APPENDIX B STABILITY ANALYSIS

APPENDIX B.3.1-1 Stability Analysis of Canal Banks

1. Slope of Concrete Lined Section

Data:

1) Friction Angle $\phi = 32^\circ$

The careful trimming should be carried out for slope behind the concrete lining by using a slope trimming machine.

2) Unit Weight (wetted) of Fill $\gamma t = 1.8\ t/m^3$

Results

Slope	Safety Factor	Remarks
1 : 2.0	1.254>1.2	Refer to Figure 1 and Table 1.
1 : 1.5	0.941<1.2	Refer to Figure 2 and Table 2

2. Slope other than Concrete Lined Section

Data:

1) Friction Angle $\phi = 32^\circ$

The outside portion of the slope will be disturbed by trimming and/or excavating operations by heavy machinery and its N values are estimated at 4-10.

The friction angle can be estimated by using Dunham's formula:

$\phi = 15 + (12 \times N)^{1/2} = 22^\circ - 26^\circ = 24^\circ$ (Ave.)

2) Unit Weight (wetted) of Fill $\gamma t = 1.8\ t/m^3$

Results

Slope	Safety Factor	Remarks
1 : 3.0	1.339>1.2	Refer to Figure 3 and Table 3.
1 : 2.5	1.116<1.2	Refer to Figure 4 and Table 4.

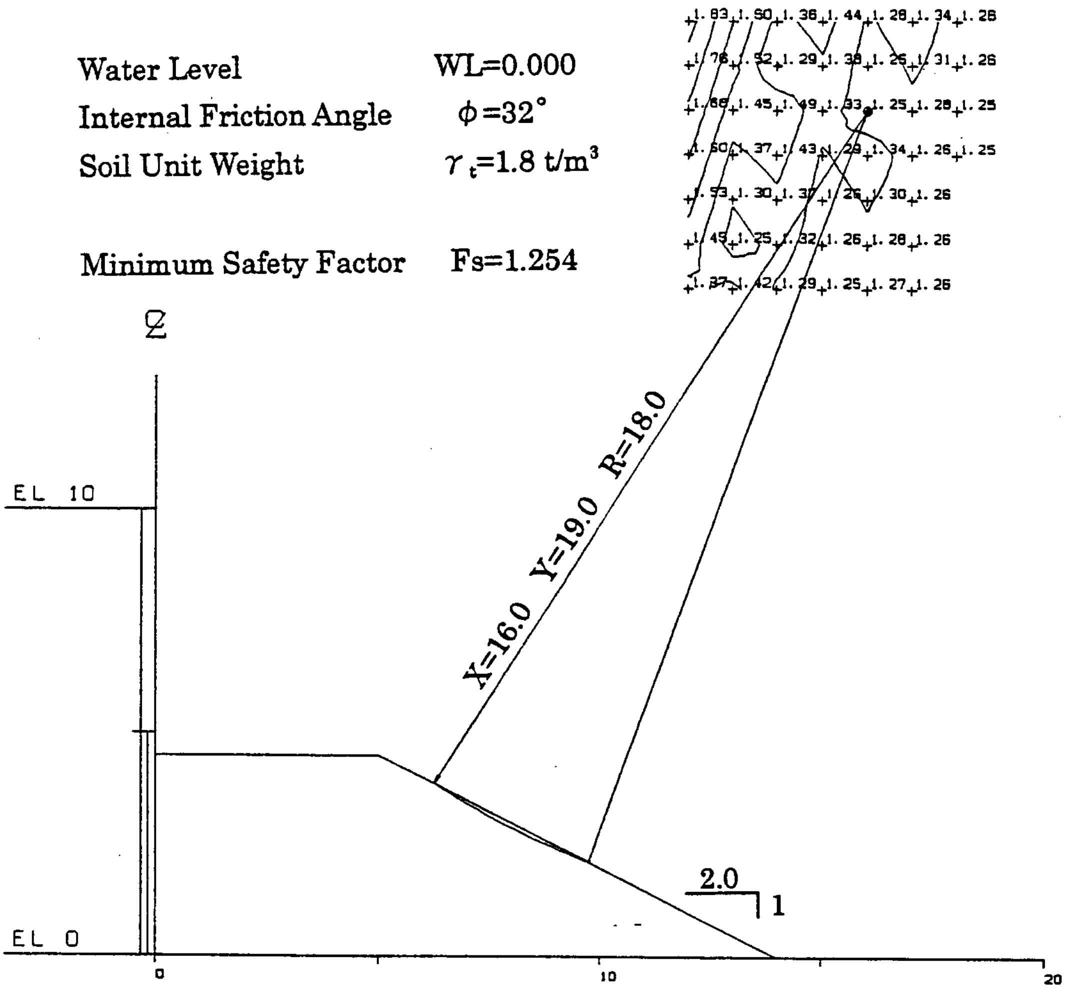


Figure 1 Stability Analysis of Lining Slope of Conveyance Canal

Table 1 Stability Analysis of Lining Slope of Conveyance Canal

X (M)	Y (M)	R (M)	CL (TON/M)	NTAN (TON/M)	T (TON/M)	F.S.
16.00	19.00	19.00	0.0	8.700	6.297	1.382
		18.00	0.0	0.279	0.222	1.254
16.00	20.00	20.00	0.0	9.826	6.894	1.425
		19.00	0.0	0.804	0.640	1.257
16.00	21.00	21.00	0.0	10.939	7.441	1.470
		20.00	0.0	1.431	1.110	1.288

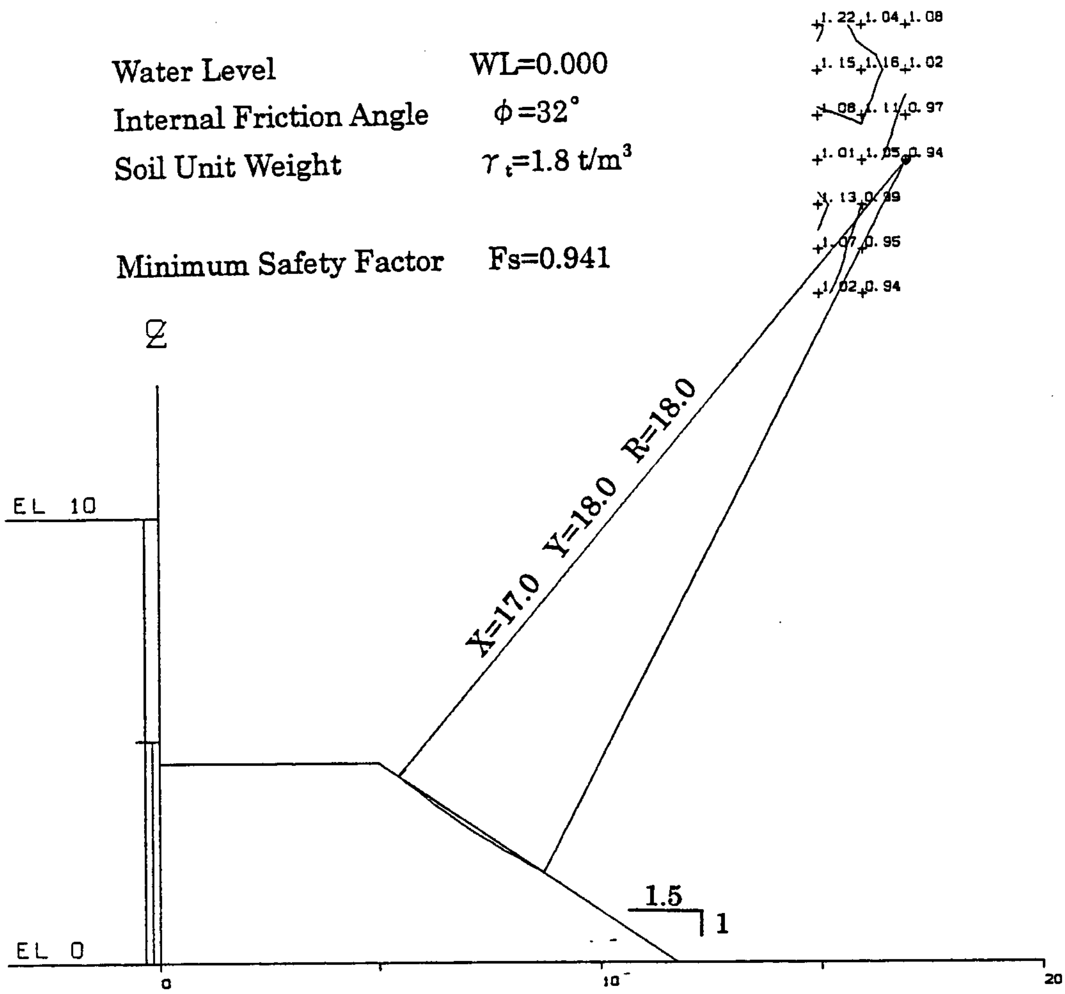


Figure 2 Stability Analysis of Lining Slope of Conveyance Canal

Table 2 Stability Analysis of Lining Slope of Conveyance Canal

X (M)	Y (M)	R (M)	CL (TON/M)	NTAN (TON/M)	T (TON/M)	F.S.
16.00	20.00	20.00	0.0	5.307	4.549	1.167
16.00	21.00	21.00	0.0	6.334	5.188	1.221
		20.00	0.0	0.271	0.260	1.046
17.00	18.00	18.00	0.0	0.256	0.272	0.941
17.00	19.00	19.00	0.0	0.998	1.024	0.975
17.00	20.00	20.00	0.0	1.832	1.779	1.030
17.00	21.00	21.00	0.0	2.715	2.501	1.085

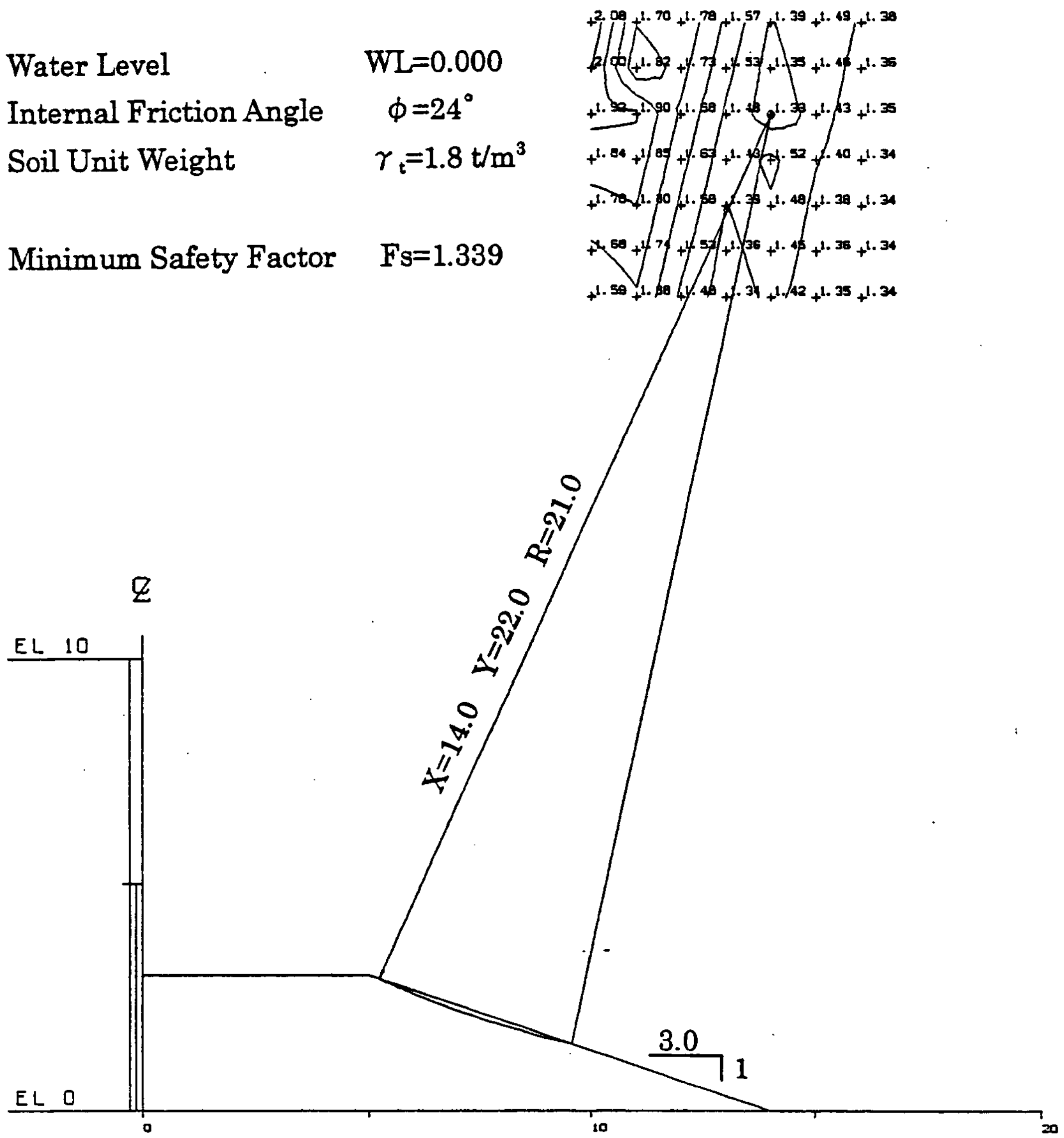


Figure 3 Stability Analysis of Embanked Slope of Conveyance Canal

Table 3 Stability Analysis of Embanked Slope of Conveyance Canal

X (M)	Y (M)	R (M)	CL (TON/M)	NTAN (TON/M)	T (TON/M)	F.S.
14.00	22.00	22.00	0.0	6.838	4.386	1.559
		21.00	0.0	0.286	0.214	1.339
14.00	23.00	23.00	0.0	7.256	4.547	1.596
		22.00	0.0	0.476	0.350	1.358
14.00	24.00	24.00	0.0	7.665	4.693	1.633
		23.00	0.0	0.676	0.487	1.390

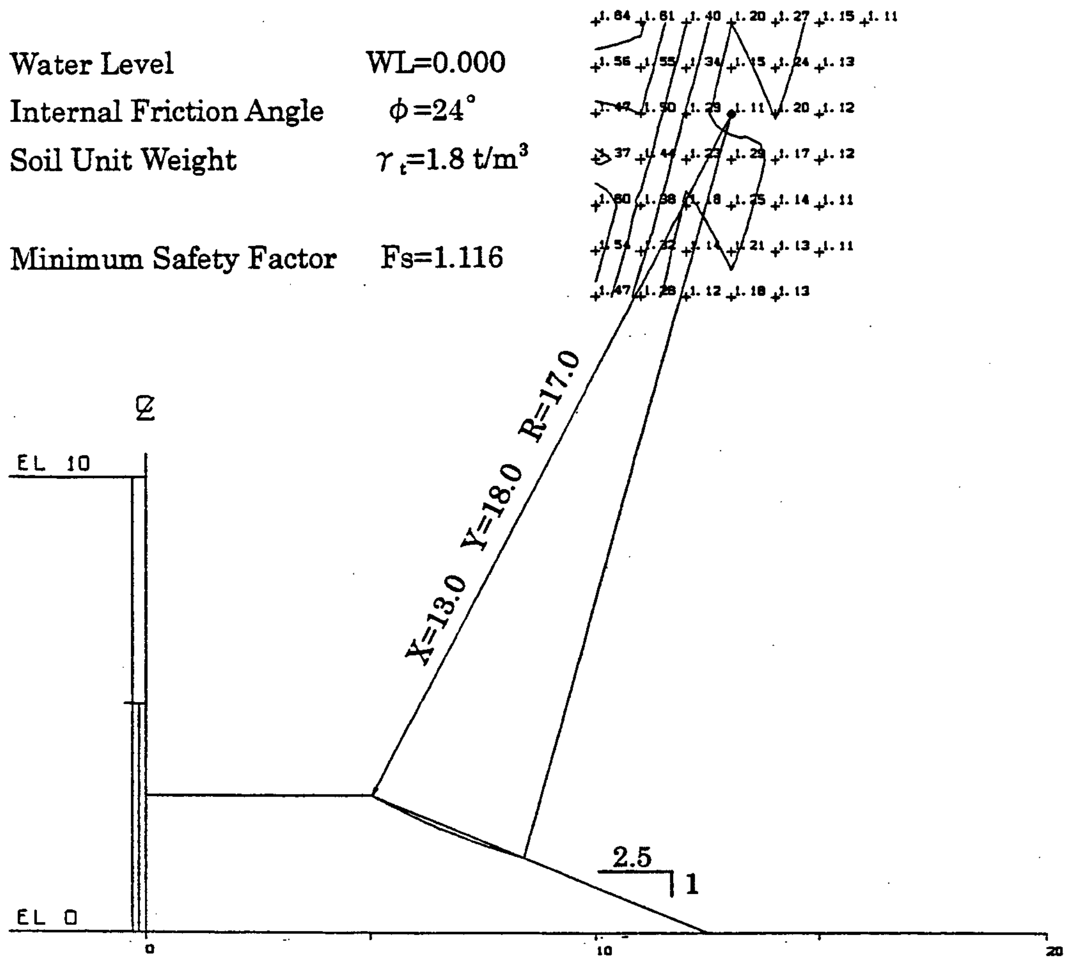
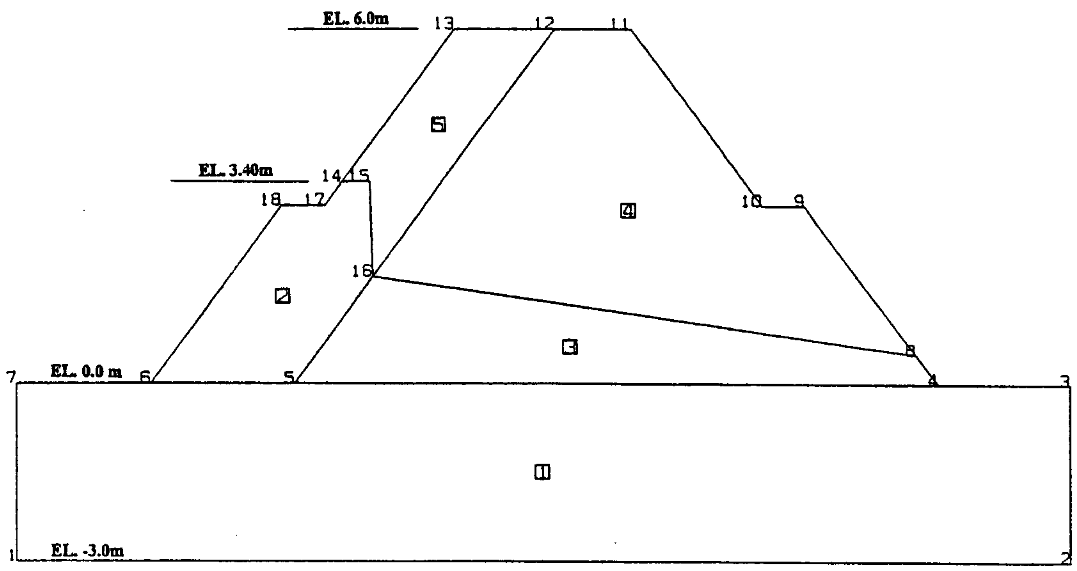


Figure 4 Stability Analysis of Embanked Slope of Conveyance Canal

Table 4 Stability Analysis of Embanked Slope of Conveyance Canal

X (M)	Y (M)	R (M)	CL (TON/M)	NTAN (TON/M)	T (TON/M)	F.S.
13.00	18.00	18.00	0.0	5.622	4.207	1.336
		17.00	0.0	0.173	0.155	1.116
13.00	19.00	19.00	0.0	6.078	4.406	1.380
		18.00	0.0	0.369	0.319	1.156
13.00	20.00	20.00	0.0	6.531	4.591	1.423
		19.00	0.0	0.586	0.488	1.201
14.00	14.00	14.00	0.0	1.510	1.337	1.130
14.00	15.00	15.00	0.0	1.961	1.733	1.131

APPENDIX B.3.2-1 Stability Analysis of Dike



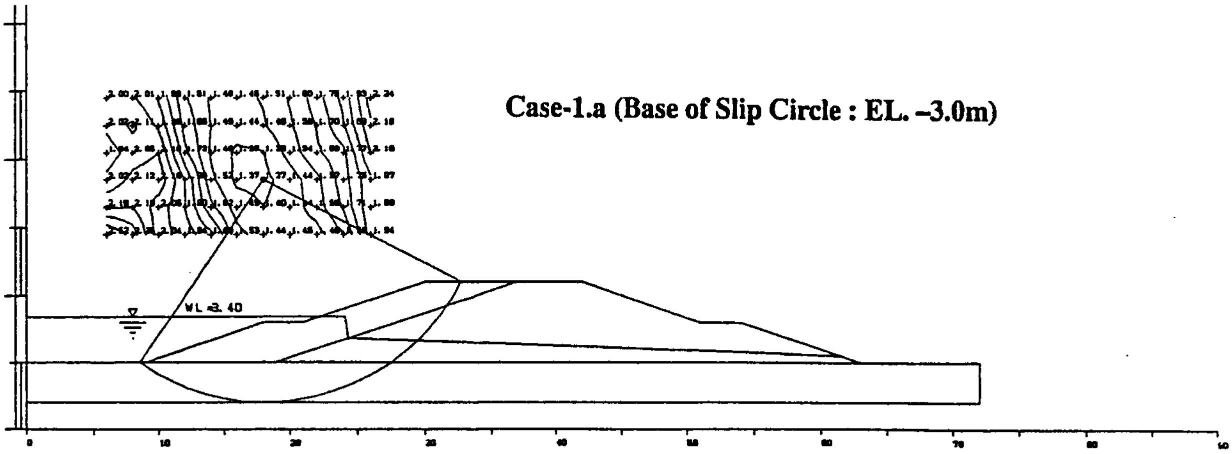
ZONING OF DIKE

DESIGN VALUES FOR STABILITY ANALYSIS

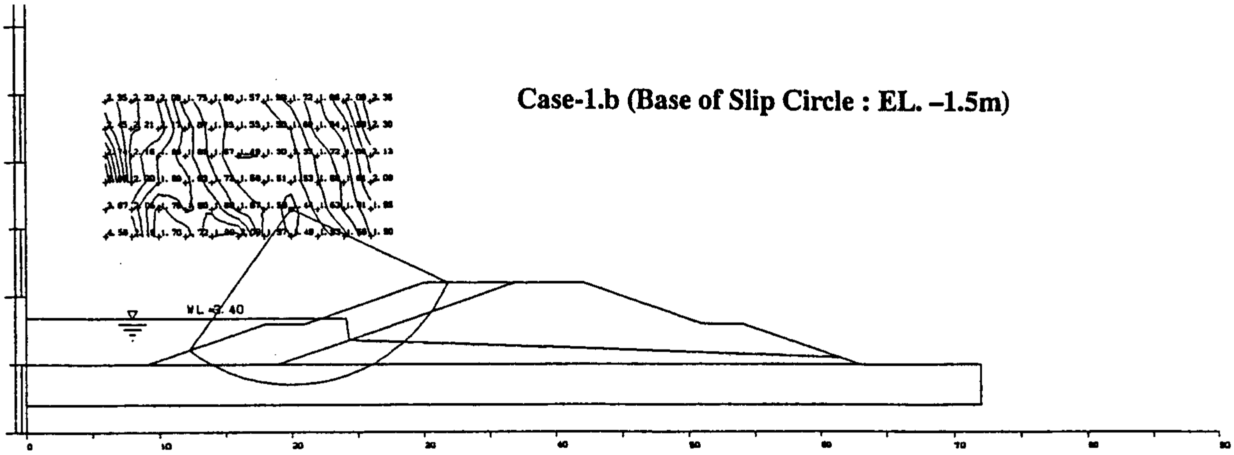
Zone	Case-1 (W.L. EL.3.40m)			Case-2 (W.L. EL.0.0m)		
	C	ϕ	γ	C	ϕ	γ
1	0	22°	0.8t/m^3	0	22°	0.8t/m^3
2	0	35°	1.0t/m^3	0	32°	1.8t/m^3
3	0	32°	0.8t/m^3	0	32°	1.8t/m^3
4	0	32°	1.8t/m^3	0	32°	1.8t/m^3
5	0	35°	2.0t/m^3	0	32°	1.8t/m^3

RESULT OF CASE-1 (W.L. EL. 3.40m)

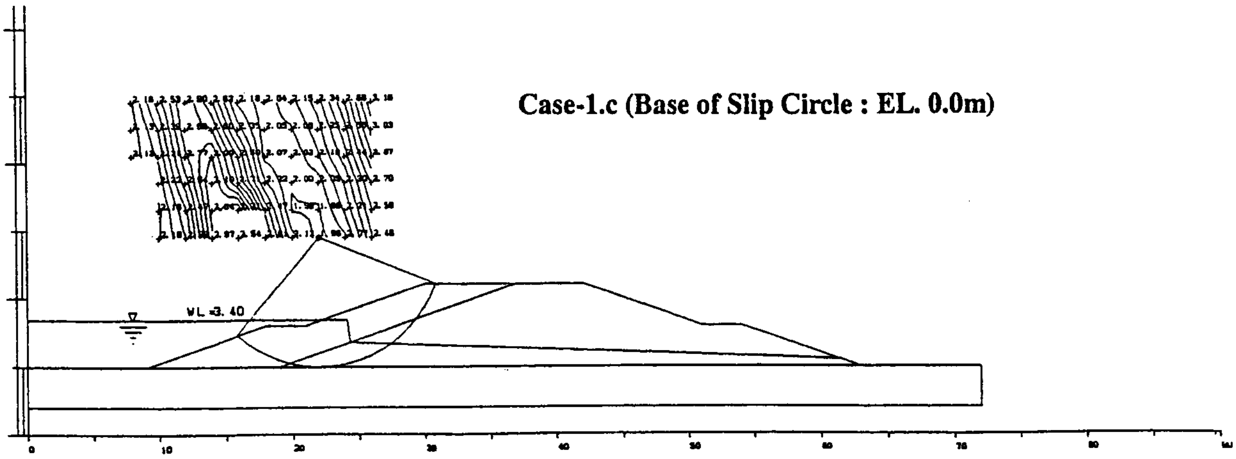
CASE	Base of Slip Circle	Safety Factor (calculated)
Case-1.a	EL. -3.0 m	1.37 < 1.5
Case-1.b	EL. -1.5 m	1.45 < 1.5
Case-1.c	EL. 0.0 m	1.96



Case-1.a (Base of Slip Circle : EL. -3.0m)



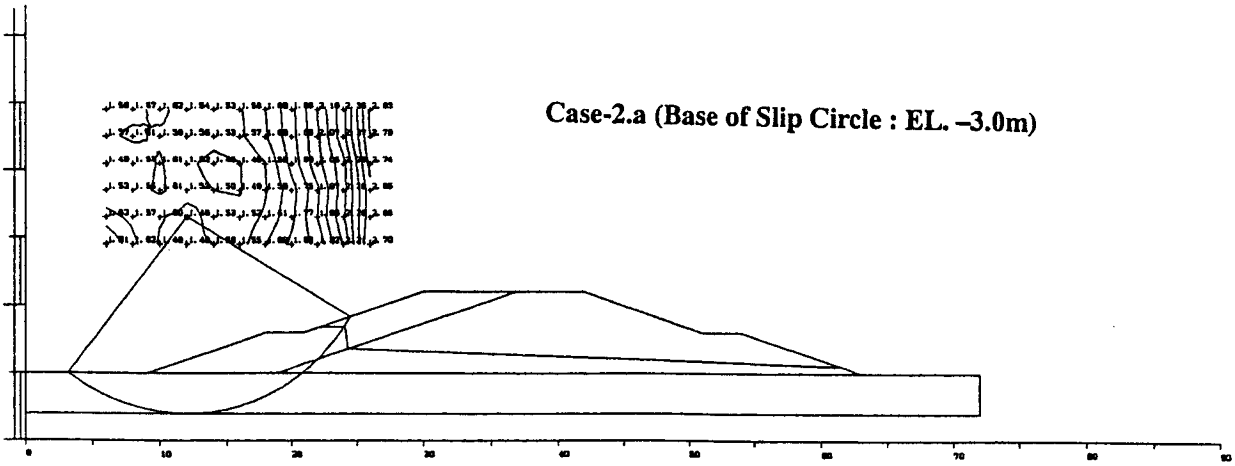
Case-1.b (Base of Slip Circle : EL. -1.5m)



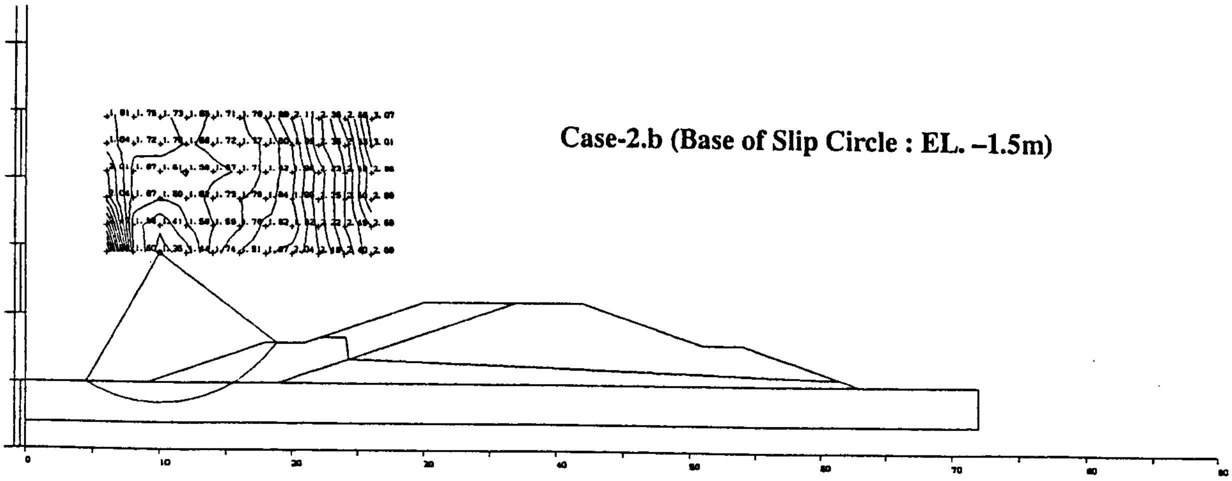
Case-1.c (Base of Slip Circle : EL. 0.0m)

RESULT OF CASE-2 (W.L. EL. 0.0m)

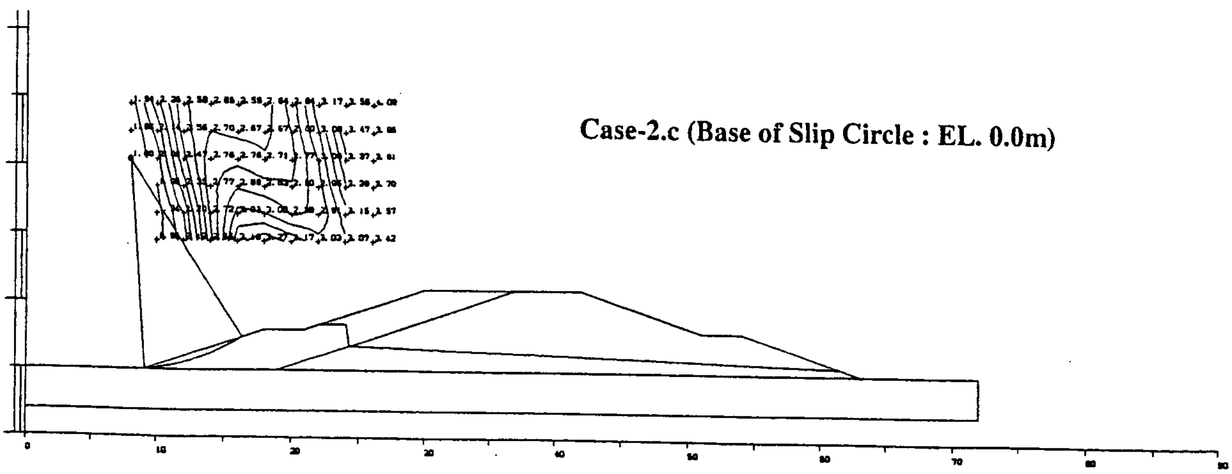
CASE	Base of Slip Circle	Safety Factor (calculated)
Case-2.a	EL. –3.0 m	1.46 < 1.5
Case-2.b	EL. –1.5 m	1.39 < 1.5
Case-2.c	EL. 0.0 m	1.90



Case-2.a (Base of Slip Circle : EL. –3.0m)



Case-2.b (Base of Slip Circle : EL. –1.5m)



Case-2.c (Base of Slip Circle : EL. 0.0m)

APPENDIX B.3.3-1 Detailed Stability Analysis for Slope of Excavation of No.7 Pumping Station

The results of detailed stability analysis for slope of excavation of No.7 Pumping Station are shown in Table B.3.3-1 ~ B.3.3-4.

Table B.3.3-1 Case 1 : Slope Stability Analysis of Excavation of No.7 P.S.

Coordination		Radius	N tan ϕ	T	Safety Factor
X (m)	Y (m)	R (m)	(tf/m)	(tf/m)	Fs
14.00	15.00	14.00	2.988	3.094	0.966
15.00	14.00	14.00	5.402	5.748	0.939
15.00	**15.00**	**14.00**	**0.275**	**0.316**	**0.871**
15.00	16.00	15.00	0.964	1.080	0.892
16.00	15.00	15.00	2.684	3.008	0.892

Table B.3.3-2 Case 2 : Slope Stability Analysis of Excavation of No.7 P.S.

Coordination		Radius	N tan ϕ	T	Safety Factor
X (m)	Y (m)	R (m)	(tf/m)	(tf/m)	Fs
14.00	20.00	18.00	2.437	1.865	1.307
15.00	19.00	18.00	6.537	5.035	1.298
15.00	**20.00**	**18.00**	**0.275**	**0.222**	**1.158**
15.00	21.00	19.00	0.679	0.568	1.195
16.00	20.00	19.00	3.814	3.124	1.221

Table B.3.3-3 Case 3 : Slope Stability Analysis of Excavation of No.7 P.S.

Coordination		Radius	N tan ϕ	T	Safety Factor
X (m)	Y (m)	R (m)	(tf/m)	(tf/m)	Fs
9.00	5.00	5.00	2.729	2.623	1.039
9.00	6.00	6.00	4.296	3.534	1.216
10.00	**6.00**	**6.00**	**0.905**	**0.991**	**0.913**
10.00	7.00	7.00	2.103	1.965	1.069
11.00	8.00	8.00	0.451	0.484	0.933

Table B.3.3-4 Case 4 : Slope Stability Analysis of Excavation of No.7 P.S.

Coordination		Radius	N tan ϕ	T	Safety Factor
X (m)	Y (m)	R (m)	(tf/m)	(tf/m)	Fs
11.00	7.00	7.00	1.210	0.933	1.296
11.00	8.00	8.00	2.313	1.732	1.336
12.00	**9.00**	**9.00**	**0.286**	**0.225**	**1.269**
12.00	10.00	10.00	1.107	0.851	1.301
12.00	11.00	11.00	2.077	1.492	1.393

APPENDIX C STRUCTURAL DESIGN

APPENDIX C.4.2-1 Selection of Type of Conveyance Structure over Depressions in the route of the Box Culvert Conduit

To secure safety against drift sand, the box culvert conduit is planned in the section between KM 94.3 and KM 101.8 where drift sand dunes are prevailing. However, the section between KM 97.251 and KM 97.509 is a depression and requests a high embankment of approx. 7 m high for the box culvert conduit that may request a comparatively high construction cost. Therefore the comparative study for selecting economic type of conveyance structure over this depression is carried out. The alternatives are :

- Plan A : Box Culvert,
- Plan B : Siphon and
- Plan C : Elevated Flume.

The estimated construction costs are shown in the table below. The plan A (Box Culvert) is selected for this conveyance canal system because of the cheapest construction cost among three alternatives. In addition, the plan A is advantage for easy construction and more safety against leakage as compared with the plan B (Siphon).

Table 1 Construction Costs of Three Alternatives

			Plan A : Box Culvert		Plan B : Siphon		Plan C : Elevated Flume	
Pay Item	Unit	Unit Price	Quantity	Amount	Quantity	Amount	Quantity	Amount
Excavation	m3	2	5,200	10,400	26,700	53,400	11,300	22,600
Fill and Backfill	m3	3	149,000	447,000	77,200	231,600	115,100	345,300
Stone Pitching	m3	104	5,500	572,000	3,100	322,400	5,100	530,400
Laterite Pavement	m2	25	13,800	345,000	10,100	252,500	5,800	145,000
Gravel Bedding	m3	43	5,600	240,800	5,800	249,400	5,400	232,200
Reinforced Concrete	m3	575	6,240	3,588,000	7,330	4,214,750	8,560	4,922,000
Plain Concrete	m3	184	920	169,280	960	176,640	1,060	195,040
Asphalt Paved Road	m	700	258	180,600	258	180,600	258	180,600
Water Stop	m	43	890	38,270	1,080	46,440	690	29,670
Stop-Log, Screen, etc	L.S.				100%	50,000		
Total				5,591,350		5,777,730		6,602,810
Order				1		2		3

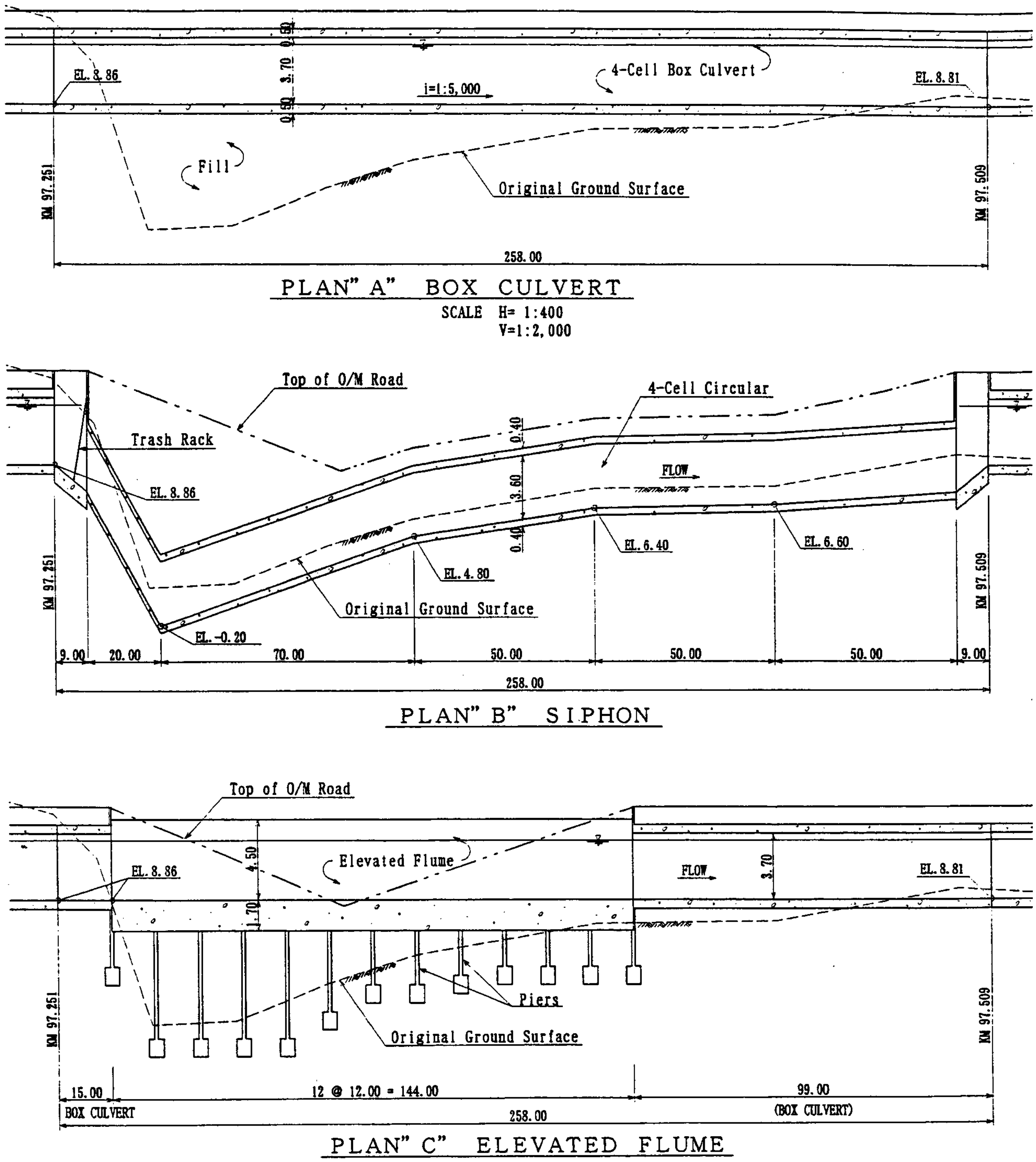
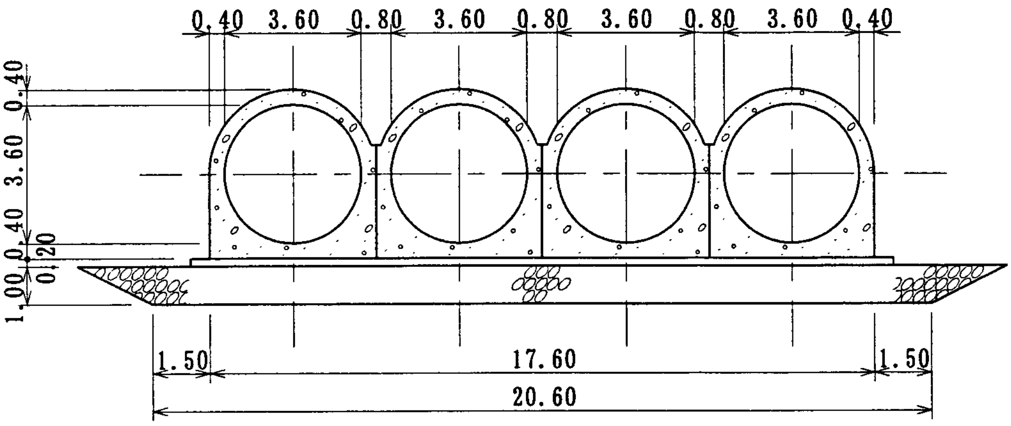
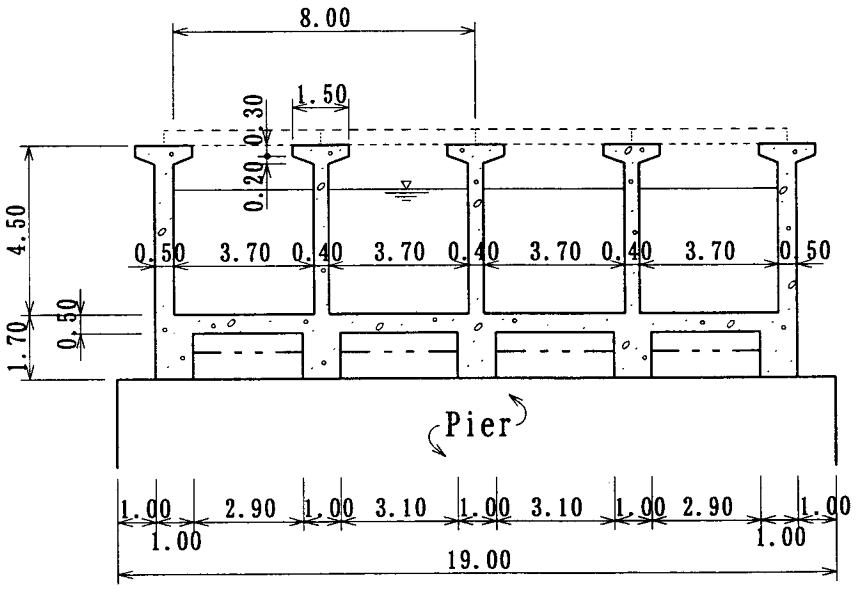


Figure 1 Profile of Three Alternative Plans



SIPHON

SCALE 1:200



ELEVATED FLUME

SCALE 1:200

Figure 2 Typical Section of Siphon and Elevated Flume

APPENDIX C.4.2-2 Structural Computation of Road Bridges

1. Design of Cast-in-place concrete beam span (T-beam type), Bridge No.1

1.1 Design Conditions

1.1.1 General Conditions

(1) Type: Cast-in-place concrete beam span (T-beam type)

(2) Total bridge length: 31.500m (3 spans)

(3) Beam length: 10.480m

(4) Span length: 9.68m

(5) Bridge width: 2sides×10.000m (side-walk)+14.000m(roadway)=14.000m(total width)

(6) Skew: Right angle 90°

(7) Live load: T.L.-70

(8) Impact coefficient: Slab i =20/(50+l)

Main beam i =0.4-0.008l

(9) Sectional slope: Roadway 2.0% slope

Side-walk 0.0% (level)

(10) Longitudinal slope: Level

(11) Pavement: Concrete pavement for roadway

Concrete pavement for side-walk

1.1.2 Construction material and allowable stresses

(1) Reinforced concrete

Material strength and allowable stress of reinforced concrete are shown in Table 1.

Table 1 Material Strength and Allowable Stress of Reinforced Concrete

Item		Allowable stress
Design concrete strength at 28 days (σ_{ck})		275 kg/cm^2
Bending compressive stress (σ_{ca})		85 kg/cm^2
Shear stress	Beams (τ_{a1})	8 kg/cm^2
	Slabs (τ_{a2})	10 kg/cm^2
Bond stress of deformed bar (τ_{oa})		16 kg/cm^2
Bearing stress		70 kg/cm^2

(2) Reinforcing bar

Material of reinforcing bar is Steel/52, and allowable tensile stresses are shown as follows.

for slabs σ_{sa}=2000 kg/cm^2

for beams σ_{sa}=1800 kg/cm^2

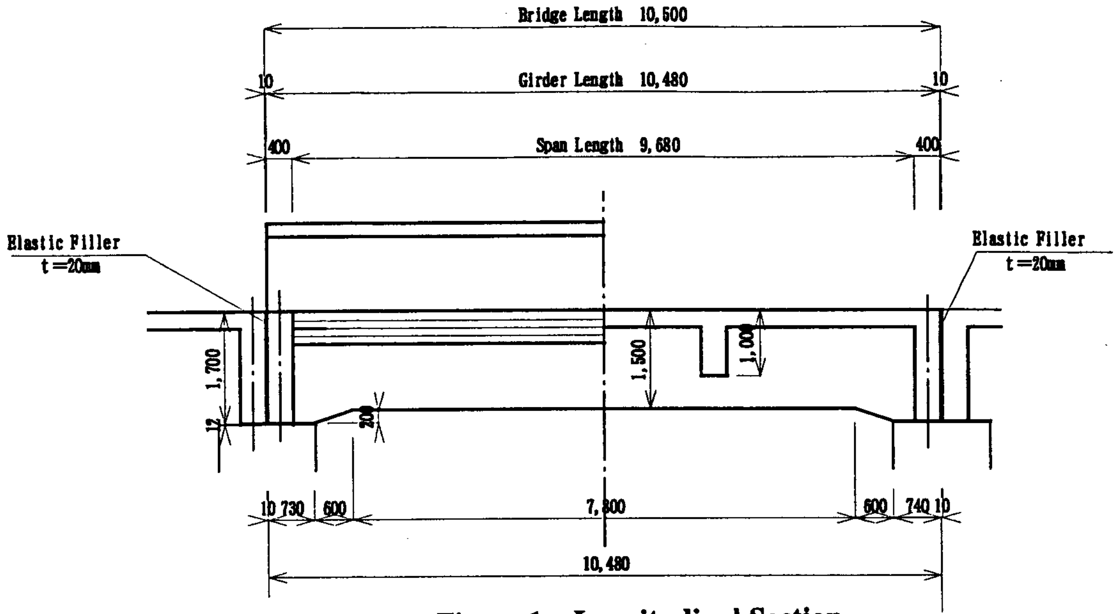


Figure 1 Longitudinal Section

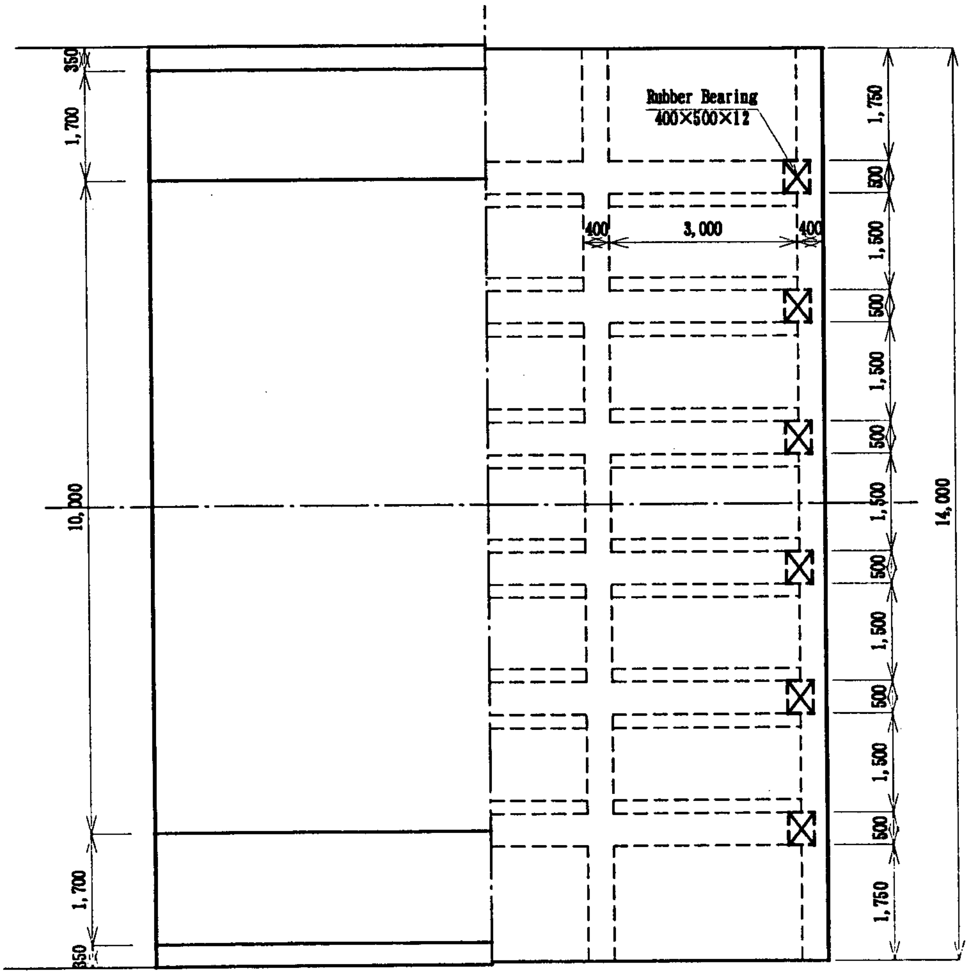


Figure 2 Plan

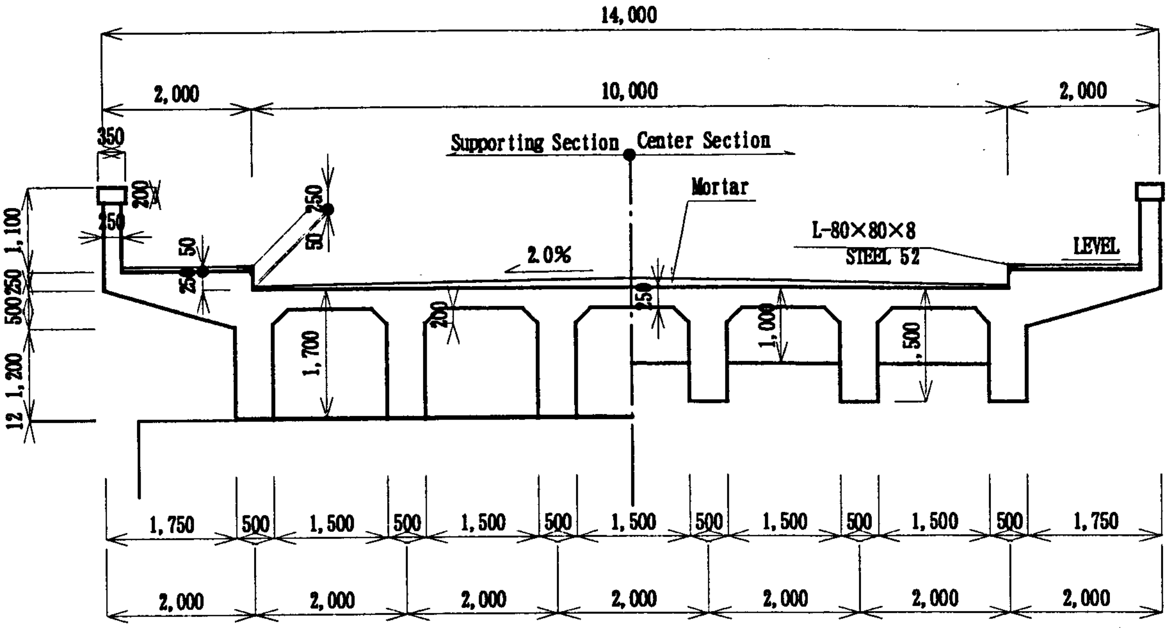


Figure 3 Center Section

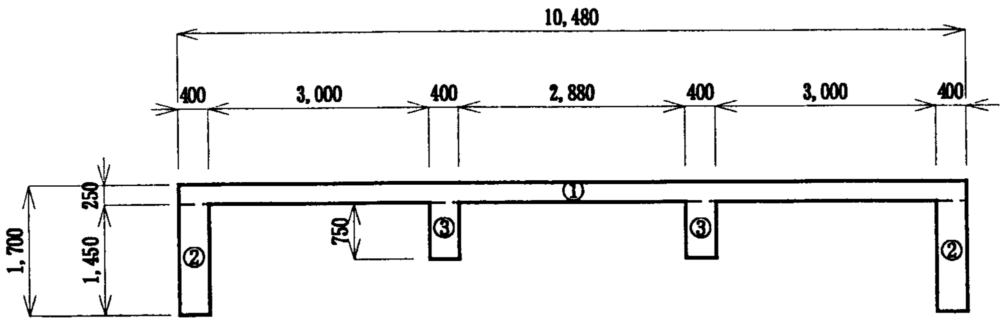


Figure 4 Partition of Cross Beam Section

1.2 Design of superstructure

1.2.1 Dimension of slab

The dimensions of the superstructure are shown in Figures 1 to 4.

1.2.2 Design of slab

(1) Loads

(a) Dead load

Pavement (7.5cm in mean thickness) $0.075 \times 2300 = 173$ kg/cm^2

Slab (25cm in thickness) $0.25 \times 2450 = 613$ kg/ cm^2

Total $\omega_d = 786$ kg/ cm^2

(b) Live load by vehicle

$P = 11700$ kg (T-70)

(c) Impact coefficient

$i = 0.4 - 0.008 \cdot L_1$

(2) Bending moment

(a) Bending moment due to dead load

① Bending moment at the beam center

$$M_{d1} = 1/10 \times 786 \times 2.00^2 = 314 \text{ kg}\cdot\text{m/m}$$

② Bending moment at supporting portion

In case of more than 3 spans

$$M_{d2} = -1/10 \times 786 \times 2.00^2 = -314 \text{ kg}\cdot\text{m/m}$$

(b) Bending moment due to live load

Impact coefficient Japanese standard; $i_1 = 20/(50 + 9.68) = 0.335$

Egyptian standard; $i_2 = 0.4 - 0.008 \times 9.68 = 0.323 \fallingdotseq i1$

Bending moment at beam center

$L \leqq 6.0$m

$$M_{l1} = (0.1 + 0.075 \cdot l) \times P \quad \text{(including impact)}$$

$$= (0.1 + 0.075 \times 2.00) \times 11700 = 2925 \text{ kg}\cdot\text{m/m}$$

① Bending moment at supporting portion

$L \leqq 6.0$m

$$M_{l2} = -(0.125 + 0.15 \cdot l) \times P$$

$$= -(0.125 + 0.15 \times 2.00) \times 11700 = -4973 \text{ kg}\cdot\text{m/m}$$

(c) Total bending moment

Span center $\Sigma M_1 = 314 + 2925 = 3239$ kg·m/m

Supporting portion $\Sigma M_2 = -(314 + 4973) = -5287$ kg·m/m

(3) Section and stress

(a) Span center

① Stress estimated at section

$\sigma_{ca} = 85$kg/cm^2, $\sigma_{sa} = 2000$kg/cm^2 $\rightarrow$ $C_1 = 0.264$

$d = 0.264 \times \sqrt{(323900/100)} = 15.0$ cm

Therefore, the section is estimated as effective height of 19cm, covering of 6cm and slab thickness of 25cm.

σ_{ca}= 85kg/cm^2, σ_{sa}=2000kg/cm^2 → j=0.870

Required reinforcing bar area is as follows.

As'=373900/(2000×0.870×9)=11.31 cm^2/m

In case of using number of 8 and D16 of diameter of reinforcing bar, reinforcing bar area is as follows.

As=8×2.01=16.08>11.31 cm^2

② Checking of stress in section

Section is checked as rectangular section in simple reinforcing bar arrangement by (1.9) formula

p=16.08×(100×19)=0.0085 → k=0.393, j=0.869

σ_c=2×323900/(0.393×0.869×100×19^2)= 52.5kg/cm^2< σ_{ca}=85 kg/ cm^2

σ_s=323900/(16.08×0.869×19)= 1220kg/ cm^2< σ_{sa}=2000kg/ cm^2

(b) Supporting portion

① Stress estimated at section

d=0.264×√(528700/100)=19.2 cm

Hunch of the supporting portion shall be added in the section.

Therefore, slab thickness including hunch is 31.67 (=25+20/3)cm. In case of effective height of 25.67cm and covering of 6cm, required reinforcing bar area is estimated as follows.

As=528700/(2000×0.870×25.67)=11.8 cm^2/m

Required reinforcing bar interval is the same as the span center.

② Checking of stress in section

p=16.08/(100×25.67)=0.0063 → k=0.350, j=0.883

σ_c=2×528700/(0.350×0.883×100×25.67^2)= 51.9kg/ cm^2< σ_{ca}=85 kg/ cm^2

σ_s=528700/(16.08×0.883×25.67)= 1451kg/ cm^2< σ_{sa}=2000kg/ cm^2

(4) Reinforcing bar arrangement at the axis direction of bridge

(a) Design bending moment

M_{l1}= α(0.1+0.075·l)×P (l≦6.0m)

α=0.66+0.04·l (l≦6.0m)

=0.66+0.04×2.0=0.74

∴ M_{l1}=0.74×(0.1+0.075×2.0)×11700=2165 kg·m/m

(b) Distribution reinforcing bar

Effective height

d=19−1/2×(1.6+1.6)=17.4 cm

Required reinforcing bar area

As'=216500/(2000×7/8×17.4)=7.11cm^2/m

In case of using number of 4 and D16 of diameter of reinforcing bar, reinforcing bar area is as follows.

As=4×2.01=8.04>7.11 cm^2/m

1.2.3 Design of slab by cantilever type at side-walk

(1) Dimension of slab

The section is checked at the end of the slab.

(3) Loads

(a) Dead load

Slab by cantilever type	$1/2\times(0.25+0.75)\times1.75\times2450$	$=2144$ kg/m
Pavement (5cm in thickness)	$0.05\times1.75\times2300$	$=201$ kg/m
Hand rail	$(0.20\times0.35+0.90\times0.25)\times2450$	$=723$ kg/m
Total	ω_d	$=3068$ kg/cm2

(b) Live load

A uniform distributed load on side-walk

$\omega=300\times1.75=525$ kg/m

(3) Bending moment at section E～E

(a) Bending moment due to dead load

Bending moment of the slab by cantilever

$$2144\times1.75/3\times(0.75+2\times0.25)/(0.25+0.75)=1563 \text{ kg·m/m}$$

Pavement	$201\times1/2\times1.75=$	176 kg·m/m
Hand rail	$723\times(1.75-1/2\times0.25)=$	1175 kg·m/m
Total		2914 kg·m/m

(b) Bending moment due to live load

$M_\omega=525\times(-1.75/2)=-459$ kg·m/m

(c) Total bending moment

$\Sigma M=-(2914+459)=-3373$ kg·m/m >-5287 kg·m/m

Total bending moment of 3373 kg·m/m is smaller than that at the supporting potion in the middle of the slab.

Therefore, the section of the slab by cantilever type is the same as that in slab of the superstructure.

1.3 Design of main beam

1.3.1 Estimation of coefficient of load distribution

(1) Grid parameter

(a) Dimensions of main beam

① Radius of gyration

No.	Dimension (cm)	A (cm^2)	y (cm)	Ay (cm^3)	Ay^2 (cm^4)	Io (cm^4)
1	1400×25	35000	12.5	437500	5468750	1822916
2	6@50×145	43500	97.5	4241250	413521875	76215625
3	10@20×20	2000	31.7	63400	2009780	44444
Total		80500		4742150	421000405	78082985

$\delta = 4742150/80500 = 58.91$ cm

$I_{.} = 421000405 + 78082985 - 80500 \times 58.91^2 = 219717148$ cm^4

② Torsional moment of inertia

$\beta = 1/3 - 64/\pi^5 \times b/a$

No.	Dimension (cm)	a/b (a>b)	β	$J = \beta \times a \times b^3$ (cm^4)
1	1400×25	1400/25=56	0.330	7218750
2	6@50×145	145/50=2.9	0.261	28383750
Total				$J_{.}$ 35602500

(b) Dimensions of cross beam

① Radius of gyration

No.	Dimension (cm)	A (cm^2)	y (cm)	Ay (cm^3)	Ay^2 (cm^4)	Io (cm^4)
1	1047×25	26175	12.5	327187	4089843	1363281
2	2@40×145	11600	97.5	1131000	110272500	20324166
3	2@40×75	6000	62.5	375000	23437500	2812500
Total		43775		1833187	137799843	24499947

$\delta = 1833187/43775 = 41.88$ cm

$I_t = 137799843 + 24499947 - 43775 \times 41.88^2 = 85521311$ cm^4

② Torsional moment of inertia

$\beta = 1/3 - 64/\pi^5 \times b/a$

No.	Dimension (cm)	a/b (a>b)	β	$J = \beta \times a \times b^3$ (cm^4)
1	1047×25	1047/25=41.9	0.328	5365875
2	2@40×145	145/40=3.6	0.275	5104000
3	2@40× 75	75/40=1.9	0.223	2140800
Total				J_t 12610675

(c) Bending stiffness modulus

$\theta = b/l \times \sqrt[4]{(E\,. \cdot I\,. / E_t \cdot I_t)}$

$b = 14.00/2 = 7.00$ m

• $= 9.67$m

$E\,. = E_t = 2.67 \times 10^5 cm^4$

$I\,. = 219717148/1400 = 156941\ cm^4/cm$

$I_t = 85521311/1047 = 81682\ cm^4/cm$

$\theta = 7.00/9.68 \times \sqrt[4]{(156941/81682)} = 0.851 \fallingdotseq 0.85$

(d) Torsional stiffness modulus

$\alpha = (G\,. \cdot J\,. + G_t \cdot J_t)/2\sqrt{(E\,. \cdot I\,. \times E_t \cdot I_t)}$

$G\,. = G_t = 0.43 \times E\,.$

$J\,. = 35602500/1400 = 25430\ cm^4/m$

$J_t = 12610675/1047 = 12045\ cm^4/m$

$\alpha = 0.43 \cdot E\,. \times (25430 + 12045)/(2 \times E\,. \times \sqrt{(156941 \times 81682)}) = 0.07116$

$\sqrt{\alpha} = \sqrt{0.07116} = 0.267 \fallingdotseq 0.27$

K0

		-b	-3/4b	-1/2b	-1/4b	0	1/4b	1/2b	3/4b	b	TOTAL
0	θ=0.85	-0.370	0.330	1.050	1.700	2.050	1.700	1.050	0.330	-0.370	
	θ=0.85	-0.370	0.330	1.050	1.700	2.050	1.700	1.050	0.330	-0.370	
	θ=0.85	-0.370	0.330	1.050	1.700	2.050	1.700	1.050	0.330	-0.370	
1/4b	θ=0.85	-0.550	-0.020	0.510	1.110		2.030	1.670	1.020	0.250	
	θ=0.85	-0.550	-0.020	0.510	1.110		2.030	1.670	1.020	0.250	
	θ=0.85	-0.550	-0.020	0.510	1.110		2.030	1.670	1.020	0.250	
1/2b	θ=0.85	-0.430	-0.180	0.100				2.120	2.020	1.700	
	θ=0.85	-0.430	-0.180	0.100				2.120	2.020	1.700	
	θ=0.85	-0.430	-0.180	0.100				2.120	2.020	1.700	
3/4b	θ=0.85	-0.250	-0.230						3.200	4.180	
	θ=0.85	-0.250	-0.230						3.200	4.180	
	θ=0.85	-0.250	-0.230						3.200	4.180	
b	θ=0.85	-0.070								6.943	
	θ=0.85	-0.070								6.943	
	θ=0.85	-0.070								6.943	

K1

		-b	-3/4b	-1/2b	-1/4b	0	1/4b	1/2b	3/4b	b	TOTAL
0	θ=0.85	0.580	0.750	0.970	1.270	1.450	1.270	0.970	0.750	0.580	
	θ=0.85	0.580	0.750	0.970	1.270	1.450	1.270	0.970	0.750	0.580	
	θ=0.85	0.580	0.750	0.970	1.270	1.450	1.270	0.970	0.750	0.580	
1/4b	θ=0.85	0.350	0.470	0.640	0.920		1.500	1.370	1.150	0.970	
	θ=0.85	0.350	0.470	0.640	0.920		1.500	1.370	1.150	0.970	
	θ=0.85	0.350	0.470	0.640	0.920		1.500	1.370	1.150	0.970	
1/2b	θ=0.85	0.220	0.300	0.430				1.700	1.670	1.560	
	θ=0.85	0.220	0.300	0.430				1.700	1.670	1.560	
	θ=0.85	0.220	0.300	0.430				1.700	1.670	1.560	
3/4b	θ=0.85	0.130	0.200						2.170	2.430	
	θ=0.85	0.130	0.200						2.170	2.430	
	θ=0.85	0.130	0.200						2.170	2.430	
b	θ=0.85	0.120								3.570	
	θ=0.85	0.120								3.570	
	θ=0.85	0.120								3.570	

K α

		-b	-3/4b	-1/2b	-1/4b	0	1/4b	1/2b	3/4b	b	TOTAL
0	K 0	-0.370	0.330	1.050	1.700	2.050	1.700	1.050	0.330	-0.370	
	K 1	0.580	0.750	0.970	1.270	1.450	1.270	0.970	0.750	0.580	
	K α	-0.114	0.443	1.028	1.584	1.888	1.584	1.028	0.443	-0.114	7.886
1/4b	K 0	-0.550	-0.020	0.510	1.110		2.030	1.670	1.020	0.250	
	K 1	0.350	0.470	0.640	0.920		1.500	1.370	1.150	0.970	
	K α	-0.307	0.112	0.545	1.059	1.584	1.887	1.589	1.055	0.444	7.900
1/2b	K 0	-0.430	-0.180	0.100				2.120	2.020	1.700	
	K 1	0.220	0.300	0.430				1.700	1.670	1.560	
	K α	-0.255	-0.050	0.189	0.545	1.028	1.589	2.007	1.926	1.662	7.937
3/4b	K 0	-0.250	-0.230						3.200	4.180	
	K 1	0.130	0.200						2.170	2.430	
	K α	-0.147	-0.114	-0.050	0.112	0.443	1.055	1.926	2.922	3.708	8.074
b	K 0	-0.070								6.943	
	K 1	0.120								3.570	
	K α	-0.019	-0.147	-0.255	-0.307	-0.114	0.444	1.662	3.708	6.032	7.998

Summary K α

	-b	-3/4b	-1/2b	-1/4b	0	1/4b	1/2b	3/4b	b	TOTAL
-b	6.032	3.708	1.662	0.444	-0.114	-0.307	-0.255	-0.147	-0.019	7.998
-3/4b	3.708	2.922	1.926	1.055	0.443	0.112	-0.050	-0.114	-0.147	8.074
-1/2b	1.662	1.926	2.007	1.589	1.028	0.545	0.189	-0.050	-0.255	7.937
-1/4b	0.444	1.055	1.589	1.887	1.584	1.059	0.545	0.112	-0.307	7.900
0	-0.114	0.443	1.028	1.584	1.888	1.584	1.028	0.443	-0.114	7.886
1/4b	-0.307	0.112	0.545	1.059	1.584	1.887	1.589	1.055	0.444	7.900
1/2b	-0.255	-0.050	0.189	0.545	1.028	1.589	2.007	1.926	1.662	7.937
3/4b	-0.147	-0.114	-0.050	0.112	0.443	1.055	1.926	2.922	3.708	8.074
b	-0.019	-0.147	-0.255	-0.307	-0.114	0.444	1.662	3.708	6.032	7.998

Summary K α

Distance	0.00	1.75	3.50	5.25	7.00	8.75	10.50	12.25	14.00	TOTAL
-b	6.032	3.708	1.662	0.444	-0.114	-0.307	-0.255	-0.147	-0.019	7.998
-3/4b	3.708	2.922	1.926	1.055	0.443	0.112	-0.050	-0.114	-0.147	8.074
-1/2b	1.662	1.926	2.007	1.589	1.028	0.545	0.189	-0.050	-0.255	7.937
-1/4b	0.444	1.055	1.589	1.887	1.584	1.059	0.545	0.112	-0.307	7.900
0	-0.114	0.443	1.028	1.584	1.888	1.584	1.028	0.443	-0.114	7.886
1/4b	-0.307	0.112	0.545	1.059	1.584	1.887	1.589	1.055	0.444	7.900
1/2b	-0.255	-0.050	0.189	0.545	1.028	1.589	2.007	1.926	1.662	7.937
3/4b	-0.147	-0.114	-0.050	0.112	0.443	1.055	1.926	2.922	3.708	8.074
b	-0.019	-0.147	-0.255	-0.307	-0.114	0.444	1.662	3.708	6.032	7.998

Distributution Coefficient(Before Rivise)

Distance	2.00	4.00	6.00	8.00	10.00	12.00	TOTAL
-b	3.415	1.314	0.205	-0.224	-0.270	-0.163	4.279
-3/4b	2.780	1.677	0.793	0.254	-0.004	-0.105	5.395
-1/2b	1.937	1.887	1.349	0.752	0.291	-0.016	6.200
-1/4b	1.131	1.674	1.757	1.284	0.692	0.174	6.712
0	0.527	1.187	1.714	1.714	1.187	0.527	6.857
1/4b	0.174	0.692	1.284	1.757	1.674	1.131	6.712
1/2b	-0.016	0.291	0.752	1.349	1.887	1.937	6.200
3/4b	-0.105	-0.004	0.254	0.793	1.677	2.780	5.395
b	-0.163	-0.270	-0.224	0.205	1.314	3.415	4.279

Distributution Coefficient(After Rivise)

	Distance	2.00	4.00	6.00	8.00	10.00	12.00	TOTAL
-b	0.00	4.789	1.843	0.288	-0.314	-0.378	-0.228	6.000
-3/4b	1.75	3.091	1.865	0.882	0.283	-0.004	-0.117	6.000
-1/2b	3.50	1.875	1.826	1.305	0.728	0.281	-0.016	6.000
-1/4b	5.25	1.011	1.496	1.571	1.148	0.618	0.156	6.000
0	7.00	0.461	1.039	1.500	1.500	1.039	0.461	6.000
1/4b	8.75	0.156	0.618	1.148	1.571	1.496	1.011	6.000
1/2b	10.50	-0.016	0.281	0.728	1.305	1.826	1.875	6.000
3/4b	12.25	-0.117	-0.004	0.283	0.882	1.865	3.091	6.000
b	14.00	-0.228	-0.378	-0.314	0.288	1.843	4.789	6.000

Coefficient of load Distribution

Distance	G 1	G 2	G 3	G 4	G 5	G 6	TOTAL
0.125	4.668	1.845	0.330	-0.272	-0.351	-0.220	6.000
0.250	4.547	1.846	0.373	-0.229	-0.325	-0.212	6.000
2.000	2.918	1.859	0.942	0.346	0.036	-0.102	6.000
9.000	0.131	0.570	1.088	1.533	1.544	1.135	6.000
10.337	0.000	0.313	0.767	1.330	1.796	1.794	6.000
12.000	-0.102	0.036	0.346	0.942	1.859	2.918	6.000
13.750	-0.212	-0.325	-0.229	0.373	1.846	4.547	6.000
13.875	-0.220	-0.351	-0.272	0.330	1.845	4.668	6.000

1.3.2 Estimation of Loads

(1) Dead loads

(a) Beam weight

① G1 and G6 beam

Slab by cantilever type	$1/2 \times 0.50 \times 1.75 \times 2450$	= 1072 kg/m
	$0.25 \times 2.00 \times 2450$	= 1225 kg/m
	$1/2 \times 0.25 \times 1.50 \times 2450$	= 459 kg/m
Hunch	$1/2 \times 0.20 \times 0.20 \times 2450$	= 49 kg/m
Main Beam	$0.50 \times 1.70 \times 2450$	= 2083 kg/m
Total		= 4888 kg/m
	W_{d1}	≒ 4900 kg/m
Cross Beam	$W_{d2}=0.40 \times 0.75 \times 2450 \times 1/2 \times 1.50$	= 551 kg/m

② G2, G3 and G4 beam

Slab	$0.25 \times 1.50 \times 2450$	= 919 kg/m
Hunch	$2 \times 1/2 \times 0.20 \times 0.20 \times 2450$	= 98 kg/m
Main Beam	$0.50 \times 1.70 \times 2450$	= 2083 kg/m
Total	W_{d1}	= 3100 kg/m
Cross Beam	$W_{d2}=0.40 \times 0.75 \times 2450 \times 1.50$	= 1103 kg/m

(b) Others

Weights of pavement and hand rail are added as dead loads.

① Hand rail $P_1 = 723$ kg/m

② Pavement of side-walk (5 cm in thickness)

$W_1=0.05 \times 1.75 \times 2300= 201$ kg/m^2

③ Pavement of road way (7.5 cm in mean thickness)

$W_2=0.075 \times 2300= 173$ kg/ m^2

(2) Live loads

(a) Uniformed distribution load at side-walk $w= 300$ kg/m^2

(b) Live load at road way

	Main load	Sub load
Concentrated load	P= 8000 kg/m	P/2= 4000 kg/m
Uniformed distribution load	P= 500 kg/m^2	P/2= 250 kg/m^2

$P=23.4/(3.5-0.55)=7.93$ t/m → 8.00 t/m

(3) Impact Coefficient

$i=0.4-0.008 \cdot L=0.4-0.008 \times 9.68= 0.323$

(3)Load Distribution

Side Walk(L)								
G 1			G 2			G 3		
Position	Coefficient	Area	Position	Coefficient	Area	Position	Coefficient	Area
0.250	4.547		0.250	1.846		0.250	0.373	
1.750	3.091	5.729	1.750	1.865	2.783	1.750	0.882	0.941
2.000	2.918	0.751	2.000	1.859	0.466	2.000	0.942	0.228
TOTAL		6.480	TOTAL		3.249	TOTAL		1.169

Side Walk(R)								
G 1			G 2			G 3		
Position	Coefficient	Area	Position	Coefficient	Area	Position	Coefficient	Area
12.000	-0.102		12.000	-0.004		12.000	0.346	
12.250	-0.117	-0.027	12.250	-0.325	-0.041	12.250	0.283	0.079
13.750	-0.212	-0.247	13.750	-0.351	-0.507	13.079	0.000	0.117
						13.750	-0.229	-0.077
TOTAL		-0.274	TOTAL		-0.548	TOTAL		0.119

Road Way								
G 1			G 2			G 3		
Position	Coefficient	Area	Position	Coefficient	Area	Position	Coefficient	Area
2.000	2.918		2.000	1.859		2.000	0.942	
3.500	1.875	3.595	3.500	1.826	2.764	3.500	1.305	1.685
5.250	1.011	2.525	5.250	1.496	2.907	5.250	1.571	2.517
7.000	0.461	1.288	7.000	1.039	2.218	7.000	1.500	2.687
8.750	0.156	0.540	8.750	0.618	1.450	8.750	1.148	2.317
10.337	0.000	0.769	10.500	0.281	2.310	10.500	0.728	3.899
10.500	-0.016	-0.001	12.000	0.036	0.238	12.000	0.346	0.806
12.000	-0.102	-0.089						
TOTAL		8.627	TOTAL		11.886	TOTAL		13.910

Main Live Load								
G 1			G 2			G 3		
Position	Coefficient	Area	Position	Coefficient	Area	Position	Coefficient	Area
2.000	2.918		2.000	1.859		2.000	0.942	
3.500	1.875	3.595	3.500	1.826	2.764	3.500	1.305	1.685
5.250	1.011	2.525	5.250	1.496	2.907	5.250	1.571	2.517
7.000	0.461	1.288	7.000	1.039	2.218	7.000	1.500	2.687
8.750	0.156	0.540	8.750	0.618	1.450	8.750	1.148	2.317
9.000	0.131	0.592	9.000	0.570	1.609	9.000	1.088	2.588
TOTAL		8.540	TOTAL		10.948	TOTAL		11.794

Sub Live Load								
G 1			G 2			G 3		
Position	Coefficient	Area	Position	Coefficient	Area	Position	Coefficient	Area
9.000	0.131		9.000	0.570		9.000	1.088	
10.337	0.000	0.088	10.500	0.281	0.638	10.500	0.728	1.362
			12.000	0.036	0.238	12.000	0.346	0.806
TOTAL		0.088	TOTAL		0.876	TOTAL		2.168

Side Walk Live Load								
G 1			G 2			G 3		
Position	Coefficient	Area	Position	Coefficient	Area	Position	Coefficient	Area
0.250	4.547		0.250	1.846		0.250	0.373	
1.750	3.091	5.729	1.750	1.865	2.783	1.750	0.882	0.941
2.000	2.918	0.751	2.000	1.859	0.466	2.000	0.942	0.228
			12.000	0.036		12.000	0.346	
			12.225	0.000	0.004	12.250	0.283	0.079
						13.079	0.000	0.117
TOTAL		6.480	TOTAL		3.253	TOTAL		1.365

Strength of Other Dead Load

		G 1		G 2		G 3		G 4		G 5		G 6	
		AOR η	Wd2	AOR η	Wd2	AOR η	Wd2	AOR η	Wd2	AOR η	Wd2	AOR η	Wd2
Hand Rail(L)	723 Kg/m	4.668	3375	1.845	1334	0.330	239	-0.272	-196	-0.351	-254	-0.220	-159
Hand Rail(R)	723 Kg/m	-0.220	-159	-0.351	-254	-0.272	-196	0.330	239	1.845	1334	4.668	3375
Pavement of S-Walk(L)	201 Kg/m2	6.480	1302	3.249	653	1.169	235	1.169	235	3.249	653	6.480	1302
Pavement of S-Walk(R)	201 Kg/m2	-0.274	-55	-0.548	-110	0.119	24	0.119	24	-0.548	-110	-0.274	-55
Pavement of Roadway	173 Kg/m2	8.627	1493	11.886	2056	13.910	2406	13.910	2406	11.886	2056	8.627	1493
ΣWd3			5956		3679		2708		2708		3679		5956
ΣWd3／6			993		613		451		451		613		993
Design Load			1000		620		460		460		620		1000

		G1		G2		G3	
		A	p、P	A	p、P	A	p、P
Distributed Load	300 kg/m2	6.480	1944	3.253	976	1.365	410
	P／6		324		163		68
	Design P		330		170		70
	Side Walk						
Concentrated Load	8000 kg/m	8.540	68319	10.948	87580	11.794	94351
	4000 kg/m	0.088	350	0.876	3504	2.168	8670
	Σ		68669		91084		103021
	Σ／6		11445		15181		17170
	Design		11450		15190		17180
Uniform Distributed Load	500 kg/m2	8.540	4270	10.948	5474	11.794	5897
	250 kg/m2	0.088	22	0.876	219	2.168	542
	Σ		4292		5693		6439
	Σ／6		715		949		1073
	Design		720		950		1080

Table 2 Estimation of Bending Moment(t·m)

l=	9.68	1／6			1／3			1／2		
		G 1	G 2	G 3	G 1	G 2	G 3	G 1	G 2	G 3
Influence Line	η max	1.344			2.151			2.420		
	η m	0.807			1.613			2.420		
	A	6.507			10.411			11.713		
Dead Load	Wd1kg/m	4900	3100	3100	4900	3100	3100	4900	3100	3100
	Wd2kg	551	1103	1103	551	1103	1103	551	1103	1103
	Wd3kg/m	1000	620	460	1000	620	460	1000	620	460
	Md1	31.88	20.17	20.17	51.02	32.28	32.28	57.39	36.31	36.31
	Md2	0.44	0.89	0.89	0.89	1.78	1.78	1.33	2.67	2.67
	Md3	6.51	4.03	2.99	10.41	6.46	4.79	11.71	7.26	5.39
	TOTAL Md	38.84	25.10	24.06	62.32	40.51	38.84	70.44	46.24	44.37
Live Load	w	330	170	70	330	170	70	330	170	70
	P	11450	15190	17180	11450	15190	17180	11450	15190	17180
	p	720	950	1080	720	950	1080	720	950	1080
	i	0.323			0.323			0.323		
	Mw	2.15	1.11	0.46	3.44	1.77	0.73	3.87	1.99	0.82
	MP	15.39	20.42	23.10	24.63	32.68	36.96	27.71	36.76	41.58
	Mp	4.69	6.18	7.03	7.50	9.89	11.24	8.43	11.13	12.65
	Mi	6.49	8.59	9.73	10.38	13.75	15.57	11.67	15.47	17.51
	TOTAL ML	28.71	36.30	40.31	45.94	58.08	64.50	51.68	65.35	72.56
TOTAL ΣM		67.55	61.40	64.37	108.26	98.59	103.34	122.12	111.59	116.93

Table 3 Estimation of Shearing Strength

l=	9.68	0			1／2		
		G 1	G 2	G 3	G 1	G 2	G 3
Influence Line	η max	1.000		0.000	0.500		-0.500
	η m		0.500		0.500		-0.500
	A, −A	4.840		0.000	1.210		-1.210
	A d		4.840			0.000	
Dead Load	Wd1kg/m	4900	3100	3100	4900	3100	3100
	Wd2kg	551	1103	1103	551	1103	1103
	Wd3kg/m	1000	620	460	1000	620	460
	Sd1	23.72	15.00	15.00	-	-	-
	Sd2	0.55	1.10	1.10	0.55	1.10	1.10
	Sd3	4.84	3.00	2.23	-	-	-
	TOTAL Sd	29.11	19.11	18.33	0.55	1.10	1.10
Live Load	w	330	170	70	330	170	70
	P	11450	15190	17180	11450	15190	17180
	p	720	950	1080	720	950	1080
	i		0.323			0.323	
	S w	1.60	0.82	0.34	0.40	0.21	0.08
	S P	11.45	15.19	17.18	5.73	7.60	8.59
	S p	3.48	4.60	5.23	0.87	1.15	1.31
	S i	4.82	6.39	7.24	2.13	2.82	3.20
	TOTAL SL	21.36	27.00	29.98	9.13	11.77	13.18
TOTAL ΣS		50.46	46.11	48.32	9.68	12.88	14.28

1.3.3 Estimation of section force

(1) Bending moment

The bending moments of the main beam are estimated in Table 2.

(2) Shearing force

Shearing force is calculated as shown in Table 3.

1.3.4 Design of section

(1) Effective width of compression flange

$\lambda = l/8 \quad (\lambda \leqq b)$

$= 1300/8 = 162.5 > b_1 = 135\text{cm}$

(2) Design due to flexural stress

(a) Estimation of required reinforcing bar area

In case of T-beam type, dimensions of the section are fixed whether the necessary reinforcing bars can be arranged in the supposed web section. Therefore, as the compression stress of the reinforced concrete is hardly over the allowable value, the following formula can be used to estimate the required reinforcing bar area.

$A_s = M/(d - t/2) \times \sigma_{sa}$

Slab thickness $t = 20$ cm

The effective height is estimated $d = 159.5\text{cm}$ ($= 170 - 10.5$).

	M (t·m)	d−t/2	σ_{sa}	Required RBA	RB to be used	RBA to be used
G_1	121.91	149.5	1800	45.3	D25-10	49.1
G_2	111.41	149.5	1800	41.4	D25-10	49.1
G_3	116.75	149.5	1800	43.4	D25-10	49.1

RBA: Reinforcing bar area, RB: Reinforcing bar

(b) Bending Stress and resisted bending moment

Bending stresses are estimated by the following formula.

$x = (n \times As \times d + b \times t^2/2)/(n \times As + b \times t)$

$y = x - t/2 + t^2/(6 \times (2x - t))$

$\sigma_s = M/(As \times (d - x + y)$

$\sigma_c = \sigma_s \times x/(n \times (d - x))$

Resisting moment is estimated as follows,

$M_{rs} = As \times (d - x + y) \times \sigma_{sa}$

$M_{rc} = (d - x + y) \times n \times (d - x) \times As \times \sigma_{ca}/x$

(3) Design due to shearing stress

(a) Shearing stress at each beams

Shearing force is estimated by the following formula.

$\tau = S/(b_o \cdot Z) = S/\ b_o \cdot (d - t/2)$

Although the effective height of d is changed according to the bending of the main reinforcing bar, it is assumed the section is the same as the middle of the beam.

$\tau = S/(50 \times (159.5 - 20/2)) = 0.134\ S\ kg/cm^2$

where, S: shearing force (t)

Allowable shearing stress is 8.0 kg/cm^2, in case diagonal tensile stress is supported only by reinforced concrete. Therefore, there is no need the bend-up bars.

Estimation of Bending Stress

	G 1	G 2	G 3	
M(t・m)	122.12	111.59	116.93	
Effective Width of Slab(b)	200	200	200	
Reinforcing Bar(As)	49.1	49.1	49.1	10-D25 = 49.1cm2
x (cm)	33.2	33.2	33.2	
y (cm)	24.7	24.7	24.7	
As(d-x+y)	7410.9	7410.9	7410.9	
σ s	1647.9	1505.7	1577.8	< σ sa=1800
x/n(d-x)	0.01756	0.01756	0.01756	
σ c	28.9	26.4	27.7	< σ ca=85

t= 20
d= 159.5

Estimation of Shearing Stress

	G 1		G 2		G 3		
	Supporting	Center	Supporting	Center	Supporting	Center	
S (t)	50.46	9.68	46.11	12.88	48.32	14.28	
τ (kg/cm2)	6.76	1.30	6.18	1.73	6.47	1.91	< τ a=8.0

1.4 Design of cross beam

1.4.1 Formula of bending moment

Bending moment of the cross beam is calculated by the following Guyon – Massonnet formula.

$M_y = \Sigma u \cdot b \cdot \lambda \cdot r \cdot \sin(\pi \cdot \chi / l)$

Where λ : interval of cross beam

b: 1/2 of effective width on slub

l : span klength

x: distance between the supporting potion and the position of M_y to be estimated

r: first amplitude of which load is shown by the Fourier series

"r" is estimated by the following formula, in case of the concentrated load is effected at the point of "u" from the supporting portion.

$r = 2P/l \cdot \sin(\pi \cdot u/l)$

On the other hand, the formula is changed in case of the uniform distribution load is effected.

$r = 4p/\pi = 4p/3.14 = 1.274$

1.4.2 Loads

(1) Dead load

Slab weight	0.20×10.08/2×2450		= 2470 kg/m
Cross bean	(see design of main bean)		= 551 kg/m
Total		W_{d1}	= 3021 kg/m

(2) Other dead load

Hand rail	$P_1 =$ 723 kg/m
Pavement of side-walk	$W_1 =$ 201 kg/m²
Pavement of Road way	$W_2 =$ 173 kg/ m²

(3) Live loads

(a) Live load

Uniform distribution load of side-walk	$\omega = 300$ kg/m²
Concentrated load of road way	P=8000 kg/m
Uniform distribution load of road way	p= 500 kg/m²
Impact coefficient	$i = 0.4 - 0.008 \times 9.68 = 0.323$

1.4.3 Influence line of the bending moment

"μ_0" and "μ_1" are estimated from the figure as same as using K factor and the same grid parameter, $\theta = 0.85$, $\alpha = 0.07116$ and $\sqrt{\alpha} = 0.27$, calculated in the design of main beam. "μ_α" is estimated by the following formula.

$\mu_\alpha = \mu_0 + (\mu_1 - \mu_0)\sqrt{\alpha}$

It is practically allowed that this influence line is considered as that in middle point of the cross beam. Therefore, design shall be done used " 0 " point in diagram of the influence line.

$\mu_0 \times 10^{-4}$

		-b	-3/4b	-1/2b	-1/4b	0	1/4b	1/2b	3/4b	b	TOTAL
0	$\theta=0.85$	-720	-500	-150	420	1400	420	-150	-500	-720	
	$\theta=0.85$	-720	-500	-150	420	1400	420	-150	-500	-720	
	$\theta=0.85$	-720	-500	-150	420	1400	420	-150	-500	-720	
1/4b	$\theta=0.85$	-360	-560	-250	-20	460	1400	250	-560	-1230	
	$\theta=0.85$	-360	-560	-250	-20	460	1400	250	-560	-1230	
	$\theta=0.85$	-360	-560	-250	-20	460	1400	250	-560	-1230	
1/2b	$\theta=0.85$	-130	-150	-180	-130	60	480	1200	-300	-1700	
	$\theta=0.85$	-130	-150	-180	-130	60	480	1200	-300	-1700	
	$\theta=0.85$	-130	-150	-180	-130	60	480	1200	-300	-1700	
3/4b	$\theta=0.85$	-10	-40	-60	-60	-40	80	280	610	-1500	
	$\theta=0.85$	-10	-40	-60	-60	-40	80	280	610	-1500	
	$\theta=0.85$	-10	-40	-60	-60	-40	80	280	610	-1500	
b	$\theta=0.85$	–	–	–	–	–	–	–	–	–	
	$\theta=0.85$	–	–	–	–	–	–	–	–	–	
	$\theta=0.85$	–	–	–	–	–	–	–	–	–	

$\mu_1 \times 10^{-4}$

		-b	-3/4b	-1/2b	-1/4b	0	1/4b	1/2b	3/4b	b	TOTAL
0	$\theta=0.85$	-230	-220	-140	140	900	140	-140	-220	-230	
	$\theta=0.85$	-230	-220	-140	140	900	140	-140	-220	-230	
	$\theta=0.85$	-230	-220	-140	140	900	140	-140	-220	-230	
1/4b	$\theta=0.85$	-150	-160	-140	-100	110	900	90	-200	-320	
	$\theta=0.85$	-150	-160	-140	-100	110	900	90	-200	-320	
	$\theta=0.85$	-150	-160	-140	-100	110	900	90	-200	-320	
1/2b	$\theta=0.85$	-100	-90	-150	-90	-120	110	840	-20	-440	
	$\theta=0.85$	-100	-90	-150	-90	-120	110	840	-20	-440	
	$\theta=0.85$	-100	-90	-150	-90	-120	110	840	-20	-440	
3/4b	$\theta=0.85$	-50	-60	-70	-100	-150	-90	100	730	-430	
	$\theta=0.85$	-50	-60	-70	-100	-150	-90	100	730	-430	
	$\theta=0.85$	-50	-60	-70	-100	-150	-90	100	730	-430	
b	$\theta=0.85$	–	–	–	–	–	–	–	–	–	
	$\theta=0.85$	–	–	–	–	–	–	–	–	–	
	$\theta=0.85$	–	–	–	–	–	–	–	–	–	

$\mu_\alpha \times 10^{-4} (\mu_\alpha = \mu_0 + (\mu_1 - \mu_0)\sqrt{\alpha})$

		-b	-3/4b	-1/2b	-1/4b	0	1/4b	1/2b	3/4b	b	TOTAL
0	μ_0	-720	-500	-150	420	1400	420	-150	-500	-720	
	μ_1	-230	-220	-140	140	900	140	-140	-220	-230	
	μ_α	-588	-424	-147	344	1265	344	-147	-424	-588	
1/4b	μ_0	-360	-560	-250	-20	460	1400	250	-560	-1230	
	μ_1	-150	-160	-140	-100	110	900	90	-200	-320	
	μ_α	-303	-452	-220	-42	366	1265	207	-463	-984	
1/2b	μ_0	-130	-150	-180	-130	60	480	1200	-300	-1700	
	μ_1	-100	-90	-150	-90	-120	110	840	-20	-440	
	μ_α	-122	-134	-172	-119	11	380	1103	-224	-1360	
3/4b	μ_0	-10	-40	-60	-60	-40	80	280	610	-1500	
	μ_1	-50	-60	-70	-100	-150	-90	100	730	-430	
	μ_α	-21	-45	-63	-71	-70	34	231	642	-1211	
b	μ_0	–	–	–	–	–	–	–	–	–	
	μ_1	–	–	–	–	–	–	–	–	–	
	μ_α	–	–	–	–	–	–	–	–	–	

Coefficient of Load Distribution of Cross Beam

Estimation of $u\alpha$, in case f=0

Distance	0.000	1.750	3.500	5.250	7.000	8.750	10.500	12.250	14.000
μ_α	-0.0588	-0.0424	-0.0147	0.0344	0.1265	0.0344	-0.0147	-0.0424	-0.0588

From above table

Distance	0.125	0.250	2.000	4.024	9.976	12.000	13.750	13.875	
μ_α	-0.0576	-0.0564	-0.0385	0.0000	0.0000	-0.0385	-0.0564	-0.0576	

Distribution of Load in Cross Beam

Side-walk(L)		
Position	Coefficient	Area
0.250	-0.0564	
1.750	-0.0424	-0.074
2.000	-0.0385	-0.010
TOTAL		-0.084

Side-walk(R)		
Position	Coefficient	Area
12.000	-0.0385	
12.250	-0.0424	-0.010
13.750	-0.0564	-0.074
TOTAL		-0.084

Pavement of RoadWay		
Position	Coefficient	Area
2.000	-0.0385	
3.500	-0.0147	-0.040
5.250	0.0344	0.017
7.000	0.1265	0.141
8.750	0.0344	0.141
10.500	-0.0147	0.017
12.000	-0.0385	-0.040
TOTAL		0.236

Main Live Load		
Position	Coefficient	Area
4.024	0.0000	
5.250	0.0344	0.021
7.000	0.1265	0.141
8.750	0.0344	0.141
9.976	0.0000	0.021
TOTAL		0.324

Sub Live Load		
Position	Coefficient	Area
2.000	-0.0385	
3.500	-0.0147	-0.040
4.024	0.0000	-0.004
9.976	0.0000	
10.500	-0.0147	-0.004
12.000	-0.0385	-0.040
TOTAL		-0.088

Live Load in Side-walk		
Position	Coefficient	Area
0.250	-0.0564	
1.750	-0.0424	-0.074
2.000	-0.0385	-0.010
12.000	-0.0385	
12.250	-0.0424	-0.010
13.750	-0.0564	-0.074
TOTAL		-0.168

1.4.4 Estimation of bending moment

(1) Bending moment due to deal loads

(a) Bending moment due to beam weight

$M_{d1}=1/8\times3018\times2.00=1509$ kg·m

(b) Bending moment due to other dead load

$M_y=\Sigma\,\mu\cdot b\cdot\lambda\cdot\gamma\cdot\sin(\pi\cdot\chi/l)$

Where $b=7.00$ m

$\lambda=3.40$ m

$l=9.68$ m

$\chi=5.035$ m

$\gamma=1.274\cdot p$ (uniform distributed load)

$\chi=l/2$, $\sin((\pi\cdot l/2)/l)=\sin(\pi/2)=1$

Therefore, $M_y=\Sigma\,\mu\times7.00\times3.40\times1.274\times p$

$=\Sigma\,\mu\times30.32\times p$

Hand rail	$2\times30.32\times723\times(-0.0576)$	$=-2525$ kg·m
Pavement of side-walk	$2\times30.32\times201\times(-0.084)$	$=-1024$ kg·m
Pavement of Road way	$30.32\times173\times0.236$	$=1238$ kg·m
	ΣM_{d2}	$=-2311$ kg·m

(c) Total bending moment

$M_D=1509+(-2311)=-802$ kg·m

(2) Bending moment due to live loads

(a) Concentrated load

$M_y=\Sigma\,\mu\times7.00\times3.40\times2P/l\times\sin(\pi\cdot u/l)\cdot\sin(\pi\cdot t/l)$

$=\Sigma\,\mu\times23.80\times2P/9.68\times\sin((\pi l/2)/l)\cdot\sin((\pi l/2)/l)$

$=\Sigma\,\mu\times4922\cdot P$

Therefore,

$M_P=4.922\times8000\times0.324=12.758$ t·m

$M_P=4.922\times8000\times(-0.044)=-1.733$ t·m

(b) Uniform distributed load of road way

$M_P=30.32\times500\times0.324=4.912$ t·m

$M_P=30.32\times500\times(-0.044)=-0.667$ t·m

(c) Uniform distributed load of side-walk

$M_\omega=30.32\times300\times(-0.168)=-1.528$ t·m

(d) Bending moment due to impact

$M_i=(12.758+4.912)\times0.323=5.707$ t·m

$M_i=-(1.733+0.667)\times0.323=-0.775$ t·m

(e) Total bending moment due to live loads

$\Sigma M_i=12.252+4.912+5.389=22.871$ t·m

$\Sigma M_i=-(1.664+0.667+0.775+1.528)=-4.634$ t·m

(C) Design bending moment

Mmax$=$-0.802$+$22.871$=$22.069 t·m

Mmin$=$-0.802$-$4.634$=$5.436 t·m

5.5.5 Section and stresses

(1) Estimated Section

(a) Effective width of compression flange

$\lambda=(n-1)/6\times C$

where n: Number of main bean$=$6

C: Interval of the main beam $=$2.00m

$=(6-1)/6\times 2.00=1.67$ m

Therefore,

$b=2\times 167+40=374$ cm

(b) Required reinforcing bar area

(i) Positive reinforcing bar

M$=$22.07 t·m

Effective height $d=100.0-7.0=93.0$ cm

Thickness of the slab $t=25.0$ cm

$As'=M/((d-t/2)\cdot\sigma_{sa})=2207000/((93.0-25/2)\times 1800)=15.23\ cm^2$

In case of using number of 4 and D25 of diameter of reinforcing bar, reinforcing bar area is as follows.

$As=4\times 4.91=19.64\ cm^2>15.23\ cm^2$

Therefore, number of 4 and D25 of diameter of reinforcing bar is arranged.

(ii) Negative reinforcing bar

M$=$-5.44 t·m

Effective height $d=100.0-7.0=93.0$ cm

Width of the web $b_0=40.0$ cm

Section is estimated as the rectangle.

$As'=M/(7/8\cdot d\cdot\sigma_{sa})=544000/(7/8\times 93\times 1800)=3.79\ cm^2$

In case of using number of 2 and D19 of diameter of reinforcing bar, reinforcing bar area is as follows.

$As=2\times 2.835=5.67\ cm^2>3.68\ c\ m^2$

Therefore, number of 2 and D19 of diameter of the positive reinforcing bar is arranged.

(2) Stresses

(a) Stress due to positive bending moment

Reinforcing bar area to be used $As=15.20\ cm^2$

$\chi=(n\cdot As\cdot d+b\cdot t^2/2)/(n\cdot As+b\cdot t)$

$=(15\times 15.20\times 93+374\times 25^2/2)/(15\times 15.20+374\times 25)=14.42\ cm< t=25\ cm$

It is estimated as a rectangular section because neutral axis is in the flange.

$p=As/(b\cdot d)=15.20/(374\times 93)=0.00044$

$k=\sqrt{(2\cdot n\cdot p+(n\cdot p)^2)}-n\cdot p=0.108$

$j=1-k/3=1-0.108/3=0.964$

$\sigma_c=2M/(k\cdot j\cdot b\cdot d^2)=2\times 2207000/(0.108\times 0.964\times 374\times 93^2)$

$=13.1\ kg/cm^2<\sigma_{ca}=80\ kg/cm^2$

$\sigma_s=M/(As\cdot j\cdot d)=2207000/(15.2\times 0.964\times 93)$

$=1620\ kg/cm^2<\sigma_{sa}=1800\ kg/cm^2$

(b) Stress due to negative bending moment

Reinforcing bar area to be used $As=5.67\ cm^2$

$p=As/(b\cdot d)=5.67/(40\times 93)=0.0015$

$k=\sqrt{(2\cdot n\cdot p+(n\cdot p)^2)}-n\cdot p=0.191$

$j=1-k/3=1-0.191/3=0.936$

$\sigma_c=2M/(k\cdot j\cdot b\cdot d^2)=2\times 544000/(0.191\times 0.936\times 40\times 93^2)$

$=17.6\ kg/\ cm^2<\sigma_{ca}=80\ kg/\ cm^2$

$\sigma_s=M/(As\cdot j\cdot d)=544000/(5.67\times 0.936\times 93)$

$=1102\ kg/\ cm^2<\sigma_{sa}=1800\ kg/\ cm^2$

(3) Bar arrangement and stress

Reinforcing bar arrangement and stress are summarized as follows

Name		No.1				No.2				No.3			
Span Length		9.68				10.67				11.392			
Position		Bar arrangement	σ_{ca}	σ_{sa}	τ_a	Bar arrangement	σ_{ca}	σ_{ca}	τ_a	Bar arrangement	σ_{sa}	σ_{ca}	τ_a
Middle of the span in the slab	Lower	8-D16	52.5	1220		8-D16	52.5	1220		8-D16	52.5	1220	
	Upper	4-D16				4-D16				4-D16			
Supporting position of the slab	Lower	4-D16				4-D16				4-D16			
	Upper	8-D16	51.9	1451		8-D16	51.9	1451		8-D16	51.9	1451	
Distribution bar of the slab		4-D16				4-D16				4-D16			
Slab by cantilever type of side-walk		8-D16				8-D16				8-D16			
Main beam (G1, G6)		10−D25	28.9	1645.0	6.76	10−D28	32.5	1555.3	7.25	10−D28	36.5	1748.5	7.64
Main beam (others)		10−D25	27.7	1575.3	6.47	10−D25	30.2	1446.7	6.75	10−D28	33.1	1582.6	6.91
Cross beam	Lower	4−D25	13.1	1620		4−D25	13.8	1484		4−D25	13.7	1318	
	Upper	2−D19	17.6	1102		2−D22	30.1	1601		2−D22	27.4	1454	

Allowable bending compressive stress; σ_{ca}=85 kg/cm^2

Allowable shearing stress (beam); τ_a=8.0 kg/cm^2

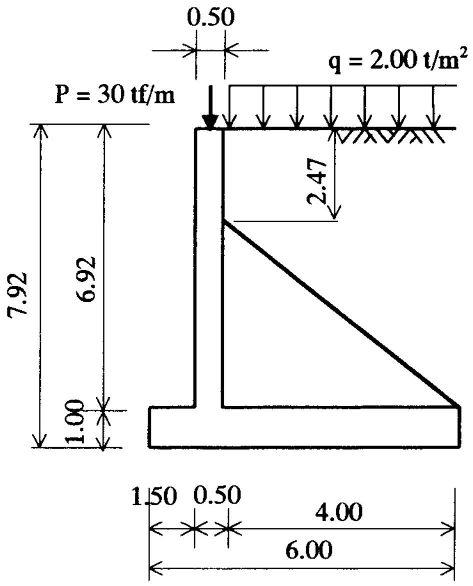
Allowable tensile stress (slab); σ_{sa}=2000 kg/cm^2

Allowable tensile stress (main beam); σ_{sa}=1800 kg/cm^2

APPENDIX C.4.2-3Structural Analysis of Abutment

(1) Design Criteria

(a) Sectional Dimension for Analysis



(b) Coefficient of Earth Pressure ; Ka=0.333

(c) Live Load ; q=2.00 tf/m^2

(d) Load of Bridge ; P=30.0 tf/m

(2) Check of Stability

The Wall stability analysis has been made to show the results in Table 1.

Table 1 Result of Stability Analysis

Load	Vertical Force (tf)	Arm Length (m)	Stabilizing Moment (tf•m)	Horizontal Force (tf)	Arm Length (m)	Overturning Moment (tf•m)
Wall	64.340	2.650	170.500			
Weight of Earth	173.200	4.030	697.590			
Live Load	19.200	4.000	76.800			
Earth Pressure				-57.780	2.930	-169.240
Bridge	72.000	1.750	126.000			
Total	328.740		1070.890	-57.780		-169.240

(Calculated about 2.4m span)

Total of Moment $\Sigma M=$ 901.650 (tf•m)
Total of Vertical Force $\Sigma V=$ 328.740 (tf)
Total of Horizontal Force $\Sigma H=$ -57.780 (tf)

Stability Against Overturning and Soil Reaction
Case-1 : After Construction
$\chi = (\Sigma M/\Sigma V) =$ 2.740 (m)
Width of Invert $L =$ 6.00 (m)
Eccentric Length $e =$ 0.260 (m) $<$ $L/6 =$ 1.000 (m)
Soil Reaction $Q_1 =$ 28.760 (tf/m^2)
$Q_2 =$ 16.890 (tf/m^2)
Case-2 : Under Construction
$\chi' = M/V =$ 2.650
Eccentric Length $e' =$ 0.350 $<$ $L/6 =$ 1.000 (m)
Soil Reaction $Q_1' =$ 6.032 (tf/m^2)
$Q_2' =$ 2.904 (tf/m^2)

Factor of Safety Against Sliding
Coefficient of friction $f=$ 0.600
Cohesion $C=$ 0.000 (tf/m^2)
Total of Vertical Force $\Sigma V=$ 328.740 (tf)
Width of Invert $L=$ 6.000 (m)
Total of Horizontal Force $\Sigma H=$ 57.780 (tf)

Factor of Safety $Fs=$ 3.41 = 1.50 OK

(3) Result of Structural Analysis

Table 2 Result of Structural Analysis

1. Calculation of stem

Height (m)	Bending Moment (tf·m)	Shearing Force (tf)	Effective Depth (m)	Required Effective Depth (m)	Required Area of Tension Reinforcement (cm^2)	Reinforcing Bar Schedule (cm^2)
1.00	0.730	1.520	0.430	0.070	1.090	3-D16=6.03
3.00	1.420	2.960	0.430	0.100	2.130	3-D16=6.03
6.92	2.770	5.780	0.430	0.140	4.160	3-D16=6.03

2. Calculation of Counterfort

Height (m)	Bending Moment (tf·m)	Shearing Force (tf)	Effective Depth (m)	Required Effective Depth (m)	Required Area of Tension Reinforcement (cm^2)	Reinforcing Bar Schedule (cm^2)
2.50	3.640	3.540	0.460	0.160	5.100	2-D22=7.60
6.00	33.570	14.790	2.550	0.470	7.330	3-D22=11.40
6.92	49.060	18.960	3.160	0.570	8.620	3-D22=11.40

3. Calculation of Toe

Height (m)	Bending Moment (tf·m)	Shearing Force (tf)	Effective Depth (m)	Required Effective Depth (m)	Required Area of Tension Reinforcement (cm^2)	Reinforcing Bar Schedule (cm^2)
0.50	2.010	8.570	0.930	0.120	1.390	3-D16+ 3-D19=14.54
1.50	18.090	24.710	0.930	0.350	12.540	3-D16+ 3-D19=14.54

4. Calculation of Heel

Height (m)	Bending Moment (tf·m)	Shearing Force (tf)	Effective Depth (m)	Required Effective Depth (m)	Required Area of Tension Reinforcement (cm^2)	Reinforcing Bar Schedule (cm^2)
0.00	0.840	1.740	0.930	0.070	0.510	3-D16=6.03
2.00	0.980	2.030	0.930	0.080	0.610	3-D16=6.03
4.00	1.110	2.320	0.930	0.120	0.700	3-D16=6.03

5. Calculation of Connection of Stem and Counterfort

Height (m)	Shearing Force (tf)	Required Area of Tension Reinforcement (cm^2)	Reinforcing Bar Schedule (cm^2)
2.50	2.600	2.890	3-D16=6.03
6.00	5.110	5.680	3-D19=8.51
6.92	5.780	6.420	3-D19=8.51

6. Calculation of Connection of Heel and Counterfort

Height (m)	Shearing Force (tf)	Required Area of Tension Reinforcement (cm^2)	Reinforcing Bar Schedule (cm^2)
0.00	1.740	1.940	3-D16=6.03
4.00	2.320	2.580	3-D16=6.03