-2.14	0.00
	My	I	-3.49	2.97	6.25	3.38	-4.16
		II	-2.25	2.03	4.10	2.56	-2.88
		III	0.00	-10.48	-18.99	-12.78	0.00

Required Re-bar a = 2700 kg/cm2 Re-bar covering depth 10 cm

			1	2	3	4	5
Distance(m)			0	1.78	3.55	5.33	7.10
Thickness (cm)			100	100	100	100	100
Required Re-bar for	Mx	I	-9.8	1.3	4.8	2.1	-11.7
		II	-6.4	0.9	3.0	1.5	-8.1
		III	0.0	-0.8	-1.5	-1.0	0.0
	My	I	-1.6	1.4	2.9	1.6	-2.0
		II	-1.1	1.0	1.9	1.2	-1.4
		III	0.0	-4.9	-8.9	-6.0	0.0

Fig. 3.7.5(5/9) Bending Moment of Flat Slabs

W-5 : Powerhouse Upstream Side Wall (12.60 x 9.00) : Normal Condition

1) Moment Coefficient

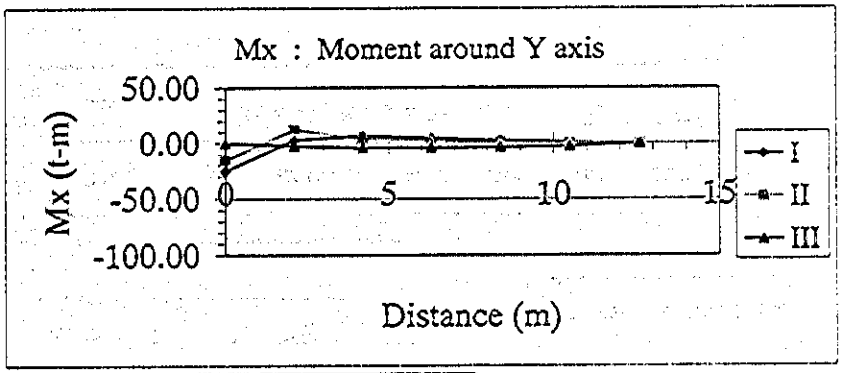
lx = 12.60 m Uniform Load w = 1.26 t/m2
ly = 9.00 m Trangulation Load q = 5.77 t/m2
= 1.400

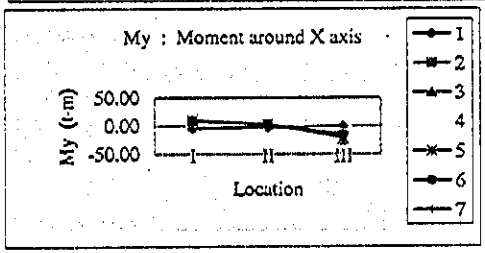
Load Type		No.	1	2	3	4	5	6	7
Uniform Load	Kx	I	-0.0568	0.0029	0.0137	0.0119	0.0088	0.0059	0.0000
		II	-0.0344	0.0019	0.0068	0.0049	0.0028	0.0012	0.0000
		III	0.0000	-0.0053	-0.0106	-0.0130	-0.0137	-0.0139	0.0000
	Ky	I	-0.0095	0.0121	0.0289	0.0372	0.0403	0.0415	0.0434
		II	-0.0057	0.0047	0.0096	0.0107	0.0106	0.0105	0.0106
		III	0.0000	-0.0319	-0.0636	-0.0778	-0.0821	-0.0835	-0.0834
Trangulation Load	Kx	I	-0.0408	0.0053	0.0113	0.0076	0.0033	0.0003	0.0000
		II	-0.0260	0.0262	0.0061	0.0035	0.0011	-0.0003	0.0000
		III	0.0000	-0.0040	-0.0066	-0.0063	-0.0047	-0.0029	0.0000
	Ky	I	-0.0068	0.0082	0.0168	0.0175	0.0140	0.0097	0.0071
		II	-0.0043	0.0040	0.0064	0.0055	0.0037	0.0020	0.0007
		III	0.0000	-0.0237	-0.0392	-0.0376	-0.0282	-0.0174	-0.0067

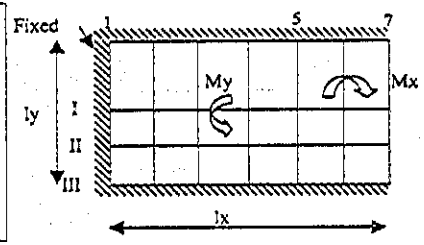
Note; The above coefficients were calculated in proportion to those tabulated in "The Technical Standards for Port and Harbor Facilities" by The Japanese Port and Harbor Association.

2) Moment > 1 : Mx = Kx·q·ly^2 (around y axis)
: My = Ky·q·ly^2 (around x axis)

Load Type		No.	1	2	3	4	5	6	7
Uniform Load	Mx	I	-5.7929	0.2939	1.3941	1.2145	0.9002	0.6042	0.0000
		II	-3.5109	0.1898	0.6981	0.5042	0.2878	0.1245	0.0000
		III	0.0000	-0.5430	-1.0859	-1.3247	-1.3962	-1.4186	0.0000
	My	I	-0.9696	1.2329	2.9455	3.7966	4.1171	4.2334	4.4335
		II	-0.5817	0.4838	0.9798	1.0941	1.0859	1.0696	1.0798
		III	0.0000	-3.2578	-6.4931	-7.9382	-8.3832	-8.5220	-8.5077
Trangulation Load	Mx	I	-19.0874	2.4771	5.2906	3.5427	1.5236	0.1215	0.0000
		II	-12.1516	12.2638	2.8603	1.6358	0.5141	-0.1496	0.0000
		III	0.0000	-1.8508	-3.0659	-2.9257	-2.1966	-1.3460	0.0000
	My	I	-3.1781	3.8324	7.8612	8.1883	6.5432	4.5241	3.2996
		II	-2.0190	1.8788	3.0005	2.5799	1.7480	0.9254	0.3178
		III	0.0000	-11.0954	-18.3303	-17.5918	-13.1798	-8.1322	-3.1407







Load Type		No.	1	2	3	4	5	6	7
Uniform + Traiangulation Loads	Mx	I	-24.88	2.77	6.68	4.76	2.42	0.73	0.00
		II	-15.66	12.45	3.56	2.14	0.80	-0.03	0.00
		III	0.00	-2.39	-4.15	-4.25	-3.59	-2.76	0.00
	My	I	-4.15	5.07	10.81	11.98	10.66	8.76	7.73
		II	-2.60	2.36	3.98	3.67	2.83	1.99	1.40
		III	0.00	-14.35	-24.82	-25.53	-21.56	-16.65	-11.65

Required Re-bar a = 1800 kg/cm2 Re-bar covering depth 10 cm

			1	2	3	4	5	6	7
Distance(m)			0	2.10	4.20	6.30	8.40	10.50	12.60
Thickness (cm)			200	200	200	200	200	200	200
Required Re-bar for	Mx	I	-8.3	0.9	2.2	1.6	0.8	0.2	0.0
		II	-5.2	4.2	1.2	0.7	0.3	0.0	0.0
		III	0.0	-0.8	-1.4	-1.4	-1.2	-0.9	0.0
	My	I	-1.4	1.7	3.6	4.0	3.6	2.9	2.6
		II	-0.9	0.8	1.3	1.2	0.9	0.7	0.5
		III	0.0	-4.8	-8.3	-8.5	-7.2	-5.6	-3.9

Fig. 3.7.5(6/9) Bending Moment of Flat Slabs

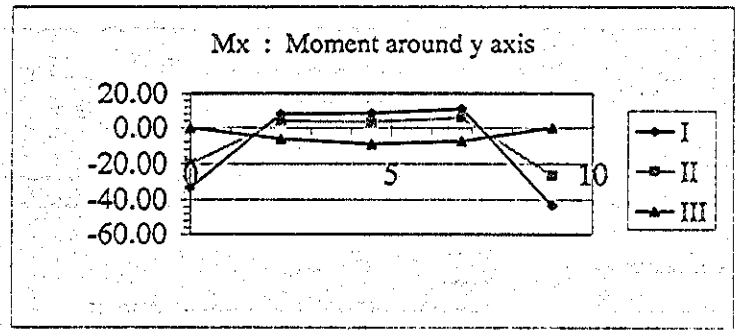
B-1 : Powerhouse Bottom Slab at EL.84.9 (9.00 x 5.00) : Flood Condition

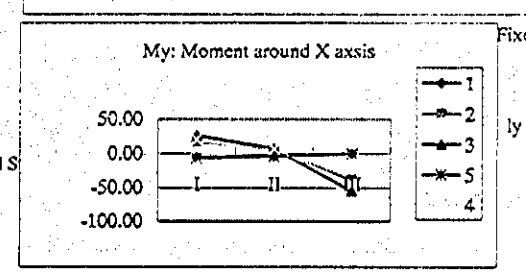
1) Moment Coefficient

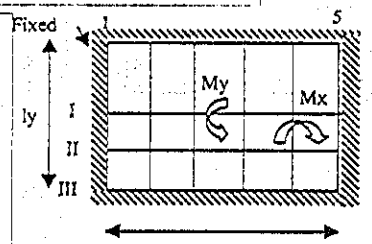
lx =	9.00 m	Uniform Load	w =	20.9	t/m2
ly =	5.00 m	Trangulation Load	q =	12.1	t/m2
=	1.800				

Load Type		No.	1	2	3	4	5
Uniform Load	Kx	I	-0.0571	0.0139	0.0123	0.0139	-0.0571
		II	-0.0346	0.0070	0.0049	0.0070	-0.0346
		III	0.0000	-0.0104	-0.0136	-0.0104	0.0000
	Ky	I	-0.0095	0.0282	0.0393	0.0282	-0.0095
		II	-0.0058	0.0095	0.0112	0.0095	-0.0058
		III	0.0000	-0.0623	-0.0810	-0.0623	0.0000
Tranguiation Load	Kx	I	-0.0125	0.0018	0.0062	0.0121	-0.0445
		II	-0.0066	0.0006	0.0025	0.0064	-0.0280
		III	0.0000	-0.0031	-0.0067	-0.0073	0.0000
	Ky	I	-0.0021	0.0091	0.0196	0.0191	-0.0075
		II	-0.0011	0.0024	0.0056	0.0071	-0.0046
		III	0.0000	-0.0186	-0.0405	-0.0437	0.0000

Note; The above coefficients were calculated in proportion to those tabulated in "The Technical S
for Port and Harbor Facilities" by The Japanese Port and Harbor Association.







2) Moment > 1 : Mx = Kx·q·ly^2 (around y axis)

: My = Ky·q·ly^2 (around x axis)

Load Type		No.	1	2	3	4	5
Uniform Load	Mx	I	-29.82	7.26	6.45	7.26	-29.82
		II	-18.07	3.68	2.57	3.68	-18.07
		III	0.00	-5.43	-7.09	-5.43	0.00
	My	I	-4.96	14.71	20.51	14.71	-4.96
		II	-3.03	4.97	5.83	4.97	-3.03
		III	0.00	-32.53	-42.30	-32.53	0.00
Tranguiation Load	Mx	I	-3.78	0.54	1.86	3.66	-13.47
		II	-2.00	0.18	0.74	1.95	-8.46
		III	0.00	-0.94	-2.04	-2.20	0.00
	My	I	-0.62	2.75	5.92	5.77	-2.26
		II	-0.33	0.73	1.69	2.15	-1.40
		III	0.00	-5.61	-12.26	-13.22	0.00

Load Type		No.	1	2	3	4	5
Uniform + Tralangulation Loads	Mx	I	-33.60	7.81	8.31	10.92	-43.29
		II	-20.07	3.86	3.31	5.63	-26.53
		III	0.00	-6.38	-9.12	-7.64	0.00
	My	I	-5.59	17.46	26.44	20.49	-7.22
		II	-3.36	5.71	7.52	7.12	-4.43
		III	0.00	-38.15	-54.56	-45.75	0.00

Required Re-bar a = 1800 kg/cm2 Re-bar covering depth 15

			1	2	3	4	5
Distance(m)			0	2.25	4.5	6.75	9
Thickness (cm)			210	210	210	210	210
Required Re-bar for	Mx	I	-10.9	2.5	2.7	3.6	-14.1
		II	-6.5	1.3	1.1	1.8	-8.6
		III	0.0	-2.1	-3.0	-2.5	0.0
	My	I	-1.8	5.7	8.6	6.7	-2.4
		II	-1.1	1.9	2.4	2.3	-1.4
		III	0.0	-12.4	-17.8	-14.9	0.0

Fig. 3.7.5(7/9) Bending Moment of Flat Slabs

B-2 : Powerhouse Bottom Slab at EL.84.9 (9.00 x 15.00) : Flood Condition

1) Moment Coefficient

lx =	9.00 m	Uniform Load	w =	18.5 t/m2
ly =	15.00 m	Trangulation Load	q =	8.0 t/m2
=	0.600			

Load Type		No.	1	2	3	4	5
Uniform Load	Kx	I	-0.0777	0.0110	0.0371	0.0110	-0.0777
		II	-0.0592	0.0091	0.0265	0.0091	-0.0592
		III	0.0000	-0.0058	-0.0095	-0.0058	0.0000
	Ky	I	-0.0130	0.0058	0.0135	0.0058	-0.0130
		II	-0.0099	0.0068	0.0133	0.0068	-0.0099
		III	0.0000	-0.0345	-0.0568	-0.0345	0.0000
Trangulation Load	Kx	I	-0.0306	0.0019	0.0185	0.0092	-0.0471
		II	-0.0219	0.0013	0.0132	0.0078	-0.0373
		III	0.0000	-0.0021	-0.0048	-0.0037	0.0000
	Ky	I	-0.0051	0.0023	0.0067	0.0036	-0.0079
		II	-0.0036	0.0025	0.0067	0.0043	-0.0062
		III	0.0000	-0.0127	-0.0284	-0.0219	0.0000

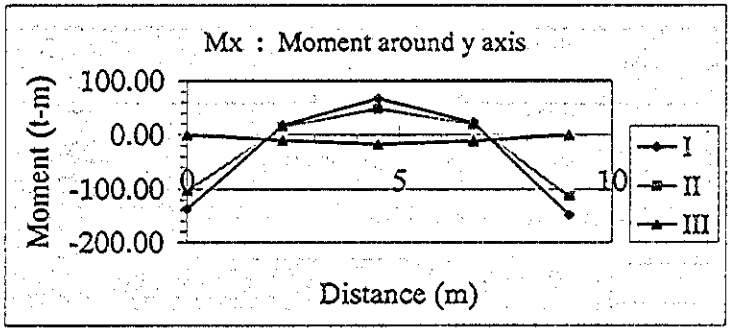
Note: The above coefficients were calculated in proportion to those tabulated in "The Technical S for Port and Harbor Facilities" by The Japanese Port and Harbor Association.

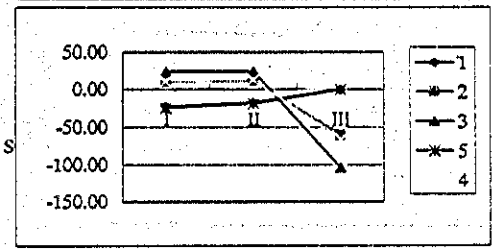
2) Moment

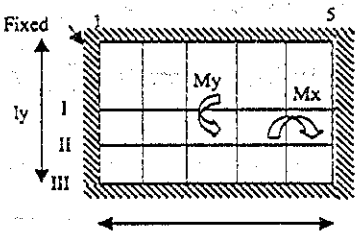
< 1 : Mx = Kx·q·lx^2 (around y axis)

: My = Ky·q·lx^2 (around x axis)

Load Type		No.	1	2	3	4	5
Uniform Load	Mx	I	-116.4634	16.4835	55.6543	16.4835	-116.4634
		II	-88.7412	13.6663	39.6803	13.6663	-88.7412
		III	0.0000	-8.6913	-14.1758	-8.6913	0.0000
	My	I	-19.4206	8.7512	20.1698	8.7512	-19.4206
		II	-14.8651	10.2497	19.9900	10.2497	-14.8651
		III	0.0000	-51.6983	-85.1148	-51.6983	0.0000
Trangulation Load	Mx	I	-19.8158	1.2571	12.0139	5.9357	-30.5467
		II	-14.2042	0.8424	8.5795	5.0285	-24.1704
		III	0.0000	-1.3608	-3.0845	-2.3717	0.0000
	My	I	-3.3048	1.4645	4.3546	2.3198	-5.0933
		II	-2.3069	1.5941	4.3286	2.7994	-4.0306
		III	0.0000	-8.2037	-18.4162	-14.1653	0.0000







Load Type		No.	1	2	3	4	5
Uniform + Traiangulation Loads	Mx	I	-136.28	17.74	67.67	22.42	-147.01
		II	-102.95	14.51	48.26	18.69	-112.91
		III	0.00	-10.05	-17.26	-11.06	0.00
	My	I	-22.73	10.22	24.52	11.07	-24.51
		II	-17.17	11.84	24.32	13.05	-18.90
		III	0.00	-59.90	-103.53	-65.86	0.00

Required Re-bar a = 1800 kg/cm2 Re-bar covering depth 15 cm

			1	2	3	4	5
Distance(m)			0	2.25	4.5	6.75	9
Thickness (cm)			210	210	210	210	210
Required Re-bar for	Mx	I	-44.4	5.8	22.0	7.3	-47.9
		II	-33.5	4.7	15.7	6.1	-36.8
		III	0.0	-3.3	-5.6	-3.6	0.0
	My	I	-7.4	3.3	8.0	3.6	-8.0
		II	-5.6	3.9	7.9	4.2	-6.2
		III	0.0	-19.5	-33.7	-21.4	0.0

Fig. 3.7.5(8/9) Bending Moment of Flat Slabs

F-1 : Powerhouse Control Room at EL.97.5 (5.00 x 9.00) : Normal Condition

1) Moment Coefficient

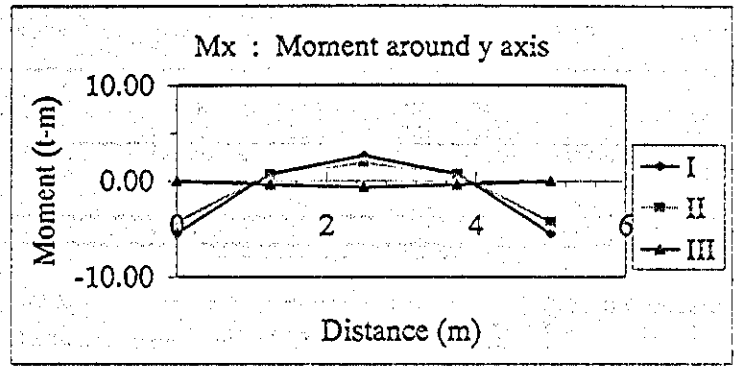
lx = 5.00 m Uniform Load w = 2.75 t/m2

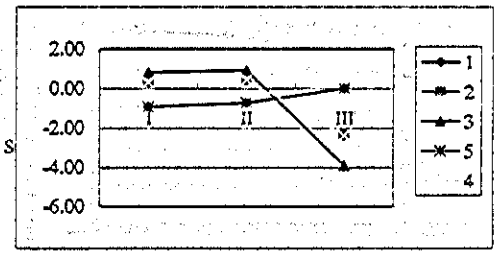
ly = 9.00 m Trangulation Load q = 0 t/m2

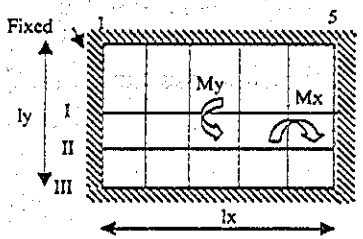
= 0.556

Load Type		No.	1	2	3	4	5
Uniform Load	Kx	I	-0.0800	0.0110	0.0387	0.0110	-0.0800
		II	-0.0626	0.0095	0.0284	0.0095	-0.0626
		III	0.0000	-0.0058	-0.0095	-0.0058	0.0000
	Ky	I	-0.0133	0.0049	0.0121	0.0049	-0.0133
		II	-0.0105	0.0068	0.0136	0.0068	-0.0105
		III	0.0000	-0.0345	-0.0569	-0.0345	0.0000
Trangulation Load	Kx	I	-0.0317	0.0019	0.0193	0.0092	-0.0483
		II	-0.0235	0.0014	0.0142	0.0080	-0.0392
		III	0.0000	-0.0021	-0.0048	-0.0037	0.0000
	Ky	I	-0.0053	0.0018	0.0060	0.0031	-0.0081
		II	-0.0038	0.0025	0.0068	0.0043	-0.0065
		III	0.0000	-0.0127	-0.0285	-0.0219	0.0000

Note: The above coefficients were calculated in proportion to those tabulated in "The Technical S
for Port and Harbor Facilities" by The Japanese Port and Harbor Association.







2) Moment < 1 : Mx = Kx·q·lx^2 (around y axis)

: My = Ky·q·lx^2 (around x axis)

Load Type		No.	1	2	3	4	5
Uniform Load	Mx	I	-5.4985	0.7563	2.6622	0.7563	-5.4985
		II	-4.3060	0.6539	1.9525	0.6539	-4.3060
		III	0.0000	-0.3988	-0.6516	-0.3988	0.0000
	My	I	-0.9167	0.3392	0.8349	0.3392	-0.9167
		II	-0.7211	0.4690	0.9342	0.4690	-0.7211
		III	0.0000	-2.3719	-3.9111	-2.3719	0.0000
Trangulation Load	Mx	I	0.0000	0.0000	0.0000	0.0000	0.0000
		II	0.0000	0.0000	0.0000	0.0000	0.0000
		III	0.0000	0.0000	0.0000	0.0000	0.0000
	My	I	0.0000	0.0000	0.0000	0.0000	0.0000
		II	0.0000	0.0000	0.0000	0.0000	0.0000
		III	0.0000	0.0000	0.0000	0.0000	0.0000

Load Type		No.	1	2	3	4	5
Uniform + Trainangulation Loads	Mx	I	-5.50	0.76	2.66	0.76	-5.50
		II	-4.31	0.65	1.95	0.65	-4.31
		III	0.00	-0.40	-0.65	-0.40	0.00
	My	I	-0.92	0.34	0.83	0.34	-0.92
		II	-0.72	0.47	0.93	0.47	-0.72
		III	0.00	-2.37	-3.91	-2.37	0.00

Required Re-bar a = 1800 kg/cm2 Re-bar covering depth 10 cm

			1	2	3	4	5
Distance(m)			0	1.25	2.5	3.75	5
Thickness (cm)			70	70	70	70	70
Required Re-bar for	Mx	I	-5.8	0.8	2.8	0.8	-5.8
		II	-4.6	0.7	2.1	0.7	-4.6
		III	0.0	-0.4	-0.7	-0.4	0.0
	My	I	-1.0	0.4	0.9	0.4	-1.0
		II	-0.8	0.5	1.0	0.5	-0.8
		III	0.0	-2.5	-4.1	-2.5	0.0

Fig. 3.7.5(9/9) Bending Moment of Flat Slabs

F-2 : Powerhouse Cable Room at EL.93.0 (5.00 x 9.00) : Normal Condition

1) Moment Coefficient

lx =	5.00 m	Uniform Load	w =	3.5 t/m2
ly =	9.50 m	Trangulation Load	q =	0 t/m2
=	0.526			

Load Type		No.	1	2	3	4	5
Uniform Load	Kx	I	-0.0815	0.0110	0.0398	0.0110	-0.0815
		II	-0.0649	0.0098	0.0297	0.0098	-0.0649
		III	0.0000	-0.0058	-0.0095	-0.0058	0.0000
	Ky	I	-0.0136	0.0043	0.0113	0.0043	-0.0136
		II	-0.0109	0.0068	0.0138	0.0068	-0.0109
		III	0.0000	-0.0345	-0.0569	-0.0345	0.0000
Trangulation Load	Kx	I	-0.0324	0.0019	0.0198	0.0092	-0.0490
		II	-0.0245	0.0014	0.0148	0.0082	-0.0404
		III	0.0000	-0.0021	-0.0048	-0.0037	0.0000
	Ky	I	-0.0054	0.0016	0.0056	0.0028	-0.0082
		II	-0.0040	0.0025	0.0069	0.0042	-0.0067
		III	0.0000	-0.0127	-0.0285	-0.0219	0.0000

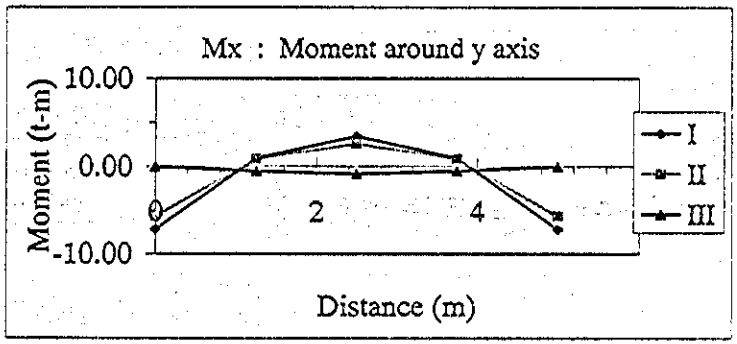
Note; The above coefficients were calculated in proportion to those tabulated in "The Technical Stand for Port and Harbor Facilities" by The Japanese Port and Harbor Association.

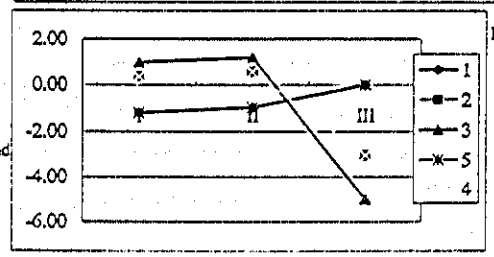
2) Moment

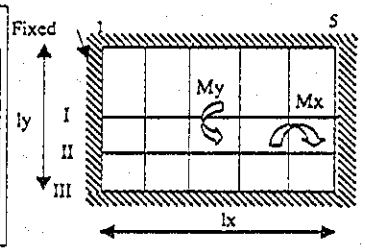
< 1 : Mx = Kx·q·lx^2 (around y axis)

: My = Ky·q·lx^2 (around x axis)

Load Type		No.	1	2	3	4	5
Uniform Load	Mx	I	-7.1280	0.9625	3.4793	0.9625	-7.1280
		II	-5.6769	0.8547	2.5955	0.8547	-5.6769
		III	0.0000	-0.5075	-0.8303	-0.5075	0.0000
	My	I	-1.1882	0.3795	0.9869	0.3795	-1.1882
		II	-0.9505	0.5959	1.2034	0.5959	-0.9505
		III	0.0000	-3.0188	-4.9829	-3.0188	0.0000
Trangulation Load	Mx	I	0.0000	0.0000	0.0000	0.0000	0.0000
		II	0.0000	0.0000	0.0000	0.0000	0.0000
		III	0.0000	0.0000	0.0000	0.0000	0.0000
	My	I	0.0000	0.0000	0.0000	0.0000	0.0000
		II	0.0000	0.0000	0.0000	0.0000	0.0000
		III	0.0000	0.0000	0.0000	0.0000	0.0000



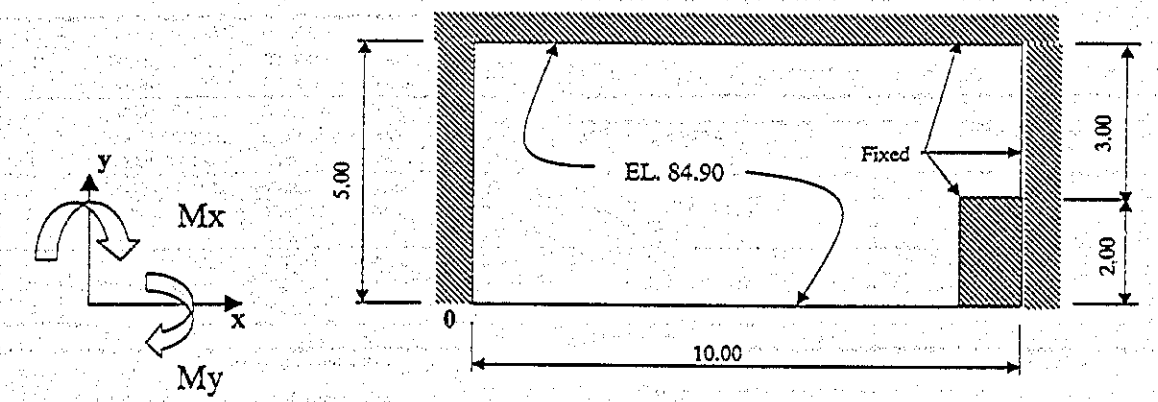


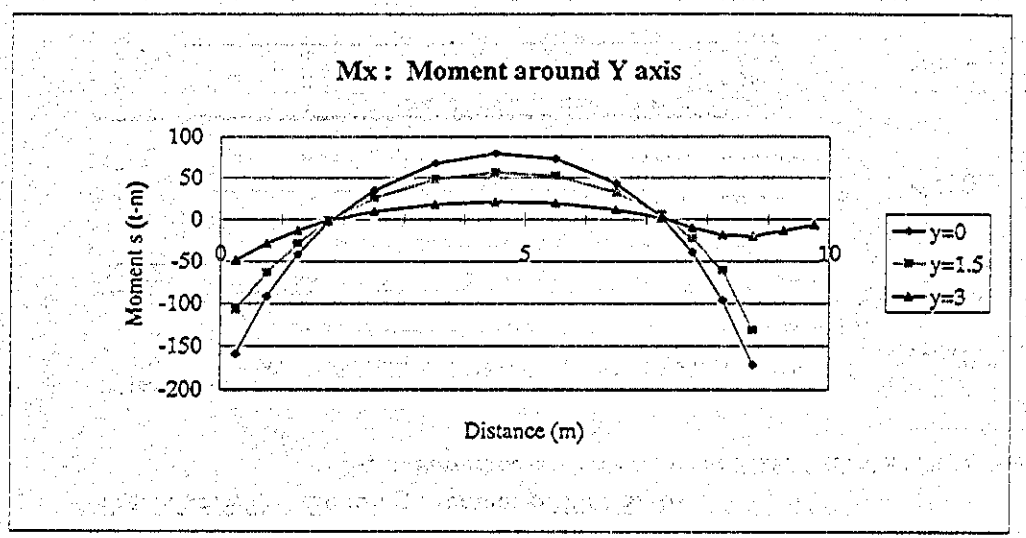


Load Type		No.	1	2	3	4	5
Uniform + Traiangulation Loads	Mx	I	-7.13	0.96	3.48	0.96	-7.13
		II	-5.68	0.85	2.60	0.85	-5.68
		III	0.00	-0.51	-0.83	-0.51	0.00
	My	I	-1.19	0.38	0.99	0.38	-1.19
		II	-0.95	0.60	1.20	0.60	-0.95
		III	0.00	-3.02	-4.98	-3.02	0.00

Required Re-bar a = 1800 kg/cm2 Re-bar covering depth 10 cm

			1	2	3	4	5
Distance(m)			0	1.25	2.5	3.75	5
Thickness (cm)			100	100	100	100	100
Required Re-bar for	Mx	I	-5.0	0.7	2.5	0.7	-5.0
		II	-4.0	0.6	1.8	0.6	-4.0
		III	0.0	-0.4	-0.6	-0.4	0.0
	My	I	-0.8	0.3	0.7	0.3	-0.8
		II	-0.7	0.4	0.8	0.4	-0.7
		III	0.0	-2.1	-3.5	-2.1	0.0





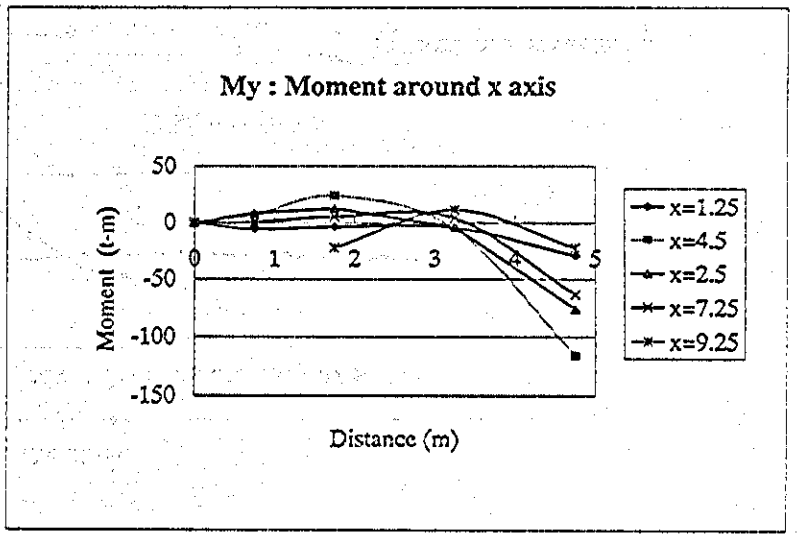


Fig. 3.7.6 Sectional Forces of Bottom Slab C at EL. 84.90 : BS - 1 in Normal Condition

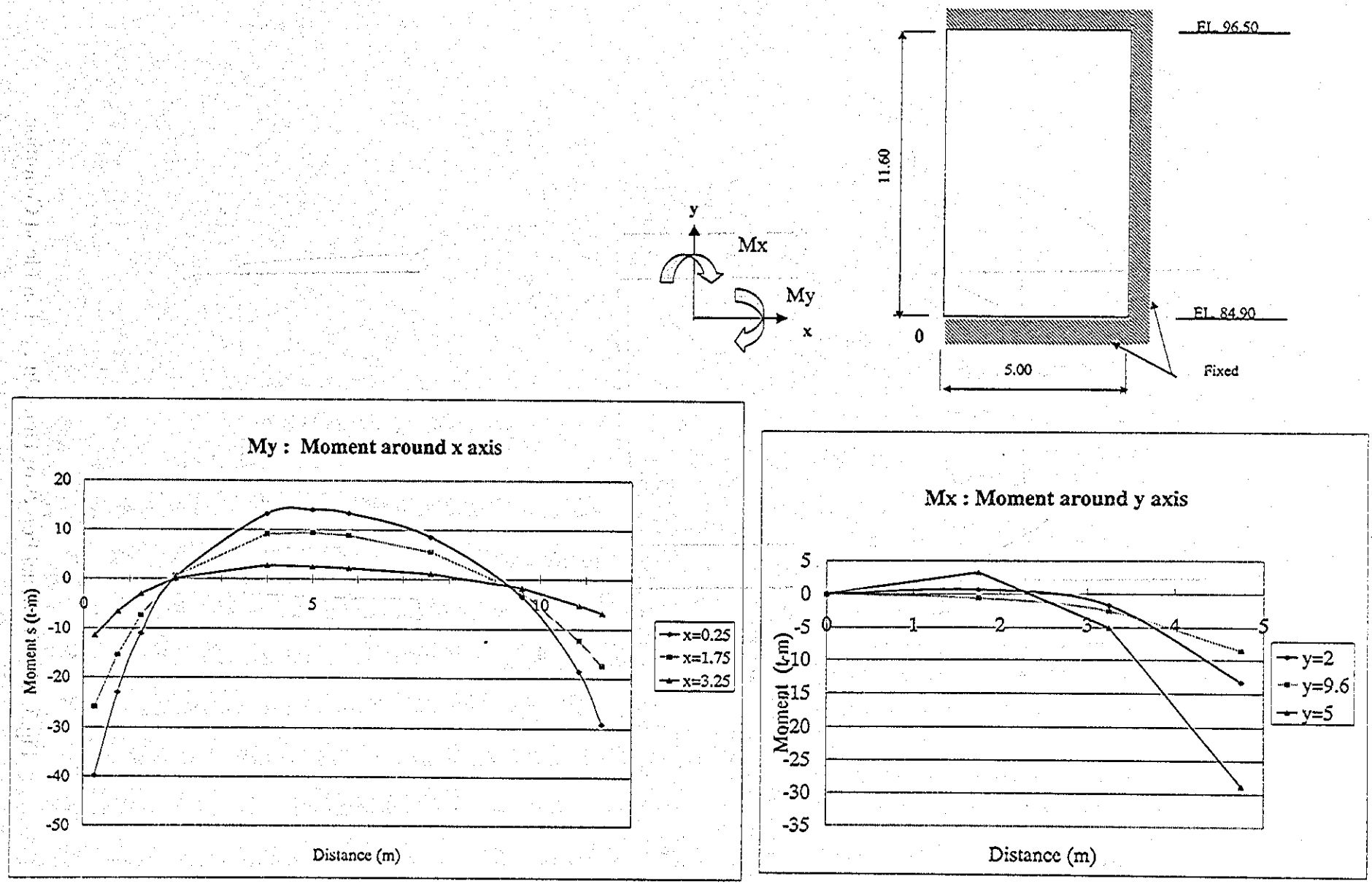
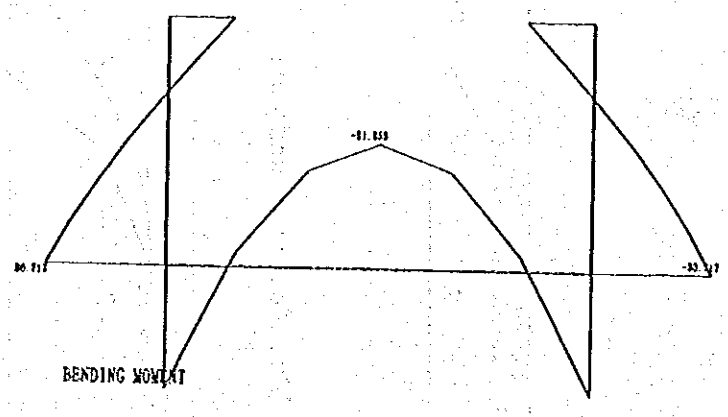
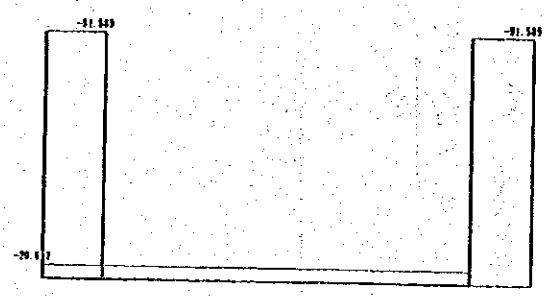


Fig. 3.7.7 Sectional Forces of Outer Wall on the Spillway Side : WS - 1 in Normal Condition





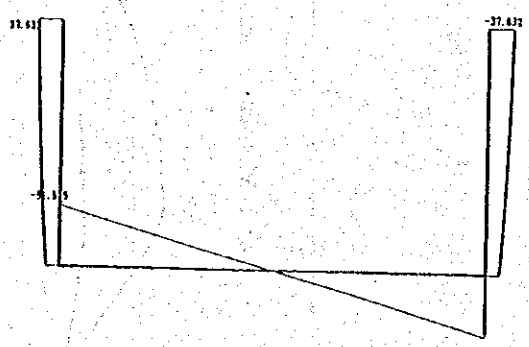
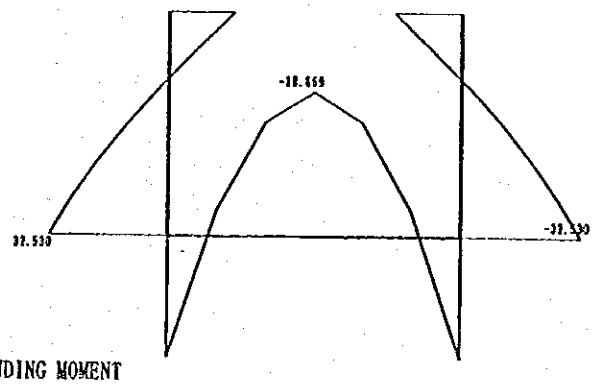
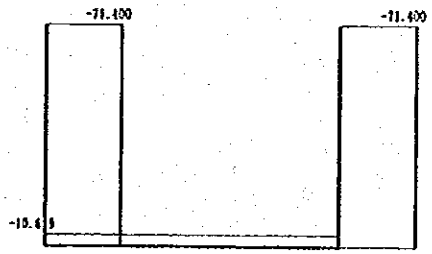


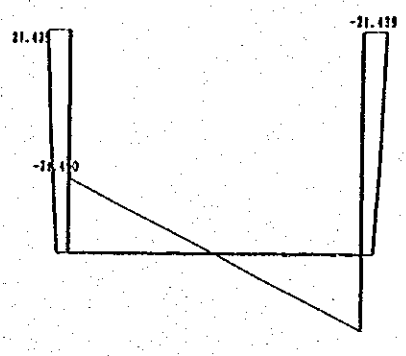
Fig.3.7.8 Sectional Forces of Draft Pit : S-1 in Flood Condition



BENDING MOMENT



NORMAL FORCE



SHEAR FORCE

Fig.3.7.9 Sectional Forces of Tailrace : S-2 in Flood Condition

3.8 Design of Tailrace Structures

3.8.1 Design of Tailrace

(1) General

The tailrace is located in the spillway wing wall. The layout, configuration and dimensions are shown in Figs.3.1.4～3.1.6. A typical section of tailrace is schematically shown in Fig. 3.8.1.

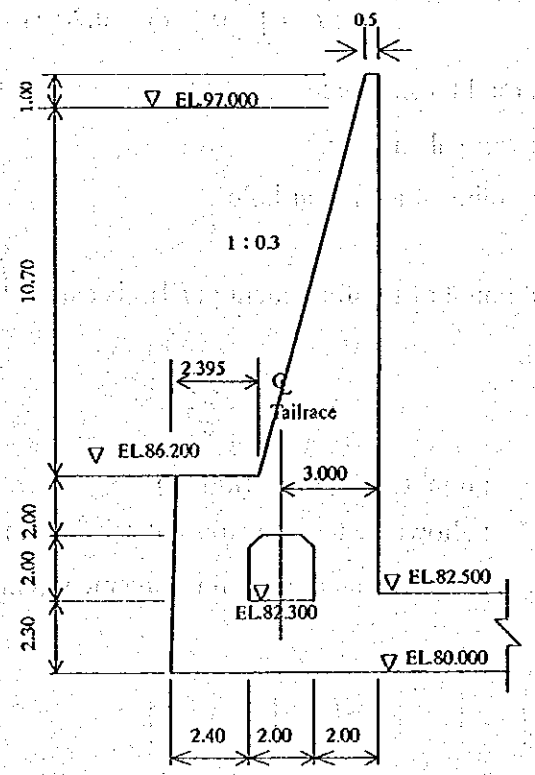


Fig. 3.8.1 Typical Section of Tailrace

(2) Design method

Concentration stress generates around the open portion in the wing wall due to internal stress, therefore reinforcement is required in case generating stress exceeds allowable tensile strength of concrete. Required amount of reinforcement is calculated using the following theory.

In case that the uniform compressive stress is acting in the y-axis direction on an infinite plate having a circular hole as shown in Fig.3.8.2, the stress around

circular hole is expressed as follows:

on X-axis : $\sigma_x = \frac{1}{2} \cdot \sigma \left(\frac{r^2}{y^2} - 3\frac{r^4}{y^4} \right)$

on Y-axis : $\sigma_y = \frac{1}{2} \cdot \sigma \left(2 + \frac{r^2}{x^2} + 3\frac{r^4}{x^4} \right)$

As shown in Fig.3.8.2, the tensile stress generates in the upper and lower portions within distance of $\pm\sqrt{3}$ r.

The total tensile force around a circular hole is calculated approximately by the following formula.

$$T = \int_r^{\sqrt{3}} \sigma_x \cdot dy = 0.2\sigma \cdot r$$

Where, T : total tensile force

σ : vertical stress

r : radius of a circular hole

The required amount of reinforcement per 1m is calculated as follow:

$$A_s = \frac{T}{\sigma_{sa}}$$

Where, T : total tensile force (kgf/m)

σ_{sa} : allowable tension stress of reinforcement (kgf/cm^2)

A_s : required amount of reinforcement (cm^2/m)

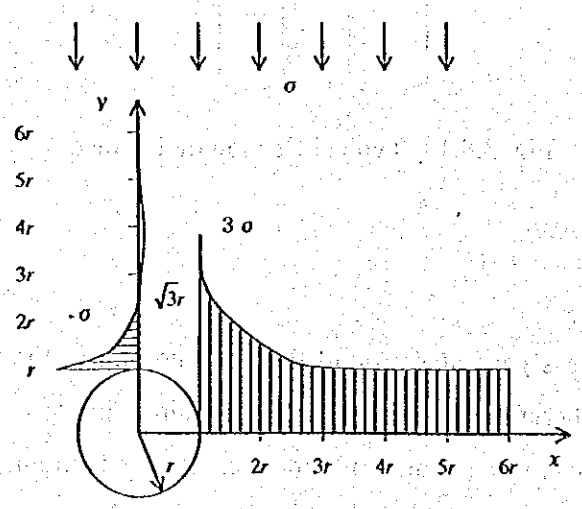


Fig. 3.8.2 Stress Distribution of an Infinite Plate

(3) Calculation results

Calculation results are given in the following table and arrangement of reinforcement is shown in Fig. 3.8.3.

σ (kgf/cm²)	r :radius (cm)	T (kgf/m)	As (cm²/m)	Amount of Reinforcement
3.875	100	-7,750 59.8	4.306	D16@0.2 m c/c 19ϕ@0.2 m c/c (10.05cm²/m)

Notes : Unit weight of reinforced concrete: 2.5 tf/m³

Allowable tensile strength of reinforcement (SD 30) : σ_{sa} = 1,800 kgf/cm²

Top elevation of spillway wing wall: EL 98.000 m.

Invert elevation of tailrace: EL 85.300 m

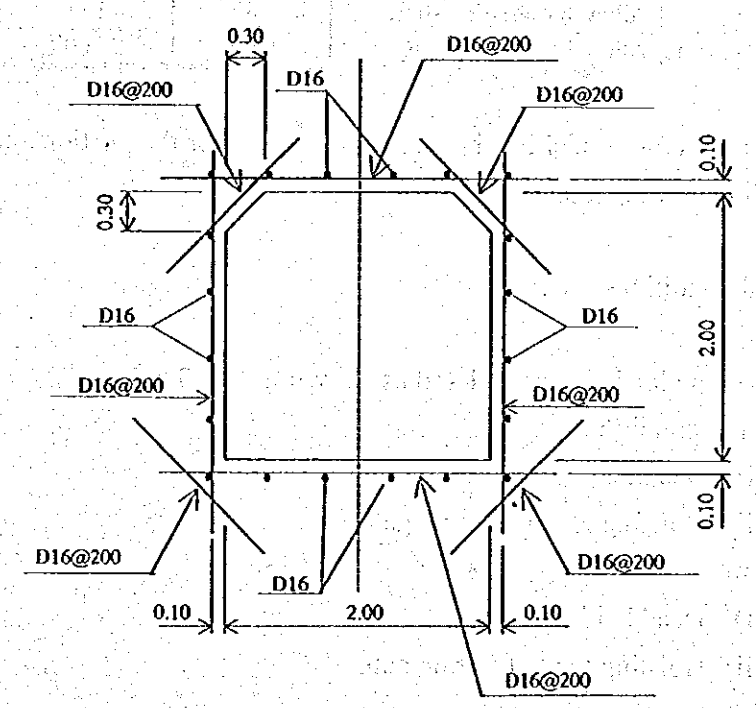


Fig. 3.8.3 Reinforcement Arrangement of Typical Section of Tailrace

3.8.2 Design of Tailrace Gate Tower

(1) General

The tailrace gate tower is located about 9.0m upstream of the exit of the tailrace. The dimensions of the tailrace gate tower are shown in Figs.3.1.4~3.1.7. Typical section of the tailrace gate tower is shown in Fig. 3.8.4.

(2) Design parameters and structural model

Unit weight of reinforced concrete	2.5 tf/m^3
Weight of a slide gate	about 1.0 ton
Horizontal seismic coefficient (K_h)	0.16

Allowable Strengths of Concrete and Reinforcement

	Item	Normal (kgf/cm^2)	Earthquake (kgf/cm^2)	Remarks
Concrete (K-225 class)	Compressive strength, σ_{tk} (28th day)	225		
	Allowable bending compressive stress	75*1	112*2	*1 $\sigma ca = \sigma_{tk}/3$ *2 $\sigma cy = 1.5\ \sigma ca$
	Allowable shearing stress	8	12*3	*3 $\tau cy = 1.5\ \tau ca$
Reinforcement	Allowable tensile stress	1,800	2,700*4	*4 $\sigma sy = 1.5\ \sigma sa$

The structural model for the analysis is made for the portion above EL 90.000 m as shown in Fig. 3.8.5

(3) Load conditions

Following loads are considered as shown in Fig. 3.8.5.

Normal condition

(i) Dead load

(ii) Hoisting load of a slide gate

Seismic condition

(i) Dead load

(ii) Hoisting load of a slide gate

(iii) Seismic force

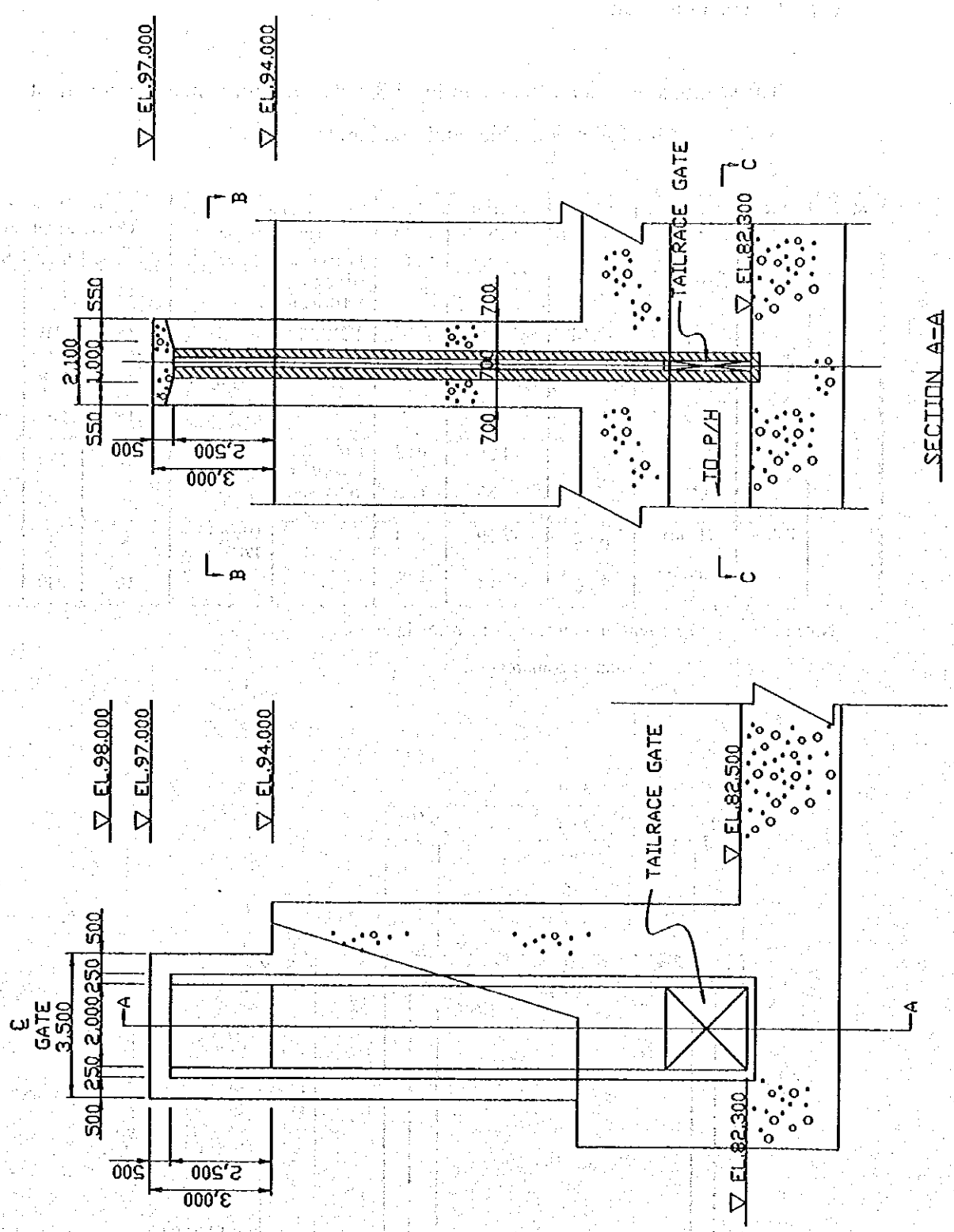


Fig. 3.8.4 Profile and Section of Tailrace Gate Tower

(4) Calculation results

The sectional forces are shown in Fig.3.8.6. The reinforcement arrangement is shown in the following tables and Fig. 3.8.7.

Mem.	Sec.	Load case	Moment *1 (tf-m)	Normal Force*2 (tf)	Shear Force (tf)	Amount of Reinforcement		Stress of Steel and Concrete (kgf/cm²)		
						Outer side	Inner side	Concrete	Steel	Shear
Top	Side	Normal	-1.54	-0.83	4.25	D16@0.3 m 19D@0.2 m (4.223cm²/m)	-	14	880	1.0
		Earthquake	-2.44	-1.43	4.85		-	18	1110	1.0
	Center	Normal	2.03	-0.83	0.50	-	D16@0.3 m 19D@0.2 m (4.223cm²/m)	15	960	0.1
		Earthquake	2.03	-0.83	-1.10	-		15	960	0.2
Side	Upper	Normal	-1.54	-4.25	0.83	D16@0.3 m 19D@0.2 m (4.223cm²/m)	-	12	520	0.3
		Earthquake	-2.44	-4.85	1.43		-	16	780	0.5
	Bottom	Normal	0.76	-7.69	0.84	-	D16@0.3 m 19D@0.2 m (4.223cm²/m)	3	5	0.3
		Earthquake	2.26	-8.29	1.98	-		10	240	0.5

Note : *1 : + tension in inside - : tension in outside

*2 : + tension - : compression

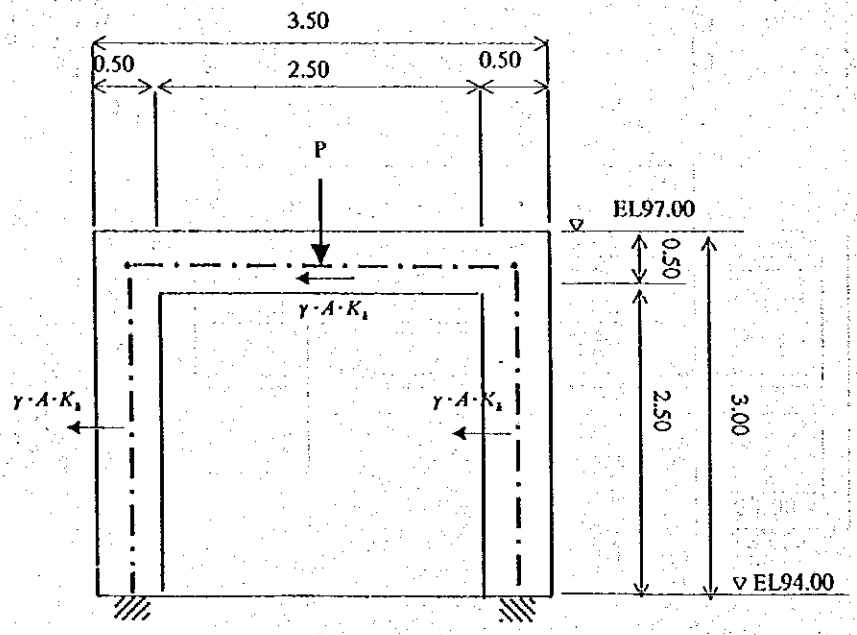
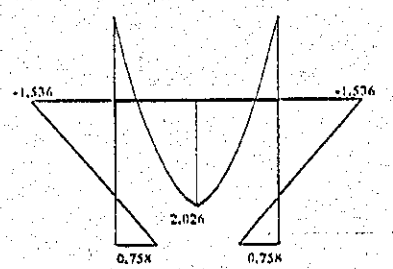
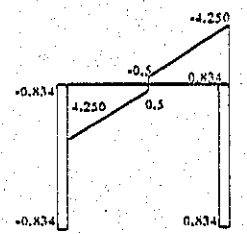
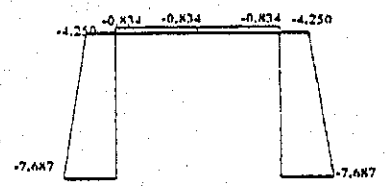


Fig. 3.8.5 Structural Model







BEENING MOMENT SCALE

COMPOSITE FOAD NO.1 0 2.0 m 0 2.0 ton

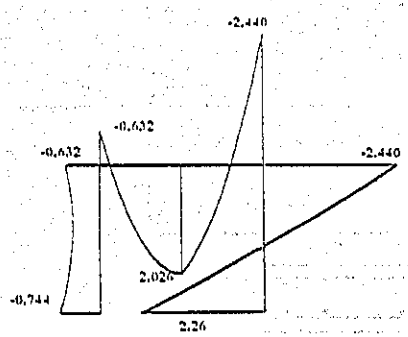
SHEAR FORCE SCALE

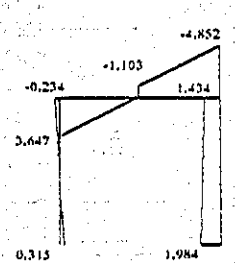
COMPOSITE FOAD NO.1 0 2.0 m 0 8.0 ton

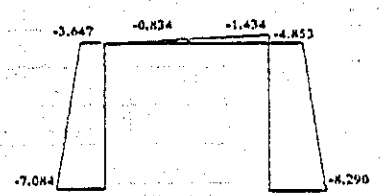
NORMAL FORCE SCALE

COMPOSITE FOAD NO.1 0 2.0 m 0 16.0 ton

Fig.3.8.6 (1/2) Sectional Force(Normal condition)







BEENING MOMENT SCALE

COMPOSITE FOAD NO.1 0 2.0 m 0 2.0 ton

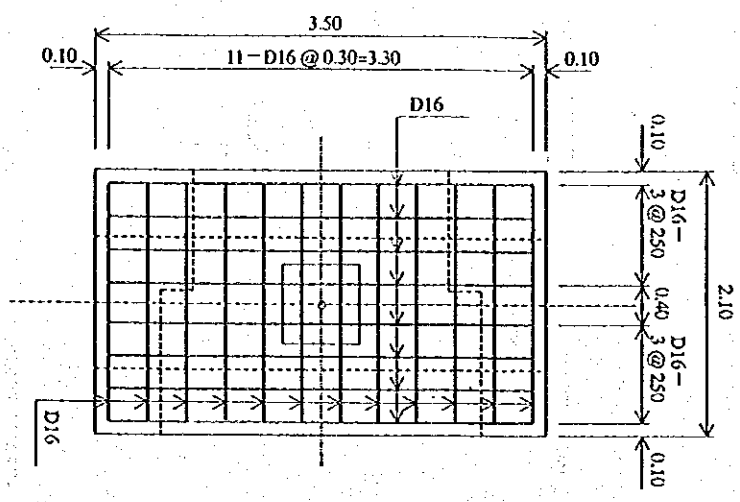
SHEAR FORCE SCALE

COMPOSITE FOAD NO.1 0 2.0 m 0 10.0 ton

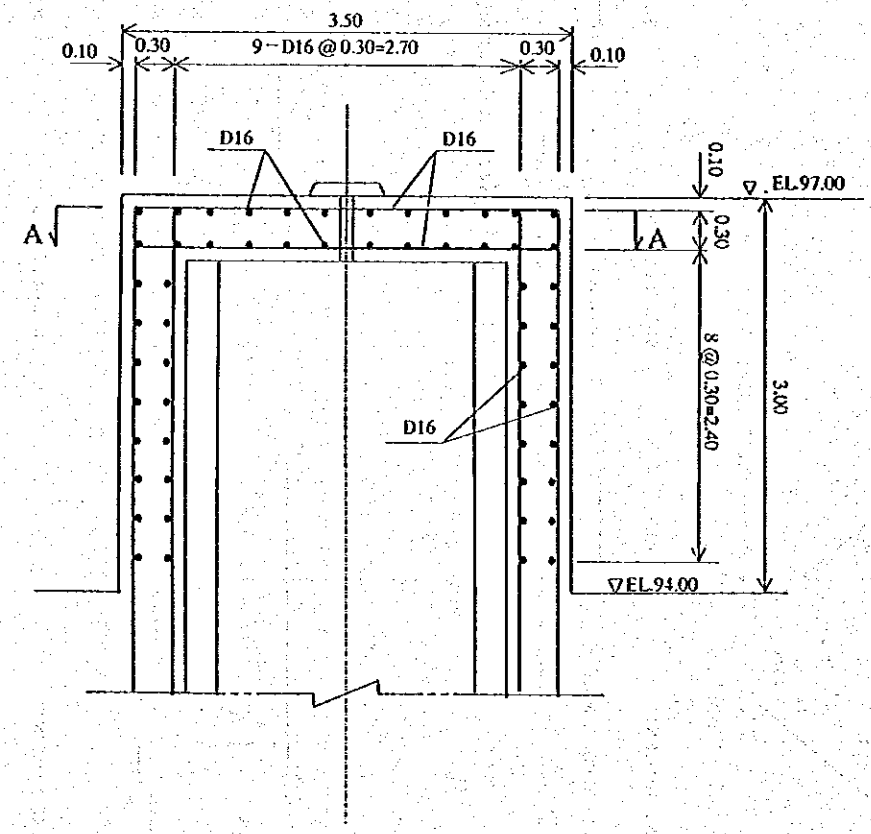
NORMAL FORCE SCALE

COMPOSITE FOAD NO.1 0 2.0 m 0 16.0 ton

Fig.3.8.6 (2/2) Sectional Force(Normal condition)



Plan A－A



Section

Fig. 3.8.7 Reinforcement Arrangement of Gate Tower

3.8.3 Design of Air Shaft

(1) General

The air shaft is located in the downstream side of powerhouse and its structure is connected to the spillway wing wall. Construction joint is provided between powerhouse structure and air shaft structure. The dimensions of the air shaft are shown in Figs.3.1.4~3.1.6. The typical section of the air shaft is shown in Fig.3.8.8.

(2) Design parameter and structural model

The calculated parameters and formulas are given as follows:

Unit weight

Unit weight of reinforced concrete	2.5 tf/m^3
Unit weight of embankment (saturated)	1.9 tf/m^3
Unit weight of embankment (in water)	0.9tf/m^3
Water	1.0 tf/m^3

Earth pressure coefficient

Coefficient of earth pressure under normal condition	0.335
Coefficient of earth pressure under seismic condition	0.444

Embankment and ground water levels

Embankment surface level: EL.97.0 m.

Ground water level: EL 84.798 m (assumed value for100 year return period flood)

Allowable strength of concrete and reinforcement

See Chapter 3.8.2.

Earth pressure

Earth pressure acting on gate shaft structure is considered as follows:

<u>Under normal condition</u>

The active earth pressure (Pa) is given as follows;

$$Pa = K_a \cdot \gamma \cdot h + K_a \cdot q$$

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

Where, K_a : coefficient of active earth pressure

γ : unit weight of soil (1.9tf/m^3)

h : earth depth to acting point of earth pressure

ϕ : internal friction angle (=35°)

θ : angle between wall backside surface and vertical plane

α : angle between ground surface and horizontal plane

δ : friction angle of soil to concrete (δ =2/3 ϕ)

q : surcharge in normal condition (1.0tf/m^2)

Under earthquake conditions

The active horizontal earth pressure (Pae) is given as follows;

$$Pae = K_{ea} \cdot \gamma \cdot h + K_{ea} \cdot q'$$

$$K_{ae} = \frac{\cos^2(\phi - \theta_0 - \theta)}{\cos^2\theta_0 \cdot \cos^2\theta \cdot \cos(\theta + \theta_0 + \delta) \cdot \left[1 + \sqrt{\frac{\sin(\phi+\delta)\cdot\sin(\phi-\alpha-\theta_0)}{\cos(\theta+\theta_0+\delta)\cdot\cos(\theta-\alpha)}}\right]^2}$$

Where, K_{ae} : coefficient of active earth pressure under earthquake conditions

θ_0 : angle between wall backside surface and vertical plane

$$\tan\theta_0 = \frac{K_h}{1 - K_V}$$

K_v : seismic coefficient in vertical direction (=0.0)

K_h : seismic coefficient in horizontal direction (=0.16)

α : angle between ground surface and horizontal plane

q' : surcharge under earthquake condition

The structural model for analysis is shown in Fig. 3.8.9.

(3) Load conditions

Following loads are considered as shown in Fig. 3.8.9.

Normal condition

(i) Dead Load

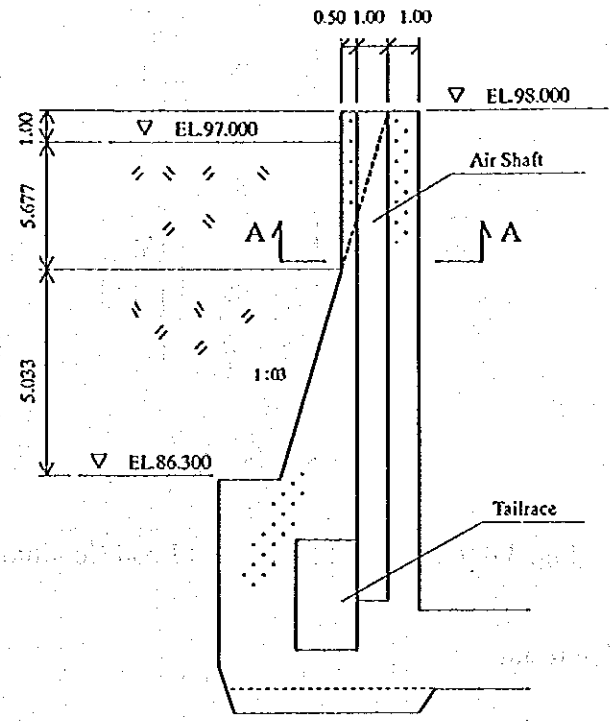
(ii) Earth Pressure

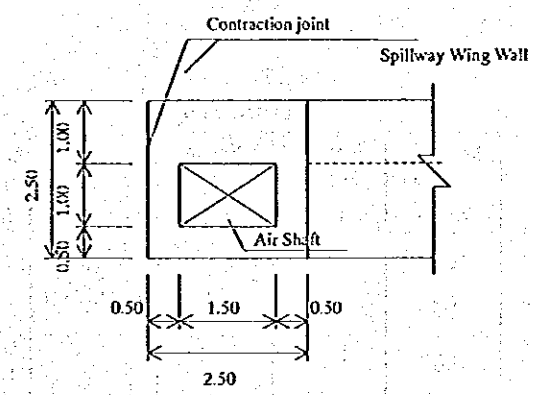
Earthquake condition

(i) Dead Load

(ii) Earth Pressure

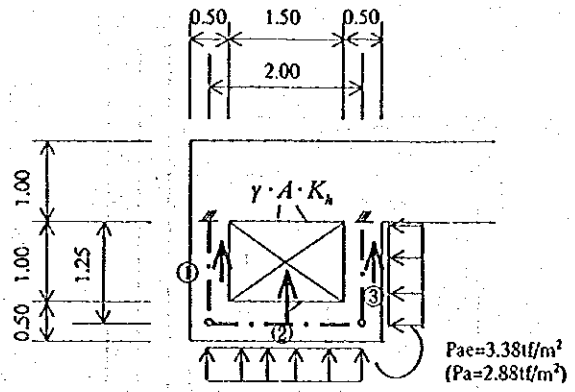
(ii) Seismic Force





Section A-A

Fig. 3.8.8 Section of Airshaft



Section A-A

Fig. 3.8.9 Structural Model and Load Conditions of Air Shaft

(4) Calculation results

The sectional forces under normal and seismic conditions are shown in Figs. 3.8.10. The reinforcement arrangement is given in the following table and Fig.3.8.11.

Section A-A

Mem.	Sec.	Load Case	Moment *1 (tf-m)	Normal Force*2 (tf)*1	Shear Force (tf)	Amount of Reinforcement		Stress of Steel and Concrete (kgf/cm²)		
						Outer side	Inner side	Concrete	Steel	Shear
②	Side	Normal	-1.04	1.53	3.18	D16@0.3 m 19D@0.2 m (4.223cm²/m))	-	13	450	0.7
		Seismic	-1.27	1.84	3.92		-	10	550	0.9
	Center	Normal	0.69	1.53	0.29	-	D16@0.3 m 19D@0.2 m (4.223cm²/m))	5	230	0.1
		Seismic	0.86	1.84	0.34	-		6	300	0.1
① (P/H side)	Top	Normal	-1.04	3.18	1.53	D16@0.3 m 19D@0.2 m (4.223cm²/m))	-	7	250	0.3
		Seismic	-1.27	3.92	1.84		-	9	300	0.4
	Botto m	Normal	0.87	3.18	1.53	-	D16@0.3 m 19D@0.2 m (4.223cm²/m))	5	150	0.3
		Seismic	0.92	4.17	1.84	-		5	90	0.3

Notes: *1 : + tension in inside - : tension in outside

*2 : + tension - : compression

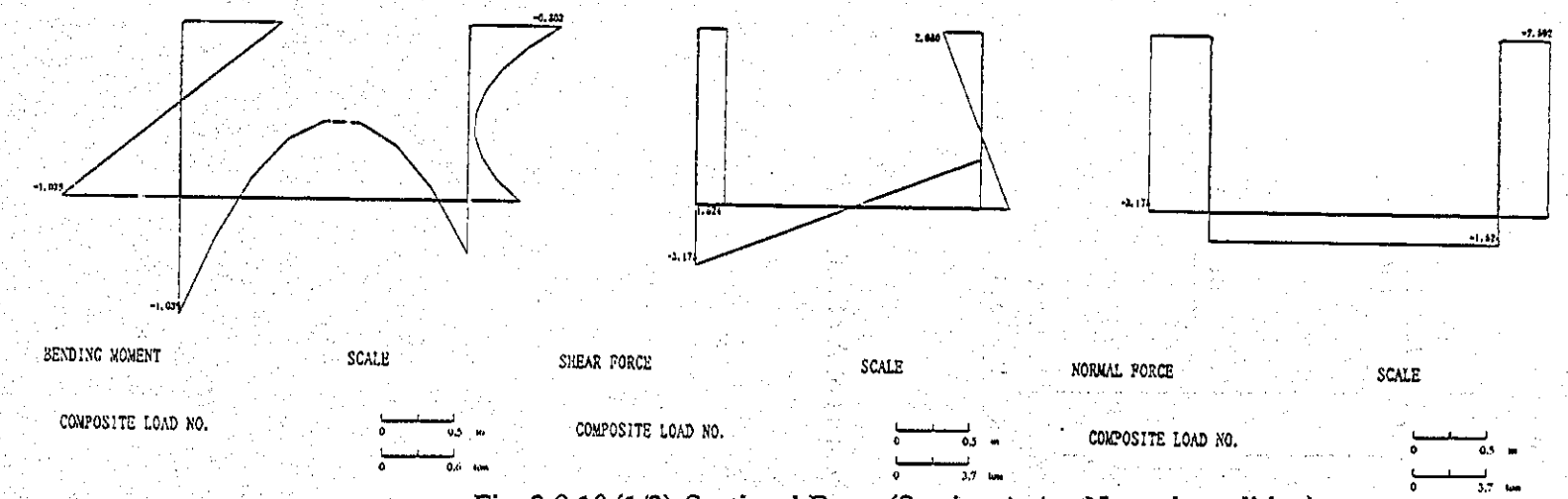


Fig. 3.8.10 (1/2) Sectional Force (Section A-A : Normal condition)

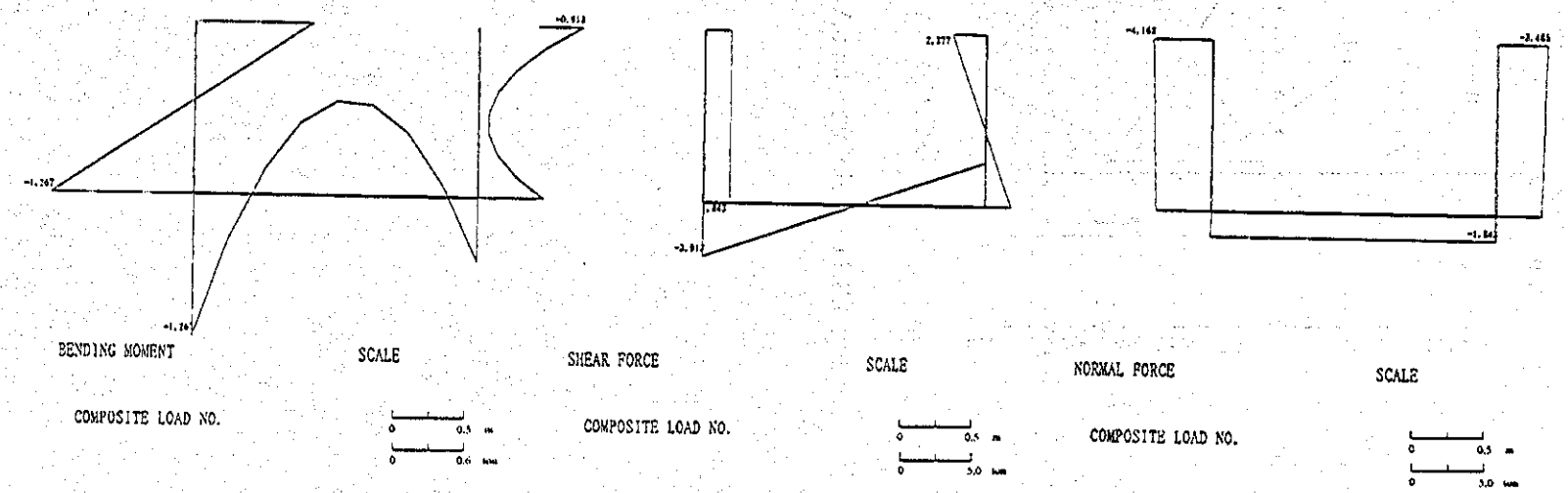


Fig. 3.8.10 (2/2) Sectional Force (Section A-A : Seismic condition)

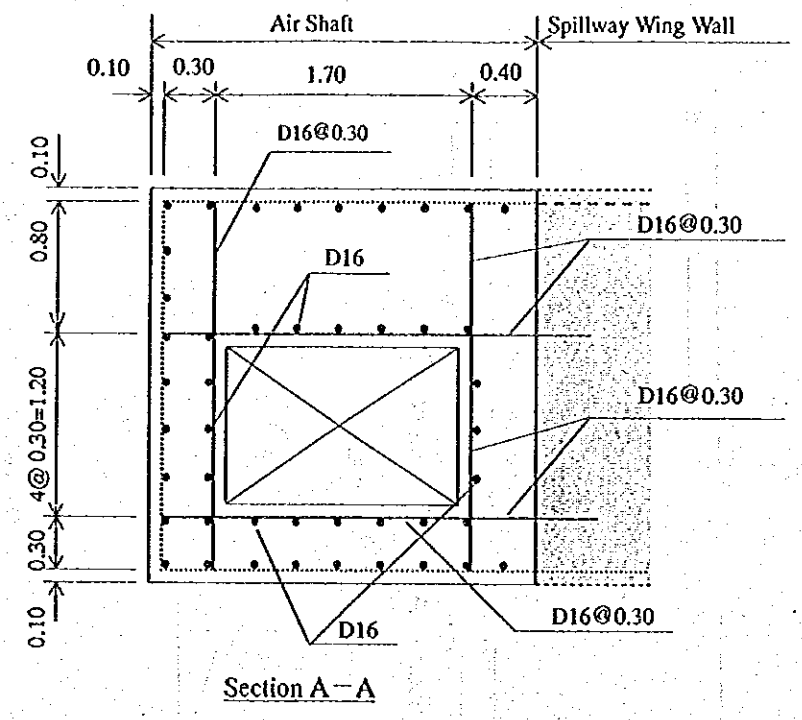


Fig.3.8.11 Reinforcement Arrangement of Air Shaft

3.8.4 Design of Tailrace Gate

(1) General

A steel slide gate is selected as the tailrace gate because gate size and hydrostatic pressure are small. The typical section of tailrace gate is shown in Fig.3.1.7.

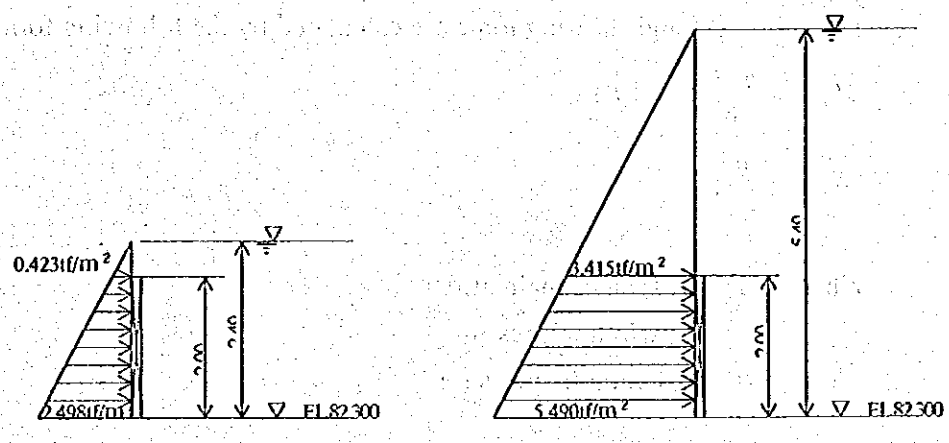
(2) Design Conditions

The design parameters for tailrace gate are given as follows.

Design Conditions		Remarks
Type	Steel slide gate	
Clear span	2.000 m	
Clear height	2.000 m	
Gate size	W 2.150 m H 2.075 m	
Design head	2.498 m (100 year return period flood) 5.490 m (PMF)	See Fig.3.8.12
Hoisting	Manual operation	

Allowable Stresses for Structural Steel (SS400)

Item	100 year return period flood	PMF	Remarks
Tensile stress	1,350kgf/cm^2	2,025kgf/cm^2 *1	*1 σsy = 1.5 σsa
Shearing stress	800kgf/cm^2	1,200kgf/cm^2 *2	*2 τ sy=1.5 τ sa
Deflection	1/800		



(1) Under 100 year return period (2) Under PMF

Fig 3.8.12 Design Head for Tailrace Gate

(3) Calculation of main girder

Reaction forces of auxiliary girders by hydraulic pressure are calculated as follows:

$$q_{1U} = \frac{(p_1 + 2p_2)l_1}{6}$$

$$q_{1L} = \frac{(2p_1 + p_2)l_1}{6}$$

$$q_{2U} = \frac{(2p_2 + p_3)l_2}{6}$$

$$q_{2L} = \frac{(p_2 + 2p_3)l_2}{6}$$

$$q_{3U} = \frac{(p_3 + p_5)(l_3 + l_4)}{2} - q_{3L}$$

$$q_{3L} = \frac{(p_3 + 2p_5)(l_3 + l_4)^2}{6l_3}$$

Where, l : span of auxiliary girders
p : hydraulic pressure at the center point of main girders
q : reaction of auxiliary girders
(see Fig. 3.8.13)

Loads of each main girders are :

$q_1 = q_{1U}$
$q_2 = q_{1L} + q_{2U}$
$q_3 = q_{2L} + q_{3U}$
$q_4 = q_{3L}$

Bending moment and shearing force are calculated by the following formulae:

$$M_{max} = \frac{q \cdot B}{8}(2L - B)$$

$$S_{max} = \frac{q \cdot B}{2}$$

Where, L : span of main girder
B : width of pressure
q : hydraulic pressure

Bending stress and shearing stress are calculated by the following formulae:

$$\sigma_{max} = \frac{M_{max}}{Z}$$

$$\tau_{max} = \frac{S_{max}}{A_{web}}$$

Where, Z : section modulus

A_{web} : area of web plate

Deflection of main girder is calculated by the following formula :

$$\delta_{max} = \frac{q \cdot B}{48 E \cdot I}\left(L^3 - \frac{L \cdot B^2}{2} + \frac{B^3}{8}\right)$$

Where, L : span of main girders

B : width of pressure

q : hydraulic pressure

E : elasticity modulus of steel

I : moment of inertia

Calculation sheets of main girders are given below:

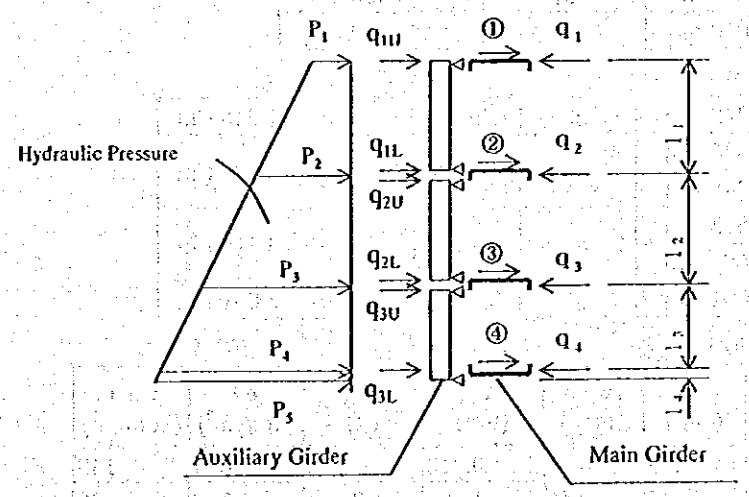


Fig. 3.8.13 Design Loads of Main Girders

Calculation Results of Main Girder under 100 year return period flood

Span of main girders and hydrostatic pressure

		p_1	0.423 t/m²
l_1	0.685 m	p_2	1.108 t/m²
l_2	0.670 m	p_3	1.778 t/m²
l_3	0.680 m	p_4	2.458 t/m²
l_4	0.040 m	p_5	2.498 t/m²

Loads for each main girder

q_{1U}	0.223 t/m	q_1	q_{1U}	0.223 t/m
q_{1L}	0.301 t/m	q_2	$q_{1L} + q_{2U}$	0.747 t/m
q_{2U}	0.446 t/m			
q_{2L}	0.521 t/m	q_3	$q_{2L} + q_{3U}$	1.199 t/m
q_{3U}	0.679 t/m			
q_{3L}	0.861 t/m	q_4	q_{3L}	0.861 t/m

Bending stress, Shear stress and Deflection

Member	I (cm⁴)	Z (cm³)	Aweb (cm²)	M_{max} (t-m)	S_{max} (tf)	σ t (kgf/cm²)	τ max (kgf/cm²)	δ_{max} (cm)	δ_{max}/L
No.1	864	115	8.45	0.128	0.223	112	26	0.0340	1/ 6322
No.2	864	115	8.45	0.430	0.747	374	88	0.1139	1/ 1887
No.3	864	115	8.45	0.690	1.199	600	142	0.1829	1/ 1176
No.4	864	115	8.45	0.495	0.861	430	102	0.1312	1/ 1639

Calculation Results of Main Girder under PMF

Span of main girders and hydrostatic pressure

		p_1	3.415 t/m²
l_1	0.685 m	p_2	4.100 t/m²
l_2	0.670 m	p_3	4.770 t/m²
l_3	0.680 m	p_4	5.450 t/m²
l_4	0.040 m	p_5	5.490 t/m²

Loads for each main girder

q_{1U}	1.248 t/m	q_1	q_{1U}	1.248 t/m
q_{1L}	1.326 t/m	q_2	$q_{1L} + q_{2U}$	2.774 t/m
q_{2U}	1.448 t/m			
q_{2L}	1.523 t/m	q_3	$q_{2L} + q_{3U}$	3.216 t/m
q_{3U}	1.692 t/m			
q_{3L}	2.001 t/m	q_4	q_{3L}	2.001 t/m

Bending stress, Shear stress

Member	I (cm⁴)	Z (cm³)	Aweb (cm²)	M_{max} (t-m)	S_{max} (tf)	σ t (kgf/cm²)	τ max (kgf/cm²)
No.1	864	115	8.45	0.718	1.248	624	148
No.2	864	115	8.45	1.595	2.774	1,387	328
No.3	864	115	8.45	1.849	3.216	1,608	381
No.4	864	115	8.45	1.151	2.001	1,001	237

Member	A (cm²)	Aweb (cm²)	I (cm⁴)	Z (cm³)
[150X75X6.5X10	23.710	8.45	864	115

(4) Calculation of auxiliary girder and side girder

Bending moment and shearing force are calculated by the following formulae:

$$M_{max} = \frac{p \cdot m}{24}(3l^2 - m^2) \quad (l > m)$$

$$S_{max} = \frac{p \cdot m}{2}\left(l - \frac{m}{2}\right)$$

Where, l : span of auxiliary girders
m : pitch of auxiliary girders
p : hydraulic pressure
(see Fig. 3.8.14 and 3.8.15)

Bending stress and shearing stress are calculated by the following formulae:

$$\sigma_{max} = \frac{M_{max}}{Z}$$

$$\tau_{max} = \frac{S_{max}}{A_{web}}$$

Where, Z : section modulus
E : elasticity modulus of steel
I : moment of inertia

Calculation sheets of auxiliary girders are given below;

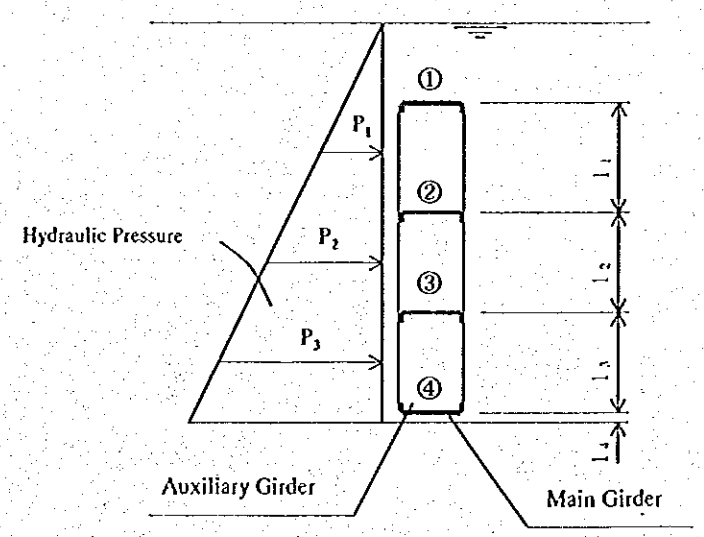


Fig. 3.8.14 Design Loads of Auxiliary Girders

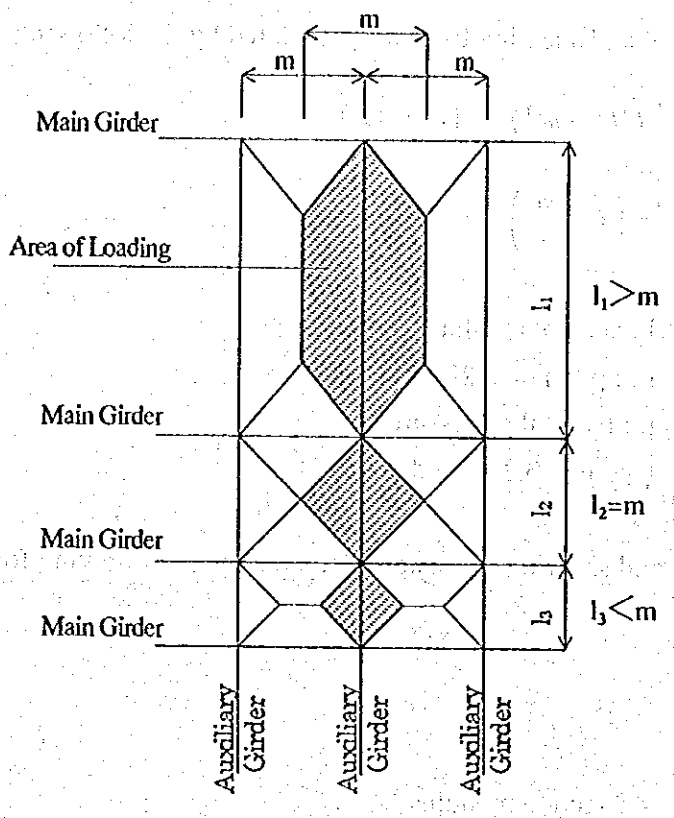


Fig. 3.8.15 Area of Loading for Auxiliary Girders

Calculation Results of Auxiliary Girder under 100 year return period flood

Span of main girders and hydrostatic pressure

		p_1	0.423 t/m²
l_1	0.685 m	p_2	1.108 t/m²
l_2	0.670 m	p_3	1.778 t/m²
l_3	0.680 m	p_4	2.458 t/m²
l_4	0.040 m	p_5	2.498 t/m²

Loads for each auxiliary girder

p'_1	0.766 t/m
p'_2	1.443 t/m
p'_3	2.118 t/m

Bending stress, Shear stress

Member			m (m)	l (m)	M (tf-m)	S (tf)	σ t (kgf/cm²)	τ max (kgf/cm²)
m_2, m_3	①~②	PL 150X6	0.500	0.685	0.018	0.083	82	9
	②~③	PL 150X6	0.500	0.670	0.033	0.152	147	17
	③~④	PL 150X6	0.500	0.680	0.050	0.228	223	25
m_1, m_4	①~②	[150X75X6.5X10	0.500	0.685	0.018	0.083	16	10
	②~③	[150X75X6.5X10	0.500	0.670	0.033	0.152	29	18
	③~④	[150X75X6.5X10	0.500	0.680	0.050	0.228	44	27

Calculation Results of Auxiliary Girder under PMP

Span of main girders and hydrostatic pressure

		p_1	3.415 t/m²
l_1	0.685 m	p_2	4.100 t/m²
l_2	0.670 m	p_3	4.770 t/m²
l_3	0.680 m	p_4	5.450 t/m²
l_4	0.040 m	p_5	5.490 t/m²

Loads for each auxiliary girder

p'_1	3.758 t/m
p'_2	4.435 t/m
p'_3	5.110 t/m

Bending stress, Shear stress

Member			m (m)	l (m)	M (tf-m)	S (tf)	σ t (kgf/cm²)	τ max (kgf/cm²)
m_2, m_3	①~②	PL 150X6	0.500	0.685	0.091	0.409	403	45
	②~③	PL 150X6	0.500	0.670	0.101	0.466	450	52
	③~④	PL 150X6	0.500	0.680	0.121	0.549	538	61
m_1, m_4	①~②	[150X75X6.5X10	0.500	0.685	0.091	0.409	79	48
	②~③	[150X75X6.5X10	0.500	0.670	0.101	0.466	88	55
	③~④	[150X75X6.5X10	0.500	0.680	0.121	0.549	105	65

Member		A (cm²)	Aweb (cm²)	I (cm⁴)	Z (cm³)
m_2, m_3	PL 150X6	9.000	9.000	168.75	22.5
m_1, m_4	[150X75X6.5X10	23.710	8.450	864	115

(5) Calculation of skin plate

The bending stress generated in a flat plate by hydraulic pressure shall be calculated by the following formula:

$$\sigma = \frac{1}{100} k \cdot a^2 \cdot \frac{p}{(t-\varepsilon)^2}$$

Where,
- a : short side of a rectangle
- b : long side of a rectangle
- p : hydraulic pressure
- t : plate thickness
- k : factor in table below
- ε : corrosion allowance (=1mm)
- σ: bending stress (see Fig. 3.8.16)

Value of k				
b/a	σ_1	σ_2	σ_3	σ_4
1.0	30.90	13.70	13.70	30.90
1.3	40.30	18.80	13.50	33.90
1.5	45.50	22.10	12.20	34.30
1.8	48.40	23.90	10.80	34.30
2.0	49.90	24.70	9.50	34.30
2.5	50.00	25.00	8.00	34.30
3.0	50.00	25.00	7.50	34.30
∞	50.00	25.00	7.50	34.30

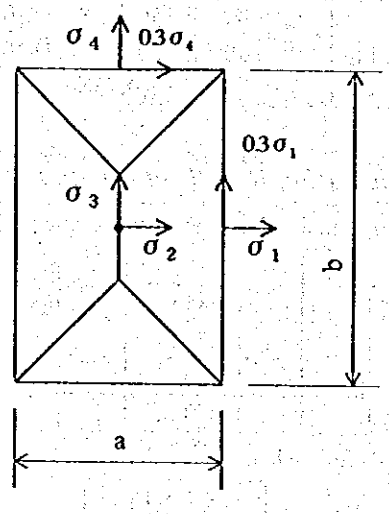


Fig.3.8.16 Stress of Skin Plates

Minimum thickness of skin plate is 6 mm. Calculation results are given below and shown in Fig.3.8.17.

Material list of tailrace gate is as shown below :

Items		Dimension	Remarks
Main girder		[150 × 75 × 6.5 × 10	
Auxiliary girder	Side	[150 × 75 × 6.5 × 10	
	Center	Plate 150 × 6	
Skin plate		t = 6mm	

Calculation Results of Skin plate under 100 year return period flood

Span of main girders and hydrostatic pressure

		p_1	0.423 t/m²
l_1	0.685 m	p_2	1.108 t/m²
l_2	0.670 m	p_3	1.778 t/m²
l_3	0.680 m	p_4	2.458 t/m²
l_4	0.040 m	p_5	2.498 t/m²

Loads for each auxiliary girder

p'_1	0.766 t/m
p'_2	1.443 t/m
p'_3	2.118 t/m

Stress

Location	Member	a	b	b/a	k	p'	t	σ t	t_0
		Short side of a rectangle	Long side of a rectangle		Factor	Hydraulic pressure	Plate thickness	Stress	$t_0=t+\varepsilon$
		cm	cm			kgf/cm²	cm	kgf/cm²	
Center	①~②	57.250	57.500	1.004	31.1	0.077	0.5	312	0.6
	②~③	57.500	59.500	1.035	32.2	0.144	0.5	615	0.6
	③~④	56.750	57.500	1.013	31.4	0.212	0.5	857	0.6
Side	①~②	50.000	57.250	1.145	36.4	0.077	0.5	278	0.6
	②~③	50.000	59.500	1.190	38.0	0.144	0.5	549	0.6
	③~④	50.000	56.750	1.135	36.0	0.212	0.5	762	0.6

ε : allowance (=1mm)

Calculation Results of Skin plate under PMF

Span of main girders and hydrostatic pressure

		p_1	3.415 t/m²
l_1	0.685 m	p_2	4.100 t/m²
l_2	0.670 m	p_3	4.770 t/m²
l_3	0.680 m	p_4	5.450 t/m²
l_4	0.040 m	p_5	5.490 t/m²

Loads for each skin plate

p'_1	3.758 t/m
p'_2	4.435 t/m
p'_3	5.110 t/m

Stress

Location	Member	a	b	b/a	k	p'	t	σ t	t_0
		Short side of a rectangle	Long side of a rectangle		Factor	Hydraulic pressure	Plate thickness	Stress	$t_0=t+\varepsilon$
		cm	cm			kgf/cm²	cm	kgf/cm²	
Center	①~②	57.250	57.500	1.004	31.1	0.376	0.5	1530	0.6
	②~③	57.500	59.500	1.035	32.2	0.444	0.5	1889	0.6
	③~④	56.750	57.500	1.013	31.4	0.511	0.5	2067	0.6
Side	①~②	50.000	57.250	1.145	36.4	0.376	0.5	1366	0.6
	②~③	50.000	59.500	1.190	38.0	0.444	0.5	1687	0.6
	③~④	50.000	56.750	1.135	36.0	0.511	0.5	1838	0.6

ε : allowance (=1mm)

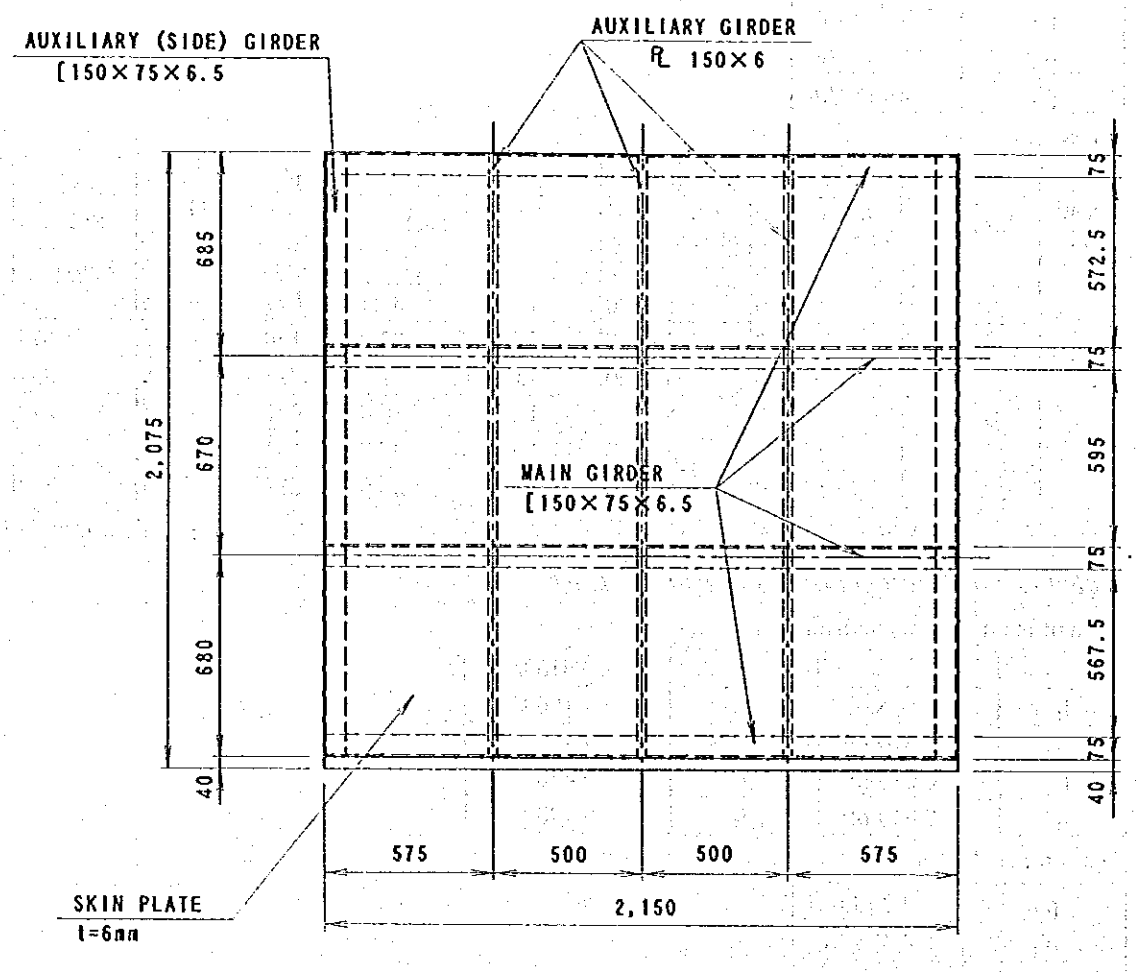


Fig.3.8.17 Dimension of Tailrace Gate