NO. 6

JAPAN INTERNATIONAL COOPERATION AGENCY (JICA)

THE GOVERNMENT OF THE REPUBLIC OF INDONESIA
MINISTRY OF PUBLIC WORKS
DIRECTORATE GENERAL OF WATER RESOURCES DEVELOPMENT

THE DETAILED DESIGN STUDY ON MEDAN FLOOD CONTROL PROJECT

FINAL REPORT

VOL. III - 1

DESIGN NOTES

RIVER AND RIVER STRUCTURES

OCTOBER 1996

CTI ENGINEERING CO., LTD.

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THE DETAILED DESIGN STUDY
ON MEDAN FLOOD CONTROL PROJECT

FINAL REPORT

VOL. III - 1

DESIGN NOTES

RIVER AND RIVER STRUCTURES

TABLE OF CONTENTS

1. RIVER CHANNEL AND SLOPE PROTECTION

1.1 RIVER HYDRAULICS

1.1.1 PERCUT RIVER

<u>Preparation of Standard Cross Section</u>

A standard cross section of the river channel is prepared for the detailed design of the channel such as alignment, longitudinal profile and cross section.

(1) Conditions on Hydraulic Calculation

(a) Location

Based on the river characteristics and river plan, the target river stretch can be divided into five (5) separable portions as follows:

[Lower Percut River] ------ River stretch with diking system

- River mouth to PE14
- PE14 to 46
- PE46 to PE106
- PE106 to PE129+43

[Upper Percut River] ------ River stretch without diking system

- PE129+43 to PE274

(b) Method

Uniform steady flow calculation is used.

(c) Design Discharge

- Urgent Plan (40-year return period): 320 m^3/s
- Immediate Plan (25-year return period) : 320 m^3/s

(d) Roughness Coefficient

- Low water channel (existing): 0.035
- Low water channel (excavated): 0.033
- Flood plain : 0.040

(e) Initial Water Level

The mean high water spring is used. (EL. 1.300 m)

(f) Dimensions of Cross-sectional Form

The slopes of the river-bank and dike are set at 1 : 2. The width and height of low water channel and the distances between river-banks and dikes are determined based on the hydraulic calculation.

(2) Calculation Results

The calculation results together with channel dimensions are shown in Fig 1.1.1.

(3) Non-Uniform Flow Calculation

Using the standard cross sections, the non-uniform flow calculation for the whole river channel from river mouth to PE274 were conducted and results are shown in Calculation Sheet-1.1.1.

<u>**Cross Sections at Titi Tuntuh, Railway and National Road Bridge**</u>

(1) Purpose of Hydraulic Calculation

Protection works are provided for the Titi Tuntuh Bridge, Railway Bridge, Tollway Bridge and National Road Bridge instead of river widening. Prior to the detailed design of protection works, cross-sectional forms of river channel at the bridge sites are prepared. As to the Tollway Bridge, the river cross section has enough river width and clearance to bridge girder. Therefore, a standard cross section will be applied.

(1) Conditions on Hydraulic Calculation

(a) Water Depth

The water depth under design flood shall be the same depth of standard river section (d=6.11 m) except the cross-section of National Road Bridge.

(b) Calculation Method

Uniform steady flow calculation is used for the Titi Tuntuh Bridge and Railway Bridge. On the other hand a non-uniform flow calculation is employed for the National Road Bridge because this river area forms a bottle-neck in the river course.

(c) Flow Velocity

The flow velocity shall not be more than 3.0 m/s except that of National Road Bridge.

(d) Roughness Coefficient

- Low water channel (protected by revetment for side slope and gabion mattress for riverbed): 0.030
- Low water channel (protected by leaning wall for side slope and concrete Block for riverbed) : 0.025

(2) Calculation Results

The cross-sections determined are shown in Figs.1.1.2 and 1.1.3, and the result of non-uniform flow calculation for the river section around the Railway Bridge is listed in Calculation Sheet-1.1.2.

<u>Water Surface Profile around National Road Bridge</u>

(1) Purpose of the Study

Hydraulics at bottle-neck portion is studied to design the protection works.

(2) Study Results

Based on the non-uniform calculation the water surface profile is estimated as shown in Fig.1.1.4 and Table 1.1.1. The maximum fall in water level is about 0.45 m and the maximum flow velocity is estimated at 3.7 m/s. Besides, as can be seen from Fig. 1.1.4, back-water effect to the upstream by the bottle-neck portion is only 5 cm rise in water level. Though the water level in the upstream exceeds the design high water level, its amount is so small that no impact to the upstream is expected.

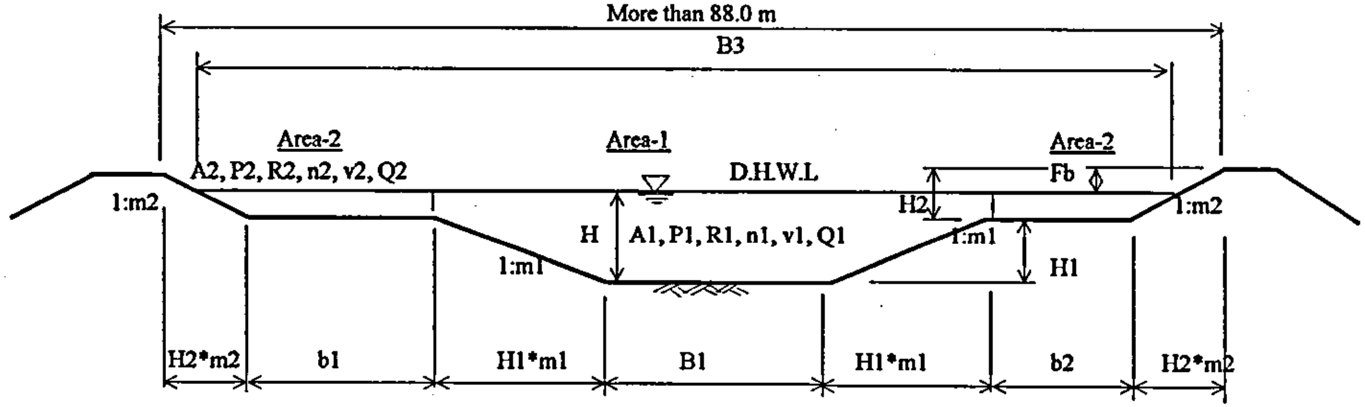
The water level at FW3, which is the starting section of Floodway, is EL.30.708 m and this is lower than the design elevation of FW3 (EL.30.730 m).(See Table 1.1.1)

Uniform Flow Calculation **Lower Percut River**

Section : (PE14 to PE46)

Roughness Coeff.	1	n1 =	0.033	
Roughness Coeff.	2	n2 =	0.035	
Channel Bed Slope		i =	0.000125	(I = 1 / 8,000)
Channel Bed Width		B1 =	40.00 m	
Height 1		H1 =	3.50 m	
Height 2		H2 =	3.00 m	
Slope Gradient 1		m1 =	2.00	
Slope Gradient 2		m2 =	2.00	
Width 1		b1 =	6.00 m	
Width 2		b2 =	16.00 m	
Freeboard for Dike		Fb =	0.80 m	

Hydraulic Radius (m) : R = A / P
Velocity (m/s) : v = 1/n * R ^(2/3) * I^(1/2)
Discharge (m3/s) : Q = v x A
Design Discharge
Immediate Plan : 320.00 (m3/s)
Urgent Plan : 320.00 (m3/s)



Calculation Table

	H	(B1)	(B2)	(B3)	A1	A2	P1	P2	R1	R2	n1	n2	i	v1	v2	Q1	Q2	Q
	m	m	m	m	m2	m2	m	m	m	m				m/s	m/s	m3/s	m3/s	m3/s
1	5.70	40.00	62.80	84.80	292.98	48.40	55.65	31.84	5.26	1.52	0.033	0.035	0.000125	1.03	0.42	300.39	20.44	320.83
2	5.70	40.00	62.80	84.80	292.98	48.40	55.65	31.84	5.26	1.52	0.033	0.035	0.000125	1.03	0.42	300.39	20.44	320.83
3	3.50	40.00	54.00		164.50		55.65		2.96		0.033		0.000125	0.70		114.79	0.00	114.79

1 Immediate Plan Q = 320.83 > 320.00 (m3/s) OK
2 Urgent Plan Q = 320.83 > 320.00 (m3/s) OK

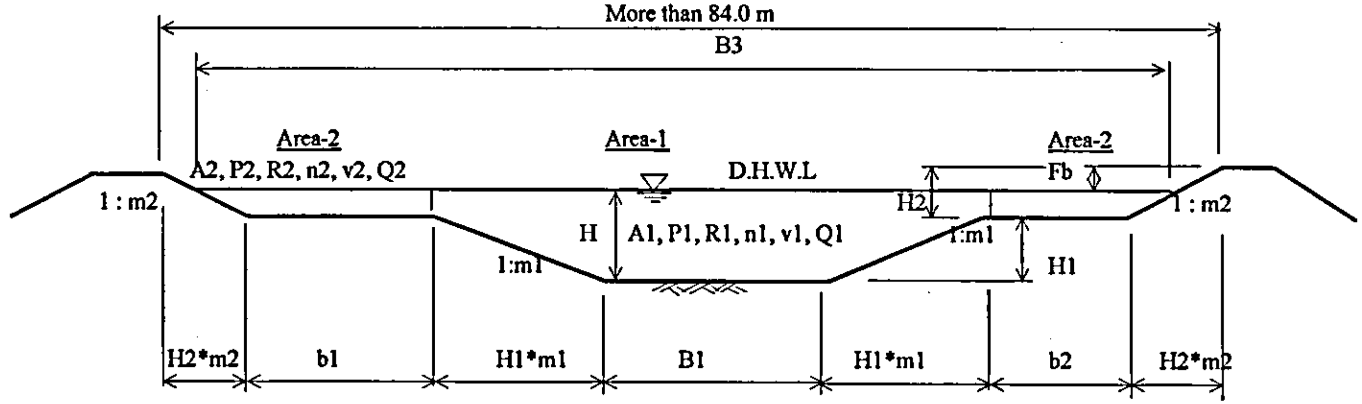
Fig. 1. 1. 1 (1/4) HYDRAULIC DESIGN CALCULATION FOR PERCUT RIVER CHANNEL

Uniform Flow Calculation — **Lower Percut River**

Section : (PE 46 to PE 106)

Roughness Coeff.	1	n1 =	0.033	
Roughness Coeff.	2	n2 =	0.040	
Channel Bed Slope		i =	0.000833	(I = 1 / 1,200)
Channel Bed Width		B1 =	13.00 m	
Height 1		H1 =	5.00 m	
Height 2		H2 =	1.50 m	
Slope Gradient 1		m1 =	2.00	
Slope Gradient 2		m2 =	2.00	
Width 1		b1 =	23.00 m	
Width 2		b2 =	22.00 m	
Freeboard for Dike		Fb =	0.80 m	

Hydraulic Radius	(m)	: R = A / P
Velocity	(m/s)	: v = 1/n * R ^(2/3) * I^(1/2)
Discharge	(m3/s)	: Q = v x A
Design Discharge		
	Immediate Plan :	320.00 (m3/s)
	Urgent Plan :	320.00 (m3/s)



Calculation Table

	H	(B1)	(B2)	(B3)	A1	A2	P1	P2	R1	R2	n1	n2	i	v1	v2	Q1	Q2	Q
	m	m	m	m	m2	m2	m	m	m	m				m/s	m/s	m3/s	m3/s	m3/s
1	5.70	13.00	35.80	80.80	139.08	31.50	35.36	48.13	3.93	0.65	0.033	0.040	0.000833	2.18	0.54	303.15	17.14	320.29
2	5.70	13.00	35.80	80.80	139.08	31.50	35.36	48.13	3.93	0.65	0.033	0.040	0.000833	2.18	0.54	303.15	17.14	320.29
3	5.00	13.00	33.00		115.00		35.36		3.25		0.033		0.000833	1.92		220.82	0.00	220.82

1	Immediate Plan	Q =	320.29	>	320.00	(m3/s)	OK
2	Urgent Plan	Q =	320.29	>	320.00	(m3/s)	OK

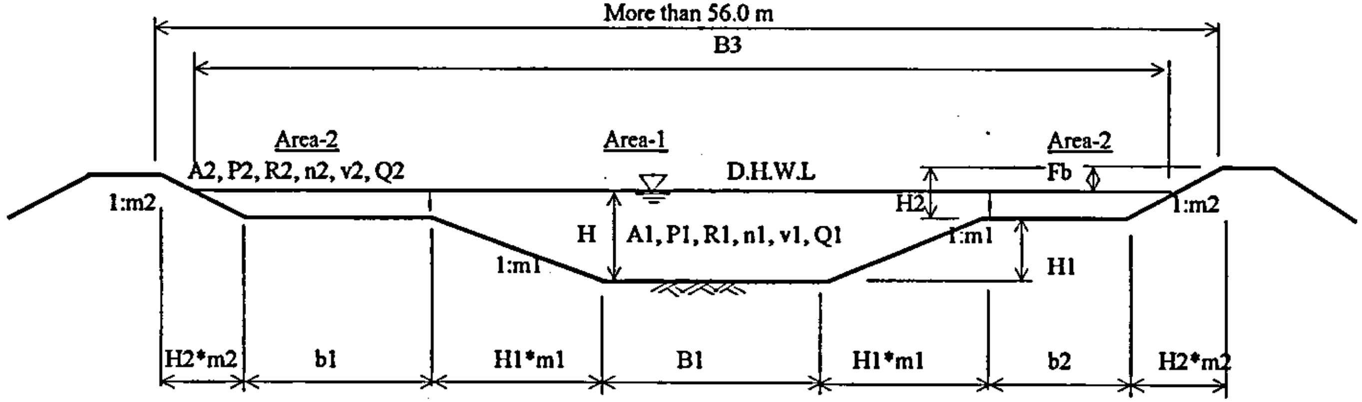
Fig. 1. 1. 1 (2/4) HYDRAULIC DESIGN CALCULATION FOR PERCUT RIVER CHANNEL

Uniform Flow Calculation

Section : (PE 106 to PE 129+43)

Roughness Coeff.	1	n1 =	0.033	
Roughness Coeff.	2	n2 =	0.040	
Channel Bed Slope		i =	0.001212	(I = 1 / 825)
Channel Bed Width		B1 =	10.00 m	
Height 1		H1 =	5.00 m	
Height 2		H2 =	1.50 m	
Slope Gradient 1		m1 =	2.00	
Slope Gradient 2		m2 =	2.00	
Width 1		b1 =	12.00 m	
Width 2		b2 =	8.00 m	
Freeboard for Dike		Fb =	0.80 m	

Hydraulic Radius (m) : R = A / P
Velocity (m/s) : v = 1/n * R ^(2/3) * I^(1/2)
Discharge (m3/s) : Q = v x A
Design Discharge
Immediate Plan : Qa = 320.00 (m3/s)
Urgent Plan : Qa = 320.00 (m3/s)
Design High Water Level



Calculation Table

	H	(B1)	(B2)	(B3)	A1	A2	P1	P2	R1	R2	n1	n2	i	v1	v2	Q1	Q2	Q
	m	m	m	m	m2	m2	m	m	m	m				m/s	m/s	m3/s	m3/s	m3/s
1	5.70	10.00	32.80	52.80	121.98	14.00	32.36	23.13	3.77	0.61	0.033	0.040	0.001212	2.56	0.62	311.69	8.72	320.41
2	5.70	10.00	32.80	52.80	121.98	14.00	32.36	23.13	3.77	0.61	0.033	0.040	0.001212	2.56	0.62	311.69	8.72	320.41
3	5.00	10.00	30.00		100.00		32.36		3.09		0.033		0.001212	2.24		223.83	0.00	223.83

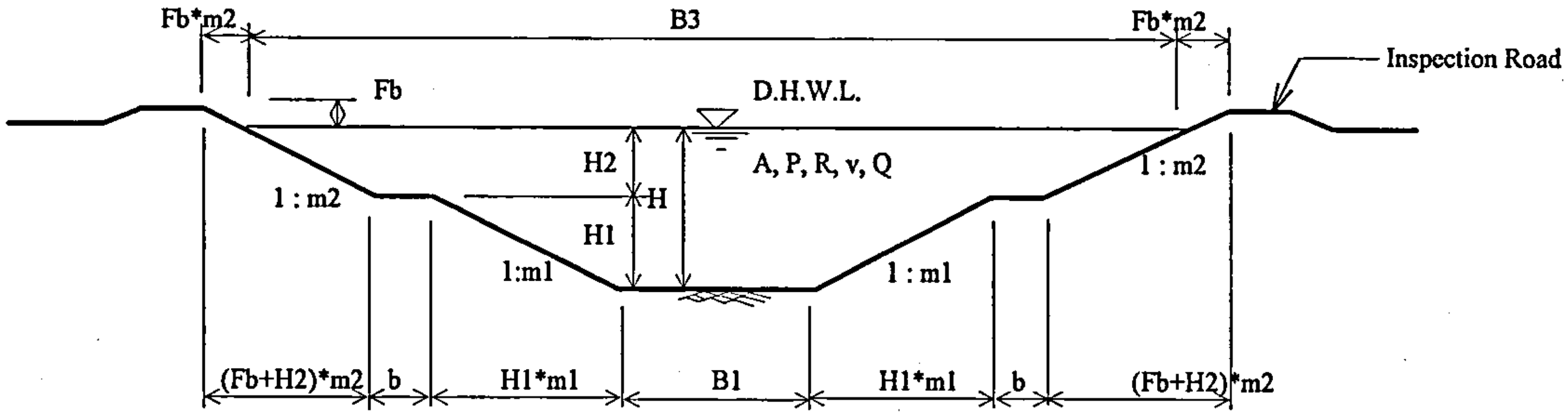
1	Immediate Plan	Q =	320.41	>	320.00	(m3/s)	OK
2	Urgent Plan	Q =	320.41	>	320.00	(m3/s)	OK

Fig. 1. 1. 1 (3/4) HYDRAULIC DESIGN CALCULATION FOR PERCUT RIVER CHANNEL

Uniform Flow Calculation **Upper Percut River** **Section : (PE129+43n to PE 274)**

Roughness Coefficient	n =	0.033	
River Bed Slope	i =	0.001212	(I = 1 / 825)
River Bed Width	B1 =	8.00 m	
Height 1	H1 =	3.00 m	
Height 2	H2 =	3.11	
Slope Gradient 1	m1 =	2.00	
Slope Gradient 2	m2 =	2.00	
Width of Berm	b =	1.50 m	
Freeboard for Inspection Road	Fb =	0.80 m	

Hydraulic Radius (m) : R = A/P
Velocity (m/s) : v = 1/n * R ^(2/3) * I^(1/2)
Discharge (m3/s) : Q = v x A
Design Discharge
Immediate Plan Qa = 320.00 (m3/s)
Urgent Plan Qa = 320.00 (m3/s)



Calculation Table

	H	B1	B2	B3	A1	A2	P	R	n	i	v	Q
	m	m	m	m	m2	m2	m	m			m/s	m3/s
1	6.11	8.00	20.00	35.44	42.00	132.87	38.32	3.47	0.033	0.001212	2.42	321.12
2	6.11	8.00	20.00	35.44	42.00	132.87	38.32	3.47	0.033	0.001212	2.42	321.12
3	3.00	8.00	20.00		42.00		21.42	1.96	0.033	0.001212	1.65	69.42

1. Immediate Plan : Q = 321.12 > 320.00 (m3/s)
2. Urgent Plan : Q = 321.12 > 320.00 (m3/s)

Fig. 1. 1. 1 (4/4) HYDRAULIC DESIGN CALCULATION FOR PERCUT RIVER CHANNEL

Calculation Sheet-1. 1. 1

NON-UNIFORM FLOW CALCULATION

1

CASE 1: PERCUT RIVER (Q=320m3/s, Ho=1.30)-FINAL- 1995/12/24 by PERCUT9.DAT PAGE = 1

** INPUT DATA **

* BASIC DATA *

Number of Section		Design Discharge	Initial Water Level	Initial Riverbed Elevation
KUKAN-SU = 21	ALPHA = 1.10	QO = 320.00 M3/S	HO = 1.300 M	ZO = -1.800 M

JCO = 0 KEY = 0 IPT = 0

* SEGMENT DATA *

	Cross-Section DANMEN NO.	BUNKATSU SUU	DANMEN KEIJYO		LOSS TYPE	Roughtness Coeff. SODO KEISU	Distance KUKAN KYORI(M)	Slope Gradient KASYO KOOBAI(1/I)	RAKUSA (M)	RYUNYU RYO(M3/S)
1	NO. -0.50	2	2	1	0	.0330	500.00	1000000.00	.000	.0(
2	NO. 0.00	5	2	1	0	.0330	1000.00	8000.00	.000	.0(
3	NO. 1.00	2	2	1	0	.0330	300.00	8000.00	.000	.0(
4	NO. 1.30	2	2	1	0	.0330	300.00	8000.00	.000	.0(
5	NO. 1.60	15	2	1	0	.0330	3000.00	8000.00	.000	.0(
6	NO. 4.60	2	2	1	0	.0330	200.00	1200.00	.000	.0(
7	NO. 4.80	12	2	1	0	.0330	2400.00	1200.00	.000	.0(
8	NO. 7.20	2	2	1	0	.0330	200.00	1200.00	.000	.0(
9	NO. 7.40	16	2	1	0	.0330	3200.00	1200.00	.000	.0(
10	NO. 10.6	2	2	1	0	.0330	200.00	825.00	.000	.0(
11	NO. 10.8	12	2	1	0	.0330	2444.00	825.00	.000	.0(
12	NO. 13.244	2	2	1	0	.0330	156.00	825.00	.000	.0(
13	NO. 13.4	10	2	1	0	.0330	4580.00	825.00	.000	.0(
14	NO. 17.98	2	2	1	0	.0300	20.00	825.00	.000	.0(
15	NO. 18.00	2	2	1	0	.0300	20.00	825.00	.000	.0(
16	NO. 18.02	10	2	1	0	.0330	9425.00	825.00	.000	.0(
17	NO. 27.445	5	2	1	0	.0300	20.00	825.00	.000	.0(
18	NO. 27.465	10	2	1	0	.0250	40.00	825.00	.000	.0(
19	NO. 27.505	10	2	1	0	.0300	15.00	825.00	.000	.0(
20	NO. 27.520	5	2	1	0	.0330	400.00	825.00	.000	.0(
21	NO. 28.0	5	2	1	0	.0300	150.00	825.00	.000	.0(

*** Cross sectional form ***

Segment KUKAN	KEIJYO		BO(R)	M1	N1	B1	B2	HP(B3)	M2	N2
1	2	1	100.000	2.000	2.000	.001	.001	2.000	2.000	2.000
			40.000	2.000	2.000	40.000	20.000	2.000	2.000	2.000
2	2	1	40.000	2.000	2.000	40.000	20.000	2.000	2.000	2.000
			40.000	2.000	2.000	20.000	15.000	2.000	2.000	2.000
3	2	1	40.000	2.000	2.000	20.000	15.000	2.000	2.000	2.000
			40.000	2.000	2.000	10.000	16.000	3.000	2.000	2.000
4	2	1	40.000	2.000	2.000	10.000	16.000	3.000	2.000	2.000
			40.000	2.000	2.000	6.000	16.000	3.500	2.000	2.000
5	2	1	40.000	2.000	2.000	6.000	16.000	3.500	2.000	2.000
			40.000	2.000	2.000	6.000	16.000	3.500	2.000	2.000
6	2	1	40.000	2.000	2.000	6.000	16.000	3.500	2.000	2.000
			13.000	2.000	2.000	23.000	22.000	5.000	2.000	2.000
7	2	1	13.000	2.000	2.000	23.000	22.000	5.000	2.000	2.000
			13.000	2.000	2.000	23.000	22.000	5.000	2.000	2.000
8	2	1	13.000	2.000	2.000	23.000	22.000	5.000	2.000	2.000
			12.000	2.000	2.000	26.000	21.000	4.800	2.000	2.000
9	2	1	12.000	2.000	2.000	26.000	21.000	4.800	2.000	2.000
			12.000	2.000	2.000	26.000	21.000	4.800	2.000	2.000
10	2	1	12.000	2.000	2.000	26.000	21.000	4.800	2.000	2.000
			10.000	2.000	2.000	12.000	8.000	5.000	2.000	2.000
11	2	1	10.000	2.000	2.000	12.000	8.000	5.000	2.000	2.000
			10.000	2.000	2.000	12.000	8.000	5.000	2.000	2.000
12	2	1	10.000	2.000	2.000	12.000	8.000	5.000	2.000	2.000
			8.000	2.000	2.000	1.500	1.500	3.000	2.000	2.000
13	2	1	8.000	2.000	2.000	1.500	1.500	3.000	2.000	2.000

			8.000	2.000	2.000	1.500	1.500	3.000	2.000	2.000
14	2	1	8.000	2.000	2.000	1.500	1.500	3.000	2.000	2.000
			15.500	.500	.500	3.000	3.000	6.910	6.910	.001
15	2	1	15.500	.500	.500	3.000	3.000	6.910	.001	.001
			8.000	2.000	2.000	1.500	1.500	3.000	2.000	2.000
16	2	1	8.000	2.000	2.000	1.500	1.500	3.000	2.000	2.000
			8.000	2.000	2.000	1.500	1.500	3.000	2.000	2.000
17	2	1	8.000	2.000	2.000	1.500	1.500	3.000	2.000	2.000
			8.000	.790	.790	4.700	4.700	3.800	1.000	1.000
18	2	1	8.000	.790	.790	4.000	4.000	3.800	1.000	1.000
			8.000	.790	.790	4.000	4.000	3.800	1.000	1.000
19	2	1	8.000	.790	.790	4.000	4.000	3.800	1.000	1.000
			8.000	2.000	2.000	1.500	1.500	3.000	2.000	2.000
20	2	1	8.000	2.000	2.000	1.500	1.500	3.000	2.000	2.000
			8.000	2.000	2.000	1.500	1.500	3.000	2.000	2.000
21	2	1	8.000	2.000	2.000	1.500	1.500	3.000	2.000	2.000
			10.000	2.000	2.000	1.500	1.500	3.000	2.000	2.000

* LOSS DATA *

KUKAN LOSS TYPE FL1 FL2

1

CASE 1: PERCUT RIVER (Q=320m3/s, Ho=1.30)-FINAL- 1995/12/24 by PERCUT9.DAT PAGE = 2

NO.	H	A	R	V	N	Q	DX	FROUD	IE	Z	H-Z
NO. -0.50	1.300F	329.22	2.891	.972	.0330	320.00	.00	.1903	.2498E-03	-1.800	3.100
+ 250.00	1.365F	276.52	2.422	1.157	.0330	320.00	250.00	.2475	.4483E-03	-1.800	3.165
+ 500.00	1.491F	230.77	2.012	1.387	.0330	320.00	250.00	.3253	.8246E-03	-1.799	3.291
NO. 0.00	1.491F	230.76	2.012	1.387	.0330	320.00	.00	.3254	.8247E-03	-1.799	3.291
+ 200.00	1.651F	238.88	2.165	1.340	.0330	320.00	200.00	.3027	.6976E-03	-1.775	3.425
+ 400.00	1.786F	243.23	2.299	1.316	.0330	320.00	200.00	.2884	.6213E-03	-1.750	3.536
+ 600.00	1.907F	245.06	2.421	1.306	.0330	320.00	200.00	.2788	.5713E-03	-1.724	3.632
+ 800.00	2.018F	245.03	2.536	1.306	.0330	320.00	200.00	.2723	.5371E-03	-1.699	3.717
+1000.00	2.122F	243.54	2.648	1.314	.0330	320.00	200.00	.2679	.5132E-03	-1.674	3.796
NO. 1.00	2.122F	243.57	2.648	1.314	.0330	320.00	.00	.2678	.5131E-03	-1.674	3.796
+ 150.00	2.190F	224.50	2.560	1.425	.0330	320.00	150.00	.2954	.6319E-03	-1.656	3.846
+ 300.00	2.276F	210.88	2.583	1.517	.0330	320.00	150.00	.3163	.7074E-03	-1.637	3.913
NO. 1.30	2.276F	210.86	2.583	1.518	.0330	320.00	.00	.3164	.7075E-03	-1.637	3.913
+ 150.00	2.376F	209.56	2.771	1.527	.0330	320.00	150.00	.3160	.6524E-03	-1.618	3.994
+ 300.00	2.469F	208.36	2.956	1.536	.0330	320.00	150.00	.3154	.6055E-03	-1.599	4.068
NO. 1.60	2.469F	208.34	2.956	1.536	.0330	320.00	.00	.3154	.6056E-03	-1.599	4.068
+ 200.00	2.595F	216.30	2.956	1.479	.0330	320.00	200.00	.2989	.5619E-03	-1.574	4.170
+ 400.00	2.712F	223.53	2.956	1.432	.0330	320.00	200.00	.2852	.5261E-03	-1.549	4.261
+ 600.00	2.821F	230.17	2.956	1.390	.0330	320.00	200.00	.2735	.4962E-03	-1.524	4.345
+ 800.00	2.923F	236.31	2.956	1.354	.0330	320.00	200.00	.2635	.4707E-03	-1.499	4.422
+1000.00	3.020F	242.04	2.956	1.322	.0330	320.00	200.00	.2546	.4487E-03	-1.474	4.494
+1200.00	3.111F	247.33	3.002	1.294	.0330	320.00	200.00	.2469	.4210E-03	-1.449	4.560
+1400.00	3.196F	252.17	3.051	1.269	.0330	320.00	200.00	.2402	.3964E-03	-1.424	4.621
+1600.00	3.276F	256.63	3.095	1.247	.0330	320.00	200.00	.2343	.3754E-03	-1.399	4.676
+1800.00	3.352F	260.75	3.136	1.227	.0330	320.00	200.00	.2290	.3573E-03	-1.374	4.727
+2000.00	3.425F	264.58	3.174	1.209	.0330	320.00	200.00	.2243	.3415E-03	-1.349	4.774
+2200.00	3.494F	268.16	3.210	1.193	.0330	320.00	200.00	.2201	.3275E-03	-1.324	4.818
+2400.00	3.560F	271.51	3.243	1.179	.0330	320.00	200.00	.2162	.3152E-03	-1.299	4.859
+2600.00	3.624F	274.67	3.274	1.165	.0330	320.00	200.00	.2127	.3041E-03	-1.274	4.898
+2800.00	3.685F	277.65	3.303	1.153	.0330	320.00	200.00	.2095	.2941E-03	-1.249	4.935
+3000.00	3.745F	280.46	3.330	1.141	.0330	320.00	200.00	.2065	.2851E-03	-1.224	4.969
NO. 4.60	3.745F	280.47	3.330	1.141	.0330	320.00	.00	.2065	.2851E-03	-1.224	4.969
+ 100.00	3.712F	195.93	3.269	1.633	.0330	320.00	100.00	.3484	.5988E-03	-1.141	4.853
+ 200.00	3.436F	98.79	2.985	3.239	.0330	320.00	100.00	.6076	.2658E-02	-1.058	4.494
NO. 4.80	3.436F	98.82	2.986	3.238	.0330	320.00	.00	.6074	.2656E-02	-1.058	4.494
+ 200.00	4.021F	112.11	3.206	2.854	.0330	320.00	200.00	.5161	.1877E-02	-.891	4.912
+ 400.00	4.461F	129.55	3.252	2.470	.0330	320.00	200.00	.6452	.1379E-02	-.725	5.186
+ 600.00	4.768F	140.62	3.252	2.276	.0330	320.00	200.00	.5726	.1170E-02	-.558	5.326
+ 800.00	5.017F	147.18	3.252	2.174	.0330	320.00	200.00	.5358	.1068E-02	-.391	5.408
+1000.00	5.240F	151.66	3.252	2.110	.0330	320.00	200.00	.5130	.1006E-02	-.224	5.464
+1200.00	5.447F	154.92	3.252	2.066	.0330	320.00	200.00	.4974	.9643E-03	-.058	5.505
+1400.00	5.645F	157.37	3.252	2.033	.0330	320.00	200.00	.4862	.9345E-03	.109	5.536
+1600.00	5.835F	159.26	3.252	2.009	.0330	320.00	200.00	.4778	.9125E-03	.276	5.559
+1800.00	6.020F	160.72	3.252	1.991	.0330	320.00	200.00	.4715	.8959E-03	.442	5.578
+2000.00	6.201F	161.88	3.252	1.977	.0330	320.00	200.00	.4666	.8831E-03	.609	5.592
+2200.00	6.379F	162.80	3.252	1.966	.0330	320.00	200.00	.4628	.8732E-03	.775	5.603

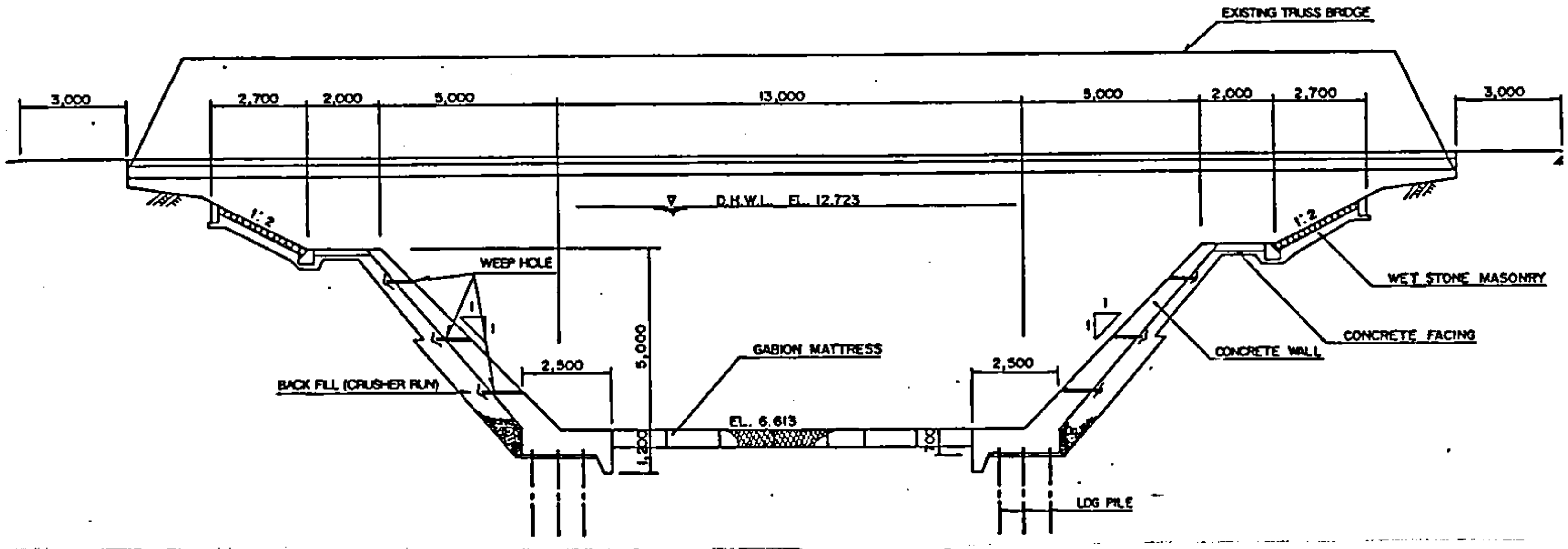
NO.	H	A	R	V	N	Q	DX	FROUD	IE	Z	H-Z
+2400.00	6.555F	163.54	3.252	1.957	.0330	320.00	200.00	.4598	.8654E-03	.942	5.613
NO.7.20	6.555F	163.54	3.252	1.957	.0330	320.00	.00	.4598	.8653E-03	.942	5.613
+ 100.00	6.649F	166.86	3.175	1.918	.0330	320.00	100.00	.4476	.8582E-03	1.026	5.624
+ 200.00	6.743F	170.32	3.098	1.879	.0330	320.00	100.00	.4355	.8511E-03	1.109	5.634
NO.7.40	6.743F	170.33	3.098	1.879	.0330	320.00	.00	.4355	.8511E-03	1.109	5.634
+ 200.00	6.914F	170.65	3.098	1.875	.0330	320.00	200.00	.4343	.8479E-03	1.275	5.638
+ 400.00	7.084F	170.91	3.098	1.872	.0330	320.00	200.00	.4333	.8453E-03	1.442	5.642
+ 600.00	7.253F	171.13	3.098	1.870	.0330	320.00	200.00	.4325	.8431E-03	1.609	5.644
+ 800.00	7.422F	171.31	3.098	1.868	.0330	320.00	200.00	.4319	.8414E-03	1.776	5.646
+1000.00	7.590F	171.46	3.098	1.866	.0330	320.00	200.00	.4314	.8399E-03	1.942	5.648
+1200.00	7.759F	171.58	3.098	1.865	.0330	320.00	200.00	.4309	.8387E-03	2.109	5.650
+1400.00	7.926F	171.68	3.098	1.864	.0330	320.00	200.00	.4306	.8378E-03	2.276	5.651
+1600.00	8.094F	171.76	3.098	1.863	.0330	320.00	200.00	.4303	.8370E-03	2.442	5.652
+1800.00	8.262F	171.82	3.098	1.862	.0330	320.00	200.00	.4300	.8363E-03	2.609	5.653
+2000.00	8.429F	171.88	3.098	1.862	.0330	320.00	200.00	.4298	.8358E-03	2.776	5.653
+2200.00	8.596F	171.92	3.098	1.861	.0330	320.00	200.00	.4297	.8354E-03	2.942	5.654
+2400.00	8.763F	171.96	3.098	1.861	.0330	320.00	200.00	.4295	.8350E-03	3.109	5.654
+2600.00	8.930F	171.99	3.098	1.861	.0330	320.00	200.00	.4294	.8347E-03	3.276	5.655
+2800.00	9.097F	172.02	3.098	1.860	.0330	320.00	200.00	.4293	.8345E-03	3.442	5.655
+3000.00	9.264F	172.04	3.098	1.860	.0330	320.00	200.00	.4292	.8343E-03	3.609	5.655
+3200.00	9.431F	172.05	3.098	1.860	.0330	320.00	200.00	.4292	.8341E-03	3.776	5.656
NO.10.6	9.431F	172.06	3.098	1.860	.0330	320.00	.00	.4292	.8341E-03	3.776	5.656
+ 100.00	9.452F	144.77	3.097	2.210	.0330	320.00	100.00	.5028	.1179E-02	3.897	5.555
+ 200.00	9.493F	124.19	3.090	2.577	.0330	320.00	100.00	.5581	.1606E-02	4.018	5.475
NO.10.8	9.493F	124.20	3.090	2.577	.0330	320.00	.00	.5580	.1606E-02	4.018	5.475
+ 203.67	9.836F	129.19	3.090	2.477	.0330	320.00	203.67	.5279	.1484E-02	4.265	5.571
+ 407.33	10.148F	132.61	3.090	2.413	.0330	320.00	203.67	.5089	.1409E-02	4.512	5.636
+ 611.00	10.441F	135.07	3.090	2.369	.0330	320.00	203.67	.4959	.1358E-02	4.759	5.683
+ 814.67	10.722F	136.88	3.090	2.338	.0330	320.00	203.67	.4867	.1322E-02	5.005	5.717
+1018.33	10.995F	138.25	3.090	2.315	.0330	320.00	203.67	.4800	.1296E-02	5.252	5.743
+1222.00	11.262F	139.29	3.090	2.297	.0330	320.00	203.67	.4750	.1277E-02	5.499	5.763
+1425.67	11.524F	140.09	3.090	2.284	.0330	320.00	203.67	.4712	.1262E-02	5.746	5.778
+1629.33	11.782F	140.71	3.090	2.274	.0330	320.00	203.67	.4683	.1251E-02	5.993	5.789
+1833.00	12.038F	141.20	3.090	2.266	.0330	320.00	203.67	.4660	.1243E-02	6.240	5.798
+2036.67	12.292F	141.57	3.090	2.260	.0330	320.00	203.67	.4643	.1236E-02	6.487	5.806
+2240.33	12.545F	141.87	3.090	2.256	.0330	320.00	203.67	.4630	.1231E-02	6.733	5.811
+2444.00	12.796F	142.10	3.090	2.252	.0330	320.00	203.67	.4619	.1227E-02	6.980	5.815
NO.13.244	12.796F	142.10	3.090	2.252	.0330	320.00	.00	.4619	.1227E-02	6.980	5.815
+ 78.00	12.888F	140.77	3.027	2.273	.0330	320.00	78.00	.4246	.1285E-02	7.075	5.813
+ 156.00	12.888F	119.30	3.262	2.682	.0330	320.00	78.00	.4788	.1619E-02	7.169	5.718
NO.13.4	12.888F	119.30	3.262	2.682	.0330	320.00	.00	.4788	.1619E-02	7.169	5.718
+ 458.00	13.620F	125.38	3.355	2.552	.0330	320.00	458.00	.4491	.1412E-02	7.725	5.896
+ 916.00	14.264F	128.44	3.401	2.491	.0330	320.00	458.00	.4353	.1321E-02	8.280	5.984
+1374.00	14.867F	130.13	3.427	2.459	.0330	320.00	458.00	.4280	.1275E-02	8.835	6.032
+1832.00	15.450F	131.11	3.441	2.441	.0330	320.00	458.00	.4239	.1249E-02	9.390	6.060
+2290.00	16.021F	131.68	3.450	2.430	.0330	320.00	458.00	.4216	.1234E-02	9.945	6.076
+2748.00	16.586F	132.02	3.455	2.424	.0330	320.00	458.00	.4202	.1225E-02	10.500	6.086
+3206.00	17.147F	132.23	3.458	2.420	.0330	320.00	458.00	.4193	.1220E-02	11.056	6.092
+3664.00	17.706F	132.35	3.459	2.418	.0330	320.00	458.00	.4188	.1217E-02	11.611	6.095
+4122.00	18.263F	132.42	3.460	2.416	.0330	320.00	458.00	.4185	.1215E-02	12.166	6.097
+4580.00	18.819F	132.47	3.461	2.416	.0330	320.00	458.00	.4183	.1214E-02	12.721	6.098
NO.17.98	18.819F	132.46	3.461	2.416	.0300	320.00	.00	.4184	.1003E-02	12.721	6.098
+ 10.00	18.781F	123.48	3.220	2.592	.0300	320.00	10.00	.4596	.1271E-02	12.733	6.048
+ 20.00	18.695F	109.93	3.816	2.911	.0300	320.00	10.00	.4308	.1279E-02	12.745	5.950
NO.18.00	18.695F	109.93	3.816	2.911	.0300	320.00	.00	.4308	.1279E-02	12.745	5.950
+ 10.00	18.791F	120.96	3.440	2.646	.0300	320.00	10.00	.4472	.1213E-02	12.757	6.033
+ 20.00	18.867F	132.43	3.460	2.416	.0300	320.00	10.00	.4185	.1004E-02	12.769	6.097
NO.18.02	18.867F	132.43	3.461	2.416	.0330	320.00	.00	.4185	.1215E-02	12.769	6.097
+ 942.50	20.011F	132.50	3.462	2.415	.0330	320.00	942.50	.4182	.1213E-02	13.912	6.099
+1885.00	21.154F	132.52	3.462	2.415	.0330	320.00	942.50	.4181	.1212E-02	15.054	6.100
+2827.50	22.297F	132.53	3.462	2.415	.0330	320.00	942.50	.4181	.1212E-02	16.197	6.100
+3770.00	23.439F	132.53	3.462	2.414	.0330	320.00	942.50	.4181	.1212E-02	17.339	6.100
+4712.50	24.582F	132.54	3.462	2.414	.0330	320.00	942.50	.4181	.1212E-02	18.482	6.100
+5655.00	25.724F	132.54	3.462	2.414	.0330	320.00	942.50	.4181	.1212E-02	19.624	6.100
+6597.50	26.867F	132.54	3.462	2.414	.0330	320.00	942.50	.4181	.1212E-02	20.766	6.100
+7540.00	28.009F	132.54	3.462	2.414	.0330	320.00	942.50	.4181	.1212E-02	21.909	6.100
+8482.50	29.152F	132.54	3.462	2.414	.0330	320.00	942.50	.4181	.1212E-02	23.051	6.100
+9425.00	30.294F	132.54	3.462	2.414	.0330	320.00	942.50	.4181	.1212E-02	24.194	6.100
NO.27.445	30.294F	132.53	3.462	2.415	.0300	320.00	.00	.4181	.1002E-02	24.194	6.100
+ 4.00	30.263F	125.97	3.402	2.540	.0300	320.00	4.00	.4412	.1135E-02	24.199	6.065
+ 8.00	30.222F	118.70	3.315	2.696	.0300	320.00	4.00	.4709	.1323E-02	24.203	6.019
+ 12.00	30.167F	110.63	3.194	2.893	.0300	320.00	4.00	.5101	.1601E-02	24.208	5.958
+ 16.00	30.086F	101.57	3.026	3.151	.0300	320.00	4.00	.5640	.2041E-02	24.213	5.873
+ 20.00	29.960F	91.02	2.794	3.516	.0300	320.00	4.00	.6449	.2827E-02	24.218	5.742

NO.	H	A	R	V	N	Q	DX	FROUD	IE	Z	H-Z
NO. 27.465	29.881F	86.28	2.787	3.709	.0250	320.00	.00	.6786	.2192E-02	24.218	5.663
+ 4.00	29.893F	86.47	2.791	3.701	.0250	320.00	4.00	.6766	.2178E-02	24.223	5.671
+ 8.00	29.905F	86.65	2.795	3.693	.0250	320.00	4.00	.6746	.2165E-02	24.228	5.678
+ 12.00	29.917F	86.83	2.799	3.686	.0250	320.00	4.00	.6727	.2152E-02	24.232	5.685
+ 16.00	29.929F	87.00	2.803	3.678	.0250	320.00	4.00	.6709	.2139E-02	24.237	5.691
+ 20.00	29.940F	87.17	2.807	3.671	.0250	320.00	4.00	.6691	.2127E-02	24.242	5.698
+ 24.00	29.952F	87.34	2.811	3.664	.0250	320.00	4.00	.6673	.2115E-02	24.247	5.705
+ 28.00	29.963F	87.51	2.815	3.657	.0250	320.00	4.00	.6655	.2103E-02	24.252	5.711
+ 32.00	29.974F	87.67	2.818	3.650	.0250	320.00	4.00	.6638	.2091E-02	24.257	5.717
+ 36.00	29.985F	87.84	2.822	3.643	.0250	320.00	4.00	.6622	.2080E-02	24.262	5.724
+ 40.00	29.996F	87.99	2.825	3.637	.0250	320.00	4.00	.6605	.2069E-02	24.266	5.730
NO. 27.505	29.996F	87.99	2.825	3.637	.0300	320.00	.00	.6605	.2980E-02	24.266	5.730
+ 1.50	30.094F	94.18	2.959	3.398	.0300	320.00	1.50	.6086	.2446E-02	24.268	5.826
+ 3.00	30.168F	99.76	3.068	3.208	.0300	320.00	1.50	.5687	.2077E-02	24.270	5.898
+ 4.50	30.227F	104.94	3.159	3.049	.0300	320.00	1.50	.5365	.1806E-02	24.272	5.955
+ 6.00	30.275F	109.82	3.235	2.914	.0300	320.00	1.50	.5096	.1597E-02	24.274	6.001
+ 7.50	30.315F	114.44	3.299	2.796	.0300	320.00	1.50	.4868	.1433E-02	24.276	6.039
+ 9.00	30.349F	118.85	3.353	2.692	.0300	320.00	1.50	.4671	.1300E-02	24.277	6.071
+ 10.50	30.378F	123.06	3.399	2.600	.0300	320.00	1.50	.4500	.1191E-02	24.279	6.099
+ 12.00	30.403F	127.10	3.437	2.518	.0300	320.00	1.50	.4348	.1100E-02	24.281	6.122
+ 13.50	30.426F	130.97	3.468	2.443	.0300	320.00	1.50	.4214	.1024E-02	24.283	6.143
+ 15.00	30.445F	134.68	3.494	2.376	.0300	320.00	1.50	.4095	.9584E-03	24.285	6.161
NO. 27.520	30.445F	134.68	3.494	2.376	.0330	320.00	.00	.4095	.1160E-02	24.285	6.161
+ 80.00	30.538F	134.51	3.491	2.379	.0330	320.00	80.00	.4102	.1164E-02	24.382	6.156
+ 160.00	30.630F	134.35	3.489	2.382	.0330	320.00	80.00	.4108	.1168E-02	24.479	6.152
+ 240.00	30.723F	134.20	3.487	2.384	.0330	320.00	80.00	.4114	.1171E-02	24.576	6.147
+ 320.00	30.816F	134.07	3.485	2.387	.0330	320.00	80.00	.4119	.1174E-02	24.672	6.144
+ 400.00	30.910F	133.95	3.483	2.389	.0330	320.00	80.00	.4124	.1177E-02	24.769	6.140
NO. 28.0	30.910F	133.95	3.483	2.389	.0300	320.00	.00	.4124	.9730E-03	24.769	6.140
+ 30.00	30.950F	136.55	3.512	2.343	.0300	320.00	30.00	.4030	.9257E-03	24.806	6.144
+ 60.00	30.988F	139.08	3.540	2.301	.0300	320.00	30.00	.3943	.8831E-03	24.842	6.146
+ 90.00	31.025F	141.53	3.566	2.261	.0300	320.00	30.00	.3862	.8445E-03	24.879	6.146
+ 120.00	31.059F	143.91	3.591	2.224	.0300	320.00	30.00	.3787	.8093E-03	24.915	6.144
+ 150.00	31.091F	146.22	3.614	2.188	.0300	320.00	30.00	.3716	.7772E-03	24.951	6.140

Uniform Flow Calculation **Upper Percut River** **Cross Section at TITI RUNTUH BRIDGE**

Roughness Coefficient	n =	0.03	
River Bed Slope	i =	0.0012121	(I = 1 / 825)
River Bed Width	B1 =	13.00 m	
Height 1	H1 =	5.00 m	
Height 2	H2 =	1.11	
Slope Gradient 1	m1 =	1.00	
Slope Gradient 2	m2 =	2.00	
Width of Berm	b =	2.00 m	
Freeboard	Fb =	0.80 m	

Hydraulic Radius (m) : R = A/P
Velocity (m/s) : v = 1/n * R ^(2/3) * I^(1/2)
Discharge (m3/s) : Q = v x A
Design Discharge
Immediate Plan Qa = 320.00 (m3/s)
Urgent Plan Qa = 320.00 (m3/s)



Calculation Table

	H	B1	B2	B3	A1	A2	P	R	n	i	v	Q
	m	m	m	m	m2	m2	m	m			m/s	m3/s
1	6.11	13.00	23.00	31.44	90.00	122.43	36.11	3.39	0.030	0.001212	2.62	320.70
2	6.11	13.00	23.00	31.44	90.00	122.43	36.11	3.39	0.030	0.001212	2.62	320.70
3	5.00	13.00	23.00		90.00		27.14	3.32	0.030	0.001212	2.58	232.25

1. Immediate Plan : Q = 320.70 > 320.00 (m3/s) OK
2. Urgent Plan : Q = 320.70 > 320.00 (m3/s) OK

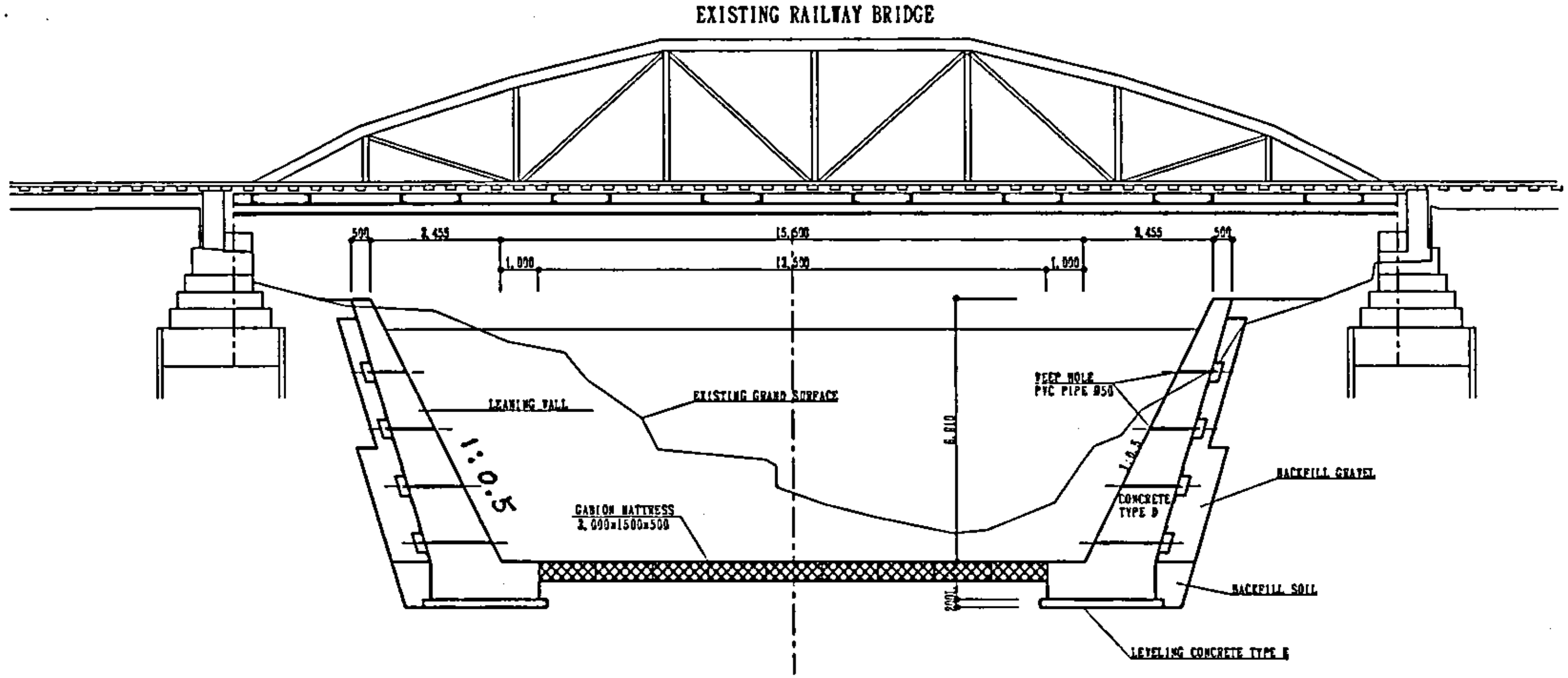
Fig. 1. 1. 2 HYDRAULIC DESIGN CALCULATION AT TITI RUNTUH BRIDGE

Uniform Flow Calculation **Upper Percut River**

Roughness Coefficient	n =	0.03	
River Bed Slope	i =	0.0012121	(I = 1 / 825)
River Bed Width	B1 =	15.50 m	
Height 1	H1 =	6.91 m	
Height 2	H2 =	0.00	
Slope Gradient 1	m1 =	0.50	
Slope Gradient 2	m2 =	0.00	
Width of Berm	b =	0.00 m	
Freeboard	Fb =	0.80 m	

Cross Section at RAILWAY BRIDGE

Hydraulic Radius	(m) :	R = A/P		
Velocity	(m/s) :	v = 1/n * R ^(2/3) * I^(1/2)		
Discharge	(m3/s) :	Q = v x A		
Design Discharge				
Immediate Plan		Qa =	320.00	(m3/s)
Urgent Plan		Qa =	320.00	(m3/s)



Calculation Table

	H	B1	B2	B3	A1	A2	P	R	n	i	v	Q
	m	m	m	m	m2	m2	m	m			m/s	m3/s
1	6.11	15.50	21.61		113.37		29.16	3.89	0.030	0.001212	2.87	325.29
2	6.11	15.50	21.61		113.37		29.16	3.89	0.030	0.001212	2.87	325.29
3	5.00	15.50	20.50		90.00		26.68	3.37	0.030	0.001212	2.61	234.92

1. Immediate Plan : Q = 325.29 > 320.00 (m3/s) OK
2. Urgent Plan : Q = 325.29 > 320.00 (m3/s) OK

Fig. 1. 1. 2 HYDRAULIC DESIGN CALCULATION AT RAILWAY BRIDGE

Calculation Sheet - 1. 1. 2

1

Railway Bridge

** INPUT DATA **

* BAISIC DATA *

KUKAN-SU = 5 ALPHA = 1.10 QO = 320.00 M3/S HO = 18.279 M ZO = 12.169 M

JCO = 0 KEY = 0 IPT = 0

* KUKAN DATA *

	DANMEN NO.	BUNKATSU SUU	DANMEN KEIJYO		LOSS TYPE	SODO KEISU	KUKAN KYORI(M)	KASYO KOOBAI(1/I)	RAKUSA (M)	RYUNYU RYO(M3/S)
1	NO.-150	5	2	0	0	.0330	100.00	825.00	.000	.00
2	NO.- 50	5	2	1	0	.0300	25.00	825.00	.000	.00
3	NO.- 25	5	2	0	0	.0300	50.00	825.00	.000	.00
4	NO.+ 25	5	2	1	0	.0300	25.00	825.00	.000	.00
5	NO.-150	5	2	0	0	.0330	100.00	825.00	.000	.00

* KEIJYO DATA *

KUKAN	KEIJYO		BO(R)	M1	N1	B1	B2	HP(B3)	M2	N2
1	2	0	8.000	2.000	2.000	1.500	1.500	3.000	2.000	2.000
2	2	1	8.000	2.000	2.000	1.500	1.500	3.000	2.000	2.000
			15.500	.500	.500	.000	.000	3.000	.500	.500
3	2	0	15.500	.500	.500	.000	.000	3.000	.500	.500
4	2	1	15.500	.500	.500	.000	.000	3.000	.500	.500

			8.000	2.000	2.000	1.500	1.500	3.000	2.000	2.000
5	2	0	8.000	2.000	2.000	1.500	1.500	3.000	2.000	2.000

* LOSS DATA *

KUKAN LOSS TYPE FL1 FL2

1

Railway Bridge PAGE = 2

NO.	H	A	R	V	N	Q	DX	FROUD	IE	Z	H-Z
NO.-150	18.279•	132.87	3.467	2.408	.0330	320.00	.00	.4167	.1204E-02	12.169	6.110
+ 20.00	18.303•	132.87	3.467	2.408	.0330	320.00	20.00	.4167	.1204E-02	12.193	6.110
+ 40.00	18.327•	132.86	3.467	2.409	.0330	320.00	20.00	.4168	.1204E-02	12.217	6.110
+ 60.00	18.351•	132.85	3.467	2.409	.0330	320.00	20.00	.4168	.1204E-02	12.242	6.109
+ 80.00	18.375•	132.85	3.467	2.409	.0330	320.00	20.00	.4168	.1204E-02	12.266	6.109
+ 100.00	18.399•	132.84	3.467	2.409	.0330	320.00	20.00	.4168	.1204E-02	12.290	6.109
NO.- 50	18.399•	132.84	3.467	2.409	.0300	320.00	.00	.4168	.9954E-03	12.290	6.109
+ 5.00	18.380•	128.11	3.569	2.498	.0300	320.00	5.00	.4220	.1030E-02	12.296	6.084
+ 10.00	18.358•	123.46	3.670	2.592	.0300	320.00	5.00	.4263	.1068E-02	12.302	6.056
+ 15.00	18.334•	118.89	3.761	2.691	.0300	320.00	5.00	.4294	.1115E-02	12.308	6.026
+ 20.00	18.308•	114.43	3.821	2.796	.0300	320.00	5.00	.4307	.1178E-02	12.314	5.993
+ 25.00	18.278•	110.10	3.820	2.907	.0300	320.00	5.00	.4299	.1273E-02	12.321	5.958
NO.- 25	18.278•	110.10	3.820	2.907	.0300	320.00	.00	.4299	.1273E-02	12.321	5.958
+ 10.00	18.291•	110.11	3.820	2.906	.0300	320.00	10.00	.4298	.1273E-02	12.333	5.959
+ 20.00	18.304•	110.13	3.820	2.906	.0300	320.00	10.00	.4297	.1272E-02	12.345	5.959
+ 30.00	18.317•	110.14	3.821	2.905	.0300	320.00	10.00	.4296	.1272E-02	12.357	5.960
+ 40.00	18.330•	110.16	3.821	2.905	.0300	320.00	10.00	.4296	.1271E-02	12.369	5.961
+ 50.00	18.343•	110.17	3.821	2.904	.0300	320.00	10.00	.4295	.1271E-02	12.381	5.962
NO.+ 25	18.343•	110.17	3.821	2.904	.0300	320.00	.00	.4295	.1271E-02	12.381	5.962
+ 5.00	18.384•	114.53	3.823	2.794	.0300	320.00	5.00	.4303	.1176E-02	12.387	5.997

+ 10.00	18.422·	118.97	3.762	2.690	.0300	320.00	5.00	.4290	.1113E-02	12.393	6.029
+ 15.00	18.457·	123.50	3.671	2.591	.0300	320.00	5.00	.4261	.1067E-02	12.399	6.057
+ 20.00	18.488·	128.09	3.569	2.498	.0300	320.00	5.00	.4221	.1030E-02	12.405	6.083
+ 25.00	18.518·	132.74	3.465	2.411	.0300	320.00	5.00	.4172	.9976E-03	12.411	6.106
NO.-150	18.518·	132.74	3.465	2.411	.0330	320.00	.00	.4172	.1207E-02	12.411	6.106
+ 20.00	18.542·	132.73	3.465	2.411	.0330	320.00	20.00	.4173	.1207E-02	12.436	6.106
+ 40.00	18.566·	132.73	3.465	2.411	.0330	320.00	20.00	.4173	.1207E-02	12.460	6.106
+ 60.00	18.590·	132.73	3.465	2.411	.0330	320.00	20.00	.4173	.1207E-02	12.484	6.106
+ 80.00	18.614·	132.72	3.465	2.411	.0330	320.00	20.00	.4173	.1207E-02	12.508	6.106
+ 100.00	18.638·	132.72	3.465	2.411	.0330	320.00	20.00	.4173	.1208E-02	12.533	6.106

Water Level Profile around National Road Bridge

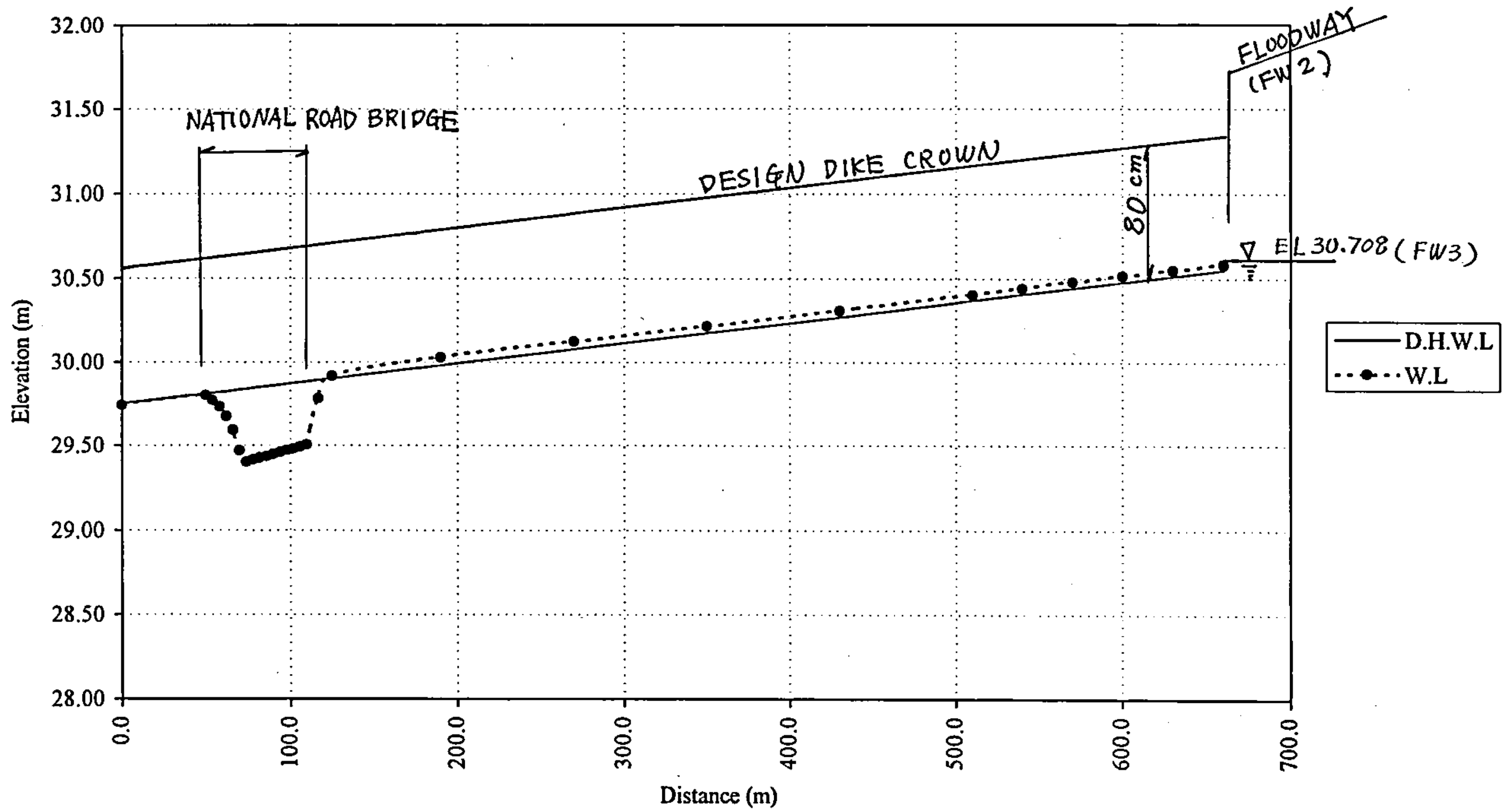


Fig. 1. 1. 4 WATER SURFACE PROFILE OF UPSTREAM CHANNEL FROM NATIONAL ROAD BRIDGE

Table 1. 1. 1 WATER LEVEL AND VELOSITY OF UPSTREAM CHANNEL FROM NATIONAL ROAD BRIDGE

<u>Water Level Profile and Velosity</u>
(National Road Bridge to Floodway)

	Distance	D.H.W.L	W.L	Difference	Velosity
	0.0	29.750	29.740	0.01	2.41
	50.0	29.811	29.801	0.01	2.42
	54.0	29.815	29.770	0.04	2.54
	58.0	29.820	29.729	0.09	2.70
	62.0	29.825	29.673	0.15	2.89
	66.0	29.830	29.593	0.24	3.15
	70.0	29.835	29.467	0.37	3.52
	74.0	29.840	29.401	0.44	3.70
	78.0	29.845	29.413	0.43	3.69
	82.0	29.849	29.424	0.43	3.69
	86.0	29.854	29.435	0.42	3.68
National Load	90.0	29.859	29.447	0.41	3.67
	94.0	29.864	29.459	0.40	3.66
	98.0	29.869	29.470	0.40	3.66
	102.0	29.874	29.481	0.39	3.65
	106.0	29.878	29.492	0.39	3.64
	110.0	29.883	29.503	0.38	3.64
	117.0	29.892	29.783	0.11	2.91
	125.0	29.902	29.914	-0.01	2.44
	190.0	29.980	30.026	-0.05	2.38
	270.0	30.077	30.119	-0.04	2.38
	350.0	30.174	30.211	-0.04	2.38
	430.0	30.271	30.305	-0.03	2.39
	510.0	30.368	30.398	-0.03	2.39
	540.0	30.405	30.439	-0.03	2.34
	570.0	30.441	30.477	-0.04	2.30
	600.0	30.477	30.513	-0.04	2.26
	630.0	30.514	30.548	-0.03	2.22
FW2	660.0	30.550	30.580	-0.03	2.19
FW3	766.0	30.730	30.708	0.02	

(Non - Uniform flow calculation)

Hydraulics at Bend

(1) Estimation of Super-Elevation at Bend

The rise in water level at a bending section is an unavoidable phenomenon. For designing the cross section of the channel at bending section, A maximum super-elevation at bend was estimated as follows:

(a) Conditions

- Channel Cross-section: Standard cross section
- Riverbed slope gradient: 1 / 825
- Width of channel: 35.44 m (width of water surface)
- Average flow velocity: 2.42 m/s
- Curve radius of river channel 40 m (minimum curve radius)

(b) Calculation formula

$$\Delta h = m \cdot \frac{b \cdot v^2}{g \cdot \gamma_c}$$

where, $b = 35.44$ m (width of channel)

$v = 2.42$ m/s (average flow velocity)

$\gamma_c = 40$m (curve radius)

$g = 9.8$ m/s^2 (acceleration of gravity)

$m = 1$ (coefficient of flow)

therefore, $$\Delta h = 1 \times \frac{35.44 \times 2.42^2}{9.8 \times 40} = 0.53 \text{ m}$$

(2) Required width of channel at bend

The maximum super-elevation of flow at bend is calculated to be 0.53 m. This means that the water level at concave side of a vend is 0.53 m higher than the average water level. In order to lower the excess water rise to the average high water level, the flow area of channel shall be enlarged. Degree of enlarging channel width is calculated based on the uniform flow calculation formula as shown the Fig.1.1.5..

Result:

Width of original water surface = 35.44 m

Required width of water surface = 56.32 m

Enlarging rate = (56.32 - 35.44) / 35.44 = 0.59, rounded to 0.6 (60%)

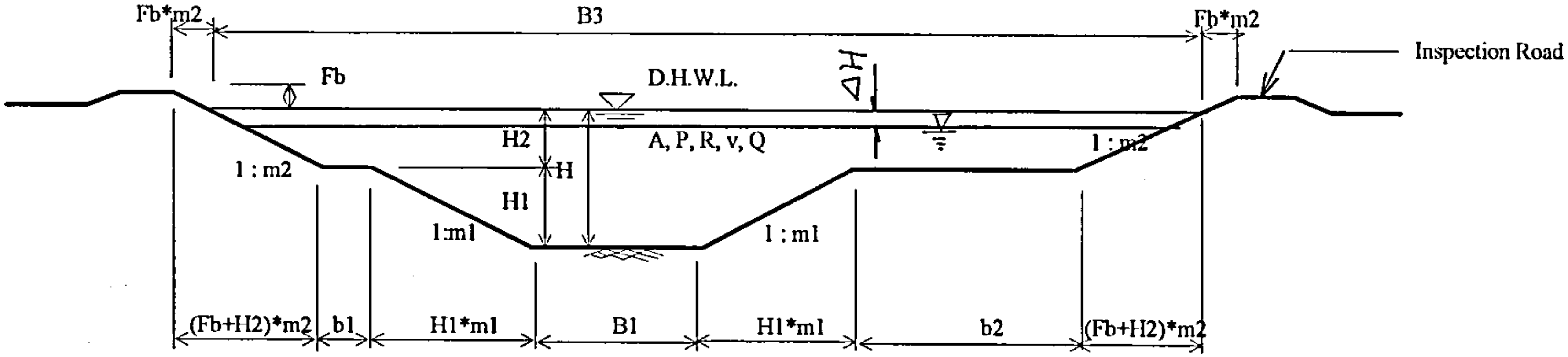
The bending section is widened by maximum rate of 60 % of a width of the standard cross section.

Uniform Flow Calculation **Upper Percut River** **Bending Section**

Roughness Coefficient	n =	0.038	
River Bed Slope	i =	0.001212	(I = 1 / 825)
River Bed Width	B1 =	8.00 m	
Height 1	H1 =	3.00 m	
Height 2	H2 =	2.58	
Slope Gradient 1	m1 =	2.00	
Slope Gradient 2	m2 =	2.00	
Width of Berm	b =	13.00 m	
Freeboard for Inspection Road	Fb =	0.80 m	

Hydraulic Radius	(m)	:	R = A/P
Velocity	(m/s)	:	v = 1/n * R ^(2/3) * I^(1/2)
Discharge	(m3/s)	:	Q = v x A
Design Discharge			

Immediate Plan	Qa =	320.00	(m3/s)
Urgent Plan	Qa =	320.00	(m3/s)



Calculation Table

$\boxed{\Delta H = 6.11 - 0.53 = 5.58\text{ m}}$

	H	B1	B2	B3	A1	A2	P	R	n	i	v	Q
	m	m	m	m	m2	m2	m	m			m/s	m3/s
1	5.58	8.00	20.00	56.32	42.00	173.99	58.95	2.95	0.038	0.001212	1.89	327.99
2	6.11	8.00	20.00	58.44	42.00	204.40	61.32	3.33	0.038	0.001212	2.04	417.88

1. D.H.W.L :	Q =	417.88	>	320.00	(m3/s)
2. D.H.W.L - 0.53m:	Q =	327.99	>	320.00	(m3/s)

Fig. 1. 1. 5 HYDRAULIC DESIGN CALCULATION FOR BENDING SECTION

1.1.2 MEDAN FLOODWAY

<u>Preparation of Standard Cross Section</u>

A standard cross section of the river channel is prepared for the detailed design of the Floodway such as alignment, longitudinal profile and cross section.

(1) Conditions on Hydraulic Calculation

(a) Location

Floodway is designed as a open channel with two types of channel, namely a single trapezoidal cross section for the downstream FW3 to FW29 and a double trapezoidal cross section for the upstream FW29 to the diversion weir.

(b) Method

Uniform steady flow calculation is used.

(c) Design Discharge

- Urgent Plan (40-year return period): 320 m^3/s
- Immediate Plan (25-year return period) : 320 m^3/s

(d) Roughness Coefficient

- Downstream Channel (with wet stone masonry): 0.030
- Upstream Channel (with leaning wall and channel-bed lining): 0.025

(e) Initial Water Level

The design high water level of FW3 (EL.30.730m) is used.

(f) Dimensions of Cross-sectional Form

The side slope of the Floodway is set as mentioned below in consideration of the decrease of social impair and relatively good ground/soil conditions.

- Downstream channel: 1: 1.5 (Single trapezoidal cross section)
- Upstream channel: 1: 0.5 (Double trapezoidal cross sections)

The water depth is set 5.8 m

(2) Calculation Results

The hydraulic designs are made as shown in Fig 1.1.6.

(3) Non-Uniform Flow Calculation

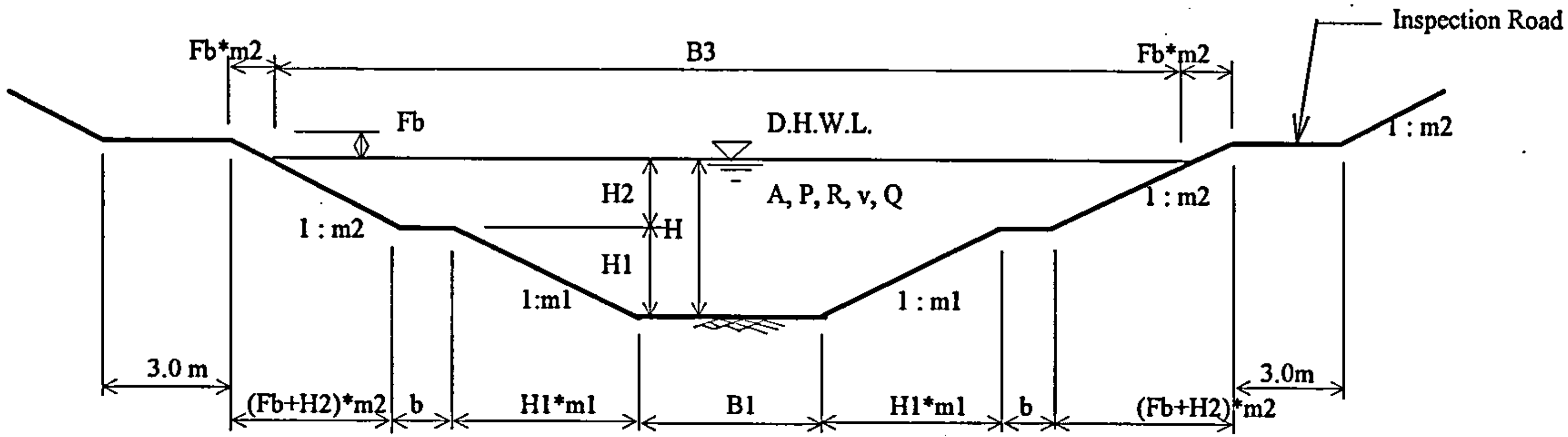
Using the standard cross sections, the non-uniform flow calculation for the Floodway were conducted in order to estimate the water surface profile and results are shown in Fig. 1.1.7 and Calculation Sheet-1.1.3.

Uniform Flow Calculation

Floodway (Channel Type-I) : (FW0 to FW28+50)

Roughness Coefficient	n =	0.0300	
River Bed Slope	i =	0.000426	(I = 1 / 2,350)
River Bed Width	B1 =	5.00 m	
Height 1	H1 =	3.00 m	
Height 2	H2 =	1.60 /2.80	
Slope Gradient 1	m1 =	1.50	
Slope Gradient 2	m2 =	1.50	
Width of Berm	b =	1.00 m	
Freeboard for Inspection Road	Fb =	0.40 m	

Hydraulic Radius	(m) :	R = A/P		
Velocity	(m/s) :	v = 1/n * R ^(2/3) * I^(1/2)		
Discharge	(m3/s) :	Q = v x A		
Design Discharge				
Immediate Plan		Qa =	70.00	(m3/s)
Urgent Plan		Qa =	120.00	(m3/s)



Calculation Table

	H m	B1 m	B2 m	B3 m	A1 m2	A2 m2	P m	R m	n	i	v m/s	Q m3/s
1	4.60	5.00	14.00	20.80	28.50	57.94	23.59	2.46	0.030	0.000426	1.25	72.53
2	5.80	5.00	14.00	24.40	28.50	85.06	27.91	3.05	0.030	0.000426	1.45	122.94

Immediate Plan :	Q =	72.53	>	70.00	(m3/s)
Urgent Plan :	Q =	122.94	>	120.00	(m3/s)

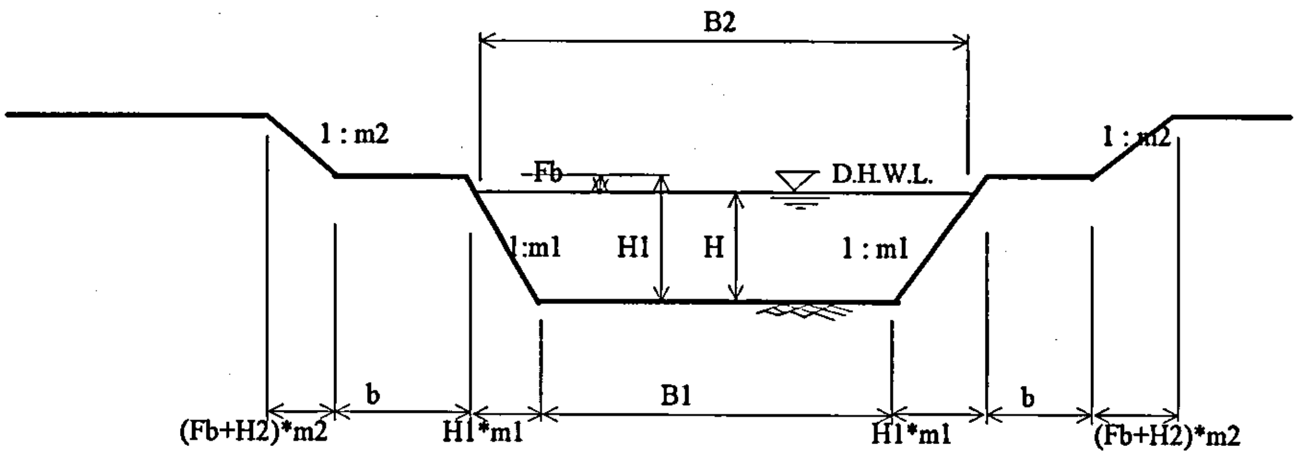
Fig. 1. 1. 6(1/2) HYDRAULIC DESIGN CALCULATION FOR FLOODWAY

Uniform Flow Calculation

Floodway (Channel Type-II) : (FW 29 to FW 3.90)

Roughness Coefficient	n =	0.0250	
River Bed Slope	i =	0.000426	(I = 1 / 2,350)
River Bed Width	B1 =	8.90 m	
Height 1	H1 =	6.60 m	
Height 2	H2 =	7.00	
Slope Gradient 1	m1 =	0.50	
Slope Gradient 2	m2 =	0.50	
Width of Berm	b =	5.00 m	
Freeboard	Fb =	0.80 m	

Hydraulic Radius (m) : R = A/P
Velocity (m/s) : v = 1/n * R ^(2/3) * I^(1/2)
Discharge (m3/s) : Q = v x A
Design Discharge
Immediate Plan Qa = 70.00 (m3/s)
Urgent Plan Qa = 120.00 (m3/s)



Calculation Table

	H	B1	B2	B3	A1	A2	P	R	n	i	v	Q
	m	m	m	m	m2	m2	m	m			m/s	m3/s
1	4.20	8.90	13.10		46.20		18.29	2.53	0.025	0.000426	1.53	70.70
2	5.80	8.90	14.70		68.44		21.87	3.13	0.025	0.000426	1.77	120.82

Immediate Plan : Q = 70.70 > 70.00 (m3/s)
Urgent Plan : Q = 120.82 > 120.00 (m3/s)

Fig. 1. 1. 6(2/2) HYDRAULIC DESIGN CALCULATION FOR FLOODWAY

Water Surface Profile

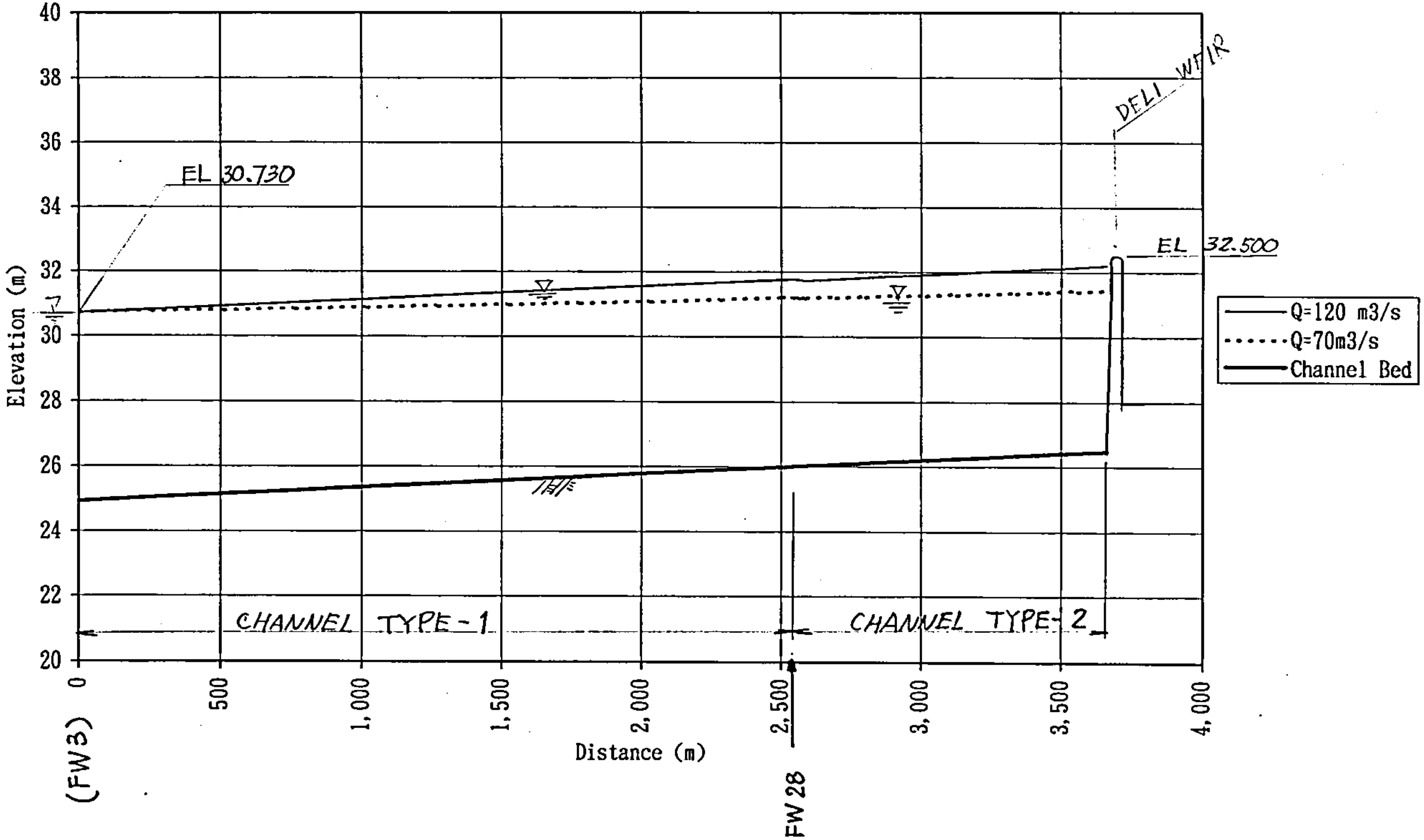


Fig. 1. 1. 7 WATER SURFACE PROFILE OF FLOODWAY

Calculation Sheet - 1. 1. 3

NON-UNIFORM FLOW CALCULATION

CASE-1 FLOODWAY WATER SURFACE PROFILE (Q=120m3/s, Ho=30.730) -FINAL- 1996/7/16 PAGE = 1

** INPUT DATA **

* BAISIC DATA *

KUKAN-SU = 3 ALPHA = 1.10 QO = 120.00 M3/S HO = 30.730 M ZO = 24.930 M

JCO = 0 KEY = 0 IPT = 0

* KUKAN DATA *

DANMEN NO.	BUNKATSU SUU	DANMEN KEIJYO		LOSS TYPE	SODO KEISU	KUKAN KYORI(M)	KASYO KOOBAI(1/I)	RAKUSA (M)	RYUNYU RYO(M3/S)
1 FW.3	10	2	1	0	.0300	2535.50	2350.00	.000	.00
2 FW.28	5	2	1	0	.0280	50.00	2350.00	.000	.00
3 FW.28+50	10	1	1	0	.0250	1075.50	2350.00	.000	.00

* KEIJYO DATA *

KUKAN	KEIJYO		BO(R)	M1	N1	B1	B2	HP(B3)	M2	N2
1	2	1	5.000	1.500	1.500	1.000	1.000	3.000	1.500	1.500
			5.000	1.500	1.500	1.000	1.000	3.000	1.500	1.500
2	2	1	5.000	1.500	1.500	1.000	1.000	3.000	1.500	1.500
			8.900	.500	.500	.000	.000	3.000	.500	.500
3	1	1	8.900	.500	.500	.000	.000	.000	.000	.000
			8.900	.500	.500	.000	.000	.000	.000	.000

* LOSS DATA *

KUKAN LOSS TYPE FL1 FL2

FLOODWAY WATER SURFACE PROFILE (Q=120m3/s, Ho=30.730) -FINAL- 1996/7/16 PAGE = 2

NO.	H	A	R	V	N	Q	DX	FROUD	IE	Z	H-Z
FW.3	30.730	85.06	3.047	1.411	.0300	120.00	.00	.2531	.4054E-03	24.930	5.800
+ 253.55	30.833	84.93	3.045	1.413	.0300	120.00	253.55	.2536	.4071E-03	25.038	5.795
+ 507.10	30.936	84.82	3.043	1.415	.0300	120.00	253.55	.2541	.4086E-03	25.146	5.790
+ 760.65	31.039	84.71	3.041	1.417	.0300	120.00	253.55	.2545	.4100E-03	25.254	5.786
+1014.20	31.143	84.61	3.039	1.418	.0300	120.00	253.55	.2549	.4113E-03	25.362	5.782
+1267.75	31.247	84.52	3.037	1.420	.0300	120.00	253.55	.2552	.4125E-03	25.469	5.778
+1521.30	31.352	84.44	3.035	1.421	.0300	120.00	253.55	.2555	.4136E-03	25.577	5.774
+1774.85	31.457	84.36	3.034	1.422	.0300	120.00	253.55	.2558	.4146E-03	25.685	5.771

+2028.40	31.562	84.30	3.032	1.424	.0300	120.00	253.55	.2561	.4156E-03	25.793	5.769
+2281.95	31.667	84.23	3.031	1.425	.0300	120.00	253.55	.2563	.4164E-03	25.901	5.766
+2535.50	31.773	84.17	3.030	1.426	.0300	120.00	253.55	.2566	.4172E-03	26.009	5.764
FW.28	31.773	84.18	3.030	1.425	.0280	120.00	.00	.2565	.3633E-03	26.009	5.764
+ 10.00	31.766	80.69	3.074	1.487	.0280	120.00	10.00	.2621	.3880E-03	26.013	5.753
+ 20.00	31.759	77.22	3.110	1.554	.0280	120.00	10.00	.2675	.4171E-03	26.017	5.741
+ 30.00	31.750	73.77	3.132	1.627	.0280	120.00	10.00	.2725	.4527E-03	26.022	5.729
+ 40.00	31.740	70.35	3.131	1.706	.0280	120.00	10.00	.2769	.4981E-03	26.026	5.714
+ 50.00	31.729	66.96	3.094	1.792	.0280	120.00	10.00	.2804	.5586E-03	26.030	5.698
FW.28+50	31.729	66.96	3.094	1.792	.0250	120.00	.00	.2804	.4453E-03	26.030	5.698
+ 107.55	31.777	66.99	3.094	1.791	.0250	120.00	107.55	.2802	.4448E-03	26.076	5.701
+ 215.10	31.825	67.02	3.095	1.791	.0250	120.00	107.55	.2800	.4442E-03	26.122	5.703
+ 322.65	31.872	67.05	3.096	1.790	.0250	120.00	107.55	.2798	.4436E-03	26.168	5.705
+ 430.20	31.920	67.08	3.097	1.789	.0250	120.00	107.55	.2797	.4431E-03	26.213	5.707
+ 537.75	31.968	67.11	3.097	1.788	.0250	120.00	107.55	.2795	.4426E-03	26.259	5.709
+ 645.30	32.016	67.14	3.098	1.787	.0250	120.00	107.55	.2793	.4421E-03	26.305	5.711
+ 752.85	32.063	67.17	3.099	1.787	.0250	120.00	107.55	.2792	.4416E-03	26.351	5.713
+ 860.40	32.111	67.19	3.099	1.786	.0250	120.00	107.55	.2790	.4411E-03	26.396	5.715
+ 967.95	32.159	67.22	3.100	1.785	.0250	120.00	107.55	.2789	.4406E-03	26.442	5.717
+1075.50	32.206	67.25	3.101	1.785	.0250	120.00	107.55	.2788	.4402E-03	26.488	5.718

1

FLOODWAY WATER SURFACE PROFILE (Q=70m3/s, Ho=30.730) -FINAL- 1996/7/16 PAGE = 3

** INPUT DATA **

* BAISIC DATA *

KUKAN-SU = 3 ALPHA = 1.10 QO = 70.00 M3/S HO = 30.730 M ZO = 24.930 M

JCO = 0 KEY = 0 IPT = 0

* KUKAN DATA *

DANMEN NO.	BUNKATSU SUU	DANMEN KEIJYO		LOSS TYPE	SODO KEISU	KUKAN KYORI(M)	KASYO KOOBAI(1/I)	RAKUSA (M)	RYUNYU RYO(M3/S)
1 FW.3	10	2	1	0	.0300	2535.50	2350.00	.000	.00
2 FW.28	5	2	1	0	.0280	50.00	2350.00	.000	.00
3 FW.28+50	10	1	1	0	.0250	1075.50	2350.00	.000	.00

* KEIJYO DATA *

KUKAN	KEIJYO		BO(R)	M1	N1	B1	B2	HP(B3)	M2	N2
1	2	1	5.000	1.500	1.500	1.000	1.000	3.000	1.500	1.500
			5.000	1.500	1.500	1.000	1.000	3.000	1.500	1.500
2	2	1	5.000	1.500	1.500	1.000	1.000	3.000	1.500	1.500
			8.900	.500	.500	.000	.000	3.000	.500	.500

3	1	1	8.900	.500	.500	.000	.000	.000	.000	.000
			8.900	.500	.500	.000	.000	.000	.000	.000

* LOSS DATA *

KUKAN LOSS TYPE FL1 FL2

FLOODWAY WATER SURFACE PROFILE (Q=70m3/s, Ho=30.730) -FINAL- 1996/7/16 PAGE = 4

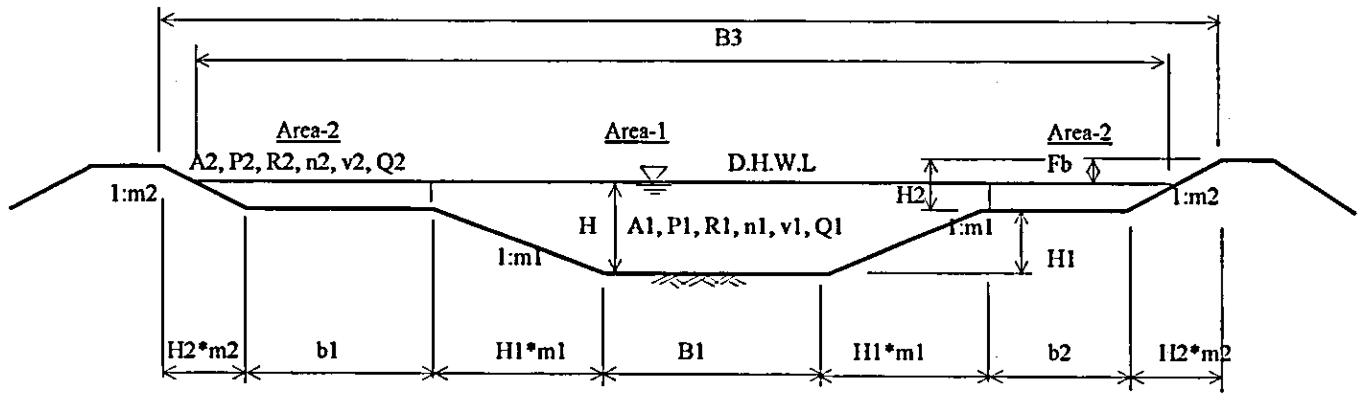
NO.	H	A	R	V	N	Q	DX	FROUD	IE	Z	H-Z
FW.3	30.730	85.06	3.047	.823	.0300	70.00	.00	.1477	.1380E-03	24.930	5.800
+ 253.55	30.764	83.27	3.012	.841	.0300	70.00	253.55	.1518	.1462E-03	25.038	5.726
+ 507.10	30.801	81.55	2.978	.858	.0300	70.00	253.55	.1559	.1548E-03	25.146	5.655
+ 760.65	30.839	79.90	2.944	.876	.0300	70.00	253.55	.1600	.1637E-03	25.254	5.586
+1014.20	30.880	78.32	2.912	.894	.0300	70.00	253.55	.1642	.1729E-03	25.362	5.519
+1267.75	30.924	76.80	2.880	.911	.0300	70.00	253.55	.1684	.1824E-03	25.469	5.454
+1521.30	30.969	75.35	2.850	.929	.0300	70.00	253.55	.1726	.1922E-03	25.577	5.392
+1774.85	31.018	73.98	2.821	.946	.0300	70.00	253.55	.1768	.2022E-03	25.685	5.332
+2028.40	31.068	72.67	2.793	.963	.0300	70.00	253.55	.1809	.2124E-03	25.793	5.275
+2281.95	31.122	71.43	2.766	.980	.0300	70.00	253.55	.1849	.2226E-03	25.901	5.221
+2535.50	31.178	70.25	2.740	.996	.0300	70.00	253.55	.1889	.2330E-03	26.009	5.169
FW.28	31.178	70.25	2.740	.996	.0280	70.00	.00	.1889	.2030E-03	26.009	5.169
+ 10.00	31.176	67.95	2.795	1.030	.0280	70.00	10.00	.1910	.2114E-03	26.013	5.163
+ 20.00	31.174	65.66	2.844	1.066	.0280	70.00	10.00	.1927	.2212E-03	26.017	5.156
+ 30.00	31.171	63.39	2.882	1.104	.0280	70.00	10.00	.1939	.2332E-03	26.022	5.150
+ 40.00	31.169	61.13	2.901	1.145	.0280	70.00	10.00	.1946	.2485E-03	26.026	5.143
+ 50.00	31.165	58.89	2.889	1.189	.0280	70.00	10.00	.1944	.2692E-03	26.030	5.135
FW.28+50	31.165	58.89	2.889	1.189	.0250	70.00	.00	.1944	.2146E-03	26.030	5.135
+ 107.55	31.188	58.56	2.880	1.195	.0250	70.00	107.55	.1959	.2179E-03	26.076	5.112
+ 215.10	31.211	58.24	2.872	1.202	.0250	70.00	107.55	.1974	.2212E-03	26.122	5.089
+ 322.65	31.234	57.92	2.863	1.209	.0250	70.00	107.55	.1988	.2245E-03	26.168	5.066
+ 430.20	31.257	57.61	2.855	1.215	.0250	70.00	107.55	.2003	.2278E-03	26.213	5.044
+ 537.75	31.281	57.30	2.847	1.222	.0250	70.00	107.55	.2017	.2311E-03	26.259	5.022
+ 645.30	31.305	57.00	2.839	1.228	.0250	70.00	107.55	.2032	.2345E-03	26.305	5.000
+ 752.85	31.330	56.71	2.831	1.234	.0250	70.00	107.55	.2046	.2378E-03	26.351	4.979
+ 860.40	31.354	56.42	2.823	1.241	.0250	70.00	107.55	.2060	.2412E-03	26.396	4.958
+ 967.95	31.380	56.13	2.815	1.247	.0250	70.00	107.55	.2074	.2445E-03	26.442	4.938
+1075.50	31.405	55.86	2.807	1.253	.0250	70.00	107.55	.2088	.2479E-03	26.488	4.917

Uniform Flow Calculation

Roughness Coeff.	1	n1 =	0.033	
Roughness Coeff.	2	n2 =	0.040	
Channel Bed Slope		i =	0.001111	(I = 1 / 900)
Channel Bed Width		B1 =	10.00 m	
Height 1		H1 =	3.00 m	
Height 2		H2 =	3.00 m	
Slope Gradient 1		m1 =	2.00	
Slope Gradient 2		m2 =	2.00	
Width 1		b1 =	0.00 m	
Width 2		b2 =	0.00 m	
Free Board		Fb =	0.80 m	

Deli River Retarding Channel

Hydraulic Radius	(m)	: R = A / P
Velocity	(m/s)	: v = 1/n * R ^(2/3) * I^(1/2)
Discharge	(m3/s)	: Q = v x A
Design Discharge		
	Immediate Plan :	300.0 (m3/s)
	Urgent Plan :	320.0 (m3/s)



Calculation Table

	H	(B1)	(B2)	(B3)	A1	A2	P1	P2	R1	R2	n1	n2	i	v1	v2	Q1	Q2	Q
	m	m	m	m	m2	m2	m	m	m	m				m/s	m/s	m3/s	m3/s	m3/s
1	1.00	10.00	14.00		12.00	0.00	14.47	0.00	0.83	0.00	0.033	0.040	0.001111	0.89		10.70	0.00	10.70
2	2.00	10.00	18.00		28.00	0.00	18.94	0.00	1.48	0.00	0.033	0.040	0.001111	1.31		36.70	0.00	36.70
3	3.00	10.00	22.00		48.00	0.00	23.42	0.00	2.05	0.00	0.033	0.040	0.001111	1.63		78.24	0.00	78.24

Probability	Q
2 years	156.00
1 year	134.00
2 times/y	120.00
5 times/y	98.00
10 times/y	78.00

Fig. 1.1.8 HYDRAULIC DESIGN CALCULATION FOR DELI RIVER RETARDING CHANNEL

1.1.3 DELI RIVER

(1) Purpose of Hydraulic Calculation

The river hydraulics of Deli Retarding Channel is discussed in the main report Chapter 3 'Investigation and Analysis'. Therefore the study item handled in this section is the design of low water channel.

(2) Conditions of Calculation

Though the ponding takes place in flood time in the retarding channel, the low water channel is designed without taking account the ponding effect.

(3) Hydraulic Design of River Channel

The low water channel was designed as shown in Fig.1.1.8 and summarized below:

- Width of river-bed:	10 m
- Height of channel:	3.0 m
- Side slopes:	1 : 2
- Bankfull discharge:	78.24 m3/s (Return Period = 10 times/year)

1.1.4 OTHER CHANNEL

Irrigation /Drainage Channel connecting with SL.2

(1) Location

The irrigation/drainage channel which flows along Percut River at the left dike in the stretch from PE96 to PE103, is relocated by about 7 m to the left due to the left dike construction. The new channel is aligned in parallel with the dike, connecting with the sluice SL.2.

(2) Design Conditions

- Length : 710 m
- Design Discharge: 15 m3/s
- Channel Type: Earth open channel (Trapezoidal cross section)
- Roughness Coefficient: n = 0.033

(3) Hydraulic Design

The channel is designed as shown in Fig.1.1.9 and summarized below.

- Width of river-bed: 4.5 m
- Water depth: 1.50 m
- Side slopes: 1 : 1.5
- Free board: 30 cm

Uniform Flow Calculation

Irrigation/drainage Channel (PE96 toPE103)

Roughness Coeff.	1	n1 =	0.033	
Roughness Coeff.	2	n2 =	0.040	
Channel Bed Slope		i =	0.002	(I = 1 / 500)
Channel Bed Width		B1 =	4.50	m
Height 1		H1 =	1.50	m
Height 2		H2 =	-	m
Slope Gradient 1		m1 =	2.00	
Slope Gradient 2		m2 =	-	
Width 1		b1 =	-	m
Width 2		b2 =	-	m
Freeboard for Dike		Fb =	0.80	m

Hydraulic Radius (m) : R = A / P

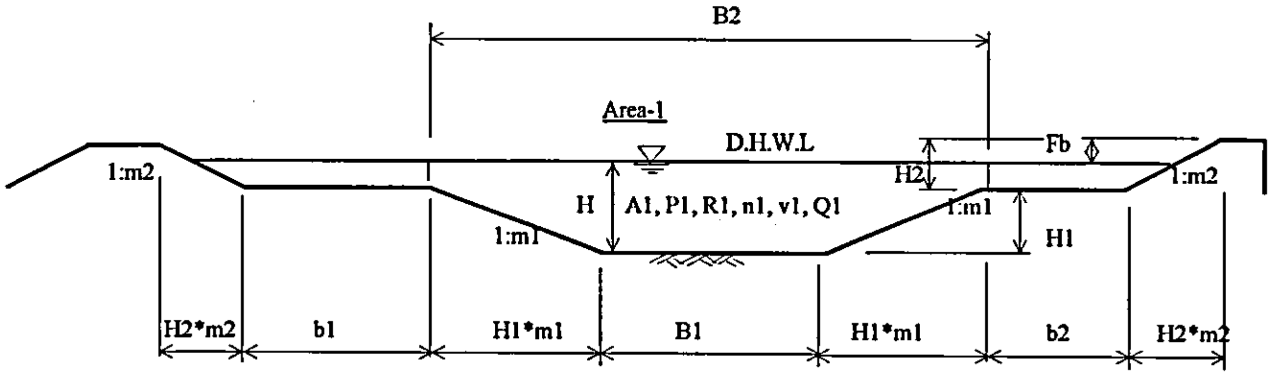
Velocity (m/s) : v = 1/n * R ^(2/3) * I^(1/2)

Discharge (m3/s) : Q = v x A

Design Discharge

Qa = 15.00 (m3/s)

Design High Water Level



Calculation Table

	H	(B1)	(B2)	(B3)	A1	A2	P1	P2	R1	R2	n1	n2	i	v1	v2	Q
	m	m	m	m	m2	m2	m	m	m	m				m/s	m/s	m3/s
1	1.50	4.50	10.50	-	11.25	-	11.21	-	1.00	-	0.033	0.040	0.002000	1.36	-	15.28
2	1.50	4.50	10.50	-	11.25	-	11.21	-	1.00	-	0.033	0.040	0.002000	1.36	-	15.28

Q = 15.28 > 15.00 (m3/s) OK

Fig. 1.1.9 HYDRAULIC DESIGN CALCULATION FOR IRRIGATION/DRAINAGE CHANNEL FOR SL.2

1.2 SLOPE AND STRUCTURAL STABILITY ANALYSIS

1.2.1 PERCUT RIVER

Slope Stability on River-Bank and Dike

(1) Purpose of Study

The study is made to verify the stability of river-bank slope and dike slope of the design cross sections.

(2) Location

The study cross sections were selected from cross sections which are involved in soft ground area. Selected cross sections are as follows:

Section-A: A representative cross section from the river stretch PE130 to PE160

Section-B: A representative cross section from the river stretch PE170 to PE230

Section-C : A representative dike cross section from the river stretch PE27 to PE42

(3) Calculation Method

A circular slip method is employed.

(4) Conditions on Calculation

(a) Soil Layer and Soil Property:

Section	Embankment	First Layer	Second Layer
Section-A	Sandy material γ=1.8 t/m3 ϕ=30°, c=0 t/m2	CH1, Alluvium Soft Clay 5 - 10 m thick	CL3, Diluvium Low stiffness Clay 5 - 15 m thick
Section-B	Sandy material γ=1.8 t/m3 ϕ=30°, c=0 t/m2	SC2, Alluvium Low Stiffness Clayey sand 7 - 15 m thick	SP4, Diluvium Sand Medium stiffness 3 - 10 m thick
Section-C	Sandy material γ=1.8 t/m3 ϕ=30°, c=0 t/m2	CL1, Alluvium Low Stiffness Silty Cray 2 - 5 m thick	SP1, Alluvium Low Density Fine Sand 1 - 5 m thick

Soil properties of each layer are determined as shown in the calculation chart based on the soil test.

(b) Calculation Case

The calculation is made using the water level conditions just after design flood. This case is most critical for the slope stability.

- River side water level : Low water level (2.0 m above riverbed)
- Land side ground water level: 1.0 m lower than ground surface

(c) Loading condition

A uniform load of 1.0 t/m2 is considered on the dike crest.

(5) Results

Calculation results are presented in Figs.1.2.1 to 1.2.3 and Calculation Sheets-1.2.1 to 1.2.3 and summarized below.

Section	Minimum Safety Factor	Allowable Safety Factor	Evaluation
Section-A	1.54	1.20	Stable
Section-B	1.32	1.20	Stable
Section-C	1.22	1.20	Stable

The river-bank slopes and dike slope of design sections are proved stable.

ANALYSIS OF SLOPE STABILITY (SECTION-A)

PROPERTY OF SOIL

	γ (tf/m^3)	c (tf/m^2)	ϕ (°)
1	1.00	0.0	0.0
2	1.65	2.5	10.0
3	1.65	2.5	10.0
4	1.65	2.5	10.0

1. LOCATION : PERCUT RIVER
2. CALCULATION SECTION : FROM PE130 TO PE160
3. BORING DATA : B15 AND B17
4. CALCULATION CONDITION : AFTER FLOOD

CALCULATION RESULTS

Minimum Safety Factor

$$SF = \frac{PV}{PH} = \frac{74.51}{47.69}$$

$= 1.56 >$ Allowable Safety Factor $= 1.2$

Radius = 10.96 m

MINIMUM SAFETY FACTOR

1.74	1.72	1.74	1.87
1.74	1.69	1.69	1.70
1.73	1.70	1.65	1.71
1.69	1.61	1.63	1.69
1.70	1.54	1.56	1.67

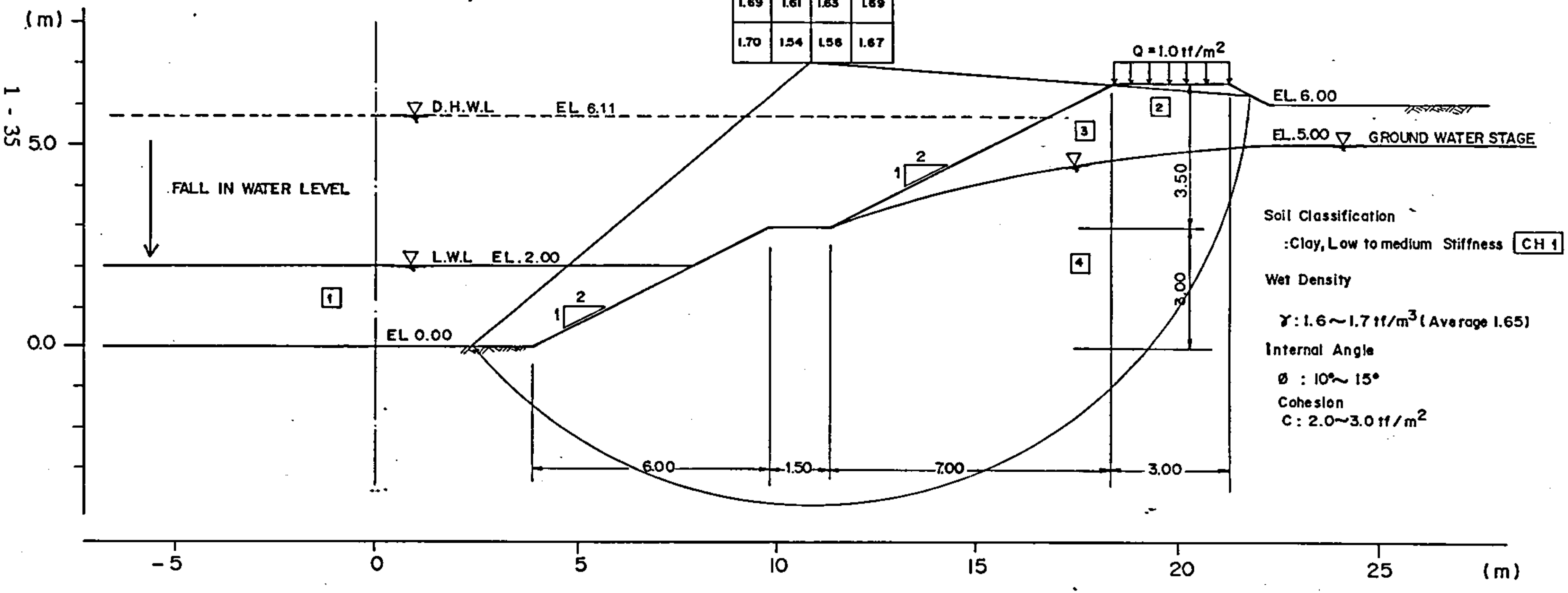


Fig. 1. 2. 1 RESULT OF SLOPE STABILITY ANALYSIS IN LOWER PERCUT RIVER

ANALYSIS ON SLOPE STABILITY (SECTION-B)

1. LOCATION : PERCUT RIVER
2. CALCULATION SECTION : FROM PE 170 TO PE 230
3. BORING DATA : B 19, B 20, B 21 AND B 24
4. CALCULATION CONDITION : AFTER FLOOD

PROPERTY OF SOIL

	γ (tf/m³)	C (tf/m²)	ø (°)
1	1.0	0.0	0.0
2	1.8	0.8	25.0
3	1.8	0.8	25.0
4	2.0	0.8	25.0

MINIMUM SAFETY FACTOR

1.41	1.43	1.37	1.43
1.48	1.39	1.32	1.45
1.50	1.42	1.37	1.38
1.44	1.43	1.40	1.40

CALCULATION RESULTS

Minimum Safety Factor

$$SF = \frac{PV}{PH} = \frac{50.72}{38.35}$$

≒ 1.32 > Allowable Safety Factor = 1.2

Radius = 11.90m

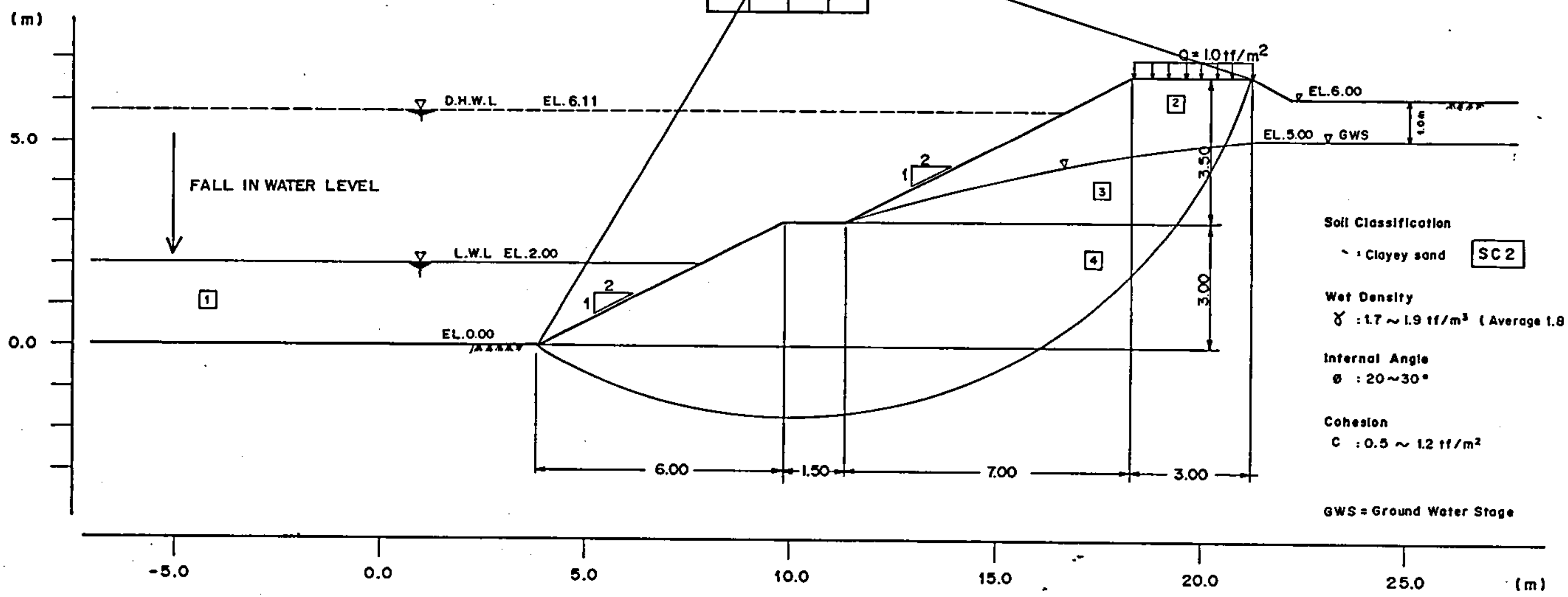


Fig. 1. 2. 2 RESULT OF SLOPE STABILITY ANALYSIS IN UPPER PERCUT RIVER

ANALYSIS ON SLOPE STABILITY (SECTION-C)

1. LOCATION : PERCUT RIVER
2. CALUCULATION SECTION : PE27, PE42
3. BORING DATA : B3
4. CALUCULATION CONDITION : AFTER FLOOD

PROPERTY OF SOIL

	γ_t (tf/m³)	γ_{sat} (tf/m³)	C (tf/m²)	Ø (°)
1	1.8	2.0	0.0	30
2	1.8	2.0	0.1	30
3	1.50	1.90	1.5	10
4	1.75	2.0	0.0	26

γ_t : Wet Density of Soil
γ_{sat} : Saturated Density of Soil
C : Cohesion
Ø : Internal Angle

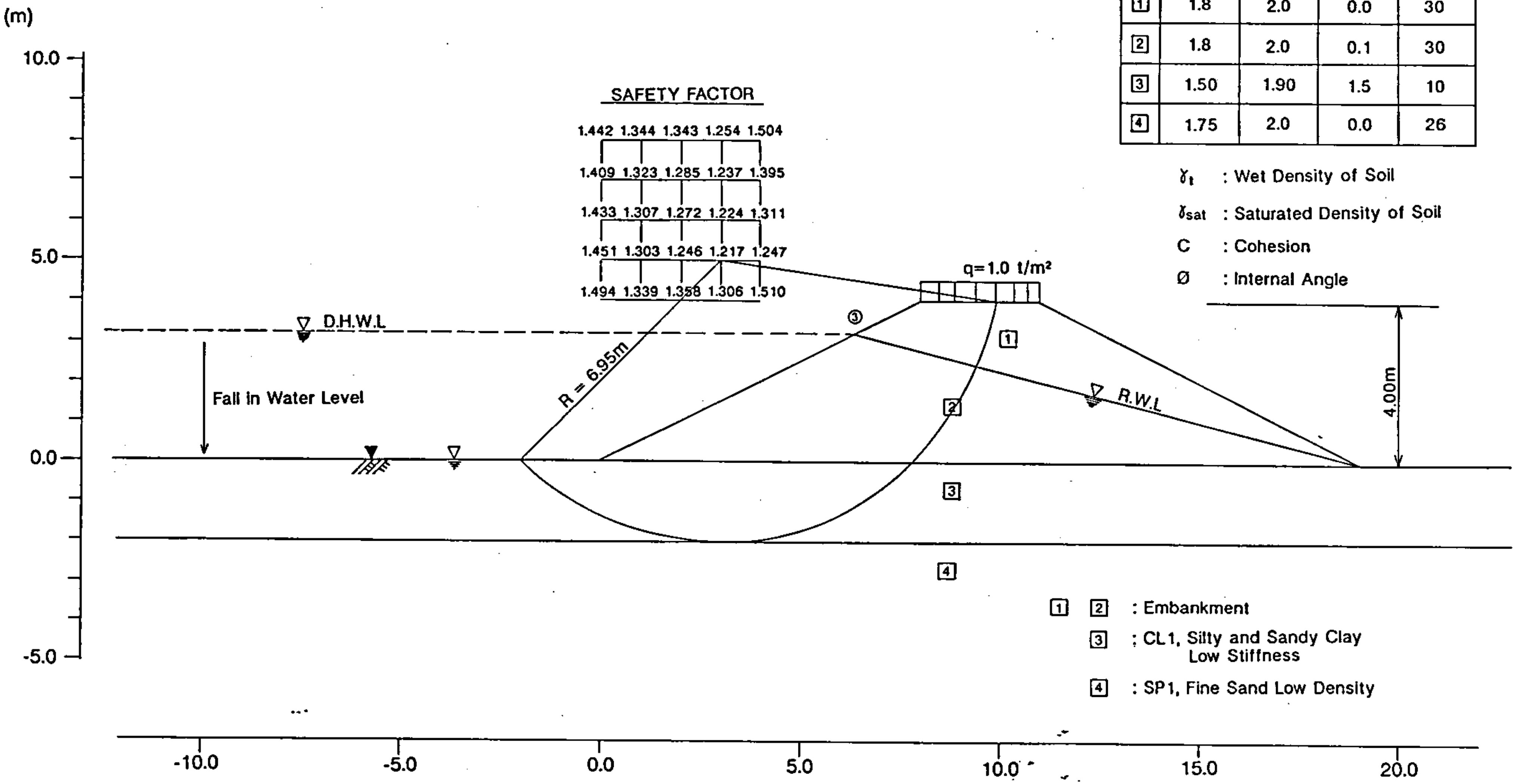


Fig. 1. 2. 3 RESULT OF DIKE SLOPE STABILITY ANALYSIS IN LOWER PERCUT RIVER

Calculation Sheet -1. 2. 1

SLOPE STABILITY ANALYSIS ** UPPER PERCUT RIVER (1) **
THE DETAILED DESIGN STUDY ON MEDAN FLOOD CONTROL PROJECT

** BASIC DATA **

METHOD OF CALCULATION	0	
LAYER NUMBERS	4	
MESH BLOCK NUMBERS	1	
CONDITION OF CALCULATION	0	
CASE OF EARTHQUAKE	0	
CASE OF COHESION	0	
UPPER LAYER	4	
WATER LAYER	1	
GROUND WATER	1	
NON-PASS-THROUGH OBJECT NO.	4	
UNIT WEIGHT OF WATER	1.000	(T/M3)
EARTHQUAKE COEFFICIENT	.000	
SLICE INTERVAL	1.000	(M)
LEFT EDGE OF SLICE	-20.000	(M)
RIGHT EDGE OF SLICE	42.500	(M)
LOWER LIMIT Y-COORDINATE	-10.000	(M)
RADIUS PITCH	1.000	(M)
ADDITIONAL INITIAL RADIUS	1.000	(M)

** CENTER DATA **

BLOCK	XG(M)	YG(M)	D(M)	NX	NY	TCH-L	TCH-N
1	7.000	7.000	1.000	6	6	0	0

** NON-PASS-THROUGH OBJECT DATA **

NUMBERS	COORDINATE NUMBER			
4	3	4	5	6

** LAYER DATA **

LAYER	W(T/M3)	PHI(DEG)	C(T/M2)	KH	N	LAYER COMPONENTS
1	1.0000	.00	.0000	.0000	11	16 4 5 6 17 7 8 9 10 11 12
2	1.6500	10.00	2.5000	.0000	13	1 2 3 4 5 6 17 7 8 9 10 11 12
3	1.6500	10.00	2.5000	.0000	10	1 2 3 4 5 6 17 7 8 14
4	1.9000	10.00	2.5000	.0000	9	1 2 3 4 5 6 17 18 15

** GROUND WATER COMPONENTS **

N = 7 LINE = 16 4 5 6 17 18 15

** COORDINATE DATA **

NO	X	Y
1	-20.000	.000
2	.000	.000
3	4.000	.000
4	8.000	2.000
5	10.000	3.000
6	11.500	3.000
7	15.500	5.000
8	16.900	5.700
9	18.500	6.500
10	21.500	6.500
11	22.500	6.000
12	42.500	6.000
13	-20.000	5.700
14	42.500	5.700
15	42.500	5.000
16	-20.000	2.000
17	11.700	3.100
18	22.500	5.000

EXTERNAL FORCE

N	XA	YA	QA	XB	YB	QB	ALPHA
1	18.500	6.500	1.000	21.500	6.500	1.000	-90.000

SAFETY FACTOR

CX	CY	R	SF	PV	PH	NF	SNF	UF	CL	T	ST	FOR	NEF	TE
7.000	7.000	17.00	1.819	146.24	80.39	90.37	17.01	-67.16	106.02	68.10	10.00	2.29	.00	.00
8.000	7.000	17.00	1.771	147.50	83.26	91.92	17.36	-67.80	106.02	70.22	10.92	2.12	.00	.00
9.000	7.000	13.02	1.699	98.00	57.66	47.71	7.83	-33.73	76.20	49.80	5.35	2.51	.00	.00
10.000	7.000	11.00	1.591	72.72	45.71	31.16	4.49	-20.99	58.07	40.45	3.05	2.20	.00	.00
11.000	7.000	10.96	1.562	74.51	47.69	32.46	4.71	-21.42	58.75	41.76	3.46	2.46	.00	.00
12.000	7.000	11.79	1.671	89.19	53.39	40.87	6.48	-27.26	69.10	46.23	5.13	2.04	.00	.00
7.000	8.000	18.00	1.843	151.02	81.95	94.83	16.77	-69.12	108.54	70.12	9.66	2.17	.00	.00
8.000	8.000	18.00	1.800	152.29	84.61	96.38	17.10	-69.73	108.54	72.13	10.48	2.00	.00	.00
9.000	8.000	13.00	1.637	87.44	53.41	40.72	5.72	-27.75	68.74	47.24	3.65	2.52	.00	.00
10.000	8.000	12.00	1.601	76.67	47.90	33.63	4.39	-22.01	60.66	42.53	2.87	2.50	.00	.00
11.000	8.000	11.95	1.625	78.46	48.28	34.78	4.58	-22.35	61.45	42.78	3.24	2.26	.00	.00
12.000	8.000	12.97	1.636	91.60	55.98	45.22	6.54	-29.48	69.33	49.02	5.10	1.86	.00	.00
7.000	9.000	19.00	1.866	155.65	83.43	99.11	16.47	-70.96	111.02	72.07	9.31	2.05	.00	.00
8.000	9.000	14.57	1.764	100.84	57.17	47.25	6.27	-32.25	79.57	51.07	3.63	2.48	.00	.00
9.000	9.000	14.08	1.728	95.04	55.01	44.14	5.73	-29.45	74.64	49.09	3.58	2.34	.00	.00
10.000	9.000	17.00	1.703	131.46	77.19	77.42	12.04	-53.31	95.31	67.21	8.22	1.76	.00	.00
11.000	9.000	12.23	1.653	75.29	45.54	30.95	3.41	-19.03	59.96	41.29	2.08	2.17	.00	.00
12.000	9.000	13.66	1.713	94.47	55.16	44.76	5.74	-28.37	72.33	49.11	4.31	1.74	.00	.00

CX	CY	R	SF	PV	PH	NF	SNF	UF	CL	T	ST	FOR	NEF	TE
7.000	10.000	16.77	1.847	119.29	64.58	61.32	8.15	-42.16	91.98	57.85	4.40	2.33	.00	.00
8.000	10.000	17.58	1.799	131.40	73.02	72.51	10.17	-49.78	98.50	64.81	6.15	2.06	.00	.00
9.000	10.000	13.07	1.735	73.76	42.51	28.24	2.70	-17.69	60.51	38.93	1.06	2.52	.00	.00
10.000	10.000	13.10	1.687	75.82	44.95	30.06	2.92	-18.46	61.30	41.23	1.41	2.31	.00	.00
11.000	10.000	13.12	1.689	77.38	45.83	31.76	3.14	-19.17	61.65	42.01	1.75	2.07	.00	.00
12.000	10.000	13.74	1.704	86.36	50.69	38.67	4.20	-23.47	66.96	46.13	2.82	1.75	.00	.00
7.000	11.000	18.99	1.869	140.09	74.96	79.38	10.64	-54.35	104.42	67.06	5.85	2.06	.00	.00
8.000	11.000	17.15	1.822	116.09	63.72	58.72	7.05	-39.03	89.36	57.59	4.04	2.09	.00	.00
9.000	11.000	15.16	1.739	91.31	52.49	39.72	4.02	-25.21	72.78	48.20	2.10	2.19	.00	.00
10.000	11.000	15.20	1.716	92.94	54.16	41.62	4.28	-26.07	73.10	49.65	2.53	1.99	.00	.00
11.000	11.000	15.90	1.719	103.21	60.03	50.08	5.59	-31.45	78.98	54.60	3.73	1.70	.00	.00
12.000	11.000	15.88	1.740	104.17	59.88	51.44	5.80	-31.85	78.78	54.38	3.98	1.52	.00	.00
7.000	12.000	16.59	1.867	97.50	52.23	42.78	4.15	-27.77	78.34	48.24	1.62	2.37	.00	.00
8.000	12.000	17.32	1.802	108.49	60.22	51.99	5.47	-33.73	84.76	55.27	2.86	2.09	.00	.00
9.000	12.000	18.06	1.776	119.56	67.31	61.76	6.97	-40.06	90.88	61.31	4.17	1.83	.00	.00
10.000	12.000	14.10	1.754	68.92	39.28	25.18	1.77	-14.38	56.35	36.94	.20	2.14	.00	.00
11.000	12.000	15.89	1.764	93.24	52.86	42.16	3.96	-25.46	72.58	48.86	2.30	1.70	.00	.00
12.000	12.000	15.91	1.786	94.80	53.07	43.98	4.21	-26.20	72.82	48.99	2.56	1.52	.00	.00

*** MINIMUM SAFETY FACTOR ***

11.000	7.000	10.96	1.562	74.51	47.69	32.46	4.71	-21.42	58.75	41.76	3.46	2.46	.00	.00

Calculation Sheet-1. 2. 2

SLOPE STABILITY ANALYSIS ** UPPER PERCUT RIVER (2) **
THE DETAILED DESIGN STUDY ON MEDAN FLOOD CONTROL PROJECT

** BASIC DATA **

METHOD OF CALCULATION	0	
LAYER NUMBERS	4	
MESH BLOCK NUMBERS	1	
CONDITION OF CALCULATION	0	
CASE OF EARTHQUAKE	0	
CASE OF COHESION	0	
UPPER LAYER	4	
WATER LAYER	1	
GROUND WATER	1	
NON-PASS-THROUGH OBJECT NO.	4	
UNIT WEIGHT OF WATER	1.000	(T/M3)
EARTHQUAKE COEFFICIENT	.000	
SLICE INTERVAL	1.000	(M)
LEFT EDGE OF SLICE	-20.000	(M)
RIGHT EDGE OF SLICE	42.500	(M)
LOWER LIMIT Y-COORDINATE	-10.000	(M)
RADIUS PITCH	1.000	(M)
ADDITIONAL INITIAL RADIUS	1.000	(M)

** CENTER DATA **

BLOCK	XG(M)	YG(M)	D(M)	NX	NY	TCH-L	TCH-N
1	7.000	7.000	1.000	6	6	0	0

NON-PASS-THROUGH OBJECT DATA

NUMBERS	COORDINATE NUMBER			
4	3	4	5	6

LAYER DATA

LAYER	W(T/M3)	PHI(DEG)	C(T/M2)	KH	N	LAYER COMPONENTS
1	1.0000	.00	.0000	.0000	11	16 4 5 6 17 7 8 9 10 11 12
2	1.8000	25.00	1.0000	.0000	13	1 2 3 4 5 6 17 7 8 9 10 11 12
3	1.8000	25.00	1.0000	.0000	10	1 2 3 4 5 6 17 7 8 14
4	2.0000	25.00	1.0000	.0000	9	1 2 3 4 5 6 17 18 15

GROUND WATER COMPONENTS

N = 7 LINE = 16 4 5 6 17 18 15

COORDINATE DATA

NO	X	Y
1	-20.000	.000
2	.000	.000
3	4.000	.000
4	8.000	2.000
5	10.000	3.000
6	11.500	3.000
7	15.500	5.000
8	16.900	5.700
9	18.500	6.500
10	21.500	6.500
11	22.500	6.000
12	42.500	6.000
13	-20.000	5.700
14	42.500	5.700
15	42.500	5.000
16	-20.000	2.000
17	11.700	3.100
18	22.500	5.000

EXTERNAL FORCE

N	XA	YA	QA	XB	YB	QB	ALPHA
1	18.500	6.500	1.000	21.500	6.500	1.000	-90.000

SAFETY FACTOR

CX	CY	R	SF	PV	PH	NF	SNF	UF	CL	T	ST	FOR	NEF	TE
7.000	7.000	10.92	1.658	55.47	33.46	72.67	9.87	-49.94	22.87	31.95	1.51	.00	.00	.00
8.000	7.000	11.97	1.553	74.14	47.73	101.27	14.88	-68.32	26.30	42.89	3.30	1.55	.00	.00
9.000	7.000	10.02	1.438	50.45	35.07	62.39	7.77	-39.95	20.24	33.00	1.48	.59	.00	.00
10.000	7.000	11.00	1.366	66.41	48.61	86.82	11.87	-55.51	23.23	43.35	3.05	2.20	.00	.00
11.000	7.000	9.96	1.364	56.68	41.56	69.90	8.83	-42.22	20.17	37.15	2.26	2.15	.00	.00
12.000	7.000	10.99	1.446	73.69	50.97	95.55	13.32	-58.76	23.57	44.83	3.93	2.20	.00	.00
7.000	8.000	13.96	1.599	91.22	57.03	130.21	18.41	-88.01	30.61	51.25	3.66	2.12	.00	.00
8.000	8.000	12.09	1.494	66.24	44.33	86.75	10.70	-56.36	25.14	40.72	2.05	1.56	.00	.00
9.000	8.000	11.10	1.432	56.14	39.22	69.60	7.78	-43.13	21.89	36.41	1.42	1.38	.00	.00
10.000	8.000	12.00	1.399	71.36	51.02	93.69	11.61	-58.21	24.26	45.65	2.87	2.50	.00	.00
11.000	8.000	11.05	1.398	62.63	44.80	77.47	8.76	-45.50	21.90	40.25	2.08	2.47	.00	.00
12.000	8.000	11.47	1.498	72.18	48.18	90.60	10.79	-53.33	24.11	43.11	3.02	2.06	.00	.00
7.000	9.000	13.41	1.586	72.52	45.72	96.57	11.11	-62.67	27.51	42.21	1.80	1.72	.00	.00
8.000	9.000	13.27	1.501	73.99	49.29	97.68	11.13	-61.98	27.16	44.88	2.12	2.29	.00	.00
9.000	9.000	10.98	1.420	44.86	31.60	51.66	4.35	-30.29	19.14	30.26	.15	1.19	.00	.00
10.000	9.000	10.90	1.374	47.66	34.69	54.67	4.61	-30.75	19.13	32.32	.44	1.93	.00	.00
11.000	9.000	11.43	1.379	58.50	42.41	69.75	6.54	-39.11	21.31	38.80	1.29	2.32	.00	.00
12.000	9.000	12.26	1.494	72.85	48.76	91.37	9.73	-52.41	24.15	44.36	2.48	1.93	.00	.00
7.000	10.000	14.57	1.582	79.64	50.33	107.00	11.46	-67.98	29.15	46.11	1.86	2.36	.00	.00
8.000	10.000	12.18	1.480	47.52	32.10	55.59	4.36	-33.05	20.61	30.95	.00	1.15	.00	.00
9.000	10.000	12.77	1.388	59.42	42.81	72.51	6.31	-42.46	23.06	39.47	.81	2.53	.00	.00

CX	CY	R	SF	PV	PH	NF	SNF	UF	CL	T	ST	FOR	NEF	TE
10.000	10.000	11.90	1.322	50.72	38.35	58.57	4.44	-32.24	19.95	35.67	.20	2.48	.00	.00
11.000	10.000	12.72	1.450	65.95	45.49	80.43	7.14	-44.85	23.23	42.00	1.36	2.13	.00	.00
12.000	10.000	13.74	1.556	83.76	53.84	107.94	11.11	-62.07	26.78	49.28	2.82	1.75	.00	.00
7.000	11.000	13.44	1.565	51.07	32.62	61.03	4.56	-36.87	22.34	31.60	-.12	1.14	.00	.00
8.000	11.000	13.25	1.405	51.37	36.56	60.70	4.38	-35.49	21.78	34.93	-.07	1.70	.00	.00
9.000	11.000	13.96	1.432	66.00	46.08	81.73	6.71	-47.19	24.75	42.83	.89	2.35	.00	.00
10.000	11.000	13.30	1.371	59.36	43.29	70.80	5.28	-39.15	22.43	40.54	.47	2.27	.00	.00
11.000	11.000	13.20	1.432	61.71	43.10	73.38	5.51	-39.45	22.27	40.39	.70	2.01	.00	.00
12.000	11.000	13.63	1.571	71.12	45.28	87.11	7.21	-47.12	23.92	42.07	1.46	1.75	.00	.00
7.000	12.000	15.39	1.528	68.79	45.01	87.63	6.99	-52.71	26.88	41.95	.56	2.50	.00	.00
8.000	12.000	14.12	1.425	52.32	36.72	61.48	4.00	-35.31	22.16	34.80	-.24	2.17	.00	.00
9.000	12.000	13.16	1.374	42.74	31.10	46.41	2.44	-24.40	18.30	29.91	-.72	1.91	.00	.00
10.000	12.000	13.70	1.416	54.79	38.70	62.72	3.82	-32.63	20.88	36.63	-.09	2.16	.00	.00
11.000	12.000	14.59	1.495	70.07	46.86	85.87	6.38	-46.58	24.40	44.11	.93	1.83	.00	.00
12.000	12.000	14.61	1.569	73.85	47.08	90.91	6.95	-48.55	24.55	44.24	1.20	1.63	.00	.00

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10.000	10.000	11.90	1.322	50.72	38.35	58.57	4.44	-32.24	19.95	35.67	.20	2.48	.00	.00

Calculation Sheet-1. 2. 3

SLOPE STABILITY ANALYSIS ** DIKE OF LOWER PERCUT RIVER **

THE DETAILED DESIGN STUDY ON MEDAN FLOOD CONTROL PROJECT

** BASIC DATA **

METHOD OF CALCULATION	0	
LAYER NUMBERS	4	
MESH BLOCK NUMBERS	1	
CONDITION OF CALCULATION	0	
CASE OF EARTHQUAKE	0	
CASE OF COHESION	0	
UPPER LAYER	3	
WATER LAYER	2	
GROUND WATER	1	
NON-PASS-THROUGH OBJECT NO.	3	
UNIT WEIGHT OF WATER	1.000	(T/M3)
EARTHQUAKE COEFFICIENT	.000	
SLICE INTERVAL	.500	(M)
LEFT EDGE OF SLICE	-12.000	(M)
RIGHT EDGE OF SLICE	23.000	(M)
LOWER LIMIT Y-COORDINATE	-10.000	(M)
RADIUS PITCH	.500	(M)
ADDITIONAL INITIAL RADIUS	.500	(M)

** CENTER DATA **

BLOCK	XG(M)	YG(M)	D(M)	NX	NY	TCH-L	TCH-N
1	.000	4.000	1.000	5	5	0	0

\#\# NON-PASS-THROUGH OBJECT DATA \#\#

NUMBERS	COORDINATE NUMBER		
3	2	3	4

\#\# LAYER DATA \#\#

LAYER	W(T/M3)	PHI(DEG)	C(T/M2)	KH	N	LAYER COMPONENTS						
1	1.8000	30.00	.0000	.0000	7	1	2	3	4	5	6	7
2	2.0000	30.00	.1000	.0000	5	1	2	3	6	7		
3	1.9000	10.00	1.5000	.0000	4	1	2	6	7			
4	2.0000	26.00	.5000	.0000	2	8	9					

\#\# GROUND WATER COMPONENTS \#\#

N = 5 LINE = 1 2 3 6 7

\#\# COORDINATE DATA \#\#

NO	X	Y
1	-12.000	.000
2	.000	.000
3	6.400	3.200
4	8.000	4.000
5	11.000	4.000
6	19.000	.000
7	23.000	.000
8	-12.000	-2.000
9	23.000	-2.000

\#\# EXTERNAL FORCE \#\#

N	XA	YA	QA	XB	YB	QB	ALPHA
1	8.000	4.000	1.000	11.000	4.000	1.000	-90.000

STABILITY FACTOR

CX	CY	R	SF	PV	PH	NF	SNF	UF	CL	T	ST	FOR	NEF	TE
.000	4.000	8.00	1.494	30.20	20.21	40.17	2.60	-25.55	12.98	14.25	5.96	.00	.00	.00
1.000	4.000	7.45	1.339	27.27	20.37	32.98	1.83	-20.29	12.75	15.01	5.85	-.48	.00	.00
2.000	4.000	6.05	1.358	18.23	13.42	10.97	.39	-6.51	13.38	8.13	5.30	.00	.00	.00
3.000	4.000	5.50	1.306	15.50	11.87	6.35	.36	-3.68	12.47	7.38	4.97	-.48	.00	.00
4.000	4.000	5.85	1.510	19.23	12.74	9.04	.99	-5.14	14.34	9.97	4.40	-1.63	.00	.00
.000	5.000	8.47	1.451	28.08	19.35	34.33	1.45	-20.89	13.18	13.45	5.47	.43	.00	.00
1.000	5.000	8.45	1.303	30.09	23.09	35.08	1.63	-20.88	14.26	16.24	5.54	1.31	.00	.00
2.000	5.000	8.45	1.246	32.21	25.85	38.12	2.15	-22.32	14.26	18.43	5.35	2.07	.00	.00
3.000	5.000	6.95	1.217	20.59	16.93	9.15	.71	-5.26	16.00	10.60	4.69	1.64	.00	.00
4.000	5.000	6.95	1.247	21.70	17.40	10.43	1.02	-5.74	16.00	11.03	4.08	2.29	.00	.00
.000	6.000	9.45	1.433	30.45	21.25	36.01	1.29	-21.32	14.48	14.80	5.28	1.17	.00	.00
1.000	6.000	9.45	1.307	32.23	24.65	38.86	1.63	-22.74	14.48	17.50	5.20	1.96	.00	.00
2.000	6.000	7.98	1.272	21.43	16.85	8.96	.43	-5.16	17.20	10.50	4.85	1.50	.00	.00
3.000	6.000	7.98	1.224	22.48	18.37	10.23	.72	-5.67	17.20	11.78	4.40	2.19	.00	.00
4.000	6.000	7.98	1.311	23.59	17.99	11.53	.98	-6.13	17.20	12.14	3.78	2.06	.00	.00
.000	7.000	10.45	1.409	32.42	23.00	39.57	1.31	-23.11	14.64	16.18	5.05	1.77	.00	.00
1.000	7.000	10.45	1.323	34.36	25.97	42.61	1.68	-24.57	14.64	18.68	4.85	2.43	.00	.00
2.000	7.000	9.45	1.285	26.46	20.59	24.33	.79	-13.43	14.77	13.72	4.51	2.37	.00	.00
3.000	7.000	7.95	1.237	14.59	11.79	5.70	.08	-2.97	11.78	5.11	4.81	1.88	.00	.00
4.000	7.000	8.47	1.395	20.53	14.72	9.64	.60	-4.93	15.22	9.04	3.73	1.94	.00	.00
.000	8.000	11.50	1.442	35.41	24.55	42.05	1.26	-24.07	16.18	17.48	4.80	2.27	.00	.00

CX	CY	R	SF	PV	PH	NF	SNF	UF	CL	T	ST	FOR	NEF	TE
1.000	8.000	10.47	1.344	26.73	19.88	23.02	.49	-12.75	15.96	13.10	4.63	2.15	.00	.00
2.000	8.000	10.47	1.343	28.51	21.23	25.70	.79	-13.94	15.96	14.87	4.22	2.14	.00	.00
3.000	8.000	8.85	1.254	14.50	11.56	5.58	.05	-2.87	11.73	4.59	4.79	2.18	.00	.00
4.000	8.000	9.87	1.504	25.57	17.00	12.55	.83	-6.37	18.55	12.04	3.31	1.66	.00	.00

*** MINIMUM SAFETY FACTOR ***

3.000	5.000	6.95	1.217	20.59	16.93	9.15	.71	-5.26	16.00	10.60	4.69	1.64	.00	.00

1.2.2 MEDAN FLOODWAY

Stability on Channel-Bank

(1) Purpose of Study

The study is made to verify the stability of design channel-bank slope.

(2) Location

Floodway is composed of two types of open channel, namely the downstream channel with a single trapezoidal section and the upstream channel with a double trapezoidal section. Therefore, two cross sections, one from the downstream and one from the upstream, are selected for the analysis. The cross sections shall have a longest channel slope.

Section-A: A representative cross section from the downstream stretch FW3 to FW29 (Section FW19)

Section-B: A representative cross section from the upstream stretch FW29 to FW39 (Section FW32)

(3) Calculation Method

A circular slip method is employed.

(4) Conditions on Calculation

Section	Surface Layer	Second Layer	Third Layer	Fourth Layer
Section-A	CH2, Diluvium Low stiffness Clay 2 - 5 m thick	SC2 ,Diluvium Low to Medium Clayey Sand 3 - 5 m thick	SP6, Diluvium Medium to High Coarse Sand 2 - 3 m thick	SC3, Diluvium Medium to High Clayey Sand 7 - 10 m thick
Section-B	CH2, Diluvium Low stiffness Clay 2 - 5 m thick	SC2, Diluvium Low to Medium Clayey Sand 2 - 5 m thick	SW1, Diluvium Medium to High Sand 3 - 9 m thick	SP6, Diluvium Medium to High Coarse Sand 6 - 10 m thick

Soil properties of each layer are determined as shown in the calculation chart based on the soil test.

(b) Calculation Case

The calculation is made using the water level conditions just after design flood. This case is most critical for the slope stability.

- River side water level : Low water level (1.0 m above riverbed)
- Land side ground water level: 3.0 m lower than ground surface

(c) Loading condition

A uniform load of 1.0 t/m2 is considered on the inspection road..

(5) Results

Calculation results are presented in Figs.1.2.4 to 1.2.5 and Calculation Sheets-1.2.4 to 1.2.5 and summarized below.

Section	Minimum Safety Factor	Allowable Safety Factor	Evaluation
Section-A	1.216	1.20	Stable
Section-B	1.222	1.20	Stable

Judging from the results, slopes of floodway are stable. For the stability of channel slope of upstream, the ground water level must be lowered by means of the drain pipes installed in the leaning wall.

ANALYSIS ON SLOPE STABILITY

1. LOCATION : FLOODWAY
2. CALUCULATION SECTION : FW19
3. BORING DATA : B 31
4. CALUCULATION CONDITION : AFTER FLOOD

PROPERTY OF SOIL

LAYER	γ (tf/m³)	Ø (°)	C (tf/m²)
1	1.0	0	0.0
2	1.5	15	2.5
3	1.9		
4	1.8	24	1.0
5	2.0		
6	1.9	35	0.5
7	2.0		
8	2.0	30	0.5

CALCULATED MINIMUM SAFETY FACTOR

$$SF = \frac{Pv}{Ph} = \frac{110.34}{90.75} = 1.216$$

Radius = 17.55m

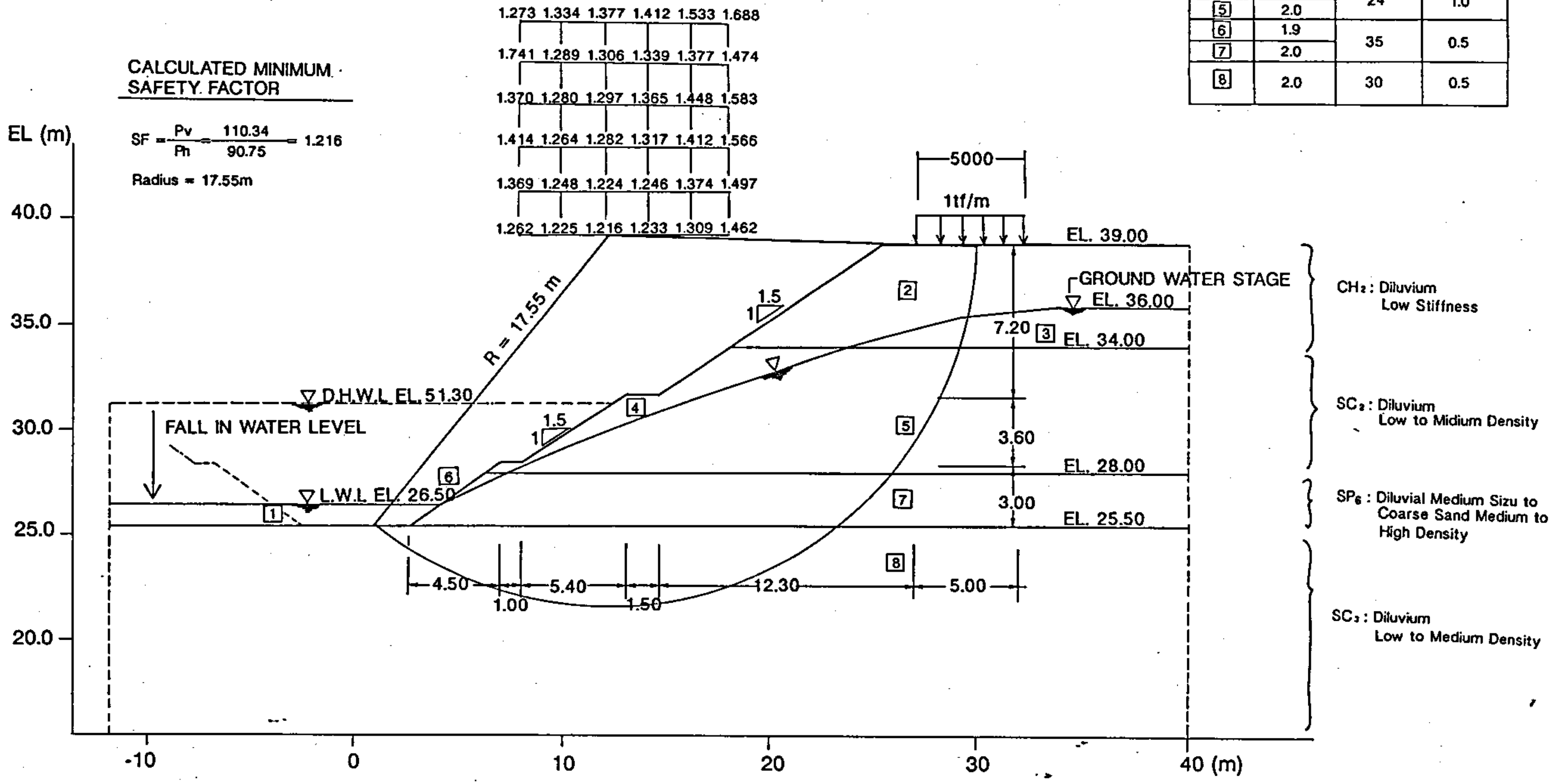

Fig. 1. 2. 4 RESULT OF SLOPE STABILITY ANALYSIS IN LOWER FLOODWAY

ANALYSIS ON SLOPE STABILITY

1. LOCATION : FLOODWAY
2. CALCULATION SECTION : FW32
3. BORING DATA : B34
4. CALCULATION CONDITION : AFTER FLOOD

CALCULATION RESULTS

Minimum Safety Factor

$$SF = \frac{Pv}{Ph} = \frac{196.53}{161.67}$$

$$= 1.216$$

Radius = 18.90m

PROPERTY OF SOIL

LAYER	γ (tf/m³)	Ø(°)	C(tf/m²)
1	1.0	0	0.0
2	1.5	15	2.5
3	1.9		
4	1.8	24	1.0
5	2.0		
6	1.8	30	1.0
7	2.0		
8	2.0	35.0	1.5

SATETY FACTOR

1.338	1.331	1.359	1.506	1.706
1.306	1.294	1.310	1.442	1.659
1.268	1.243	1.276	1.373	1.581
1.241	1.222	1.222	1.330	1.529
1.272	1.216	1.236	1.317	1.508

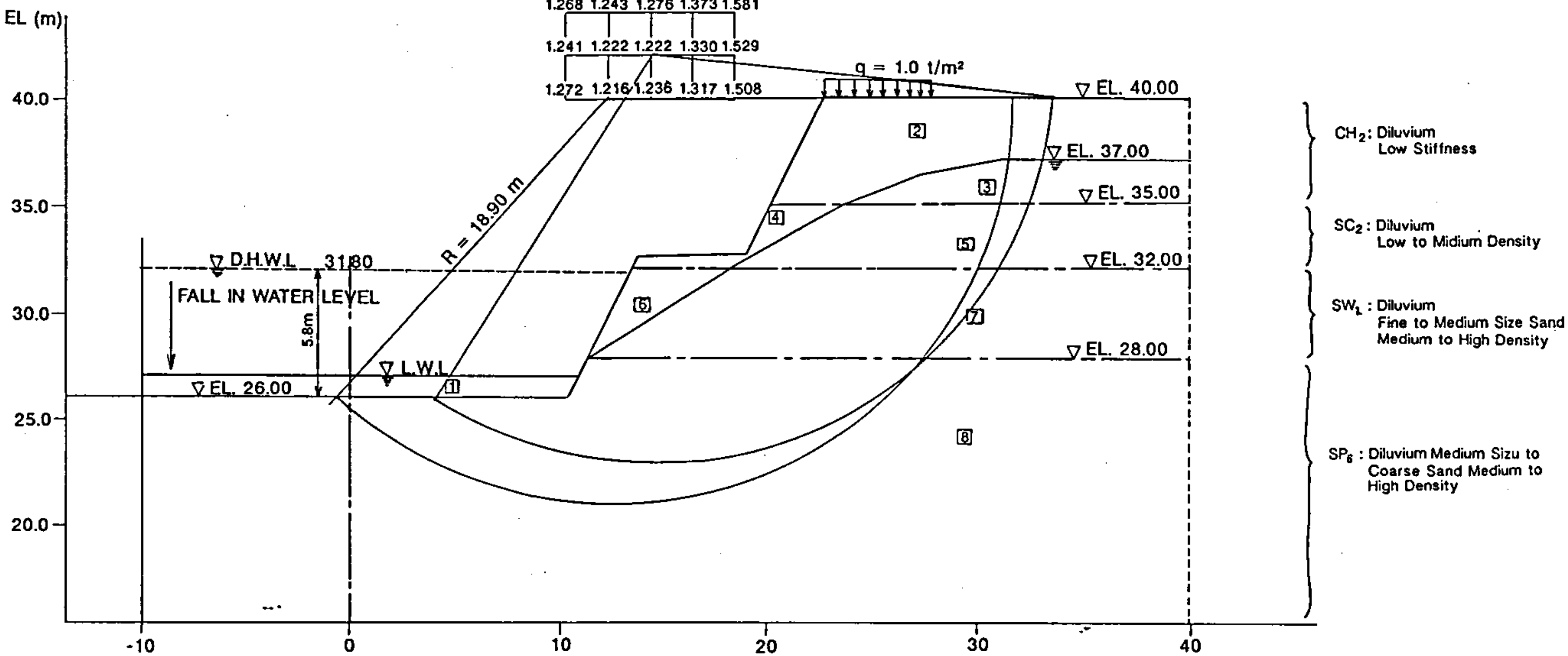

Fig. 1. 2. 5 RESULT OF SLOPE STABILITY ANALYSIS IN UPPER FLOODWAY

SLOPE STABILITY ANALISIS ***** MEDAN FLOODWAY (CHANNEL TYPE-I) *****
THE DETAILED DESIGN STUDY ON MEDAN FLOOD CONTROL PROJECT

** BASIC DATA **

METHOD OF CALCULATION	0	
LAYER NUMBERS	8	
MESH BLOCK NUMBERS	1	
CONDITION OF CALCULATION	0	
CASE OF EARTHQUAKE	0	
CASE OF COHESION	0	
UPPER LAYER	8	
WATER LAYER	1	
GROUND WATER	1	
NON-PASS-THROUGH OBJECT NO.	2	
UNIT WEIGHT OF WATER	1.000	(T/M3)
EARTHQUAKE COEFFICIENT	.000	
SLICE INTERVAL	1.000	(M)
LEFT EDGE OF SLICE	-20.000	(M)
RIGHT EDGE OF SLICE	50.000	(M)
LOWER LIMIT Y-COORDINATE	-10.000	(M)
RADIUS PITCH	1.000	(M)
ADDITIONAL INITIAL RADIUS	1.000	(M)

** CENTER DATA **

BLOCK	XG(M)	YG(M)	D(M)	NX	NY	TCH-L	TCH-N
1	8.000	14.000	2.000	7	7	0	0

** NON-PASS-THROUGH OBJECT DATA **

NUMBERS	COORDINATE NUMBER	
2	1	2

** LAYER DATA **

LAYER	W(T/M3)	PHI(DEG)	C(T/M2)	KH	N	LAYER COMPONENTS
1	1.0000	.00	.0000	.0000	11	16 5 6 7 8 9 10 11 12 13 14
2	1.5000	15.00	2.5000	.0000	14	1 2 3 4 5 6 7 8 9 10 11 12 13 14
3	1.9000	15.00	2.5000	.0000	16	1 2 3 4 5 6 7 8 9 10 11 12 19 20 21 22
4	1.8000	24.00	1.0000	.0000	14	1 2 3 4 5 6 7 8 9 10 11 12 19 23
5	2.0000	24.00	1.0000	.0000	10	1 2 3 4 5 6 17 18 19 23
6	1.8000	35.00	.5000	.0000	8	1 2 3 4 5 6 17 24
7	2.0000	35.00	.5000	.0000	7	1 2 3 4 5 17 24
8	2.0000	30.00	.5000	.0000	5	1 2 3 4 25

** GROUND WATER COMPONENTS **

N = 8   LINE = 16 5 17 18 19 20 21 22

** COORDINATE DATA **

NO	X	Y
1	-20.000	.000
2	.000	.000
3	2.500	.000
4	2.500	.000
5	4.000	1.000
6	6.250	2.500
7	7.000	3.000
8	8.000	3.000
9	12.200	5.800
10	12.950	6.300
11	14.430	6.300

12	17.750	8.500
13	25.250	13.500
14	50.000	13.500
15	-20.000	5.800
16	-20.000	1.000
17	7.300	2.500
18	15.500	6.300
19	22.250	8.500
20	28.000	10.000
21	32.750	10.500
22	50.000	10.500
23	50.000	8.500
24	50.000	2.500
25	50.000	.000

EXTERNAL FORCE

N	XA	YA	QA	XB	YB	QB	ALPHA
1	26.750	13.500	1.000	31.750	13.500	1.000	-90.000

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CX	CY	R	SF	PV	PH	NF	SNF	UF	CL	T	ST	FOR	NEF	TE
8.000	14.000	16.10	1.262	50.64	40.13	84.63	.61	-42.91	8.31	20.20	19.93	.00	.00	.00
10.000	14.000	16.95	1.225	82.51	67.34	137.21	3.39	-68.23	10.15	38.77	28.33	.24	.00	.00
12.000	14.000	17.55	1.216	110.34	90.75	183.35	6.75	-91.13	11.36	51.92	36.13	2.70	.00	.00
14.000	14.000	18.90	1.233	160.89	130.44	269.23	15.04	-137.30	13.92	79.68	46.75	4.01	.00	.00
16.000	14.000	20.90	1.309	237.05	181.10	398.14	32.67	-211.25	17.48	120.75	57.20	3.15	.00	.00
18.000	14.000	20.90	1.462	256.74	175.65	424.37	37.93	-223.06	17.48	114.72	58.24	2.68	.00	.00
20.000	14.000	20.90	1.679	275.71	164.22	449.80	43.55	-235.12	17.48	104.58	57.44	2.20	.00	.00
8.000	16.000	17.25	1.369	34.45	25.17	56.82	-.16	-28.81	6.61	8.88	16.29	.00	.00	.00
10.000	16.000	18.70	1.248	81.54	65.31	135.46	2.41	-66.50	10.17	35.14	28.58	1.59	.00	.00
12.000	16.000	20.00	1.224	125.16	102.23	212.88	7.62	-108.20	12.87	58.14	39.77	4.31	.00	.00

CX	CY	R	SF	PV	PH	NF	SNF	UF	CL	T	ST	FOR	NEF	TE
14.000	16.000	20.95	1.246	174.28	139.85	291.22	14.10	-145.73	14.70	87.38	48.84	3.63	.00	.00
16.000	16.000	20.95	1.374	193.16	140.57	317.59	17.69	-156.83	14.70	85.77	51.64	3.15	.00	.00
18.000	16.000	23.00	1.497	273.69	182.85	453.73	37.16	-235.63	18.43	121.68	58.72	2.45	.00	.00
20.000	16.000	23.00	1.704	293.30	172.13	479.85	42.49	-247.48	18.43	111.99	58.13	2.01	.00	.00
8.000	18.000	19.10	1.411	34.08	24.16	55.46	-.27	-27.62	6.51	8.02	16.14	.00	.00	.00
10.000	18.000	20.50	1.264	80.61	63.78	134.04	1.71	-65.37	10.23	32.06	28.90	2.83	.00	.00
12.000	18.000	20.90	1.282	107.36	83.73	176.37	3.73	-83.88	11.14	43.60	36.03	4.11	.00	.00
14.000	18.000	21.95	1.317	155.10	117.74	255.19	8.90	-122.36	13.37	68.61	45.67	3.47	.00	.00
16.000	18.000	24.00	1.412	231.28	163.83	388.56	23.25	-197.87	17.34	105.15	55.91	2.76	.00	.00
18.000	18.000	23.90	1.566	249.32	159.23	411.31	26.94	-206.09	17.16	100.32	56.57	2.34	.00	.00
20.000	18.000	25.90	1.720	335.99	195.30	555.95	48.72	-289.46	20.78	133.22	60.29	1.78	.00	.00
8.000	20.000	21.00	1.370	33.72	24.62	54.64	-.33	-27.10	6.51	7.30	16.20	1.12	.00	.00
10.000	20.000	22.10	1.280	68.93	53.84	117.18	.85	-58.81	9.71	22.18	28.00	3.66	.00	.00
12.000	20.000	23.10	1.297	119.85	92.38	197.22	3.85	-93.33	12.10	50.34	38.30	3.75	.00	.00
14.000	20.000	23.70	1.365	153.77	112.69	253.30	7.32	-120.27	13.42	63.54	45.96	3.19	.00	.00
16.000	20.000	24.75	1.448	206.03	142.27	339.20	14.84	-163.60	15.59	86.78	52.83	2.66	.00	.00
18.000	20.000	25.95	1.583	264.04	166.85	435.13	26.28	-215.29	17.92	107.85	56.83	2.16	.00	.00
20.000	20.000	27.95	1.741	352.30	202.41	584.14	47.30	-300.75	21.61	140.41	60.36	1.65	.00	.00
8.000	22.000	23.20	1.289	40.04	31.06	66.03	-.46	-33.02	7.49	9.85	18.83	2.38	.00	.00
10.000	22.000	24.10	1.306	78.66	60.25	129.47	.72	-61.66	10.13	27.27	28.97	4.01	.00	.00
12.000	22.000	25.00	1.339	119.22	89.01	198.17	3.25	-94.57	12.37	46.51	39.04	3.45	.00	.00
14.000	22.000	26.00	1.377	168.55	122.37	280.71	7.70	-134.47	14.61	70.40	49.03	2.93	.00	.00
16.000	22.000	26.95	1.474	221.49	150.30	366.06	15.21	-176.37	16.59	93.64	54.20	2.45	.00	.00

CX	CY	R	SF	PV	PH	NF	SNF	UF	CL	T	ST	FOR	NEF	TE
18.000	22.000	26.95	1.642	240.53	146.45	393.05	18.67	-187.78	16.59	88.97	55.40	2.08	.00	.00
20.000	22.000	28.75	1.794	322.24	179.58	529.93	35.54	-263.34	20.11	120.02	57.96	1.60	.00	.00
8.000	24.000	25.10	1.273	39.69	31.17	65.01	-.51	-32.27	7.46	9.11	18.80	3.26	.00	.00
10.000	24.000	26.00	1.334	77.90	58.38	128.94	.45	-61.75	10.26	25.28	29.40	3.70	.00	.00
12.000	24.000	26.70	1.377	117.58	85.38	192.85	2.34	-89.73	12.11	43.55	38.58	3.24	.00	.00
14.000	24.000	27.60	1.412	165.76	117.37	272.37	5.93	-126.80	14.25	66.32	48.27	2.77	.00	.00
16.000	24.000	28.10	1.533	201.16	131.22	329.12	10.45	-153.79	15.37	76.44	52.41	2.37	.00	.00
18.000	24.000	30.00	1.688	280.99	166.47	468.28	24.78	-231.37	19.30	107.46	57.13	1.88	.00	.00
20.000	24.000	30.95	1.823	340.25	186.61	562.04	35.83	-278.77	21.15	126.80	58.32	1.49	.00	.00
8.000	26.000	27.00	1.273	39.46	31.01	63.89	-.52	-31.28	7.37	8.47	18.60	3.94	.00	.00
10.000	26.000	27.00	1.383	48.82	35.31	79.33	-.15	-37.74	7.37	8.69	23.06	3.56	.00	.00
12.000	26.000	28.00	1.431	91.02	63.60	152.17	.97	-72.77	10.65	24.75	35.77	3.08	.00	.00
14.000	26.000	29.30	1.449	163.76	112.98	266.69	4.70	-121.67	14.04	62.48	47.89	2.61	.00	.00
16.000	26.000	29.90	1.578	199.65	126.55	327.20	9.14	-152.13	15.44	71.84	52.51	2.21	.00	.00
18.000	26.000	30.90	1.719	254.47	148.00	417.65	17.13	-197.94	17.64	90.57	55.62	1.81	.00	.00
20.000	26.000	31.95	1.885	313.34	166.20	513.92	27.35	-247.74	19.81	108.00	56.76	1.45	.00	.00

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12.000	14.000	17.55	1.216	110.34	90.75	183.35	6.75	-91.13	11.36	51.92	36.13	2.70	.00	.00

Calculation Sheet-1. 2. 5

SLOPE STABILITY ANALYSIS ### FLOODWAY CHANNEL TYPE-II ##

THE DETAILED DESIGN STUDY ON MEDAN FLOOD CONTROL PROJECT

BASIC DATA

METHOD OF CALCULATION	0	
LAYER NUMBERS	8	
MESH BLOCK NUMBERS	1	
CONDITION OF CALCULATION	0	
CASE OF EARTHQUAKE	0	
CASE OF COHESION	0	
UPPER LAYER	8	
WATER LAYER	1	
GROUND WATER	1	
NON-PASS-THROUGH OBJECT NO.	3	
UNIT WEIGHT OF WATER	1.000	(T/M3)
EARTHQUAKE COEFFICIENT	.000	
SLICE INTERVAL	1.000	(M)
LEFT EDGE OF SLICE	-20.000	(M)
RIGHT EDGE OF SLICE	50.000	(M)
LOWER LIMIT Y-COORDINATE	-30.000	(M)
RADIUS PITCH	1.000	(M)
ADDITIONAL INITIAL RADIUS	1.000	(M)

CENTER DATA

BLOCK	XG(M)	YG(M)	D(M)	NX	NY	TCH-L	TCH-N
1	8.000	40.000	2.000	7	7	0	0

** NON-PASS-THROUGH OBJECT DATA **

NUMBERS	COORDINATE NUMBER
3	19 20 4

** LAYER DATA **

LAYER	W(T/M3)	PHI(DEG)	C(T/M2)	KH	N	LAYER COMPONENTS
1	1.0000	.00	.0000	.0000	10	14 15 16 2 3 4 5 6 7 8
2	1.5000	15.00	2.5000	.0000	12	18 19 20 15 16 2 3 4 5 6 7 8
3	1.9000	15.00	2.5000	.0000	13	18 19 20 15 16 2 3 4 5 6 11 9 10
4	1.8000	24.00	1.0000	.0000	12	18 19 20 15 16 2 3 4 5 6 11 12
5	2.0000	24.00	1.0000	.0000	10	18 19 20 15 16 2 3 17 11 12
6	1.8000	30.00	1.0000	.0000	9	18 19 20 15 16 2 3 17 13
7	2.0000	30.00	1.0000	.0000	7	18 19 20 15 16 17 13
8	2.0000	35.00	1.5000	.0000	6	18 19 20 15 16 21

** GROUND WATER COMPONENTS **

N = 9 LINE = 18 19 20 15 16 17 11 9 10

** COORDINATE DATA **

NO	X	Y
1	-10.000	31.800
2	12.900	31.800
3	13.000	32.000
4	13.300	32.600
5	18.300	32.750
6	19.425	35.000
7	22.000	40.000
8	40.000	40.000
9	26.500	37.000
10	40.000	37.000

11	22.950	35.000
12	40.050	35.000
13	40.000	32.000
14	-10.000	27.000
15	10.500	27.000
16	11.000	28.000
17	17.500	32.000
18	-10.000	26.000
19	8.500	26.000
20	10.000	26.000
21	40.000	28.000

EXTERNAL FORCE

N	XA	YA	QA	XB	YB	QB	ALPHA
1	22.000	40.000	1.000	27.000	40.000	1.000	-90.000

SAFETY FACTOR

CX	CY	R	SF	PV	PH	NF	SNF	UF	CL	T	ST	FOR	NEF	TE
8.000	40.000	21.27	1.363	236.65	173.66	366.38	15.61	-203.50	58.16	137.78	39.74	-3.86	.00	.00
10.000	40.000	19.74	1.272	203.04	159.58	304.47	9.43	-161.18	50.32	124.00	39.21	-3.64	.00	.00
12.000	40.000	18.90	1.216	196.55	161.67	290.27	8.40	-148.02	45.90	122.65	42.31	-3.29	.00	.00
14.000	40.000	16.70	1.236	150.03	121.33	209.43	2.51	-95.61	33.70	87.92	36.52	-3.11	.00	.00
16.000	40.000	15.95	1.317	150.19	114.06	209.31	3.07	-91.35	29.15	77.48	39.24	-2.66	.00	.00
18.000	40.000	18.90	1.508	272.77	180.93	392.81	30.99	-196.93	45.90	132.91	49.73	-1.71	.00	.00
20.000	40.000	18.90	1.714	298.44	174.15	426.83	38.80	-213.08	45.90	125.62	49.71	-1.18	.00	.00
8.000	42.000	23.22	1.326	254.91	192.22	394.57	14.18	-214.21	60.37	145.90	42.78	3.54	.00	.00
10.000	42.000	20.94	1.241	194.80	156.98	284.02	4.90	-142.44	48.32	116.29	37.24	3.45	.00	.00
12.000	42.000	18.90	1.222	147.99	121.11	203.23	.71	-92.73	36.79	85.71	32.11	3.29	.00	.00
14.000	42.000	18.90	1.222	167.85	137.42	236.94	2.56	-108.43	36.79	94.32	40.33	2.76	.00	.00

CX	CY	R	SF	PV	PH	NF	SNF	UF	CL	T	ST	FOR	NEF	TE
16.000	42.000	17.70	1.330	149.74	112.57	209.31	1.78	-90.56	29.21	70.27	39.93	2.37	.00	.00
18.000	42.000	18.90	1.529	216.37	141.47	305.66	14.61	-140.69	36.79	95.81	43.95	1.71	.00	.00
20.000	42.000	20.90	1.729	315.68	182.63	452.80	36.89	-222.14	48.13	132.40	49.16	1.07	.00	.00
8.000	44.000	25.23	1.369	275.61	201.30	425.29	14.50	-227.00	62.81	153.51	44.53	3.26	.00	.00
10.000	44.000	23.03	1.268	212.99	167.96	312.66	4.64	-155.30	51.00	124.08	40.73	3.15	.00	.00
12.000	44.000	20.95	1.243	163.28	131.31	225.26	-.08	-100.83	38.92	92.68	35.65	2.98	.00	.00
14.000	44.000	20.95	1.276	184.53	144.60	259.40	3.08	-116.88	38.92	100.60	41.51	2.50	.00	.00
16.000	44.000	19.75	1.373	164.83	120.03	228.83	2.27	-97.39	31.12	77.16	40.74	2.14	.00	.00
18.000	44.000	20.95	1.581	233.08	147.46	328.52	14.47	-148.83	38.92	101.94	43.98	1.55	.00	.00
20.000	44.000	23.00	1.785	335.62	188.02	483.23	36.08	-234.52	50.82	138.13	48.91	.98	.00	.00
8.000	46.000	25.15	1.372	213.65	155.73	309.58	2.68	-152.34	53.72	116.28	36.18	3.27	.00	.00
10.000	46.000	23.78	1.306	183.86	140.83	257.00	-.04	-118.87	45.77	102.11	35.69	3.03	.00	.00
12.000	46.000	23.00	1.294	165.96	128.27	234.79	-.96	-108.86	41.00	86.07	39.48	2.72	.00	.00
14.000	46.000	21.95	1.310	159.82	122.03	219.04	-.79	-92.47	34.04	80.58	39.07	2.39	.00	.00
16.000	46.000	21.95	1.442	182.77	126.78	253.66	3.49	-108.42	34.04	83.52	41.32	1.93	.00	.00
18.000	46.000	23.90	1.659	279.74	168.66	400.08	20.25	-187.09	46.50	121.44	45.86	1.35	.00	.00
20.000	46.000	25.90	1.848	387.39	209.62	565.37	42.43	-278.32	57.91	158.00	50.75	.87	.00	.00
8.000	48.000	25.97	1.426	184.66	129.50	253.77	-.55	-117.34	48.79	95.37	30.98	3.16	.00	.00
10.000	48.000	25.97	1.338	202.86	151.65	287.12	.08	-133.13	48.80	109.40	39.47	2.79	.00	.00
12.000	48.000	24.70	1.331	163.47	122.85	230.09	-2.02	-105.32	40.72	80.16	40.18	2.51	.00	.00
14.000	48.000	23.50	1.359	141.85	104.38	196.13	-2.23	-84.19	32.15	62.99	39.17	2.22	.00	.00
16.000	48.000	23.70	1.506	181.09	120.28	250.87	2.54	-106.00	33.68	77.39	41.10	1.78	.00	.00
18.000	48.000	25.95	1.706	297.05	174.11	424.44	19.96	-196.00	48.64	127.16	45.70	1.25	.00	.00

CX	CY	R	SF	PV	PH	NF	SNF	UF	CL	T	ST	FOR	NEF	TE
20.000	48.000	27.95	1.896	406.65	214.52	594.65	41.31	-289.59	60.28	163.43	50.29	.80	.00	.00
8.000	50.000	28.15	1.434	203.06	141.64	283.26	-1.02	-130.86	51.68	103.33	35.39	2.93	.00	.00
10.000	50.000	26.92	1.372	175.35	127.77	236.83	-2.39	-102.89	43.80	90.03	35.06	2.68	.00	.00
12.000	50.000	26.15	1.361	157.36	115.60	214.87	-2.78	-93.14	38.42	75.13	38.08	2.39	.00	.00
14.000	50.000	25.10	1.401	137.86	98.38	187.30	-2.61	-76.92	30.10	58.49	37.78	2.10	.00	.00
16.000	50.000	25.90	1.569	198.94	126.83	275.68	3.59	-116.90	36.57	83.70	41.49	1.63	.00	.00
18.000	50.000	26.95	1.754	266.70	152.01	376.64	13.22	-167.14	43.98	107.18	43.63	1.20	.00	.00
20.000	50.000	28.75	1.948	368.58	189.21	532.39	30.11	-249.22	55.30	141.36	47.07	.78	.00	.00
8.000	52.000	29.74	1.455	197.83	135.95	271.38	-2.15	-122.17	50.77	98.06	35.13	2.76	.00	.00
10.000	52.000	29.22	1.402	195.06	139.12	269.03	-2.14	-119.21	47.38	97.43	39.20	2.49	.00	.00
12.000	52.000	28.00	1.391	156.04	112.15	213.87	-3.30	-93.21	38.69	70.41	39.51	2.23	.00	.00
14.000	52.000	27.00	1.427	136.69	95.77	187.18	-2.97	-77.85	30.33	54.56	39.26	1.94	.00	.00
16.000	52.000	28.10	1.629	217.06	133.23	301.16	4.57	-128.12	39.45	89.90	41.81	1.52	.00	.00
18.000	52.000	30.95	1.832	366.05	199.81	531.22	25.63	-249.14	58.34	151.58	47.18	1.05	.00	.00
20.000	52.000	30.95	2.001	390.26	195.00	566.14	30.59	-264.80	58.34	147.13	47.14	.73	.00	.00

*** MINIMUM SAFETY FACTOR ***

12.000	40.000	18.90	1.216	196.55	161.67	290.27	8.40	-148.02	45.90	122.65	42.31	-3.29	.00	.00

1.2.3 DELI RIVER

Stability on Channel-Bank

(1) Purpose of Study

An embankment for Zone-D is made in the river stretch UD14 to UD21 Therefore, the slope stability of right side embankment (Zone-D) is assessed.

(2) Location and Description of Slope

The cross section UD15 is used for the study. The embankment height is about 5.0 m and the distance between design riverbed and top of embankment comes to about 10.0 m. The slope gradient is 1 : 2.

(3) Calculation Method

A circular slip method is employed.

(4) Conditions on Calculation

Section	Embankment	First Layer	Second Layer	Third Layer
Section-A	Sandy material γ=1.8 t/m3 ϕ=30°,c=0 t/m2	CL3, Diluvium Low stiffness Silty & Sandy Clay, 5 m thick	SP6, Diluvium Medium to High Coarse Sand 2 - 3 m thick	SW2, Diluvium High density Medium size Sand, 10 m thick

Soil properties of each layer are determined as shown in the calculation chart based on the soil test.

(b) Calculation Case

The calculation is made using the water level conditions just after design flood. This case is most critical for the slope stability.

- River side water level : Low water level (3.0 m above riverbed)
- Land side ground water level: 1.0 m below ground surface line

(c) Loading condition

A uniform load of 1.0 t/m2 is considered on the top of embankment

(5) Results

Calculation results are presented in Fg.1.2.6 and Calculation Sheet-1.2.6 and summarized below.

Section	Minimum Safety Factor	Allowable Safety Factor	Evaluation
Section-A	1.486	1.20	Stable

Judging from the results, the slope of embankment is stable enough.

ANALYSIS ON SLOPE STABILITY

1. LOCATION : DELI RETARDING CHANNEL
2. CALUCULATION SECTION : UD15
3. BORING DATA : B36
4. CALCULATION CONDITION : AFTER FLOOD

CALCULATION RESULTS

Minimum Safety Factor

$$SF = \frac{Pv}{Ph} = \frac{103.21}{69.46} = 1.486 > 1.20$$

Radius = 15.95 m

PROPERTY OF SOIL

	γ (tf/m³)	C (tf/m²)	Ø (°)
1	1.0	0.0	0.0
2	1.8	0.0	30.0
3	2.0	0.0	30.0
4	1.6	3.0	0.0
5	1.9	3.0	0.0
6	2.0	0.0	30.0
7	2.0	0.0	35.0

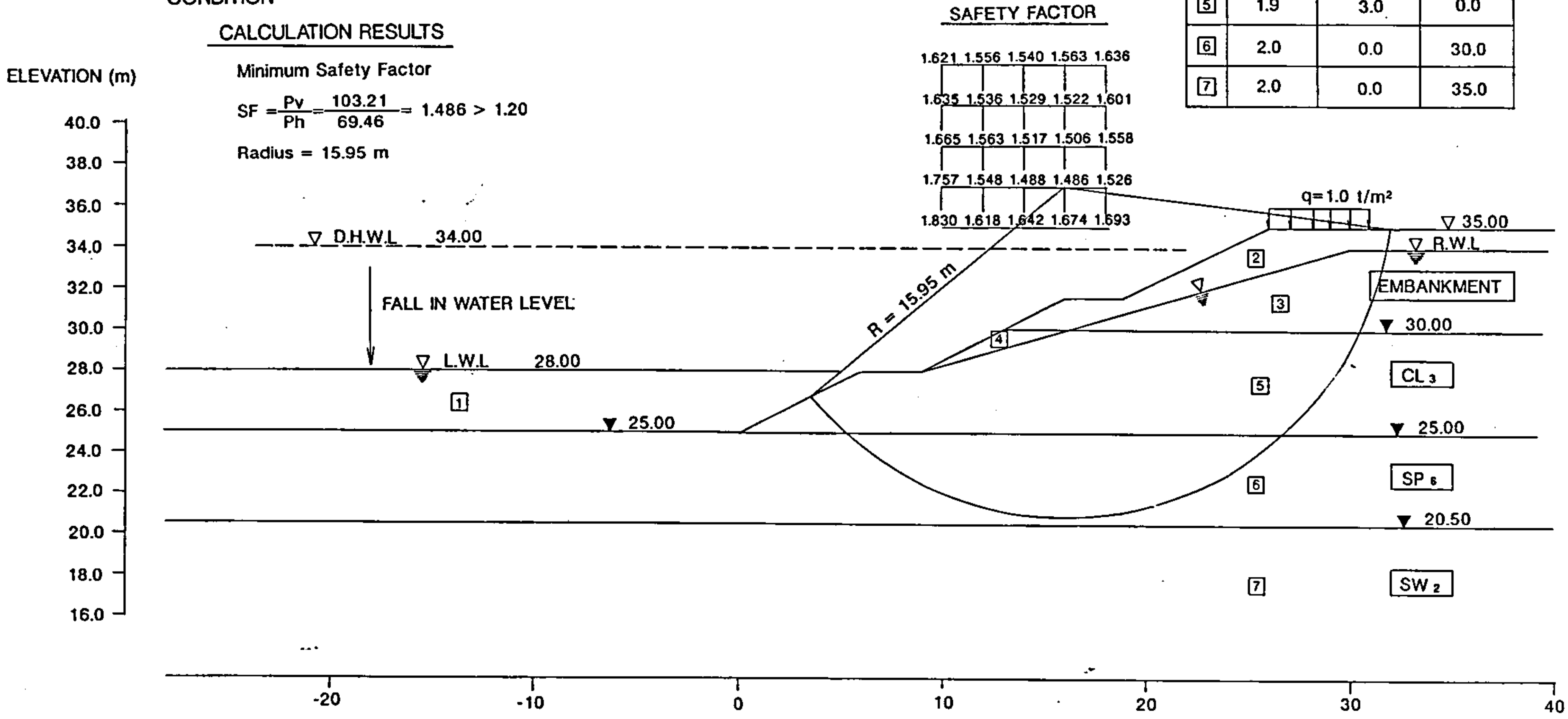


Fig. 1. 2. 6 RESULT OF SLOPE STABILITY ANALYSIS IN DELI RIVER RETARDING CHANNEL

Calculation Sheet-1. 2. 6

SLOPE STABILITY ANALYSIS **** DELI RIVER RETARDING CHANNEL ****
THE DETAILED DESIGN STUDY ON MEDAN FLOOD CONTROL PROJECT

** BASIC DATA **

METHOD OF CALCULATION	0	
LAYER NUMBERS	7	
MESH BLOCK NUMBERS	1	
CONDITION OF CALCULATION	0	
CASE OF EARTHQUAKE	0	
CASE OF COHESION	0	
UPPER LAYER	6	
WATER LAYER	1	
GROUND WATER	1	
NON-PASS-THROUGH OBJECT NO.	5	
UNIT WEIGHT OF WATER	1.000	(T/M3)
EARTHQUAKE COEFFICIENT	.000	
SLICE INTERVAL	1.000	(M)
LEFT EDGE OF SLICE	-20.000	(M)
RIGHT EDGE OF SLICE	50.000	(M)
LOWER LIMIT Y-COORDINATE	12.000	(M)
RADIUS PITCH	1.000	(M)
ADDITIONAL INITIAL RADIUS	1.000	(M)

** CENTER DATA **

BLOCK	XG(M)	YG(M)	D(M)	NX	NY	TCH-L	TCH-N
1	8.000	35.000	2.000	7	7	0	0

** NON-PASS-THROUGH OBJECT DATA **

NUMBERS	COORDINATE NUMBER				
5	3	4	5	6	7

** LAYER DATA **

LAYER	W(T/M3)	PHI(DEG)	C(T/M2)	KH	N	LAYER COMPONENTS								
1	1.0000	.00	.0000	.0000	8	1	2	3	4	5	6	7	8	
2	1.8000	30.00	.0000	.0000	9	12	13	2	3	4	5	6	7	8
3	2.0000	30.00	.0000	.0000	8	12	13	2	3	4	9	10	17	
4	1.6000	.00	3.0000	.0000	7	12	13	2	3	4	9	11		
5	1.9000	.00	3.0000	.0000	6	12	13	2	3	9	11			
6	2.0000	30.00	.0000	.0000	3	12	13	14						
7	2.0000	35.00	.0000	.0000	2	15	16							

** GROUND WATER COMPONENTS **

N = 7 LINE = 12 13 2 3 9 10 17

** COORDINATE DATA **

NO	X	Y
1	-28.000	28.000
2	6.000	28.000
3	9.000	28.000
4	13.000	30.000
5	16.000	31.500
6	19.000	31.500
7	26.000	35.000
8	40.000	35.000
9	16.000	30.000
10	30.000	34.000
11	40.000	30.000
12	-28.000	25.000
13	.000	25.000

14	40.000	25.000
15	-28.000	20.500
16	40.000	20.500
17	40.000	34.000

** EXTERNAL FORCE **

N	XA	YA	QA	XB	YB	QB	ALPHA
1	26.000	35.000	1.000	31.000	35.000	1.000	-90.000

*** SAFETY FACTOR AND COMPONENTS OF FORCE ***

CX	CY	R	SF	PV	PH	NF	SNF	UF	CL	T	ST	FOR	NEF	TE
8.000	35.000	21.95	1.999	301.96	151.04	596.26	68.40	-362.70	.00	107.96	46.72	-3.64	.00	.00
10.000	35.000	16.90	1.830	153.76	84.02	295.18	25.70	-167.12	.00	56.48	28.51	-.97	.00	.00
12.000	35.000	14.00	1.618	78.42	48.48	151.17	10.71	-83.46	.00	27.53	20.95	.00	.00	.00
14.000	35.000	13.90	1.642	83.98	51.16	161.08	11.01	-88.11	.00	30.20	22.82	-1.86	.00	.00
16.000	35.000	13.90	1.674	91.99	54.94	174.62	11.58	-94.21	.00	33.16	25.21	-3.43	.00	.00
18.000	35.000	13.90	1.693	101.57	60.00	188.86	13.26	-100.55	.00	34.43	29.32	-3.75	.00	.00
20.000	35.000	14.50	1.714	123.90	72.30	228.63	18.91	-123.64	.00	40.50	34.75	-2.95	.00	.00
8.000	37.000	22.90	1.902	287.62	151.23	563.24	54.37	-329.99	.00	103.48	43.38	4.37	.00	.00
10.000	37.000	18.90	1.757	166.35	94.68	318.25	23.50	-175.40	.00	61.73	30.29	2.66	.00	.00
12.000	37.000	15.95	1.548	87.45	56.50	165.50	9.59	-87.64	.00	32.10	22.65	1.75	.00	.00
14.000	37.000	15.95	1.488	94.71	63.64	179.48	10.43	-95.20	.00	34.91	25.35	3.38	.00	.00
16.000	37.000	15.95	1.486	103.21	69.46	194.39	10.94	-102.11	.00	37.87	27.69	3.91	.00	.00
18.000	37.000	15.95	1.527	112.54	73.69	209.10	12.16	-108.72	.00	38.90	31.51	3.28	.00	.00
20.000	37.000	15.95	1.646	122.00	74.11	223.76	14.26	-116.02	.00	36.82	34.63	2.66	.00	.00

CX	CY	R	SF	PV	PH	NF	SNF	UF	CL	T	ST	FOR	NEF	TE
8.000	39.000	18.90	1.888	109.07	57.77	205.09	9.79	-105.81	.00	36.35	20.86	.55	.00	.00
10.000	39.000	17.55	1.665	79.71	47.87	148.67	6.39	-75.35	.00	27.10	19.61	1.17	.00	.00
12.000	39.000	17.55	1.563	85.72	54.83	161.71	7.34	-83.33	.00	29.17	22.79	2.86	.00	.00
14.000	39.000	17.55	1.517	93.14	61.38	175.95	8.01	-90.81	.00	31.73	25.55	4.10	.00	.00
16.000	39.000	18.20	1.506	116.07	77.07	219.62	11.48	-115.03	.00	42.11	31.51	3.45	.00	.00
18.000	39.000	18.20	1.558	124.65	80.01	234.92	12.14	-122.41	.00	42.66	34.45	2.90	.00	.00
20.000	39.000	17.50	1.688	107.99	63.96	207.73	10.15	-109.89	.00	25.67	35.90	2.40	.00	.00
8.000	41.000	19.75	1.797	84.60	47.09	155.22	5.40	-76.02	.00	27.95	18.23	.90	.00	.00
10.000	41.000	19.75	1.635	90.09	55.10	167.53	6.32	-83.76	.00	30.93	21.56	2.61	.00	.00
12.000	41.000	19.75	1.536	96.82	63.02	181.79	7.34	-92.31	.00	33.75	25.14	4.12	.00	.00
14.000	41.000	19.75	1.529	105.05	68.71	197.59	8.18	-100.72	.00	36.62	28.44	3.64	.00	.00
16.000	41.000	20.40	1.522	128.70	84.54	243.67	11.68	-126.66	.00	46.54	34.91	3.08	.00	.00
18.000	41.000	20.35	1.601	137.11	85.65	258.02	12.61	-133.52	.00	46.65	36.41	2.59	.00	.00
20.000	41.000	19.10	1.728	106.17	61.45	202.79	7.78	-104.39	.00	23.52	35.70	2.24	.00	.00
8.000	43.000	21.95	1.773	94.84	53.49	174.01	5.40	-84.58	.00	31.30	20.01	2.18	.00	.00
10.000	43.000	21.95	1.621	101.05	62.34	187.37	6.29	-92.61	.00	35.07	23.54	3.73	.00	.00
12.000	43.000	21.95	1.556	108.60	69.78	202.74	7.29	-101.43	.00	38.71	27.32	3.75	.00	.00
14.000	43.000	21.95	1.540	117.34	76.21	219.78	8.28	-110.72	.00	41.73	31.18	3.30	.00	.00
16.000	43.000	21.95	1.563	126.06	80.67	236.54	9.03	-119.51	.00	43.26	34.57	2.84	.00	.00
18.000	43.000	21.90	1.636	122.07	74.61	239.30	8.84	-126.08	.00	35.46	36.77	2.39	.00	.00
20.000	43.000	20.70	1.785	104.87	58.75	196.97	5.84	-97.95	.00	21.70	35.01	2.03	.00	.00
8.000	45.000	23.70	1.787	93.36	52.26	171.40	4.35	-82.40	.00	28.99	20.19	3.08	.00	.00
10.000	45.000	23.70	1.656	99.57	60.13	184.88	5.11	-90.42	.00	32.48	23.77	3.87	.00	.00

CX	CY	R	SF	PV	PH	NF	SNF	UF	CL	T	ST	FOR	NEF	TE
12.000	45.000	23.70	1.601	107.17	66.93	200.40	5.98	-99.21	.00	35.85	27.63	3.45	.00	.00
14.000	45.000	23.70	1.582	115.95	73.28	217.61	6.86	-108.52	.00	38.65	31.60	3.03	.00	.00
16.000	45.000	24.20	1.572	138.80	88.28	261.39	9.44	-132.03	.00	47.68	38.00	2.59	.00	.00
18.000	45.000	23.00	1.696	106.98	63.07	205.89	5.31	-104.23	.00	26.98	33.82	2.28	.00	.00
20.000	45.000	23.00	1.852	117.08	63.20	221.10	7.35	-111.37	.00	25.53	35.82	1.85	.00	.00
8.000	47.000	25.90	1.776	104.04	58.58	190.71	4.42	-91.09	.00	32.66	21.97	3.94	.00	.00
10.000	47.000	25.90	1.678	110.99	66.15	205.24	5.16	-99.41	.00	36.85	25.74	3.56	.00	.00
12.000	47.000	25.90	1.616	119.37	73.88	221.84	6.01	-108.48	.00	40.93	29.78	3.17	.00	.00
14.000	47.000	25.90	1.592	128.24	80.56	239.61	6.95	-118.32	.00	43.67	34.10	2.79	.00	.00
16.000	47.000	26.35	1.603	151.31	94.40	283.20	9.81	-141.70	.00	52.39	39.63	2.38	.00	.00
18.000	47.000	25.30	1.698	118.15	69.57	230.69	5.66	-118.20	.00	30.69	36.79	2.08	.00	.00
20.000	47.000	24.70	1.890	115.62	61.17	216.45	5.86	-106.68	.00	23.78	35.69	1.71	.00	.00

SAISYOU ANZENRITSU

CX	CY	R	SF	PV	PH	NF	SNF	UF	CL	T	ST	FOR	NEF	TE
16.000	37.000	15.95	1.486	103.21	69.46	194.39	10.94	-102.11	.00	37.87	27.69	3.91	.00	.00

1.2.4 STABILITY ANALYSIS OF RIPARIAN STRUCTURE

Stability Analysis On Leaning Wall

The leaning walls constructed in the Floodway are designed through the stability analysis such as sliding, over turning and bearing capacity. The stability calculation is made under normal and earthquake conditions.

(1) Dimensions of Leaning Wall

The cross section of leaning wall with a maximum height is selected for the stability analysis.

Location :	upper side leaning wall
Height :	7.10 m (top surface of footing to wall crown)
Length of Footing (B)	2.85 m (middle third : B/6=0.475)
Thickness of Footing	1.10 m
Slope gradient	1 : 0.5 (front surface of wall)

(2) Over Turning and Sliding

The study was made on Calculation Sheet-1.2.7. The results are as follows:

[Over turning]

Normal time :	e = 0.194 m < B/6 (middle third) ------ OK
Earthquake time:	e = -0.159 m < B/6 (middle third) ------ OK

[Sliding]

Normal time :	FS = 1.652 > 1.50 ------ OK
Earthquake time:	FS = 1.376 > 1.20 ------ OK

The leaning wall is stable against over turning and sliding.

(3) Ground Reaction and Bearing Capacity of Soil

Based on the soil survey results, the ultimate soil bearing capacity of the foundation ground is calculated as follows:

[Calculation Formula]

$$Q_u = A' \cdot \{\alpha \cdot \kappa \cdot c \cdot Nc + \kappa \cdot q \cdot Nq + 1/2 \cdot \gamma_i \cdot \beta \cdot B' \cdot Nr\}$$

[Calculation Conditions]

- Height of wall = 7.10 m
- Angle of internal friction = 30 degree (ϕ=30 to 40, used minimum angle)
- Cohesion = 1.0 t/m2 (Layers SW1 and SP6 are diluvial deposits. Therefore, cohesive effect of soil can be expected.)

- Eccentric distance caused by loading (e_b) = 0.194 m (Normal condition)

= -0.159 m (Earthquake condition)

- Embedment length of footing (D_f) = 1.1m
- Assuming that ground water level is well lowered by the drain pipe
- Earthquake load Horizontal earthquake factor (k_h) = 0.15, The earth pressure under earthquake time shall be the same as that of normal time.

[Calculation]

Case : Normal condition

$\alpha = 1.0,\ \beta = 1.0$

$\kappa = 1 + 0.3 \times 1.10 / (2.85-2 \times 0.194) = 1.134$

$\tan\theta = H_B/v = 8.63 / 24.69 = 0.349$

$q = r_2 \times D_f = 1.0 \times 1.1 = 1.1$ tf/m2

$A' = B' \times 1.0 = (2.85-2 \times 0.194) \times 1.0 = 2.46$ m2

using figures above Nc, Nq and Nr are given as follows:

Nc = 15, Nq = 8, Nr = 3 , Then Qu is obtained as follows:

$Qu = 2.46 \times \{1.0 \times 1.134 \times 1.0 \times 15 + 1.134 \times 1.1 \times 8 + 1/2 \times 1.0 \times 1.0 \times 2.46 \times 3\}$

$= 75.5$ tf

$Qa = Qu/3 = 75.5 / 3 = 25.1$ tf

Reaction of foundation caused by vertical loads

$= 1/2 \times (5.12+12.21) \times 2.85 = 24.7$t/m2 < Qa --- OK

Case : Earthquake condition

$\alpha = 1.0,\ \beta = 1.0$

$\kappa = 1 + 0.3 \times 1.10 / \{2.85-2 \times (-0.159)\} = 1.104$

$\tan\theta = H_B/v = 11.61 / 24.61 = 0.47$

$q = r_2 \times D_f = 1.0 \times 1.1 = 1.1$ tf/m2

$A' = B' \times 1.0 = (2.85-2 \times 0.159) \times 1.0 = 2.53$ m2

using figures above Nc, Nq and Nr are given as follows:

Nc = 11, Nq = 5, Nr = 1.5 , Then Qu is obtained as follows:

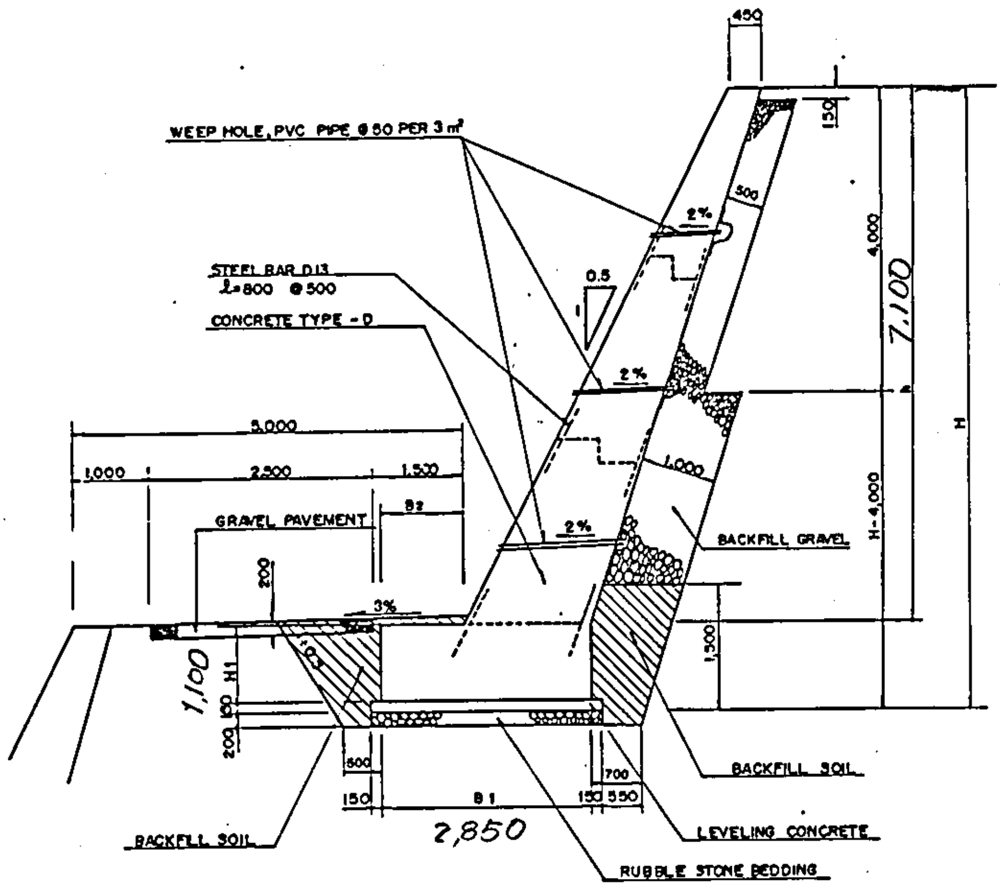
$Qu = 2.53 \times \{ 1.0 \times 1.104 \times 1.0 \times 11 + 1.104 \times 1.1 \times 5 + 1/2 \times 1.0 \times 1.0 \times 2.53 \times 1.5 \}$

$= 50.89$ tf

$Qa = Qu / 2 = 50.89 / 2 = 25.44$ tf

Reaction of foundation caused by vertical loads

$= 1/2 \times (5.75+11.53) \times 2.85 = 24.62$ t/m2 < Qa --- OK



CROSS SECTION OF UPPER WALL

SCALE 1:50

Calculation Sheet-1. 2. 7(1/2) :Normal Time

Structural Design of Leaning Wall

Case : Normal condition

Design Condition

Backfill Material					
Unit Weight	Gs	2.000 t/m3			
Internal Friction Angle	fs	35.000			
Coefficient of Earth Pressure	Kh	0.171			
	Kv	0.025			
Water Height Back Side	Hwb	0.000 m			
Front Side	Hwf	0.000			
Load on Backfill	q	1.000 t/m2			
Features of Wall					
Height of Wall	H	6.000			
Slope Front	nf	0.500			
Back	nb	-0.270			
Top Width	B	0.450 m	B/6-e	0.281	
Toe Length	Lt	1.020 m	Fs	1.652	
Toe Height	Ht	1.100 m	Qumax	12.206	
Foundation					
Internal Friction Angle	ff	30.000			
Cohesion	Cf	0.000 t/m2			
Concrete					
Unit Weight	Gc	2.350 t/m3			
Design Strength	Sck	160.000 kg/cm2			
Coefficient of Earthquake	k	0.000			
Factor of Safety		1.500			
Bearing Strength	qa	23.600			

Load			Vertical	Horizontal	Arm	Moment	
Self Weight	Wall	W1	6.345		4.245	26.935	
		W2	21.150		3.020	63.873	
		W3	-11.421		3.930	-44.885	23.441
	Footing	W3	7.367		1.425	10.498	56.421
Water Pressure	Horizontal	Pw	0.000		-0.367	0.000	
		Pw1"	0.000		0.000	0.000	
		Pwf	0.000		0.000	0.000	
		Pw2	0.000		0.733	0.000	
	Vertical	Ph2	0.000		2.949	0.000	
Water Weight		Ww1	0.000		0.510	0.000	
		Ww2	0.000		0.917	0.000	
Seismic Force	Wall	Ke1		0.000	4.100	0.000	
		Ke2		0.000	3.100	0.000	
		Ke3		0.000	3.100	0.000	
	Footing	Ke4		0.000	0.550	0.000	
Earth Pressure	Horizontal	Ph		-8.630	2.367	-20.425	
	Vertical	Pv	1.247		3.192	3.981	
Uplift		U1	0.000		1.425	0.000	
		U2		0.000	0.950	0.000	
Total			24.689	-8.630		39.977	

Stability Calculation 1.619

Middle third	**B=**	2.850 m	
	B/6 =	**0.475 m**	
	e =	**0.194 m**	**B/6>e OK**
	(B/6-e	0.281)	
Sliding	**FS=**	**1.652 > 1.50**	**OK**
Bearing strength	**U1=**	**5.120 t/m2**	**OK**
	U2=	**12.206 t/m2**	**OK**

Basic Dimensions

Wall Height	H	7.100 m
Width of Wall Bottom	Bb	1.830 m
Width of Footing	Bf	2.850 m
Back Slope Angle		-15.110

			Normal	Earthquake
Conditions				
Wall height	H	m	7.100	7.100
Wall slope at backside	m	1:	-0.270	-0.270
Height of backfill	dH	m	0.000	0.000
Slope of backfill	n	1:	##########	##########
Unit weight of soil	γs	t/m^3	2.000	2.000
Surcharge in normal condition	q	tf/m^2	1.000	0.000
Internal friction angle of soil	ϕ	°	35.000	35.000
Friction angle of soil to concrete (2/3f)	δ	°	23.333	23.333
Trial				
Slip angle	α	°	53.094	52.462
	Pa-Pa'		0.000	0.005
Total weight of soil wedge	W	tf/m	27.662	25.124
	Pa	tf/m	8.720	7.638
Soil Pressure				
Active earth pressure (Horizontal)	P_H	tf/m	8.630	7.559
Active earth pressure (Vertical)	P_V	tf/m	1.247	1.093
Coefficient of active earth pressure	K_H		0.171	0.150
Coefficient of active earth pressure	K_V		0.025	0.022

Total weight of soil wedge	W'	tf/m	27.977	25.404
Slip angle	α'	°	52.894	52.262
	P_{a-1}	tf/m	8.720	7.633
angle (wall)	j	°	-15.110	-15.110
angle (Backfill)		°	0.000	0.000
	L1+L2		3.415	3.539
			3.454	3.578
	L1		0.000	0.000
			0.000	0.000
	L2		3.415	3.539
			3.454	3.578
	L3		-1.917	-1.917
			-1.917	-1.917
	L4		5.332	5.456
			5.371	5.495
	L3+L4		3.415	3.539
			3.454	3.578

Structural Design of Leaning Wall

Case : Earthquake condition

Design Condition

Backfill Material						
Unit Weight		Gs	2.000	t/m3		
Internal Friction Angle		fs	35.000			
Coefficient of Earth Pressure		Kh	0.161			
		Kv	0.023			
Water Height	Back Side	Hwb	0.000	m		
	Front Side	Hwf	0.000			
Load on Backfill		q	0.500	t/m2		
Features of Wall						
Height of Wall		H	6.000			
Slope	Front	nf	0.500			
	Back	nb	-0.270			
Top Width		B	0.450	m	B/6-e	0.316
Toe Length		Lt	1.020	m	Fs	1.376
Toe Height		Ht	1.100	m	Qumax	11.526
Foundation						
Internal Friction Angle		ff	33.000			
Cohesion		Cf	0.000	t/m2		
Concrete						
Unit Weight		Gc	2.350	t/m3		
Design Strength		Sck	160.000	kg/cm2		
Coefficient of Earthquake		k	0.150			
Factor of Safety			1.200			
Bearing Strength		qa				

Load			Vertical	Horizontal	Arm	Moment
Self Weight	Wall	W1	6.345		4.245	26.935
		W2	21.150		3.020	63.873
		W3	-11.421		3.930	-44.885
	Footing	W3	7.367		1.425	10.498
Water Pressure	Horizontal	Pw	0.000		-0.367	0.000
		Pw1"	0.000		0.000	0.000
		Pwf	0.000		0.000	0.000
		Pw2	0.000		0.733	0.000
	Vertical	Ph2	0.000		2.949	0.000
Water Weight		Ww1	0.000		0.510	0.000
		Ww2	0.000		0.917	0.000
Seismic Force	Wall	Ke1		-0.952	4.100	-3.902
		Ke2		-3.173	3.100	-9.835
		Ke3		1.713	3.100	5.311
	Footing	Ke4		-1.105	0.550	-0.608
Earth Pressure	Horizontal	Ph		-8.097	2.367	-19.164
	Vertical	Pv	1.170		2.508	2.935
Uplift		U1	0.000		1.425	0.000
		U2		0.000	0.950	0.000
Total			24.612	-11.614		31.158

Stability Calculation

Middle third	**B=**	**2.850 m**		
	B/6 =	**0.475 m**		
	e =	**-0.159 m**		**B/6>e OK**
	(B/6-e	0.316)		
Sliding	**FS=**	**1.376**	**> 1.20**	**OK**
Bearing strength	**U1=**	**11.526 t/m2**		
	U2=	**5.745 t/m2**		

Basic Dimensions

Wall Height	H	7.100 m
Width of Wall Bottom	Bb	1.830 m
Width of Footing	Bf	2.850 m
Back Slope Angle		-15.110

			Normal	Earthquake
Conditions				
Wall height	H	m	7.100	7.100
Wall slope at backside	m	1:	-0.270	-0.270
Height of backfill	dH	m	0.000	0.000
Slope of backfill	n	1:	##########	##########
Unit weight of soil	γs	t/m^3	2.000	2.000
Surcharge in normal condition	q	tf/m^2	0.500	0.500
Internal friction angle of soil	φ	°	35.000	35.000
Friction angle of soil to concrete (2/3f)	δ	°	23.333	23.333
Trial				
Slip angle	α	°	53.145	53.145
	Pa-Pa'		-0.000	-0.000
Total weight of soil wedge	W	tf/m	25.879	25.879
	Pa	tf/m	8.182	8.182
Soil Pressure				
Active earth pressure (Horizontal)	P_H	tf/m	8.097	8.097
Active earth pressure (Vertical)	P_V	tf/m	1.170	1.170
Coefficient of active earth pressure	K_H		0.161	0.161
Coefficient of active earth pressure	K_V		0.023	0.023

	Total weight of soil wedge	W'	tf/m	26.174	26.174
	Slip angle	α'	°	52.945	52.945
		P_{a-1}	tf/m	8.182	8.182
	angle (wall)	j	°	-15.110	-15.110
	angle (Backfill)		°	0.000	0.000
		L1+L2		3.405	3.405
				3.444	3.444
		L1		0.000	0.000
				0.000	0.000
23.441		L2		3.405	3.405
56.421				3.444	3.444
		L3		-1.917	-1.917
				-1.917	-1.917
		L4		5.322	5.322
				5.361	5.361
		L3+L4		3.405	3.405
				3.444	3.444

2. DESIGN OF RIVER STRUCTURES

2.1 Groundsill (PE 55 + 00)

2.1.1 General Discription

Location and Purpose

After construction of the existing Bandar Sidoras Weir, the downstream riverbed of the weir has lowered because the supply of sediment has been stopped by the fixed weir. Therefore, a groundsill located at 100 m downstream from the Titi Beshi Bridge (PE55+00) is proposed to restore the riverbed elevation up to the design riverbed level and also to stabilize the substructures of Titi Beshi Bridge and revetments. This groundsill will be well functioned in harmony with the movable weir to be constructed which can supply a sediment during floods.

Structural Dimensions

Since the design riverbed elevation is about 1.0 m above the existing riverbed, the crest elevation of the groundsill is designed to conform to the design riverbed, resulting in a groundsill with a height of 1.0 m. This groundsill is of concrete gravity type with an apron to safeguard its own body from hydraulic force during flood time.

The structural features of the groundsill are driven from the next paragraph and summarized below and shown in the Fig. 2.1.1.

Work Item		Dimensions
Main body		
Crest Elevation		EL. -0.44 m
Elevation of Apron		EL. -1.44 m
Elevation of foundation		EL. -2.24 m
Height of Groundsill		1.00m
Crest Width		1.00 m
Downstream Slope		1:0.8
Length of Apron		4.50 m
Thickens of Apron		0.80 m
Foundation		
Waterstop (Steel sheet pile Type II)		2.0 m long at 2 locations
Foundation Treatment		PC Pile f=400, L=9m, nos.
Riverbed Protection		
Gabion Mattress	Upstream	L = 6.0 m
	Downstream	L = 12.0 m

2.1.2 Design of Groundsill

(1) Stability Analysis

(a) Design Factors

Design discharge	10.6 m^3/s (25% discharge)	
Earthquake load	k=0.11 (Allvium, 1/100 year)	
Foundation	Fine Sand (Co=18)	
	Internal Friction Angle	f = 0.5
	Cohesion	C = 1.0
	Bearing Capacity	qa = 6 tf/m^2

Upstream Water Level

$$Q = 2/15\, C\, (2\, g)^{1/2} (3\, B_1 + 2\, B_2)\, h^{3/2}$$

where, Q : design discharge (Q = 10.6 m^3/s)
C : coefficient (C = 0.6)
g : gravity accerration (g = 9.8 m/sec^2)
B_1: crest width (B1 = 13 m)
B_2: width of overflow water surface (13m + 4 h)
h : overflow depth (m)

h = 0.57 m (EL. -0.44 + 0.57 =EL. 0.13 m)

Downstream water level
MLWL ; EL. -0.930m

(b) Loading Combination and Safety Factor

The stability analysis of weir are conducted under the following load combinations and safety conditions.

	Loading Combination	Increase in Allowable Stress	Safety Factor	
			Sliding	Bearing Capacity
Flood	$M+T+Th_b+U$	0 %	1.5	3
Earthquake	$M+T+Th_n+G+U$	20 %	1.2	2
Self weight	M	0 %	-	3

where,
M : dead load
T : sediment pressure
Thn : normal water pressure
Thb : water pressure during flood
G : earthquake load
U : uplift

(c) Safety of Weir

Sliding

The factor of safety against sliding is determined using the following formula:

$SF \quad = \quad Rc / \Sigma H$

where, H : total horizontal forces
Rc : resisting capacity
SF : safety factor given in the following
$SF \geq 1.5$ (under normal condition)
$SF \geq 1.2$ (under floods and earthquake condition)

Overturning

All forces acting on part of the structure above any horizontal plane should fall within the middle third of the structure base.

For this purpose, the following condition should be satisfied :

$$e = \frac{b}{2} - \frac{M}{N} < \frac{b}{6}$$

where,
b : width of base (m)
M : total moment about point A (tf . m)
N : total vertical forces (tf)
e : eccentricity (m)

Bearing Capacity of Pile Foundation

The maximum principal stress in the spread foundation must be kept within allowable soil bearing capacity which is derived from the following :

Qa = Qu/SF

where,

Qa : allowable soil bearing capacity
Qu : ultimate soil bearing capacity
$Qu_1 = 55$ tf/m^2 for long term load,
$Qu_2 = 73$ tf/m^2 for short term load.
$SF \geq 3$ (under the normal condition)
$SF \geq 2$ (under the earthquake condition)

The allowable soil bearing capacity for the diversion weirs are;

$Qa_1 = 6$ tf/m^2 / 3 = 2 tf/m^2 for nomal condition,
$Qa_2 = 6$ tf/m^2 / 2 = 3 tf/m^2 for earthquake condition.

(d) Results of Calculation

The results of stability analysis is presented in Table 2.1.1 and summarized in the table below:

	SF	e-B/6	qmax
Normal Condition	1.799	0.226	1.706
Earthquake Condition	1.250	0.019	3.009
Maximum	3.460	0.278	3.923

As shown above table, the vertical load on the groundsill foundation is bigger than allowable soil bearing capacity (Qa). So as to increase the bearing capacity, the pile foundation will be applied.

(2) Design of Pile Foundation

The N value in the foundation of the groundsill is summarized as bellow;

from ground surface to EL. -6 m to -8 m	$N \leq 5$
from EL. -6 m to-10 m	N = about 20.
deeper than EL. -10 m	$N \geq 20$

As for the foundation of the ground sill, PC piles f 300 mm with 9 m (EL.-2 m-EL.-10 m + 1 m allowance) long are to be driven to increase the bearing capacity of the subsurface layer to support the weir body.

The bearing capacity (Ra) of pile (PC ϕ300) is estimated as shown below:

Upper layer (clay, $L_1 = 4.0$ m, C=2.0 t/m$_2$)

R_{u1} $= D\pi \times L_1 \times C$
$= 0.3$ p x 4.0 m x 2.0 t/m$_2$
$= 7.54$ tf/m^2

Lower layer (sand, L2 = 5 m, N = 20)

R_{u2} $= D\pi \times L_2 \times 0.2 \times C$
$= 0.3$ p x 5.0 m x 0.2 x 20 t/m$_2$
$= 18.85$ tf/m^2

R_u $= R_{u1} + R_{u2}$
$= 7.54 + 18.85 = 26.39$ t/m$_2$

R_a $= \gamma/n \times R_u = 1.0/4 \times 26.39 = 6.6$ tf/nos.

Since the self weight of groundsill including apron is 220 t, the required number of piles is estimated at;

220.054 t /6.6 t/nos. = 33.3 nos.

The arrangement of piles are shown in Fig. 2.1.1 and the total number of pile is 36.

(3) Length of apron and channelbed protection

To protect the channel bed erosion, the concrete lined apron will be provided at the downstream. The length of apron (La) is set at same length of hydraulic jump:

$L > 4.5 \times Hj$ or $6(Hj - h1)$

$Hj = 0.84$ m (see Table 2.1.2)

$L = 4.5Hj = 3.78$ m or
$6(Hj - h1) = 4.10$ m

≤ 4.5m

The length of channelbed protection works of downstream channel is generally set at 3 times of apron length and that of upstream channel is set at half of that of downstream. Therefore, the length of channelbed protection works are:

Downstream : $4.5 \times 3 = 14.5$ m ≤ 15 m
Upstream : $15/2 = 7.5$ m ≤ 8 m

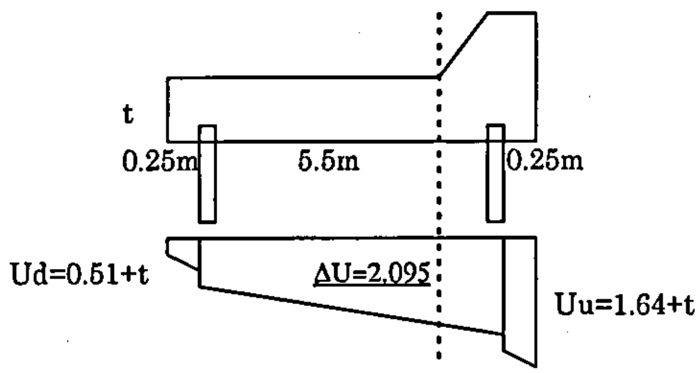
(4) Thickness of Apron

To withstand the scouring force and uplift pressure, the thickness of apron is estimated by the following formula.

$t = 4/3\ (\Delta h - \Delta u)/(Wc-1)$

where, Δh : max. difference between water heads (m), $\Delta H = 1.13$ m
Δu : uplift at the toe of main body ($\Delta u = 2.095\ t/m^2$)
Wc : unit weight of concrete ($Wc = 2.35\ t/m^3$)

$t = 4/3\ (2.095-1.31)/(2.35-1) = 0.775 \leq 0.8$ m



(5) Creep Length

For the prevention of piping at the foundation of groundsill, cut off wall, steel sheet piles are normally employed to the cut off wall, will be provided to increase the vertical creep length.

The required length of sheet piles are calculated by Lane's method as shown bellow.

$C \leq (Lh/3 + \Sigma Lv)/\Delta H$

where, C : Lane's creep ratio, C = 8.5 for silty sand/clay

Lh : creep length of horizontal direction (m), Lh = 6 m

Lv : creep length of vertical direction (m)

ΔH : max. difference between water heads (m), ΔH= 1.13 m

$\Sigma Lv \geq C\ \Delta H - Lh/3 = 8.5 \times 1.13 - 6/3) = 7.605$ m

To obtain a 8 m vertical creep length, 2 m long steel sheet piles shall be driven at the downstream edge of apron and upstream edge of main body.

Gabion mattress for riverbed protection are placed on upstream and downstream riverbeds of the groundsill with appropriate length.

Table 2.1.1 (1/3) STABILITY ANALYSIS (NORMAL CONDITION)

Stability Analysis

Groundsill (PE55+00)
Nomal Condition

Basic Condition				Stability Condition		
Water Level	Upstream	EL.	5.260 m	U.W.. of Concrete		2.350 t/m3
	Downstream	EL.	4.260 m	Coefficient of Earth quake		0.000
EL. of Crest		EL.	-0.440 m	U.W. of Sediment		1.800
EL. of Foundation		EL.	-2.240 m	Coefficient of Sediment P.		0.400
Crest Width		B1	1.000 m	Foundation	C	1.000
Slope	Downstream	1:	0.800		tan φ	0.600
U.P. Slope	Upstream	1:	0.000		Bearing strength	2.000 t/m2
Orifice	Height	Ho	0.000 m	Required Factor of Safety		1.500
	Width	Wo	0.000 m	Uplift C.	Sheetpile L	m
	Length	Lo	0.000 m		Up1	
	Orifice Center	EL.	0.000 m		Up2	
Block Width		Wb	1.000 m		Up2'	
Weir Height		Hw	1.800 m		Up3	
EL. of Sedimentation			-0.440			

Load			Vertical	Horizontal	Arm	Moment	
Self Weight	Center	W1	4.230		0.500	2.115	
	Downstream	W2	3.046		1.480	4.507	
	Upstream	W3	0.000		0.000	0.000	
	Orifice	Wo	0.000		0.000	0.000	
Hydraulic Pressure	Hrizontal US1	P1		10.260	0.900	9.234	
	Hrizontal US2	P2		1.620	0.600	0.972	
	Hrizontal DS1	P3		-1.620	0.600	-0.972	
	Hrizontal DS2	P4		-8.460	0.900	-7.614	
Water Weight	Crest	P3	5.200		0.333	1.733	5.700 4.700
	Down Stream	P4	6.768		1.720	11.641	
	Down Stream1	P6	0.720		1.960	1.411	
Seismic Force	Center	Ke1		0.000	0.900	0.000	
	Downstream	Ke2		0.000	0.600	0.000	
	Upstream	Ke3		0.000	0.600	0.000	
	Orifice	Keo		0.000	2.240	0.000	
Sediment Pressure		Ps		0.518	0.600	0.311	
Uplift		U1	-15.860		1.220	-19.349	17.080
		U2	-1.220		0.813	-0.992	
Total			2.884	2.318		2.998	

Stability Calculation -17.080 -20.341

Middle third	**B=**	**2.440 m**	
	B/6 =	**0.407 m**	
	e =	**-0.180 m**	**B/6>e OK**
	(B/6-e	**0.226)**	
Sliding	**FS=**	**1.799**	**OK**
Bearing strength	**U1=**	**1.706 t/m2**	**OK**
	U2=	**0.657 t/m2**	**OK**

Table 2.1.1 (2/3) STABILITY ANALYSIS (EARTHQUAKE CONDITION)

Stability Analysis

Groundsill (PE55+00)
Earthquake Condition

Basic Condition				Stability Condition		
Water Level	Upstream	EL.	0.130 m	U.W.. of Concrete		2.350 t/m3
	Downstream	EL.	-0.930 m	Coefficient of Earth quake		0.110
EL. of Crest		EL.	-0.440 m	U.W. of Sediment		1.800
EL. of Foundation		EL.	-2.240 m	Coefficient of Sediment P.		0.400
Crest Width		B1	1.000 m	Foundation	C	1.000
Slope	Downstream	1:	0.800		tan ϕ	0.600
U.P. Slope	Upstream	1:	0.000		Bearing strength	3.000 t/m2
Orifice	Height	Ho	0.000 m	Required Factor of Safety		1.200
	Width	Wo	0.000 m	Uplift C.	Sheetpile L	m
	Length	Lo	0.000 m		Up1	
	Orifice Center	EL.	0.000 m		Up2	
Block Width		Wb	1.000 m		Up2'	
Weir Height		Hw	1.800 m		Up3	
EL. of Sedimentation			-0.440			

Load			Vertical	Horizontal	Arm	Moment
Self Weight	Center	W1	4.230		0.500	2.115
	Downstream	W2	3.046		1.480	4.507
	Upstream	W3	0.000		0.000	0.000
	Orifice	Wo	0.000		0.000	0.000
Hydraulic Pressure	Hrizontal US1	P1		1.026	0.900	0.923
	Hrizontal US2	P2		1.620	0.600	0.972
	Hrizontal DS	P5		-0.858	0.437	-0.375
	Orifice	Po		0.000	2.240	0.000
Water Weight	Crest	P3	0.285		0.333	0.095
	Down Stream	P4	0.686		2.091	1.435
	Upstream	P6	0.000		0.000	0.000
Seismic Force	Center	Ke1		0.465	0.900	0.419
	Downstream	Ke2		0.335	0.600	0.201
	Upstream	Ke3		0.000	0.600	0.000
	Orifice	Keo		0.000	2.240	0.000
Sediment Pressure		Ps		1.166	0.600	0.700
Uplift		U1	-3.196		1.220	-3.900
		U2	-1.293		0.813	-1.052
Total			3.757	3.755		6.042

Stability Calculation

Middle third	**B=**	**2.440 m**	
	B/6 =	**0.407 m**	
	e =	**0.388 m**	**B/6>e OK**
	(B/6-e	**0.019)**	
Sliding	**FS=**	**1.250**	**OK**
Bearing strength	**U1=**	**0.071 t/m2**	**OK**
	U2=	**3.009 t/m2**	**NG**

Table 2.1.1 (3/3) STABILITY ANALYSIS (EXCEPT UPLIFT)

Stability Analysis

Groundsill (PE55+00)
Without Water Pressure and Uplift (Check of Bearing Capacity)

Basic Condition				Stability Condition		
Water Level	Upstream	EL.	-2.240 m	U.W.. of Concrete		2.350 t/m3
	Downstream	EL.	-2.240 m	Coefficient of Earth quake		0.110
EL. of Crest		EL.	-0.440 m	U.W. of Sediment		1.800
EL. of Foundation		EL.	-2.240 m	Coefficient of Sediment P.		0.400
Crest Width		B1	1.000 m	Foundation	C	1.000
Slope	Downstream	1:	0.800		tan ϕ	0.600
U.P. Slope	Upstream	1:	0.000		Bearing strength	2.000 t/m2
Orifice	Height	Ho	0.000 m	Required Factor of Safety		1.200
	Width	Wo	0.000 m	Uplift C.	Sheetpile L	m
	Length	Lo	0.000 m		Up1	
	Orifice Center	EL.	0.000 m		Up2	
Block Width		Wb	1.000 m		Up2'	
Weir Height		Hw	1.800 m		Up3	
EL. of Sedimentation			-0.440			

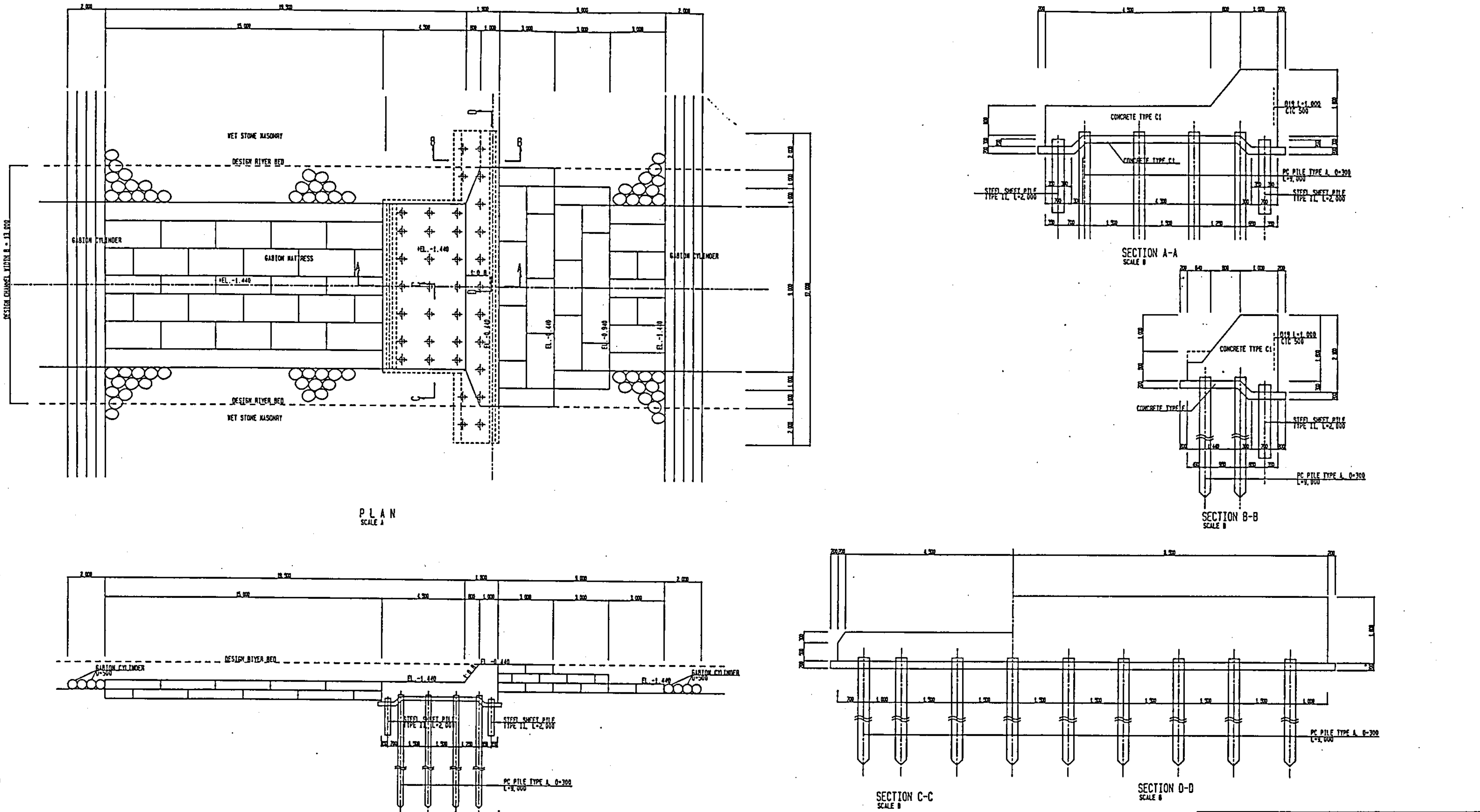
Load			Vertical	Horizontal	Arm	Moment
Self Weight	Center	W1	4.230		0.500	2.115
	Downstream	W2	3.046		1.480	4.507
	Upstream	W3	0.000		0.000	0.000
	Orifice	Wo	0.000		0.000	0.000
Hydraulic Pressure	Hrizontal US1	P1		0.000	0.900	0.000
	Hrizontal US2	P2		0.000	0.000	0.000
	Hrizontal DS	P5		0.000	0.000	0.000
	Orifice	Po		0.000	2.240	0.000
Water Weight	Crest	P3	0.000		0.333	0.000
	Down Stream	P4	0.000		2.440	0.000
	Upstream	P6	0.000		0.000	0.000
Seismic Force	Center	Ke1		0.465	0.900	0.419
	Downstream	Ke2		0.335	0.600	0.201
	Upstream	Ke3		0.000	0.600	0.000
	Orifice	Keo		0.000	2.240	0.000
Sediment Pressure		Ps		1.166	0.600	0.700
Uplift		U1	0.000		1.220	0.000
		U2	0.000		0.813	0.000
Total			7.276	1.967		7.942

Stability Calculation

Middle third	B=	2.440 m	
	B/6 =	0.407 m	
	e =	-0.128 m	B/6>e OK
	(B/6-e	0.278)	
Sliding	FS=	3.460	OK
Bearing strength	U1=	3.923 t/m2	NG
	U2=	2.040 t/m2	NG

Table 2.1.2 CALCULATION OF HYDRAULIC JUMP

		Unit	Groundsil Weir	Groundsil Weir	Note
Discharge	Q	m3/s	55.00	10.60	
Width	B	m	13.00	13.00	
U.S. WL		EL. m	1.47	0.20	
Crest EL.			-0.44	-0.44	
Apron EL.			-1.44	-1.44	
Unit Q	q	m3/s/m	4.23	0.82	
	Z	m	1.96	1.32	
	h1	m	0.68	0.16	
	Z-h1	m	1.27	1.16	
	V1	m/s	6.19	5.08	
	Fr		2.39	4.06	
	hj	m	2.00	0.84	0.56
6x(h2-h1)	L	m	7.87	4.10	
4.5xhj		m	8.98	3.79	



SCALE B (1:50)
SCALES SCALE A (1:100)

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MINISTRY OF PUBLIC WORKS
DIRECTORATE GENERAL OF WATER RESOURCES DEVELOPMENT
DIRECTORATE OF TECHNICAL GUIDANCE

MEDAN FLOOD CONTROL PROJECT

GROUNDSILL
GROUNDSILL FOR PERCUT RIVER (PE55+00)
STRUCTURAL DETAILS

NO.	DATE	REVISIONS	ORIGINATED	APPROVED

PREPARED	CHECKED	SUBMITTED	CERTIFIED	APPROVED	DRAWING NO.
					RS - 001
JAPAN INTERNATIONAL COOPERATION AGENCY CTI ENGINEERING CO., LTD			PROYEK PENGELOLAAN SUMBER AIR DAN PENGENDALIAN BANJIL SUMATERA UTARA		SHEET NO. 211 / REV.

Fig. 2.1.1 GENRAL DRAWING OF GROUNFDSILL

2.2 Drainage Outlets

2.2.1 Confluence Treatment of Batuan River

Batuan River which is a small tributary of Deli River crosses the proposed Floodway at FW 25+24. The Catchment area of Batuan River, which is included in the Deli river basin, is small. Therefore, the runoff discharge is faster than that of the Upper Deli River, resulting in no additional discharge to the Floodway.

The improvement of Batuan River is proposed from the intake weir located at 85 m upstream from the confluence to the Floodway with the design discharge of 16 m^3/s corresponding to a 5-year return period flood..

Since the riverbed elevation of Batuan River is about 5.1 m higher than the channel bed of Floodway, a step-wise fall structure with double box culvert (B x H = 2.0 m x 2.0 m) is provided at the confluence to dissipate the energy of river flow.

The structural detail is shown in Fig. 2.2.1.

<u>Hydraulic Design</u>

The basic dimensions of the upstream channel and sluice is obtained by the Manning's formula:

$$Q = A \, 1/n \, I^{1/2} R^{2/3}$$

where, Q : design discharge (Q=16 m^3/s)
A : flow area (m^2)
n : roughness coefficient (n = 0.025 for wet masonry
n = 0.023 for concrete lining)
I : slope
R : hydraulic radius (m)

(a) Upstream Channel

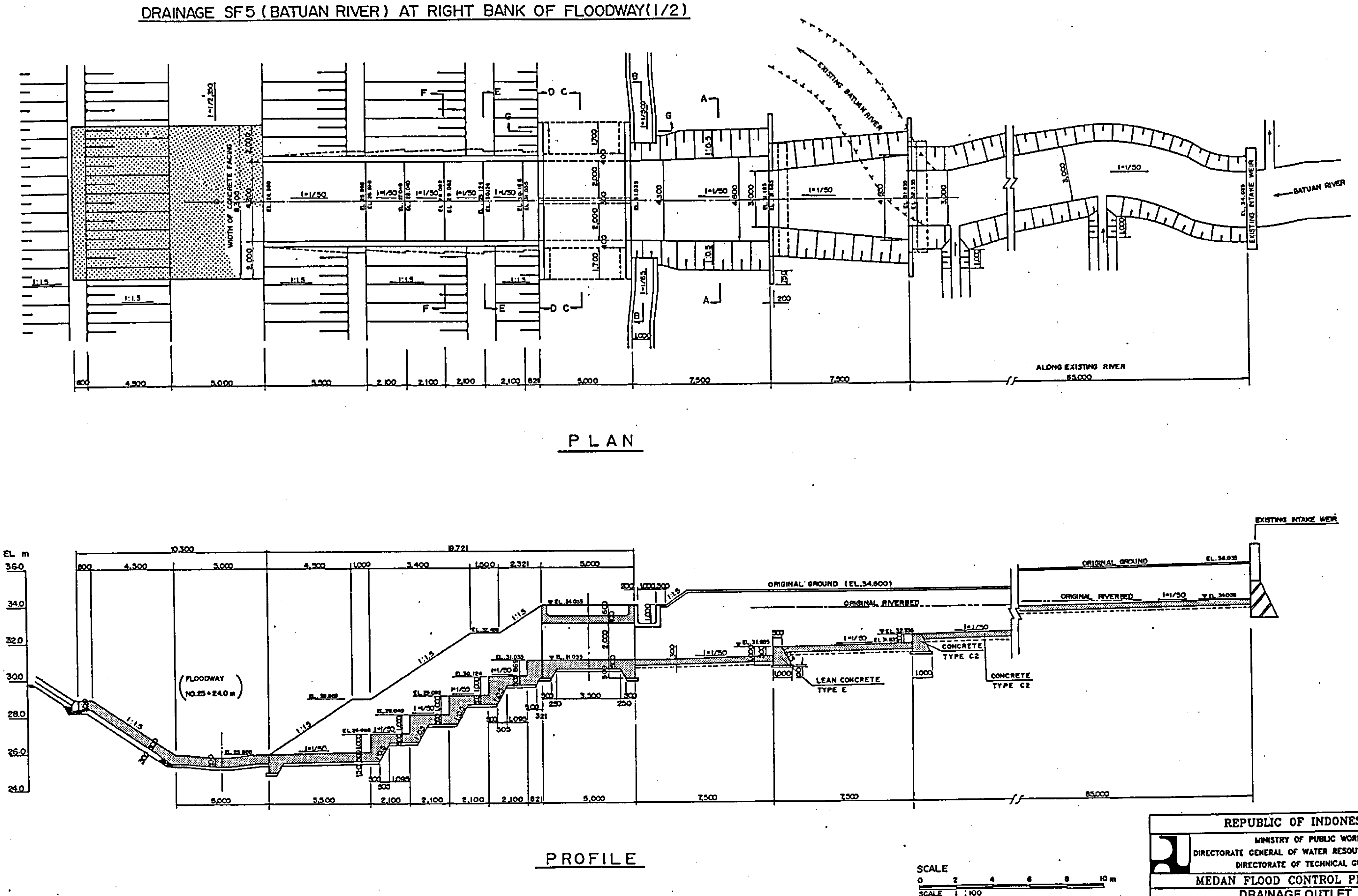
The feature of channel is:

bottom width	B = 3 m,
water depth	h = 1.1 m,
hydraulic radius	R = 0.72 m
flow area	A = 3.9 m^2/unit

The slope of channel is set at I = 1/50 to make the flow velocity less than 5 m/s and 50 cm high dorop structures with 7.5 m interval are provided to connect the upstream end of the channel to the existing intake weir.

The flow capacity of the channel is calculated as shown bellow:

$$Q = 1/0.023\ (1/50)^{1/2}\ 0.524^{2/3} = 17.7\ m^3/s > 16\ m^3/s, \text{OK}$$



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MINISTRY OF PUBLIC WORKS
DIRECTORATE GENERAL OF WATER RESOURCES DEVELOPMENT
DIRECTORATE OF TECHNICAL GUIDANCE

MEDAN FLOOD CONTROL PROJECT

DRAINAGE OUTLET
SF5 (BATUAN RIVER)
PLAN AND PROFILE

PREPARED	CHECKED	SUBMITTED	CERTIFIED	APPROVED	DRAWING NO.
					DO - 042

		SHEET NO.	REV.
JAPAN INTERNATIONAL COOPERATION AGENCY CTI ENGINEERING CO., LTD	PROYEK PENGELOLAAN SUMBER AIR DAN PENGENDALIAN BANJIR SUMATERA UTARA	266	

NO.	DATE	REVISIONS	ORIGINATED	APPROVED

Fig. 2.2.1 CONFLUENCE TREATMENT OF BATUAN RIVER

(b) Sluice

Since the water level at the inlet of the sluice is 1.1 m, the width of sluice is calculated as bellow:

The feature of channel is:

bottom width	B = 2 m x 2 units,
water depth	h = 1.1 m,
hydraulic radius	R = 0.524 m
flow area	A = 2.2 m^2/unit

The flow capacity of the channel is:

$Q_1 = 2.2 \times 1/0.023 \times (1/50)^{1/2} \times 0.546^{2/3} = 8.8\ m^3/s/unit$
$Q = 8.8 \times 2\ unit = 17.6 > 16\ m^3/s$, OK

The water level inside of the box culvert is assumed as same height as height of hydraulic jump.

water depth at inlet	: $h_1 = 1.1$ m
flow velocity at inlet	: $v_1 = 3.995$ m/s
Froude number	: $F_1^2 = v_1^2/gh_1 = 1.481$
height of hydraulic jump	: $hj = ((1+8\ F_1^2)^{1/2}-1)\ h_1/2 = 1.42 = \underline{1.5\ m}$

The height of box culvert is set at 2m high including 50 cm freeboard.

(3) Step-wise Fall Structure

Since there is a 5.5 m water head from the outlet of box culvert to the floodway channel bed and there is no water in the floodway during small floods, it is necessary to dissipate the enargy of the flow. For this perpose, a falling structure is provided at the confluence.

The height and length of each step are determined under the following conditions:

- Average slope is set at about 1:2.0 considering the average bank slope.
- Water vein from the upper step should be fallen on the lower step at least 2/3 of the step length or near.
- Height of step shall be minimized to avoide the vibration by the water drop.

The water depth at the edge of step (h_3) is:

$h_3 = (Q/CB)^{2/3} = (16.9/(1.77 \times 4.3))^{2/3} = 1.7$ m.

The initial velocity of water vain is:

$Vo = Q/(Bh_3) = 16.9/(4.3 \times 1.7) = 2.31$ m/s.

The distance from the upper step to the drop point of water vein (L_1) is:

$L_1 = Vo(2\ (\Delta H+h_3/2)/g)^{1/2} = 2.31\ (2\ (1.0 + 1.7/2)/9.8)1/2 = 1.42$ m.

where ΔH : step height ($\Delta H = 1.0$ m)

The length of step (L)is:

$L = 3/2\ L1 = 3/2 \times 1.42 = 2.1$ m

(4) Bank Protection at Water Collidding Slope at Floodway Channel

To protect the slope erosion at the bottom of the falling works and left bank slope of the floodway, a concrete faced bank protection will be provided.

The area to be faced by concrete is determined by the width of water vain at the left bank slope of floodway in front of the falling works outlet. The width of water vain at the left bank slope is assumed as fillows:

angle of spreading	$\theta = 11^\circ$
width of water vain	$w = 2 \times \tan\theta \times 5$ m + 4.3 m =6.24 m

The width of bank protection is set at 8.3 m including the 1 m allowance at the both sides.

2.2.2 Drainage Outlet

River improvement works such as channel excavation, widening and diking usually affect the drainage system around the river channel and it is anticipated that the excavation and channel widening will slightly affect the outlet structures, while diking will change the drainage condition in the adjacent area. Besides, the capacity of the existing drainage outlet is not adequate for the standard of drainage improvement as carried out for Deli River and its tributaries under MUDP II.

Therefore, the drainage improvement will focus on the outlet portion, i.e., the sluice connecting to Percut River and Medan Floodway. Since the inspection roads will be constructed in succession along the riverbanks, the sluice is required to be modified to either a box culvert or a pipe culvert embedded in the riverbank.

(1) Existing Condition of Drainage Outlet

Many closed and open drainage channels discharge storm runoff and domestic wastewater into Percut River. Further, some tributaries and drainage channels will be cut by the proposed Floodway. Based on the field survey and hydrographic study with a 1/5,000 topo-map, the existing drainage areas and networks were identified, as shown in Fig. 2.2.2. There are 44 drainage areas in total; 12 drainage channels are found to the right side of Percut River and 25 channels are to the left side; and 7 channels shall bring the storm runoff to the Floodway.

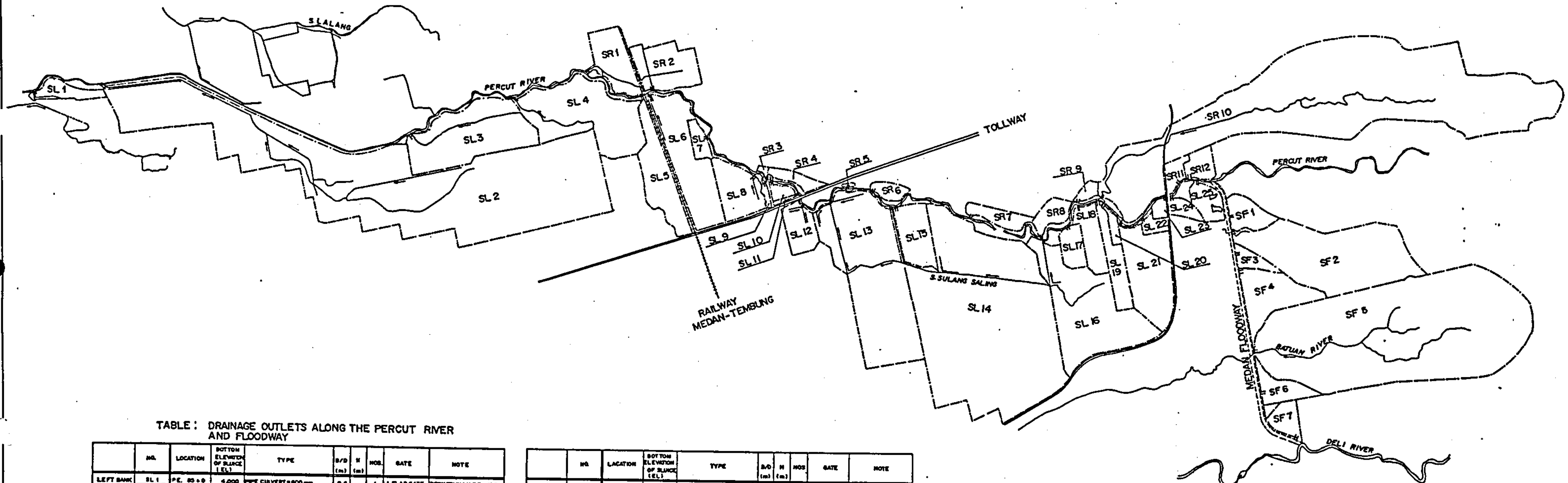

TABLE : DRAINAGE OUTLETS ALONG THE PERCUT RIVER AND FLOODWAY

	NO.	LOCATION	BOTTOM ELEVATION OF SLUICE (EL)	TYPE	B/D (m)	H (m)	NOS.	GATE	NOTE
LEFT BANK	SL 1	PE. 85+0	4.000	PIPE CULVERT ø600 mm	0.6		1	1 FLAP GATE	PERKEBUNAN BR. +100m
	SL 2	PE. 99+35	8.900	BOX CULVERT 2.0×1.5×1	2.0	1.5	2	2 SLIDE GATES	
	SL 3	PE. 138+35	11.000	BOX CULVERT 1.5×1.5×1	1.5	1.5	1	1 SLIDE GATE	PAYUNG BR. 110 m
	SL 4	PE. 156+90	12.900	BOX CULVERT 2.0×1.5×1	2.0	1.5	1	1 SLIDE GATE	UNDER CONSTRUCTION
	SL 5	PE. 176+50	16.000	BOX CULVERT 1.5×1.5×1	1.5	1.5	1		RAILWAY BR
	SL 6	PE. 176+80	16.000	BOX CULVERT 2.0×1.5×1	2.0	1.5	1		RAILWAY BR
	SL 7	PE. 189+40	18.000	PIPE CULVERT D=600mm			1		
	SL 8	PE. 196+35	20.300	PIPE CULVERT D=1000mm×2			2		
	SL 9	PE. 200+25	20.900	PIPE CULVERT D=600mm	0.6		1		DENAI BR
	SL 10	PE. 200+40	20.300	PIPE CULVERT D=600mm	0.6		1		DENAI BR
	SL 11	PE. 205+0	20.300	PIPE CULVERT D=600mm	0.6		1		TOLL-WAY BR
	SL 12	PE. 206+30	20.700	OPEN DITCH D=1000mm	1.0	1.0	1		TOLL-WAY BR
	SL 13	PE. 212+0	20.800	BOX CULVERT 1.5×1.5×2	1.5	1.5	2		JL. LONG
	SL 14	PE. 222+0	20.700	BOX CULVERT 2.1×2.4×2	2.1	2.4	2		BINJAI BR
	SL 15	PE. 222+15	21.300	PIPE CULVERT D=1000mm			1		JL. TIMUR
	SL 16	PE. 246+40	23.900	BOX CULVERT 2.0×2.0×1	2.0	2.0	1		AMPLAS BR
	SL 17	PE. 250+90	26.000	PIPE CULVERT D=600mm	0.6		1		
	SL 18	PE. 258+15	26.800	PIPE CULVERT D=600mm			2		PIPE BRIDGE
	SL 19	PE. 258+25	27.600	PIPE CULVERT D=600mm	0.6		1		
	SL 20	PE. 259+60	28.000	PIPE CULVERT D=600mm	0.6		1		
	SL 21	PE. 262+90	28.300	PIPE CULVERT D=1000mm×2			2		
	SL 22	PE. 264+90	28.800	PIPE CULVER D=600mm	0.6		1		
	SL 23	PE. 269+80	28.800	OPEN DITCH B=600mm	0.6	1.0	1		NATIONAL ROAD BR
	SL 24	PE. 269+80	29.300	OPEN DITCH B=600mm	0.6	1.0	1		NATIONAL ROAD BR
	SL 25	PE. 274+98	29.800	PIPE CULVERT D=600mm			1		

	NO.	LACATION	BOTTOM ELEVATION OF SLUICE (EL)	TYPE	B/D (m)	H (m)	NOS	GATE	NOTE
RIGHT BANK	SR 1	PE. 166+80	14.300	PIPE CULVERT D=600mm×2			2		
	SR 2	PE. 176+85	15.900	PIPE CULVERT D=600mm×2			2		RAILWAY BR
	SR 3	PE. 200+10	21.000	OPEN DITCH B=600mm	0.6	1.0	1		DENAI BR
	SR 4	PE. 200+25	20.400	PIPE CULVERT D=1000 mm			1		DENAI BR
	SR 5	PE. 216+0	22.900	OPEN DITCH B=600mm	0.6	1.0	1		
	SR 6	PE. 216+40	23.000	PIPE CULVERT D=600mm			1		
	SR 7	PE. 234+20	24.000	PIPE CULVERT D=600mm			1		
	SR 8	PE. 246+30	26.300	PIPE CULVERT D=600mm			1		AMPLAS BR
	SR 9	PE. 253+20	27.300	PIPE CULVERT D=600mm			1		
	SR 10	PE. 239+0	24.307	BOX CULVERT 2.0×2.2×2	2.0	2.0	2		RIVER
	SR 11	PE. 271+40	28.000	PIPE CULVERT D=600 mm	0.6		1		
	SR 12	PE. 272+86	29.300	PIPE CULVERT D=1000 mm			1		
FLOODWAY	SF 1	FW. 6+80	31.477	PIPE CULVERT D=1000 mm	1.0		1		
	SF 2	FW. 9+81	30.579	BOX CULVERT 2.0×2.0×1	2.0	2.0	1		
	SF 3	FW. 13+0	34.167	PIPE CULVERT D=1000 mm	1.0		1		
	SF 4	FW. 18+0	33.147	PIPE CULVERT D=1000 mm	1.0		1		
	SF 5	FW. 25+24	31.035	BOX CULVERT 2.0×2.0×2	2.0		2		BUTUAN RIVER
	SF 6	FW. 30+0	36.938	PIPE CULVERT D=1000 mm	1.0		1		
	SF 7	FW. 36+50	32.800	PIPE CULVER D=1000 mm	1.0		1		

LEGEND

- : BOUNDARY OF DRAINAGE AREA
- : ROAD
- : RAILWAY
- : DRAINAGE CHANNEL
- : RIVER

0 0.5 1.0 1.5 2.0 2.5 3.0 Km
SCALE 1 : 25.000

NO.	DATE	REVISIONS	ORIGINATED	APPROVED

REPUBLIC OF INDONESIA
MINISTRY OF PUBLIC WORKS
DIRECTORATE GENERAL OF WATER RESOURCES DEVELOPMENT
DIRECTORATE OF TECHNICAL GUIDANCE
MEDAN FLOOD CONTROL PROJECT
DRAINAGE OUTLET
LOCATION OF DRAINAGE OUTLETS

PREPARED	CHECKED	SUBMITTED	CERTIFIED	APPROVED	DRAWING NO.
					DO - 001

JAPAN INTERNATIONAL COOPERATION AGENCY
CTI ENGINEERING CO., LTD

PROYEK PENGELOLAAN ... 2 - 16 ... UTARA

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225	

Fig. 2.2.2 LOCATION OF DRAINAGE OUTLET

In the course of project construction, all existing drainage channels shall be improved or reconstructed, particularly, their outlet structures. For the drainage channel itself, the improvement works are expected under the MMUDP (MUDP III).

(2) Planning Criteria

(a) Design Scale and Discharge

Based on the Flood Control Manual, the guideline for the design scale is as shown below. The design scale of drainage improvement in terms of return period is a 5-year return period for the urban area and a 2-year for the rural area.

Conveyance System	Project Type **	Return Period (yr) *	
		Initial Phase	Final Phase
Primary Drainage System (CA>500 ha)	Rural	2	5
	Urban P<500,000	5	10
	Urban 500,000<P<2,000,000	5	15
	Urban 2,000,000<P	10	25
Secondary Drainage System (CA<500 ha)	Rural	1	2
	Urban P<500,000	2	5
	Urban 500,000<P<2,000,000	2	5
	Urban 2,000,000<P	5	10
Tertiary Drainage System (CA<10 ha)	Rural and Urban	1	2

(Note) * Higher design flood standard shall be applied if economic analysis indicates desirability, or if flooding is a significant risk to human life.
** P = Total Urban Population

The design discharges of drainage outlet is formulated and the detail of analysis on the drainage discharge is presented in the section 3.5.3 of Main Report.

(b) Promises of Design for Drainage Outlet

a. Gravity flow is principally employed to avoid the big O&M cost to be brought by a pumping station.

b. Where the design high water level is higher than the ground elevation of the drainage area, a control gate is employed for the sluice to stop the reverse flow from the river/floodway. No gate is provided for the sluice where the design high water level is lower than the ground elevation.

c. Sluices to be placed close to each other are combined to a certain size of box/pipe culvert to reduce the number of structures in the riverbank.

d. In case the landside area is low in ground elevation, side ditches are provided along the proposed dike/inspection road to drain inland water.

e. Either type of sluice, box culvert, or pipe, could be classified according to the drainage discharge having a flow velocity of 2.5 m^3/s to 3.0 m^3/s. In addition to this criteria, in case earth covering depth is less than 50cm, open ditch type culvert is applied to prevent the occurance of cracks on RC pipes.

Type	Dia./Width (m)	Height (m)	Quantity	Flow Area (m^2)	Design Discharge (m^3/s)
Pipe Culvert	0.600	-	1	0.283	0.707 - 0.989
	0.800	-	1	0.502	0.989 - 1.748
	1.000	-	1	0.785	1.758 - 2.748
	0.800	-	2	1.005	2.748 - 3.517
	1.000	-	2	1.570	3.517 - 5.495
Open Ditch	0.600	1.000	1		0.000 - 0.989
	1.000	1.000	1		1.758 - 2.748
Box Culvert	1.500	1.500	1	2.250	5.495 - 7.875
	2.000	1.500	1	3.000	7.875 - 10.500
	2.000	2.000	1	4.000	10.500 - 14.000
	2.000	2.000	2	8.000	14.000 - 28.000

Through the basic design conditions, the structural dimensions of all the 42 sluices are estimated, as shown in Table 2.2.1.

Structural Detail

Since the inspection roads will be constructed in succession along the riverbanks, ditches are required to be modified to box culverts embedded in the riverbank. If the bottom elevation of drainage ditch is higher than 1 m below the crest elevation of dike and it is impossible to have a enough thickens of cover soil, a open ditch type sluice is applied.

(1) Box Culvert

The following thickness of each members is applied to the box culvert.

side wall and top slab	40 cm,
base slab	50cm and
center wall	30 cm.

In case the total length of box culvert is more than 20 m, the culvert is divided into two portion and a joint is provided.

Wing walls with 3 m wide and 1.0 m higher than height of box culvert provided at inlet and outlet.

Since the ground elevations of the basin of the SL 2, SL 3 and SL 4 are lower than design high water level, sluice gates are installed to prevent the counter flow. A spindle type steel slide gate and manual operation system are applied to the gate.

To prevent the piping, steel sheet piles with 3 m long is provided at the outlet and a cut off wall at the center for the box culvert with gates.

(b) Pipe Culvert

Centrifugal RC pipe which is factory product is applied to the pipe for the pipe culverts. To make easy the maintenance of the culvert, the minimum diameter of the pipe is set at 60 cm. The base of pipes is fixed 180 degree by plain concrete considering the foundation condition and thickness of cover soil. The joint of pipes is covered with in situ reinforced concrete.

Since the ground elevations of the basin of the SL 1 is lower than design high water level, a operation free flap gate is employed to prevent the counter flow from the river side during floods.

(c) Open Ditch Type Sluice

The height of ditch is fixed at 1.0 m and the ditch width of 0.6 m and 1.0 m is appreciable depending on the design discharge. For the inspection road, a reinforced concrete cover with 3 m wide is provided.

2.2.3 Structural Design of Drainadge Outlets

<u>Pipe Culvert</u>

(1) Soil Pressure of Embankment

$Qd = Ce\ r\ Bc^2$

where,	Qd	: soil pressure of embankment (kg/m)
	Ce	: $H<He$ $Ce = (\exp(0.8\ H/Bc)-1)/0.8$
		: $H \geq He$ $Ce = (\exp(0.8\ H/Bc)-1)/0.8+((H-He)/Bc)\exp(0.8He/Bc)$
	r	: unit weight of soil 2,000 kg/m^3
	Bc	: diameter of pipe
	He	: 1.12 Bc
	H	: maximum soil depth (3.9 m)

(2) Active Load

$Ql = (2\ P\ Be\ (1+i))/(2.75(0.2+2H\tan\theta))$

where	Ql	: active load (kg/m)
	P	: wheel load (P=8000kg)
	i	: impact coefficient (i=0.65-0.1H)
	θ	: 45°

(3) Maximum Morment

$M = k\ (Qd+Ql)\ Bc/2$

where, M : maximum bending moment
k : constant k = 0.152 for 90 degrees fix
k = 0.110 for 180 degrees fix

(4) Allowable Bending Moment (Mra)

$$Mr = 0.318\ Pr\ R + 0.239\ W\ R\ (kgm)$$

$$Mra = Mr/Fs$$

where, Pr : cruck load (kg/m)
W : self weight (kg/m)
Mra : allowable bending moment (kgm)
Fs : factor of safety (Fs = 1.25)

(5) Type of Pipe Foundation

The results of calculation of the bending moments for pipe culverts are given bellow.

		φ600	φ800	φ1000
H	m	3.00	3.00	3.00
He	m	1.00	1.33	1.66
K		0.40	0.40	0.40
Ce		8.84	6.42	4.96
r	kg/m^3	1,800.00	1,800.00	1,800.00
Qd	kg/m	5,731.22	7,391.57	8,926.88
Ql	kg/m	845.23	1,126.97	1,408.71
Total	kg/m	6,576.45	8,518.53	10,335.59
M_{90}	kgm	299.89	517.93	785.51
M_{180}	kgm	217.02	374.82	568.46
Pr		3,000.00	3,600.00	4,200.00
W		283.26	502.15	502.15
Mr		306.51	505.93	727.81
Mra	kgm	245.21	404.74	582.25

Note : M_{90} - bending moment of 90° fix foundation.
: M_{180} - bending moment of 180° fix foundation.

As shown in above table, the base of pipes shall be fixed 180 degree by plain concrete to prevent the crack on the RC pipe.

Step-wise Fall Structure for Batuan River

(1) Structural Dimensions

The dimensions of structures and rahment frame for analysis are shown in Fig. 2.2.3.

(2) Loading Condition

The load to be considered for the structural design of box culverts are:

a. Earth pressure : γt = 1.9 t/m3
Ka = 0.341 (see Table 2.2.2)

b. Live load : Pvl = 1 t/m^2

c. Dead load : reinforced concrete γc = 2.5 t/m^3

(3) Results of Calculation

The moment, sher force and axial force are calculated by rahment analysis and the out puts from computer is shown in the following computer out put.

The results of stress calculation are presented in Table 2.2.3 and the reinforcement arrangement are as shown in Fig. 2.2.4.

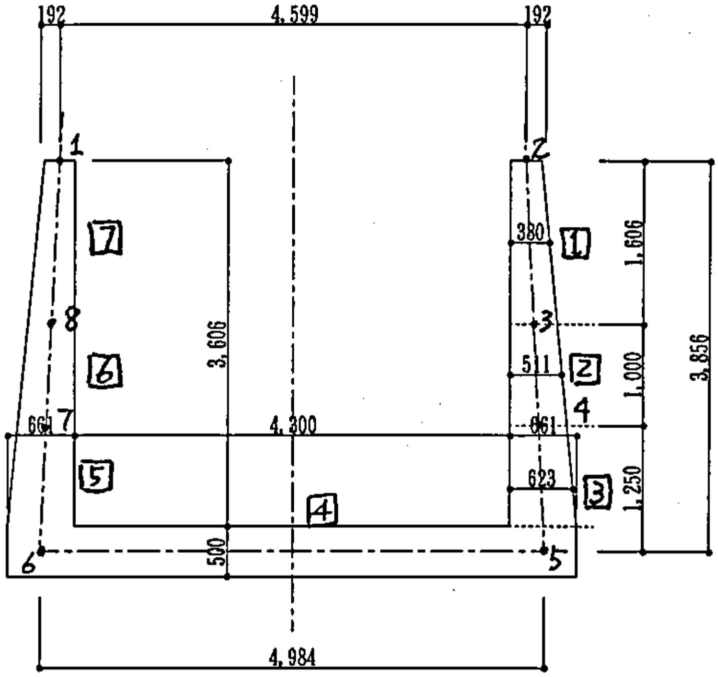


Fig. 2.2.3 Dimensions and Frame

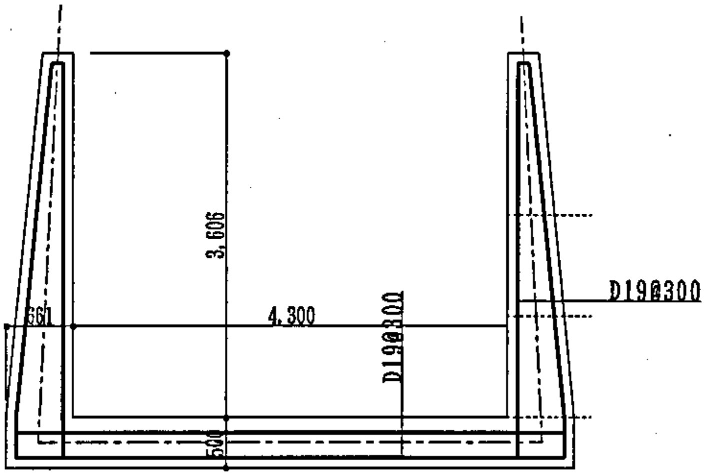


Fig. 2.2.4 Reinforcement Arrangement

Table 2.2.2 CALCULATION OF EARTH PRESSURE

			Wall 1	Wall 2	Wall3
Basic Conditions					
Earth depth to acting point	h	(m)	1.606	2.606	3.856
Unit weight of soil	γ	(tf/m^3)	1.900	1.900	1.900
Soil cohesion	c	(tf/m^2)	0.000	0.000	0.000
Internal friction angle of soil	φ	(°)	30.000	30.000	30.000
Back side wall slope	m	1:	0.100	0.100	0.100
Angle between back side and vertical plane	θ	(°)	5.711	5.711	5.711
Back-fill ground surface slope	n	1:	##########	##########	##########
Angle between ground surface and horizontal plane	α	(°)	0.000	0.000	-0.000
Surcharge in normal condition	q	(tf/m^2)	1.000	1.000	1.000
Surcharge in seismic case	q'	(tf/m^2)	0.000	0.000	0.000
Seismic coefficient in vertical direction	Kv		0.000	0.000	0.010
Seismic coefficient in horizontal direction	Kh		0.183	0.183	0.183
Friction angle of soil to concrete	δ	(°)	20.000	20.000	20.000
Angle θo=Kh/(1-Kv)	θo	(°)	10.370	10.370	10.473
Unit weight of saturated soil	γ'	(tf/m^3)	1.900	1.900	1.900
Seismic coefficient in horizontal direction (submarged)	Kh'		0.300	0.300	0.386
Calculation of Coefficient of each Earth Pressure					
Coefficient of active earth pressure	Ka	(tf/m^2)	0.341	0.341	0.341
Coefficient of passive earth pressure	Kp	(tf/m^2)	1.558	1.558	1.558
Coefficient of steady earth pressure	Ks	(tf/m^2)	0.500	0.500	0.500
Coefficient of active earth pressure	Kae	(tf/m^2)	0.488	0.488	0.490
Coefficient of passive earth pressure	Kpe	(tf/m^2)	0.560	0.560	0.686
Earth Pressure					
Active earth pressure	Pa		1.380	2.027	2.836
Passive earth pressure	Pp		6.311	9.271	12.970
Steady earth pressure	Ps		2.026	2.976	4.163
Active earth pressure under easrthquake	Pae		1.490	2.418	3.591
Passive earth pressure under earthquake	Ppe		1.708	2.771	5.023

1

****** SL-5 U-CHANNEL 4300 X 3606 ******** PAGE - 1

** BAISIC-DATA LIST **

NUMBER OF NODES	8	
NUMBER OF MEMBERS	7	
NUMBER OF BAISIC LOAD	1	CASES
NUMBER OF LOAD COMBINATION	0	CASES
ALLOWABLE STRESS (CONCRETE)	70.0	KG/CM**2
ALLOWABLE STRESS (STEEL)	1600.0	KG/CM**2
MAX-DAMMEN-RYOKU	CASE= 0 - CASE= 0	
OUT PUT FORM	F TYPE (UNIT = MM, 1/1000RAD, TON, TON-M)	
KANZAN KEISUU	NODAL-DISPLASEMENT, NODAL-REACTION, MEMBER-DISPLASEMENT, MEMBER-FORCE	

1

****** SL-5 U-CHANNEL 4300 X 3606 ******** PAGE - 2

* NODE COODINATE *

NO	X	Y
1	.000	.000

2	4.599	.000
3	4.679	1.606
4	4.729	2.606
5	4.791	3.856
6	-.192	3.856
7	-.130	2.606
8	-.080	1.606

1

** MEMBER-DATA LIST **

NO	NODE-NO L	NODE-NO R	CONDITION L	CONDITION R	DANMENSEKI A(M**2)	YOUNG MODULUS E(TON/M**2)	NIJI MOMENT I(M**4)	BUZAICHO L(M)	THETA (DEG)	T.E.COEF (M/M/DEG)	JIBAN HANRYOKU COEF KV(T/M**2)	JIBAN HANRYOKU COEF KS(T/M**2)
1	2	3	FIX	FIX	.380	.25000E+07	.45700E-02	1.608	87.15	.0000E+00	.0	.0
2	3	4	FIX	FIX	.511	.25000E+07	.11120E-01	1.001	87.14	.0000E+00	.0	.0
3	4	5	FIX	FIX	.623	.25000E+07	.20150E-01	1.252	87.16	.0000E+00	.0	.0
4	5	6	FIX	FIX	.500	.25000E+07	.10100E-01	4.983	.00	.0000E+00	10000.0	.0
5	6	7	FIX	FIX	.623	.25000E+07	.20150E-01	1.252	-87.16	.0000E+00	.0	.0
6	7	8	FIX	FIX	.511	.25000E+07	.11120E-01	1.001	-87.14	.0000E+00	.0	.0
7	8	1	FIX	FIX	.380	.25000E+07	.45700E-02	1.608	-87.15	.0000E+00	.0	.0

1

** BAISIC LOAD DATA **

	CASE	TYPE	N	P1	(P3)	P2	L1	L2	ANGLE	HANTEI
	1									
+		DISTRIBU	1	1.000		2.380	.000	1.606	.00	0
+		DISTRIBU	2	1.380		3.027	.000	1.000	.00	0

+	DISTRIBU	3	3.027	3.836	.000	1.250	.00	0
+	DISTRIBU	5	3.836	3.027	.000	1.250	.00	0
+	DISTRIBU	6	3.027	1.380	.000	1.000	.00	0
+	DISTRIBU	7	1.380	1.000	.000	1.606	.00	0
+	DISTRIBU	1	.950	.950	.000	1.606	90.00	0
+	DISTRIBU	2	1.278	1.278	.000	1.000	90.00	0
+	DISTRIBU	3	1.558	1.558	.000	1.250	90.00	0
+	DISTRIBU	5	1.558	1.558	.000	1.250	-90.00	0
+	DISTRIBU	6	1.278	1.278	.000	1.000	-90.00	0
+	DISTRIBU	7	.950	.950	.000	1.606	-90.00	0
+	DISTRIBU	4	1.898	1.898	.000	4.984	.00	0

1

** NODAL DISPLASEMENT **

CASE	N	DX	DY
1			
	1	25.66862	-.00025
	2	25.66128	-.00025
	3	25.66303	-.00033
	4	25.66401	-.00038

5	25.66502	-.00044
6	25.66506	-.00043
7	25.66604	-.00038
8	25.66697	-.00033

1

** LOAD CASE = 1 **

NO.	DIST	DEFORMATION	DEFLECTION	DEFL-ANGLE	AXIAL-FORCE	SHEAR-FORCE	MOMENT	REACTION
1	(2- 3)							
	.000	1.27644	-25.62951	-.00111	-.090	.098	-.047	
	.402	1.27644	-25.62996	-.00111	-.472	-.373	-.098	
	.804	1.27644	-25.63041	-.00110	-.854	-.984	-.366	
	1.206	1.27644	-25.63085	-.00108	-1.235	-1.733	-.908	
	1.608	1.27644	-25.63127	-.00104	-1.615	-2.616	-1.778	
2	(3- 4)							
	.000	1.28122	-25.63103	-.00103	-1.634	-3.243	-1.999	
	.250	1.28122	-25.63129	-.00101	-1.954	-3.640	-2.858	
	.501	1.28122	-25.63153	-.00098	-2.273	-4.140	-3.830	
	.751	1.28122	-25.63178	-.00094	-2.593	-4.743	-4.939	
	1.001	1.28122	-25.63200	-.00089	-2.912	-5.446	-6.213	
3	(4- 5)							
	.000	1.27098	-25.63252	-.00090	-3.076	-2.962	-5.817	
	.313	1.27098	-25.63280	-.00086	-3.564	-3.940	-6.895	
	.626	1.27098	-25.63306	-.00082	-4.051	-4.982	-8.289	
	.939	1.27098	-25.63330	-.00076	-4.538	-6.088	-10.020	
	1.252	1.27098	-25.63353	-.00069	-5.024	-7.251	-12.106	
4	(5- 6)							
	.000	-25.66502	.00044	-.00069	-8.944	5.303	-13.106	4.38390

1.246	-25.66503	-.00009	-.00021	-8.944	4.520	-6.319	-.86226
2.491	-25.66504	-.00020	.00000	-8.944	.091	-3.301	-2.01600
3.737	-25.66505	-.00008	.00021	-8.944	-4.313	-6.080	-.83521
4.983	-25.66506	.00043	.00067	-8.944	-5.114	-12.606	4.28951
5 (6- 7)							
.000	1.27185	25.63352	.00067	-4.619	9.352	-13.204	
.313	1.27185	25.63375	.00074	-4.131	8.184	-10.462	
.626	1.27185	25.63399	.00080	-3.644	7.078	-8.076	
.939	1.27185	25.63424	.00084	-3.156	6.037	-6.026	
1.252	1.27185	25.63451	.00087	-2.671	5.063	-4.292	
6 (7- 8)							
.000	1.28208	25.63400	.00087	-2.752	5.109	-4.925	
.250	1.28208	25.63422	.00091	-2.432	4.403	-3.736	
.501	1.28208	25.63446	.00094	-2.113	3.800	-2.712	
.751	1.28208	25.63469	.00096	-1.793	3.300	-1.826	
1.001	1.28208	25.63494	.00097	-1.474	2.905	-1.051	
7 (8- 1)							
.000	1.27730	25.63517	.00097	-1.573	1.874	-1.474	
.402	1.27730	25.63557	.00101	-1.191	1.338	-.830	
.804	1.27730	25.63598	.00103	-.809	.841	-.393	
1.206	1.27730	25.63640	.00104	-.427	.382	-.148	
1.608	1.27730	25.63682	.00104	-.047	-.037	-.081	

Table 2.2.3 STRESS CALCULATION

Type		Batuan River					
		Side Wall 1		Side Wall 2		Side Wall 3	
N	t	1.62		2.91		5.20	
M	tm	1.78		6.21		12.02	
Q	t	2.62		5.45		7.25	
d	cm	30.00		46.06		66.10	
d'	cm	10.00		10.00		10.00	
u	cm	10.00		18.03		28.05	
b	cm	100.00		100.00		100.00	
r	A's/As	1.00		1.00		1.00	
n		15.00		15.00		15.00	
Sca	kg/cm2	70.00		70.00		70.00	
Ssa	kg/cm2	1,800.00		1,800.00	0.18	1,800.00	
Tca	kg/cm2	4.25		4.25		4.25	
M'		1.94		6.74		13.48	
					1.90		
C		32.48	32.48	22.04	22.04	22.70	22.70
k (C)		0.07	0.001	0.10	0.001	0.00	0.000
np (C)		0.0024	0.0018	0.0049	0.0043	-0.0001	0.0000
S		55.68	55.68	37.78	37.78	38.91	38.91
k (S)		0.18	0.000	0.21	0.000	0.20	-0.001
np (S)		0.0140	0.0139	0.0229	0.0232	0.0203	0.0212
d'/d		0.33		0.22		0.15	
f	cm	120.09		231.39		258.95	
f/d		4.00		5.02		3.92	
np		0.0140		0.0229		0.0203	
r-As		2.80		7.04		8.97	
r-As'		2.80		7.04		8.97	
Re-Bar Arrangement							
As1	Diameter	D16		D19		D19	
	Area (cm2)	1.99		2.87		2.87	
	Pitch (cm)	30.00		30.00		30.00	
	As1	6.62		9.55		9.55	
As2	Diameter	0.00		0.00		0.00	
	Area (cm2)	0.00		0.00		0.00	
	Pitch (cm)	30.00		30.00		30.00	
	As2 (cm2)	0.00		0.00		0.00	
Total of As		6.62	(OK)	9.55	(OK)	9.55	(OK)
		-2.80		-7.04		-8.97	
As'1	Diameter	D16		D19		D19	
	Area (cm2)	1.99		2.87		2.87	
	Pitch (cm)	30.00		30.00		30.00	
	As'1 (cm2)	6.62		9.55		9.55	
As'2	Diameter	0.00		0.00		0.00	
	Area (cm2)	0.00		0.00		0.00	
	Pitch (cm)	25.00		25.00		25.00	
	As'2 (cm2)	0.00		0.00		0.00	
Total of As'		6.62	(OK)	9.55	(OK)	9.55	(OK)
		-2.80		-7.04		-8.97	
Check	np	0.03310		0.03110		0.02167	
	k	0.26	0.000	0.24	0.000	0.21	0.000
	A's/As	1.00		1.00		1.00	
	C	8.82		8.95		9.89	
	S	24.88		28.64		37.99	
	Z	1.05	0.24	1.08	0.22	1.07	0.19
	Sc	19.01	OK	28.43	OK	30.51	OK
	Ss	804.37	OK	1,364.65	OK	1,757.47	OK
	Tc	0.87	OK	1.18	OK	1.10	OK
	x	7.85		10.97		13.66	

Open Ditch Type

(1) Structural Dimensions

The dimensions of structures and rahment frame for analysis are shown in Fig. 2.2.5.

(2) Loading Condition

The load to be considered for the structural design of box culverts are:

a. Earth pressure : $\gamma t = 1.9$ t/m3
$Ka = 0.341$

b. Live load : $Pvl = 1$ t/m^2

c. Dead load : reinforced concrete $\gamma c = 2.5$ t/m^3

(3) Results of Calculation

The moment, sher force and axial force are calculated by rahment analysis and the out puts from computer is shown in the following computer out put.

The results of stress calculation are presented in Table 2.2.4 and the reinforcement arrangement are as shown in Fig. 2.2.6.

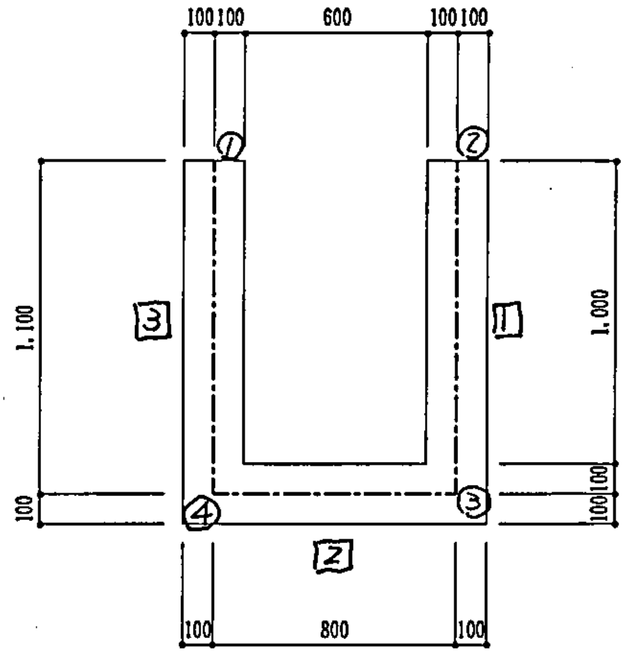


Fig. 2.2.5 Dimensions & Frame

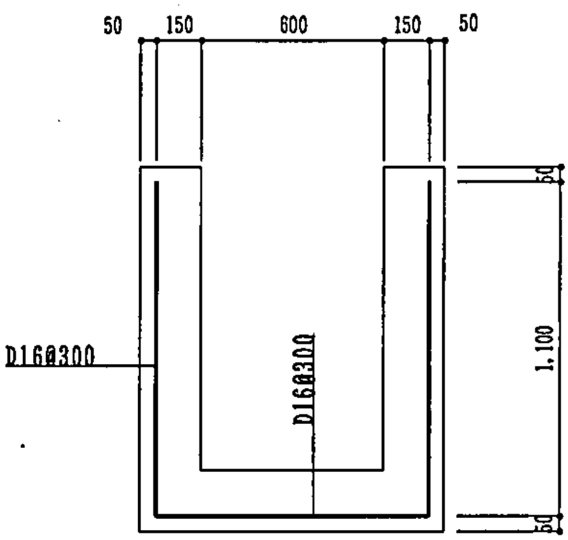


Fig. 2.2.6 Reinforcement Arrangement

```
1

        ****** OPEN DITCH  600 X 1000 ********                                  PAGE -  1

   ** BAISIC-DATA LİST **

        NUMBER OF NODES                          4

        NUMBER OF MEMBERS                        3

        NUMBER OF BAISIC LOAD                    1      CASES

        NUMBER OF LOAD COMBINATION               0      CASES

        ALLOWABLE STRESS (CONCRETE)           70.0      KG/CM**2

        ALLOWABLE STRESS (STEEL)            1600.0      KG/CM**2

        MAX-DAMMEN-RYOKU               CASE=  0 - CASE=  0

        OUT PUT FORM                   F TYPE ( UNIT = MM, 1/1000RAD, TON, TON-M )

        KANZAN KEISUU                  NODAL-DISPLASEMENT, NODAL-REACTION, MEMBER-DISPLASEMENT, MEMBER-FORCE
1

        ****** OPEN DITCH  600 X 1000 ********                                  PAGE -  2

 * NODE COODINATE *

   NO     X        Y

    1    .000     .000
```

2	.900	.000
3	.900	1.100
4	.000	1.100

1

** MEMBER-DATA LIST **

NO	NODE-NO L	R	CONDITION L	R	DANMENSEKI A(M**2)	YOUNG MODULUS E(TON/M**2)	NIJI MOMENT I(M**4)	BUZAICHO L(M)	THETA (DEG)	T.E.COEF (M/M/DEG)	JIBAN HANRYOKU COEF KV(T/M**2)	KS(T/M**2)
1	2	3	FIX	FIX	.200	.25000E+07	.66000E-03	1.100	90.00	.0000E+00	.0	.0
2	3	4	FIX	FIX	.200	.25000E+07	.66000E-03	.900	.00	.0000E+00	10000.0	.0
3	4	1	FIX	FIX	.200	.25000E+07	.66000E-03	1.100	-90.00	.0000E+00	.0	.0

1

** BAISIC LOAD DATA **

	CASE	TYPE	N	P1	(P3)	P2	L1	L2	ANGLE	HANTEI
	1									
+		DISTRIBU	1	1.000		3.090	.000	1.100	.00	0
+		DISTRIBU	2	1.500		1.500	.000	.900	.00	0
+		DISTRIBU	3	3.090		1.000	.000	1.100	.00	0

1

** NODAL DISPLASEMENT **

CASE	N	DX	DY
1			
	1	.00048	-.00019
	2	-.00048	-.00019
	3	.00000	-.00019
	4	.00000	-.00019

1

****** OPEN DITCH 600 X 1000 ********

** LOAD CASE = 1 **

NO.	DIST	DEFORMATION	DEFLECTION	DEFL-ANGLE	AXIAL-FORCE	SHEAR-FORCE	MOMENT	REACTION
1	(2- 3)							
	.000	-.00019	.00048	-.00048	.000	.000	.000	
	.275	-.00019	.00035	-.00048	.000	-.347	-.044	
	.550	-.00019	.00022	-.00046	.000	-.837	-.204	
	.825	-.00019	.00010	-.00040	.000	-1.472	-.518	
	1.100	-.00019	.00000	-.00028	.000	-2.250	-1.026	
2	(3- 4)							
	.000	.00000	.00019	-.00028	-2.249	.000	-1.026	1.91696
	.225	.00000	.00014	-.00014	-2.249	.035	-1.021	1.44780
	.450	.00000	.00013	.00000	-2.249	.000	-1.016	1.29182
	.675	.00000	.00014	.00014	-2.249	-.035	-1.021	1.44780
	.900	.00000	.00019	.00028	-2.249	.000	-1.026	1.91697
3	(4- 1)							
	.000	.00019	.00000	.00028	.000	2.249	-1.026	
	.275	.00019	.00010	.00040	.000	1.472	-.518	
	.550	.00019	.00022	.00046	.000	.837	-.204	
	.825	.00019	.00035	.00048	.000	.347	-.044	
	1.100	.00019	.00048	.00048	.000	.000	.000	

Table 2.2.4 STRESS CALCULATION OF OPEN DITCH TYPE

Type		Open Ditch			
		Side Wall		Base Concrete	
N	t	0.00		2.25	
M	tm	1.03		1.03	
Q	t	2.25		0.00	
d	cm	20.00		20.00	
d'	cm	5.00		5.00	
u	cm	7.50		7.50	
b	cm	100.00		100.00	
r	A's/As	1.00		1.00	
n		15.00		15.00	
Sca	kg/cm2	70.00		70.00	
Ssa	kg/cm2	1,600.00		1,600.00	
Tca	kg/cm2	4.25		4.25	
M'		1.03		1.19	
C		27.29	32.96	23.44	23.44
k (C)		0.07	0.000	0.00	0.000
np (C)		0.0029	0.0024	-0.0002	0.0000
S		41.59	41.59	35.71	35.71
k (S)		-0.26	0.000	0.22	0.001
np (S)		0.0194	0.0196	0.0205	0.0195
d'/d		0.25		0.25	
f	cm	9,999.99		53.12	
f/d		500.00		2.66	
np		0.0194		0.0205	
r-As		2.59		2.73	
r-As'		2.59		2.73	
Re-Bar Arrangement					
As1	Diameter	D16		D16	
	Area (cm2)	1.99		1.99	
	Pitch (cm)	30.00		30.00	
	As1	6.62		6.62	
As2	Diameter	0.00		0.00	
	Area (cm2)	0.00		0.00	
	Pitch (cm)	30.00		30.00	
	As2 (cm2)	0.00		0.00	
Total of As		6.62	(OK)	6.62	(OK)
		-2.59		-2.73	
As'1	Diameter	D16		D16	
	Area (cm2)	1.99		1.99	
	Pitch (cm)	30.00		30.00	
	As'1 (cm2)	6.62		6.62	
As'2	Diameter	0.00		0.00	
	Area (cm2)	0.00		0.00	
	Pitch (cm)	25.00		25.00	
	As'2 (cm2)	0.00		0.00	
Total of As'		6.62	(OK)	6.62	(OK)
		-2.59		-2.73	
Check	np	0.04965		0.04965	
	k	0.27	0.001	0.31	0.001
	A's/As	1.00		1.00	
	C	8.07		6.81	
	S	22.16		15.06	
	Z	1.10	0.27	1.10	0.27
	Sc	20.69	OK	20.34	OK
	Ss	852.52	OK	674.63	OK
	Tc	1.13	OK	0.00	OK
	x	5.34		6.23	

Box Culvert

(1) Structural Dimensions

Following types of box culvert are emproyed in the project. The dimensions of structures and rahment frame for analysis are shown in Fig. 2.2.7.

Case No.	Type	h b
Case 1	Single Section	1.5 m x 1.5 m
Case 2	Single Section	1.5 m x 2.0 m
Case 3	Single Section	2.0 m x 2.0 m
Case 4	Double Section	1.5 m x 1.5 m x 2
Case 5	Double Section	1.5 m x 2.0 m x 2
Case 6	Double Section	2.0 m x 2.0 m x 2

(2) Loading Condition

The load to be considered for the structural design of box culverts are:

a. Earth covering : He = 3.2 m

b. Water level : Hw = 2/3 h m

c. Active load : Pvl = 1 t/m^2

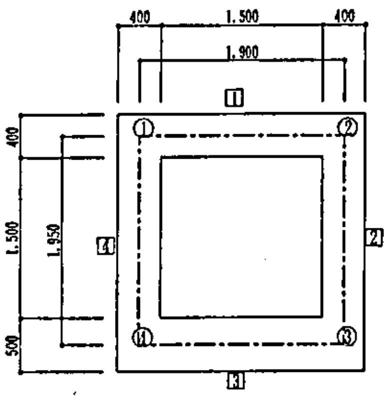
d. Dead load : reinforced concrete $\gamma c = 2.5\ t/m^3$

The results of load calculation are shown in Table 2.2.5.

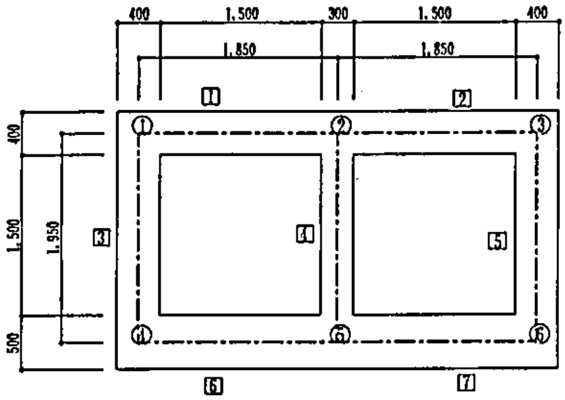
(3) Results of Calculation

The moment, sher force and axial force are calculated by rahment analysis and the out puts from computer is attached bellow.

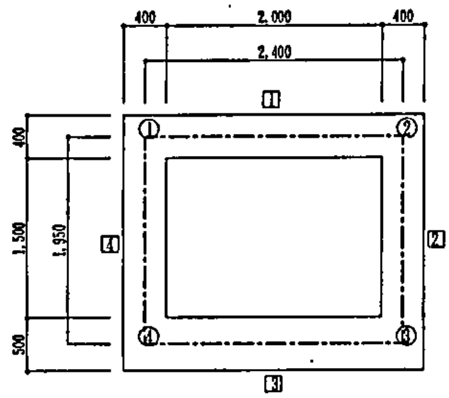
The results of stress calculation are presented in Table 2.2.4 and the reinforcement arrangement are as shown in Fig. 2.2.8.



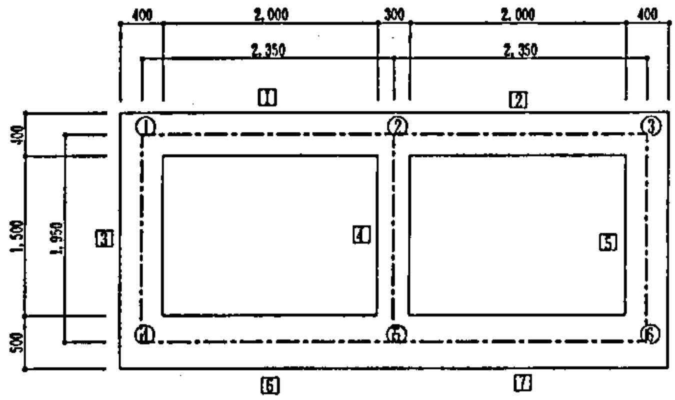
Single Section
1.5m x 1.5m



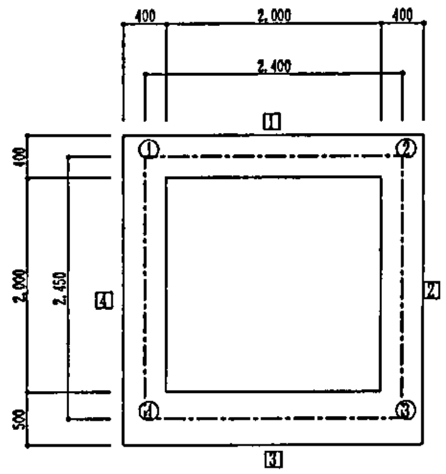
Double Section
1.5m x 1.5m x 2



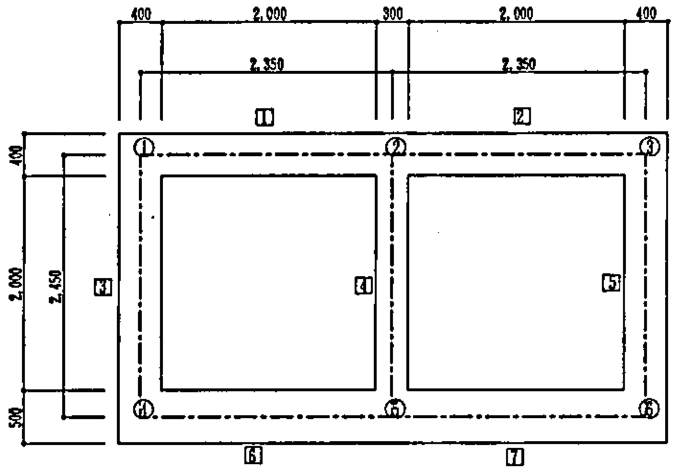
Single Section
1.5m x 2.0m



Double Section
1.5m x 2.0m x 2



Single Section
2.0m x 2.0m



Double Section
2.0m x 2.0m x 2

Fig. 2.2.7 DIMENSIONS AND FRAME OF RAHMEN

Table 2.2.5 LOAD COMBINATION OF BOX CULVERTS

Load Conditions of Box Culvert			Single			Double	Double	Double
			1.5x1.5x1	1.5x2.0x1	2.0x2.0x1	1.5x1.5x1	1.5x2.0x1	2.0x2.0x1
Frame Length								
Top Slab			1.900	2.400	2.400	1.850	2.350	2.350
Side Wall			1.950	1.950	2.450	1.950	1.950	2.450
Height of Box	h_0	m	1.500	1.500	2.000	1.500	2.000	2.000
Width of Box	L_0	m	1.500	2.000	2.000	1.500	1.500	2.000
Soil Depth	h_1	m	3.200	3.200	3.200	3.200	3.200	3.200
	h_2	m	5.150	5.150	5.650	5.150	5.650	5.650
	γs	t/m^3	1.900	1.900	1.900	1.900	1.900	1.900
1 Vertical Earth Pressure								
	P_{vd1}	t/m^2	6.080	6.080	6.080	6.080	6.080	6.080
2 Horizontal Earth Pressure								
Coefficient of E.P.	k		0.500	0.500	0.500	0.500	0.500	0.500
	P_{hd1}	t/m^2	3.040	3.040	3.040	3.040	3.040	3.040
	P_{hd2}	t/m^2	4.893	4.893	5.368	4.893	5.368	5.368
3 Water Pressure								
Water Depth	h_w	m	1.000	1.000	1.250	1.000	1.250	1.250
	P_{w1}	t/m^2	0.000	0.000	0.000	0.000	0.000	0.000
	P_{w2}	t/m^2	1.000	1.000	1.250	1.000	1.250	1.250
4 Active Load								
	P_{vl}		1.000	1.000	1.000	1.000	1.000	1.000
4 Dead Load								
Unit Uweight								
Reinforced Conc.	γ_c	t/m^3	2.500	2.500	2.500	2.500	2.500	2.500
Self Weight (Top)	D_1	t/m	2.300	2.800	2.800	2.050	2.050	2.550
Self Weight (Side)	D_2	t/m	3.000	3.000	4.000	2.063	2.750	2.750
5 Summary of Loads								
Load 1 (Top Beem)			8.291	8.247	8.247	8.188	7.952	8.165
Load 2 (Side wall, Soil)			3.040	3.040	3.040	3.040	3.040	3.040
			4.893	4.893	5.368	4.893	5.368	5.368
Load 2 (Side wall, Water)			0.000	0.000	0.000	0.000	0.000	0.000
			1.000	1.000	1.250	1.000	1.250	1.250
Water Depth			1.000	1.000	1.250	1.000	1.250	1.250
Load 5 (Bottom Beem)			9.869	9.497	9.913	8.746	8.537	8.750

1

** BAISIC-DATA LIST **

NUMBER OF NODES	4	
NUMBER OF MEMBERS	4	
NUMBER OF BAISIC LOAD	1	CASES
NUMBER OF LOAD COMBINATION	0	CASES
ALLOWABLE STRESS (CONCRETE)	70.0	KG/CM**2
ALLOWABLE STRESS (STEEL)	1600.0	KG/CM**2
MAX-DAMMEN-RYOKU	CASE= 0 - CASE= 0	
OUT PUT FORM	F TYPE (UNIT = MM, 1/1000RAD, TON, TON-M)	
KANZAN KEISUU	NODAL-DISPLASEMENT, NODAL-REACTION, MEMBER-DISPLASEMENT, MEMBER-FORCE	

1

* NODE COODINATE *

NO	X	Y
1	.000	.000
2	1.900	.000
3	1.900	1.950
4	.000	1.950

1

****** BOX CULVERT 1.5 X 1.5 ******** PAGE - 3

** MEMBER-DATA LIST **

NO	NODE-NO L	NODE-NO R	CONDITION L	CONDITION R	DANMENSEKI A(M**2)	YOUNG MODULUS E(TON/M**2)	NIJI MOMENT I(M**4)	BUZAICHO L(M)	THETA (DEG)	T.E.COEF (M/M/DEG)	JIBAN HANRYOKU COEF KV(T/M**2)	JIBAN HANRYOKU COEF KS(T/M**2)
1	1	2	FIX	FIX	.400	.25000E+07	.53300E-02	1.900	.00	.0000E+00	.0	.0
2	2	3	FIX	FIX	.400	.25000E+07	.53300E-02	1.950	90.00	.0000E+00	.0	.0
3	3	4	FIX	FIX	.500	.25000E+07	.10400E-01	1.900	.00	.0000E+00	10000.0	.0
4	4	1	FIX	FIX	.400	.25000E+07	.53300E-02	1.950	-90.00	.0000E+00	.0	.0

1

****** BOX CULVERT 1.5 X 1.5 ******** PAGE - 4

** BAISIC LOAD DATA **

	CASE	TYPE	N	P1	(P3)	P2	L1	L2	ANGLE	HANTEI
	1									
+		DISTRIBU	1	7.080		7.080	.000	1.900	.00	0
+		DISTRIBU	2	3.040		4.890	.000	1.950	.00	0

+	DISTRIBU	3	9.900	9.900	.000	1.900	.00	0
+	DISTRIBU	4	4.890	3.040	.000	1.950	.00	0
+	DISTRIBU	2	.000	1.000	.950	1.950	.00	0
+	DISTRIBU	4	1.000	.000	.000	1.000	.00	0
+	DISTRIBU	2	1.000	1.000	.000	1.950	90.00	0
+	DISTRIBU	4	1.000	1.000	.000	1.950	-90.00	0

1

** NODAL DISPLASEMENT **

CASE	N	DX	DY
1			
	1	.00001	-.00005
	2	.00000	-.00005
	3	.00000	-.00006
	4	.00001	-.00006

1

** LOAD CASE = 1 **

NO.	DIST	DEFORMATION	DEFLECTION	DEFL-ANGLE	AXIAL-FORCE	SHEAR-FORCE	MOMENT	REACTION

				N	S	M	
1 (1- 2)							
.000	.00001	-.00005	.00003	-3.562	6.726	-1.675	
.475	.00000	-.00002	.00004	-3.562	3.363	.721	
.950	.00000	-.00001	.00000	-3.562	.000	1.520	
1.425	.00000	-.00002	-.00004	-3.562	-3.363	.721	
1.900	.00000	-.00005	-.00003	-3.562	-6.726	-1.675	
2 (2- 3)							
.000	-.00005	.00000	-.00003	-6.726	3.562	-1.675	
.488	-.00005	.00000	.00000	-7.213	1.967	-.318	
.975	-.00005	.00000	.00000	-7.701	.146	.206	
1.463	-.00006	.00000	.00000	-8.188	-2.030	-.234	
1.950	-.00006	.00000	.00003	-8.676	-4.670	-1.849	
3 (3- 4)							
.000	.00000	.00006	.00003	-4.670	8.676	-1.849	.60153
.475	.00000	.00008	.00003	-4.670	4.302	1.230	.78368
.950	.00000	.00009	.00000	-4.670	.000	2.250	.87395
1.425	.00000	.00008	-.00003	-4.670	-4.302	1.230	.78367
1.900	-.00001	.00006	-.00003	-4.670	-8.676	-1.849	.60152
4 (4- 1)							
.000	.00006	.00001	-.00003	-8.676	4.670	-1.849	
.488	.00006	.00000	.00000	-8.189	2.030	-.234	
.975	.00005	.00000	.00000	-7.701	-.146	.206	
1.463	.00005	.00000	.00000	-7.214	-1.967	-.318	
1.950	.00005	.00001	.00003	-6.726	-3.562	-1.675	

1

****** BOX CULVERT 1.5 X 2.0 ******** PAGE - 1

** BAISIC-DATA LIST **

NUMBER OF NODES	4	
NUMBER OF MEMBERS	4	
NUMBER OF BAISIC LOAD	1	CASES
NUMBER OF LOAD COMBINATION	0	CASES
ALLOWABLE STRESS (CONCRETE)	70.0	KG/CM**2
ALLOWABLE STRESS (STEEL)	1600.0	KG/CM**2
MAX-DAMMEN-RYOKU	CASE= 0 - CASE= 0	
OUT PUT FORM	F TYPE (UNIT = MM, 1/1000RAD, TON, TON-M)	
KANZAN KEISUU	NODAL-DISPLASEMENT, NODAL-REACTION, MEMBER-DISPLASEMENT, MEMBER-FORCE	

1

****** BOX CULVERT 1.5 X 2.0 ******** PAGE - 2

* NODE COODINATE *

NO	X	Y
1	.000	.000
2	2.400	.000
3	2.400	1.950
4	.000	1.950

1

** MEMBER-DATA LIST **

NO	NODE-NO L	NODE-NO R	CONDITION L	CONDITION R	DANMENSEKI A(M**2)	YOUNG MODULUS E(TON/M**2)	NIJI MOMENT I(M**4)	BUZAICHO L(M)	THETA (DEG)	T.E.COEF (M/M/DEG)	JIBAN HANRYOKU COEF KV(T/M**2)	KS(T/M**2)
1	1	2	FIX	FIX	.400	.25000E+07	.53300E-02	2.400	.00	.0000E+00	.0	.0
2	2	3	FIX	FIX	.400	.25000E+07	.53300E-02	1.950	90.00	.0000E+00	.0	.0
3	3	4	FIX	FIX	.500	.25000E+07	.10400E-01	2.400	.00	.0000E+00	10000.0	.0
4	4	1	FIX	FIX	.400	.25000E+07	.53300E-02	1.950	-90.00	.0000E+00	.0	.0

1

** BAISIC LOAD DATA **

	CASE	TYPE	N	P1	(P3)	P2	L1	L2	ANGLE	HANTEI
	1									
+		DISTRIBU	1	7.080		7.080	.000	2.400	.00	0
+		DISTRIBU	2	3.040		4.890	.000	1.950	.00	0

+	DISTRIBU	3	9.500	9.500	.000	2.400	.00	0
+	DISTRIBU	4	4.890	3.040	.000	1.950	.00	0
+	DISTRIBU	2	.000	1.000	.950	1.950	.00	0
+	DISTRIBU	4	1.000	.000	.000	1.000	.00	0
+	DISTRIBU	2	1.000	1.000	.000	1.950	90.00	0
+	DISTRIBU	4	1.000	1.000	.000	1.950	-90.00	0

1

** NODAL DISPLASEMENT **

CASE	N	DX	DY
1			
	1	.00000	-.00001
	2	-.00001	-.00001
	3	-.00001	-.00003
	4	.00000	-.00003

1

** LOAD CASE = 1 **

NO.	DIST	DEFORMATION	DEFLECTION	DEFL-ANGLE	AXIAL-FORCE	SHEAR-FORCE	MOMENT	REACTION

				N	S	M	
1 (1- 2)							
.000	.00000	-.00001	.00008	-3.673	8.496	-2.459	
.600	.00000	.00005	.00010	-3.673	4.248	1.364	
1.200	.00000	.00008	.00000	-3.673	.000	2.639	
1.800	-.00001	.00005	-.00010	-3.673	-4.248	1.364	
2.400	-.00001	-.00001	-.00008	-3.673	-8.496	-2.459	
2 (2- 3)							
.000	-.00001	.00001	-.00008	-8.496	3.673	-2.459	
.488	-.00002	-.00002	-.00002	-8.984	2.079	-1.048	
.975	-.00002	-.00002	.00000	-9.471	.258	-.469	
1.463	-.00003	-.00001	.00002	-9.958	-1.918	-.855	
1.950	-.00003	.00001	.00008	-10.446	-4.558	-2.415	
3 (3- 4)							
.000	.00001	.00003	.00008	-4.558	10.446	-2.415	.32726
.600	.00001	.00008	.00007	-4.558	5.099	2.233	.84293
1.200	.00000	.00011	.00000	-4.558	.000	3.755	1.08541
1.800	.00000	.00008	-.00007	-4.558	-5.099	2.233	.84294
2.400	.00000	.00003	-.00008	-4.558	-10.446	-2.415	.32727
4 (4- 1)							
.000	.00003	.00000	-.00008	-10.446	4.558	-2.415	
.488	.00003	-.00002	-.00002	-9.958	1.918	-.855	
.975	.00002	-.00003	.00000	-9.471	-.258	-.469	
1.463	.00002	-.00002	.00002	-8.983	-2.079	-1.048	
1.950	.00001	.00000	.00008	-8.496	-3.673	-2.459	

1

****** BOX CULVERT 2.0 X 2.0 ******** PAGE - 1

** BAISIC-DATA LIST **

```
NUMBER OF NODES                      4
NUMBER OF MEMBERS                    4
NUMBER OF BAISIC LOAD                1      CASES
NUMBER OF LOAD COMBINATION           0      CASES
ALLOWABLE STRESS (CONCRETE)       70.0      KG/CM**2
ALLOWABLE STRESS (STEEL)        1600.0      KG/CM**2
MAX-DAMMEN-RYOKU              CASE=  0 - CASE=  0
OUT PUT FORM                  F TYPE ( UNIT = MM, 1/1000RAD, TON, TON-M )
KANZAN KEISUU                 NODAL-DISPLASEMENT, NODAL-REACTION, MEMBER-DISPLASEMENT, MEMBER-FORCE
```

1

****** BOX CULVERT 2.0 X 2.0 ******** PAGE - 2

* NODE COODINATE *

NO	X	Y
1	.000	.000
2	2.400	.000
3	2.400	2.450
4	.000	2.450

1

** MEMBER-DATA LIST **

NO	NODE-NO L	NODE-NO R	CONDITION L	CONDITION R	DANMENSEKI A(M**2)	YOUNG MODULUS E(TON/M**2)	NIJI MOMENT I(M**4)	BUZAICHO L(M)	THETA (DEG)	T.E.COEF (M/M/DEG)	JIBAN HANRYOKU COEF KV(T/M**2)	JIBAN HANRYOKU COEF KS(T/M**2)
1	1	2	FIX	FIX	.400	.25000E+07	.53300E-02	2.400	.00	.0000E+00	.0	.0
2	2	3	FIX	FIX	.400	.25000E+07	.53300E-02	2.450	90.00	.0000E+00	.0	.0
3	3	4	FIX	FIX	.500	.25000E+07	.10400E-01	2.400	.00	.0000E+00	10000.0	.0
4	4	1	FIX	FIX	.400	.25000E+07	.53300E-02	2.450	-90.00	.0000E+00	.0	.0

1

** BAISIC LOAD DATA **

	CASE	TYPE	N	P1	(P3)	P2	L1	L2	ANGLE	HANTEI
	1									
+		DISTRIBU	1	7.080		7.080	.000	2.400	.00	0
+		DISTRIBU	2	3.040		5.370	.000	2.450	.00	0

+	DISTRIBU	3	9.910	9.910	.000	2.400	.00	0
+	DISTRIBU	4	5.370	3.040	.000	2.450	.00	0
+	DISTRIBU	2	.000	1.000	1.200	2.450	.00	0
+	DISTRIBU	4	1.000	.000	.000	1.250	.00	0
+	DISTRIBU	2	1.000	1.000	.000	2.450	90.00	0
+	DISTRIBU	4	1.000	1.000	.000	2.450	-90.00	0

1

** NODAL DISPLASEMENT **

CASE	N	DX	DY
1			
	1	.51613	-.00001
	2	.51612	-.00002
	3	.51613	-.00004
	4	.51614	-.00004

1

** LOAD CASE = 1 **

NO.	DIST	DEFORMATION	DEFLECTION	DEFL-ANGLE	AXIAL-FORCE	SHEAR-FORCE	MOMENT	REACTION

				N	S	M	
1 (1- 2)							
.000	.51613	-.00001	.00006	-4.619	8.499	-2.719	
.600	.51613	.00004	.00009	-4.619	4.251	1.106	
1.200	.51613	.00007	.00000	-4.619	.003	2.382	
1.800	.51613	.00004	-.00009	-4.619	-4.245	1.110	
2.400	.51612	-.00002	-.00006	-4.619	-8.493	-2.711	
2 (2- 3)							
.000	-.00002	-.51612	-.00006	-8.493	4.657	-2.713	
.613	-.00002	-.51614	.00001	-9.105	2.616	-.467	
1.225	-.00003	-.51613	.00000	-9.718	.219	.419	
1.838	-.00003	-.51614	-.00001	-10.330	-2.697	-.306	
2.450	-.00004	-.51613	.00006	-10.943	-6.270	-3.019	
3 (3- 4)							
.000	-.51613	.00004	.00006	-6.279	10.943	-3.019	.40160
.600	-.51613	.00008	.00007	-6.279	5.366	1.860	.83517
1.200	-.51613	.00010	.00000	-6.279	.006	3.465	1.04895
1.800	-.51614	.00008	-.00007	-6.279	-5.359	1.866	.81983
2.400	-.51614	.00004	-.00006	-6.279	-10.949	-3.012	.37010
4 (4- 1)							
.000	.00004	.51614	-.00006	-10.949	6.264	-3.010	
.613	.00003	.51613	.00001	-10.337	2.691	-.301	
1.225	.00002	.51613	.00000	-9.724	-.225	.420	
1.838	.00002	.51612	-.00001	-9.112	-2.622	-.470	
2.450	.00001	.51613	.00006	-8.499	-4.663	-2.719	

1

****** BOX CULVERT 1.5 X 1.5 X 2 ******** PAGE - 1

** BAISIC-DATA LIST **

NUMBER OF NODES	6	
NUMBER OF MEMBERS	7	
NUMBER OF BAISIC LOAD	1	CASES
NUMBER OF LOAD COMBINATION	0	CASES
ALLOWABLE STRESS (CONCRETE)	70.0	KG/CM**2
ALLOWABLE STRESS (STEEL)	1600.0	KG/CM**2
MAX-DAMMEN-RYOKU	CASE= 0 - CASE= 0	
OUT PUT FORM	F TYPE (UNIT = MM, 1/1000RAD, TON, TON-M)	
KANZAN KEISUU	NODAL-DISPLASEMENT, NODAL-REACTION, MEMBER-DISPLASEMENT, MEMBER-FORCE	

1

****** BOX CULVERT 1.5 X 1.5 X 2 ******** PAGE - 2

* NODE COODINATE *

NO	X	Y
1	.000	.000
2	1.850	.000
3	3.700	.000
4	.000	1.950
5	1.850	1.950
6	3.700	1.950

1

** MEMBER-DATA LIST **

NO	NODE-NO L	NODE-NO R	CONDITION L	CONDITION R	DANMENSEKI A(M**2)	YOUNG MODULUS E(TON/M**2)	NIJI MOMENT I(M**4)	BUZAICHO L(M)	THETA (DEG)	T.E.COEF (M/M/DEG)	JIBAN HANRYOKU COEF KV(T/M**2)	JIBAN HANRYOKU COEF KS(T/M**2)
1	1	2	FIX	FIX	.400	.25000E+07	.53300E-02	1.850	.00	.0000E+00	.0	.0
2	2	3	FIX	FIX	.400	.25000E+07	.53300E-02	1.850	.00	.0000E+00	.0	.0
3	1	4	FIX	FIX	.400	.25000E+07	.53300E-02	1.950	90.00	.0000E+00	.0	.0
4	2	5	FIX	FIX	.300	.25000E+07	.73000E-03	1.950	90.00	.0000E+00	.0	.0
5	3	6	FIX	FIX	.400	.25000E+07	.53300E-02	1.950	90.00	.0000E+00	.0	.0
6	4	5	FIX	FIX	.500	.25000E+07	.10420E-01	1.850	.00	.0000E+00	10000.0	.0
7	5	6	FIX	FIX	.500	.25000E+07	.10420E-01	1.850	.00	.0000E+00	10000.0	.0

1

** BAISIC LOAD DATA **

CASE	TYPE	N	P1	(P3)	P2	L1	L2	ANGLE	HANTEI

1

+	DISTRIBU	1	7.080	7.080	.000	1.850	.00	0
+	DISTRIBU	2	7.080	7.080	.000	1.850	.00	0
+	DISTRIBU	3	-3.040	-4.890	.000	1.950	.00	0
+	DISTRIBU	5	3.040	4.890	.000	1.950	.00	0
+	DISTRIBU	3	.000	-1.000	.950	1.950	.00	0
+	DISTRIBU	5	.000	1.000	.950	1.950	.00	0
+	DISTRIBU	3	1.000	1.000	.000	1.950	90.00	0
+	DISTRIBU	4	.750	.750	.000	1.950	90.00	0
+	DISTRIBU	5	1.000	1.000	.000	1.950	90.00	0
+	DISTRIBU	6	-8.530	-8.530	.000	1.850	.00	0
+	DISTRIBU	7	-8.530	-8.530	.000	1.850	.00	0

1

** NODAL DISPLASEMENT **

CASE	N	DX	DY
1			
	1	.00000	.00004

2	-.00001	.00003
3	-.00001	.00004
4	.00000	.00002
5	-.00001	.00000
6	-.00001	.00002

1

** LOAD CASE = 1 **

NO.	DIST	DEFORMATION	DEFLECTION	DEFL-ANGLE	AXIAL-FORCE	SHEAR-FORCE	MOMENT	REACTION
1	(1- 2)							
	.000	.00000	.00004	.00002	-3.330	5.988	-1.358	
	.463	.00000	.00005	.00003	-3.330	2.714	.654	
	.925	.00000	.00006	-.00001	-3.330	-.561	1.152	
	1.388	-.00001	.00005	-.00003	-3.330	-3.835	.135	
	1.850	-.00001	.00003	.00000	-3.330	-7.110	-2.396	
2	(2- 3)							
	.000	-.00001	.00003	.00000	-3.330	7.110	-2.396	
	.463	-.00001	.00005	.00003	-3.330	3.835	.135	
	.925	-.00001	.00006	.00001	-3.330	.561	1.152	
	1.388	-.00001	.00005	-.00003	-3.330	-2.714	.654	
	1.850	-.00001	.00004	-.00002	-3.330	-5.988	-1.358	
3	(1- 4)							
	.000	.00004	.00000	.00002	-5.988	-3.330	1.358	
	.488	.00004	.00000	.00000	-6.476	-1.736	.114	
	.975	.00003	.00000	.00000	-6.963	.085	-.298	
	1.463	.00003	.00000	.00001	-7.451	2.261	.255	

1.950	.00002	.00000	-.00003	-7.938	4.901	1.982	
4 (2- 5)							
.000	.00003	.00001	.00000	-14.220	.000	.000	
.488	.00003	.00001	.00000	-14.586	.000	.000	
.975	.00002	.00001	.00000	-14.951	.000	.000	
1.463	.00001	.00001	.00000	-15.317	.000	.000	
1.950	.00000	.00001	.00000	-15.682	.000	.000	
5 (3- 6)							
.000	.00004	.00001	-.00002	-5.988	3.330	-1.358	
.488	.00004	.00001	.00000	-6.476	1.736	-.114	
.975	.00003	.00001	.00000	-6.963	-.085	.298	
1.463	.00003	.00001	-.00001	-7.451	-2.261	-.255	
1.950	.00002	.00001	.00003	-7.938	-4.901	-1.982	
6 (4- 5)							
.000	.00000	.00002	-.00003	-4.901	-7.938	1.982	.24726
.463	.00000	.00001	-.00004	-4.901	-3.920	-.756	.06312
.925	.00000	-.00001	-.00002	-4.901	.019	-1.656	-.07149
1.388	-.00001	-.00001	.00001	-4.901	3.924	-.744	-.08251
1.850	-.00001	.00000	.00000	-4.901	7.841	1.976	-.04481
7 (5- 6)							
.000	-.00001	.00000	.00000	-4.901	-7.841	1.976	-.04481
.463	-.00001	-.00001	-.00001	-4.901	-3.924	-.744	-.08251
.925	-.00001	-.00001	.00002	-4.901	-.019	-1.656	-.07149
1.388	-.00001	.00001	.00004	-4.901	3.920	-.756	.06312
1.850	-.00001	.00002	.00003	-4.901	7.938	1.982	.24727

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1
            ****** BOX CULVERT 1.5 X 2.0 X 2 ********                    PAGE -  1

   ** BAISIC-DATA LIST **

        NUMBER OF NODES                        6

        NUMBER OF MEMBERS                      7

        NUMBER OF BAISIC LOAD                  1     CASES

        NUMBER OF LOAD COMBINATION             0     CASES

        ALLOWABLE STRESS (CONCRETE)         70.0     KG/CM**2

        ALLOWABLE STRESS (STEEL)          1600.0     KG/CM**2

        MAX-DAMMEN-RYOKU               CASE=  0 - CASE=  0

        OUT PUT FORM                   F TYPE ( UNIT = MM, 1/1000RAD, TON, TON-M )

        KANZAN KEISUU                  NODAL-DISPLASEMENT, NODAL-REACTION, MEMBER-DISPLASEMENT, MEMBER-FORCE
1
            ****** BOX CULVERT 1.5 X 2.0 X 2 ********                    PAGE -  2

  * NODE COODINATE *
```

NO	X	Y
1	.000	.000
2	2.350	.000
3	4.700	.000
4	.000	1.950
5	2.350	1.950
6	4.700	1.950

1

** MEMBER-DATA LIST **

NO	NODE-NO L	NODE-NO R	CONDITION L	CONDITION R	DANMENSEKI A(M**2)	YOUNG MODULUS E(TON/M**2)	NIJI MOMENT I(M**4)	BUZAICHO L(M)	THETA (DEG)	T.E.COEF (M/M/DEG)	JIBAN HANRYOKU COEF KV(T/M**2)	KS(T/M**2)
1	1	2	FIX	FIX	.400	.25000E+07	.53300E-02	2.350	.00	.0000E+00	.0	.0
2	2	3	FIX	FIX	.400	.25000E+07	.53300E-02	2.350	.00	.0000E+00	.0	.0
3	1	4	FIX	FIX	.400	.25000E+07	.53300E-02	1.950	90.00	.0000E+00	.0	.0
4	2	5	FIX	FIX	.300	.25000E+07	.73000E-03	1.950	90.00	.0000E+00	.0	.0
5	3	6	FIX	FIX	.400	.25000E+07	.53300E-02	1.950	90.00	.0000E+00	.0	.0
6	4	5	FIX	FIX	.500	.25000E+07	.10420E-01	2.350	.00	.0000E+00	10000.0	.0
7	5	6	FIX	FIX	.500	.25000E+07	.10420E-01	2.350	.00	.0000E+00	10000.0	.0

1

** BAISIC LOAD DATA **

CASE	TYPE	N	P1	(P3)	P2	L1	L2	ANGLE	HANTEI

1

+	DISTRIBU	1	7.080	7.080	.000	2.350	.00	0
+	DISTRIBU	2	7.080	7.080	.000	2.350	.00	0
+	DISTRIBU	3	-3.040	-4.890	.000	1.950	.00	0
+	DISTRIBU	5	3.040	4.890	.000	1.950	.00	0
+	DISTRIBU	3	.000	-1.000	.950	1.950	.00	0
+	DISTRIBU	5	.000	1.000	.950	1.950	.00	0
+	DISTRIBU	3	1.000	1.000	.000	1.950	90.00	0
+	DISTRIBU	4	.750	.750	.000	1.950	90.00	0
+	DISTRIBU	5	1.000	1.000	.000	1.950	90.00	0
+	DISTRIBU	6	-8.220	-8.220	.000	2.350	.00	0
+	DISTRIBU	7	-8.220	-8.220	.000	2.350	.00	0

1

** NODAL DISPLASEMENT **

CASE	N	DX	DY
1			
	1	.00001	.00007

2	.00000	.00005
3	-.00001	.00007
4	.00001	.00005
5	.00000	.00000
6	-.00001	.00005

1

** LOAD CASE = 1 **

NO.	DIST	DEFORMATION	DEFLECTION	DEFL-ANGLE	AXIAL-FORCE	SHEAR-FORCE	MOMENT	REACTION
1	(1- 2)							
	.000	.00001	.00007	.00005	-3.309	7.313	-1.783	
	.587	.00001	.00011	.00005	-3.309	3.154	1.292	
	1.175	.00001	.00012	-.00003	-3.309	-1.006	1.922	
	1.762	.00000	.00008	-.00008	-3.309	-5.165	.110	
	2.350	.00000	.00005	.00000	-3.309	-9.325	-4.147	
2	(2- 3)							
	.000	.00000	.00005	.00000	-3.309	9.325	-4.147	
	.587	.00000	.00008	.00008	-3.309	5.165	.110	
	1.175	.00000	.00012	.00003	-3.309	1.006	1.922	
	1.762	.00000	.00011	-.00005	-3.309	-3.154	1.292	
	2.350	-.00001	.00007	-.00005	-3.309	-7.313	-1.783	
3	(1- 4)							
	.000	.00007	-.00001	.00005	-7.313	-3.309	1.783	
	.488	.00007	.00000	.00001	-7.801	-1.714	.550	
	.975	.00006	.00001	.00000	-8.288	.107	.149	
	1.463	.00006	.00000	-.00001	-8.776	2.283	.712	

1.950	.00005	-.00001	-.00007	-9.263	4.923	2.450	
4 (2- 5)							
.000	.00005	.00000	.00000	-18.650	.000	.000	
.488	.00004	.00000	.00000	-19.015	.000	.000	
.975	.00002	.00000	.00000	-19.381	.000	.000	
1.463	.00001	.00000	.00000	-19.747	.000	.000	
1.950	.00000	.00000	.00000	-20.112	.000	.000	
5 (3- 6)							
.000	.00007	.00001	-.00005	-7.313	3.309	-1.783	
.488	.00007	-.00001	-.00001	-7.801	1.714	-.550	
.975	.00006	-.00001	.00000	-8.288	-.107	-.149	
1.463	.00006	-.00001	.00001	-8.776	-2.283	-.712	
1.950	.00005	.00001	.00007	-9.263	-4.923	-2.450	
6 (4- 5)							
.000	.00001	.00005	-.00007	-4.923	-9.263	2.450	.53600
.587	.00001	.00001	-.00007	-4.923	-4.247	-1.506	.09499
1.175	.00001	-.00002	-.00002	-4.923	.542	-2.586	-.17825
1.762	.00000	-.00001	.00003	-4.923	5.265	-.881	-.13954
2.350	.00000	.00000	.00000	-4.923	10.056	3.616	-.01487
7 (5- 6)							
.000	.00000	.00000	.00000	-4.923	-10.056	3.616	-.01487
.587	.00000	-.00001	-.00003	-4.923	-5.265	-.881	-.13954
1.175	.00000	-.00002	.00002	-4.923	-.542	-2.586	-.17825
1.762	.00000	.00001	.00007	-4.923	4.247	-1.506	.09499
2.350	-.00001	.00005	.00007	-4.923	9.263	2.450	.53601

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1
                 ****** BOX CULVERT 2.0 X 2.0 X 2 ********                          PAGE -  1

      ** BAISIC-DATA LIST **

            NUMBER OF NODES                      6

            NUMBER OF MEMBERS                    7

            NUMBER OF BAISIC LOAD                1     CASES

            NUMBER OF LOAD COMBINATION           0     CASES

            ALLOWABLE STRESS (CONCRETE)       70.0     KG/CM**2

            ALLOWABLE STRESS (STEEL)        1600.0     KG/CM**2

            MAX-DAMMEN-RYOKU              CASE=  0 - CASE=  0

            OUT PUT FORM                  F TYPE ( UNIT = MM, 1/1000RAD, TON, TON-M )

            KANZAN KEISUU                 NODAL-DISPLASEMENT, NODAL-REACTION, MEMBER-DISPLASEMENT, MEMBER-FORCE
1
                 ****** BOX CULVERT 2.0 X 2.0 X 2 ********                          PAGE -  2

    * NODE COODINATE *
```

NO	X	Y
1	.000	.000
2	2.350	.000
3	4.700	.000
4	.000	2.450
5	2.350	2.450
6	4.700	2.450

1

** MEMBER-DATA LIST **

NO	NODE-NO L	NODE-NO R	CONDITION L	CONDITION R	DANMENSEKI A(M**2)	YOUNG MODULUS E(TON/M**2)	NIJI MOMENT I(M**4)	BUZAICHO L(M)	THETA (DEG)	T.E.COEF (M/M/DEG)	JIBAN HANRYOKU COEF KV(T/M**2)	KS(T/M**2)
1	1	2	FIX	FIX	.400	.25000E+07	.53300E-02	2.350	.00	.0000E+00	.0	.0
2	2	3	FIX	FIX	.400	.25000E+07	.53300E-02	2.350	.00	.0000E+00	.0	.0
3	1	4	FIX	FIX	.400	.25000E+07	.53300E-02	2.450	90.00	.0000E+00	.0	.0
4	2	5	FIX	FIX	.300	.25000E+07	.73000E-03	2.450	90.00	.0000E+00	.0	.0
5	3	6	FIX	FIX	.400	.25000E+07	.53300E-02	2.450	90.00	.0000E+00	.0	.0
6	4	5	FIX	FIX	.500	.25000E+07	.10420E-01	2.350	.00	.0000E+00	10000.0	.0
7	5	6	FIX	FIX	.500	.25000E+07	.10420E-01	2.350	.00	.0000E+00	10000.0	.0

1

** BAISIC LOAD DATA **

CASE	TYPE	N	P1	(P3)	P2	L1	L2	ANGLE	HANTEI

1

+	DISTRIBU	1	7.080	7.080	.000	2.350	.00	0
+	DISTRIBU	2	7.080	7.080	.000	2.350	.00	0
+	DISTRIBU	3	-3.040	-5.370	.000	2.450	.00	0
+	DISTRIBU	5	3.040	5.370	.000	2.450	.00	0
+	DISTRIBU	3	.000	-1.250	1.200	2.450	.00	0
+	DISTRIBU	5	.000	1.250	1.200	2.450	.00	0
+	DISTRIBU	3	1.000	1.000	.000	2.450	90.00	0
+	DISTRIBU	4	.750	.750	.000	2.450	90.00	0
+	DISTRIBU	5	1.000	1.000	.000	2.450	90.00	0
+	DISTRIBU	6	-8.520	-8.520	.000	2.350	.00	0
+	DISTRIBU	7	-8.520	-8.520	.000	2.350	.00	0

1

** NODAL DISPLASEMENT **

CASE	N	DX	DY
1			
	1	.00002	.00008

2	.00001	.00006
3	.00000	.00008
4	.00002	.00005
5	.00001	-.00001
6	.00000	.00005

1

****** BOX CULVERT 2.0 X 2.0 X 2 ********

** LOAD CASE = 1 **

NO.	DIST	DEFORMATION	DEFLECTION	DEFL-ANGLE	AXIAL-FORCE	SHEAR-FORCE	MOMENT	REACTION
1	(1- 2)							
	.000	.00002	.00008	.00003	-4.407	7.592	-2.213	
	.587	.00002	.00011	.00005	-4.407	3.433	1.025	
	1.175	.00002	.00012	-.00002	-4.407	-.727	1.820	
	1.762	.00001	.00009	-.00007	-4.407	-4.886	.171	
	2.350	.00001	.00006	.00000	-4.407	-9.046	-3.921	
2	(2- 3)							
	.000	.00001	.00006	.00000	-4.407	9.046	-3.921	
	.587	.00001	.00009	.00007	-4.407	4.886	.171	
	1.175	.00001	.00012	.00002	-4.407	.727	1.820	
	1.762	.00000	.00011	-.00005	-4.407	-3.433	1.025	
	2.350	.00000	.00008	-.00003	-4.407	-7.592	-2.213	
3	(1- 4)							
	.000	.00008	-.00002	.00003	-7.592	-4.407	2.213	
	.613	.00007	-.00002	-.00002	-8.205	-2.367	.120	
	1.225	.00007	-.00003	.00000	-8.817	.031	-.613	
	1.838	.00006	-.00002	.00002	-9.430	2.987	.273	

2.450	.00005	-.00002	-.00005	-10.042	6.676	3.195	
4 (2- 5)							
.000	.00006	-.00001	.00000	-18.092	.000	.000	
.613	.00004	-.00001	.00000	-18.551	.000	.000	
1.225	.00003	-.00001	.00000	-19.011	.000	.000	
1.838	.00001	-.00001	.00000	-19.470	.000	.000	
2.450	-.00001	-.00001	.00000	-19.929	.000	.000	
5 (3- 6)							
.000	.00008	.00000	-.00003	-7.592	4.407	-2.213	
.613	.00007	.00000	.00002	-8.205	2.367	-.120	
1.225	.00007	.00001	.00000	-8.817	-.031	.613	
1.838	.00006	.00000	-.00002	-9.430	-2.987	-.273	
2.450	.00005	.00000	.00005	-10.042	-6.676	-3.195	
6 (4- 5)							
.000	.00002	.00005	-.00005	-6.676	-10.042	3.195	.53689
.587	.00002	.00001	-.00007	-6.676	-4.840	-1.164	.11803
1.175	.00002	-.00002	-.00002	-6.676	.134	-2.537	-.17958
1.762	.00001	-.00002	.00002	-6.676	5.022	-1.023	-.17282
2.350	.00001	-.00001	.00000	-6.676	9.965	3.376	-.06235
7 (5- 6)							
.000	.00001	-.00001	.00000	-6.676	-9.965	3.376	-.06235
.587	.00001	-.00002	-.00002	-6.676	-5.022	-1.023	-.17282
1.175	.00000	-.00002	.00002	-6.676	-.134	-2.537	-.17958
1.762	.00000	.00001	.00007	-6.676	4.840	-1.164	.11803
2.350	.00000	.00005	.00005	-6.676	10.042	3.195	.53690

Table 2.2.6 (1/2) STRESS CALCULATION OF SINGLE BOX CULVERTS

Type		Box 1.5 x 1.5 x 1				Box 1.5 x 2.0 x 1				Box 2.0 x 2.0 x 1			
		Top Slab		Side Wall		Top Slab		Side Wall		Top Slab		Side Wall	
N	t	3.56		8.68		3.67		8.68		4.62		10.95	
M	tm	1.68		1.85		2.46		1.85		2.72		3.01	
Q	t	6.73		4.67		8.50		4.67		8.49		6.26	
d	cm	30.00		30.00		30.00		30.00		30.00		30.00	
d'	cm	10.00		10.00		10.00		10.00		10.00		10.00	
u	cm	10.00		1.00		10.00		1.00		10.00		10.00	
b	cm	100.00		100.00		100.00		100.00		100.00		100.00	
r	A's/As	1.00		1.00		1.00		0.00		1.00		1.00	
n		15.00		15.00		15.00		15.00		15.00		15.00	
Sca	kg/cm2	70.00		70.00		70.00		70.00		70.00		70.00	
Ssa	kg/cm2	1,600.00		1,600.00		1,600.00		1,600.00		1,600.00		1,600.00	
Tca	kg/cm2	4.25		4.25		4.25		4.25		4.25		4.25	
M'		2.03		1.94		2.83		1.94		3.18		4.10	
C		31.02	31.02	32.55	32.55	22.29	22.29	32.55	32.55	19.81	19.81	15.35	15.35
k (C)		0.07	-0.001	0.06	0.001	0.11	0.001	0.06	0.000	0.12	0.001	0.14	0.000
np (C)		0.0001	0.0010	0.0000	-0.0006	0.0040	0.0032	-0.0007	-0.0007	0.0044	0.0038	0.0021	0.0025
S		47.26	47.26	49.59	49.59	33.97	33.97	49.59	49.59	30.18	30.18	23.39	23.39
k (S)		0.20	0.000	0.19	0.006	0.23	0.000	0.19	0.000	0.24	0.000	0.27	0.000
np (S)		0.0113	0.0110	0.0000	-0.0055	0.0196	0.0196	-0.0056	-0.0056	0.0208	0.0209	0.0128	0.0124
d'/d		0.33		0.33		0.33		0.33		0.33		0.33	
f	cm	57.02		22.31		76.95		22.31		68.87		37.49	
f/d		1.90		0.74		2.56		0.74		2.30		1.25	
np		0.0113		0.0000		0.0196		-0.0007		0.0208		0.0128	
r-As		2.27		0.00		3.91		-0.13		4.16		2.57	
r-As'		2.27		0.00		3.91		0.00		4.16		2.57	
Re-Bar Arrangement													
As1	Diameter	D13		D13		D16		D16		D16		D16	
	Area (cm2)	1.27		1.27		1.99		1.99		1.99		1.99	
	Pitch (cm)	30.00		30.00		30.00		30.00		30.00		30.00	
	As1	4.22		4.22		6.62		6.62		6.62		6.62	
As2	Diameter	0.00		0.00		0.00		0.00		0.00		0.00	
	Area (cm2)	0.00		0.00		0.00		0.00		0.00		0.00	
	Pitch (cm)	30.00		30.00		30.00		30.00		30.00		30.00	
	As2 (cm2)	0.00		0.00		0.00		0.00		0.00		0.00	
Total of As		4.22	(OK)	4.22	(OK)	6.62	(OK)	6.62	(OK)	6.62	(OK)	6.62	(OK)
		-2.27		-0.00		-3.91		0.13		-4.16		-2.57	
As'1	Diameter	D13		D13		D16		D16		D16		D16	
	Area (cm2)	1.27		1.27		1.99		1.99		1.99		1.99	
	Pitch (cm)	30.00		30.00		25.00		25.00		30.00		30.00	
	As'1 (cm2)	4.22		4.22		7.94		7.94		6.62		6.62	
As'2	Diameter	0.00		0.00		0.00		0.00		0.00		0.00	
	Area (cm2)	0.00		0.00		0.00		0.00		0.00		0.00	
	Pitch (cm)	25.00		25.00		25.00		25.00		25.00		25.00	
	As'2 (cm2)	0.00		0.00		0.00		0.00		0.00		0.00	
Total of As'		4.22	(OK)	4.22	(OK)	7.94	(OK)	7.94	(OK)	6.62	(OK)	6.62	(OK)
		-2.27		-0.00		-3.91		0.00		-4.16		-2.57	
Check	np	0.02112		0.02112		0.03310		0.03310		0.03310		0.03310	
	k	0.25	0.000	0.79	0.000	0.28	0.000	0.80	0.000	0.29	0.000	0.37	0.000
	A's/As	1.00		1.00		1.20		1.20		1.00		1.00	
	C	8.91		3.33		8.19		3.24		7.96		6.09	
	S	26.08		0.87		21.04		0.81		19.91		10.39	
	Z	1.01	0.20	0.25	0.41	1.04	0.24	0.35	0.42	1.04	0.24	1.00	0.26
	Sc	20.12	OK	7.17	OK	25.73	OK	6.97	OK	28.15	OK	27.78	OK
	Ss	883.01	OK	28.05	OK	991.23	OK	26.12	OK	1,055.74	OK	711.00	OK
	Tc	2.24	OK	1.56	OK	2.83	OK	1.56	OK	2.83	OK	2.09	OK
	x	7.64		23.80		8.41		24.00		8.57		11.09	

Table 2.2.6 (2/2) STRESS CALCULATION OF DOUBLE BOX CULVERTS

Type		Box 1.5 x 1.5 x 2				Box 1.5 x 2.0 x 2				Box 2.0 x 2.0 x 2			
		Top Slab		Side Wall		Top Slab		Side Wall		Top Slab		Top Slab	
N	t	3.33		5.99		3.31		7.31		4.41		0.00	
M	tm	2.40		1.36		4.15		1.78		3.92		2.72	
Q	t	7.11		3.33		9.33		3.31		9.05		8.50	
d	cm	29.00		29.00		30.00		30.00		30.00		30.00	
d'	cm	10.00		10.00		10.00		10.00		10.00		10.00	
u	cm	9.50		9.50		10.00		10.00		10.00		10.00	
b	cm	100.00		100.00		100.00		100.00		100.00		100.00	
r	A's/As	1.00		1.00		1.00		1.00		1.00		1.00	
n		15.00		15.00		15.00		15.00		15.00		15.00	
Sca	kg/cm2	70.00		70.00		70.00		70.00		70.00		70.00	
Ssa	kg/cm2	1,600.00		1,600.00		1,600.00		1,600.00		1,600.00		1,600.00	
Tca	kg/cm2	4.25		4.25		4.25		4.25		4.25		4.25	
M'		2.71		1.93		4.48		2.51		4.36		2.72	
C		21.70	21.70	30.55	30.55	14.07	14.07	25.06	25.06	14.44	14.44	23.17	28.41
k (C)		0.11	0.000	0.07	0.000	0.17	0.000	0.09	0.001	0.16	0.000	0.09	0.001
np (C)		0.0033	0.0034	0.0000	0.0003	0.0115	0.0113	0.0012	0.0006	0.0090	0.0093	0.0043	0.0034
S		33.07	33.07	46.56	46.56	21.44	21.44	38.18	38.18	22.01	22.01	35.31	35.31
k (S)		0.21	-0.062	0.19	0.000	0.28	0.000	0.21	0.000	0.27	0.000	0.23	0.000
np (S)		-0.0461	0.0158	0.0038	0.0034	0.0404	0.0403	0.0049	0.0051	0.0354	0.0355	0.0292	0.0294
d'/d		0.34		0.34		0.33		0.33		0.33		0.33	
f	cm	81.45		32.18		135.32		34.38		98.97		9,999.99	
f/d		2.81		1.11		4.51		1.15		3.30		333.33	
np		0.0033		0.0038		0.0404		0.0049		0.0354		0.0292	
r-As		0.64		0.74		8.07		0.98		7.09		5.84	
r-As'		0.64		0.74		8.07		0.98		7.09		5.84	
Re-Bar Arrangement													
As1	Diameter	D16		D16		D19		D19		D19		D19	
	Area (cm2)	1.99		1.99		2.87		2.87		2.87		2.87	
	Pitch (cm)	30.00		30.00		30.00		30.00		30.00		30.00	
	As1	6.62		6.62		9.55		9.55		9.55		9.55	
As2	Diameter	0.00		0.00		0.00		0.00		0.00		0.00	
	Area (cm2)	0.00		0.00		0.00		0.00		0.00		0.00	
	Pitch (cm)	30.00		30.00		30.00		30.00		30.00		30.00	
	As2 (cm2)	0.00		0.00		0.00		0.00		0.00		0.00	
Total of As		6.62	(OK)	6.62	(OK)	9.55	(OK)	9.55	(OK)	9.55	(OK)	9.55	(OK)
		-0.64		-0.74		-8.07		-0.98		-7.09		-5.84	
As'1	Diameter	D16		D16		D19		D19		D19		D19	
	Area (cm2)	1.99		1.99		2.87		2.87		2.87		2.87	
	Pitch (cm)	30.00		30.00		30.00		30.00		30.00		30.00	
	As'1 (cm2)	6.62		6.62		9.55		9.55		9.55		9.55	
As'2	Diameter	0.00		0.00		0.00		0.00		0.00		0.00	
	Area (cm2)	0.00		0.00		0.00		0.00		0.00		0.00	
	Pitch (cm)	25.00		25.00		25.00		25.00		25.00		25.00	
	As'2 (cm2)	0.00		0.00		0.00		0.00		0.00		0.00	
Total of As'		6.62	(OK)	6.62	(OK)	9.55	(OK)	9.55	(OK)	9.55	(OK)	9.55	(OK)
		-0.64		-0.74		-8.07		-0.98		-7.09		-5.84	
Check	np	0.03424		0.03424		0.04775		0.04775		0.04775		0.04775	
	k	0.28	0.000	0.41	0.000	0.30	0.000	0.44	0.001	0.31	0.001	0.27	0.000
	A's/As	1.00		1.00		1.00		1.00		1.00		1.00	
	C	8.24		5.50		7.72		5.10		7.41		8.50	
	S	21.27		7.83		18.34		6.47		16.75		22.50	
	Z	1.04	0.24	0.96	0.27	1.08	0.27	1.00	0.30	1.07	0.28	1.08	0.27
	Sc	26.58	OK	12.61	OK	38.42	OK	14.26	OK	35.91	OK	25.67	OK
	Ss	1,028.86	OK	269.06	OK	1,369.01	OK	271.08	OK	1,217.36	OK	1,019.51	OK
	Tc	2.45	OK	1.15	OK	3.11	OK	1.10	OK	3.02	OK	2.83	OK
	x	8.10		11.97		8.89		13.23		9.20		8.22	

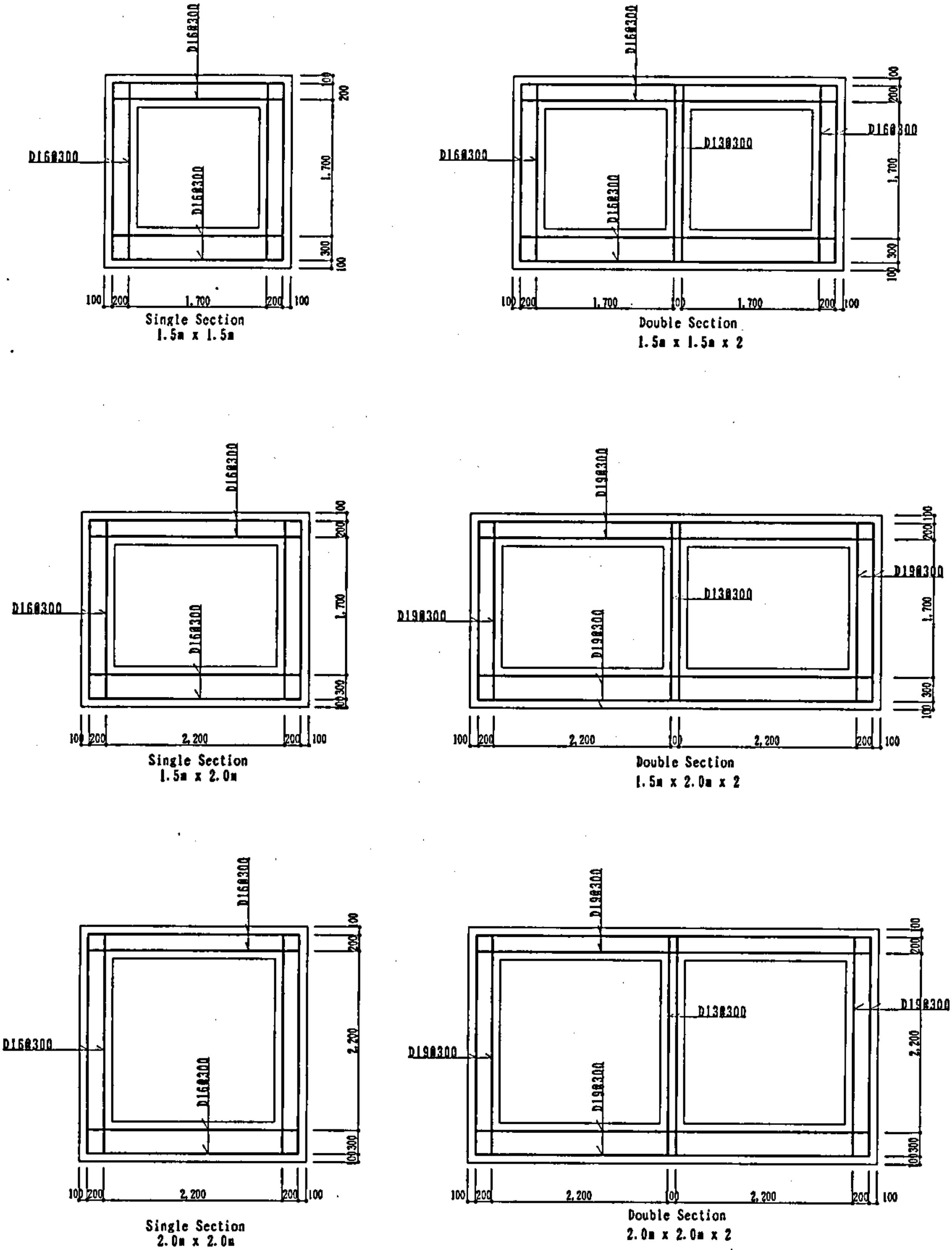


Fig. 2.2.8 Reinforcement Arrangement for Box Culvert

Steel Slide Gates for Sluice

(1) Requirements and Design Loads

Since the ground elevations of the basin of the SL 2, SL 3 and SL 4 are lower than design high water level, sluice gates are installed to prevent the counter flow. A spindle type steel slide gate and manual operation system are applied to the gate.

The design water level for main beams and skin plate is set at considering the time lag of flood concentration. The design water levels are:

River side : Design high water level
Land side : Sluice bed

For the design of opration equipment such as spindols, the diferance between river side and land side is set at 1.0 m.

(2) Basic Dimensions

Following types of steel slide gates are emproyed in the project.

	Type 1	Type 2
Clear Span (W)	1.5 m	2.0 m
Clear Height (H)	1.5 m	1.5 m

The gate width and height shall be calculated as follows:

Gate height $H + \Delta H$
Gate width $H + 2\ \Delta H$

where, DH : $\Sigma P \leq 5\ t$: $\Delta t = 5.0$ cm
$\Sigma P > 5\ t$: $\Delta t = 7.5$ cm

Two (2) middle main beams shall be provided besides main beams provided at the bottom and top gate. The layout of the main beams shall determined to load the same waterpresser at the each mainbeams.

The design loads and other requirements for design of gates are summarized in Fig 2.2.9.

Flap Gate

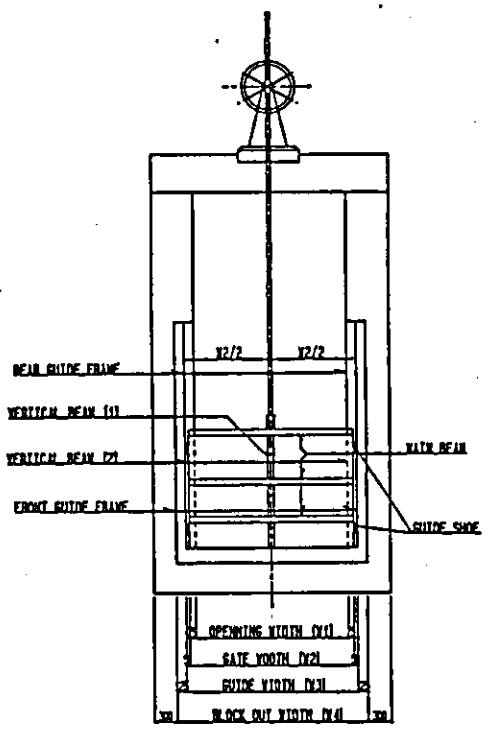
A flap gate shall be installed at the outlet of the pipe culvert SL 1 to prevent the counterflow from the river to land side. The design conditions are:

1. design water level is:
   River side : Design high water level (EL. 7.77)
   Land side : Sluice bed (EL. 4.00)

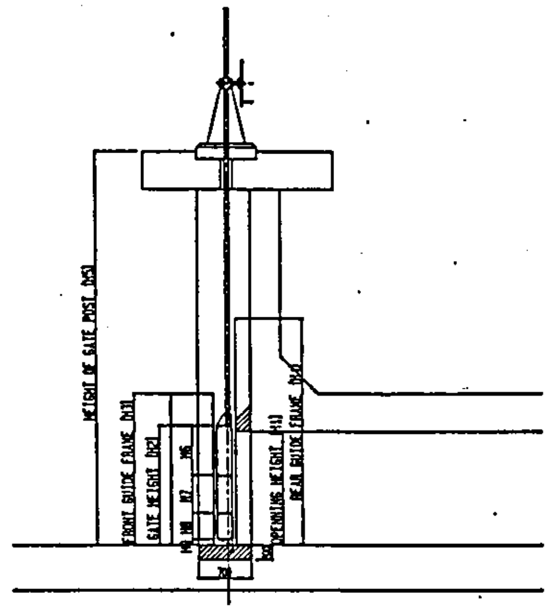
2. openning water pressure is 1/3 of gate clear diameter (20cm).

3. material of gate leaf shall be fablic casting iron.

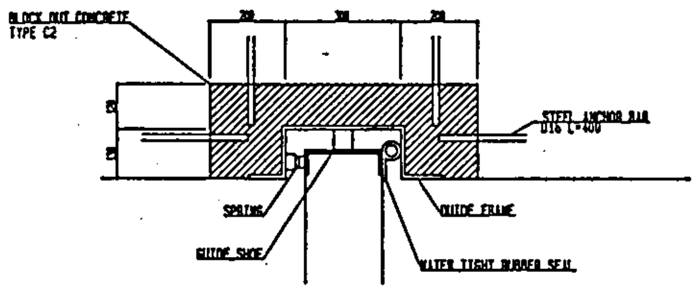
The detail of flap gate is shown in Fig. 2.2.10.



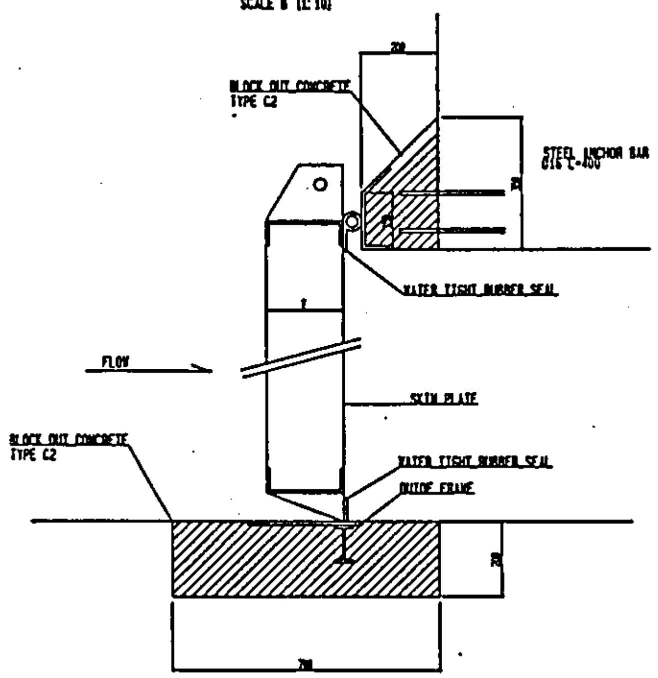
FRONT VIEW
SCALE A (1:50)



SECTION
SCALE A (1:50)



SIDE GUIDE
SCALE B (1:10)



BOTTOM AND UPPER GUIDE
SCALE B (1:10)

DIMENSION OF GATES

(UNIT meter)

		DRAINAGE OUTLET			IRRIGATION INTAKE	
		SL 2	SL 3	SL4	RIGHT	LEFT
WIDTH	W1	2.000	1.500	2.000	1.000	1.250
	W2	2.150	1.650	2.150	1.100	1.350
	W3	2.260	1.760	2.260	1.260	1.510
	W4	2.500	2.000	2.500	1.500	1.750
HEIGHT	H1	1.500	1.500	1.500	1.000	1.000
	H2	1.575	1.575	1.575	1.050	1.050
	H3	2.000	2.000	2.000	1.500	1.500
	H4	3.000	3.000	3.000	2.000	2.000
	H5	4.650	4.650	4.650	6.170	6.170
	H6	0.630	0.630	0.630	0.568	0.568
	H7	0.508	0.508	0.508	0.000	0.000
	H8	0.508	0.508	0.508	0.432	0.432
	H9	0.750	0.750	0.750	0.500	0.500

DESIGN REQUIREMENTS

(UNIT meter)

	DRAINAGE OUTLET			IRRIGATION INTAKE	
	SL 2	SL 3	SL4	RIGHT	LEFT
TYPE OF GATE	FABRICATED STEEL SLIDE GATE				
CLEAR SPAN	2.000	1.500	2.000	1.250	1.000
CLEAR HEIGHT	1.500	1.500	1.500	1.000	1.000
DESIGN WATER DEPTH					
RIVER SIDE	3.625	3.625	3.625	3.720	3.720
LAND SIDE	0.000	0.000	0.000	0.000	0.000
OPERATING WATER DEPTH					
RIVER SIDE	1.575	1.575	1.575	1.050	1.050
LAND SIDE	0.575	0.575	0.575	0.050	0.050
WATER SEAL	RUBBER SEALS ON ALL SIDE (LAND SIDE)				
TYPE OF HOIST	MANUAL DRIVEN SPINDLE				
OPERATING SPEED	MORE THAN 20 cm/min.				
STRUCTURAL STEEL					
MAIN BODY	SS41				
MINIMUM THICKNESS					
PLATE	0.006				
BEAM	0.005				
SKIN PLATE	0.008				
ALLOWANCE FOR CORROSION					
ONE SIDE FACING WATER	0.0005				
BOTH SIDES FACING WATER	0.001				
SLENDERNESS RATIO					
COMPRESSION MEMBERS	120				
TENSION MEMBERS	200				
DEFLECTION OF MAIN BEAM	W2 / 800				

SCALES SCALE B (1:10) SCALE A (1:50)

NO.	DATE	REVISIONS	ORIGINATED	APPROVED

REPUBLIC OF INDONESIA
MINISTRY OF PUBLIC WORKS
DIRECTORATE GENERAL OF WATER RESOURCES DEVELOPMENT
DIRECTOTATE OF TECHNICAL GUIDANCE

MEDAN FLOOD CONTROL PROJECT
IRRIGATION FACILITIES
STEEL SLIDE GATE
LIST OF DIMENSIONS & DESIGN REQUIREMENT

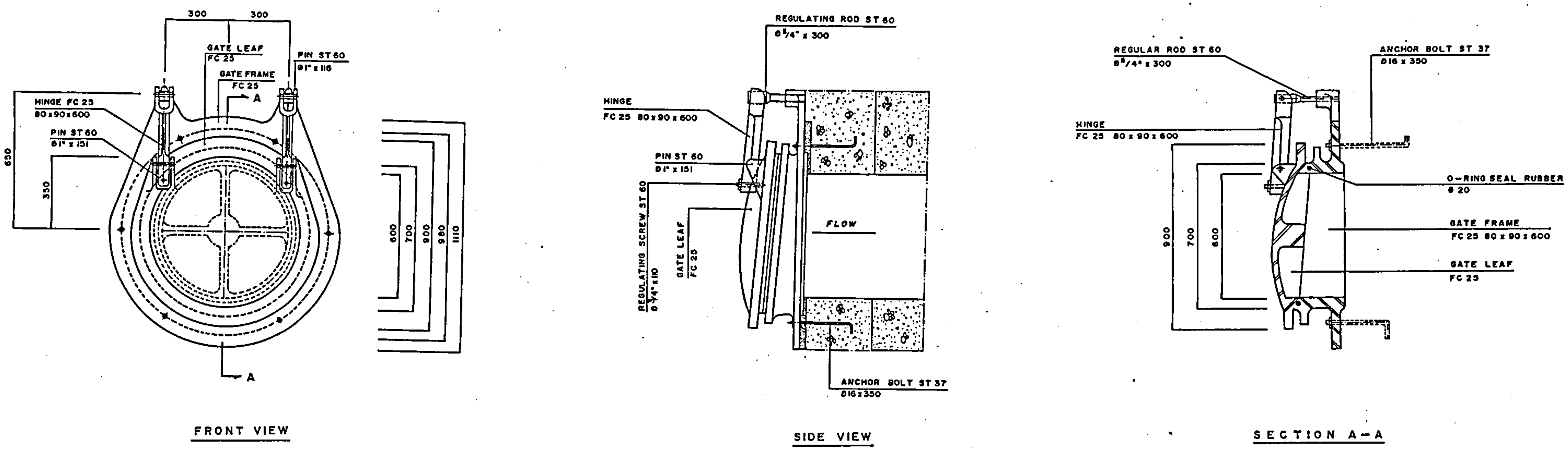
PREPARED	CHECKED	SUBMITTED	CERTIFIED	APPROVED	DRAWING NO.
					RD - 027

JAPAN INTERNATIONAL COOPERATION AGENCY
CTI ENGINEERING CO., LTD

PROYEK PENGELOLAAN SUMBER AIR DAN PENGENDALIAN BANJIR SUMATERA UTARA

SHEET NO. 297 REV.

Fig. 2.2.9 STEEL SLIDE GATE FOR DRAINAGE SLUICE



DETAIL OF FLAPE GATE

SCALE. D

Qty.	Part no.	Description	Material	Dimension	Weight	Remarks
4	9	ANCHOR BOLT	ST 37	Ø16 x 350	4	Galvanized
1	8	O-RING SEAL	RUBBER	Ø 20mm		
2	7	REGULATING SCREW	ST 60	Ø3/4" x 110	0.6	Galvanized
2	6	PIN	ST 60	Ø1" x 116	1.1	Galvanized
2	5	PIN	ST 60	Ø1" x 151	1.2	Galvanized
2	4	REGULATING ROD	ST 60	Ø1 1/4" x 300	22	Galvanized
2	3	HINGE	FC 25	80 x 90 x 600	45	
1	2	GATE FRAME	FC 25	-	-	See Table
1	1	GATE LEAF	FC 25	-	-	See Table

SCALE:

0 0.2 0.4 0.6 0.8 1 m

SCALE. D (1:10)

NO.	DATE	REVISIONS	ORIGINATED	APPROVED

REPUBLIC OF INDONESIA

MINISTRY OF PUBLIC WORKS
DIRECTORATE GENERAL OF WATER RESOURCES DEVELOPMENT
DIRECTORATE OF TECHNICAL GUIDANCE

MEDAN FLOOD CONTROL PROJECT

DRAINAGE OUTLET
SL1
DETAIL OF FLAP GATE

PREPARED	CHECKED	SUBMITTED	CERTIFIED	APPROVED	DRAWING NO.
					DO - 011

JAPAN INTERNATIONAL COOPERATION AGENCY
CTI ENGINEERING CO., LTD

PROYEK PENGELOLAAN SUMBER AIR DAN PENGENDALIAN BANJIR .... UTARA

SHEET NO.	REV.
235	

Fig. 2.2.10 FLAP GATE FOR DRAINAGE OUTLET

2.3 Bandar Sidoras Intake Weir

The existing Bandar Sidoras Weir forms an obstruction during flood time and causes substantial inundation upstream of the weir. This weir, therefore, need to be reconstructed in line with the river improvement plan.

2.3.1 Hydraulic Requirements

Features and Functions of Existing Weir

The features and functions of existing intake facilities and the future plan of irrigation in this area are;

(1) Intake Weir:

- Type of weir : Concrete-made ogee weir
- Crest elevation : EL. 3.934 m
- Length of overflow section : 28.0 m (14.0 m x 2)
- Weir height : about 5.8 m
- Scouring sluices : B=2.0 m at the both sides of weir

(2) Irrigation Channel

- Irrigation channel left bank : Earth channel, B=2.95m, H=0.72m
  right bank : Earth channel, B=3.30m, H=0.83m
- Water level at outlets of box culvert
  left bank : EL. 3.4 m
  right bank : EL. 3.8 m
- Existing irrigation system
  Irrigation area left bank : 1,364 ha (to be expanded to 1,680 ha)
  right bank : 2,093 ha
  Required water left bank : 1.36 m^3/s (to be increased to 1.68 m^3/s)
  right bank : 2.09 m^3/s

(3) Function and Operation Method

- Under the normal flow conditions, the weir maintains the water level by 30 cm above the weir crest and provides the required irrigation water to the downstream irrigation area.
- The opening of the control gates at the inlet of the box culverts is controlled keeping the required water level of irrigation channel.

- If floods occurs, the scouring sluices are opened to release the excess water into the downstream so that the water level at the weir be lowered to the regulated level for irrigation.
- The scouring sluices are periodically opened to flush sediment which settled in front of intake gates.

<u>**Reconstruction Plan**</u>

(1) Type of Weir

The requirements of the new weir to be followed are:

- To have sufficient flow capacity for the design flood discharge; and
- To maintain the existing function as irrigation facility as well as to meet the feature requirement of the irrigation plan.

To fulfill these requirements, construction of movable weir or fixed weir with large capacity sluice gates are conceivable. Judging from the large design flood discharge of 320 m^3/s, movable type of weir is essential.

The following three types of gates are generally employed for the river weir which has a purpose of controlling upstream water level for irrigation:

- Inflatable rubber gate
- Steel roller gate
- Steel tilting gate

To select the suitable type of gate, a comparative study focusing on operation, maintenance and construction cost has been made and the inflatable rubber gate is proposed for the following reasons:

- Construction cost and maintenance cost are low.
- Operating devices are simple and easy to handle and further, automatic operation for gate deflation is performed without any external power source.
- Only a short time is required for mere periodical maintenance work.
- Construction and operation have been experienced successfully in Indonesia in recent years.

(2) Location of Weir

The weir is to be constructed on the flood channel at the point of 150 m upstream of the existing one (PE71+00) from the following reasons:

- The weir can be constructed under the dry condition.

- Since this flood channel is planed to excavate for short cut channel, the construction cost will not be increased.

The existing irrigation channels will be extended to the upstream to connect with the proposed box culvert as well.

(3) Hydraulic Design of Intake Weir

The new dam shall be designed to meet the future irrigation plan. The hydraulic requirements are;

- To provide the required irrigation water which is 2.09 m^3/s for the right channel and 1.68 m^3/s for the left channel under the condition of the river discharge 4.32 m^3/s corresponding to the 95 % flow frequency a year.

- The volume of intake water will be controlled by the control gates at the inlet of the box culverts to maintain the water level of irrigation channel.

- To minimize the inflow of excess water to the drainage channel during the floods, an orifice type intake structure shall be employed.

(a) Water Level at Control Gates

Since water levels of the irrigation channels and box culverts are influenced by the downstream water level, a non-uniform flow method is employed to compute the water level at control gates.

The boundary conditions for the non-uniform flow calculation and results are shown in table bellow:

	Left	**Right**
Boundary Condition		
Discharge	1.68	2.09
Initial Water Level	3.40	3.80
Roughness Coefficient		
Existing Channel	0.030	0.030
Connecting Channel	0.025	0.025
Box Culvert	0.015	0.015
Results of Calculation		
Water Level at		
Outlet of Box Culvert	EL. 3.65 m	EL. 4.04 m
Inlet of Box Culvert	EL. 3.70 m	EL. 4.08 m

Since the water level at the right gates is higher than left one, the hydraulic calculation to determine the water stages of weir and intake facilities is carried out using the right side condition.

(b) Crest Elevation of Weir

The water level at the downstream of the right control gates are higher than left one so that the right side condition will give the critical condition to set the crest elevation.

The crest elevation of the weir is derived from the water level of the control gates, orifice loss (Δh) and overflow depth of weir(h). The relations between them are as shown below:

Crest Elevation = Upstream Water Level of Control Gate
+ Orifice loss - overflow depth of weir

The orifice loss is calculated by the following formula:

$$\Delta h = h_1 - h_2 = \{Q/(C \,.\, B \,.\, h)\}^2/(2 \,.\, g) - h_2$$

where,

Δh	:	orifice loss, difference of water head between upstream and downstream (m)
h_1	:	upstream water height (m)
h_2	:	downstream water height (m)
Q	:	volume of intake water (2.09 m^3/s)
C	:	coefficient of orifice (= 0.253)
B	:	width of orifice (= 1.25 m x 2)
h	:	height of orifice (= 0.70 m, opening 70 %)
g	:	gravity acceleration

An overflow depth of rubber made dam can be obtained from the equation below:

$$hd = \{Q / (Cd \,.\, B)\}^{(2/3)}$$

where,

hd	:	energy head above crest
Q	:	discharge rubber made crest (m^3/s)
Cd	:	discharge coefficient, $Cd = 1.77 \times h/H + 1.05$ (experimental formula for rubber made dam crest)
B	:	length of crest (surface water width at crest)
hd	:	energy head above crest
H	:	upstream water depth from bottom of weir

Orifice loss of the right control gates and overflow depth of the weir are calculated as below:

$\Delta h = 0.09$ m

$hd = 0.11$m

From the results of above calculations, The crest elevation of weir is set at EL. 4.06m (EL. 4.08 m + 0.09 - 0.11 m = EL. 4.06 m).

(c) Auto-deflation Water Level (Maximum Overflow Water Level)

To be free from the vibration and deformation of the dam body caused by overflow water, the overflow water depth of inflatable rubber made dam is limited about 20 % of dam height. Therefore, the inflatable rubber made dam is designed to deflate automatically depending on upstream water level.

On the other hand, to prevent the sedimentation at the upstream of dam, the weir required to deflate a certain extent time. The existing scouring sluice gates have been opened once a month to flash out the sediment. To maintain the equipment of inflatable rubber made dam, the gate operation will be conducted twice a year as a maintenance operation.

The occurrence of auto-deflation concentrates in the rainy season especially on November to December.

Considering the above matters, the maximum overflow water depth will be decided by the structural requirement (H/D<1.4) and number of deflecting time. the number of auto-deflation time is set at 10 times a year.

The number of auto-deflation times corresponding to the river discharge are shown in the following table and the auto-deflection water level is set at EL. 4.87 m.

River Discharge (m^3/s)	Overflow Discharge (m^3/s)	Upstream Water Level (EL. m)	Auto-deflection Time	H/D
20	16.23	4.63	28	1.182
25	21.23	4.72	20	1.210
30	26.23	4.80	15	1.236
35	31.23	4.87	10	1.258
40	36.23	4.93	7	1.277

(d) Bottom Elevation of Orifice

To ensure the orifice flow at the control gate, the upstream water depth shall be more than 1.8 times of orifice height. The bottom elevation is calculated as bellow:

EL. 4.17 m - 0.7 x 1.8 = 2.91 $\geq$ EL. 2.90 m

(e) Hydraulic Design of Box culvert and Connecting Irrigation Channel

As mentioned above, the connecting irrigation channel including the box culvert is designed under following conditions:

a. channel-bed elevation at the outlet of box culvert is EL. 3.13 m.

b. water level at the control gate is EL. 4.08 m.

c. bottom elevation at the control gate is EL. 2.90 m.

d. to ensure the water level at left control gate, a drop structure is provided at the end of winding portion of channel.

e. the dimensions of drop structure are:

bottom elevation of weir	: EL. 3.13 m
upstream water level	: EL. 4.062 m
overflow depth	: H = 0.95 m
width of weir	: $B = Q/(C\ h^{3/2}) = 1.68/(1.75\ 0.95^{3/2})$
	$= 1.04 = 1.05$ m

The basic dimensions of connecting irrigation channels are summarized as bellow.

	Left Channel	Right Channel
Cannel-bed Elevation		
at Box Outlet	EL. 3.13 m	
at Box Inlet	EL. 2.90 m	
Discharge	1.68 m³/s	2.09 m³/s
Connecting Channel		
Channel Width	2.80 m	3.30 m
Water Depth	1.10 m	1.10 m
Channel Slope	1/1,000	1/2,000
Channel Length	190 m	258 m
Channel Lining	Wet Stone Masonry (n = 1:1.0)	
Approach Channel		-
Channel Width	2.80 m	
Water Depth	1.10 m	
Channel Slope	level (EL.3.13 m)	
Channel Length	28.5 m	
Drop Structure drop height drop width upstream WL.	 H = 0.63 m W = 1.05 m EL. 4.062 m	-
Box Culvert		
Channel Width	1.25 x 1.5 x 2	1. 5 x 1.5 x 2
Water Depth	1.10 m	1.10 m
Channel Slope	level (EL. 3.13 m)	
Channel Length	36.78 m	60.60 m

The results of non-uniform calculation is shown in next pages.

1

** INPUT DATA **

* BAISIC DATA *

KUKAN-SU = 5 ALPHA = 1.10 QO = 1.68 M3/S HO = 4.062 M ZO = 3.130 M

JCO = 0 KEY = 0 IPT = 0

* KUKAN DATA *

	DANMEN NO.	BUNKATSU SUU	DANMEN KEIJYO		LOSS TYPE	SODO KEISU	KUKAN KYORI(M)	KASYO KOOBAI(1/I)	RAKUSA (M)	RYUNYU RYO(M3/S)
1	NO.235	6	1	0	0	.0250	28.501000000000.00		.000	.00
2	NO.263	6	1	0	0	.0150	3.401000000000.00		.000	.00
3	NO.235	6	1	0	0	.0150	25.881000000000.00		.000	.84
4	NO.235	6	1	0	0	.0150	2.30	-10.00	.000	.00
5	NO.235	6	1	0	0	.0150	5.001000000000.00		.000	.00

* KEIJYO DATA *

KUKAN	KEIJYO		BO(R)	M1	N1	B1	B2	HP(B3)	M2	N2
1	1	0	2.800	1.000	1.000	.000	.000	.000	.000	.000
2	1	0	2.800	.000	.000	.000	.000	.000	.000	.000
3	1	0	1.250	.000	.000	.000	.000	.000	.000	.000
4	1	0	1.250	.000	.000	.000	.000	.000	.000	.000
5	1	0	1.250	.000	.000	.000	.000	.000	.000	.000

* LOSS DATA *

KUKAN LOSS TYPE FL1 FL2

Bandar Sidoras Weir Irrigation Channel (LEFT)

	NO.	H	A	R	V	N	Q	DX	FROUD	IE	Z	H-Z
Drop Structure	NO. 235	4.062t	3.48	.640	.483	.0250	1.68	.00	.1874	.2645E-03	3.130	.932
	+ 4.75	4.063t	3.48	.641	.482	.0250	1.68	4.75	.1869	.2632E-03	3.130	.933
	+ 9.50	4.065t	3.49	.641	.481	.0250	1.68	4.75	.1865	.2619E-03	3.130	.935
	+ 14.25	4.066t	3.50	.642	.480	.0250	1.68	4.75	.1861	.2606E-03	3.130	.936
	+ 19.00	4.067t	3.50	.643	.480	.0250	1.68	4.75	.1857	.2594E-03	3.130	.937
	+ 23.75	4.068t	3.51	.643	.479	.0250	1.68	4.75	.1852	.2581E-03	3.130	.938
	+ 28.50	4.070t	3.51	.644	.478	.0250	1.68	4.75	.1848	.2569E-03	3.130	.940
	NO. 263	4.059t	2.60	.558	.646	.0150	1.68	.00	.2244	.2040E-03	3.130	.929
	+ .57	4.059t	2.60	.559	.646	.0150	1.68	.57	.2244	.2039E-03	3.130	.929
	+ 1.13	4.059t	2.60	.559	.646	.0150	1.68	.57	.2244	.2039E-03	3.130	.929
	+ 1.70	4.059t	2.60	.559	.646	.0150	1.68	.57	.2243	.2038E-03	3.130	.929
	+ 2.27	4.060t	2.60	.559	.645	.0150	1.68	.57	.2243	.2037E-03	3.130	.930
	+ 2.83	4.060t	2.60	.559	.645	.0150	1.68	.57	.2242	.2036E-03	3.130	.930
	+ 3.40	4.060t	2.60	.559	.645	.0150	1.68	.57	.2242	.2036E-03	3.130	.930
Box Inlet	NO. 235	4.054t	1.15	.373	.728	.0150	.84	.00	.2537	.4441E-03	3.130	.924
	+ 4.31	4.056t	1.16	.373	.726	.0150	.84	4.31	.2528	.4416E-03	3.130	.926
	+ 8.63	4.058t	1.16	.373	.724	.0150	.84	4.31	.2520	.4392E-03	3.130	.928
	+ 12.94	4.060t	1.16	.374	.723	.0150	.84	4.31	.2512	.4367E-03	3.130	.930
	+ 17.25	4.062t	1.16	.374	.721	.0150	.84	4.31	.2504	.4344E-03	3.130	.932
	+ 21.57	4.064t	1.17	.374	.720	.0150	.84	4.31	.2496	.4320E-03	3.130	.934
	+ 25.88	4.066t	1.17	.375	.718	.0150	.84	4.31	.2488	.4297E-03	3.130	.936
	NO. 235	4.066t	1.17	.375	.718	.0150	.84	.00	.2488	.4297E-03	3.130	.936
	+ .38	4.068t	1.22	.381	.688	.0150	.84	.38	.2333	.3857E-03	3.092	.976
	+ .77	4.070t	1.27	.387	.661	.0150	.84	.38	.2195	.3482E-03	3.053	1.017
	+ 1.15	4.072t	1.32	.393	.636	.0150	.84	.38	.2071	.3160E-03	3.015	1.057
	+ 1.53	4.074t	1.37	.398	.612	.0150	.84	.38	.1959	.2880E-03	2.977	1.097
	+ 1.92	4.076t	1.42	.403	.591	.0150	.84	.38	.1856	.2636E-03	2.938	1.137
	+ 2.30	4.077t	1.47	.408	.571	.0150	.84	.38	.1763	.2422E-03	2.900	1.177
	NO. 235	4.077t	1.47	.408	.571	.0150	.84	.00	.1763	.2422E-03	2.900	1.177

	+	.83	4.077t	1.47	.408	.571	.0150	.84	.83	.1763	.2421E-03	2.900	1.177
	+	1.67	4.077t	1.47	.408	.571	.0150	.84	.83	.1762	.2420E-03	2.900	1.177
	+	2.50	4.078t	1.47	.408	.571	.0150	.84	.83	.1762	.2419E-03	2.900	1.178
	+	3.33	4.078t	1.47	.408	.571	.0150	.84	.83	.1761	.2418E-03	2.900	1.178
	+	4.17	4.078t	1.47	.408	.570	.0150	.84	.83	.1761	.2417E-03	2.900	1.178
Control Gate	+	5.00	4.078t	1.47	.408	.570	.0150	.84	.83	.1760	.2416E-03	2.900	1.178

** INPUT DATA **

* BAISIC DATA *

KUKAN-SU = 6 ALPHA = 1.10 QO = 2.09 M3/S HO = 3.950 M ZO = 3.300 M

JCO = 0 KEY = 0 IPT = 0

* KUKAN DATA *

	DANMEN NO.	BUNKATSU SUU	DANMEN KEIJYO		LOSS TYPE	SODO KEISU	KUKAN KYORI(M)	KASYO KOOBAI(1/I)	RAKUSA (M)	RYUNYU RYO(M3/S)
1	NO.121	6	1	0	0	.0250	1.50	-5.00	.000	.00
2	NO.136	6	1	0	0	.0250	258.00	2000.00	.000	.00
3	NO.394	6	1	0	0	.0150	3.401000000000.00		.000	.00
4	NO.397	6	1	0	0	.0150	62.401000000000.00		.000	1.04
5	NO.459	6	1	0	0	.0150	2.30	-10.00	.000	.00
6	NO.462	6	1	0	0	.0150	5.001000000000.00		.000	.00

* KEIJYO DATA *

KUKAN	KEIJYO		BO(R)	M1	N1	B1	B2	HP(B3)	M2	N2
1	1	0	3.300	1.000	1.000	.000	.000	.000	.000	.000
2	1	0	3.300	1.000	1.000	.000	.000	.000	.000	.000
3	1	0	3.300	1.000	1.000	.000	.000	.000	.000	.000
4	1	0	1.500	.000	.000	.000	.000	.000	.000	.000
5	1	0	1.500	.000	.000	.000	.000	.000	.000	.000
6	1	0	1.500	.000	.000	.000	.000	.000	.000	.000

* LOSS DATA *

KUKAN	LOSS TYPE	FL1	FL2

1

Bandar Sidoras Weir Irrigation Channel (RIGHT)

PAGE = 8

	NO.	H	A	R	V	N	Q	DX	FROUD	IE	Z	H-Z
End of Channel	NO. 121	3.950t	2.57	.499	.815	.0250	2.09	.00	.3654	.1047E-02	3.300	.650
	+ .25	3.957t	2.83	.534	.738	.0250	2.09	.25	.3192	.7861E-03	3.250	.706
	+ .50	3.962t	3.09	.567	.676	.0250	2.09	.25	.2826	.6077E-03	3.200	.762
	+ .75	3.966t	3.36	.599	.623	.0250	2.09	.25	.2528	.4802E-03	3.150	.816
	+ 1.00	3.969t	3.62	.629	.577	.0250	2.09	.25	.2280	.3861E-03	3.100	.869
	+ 1.25	3.972t	3.89	.659	.537	.0250	2.09	.25	.2070	.3150E-03	3.050	.921
	+ 1.50	3.974t	4.16	.687	.502	.0250	2.09	.25	.1890	.2601E-03	3.000	.973
	NO. 136	3.974t	4.16	.687	.502	.0250	2.09	.00	.1890	.2601E-03	3.000	.973
	+ 43.00	3.985t	4.11	.682	.509	.0250	2.09	43.00	.1924	.2701E-03	3.022	.963
	+ 86.00	3.996t	4.05	.676	.516	.0250	2.09	43.00	.1958	.2801E-03	3.043	.953
	+ 129.00	4.008t	4.00	.671	.522	.0250	2.09	43.00	.1991	.2902E-03	3.065	.943
	+ 172.00	4.021t	3.96	.666	.528	.0250	2.09	43.00	.2024	.3002E-03	3.086	.934
	+ 215.00	4.033t	3.91	.661	.534	.0250	2.09	43.00	.2055	.3102E-03	3.108	.925
	+ 258.00	4.047t	3.87	.656	.540	.0250	2.09	43.00	.2086	.3201E-03	3.129	.917
	NO. 394	4.047t	3.87	.656	.540	.0150	2.09	.00	.2086	.1152E-03	3.129	.917
	+ .57	4.047t	3.87	.656	.540	.0150	2.09	.57	.2086	.1152E-03	3.129	.917
	+ 1.13	4.047t	3.87	.656	.540	.0150	2.09	.57	.2085	.1152E-03	3.129	.917
	+ 1.70	4.047t	3.87	.656	.540	.0150	2.09	.57	.2085	.1151E-03	3.129	.917
	+ 2.27	4.047t	3.87	.656	.540	.0150	2.09	.57	.2085	.1151E-03	3.129	.917
	+ 2.83	4.047t	3.87	.656	.540	.0150	2.09	.57	.2085	.1151E-03	3.129	.917
	+ 3.40	4.047t	3.87	.656	.540	.0150	2.09	.57	.2084	.1150E-03	3.129	.918
Box. Outlet	NO. 397	4.030t	1.35	.409	.774	.0150	1.04	.00	.2732	.4434E-03	3.129	.900
	+ 10.40	4.035t	1.36	.410	.770	.0150	1.04	10.40	.2710	.4372E-03	3.129	.905
	+ 20.80	4.040t	1.37	.411	.765	.0150	1.04	10.40	.2688	.4312E-03	3.129	.910
	+ 31.20	4.044t	1.37	.412	.761	.0150	1.04	10.40	.2667	.4253E-03	3.129	.915
	+ 41.60	4.049t	1.38	.413	.758	.0150	1.04	10.40	.2646	.4196E-03	3.129	.920
	+ 52.00	4.054t	1.39	.414	.754	.0150	1.04	10.40	.2626	.4142E-03	3.129	.924
	+ 62.40	4.058t	1.39	.415	.750	.0150	1.04	10.40	.2607	.4089E-03	3.129	.929
	NO. 459	4.058t	1.39	.415	.750	.0150	1.04	.00	.2607	.4089E-03	3.129	.929
	+ .38	4.061t	1.46	.423	.718	.0150	1.04	.38	.2443	.3655E-03	3.091	.970

	+ .77	4.063t	1.52	.431	.689	.0150	1.04	.38	.2297	.3288E-03	3.053	1.011
	+ 1.15	4.066t	1.58	.438	.663	.0150	1.04	.38	.2166	.2973E-03	3.014	1.051
	+ 1.53	4.068t	1.64	.445	.638	.0150	1.04	.38	.2047	.2702E-03	2.976	1.091
	+ 1.92	4.069t	1.70	.451	.616	.0150	1.04	.38	.1939	.2466E-03	2.938	1.131
	+ 2.30	4.071t	1.76	.457	.595	.0150	1.04	.38	.1841	.2260E-03	2.900	1.171
	NO.462	4.071t	1.76	.457	.595	.0150	1.04	.00	.1841	.2260E-03	2.900	1.171
	+ .83	4.071t	1.76	.457	.595	.0150	1.04	.83	.1841	.2259E-03	2.900	1.172
	+ 1.67	4.071t	1.76	.457	.595	.0150	1.04	.83	.1840	.2258E-03	2.900	1.172
	+ 2.50	4.071t	1.76	.457	.594	.0150	1.04	.83	.1840	.2257E-03	2.900	1.172
	+ 3.33	4.071t	1.76	.457	.594	.0150	1.04	.83	.1839	.2256E-03	2.900	1.172
	+ 4.17	4.072t	1.76	.457	.594	.0150	1.04	.83	.1839	.2255E-03	2.900	1.172
Control Gate	+ 5.00	4.072t	1.76	.457	.594	.0150	1.04	.83	.1838	.2254E-03	2.900	1.172

2.3.2 Structural Design

Spillway Bed Structure (Main Dam Body)

(1) Length of Apron and Riverbed Protection

The length of apron are determined by using the Bligh's Formula.

$$W > 0.6\ Co\ D^{1/2}$$

where, W : length of apron
L : total length of apron and riverbed protection works
Co: creep ratio (Co = 18 for fine sand and clay)
D : height of weir (D = 3.14 m)

$$W > 0.6 \times 18 \times 3.14^{1/2} = 19.14 \text{ m}$$

Since the length of down stream spillway bed is 7.479m and the distance between dam axis and water falling point is about 4.0 m, the apron length (La) is calculated as bellow:

$$La = 19.14 \text{ m} - (7.479 \text{ m} - 4.0 \text{ m}) = 15.66 < 16 \text{ m}$$

The length of upstream apron is generally set at half length of that of downstream one. The length of riverbed protection works is set at the same length of apron.

The required length of apron is, spillway bed and protection works are summarized bellow:

	upstream	downstream
length of spillway bed	4.521 m	7.479 m
length of apron	8 m	16 m
length of protection works	10 m	20 m

(2) Thickens of Apron

To withstand the scouring force and uplift pressure, the thickness of spillway bed and apron is set at 1.6 m and 1.2 m as bellow.

$$t = 4/3\ (\Delta h - \Delta u)/(Wc - 1)$$

where, Δh : max. difference between water heads (m)
Δu : uplift at the toe of main body (t/m^2)
Wc : unit weight of concrete ($Wc = 2.35\ t/m^3$)

	US Water Level (EL. m)	DS Water Level (EL. m)	Thickness (m)	
			Spillway Bed	Apron
Normal Condition	4.06	0.92	1.650	1.180
Flood Condition	4.36	1.55	1.463	1.069

(3) Prevention of piping

The required creep length against piping can be obtained by Lane's method.

$C \leq (Lh/3 + \Sigma Lv)/\Delta H$

where, C : Lane's creep ratio, C = 8.5 for silty sand/clay
Lh : creep length of horizontal direction,
(Lh = 8+16+12= 36 m)
Lv : creep length of vertical direction (m)
ΔH : max. difference between water heads (m), ΔH= 3.14 m

$\Sigma Lv \geq C\ \Delta H - Lh/3 = 8.5 \times 3.14 - 36/3) = 14.7$ m

To obtain a more than 14.7 m vertical creep length, the steel sheet piles with 2 m long will be driven at four (4) locations which is upstream and downstream edge of apron and spillway bed structure.

(4) Foundation

The foundation of weir is consisted of fine sand whose N value is 16 in average. The supporting layer which has the N value more than 50 observed below EL. -10.0 m in boring Bo.07.

As for the foundation of the rubber made dam, PC piles ϕ 600 mm are to be driven to increase the bearing capacity of the subsurface layer to support the weir body. The detail of foundation design is presented in 2.3.3 Design Calculation.

(2) Inflatable Rubber Made Dam

An air filled inflatable/deflectable rubber made dam with automatically deflation system is employed. The power for operation of rubber made dam is domestic power consumption. One set of diesel engine generator is prepared for the emergency power source.

The detail of design for dam body is shown in 2.3.3 Design Calculation.

(3) Machine Room and Control House

A control house is provided at the left side of the left dike. The control house is composed of the entrance hall, the operator room and the operation room and at the basement, machine room is provided. The minimum area for each space is;

Entrance hall	2.5 m^2
Operator room	14.5 m^2
Operation room	8.0 m^2
Machine room	8.0 m^2
Total	33.0m^2

(4) Inspection Bridge

To approach the control gates for irrigation, a inspection bridge is provided at the upstream of rubber made dam. The bridge is designed as a steel plate girder pedestrian bridge. The basic dimensions of bridge are;

Span	L_1 = 26.4 m (control house to left control gate)
	L_2 = 36.0 m (left control gate to right control gate)
Steel plate girder	I Beam (120 cm high, 25cm wide)
Width	1.5 m wide

The design calculation of the inspection bridge is attached in 2.3.3 Design Calculation.

(5) Flood Channel Protection Work

For the flood channel protection, the crib-type concrete blocks (1.5 m x 1.5 m x 0.2 m, 0.5 ton/each) with boulder filling are provided to prevent erosion along the dam body and intake facilities for irrigation. The protection work covers the area along the low water channel and above the box culverts for irrigation intake. The area to be covered by protection works is 3,000 m^2 in total.

(6) Structural Features of Proposed Intake Weir

The determined structural features of the proposed weir and related irrigation structures through the hydraulic design are given below and shown in the DRW **.

Work Item		Dimensions
Main Rubber Dam		
Crest Elevation		EL. 4.06 m
Bottom Elevation		EL. 0.92 m
Auto-deflection Water Level		EL. 4.87 m
Height of Rubber Body		3.14 m
Width of Rubber Body		13.00 m at the bottom
Maximum Water Head		3.95 m at the auto-deflation water level
Design Overflow Depth		0.81 m
Concrete Footing with Steel Pipe Pile		28.0 m long
Waterstop (Steel sheet pile Type II)		2.0 m long at 4 locations
Foundation Treatment		PC Pile ϕ=600, L=12m-14m, 76 nos. PC Pile ϕ=400, L=12m, 62 nos.
Intake Facilities		
Intake Gates (Steel slide gate)	Right Left	HxB=1.00 m × 1.25 m, 2 nos. HxB=1.00 m × 1.00 m, 2 nos.
Size of Box Culvert	Right Left	HxBxL=1.50 m × 1.50 m × 37.3m, 2 nos. HxBxL=1.50 m × 1.25 m × 73.3m, 2 nos.
Irrigation Channel		
Shape and Type		Trapezoid-section and wet masonry type
Channel Size	Right Left	Bottom Width B=3.3m, L=257 m Bottom Width B=2.8m, L=218 m
Maintenance Bridge (steel)		W = 1.1 m, L1 = 66 m, L2=
Control House		1 site
Protection Works for Flood Channel		3,000 m^2

2.3.3 Design Calculation

<u>Calculation Conditions</u>

(1) Section for Structural Calculation

The following nine (9) sections are subject to the design calculation.

<u>Main body</u>

a. Spillway Bed (flow direction)
b. Spillway Bed (cross section)
c. Downstream Apron (flow direction)
d. Downstream Apron (cross section)
e. Upstream Apron (flow direction)
f. Upstream Apron (cross section)

<u>Irrigation Intake Structures</u>

a. Inlet Structure
   - Inlet
   - Gate Post
b. Sluice-way
   - Box Culvert

The structural dimensions of above sections are shown in Fig. 2.3.1.

(2) Safety Conditions

<u>Sliding</u>

The factor of safety against sliding is determined using the following formula:

$$SF = Rc/\Sigma H$$

where, H : total horizontal forces
Rc : resisting capacity ($Rc = \Sigma V \tan \phi$, $\phi = 30^\circ$)
SF : safety factor given in the following
$SF \geq 1.5$ (under normal condition)
$SF \geq 1.2$ (under floods and earthquake condition)

<u>Overturning</u>

All forces acting on part of the structure above any horizontal plane should fall within the middle third of the structure base.

For this purpose, the following condition should be satisfied :

$$e = \frac{b}{2} - \frac{M}{N} < \frac{b}{6}$$

where, b : width of base (m)
M : total moment about point A (tf . m)
N : total vertical forces (tf)
e : eccentricity (m)

Bearing Capacity of Pile Foundation

The maximum principal stress in the spread foundation must be kept within allowable soil bearing capacity which is derived from the following :

Qa = Qu/SF

where, Qa : allowable soil bearing capacity
Qu : ultimate soil bearing capacity
$Qu_1 = 55$ tf/m^2 for long term load,
$Qu_2 = 73$ tf/m^2 for short term load.
SF $\geq$ 3 (under the normal condition)
SF $\geq$ 2 (under the earthquake condition)

The allowable soil bearing capacity for the diversion weirs are;

$Qa_1 = 20$ tf/m^2 / 3 = 10 tf/m^2 for normal condition,
$Qa_2 = 30$ tf/m^2 / 2 = 15 tf/m^2 for earthquake condition.

(3) Load Combination

The load combination for the stability analysis and stress calculation is shown in table bellow.

Cross Section

	after const.	normal	dry	flood	earthquake
Self Weight	X	X	X	X	X
Earthquake Load	-	-	-	-	X
Gate Load	rubber	X	X	X	X
Soil Pressure	-	-	-	-	X
Water Weight (upstream)	-	X	X	X	X
Water Weight (downstream)	-	X	-	X	X
Uplift	-	X	X	X	X

Flow Direction

	after const.	normal	dry	flood	earthquake
Self Weight	X	X	X	X	X
Earthquake Load	-	-	-	-	X
Gate Load	rubber	X	X	X	X
Water Weight (upstream)	-	X	X	X	X
Water Weight (downstream)	-	X	-	X	X
Water Perssure	X	X	X	X	X
Hydrodynamic Pressure	-	-	-	-	X
Uplift	-	X	X	X	X

Stability Analysis

The results of load calculation and stability analysis of all calculation conditions are given bellow.

(1) Main Body - Cross Section

(a) After Construction

	V	VX	H	HZ	
Self Weight	1,876.558	17,733.469			
Gate	2.21	20.88			
Uplift	0	0	0	0	
Total	1,878.768	17,754.349	0	0	

V = 1,878.768

e=18.9/2−17,754.349/1,878.768=0.000<B/6

(b) normal

	V	VX	H	HZ	
Self weight	1,876.558	17,733.469			
Gate	2.21	20.88			
Water Weight (upstream)	224.242	2,119.087			
Water Weight (downstream)	44.272	418.370			
Uplift	-568.243	-5,369.896			
Total	1,579.039	14,921.910			

V = 1,579.039

e = 9.45−14,921.910/1,579.039=0.000<B/6

(c) dry condition

	V	VX	H	HZ	
Self weight	1,876.558	17,733.469			
Gate	2.21	20.88			
Water Weight (upstream)	198.508	1,875.9			
Uplift	445.77	4,212.527			
Total	1,631.506	15,417.722			

V = 1,631.506

e = 9.45−15,417.722/1,631.506=0.000

(d) Flood Condition

	V	VX	H	HZ	
Self weight	1,876.558	17,733.469			
Gate	2.21	20.88			
Water Weight (upstream)	253.912	2,399.468			
Water Weight (downstream)	103.543	978.481			
Uplift	-708.600	-6,696.27			
Total	1,527.623	14,436.028			

V = 1,527.623

e = 9.45 − 14,436.028/1,527.623 = 0.000

(e) earthquakes

	V	V•x	H	H•z	
Self Weight	1,876.558	17,733.469	375.312	729.042	
Gate	2.21	20.88	0.44	0.704	
Water Weight (upstream)	224.242	2,119.087	0.738	0.939	
Water Weight (downstream)	44.272	418.370	0.045	0.011	
Uplift	-568.243	-5,369.896			
Acting Earth Pressure			28.506	54.537	
Passive Earth Pressure			-113.742	-45.137	
Total	1,579.039	14,921.91	291.299	740.096	

V = 1,579.039

H = 291.299

n = 14,921.91 + 740.096 = 15,662.006

e = 0.469 < B/3 = 6.3

Fs = 1,579.039 x tan 20/291.29 = 1.97 > 1.5

(2) Main Body - Flow Direction

(a) After Construction

	V	VY	H	HZ	
Self Weight	1,876.558	11,287.619			
Gate	2.21	9.39			
Total	1,878.768	11.297.009			

V = 1,878.768

e = 11,297.009/1,878.768 − B/2 = 0.013 < B/6

(b) Normal Condition

	V	VY	H	HZ	
Self Weight	1,876.558	11,287.619			
Gate	2.21	9.39			
Water Weight (upstream)	224.242	367.757	381.949	603.479	
Water Weight (downstream)	44.272	422.178	-43.52	-23.196	
Uplift	-568.243	-3,262.283			
Total	1,579.039	8,824.661	338.429	580.283	

V = 1,579.039

H = 338.429

e = (8,824.661 + 580.283)/1,579.039 − 6.0 = -0.044 > -B/6

Fs = 1,579.039 × tan 20/338.429 = 1.698 > 1.5

(c) Dry Condition

	V	V Y	H	H Z	
Self Weight	1,876.558	11,287.619			
Gate	2.21	9.39			
Water Weight (upstream)	198.508	325.553	381.949	603.479	
Water Weight (downstream)	0	0	-43.52	-23.196	
Uplift	-445.770	-3,262.283			
Total	1,631.506	8,360.279	338.429	580.283	

V = 1,631.506
H = 338.429
e = (8,360.279 + 580.283)/1,631.506 − 6.0 = −0.520 > -B/6
Fs = 1,631.506 × tan20/338.429 = 1.755 > 1.5

(d) Flood Condition

	V	V Y	H	H Z	
Self Weight	1,876.558	11,287.619			
Gate	2.21	9.39			
Water Weight (upstream)	253.912	416.416	381.949	603.479	
Water Weight (downstream)	103.543	987.386	-43.52	-23.196	
Uplift	-708.600	-4,121.926			
Total	1,527.623	8,578.885	338.429	580.283	

V = 1,527.623
H = 338.429
e = (8,578.885 + 580.283)/1,527.623 − 6.0 = -0.004 > B/6
Fs = 1,527.623 × tan20/338.429 = 1.643 > 1.5

(e) Earthquake Condition

	V	V Y	H	H Z	
Self Weight	1,876.558	11,287.619	±375.312	±729.042	
Gate	2.21	9.39	±0.44	±0.704	
Water Weight (upstream)	224.242	367.757	381.949	603.479	
Water Weight (downstream)	44.272	422.178	-43.52	-23.196	
Uplift	-568.243	-3,262.283			
Hydrodynamic Pressure			22.178	63.340	
			-0	-0	
Total	1,579.039	8,824.661	736.359	1,373.369	
			-38.323	-149.463	

V = 1,579.039
H = 736.359(up → down)
=-38.323(down → up)
e = (8,824.661 + 1,373.369)/1,579.039 − 6.0 = 0.458 < B/3
= (8,824.661 − 149.463)/1,579.039 − 6.0 = -0.506 > B/3
Fs = 1,579.039 × tan20/736.359 = 0.78 < 1.2

(3) Downstream Apron - Cross Section

(a) After Construction

	V	V•x	H	H•z	
Self Weight	1,755.072	14,961.988			
Total	1,755.072	14,961.988			

V = 1,755.072

e = 14,961.188/1,755.072 − B/2 = = 0.000 < B/6

(b) Normal Condition (flood)

	V	V•x	H	H•z	
Self Weight	1,755.072	14,961.988			
Water Weight	1,238.973	10,562.243			
Uplift	-584.43	-4,982.266			
Total	2,409.615	20,541.965			

$W = 1/2 \times (13.0 + 28.08) \times 3.77 \times 16 = 1,238.973$

V = 2,409.615

e = 0.000 < B/6

(c) Dry Condition

	V	V•x	H	H•z	
Self Weight	1,755.072	14,961.988			
Uplift	-216.570	-1,846.259			
Total	1,538.502	13,115.729			

V = 1,538.502

e = 0.000

(d) Earthquake Condition

	V	V•x	H	H•z	
Self Weight	1,755.072	14,961.988	351.014	667.844	
Water Weight	143.741	1,225.392			
Uplift	-384.826	-3,280.642			
Total	1,513.987	12,906.738	351.014	667.844	

$W = 1/2 \times (13.0 + 15.52) \times 0.63 \times 16 = 143.741$

V = 1,513.987

e = 0.441

$Fs = 1,513.987 \times \tan 20 / 351.014 = 1.57 > 1.2$

(4) Downstream Apron - Flow Direction

(a) After Construction

	V	V•y	H	H•z	
Self Weight	1,755.072	14,972.138			
Total	1,755.072	14,972.138			

V = 1,755.072

e = 14,972.138/1,755.072 − B/2 = 0.531 < B/6

(b) Normal Condition

	V	V•y	H	H•z	
Self Weight	1,755.072	14,972.138			
Water Weight	1,238.973	9,911.784			
Uplift	-584.43	-4,396.082			
Total	2,409.615	20,487.84			

V = 2,409.615

e = 0.503 < B/6

(c) Dry Condition

	V	V•y	H	H•z	
Self Weight	1,755.072	14,972.138			
Uplift	-216.570	-1,390.596			
Total	1,538.502	13,581.542			

V = 1,538.502

e = 0.828

(d) earthquake

	V	V•y	H	H•z	
Self Weight	1,755.072	14,972.138	±351.014	±667.844	
Water Weight	143.741	1,149.928			
Uplift	-384.826	-2,765.360			
Total	1,513.987	13,356.706	±351.014	±667.844	

V = 1,513.987

H = 351.014

e = 1.263

=0.381

Fs = 1,513.987 × tan20/351.014 = 15.737 > 1.5

(4) Upstream Apron - Flow Direction

(a) Normal Condition (Flood Condition)

	V	V•x	H	H•z	
Self Weight	156.0	1,014.0			
Water Weight	619.486	4,026.659			
Uplift	-352.04	-2,288.26			
Total	423.446	2,752.399			

$W = /2 \times (13.0 + 28.08) \times 3.77 \times 8.0 = 619.486$

$V = 423.446$

$e = 2{,}752.399/423.446 - B/2 = 0$

(b) Earthquake Condition

	V	V•x	H	H•z	
Self Weight	156.0	1,014.0	±31.2	±9.36	
Water Weight	547.098	3,556.134			
Uplift	-311.48	-2,024.62			
Total	391.618	2,545.514	±31.2	±9.36	

$W = 1/2 \times (13.0 + 26.76) \times 3.44 \times 8.0 = 547.098$

$V = 391.618$

$H = 31.2$

$e = 0.024$

$$Fs = \frac{391.618 \times \tan 20}{31.2} = 4.57 > 1.5$$

(5) Upstream Apron - Flow Direction

(a) normal(flood)

	V	V•y	remarks
Self Weight	156.0	624.0	
Water Weight	619.486	2,477.944	
Uplift	-352.04	-1.385.277	
Total	423.446	1,716.667	

$V = 423.446$

$$e = \frac{1{,}716.667}{423.446} - \frac{8.0}{2} = 0.054$$

(b) earthquake

	V	V•y	H	H•z	remarks
Self Weight	156.0	624.0	31.2	9.36	
Water Weight	547.098	2,188.392			
Uplift	-311.48	-1,218.821			
Total	391.618	1.593.571	31.2	9.36	

$V = 391.618$
$H = 31.2$
$e = 0.093$

$$Fs = \frac{391.618 \times \tan 20}{31.2} = 4.57 > 1.2$$

Design of Foundation

(1) Main Dam (Spillway Bed)

(a) Selection of Foundation Type

The ultimate bearing capacity of the foundation is estimated by Terzaghi s formula.

$$Qu = A' \times \frac{1}{2} \gamma \beta B' N_\gamma$$

where, Qu : ultimate bearing capacity of the foundation (tf/m^2)
A : effective area (m^2)
$A' = (18.9 - 2 \times 0.409) \times (12 - 2 \times 0.506) = 197.366$
γ : unit weight of foundation soil (= 0.8 tf/m^3)
B : B - 2eB = 12 − 2 x 0.506 = 10.988
Nr : 0

$\therefore$ Qu = 0

Since the ultimate bearing capacity of the foundation is estimated at 0 tf/m^2, a pile foundation shall be employed to the structures.

The type and diameter of piles are selected through the cost comparison including the driving cost. The result of the study is shown in the following table:

	Nos. of Pile	Dia. of Pile Main Body	Dia. of Pile DS. Apron	Construction Cost Rp.
1	76	ϕ600	ϕ600	154,668,436
2	50	ϕ500	ϕ500	169,855,861
3	37	ϕ600	ϕ400	168,883,585
4	78	ϕ500	ϕ500	182,109,304
5	54	ϕ500	ϕ400	181,137,028
6	39	ϕ400	ϕ400	184,769,354

As shown in the table above, PC pile ϕ600 is employed to the main body and the downstream apron.

The reaction of pile and moment at the top of pile were calculated by computer. The result of calculation is summarized bellow:

			Pn_{max}	Pn_{min}	H (tf)	n (tm)	σ_{max}	σ_{min}
Main body	Cross Section	After Const.	49.692	49.692	0.0	0.0	31.6	31.6
		earthquake	41.546	31.898	6.774	1.839	38.8	8.0
	Flow Direction	After Const	43.868	43.517	0.0	0.019	28.05	25.6
		dry	43.178	32.706	7.870	3.389	50.2	-1.9
		earthquake(1)	43.489	29.955	17.125	4.292	56.5	-9.7
		earthquake(2)	42.522	30.922	0.891	0.478	30.3	16.5
Downstream apron	Cross Section	flood	54.764	54.764	0.0	0.0	34.9	34.9
		earthquake	41.234	27.584	7.978	2.100	40.3	3.5
	Flow Direction	flood	57.822	52.216	0.0	0.221	38.3	31.8
		earthquake	38.468	30.438	0.796	0.137	25.4	18.5
Upstream apron	Cross Section	flood	23.525	23.525	0.0	0.0	30.7	30.7
		earthquake	22.076	21.437	1.733	0.3374	35.9	20.9
	Flow Direction	flood	24.128	22.921	0.0	0.0639	32.9	28.6
		earthquake	22.878	20.635	1.733	0.2171	34.4	22.4

(2) Inlet Structure

(a) Load Calculation

The result of load calculation and combination of load are summarized bellow:

Normal Cross Section

	V	V*x	V*y	H	H*z	remarks
Main	399.009	1926.843				
Gate	3.92	12.17552				
Water	33.93	198.4905				
Water Press.						
Uplift						
Wind Press.						
Total	436.859	2137.5090	0	0	0	

Normal+Wind Cross Section

	V	V*x	V*y	H	H*z	remarks
Main	399.009	1926.843				
Gate	3.92	12.17552				
Water	33.93	198.4905				
Water Press.						
Uplift						
Wind Press.				2.387	12.49269	
Total	436.859	2137.5090	0	2.387	12.49269	

M= 2137.509+12.493=2140.002

Flood1 Cross Section

	V	V*x	V*y	H	H*z	remarks
Main	399.009	1926.843				
Gate	3.92	12.17552				
Water	20.25	27.3375				
Water Press.	4.56	8.23992				
Uplift						
Wind Press.				2.387	12.49269	
Total	427.739	1974.5959	0	2.387	12.49269	

M= 1974.596+12.493=1987.089

Flood2 Cross Section

	V	V*x	V*y	H	H*z	remarks
Main	399.009	1926.843				
Gate	3.92	12.17552				
Water	59.175	307.5975				
Water Press.						
Uplift						
Wind Press.				2.387	12.49269	
Total	462.104	2246.6160	0	2.387	12.49269	

M= 2246.616+12.493=2259.109

Earthqueke Cross Section

	V	V*x	V*y	H	H*z	remarks
Main	399.009	1926.843		79.804	147.976	
Gate	0.49	1.4455		0.098	0.2254	
Water	33.93	198.4905				
Water Press.	0.912	1.186				
Uplift						
Wind Press.						
Total	434.341	2127.965	0	79.902	148.2014	

M= 2127.965+148.201=2276.166

under Cross Section

	V	V*x	V*y	H	H*z	remarks
Main	399.009	1926.843				
Gate	0.49	1.4455				
Water						
Water Press.						
Uplift						
Wind Press.						
Total	399.499	1928.2885	0	0	0	

Normal Flow Direction

	V	V*x	V*y	H	H*z	remarks
Main	399.009		1577.896			
Gate	3.92		15.68			
Water	33.93		135.72			
Water Press.						
Uplift						
Wind Press.						
Total	436.859	0	1729.296	0	0	

Normal+win Flow Direction

	V	V*x	V*y	H	H*z	remarks
Main	399.009		1577.896			
Gate	3.92		15.68			
Water	33.93		135.72			
Water Press.						
Uplift						
Wind Press.				-2.311	-12.242895	
Total	436.859	0	1729.296	-2.311	-12.242895	

M=1729.296-12.243=1717.053

Flood Flow Direction

	V	V*x	V*y	H	H*z	remarks
Main	399.009		1577.896			
Gate	3.92		15.68			
Water	59.175		239.5			
Water Press.						
Uplift						
Wind Press.				-2.311	-12.242895	
Total	462.104	0	1833.076	-2.311	-12.242895	

M=1833.076-12.243=1820.833

Earthquake Flow

	V	V*x	V*y	H	H*z	remarks
Main	399.009		1577.896	-79.804	-147.976	
Gate	0.49		1.96	0.098	0.2254	
Water	33.93		135.72			
Water Press.						
Uplift						
Wind Press.						
Total	433.429	0	1715.576	-79.706	-147.7506	

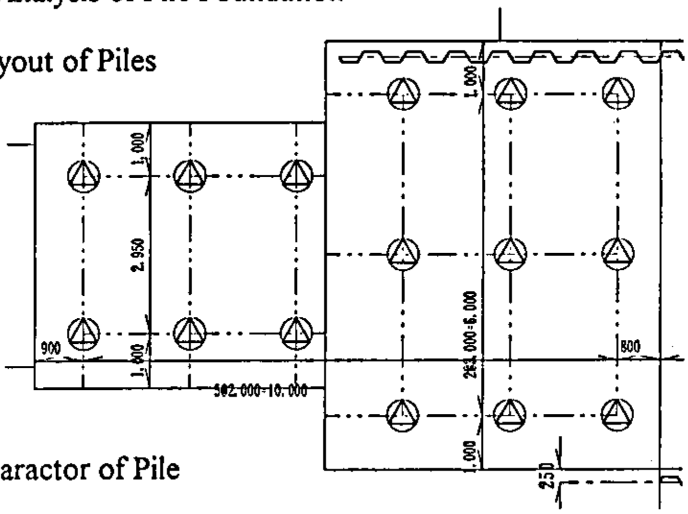
M=1715.576-147.751=1567.825

under Flow Direction

	V	V*x	V*y	H	H*z	remarks
Main	399.009		1577.896			
Gate	0.49		1.96			
Water						
Water Press.						
Uplift						
Wind Press.						
Total	399.499	0	1579.856	0	0	

(b) Stability Analysis of Pile Foundation

(i) Layout of Piles



(ii) Charactor of Pile

- Pile Material PC Pile ϕ 600,
- Unit Weight 4.21 t/nos = 0.30 tf/m
- Length & Area L = 14 m, A = 1570.8 m^2
- Polar moment of inertia I = 0.005105 m^4
- Young's modulus E = 400,000 kg/cm^2
- Kv = 50,116 tf/m
- Kh = 15,372 tf/m^3 (h = 0.6 m)
  = 5,593 tf/m^3 (h = 2.0 m)
  = 8,347 tf/m^3 (h = 3.45 m)
  = 1,895 tf/m^3 (h = 6.0 m)
- Allowable vertical bearing capacity of pile : 68.4 tf/nos.

(iii) Stability Calculation

Summary of stability calculation of piles are shown bellow:

		PN_{max}	PN_{min}	H	M	σ_{MAX}	σ_{MIN}
Cross Section	Normal	50.52	12.31	0	-0.688	96.2	63.79
	Normal+wind	50.36	12.44	0.159	-0.855	97.08	62.9
	Flood1	52.82	9.42	0.159	-0.954	99.23	60.39
	Flood2	53.42	13.04	0.159	-0.899	99.29	63.02
	Earthquake	42.93	17.97	5.327	-5.351	118.78	40
	under construction	47.06	10.58	0	-0.657	93.82	62.87
Flow Direct.	Normal	29.9	28.34	0	-0.047	79.31	77.77
	Normal+wind	30.54	27.71	-0.154	0.082	79.92	77.16
	Flood	32.1	29.51	-0.154	0.089	80.96	78.26
	Earthquake	39.07	18.72	-5.314	4.098	108.95	47.84
	under construction	27.41	25.85	0	-0.047	77.73	76.18

I = 510500 (cm^4)
Z = 510500*2/60 =17017 (cm^3)

(2) Sluice way for Irrigation

The total load of Sluice way (ΣW) is:

$\Sigma W1$ = Wc + Ws
$\Sigma W2$ = Wc +Ww + Ws +q

where, $\Sigma W1$: Load on after construction
$\Sigma W2$: Load on normal condition
Wc : Self weight (γc = 2.5 tf/m^3)
Ww : Water weight (γw = 1.0 tf/m^3)
Ws : Soil weight (γs = 1.8 tf/m^3)
q : Live Load (q = 1 tf/m^2)

$\Sigma W1$ = 18.352 tf
$\Sigma W2$ = 24.222 tf.

The ultimate bearing capacity of Terzaghi is calculated:

Qu_1 (for after construction) 55.97 tf/m^2
Qu_1 (for normal condition) 306.97 tf/m^2

The allowable bearing capacity is

Qu_1 (for after construction) 55.97/3 = 18.66 tf/m^2 > 18.352 tf/m^2
Qu_1 (for normal condition) 306.97/3 = 102.32 tf/m^2 > 24.222 tf/m^2

A spread foundation can be applied to the sluice way.

Structural Calculation

(1) Spillway Bed

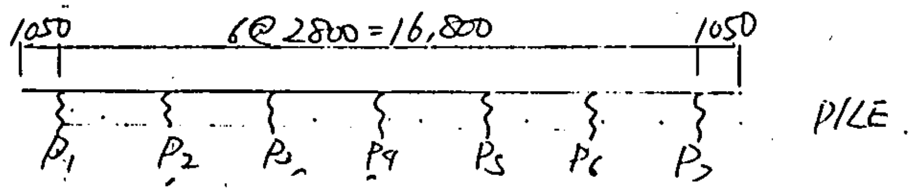
The apron slab regard as a elastic beam supported by the piles and the wing wall is cantilever fixed at the apron slab.

1) apron slab (per m)

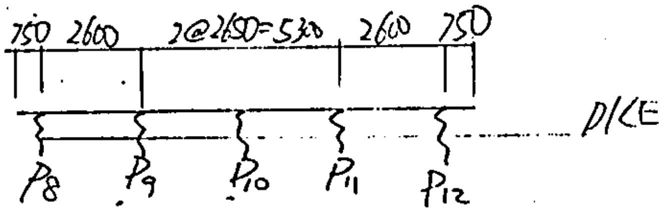
a) beam

i) dimension

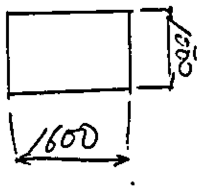
• cross section



• flow direction



ii) crossing sectional area



$$A = 1.6 \times 1.0 = 1.6(m^2)$$

iii) moment of inertia of area

$$I = 1.0 \times 1.6^3/12 = 0.3413(m^4)$$

b) load(per m)

i) bottom beam

$$w_1 = 1.6 \times 2.5 = 4.0(t/m)$$
$$H_{w1} = 4.0 \times 0.2 = 0.8(t/m)$$

ii) spring force of pile

- case of normal, flood, dry and just construct

$k_V = 19{,}200(tf/m)$(right angle of stream)
$= 17{,}900(tf/m)$(direction of stream)
$k_H = 1/3 \times k_V$

- case of earthquake

$k_V = 38{,}400(tf/m)$(right angle of stream)
$= 35{,}800(tf/m)$(direction of stream)
$k_H = 1/3 \times k_V$

iii) weight of water

- case of normal and earthquake

w_2 up $= 3.44 \times 1.00 = 3.44(t/m)$
w_2 down $= 0.63 \times 1.00 = 0.63(t/m)$

- case of dry Condition

w_3 up $= 3.14(t/m)$

- case of flood

w_4 up $= 3.77(t/m)$
w_4 down $= 1.34(t/m)$

iv) weight of gate

$w_5 = 0.1(t/m)$

v) water pressure

- gate
$H_2 = 1/2 \times 3.14^2 \times 1.0 = 4.930(tf)$
$n_{H2} = 4.930 \times (0.8 + 3.14/3) = 9.105(tf \cdot m)$

- normal and earthquake
H_8 up $= 1/2 \times 3.44^2 = 5.917(tf)$
n_{H8} up $= 10.335(tf \cdot m)$
H_8 down $= 1/2 \times 0.63^2 = 0.198(tf)$
n_{H8} down $= 0.160(tf \cdot m)$

• Dray condition

H_9 up $=1/2\times 3.14^2=4.930$(tf)
n_{H9} up $=8.118$(tf•m)

• flood

H_{10} up $=1/2\times 3.77^2=7.106$(tf)
n_{H10} up $=13.193$(tf•m)

vi) earth pressure in earthquake

$H_3=2.376$(tf)
$n_{H3}=4.545$(tf•m)
$H_4=3.761$(tf)
$n_{H4}=9.479$(tf•m)

vii) water pressure in earthquake

$H_5=7/12\times 1.0\times 0.2\times 3.14^2=1.150$(tf)(direction of stream)
$n_{H5}=1.150\times 2.056=2.364$(tf•m)
$H_6=0.738$(tf)(right angle of stream)
$n_{H6}=0.738\times 2.072=1.529$(tf•m)
$H_7=0.006$(tf)(right angle of stream)
$n_{H7}=0.006\times 1.055=0.006$(tf•m)

viii) uplifting pressure of water

• right angle of stream

$u_1=1.795$(t/m)(Dray Condition)
$u_2=2.24$(t/m)(normal)
$u_3=2.73$(t/m)(flood)

• direction of stream

$u_4=2.12$(t/m)(Dray Condition)
$=1.47$

$u_5=2.53$(t/m)(normal)
$=1.95$

$u_6=2.98$(t/m)(flood)
$=2.48$

ix) load at end of beam

<u>load from wing wall</u>

・weight of wall

	W	x	y	H	W•x	W•y	z	H•z
1	16.405	2.208		3.281	36.222		1.47	4.823
2	7.027	4.708		1.405	33.083		2.47	3.470
3	0.313	6.958		0.063	2.178		4.083	0.137
Total	23.745			4.749	71.483			8.43

・water pressure

	H	z	H•z
in	2.576	1.507	3.882
out	-3.158	0.089	-0.281
Total	0.582		3.601

・weight of water

w = 5.153 　　　　w x = 5.864
x = 1.138 　　　　w y =
y =

・uplifting pressure

w = -4.737 　　　　w x = -12.193
x = 2.574 　　　　w y =
y =

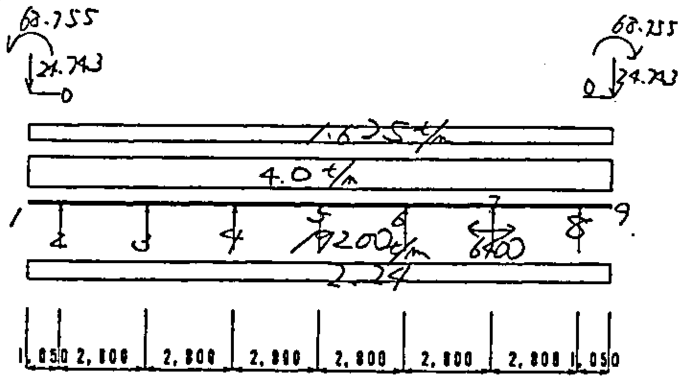
・total(right angle of stream)

V = 24.743(normal) 　　　n = 68.755(normal)
H = 0.582(normal) 　　　= 77.185(earthquake)
= 5.331(earthquake)

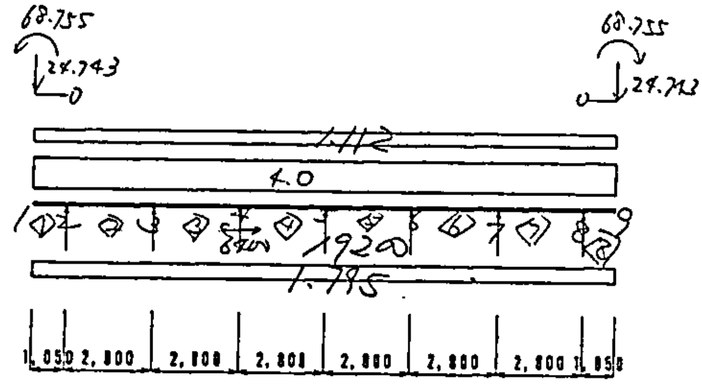
(x) combination of loads

The combination of loads for each conditions are presented in next pages.

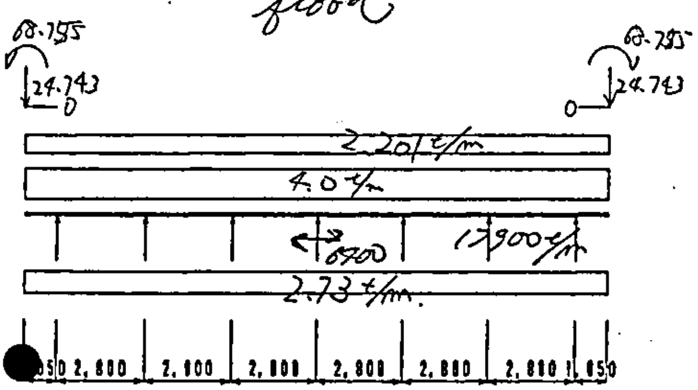
normal.



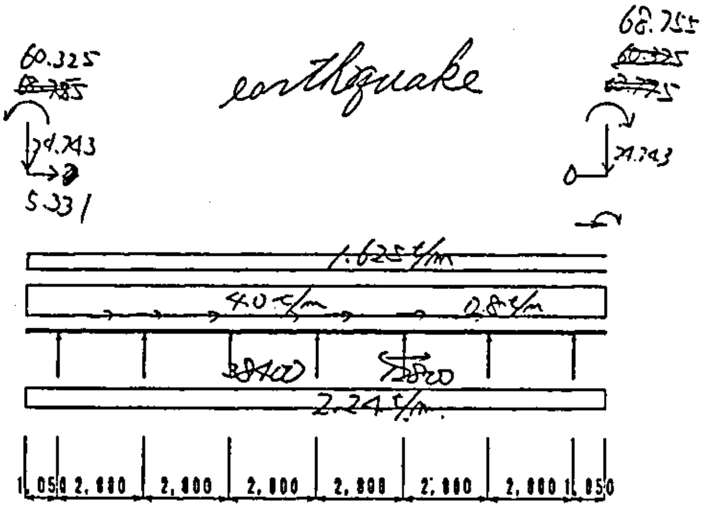
dry season



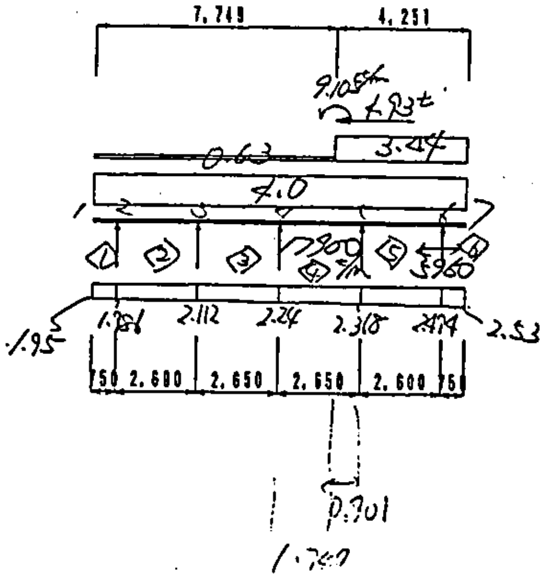
flood



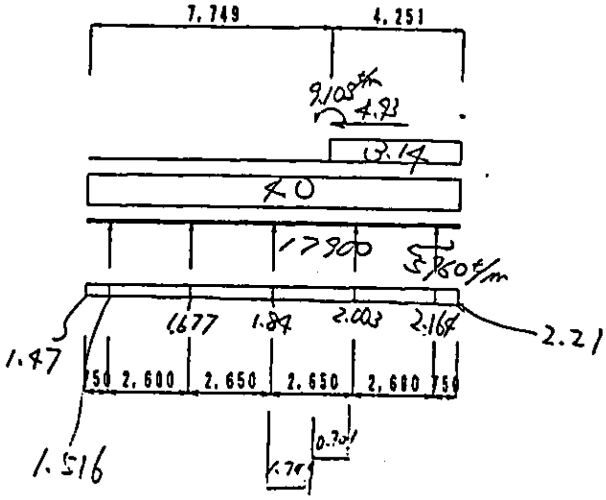
earthquake



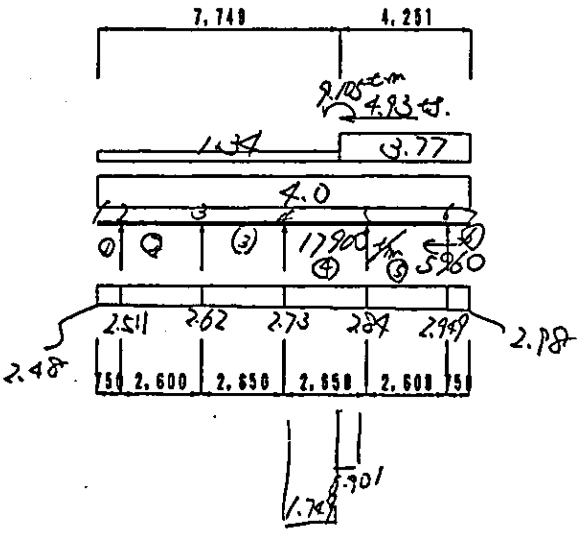
normal



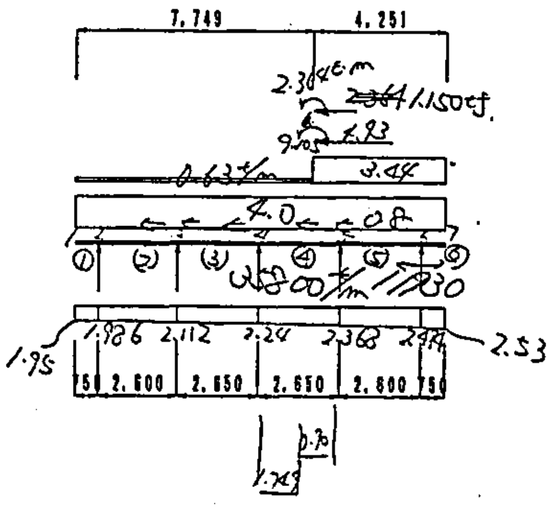
dry season



flood



earthquake



c) Result of Calculation

i) maximum moment

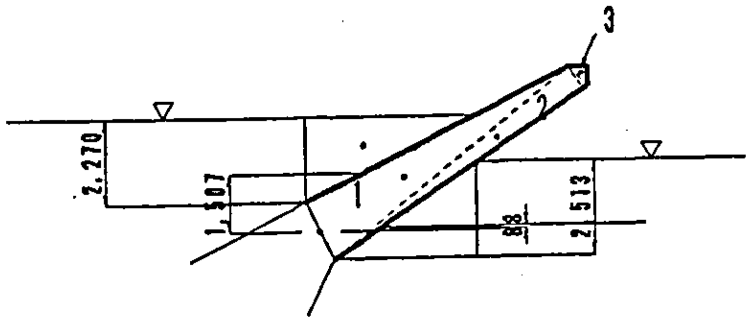
	situation	moment max	converted moment	ordar of beam
right angle	normal	-80.5	-80.5	②and⑦
	dryseason	-80.6	-80.6	②and⑦
	flood	-80.5	-80.5	②and⑦
	earthquake	-70.3	-41.7	②and⑦
stream	normal	7.7	7.7	④
	dryseason	7.3	7.3	④
	flood	8.0	8.0	④
	earthquake	8.4	5.0	④

※ S=16.5

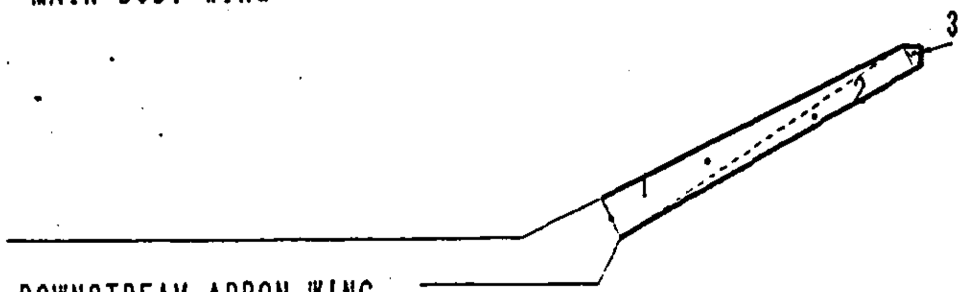
ii) Stress Calculation

( ) KEIJO	MAIN-RA KUKEI	MAIN-ST KEI	MAIN-RA KUKEI	DOWN-RA KUKEI	DOWN-ST KUKEI	DOWN-RA KUKEI	DOWN-RA KUKEI	DOWN-RA KUKEI
M (T-M)	80.500	8.000	44.800	86.200	3.500	75.900	56.900	40.500
N (TON)	.000	.000	.000	.000	.000	.000	.000	.000
S (TON)	16.500	16.500	16.500	19.100	19.100	19.100	19.100	19.100
B (CM)	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
H (CM)	150.0	150.0	150.0	130.0	120.0	120.0	120.0	120.0
D (CM)	142.5	135.0	142.5	122.5	105.0	112.5	112.5	112.5
DD (CM)	7.5	15.0	7.5	7.5	15.0	7.5	7.5	7.5
DG (CM)	9.0	16.5	9.0	9.0	16.5	9.0	9.0	9.0
BO , R (CM)	.0	.0	.0	.0	.0	.0	.0	.0
HO , RO (CM)	.0	.0	.0	.0	.0	.0	.0	.0
AC (CM**2)	15000.0	15000.0	15000.0	13000.0	12000.0	12000.0	12000.0	12000.0
AS , AS1 (CM**2)	39.49	3.91	21.50	50.27	2.20	48.45	35.81	25.12
	D25@125		D19@125	D29@125		D29@125	D22@125	D22@125
AS', AS2 (CM**2)	.00	.00	.00	.00	.00	.00	.00	.00
P , P1	.00263	.00026	.00143	.00387	.00018	.00404	.00298	.00209
P' , P2	.00000	.00000	.00000	.00000	.00000	.00000	.00000	.00000
N = ES/EC	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0
XO (CM)	35.4	11.9	27.1	35.8	7.9	33.6	29.6	25.4
K = XO/H	.236	.080	.181	.276	.066	.280	.246	.212
M/B*H**2(KG/CM2)	3.578	.356	1.991	5.101	.243	5.271	3.951	2.813
S/B*H (KG/CM2)	1.100	1.100	1.100	1.469	1.592	1.592	1.592	1.592
(C)	9.845	29.103	12.582	8.648	35.935	8.600	9.628	11.059
(S)	29.814	300.000	53.571	20.913	438.857	20.237	26.995	37.926
(Z)	1.161	1.158	1.137	1.192	1.190	1.202	1.186	1.170
SIGMA-C (KG/CM2)	35.2	10.3	25.1	44.1	8.7	45.3	38.0	31.1
SIGMA-S (KG/CM2)	1600.	1600.	1600.	1600.	1600.	1600.	1600.	1600.
TAU-MAX (KG/CM2)	1.28	1.27	1.25	1.75	1.89	1.91	1.89	1.86
TAU-AVE (KG/CM2)	1.17	1.24	1.17	1.58	1.85	1.72	1.72	1.72
SIGMA-CA(KG/CM2)	75.0	75.0	75.0	75.0	75.0	75.0	75.0	75.0
SIGMA-SA(KG/CM2)	1600.	1600.	1600.	1600.	1600.	1600.	1600.	1600.
TAU-A (KG/CM2)	3.60	3.60	3.60	3.60	3.60	3.60	3.60	3.60

2) wing wall



MAIN BODY WING



DOWNSTREAM APRON WING

a) weight of wall(per m)

	discription	$W_{(t)}$	wx	wy	H	Hx	Hy	nx	ny
1	1/2×1.677×7.826×2.5	16.405	14.673	7.337	3.281	1.467	-2.935	38.282	-2.504
								42.109	-1.230
2	1/2×8.112×0.693×2.5	7.027	6.285	3.143	1.405	0.628	-1.257	33.260	-0.176
								36.584	-0.105
3	1/2×0.5×0.5×2.5	0.313	0.280	0.140	0.063	0.028	-0.056	2.254	-0.076
								2.479	-0.045
total			21.238	10.62		2.123	-4.248	73.796	-2.756
								81.172	-1.38

earthquake $V=10.62-4.248=6.372$ $\quad V=10.62$

$H=21.238+2.123=23.361$ $\quad H=21.238$

$n=81.172-1.38=79.792$ $\quad n=71.04$

b) water pressure

	EXPRESSION	H	Hx	Hy	nx	ny
in	$1/2\times2.77^2$	2.576	1.152	-2.304	1.949	1.933
out	$1/2\times2.513^2$	-3.158	-1.412	2.825	-2.115	1.839
total			-0.26	0.521	-0.166	3.772

$V=0.521$

$H=-0.26$

$n=3.606$

c) weight of water

$$W = 1/2 \times 2.27 \times 4.54 = 5.153(tf)$$
$$V = 2.304(tf)$$
$$H = 4.609(tf)$$
$$n = 4.609 \times 2.03 - 2.304 \times 0.839 = 7.423(t \cdot m)$$

d) uplifting pressure

$$U = 1/2 \times 3.77 \times 2.513 = 4.737(tf)$$
$$V = -2.118(tf)$$
$$H = -4.237(tf)$$
$$n = -4.237 \times 2.622 - 2.118 \times 0.511 = -12.192(t \cdot m)$$

e) total amount

(a) normal

$$V = 10.62 + 0.521 + 2.304 - 2.118 = 11.327(tf)$$
$$H = 21.238 - 0.26 + 4.609 - 4.237 = 21.35(tf)$$
$$n = 71.04 + 3.606 + 7.423 - 12.192 = 69.877(t \cdot m)$$

(b) earthquake

$$V = 6.372 + 0.521 + 2.304 - 2.118 = 7.079(tf)$$
$$H = 23.361 - 0.26 + 4.609 - 4.237 = 23.473(tf)$$
$$n = 79.792 + 3.606 + 7.423 - 12.192 = 78.629(t \cdot m)$$

f) necessary amount of re-bar

$$A_s = 26.74(cm^2/m)$$

$\therefore$ D22 pitch 125mm

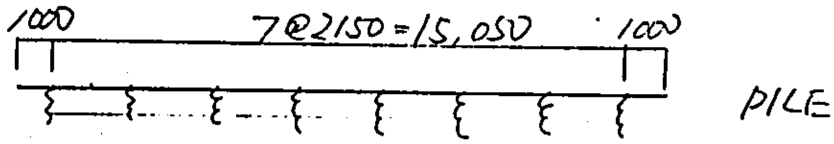
(b) Downstream Apron

(i) Apron Slab

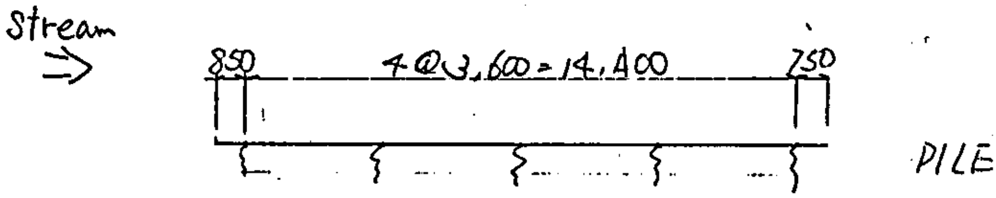
1) beam

a) dimension

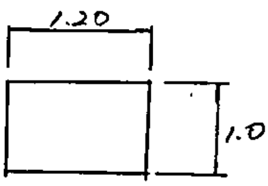
• Cross section



• Flow Direction



b) crossing sectional area



$A = 1.2 \times 1.0 = 1.2(m^2)$

c) crossing sectional quadratic moment

$I = 1.0 \times 1.2^3/12 = 0.144(m^4)$

2) load (per m)

a) bottom beam

$w_1 = 1.2 \times 2.5 = 3.0(t/m)$
$H_{w1} = 3.0 \times 0.2 = 0.6(t/m)$

b) spring of pile

• case of normal,flood,dry and just constract

k_V =13,921(tf/m)(right angle of stream)
=23,310(tf/m)(direction of stream)
k_H =1/3× k_V

- case of earthquake

k_V =27,842(tf/m)(right angle of stream)
=46,620(tf/m)(direction of stream)
k_H =1/3× k_V

c) weight of water

- case of normal and earthquake
w_2 =0.63×1.00=0.63(t/m)
- case of flood
w_2 =1.34(t/m)

d) earth pressure in earthquake

H_2 =2.376(tf)
n_{H2}=4.545(t•m)
H_3 =3.761(tf)
n_{H3}=9.479(t•m)

e) water pressure

H_4 =1/2×0.63²=0.198(tf)(normal,earthquake)
n_{H4}=0.160(t•m)
H_5 =1/2×1.34²=0.898(tf)(flood)
n_{H5}=0.940(t•m)

f) water pressure in earthquake

H_6 =7/12×1.0×0.2×0.63²=0.046
n_{H6}=0.039(t•m)

g) uplifting pressure of water

- right angle of stream

u_1 =0.76(t/m)(dryseason)
u_2 =1.31(t/m)(normal)
u_3 =1.925(t/m)(flood)

• direction of stream

u_4 = 0.31(t/m)(dryseason)
= 1.21
u_5 = 0.91(t/m)(normal)
= 1.71
u_6 = 2.58(t/m)(flood)
= 2.27

h) load of beam s ends

• weight of wall

	W	x	y	H	z	W•x	W•y	H•z
1	12.961	2.546		2.592	1.492	32.999		3.867
2	7.732	5.346		1.546	2.642	41.335		4.085
3	0.313	7.971		0.063	4.308	2.495		0.271
total	21.006			4.201		76.829		8.223

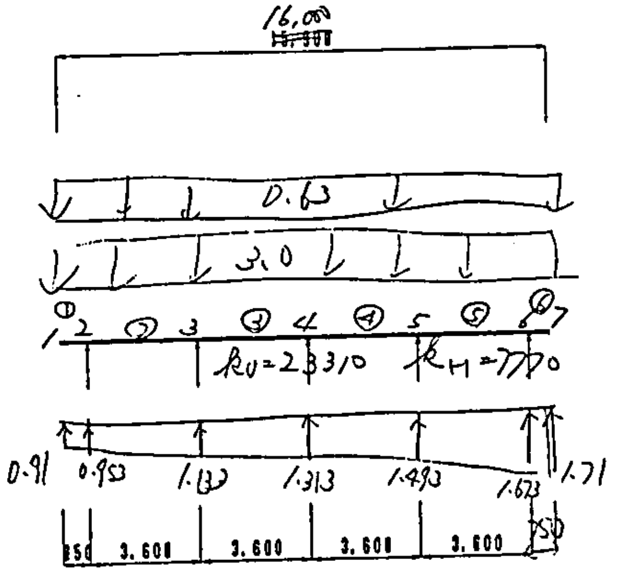
V = 21.006(normal)
= 0(earthquake)
H = 4.021
n = 76.829(normal)
= 85.052(earthquake)

i) combination of loads

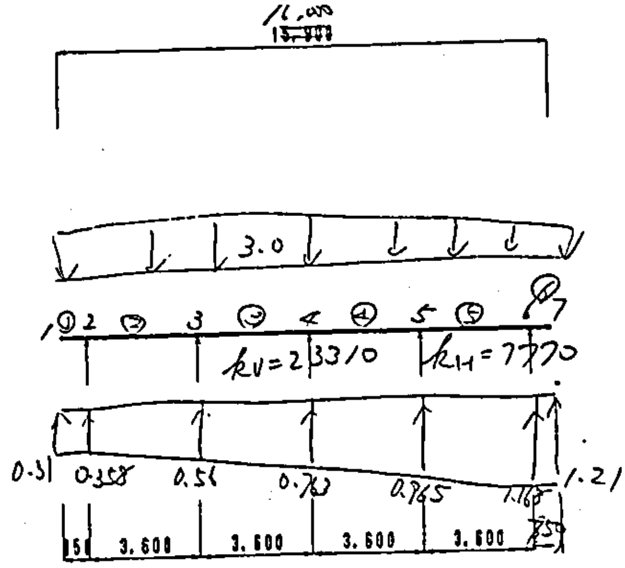
The combination of load by each conditions are shown in following pages.

CROSS SECTION

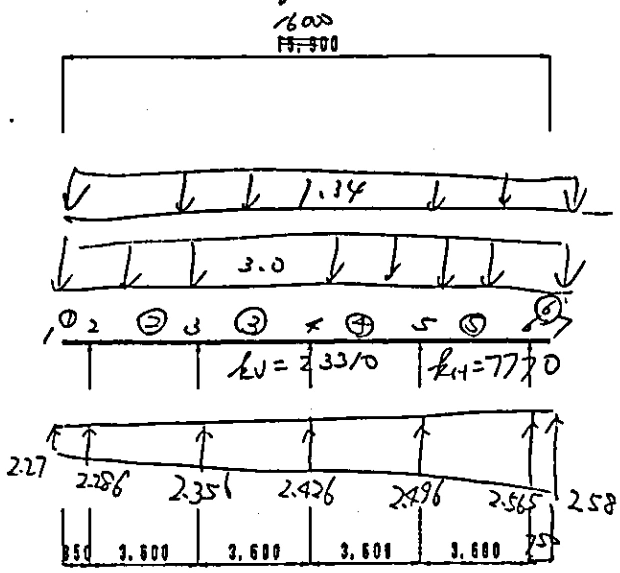
normal



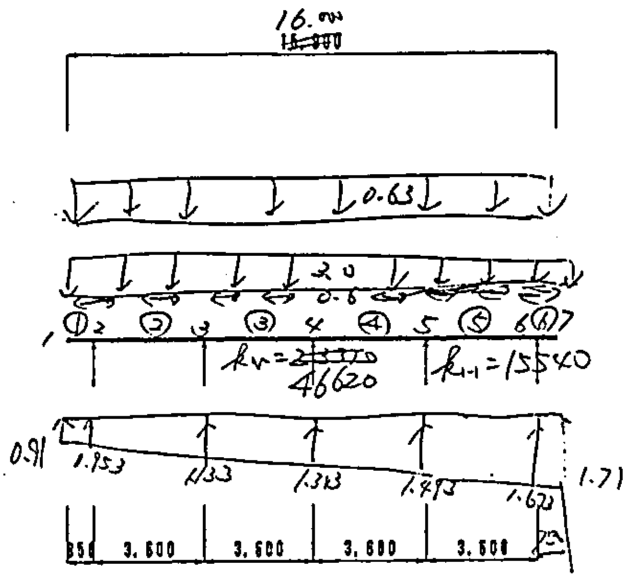
dry season



flood

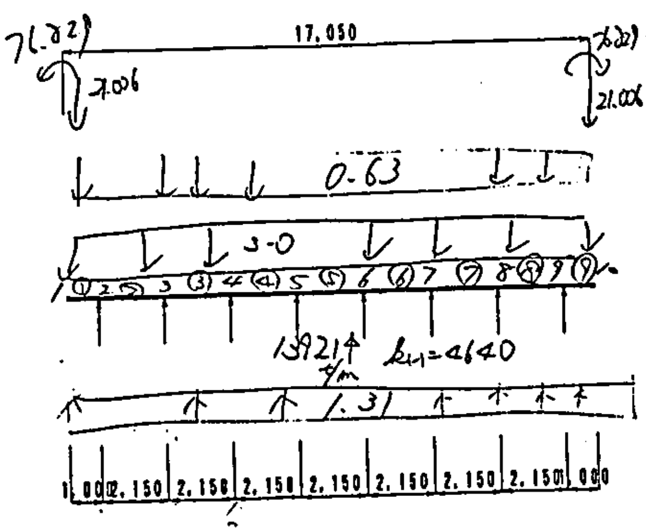


earthquake

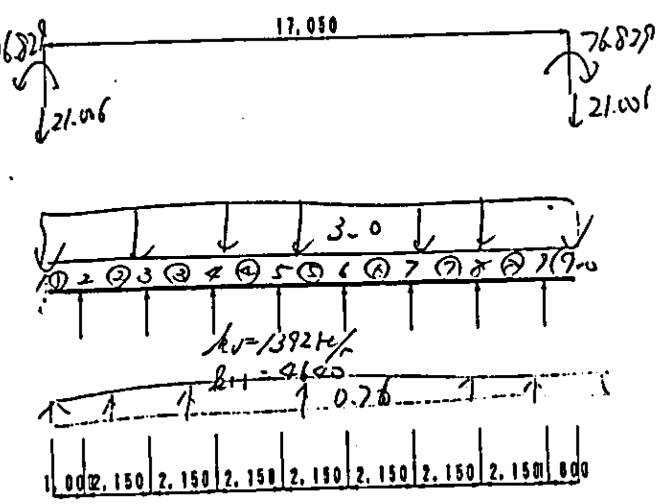


Flow Direction

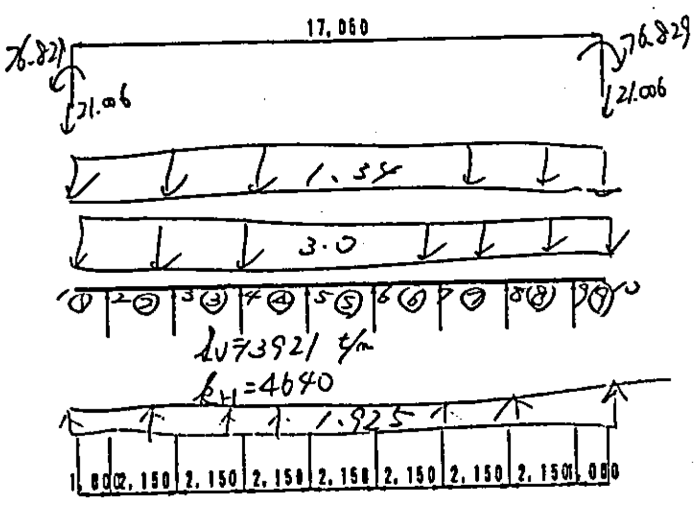
normal



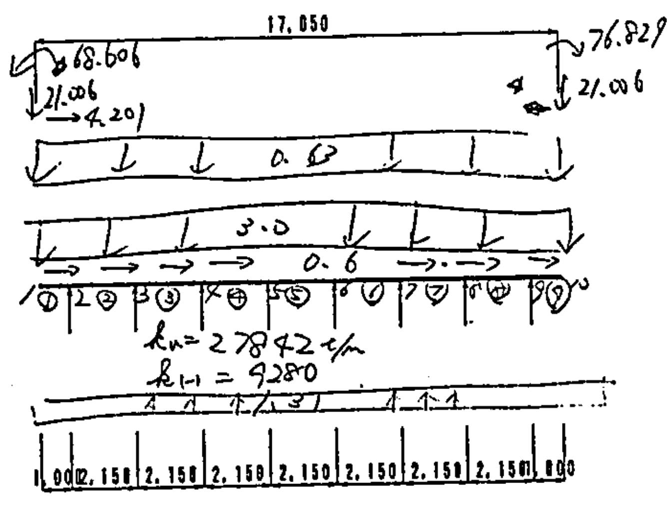
dry season



flood



earthquake



3) result of calculation

a) maximum moment

	situation	moment max	converted moment	order of beam
right angle	normal	86.2	86.2	②and⑧
	dryseason	86.2	86.2	②and⑧
	flood	86.1	86.1	②and⑧
	earthquake	80.1	47.5	②and⑧
stream	normal	3.5	3.5	②
	dryseason	3.3	3.3	②
	flood	2.6	2.6	②
	earthquake	3.0	1.8	②

※ S = 19.1

b) Stress Calculation

The calculation sheet of stress calculation are attached in following pages.

( ) KEIJO	MAIN-RA KUKEI	MAIN-ST KUKEI	MAIN-RA KUKEI	DOWN-RA KUKEI	DOWN-ST KUKEI	DOWN-RA KUKEI	DOWN-RA KUKEI	DOWN-RA KUKEI	DOWN-RA KUKEI
M (T-M)	80.500	8.000	44.800	86.200	3.500	75.900	56.900	40.500	28.100
N (TON)	.000	.000	.000	.000	.000	.000	.000	.000	.000
S (TON)	16.500	16.500	16.500	19.100	19.100	19.100	19.100	19.100	19.100
B (CM)	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
H (CM)	150.0	150.0	150.0	130.0	120.0	120.0	120.0	120.0	120.0
D (CM)	142.5	135.0	142.5	122.5	105.0	112.5	112.5	112.5	112.5
DD (CM)	7.5	15.0	7.5	7.5	15.0	7.5	7.5	7.5	7.5
DG (CM)	9.0	16.5	9.0	9.0	16.5	9.0	9.0	9.0	9.0
BO , R (CM)	.0	.0	.0	.0	.0	.0	.0	.0	.0
HO , RO (CM)	.0	.0	.0	.0	.0	.0	.0	.0	.0
AC (CM**2)	15000.0	15000.0	15000.0	13000.0	12000.0	12000.0	12000.0	12000.0	12000.0
AS , AS1 (CM**2)	39.49	3.91	21.50	50.27	2.20	48.45	35.81	25.12	17.20
	D25@125		D19@125	D29@125		D29@125	D22@125	D22@125	D19@125
AS', AS2 (CM**2)	.00	.00	.00	.00	.00	.00	.00	.00	.00
P , P1	.00263	.00026	.00143	.00387	.00018	.00404	.00298	.00209	.00143
P' , P2	.00000	.00000	.00000	.00000	.00000	.00000	.00000	.00000	.00000
N = ES/EC	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0
XO (CM)	35.4	11.9	27.1	35.8	7.9	33.6	29.6	25.4	21.5
K = XO/H	.236	.080	.181	.276	.066	.280	.246	.212	.179
M/B*H**2(KG/CM2)	3.578	.356	1.991	5.101	.243	5.271	3.951	2.813	1.951
S/B*H (KG/CM2)	1.100	1.100	1.100	1.469	1.592	1.592	1.592	1.592	1.592
(C)	9.845	29.103	12.582	8.648	35.935	8.600	9.628	11.059	12.907
(S)	29.814	300.000	53.571	20.913	438.857	20.237	26.995	37.926	54.662
(Z)	1.161	1.158	1.137	1.192	1.190	1.202	1.186	1.170	1.156
SIGMA-C (KG/CM2)	35.2	10.3	25.1	44.1	8.7	45.3	38.0	31.1	25.2
SIGMA-S (KG/CM2)	1600.	1600.	1600.	1600.	1600.	1600.	1600.	1600.	1600.
TAU-MAX (KG/CM2)	1.28	1.27	1.25	1.75	1.89	1.91	1.89	1.86	1.84
TAU-AVE (KG/CM2)	1.17	1.24	1.17	1.58	1.85	1.72	1.72	1.72	1.72
SIGMA-CA(KG/CM2)	75.0	75.0	75.0	75.0	75.0	75.0	75.0	75.0	75.0
SIGMA-SA(KG/CM2)	1600.	1600.	1600.	1600.	1600.	1600.	1600.	1600.	1600.
TAU-A (KG/CM2)	3.60	3.60	3.60	3.60	3.60	3.60	3.60	3.60	3.60

(ii) wing wall

1) weight of wall (per m)

	description	$W_{(tf)}$	Wx	Wy	$H_{(tf)}$	Hx	Hy	nx	ny
1	1/2×8.832×1.174×2.5	12.961	11.593	5.796	2.592	1.159	-2.318	34.130	-1.136
								37.542	-0.682
2	1/2×9.07×0.682×2.5	7.732	6.916	3.458	1.546	0.691	-1.383	41.240	0.097
								45.361	0.069
3	1/2×0.5×0.5×2.5	0.313	0.280	0.140	0.063	0.028	-0.056	2.536	-0.040
								2.789	-0.024
total			18.789	9.394		1.878	-3.757	77.906	-1.079
								85.691	-0.637

normal V = 9.394
H = 18.789
n = 76.827

earthquake V = 5.637
earthquake H = 20.667
earthquake n = 85.054

2) Total amount

a) normal

V = 9.394(tf)
H = 18.789(tf)
n = 76.827(t・m)

b) earthquake

V = 5.637(tf)
H = 20.667(tf)
n = 85.054(t・m)

3) required amount of re-bar

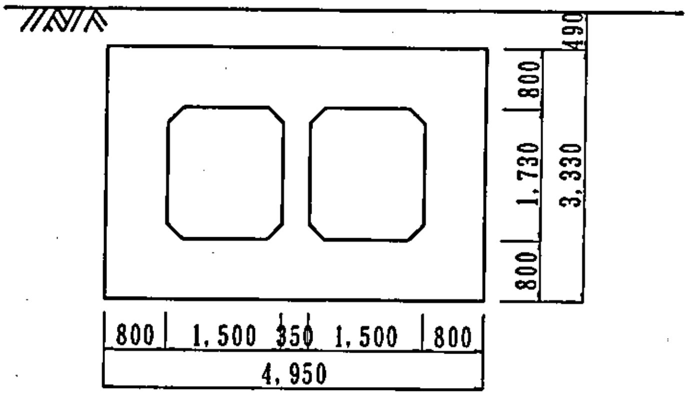
$A_s = 47.85(cm^2/m)$

∴ D29 pitch 125mm

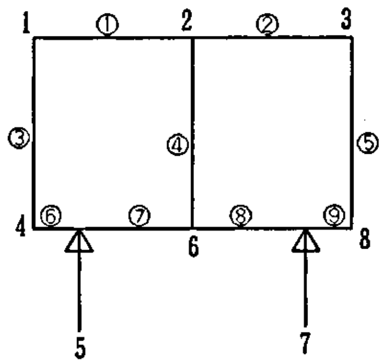
(c) Inlet Structure

(i) Box culvert section

1) Dimension



2) Frame of Rahmen



3) Members

1,2,3,5,6,7,8,9 : $A = 0.8\ m^2$
: $I = 0.0427\ m^4$
4 : $A = 0.35\ m^2$
: $I = 0.0036\ m^4$

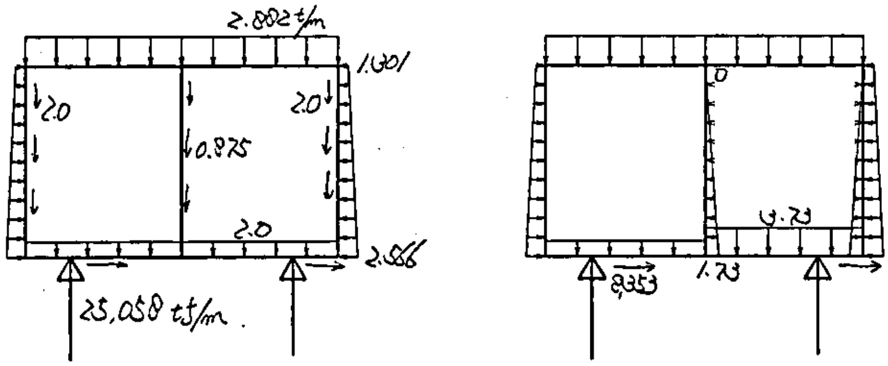
4) Rahmen Analysis

Self weight	:	2.875 tf/m
Soil weight	:	0.882 tf/m
Actve load	:	1.0 tf/m
Earth Pressure	:	3.867 tf/m
Reaction of Pile	:	Kv = 25.058 tf/m/m
		Kh = 1/3 kv = 8.353 tf/m/m
Water Pressure	:	Pw = 1.73 t/m
Water Weight	:	Ww = 1.73 t/m

Result of the rahmen analysis is summarized folloing figures.

Gate Standard

Frame	Left	Right	Status	Left Axial (t)	Left Shear (t)	Left Moment(tm)	Right Axial (t)	Right Shear (t)	Right Moment(tm)
1	1	2	Normal	2.108	3.856	-1.57	2.108	2.124	0.227
			Flood	2.18	3.873	-1.631	2.18	2.107	0.201
1	Center		Normal	2.108	0.268	0.997			
			Flood	2.18	0.285	0.957			
2	2	3	Normal	2.109	2.126	0.227	2.109	3.854	-1.567
			Flood	1.515	1.999	0.57	1.515	3.981	-1.486
2	Center		Normal	2.109	0.266	0.998			
			Flood	1.515	0.205	1.256			
3	4	1	Normal	8.916	2.771	-1.739	3.856	2.108	-1.57
			Flood	8.933	2.699	-1.619	3.873	2.18	-1.631
3	Center		Normal	6.386	0.066	-0.112			
			Flood	6.403	0.137	-0.082			
4	6	2	Normal	6.463	0.001	0.001	4.25	0.001	0.001
			Flood	6.32	1.523	0.533	4.106	0.665	0.37
4	Center		Normal						
			Flood	5.213	0.118	-0.241			
5	8	3	Normal	8.914	2.783	1.745	3.854	2.109	1.567
			Flood	9.041	1.189	1.322	3.981	1.515	1.486
5	Center		Normal	6.384	0.063	0.109			
			Flood	6.511	0.016	0.549			
6	4	5	Normal	2.771	8.196	1.739	2.771	10.116	-3.971
			Flood	2.699	8.933	1.619	2.699	10.133	-4.101
7	5	6	Normal	2.759	6.185	-3.971	2.759	3.235	2.976
			Flood	2.693	6.7	-4.101	2.693	3.75	3.607
8	6	7	Normal	2.759	3.229	2.975	2.759	6.179	-3.963
			Flood	1.17	2.57	3.074	1.17	8.071	-4.774
9	7	8	Normal	2.783	10.114	-3.963	2.783	8.914	1.745
			Flood	1.189	11.279	-4.774	1.189	9.041	1.322



5) Stress calculation

Calculation of Required Re-Bar

Type of Section			1LEFT-1	1LEFT-2
Dimension	B H	cm cm	100.0 80.0	100.0 80.0
Force	M N	tf·m tf	1.631 2.180	0.997 2.108
Required Re-Bar	d1 As1	cm cm²	71.0 0.731	71.0 0.172
	Total	cm²	0.731	0.172
Stress Intensity	σ c σ s	kgf/cm² kgf/cm²	10.5 1600.0	8.8 1600.0
Allowable Stress	σ ca σ sa	kgf/cm² kgf/cm²	70.0 1600.0	70.0 1600.0
Neutral Axis	x	cm	6.370	5.418
Young s Modules (n = Es/Ec)			15.0	15.0

X = Distance from the Edge of the Compressive Side

Calculation of Required Re-Bar

Type of Section			2RIGHT	2CENTER
Dimension	B H	cm cm	100.0 80.0	100.0 80.0
Force	M N	tf·m tf	1.567 2.109	1.300 1.515
Required Re-Bar	d1 As1	cm cm²	71.0 0.696	71.0 0.654
	Total	cm²	0.696	0.654
Stress Intensity	σ c σ s	kgf/cm² kgf/cm²	10.3 1600.0	9.1 1600.0
Allowable Stress	σ ca σ sa	kgf/cm² kgf/cm²	70.0 1600.0	70.0 1600.0
Neutral Axis	x	cm	6.254	5.601
Young s Modules (n = Es/Ec)			15.0	15.0

X = Distance from the Edge of the Compressive Side

Gate Standard - 3

Calculation of Required Re-Bar

Type of Section				3LEFT	3RIGHT
Dimension	B	cm		100.0	100.0
	H	cm		80.0	80.0
Force	M	tf·m		1.739	1.631
	N	tf		8.916	3.873
Required Re-Bar	d1	cm		71.0	71.0
	As1	cm²		0.000	0.155
	Total	cm²		0.000	0.155
Stress Intensity	σc	kgf/cm²		0.0	11.7
	σs	kgf/cm²		0.0	1600.0
Allowable Stress	σca	kgf/cm²		70.0	70.0
	σsa	kgf/cm²		1600.0	1600.0
Neutral Axis	x	cm		10.090	7.036
Young s Modules (n = Es/Ec)				15.0	15.0

X = Distance from the Edge of the Compressive Side

Calculation of Required Re-Bar

Type of Section			4LEFT	4RIGHT	4CENTER
Dimension	B	cm	100.0	100.0	100.0
	H	cm	35.0	35.0	35.0
Force	M	tf·m	0.533	0.370	0.241
	N	tf	6.320	4.106	5.213
Required Re-Bar	d1	cm	26.0	26.0	26.0
	As1	cm²	0.000	0.000	0.000
	Total	cm²	0.000	0.000	0.000
Stress Intensity	σc	kgf/cm²	0.0	0.0	0.0
	σs	kgf/cm²	0.0	0.0	0.0
Allowable Stress	σca	kgf/cm²	70.0	70.0	70.0
	σsa	kgf/cm²	1600.0	1600.0	1600.0
Neutral Axis	x	cm	4.989	4.105	4.577
Young s Modules (n = Es/Ec)			15.0	15.0	15.0

X = Distance from the Edge of the Compressive Side

Gate standard-5

Calculation of Required Re-Bar

Type of Section				5LEFT	5RIGHT
Dimension		B H	cm cm	100.0 80.0	100.0 80.0
Force		M N	tf·m tf	1.745 8.914	1.567 3.854
Required Re-Bar		d1 As1	cm cm²	71.0 0.000	71.0 0.103
	Total		cm²	0.000	0.103
Stress Intensity		σ c σ s	kgf/cm² kgf/cm²	0.0 0.0	11.6 1600.0
Allowable Stress		σ ca σ sa	kgf/cm² kgf/cm²	70.0 1600.0	70.0 1600.0
Neutral Axis		x	cm	10.090	6.950
Young s Modules (n = Es/Ec)				15.0	15.0

X = Distance from the Edge of the Compressive Side

Calculation of Required Re-Bar

Type of Section				6LEFT	6RIGHT	7RIGHT	8LEFT
Dimension		B H	cm cm	100.0 80.0	100.0 80.0	100.0 80.0	100.0 80.0
Force		M N	tf·m tf	1.739 2.771	4.101 2.699	3.607 2.693	3.074 1.170
Required Re-Bar		d1 As1	cm cm²	63.5 0.703	71.0 2.856	63.5 2.688	63.5 2.698
	Total		cm²	0.703	2.856	2.688	2.698
Stress Intensity		σ c σ s	kgf/cm² kgf/cm²	12.1 1600.0	15.8 1600.0	16.5 1600.0	14.5 1600.0
Allowable Stress		σ ca σ sa	kgf/cm² kgf/cm²	70.0 1600.0	70.0 1600.0	70.0 1600.0	70.0 1600.0
Neutral Axis		x	cm	6.456	9.178	8.488	7.591
Young s Modules (n = Es/Ec)				15.0	15.0	15.0	15.0

X = Distance from the Edge of the Compressive Side

Gate - Standard/1

Calculation of Required Re-Bar

Type of Section			6LEFT	6RIGHT	7RIGHT	8LEFT
Dimension	B	cm	100.0	100.0	100.0	100.0
	H	cm	80.0	80.0	80.0	80.0
Force	M	tf·m	1.739	4.101	3.607	3.074
	N	tf	2.771	2.699	2.693	1.170
	S	tf	8.196	10.133	3.750	2.570
Required Re-Bar	d1	cm	63.5	71.0	63.5	63.5
	As1	cm^2	7.944 (4-D16)	7.944 (4-D16)	7.944 (4-D16)	7.944 (4-D16)
	Total	cm^2	7.944	7.944	7.944	7.944
Stress Intensity	σc	kgf/cm^2	4.6	10.3	10.5	9.1
	σs	kgf/cm^2	174.5	599.8	567.0	562.9
	$\sigma s'$	kgf/cm^2	0.0	0.0	0.0	0.0
	τ	kgf/cm^2	1.29	1.43	0.59	0.40
Allowable Stress	σca	kgf/cm^2	70.0	70.0	70.0	70.0
	σsa	kgf/cm^2	1600.0	1600.0	1600.0	1600.0
	τa	kgf/cm^2	3.60	3.60	3.60	3.60
Neutral Axis	x	cm	18.027	14.519	13.762	12.395
Young s Modules (n = Es/Ec)			15.0	15.0	15.0	15.0

X = Distance from the Edge of the Compressive Side

Gate - Standard - 7

Calculation of Required Re-Bar

Type of Section			8RIGHT	9RIGHT
Dimension	B	cm	100.0	100.0
	H	cm	80.0	80.0
Force	M	tf·m	4.774	1.745
	N	tf	1.170	2.783
Required Re-Bar	d1	cm	71.0	63.5
	As1	cm²	3.995	0.704
	Total	cm²	3.995	0.704
Stress Intensity	σ c	kgf/cm²	16.2	12.1
	σ s	kgf/cm²	1600.0	1600.0
Allowable Stress	σ ca	kgf/cm²	70.0	70.0
	σ sa	kgf/cm²	1600.0	1600.0
Neutral Axis	x	cm	9.355	6.468
Young s Modules (n = Es/Ec)			15.0	15.0

X = Distance from the Edge of the Compressive Side

Calculation of Required Re-Bar

Type of Section			8RIGHT	9RIGHT
Dimension	B	cm	100.0	100.0
	H	cm	80.0	80.0
Force	M	tf·m	4.774	1.745
	N	tf	1.170	2.783
	S	tf	8.071	11.279
Required Re-Bar	d1	cm	71.0	63.5
	As1	cm²	7.944	7.944
			(4-D16)	(4-D16)
	Total	cm²	7.944	7.944
Stress Intensity	σ c	kgf/cm²	12.0	4.6
	σ s	kgf/cm²	821.6	175.0
	σ s'	kgf/cm²	0.0	0.0
	τ	kgf/cm²	1.14	1.78
Allowable Stress	σ ca	kgf/cm²	70.0	70.0
	σ sa	kgf/cm²	1600.0	1600.0
	τ a	kgf/cm²	3.60	3.60
Neutral Axis	x	cm	12.791	18.035
Young s Modules (n = Es/Ec)			15.0	15.0

X = Distance from the Edge of the Compressive Side

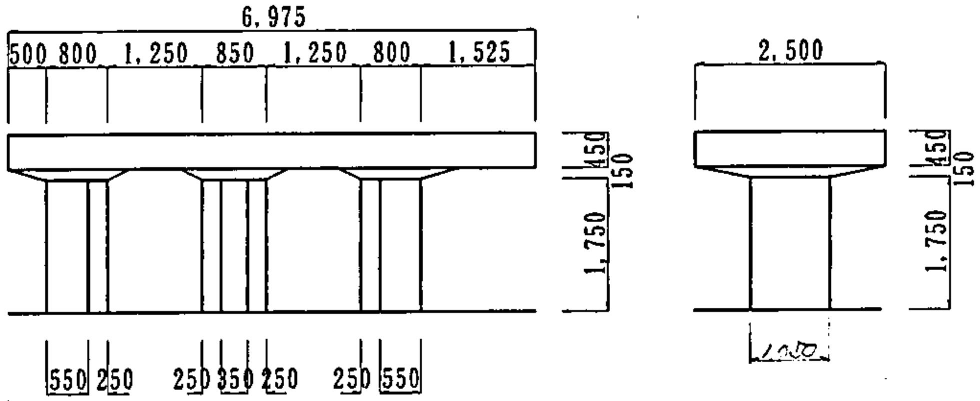
(ii) Gate Post

The gate post is regarded as a portal bracing.

1) Load Combination

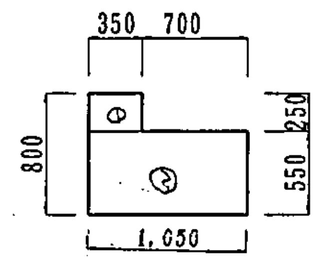
Case	Condition	Load	応力度の割増
1	Normal	self weight, gate, load by pepol	0
2 · 3	Nolmal+Temp.	normal condition + (± 1 5 ℃)	1 5 %
4	Normal+Wind	normal condition + wind	0
5 · 6	Wind+Temp.	normal condition + wind + (± 1 5 ℃)	1 5 %
7	Earthquake	self weight, gate	5 0 %
8 · 9	Earthquake+Temp.	self weight, gate + (± 1 5 ℃)	6 5 %

2) Dimension



3) Members

a) End Columun



	A(m2)	y (m)	A · y (m2)	A · y²(m3)	I (m4)
①	0.35×0.25 = 0.0875	0.125	0.0109	0.0014	0.0005
②	1.05×0.55 = 0.5775	0.275	0.1588	0.0437	0.0146
Total	0.645		0.1697	0.0451	0.0196

$$e=(\Sigma A y/\Sigma A)=0.1697/0.645=0.2631$$

Moment of inertia of area

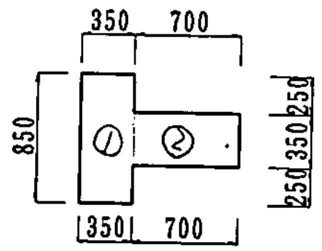
$$I_1=\Sigma A y^2+\Sigma I-\Sigma A \cdot e^2$$

$= 0.0451 + 0.0196 - 0.645 \times 0.2631^2 = 0.0201\ m^4$

Sectional Area

$A_1 = 0.645\ m^2$

b) Center Columun



	A(m2)	I (m4)
①	$0.35 \times 0.85 = 0.2975$	0.0179
②	$0.70 \times 0.35 = 0.245$	0.0025
Total	0.5425	0.0204

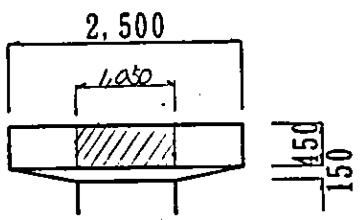
Moment of incrtia of area

$I_2 = 0.0451 + 0.0196 - 0.645 \times 0.2631^2 = 0.0204$ m4

Sectional Area

$A2 = 0.5425\ m^2$

c) Operation Deck



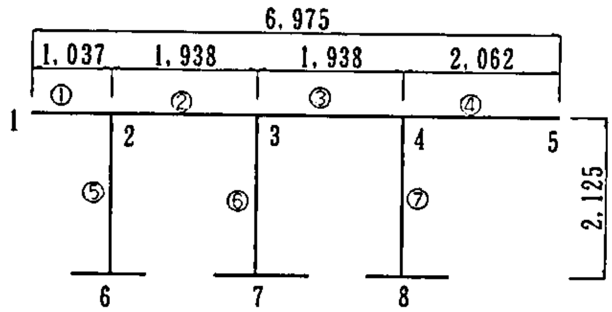
Moment of inertia of area

$I3 = 1.05 \times 0.45^3 \times 1/12 = 0.0080$ m4

Sectional Area

$A3 = 1.05 \times 0.45 = 0.4725$ m2

Frame for rahmen analysis



4) Rahmen Analysis

Gate-gate Rahmen

				Left			Right		
Frame	Left	Right	Status	Axial (t)	Shear (t)	Moment(tm)	Axial (t)	Shear (t)	Moment(tm)
1	1	2	Normal	0	0	0	0	3.695	-1.916
			N+wind	0	0	0	0	4.083	-2.318
			Earthquake	0	0	0	0.258	2.917	-1.513
			N+15	0	0	0	0	3.695	-1.916
			N-15	0	0	0	0	3.695	-1.916
			N+w+15	0	0.388	0	0	4.083	-2.318
			N+w-15	0	0.388	0	0	4.083	-2.318
			E+15	0	0	0	0.258	2.917	-1.513
			E-15	0	0	0	0.258	2.917	-1.513
2	2	3	Normal	-0.958	4.411	-1.238	0.958	5.454	-2.193
			N+wind	-1.082	4.384	-1.241	-1.082	5.481	-2.248
			Earthquake	-1.381	2.271	-0.023	-0.649	4.426	-2.088
			N+15	6.618	8.089	-6.165	6.618	1.776	0.009
			N-15	-8.533	0.732	3.689	-8.533	9.133	-4.394
			N+w+15	6.494	8.062	-6.168	6.494	1.803	-0.046
			N+w-15	-8.657	0.706	3.686	-8.657	9.159	-4.449
			E+15	6.195	5.949	-4.949	6.926	0.748	0.114
			E-15	-8.956	1.408	4.904	-8.225	8.104	-4.289
2	Center		Normal	-0.958	0.958	1.364			
			N+wind	-1.082	0.931	1.335			
			Earthquake	-1.188	0.09	0.892			
			N+15	6.618	0.296	0.44			
			N-15	-8.533	0.042	3.764			
			N+w+15	6.494	0.269	0.4			
			N+w-15	-8.657	0.015	3.756			
			E+15	6.878	0.203	0.206			
			E-15						
3	3	4	Normal	-2.797	2.463	-0.311	-2.797	7.402	-4.858
			N+wind	-2.875	2.358	-0.215	-2.875	7.507	-4.964
			Earthquake	-3.246	0.597	0.715	-2.514	6.1	4.517
			N+15	4.778	1.216	1.891	4.778	11.081	-9.784
			N-15	-10.373	6.141	-2.513	-10.373	3.724	0.069
			N+w+15	4.701	1.32	1.987	4.701	11.185	-9.891
			N+w-15	-10.45	6.037	-2.417	-10.45	3.829	-0.037
			E+15	4.33	3.082	2.917	5.061	9.778	-9.443
			E-15	-10.821	4.275	-1.487	-10.09	2.421	0.41
3	Center		Normal	-2.797	0.299	0.528			
			N+wind	-2.875	0.287	0.554			
			Earthquake	-3.197	0.052	0.778			
			N+15	-10.373	0.962	1.885			
			N-15						
			N+w+15						
			N+w-15	-10.45	1.067	1.86			
			E+15						
			E-15	-10.283	0.241	1.442			
4	4	5	Normal	0	8.347	-9.637	0	0	0
			N+wind	0	8.347	-9.637	0	0	0
			Earthquake	-0.713	6.8	-8.042	0	0	0
			N+15	0	8.347	-9.637	0	0	0
			N-15	0	8.347	-9.637	0	0	0
			N+w+15	0	8.347	-9.637	0	0	0
			N+w-15	0	8.347	-9.637	0	0	0
			E+15	-0.713	6.8	-8.042	0	0	0
			E-15	-0.713	6.8	-8.402	0	0	0

Gate-gate Rahmen

Frame	Left	Right	Status	Left Axial (t)	Left Shear (t)	Left Moment(tm)	Right Axial (t)	Right Shear (t)	Right Moment(tm)
5	6	2	Normal	12.426	0.958	-1.357	8.106	0.958	0.678
			N+wind	12.787	1.751	-1.933	8.467	1.082	1.077
			Earthquake	9.508	2.504	-2.912	5.188	1.639	1.49
			N+15	16.104	6.618	9.813	11.784	6.618	-4.249
			N-15	8747	8.533	-12.528	4.427	8.533	5.605
			N+w+15	16.465	5.824	9.238	12.145	6.494	-3.849
			N+w-15	9.109	9.327	-13.104	4.788	8.657	6.004
			E+15	13.186	5.072	8.259	8.866	5.936	-3.437
			E-15	5.829	10.079	-14.083	1.509	9.214	6.417
6	7	3	Normal	12.373	1.84	-2.028	7.917	1.84	1.881
			N+wind	12.295	2.462	-2.489	7.839	1.793	2.033
			Earthquake	9.479	3.487	-3.661	5.023	2.596	2.083
			N+15	5.016	1.84	-2.028	0.56	1.84	1.881
			N-15	19.73	1.84	-2.028	15.274	1.84	1.881
			N+w+15	4.939	2.462	-2.489	0.482	1.793	2.033
			N+w-15	19.652	2.462	-2.489	15.196	1.793	2.033
			E+15	2.122	3.487	-3.661	-2.334	2.596	2.803
			E-15	16.836	3.487	-3.661	12.38	2.596	2.803
7	8	4	Normal	20.069	2.797	1.165	15.749	2.797	-4.779
			N+wind	20.174	2.205	0.725	15.854	2.875	4.672
			Earthquake	17.22	0.936	-0.618	12.9	1.801	-3.526
			N+15	23.748	4.778	-10.006	19.428	4.778	0.148
			N-15	16.391	10.373	12.336	12.071	10.373	-9.706
			N+w+15	23.852	5.37	-10.446	19.532	4.701	0.255
			N+w-15	16.496	9.781	11.896	12.175	10.45	-9.599
			E+15	20.899	6.64	-11.789	16.579	5.775	1.401
			E-15	13.542	8.511	10.553	9.222	9.376	-8.453

5) Stress Calculation

a) Distribution of Load

<u>End Columun</u>

	Axis force, Shear force		Bending moment	
	Area(m2)	Ratio	Inertia	ratio
①	0.28	0.421	0.0149	0.606
②	0.385	0.579	0.0097	0.394
Total	0.665	1.000	0.0246	1.000

<u>Center Columun</u>

	Axis force, Shear force		Bending moment	
	Area(m2)	Ratio	Inertia	ratio
①	0.2975	0.548	0.0179	0.877
②	0.245	0.452	0.0025	0.123
Total	0.5425	1.000	0.0204	1.000

<u>Operation Deck</u>

No distribution

b) Result of calculation

Calculation of Required Re-Bar

Type of Section			1-RIGHT	2-RIGHT	2-CENTER	3-LEFT
Dimension	B	cm	105.0	105.0	105.0	105.0
	H	cm	45.0	45.0	45.0	45.0
Force	M	tf·m	2.318	3.869	3.300	2.185
	N	tf	0.000	-7.528	-7.420	-9.020
	S	tf	4.038	7.964	0.958	5.340
Required Re-Bar	d1	cm	38.5	38.5	38.5	38.5
	As1	cm²	13.902	13.902	13.902	13.902
			(7-D16)	(7-D16)	(7-D16)	(7-D16)
	Total	cm²	13.902	13.902	13.902	13.902
Stress Intensity	σc	kgf/cm²	12.0	18.2	15.1	7.8
	σs	kgf/cm²	476.6	1075.3	955.5	793.6
	$\sigma s'$	kgf/cm²	0.0	0.0	0.0	0.0
	τ	kgf/cm²	1.00	1.97	0.24	1.32
Allowable Stress	σca	kgf/cm²	70.0	70.0	70.0	70.0
	σsa	kgf/cm²	1800.0	1800.0	1800.0	1800.0
	τa	kgf/cm²	3.60	3.60	3.60	3.60
Neutral Axis	x	cm	10.539	7.782	7.386	4.931
Young s Modules (n = Es/Ec)			15.0	15.0	15.0	15.0

X = Distance from the Edge of the Compressive Side

Calculation of Required Re-Bar

Type of Section			3-RIGHT	4-LEFT	5-L-1	5-L-2
Dimension	B	cm	105.0	105.0	35.0	70.0
	H	cm	45.0	45.0	80.0	55.0
Force	M	tf·m	8.601	9.637	6.138	3.991
	N	tf	4.087	0.000	2.964	4.077
	S	tf	9.726	8.347	3.927	4.696
Required Re-Bar	d1	cm	38.5	38.5	11.0	11.0
	As1	cm^2	13.902	20.055	5.730	9.930
			(7-D16)	(7-D19)	(2-D19)	(5-D16)
	d2	cm			69.0	44.0
	As2	cm^2			5.730	9.930
					(2-D19)	(5-D16)
	Total	cm^2	13.902	20.055	11.460	19.860
Stress Intensity	σc	kgf/cm^2	44.9	43.5	32.8	23.6
	σs	kgf/cm^2	1622.5	1396.3	1432.5	790.7
	$\sigma s'$	kgf/cm^2	0.0	0.0	-184.9	-68.0
	τ	kgf/cm^2	2.41	2.06	1.63	1.52
Allowable Stress	σca	kgf/cm^2	70.0	70.0	70.0	70.0
	σsa	kgf/cm^2	1800.0	1800.0	1600.0	1600.0
	τa	kgf/cm^2	3.60	3.60	3.60	3.60
Neutral Axis	x	cm	11.298	12.262	17.629	13.614
Young s Modules (n = Es/Ec)			15.0	15.0	15.0	15.0

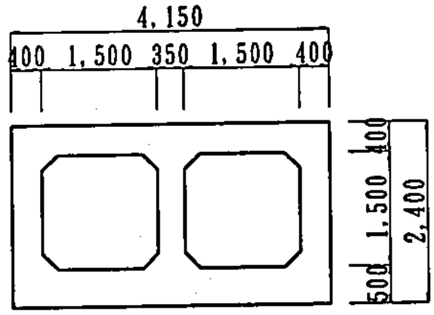
X = Distance from the Edge of the Compressive Side

(d) Box Culvert

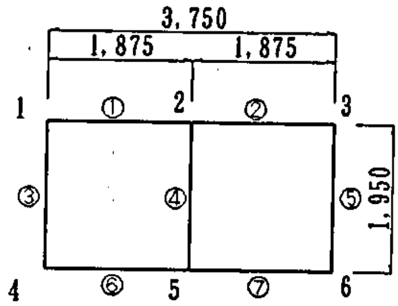
(i) Structural Calculation of Sluice

The structural calculation of sluice was carried out for the right side sluice which is bigger than left side one.

1) Dimension



2) Frame for Rahmen Analysis



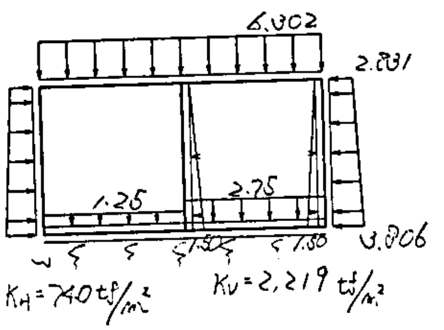
3) Members

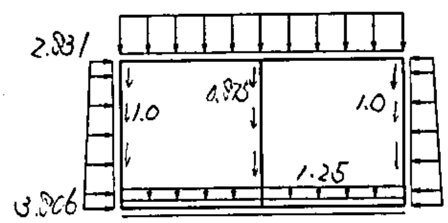
①②③⑤	:	$A = 0.4 \times 1.0 = 0.4\ (m2)$
		$I = (1.0 \times 0.4^3)/12 = 0.0053\ (m4)$
④	:	$A = 0.35 \times 1.0 = 0.35\ (m2)$
		$I = (1.0 \times 0.35^3)/12 = 0.0036\ (m4)$
⑦	:	$A = 0.5 \times 1.0 = 0.5\ (m2)$
		$I = (1.0 \times 0.53)/12 = 0.0104\ (m4)$

4) Rahamen Analysis

Sluice

				Left			Right		
Frame	Left	Right	Status	Axial (t)	Shear (t)	Moment(tm)	Axial (t)	Shear (t)	Moment(tm)
1	1	2	Normal	2.761	5.252	-1.097	2.761	6.565	-2.327
			Flood						
1	Center		Normal	2.761	0.525	1.07			
			Flood						
2	2	3	Normal	2.761	6.565	-2.327	2.761	5.252	-1.096
			Flood						
2	Center		Normal	2.761	0.525	1.07			
			Flood						
3	4	1	Normal	7.202	3.71	-1.714	5.252	2.761	-1.096
			Flood						
3	Center		Normal	6.227	0.237	0.172			
			Flood						
4	5	2	Normal	14.835	0	0	13.129	0	0
			Flood						
4	Center		Normal						
			Flood						
5	6	3	Normal	7.202	3.71	1.714	5.252	2.761	1.097
			Flood						
5	Center		Normal	6.227	0.237	-0.172			
			Flood						
6	4	5	Normal	3.71	7.202	1.714	3.706	7.418	1.935
			Flood						
6	Cente		Normal	3.707	0.123	-1.598			
			Flood						
7	5	6	Normal	3.706	7.418	1.936	3.71	7.202	1.714
			Flood						
7	Center		Normal	3.707	0.123	-1.598			
			Flood						





5) Stress Calculation

Calculation of Required Re-Bar

Type of Section				6RIGHT	6CENTER
Dimension		B	cm	100.0	100.0
		H	cm	50.0	50.0
Force		M	tf·m	1.935	1.600
		N	tf	3.706	3.707
		S	tf	7.418	0.000
Required Re-Bar		d1	cm	38.5	41.0
		As1	cm²	7.944	7.944
				(4-D16)	(4-D16)
	Total		cm²	7.944	7.944
Stress Intensity		σ c	kgf/cm²	12.1	9.1
		σ s	kgf/cm²	418.9	287.6
		σ s'	kgf/cm²	0.0	0.0
		τ	kgf/cm²	1.93	0.00
Allowable Stress		σ ca	kgf/cm²	70.0	70.0
		σ sa	kgf/cm²	1600.0	1600.0
		τ a	kgf/cm²	3.60	3.60
Neutral Axis		x	cm	11.633	13.185
Young s Modules (n = Es/Ec)				15.0	15.0

X = Distance from the Edge of the Compressive Side

Calculation of Required Re-Bar

Type of Section				1RIGHT	1CENTER	3LEFT	3RIGHT
Dimension		B	cm	100.0	100.0	100.0	100.0
		H	cm	40.0	40.0	40.0	40.0
Force		M	tf·m	2.327	1.100	1.714	1.100
		N	tf	2.761	2.761	7.202	5.252
		S	tf	6.565	0.000	3.710	0.000
Required Re-Bar		d1	cm	31.0	31.0	31.0	31.0
		As1	cm²	7.944	7.944	7.944	7.944
				(4-D16)	(4-D16)	(4-D16)	(4-D16)
	Total		cm²	7.944	7.944	7.944	7.944
Stress Intensity		σ c	kgf/cm²	21.6	9.8	14.2	8.8
		σ s	kgf/cm²	830.7	293.9	281.4	147.3
		σ s'	kgf/cm²	0.0	0.0	0.0	0.0
		τ	kgf/cm²	2.12	0.00	1.20	0.00
Allowable Stress		σ ca	kgf/cm²	70.0	70.0	70.0	70.0
		σ sa	kgf/cm²	1600.0	1600.0	1600.0	1600.0
		τ a	kgf/cm²	3.60	3.60	3.60	3.60
Neutral Axis		x	cm	8.685	10.361	13.333	14.631
Young s Modules (n = Es/Ec)				15.0	15.0	15.0	15.0

X = Distance from the Edge of the Compressive Side

Maintenance Bridge

I. Design Criteria

(1) GENERAL

(a) Scope

These calculations cover the design of the Inspection Bridge for the Bandar Sidoras Intake Weir.

(b) Applicable Codes and Standards

The following codes and standards were applied for the design, namely:

a. America Association of State Highway and Transportation Officials (AASHTO)
b. American Institute of Steel Construction (AISC)
c. American Welding Society (AWS)
d. American National Standards Institute (ANSI)
e. American Concrete Institute (ACI)

(2) STRUCTURAL DESIGN

(a) Structural Assumptions and Model

a. Structural model of the pipe bridge is shown in Figure 1, Section IV

b. Support is assumed pinned at Abutment A and fixed but free to translate along the x-axis at Abutment B .

c. Member properties are shown on page 2 of Section IV.

d. Loading calculations are shown in Section II.

(b) Design Principle

(i) Method of Analysis

The structural analysis and design was performed using STAADIII. In addition, axial, flexural and shear deformations were considered in the analysis.

(ii) Basis of Design

Steel members were designed in accordance with AISC specification and concrete structures were designed in accordance with ACI Code.

(3) Material

(a) Structural Steel

ASTM A36
Minimum yield stress Fy = 248 Mpa
Allowable stresses for tension, shear, compression, bending and bearing were based on AISC Specification.

(b) High-Strength Bolt (Friction-type)

ASTM A-325
Allowable tensile stress 304 Mpa
Allowable shear stress 118 Mpa

(c) Welds

Base metal ASTM A36
Welding electrode AWS E-70

(i) Complete-penetration groove welds

Shear on effective area : Fv = 0.30 Fu, except shear stress on base metal shall not exceed 0.40Fy of base metal.

Stress except shear : same as base metal

(ii) Fillet welds

Shear on effective area: Fv = 0.30 Fu, except shear stress on base metal shall not exceed 0.40Fy of base metal.

Fy shall be the minimum yield stress of base metal.

(d) Deformed Reinforcing Steel Bars

ASTM A615 Grade 40
fy = 275.80 Mpa

(e) Structural Concrete

Minimum compressive strength @ 28 days, f c = 21 Mpa
Allowable stresses were based on ACI Code requirements.

(4) Loads and External Forces

(a) Dead Load (DL)

Dead loads acting on the pipe bridge were based on the pipe load provided by the Client together with the self-weight of the bridge.

(b) Live Load (LL)

A uniform live load of 1.437 kN/m^2 (30psf) was adopted based on ANSI requirement.

(c) Seismic Load (EQ)

Seismic load acting on the bridge was determined in accordance with AASHTO specification.

(d) Earth Load (E)

Earth load on the abutment was considered using the following soil data provided by the Client, namely:

Soil unit weight............................ 17.2 kN/m^3

Angle of internal friction.............. 30°

Due to the absence of data, piles were designed based on the following assumed data, namely:

a. Bearing capacity............................500 kN

b. Lateral sub-grade reaction, kh...........15,000 kN/m^3

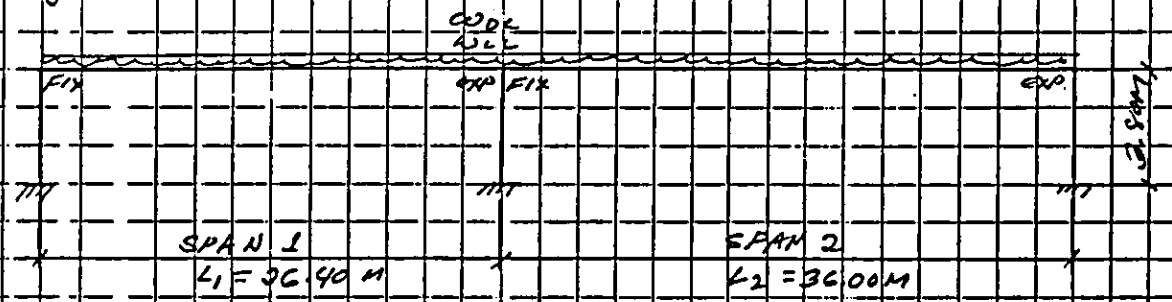
SECTION II

LOAD CALCULATIONS

NAME OF PROJECT	DESIGNED BY: AMM	SHEET NO.
MAINTENANCE BRIDGE	DATE: 6/24/96	OF

I. LOAD CALCULATIONS

Bridge overall width = 1.50 M.



Load Data: (Provided by client)

$DL = 3.43\ kN/m^2$

$LL = 2.90\ kN/m^2$

Uniform Load:

$\omega_{DL} = 3.43 \times 1.5 \approx 5.20\ kN/m$

$\omega_{LL} = 2.90 \times 1.5 \approx 4.40\ kN/m$

FOR EARTHQUAKE LOAD, SEE SUCCEEDING CALCULATIONS.

DESIGNATION	AREA	WEB		FLANGE		DIST.	b/2tf	h/tw	Ix	Iy	rx	ry	Wt.
	cm^2	H, mm	tw, mm	B, mm	tf, mm	h, mm			cm^4	cm^4	cm	cm	kg/m
BH 560X358.24	455.94	560	12.7	450	44	472	4.97	37.17	275,361	66,833	24.58	12.11	358.24
BH 560X375.19	477.52	560	28	450	38	484	5.55	17.29	259,841	57,801	23.33	11.00	375.19
BH 560X363.79	463.00	560	25	450	38	484	5.59	19.36	257,006	57,776	23.56	11.17	363.79
BH 560X352.38	448.48	560	22	450	38	484	5.63	22.00	254,172	57,755	23.81	11.35	352.38
BH 560X340.97	433.96	560	19	450	38	484	5.67	25.47	251,337	57,740	24.07	11.53	340.97
BH 560X329.56	419.44	560	16	450	38	484	5.71	30.25	248,503	57,729	24.34	11.73	329.56
BH 560X317.01	403.47	560	12.7	450	38	484	5.75	38.11	245,385	57,721	24.66	11.96	317.01
BH 560X360.27	458.52	560	28	425	38	484	5.22	17.29	246,875	48,707	23.20	10.31	360.27
BH 560X348.86	444.00	560	25	425	38	484	5.26	19.36	244,040	48,681	23.44	10.47	348.86
BH 560X337.45	429.48	560	22	425	38	484	5.30	22.00	241,206	48,661	23.70	10.64	337.45
BH 560X326.04	414.96	560	19	425	38	484	5.34	25.47	238,371	48,646	23.97	10.83	326.04
BH 560X314.63	400.44	560	16	425	38	484	5.38	30.25	235,537	48,635	24.25	11.02	314.63
BH 560X302.08	384.47	560	12.7	425	38	484	5.43	38.11	232,419	48,626	24.59	11.25	302.08
BH 560X333.39	424.32	560	28	380	38	484	4.63	17.29	223,536	34,841	22.95	9.06	333.39
BH 560X321.99	409.80	560	25	380	38	484	4.67	19.36	220,702	34,815	23.21	9.22	321.99
BH 560X310.58	395.28	560	22	380	38	484	4.71	22.00	217,867	34,795	23.48	9.38	310.58
BH 560X299.17	380.76	560	19	380	38	484	4.75	25.47	215,033	34,780	23.76	9.56	299.17
BH 560X287.76	366.24	560	16	380	38	484	4.79	30.25	212,198	34,769	24.07	9.74	287.76
BH 560X275.21	350.27	560	12.7	380	38	484	4.83	38.11	209,080	34,761	24.43	9.96	275.21
BH 560X321.45	409.12	560	28	360	38	484	4.37	17.29	213,164	29,637	22.83	8.51	321.45
BH 560X310.04	394.60	560	25	360	38	484	4.41	19.36	210,329	29,612	23.09	8.66	310.04
BH 560X298.63	380.08	560	22	360	38	484	4.45	22.00	207,495	29,592	23.37	8.82	298.63
BH 560X287.23	365.56	560	19	360	38	484	4.49	25.47	204,660	29,576	23.66	8.99	287.23
BH 560X275.82	351.04	560	16	360	38	484	4.53	30.25	201,826	29,565	23.98	9.18	275.82
BH 560X263.27	335.07	560	12.7	360	38	484	4.57	38.11	198,708	29,557	24.35	9.39	263.27
BH 560X290.15	369.28	560	28	360	32	496	5.19	17.71	189,248	24,974	22.64	8.22	290.15
BH 560X278.46	354.40	560	25	360	32	496	5.23	19.84	186,198	24,948	22.92	8.39	278.46
BH 560X266.77	339.52	560	22	360	32	496	5.28	22.55	183,147	24,927	23.23	8.57	266.77
BH 560X255.07	324.64	560	19	360	32	496	5.33	26.11	180,097	24,912	23.55	8.76	255.07
BH 560X243.38	309.76	560	16	360	32	496	5.38	31.00	177,046	24,900	23.91	8.97	243.38
BH 560X230.52	293.39	560	12.7	360	32	496	5.43	39.06	173,690	24,892	24.33	9.21	230.52
BH 560X257.40	327.60	560	25	360	28	504	5.98	20.16	169,447	21,838	22.74	8.16	257.40
BH 560X245.52	312.48	560	22	360	28	504	6.04	22.91	166,247	21,818	23.07	8.36	245.52
BH 560X233.64	297.36	560	19	360	28	504	6.09	26.53	163,046	21,802	23.42	8.56	233.64
BH 560X221.76	282.24	560	16	360	28	504	6.14	31.50	159,846	21,790	23.80	8.79	221.76
BH 560X208.69	265.61	560	12.7	360	28	504	6.20	39.69	156,325	21,781	24.26	9.06	208.69
BH 560X248.29	316.00	560	25	300	32	496	4.30	19.84	159,402	14,465	22.46	6.77	248.29
BH 560X236.59	301.12	560	22	300	32	496	4.34	22.55	156,351	14,444	22.79	6.93	236.59
BH 560X224.90	286.24	560	19	300	32	496	4.39	26.11	153,301	14,428	23.14	7.10	224.90
BH 560X213.21	271.36	560	16	300	32	496	4.44	31.00	150,250	14,417	23.53	7.29	213.21
BU 1100X163.55	208.16	1100	12.0	250	16	1068	7.44	89.00	356,847	4,182	41.40	4.48	163.55
BU 1200X172.98	220.16	1200	12.0	250	16	1168	7.44	97.33	439,730	4,183	44.69	4.36	172.98 1.7 kN/m
BU 1200X224.40	285.60	1200	16.0	250	20	1160	5.85	72.50	556,253	5,248	44.13	4.29	224.40 2.2 kN/m

DL-6.9 DL-7.4
LL-4.4 LL 4.4

26.4 36

$K_{DL} = 6.9 \times 13.2 + 7.4 \times 18 + 1 \times 1 \times 3.8 \times 24 = 315.48$

$R_{LL} = 4.4(13.2 + 18) = 137.28$

SEISMIC LOADING INTENSITY FOR TRANSVERSE EARTHQUAKE LOADING

Acceleration Coefficient, A = 0.15
Site Coefficient, S = 1.00

NODE NO	Displacements vs(x) (mm)	vs(x) (m)	vs(x)^2 (m^2)	dx (m)	vs(x)*dx	vs(x)^2*dx	Load Intensity, Pe(x) (kN/m) NODE NO.	Pe(x)
0L1	0.1380	0.00014	1.90E-08	1.05	0.0001	2.01E-08	0L1	0.00
.1L1	9.2270	0.00923	8.51E-05	2.11	0.0194	1.79E-04	.1L1	0.27
.2L1	29.7070	0.02971	8.83E-04	2.11	0.0626	1.86E-03	.2L1	0.86
.3L1	51.4690	0.05147	2.65E-03	2.11	0.1084	5.58E-03	.3L1	1.50
.4L1	67.3080	0.06731	4.53E-03	2.11	0.1418	9.54E-03	.4L1	1.96
.5L1	72.9210	0.07292	5.32E-03	2.11	0.1536	1.12E-02	.5L1	2.12
.6L1	66.9110	0.06691	4.48E-03	2.11	0.1410	9.43E-03	.6L1	1.94
.7L1	50.7850	0.05079	2.58E-03	2.11	0.1070	5.43E-03	.7L1	1.48
.8L1	28.9530	0.02895	8.38E-04	2.11	0.0610	1.77E-03	.8L1	0.84
.9L1	8.7280	0.00873	7.62E-05	2.11	0.0184	1.60E-04	.9L1	0.25
L1	0.3290	0.00033	1.08E-07	2.11	0.0007	2.28E-07	L1	0.01
.1L2	24.9330	0.02493	6.22E-04	2.11	0.0525	1.31E-03	.1L2	0.72
.2L2	81.5230	0.08152	6.65E-03	2.11	0.1717	1.40E-02	.2L2	2.37
.3L2	142.0470	0.14205	2.02E-02	2.11	0.2992	4.25E-02	.3L2	4.13
.4L2	186.4560	0.18646	3.48E-02	2.11	0.3928	7.32E-02	.4L2	5.42
.5L2	202.7040	0.20270	4.11E-02	1.05	0.2135	4.33E-02	.5L2	5.89
.6L2	186.7500	0.18675	3.49E-02	1.05	0.1967	3.67E-02	.6L2	5.43
.7L2	142.5560	0.14256	2.03E-02	1.05	0.1502	2.14E-02	.7L2	4.14
.8L2	82.0850	0.08208	6.74E-03	1.05	0.0865	7.10E-03	.8L2	2.39
.9L2	25.3060	0.02531	6.40E-04	1.05	0.0267	6.75E-04	.9L2	0.74
L2	0.1890	0.00019	3.57E-08	1.05	0.0002	3.76E-08	L2	0.01
					2.4039	0.285399401		

w(x) = 15.192
alpha = 2.4039
beta = 36.522
gamma = 4E+00
T = 0.7055
$Cs = 1.2AS/T^{2/3}$ = 0.2271 use Cs = 0.227
Pe(x) = 29.062 * vs(x)

06/09/96
FN: EQINTENS

NODAL LOAD DUE TO TRANSVERSE EARTHQUAKE LOADING:

NODE NO.	Displacements* vs(x) (mm)	vs(x) (m)	x (m)	Load Intensity, Pe(x) (kN/m)	Nodal Load, Pe (kN) Pe	
0L1	0.1380	0.00014		0.00	0.12	
.1L1	9.2270	0.00923	2.64	0.27	0.85	
.2L1	29.7070	0.02971	2.64	0.86	2.30	
.3L1	51.4690	0.05147	2.64	1.50	3.87	
.4L1	67.3080	0.06731	2.64	1.96	5.03	
.5L1	72.9210	0.07292	2.64	2.12	5.45	
.6L1	66.9110	0.06691	2.64	1.94	5.00	
.7L1	50.7850	0.05079	2.64	1.48	3.82	
.8L1	28.9530	0.02895	2.64	0.84	2.24	
.9L1	8.7280	0.00873	2.64	0.25	0.82	
L1	0.3290	0.00033	2.64	0.01	0.57	SUM L1 30.08 kN
.1L2	24.9330	0.02493	3.60	0.72	3.17	
.2L2	81.5230	0.08152	3.60	2.37	8.60	
.3L2	142.0470	0.14205	3.60	4.13	14.58	
.4L2	186.4560	0.18646	3.60	5.42	15.49	
.5L2	202.7040	0.20270	3.60	5.89	10.32	
.6L2	186.7500	0.18675	3.60	5.43	10.05	
.7L2	142.5560	0.14256	3.60	4.14	8.23	
.8L2	82.0850	0.08208	3.60	2.39	5.35	
.9L2	25.3060	0.02531	3.60	0.74	2.31	
L2	0.1890	0.00019	3.60	0.01	0.45	SUM L2 78.54 kN

Total EQ Force = 108.62 kN

NOTE:
* From STAAD output on displacements calculations due to 1 kN/m transverse load.

06/09/96
FN:NODALEQ

SECTION III

STRESS, DISPLACEMENTS AND REACTIONS CALCULATIONS USING STAAD III

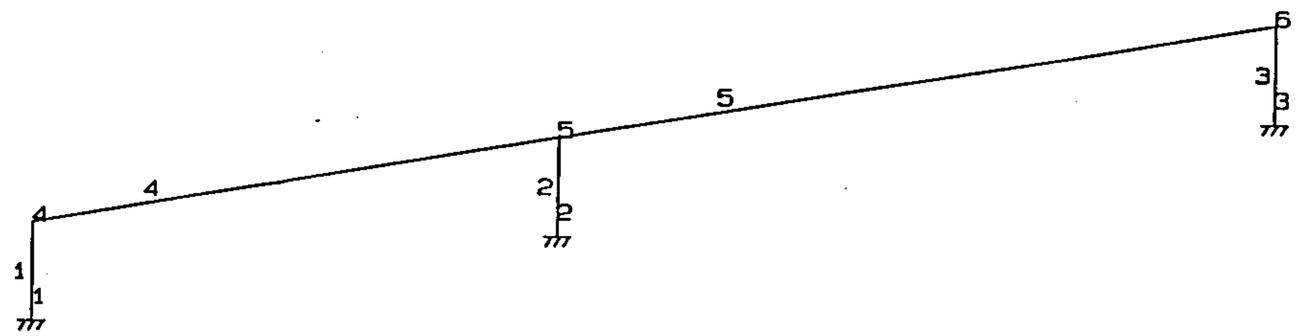
STRUCTURE DATA

TYPE = SPACE

NJ =	6
NM =	5
NE =	0
NS =	3
NL =	9
XMAX=	62.4
YMAX=	3.8
ZMAX=	0.0

1

MN/ELEM



MAINTENANCE BRIDGE GEOMETRY

J-6, M-5

UNIT MET MNS

S T A A D P O S T - P L O T (REV: 20.2)

INC. TITLE: CTI-MAINTENANCE BRIDGE

DATE: JUN 24, 1996

SECTION IV

DESIGN OF PLATE GIRDERS

DESIGN OF PLATE GIRDER

When no transverse stiffeners are provided or when traverse stiffeners are spaced more than 1-1/2 times the distance between flanges.

$$\frac{h}{tw} \leq \frac{14000}{\{fy(FYf+165)]^{1/2}}$$

When transverse stiffenners are provided, spaced not more than 1-1/2 times the distance between flanges

$$\frac{h}{tw} \leq \frac{2000}{(Fyf)^{1/2}}$$

ALLOWABLE BENDING STRESS

When the web depth to thickness ratio exceeds 760/(Fb)^1/2, the maximum bending stress in the compression flanges shall not exceeds

$$Fb' \leq Fb\ Rpg\ Re$$

where:

Fb	=	applicable bending stress given as follows,
Fb	=	0.75Fy for compact section
Fb	=	0.60Fy for non-compact section

$$Rpg = 1 - 0.005(Aw/Af)(h/t - 760/Fb^{1/2} \leq 1.0$$

$$Re = \frac{12 + Aw/Af(3alpha-alpha^3)}{12 + 2(Aw/Af)} \leq 1.0$$

for non hybrid girders, Re = 1.0

Aw	=	area of web at the section under investigation
Af	=	area of compression flanges, in.^2
alpha	=	$0.60Fyw/Fb \leq 1.0$

ALLOWABLE SHEAR STRESS WITH TENSION FIELD ACTION (Mpa)

fv in Mpa or Ksi

Fv	=	$(Fy/2.89)Cv \leq 0.40fy$	
Cv	=	$4500Kv/[(Fy)(h/tw)]$,	when Cv is less than 0.80
Cv	=	$190/(h/tw)[(Kv/Fy)]^{1/2}$,	when Cv is more than 0.8
Kv	=	$4.00 + [5.34/(a/h)^2]$,	when a/h is less than 1.00
Kv	=	$5.34 + [4.00/(a/h)^2]$,	when a/h is less than 1.00
tw	=	thickness of web	
a	=	clear distance between traverse stiffener	
h	=	clear distance between flanges at the section under investigation	

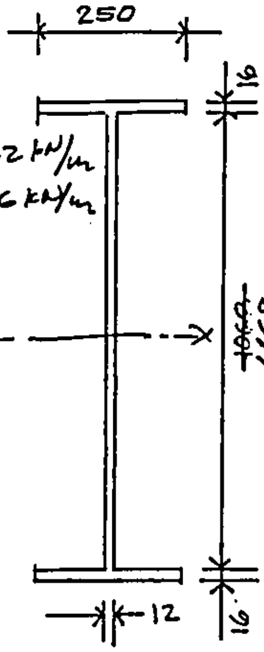
For girder other than hybrid girder if intermediate stiffener are provided and spaced to satisfy the provisions given and Cv is lesser than or equal to 1.00, the allowable shear including tension field action is given by the equation,

$$Fv = \frac{Fy}{2.89}\{ Cv + [(1 - Cv)/1.15[1+(a/h)^{1/2}]\} \leq 0.40Fy$$

DESIGN OF PLATE GIRDER

Design Criteria:

Fy	=	248.22 Mpa	(36Ksi)
E	=	200000 Mpa	(29000Ksi)
L	=	~~29.00~~ m. 26.40 m. (c to c of pier)	



Loads:

Liveload = 2.90 Kpa ; $w_{LL} = 2.9 \times 1.5/2 = 2.2$ kN/m

Deadload = 3.43 Kpa ; $w_{DL} = 3.43 \times 1.5/2 = 2.6$ kN/m

Properties

A	=	22016 mm^2
I_{xx}	=	~~3568.469~~ 4397.30 X 10^6 mm^4
S	=	~~6.488~~ 7.329 X 10^6 mm^3
Wt.	=	~~1.94~~ 1.70 Kn /m.

$w_{TOTAL} = 2.2 + 2.6 + 1.7 = 6.5$ kN/m

Check if section is Compact

Web maximum thickness ratio:

$$\frac{640}{(36)^{1/2}} = 107$$

Actual ratio:

$$\frac{d}{tw} = \frac{\cancel{1068}\ 1168}{12} = \cancel{89}\ 97.33 < 106.7 \text{ OK!}$$

Flange maximum width thickness ratio:

$$\frac{65}{(36)^{1/2}} = 10.8$$

Actual ratio:

$$\frac{b}{2tf} = \frac{250}{2(16)} = 7.8 < 10.80 \text{ OK!}$$

Therefore, The section is Compact!

Fb = 0.66Fy = .66(248.22) = 163.8 Mpa

STAAD OUTPUT:

from ~~continuous beam analysis~~

Mdl	=	747.71/2 ~~360.58~~	Kn-m	=	373.86
Mll	=	~~304.86~~ 383.33/2	Kn-m	=	191.66
M TOTAL	=	~~1.4(360.58)+1.7(304.86)~~ = 373.86 + 191.66			
	=	~~1023.07~~ 565.52	Kn-m		

$$fb = \frac{M}{S} = \frac{\cancel{1023.07}\ 565.52 \times 10^6}{\cancel{6.488}\ 7.329 \times 10^6} = \cancel{157.7}\ 77.16 \text{ Mpa}$$

Fb > fb, therefore OK!

Check for Shear:

from continuous beam analysis,

Vdl	=	~~59.34~~ Kn 113.29/2 = 56.64
Vll	=	~~50.17~~ Kn = 58.08/2 = 29.04
Vu	=	~~1.4(59.34)+1.7(50.17)~~
	=	~~168.365~~ 85.68 Kn

$$fv = \frac{V}{dtw} = \frac{\cancel{168.365}\ 85.68 \times 10^3}{\cancel{1068}\ 1168 \times 12} = \cancel{13.14}\ 6.11 \text{ Mpa}$$

Fv = 0.45Fy = 0.45(248.22)
= 111.7 Mpa

Fv > fv, therefore OK!

Check allowable deflection:

$$Y\ allow. = \frac{L}{360} = \frac{\cancel{29000}\ 26400}{360} = \cancel{80.6}\ 73.33 \text{ mm.}$$

$$Y\ actual = \frac{5wL^4}{384EI} = \frac{5(\cancel{6.224}\ 6.5)(\cancel{29000}\ 26400)^4}{384(200000)(\cancel{3568.469}\ 4397.30 \times 10^6)}$$

$$= \cancel{80.31}\ 46.75 \text{mm} < \cancel{80.55}\ 73.33 \text{mm.,}$$

therefore, OK!

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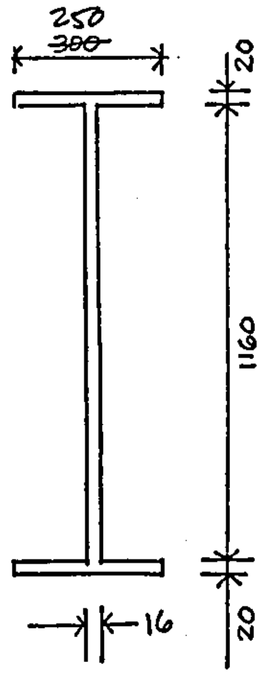
Consider span L = ~~35.00~~ 36.00m.



Section Properties:

A = ~~30560~~ 28560 mm^2

I_{xy} = ~~6260.00~~ 5562.53 X !0^6 mm^4

S = ~~10.433~~ 9.271 X 10^6 mm^3

Wt. = ~~2.694~~ 2.20 Kn /m.

w_{TOTAL} = 2.2 + 2.6 + 2.2 = 7.00 kN/m

Check if section is Compact

Web maximum thickness ratio:

$$\frac{640}{(36)^{1/2}} = 107$$

Actual ratio:

$$\frac{d}{tw} = \frac{1160}{16} = 73 < 107 \text{ OK!}$$

Flange maximum width thickness ratio:

$$\frac{65}{(36)^{1/2}} = 10.8$$

Actual ratio:

$$\frac{b}{2tf} = \frac{\cancel{300}\ 250}{2(20)} = \cancel{7.5}\ 6.25 < 10.80 \text{ OK!}$$

Therefore, The section is Compact!

Fb = 0.66Fy = .66(248.22) = 163.8 Mpa

from continuous beam analysis

Mdl = 1553.24/2 ~~625.22~~ = 776.62 Kn-m

Mll = 712.8/2 ~~444.06~~ 356.4 Kn-m

Mu = ~~1.4(525.22)+1.7(444.06)~~ 776.62 + 356.4

= ~~1490.21~~ 1133.02 Kn-m

$$fb = \frac{M}{S} = \frac{\cancel{1490.20}\ 1133.02 \times 10^6}{\cancel{10.43}\ 9.271 \times 10^6} = \cancel{142.8}\ 122.21 \text{ Mpa}$$

Fb > fb, therefore OK!

Check for Shear:

from continuous beam analysis,

$V_{dl} = \cancel{59.34}$ Kn   172.58/2 = 86.29 kN

$V_{ll} = \cancel{50.17}$ Kn   79.2/2 = 39.60 kN

$V_u = \cancel{1.4(59.34)+1.7(50.17)}$ 86.29 + 39.60

$= 125.890$ Kn

$$f_v = \frac{V}{d t_w} = \frac{\overset{125.89}{\cancel{168.365}} \times 10^3}{1160 \times 16} = \overset{6.78}{\cancel{9.07}} \text{ Mpa}$$

$F_v = 0.45F_y = 0.45(248.22)$

$= 111.7$ Mpa

$F_v > f_v$, therefore OK!

Check allowable deflection:

$$Y \text{ allow.} = \frac{L}{360} = \frac{\overset{36,000}{\cancel{35000}}}{360} = \overset{100}{\cancel{97.22}}\text{mm.}$$

$$Y \text{ actual} = \frac{5wL^4}{384EI} = \frac{5(\overset{7.00}{\cancel{6.224}})(\overset{36000}{\cancel{35000}})^4}{384(200{,}000)(\overset{5562.53}{\cancel{6260}}\times 10^6)}$$

$= \overset{}{\cancel{105.37}}$ 137.61 $>$ 100 $\cancel{97.22}$ mm

$>$ 90%

therefore, OK!

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SECTION V

DESIGN OF CORBEL

DESIGN OF CORBEL

1. DETERMINE SIZE OF BEARING PLATE

Vu =	1.4	DL	+	1.7	LL	+	1.87	EQ.
	DL=	120.05		LL=	101.5		EQ=	84
Vu =		168.07	+		172.55	+		157.1
Vu =		497.7	KN					

Vu ≤ phiPnb = phi(0.85f'cA1) ; f'c = 27.58 Mpa
phi= 0.7

A1 = 3033mm^2

width of girder= 300mm

L = A1/300 = 101mm

USE : 150mm X 300mm X 20mm thk. BEARING PLATE

2. DETERMINE a

Assume beam reaction to act at outer third point of bearing plate. Assume 25mm gap between back edge of bearing plate and column face.

Hence; a = 25+2/3(150) = 125 mm

3. DETERMINE TOTAL DEPTH OF CORBEL BASED ON LIMITING SHEAR-TRANSFER STRENGTH Vn. FOR EASIER PLACEMENT OF REINFORCEMENT AND CONCRETE.

a/d	=	0.22		
d	=	568.18mm		
h	=	d + 25 = 593.18mm		say 600mm.
Vn	=	[800-280(a/d)]bwd KIPS		where: a, d, bw in inches
	=	200.6 KIPS		
phiVn	=	0.85(Vn) = 170.51 KIPS	=	758.43 KN
		phi(Vn) > Vu,		

therefore, OK!

4. DETERMINE SHEAR FRICTION REINFORCEMENT, Avf

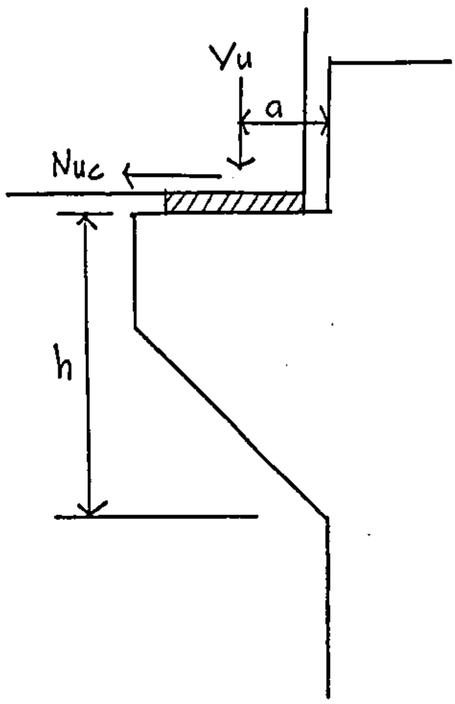
Vu	$\leq$ phi(Vn) $\leq$	phi(0.8Avf fy + 0.20bwd)	
Solving for Avf,			
Avf	=	(Vu - phi 0.2bwd)/(phi 0.8fy) ;	but not less than
		0.20bwd/fy	fy = 414 Mpa
			phi = 0.85
Avf	=	1663.736 mm^2 > 0.20bwd/fy	.= 83.33mm^2 OK!

5. DETERMINE FLEXURAL REINFORCEMENT, Af

Mu	=	(Vu X a) + Nuc(h-d);	Nuc = 1.7(100) = 170 KN
	=	66.425 KN-m	
Af	=	Mu/phi fy j d	66.425 X 10^3/0.85(414)(0.9)(575)
Af	=	364.96 mm^2	

6. DETERMINE PRIMARY TENSION REINFORCEMENT, As

2/3Avf	=	1109.16 mm^2 > Af,
		therefore, Avf controls design
As	=	2/3Avf + An



7. DETERMINE DIRECT TENSION REINFORCEMENT, An

An	=	Nuc/phi fy;	phi = 0.85
An	=	483.09mm^2	Thus,
As	=	1592.25mm^2	
		USE:	4 - 28 mm dia. RSB

8. DETERMINE SHEAR REINFORCEMENT, Ah

Ah	=	0.50(As - An) =	554.58mm^2
		USE:	6 -12 mm. dia. RSB

SECTION VI

DESIGN OF COLUMN

DESIGN OF COLUMN

f'c = 20.7 Mpa
fy = 275.8 ~~414~~ Mpa
Wc = 24 Kn/cu.m.

Desired section: 1000 X 1000

From Microfeap analysis

Pu = 1076.08 Kn
phiPn; phi = 0.70
Pn = 0.80Po

thus,
Pu = 0.70(0.80 X Po)

for short tied column,

Po = Ag[(0.85f'c+pg(fy-0.85f'c)]

assume pg = ~~0.03~~ 0.01

.01 275.8
Po = 1000 X 1000[0.85(20.7)+0.~~03~~(~~414~~-0.85X20.7)]1/10^3
20177.05
= ~~29487.15~~ Kn 20177.05
Pu = 0.70 X 0.80 (~~29487.15~~)
Pu = ~~16512.804~~ Kn - m > 1076.08 Kn.
11299.15

therefore, OK!

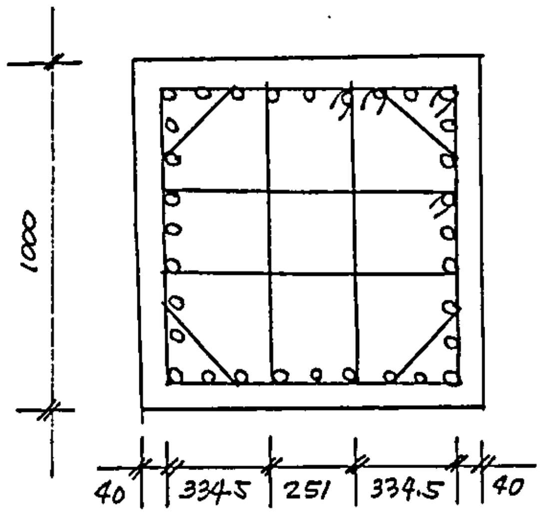
.01
As = pgAg = 0.~~03~~(1000 X 1000)
As = ~~30000~~ sq. m.
10000
32-20
use: ~~30 - 36~~mm. dia.RSB

DESIGN OF HOOP CONFINEMENT

Size of column			(mm.)	1000 x 1000
Height from footing			(mm.)	5750
h = 1/6 total height			(mm.)	958
Maximum column dimension			(mm.)	1000
h	$\geq$	450	(mm.)	450
lh			(mm.)	251
Ag			(sq.m)	1000 X !0^3
Ac			(sq.m)	846.4 x 10^3
Ps	=	0.45[(Ag/Ac) - 1][f'c/fy]		4.08 x 10^-3
Psmin	=	0.12(f'c/fy)		6.00 X 10^-3
Tie rebars	=			R 12
sh	=	4Ash/(lhps) (mm)		826
shmax.	=		(mm.)	100
smax.	=	least dimension	(mm.)	1000
smax.	=	16 dia. vertical bar	(mm.)	576
smax.	=	48 dia ties	(mm.)	576
smax.	=	d/2	(mm.)	467

Remarks:

Use: 4 R 12 ties, 10 @ 100mm. rest @ 300mm. O.C. to midheight



SECTION VII

DESIGN OF PILES & FOOTING

Design of Piles and Pier Footing

A. Design of Piles

A.1 Compute number of piles (assume 400 x 400 RC Piles)

Assumed allow. pile cap. @ 15.00m depth & F.S.= 3, . Qa = 500.00 kN

Soil unit weight, w_s = 17.20 kN/m^3

Concrete unit weight, w_c = 24.00 kN/m^3

Soil internal angle of friction, ø = 30.00 °

Coef. of active soil pressure, K_A = $\frac{(1 - \sin ø)}{(1 + \sin ø)}$ = 0.33

P_{DL} = 224.28 kN

P_{LL} = 137.28 kN

EQ_L = 102.12 kN

EQ_T = 78.55 kN

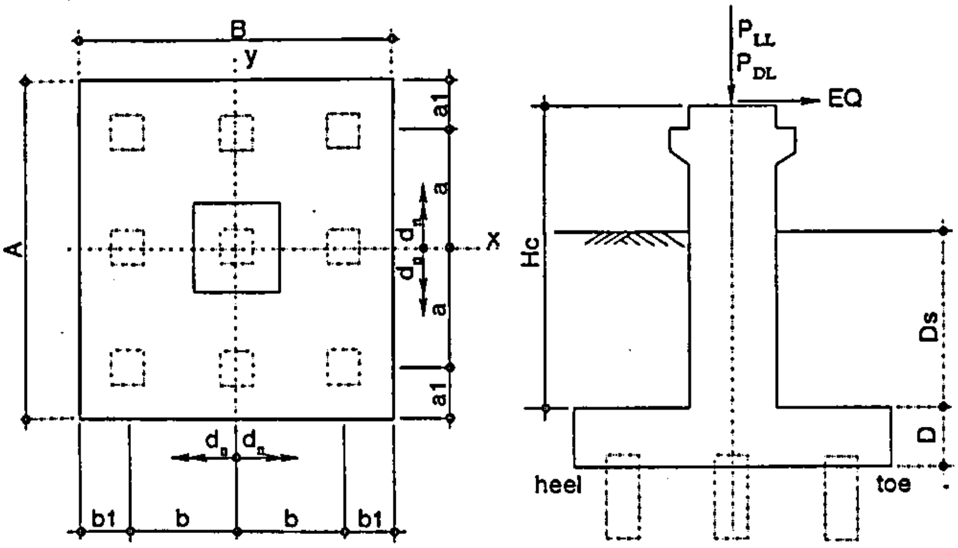
TRIAL DIMENSIONS:

A	=	3.00 m	a	=	1.00 m
B	=	4.00 m	b	=	1.50 m
D	=	0.80 m	a_1	=	0.50 m
D_S	=	2.10 m	b_1	=	0.50 m
H_C	=	3.80 m			

Loads carried by piles:

DL (superstructure + pier)	=	315.48 kN
LL	=	137.28 kN
Footing	=	230.40 kN
Soil	=	397.32 kN
		1080.4 kN say 1100 kN

Estimated no. of piles = 2.2 Try no. of piles = 9 spaced as shown



(FN: PILEDES.WK1)

At x–axis direction:

d_1 = 1.50 n_1 = 3.00 $\frac{n \times d^2}{6.75}$

$Sum(nd^2)_{(Y-Y)}$ = 13.5

At y–axis direction:

d_1 = 1 n_1 = 3 $\frac{n \times d^2}{3.00}$

$Sum(nd^2)_{(X-X)}$ = 6

Total vert. forces, F_V = 1051.6 kN
Long. overturning mom., M_{OL} = 388.06 kN–m
Trans. overturning mom., M_{OT} = 298.49 kN–m
Righting moment, M_{RL}, long. = 2103.3 kN–m
Righting moment, M_{RT}, transverse = 1577.5 kN–m

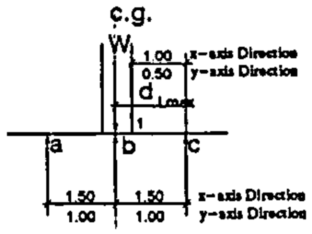
$x = \frac{M_R - M_O}{F_V} =$ 1.631 m

e = 0.369 m

Compute maximum pile load: $P = W_T/n \pm M \times$ dist. fr. c.g./$Sum(nd^2)$

Along x–axis: $M_{C.G.}$ = 388.06 d_{MAX} = 1.5 m
Along y–axis: $M_{C.G.}$ = 298.49 d_{MAX} = 1 m

Along the longitudinal direction:



$P_{MAX}=P_c$ = 159.97 kN < Qa, = 500 OK!
$P_{MIN}=P_a$ = 73.736 kN < Qa, = 500 OK!
P_b = 116.85 kN < Qa, = 500 OK!

Along the transverse direction:

$P_{MAX}=P_c$ = 166.60 kN < Qa, = 500 OK!
$P_{MIN}=P_a$ = 67.105 kN < Qa, = 500 OK!
P_b = 116.85 kN < Qa, = 500 OK!

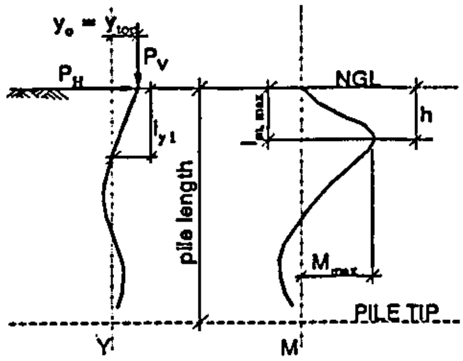
therefore,

PROVIDE 9–400x400x15000 RC PILES

A.2 Ultimate Pile Capacity Analysis & Design (Chang's Method)

$$y_{TOP} = \frac{P_H}{2EI\beta^3}$$

$$y_O = y_{TOP}$$



ly_1 = depth point of first immovable pt.

$$= \frac{\pi}{2\beta}$$

lm = depth of ground where the max. bending moment occurs

$$= \frac{\pi}{4\beta}$$

$\beta = (k_h B/4EI)^{.25}$

where k_h = coefficient of lateral subgrade reaction
= 15000.00 kN/m^3 (assumed)
B = pile width = 0.40 m
E = 57000 SQRT(fc') fc' = 3000.00 psi
= 3122018.6 psi
= 21526318.1 kN / m^2
I = $B^4/12$ = 0.002 m^4
EI = pile flexural rigidity = 45922.81 kN−m^2

therefore

β = 0.4251 /m
ly_1 = 3.695 m
lm = 1.847 m

n = number of piles = 9

W_{DL} = total dead load = 545.88 kN
W_{LL} = total live load = 137.28 kN
W_E = total earth load = 397.16 kN

EQ_F = 102.12 kN , GOVERNS
EQ_{min} = .1(DL+.5LL+E) = 101.17 kN

EQ = 102.12 kN
P_E = 0.00 kN

$$P_{HEQ} = \frac{EQ}{n} = 11.35 \text{ kN}$$

$$P_{HE} = \frac{P_E}{n} = 0.00 \text{ kN}$$

$$P_H = \frac{EQ + P_E}{n} = 11.35 \text{ kN}$$

M_{max} = maximum bending moment in the ground due to lateral load P_H

$$= \frac{-0.322 \times P_H}{\beta}$$

= −8.594259 kN−m

y_{TOP} = 0.0016079 m = 1.607913 mm < 25 mm, OK!

Ultimate Stresses

$Pv_{DL} = W_{DL} / n = 60.65$ kN
$Pv_{LL} = W_{LL} / n = 15.25$ kN
$Pv_E = W_E / n = 44.13$ kN

$M_{DL} = Pv_{DL} \times y_{TOP} = 0.097525$ kN−m
$M_{LL} = Pv_{LL} \times y_{TOP} = 0.024526$ kN−m
$M_{EQ} = \dfrac{-0.322 \times P_{HEQ}}{\beta} = -8.59425$ kN−m

$M_{U1} = 1.3(.75M_{DL}+1.3M_E+M_{EQ}) = 11.27$ kN−m, GOVERNS
$M_{U2} = .75(1.4M_{DL}+1.7M_{LL}+1.87M_{EQ}) = 12.19$ kN−m
$P_{U1} = 1.3(.75Pv_{DL}+1.3Pv_E) = 133.71$ kN, GOVERNS
$P_{U2} = .75(1.4Pv_{DL}+1.7Pv_{LL}) = 83.13$ kN

Check Slenderness Effect:

Effective length factor, $K = 2.10$
$l_U = ly_1 = 3.695$ m.
$r = 0.30 \times B = 0.120$ m.

$Kl_U/r = 64.660972 > 22$, long column (consider slenderness effect)

$\beta_d = [\text{Factored } M_{DL}/(\text{Factored } M_{TOTAL})] = 1.4M_{DL}/M_U = 0.012$

$EI = \dfrac{(E_c I_g)/2.5}{1 + \beta_d} = 18149.20$ kN−m^2

$P_c = \dfrac{\pi^2 EI}{(Kl_U)^2} = 2975.16$ kN

$C_m = 1.00$

$\text{Delta} = \dfrac{C_m}{(1 - P_u/\text{phi} \times P_c)} = 1.041577$

$M_c = \text{delta} \times M_U = 11.73610$ kN−m.

FROM INTERACTION DIAGRAM, 400 x 400 RC PILES will SUFFICE !

ADOPT 9 − 400 x 400 x 15.00M LONG PRE−CAST RC PILES

B. Design of Footing

Compute piles reaction: No. of piles per row, nr = 3

Along the longitudinal direction:

P_a = 221.21 kN

P_b = 350.56 kN

P_c = 479.91 kN

Along the transverse direction:

P_a = 201.32 kN

P_b = 350.56 kN

P_c = 499.81 kN

Compute Moments:

Along the longitudinal direction:

M_1 = 479.91 kN−m

M_{U1} = 719.87 kN−m

M_{U1}/m = 239.95 kN−m/m, GOVERNS

Along the transverse direction:

M_1 = 249.90 kN−m

M_{U1} = 374.85 kN−m

M_{U1}/m = 93.713 kN−m/m

Compute Reinforcements:

fy =

Pile embed. to footing = 0.15 m

Assuming rebars = 25 Ø
and using f'c = 20.685 Mpa
fy = 275.8 Mpa

Effective depth, d = 0.4625 m
Rn = 1246.42
p = 0.00469
p_{min} = 0.00500 , governs

therefore

As = 2314.18 mm^2/m
Spacing = 0.2121 m

As_{MIN} = 2314.18 mm^2/m
Spacing = 0.2121 m

Provide Ø 25 @ 0.20m O.C. main longitudinal bars and
Ø 25 @ 0.20m O.C. transverse bars

Inflatable Rubber Made Dam

I. Design Feature of Inflatable Rubber Made Dam

1 Size of Rubber Dam

1) Height x Length (along foundation) : 3.1400 m x 13.000 m x 1 span

2) Medium of Inflation : Air

3) Batter of Side Slope (V:H) (LEFT SIDE) : 1:2.0

(RIGHT SIDE) : 1:2.0

4) Fixing Line : Double Anchoring Line

2 Rubber Body

1) Rubber Sheet Size (width:Length) : 6.315 m x 33.22 m x 1 span

2) Thickness of Rubber Body : 15.8 mm

3) Weight of Rubber Body : 249 kgf/m

4) Reinforcement : 4 Layers of nylon canvas

5) Tensile Strength

(in watercourse direction) : 590 kgf/cm or more

(in longitudinal direction) : 393 kgf/cm or more

6) Protective Outer Rubber Compound	: EPDM (Etylene Propylen Diene Monomer) compounded
7) Thickness of Outer Cover Rubber	: 7.0 mm
8) Physical Properties of Rubber Materials	
a. Initial Strength	
Tensile Breaking Point (TB)	: more than 120 kgf/cm^2
Elongation at Breaking (EB)	: more than 400%
b. Heat Aging Test	(100°C for 4 days)
Tensile Breaking Point	: more than 100 kgf/cm^2
(EB)	: more than 300%
c. Ozone Test	(100 pphm, 40°C , 96 hrs, under 50% elongation)
Observation	: No craking
d. Cold Test	
Brittling Temperature	: less than -40°C
e. Hot Water-Resistance Test	(70°C for 4 days)
Volume Change	: less than 20%
f. Abrasion-Resistance Test	(Abrasion test (Disc H18) with load weight 1000 gr -1,000 rotations)
Abrased Volume	: 0.5 ml or less

3 Anchor Parts

1) Clamping Plate	: Galvanized ductile cast iron
	: JIS FCD500/ASTM A536 Gr.80
2) Embedded Plate	: Galvanized rolled steel
	: JIS SS400/ASTM A36
3) Anchor Bolt (M 30)	: Galvanized carbon steel
	: JIS S45C/ASTM A575 Gr.1045
4) Anchor Nut (M 30)	: Galvanized carbon steel
	: JIS S45C/ASTM A575 Gr.1045
5) Spring Washer	: Stainless steel
	: JIS SUS304/AISI 304

4 Spacer Parts in Rubber Body

1) Spacer on River Bed	: 50 OD. x 4 lines
	Braided rubber hose set
2) Connection Parts	: Mild Steel Clamp / Stainless Steel Clip

5 Pipelines (Galvanized Steel Pipe)

1) Air Supply & Exhaust Pipeline	: 100 mm diameter
	JIS SGPW/ASTM A120
2) Water Inlet Pipeline	: 100 mm diameter
	JIS SGPW/ASTM A120
3) Pressure Sensing Pipeline	: 50 mm diameter
	JIS SGPW/ASTM A120

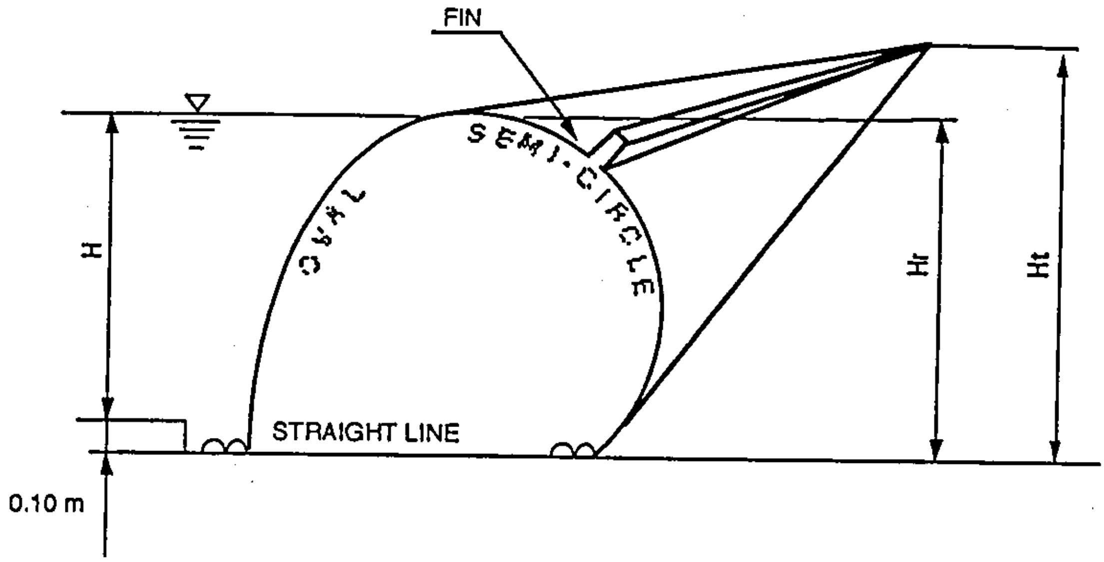
6 Operation Equipment

1) Required Space of Control Room	: 2100 mm wide x 3200 mm long
2) Air Compressor	:
a. Type	: Rotary Blower BH125A / 1 Units
b. Max. Discharge Static Pressure	: 4000 mmAq
c. Effective discharge Capacity	: Min. 12.0 m^3/min
d. Motor Electric Power	: 15.0 kw
e. Electric Voltage	: 380V-3phase-50Hz
3) System control of auto-deflation	: Electrical auto-deflation +Bucket system
4) Control Board	: Design shall be submitted later

II. SHAPE OF RUBBER DAM

1 Rubber Body: Inflated

With a full head of water, the Rubber Dam takes the shape shown below. A fin, of the downstream side, indicated in the drawing below, minimizes oscillation.



In case of 20% or less over-topping height of design rubber dam height;

SIDE SLOPE (1 / n)	Ht / Hr
1 / 0	1.2
1 / 0.3	1.17
1 / 0.5	1.15
LESS THAN 1 / 0.5	1.1

In case of 20% or more over-topping height of design rubber dam height;
Ht = Auto deflation Water height

2 Rubber Body: Deflated

The Rubber Dam appears flat when deflated. The fin shown above becomes the flat trailing edge of the rubber body.

III CALCULATION OF STRENGTH OF RUBBER DAM BODY

1 Internal Pressure

Design Internal pressure under full water is calculated as follows;

$Po = \alpha \times \omega \times Hr$

$= 1.0 \times 0.001 \times 324$

$= 0.324$ kgf/cm^2

where

α : Internal pressure coefficient ($\alpha = 1.0$)

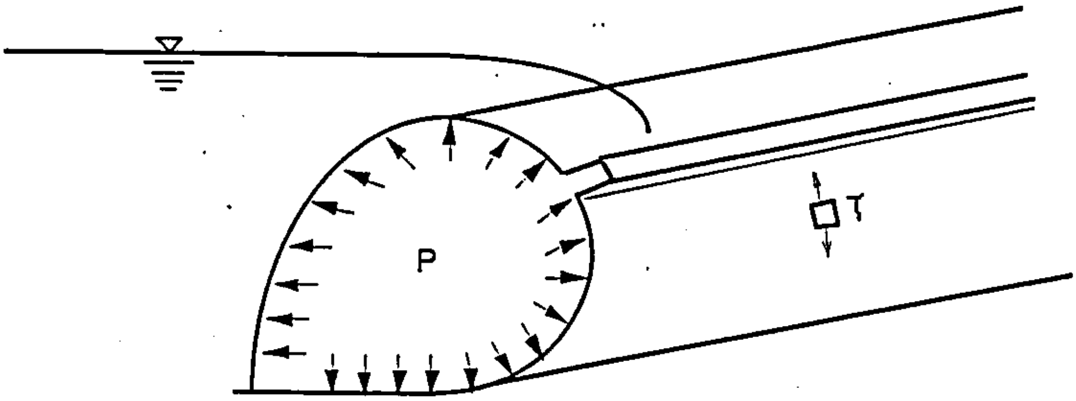
ω : Specific gravity of water ($\omega = 0.001$ kgf/cm^3)

Hr : Effective height of dam: 324 cm = 314 + 10

2 Tension on Rubber Dam Body

The Max. working tension is generated at max. over flow.

Tension on the dam body is applied only in the direction of water flow.



Tension (T) per unit width is obtained by following formulas;

$T = 0.5 \times \omega \times Hr^2 \times k_1 + hu \times k_2$

$= 0.5 \times 0.001 \times 324^2 \times 1.10 + 81 \times 0.031$

$= 60.2$ kgf/cm

where, k1: Correction factor for computer tension analysis(= 1.10)

k2:Correction factor for computer tension analysis(= 0.031)

hu:Maximum overflow height (=81cm)

3 Safety Factor (Sf)

$Sf = Ts / T$

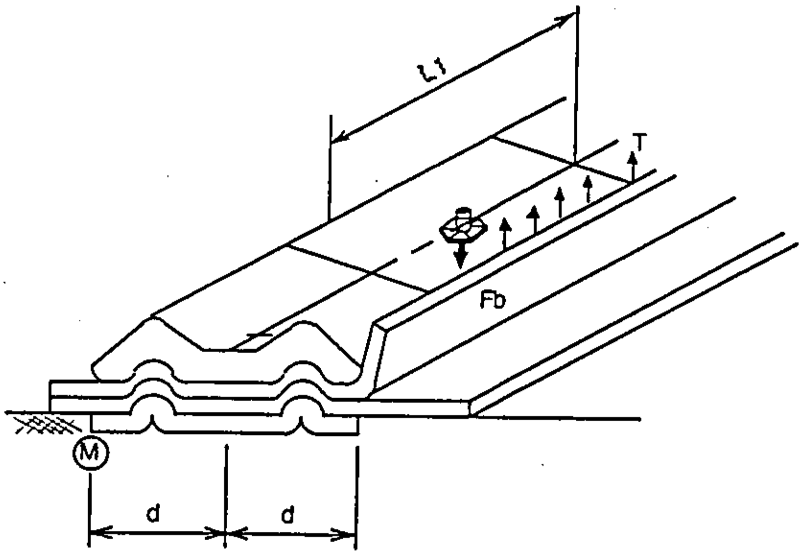
$= 590 / 60.2$

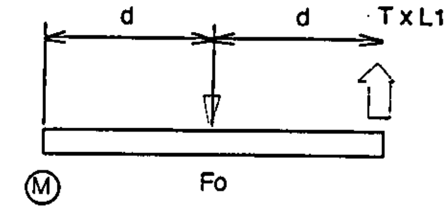
$= 9.8 > 8.0$ (Required Safety Factor)

where, Ts = Tensile strength of Rubber Dam body (min. 590 kgf/cm)

IV CALCULATION OF STRENGTH OF ANCHOR BOLTS

1 Clamping Force Per Anchor Bolt (Fb)





< MOMENT ABOUT M ON TENSION OF RUBBER >

$$Fo = 2\,T\,L1$$

Relationship between applied torque (Fb) and clamping force on the anchor bolt is taken based on results of experiment as follows:

Tr = 35.3 kgf-m	Anchor bolt torque
Fa = 5890 kgf	Clamping force
Ab = 5.61 cm^2 / M 30	Stress Area

$$Fb = Fa + 2 \cdot T \cdot L1$$
$$= 5890 + 2 \times 60.2 \times 20$$
$$= 8300 \text{ kgf}$$

2 Stress of the Anchor Bolt (sb) & Safety Factor (Sf)

$$\sigma bt = Fb/Ab$$
$$= 8300 / 5.61$$
$$= 1479 \text{ kgf/cm}^2$$

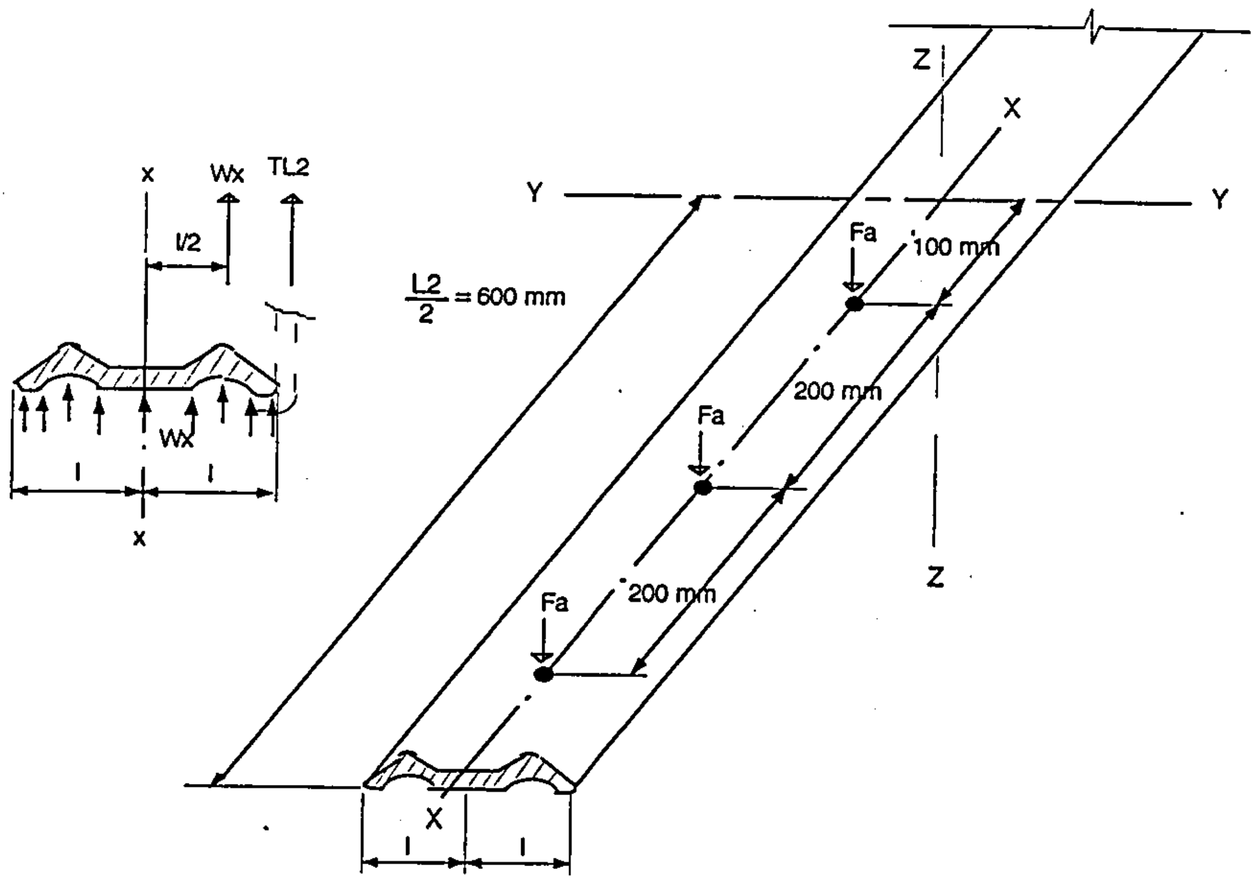
Breaking stress for anchor bolt : $\sigma bB = 5800 \text{ kgf/cm}^2$

Safety Factor : $Sf = \sigma bB / \sigma bt$

$$= 5800 / 1479$$
$$= 3.9 > 3.0 \text{ (Required Safety Factor)}$$

V CALCULATION OF STRENGTH OF CLAMPING PLATES

STRENGTH IN FLOW DIRECTION



1 Maximum Bending Moment About X-X Axis for One Clamping Plate

Mx = (T x L2 x l) + (Wx x l x l / 2)
= (60.2 x 120 x 12.1) + (1460 x 12.1 x 12.1 / 2)
= 194383.3 kgf-cm

Wx: Uniform Load of reaction force of one clamping plate secured with 6 bolts.
Wx = 6 x 5890 / (2 x l)
= 6 x 5890 / (2 x 12.1)
= 1460 kgf/cm

2 Bending Stress & Safety Factor

$\sigma s = Mx / Zx$

$= 194383.3 / 130.1$

$= 1494.1 \text{ kgf/cm}^2$

Zx: Sectional Modulus ($Zx = 130.1 \text{ cm}^3$)

Breaking stress for clamping plate: $\sigma cB = 5000 \text{ kgf/cm}^2$

$Sf = \sigma cB / \sigma s$

$= 5000 / 1494.1$

$= 3.3 > 3.0$ (Required safety factor)

VII CALCULATION OF INFLATION AND DEFLATION TIMES

1 Specification of Air Compresser

Pressure of air compressor should be more than the water head pressure. Taking into account some marginal pressure, air compressor specifications are determined as follows:

1) Maximum static pressure : 4000 mmAq of waterhead

2) Effective discharge capacity: Qs= 12.0 m^3/min

2 Time Needed for Inflation Dam

Inflation time depends on the capacity of the air compressor and the internal volume of the Rubber Dam:

$$t = Vo \times (1.033 + Po) / (1.033 \times Qs \times Kq)$$
$$= 198.7 \times (1.033 + 0.324) / (1.033 \times 12.0 \times 0.9)$$
$$= \text{Approx. } 26 \text{ minutes } (< 30 \text{ min.})$$

where:
Qs: Discharge capacity (12.0 m^3/min)
Kq: Loss rate (0.9)
Vo: Internal volume of Rubber Dam of each span
($Vo = 0.97 \cdot Hr^2 \cdot L$) $= 0.97 \times 3.24^2 \times 19.5 = 198.7$ m^3
Po: Design pressure (= 0.324 kgf/cm^2) .

The representative length (L) depends on the slope and it is calculated as follows, assuming the length of the riverbed to be Lo:

Bank Slop	1 /0	1 /0.3	1 /0.5	1 /1	1 /1.5	1 /2
(L)	Lo	Lo + 0.3 · Hr	Lo + 0.5 · Hr	Lo + 1.0 · Hr	Lo + 1.5 · Hr	Lo + 2 · Hr

3 Time Needed for Deflating Dam

Deflation time depends on the upstream water level and pressure loss of pipeline. It is calculated by the formula below.

$$t = \frac{V}{\pi / 4 \times d^2 \times u \times 60} \quad \text{(min.)}$$

where:

V: Internal Air volume of dam (m^3) = (1.033 + 0.324) / 1.033 x 198.7 = 261.0 m^3

d: Diameter of pipe (0.10 m)

u: Average velocity of exhausting air (m/s)

$$u = [\, 2 \times Pav / \{ \rho \times (1 + \lambda \times L2 / d + \Sigma\zeta) \}]^{1/2}$$

(where)

Pav: Average pressure (= 0.2 x Pd x g = 7938 kgf/m·sec^2)

g : Gravity acceleration(= 9.8 m/sec^2)

Pd : Inner pressure at deflation start (=(Hr + hu) waterhead =4.05 mAq = 4050 kgf/m^2)

λ : Frictional coefficient as per pipe size (= 0.02)

$\Sigma\zeta$: Pressure loss coefficient by elbow (= 4.80)

L2 : Length of piping (approx. 50 m)

ρ : Density of exhausting air (= 1.2 kgf/m^3)

$$u = [\, 2 \times 7938 / \{ 1.2 \times (1 + 0.02 \times 50 / 0.10 + 4.80) \}]^{1/2}$$
$$= 28.9 \text{ m/s}$$

$$t = 261.0 / (\pi/4 \times 0.10^2 \times 28.9 \times 60)$$
$$= \text{Approx. } 19 \text{ minutes } (< 30 \text{ min.})$$

2.4 Diversion Weirs

The Deli River Weir and the Floodway Weir are planed at the outlet of retarding channel to Deli river and Floodway.. These diversion weirs are designed to meet the basic conditions such as hydraulic requirements, structural stability, topographic constraints and economy.

2.4.1 Design Conditions

<u>Hydraulic Requirements</u>

The conditions for hydraulic design of weirs are as follows:

(a) The design water level in the retarding channel is set at EL. 34.00 m for both Immediate and Urgent plans.

(b) The discharge of 70 m^3/s out of 300 m^3/s is diverted into the Floodway through the Floodway Diversion Weir and the remaining discharge of 230 m^3/s into the downstream channel of Deli River in the Immediate Plan.

(c) In the Urgent Plan stage, the diversion discharge for the Floodway will be increased to 120 m^3/s out of 320 m^3/s, while the discharge for the Deli River will be decreased to 200 m^3/s. Therefore, both weirs are required to be modified to meet the change of design discharge between the two stages.

(d) In ordinary time, the whole discharge of Deli River flows into its downstream without diversion to the Floodway to maintain the current water uses in the downstream area.

<u>Premises on Weir Design</u>

In compliance with the above hydraulic requirements, the basic design is prepared based on the following premises.

(a) The B-P Study has recommended the use of fixed weir for both the Deli River and Floodway weirs through a comparative study. In this D/D Study, further comparative study was made, and the fixed weir is justified as the suitable type for the diversion weirs.

(b) To avoid a so-called man-made flood, gates which serve for regulating diversion, are not provided for the Deli River Weir.

(c) An orifice is provided in the weir body to enable the low water to flow down towards the downstream of Deli River. The orifice is designed on condition that a discharge of 10.6 m^3/s, which corresponds to 95-day discharge (occurrence probability of 25%), is carried with some vertical clearance in the orifice.

(d) Modification of the weirs, resulting from the change of diversion discharges between Immediate and Urgent plans, is made by adjusting the crest elevation of weirs.

(e) To confirm the perfect overflow at the crest, the downstream water level shall be lower than the elevation of 15 % of the over flow depth.

(f) An overflow depth shall be less than 3 m.

Following the above premise, the boundary condition of the Diversion weirs are;

		Downstream Water Level DL	Maximum Overflow Depth OD =3/2x(EL.34.0-DL)	Lowest Crest Elevation ME =EL.34.0m-OD
Deli River Weir	Immediate Plan	EL. 31.10 m	3.00 m (3.41 m)	EL. 31.0 m
	Urgent Plan	EL. 30.60 m	3.00 m (4.00m)	EL. 31.0 m
Floodway Weir	Immediate Plan	EL. 31.65 m	2.76 m	EL. 31.24 m
	Urgent Plan	EL. 32.30 m	2.00 m	EL. 32.00 m

Location of Weirs

(a) Deli River Weir

To avoid construction of weir in the narrow and deep downstream channel, the weir is placed at River Section UD. 12.00 (refer to Fig. 4.2.10) which is the entrance portion to the narrow downstream from the retarding channel.

(b) Floodway Weir

The location of the weir is at the entrance portion to the Floodway, which is about 40 m away from the existing low water channel of Deli River.

Type of Weir

There are three types of fixed weirs applicable for the site, as follows:

(a) Gravity Type, trapezoid-shaped, ogee and sharp-crested
(b) Armored Dike, trapezoid-shaped and broad-crested
(c) Concrete Step Type (semi-concrete gravity)

Of these weirs, gravity type trapezoid-shaped weir and armoured earth dike type are compared. As a result, gravity type trapezoid-shaped weir is proposed as the most suitable type for the following reasons:

(a) The maximum water heads are 9.8 m for the Deli River Weir and 7.5 m for the Floodway Weir. These water heads can induce enormous hydrostatic force and uplift pressure. To resist these forces and to keep structural stability, the weir body should be of a rigid structure with heavy weight. In this context, the gravity type is preferred for these weirs.

(b) The gravity type trapezoid-shaped weir is more economical than the armoured earth dike type in terms of construction cost.

(c) There exists the uncemented Toba Tuff beneath the riverbed surface, which has a high bearing capacity. This layer is evaluated as a foundation for the gravity type of weir.

(d) To obtain a precise and safe diversion of flood flow with a big overflow depth, the trapezoid-shaped fixed weir is preferred, since this type of weir has been applied in many projects in Indonesia and Japan.

(e) The Deli River weir is provided with orifice in the weir body. These openings can cause the decrease of weir stability and safety. To ensure the weir's stability and safety, the gravity type should be employed.

(f) The gravity type is easy to be modified at the portion of the weir crest for the modification from the Immediate Plan to the Urgent Plan.

Foundation of Weirs

Toba Tuff, which was formed in diluvium stratum and is stiff sandy soil, will be applied to the bearing foundation of weirs. The results of SPT of Toba Tuff shows N=30 to more than 50 and the coefficient of permeability is $k = 10^{-4}$ to 10^{-5} cm/s. Judging form these soil properties, Toba Tuff is evaluated an adequate stratum to the spread foundation for 5 to 10 m high gravity weir.

The details of foundation design of both diversion weirs are;

(1) Deli River Weir

A outcrop of stiff Toba Tuff stratum is observed at the riverbed and the stratum which has a high bearing capacity (N value is mostly more than 40) is confirmed at EL. 22 to 23.0 m in the Boring No. B35, B38 and B39. The base elevation of the weir is set at EL. 21.7 m which is 2.5 m bellow the surface of apron.

An un-consolidated clay layer which N value is about 10 and whose thickness is about 2 to 4 m, was found in the Deli river weir foundation at EL. 8 m. Considering the foundation settlement due to the consolidation by the construction of the weir , consolidation test to the clay layer was carried out during this study.

The additional vertical load in clay layer by weir ($\Delta\sigma z$ kgf/cm^2) is estimated by following formula.

$\Delta\sigma z = qBL/((B+2\ Z \tan\theta)(L+2\ Z \tan\theta))$

where,	Z	:	depth of cover soil (EL. 21.5m - EL. 8.0 m = 13.5 m)
	B	:	width of weir (B = 17.5 m)
	L	:	length of weir (L = 12.0 m)
	q	:	extra load ($q = 0.604\ kg/m^2$)
	θ	:	angle of load spreading ($\theta = 30°$)

$\Delta\sigma z = 0.139\ kgf/cm^2$

The effective soil load (Po) is calculated as bellow.

$Po = Z \times \gamma sub = 1350\ cm \times (0.0019 - 0.001)\ kgf/cm^3 = 1.215\ kgf/cm^2$

The total load at the clay layer after construction of the weir (P) is:

$P = Po + \Delta\sigma z = 1.354\ kgf/cm^2$

On the other hand, the consolidation yield stress (Pc) is obtained by the settlement test.

log Pc = 0.85 $\qquad$ $Pc = 7.08\ lgf/cm^2$

Since P is smaller than Pc, the settlement will not be occurred. Therefore, the spreading foundation can be applied to the Deli river weir foundation.

(2) Floodway Weir

The results of boring B37 and additional boring B40 and 42 along the Floodway axis show that the stiff and uncemented Toba Tuff as a supporting layer (N value is more than 40) appears at EL. 20.0 m to 23.0m. If spread foundation is applied to the weir, the embedment depth of weir reaches 5 m in spite of only 6 m weir height.

On the other hand, if pile foundation is applied to the Floodway weir, the required pile length is only 3 to 4 m. To provide such short piles has an economical disadvantage.

Therefore, to ensure the strength of the foundation, a foundation improvement works by cement-treated material is employed from the economical point of view. The thickness of to be improved foundation is 3 m and the area is 2 m wider than the weir base. The required strength of improved foundation is 70 tf/m^2 and it is assumed that about 70 to 100 kg/m^3 of cement will be sprinkled.

2.4.2 Hydraulic Design

(1) Hydraulic Model Test and Modification of Tested Design

Previous to the structural design of the diversion weirs, the hydraulic model test had been conducted to confirm the flow condition of the retarding channel and to examine the successful diversion by the weirs. As a result of the hydraulic model test, the following modifications are proposed.

- Crest elevation of Floodway weir
- Baffles for orifice
- Length of apron of Deli River weir
- Approach wall (wing walls) of Deli River Weir

In addition, a hollow was observed on the vein of overflow water at the Floodway weir crest.

To check the affection of this hollow, the tested and the calculated overflow coefficients of Floodway weir are compared as shown bellow.

Discharge Q	Water Level EL. m	Water Depth h	Overflow Coefficient	
			Tested	Calculated
99.0	34.32	1.82	1.52	1.73
70.0	33.98	1.48	1.47	1.67
29.5	33.44	0.94	1.22	1.50

Note: Overflow coefficients are calculated with Beresinski's formula.

The tested coefficients of overflow are 14 % to 23 % of the calculated. This results shows that this hollow obstruct the smooth and effective flow at the crest and these difference are caused by the warped shape of crest.

Therefore, the rounded crest is employed to avoid the occurrence of cabitation at the crest edge. The method of raising/cutting of weir crest for the urgent plan is also changed as shown in Fig. 2.4.1.

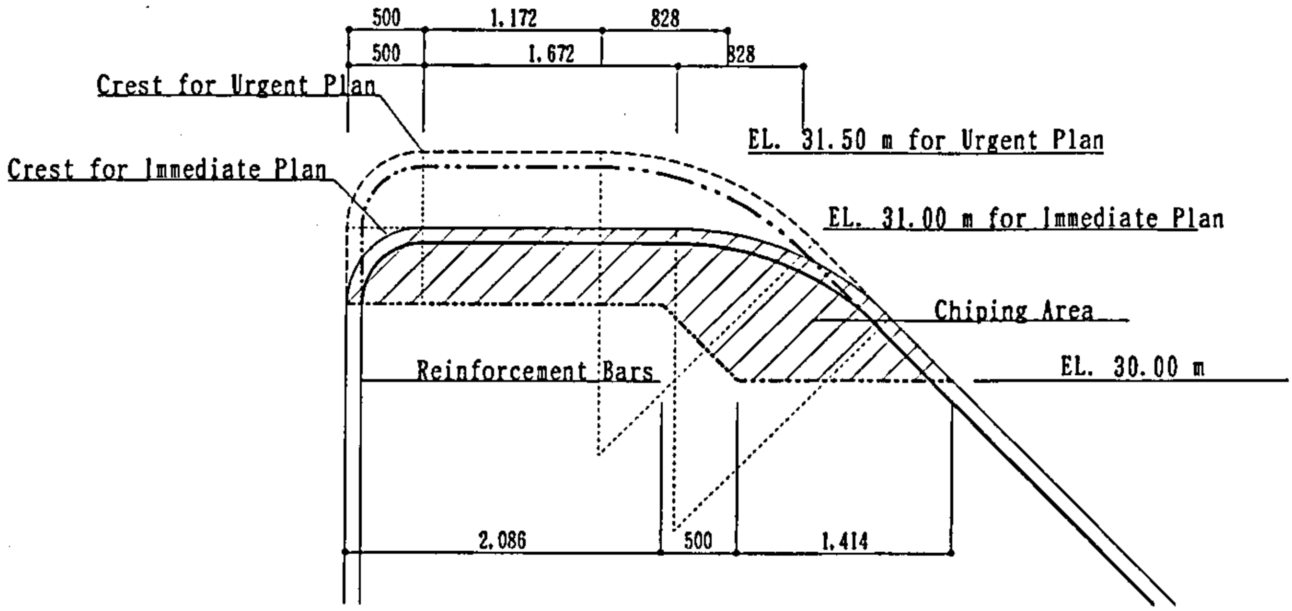
The dimensions of weirs are recalculated as bellow.

(3) Discharge Equations of Flow Over/Below Weir

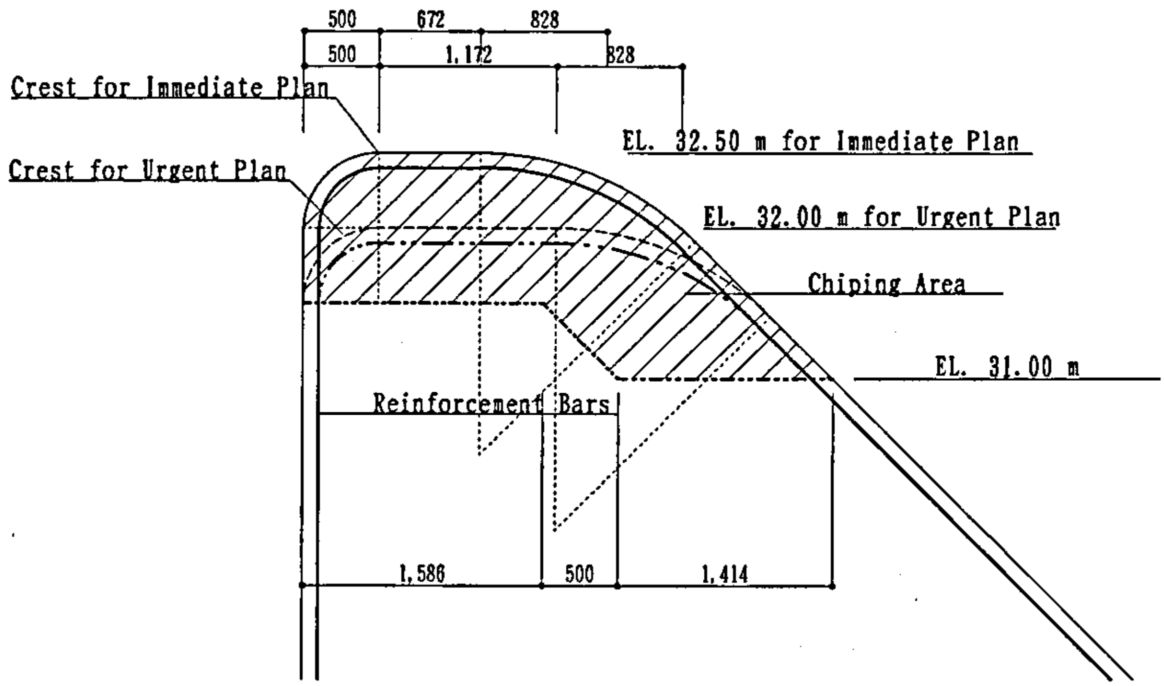
The following equations are employed to the hydraulic designs for both diversion weirs.

(a) Flow Over Trapezoid Shaped Weir

The discharge equation for flow over the rounded trapezoid-shaped weir is as follows (Beresinski's Formula):



Deli River Weir



Floodway Weir

Fig. 2.4.1 MODIFICATION PLAN OF DIVERSION WEIR CREST

$$Q = C \times B \times h^{3/2} \qquad \text{(for non-submerged flow)}$$

where,

- Q : discharge (m^3/s)
- C : discharge coefficient
  - $2.5<L/h<10$;
    - $C=1.706\{1+1.146(W/h)\}/\{1+1.250(W/h)\}$
  - $0.3<L/h<2.5$ and $W/h>2$to 2.5;
    - $C=1.373(0.984+L/h)/(0.500+L/h)$
- B : length of the weir (m)
- h : energy head on the weir in upstream side (m)
- W : height of weir (m)
- L : Length of crest (m)

(b) Flow of Orifice

The following formula is used to calculate the discharge flowing through the orifice of the weir:

$$Q = a\,b\,C\,(2\,g\,h_1)^{1/2}$$

where,

- Q : discharge (m^3/s)
- a : opening of orifice (m)
- b : width of orifice (m)
- g : acceleration of gravity (9.8 m/s^2)
- h_1 : water depth in front of the weir
- C : discharge coefficient obtained as shown below:

- When flow through the orifice is free efflux, C is given as follows:

$$C = C_c\,[(1-C_c\,a/\,h_1)/\{1-(C_c\,a/h_1)^2\}]^{0.5}$$

where,
C_c: shrinkage factor (0.606)

- When submerged flow occurs, the following equations are used:

$$F(C)=AC^4+BC^2+D$$

where,

$$A=\{a/h_1-h_1/(C_c^2\,a)\}^2$$

$$B=2\{a/h_1-h_1/(C_c^2\,a)\}h_1/a-4\{h_1/h_2-h_1/(C_c a)\}$$

$D=(h_1/a)^2-(h_2/a)^2$

h_1 : downstream water depth (m)

(4) Deli River Weir

Immediate Plan

As mentioned above sections, the lowest crest elevation and maximum overflow depth for immediate plan of Deli River weir are set at;

Crest Elevation	EL. 31.00 m
Overflow Depth	3.0 m

The weir length are calculated as below:

- Discharge from orifice
  The discharge form orifice is restricted by the downstream water level. Since the downstream water level under the 230m^3/s flow is EL. 31.1 m, the discharge from orifice Q = 68.9 m^3/s is obtained from the above formula.

- Discharge from overflow section
  Since the discharge of 68.9 m^3/s out of 230 m^3/s is released form the orifice, the remaining discharge of 161.1 m^3/s will be discharged from the overflow section.

- Required weir length
  The required weir length is calculated as below;
  $B=Q/Ch^{3/2}$ = 161.1/(1.816 x $3^{3/2}$) = 17.10 = <u>17.5 m</u>

  where
  C=1.816 (L = 3.0 m, h = 3.0m)

Urgent Plan

For the modification of the weirs from Immediate plan to Urgent plans, only the crest elevation of the weir is adjusted and the weir length will not be changed. The crest elevation for Urgent plan is calculated as bellow:

- Discharge from orifice (refer to Fig. 3.2.*)
  Downstream water level EL. 30.60 m
  Discharge Q = 78.5 m^3/s

- Discharge from overflow section is;
  Q = 200 m^3/s - 78.5 m^3/s = 121.5 m^3/s

- Required overflow depth is;
  $h=(Q/CB)^{2/3}=\{121.5/(1.816 \times 17.5)\}^{2/3}=2.45=\underline{2.5\ m}$
  where
  C=1.816 (L = 2.5 m, h = 2.5m)

- Crest Elevation is;
  EL. 34.0 m - 2.5 m = EL. 31.5 m

(4) Floodway Weir

Urgent Plan

The dimensions of Floodway weir are determined by the condition of the urgent plan as mentioned above. To obtain the perfect overflow at the crest, the lowest crest elevation and maximum overflow depth for urgent plan of Floodway weir are set at;

Crest Elevation	EL. 32.00 m
Overflow Depth	2.0 m

The required weir length is calculated as below;

$B=Q/Ch^{3/2}=120/(1.709 \times 2.0^{3/2})=24.83=\underline{25\ m}$
where
C=1.709 (L = 2.5 m, h = 2.0m)

Immediate Plan

To discharge the design discharge of immediate plan with 25 m wide weir, the required overflow water depth is;

$h=(Q/CB)^{2/3}=\{70/(1.692 \times 25.0)\}^{3/2}=1.46=\underline{1.5\ m}$
where
C = 1.692 (L = 2.5 m, h = 1.5m)

- The crest elevation is;
  EL. 34.0 m - 1.5 m = EL. 32.5 m

Structural Dimensions

The dimensions determined through the hydraulic design calculation are as follows:

(1) Diversion weir

Item	Dimensions
Structural Type	Gravity, Trapezoid-Shaped Weir
Design Water Level	EL. 34.0 m
Elevation of Crest	EL. 31.0 m (EL. 31.5 m)
Elevation of Apron	EL. 24.2 m
Elevation of Foundation	EL. 21.7 m
Length of Overflow Weir Crest	17.50 m
Width of Overflow Weir Crest	3.00 m (2.50 m)
Height of Overflow Weir Crest	6.80 m (7.30 m)
Orifice for Low Water	3.00 m x 2.00 m
Number of Orifice	2
Bottom Elevation of Orifice	EL. 24.7 m
Overflow Depth	3.00 m (2.50 m)
Inclination of Weir Face	Vertical (up) and 1 : 1 (down)
Modification of Weir from Immediate to Urgent Plan	0.50 m (Raising weir crest)

Note: Figures in parentheses are for the Urgent Plan.

(b) Floodway Weir

Item	Dimensions
Structural Type	Gravity, Trapezoid-Shaped Weir
Design Water Level	EL. 34.0 m
Elevation of Crest	EL. 32.5 m (EL. 32.0 m)
Elevation of Apron	EL. 26.5 m
Elevation of Foundation	EL. 24.0 m
Length of Overflow Weir Crest	25.00 m
Width of Overflow Weir Crest	2.00 m (2.50 m)
Height of Overflow Weir Crest	6.00 m (5.50 m)
Overflow Depth	1.50 m (2.00 m)
Inclination of Weir Face	Vertical (up) and 1 : 1 (down)
Modification of Weir from Immediate to Urgent Plan	0.50 m (Lowering weir crest)

Note: Figures in parentheses are for the Urgent Plan.

2.4.3 Structural Design

<u>Stability Analysis of Weirs</u>

(1) Loading Combination and Safety Factor

The stability analysis of weir are conducted under the following load combinations and safety conditions.

	Loading Combination	Increase in Allowable Stress	Safety Factor	
			Sliding	Bearing Capacity
Case 1	$M+T+Th_n+U$	0 %	1.5	3
Case 2	$M+T+Th_n+G+U$	20 %	1.3	2
Case 3	$M+T+Th_b+U$	20 %	1.3	3
Case 4	$M+T+Th_b+G+U$	50 %	1.1	2
Case 5	M	0 %	-	3

where

M : dead load
T : sediment pressure
Thn : normal water pressure
Thb : water pressure during flood
G : earthquake load
U : uplift

(2) Design Factors

(a) Horizontal Earthquake Factor

The horizontal earthquake factor is calculated using the following equations.

$$E = ad / g$$
$$ad = n\,(ac \cdot Z)^{m}$$

where; E : horizontal earthquake factor
ad : design shock acceleration (cm/s^2)
ac : basic shock acceleration (= 160 cm/s^2 corresponding to a 100 year return period)
g : acceleration of gravity (980 cm/s^2)
Z : factor depending on geographic position (= 1.0 at Medan City)
n, m : coefficients for soil types.
For the diluvium foundation, following values are employed:

	n	m
Diluvium	0.87	1.05

The following horizontal earthquake factor is applied to the calculation of the earthquake load.

$E = 0.87 \times (160 \times 1.0)^{1.05} / 980 = \underline{\mathbf{0.183}}$

(b) Internal Friction Angle

The internal friction angle of cohesionless soil shall be determined either by direct shear test, unconfined compression test or tri-axial test, but for sandy soil or if there is no data available, it can be estimated by the following formula.

$$\phi = 15 + (15 \times N)^{0.5} \leq 45° \quad (\text{for } N > 5)$$

where, ϕ : internal friction angle (degree)

N : "N" value by standard penetration test

Since the foundation of weirs is set on the stiff and uncemented Toba Tuff which has the N value of more than 40, the internal friction angle of the weir foundation is calculated as;

$$\phi = 15 + (15 \times 40)^{0.5} = \underline{40°}.$$

(c) Allowable Soil Bearing Capacity

The ultimate soil bearing capacity of a foundation is calculated by the following formula :

$$Qu_1 = 1/3\ (\alpha \cdot c \cdot Nc + \beta \cdot \gamma_1 \cdot B \cdot Nr + \gamma_2 \cdot Df \cdot Nq\}$$
$$Qu_2 = 2/3\ (\alpha \cdot c \cdot Nc + \beta \cdot \gamma_1 \cdot B \cdot Nr + 1/2\ \gamma_2 \cdot Df \cdot Nq\}$$

where,

Qu_1 : ultimate bearing capacity for long term load

Qu_2 : ultimate bearing capacity for short term load

α, β : coefficient depending on shape of footing as shown in the following table.

Shape of Footing	α	β
Excessively long rectangle	0.1	0.5

c : cohesion of foundation ground (= 0 tf/m^2)

γ_1 : unit weight of soil of below the foundation (= 0.9 tf/m^3, submerged)

γ_2 : unit weight of soil of above the foundation (= 0.7 tf/m^3, submerged)

B : minimum width of effective loading area (= 1 m)

Df : depth from ground surface to bottom of footing (= 2.5 m)

Nc, Nq, Ng : bearing capacity factors corresponding to the internal friction angle (ϕ = 40°, Nc=95.7, Nq = 83.2, Nr = 114.0)

From the above equations, the following ultimate soil bearing capacities of the weir foundation are obtained.

$Qu_1 = 65.6$ tf/m^2,
$Qu_2 = 82.7$ tf/m^2.

(3) Safety of Weir

The main body with direct foundation should be safe against sliding, overturning, failure of foundation ground and uplift.

(a) Sliding

The factor of safety against sliding is determined by using the following formula:

$$SF = \frac{C.A + \sum(V - U).\tan\phi}{\sum H}$$

where,
H : total horizontal forces
V : vertical forces
U : uplift forces
ϕ : coefficient of internal friction
C : cohesion value
A : structure base area
SF : safety factor given in the following
$SF \geq 1.5$ (under normal condition)
$SF \geq 1.2$ (under floods and earthquake condition)

(b) Overturning

All forces acting on part of the structure above any horizontal plane should fall within the middle third of the structure base.

For this purpose, the following condition should be satisfied :

$$e = \frac{b}{2} - \frac{M}{N} < \frac{b}{6}$$

where,
b : width of base (m)
M : total moment about point A (tf . m)
N : total vertical forces (tf)
e : eccentricity (m)

(c) Bearing Capacity of Spread Foundation

The maximum principal stress in the spread foundation must be kept within allowable soil bearing capacity which is derived from the following :

$$Qa = \frac{Qu}{SF}$$

where,

Qa	:	allowable soil bearing capacity
Qu	:	ultimate soil bearing capacity
		$Qu_1 = 55$ tf/m^2 for long term load,
		$Qu_2 = 73$ tf/m^2 for short term load.
SF	$\geq$	3 (under the normal condition)
SF	$\geq$	2 (under the earthquake condition)

The allowable soil bearing capacity for the

diversion weirs are;

$Qa_1 = 65.6$ tf/m^2 / 3 = 21.8 tf/m^2 for long term load,
$Qa_2 = 82.7$ tf/m^2 / 2 = 41.3 tf/m^2 for short term load.

(4) Results of Calculation

The results of stability analysis is summarized in Table 2.4.1 and calculation sheets for each case are attached following pages.

Table 2.4.1 SUMMARY OF STABILITY ANALYSIS OF DIVERSION WEIRS

Floodway Weir						
Immediate		CASE 1	CASE 2	CASE 3	CASE 4	CASE 5
	B=	9.500	9.500	9.500	9.500	9.500
	B/6 =	1.583	1.583	1.583	1.583	1.583
	e =	-1.336	-0.994	-0.321	-0.338	-1.427
	B/6-e	0.247	0.589	1.262	1.245	0.156
	FS=	2.448	3.636	3.341	1.228	20.982
	U1=	10.582	15.117	8.300	6.963	20.285
	U2=	0.895	3.456	5.500	4.514	1.050
Urgent	B=	10.000	10.000	10.000	10.000	10.000
	B/6 =	1.667	1.667	1.667	1.667	1.667
	e =	-1.480	-1.017	-0.649	-0.432	-1.463
	B/6-e	0.186	0.650	1.017	1.234	0.203
	FS=	2.591	3.677	6.039	1.246	22.806
	U1=	10.897	15.502	10.490	7.268	20.687
	U2=	0.645	3.754	4.607	4.273	1.345
Deli River Weir						
Immediate	B=	12.300	12.300	12.300	12.300	12.300
	B/6 =	2.050	2.050	2.050	2.050	2.050
	e =	-1.177	-1.103	-0.687	0.323	-1.935
	B/6-e	0.873	0.947	1.363	1.727	0.115
	FS=	2.546	2.537	3.135	1.138	8.074
	U1=	8.846	13.190	6.478	4.736	19.204
	U2=	2.393	3.961	3.225	6.504	0.552
Urgent	B=	12.300	12.300	12.300	12.300	12.300
	B/6 =	2.050	2.050	2.050	2.050	2.050
	e =	-1.003	-1.144	-0.424	0.186	-1.977
	B/6-e	1.047	0.906	1.626	1.864	0.073
	FS=	2.075	2.531	2.414	1.176	7.546
	U1=	8.216	13.771	5.730	5.363	19.922
	U2=	2.817	3.906	3.767	6.432	0.360

Stability Analysis

Deli River Weir (Immediate Plan)
Case 1 , Nomal Condition (WL. EL. 31.0 m, k=0.000)

Basic Condition				Stability Condition		
Water Level	Upstream	EL.	31.000 m	U.W.. of Concrete		2.350 t/m3
	Downstream	EL.	28.310 m	Coefficient of Earth quake		0.000
EL. of Crest		EL.	31.000 m	U.W. of Sediment		1.800
EL. of Foundation		EL.	21.700 m	Coefficient of Sediment P.		0.500
Crest Width		B1	3.000 m	Foundation	C	0.000
Slope	Downstream	1:	1.000		tan ϕ	0.838
U.P. Slope	Upstream	1:	0.000		Bearing strength	21.867 t/m2
Orifice	Height	Ho	2.000 m	Required Factor of Safety		1.500
	Width	Wo	6.000 m	Uplift C.	L1=apron+base	39.800 m
	Length	Lo	8.300 m		L2=base	12.300
	Orifice Center	EL.	25.700 m		μ	1.00
Block Width		Wb	17.500 m		Up1	100%
Weir Height		Hw	9.300 m		Up2	26%
EL. of Sedimentation			27.900			

Load			Vertical	Horizontal	Arm	Moment
Self Weight	Center	W1	1147.388		1.500	1,721.081
	Downstream	W2	1,778.451		6.100	10,848.549
	Upstream	W3	0.000		0.000	0.000
	Orifice	Wo	-234.060		4.150	-971.349
Hydraulic	Horizontal US	P1		0.000	4.650	0.000
Pressure	Horizontal US	P2		756.788	3.100	2,346.041
	Horizontal DS	P5		-382.306	2.203	-842.347
	Orifice	Po		-63.600	4.000	-254.400
Water Weight	Crest	P3	0.000		1.000	0.000
	Down Stream	P4	382.306		10.097	3,860.015
	Upstream	P6	0.000		0.000	0.000
Seismic Force	Center	Ke1		0.000	4.650	0.000
	Downstream	Ke2		0.000	3.100	0.000
	Upstream	Ke3		0.000	3.100	0.000
	Orifice	Keo		0.000	4.000	0.000
Sediment Pressure		Ps		87.451	2.067	180.732
Uplift		U1	-289.511		4.100	-1,186.996
		U2	-1,574.905		6.150	-9,685.666
Total			1,209.668	398.333		6,015.660

Stability Calculation

4.973

Middle third	B=	12.300 m		6.150
	B/6 =	2.050 m		
	e =	-1.177 m	B/6>e OK	
	(B/6-e	0.873)		
Sliding	FS=	2.546	OK	
Bearing strength	U1=	8.846 t/m2	OK	
	U2=	2.393 t/m2	OK	

Stability Analysis

Deli River Weir (Immediate Plan)
Case 2 , Earthquake Condition (WL. EL. 26.5m, k= 0.183)

Basic Condition				Stability Condition		
Water Level	Upstream	EL.	26.500 m	U.W.. of Concrete		2.350 t/m3
	Downstream	EL.	26.000 m	Coefficient of Earth quake		0.183
EL. of Crest		EL.	31.000 m	U.W. of Sediment		1.800
EL. of Foundation		EL.	21.700 m	Coefficient of Sediment P.		0.500
Crest Width		B1	3.000 m	Foundation	C	0.000
Slope	Downstream	1:	1.000		tan ϕ	0.838
U.P. Slope	Upstream	1:	0.000		Bearing strength	41.350 t/m2
Orifice	Height	Ho	2.000 m	Required Factor of Safety		1.300
	Width	Wo	6.000 m	Uplift C.	L1=apron+base	39.800 m
	Length	Lo	8.300 m		L2=base	12.300
	Orifice Center	EL.	25.700 m		μ	1.00
Block Width		Wb	17.500 m		Up1	100%
Weir Height		Hw	9.300 m		Up2	26%
EL. of Sedimentation			27.900			

Load			Vertical	Horizontal	Arm	Moment
Self Weight	Center	W1	1147.388		1.500	1,721.081
	Downstream	W2	1,778.451		6.100	10,848.549
	Upstream	W3	0.000		0.000	0.000
	Orifice	Wo	-234.060		4.150	-971.349
Hydraulic Pressure	Horizontal US	P1		0.000	4.650	0.000
	Horizontal US	P2		201.600	1.600	322.560
	Horizontal DS	P5		-161.788	1.433	-231.895
	Orifice	Po		-9.720	4.000	-38.880
Water Weight	Crest	P3	0.000		1.000	0.000
	Down Stream	P4	161.788		10.867	1,758.091
	Upstream	P6	0.000		0.000	0.000
Seismic Force	Center	Ke1		209.972	4.650	976.369
	Downstream	Ke2		325.456	3.100	1,008.915
	Upstream	Ke3		0.000	3.100	0.000
	Orifice	Keo		-42.833	4.000	-171.332
Sediment Pressure		Ps		87.451	2.067	180.732
Uplift		U1	-53.813		4.100	-220.631
		U2	-953.847		6.150	-5,866.158
Total			1,845.906	610.139		9,316.052

Stability Calculation

5.047

Middle third	**B=**	12.300 m		6.150
	B/6 =	**2.050 m**		
	e =	**-1.103 m**	**B/6>e OK**	
	(B/6-e	0.947)		
Sliding	**FS=**	**2.537**	**OK**	
Bearing strength	**U1=**	**13.190 t/m2**	**OK**	
	U2=	**3.961 t/m2**	**OK**	

Stability Analysis

Deli River Weir (Immediate Plan)
Case 3 , Flood Condition (WL. EL. 34.0m k=0.000)

Basic Condition				Stability Condition		
Water Level	Upstream	EL.	34.000 m	U.W.. of Concrete		2.350 t/m3
	Downstream	EL.	31.100 m	Coefficient of Earth quake		0.000
EL. of Crest		EL.	31.000 m	U.W. of Sediment		0.000
EL. of Foundation		EL.	21.700 m	Coefficient of Sediment P.		0.500
Crest Width		B1	3.000 m	Foundation	C	0.000
Slope	Downstream	1:	1.000		tan ϕ	0.838
U.P. Slope	Upstream	1:	0.000		Bearing strength	21.867 t/m2
Orifice	Height	Ho	2.000 m	Required Factor of Safety		1.300
	Width	Wo	6.000 m	Uplift C.	L1=apron+base	39.800 m
	Length	Lo	8.300 m		L2=base	12.300
	Orifice Center	EL.	25.700 m		μ	1.00
Block Width		Wb	17.500 m		Up1	100%
Weir Height		Hw	9.300 m		Up2	26%
EL. of Sedimentation			27.900			

Load			Vertical	Horizontal	Arm	Moment
Self Weight	Center	W1	1147.388		1.500	1,721.081
	Downstream	W2	1,778.451		6.100	10,848.549
	Upstream	W3	0.000		0.000	0.000
	Orifice	Wo	-234.060		4.150	-971.349
Hydraulic Pressure	Horizontal US	P1		488.250	4.650	2,270.363
	Horizontal US	P2		756.788	3.100	2,346.041
	Horizontal DS	P5		-756.788	3.100	-2,346.041
	Orifice	Po		-99.600	4.000	-398.400
Water Weight	Crest	P3	78.750		1.000	78.750
	Down Stream	P4	773.150		9.200	7,112.980
	Upstream	P6	0.000		0.000	0.000
Seismic Force	Center	Ke1		0.000	4.650	0.000
	Downstream	Ke2		0.000	3.100	0.000
	Upstream	Ke3		0.000	3.100	0.000
	Orifice	Keo		0.000	4.000	0.000
Sediment Pressure		Ps		-109.314	2.067	-225.915
Uplift		U1	-312.113		4.100	-1,279.661
		U2	-2,187.327		6.150	-13,452.059
Total			1,044.239	279.336		5,704.338

Stability Calculation

5.463

Middle third	B=	12.300 m		6.150
	B/6 =	2.050 m		
	e =	-0.687 m	B/6>e OK	
	(B/6-e	1.363)		
Sliding	FS=	3.135	OK	
Bearing strength	U1=	6.478 t/m2	OK	
	U2=	3.225 t/m2	OK	

Stability Analysis

Deli River Weir (Immediate Plan)
Case 4 , Critical Condition (WL. EL. 31.0m k=0.183)

Basic Condition				Stability Condition		
Water Level	Upstream	EL.	31.000 m	U.W.. of Concrete		2.350 t/m3
	Downstream	EL.	28.310 m	Coefficient of Earth quake		0.183
EL. of Crest		EL.	31.000 m	U.W. of Sediment		1.800
EL. of Foundation		EL.	21.700 m	Coefficient of Sediment P.		0.500
Crest Width		B1	3.000 m	Foundation	C	0.000
Slope	Downstream	1:	1.000		tan ϕ	0.838
U.P. Slope	Upstream	1:	0.000		Bearing strength	41.350 t/m2
Orifice	Height	Ho	2.000 m	Required Factor of Safety		1.100
	Width	Wo	6.000 m	Uplift C.	L1=apron+base	39.800 m
	Length	Lo	8.300 m		L2=base	12.300
	Orifice Center	EL.	25.700 m		μ	1.00
Block Width		Wb	17.500 m		Up1	100%
Weir Height		Hw	9.300 m		Up2	26%
EL. of Sedimentation			27.900			

Load			Vertical	Horizontal	Arm	Moment
Self Weight	Center	W1	1147.388		1.500	1,721.081
	Downstream	W2	1,778.451		6.100	10,848.549
	Upstream	W3	0.000		0.000	0.000
	Orifice	Wo	-234.060		4.150	-971.349
Hydraulic Pressure	Horizontal US1	P1		0.000	4.650	0.000
	Horizontal US2	P2		756.788	3.100	2,346.041
	Horizontal DS	P5		-382.306	2.203	-842.347
	Orifice	Po		-63.600	4.000	-254.400
Water Weight	Crest	P3	0.000		1.000	0.000
	Down Stream	P4	382.306		10.097	3,860.015
	Upstream	P6	0.000		0.000	0.000
Seismic Force	Center	Ke1		209.972	4.650	976.369
	Downstream	Ke2		325.456	3.100	1,008.915
	Upstream	Ke3		0.000	3.100	0.000
	Orifice	Keo		-42.833	4.000	-171.332
Sediment Pressure		Ps		87.451	2.067	180.732
Uplift		U1	-289.511		4.100	-1,186.996
		U2	-1,574.905		6.150	-9,685.666
Total			1,209.668	890.928		7,829.613

Stability Calculation

6.473

Middle third	B=	12.300 m		6.150
	B/6 =	2.050 m		
	e =	0.323 m	B/6>e OK	
	(B/6-e	1.727)		
Sliding	FS=	1.138	OK	
Bearing strength	U1=	4.736 t/m2	OK	
	U2=	6.504 t/m2	OK	

Stability Analysis

Deli River Weir (Immediate Plan)
Case 5 , Nomal Condition (WL. EL. 25.0m, k=0.000)

Basic Condition				Stability Condition		
Water Level	Upstream	EL.	24.700 m	U.W.. of Concrete		2.350 t/m3
	Downstream	EL.	24.200 m	Coefficient of Earth quake		0.000
EL. of Crest		EL.	31.000 m	U.W. of Sediment		1.800
EL. of Foundation		EL.	21.700 m	Coefficient of Sediment P.		0.500
Crest Width		B1	3.000 m	Foundation	C	0.000
Slope	Downstream	1:	1.000		tan ϕ	0.838
U.P. Slope	Upstream	1:	0.000		Bearing strength	21.867 t/m2
Orifice	Height	Ho	2.000 m	Required Factor of Safety		1.500
	Width	Wo	6.000 m	Uplift C.	L1=apron+base	39.800 m
	Length	Lo	8.300 m		L2=base	12.300
	Orifice Center	EL.	25.700 m		μ	1.00
Block Width		Wb	17.500 m		Up1	100%
Weir Height		Hw	9.300 m		Up2	26%
EL. of Sedimentation			27.900			

Load			Vertical	Horizontal	Arm	Moment
Self Weight	Center	W1	1147.388		1.500	1,721.081
	Downstream	W2	1,778.451		6.100	10,848.549
	Upstream	W3	0.000		0.000	0.000
	Orifice	Wo	-234.060		4.150	-971.349
Hydraulic Pressure	Horizontal US	P1		0.000	4.650	0.000
	Horizontal US	P2		78.750	1.000	78.750
	Horizontal DS	P5		-54.688	0.833	-45.573
	Orifice	Po		0.000	4.000	0.000
Water Weight	Crest	P3	0.000		1.000	0.000
	Down Stream	P4	54.688		11.467	627.083
	Upstream	P6	0.000		0.000	0.000
Seismic Force	Center	Ke1		0.000	4.650	0.000
	Downstream	Ke2		0.000	3.100	0.000
	Upstream	Ke3		0.000	3.100	0.000
	Orifice	Keo		0.000	4.000	0.000
Sediment Pressure		Ps		196.765	2.067	406.647
Uplift		U1	-53.813		4.100	-220.631
		U2	-566.397		6.150	-3,483.341
Total			2,126.256	220.827		8,961.217

Stability Calculation

4.215

Middle third	B=	12.300 m		6.150
	B/6 =	2.050 m		
	e =	-1.935 m	B/6>e OK	
	(B/6-e	0.115)		
Sliding	FS=	8.074	OK	
Bearing strength	U1=	19.204 t/m2	OK	
	U2=	0.552 t/m2	OK	

Stability Analysis

Deli River Weir (Immediate Plan)
Case 1 , Nomal Condition (WL. EL. 31.0 m, k=0.000)

Basic Condition				Stability Condition		
Water Level	Upstream	EL.	31.500 m	U.W.. of Concrete		2.350 t/m3
	Downstream	EL.	28.360 m	Coefficient of Earth quake		0.000
EL. of Crest		EL.	31.500 m	U.W. of Sediment		1.800
EL. of Foundation		EL.	21.700 m	Coefficient of Sediment P.		0.500
Crest Width		B1	2.500 m	Foundation	C	0.000
Slope	Downstream	1:	1.000		tan ϕ	0.838
U.P. Slope	Upstream	1:	0.000		Bearing strength	21.867 t/m2
Orifice	Height	Ho	2.000 m	Required Factor of Safety		1.500
	Width	Wo	6.000 m	Uplift C.	L1=apron+base	39.800 m
	Length	Lo	8.300 m		L2=base	12.300
	Orifice Center	EL.	25.700 m		μ	1.00
Block Width		Wb	17.500 m		Up1	100%
Weir Height		Hw	9.800 m		Up2	26%
EL. of Sedimentation			28.233			

Load			Vertical	Horizontal	Arm	Moment
Self Weight	Center	W1	1007.563		1.250	1,259.453
	Downstream	W2	1,974.823		5.767	11,388.143
	Upstream	W3	0.000		0.000	0.000
	Orifice	Wo	-234.060		4.150	-971.349
Hydraulic Pressure	Horizontal US1	P1		0.000	4.900	0.000
	Horizontal US2	P2		840.350	3.267	2,745.143
	Horizontal DS	P5		-388.112	2.220	-861.608
	Orifice	Po		-69.600	4.000	-278.400
Water Weight	Crest	P3	0.000		0.833	0.000
	Down Stream	P4	388.112		10.080	3,912.164
	Upstream	P6	0.000		0.000	0.000
Seismic Force	Center	Ke1		0.000	4.900	0.000
	Downstream	Ke2		0.000	3.267	0.000
	Upstream	Ke3		0.000	3.267	0.000
	Orifice	Keo		0.000	4.000	0.000
Sediment Pressure		Ps		97.107	2.178	211.478
Uplift		U1	-337.943		4.100	-1,385.564
		U2	-1,611.112		6.150	-9,908.340
Total			1,187.382	479.746		6,111.120

Stability Calculation

Middle third	**B=**	**12.300 m**	
	B/6 =	**2.050 m**	
	e =	**-1.003 m**	**B/6>e OK**
	(B/6-e	**1.047)**	
Sliding	**FS=**	**2.075**	**OK**
Bearing strength	**U1=**	**8.216 t/m2**	**OK**
	U2=	**2.817 t/m2**	**OK**

Stability Analysis

Deli River Weir (Immediate Plan)
Case 2 , Earthquake Condition (WL. EL. 26.5m, k= 0.183)

Basic Condition				Stability Condition		
Water Level	Upstream	EL.	26.500 m	U.W.. of Concrete		2.350 t/m3
	Downstream	EL.	26.000 m	Coefficient of Earth quake		0.183
EL. of Crest		EL.	31.500 m	U.W. of Sediment		1.800
EL. of Foundation		EL.	21.700 m	Coefficient of Sediment P.		0.500
Crest Width		B1	2.500 m	Foundation	C	0.000
Slope	Downstream	1:	1.000		tan ϕ	0.838
U.P. Slope	Upstream	1:	0.000		Bearing strength	41.350 t/m2
Orifice	Height	Ho	2.000 m	Required Factor of Safety		1.300
	Width	Wo	6.000 m	Uplift C.	L1=apron+base	39.800 m
	Length	Lo	8.300 m		L2=base	12.300
	Orifice Center	EL.	25.700 m		μ	1.00
Block Width		Wb	17.500 m		Up1	100%
Weir Height		Hw	9.800 m		Up2	26%
EL. of Sedimentation			28.233			

Load			Vertical	Horizontal	Arm	Moment
Self Weight	Center	W1	1007.563		1.250	1,259.453
	Downstream	W2	1,974.823		5.767	11,388.143
	Upstream	W3	0.000		0.000	0.000
	Orifice	Wo	-234.060		4.150	-971.349
Hydraulic Pressure	Horizontal US1	P1		0.000	4.900	0.000
	Horizontal US2	P2		201.600	1.600	322.560
	Horizontal DS	P5		-161.788	1.433	-231.895
	Orifice	Po		-9.720	4.000	-38.880
Water Weight	Crest	P3	0.000		0.833	0.000
	Down Stream	P4	161.788		10.867	1,758.091
	Upstream	P6	0.000		0.000	0.000
Seismic Force	Center	Ke1		184.384	4.900	903.481
	Downstream	Ke2		361.393	3.267	1,180.549
	Upstream	Ke3		0.000	3.267	0.000
	Orifice	Keo		-42.833	4.000	-171.332
Sediment Pressure		Ps		97.107	2.178	211.478
Uplift		U1	-53.813		4.100	-220.631
		U2	-953.847		6.150	-5,866.158
Total			1,902.453	630.143		9,523.509

Stability Calculation

Middle third	**B=**	12.300 m	
	B/6 =	**2.050 m**	
	e =	**-1.144 m**	**B/6>e OK**
	(B/6-e	**0.906)**	
Sliding	**FS=**	**2.531**	**OK**
Bearing strength	**U1=**	**13.771 t/m2**	**OK**
	U2=	**3.906 t/m2**	**OK**

Stability Analysis

Deli River Weir (Immediate Plan)
Case 3 , Flood Condition (WL. EL. 34.0m k=0.000)

Basic Condition				Stability Condition		
Water Level	Upstream	EL.	34.000 m	U.W.. of Concrete		2.350 t/m3
	Downstream	EL.	30.600 m	Coefficient of Earth quake		0.000
EL. of Crest		EL.	31.500 m	U.W. of Sediment		0.000
EL. of Foundation		EL.	21.700 m	Coefficient of Sediment P.		0.500
Crest Width		B1	2.500 m	Foundation	C	0.000
Slope	Downstream	1:	1.000		tan ϕ	0.838
U.P. Slope	Upstream	1:	0.000		Bearing strength	21.867 t/m2
Orifice	Height	Ho	2.000 m	Required Factor of Safety		1.300
	Width	Wo	6.000 m	Uplift C.	L1=apron+base	39.800 m
	Length	Lo	8.300 m		L2=base	12.300
	Orifice Center	EL.	25.700 m		μ	1.00
Block Width		Wb	17.500 m		Up1	100%
Weir Height		Hw	9.800 m		Up2	26%
EL. of Sedimentation			28.233			

Load			Vertical	Horizontal	Arm	Moment
Self Weight	Center	W1	1007.563		1.250	1,259.453
	Downstream	W2	1,974.823		5.767	11,388.143
	Upstream	W3	0.000		0.000	0.000
	Orifice	Wo	-234.060		4.150	-971.349
Hydraulic Pressure	Horizontal US1	P1		428.750	4.900	2,100.875
	Horizontal US2	P2		840.350	3.267	2,745.143
	Horizontal DS	P5		-693.088	2.967	-2,056.160
	Orifice	Po		-99.600	4.000	-398.400
Water Weight	Crest	P3	54.688		0.833	45.573
	Down Stream	P4	693.088		9.333	6,468.817
	Upstream	P6	0.000		0.000	0.000
Seismic Force	Center	Ke1		0.000	4.900	0.000
	Downstream	Ke2		0.000	3.267	0.000
	Upstream	Ke3		0.000	3.267	0.000
	Orifice	Keo		0.000	4.000	0.000
Sediment Pressure		Ps		-121.384	2.178	-264.347
Uplift		U1	-365.925		4.100	-1,500.293
		U2	-2,107.974		6.150	-12,964.037
Total			1,022.201	355.029		5,853.419

Stability Calculation

Middle third	**B=**	**12.300 m**	
	B/6 =	**2.050 m**	
	e =	**-0.424 m**	**B/6>e OK**
	(B/6-e	**1.626)**	
Sliding	**FS=**	**2.414**	**OK**
Bearing strength	**U1=**	**5.730 t/m2**	**OK**
	U2=	**3.767 t/m2**	**OK**

Stability Analysis

Deli River Weir (Immediate Plan)
Case 4 , Critical Condition (WL. EL. 31.5m k=0.183)

Basic Condition				Stability Condition		
Water Level	Upstream	EL.	31.000 m	U.W.. of Concrete		2.350 t/m3
	Downstream	EL.	28.360 m	Coefficient of Earth quake		0.183
EL. of Crest		EL.	31.500 m	U.W. of Sediment		1.800
EL. of Foundation		EL.	21.700 m	Coefficient of Sediment P.		0.500
Crest Width		B1	2.500 m	Foundation	C	0.000
Slope	Downstream	1:	1.000		tan ϕ	0.838
U.P. Slope	Upstream	1:	0.000		Bearing strength	41.350 t/m2
Orifice	Height	Ho	2.000 m	Required Factor of Safety		1.100
	Width	Wo	6.000 m	Uplift C.	L1=apron+base	39.800 m
	Length	Lo	8.300 m		L2=base	12.300
	Orifice Center	EL.	25.700 m		μ	1.00
Block Width		Wb	17.500 m		Up1	100%
Weir Height		Hw	9.800 m		Up2	26%
EL. of Sedimentation			28.233			

Load			Vertical	Horizontal	Arm	Moment
Self Weight	Center	W1	1007.563		1.250	1,259.453
	Downstream	W2	1,974.823		5.767	11,388.143
	Upstream	W3	0.000		0.000	0.000
	Orifice	Wo	-234.060		4.150	-971.349
Hydraulic Pressure	Horizontal US1	P1		0.000	4.900	0.000
	Horizontal US2	P2		756.788	3.100	2,346.041
	Horizontal DS	P5		-388.112	2.220	-861.608
	Orifice	Po		-63.600	4.000	-254.400
Water Weight	Crest	P3	0.000		0.833	0.000
	Down Stream	P4	388.112		10.080	3,912.164
	Upstream	P6	0.000		0.000	0.000
Seismic Force	Center	Ke1		184.384	4.900	903.481
	Downstream	Ke2		361.393	3.267	1,180.549
	Upstream	Ke3		0.000	3.267	0.000
	Orifice	Keo		-42.833	4.000	-171.332
Sediment Pressure		Ps		97.107	2.178	211.478
Uplift		U1	-284.130		4.100	-1,164.933
		U2	-1,582.840		6.150	-9,734.468
Total			1,269.466	905.127		8,043.220

Stability Calculation

Middle third	**B=**	**12.300 m**	
	B/6 =	**2.050 m**	
	e =	**0.186 m**	**B/6>e OK**
	(B/6-e	1.864)	
Sliding	**FS=**	**1.18**	**OK**
Bearing strength	**U1=**	**5.363 t/m2**	**OK**
	U2=	**6.432 t/m2**	**OK**

Stability Analysis

Deli River Weir (Immediate Plan)
Case 5 , Nomal Condition (WL. EL. 25.0m, k=0.000)

Basic Condition				Stability Condition		
Water Level	Upstream	EL.	24.700 m	U.W.. of Concrete		2.350 t/m3
	Downstream	EL.	24.200 m	Coefficient of Earth quake		0.000
EL. of Crest		EL.	31.500 m	U.W. of Sediment		1.800
EL. of Foundation		EL.	21.700 m	Coefficient of Sediment P.		0.500
Crest Width		B1	2.500 m	Foundation	C	0.000
Slope	Downstream	1:	1.000		tan ϕ	0.838
U.P. Slope	Upstream	1:	0.000		Bearing strength	21.867 t/m2
Orifice	Height	Ho	2.000 m	Required Factor of Safety		1.500
	Width	Wo	6.000 m	Uplift C.	L1=apron+base	39.800 m
	Length	Lo	8.300 m		L2=base	12.300
	Orifice Center	EL.	25.700 m		μ	1.00
Block Width		Wb	17.500 m		Up1	100%
Weir Height		Hw	9.800 m		Up2	26%
EL. of Sedimentation			28.233			

Load			Vertical	Horizontal	Arm	Moment
Self Weight	Center	W1	1007.563		1.250	1,259.453
	Downstream	W2	1,974.823		5.767	11,388.143
	Upstream	W3	0.000		0.000	0.000
	Orifice	Wo	-234.060		4.150	-971.349
Hydraulic Pressure	Horizontal US1	P1		0.000	4.900	0.000
	Horizontal US2	P2		78.750	1.000	78.750
	Horizontal DS	P5		-54.688	0.833	-45.573
	Orifice	Po		0.000	4.000	0.000
Water Weight	Crest	P3	0.000		0.833	0.000
	Down Stream	P4	54.688		11.467	627.083
	Upstream	P6	0.000		0.000	0.000
Seismic Force	Center	Ke1		0.000	4.900	0.000
	Downstream	Ke2		0.000	3.267	0.000
	Upstream	Ke3		0.000	3.267	0.000
	Orifice	Keo		0.000	4.000	0.000
Sediment Pressure		Ps		218.491	2.178	475.825
Uplift		U1	-53.813		4.100	-220.631
		U2	-566.397		6.150	-3,483.341
Total			2,182.803	242.554		9,108.361

Stability Calculation

Middle third	**B=**	**12.300 m**	
	B/6 =	**2.050 m**	
	e =	**-1.977 m**	**B/6>e OK**
	(B/6-e	**0.073)**	
Sliding	**FS=**	**7.546**	**OK**
Bearing strength	**U1=**	**19.922 t/m2**	**OK**
	U2=	**0.360 t/m2**	**OK**

Stability Analysis

Floodway Weir (Immediate Plan)
Case 1 , Nomal Condition (WL. EL. 31.0 m, k=0.000)

Basic Condition				Stability Condition		
Water Level	Upstream	EL.	31.000 m	U.W.. of Concrete		2.350 t/m3
	Downstream	EL.	26.500 m	Coefficient of Earth quake		0.000
EL. of Crest		EL.	32.500 m	U.W. of Sediment		1.800
EL. of Foundation		EL.	25.000 m	Coefficient of Sediment P.		0.500
Crest Width		B1	2.000 m	Foundation	C	0.000
Slope	Downstream	1:	1.000		tan ϕ	0.838
U.P. Slope	Upstream	1:	0.000		Bearing strength	21.867 t/m2
Orifice	Height	Ho	0.000 m	Required Factor of Safety		1.500
	Width	Wo	0.000 m	Uplift C.	L1=apron+base	28.000 m
	Length	Lo	0.000 m		L2=base	9.500
	Orifice Center	EL.	0.000 m		μ	1.00
Block Width		Wb	1.000 m		Up1	100%
Weir Height		Hw	7.500 m		Up2	29%
EL. of Sedimentation			28.000			

Load			Vertical	Horizontal	Arm	Moment
Self Weight	Center	W1	35.250		1.000	35.250
	Downstream	W2	66.094		4.500	297.422
	Upstream	W3	0.000		0.000	0.000
	Orifice	Wo	0.000		0.000	0.000
Hydraulic Pressure	Horizontal US	P1		0.000	3.750	0.000
	Horizontal US	P2		18.000	2.000	36.000
	Horizontal DS	P5		-1.125	0.500	-0.563
	Orifice	Po		0.000	-25.000	0.000
Water Weight	Crest	P3	0.000		0.667	0.000
	Down Stream	P4	1.125		9.000	10.125
	Upstream	P6	0.000		0.000	0.000
Seismic Force	Center	Ke1		0.000	3.750	0.000
	Downstream	Ke2		0.000	2.500	0.000
	Upstream	Ke3		0.000	2.500	0.000
	Orifice	Keo		0.000	-25.000	0.000
Sediment Pressure		Ps		1.800	1.000	1.800
Uplift		U1	-21.375		3.167	-67.688
		U2	-26.579		4.750	-126.249
Total			54.515	18.675		186.098

Stability Calculation

3.414

Middle third	**B=**	**9.500 m**		4.750
	B/6 =	**1.583 m**		
	e =	**-1.336 m**	**B/6>e OK**	
	(B/6-e	**0.247)**		
Sliding	**FS=**	**2.448**	**OK**	
Bearing strength	**U1=**	**10.582 t/m2**	**OK**	
	U2=	**0.895 t/m2**	**OK**	

Stability Analysis

Floodway Weir (Immediate Plan)
Case 2 , Earthquake Condition (WL. EL. 31.0m, k= 0.183)

Basic Condition				Stability Condition		
Water Level	Upstream	EL.	26.500 m	U.W.. of Concrete		2.350 t/m3
	Downstream	EL.	26.500 m	Coefficient of Earth quake		0.183
EL. of Crest		EL.	32.500 m	U.W. of Sediment		1.800
EL. of Foundation		EL.	25.000 m	Coefficient of Sediment P.		0.500
Crest Width		B1	2.000 m	Foundation	C	0.000
Slope	Downstream	1:	1.000		tan ϕ	0.838
U.P. Slope	Upstream	1:	0.000		Bearing strength	41.350 t/m2
Orifice	Height	Ho	0.000 m	Required Factor of Safety		1.300
	Width	Wo	0.000 m	Uplift C.	L1=apron+base	28.000 m
	Length	Lo	0.000 m		L2=base	9.500
	Orifice Center	EL.	0.000 m		μ	1.00
Block Width		Wb	1.000 m		Up1	100%
Weir Height		Hw	7.500 m		Up2	29%
EL. of Sedimentation			28.000			

Load			Vertical	Horizontal	Arm	Moment
Self Weight	Center	W1	35.250		1.000	35.250
	Downstream	W2	66.094		4.500	297.422
	Upstream	W3	0.000		0.000	0.000
	Orifice	Wo	0.000		0.000	0.000
Hydraulic Pressure	Horizontal US	P1		0.000	3.750	0.000
	Horizontal US	P2		1.125	0.500	0.563
	Horizontal DS	P5		-1.125	0.500	-0.563
	Orifice	Po		0.000	-25.000	0.000
Water Weight	Crest	P3	0.000		0.667	0.000
	Down Stream	P4	1.125		9.000	10.125
	Upstream	P6	0.000		0.000	0.000
Seismic Force	Center	Ke1		6.451	3.750	24.190
	Downstream	Ke2		12.095	2.500	30.238
	Upstream	Ke3		0.000	2.500	0.000
	Orifice	Keo		0.000	-25.000	0.000
Sediment Pressure		Ps		1.800	1.000	1.800
Uplift		U1	0.000		3.167	0.000
		U2	-14.250		4.750	-67.688
Total			88.219	20.346		331.338

Stability Calculation

3.756

Middle third **B=** 9.500 m 4.750
B/6 = **1.583 m**
e = -0.994 m **B/6>e OK**
(B/6-e 0.589)

Sliding **FS=** 3.636 **OK**

Bearing strength **U1=** 15.117 t/m2 **OK**
U2= 3.456 t/m2 **OK**

Stability Analysis

Floodway Weir (Immediate Plan)
Case 3 , Flood Condition (WL. EL. 34.0m k=0.000)

Basic Condition				Stability Condition		
Water Level	Upstream	EL.	34.000 m	U.W.. of Concrete		2.350 t/m3
	Downstream	EL.	31.430 m	Coefficient of Earth quake		0.000
EL. of Crest		EL.	32.500 m	U.W. of Sediment		0.000
EL. of Foundation		EL.	25.000 m	Coefficient of Sediment P.		0.500
Crest Width		B1	2.000 m	Foundation	C	0.000
Slope	Downstream	1:	1.000		tan ϕ	0.838
U.P. Slope	Upstream	1:	0.000		Bearing strength	21.867 t/m2
Orifice	Height	Ho	0.000 m	Required Factor of Safety		1.300
	Width	Wo	0.000 m	Uplift C.	L1=apron+base	28.000 m
	Length	Lo	0.000 m		L2=base	9.500
	Orifice Center	EL.	0.000 m		μ	0.72
Block Width		Wb	1.000 m		Up1	100%
Weir Height		Hw	7.500 m		Up2	29%
EL. of Sedimentation			28.000			

Load			Vertical	Horizontal	Arm	Moment
Self Weight	Center	W1	35.250		1.000	35.250
	Downstream	W2	66.094		4.500	297.422
	Upstream	W3	0.000		0.000	0.000
	Orifice	Wo	0.000		0.000	0.000
Hydraulic Pressure	Horizontal US	P1		11.250	3.750	42.188
	Horizontal US	P2		28.125	2.500	70.313
	Horizontal DS	P5		-20.672	2.143	-44.308
	Orifice	Po		0.000	-25.000	0.000
Water Weight	Crest	P3	1.500		0.667	1.000
	Down Stream	P4	20.672		7.357	152.080
	Upstream	P6	0.000		0.000	0.000
Seismic Force	Center	Ke1		0.000	3.750	0.000
	Downstream	Ke2		0.000	2.500	0.000
	Upstream	Ke3		0.000	2.500	0.000
	Orifice	Keo		0.000	-25.000	0.000
Sediment Pressure		Ps		-2.250	1.000	-2.250
Uplift		U1	-8.809		3.167	-27.895
		U2	-49.160		4.750	-233.510
Total			65.547	16.453		290.289

Stability Calculation 4.429

Middle third	B=	9.500 m		4.750
	B/6 =	1.583 m		
	e =	-0.321 m	B/6>e OK	
	(B/6-e	1.262)		
Sliding	FS=	3.341	OK	
Bearing strength	U1=	8.300 t/m2	OK	
	U2=	5.500 t/m2	OK	

Stability Analysis

Floodway Weir (Immediate Plan)
Case 4 , Critical Condition (WL. EL. 31.0m k=0.183)

Basic Condition				Stability Condition		
Water Level	Upstream	EL.	31.000 m	U.W.. of Concrete		2.350 t/m3
	Downstream	EL.	26.500 m	Coefficient of Earth quake		0.183
EL. of Crest		EL.	32.500 m	U.W. of Sediment		1.800
EL. of Foundation		EL.	25.000 m	Coefficient of Sediment P.		0.500
Crest Width		B1	2.000 m	Foundation	C	0.000
Slope	Downstream	1:	1.000		tan ϕ	0.838
U.P. Slope	Upstream	1:	0.000		Bearing strength	41.350 t/m2
Orifice	Height	Ho	0.000 m	Required Factor of Safety		1.100
	Width	Wo	0.000 m	Uplift C.	L1=apron+base	28.000 m
	Length	Lo	0.000 m		L2=base	9.500
	Orifice Center	EL.	0.000 m		μ	1.00
Block Width		Wb	1.000 m		Up1	100%
Weir Height		Hw	7.500 m		Up2	29%
EL. of Sedimentation			28.000			

Load			Vertical	Horizontal	Arm	Moment
Self Weight	Center	W1	35.250		1.000	35.250
	Downstream	W2	66.094		4.500	297.422
	Upstream	W3	0.000		0.000	0.000
	Orifice	Wo	0.000		0.000	0.000
Hydraulic Pressure	Horizontal US	P1		0.000	3.750	0.000
	Horizontal US	P2		18.000	2.000	36.000
	Horizontal DS	P5		-1.125	0.500	-0.563
	Orifice	Po		0.000	-25.000	0.000
Water Weight	Crest	P3	0.000		0.667	0.000
	Down Stream	P4	1.125		9.000	10.125
	Upstream	P6	0.000		0.000	0.000
Seismic Force	Center	Ke1		6.451	3.750	24.190
	Downstream	Ke2		12.095	2.500	30.238
	Upstream	Ke3		0.000	2.500	0.000
	Orifice	Keo		0.000	-25.000	0.000
Sediment Pressure		Ps		1.800	1.000	1.800
Uplift		U1	-21.375		3.167	-67.688
		U2	-26.579		4.750	-126.249
Total			54.515	37.221		240.526

Stability Calculation

4.412

Middle third	**B=**	**9.500 m**		**4.750**
	B/6 =	**1.583 m**		
	e =	**-0.338 m**	**B/6>e OK**	
	(B/6-e	**1.245)**		
Sliding	**FS=**	**1.228**	**OK**	
Bearing strength	**U1=**	**6.963 t/m2**	**OK**	
	U2=	**4.514 t/m2**	**OK**	

Stability Analysis

Floodway Weir (Immediate Plan)
Case 5 , Nomal Condition (WL. EL. 26.5m, k=0.000)

Basic Condition				Stability Condition		
Water Level	Upstream	EL.	25.000 m	U.W.. of Concrete		2.350 t/m3
	Downstream	EL.	25.000 m	Coefficient of Earth quake		0.000
EL. of Crest		EL.	32.500 m	U.W. of Sediment		1.800
EL. of Foundation		EL.	25.000 m	Coefficient of Sediment P.		0.500
Crest Width		B1	2.000 m	Foundation	C	0.000
Slope	Downstream	1:	1.000		tan ϕ	0.838
U.P. Slope	Upstream	1:	0.000		Bearing strength	21.867 t/m2
Orifice	Height	Ho	0.000 m	Required Factor of Safety		1.500
	Width	Wo	0.000 m	Uplift C.	L1=apron+base	28.000 m
	Length	Lo	0.000 m		L2=base	9.500
	Orifice Center	EL.	0.000 m		μ	1.00
Block Width		Wb	1.000 m		Up1	100%
Weir Height		Hw	7.500 m		Up2	29%
EL. of Sedimentation			28.000			

Load			Vertical	Horizontal	Arm	Moment
Self Weight	Center	W1	35.250		1.000	35.250
	Downstream	W2	66.094		4.500	297.422
	Upstream	W3	0.000		0.000	0.000
	Orifice	Wo	0.000		0.000	0.000
Hydraulic Pressure	Horizontal US	P1		0.000	3.750	0.000
	Horizontal US	P2		0.000	0.000	0.000
	Horizontal DS	P5		0.000	0.000	0.000
	Orifice	Po		0.000	-25.000	0.000
Water Weight	Crest	P3	0.000		0.667	0.000
	Down Stream	P4	0.000		9.500	0.000
	Upstream	P6	0.000		0.000	0.000
Seismic Force	Center	Ke1		0.000	3.750	0.000
	Downstream	Ke2		0.000	2.500	0.000
	Upstream	Ke3		0.000	2.500	0.000
	Orifice	Keo		0.000	-25.000	0.000
Sediment Pressure		Ps		4.050	1.000	4.050
Uplift		U1	0.000		3.167	0.000
		U2	0.000		4.750	0.000
Total			101.344	4.050		336.722

Stability Calculation

3.323

Middle third	**B=**	**9.500 m**		**4.750**
	B/6 =	**1.583 m**		
	e =	**-1.427 m**	**B/6>e OK**	
	(B/6-e	**0.156)**		
Sliding	**FS=**	**20.982**	**OK**	
Bearing strength	**U1=**	**20.285 t/m2**	**OK**	
	U2=	**1.050 t/m2**	**OK**	

Stability Analysis

Floodway Weir (Immediate Plan)
Case 1 , Nomal Condition (WL. EL. 31.0 m, k=0.000)

Basic Condition				Stability Condition		
Water Level	Upstream	EL.	31.000 m	U.W.. of Concrete		2.350 t/m3
	Downstream	EL.	26.500 m	Coefficient of Earth quake		0.000
EL. of Crest		EL.	32.500 m	U.W. of Sediment		1.800
EL. of Foundation		EL.	25.000 m	Coefficient of Sediment P.		0.500
Crest Width		B1	2.500 m	Foundation	C	0.000
Slope	Downstream	1:	1.000		tan ϕ	0.838
U.P. Slope	Upstream	1:	0.000		Bearing strength	21.867 t/m2
Orifice	Height	Ho	0.000 m	Required Factor of Safety		1.500
	Width	Wo	0.000 m	Uplift C.	L1=apron+base	28.000 m
	Length	Lo	0.000 m		L2=base	10.000
	Orifice Center	EL.	0.000 m		μ	1.00
Block Width		Wb	1.000 m		Up1	100%
Weir Height		Hw	7.500 m		Up2	36%
EL. of Sedimentation			28.000			

Load			Vertical	Horizontal	Arm	Moment
Self Weight	Center	W1	44.063		1.250	55.078
	Downstream	W2	66.094		5.000	330.469
	Upstream	W3	0.000		0.000	0.000
	Orifice	Wo	0.000		0.000	0.000
Hydraulic Pressure	Hrizontal US1	P1		0.000	3.750	0.000
	Hrizontal US2	P2		18.000	2.000	36.000
	Hrizontal DS	P5		-1.125	0.500	-0.563
	Orifice	Po		0.000	-25.000	0.000
Water Weight	Crest	P3	0.000		0.833	0.000
	Down Stream	P4	1.125		9.500	10.688
	Upstream	P6	0.000		0.000	0.000
Seismic Force	Center	Ke1		0.000	3.750	0.000
	Downstream	Ke2		0.000	2.500	0.000
	Upstream	Ke3		0.000	2.500	0.000
	Orifice	Keo		0.000	-25.000	0.000
Sediment Pressure		Ps		1.800	1.000	1.800
Uplift		U1	-22.500		3.333	-75.000
		U2	-31.071		5.000	-155.357
Total			57.710	18.675		203.115

Stability Calculation

Middle third	**B=**	10.000 m	
	B/6 =	**1.667 m**	
	e =	**-1.480 m**	**B/6>e OK**
	(B/6-e	0.186)	
Sliding	**FS=**	**2.591**	**OK**
Bearing strength	**U1=**	**10.897 t/m2**	**OK**
	U2=	**0.645 t/m2**	**OK**

Stability Analysis

Floodway Weir (Immediate Plan)
Case 2 , Earthquake Condition (WL. EL. 31.0m, k= 0.183)

Basic Condition				Stability Condition		
Water Level	Upstream	EL.	26.500 m	U.W.. of Concrete		2.350 t/m3
	Downstream	EL.	26.500 m	Coefficient of Earth quake		0.183
EL. of Crest		EL.	32.500 m	U.W. of Sediment		1.800
EL. of Foundation		EL.	25.000 m	Coefficient of Sediment P.		0.500
Crest Width		B1	2.500 m	Foundation	C	0.000
Slope	Downstream	1:	1.000		tan ϕ	0.838
U.P. Slope	Upstream	1:	0.000		Bearing strength	41.350 t/m2
Orifice	Height	Ho	0.000 m	Required Factor of Safety		1.300
	Width	Wo	0.000 m	Uplift C.	L1=apron+base	28.000 m
	Length	Lo	0.000 m		L2=base	10.000
	Orifice Center	EL.	0.000 m		μ	1.00
Block Width		Wb	1.000 m		Up1	100%
Weir Height		Hw	7.500 m		Up2	36%
EL. of Sedimentation			28.000			

Load			Vertical	Horizontal	Arm	Moment
Self Weight	Center	W1	44.063		1.250	55.078
	Downstream	W2	66.094		5.000	330.469
	Upstream	W3	0.000		0.000	0.000
	Orifice	Wo	0.000		0.000	0.000
Hydraulic Pressure	Hrizontal US1	P1		0.000	3.750	0.000
	Hrizontal US2	P2		1.125	0.500	0.563
	Hrizontal DS	P5		-1.125	0.500	-0.563
	Orifice	Po		0.000	-25.000	0.000
Water Weight	Crest	P3	0.000		0.833	0.000
	Down Stream	P4	1.125		9.500	10.688
	Upstream	P6	0.000		0.000	0.000
Seismic Force	Center	Ke1		8.063	3.750	30.238
	Downstream	Ke2		12.095	2.500	30.238
	Upstream	Ke3		0.000	2.500	0.000
	Orifice	Keo		0.000	-25.000	0.000
Sediment Pressure		Ps		1.800	1.000	1.800
Uplift		U1	0.000		3.333	0.000
		U2	-15.000		5.000	-75.000
Total			96.281	21.959		383.510

Stability Calculation

Middle third	**B=**	**10.000 m**	
	B/6 =	**1.667 m**	
	e =	**-1.017 m**	**B/6>e OK**
	(B/6-e	**0.650)**	
Sliding	**FS=**	**3.677**	**OK**
Bearing strength	**U1=**	**15.502 t/m2**	**OK**
	U2=	**3.754 t/m2**	**OK**

Stability Analysis

Floodway Weir (Immediate Plan)
Case 3 , Flood Condition (WL. EL. 34.0m k=0.000)

Basic Condition				Stability Condition		
Water Level	Upstream	EL.	34.000 m	U.W.. of Concrete		2.350 t/m3
	Downstream	EL.	32.300 m	Coefficient of Earth quake		0.000
EL. of Crest		EL.	32.500 m	U.W. of Sediment		0.000
EL. of Foundation		EL.	25.000 m	Coefficient of Sediment P.		0.500
Crest Width		B1	2.500 m	Foundation	C	0.000
Slope	Downstream	1:	1.000		tan ϕ	0.838
U.P. Slope	Upstream	1:	0.000		Bearing strength	21.867 t/m2
Orifice	Height	Ho	0.000 m	Required Factor of Safety		1.300
	Width	Wo	0.000 m	Uplift C.	L1=apron+base	28.000 m
	Length	Lo	0.000 m		L2=base	10.000
	Orifice Center	EL.	0.000 m		μ	0.72
Block Width		Wb	1.000 m		Up1	100%
Weir Height		Hw	7.500 m		Up2	36%
EL. of Sedimentation			28.000			

Load			Vertical	Horizontal	Arm	Moment
Self Weight	Center	W1	44.063		1.250	55.078
	Downstream	W2	66.094		5.000	330.469
	Upstream	W3	0.000		0.000	0.000
	Orifice	Wo	0.000		0.000	0.000
Hydraulic Pressure	Hrizontal US1	P1		11.250	3.750	42.188
	Hrizontal US2	P2		28.125	2.500	70.313
	Hrizontal DS	P5		-26.645	2.433	-64.836
	Orifice	Po		0.000	-25.000	0.000
Water Weight	Crest	P3	1.875		0.833	1.563
	Down Stream	P4	26.645		7.567	201.614
	Upstream	P6	0.000		0.000	0.000
Seismic Force	Center	Ke1		0.000	3.750	0.000
	Downstream	Ke2		0.000	2.500	0.000
	Upstream	Ke3		0.000	2.500	0.000
	Orifice	Keo		0.000	-25.000	0.000
Sediment Pressure		Ps		-2.250	1.000	-2.250
Uplift		U1	-6.134		3.333	-20.445
		U2	-57.058		5.000	-285.291
Total			75.484	10.480		328.401

Stability Calculation

Middle third	**B=**	**10.000 m**	
	B/6 =	**1.667 m**	
	e =	**-0.649 m**	**B/6>e OK**
	(B/6-e	**1.017)**	
Sliding	**FS=**	**6.039**	**OK**
Bearing strength	**U1=**	**10.490 t/m2**	**OK**
	U2=	**4.607 t/m2**	**OK**

Stability Analysis

Floodway Weir (Immediate Plan)
Case 4 , Critical Condition (WL. EL. 31.0m k=0.183)

Basic Condition			
Water Level	Upstream	EL.	31.000 m
	Downstream	EL.	26.500 m
EL. of Crest		EL.	32.500 m
EL. of Foundation		EL.	25.000 m
Crest Width		B1	2.500 m
Slope	Downstream	1:	1.000
U.P. Slope	Upstream	1:	0.000
Orifice	Height	Ho	0.000 m
	Width	Wo	0.000 m
	Length	Lo	0.000 m
	Orifice Center	EL.	0.000 m
Block Width		Wb	1.000 m
Weir Height		Hw	7.500 m
EL. of Sedimentation			28.000

Stability Condition		
U.W.. of Concrete		2.350 t/m3
Coefficient of Earth quake		0.183
U.W. of Sediment		1.800
Coefficient of Sediment P.		0.500
Foundation	C	0.000
	tan φ	0.838
	Bearing strength	41.350 t/m2
Required Factor of Safety		1.100
Uplift C.	L1=apron+base	28.000 m
	L2=base	10.000
	μ	1.00
	Up1	100%
	Up2	36%

Load			Vertical	Horizontal	Arm	Moment
Self Weight	Center	W1	44.063		1.250	55.078
	Downstream	W2	66.094		5.000	330.469
	Upstream	W3	0.000		0.000	0.000
	Orifice	Wo	0.000		0.000	0.000
Hydraulic Pressure	Hrizontal US1	P1		0.000	3.750	0.000
	Hrizontal US2	P2		18.000	2.000	36.000
	Hrizontal DS	P5		-1.125	0.500	-0.563
	Orifice	Po		0.000	-25.000	0.000
Water Weight	Crest	P3	0.000		0.833	0.000
	Down Stream	P4	1.125		9.500	10.688
	Upstream	P6	0.000		0.000	0.000
Seismic Force	Center	Ke1		8.063	3.750	30.238
	Downstream	Ke2		12.095	2.500	30.238
	Upstream	Ke3		0.000	2.500	0.000
	Orifice	Keo		0.000	-25.000	0.000
Sediment Pressure		Ps		1.800	1.000	1.800
Uplift		U1	-22.500		3.333	-75.000
		U2	-31.071		5.000	-155.357
Total			57.710	38.834		263.591

Stability Calculation

Middle third	**B=**	**10.000 m**	
	B/6 =	**1.667 m**	
	e =	**-0.432 m**	**B/6>e OK**
	(B/6-e	**1.234)**	
Sliding	**FS=**	**1.246**	**OK**
Bearing strength	**U1=**	**7.268 t/m2**	**OK**
	U2=	**4.273 t/m2**	**OK**

Stability Analysis

Floodway Weir (Immediate Plan)
Case 5 , Nomal Condition (WL. EL. 26.5m, k=0.000)

Basic Condition				Stability Condition		
Water Level	Upstream	EL.	25.000 m	U.W.. of Concrete		2.350 t/m3
	Downstream	EL.	25.000 m	Coefficient of Earth quake		0.000
EL. of Crest		EL.	32.500 m	U.W. of Sediment		1.800
EL. of Foundation		EL.	25.000 m	Coefficient of Sediment P.		0.500
Crest Width		B1	2.500 m	Foundation	C	0.000
Slope	Downstream	1:	1.000		tan ϕ	0.838
U.P. Slope	Upstream	1:	0.000		Bearing strength	21.867 t/m2
Orifice	Height	Ho	0.000 m	Required Factor of Safety		1.500
	Width	Wo	0.000 m	Uplift C.	L1=apron+base	28.000 m
	Length	Lo	0.000 m		L2=base	10.000
	Orifice Center	EL.	0.000 m		μ	1.00
Block Width		Wb	1.000 m		Up1	100%
Weir Height		Hw	7.500 m		Up2	36%
EL. of Sedimentation			28.000			

Load			Vertical	Horizontal	Arm	Moment
Self Weight	Center	W1	44.063		1.250	55.078
	Downstream	W2	66.094		5.000	330.469
	Upstream	W3	0.000		0.000	0.000
	Orifice	Wo	0.000		0.000	0.000
Hydraulic Pressure	Hrizontal US1	P1		0.000	3.750	0.000
	Hrizontal US2	P2		0.000	0.000	0.000
	Hrizontal DS	P5		0.000	0.000	0.000
	Orifice	Po		0.000	-25.000	0.000
Water Weight	Crest	P3	0.000		0.833	0.000
	Down Stream	P4	0.000		10.000	0.000
	Upstream	P6	0.000		0.000	0.000
Seismic Force	Center	Ke1		0.000	3.750	0.000
	Downstream	Ke2		0.000	2.500	0.000
	Upstream	Ke3		0.000	2.500	0.000
	Orifice	Keo		0.000	-25.000	0.000
Sediment Pressure		Ps		4.050	1.000	4.050
Uplift		U1	0.000		3.333	0.000
		U2	0.000		5.000	0.000
Total			110.156	4.050		389.597

Stability Calculation

Middle third	**B=**	10.000 m	
	B/6 =	**1.667 m**	
	e =	**-1.463 m**	**B/6>e OK**
	(B/6-e	0.203)	
Sliding	**FS=**	**22.806**	**OK**
Bearing strength	**U1=**	**20.687 t/m2**	**OK**
	U2=	**1.345 t/m2**	**OK**

Structural Details

(1) Deli River Weir

The Deli River Weir is composed of the following major structural components such as main weir body, apron, breast wall, wing wall, riverbed protection and revetments. The basic features of the structural components are presented in Fig. 2.4.2, and the design concepts are described below.

(a) Direction

The weir axis is placed at right angle with the flow direction of the existing channel.

(b) Elevation of Apron Bed

The elevation of apron bed is set at EL. 24.2 m which conform to the existing riverbed elevation at Section UD. 12.00.

(c) Length and Thickness of Apron

The apron with 30 m long is proposed by the hydraulic model test to ensure the dissipation of flow energy. With this length, the required creep length against piping by Lane's method can be obtained.

The required creep length against piping can be obtained by Lane's method.

$C \leq (Lh/3 + \Sigma Lv)/\Delta H$

where, C : Lane's creep ratio, C = 6 for sand (D50 = 0.2 to 0.4)
Lh : creep length of horizontal direction,
(Lh = 39.8 +5 m)
Lv : creep length of vertical direction (ΣLv = 11 m)
ΔH : max. difference between water heads (m),
design high water level
ΔH= 3.4 m for urgent plan
ΔH= 2.8 m for immediate plan
water level at crest
ΔH= 2.5 m for urgent plan
ΔH= 1.7 m for immediate plan

$C = (44.8/3 \times 11.0)/3.4 = 7.6 > 6$ ok

Therefore the cut off wall such as sheet piles is not provided.

The apron thickness is determined to withstand the scouring force and uplift pressure. To withstand the scouring force and uplift pressure, the thickness of spillway bed and apron is set at 2.0 m as bellow.

$t = 4/3\ (\Delta h - \Delta u)/(Wc-1)$

where, Δh : maximum difference between water heads (m)
Δu : uplift at the toe of main body (t/m^2)
Wc : unit weight of concrete ($Wc = 2.35\ t/m^3$)

		Δh (EL. m)	Δu (EL. m)	Thickness (m)
design high water level	immediate plan		10.69	
	urgent plan		8.12	
water level at crest	immediate plan			
	urgent plan			

(d) Breast Wall

A gravity type of breast wall is provided at the both sides of main weir body to contact the weir body and impermeable foundation to prevent horizontal seepage.

(e) Wing Wall

To minimize the excavation volume and its construction cost and to ensure the connection between wall and foundation, a semi-gravity type of wing wall and a gravity type wall are applied to the upstream and downstream wing wall respectively.

(f) Protection Works

Concrete blocks as a flexible riverbed protection are provided for the upstream and downstream riverbeds of weir. For the upstream protection, the crib-type concrete blocks (1.5 m x 1.5 m, 0.5 ton/each) with boulder filling are provided and for the downstream, square type concrete block (1 m x 1 m, 1.0 ton/each) corresponding to the water velocity. The length of downstream protection works is calculated by the Bligh's formula. It is 45 m from the end of apron. The length of upstream protection is set at about half of the downstream length. Slope protections are also provided to prevent side slope faces from erosion caused by overflow discharge.

(g) Low Water Passage (Orifice)

To flow down the low water discharge of 10.6 m^3/s with an open channel flow, two orifices with 2.0 m high and 3 m wide are is provided at the center of the weir. Bed elevation of the orifice is raised by 50 cm from the bed elevation of the apron. At the outlet of the orifice, two column type of baffles are provided to dissipate the energy of flow from orifices.

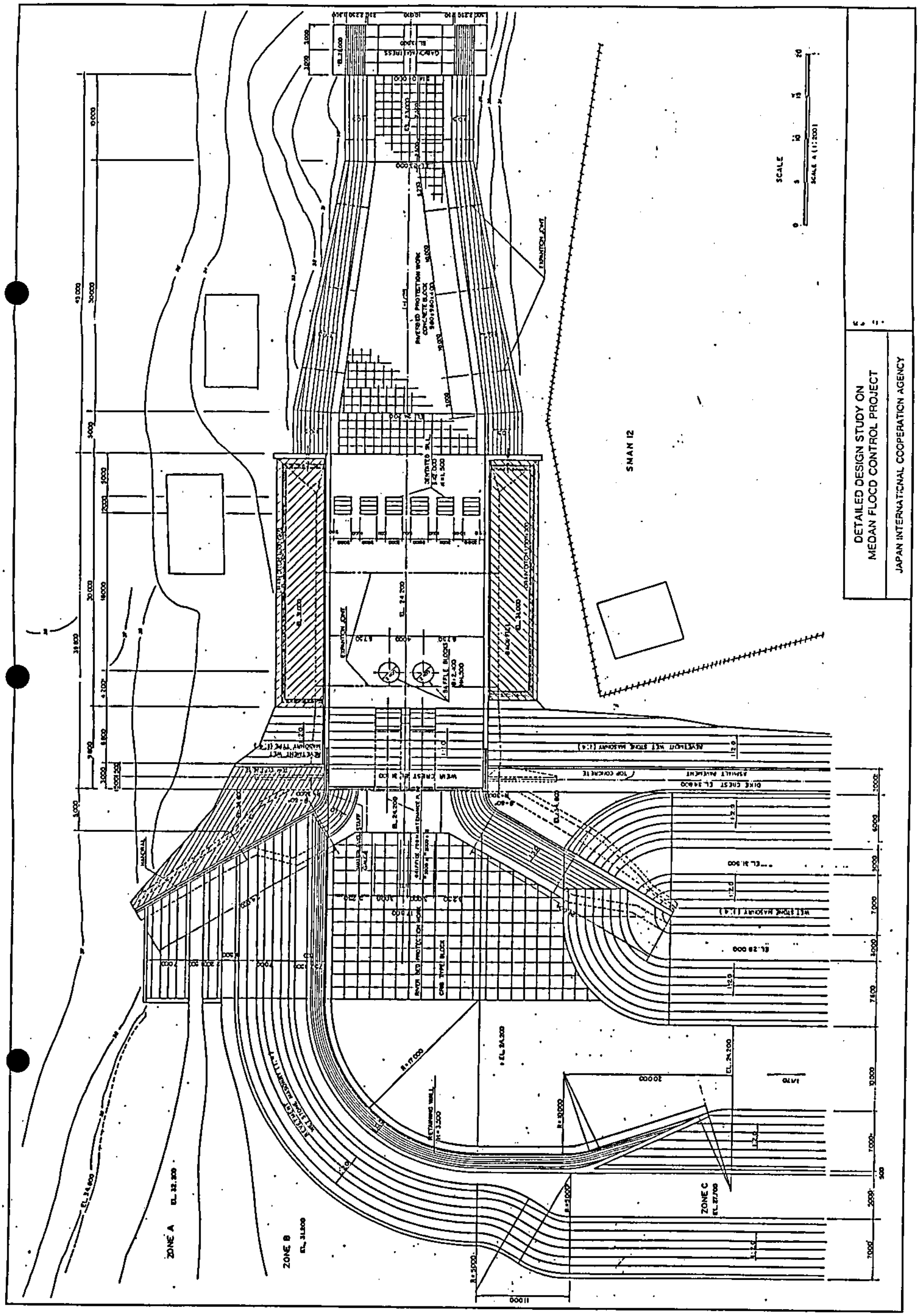


Fig. 2.4.2 (1/2) DELI RIVER WEIR, PLAN

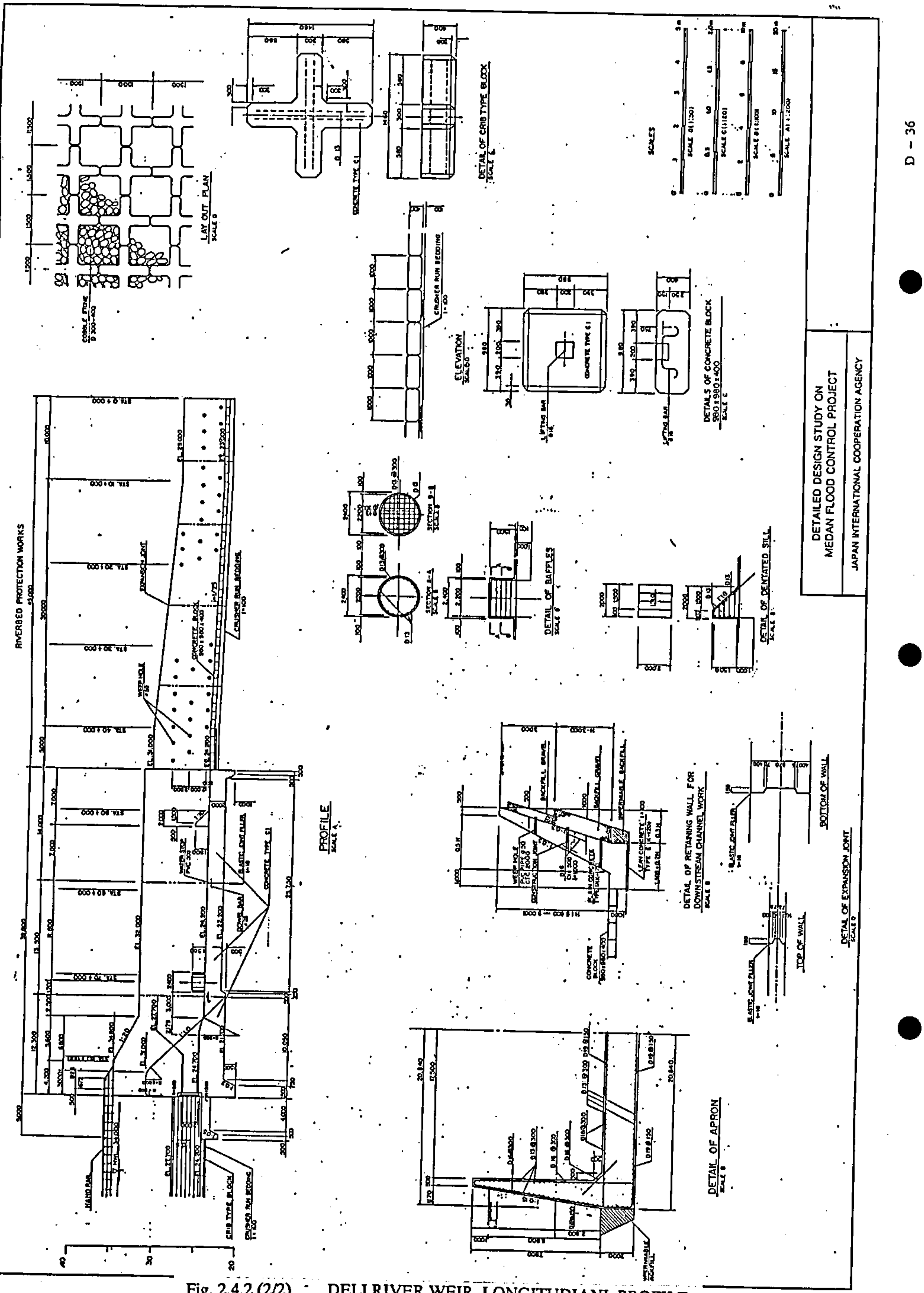


Fig. 2.4.2 (2/2) DELI RIVER WEIR, LONGITUDIANL PROFILE

(2) Floodway Weir

The structural components are almost the same as the Deli River Weir. The basic design results are shown in fIG. 2.4.3, and the design concepts are as follows:

(a) Direction

The weir axis is placed at right angle with the Floodway channel centerline which is nearly parallel to the flow direction of the Deli River channel.

(b) Height of Apron Bed

The height of weir apron bed is set at the elevation of 26.50 m which is about 1.0 m higher than the nearby riverbed elevation.

(c) Length and Thickness of Apron

The same concepts as those of the Deli River Weir are applied.

The apron with 30 m long is proposed by the hydraulic model test to ensure the dissipation of flow energy. With this length, the required creep length against piping by Lane's method can be obtained as bellow.

The required creep length against piping can be obtained by Lane's method.

$C \leq (Lh/3 + \Sigma Lv)/\Delta H$

where, C : Lane's creep ratio, C = 6 for sand (D50 = 0.2 to 0.4)
Lh : creep length of horizontal direction,
(Lh = 34 m)
Lv : creep length of vertical direction (ΣLv = 12.5 m)
ΔH : max. difference between water heads (m),
Design High water level
ΔH= 1.7 m for urgent plan
ΔH= 3.0 m for immediate plan
water level at crest
ΔH= 3.3 m for urgent plan
ΔH= 3.9 m for immediate plan

$C = (34.0/3 + 12.5)/3.9 = 6.1 > 6$ ok

Therefore the cut off wall such as sheet piles is not provided.

(d) Breast Wall

A gravity type of breast wall is provided at the both sides of main weir body to contact the weir body and impermeable foundation to prevent horizontal seepage.

(e) Wing Wall

A reinforced concrete wall (inverse T type wall) is provided along the approach channel of the Floodway Weir to retaining the both bank embankment.

To ensure the connection between wall and foundation, a gravity type wall are applied to the downstream wing wall.

(f) Protection Works

The same concepts as those of the Deli River Weir are applied.

As a results of study, a 40 m long downstream protection works is provided. For the approach channel bed, crib type concrete blocks with boulder filling is applied.

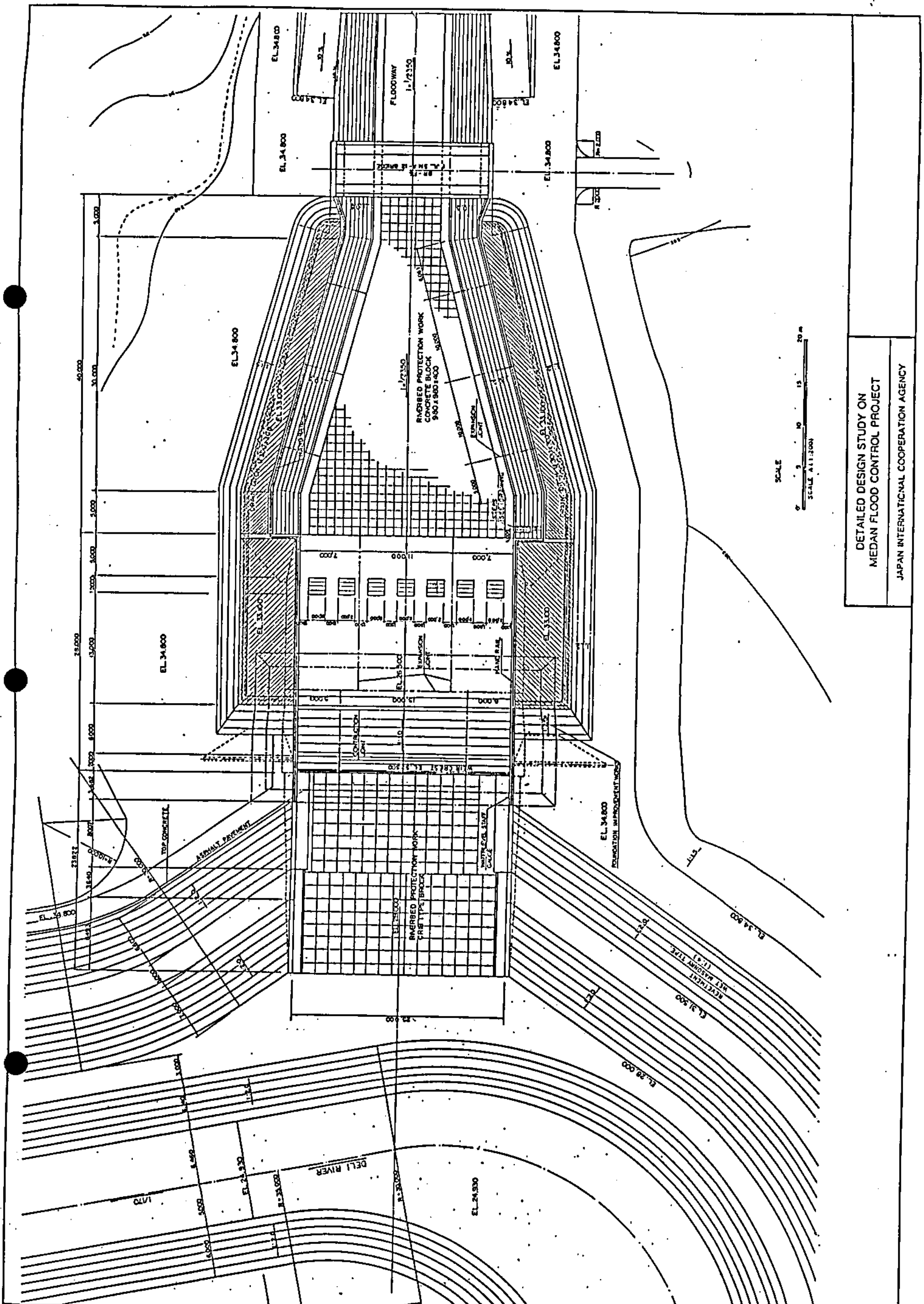


Fig. 2.4.3 (1/2) FLOODWAY WEIR, PLAN

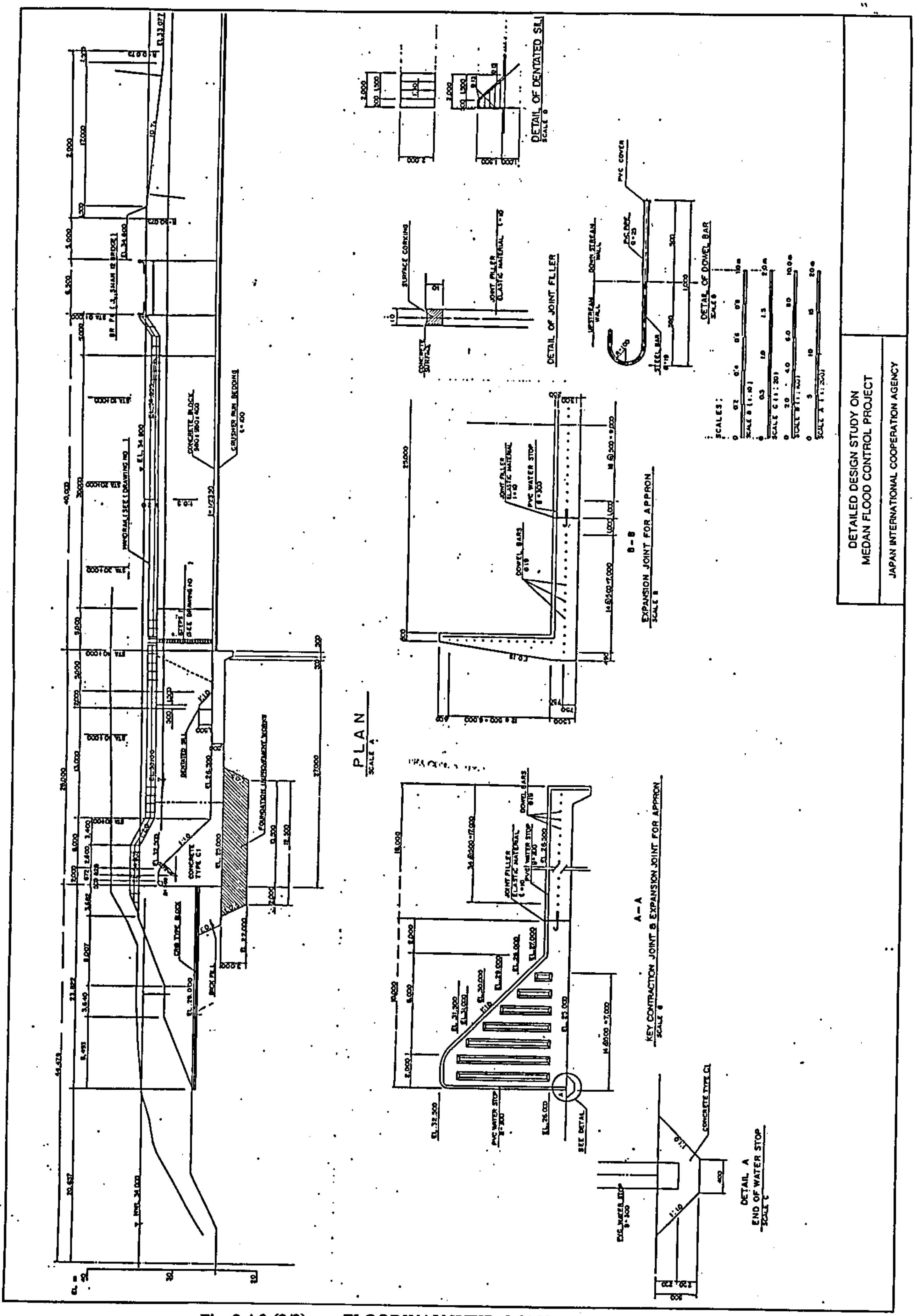


Fig. 2.4.3 (2/2) FLOODWAY WEIR, LONGITUDIANL PROFILE

2.4.4 Design Calculation

The stability analysis and stress calculation for the apron wall and upstream and downstream wing wall wrer conducted in this section.

(1) Stability Analysis

(a) Safety of Wall

The walls were designed to meet the following requirements of safty.

Sliding

The factor of safety against sliding is determined using the following formula:

$$SF = Rc / \Sigma H$$

where, H : total horizontal forces
Rc : resisting capacity
SF : safety factor given in the following
$SF \geq 1.5$ (under normal condition)
$SF \geq 1.2$ (under floods and earthquake condition)

Overturning

All forces acting on part of the structure above any horizontal plane should fall within the middle third of the structure base.

For this purpose, the following condition should be satisfied :

$$e = \frac{b}{2} - \frac{M}{N} < \frac{b}{6} \quad \text{(under the normal condition)}$$

$$e = \frac{b}{2} - \frac{M}{N} < \frac{b}{3} \quad \text{(under the earthquake condition)}$$

where,
b : width of base (m)
M : total moment about point A (tf . m)
N : total vertical forces (tf)
e : eccentricity (m)

Bearing Capacity of Pile Foundation

The maximum principal stress in the spread foundation must be kept within allowable soil bearing capacity. The allowable soil bearing capacity at the weir site (Qa) is calculated as follows:

$Qa_1 = 65.6\ tf/m^2 / 3 = 21.8\ tf/m^2$ for nomal condition,
$Qa_2 = 82.7\ tf/m^2 / 2 = 41.3\ tf/m^2$ for earthquake condition.

(b) Loading Combination

The loading combination for the stability analysis is:

Condition	Loading Combination	Increase in Allowable Stress	Safety Factor	
			Sliding	Bearing Capacity
Normal	$M+T+Th_n+U$	0 %	1.5	3
Flood	$M+T+Th_b+U$	0 %	1.2	2
Earthquake	$M+T+Th_n+G+U$	20 %	1.2	2

where

M : dead load ($\gamma c = 2.5\ t/m^3$)
T : soil pressure($\gamma c = 1.9\ t/m^3$)
Thn : normal water pressure
Thb : water pressure during flood (hw = H/3, H = wall height)
G : earthquake load (k = 0.183)
U : uplift

(2) Stress Calculation

For the wall and footing slab of walls to arrange the reinforcement bars

(a) Loading Condition

The load to be considered for the structural design of box culverts are:

a. Earth pressure : $\gamma t = 1.9$ t/m3
b. Live load : Pvl = 1 t/m^2
c. Dead load : reinforced concrete $\gamma c = 2.5\ t/m^3$

(b) Allowable Stress

a. Reinforcement Bar
- Allowable tensile stress (σsa) : 1,800 kg/cm^2
- Allowable compressive stress : 1,800 kg/cm^2

b. Concrete (Type C_1 & Type C_2)
- Allowable compressive stress : 225/3 = 75 kg/cm^2
- Allowable shear stress : 4.25 kg/cm^2

(3) Results of Calculation

The dimension, Stablity Analysis, Load Calclation, Stress Calculation and Reinforcement bar arrangement of the apron wall and upstream and downstream wing walls are as shown in following pages.

(a) Apron Wall of Deli River Weir

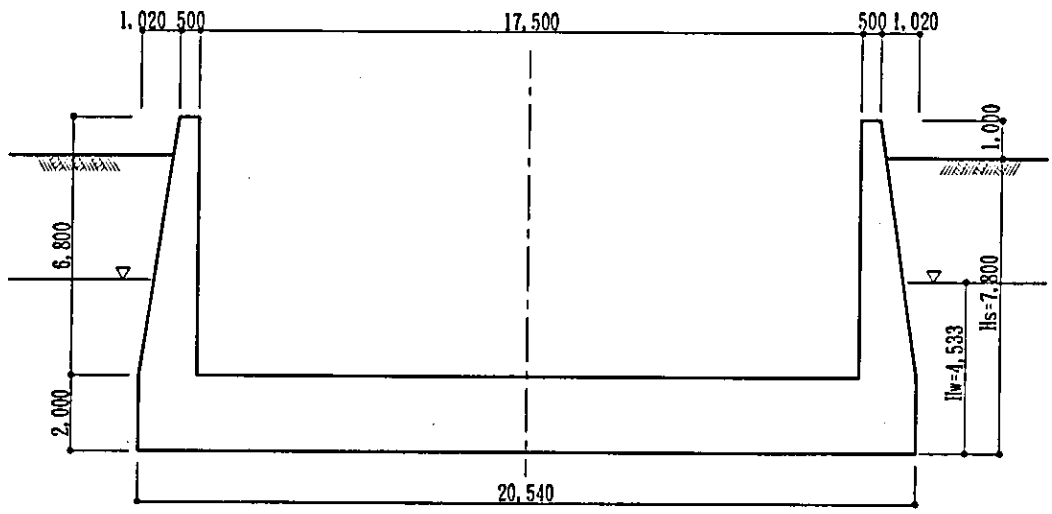


Fig. 2.4.4 BASIC DIMENSIONS
DELI RIVER WEIR - APRON WALL

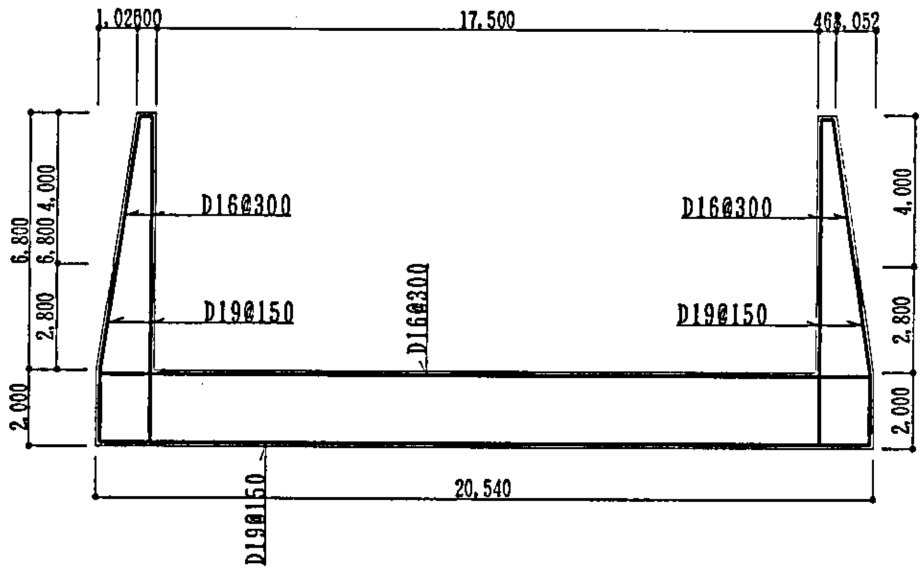


Fig. 2.4.5 REINFORCEMENT BAR ARRANGEMENT
DELI RIVER WEIR - APRON WALL

Table 4.2. Load Calculation (Dell River Weir, Apron Wall)

			Apron Wall		Apron Wall		Apron Footing		Apron Footing	
			Normal H=6.8m	Earthquake H=6.8m	Normal H=4.0m	Earthquake H=4.0m	Normal Corner	Earthquake Corner	Normal Center	Earthquake Center
Wall Height	H	m	6.800	6.800	4.000	4.000	6.800	6.800		
Top Width	B1	m	0.500	0.500	0.500	0.500	0.500	0.500		
Back-side Slope	n	m	0.150	0.150	0.150	0.150	0.150	0.150		
C. of EQ	k		0.000	0.183	0.000	0.183	0.000	0.183		
C. of E.P	kh		0.364	0.364	0.364	0.364	0.364	0.364		
U.W. of Conc.	rc	t/m^3	2.500	2.500	2.500	2.500	2.500	2.500		
U.W. of Soil	rs	t/m^4	1.900	1.900	1.900	1.900	1.900	1.900		
Water Depth	hw	m	4.533	0.000	2.667	0.000	4.533	0.000		
Self Weight	W1	t	8.500	8.500	5.000	5.000	8.500	8.500		
	W2	t	8.670	8.670	3.000	3.000	8.670	8.670	1.805	1.805
Earthquake Load	Ke1	t	0.000	1.556	0.000	0.915	0.000	1.556		
	Ke2	t	0.000	1.587	0.000	0.549	0.000	1.587		
Water Pressure	Wp	t	10.276	0.000	3.556	0.000	10.276	0.000		
Earth Pressure	We	t	15.991	15.991	5.533	5.533	15.991	15.991		
Arm Length	Lke1	m	3.400	3.400	2.000	2.000	3.400	3.400		
	Lke2	m	2.267	2.267	1.333	1.333	2.267	2.267		
	Lwp	m	1.511	0.000	0.889	0.000	1.511	0.000		
	Lwe	m	2.267	2.267	1.333	1.333	2.267	2.267		
Moment	Mke1	t·m	0.000	5.289	0.000	1.830	0.000	5.289		
	Mke2	t·m	0.000	3.596	0.000	0.732	0.000	3.596		
	Mwp	t·m	15.528	0.000	3.160	0.000	15.528	0.000		
	Mwe	t·m	36.247	36.247	7.378	7.378	36.247	36.247		
Axis Force	N	t	17.170	17.170	8.000	8.000	0.000	0.000	0.000	0.000
Moment	M	t·m	51.775	45.132	10.538	9.940	51.775	45.132	-17.341	-23.983
Shear Force	S	t	26.267	19.134	9.089	6.997	26.267	19.134	0.000	0.000
Effective Thikness	d	cm	142.000	142.000	100.000	100.000	190.000	190.000	190.000	190.000
	d'	cm	10.000	10.000	10.000	10.000	10.000	10.000	10.000	10.000
	u	cm	54.792	54.792	41.875	41.875	100.000	100.000	0.000	0.000
	b	cm	100.000	100.000	100.000	100.000	100.000	100.000	100.000	100.000

Table 4.2. STRESS CALCULATION (DELI RIVER WEIR, APRON WALL)

Type		Wing Wall (Upstream)				Wing Wall (Upstream)				Wing Wall Footing (Upstream)				Wing Wall Footing (Upstream)			
		Normal		Earthquake		Normal		Earthquake		Normal		Earthquake		Normal		Earthquake	
N	t	17.17		17.17		8.00		8.00		0.00		0.00		0.00		0.00	
M	tm	51.77		45.13		10.51		9.91		51.77		45.13		17.31		23.98	
Q	t	26.27		19.13		9.00		7.00		26.27		19.13		0.00		0.00	
d	cm	112.00		112.00		100.00		100.00		100.00		100.00		100.00		100.00	
d'	cm	10.00		10.00		10.00		10.00		10.00		10.00		10.00		10.00	
u	cm	51.79		51.79		41.88		41.88		100.00		100.00		0.00		0.00	
b	cm	100.00		100.00		100.00		100.00		100.00		100.00		100.00		100.00	
r	A's/As	1.00		1.00		1.00		1.00		1.00		1.00		1.00		1.00	
n		15.00		15.00		15.00		15.00		15.00		15.00		15.00		15.00	
Sca	kg/cm2	70.00		105.00		70.00		105.00		70.00		105.00		70.00		105.00	
Ssa	kg/cm2	1,600.00		2,400.00		1,600.00		2,400.00		1,600.00		2,400.00		1,600.00		2,400.00	
Tca	kg/cm2	4.25		6.38		4.25		6.38		4.25		6.38		4.25		6.38	
M'		61.18		54.51		13.89		13.29		51.77		45.13		17.31		23.98	
C		23.07	23.07	38.82	38.82	50.10	50.10	79.01	79.01	18.81	63.07	83.99	78.91	115.72	63.07	158.05	78.91
k (C)		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0.033	[illegible]	[illegible]	[illegible]	0.033	[illegible]	[illegible]	[illegible]
np (C)		0.0000	0.0000	0.0008	0.0008	0.0000	0.0000	0.0000	0.0000	0.0006	0.0005	0.0003	0.0003	0.0006	0.0005	0.0001	0.0003
S		35.15	35.15	59.15	59.15	76.80	76.80	120.39	120.39	71.37	71.37	127.98	127.98	222.06	222.06	210.83	210.83
k (S)		0.208	[illegible]	0.17	[illegible]	0.151	[illegible]	[illegible]	[illegible]	0.147	[illegible]	0.12	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
np (S)		0.0192	0.0192	0.0101	0.0101	0.0062	0.0062	0.0029	0.0029	0.0139	0.0139	0.0080	0.0080	0.0046	0.0046	0.0012	0.0012
d'/d		0.07		0.07		0.10		0.10		0.05		0.05		0.05		0.05	
f	cm	356.33		317.65		173.60		166.12		9,999.99		9,999.99		9,999.99		9,999.99	
f/d		2.51		2.23		1.74		1.66		52.63		52.63		52.63		52.63	
np		0.0192		0.0101		0.0062		0.0029		0.0139		0.0080		0.0046		0.0012	
r-As		18.21		9.81		4.15		1.90		17.59		10.12		5.77		5.32	
r-As'		18.21		9.81		4.15		1.90		17.59		10.12		5.77		5.32	
Re-Bar Arrangement																	
As1	Diameter	D19		D19		D16		D16		D19		D19		D16		D16	
	Area (cm2)	2.87		2.87		1.99		1.99		2.87		2.87		1.99		1.99	
	Pitch (cm)	30.00		30.00		30.00		30.00		30.00		30.00		30.00		30.00	
	As1	9.55		9.55		6.62		6.62		9.55		9.55		6.62		6.62	
As2	Diameter	D19		D19		0.00		0.00		D19		D19		0.00		0.00	
	Area (cm2)	2.87		2.87		0.00		0.00		2.87		2.87		0.00		0.00	
	Pitch (cm)	30.00		30.00		30.00		30.00		30.00		30.00		30.00		30.00	
	As2 (cm2)	9.55		9.55		0.00		0.00		9.55		9.55		0.00		0.00	
Total of As		19.10	(OK)	19.10	(OK)	6.62	(OK)	6.62	(OK)	19.10	(OK)	19.10	(OK)	6.62	(OK)	6.62	(OK)
		-18.21		-9.81		-4.15		-1.90		-17.59		-10.12		-5.77		-5.32	
As'1	Diameter	D19		D19		D16		D16		D16		D16		D16		D16	
	Area (cm2)	2.87		2.87		1.99		1.99		1.99		1.99		1.99		1.99	
	Pitch (cm)	30.00		30.00		30.00		30.00		30.00		30.00		30.00		30.00	
	As'1 (cm2)	9.55		9.55		6.62		6.62		6.62		6.62		6.62		6.62	
As'2	Diameter	D19		D19		0.00		0.00		0.00		0.00		0.00		0.00	
	Area (cm2)	2.87		2.87		0.00		0.00		0.00		0.00		0.00		0.00	
	Pitch (cm)	25.00		25.00		25.00		25.00		25.00		25.00		25.00		25.00	
	As'2 (cm2)	11.46		11.46		0.00		0.00		0.00		0.00		0.00		0.00	
Total of As'		21.01	(OK)	21.01	(OK)	6.62	(OK)	6.62	(OK)	6.62	(NG)	6.62	(NG)	6.62	(OK)	6.62	(OK)
		-18.21		-9.81		-4.15		-1.90		-17.59		-10.12		-5.77		-5.32	
Check	np	0.02018		0.02018		0.00993		0.00993		0.01508		0.01508		0.00523		0.00523	
	k	0.21	0.000	0.22	0.000	0.18	0.000	0.19	0.000	0.16	0.000	0.16	0.000	0.10	0.000	0.10	0.000
	A's/As	1.10		1.10		1.00		1.00		0.35		0.35		1.00		1.00	
	C	8.96		8.70		11.10		10.86		12.84		12.84		20.58		20.58	
	S	33.56		31.27		49.46		47.00		68.73		68.73		194.21		194.21	
	Z	1.06	0.17	1.05	0.17	1.03	0.14	1.03	0.14	1.05	0.16	1.05	0.16	1.03	0.09	1.03	0.09
	Sc	27.17	OK	23.54	OK	15.41	OK	14.43	OK	18.42	OK	16.05	OK	9.88	OK	13.67	OK
	Ss	1,527.28	OK	1,268.69	OK	1,030.34	OK	937.03	OK	1,478.61	OK	1,288.91	OK	1,399.38	OK	1,935.41	OK
	Tc	1.85	OK	1.35	OK	0.91	OK	0.70	OK	1.38	OK	1.01	OK	0.00	OK	0.00	OK
	x	29.91		30.91		18.33		18.76		29.91		29.91		18.20		18.20	

(b) Upstream Wing Wall of Deli River Weir

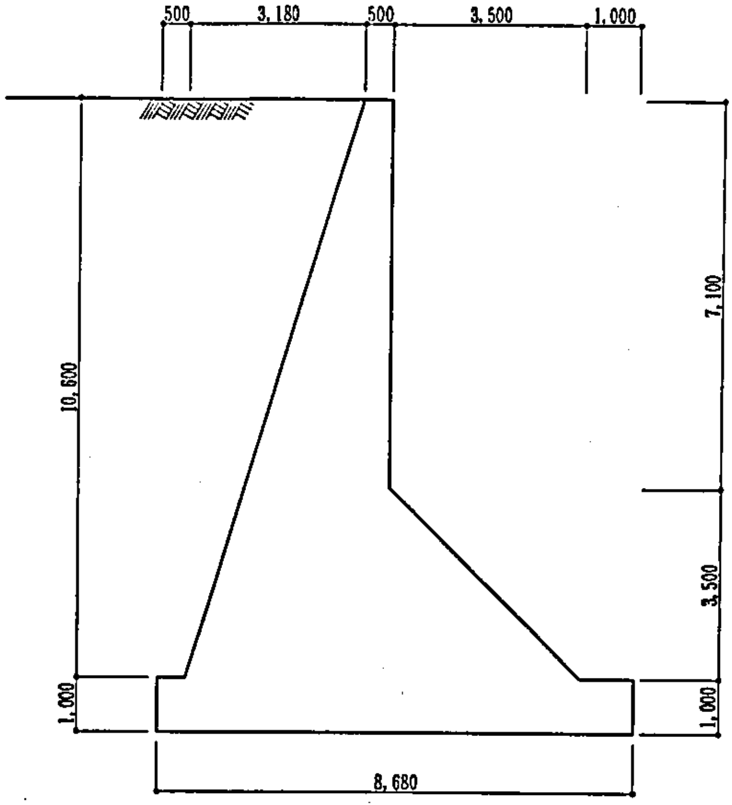


Fig. 2.4.6 BASIC DIMENSIONS
DELI RIVER WEIR - WING WALL

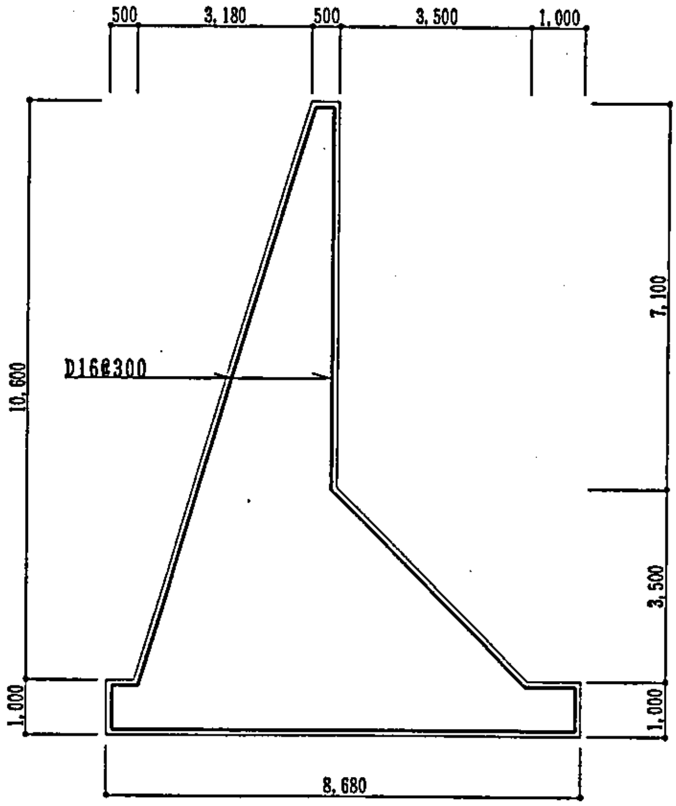


Fig. 2.4.7 BASIC DIMENSIONS
DELI RIVER WEIR - WING WALL

TABLE 4.2. STABILITY CALCULATION OF WING WALL
DELI RIVER WEIR. UPSTREAM WING WALL

Design Condition	Flood Condition				
Backfill Material					
Height		hb	0.000		
Slope		n 1:	#########		
Unit Weight		Gs	1.900 t m3		
Internal Friction Angle		fs	30.000		
Water Height	Back Side	Hwb	3.533 m		
	Front Side	Hwf	0.500		
Load on Backfill		q	1.000 t m2		
Features of Wall					
Height of Wall		H	10.600		
Slope	Front slope	nf	1.000		
	Height	hf	3.500		
	Back slope	nb	0.300		
Top Width		B	0.500 m	B 6-e	1.181
Toe Length		Lt	1.000 m	Fs	1.367
Toe Height		Ht	1.000 m	Qumax	14.100
Heel Length		La	0.500 m		4.948
Foundation					
Internal Friction Angle		ff	35.000		
Coheson		Cf	0.000 t m2		
Concrete					
Unit Weight		Gc	2.350 t m3		
Design Strength		σck	225.000 kg cm2		
Coefficient of Earthquake		k	0.000		
Factor of Safty					
Bearing Strength		qa	21.900		

Load			Vertical	Horizontal	Arm	Moment
Self Weight	Wall	W1	39.607		6.060	240.018
		W2	12.455		4.750	59.161
		W3	14.394		3.333	47.979
	Footing	W4	20.398		4.340	88.527
	Backfill	We1	0.000		5.000	0.000
		We2	0.000		6.840	0.000
		We3	14.232		6.413	91.276
		We4	20.946		7.900	165.470
		We5	1.685		7.827	13.191
		We6	1.590		8.430	13.404
	Water	Ww1	0.000		2.250	0.000
		Ww2	0.500		0.500	0.250
		Ww3	0.125		3.333	0.417
		Ww4	1.873		7.827	14.657
		Ww5	1.767		8.430	14.893
Water Pressure	Front side	Pw1		1.125	0.500	0.563
	Back side	Pw2		-10.276	1.511	-15.528
Seismic Force	Wall	Ke1		0.000	4.533	0.000
		Ke2		0.000	6.300	0.000
		Ke3		0.000	2.167	0.000
	Footing	Ke4		0.000	0.500	0.000
	Backfill	We1		0.000	11.600	0.000
		We2		0.000	11.600	0.000
		We3		0.000	9.244	0.000
		We4		0.000	8.067	0.000
		We5		0.000	3.356	0.000
		We6		0.000	2.767	0.000
Earth Pressure	Horizontal	Phd		-18.169	6.889	-125.165
(dry area)	Vertical	Pvd	0.000		8.680	0.000
Earth Pressure	Horizontal	Phw		-3.838	1.511	-5.800
(submarged 1)	Vertical	Pvw	0.000		8.680	0.000
Earth Pressure	Horizontal	Phw'		-21.800	2.267	-49.414
(submarged 2)	Vertical	Pvw'	0.000		8.680	0.000
Uplift		U1	-13.020		4.340	-56.507
		U2	-13.165		5.787	-76.180
Total			103.387	-52.958		421.212

Stability Calculation

Middle third	**B=**	**8.680 m**	
	B/6 =	**1.447 m**	
	e =	**-0.266 m**	**B/6>e OK**
	(B/6-e	**1.181)**	
Sliding	**FS=**	**1.367**	**OK**
Bearing strength	**U1=**	**14.100 t/m2**	**OK**
	U2=	**9.722 t/m2**	**OK**
Floating	**FS=**	**4.948**	

TABLE 4.2. STABILITY CALCULATION OF WING WALL
DELI RIVER WEIR, UPSTREAM WING WALL

Design Condition	Earthquake Condition				
Backfill Material					
Height		hb	0.000		
Slope		n=1:	#########		
Unit Weight		Gs	1.900 t/m3		
Internal Friction Angle		fs	30.000		
Water Height	Back Side	Hwb	0.500 m		
	Front Side	Hwf	0.500		
Load on Backfill		q	1.000 t/m2		
Features of Wall					
Height of Wall		H	10.600		
Slope	Front slope	nf	1.000		
	Height	hf	3.500		
	Back slope	nb	0.300		
Top Width		B	0.500 m	B/6-e	1.191
Toe Length		Lt	1.000 m	Fs	1.286
Toe Height		Ht	1.000 m	Qumax	18.672
Heel Length		Lt	0.500 m		11.581
Foundation					
Internal Friction Angle		ff	35.000		
Coheson		Cf	0.000 t/m2		
Concrete					
Unit Weight		Gc	2.350 t/m3		
Design Strength		σck	225.000 kg/cm2		
Coefficient of Earthquake		k	0.183		
Factor of Safty					
Bearing Strength		qa	41.350		

Load			Vertical	Horizontal	Arm	Moment
Self Weight	Wall	W1	39.607		6.060	240.018
		W2	12.455		4.750	59.161
		W3	14.394		3.333	47.979
	Footing	W4	20.398		4.340	88.527
	Backfill	We1	0.000		5.000	0.000
		We2	0.000		6.840	0.000
		We3	29.073		7.020	204.091
		We4	12.474		8.355	104.216
		We5	0.034		8.130	0.274
		We6	0.225		8.430	1.897
	Water	Ww1	0.000		2.250	0.000
		Ww2	0.500		0.500	0.250
		Ww3	0.125		3.333	0.417
		Ww4	0.038		8.130	0.305
		Ww5	0.250		8.430	2.108
Water Pressure	Front side	Pw1		1.125	0.500	0.563
	Back side	Pw2		-1.125	0.500	-0.563
Seismic Force	Wall	Ke1		-7.248	4.533	-32.858
		Ke2		-2.279	6.300	-14.359
		Ke3		-2.634	2.167	-5.707
	Footing	Ke4		-3.733	0.500	-1.866
	Backfill	We1		0.000	11.600	0.000
		We2		0.000	11.600	0.000
		We3		-5.320	8.233	-43.804
		We4		-2.283	6.550	-14.951
		We5		-0.006	1.333	-0.008
		We6		-0.041	1.250	-0.051
Earth Pressure (dry area)	Horizontal	Phd		-39.357	4.867	-191.539
	Vertical	Pvd	16.220		8.680	140.789
Earth Pressure (submarged 1)	Horizontal	Phw		-0.411	0.500	-0.206
	Vertical	Pvw	0.169		8.680	1.471
Earth Pressure (submarged 2)	Horizontal	Phw'		-11.690	0.750	-8.768
	Vertical	Pvw'	4.818		8.680	41.818
Uplift		U1	-13.020		4.340	-56.507
		U2	0.000		5.787	0.000
Total			137.758	-75.003		562.696

Stability Calculation

Middle third	B=	8.680 m	
	B/6 =	1.447 m	
	e =	-0.255 m	B/6>e OK
	(B/6-e	1.191)	
Sliding	FS=	1.286	OK
Bearing strength	U1=	18.672 t/m2	OK
	U2=	13.070 t/m2	OK
Floating	FS=	11.581	

TABLE 4.2. (1/3) STABILITY CALCULATION OF WING WALL
DELI RIVER WEIR, UPSTREAM WING WALL

Design Condition	Normal Condition		
Backfill Material			
Height		hb	0.000
Slope		n=1:	#########
Unit Weight		Gs	1.900 t/m3
Internal Friction Angle		fs	30.000
Water Height	Back Side	Hwb	0.500 m
	Front Side	Hwf	0.500
Load on Backfill		q	1.000 t/m2
Features of Wall			
Height of Wall		H	10.600
Slope	Front slope	nf	1.000
	Height	hf	3.500
	Back slope	nb	0.300
Top Width		B	0.500 m
Toe Length		Lt	1.000 m
Toe Height		Ht	1.000 m
Heel Length		Lt	0.500 m
Foundation			
Internal Friction Angle		ff	35.000
Coheson		Cf	0.000 t m2
Concrete			
Unit Weight		Gc	2.350 t m3
Design Strength		σck	225.000 kg cm2
Coefficient of Earthquake		k	0.000
Factor of Safty			
Bearing Strength		qa	21.900

B 6-e	1.400
Fs	1.761
Qumax	13.861
	9.952

Load			Vertical	Horizontal	Arm	Moment
Self Weight	Wall	W1	39.607		6.060	240.018
		W2	12.455		4.750	59.161
		W3	14.394		3.333	47.979
	Footing	W4	20.398		4.340	88.527
	Backfill	We1	0.000		5.000	0.000
		We2	0.000		6.840	0.000
		We3	29.073		7.020	204.091
		We4	12.474		8.355	104.216
		We5	0.034		8.130	0.274
		We6	0.225		8.430	1.897
	Water	Ww1	0.000		2.250	0.000
		Ww2	0.500		0.500	0.250
		Ww3	0.125		3.333	0.417
		Ww4	0.038		8.130	0.305
		Ww5	0.250		8.430	2.108
Water Pressure	Front side	Pw1		1.125	0.500	0.563
	Back side	Pw2		-1.125	0.500	-0.563
Seismic Force	Wall	Ke1		0.000	4.533	0.000
		Ke2		0.000	6.300	0.000
		Ke3		0.000	2.167	0.000
	Footing	Ke4		0.000	0.500	0.000
	Backfill	We1		0.000	11.600	0.000
		We2		0.000	11.600	0.000
		We3		0.000	8.233	0.000
		We4		0.000	6.550	0.000
		We5		0.000	1.333	0.000
		We6		0.000	1.250	0.000
Earth Pressure	Horizontal	Phd		-35.670	4.867	-173.593
(dry area)	Vertical	Pvd	0.000		8.680	0.000
Earth Pressure	Horizontal	Phw		-0.588	0.500	-0.294
(submarged 1)	Vertical	Pvw	0.000		8.680	0.000
Earth Pressure	Horizontal	Phw'		-10.095	0.750	-7.571
(submarged 2)	Vertical	Pvw'	0.000		8.680	0.000
Uplift		U1	-13.020		4.340	-56.507
		U2	0.000		5.787	0.000
Total			116.551	-46.352		511.278

Stability Calculation

Middle third	B=	8.680 m	
	B/6 =	1.447 m	
	e =	0.047 m	B/6>e OK
	(B/6-e	1.400)	
Sliding	FS=	1.761	OK
Bearing strength	U1=	12.994 t/m2	OK
	U2=	13.861 t/m2	OK
Floating	FS=	9.952	

Table 4.2. Load Calculation (Deli River Weir, Apron Wall)

			Wing Wall		Wing Wall	
			Normal	Earthquake	Normal	Earthquake
			H=7.1m	H=7.1m	H=4.0m	H=4.0m
Wall Height	H	m	7.100	7.100	4.000	4.000
Top Width	B1	m	0.500	0.500	0.500	0.500
Back-side Slope	n	m	0.300	0.300	0.300	0.300
C. of EQ	k		0.000	0.183	0.000	0.183
C. of E.P	kh		0.442	0.442	0.442	0.442
U.W. of Conc.	rc	t/m^3	2.500	2.500	2.500	2.500
U.W. of Soil	rs	t/m^4	1.900	1.900	1.900	1.900
Water Depth	hw	m	2.367	0.000	1.333	0.000
Self Weight	W1	t	8.875	8.875	5.000	5.000
	W2	t	18.904	18.904	6.000	6.000
Earthquake Load	Ke1	t	0.000	1.624	0.000	0.915
	Ke2	t	0.000	3.459	0.000	1.098
Water Pressure	Wp	t	2.801	0.000	0.889	0.000
Earth Pressure	We	t	21.180	21.180	6.722	6.722
Arm Length	Lke1	m	3.550	3.550	2.000	2.000
	Lke2	m	2.367	2.367	1.333	1.333
	Lwp	m	0.789	0.000	0.444	0.000
	Lwe	m	2.367	2.367	1.333	1.333
Moment	Mke1	t·m	0.000	5.766	0.000	1.830
	Mke2	t·m	0.000	8.187	0.000	1.464
	Mwp	t·m	2.209	0.000	0.395	0.000
	Mwe	t·m	50.126	50.126	8.963	8.963
Axis Force	N	t	27.779	27.779	11.000	11.000
Moment	M	t·m	52.335	64.078	9.358	12.257
Shear Force	S	t	23.980	26.263	7.611	8.735
Effective Thikness	d	cm	253.000	253.000	160.000	160.000
	d'	cm	10.000	10.000	10.000	10.000
	u	cm	90.329	90.329	60.455	60.455
	b	cm	100.000	100.000	100.000	100.000

Table 2.4. STRESS CALCULATION, DRLI RIVER WEIR
(UPSTREAM WING WALL)

Type		Wing Wall (Upstream, H=6.6m)			
		Normal		Earthquake	
N	t	27.78		27.78	
M	tm	52.33		64.08	
Q	t	23.98		26.26	
d	cm	253.00		253.00	
d'	cm	10.00		10.00	
u	cm	90.33		90.33	
b	cm	100.00		100.00	
r	A's/As	1.00		1.00	
n		15.00		15.00	
Sca	kg/cm2	75.00		112.50	
Ssa	kg/cm2	1,800.00		2,700.00	
Tca	kg/cm2	4.25		6.38	
M'		77.43		89.17	
C		62.00	62.00	80.76	80.76
k (C)		0.000	0.000	0.00	0.000
np (C)		0.0000	0.0000	0.0000	0.0000
S		99.20	99.20	129.21	129.21
k (S)		0.134	0.000	0.12	0.000
np (S)		0.0014	0.0014	0.0020	0.0020
d'/d		0.04		0.04	
f	cm	278.73		321.00	
f/d		1.10		1.27	
np		0.0014		0.0020	
r-As		2.36		3.30	
r-As'		2.36		3.30	
Re-Bar Arrangement					
As1	Diameter	D16		D16	
	Area (cm2)	1.99		1.99	
	Pitch (cm)	30.00		30.00	
	As1	6.62		6.62	
As2	Diameter	0.00		0.00	
	Area (cm2)	0.00		0.00	
	Pitch (cm)	30.00		30.00	
	As2 (cm2)	0.00		0.00	
Total of As		6.62	(OK)	6.62	(OK)
		-2.36		-3.30	
As'1	Diameter	D16		D16	
	Area (cm2)	1.99		1.99	
	Pitch (cm)	30.00		30.00	
	As'1 (cm2)	6.62		6.62	
As'2	Diameter	0.00		0.00	
	Area (cm2)	0.00		0.00	
	Pitch (cm)	25.00		25.00	
	As'2 (cm2)	0.00		0.00	
Total of As'		6.62	(OK)	6.62	(OK)
		-2.36		-3.30	
Check	np	0.00392		0.00392	
	k	0.20	0.000	0.16	0.000
	A's/As	1.00		1.00	
	C	10.37		12.83	
	S	41.48		68.05	
	Z	0.91	0.12	0.98	0.10
	Sc	12.55	OK	17.87	OK
	Ss	752.66	OK	1,422.04	OK
	Tc	0.95	OK	1.04	OK
	x	50.62		40.13	

(c) Downstream Wing Wall of Deli River Weir

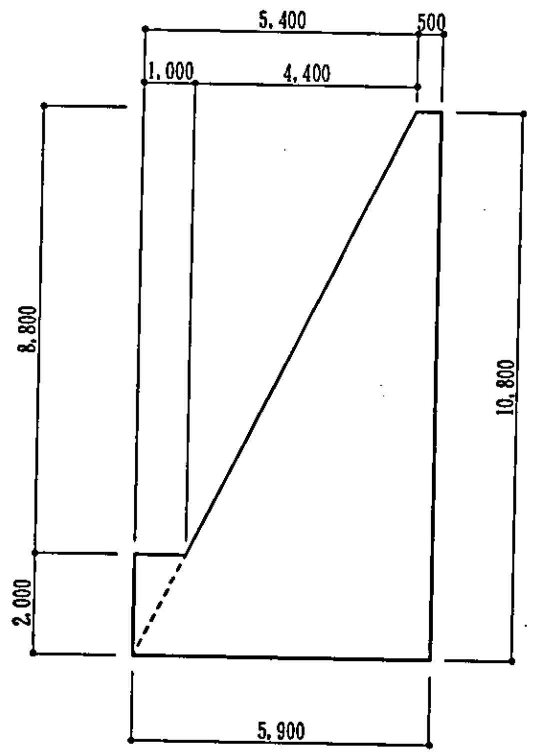


Fig. 2.4.8 DIMENSIONS OF DOWNSTREAM WING WALL (DELI RIVR WEIR)

Table DOWN STREAM WING WALL
(DELI RIVER WEIR)

Design Condition	Normal Condition			
Backfill Material				
Height		hb	0.000	
Slope		n=1:	#######	
Unit Weight		Gs	1.900	t/m3
Internal Friction Angle		fs	30.000	
Water Height	Back Side	Hwb	3.000	m
	Front Side	Hwf	2.000	
Load on Backfill		q	1.000	t/m2
Features of Wall				
Height of Wall		H	8.800	
Slope	Front slope	nf	0.000	
	Height	hf	0.000	
	Back slope	nb	0.500	
Top Width		B	0.500	m
Toe Length		Lt	0.000	m
Toe Height		Ht	2.000	m
Heel Length		Lt	1.000	m
Foundation				
Internal Friction Angle		ff	40.000	
Coheson		Cf	0.000	t/m2
Concrete				
Unit Weight		Gc	2.350	t/m3
Design Strength		σck	160.000	kg/cm2
Coefficient of Earthquake		k	0.000	
Factor of Safty			1.500	
Bearing Strength		qa	65.600	

B/6-e	0.304
Fs	3.273
Qumax	36.310

Load			Vertical	Horizontal	Arm	Moment
Self Weight	Wall	W1	45.496		1.967	89.475
		W2	10.340		0.250	2.585
		W3	0.000		0.000	0.000
	Footing	W4	27.730		2.950	81.804
	Backfill	We1	0.000		0.500	0.000
		We2	0.000		3.200	0.000
		We3	15.979		2.433	38.882
		We4	27.550		4.650	128.108
		We5	2.025		4.400	8.910
		We6	2.700		5.400	14.580
	Water	Ww1	0.000		0.000	0.000
		Ww2	0.000		0.000	0.000
		Ww3	0.000		0.000	0.000
		Ww4	2.250		4.400	9.900
		Ww5	3.000		5.400	16.200
Water Pressure	Front side	Pw1		8.000	1.333	10.667
	Back side	Pw2		-12.500	1.667	-20.833
Seismic Force	Wall	Ke1		0.000	4.933	0.000
		Ke2		0.000	6.400	0.000
		Ke3		0.000	2.000	0.000
	Footing	Ke4		0.000	1.000	0.000
	Backfill	We1		0.000	10.800	0.000
		We2		0.000	10.800	0.000
		We3		0.000	8.867	0.000
		We4		0.000	7.900	0.000
		We5		0.000	4.000	0.000
		We6		0.000	3.500	0.000
Earth Pressure	Horizontal	Phd		-8.971	6.933	-62.199
(dry area)	Vertical	Pvd	5.179		5.900	30.559
Earth Pressure	Horizontal	Phw		-3.539	1.667	-5.898
(submarged 1)	Vertical	Pvw	2.043		5.900	12.054
Earth Pressure	Horizontal	Phw		-15.467	2.500	-38.668
(submarged 2)	Vertical	Pvw	8.930		5.900	52.687
Uplift		U1	-23.600		2.950	-69.620
		U2	-2.950		3.933	-11.603
Total			126.673	-32.477		287.588

Stability Calculation

Middle third	B=	5.900 m	
	B/6 =	0.983 m	
	e =	-0.680 m	B/6>e OK
	(B/6-e	0.304)	
Sliding	FS=	3.273	OK
Bearing strength	U1=	36.310 t/m2	OK
	U2=	6.630 t/m2	OK

Table DOWN STREAM WING WALL
(DELI RIVER WEIR)

Design Condition	Flood Condition				
Backfill Material					
Height		hb	0.000		
Slope		n=1:	#######		
Unit Weight		Gs	1.900 t/m3		
Internal Friction Angle		fs	30.000		
Water Height	Back Side	Hwb	4.933 m		
	Front Side	Hwf	3.000		
Load on Backfill		q	1.000 t/m2		
Features of Wall					
Height of Wall		H	8.800		
Slope	Front slope	nf	0.000		
	Height	hf	0.000		
	Back slope	nb	0.500		
Top Width		B	0.500 m	B/6-e	0.027
Toe Length		Lt	0.000 m	Fs	2.641
Toe Height		Ht	2.000 m	Qumax	38.980
Heel Length		Lt	1.000 m		
Foundation					
Internal Friction Angle		ff	40.000		
Coheson		Cf	0.000 t/m2		
Concrete					
Unit Weight		Gc	2.350 t/m3		
Design Strength		σck	210.000 kg/cm2		
Coefficient of Earthquake		k	0.000		
Factor of Safty			1.500		
Bearing Strength		qa	65.600		

Load			Vertical	Horizontal	Arm	Moment
Self Weight	Wall	W1	45.496		1.967	89.475
		W2	10.340		0.250	2.585
		W3	0.000		0.000	0.000
	Footing	W4	27.730		2.950	81.804
	Backfill	We1	0.000		0.500	0.000
		We2	0.000		3.200	0.000
		We3	7.102		1.789	12.704
		We4	25.468		4.167	106.119
		We5	5.476		4.078	22.330
		We6	4.440		5.400	23.976
	Water	Ww1	0.000		0.000	0.000
		Ww2	0.000		0.000	0.000
		Ww3	0.000		0.000	0.000
		Ww4	6.084		4.078	24.811
		Ww5	4.933		5.400	26.640
Water Pressure	Front side	Pw1		12.500	1.667	20.833
	Back side	Pw2		-24.036	2.311	-55.549
Seismic Force	Wall	Ke1		0.000	4.933	0.000
		Ke2		0.000	6.400	0.000
		Ke3		0.000	2.000	0.000
	Footing	Ke4		0.000	1.000	0.000
	Backfill	We1		0.000	10.800	0.000
		We2		0.000	10.800	0.000
		We3		0.000	9.511	0.000
		We4		0.000	8.867	0.000
		We5		0.000	5.289	0.000
		We6		0.000	4.467	0.000
Earth Pressure	Horizontal	Phd		-4.153	8.222	-34.147
(dry area)	Vertical	Pvd	2.398		5.900	14.147
Earth Pressure	Horizontal	Phw		-6.459	2.311	-14.928
(submarged 1)	Vertical	Pvw	3.729		5.900	22.003
Earth Pressure	Horizontal	Phw'		-14.893	3.467	-51.631
(submarged 2)	Vertical	Pvw'	8.599		5.900	50.733
Uplift		U1	-29.500		2.950	-87.025
		U2	-5.703		3.933	-22.433
Total			116.592	-37.041		232.447

Stability Calculation

Middle third	B=	5.900 m	
	B/6 =	0.983 m	
	e =	-0.956 m	B/6>e OK
	(B/6-e	0.027)	
Sliding	FS=	2.641	OK
Bearing strength	U1=	38.980 t/m2	OK
	U2=	0.543 t/m2	OK

Table DOWN STREAM WING WALL
(DELI RIVER WEIR)

Design Condition	Earthquake Condition					
Backfill Material						
Height		hb	0.000			
Slope		n=1:	#######			
Unit Weight		Gs	1.900 t/m3			
Internal Friction Angle		fs	30.000			
Water Height	Back Side	Hwb	2.000 m			
	Front Side	Hwf	2.000			
Load on Backfill		q	0.000 t/m2			
Features of Wall						
Height of Wall		H	8.800			
Slope	Front slope	nf	0.000			
	Height	hf	0.000			
	Back slope	nb	0.500			
Top Width		B	0.500 m		B/6-e	0.220
Toe Length		Lt	0.000 m		Fs	1.882
Toe Height		Ht	2.000 m		Qumax	67.856
Heel Length		Lt	1.000 m			
Foundation						
Internal Friction Angle		ff	40.000			
Coheson		Cf	0.000 t/m2			
Concrete						
Unit Weight		Gc	2.350 t/m3			
Design Strength		σck	210.000 kg/cm2			
Coefficient of Earthquake		k	0.183			
Factor of Safty			1.200			
Bearing Strength		qa	82.700			

Load			Vertical	Horizontal	Arm	Moment
Self Weight	Wall	W1	45.496		1.967	89.475
		W2	10.340		0.250	2.585
		W3	0.000		0.000	0.000
	Footing	W4	27.730		2.950	81.804
	Backfill	We1	0.000		0.500	0.000
		We2	0.000		3.200	0.000
		We3	21.964		2.767	60.767
		We4	25.840		4.900	126.616
		We5	0.900		4.567	4.110
		We6	1.800		5.400	9.720
	Water	Ww1	0.000		0.000	0.000
		Ww2	0.000		0.000	0.000
		Ww3	0.000		0.000	0.000
		Ww4	1.000		4.567	4.567
		Ww5	2.000		5.400	10.800
Water Pressure	Front side	Pw1		0.000	1.333	0.000
	Back side	Pw2		0.000	1.333	0.000
Seismic Force	Wall	Ke1		-8.326	4.933	-41.074
		Ke2		-1.892	6.400	-12.110
		Ke3		0.000	2.000	0.000
	Footing	Ke4		-5.075	1.000	-5.075
	Backfill	We1		0.000	10.800	0.000
		We2		0.000	10.800	0.000
		We3		-4.019	8.533	-34.299
		We4		-4.729	7.400	-34.993
		We5		-0.165	3.333	-0.549
		We6		-0.329	3.000	-0.988
Earth Pressure	Horizontal	Phd		-39.780	3.267	-129.947
(dry area)	Vertical	Pvd	7.147		5.900	42.167
Earth Pressure	Horizontal	Phw		0.000	1.333	0.000
(submarged 1)	Vertical	Pvw	0.000		5.900	0.000
Earth Pressure	Horizontal	Phw'		0.000	2.000	0.000
(submarged 2)	Vertical	Pvw'	0.000		5.900	0.000
Uplift		U1	0.000		2.950	0.000
		U2	0.000		3.933	0.000
Total			144.217	-64.315		173.576

Stability Calculation

Middle third	B=	5.900 m	
	B/3=	1.967 m	
	e=	-1.746 m	B/3>e OK
	(B/6-e	0.220)	
Sliding	FS=	1.882	OK
Bearing strength	U1=	67.856 t/m2	OK
	U2=	-18.969 t/m2	OK

(d) Apron Wall of Floodway Weir

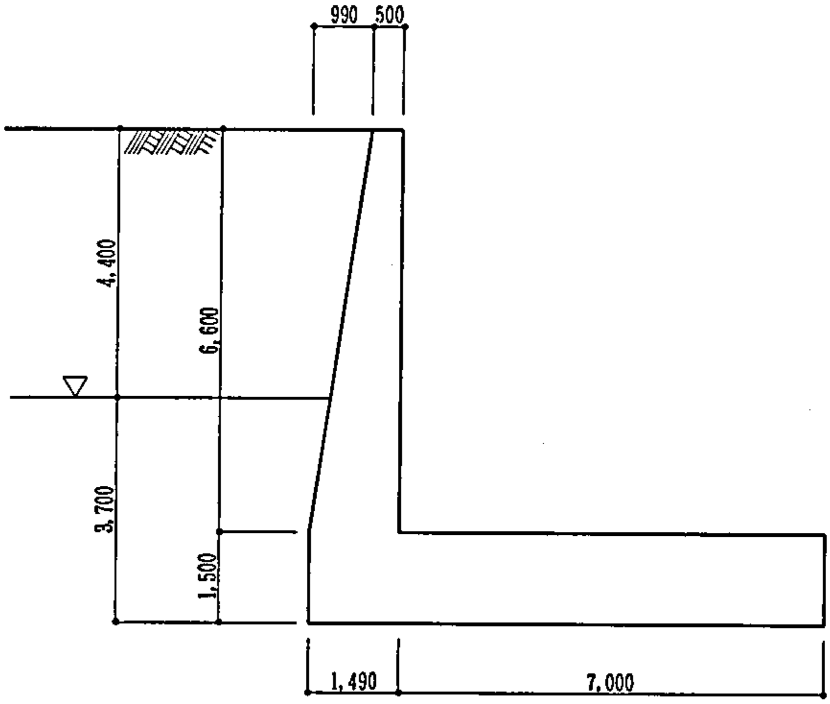


Fig. 2.4.9 BASIC DIMENSIONS
FLOODWAY WEIR - APRON WALL

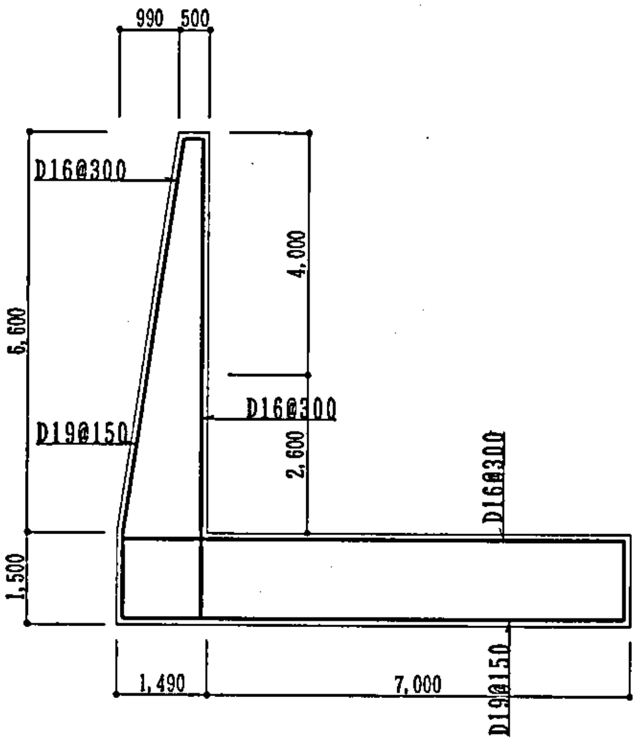


Fig. 2.4.10 REINFORCEMENT ARRANGEMENT
FLOODWAY WEIR - APRON WALL

Table 2.4. STABILITY CALCULATION

Floodway Weir, Apron Wall H=6.6 m (Nomal Condition)

Design Condition				
Backfill Material				
Height		hb		0.000
Slope		n=1:		########
Unit Weight		Gs		1.900 t/m3
Internal Friction Angle		fs		30.000
Slip angle	α	°		56.000
	Pa-Pa'			0.000
Coefficient of Earth Pressure		Kh		0.322
		Kv		0.117
Water Height Back Side		Hwb		-1.500 m
Front Side		Hwf		-1.500
Load on Backfill		q		1.000 t/m2
Features of Wall				
Height of Wall		H		6.600
Slope	Front	nf		0.000
	Height	hf		6.600
	Back	nb		0.150
				6.600
Top Width		B		0.500 m
Toe Length		Lt		7.000 m
Toe Height		Ht		1.500 m
Heel Length		Lt		0.000 m
Heel Height		Ht		1.500 m
Foundation				
Internal Friction Angle		ff		40.000
Coheson		Cf		0.000 t/m2
Concrete				
Unit Weight		Gc		2.500 t/m3
Design Strength		σck		210.000 kg/cm2
Coefficient of Earthquake		k		0.000
Factor of Safty				1.500
Bearing Strength		qa		21.867

B/6-e	0.186
Fs	2.142
Qumax	11.283
	19.302
	9.659
	1.388

Load			Vertical	Horizontal	Arm	Moment
Self Weight	Wall	W1	0.000		7.000	0.000
		W2	8.250		7.250	59.813
		W3	8.168		7.830	63.952
	Footing	W4	31.838		4.245	135.150
	Backfill	We1	0.000		7.500	0.000
		We2	0.000		7.995	0.000
		We3	9.349		8.310	77.694
		We4	-3.463		8.603	-29.788
		We5	0.152		8.565	1.301
	54.294	We6	0.000		8.490	0.000
	Water	Ww1	-10.500		3.500	-36.750
		Ww2	0.000		7.000	0.000
		Ww3	0.169		8.565	1.445
		Ww4	0.000		8.490	0.000
Water Pressure	Front side	Pw1		0.000	0.000	0.000
	Back side	Pw2		0.000	0.000	0.000
Seismic Force	Wall	Ke1		0.000	3.700	0.000
		Ke2		0.000	4.800	0.000
		Ke3		0.000	3.700	0.000
	Footing	Ke4		0.000	0.750	0.000
	Backfill	We1		0.000	8.100	0.000
		We2		0.000	8.100	0.000
		We3		0.000	5.900	0.000
		We4		0.000	4.800	0.000
		We5		0.000	1.000	0.000
		We6		0.000	0.750	0.000
Earth Pressure	Horizontal	Ph		-20.087	2.700	-54.235
	Vertical	Pv	7.311		8.490	62.071
Uplift		U1	0.000		4.245	0.000
		U2	0.000		5.660	0.000
Total			51.273	-20.087		280.652

Stability Calculation

Middle third	**B=**	**8.490 m**		
	B/6 =	**1.415 m**		
	e =	**1.229 m**	**B/6>e OK**	**5.474**
	(B/6-e	**0.186)**		
Sliding	**FS=**	**2.142**	**OK**	
Bearing strength	**U1=**	**0.795 t/m2**	**OK**	
	U2=	**11.283 t/m2**	**OK**	

Table 2.4. STABILITY CALCULATION

Floodway Weir, Apron Wall H=6.6 m Flood Condition)

Design Condition					
Backfill Material					
Height		hb	0.000		
Slope		n=1:	########		
Unit Weight		Gs	1.900 t/m3		
Internal Friction Angle		fs	30.000		
Slip angle	α	°	56.000		
	Pa-Pa'		0.000		
Coefficient of Earth Pressure		Kh	0.322		
		Kv	0.117		
Water Height	Back Side	Hwb	2.200 m		
	Front Side	Hwf	0.000		
Load on Backfill		q	0.000 t/m2		
Features of Wall					
Height of Wall		H	6.600		
Slope	Front	nf	0.000		
	Height	hf	6.600		
	Back	nb	0.150		
			6.600		
Top Width		B	0.500 m	B/6-e	0.535
Toe Length		Lt	7.000 m	Fs	1.326
Toe Height		Ht	1.500 m	Qumax	7.583
Heel Length		Lt	0.000 m		
Heel Height		Ht	1.500 m		19.302
Foundation					
Internal Friction Angle		ff	40.000		9.659
Coheson		Cf	0.000 t/m2		1.388
Concrete					
Unit Weight		Gc	2.500 t/m3		
Design Strength		σck	210.000 kg/cm2		
Coefficient of Earthquake		k	0.000		
Factor of Safty			1.300		
Bearing Strength		qa	21.867		

Load			Vertical	Horizontal	Arm	Moment
Self Weight	Wall	W1	0.000		7.000	0.000
		W2	8.250		7.250	59.813
		W3	8.168		7.830	63.952
	Footing	W4	31.838		4.245	135.150
	Backfill	We1	0.000		7.500	0.000
		We2	0.000		7.995	0.000
		We3	2.759		7.940	21.905
		We4	2.759		8.325	22.967
		We5	0.327		8.380	2.738
		We6	0.000		8.490	0.000
	Water	Ww1	0.000		3.500	0.000
		Ww2	0.000		7.000	0.000
		Ww3	0.363		8.380	3.042
		Ww4	0.000		8.490	0.000
Water Pressure	Front side	Pw1		1.125	0.500	0.563
	Back side	Pw2		-6.161	1.233	-7.598
Seismic Force	Wall	Ke1		0.000	3.700	0.000
		Ke2		0.000	4.800	0.000
		Ke3		0.000	3.700	0.000
	Footing	Ke4		0.000	0.750	0.000
	Backfill	We1		0.000	8.100	0.000
		We2		0.000	8.100	0.000
		We3		0.000	5.900	0.000
		We4		0.000	4.800	0.000
		We5		0.000	2.233	0.000
		We6		0.000	2.600	0.000
Earth Pressure	Horizontal	Ph		-20.087	2.700	-54.235
	Vertical	Pv	7.311		8.490	62.071
Uplift		U1	-12.735		4.245	-54.060
		U2	-9.339		5.660	-52.859
Total			39.699	-25.122		203.448

Stability Calculation

Middle third	B=	8.490 m		
	B/6 =	1.415 m		
	e =	0.880 m	B/6>e OK	5.125
	(B/6-e	0.535)		
Sliding	FS=	1.326	OK	
Bearing strength	U1=	1.769 t/m2	OK	
	U2=	7.583 t/m2	OK	

Table 2.4. STABILITY CALCULATION

Floodway Weir, Apron Wall H=6.6 m (Earthquake Condition)

Design Condition				
Backfill Material				
Height		hb		0.000
Slope		n=1:		########
Unit Weight		Gs		1.900 t/m3
Internal Friction Angle		fs		30.000
Slip angle	α		°	56.000
	Pa-Pa'			0.000
Coefficient of Earth Pressure		Kh		0.322
		Kv		0.117
Water Height	Back Side	Hwb		-1.500 m
	Front Side	Hwf		-1.500
Load on Backfill		q		1.000 t/m2
Features of Wall				
Height of Wall		H		6.600
Slope	Front	nf		0.000
	Height	hf		6.600
	Back	nb		0.150
				6.600
Top Width		B		0.500 m
Toe Length		Lt		7.000 m
Toe Height		Ht		1.500 m
Heel Length		Lt		0.000 m
Heel Height		Ht		1.500 m
Foundation				
Internal Friction Angle		fF		40.000
Coheson		Cf		0.000 t/m2
Concrete				
Unit Weight		Gc		2.500 t/m3
Design Strength		σck		210.000 kg/cm2
Coefficient of Earthquake		k		0.183
Factor of Safty				1.100
Bearing Strength		qa		21.867

B/6-e	0.659
Fs	1.433
Qumax	9.266
	19.302
	9.659
	1.388

Load			Vertical	Horizontal	Arm	Moment
Self Weight	Wall	W1	0.000		7.000	0.000
		W2	8.250		7.250	59.813
		W3	8.168		7.830	63.952
	Footing	W4	31.838		4.245	135.150
	Backfill	We1	0.000		7.500	0.000
		We2	0.000		7.995	0.000
		We3	9.349		8.310	77.694
		We4	-3.463		8.603	-29.788
		We5	0.152		8.565	1.301
	54.294	We6	0.000		8.490	0.000
	Water	Ww1	-10.500		3.500	-36.750
		Ww2	0.000		7.000	0.000
		Ww3	0.169		8.565	1.445
		Ww4	0.000		8.490	0.000
Water Pressure	Front side	Pw1		0.000	0.000	0.000
	Back side	Pw2		0.000	0.000	0.000
Seismic Force	Wall	Ke1		0.000	3.700	0.000
		Ke2		-1.510	4.800	-7.247
		Ke3		-1.495	3.700	-5.530
	Footing	Ke4		-5.826	0.750	-4.370
	Backfill	We1		0.000	8.100	0.000
		We2		0.000	8.100	0.000
		We3		-1.711	5.900	-10.095
		We4		0.634	4.800	3.042
		We5		-0.028	1.000	-0.028
		We6		0.000	0.750	0.000
Earth Pressure	Horizontal	Ph		-20.087	2.700	-54.235
	Vertical	Pv	7.311		8.490	62.071
Uplift		U1	0.000		4.245	0.000
		U2	0.000		5.660	0.000
Total			51.273	-30.023		256.424

Stability Calculation

Middle third	B=	8.490 m		
	B/6 =	1.415 m		
	e =	0.756 m	B/6>e OK	5.001
	(B/6-e	0.659)		
Sliding	FS=	1.433	OK	
Bearing strength	U1=	2.812 t/m2	OK	
	U2=	9.266 t/m2	OK	

Table 4.2. Load Calculation (Floodway Weir, Apron Wall)

			Apron Wall		Apron Wall	
			Normal	Earthquake	Normal	Earthquake
			H=6.6m	H=6.6m	H=4.0m	H=4.0m
Wall Height	H	m	6.600	6.600	4.000	4.000
Top Width	B1	m	0.500	0.500	0.500	0.500
Back-side Slope	n	m	0.150	0.150	0.150	0.150
C. of EQ	k		0.000	0.183	0.000	0.183
C. of E.P	kh		0.364	0.364	0.364	0.364
U.W. of Conc.	rc	t/m^3	2.500	2.500	2.500	2.500
U.W. of Soil	rs	t/m^4	1.900	1.900	1.900	1.900
Water Depth	hw	m	4.400	0.000	2.667	0.000
Self Weight	W1	t	8.250	8.250	5.000	5.000
	W2	t	8.168	8.168	3.000	3.000
Earthquake Load	Ke1	t	0.000	1.510	0.000	0.915
	Ke2	t	0.000	1.495	0.000	0.549
Water Pressure	Wp	t	9.680	0.000	3.556	0.000
Earth Pressure	We	t	15.065	15.065	5.533	5.533
Arm Length	Lke1	m	3.300	3.300	2.000	2.000
	Lke2	m	2.200	2.200	1.333	1.333
	Lwp	m	1.467	0.000	0.889	0.000
	Lwe	m	2.200	2.200	1.333	1.333
Moment	Mke1	t·m	0.000	4.982	0.000	1.830
	Mke2	t·m	0.000	3.288	0.000	0.732
	Mwp	t·m	14.197	0.000	3.160	0.000
	Mwe	t·m	33.142	33.142	7.378	7.378
Axis Force	N	t	16.418	16.418	8.000	8.000
Moment	M	t·m	47.339	41.412	10.538	9.940
Shear Force	S	t	24.745	18.069	9.089	6.997
Effective Thikness	d	cm	139.000	139.000	100.000	100.000
	d'	cm	10.000	10.000	10.000	10.000
	u	cm	53.854	53.854	41.875	41.875
	b	cm	100.000	100.000	100.000	100.000

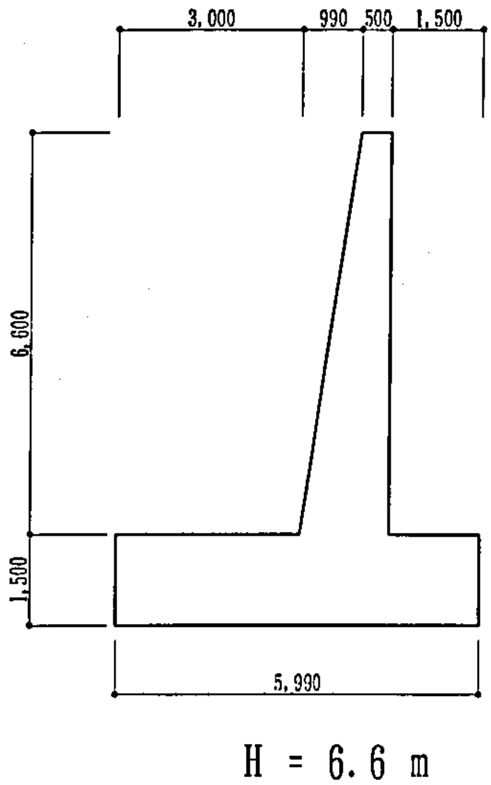
Table 4.2. Load Calculation, Floodway Weir,
(Footing for Apron Wall)

			Footing for Apron Wall		
			Normal	Earthquake	Earthquake
			L=7m	L=7m	L=7m
		L	7.000	7.000	7.000
Slf Weight		Wc	26.250	26.250	26.250
Reaction		q1	0.795	2.821	1.769
		q2	11.283	9.266	7.583
		Q	42.273	42.305	32.732
Arm Length	Lke1	m	3.500	3.500	3.500
	Lke2	m	4.974	5.756	5.549
Moment	Mke1	t·m	91.875	91.875	91.875
	Mke2	t·m	-85.652	-52.634	-47.481
Axis Force	N	t	0.000	0.000	0.000
Moment	M	t·m	6.223	39.241	44.394
Shear Force	S	t	42.273	42.305	32.732
Effective Thikness	d	cm	140.000	140.000	140.000
	d'	cm	10.000	10.000	10.000
	u	cm	75.000	75.000	75.000
	b	cm	100.000	100.000	100.000

Table STRESS CALCULATION FOR APRON WALL OF FLOODWAY WEIR

Type		Wing Wall (Upstream)				Wing Wall (Upstream)				Wing Wall Footing (Upstream)			
		Normal		Earthquake		Normal		Earthquake		Normal		Earthquake	
N	t	16.42		16.42		8.00		8.00		0.00		0.00	
M	tm	47.34		41.41		10.54		9.94		39.24		44.39	
Q	t	24.74		18.07		9.09		7.00		42.30		32.73	
d	cm	139.00		139.00		100.00		100.00		140.00		140.00	
d'	cm	10.00		10.00		10.00		10.00		10.00		10.00	
u	cm	53.85		53.85		41.88		41.88		75.00		75.00	
b	cm	100.00		100.00		100.00		100.00		100.00		100.00	
r	A's/As	1.00		1.00		1.00		1.00		1.00		1.00	
n		15.00		15.00		15.00		15.00		15.00		15.00	
Sca	kg/cm2	70.00		105.00		70.00		105.00		70.00		105.00	
Ssa	kg/cm2	1,600.00		2,400.00		1,600.00		2,400.00		1,600.00		2,400.00	
Tca	kg/cm2	4.25		6.38		4.25		6.38		4.25		6.38	
M'		56.18		50.25		13.89		13.29		39.24		44.39	
C		24.07	24.07	40.37	40.37	50.40	50.40	79.01	79.01	34.96	64.24	46.36	80.39
k (C)		0.000	0.000	0.05	0.000	0.033	-0.002	0.03	0.000	0.033	0.000	0.03	0.000
np (C)		0.0000	0.0000	0.0008	0.0007	-0.0020	0.0002	0.0001	0.0001	0.0006	0.0005	0.0004	0.0003
S		36.68	36.68	61.51	61.51	76.80	76.80	120.39	120.39	53.28	53.28	70.64	70.64
k (S)		0.204	0.000	0.16	0.000	0.136	0.067	0.12	0.000	0.171	0.000	0.15	0.000
np (S)		0.0182	0.0182	0.0098	0.0098	0.0716	0.0050	0.0037	0.0037	0.0197	0.0197	0.0147	0.0147
d'/d		0.07		0.07		0.10		0.10		0.07		0.07	
f	cm	342.20		306.10		173.60		166.12		9,999.99		9,999.99	
f/d		2.46		2.20		1.74		1.66		71.43		71.43	
np		0.0182		0.0098		0.0716		0.0037		0.0197		0.0147	
r-As		16.86		9.11		47.73		2.44		18.37		13.76	
r-As'		16.86		9.11		47.73		2.44		18.37		13.76	
Re-Bar Arrangement													
As1	Diameter	D19		D19		D13		D13		D19		D19	
	Area (cm2)	2.87		2.87		1.27		1.27		2.87		2.87	
	Pitch (cm)	30.00		30.00		30.00		30.00		30.00		30.00	
	As1	9.55		9.55		4.22		4.22		9.55		9.55	
As2	Diameter	D19		0.00		0.00		0.00		D19		D19	
	Area (cm2)	2.87		0.00		0.00		0.00		2.87		2.87	
	Pitch (cm)	30.00		30.00		30.00		30.00		30.00		30.00	
	As2 (cm2)	9.55		0.00		0.00		0.00		9.55		9.55	
Total of As		19.10	(OK)	9.55	(OK)	4.22	(NG)	4.22	(OK)	19.10	(OK)	19.10	(OK)
		-16.86		-9.11		-47.73		-2.44		-18.37		-13.76	
As'1	Diameter	D22		D19		D13		D13		D13		D13	
	Area (cm2)	3.87		2.87		1.27		1.27		1.27		1.27	
	Pitch (cm)	30.00		30.00		30.00		30.00		30.00		30.00	
	As'1 (cm2)	12.90		9.55		4.22		4.22		4.22		4.22	
As'2	Diameter	0.00		0.00		0.00		0.00		0.00		0.00	
	Area (cm2)	0.00		0.00		0.00		0.00		0.00		0.00	
	Pitch (cm)	25.00		25.00		25.00		25.00		25.00		25.00	
	As'2 (cm2)	0.00		0.00		0.00		0.00		0.00		0.00	
Total of As'		12.90	(NG)	9.55	(OK)	4.22	(NG)	4.22	(OK)	4.22	(NG)	4.22	(NG)
		-16.86		-9.11		-47.73		-2.44		-18.37		-13.76	
Check	np	0.02061		0.01031		0.00634		0.00634		0.02046		0.02046	
	k	0.20	-0.015	0.17	0.000	0.18	0.008	0.16	0.000	0.18	0.000	0.18	0.000
	A's/As	0.68		1.00		1.00		1.00		0.22		0.22	
	C	9.86		11.83		11.68		13.20		11.39		11.39	
	S	39.52		58.83		54.44		71.70		51.35		51.35	
	Z	1.06	0.18	1.04	0.13	1.01	0.12	1.02	0.11	1.06	0.18	1.06	0.18
	Sc	28.66	OK	30.78	OK	16.22	OK	17.54	OK	22.80	OK	25.79	OK
	Ss	1,723.71	NG	2,295.35	OK	1,134.17	OK	1,429.36	OK	1,542.00	OK	1,744.49	OK
	Tc	1.78	OK	1.30	OK	0.91	OK	0.70	OK	3.02	OK	2.34	OK
	x	27.75		23.28		17.67		15.55		25.41		25.41	

(e) Upstream Wing Wall of Floodway Weir



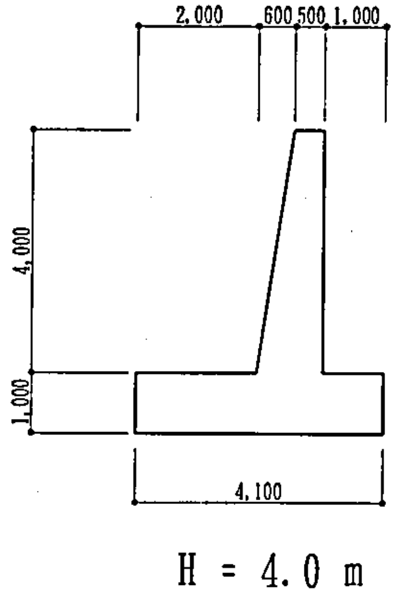
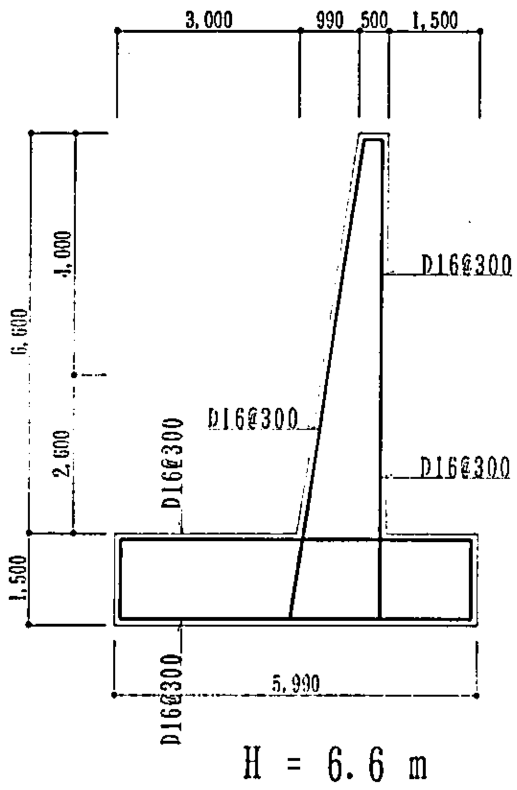


Fig. 2.4.11 BASIC DIMENSIONS
FLOODWAY WEIR - WING WALL



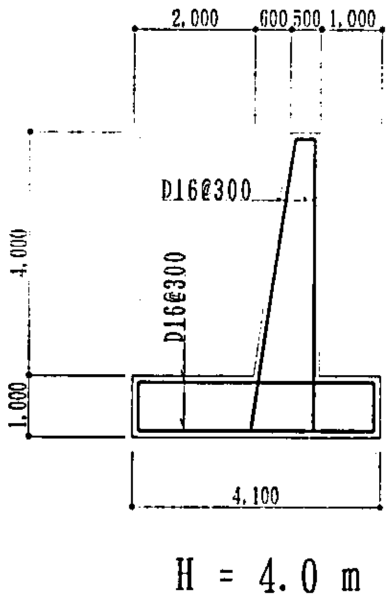


Fig. 2.4.12 BASIC DIMENSIONS
FLOODWAY WEIR - WING WALL

Table 2.4. STABILITY CALCULATION

Floodway Weir, Upstream Wing Wall H=6.8 m (Nomal Condition)

Design Condition					
Backfill Material					
Height		hb	0.000		
Slope		n=1:	########		
Unit Weight		Gs	1.900 t/m3		
Internal Friction Angle		fs	30.000		
Slip angle	α		56.000		
	Pa-Pa'		-0.000		
Coefficient of Earth Pressure		Kh	0.279		
		Kv	0.102		
Water Height	Back Side	Hwb	2.400 m		
	Front Side	Hwf	0.000		
Load on Backfill		q	1.000 t/m2		
Features of Wall					
Height of Wall		H	7.200		
Slope	Front	nf	0.000		
	Height	hf	7.200		
	Back	nb	0.150		
			7.200		
Top Width		B	0.500 m	B/6-e	0.888
Toe Length		Lt	1.500 m	Fs	1.826
Toe Height		Ht	1.500 m	Qumax	14.941
Heel Length		Lt	3.000 m		
Heel Height		Ht	1.500 m		16.608
Foundation					
Internal Friction Angle		ff	30.964		7.601
Coheson		Cf	0.000 t/m2		0.986
Concrete					
Unit Weight		Gc	2.500 t/m3		
Design Strength		σck	210.000 kg/cm2		
Coefficient of Earthquake		k	0.000		
Factor of Safty			1.500		
Bearing Strength		qa	21.867		

Load			Vertical	Horizontal	Arm	Moment
Self Weight	Wall	W1	0.000		1.500	0.000
		W2	9.000		1.750	15.750
		W3	9.720		2.360	22.939
	Footing	W4	22.800		3.040	69.312
	Backfill	We1	0.000		2.000	0.000
		We2	0.000		4.040	0.000
		We3	3.283		2.480	8.142
		We4	30.643		4.400	134.830
		We5	0.389		2.960	1.151
	82.315	We6	6.480		4.580	29.678
	Water	Ww1	0.000		0.750	0.000
		Ww2	0.000		1.500	0.000
		Ww3	0.432		2.960	1.279
		Ww4	7.200		4.580	32.976
Water Pressure	Front side	Pw1		1.125	0.500	0.563
	Back side	Pw2		-7.605	1.300	-9.887
Seismic Force	Wall	Ke1		0.000	3.900	0.000
		Ke2		0.000	5.100	0.000
		Ke3		0.000	3.900	0.000
	Footing	Ke4		0.000	0.750	0.000
	Backfill	We1		0.000	8.700	0.000
		We2		0.000	8.700	0.000
		We3		0.000	6.300	0.000
		We4		0.000	5.100	0.000
		We5		0.000	2.300	0.000
		We6		0.000	2.700	0.000
Earth Pressure	Horizontal	Ph		-20.089	2.900	-58.259
	Vertical	Pv	7.312		6.080	44.456
Uplift		U1	-9.120		3.040	-27.725
		U2	-7.296		4.053	-29.573
Total			80.843	-26.569		235.633

Stability Calculation

Middle third	**B=**	**6.080 m**		
	B/6 =	**1.013 m**		
	e =	**-0.125 m**	**B/6>e OK**	**2.915**
	(B/6-e	**0.888)**		
Sliding	**FS=**	**1.826**	**OK**	
Bearing strength	**U1=**	**14.941 t/m2**	**OK**	
	U2=	**11.652 t/m2**	**OK**	

Table 2.4. STABILITY CALCULATION

Floodway Weir, Upstream Wing Wall H=6.8 m (Earthquake)

Design Condition						
Backfill Material						
Height		hh	0.000			
Slope		n=1:	########			
Unit Weight		Gs	1.900	t m3		
Internal Friction Angle		fs	30.000			
Slip angle	α	˚	55.056			
	Pa-Pa'		0.001			
Coefficient of Earth Pressure		Kh	0.279			
		Kv	0.102			
Water Height	Back Side	Hwb	-1.500	m		
	Front Side	Hwf	-1.500			
Load on Backfill		q	0.000	t m2		
Features of Wall						
Height of Wall		H	7.200			
Slope	Front	nf	0.000			
	Height	hf	7.200			
	Back	nb	0.150			
			7.200			
Top Width		B	0.500	m	B/6-e	0.367
Toe Length		Lt	1.500	m	Fs	1.527
Toe Height		Ht	1.500	m	Qumax	25.594
Heel Length		Lt	3.000	m		
Heel Height		Ht	1.500	m		16.608
Foundation						
Internal Friction Angle		ff	30.964			7.601
Coheson		Cf	0.000	t/m2		0.986
Concrete						
Unit Weight		Gc	2.500	t/m3		
Design Strength		σck	210.000	kg/cm2		
Coefficient of Earthquake		k	0.183			
Factor of Safty			1.100			
Bearing Strength		qa	41.350			

Load			Vertical	Horizontal	Arm	Moment
Self Weight	Wall	W1	0.000		1.500	0.000
		W2	9.000		1.750	15.750
		W3	9.720		2.360	22.939
	Footing	W4	22.800		3.040	69.312
	Backfill	We1	0.000		2.000	0.000
		We2	0.000		4.040	0.000
		We3	10.786		2.870	30.955
		We4	45.871		4.693	215.248
		We5	0.152		3.155	0.479
	94.278	We6	-4.050		4.580	-18.549
	Water	Ww1	-2.250		0.750	-1.688
		Ww2	0.000		1.500	0.000
		Ww3	0.169		3.155	0.532
		Ww4	-4.500		4.580	-20.610
Water Pressure	Front side	Pw1		0.000	0.000	0.000
	Back side	Pw2		0.000	0.000	0.000
Seismic Force	Wall	Ke1		0.000	3.900	0.000
		Ke2		-1.647	5.100	-8.400
		Ke3		-1.779	3.900	-6.937
	Footing	Ke4		-4.172	0.750	-3.129
	Backfill	We1		0.000	8.700	0.000
		We2		0.000	8.700	0.000
		We3		-1.974	6.300	-12.435
		We4		-8.394	5.100	-42.811
		We5		-0.028	1.000	-0.028
		We6		0.741	0.750	0.556
Earth Pressure	Horizontal	Ph		-20.073	2.900	-58.213
	Vertical	Pv	7.306		6.080	44.421
Uplift		U1	0.000		3.040	0.000
		U2	0.000		4.053	0.000
Total			95.003	-37.326		227.394

Stability Calculation

Middle third	B=	6.080 m		
	B/6 =	1.013 m		
	e =	-0.646 m	B/6>e OK	2.394
	(B/6-e	0.367)		
Sliding	FS=	1.527	OK	
Bearing strength	U1=	25.594 t/m2	OK	
	U2=	5.657 t/m2	OK	

Table 2.4. STABILITY CALCULATION

Floodway Weir, Upstream Wing Wall H=4.0 m (Normal)

Design Condition						
Backfill Material						
Height		hb	3.000			
Slope		n=1:	4.000			
Unit Weight		Gs	1.900 t/m3			
Internal Friction Angle		fs	30.000			
Slip angle	α		46.818			
	Pa-Pa'		0.001			
Coefficient of Earth Pressure		Kh	0.332			
		Kv	0.092			
Water Height Back Side		Hwb	1.333 m			
Front Side		Hwf	0.000			
Load on Backfill		q	1.000 t/m2			
Features of Wall						
Height of Wall		H	4.000			
Slope	Front	nf	0.000			
	Height	hf	4.000			
	Back	nb	0.150			
			4.000			
Top Width		B	0.500 m	B/6-e		0.546
Toe Length		Lt	1.000 m	Fs		1.627
Toe Height		Ht	1.000 m	Qumax		9.750
Heel Length		Lt	2.000 m			
Heel Height		Ht	1.000 m			7.300
Foundation						
Internal Friction Angle		ff	30.964			5.340
Coheson		Cf	0.000 t/m2			1.210
Concrete						
Unit Weight		Gc	2.500 t/m3			
Design Strength		σck	210.000 kg/cm2			
Coefficient of Earthquake		k	0.000			
Factor of Safty			1.500			
Bearing Strength		qa	21.867			

Load			Vertical	Horizontal	Arm	Moment
Self Weight	Wall	W1	0.000		1.000	0.000
		W2	5.000		1.250	6.250
		W3	3.000		1.700	5.100
	Footing	W4	10.250		2.050	21.013
	Backfill	We1	1.606		3.233	5.191
		We2	0.000		4.100	0.000
		We3	1.013		1.767	1.790
		We4	11.147		3.000	33.440
		We5	0.120		2.033	0.244
	34.536	We6	2.400		3.100	7.440
	Water	Ww1	0.000		0.500	0.000
		Ww2	0.000		1.000	0.000
		Ww3	0.133		2.033	0.271
		Ww4	2.667		3.100	8.267
Water Pressure	Front side	Pw1		0.500	0.333	0.167
	Back side	Pw2		-2.722	0.778	-2.117
Seismic Force	Wall	Ke1		0.000	2.333	0.000
		Ke2		0.000	3.000	0.000
		Ke3		0.000	2.333	0.000
	Footing	Ke4		0.000	0.500	0.000
	Backfill	We1		0.000	5.217	0.000
		We2		0.000	5.325	0.000
		We3		0.000	3.667	0.000
		We4		0.000	3.000	0.000
		We5		0.000	1.444	0.000
		We6		0.000	1.667	0.000
Earth Pressure	Horizontal	Ph		-10.058	1.883	-18.943
	Vertical	Pv	2.796		4.100	11.464
Uplift		U1	-4.100		2.050	-8.405
		U2	-2.733		2.733	-7.471
Total			33.298	-12.281		63.699

Stability Calculation

Middle third	B=	4.100 m		
	B/6 =	0.683 m		
	e =	-0.137 m	B/6>e OK	1.913
	(B/6-e	0.546)		
Sliding	FS=	1.627	OK	
Bearing strength	U1=	9.750 t/m2	OK	
	U2=	6.493 t/m2	OK	

Table 2.4. STABILITY CALCULATION

Floodway Weir, Upstream Wing Wall H=4.0 m (Earthquake)

Design Condition					
Backfill Material					
Height		hb	3.000		
Slope		n=1:	3.000		
Unit Weight		Gs	1.900 t/m3		
Internal Friction Angle		fs	30.000		
Slip angle	α	°	48.180		
	Pa-Pa'		0.001		
Coefficient of Earth Pressure		Kh	0.384		
		Kv	0.121		
Water Height	Back Side	Hwb	-1.000 m		
	Front Side	Hwf	-1.000		
Load on Backfill		q	1.000 t/m2		
Features of Wall					
Height of Wall		H	4.000		
Slope	Front	nf	0.000		
	Height	hf	4.000		
	Back	nb	0.150		
			4.000		
Top Width		B	0.500 m	B/6-e	0.176
Toe Length		Lt	1.000 m	Fs	1.235
Toe Height		Ht	1.000 m	Qumax	17.353
Heel Length		Lt	2.000 m		
Heel Height		Ht	1.000 m		7.300
Foundation					
Internal Friction Angle		ff	30.964		5.340
Coheson		Cf	0.000 t/m2		1.210
Concrete					
Unit Weight		Gc	2.500 t/m3		
Design Strength		σck	210.000 kg/cm2		
Coefficient of Earthquake		k	0.183		
Factor of Safty			1.100		
Bearing Strength		qa	41.350		

Load			Vertical	Horizontal	Arm	Moment
Self Weight	Wall	W1	0.000		1.000	0.000
		W2	5.000		1.250	6.250
		W3	3.000		1.700	5.100
	Footing	W4	10.250		2.050	21.013
	Backfill	We1	2.141		3.233	6.921
		We2	0.000		4.100	0.000
		We3	3.563		2.000	7.125
		We4	17.575		3.175	55.801
		We5	0.068		2.150	0.145
	39.796	We6	-1.800		3.100	-5.580
	Water	Ww1	-1.000		0.500	-0.500
		Ww2	0.000		1.000	0.000
		Ww3	0.075		2.150	0.161
		Ww4	-2.000		3.100	-6.200
Water Pressure	Front side	Pw1		0.000	0.000	0.000
	Back side	Pw2		0.000	0.000	0.000
Seismic Force	Wall	Ke1		0.000	2.333	0.000
		Ke2		-0.915	3.000	-2.745
		Ke3		-0.549	2.333	-1.281
	Footing	Ke4		-1.876	0.500	-0.938
	Backfill	We1		-0.392	5.289	-2.072
		We2		0.000	5.433	0.000
		We3		-0.652	3.667	-2.390
		We4		-3.216	3.000	-9.649
		We5		-0.012	0.667	-0.008
		We6		0.329	0.500	0.165
Earth Pressure	Horizontal	Ph		-12.556	1.956	-24.555
	Vertical	Pv	3.952		4.100	16.201
Uplift		U1	0.000		2.050	0.000
		U2	0.000		2.733	0.000
Total			40.822	-19.839		62.964

Stability Calculation

Middle third	B=	4.100 m		
	B/6 =	0.683 m		
	e=	-0.508 m	B/6>e OK	1.542
	(B/6-e	0.176)		
Sliding	FS=	1.235	OK	
Bearing strength	U1=	17.353 t/m2	OK	
	U2=	2.561 t/m2	OK	

Table 4.2. Load Calculation (Floodway Weir, Upstream Wing Wall)

			Wing Wall		Wing Wall	
			Normal	Earthquake	Normal	Earthquake
			H=6.6m	H=6.6m	H=4.0m	H=4.0m
Wall Height	H	m	6.600	6.600	4.000	4.000
Top Width	B1	m	0.500	0.500	0.500	0.500
Back-side Slope	n	m	0.150	0.150	0.150	0.150
C. of EQ	k		0.000	0.183	0.000	0.183
C. of E.P	kh		0.364	0.364	0.364	0.364
U.W. of Conc.	rc	t/m³	2.500	2.500	2.500	2.500
U.W. of Soil	rs	t/m⁴	1.900	1.900	1.900	1.900
Water Depth	hw	m	2.200	0.000	1.333	0.000
Self Weight	W1	t	8.250	8.250	5.000	5.000
	W2	t	8.168	8.168	3.000	3.000
Earthquake Load	Ke1	t	0.000	1.510	0.000	0.915
	Ke2	t	0.000	1.495	0.000	0.549
Water Pressure	Wp	t	2.420	0.000	0.889	0.000
Earth Pressure	We	t	15.065	15.065	5.533	5.533
Arm Length	Lke1	m	3.300	3.300	2.000	2.000
	Lke2	m	2.200	2.200	1.333	1.333
	Lwp	m	0.733	0.000	0.444	0.000
	Lwe	m	2.200	2.200	1.333	1.333
Moment	Mke1	t·m	0.000	4.982	0.000	1.830
	Mke2	t·m	0.000	3.288	0.000	0.732
	Mwp	t·m	1.775	0.000	0.395	0.000
	Mwe	t·m	33.142	33.142	7.378	7.378
Axis Force	N	t	16.418	16.418	8.000	8.000
Moment	M	t·m	34.917	41.412	7.773	9.940
Shear Force	S	t	17.485	18.069	6.422	6.997
Effective Thikness	d	cm	139.000	139.000	100.000	100.000
	d'	cm	10.000	10.000	10.000	10.000
	u	cm	53.854	53.854	41.875	41.875
	b	cm	100.000	100.000	100.000	100.000

Table STRESS CALCULATION FOR UPSTREAM WING WALL OF FLOODWAY WEIR

Type		Wing Wall (Upstream)				Wing Wall (Upstream)			
		Normal		Earthquake		Normal		Earthquake	
N	t	16.42		16.42		8.00		8.00	
M	tm	34.92		41.41		7.77		9.94	
Q	t	17.48		18.07		6.42		7.00	
d	cm	139.00		139.00		100.00		100.00	
d'	cm	10.00		10.00		10.00		10.00	
u	cm	53.85		53.85		41.88		41.88	
b	cm	100.00		100.00		100.00		100.00	
r	A's/As	1.00		1.00		1.00		1.00	
n		15.00		15.00		15.00		15.00	
Sca	kg/cm2	75.00		112.50		70.00		105.00	
Ssa	kg/cm2	1,800.00		2,700.00		1,800.00		2,700.00	
Tca	kg/cm2	4.25		6.38		4.25		6.38	
M'		43.76		50.25		11.12		13.29	
C		33.12	33.12	43.25	43.25	62.93	62.93	79.01	79.01
k (C)		0.000	0.000	0.05	0.004	0.045	0.005	0.04	0.003
np (C)		0.0000	0.0000	0.0051	0.0007	0.0058	0.0003	0.0030	0.0003
S		52.98	52.98	69.20	69.20	107.89	107.89	135.44	135.44
k (S)		0.176	0.000	0.16	-0.018	0.157	-0.080	0.14	-0.089
np (S)		0.0102	0.0102	-0.0078	0.0098	-0.0756	0.0048	-0.0836	0.0050
d'/d		0.07		0.07		0.10		0.10	
f	cm	266.53		306.10		139.04		166.12	
f/d		1.92		2.20		1.39		1.66	
np		0.0102		0.0051		0.0058		0.0030	
r-As		9.47		4.71		3.86		1.98	
r-As'		9.47		4.71		3.86		1.98	
Re-Bar Arrangement									
As1	Diameter	D19		D19		D16		D16	
	Area (cm2)	2.87		2.87		1.99		1.99	
	Pitch (cm)	30.00		30.00		30.00		30.00	
	As1	9.55		9.55		6.62		6.62	
As2	Diameter	0.00		0.00		0.00		0.00	
	Area (cm2)	0.00		0.00		0.00		0.00	
	Pitch (cm)	30.00		30.00		30.00		30.00	
	As2 (cm2)	0.00		0.00		0.00		0.00	
Total of As		9.55	(OK)	9.55	(OK)	6.62	(OK)	6.62	(OK)
		-9.47		-4.71		-3.86		-1.98	
As'1	Diameter	D22		D19		D16		D16	
	Area (cm2)	3.87		2.87		1.99		1.99	
	Pitch (cm)	30.00		30.00		30.00		30.00	
	As'1 (cm2)	12.90		9.55		6.62		6.62	
As'2	Diameter	0.00		0.00		0.00		0.00	
	Area (cm2)	0.00		0.00		0.00		0.00	
	Pitch (cm)	25.00		25.00		25.00		25.00	
	As'2 (cm2)	0.00		0.00		0.00		0.00	
Total of As'		12.90	(OK)	9.55	(OK)	6.62	(OK)	6.62	(OK)
		-9.47		-4.71		-3.86		-1.98	
Check	np	0.01031		0.01031		0.00993		0.00993	
	k	0.31	0.079	0.17	0.000	0.20	-0.003	0.17	-0.006
	A's/As	1.35		1.00		1.00		1.00	
	C	6.67		11.83		10.24		12.10	
	S	14.70		58.83		41.06		60.15	
	Z	0.86	0.18	1.04	0.13	1.02	0.14	1.04	0.13
	Sc	15.12	OK	30.78	OK	11.39	OK	16.08	OK
	Ss	499.46	OK	2,295.35	OK	685.06	OK	1,199.13	OK
	Tc	1.26	OK	1.30	OK	0.64	OK	0.70	OK
	x	43.40		23.28		19.96		16.75	

(f) Downstream Wing Wall of Floodway Weir

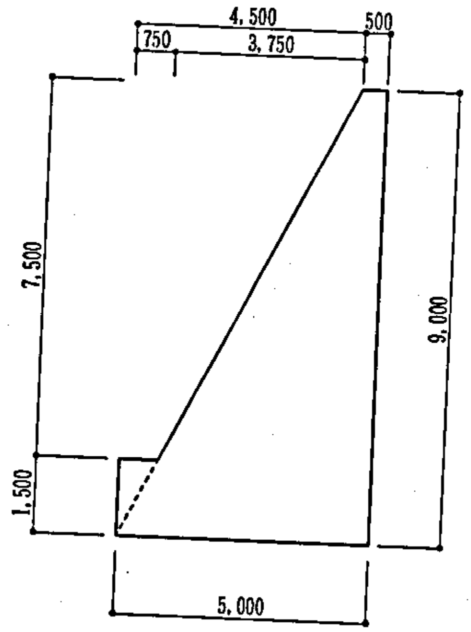


Fig. 2.4.13 DIMENSIONS OF DOWNSTREAM WING WALL (FLOODWAY WEIR)

Table DOWN STREAM WING WALL
(FLOODWAY WEIR)

Design Condition	Normal Condition			
Backfill Material				
Height		hb	0.000	
Slope		n=1:	#######	
Unit Weight		Gs	1.900	t/m3
Internal Friction Angle		fs	30.000	
Water Height	Back Side	Hwb	3.000	m
	Front Side	Hwf	2.000	
Load on Backfill		q	1.000	t/m2
Features of Wall				
Height of Wall		H	6.600	
Slope	Front slope	nf	0.000	
	Height	hf	0.000	
	Back slope	nb	0.500	
Top Width		B	0.500	m
Toe Length		Lt	0.000	m
Toe Height		Ht	1.500	m
Heel Length		Lt	0.750	m
Foundation				
Internal Friction Angle		ff	40.000	
Coheson		Cf	0.000	t/m2
Concrete				
Unit Weight		Gc	2.350	t/m3
Design Strength		σck	160.000	kg/cm2
Coefficient of Earthquake		k	0.000	
Factor of Safty			1.500	
Bearing Strength		qa	65.600	

B/6-e	0.174
Fs	3.005
Qumax	27.353

Load			Vertical	Horizontal	Arm	Moment
Self Weight	Wall	W1	25.592		1.600	40.946
		W2	7.755		0.250	1.939
		W3	0.000		0.000	0.000
	Footing	W4	16.039		2.275	36.488
	Backfill	We1	0.000		0.500	0.000
		We2	0.000		2.525	0.000
		We3	6.156		1.700	10.465
		We4	15.390		3.425	52.711
		We5	2.025		3.300	6.683
		We6	2.025		4.175	8.454
	Water	Ww1	0.000		0.000	0.000
		Ww2	0.000		0.000	0.000
		Ww3	0.000		0.000	0.000
		Ww4	2.250		3.300	7.425
		Ww5	2.250		4.175	9.394
Water Pressure	Front side	Pw1		6.125	1.167	7.146
	Back side	Pw2		-10.125	1.500	-15.188
Seismic Force	Wall	Ke1		0.000	3.700	0.000
		Ke2		0.000	4.800	0.000
		Ke3		0.000	1.500	0.000
	Footing	Ke4		0.000	0.750	0.000
	Backfill	We1		0.000	8.100	0.000
		We2		0.000	8.100	0.000
		We3		0.000	6.900	0.000
		We4		0.000	6.300	0.000
		We5		0.000	3.500	0.000
		We6		0.000	3.000	0.000
Earth Pressure	Horizontal	Phd		-3.632	5.700	-20.702
(dry area)	Vertical	Pvd	2.097		4.550	9.541
Earth Pressure	Horizontal	Phw		-2.924	1.500	-4.386
(submarged 1)	Vertical	Pvw	1.688		4.550	7.682
Earth Pressure	Horizontal	Phw'		-9.080	2.250	-20.429
(submarged 2)	Vertical	Pvw'	5.242		4.550	23.852
Uplift		U1	-15.925		2.275	-36.229
		U2	-2.275		3.033	-6.901
Total			70.309	-19.636		118.890

Stability Calculation

Middle third	B=	4.550 m	
	B/6 =	0.758 m	
	e =	-0.584 m	B/6>e OK
	(B/6-e	0.174)	
Sliding	FS=	3.005	OK
Bearing strength	U1=	27.353 t/m2	OK
	U2=	3.552 t/m2	OK

Table **DOWN STREAM WING WALL**
(FLOODWAY WEIR)

Design Condition	Flood Condition			
Backfill Material				
Height		hb	0.000	
Slope		n=1:	#######	
Unit Weight		Gs	1.900	t/m3
Internal Friction Angle		fs	30.000	
Water Height	Back Side	Hwb	4.200	m
	Front Side	Hwf	3.000	
Load on Backfill		q	1.000	t/m2
Features of Wall				
Height of Wall		H	6.600	
Slope	Front slope	nf	0.000	
	Height	hf	0.000	
	Back slope	nb	0.500	
Top Width		B	0.500	m
Toe Length		Lt	0.000	m
Toe Height		Ht	1.500	m
Heel Length		Lt	0.750	m
Foundation				
Internal Friction Angle		ff	40.000	
Coheson		Cf	0.000	t/m2
Concrete				
Unit Weight		Gc	2.350	t/m3
Design Strength		σck	210.000	kg/cm2
Coefficient of Earthquake		k	0.000	
Factor of Safty			1.500	
Bearing Strength		qa	65.600	

B/6-e	0.013
Fs	2.644
Qumax	28.143

Load			Vertical	Horizontal	Arm	Moment
Self Weight	Wall	W1	25.592		1.600	40.946
		W2	7.755		0.250	1.939
		W3	0.000		0.000	0.000
	Footing	W4	16.039		2.275	36.488
	Backfill	We1	0.000		0.500	0.000
		We2	0.000		2.525	0.000
		We3	2.736		1.300	3.557
		We4	12.996		3.125	40.613
		We5	3.969		3.100	12.304
		We6	2.835		4.175	11.836
	Water	Ww1	0.000		0.000	0.000
		Ww2	0.000		0.000	0.000
		Ww3	0.000		0.000	0.000
		Ww4	4.410		3.100	13.671
		Ww5	3.150		4.175	13.151
Water Pressure	Front side	Pw1		10.125	1.500	15.188
	Back side	Pw2		-16.245	1.900	-30.866
Seismic Force	Wall	Ke1		0.000	3.700	0.000
		Ke2		0.000	4.800	0.000
		Ke3		0.000	1.500	0.000
	Footing	Ke4		0.000	0.750	0.000
	Backfill	We1		0.000	8.100	0.000
		We2		0.000	8.100	0.000
		We3		0.000	7.300	0.000
		We4		0.000	6.900	0.000
		We5		0.000	4.300	0.000
		We6		0.000	3.600	0.000
Earth Pressure	Horizontal	Phd		-1.717	6.500	-11.161
(dry area)	Vertical	Pvd	0.991		4.550	4.511
Earth Pressure	Horizontal	Phw		-4.496	1.900	-8.543
(submarged 1)	Vertical	Pvw	2.596		4.550	11.811
Earth Pressure	Horizontal	Phw'		-8.156	2.850	-23.245
(submarged 2)	Vertical	Pvw'	4.709		4.550	21.426
Uplift		U1	-20.475		2.275	-46.581
		U2	-2.730		3.033	-8.281
Total			64.573	-20.490		98.764

Stability Calculation

Middle third	B=	4.550 m	
	B/6 =	0.758 m	
	e =	-0.745 m	B/6>e OK
	(B/6-e	0.013)	
Sliding	FS=	2.644	OK
Bearing strength	U1=	28.143 t/m2	OK
	U2=	0.240 t/m2	OK

Table DOWN STREAM WING WALL
(FLOODWAY WEIR)

Design Condition	Earthquake Condition			
Backfill Material				
Height		hb	0.000	
Slope		n=1:	#######	
Unit Weight		Gs	1.900	t/m3
Internal Friction Angle		fs	30.000	
Water Height	Back Side	Hwb	2.000	m
	Front Side	Hwf	2.000	
Load on Backfill		q	0.000	t/m2
Features of Wall				
Height of Wall		H	6.600	
Slope	Front slope	nf	0.000	
	Height	hf	0.000	
	Back slope	nb	0.500	
Top Width		B	0.500	m
Toe Length		Lt	0.000	m
Toe Height		Ht	1.500	m
Heel Length		Lt	0.750	m
Foundation				
Internal Friction Angle		ff	40.000	
Coheson		Cf	0.000	t/m2
Concrete				
Unit Weight		Gc	2.350	t/m3
Design Strength		ock	210.000	kg/cm2
Coefficient of Earthquake		k	0.183	
Factor of Safty			1.200	
Bearing Strength		qa	82.700	

B/6-e	0.041
Fs	1.715
Qumax	54.593

Load			Vertical	Horizontal	Arm	Moment
Self Weight	Wall	W1	25.592		1.600	40.946
		W2	7.755		0.250	1.939
		W3	0.000		0.000	0.000
	Footing	W4	16.039		2.275	36.488
	Backfill	We1	0.000		0.500	0.000
		We2	0.000		2.525	0.000
		We3	10.051		2.033	20.437
		We4	15.295		3.675	56.209
		We5	0.900		3.467	3.120
		We6	1.350		4.175	5.636
	Water	Ww1	0.000		0.000	0.000
		Ww2	0.000		0.000	0.000
		Ww3	0.000		0.000	0.000
		Ww4	1.000		3.467	3.467
		Ww5	1.500		4.175	6.263
Water Pressure	Front side	Pw1		0.000	1.167	0.000
	Back side	Pw2		0.000	1.167	0.000
Seismic Force	Wall	Ke1		-4.683	3.700	-17.328
		Ke2		-1.419	4.800	-6.812
		Ke3		0.000	1.500	0.000
	Footing	Ke4		-2.935	0.750	-2.201
	Backfill	We1		0.000	8.100	0.000
		We2		0.000	8.100	0.000
		We3		-1.839	6.567	-12.078
		We4		-2.799	5.800	-16.234
		We5		-0.165	2.833	-0.467
		We6		-0.247	2.500	-0.618
Earth Pressure	Horizontal	Phd		-27.176	2.700	-73.374
(dry area)	Vertical	Pvd	4.835		4.550	21.998
Earth Pressure	Horizontal	Phw		0.000	1.167	0.000
(submarged 1)	Vertical	Pvw	0.000		4.550	0.000
Earth Pressure	Horizontal	Phw'		0.000	1.750	0.000
(submarged 2)	Vertical	Pvw'	0.000		4.550	0.000
Uplift		U1	0.000		2.275	0.000
		U2	0.000		3.033	0.000
Total			84.316	-41.263		67.390

Stability Calculation

Middle third	B=	4.550 m	
	B/3 =	1.517 m	
	e =	-1.476 m	B/3>e OK
	(B/6-e	0.041)	
Sliding	FS=	1.715	OK
Bearing strength	U1=	54.593 t/m2	OK
	U2=	-17.531 t/m2	OK