

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 - 2

DESIGN NOTES

BRIDGES

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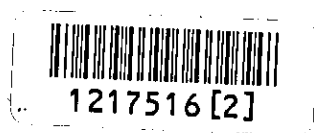
VOL. III - 2

DESIGN NOTES

BRIDGES

OCTOBER 1996

CTI ENGINEERING CO., LTD.



LIST OF REPORTS

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VOLUME II	MAIN REPORT
VOLUME III	DESIGN NOTES
VOLUME IV	WORK QUANTITIES
VOLUME V	COST ESTIMATE
VOLUME VI	DATA BOOK

**THE DETAILED DESIGN STUDY
ON MEDAN FLOOD CONTROL PROJECT**

FINAL REPORT

VOL. III - 2

DESIGN NOTES

BRIDGES

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ANNEX

Annex

DESIGN CRITERIA

4. BRIDGE WORKS

4.1 ROAD BRIDGE

4.1.1 LOCATION AND MAJOR DIMENSIONS OF BRIDGE

The detailed design was made for the following road bridges including pedestrian and railway bridges.

Road and Pedestrian Bridge in Percut River

No	Name of Bridge	Location	Length of Bridge (m)			Width (m)	Remarks
			Left	Center	Right		
P1	Titi Besi	PE57+5	25.6	31.6	25.6	7.0	Road
P2	Perkebunan	PE84+28	31.6	40.8	31.6	7.0	Road
P3	Titi Gantung	PE115+5	16.6	40.8	-	7.0	Road
P5	Payung	PE137+45	-	40.8	-	7.0	Road
P6	Pedestrian	PE147+58	-	40.8	-	2.0	Pedestrian
P7	Medan Tembung	PE169+59	-	40.8	-	9.0	Road
P9	Medan Denai	PE200+25	-	40.8	-	16.0	Road
P11	Binjei	PE222+0	-	40.8	-	16.0	Road
P13	Amplas	PE246+57	-	40.8	-	16.0	Road

Road, Pedestrian and Railway Bridge in Medan Floodway

No	Name of Bridge	Location	Length of Bridge (m)			Width (m)	Remarks
			Left	Center	Right		
F1	Jalan Bajak	FW6+90	-	31.6	-	7.0	Road
F2	PTP IX	FW20+45	-	31.6	-	9.0	Road
F3	JL. STM Ujung	FW28+22	-	31.6	-	9.0	Road
F4	Railway	FW32+0	-	31.6	-	3.5	Railway
F5	JL.Deli Tua	FW33+65	-	31.6	-	16.0	Road
F6	Pedestrian	FW37+60	-	31.6	-	3.0	Pedestrian
F7	JL. SMA-12	FW38+78	-	19.6	-	4.5	Road
F8	Gg.Seksuma	UD19+0	13.6	31.6	13.6	2.0	Pedestrian

The detailed design was made for the superstructure and substructure of respective bridges. Each bridge was categorized by the span length and bridge width, and the design contents are presented. The category of bridge is as follows:

Bridge Span (m) (Clear Span of Beam)	Bridge Width (m)
40.0	16.0 (Road)
31.0	9.0 (Road)
25.0	7.0 (Road)
19.0	4.50 (Road)
16.0	3.5 (Railway)
13.0	2.0 (Pedestrian)

The design results of superstructure are compiled for the following six (6) cases depending on the length of beam.

Superstructure

Case-A PC Beam 40.0 m long
Case-B PC Beam 31.0 m long
Case-C PC Beam 25.0 m long
Case-D PC Beam 19.0 m long
Case-E PC Beam 16.0 m long
Case-F PC Beam 13.0 m long

On the other hand, the design of substructure were performed for abutment and pier structures, and pile foundation. The results are compiled as follows:

Substructure

Case	Bridge Span (m)	Bridge Width (m)
Case-1	40.0	16.0
	40.0	9.0
	40.0	7.0
Case-2	31.0	16.0
	31.0	9.0
	31.0	7.0
Case-3	25.0	7.0
Case-4	19.0	4.5
Case-5	16.0	7.0
Pedestrian	31.0/16.0	2.0

4.1.2 SUPERSTRUCTURE (PC BEAM 40.0M LONG)

**PRESTRESSED CONCRETE BRIDGE COMPUTATION
CHANNEL PERCUT RIVER**

Br - P7

Station : Pe 169 + 59

span
40.00

I SUPERSTRUCTURE

- I.1 Post railing**
- I.2 Side walks**
- I.3 Slab deck**
- I.4 Beam**
 - I.4.1 Section properties of beam**
 - I.4.2 Loading on a beam**
 - I.4.3 Prestress force and number of tendon available**
 - I.4.4 Prestress loss**
 - I.4.5 Centre gravity of tendon**
 - I.4.6 Stresses control**
 - I.4.7 Layout of tendons**
 - I.4.8 Shear force and tensile stress**
 - I.4.9 Deflection**
- I.5 Loading on Abutment and Pier**

Project : Medan Flood Control Channel : Percut River Structure: Br - P 7 Station : PE 169 + 59		Japan International Cooperation Agency (JICA)		Calculation sheet Date Nos of sheet Prepared by	
I. SUPERSTRUCTURE I.1 Post railing					
arm force moment shear	= = = =	0.9 m 0.1 ton 0.1 x 0.1 ton	0.9 = 0.09 t.m		
description					
allowable working stress		value		unit	
concrete compressive		f_c' $f_c = 0.33 \times f_c''$	225 15.00	kg/cm2	
steel tensile		f_s	75	kg/cm2	
concrete shear without steel bar		$tou = 0.54 \times \sqrt{f_c'}$	1,400	kg/cm2	
concrete shear with steel bar		$tou = 1.35 \times \sqrt{f_c'}$	8	kg/cm2	
modulus of elasticity steel		E_s	20	kg/cm2	
modulus of elasticity concrete		$E_c = 6400 \times \sqrt{f_c'}$	2,100,000	kg/cm2	
modular ratio		$n = E_s/E_c$	96,000	kg/cm2	
forces		M	0.09	t.m	
moment (kg.cm)		S	9,000	kg.cm	
shear (kg)			0.1	ton	
			100	kg	
dimension of section					
width of section		b	15	cm	
depth of section		ht	24.00	cm	
distance of compression steel to top fiber extrem		d'	2.5	cm	
effective height of section		h	21.50	cm	
area of section		b.h	323	cm2	
Ca = section coefficient		$n.M$	196.875	kg.cm	
		$b.f_s$	21,000	kg/cm	
		$n.M/(b.f_s)$	9.38		
		$Ca = h / \sqrt{n.M/(b.f_s)}$	3.06		
		$n.f_c$	7.022		
		$ao = f_s/(n.f_c)$	1.641		
		delta	0.8533	kg/cm2	
			0.20	assumed	
Calculation of steel bar area		Ca	7.053		
		$a > ao$	4.405		
		psi	9.588		
		100 mw	0.938		
		rw	2.144		
		w	0.02144		
A = w.bh A' = delta.A		0.001 x 0.20 x	323 = 0.316 0.315 = 0.063	cm2	
steel bar full working					
compressive concrete		$n.a$	96.36		
tensile bar		$f_c' = f_s / (n.a)$	14.53 < 74.99	kg/cm2 ok	
		$f_s' = f_s / a'$	146.02 < 1400	kg/cm2 ok	
		$7/8 \times b \times h$	282.19	cm2	
shear of concrete		$tou ca = S / (7/8 b.h)$	0.35 < 8	kg/cm2 ok	
area of tensional steel bar					
A = used steel bar dia. 13 mm					
area @ bar dia. 13 mm		=	1.267 2 nos	cm2,	
total area		=	2.534 > 0.32	cm2 ok	
area of compressive steel bar					
A' = delta.A		=	0.063 cm2	cm2	
A' = used steel bar dia. 13 mm					
area @ bar dia. 13 mm		=	1.267 2 nos	cm2,	
total area		=	2.534 > 0.06	cm2 ok	

Project : Medan Flood Control Channel : Perut River Structure: Br -- P 7 Station : Pe 159 + 69		Japan International Cooperation Agency (JICA)		Calculation sheet Date Nos of sheet Prepared by		
I.2. Side Walk						
a. Dead load						
description	width m	length m	depth m	Yc t/m ³	weight ton	
a). hand railing						
up hand railing	P1 = 0.150	0.180	0.550	2.400	0.036	
down hand railing	P2 = 0.150	0.210	0.450	2.400	0.034	
slab Rail	P3 = 0.100	1.000	0.450	2.400	0.108	
b). slab						
slab side walk (1)	P4 = 1.00	1.410	0.150	2.400	0.508	
slab side walk (2)	P5 = 1.00	0.623	0.050	2.400	0.075	
slab deck	P6 = 1.00	0.300	0.275	2.400	0.198	
total shear					0.958	
Moment to point A						
a by dead load						
description	Load ton	Distance m	Moment t.m			
up hand railing	P1 = 0.036	1.410	0.050			
down hand railing	P2 = 0.034	1.350	0.046			
slab rail	P3 = 0.108	1.405	0.152			
slab side walk (1)	P4 = 0.508	0.705	0.358			
slab side walk (2)	P5 = 0.075	0.670	0.050			
slab deck	P6 = 0.198	0.150	0.030			
Total moment by dead load (dl)			0.686			
b. Live load						
due to pedestrian	= 0.500 ton/m ²					
due to railing force	= 0.100 ton					
due to curb force	= 0.500 ton					
c. Live load						
description	Load ton	Distance m	Moment t.m			
pedestrian	= 0.500	1.254	0.314			
railing force	= 0.100	0.900	0.090			
curb force	= 0.500	0.250	0.125			
sum shear by live load (ll)	= 1.600					
sum moment by live load (ll)			0.529			
total shear (dl + ll)	= 1.600 +	0.958	= 2.558 ton			
total moment (dl + ll)	= 0.686 +	0.529	= 1.214 ton.m			

side-wk

Project : Medan Flood Control Channel : Perent River Structure : Br - P 7 Station : Pe 169 + 59	Japan International Cooperation Agency (JICA)	Calculation Date Nos of sheet Prepared by
1.3. Slab deck		
a. Data		
thick of slab = 0.175 m		
thick of asphalt = 0.050 m		
distance each beam = lx = 1.750 m		
dead load		
Uraian	width x thick x length	Unit weight = Weight t/m ³ = t/m ²
slab deck		
asphalt	= 1.000 x 0.050 x 1.000 x	2.300 = 0.115
slab	= 1.000 x 0.175 x 1.000 x	2.500 = 0.438
		g = 0.553
Fixed End Moment		
Mge	= Mge	
	= 1/10 x g x lx ²	
	= 0.100 x 0.553 x 3.063	
	= 0.169 t.m	
Span moment		
Mgs	= Mgs	
	= 1/12 x g x lx ²	
	= 0.083 x 0.553 x 3.063	
	= 0.141 t.m	
Shear by dead load		
	= 1/2 x g x lx	
	= 0.500 x 0.553 x 1.750	
	= 0.483 ton	
Live Load		
T	= 10.000 ton	
a	= 0.200 m	
b	= 0.500 m	
a1	= 0.200 + (2.000 x (0.050 + 0.088)) = 0.475 m	
b1	= 0.500 + (2.000 x (0.050 + 0.088)) = 0.775 m	
width of wheel effective		
lx	= 1.750 m	
ly	= 31.000 m	
r	= 0.500	
3.r	= 1.500	
ly/lx	= 17.714	
	> 3.r	
	3/4 . a1 + 3/4 . r lx	
Sa	= 3/4 x a1 + 3/4 x r lx	
	= 0.750 x 0.475 + 0.750 x 0.875	
	= 1.013 m	
Area of parabola		
Ap	= Ap	
	= 4/3 x lx/2 x (Sa - a1) / 2	
	= 1.333 x 0.875 x 0.269	
	= 0.314 m ²	
Area of tyred contact = Ac		
Ac	= 2 x Ap + lx . a1	
	= 2.000 x 0.314 + 0.831	
	= 1.458 m ²	
slab		

q	T	10.000	=	6.857 t/m ²
	Ac		=	1.458
Moment maximum	Mo			
	= 0.125 x	6.857 x	3.063	
	= 2.625 t.m			
Fixed End Moment	Mle			
	= 1/10 x	q x	lx ²	
	= 0.100 x	6.857 x	3.063	
	= 2.100 t.m			
Span moment	Mls			
	Mo	2.625	=	2.593 t.m
	Sa	1.013	=	
Shear by live load				
	= 1/2 x	q x	lx	
	= 0.500 x	6.857 x	1.750	
	= 6.000 ton			
Total moment				
Fixed end moment	Mge	+	Mle	
	= 0.169 +	2.100		
	= 2.269 t.m			
Span moment	Mgs	+	Mls	
	= 0.141 +	2.593		
	= 2.734 t.m			
Total shear				
Shear by dead load		=	0.483 ton	
Shear by live load		=	6.000 ton	
Total		=	6.483 ton	
Reinforcing Bar				
Allowance stresses				
concrete comp	fc	K - 350	kg/cm ²	115.5
steel tensile	fs		kg/cm ²	1400
concrete shear	without steel bar		kg/cm ²	8.04
ton with steel bar			kg/cm ²	20.20
E steel			kg/cm ²	1,897,200
E concrete			kg/cm ²	160,577
transformand n = Es/Ec				11.81
Moment (kg.c	M		t.m	2.734
Shear (kg)	S		kg.cm	273,360
			ton	6.483
Normal (kg)	N		kg	6,483
			ton	0
dimension	b		kg	0
	ht		cm	100
	d'		cm	17.50
	h		cm	5
	b.h		cm ²	12.50
	a.M		cm ²	1,250
	b.fs		kg.cm	3,229,720
			kg/cm	140,000
slab				

$$\begin{aligned} \text{bottom fiber } k_b &= \frac{Wt}{Ac} = \frac{284,386}{6,835.00} = 41.61 \text{ cm} \end{aligned}$$

Composite section

width of flange = (be)
ACI

Effective flange width (be) lesser than	
1	S = 140.0 cm
2	1/4 = 1,000.0 cm
	bw = 20.0 cm
	ht = 17.5 cm
3	bw + 16 ht = 300.0 cm

be = 140.0 cm
1 Mpa = 10.2 kg/cm²
1 kg/cm² = 0.0980 Mpa

Width of transformed flange = bt
Concrete Class of deck K - 350

= 350 kg/cm²
= 34.31 Mpa

Concrete Class.

Precast Prestressed K - 400 = 400 kg/cm²
Concrete specific = 39.22 Mpa

Ratio of moduli (n) = 350
= 400

bt = n x be
bt = 0.88 x 140
say = 122.50 cm
= 130.0 cm

Section properties of Composite section moment static to bottom fiber

part	depth t	width w	area Ai'	distance to Yb	AI',Yi'	AI',Yi'^2
	cm	cm	cm ²	cm	cm ³	cm ⁴
(1)	(2)	(3)	(4)	(5)	(6)	(8)
slab	17.5	130.0	2,275.0	185.8	422,581.3	78,494,467.2
panel	7.0	130.0	910.0	173.5	157,885.0	27,393,047.5
beam			6,835.0	83.4	570,018.3	47,537,805.5
total	24.5		10,020.0		1,150,484.6	

Ii'	Ib'
cm ⁴	cm ⁴
(9)	(10)
1/12 b h ^ 3	(8) + (9)
58,060	78,552,527
3,716	27,396,763
24,628,730	72,166,536
	178,115,826

$$\begin{aligned} Yb' &= \frac{Sx'}{Ac'} = \frac{1,150,485}{10,020.00} = 114.82 \text{ cm} \end{aligned}$$

$$\begin{aligned} Yi' &= Ht + ht - Yb' = 170.00 + 24.50 - 114.82 = 79.68 \text{ cm} \\ Ix' &= \text{tot. Ixi'} - \text{tot. Ai'} \times Yb'^2 = 178,115,826 - 10,020 \times 13,183 = 178,105,806 \\ &= 46,008,523 \text{ cm}^4 \end{aligned}$$

Section modulus

$$\begin{aligned} \text{top fiber } Wt' &= \frac{Ix'}{Yt'} = \frac{46,008,523}{79.7} = 577,407.7 \text{ cm} \\ \text{bottom fiber } Wb' &= \frac{Ix'}{Yb'} = \frac{46,008,523}{114.8} = 400,705.4 \text{ cm} \end{aligned}$$

Kern

$$\begin{aligned} \text{top fiber } kb' &= \frac{Wb'}{Ac'} = \frac{400,705.4}{10,020.0} = 40.0 \text{ cm} \\ \text{bottom fiber } kb' &= \frac{Wt'}{Ac'} = \frac{577,407.7}{10,020.0} = 57.6 \text{ cm} \end{aligned}$$

1.4.2. Load on a beam

Span = 40.80 m thickness of asphalt = 0.050 m
Clear span = 40.00 m thickness of slab = 0.245 m
nos of beam = 7.00 unit weight of asphalt = 2.300 t/m³
total weight cross beam = 10.56 ton unit weight of concrete = 2.500 t/m³

Width effective of beam
be = 1.40 m

description	width m	x	thick m	area m ²	x	length m	x	unit weight t/m ³	weight t/m ²
a.1 Beam	gb			0.6835 x	1.00 x	2.500 =	1.709		
a.2 Cross beam				10.56 : (	7.00 x	40.00) =	0.038		
a.3 Slab	gs	1.40 x	0.245 x	0.34 x	1.00 x	2.500 =	0.857		
a.4 Asphalt	ga	1.40 x	0.050 x	0.07 x	1.00 x	2.300 =	0.161		

b. Live load

b.1 Line load = p

Impact coefficient

$$k = 1 + \frac{20}{(50 + L)}$$

p = 12.00 ton/lane

be

2.75

1.40

2.75

12.00 x 1.222

7.467 t/m'

Uniform Load = q

Span

Q = 2.20 ton / lane

40.00 < 30.00 m

Q = 2.20 ton / lane

1.10

Q = 2.20 -

60x(L - 30)

60x(L - 30)

60.00 x 10.00

600.00

2.20 -

1.10

600

2.20 - 0.0018

2.198 t/m'

be

q =

2.75

Q

2. Influence line of moment and moment

2.1 Beam
Length of span
Interval distance
g = 1.746 t/m'
= 40.00 m
= 2.50 m

Point	Distance m	Load t/m'	Reaction ton	I.L. Moment t.m	Area of I.L. m ²	Moment t.m
1	2.50	1.746	0.938	2.34	46.88	81.866
2	5.00	1.746	0.875	4.38	87.50	152.816
3	7.50	1.746	0.813	6.09	121.88	212.850
4	10.00	1.746	0.750	7.50	150.00	261.970
5	12.50	1.746	0.688	8.59	171.88	300.174
6	15.00	1.746	0.625	9.38	187.50	327.462
7	17.50	1.746	0.563	9.84	196.88	343.835
8	20.00	1.746	0.500	10.00	200.00	349.293

2.2 Slab
gs = 0.857 t/m'

Point	Distance m	Load t/m'	Reaction ton	I.L. Moment t.m	Area of I.L. m ²	Moment t.m
1	2.50	0.857	0.938	2.34	46.88	40.195
2	5.00	0.857	0.875	4.38	87.50	75.031
3	7.50	0.857	0.813	6.09	121.88	104.508
4	10.00	0.857	0.750	7.50	150.00	128.625
5	12.50	0.857	0.688	8.59	171.88	147.303
6	15.00	0.857	0.625	9.38	187.50	160.781
7	17.50	0.857	0.563	9.84	196.88	166.820
8	20.00	0.857	0.500	10.00	200.00	171.500

2.3 asphalt
gas = 0.161 t/m'

Point	Distance m	Load t/m'	Reaction ton	I.L. Moment t.m	Area of I.L. m ²	Moment t.m
1	2.50	0.161	0.938	2.34	46.88	7.547
2	5.00	0.161	0.875	4.38	87.50	14.087
3	7.50	0.161	0.813	6.09	121.88	19.622
4	10.00	0.161	0.750	7.50	150.00	24.150
5	12.50	0.161	0.688	8.59	171.88	27.672
6	15.00	0.161	0.625	9.38	187.50	30.187
7	17.50	0.161	0.563	9.84	196.88	31.697
8	20.00	0.161	0.500	10.00	200.00	32.200

2.4 uniform load
q = 1.119 t/m'

Point	Distance m	Load t/m'	Reaction ton	I.L. Moment t.m	Area of I.L. m ²	Moment t.m
1	2.50	1.119	0.938	2.34	46.88	52.456
2	5.00	1.119	0.875	4.38	87.50	97.918
3	7.50	1.119	0.813	6.09	121.88	136.386
4	10.00	1.119	0.750	7.50	150.00	167.860
5	12.50	1.119	0.688	8.59	171.88	192.340
6	15.00	1.119	0.625	9.38	187.50	209.825
7	17.50	1.119	0.563	9.84	196.88	220.316
8	20.00	1.119	0.500	10.00	200.00	223.613

Project : Medan Flood Control Channel : Percut River Structure : Br - P 7 Station : Pe 169 + 59	Japan International Cooperation Agency (JICA)	Calculation sheet Date Nos of sheet Prepared by
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1.4.3 Prestress Force and Numbers of Tendon available

Mg	=	350,550 Lm
M after transfer	=	35,055,000.000 kg.cm
M dead load	=	35,055,000.000 kg.cm
M composite	=	522,050 Lm
	=	52,205,000 kg.cm
	=	330,680 Lm
	=	33,068,000 kg.cm

Characteristic of Concrete

Compression strength		
f _{cu} at transfer	N'	0 kg/cm ²
	f _{cu} '	190 kg/cm ²
	f _{cu} '	0 kg/cm ²
	f _{cu} '	190 kg/cm ²
	1 Mpa	10.2 kg/cm ²
	1 kg/cm ²	0.0980 Mpa
	1 kN	102 kg
	1 kg	0.0098 kN

Section properties

Precast Prestressed Beam	Composite Section
d	thick of slab = 17.50 cm
A	d' = 195 cm
S _x	Ac' = 9,773.0 cm ²
Y ₁	S _x ' = 1,105,133.9 cm ³
Y ₂	Yb' = 113.13 cm
I _x	Yt' = 81.37 cm
W ₁	I _x ' = 44,154,355.9 cm ⁴
W ₂	Wb' = 551,233.00 cm ³
I _y	I _y ' = 396,491.00 cm ⁴
I _y	I _y ' = 40.40 cm ³
I _y	I _y ' = 41.61 cm
I _y	I _y ' = 59.40 cm ³
I _y	I _y ' = 0.75

1. Initial case

PI	>	f _{cu} Wt - Mg
		(kb - e)
Mg	=	35,055,000 kg.cm
PI	=	0.00 kg/cm ²
Wt	=	284,386.81 cm ³
f _{cu} Wt	=	0.00 cm ³
kb	=	41.61 cm
f _{cu} Wt - M min	=	0.00 - 35,055,000 = (35,055,000) kg.cm
kb - e	=	41.61 - 70.90 = (29.29) cm
PI	>	(35,055,000) = 1,196,720 (29.29)
PI	>	1,196.72 ton

Computation prestressing force

1st equation top fiber

2.5 line load	P = 7.47 t/m'
---------------	---------------

Point	Distance m	Load t/m'	Reaction ton	I.L Moment t.m	Moment Lm
1	2.50	7.467	0.938	2.34	17,500
2	5.00	7.467	0.875	4.38	32,667
3	7.50	7.467	0.813	6.09	45,500
4	10.00	7.467	0.750	7.50	56,000
5	12.50	7.467	0.688	8.59	64,167
6	15.00	7.467	0.625	9.38	70,000
7	17.50	7.467	0.563	9.84	73,500
8	20.00	7.467	0.500	10.00	74,667

(1). Initial case by self weight of beam = Mg

Distance	Mg
2.50	61,566
5.00	152,816
7.50	212,850
10.00	261,970
12.50	300,174
15.00	327,462
17.50	343,635
20.00	349,293

(2). Transfer case (Mg + M slab) = Mdl

Distance	Mg	M slab	Mdl = Mg + Ms
2.50	61,67	40.20	122,061
5.00	152,82	75.03	227,847
7.50	212,85	104.51	317,358
10.00	261,97	128.63	390,595
12.50	300,17	147.38	447,556
15.00	327,46	160.78	488,243
17.50	343,64	168.82	512,655
20.00	349,29	171.50	520,793

(3). Final case (composite section)

Distance	Mg	Mp	Ml	Ma	Mc = Ma + Ml
2.50	52.46	17.50	69.96	7.55	77,503
5.00	97.92	32.67	130.59	14.09	144,673
7.50	136.39	45.50	181.89	19.82	201,508
10.00	167.86	56.00	223.86	24.15	249,010
12.50	192.34	64.17	256.51	27.67	284,178
15.00	209.83	70.00	279.83	30.19	310,013
17.50	220.32	73.50	293.82	31.70	325,513
20.00	223.81	74.67	298.48	32.20	330,680

Mg = moment support by precast concrete prestressed
Mdl = moment support by precast concrete prestressed (beam + concrete insitu + cross beam + side walks)
Ml = moment support by composite section (DI + LL)
Mc = moment support by composite section (DI + LL + Asphalt)

3. Influence line of shear and shear

Load	Distance m	Reaction ton	Area of I.L m ²	Shear ton
3.1 self weight	1.746	0.00	1.000	34,929
3.2 slab	0.857	0.00	1.000	20,00
3.3 asphalt	0.161	0.00	1.000	17,150
3.4 uniform load	1.119	0.00	1.000	3,220
3.5 line load	7.467	0.00	1.000	22,381
				7,467
				85,147

2nd equation bottom fiber		by self weight of beam	
PI	<	$f_{cl}W_b + M_g$	$n(e + kt)$
Mg	=	35,055,000 kg.cm	
fcl	=	180.00 kg/cm	
Wb	=	295,318.87 kg/cm ²	
fclWb	=	58,110,598.16 cm ³	
kt	=	43.21 cm ³	
•	=	70.90 cm	
M min + fclWb	=	35,055,000 + 58,110,598 = 93,165,598 kg.cm	
kt + •	=	43.21 + 70.9 = 114.11 cm	
PI	<	93,165,598	
PI	<	114.11	
PI	<	798.85 ton	
1st equation top fiber		2. After transfer	
Top fiber		after prestress losses	
PI	>	$f_{cl}W_t - M_g$	$n(kb - e)$
Mg	=	35,055,000 kg.cm	
fcl	=	0.00 kg/cm ²	
Wt	=	284,386.81 cm ³	
fclWt	=	0.00 cm ³	
kb	=	41.61 cm	
•	=	70.90 cm	
(kb - e)	=	41.61 - 70.90 = -29.29 cm	
fclWt - M_g	=	0.00 - 35,055,000 = -35,055,000 cm	
n(kb - e)	=	0.75 x (29.29) = -21.97 cm	
PI	>	(35,055,000)	
PI	>	-21,969,425	
PI	>	1,595.63 ton	
2nd equation bottom fiber		after prestress losses	
PI	<	$f_{cl}W_b + M_g$	$n(e + kt)$
Mg	=	35,055,000 kg.cm	
fcl	=	180.00 kg/cm	
Wb	=	295,318.87 kg/cm ²	
fclWb	=	190 x 295,318.87 = 54,110,596 cm	
kt	=	43.21 cm ³	
•	=	70.90 cm	
(kt + e)	=	43.21 + 70.90 = 114.11 cm	
fclWb + M_g	=	58,110,596 + 35,055,000 = 93,165,596 cm	
n(kt + e)	=	0.75 x 114.11 = 85.58 cm	
PI	<	93,165,596	
PI	<	85.58	
PI	<	1,065.27 ton	

3. Dead load		1st equation top fiber	
PI	>	$f_{cl}W_t - M_{dl}$	$n(kb - e)$
Mdl	=	52,205,000 kg.cm	
fcl	=	0.00 kg/cm ²	
Wt	=	284,386.81 cm ³	
fclWt	=	0 x 284,387 = 0.00 cm	
kb	=	41.61 cm	
•	=	70.90 cm	
(kb - e)	=	41.61 - 70.90 = -29.29 cm	
fclWt - Mdl	=	0 - 52,205,000 = -52,205,000 kg.cm	
n(kb - e)	=	0.75 x (29.29) = -21.97 cm	
PI	>	(52,205,000)	
PI	>	-21,97	
PI	>	2,376.26 ton	
2nd equation bottom fiber		by beam and concrete inertia	
PI	<	$f_{cl}W_b + M_{dl}$	$n(e + kt)$
Mdl	=	52,205,000 kg.cm	
fcl	=	0.00 kg/cm	
Wb	=	295,318.87 kg/cm ²	
fclWb	=	0 x 295,318.87 = 0.00 cm	
kt	=	43.21 cm	
•	=	70.90 cm	
(e + kt)	=	70.9 + 43.21 = 114.11 cm	
fclWb + Mdl	=	0 + 52,205,000 = 52,205,000 cm	
n(e + kt)	=	0.75 x 114.11 = 85.58 cm	
PI	<	52,205,000	
PI	<	85.58	
PI	<	610.01 ton	
4. Service load		3rd equation top fiber	
PI	<	$f_{cl}W_t - M_{dl} - M_{s}W/Wt$	$n(kb - e)$
Mdl	=	33,068,000 kg.cm	
Mdl	=	52,205,000 kg.cm	
fcl	=	180.00 kg/cm ²	
Wt	=	284,386.81 cm ³	
Wt	=	551,233.00 cm ³	
Wb	=	551,233.00 cm ³	
kb	=	41.61 cm	
•	=	70.90 cm	
(kb - e)	=	41.61 - 70.90 = -29.29 cm	
W/Wt	=	284,386.81 : 551,233.00 = 0.52	
Mdl/Wt	=	33,068,000.00 x 0.52 = 17,394,124 kg.cm	
fclWt	=	180.00 x 284,386.81 = 51,189,626 kg.cm	
fclWt - Mdl	=	51,189,626.02 - 33,068,000 = 18,121,626 kg.cm	
fclWt - Mdl - Mdl/Wt	=	(18,121,626 kg.cm) - 17,394,124 = 727,502 kg.cm	

$$n.(k_b - e) = 0.75 \times (29.29) = (21.97) \text{ kg/cm}$$

$$P_i < \frac{(18,075,497.51)}{(21.97)} = 822,756.99$$

$$P_i < 822.76 \text{ ton}$$

4th equation bottom fiber

$$P_i > \frac{f_{1a} W_b + M_{dl} + M_c W_b / W_b'}{n.(e + k_t)}$$

by service load

Mc	=	33,068,000.00 kg.cm	
Mdl	=	52,205,000.00 kg.cm	
f _{1a}	=	0.00 kg/cm ²	
W _b	=	551,233.00 cm ³	
W _b	=	295,318.87 cm ³	
W _b	=	398,491.00 cm ³	
k _t	=	43.21 cm	
e	=	70.90 cm	
(e + k _t)	=	70.90 + 43.21 = 114.11 cm	
W _b /W _b '	=	295,318.87 : 398,491.00 = 0.74	
M _c W _b /W _b '	=	33,068,000 x 0.74 = 24,430,078.71 kg.cm	
f _{1a} W _b	=	0 x 295,318.87 = 0 kg.cm	
f _{1a} W _b + M _c	=	0 + 24,430,078.71 = 24,430,078.71 kg.cm	
n.(e + k _t)	=	522,050.00 + 24,430,078.71 = 24,952,128.71 kg.cm	
	=	0.75 x 114.11 = 85.58 cm	

$$f_{1a} W_b + M_{dl} + M_c W_b / W_b'$$

$$P_i > \frac{76,835,079}{85.58} = 897,814 \text{ kg}$$

Initial case

Top fibre	P _i	>	1,196.72 ton
Bottom fib	P _i	<	798.95 ton

Transfer

Top fibre	P _i	>	1,595.83 ton
Bottom fib	P _i	<	1,065.27 ton

Dead load

Top fibre	P _i	<	2,376.26 ton
Bottom fib	P _i	>	610.01 ton

Service case

Top fibre	P _i	<	622.76 ton
Bottom fib	P _i	>	897.81 ton

$$610.01 < P_i < 798.95 \text{ ton}$$

Tendon specification

P _i	=	798,949 kg
Area	=	0.842 cm ²
f _u	=	1,860 Mpa
	=	1,860 x 10.2

breaking load

P _u	=	A _s x f _u
	=	0.842 x 18,972
	=	17,872 kg / strand
	=	175 kN / strand

nos of strand tendon 18

nos of strand tendon 38

sum	=	19.00 per tendon
	=	12.00 per tendon
	=	31.00 nos of strand

$$\text{Jacking force } P_i - 19 = 0.80 \times P_u / \text{strand} \times \text{nos of strand}$$

$$= 0.8 \times 17,872 \times 19.00$$

$$= 271,849 \text{ kg / tendon}$$

$$= 2,663 \text{ kN / tendon}$$

$$P_i - 12 = 0.80 \times P_u / \text{strand} \times \text{nos of strand}$$

$$= 0.8 \times 17,872 \times 12.00$$

$$= 171,568 \text{ kg / tendon}$$

$$= 1,682 \text{ kN / tendon}$$

$$\text{initial force } P_i = 0.7 \times P_u$$

$$= 0.7 \times 17,872$$

$$= 12,510 \text{ kg / strand}$$

$$\text{Force support by tendon 19 strand} = \frac{19}{31} \times 798,949$$

$$= 489,678.58 \text{ kg}$$

$$\text{nos of tendon, strand 19} = \frac{\text{total force}}{P_i}$$

$$= \frac{489,679}{271,849} = 1.80$$

$$\text{say } 2.00$$

$$\text{Force support by tendon 12 strand} = \frac{12}{31} \times 798,949$$

$$= 309,271$$

$$\text{nos of tendon, strand 19} = \frac{\text{total force}}{P_i}$$

$$= \frac{309,271}{171,568} = 1.80$$

$$\text{say } 2.00$$

Using tendon with this number :

$$\text{number of tendon 1.2} = 2.00$$

$$\text{number of strand / tendon 1.2} = 19.00$$

$$\text{sum nos of strand 1.2} = 2.00 \times 19.00 = 38.00$$

P_i (1.2)

$$\text{number of tendon 3.4} = 2.00$$

$$\text{number of strand / tendon 3.4} = 12.00$$

$$\text{sum nos of strand 3.4} = 2.00 \times 12.00 = 24.00$$

P_i (3.4)

$$\text{sum } P_i = 0.7 \times 17,872 \times 31.00$$

$$= 300,243 \text{ kg / tendon}$$

$$= 475,385 + 300,243$$

$$= 775,628 \text{ kg}$$

pres-40

pres-40

Project : Medan Flood Control Channel : Percut River Structure : Br - P 7 Station : Pe 169 + 59		Japan International Cooperation Agency (JICA)		Calculation sheet Date Nos of sheet Prepared by	
1.4.4 Prestress losses					
1. Due to elastic shortening					
span		=		40.00 m	
section properties					
Ht	=	170.00 cm	e	=	66.40 cm
Ac	=	6,835.00 cm ²	Yt	=	86.60 cm
Ix	=	24,628,730 cm ⁴	Yb	=	83.40 cm
Wt	=	284,388 cm ³			
Wb	=	295,319 cm ³			
Concrete class K -400					
(fc)	=	400 kg/cm ²			
Nos strand / tendon 1 and 2	=	19.00 nos			
nos of tendon 1 and 2	=	2.00 nos			
Nos strand / tendon 3 and 4	=	12.00 nos			
nos of tendon 3 and 4	=	2.00 nos			
Area of strand	=	94.20 mm ²			
Total area	=	0.9420 cm ²			
Ec	=	343,327 kg/cm ²			
Es	=	1,897,200 kg/cm ²			
modular ratio					
n	=	-----	Es		
		Ec			
		1,897,200			
		343,327			
		5.53			
Allowance stress in prestressing tendon = fpu					
f _u	=	1,860 Mpa			
	=	1,860 x	10.2		
	=	18,972 kg/cm ²			
Stress at jacking					
	=	0.8 x	18,972		
	=	15,178 kg/cm ²			
Jacking force / strand					
	=	0.942 x	15,178		
	=	14,297 kg / strand			
Jacking force tendon 1 and 2					
	=	14,297 x	19 x	2	
	=	543,297 kg			
Jacking force tendon 3 and 4					
	=	14,297 x	12 x	2	
	=	343,135 kg			
total jacking force					
	=	543,297 +	343,135		
	=	886,433 kg / beam			
all the jacking					
tendon	Yb	Ys at midle	e	Pi	PI/AC (e Ac/Wb) (1+e Ac/Wb)
no	cm	cm	cm	kg	kg/cm ² kg/cm ²
4	83.40	20.00	63.4	886,433	6835
3	83.40	10.00	73.4	886,433	6835
2	83.40	10.00	73.4	886,433	6835
1	83.40	10.00	73.4	886,433	6835

plus-40

tendon		nos of tendon		= 4.00	
no	kg/cm ²	n	n.fc	kg/cm ²	Stress losses
4	319.99	5.53	1,768.25	15,178	n.fc / f x 100
3	350.01	5.53	1,934.12	15,178	11.65
2	350.01	5.53	1,934.12	15,178	12.74
1	350.01	5.53	1,934.12	15,178	12.74
average		49.880	4	12.4700	%
2. Due to friction					
fx	=	f x a ⁿ (-ua + Ks)			
	=	stress in the strand at the jacking end or live anchorage			
fj	=	stress at dead anchorage			
u	=	coefficient of angular friction			
alpha	=	change in the angle between the force at anchorage to section X			
x	=	curvilinear length of the tendon anchorage to section x			
f	=	depth of parabola			
tendon parabola curve					
Y	=	4 x f / l ² x (l-x)			
angle of tendon parabola to horizontal line					
dx/dy = 0					
x	=	0.00 m			
e	=	0.664 m			
l	=	40.00 m			
alpha 1	=	4 x f / l	radial		
alpha 1	=	4.00 x	0.664		
	=	40.00			
	=	0.0664	radial		
alpha	=	2.00 x	alpha 1		
	=	2.00 x	0.0664		
	=	0.1328			
no.tendon value of sag parabola each tendon					
		H	=	distance tendon to bottom	= f
4		1.350	=	0.200	= 1.150 m
3		1.000	=	0.100	= 0.900 m
2		0.650	=	0.100	= 0.550 m
1		0.350	=	0.100	= 0.250 m
no.tendon value of depth parabola each tendon					
alpha 1	=	4	x	f	= alpha 1 radial
alpha 1	=	4.00 x	1.150	40.00	= 0.115 radial
alpha 1	=	4.00 x	0.900	40.00	= 0.09 radial
alpha 1	=	4.00 x	0.550	40.00	= 0.055 radial
alpha 1	=	4.00 x	0.250	40.00	= 0.025 radial
if x = 0, alpha = alpha 1					
if x = l/2, alpha = 2 x alpha 1					

plus-40

tandon - 1	
	x (m)
	0.000
alpha	0.0250
u	0.2000
u.alhpa	0.0050
K	0.0041
Kx	0.0000
alpha + K X	0.0050
e	2.7183
g ~ - (u.alpha + K X)	2.7183
ix	0.9950
	0.0500
	0.2000
	0.0100
	0.0041
	0.0820
	0.0920
	0.3121
	0.9121

tandon no.	Σx	Σx^2
4	0.977	0.880
3	0.982	0.889
2	0.989	0.901
1	0.995	0.912
sum	3.943	3.582
average	0.986	0.895
average of Σx	0.986	0.895
fs jacking	0.9407	15.178
stress lost	16.135	957.25
percentage stress lost	15.177.6	6.307 %

	3. Due to anchorage set	
stress losses	=	delta / l x Es
Es	=	1,897,200 kg/cm ²
assumed slip	=	0.5 cm
stress	=	delta / l x Es
	=	0.50
	=	1,897,200 x 0.50
	=	4,000
	=	237 kg/cm ²
stress losses	=	237 x 100
	=	15,178
	=	1,563 %
total stress lost	=	12,470 + 6,307
	=	20.34 %
Initial forces / tendon 1 and 2	=	1.00 - 0.2034
P i (1 and 2)	=	0.797 x 543,297
	=	432,793 kg
Initial forces / tendon 3 and 4	=	1.00 - 0.2034
P i (3 and 4)	=	0.797 x 343,135
	=	273,343 kg
initial force after transfer	=	432,793 + 273,343
P initial	=	706,136 kg
P j	=	866,433 kg

Project : Medan Flood Control	Japan International Cooperation Agency (JICA)	Calculation sheet
Channel : Percut River		Date
Structure : Br - P 7		Nos of sheet
Station : Pe 169 + 59		Prepared by

1.4.5 Centre Gravity of tendon

cgs = distance from centre gravity of strand (cgs)
to the centre gravity of concrete (cgc)

dia. of strand = 12.70 mm

Ht = 170.00 cm

area of a strand = 94.20 mm²

area = 0.942 cm²

nos of strand = 7.00 per tendon

area of a tendon = 6.5940 cm²

Yt = 86.60 cm

Yb = 83.40 cm

cgs tendon

a. at stressing of tendon

when stressing tendon no.4

a.1 on edge of beam

tendon no	distance to each cm	distance to tendon no cgc cm	area of tendon As cm ²	static moment Y cm	As.Y cm ³
top fiber to no.4	35.00				
No.4 to No.3	35.00	No.4	6.5940 x	51.60	340.25
No.3 to cgc	16.60	No.3	6.5940 x	16.60	108.46
	51.600		13.188		448.711

Ys = ei2 = 449.71
13.188

a.2 on middle of beam

Strand no	distance to each cm	distance to bottom fiber cm	area of strand As cm ²	static moment Y cm	As.Y cm ³
No.4 to No.3	10.00	No.4	6.5940 x	20.00	131.88
No.3 to No.2	0.00	No.3	6.5940 x	10.00	65.94
No.2 to No.1	0.00	No.2	6.5940 x	10.00	65.94
No.1 to outside	10.00	No.1	6.5940 x	10.00	65.94
	20.00		26.376		329.700

Ys = 329.70
26.376

ei1 = Yb - Ys
83.40 - 12.50
70.90 cm

cgs-40

b. at transfer b.1 at edge of beam

Strand no	distance to each cm	Strand no	distance to bot. fiber cm	area of strand As cm ²	static moment Y cm	As.Y cm ³
top fiber	35.00	Ht	170.00			
no.4	35.00	no.4	135.00	0.942 x	135.00	127.17
no.3	35.00	no.3	100.00	0.942 x	100.00	94.20
no.2	35.00	no.2	65.00	0.942 x	65.00	61.23
no.1	35.00	no.1	30.00	0.942 x	30.00	28.26
bot.fiber	30.00	bot.fiber	0.00			
	170.00			3.768		310.860

Ys = 310.86
3.768

ei2 = Yb - Ys
83.40 - 82.50
0.90 cm

b.2 cgs tendon at middle of span

Strand no	distance to each cm	Strand no	distance to bot.fiber cm	area of strand As cm ²	static moment Y cm	As.Y cm ³
no.4 to no.3	10.00	No.4	20.00	0.942 x	20.00	18.840
no.3 to no.2	0.00	No.3	10.00	0.942 x	10.00	9.420
no.2 to no.1	0.00	No.2	10.00	0.942 x	10.00	9.420
no.2 to bot.	10.00	No.1	10.00	0.942 x	10.00	9.420
	20.00			3.768		47.100

Ys = 47.10
3.768

ei1 = Yb - Ys
83.40 - 12.50
70.90 cm

cgs-40

1.4.6 Stress Control Working_moment

M minimum	■	350,550	l.m
	■	35,055,000	kg.cm
M after transfer	■	350,550	l.m
	■	35,055,000	kg.cm
M dead load	■	\$22,050	l.m
	■	\$2,205,000	kg.cm
M composite	■	\$30,660	l.m
	■	\$3,066,000	kg.cm

Allowable stresses in prestressed concrete

Initial case		Final case	
f_i	f_f	f_i	f_f
190.0 kg/cm ²	180.0 kg/cm ²	190.0 kg/cm ²	180.0 kg/cm ²
0.0 kg/cm ²	0.0 kg/cm ²	0.0 kg/cm ²	0.0 kg/cm ²

Section properties

Girder	Precast Prestressed Beam	Composite Section	thick of slab	thick of parn	
d	=		170.00	cm	17.50 cm
A	=		6,835.00	cm ²	7 cm
Sx	=		591,931.00	cm ³	195 cm
Yt	=		86.60	cm	9,775.0 cm ²
Yb	=		83.40	cm	1,105,833.0 cm ³
Ix	=		24,628,730	cm ⁴	113.13 cm
Wt	=		284,366.61	cm ³	61.37 cm
Wb	=		295,316.87	cm ³	44,854,585.0 cm ⁴
k1	=		43.21	cm	551,239.00 cm ³
k2	=		41.61	cm	396,491.00 cm ³
kb	=		70.90	cm	40.60 cm ³
e	=			cm	56.40 cm ³
	=			cm	0.75

Computation of stresses

Centre gravity of steel		50 % total P jacking	
at jacking			
at edge of beam	=	34.10	cm
at middle of beam	=	70.90	cm
at transfer			
at edge of beam	=	0.90	cm
at middle of beam	=	70.90	cm
Force at jacking	=	Pj	kg
Pj	=	443,217	kg
Pj	=	706,136	kg
n.Pj = P	=	n	x
	=	0.75	x
	=	529,602	kg
450374			

Plack x a	Initial case	Moment by eccentricity tendon
■	443,217	x 70.90
■	31,424,050	kg.cm

Transfer		
P initial x e	=	706,135 x 70.90
	=	50,065,042 kg.cm

	P final x e	=	Final case =	529,602 x 37,548,762 kg.cm	70.90
Actually working stresses					
1. Initial case					
			<u>top fibre</u>		
f _i	Pj	-	Pj x e	Mg	>
	Ac	-	Wt	Wt	
	443,217	-	31,424,050	35,055,000	
	6,835	-	284,387	284,387	
	64.85	-	110.50	+ 123.27	>
	77.61	>	0 kg/cm ²		
			<u>bottom fibre</u>		
f _b	Pj	+	Pj x e	Mg	<
	Ac	+	Wb	Wb	
	443,217	+	31,424,050	35,055,000	
	6,835	+	295,319	295,319	
	64.85	+	106.41	- 118.70	<
	52.55	<	190 kg/cm ²		
2. after transfer					
			<u>top fibre</u>		
f _i	Pj	-	Pj e	Mtr	>
	Ac	-	Wt	Wt	
	706,136	-	50,065,042	35,055,000	
	6,835	-	284,387	284,387	
	103.31	-	176.05	+ 123.27	>
	50.53	>	0 kg/cm ²		
			<u>bottom fibre</u>		
f _b	P	+	P e	M tr	<
	Ac	+	Wb	Wb	
	706,136	+	50,065,042	35,055,000	
	6,835	+	295,319	295,319	
	103.31	+	169.53	- 118.70	<
	154.14	<	180.00 kg/cm ²		
3. Dead load (self weight beam + concrete cast in place)					
			<u>top fibre</u>		
f _i	P	-	P e	Mdl	>
	Ac	-	Wt	Wt	

Project : Medan Flood Control Channel : Percut River Structure : Br - P 7 Station : Pc 169 + 59		Japan International Cooperation Agency (JICA)		Calculation sheet					
		Date		Nos of sheet					
		Prepared by							
1.4.7 Layout of tendon									
description		Combined moment (t.m) at various length (m)							
		2.50	5.00	7.50	10.00	12.50	15.00	17.50	20.00
2.1.	beam	92.573	153.368	213.616	262.913	301.254	328.641	345.073	350.550
2.2	slab	40.195	75.031	104.508	128.625	147.383	160.781	168.820	171.500
2.3	asphalt	7.547	14.087	19.622	24.150	27.672	30.186	31.697	32.200
2.4	uniform load	52.458	97.918	136.388	167.860	192.340	209.825	220.316	223.813
2.5	line load	17.500	32.667	45.500	56.000	64.167	70.000	73.500	74.667
(1) M minimum = 2.1		92.573	153.368	213.616	262.913	301.254	328.641	345.073	350.550
(2) M dead load = 2.1 + 2.2		122.768	228.397	318.124	391.538	448.637	489.422	513.893	522.050
(3) M live load = 2.4+2.5		69.956	130.585	181.868	223.860	258.507	279.825	293.816	298.480
(4) M complete = (3) + 2.3		77.503	144.672	201.508	248.010	284.179	310.011	325.513	330.680
P jacking		= 886.433 kg							
P initial		= 685.492 kg							
n		= 0.75							
n.PI = P		= 514.119 kg							
case		Prestressing force (kg)							
		Pia		P-n.PI					
Jacking				886.433					
After transfer				685.492					
Dead load				514.119					
Live load				514.119					
Composite				514.119					
Arm		Moment							
		Force							
		at Jacking							
Distance	m	2.5	5	7.5	10	12.5	15	17.50	20.00
Mg	t.m	82.57	153.37	213.62	262.91	301.25	328.64	345.07	350.55
kg.cm	kg.cm	8,257,300	15,336,600	21,361,600	26,291,300	30,125,400	32,864,100	34,507,300	35,055,000
Prestressed force	kg	886.433	886.433	886.433	886.433	886.433	886.433	886.433	886.433
arm	cm	9.32	17.30	24.10	29.68	33.98	37.07	38.93	39.55
		After transfer							
Distance	m	2.5	5	7.5	10	12.5	15	17.50	20.00
M transfer	t.m	82.57	153.37	213.62	262.91	301.25	328.64	345.07	350.55
kg.cm	kg.cm	8,257,300	15,336,600	21,361,600	26,291,300	30,125,400	32,864,100	34,507,300	35,055,000
Prestressed force	kg	685.492	685.492	685.492	685.492	685.492	685.492	685.492	685.492
Arm	cm	12.05	22.37	31.16	38.35	43.95	47.94	50.34	51.14
		Dead load							
Distance	m	2.5	5	7.5	10	12.5	15	17.50	20.00
M dead load	t.m	122.77	228.40	318.12	391.54	448.64	489.42	513.89	522.05
kg.cm	kg.cm	12,276,800	22,839,700	31,812,400	39,153,800	44,863,700	48,942,200	51,389,300	52,205,000
Prestressed force	kg	514.119	514.119	514.119	514.119	514.119	514.119	514.119	514.119
Arm	cm	23.88	44.42	61.88	76.16	87.26	95.20	99.98	101.54
		Service load							
Distance	m	2.5	5	7.5	10	12.5	15	17.50	20.00
M complete	t.m	77.50	144.67	201.51	248.01	284.18	310.01	325.51	330.68
kg.cm	kg.cm	7,750,300	14,467,200	20,150,800	24,801,000	28,417,900	31,001,100	32,551,300	33,068,000
Prestressed force	kg	514.119	514.119	514.119	514.119	514.119	514.119	514.119	514.119
Arm	cm	15.07	28.14	39.19	48.24	55.27	60.30	63.31	64.32
lay-40									

529.602		37,548,762	52,205,000	>	0 kg/cm ²
6.835		284,387	284,387	>	0 kg/cm ²
77.46		132.03	183.57	>	0 kg/cm ²
129.02		0.00 kg/cm ²			
P		bottom fibre			
Ac		P.e	Mdl	Wb	
706.136		50,065,042	52,205,000	<	180 kg/cm ²
6.835		295,319	295,319	<	180 kg/cm ²
103.31		169.53	176.76	<	180 kg/cm ²
96.07		180 kg/cm ²			
4. at service load beam unshored					
top fibre					
M composite		Y' = depth of faa	Y' = depth of slab	Y' = Y' - h _s	
Wt'		h _s =	81.37 cm	22 cm	
33,066,000			59.37 cm		
551,233					
59.99		<	180.00 kg/cm ² on top of slab deck		
faa : fa = Y' : Y'					
fa x Y' = faa x Y'					
fa = faa x Y'/Y'					
59.99		x	85.60		
63.84		<	180 kg/cm ² (on bottom of slab deck)		
concrete compressive on top of girder					
fa dl		+	fa composite	<	180 kg/cm ²
129.02		+	63.84	=	192.86 kg/cm ²
concrete compressive on bottom of girder in composite					
M composite					
Wb'					
33,066,000					
396,491					
83.40		kg/cm ²			
fb dl		-	fb c	>	0 kg/cm ²
96.07		-	83.40	=	12.67 kg/cm ²
stress-40					

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Station : Pe 169 + 59		Prepared by

1.4.8 Shear

Shear force by each load				
Beam	Slab	Asphalt	Uniform	Line
load ton	load ton	load ton	load ton	load ton
35.055	17.150	3.220	22.381	7.467
				85.273

Inertia	=	24,828,730 cm ⁴
Ht	=	170.00 cm
Yt	=	86.80 cm
Yb	=	83.20 cm
e	=	70.90 cm
Area of section	=	6,835.00 cm ²
Thick of web section	=	20.00 cm
Span	=	40.00 m
V	=	85.27 x
	=	85,273.00 kg

Part	Depth	Width	Area	Distance	AlYi
i	t	w	Aj	to top = Yi	
(1)	(2)	(3)	(4)	(5)	(6)
1	20.00	80.00	1,600.00	10.00	16,000.00
2	86.80	20.00	1,736.00	43.40	75,342.40
4	12.00	30.00	360.00	24.00	8,640.00
		AY	3,696.00		99,982.40

Y	=	Sy	99,982.40
	=	tot.Ay	3,696.00

AYy	=	3,696 x	27.05
	=	99,982 cm ²	

b.I	=	20.00 x	24,828,730
	=	492,574,600 cm ⁵	

e	=	70.90 cm	
4 x e	=	4.00 x	70.90
	=	283.60 cm	

P jacking	=	886,443 kg	
n	=	0.75	
P	=	664,832 kg	
	=	4 x e	
Vt	=	----- x	P
	=	2,9360	
	=	----- x	664,832
	=	4,000	
	=	471.37 kg	

V net	=	V	=	Vt
	=	85,273.00	=	471.37
	=	84,801.63 kg		
vc	=	the shear resistance on the concrete		
AY	=	area of the portion of section outside the shear plane		
	=	outside the shear plane considered		
Y	=	distance from neutral axis to centroid as AY		
by	=	width of section at level y of the shear plane considered		
I	=	moment of Inertia of the cross section		
shear stress	=	AYy	=	V net
	=	b.I		
vc	=	99,982.40	=	84,802
	=	492,574,600		
vc^2	=	17.21	=	kg/cm ²
	=	298.29 kg/cm ²		
axial force	=	tx		
tx	=	P	=	664,832.25
	=	Ac	=	6,835
	=		=	97.27 kg/cm ²
tx/2	=	97.27	=	48.63 kg/cm ²
(tx/2)^2	=	2.00		
	=	2,365.30		
vc^2	=	17.21		
(tx/2)^2	=	2,365.30		
vc^2 + (tx/2)^2	=	2,382.52		
R	=	@sqrt (vc^2 + (tx/2)^2)		
	=	48.81		
Principle compressive force = f				
f1 max	=	+/- tx/2 + R		
	=	48.63 + 48.81		
	=	97.45 kg/cm ²	<	190.00 kg/cm ²
f2 min	=	(48.63) + 48.81		
	=	0.18 kg/cm ²	<	0.00 kg/cm ²
1 kg/cm ²	=	14.21 psi		
fc	=	400.00 kg/cm ²		
	=	5,684.24 psi		
vc min allowance = 2 @sqrt(fc)				
	=	150.79 psi		
vc max allowance = 5 @sqrt(fc)				
	=	376.97 psi		
	=	28.53 kg/cm ²		

ERR

1.4.9 Deflection of beam

I. deflection computation

deflection is moment at any point by "bending moment" as load

I.a. deflection due to uniform load

load g
 $Mo = 1/8 \cdot g \cdot l^2$
 $Aa = 4/3 \cdot a \cdot b$
 $a = l/2$
 $b = l/2$
 $Mo = 4/3 \cdot (l/2) \cdot Mo$
 $Aa = 2/3 \cdot l \cdot Mo$
 $R = 1/2 \cdot Aa$
 $Mo = 2/8 \cdot l \cdot Mo$
 $xp1 = 3 \cdot a/8$
 $= 3 \cdot l/16$
 $= 3/16 \cdot l$

$M_{max} = Ya$
 $= R \cdot l/2 - Aa \cdot l/2 \cdot xp1$
 $= 2/6 \cdot l \cdot Mo \cdot l/2 - 2/3 \cdot l \cdot Mo \cdot (3/16 \cdot l)$
 $= Mo \cdot l^2 \cdot (1/8 - 1/16)$
 $= Mo \cdot l^2 \cdot (8/48 - 3/48)$
 $= Mo \cdot (5/48 \cdot l^2)$

$Ya = 5 \cdot Mo \cdot l^2 / 48 \cdot EI$

I.b. deflection due to tendon forces

load P
 $Mo = P \cdot e2$
 $Ab = Mo \cdot l$
 $R = (Mo \cdot l) / 2$
 $xp2 = 1/4$
 $M_{max} = Yb$
 $= Re2 \cdot l/2 - Ab \cdot l/2 \cdot xp2$
 $= (Mo \cdot l) / 2 \cdot l/2 - (P \cdot e2 \cdot l) / 2 \cdot l/4$
 $= (Mo \cdot l) \cdot l/4 - l/8$
 $= (Mo \cdot l) \cdot l \cdot (2/8 - 1/8)$
 $= (Mo \cdot l) \cdot l \cdot (2/8 - 1/8)$

$Yb = Mo \cdot l^2 / 8 \cdot EI$

I.c. deflection due to knife load

load Pk
 $Mo = 1/4 \cdot Pk \cdot l$
 $Ac = Mo \cdot l/2 \cdot l/2$
 $R = Mo/4 \cdot l$
 $= 1/3 \cdot l/2$
 $= l/6$
 $M_{max} = Yc$
 $= Mo \cdot l/4 \cdot l/2 - Ac \cdot l/2 \cdot l/6$
 $= Mo \cdot l/4 \cdot l/2 - Mo \cdot l/4 \cdot l/6$
 $= Mo \cdot (l^2/8 - l^2/24)$
 $= Mo \cdot l^2 \cdot (1/8 - 1/24)$

$Yc = Mo \cdot l^2 / 12 \cdot EI$

I.d. data
 span = 40.00 m
 $E \cdot c = 343,327 \text{ kg/cm}^2$
 Inertia = 24,828,730 cm⁴
 $E \cdot x \cdot l = 8,46E+12 \text{ kg cm}^2$

II. loading
 II.1 dead load
 weight of beam = 1.753 t/m³
 weight of slab = 0.857 t/m³
 asphalt = 0.161 t/m³
 II.2 live load
 uniform load = 1.119 t/m³
 line load = 7.467 ton / line

III. moment

III.1 by dead load
 load coefficient x load l^2 moment
 by beam 0.125 x 1.753 x 1,600.0 = 35,060,000
 slab deck 0.125 x 0.857 x 1,600.0 = 17,140,000
 asphalt 0.125 x 0.161 x 1,600.0 = 3,220,000

III.2 moment due to prestressed force

III.2.1 at bracing of tendon no.3
 P Jacking (15) = 271,649 kg / tendon
 P Jacking (12) = 171,568 kg / tendon
 average = 221,609 kg / tendon

P Jacking
 P initial = 886,433 kg
 P final = 691,744 kg
 P final = 518,608 kg
 e1 edge = 0.3410 m
 e2 middle = 0.7840 m
 e1 + e2 = 1.1250 m

at edge $M2 = P \cdot x \cdot e2$
 $= 886,433 \cdot x \cdot 0.7840 = 69,496,347 \text{ kg cm}$
 at middle $M1 = -P \cdot x \cdot (e1 + e2)$
 $= (886,433) \cdot x \cdot 1.1250 = (99,723,713) \text{ kg cm}$

III.2.2 at transfer
 e1 edge = 0.0900 m
 e2 middle = 0.7090 m
 e1 + e2 = 0.7990 m
 at edge $M2 = P \cdot x \cdot e2$
 $= 691,744 \cdot x \cdot 0.7090 = 49,044,650 \text{ kg cm}$
 at middle $M1 = -P \cdot x \cdot (e1 + e2)$
 $= (691,744 \cdot x) \cdot 0.7990 = (55,270,346) \text{ kg cm}$

IV. deflection computation

IV.1 by dead load

load	5/48	x	Mo	x	l ²	x	EI	=	Y
self weight beam	(1)	0.1042	x	35,060,000	x	16,000,000	:	8.46E+12	= 69.11
slab deck	(2)	0.1042	x	17,140,000	x	16,000,000	:	8.46E+12	= 33.79
asphalt	(3)	0.1042	x	3,220,000	x	16,000,000	:	8.46E+12	= 6.35

IV.1 by prestressing forces

IV.2.1 at stressing of tendon no.3

at edge	1/8	x	M2	x	1 ²	:	EI	=	y
Ye5 =	0.125	x	69,496,347	x	16,000,000	:	8,46E+12	=	164.38
at middle	5/48	x	M1	x	1 ²	:	EI	=	y
Ym5 =	0.1042	x	(59,723,713)	x	16,000,000	:	8,46E+12	=	(196.56)
balance			(197)	+	164.38	=	(32.18)		mm
by self weight beam									mm
total				+	69.11	=	36.92		mm

IV.2.2 at transfer

at edge	1/8	x	M2	x	1 ²	:	EI	=	y
Ye =	0.125	x	49,044,650	x	16,000,000	:	8,46E+12	=	116.00
at middle	5/48	x	M1	x	1 ²	:	EI	=	y
Ym =	0.1042	x	(55,270,346)	x	16,000,000	:	8,46E+12	=	(106.94)

balance	=	(106.94)	+	116.00	=	7.06	mm
by self weight beam							mm
total deflection by tendon	=	7.06	+	69.11	=	76.17	mm

V. short term deflection

V.1 uncracked beam

coefficient	load	l	Moment
	t.m	m	kg.cm
uniform load	0.125 x	1.119 x	1,800.0 = 22,380,000
line load	0.250 x	7.467 x	40.0 = 7,467,000

deflection

load	coefficient	x	Mo	x	1 ²	:	EI	=	y
(4)	5/48	x	Mo	x	1 ²	:	kg.cm ²		mm
uniform load	0.1042	x	22,380,000	x	16,000,000	:	8,4557E+12	=	44.11
line load	1/12	x	Mo	x	1 ²	:	EI	=	y
	0.0833	x	7,467,000	x	16,000,000	:	8,46E+12	=	11.77
			total						55.89

VI. long term deflection

VI.1 cracked beam

Cracking moment is the moment for which the tensile stress on the extreme fibre of the concrete

section reaches a value equal to the modulus of rupture of the concrete.

modulus of rupture of the concrete

$$f_r = 7.5 \cdot \sqrt{f_c'} \text{ psi}$$

$$f_r = 7.5 \cdot \sqrt{40 \text{ Mpa}}$$

$$f_r = 145.0326 \text{ psi}$$

$$f_r = 5901.304 \text{ psi}$$

$$f_r = (571.25) \text{ psi}$$

$$f_r = 0.07 \text{ kg/cm}^2$$

$$f_r = -571.247188 \text{ x } 0.07 = (39.99) \text{ kg/cm}^2$$

$$M_{cr} = n \cdot P \cdot (k + e) + f_r \cdot wb$$

$$M_{cr} = 2.00$$

$$P = 691,744 \text{ kg}$$

$$n = 0.75$$

$$n \cdot P = 0.75 \text{ x } 691,744 = 518,808 \text{ kg}$$

$$k = 24.28 \text{ cm}$$

$$e = 27.55 \text{ cm}$$

$$(k + e) = 51.81 \text{ cm}$$

$$wb = 62,399.00 \text{ cm}^3$$

$$f_r \cdot wb = (39.99) \text{ x } 62,399.00$$

$$= (2,495,167.73) \text{ kg.cm}$$

$$M_{cr} = n \cdot P \cdot (k + e) + f_r \cdot wb$$

$$= 518,808.00 \text{ x } 51.81 + (2,495,167.73)$$

$$= 26,879,442.48 + (2,495,167.73)$$

$$= 24,384,274.75 \text{ kg.cm}$$

deflection by cracking moment = y crack

$$y_{crack} = 5/48 \times Mo \times 1^3 / EI$$

5/48	x	Mo	x	1 ³	:	EI	=	yc
coefficient		kg.cm		cm ³		kg.cm ²		mm
0.1042	x	24,384,275	x	16,000,000	:	8,4557E+12	=	48.08
deflection by tendon						4,06E+13	:	8,4557E+12
								76.17 mm

total deflection = 48.08 + 76.17

= 124.23 mm

permissible deflections

= span

= 300

= 4,000

= 300

= 133.33 mm

ok

del-40

4.1.3 SUPERSTRUCTURE (PC BEAM 31.0M LONG)

1.4.12 Load on a beam

2. Influence line of moment and moment

2.1 Beam $g = 1.310 \text{ t/m}^3$
Length of span $= 31.00 \text{ m}$
Interval distance $= 2.50 \text{ m}$

Point	Distance m	Load t/m ³	Reaction ton	I.L. Moment t.m	Area of I.L. m ²	Moment t.m
1	2.50	1.310	0.919	2.30	35.63	46.67
2	5.00	1.310	0.839	4.19	65.00	85.15
3	7.50	1.310	0.758	5.69	88.13	115.44
4	10.00	1.310	0.677	6.77	105.00	137.55
5	12.50	1.310	0.597	7.46	115.63	151.47
6	15.00	1.310	0.516	7.74	120.00	157.20
7	15.50	1.310	0.500	7.75	120.13	157.36

2.2 Slab $g_s = 1.072 \text{ t/m}^3$

Point	Distance m	Load t/m ³	Reaction ton	I.L. Moment t.m	Area of I.L. m ²	Moment t.m
1	2.50	1.072	0.919	2.30	35.63	38.19
2	5.00	1.072	0.839	4.19	65.00	69.67
3	7.50	1.072	0.758	5.69	88.13	94.46
4	10.00	1.072	0.677	6.77	105.00	112.55
5	12.50	1.072	0.597	7.46	115.63	123.94
6	15.00	1.072	0.516	7.74	120.00	128.63
7	15.50	1.072	0.500	7.75	120.13	128.78

2.3 asphalt $g_{as} = 0.201 \text{ t/m}^3$

Point	Distance m	Load t/m ³	Reaction ton	I.L. Moment t.m	Area of I.L. m ²	Moment t.m
1	2.50	0.201	0.919	2.30	35.63	7.17
2	5.00	0.201	0.839	4.19	65.00	13.08
3	7.50	0.201	0.758	5.69	88.13	17.74
4	10.00	0.201	0.677	6.77	105.00	21.13
5	12.50	0.201	0.597	7.46	115.63	23.27
6	15.00	0.201	0.516	7.74	120.00	24.15
7	15.50	0.201	0.500	7.75	120.13	24.18

2.4 uniform load $q = 1.388 \text{ t/m}^3$

Point	Distance m	Load t/m ³	Reaction ton	I.L. Moment t.m	Area of I.L. m ²	Moment t.m
1	2.50	1.388	0.919	2.30	35.63	49.46
2	5.00	1.388	0.839	4.19	65.00	90.24
3	7.50	1.388	0.758	5.69	88.13	122.35
4	10.00	1.388	0.677	6.77	105.00	145.77
5	12.50	1.388	0.597	7.46	115.63	160.53
6	15.00	1.388	0.516	7.74	120.00	166.60
7	15.50	1.388	0.500	7.75	120.13	166.77

load

1.4.12 Load on a beam

Span $= 31.00 \text{ m}$ thickness of asphalt $= 0.050 \text{ m}$
Clear span $= 31.00 \text{ m}$ thickness of slab $= 0.245 \text{ m}$
width of Road way $= 7.00 \text{ m}$ unit weight of asphalt $= 2.300 \text{ t/m}^3$
a Dead load unit weight of concrete $= 2.500 \text{ t/m}^3$
Load per width effective of beam $b_e = 1.75 \text{ m}$

description	width m	thick m	area m ²	length m	x unit weight t/m ³	weight t/m ²
a.1 Beam	gb					
a.2 Cross beam			0.4774	1.00	2.500	1.194
a.3 Slab	gs	0.245	0.43	1.00	g tot.	0.120
a.4 Asphalt	ga	0.050	0.09	1.00	2.500	1.314
					2.500	1.072
					2.300	0.201

b. Live load

b.1 Line load $= p$

Impact coefficient

$$k = 1 + \frac{20}{(50 + L)}$$

$$p = 12.00 \text{ ton / lane}$$

b.2 Uniform Load $= q$

$$q = 9.522 \text{ t/m}^3$$

$$Q = 31.00 < 30.00 \text{ m}$$

$$Q = 2.20 - \frac{60 \times (1-30)}{1.10}$$

$$Q = 2.20 - \frac{60.00}{60.00}$$

$$Q = 2.20 - 0.02$$

$$Q = 2.18 \text{ t / lane}$$

$$Q = 2.18 \text{ t / lane}$$

$$q = \frac{b_e}{2.75} \times Q$$

$$q = \frac{1.75}{2.75} \times 2.18$$

$$q = 1.39 \text{ t / m}^3$$

load

1.4.12 Load on a beam

Point	Distance m	Load t/m ²	Reaction ton	I.L Moment t.m	Moment t.m
1	2.50	9.522	0.919	2.30	21.88
2	5.00	9.522	0.839	4.19	39.93
3	7.50	9.522	0.758	5.69	54.14
4	10.00	9.522	0.677	6.77	64.50
5	12.50	9.522	0.597	7.46	71.03
6	15.00	9.522	0.516	7.74	73.72
7	15.50	9.522	0.500	7.75	73.79

(1). Initial case by self weight of beam = Mg

Distance	Mg
2.50	46.67
5.00	85.15
7.50	115.44
10.00	137.55
12.50	151.47
15.00	157.20
15.50	157.36

(2). Transfer case (Mg + M slab) = Mdl

Distance	Mg	M slab	Mdl = Mg + Ms
2.50	46.67	38.19	84.85
5.00	85.15	69.67	154.82
7.50	115.44	94.46	209.90
10.00	137.55	112.55	250.10
12.50	151.47	123.94	275.40
15.00	157.20	128.63	285.83
15.50	157.36	128.76	286.12

(3). Final case (composite section)

Distance	Moment support by composite section			
	Mg uniform load	Mp line load	Mll live load	Ma by asphalt
2.50	49.46	21.88	71.34	78.51
5.00	90.24	39.93	130.17	143.25
7.50	122.35	54.14	176.48	194.22
10.00	145.77	64.50	210.28	231.41
12.50	160.53	71.03	231.56	254.83
15.00	166.60	73.72	240.32	264.47
15.50	166.77	73.79	240.57	264.74

Mg = moment support by precast concrete prestressed
Mdl = moment support by precast concrete prestressed (beam + concrete Insitu + cross beam + side walks)
Ml = moment support by composite section (DI + LL)
Mc = moment support by composite section (DI + LL + Asphalt)

3. Influence line of shear and shear

Load	Distance m	Reaction ton	Area of I.L m ²	Shear ton
3.1 self weight	1.310	0.00	1.000	15.50
3.2 dead load	1.072	0.00	1.000	15.50
3.3 asphalt	0.201	0.00	1.000	15.50
3.4 uniform load	1.388	0.00	1.000	15.50
3.5 line load	9.522	0.00	1.000	9.522
				71.079

load

1.4.13 Prestress Force and numbers of tendon

Span	=	31.00 m
Mg (moment min.)	=	157.78 t.m
M transfer	=	15,778,000 kg.cm
M dead load	=	15,778,000 kg.cm
M composite	=	286.54 t.m
	=	28,654,000 kg.cm
	=	264.74 t.m
	=	26,474,000 kg.cm

Characteristic of Concrete

specified strength (fc)
Concrete class, K400

ft'	=	0 kg/cm ²
ksi'	=	190 kg/cm ²
ksi'	=	180 kg/cm ²
ksi'	=	0 kg/cm ²

Compression strength

Section properties

Beam		Composite	
h	=	160.00 cm	thick of slab = 17.50 cm
A	=	4,774.00 cm ²	thick of panel = 7 cm
Yt	=	69.86 cm	h' = 184.50 cm
Yb	=	71.14 cm	Ac' = 8,449.00 cm ²
I	=	14,611,103.00 cm ⁴	Yt' = 69.38 cm
Wt	=	164,424.00 cm ³	Yb' = 115.12 cm
Wb	=	205,390.50 cm ³	I' = 36,015,431 cm ⁴
kt	=	43.02 cm	Wt' = 519,099 cm ³
kb	=	34.44 cm	Wb' = 312,853 cm ³
e	=	51.14 cm	kt' = 37.00 cm ³
			kb' = 61.40 cm ³
			n = 0.75

Computation prestressing force

1. Initial case

1st equation top fiber

Pi	>	f _{ti} Wt - Mg (kb - e)
----	---	-------------------------------------

by self weight of beam

Mg	=	15,778,000 kg.cm
f _{ti}	=	0.00 kg/cm ²
Wt	=	164,424.00 cm ³
f _{ti} Wt	=	0.00 cm ³
kb	=	34.44 cm
e	=	51.14 cm
f _{ti} Wt - M min	=	0.00
kb - e	=	34.44
	=	51.14
Pi	>	(15,778,000)
	=	944.790
	>	944.79 ton

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1.4.13 Prestress Force and numbers of tendon

2nd equation bottom fiber		by self weight of beam	
Pi	<	$f_{ci}W_b + Mg$	
		$n(e + kt)$	
Mg	=	15,778,000	kg.cm
fci	=	190.00	kg/cm
Wb	=	205,390.50	kg/cm ²
fciWb	=	39,024,195.00	cm ³
kt	=	43.02	cm ³
e	=	51.14	cm
M min + fciWb	+	15,778,000	cm
kt + e	+	43.02	cm
	=	39,024,195	cm
	=	51.14	cm
	=	54,802,195	kg.cm
	=	94.16	cm
Pi	<	54,802,195	ton
	<	94.16	cm
	<	582.01	ton
2. After transfer			
1st equation top fiber Top fiber		after prestress losses	
Pi	>	$f_{ti}W_t - Mg$	
		$n(kb - e)$	
Mg	=	15,778,000	kg.cm
fci	=	0.00	kg/cm ²
Wt	=	164,424.00	cm ³
fciWt	=	0.00	cm ³
kb	=	34.44	cm
e	=	51.14	cm
(kb - e)	=	34.44	cm
fciWt - M min	=	0.00	cm
n(kb - e)	=	0.75	cm
	=	15,778,000	cm
	=	51.14	cm
	=	15,778,000	cm
	=	16.70	cm
	=	(15,778,000)	cm
	=	(12.53)	cm
Pi	>	1,259,721	kg
	>	-12.525	ton
	>	1,259.72	ton
2nd equation bottom fiber		after prestress losses	
Pi	<	$f_{ci}W_b + Mg$	
		$n(e + kt)$	
Mg	=	15,778,000	kg.cm
fci	=	190.00	kg/cm
Wb	=	205,390.50	kg/cm ²
fciWb	=	39,024,195	cm ³
kt	=	43.02	cm ³
e	=	51.14	cm
(kt + e)	=	43.02	cm
fciWb + M trans.	+	39,024,195	cm
n(kt + e)	+	0.75	cm
	=	15,778,000	cm
	=	51.14	cm
	=	54,802,195	kg.cm
	=	94.16	cm
	=	582.01	ton

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1.4.13 Prestress Force and numbers of tendon

3. Dead load 1st equation top fiber		by beam and concrete in situ	
Pi	<	$f_{cs}W_t - M dl$	
		$n(kb - e)$	
M dl	=	28,654,000	kg.cm
fci	=	0.00	kg/cm ²
Wt	=	164,424.00	cm ³
fciWt	=	0	cm ³
kb	=	34.44	cm
e	=	51.14	cm
(kb - e)	=	34.44	cm
fciWt - M dl	=	0	cm
n(kb - e)	=	0.75	cm
	=	28,654,000	kg.cm
	=	51.14	cm
	=	28,654,000	kg.cm
	=	16.70	cm
	=	(28,654,000)	kg.cm
	=	(12.53)	cm
Pi	>	2,287,745	kg
	>	(12.53)	ton
2nd equation bottom fiber		by beam and concrete in situ	
Pi	<	$f_{ti}W_b + M dl$	
		$n(e + kt)$	
M dl	=	28,654,000	kg.cm
fci	=	0.00	kg/cm ²
Wb	=	205,390.50	kg/cm ²
fciWb	=	0	cm ³
kt	=	43.02	cm ³
e	=	51.14	cm
(e + kt)	=	51.14	cm
fciWb + M dl	+	43.02	cm
n(e + kt)	+	0	cm
	=	28,654,000	kg.cm
	=	51.14	cm
	=	28,654,000	kg.cm
	=	94.16	cm
	=	70.62	cm
Pi	<	405,749	kg
	<	70.62	ton
	<	405.75	ton

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1.4.13 Prestress Force and numbers of tendon

4. Service load 3 rd equation top fiber

PI	<	$f_{cs}W_t - M_{dl} - M_c W_t / W_t'$	$n(kb - e)$	by service load
Mc	=	26,474,000	kg.cm	
Mdl	=	28,654,000	kg.cm	
fcs	=	180.00	kg/cm ²	
Wt	=	164,424.00	cm ³	
Wt'	=	519,099.00	cm ³	
Wb	=	519,099.00	cm ³	
kb	=	34.44	cm	
e	=	51.14	cm	
(kb - e)	=	34.44	cm	
Wt/Wt'	=	164,424.00	:	51.14 = (16.70) cm
Mc.Wt / Wt'	=	26,474,000.00	x	519,099.00 = 0.32
fcs.Wt	=	180.00	x	8,385,608.48 kg.cm
fcs.Wt - Mdl	=	29,596,320.00	kg.cm	29,596,320.00 kg.cm
n.(kb - e)	=	942,320.00	cm	942,320.00 kg.cm
	=	7,443,288.48	kg.cm	(7,443,288.48) kg.cm
	=	0.75	x	(16.70) = (12.53) kg.cm
PI	<	(7,443,288.48)	=	594,274.53
	<	(12.53)	=	

4 th equation bottom fiber

PI	>	$f_{ts}W_b + M_{dl} + M_c W_b / W_b'$	$n(e + kt)$	by service load
Mc	=	26,474,000.00	kg.cm	
Mdl	=	28,654,000.00	kg.cm	
fcs	=	0.00	kg/cm ²	
Wt'	=	519,099.00	cm ³	
Wb	=	205,390.50	cm ³	
Wb'	=	312,853.00	cm ³	
kt	=	43.02	cm	
e	=	51.14	cm	
(e + kt)	=	51.14	+	43.02 = 94.16 cm
Wb/Wb'	=	205,391	:	312,853 = 0.66
Mc.Wb/Wb'	=	26,474,000	x	17,380,393.02 kg.cm
fcs.Wb	=	0	x	0 kg.cm
fcs.Wb + Mdl	=	28,654,000	+	28,654,000 kg.cm
n.(e + kt)	=	48,034,393	+	46034393.018 kg.cm
	=	0.75	x	94.16 = 70.62 cm
PI	>	(48,034,393)	=	651,861 kg
	>	70.62	=	
	>	651.86	=	

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1.4.13 Prestress Force and numbers of tendon

Initial case	PI	>	944.79 ton
Top fibre	PI	<	582.01 ton
Bottom fibre	PI	>	1,259.72 ton
Transfer	PI	<	776.02 ton
Dead load	PI	<	2,287.74 ton
Top fibre	PI	<	405.75 ton
Bottom fibre	PI	>	594.27 ton
Service case	PI	<	651.86 ton
Top fibre	PI	<	582.01
Bottom fibre	PI	<	582.01

PI	=	582,011	kg
1 Mpa	=	10.2	kg/cm ²
1 kg/cm ²	=	0.098	Mpa
1 kN	=	102	Kg
1 kg	=	0.0098	kN
used strand rate 270	=	12.7	mm
nominal diameter	=	98.71	mm ²
nominal area	=	0.9871	cm ²
ultimate	=	1,860	Mpa
at jacking	=	18,972	kg/cm ²
at jacking	=	1,488	Mpa
P jacking	=	15,178	kg/cm ²
nos of strand / tendon	=	15177.6 x 0.9871	
PI / tendon	=	14,982 kg / strand	
PI / tendon	=	7.00	
PI / tendon	=	14,982 x 7	
PI / tendon	=	104,873 kg / tendon	
total force	=	582,011	kg
total force	=	5.55	
total force	=	6.00	nos

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1.4.14 Prestress losses

	tanlon - 3	tanlon - 2	tanlon - 1
	x (m)	x (m)	x (m)
K	0.0041	0.0041	0.0041
Kx	0.0000	0.0000	0.0000
u.alpha + K X	0.0238	0.1111	0.0636
e	2.7183	2.7183	0.1251
e - (u.alpha + K X)	0.9765	2.7183	2.7183
fx	0.8948	0.9807	0.8824
	0.9765	0.9807	0.8824

	tanlon - 3	tanlon - 2	tanlon - 1
	x (m)	x (m)	x (m)
X	0.000	0.000	0.000
alpha	0.1151	0.0275	0.0307
u	0.2000	0.0550	0.0614
u.alpha	0.0230	0.2000	0.2000
K	0.0041	0.0055	0.0061
Kx	0.0000	0.0041	0.0041
u.alpha + K X	0.0230	0.0055	0.0061
e	2.7183	0.0055	0.0061
e - (u.alpha + K X)	0.9772	2.7183	2.7183
fx	0.8962	0.9945	0.9270
	0.9772	0.9945	0.9270

$$X = 0 \quad X = 1/2$$

tanlon no.	fx	fx
6	0.976	0.895
5	0.961	0.902
4	0.970	0.882
3	0.977	0.896
2	0.995	0.928
1	0.994	0.927
sum	5.892	5.431
average	6	6
	0.982	0.905
average of fx	0.982 +	0.905
	2.00	
	0.9436 kg	
	fx	
	0.944	
	14.060	
	0.944	
	14.900 kg/cm2	
stress lost	14.900 -	14.060
	839.93 kg/cm2	
percentage stress lost	839.93	100
	14060	
	5.974 %	

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1.4.14 Prestress losses

stress losses	3. Due to anchorage set
Es	= delta / l x Es
assumed slip	= 1.897/200 kg/cm2
	= 0.5 cm
stress	= delta x Es
	= 0.50
	= 3.100
	= 1.897/200
	= 308 kg/cm2
	= 308
	= 100
	= 14.060
	= 2.176 %
	= 1.271 +
	= 5.974 +
	= 2.176
	= 9.42 %
	= (1.00 -
	= 0.908 x
	= 82.369 kg
	= Initial stress x
	= 82.369 x
	= 294.215 kg
	= 545.822 kg

Initial stress / tendon
PI / strand
PI / strand

Initial force after prestress loss

PI
PI

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1.4.15 Centre Gravity of Tendon

for span	=	31.00	m			
cgs	=	centre gravity of tendon (cgs) to the centre gravity of concrete (cgc)				
dia. of strand	=	12.70	mm			
depth	=	160.0	cm			
area	=	92.40	mm ²			
Nos of strand	=	0.924	cm ²			
area per tendon	=	7.00	cm ²			
Y _b	=	6.468	cm			
Y _t	=	71.14	cm			
thick of top cover concrete	=	88.86	cm			
	=	20.00	cm			
a. at stressing of tendon						
cgs tendon at stressing tendon no.5 at edge of span						
Strand no	distance to each	Strand no	distance to tendon outside	area of tendon	static moment	
	cm		cm	cm ²	Y	As.Y
No.6 to No.5	= 30.00	No.6	68.86	6.4680	x 68.86	= 445.39
No.5 to No.4	= 30.00	No.5	38.86	6.4680	x 38.86	= 251.35
No.4 to cgc	= 8.86	No.4	8.86	6.4680	x 8.86	= 57.31
				19.40		754.04

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1.4.15 Centre Gravity of Tendon

b. at transfer						
cgs tendon at edge of span						
Strand no	distance to each cm	Strand no	distance to outside cm	area of strand As cm2	static moment	
					Y cm	As.Y cm3
No.6 to No.5	=	30.00	No.6	140.00	0.9240 x	140.00 = 129.36
No.5 to No.4	=	30.00	No.5	110.00	0.9240 x	110.00 = 101.64
No.4 to No.3	=	30.00	No.4	80.00	0.9240 x	80.00 = 73.92
No.3 to No.2	=	30.00	No.3	50.00	0.9240 x	50.00 = 46.20
No.2.1 to our side	=	20.00	No.2	20.00	0.9240 x	20.00 = 18.48
			No.1	20.00	0.9240 x	20.00 = 18.48
				140.00	5.544	388.08

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1.4.17 Stress Control

Working moment

M minimum	=	157.76	t.m
M after transfer	=	15,776,000	kg.cm
M dead load	=	15,776,000	kg.cm
M composite	=	28,654,000	kg.cm
	=	28,654,000	kg.cm
	=	26,474,000	kg.cm

Allowable stresses in prestressed concrete

Initial case			
f _{ci}	=	192.0	kg/cm ²
f _{ti}	=	0.0	kg/cm ²
Final case			
f _{cs}	=	180.0	kg/cm ²
f _{ts}	=	0.0	kg/cm ²

Section properties

Girder	Composite	Thick of slab	Thick of panel
h	=	160.00	cm
Ac	=	4,774.00	cm ²
Y _t	=	68.86	cm
Y _b	=	71.14	cm
I	=	14,611,103	cm ⁴
W _t	=	164,425.00	cm ³
W _b	=	205,391.00	cm ³
W _c	=	43.02	cm
W _b	=	34.44	cm
W _c	=	50.47	cm
W _b	=	17.50	cm
W _c	=	7.00	cm
W _b	=	184.50	cm ²
W _c	=	8,449.00	cm ²
W _b	=	69.38	cm
W _c	=	115.12	cm
W _b	=	36,015.431	cm ⁴
W _c	=	519,099.00	cm ³
W _b	=	312,853.00	cm ³
W _c	=	37.00	cm
W _b	=	61.40	cm
W _c	=	0.75	cm

Computation of stresses

Force at jacking			
P _j	=	545,622	kg
P _i	=	494,215	kg
e jacking	=	38.86	cm
e transfer	=	50.47	cm
P	=	n	x
	=	0.75	x
	=	370,661	kg

Moment by eccentricity tendon

P _j x e	=	545,622	x	38.86
	=	21,202,871	kg.cm	
P _i x e	=	494,215	x	50.47
	=	24,943,031	kg.cm	
P x e	=	370,661	x	50.47
	=	18,707,273	kg.cm	

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1.4.17 Stress Control

Actually working stresses

1. Initial case									
top fibre									
f _{ti}	=	P _j	-	P _j x e	+	M _j	-	0.00	kg/cm ²
	=	Ac	-	W _t	+	W _t	-	0.00	kg/cm ²
	=	545,622	-	21,202,871	+	15,776,000	-	0.00	kg/cm ²
	=	4,774	-	164,425	+	164,425	-	0.00	kg/cm ²
	=	114.29	-	128.95	+	95.95	-	0.00	kg/cm ²
	=	81.29	-	0	+	0	-	0.00	kg/cm ²
bottom fibre									
f _{bi}	=	P _j	+	P _j x e	-	M _j	-	192	kg/cm ²
	=	Ac	+	W _b	-	W _b	-	192	kg/cm ²
	=	545,622	+	21,202,871	-	15,776,000	-	192	kg/cm ²
	=	4,774	+	205,391	-	205,391	-	192	kg/cm ²
	=	114.29	+	103.23	-	76.81	-	192	kg/cm ²
	=	140.71	+	192	-	192	-	192	kg/cm ²
2. after transfer case									
top fibre									
f _{tr}	=	P _i	-	P _i x e	+	M _{tr}	-	0	kg/cm ²
	=	Ac	-	W _t	+	W _t	-	0	kg/cm ²
	=	370,661	-	24,943,031	+	15,776,000	-	0	kg/cm ²
	=	4,774	-	164,425	+	164,425	-	0	kg/cm ²
	=	77.64	-	151.70	+	95.95	-	0	kg/cm ²
	=	21.69	-	0	+	0	-	0	kg/cm ²
bottom fibre									
f _{tr}	=	P _i	+	P _i x e	-	M _{tr}	-	180	kg/cm ²
	=	Ac	+	W _b	-	W _b	-	180	kg/cm ²
	=	370,661	+	24,943,031	-	15,776,000	-	180	kg/cm ²
	=	4,774	+	205,391	-	205,391	-	180	kg/cm ²
	=	77.64	+	121.44	-	76.81	-	180	kg/cm ²
	=	122.27	+	180.00	-	180.00	-	180	kg/cm ²

Project : Median Flood Control	Japan International Cooperation Agency (JICA)	Calculation sheet
Channel : Percut River		Date
Structure : Br - P1		Nos of sheet
Station : PE 57 + 05		Prepared by

1.4.17 Stress Control

3. Dead load (self weight beam + concrete cast in place)

ft	P	Ac	WT	Mdl	Wt	>	0 kg/cm2
	370.661		18,707.273	28,854,000		>	0 kg/cm2
	4,774		164,425	164,425			
	77.64		113.77	174.27		>	0 kg/cm2
	138.14		0.00 kg/cm2				

bottom fibre

P	Ac	Wb	Mdl	Wb	<	180 kg/cm2
494.215		24,943,031	28,854,000		<	180 kg/cm2
4,774		205,391	205,391			
103.52		121.44	139.51		<	180 kg/cm2
85.45		180 kg/cm2				

4. At service load beam unshare top fibre

ft aa	M composite	Wt	Yt = depth of faa	hs =	Yt = Yt - hs	=	69. cm
	28,474,000						22 cm
	519,099						47. cm

51.00 < 180.00 kg/cm2 on top of slab deck

$$faa : fa = Yt : Yt$$

$$fa \times Yt = faa \times Yt$$

$$fa = faa \times Yt / Yt$$

fa	47.38	x	51.00
	69.38		

34.83 < 180 kg/cm2 (on bottom of slab deck)

stres-31
total fa = concrete compressive on top of girder
total fa = fa dl + fa composite < 180 kg/cm2

total fa	138.14	+	34.83	=	172.96	kg/cm2	<	180 kg/cm2
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Project : Median Flood Control	Japan International Cooperation Agency (JICA)	Calculation sheet
Channel : Percut River		Date
Structure : Br - P1		Nos of sheet
Station : PE 57 + 05		Prepared by

1.4.17 Stress Control

ft = concrete compressive on bottom of girder in composite

M composite
Wb

28,474,000
312,853

84.62 kg/cm2

fb	fb dl	-	fb c	>	0 kg/cm2
	85.45	-	84.62	=	0.83 kg/cm2

0 kg/cm2

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Project : Medan Flood Control	Japan International Cooperation Agency (JICA)	Calculation sheet
Channel : Percut River		Date
Structure : Br - P1		Nos of sheet
Station : 57 + 05		Prepared by

1.4.18 Deflection of beam

I. deflection computation

deflection is "bending moment" as load

La deflection due to bending moment as parabola

$$y_a = \frac{5 \cdot Mo \cdot l^3}{48 \cdot EI}$$

Lb deflection due to bending moment as perpendicular

$$y_b = \frac{Mo \cdot l^3}{8 \cdot EI}$$

Lc deflection due to bending moment as triangle

$$y_c = \frac{Mo \cdot l^3}{12 \cdot EI}$$

II. data

span	=	31.00 m
E - c	=	3,100 cm
Inertia of beam	=	317,989 kg/cm ²
E x I	=	14,811,103 cm ⁴
Inertia of composite	=	4,65E+12 kg.cm ²
E x I	=	36,015,431 cm ⁴
	=	1.145E+13

III. loading

a. dead load		
weight of beam	=	1.31 t/m'
weight of slab	=	1.07 t/m'
asphalt	=	0.201 t/m'

b. live load		
uniform load	=	1,390 t/m'
line load	=	9.52 ton / line

III. moment

a. moment due to external load = Mo

coefficient	x	load	t/m'	x	l ²	m	moment	kg.cm
self weight of girder	0.125	x	1.310	x	961.0	=	15,736,375	
slab deck	0.125	x	1.070	x	961.0	=	12,853,375	
asphalt	0.125	x	0.201	x	961.0	=	2,414,513	

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Project : Medan Flood Control	Japan International Cooperation Agency (JICA)	Calculation sheet
Channel : Percut River		Date
Structure : Br - P1		Nos of sheet
Station : 57 + 05		Prepared by

1.4.18 Deflection of beam

b. moment due to prestressed force

b.1 at stressing of tendon no.5

jacking force	=	545,622 ton
initial force	=	494,215 ton
n	=	0.75
final	=	370,661 ton
e1	=	0.3886 m eccentricitas cable at middle span
e2	=	0.3684 m eccentricitas cable at edge span
e1 + e2	=	0.7570 m

at edge	M2	=	P	x	e2
		=	545,622 x	0.3684	= 20,100,714 kg.cm

at middle	M1	=	-P	x	(e1 + e2)
		=	(545,622) x	0.7570	= (41,303,585) kg.cm

b.2 at transfer

e1	=	0.5047 m	eccentricitas cable at middle span
e2	=	0.0114 m	eccentricitas cable at edge span
e1 + e2	=	0.5161 m	
at edge	M2	P	x e2
		494.215 x	0.0114 = 563.405

at middle	M1	=	-P	x	(e1 + e2)
		=	(494,215) x	0.5161	= (25,506,436) kg.cm

IV. deflection computation

a. by external load

load	5/48	x	Mo	x	l ²	:	E.I	=	y
	coefficient		kg.cm		cm ²		kg.cm ²		mm

a. dead load									
self weight (1)	0.1042	x	15,736,375	x	9,610,000	:	1.15E+13	=	13.75
slab deck (2)	0.1042	x	12,853,375	x	9,610,000	:	1.15E+13	=	11.23
asphalt (3)	0.1042	x	2,414,513	x	9,610,000	:	1.15E+13	=	2.11

b. by prestressing force

b.1 at stressing of tendon no.5

at edge	1/8	x	M2	x	l ²	:	E.I	=	y
at middle	5/48	x	M1	x	l ²	:	E.I	=	y
	0.1042	x	(41,303,585)	x	9,610,000	:	4,65E+12	=	(86.99)

balance	=	(88.99) +	51.97	=	(37.02)	mm
by self weight beam						
total	=	(37.02) +	13.75	=	(23.27)	mm

b.2 at transfer

at edge	1/8	x	M2	x	l ²	:	E.I	=	y
	0.125	x	563,405	x	9,610,000	:	4,65E+12	=	1.48
at middle	5/48	x	M1	x	l ²	:	E.I	=	y
	0.1042	x	(25,506,436)	x	9,610,000	:	4,65E+12	=	(54.95)

del-31

Project : Medan Flood Control	Japan International Cooperation Agency (JICA)	Calculation sheet
Channel : Percut River		Date
Structure : Br - P1		Nos of sheet
Station : 57 + 05		Prepared by

1.4.18 Deflection of beam

balance	=	(54.95) +	1.46	=	(53.50)	mm
by self weight beam				=	13.75	mm
total	=	(53.50) +	13.75	=	(39.74)	mm
V. short term deflection						
un-cracked beam						
coefficient	load	l	Moment			
	tm	m	kg.cm			
uniform load	0.125 x	1.390 x	961.0	=	16,697.375	
line load	0.250 x	9.520 x	31.0	=	7,378,000	
deflection						
load	coefficient	x	Mo	x	l ³ /2	: EI = y
			kg.cm		cm ²	kg.cm ²
uniform load	5/48 x		Mo	x	l ³ /2	: EI = y
			kg.cm		cm ²	kg.cm ²
line load	1/12 x		Mo	x	l ³ /2	: EI = y
			kg.cm		cm ²	kg.cm ²
	0.0833 x		7,378,000 x		9,610,000	: 1.15E+13 = 5.16
			total			= 19.75

VI. long term deflection

cracked beam

Cracking moment

Cracking moment is the moment for which the tensile stress on the extreme fibre of the concrete reaches a value equal to the modulus of rupture of the concrete.

modulus of rupture of the concrete

$$f_r = 7.5 \cdot \sqrt{f_c'} \text{ psi}$$

$$f_c' = 40 \text{ Mpa}$$

$$1 \text{ Mpa} = 145.0328 \text{ psi}$$

$$40 \text{ Mpa} = 5801.304 \text{ psi}$$

$$f_r = 571.25 \text{ psi}$$

$$1 \text{ psi} = 0.07 \text{ kg/cm}^2$$

$$f_r = 39.99 \text{ kg/cm}^2$$

$$M_{cr} = n \cdot P \cdot (k - e) + f_r \cdot w_b$$

$$P = 107,100 \text{ x}$$

$$n = 0.75$$

$$k = 0.75 \text{ x}$$

$$e = 43.02 \text{ cm}$$

$$(k + e) = 50.47 \text{ cm}$$

$$w_b = 93.49 \text{ cm}$$

$$f_r \times w_b = 205,390.50 \text{ cm}^3$$

$$= 39.99 \text{ x } 205,390.50$$

$$= 8,213,012.19 \text{ kg.cm}$$

$$M_{cr} = n \cdot P \cdot (k - e) + f_r \cdot w_b$$

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$$k = 0.75 \text{ x}$$

$$e = 43.02 \text{ cm}$$

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$$k = 0.75 \text{ x}$$

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$$= 39.99 \text{ x } 205,390.50$$

$$= 8,213,012.19 \text{ kg.cm}$$

Project : Medan Flood Control	Japan International Cooperation Agency (JICA)	Calculation sheet
Channel : Percut River		Date
Structure : Br - P1		Nos of sheet
Station : 57 + 05		Prepared by

1.4.18 Deflection of beam

$$\text{deflection by cracking moment} = y_c$$

$$y_c = 5/48 \times Mo \times l^3 / EI$$

5/48 x	Mo	x	l ³ /2	:	EI	=	y _c
	kg.cm		cm ²		kg.cm ²		mm
0.1042 x	53,270,518 x		9,610,000	:	1.145E+13	=	46.56
deflection by tendon							(39.74)

$$\text{total deflection} = 46.56 + 39.74 = 86.30 \text{ mm}$$

$$\text{permissible deflections} = \text{span} / 300 = 31,000 / 300 = 103.33 \text{ mm}$$

$$86.30 \text{ mm} < 103.33 \text{ mm} \text{ ok}$$

Project : Medan Flood Control	Japan International Cooperation Agency (JICA)	Calculation sheet
Channel : Perout River		Date
Structure : Br - P1		Nos of sheet
Station : PE 05 + 705 m		Prepared by

1.4.19 Shear

Computation of shear on distance 1.2 m from bearing

Clear span	=	31.00 m
Inertia	=	14,611,104 cm ⁴
Y ₁	=	88.86 cm
Y ₂	=	71.14 cm
Area of section	=	4,774.00 cm ²
Thick of web section	=	18.00 cm

Shear force

description	load per m ² l/m	q x l ton	Reaction ton
beam	1.310	31.00	40.61
slab	1.070	31.00	33.17
uniform load	1.390	31.00	43.09
line load	9.520	31.00	295.20
asphalt	0.201	31.00	6.23
Shear force = V			71.07

$$V = 71.07 \times 1,000 = 71,070.50 \text{ kg}$$

Part	Depth	Width	Area	Distance	A ₁ Y ₁
(1)	(2)	(3)	(4)	(5)	(6)
1	12.50	55.00	687.50	6.25	4,296.88
2	88.86	18.00	1,599.48	44.43	71,064.90
4	7.50	18.50	138.75	15.00	2,081.25
			2,425.73		77,443.02

$$Y = \frac{S_y}{A_{1Y}} = \frac{77,443.02}{2,425.73} = 31.93 \text{ cm}$$

$$A_{1Y} = 2,425.73 \times 31.93 = 77,443 \text{ cm}^2$$

$$b_1 I = 18.00 \times 14,611,104 = 262,999,872 \text{ cm}^5$$

$$e = 50.47 \text{ cm}$$

$$4 \times e = 201.88 \text{ cm}$$

$$P_{jacking} = 104,370 \text{ kg}$$

$$P = 78,278 \text{ kg}$$

$$V_1 = 4 \times e \times P = 2,0188 \times 78,278 = 1,580,310$$

$$V_{net} = V - V_1 = 71,070.50 - 1,580,310 = -1,509,239.5$$

$$V = 71,070.50 \text{ kg}$$

$$V_1 = 1,580,310 \text{ kg}$$

$$V_{net} = 71,070.50 - 1,580,310 = -1,509,239.5$$

$$V = 71,070.50 \text{ kg}$$

$$V_1 = 1,580,310 \text{ kg}$$

$$V_{net} = 71,070.50 - 1,580,310 = -1,509,239.5$$

Project : Medan Flood Control	Japan International Cooperation Agency (JICA)	Calculation sheet
Channel : Perout River		Date
Structure : Br - P1		Nos of sheet
Station : PE 05 + 705 m		Prepared by

1.4.19 Shear

shear stress	=	$\frac{A_{1Y}}{b_1 I}$	x	V _{net}
vc	=	$\frac{77,443.02}{262,999,872}$	x	85,973
vc ²	=	19.43	kg/cm ²	
axial force	=	377.38	kg/cm ²	
bx	=	P		78,277.50
bx	=	Ac		4,774
bx/2	=	18.40	kg/cm ²	
(bx/2) ²	=	2.00		
vc ²	=	67.21		
(bx/2) ²	=	19.43		
vc ² + (bx/2) ²	=	86.64		
R	=	@sqrt (vc ² + (bx/2) ²)		9.31
Principle compressive force = f				
f ₁ max	=	+/- bx/2 + R		
f ₂ min	=	8.20 + 17.51	kg/cm ²	9.31
1 kg/cm ²	=	(8.20) + 1.11	kg/cm ²	9.31
fc	=	14.21	psi	
vc min allowance	=	2 @sqrt(fc)		
vc max allowance	=	5 @sqrt(fc)		
	=	150.79	psi	
	=	10.61	kg/cm ²	
	=	378.97	psi	
	=	26.53	kg/cm ²	

Project : Median Flood Control	Japan International Cooperation Agency (JICA)	Calculation sheet
Channel : Perout River		Date
Structure : Bt - P1		Nos of sheet
Station : 57 + 05		Prepared by

1.4.20 Load of Superstructure

For span = 31.00 m									
span	=	31.00 m	Thickness of asphalt	=	0.050 m				
clear span	=	31.00 m	Unit weight of asphalt	=	2.300 t/m ³				
width of road way	=	7.00 m	Thickness of concrete deck	=	0.245 m				
numbers of lane	=	1.00							
Width of traffic separator	=	0.00 m	Unit weight of concrete K-225	=	2.400 t/m ³				
width of side walk 2 x 1.5 m	=	3.00 m	Unit weight of concrete K-400	=	2.500 t/m ³				
total width of slab	=	10.00 m							
nos = n	=	5.00 m	End block section area	=	0.960 m ²				
space = S	=	1.75 m	Prestressed section area	=	0.477 m ²				
thick of web = tw	=	0.18 m	Additional area on end block	=	0.483 m ²				
depth	=	1.80 m							
depth of flange	=	0.525 m							
cross beam									
length	=	(S - tw) x nos of beam - 1							
	=	4.00							
length	=	1.75 - 0.18 =	1.570 m						
width	=	1.57 x 4.00 =	6.280 m						
depth	=	0.20 m							
area	=	1.600 - 0.525 =	1.075 m						
	=	0.200 x 1.075 =	0.215 m						
end block									
length	=	2.00 x 1.60 =	3.200 m						
volume	=	0.48 x 3.20 =	1.544 m						
volume / m run	=	1.54 : 31.00 =	0.050 m						
area	x	length x volume x nos	x unit weight						
	=	0.59 x 31.00 =	204.30 ton						
a1.beam									
load of a beam	=	0.4774 x 1.00 x 0.4774 x 5.00 x 2.500 =	5.97						
end block	=	0.050 x 5.00 x 2.500 =	0.62						
Gbm	=	weight / m' x length							
	=	6.59 x 31.00 =	204.30 ton						
Description									
width	x	thick x length x nos	x unit weight						
	=	10.00 x 0.245 x 1.00 x 1.00 x 2.500 =	6.13						
a.2 Slab	gs =								
	=	6.13 x 31.80 =	193.55 ton						
a.3 Asphalt									
ga =		7.00 x 0.05 x 1.00 x 1.00 x 2.300 =	0.80						
	=	0.80 x 31.80 =	25.44 ton						
a.4 Side walk									
gsw =		1.00 x 0.250 x 1.00 x 2.00 x 2.400 =	1.20						
	=	1.20 x 31.80 =	37.92 ton						
a.5 Cross Beam									
gcb =		0.215 x 6.280 x 3.00 x 2.500 =	10.13						
a.6 Hand Rail									
down		0.16 x 0.10 x 0.55 x 33.00 x 2.400 =	0.70						
	=	0.16 x 0.30 x 0.70 x 33.00 x 2.400 =	2.66						
a.7 Slab Rail									
gsr =		0.10 x 0.70 x 1.00 x 2.00 x 2.400 =	0.34						
	=	0.34 x 31.80 =	10.62 ton						

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Project : Median Flood Control	Japan International Cooperation Agency (JICA)	Calculation sheet
Channel : Perout River		Date
Structure : Bt - P1		Nos of sheet
Station : 57 + 05		Prepared by

1.4.20 Load of Superstructure

Total

=	a.1	+	a.2	+	a.3	+	a.4	+	a.5	+	a.6
=	204.30	+	193.55	+	25.44	+	37.92	+	10.13	+	3.36
+	10.62	=	485.31 ton								

b. Live load
Impact coefficient

$$k = 1.00$$

b.1 line load

$$P = \frac{P}{2.75} \times \left\{ 5.5 \times k \right\} + \left\{ \frac{P}{2.75} \times (W - 5.5) \times k \right\} \times 50\%$$

total width of road way

$$= 7.00 \text{ m}$$

width of road way per lane

$$= 5.50 \text{ m}$$

W

$$= 1.50 \text{ m}$$

$$P = \frac{12.00}{2.75} \times 5.5 \times 1.00 \times 1.00 + \left\{ \frac{12.00}{2.75} \times 1.50 \times 1.00 \right\} \times 0.50$$

$$= 24.00 + 3.27$$

Gp

$$= 27.27 \text{ ton}$$

Gp tot.

$$= 1.00 \times 27.27 = 27.27 \text{ ton}$$

b-2. Uniform Load = q

$$\text{Span} = 31.00 \text{ m} > 30.00 \text{ m}$$

$$Q = 2.20 - \frac{1.1}{60 \times (L - 30)}$$

$$= 2.20 - 0.02$$

$$= 2.18 \text{ t/m'}$$

Q

$$q = \frac{Q}{2.75} \times 7.00 \times k \left\{ \right\} + \left\{ \frac{Q}{2.75} \times (W - 5.5) \times k \right\} \times 50\%$$

$$= \frac{2.182}{2.75} \times 7.00 \times 1.00 + \left\{ \frac{2.182}{2.75} \times 1.50 \times 1.00 \right\} \times 0.50$$

$$= 5.55 + 0.60$$

$$= 6.15 \text{ ton/m'}$$

Gq

$$= q \times l$$

$$= 6.15 \times 31.00$$

$$= 190.6 \text{ ton}$$

Gp tot.

$$= 1.00 \times 190.60 = 190.60 \text{ ton}$$

$$= 0.50 \text{ ton/m}^2$$

$$q_t = q_t \times N_{os} \times 60\%$$

$$= 0.50 \times 2.00 \times 0.60$$

$$= 0.60 \text{ ton}$$

Gpd

$$= q_p \times l$$

$$= 0.60 \times 31.00$$

$$= 18.60 \text{ ton}$$

b.3 .by pedestrian

$$q_t = 0.50 \text{ ton/m}^2$$

$$q_{pd} = q_t \times N_{os} \times 60\%$$

$$= 0.50 \times 2.00 \times 0.60$$

$$= 0.60 \text{ ton}$$

Gpd

$$= q_p \times l$$

$$= 0.60 \times 31.00$$

$$= 18.60 \text{ ton}$$

Total load

load on Abutment	by dead load	by line load	by uniform load	by pedes-rian load	total ton
	485.31	27.27	190.60	18.60	721.78

sup-31

4.1.4 SUPERSTRUCTURE (PC BEAM 25.0M LONG)

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I. SUPERSTRUCTURE I.1 Post railing

I.1.1 Moment, shear and steel bar		value	unit
allowable working stress			
concrete compressive	f_c' $@\sqrt{f_c' \times f_c'}$ $f_c = 0.33 \times f_c'$	225	kg/cm ²
steel tensile	f_s	15.00	kg/cm ²
concrete shear without steel bar	$tou = 0.54 \times @\sqrt{f_c'}$	75	kg/cm ²
concrete shear with steel bar	$tou = 1.35 \times @\sqrt{f_c'}$	1,400	kg/cm ²
modulus of elasticity steel	E_s	20	kg/cm ²
modulus of elasticity concrete	$E_c = 6400 \times @\sqrt{f_c'}$	2,100,000	kg/cm ²
modular ratio	$n = E_s/E_c$	96,000	kg/cm ²
forces		22	
moment (kg.cm)	M	0.09	t.m
shear (kg)	S	9,000	kg.cm
		0.1	ton
		100	kg
dimension of section			
width of section	b	15	cm
depth of section	h	24.00	cm
distance of compression steel to top fiber extrem	d'	2.5	cm
effective height of section	h	21.50	cm
area of section	$b \times h$	323	cm ²
Ca = section coefficient	n.M	196,875	kg.cm
	b.f.s	21,000	kg/cm
	$n.M/(b.f.s)$	9.38	
	$@\sqrt{f_c' \times n.M/(b.f.s)}$	3.06	
	$Ca = h / @\sqrt{f_c' \times n.M/(b.f.s)}$	7.022	
Calculation of steel bar area	n.f.c	1,641	kg/cm ²
	$so = f_s / (n.f.c)$	0.8533	
	delta	0.20	assumed
	Ca	7.053	
	$a > so$	4.405	
steel bar full working	e'	9.588	
	psi	0.938	
	100 mw	2.144	
	nw	0.02144	
	w	0.001	
A = with		0.001 x	323 = 0.316 cm ²
A' = delta.A		0.20 x	0.316 = 0.063 cm ²
compressive concrete		n.e	96.36
tensile bar		$f_c' = f_s / (n.e)$	14.53 < 74.99 kg/cm ² ok
shear of concrete		$f_s' = f_s / e'$	146.02 < 1400 kg/cm ² ok
		$7/8 \times b \times h$	282.19 cm ²
		$tou ca = S / (7/8 \times b \times h)$	0.35 < 8 kg/cm ² ok

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I. SUPERSTRUCTURE I.1 Post railing

area of tensional steel bar				
A - used steel bar dia. 13 mm				
area @ bar dia. 13 mm	=	1.267	2 nos	cm ²
total area	=	2.534	> 0.32	cm ²
area of compressive steel bar				
A' - used steel bar dia. 13 mm				
area @ bar dia. 13 mm	=	1.267	2 nos	cm ²
total area	=	2.534	> 0.06	cm ²

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I.2. Side Walk

I.2.1. Moment and Shear

description	a. Dead load				Yc	= weight
	width	height	depth	x		
	m	m	m	m	t/m ³	ton
a). hand railing						
up hand railing	P1 = 0.150	x 0.180	x 0.550	x	2.400	= 0.036
down hand railing	P2 = 0.150	x 0.210	x 0.450	x	2.400	= 0.034
slab Rail	P3 = 0.100	x 1.000	x 0.450	x	2.400	= 0.108
b). slab						
slab side walk (1)	P4 = 1.00	x 1.410	x 0.150	x	2.400	= 0.508
slab side walk (2)	P5 = 1.00	x 0.623	x 0.050	x	2.400	= 0.075
slab deck	P6 = 1.00	x 0.300	x 0.275	x	2.400	= 0.198
						0.958
total shear						
Moment to point A						
a by dead load						
description	Load	x	Distance	=	Moment	
	ton		m		t.m	
up hand railing	P1 = 0.036	x	1.410	=	0.050	
down hand railing	P2 = 0.034	x	1.350	=	0.046	
slab rail	P3 = 0.108	x	1.405	=	0.152	
slab side walk (1)	P4 = 0.508	x	0.705	=	0.358	
slab side walk (2)	P5 = 0.075	x	0.670	=	0.050	
slab deck	P6 = 0.198	x	0.150	=	0.030	
Total moment by dead load (dl)				=	0.686	
b. Live load						
due to pedestrian	= 0.500 ton/m ²					
due to railing force	= 0.100 ton					
due to curb force	= 0.500 ton					
description	Load	x	Distance	=	Moment	
	ton		m		t.m	
pedestrian	= 0.500	x	0.500	x	1.254	= 0.314
railing force	= 0.100	x	0.900	=	0.090	
curb force	= 0.500	x	0.250	=	0.125	
sum shear by live load (ll)	= 1.600					
sum moment by live load (ll)				=	0.529	
total shear (dl + ll)	= 1.600	+	0.958	=	2.558 ton	
total moment (dl + ll)	= 0.686	+	0.529	=	1.214 ton.m	

side-wk

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I.2.2 Reinforcing steel bar of side walks

Item	Description	unit	Calculation
Allowance stresses			
concrete compressive	fc K - 350	kg/cm ²	115.5
steel tensile	fs	kg/cm ²	1400
concrete shear	tau without steel bar	kg/cm ²	8.04
	tau with steel bar	kg/cm ²	20.20
	E steel	kg/cm ²	1,897,200
	E concrete	kg/cm ²	160,577
transformend	n = Es/Ec		11.61
Moment (kg.cm)	M	kg.cm	273,400
Shear (kg)	S	kg	6,483
Normal (kg)	N	kg	0
dimension	b	cm	100
	ht	cm	17.50
	d'	cm	5
	h	cm	12.50
	b.h	cm ²	1,250
	n.M	kg.cm	3,230,198
	b.fs	kg/cm	140,000
	n.M/(b.fs)		23.07
	@sqrt(n.M'/(b.fs))		4.80
	Ca=h/@sqrt(n.M'/(b.fs))		2.602
Calculation for reinfor- cement bar with steel full working	n.fc		1,365
	so = fs/(n.fc)		1.028
	delta		0.20
	see table of PBI		
	from table fined Ca		2.602
	fine ø > so		1.381
	ø'		1.812
	j		0.420
	100 nw		17.09
	nw		0.1709
	w		0.014
	A = wbh		18.08
	A' = delta.A		3.616
A - used bar dia. 16 mm @ 12.5 cm.			
Area @ bar	=	2.011	cm ²
nos of bar	=	9	
total area	=	18.099	> 2.011
	=	18.099	> 18.08
A' - used bar dia. 13 mm @ 12.5 cm.			
Area @ bar	=	1.327	cm ²
nos of bar	=	9	
total area	=	11.94	> 1.327
	=	11.94	> 3.616
steel bar full warking	n.e		16.32
	fs		1400
compressive concrete	fc' = fs / (n.e)	kg/cm ²	85.80
tensile bar	fs' = fs / ø'	kg/cm ²	772.63
	7/8.b.h		1,093.75
shear concrete	Tca = S / (7/8.b.h)	kg/cm ²	5.93

bar-sw

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I.3. Slab deck

I.3.1 Moment and Shear

a. Data

thick of slab = 0.175 m
 thick of asphalt = 0.050 m
 distance each beam = $l_x = 1.750$ m

dead load

Uraian	width	x	thick	x	length	Unit weight	Weight
	m		m		m	t/m^3	t/m^2
slab deck							
asphalt	=	1.000	x	0.050	x	1.000	= 2.300 = 0.115
slab	=	1.000	x	0.175	x	1.000	= 2.500 = 0.438
						$\frac{g}{g}$	= 0.553

Fixed End Moment	Mge	=	$\frac{1}{10} \times g \times l_x^2$	=	3.063	
		=	$0.100 \times g \times 0.553 \times 3.063$			
		=	0.169 t.m			
Span moment	Mgs	=	$\frac{1}{12} \times g \times l_x^2$	=	3.063	
		=	$0.083 \times g \times 0.553 \times 3.063$			
		=	0.141 t.m			
Shear by dead load		=	$\frac{1}{2} \times g \times l_x$	=	1.750	
		=	$0.500 \times g \times 0.553 \times 1.750$			
		=	0.483 ton			

Live Load

T	=	10.000 ton				
a	=	0.200 m				
b	=	0.500 m				
a1	=	$0.200 + (2.000 \times (0.050 + 0.050))$	=	0.088	=	0.475 m
b1	=	$0.500 + (2.000 \times (0.050 + 0.088))$	=	0.088	=	0.775 m
width of wheel effective						
l_x	=	1.750 m				
l_y	=	31.000 m				
r	=	0.500				
$3r$	=	1.500				
l_y/l_x	=	17.714				
	>	$3r$				
Sa	=	$\frac{3}{4} \cdot a1 + \frac{3}{4} \cdot r \cdot l_x$				
	=	$\frac{3}{4} \times a1 + \frac{3}{4} \times 0.750 \times 0.475$				$r \cdot l_x$
	=	$0.750 \times 0.475 + 1.013$				0.875

sl-deck

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I.3. Slab deck

I.3.1 Moment and Shear

Area of parabola	$Ap = \frac{4}{3} \times l_x/2 \times (Sa - a1)/2$	=	0.269
	$= \frac{1.333 \times 0.875 \times 0.314}{2}$		
Area of tyred contact = Ac	$Ac = 2 \times Ap + l_x \cdot a1$	=	0.831
	$= 2.000 \times 0.314 + 1.458 \text{ m}^2$		
	$q = \frac{T}{Ac} = \frac{10.000}{1.458}$	=	6.857 t/m^2
Moment maximum	$Mo = \frac{0.125 \times 6.857 \times 2.625 \text{ t.m}}$	=	3.063
Fixed End Moment	$Mle = \frac{1}{10} \times q \times l_x^2$	=	3.063
	$= \frac{0.100 \times 6.857 \times 2.100 \text{ t.m}}$		
Span moment	$Mls = \frac{Mo}{Sa} = \frac{2.625}{1.013}$	=	2.593 t.m
Shear by live load	$= \frac{1}{2} \times q \times l_x$	=	1.750
	$= \frac{0.500 \times 6.857 \times 6.000 \text{ ton}}$		
Fixed end moment	Total moment		
	$= Mge + Mle$	=	2.100
	$= 0.169 + 2.269 \text{ t.m}$		
Span moment	$= Mgs + Mls$	=	2.593
	$= 0.141 + 2.734 \text{ t.m}$		
Shear by dead load	Total shear		
Shear by live load	$= 0.483 \text{ ton}$		
Total	$= 6.000 \text{ ton}$		
	$= 6.483 \text{ ton}$		

sl-deck

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1.4 Beam
For span = 25.00 m
1.4.1 Section Properties

Concrete Class of deck K - 350		=	350.00	kg/cm ²
		=	34.31	Mpa
Concrete Class of Precast Prestressed Concrete specific		=	400	kg/cm ²
		=	39.22	Mpa
Ratio of moduli (n)		=	350	
		=	400	
		=	0.875	
bt	=	n	x	be
bt	=	0.875	x	175
bt	=	153.15	cm	
say	=	150.0	cm	
Section properties of Composite section moment static to bottom fiber				
part	depth	width	area	distance to
t	cm	w	Al'	Yb
(1)	(2)	(3)	(4)	(5)
slab	17.5	150.0	2,625.0	140.8
panel	7.0	150.0	1,050.0	128.5
beam			3,187.5	51.9
total	24.5		6,842.5	
			668,887.50	
Yb'	=	Ac'	=	97.75
Yt'	=	Ht'	=	Yb'
	=	125.00	=	97.75
	=	51.75	=	
lx'	=	tot.lx'	=	Yb'^2
	=	83,450,569.3	=	9,556.0
	=	83,443,726.8	=	65,386,991.3
	=	18,056,795.5	=	
Section modulus				
top fiber	Wt'	lx'	18,056,795.5	
bottom fiber	Wb'	Yt'	51.7	348,955.1
		lx'	18,056,795.5	
		Yb'	97.8	184,714.5
Kern				
top fiber	kt'	Wb'	184,714.5	
bottom fiber	kb'	Ac'	6,842.5	27.0
		Wt'	348,955.1	
		Ac'	6,842.5	51.0

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1.4.2 Load on a beam

Span	=	25.60	m	thickness of asphalt	=	0.050	m
Clear span	=	25.00	m	thickness of slab	=	0.245	m
width of Road way	=	7.00	m	unit weight of asphalt	=	2.300	t/m ³
a Dead load				unit weight of concrete	=	2.500	t/m ³
Load per width effective of beam							
be	=	1.75	m				

description	width	x	thick	x	area	x	length	x	unit weight	weight
	m		m		m ²		m		t/m ³	t/m ²
a.1 Beam	gb				0.3168	x	1.00	x	2.500	= 0.792
a.2 Cross beam										= 0.080
a.3 Slab	gs	1.75	x	0.245	x	0.43	x	1.00	x	= 0.872
a.4 Asphalt	ga	1.75	x	0.050	x	0.09	x	1.00	x	= 1.072
										= 0.201

b. Live load

b.1 Line load = p

Impact coefficient

$$k = 1 + \frac{20}{(50 + L)}$$

p = 12.00 ton / lane

$$be = \frac{2.75}{1.75} \times p \times k$$

= 9.673 t/m'

b.2 Uniform Load = q

Span = 25.00 < 30.00 m

Q = 2.20 t / lane

$$q = \frac{be}{2.75} \times Q$$

$$= \frac{1.75}{2.75} \times 2.20$$

$$= 1.40 \text{ t / m'}$$

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1.4.2 Load on a beam

2. Influence line of moment and moment

2.1 Beam		g = 0.872 t/m'	
Length of span		= 25.00 m	
Interval distance		= 2.50 m	
Point	Distance m	Load t/m'	Reaction ton
1	2.50	0.872	0.900
2	5.00	0.872	0.800
3	7.50	0.872	0.700
4	10.00	0.872	0.600
5	12.50	0.872	0.500
2.2 Slab		gs = 1.072 t/m'	
Point	Distance m	Load t/m'	Reaction ton
1	2.50	1.072	0.900
2	5.00	1.072	0.800
3	7.50	1.072	0.700
4	10.00	1.072	0.600
5	12.50	1.072	0.500

Point	Distance m	Load t/m'	Reaction ton	I.L. Moment t.m	Area of I.L. m ²	Moment t.m
1	2.50	1.072	0.900	2.25	28.13	30.146
2	5.00	1.072	0.800	4.00	50.00	53.594
3	7.50	1.072	0.700	5.25	65.63	70.342
4	10.00	1.072	0.600	6.00	75.00	80.391
5	12.50	1.072	0.500	6.25	78.13	83.740

2.3 asphalt		gas = 0.201 t/m'	
Point	Distance m	Load t/m'	Reaction ton
1	2.50	0.201	0.900
2	5.00	0.201	0.800
3	7.50	0.201	0.700
4	10.00	0.201	0.600
5	12.50	0.201	0.500

2.4 uniform load		q = 1.400 t/m'	
Point	Distance m	Load t/m'	Reaction ton
1	2.50	1.400	0.900
2	5.00	1.400	0.800
3	7.50	1.400	0.700
4	10.00	1.400	0.600
5	12.50	1.400	0.500

2.5 line load		p = 9.673 t/m'	
Point	Distance m	Load t/m'	Reaction ton
1	2.50	9.673	0.900
2	5.00	9.673	0.800
3	7.50	9.673	0.700
4	10.00	9.673	0.600
5	12.50	9.673	0.500

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1.4.2 Load on a beam

(1). Initial case by self weight of beam = Mg

Distance	Mg
2.50	24.525
5.00	43.600
7.50	57.225
10.00	65.400
12.50	68.125

(2). Transfer case (Mg + M slab) = Mdl

Distance	Mg	M slab	Mdl = Mg + Ms
2.50	24.53	30.15	54.671
5.00	43.60	53.59	97.194
7.50	57.23	70.34	127.567
10.00	65.40	80.39	145.791
12.50	68.13	83.74	151.865

(3). Final case (composite section)

Moment support by composite section				
Distance	Mq	Ml	Ma	Mc = Ma + Ml
	uniform	line	live	asphalt
2.50	39.38	21.76	61.14	5.66
5.00	70.00	38.69	108.69	10.06
7.50	91.88	50.78	142.66	13.21
10.00	105.00	58.04	163.04	15.09
12.50	109.38	60.45	169.83	15.72

Mg = moment support by precast concrete prestressed
 Mdl = moment support by precast concrete prestressed (beam + concrete in situ + cross beam + side walks)
 Ml = moment support by composite section (DI + LL)
 Mc = moment support by composite section (DI + LL + Asphalt)

3. Influence line of shear and shear

Load	Distance m	Reaction ton	Area of I.L. m ²	Shear ton
3.1 self weight	0.872	0.00	1.000	12.50
3.2 dead load	1.072	0.00	1.000	12.50
3.3 asphalt	0.201	0.00	1.000	12.50
3.4 uniform load	1.400	0.00	1.000	17.500
3.5 line load	9.673	0.00	1.000	9.673
				53.987

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1.4.3 Prestress Force and numbers of tendon

PI	<	26,923,840	=	514,784 kg
	<	52.30	ton	
	<	514.78	ton	

3. Dead load		by beam and concrete insitu	
1st equation top fiber			
M dl	kg.cm	15,187,000	
f _{ti}	kg/cm ²	0.00	
Wt	cm ³	75,220.00	0.00 cm
f _{ti} Wt	cm	0	
kb	cm	23.75	
e	cm	36.33	
(kb - e)	cm	23.75	
f _{ti} Wt - M dl	cm	23.75	
n.(kb - e)	x	0.75	
		(15,187,000)	
PI	>	(9.43)	
	>	1,610.28	ton

2nd equation bottom fiber

PI		PI		PI	
<		<		<	
f _{ts} Wb + M dl		f _{ts} Wb + M dl		f _{ts} Wb + M dl	
n.(e + kt)		n.(e + kt)		n.(e + kt)	
M dl	kg.cm	15,187,000		M dl	kg.cm
f _{ti}	kg/cm ²	0.00		f _{ti}	kg/cm ²
Wb	cm ³	105,836.00		Wb	cm ³
f _{ti} Wb	x	0		f _{ti} Wb	x
kt	cm	33.41		kt	cm
e	cm	36.33		e	cm
(e + kt)	cm	36.33		(e + kt)	cm
f _{ti} Wb + M dl	+	36.325		f _{ti} Wb + M dl	+
n.(e + kt)	x	0.75		n.(e + kt)	x
		15,187,000			
PI	<	52.30		PI	<
	<	290.38	ton		<

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1.4.3 Prestress Force and numbers of tendon

4. Service load		by service load	
3rd equation top fiber			
PI	<	f _{ts} Wt - M dl - Mc.Wt/Wt	n.(kb - e)
Mc	kg.cm	19,555,000	
M dl	kg.cm	15,187,000	
f _{ts}	kg/cm ²	180.00	
Wt	cm ³	75,220.00	
Wt'	cm ³	348,955.00	
Wb	cm ³	348,955.00	
kb	cm	23.75	
e	cm	36.33	
(kb - e)	cm	23.75	
Wt/Wt'	x	75,220.00	
Mc.Wt/Wt'	x	18,555,000.00	
f _{ts} Wt	x	180.00	
f _{ts} Wt - M dl	x	13,539,600.00	
n.(kb - e)	x	(1,647,400.00)	
		0.75	
		(5,647,076.46)	
PI	<	(9.43)	
	<	598.76	

4th equation bottom fiber

PI		PI		PI	
<		<		<	
f _{ts} Wb + M dl + Mc.Wb/Wb'		f _{ts} Wb + M dl + Mc.Wb/Wb'		f _{ts} Wb + M dl + Mc.Wb/Wb'	
n.(e + kt)		n.(e + kt)		n.(e + kt)	
Mc	kg.cm	18,555,000.00		Mc	kg.cm
M dl	kg.cm	15,187,000.00		M dl	kg.cm
f _{ts}	kg/cm ²	0.00		f _{ts}	kg/cm ²
Wt'	cm ³	348,955.00		Wt'	cm ³
Wb	cm ³	105,836.00		Wb	cm ³
Wb'	cm ³	184,714.00		Wb'	cm ³
kt	cm	33.41		kt	cm
e	cm	36.33		e	cm
(e + kt)	cm	36.325		(e + kt)	cm
Wb/Wb'	x	105.836		Wb/Wb'	x
Mc.Wb/Wb'	x	18555000		Mc.Wb/Wb'	x
f _{ts} Wb	x	0		f _{ts} Wb	x
f _{ts} Wb + M dl	+	0		f _{ts} Wb + M dl	+
n.(e + kt)	x	0.75		n.(e + kt)	x
		25,818,500			
PI	>	52.30		PI	>
	>	493.85	ton		>

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1.4.4 Prestress loss

2. Due to friction
- $f_x = f_i e^{-(\mu \theta + Ks)}$
 - η = stress in the strand at the jacking end or live anchorage
 - u = stress at dead anchorage
 - α = coefficient of angular friction
 - x = change in the angle between the force at anchorage to section X
 - f = curvilinear length of the tendon anchorage to section x
 - r = depth of parabola

tendon parabola curve
 $y = 4x(l-x)/l^2$
 angle of tendon parabolic to horizontal line
 $dx/dy = 0$

x	=	0.00 m
e	=	0.416 m
l	=	25.00 m
α	=	$4x/l$ radial
α	=	4.00 x 0.416
α	=	1.664
α	=	0.0666 radial
α	=	2.00 x 0.0666
α	=	0.1331

no tendon		value of sag parabola each tendon		
	f	H	distance tendon to bottom	top cover
4	f	1.25		0.35 m
3	f	0.900		0.155
2	f	0.600		0.090
1	f	0.350		0.065
	f	0.150		0.065

no tendon		value of depth parabola each tendon		
4	α	4.00 x	0.745	25.00 = 0.1192 radial
3	α	4.00 x	0.510	25.00 = 0.0816 radial
2	α	4.00 x	0.285	25.00 = 0.0456 radial
1	α	4.00 x	0.065	25.00 = 0.0136 radial

If $x = 0$, $\alpha = \alpha$
 If $x = l/2$, $\alpha = 2x \alpha$

description	tendon - 4		tendon - 3	
	x (m)	x (m)	x (m)	x (m)
α	0.0666	0.1331	0.0816	0.1632
u	0.2000	0.2000	0.2000	0.2000
u	0.0133	0.0266	0.0163	0.0326
K	0.0041	0.0041	0.0041	0.0041
Kx	0.0000	0.0000	0.0000	0.0000
u	0.0133	0.0266	0.0163	0.0326
e	2.7183	2.7183	2.7183	2.7183
f_x	0.9568	0.9251	0.9338	0.9195

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1.4.4 Prestress loss

X	tendon - 2		tendon - 1	
	x (m)	x (m)	x (m)	x (m)
α	0.0000	12.50	0.0000	12.50
u	0.0456	0.0912	0.0136	0.0272
u	0.2000	0.2000	0.2000	0.2000
u	0.0091	0.0182	0.0027	0.0054
K	0.0041	0.0041	0.0041	0.0041
Kx	0.0000	0.0513	0.0000	0.0513
u	0.0091	0.0695	0.0027	0.0567
e	2.7183	2.7183	2.7183	2.7183
e	0.9909	0.9329	0.9973	0.9449
f_x	0.9909	0.9329	0.9973	0.9449

$X = 0$ $X = l/2$

tendon no.	f_x	f_x
4	0.987	0.925
3	0.984	0.920
2	0.991	0.933
1	0.997	0.945
sum	3.959	3.722

average = 4 3.959 3.722

= 0.990 0.931

average of f_x = 0.990 + 0.931

= 2.00 0.9601 f_x

f_x = 0.960

= 14.060

= 0.960

= 14.844 kg/cm²

= 14.844 - 14.060

= 583.62 kg/cm²

= 583.62 x 100

= 14060

= 4.151 %

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1.4.4 Prestress loss

3. Due to anchorage set	
stress losses	= $\frac{\Delta}{L} \times E_s$
E_s	= 1,897,200 kg/cm ²
assumed slip	= 0.5 cm
stress	= $\frac{\Delta}{L} \times E_s$
	= $\frac{0.5}{2,500} \times 1,897,200$
	= 379 kg/cm ²
stress losses	= $\frac{379}{100} \times 100$
	= 14,060
	= 2,699 %
total stress lost	= 8,685 + 4,151 + 2,699
	= 15,54 %
initial stress / tendon	= (1.00 - 0.1554) x P jacking
	= 0.845 x 103,928
	= 87,783 kg
initial force after prestress loss	= initial stress x nos tendon
	= 87,783 x 4
Pi	= 351,131 kg
Pj	= 415,712 kg

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Station : PE 57 + 05		Prepared by

1.4.5 Centre Gravity of Tendon

cgs	for span	= 25.00 m	distance from centre gravity of strand (cgs) to the centre gravity of concrete (cgc)
depth of beam	= 125.00 mm		
dia. of strand	= 12.70 mm		
nos of strand	= 7.00		
area	= 92.40 mm ²		
area a tendon	= 0.9240 cm ²		
Yb	= 51.93 cm		
Yt	= 73.07 cm		
top concrete cover	= 30.00 cm		

a. at stressing of tendon

Strand no	distance to each	tendon no	distance to outside	area of a tendon	static moment
	cm		cm	As cm ²	Y cm
No.4 to No.3	= 25.00	No.4	43.07	6.468 x	43.07 = 278.58
No.3 to cgc	= 18.07	No.3	18.07	6.468 x	18.07 = 116.88
	43.07			12.936	395.45

e se = 30.57 cm

cgs tendon no.4 at middle of span

Strand no	distance to each	tendon no	distance to fiber bottom	area of a tendon	static moment
	cm		cm	As cm ²	Y cm
No.4 to No.3	= 6.50	No.4	15.50	6.468 x	15.50 = 100.25
No.3 to outside	= 9.00	No.3	9.00	6.468 x	9.00 = 58.21
	15.50			12.936	158.47

Ys = 158.47 cm

e sm = 12.25 cm

Yb = 51.93 cm

Yt = 73.07 cm

b. at transfer

cgs tendon at edge of span

Strand no	distance to each	tendon no	distance to fiber bottom	area of a tendon	static moment
	cm		cm	As cm ²	Y cm
No.4 to No.3	= 30.00	No.4	90.00	0.9240 x	90.00 = 83.16
No.3 to no.2	= 25.00	No.3	60.00	0.9240 x	60.00 = 55.44
No.2 to no.1	= 20.00	No.2	35.00	0.9240 x	35.00 = 32.34
No.1 to outside	= 15.00	No.1	15.00	0.9240 x	15.00 = 13.86
	90.00			3.70	184.80

cgc-25

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1.4.5 Centre Gravity of Tendon

Ys	184.80	50.00	cm
e	3.696		
e to Ys	Yb	Ys	
	51.93	50.00	
	1.93	cm	

cgs tendon at middle of span					
Strand no	distance to each	tendon no	distance to fiber to bottom	area of a tendon	
				As	As.Y
No.4 to No.3	8.50	No.4	15.50	0.9240 x 15.50	14.32
No.3 to No.2.1	2.50	No.3	9.00	0.9240 x 9.00	8.32
No.2.1 to out side	8.50	No.2	6.50	0.9240 x 6.50	6.01
	15.50			2.7720	28.6440

Ys	28.644	10.333	cm
e	2.772		
e to Ys	Yb	Ys	
	51.93	10.33	
	41.60	cm	

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1.4.6 Layout of tendon

description	span		25.00		m	
	2.50	5.00	7.50	10.00	12.50	
2.1. beam	24.53	43.60	57.23	65.40	68.13	
2.2. slab	30.15	53.59	70.94	80.39	83.39	
2.3. asphalt	5.66	10.08	15.21	15.09	15.72	
2.4. uniform load	39.38	70.00	91.88	105.00	109.38	
2.5. line load	21.76	38.69	50.76	58.04	60.46	

(1) M minimum = 2.1	24.53	43.60	57.23	65.40	68.13
(2) M dead load = 2.1+2.2	54.67	97.19	128.17	145.79	151.52
(3) M live load = 2.5+2.6	61.14	106.69	142.84	163.04	169.83
(4) M composite = (3) + 2.4	66.80	116.75	157.84	178.13	185.55

case	Pia		P=n.Pi	
	Initial case	After transfer	Dead load	Composite
Distance	2.5	5	7.5	10
Mg	24.53	43.60	57.23	65.40
Pi	2,452,500	4,360,000	5,722,500	6,540,000
arm	415,712	415,712	415,712	415,712
	5.90	10.49	13.77	15.73

case	Pia		P=n.Pi	
	Initial case	After transfer	Dead load	Composite
Distance	2.5	5	7.5	10
M transfer	24.53	43.60	57.23	65.40
P	2,452,500	4,360,000	5,722,500	6,540,000
Arm	263,348	263,348	263,348	263,348
	9.31	16.56	21.73	24.83

case	Pia		P=n.Pi	
	Initial case	After transfer	Dead load	Composite
Distance	2.5	5	7.5	10
M dead load	24.53	43.60	57.23	65.40
P	2,452,500	4,360,000	5,722,500	6,540,000
Arm	263,348	263,348	263,348	263,348
	20.76	36.91	48.67	55.36

case	Pia		P=n.Pi	
	Initial case	After transfer	Dead load	Composite
Distance	2.5	5	7.5	10
Mc	24.53	43.60	57.23	65.40
P	2,452,500	4,360,000	5,722,500	6,540,000
Arm	263,348	263,348	263,348	263,348
	20.76	36.91	48.67	55.36

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1.4.7 Stress Control

Actually working stresses									
1. Initial case									
top fibre									
ft	Pj	Pj x e	Mg	Wt	kg/cm2				
	Ac								
	415,712	16,487,138	6,813,000						
	3,168	75,220	75,220		0.00				
	131.22	219.19	90.57		0.00				
	2.61	0	0		0.00				
bottom fibre									
fb	Pj	Pj x e	Mg	Wb	kg/cm2				
	Ac								
	415,712	16,487,138	6,813,000		192				
	3,168	105,836	105,836		192				
	131.22	155.78	64.37		192				
	222.63	192	192		192				
2. after transfer case									
top fibre									
ft	Pi	Pi x e	Mtr	Wt	kg/cm2				
	Ac								
	351,131	16,487,138	6,813,000		0				
	3,168	75,220	75,220		0				
	110.84	4.67	90.57		0				
	196.74	0	0		0				
bottom fibre									
fb	P	P x e	Mtr	Wb	kg/cm2				
	Ac								
	351,131	16,487,138	6,813,000		180				
	3,168	105,836	105,836		180				
	110.84	3.32	64.37		180				
	49.78	180.00	180.00		180				

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1.4.7 Stress Control

3. Dead load (self weight beam + concrete cast in place)									
top fibre									
ft	P	P x e	Mdl	Wt	kg/cm2				
	Ac								
	263,348	10,955,287	15,187,000		0				
	3,168	75,220	75,220		0				
	83.13	145.84	201.90		0				
	139.39	0.00	0.00		0				
bottom fibre									
fb	P	P x e	Mdl	Wb	kg/cm2				
	Ac								
	351,131	14,607,050	15,187,000		180				
	3,168	105,836	105,836		180				
	110.84	138.02	143.50		180				
	105.36	180	180		180				
4. at service load beam unshored									
top fibre									
ftaa	M composite	Yt' = depth of faa	hs = depth of slab	Yt = Yt' - hs	cm				
	Wt'								
	18,555,200				51.75				
	348,955				22				
	53.17				29.75				
	faa : fa = Yt' : Yt								
	fa x Yt' = faa x Yt								
	fa = faa x Yt/Yt'								
bottom fibre									
fa	M composite	Yt' = depth of faa	hs = depth of slab	Yt = Yt' - hs	cm				
	Wt'								
	29.75				53.17				
	51.75								
	30.57				180				
	concrete compressive on top of girder				180				
	fa dl + fa composite				180				
	139.39				169.95				
	30.57				180				

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1.4.8 Deflection of beam

b. moment due to prestressed force									
a. at stressing of tendon no.4									
jacking force	=	415.712 ton							
initial force	=	351.131 ton							
final	=	263.348 ton							
e1	=	0.3968 m	eccentricitas cable at middle span						
e2	=	0.3057 m	eccentricitas cable at edge span						
e1 + e2	=	0.7025 m							
at edge	M2	P	x	e2					
		415.712 x		0.3057	=	12,708,316 kg.cm			
at middle	M1	-P	x	(e1 + e2)	=	(29,203,768) kg.cm			
		(415.712) x		0.7025	=				
b. at transfer									
e1	=	0.4160 m	eccentricitas cable at middle span						
e2	=	0.0193 m	eccentricitas cable at edge span						
e1 + e2	=	0.4353 m							
at edge	M2	P	x	e2					
		351.131 x		0.0193	=	677,683 kg.cm			
at middle	M1	-P	x	(e1 + e2)					
		(351.131) x		0.4353	=	(15,284,732) kg.cm			
VI. deflection computation									
a. due to external load									
load	5/48 x	Mo	x	l ²	:	E.I	=	Y	
	coefficient	kg.cm		cm ²		kg.cm ²		mm	
a. dead load									
self weight beam	(1)	0.1042 x	6,812,500 x	6,250,000	:	6,20E+12	=	7.15	
slab deck	(2)	0.1042 x	8,375,000 x	6,250,000	:	6,20E+12	=	8.80	
asphalt	(3)	0.1042 x	1,570,313 x	6,250,000	:	6,20E+12	=	1.65	
b. due to prestressing force									
b.1 at stressing of tendon no.4									
at edge	1/8 x	M2	x	l ²	:	E.I	=	Y	
	0.125 x	12,708,316 x		6,250,000	:	1,89E+12	=	52.61	
at middle	5/48 x	M1	x	l ²	:	E.I	=	Y	
	0.1042 x	(29,203,768) x		6,250,000	:	1,89E+12	=	(100.76)	
balance	=	(100.76) +	52.61	=		(48.14)		mm	
by self weight beam						7.15		mm	
total	=	(48.14) +	7.15	=		(40.99)		mm	
b.2 at transfer									
at edge	1/8 x	M2	x	l ²	:	E.I	=	Y	
	0.125 x	677,683 x		6,250,000	:	1,89E+12	=	2.81	

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1.4.8 Deflection of beam

at middle	5/48 x	M1	x	l ²	:	E.I	=	Y	
	0.1042 x	(15,284,732) x		6,250,000	:	1,89E+12	=	(52.73)	
balance	=	(52.73) +	2.81	=		(49.93)		mm	
by self weight beam						7.15		mm	
total	=	(49.93) +	7.15	=		(42.77)		mm	
VII. short term deflection									
un-cracked beam	coefficient	load	l	Moment					
		kg.cm	m	kg.cm					
uniform load	0.125 x	1,400 x		625.0	=	10,937,500			
line load	0.250 x	9,670 x		250	=	6,043,750			
deflection	coefficient	Mo	x	l ²	:	E.I	=	Y	
		kg.cm		cm ²		kg.cm ²		mm	
uniform load	5/48 x	Mo	x	l ²	:	E.I	=	Y	
	0.1042 x	10,937,500 x		6,250,000	:	6,199E+12	=	11.49	
line load	1/12 x	Mo	x	l ²	:	E.I	=	Y	
	0.0833 x	6,043,750 x		6,250,000	:	6,20E+12	=	5.08	
		total						16.56	
VIII. long term deflection									
cracked beam									
Cracking moment									
Cracking moment is the moment for which the tensile stress on the extreme fibre of the concrete section reaches a value equal to the modulus of rupture of the concrete.									
modulus of rupture of the concrete									
fr	=	7.5 . @sqrt (fc') psi							
fc'	=	40 Mpa							
1 Mpa	=	145,0326 psi							
40 Mpa	=	5801,304 psi							
fr	=	571.25 psi							
1 psi	=	0.07 kg/cm ²							
fr	=	39.99 kg/cm ²							
Mcr	=	n.P. (kt - e)	+	fr . wb					
P	=	107,100 x							
n	=	0.75							
n.P	=	0.75 x							
kt	=	43.02 cm							
e	=	50.47 cm							
(kt + e)	=	93.49 cm							
wb	=	205,390.50 cm ³							
fr x wb	=	39.99 x		205,390.50					
	=	8,213,012.19 kg.cm							
Mcr	=	n.P x (kt - e)	+	fr . wb					
	=	481,950.00 x		93.49	+	8,213,012.19			
	=	45,057,505.50	+	8,213,012.19					
	=	53,270,517.69 kg.cm							

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1.4.8 Deflection of beam

deflection by cracking moment					yc
$yc = 5/48 \times Mo \times l^2 / EI$					
5/48	x	Mo	x	l^2	= yc
coefficient		kg.cm		cm ²	= mm
0.1042	x	53,270.518	x	6,250,000	= 55.94
deflection by tendon					(42.77)
total deflection					= 55.94 + 13.17 mm = 69.11 mm
permissible deflections					= 25,000 / 300 = 83.33 mm ok

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1.4.9 Shear

span	=	25.00 m
Computation of shear on distance 1.2 m from bearing		
Reaction	=	$0.5 \times q \times l - q \times X$
Clear span	=	25.00 m
Inertia	=	5,496,255 cm ⁴
H	=	125.00 cm
Yt	=	73.07 cm
Yb	=	51.93 cm
Area of section	=	3,168.00 cm ²
Thick of web section	=	17.00 cm
Shear force	=	

description	load per m'	l	q x l	Reaction
	V/m'	m	ton	ton
beam	=	0.872	25.00	21.80
slab	=	1.072	25.00	26.80
uniform load	=	1.400	25.00	35.00
line load	=	9.670		17.500
asphalt	=	0.201	25.00	5.03
Shear force = V				2513
				53.98

$$V = 53.98 \times 1,000 = 53,982.50 \text{ kg}$$

Part	Depth	Width	Area	Distance	Al Yt
(1)	(2)	(3)	(4)	(5)	(6)
depth of section					
1	12.50	55.00	687.50	6.25	4,298.88
2	73.07	18.00	1,315.26	36.54	48,053.02
4	7.50	18.50	138.75	15.00	2,081.25
			2,141.51		54,431.15

Y	Sy	=	54,431.15
	totAy	=	2,141.51
Ay.y	=	2,141.51 x	25.42 cm
b.I	=	54,431 cm ²	
	=	17.00 x	5,496,255
	=	93,436,335 cm ⁵	
e	=	50.47 cm	
4 x e	=	4.00 x	50.47
	=	201.88 cm	
Packing	=	415,712 kg	
P initial	=	351,131 kg	
n	=	0.75	
P	=	263,348 kg	
Vt	=	4 x e	x P
	=	-----	I

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I.4.9 Shear

$$= \frac{2.0188}{25.00} \times 263,348$$

$$= 21,265.90 \text{ kg}$$

$$= V - V_t$$

$$= 53,982.50 - 21,265.90$$

$$= 32,716.60 \text{ kg}$$

V_{net} = the shear resistance on the concrete
 A_y = area of the portion of section outside the shear plane outside the shear plane considered
 y = distance from neutral axis to centroid as A_y
 I = width of section at level y of the shear plane considered
 I = moment of inertia of the cross section

$$= \frac{A_y y}{b I} \times V_{net}$$

$$= \frac{54,431.15}{93,436,335} \times 32,717$$

$$= 19.06 \text{ kg/cm}^2$$

$$= 363.25 \text{ kg/cm}^2$$

axial force = f_x

$$= \frac{P}{A_c}$$

$$= \frac{263,348.25}{83.13 \text{ kg/cm}^2}$$

$$= 83.13$$

$$= \frac{2.00}{41.56 \text{ kg/cm}^2}$$

$$= \frac{1,727.55}{19.06}$$

$$= \frac{1,727.55}{1,746.61}$$

$$= \sqrt{vc^2 + (fx/2)^2}$$

$$= \sqrt{41.79^2 + (19.06/2)^2}$$

Project : Medan Flood Control	Japan International Cooperation Agency (JICA)	Calculation sheet
Channel : Percut River		Date
Structure : Br - P1		Nos of sheet
Station : 05 + 705		Prepared by

I.4.9 Shear

Principle compressive force = f

$$f_1 \text{ max} = +/ - \frac{fx}{2} + R$$

$$= 41.56 + 41.79$$

$$= 83.36 \text{ kg/cm}^2$$

$$f_2 \text{ min} = (41.56) + 41.79$$

$$= 0.23 \text{ kg/cm}^2$$

$$1 \text{ kg/cm}^2 = 14.21 \text{ psi}$$

$$f_c = 400.00 \text{ kg/cm}^2$$

$$= 5,684.24 \text{ psi}$$

$$vc \text{ min allowance} = 2 @ \sqrt{f_c}$$

$$= 150.79 \text{ psi}$$

$$= 10.61 \text{ kg/cm}^2$$

$$vc \text{ max allowance} = 5 @ \sqrt{f_c}$$

$$= 376.97 \text{ psi}$$

$$= 26.53 \text{ kg/cm}^2$$

Project : Medan Flood Control Channel : Percut River Structure : Br - P1 Station : 57 + 05	Japan International Cooperation Agency (JICA)		Calculation sheet	
			Date	
			Nos of sheet	
			Prepared by	

1.4.10 Load of Superstructure

span	=	25.60 m	Thickness of asphalt	=	0.050 m
clear span	=	25.00 m	Unit weight of asphalt	=	2,300 t/m ³
width of road way	=	7.00 m	Thickness of concrete deck	=	0.245 m
numbers of lane	=	1.00			
Width of traffic separator	=	0.00 m	Unit weight of concrete K-225	=	2,400 t/m ³
width of side walk 2 x 1.5 m	=	3.00 m	Unit weight of concrete K-400	=	2,500 t/m ³
total width of slab	=	10.00 m			
nos = n	=	5.00 m	End block section area	=	0.825 m ²
space = S	=	1.75 m	Prestressed section area	=	0.317 m ²
thick of web = tw	=	0.17 m	Additional area on end block	=	0.308 m ²
depth	=	1.25 m			
depth of flens	=	0.375 m			
length	=	(S - tw) x nos of beam - 1			
	=	4.00			
length	=	1.75	0.17	=	1.580 m
width	=	1.58	4.00	=	6.320 m
depth	=	0.20			
area	=	1.250	0.375	=	0.875 m
	=	0.200	0.875	=	0.175 m
length	=	2.00	1.25	=	2.500 m
volume	=	0.31	2.50	=	0.771 m
volume / m run	=	0.77	25.00	=	0.031 m
area x length	=	volume	x nos	x unit weight	weight
m ²	=	m	m ³	t/m ³	ton/m ³
a1 beam					
load of a beam	0.3168 x	1.00 x	0.3168 x	5.00 x	2,500 = 3.96
end block			0.031 x	5.00 x	2,500 = 0.39
Gbm	=	weight / m' x length			gb / m' = 4.35
	=	4.35 x 25.00	=	108.63 ton	
Description	width x	thick x	length x	nos x	unit weight
	m	m	m ³		t/m ³
a.2 Slab	gs =	10.00 x	0.245 x	1.00 x	2,500 = 6.13
	Gs =	6.13 x	25.60 =	156.80 ton	
a.3 Asphalt					
ga =	7.00 x	0.05 x	1.00 x	1.00 x	2,300 = 0.80
	Ga =	0.80 x	25.60 =	20.61 ton	
a.4 Side walk					
gaw =	1.00 x	0.250 x	1.00 x	2.00 x	2,400 = 1.20
	Gaw =	1.20 x	25.60 =	30.72 ton	
a.5 Cross Beam					
gcb =	0.175 x	6.320 x	3.00 x	2,500 =	8.30
a.6 Hand Rail					
down	0.16 x	0.10 x	0.55 x	27.00 x	2,400
	Ghr =	0.16 x	0.30 x	0.70 x	2,400
					2.18
					2.75
a.7 Slab Rail					
gar =	0.10 x	0.70 x	1.00 x	2.00 x	2,400
	Gsr =	0.34 x	25.60 =	8.60 ton	
Total					
=	a.1 +	a.2 +	a.3 +	a.4 +	a.5 + a.6
=	a.7				
=	108.63 +	156.80 +	20.61 +	30.72 +	8.30 +
=	8.80 =	336.40 ton			2.75

sup-25

Project : Median Flood Control
Channel : Percut River
Structure : Br - P1
Station : 57 + 05

Japan International
Cooperation Agency
(JICA)

Calculation sheet
Date
Nos of sheet
Prepared by

1.4.10 Load of Superstructure

b. Live load		k = 1.00			
Impact coefficient					
b.1 line load					
P	=	$\frac{2.75}{5.5} \times (5.5 \times k) + \{ (W - 5.5) \times k \} \times 50\%$			
		2.75		2.75	
total width of road way	=	7.00 m			
width of road way per lane	=	5.50 m			
W	=	1.50 m			
P	=	$\frac{12.00}{5.5} \times 5.5 \times 1.000 + \{ (12.00 - 5.5) \times 1.50 \} \times 50\%$			
		2.75		2.75	
Gp	=	24.00 + 3.27			
Gp tot.	=	1.00 x 27.27 = 27.27 ton			
b-2. Uniform Load = q					
Span	=	25.00 m	<	30.00 m	
Q	=	2.20 t/m'			
q	=	$\frac{2.75}{2.200} \times 7.00 \times k + \{ (W - 5.5) \times k \} \times 50\%$			
		2.75		2.75	
		2.200		2.200	
		$\frac{2.75}{5.60} \times 7.00 \times 1.00 + \{ (W - 5.5) \times 1.50 \} \times 50\%$			
		5.60 + 0.60		2.75	
		6.20 ton/m'			
Gq	=	q x l			
		6.20 x 25.00			
		155.0 ton			
Gp tot.	=	1.00 x 155.00 = 155.00 ton			
b.3 by pedestrian					
qt	=	0.50 ton/m ²			
qpd	=	qt x Nos x 60 %			
		0.50 x 2.00 x 0.60			
		0.60 ton			
Gpd	=	qp x l			
		0.60 x 25.00			
		15.00 ton			

Total load				
load on Abutment	by dead load	by line load	by uniform load	by pedest. load
	336.40	27.27	155.00	15.00
				total ton
				533.676

sup-25

4.1.5 SUPERSTRUCTURE (PC BEAM 19.0M LONG)

**PRESTRESSED CONCRETE BEAM COMPUTATION
CHANNEL, MEDAN FLOOD WAY
STRUCTURE, BR – F7
STATION, FW 38 + 78
WITH
CLEAR SPAN = 19.00 M**

1.4 Beam

- 1.4.1 Section Properties of Beam**
- 1.4.2 Loading on a Beam**
- 1.4.3 Prestress force and Number Of Tendon Available**
- 1.4.4 Prestress Loss**
- 1.4.5 Centre Gravity of Tendon**
- 1.4.6 Layout of Tendons**
- 1.4.7 Stresses Control**
- 1.4.8 Deflection of a Beam**
- 1.4.9 Shear Force and Tensile Stress**

Project : Medan Flood Control		Japan International Cooperation Agency (JICA)		Calculation sheet	
Channel : Perout River				Date	
Structure : Br - F7				Nos of sheet	
Station : FW 38 + 78				Prepared by	
L4 Main Beam					
L4.1 Section Properties of beam					
Span	=	19.00 m			
Width of road way	=	3.50 m			
Nos of beam	=	3.00 m			
Space of a beam	=	1.75 m			
Thick of slab	=	0.175 m			
Section Properties					
moment static to top fiber					
part	depth	width	area	distance to	AI.YI
i	t	w	AI	top fiber (Yi)	AI.YI~2
	cm	cm	cm2	cm	cm3
(1)	(2)	(3)	(4)	(5)	(6)
	depth of beam	90	cm		cm4
1	7.5	35.0	262.5	3.8	984
3	12.5	65.0	812.5	83.8	68,047
2	70.0	17.0	1,190.0	42.5	50,575
4	7.5	9.0	67.5	10.0	675
5	10.0	24.0	240.0	74.2	17,800
					1,320,167
					138,081

li	lx
cm4	cm4
(9)	(10)
1/12.b.h^3	(8) + (9)
1,230	4,922
10,579	5,709,505
485,917	2,635,354
211	6,961
1,333	1,321,500
	9,678,242

Yt	=	Sx	138,081	=	53.68 cm
Yb	=	Ac	2,572.50	=	
	=	Ht	Yt	=	
	=	90.00	53.68	=	36.32 cm
Ix	=	tot.Ixi	x	Yt~2	
	=	9,678,242	x	2,881	
	=	9,678,242	x	7,411,635	
	=	2,266,607 cm4			
Section modulus					
top fiber	Wt	Ix	2,266,607	=	42,228 cm3
		Yt	53.68	=	
bottom fiber	Ix	Ix	2,266,607	=	62,400 cm3
Wb		Yb	36.32	=	

top fiber	Wb	62,400	=	24.26 cm
kt	Ac	2,573	=	
bottom fiber	Wt	42,228	=	16.42 cm
kb	Ac	2,573	=	

Composite section

width of flange = (be)
ACI

Effective flange width (be) lesser than

1	S	=	175.0 cm
2	1/4	=	475.0 cm
	bw	=	17.0 cm
	ht	=	17.5 cm
3	bw + 16 ht	=	297.0 cm

be	=	175.0 cm
1 Mpa	=	10.2 kg/cm2
1 kg/cm2	=	0.0980 Mpa

Width of transformed flange = bt
Concrete Class of deck K - 350

	=	350 kg/cm2
	=	34.31 Mpa

Concrete Class .

Precast Prestressed K - 400	=	400 kg/cm2
Concrete specific	=	39.22 Mpa
	=	350

Ratio of moduli (n)

	=	400
	=	0.875
bt	=	n
	=	0.88
bt	=	153.13 cm
say	=	150.0 cm

Section properties of Composite section

moment static to bottom fiber

part	depth	width	area	distance to	AI.YI'	AI.YI'^2
t	w	AI'	Yb	cm	cm3	cm4
(1)	(2)	(3)	(4)	(5)	(6)	(7)
slab	17.5	150.0	2,625.0	105.8	277,593.8	29,355,539.1
panel	7.0	150.0	1,050.0	93.5	98,175.0	9,179,362.5
beam			2,572.5	36.3	93,443.8	3,394,260.2
total	24.5		6,247.5		469,212.5	

li'	lx'
cm4	cm4
(9)	(10)
1/12.b.h^3	(8) + (9)
66,992	29,422,531
4,288	9,183,650
2,266,607	5,660,867
	44,267,048

ior-19

I.4.2. Load on a beam

Span	=	19.60 m	thickness of asphalt	=	0.050 m
Clear span	=	19.00 m	thickness of slab	=	0.245 m
nos of beam	=	3.00	unit weight of asphalt	=	2.300 t/m3
			unit weight of concrete	=	2.500 t/m3

1. Loading

Width effective of beam = be = 1.75 m

a. Dead load

description	width m	x	thick m	area m ²	x	length m	x	unit weight t/m3	weight t/m3
a.1 Beam	gb			0.2570	x	1.00	x	2.500	= 0.643
a.2 Slab	gs	1.75	x	0.245	x	1.00	x	2.500	= 1.072
a.3 Asphalt	ga	1.75	x	0.050	x	1.00	x	2.300	= 0.201

b. Live load

b.1 Line load = p

Impact coefficient

$$k = 1 + \frac{20}{(50 + L)}$$

$$P = 12.00 \text{ ton/lane}$$

$$Q = 2.20 \text{ ton/lane}$$

$$q = 2.75 \text{ t/m}$$

$$g = 0.643 \text{ t/m'}$$

$$g = 1.900 \text{ m}$$

$$g = 2.50 \text{ m}$$

$$g = 0.643 \text{ t/m'}$$

$$g = 1.900 \text{ m}$$

$$g = 2.50 \text{ m}$$

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$$g = 0.643 \text{ t/m'}$$

$$g = 1.900 \text{ m}$$

Project : Median Flood Control	Japan International	Calculation sheet
Channel : Perout River	Cooperation Agency	Date
Structure : Br - F7	(JICA)	Nos of sheet
Station : FW 38 + 78		Prepared by

1.4.3. Prestress Force and numbers of tendon

Span	=	19.00 m
M minimum	=	28,993 kg.cm
M transfer	=	28,993 kg.cm
M dead load	=	77,363 kg.cm
M composite	=	119,043 kg.cm

Characteristic of Concrete

specified strength (fc)	=	0.00 kg/cm ²
Concrete class, K-400	=	190.00 kg/cm ²
fc'	=	180.00 kg/cm ²
fc*	=	0.00 kg/cm ²

Compression strength

Section properties

	Beam	Composite
h	90 cm	thick of slab = 17.50 cm
A	2,572 cm ²	thick of panel = 7 cm
Yt	53.68 cm	h' = 114.50 cm
Yb	36.32 cm	Ac' = 6.248 cm ²
I	2,266,607 cm ⁴	Yt' = 39.40 cm
Wt	42,227 cm ³	Yb' = 75.10 cm
Wb	62,399 cm ³	I' = 9,021,046 cm ⁴
kt	24.26 cm	Wt' = 228,984 cm ³
kb	16.42 cm	Wb' = 120,114 cm ³
e	25.07 cm	kt' = 19.20 cm ³
		kb' = 36.70 cm ³
		n = 0.75

Computation prestressing force

1. Initial case

1st equation top fiber

Pi	>	$\frac{f_t W_t - M_g}{(k_b - e)}$
----	---	-----------------------------------

by self weight of beam

Mg = 2,899,300 kg.cm

ft = 0.00 kg/cm²

Wt = 42,227.00 cm³

ft.Wt = 0.00 cm³

kb = 16.42 cm

e = 25.07 cm

ft.Wt - M mi = 0.00 -

kb - e = 16.42 -

Pi > $\frac{(2,899,300)}{(8.65)}$ = 335,179

> 335.18 ton

2,899,300 = (2,899,300) kg.cm

25.07 = (8.65) cm

335.18 ton

2.3 asphalt

gas = 0.201 t/m³

Point	Distance m	Load t/m ²	Reaction ton	I.L Moment t.m	Area of I.L m ²	Moment t.m
1	2.50	0.201	0.868	2.17	20.63	4.151
2	5.00	0.201	0.737	3.68	35.00	7.044
3	7.50	0.201	0.605	4.54	43.13	8.679
4	9.50	0.201	0.500	4.75	45.13	9.081

2.4 uniform load

q = 1.400 t/m²

Point	Distance m	Load t/m ²	Reaction ton	I.L Moment t.m	Area of I.L m ²	Moment t.m
1	2.50	1.400	0.868	2.17	20.63	28.875
2	5.00	1.400	0.737	3.68	35.00	49.000
3	7.50	1.400	0.605	4.54	43.13	60.375
4	9.50	1.400	0.500	4.75	45.13	63.175

2.5 line load

p = 9.85 t/m²

Point	Distance m	Load t/m ²	Reaction ton	I.L Moment t.m	Moment t.m
1	2.50	9.850	0.868	2.17	21.384
2	5.00	9.850	0.737	3.68	36.289
3	7.50	9.850	0.605	4.54	44.713
4	9.50	9.850	0.500	4.75	46.787

(1). Initial case by self weight of beam = Mg

Distance	Mg
2.50	13.252
5.00	22.483
7.50	27.708
9.50	28.993

(2). Transfer case (Mg + M slab) = Mdl

Distance	Mg	M slab	Mdl = Mg + Ms
2.50	13.25	22.11	35.359
5.00	22.49	37.52	60.003
7.50	27.71	46.22	73.932
9.50	28.99	48.37	77.361

(3). Final case (composite section)

Distance	Mg uniform load	Mp live load	Ml live load	Ma by asphalt	Mc = Ma + Ml
2.50	28.88	21.38	50.26	4.15	54.410
5.00	49.00	36.29	85.29	7.04	92.332
7.50	60.38	44.71	105.09	8.68	113.767
9.50	63.18	46.79	109.96	9.08	119.043

Mg = moment support by precast concrete prestressed

Mdl = moment support by precast concrete prestressed (beam + concrete institu + cross beam + side walks)

Ml = moment support by composite section (DI + LL)

Mc = moment support by composite section (DI + LL + Asphalt)

3. Influence line of shear and shear

	Load	Distance m	Reaction ton	Area of I.L m ²	Shear ton
3.1 self weight	0.643	0.00	1.000	9.50	6.104
3.2 slab	1.072	0.00	1.000	9.50	10.183
3.3 asphalt	0.201	0.00	1.000	9.50	1.912
3.4 uniform load	1.400	0.00	1.000	9.50	13.900
3.5 line load	9.850	0.00	1.000	9.50	9.850
					41.348

3. Dead load
1st equation top fiber

PI	>	$f_{cs} W_t - M d$ ----- $n_i (k_b - e)$	by beam and concrete insitu
M d l	=	7,736,300 kg.cm	
f _{ti}	=	0.00 kg/cm ²	
W _b	=	42,227.00 cm ³	
f _{ti} W _b	=	0 x	42,227 = 0.00 cm
k _b	=	16.42 cm	
e	=	25.07 cm	
(k _b - e)	=	16.42 -	25.07 = (8.65) cm
f _{ti} W _b - M d l	=	0 -	7,736,300 = (7,736,300) kg.cm
n _i (k _b - e)	=	0.75 x	(8.65) = (6.49) cm
PI	>	$\frac{(7,736,300)}{(6.49)}$	= 1,192,493 kg
	>	1,192.49 ton	

2nd equation bottom fiber

PI	<	$f_{ts} W_b + M d$ ----- $n_i (e + k_t)$	by beam and concrete insitu
M d l	=	7,736,300 kg.cm	
f _{ti}	=	0.00 kg/cm ²	
W _b	=	62,399.00 kg/cm ²	
f _{ti} W _b	=	0 x	62,399.00 = 0.00 cm
k _t	=	24.26 cm	
e	=	25.07 cm	
(e + k _t)	=	25.07 +	24.26 = 49.33 cm
f _{ti} W _b + M d	=	0 +	7,736,300 = 7,736,300 cm
n _i (e + k _t)	=	0.75 x	49.33 = 37.00 cm
PI	<	$\frac{7,736,300}{37.00}$	= 209,103 kg
	<	209.10 ton	

4. Service load
3 rd equation top fiber

PI	<	$f_{cs} W_t - M d l - M c W_t / W_t'$ ----- $n_i (k_b - e)$	by service load
M c	=	11,904,300 kg.cm	
M d l	=	7,736,300 kg.cm	
f _{cs}	=	180.00 kg/cm ²	
W _t	=	42,227.00 cm ³	
W _{t'}	=	228,984.00 cm ³	
W _b	=	228,984.00 cm ³	
k _b	=	16.42 cm	
e	=	25.07 cm	
(k _b - e)	=	16.42 -	25.07 = (8.65) cm
W _t W _{t'}	=	42,227.00 x	228,984.00 = 0.18

2nd equation bottom fiber

PI	<	$f_{cl} W_b + M g$ ----- $(e + k_t)$	by self weight of beam
M g	=	2,899,300 kg.cm	
f _{cl}	=	190.00 kg/cm ²	
W _b	=	62,399.00 kg/cm ²	
f _{cl} W _b	=	11,855,810 cm ³	
k _t	=	24.26 cm ³	
e	=	25.07 cm	
M min + f _{cl}	=	11,855,810 +	14,755,110 kg.cm
k _t + e	=	24.26 +	25.07 = 49.33 cm
PI	<	$\frac{14,755,110}{49.33}$	= 299,110
	<	299.11 ton	

2. After transfer

1st equation top fiber

PI	>	$f_{ti} W_t - M g$ ----- $n_i (k_b - e)$	after prestress losses
M g	=	2,899,300 kg.cm	
f _{ti}	=	0.00 kg/cm ²	
W _t	=	42,227.00 cm ³	
f _{ti} W _t	=	0.00 cm ³	
k _b	=	16.42 cm	
e	=	25.07 cm	
(k _b - e)	=	16.42 -	25.07 = (8.65) cm
f _{ti} W _t - M g	=	0.00 -	2,899,300 = (2,899,300) cm
n _i (k _b - e)	=	0.75 x	(8.65) = (6.49) cm
PI	>	$\frac{(2,899,300)}{(6.49)}$	= 446,906 kg
	>	446.91 ton	

2nd equation bottom fiber

PI	<	$f_{cl} W_b + M g$ ----- $n_i (e + k_t)$	after prestress losses
M g	=	2,899,300 kg.cm	
f _{cl}	=	190.00 kg/cm ²	
W _b	=	62,399.00 kg/cm ²	
f _{cl} W _b	=	190 x	62,399.00 = 11,855,810 cm
k _t	=	24.26 cm ³	
e	=	25.07 cm	
(k _t + e)	=	24.26 +	25.07 = 49.33 cm
f _{cl} W _b + M g	=	11,855,810 +	2,899,300 = 14,755,110 cm
n _i (k _t + e)	=	0.75 x	49.33 = 37.00 cm
PI	<	$\frac{14,755,110}{37.00}$	= 398,814 kg
	<	398.81 ton	

Mc.Wt / Wt'	=	11,904,300.00 x	0.18	=	2,20E+06 kg.cm
fcs.Wt	=	180.00 x	42,227.00	=	7,60E+06 kg.cm
fcs.Wt - Md	=	7,600,860.00 -	7,74E+06	=	 kg.cm
n.(Wb - e)	=	(135,440.00) -	2,195,275.11	=	 kg.cm
	=	0.75 x	(8.65)	=	(6.49) kg.cm
Pj	<	-2,33E+06	=	359,262.44	
		(6.49)			
4th equation bottom fiber	<	359.26			
Pj	>	$\frac{f_{ts}W_b + Mdl + Mc.W_b/W_b'}{n(e + kt)}$			
Mc	=	11,904,300.00 kg.cm			
Mdl	=	7,736,300.00 kg.cm			
fcs	=	0.00 kg/cm2			
Wt'	=	228,984.00 cm3			
Wb	=	62,399.00 cm3			
Wb'	=	120,114.00 cm3			
kt	=	24.26 cm			
e	=	25.07 cm			
(e + kt)	=	25.07 +	24.26	=	49.33 cm
Wb/Wb'	=	62,399 :	120,114	=	0.52
Mc.Wb/Wb'	=	11,904,300 x	0.52	=	6,18E+06 kg.cm
fcs.Wb	=	0 x	62,399	=	0 kg.cm
fcs.Wb + Mk	=	0 +	773,6300	=	7,74E+06 kg.cm
n.(e + kt)	=	773,6300 +	618,4261.74	=	1,39E+07 kg.cm
	=	0.75 x	49.33	=	37.00 cm
Pj	>	13,920,562	=	376,257 kg	
		37.00			
	>	376.26 ton			

Initial case	
Top fibre	Pj > 335.18 ton
Bottom fibre	Pj < 299.11 ton
Transfer	
Top fibre	Pj > 446.91 ton
Bottom fibre	Pj < 398.81 ton
Dead load	
Top fibre	Pj < 1,192.49 ton
Bottom fibre	Pj > 209.10 ton
Service case	
Top fibre	Pj < 359.26 ton
Bottom fibre	Pj > 376.26 ton
335.18 < Pj <	299.11

Pj	=	299,110 kg
1 Mpa	=	102 kg/cm2
1 kg/cm2	=	0.098 Mpa
1 kN	=	102 Kg
1 kg	=	0.0098 kN

used strand rade 270					
nominal diameter	=	12.7 mm			
nominal area	=	94.2 mm2			
	=	0.9420 cm2			
fu	=	1,860 Mpa			10.2
	=	1860 x			
	=	18,972 kg/cm2			
at jacking					
$\eta = 0.80 \times fu$	=	0.8 x	fu		18972
	=	0.8 x			
	=	1,488 Mpa			
	=	15,178 kg/cm2			
P jacking	=	Pj	x	As	
	=	η	x		0.942
	=	15177.6 x			
	=	14,297 kg / strand			
nos of strand / tendon	=	7.00			
Pj / tendon	=	Pj / strand x nos of strand			7
	=	14,297 x			
	=	100,081 kg / tendon			
nos of tendon	=	total force : Pj			
	=	299,110 : 100,081			
	=	2.99			
	=	3.00 nos			
say					

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1.4.4 Prestress losses

1. Due to elastic shortening

span = 19.00 m

section properties

Ht = 90.00 cm

Ac = 2,573 cm²

Ix = 2,266,607 cm⁴

Wt = 42,228 cm³

Wb = 62,399 cm³

Yt = 53.68 cm

Yb = 36.32 cm

Concret class K - 400

(fc) = 400 kg/cm²

Nos strand / tendon = 7.00 nos

nos of tendon = 3.00 nos

Area of strand = 94.2 mm²

Total area = 0.942 cm²

Ec = 343,327 kg/cm²

Es = 1,997,200 kg/cm²

modular ratio n = -----

Es

Ec

1,997,200

343,327

5.53

Allowance stress in prestressing tendon = fpu

fpu = 1,660 Mpa

= 1,860 x 10.2

= 18,972 kg/cm²

Stress at jacking

= 0.8 x 18,972

= 15,178 kg/cm²

Jacking force

= 15,177.60 x 0.942

= 14,297 kg / strand

Jacking force

= 7.00 x 14,297

P jack = 100,081 kg / tendon

total force at jacking per tendon = P jt

P jt = P jack x nos of tendon

= 100,081 x 3.00

= 300,243 kg

at the jacking

tendon	Yb	Ys	e	Pj	Ac	Pj/Ac	(e Ac/Wb) (1 + e Ac/Wb)
no	cm	cm	cm	kg	cm ²	kg/cm ²	kg cm
3	36.32	13.00	23.32	100,081	2573	38.90	0.9616
2	36.32	6.50	29.82	100,081	2573	38.90	1.2296
1	36.32	6.50	29.82	100,081	2573	38.90	1.2296

Stress losses

tendon	fcj	n	n.fci	fs	n.fci / ft x 100
no	kg/cm ²		cm	kg/cm ²	
3	76.30	5.53	421.62	15,178	2.76
2	86.72	5.53	479.23	15,178	3.16
1	86.72	5.53	479.23	15,178	3.16

9.093

nos - 19

average 9.093 : 3 = 3.0310 %

2. Due to friction

$f_x = f_j \times e^{(-u\alpha + Ks)}$
 f_j = stress in the strand at the jacking end or live anchorage
 u = stress at dead anchorage
 α = coefficient of angular friction
 x = change in the angle between the force at anchorage to section X
 f = curvilinear length of the tendon anchorage to section x
 f = depth of parabola

tendon parabola curve

$$Y = 4x/l^2 \times (l-x)$$

angle of tendon parabola to horizontal line

$$dx/dy = 0$$

$x = 0.00 \text{ m}$
 $e = 0 \text{ m}$
 $l = 19.00 \text{ m}$

$$\alpha_1 = 4x/l \text{ radial}$$

$$\alpha_1 = 4.00 \times 0$$

$$\alpha_1 = 19.00$$

$$\alpha_1 = 0.0000 \text{ radial}$$

$$\alpha = 2.00 \times \alpha_1$$

$$\alpha = 2.00 \times 0.0000$$

$$\alpha = 0.0000$$

no tendon value of sag parabola each tendon

tendon no.	Distance	H	Ys	f
top fiber to no.3	0.20			
3 no.3 to no.2	0.25	0.700	0.130	0.570 m
2 no.2 to no.1	0.00	0.450	0.065	0.385 m
1 no.1 to outside	0.25	0.200	0.065	0.135 m

no tendon value of depth parabola each tendon

alpha 1	=	4	x	f	L	=	alpha 1	radial
3	alpha 1	=	4.00 x	0.570	19.00	=	0.12	radial
2	alpha 1	=	4.00 x	0.385	19.00	=	0.081052	radial
1	alpha 1	=	4.00 x	0.135	19.00	=	0.028421	radial

if $x = 0$, $\alpha = \alpha_1$

if $x = l/2$, $\alpha = 2 \times \alpha_1$

value of Px each tendon

description	tendon - 3		tendon - 2		tendon - 1	
	x (m)		x (m)		x (m)	
alpha	0.000	9.500	0.000	9.50	0.000	9.50
u	0.1200	0.2400	0.0811	0.1621	0.0284	0.0568
u.alpha	0.2000	0.2000	0.2000	0.2000	0.2000	0.2000
K	0.0240	0.0480	0.0162	0.0324	0.0057	0.0114
Kx	0.0041	0.0041	0.0041	0.0041	0.0041	0.0041
Kx	0.0000	0.0390	0.0000	0.0390	0.0000	0.0390
u.alpha + K x	0.0240	0.0870	0.0162	0.0714	0.0057	0.0503
e	2.7183	2.7183	2.7183	2.7183	2.7183	2.7183
$e^{-u(\alpha + Kx)}$	0.9763	0.9167	0.9839	0.9311	0.9943	0.9509
fx	0.9763	0.9167	0.9839	0.9311	0.9943	0.9509

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1.4.5 Centre Gravity of tendon

for span = 19.00 m

Y = centre gravity of tendon (cgs) to the centre gravity of concrete (cgc)

dia. of strand = 12.70 mm

depth = 90.00 cm

area = 94.20 mm²

= 0.9420 cm²

Nos of strand = 6.00

area per tendon = 5.6520 cm²

Yb = 36.32 cm

Yt = 53.68 cm

top cover concrete = tc

tc = 20.00 cm

a. at stressing of tendon

a.1 cgs tendon at stressing tendon no.3 at edge of span

tendon no	distance to each cm	tendon no	distance to tendon cgc cm	area of tendon As cm ²	static moment		
No.3 to no.2	= 25.00	No.3	33.68	5.6520 x	33.68	=	190.38
No.3 to cgc	= 8.68	No.2	25.00	5.6520 x	25.00	=	141.30
	33.68						331.66
e at edge	= 331.66		29.34 cm				
	= 11.304						

a.2 cgs tendon at stressing tendon no.3 at middle of span

tendon no	distance to each cm	tendon no	distance to tendon cgc cm	area of tendon As cm ²	static moment		
No.3 to no.2	= 6.50	No.3	13.00	5.652 x	13.00	=	73.476
No.2 to outside	= 6.50	No.2	6.50	5.652 x	6.50	=	36.738
No.1 to outside	= 6.50	No.1	6.50	5.652 x	6.50	=	36.738
	13.00			5.652			73.476
Ys	= 73.48		13.00 cm				
e jack at midlle	= 5.652	Yb =	Ys				
	= 36.32	-	13.00				
	= 23.32 cm						

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No.1 to outside	= 6.50	No.1	6.50	5.652 x	6.50	=	36.738
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Ys	= 73.48		13.00 cm				
e jack at midlle	= 5.652	Yb =	Ys				
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	13.00			5.652			73.476
Ys	= 73.48		13.00 cm				
e jack at midlle	= 5.652	Yb =	Ys				
	= 36.32	-	13.00				
	= 23.32 cm						

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area = 94.20 mm²

= 0.9420 cm²

Nos of strand = 6.00

area per tendon = 5.6520 cm²

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Yt = 53.68 cm

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tc = 20.00 cm

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a.1 cgs tendon at stressing tendon no.3 at edge of span

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No.1 to outside	= 6.50	No.1	6.50	5.652 x	6.50	=	36.738
	13.00			5.652			73.476
Ys	= 73.48		13.00 cm				
e jack at midlle	= 5.652	Yb =	Ys				
	= 36.32	-	13.00				
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Yb = 36.32 cm

Yt = 53.68 cm

top cover concrete = tc

tc = 20.00 cm

a. at stressing of tendon

a.1 cgs tendon at stressing tendon no.3 at edge of span

tendon no	distance to each cm	tendon no	distance to tendon cgc cm	area of tendon As cm ²	static moment		
No.3 to no.2	= 25.00	No.3	33.68	5.6520 x	33.68	=	190.38
No.3 to cgc	= 8.68	No.2	25.00	5.6520 x	25.00	=	141.30
	33.68						331.66
e at edge	= 331.66		29.34 cm				
	= 11.304						

a.2 cgs tendon at stressing tendon no.3 at middle of span

tendon no	distance to each cm	tendon no	distance to tendon cgc cm	area of tendon As cm ²	static moment		
No.3 to no.2	= 6.50	No.3	13.00	5.652 x	13.00	=	73.476
No.2 to outside	= 6.50	No.2	6.50	5.652 x	6.50	=	36.738
No.1 to outside	= 6.50	No.1	6.50	5.652 x	6.50	=	36.738
	13.00			5.652			73.476
Ys	= 73.48		13.00 cm				
e jack at midlle	= 5.652	Yb =	Ys				
	= 36.32	-	13.00				
	= 23.32 cm						

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Calculation sheet

1.4.5 Centre Gravity of tendon

for span = 19.00 m

Y = centre gravity of tendon (cgs) to the centre gravity of concrete (cgc)

dia. of strand = 12.70 mm

depth = 90.00 cm

area = 94.20 mm²

= 0.9420 cm²

Nos of strand = 6.00

area per tendon = 5.6520 cm²

Yb = 36.32 cm

Yt = 53.68 cm

top cover concrete = tc

tc = 20.00 cm

a. at stressing of tendon

a.1 cgs tendon at stressing tendon no.3 at edge of span

tendon no	distance to each cm	tendon no	distance to tendon cgc cm	area of tendon As cm ²	static moment		
No.3 to no.2	= 25.00	No.3	33.68	5.6520 x	33.68	=	190.38
No.3 to cgc	= 8.68	No.2	25.00	5.6520 x	25.00	=	141.30
	33.68						331.66
e at edge	= 331.66		29.34 cm				
	= 11.304						

a.2 cgs tendon at stressing tendon no.3 at middle of span

tendon no	distance to each cm	tendon no	distance to tendon cgc cm	area of tendon As cm ²	static moment		
No.3 to no.2	= 6.50	No.3	13.00	5.652 x	13.00	=	73.476
No.2 to outside	= 6.50	No.2	6.50	5.652 x	6.50	=	36.738
No.1 to outside	= 6.50	No.1	6.50	5.652 x	6.50	=	36.738
	13.00			5.652			73.476
Ys	= 73.48		13.00 cm				
e jack at midlle	= 5.652	Yb =	Ys				
	= 36.32	-	13.00				
	= 23.32 cm						

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1.4.5 Centre Gravity of tendon

for span = 19.00 m

Y = centre gravity of tendon (cgs) to the centre gravity of concrete (cgc)

dia. of strand = 12.70 mm

depth = 90.00 cm

area = 94.20 mm²

= 0.9420 cm²

Nos of strand = 6.00

area per tendon = 5.6520 cm²

Yb = 36.32 cm

Yt = 53.68 cm

top cover concrete = tc

tc = 20.00 cm

a. at stressing of tendon

a.1 cgs tendon at stressing tendon no.3 at edge of span

tendon no	distance to each cm	tendon no	distance to tendon cgc cm	area of tendon As cm ²	static moment		
No.3 to no.2	= 25.00	No.3	33.68	5.6520 x	33.68	=	190.38
No.3 to cgc	= 8.68	No.2	25.00	5.6520 x	25.00	=	141.30
	33.68						331.66
e at edge	= 331.66		29.34 cm				
	= 11.304						

a.2 cgs tendon at stressing tendon no.3 at middle of span

tendon no	distance to each cm	tendon no	distance to tendon cgc cm	area of tendon As cm ²	static moment		
No.3 to no.2	= 6.50	No.3	13.00	5.652 x	13.00	=	73.476
No.2 to outside	= 6.50	No.2	6.50	5.652 x	6.50	=	36.738
No.1 to outside	= 6.50	No.1	6.50	5.652 x	6.50	=	36.738
	13.00			5.652			73.476
Ys	= 73.48		13.00 cm				
e jack at midlle	= 5.652	Yb =	Ys				
	= 36.32	-	13.00				
	= 23.32 cm						

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Date

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Calculation sheet

1.4.5 Centre Gravity of tendon

for span = 19.00 m

Y = centre gravity of tendon (cgs) to the centre gravity of concrete (cgc)

dia. of strand = 12.70 mm

depth = 90.00 cm

area = 94.20 mm²

= 0.9420 cm²

Nos of strand = 6.00

area per tendon = 5.6520 cm²

Yb = 36.32 cm

Yt = 53.68 cm

top cover

	X = 0	X = l/2
tendon no.	fx	fx
3	0.976	0.917
2	0.984	0.931
1	0.994	0.951
sum	2.955	2.799
average	= 0.985	+ 0.933
	= 0.985	+ 0.933
average of fx	= 2.00	
	= 0.9589	fx jacking
fx jacking	= 0.9589	
	15.178	
	0.9589	
	15.828 kg/cm ²	
stress lost	= 15.828	- 15.178
	= 650.81	kg/cm ²
percentage stress lost	= 650.81	x 100
	= 15177.8	
	= 4.288	%
3. Due to anchorage set		
stress losses	= delta / l x Es	
Es	= 1,897,200	
assumed slip	= 0.5	
stress	= delta	
	= 0.50	
	= 1,900	
	= 499	
stress losses	= 15,178	
	= 3,289	
total stress lost	= 3,031	
	= 10.61	
Initial forces / tendon	= (1.00 - 0.1061) x P jacking	
P i / tendon	= 0.894 x 100,081	
	= 89,464	
Initial force after transfer	= Initial forces x nos tendon	
	= 89,464 x 3.00	
P initial	= 268,392	
Pj	= 300,243	

b. at transfer									
b.1 cgs tendon at edge of span									
tendon no	distance to tendon		distance to outside		area of tendon		static moment		As.Y
	each	no	cm	cm	cm ²	cm	Y	cm	
No.3 to No.2	=	25.00	No.3	70.00	5.6520 x	70.00	=	395.64	
No.2 to No.1	=	25.00	No.2	45.00	5.6520 x	45.00	=	254.34	
No.1 to outside	=	20.00	No.1	20.00	5.6520 x	20.00	=	113.04	
		70.00			16.96			763.02	
Ys	=	763.02		45.00	cm				
		16.956							
e at edge	=	Yb	-	Ys					
	=	36.32	-	45.00					
	=	(8.68)	cm						
b.2 cgs tendon at middle of span									
Strand no	distance to strand		distance to outside		area of strand		static moment		As.Y
	each	no	cm	cm	cm ²	cm	Y	cm	
No.3 to No.2	=	6.50	No.3	13.00	5.6520 x	13.00	=	73.48	
No.2 to No.1	=	0.00	No.2	6.50	5.6520 x	6.50	=	36.74	
No.1 to outside	=	6.50	No.1	6.50	5.6520 x	6.50	=	36.74	
		13.00			16.96			146.95	
Ys	=	146.95		8.67	cm				
		16.956							
e at middle	=	Yb	-	Ys					
	=	36.32	-	8.67					
	=	27.65	cm						

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1.4.6 Layout of tendon

span	=	19.00	m	
description				
Combined moment (tm) at various length (m)				
		2.50	5.00	7.50
2.1. beam		13.252	22.490	27.708
2.2. slab		37.516	46.225	48.370
2.3. asphalt		4.150	7.044	8.679
2.4. uniform load		28.875	49.000	60.375
2.5. line load		50.260	89.290	105.090

(1) M minimum = 2.1	13.25	22.49	27.71	28.99
(2) M dead load = 2.1+2.2	35.36	60.01	73.93	77.36
(3) M live load = 2.4+2.5	79.14	138.29	165.47	171.14
(4) M composite = (3) + 2.3	83.29	145.33	174.14	180.22

Pj	=	300,243	kg
Pi	=	268,392	kg
n	=	0.75	
P	=	0.75	x
	=	225,182	kg
	=	300,243	kg

case	Pia	P=n.Pi
Initial case	300,243	
After transfer	268,392	
Dead load	225,182	
Live load	225,182	
Composite	225,182	

Distance	m	2.50	5.00	7.50	8.00
Mg	tm	13.25	22.49	27.71	28.99
Prestressed force	kg	1,325,200	2,249,000	2,770,800	2,899,300
arm	cm	300,243	300,243	300,243	300,243
	cm	4.41	7.49	9.23	9.66

After transfer					
Distance	m	2.50	5.00	7.50	8.00
M transfer	tm	13.25	22.49	27.71	28.99
Prestressed force	kg	1,325,200	2,249,000	2,770,800	2,899,300
Arm	cm	268,392	268,392	268,392	268,392
	cm	4.94	8.38	10.32	10.80

Dead load					
Distance	m	2.50	5.00	7.50	8.00
M dead load	tm	35.36	60.01	73.93	77.36
Prestressed force	kg	3,536,200	6,000,600	7,393,300	7,736,300
Arm	cm	225,182	225,182	225,182	225,182
	cm	15.70	26.65	32.83	34.36

Service load					
Distance	m	2.50	5.00	7.50	8.00
M composite	tm	83.29	145.33	174.14	180.22
Prestressed force	kg	8,328,500	14,533,400	17,414,400	18,022,000
Arm	cm	225,182	225,182	225,182	225,182
	cm	36.99	64.54	77.33	80.03

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1.4.7 Stress Control

Working moment

M minimum	=	26,993	t.m
M after transfer	=	2,899,300	kg.cm
M dead load	=	28,993	t.m
M composite	=	2,899,300	kg.cm
	=	77,361	t.m
	=	7,736,100	kg.cm
	=	116,075	t.m
	=	119,043	kg.cm
	=	27.32	cm

Alloable stresses in prestressed concrete

Initial case			
fci	=	190.0	kg/cm ²
fts	=	0.0	kg/cm ²
Final case			
fcs	=	180.0	kg/cm ²
fts	=	0.0	kg/cm ²

Section properties

Girder	Composite	thick of slab	thick of pan
h	=	90	cm
Ac	=	2,573	cm ²
Yt	=	53.68	cm
Yb	=	36.32	cm
I	=	2,266,607	cm ⁴
Wt	=	42,228	cm ³
Wb	=	62,399	cm ³
kt	=	24.26	cm
kb	=	16.42	cm
e	=	27.32	cm
n	=	0.75	

Computation of stresses

Force at jacking			
Pj	=	300,243	kg
Pl	=	268,392	kg
P	=	n	x
	=	0.75	x
	=	225,182	kg

Moment by eccentricity tendon

Initial case			
Pjack x e	=	300,243	x
	=	7,001,667	kg.cm
Transfer			
P initial x e	=	268,392	x
	=	7,421,039	kg.cm
Final case			
P final x e	=	225,182	x
	=	6,226,289	kg.cm

Actually working stresses

1. Initial case

		top fibre			
Pj	=	Pj x e	=	Mg	>
Ac	=	Wt	=	Wt	>
300,243	=	7,001,667	=	2,899,300	>
2,573	=	42,228	=	42,228	>
116.69	=	165.81	=	68.66	>
19.54	=	0	=	0	>

bottom fibre

Pj	=	Pj x e	=	Mg	<
Ac	=	Wb	=	Wb	<
300,243	=	7,001,667	=	2,899,300	<
2,573	=	62,399	=	62,399	<
116.69	=	112.21	=	46.46	<
182.43	=	190	=	190	<

2. after transfer

		top fibre			
Pl	=	Pl x e	=	Mtr	>
Ac	=	Wt	=	Wt	>
268,392	=	7,421,039	=	2,899,300	>
2,573	=	42,228	=	42,228	>
104.31	=	175.74	=	68.66	>
(2.77)	=	0	=	0	>

bottom fibre

P	=	P x e	=	Mtr	<
Ac	=	Wb	=	Wb	<
268,392	=	7,421,039	=	2,899,300	<
2,573	=	62,399	=	62,399	<
104.31	=	118.93	=	46.46	<
176.78	=	180.00	=	180	<

3. Dead load (self weight beam + concrete cast in place)

		top fibre			
P	=	P x e	=	Mdl	>
Ac	=	Wt	=	Wt	>
225,182	=	6,226,289	=	7,736,100	>
2,573	=	42,228	=	42,228	>

=	87.52	-	147.44	+	183.20	>	0 kg/cm ²
=	123.27	>	0.00	kg/cm ²			
<u>bottom fibre</u>							
fb	P	+	Wb	-	Mdl	<	180 kg/cm ²
	Ac				Wb		
	268,392	+	7,421,039	-	7,738,100	<	180 kg/cm ²
	2,573		62,399		62,399		
	104.31	+	118.93	-	123.98	<	180 kg/cm ²
	99.26	<	180	kg/cm ²			
4. at service load							
beam unshore							
<u>top fibre</u>							
ftaa	M composite		Yt =	depth of faa	=	39.40	cm
	Wt		fs =	depth of slab	=	22.00	cm
	119,043		Yt = Yt - fs		=	17.40	cm
	228,984						
=	0.52	<	180.00	kg/cm ²	on top of slab deck		
faa : fa = Yt : Yt							
fa x Yt = faa x Yt							
fa = faa x Yt/Yt							
fa	17.4	x	0.52				
	39.4						
=	0.23	<	180	kg/cm ²	(on bottom of slab deck)		
total fa	concrete compressive on top of girder						
total fa	fa dl	+	fa composite	<	180	kg/cm ²	
total fa	123.27	+	0.23	=	123.50	kg/cm ²	
	<u>bottom fibre</u>						
fb	concrete compressive on bottom of girder in composite						
fb	M composite		fb c	>	0	kg/cm ²	
	Wb			=	98.27	kg/cm ²	
	119,043			>	0.00	kg/cm ²	
	120,114						
	0.99	kg/cm ²					
fb	fb dl	-	fb c	=	98.26	kg/cm ²	
	98.26	-	0.99	=	99.25	kg/cm ²	

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1.4.6 Deflection of beam

1. deflection computation

deflection is moment at any point by * bending moment * as load

1.a deflection due to uniform load

load	=	g
Mo	=	1/8 . g . l ² / 2
Aa	=	4/3 . a . b
a	=	l/2
b	=	Mo
Aa	=	4/3 . (l/2) . Mo
R	=	2/3 . l . Mo
	=	1/2 . Aa
	=	2/6 . l . Mo
xp1	=	xp1
	=	3 . a / 8
	=	3 . l / 16
	=	3 / 16 . l
M max	=	ya
	=	R . l/2 - Aa / 2 . xp1
	=	2/6 . l . Mo . l/2 - 2/6 . l . Mo . (3/16 . l)
	=	Mo . l ² / 2 . (1/6 - 1/16)
	=	Mo . l ² / 2 . (8/48 - 3/48)
	=	Mo . (5/48 . l ²)

$$ya = \frac{5 \cdot Mo \cdot l^2}{48 \cdot EI}$$

1.b deflection due to tendon forces

load	=	P
Mo	=	P . e2
Ab	=	Mo . l
R	=	(Mo . l) / 2
axis centroid	=	xp2
	=	l/4
M max	=	yb
	=	Re2 . l/2 - Ab . l/2 . xp2
	=	(Mo . l) / 2 . l/2 - (P . e2 . l) / 2 . l/4
	=	(Mo . l) . l/4 - l/8
	=	(Mo . l) . l . (2/8 - 1/8)
	=	(Mo . l) . l . (2/8 - 1/8)

$$yb = \frac{Mo \cdot l^2}{8 \cdot EI}$$

1.c deflection due to knife load

load	=	Pk
Mo	=	1/4 Pk . l
Ac	=	Mo / 2 . l/2
	=	Mo / 4 . l
	=	1/3 . l/2
	=	l/6

$$M_{max} = \frac{Yc}{4} = \frac{Mo}{4} \cdot \frac{1}{1/2} - \frac{Ac}{2} \cdot \frac{1}{1/6}$$

$$= \frac{Mo}{4} \cdot \frac{1}{1/2} - \frac{Mo}{4} \cdot \frac{1}{1/6}$$

$$= \frac{Mo}{4} \cdot \left(\frac{1}{1/2} - \frac{1}{1/6} \right)$$

$$= \frac{Mo}{4} \cdot \frac{1}{1/2} \left(1 - \frac{1}{3} \right)$$

$$= \frac{Mo}{12} \cdot \frac{1}{1/2}$$

$$Yc = \frac{Mo \cdot 1}{12EI}$$

l.d data	
span	19.00 m
E - c	1900 cm
Inertia	343,327 kg/cm ²
Ext	2,266,607 cm ⁴
	7,78E+11 kg/cm ²

II. loading	
weight of beam	0.643 t/m'
weight of slab	1.072 t/m'
asphalt	0.201 t/m'
uniform load	1.400 t/m'
line load	9.950 ton / line

III.1 by dead load							
load	coefficient	x	load	x	l^2	=	moment
			t/m'		m		kg.cm
by beam	0.125	x	0.643	x	361.0	=	2,901,538
slab deck	0.125	x	1.072	x	361.0	=	4,837,400
asphalt	0.125	x	0.201	x	361.0	=	907,013

III.2 moment due to prestressed force

III.2.1 at stressing of tendon no.3									
P Jacking		=	300.243 ton						
		=	300.243 kg						
P Initial		=	268.392 ton						
		=	268.392 kg						
e1		=	29.34 cm						
e2		=	23.32 cm						
e1 + e2		=	52.66 cm						
at edge		M2	=	P	x	e2			
			=	300.243	x	23.32	=	7,001,667 kg.cm	
at middle		M1	=	- P	x	(e1 + e2)	=		
			=	(300.243)	x	52.66	=	(15,810,796)kg.cm	
III.2.2 at transfer									
e1		=	(5.68) cm						
e2		=	27.65 cm						
e1 + e2		=	18.97 cm						
at edge		M2	=	P	x	e2			
			=	268.392	x	27.65	=	7,421,039 kg.cm	
at middle		M1	=	- P	x	(e1 + e2)	=		
			=	(268.392)	x	18.97	=	(5,091,396)kg.cm	

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IV. deflection computation

IV.1 by dead load

load	5/48	x	Mo	x	l ²	=	EI	=	Y
			coefficient		kg.cm		cm ²		mm
self weight bea	(1)	0.1042	x	2,901,538	x	3,610,000	:	7,78E+11	= 14.02
slab deck	(2)	0.1042	x	4,837,400	x	3,610,000	:	7,78E+11	= 23.38
asphalt	(3)	0.1042	x	907,013	x	3,610,000	:	7,78E+11	= 4.38

IV.1 by prestressing forces

IV.2.1 at stressing of tendon no.3

at edge	1/8	x	M2	x	l ²	=	EI	=	Y
			0.125	x	7,00E+06	x	3,610,000	:	7,78E+11
Ye5 =									40.60
at middle	5/48	x	M1	x	l ²	=	EI	=	Y
			0.1042	x	-1.58E+07	x	3,610,000	:	7,78E+11
Ym5 =									(76.40)
balance	=		(76.40)	+	40.60	=			mm
by self weight beam									14.02
total	=		(35.80)	+	14.02	=			mm

IV.2.2 at transfer

at edge	1/8	x	M2	x	l ²	=	EI	=	Y
			0.125	x	7,421,039	x	3,610,000	:	7,78E+11
Ye5 =									43.03
at middle	5/48	x	M1	x	l ²	=	EI	=	Y
			0.1042	x	(5,091,396)	x	3,610,000	:	7,78E+11
Yem =									(24.60)
balance	=		(24.60)	+	43.03	=			mm
by self weight beam									14.02
total deflection by tendon	=		18.43	+	14.02	=			mm

balance	=	(24.60)	+	43.03	=	16.43	mm
by self weight beam					=	14.02	mm
total deflection by tendon	=	18.43	+	14.02	=	32.45	mm

V. long term deflection

V.1 cracked beam

load	5/48	x	Mo	x	l ²	=	EI	=	Y
			coefficient		kg.cm		cm ²		mm
uniform load	(4)	0.1042	x	6,317,500	x	3,610,000	:	7,78E+11	= 30.53
line load	(5)	1/12	x	Mo	x	l ²	:	EI	= Y
			0.0833	x	4,728,250	x	3,610,000	:	7,78E+11
					total				18.27
									48.80

VI. long term deflection

VI.1 cracked beam

Cracking moment
Cracking moment is the moment for which the tensile stress on the extreme fibre of the concrete section reaches a value equal to the modulus of rupture of the concrete.

$$\text{modulus of rupture of the concrete} = \frac{fr}{tr} = \frac{7.5 \cdot \sqrt{f'c}}{40} \text{ psi}$$

$$f'c = 40 \text{ Mpa}$$

$$1 \text{ Mpa} = 145,0326 \text{ psi}$$

$$40 \text{ Mpa} = 5801,304 \text{ psi}$$

$$fr = (571.25) \text{ psi}$$

$$1 \text{ psi} = 0.07 \text{ kg/cm}^2$$

$$fr = (571.25) \text{ x } 0.07 = 39.99 \text{ kg/cm}^2$$

$$Mcr = n \cdot P \cdot (k + e) + fr \cdot wb$$

def-16

P	=	268,392	x	1,000	=	268,392	kg
n	=	0.75					
nP	=	0.75	x	268,392	=	201,294	kg
kt	=	24.26	cm				
e	=	27.55	cm				
(kt+e)	=	51.81	cm				
wb	=	62,400	cm ³				
fr x wb	=	(39.99)	x	62,400			
	=	(2,495,207.72)	kg.cm				
Mcr	=	nP	x	(kt+e)	+	fr wb	
	=	201,294	x	51.81	+	(2,495,208)	
	=	10,429,042	+	(2,495,207.72)			
	=	7,933,834	kg.cm				

deflection by cracking moment = y crack

y crack = $5/48 \times Mo \times l^2 : EI$

S/48	x	Mo	x	l^2	:	EI	=	yc
coefficient		kg.cm		cm ²		kg.cm ²		mm
0.1042	x	7,933,834	x	3,610,000	:	7,782E+11	=	
				2,98E+12	:	7,782E+11	=	36.34

deflection by tendon	=	32.45	mm	
total deflection	=	38.34	+	32.45
	=	70.79	mm	
permissible deflections	=	span		
	=	-----		
	=	300		
	=	19,000		
	=	-----		
	=	300		
	=	63.33	mm	ok

Project : Median Flood Control	Japan International Cooperation Agency (JICA)	Calculation sheet
Channel : Percut River		Date
Structure : Br - F7		Nos of sheet
Station : FW 38 + 76		Prepared by

1.4.9 Shear

Shear force by each load

Shear force	=	41,935	ton
Inertia	=	2,266,607	cm ⁴
Ht	=	90.00	cm
Yt	=	53.68	cm
Yb	=	36.32	cm
e	=	27.65	cm
Area of section	=	2,573	cm ²
Thick of web section	=	17	cm
Span	=	19.0	m
V	=	41.94	x
	=	41,935.00	kg

Part	Depth	Width	Area	Distance	At.Yi
i	t	w	Ai	to top = Yi	
(1)	(2)	(3)	(4)	(5)	(6)
depth of section					= 90.00
1	7.50	35.00	262.50	3.75	984.38
2	28.82	17.00	489.94	14.41	7,060.04
4	7.50	9.00	67.50	10.00	675.00
		Ay	819.94		8,719.41

Y	=	Sy	8,719.41
	=	tot.Ay	819.94
	=		10.63
Av.y	=	820	x
	=	8,719	cm ²
b.I	=	17.00	x
	=	38,532,319	cm ⁵
e	=	27.65	cm
4 x e	=	4.00	x
	=	110.60	cm
P jacking	=	257,351	kg
n	=	0.75	
P	=	193,013	kg
Vt	=	4 x e	
	=	1,1060	
	=	1,900	
	=	112.35	kg
V net	=	V	-
	=	41,935.00	-
	=	41,822.65	kg

vc = the shear resistance on the concrete
 Ay = area of the portion of section outside the shear plane
 outside the shear plane considered
 y = distance from neutral axis to centroid of Ay
 by = width of section at level y of the shear plane considered
 I = moment of inertia of the cross section

shear stress	Ay/y	x	V net
	b, I		
vc	8,719.41	x	41,823
	38,532.319		
vc^2	9.46	kg/cm2	
axial force	89.57	kg/cm2	
fx	P	193,013	
	Ac	2,573	
fx/2	75.01	kg/cm2	
(fx/2)^2	2.00		
vc^2	1,406.81		
(fx/2)^2	9.46		
vc^2 + (fx/2)^2	1,406.81		
	1,418.27		
R	@sqrt (vc^2 + (fx/2)^2)		
	37.63		
Principle compressive force = f			
f1 max	+/- fx/2 + R		
	37.51 + 37.63		
f2 min	75.14 kg/cm2	<	190.00 kg/cm2
	(37.51) + 37.63		
1 kg/cm2	0.13 kg/cm2	<	0.00 kg/cm2
fc	14.21 psi		
	400.00 kg/cm2		
vc min allowance = 2 @sqrt(fc)	5,684.24 psi		
	150.79 psi		
vc max allowance = 5 @sqrt(fc)	10.61 kg/cm2	<	190.00 kg/cm2
	376.97 psi		
	26.53 kg/cm2	<	190.00

4.1.6 SUPERSTRUCTURE (PC BEAM 16.0M LONG)

**PRESTRESSED CONCRETE BEAM COMPUTATION
CHANNEL, PERCUR RIVER
STRUCTURE, BR – P3
STATION, PE 115 + 06
WITH
CLEAR SPAN = 16.00 M**

I.4 Beam

- I.4.1 Section Properties of Beam**
- I.4.2 Loading on a Beam**
- I.4.3 Prestress force and Number Of Tendon Available**
- I.4.4 Prestress Loss**
- I.4.5 Centre Gravity of Tendon**
- I.4.6 Layout of Tendons**
- I.4.7 Stresses Control**
- I.4.8 Deflection of a Beam**
- I.4.9 Shear Force and Tensile Stress**

Project : Median Flood Control	Japan International Cooperation Agency (JICA)	Calculation sheet
Channel : Perout River		Date
Structure : Br. P3		Nos of sheet
Station : PE 115 + 08		Prepared by

L4 Main Beam									
14.1 Section Properties of beam									
Span	=	16.00 m							
Width of road way	=	7.00 m							
Nos of beam	=	5.00 m							
Space of a beam	=	1.75 m							
Thick of slab	=	0.175 m							

Section Properties									
moment static to top fiber									
part	depth	width	area	distance to	top fiber (Y)	AI	AI ²	AI ³	AI ⁴
1	cm	cm	cm ²	cm	cm	cm ²	cm ³	cm ⁴	cm ⁵
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
depth of beam									
1	7.5	35.0	262.5	3.8	984	3,691			
3	12.5	65.0	812.5	83.8	68,047	5,698,926			
2	70.0	17.0	1,190.0	42.5	50,575	2,149,438			
4	7.5	9.0	67.5	10.0	675	6,750			
5	10.0	24.0	240.0	74.2	17,800	1,320,167			
total						2,572.5	138,081		

li	cm ⁴	lx	cm ⁴	ly	cm ⁴
(9)	(10)	(11)	(12)	(13)	(14)
1/12.b.h ³	(8) + (9)				
1,230	4,922				
10,579	5,709,505				
485,917	2,635,354				
211	6,961				
1,333	1,321,500				
total		9,678,242			

Yt	=	Sx	138,081	=	53.68 cm
Yb	=	Ac	2,572.50	=	Yt
Yt	=	Ht	90.00	=	53.68 =
lx	=	tot.lx	9,678,242	=	Yt ² x
ly	=	9,678,242	2,573 x	=	2,881
lx	=	7,411,635			
ly	=	2,266,607 cm ⁴			

Section modulus			
top fiber	lx	2,266,607	=
bottom fiber	ly	53.68	=
Yt	lx	2,266,607	=
Yb	ly	36.32	=

top fiber	lt	Wb	Kern	62,400	24.26 cm
		Ac		2,573	
bottom fiber	lb	Wt		42,228	16.42 cm
		Ac		2,573	

Composite section	
width of flange = (be)	
ACI	
1	S
2	1/4
3	bw + 16 ht

Effective flange width (be) lesser than	
1	175.0 cm
2	400.0 cm
3	17.0 cm
	17.5 cm

Width of transformed flange = bt	
Concrete Class of deck K - 350	
Concrete Class	
Precast Prestressed K - 400	400 kg/cm ²
Concrete specific	39.22 Mpa
Ratio of moduli (n)	350
	400
	0.875
bt	n
	0.88
bt	153.13 cm
say	150.0 cm

Section properties of Composite section			
moment static to bottom fiber			
part	depth	width	area
1	cm	cm	cm ²
(1)	(2)	(3)	(4)
slab	17.5	150.0	2,625.0
panel	7.0	150.0	1,050.0
beam			2,572.5
total	24.5		6,247.5

li ²	lx ²
cm ⁴	cm ⁴
(9)	(10)
1/12.b.h ³	(8) + (9)
66,992	29,422,531
4,288	9,183,650
2,266,607	5,660,867
44,267,048	

Project : Medan Flood Control Channel : Percut River Structure : Br - P3 Station : PE 115 + 06	Japan International Cooperation Agency (JICA)	Calculation sheet
Date		Nos of sheet
Prepared by		

1.4.2. Load on a beam

Span	=	16.00 m	thickness of asphalt	=	0.050 m					
Clear span	=	16.00 m	thickness of slab	=	0.245 m					
nos of beam	=	3.00	unit weight of asphalt	=	2.300 t/m3					
1. Loading			unit weight of concrete	=	2.500 t/m3					
Width effective of beam	be	=								
Dead load										
a. Dead load										
description	width	x	thick	x	area	x	length	x	unit weight	weight
	m		m		m2		m		t/m3	t/m2
a.1 Beam	gb				0.2573 x	1.00 x	2.500	=	0.643	
a.2 Slab	gs	1.75 x	0.245 x	0.43 x	1.00 x	2.500	=	1.072		
a.3 Asphalt	ga	1.75 x	0.050 x	0.09 x	1.00 x	2.300	=	0.201		
b. Live load										
b.1 Line load = p										
Impact coefficient										
k	=	1	+	20						
					(50 + L)					
					20.00	+	1.303			
P	=	12.00 ton/lane								
		be								
		2.75	x	p	x	k				
		1.75	x	12.00 x	1.303					
		2.75								
		9.950 t/m'								
b.2 Uniform Load = q										
Span		16.00	<	30.00 m						
Q	=	2.20 ton / lane								
		be								
q	=	2.75	x	Q						
		1.75	x	2.20						
		2.75								
		1.40 t / m'								
2. Influence line of moment and moment										
2.1 Beam										
Length of span	g	=	0.643 t/m'							
Interval distance		=	16.00 m							
		=	2.50 m							
Point	Distance	Load	Reaction	I.L. Moment	Area of I.L.	Moment				
	m	t/m'	ton	t.m	m2	t.m				
1	2.50	0.643	0.844	2.11	16.88	10.855				
2	5.00	0.643	0.888	3.44	27.50	17.689				
3	7.50	0.643	0.531	3.98	31.88	20.504				
4	8.00	0.643	0.500	4.00	32.00	20.584				
2.2 Slab										
		gs	=	1.072 t/m'						
Point	Distance	Load	Reaction	I.L. Moment	Area of I.L.	Moment				
	m	t/m'	ton	t.m	m2	t.m				
1	2.50	1.072	0.844	2.11	16.88	18.089				
2	5.00	1.072	0.688	3.44	27.50	29.477				
3	7.50	1.072	0.531	3.98	31.88	34.166				
4	8.00	1.072	0.500	4.00	32.00	34.300				

bm-10d16

Project : Median Flood Control Channel : Percut River Structure : Br. P3 Station : PE 115 + 06	Japan International Cooperation Agency (JICA)	Calculation sheet Date Nos of sheet Prepared by
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1.4.3. Prestress Force and numbers of tendon	
Span	16.00 m
M minimum	20,584 kg.cm
M transfer	2,058,400 kg.cm
M dead load	20,584 t.m
M composite	2,058,400 kg.cm
	54,884 t.m
	5,488,400 kg.cm
	91,042 t.m
	9,104,200 kg.cm

Characteristic of Concrete

specified strength (fc)

Concrete class, K 400

ft'	0.00 kg/cm2
fc'	190.00 kg/cm2
fs'	180.00 kg/cm2
fs'	0.00 kg/cm2

Compression strength

Section properties

	Beam	Composite	
h	90 cm	thick of slab 17.50 cm	
A	2,573 cm2	thick of panel 7 cm	
Yt	53.68 cm	h'	114.50 cm
Yb	36.32 cm	Ac'	6,248.00 cm2
I	2,286,607 cm4	Yt'	39.40 cm
Wt	42,228 cm3	Yb'	75.10 cm
Wb	62,400 cm3	I'	9,021,046 cm4
kt	24.28 cm	Wt'	228,984 cm3
kb	16.42 cm	Wb'	120,114 cm3
e	25.07 cm	kt'	19.20 cm3
		kb'	36.70 cm3
		n	0.75

Computation prestressing force

1. Initial case

1st equation top fiber

Pt	>	fc.Wt - Mg
		(kb - e)

Mg	2,058,400 kg.cm	
fc	0.00 kg/cm2	
Wt	42,228.00 cm3	
fc.Wt	0.00 cm3	
kb	16.42 cm	
e	25.07 cm	
fc.Wt - M min	0.00 -	
kb - e	16.42 -	
Pt	>	(2,058,400)
		(8.65)
		237.965

by self weight of beam

2,058,400	=	(2,058,400)	kg.cm
25.07	=	(8.65)	cm

2nd equation bottom fiber

Pt	<	fc.Wb + Mg
		(e + kt)

Mg	2,058,400 kg.cm
fc	190.00 kg/cm
Wb	62,400.00 kg/cm2
fc.Wb	11,856,000.00 cm3
kt	24.28 cm3
e	25.07 cm
M min + fc.Wb	2,058,400 +
kt + e	24.28 +

by self weight of beam

11,856,000	=	13,914,400	kg.cm
25.07	=	49.33	cm

es-16

pres-16

2.3 asphalt $g_{as} = 0.201 \text{ t/m'}$						
Point	Distance m	Load t/m'	Reaction ton	I.L Moment t.m	Area of I.L m ²	Moment t.m
1	2.50	0.201	0.844	2.11	16.88	3.396
2	5.00	0.201	0.688	3.44	27.50	5.534
3	7.50	0.201	0.531	3.98	31.88	6.415
4	8.00	0.201	0.500	4.00	32.00	6.440
2.4 uniform load $q = 1.400 \text{ t/m'}$						
Point	Distance m	Load t/m'	Reaction ton	I.L Moment t.m	Area of I.L m ²	Moment t.m
1	2.50	1.400	0.844	2.11	16.88	23.625
2	5.00	1.400	0.688	3.44	27.50	38.500
3	7.50	1.400	0.531	3.98	31.88	44.825
4	8.00	1.400	0.500	4.00	32.00	44.800
2.5 line load $P = 9.95 \text{ t/m'}$						
Point	Distance m	Load t/m'	Reaction ton	I.L Moment t.m	Moment t.m	
1	2.50	9.950	0.844	2.11	20.989	
2	5.00	9.950	0.688	3.44	34.205	
3	7.50	9.950	0.531	3.98	39.846	
4	8.00	9.950	0.500	4.00	39.802	

(1). Initial case by self weight of beam = Mg

Distance	Mg
2.50	10.855
5.00	17.689
7.50	20.504
8.00	20.584

(2). Transfer case (Mg + M slab) = Mdl

Distance	Mg	M slab	Mdl = Mg + Ms
2.50	10.85	18.09	28.943
5.00	17.69	29.48	47.166
7.50	20.50	34.17	54.670
8.00	20.58	34.30	54.884

(3). Final case (composite section)

Moment support by composite section					
Distance	Mg uniform load	MP line load	MIL live load	Ma by asphalt	Mc = Ma + MIL
2.50	23.63	20.99	44.61	3.40	48.010
5.00	38.50	34.20	72.70	5.53	78.239
7.50	44.63	39.65	84.27	6.41	90.686
8.00	44.90	39.80	84.60	6.44	91.042

Mg = moment support by precast concrete prestressed
 Mdl = moment support by precast concrete prestressed (beam + concrete in situ + cross beam + side walks)
 Ml = moment support by composite section (DI + LL)
 Mc = moment support by composite section (DI + LL + Asphalt)

3. Influence line of shear and shear

Load	Distance m	Reaction ton	Area of I.L m ²	Shear ton
3.1 self weight	0.643	0.00	1.000	8.00
3.2 slab	1.072	0.00	1.000	8.00
3.3 asphalt	0.201	0.00	1.000	8.00
3.4 uniform load	1.400	0.00	1.000	8.00
3.5 line load	9.950	0.00	1.000	8.00
				36.481

bm-bd16

bm-jod16

PI	<	13,914.400	=	282,068
	<	49.33		
	<	282.07	ton	

1st equation top fiber		Top fiber	after prestress losses
PI	>	$f_{ti}W_t - M_g$	
		$n_s(lb - e)$	
Mg	=	2,058,400 kg.cm	
f _{ti}	=	0.00 kg/cm ²	
W _t	=	42,228.00 cm ³	
f _{ti} W _t	=	0.00 cm ³	
lb	=	16.42 cm	
e	=	25.07 cm	
(lb - e)	=	16.42 -	(8.65) cm
f _{ti} W _t - M _{min}	=	0.00 -	(2,058,400) cm
n _s (lb - e)	=	0.75 x	(8.65) -
		(2,058,400)	
PI	>	317,287	kg
	>	-8.4875	
	>	317.29	ton

2nd equation bottom fiber		after prestress losses	
PI	<	$f_{ci}W_b + M_g$	
		$n_s(e + kt)$	
Mg	=	2,058,400 kg.cm	
f _{ci}	=	190.00 kg/cm ²	
W _b	=	62,400.00 kg/cm ²	
f _{ci} W _b	=	190 x	
kt	=	24.26 cm ³	
e	=	25.07 cm	
(kt + e)	=	24.26 +	
f _{ci} W _b + M _{trans.}	=	11,856,000 +	
n _s (kt + e)	=	0.75 x	
		13,914.400	
PI	<	376,090	kg
	<	37.00	
	<	376.09	ton

3. Dead load		by beam and concrete Insltu	
1st equation top fiber			
PI	>	$f_{ci}W_t - M_{dl}$	
		$n_s(lb - e)$	
M _{dl}	=	5,488,400 kg.cm	
f _{ti}	=	0.00 kg/cm ²	
W _t	=	42,228.00 cm ³	
f _{ti} W _t	=	0 x	
lb	=	16.42 cm	
e	=	25.07 cm	
(lb - e)	=	16.42 -	
f _{ti} W _t - M _{dl}	=	0 -	
n _s (lb - e)	=	0.75 x	
		(5,488,400)	
PI	>	845,998	kg
	>	(8.65)	
	>	846.00	ton

2nd equation bottom fiber		by beam and concrete Insltu
PI	<	$f_{ci}W_b + M_{dl}$
		$n_s(e + kt)$

M _{dl}	=	5,488,400 kg.cm	
f _i l	=	0.00 kg/cm ²	
W _b	=	62,400.00 kg/cm ²	
f _i l.W _b	=	0 x	
kt	=	24.26 cm	
e	=	25.07 cm	
(e + kt)	=	25.07 +	
f _i l.W _b + M _{dl}	=	0 +	
n _s (e + kt)	=	0.75 x	
		5,488,400	
PI	<	148,345 kg	
	<	37.00	
	<	148.35 ton	

4. Service load		by service load
3rd equation top fiber		
PI	<	$f_{ci}W_t - M_{dl} - M_cW_t/W_b$
		$n_s(lb - e)$
M _c	=	9,104,200 kg.cm
M _{dl}	=	5,488,400 kg.cm
f _i l	=	180.00 kg/cm ²
W _t	=	42,228.00 cm ³
W _t	=	228,984.00 cm ³
W _b	=	228,984.00 cm ³
lb	=	16.42 cm
e	=	25.07 cm
(lb - e)	=	16.42 -
W _t /W _b	=	42,228.00 :
M _c W _t /W _b	=	9,104,200.00 x
f _i l.W _t	=	180.00 x
f _i l.W _t - M _{dl}	=	7,601,040.00 -
		2,112,640.00
n _s (lb - e)	=	0.75 x
		433,692.31
PI	<	(66,850.45)
	<	(8.49)
	<	(66.85)

4th equation bottom fiber		by service load
PI	>	$f_{ci}W_b + M_{dl} + M_cW_b/W_b$
		$n_s(e + kt)$
M _c	=	9,104,200.00 kg.cm
M _{dl}	=	5,488,400.00 kg.cm
f _i l	=	0.00 kg/cm ²
W _t	=	228,984.00 cm ³
W _b	=	62,400.00 cm ³
W _b	=	120,114.00 cm ³
kt	=	24.26 cm
e	=	25.07 cm
(e + kt)	=	25.07 +
W _b /W _t	=	62,400 :
M _c W _b /W _t	=	9104200 x
f _i l.W _b	=	0 x
f _i l.W _b + M _{dl}	=	0 +
		5,488,400
n _s (e + kt)	=	0.75 x
		4729690.7937
PI	>	276,163 kg
	>	10,218.091
	>	37.00
	>	276.16 ton

Project : Medan Flood Control Channel : Perout River	Japan International Cooperation Agency (JICA)	Date	Calculation sheet
Structure : Br. P3		Nos of sheet	
Station : PE 115 + 06		Prepared by	

1.4.4 Prestress losses

1. Due to elastic shortening
span = 16.00 m

section properties

Ht =	90.00 cm	Yt =	53.68 cm
Ac =	2,573.00 cm ²	Yb =	36.32 cm
Ix =	2,266,607 cm ⁴		
Wt =	42,228 cm ³		
Wb =	62,399 cm ³		

Concret class K-400

(ic)	=	400 kg/cm ²
Nos strand / tendon	=	6.00 nos
nos of tendon	=	3.00 nos
Area of strand	=	94.2 mm ²
Total area	=	0.942 cm ²

Ec =	343,327 kg/cm ²
Es =	1,897,200 kg/cm ²
Es	

modular ratio n = -----

Ec

1,897,200

343,327

5.53

Allowance stress in prestressing tendon = fpu

fpu = 1,860 Mpa

1,860 x

18,972 kg/cm²

10.2

Stress at jacking

= 0.8 x 18,972

= 15,178 kg/cm²

Jacking force

= 15,178.60 x 0.942

= 14,297 kg / strand

Jacking force

= 6.00 x 14,297

P jacking = 85,784 kg / tendon

total force at jacking per tendon = P j t

P j t = P jacking x nos of tendon

= 85,784 x 3.00

= 257,351 kg

at the jacking

tendon no	Yb cm	Ys cm	e cm	Pj kg	Ac cm ²	Pj/Ac (eAc/Wb) (1+eAc/Wb) kg/cm ²
3	36.32	19.00	23.32	85,784	2573	33.34
2	36.32	6.50	29.82	85,784	2573	33.34
1	36.32	6.50	29.82	85,784	2573	33.34
tendon no	Yb cm	Ys cm	e cm	Pj kg	Ac cm ²	Pj/Ac (eAc/Wb) (1+eAc/Wb) kg/cm ²
3	65.40	5.53	361.39	15,178	2.36	
2	74.34	5.53	410.77	15,178	2.71	
1	74.34	5.53	410.77	15,178	2.71	
average	7.794	3	2.5980	7.794		

Initial case	Top fibre	P1 >	237.97 ton
	Bottom fibre	P1 <	282.07 ton
Transfer	Top fibre	P1 >	317.29 ton
	Bottom fibre	P1 <	376.09 ton
Dead load	Top fibre	P1 <	646.00 ton
	Bottom fibre	P1 >	148.35 ton
Service case	Top fibre	P1 <	(66.85) ton
	Bottom fibre	P1 >	276.18 ton

237.97 < P1 < 282.07

P1	=	282,068 kg
1 Mpa	=	10.2 kg/cm ²
1 kg/cm ²	=	0.098 Mpa
1 kN	=	102 Kg
1 kg	=	0.0098 kN

used strand grade 270

nominal diameter

nominal area

fu = 1,860 Mpa

1860 x

18,972 kg/cm²

10.2

at jacking

f = 0.80 x fu

0.8 x

18,972

1,488 Mpa

15,178 kg/cm²

P jacking

P j =

15,177.6 x

14,297 kg / strand

6.00

P j / strand x nos of strand

14,297 x

85,784 kg / tendon

total force : P j

282,068 : 85,784

3.29

3.00 nos

say

2. Due to friction
- $f \times e^{-(ua + Ks)}$
 - = stress in the strand at the jacking end or live anchorage
 - f = stress at dead anchorage
 - u = coefficient of angular friction
 - α = change in the angle between the force at anchorage to section X
 - x = curvilinear length of the tendon anchorage to section x
 - f = depth of parabola

tendon parabola curve

$$Y = 4x(l)^{-2} \times (l-x)$$

angle of tendon parabola to horizontal line

$$dx/dy = 0$$

x	=	0.00 m
e	=	0 m
l	=	16.00 m
α	=	$4 \times l / l$ radial
α	=	4.00×0
α	=	16.00 radial
α	=	0.0000 radial

α	=	2.00 x α
α	=	2.00 x 0.0000
α	=	0.0000

tendon no.	Distance	Ys	no tendon	value of sag parabola each tendon	H	Ys	f
3	top fiber to no.3	0.20					
2	no.3 to no.2	0.25			0.700	0.130	0.570 m
1	no.2 to no.1	0.00			0.450	0.065	0.395 m
1	no.1 to outside	0.25			0.200	0.065	0.135 m

no tendon	value of depth parabola each tendon	f	L	α	radial
3	α	4.00 x	0.570	16.00	0.1425 radial
2	α	4.00 x	0.385	16.00	0.0962 radial
1	α	4.00 x	0.135	16.00	0.0337 radial

if $x = 0$, $\alpha = \alpha$

if $x = l/2$, $\alpha = 2 \times \alpha$

description	value of Px each tendon					
	tendon - 3	tendon - 2	tendon - 1	tendon - 2	tendon - 1	tendon - 1
α	0.000	0.000	0.000	0.000	0.000	0.000
u	0.1425	0.2850	0.0962	0.1925	0.0337	0.0675
$u \alpha$	0.2000	0.2000	0.2000	0.2000	0.2000	0.2000
K	0.0285	0.0570	0.0193	0.0385	0.0067	0.0135
Kx	0.0041	0.0041	0.0041	0.0041	0.0041	0.0041
$u \alpha + Kx$	0.0000	0.0328	0.0328	0.0328	0.0000	0.0328
$e^{-u \alpha - (u \alpha + Kx)}$	0.0285	0.0698	0.0193	0.0713	0.0087	0.0463
f	2.7183	2.7183	2.7183	2.7183	2.7183	2.7183
f	0.9719	0.9141	0.9809	0.9312	0.9933	0.9548
f	0.9719	0.9141	0.9809	0.9312	0.9933	0.9548

$X = 0$

$X = l/2$

tendon no.	f	X
3	0.972	0.914
2	0.981	0.931
1	0.993	0.955
sum	2.946	2.800

average

average of f

f jacking

stress lost

percentage stress lost

0

stress losses

E_s

assumed slip

stress

stress losses

total stress lost

Initial forces / tendon

P / tendon

Initial force after transfer

P Initial

P_j

2.946 + 2.000

3

0.982 + 0.933

0.982 + 0.933

2.00

0.9577 fs jacking

f 15.178

0.9577 0.9577

15.848 kg/cm2

15.848 - 15.178

670.48 kg/cm2

670.48 x 100

15177.6

4.418 %

3. Due to anchorage set

delta / $l \times E_s$

1,897.200 kg/cm2

0.5 cm

delta x E_s

0.50 x 1,897.200

1,600

593 kg/cm2

593 x 100

15,178

3.906 %

2.598 + 4.418 + 3.906

10.92 %

1.00 - 0.1092 x P jacking

0.891 x 85,784

76,415 kg

Initial forces x nos tendon

76,415 x 3.00

229,244 kg

257,351 kg

1.4.5 Centre Gravity of tendon

for span = 16.00 m

Y = centre gravity of tendon (cgs)
to the centre gravity of concrete (cgc)

dia. of strand = 12.70 mm
depth = 90.00 cm
area = 94.20 mm²
= 0.9420 cm²

Nos of strand = 6.00
area per tendon = 5.6520 cm²
Yb = 36.22 cm
Yt = 53.78 cm

top cover concrete = tc
to = 20.00 cm

a. at stressing of tendon

a.1 cgs tendon at stressing tendon no.3 at edge of span

tendon no	distance to each cm	tendon no	distance to cgc cm	area of tendon As cm ²	static moment Y cm	As.Y cm ³
No.3 to no.2	25.00	No.3	33.78	5.6520	33.78	190.92
No.3 to cgc	8.78	No.2	25.00	5.6520	25.00	141.30
	33.78			11.30		332.22

e at edge = 332.22
= 29.39 cm
11.304

a.2 cgs tendon at stressing tendon no.3 at middle of span

tendon no	distance to each cm	tendon no	distance to cgc cm	area of tendon As cm ²	static moment Y cm	As.Y cm ³
No.3 to no.2	13.00	No.3	13.00	5.652	13.00	73.476
No.2 to outside	6.50	No.2	6.50	5.652	6.50	36.738
No.1 to outside	6.50	No.1	6.50	5.652	6.50	36.738
	13.00			5.652		73.476

Ys = 73.48
= 5.652
13.00 cm

e jack at middle = Yb - Ys
= 36.22 - 13.00
= 23.22 cm

b. at transfer b.1 cgs tendon at edge of span

tendon no	distance to each cm	tendon no	distance to outside cm	area of tendon As cm ²	static moment Y cm	As.Y cm ³
No.3 to No.2	25.00	No.3	70.00	5.6520	70.00	395.64
No.2 to No.1	25.00	No.2	45.00	5.6520	45.00	254.34
No.1 to outside	20.00	No.1	20.00	5.6520	20.00	113.04
	70.00			16.96		763.02

Ys = 763.02
= 45.00 cm
16.956

e at edge = Yb - Ys
= 36.22 - 45.00
= (8.78) cm

b.2 cgs tendon at middle of span

Strand no	distance to each cm	Strand no	distance to cgc cm	area of strand As cm ²	static moment Y cm	As.Y cm ³
No.3 to No.2	6.50	No.3	13.00	5.6520	13.00	73.48
No.2 to No.1	0.00	No.2	6.50	5.6520	6.50	36.74
No.1 to outside	6.50	No.1	6.50	5.6520	6.50	36.74
	13.00			16.96		146.95

Ys = 146.95
= 8.67 cm
16.956

e at middle = Yb - Ys
= 36.22 - 8.67
= 27.55 cm

Project : Median Flood Control

Channel : Percut River

Structure : Br. P3

Station : PE 115 + 06

Japan International Cooperation Agency (JICA)

Calculation sheet

Date

Nos of sheet

Prepared by

1.4.6 Layout of tendon

span = 16.00 m

description	Combined moment (t.m) at various length (m)			
	2.50	5.00	7.50	8.00
2.1. beam	10.86	17.69	20.50	20.58
2.2. slab	18.09	29.48	34.17	34.30
2.3. asphalt	3.40	5.53	6.41	6.44
2.4. uniform load	23.63	39.65	44.63	44.80
2.5. line load	20.99	34.21	39.65	39.80

(1) M minimum = 2.1	10.86	17.69	20.50	20.58
(2) M dead load = 2.1+2.2	28.94	47.17	54.67	54.88
(3) M live load = 2.4+2.5	44.61	73.86	84.27	84.60
(4) M composite = (3)+2.3	48.01	79.39	90.68	91.04

Pi	=	257,351	kg
Pi	=	229,244	kg
n	=	0.75	
P	=	0.75	x
P	=	193,013	kg

case	Pia	P=n.Pi
Initial case	257,351	
After transfer	193,013	
Dead load	193,013	
Live load		193,013
Composite		193,013

Arm	=	Moment
	=	Force

Distance	m	2.50	5.00	7.50	8.00
Mg	t.m	10.86	17.69	20.50	20.58
Prestressed force	kg.cm	1,085,500	1,768,900	2,050,400	2,058,400
arm	cm	257,351	257,351	257,351	257,351
	cm	4.22	6.87	7.97	8.00

Distance	m	2.50	5.00	7.50	8.00
M transfer	t.m	10.86	17.69	20.50	20.58
Prestressed force	kg.cm	1,085,500	1,768,900	2,050,400	2,058,400
Arm	cm	193,013	193,013	193,013	193,013
	cm	5.62	9.16	10.62	10.66

Distance	m	2.50	5.00	7.50	8.00
M dead load	t.m	28.94	47.17	54.67	54.88
Prestressed force	kg.cm	2,894,300	4,716,600	5,467,000	5,488,400
Arm	cm	193,013	193,013	193,013	193,013
	cm	15.00	24.44	28.32	28.44

Distance	m	2.50	5.00	7.50	8.00
M composite	t.m	48.01	79.39	90.68	91.04
Prestressed force	kg.cm	4,801,000	7,938,900	9,068,100	9,104,200
Arm	cm	193,013	193,013	193,013	193,013
	cm	24.87	41.13	46.98	47.17

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Project : Median Flood Control Channel : Percut River Structure : Br. P3 Station : PE 115 + 06	Japan International Cooperation Agency (JICA)	Calculation sheet Date Nos of sheet Prepared by
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1.4.7 Stress Control	
Working moment	
M minimum	= 20,584 t.m
M after transfer	= 2,058,400 kg.cm
M dead load	= 2,058,400 kg.cm
M composite	= 5,488,400 kg.cm
e	= 91,042 t.m
e	= 27.32 cm

Allowable stresses in prestressed concrete	
Initial case	
fci	= 190.0 kg/cm ²
fti	= 0.0 kg/cm ²
Final case	
fcs	= 180.0 kg/cm ²
fts	= 0.0 kg/cm ²

Girder	Section properties	Composite thick of slab thick of panel
h	= 90 cm	h' = 17.50 cm
Ac	= 2,266 cm ²	Ac' = 7 cm
Yt	= 53.68 cm	Yt' = 114.50 cm
Yb	= 36.32 cm	Yb' = 6,248.00 cm ²
I	= 2,266,607 cm ⁴	I' = 39.40 cm
Wt	= 42,227 cm ³	Wt' = 75.10 cm
Wb	= 62,399 cm ³	Wb' = 9,021,046 cm ⁴
kt	= 24.26 cm	kt' = 228,984 cm ³
kb	= 16.42 cm	kb' = 120,114 cm ³
e	= 27.32 cm	e = 19.20 cm ³
		e = 36.70 cm ³
		n = 0.75

Computation of stresses	
Force at jacking	Pj
Pj	= 257,351 kg
Pi	= 229,244 kg
P	= n x x
P	= 0.75 x
P	= 193,013 kg

Moment by eccentricity tendon	
Initial case	
Piack x e	= 257,351 x 23.22
Pi	= 5,975,690 kg.cm
P initial x e	= 229,244 x 27.55
P	= 6,315,672 kg.cm
P final x e	= 193,013 x 27.55
P	= 5,317,515 kg.cm

stres-16

Actually working stresses									
1. Initial case									
f _i	top fibre		bottom fibre		top fibre		bottom fibre		kg/cm ²
	P _i	P _i x e	P _i x e	W _i	M _i	W _i	P _i x e	W _i	
=	---	---	---	---	---	---	---	---	0.00
=	257,351	5,975,690	2,058,400	42,227	2,058,400	42,227	5,975,690	2,058,400	0.00
=	2,266	141.51	48.75	32.99	48.75	32.99	141.51	48.75	0.00
=	113.57	101.17	87.96	180	87.96	180	101.17	87.96	180
=	20.80	0	0	0	0	0	0	0	0
f _{bd}	top fibre		bottom fibre		top fibre		bottom fibre		kg/cm ²
	P _i	P _i x e	P _i x e	W _i	M _i	W _i	P _i x e	W _i	
=	---	---	---	---	---	---	---	---	180
=	257,351	5,975,690	2,058,400	42,227	2,058,400	42,227	5,975,690	2,058,400	180
=	2,266	141.51	48.75	32.99	48.75	32.99	141.51	48.75	180
=	113.57	101.17	87.96	180	87.96	180	101.17	87.96	180
=	176.35	0	0	0	0	0	0	0	0
f _t	top fibre		bottom fibre		top fibre		bottom fibre		kg/cm ²
	P _i	P _i x e	P _i x e	W _i	M _i	W _i	P _i x e	W _i	
=	---	---	---	---	---	---	---	---	0
=	257,351	5,975,690	2,058,400	42,227	2,058,400	42,227	5,975,690	2,058,400	0
=	2,266	141.51	48.75	32.99	48.75	32.99	141.51	48.75	0
=	113.57	101.17	87.96	180	87.96	180	101.17	87.96	0
=	0.35	0	0	0	0	0	0	0	0
f _b	top fibre		bottom fibre		top fibre		bottom fibre		kg/cm ²
	P _i	P _i x e	P _i x e	W _i	M _i	W _i	P _i x e	W _i	
=	---	---	---	---	---	---	---	---	180
=	257,351	5,975,690	2,058,400	42,227	2,058,400	42,227	5,975,690	2,058,400	180
=	2,266	141.51	48.75	32.99	48.75	32.99	141.51	48.75	180
=	113.57	101.17	87.96	180	87.96	180	101.17	87.96	180
=	0.35	0	0	0	0	0	0	0	0
f _r	top fibre		bottom fibre		top fibre		bottom fibre		kg/cm ²
	P _i	P _i x e	P _i x e	W _i	M _i	W _i	P _i x e	W _i	
=	---	---	---	---	---	---	---	---	0
=	257,351	5,975,690	2,058,400	42,227	2,058,400	42,227	5,975,690	2,058,400	0
=	2,266	141.51	48.75	32.99	48.75	32.99	141.51	48.75	0
=	113.57	101.17	87.96	180	87.96	180	101.17	87.96	0
=	0.35	0	0	0	0	0	0	0	0

2. after transfer									
3. Dead load (self weight beam + concrete cast in place)									
f _i	top fibre		bottom fibre		top fibre		bottom fibre		kg/cm ²
	P _i	P _i x e	P _i x e	W _i	M _i	W _i	P _i x e	W _i	
=	---	---	---	---	---	---	---	---	0
=	257,351	5,975,690	2,058,400	42,227	2,058,400	42,227	5,975,690	2,058,400	0
=	2,266	141.51	48.75	32.99	48.75	32.99	141.51	48.75	0
=	113.57	101.17	87.96	180	87.96	180	101.17	87.96	0
=	0.35	0	0	0	0	0	0	0	0
f _{bd}	top fibre		bottom fibre		top fibre		bottom fibre		kg/cm ²
	P _i	P _i x e	P _i x e	W _i	M _i	W _i	P _i x e	W _i	
=	---	---	---	---	---	---	---	---	180
=	257,351	5,975,690	2,058,400	42,227	2,058,400	42,227	5,975,690	2,058,400	180
=	2,266	141.51	48.75	32.99	48.75	32.99	141.51	48.75	180
=	113.57	101.17	87.96	180	87.96	180	101.17	87.96	180
=	0.35	0	0	0	0	0	0	0	0
f _t	top fibre		bottom fibre		top fibre		bottom fibre		kg/cm ²
	P _i	P _i x e	P _i x e	W _i	M _i	W _i	P _i x e	W _i	
=	---	---	---	---	---	---	---	---	0
=	257,351	5,975,690	2,058,400	42,227	2,058,400	42,227	5,975,690	2,058,400	0
=	2,266	141.51	48.75	32.99	48.75	32.99	141.51	48.75	0
=	113.57	101.17	87.96	180	87.96	180	101.17	87.96	0
=	0.35	0	0	0	0	0	0	0	0
f _b	top fibre		bottom fibre		top fibre		bottom fibre		kg/cm ²
	P _i	P _i x e	P _i x e	W _i	M _i	W _i	P _i x e	W _i	
=	---	---	---	---	---	---	---	---	180
=	257,351	5,975,690	2,058,400	42,227	2,058,400	42,227	5,975,690	2,058,400	180
=	2,266	141.51	48.75	32.99	48.75	32.99	141.51	48.75	180
=	113.57	101.17	87.96	180	87.96	180	101.17	87.96	180
=	0.35	0	0	0	0	0	0	0	0

1.4.8 Deflection of beam

I. deflection computation

deflection is moment at any point by "bending moment" as load

I.a. deflection due to uniform load

$$\begin{aligned} \text{load} &= g \\ \text{Mo} &= 1/8 \cdot g \cdot l^2 \\ \text{Aa} &= 4/3 \cdot a \cdot b \\ a &= l/2 \\ b &= Mo \\ \text{Aa} &= 4/3 \cdot (l/2) \cdot Mo \\ R &= 2/3 \cdot l \cdot Mo \\ &= 1/2 \cdot Aa \\ &= 2/6 \cdot l \cdot Mo \\ xp1 &= 3 \cdot a/8 \\ &= 3 \cdot l/16 \\ &= 3/16 \cdot l \\ M \max &= ya \\ &= R \cdot l/2 - Aa/2 \cdot xp1 \\ &= 2/6 \cdot l \cdot Mo \cdot l/2 - 2/6 \cdot l \cdot Mo \cdot (3/16 \cdot l) \\ &= Mo \cdot l^2 \cdot (1/6 - 1/16) \\ &= Mo \cdot l^2 \cdot (8/48 - 3/48) \\ &= Mo \cdot (5/48 \cdot l^2) \end{aligned}$$

$$ya = \frac{5 \cdot Mo \cdot l^2}{48 \cdot EI}$$

I.b. deflection due to tendon forces

$$\begin{aligned} \text{load} &= P \\ \text{Mo} &= P \cdot e2 \\ \text{Ab} &= Mo \cdot l \\ R &= (Mo \cdot l)/2 \\ xp2 &= l/4 \\ M \max &= yb \\ &= Pe2 \cdot l/2 - Ab \cdot l/2 \cdot xp2 \\ &= (Mo \cdot l)/2 \cdot l/2 - (P \cdot e2 \cdot l)/2 \cdot l/4 \\ &= (Mo \cdot l) \cdot l/4 - l/8 \\ &= (Mo \cdot l) \cdot l \cdot (2/8 - 1/8) \\ &= (Mo \cdot l) \cdot l \cdot (2/8 - 1/8) \end{aligned}$$

$$yb = \frac{Mo \cdot l^2}{8 \cdot EI}$$

I.c. deflection due to knife load

$$\begin{aligned} \text{load} &= Pk \\ \text{Mo} &= 1/4 Pk \cdot l \\ \text{Ac} &= Mo/2 \cdot l/2 \\ &= Mo/4 \cdot l \\ &= 1/3 \cdot l/2 \\ &= l/6 \end{aligned}$$

$$\begin{aligned} M \max &= yc \\ &= Mo/4 \cdot l/2 - Ac/2 \cdot l/6 \\ &= Mo/4 \cdot l/2 - Mo/4 \cdot l/6 \\ &= Mo \cdot (l^2/8 - l^2/24) \\ &= Mo \cdot l^2 \cdot (1/8 - 1/24) \\ &= Mo/12 \cdot l^2 \end{aligned}$$

$$yc = \frac{Mo \cdot l^2}{12 \cdot EI}$$

$$\begin{aligned} \text{I.d. data} &= \\ \text{span} &= 16.00 \text{ m} \\ E-c &= 1,600 \text{ cm} \\ \text{Inertia} &= 343,327 \text{ kg/cm}^2 \\ Ex1 &= 2,268,607 \text{ cm}^4 \\ &= 7,78E+11 \text{ kg/cm}^2 \end{aligned}$$

II. loading

$$\begin{aligned} \text{II.1 dead load} &= \\ \text{weight of beam} &= 0.643 \text{ t/m} \\ \text{weight of slab} &= 1.072 \text{ t/m} \\ \text{asphalt} &= 0.201 \text{ t/m} \end{aligned}$$

$$\begin{aligned} \text{II.2 live load} &= \\ \text{uniform load} &= 1,400 \text{ t/m} \\ \text{line load} &= 9,950 \text{ ton/line} \end{aligned}$$

III. moment

III.1 by dead load

load	coefficient	x	load	x	l ²	=	moment
by beam	0.125	x	0.643	x	256.0	=	2,057,600
slab deck	0.125	x	1.072	x	256.0	=	3,430,400
asphalt	0.125	x	0.201	x	256.0	=	643,200

III.2 moment due to prestressed force

III.2.1 at stressing of tendon no.3

$$\begin{aligned} P \text{ jacking} &= 257,351 \text{ ton} \\ P \text{ initial} &= 266,151 \text{ ton} \\ P \text{ final} &= 199,613 \text{ ton} \\ e1 &= \text{edge} = 0.2939 \text{ m} \\ e2 &= \text{middle} = 0.2322 \text{ m} \\ e1 + e2 &= 0.5261 \text{ m} \end{aligned}$$

$$\begin{aligned} \text{at edge} &= M2 = P \cdot x \cdot e2 \\ &= 257,351 \cdot x \cdot 0.2322 = 5,975,690 \text{ kg.cm} \\ \text{at middle} &= M1 = -P \cdot x \cdot (e1 + e2) \\ &= -(257,351) \cdot x \cdot 0.5261 = (13,539,236) \text{ kg.cm} \end{aligned}$$

III.2.2 at transfer

$$\begin{aligned} e1 &= \text{edge} = 0.0868 \text{ m} \\ e2 &= \text{middle} = 0.2755 \text{ m} \\ e1 + e2 &= 0.3623 \text{ m} \\ \text{at edge} &= M2 = P \cdot x \cdot e2 \\ &= 266,151 \cdot x \cdot 0.2755 = 7,332,460 \text{ kg.cm} \\ \text{at middle} &= M1 = -P \cdot x \cdot (e1 + e2) \\ &= -(266,151) \cdot x \cdot 0.3623 = (9,642,651) \text{ kg.cm} \end{aligned}$$

IV. deflection computation.

IV.1 by dead load

load	5/48	x	Mo	kg.cm	x	I ²	cm ²	EI	kg.cm ²	=	Y	mm
self weight beam	(1)	0.1042	x	2,057,600	x	2,560,000	:	7,78E+11	=	7.05		
slab deck	(2)	0.1042	x	3,430,400	x	2,560,000	:	7,78E+11	=	11.76		
asphalt	(3)	0.1042	x	843,200	x	2,560,000	:	7,78E+11	=	2.20		

IV.1 by prestressing forces

IV.2.1 at stressing of tendon no.3

steel, supporting dimensions									
at edge	1/8	x	M2	x	I ²	:	EI	=	Y
Ye5 =	0.125	x	5975.690	x	2,560,000	:	7.78E+11	=	24.57
at middle	5/48	x	M1	x	I ²	:	EI	=	Y
Ym5 =	0.1042	x	(13,539,236)	x	2,560,000	:	7.78E+11	=	(46.40)
balance	=		(46.40)	+	24.57	=	(21.82)		mm
by self weight beam									mm
total	=		(21.82)	+	7.05	=	(14.77)		mm

IV.2.2 at transfer

at edge	1/8	x	M2	x	I ²	:	EI	=	Y
Yet =	0.125	x	7,332,480	x	2,560,000	:	7,78E+11	=	30.15
at midle	5/48	x	M1	x	I ²	:	EI	=	Y
Yem =	0.1042	x	(9,642,651)	x	2,560,000	:	7,78E+11	=	(33.04)

balance	=	(33.04)	+	30.15	=	(2.89)				
by self weight beam									7.05	
total deflection by tendon	=	(2.89)	+	7.05	=	4.16				

V. short term deflection

V.1 uncracked beam

S.I. ENGINEERING DATA				
	coefficient	load	l	Moment
		kg/cm	m	kg.cm
uniform load	0.125	x	1.400	x
line load	0.250	x	9.950	x
			256.0	= 4,480,000
			16.0	= 3,980,000

	load	deflection coefficient	x	Mo kg.cm	x	I^2 cm ²	:	EI kg.cm ²	=	Y mm
uniform load	(4)	5/48	x	Mo	x	I^2	:	EI	=	Y
				4,480,000	x	2,560,000	:	7.78E+11	=	15.35
line load	(5)	1/12	x	Mo	x	I^2	:	EI	=	Y
		0.0833	x	3,980,000	x	2,560,000	:	7.78E+11	=	10.91
				total					=	26.26

VI. long term deflection

VI.1 cracked beam

Cracking moment

Cracking moment is the moment for which the tensile stress on the extreme fibre of the concrete section reaches a value equal to the modulus of rupture of the concrete.

modulus of rupture of the concrete

fr	=	7.5 . @sqrt (fc') psi											
fc'	=	40 Mpa											
1 Mpa	=	145,0328 psi											
40 Mpa	=	5801,304 psi											
fr	=	571.25 psi											
1 psi	=	0.07 kg/cm ²											
fr	=	571,247,19818 x											
Mcr	=	n.P. (I ₀ - e)	+	fr . wb									

P	=	266,151	x	1,000	=	266,151	kg
n	=	0.75					
n.P	=	0.75	x	266,151	=	199,613	kg
kl	=	24.26	cm				
e	=	27.55	cm				
(kl + e)	=	51.81	cm				
wb	=	62,399.00	cm ³				
fr x wb	=	39.99	x	62,399.00			
	=	2,495,167.73	kg.cm				
Mcr	=	n.P	x	(I ₀ - e)	+	fr.wb	
	=	199,613.25	x	51.81	+	2,495,167.73	
	=	10,341,962.48	+	2,495,167.73			
	=	12,837,130.21	kg.cm				

deflection by cracking moment = y crack

y crack = 5/48 x Mo x I²:EI

5/48	x	Mo	kg.cm	x	I ²	:	EI	kg.cm ²	=	Yc	mm
coefficient											
0.1042	x	12,837,130	x	2,560,000	:	7,78E+11	=				
deflection by tendon											

total deflection = 43.99 + 4.16

= 48.15 mm

permissible deflections = span

300

16,000

900

53.33 mm ok

Project : Medan Flood Control Channe : Percut River Structur : Br . P3 Station : PE 115 + 06	Japan International Cooperation Agency (JICA)	Calculation sheet
Date		Nos of sheet
		Prepared by

1.4.9 Shear

Shear force	=	36.481 ton
Inersia	=	2,268,807 cm4
Ht	=	90.00 cm
Yt	=	53.66 cm
Yb	=	36.32 cm
e	=	27.55 cm
Area of section	=	2,573.00 cm2
Thick of web section	=	17.00 cm
Span	=	16.00 m
V	=	36.48 x
	=	36,481.00 kg

Part	Depth	Width	Area	Distance	Al.Yt
i	t	w	Al	to top = Yi	
(1)	(2)	(3)	(4)	(5)	(6)
	depth of section		=	90.00	
1	7.50	35.00	262.50	3.75	984.38
2	28.82	17.00	489.94	14.41	7,060.04
4	7.50	9.00	67.50	10.00	675.00
			Ay	819.94	8,719.41

Y	=	Sy	=	8,719.41	
		toAy	=	819.94	
			=	10.63	cm
Ay.y	=	820 x		10.63	
	=	8,719 cm2			
b.J	=	17.00 x		2,268.607	
	=	38,532.319 cm5			
e	=	27.55 cm			
4 x e	=	4.00 x		27.55	
	=	110.20 cm			
P jacking	=	257,351 kg			
n	=	0.75			
P	=	193,013 kg			
	=	4 x e		P	
Vt	=	1,1020		193,013	
	=	1,600			
	=	132.94 kg			
V net	=	V		Vt	
	=	36,481.00		132.94	
	=	36,348.06 kg			

Shear-16

shear-16

vc	=	the shear resistance on the concrete
Ay	=	area of the portion of section outside the shear plane
y	=	distance from neutral axis to centroid as Ay
by	=	width of section at level y of the shear plane considered
I	=	moment of inertia of the cross section
shear stress	=	$\frac{Ay.y}{b.J}$ x V net
vc	=	$\frac{8,719.41}{38,532.319}$ x 36,348
vc ^ 2	=	8.23 kg/cm ²
axial force	=	67.65 kg/cm ²
fx	=	fx
	=	P 193,013.25
	=	Ac 2,573
	=	75.01 kg/cm ²
fx/2	=	75.01
	=	2.00
(fx/2) ^ 2	=	1,406.81
vc ^ 2	=	8.23
(fx/2) ^ 2	=	1,406.81
vc ^ 2 + (fx/2) ^ 2	=	1,415.03
R	=	@sqrt (vc ^ 2 + (fx/2) ^ 2)
	=	37.62
Principle compressive force = f		
f1 max	=	+/- fx/2 + R
	=	37.51 + 37.62
	=	75.12 kg/cm ² < 190.00 kg/cm ²
f2 min	=	(37.51) + 37.62
	=	0.11 kg/cm ² < 0.00 kg/cm ²
1 kg/cm ²	=	14.21 psi
fc	=	400.00 kg/cm ²
	=	5,684.24 psi
vc min allowance = 2 @sqrt(fc)	=	150.79 psi
	=	10.81 kg/cm ² < 190.00 kg/cm ²
vc max allowance = 5 @sqrt(fc)	=	376.37 psi
	=	26.53 kg/cm ² < 190.00 kg/cm ²

shear-16

4.1.7 SUPERSTRUCTURE (PC BEAM 13.0M LONG)

**PRESTRESSED CONCRETE BEAM COMPUTATION
CHANNEL, MEDAN FLOOD WAY
STRUCTURE, BR – MF 8
STATION, DU. 19 + 00
WITH
CLEAR SPAN = 13.00 M**

I.4 Beam

- I.4.1 Section Properties of Beam**
- I.4.2 Loading on a Beam**
- I.4.3 Prestress force and Number Of Tendon Available**
- I.4.4 Prestress Loss**
- I.4.5 Centre Gravity of Tendon**
- I.4.6 Layout of Tendons**
- I.4.7 Stresses Control**
- I.4.8 Deflection of a Beam**
- I.4.9 Shear Force and Tensile Stress**

Project : Medan Flood Control	Japan International Cooperation Agency (JICA)	Calculation sheet
Channel : Percut River		Date
Structure : Br - MF 8		Nos of sheet
Station : DJ.19 + 00		Prepared by

I.4 Main Beam	
I.4.1 Section Properties of beam	
Span	= 13.00 m
Width of road way	= 2.00 m
Nos of beam	= 1.00 m
Space of a beam	= 2.00 m
Thick of slab	= 0.20 m

Section Properties					
moment static to top fiber					
part	depth	width	area	distance to top fiber (Y)	ALY ²
i	t	w	Ai	cm	cm ⁴
(1)	(2)	(3)	(4)	(5)	(6)
1	depth of beam	90			
1	7.5	35.0	262.5	3.8	984.4
3	12.5	65.0	812.5	83.8	68,046.9
2	70.0	17.0	1,190.0	42.5	50,575.0
4	7.5	9.0	67.5	10.0	675.0
5	10.0	24.0	240.0	74.2	17,800.0
	total		2,572.5		138,081.3

II	bt
cm ⁴	cm ⁴
(9)	(10)
1/12.b.h ³	(8) + (9)
1,230	4,922
10,579	5,709,505
485,917	2,635,334
211	6,961
1,333	1,321,500
	9,678,242

Yt	=	Sx	=	138,081.25	=	53.68 cm
Yb	=	Ac	=	2,572.50	=	
	=	Iit	=	Yt	=	
Ix	=	90.00	=	53.68	=	36.32 cm
	=	tot.Ixi	=	tot.Ai x Yt ²	=	
	=	9,678,242	=	2,573 x 2,881	=	
	=	9,678,242	=	7,411,635	=	
	=	2,266,607 cm ⁴	=		=	

Section modulus	
top fiber	Ix
Wt	= 2,266,607
	= 42,228 cm ³
bottom fiber	Yb
Wb	= 2,266,607
	= 62,400 cm ³
	Yb
	= 36.32

top fiber	Wb
kt	= 62,400
	= 24.26 cm
	Ac
	= 2,572.50

bottom fiber	Wt	= 42,228	= 16.42 cm
kt	Ac	= 2,572.50	

Composite section
width of flange = (be)
ACI

Effective flange width (be) lesser than	
1	S
2	1/4
	bw
	ht
3	bw + 16 ht

be	= 200.0 cm
1 Mpa	= 10.2 kg/cm ²
1 kg/cm ²	= 0.0980 Mpa
Width of transformed flange	= bt
Concrete Class of deck K - 350	= 350 kg/cm ²
	= 34.31 Mpa
Concrete Class.	
Precast Prestressed K - 400	= 400 kg/cm ²
Concrete specific	= 39.22 Mpa
	= 350
Ratio of moduli (n)	= 400
	= 0.875
bt	= n
	= 0.88
bt	= 175.00 cm
say	= 175.0 cm

Section properties of Composite section
moment static to bottom fiber

part	depth	width	area	distance to	ALY ²	ALY ² ~ 2
	t	w	Ai	Yb	(4)	(5)
	cm	cm	cm ²	cm	cm ³	cm ⁴
(1)	(2)	(3)	(4)	(5)	(6)	(8)
slab	20.0	175.0	3,500.0	100.0	350,000.0	35,000,000.0
panel	0.0	175.0	0.0	90.0	0.0	0.0
beam			2,572.5	36.3	93,443.8	3,394,260.2
total	20.0		6,072.5		443,443.8	

Ii	Ixi
cm ⁴	cm ⁴
(9)	(10)
1/12.b.b ³	(8) + (9)
116,667	35,116,667
0	0
2,266,607	5,660,867
	40,777,534

Yb	=	Sx	=	443,444	=	73.02 cm
	=	Ac	=	6,072.50	=	

1.4.2. Load on a beam

Span	=	13.60 m	thickness of asphalt	=	0.050
Clear span	=	13.00 m	thickness of slab	=	0.245
Width of slab	=	2.00 m	unit weight of asphalt	=	2.300
			unit weight of concrete	=	2.500

description	width m	x m	thick m	area m ²	x m	length m	x m	unit weight t/m ³	weight t/m
a.1 Beam	gb								
a.2 Slab	gs	2.00	x	0.200	x	1.00	x	2.500	= 1.000
a.3 Hand rail	up	0.16	x	0.100	x	0.02	x	2.500	= 0.022
	down	0.16	x	0.300	x	0.05	x	2.500	= 0.084
				w	= (	0.022 + 0.084	);	2.00	= 0.05
a.4 Slab rail		0.10	x	0.700	x	0.07	x	2.500	= 0.175
sum of dead load (hand slab and slab rail)									
									= 0.228

b.2 Pedestrian load

$$p = 0.50 \text{ ton / m}^2$$

$$q = 0.50 \times 2.00 = 1.00 \text{ t/m'}$$

2. Influence line of moment and moment

2.1 Beam

$$\text{Length of span} = 13.00 \text{ m}$$

$$\text{Interval distance each} = 2.50 \text{ m}$$

Point	Distance m	Load t/m'	Reaction ton	I.L. Moment t.m	Area of I.L. m ²	Moment t.m
1	2.50	0.643	0.808	2.02	13.13	6.441
2	5.00	0.643	0.615	3.08	20.00	12.662
3	6.50	0.643	0.500	3.25	21.13	13.586

2.2 Slab

$$gs = 1.000 \text{ t/m'}$$

Point	Distance m	Load t/m'	Reaction ton	I.L. Moment t.m	Area of I.L. m ²	Moment t.m
1	2.50	1.000	0.808	2.02	13.13	13.125
2	5.00	1.000	0.615	3.08	20.00	20.000
3	6.50	1.000	0.500	3.25	21.13	21.125

2.3 rail and slab rail

$$gas = 0.228 \text{ t/m'}$$

Point	Distance m	Load t/m'	Reaction ton	I.L. Moment t.m	Area of I.L. m ²	Moment t.m
1	2.50	0.228	0.808	2.02	13.13	2.992
2	5.00	0.228	0.615	3.08	20.00	4.560
3	6.50	0.228	0.500	3.25	21.13	4.817

2.4 pedestrian load

$$q = 1.000 \text{ t/m'}$$

Point	Distance m	Load t/m'	Reaction ton	I.L. Moment t.m	Area of I.L. m ²	Moment t.m
1	2.50	1.000	0.808	2.02	13.13	13.125
2	5.00	1.000	0.615	3.08	20.00	20.000
3	6.50	1.000	0.500	3.25	21.13	21.125

(t). Initial case by self weight of beam = Mg

Point	Distance m	Mg t.m
1	2.50	8.441
2	5.00	12.662
3	6.50	13.586

$$Yt' = It' + ht - Yb' = 73.02$$

$$Ix' = tot.Ix' - tot.Aj' \times Yb' \sim 2$$

$$= 40,777,534 - 6,073 \times 5,333$$

$$= 40,771,461 - 32,382,439$$

$$= 8,389,023 \text{ cm}^4$$

Section modulus

$$\text{top fiber } Wt' = \frac{Ix'}{Yt'}$$

$$Yt' =$$

$$8,389,023$$

$$= 37.0$$

$$= 226,883.1 \text{ cm}$$

$$\text{bottom fiber } Wb' = \frac{Ix'}{Yb'}$$

$$Yb' =$$

$$8,389,023$$

$$= 73.0$$

$$= 114,878.9 \text{ cm}$$

Kern

$$\text{top fiber } kt' = \frac{Wb'}{Ac'}$$

$$Ac' =$$

$$114,878.9$$

$$= 6,072.5$$

$$= 18.9 \text{ cm}$$

$$\text{bottom fiber } kb' = \frac{Wt'}{Ac'}$$

$$Ac' =$$

$$226,883.1$$

$$= 6,072.5$$

$$= 37.4 \text{ cm}$$

Point	Distance m	Mg t.m	M slab t.m	M rail t.m	Total t.m
1	2.50	8.44	13.13	2.992	24.559
2	5.00	12.86	20.00	4.560	37.423
3	6.50	13.59	21.13	4.817	39.528

Moment support by composite section

	Point Distance m	Mq t.m
1	2.50	13.13
2	5.00	20.00
3	6.50	21.13

Mg	=	moment support by precast concrete prestressed
Md	=	moment support by precast concrete prestressed (beam + concrete insitu)
Ml	=	moment support by composite section (DI + LL)
Mc	=	moment support by composite section (DI + LL + Asphalt)

3. Influence line of shear and shear

Load	Distance m	Reaction ton	Area of I.L. m ²	Shear ton
3.1 self weight	0.643	1.000	6.50	4.180
3.2 slab	1.000	1.000	6.500	6.500
3.3 rail	0.228	1.000	6.50	1.482
3.4 pedestrian	1.000	1.000	6.50	6.500
				18.662

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Project	: Median Flood Control	Japan International Cooperation Agency (JICA)	Calculation sheet
Channel	: Median Flood Way		
Structure	: Br - P8		
Station	: DU. 19 + 00		
		Date	Nos of sheet
			Prepared by

1.4.3. Prestress Force and numbers of tendon

M minimum	■	19,856 t.m
M transfer	■	1,985,600 kg.cm
M dead load	■	19,856 t.m
	■	1,985,600 kg.cm
M composite	■	57,771 t.m
	■	5,777,100 kg.cm
	■	30,880 t.m
	■	3,088,000 kg.cm

Characteristic of Concrete

specified strength (fc) Concrete class, K 400	ft ²	kg/cm ²
fc ¹	190.00	190.00
fc ²	180.00	180.00
fc ³	0.00	0.00

Compression strength

Section properties		Beam		Composite	
h	90.00	cm	thick of slab	=	20.00
A	2,572.00	cm ²	thick of panel	=	0
Yt	53.68	cm	h'	=	110.00
Yb	36.32	cm	Ac	=	6,072.00
I	2,265,607.00	cm ⁴	Yt'	=	36.98
Wt	42,228.00	cm ³	Yb'	=	73.02
Wb	62,400.00	cm ³	I'	=	6,389,023
kt	24.26	cm	Wt'	=	228,883
kb	16.42	cm	Wb'	=	114,478
•	25.07	cm	kt'	=	16.90
			kb'	=	37.40
			n	=	0.75

Computation prestressing force

1. Initial case

1st equation top fiber

Top row	by self weight of beam
Pl	Pd.Wt - Mg

	(4th - 5)

by self weight of beam

Mg	1,985,600 kg, cm	
ra	0.00 kg/cm ²	
wt	42,228.00 cm ³	
rd, wt	0.00 cm ³	
kb	16.42 cm	
	25.07 cm	
rd, wt - M min	0.00 -	1,985,600 = (1,985,600) kg, cm
kb - ●	16.42 -	25.07 = (8.65) cm
	(1,985,600)	
Pi	-----	229,549
	(8.65)	
		229.55 ton

2nd equation bottom fiber

Pt	$f_{ci}W_b + Mg$ ----- by self weight of beam
----	---

by self weight of beam

Mg	1,985,600 kg.cm	
Fcl	190.00 kg.cm	
Wb	62,400.00 kg/cm ²	
Fcl.Wb	11,856,000.00 cm ³	
kt	24.26 cm ³	
•	25.07 cm	
M min + Fcl.Wb	11,856,000	13,841,600 kg.cm
kt + e	24.26 +	49.33 cm
Pi	13,841,600	280,592
	49.33	
	280.59 ton	

1st equation top fiber

2. After transfer

Pi	$\frac{F_{cl}W_b + Mg}{n(e + kt)}$	
----	------------------------------------	--

after prestress losses

Mg	1,985,600 kg.cm	
Fcl	190.00 kg.cm	
Wb	62,400.00 kg/cm ²	
Fcl.Wb	11,856,000.00 cm ³	
kt	24.26 cm ³	
•	25.07 cm	
(kt + e)	18.42 +	49.33 cm
Fcl.Wb + M min	11,856,000	13,841,600 kg.cm
n.(kt + e)	0.75 x	49.33 cm
Pi	13,841,600	280,592
	49.33	
	280.59 ton	

2nd equation bottom fiber

Pi	$\frac{F_{cl}W_b + Mg}{n(e + kt)}$	
----	------------------------------------	--

after prestress losses

Mg	1,985,600 kg.cm	
Fcl	190.00 kg.cm	
Wb	62,400.00 kg/cm ²	
Fcl.Wb	11,856,000.00 cm ³	
kt	24.26 cm ³	
•	25.07 cm	
(kt + e)	18.42 +	49.33 cm
Fcl.Wb + M trans.	11,856,000	13,841,600 kg.cm
n.(kt + e)	0.75 x	49.33 cm
Pi	13,841,600	280,592
	49.33	
	280.59 ton	

3. Dead load

1st equation top fiber

Pi	$\frac{F_{cl}W_b + M_{dl}}{n(e + kt)}$	
----	--	--

by beam and concrete insitu

Mdl	5,777,100 kg.cm	
Fcl	190.00 kg.cm	
Wb	62,400.00 kg/cm ²	
Fcl.Wb	11,856,000.00 cm ³	
kt	24.26 cm ³	
•	25.07 cm	
(kt + e)	18.42 +	49.33 cm
Fcl.Wb + M dl	11,856,000	13,841,600 kg.cm
n.(kt + e)	0.75 x	49.33 cm
Pi	13,841,600	280,592
	49.33	
	280.59 ton	

kb	18.42 cm	
•	25.07 cm	
(kb + e)	18.42 +	49.33 cm
Fcl.Wb + M dl	11,856,000	13,841,600 kg.cm
n.(kb + e)	0.75 x	49.33 cm
Pi	13,841,600	280,592
	49.33	
	280.59 ton	

2nd equation bottom fiber

Pi	$\frac{F_{cl}W_b + M_{dl}}{n(e + kt)}$	
----	--	--

by beam and concrete insitu

Mdl	5,777,100 kg.cm	
Fcl	190.00 kg.cm	
Wb	62,400.00 kg/cm ²	
Fcl.Wb	11,856,000.00 cm ³	
kt	24.26 cm ³	
•	25.07 cm	
(e + kt)	25.07 +	49.33 cm
Fcl.Wb + M dl	11,856,000	13,841,600 kg.cm
n.(e + kt)	0.75 x	49.33 cm
Pi	13,841,600	280,592
	49.33	
	280.59 ton	

4. Service load

3rd equation top fiber

Pi	$\frac{F_{cs}W_t + M_{dl} - M_{c,Wt/Wt}}{n(kb + e)}$	
----	--	--

by service load

Mc	3,088,000 kg.cm	
Mdl	5,777,100 kg.cm	
Fcs	180.00 kg/cm ²	
Wt	42,228.00 cm ³	
Wt'	228,883.00 cm ³	
Wb	228,883.00 cm ³	
kb	16.42 cm	
•	25.07 cm	
(kb + e)	16.42 +	49.33 cm
Wt/Wt'	42,228.00	228,883.00
Mc.Wt/Wt'	3,088,000.00	7,601,040.00 kg.cm
Fcs.Wt	7,601,040.00	13,841,600 kg.cm
Fcs.Wt - Mdl	1,823,940.00	1,249,194.14 kg.cm
n.(kb + e)	0.75 x	49.33 cm
Pi	1,249,194.14	192,554.01
	49.33	
	192.55 ton	

4th equation bottom fiber

Pi	$\frac{F_{cs}W_t + M_{dl} + M_{c,Wt/Wb}}{n(e + kt)}$	
----	--	--

by service load

Mc	3,088,000 kg.cm	
Mdl	5,777,100 kg.cm	
Fcs	180.00 kg/cm ²	
Wt	42,228.00 cm ³	
Wb	62,400.00 cm ³	
Wb'	114,478.00 cm ³	

Project : Medan Flood Control

Channel : Medan Flood Way

Structure : Br - F 8

Station : DU -19 + 00

Japan International

Cooperation Agency

(JICA)

Calculation sheet

Date

Nos of sheet

Prepared by

1.4.5 Centre Gravity of tendon

for span = 13.00 m

Y = centre gravity of tendon (cgs)

to the centre gravity of concrete (cgc)

dia. of strand = 12.70 mm

depth = 90.00 cm

area = 94.20 mm²

= 0.9420 cm²

Nos of strand = 6.00

area per tendon = 5.6520 cm²

Yb = 36.22 cm

Yt = 53.78 cm

top cover concrete = tc

tc = 20.00 cm

a. at stressing of tendon

a.1 cgs tendon at stressing tendon no.3 at edge of span

tendon no	distance to each cm	tendon no	distance to cgc cm	area of tendon As cm ²	static moment Y cm	As Y cm ³
No.3 to no.2	= 25.00	No.3	33.78	5.6520 x	33.78 =	190.92
No.3 to cgc	= 8.78	No.2	25.00	5.6520 x	25.00 =	141.30
			33.78		11.30	332.22
e at edge	= 332.22		29.39 cm			
	= 11.304					

a.2 cgs tendon at stressing tendon no.3 at middle of span

tendon no	distance to each cm	tendon no	distance to cgc cm	area of tendon As cm ²	static moment Y cm	As Y cm ³
No.3 to no.2	= 13.00	No.3	13.00	5.652 x	13.00 =	73.476
No.2 to outside	= 6.50	No.2	6.50	5.652 x	6.50 =	36.738
No.1 to outside	= 6.50	No.1	6.50	5.652 x	6.50 =	36.738
			13.00	5.652		73.476
Ys	= 73.48		13.00 cm			
	= 5.652					
e at middle	= Yb -	Ys				
	= 36.22 -	13.00				
	= 23.22 cm					

cgs-13

b. at transfer		b.1 cgs tendon at edge of span			
tendon no		distance to each	tendon no	distance to outside	area of tendon As
No.3 to No.2	=	25.00	No.3	70.00	5.6520 x 70.00 = 395.64
No.2 to No.1	=	25.00	No.2	45.00	5.6520 x 45.00 = 254.34
No.1 to outside	=	20.00	No.1	20.00	5.6520 x 20.00 = 113.04
		70.00		16.96	763.02
Ys	=	763.02			
	=	16.956			
e at edge	=	Yb	-	Ys	
	=	36.22	-	45.00	
	=	(8.76) cm			
b.2 cgs tendon at middle of span					
Strand no		distance to each	Strand no	distance to cgc	area of strand As
No.3 to No.2	=	6.50	No.3	13.00	5.6520 x 13.00 = 73.48
No.2 to No.1	=	0.00	No.2	6.50	5.6520 x 6.50 = 36.74
No.1 to outside	=	6.50	No.1	6.50	5.6520 x 6.50 = 36.74
		13.00		16.96	146.95
Ys	=	146.95			
	=	16.956			
e at middle	=	Yb	-	Ys	
	=	36.22	-	8.67	
	=	27.55	cm		

bottom fibre									
f _a	P _i	+	-----	M _g	-----	-----	-----	-----	190 kg/cm ²
	A _c		-----	W _b	-----	-----	-----	-----	
=	257.351	+	-----	5,975,690	-----	1,985,600	-----	-----	190 kg/cm ²
	2,572	+	-----	62,400	-----	62,400	-----	-----	
=	100.06	+	-----	95.76	-----	31.82	-----	-----	190 kg/cm ²
	164.00	<	-----	190 kg/cm ²	-----	-----	-----	-----	
2. after transfer top fibre									
f _t	P _i	-----	-----	M _{tr}	-----	-----	-----	-----	0 kg/cm ²
	A _c	-----	-----	W _t	-----	-----	-----	-----	
=	228.065	-----	-----	6,228,091	-----	1,985,600	-----	-----	0 kg/cm ²
	2,572	-----	-----	42,228	-----	42,228	-----	-----	
=	87.89	-----	-----	147.49	-----	47.02	-----	-----	0 kg/cm ²
	(12.57)	>	-----	0 kg/cm ²	-----	-----	-----	-----	
bottom fibre									
f _b	P	-----	-----	M _{tr}	-----	-----	-----	-----	180 kg/cm ²
	A _c	-----	-----	W _b	-----	-----	-----	-----	
=	228.065	-----	-----	6,228,091	-----	1,985,600	-----	-----	180 kg/cm ²
	2,572	-----	-----	62,400	-----	62,400	-----	-----	
=	87.89	-----	-----	99.81	-----	31.82	-----	-----	180 kg/cm ²
	155.88	<	-----	180.00 kg/cm ²	-----	-----	-----	-----	
3. Dead load (self weight beam + concrete cast in place) top fibre									
f _t	P	-----	-----	M _{dl}	-----	-----	-----	-----	0 kg/cm ²
	A _c	-----	-----	W _t	-----	-----	-----	-----	
=	193.013	-----	-----	5,317,515	-----	5,777,100	-----	-----	0 kg/cm ²
	2,572	-----	-----	42,228	-----	42,228	-----	-----	
=	75.04	-----	-----	125.92	-----	136.81	-----	-----	0 kg/cm ²
	85.93	>	-----	0.00 kg/cm ²	-----	-----	-----	-----	
bottom fibre									
f _b	P	-----	-----	M _{dl}	-----	-----	-----	-----	180 kg/cm ²
	A _c	-----	-----	W _b	-----	-----	-----	-----	
=	228.065	-----	-----	6,228,091	-----	5,777,100	-----	-----	180 kg/cm ²
	2,572	-----	-----	62,400	-----	62,400	-----	-----	
=	87.89	-----	-----	99.81	-----	92.58	-----	-----	180 kg/cm ²
	95.12	<	-----	180 kg/cm ²	-----	-----	-----	-----	
4. at service load beam unshored top fibre									
f _a	M composite	-----	-----	Y _t = depth of f _a	-----	-----	-----	-----	36.98 cm
	W _t	-----	-----	h _s = depth of slab	-----	-----	-----	-----	22.00 cm
=	3,098,000	-----	-----	Y _t = Y _t - h _s	-----	-----	-----	-----	14.98 cm
	228.853	-----	-----	-----	-----	-----	-----	-----	
=	13.61	<	-----	180.00 kg/cm ² on top of slab deck	-----	-----	-----	-----	

f _a : f _a = Y _t : Y _t f _a x Y _t = f _a x Y _t f _a = f _a x Y _t /Y _t									
f _a	-----	-----	-----	-----	-----	-----	-----	-----	-----
	14.98	x	-----	-----	-----	-----	-----	-----	-----
=	36.98	-----	-----	-----	-----	-----	-----	-----	-----
	5.51	<	-----	180 kg/cm ² (on bottom of slab deck)	-----	-----	-----	-----	
total f _a	-----	-----	-----	-----	-----	-----	-----	-----	-----
	concrete compressive on top of girder	-----	-----	-----	-----	-----	-----	-----	-----
total f _a	-----	-----	-----	-----	-----	-----	-----	-----	-----
	f _a dl	+	-----	f _a composite	-----	-----	-----	-----	-----
total f _a	-----	-----	-----	-----	-----	-----	-----	-----	-----
	85.83	+	-----	5.51	-----	91.44 kg/cm ²	-----	-----	180 kg/cm ²
bottom fibre									
f _b	-----	-----	-----	-----	-----	-----	-----	-----	-----
	concrete compressive on bottom of girder in composite	-----	-----	-----	-----	-----	-----	-----	-----
f _b	-----	-----	-----	-----	-----	-----	-----	-----	-----
	M composite	-----	-----	-----	-----	-----	-----	-----	-----
=	3,098,000	-----	-----	-----	-----	-----	-----	-----	-----
	114.875	-----	-----	-----	-----	-----	-----	-----	-----
=	26.88	-----	-----	-----	-----	-----	-----	-----	-----
	95.12	-----	-----	-----	-----	-----	-----	-----	-----
f _b	-----	-----	-----	-----	-----	-----	-----	-----	-----
	f _b dl	-----	-----	-----	-----	-----	-----	-----	-----
=	95.12	-----	-----	-----	-----	-----	-----	-----	-----
	26.88	-----	-----	-----	-----	-----	-----	-----	-----
=	0 kg/cm ²	-----	-----	-----	-----	-----	-----	-----	-----
	68.24 kg/cm ²	-----	-----	-----	-----	-----	-----	-----	-----
=	0.00 kg/cm ²	-----	-----	-----	-----	-----	-----	-----	-----

1.4.8 Deflection of beam

I. deflection computation

deflection is moment at any point by "bending moment" as load

I.a. deflection due to uniform load

load = g
 $Mo = 1/8 \cdot g \cdot l^2$
 $Aa = 4/3 \cdot a \cdot b$
 $a = l/2$
 $b = Mo$
 $Aa = 4/3 \cdot (l/2) \cdot Mo$
 $R = 2/3 \cdot l \cdot Mo$
 $= 1/2 \cdot Aa$
 $= 2/6 \cdot l \cdot Mo$
 $xp1 = 3 \cdot a/8$
 $= 3 \cdot l/16$
 $= 3/16 \cdot l$
 $ya = R \cdot l/2 - Aa/2 \cdot xp1$
 $= 2/6 \cdot l \cdot Mo \cdot l/2 - 2/6 \cdot l \cdot Mo \cdot (3/16 \cdot l)$
 $= Mo \cdot l^2 \cdot (1/8 - 1/16)$
 $= Mo \cdot l^2 \cdot (8/48 - 3/48)$
 $= Mo \cdot (5/48 \cdot l^2)$

$ya = 5 \cdot Mo \cdot l^2 / 48 \cdot EI$

I.b. deflection due to tendon forces

load = P
 $Mo = P \cdot e2$
 $Ab = Mo \cdot l$
 $R = (Mo \cdot l) / 2$
 $xp2 = l/4$
 $ya = Re2 \cdot l/2 - Ab \cdot l/2 \cdot xp2$
 $= (Mo \cdot l) / 2 \cdot l/2 - (P \cdot e2 \cdot l) / 2 \cdot l/4$
 $= (Mo \cdot l) \cdot l/4 - l/8$
 $= (Mo \cdot l) \cdot l \cdot (2/8 - 1/8)$
 $= (Mo \cdot l) \cdot l \cdot (2/8 - 1/8)$

$yb = Mo \cdot l^2 / 8 \cdot EI$

I.c. deflection due to knife load

load = Pk
 $Mo = 1/4 Pk \cdot l$
 $Ac = Mo \cdot l/2$
 $= Mo/4 \cdot l$
 $= l/3 \cdot l/2$
 $= l/6$

M max = yc
 $= Mo/4 \cdot l/2 - Ac/2 \cdot l/6$
 $= Mo/4 \cdot l/2 - Mo/4 \cdot l/6$
 $= Mo \cdot (l^2/8 - l^2/24)$
 $= Mo \cdot l^2 \cdot (1/8 - 1/24)$
 $= Mo/12 \cdot l^2$

$yc = Mo \cdot l^2 / 12 \cdot EI$

I.d. data
span = 13.00 m
= 1.300 cm
E-c = 343.327 kg/cm²
Inertia = 2,266,607 cm⁴
Ex1 = 7.78E+11 kg.cm²

II. loading

II.1 dead load
weight of beam = 0.643 t/m³
weight of slab = 1.000 t/m³
rail = 0.288 t/m³

II.2 live load = 1.000 t/m³

III. moment

III.1 by dead load

load	coefficient	x	load	x	l ²	moment
			t/m ³		m	kg.cm
by beam	0.125	x	0.643	x	169.0	1,358,138
slab deck	0.125	x	1.000	x	169.0	2,112,500
asphalt	0.125	x	0.288	x	169.0	608,400

III.2 moment due to prestressed force

III.2.1 at tressing of tendon no.3

P jacking = 85.784 ton / tendon
P initial = 75.355 ton / tendon
Nos. of tendon = 3.00 ton
P jacking = 257.352 ton
P initial = 226.065 ton
P final = 169.549 ton

e1 edge = 0.2939 m
e2 middle = 0.2322 m
e1 + e2 = 0.5261 m

at edge M2 = $P \cdot x \cdot e2$
= 257.352 x 0.2322 = 5,975.713 kg.cm

at middle M1 = $-P \cdot x \cdot (e1 + e2)$
= (257.352) x 0.5261 = (13,539.289 kg.cm)

III.2.2 at transfer

e1 edge = (0.0868) m
e2 middle = 0.2755 m
e1 + e2 = 0.1887 m

at edge M2 = $P \cdot x \cdot e2$
= 226.065 x 0.2755 = 6,228.091 kg.cm

at middle M1 = $-P \cdot x \cdot (e1 + e2)$
= (226.065) x 0.1887 = (4,265.847 kg.cm)

IV. deflection computation.

IV.1 by dead load

load	5/48 x	Mo	x	1 ²	:	EI	=	y
	coefficient	kg.cm		cm ²		kg.cm ²		mm
self weight beam	(1) 0.1042 x	1,358,338 x	1,690,000 :	7,78E+11 =		3.07		
slab deck	(2) 0.1042 x	2,112,500 x	1,690,000 :	7,78E+11 =		4.78		
asphalt	(3) 0.1042 x	608,400 x	1,690,000 :	7,78E+11 =		1.38		

IV.1 by prestressing forces

IV.2.1 at stressing of tendon no.3

at edge	1/8 x	M2	x	1 ²	:	EI	=	y
	coefficient	kg.cm		cm ²		kg.cm ²		mm
Yes =	0.125 x	5,975,713 x	1,690,000 :	7,78E+11 =		16.22		
at middle	5/48 x	M1	x	1 ²	:	EI	=	y
	coefficient	kg.cm		cm ²		kg.cm ²		mm
Yes =	0.1042 x	(13,539,289) x	1,690,000 :	7,78E+11 =		(30.63)		
balance	=	(30.63) +	16.22 =	(14.41)				
by self weight beam	=	(14.41) +	3.07 =	(11.33)				
total	=							

IV.2.2 at transfer

at edge	1/8 x	M2	x	1 ²	:	EI	=	y
	coefficient	kg.cm		cm ²		kg.cm ²		mm
Yes =	0.125 x	6,228,091 x	1,690,000 :	7,78E+11 =		16.91		
at middle	5/48 x	M1	x	1 ²	:	EI	=	y
	coefficient	kg.cm		cm ²		kg.cm ²		mm
Yes =	0.1042 x	(4,265,847) x	1,690,000 :	7,78E+11 =		(9.65)		

balance	=	(9.65) +	16.91 =	7.26				
by self weight beam	=	7.26 +	3.07 =	10.33				
total deflection by tendon	=							

V. short term deflection

V.1 uncracked beam

load	coefficient	load	l ³	:	EI	=	y
		kg.cm	m		kg.cm ²		mm
pedestrian load	0.125 x	1,000 x	1.0 =	12,500			

load	coefficient	Mo	x	1 ²	:	EI	=	y
		kg.cm		cm ²		kg.cm ²		mm
uniform load	(4) 5/48 x	Mo	x	1 ²	:	EI	=	y
	coefficient	kg.cm		cm ²		kg.cm ²		mm
line load	(5) 1/12 x	12,500 x	1,690,000 :	7,78E+11 =		0.03		
	coefficient	kg.cm		cm ²		kg.cm ²		mm
	0.0833 x	0	x	1,690,000 :	7,78E+11 =	0.00		
		total						
								0.03

VI. long term deflection

VI.1 cracked beam

Cracking moment

Cracking moment is the moment for which the tensile stress on the extreme fibre of the concrete

section reaches a value equal to the modulus of rupture of the concrete.

modulus of rupture of the concrete

$$f_r = 7.5 \cdot \sqrt{f_c'} \text{ (psi)}$$

$$f_r = 7.5 \cdot \sqrt{f_c'} \text{ (kg/cm}^2\text{)}$$

$$f_r = 7.5 \cdot \sqrt{f_c'} \text{ (kg/cm}^2\text{)}$$

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$$f_r = 7.5 \cdot \sqrt{f_c'} \text{ (kg/cm}^2\text{)}$$

$$f_r = 7.5 \cdot \sqrt{f_c'} \text{ (kg/cm}^2\text{)}$$

n.P	=	0.75 x	226,065	=	169,549 kg
kt	=	24.26 cm			
e	=	27.55 cm			
(kt+e)	=	51.81 cm			
fr x wb	=	62,399.00 cm ³			
fr x wb	=	39.99 x	62,399.00		
M cr	=	2,495,167.73 kg.cm			
	=	n.P	(kt-e) +	fr.wb	
	=	169,548.75 x	51.81 +	2,495,167.73	
	=	8,784,320.74 +	2,495,167.73		
	=	11,279,488.47 kg.cm			

deflection by cracking moment

$$y_{crack} = 5/48 \times Mo \times l^3 / EI$$

5/48	x	Mo	x	1 ²	:	EI	=	yc
	coefficient	kg.cm		cm ²		kg.cm ²		mm
0.1042	x	11,279,488	x	1,690,000 :	7,78E+11 =			
				1,99E+12 :	7,78E+11 =			
					10.33 mm			

deflection by tendon

$$y_{tendon} = 5/48 \times Mo \times l^3 / EI$$

$$y_{tendon} = 5/48 \times Mo \times l^3 / EI$$

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$$y_{tendon} = 5/48 \times Mo \times l^3 / EI$$

$$y_{tendon} = 5/48 \times Mo \times l^3 / EI$$

1.4.9 Shear

Shear force by each load

Beam	Slab	Rail	Pedestrian	Total
load	load	load	load	load
ton	ton	ton	ton	ton
5.145	8.000	1.824	8.000	22.969

Inertia

Ht = 2,266,607 cm⁴
 Yt = 90.00 cm
 Yb = 53.68 cm
 e = 36.32 cm
 e = 27.55 cm

Area of section

= 2,573.00 cm²
 = 20.00 cm

Span

= 13.00 m
 = 22.97 x
 = 22,969.00 kg

Part	Depth	Width	Area	Distance	Al.Yl
t	w	Al	to top = Yl		
(1)	(2)	(3)	(4)	(5)	(6)
1	7.50	35.00	262.50	3.75	984.38
2	28.82	20.00	576.40	14.41	8,305.92
4	7.50	7.50	56.25	10.00	562.50
		AY	895.15		9,852.80

Sy = 9,852.80

tot.Ay = 895.15

Y = 11.01 cm

Av:y = 895 x 11.01

b.l = 9,853 cm²

20.00 x 2,266,607

45,332,140 cm⁵

e = 27.55 cm

4 x e = 4.00 x 27.55

110.20 cm

P jacking = 257,351 kg

n = 0.75

P = 193,013 kg

4 x e

Vt = 1,1020 x 193,013

1,300

163.62 kg

V = 22,969.00

Vt = 163.62

22,805.38 kg

vc = the shear resistance on the concrete
 Ay = area of the portion of section outside the shear plane
 outside the shear plane considered
 y = distance from neutral axis to centroid as Ay
 by = width of section at level y of the shear plane considered
 I = moment of inertia of the cross section

shear stress = $\frac{A_v y}{b \cdot I}$ x V net

vc = $\frac{9,852.80}{45,332,140}$ x 22,805

= 4.96 kg/cm²

vc ^ 2 = 24.57 kg/cm²

axial force = fx

P = 193,013.25

fx = $\frac{A_c}{P}$ = $\frac{2,573}{75.01}$ kg/cm²

fx/2 = $\frac{75.01}{2.00}$ = 37.51 kg/cm²

(fx/2) ^ 2 = 1,406.81

vc ^ 2 = 4.96

(fx/2) ^ 2 = 1,406.81

vc ^ 2 + (fx/2) ^ 2 = 1,411.76

R = $\sqrt{vc^2 + (fx/2)^2}$ = 37.57

Principle compressive force = f

f1 max = $\frac{+/- fx/2}{R}$ + $\frac{R}{37.51 + 37.57}$

= $\frac{75.08}{75.08}$ kg/cm² < 190.00 kg/cm²

f2 min = $\frac{(37.51) + 37.57}{0.07}$ kg/cm²

1 kg/cm² = 14.21 psi

fc = 400.00 kg/cm²

vc min allowance = 2 @sqrt(fc) = 5,864.24 psi

= 150.79 psi

= 10.61 kg/cm²

vc max allowance = 5 @sqrt(fc) = 376.97 psi

= 26.53 kg/cm²

4.1.8 SUBSTRUCTURE (BRIDGE SPAN 40.0M)

Case-1: Bridge Width = 16.0 m

Bridge Width = 9.0 m

Bridge Width = 7.0 m

PRESTRESSED CONCRETE BRIDGE COMPUTATION
CHANNEL PERCUT RIVER

Br - P 9

Station : Pe 200 + 25

span
Midle
40.00

Width

16.0 m

II SUB STRUCTURE

Abutment

- II.1 Neutral axis of Abutment
- II.2 Earthquake coefficient
- II.3 Axial forces on Abutment
- II.4 Coulumb coefficient
- II.5 Horizontal force
 - II.5.1 Case - I, Normal condition (HWL)
 - II.5.2 Case - II, Normal condition (LWL)
 - II.5.3 Case - III, Earthquake condition (HWL)
 - II.5.4 Case - IV, Earthquake condition (LWL)
- II.6 Combined Load on Abutment
 - II.6.1 Case - I, Normal condition (HWL)
 - II.6.2 Case - II, Normal condition (LWL)
 - II.6.3 Case - III, Earthquake condition (HWL)
 - II.6.4 Case - IV, Earthquake condition (LWL)

Project : Medan Flood Control Channel : Percut River Structure: Br - P9 Station : PE 200 + 25 m	Japan International Cooperation Agency (JICA)	Calculation sheet
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II. SUBSTRUCTURE

ABUTMENT

II.1 Neutral axis, Left side Abutment

depth of abutment = Ht	=	9.00 m				
depth beam	=	1.70 m				
depth slab	=	0.245 m				
depth of beam to deck	=	1.945 m				
depth of beam and deck	=	1.700 +	0.245	=	1.945 m	
depth bearing pad	=	1.945 +	0.100	=	2.045 m	
depth back wall	=	1.000 m				
depth of footing	=	9.000 -	2.045	=	1.000 = 5.955 m	
H1	=	2.045 m	W1	=	0.300 m	
H2	=	5.955 m	W1	=	2.300 m	
H3	=	1.000 m	W2	=	1.000 m	
Ht	=	9.000 m	W3	=	1.800 m	
			W3	=	5.100 m	
Depth of back fill						
H4	=	9.000 -	1.000	=	8.000 m	
H5	=	0.500 m	W4	=	0.550 m	
H6	=	0.250 m	W5	=	0.550 m	
H7	=	0.500 m	W6	=	0.200 m	
H8	=	0.250 m	W7	=	0.200 m	
H9	=	0.300 m	W8	=	0.300 m	
			W9	=	1.800 m	
A1	W1	= 0.300	X1 =	W1/2 + W2 + W3	= 1.800	= 2.950 m
	H1	= 2.045	Y1 =	H1/2	= 1.023	= 7.978 m
A2	W2	= 1.000	X2 =	W2/2 + W3	= 1.800	= 2.300 m
	H2	= 5.955	Y2 =	H2/2 + H3	= 1.000	= 3.978 m
A3	W3	= 5.100	X3 =	W3/2	= 2.550	= 2.550 m
	H3	= 1.000	Y3 =	H3/2	= 0.500	= 0.500 m
A4	W4	= 0.550	X4 =	W4/2 + W2 + W3	= 1.800	= 3.075 m
	H4	= 0.500	Y4 =	H4/2 + H3	= 1.000	= 6.455 m
A5	W5	= 0.550	X5 =	W5/3 + W2 + W3	= 1.000	= 2.983 m
	H5	= 0.250	Y5 =	H5/3 + H3	= 0.833 + 1.000	= 6.622 m
A6	W6	= 0.200	X6 =	W6/3 + W2 + W3	= 0.667 + 1.000	= 1.700 m
	H6	= 0.500	Y6 =	H6/3 + H3	= 0.167 + 1.000	= 5.788 m
A7	W7	= 0.200	X7 =	W7/3 + W2 + W3	= 0.667 + 1.000	= 1.733 m
	H7	= 0.250	Y7 =	H7/3 + H3	= 0.083 + 1.000	= 6.622 m
A8	W8	= 0.300	X8 =	W8/3 + W2 + W3	= 1.000	= 3.567 m
	H8	= 0.300	Y8 =	H8/3 + H3	= 0.100 + 1.000	= 1.100 m
A9	W9	= 1.800	X9 =	W9/3	= 1.200	= 1.200 m
	H9	= 0.300	Y9 =	H9/3 + H3	= 0.100 + 1.000	= 1.100 m

x-y

Wing wall									
Hgw	=	1.200 m	H back fill	=	8.00 m				
Ww1	=	3.000 m	Hw1	=	3.00 m				
Ww2	=	2.30 m	Hw2	=	1.00 m				
Ww3	=	0.70 m	Hw3	=	1.00 m				
Ww4	=	2.30 m	Hw4	=	3.70 m				
Ww5	=	2.30 m	Hw5	=	0.30 m				
Ww1	=	3.000	Xw1	=	Ww1/2 + W2 + W3	=	4.300 m		
Hw1	=	3.000	Yw1	=	Hw1/2	=	1.500		
Ww2	=	2.300	Xw2	=	Ww2/2 + W2 + W3	=	7.500 m		
Hw2	=	1.000	Yw2	=	Hw1 - Hw2/2	=	3.950 m		
Ww3	=	0.700	Xw3	=	W3 - Ww3/2	=	5.500 m		
Hw3	=	1.000	Yw3	=	Hw1 - Hw3	=	4.750 m		
Ww4	=	2.300	Xw4	=	Ww4/2 + W2 + W3	=	5.667 m		
Hw4	=	3.700	Yw4	=	Hw4/2 + Hw5 + H3	=	3.950 m		
Ww5	=	2.300	Xw5	=	W3 - Ww5/2	=	3.150 m		
Hw5	=	0.300	Yw5	=	Hw5/3 + H3	=	3.567 m		
				=	0.100 + 1.000	=	1.100 m		
Guide wall									
Gw	Wgw	=	3.000	Xgw	=	Wgw/2 + W2 + W3	=	4.300 m	
	Hgw	=	1.200	Ygw	=	Ht + Hgw/2	=	9.600 m	
Approach slab									
Apr	Wapr	=	3.000	Xapr	=	Wapr/2 + W2 + W3	=	4.300 m	
	Hapr	=	0.200	Yapr	=	Ht - H1 + Hapr/2	=	7.055 m	
Asphalt									
Asp	Wasp	=	3.000	Xasp	=	Wap/2 + W2 + W3	=	4.300 m	
	Hasp	=	0.050	Yasp	=	Ht - Hasp/2	=	8.975 m	
Back fill HWL									
a. In case High Water Level									
Top deck elevation	=	24.261 m							
HWL elevation	=	21.316 m							
Base foiling elev.	=	24.261 m							
Top foiling elev.	=	15.261 m							
Top foiling elev	> or	higher than Elv. of LWL							
Depth of wet soil									
H well	=	24.261 m							
H wellr	=	21.316 m							
Depth of saturated soil									
H satd	=	8.000 m							

a.1 soil wet condition									
WwetL	=	2.300	X1 =	WwetL/2	+	W2	+	W/3	
HwetL	=	2.945	Y1 =	HwetL/2	+	H3	+	1.800	= 3.950 m
WwetR	=	1.800	X2 =	WwetR/2	+	H3	+	1.473	= 7.528 m
HwetR	=	0.900	Y2 =	HwetR/2	+	H3	+	1.000	= 0.900 m
									= 1.450 m
a.2 soil saturated condition									
WsatL	=	2.300	X1 =	WsatL/2	+	W2	+	W/3	
HsatL	=	5.055	Y1 =	HsatL/2	+	H3	+	1.800	= 3.950 m
WsatR	=	1.800	X2 =	WsatR/2	+	H3	+	1.000	= 3.528 m
HsatR	=	5.055	Y2 =	HsatR/2	+	H3	+	1.000	= 0.900 m
									= 3.528 m
b. In case Low Water Level									
Lwl elevation	=	17.674	m						
Depth of wet soil									
HwetL	=	Top elev.	=	Lwlev.	=	17.674	=	6.587 m	
HwetR	=	24.261	=	Hwe	=	17.674	=	2.842 m	
Depth of saturated soil									
Hsat	=	20.516	=	8.000	=	6.587	=	1.413 m	
b.1 soil wet condition									
WwetL	=	2.300	X1 =	WwetL/2	+	W2	+	W/3	
HwetL	=	6.587	Y1 =	HwetL/2	+	H3	+	1.800	= 3.950 m
WwetR	=	1.800	X2 =	WwetR/2	+	H3	+	1.000	= 4.294 m
HwetR	=	2.842	Y2 =	HwetR/2	+	H3	+	1.000	= 0.900 m
									= 2.421 m
b.2 soil saturated condition									
WsatL	=	2.300	X1 =	WsatL/2	+	W2	+	W/3	
HsatL	=	1.413	Y1 =	HsatL/2	+	H3	+	1.800	= 3.950 m
WsatR	=	1.800	X2 =	WsatR/2	+	H3	+	1.000	= 1.707 m
HsatR	=	1.413	Y2 =	HsatR/2	+	H3	+	1.000	= 0.900 m
									= 1.707 m

Project : Medan Flood Control

Channel : Percut River

Structure : Br - P 9

Station : PE 200 + 25

Japan International Cooperation Agency (JICA)

Calculation sheet

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II.2 Lateral earth pressure due to earthquake

Kh = C x I x S

Kh = coefficient of horizontal seismic loading

C = the base shear coefficient, base on the data : seismic zone and soil condition

Sismic zone = 4

soil condition soft soil C = 0.15

S = the structure type factor = 1

I = the importance factor = 1

for bridge carrying more than 2,000 vehicle per day

I = 1.2

for temporary bridge

I = 0.8

for all other permanent bridge

I = 1

C	I	S	kh
0.15	1.2	1	0.18

C	I	S	kh
0.15	0.8	1	0.12

C	I	S	kh
0.15	1	1	0.15

soil condition firm soil C = 0.10

for bridge carrying more than 2,000 vehicle per day

for all other permanent bridge

C	I	S	kh
0.1	1.2	1	0.12

C	I	S	kh
0.1	1.00	1	0.10

for temporary bridge

C	I	S	kh
0.1	0.8	1	0.08

source, Bridge Management System

EO

Project : Medan Flood Control	Japan International Cooperation Agency (JICA)	Date	Calculation sheet
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Station : PE 200 + 25			

II.4 Column's coefficient

Right side Abutment

Normal condition

θ	=	30.00 deg.	$\theta - \alpha$	=	30.00 -	0.0 = 30.00	deg.
α	=	0.0 deg.	$\theta + \delta$	=	30.00 +	30.00 = 60.00	deg.
δ	=	0.0 deg.	$\theta - \beta$	=	30.00 -	0.0 = 30.00	deg.
δ	=	30.00 deg.	$\alpha + \delta$	=	0.0 +	30.00 = 30.00	deg.
β	=	0.0 deg.	$\alpha - \beta$	=	0.0 -	0.00 = 0.00	deg.

$$\cos^2(\theta - \alpha)$$

$$\cos^2 \alpha \cos(\alpha + \delta) \cdot \{1 + \sqrt{1 - \sin^2(\theta - \beta)}\}^2$$

$$\cos(\alpha + \delta) \cdot \cos(\alpha - \beta)$$

$\cos(\alpha)$	=	1.000	$\sin(\theta + \delta)$	=	0.866
$\cos^2(\alpha)$	=	1.000	$\sin(\theta - \beta)$	=	0.500
$\cos(\theta - \alpha)$	=	0.866	$\cos(\alpha + \delta)$	=	0.866
$\cos^2(\theta - \alpha)$	=	0.750	$\cos(\alpha - \beta)$	=	1.000
A	=	$\sin(\theta + \delta) \cdot \sin(\theta - \beta)$	B	=	$\cos(\alpha + \delta) \cdot \cos(\alpha - \beta)$
	=	0.866 x 0.500		=	0.866 + 1.000
	=	0.433		=	0.866
C	=	$\cos^2(\alpha) \cdot \cos(\alpha + \delta)$	$\sqrt{1 - \sin^2(\theta - \beta)}$	=	0.71
	=	1.000 x 0.866		=	
	=	0.866		=	
D	=	$(1 + \sqrt{1 - \sin^2(\theta - \beta)})^2$	C.D	=	0.866 x 2.914
	=	(1.000 + 0.707)^2		=	2.524
	=	2.914		=	
Ka	=	$\cos^2(\theta - \alpha)$	0.75	=	0.297
	=	C.D	2.52	=	

Earth quake condition

$$\cos(\theta - \alpha) - \cos \theta$$

$$\sin(\theta + \delta) \cdot \sin(\theta - \beta)$$

$$\cos \theta \cdot \cos^2 \alpha \cos(\alpha + \delta) \cdot \{1 + \sqrt{1 - \sin^2(\theta - \beta)}\}^2$$

$$\cos(\alpha + \delta) \cdot \cos(\alpha - \beta)$$

composite seismic angle

$\tan^{-1} k$

Y

Y - 1

seismic coefficient in air

0.15

2.00

1.00

Y

Y - 1

2.00

1.00

0.15

0.30

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1.5.2 Normal Condition
Case - II, Low Water Level

Depth of abutment	=	9.00 m
Surcharge load	=	1.00 t/m ²
Unit weight wet soil	=	1.85 t/m ³
Unit weight saturated soi	=	2.00 t/m ³
Unit weight of water	=	1.00 t/m ³
Width of footing		
W/F1	=	2.30 m
W/F2	=	1.00 m
W/F3	=	1.80 m
W/F3	=	5.10 m
Elevation of Hwl	=	21.316 m
Elevation of Lwl	=	17.674 m
Top slab elevation	=	24.261 m
Depth of footing	=	1.000 m
Elv of base footing	=	24.261 - 9.00 = 15.261 m
Elv of top footing	=	15.261 + 1.00 = 16.261 m
Depth of back fill	=	24.261 - 16.261 = 8.000 m
Depth of wet soil = H wet		
H wet = Top elv. - Elevation of Lwl		
H wet = 24.261 - 17.674 =		6.587 m
Depth of saturated soil = H sat		
H sat = 8.00 - 6.587 =		1.413 m

A Active earth pressure			
By live load			
Surcharge load = (q)			

q	=	1.000 t/m ²	Sin(alpha)	=	0.000
delta	=	0.000	L	=	1.000 m
alpha	=	0.000	H	=	9.000 m
Cos(alp	=	1.000	B	=	5.100 m

coefficient of active pressure Ka = 0.297			
Pq	=	q x H x L x Ka	
Pq	=	1.000 x 8.000 x 1.000 x 0.297	
Pq	=	2.376 ton	

Pqv	=	Hq x Sin(alpha)	Pqb	=	Hq x Cos(alpha)
	=	2.376 x 0.000		=	2.376 x 1.000
	=	0.000 ton		=	2.376 ton
X	=	5.100 m	Y	=	8.000 : 2.000
				=	4.000 m

B By earth B-1 Wet soil			
Length	=	1.000 m	
Hwet	=	6.587 m	
Hsat	=	1.413 m	

1. Load triangular			
Pw	=	0.500 x 5.000 x Hwet^2 x L x Ka	
	=	0.500 x 1.850 x 43.389 x 1.000 x 0.297	
	=	11.920 ton	

Hsat = 5.055 m		Pw(h) = Pw x Cos(alpha)	
Pw(v) = Pw x Sin(alpha)	=	Pw x 0.000	=
Pw(v)	=	2.383 x 1.000	=
Pw(v)	=	0.000 ton	=
X	=	5.100 m	Y = 2.945 : 3.000 + 5.055
			= 6.037 m
B-2 Saturated			
Hsat = 5.055 m			
2. Load rectangular			
Psl	=	Ysat x Hsat x L x Ka	
Psl	=	2.000 x 5.055 x 1.000 x 0.297	
Psl	=	3.003 ton	
Psl(v) = Hsat x Sin(alpha)	=	Hsat x Cos(alpha)	
Psl(v)	=	3.003 x 0.000	=
Psl(v)	=	0.000 ton	=
X	=	5.100 m	Y = 5.055 : 2.000
			= 2.528 m
3. Load triangular			
Psl	=	0.500 x Ysat x Hsat^2 x L x Ka	
Psl	=	0.500 x 2.000 x 25.553 x 1.000 x 0.297	
Psl	=	7.589 ton	
Psl(v) = Hsat x Sin(alpha)	=	Hsat^2 x Cos(alpha)	
Psl(v)	=	7.589 x 0.000	=
Psl(v)	=	0.000 ton	=
X	=	5.100 m	Y = 5.055 : 3.000
			= 1.685 m
4. Water			
Hw = 5.055 m			
Pw	=	0.500 x Yw x Hw^2 x L	
Pw	=	0.500 x 1.000 x 25.553 x 1.000	
Pw	=	12.777 ton	
X	=	5.100 m	
Y	=	5.055 : 3.000	
			= 1.685 m
Pw	=	0.500 x Yw x Hw^2 x L	
Pw	=	0.500 x 1.000 x 28.676 x 1.000	
Pw	=	14.338 ton	
X	=	0.500 m	
Y	=	5.355 : 3.000	
			= 1.785 m
5. Uplift HWL			
B = 5.100 m			
Pu	=	Hw x B x Yw	
Pu	=	5.055 x 5.100 x 1.000	
Pu	=	25.780 ton/m ²	
X	=	5.100 : 2.000	
			= 2.550 m

$P_w(v) =$	$P_w \times \sin(\alpha)$	$P_w(h) =$	$P_w \times \cos(\alpha)$
$=$	11.920×0.000	$=$	11.920×1.000
$=$	0.000 ton	$=$	11.920 ton
$X =$	5.100 m	$Y =$	$6.587 : 3.000 + 1.413$
			$= 3.609 \text{ m}$
B-2 Saturated			
2. Load rectangular			
$P_{s1} =$	$Y_{sat} \times H_{sat} \times L \times K_a$		
$=$	$2.000 \times 1.413 \times 1.000 \times 0.297$		
$=$	0.839 ton		
$P_{s1}(v) =$	$H_{sat} \times \sin(\alpha)$	$P_{s1}(h) =$	$H_{sat} \times \cos(\alpha)$
$=$	0.839×0.000	$=$	0.839×1.000
$=$	0.000 ton	$=$	0.839 ton
$X =$	5.100 m	$Y =$	$1.413 : 2.000$
			$= 0.707 \text{ m}$
3. Load triangular			
$P_{s2} =$	$0.500 \times Y_{sat} \times H_{sat}^2 \times L \times K_a$		
$=$	$0.500 \times 2.000 \times 1.997 \times 1.000 \times 0.297$		
$=$	0.593 ton		
$P_{s2}(v) =$	$H_{sat}^2 \times \sin(\alpha)$	$P_{s2}(h) =$	$H_{sat}^2 \times \cos(\alpha)$
$=$	0.593×0.000	$=$	0.593×1.000
$=$	0.000 ton	$=$	0.593 ton
$X =$	5.100 m	$Y =$	$1.413 : 3.000$
			$= 0.471 \text{ m}$
4. Water			
$P_{w \text{ in}} =$	$H_w \times Y_w \times H_w^2 \times L$		
$=$	$0.500 \times 1.000 \times 1.413^2 \times 1.000$		
$=$	$0.500 \times 1.000 \times 1.997 \times 1.000$		
$=$	0.998 ton		
$X =$	5.100 m		
$Y =$	$1.413 : 3.000$		
	$= 0.471 \text{ m}$		
$P_{w \text{ out}} =$	$H_w \times Y_w \times H_w^2 \times L$		
$=$	$0.500 \times 1.000 \times 1.713^2 \times 1.000$		
$=$	$0.500 \times 1.000 \times 2.934 \times 1.000$		
$=$	1.467 ton		
$X =$	0.500 m		
$Y =$	$1.713 : 3.000$		
	$= 0.571 \text{ m}$		
5. Uplift HWL			
$P_u =$	$H_w \times B \times Y_w$		
$=$	$1.413 \times 5.100 \times 1.000$		
$=$	7.206 ton/m^2		
$X =$	$5.100 : 2.000$		
	$= 2.550 \text{ m}$		

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11.5.3 Earthquake Condition		
Case - III, High Water Level		
Depth of abutment	$= 9.00 \text{ m}$	
Surcharge load	$= 0.50 \text{ t/m}^2$	
Unit weight wet soil	$= 1.85 \text{ t/m}^3$	
Unit weight saturated soil	$= 2.00 \text{ t/m}^3$	
Unit weight of water	$= 1.00 \text{ t/m}^3$	
Width of footing	$= 2.30 \text{ m}$	
$W/1$	$= 1.00 \text{ m}$	
$W/2$	$= 1.80 \text{ m}$	
$W/3$	$= 5.10 \text{ m}$	
Elevation of Hwl	$= 21.316 \text{ m}$	
Elevation of Lwl	$= 17.674 \text{ m}$	
Top slab elevation	$= 24.261 \text{ m}$	
Depth of footing	$= 1.000 \text{ m}$	
Elev. of base footing	$= 24.261 - 1.000 = 15.261 \text{ m}$	
Elev. of top footing	$= 15.261 + 1.000 = 16.261 \text{ m}$	
Depth of back fill	$= 24.261 - 16.261 = 8.000 \text{ m}$	
Depth of wet soil = H wet		
H wet = Top elev. - Elevation of Hwl	$= 24.261 - 21.316 = 2.945 \text{ m}$	
Depth of saturated soil = H sat	$= 8.00 - 2.945 = 5.055 \text{ m}$	
A Active earth pressure		
By live load		
Surcharge load = (q)		
$q = 0.500 \text{ t/m}^2$	$\sin(\alpha) = 0.000$	
$\delta = 0.000$	$L = 1.000 \text{ m}$	
$\alpha = 0.000$	$H = 9.000 \text{ m}$	
$\cos(\alpha) = 1.000$	$B = 5.100 \text{ m}$	
coefficient of active pressure	$K_e = 0.407$	
$P_q = q \times H \times L \times K_e$		
$P_q = 0.500 \times 8.000 \times 1.000 \times 0.407$		
$P_q = 1.628 \text{ ton}$		
$P_{qv} = H_q \times \sin(\alpha)$	$P_{qh} = H_q \times \cos(\alpha)$	
$= 1.628 \times 0.000$	$= 1.628 \times 1.000$	
$= 0.000 \text{ ton}$	$= 1.628 \text{ ton}$	
$X = 5.100 \text{ m}$	$Y = 8.000 : 2.000$	
	$= 4.000 \text{ m}$	
B By earth		
B-1 Wet soil		
Length	$= 1.000 \text{ m}$	
H wet	$= 2.945 \text{ m}$	
H sat	$= 5.055 \text{ m}$	
1. Load triangular		
$P_w = 0.500 \times Y_{wet} \times H_{wet}^2 \times L \times K_e$		
$= 0.500 \times 1.850 \times 8.673 \times 1.000 \times 0.407$		
$= 3.265 \text{ ton}$		

$Pw(v) =$	$Pw \times \sin(\alpha)$	$Pw(h) =$	$Pw \times \cos(\alpha)$
$=$	3.265×0.000	$=$	3.265×1.000
$=$	0.000 ton	$=$	3.265 ton
$X =$	5.100 m	$Y =$	$2.945 : 3.000 + 5.055$
$=$	6.037 m	$=$	6.037 m
B-2 Saturated			
2. Load rectangular			
$Ps1 =$	$Y_{sat} \times H_{sat} \times L \times K_c$	$H_{sat} =$	5.055 m
$=$	$2.000 \times 5.055 \times 1.000 \times 0.407$		
$=$	4.115 ton		
$Ps1(v) =$	$H_{sat} \times \sin(\alpha)$	$Ps1(h) =$	$H_{sat} \times \cos(\alpha)$
$=$	4.115×0.000	$=$	4.115×1.000
$=$	0.000 ton	$=$	4.115 ton
$X =$	5.100 m	$Y =$	$5.055 : 2.000$
$=$	2.528 m	$=$	2.528 m
3. Load triangular			
$Ps2 =$	$0.500 \times Y_{sat} \times H_{sat}^2 \times L \times K_c$	$H_{sat}^2 =$	$5.055^2 \times 1.000 \times 0.407$
$=$	$0.500 \times 2.000 \times 25.553 \times 1.000 \times 0.407$		
$=$	10.400 ton		
$Ps2(v) =$	$H_{sat}^2 \times \sin(\alpha)$	$Ps2(h) =$	$H_{sat}^2 \times \cos(\alpha)$
$=$	10.400×0.000	$=$	10.400×1.000
$=$	0.000 ton	$=$	10.400 ton
$X =$	5.100 m	$Y =$	$5.055 : 3.000$
$=$	1.685 m	$=$	1.685 m
4. Water			
$Pw \text{ in} =$	$0.500 \times Y_w \times H_w^2 \times L$	$H_w =$	5.055 m
$=$	$0.500 \times 1.000 \times 25.553 \times 1.000$		
$=$	12.777 ton		
$X =$	5.100 m		
$Y =$	$5.055 : 3.000$		
$=$	1.685 m		
$Pw \text{ out} =$	$0.500 \times Y_w \times H_w^2 \times L$	$H_w =$	5.355 m
$=$	$0.500 \times 1.000 \times 28.676 \times 1.000$		
$=$	14.338 ton		
$X =$	0.500 m		
$Y =$	$5.355 : 3.000$		
$=$	1.785 m		
5. Uplift IIWL			
$Pu =$	$H_w \times B \times Y_w$	$B =$	5.100 m
$=$	$5.055 \times 5.100 \times 1.000$		
$=$	25.780 ton/m^2		
$X =$	$5.100 : 2.000$		
$=$	2.550 m		

Project : Medan Flood Control	Japan International Cooperation Agency (JICA)	Calculation sheet
Channel : Percut River		Date
Structure : Br - P9		Nos of sheet
Station : PE 200 + 25		Prepared by

II.6 Combined load on Abutment left side
II.6.1 Normal condition
Case - I, High Water Level

Skew angle	$\alpha =$	56.00	degree
Length	$\sin \alpha =$	0.848	
Width of footing	$L =$	19.00	m
Thick of footing	$W =$	5.10	m
Length of skew	$L_{skew} =$	22.40	m
	$W_{skew} =$	6.01	m

Item	V load/m'	H load /m'	Xo	Yo	Vxo	Hy
Super structure						
Abutment	V-sp 44.781		2.200		98.518	
Back fill	V-abt 39.829		2.557		101.843	
	V-bf 56.979		2.704		154.071	
Active earth pressure						
Surcharge load		2.376		4.000		9.504
Wet soil triangle load		2.383		6.000		14.298
Sat.soil rectangular load		3.001		2.528		7.587
triangle load		7.589		1.685		12.787
Water		(12.777)		1.685		(21.529)
Up life		14.338		1.785		25.593
	(25.780)		2.550		(65.739)	
Total	115.809	16.910			288.693	48.240

$X' =$	$VoXo$	$HoYo$
$=$	-----	-----
$=$	$Vo \text{ tot}$	$Ho \text{ tot}$
$=$	288.693	48.240
$=$	-----	-----
$=$	115.809	16.910
$=$	2.493	2.853
$V =$	V/m'	Ls
$=$	115.81	22.40
$=$	2,594.63	ton

$H =$	H/m'	Ls
$=$	16.91	22.40
$=$	378.86	ton

$M =$	$V(X - (0.5 \times Ws))$	$H(Y - tb)$		
$=$	$0.5 \times W$	$=$	$0.50 \times$	6.01
$=$	$(X - (0.5 \times Ws))$	$=$	3.01 m	
$=$	$(Y - tb)$	$=$	$2.49 -$	3.01
$=$	$H \times (Y - tb)$	$=$	$(0.51) \text{ m}$	
$=$	$V \times (X - (0.5 \times Ws))$	$=$	$2.853 -$	1.00
$=$	$2,594.63 \times$	$=$	1.85 m	
$=$	$(1.33381) -$	$=$	$378.86 \times$	1.85
$=$	$(2.03574) \text{ t.m}$	$=$	701.93 t.m	
		$=$	$(H \times (Y - tb))$	
		$=$	$2,594.63 \times$	701.93
		$=$	$(0.51) -$	
		$=$	$(1.33381) -$	701.93
		$=$	$(2.03574) \text{ t.m}$	

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Project : Medan Flood Control			Japan International Cooperation Agency (JICA)			Calculation sheet		
Channel : Percut River						Date		
Structure : Br - P9						Nos of sheet		
Station : PE 200 + 25						Prepared by		
II.6 Combined load on Abutment left side								
II.6.3 Earthquake condition								
Case - III, High Water Level								
Skew angle	alpha =	58.00	degree					
	sin alpha =	0.85	degree					
Length	L =	19.00	m					
Width of footing	W =	5.10	m					
Thick of footing	H =	1.00	m					
Length of skew	Ls =	22.40	m					
	Ws =	6.01	m					
Item	V load/m'	H load /m'	Xo	Yo	Vxo	Hyo		
Super structure	44.781	6.140	2.100	6.955	94.040	42.704		
Abutment	39.829	7.169	2.557	2.730	101.843	19.571		
Back fill	56.979	10.256	2.730	4.298	155.553	44.080		
Active earth pressure								
Surcharge load	Pq	1.628		4.000		6.512		
triangle load	Pw	3.265		6.000		19.590		
rectangular load	Ps1	4.115		2.528		10.403		
triangle load	Ps2	10.400		1.685		17.524		
Water	Pw in	(12.777)		1.685		(21.529)		
	Pw out	14.338		1.785		25.593		
Up life	Pu	(25.78)	2.550		(65.739)			
Total		115.809	44.534		285.697	*****		
X'		VoXo		Y'	HoYo			
		-----			-----			
		Vo tot			Ho tot			
		285.697			164.448			
		-----			-----			
		115.809			44.534			
		2.467	m		3.693	m		
V		V/m'	x	Ls				
		115.81	x	22.40				
		2,594.63	ton					
H		H/m'	x	Ls				
		44.53	x	22.40				
		997.76	ton					
M		V x (X - (0.5 x W))		H x (Y - tb)				
		0.5 x W			0.50 x 6.01			
		(X - (0.5 x W))			3.01 m			
		(Y - tb)			2.47 - 3.01			
					(0.54) m			
		H x (Y - tb)			3.693 - 1.00			
					2.69 m			
					997.76 x 2.69			
					2,686.60 t.m			
		V x (X - (0.5 x W))			(H x (Y - tb))			
		2,594.63 x (0.54)			1,400.95 - 2,686.60			
					(4,087.55) t.m			

cld-aq

II.6.2 Case - II, Low Water Level									
Item	Vo/m'	Ho/m'	Xo	Yo	VoXo	HoYo			
Super structure	44.781		2.200		98.518				
Abutment	39.829		2.557		101.843				
Back fill	48.209		2.704		130.357				
Active earth pressure									
Hq	2.376					9.504			
triangle load	11.920					43.019			
rectangular load	0.839					0.593			
Hsat-1	0.593					0.279			
Hsat-2	(0.598)					(0.470)			
Pw in	1.467					0.838			
Pw out	(7.206)					0.000			
Pu									
Up life									
Total	125.613	16.197			330.718	53.763			
X'		VoXo		Y'	HoYo				
		-----			-----				
		Vo tot			Ho tot				
		330.718			53.763				
		-----			-----				
		125.613			16.197				
		2.833	m		3.319	m			
V		V/m'	x	Ls					
		125.61	x	22.40					
		2,814.28	ton						
H		H/m'	x	Ls					
		16.20	x	22.40					
		362.88	ton						
M		V x (X - (0.5 x Ws))		(H x (Y - tb))					
		0.5 x Ws			0.500 x 6.01				
					3.01 m				
		(X - (0.5 x Ws))			2.633 - 3.01				
					(0.37) m				
		(Y - tb)			3.319 - 1.00				
					2.32 m				
		H x (Y - tb)			362.88 x 2.32				
					841.65 m				
		V x (X - (0.5 x Ws))		(H x (Y - tb))					
		2,814.28 x (0.37)			841.65				
		(1,052.74) - 841.65							
		(1,894.39) t.m							

cld-an

II.6.4 Earthquake condition
Case - IV, Low Water Level

Item	Vo/m'	Ho/m'	Xo	Yo	Vo.Xo	Ho.Yo
Super structure	44.761	6.140	2.100	6.955	94.040	42.704
Abutment	39.829	7.169	2.557	2.730	101.843	19.571
Back fill	48.209	8.678	2.704	3.151	130.357	27.344
Active earth pressure						
Surcharge load		1.828		4.000		6.512
Wet soil triangle load		16.335		3.609		58.953
Sat. soil rectangular load		1.150		0.707		0.813
triangle load		0.813		0.471		0.383
Water		(0.998)		0.471		(0.470)
Pw in		1.467		0.571		0.838
Pu	(7.206)			2.55		0.000
Up life						
Total	125.613	42.382			326.240	*****
X' =	Vo.Xo		Y'	=	Ho.Yo	
	-----			=	-----	
	Vo tot			=	Ho tot	
	326.240			=	156.648	
	-----			=	-----	
	125.613			=	42.382	
V =	2.597 m			=	3.696 m	
V =	V/m' x Ls					
=	125.61 x 22.40					
=	2,814.28 ton					
H =	H/m' x Ls					
=	42.38 x 22.40					
=	949.54 ton					
M =	V x (X - (0.5 x W)) - (H x (Y - tb))					
0.5 x W	= 0.500 x 5.10					
	= 2.55 m					
(X - (0.5 x W))	= 2.597 - 2.55					
	= 0.05 m					
(Y - tb)	= 3.696 - 1.00					
	= 2.70 m					
H x (Y - tb)	= 949.54 x 2.70					
	= 2,560.06 m					
=	V x (X - (0.5 x W)) - (H x (Y - tb))					
=	2,814.28 x 0.05 - 2,560.06					
=	132.79 - 2,560.06					
=	(2,427.27) t.m					

**PRESTRESSED CONCRETE BRIDGE COMPUTATION
CHANNEL PERCUT RIVER**

Br – P 9

Station : Pe 200 + 25

span
40.00

III FOUNDATION

Pile for Abutment

- III.1 Charateristic of pile
 - III.1.1 Normal condition
 - III.1.2 Earthquake condition
- III.2 Axial bearing load allowance for a pile
- III.3 Spring constant of a pile
 - III.3.1 Normal condition
 - III.3.2 Earthquake condition
- III.4 Displacement of a head pile
 - III.4.1 Normal condition
 - III.4.2 Earthquake condition
- III.5 Coeficient displacement of footing
 - III.5.1 Case – I,Normal condition (HWL)
 - III.5.2 Case – II,Normal condition (LWL)
 - III.5.3 Case – III,Earthquake condition (HWL)
 - III.5.4 Case – IV,Earthquake condition (LWL)
- III.6 Forces at the i–th pile head
 - III.6.1 Case – I,Normal condition (HWL)
 - III.6.2 Case – II,Normal condition (LWL)
 - III.6.3 Case – III,Earthquake condition (HWL)
 - III.6.4 Case – IV,Earthquake condition (LWL)
- III.7 Control stresses on a head pile
 - III.7.1 Normal condition
 - III.7.2 Earthquake condition
- III.8 Combined load at Abutment section
 - III.8.1 Case – I,Normal condition (HWL)
 - III.8.2 Case – II,Normal condition (LWL)
 - III.8.3 Case – III,Earthquake condition (HWL)
 - III.8.4 Case – IV,Earthquake condition (LWL)
- III.9 Resume Load on Abutment Section
- III.10 Reinforcing steel bar

char - a

Char-a-

Project : Medan Flood Control Channel : Perak River Structure : Br. P - 9 Station : PG 200 + 25 m	Japan International Cooperation Agency (JICA)	Date Nos of sheet Prepared by	Calculation sheet
III.1 File of Left side Abutment			
III.1.2 Characteristic of steel pipe pile Earthquake condition			
t' = thick of steel pipe corrosion.			
d _o = outer diameter of steel pipe			
t = thick of steel pipe after corrosion			
d _i = Inner diameter of steel pipe			
Beta = characteristic value of pile			
d = outside diameter of pile before corrosion			
d _e = outside diameter of pile after corrosion			
delta = the standard amount of displacement (cm)			
k = coefficient of horizontal subgrade reaction (kg/cm ²)			
E _o = modulus of deformation of the ground obtained from table 3.5			
I _c = Inertia concrete			
I _s = Inertia steel			
Modulus elasticity			
E _s = pile/steel			
E _c = concrete			
thick of corrosion = t'			
normal thick = t _e			
d _e effective			
d _i			
d _e ^ 4			
d _i ^ 4			
d _e ^ 4 - d _i ^ 4			
Beta			
k _{ho}			
Alpha			
E _o			
Temporary load			
E _c (K-225)			
E _s			
I/n			
I _{sc}			
I _{ss}			

Project : Median Flood Control	Japan International Cooperation Agency (JICA)	Calculation sheet
Channel : Percut River		Date
Structure : Br. P 9		Nos of sheet
Station : PE 200 + 25		Prepared by

III.4 Coefficient deflection of footing

III.4.1 Normal condition

$$Axx = \text{tot}(K1 \times \cos^2 \theta + Kv \times \sin^2 \theta)$$

$$Ayy = \text{tot}(Kv - K1) \times \sin \theta \times \cos \theta$$

$$Ayx = \text{tot}(Kv - K1) \times \sin \theta \times \cos \theta$$

$$Aza = \text{tot}(Kv - K1) \times Xi \times (\sin \theta \times \cos \theta) - (K2 \times \cos \theta)$$

$$Aax = \text{tot}(Kv - K1) \times Xi \times (\sin \theta \times \cos \theta) - (K2 \times \cos \theta)$$

$$Ayy = \text{tot}(Kv \times \cos^2 \theta + K1 \times \sin^2 \theta)$$

$$Aya = \text{tot}(Kv \times \cos^2 \theta + K1 \times \sin^2 \theta) \times Xi + K2 \times \sin \theta$$

$$Aay = \text{tot}(Kv \times \cos^2 \theta + K1 \times \sin^2 \theta) \times Xi + K2 \times \sin \theta$$

$$Aaa = \text{tot}(Kv \times \cos^2 \theta + K1 \times \sin^2 \theta) \times Xi^2 + (K2 + K3) \times Xi \times \sin \theta + K4$$

Nos. of row	Rows				Total
	(1)	(2)	(3)	(4)	
side	L/a	L/a	R/a	R/a	
sign	(-)	(-)	(+)	(+)	
angle	> 0	0	< 0	< 0	
Xi	(1.875)	(0.625)	0.625	1.875	
n	16.00	16.00	16.00	16.00	
angle degree	0.0000	0.0000	0.0000	0.0000	5.71
sin θ	0.0000	0.00	0.00	0.10	0.9950
cos θ	1.0000	1.0000	1.0000	1.0000	0.9950
sin θ × cos θ	0.0000	0.00	0.00	0.00	0.10
sin ² θ	0.0000	0.00	0.00	0.00	0.01
cos ² θ	1.0000	1.0000	1.0000	1.0000	0.9901
Xi ²	3.5156	0.3906	0.3906	3.5156	
K1	1.914	1.914	1.914	1.914	
K2	1.902	1.902	1.902	1.902	
K3	1.902	1.902	1.902	1.902	
K4	3.782	3.782	3.782	3.782	
Kv	19.009	19.009	19.009	19.009	
n.K1	30.624	30.624	30.624	30.624	
n.K2	30.432	30.432	30.432	30.432	
n.K3	30.432	30.432	30.432	30.432	
n.K4	60.512	60.512	60.512	60.512	
n.Kv	304.144	304.144	304.144	304.144	
n.(K2 + K3)	60.864	60.864	60.864	60.864	
Xi.(n.(K2 + K3))	(114.120)	(38.040)	38.040	114.120	
Xi.(n.(K2 + K3)).sin θ	0	0	0	11.354	
n.(Kv - K1)	273.520	273.520	273.520	273.520	
n.(Kv - K1).sin θ.cos θ	0	0	0	27.078	
n.K1.cos ² θ	30.624	30.624	30.624	30.624	
n.Kv.sin ² θ	0	0	0	0	
n.Kv.cos ² θ	304.144	304.144	304.144	304.144	
n.K1.sin ² θ	0	0	0	0	
n.K1.sin θ.cos θ	0	0	0	0	
n.Kv.sin θ.cos θ	0	0	0	0	
Xi.(n.K1.sin θ.cos θ)	0	0	0	0	
Xi.(n.Kv.sin θ.cos θ)	0	0	0	0	

cos = -disn

As1	=	1/4 x 3.14 x di ² x	3.14 x	56.52
	=	0.25 x		
As2	=	1/4 x 3.14 x (di ² - di ²) x	3.14 x	140.51
	=	0.25 x		
A tot.	=	110.35 cm ²		
	=	As1 + As2		
	=	44.39 + 110.35		
	=	154.75 cm ²		
	=	0.01547 m ²		
Lp	=	27.00 m		

Kv	=	A tot.	x	Ep
	=	Lp		

a	=	0.027 x	de	0.2
	=	2.700		
	=	38.78		
	=	1.87963 + 0.2000		
	=	2.0796		

Spring constants on axial force = Kv

Kv	=	A tot.	x	Ep
	=	Lp		

EI	=	154.75 x 1.897.200		
	=	2.0796 x		
	=	2.08 x 108.735		
	=	226.128.68 kg/cm		
	=	22.613 ton/m		
EI	=	1.897.200 x 19.937		
	=	3.78E+10 kg.cm ²		
	=	3.783 km ²		
2EI	=	2 x 3.78E+03		
	=	7.565 km ²		
4EI	=	4 x 3.783		
	=	15.130 km ²		
Beta	=	0.00477 cm ⁻¹		
Beta	=	0.47706 m ⁻¹		

Spring constants K1, K2 and K3

K1	=	4.EI.Beta ³		
	=	1.51E+04 x 0.108576		
	=	1.643 tm		
K2=K3	=	2.EI.Beta ²		
	=	7.565 x 0.227591		
	=	1.722 t/m		
K4	=	2.EI.Beta		
	=	7.565.00 x 0.477066		
	=	3.609 tm/m		

kg-q

n.K2.cos θ	=	30.432	30.432	30.432	30.281
n.K2.sin θ	=	0	0	0	3.028
n.K3.sin θ	=	0	0	0	3.028
Xi(n.Kv.cos $\sim 2\theta$)	=	(570.270)	(190.090)	190.090	564.625
Xi(n.K1.sin $\sim 2\theta$)	=	0	0	0	568
Xi ~ 2 (n.Kv.cos $\sim 2\theta$)	=	1,069.256	118.806	118.806	1,058.672
Xi ~ 2 (n.K1.sin $\sim 2\theta$)	=	0	0	0	1,066
Xi(n.K2.sin θ)	=	0	0	0	5,677
Xi(n.K3.sin θ)	=	0	0	0	0
Axx = tot(K1 x cos $\sim 2\theta$ + Kv x sin $\sim 2\theta$)					
n.K1.cos $\sim 2\theta$	=	30.624	30.624	30.624	30.321
n.Kv.sin $\sim 2\theta$	=	0	0	0	3,011
total		30.624	30.624	30.624	33.332
Ayy = tot(Kv - K1) x sin θ cos θ					
n.(Kv - K1).sin θ cos θ	=	0	0	0	27,078
Axy	=	0	0	0	27,078
Ayz	=	0	0	0	27,078
Axx = tot(KV - K1) x Xi x (sin θ x cos θ) - (K2 x cos θ)					
- n.K2.cos θ	=	(30.432)	(30.432)	(30.432)	(30.281)
Axx	=	(30.432)	(30.432)	(30.432)	(30.432)
Axx	=	(30.432)	(30.432)	(30.432)	(30.432)
Ayy = tot(KV x cos $\sim 2\theta$ + (K1 x sin $\sim 2\theta$))					
n.Kv.cos $\sim 2\theta$	=	304.144	304.144	304.144	301.133
n.K1.sin $\sim 2\theta$	=	0	0	0	303
Ayy	=	304.144	304.144	304.144	301.436
Ayy	=	304.144	304.144	304.144	301.436
Ayy = tot(KV x cos $\sim 2\theta$ + K1 x sin $\sim 2\theta$) x Xi + K2 x sin θ					
Xi(n.Kv.cos $\sim 2\theta$)	=	(570.270)	(190.090)	190.090	564.625
Xi(n.K1.sin $\sim 2\theta$)	=	0	0	0	568
n.K2.sin θ	=	0	0	0	3,028
Ayy	=	(570.270)	(190.090)	190.090	568.221
Ayy	=	(570.270)	(190.090)	190.090	568.221
Ayy	=	(570.270)	(190.090)	190.090	568.221
Axx = tot(KV x cos $\sim 2\theta$ + K1 x sin $\sim 2\theta$) x Xi ~ 2 + (K2 + K3) Xi sin θ + K4					
Xi ~ 2 (n.Kv.cos $\sim 2\theta$)	=	1,069.256	118.806	118.806	1,058.672
Xi ~ 2 (n.K1.sin $\sim 2\theta$)	=	0	0	0	1,066
n.(K2 + K3) Xi sin θ	=	0	0	0	11,354
n.K4	=	60,512	60,512	60,512	60,512
Axx	=	1,129,768	179,218	179,218	1,131,604
Axx	=	1,129,768	179,218	179,218	1,131,604

coe - disq

Project : Medan Flood Control	Japan International Cooperation Agency (JICA)	Calculation sheet
Channel : Perut River		Date
Structure : Br. P - 9		Nos of sheet
Station : PE 200 + 25 m		Prepared by

III.4.2 Earthquake condition

$$\begin{aligned}
 Axx &= \text{tot}(K1 \times \cos \sim 2 \theta + Kv \times \sin \sim 2 \theta) \\
 Ayy &= \text{tot}(Kv - K1) \times \sin \theta \times \cos \theta \\
 Ayx &= \text{tot}(Kv - K1) \times \sin \theta \times \cos \theta \\
 Axx &= \text{tot}(Kv - K1) \times Xi \times (\sin \theta \times \cos \theta) - (K2 \times \cos \theta) \\
 Axx &= \text{tot}(Kv - K1) \times Xi \times (\sin \theta \times \cos \theta) - (K2 \times \cos \theta) \\
 Ayy &= \text{tot}(Kv \times \cos \sim 2 \theta + (K1 \times \sin \sim 2 \theta)) \\
 Ayy &= \text{tot}(Kv \times \cos \sim 2 \theta + K1 \times \sin \sim 2 \theta) \times Xi + K2 \times \sin \theta \\
 Ayy &= \text{tot}(Kv \times \cos \sim 2 \theta + K1 \times \sin \sim 2 \theta) \times Xi + K2 \times \sin \theta \\
 Axx &= \text{tot}(Kv \times \cos \sim 2 \theta + K1 \times \sin \sim 2 \theta) \times Xi \sim 2 + (K2 + K3) \times Xi \times \sin \theta + K4
 \end{aligned}$$

Total				
Rows				
Nos. of row	(1)	(2)	(3)	(4)
side	L/s	L/s	R/s	R/s
sign	(-)	(-)	(+)	(+)
angle	> 0	0	< 0	< 0
Xi	= (1.875)	(0.625)	0.625	1.875
n	= 16.00	16.00	16.00	16.00
angle degree	= 0.0000	0.0000	0.0000	5.71
sin θ	= 0.0000	0.00	0.00	0.10
cos θ	= 1.0000	1.0000	1.0000	0.9950
sin θ x cos θ	= 0.0000	0.00	0.00	0.10
sin $\sim 2 \theta$	= 0.0000	0.00	0.00	0.01
cos $\sim 2 \theta$	= 1.0000	1.0000	1.0000	0.9901
Xi ~ 2	= 3.5156	0.3906	0.3906	3.5156
K1	= 1,643.00	1,643.00	1,643.00	1,643.00
K2	= 1,722.00	1,722.00	1,722.00	1,722.00
K3	= 1,722.00	1,722.00	1,722.00	1,722.00
K4	= 3,609.00	3,609.00	3,609.00	3,609.00
Kv	= 22,613.00	22,613.00	22,613.00	22,613.00
n.K1	= 26,288	26,288	26,288	26,288
n.K2	= 27,552	27,552	27,552	27,552
n.K3	= 27,552	27,552	27,552	27,552
n.K4	= 57,744	57,744	57,744	57,744
n.Kv	= 361,808	361,808	361,808	361,808
n.(K2 + K3)	= 55,104	55,104	55,104	55,104
Xi(n.(K2 + K3))	= (103,320)	(34,440)	34,440	103,320
Xi(n.(K2 + K3))sin θ	= 0	0	0	10,280
n.(Kv - K1)	= 335,520	335,520	335,520	335,520
n.(Kv - K1)sin θ cos θ	= 0	0	0	33,216
n.K1.cos $\sim 2 \theta$	= 26,288	26,288	26,288	26,028
n.Kv.sin $\sim 2 \theta$	= 0	0	0	3,582
n.Kv.cos $\sim 2 \theta$	= 361,808	361,808	361,808	358,226
n.K1.sin $\sim 2 \theta$	= 0	0	0	260
n.K1.sin θ cos θ	= 0	0	0	2,603
n.Kv.sin θ cos θ	= 0	0	0	35,819
Xi(n.K1 sin θ cos θ)	= 0	0	0	4,880
Xi(n.Kv sin θ cos θ)	= 0	0	0	67,160
n.K2.cos θ	= 27,552	27,552	27,552	27,415
n.K2.sin θ	= 0	0	0	2,741
n.K3.sin θ	= 0	0	0	2,741
Xi(n.Kv.cos $\sim 2 \theta$)	= (678,390)	(226,130)	226,130	671,675
Xi(n.K1 sin $\sim 2 \theta$)	= 0	0	0	488

coe - disq

Project	Median Flood Control	Japan International Cooperation Agency (JICA)	Calculation sheet
Channel	Perot River		Date
Structure	Br. P - B		No. of sheet
Station	PE 200 + 25 m		Prepared by

M.S. Displacement of head pile										
M.S.1 Internal condition										
Case - 1 High Water Level										
	dx	dy	dz	dx	dy	dz	dx	dy	dz	
125204	+	1213800	+	27078	dy	+	170.800	dz	+	378.08
27078	dx	+	1213800	dy	+	2.049	dz	+	2.594	03
170.800	dx	+	2.049	dy	+	2.050	dz	+	2.053	74
2.050	dx	+	2.053	dy	+	2.056	dz	+	2.059	03
2.056	dx	+	2.059	dy	+	2.062	dz	+	2.065	03
2.062	dx	+	2.065	dy	+	2.068	dz	+	2.071	03
2.068	dx	+	2.071	dy	+	2.074	dz	+	2.077	03
2.074	dx	+	2.077	dy	+	2.080	dz	+	2.083	03
2.080	dx	+	2.083	dy	+	2.086	dz	+	2.089	03
2.086	dx	+	2.089	dy	+	2.092	dz	+	2.095	03
2.092	dx	+	2.095	dy	+	2.098	dz	+	2.101	03
2.098	dx	+	2.101	dy	+	2.104	dz	+	2.107	03
2.104	dx	+	2.107	dy	+	2.110	dz	+	2.113	03
2.110	dx	+	2.113	dy	+	2.116	dz	+	2.119	03
2.116	dx	+	2.119	dy	+	2.122	dz	+	2.125	03
2.122	dx	+	2.125	dy	+	2.128	dz	+	2.131	03
2.128	dx	+	2.131	dy	+	2.134	dz	+	2.137	03
2.134	dx	+	2.137	dy	+	2.140	dz	+	2.143	03
2.140	dx	+	2.143	dy	+	2.146	dz	+	2.149	03
2.146	dx	+	2.149	dy	+	2.152	dz	+	2.155	03
2.152	dx	+	2.155	dy	+	2.158	dz	+	2.161	03
2.158	dx	+	2.161	dy	+	2.164	dz	+	2.167	03
2.164	dx	+	2.167	dy	+	2.170	dz	+	2.173	03
2.170	dx	+	2.173	dy	+	2.176	dz	+	2.179	03
2.176	dx	+	2.179	dy	+	2.182	dz	+	2.185	03
2.182	dx	+	2.185	dy	+	2.188	dz	+	2.191	03
2.188	dx	+	2.191	dy	+	2.194	dz	+	2.197	03
2.194	dx	+	2.197	dy	+	2.200	dz	+	2.203	03
2.200	dx	+	2.203	dy	+	2.206	dz	+	2.209	03
2.206	dx	+	2.209	dy	+	2.212	dz	+	2.215	03
2.212	dx	+	2.215	dy	+	2.218	dz	+	2.221	03
2.218	dx	+	2.221	dy	+	2.224	dz	+	2.227	03
2.224	dx	+	2.227	dy	+	2.230	dz	+	2.233	03
2.230	dx	+	2.233	dy	+	2.236	dz	+	2.239	03
2.236	dx	+	2.239	dy	+	2.242	dz	+	2.245	03
2.242	dx	+	2.245	dy	+	2.248	dz	+	2.251	03
2.248	dx	+	2.251	dy	+	2.254	dz	+	2.257	03
2.254	dx	+	2.257	dy	+	2.260	dz	+	2.263	03
2.260	dx	+	2.263	dy	+	2.266	dz	+	2.269	03
2.266	dx	+	2.269	dy	+	2.272	dz	+	2.275	03
2.272	dx	+	2.275	dy	+	2.278	dz	+	2.281	03
2.278	dx	+	2.281	dy	+	2.284	dz	+	2.287	03
2.284	dx	+	2.287	dy	+	2.290	dz	+	2.293	03
2.290	dx	+	2.293	dy	+	2.296	dz	+	2.299	03
2.296	dx	+	2.299	dy	+	2.302	dz	+	2.305	03
2.302	dx	+	2.305	dy	+	2.308	dz	+	2.311	03
2.308	dx	+	2.311	dy	+	2.314	dz	+	2.317	03
2.314	dx	+	2.317	dy	+	2.320	dz	+	2.323	03
2.320	dx	+	2.323	dy	+	2.326	dz	+	2.329	03
2.326	dx	+	2.329	dy	+	2.332	dz	+	2.335	03
2.332	dx	+	2.335	dy	+	2.338	dz	+	2.341	03
2.338	dx	+	2.341	dy	+	2.344	dz	+	2.347	03
2.344	dx	+	2.347	dy	+	2.350	dz	+	2.353	03
2.350	dx	+	2.353	dy	+	2.356	dz	+	2.359	03
2.356	dx	+	2.359	dy	+	2.362	dz	+	2.365	03
2.362	dx	+	2.365	dy	+	2.368	dz	+	2.371	03
2.368	dx	+	2.371	dy	+	2.374	dz	+	2.377	03
2.374	dx	+	2.377	dy	+	2.380	dz	+	2.383	03
2.380	dx	+	2.383	dy	+	2.386	dz	+	2.389	03
2.386	dx	+	2.389	dy	+	2.392	dz	+	2.395	03
2.392	dx	+	2.395	dy	+	2.398	dz	+	2.401	03
2.398	dx	+	2.401	dy	+	2.404	dz	+	2.407	03
2.404	dx	+	2.407	dy	+	2.410	dz	+	2.413	03
2.410	dx	+	2.413	dy	+	2.416	dz	+	2.419	03
2.416	dx	+	2.419	dy	+	2.422	dz	+	2.425	03
2.422	dx	+	2.425	dy	+	2.428	dz	+	2.431	03
2.428	dx	+	2.431	dy	+	2.434	dz	+	2.437	03
2.434	dx	+	2.437	dy	+	2.440	dz	+	2.443	03
2.440	dx	+	2.443	dy	+	2.446	dz	+	2.449	03
2.446	dx	+	2.449	dy	+	2.452	dz	+	2.455	03
2.452	dx	+	2.455	dy	+	2.458	dz	+	2.461	03
2.458	dx	+	2.461	dy	+	2.464	dz	+	2.467	03
2.464	dx	+	2.467	dy	+	2.470	dz	+	2.473	03
2.470	dx	+	2.473	dy	+	2.476	dz	+	2.479	03
2.476	dx	+	2.479	dy	+	2.482	dz	+	2.485	03
2.482	dx	+	2.485	dy	+	2.488	dz	+	2.491	03
2.488	dx	+	2.491	dy	+	2.494	dz	+	2.497	03
2.494	dx	+	2.497	dy	+	2.500	dz	+	2.503	03
2.500	dx	+	2.503	dy	+	2.506	dz	+	2.509	03
2.506	dx	+	2.509	dy	+	2.512	dz	+	2.515	03
2.512	dx	+	2.515	dy	+	2.518	dz	+	2.521	03
2.518	dx	+	2.521	dy	+	2.524	dz	+	2.527	03
2.524	dx	+	2.527	dy	+	2.530	dz	+	2.533	03
2.530	dx	+	2.533	dy	+	2.536	dz	+	2.539	03
2.536	dx	+	2.539	dy	+	2.542	dz	+	2.545	03
2.542	dx	+	2.545	dy	+	2.548	dz	+	2.551	03
2.548	dx	+	2.551	dy	+	2.554	dz	+	2.557	03
2.554	dx	+	2.557	dy	+	2.560	dz	+	2.563	03
2.560	dx	+	2.563	dy	+	2.566	dz	+	2.569	03
2.566	dx	+	2.569	dy	+	2.572	dz	+	2.575	03
2.572	dx	+	2.575	dy	+	2.578	dz	+	2.581	03
2.578	dx	+	2.581	dy	+	2.584	dz	+	2.587	03
2.584	dx	+	2.587	dy	+	2.590	dz	+	2.593	03
2.590	dx	+	2.593	dy	+	2.596	dz	+	2.599	03
2.596	dx	+	2.599	dy	+	2.602	dz	+	2.605	03
2.602	dx	+	2.605	dy	+	2.608	dz	+	2.611	03
2.608	dx	+	2.611	dy	+	2.614	dz	+	2.617	03
2.614	dx	+	2.617	dy	+	2.620	dz	+	2.623	03
2.620	dx	+	2.623	dy	+	2.626	dz	+	2.629	03
2.626	dx	+	2.629	dy	+	2.632	dz	+	2.635	03
2.632	dx	+	2.635	dy	+	2.638	dz	+	2.641	03
2.638	dx	+	2.641	dy	+	2.644	dz	+	2.647	03
2.644	dx	+	2.647	dy	+	2.650	dz	+	2.653	03
2.650	dx	+	2.653	dy	+	2.656	dz	+	2.659	03
2.656	dx	+	2.659	dy	+	2.662	dz	+	2.665	03
2.662	dx	+	2.665	dy	+	2.668	dz	+	2.671	03
2.668	dx	+	2.671	dy	+	2.674	dz	+	2.677	03
2.674	dx	+	2.677	dy	+	2.680	dz	+	2.683	03
2.680	dx	+	2.683	dy	+	2.686	dz	+	2.689	03
2.686	dx	+	2.689	dy	+	2.692	dz	+	2.695	03
2.692	dx	+	2.695	dy	+	2.698	dz	+	2.701	03
2.698	dx	+	2.701	dy	+	2.704	dz	+	2.707	03
2.704	dx	+	2.707	dy	+	2.710	dz	+	2.713	03
2.710	dx	+	2.713	dy	+	2.716	dz	+	2.719	03
2.716	dx	+	2.719	dy	+	2.722	dz	+	2.725	03
2.722	dx	+	2.725	dy	+	2.728	dz	+	2.731	03
2.728	dx	+	2.731	dy	+	2.734	dz	+	2.737	03
2.734	dx	+	2.737	dy	+	2.740	dz	+	2.743	03
2.740	dx	+	2.743	dy	+	2.746	dz	+	2.749	03
2.746	dx	+	2.749	dy	+	2.752	dz	+	2.755	03
2.752	dx	+	2.755	dy	+	2.758	dz	+	2.761	03
2.758	dx	+	2.761	dy	+	2.764	dz	+	2.767	03
2.764	dx	+	2.767	dy	+	2.770	dz	+	2.773	03
2.770	dx	+	2.773	dy	+	2.776	dz	+	2.779	03
2.776	dx	+	2.779	dy	+	2.782	dz	+	2.785	03
2.782	dx	+	2.785	dy	+	2.788	dz	+	2.791	03
2.788	dx	+	2.791	dy	+	2.794	dz	+	2.797	03
2.794	dx	+	2.797	dy	+	2.800	dz	+	2.803	03
2.800	dx	+	2.803	dy	+	2.806	dz	+	2.809	03
2.806	dx	+	2.809	dy	+	2.812	dz	+	2.815	03
2.812	dx	+	2.815	dy	+	2.818	dz	+	2.821	03
2.818	dx	+	2.821	dy	+	2.824	dz	+	2.827	03
2.824	dx	+	2.827	dy	+	2.830	dz	+	2.833	03
2.830	dx	+	2.833	dy	+	2.836	dz	+	2.839	03
2.836	dx	+	2.839	dy	+	2.842	dz	+	2.845	03
2.842	dx	+	2.845	dy	+	2.848	dz	+	2.851	03
2.848	dx	+	2.851	dy	+	2.854	dz	+	2.857	03

[illegible]

23-100

manh-1

b-6a

b-6a

total area of steel bar		P		16.08		3.14 x 400.00	
@pi.ro^2		16.08		3.14 x 400.00		16.08	
1.256.64		1.256.64		0.01280		7.37 x 0.01280	
1.54810		1.54810		18.97		20.00	
0.95		0.95		9.53 cm		9.53	
20.00		20.00		0.4765		0.4765	
pi/4 - (5/12 - 1/6 cos^2 pi) sin pi cos pi + (n.@pi.p/2)(nitro)^2		pi/4 - (5/12 - 1/6 cos^2 pi) sin pi cos pi + (n.@pi.p/2)(nitro)^2		pi/4 - (5/12 - 1/6 cos^2 pi) sin pi cos pi + (n.@pi.p/2)(nitro)^2		pi/4 - (5/12 - 1/6 cos^2 pi) sin pi cos pi + (n.@pi.p/2)(nitro)^2	
sin pi/2 (2 + cos^2 pi) - pi cos pi - n.@pi.p cos pi		sin pi/2 (2 + cos^2 pi) - pi cos pi - n.@pi.p cos pi		sin pi/2 (2 + cos^2 pi) - pi cos pi - n.@pi.p cos pi		sin pi/2 (2 + cos^2 pi) - pi cos pi - n.@pi.p cos pi	
trial value of pi ----->>>>		pi = 142.8261 degree		pi = 2.4928 rad.		pi = 142.8261 degree	
s = pi/4 - (5/12 - 1/6 cos^2 pi) sin pi cos pi + (n.@pi.p/2)(nitro)^2		s = pi/4 - (5/12 - 1/6 cos^2 pi) sin pi cos pi + (n.@pi.p/2)(nitro)^2		s = pi/4 - (5/12 - 1/6 cos^2 pi) sin pi cos pi + (n.@pi.p/2)(nitro)^2		s = pi/4 - (5/12 - 1/6 cos^2 pi) sin pi cos pi + (n.@pi.p/2)(nitro)^2	
pi/4 = A		pi/4 = A		pi/4 = A		pi/4 = A	
n		n		n		n	
alpha		alpha		alpha		alpha	
cos pi		cos pi		cos pi		cos pi	
sin pi		sin pi		sin pi		sin pi	
cos^2 pi		cos^2 pi		cos^2 pi		cos^2 pi	
1/6 cos^2 pi		1/6 cos^2 pi		1/6 cos^2 pi		1/6 cos^2 pi	
(5/12 - 1/6 cos^2 pi)		(5/12 - 1/6 cos^2 pi)		(5/12 - 1/6 cos^2 pi)		(5/12 - 1/6 cos^2 pi)	
sin pi cos pi		sin pi cos pi		sin pi cos pi		sin pi cos pi	
(5/12 - 1/6 cos^2 pi) x sin pi cos pi = B		(5/12 - 1/6 cos^2 pi) x sin pi cos pi = B		(5/12 - 1/6 cos^2 pi) x sin pi cos pi = B		(5/12 - 1/6 cos^2 pi) x sin pi cos pi = B	
(n.@pi.p)		(n.@pi.p)		(n.@pi.p)		(n.@pi.p)	
(n.@pi.p)/2		(n.@pi.p)/2		(n.@pi.p)/2		(n.@pi.p)/2	
(n.@pi.p)/2 x (nitro)^2 = C		(n.@pi.p)/2 x (nitro)^2 = C		(n.@pi.p)/2 x (nitro)^2 = C		(n.@pi.p)/2 x (nitro)^2 = C	
sum A - B + C		sum A - B + C		sum A - B + C		sum A - B + C	
ro = sin pi/2 (2 + cos^2 pi) - pi cos pi - n.@pi.p cos pi		ro = sin pi/2 (2 + cos^2 pi) - pi cos pi - n.@pi.p cos pi		ro = sin pi/2 (2 + cos^2 pi) - pi cos pi - n.@pi.p cos pi		ro = sin pi/2 (2 + cos^2 pi) - pi cos pi - n.@pi.p cos pi	
(2 + cos^2 pi)		(2 + cos^2 pi)		(2 + cos^2 pi)		(2 + cos^2 pi)	
sin pi/2 (2 + cos^2 pi)		sin pi/2 (2 + cos^2 pi)		sin pi/2 (2 + cos^2 pi)		sin pi/2 (2 + cos^2 pi)	
n.@pi.p		n.@pi.p		n.@pi.p		n.@pi.p	
n.@pi.p cos pi		n.@pi.p cos pi		n.@pi.p cos pi		n.@pi.p cos pi	
e		e		e		e	
ro		ro		ro		ro	
e		e		e		e	
ro		ro		ro		ro	
pi		pi		pi		pi	
e		e		e		e	
ro		ro		ro		ro	

leg-q

C =		2 sin^3 pi/3 - pi cos pi + sin pi cos^2 pi + pi/4 - sin pi cos pi/4 - sin^3 pi cos pi/6 @pi.ap (alpha^2/2 - cos pi)		1 - cos pi	
Cos pi		Cos pi		(0.797)	
sin pi		sin pi		0.604	
sin^3 pi		sin^3 pi		0.221	
sin pi cos pi		sin pi cos pi		0.604 x	
Cos^2 pi		Cos^2 pi		0.635	
1 - cos pi		1 - cos pi		1.000 -	
2 sin^3 pi/3		2 sin^3 pi/3 = A		(0.797) =	
2 sin^3 pi/3		2 sin^3 pi/3 = A		0.221 =	
pi cos pi = B		pi cos pi = B		3.000	
sin pi cos^2 pi = C		sin pi cos^2 pi = C		2.493 x	
pi/4 = D		pi/4 = D		0.604 x	
Cos pi/4		Cos pi/4		0.623	
sin pi cos pi/4 = E		sin pi cos pi/4 = E		(0.797) :	
(sin^3 pi cos pi)		(sin^3 pi cos pi)		0.604 x	
(sin^3 pi cos pi)/6 = F		(sin^3 pi cos pi)/6 = F		0.221 x	
n.p		n.p		(0.176) :	
@pi.ap		@pi.ap		6.000	
alpha		alpha		1.548	
alpha^2		alpha^2		3.142 x	
0.900		0.900		1.548 =	
alpha^2/2		alpha^2/2		2.000 =	
alpha^2/2 - cos pi		alpha^2/2 - cos pi		(0.797) =	
@pi.ap x alpha^2/2		@pi.ap x alpha^2/2		4.864 x	
@pi.ap x alpha^2/2 - cos pi = G		@pi.ap x alpha^2/2 - cos pi = G		2.188 =	
sum		sum		2.188 -	
A - B + C + D - E - F + G		A - B + C + D - E - F + G		(0.797)	
2.985		2.985		6.274	
C =		2 sin^3 pi/3 - pi cos pi + sin pi cos^2 pi + pi/4 - sin pi cos pi/4 - sin^3 pi cos pi/6 @pi.ap (alpha^2/2 - cos pi)		1 - cos pi	
C		C		1.796805	
S		S		6.274	
alpha + cos pi		alpha + cos pi		0.28637	
1 - cos pi		1 - cos pi		0.948 +	
0.286 x		0.286 x		(0.797)	
0.286 x		0.286 x		1.797	
0.024		0.024		0.084	
1.976.800		1.976.800		kg cm	
20.000		20.000		cm	
M'		M'		C	
ro		ro		ro^3	
1.976.800		1.976.800		0.266	
8.000		8.000		0.256	
247.100 x		247.100 x		kg/cm2	
70.76		70.76		170 kg/cm2	
stress on steel = fs		stress on steel = fs		stress on steel = fs	
fs		fs		S	
ro^3		ro^3		1.976.800	
8.000		8.000		0.024 x	
247.100 x		247.100 x		7 kg/cm2	
44 kg/cm2		44 kg/cm2		7 kg/cm2	

leg-q

Project : Medan Flood Control	Japan International Cooperation Agency (JICA)	Calculation sheet
Channel : Perak River		Date
Structure : Br. P 9		Nos of sheet
Station : PB200 + 25		Prepared by
III.7 Control stresses on pile		
III.7.2 Earthquake Condition		
Case - III Low Water Level		
Depth embedded of pile	V _i	27.00 m
Axial load	M _i	87,530 kg
Moment	M _i	18,440 kg.cm
diameter	d _i	40.00 cm
outer radius	r _i	20.00 cm
	M _i	M _x + V _i x r _i
		1,844,000 + 87,530 x 20.00
		1,844,000 + 1,750,600
		3,594,600 kg.cm
eccentricitas	e	M _x
		V _i
		1,844,000 kg.cm
		87,530 kg
		21.07 cm
concrete covering	a'	2.50 cm
	l	1.03 cm
Inner diameter	d _i	40.00 - 2.06
Inner radius	r _i	37.94
		18.97 cm
r _i ²		400.00
r _i ³		359.86
r _i ² -r _i ²		400.00 - 359.86
r _i ² -r _i ²		40.14 cm ²
area of steel bar	A _s	8.00 x 20.06 = 16.08 cm ²
	A _s	16.08 cm ²
Area of bar. Y bar	Y _s	16.08
Area of bar		305.13
		16.08
		1,897.200
		257.495
		7.37

P	tot. A rea of steel bar @pi.ro ²	16.08	3.14 x 400.00
		16.08	1.256.64
P		0.01280	
a.p		7.37 x 0.01280	
		1.54810	
alpha	ri	18.97	0.95
	ro	20.00	
		21.07 cm	
e		21.07	1.0534
ro		20.00	
e	pi/4 - (5/12 - 1/6 cos ² 2 pi) sin pi cos pi + (n.@pi.p/2)(ri/ro) ²		
ro	sin pi/2 (2 + cos ² 2 pi) - pi cos pi - a.@pi.p cos pi		
pi	pi = 108.0001 degree		
pi	pi = 1.8850 rad.		
trial value of pi ->>>>>			
a	pi/4 - (5/12 - 1/6 cos ² 2 pi) sin pi cos pi + (n.@pi.p/2)(ri/ro) ²		
pi/4 = A		1.8850	4.00 = 0.47
n		7.37	
alpha		0.95 cm	
alpha ²		0.90 cm ²	
cos pi		(0.309019)	
sin pi		0.951056	
cos ² 2 pi		0.095493	
1/6 cos ² 2 pi		0.166667 x	0.095493 = 0.01592
(5/12 - 1/6 cos ² 2 pi)		0.416667 -	0.015915 = 0.400751
sin pi cos pi		0.951056 x	(0.309019) = (0.29)
(5/12 - 1/6 cos ² 2 pi) x sin pi cos pi = B		0.400751 x	(0.309019) = (0.12384)
(n.@pi.p)		1.55 x	3.14 = 4.86
(n.@pi.p/2)		4.86 :	2.00 = 2.43
(n.@pi.p/2) x (ri/ro) ² = C		2.43 x	0.90 = 2.19
sum	A - B + C	0.47 - (0.12) +	2.19 = 2.78
ro = sin pi/2 (2 + cos ² 2 pi) - pi cos pi - a.@pi.p cos pi			
(Sin pi)/2		0.317	
(2 + cos ² 2 pi)		2.000 +	0.095 = 2.095
sin pi/2 (2 + cos ² 2 pi)		0.317 x	2.095 = 0.664
pi cos pi		1.885 x	(0.309) = (0.582)
a.@pi.p		1.548 x	3.142 = 4.864
a.@pi.p cos pi		4.864 x	(0.309) = (1.503)
		2.78	2.750
e		2.78	
ro		2.75	pi = 108.00 rad
e			e
ro		1.0120	ro

Load	Description							Vo	Ho	Xo	Yo	M
	W	Y	X	H	W	W	Weight					
C1	2.40	2.40	2.045	1.00	0.30	1.472	1.472	1.472				
C4	2.40	2.40	1.000	1.00	1.00	2.400	2.400	2.400				
C5	0.5	2.40	0.500	0.55	0.55	0.330	0.330	0.330				
C2	2.40	2.40	1.000	0.55	1.320	1.320	1.320	1.320				
C6	2.40	2.40	0.500	0.20	0.240	0.240	0.240	0.240				
C7	0.5	2.40	0.250	0.20	0.060	0.060	0.060	0.060				
Pe							0.00		0.00		1.00	0.00
Pq	1.00	1.85	2.545		0.297	2.545	2.545		2.545		1.273	3.239
Pa	0.5					0.699	0.699		0.699		0.848	0.593
Rv	sum					44.781	44.781	44.781				3.832

Section 4 - 4

Top deck	24.261 m
Hwd elv.	21.316 m
H back fill	H3 = 9.00 = 1.00 = 8.00 m
Top footing	24.261 = 8.000 = 16.261 m
H wet soil	24.261 = 21.316 = 2.945 m
H sat soil	8.00 = 2.945 = 5.055 m
H1	2.045 m W1 = 0.300 m
H2	3.655 m W2 = 1.000 m
H4	0.500 m W4 = 0.550 m
H5	0.500 m W5 = 0.550 m
H6	0.500 m W6 = 0.200 m
H7	0.250 m W7 = 0.200 m
H wet	2.945 = 3 + 5.055 = 6.037 m

Load	Description							Vo	Ho	Xo	Yo	M
	W	Y	X	H	W	W	Weight					
C1	2.40	2.40	2.05	1.00	0.30	1.472	1.472	1.472				
C4	2.40	2.40	0.50	1.00	1.00	1.200	1.200	1.200				
C5	0.5	2.40	0.50	0.55	0.55	0.330	0.330	0.330				
C2	2.40	2.40	3.66	0.55	4.825	4.825	4.825	4.825				
C6	2.40	2.40	0.50	0.20	0.240	0.240	0.240	0.240				
C7	0.5	2.40	0.25	0.20	0.060	0.060	0.060	0.060				
Pe							0		0.000		3.655	0.000
Pq1	1.00	1.85	8.00		0.297	8.000	8.000		8.000		4.000	32.000
Pw1	0.5	2.40	8.673		2.383	2.383	2.383		2.383		6.037	14.384
Ps1	0.5	2.00	25.553		7.589	7.589	7.589		7.589		1.685	12.788
Ps2	2.00	2.00	5.055		3.003	3.003	3.003		3.003		2.528	7.589
Pw1	0.5	1.00	28.676		14.338	14.338	14.338		14.338		1.785	25.593
Pwr	0.5	1.00	25.553		12.777	12.777	12.777		12.777		1.685	21.528
Rv	sum					44.781	44.781	44.781				70.826

Section 5 - 5

H1	=	0.300	m	W1	=	1.80	m
H2	=	1.000	m	W2	=	1.80	m
H wet	=	2.945	-	2.045	=	0.90	m
H sat	=	5.055	m				

Load	Description							Vo	Ho	Xo	Yo	M
	W	Y	X	H	W	W	Weight					
C1	0.5	2.40	2.40	0.30	1.80	0.648	0.648	0.648		0.600		0.389
C2	2.40	2.40	1.00	1.80	4.320	4.320	4.320			0.900		3.888
El(wet)	1.85	2.40	0.90	1.80	2.997	2.997	2.997			0.900		2.697
El(sat)	2.00	2.00	5.055	1.80	18.198	18.198	18.198			0.900		16.378
U(uplift)	1.00	2.00	5.055	1.80	9.099	9.099	9.099			0.900		8.189
Re 4							(17.550)	(17.550)		1.175		(20.621)
Re 3							(31.170)	(31.170)		0.000		0.000
	total						(31.656)	(31.656)				(3.456)

Section 6 - 6

H 3.1	=	0.300 m	W1	=	2.30 m
H 3.2	=	1.000 m	W2	=	2.30 m
H wet	=	2.945 m			
H sat	=	5.055 m			

Load	Description							Vo	Ho	Xo	Yo	M
	W	Y	X	H	W	W	Weight					
C3.1	0.5	2.40	2.40	0.30	2.30	0.828	0.828	0.828		0.767		0.635
C3.2	2.40	2.40	1.00	2.30	5.520	5.520	5.520			1.150		6.348
El(wet)	1.85	2.40	2.945	2.30	12.531	12.531	12.531			1.150		14.411
El(sat)	2.00	2.00	5.055	2.30	23.253	23.253	23.253			1.150		26.741
U(uplift)	1.00	2.00	5.055	2.30	11.626	11.626	11.626			1.150		(13.370)
Re 1							(65.250)	(65.250)		1.675		(109.294)
Re 2							(48.200)	(48.200)		0.425		(20.485)
	total						(82.945)	(82.945)				(95.015)

Project : Medan Flood Control	Japan International Cooperation Agency (JICA)	Calculation sheet
Channel : Perak River		Date
Structure : Br. P - 9		Nos of sheet
Station : PE 200 + 25 m		Prepared by

III.8.2 Normal Condition Case - II, Low Water Level

Ka = 0.297	Ywet = 1.850 t/m ³
Ht = 9.000 m	Ysat = 2.000 t/m ³
W = 19.00 m	Yw = 1.000 t/m ³
q = 1.000 t/m ²	Yc = 2.400 t/m ³
alpha = 58.00 degree	sin alpha = 0.848 degree
	skew width = 22.40 m

Description	Total load (ton)			
	Dead load	Line load	Pedestrian load	Total load
weight	1528.38	54.55	399.67	24.00
per Abt	764.19	27.275	199.8	2006.60
perm'	34.109	1.217	8.919	1003.3
				44.781

Section 1 - 1 H1 = 2.045 m W1 = 0.300 m

Load	Description									
	Y	x	H	W	Wn	Weight	Vo	Ho	Xo	Yo
C1	2.40	x	2.045	x	0.30	1.472	1.472			
Pq	q	x	H							
Pa1	0.5 x Y	x	H1 - 2 x Ka			2.045		2.045		1.023
	0.5 x Y	x	1.85 x 4.18 x 0.297			1.149		1.149		0.682
	total						1.472	3.194		2.091

Section 2 - 2

H1 = 2.045 m W1 = 0.30 m
H4 = 0.500 m W4 = 0.55 m
H5 = 0.500 m W5 = 0.55 m

Load	Description									
	Yc	x	H	W	Wn	Weight	Vo	Ho	Xo	Yo
C1	2.40	x	2.045	x	0.30	1.472	1.472			
C2	2.40	x	0.500	x	0.55	0.660	0.660			
C3	0.5 x q	x	0.500	x	0.55	0.330	0.330			
E1	q	x	H	x	W					
	1.00	x	2.045	x	0.25	0.511	0.511			
	total						2.974			

Section 3 - 3

Pe = 0.000 ton (by earthquake)

H1 = 2.045 m	W1 = 0.300 m	H1 + H4 = 2.545 m
H2 = 1.000 m	W2 = 1.000 m	He = H4 + H5 = 1.000 m
H4 = 0.500 m	W4 = 0.550 m	
H5 = 0.500 m	W5 = 0.550 m	
H6 = 0.500 m	W6 = 0.200 m	
H7 = 0.500 m	W7 = 0.200 m	

Load	Description									
	W	Y	x	H	W	Wn	Weight	Vo	Ho	Yo
C1	2.40	x	2.045	x	0.30	1.472	1.472			
C4	2.40	x	1.000	x	1.00	2.400	2.400			
C5	0.5 x	2.40	x	0.500	x	0.330	0.330			
C2	2.40	x	0.500	x	0.55	0.660	0.660			
C6	2.40	x	0.500	x	0.20	0.240	0.240			
C7	0.5 x	2.40	x	0.500	x	0.120	0.120			
Pe							0.000			
Pq	1.00	x	2.545	x	0.297	2.545	2.545			
Pa	0.5 x	1.85	x	2.545	x	0.699	0.699			
Rv							44.781	44.781		
	sum						50.004	3.244		
										3.832

Section 4 - 4

Top deck = 24.261 m
Lwl elev. = 17.674 m
H back fill = Ht = 9 - 1.00 = 8.00 m
Top footing = 24.261 - 8.000 = 16.261 m
H wet soil = 24.261 - 17.674 = 6.587 m under of footing
H wet soil = 24.261 - 16.261 = 8.000 m
H sat soil = 8.00 - 8 = 0.00 m

H1 = 2.045 m	W1 = 0.300 m
H2 = 3.655 m	W2 = 1.000 m
H4 = 0.500 m	W4 = 0.550 m
H5 = 0.500 m	W5 = 0.550 m
H6 = 0.500 m	W6 = 0.200 m
H7 = 0.250 m	W7 = 0.200 m
H wet = 8 :	3 + 0.00 = 2.667 m

Load	Description									
	Y	x	H	W	Wn	Weight	Vo	Ho	Xo	Yo
C1	2.40	x	2.05	x	0.30	1.472	1.472			
C4	2.40	x	0.50	x	1.00	1.200	1.200			
C5	0.5 x	2.40	x	0.50	x	0.330	0.330			
C2	2.40	x	3.66	x	0.55	4.825	4.825			
C6	2.40	x	0.50	x	0.20	0.240	0.240			
C7	0.5 x	2.40	x	0.25	x	0.060	0.060			
Pe						0	0			
Pq1	1.00	x	8.00			8.000	8.000			
Pw1	0.5 x	1.85	x	64.000	x	17.582	17.582			
Pa1	0.5 x	2.00	x	0.00	x	0.000	0.000			
Pa2	2.00	x	0.00	x	0.297	0.000	0.000			
Pw4	0.5 x	1.00	x	0.090	x	0.045	0.045			
Pw7	0.5 x	1.00	x	0.000	x	0.000	0.000			
Rv						44.781	44.781			
	sum						52.908	25.627		
										78.891

Section 5 - 5

H1 = 0.300 m W1 = 1.80 m
H2 = 1.000 m W2 = 1.80 m
H wet = 8.000 - 2.045 = 5.96 m
H sat = 0.000 m

Load	Description						Y _o	X _o	H _o	Y _o	M = V _o X _o
	Y	X	H	W	Weight						
C1	0.5	2.40	0.30	1.80	0.648	0.648	0.600				0.389
C2	2.40	2.40	1.00	1.80	4.320	4.320	0.900				3.888
El(wet)	1.85	5.96	1.80	1.80	19.830	19.830	0.900				17.847
El(sat)	2.00	0.000	1.80	1.80	0.000	0.000	0.900				0.000
Ul(uplift)	1.00	0.000	1.80	1.80	0.000	0.000	0.900				0.000
Rp 4					(22.570)	(22.570)	1.175				(26.520)
Rp 3					(35.270)	(35.270)	0.000				0.000
total					(33.042)	(33.042)					(4.396)

Section 6 - 6

H31 = 0.300 m W1 = 2.30 m

H32 = 1.000 m W2 = 2.30 m

Hwet = 8.000 m

Hsat = 0.000 m

Load	Description						Y _o	X _o	H _o	Y _o	M = V _o X _o
	Y	X	H	W	Weight						
C31	0.5	2.40	0.300	2.30	0.828	0.828	0.767				0.635
C32	2.40	2.40	1.000	2.30	5.520	5.520	1.150				6.348
El(wet)	1.85	8.000	2.30	2.30	34.040	34.040	1.150				39.146
El(sat)	2.00	0.000	2.30	2.30	0.000	0.000	1.150				0.000
Ul(uplift)	1.00	0.000	2.30	2.30	0.000	0.000	1.150				0.000
Ro 1					(66.940)	(66.940)	1.675				(112.124)
Rp 2					(51.110)	(51.110)	0.425				(21.722)
total					(77.662)	(77.662)					(87.717)

mch-2

Project : Median Flood Control	Japan International Cooperation Agency (JICA)	Calculation sheet
Channel : Perut River		Date
Structure : Br. P - 9		Nos of sheet
Station : PE 200 + 25 m		Prepared by

III.8.3 Earthquake condition Case - III. High Water Level

Ka = 0.407	Ywet = 1.850	tm3
Ht = 9.000	Ysat = 2.000	tm3
W = 19.000	Yw = 1.000	tm3
q = 0.500	Yc = 2.400	tm3
alpha = 58.00 degree	sin alpha = 0.848	degree
	skew width = 22.40	m

Total load (ton)

Description	Dead load	Line load	Uniform load	Pedestrian load	Total load	Width
weight	1528.38	54.55	399.67	24.00	2006.60	22.40
per Abt.	764.19	27.275	199.8	12.00	1003.3	
per m'	34.109	1.217	8.919	0.536	44.781	

Section 1 - 1 H1 = 2.045 m W1 = 0.300 m

Load	Description						V _o	H _o	X _o	Y _o	M = V _o Y _o
	Y	X	H	W	Weight						
C1	2.40	2.40	2.045	0.30	1.472	1.472	1.472				
Pq	q	x	H								
Pa1	0.5	x	Y	2.045	1.023	1.023		1.023		1.023	1.046
	0.5	x	1.85	x	4.18	4.07	1.574	1.574		0.682	
total							1.472	2.597			1.046

Section 2 - 2

H1 = 2.045 m W1 = 0.30 m

H2 = 0.500 m W2 = 0.55 m

H3 = 0.500 m W3 = 0.55 m

Load	Description						V _o	H _o	X _o	Y _o	M = V _o Y _o
	Yc	x	H	x	W	Weight					
C1	2.40	x	2.045	x	0.30	1.472	1.472		0.150		0.221
C2	2.40	x	0.500	x	0.55	0.660	0.660		0.275		0.182
C3	2.40	x	0.500	x	0.55	0.330	0.330		0.275		0.091
E1	q	x	H	x	W						
	0.50	x	2.045	x	0.25	0.256	0.256		0.275		0.070
total							2.718				0.563

Section 3 - 3

Pe = 6.140 ton (by earthquake)

H1 = 2.045 m	W1 = 0.300 m	H1 + H4 = 2.545 m
H2 = 1.000 m	W2 = 0.550 m	He = H4 + H5 = 1.000 m
H4 = 0.500 m	W4 = 0.550 m	
H5 = 0.500 m	W5 = 0.200 m	
H6 = 0.500 m	W6 = 0.200 m	
H7 = 0.250 m	W7 = 0.200 m	

mnh-2

Load	Description						Vo	Ho	Xo	Yo	M
	W	Y	X	H	W	Weight					
C1		2.40	x	2.045	x	0.30	1.472	1.472			
C4		2.40	x	0.500	x	1.00	1.200	1.200			
C5		0.5	x	2.40	x	0.55	0.330	0.330			
C2		2.40	x	1.000	x	0.55	1.320	1.320			
C6		2.40	x	0.500	x	0.20	0.240	0.240			
C7		0.5	x	2.40	x	0.20	0.060	0.060			
Pe						6.14			6.14	1.00	6.14
Pq		0.50	x	2.545	x		1.273	1.273		1.273	1.619
Pa		0.5	x	1.85	x	2.545	0.958	0.958		0.848	0.813
Rv		sum				44.781	44.781	44.781			8.572

Section 4 - 4											
Top deck elv	=	24.261	m								
Hwd elv	=	21.316	m								
H back fill	=	H1	-	H3							
	=	9.00	-	1.00	=	8.00	m				
Top footing elv	=	24.261	-	8.000	=	16.261	m				
H wet soil	=	24.261	-	21.316	=	2.945	m				
H sat soil	=	8.00	-	2.945	=	5.055	m				
		H1	=	2.045	m	W1	=	0.300	m		
		H2	=	3.655	m	W2	=	1.000	m		
		H4	=	0.500	m	W4	=	0.550	m		
		H5	=	0.500	m	W5	=	0.550	m		
		H6	=	0.500	m	W6	=	0.200	m		
		H7	=	0.250	m	W7	=	0.200	m		
		H wet	=	2.945	:	3	+	5.055	=	6.037	m

Load	Description						Vo	Ho	Xo	Yo	M = Ho.Yo
	Y	X	Hn	X	Wn	Weight					
C1		2.40	x	2.05	x	0.30	1.472	1.472			
C4		2.40	x	0.50	x	1.00	1.200	1.200			
C5		0.5	x	2.40	x	0.55	0.330	0.330			
C2		2.40	x	3.66	x	0.55	4.825	4.825			
C6		2.40	x	0.50	x	0.20	0.240	0.240			
C7		0.5	x	2.40	x	0.20	0.060	0.060			
Pe						6.14			6.140	3.65	22.442
Pq1		0.50	x	8.00			4.000	4.000		4.000	16.000
Pw1		0.5	x	1.85	x	8.673	3.265	3.265		6.037	19.711
Ps1		0.5	x	2.00	x	25.553	0.407	0.407		1.685	17.524
P2		0.5	x	2.00	x	5.055	0.407	0.407		2.528	10.400
Pw		0.5	x	1.00	x	28.676	14.338	14.338		1.785	25.593
Pwr		0.5	x	1.00	x	25.553	12.777	12.777		1.685	21.528
Rv		sum				44.781	44.781	44.781			90.142

Section 5 - 5											
H1	=	0.300	m	W1	=	1.80	m				
H2	=	1.000	m	W2	=	1.80	m				
H wet	=	2.945	-	2.045	=	0.90	m				
H sat	=	5.055	m								

Load	Description						Vo	Ho	Xo	Yo	M = Vo.Xo
	Y	X	H	X	W	Weight					
C1		0.5	x	2.40	x	1.80	0.648	0.648		0.600	0.389
C2		2.40	x	1.00	x	1.80	4.320	4.320		0.900	3.888
El(wet)		1.85	x	0.90	x	1.80	2.997	2.997		0.900	2.697
El(sat)		2.00	x	5.055	x	1.80	18.198	18.198		0.900	16.378
U1(up lift)		1.00	x	5.055	x	1.80	(9.099)	(9.099)		0.900	(8.189)
Rp4						(27.800)	(27.800)		1.175		(32.665)
Rp3		total				(30.470)	(30.470)		0.000		(17.502)

Section 6 - 6											
H3.1	=	0.300	m	W1	=	2.30	m				
H3.2	=	1.000	m	W2	=	2.30	m				
H wet	=	2.945	m								
H sat	=	5.055	m								

Load	Description						Vo	Ho	Xo	Yo	M = Vo.Xo
	Y	X	H	X	W	Weight					
C3.1		0.5	x	2.40	x	0.300	0.828	0.828		0.767	0.635
C3.2		2.40	x	1.000	x	2.30	5.520	5.520		1.150	6.348
El(wet)		1.85	x	2.945	x	2.30	12.531	12.531		1.150	14.411
El(sat)		2.00	x	5.055	x	2.30	23.253	23.253		1.150	26.741
U1(up lift)		1.00	x	5.055	x	2.30	(11.626)	(11.626)		1.150	(13.370)
Rp1						(68.260)	(68.260)		1.675		(114.336)
Rp2		total				(49.360)	(49.360)		0.425		(20.978)

Project : Medan Flood Control	Japan International Cooperation Agency (JICA)	Calculation sheet
Channel : Perut River		
Structure : Br. P - 9		
Station : PE 200 + 25 m		

III.2.4 Earthquake condition Case - IV, Low Water Level

Ka	=	0.407	Ywet	=	1.850	tm ³
Ht	=	9.000	Ysat	=	2.000	tm ³
W	=	19.00	Yw	=	1.000	tm ³
q	=	0.500	Yc	=	2.400	tm ³
alpha	=	58.00 degree	sin alpha	=	0.848	degree
			skew width	=	22.40	m

Description	Total load (ton)				
	Dead load	Line load	Uniform load	Pedestrian load	Total Width
weight	1528.38	54.55	399.67	24.00	2006.60
per Abc.	764.19	27.275	199.8	12.00	1003.3
per m'	34.109	1.217	8.919	0.536	44.781

Section 1 - 1 H1 = 2.045 m W1 = 0.300 m

Load	Description						Vo	Ho	Xo	Yo	M = Vo.Yo
	Y	x	H1	x	W1	Weight					
C1	2.40	x	2.045	x	0.30	1.472	1.472				
Pq	0.50	x	2.045			1.023		1.023		1.023	1.046
Pa1	0.5 x	Y	x	H1^2	x	Ka					
	0.5 x	1.85	x	4.18	x	0.407	1.574	1.574		0.682	
	total						1.472	2.597			1.046

Section 2 - 2

H1	=	2.045 m	W1	=	0.30 m
H2	=	0.500 m	W2	=	0.55 m
H3	=	0.500 m	W3	=	0.55 m

Load	Description						Vo	Ho	Xo	Yo	M = Vo.Yo
	Yc	x	H	x	W	Weight					
C1	2.40	x	2.045	x	0.30	1.472	1.472				0.221
C2	2.40	x	0.500	x	0.55	0.660	0.660	0.275		0.275	0.182
C3	0.5 x	2.40	x	0.500	x	0.55	0.330	0.275		0.275	0.091
E1	q	x	H	x	W						0.070
	0.50	x	2.045	x	0.25	0.256	0.256	0.275			0.563
	total						2.718				

Section 3 - 3

Pe = 6.140 ton (by earthquake)								
H1	=	2.045 m	W1	=	0.300 m	H1 + H4	=	2.545 m
H2	=	1.000 m	W2	=	1.000 m	He=H4+H5	=	1.000 m
H4	=	0.500 m	W4	=	0.550 m			
H5	=	0.500 m	W5	=	0.550 m			
H6	=	0.500 m	W6	=	0.200 m			
H7	=	0.250 m	W7	=	0.200 m			

Page 4

Load	W			Description			Weight		Vo ton	Ho ton	Xo m	Yo m	M = Ho.Yo Lm
	Y m	X m	H m	W t/m ³	W ton								
C1	=	2.40	x	2.045	x	0.30	=	1.472	1.472				
C4	=	2.40	x	0.500	x	1.00	=	1.200	1.200				
C5	=	0.5	x	2.40	x	0.55	=	0.330	0.330				
C2	=	2.40	x	1.000	x	0.55	=	1.320	1.320				
C6	=	2.40	x	0.500	x	0.20	=	0.240	0.240				
C7	=	0.5	x	2.40	x	0.20	=	0.060	0.060				
Pe	=						=	6.14	6.14			1.00	6.14
Pq	=	0.5	x	1.85	x	2.545	x	0.407	1.273			1.273	1.619
Pa	=						=	0.958	0.958			0.848	0.813
Rv	=						=	44.781	44.781				
sum									49.404	8.371			8.572
Section 4 - 4													
Top deck	=	24.261	m										
Lwl elv.	=	17.674	m										
H back fill	=	Ht	=	H3									
	=	9	=	1.00	=	8.00	m						
Top footin	=	24.261	=	8.000	=	16.261	m						
H wet soil	=	24.261	=	17.674	=	6.587	m	under of footing					
H wet soil	=	24.261	=	16.261	=	8.000	m						
H sat soil	=	8.00	=	8	=	0.00	m						
H1	=	2.045	m	W1	=	0.300	m						
H2	=	3.655	m	W2	=	1.000	m						
H4	=	0.500	m	W4	=	0.550	m						
H5	=	0.500	m	W5	=	0.550	m						
H6	=	0.500	m	W6	=	0.200	m						
H7	=	0.250	m	W7	=	0.200	m						
H wet	=	8	=	3	=	0.00	m	2.667					
Section 5 - 5													
C1	=	2.40	x	2.05	x	0.30	=	1.472	1.472				
C4	=	2.40	x	0.50	x	1.00	=	1.200	1.200				
C5	=	0.5	x	2.40	x	0.55	=	0.330	0.330				
C2	=	2.40	x	3.66	x	0.55	=	4.825	4.825				
C6	=	2.40	x	0.50	x	0.20	=	0.240	0.240				
C7	=	0.5	x	2.40	x	0.20	=	0.060	0.060				
Pe	=						=	6.14	6.14			3.655	22.442
Pq1	=	0.50	x	8.00	x		=	4.000	4.000			4.000	16.000
Pw1	=	1.85	x	64.000	x	0.407	=	24.094	24.094			2.667	64.252
Pa1	=	0.5	x	2.00	x	0.00	x	0.000	0.000			0.000	0.000
Pa2	=	0.5	x	2.00	x	0.00	x	0.000	0.000			0.000	0.000
Pw2	=	0.5	x	1.00	x	0.090	=	0.045	0.045			0.100	0.005
Pw3	=	0.5	x	1.00	x	0.000	=	0.000	0.000			0.000	0.000
Rv	=						=	44.781	44.781				
sum									52.908	34.279			102.698

Project : Medan Flood Control	Japan International Cooperation Agency (JICA)	Calculation sheet
Channel : Perout River		Date
Structure : Br - P 9		Nos of sheet
Station : Pe 200 + 25		Prepared by

III.9 Resume forces on Abutment Section									
Case - I, Normal Condition (H.W.)									
Case	Forces	Unit	1	2	3	4	5	6	
Case - I	Moment	tm	2.091	0.634	3.832	70.826	5.458	95.015	
		kg.cm	209,100	63,400	383,200	7,082,600	545,800	9,501,500	
	Shear	ton	3.194	2.974	3.244	22.536	31.656	82.945	
		kg	3,194	2,974	3,244	22,536	31,656	82,945	
	Normal	ton	1.472	0.0	50.604	52.908	0.0	0.0	
		kg	1,472	0.0	50,604	52,908	0.0	0.0	

Case - II, Normal Condition (L.W.)									
Case	Forces	Unit	1	2	3	4	5	6	
Case - II	Moment	tm	2.091	0.634	3.832	78.891	4.396	87.717	
		kg.cm	209,100	63,400	383,200	7,889,100	439,600	8,771,700	
	Shear	ton	3.194	1.472	3.244	25.627	33.042	77.062	
		kg	3,194	1,472	3,244	25,627	33,042	77,062	
	Normal	ton	1.472	0	50.004	52.908	0	0	
		kg	1,472	0	50,004	52,908	0	0	

Case - III, Earthquake Condition (H.W.)									
Case	Forces	Unit	1	2	3	4	5	6	
Case - III	Moment	tm	1.046	0.563	8.572	90.142	17.502	100.55	
		kg.cm	104,600	56,300	857,200	9,014,200	1,750,200	10,055,000	
	Shear	ton	2.597	2.718	8.371	29.48	41.206	87.115	
		kg	2,597	2,718	8,371	29,482	41,206	87,115	
	Normal	ton	1.472	0	49.404	52.908	0	0	
		kg	1,472	0	49,404	52,908	0	0	

Case - IV, Earthquake Condition (L.W.)									
Case	Forces	Unit	1	2	3	4	5	6	
Case - IV	Moment	tm	1.046	0.563	8.572	102.698	4.396	87.717	
		kg.cm	104,600	56,300	857,200	10,269,800	439,600	8,771,700	
	Shear	ton	2.597	2.718	8.371	34.279	33.042	77.662	
		kg	2,597	2,718	8,371	34,279	33,042	77,662	
	Normal	ton	1.472	0	49.404	52.908	0	0	
		kg	1,472	0	49,404	52,908	0	0	

mmh

Load	Description									
	Y	X	H	X	W	Weight	Vo	Ho	Xo	Yo
	m	m	m	m	tm ³	ton	ton	ton	m	m
C1	0.5	2.40	0.30	1.80	0.648	0.648	0.600	0.000	0.000	0.389
C2	1.85	2.40	1.00	1.80	4.320	4.320	4.320	0.000	0.000	3.888
El(wet)	1.85	5.96	1.80	1.80	19.830	19.830	19.830	0.000	0.000	17.847
El(sat)	2.00	0.000	1.80	1.80	0.000	0.000	0.000	0.000	0.000	0.000
U1(up lift)	1.00	0.000	1.80	1.80	0.000	0.000	0.000	0.000	0.000	0.000
Rp 4										
Rp 3										
	total									
						(35.270)	(35.270)		0.000	0.000
							(33.042)			(4.396)

Section 6 - 6									
H 3.1	0.300	m	W1	2.30	m				
H 3.2	1.000	m	W2	2.30	m				
H wet	8.000	m							
H sat	0.000	m							

Load	Description									
	Y	X	H	X	W	Weight	Vo	Ho	Xo	Yo
	m	m	m	m	tm ³	ton	ton	ton	m	m
C 3.1	0.5	2.40	0.300	2.30	0.828	0.828	0.828	0.767	0.000	0.635
C 3.2	1.85	2.40	1.000	2.30	5.520	5.520	5.520	1.150	0.000	6.348
El(wet)	1.85	8.000	2.30	2.30	34.040	34.040	34.040	1.150	0.000	39.146
El(sat)	2.00	0.000	2.30	2.30	0.000	0.000	0.000	0.000	0.000	0.000
U1(up lift)	1.00	0.000	2.30	2.30	0.000	0.000	0.000	0.000	0.000	0.000
Rp 1										
						(66.940)	(66.940)	1.675		(112.124)
Rp 2										
						(51.110)	(51.110)	0.425		(21.722)
	total									
							(77.662)			(87.717)

Project : Median Flood Control Channel : Pericut River Structure : Br - P 9 Station : Pe 200 + 25		Japan International Cooperation Agency (JICA)		Calculation sheet	
				Date	Nos of sheet
				Prepared by	
III.10 Reinforcing steel bar of Abutment section					
Section - 4					
Item	Description	unit	Allowance stresses		
			Normal condition	Earth quake condition	Calcu-lation
concrete compressive steel tensile concrete shear	fc = K - 225 fs tens without r bar tens with r bar n = Es/Ec	kg/cm2 kg/cm2 kg/cm2 kg/cm2	75 1400 8 20 21	75 1400	75 1400
	M	kg.cm	70.826 7,082,600	102,696 10,269,800	
Shear	S	ton	22,536 29,482	29,482	
Normal	N	ton	52,908 52,908	52,908	
dimension	b	cm	100	100	100
	ht	cm	100	100	100
	d'	cm	10	10	10
	h	cm	90	90	90
	b.h	cm2	9,000	9,000	9,000
	eo1 = M/N	cm	133.87	194.11	
	eo2 = 1/30 ht > 2 cm	cm	3.33	3.33	
	used eo2	cm	2.00	2.00	
	eo = eo1 + eo2	cm	135.87	196.11	
	eo/ht (see table)	cm	1.36	1.96	
	C	cm	7.70	7.70	
	Ik	cm	365.5	365.5	
100.Ik	cm	10,000	10,000		
Ik/100.Ik	cm	0.0366	0.0366		
[Ik/100.Ik]^2	cm	0.0013	0.0013		
e1 = C.[Ik/(100.Ik)]^2	cm	1.029	1.029		
e2 = 0.15 ht	cm	15.00	15.00		
e = eo + e1 + e2	cm	151.89	212.14		
ea = eo + 1/2.M - d'	cm	191.89	252.14		
M' = N.ea	kg.cm	10,152,780	13,336,980		
n.M'	kg.cm	213,206,370	280,139,570		
b.f.s	kg/cm	140,000	140,000		
n.M'/(b.f.s)		1,522.92	2,001.00		
@sqrt(n.M'/(b.f.s))		39.02	44.73		
Ca = h./@sqrt(n.M'/(b.f.s)) see PBI 1971 N.I.-2		2.306	2.012		
n.f.c		1,575	1,575		
eo = fs / (n.f.c)		0.8869	0.8869		
delta		0.60	0.60		
from table fined Ca		2.554	2.294		
fine e > eo		1,564	1,410		
e'		2,103	1,857		
ksi()		0.390	0.415		
zeta		0.879	0.874		
100 nw		16.69	21.75		
nw		0.1669	0.2175		
w		0.00795	0.01036		
ea/h		2.13	2.80		
from table fined i		4.59	2,310		
LA = w.h		71.53	93.21		
A	cm2	15.62	40.35		
A' = delta. cm2		42.917	55.929		

= 1.327		A' - used bar dia. 25 mm @ 12.5 cm,		Area @ bar = 4.900	
= 10		nos of bar =		10	
= 10 x		total area =		10 x	
= 13.27 cm >		15.62 cm2		49.00 cm2 >	
		steel bar full working			
		n.e		32.84	
tension of bar		kg/cm2		1,400.00	
compressive concrete		kg/cm2		42.63	
tensile of bar		kg/cm2		665.72	
		7/8 b.h		7,875.00	
shear concrete		kg/cm2		2.86	
		Tca = S / (7/8 b.h)			
Section - 5					
Item	Description	unit	Normal condition	Earth quake condition	
			Calcu-lation	Calcu-lation	
Allowance stresses					
concrete compressive	fc = K - 225	kg/cm2	75	75	
steel tensile	fs	kg/cm2	1400	1400	
concrete shear	tau without r.bar	kg/cm2	8	8	
	tau with r.bar	kg/cm2	20	20	
transformend	n = Es/Ec		21	21	
Moment	M	tm	95.015	100.55	
		kg cm	9,501,500	10,055,000	
Shear	S	ton	82,945	87,115	
		kg	82945	87115	
dimension	b	cm	100	100	
	ht	cm	1,300	1,300	
	d'	cm	10	10	
	h	cm	1,290	1,290	
	b.h	cm2	129,000	129,000	
	M	kg.cm	9,501,500	10,055,000	
	n.M	kg.cm	199,531,500	211,155,000	
	b.fs	kg/cm	140,000	140,000	
	n.M / (b.fs)		1,425.23	1,508.25	
	@sqrt(n.M / (b.fs))		37.75	38.84	
	Ca = h / @sqrt(n.M / (b.fs)) see PBI 1971 N.I.-2		34.170	33.216	
	n.f.c		1.575	1.575	
	eo = fs / (n.f.c)		0.8889	0.8889	
	delta		1.00	1.00	
	from table fined Ca		12,290	12,290	
	fine e > eo		8.091	8.091	
	e'		89,000	89,000	
	ksi()		0.110	0.110	
	zeta		0.963	0.963	
	100 nw		0.688	0.688	
	nw		0.00688	0.00688	
	w		0.00033	0.00033	
	A	cm2	42.26	42.26	
	A' = delta.I.A	cm2	42.26	42.26	
A - used bar dia. 25 mm @ 12.5 cm,					
Area @ bar = 4.900					
nos of bar = 10					
total area = 10 x					
[cm2] = 49.00 cm2 >					
steel bar full working					
n.e					
kg/cm2					
1,400.00					
tension of bar					
compressive concrete					
tensile of bar					
kg/cm2					
8.24					
7/8 b.h					
kg/cm2					
15.73					
Tca = S / (7/8 b.h)					
cm2					
112,875.00					
kg/cm2					
0.73					
bar					

**PRESTRESSED CONCRETE BRIDGE COMPUTATION
CHANNEL PERCUT RIVER**

Br - P 7

Station : Pe 169 + 59

span
Midle
40.00

width

9.0 m

II SUB STRUCTURE

Abutment

- II.1 Neutral axis of Abutment**
- II.2 Earthquake coefficient**
- II.3 Axial forces on Abutment**
- II.4 Coulumb coefficient**
- II.5 Horizontal force**
 - II.5.1 Case -- I,Normal condition (HWL)**
 - II.5.2 Case -- II,Normal condition (LWL)**
 - II.5.3 Case -- III,Earthquake condition (HWL)**
 - II.5.4 Case -- IV,Earthquake condition (LWL)**
- II.6 Combined Load on Abutment**
 - II.6.1 Case -- I,Normal condition (HWL)**
 - II.6.2 Case -- II,Normal condition (LWL)**
 - II.6.3 Case -- III,Earthquake condition (HWL)**
 - II.6.4 Case -- IV,Earthquake condition (LWL)**

Project : Medan Flood Control Channel : Medan Flood Way Structure: Br - P7 Station : PE 169 + 39 m			Japan International Cooperation Agency (JICA)		Calculation sheet	
					Date	Nos of sheet
					Prepared by	
II ABUTMENT						
II.1 Neutral axis						
depth of abutment = Ht	=	6.70	m			
depth beam	=	1.70	m			
depth slab	=	0.245	m			
depth of beam to deck	=	1.945	m			
depth of beam and deck	=	1.700	+	0.245	=	1.945 m
depth bearing pad	=	1.945	+	0.100	=	2.045 m
depth back wall	=	1.000	m			
depth of footing	=	6.700	-	2.045	=	1.000 = 3.655 m
depth of stream						
H1	=	2.045	m	W1	=	0.300 m
H2	=	3.655	m	W1	=	2.300 m
H3	=	1.000	m	W2	=	1.000 m
Ht	=	6.700	m	W3	=	1.800 m
				W3	=	5.100 m
Depth of back fill						
H4	=	6.700	-	1.000	=	5.700 m
H5	=	0.500	m	W4	=	0.550 m
H6	=	0.250	m	W5	=	0.550 m
H7	=	0.500	m	W6	=	0.200 m
H8	=	0.250	m	W7	=	0.200 m
H9	=	0.300	m	W8	=	0.300 m
				W9	=	1.800 m
W1	=	0.300	+	W2	+	W3
H1	=	2.045	+	H1/2	+	1.800
				Ht	=	6.700
W2	=	1.000	+	W3	+	1.025
H2	=	3.655	+	H2/2	+	1.800
W3	=	5.100	+	W3/2	+	0.500
H3	=	1.000	+	H3/2	+	0.500
W4	=	0.550	+	W4/2	+	W3
H4	=	0.500	+	H4/2	+	H3
W5	=	0.550	+	W5/3	+	W2
H5	=	0.250	+	H5/2	+	1.000
W6	=	0.200	+	W6/2	+	W3
H6	=	0.500	+	H6/2	+	H3
W7	=	0.200	+	W7/3	+	W3
H7	=	0.250	+	H7/2	+	H3
W8	=	2.300	+	W8/2/3	+	W2
H8	=	0.300	+	H8/3	+	H3
				0.100	+	1.000

A9	W9 = 1.800	X9 = W9/2/3	1.200	H9 = 0.300	Y9 = H9/3	+	H3	= 1.200	m
								= 1.100	m
Wing wall									
	Hgw = 1.200	m	H back fill = 5.70	m					
	Ww1 = 3.00	m	Hw1 = 3.00	m					
	Ww2 = 2.30	m	Hw2 = 1.00	m					
	Ww3 = 0.70	m	Hw3 = 1.00	m					
	Ww4 = 2.30	m	Hw4 = 1.40	m					
	Ww5 = 2.30	m	Hw5 = 0.30	m					
Ww1	= 3.000	Xw1 = Ww1/2	1.500	+	W2	+	W3	= 4.300	m
Hw1	= 3.000	Yw1 = Ht	6.700	-	Hw1/2	-	1.500	= 5.200	m
Ww2	= 2.300	Xw2 = Ww2/2	1.150	+	W2	+	W3	= 3.950	m
Hw2	= 1.000	Yw2 = Ht	6.700	-	Hw1	-	Hw2/2	= 3.200	m
Ww3	= 0.700	Xw3 = W3	5.100	-	Ww3/2	-	0.350	= 4.750	m
Hw3	= 1.000	Yw3 = Ht	6.700	-	Hw1	-	Hw3/2	= 3.367	m
Ww4	= 2.300	Xw4 = Ww4/2	1.150	+	W2	+	W3	= 3.950	m
Hw4	= 1.400	Yw4 = Hw4/2	0.700	+	Hw5	+	H3	= 2.000	m
Ww5	= 2.300	Xw5 = W3	5.100	-	Ww5/2/3	-	1.533	= 3.567	m
Hw5	= 0.300	Yw5 = Hw5/3	0.100	+	H3	+	1.000	= 1.100	m
Guide wall									
Gw	Wgw = 3.000	Xgw = Wgw/2	1.500	+	W2	+	W3	= 4.300	m
Hgw	= 1.200	Ygw = Ht	6.700	+	Hgw/2	+	0.600	= 7.300	m
Approach slab									
Apr	Wapr = 3.000	Xapr = Wapr/2	1.500	+	W2	+	W3	= 4.300	m
Hapr	= 0.200	Yapr = Ht	6.700	-	H1	+	Hapr/2	= 4.755	m
Asphalt									
Asp	Wasp = 3.000	Xasp = W9/2	1.500	+	W2	+	W3	= 4.300	m
Hasp	= 0.050	Yasp = Ht	6.700	-	Hasp/2	-	0.025	= 6.675	m
Back fill HWL									
a. In case High Water Level									
HWL elevation	=	17.571	m						
LWL elevation	=	13.461	m						
Top deck elevation	=	20.516	m						
Base foiling elev.	=	20.516	-	6.700	=	13.816	m		
Top foiling elev.	=	13.816	+	1.000	=	14.816	m		
Depth of back fill (H b5)	=	20.516	-	14.816	=	5.700	m		

Depth of wet soil HwetL	=	Top deck elv.	=	Hwl elv. --->Hwl elv. under base footing elv.	
	=	20.516	=	17.571	= 2.945 m
HwetR	=	Hwe	=	Top footing elv.	
	=	16.771	=	14.816	= 1.955 m under footing elv.
Depth of saturated soil HsatL	=	Hbf	=	Hwet	
	=	5.700	=	2.945	= 2.755 m
a. soil wet condition					
wetL	=	WwetL/2	=	X1 = WwetL/2 + W2 + W3	
	=	2.300	=	1.150 + 1.000 + 1.800	= 3.950 m
HwetL	=	Y1 = Ht	=	HwetL/2	
	=	2.945	=	6.700 - 1.472	= 5.228 m
wetR	=	WwetR	=	W3/2	
	=	1.800	=	0.900	= 0.900 m
HwetR	=	Y2 = HwetL/2	=	H3	
	=	1.955	=	0.978 + 1.000	= 1.978 m
b. soil saturated condition					
satL	=	WsatL	=	X1 = WsatL/2 + W2 + W3	
	=	2.300	=	1.150 + 1.000 + 1.800	= 3.950 m
HsatL	=	Y1 = HsatL/2	=	H3	
	=	2.755	=	1.378 + 1.000	= 2.378 m
satR	=	WsatR	=	X2 = WsatR/2	
	=	1.800	=	0.900	= 0.900 m
HsatR	=	Y2 = HsatL/2	=	H3	
	=	2.755	=	1.378 + 1.000	= 2.378 m
b. In case Low Water Level					
Depth of wet soil on left side footing					
HwetL	=	Top footing elv.	=	Lwl elv. < Top footing elv.	
	=	20.516	=	14.816	= 5.700 m
Depth of wet soil on right side footing					
HwetR	=	Hwe elv.	=	Top footing elv.	
	=	16.771	=	14.816	= 1.955 m
Depth of saturated soil HWL = Hsat					
Hsat	=	5.700	=	5.700	= 0.000 m
a. soil wet condition					
wetL	=	WwetL	=	X1 = WwetL/2 + W2 + W3	
	=	2.300	=	1.150 + 1.000 + 1.800	= 3.950 m
HwetL	=	Y1 = HwetL/2	=	H3	
	=	5.700	=	2.850 + 1.000	= 3.850 m
wetR	=	WwetR	=	X2 = WwetR/2	
	=	1.800	=	0.900	= 0.900 m
HwetR	=	Y2 = HwetL/2	=	H3	
	=	1.955	=	0.978 + 1.000	= 1.978 m
b. soil saturated condition					
satL	=	WsatL	=	X1 = WsatL/2 + W2 + W3	
	=	2.300	=	1.150 + 1.000 + 1.800	= 3.950 m
HsatL	=	Y1 = HsatL/2	=	H3	
	=	0.000	=	0.000 + 1.000	= 1.000 m
satR	=	WsatR	=	X2 = WsatR/2	
	=	1.800	=	0.900	= 0.900 m
HsatR	=	Y2 = HsatL/2	=	H3	
	=	0.000	=	0.000 + 0.200	= 0.200 m

x-y

Project : Medan Flood Control
Channel : Percut River
Structure : Br - P 7
Station : Pe 169 + 59

Japan International
Cooperation Agency
(JICA)

Calculation sheet
Date
Nos of sheet
Prepared by

II.2 Lateral earth pressure due to earthquake

Kh = C x I x S
Kh = coefficient of horizontal seismic loading
C = the base shear coefficient, base on the data :
seismic zone and soil condition
Seismic zone = 4

soil condition soft soil C = 0.15

S = the structure type factor = 1
I = the importance factor
for bridge carrying more than
2,000 vehicle per day

for all other permanent bridge

C	I	S	kh
0.15	1.2	1	0.18

for temporary bridge

C	I	S	kh
0.15	0.8	1	0.12

soil condition firm soil C = 0.10

for bridge carrying more than
2,000 vehicle per day

for all other permanent bridge

C	I	S	kh
0.1	1.2	1	0.12

for temporary bridge

C	I	S	kh
0.1	0.8	1	0.08

source, Bridge Management System

EQ

Project : Medan Flood Control	Japan International Cooperation Agency (JICA)	Calculation sheet
Channel : Percut River		Date
Structure : Br - P7		Nos of sheet
Station : PE 169 + 59		Prepared by

II.3 Axial load to Abutment

a. Superstructure

Vertical load/m'
 Length of abutment = 12.00 m
 Nos of bearing = 2.00
 Depth of abutment = 6.70 m
 Width of footing = 5.10 m

Vertical load on abutment :

Description	Dead load	Line load	Uniform load	Pedestrian load	total load
Total load	936.42	31.64	343.71	24.00	1,335.77
load to Abt.	468.21	15.82	171.86	12.00	667.89
load / m'	39.02	1.32	14.32	1.00	333.94

Seismic coefficient = 0.18

Horizontal load
 Heq = 0.180 x Rdl
 = 0.180 x 39.018
 = 7.023 ton/m'

Acting point of seismic load

WF3	=	1.80	m	X	=	0.300 + WF3
H3	=	1.00	m		=	0.300 + 1.80
				Y	=	H2 + H3
					=	3.655 + 1.00
					=	4.655 m

b. Abutment section

Sec- tion	width m	depth m	length m	nos	Yc t/m3	G / m'	Gv/m'	X m	Y m	GX m3	G.Y m3
A1	0.30	1.595	1.00	1.00	2.40	1.148	1.148	2.950	5.678	3.368	3.368
A2	1.00	3.655	1.00	1.00	2.40	8.772	8.772	2.300	2.828	20.176	20.176
A3	5.10	1.000	1.00	1.00	2.40	12.240	12.240	2.550	0.500	31.212	31.212
A4	0.55	0.500	1.00	1.00	2.40	0.660	0.660	3.075	4.155	2.030	2.030
A5	0.55	0.250	1.00	1.00	2.40	0.165	0.165	2.983	4.322	0.492	0.492
A6	0.20	0.500	1.00	1.00	2.40	0.240	0.240	1.700	3.488	0.408	0.408
A7	0.20	0.500	1.00	1.00	2.40	0.120	0.120	3.567	4.322	0.208	0.208
A8	2.30	0.300	1.00	1.00	2.40	0.828	0.828	3.567	1.100	2.953	2.953
A9	1.80	0.300	1.00	1.00	2.40	0.648	0.648	1.200	1.100	0.778	0.778
					sum		24.821			61.64	61.64

Wing wall and guard wall

Ww1	3.000	3.000	0.40	2.00	2.400	1.440	0.120	4.300	5.200	0.516	0.516
Ww2	2.300	1.000	0.40	2.00	2.400	0.368	0.031	3.950	3.200	0.121	0.098
Ww3	0.400	1.000	0.40	2.00	2.400	0.064	0.005	4.750	3.367	0.025	0.018
Ww4	2.300	1.400	0.40	2.00	2.400	0.515	0.043	3.950	2.000	0.170	0.086
Ww5	2.300	0.300	0.40	2.00	2.400	0.110	0.009	3.567	1.100	0.033	0.010
Gw	3.00	1.200	0.40	2.00	2.000	0.480	0.040	4.300	7.300	0.172	0.292
					sum		0.208			0.865	0.728

Miscellaneous load

Apr	3.00	0.20	1.00	1.00	2.400	1.44	1.440	4.300	4.755	6.192	6.847
asp	3.00	0.05	1.00	1.00	2.300	0.35	0.345	4.300	6.675	1.483	2.303
					sum		1.785			7.675	9.150
					total		26.815			70.18	71.52

axl-lod H-filling = 0.180 x 26.815 = 4.827 ton

Project : Medan Flood Control	Japan International Cooperation Agency (JICA)	Calculation sheet
Channel : Percut River		Date
Structure : Br - P7		Nos of sheet
Station : PE 169 + 59		Prepared by

II.3 Axial load to Abutment

a. Superstructure

Vertical load/m'
 Length of abutment = 12.00 m
 Nos of bearing = 2.00
 Depth of abutment = 6.70 m
 Width of footing = 5.10 m

Vertical load on abutment :

Description	Dead load	Line load	Uniform load	Pedestrian load	total load
Total load	936.42	31.64	343.71	24.00	1,335.77
load to Abt.	468.21	15.82	171.86	12.00	667.89
load / m'	39.02	1.32	14.32	1.00	333.94

Seismic coefficient = 0.18

Horizontal load
 Heq = 0.180 x Rdl
 = 0.180 x 39.018
 = 7.023 ton/m'

Acting point of seismic load

WF3	=	1.80	m	X	=	0.300 + WF3
H3	=	1.00	m		=	0.300 + 1.80
				Y	=	H2 + H3
					=	3.655 + 1.00
					=	4.655 m

b. Abutment section

Sec- tion	width m	depth m	length m	nos	Yc t/m3	G / m'	Gv/m'	X m	Y m	GX m3	G.Y m3
A1	0.30	1.595	1.00	1.00	2.40	1.148	1.148	2.950	5.678	3.368	3.368
A2	1.00	3.655	1.00	1.00	2.40	8.772	8.772	2.300	2.828	20.176	20.176
A3	5.10	1.000	1.00	1.00	2.40	12.240	12.240	2.550	0.500	31.212	31.212
A4	0.55	0.500	1.00	1.00	2.40	0.660	0.660	3.075	4.155	2.030	2.030
A5	0.55	0.250	1.00	1.00	2.40	0.165	0.165	2.983	4.322	0.492	0.492
A6	0.20	0.500	1.00	1.00	2.40	0.240	0.240	1.700	3.488	0.408	0.408
A7	0.20	0.500	1.00	1.00	2.40	0.120	0.120	3.567	4.322	0.208	0.208
A8	2.30	0.300	1.00	1.00	2.40	0.828	0.828	3.567	1.100	2.953	2.953
A9	1.80	0.300	1.00	1.00	2.40	0.648	0.648	1.200	1.100	0.778	0.778
					sum		24.821			61.64	61.64

Wing wall and guard wall

Ww1	3.000	3.000	0.40	2.00	2.400	1.440	0.120	4.300	5.200	0.516	0.516
Ww2	2.300	1.000	0.40	2.00	2.400	0.368	0.031	3.950	3.200	0.121	0.098
Ww3	0.400	1.000	0.40	2.00	2.400	0.064	0.005	4.750	3.367	0.025	0.018
Ww4	2.300	1.400	0.40	2.00	2.400	0.515	0.043	3.950	2.000	0.170	0.086
Ww5	2.300	0.300	0.40	2.00	2.400	0.110	0.009	3.567	1.100	0.033	0.010
Gw	3.00	1.200	0.40	2.00	2.000	0.480	0.040	4.300	7.300	0.172	0.292
					sum		0.208			0.865	0.728

Miscellaneous load

Apr	3.00	0.20	1.00	1.00	2.400	1.44	1.440	4.300	4.755	6.192	6.847
asp	3.00	0.05	1.00	1.00	2.300	0.35	0.345	4.300	6.675	1.483	2.303
					sum		1.785			7.675	9.150
					total		26.815			70.18	71.52

axl-lod H-filling = 0.180 x 26.815 = 4.827 ton

Project : Median Flood Control	Japan International Cooperation Agency (JICA)	Calculation sheet
Channel : Median Flood Way		Date
Structure : Br - P7		No. of sheet
Station : PE 169 + 59		Prepared by

II.5 Horizontal force at Abutment

II.5.1 Normal Condition

Case - I, High Water Level

Depth of abutment	=	6.70 m
Surcharge load	=	1.00 t/m ²
Unit weight wet soil	=	1.85 t/m ³
Unit weight saturated soil	=	2.00 t/m ³
Unit weight of water	=	1.00 t/m ³
Width of footing		
W/1	=	2.30 m
W/2	=	1.00 m
W/3	=	1.80 m
W3	=	5.10 m
Elevation of Hwl	=	17.571 m
Elevation of Lwl	=	13.461 m
Top slab elevation	=	20.516 m
Depth of footing	=	1.000 m
Elv. of base footing	=	20.516 - 6.70 = 13.816 m
Elv. of top footing	=	13.816 + 1.00 = 14.816 m
Depth of back fill	=	20.516 - 14.816 = 5.700 m

Depth of wet soil

H wet	=	Top elv. - Elevation of Hwl
H wet	=	20.516 - 17.571 = 2.945 m
Depth of saturated soil	=	H sat
H sat	=	5.70 - 2.945 = 2.755 m

A Active earth pressure

By live load

Surcharge load = (q)

q	=	1.000 t/m ²	Sin(alpha)	=	0.000
delta	=	0.000	L	=	1.000 m
alpha	=	0.000	H	=	6.700 m
Cos(alpha)	=	1.000	B	=	5.100 m

coefficient of active pressure Ka = 0.297

Pq	=	q x H x L x Ka
Pq	=	1.000 x 5.700 x 1.000 x 0.297
Pq	=	1.693 ton

Pqv	=	Ilq x Sin(alpha)	Pqb	=	Ilq x Cos(alpha)
	=	1.693 x 0.000		=	1.693 x 1.000
	=	0.000 ton		=	1.693 ton
X	=	5.100 m	Y	=	5.700 : 2.000

B By earth

B-1

Wet soil	
Length	= 1.000 m
Ilwet	= 2.945 m
Ilstat	= 2.755 m

1. Load triangular

Pw	=	0.500 x Ywet x Hwet^2 x L x Ka
	=	0.500 x 1.850 x 8.673 x 1.000 x 0.297
	=	2.383 ton

Pw(v)	=	Pw x Sin(alpha)	Pw(h)	=	Pw x Cos(alpha)
	=	2.383 x 0.000		=	2.383 x 1.000
	=	0.000 ton		=	2.383 ton

X	=	5.100 m	Y	=	2.945 : 3.000 + 2.755
	=			=	3.737 m

B-2 Saturated

H sat = 2.755 m

2. Load rectangular

Ps1	=	Ysat x H sat x L x Ka
	=	2.000 x 2.755 x 1.000 x 0.297
	=	1.636 ton

Ps1(v)	=	H sat x Sin(alpha)	Ps1(h)	=	H sat x Cos(alpha)
	=	1.636 x 0.000		=	1.636 x 1.000
	=	0.000 ton		=	1.636 ton

X	=	5.100 m	Y	=	2.755 : 2.000
	=			=	1.378 m

3. Load triangular

Ps2	=	0.500 x Ysat x H sat^2 x L x Ka
	=	0.500 x 2.000 x 7.590 x 1.000 x 0.297
	=	2.254 ton

Ps2(v)	=	H sat-2 x Sin(alpha)	Ps2(h)	=	H sat-2 x Cos(alpha)
	=	2.254 x 0.000		=	2.254 x 1.000
	=	0.000 ton		=	2.254 ton

X	=	5.100 m	Y	=	2.755 : 3.000
	=			=	0.918 m

4. Water

Pwin	=	0.500 x Yw x Hw^2 x L
	=	0.500 x 1.000 x 7.590 x 1.000
	=	3.795 ton

X = 5.100 m

Y = 2.755 : 3.000

= 0.918 m

Pw out	=	0.500 x Yw x Hw^2 x L
	=	0.500 x 1.000 x 9.333 x 1.000
	=	4.667 ton
X	=	0.500 m
Y	=	3.055 : 3.000
	=	1.018 m

5. Uplift HWL

B	=	5.100 m
B-1	=	Hw x B x Yw
	=	2.755 x 5.100 x 1.000
	=	14.051 ton/m ²
X	=	5.100 : 2.000
	=	2.550 m

Project : Median Flood Control	Japan International Cooperation Agency (JICA)	Calculation sheet
Channel : Median Flood Way		Date
Structure : Br - P7		Nos of sheet
Station : PE169 + 59		Prepared by

IL5.2 Normal Condition

Case - II, Low Water Level

Depth of abutment	=	6.70 m
Surcharge load	=	1.00 t/m ²
Unit weight wet soil	=	1.85 t/m ³
Unit weight saturated soil	=	2.00 t/m ³
Unit weight of water	=	1.00 t/m ³
Width of footing		
W1	=	2.30 m
W2	=	1.00 m
W3	=	1.80 m
	=	5.10 m
Elevation of Hwl	=	17.571 m
Elevation of Lwl	=	13.461 m
Top slab elevation	=	20.516 m
Depth of footing	=	1.000 m
Elv. of base footing	=	20.516 - 1.00 = 19.516 m
Elv. of top footing	=	13.816 + 1.00 = 14.816 m
Depth of back fill	=	20.516 - 14.816 = 5.700 m

Depth of wet soil = H wet

H wet	=	Top elv. - Elv. of top footing
H wet	=	20.516 - 14.816 = 5.700 m
Depth of saturated soil	=	H sat
H sat	=	5.70 - 5.700 = 0.000 m

A Active earth pressure

By live load

Surcharge load = (q)

q	=	1.000 t/m ²
delta	=	0.000
alpha	=	0.000
Cos(alpha)	=	1.000
Sin(alpha)	=	0.000
L	=	1.000 m
H	=	6.700 m
B	=	5.100 m

coefficient of active pressure Ka = 0.297

Pq	=	q x H x L x Ka
Pq	=	1.000 x 5.700 x 1.000 x 0.297
Pq	=	1.693 ton

Pqv	=	Hq x Sin(alpha)
	=	1.693 x 0.000
	=	0.000 ton
X	=	5.100 m
	=	5.700 : 2.000

B By earth

B-1

Wet soil	
Length	= 1.000 m
H wet	= 5.700 m
H sat	= 0.000 m

1. Load triangular

Pw	=	0.500 x Ywet x Hwet^2 x L x Ka
	=	0.500 x 1.850 x 32.490 x 1.000 x 0.297
	=	8.926 ton

Pw(v)	=	Pw x Sin(alpha)
	=	8.926 x 0.000
	=	0.000 ton
X	=	5.100 m
	=	Y = 5.700 : 3.000 + 0.000
	=	1.900 m

B-2 Saturated

H sat = 0.000 m

2. Load rectangular

Ps1	=	Ysat x Hsat x L x Ka
	=	2.000 x 0.000 x 1.000 x 0.297
	=	0.000 ton

Ps1(v)	=	Hsat x Sin(alpha)
	=	0.000 x 0.000
	=	0.000 ton
X	=	5.100 m
	=	Y = 0.000 : 2.000
	=	0.000 m

3. Load triangular

Ps2	=	0.500 x Ysat x Hsat^2 x L x Ka
	=	0.500 x 2.000 x 0.000 x 1.000 x 0.297
	=	0.000 ton

Ps2(v)	=	Hsat-2 x Sin(alpha)
	=	0.000 x 0.000
	=	0.000 ton
X	=	5.100 m
	=	Y = 0.000 : 3.000
	=	0.000 m

4. Water

Pwin	=	0.500 x Yw x Hw^2 x L
	=	0.500 x 1.000 x 0.000 x 1.000
	=	0.000 ton
X	=	5.100 m
Y	=	0.000 : 3.000
	=	0.000 m

Pwout	=	0.500 x Yw x Hw^2 x L
	=	0.500 x 1.000 x 0.000 x 1.000
	=	0.000 ton
X	=	5.100 m
Y	=	0.000 : 3.000
	=	0.000 m

5. Uplift H/WL

Pu	=	B x Hw x B x Yw
	=	0.000 x 5.100 x 1.000
	=	0.000 ton/m ²
X	=	5.100 : 2.000
	=	2.550 m

Project	: Median Flood Control	Japan International Cooperation Agency (JICA)	Calculation sheet
Channel	: Perot River		Date
Structure	: Br - P7		No. of sheet
Station	: PE 169 + 59 m		Prepared by

II.5.3 Earthquake Condition

Case - III, High Water Level

Depth of abutment	=	6.70 m
Surcharge load	=	0.50 t/m ²
Unit weight wet soil	=	1.85 t/m ³
Unit weight saturated soil	=	2.00 t/m ³
Unit weight of water	=	1.00 t/m ³
Width of footing		
W/1	=	2.30 m
W/2	=	1.00 m
W/3	=	1.80 m
	=	5.10 m
Elevation of Hwl	=	17.571 m
Elevation of Lwl	=	13.461 m
Top slab elevation	=	20.516 m
Depth of footing	=	1.000 m
Elv. of base footing	=	20.516 - 6.70 = 13.816 m
Elv. of top footing	=	13.816 + 1.00 = 14.816 m
Depth of back fill	=	20.516 - 14.816 = 5.700 m

Depth of wet soil = H wet

H wet	=	Top elev. - Elevation of Hwl
H wet	=	20.516 - 17.571 = 2.945 m
Depth of saturated soil	=	H sat
H sat	=	5.70 - 2.945 = 2.755 m

A Active earth pressure
By live load
Surcharge load = (q)

q	=	0.500 t/m ²
delta	=	0.000
alpha	=	0.000
Cos(alpha)	=	1.000
Sin(alpha)	=	0.000
L	=	1.000 m
H	=	6.700 m
B	=	5.100 m

coefficient of active pressure Ke = 0.407

Pq	=	q x H x L x Ke
Pq	=	0.500 x 5.700 x 1.000 x 0.407
Pq	=	1.160 ton

Pqv	=	Hq x Sin(alpha)
	=	1.160 x 0.000
	=	0.000 ton
X	=	5.100 m
Y	=	5.700 : 2.850 m

B By earth

B-1	Wet soil
Length	= 1.000 m
Ywet	= 2.945 m
Hsat	= 2.755 m

1. Load triangular

Pw	=	0.500 x Ywet x Hwet^2 x L x Ke
	=	0.500 x 1.850 x 8.673 x 1.000 x 0.407
	=	3.265 ton

Pw(v)	=	Pw x Sin(alpha)
	=	3.265 x 0.000
	=	0.000 ton

X	=	5.100 m
Y	=	2.945 : 3.000 + 2.755
	=	3.737 m

B-2 Saturated

Hsat = 2.755 m

2. Load rectangular

Ps1	=	Ysat x Hsat x L x Ke
	=	2.000 x 2.755 x 1.000 x 0.407
	=	2.243 ton

Ps1(v)	=	Hsat x Sin(alpha)
	=	2.243 x 0.000
	=	0.000 ton

X	=	5.100 m
Y	=	2.755 : 2.000
	=	1.378 m

3. Load triangular

Ps2	=	0.500 x Ysat x Hsat^2 x L x Ke
	=	0.500 x 2.000 x 7.590 x 1.000 x 0.407
	=	3.089 ton

Ps2(v)	=	Hsat-2 x Sin(alpha)
	=	3.089 x 0.000
	=	0.000 ton

X	=	5.100 m
Y	=	2.755 : 3.000
	=	0.918 m

4. Water

Pwin	=	0.500 x Yw x Hw^2 x L
	=	0.500 x 1.000 x 7.590 x 1.000
	=	3.795 ton

X = 5.100 m

Y = 2.755 : 3.000

= 0.918 m

Pwout	=	0.500 x Yw x Hw^2 x L
	=	0.500 x 1.000 x 9.333 x 1.000
	=	4.667 ton

X = 5.100 m

Y = 3.055 : 3.000

= 1.018 m

5. Uplift HWL

Pu	=	Hw x B x Yw
	=	2.755 x 5.100 x 1.000
	=	14.051 ton/m ²
X	=	5.100 : 2.000
	=	2.550 m

Project : Medan Flood Control Channel : Medan Flood Way Structure : Br - P7 Station : PE 169 + 59	Japan International Cooperation Agency (JICA)	Calculation sheet Date Nos of sheet Prepared by
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II.5.4 Earthquake condition
Case - IV, Low Water Level

Depth of abutment	= 6.70 m
Surcharge load	= 0.50 t/m ²
Unit weight wet soil	= 1.85 t/m ³
Unit weight saturated soil	= 2.00 t/m ³
Unit weight of water	= 1.00 t/m ³
Width of footing	
W/F1	= 2.30 m
W/F2	= 1.00 m
W/F3	= 1.80 m
W3	= 5.10 m
Elevation of Hwl	= 17.571 m
Elevation of Lwl	= 13.461 m
Top slab elevation	= 20.516 m
Depth of footing	= 1.000 m
Elv. of base footing	= 20.516 m
Elv. of top footing	= 13.816 + 6.70 = 20.516 m
Depth of back fill	= 20.516 - 14.816 = 5.700 m
Depth of wet soil = H wet	
H wet	= Top elv. - Elv. of top footing
H wet	= 20.516 - 14.816 = 5.700 m
Depth of saturated soil = H sat	
H sat	= 5.70 - 5.700 = 0.000 m

A Active earth pressure
By live load
Surcharge load = (q)

q	= 0.500 t/m ²	Sin(alpha)	= 0.000
delta	= 0.000	L	= 1.000 m
alpha	= 0.000	H	= 6.700 m
Cos(alpha)	= 1.000	B	= 5.100 m

coefficient of active pressure Ke = 0.407

Pq	= q x H x L x Ke
Pq	= 0.500 x 5.700 x 1.000 x 0.407
Pq	= 1.160 ton

Pqv	= Hq x Sin(alpha)	Pqb	= Hq x Cos(alpha)
= 1.160 x 0.000	= 1.160 x 1.000		
= 0.000 ton	= 1.160 ton		
X	= 5.100 m	Y	= 5.700 : 2.000

B By earth B-1 Wet soil

Length	= 1.000 m
Hwet	= 5.700 m
Hsat	= 0.000 m

1. Load triangular

$$Pw = 0.500 \times Ywet \times Hwet^2 \times L \times Ke$$

$$= 0.500 \times 1.850 \times 32.490 \times 1.000 \times 0.407$$

$$= 12.232 \text{ ton}$$

$Pw(v)$	=	$Pw \times \sin(\alpha)$	$Pw(h)$	=	$Pw \times \cos(\alpha)$
=		12.232×0.000	=		12.232×1.000
=		0.000 ton	=		12.232 ton
X	=	5.100 m	Y	=	5.700 : 3.000 + 0.000
					= 1.900 m

B-2 Saturated

2. Load rectangular

$$Ps1 = Ysat \times Hsat \times L \times Ke$$

$$= 2.000 \times 0.000 \times 1.000 \times 0.407$$

$$= 0.000 \text{ ton}$$

$Ps1(v)$	=	$Hsat \times \sin(\alpha)$	$Ps1(h)$	=	$Hsat \times \cos(\alpha)$
=		0.000×0.000	=		0.000×1.000
=		0.000 ton	=		0.000 ton
X	=	5.100 m	Y	=	0.000 : 2.000
					= 0.000 m

3. Load triangular

$$Ps2 = 0.500 \times Ysat \times Hsat^2 \times L \times Ke$$

$$= 0.500 \times 2.000 \times 0.000 \times 1.000 \times 0.407$$

$$= 0.000 \text{ ton}$$

$Ps2(v)$	=	$Hsat \times \sin(\alpha)$	$Ps2(h)$	=	$Hsat \times \cos(\alpha)$
=		0.000×0.000	=		0.000×1.000
=		0.000 ton	=		0.000 ton
X	=	5.100 m	Y	=	0.000 : 3.000
					= 0.000 m

4. Water

$Pwin$	=	$0.500 \times Yw \times Hw^2 \times L$
=		$0.500 \times 1.000 \times 0.000 \times 1.000$
=		0.000 ton
X	=	5.100 m
Y	=	0.000 : 3.000
		= 0.000 m

$Pwout$	=	$0.500 \times Yw \times Hw^2 \times L$
=		$0.500 \times 1.000 \times 0.000 \times 1.000$
=		0.000 ton
X	=	5.100 m
Y	=	0.000 : 3.000
		= 0.000 m

5. Uplift HWL

Pu	=	$Hw \times B \times Yw$
=		$0.000 \times 5.100 \times 1.000$
=		0.000 ton/m^2
X	=	5.100 : 2.000
		= 2.550 m

Project : Median Flood Control Channel : Median Flood Way Structure : Br - P7 Station : Ps 169 + 59

Japan International Cooperation Agency (JICA)

Calculation sheet
Date
Nos of sheet
Prepared by

II.6 Combined load

II.6.1 Normal condition

Case - I, High Water Level

Length	L =	12.00	m
Width of footing	W3 =	5.10	m
Thick of footing	H3 =	1.00	m
Item	V load/m ²	H load/m ²	
Super structure V-sp	50.894		
Abutment V-abt	26.815		
Back fill V-bf	44.013		
		Xo	Yo
		2.100	
		2.519	
			Vxo
			106.877
			70.202
			110.869
Active earth pressure			
Surcharge load Pq		1.693	2.850
Wet soil triangle load Pw		2.393	3.737
Sat.soil rectangular load Ps1		1.836	1.378
triangle load Ps2		2.254	0.918
Water Pw in		(3.795)	0.918
Pw out		4.667	0.918
Pu		(14.051)	2.550
Up life			
Total		121.722	287.948
		(5.213)	(16.976)
X	Vo.Xo		Ho.Yo
	-----		-----
	Vo tot		Ho tot
	287.948		(16.976)
	-----		-----
	121.722		(5.213)
V	V/m ² x L		
	121.72 x 12.00		
	1,460.66 ton		
H	H/m ² x L		
	(5.21) x 12.00		
	(62.56) ton		
M	V x (X - (0.5 x W))		- H x (Y - tb)
	0.5 x W		
	= 0.50 x 5.10		
	= 2.55 m		
	X - (0.5 x W)		
	= 2.37 - 2.55		
	= (0.18) m		
	Y - tb		
	= 3.258 - 1.00		
	= 2.26 m		
	H x (Y - tb)		
	= (62.56) x 2.26		
	= (141.15) k.m		
	V x (X - (0.5 x W))		- H x (Y - tb)
	= 1,460.66 x (0.18) - (141.15)		
	= (269.32) - (141.15)		
	= (128.17) k.m		

id - an

cid - an

II.6.2 Normal condition Case - II, Low Water Level						
Item	Vo/m'	Ho/m'	Xo	Yo	Vo.Xo	Ho.Yo
Super structure	50.894		2.100		106.877	
Abutment	26.815		2.618		70.202	
Back fill	34.723		2.881		100.037	
Active earth pressure						
Surcharge load		1.693		2.850		4.825
triangle load		8.926		1.900		16.959
rectangular load						
Hsat-1						
triangle load						
Hsat-2						
Water						
Up life						
Total	112.432	10.619			277.116	21.784
Wet soil						
Sat.soil						
X'	Vo.Xo		Y'	=	Ho.Yo	
	-----				-----	
	Vo tot				Ho tot	
	277.116				21.784	
	-----				-----	
	112.432				10.619	
V	V/m' x L					m
	= 112.43 x 12.00					
	= 1,349.16 ton					
H	H/m' x L					
	= 10.82 x 12.00					
	= 127.43 ton					
M	V x (X-(0.5 x W))			-(H x (Y-tb))		
	0.5 x W			= 0.500 x 5.10		
				= 2.55 m		
	(X -(0.5xW))			= 2.465 - 2.55		
				= (0.09)m		
	(Y -tb)			= 2.051 - 1.00		
				= 1.05 m		
	H x (Y - tb)			= 127.43 x 1.05		
				= 133.99 m		
	V x (X-(0.5 x W))			-(H x (Y-tb))		
	= 1,349.16 x (0.09) -			133.99		
	= (115.03) - 133.99					
	= (249.01)t.m					

Id - an

cid - an

PRESTRESSED CONCRETE BRIDGE COMPUTATION

CHANNEL PERCUT RIVER

Br - P 2

Station : Pe 169 + 59

span
40.00

III FOUNDATION

Pile for Abutment

- III.1 Characteristic of pile
 - III.1.1 Normal condition
 - III.1.2 Earthquake condition
- III.2 Axial bearing load allowance for a pile
- III.3 Spring constant of a pile
 - III.3.1 Normal condition
 - III.3.2 Earthquake condition
- III.4 Displacement of a head pile
 - III.4.1 Normal condition
 - III.4.2 Earthquake condition
- III.5 Coefficient displacement of footing
 - III.5.1 Case - I, Normal condition (HWL)
 - III.5.2 Case - II, Normal condition (LWL)
 - III.5.3 Case - III, Earthquake condition (HWL)
 - III.5.4 Case - IV, Earthquake condition (LWL)
- III.6 Forces at the i-th pile head
 - III.6.1 Case - I, Normal condition (HWL)
 - III.6.2 Case - II, Normal condition (LWL)
 - III.6.3 Case - III, Earthquake condition (HWL)
 - III.6.4 Case - IV, Earthquake condition (LWL)
- III.7 Control stresses on a head pile
 - III.7.1 Normal condition
 - III.7.2 Earthquake condition
- III.8 Combined load at Abutment section
 - III.8.1 Case - I, Normal condition (HWL)
 - III.8.2 Case - II, Normal condition (LWL)
 - III.8.3 Case - III, Earthquake condition (HWL)
 - III.8.4 Case - IV, Earthquake condition (LWL)
- III.9 Resume Load on Abutment Section
- III.10 Reinforcing steel bar

Project : Medan Flood Control	Japan International	Calculation sheet
Channel : Percut River	Cooperation Agency	
Structure : Br. P - 5	(JICA)	
Station : PE 137 + 39 m		
	Date	
	Nos of sheet	
	Prepared by	

III.1.2 Earthquake condition

- t' = thick of steel pipe corrosion.
- do = outer diameter of steel pipe
- t = thick of steel pipe after corrosion
- di = Inner diameter of steel pipe
- Beta = characteristic value of pile
- d = outside diameter of pile before corrosion
- de = outside diameter of pile after corrosion
- delta = the standard amount of displacement (cm)
- k = coefficient of horizontal subgrade reaction (kg/cm²)
- Eo = modulus of deformation of the ground obtained from table 3.5

Inertia
 Ic = Inertia concrete
 Is = Inertia steel

Modulus elasticity
 Es = pile/steel
 Ec = concrete

thick of corrosion = t'
 normal thick = te

de effective
 do = 2 x te

di
 (de - (2 x t)) cm

de ^ 4
 di ^ 4
 de ^ 4 - di ^ 4

Beta
 = (-----)^0.25
 4.EI

kho
 Alpha
 Eo

Temporary load
 Ec (K-225)
 Es

1/n

lxc	=	@pi64 x de ^ 4 x 1/n	2,116,967	x	0.20359
lxs	=	21,156 cm ^ 4			
lxtot	=	@pi64 x (de ^ 4 - di ^ 4)	382,878		
EI	=	18,794 cm ⁴			
4.EI	=	lxc + lxs	18,794		
kho	=	39,950 cm ²			
Beta	=	1,897,200 x	39,950		
Alpha	=	75,793,769,494 kg/cm ²			
Eo	=	303,175,077,578 kg/cm ²	75,793,769,494		
1/n	=	1/30			
Ec	=	0.03 x	Alpha	x	Eo
Es	=	9.33 kg/cm ³	1	x	280
(1)-----equation					
kh	=	kho x {(Bh)/30} ^ -0.75			
Bh	=	@sqrt (de / Beta)			
de	=	38.1442			
Beta	=	0.004771			7,995.58
Bh	=	89.42			----->>> Beta find by trial
Bh/30	=	89.42			
(Bh/30) ^ -0.75	=	30.00			2.98
kh	=	0.44			
	=	9.33 x			0.44
	=	4.1144			
(2)-----equation					
Beta	=	kh x de			
kh	=	Beta ^ 4 x 4.EI			
kh	=	de			
kh	=	5.18E-10			
	=	38.1442			303,175,077,976
	=	4.1144			
	=	4.1170 kg/cm ³			
By trial method, kh left side equal to kh right side					
kh	=	4,116.97 ton / m ³			
Beta	=	0.004771 cm ⁻¹			
Beta	=	0.477066 m ⁻¹			
kh.D	=	4.12 x			38.14
Allowance Horizontal Force	=	157.04 kg/cm ²			
Han	=	kh.D			delta
Han	=	Beta			1.00
	=	157.04			
	=	0.00			
	=	32,918 kg			
	=	32,918 ton			

Project : Medan Flood Control		Japan International Cooperation Agency (JICA)		Calculation sheet	
Channel : Percut River				Date	
Structure: Br - P7				Nos of sheet	
Station : PB 169 + 59				Prepared by	

III.2 Axial Bearing Load Allowable for a pile

do

di

t

te

de

re

di

de²

di²

de²-di²

= outer diameter of a pile (cm)

= inner diameter of a pile (cm)

= normal thick

= effective thick

= corrosion thick

= dia. effective

= area of a pile (cm²)

= Length of pile

= 40.64 cm

= 1.031 cm

= 0.10 x t

= 0.10

= 0.1031 cm

= t - t'

= 1.0310

= 0.9279 cm

= do - 2 x te

= 40.64

= 38.78 cm

= 0.1939 m

= (de - (2 x te))

= 38.78

= 36.93 cm

= 1504.21 cm²

= 1363.7067 cm²

= 1504.2142

= 140.5074 cm²

= 0.0141 m²

= 0.25 x

= 0.25 x

= 110.35 cm²

= 0.0110 m²

Steel

Ep

Concrete K225

Ec

1/n

= 1,897,200 kg/cm²

= 18,972,000 t/m²

= 257,495 kg/cm²

= 2,574,950 t/m²

= Ec

di-transf

dt

dt²

de²-dt²

= 36.93 x

= 5.0121 cm

= 25.1207 cm²

= 1504.21

= 1479.09

0.1357

0.1357

25.12

Ap2	=	@pi/64 x (de ² - (di ² - 2))	3.14	x	
	=	1.16168 cm ²			
	=	0.1162 m ²			
Ap	=	Ap1 +	Ap2		
	=	0.0110 +	0.1162		
	=	0.1272 m ²			
	Bottom footing elv.	=	13.800		
	Bottom pile elv.	=	(8.916)		
	Depth of pile	=	22.72		
	say	=	24.00 m		
Lp	=	24.00 m			
N1	=	50.00			
N	=	40.00			

N - value of embedded									
N-value	top	bot.	Elv.	top	bot.	Elv.	avg depth	tot depth	tot N-value
27.00	27.00	13.084	12.084	1.00	1.00	1.00	1.00	1.00	27.00
27.00	14.00	12.084	11.084	1.00	2.00	2.00	2.00	2.00	27.00
14.00	14.00	11.084	10.084	1.00	3.00	3.00	3.00	3.00	14.00
14.00	25.00	10.084	9.084	1.00	4.00	4.00	4.00	4.00	14.00
25.00	22.00	9.084	8.084	1.00	5.00	5.00	5.00	5.00	25.00
22.00	12.00	8.084	7.084	1.00	6.00	6.00	6.00	6.00	22.00
12.00	15.00	7.084	6.084	1.00	7.00	7.00	7.00	7.00	12.00
13.00	9.00	6.084	5.084	1.00	8.00	8.00	8.00	8.00	13.00
9.00	12.00	5.084	4.084	1.00	9.00	9.00	9.00	9.00	9.00
12.00	21.00	4.084	3.084	1.00	10.00	10.00	10.00	10.00	12.00
21.00	24.00	3.084	2.084	1.00	11.00	11.00	11.00	11.00	21.00
24.00	26.00	2.084	1.084	1.00	12.00	12.00	12.00	12.00	24.00
26.00	31.00	1.084	0.084	1.00	13.00	13.00	13.00	13.00	26.00
31.00	29.00	0.084	(0.916)	1.00	14.00	14.00	14.00	14.00	31.00
29.00	33.00	(0.916)	(1.916)	1.00	15.00	15.00	15.00	15.00	29.00
33.00	36.00	(1.916)	(2.916)	1.00	16.00	16.00	16.00	16.00	33.00
36.00	50.00	(2.916)	(3.916)	1.00	17.00	17.00	17.00	17.00	36.00
50.00	50.00	(3.916)	(4.916)	1.00	18.00	18.00	18.00	18.00	50.00
50.00	24.00	(4.916)	(5.916)	1.00	19.00	19.00	19.00	19.00	50.00
24.00	26.00	(5.916)	(6.916)	1.00	20.00	20.00	20.00	20.00	24.00
26.00	47.00	(6.916)	(7.916)	1.00	21.00	21.00	21.00	21.00	26.00
47.00	50.00	(7.916)	(8.916)	1.00	22.00	22.00	22.00	22.00	47.00
50.00	50.00	(8.916)	(9.916)	1.00	23.00	23.00	23.00	23.00	50.00
50.00	50.00	(9.916)	(10.916)	1.00	24.00	24.00	24.00	24.00	50.00
N - value average		sum	685.50	:	24.00	:	685.50	:	28.56

Material is fine sand									
N	=	N1 + N2							
N2'	=	mean N-value in the range from the tip of the pile to 4 x D above.							
N2'	=	50.00 +	50.00	=	50.00				
N2'	=	50.00							
N	=	50.00 +	50.00	=	100.00				

Raq	=	1.00	x(	198.62	-	5.67	) +	5.67	-	2.14

	=	2.00								
	=	0.50	x(	192.94	) +	3.53				
	=	96.47	+	3.53						
	=	100.00	ton/haos							

Case		V ton	H ton	M ton.m	Allowance bearing pile		Nos of pile	
					Vertical	Horizontal	Vertical	Horizontal
		Normal condition						
Case - I	Hwl	1460.88	-27.48	-125.28	67.84	31.66	21.53	-0.87
Case - II	Lwl	1349.40	127.43	-249.00	67.84	31.66	19.89	4.02
		Earthquake condition						
Case - III	Hwl	1460.88	231.31	-74.63	100.00	32.69	14.61	7.08
Case - IV	Lwl	1349.40	377.94	-842.07	100.00	32.69	13.49	11.56

Ru	= Ultimate bearing capacity of the statum thickness with the skin friction power taken into account (m)
f _i	= maximum unit area skin friction power of statum with the skin friction power taken into account (tm ²)
4.D	= 4.00 x 0.3878 = 1.55 m
I/D	= 0.50 x 1.55 = 0.78 m
qd/N	= 0.78 : 0.39 = 2.00
qd	= 8.00
N	= 8.00 x N
x	= 8.00 x 40.00
m	= 320.00 ton/m ²
qdAp	= 320 x 0.1272
t	= 40.71 ton
U	= 2 x @pi x re 2.00 x 3.14 x 0.1939
m	= 1.22 m
Ru	= qd.A + U (total li.ft) 40.71 + 1.22 x 129.60
t	= 198.615 ton
Lc	= depth of concrete which filled in the pile
m	= 8.00 m
Y soil	= 2.00 t/m ³
Ws	= Effective weight of soil which replace by pile
m	= 0.25 x @pi x de ^2 x Lp 0.25 x 3.14 x 0.15 x 24.00
m	= 0.25 x 3.14 x 0.15 x 24.00 x Ysoil 2.00
m	= 5.67 ton
W	= Effective weight of concrete which filled in the pile
m	= 0.25 @pi di ^2 Lc 0.25 x 3.14 x 0.1364 x 8.00
m	= 0.25 x 3.14 x 0.1364 x 8.00 x Yconcrete 2.50
m	= 2.14 ton

Force bearing capacity of a pile in Pier

Permanent load safety factor

Temporary load safety factor

Ran	=	----- Sn	x(	Ru	-	Ws)	+	Ws	-	W
		1.00								
	=	----- 3.00	x(	198.62	-	5.67	)+	5.67	-	2.14
Ran	=	0.33	x(	192.94	)+	3.53				
	=	64.31	+	3.53						
	=	67.84	ton/hrs							

by material of pile

Rp	=	Ap 1 x 15	=	110.35	x	1.400	=	154.496	kg
----	---	-----------	---	--------	---	-------	---	---------	----

Rp = 154.50 ton

67.84 ton

Project : Medan Flood Control		Japan International Cooperation Agency (JICA)		Calculation sheet	
Channel : Percut River				Date	Noe of sheet
Structure : Br. P - 7				Prepared by	
Station : PE 169 + 59 m					
III.3 Spring Constante of steel pipe pile					
III.3.1 Normal condition					
h	=	axial length of pile above the design groung surface			
Beta	=	characteristic value of pile			
k	=	coefficient of horizontal sub grade reaction (kg/cm ²)			
IE	=	flexural rigidity of pile			
K1, K2, K3	=	spring constants			
Kv	=	Axial force (axial spring constant)			
t	=	thick of steel pipe before corrosion (cm)			
t'	=	thick of steel pipe corrosion (cm)			
te	=	thick of steel pipe after corrosion (cm)			
d	=	40.64 cm			
t	=	1.031 cm			
t'	=	0.103 cm			
te	=	1.0310 - 0.1031			
	=	0.9279 cm			
de	=	d - 2 x te			
	=	40.64 - 1.8558			
	=	38.78 cm			
de ²	=	1,504.21 cm ²			
di	=	(de - (2 x te)) cm			
	=	38.78 - 1.8558			
	=	36.93 cm			
di ²	=	1,363.71 cm ²			
de ² - di ²	=	1,504.21 - 1,363.71			
	=	140.51 cm ²			
de ⁴	=	2,262.660 cm ⁴			
di ⁴	=	1,859.696 cm ⁴			
de ⁴ - di ⁴	=	2,262.660 - 1,859.696			
	=	402.964 cm ⁴			
Modulus Elasticity					
Permanent load					
Ec (K-225)	=	257,495 kg/cm ²			
Es	=	1,897,200 kg/cm ²			
1/n	=	----- = 0.1357			
		1,897,200			
dt	=	di x 1/n			
	=	36.9284 x 0.1357			
	=	5.0121 cm			
dt ²	=	25.1207 cm ²			
Ixc	=	@pi/64 x (di ⁴)			
	=	0.0491 x 631			
	=	30.98 cm ⁴			
Ixc	=	@pi/64 x (de ⁴ - di ⁴)			
	=	0.04909 x 402.964			
	=	19.780 cm ⁴			
I tot	=	Ixc + Ixc			
	=	30.9767 + 19.780.46			
	=	19.811.44 cm ⁴			
	=	0.000198114 m ⁴			

ks-n

As1	=	1/4 x 3.14 x dt^2	0.25 x	3.14 x	25.12
	=		19.73 cm2		
As2	=	1/4 x 3.14 (de^2 - di^2)	0.25 x	3.14 x	140.51
	=		110.35 cm2		
A tot	=	As1 + As2			
	=	19.73 + 110.35			
	=	130.08 cm2			
	=	0.013008406 m2			
Lp	=	24.00 m			

Kv	=	a	x	A tot	x	Ep
				Lp		

a	=	0.027 x	de		0.2
	=	0.027 x	2400.00		0.2
	=		38.7842		
	=	1.67078 +	0.2000		
	=	1.8708			

Spring constants on axial force = Kv

Kv	=	a	x	A tot	x	Ep
				Lp		
	=	1.87 x	130.08 x	1.897,200		
	=		2.400			
	=	1.87 x	102.831			
	=	192,375.39 kg/cm				
	=	19,238 ton/m				
EI	=	1.897,200 x	19.811			
	=	3.76E+10 kg.cm2				
	=	3.759 tm2				
2EI	=	2.00 x	3.76E+03			
	=	7.517 tm2				
4EI	=	4 x	3.759			
	=	15.035 tm2				
Beta	=	0.00503058 cm^-1				
Beta	=	0.503058 m^-1				

Spring constants K1, K2 and K3

K1	=	4.EI.Beta^3			
	=	1.50E+04 x	0.127307		
K2=K3	=	1.914 t/m			
	=	2.EI.Beta^2			
	=	7.517 x	0.253067		
	=	1.902 t/rad			
K4	=	2.EI.Beta			
	=	7.51725 x	0.503058		
	=	3.782 tm/rad			

ks-n

Project : Medan Flood Control		Japan International Cooperation Agency (JICA)		Calculation sheet	
Channel : Percut River				Date	Nos of sheet
Structure : Br. P - 7				Prepared by	
Station : PE 169 + 59 m					

III.5.2 Earthquake condition	
h	= axial length of pile above the design ground surface
Beta	= characteristic value of pile
k	= coefficient of horizontal sub grade reaction (kg/cm ²)
IE	= flexural rigidity of pile
K1,K2,K3&	= spring constante
Kv	= Axial force (axial spring constante)
t	= thick of steel pipe before corrosion (cm)
t'	= thick of steel pipe corrosion (cm)
te	= thick of steel pipe after corrosion (cm)
d	= 40.64 cm
t	= 1.031 cm
t'	= 0.103 cm
te	= 1.0310 - 0.1031
	= 0.9279 cm
de	= d - 2 x te
	= 40.64 - 1.8558
	= 38.78 cm
de ²	= 1.504.21 cm ²
di	= (de - (2 x te) cm
	= 38.78 - 1.8558
	= 36.93 cm
di ²	= 1.363.71 cm ²
de ² - di	= 1.504.21 - 1.363.71
	= 140.51 cm ²
de ⁴	= 2.262.660 cm ⁴
di ⁴	= 1.859.696 cm ⁴
de ⁴ - di ⁴	= 2.262.660 - 1.859.696
	= 402.964 cm ⁴
Modulus Elasticity	
Temporary load	
Ec (K-225	= 386.242 kg/cm ²
Es	= 1.897.200 kg/cm ²
1/n	= 386.242
	= 1.897.200
dt	= di x 1/n
	= 36.9284 x 0.2036
	= 7.5181 cm
dt ²	= 56.5215 cm ²
I1	= @pi/64 x (dt ⁴)
	= 0.0491 x 3.195
	= 156.82 cm ⁴
I2	= @pi/64 x (de ⁴ - di ⁴)
	= 0.04909 x 402.964
	= 19.780 cm ⁴
I tot.	= I1 + I2
	= 157 + 19.780.46
	= 19.937.28 cm ⁴
	= 0.00019 m ⁴

As1	=	1/4 x 3.14 x dt ²			
	=	0.25 x 3.14 x			
	=	44.39 cm ²			
As2	=	1/4 x 3.14 (de ² - dt ²)			
	=	0.25 x 3.14 x			
	=	110.35 cm ²			
A totL	=	As1 + As2			
	=	44.39 + 110.35			
	=	154.75 cm ²			
	=	0.01547 m ²			
Lp	=	24.00 m			
Kv = a x A tot ----- x Ep Lp					
a	=	0.027 x			
	=	Lp			
	=	de			
	=	0.2			
	=	2.400.00			
	=	38.78			
	=	1.67078 + 0.2000			
	=	1.8708			
Spring constante on axial force = Kv					
Kv	=	a x			
	=	Lp			
	=	154.75 x 1.897.200			
	=	2.400			
	=	1.87 x 122.327			
	=	228.847.05 kg/cm			
	=	22.883 ton/m			
EI	=	1.897.200 x			
	=	3.78E+10 kg.cm ²			
	=	3.783 Lm ²			
2.EI	=	2 x 3.78E+03			
	=	7.565 Lm ²			
4.EI	=	4 x 3.783			
	=	15.130 Lm ²			
Beta	=	0.00477 cm ⁻¹			
Beta	=	0.47706 m ⁻¹			
Spring constante K1,K2 and K3					
K1	=	4.EI.Beta ³			
	=	1.51E+04 x 0.108576			
	=	1.643 Lm			
K2=K3	=	2.EI.Beta ²			
	=	7.565 x 0.227591			
	=	1.722 Lm			
K4	=	2.EI.Beta			
	=	7.565.00 x 0.477066			
	=	3.609 Lm/rad			

exp-dict

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Project : Median Flood Control	Japan International Cooperation Agency (JICA)	Calculation sheet
Channel : Perut River		Date
Structure : Br. P - 7		No. of sheet
Station : PE 167 + 59 m		Prepared by

III.4.2 Earthquake condition

$$Axx = \text{tot}(K1 \times \cos^2 \theta + Kv \times \sin^2 \theta)$$

$$Ayy = \text{tot}(Kv - K1) \times \sin \theta \times \cos \theta$$

$$Ayz = \text{tot}(Kv - K1) \times \sin \theta \times \cos \theta$$

$$Axx = \text{tot}(Kv - K1) \times \sin \theta \times \cos \theta - (K2 \times \cos \theta)$$

$$Ayy = \text{tot}(Kv - K1) \times \sin \theta \times \cos \theta - (K2 \times \cos \theta)$$

$$Ayz = \text{tot}(Kv - K1) \times \sin \theta \times \cos \theta$$

$$Axx = \text{tot}(Kv \times \cos^2 \theta + K1 \times \sin^2 \theta) \times \sin \theta \times \cos \theta$$

$$Ayy = \text{tot}(Kv \times \cos^2 \theta + K1 \times \sin^2 \theta) \times \sin \theta \times \cos \theta$$

$$Ayz = \text{tot}(Kv \times \cos^2 \theta + K1 \times \sin^2 \theta) \times \sin \theta \times \cos \theta$$

Nos. of row	(1)	(2)	(3)	(4)
side	L/A	L/A	R/A	R/A
sign	(-)	(-)	(+)	(+)
angle	> 0	0	< 0	< 0
Xi	(1.875)	(0.625)	0.625	1.875
n	9.00	9.00	9.00	9.00
angle degree	0.0000	0.0000	0.0000	5.71
sin θ	0.0000	0.00	0.00	0.10
cos θ	1.0000	1.0000	1.0000	0.9950
sin $\theta \times \cos \theta$	0.0000	0.00	0.00	0.10
sin θ^2	0.0000	0.00	0.00	0.01
cos θ^2	1.0000	1.0000	1.0000	0.9901
Xi θ^2	3.5156	0.3906	0.3906	3.5156
K1	1.643.00	1.643.00	1.643.00	1.643.00
K2	1.722.00	1.722.00	1.722.00	1.722.00
K3	1.722.00	1.722.00	1.722.00	1.722.00
K4	3.609.00	3.609.00	3.609.00	3.609.00
Kv	22.885.00	22.885.00	22.885.00	22.885.00
n.K1	14.787	14.787	14.787	14.787
n.K2	15.498	15.498	15.498	15.498
n.K3	15.498	15.498	15.498	15.498
n.K4	32.481	32.481	32.481	32.481
n.Kv	205.965	205.965	205.965	205.965
n.(K2 + K3)	30.996	30.996	30.996	30.996
Xi.(n.(K2 + K3))	(58.118)	(19.373)	19.373	58.118
Xi.(n.(K2 + K3)).sin θ	0	0	0	5.782
n.(Kv - K1)	191.178	191.178	191.178	191.178
n.(Kv - K1).sin $\theta \cos \theta$	0	0	0	18.927
n.K1.cos θ^2	14.787	14.787	14.787	14.641
n.Kv.sin θ^2	0	0	0	2.039
n.Kv.cos θ^2	205.965	205.965	205.965	203.926
n.K1.sin θ^2	0	0	0	1.46
n.K1.sin $\theta \cos \theta$	0	0	0	1.464
n.Kv.sin $\theta \cos \theta$	0	0	0	20.390
Xi.(n.K1.sin $\theta \cos \theta$)	0	0	0	2.745
Xi.(n.Kv.sin $\theta \cos \theta$)	0	0	0	38.232
n.K2.cos θ	15.498	15.498	15.498	15.421
n.K2.sin θ	0	0	0	1.542
n.K3.sin θ	0	0	0	1.542
Xi.(n.Kv.cos θ^2)	(386.184)	(128.728)	128.728	382.362
Xi.(n.K1.sin θ^2)	0	0	0	274

$Xi^2(n.Kv \cos^2 \theta) =$	724.096	80.455	80.455	716.928
$Xi^2(n.K1 \sin^2 \theta) =$	0	0	0	515
$Xi(n.K2 \sin \theta) =$	0	0	0	2.891
$Xi(n.K3 \sin \theta) =$	0	0	0	0
$Axx = \text{tot}(K1 \times \cos^2 \theta + Kv \times \sin^2 \theta)$				
$n.K1 \cos^2 \theta =$	14.787	14.787	14.787	14.641
$n.Kv \sin^2 \theta =$	0	0	0	2.039
total	14.787	14.787	14.787	16.679
$Ayy = \text{tot}(Kv - K1) \times \sin \theta \times \cos \theta$				
$n.(Kv - K1) \sin \theta \cos \theta =$	0	0	0	18.927
$Ayz =$	0	0	0	18.927
$Ayz =$	0	0	0	18.927
$Axx = \text{tot}(Kv - K1) \times \sin \theta \times \cos \theta - (K2 \times \cos \theta)$				
$Xi(n.(Kv - K1) \sin \theta \cos \theta) =$	0	0	0	35.487
$- n.K2 \cos \theta =$	(15.498)	(15.498)	(15.498)	(15.421)
$Axx =$	(15.498)	(15.498)	(15.498)	20.066
$Axx =$	(15.498)	(15.498)	(15.498)	20.066
$Ayy = \text{tot}(Kv \times \cos^2 \theta + (K1 \times \sin^2 \theta))$				
$n.Kv \cos^2 \theta =$	205.965	205.965	205.965	203.926
$n.K1 \sin^2 \theta =$	0	0	0	1.46
$Ayy =$	205.965	205.965	205.965	204.073
$Ayy = \text{tot}(Kv \times \cos^2 \theta + K1 \times \sin^2 \theta) \times \sin \theta \times \cos \theta$				
$Xi(n.Kv \cos^2 \theta) =$	(386.184)	(128.728)	128.728	382.362
$Xi(n.K1 \sin^2 \theta) =$	0	0	0	274
$n.K2 \sin \theta =$	0	0	0	1.542
$Ayy =$	(386.184)	(128.728)	128.728	384.178
$Ayy =$	(386.184)	(128.728)	128.728	384.178
$Aaa = \text{tot}(Kv \times \cos^2 \theta + K1 \times \sin^2 \theta) \times \sin^2 \theta + (K2 + K3) \times \sin \theta \times \cos \theta$				
$Xi^2(n.Kv \cos^2 \theta) =$	724.096	80.455	80.455	716.928
$Xi^2(n.K1 \sin^2 \theta) =$	0	0	0	515
$n.(K2 + K3) \times \sin \theta =$	32.481	32.481	32.481	32.481
$Aaa =$	756.577	112.936	112.936	751.706
				1.748.155

Project : Median Flood Control	Japan International Cooperation Agency (JICA)	Calculation sheet
Channel : Perai River		Date
Structure : B - P 7		Year of sheet
Station : PE 109 + 59		Prepared by

III.5.2 Normal condition Case - B Low Water Level

	Aux	dy	Aux	dy	Aux	dy	H
	dx	+	dy	+	dx	+	
	Aux	dx	Aux	dy	Aux	dx	V
	Aux	dx	Aux	dy	Aux	dx	M
70.447 dx +	15.438 dy +	69.445 dx +	127.43 H				
15.438 dx +	69.102 dy +	(1.19) dx +	1.348.40 V				
(69.445) dx +	(1.19) dy +	1.400.785 dx +	(249.00) M				

70.447 dx +	15.438 dy +	69.445 dx +	127.43 H
15.438 dx +	69.102 dy +	(1.19) dx +	1.348.40 V
(69.445) dx +	(1.19) dy +	1.400.785 dx +	(249.00) M

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70.447 dx +	15.438 dy +	69.445 dx +	127.43 H
15.438 dx +	69.102 dy +	(1.19) dx +	1.348.40 V
(69.445) dx +	(1.19) dy +	1.400.785 dx +	(249.00) M

70.447 dx +	15.438 dy +	69.445 dx +	127.43 H
15.438 dx +	69.102 dy +	(1.19) dx +	1.348.40 V
(69.445) dx +	(1.19) dy +	1.400.785 dx +	(249.00) M

70.447 dx +	15.438 dy +	69.445 dx +	127.43 H
15.438 dx +	69.102 dy +	(1.19) dx +	1.348.40 V
(69.445) dx +	(1.19) dy +	1.400.785 dx +	(249.00) M

70.447 dx +	15.438 dy +	69.445 dx +	127.43 H
15.438 dx +	69.102 dy +	(1.19) dx +	1.348.40 V
(69.445) dx +	(1.19) dy +	1.400.785 dx +	(249.00) M

70.447 dx +	15.438 dy +	69.445 dx +	127.43 H
15.438 dx +	69.102 dy +	(1.19) dx +	1.348.40 V
(69.445) dx +	(1.19) dy +	1.400.785 dx +	(249.00) M

70.447 dx +	15.438 dy +	69.445 dx +	127.43 H
15.438 dx +	69.102 dy +	(1.19) dx +	1.348.40 V
(69.445) dx +	(1.19) dy +	1.400.785 dx +	(249.00) M

70.447 dx +	15.438 dy +	69.445 dx +	127.43 H
15.438 dx +	69.102 dy +	(1.19) dx +	1.348.40 V
(69.445) dx +	(1.19) dy +	1.400.785 dx +	(249.00) M

70.447 dx +	15.438 dy +	69.445 dx +	127.43 H
15.438 dx +	69.102 dy +	(1.19) dx +	1.348.40 V
(69.445) dx +	(1.19) dy +	1.400.785 dx +	(249.00) M

70.447 dx +	15.438 dy +	69.445 dx +	127.43 H
15.438 dx +	69.102 dy +	(1.19) dx +	1.348.40 V
(69.445) dx +	(1.19) dy +	1.400.785 dx +	(249.00) M

Project : Median Flood Control	Japan International Cooperation Agency (JICA)	Calculation sheet
Channel : Perai River		Date
Structure : B - P 7		Year of sheet
Station : PE 109 + 59		Prepared by

III.5.3 Normal condition Case - B High Water Level

	Aux	dy	Aux	dy	Aux	dy	H
	dx	+	dy	+	dx	+	
	Aux	dx	Aux	dy	Aux	dx	V
	Aux	dx	Aux	dy	Aux	dx	M
70.447 dx +	15.438 dy +	69.445 dx +	127.43 H				
15.438 dx +	69.102 dy +	(1.19) dx +	1.400.08 V				
(69.445) dx +	(1.19) dy +	1.400.785 dx +	(128.17) M				

70.447 dx +	15.438 dy +	69.445 dx +	127.43 H
15.438 dx +	69.102 dy +	(1.19) dx +	1.400.08 V
(69.445) dx +	(1.19) dy +	1.400.785 dx +	(128.17) M

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70.447 dx +	15.438 dy +	69.445 dx +	127.43 H
15.438 dx +	69.102 dy +	(1.19) dx +	1.400.08 V
(69.445) dx +	(1.19) dy +	1.400.785 dx +	(128.17) M

70.447 dx +	15.438 dy +	69.445 dx +	127.43 H
15.438 dx +	69.102 dy +	(1.19) dx +	1.400.08 V
(69.445) dx +	(1.19) dy +	1.400.785 dx +	(128.17) M

70.447 dx +	15.438 dy +	69.445 dx +	127.43 H
15.438 dx +	69.102 dy +	(1.19) dx +	1.400.08 V
(69.445) dx +	(1.19) dy +	1.400.785 dx +	(128.17) M

70.447 dx +	15.438 dy +	69.445 dx +	127.43 H
15.438 dx +	69.102 dy +	(1.19) dx +	1.400.08 V
(69.445) dx +	(1.19) dy +	1.400.785 dx +	(128.17) M

70.447 dx +	15.438 dy +	69.445 dx +	127.43 H
15.438 dx +	69.102 dy +	(1.19) dx +	1.400.08 V
(69.445) dx +	(1.19) dy +	1.400.785 dx +	(128.17) M

70.447 dx +	15.438 dy +	69.445 dx +	127.43 H
15.438 dx +	69.102 dy +	(1.19) dx +	1.400.08 V
(69.445) dx +	(1.19) dy +	1.400.785 dx +	(128.17) M

70.447 dx +	15.438 dy +	69.445 dx +	127.43 H
15.438 dx +	69.102 dy +	(1.19) dx +	1.400.08 V
(69.445) dx +	(1.19) dy +	1.400.785 dx +	(128.17) M

70.447 dx +	15.438 dy +	69.445 dx +	127.43 H
15.438 dx +	69.102 dy +	(1.19) dx +	1.400.08 V
(69.445) dx +	(1.19) dy +	1.400.785 dx +	(128.17) M

70.447 dx +	15.438 dy +	69.445 dx +	127.43 H
15.438 dx +	69.102 dy +	(1.19) dx +	1.400.08 V
(69.445) dx +	(1.19) dy +	1.400.785 dx +	(128.17) M

mvh-2

၁၁-၁

Project : Medan Flood Control
Channel : Perant River
Structure : Br. P 7
Station : PE 169 + 59

Japan International
Cooperation Agency
(JICA)

Calculation sheet
Date
Nos of sheet
Prepared by

III.8.4 Earthquake condition Case - IV, Low Water Level

$$dd = (dd \cdot \cos \theta) - (dy + \alpha \cdot X) \cdot \sin \theta$$

$$dyl = (dd \cdot \sin \theta) + (dy + \alpha \cdot X) \cdot \cos \theta$$

Row of pile

		(1)	(2)	(3)	(4)	
	Side sign	L/s	L/s	P/s	P/s	
	degree	< 0	> 0	(+)	(-)	
Xi	m	(1.875)	(0.625)	0.625	1.875	
n-nos pile/rows	degree	9.00	9.00	9.00	9.00	
cos e	degree	1.000	0.00	0.00	5.71	
sin e	degree	0.000	1.000	1.000	0.995	
K1	tm/rad	1.643	1.643	1.643	1.643	
K2	tm/rad	1.722	1.722	1.722	1.722	
K3	tm/rad	1.722	1.722	1.722	1.722	
K4	tm	3.609	3.609	3.609	3.609	
Kv	tm	22.885	22.885	22.885	22.885	
dx	m	0.00554645	0.00554645	0.00554645	0.00554645	
dy	m	0.00151298	0.00151298	0.00151298	0.00151298	
alpha	rad	(0.00039838)	(0.00039838)	(0.00039838)	(0.00039838)	
dx cos e	=	0.00555	0.00555	0.00555	0.00552	
dy	=	0.00151	0.00151	0.00151	0.00151	
alpha	=	(0.00040)	(0.00040)	(0.00040)	(0.00040)	
Alpha x Xi	=	0.00075	0.00025	(0.00025)	(0.00075)	
dy + Alpha x Xi	=	0.00228	0.00178	0.00126	0.00077	
(dy + alpha.X).sin e	=	0.00000	0.00000	0.00000	0.00008	
dx	=	0.00555	0.00555	0.00555	0.00544	
dx sin e + (dy + alpha.X).cos e	=	0.00000000	0.00000000	0.00000000	0.0005518	
dy	=	0.0015130	0.0015130	0.0015130	0.0015130	
alpha	=	(0.0003984)	(0.0003984)	(0.0003984)	(0.0003984)	
Alpha x Xi	=	0.0007470	0.0002480	(0.0002480)	(0.0007470)	
dy + Alpha x Xi	=	0.0022599	0.0017820	0.0012640	0.0007660	
(dy + alpha.X).cos e	=	0.0022599	0.0017820	0.0012640	0.0007622	
dx	=	0.0022599	0.0017820	0.0012640	0.0013141	
P n = Kv.dyl	=	51.72	40.32	28.93	30.07	
K1.dxl	=	9.11	9.11	9.11	8.94	
K2.alpha	=	(0.69)	(0.69)	(0.69)	(0.69)	
P.Hi = K1.dxl - K2.alpha	=	9.80	9.80	9.80	9.83	
K3.dxl	=	9.55	9.55	9.55	9.37	
K4.alpha	=	(1.44)	(1.44)	(1.44)	(1.44)	
M s = -K3.dxl + K4.alpha	=	(10.99)	(10.99)	(10.99)	(10.81)	
M s x n	=	(98.90)	(98.90)	(98.90)	(97.29)	
V1 = P.Hi cos e - Phi sin e	=	51.72	40.32	28.93	29.92	
Phi cos e	=	0.00	0.00	0.00	0.96	
Phi sin e	=	51.72	40.32	28.93	28.98	
V1	=	485.47	382.90	269.34	260.68	
V1n	=	0.0	0.0	0.0	3.0	
Hi = P.Hi sin e + Phi cos e	=	9.80	9.80	9.80	9.6	
Phi sin e	=	88.19	88.19	88.19	12.57	
Hi	=	(672.5)	(228.61)	162.71	113.15	
V1n x	=	(98.90)	(98.90)	(98.90)	488.78	
M s n	=	(971.65)	(325.71)	63.91	(97.29)	
M s + M s n + V1n x	=				391.49	

Total forces at footing of Abutment

Ho	=	377.72	ton
Va	=	1,349.40	ton
Mo	=	(842.06)	tm

Total forces at pile head

Hu = Ho		377.72	ton
Vu = Va		1,349.40	ton
Mu = Mo		(842.06)	tm

mm-4

Project : Median Flood Control Channel : Peralt River Structure : Br. P-5 Station : P+137 + 39 m	Japan International Cooperation Agency (JICA)	Date Nos of sheet Prepared by	Calculation sheet
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III.8.3 Earthquake condition Case -III, High Water Level

$$\text{dd} = (\text{dd} \cdot \sin \theta) - (\text{dy} + \text{alpa} \cdot X) \cdot \sin \theta$$

$$\text{dy} = (\text{dd} \cdot \sin \theta) + (\text{dy} + \text{alpa} \cdot X) \cdot \cos \theta$$

	(1)	(2)	(3)	(4)		
	L/a	L/a	R/a	R/a		
	(-)	(-)	(+)	(+)		
	< 0			> 0		
Xi	m	(1.875)	(0.625)	0.625	1.875	
n-rocs pile/rows		9.00	9.00	9.00	9.00	
°	degree	0.00	0.00	0.00	5.71	
cos °	degree	1.000	1.000	1.000	0.995	
sin °	degree	0.000	0.000	0.000	0.099	
K1	Lm	1.643	1.643	1.643	1.643	
K2	Lm/rad	1.722	1.722	1.722	1.722	
K3	Lm/rad	1.722	1.722	1.722	1.722	
K4	Lm	3.609	3.609	3.609	3.609	
Kv	Lm	22.885	22.885	22.885	22.885	
dx	m	0.00251137	0.00251137	0.00251137	0.00251137	
dy	m	0.00171851	0.00171851	0.00171851	0.00171851	
alpa	rad	(0.00039426)	(0.00039426)	(0.00039426)	(0.00039426)	
dx cos ° - (dy + alpa · X) sin °						
dx cos °		0.00251	0.00251	0.00251	0.00250	
dy		0.00172	0.00172	0.00172	0.00172	
alpa		(0.00039)	(0.00039)	(0.00039)	(0.00039)	
Alpa x Xi		0.00074	0.00025	(0.00025)	(0.00074)	
dy + Alpa x Xi		0.00246	0.00198	0.00147	0.00098	
(dy + alpa · X) sin °		0.00000	0.00000	0.00000	0.00010	
dd		0.00251	0.00251	0.00251	0.00240	
dx sin ° + (dy + alpa · X) cos °						
dx sin °		0.00000000	0.00000000	0.00000000	0.0002499	
dy		0.0017185	0.0017185	0.0017185	0.0017185	
alpa		(0.0003943)	(0.0003943)	(0.0003943)	(0.0003943)	
Alpa x Xi		0.0007362	0.0002464	(0.0002464)	(0.0007362)	
dy + Alpa x Xi		0.0024577	0.0018649	0.0014721	0.0009793	
(dy + alpa · X) cos °		0.0024577	0.0019649	0.0014721	0.0009744	
dy		0.0024577	0.0019649	0.0014721	0.0012243	
P = K1 · dy		56.25	44.97	33.69	28.02	
K1 · dd		4.13	4.13	4.13	3.95	
K2 · alpa		(0.68)	(0.68)	(0.68)	(0.68)	
P = K1 · dd - K2 · alpa		4.81	4.81	4.81	4.02	
K3 · dd		4.32	4.32	4.32	4.14	
K4 · alpa		(1.42)	(1.42)	(1.42)	(1.42)	
M = -K3 · dd + K4 · alpa		(5.75)	(5.75)	(5.75)	(5.56)	
M · X		(51.73)	(51.73)	(51.73)	(50.02)	
V = P · X cos ° - P · X sin °						
P · X cos °		56.25	44.97	33.69	27.88	
P · X sin °		0.00	0.00	0.00	0.46	
V		56.25	44.97	33.69	27.42	
V · X		506.21	404.71	303.20	246.77	
H = P · X sin ° + P · X cos °						
P · X sin °		0.00	0.00	0.00	2.8	
P · X cos °		4.8	4.8	4.8	4.6	
H		4.81	4.81	4.81	7.39	
H · X		43.25	43.25	43.25	66.50	
V · X		(949.14)	(252.94)	189.50	462.69	
M · X		(51.73)	(51.73)	(51.73)	(50.02)	
M = M · X + V · X		(1,000.87)	(304.67)	137.77	412.66	
Total forces at pile head						
H _{tot}	=	196.24	ton	H _{tot} = H _o	196.24	ton
V _{tot}	=	1,460.88	ton	V _{tot} = V _o	1,460.88	ton
M _{tot}	=	(755.10)	t·m	M _{tot} = M _o	(755.10)	t·m

Total forces at pile head

H _{tot}	=	196.24	ton	H _{tot} = H _o	196.24	ton
V _{tot}	=	1,460.88	ton	V _{tot} = V _o	1,460.88	ton
M _{tot}	=	(755.10)	t·m	M _{tot} = M _o	(755.10)	t·m

mm-3

Project : Medan Flood Control Channel : Perot River Structure : Br. P 7 Station : P+167 + 59		Japan International Cooperation Agency (JICA)		Calculation sheet	
				Date	Nos of sheet
				Prepared by	
III.7 Control stresses on pile					
III.7.1 Normal Condition					
Case - I High Water Level					
Depth embedded of pile	VI	=	24.00 m		
Axial load	VI	=	45,680 ton		
		=	45,680 kg		
Moment	MI	=	2,250 t.m		
		=	225,000 kg.cm		
diameter	do	=	40.00 cm		
outer radius	ro	=	20.00 cm		
M'	Mx	+	VI x ro		
		=	225,000 + 45,680 x 20.00		
		=	225,000 + 913,600		
		=	1,138,600 kg.cm		
eccentricities	e	=	Mx		
		=	VI		
		=	225,000 kg.cm		
		=	45,680 kg		
		=	4.93 cm		
concrete covering	a'	=	2.50 cm		
	t	=	1.03 cm		
Inner diameter	di	=	40.00 - 2.06		
		=	37.94		
Inner radius	ri	=	18.97 cm		
	ro^2	=	400.00		
	ri^2	=	359.86		
	ro^2-ri^2	=	40.00 - 359.86		
	ro^2-ri^2	=	40.14 cm2		
area of steel bar	As	=	8 x 16		
		=	8.00 x 20106 = 16.08 cm2		
	As	=	16.08 cm2		
Area of bar, Y bar	Ys	=	Area of bar		
		=	Area of bar		
		=	16.08 x 18.97		
		=	16.08		
		=	305.13		
		=	16.08		
		=	1,897,200		
		=	257,495		
		=	7.37		

tot. Area of steel bar

@pi.ro^2

16.08

3.14 x 400.00

16.08

1,256.64

0.01280

7.37 x 0.01280

1.54810

ri

ro

4.93 cm

4.93

20.00

0.2463

pi/4 - (5/12 - 1/6 cos^2 pi) sin pi cos pi + (n.@pi.p/2)(alpha)^2

sin pi/2(2 + cos^2 pi) - pi cos pi - n.@pi.p cos pi

pi = 169.1261 degree

pi = 2.9518 rad.

pi/4 = A

n

alpha

alpha^2

cos pi

sin pi

cos^2 pi

1/6 cos^2 pi

(5/12 - 1/6 cos^2 pi)

sin pi cos pi

(5/12 - 1/6 cos^2 pi) x sin pi cos pi = B

(n.@pi.p)

(n.@pi.p)/2

(n.@pi.p)/2 x (alpha)^2 = C

sum

A - B + C

16.08

3.14 x 400.00

16.08

1,256.64

0.01280

7.37 x 0.01280

1.54810

ri

ro

4.93 cm

4.93

20.00

0.2463

pi/4 - (5/12 - 1/6 cos^2 pi) sin pi cos pi + (n.@pi.p/2)(alpha)^2

sin pi/2(2 + cos^2 pi) - pi cos pi - n.@pi.p cos pi

pi = 169.1261 degree

pi = 2.9518 rad.

pi/4 = A

n

alpha

alpha^2

cos pi

sin pi

cos^2 pi

1/6 cos^2 pi

(5/12 - 1/6 cos^2 pi)

sin pi cos pi

(5/12 - 1/6 cos^2 pi) x sin pi cos pi = B

(n.@pi.p)

(n.@pi.p)/2

(n.@pi.p)/2 x (alpha)^2 = C

sum

A - B + C

16.08

3.14 x 400.00

16.08

1,256.64

0.01280

7.37 x 0.01280

1.54810

ri

ro

4.93 cm

4.93

20.00

0.2463

pi/4 - (5/12 - 1/6 cos^2 pi) sin pi cos pi + (n.@pi.p/2)(alpha)^2

sin pi/2(2 + cos^2 pi) - pi cos pi - n.@pi.p cos pi

pi = 169.1261 degree

pi = 2.9518 rad.

pi/4 = A

n

alpha

alpha^2

cos pi

sin pi

cos^2 pi

1/6 cos^2 pi

(5/12 - 1/6 cos^2 pi)

sin pi cos pi

(5/12 - 1/6 cos^2 pi) x sin pi cos pi = B

(n.@pi.p)

(n.@pi.p)/2

(n.@pi.p)/2 x (alpha)^2 = C

sum

A - B + C

16.08

3.14 x 400.00

16.08

1,256.64

0.01280

7.37 x 0.01280

1.54810

ri

ro

4.93 cm

4.93

20.00

0.2463

pi/4 - (5/12 - 1/6 cos^2 pi) sin pi cos pi + (n.@pi.p/2)(alpha)^2

sin pi/2(2 + cos^2 pi) - pi cos pi - n.@pi.p cos pi

pi = 169.1261 degree

pi = 2.9518 rad.

pi/4 = A

n

alpha

alpha^2

cos pi

sin pi

cos^2 pi

1/6 cos^2 pi

(5/12 - 1/6 cos^2 pi)

sin pi cos pi

(5/12 - 1/6 cos^2 pi) x sin pi cos pi = B

(n.@pi.p)

(n.@pi.p)/2

(n.@pi.p)/2 x (alpha)^2 = C

sum

A - B + C

16.08

3.14 x 400.00

16.08

1,256.64

0.01280

7.37 x 0.01280

1.54810

ri

ro

4.93 cm

4.93

20.00

0.2463

pi/4 - (5/12 - 1/6 cos^2 pi) sin pi cos pi + (n.@pi.p/2)(alpha)^2

sin pi/2(2 + cos^2 pi) - pi cos pi - n.@pi.p cos pi

pi = 169.1261 degree

pi = 2.9518 rad.

pi/4 = A

n

alpha

alpha^2

cos pi

sin pi

cos^2 pi

1/6 cos^2 pi

(5/12 - 1/6 cos^2 pi)

sin pi cos pi

(5/12 - 1/6 cos^2 pi) x sin pi cos pi = B

(n.@pi.p)

(n.@pi.p)/2

(n.@pi.p)/2 x (alpha)^2 = C

sum

A - B + C

16.08

3.14 x 400.00

16.08

1,256.64

0.01280

7.37 x 0.01280

1.54810

ri

ro

4.93 cm

4.93

20.00

0.2463

pi/4 - (5/12 - 1/6 cos^2 pi) sin pi cos pi + (n.@pi.p/2)(alpha)^2

sin pi/2(2 + cos^2 pi) - pi cos pi - n.@pi.p cos pi

pi = 169.1261 degree

pi = 2.9518 rad.

pi/4 = A

n

alpha

alpha^2

cos pi

sin pi

cos^2 pi

1/6 cos^2 pi

(5/12 - 1/6 cos^2 pi)

sin pi cos pi

(5/12 - 1/6 cos^2 pi) x sin pi cos pi = B

(n.@pi.p)

(n.@pi.p)/2

(n.@pi.p)/2 x (alpha)^2 = C

sum

A - B + C

16.08

3.14 x 400.00

16.08

1,256.64

0.01280

7.37 x 0.01280

1.54810

ri

ro

4.93 cm

4.93

20.00

0.2463

pi/4 - (5/12 - 1/6 cos^2 pi) sin pi cos pi + (n.@pi.p/2)(alpha)^2

sin pi/2(2 + cos^2 pi) - pi cos pi - n.@pi.p cos pi

pi = 169.1261 degree

pi = 2.9518 rad.

pi/4 = A

n

alpha

alpha^2

cos pi

sin pi

cos^2 pi

1/6 cos^2 pi

(5/12 - 1/6 cos^2 pi)

sin pi cos pi

(5/12 - 1/6 cos^2 pi) x sin pi cos pi = B

(n.@pi.p)

Project : Medan Flood Control Channel : Perant River Structure : Br. P 7 Station : Pe 167 + 59		Japas International Cooperation Agency (JICA)	Calculation sheet	
			Date	Nos of sheet
			Prepared by	
III.7 Control stresses on pile				
III.7.2 Earthquake Condition				
Case - III High Water Level				
Depth embedded of pile	Vi	=	27.00 m	
Asial load		=	56,250 ton	
		=	56,250 kg	
Moment	M	=	5,750 Lm	
		=	575,000 kg.cm	
diameter	do	=	40.00 cm	
outer radius	ro	=	20.00 cm	
		=	Mx +	Vi x ro
		=	575,000 +	56,250 x 20.00
		=	575,000 +	1,125,000
		=	1,700,000	kg.cm
eccentricities	e	=	Mx	
		=	Vi	
concrete covering	a'	=	575,000 kg.cm	
		=	56,250 kg	
		=	10.22 cm	
inner diameter	di	=	2.50 cm	
inner radius	ri	=	1.03 cm	
		=	40.00 -	2.06
		=	37.94	
		=	18.97 cm	
		=	400.00	
		=	359.86	
		=	400.00 -	359.86
		=	40.14 cm	
area of steel bar	As	=	8.00 x 2.0106 =	16.08 cm ²
		=	16.08	
		=	305.13	
		=	16.08	
		=	1,897,200	
		=	257,495	
		=	7.37	

C =		1 - cos pi	
2 sin ^ 3 pi/3 - pi cos pi + sin pi cos ^ 2 pi + pi/4 - sin pi cos pi/4 - sin ^ 3 pi cos pi/6 @ pi.np.(alpha ^ 2/2 - cos pi)			
Cos pi	=	(0.982)	
sin pi	=	0.189	
sin ^ 3 pi	=	0.007	
sin pi cos pi	=	0.189 x	(0.982) =
Cos ^ 2 pi	=	0.964	
1 - cos pi	=	1.000 -	(0.982) =
2 sin ^ 3 pi/3	=	2.000 x	0.007 =
2 sin ^ 3 pi/3 = A	=	0.013	
pi cos pi = B	=	2.932 x	(0.982) =
sin pi cos pi = C	=	0.189 x	0.964 =
pi/4 = D	=	0.738	
Cos pi/4	=	(0.982) :	4.000 =
sin pi cos pi/4 = E	=	0.189 x	(0.246) =
(sin ^ 3 pi cos pi)	=	0.007 x	(0.982) =
(sin ^ 3 pi cos pi)/6 = F	=	(0.007) :	6.000 =
ap	=	1.548	
@ pi.np	=	3.142 x	1.548 =
alpha	=	0.948	
alpha ^ 2	=	0.900	
alpha ^ 2/2	=	0.900 :	2.000 =
alpha ^ 2/2 - cos pi	=	0.450 -	(0.982) =
@ pi.np x alpha ^ 2/2	=	4.864 x	0.450 =
@ pi.np x alpha ^ 2/2 - cos pi = G	=	2.188 -	(0.982) =
sum	=	A - B + C + D - E - F + G	=
			3.170
			7.040
C =		1 - cos pi	
2 sin ^ 3 pi/3 - pi cos pi + sin pi cos ^ 2 pi + pi/4 - sin pi cos pi/4 - sin ^ 3 pi cos pi/6 @ pi.np.(alpha ^ 2/2 - cos pi)			
C	=	1.982045	
S	=	7.040	
S	=	alpha + cos pi	
	=	C x	1 - cos pi
	=	0.282 x	0.948 +
	=	0.282 x	1.982
	=	(0.005)	(0.017)
M'	=	1,138,600	kg.cm
ro	=	20.00	cm
stress on concrete = fc	=		
fc	=	M'	
	=	ro ^ 3	
	=	1,138,600	
	=	0.282	
	=	8,000	
	=	142,325 x	0.282
	=	40.07	kg/cm ²
stress on steel = fs	=		
fs	=	M'	
	=	ro ^ 3	
	=	1,138,600	
	=	(0.005) x	
	=	8,000	
	=	142,325 x	(0.005) x
	=	(5) kg/cm ²	

total area of steel bar		
P	@pi.ro^2	
	16.08	
	3.14 x 400.00	
	16.08	
	1.256.64	
P	0.01280	
s.p	7.37 x 0.01280	
	1.54810	
alpha	18.97	0.95
	20.00	
e	10.22 cm	
e	10.22	0.5111
ro	20.00	
e	pi/4 - (5/12 - 1/6 cos^2 pi) sin pi cos pi + (n.@pi.p/2.(tho))^2	
ro	sin pi/2 (2 + cos^2 pi) - pi cos pi - n.@pi.p cos pi	
trial value of pi ->>>>>		
pi	136.6281 degree	
pi	2.3881 rad.	
e = pi/4 - (5/12 - 1/6 cos^2 pi) sin pi cos pi + (n.@pi.p/2.(elap))^2		
pi/4 = A	2.3881	4.00 = 0.60
n	7.37	
alpha	0.95 cm	
alpha^2	0.90 cm^2	
cos pi	(0.729281)	
sin pi	0.684215	
cos^2 pi	0.531850	
1/6 cos^2 pi	0.166667	0.531850 = 0.08664
(5/12 - 1/6 cos^2 pi)	0.416667	0.088642 = 0.328025
sin pi cos pi	0.684215 x (0.729281)	(0.50)
(5/12 - 1/6 cos^2 pi) x sin pi cos pi = B	0.32802 x (0.72928)	(0.23922)
(n.@pi.p)	1.55 x 3.14 = 4.86	
(n.@pi.p/2	4.86 : 2.00 = 2.43	
(n.@pi.p/2 x elap)^2 = C	2.43 x 0.90	2.19
sum A - B + C	0.60 - (0.24) + 2.19	3.02
ro = sin pi/2 (2 + cos^2 pi) - pi cos pi - n.@pi.p cos pi		
(sin pi/2	0.228	
(2 + cos^2 pi)	2.000 +	0.532 = 2.532
sin pi/2 x (2 + cos^2 pi)	0.228 x 2.532	0.577
pi cos pi	2.388 x (0.729)	(1.742)
n.@pi.p	1.548 x 3.142	4.864
n.@pi.p cos pi	4.864 x (0.729)	3.547
		5.866
e	3.02	
ro	5.87	pi = 136.83 rad
e		e
ro	0.5155	0.5111

C =		
2 sin^3 pi/2 - pi cos pi + sin pi cos^2 pi + pi/4 - sin pi cos pi/4 - sin^3 pi cos pi/6 @pi.ap (elap^2/2 - cos pi)		1 - cos pi
cos pi	(0.729)	
sin pi	0.684	
sin^3 pi	0.320	
sin pi cos pi	0.684 x	(0.729) = (0.499)
cos^2 pi	0.532	
1 - cos pi	1.000 -	(0.729) = 1.729
2 sin^3 pi/2	2.000 x	0.320 = 0.641
2 sin^3 pi/2 = A	0.641 :	3.000
pi cos pi = B	2.388 x	(0.729)
sin pi cos^2 pi = C	0.684 x	0.532
pi/4 = D	0.597	
cos pi/4	(0.729) :	4.000 = (0.182)
sin pi cos pi/4 = E	0.684 x	(0.182)
(sin^3 pi cos pi)	0.320 x	(0.729) = (0.234)
(sin^3 pi cos pi)/6 = F	(0.234) :	6.000
n.p	1.548	
@pi.ap	3.142 x	1.548 = 4.864
alpha	0.948	
alpha^2	0.900	
0.00		
alpha^2/2	0.900 :	2.000 = 0.450
alpha^2/2 - cos pi	0.450 -	(0.729) = 1.179
@pi.ap x alpha^2/2	4.864 x	0.450 = 2.188
@pi.ap x alpha^2/2 - cos pi = G	2.188 -	(0.729)
sum A - B + C + D - E - F + G		2.917
	1 - cos pi	5.997
C =		
2 sin^3 pi/2 - pi cos pi + sin pi cos^2 pi + pi/4 - sin pi cos pi/4 - sin^3 pi cos pi/6 @pi.ap (elap^2/2 - cos pi)		1.729281
C	5.997	0.2837
S	C x	alpha + cos pi
	0.288 x	1 - cos pi
	0.288 x	0.948 + (0.729)
	0.037	1.729
M'	1,700,000	kg cm
ro	20.000	cm
stress on concrete = fc		
fc	M'	C
	ro^3	1,700,000
	8,000	0.288
	212,500 x	0.288
stress on steel = fs	61.28	kg/cm2 > 170 kg/cm2
fs	M'	S x
	ro^3	1,700,000
	8,000	0.037 x
	212,500 x	0.037 x
	57 kg/cm2	7 kg/cm2

Project : Medan Flood Control Channel : Percut River Structure : Br. P - 7 Station : PE.167 + 59 m				Japan International Cooperation Agency (JICA)				Calculation sheet																																																																																						
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III.1.8 Combined load at Abutment section III.1.8.1 Normal Condition Case - 1, High Water Level																																																																																														
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Project : Medan Flood Control

Channel : Percut River

Structure : Br. P - 7

Station : PE 167 + 59 m

Japan International Cooperation Agency (JICA)

Date

Nos of sheet

Prepared by

Calculation sheet

Combined load at Abutment section

Normal Condition

Case - II. Low Water Level

Ka	=	0.297		Ywet	=	1.850	t/m3
Ht	=	6.700	m	Ysat	=	2.000	t/m3
Yw	=	1.000	t/m3	q	=	1.000	t/m2
Yc	=	2.400	t/m3				

Description		Dead load	Line load	Uniform load	Pedestrian load	Total load	Nos of bearing
Left side app		936.42	31.64	343.71	24.00	1335.77	2.00
load per m w		46.821	1.582	17.185	1.200	66.789	10.00

Section 1 - 1

H1 = 2.045 m W1 = 0.300 m

Load	Description						
	Y	x	H	x	W	Weight	
C1	2.40	x	2.045	x	0.30	=	1.472
Pq	q	x	H				
Pa1	1.00	x	2.045			=	2.045
	0.5 x Y	x	H1^2	x	Ka		
	0.5 x	1.85	x	4.18	x	0.297	=
	total						1.149
							1.472
							3.194
							2.091

Section 2 - 2

H1 = 2.045 m W1 = 0.30 m

H2 = 0.500 m W2 = 0.55 m

H3 = 0.500 m W3 = 0.55 m

Load	Description						
	Yc	x	H	x	W	Weight	
C1	2.40	x	2.045	x	0.30	=	1.472
C2	2.40	x	0.500	x	0.55	=	0.660
C3	2.40	x	0.500	x	0.55	=	0.330
E1	q	x	H	x	W		
	1.00	x	2.045	x	0.25	=	0.511
	total						2.974
							0.634

Section 3 - 3

H1 = 2.045 m W1 = 0.300 m H1 + H4 = 2.545 m

H2 = 1.000 m W2 = 1.000 m Pe = 7.023 ton (by earthquake)

H4 = 0.500 m W4 = 0.550 m He = H4 + H5 = 1.000 m

H5 = 0.500 m W5 = 0.550 m

H6 = 0.500 m W6 = 0.200 m

H7 = 0.250 m W7 = 0.200 m

Load	Description						
	W	x	H	x	W	Weight	
C1	H1	=	2.045	m	W1	=	0.300 m
	H2	=	1.000	m	W2	=	1.000 m
	H4	=	0.500	m	W4	=	0.550 m
	H5	=	0.500	m	W5	=	0.550 m
	H6	=	0.500	m	W6	=	0.200 m
	H7	=	0.250	m	W7	=	0.200 m
	H1 + H4						= 2.545 m
	Pe						= 7.023 ton (by earthquake)
	He = H4 + H5						= 1.000 m
Load	Description						
	Y	x	H	x	W	Weight	
C1	2.40	x	2.045	x	0.30	=	1.472
C4	2.40	x	1.000	x	1.00	=	2.400
C5	0.5 x	2.40	x	0.500	x	0.55	= 0.330

Section 6 - 6									
H3.1	=	0.300	m	W1	=	2.30	m		
H3.2	=	1.000	m	W2	=	2.30	m		
H wet	=	2.945	m						
H sat	=	2.755	m						
Load	Description								
	Y	x	H	x	W	Weight			
C3.1	0.5 x	2.40	x	0.300	x	2.30	=	0.828	
C3.2	2.40	x	1.000	x	2.30	=	5.520		
E1(wet)	1.85	x	2.945	x	2.30	=	12.531		
E1(sat)	2.00	x	2.755	x	2.30	=	12.673		
U1(up lift)	1.00	x	2.755	x	2.30	=	6.337		
Rp 1							(45.680)		
Rp 2							(42.750)		
	total						(63.215)		
							(76.514)		
							(18.169)		
							(66.002)		

Project : Medan Flood Control

Channel : Percut River

Structure : Br. P - 7

Station : PE.167 + 59 m

Japan International Cooperation Agency (JICA)

Date

Nos of sheet

Prepared by

Calculation sheet

III.8.3 Earthquake condition

Case - III. High Water Level

Ka = 0.407

Ht = 6.700 m

Yw = 1.000 t/m³

Yc = 2.400 t/m³

Ywet = 1.850 t/m³

Ysat = 2.000 t/m³

q = 0.500 t/m²

Total load (ton)

Description	Dead load	Line load	Uniform load	Pedestrian load	Total Nos of load bearing
Left side span	936.42	31.64	343.71	24.00	1335.77
load per m wid	46.821	1.582	17.185	1.200	66.789

Section 1 - 1

H1 = 2.045 m W1 = 0.300 m

Load	Description			Weight		Vo ton	Ho ton	Xo m	Yo m	M = Vo.Yo t.m
	Y m	X m	H m	Wn t/m ³	Weight ton					
C1	2.40	x	2.045	x	0.30	1.472				
Pq	q	x	H							
Pa1	0.50	x	2.045		1.023		1.023		1.023	1.046
	0.5 x	Y	H1 ~ 2 x	Ka						
	0.5 x	1.85	x	4.18 x	0.407	1.574				
	total					1.472	2.597			1.046

Section 2 - 2

H1 = 2.045 m W1 = 0.30 m

H2 = 0.500 m W2 = 0.55 m

H3 = 0.500 m W3 = 0.55 m

Load	Description			Weight		Vo ton	Ho ton	Xo m	Yo m	M = Vo.Yo t.m
	Yc m	X m	H m	W t/m ³	Weight ton					
C1	2.40	x	2.045	x	0.30	1.472				0.221
C2	2.40	x	0.500	x	0.55	0.660		0.150		0.182
C3	2.40	x	0.500	x	0.55	0.330		0.275		0.091
E1	q	x	H		W			0.275		
	0.50	x	2.045	x	0.25	0.256				0.070
	total					2.718				0.563

Section 3 - 3

H1 = 2.045 m W1 = 0.300 m H1 + H4 = 2.545 m

H2 = 1.000 m W2 = 1.000 m Pe = 7.023 ton (by earthquake)

H4 = 0.500 m W4 = 0.550 m He = H4 + H5 = 1.000 m

H5 = 0.500 m W5 = 0.550 m

H6 = 0.500 m W6 = 0.200 m

H7 = 0.250 m W7 = 0.200 m

Load	Description			Weight		Vo ton	Ho ton	Xo m	Yo m	M t.m
	W m	Y m	H m	W t/m ³	Weight ton					
C1	2.40	x	2.045	x	0.30	1.472				
C4	2.40	x	0.500	x	1.00	1.200				
C5	2.40	x	0.500	x	0.55	0.330				
C2	0.5 x	2.40	x	1.000	x	1.320				

C6	=	0.5 x	2.40 x	0.500 x	0.20 =	0.240	0.240	7.023	1.00	7.023
C7	=				0.20 =	0.060	0.060			
Pe	=							7.023	1.273	1.619
Pq	=								0.958	0.813
Pa	=	0.5 x	1.85 x	2.545 x	0.407 =	66.789	66.789			
Rv	=									
	sum					71.411	9.254			9.455
Section 4 - 4										
Top deck	=	20.516 m								
Hwd elv.	=	17.571 m								
H back fill	=	Ht								
	=	6.70		1.00	=	5.70 m				
Top footin	=	20.516		5.700	=	14.816 m				
H wet soil	=	20.516		17.571	=	2.945 m				
H sat soil	=	5.70		2.945	=	2.755 m				
H1	=	2.045 m	W1	=	0.300 m					
H2	=	3.655 m	W2	=	1.000 m					
H4	=	0.500 m	W4	=	0.550 m					
H5	=	0.500 m	W5	=	0.550 m					
H6	=	0.500 m	W6	=	0.200 m					
H7	=	0.250 m	W7	=	0.200 m					
H wet	=	2.945		3 +	2.755 =	3.737 m				
Load	Description			Weight						
	Y	X	H	Wn	Vo	Ho	Xo	Yo	M = Ho.Yo	
C1	2.40	x	2.05	x	0.30 = 1.472					
C4	2.40	x	0.50	x	1.00 = 1.200					
C5	2.40	x	0.50	x	0.55 = 0.330					
C2	2.40	x	3.66	x	0.55 = 4.825					
C6	2.40	x	0.50	x	0.20 = 0.240					
C7	2.40	x	0.50	x	0.20 = 0.120					
Pe	0.50	x	5.70		7.023				3.655	25.669
Pq1	1.85	x	8.673	x	0.407 = 3.265				2.850	8.123
Pa1	2.00	x	7.590	x	0.407 = 3.089				3.737	12.201
Pa2	2.00	x	2.755	x	0.407 = 2.243				0.918	2.837
Pw	1.00	x	9.333		4.667				1.378	3.089
Pwr	1.00	x	7.590		3.795				1.018	4.752
Rv					66.789				0.918	3.485
	sum				74.976	26.931				60.156
Section 5 - 5										
H1	0.300 m	W1	=	1.80 m						
H2	1.000 m	W2	=	1.80 m						
H wet r	2.945	=	2.045	=	0.90 m					
H sat r	2.755	=								
Load	Description			Weight						
	Y	X	H	W	Vo	Ho	Xo	Yo	M = Vo.Xo	
C1	0.5 x	2.40 x	0.30 x	1.80 =	0.648				0.600	0.389
C2	2.40 x	1.00 x	1.80 x	1.80 =	4.320				0.900	3.888
E1(wet)	1.85 x	0.90 x	1.80 =	2.997					0.900	2.697
E1(sat)	2.00 x	2.755 x	1.80 =	9.918					0.900	8.926
U1(up lift)	1.00 x	2.755 x	1.80 =	(4.959)					0.900	(4.463)
Rp 4				(27.420)					1.175	(32.219)
Rp 3				(33.690)					0.000	(20.781)
	total				(48.186)					

Section 6 - 6									
H3.1	=	0.300 m	W1	=	2.30 m				
H3.2	=	1.000 m	W2	=	2.30 m				
H wet	=	2.945 m							
H sat	=	2.755 m							
Description									
Load	Y	x	H	x	W	=	Weight		
	m		m		t/m3		ton		M = Vo.Xo
C3.1	=	0.5 x	2.40 x	0.900 x	2.30	=	0.828		0.635
C3.2	=	2.40 x	2.40 x	1.000 x	2.30	=	5.520		6.348
El(wet)	=	1.85 x	2.945 x	2.30	=	12.531			14.411
El(sat)	=	2.00 x	2.755 x	2.30	=	12.673			14.574
U1(up lift)	=	1.00 x	2.755 x	2.30	=	6.337			(7.287)
Rp 1							(36.250)		(94.219)
Rp 2							(44.970)		(19.112)
total									(84.651)

Project : Median Flood Control	Japan International Cooperation Agency (JICA)	Calculation sheet
Channel : Percut River		Date
Structure : Br. P - 7		Nos of sheet
Station : PE 167 + 59 m		Prepared by

III.8.4 Earthquake condition Case - IV, Low Water Level

Ka	=	0.407	Ywet	=	1.850 t/m3
Ht	=	6.700 m	Ysat	=	2.000 t/m3
Yw	=	1.000 t/m3	q	=	0.500 t/m2
Ye	=	2.400 t/m3			

Total load (ton)				
Description	Dead load	Uniform load	Pedestrian load	Total Nos of bearing
Left side span	936.42	31.64	343.71	24.00
load per m wid	46.821	1.582	17.185	1.200
				66.789

Section 1 - 1 H1 = 2.045 m W1 = 0.300 m

Load	Description											
	Y	x	H	x	W	=	Weight	Vo	Ho	Xo	Yo	M = Vo.Yo
	m		m		tm ³		ton					
C1	=	2.40	x	2.045	x	0.30	=	1.472	1.472			
Pq	=	q	x	H								
Pa1	=	0.5	x	Y	x	H1 ~ 2	x	Ka	1.023		1.023	1.046
		0.5	x	1.85	x	4.18	x	0.407	=	1.574		
		total							1.472	2.597		1.046

Section 2 - 2

H1 = 2.045 m W1 = 0.30 m
H2 = 0.500 m W2 = 0.55 m
H3 = 0.500 m W3 = 0.55 m

Load	Description					Vo	Ho	Xo	Yo	M = Vo.Yo
	Ye	x	H	x	W	ton	ton	m	m	Lm
C1	=	2.40 x	2.045 x	0.30	=	1.472				
C2	=	2.40 x	0.500 x	0.55	=	0.660				
C3	=	0.5 x	2.40 x	0.500 x	0.55	=	0.330			
E1	=	q	x	H	x	W				
		0.50 x	2.045 x	0.25	=	0.256				
total										
							2.718			
									0.275	0.221
									0.275	0.182
									0.275	0.091
									0.275	0.070
										0.563

Section 3 - 3

H1 = 2.045 m W1 = 0.300 m H1 + H4 = 2.545 m
H2 = 1.000 m W2 = 1.000 m Pe = 7.023 ton (by earthquake)
H4 = 0.500 m W4 = 0.550 m He = H4 + H5 = 1.000 m
H5 = 0.500 m W5 = 0.550 m
H6 = 0.500 m W6 = 0.200 m
H7 = 0.250 m W7 = 0.200 m

Load	Description					Vo	Ho	Xo	Yo	M
	Y	x	H	x	W	ton	ton	m	m	Lm
C1	=	2.40 x	2.045 x	0.30	=	1.472				
C4	=	2.40 x	0.500 x	1.00	=	1.200				
C5	=	0.5 x	2.40 x	0.500 x	0.55	=	0.330			
C7	=	2.40 x	1.000 x	0.55	=	1.320				

C6	=	0.5	x	2.40	x	0.500	x	0.20	=	0.240	0.240	7.023	1.00	7.023
C7	=			2.40	x	0.250	x	0.20	=	0.060	0.060			
Pe	=									7.023		1.273	1.619	
Pq	=			0.50	x	2.545	x		=	1.273		0.958	0.848	0.813
Pa	=	0.5	x	1.85	x	2.545	x	0.407	=	0.958	66.789			
Rv	=									66.789	71.411	9.254		9.455
sum														
Section 4 - 4														
Top deck	=	20.516	m											
Lwl elev.	=	13.461	m											
H back fill	=	Ht	=	H3										
	=	6.70	=	1.00	=	5.70	m							
Top footin	=	20.516	=	5.700	=	14.816	m							
H wet soil	=	20.516	=	14.816	=	5.700	m							
	=	5.70	=	5.7	=	0.00	m							
H1	=	2.045	m	W1	=	0.300	m							
H2	=	3.655	m	W2	=	1.000	m							
H4	=	0.500	m	W4	=	0.550	m							
H5	=	0.500	m	W5	=	0.550	m							
H6	=	0.500	m	W6	=	0.200	m							
H7	=	0.250	m	W7	=	0.200	m							
H wet	=	5.7	:	3 + 9E-16	=	1.900	m							
Description														
Load		Y	x	H	x	W	x	W	x	W	x	W	x	W
		m		m		m		m		m		m		m
C1	=	2.40	x	2.05	x	0.30	=	1.472	1.472					
C4	=	2.40	x	0.50	x	1.00	=	1.200	1.200					
C5	=	0.5	x	2.40	x	0.50	x	0.55	=	0.330	0.330			
C2	=	2.40	x	3.66	x	0.55	=	4.825	4.825					
C6	=	2.40	x	0.50	x	0.20	=	0.240	0.240					
C7	=	0.5	x	2.40	x	0.25	x	0.060	0.060					
Pe	=	0.50	x	5.70				7.023						
Pq1	=	1.85	x	32.490	x	0.407	=	2.850						
Pw1	=	2.00	x	0.000	x	0.407	=	12.232						
Pd1	=	2.00	x	0.000	x	0.407	=	0.000						
Pd2	=	2.00	x	0.000	x	0.407	=	0.000						
Pw4	=	1.00	x	0.090				0.045						
Pwr	=	1.00	x	0.000				0.000						
Rv	=							66.789	66.789					
sum														
Section 5 - 5														
H1	=	0.300	m	W1	=	1.80	m							
H2	=	1.000	m	W2	=	1.80	m							
H wet	=	5.700	=	2.045	=	3.65	m							
H sat	=	0.000	m											
Description														
Load		Y	x	H	x	W	x	W	x	W	x	W	x	W
		m		m		m		m		m		m		m
C1	=	0.5	x	2.40	x	0.30	x	1.80	=	0.648	0.648			
C2	=	2.40	x	1.00	x	1.80	=	4.320	4.320					
Et(wet)	=	1.85	x	3.65	x	1.80	=	12.171	12.171					
Et(sat)	=	2.00	x	0.000	x	1.80	=	0.000	0.000					
Ul(up lift)	=	1.00	x	0.000	x	1.80	=	(0.000)	(0.000)					
Rp 4	=							(28.960)	(28.960)					
Rp 3	=							(28.930)	(28.930)					
total														
ERR														

Project : Medan Flood Control Channel : Percut River Structure : Br - P 7 Station : Pe 167 + 59	Japan International Cooperation Agency (JICA)	Calculation sheet Date Nos of sheet Prepared by
III.9 Resume forces on Abutment Section		
Case - I Normal Condition (H-WL)		
Case	Forces	Unit
Case - I	Moment	tm
	Shear	kg.cm
	Normal	ton
	Normal	kg
Case - II Normal Condition (LWL)		
Case	Forces	Unit
Case - II	Moment	tm
	Shear	kg.cm
	Normal	ton
	Normal	kg
Case - III Earthquake Condition (H-WL)		
Case	Forces	Unit
Case - III	Moment	tm
	Shear	kg.cm
	Normal	ton
	Normal	kg
Case - IV Earthquake Condition (LWL)		
Case	Forces	Unit
Case - IV	Moment	tm
	Shear	kg.cm
	Normal	ton
	Normal	kg

Project : Medan Flood Control Channel : Percut River Structure : Br - P 7 Station : Pe 167 + 59	Japan International Cooperation Agency (JICA)	Calculation sheet Date Nos of sheet Prepared by
III.10 Reinforcing steel bar of Abutment section		
Section - 4		
Item	Description	unit
concrete compressive steel tensile concrete shear transformend Moment Shear Normal dimension	$f_c = K - 225$	kg/cm ²
	f_s	kg/cm ²
	ton without r bar	kg/cm ²
	ton with r bar	kg/cm ²
	$n = E_s/E_c$	
	M	tm
	S	kg.cm
	N	ton
	b	cm
	ht	cm
C	$100 \cdot ht$	cm
	$100 \cdot ht$	cm
	$(W/100 \cdot ht)^2$	cm
	$e_1 = C \cdot [W/(100 \cdot ht)]^2$	cm
	$e_2 = 0.15 \cdot ht$	cm
	$e = e_0 + e_1 + e_2$	cm
	$e_s = e + 1/2 \cdot ht - d'$	cm
	$M' = N \cdot e$	kg.cm
	$n \cdot M'$	kg.cm
	$b \cdot f_s$	kg/cm
Calculation for reinfor - cement bar with steel full working	$n \cdot M'/(b \cdot f_s)$	
	$@ \sqrt{n \cdot M'/(b \cdot f_s)}$	
	$Ca = h \cdot (@ \sqrt{n \cdot M'/(b \cdot f_s)})$ see PBI 1971 N1-2	
	$n \cdot f_c$	
	$e_0 = f_s / (n \cdot f_c)$	
	delta	
	from table lined Ca	
	fine $e > e_0$	
	e'	
	ksi	
	zeta	
	100 rw	
	rw	
	w	
	ea/h	
	from table lined i	
	i.A = w.bh	
	A	
	A' = delta.	

= 1.327 = 10 = 10 x = 13.27 cm > 15.62 cm2 A' - used bar dia. 25 mm @ 12.5 cm, Area @ bar = 4.900 nos of bar = 10 total area = 49.00 cm2 >			
steel bar full working			
tension of bar compressive concrete tensile of bar shear concrete	n.e	kg/cm2	32.84
	fs	kg/cm2	1,400.00
	fc' = fs / (n.e)	kg/cm2	42.83
	fs' = fs / e'	kg/cm2	665.72
7/8.b.h	cm2		7.875.00
Tca = S / (7/8.b.h)	kg/cm2		2.59

Section - 5				
Item	Description	unit	Normal condition	Earth quake condition
			Calculation	Calculation
Allowance stresses				
concrete compressive steel tensile concrete shear transformend Moment	fc = K - 225	kg/cm2	75	75
	fs	kg/cm2	1400	1400
	tau without r bar	kg/cm2	8	8
	tau with r bar	kg/cm2	20	20
Shear	n = Es/Ec	Lm	21	21
	M	kg.cm	28.607	20.781
	S	ton	2,860.700	2,078.100
		kg	60.968	48.186
dimension	b	cm	100	100
	ht	cm	1,300	1,300
	d'	cm	10	10
	h	cm	1,290	1,290
	b.h	cm2	129.000	129.000
	M	kg.cm	2,860.700	2,078.100
	n.M	kg.cm	60,074.700	43,640.100
	b.fs	kg/cm	140.000	140.000
	n.M / (b.fs)		429.11	311.72
	@sqrt(n.M / (b.fs))		20.71	17.66
	Ca = h / @sqrt(n.M / (b.fs)) see PBI 1971 N.I-2		62.274	73.065
	n.fc		1.575	1.575
Calculation for reinfor- cement bar with steel full working	eo = fs / (n.fc)		0.8889	0.8889
	delta		1.00	1.00
	from table fined Ca		12.290	12.290
	fine eo > eo		8.091	8.091
	e'		89.000	89.000
	ksi()		0.110	0.110
	zeta		0.963	0.963
	100 nw		0.688	0.688
	nw		0.00688	0.00688
	w		0.00033	0.00033
	A	cm2	42.26	42.26
	A' =delta.A	cm2	42.26	42.26
A - used bar dia. 25 mm @ 12.5 cm, Area @ bar = 4.900 nos of bar = 10 total area = 49.00 cm2 >				
steel bar full working				
n.e				
tension of bar compressive concrete tensile of bar shear concrete	fs	kg/cm2	1,400.00	169.91
	fc' = fs / (n.e)	kg/cm2	8.24	8.24
	fs' = fs / e'	kg/cm2	15.73	15.73
	7/8.b.h	cm2	112.875.00	0.54
Tca = S / (7/8.b.h)	kg/cm2			

PRESTRESSED CONCRETE BRIDGE COMPUTATION
CHANNEL PERCUT RIVER

Br - P 2

Station : Pe 84 + 28

span		
Left side	Midle	Right side
31.00	40.00	31.00

Width

7.0 m

II SUB STRUCTURE

A. Abutment

II.A.1 Neutral axis of Abutment

II.A.2 Earthquake coefficient

II.A.3 Axial forces on Abutment

II.A.4 Coulumb coefficient

II.A.5 Horizontal force

II.A.5.1 Case - I, Normal condition (HWL)

II.A.5.2 Case - II, Normal condition (LWL)

II.A.5.3 Case - III, Earthquake condition (HWL)

II.A.5.4 Case - IV, Earthquake condition (LWL)

II.A.6 Combined Load on Abutment

II.A.6.1 Case - I, Normal condition (HWL)

II.A.6.2 Case - II, Normal condition (LWL)

II.A.6.3 Case - III, Earthquake condition (HWL)

II.A.6.4 Case - IV, Earthquake condition (LWL)

B P I E R

II.B.1 Neutral axis of Pier

II.B.2 Axial forces on Pier

II.B.3 Combined Load on Pier

II.B.3.1 Case - I, Normal condition (HWL)

II.B.3.2 Case - II, Normal condition (LWL)

II.B.3.3 Case - III, Earthquake condition (HWL)

II.B.3.4 Case - IV, Earthquake condition (LWL)

Project : Median Flood Control

Channel : Percut River

Structure : Br. P-2

Station : Pe 84 + 28

Japan International Cooperation Agency (JICA)

Date

Nos of sheet

Prepared by

II.SUB STRUCTURE

A. ABUTMENT

II.A.1 Neutral axis of Left Side Abutment

Top deck elevation

Bottom footing elevation

depth of abutment = Ht

depth beam

depth slab

depth of beam to deck

depth of beam and deck

depth bearing pad

depth back wall

depth of footing

depth of steam

Depth of back fill

10.851 m

4.351 m

6.30 m

1.60 m

0.245 m

1.845 m

1.800 +

1.845 +

1.000 m

6.300 -

1.945 +

1.945 m

3.355 m

W1 = 0.300 m

W1 = 2.300 m

W2 = 1.000 m

W3 = 1.800 m

W3 = 5.100 m

W4 = 0.550 m

W5 = 0.550 m

W6 = 0.200 m

W7 = 0.200 m

W8 = 0.300 m

W9 = 1.800 m

W1 = 0.300

H1 = 1.945

W2 = 1.800

H2 = 3.355

W3 = 5.100

H3 = 1.000

W4 = 0.550

H4 = 0.500

W5 = 0.550

H5 = 0.250

W6 = 0.200

H6 = 0.500

W7 = 0.200

H7 = 0.250

X1 = W1/2 + W2 + W3

Y1 = H1 - H1/2

X2 = W2/2 + W3

Y2 = H2/2 + H3

X3 = W3/2

Y3 = H3/2

X4 = W4/2 + W2 + W3

Y4 = H2 - H4/2 + H3

X5 = W5/3 + W2 + W3

Y5 = H2 - H4/2 - H5/3 + H3

X6 = W3 - W6/2

Y6 = H2 - H6/2 + H3

X7 = W3 - W7/3

Y7 = H2 - H6/2 - H7/3 + H3

= 0.150 + 1.000 + 1.800

= 0.900 + 1.800

= 1.877 + 1.000

= 2.550

= 0.500

= 1.800 + 1.800 + 1.800

= 0.250 - 0.250 + 0.083 + 1.000

= 1.800 - 0.100

= 1.800 - 0.067

= 0.250 - 0.083 + 1.000

= 0.250 - 0.083 + 1.000

= 0.250 - 0.067 + 1.733

= 0.250 - 0.083 + 1.000

A1

A2

A3

A4

A5

A6

A7

= 2.950 m

= 5.328 m

= 2.700 m

= 2.677 m

= 2.550 m

= 0.500 m

= 3.875 m

= 3.855 m

= 3.783 m

= 4.022 m

= 1.700 m

= 3.186 m

= 1.733 m

= 4.022 m

II-XI

al-xy

A8	W8 = 2.300	X8 = W8/3	+ W2	+ W3	= 3.567 m
	H8 = 0.300	Y8 = H8/3	+ H3		= 1.100 m
A9	W9 = 1.800	X9 = W9/2/3			
	H9 = 0.300	Y9 = H9/3	+ H3		= 1.200 m
			0.100 + 1.000		= 1.100 m
Wing wall					
	Hgw = 1.200 m	H back fill = 5.30 m			
	Ww1 = 3.000	Hw1 = 3.00 m			
	Ww2 = 2.300	Hw2 = 2.30 m			
	Ww3 = 0.700	Hw3 = 0.70 m			
	Ww4 = 2.300	Hw4 = 2.30 m			
	Ww5 = 2.300	Hw5 = 2.30 m			
Ww1	Ww1 = 3.000	Xw1 = Ww1/2	+ W2	+ W3	= 4.300 m
	Hw1 = 3.000	Yw1 = Ht	- Hw1/2		= 4.800 m
Ww2	Ww2 = 2.300	Xw2 = Ww2/2	+ W2	+ W3	= 3.950 m
	Hw2 = 1.000	Yw2 = Ht	- Hw1 - Hw2/2		= 2.800 m
Ww3	Ww3 = 0.700	Xw3 = Ww3	- Ww3/2		= 4.750 m
	Hw3 = 1.000	Yw3 = Ht	- Hw1 - Hw3/2		= 2.967 m
Ww4	Ww4 = 2.300	Xw4 = Ww4/2	+ W2	+ W3	= 3.950 m
	Hw4 = 1.000	Yw4 = Hw4/2	+ Hw5 + H3		= 1.800 m
Ww5	Ww5 = 2.300	Xw5 = Ww5	- Ww5/2/3		= 3.567 m
	Hw5 = 0.300	Yw5 = Hw5/3	+ H3		= 1.100 m
Apr	Wapr = 3.000	Xapr = Wapr/2	+ W2	+ W3	= 4.300 m
	Hapr = 0.200	Yapr = Ht	- H1 + Hapr/2		= 4.455 m
			6.300 - 1.945 + 0.100		
Guide wall					
Gw	Wgw = 3.000	Xgw = Wgw/2	+ W2	+ W3	= 4.300 m
	Hgw = 1.200	Ygw = Ht	+ Hgw/2		= 6.900 m
Asp	Wasp = 3.000	Xasp = Wg/2	+ W2	+ W3	= 4.300 m
	Hasp = 0.050	Yasp = Ht	- Hasp/2		= 6.275 m
			6.300 - 0.025		
Asphalt					
	Ha = 0.05 m				

al-xy

Project : Median Flood Control	Japan International Cooperation Agency (JICA)	Calculation sheet
Channel : Bx P-2		Date
Station : P+84 + 28		No. of sheet
		Prepared by

a. Superstructure

II.A.3 Axial load to Abutment

Vertical load/m'	
Length of abutment	= 10.00 m
No. of bearing	= 2.00
Depth of abutment	= 6.30 m
Width of footing	= 5.10 m
Dead load	= 485.37 : 20.0 = 24.269 ton/m'
Line load	= 27.27 : 20.0 = 1.364 ton/m'
Uniform load	= 190.60 : 20.0 = 9.530 ton/m'
Pedestrian	= 18.60 : 20.0 = 0.930 ton/m'
Total load	= 36.092 ton/m'

Seismic coefficient = 0.15

Horizontal load
 $H_{eq} = 0.150 \times R_{dl}$
 $= 0.150 \times 24.269$
 $= 3.640 \text{ ton/m'}$

Acting point of seismic load

X	= 0.300 + WF3
	= 0.300 + 1.80
	= 2.100 m
Y	= H2 + H3
	= 3.355 + 1.00
	= 4.355 m

WF3 = 1.80 m
H3 = 1.00 m

b. Abutment section

Sec- tion	width m	depth m	length m	nos	Yo t/m3	G/m' m3	G/m' m3	X m	Y m	GX m3	GY m3
A1	0.30	1.945	1.00	1.00	2.40	1.400	1.400	2.950	5.328	4.131	4.131
A2	1.00	3.355	1.00	1.00	2.40	8.052	8.052	2.700	2.677	21.740	21.740
A3	5.10	1.00	1.00	1.00	2.40	12.240	12.240	2.550	0.500	31.212	31.212
A4	0.55	0.50	1.00	1.00	2.40	0.860	0.860	3.875	3.855	2.558	2.558
A5	0.55	0.25	1.00	1.00	2.40	0.165	0.165	3.783	4.022	0.624	0.624
A6	0.20	0.50	1.00	1.00	2.40	0.240	0.240	1.700	3.188	0.408	0.408
A7	0.20	0.50	1.00	1.00	2.40	0.120	0.120	1.733	4.022	0.208	0.208
A8	2.30	0.30	1.00	1.00	2.40	0.828	0.828	3.567	1.100	2.953	2.953
A9	1.80	0.30	1.00	1.00	2.40	0.848	0.848	1.200	1.100	0.778	0.778
					sum	24.35	24.35			64.61	64.61

Wing wall and guard wall

Ww1	Ww2	Ww3	Ww4	Ww5	Gw
3.000	3.000	0.40	0.40	2.00	2.00
2.300	1.000	0.40	0.40	2.00	2.00
0.400	1.000	0.40	0.40	2.00	2.00
2.300	1.000	0.40	0.40	2.00	2.00
2.300	0.300	0.40	0.40	2.00	2.00
3.000	1.200	0.40	0.40	2.00	2.00
				sum	0.282
					1.176
					0.984

Miscellaneous load

Apr	asp
3.00	3.00
0.20	0.05
1.00	1.00
2.400	2.300
1.44	0.35
1.440	0.345
4.300	4.300
4.455	8.275
8.192	1.483
6.415	7.675
	8.580
	73.46
	74.18

Centre of gravity

X	=	Gv/m' X	Y	=	Gv/m' Y
		---			---
		Gv/m'			Gv/m'
		73.463			74.176
		---			---
		26.420			26.420
		---			---
		2.808			2.808
		m			m

axi-lod

H-filling	=	0.150	x	26.420	=	3.963 ton
c. Back fill						
a. Increase High Water level						
Depth of back fill	=	5.300 m				
H wet1	=	2.945 m				
H wet2	=	1.000 m				
H saturated	=	2.355 m				

X	=	Gv/m' X	Y	=	Gv/m' Y
		---			---
		Gv/m'			Gv/m'
		113.071			118.817
		---			---
		35.172			35.172
		---			---
		3.215 m			3.372 m

H-filling = 0.150 x 35.172 = 5.276 ton

b. Low water level

Depth of H wet = 5.300 m
Depth of H saturated = 0.00 m

a. Wet condition

Sec- tion	width m	depth m	length m	nos	Yo t/m3	G/m' m3	G/m' m3	X m	Y m	GX m3	GY m3
wet-1	2.300	5.300	1.00	1.00	1.850	22.551	22.551	4.750	3.650	107.120	82.313
wet-2	1.800	3.355	1.00	1.00	1.850	11.172	11.172	0.900	1.827	10.055	20.412
					sum	33.724	33.724			117.17	102.72

Habit 0.150 x 33.724 = 5.059 ton

Centre of gravity

X	=	Gv/m' X	Y	=	Gv/m' Y
		---			---
		Gv/m'			Gv/m'
		117.175			102.724
		---			---
		33.724			33.724
		---			---
		3.475 m			3.046 m

b. Saturated condition, zero

axi-lod

II.A.4 Coulumb's coefficient

Normal condition		Flight side Abutment	
e	30.00 deg.	e - alpha	30.00 - 0.0 = 30.00 deg.
alpha	0.0 deg.	e + delta	30.00 + 30.00 = 60.00 deg.
Q	0.0 deg.	e - beta	30.00 - 0.0 = 30.00 deg.
delta	30.00 deg.	alpha + delta	0.0 + 30.00 = 30.00 deg.
beta	0.0 deg.	alpha - beta	0.0 - 0.000 = 0.00 deg.

$$\cos^2(e - \alpha) = \frac{\sin(e + \delta) \sin(e - \beta)}{\cos(\alpha + \delta) \cos(\alpha - \beta)}$$

$$\cos^2(e - \alpha) = \frac{\sin(e + \delta) \sin(e - \beta)}{\cos(\alpha + \delta) \cos(\alpha - \beta)}$$

$$\cos^2(e - \alpha) = \frac{\sin(e + \delta) \sin(e - \beta)}{\cos(\alpha + \delta) \cos(\alpha - \beta)}$$

$$\cos^2(e - \alpha) = \frac{\sin(e + \delta) \sin(e - \beta)}{\cos(\alpha + \delta) \cos(\alpha - \beta)}$$

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$$\cos^2(e - \alpha) = \frac{\sin(e + \delta) \sin(e - \beta)}{\cos(\alpha + \delta) \cos(\alpha - \beta)}$$

$$\cos^2(e - \alpha) = \frac{\sin(e + \delta) \sin(e - \beta)}{\cos(\alpha + \delta) \cos(\alpha - \beta)}$$

$$\cos^2(e - \alpha) = \frac{\sin(e + \delta) \sin(e - \beta)}{\cos(\alpha + \delta) \cos(\alpha - \beta)}$$

$$\cos^2(e - \alpha) = \frac{\sin(e + \delta) \sin(e - \beta)}{\cos(\alpha + \delta) \cos(\alpha - \beta)}$$

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$$\cos^2(e - \alpha) = \frac{\sin(e + \delta) \sin(e - \beta)}{\cos(\alpha + \delta) \cos(\alpha - \beta)}$$

$$\cos^2(e - \alpha) = \frac{\sin(e + \delta) \sin(e - \beta)}{\cos(\alpha + \delta) \cos(\alpha - \beta)}$$

$$\cos^2(e - \alpha) = \frac{\sin(e + \delta) \sin(e - \beta)}{\cos(\alpha + \delta) \cos(\alpha - \beta)}$$

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$$\cos^2(e - \alpha) = \frac{\sin(e + \delta) \sin(e - \beta)}{\cos(\alpha + \delta) \cos(\alpha - \beta)}$$

$$\cos^2(e - \alpha) = \frac{\sin(e + \delta) \sin(e - \beta)}{\cos(\alpha + \delta) \cos(\alpha - \beta)}$$

$$\cos^2(e - \alpha) = \frac{\sin(e + \delta) \sin(e - \beta)}{\cos(\alpha + \delta) \cos(\alpha - \beta)}$$

$$\cos^2(e - \alpha) = \frac{\sin(e + \delta) \sin(e - \beta)}{\cos(\alpha + \delta) \cos(\alpha - \beta)}$$

$$\cos^2(e - \alpha) = \frac{\sin(e + \delta) \sin(e - \beta)}{\cos(\alpha + \delta) \cos(\alpha - \beta)}$$

$$\cos^2(e - \alpha) = \frac{\sin(e + \delta) \sin(e - \beta)}{\cos(\alpha + \delta) \cos(\alpha - \beta)}$$

$$\cos^2(e - \alpha) = \frac{\sin(e + \delta) \sin(e - \beta)}{\cos(\alpha + \delta) \cos(\alpha - \beta)}$$

$$\cos^2(e - \alpha) = \frac{\sin(e + \delta) \sin(e - \beta)}{\cos(\alpha + \delta) \cos(\alpha - \beta)}$$

$$\cos^2(e - \alpha) = \frac{\sin(e + \delta) \sin(e - \beta)}{\cos(\alpha + \delta) \cos(\alpha - \beta)}$$

$$\cos^2(e - \alpha) = \frac{\sin(e + \delta) \sin(e - \beta)}{\cos(\alpha + \delta) \cos(\alpha - \beta)}$$

$$\cos^2(e - \alpha) = \frac{\sin(e + \delta) \sin(e - \beta)}{\cos(\alpha + \delta) \cos(\alpha - \beta)}$$

$$\cos^2(e - \alpha) = \frac{\sin(e + \delta) \sin(e - \beta)}{\cos(\alpha + \delta) \cos(\alpha - \beta)}$$

$$\cos^2(e - \alpha) = \frac{\sin(e + \delta) \sin(e - \beta)}{\cos(\alpha + \delta) \cos(\alpha - \beta)}$$

Project : Medan Flood Control	Japan International Cooperation Agency (JICA)	Calculation sheet
Channel : Percut River		Date
Structure : Br. P-2		Nos of sheet
Station : Pc 84 + 28		Prepared by

II.A.5 Horizontal force at Abutment
II.A.5.1 Normal Condition
Case - I, High Water Level

Depth of abutment	=	6.300 m			
Surcharge load	=	1.000 t/m ²			
Unit weight wet soil	=	1.850 t/m ³			
Unit weight saturated soil	=	2.000 t/m ³			
Unit weight of water	=	1.000 t/m ³			
Width of footing					
W1	=	2.300 m			
W2	=	1.000 m			
W3	=	1.800 m			
W3	=	5.100 m			
Elevation of LWL	=	4.000 m			
Elevation of HWL	=	7.706 m			
Top slab elevation	=	10.651 m			
Depth of footing	=	1.000 m			
Elv. of base footing	=	10.651 -	6.300	=	4.351 m
Elv. of top footing	=	1.000	1.000	=	5.351 m
Depth of back fill	=	10.651 -	5.351	=	5.300 m

Depth of wet soil = H wet					
H wet	=	Top elv. -	Hwl elv.		
H wet	=	10.651 -	7.706	=	2.945 m
Depth of saturated soil					
H sat	=	H sat			
H sat	=	5.300 -	2.945	=	2.355 m

A Active earth pressure
By live load
Surcharge load = (q)

q	=	1.000 t/m ²	Sin(alpha)	=	0.000
delta	=	0.000	L (length)	=	1.000 m
alpha	=	0.000	H (depth)	=	6.300 m
Co(alpha)	=	1.000	B (width of footing)	=	5.100 m

coefficient of active pressure, normal condition

Pq	=	q x H x Ka	=	0.297	
Pq	=	1.000 x 5.300 x	L x Ka	x Ka	0.297
Pq	=	1.574 ton			

Pqv	=	Hq x Sin(alpha)			
Pqv	=	1.574 x	0.000		
Pqv	=	0.000 ton			

X	=	5.100 m			
---	---	---------	--	--	--

Pqh	=	Hq x Cos(alpha)			
Pqh	=	1.574 x	1.000		
Pqh	=	1.574 ton			

Y	=	5.300 :	2.000		
Y	=	2.650 m			

B By earth
B-1

Wet soil					
Length	=	1.000 m			
Hwet	=	2.945 m			
Hsat	=	2.355 m			

4 - 177

1. Load triangular

Pw	=	0.500 x Ywet x	Hwet^2	x	L	x Ka
	=	0.500 x 1.850 x	8.673	x	1.000	x 0.297
	=	2.383 ton				

Pw(v)	=	Pw x Sin(alpha)		=	Pw	x Co(alpha)
	=	2.383 x 0.000		=	2.383	x 1.000
	=	0.000 ton		=	2.383	ton

X	=	5.100 m		Y	=	2.945	:	3.000 + 2.355
					=	5.337	m	

B-2 Saturated

H sat = 2.355 m

2. Load rectangular

Ps1	=	Ysat x H sat x	L	x Ka
	=	2.000 x 2.355 x	1.000	x 0.297
	=	1.399 ton		

Ps1(v)	=	H sat x Sin(alpha)		=	H sat	x Co(alpha)
	=	1.399 x 0.000		=	1.399	x 1.000
	=	0.000 ton		=	1.399	ton

X	=	5.100 m		Y	=	2.355	:	2.000
					=	1.178	m	

3. Load triangular

Ps2	=	0.500 x Ysat x	Hsat^2	x	L	x Ka
	=	0.500 x 2.000 x	5.546	x	1.000	x 0.297
	=	1.647 ton				

Ps2(v)	=	H sat-2 x Sin(alpha)		=	H sat-2	x Co(alpha)
	=	1.647 x 0.000		=	1.647	x 1.000
	=	0.000 ton		=	1.647	ton

X	=	5.100 m		Y	=	2.355	:	3.000
					=	0.785	m	

4. Water

Pw	=	0.500 x Yw x	Hw^2	x	L
	=	0.500 x 1.000 x	5.546	x	1.000
	=	2.775 ton			

X	=	5.100 m			
---	---	---------	--	--	--

Y	=	2.355 :	3.000		
	=	0.785 m			

5. Uplift HWL

B = 5.100 m

Pu	=	Hw x B x Yw			
	=	2.355 x 5.100 x	1.000		
	=	12.011 ton/m			

X	=	5.100 :	2.000		
	=	2.550 m			

Project : Medias Flood Control	Japan International Cooperation Agency (JICA)	Calculation sheet
Channel : Perut River		Date
Structure : Br. P-2		No. of sheet
Station : P+84+28		Prepared by

II.A.5.2 Normal Condition Case - II, Low Water Level

Depth of abutment	=	6.300 m
Surcharge load	=	1.000 t/m ²
Unit weight wet soil	=	1.850 t/m ³
Unit weight saturated soil	=	2.000 t/m ³
Unit weight of water	=	1.000 t/m ³
Width of footing		
W/1	=	2.300 m
W/2	=	1.000 m
W/3	=	1.800 m
W/3	=	5.100 m
Elevation of LWL	=	4.000 m
Elevation of HWL	=	7.706 m
Top slab elevation	=	10.651 m
Depth of footing	=	1.000 m
Elv. of base footing	=	10.651 - 1.000 = 9.651 m
Elv. of top footing	=	4.351 + 1.000 = 5.351 m
Depth of back fill	=	10.651 - 5.351 = 5.300 m

Depth of wet soil = H wet		
H wet	=	Top elv. - Elv. of top footing
H wet	=	10.651 - 5.351 = 5.300 m
H sat	=	H sat
H sat	=	5.300 - 5.300 = 0.000 m

A Active earth pressure By live load Surcharge load = (q)

q	=	1.000 t/m ²
delta	=	0.000
alpha	=	0.000
Cos(alpha)	=	1.000
Sin(alpha)	=	0.000
L (length)	=	1.000 m
H (depth)	=	6.300 m
B (width of footing)	=	5.100 m

coefficient of active pressure, normal condition

Pq	=	q x H x L x Ka
Pq	=	1.000 x 5.300 x 1.000 x 0.297
Pq	=	1.574 ton

Pqv	=	Hq x Sin(alpha)
Pqv	=	1.574 x 0.000
X	=	0.000 ton
X	=	5.100 m

Pqh	=	Hq x Cos(alpha)
Pqh	=	1.574 x 1.000
Y	=	1.574 ton

Y	=	5.300 : 2.000
Y	=	2.650 m

B By earth

B-1		
Wet soil	=	1.000 m
Length	=	5.300 m
H wet	=	0.000 m
H sat	=	0.000 m

1. Load triangular

Pw	=	0.500 x Ywet x Hwet^2 x L x Ka
Pw	=	0.500 x 1.850 x 28.090 x 1.000 x 0.297
Pw	=	7.717 ton

Pw(v)	=	Pw x Sin(alpha)
Pw(v)	=	7.717 x 0.000
Pw(v)	=	0.000 ton
X	=	5.100 m
X	=	5.300 : 3.000 + 0.000
X	=	1.767 m

B-2 Saturated

2. Load rectangular		
Ps1	=	Ysat x Hsat x L x Ka
Ps1	=	2.000 x 0.000 x 1.000 x 0.297
Ps1	=	0.000 ton

Ps1(v)	=	Hsat x Sin(alpha)
Ps1(v)	=	0.000 x 0.000
Ps1(v)	=	0.000 ton
X	=	5.100 m
X	=	Y
X	=	0.000 : 2.000

3. Load triangular

Ps2	=	0.500 x Ysat x Hsat^2 x L x Ka
Ps2	=	0.500 x 2.000 x 0.000 x 1.000 x 0.297
Ps2	=	0.000 ton

Ps2(v)	=	Hsat-2 x Sin(alpha)
Ps2(v)	=	0.000 x 0.000
Ps2(v)	=	0.000 ton
X	=	5.100 m
X	=	Y
X	=	0.000 : 3.000

4. Water

Pw	=	0.500 x Yw x Hw^2 x L
Pw	=	0.500 x 1.000 x 0.000 x 1.000
Pw	=	0.000 ton

X	=	5.100 m
Y	=	0.000 : 3.000
Y	=	0.000 m

5. Uplift HWL

B	=	5.100 m
Pu	=	Hw x B x Yw
Pu	=	0.000 x 5.100 x 1.000
Pu	=	0.000 ton/m
X	=	5.100 : 2.000
X	=	2.550 m

Project : Medan Flood Control	Japan International	Calculation sheet
Channel : Medan Flood Way	Cooperation Agency	Date
Structure : Br - P2	(JICA)	Nos of sheet
Station : 08 + 428		Prepared by

II.A.5.4 Earthquake Condition Case - IV, Low Water Level

Depth of abutment = 6.300 m
 Surcharge load = 0.500 t/m²
 Unit weight wet soil = 1.850 t/m³
 Unit weight saturated soil = 2.000 t/m³
 Unit weight of water = 1.000 t/m³

W/1 = 2.300 m
 W/2 = 1.000 m
 W/3 = 1.800 m
 W3 = 5.100 m

Elevation of LWL = 4.000 m
 Elevation of HWL = 7.706 m

Top slab elevation = 10.651 m
 Depth of footing = 1.000 m
 Elev. of base footing = 6.300 m
 Elev. of top footing = 4.351 m
 Depth of back fill = 10.651 m

Depth of wet soil = H wet
 H wet = Top elev. - Elev. of top footing = 5.351 m
 Depth of saturated soil = H sat
 H sat = H wet = 5.300 m

A Active earth pressure By live load Surcharge load = (q)

q	=	0.500 t/m ²	Sin(alpha)	=	0.000
delta	=	0.000	L (length)	=	1.000 m
alpha	=	0.000	H (depth)	=	6.300 m
Cos(alpha)	=	1.000	B (width of footing)	=	5.100 m

coefficient of active pressure, normal condition

Pq = q x H x L x Ke = 0.407
 Pq = 0.500 x 5.300 x 1.000 x 0.407
 Pq = 1.079 ton

Pqv = Hq x Sin(alpha)
 = 1.079 x 0.000
 = 0.000 ton
 X = 5.100 m

Pqh = Hq x Cos(alpha)
 = 1.079 x 1.000
 = 1.079 ton

Y = 5.300 : 2.000
 = 2.650 m

B By earth

B-1 Wet soil
 Length = 1.000 m
 Hwet = 5.300 m
 Hsat = 0.000 m

1. Load triangular

Pw = 0.500 x Ywet x Hwet² x L x Ke
 = 0.500 x 1.850 x 28.090 x 1.000 x 0.407
 = 10.575 ton

Pw(v) = Pw x Sin(alpha)
 = 10.575 x 0.000
 = 0.000 ton
 X = 5.100 m
 Y = 5.300 : 3.000 + 0.000
 = 1.767 m

B-2 Saturated

Hsat = 0.000 m
 Ps1 = Ysat x Hsat x L x Ke
 = 2.000 x 0.000 x 1.000 x 0.407
 = 0.000 ton

Ps1(v) = Hsat x Sin(alpha)
 = 0.000 x 0.000
 = 0.000 ton
 X = 5.100 m
 Y = 0.000 : 2.000
 = 0.000 m

3. Load triangular

Ps2 = 0.500 x Ysat x Hsat² x L x Ke
 = 0.500 x 2.000 x 0.000 x 1.000 x 0.407
 = 0.000 ton

Ps2(v) = Hsat-2 x Sin(alpha)
 = 0.000 x 0.000
 = 0.000 ton
 X = 5.100 m
 Y = 0.000 : 3.000
 = 0.000 m

4. Water

Pw = 0.500 x Yw x Hw² x L
 = 0.500 x 1.000 x 0.000 x 1.000
 = 0.000 ton
 X = 5.100 m
 Y = 0.000 : 3.000
 = 0.000 m

5. Uplift HWL

B = 5.100 m
 Pu = Hw x B x Yw
 = 0.000 x 5.100 x 1.000
 = 0.000 ton/m²
 X = 5.100 : 2.000
 = 2.550 m

Project : Medan Flood Control	Japan International Cooperation Agency (JICA)	Calculation sheet
Channel : Percut River		Date
Structure : Br. P-2		Nos of sheet
Station : P+84 + 28		Prepared by

II.A.6 Combined load
III.A.6.1 Normal condition
Case - I, High Water Level

Length	L	=	10.00	m		
Width of footing	W3	=	5.10	m		
Thick of footing	H3	=	1.00	m		
	Item	V load/m'	H load /m	Yo	V.Xo	H.Yo
Super structure	V-sp	36.092	2.100		75.793	
Abutment	V-abt	26.420	2.780		73.448	
Back fill in Hwt	V-bf	35.172		3.215	113.078	
Active earth pressure						
Surcharge load	Pq		1.540	2.650		4.081
Earth load triangle	Pw		2.383	3.337		7.952
Earth load rectangle	Ps1		1.399	1.178		1.648
Earth load triangle	Ps2		1.647	0.785		1.293
Water	Pw		(2.773)	0.785		(2.177)
Up life	Pu	(12.011)		2.255		(27.085)
Total		85.673	4.196		235.234	12.797

X'	Vo.Xo	Y'	Ho.Yo
=	-----	=	-----
	Vo tot		Ho tot
	235.234		12.797
	-----		-----
	85.673		4.196
V	V/m'	L	
=	2.746	x	10.00
	85.67	x	10.00
	856.73	ton	
H	H/m'	L	
=	4.20	x	10.00
	41.96	ton	
M	V.(X-(0.5.W))	H.(Y-tb)	
=	0.5 x W	=	0.50 x
	(X-(0.5xW))	=	2.55 m
	(Y-tb)	=	0.20 m
	H x (Y-tb)	=	3.050 m
	V x (X-(0.5 x	=	41.96 x
	856.73	=	86.01 t.m
	167.68	=	(H x (Y-tb))
	81.67	=	0.20
		=	86.01

II.A.6.2 Normal condition
Case - II, Low Water Level

Super structure	Item	Vo/m'	Ho/m'	Yo	Vo.Xo	Ho.Yo
Abutment	V-sp	36.092	2.100		75.793	
Back fill	V-abt	26.420	2.780		73.448	
	V-bf	33.724	3.475		117.191	
Surcharge load	Hq		1.574	2.650		4.171
Earth load triangle	Hwt		7.717	1.767		13.636
Earth load uniform	Hsat-1					
Earth load triangle	Hsat-2					
Water	Pw					
Up life	Pu					
Total		96.236	9.291		266.432	17.807

X'	Vo.Xo	Y'	Ho.Yo
=	-----	=	-----
	Vo tot		Ho tot
	266.432		17.807
	-----		-----
	96.236		9.291
V	V/m'	L	
=	96.24	x	10.00
	962.36	ton	
H	H/m'	L	
=	9.29	x	10.00
	92.91	ton	
M	V x (X-(0.5 x	(H x (Y-tb))	
	0.5 x W	=	0.500 x
	(X-(0.5xW))	=	2.55 m
	(Y-tb)	=	0.22 m
	H x (Y-tb)	=	1.917 m
	V x (X-(0.5 x	=	92.91 x
	962.36	=	85.16 t.m
	210.30	=	(H x (Y-tb))
	125.14	=	0.22
		=	85.16

cid-an

cid-an

Project : Median Flood Control	Japan International Cooperation Agency (JICA)	Calculation sheet
Channel : Perout River		Date
Structure : Gr. P-2		Nos of sheet
Station : Pe 84 + 28		Prepared by

III.A.6.3 Earthquake condition Case - III, High Water Level

Length	L	=	10.00	m			
Width of footing	W3	=	5.10	m			
Thick of footing	H3	=	1.00	m			
	Item	V load/m'	H load /m'	Xo	Yo	V.Xo	H.Yo
Super structure	V-sp	36.092	3.640	2.100	4.355	75.793	15.852
Abutment	V-abt	26.420	3.963	2.780	2.808	73.448	11.128
Back fill in Hwl	V-bf	35.172	5.276	3.215	3.372	113.078	17.791
					</		

X'	Vo.Xo	Y	Ho.Yo
	-----		-----
	Vo tot		Ho tot
	235.234		57.568
	-----		-----
	85.673		17.075
V	V/m'	L	
	2.746	m	3.371
	-----		-----
	85.67	x	10.00
	856.73	ton	
H	H/m'	L	
	17.08	x	10.00
	-----		-----
	170.75	ton	
M	V(X-(0.5W))	H(Y-b)	
	0.5 x W	=	0.50 x
	(X-(0.5xW))	=	2.55 m
	(Y-b)	=	0.20 m
		=	3.371 m
	H x (Y-b)	=	170.75 x
		=	404.93 t.m
	V x (X-(0.5 x	=	(H x (Y-b)))
	856.73	x	0.20
	167.68	-	404.93
	(237.25)	t.m	

III.A.6.3 Earthquake condition Case - IV, Low Water Level

Super structure	Item	Vo/m'	Ho/m'	Xo	Yo	Vo.Xo	Ho.Yo
Abutment	V-sp	36.092	3.640	2.100	4.355	75.793	15.852
Back fill	V-abt	26.420	3.963	2.780	2.808	73.448	11.128
	V-bf	33.724	5.559	3.475	3.046	117.191	16.933
Surcharge load	Hq		1.574		2.650		4.171
Earth load triangle	Hwet		7.717		1.767		13.636
Earth load uniform	Hsat-1						
Earth load triangle	Hsat-2						
Water	Pw						
Up life	Pu						
Total		98.236	22.453			266.432	61.720

X'	Vo.Xo	Y	Ho.Yo
	-----		-----
	Vo tot		Ho tot
	266.432		61.720
	-----		-----
	98.236		22.453
V	V/m'	L	
	98.24	x	10.00
	962.36	ton	
H	H/m'	L	
	22.45	x	10.00
	-----		-----
	224.53	ton	
M	V(X-(0.5 x	(H x (Y-b)))	
	0.5 x W	=	0.500 x
	(X-(0.5xW))	=	2.55 m
	(Y-b)	=	0.22 m
		=	2.749 m
	H x (Y-b)	=	224.53 x
		=	392.67 m
	V x (X-(0.5 x	(H x (Y-b)))	
	962.36	x	0.22
	210.30	-	392.67
	(182.37)	t.m	

B PIER

II.B.1. Neutral axis of Pier

span	=	31.00 m	span	=	40.00 m
depth slab	=	0.245 m	depth slab	=	0.245 m
depth beam	=	1.600 m	depth beam	=	1.700 m
	=	1.845 m		=	1.945 m
thick of bearing	=	0.100 m	thick of bearing	=	0.100 m
depth of back wall	=	1.945 m	depth of back wall	=	2.045 m

Hwl elv	=	7.706 m	Hwc elv.	=	7.006
Free board to base bearing	=	0.900 m			
depth of back wall	=	2.045 m			
Slab deck elv.	=	10.651 m			

Ht	=	6.800 m	Hf	=	1.300 m
H1	=	0.250 m	W1	=	0.400 m
H2	=	1.100 m	W2	=	1.000 m
H3	=	1.000 m	W3	=	0.600 m
H4	=	4.400 m	W4	=	1.000 m
H5	=	1.000 m	W5	=	5.100 m
H6	=	0.500 m	W6	=	0.500 m
H7	=	0.500 m	W7	=	0.500 m
H8	=	4.400 m	W8	=	0.200 m
H9	=	4.400 m	W9	=	0.200 m
H10	=	0.300 m	W10	=	1.850 m
H11	=	0.300 m	W11	=	1.850 m
H12	=	0.300 m	W12	=	1.400 m

Depth of steam

=	Ht	=	Hb	=	H3	=	Hf
=	6.800	=	2.045	=	1.000	=	1.300
=	2.455 m						

(1)	W1	=	0.400	X1	=	W5/2	=	W1/2																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																			
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b. In case LWL					
Low water channel elv	=	0.200 m			
High water channel elv	=	7.706 m			
Depth of earth filling	=	Hwe elv - Top footing elv.			
	=	7.006 - 3.206 =	3.800 m		
Depth of wet soil = Hwet					
Hwet	=	Hwe elv - Top footing elv.			
	=	7.006 - 3.206 =	3.800 m		
no soil portion of saturated condition Lwl < top of footing					
Depth of saturated soil	=	Hsat			
Hsat	=	Lwl elv - Top footing elv.			
	=	3.206 - 3.206 =	0.000 m		
b.1 soil wet condition					
Wwet1	=	1.850	X1 = W5/2 + W4 + Wsat1/2	=	4.475 m
Hwet1	=	3.800	Y1 = Hwet1 + Hf	=	0.925
Wwet2	=	1.850	X2 = W5/2	=	3.200 m
Hwet2	=	3.800	Y2 = Y1	=	0.925 m
					3.200 m
b.2 soil saturated condition, ----- nothing					
Wsat1	=	0.000	X1 = Wsat1/2 + W4 + W2	=	0.000 m
Hsat1	=	0.000	Y1 = Hsat1/2 + H3	=	0.925
Wsat2	=	0.000	X2 = Wsat2/2	=	0.000 m
Hsat2	=	0.000	Y2 = Hsat2/2 + H3	=	0.000 m
					0.000 + 1.300

pl-sy

Project : Medan Flood Control	Japan International Cooperation Agency (JICA)	Calculation sheet																					
Channel : Percut River		Date																					
Structure : Br. P-2		Nos of sheet																					
Station : Pe 84 + 28		Prepared by																					
II.B.2 Axial load to Pier																							
a. Superstructure																							
Vertical load/m' = 10.00 m																							
Length of pier = 5.10 m																							
Width of footing																							
Vertical load on Pier																							
Description	Dead load	Line load	Uniform load	Pedestrian load	total ton																		
L/s span = 25.00 m																							
Total weight	485.37	27.27	190.60	18.60	721.8																		
Reaction to pier	242.69	13.64	95.30	9.30	360.9																		
Load per m width	24.27	1.36	9.53	0.93	36.09																		
R/s span/s span = 31.00 m																							
Total weight	797.87	27.27	247.79	24.00	1,096.9																		
Reaction to pier	398.94	13.64	123.90	12.00	548.5																		
Load per m width	39.89	1.36	12.39	1.20	54.85																		
Total load to pier	64.16	2.73	21.92	2.13	90.94																		
Left side span																							
Acting point of seismic load																							
Seismic coefficient = 0.15																							
Horizontal load																							
Heq	=	0.150 x Rdl																					
	=	0.150 x 24.269																					
	=	3.640 ton/m'																					
W1	=	0.400 m	H2 = 1.100 m																				
W3	=	1.850 m	H4 = 4.400 m																				
Hf	=	1.300 m																					
<table><tr><td>X</td><td>=</td><td>0.300 + W1/2 + W5/2</td></tr><tr><td></td><td>=</td><td>0.300 + 0.200 + 2.550</td></tr><tr><td></td><td>=</td><td>3.050 m</td></tr><tr><td>Y</td><td>=</td><td>H2 + H4 + Hf</td></tr><tr><td></td><td>=</td><td>1.100 + 4.400 + 1.300</td></tr><tr><td></td><td>=</td><td>6.800 m</td></tr></table>						X	=	0.300 + W1/2 + W5/2		=	0.300 + 0.200 + 2.550		=	3.050 m	Y	=	H2 + H4 + Hf		=	1.100 + 4.400 + 1.300		=	6.800 m
X	=	0.300 + W1/2 + W5/2																					
	=	0.300 + 0.200 + 2.550																					
	=	3.050 m																					
Y	=	H2 + H4 + Hf																					
	=	1.100 + 4.400 + 1.300																					
	=	6.800 m																					
Right side span																							
Horizontal load																							
Heq	=	0.150 x Rdl																					
	=	0.150 x 39.894																					
	=	5.984 ton/m'																					
W1	=	0.400 m	H3 = 1.000 m																				
W5	=	5.100 m	H4 = 4.400 m																				
W3	=	0.600 m	Hf = 1.300 m																				

pl-pv

X	=	W5/2	=	W3/2
	=	2.550	=	0.300
	=	2.250	=	m
Y	=	H3	+	H4 + Hf
	=	1.000	+	4.400 + 1.300
	=	6.700	=	m

Self weight											
Sec- tion	width m	depth m	length m	nos	Yc t/m3	G / m' m3	Gv/m' m3	X m	Y m	G.X m3	G.Y m3
A1	0.40	0.250	10.00	1.00	2.40	2.400	2.400	2.350	6.925	5.640	16.62
A2	1.00	1.100	10.00	1.00	2.40	26.400	26.400	3.050	6.250	80.520	165.00
A3	0.80	1.00	10.00	1.00	2.40	19.200	19.200	2.250	6.200	43.200	119.04
A4	1.00	4.400	7.00	1.00	2.40	73.920	73.920	2.050	2.528	151.536	186.87
A5	5.10	1.00	10.00	1.00	2.40	122.40	122.40	2.550	0.500	312.120	61.20
A6	0.50	0.50	7.00	1.00	2.40	2.100	2.100	3.217	5.533	6.756	11.62
A7	0.50	0.50	7.00	1.00	2.40	2.100	2.100	2.383	5.533	5.004	11.62
A8	0.20	4.40	7.00	1.00	2.40	7.392	7.392	3.617	2.767	28.737	20.45
A9	0.20	4.40	7.00	1.00	2.40	7.392	7.392	1.483	2.767	10.962	20.45
A10	1.85	0.30	10.00	1.00	2.40	6.660	6.660	3.367	1.100	22.424	7.33
A11	1.85	0.30	10.00	1.00	2.40	6.660	6.660	1.233	1.100	8.212	7.33
A12	1.40	0.30	10.00	1.00	2.40	10.080	10.080	1.117	1.150	11.259	11.59
						sum	286.70			684.37	639.12
Total load per meter							28.670			68.44	63.91

H-pler = 28.670 x 0.150 = 4.301 ton

X	=	Gv/m'.X	=	Gv/m'.Y
	=	88.437	=	63.912
	=	28.670	=	2.229 m

Backfill

a. In case : High water level

H wet = 0.000 m
H sat = 3.800 m
Wf1 = 1.850 m
Wf2 = 1.850 m

Sec- tion	width m	depth m	length m	nos	Yc t/m3	G / m' m3	Gv/m' m3	X m	Y m	G.X m3	G.Y m3
wet.l	1.850	0.000	1.00	1.00	1.850	0.000	0.000	0.000	0.000	0.00	0.00
wet.r	1.850	0.000	1.00	1.00	1.850	0.000	0.000	0.000	0.000	0.00	0.00
sat.l	1.850	3.800	1.00	1.00	2.000	14.060	14.060	4.475	3.200	62.918	44.99
sat.r	1.850	3.800	1.00	1.00	2.000	14.060	14.060	0.925	3.200	13.006	44.99
						sum	28.120			75.924	89.984

H-filling = 28.120 x 0.150 = 4.218 ton

X	=	Gv/m'.X	=	Gv/m'.Y
	=	75.924	=	89.984
	=	28.120	=	3.200 m

b. Low water level

H wet = 3.800 m
H sat = 0.000 m
Wf1 = 1.850 m
Wf2 = 1.850 m

Sec- tion	width m	depth m	length m	nos	Yc t/m3	G / m' m3	Gv/m' m3	X m	Y m	G.X m3	G.Y m3
wet.l	1.850	3.800	1.00	1.00	1.850	13.006	13.006	4.475	3.700	58.200	48.12
wet.r	1.850	3.800	1.00	1.00	1.850	13.006	13.006	0.925	3.700	12.030	48.12
sat.l	1.850	0.000	1.00	1.00	2.000	0.000	0.000	0.000	0.000	0.000	0.00
sat.r	1.850	0.000	1.00	1.00	2.000	0.000	0.000	0.000	0.000	0.000	0.00
						sum	26.011			70.230	96.241

H-filling = 26.011 x 0.150 = 3.902 ton

Centre of gravity

X	=	Gv/m'.X	=	Gv/m'.Y
	=	70.230	=	96.241
	=	26.011	=	3.700 m

III.B.3 Combined load on Pier

III.B.3.1 Normal condition
Case - I, High Water Level

Width	L	=	10.00 m
Width of footing	W _S	=	5.10 m
Thick of footing	H _F	=	1.30 m

W _{H1}	=	1.85 m
UP lift	=	
H _{sat}	=	3.800 m
P _{up}	=	5.10 x 3.80 x 1.00
	=	19.380 ton

Description	Dead load	Line load	Uniform load	Pedestrian load	total ton
		L/s span =			
Total weight	485.37	27.27	31.00	18.60	721.84
Reaction to pier	242.69	13.64	95.30	9.30	360.92
Load per m width	24.27	1.36	9.53	0.93	36.09
		R/s span =			
Total weight	797.87	27.27	247.79	24.00	1,088.93
Reaction to pier	398.94	13.64	123.90	12.00	548.47
Load per m width	39.89	1.36	12.39	1.20	54.85
Total load to pier	64.16	2.73	21.92	2.13	90.94

Braking force = 5 % x D * load (SNI No.1725-1989-F) Dep. PU
Left side span = 0.05 x 10.89 = 0.545 ton
Right side span = 0.05 x 13.75 = 0.688 ton

Item	V load/m'	H load/m'	Xo	Yo	V.Xo	H.Yo
Super structure						
Left side						
Right side						
Pier						
Braking force L/s						
Braking force R/s						
Back fill						
Up life						
Pu1	(19.380)				(49.419)	
Total	128.349	1.232	2.550		328.428	8.311

X'	Vo.Xo	Y'	Ho.Yo
	Vo tot		Ho tot
	328.428		8.311
	128.349		1.232
	2.559 m		8.744 m

V	V/m' x L		
	128.35 x 10.00		
H	H/m' x L		
	1.23 x 10.00		
M	V(X-(0.5W - H(Y-tb)))		
	0.5 x W = 0.50 x 5.10		
	(X - (0.5W) = 2.56 - 2.55		

(V - tb)	=	6.744 - 1.30
H x (Y - tb)	=	12.32 x 5.44
V x (X - (0.5 - (H x (Y - tb))))	=	67.09 Lm
	=	1.283.49 x 0.01 - 67.09
	=	11.37 - 67.09
	=	(55.72) Lm

III.B.3.2 Normal condition
Case - II, Low Water Level

W _{H1}	=	1.85 m
H _{sat}	=	0.000 m
P _{up}	=	1.85 x 0.00 x 1.00
	=	0.00 ton

Item	V load/m'	H load/m'	Xo	Yo	V.Xo	H.Yo
Super structure						
Left side						
Right side						
Pier						
Braking force L/s						
Braking force R/s						
Back fill						
Up life						
Pu1	0.000		0.000		0.000	
Total	145.620	1.232	0.000		385.650	8.311

X'	Vo.Xo	Y'	Ho.Yo
	Vo tot		Ho tot
	385.650		8.311
	145.620		1.232
	2.648 m		8.744 m

M	V(X-(0.5W - H(Y-tb)))		
	0.5 x W = 0.50 x 5.10		
	(X - (0.5W) = 2.65 - 2.55		
	(Y - tb) = 6.744 - 1.30		
	5.44 m		
	H x (Y - tb) = 12.32 x 5.44		
	67.09 Lm		
	V x (X - (0.5 - (H x (Y - tb))))		
	1.456.20 x 0.10 - 67.09		
	143.20 - 67.09		
	76.11 Lm		

Project : Median Flood Control	Japan International Cooperation Agency (JICA)	Calculation sheet
Channel : Perout River		Date
Structure : Br. P-2		Nos of sheet
Station : Pa. 84 + 25		Prepared by

III.B.3.3 Earthquake condition
Case - III, High Water Level

Width L = 10.00 m

Width of footing WS = 5.10 m

Thick of footing Hf = 1.30 m

WT1 = 1.85 m

UP lift

H sat = 3.800 m

P up = 5.10 x 3.80 x 1.00

total load = 19,380 ton

Description	Dead load	Line load	Uniform load	Pedestrian load	total ton
Total weight	485.37	27.27	31.00	18.60	721.84
Reaction to pier	242.69	13.64	190.60	9.30	360.92
Load per m width	24.27	1.36	9.53	0.93	36.09
R/s span = 40.00					
Total weight	797.87	27.27	247.79	24.00	1,096.93
Reaction to pier	398.94	13.64	123.90	12.00	548.47
Load per m width	39.89	1.36	12.39	1.20	54.85
Total load to pier	64.16	2.73	21.92	2.13	90.94
Braking forces	= 5 % x D * load (SNI No. 1725-1989-F). Dep. PU				
Left side span	= 0.05 x 10.89 = 0.545 ton				
Right side span	= 0.05 x 13.75 = 0.688 ton				

Item	V load/m'	H load/m'	Xo	Yo	V Xo	H Yo
Super structure						
Left side						
V-sp	36.092	3.640	3.050	6.800	110.081	24.752
V-sp	54.847	5.98	2.250	6.700	123.405	40.093
Pier	28.870	4.301	2.387	2.229	68.435	9.587
Braking forces L/s		0.545		6.800	3.704	
Braking forces R/s		0.688		6.700	4.607	
Back fill	28.120	7.540	2.700	3.200	75.924	24.126
Up life						
Pu1	(19.380)				(49.419)	0.000
Total	128.349	22.697		2.550	328.426	106.871

X'	Vo.Xo	Y	Ho.Yo	
	-----		-----	
	Vo tot		Ho tot	
	328.426		106.871	
	-----		-----	
V	V/m'	L		
	128.349	10.00		
	2.559 m			

H	H/m'	L		
	22.70 x	10.00		

M	V x (X - (0.5 x W))	Y	H (Y - tb)	
	0.5 x W	0.50 x	5.10	
	-----	-----	-----	
	(X - (0.5 x W))	2.55 m	2.55	
	-----	-----	-----	
	(Y - tb)	0.01 m	1.30	
	-----	-----	-----	
	H x (Y - tb)	4.709	3.41 m	
	-----	-----	-----	
	V x (X - (0.5 x W))	226.97 x	773.64 Lm	
	-----	-----	-----	
	(H x (Y - tb))	11.37	773.64	
	-----	-----	-----	
	(782.27) Lm			

Project : Medan Flood Control Channel : Perant River Structure : Br. P-2 Station : P+84 + 28	Japan International Cooperation Agency (JICA)	Calculation sheet Date Nos of sheet Prepared by
III. FOUNDATION A. Pile for Left side Abutment III.A.1 Characteristic of steel pipe pile III.A.1.1 Normal condition		
t' = thick of steel pipe corrosion.		
do = outer diameter of steel pipe		
t = thick of steel pipe after corrosion		
di = inner diameter of steel pipe		
Beta = characteristic value of pile		
d = outside diameter of pile before corrosion		
dc = outside diameter of pile after corrosion		
delta = the standard amount of displacement (cm)		
k = coefficient of horizontal subgrade reaction (kg/cm ²)		
Eo = modulus of deformation of the ground obtained from table 3.5		
Ec = Inertia concrete		
Es = Inertia steel		
Modulus elasticity		
Es = pile/steel		
Ec = concrete		
thick of corrosion = t'		
normal thick = te		
de effective		
dc		
di		
dc^4		
di^4		
dc^4 - di^4		
Beta = (kh.de) / (4.EI)		
kho		
Alpha		
Eo		
Permanent load		
Es (K-225)		
l/n		
Ixc		

chan-n

Ixc	=	@pi/64 x (dc^4 - di^4)	=	382,878
Ixc	=	Ixc + Ixc	=	18,794 cm ⁴
EI	=	1,897,200 x	=	32,898
4.EI	=	62,414,849,390 kg/cm ²	=	18,794
kho	=	249,659,397,562 kg/cm ²	=	62,414,849,390
Beta	=	kh x ((Bh)/30) ^ -0.75	=	7,582.47
Bh	=	@sqrt (dc / Beta)	=	38,1442
de	=	0.008031	=	0.008031
Bh/30	=	87.08	=	2.90
(Bh/30) ^ -0.75	=	30.00	=	0.45
kh	=	9.33 x	=	4.1971
Beta	=	(kh x de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,562
Beta	=	(kh.de) / (4.EI)	=	0.25
kh	=	Beta ^ 4 x 4.EI	=	6.404308E-10
kh	=	6.404308E-10	=	249,659,397,56

Project : Medan Flood Control		Japan International Cooperation Agency (JICA)		Calculation sheet	
Channel : Perut River				Date	
Structure : Br. P-2				Nos of sheet	
Station : P+84 + 28				Prepared by	
III.A.2 Axial bearing Load allowable for a Pile					
do	outer diameter of a pile (cm)				
di	inner diameter of a pile (cm)				
t	normal thick				
te	effective thick				
t'	corrosion thick				
de	dia. effective				
Apl	area of a pile (cm2)				
Lp	Length of pile				
d	40.64 cm				
t	1.031 cm				
t'	0.10 x t				
te	0.10 x t				
de	1.0310				
te	0.1031				
de	1.0310				
re	0.9279				
di	1.86				
de^2	1.86				
di^2	1.86				
de^2 - di^2	1.86				
de	1.86				
re	0.1539				
di	38.78				
de^2	1504.21				
di^2	1504.21				
de^2 - di^2	1504.21				
de	1504.21				
re	140.5074				
di	0.0141				
de^2	0.25				
di^2	0.25				
de^2 - di^2	110.35				
de	0.0110				
re	1.897,200				
di	18,972,000				
de^2	242,201				
di^2	2,422,010				
de^2 - di^2	242,201				
de	1,897,200				
re	1,897,200				
di	36.93				
de^2	4,7144				
di^2	22,252				
de^2 - di^2	1504.21				
de	1481.99				
re	0.25				
di	1,163.95				
de^2	0.1164				
di^2	0.0110				
de^2 - di^2	0.1274				
de	1,481.99				
re	1,481.99				
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de^2 - di^2	1,481.99				
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di	1,481.99				
de^2	1,481.99				
di^2	1,481.99				
de^2 - di^2	1,481.99</				

Project : Medias Flood Control Channel : Perut River Structure : Br. P-2 Station : Fe E4 + 28	Japan International Cooperation Agency (JICA)	Calculation sheet
Date	Nos of sheet	Prepared by
III.A.3 Spring Constants of steel pipe pile III.A.1 Normal condition h = axial length of pile above the design ground surface Beta = characteristic value of pile k = coefficient of horizontal sub grade reaction (kg/cm²) IE = flexural rigidity of pile K1, K2, K3 = spring constants Kv = Axial force (axial spring constants) t = thick of steel pipe before corrosion (cm) t' = thick of steel pipe after corrosion (cm) te = thick of steel pipe after corrosion (cm) d = 40.64 cm t = 1.031 cm t' = 0.103 cm te = 1.0310 - 0.1031 de = 0.9279 cm d = 2 x te de^2 = 40.64 - 1.8558 di = 38.78 cm di = (de - (2 x te)) cm di^2 = 38.78 - 1.8558 di^2 = 36.93 cm de^2 - di^2 = 1.363.71 cm² de^2 - di^2 = 1.504.21 - 1.363.71 de^4 = 140.51 cm⁴ di^4 = 2,262,660 cm⁴ de^4 - di^4 = 1,859,696 cm⁴ de^4 - di^4 = 2,262,660 - 1,859,696 de^4 - di^4 = 402,964 cm⁴ Modulus Elasticity Permanent load Es (K-225) = 257,495 kg/cm² Es = 1,897,200 kg/cm² 1/n = 257,495 1/n = 0.1357 1/n = 1,897,200 dt = di x 1/n dt = 36.9234 x 0.1357 dt^2 = 5.0121 cm dt^2 = 25.1207 cm² luc = @pi/64 x (di^4) luc = 0.0491 x 631 luc = 30.98 cm⁴ luc = @pi/64 x (de^4 - di^4) luc = 0.04909 x 402,964 luc = 19,780 cm⁴ luc = luc + luc luc = 30.9767 + 19,780.46 luc = 19,811.44 cm⁴ luc = 0.000198114 m⁴		

Le = Length of pile filled by concrete Y soil = 17.00 m Ws = 2.00 m Ws = Effective weight of soil which replace by pile Ws = 0.25 x @pi x de^2 x Ls Ws = 0.25 x 3.14 x 0.15 x 17.00 x 2.00 Ws = 4.02 ton W = Effective weight of concrete which filled in the pile W = 0.25 x @pi x di^2 x Ls W = 0.25 x 3.14 x 0.1364 x 17.00 x 2.50 W = 4.55 ton Force bearing capacity of a pile in Pter Permanent load safety factor Su = 3.00 Sq = 2.00 Temporary load safety factor	Case - I, Normal Condition Ran = 1 Ran = 1.00 Ran = 3.00 Ran = 0.33 x (170.79) + -0.54 Ran = 56.93 + -0.54 Ran = 56.39 ton/ton by material of pile Rp = Ap x fs Rp = 110.35 x 1,400 Rp = 154.50 ton Rp = 56.39 ton Case - II, Earthquake Condition Raq = 1.00 Raq = 1.00 Raq = 2.00 Raq = 0.50 x (170.79) + -0.54 Raq = 85.39 + -0.54 Raq = 84.86 ton/ton	Allowance bearing pile Vertical Horizontal case - I case - II case - III case - IV	Nos of pile Vertical Horizontal 15.19 17.06 10.10 11.34	5.22 6.87
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Project : Medan Flood Control Channel : Perout River Structure : Br. P-2 Station : P+84 + 28		Japan International Cooperation Agency (JICA)		Calculation sheet	
				Date	No. of sheet
				Prepared by	
III.A.3.2 Earthquake condition					
h	=	axial length of pile above the design ground surface			
Beta	=	characteristic value of pile			
k	=	coefficient of horizontal sub grade reaction (kg/cm ²)			
IE	=	flexural rigidity of pile			
K1, K2, K3 &	=	spring constants			
Kv	=	Axial force (axial spring constants)			
t	=	thick of steel pipe before corrosion (cm)			
t'	=	thick of steel pipe corrosion (cm)			
te	=	thick of steel pipe after corrosion (cm)			
d	=	40.64 cm			
t'	=	1.031 cm			
te	=	1.0310 - 0.1031			
	=	0.9279 cm			
de	=	d	- 2 x te		
	=	40.64	- 1.8558		
de ²	=	38.78	cm		
	=	1,504.21	cm ²		
di	=	(de - (2 x te))	cm		
	=	38.78	- 1.8558		
di ²	=	1,363.71	cm ²		
de ² - di ²	=	1,504.21	- 1,363.71		
	=	140.51	cm ²		
de ⁴	=	2,262.660	cm ⁴		
di ⁴	=	1,897.696	cm ⁴		
de ⁴ - di ⁴	=	2,262.660	- 1,897.696		
	=	402.964	cm ⁴		
Modulus Elasticity					
Temporary load					
Ec (K-225)	=	386,242	kg/cm ²		
Ea	=	1,897,200	kg/cm ²		
L/n	=	386,242		0.2036	
	=	1,897,200			
dt	=	di x L/n		0.2036	
	=	36.9284 x			
	=	7.5181	cm		
dt ²	=	56.5215	cm ²		
I1	=	@pi/64 x (di ⁴)			
	=	0.0491 x		3.195	
	=	156.82	cm ⁴		
I2	=	@pi/64 x (de ⁴ - di ⁴)			
	=	0.04909 x	402.964		
	=	19.780	cm ⁴		
I tot.	=	I1 + I2			
	=	157 +	19,780.46		
	=	19,937.28	cm ⁴		
	=	0.00019	m ⁴		

As1 = 1/4 x 3.14 x di ²		0.25 x 3.14 x 25.12	
As1		19.73 cm ²	
As2 = 1/4 x 3.14 x (de ² - di ²)		0.25 x 3.14 x 140.51	
As2		110.35 cm ²	
A tot = As1 + As2		110.35	
A tot		130.08 cm ²	
Lc = 0.013008406 m ²		17.00 m	
Kv		A tot x Lc	
Kv		1,700.00	
Kv		38.7842	
Kv		0.2000	
Kv		1.8558	
Spring constants on axial force = Kv		A tot x Ep	
Kv		130.08 x 1,897,200	
Kv		1,700	
EI		138 x 145,174	
EI		200,843.86 kg/cm	
EI		20,084 ton/m	
EI		1,897,200 x 19.811	
EI		3,768 x 10 kg/cm ²	
EI		3,759 ton/m ²	
EI		2.00 x 3,768 + 03	
EI		7,517 ton/m ²	
EI		4 x 3,759	
EI		15,035 ton/m ²	
EI		0.00503058 cm ⁻¹	
EI		0.500058 m ⁻¹	
Spring constants K1, K2 and K3		4 x EI Beta ³	
K1		1,50E+04 x 0.127307	
K1		1,914 ton	
K2=K3		2 x EI Beta ²	
K2=K3		7,517 x 0.253067	
K2=K3		1,902 ton/rad	
K4		2 x EI Beta	
K4		7,517.25 x 0.500058	
K4		3,782 ton/rad	

$A_{d1} = \frac{1}{4} \times 3.14 \times d_1^2$	$0.25 \times 3.14 \times 56.52$
$A_{d2} = \frac{1}{4} \times 3.14 \times d_2^2$	$0.25 \times 3.14 \times 140.51$
$A_{tot} = A_{d1} + A_{d2}$	$44.39 + 110.35$
$L_e = 18.50 \text{ m}$	
$K_v = \frac{A_{tot}}{L_e}$	$\frac{154.75}{18.50} = 8.37$
$EI = \frac{236120.54 \text{ kg/cm}^2 \times 23.012 \text{ ton/m}}{1.49 \times 1.850}$	$\frac{5438.48}{1.49 \times 1.850} = 2000$
$2EI = 4000$	
$4EI = 8000$	
$\beta_1 = \sqrt[4]{\frac{K_v}{EI}}$	$\sqrt[4]{\frac{8.37}{2000}} = 0.0477$
$\beta_2 = \sqrt[4]{\frac{K_v}{EI}}$	$\sqrt[4]{\frac{8.37}{2000}} = 0.0477$
$K_1 = \frac{4EI\beta_1^3}{L_e}$	$\frac{4 \times 8000 \times 0.0477^3}{18.50} = 0.108576$
$K_2 = K_3 = \frac{2EI\beta_1^2}{L_e}$	$\frac{2 \times 8000 \times 0.0477^2}{18.50} = 0.227591$
$K_4 = \frac{2EI\beta_1}{L_e}$	$\frac{2 \times 8000 \times 0.0477}{18.50} = 0.477066$

Project : Medan Flood Control	Japan International Cooperation Agency (JICA)	Calculation sheet
Channel : Perent River		Date
Structure : Br. P-2		Nos of sheet
Station : Fe 84 + 28		Prepared by

III.A.4 Displacement of head pile III.A.4.1 Normal condition

$$\begin{aligned}
 A_{x1} &= \text{tot}(K_1 \times \cos^2 \theta + K_v \times \sin^2 \theta) \\
 A_{y1} &= \text{tot}(K_v - K_1) \times \sin \theta \times \cos \theta \\
 A_{y2} &= \text{tot}(K_v - K_1) \times \sin \theta \times \cos \theta \\
 A_{x2} &= \text{tot}(K_v - K_1) \times X_1 \times (\sin \theta \times \cos \theta) - (K_2 \times \cos \theta) \\
 A_{x3} &= \text{tot}(K_v - K_1) \times X_1 \times (\sin \theta \times \cos \theta) - (K_2 \times \cos \theta) \\
 A_{y3} &= \text{tot}(K_v \times \cos^2 \theta + (K_1 \times \sin^2 \theta)) \\
 A_{y4} &= \text{tot}(K_v \times \cos^2 \theta + K_1 \times \sin^2 \theta) \times X_1 + K_2 \times \sin \theta \\
 A_{y5} &= \text{tot}(K_v \times \cos^2 \theta + K_1 \times \sin^2 \theta) \times X_1 + K_2 \times \sin \theta \\
 A_{x4} &= \text{tot}(K_v \times \cos^2 \theta + K_1 \times \sin^2 \theta) \times X_1^2 + (K_2 + K_3) \times X_1 \sin \theta + K_4
 \end{aligned}$$

Nos. of row side	Rows				Total
	(1)	(2)	(3)	(4)	
X_1	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	
n	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	
angle degree	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	
$\sin \theta$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	
$\cos \theta$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	
$\sin^2 \theta$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	
$\cos^2 \theta$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	
X_1^2	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	
K_1	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	
K_2	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	
K_3	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	
K_4	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	
K_v	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	
$n.K_1$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	
$n.K_2$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	
$n.K_3$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	
$n.K_4$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	
$n.K_v$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	
$n.(K_2 + K_3)$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	
$X_1(n.(K_2 + K_3))$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	
$X_1(n.(K_2 + K_3)) \sin \theta$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	
$n.(K_v - K_1)$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	
$n.(K_v - K_1) \sin \theta \cos \theta$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	
$n.K_1 \cos^2 \theta$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	
$n.K_v \sin^2 \theta$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	
$n.K_v \cos^2 \theta$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	
$n.K_1 \sin^2 \theta$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	
$n.K_1 \sin \theta \cos \theta$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	
$X_1(n.K_1 \sin \theta \cos \theta)$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	
$X_1(n.K_v \sin \theta \cos \theta)$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	
$n.K_2 \cos \theta$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	
$n.K_2 \sin \theta$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	
$n.K_3 \sin \theta$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	
$X_1(n.K_v \cos^2 \theta)$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	
$X_1(n.K_1 \sin^2 \theta)$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{1}{2}$	

coe-disan

Project : Medan Flood Control		Japan International Cooperation Agency (JICA)		Calculation sheet	
Channel : Peret River				Date	
Structure : Br. P-2				Nos of sheet	
Station : Pe 84 + 28				Prepared by	

III.A.4.2 Earthquake condition					
$A_{xx} = \text{tot}(K1 \times \cos^2 \theta + K_v \times \sin^2 \theta)$ $A_{yy} = \text{tot}(K_v - K1) \times \sin \theta \times \cos \theta$ $A_{yz} = \text{tot}(K_v - K1) \times \sin \theta \times \cos \theta$ $A_{zz} = \text{tot}(K_v - K1) \times \sin^2 \theta - (K2 \times \cos \theta)$ $A_{xy} = \text{tot}(K_v - K1) \times \sin \theta \times \cos \theta - (K2 \times \cos \theta)$ $A_{yx} = \text{tot}(K_v - K1) \times \sin \theta \times \cos \theta - (K2 \times \cos \theta)$ $A_{yz} = \text{tot}(K_v - K1) \times \sin \theta \times \cos \theta - (K2 \times \cos \theta)$ $A_{zy} = \text{tot}(K_v - K1) \times \sin \theta \times \cos \theta - (K2 \times \cos \theta)$ $A_{xx} = \text{tot}(K_v \times \cos^2 \theta + K1 \times \sin^2 \theta) \times X1 + K2 \times \sin \theta$ $A_{yy} = \text{tot}(K_v \times \cos^2 \theta + K1 \times \sin^2 \theta) \times X1 + K2 \times \sin \theta$ $A_{zz} = \text{tot}(K_v \times \cos^2 \theta + K1 \times \sin^2 \theta) \times X1 + K2 \times \sin \theta + (K2 + K3) \times X1 \sin \theta + K4$					
Rows					
Nos of row	(1)	(2)	(3)	(4)	
side	L/A	L/A	R/A	R/A	
sign	(-)	(-)	(+)	(+)	
angle	> 0	0	< 0	< 0	
X1	1.875	0.625	1.875	0.625	1.875
n	7.00	7.00	7.00	7.00	7.00
angle degree	0.0000	0.0000	0.0000	0.0000	5.71
sin θ	0.0000	0.0000	0.0000	0.0000	0.10
cos θ	1.0000	1.0000	1.0000	1.0000	0.9950
sin $\theta \times \cos \theta$	0.0000	0.0000	0.0000	0.0000	0.10
sin θ^2	0.0000	0.0000	0.0000	0.0000	0.01
cos θ^2	1.0000	1.0000	1.0000	1.0000	0.9901
X1 θ^2	3.5156	0.3906	3.5156	0.3906	3.5156
K1	1.643.00	1.643.00	1.643.00	1.643.00	1.643.00
K2	1.722.00	1.722.00	1.722.00	1.722.00	1.722.00
K3	1.722.00	1.722.00	1.722.00	1.722.00	1.722.00
K4	3.609.00	3.609.00	3.609.00	3.609.00	3.609.00
Kv	23.612.00	23.612.00	23.612.00	23.612.00	23.612.00
n.K1	11.501	11.501	11.501	11.501	11.501
n.K2	12.054	12.054	12.054	12.054	12.054
n.K3	12.054	12.054	12.054	12.054	12.054
n.K4	25.263	25.263	25.263	25.263	25.263
n.Kv	165.284	165.284	165.284	165.284	165.284
n.(K2 + K3)	24.108	24.108	24.108	24.108	24.108
X1(n.(K2 + K3))	(45.203)	(15.068)	0	0	45.203
X1(n.(K2 + K3))sin θ	0	0	0	0	4.497
n.(Kv - K1)	153.783	153.783	153.783	153.783	153.783
n.(Kv - K1)sin $\theta \cos \theta$	0	0	0	0	15.224
n.K1cos θ^2	11.501	11.501	11.501	11.501	11.387
n.Kv sin θ^2	0	0	0	0	1.636
n.Kv cos θ^2	165.284	165.284	165.284	165.284	163.648
n.K1 sin θ^2	0	0	0	0	114
n.K1 sin $\theta \cos \theta$	0	0	0	0	1.139
n.Kv sin $\theta \cos \theta$	0	0	0	0	16.363
X1(n.K1 sin $\theta \cos \theta$)	0	0	0	0	2.135
X1(n.Kv sin $\theta \cos \theta$)	0	0	0	0	30.681
n.K2 cos θ	12.054	12.054	12.054	12.054	11.994
n.K2 sin θ	0	0	0	0	1.199
n.K3 sin θ	0	0	0	0	1.199
X1(n.Kv cos θ^2)	(309.908)	(103.303)	103.303	103.303	306.840
X1(n.K1 sin θ^2)	0	0	0	0	213

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X1 θ^2 (n.Kv cos θ^2)	=	494.255	54.917	54.917	489.362
X1 θ^2 (n.K1 sin θ^2)	=	0	0	0	466
X1(n.K2 sin θ)	=	0	0	0	2.484
X1(n.K3 sin θ)	=	0	0	0	0
Axx = tot(K1 x cos θ^2 + Kv x sin θ^2)					
n.K1 cos θ^2	=	13.398	13.398	13.398	13.265
n.Kv sin θ^2	=	0	0	0	1.392
total	=	13.398	13.398	13.398	14.657
Ayy = tot(Kv - K1) x sin θ x cos θ					
n.(Kv - K1) sin θ cos θ	=	0	0	0	12.592
Ayz	=	0	0	0	12.592
Ayx	=	0	0	0	12.592
Azz = tot(Kv - K1) x sin θ x cos θ - (K2 x cos θ)					
X1(n.(Kv - K1) sin θ cos θ)	=	0	0	0	23.610
- n.K2 cos θ	=	(13.314)	(13.314)	(13.314)	(13.248)
Aza	=	(13.314)	(13.314)	(13.314)	10.362
Aax	=	(13.314)	(13.314)	(13.314)	10.362
Ayy = tot(Kv x cos θ^2 + K1 x sin θ^2)					
n.Kv cos θ^2	=	140.588	140.588	140.588	139.196
n.K1 sin θ^2	=	0	0	0	133
Ayy	=	140.588	140.588	140.588	139.329
Aya = tot(Kv x cos θ^2 + K1 x sin θ^2) x X1 + K2 x sin θ					
X1(n.Kv cos θ^2)	=	(263.603)	(87.868)	87.868	260.993
X1(n.K1 sin θ^2)	=	0	0	0	249
n.K2 sin θ	=	0	0	0	1.325
Ayz	=	(263.603)	(87.868)	87.868	262.566
Ayx	=	(263.603)	(87.868)	87.868	262.566
Aza = tot(Kv x cos θ^2 + K1 x sin θ^2) x X1 θ^2 + (K2 + K3) X1 sin θ + K4					
X1 θ^2 (n.Kv cos θ^2)	=	494.255	54.917	54.917	489.362
X1 θ^2 (n.K1 sin θ^2)	=	0	0	0	466
n.(K2 + K3) X1 sin θ	=	0	0	0	4.967
n.K4	=	26.474	26.474	26.474	26.474
Aza	=	520.729	81.391	81.391	521.270

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Project : Median Flood Control Channel : Peret River Structure : Br. P-2 Station : Pc 84 + 28	Japan International Cooperation Agency (JICA)	Calculation sheet
Project : Median Flood Control Channel : Peret River Structure : Br. P-2 Station : Pc 84 + 28	Japan International Cooperation Agency (JICA)	Calculation sheet

III.A.6.3 Earthquake condition Case - III, High Water Level																																																																																																																																																																																																																									
$dd = (dx \cos \theta) - (dy + \alpha \sin \theta) \sin \theta$ $dy = (dx \sin \theta) + (dy + \alpha \sin \theta) \cos \theta$	<table> <tr> <th colspan="2">Side</th><th colspan="2">Row of pile</th></tr> <tr> <th>sign</th><th>degree</th><th>(1)</th><th>(2)</th></tr> <tr> <td>(-)</td><td>< 0</td><td>(-)</td><td>(+)</td></tr> <tr> <td>(+)</td><td>> 0</td><td>(+)</td><td>(-)</td></tr> </table>	Side		Row of pile		sign	degree	(1)	(2)	(-)	< 0	(-)	(+)	(+)	> 0	(+)	(-)																																																																																																																																																																																																								
Side		Row of pile																																																																																																																																																																																																																							
sign	degree	(1)	(2)																																																																																																																																																																																																																						
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(+)	> 0	(+)	(-)																																																																																																																																																																																																																						
X n no. piles/rows θ $\cos \theta$ $\sin \theta$ $K1$ $K2$ $K3$ $K4$ Kv dx dy α $dd = dx \cos \theta - (dy + \alpha \sin \theta) \sin \theta$ $dy = dx \sin \theta + (dy + \alpha \sin \theta) \cos \theta$ $dx \sin \theta$ α $dy + \alpha \sin \theta$ $dy + \alpha \sin \theta$ $(dy + \alpha \sin \theta) \sin \theta$ $(dy + \alpha \sin \theta) \cos \theta$ $Pn = Kv \cdot dy$ $K1 \cdot dd$ $K2 \cdot \alpha$ $Pn + K1 \cdot dd + K2 \cdot \alpha$ $K3 \cdot dd$ $K4 \cdot \alpha$ $M = -K3 \cdot dd + K4 \cdot \alpha$ $M \cdot x$ $V = Pn \cos \theta - Ph \sin \theta$ $Ph \sin \theta$ $Ph \cos \theta$ $H = Ph \sin \theta + Ph \cos \theta$ Vn Mn $Mt = Mn + Vn \cdot x$	<table> <tr> <td>(1.875)</td><td>(0.625)</td><td>0.625</td><td>1.875</td></tr> <tr> <td>7.00</td><td>7.00</td><td>7.00</td><td>7.00</td></tr> <tr> <td>0.00</td><td>0.00</td><td>0.00</td><td>0.00</td></tr> <tr> <td>1.000</td><td>1.000</td><td>1.000</td><td>0.995</td></tr> <tr> <td>0.000</td><td>0.000</td><td>0.000</td><td>0.099</td></tr> <tr> <td>1.943</td><td>1.943</td><td>1.943</td><td>1.943</td></tr> <tr> <td>1.722</td><td>1.722</td><td>1.722</td><td>1.722</td></tr> <tr> <td>1.722</td><td>1.722</td><td>1.722</td><td>1.722</td></tr> <tr> <td>3.609</td><td>3.609</td><td>3.609</td><td>3.609</td></tr> <tr> <td>23.612</td><td>23.612</td><td>23.612</td><td>23.612</td></tr> <tr> <td>0.00314865</td><td>0.00314865</td><td>0.00314865</td><td>0.00314865</td></tr> <tr> <td>0.00122585</td><td>0.00122585</td><td>0.00122585</td><td>0.00122585</td></tr> <tr> <td>(0.00012467)</td><td>(0.00012467)</td><td>(0.00012467)</td><td>(0.00012467)</td></tr> <tr> <td>0.00315</td><td>0.00315</td><td>0.00315</td><td>0.00313</td></tr> <tr> <td>0.00123</td><td>0.00123</td><td>0.00123</td><td>0.00123</td></tr> <tr> <td>(0.00012)</td><td>(0.00012)</td><td>(0.00012)</td><td>(0.00012)</td></tr> <tr> <td>0.00023</td><td>0.00008</td><td>(0.00008)</td><td>(0.00023)</td></tr> <tr> <td>0.00148</td><td>0.00130</td><td>0.00115</td><td>0.00098</td></tr> <tr> <td>0.00000</td><td>0.00000</td><td>0.00000</td><td>0.00010</td></tr> <tr> <td>0.00315</td><td>0.00315</td><td>0.00315</td><td>0.00303</td></tr> <tr> <td>0.0000000</td><td>0.0000000</td><td>0.0000000</td><td>0.0003135</td></tr> <tr> <td>0.0012259</td><td>0.0012259</td><td>0.0012259</td><td>0.0012259</td></tr> <tr> <td>(0.0001247)</td><td>(0.0001247)</td><td>(0.0001247)</td><td>(0.0001247)</td></tr> <tr> <td>0.0002338</td><td>0.0000779</td><td>(0.0000779)</td><td>(0.0002338)</td></tr> <tr> <td>0.0014598</td><td>0.0013038</td><td>0.0011479</td><td>0.0009821</td></tr> <tr> <td>0.0014598</td><td>0.0013038</td><td>0.0011479</td><td>0.0009872</td></tr> <tr> <td>0.0014596</td><td>0.0013038</td><td>0.0011479</td><td>0.0013004</td></tr> <tr> <td>34.46</td><td>30.78</td><td>27.10</td><td>30.71</td></tr> <tr> <td>5.17</td><td>5.17</td><td>5.17</td><td>4.99</td></tr> <tr> <td>(0.21)</td><td>(0.21)</td><td>(0.21)</td><td>(0.21)</td></tr> <tr> <td>5.39</td><td>5.39</td><td>5.39</td><td>5.20</td></tr> <tr> <td>5.42</td><td>5.42</td><td>5.42</td><td>5.23</td></tr> <tr> <td>(0.45)</td><td>(0.45)</td><td>(0.45)</td><td>(0.45)</td></tr> <tr> <td>(5.87)</td><td>(5.87)</td><td>(5.87)</td><td>(5.88)</td></tr> <tr> <td>(41.10)</td><td>(41.10)</td><td>(41.10)</td><td>(39.73)</td></tr> <tr> <td>34.46</td><td>30.78</td><td>27.10</td><td>30.55</td></tr> <tr> <td>0.00</td><td>0.00</td><td>0.00</td><td>0.82</td></tr> <tr> <td>34.46</td><td>30.78</td><td>27.10</td><td>30.04</td></tr> <tr> <td>241.25</td><td>215.49</td><td>189.73</td><td>210.25</td></tr> <tr> <td>0.0</td><td>0.0</td><td>0.0</td><td>3.1</td></tr> <tr> <td>5.4</td><td>5.4</td><td>5.4</td><td>5.2</td></tr> <tr> <td>5.39</td><td>5.39</td><td>5.39</td><td>8.23</td></tr> <tr> <td>37.72</td><td>37.72</td><td>37.72</td><td>57.61</td></tr> <tr> <td>(452.34)</td><td>(134.68)</td><td>118.58</td><td>394.23</td></tr> <tr> <td>(41.10)</td><td>(41.10)</td><td>(41.10)</td><td>(39.73)</td></tr> <tr> <td>(493.45)</td><td>(175.79)</td><td>77.48</td><td>354.50</td></tr> <tr> <td colspan="4">Total forces at footing of Abutment</td></tr> <tr> <td>Ho</td><td>= 170.75</td><td>ton</td><td></td></tr> <tr> <td>Vo</td><td>= 856.73</td><td>ton</td><td></td></tr> <tr> <td>Mo</td><td>= (237.25)</td><td>tm</td><td></td></tr> <tr> <td colspan="4">Total forces at pile head</td></tr> <tr> <td>Hn</td><td>= Ho</td><td>ton</td><td>170.75</td></tr> <tr> <td>Vn</td><td>= Vo</td><td>ton</td><td>856.73</td></tr> <tr> <td>Mn</td><td>= Mo</td><td>tm</td><td>(237.25)</td></tr> </table>	(1.875)	(0.625)	0.625	1.875	7.00	7.00	7.00	7.00	0.00	0.00	0.00	0.00	1.000	1.000	1.000	0.995	0.000	0.000	0.000	0.099	1.943	1.943	1.943	1.943	1.722	1.722	1.722	1.722	1.722	1.722	1.722	1.722	3.609	3.609	3.609	3.609	23.612	23.612	23.612	23.612	0.00314865	0.00314865	0.00314865	0.00314865	0.00122585	0.00122585	0.00122585	0.00122585	(0.00012467)	(0.00012467)	(0.00012467)	(0.00012467)	0.00315	0.00315	0.00315	0.00313	0.00123	0.00123	0.00123	0.00123	(0.00012)	(0.00012)	(0.00012)	(0.00012)	0.00023	0.00008	(0.00008)	(0.00023)	0.00148	0.00130	0.00115	0.00098	0.00000	0.00000	0.00000	0.00010	0.00315	0.00315	0.00315	0.00303	0.0000000	0.0000000	0.0000000	0.0003135	0.0012259	0.0012259	0.0012259	0.0012259	(0.0001247)	(0.0001247)	(0.0001247)	(0.0001247)	0.0002338	0.0000779	(0.0000779)	(0.0002338)	0.0014598	0.0013038	0.0011479	0.0009821	0.0014598	0.0013038	0.0011479	0.0009872	0.0014596	0.0013038	0.0011479	0.0013004	34.46	30.78	27.10	30.71	5.17	5.17	5.17	4.99	(0.21)	(0.21)	(0.21)	(0.21)	5.39	5.39	5.39	5.20	5.42	5.42	5.42	5.23	(0.45)	(0.45)	(0.45)	(0.45)	(5.87)	(5.87)	(5.87)	(5.88)	(41.10)	(41.10)	(41.10)	(39.73)	34.46	30.78	27.10	30.55	0.00	0.00	0.00	0.82	34.46	30.78	27.10	30.04	241.25	215.49	189.73	210.25	0.0	0.0	0.0	3.1	5.4	5.4	5.4	5.2	5.39	5.39	5.39	8.23	37.72	37.72	37.72	57.61	(452.34)	(134.68)	118.58	394.23	(41.10)	(41.10)	(41.10)	(39.73)	(493.45)	(175.79)	77.48	354.50	Total forces at footing of Abutment				Ho	= 170.75	ton		Vo	= 856.73	ton		Mo	= (237.25)	tm		Total forces at pile head				Hn	= Ho	ton	170.75	Vn	= Vo	ton	856.73	Mn	= Mo	tm	(237.25)
(1.875)	(0.625)	0.625	1.875																																																																																																																																																																																																																						
7.00	7.00	7.00	7.00																																																																																																																																																																																																																						
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1.722	1.722	1.722	1.722																																																																																																																																																																																																																						
1.722	1.722	1.722	1.722																																																																																																																																																																																																																						
3.609	3.609	3.609	3.609																																																																																																																																																																																																																						
23.612	23.612	23.612	23.612																																																																																																																																																																																																																						
0.00314865	0.00314865	0.00314865	0.00314865																																																																																																																																																																																																																						
0.00122585	0.00122585	0.00122585	0.00122585																																																																																																																																																																																																																						
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5.17	5.17	5.17	4.99																																																																																																																																																																																																																						
(0.21)	(0.21)	(0.21)	(0.21)																																																																																																																																																																																																																						
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5.42	5.42	5.42	5.23																																																																																																																																																																																																																						
(0.45)	(0.45)	(0.45)	(0.45)																																																																																																																																																																																																																						
(5.87)	(5.87)	(5.87)	(5.88)																																																																																																																																																																																																																						
(41.10)	(41.10)	(41.10)	(39.73)																																																																																																																																																																																																																						
34.46	30.78	27.10	30.55																																																																																																																																																																																																																						
0.00	0.00	0.00	0.82																																																																																																																																																																																																																						
34.46	30.78	27.10	30.04																																																																																																																																																																																																																						
241.25	215.49	189.73	210.25																																																																																																																																																																																																																						
0.0	0.0	0.0	3.1																																																																																																																																																																																																																						
5.4	5.4	5.4	5.2																																																																																																																																																																																																																						
5.39	5.39	5.39	8.23																																																																																																																																																																																																																						
37.72	37.72	37.72	57.61																																																																																																																																																																																																																						
(452.34)	(134.68)	118.58	394.23																																																																																																																																																																																																																						
(41.10)	(41.10)	(41.10)	(39.73)																																																																																																																																																																																																																						
(493.45)	(175.79)	77.48	354.50																																																																																																																																																																																																																						
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Vo	= 856.73	ton																																																																																																																																																																																																																							
Mo	= (237.25)	tm																																																																																																																																																																																																																							
Total forces at pile head																																																																																																																																																																																																																									
Hn	= Ho	ton	170.75																																																																																																																																																																																																																						
Vn	= Vo	ton	856.73																																																																																																																																																																																																																						
Mn	= Mo	tm	(237.25)																																																																																																																																																																																																																						

invh-3

Project : Median Flood Control
Channel : Peret River
Structure : Br. P-2
Station : Pc 84 + 28

Japan International
Cooperation Agency
(JICA)

Calculation sheet
Date
Nos of sheet
Prepared by

III.A.6.2 Normal condition Case - II, Low Water Level

$$dd = (dx \cos \theta) - (dy + \alpha \sin \theta) \sin \theta$$

$$dy = (dx \sin \theta) + (dy + \alpha \sin \theta) \cos \theta$$

		Row of pile			
		(1)	(2)	(3)	(4)
		L/s	L/s	R/s	R/s
		< 0	(-)	(+)	> 0
Xi	m	(1.875)	(0.625)	0.625	1.875
n=no. piles/rows		7.00	7.00	7.00	7.00
θ	degree	0.00	0.00	0.00	5.71
cos θ	degree	1.000	1.000	1.000	0.995
sin θ	degree	0.000	0.000	0.000	0.099
K1	tm	1.914	1.914	1.914	1.914
K2	tm/rad	1.902	1.902	1.902	1.902
K3	tm/rad	1.902	1.902	1.902	1.902
K4	tm	3.782	3.782	3.782	3.782
Kv	tm	20.084	20.084	20.084	20.084
dx	m	0.00138228	0.00138228	0.00138228	0.00138228
dy	m	0.00168439	0.00168439	0.00168439	0.00168439
α	rad	0.00013928	0.00013928	0.00013928	0.00013928
dd = dx cos θ - (dy + α sin θ) sin θ					
dx cos θ	=	0.00138	0.00138	0.00138	0.00138
dy	=	0.00168	0.00168	0.00168	0.00168
α sin θ	=	0.00014	0.00014	0.00014	0.00014
α sin θ x Xi	=	(0.00026)	(0.00009)	0.00009	0.00026
dy + α sin θ x Xi	=	0.00142	0.00160	0.00177	0.00195
(dy + α sin θ) sin θ	=	0.00000	0.00000	0.00000	0.00019
dd	=	0.00138	0.00138	0.00138	0.00118
dy = dx sin θ + (dy + α sin θ) cos θ					
dx sin θ	=	0.0000000	0.0000000	0.0000000	0.0001375
dy	=	0.0016844	0.0016844	0.0016844	0.0016844
α sin θ	=	0.0001393	0.0001393	0.0001393	0.0001393
α sin θ x Xi	=	(0.0002611)	(0.0000670)	0.0000670	0.0002611
dy + α sin θ x Xi	=	0.0014233	0.0015974	0.0017714	0.0019455
(dy + α sin θ) cos θ	=	0.0014233	0.0015974	0.0017714	0.0019455
Pn = Kv · dy	=	0.0014233	0.0015974	0.0017714	0.0020734
K1 · dd	=	28.59	32.06	35.58	41.64
K2 · α sin θ	=	2.65	2.65	2.65	2.26
Pn + K1 · dd + K2 · α sin θ	=	0.26	0.26	0.26	0.28
K3 · dd	=	2.38	2.38	2.38	2.00
K4 · α sin θ	=	2.63	2.63	2.63	2.25
M = -K3 · dd + K4 · α sin θ	=	0.53	0.53	0.53	0.53
M · x	=	(2.10)	(2.10)	(2.10)	(1.72)
V = Pn cos θ - Ph sin θ	=	(14.72)	(14.72)	(14.72)	(12.05)
Ph sin θ	=	28.59	32.06	35.58	41.64
Ph cos θ	=	0.00	0.00	0.00	0.20
H = Ph sin θ + Ph cos θ	=	28.59	32.06	35.58	41.24
Vn	=	200.10	224.57	249.04	288.65
Mn	=	0.0	0.0	0.0	4.1
Mt = Mn + Vn · x	=	2.4	2.4	2.4	2.0
Hi	=	2.38	2.38	2.38	8.13
Vn · x	=	16.67	16.67	16.67	42.91
Mt · n	=	(375.18)	(140.36)	155.65	541.23
Mt = Mt · n + Vn · x	=	(14.72)	(14.72)	(14.72)	(12.05)
Mt = Mt · n + Vn · x	=	(389.90)	(155.07)	140.93	529.18
Total forces at footing of Abutment					
Ho	=	92.91	ton		
Vo	=	962.36	ton		
Mo	=	125.14	tm		
Total forces at pile head					
Hn	=	Ho	ton		92.91
Vn	=	Vo	ton		962.36
Mn	=	Mo	tm		125.14

mmh-2

teg-n

teg-n

Project : Medan Flood Control Channel : Percut River Structure : Br. P 2 Station : Pe 84 + 25		Japan International Cooperation Agency (JICA)		Calculation sheet	
				Date	Nos of sheet
				Prepared by	
III A-7 Control stresses on pile					
III A-7.2 Earthquake Condition					
Case - IV Low Water Level					
Steel pipe pile dia.400 mm					
Depth embedded of pile					
Axial load	Vl	=	17.00 m		
Moment	Ml	=	38,400 ton		
diameter	do	=	38,400 kg		
outer radius	ro	=	7,340 cm		
	Mr	=	734,000 kg.cm		
		=	40,000 cm		
		=	20,000 cm		
		=	Mx +		
		=	734,000 +	Vl x ro	20.00
		=	734,000 +	38,400 x	768,000
		=	1,502,000	kg.cm	
eccentricities	e	=	Mx		
		=	-----		
		=	Vl		
		=	734,000 kg.cm		
concrete covering	a'	=	38,400 kg		
	t	=	19.11 cm		
Inner diameter	di	=	2,50 cm		
		=	1.03 cm		
		=	40,00 --	2.06	
inner radius	ri	=	37.94		
		=	18.97 cm		
		=	400.00		
		=	359.86		
		=	400.00 --	359.86	
		=	40.14 cm ²		
area of steel bar	As = 8 # 16	--Ø 16			
	8.00 x	2.0106 =	16.08 cm ²		
	As =	16.08 cm ²			
	Ys =	Area of bar.Y bar			
		Area of bar			
		16.08 x	18.97		

		16.08			
		305.13			

		16.08			
		18.97 cm			
		1,897.200			

		257.495			
		7.37			

		total area of steel bar			
	P =	@piro ~ 2			
		16.08			

		3.14 x	400.00		
		16.08			

		1,256.64			

$$P = 0.01280$$

$$n.p = 7.37 \times 0.01280$$

$$= 1.54810$$

$$r_i = 18.97$$

$$alpha = \frac{r_i}{r_o} = \frac{18.97}{20.00} = 0.95$$

$$e = \frac{19.11 \text{ cm}}{19.11} = 1$$

$$e = 1$$

$$r_o = 20.00$$

$$0.9557$$

$$e = \frac{pi/4 - (5/12 - 1/6 \cos^2 \pi) \sin \pi \cos \pi + (n \cdot @ \cdot pi \cdot p/2) (\pi/r_o) \pi}{\sin \pi/3 (2 + \cos^2 \pi) - \pi \cos \pi - n \cdot @ \cdot pi \cdot p \cos \pi}$$

$$pi = 109.5261 \text{ degree}$$

$$pi = 1.9116 \text{ rad.}$$

$$e = \frac{pi/4 - (5/12 - 1/6 \cos^2 \pi) \sin \pi \cos \pi + (n \cdot @ \cdot pi \cdot p/2) (\alpha \pi)^2}{\sin \pi/3 (2 + \cos^2 \pi) - \pi \cos \pi - n \cdot @ \cdot pi \cdot p \cos \pi}$$

$$pi/4 = A$$

$$n = 1.9116$$

$$alpha = 0.95$$

$$alpha^2 = 0.9025$$

$$\cos \pi = 0.314$$

$$\sin \pi = 0.942489$$

$$\cos^2 \pi = 0.0990$$

$$1/6 \cos^2 \pi = 0.0165$$

$$5/12 - 1/6 \cos^2 \pi = 0.416667$$

$$\sin \pi \cos \pi = 0.942489$$

$$(5/12 - 1/6 \cos^2 \pi) \sin \pi \cos \pi = 0.39805$$

$$(n \cdot @ \cdot pi \cdot p) = 1.55$$

$$(n \cdot @ \cdot pi \cdot p/2) = 0.775$$

$$(n \cdot @ \cdot pi \cdot p/2) \times (\alpha \pi)^2 = 0.486$$

$$\sum = A - B + C = 0.48 - (0.13) + 2.19 = 2.80$$

$$r_o = \sin \pi/3 (2 + \cos^2 \pi) - \pi \cos \pi - n \cdot @ \cdot pi \cdot p \cos \pi$$

$$(\sin \pi/3) = 0.314$$

$$(2 + \cos^2 \pi) = 2.099$$

$$\sin \pi/3 (2 + \cos^2 \pi) = 0.663$$

$$\pi \cos \pi = 0.314$$

$$n \cdot @ \cdot pi \cdot p \cos \pi = 1.912$$

$$n \cdot @ \cdot pi \cdot p \cos \pi = 1.548$$

$$n \cdot @ \cdot pi \cdot p \cos \pi = 4.864$$

$$e = 2.80$$

$$r_o = 2.93$$

$$e = 109.53 \text{ rad}$$

$$e = 0.9559$$

$$r_o = 0.9557$$

$$1 - \cos \pi$$

$$2 \sin^2 \pi/3 - \pi \cos \pi + \sin \pi \cos \pi = 2 \pi + pi/4 - \sin \pi \cos \pi/4 - \sin^2 \pi/3 \cos \pi/4 @ \pi \cdot n \cdot p (\alpha \pi)^2 - \cos \pi$$

$$\cos \pi = 0.314$$

$$\sin \pi = 0.942$$

$$\sin^2 \pi/3 = 0.837$$

$$\sin \pi \cos \pi = 0.942$$

$$\cos^2 \pi = 0.112$$

$$1 - \cos \pi = 0.686$$

$$2 \sin^2 \pi/3 = 1.674$$

$$2 \sin^2 \pi/3 = A$$

$$\pi \cos \pi = B$$

Project : Medan Flood Control	Japan International	Calculation sheet
Channel : Perout River	Cooperation Agency	Date
Structure : Br. P-2	(JICA)	Nos of sheet
Station : PE 84 + 28 m		Prepared by

III.A.8 Combined load at Abutment section
 III.A.8.1 Normal Condition
 Case - I, High Water Level

Ka = 0.297	Ywet = 1.850	tm3
Ht = 6.300	Ysat = 2.000	tm3
Yw = 1.000	q = 1.000	tm2
Yc = 2.400		tm3

Description	Total load (ton)			
	Dead load	Line load	Pedestrian load	Total Nos of load bearing
Left side span	485.37	27.27	18.60	721.84
load per m wid	24.269	1.364	0.930	36.092

Section 1 - 1 H1 = 1.945 m W1 = 0.300 m

Load	Description						Vo	ton	Yo	m	M = Vo.Yo	m
	Y	x	H	x	W	x						
C1	2.40	x	1.945	x	0.30	=	1.400	1.400				
Pq	q	x	H									
Pa1	1.00	x	1.945			=	1.945		1.945	0.973	1.892	
	0.5	x	Y	x	H1^2	x	Ka					
	0.5	x	1.85	x	3.78	x	0.297	=	1.039	0.648	1.892	
	total							1.400	2.984		1.892	

Section 2 - 2

H1 = 1.945 m W1 = 0.30 m
 H2 = 0.500 m W2 = 0.55 m
 H3 = 0.500 m W3 = 0.55 m

Load	Description						Vo	ton	Yo	m	M = Vo.Yo	tm
	Yc	x	H	x	W	x						
C1	2.40	x	1.945	x	0.30	=	1.400	1.400	0.150	0.210		
C2	2.40	x	0.500	x	0.55	=	0.660	0.660	0.275	0.182		
C3	2.40	x	0.500	x	0.55	=	0.330	0.330	0.275	0.091		
E1	q	x	H	x	W							
	1.00	x	1.945	x	0.25	=	0.486	0.486	0.275	0.134		
	total							2.877		0.616		

Section 3 - 3

H1 = 1.945 m W1 = 0.300 m H1 + H4 = 2.445 m
 H2 = 1.000 m W2 = 1.000 m Fe = 3.963 ton (by earthquake)
 H4 = 0.500 m W4 = 0.550 m He = H4 + H5 = 1.000 m
 H5 = 0.500 m W5 = 0.550 m
 H6 = 0.500 m W6 = 0.200 m
 H7 = 0.250 m W7 = 0.200 m

Load	Description										Vo	Ho	Xo	Yo	M
	W		H		x		W		Weight						
	Y	x	m	m	m	m	ton	ton		ton					
C1	=	2.40	x	1.945	x	0.30	=	1.400		1.400					
C4	=	2.40	x	1.000	x	1.00	=	2.400		2.400					
C5	=	0.5	x	2.40	x	0.500	x	0.55	=	0.330					

mqh-1

sin p/4 = D	0.942	x	0.112	=	0.105
cos p/4	0.478	=		=	0.478
sin p/4 = E	(0.334)	:	4.000	=	(0.084)
(sin^3 p/4)	0.942	x	(0.084)	=	(0.079)
(sin^3 p/4) = F	0.837	x	(0.334)	=	(0.280)
@p	(0.280)	:	6.000	=	(0.047)
@p	1.548	=		=	
alpha	3.142	x	1.548	=	4.864
alpha^2	0.948	=		=	
alpha^2/2	0.900	=		=	
alpha^2/2 - cos pi	0.900	:	2.000	=	0.450
@p x alpha^2/2	0.450	=	(0.334)	=	0.784
@p x alpha^2/2 - cos pi	4.864	x	0.450	=	2.188
@p x alpha^2/2 - cos pi = G	2.188	=	(0.334)	=	2.522
A - B + C + D - E - F + G		=		=	4.428
sum		=		=	
C =		=		=	
2 sin^3 p/3 - pi cos pi + sin pi cos pi/4 - sin^3 pi cos pi/6 @p		=		=	
C	1.334236	=		=	
	4.428	=		=	
S	0.30135	=		=	
alpha + cos pi		=		=	
1 - cos pi		=		=	
	0.301	x	0.948	=	(0.334)
	0.301	x	1.334	=	
	0.139	x	0.460	=	
M'	1.502,000	=		=	
ro	20.000	=		=	
stress on concrete = fc		=		=	
fc	M'	=		=	
	ro^3	=		=	
	1.502,000	=		=	
	8.000	=		=	
	187.750	x	0.301	=	
	56.58	=		=	
stress on steel = fs		=		=	
fs	M'	=		=	
	ro^3	=		=	
	1.502,000	=		=	
	8.000	=		=	
	187.750	x	0.139	=	
	192.000	=		=	

teg-q

[illegible]

Section 6 - 6

Load	Description													
	Y m	x m	H m	x m	W t/m ³	Weight ton	V _o ton	H _o ton	X _o m	Y _o m	M = V _o X _o t.m			
C-3.1	=	0.5	x	2.40	x	0.300	x	2.30	=	0.828	0.828	0.767	m	0.635
C-3.2	=	2.40	x	1.000	x	2.30	=	5.520	5.520	1.150	6.348	1.150		6.348
EI(wet)	=	1.85	x	2.945	x	2.30	=	12.531	12.531	1.150	14.411	1.150		14.411
EI(sat)	=	2.00	x	2.355	x	2.30	=	10.833	10.833	1.150	12.458	1.150		12.458
U1(up lift)	=	1.00	x	2.355	x	2.30	=	5.417	5.417	1.150	6.229	1.150		6.229
Rp 1							=	(27.430)	(27.430)	1.675	(45.945)			(45.945)
Rp 2							=	(29.450)	(29.450)	0.625	(18.406)			(18.406)
		total					=	(21.752)			(24.271)			(24.271)

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III.A.8.4 Earthquake condition Case - IV, Low Water Level

Ka = 0.407	Ywet = 1.850	tm3
Ht = 6.300	Ysat = 2.000	tm3
Yw = 1.000	q = 0.500	tm2
Ye = 2.400		tm3

Description	Total load (ton)			
	Dead load	Uniform load	Pedestrian load	Total Nox of load bearing
Left side span	485.37	27.27	190.60	18.60
load per m wld	24.269	1.364	9.530	0.930
				721.84
				2.00
				10.00
				36.092

Section 1 - 1									
H1 = 1.945 m W1 = 0.300 m									
Load	Description				Weight				
	Y	X	H	W	Y	X	H	W	M = Vo.Yo
C1	2.40	1.945	0.30	1.400	1.400				
Pq	0.50	1.945			0.973				
Pa1	0.5 x Y	1.945	H1^2 x Ka						0.946
	0.5 x	1.85	3.78 x	0.407 = 1.424	1.424				0.946
	total				1.400	2.397			

Section 2 - 2									
H1 = 1.945 m W1 = 0.30 m									
H4 = 0.500 m W4 = 0.55 m									
H5 = 0.500 m W5 = 0.55 m									
Load	Description				Weight				
	Ye	X	H	W	Ye	X	H	W	M = Vo.Yo
C1	2.40	1.945	0.30	1.400	1.400				
C2	2.40	0.500	0.55	0.660	0.660				0.210
C3	2.40	0.500	0.55	0.330	0.330				0.182
El	0.5 x	1.945	0.55	0.330	0.330				0.091
	total				2.433	0.275			0.067
					2.634				0.540

Section 3 - 3									
H1 = 1.945 m W1 = 0.300 m									
H2 = 1.000 m W2 = 1.000 m									
H4 = 0.500 m W4 = 0.550 m									
H5 = 0.500 m W5 = 0.550 m									
H6 = 0.500 m W6 = 0.200 m									
H7 = 0.500 m W7 = 0.200 m									
Load	Description				Weight				
	W	Y	X	H	W	Y	X	H	M
C1	2.40	1.945	0.30	1.400	1.400				
C4	2.40	1.000	1.00	2.400	2.400				
C5	0.5 x	2.40	0.500	0.55 = 0.330	0.330				

C2	2.40	0.500	0.55	0.660	0.660				
C6	2.40	0.500	0.20	0.240	0.240				
C7	2.40	0.500	0.20	0.120	0.120				
Pe	0.5 x	2.40	0.500	0.20	0.240				
Pq	0.50	2.445	0.20	0.245	0.245				
Pa	1.85	2.445	0.407	0.920	0.920				
Rv									
sum					36.092				
					41.242				
					5.128				
					5.012				

Section 4 - 4			
Top deck =	10.651 m	Lwf =	4.006 m
Hwf elv. =	7.706 m		
Hwe elv. =	7.006 m		
H back fill =	Ht	H3	
	6.3	1.00	5.30 m
Top footin =	10.651	5.300	5.351 m
H wet soil =	10.651	5.351	5.300 m
H sat soil =	5.3	5.3	0 m
H1 =	1.945 m	W1 =	0.300 m
H2 =	3.355 m	W2 =	1.000 m
H4 =	0.500 m	W4 =	0.550 m
H5 =	0.500 m	W5 =	0.550 m
H6 =	0.500 m	W6 =	0.200 m
H7 =	0.500 m	W7 =	0.200 m
H wet =	5.3	3	1.767

Project Channel Structure Station	Median Flood Control Percut River Br. P-2 Pa B4 + 28	Japan International Cooperation Agency (JICA)	Calculation sheet Date Nos of sheet Prepared by
III.B.1.2 Characteristic Value of Pile Earthquake Condition			
Beta	characteristic value of pile		
kh	coefficient of horizontal subgrade reaction (kg/cm^3)		
EI	flexural rigidity of pile		
do	outer diameter of pile (cm)		
Eo	modulus of deformation of the ground obtained from table		
t	40.00 cm		
do^2	7,500 cm		
di	1,800 cm^2		
	(do-2t)		
	40 -	15	
di^2	25 cm		
do^2-di^2	625 cm^2	625	
	1,800 -		
do^4	975 cm		
di^4	2,560,000 cm^4		
do^4-di^4	390,825 cm^4	390,825	
	2,560,000 -		
	2,169,375 cm^4		
Modulus elasticity			
Permanent load			
Concrete K.400			
Eo	514,991 kg/cm2		
Steel			
Es	1,897,200 kg/cm2		
Inertia of pile			
Ix	@pi/64(do^4-di^4)		
	0.05 x 2,169,375		
	106,489 cm^4		
Area of prestress			
Ap	@pi/4 (do^2-di^2)		
	0.79 x 975.00		
	765.76 cm^2		
(kh.do)			
Beta	(-----)^0.25		
	4.EI		
kho	1/30 x alpha x Eo		
Alpha	0.20 from table 3.5		
Eo	28 x N		
	28.00 x 10.00		
	280 kg/cm3		
E.I	514,991 x 106,489		
			5.48E+10 kg.cm2
4.E.I	4 x 5.48E+10		
			2.19E+11 kg.cm2
kho	1/30 x Alpha x Eo		
	0.03 x 1 x 280		
	9.33 kg/cm3		

kh	=	kho x {(Beta h/30)}^-0.75	(1)-----equation
Beta h	=	Bh	
Beta	=	B	
Bh	=	@sqrt(do / B)	
do	=	40.00	
B	=	0.00526039	7.604.00
Bh	=	87.20	----> B find by trial
(Bh / 30)	=	87.20	
(Bh/30)^-0.	=	30.00	2.91
kh	=	0.45	..
	=	9.33 x	0.45
	=	4.1926	
B	=	kh x do	
	=	(-----)^0.25	(2)-----equation
	=	4.EI	
kh	=	Beta^4 x 4.EI	
	=	do	
kh	=	7.66E-10	
	=	2.19E+11	
	=	40.00	
----- (1)	=	----- (2)	
4.1926	=	4.1993	kg/cm3
By trial value of "kh" left side = right side			
kh	=	4.1993 kg/cm3	
B	=	4,199.29 ton/m3	
B	=	0.00526 cm^-1	
kh x do	=	0.526039 m^-1	
	=	4.1993 x	40.00
	=	167.97	
Allowance Horizontal Force			
Normal Condition			
han	=	kh x do	x delta
	=	B	
	=	167.97	x 1.00
	=	0.01	
	=	31,931.40 kg	
	=	31.93 ton	

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III.B.2 Axial bearing load allowance for a pile									
a. by soil	do	=	outer diameter of a pile (m)						
di	do	=	inner diameter of a pile (m)						
Lb	do	=	depth of log boring from top soil to the hard layer N > 40						
As	do	=	total circumferential surface area of the pile (m ²)						
N2'	do	=	mean N-value in the range from the tip of the pile to 10xD above.						
do	do	=	diameter or width of the pile						
Ru	do	=	ultimate bearing capacity of the pile						
li	do	=	statum thickness with the skin friction power taken into account (m)						
fi	do	=	maximum unit area skin friction power of statum with the skin friction power taken into account (t/m ²)						
Ws	do	=	effective weight of soil which replace by pile						
W	do	=	effective weight of concrete which filled in the pile						
Lc	do	=	length of pile filled by concrete						
N2'	do	=	Mean N-value in the range from the tip of the pile to 10xD above.						
do	do	=	diameter or width of the pile						
	do	=	0.40 m						
	t	=	0.075 m						
	di	=	(do - (2 x t))						
	do	=	0.40 m						
	do	=	0.25 m						
	do	=	0.1600 m ²						
	di	=	0.0625 m ²						
	do	=	0.1600 m						
	do	=	0.0975 m ²						
Ap	do	=	area of a pile (m ²)						
	do	=	0.25 x						
	do	=	0.25 x						
	do	=	0.0766 m ²						
Lp	do	=	Lp						
Lp	do	=	14.00 m						
ro	do	=	0.20 m						
U	do	=	2.00 x						
	do	=	2.00 x						
	do	=	1.26 m						
As	do	=	total circumferential surface area of the pile (m ²)						
	do	=	U x						
	do	=	1.26 x						
	do	=	17.59 m ²						
N1	do	=	50.00						
N	do	=	40.00						
Length pile embedded	do	=	13.00 m						

III.2.1 Normal condition									
N - value of embedded									
N-value	Elv.	Elv.	avg	depth	tot	depth	avg	tot	Kind
top	bot	top	bot	depth	depth	depth	depth	depth	soil
2.00	22.00	3.685	2.865	0.82	0.82	12.00	9.84	12.00	silty clay
22.00	11.00	2.865	1.865	1.00	1.82	16.50	16.50	16.50	fine sand
11.00	17.00	1.865	0.865	1.00	2.82	14.00	14.00	14.00	do
17.00	19.00	0.865	(0.135)	1.00	3.82	18.00	18.00	18.00	do
19.00	12.00	(0.135)	(1.135)	1.00	4.82	15.50	15.50	15.50	do
12.00	18.00	(2.135)	(2.135)	1.00	5.82	15.00	15.00	15.00	do
18.00	12.00	(3.135)	(3.135)	1.00	6.82	14.50	14.50	14.50	do
12.00	17.00	(4.135)	(4.135)	1.00	7.82	15.00	15.00	15.00	do
17.00	13.00	(5.135)	(5.135)	1.00	8.82	15.00	15.00	15.00	do

13.00	24.00	(5.135)	(6.135)	1.00	9.82	18.50	18.50	do	5.40	5.40
24.00	25.00	(6.135)	(7.135)	1.00	10.82	24.50	24.50	do	6.80	6.80
25.00	26.00	(7.135)	(8.135)	1.00	11.82	25.50	25.50	do	6.80	6.80
26.00	26.00	(8.135)	(9.135)	1.00	12.82	26.00	26.00	do	6.80	6.80
26.00	38.00	(9.135)	(10.135)	1.00	13.82	32.00	32.00	do	6.80	6.80
38.00	46.00	(10.135)	(11.135)	1.00	14.82	42.00	42.00	do	6.80	6.80
N - value average		14.82	301.84	:	14.82	301.84	20.37	96.40	95.43	
N average of pile			301.84	:	14.82		20.37			
Material in fine sand										
Ru	=	Ultimate bearing capacity of the pile								
li	=	statum thickness with the skin friction power taken into account (m)								
fi	=	maximum unit area skin friction power of statum with the skin friction power taken into account (t/m ²)								
N1	=	20.37								
N2'	=	40.00								
N'	=	N1 + N2'								
	=	2								
N2'	=	40.00								
N'	=	20.37 + 40.00								
	=	2								
	=	30.18 < 40								
4D	=	4.00 x								
I	=	0.50 x								
I/D	=	0.80 :								
qd/N	=	8.00								
qd	=	8.00 x								
	=	320.00 ton/m ²								
A	=	0.25 @ pi . d ^ 2								
qd.A	=	320 x								
	=	40.21								
U	=	2 x @ pi x ro								
	=	2.00 x								
	=	1.26 m								
Ru	=	qd.A + U (total li.fi)								
	=	40.21 +								
	=	160.131 ton								
Lc	=	14.00 m								
Y soil	=	2.00 t/m ³								
Ws	=	Effective weight of soil which replace by pile								
	=	0.25 @ pi								
	=	0.25 x								
	=	3.52 ton								
W	=	Effective weight of concrete which filled in the pile								
	=	0.25 @ pi								
	=	0.25 x								
	=	0.25 ton								

Normal condition.

Force bearing capacity of a pile is about

Permanent load safety factor

Temporary load safety factor

$$S_a = 3.00$$

$$S_q = 2.00$$

$$R_{aa} = \frac{1}{S_a} \left(R_a - W_a \right) + W_a - W$$

$$= \frac{1}{1.00} \left(160.13 - 3.52 \right) + 3.52 - 3.00$$

$$= 156.61 + 3.27 - 3.52 + 3.52 = 160.13$$

b. by material

R_p

$$= \frac{\text{area of pile} \times \text{compressive stress}}{765.7632 \times 350.00}$$

$$= \frac{89.339.04 \text{ kg}}{89.34 \text{ ton}}$$

$$= \frac{55.48 \text{ ton}}{55.48 \text{ ton}}$$

R minimum

Earth quake condition

$$R_{aq} = \frac{1}{2.00} \left(160.13 - 3.52 \right) + 3.27$$

$$= \frac{1}{2.00} \left(156.61 \right) + 3.27$$

$$= 78.31 + 3.27 = 81.58 \text{ ton/soe}$$

number of piles

Case	V		H	M	Allowance bearing		Nos of pile	
	ton	ton	ton	ton-m	Vertical	Horiz	Vertical	Horizontal
	Normal condition							
Case-I	HWL 1505.49	12.32	-22.42		55.48	29.99	27.14	0.41
Case-II	LWL 1661.00	12.32	106.92		55.48	29.99	29.94	0.41
	Earth quake condition							
Case-III	HWL 1259.15	214.40	(673.47)		81.58	32.15	15.43	6.67
Case-IV	LWL 1477.31	210.41	-667.87		81.58	32.15	18.11	6.54

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Structure : Br. P-2

Station : Ps 84 + 28

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III.B.3 Spring Constante

III.B.3.1 Normal condition

Ep

=

Elasticity of precast prestressed pile concrete

h

=

axial length of pile above the design ground surface

Beta

=

characteristic value of pile

k

=

coefficient of horizontal sub grade reaction (kg/cm²)

IE

=

flexural rigidity of pile

K1,K2,K3&K

=

spring constante

Kv

=

Axial force (axial spring constante)

do	t	di=do-2t	do ² 2	di ² 2	do ² 4-di ² 4	do ⁴ 4	di ⁴ 4	do ⁴ 4-di ⁴ 4
cm	cm	cm	cm ²	cm ²	cm ²	cm ⁴	cm ⁴	cm ⁴
40	7.50	25.00	1,600	625	975	2,560,000	390,625	2,169,375

lp

=

@pi/64 x (do⁴-di⁴)

=

0.0491 x

=

2,169,375

=

106,489 cm⁴

Ap

=

1/4.@pi.(do²-di²)

=

0.25 x

=

3.14

=

x

=

975

Ep

=

343,327 kg/cm²

Lp

=

14.00 m

=

1,400 cm

a

=

0.04 x

=

0.04

=

x

=

1,400

=

40.0

=

0.04 x

=

35.00

=

0.27

=

0.27

Spring constante on axial force = Kv

Kv

=

a

x

Ap

x

Ep

=

1,1300

x

765.76

x

343,327

=

1,1300

x

187,791

1,400

=

212,204

kg/cm

=

21,220

ton/m

EI

=

343,327 x

=

106,489

=

3,656E+10 kg.cm²

4EI

=

4 x

=

3,656E+10

=

1,462E+11 kg.cm²

=

1,462E+11 x

=

1,00E-07

=

14,624 1.m²

2.EI	=	7,312	Ln2	
Beta	=	0.598039	m-1	
Beta^2	=	0.34579	m-2	
Beta^3	=	0.2033	m-3	
K1	=	4.EI.Beta^3	x	0.2033
	=	14,624	ton/m	
	=	2,974		
K2=K3	=	2.EI.Beta^2	x	0.3458
	=	7,312	ton/rad	
	=	2,528		
K4	=	2.EI.Beta	x	0.5880
	=	7,312	ton.m/rad	
	=	4,300		

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III.B.3.2 Earthquake condition

- = Elasticity of precast prestressed pile concrete
- = axial length of pile above the design ground surface
- = characteristic value of pile
- = coefficient of horizontal sub grade reaction (kg/cm²)
- = flexural rigidity of pile
- = spring constante
- = Axial force (axial spring constante)

do	t	di=do-2.i	do ²	di ²	do ² -di ²	do ⁴	di ⁴	do ⁴ -di ⁴
cm	cm	cm	cm ²	cm ²	cm ²	cm ⁴	cm ⁴	cm ⁴
40	7.50	25.00	1,600	625	975	2,560,000	390,625	2,169,375

Ip	=	@pi/64 x (do ⁴ -di ⁴)						
	=	0.0491	x			2,169,375		
	=	108,489	cm ⁴					
Ap	=	1/4.@pi.(do ² -di ²)						
	=	0.25	x			3.14	x	975
	=	785.78	cm ²					
Ep	=	514,991	kg/cm ²					
Le	=	14.00	m					
	=	1,400	cm					
a	=	0.04	x			Le		0.27
	=					do		
	=	0.04	x			1,400		0.27
	=					40		
	=	0.04	x			35.00		0.27
	=	1,4000	-			0.27		
	=	1,1300						

Spring constante on axial force = Kv

Kv	=	a	x			Ap	x	Ep
	=					Le		
	=	1,1300	x			786	x	514,991
	=					1,400		
	=	1,1300	x			281,687		
	=	318,306	kg/cm					
	=	31,831	ton/m					
EI	=	514,991	x			106,489		
	=	5,484E+10	kg.cm ²					
4.EI	=	4	x			5,484E+10		
	=	2,194E+11	kg.cm ²					
	=	2,194E+11	x			1,00E-07		
	=	21,936	t.m ²					

ke-pq

Project : Median Flood Control	Japan International Cooperation Agency (JICA)	Calculation sheet
Channel : Perout River		Date
Structure : Br. P-2		No. of sheet
Station : Pc 84 + 28		Prepared by

II.B.4 Displacement of a head pile

II.B.4.1 Normal Condition

$$Ax = \text{tot}(K1 \times \cos^2 \theta + Kv \times \sin^2 \theta)$$

$$Ay = \text{tot}(Kv - K1) \times \sin \theta \cos \theta$$

$$Ax' = \text{tot}(Kv - K1) \times \sin \theta \cos \theta$$

$$Ay' = \text{tot}(Kv - K1) \times \sin \theta \cos \theta - (K2 \times \cos \theta)$$

$$Ax = \text{tot}(Kv - K1) \times \sin \theta \cos \theta - (K2 \times \cos \theta)$$

$$Ay = \text{tot}(Kv \times \cos^2 \theta + K1 \times \sin^2 \theta)$$

$$Ax = \text{tot}(Kv \times \cos^2 \theta + K1 \times \sin^2 \theta) \times X1 + K2 \times \sin \theta$$

$$Ay = \text{tot}(Kv \times \cos^2 \theta + K1 \times \sin^2 \theta) \times X1 + K2 \times \sin \theta$$

$$Ax = \text{tot}(Kv \times \cos^2 \theta + K1 \times \sin^2 \theta) \times X1^2 + (K2 + K3) \times \sin \theta + K4$$

Abutment	Rows	Total
No. of row	(1) (2) (3) (4)	
side	L/s L/s R/s	
sign	(-) 0 (+)	
angle	> 0 0 < 0	
X1	(1.875) 0.625 1.875	
a	7.00 7.00 7.00	1.875
angle degree	(5.7100) 0.0000 0.0000	5.71
sin θ	(0.0995) 0.00 0.00	0.10
cos θ	0.9990 1.0000 0.9990	0.9990
sin $\theta \times \cos \theta$	(0.0990) 0.00 0.00	0.10
cos θ^2	0.0099 0.00 0.00	0.01
X1 θ^2	3.5156 0.3906 0.3906	0.9001
K1	2.974 2.974 2.974	2.974
K2	2.528 2.528 2.528	2.528
K3	2.528 2.528 2.528	2.528
K4	4.300 4.300 4.300	4.300
Kv	21.220 21.220 21.220	21.220
nK1	20.818 20.818 20.818	20.818
nK2	17.696 17.696 17.696	17.696
nK3	17.696 17.696 17.696	17.696
nK4	30.100 30.100 30.100	30.100
n(K2 + K3)	148.540 148.540 148.540	148.540
n(K2 + K3)	35.392 35.392 35.392	35.392
X1(n(K2 + K3))sin θ	(66.360) 0 0	66.360
n(Kv - K1)	127.722 127.722 127.722	127.722
n(Kv - K1)sin $\theta \cos \theta$	(12.644) 0 0	12.644
nK1cos θ^2	20.612 20.612 20.612	20.612
nKv sin θ^2	1.470 1.470 1.470	1.470
nKv cos θ^2	147.070 147.070 147.070	147.070
nK1 sin $\theta \cos \theta$	(2.061) 0 0	2.061
nK1 sin $\theta \cos \theta$	(14.705) 0 0	14.705
X1(nK1 sin $\theta \cos \theta$)	3.864 3.864 3.864	3.864
X1(nKv sin $\theta \cos \theta$)	27.573 27.573 27.573	27.573
nK2 cos θ	17.696 17.696 17.696	17.696
nK2 sin θ	(1.761) 0 0	1.761
nK3 sin θ	(1.761) 0 0	1.761
X1(nKv cos θ^2)	(92.838) 0 0	92.838
X1(nK1 sin θ^2)	(386) 0 0	386
X1 θ^2 (nKv cos θ^2)	517.042 517.042 517.042	517.042
X1 θ^2 (nK1 sin θ^2)	724 724 724	724
X1(nK2 sin θ)	3.301 3.301 3.301	3.301
X1(nK3 sin θ)	3.301 3.301 3.301	3.301

cos = d sin

kg-pg

$A_{xx} = \text{tot}(K1 \times \cos^2 \theta + K2 \times \sin^2 \theta)$							
$n.K1.\cos^2 \theta$	=	22,130	22,351	22,351	22,351	22,130	
$n.K2.\sin^2 \theta$	=	2,206	0	0	0	2,206	
total	=	24,335	22,351	22,351	22,351	24,335	91,375
$A_{xy} = \text{tot}(Kv - K1) \times \sin \theta \times \cos \theta$							
$n.(Kv - K1) \times \sin \theta \times \cos \theta$	=	(19,846)	0	0	0	19,846	
A_{yx}	=	(19,846)	0	0	0	19,846	0
A_{yz}	=	(19,846)	0	0	0	19,846	0
$A_{zx} = \text{tot}(Kv - K1) \times \sin \theta \times \cos \theta - (K2 \times \cos \theta)$							
$Xi.(n.(Kv - K1) \times \sin \theta \times \cos \theta - n.K2.\cos \theta)$	=	37,211	0	0	0	37,211	
A_{zx}	=	(21,140)	(21,245)	(21,245)	(21,245)	(21,140)	
A_{xz}	=	-16,072	(21,245)	(21,245)	(21,245)	16,072	(10,346)
A_{zz}	=	16,072	(21,245)	(21,245)	(21,245)	16,072	(10,346)
$A_{yt} = \text{tot}(Kv \times \cos^2 \theta + (K1 \times \sin^2 \theta))$							
$n.Kv.\cos^2 \theta$	=	220,611	222,817	222,817	222,817	220,611	
$n.K1.\sin^2 \theta$	=	221	0	0	0	221	
A_{yt}	=	220,833	222,817	222,817	222,817	220,833	887,299
$A_{ya} = \text{tot}(Kv \times \cos^2 \theta + K1 \times \sin^2 \theta) \times Xi + K2 \times \sin \theta$							
$Xi.(n.Kv.\cos^2 \theta + n.K1.\sin^2 \theta) \times Xi + K2 \times \sin \theta$	=	(415,646)	(139,261)	139,261	139,261	415,646	
A_{ya}	=	(415)	0	0	0	415	
$n.K2.\sin \theta$	=	(2,114)	0	0	0	2,114	
A_{ya}	=	(416,175)	(139,261)	139,261	416,175	0	
A_{yt}	=	(416,175)	(139,261)	139,261	416,175	0	
$A_{aa} = \text{tot}(Kv \times \cos^2 \theta + K1 \times \sin^2 \theta) \times Xi^2 + (K2 + K3) \times \sin \theta + K4$							
$Xi^2.(n.Kv.\cos^2 \theta + n.K1.\sin^2 \theta) \times Xi^2 + (K2 + K3) \times \sin \theta + K4$	=	775,587	87,038	87,038	87,038	775,587	
A_{aa}	=	778	0	0	0	778	
$n.K4$	=	7,927	0	0	0	7,927	
A_{aa}	=	40,390	40,390	40,390	40,390	40,390	40,390
A_{aa}	=	824,681	127,428	127,428	127,428	824,681	1,904,218

Project : Median Flood Control	Japan International Cooperation Agency (JICA)	Calculation sheet
Channel : Furusui River		
Structure : Br. P-2		
Station : P. 84 + 28		

MLR.5 Displacement of head pile									
MLR.5.1 Normal condition									
Case = High Water Level									
Axis	dx	+	Axis	dy	+	Axis	dz	+	Axis
Axis	dx	+	Axis	dy	+	Axis	dz	+	Axis
85.801 dx	+	Axis	dy	+	Axis	dz	+	Axis	
0 dx	+	Axis	dy	+	Axis	dz	+	Axis	
23.192 dx	+	Axis	dy	+	Axis	dz	+	Axis	
8.500E+04	+	Axis	dy	+	Axis	dz	+	Axis	
0	+	Axis	dy	+	Axis	dz	+	Axis	
-23.192	+	Axis	dy	+	Axis	dz	+	Axis	
-23.192	+	Axis	dy	+	Axis	dz	+	Axis	

85.801 dx	+	Axis	dy	+	Axis	dz	+	Axis	
0 dx	+	Axis	dy	+	Axis	dz	+	Axis	
23.192 dx	+	Axis	dy	+	Axis	dz	+	Axis	
8.500E+04	+	Axis	dy	+	Axis	dz	+	Axis	
0	+	Axis	dy	+	Axis	dz	+	Axis	
-23.192	+	Axis	dy	+	Axis	dz	+	Axis	
-23.192	+	Axis	dy	+	Axis	dz	+	Axis	

85.801 dx	+	Axis	dy	+	Axis	dz	+	Axis	
0 dx	+	Axis	dy	+	Axis	dz	+	Axis	
23.192 dx	+	Axis	dy	+	Axis	dz	+	Axis	
8.500E+04	+	Axis	dy	+	Axis	dz	+	Axis	
0	+	Axis	dy	+	Axis	dz	+	Axis	
-23.192	+	Axis	dy	+	Axis	dz	+	Axis	
-23.192	+	Axis	dy	+	Axis	dz	+	Axis	

85.801 dx	+	Axis	dy	+	Axis	dz	+	Axis	
0 dx	+	Axis	dy	+	Axis	dz	+	Axis	
23.192 dx	+	Axis	dy	+	Axis	dz	+	Axis	
8.500E+04	+	Axis	dy	+	Axis	dz	+	Axis	
0	+	Axis	dy	+	Axis	dz	+	Axis	
-23.192	+	Axis	dy	+	Axis	dz	+	Axis	
-23.192	+	Axis	dy	+	Axis	dz	+	Axis	

85.801 dx	+	Axis	dy	+	Axis	dz	+	Axis	
0 dx	+	Axis	dy	+	Axis	dz	+	Axis	
23.192 dx	+	Axis	dy	+	Axis	dz	+	Axis	
8.500E+04	+	Axis	dy	+	Axis	dz	+	Axis	
0	+	Axis	dy	+	Axis	dz	+	Axis	
-23.192	+	Axis	dy	+	Axis	dz	+	Axis	
-23.192	+	Axis	dy	+	Axis	dz	+	Axis	

85.801 dx	+	Axis	dy	+	Axis	dz	+	Axis	
0 dx	+	Axis	dy	+	Axis	dz	+	Axis	
23.192 dx	+	Axis	dy	+	Axis	dz	+	Axis	
8.500E+04	+	Axis	dy	+	Axis	dz	+	Axis	
0	+	Axis	dy	+	Axis	dz	+	Axis	
-23.192	+	Axis	dy	+	Axis	dz	+	Axis	
-23.192	+	Axis	dy	+	Axis	dz	+	Axis	

85.801 dx	+	Axis	dy	+	Axis	dz	+	Axis	
0 dx	+	Axis	dy	+	Axis	dz	+	Axis	
23.192 dx	+	Axis	dy	+	Axis	dz	+	Axis	
8.500E+04	+	Axis	dy	+	Axis	dz	+	Axis	
0	+	Axis	dy	+	Axis	dz	+	Axis	
-23.192	+	Axis	dy	+	Axis	dz	+	Axis	
-23.192	+	Axis	dy	+	Axis	dz	+	Axis	

85.801 dx	+	Axis	dy	+	Axis	dz	+	Axis	
0 dx	+	Axis	dy	+	Axis	dz	+	Axis	
23.192 dx	+	Axis	dy	+	Axis	dz	+	Axis	
8.500E+04	+	Axis	dy	+	Axis	dz	+	Axis	
0	+	Axis	dy	+	Axis	dz	+	Axis	
-23.192	+	Axis	dy	+	Axis	dz	+	Axis	
-23.192	+	Axis	dy	+	Axis	dz	+	Axis	

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mvh-2

Project : Median Flood Control Channel : Perout River Structure : Br. P-2 Station : Pc 84 + 28	Japan International Cooperation Agency (JICA)	Calculation sheet Date Nos of sheet Prepared by
III.B.6 Force at each pile head III.B.6.1 Normal condition Case -I High Water Level		
$dd = (dd \cos \theta) - (dy + \alpha pa.X) \sin \theta$ $dy = (dd \sin \theta) + (dy + \alpha pa.X) \cos \theta$		
	Side sign degree	Row of pile
X1	(1.875)	(0.625)
n-nos pile/rows	7.00	7.00
a	(5.71)	0.00
cos e	0.995	1.000
sin e	(0.099)	0.000
K1	2.974	2.974
K2	2.528	2.528
K3	2.528	2.528
K4	4.300	4.300
Kv	21.220	21.220
dx	0.0013252	0.0013252
dy	0.00216941	0.00216941
alpha	(0.00004096)	(0.00004096)
dd = dx cos e - (dy + alpha.X) sin e		
dx cos e	0.00013	0.00013
dy	0.00217	0.00217
alpha x X1	(0.00004)	(0.00004)
dy + Alpha x X1	0.00008	0.00003
(dy + alpha.X) sin e	0.00225	0.00214
dd	(0.00022)	0.00000
dy = dx sin e + (dy + alpha.X) cos e		
dx sin e	0.00036	0.00013
dy	(0.0000132)	0.0000000
alpha	0.0021694	0.0021694
Alpha x X1	(0.0000410)	(0.0000410)
dy + Alpha x X1	0.0000768	0.0000258
(dy + alpha.X) cos e	0.0022462	0.0021950
dy	0.0022351	0.0021950
P n1=Kv.dyl	47.15	46.58
K1.dyl	1.08	0.39
K2.alpa	(0.10)	(0.10)
P N=K1.dyl-K2.alpa	1.16	0.50
K3.dyl	0.30	0.34
K4.alpa	(0.16)	(0.16)
M t = -K3.dyl + K4.alpa	(1.07)	(0.51)
M t x n	(7.52)	(3.56)
Vi = Phi cos e - Phi sin e		
Phi cos	46.91	46.58
Phi sin e	(0.12)	0.00
Vi	47.03	46.58
V n	329.21	326.05
H = Phi sin e + Phi cos e		
Phi sin e	(4.7)	0.0
Phi cos e	1.2	0.5
H	(3.54)	0.50
H n	(24.75)	3.48
V n x	(617.27)	(203.78)
M n	(7.52)	(3.56)
M t = M n + V n x	(624.79)	(207.36)
Total forces at footing of Abutment		
Ho	= 12.32 ton	
Vo	= 1,283.49 ton	
Mo	= (55.71) t.m	
Total forces at pile head		
H n = Ho		12.32 ton
V n = Vo		1,283.49 ton
M t = Mo		(55.71) t.m

mwh-1

Project : Median Flood Control Channel : Perout River Structure : Br. P-2 Station : Pc 84 + 28	Japan International Cooperation Agency (JICA)	Calculation sheet Date Nos of sheet Prepared by																																																																																																																																																																																																																																																																																														
III.B.6.4 Earthquake condition Case - IV Low Water Level																																																																																																																																																																																																																																																																																																
$dd = (dd \cos e) - (dy + \alpha pa.X) \sin e$ $dy = (dd \sin e) + (dy + \alpha pa.X) \cos e$																																																																																																																																																																																																																																																																																																
<table> <tr> <th rowspan="2">Side sign degree</th><th colspan="3">Row of pile</th><th rowspan="2">total per row</th></tr> <tr> <th>(1) L/a (-)</th><th>(2) L/a (+)</th><th>(3) R/a (+)</th></tr> <tr> <td>< 0</td><td>(1.875)</td><td>(0.625)</td><td>(0.625)</td><td>1.875</td></tr> <tr> <td>n-nos pile/rows</td><td>7.00</td><td>7.00</td><td>7.00</td><td>7.00</td></tr> <tr> <td>a</td><td>(5.71)</td><td>0.00</td><td>0.00</td><td>5.71</td></tr> <tr> <td>cos e</td><td>0.995</td><td>1.000</td><td>1.000</td><td>0.995</td></tr> <tr> <td>sin e</td><td>(0.099)</td><td>0.000</td><td>0.000</td><td>0.099</td></tr> <tr> <td>K1</td><td>3.193</td><td>3.193</td><td>3.193</td><td>3.193</td></tr> <tr> <td>K2</td><td>3.035</td><td>3.035</td><td>3.035</td><td>3.035</td></tr> <tr> <td>K3</td><td>3.035</td><td>3.035</td><td>3.035</td><td>3.035</td></tr> <tr> <td>K4</td><td>5.770</td><td>5.770</td><td>5.770</td><td>5.770</td></tr> <tr> <td>Kv</td><td>31.631</td><td>31.631</td><td>31.631</td><td>31.631</td></tr> <tr> <td>dx</td><td>0.00233494</td><td>0.00233494</td><td>0.00233494</td><td>0.00233494</td></tr> <tr> <td>dy</td><td>0.00164116</td><td>0.00164116</td><td>0.00164116</td><td>0.00164116</td></tr> <tr> <td>alpha</td><td>(0.00022912)</td><td>(0.00022912)</td><td>(0.00022912)</td><td>(0.00022912)</td></tr> <tr> <td>dd = dx cos e - (dy + alpha.X) sin e</td><td></td><td></td><td></td><td></td></tr> <tr> <td>dx cos e</td><td>0.00232</td><td>0.00233</td><td>0.00233</td><td>0.00232</td></tr> <tr> <td>dy</td><td>0.00164</td><td>0.00164</td><td>0.00164</td><td>0.00164</td></tr> <tr> <td>Alpha x X1</td><td>(0.00023)</td><td>(0.00023)</td><td>(0.00023)</td><td>(0.00023)</td></tr> <tr> <td>dy + Alpha x X1</td><td>0.00043</td><td>0.00014</td><td>0.00014</td><td>0.00043</td></tr> <tr> <td>(dy + alpha.X) sin e</td><td>0.00207</td><td>0.00178</td><td>0.00150</td><td>0.00121</td></tr> <tr> <td>dd</td><td>(0.00021)</td><td>0.00000</td><td>0.00000</td><td>0.00012</td></tr> <tr> <td>dy = dx sin e + (dy + alpha.X) cos e</td><td></td><td></td><td></td><td></td></tr> <tr> <td>dx sin e</td><td>0.0003233</td><td>0.0000000</td><td>0.0000000</td><td>0.0003233</td></tr> <tr> <td>dy</td><td>0.0016412</td><td>0.0016412</td><td>0.0016412</td><td>0.0016412</td></tr> <tr> <td>alpha</td><td>(0.0002291)</td><td>(0.0002291)</td><td>(0.0002291)</td><td>(0.0002291)</td></tr> <tr> <td>Alpha x X1</td><td>0.0004398</td><td>0.0001432</td><td>0.0001432</td><td>0.0004398</td></tr> <tr> <td>dy + Alpha x X1</td><td>0.0009798</td><td>0.0017844</td><td>0.0014960</td><td>0.0012116</td></tr> <tr> <td>(dy + alpha.X) cos e</td><td>0.0025605</td><td>0.0017844</td><td>0.0014960</td><td>0.0012055</td></tr> <tr> <td>P n1 = Kv.dyl</td><td>59.19</td><td>56.80</td><td>47.68</td><td>45.77</td></tr> <tr> <td>K1.dyl</td><td>8.08</td><td>7.46</td><td>7.46</td><td>7.03</td></tr> <tr> <td>K2.alpa</td><td>(0.70)</td><td>(0.70)</td><td>(0.70)</td><td>(0.70)</td></tr> <tr> <td>P N = K1.dyl - K2.alpa</td><td>8.77</td><td>8.15</td><td>8.15</td><td>7.73</td></tr> <tr> <td>K3.dyl</td><td>7.68</td><td>7.09</td><td>7.09</td><td>6.69</td></tr> <tr> <td>K4.alpa</td><td>(1.32)</td><td>(1.32)</td><td>(1.32)</td><td>(1.32)</td></tr> <tr> <td>M t = -K3.dyl + K4.alpa</td><td>(8.41)</td><td>(8.41)</td><td>(8.41)</td><td>(8.01)</td></tr> <tr> <td>M t x n</td><td>(62.99)</td><td>(58.86)</td><td>(58.86)</td><td>(56.05)</td></tr> <tr> <td>Vi = Phi cos e - Phi sin e</td><td></td><td></td><td></td><td></td></tr> <tr> <td>Phi cos</td><td>57.90</td><td>56.80</td><td>47.68</td><td>45.54</td></tr> <tr> <td>Phi sin e</td><td>(0.87)</td><td>0.00</td><td>0.00</td><td>0.77</td></tr> <tr> <td>Vi</td><td>58.78</td><td>56.80</td><td>47.68</td><td>44.77</td></tr> <tr> <td>V n</td><td>411.44</td><td>397.59</td><td>333.77</td><td>313.41</td></tr> <tr> <td>H = Phi sin e + Phi cos e</td><td></td><td></td><td></td><td></td></tr> <tr> <td>Phi sin e</td><td>(5.8)</td><td>0.0</td><td>0.0</td><td>4.6</td></tr> <tr> <td>Phi cos e</td><td>8.7</td><td>8.15</td><td>8.15</td><td>7.7</td></tr> <tr> <td>H</td><td>20.57</td><td>57.06</td><td>57.06</td><td>12.24</td></tr> <tr> <td>H n</td><td>(771.44)</td><td>(248.49)</td><td>208.61</td><td>220.39</td></tr> <tr> <td>V n x</td><td>(62.99)</td><td>(58.86)</td><td>(58.86)</td><td>587.64</td></tr> <tr> <td>M n</td><td>(624.43)</td><td>(307.35)</td><td>149.75</td><td>531.59</td></tr> <tr> <td>M t = M n + V n x</td><td></td><td></td><td></td><td>(460.45)</td></tr> <tr> <td>Total forces at footing of Abutment</td><td></td><td></td><td></td><td></td></tr> <tr> <td>Ho</td><td>= 220.39 ton</td><td></td><td></td><td>220.39 ton</td></tr> <tr> <td>Vo</td><td>= 1,456.20 ton</td><td></td><td></td><td>1,456.20 ton</td></tr> <tr> <td>Mo</td><td>= (460.45) t.m</td><td></td><td></td><td>(460.45) t.m</td></tr> <tr> <td>Total forces at pile head</td><td></td><td></td><td></td><td></td></tr> <tr> <td>H n = Ho</td><td></td><td></td><td></td><td>220.39 ton</td></tr> <tr> <td>V n = Vo</td><td></td><td></td><td></td><td>1,456.20 ton</td></tr> <tr> <td>M t = Mo</td><td></td><td></td><td></td><td>(460.45) t.m</td></tr> </table>	Side sign degree	Row of pile			total per row	(1) L/a (-)	(2) L/a (+)	(3) R/a (+)	< 0	(1.875)	(0.625)	(0.625)	1.875	n-nos pile/rows	7.00	7.00	7.00	7.00	a	(5.71)	0.00	0.00	5.71	cos e	0.995	1.000	1.000	0.995	sin e	(0.099)	0.000	0.000	0.099	K1	3.193	3.193	3.193	3.193	K2	3.035	3.035	3.035	3.035	K3	3.035	3.035	3.035	3.035	K4	5.770	5.770	5.770	5.770	Kv	31.631	31.631	31.631	31.631	dx	0.00233494	0.00233494	0.00233494	0.00233494	dy	0.00164116	0.00164116	0.00164116	0.00164116	alpha	(0.00022912)	(0.00022912)	(0.00022912)	(0.00022912)	dd = dx cos e - (dy + alpha.X) sin e					dx cos e	0.00232	0.00233	0.00233	0.00232	dy	0.00164	0.00164	0.00164	0.00164	Alpha x X1	(0.00023)	(0.00023)	(0.00023)	(0.00023)	dy + Alpha x X1	0.00043	0.00014	0.00014	0.00043	(dy + alpha.X) sin e	0.00207	0.00178	0.00150	0.00121	dd	(0.00021)	0.00000	0.00000	0.00012	dy = dx sin e + (dy + alpha.X) cos e					dx sin e	0.0003233	0.0000000	0.0000000	0.0003233	dy	0.0016412	0.0016412	0.0016412	0.0016412	alpha	(0.0002291)	(0.0002291)	(0.0002291)	(0.0002291)	Alpha x X1	0.0004398	0.0001432	0.0001432	0.0004398	dy + Alpha x X1	0.0009798	0.0017844	0.0014960	0.0012116	(dy + alpha.X) cos e	0.0025605	0.0017844	0.0014960	0.0012055	P n1 = Kv.dyl	59.19	56.80	47.68	45.77	K1.dyl	8.08	7.46	7.46	7.03	K2.alpa	(0.70)	(0.70)	(0.70)	(0.70)	P N = K1.dyl - K2.alpa	8.77	8.15	8.15	7.73	K3.dyl	7.68	7.09	7.09	6.69	K4.alpa	(1.32)	(1.32)	(1.32)	(1.32)	M t = -K3.dyl + K4.alpa	(8.41)	(8.41)	(8.41)	(8.01)	M t x n	(62.99)	(58.86)	(58.86)	(56.05)	Vi = Phi cos e - Phi sin e					Phi cos	57.90	56.80	47.68	45.54	Phi sin e	(0.87)	0.00	0.00	0.77	Vi	58.78	56.80	47.68	44.77	V n	411.44	397.59	333.77	313.41	H = Phi sin e + Phi cos e					Phi sin e	(5.8)	0.0	0.0	4.6	Phi cos e	8.7	8.15	8.15	7.7	H	20.57	57.06	57.06	12.24	H n	(771.44)	(248.49)	208.61	220.39	V n x	(62.99)	(58.86)	(58.86)	587.64	M n	(624.43)	(307.35)	149.75	531.59	M t = M n + V n x				(460.45)	Total forces at footing of Abutment					Ho	= 220.39 ton			220.39 ton	Vo	= 1,456.20 ton			1,456.20 ton	Mo	= (460.45) t.m			(460.45) t.m	Total forces at pile head					H n = Ho				220.39 ton	V n = Vo				1,456.20 ton	M t = Mo				(460.45) t.m
Side sign degree		Row of pile				total per row																																																																																																																																																																																																																																																																																										
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< 0	(1.875)	(0.625)	(0.625)	1.875																																																																																																																																																																																																																																																																																												
n-nos pile/rows	7.00	7.00	7.00	7.00																																																																																																																																																																																																																																																																																												
a	(5.71)	0.00	0.00	5.71																																																																																																																																																																																																																																																																																												
cos e	0.995	1.000	1.000	0.995																																																																																																																																																																																																																																																																																												
sin e	(0.099)	0.000	0.000	0.099																																																																																																																																																																																																																																																																																												
K1	3.193	3.193	3.193	3.193																																																																																																																																																																																																																																																																																												
K2	3.035	3.035	3.035	3.035																																																																																																																																																																																																																																																																																												
K3	3.035	3.035	3.035	3.035																																																																																																																																																																																																																																																																																												
K4	5.770	5.770	5.770	5.770																																																																																																																																																																																																																																																																																												
Kv	31.631	31.631	31.631	31.631																																																																																																																																																																																																																																																																																												
dx	0.00233494	0.00233494	0.00233494	0.00233494																																																																																																																																																																																																																																																																																												
dy	0.00164116	0.00164116	0.00164116	0.00164116																																																																																																																																																																																																																																																																																												
alpha	(0.00022912)	(0.00022912)	(0.00022912)	(0.00022912)																																																																																																																																																																																																																																																																																												
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dx cos e	0.00232	0.00233	0.00233	0.00232																																																																																																																																																																																																																																																																																												
dy	0.00164	0.00164	0.00164	0.00164																																																																																																																																																																																																																																																																																												
Alpha x X1	(0.00023)	(0.00023)	(0.00023)	(0.00023)																																																																																																																																																																																																																																																																																												
dy + Alpha x X1	0.00043	0.00014	0.00014	0.00043																																																																																																																																																																																																																																																																																												
(dy + alpha.X) sin e	0.00207	0.00178	0.00150	0.00121																																																																																																																																																																																																																																																																																												
dd	(0.00021)	0.00000	0.00000	0.00012																																																																																																																																																																																																																																																																																												
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alpha	(0.0002291)	(0.0002291)	(0.0002291)	(0.0002291)																																																																																																																																																																																																																																																																																												
Alpha x X1	0.0004398	0.0001432	0.0001432	0.0004398																																																																																																																																																																																																																																																																																												
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K2.alpa	(0.70)	(0.70)	(0.70)	(0.70)																																																																																																																																																																																																																																																																																												
P N = K1.dyl - K2.alpa	8.77	8.15	8.15	7.73																																																																																																																																																																																																																																																																																												
K3.dyl	7.68	7.09	7.09	6.69																																																																																																																																																																																																																																																																																												
K4.alpa	(1.32)	(1.32)	(1.32)	(1.32)																																																																																																																																																																																																																																																																																												
M t = -K3.dyl + K4.alpa	(8.41)	(8.41)	(8.41)	(8.01)																																																																																																																																																																																																																																																																																												
M t x n	(62.99)	(58.86)	(58.86)	(56.05)																																																																																																																																																																																																																																																																																												
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V n x	(62.99)	(58.86)	(58.86)	587.64																																																																																																																																																																																																																																																																																												
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mwh-4

Project : Median Flood Control Channel : Perant River Structure : Br. P-2 Station : P+ 84 + 28	Japan International Cooperation Agency (JICA)	Calculation sheet Date Nos of sheet Prepared by
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III.B.5.3 Earthquake condition
Case -III High Water Level

$$dd = (dd \cos \theta) - (dy + \alpha \sin \theta) \sin \theta$$

$$dy = (dd \sin \theta) + (dy + \alpha \sin \theta) \cos \theta$$

			(1)	(2)	(3)	(4)	
X1	m		Ln	Ln	R/a	R/a	total per row
n=sides piers		Side sign	<0	<-	(+)	>0	
cos θ	degree	degree	7.00	(0.625)	0.625	7.00	1.875
sin θ	degree	degree	(5.71)	0.00	0.00	5.71	5.71
K1	Ln	Ln	(0.999)	1.000	1.000	0.999	0.999
K2	Ln	Ln	3.193	3.193	3.193	3.193	3.193
K3	Ln	Ln	3.035	3.035	3.035	3.035	3.035
K4	Ln	Ln	5.770	5.770	5.770	5.770	5.770
Kv	Ln	Ln	31.831	31.831	31.831	31.831	31.831
dx	m	m	0.00237697	0.00237697	0.00237697	0.00237697	0.00237697
dy	m	m	0.00144651	0.00144651	0.00144651	0.00144651	0.00144651
dx = dx cos θ - (dy + α sin θ) sin θ	rad	rad	(0.00038739)	(0.00038739)	(0.00038739)	(0.00038739)	(0.00038739)
dy = dy sin θ + (dy + α sin θ) cos θ			0.00237	0.00238	0.00238	0.00237	0.00237
α			0.00145	0.00145	0.00145	0.00145	0.00145
dy + α x X1			(0.00039)	(0.00039)	(0.00039)	(0.00039)	(0.00039)
dy + α x X2			0.00073	0.00024	(0.00024)	(0.00073)	(0.00073)
(dy + α x X3)			0.00217	0.00169	0.00120	0.00072	0.00072
dx			(0.00022)	0.00000	0.00000	0.00007	0.00007
dy = dx sin θ + (dy + α sin θ) cos θ			0.00238	0.00238	0.00238	0.00229	0.00229
dx = dx cos θ - (dy + α sin θ) sin θ			(0.0002365)	0.0000000	0.0000000	0.0002365	0.0002365
dy			0.0014465	0.0014465	0.0014465	0.0014465	0.0014465
α x X1			(0.0003874)	(0.0003874)	(0.0003874)	(0.0003874)	(0.0003874)
dy + α x X2			0.0007264	0.0002421	(0.0002421)	(0.0007264)	(0.0007264)
dy + α x X3			0.0021729	0.0016866	0.0012044	0.0007202	0.0007202
(dy + α x X4)			0.0021621	0.0016866	0.0012044	0.0007168	0.0007168
dy			0.0019258	0.0016866	0.0012044	0.0006531	0.0006531
P1 = Kv dy			61.29	53.75	38.34	30.34	30.34
K1.dd			8.24	7.59	7.59	7.59	7.59
K2.alpa			(1.18)	(1.18)	(1.18)	(1.18)	(1.18)
P1 = K1.dd - K2.alpa			9.42	8.77	8.77	8.50	8.50
K3.dd			7.53	7.21	7.21	6.98	6.98
K4.alpa			(2.24)	(2.24)	(2.24)	(2.24)	(2.24)
M1 = -K3.dd + K4.alpa			(10.07)	(9.45)	(9.45)	(9.20)	(9.20)
M1 x n			(70.49)	(68.15)	(68.15)	(64.37)	(64.37)
V1 = P1 cos θ - P1 sin θ			60.99	53.75	38.34	30.19	30.19
P1 cos θ			(0.94)	0.00	0.00	0.85	0.85
V1			61.93	53.75	38.34	29.34	29.34
V1 x n			433.48	376.26	268.36	205.39	205.39
H1 = P1 sin θ + P1 cos θ			(6.1)	0.0	0.0	3.0	3.0
P1 sin θ			3.27	8.8	8.8	8.5	8.5
H1			22.91	81.36	81.36	80.33	80.33
V1 x n			(812.78)	(235.18)	(157.72)	383.10	383.10
M1 x n			(70.49)	(68.15)	(64.37)	(64.37)	(64.37)
M1 = M1 x n + V1 x n			(883.27)	(301.30)	101.56	320.73	320.73

Total forces at footing of Abutment

Ho	= 225.95	ton
Vo	= 1283.49	ton
Mo	= (762.27)	tm

Total forces at pile head

H1n = Ho	225.95	ton
V1n = Vo	1283.49	ton
M1n = Mo	(762.27)	tm

III-B-3

Project : Madan Flood Control

Channel : Percut River

Structure : Br. P-2

Station : Pa 84 + 28

Japan International Cooperation Agency (JICA)

Calculation sheet

Date

Nos of sheet

Prepared by

III.B.5 Combined load at Pier section

III.B.5.1 Normal condition

Case - I, High water level

width of foot = 10 m

Total weight					
Description	Dead load	Line load	Uniform load	Pedestrian load	total ton
		L/s span =			
Total weight	485.37	27.27	190.60	18.60	721.84
Reaction to pier	242.69	13.64	95.30	9.30	360.92
Load per m width	24.27	1.36	9.53	0.93	36.09
	R/s span = 40.00				
Total weight	900.51	27.27	247.79	24.00	1,099.57
Reaction to pier	400.26	13.64	123.90	12.00	549.79
Load per m width	40.03	1.36	12.39	1.20	54.98
Total load to pier	64.29	2.73	21.92	2.13	91.07

Braking forces L/s = 5% x "D" load (SHA No.1725-1989-F) Dep.PU

Braking forces L/s = 0.05 x 10.893 = 0.54467 ton

Braking forces R/s = 0.05 x 13.753 = 0.68765 ton

Section 1 - 1

Load	Description					Vo ton	Ho ton	Yo m	Xo m	Mo tm
	Yo m	H m	X m	W t/m3	Weight ton					
C3	2.40	1.00	x	0.80	1.440	1.440	0.300	0.087	0.016	0.432
C7	2.40	x	0.50	x	0.20	0.240	0.000	0.300	0.000	0.016
Pq					0.000					
Rv					36.100	36.100	0.300			10.830
sum					37.780	0.000				11.278

Section 2 - 2

H1 = 0.25 m	W1 = 0.40 m	H4 = 0.50 m	W4 = 0.50 m	W4 = 1.000 m
H2 = 1.10 m	W2 = 0.80 m	H6 = 0.50 m	W6 = 0.50 m	W6 = 0.200 m
H3 = 1.00 m	W3 = 0.60 m	H7 = 0.50 m	W7 = 0.50 m	W7 = 0.200 m
Rv1 = 36.09 ton	Rv1 = 54.85 ton	H2+H6 = 1.60 m	H1+H7 = 1.500 m	
Pq1 = 0.00 ton	Pq1 = 0.00 ton			

Load	Description				Vo ton	Ho ton	Yo m	Xo m	Mo-Vo-Xo tm
	Y m	H m	X m	W t/m3	Weight ton				
C1	2.40	x	0.25	x	0.40	0.240			
C2	2.40	x	1.10	x	0.60	1.584			
C3	2.40	x	1.00	x	0.60	1.440			
C4	2.40	x	0.50	x	1.00	1.200			
C6	0.50	x	2.40	x	0.20	0.120			
C7	0.50	x	2.40	x	0.20	0.120			
Pq1					0.000				
Rv1					36.090	36.090	1.600		0.000
Rv1					54.850	54.850	1.500		0.000
Braking forces L/s					0.545		1.600		0.871
Braking forces R/s					0.688		1.500		1.031
sum					95.644	1.232			1.903

Section 3 - 3

H1	=	0.25 m	W1	=	0.40 m	H6	=	0.50 m	W6	=	0.200 m	
H2	=	1.10 m	W2	=	0.60 m	H7	=	0.50 m	W7	=	0.200 m	
H3	=	1.00 m	W3	=	0.60 m	H8	=	4.40 m	W8	=	0.200 m	
H4	=	4.40 m	W4	=	1.00 m	H9	=	4.40 m	W9	=	0.200 m	
Pq.1	=	0.00 ton				H2+H6+H4	=	6.000 m		=	6.000 m	
Pq.2	=	0.00 ton				H3+H7+H4	=	5.900 m		=	5.900 m	

Load	Description										Vo	Ho	Xo	Yo	M		
	Y	x	H	x	W	=	Weight	ton	ton	m						m	t.m
	m		m		t/m3												
C1	=	2.40 x	0.25 x	0.40		=		0.240									
C2	=	2.40 x	1.10 x	0.60		=		1.584									
C3	=	2.40 x	1.00 x	0.60		=		1.440									
C4	=	2.40 x	0.50 x	1.00		=		1.200									
C8	=	0.50 x	2.40 x	0.50 x	0.20			0.120									
C9	=	0.50 x	2.40 x	0.50 x	0.20			0.120									
Pq.1						=		0.000		0.000		6.000		0.000			
Pq.7						=		0.000		0.000		5.900		0.000			
Pv.1						=		36.090									
Pv.7						=		54.850									
Braking forces L/s						=		0.545		0.545		6.000		3.268			
Braking forces R/s						=		0.688		0.688		5.900		4.057			
sum						=		95.644		1.232		7.325					

Project : Medan Flood Control			Japan International Cooperation Agency (JICA)			Calculation sheet																																																			
Channel : Percut River						Date	Nos of sheet																																																		
Structure : Br. P-2						Prepared by																																																			
Station : Ps 84 + 28																																																									
III.B.8.2 Normal condition Case - II Low water level width of foot = 10 m																																																									
Total weight <table> <tr> <th>Description</th><th>Dead load</th><th>Live load</th><th>Uniform Pedestrian load</th><th>total load</th></tr> <tr> <td colspan="5">L/s span = 31.00</td></tr> <tr> <td>Total weight</td><td>485.37</td><td>27.27</td><td>190.60</td><td>18.60</td></tr> <tr> <td>Reaction to pier</td><td>242.69</td><td>13.64</td><td>95.30</td><td>9.30</td></tr> <tr> <td>Load per m width</td><td>24.27</td><td>1.36</td><td>9.53</td><td>0.93</td></tr> <tr> <td colspan="5">R/s span = 40.00</td></tr> <tr> <td>Total weight</td><td>800.51</td><td>27.27</td><td>247.79</td><td>24.00</td></tr> <tr> <td>Reaction to pier</td><td>400.26</td><td>13.64</td><td>123.90</td><td>12.00</td></tr> <tr> <td>Load per m width</td><td>40.03</td><td>1.36</td><td>12.39</td><td>1.20</td></tr> <tr> <td>Total load to pier</td><td>64.29</td><td>2.73</td><td>21.92</td><td>2.13</td></tr> </table> Braking forces L/s = 5 % x D * load (SM No.1725-1989-F).Dep.PU Braking forces L/s = 0.05 x 10.893 = 0.5446 ton Braking forces R/s = 0.05 x 13.753 = 0.6876 ton								Description	Dead load	Live load	Uniform Pedestrian load	total load	L/s span = 31.00					Total weight	485.37	27.27	190.60	18.60	Reaction to pier	242.69	13.64	95.30	9.30	Load per m width	24.27	1.36	9.53	0.93	R/s span = 40.00					Total weight	800.51	27.27	247.79	24.00	Reaction to pier	400.26	13.64	123.90	12.00	Load per m width	40.03	1.36	12.39	1.20	Total load to pier	64.29	2.73	21.92	2.13
Description	Dead load	Live load	Uniform Pedestrian load	total load																																																					
L/s span = 31.00																																																									
Total weight	485.37	27.27	190.60	18.60																																																					
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Load per m width	24.27	1.36	9.53	0.93																																																					
R/s span = 40.00																																																									
Total weight	800.51	27.27	247.79	24.00																																																					
Reaction to pier	400.26	13.64	123.90	12.00																																																					
Load per m width	40.03	1.36	12.39	1.20																																																					
Total load to pier	64.29	2.73	21.92	2.13																																																					
Section 1 - 1																																																									
H1 = 1.00 m W1 = 0.60 m Yconc = 2.40 t/m3 PqJ = 0.00 ton Ywet = 1.85 t/m3 Ysat = 2.00 t/m3 Yw = 1.00 t/m3																																																									
Load	Yc	x	H	x	Description	Weight	M																																																		
	m		m		t/m3	ton	t.m																																																		
C3	2.40	x	1.00	x	0.60	1.440	0.432																																																		
C7	2.40	x	0.50	x	0.20	0.240	0.016																																																		
Pq						0.000																																																			
Rv						36.100	10.830																																																		
sum						37.780	11.278																																																		
Section 2 - 2																																																									
H1 = 0.25 m H2 = 1.10 m H3 = 1.00 m RvJ = 36.09 ton PqJ = pc ton W1 = 0.40 m W2 = 0.60 m W3 = 0.60 m H4 = 0.50 m H6 = 0.50 m H7 = 0.50 m H2+H6 = 1.60 m H1+H7 = 1.60 m																																																									
Load	Y	x	H	x	Description	Weight	M																																																		
	m		m		t/m3	ton	t.m																																																		
C1	2.40	x	0.25	x	0.40	0.240	0.000																																																		
C2	2.40	x	1.10	x	0.60	1.584	1.600																																																		
C3	2.40	x	1.00	x	0.60	1.440	1.500																																																		
C4	2.40	x	0.50	x	1.00	1.200	0.000																																																		
C6	2.40	x	0.50	x	0.20	0.120	0.000																																																		
C7	2.40	x	0.50	x	0.20	0.120	0.000																																																		
PqJ						0.000	0.000																																																		
PqJ						36.090	0.000																																																		
RvJ						54.850	0.871																																																		
Braking forces L/s						0.545	0.871																																																		
Braking forces R/s						0.688	1.031																																																		
sum						95.644	1.903																																																		

Section 3 - 3													
H1 = 0.25 m	W1 = 0.40 m	H6 = 0.50 m	W6 = 0.200 m										
H2 = 1.10 m	W2 = 0.60 m	H7 = 0.50 m	W7 = 0.200 m										
H3 = 1.00 m	W3 = 0.60 m	H8 = 0.50 m	W8 = 0.200 m										
H4 = 0.40 m	W4 = 1.00 m	H9 = 0.40 m	W9 = 0.200 m										
PqJ = pc ton		H2+H6+H4 = 2.10 m											
PqJ = 0.00 ton		H3+H7+H4 = 2.10 m											
Load		Description		Weight		M		Vo		Ho		Yo	
Y	X	H	W	ton	m	ton	t.m	ton	m	ton	m	ton	m
C1	2.40 x	0.25 x	0.40	0.240	0.240	0.058		0.240					
C2	2.40 x	1.10 x	0.60	1.584	0.300	0.475		1.584					
C3	2.40 x	1.00 x	0.60	1.440	0.300	0.432		1.440					
C4	2.40 x	0.50 x	0.20	0.240	0.120	0.029		0.240					
C6	2.40 x	0.50 x	0.20	0.240	0.120	0.029		0.240					
C9	2.40 x	0.50 x	0.20	0.240	0.120	0.029		0.240					
PqJ				0.000									
PqJ				36.090	0.300	10.827		36.090					
RvJ				54.850	0.300	16.455		54.850					
				0.545	0.688	0.373		0.545					
				0.688	1.232	0.373		0.688					
				95.644	1.232	31.902		95.644					
Braking forces L/s													
Braking forces R/s													
Section 4 - 4													
H1 = 0.30 m	W1 = 1.85 m												
H2 = 1.00 m	W2 = 1.85 m												
Hwet = 3.80 m													
Hsat = 0.00 m													
Load		Description		Weight		M		Vo		Ho		Yo	
Y	X	H	W	ton	m	ton	t.m	ton	m	ton	m	ton	m
C5-1	0.50 x	2.40 x	0.30 x	1.85	0.668	0.668		1.85		0.617			
C5-2	2.40 x	1.00 x	1.85	4.440	4.440	4.440		4.440		0.925			
Ewet	1.85 x	3.80 x	1.85	13.008	13.008	13.008		13.008		0.925			
Esat	2.00 x	0.00 x	1.85	0.000	0.000	0.000		0.000		0.925			
Pq.3				(54.54)	(54.54)	(54.54)		(54.54)		1.225			
Pq.3				(53.05)	(53.05)	(53.05)		(53.05)		0.625			
				(89.479)	(89.479)	(89.479)		(89.479)					
Section 5 - 5													
H1 = 0.30 m	W1 = 1.85 m												
H2 = 1.00 m	W2 = 1.85 m												
Hwet = 3.80 m													
Hsat = 0.00 m													
Load		Description		Weight		M		Vo		Ho		Yo	
Y	X	H	W	ton	m	ton	t.m	ton	m	ton	m	ton	m
C5-1	0.50 x	2.40 x	0.30 x	1.85	0.668	0.668		1.85		0.617			
C5-2	2.40 x	1.00 x	1.85	4.440	4.440	4.440		4.440		0.925			
Ewet	1.85 x	3.80 x	1.85	13.008	13.008	13.008		13.008		0.925			
Esat	2.00 x	0.00 x	1.85	0.000	0.000	0.000		0.000		0.925			
Pq.1				(49.03)	(49.03)	(49.03)		(49.03)		1.225			
Pq.2				(51.41)	(51.41)	(51.41)		(51.41)		0.625			
				(82.328)	(82.328)	(82.328)		(82.328)					
Section 6 - 6													
H1 = 1.00 m	W1 = 1.50 m												
H2 = 1.35 m	W2 = 1.50 m												
Length = 1.80 m	Rv tot. = 90.94 ton												
mqh-2													

Project : Medan Flood Control	Japan International Cooperation Agency (JICA)	Calculation sheet
Channel : Perout River		Date
Structur : Br. P-2		Nos of sheet
Station : P+84 + 28		Prepared by

III.B.8.3 Earthquake condition
Case - III, High water level

width of foot = 10 m

Total weight

Description	Dead load	Line load	Uniform Pedestrian load	total ton
L/s span = 31.00				
Total weight	485.37	27.27	190.60	18.60
Reaction to pier	242.69	13.64	95.30	9.30
Load per m width	24.27	1.36	9.53	0.93
R/s span = 40.00				
Total weight	800.51	27.27	247.79	24.00
Reaction to pier	400.26	13.64	123.90	12.00
Load per m width	40.03	1.35	12.39	1.20
Total load to pier	64.29	2.73	21.92	2.13

Braking forces L/s = 5% x "D" * load (SNI No.1725-1989-F) Dep.PU

Braking forces L/s = 0.05 x 10.893 = 0.5446 ton

Braking forces R/s = 0.05 x 13.753 = 0.6876 ton

Section 1 - 1

H1 = 1.00 m Ywet = 1.85 t/m3
W1 = 0.60 m Ysat = 2.00 t/m3
Yconcrete = 2.40 t/m3
Pq,r = 5.98 ton

Load	Description			Weight		
	Yc	x	H	m	W	ton
C3	2.40	x	1.00	x	0.60	1.440
C7	2.40	x	0.50	x	0.20	0.240
Pq						5.984
Rr						36.100
	sum					37.780
						5.984
						10.830
						11.278

Section 2 - 2

H1 = 0.25 m W1 = 0.40 m H4 = 0.50 m W4 = 1.000 m
H2 = 1.10 m W2 = 0.60 m H6 = 0.50 m W6 = 0.200 m
H3 = 1.00 m W3 = 0.60 m H7 = 0.50 m W7 = 0.200 m
Rv,l = 36.09 ton Rv,r = 54.98 ton H2+H6 = 1.60 m H1+H7 = 1.500 m
Pq,l = 3.64 ton Pq,r = 6.00 ton

Load	Description			Weight		
	Y	x	H	m	W	ton
C1	2.40	x	0.25	x	0.40	0.240
C2	2.40	x	1.10	x	0.60	1.584
C3	2.40	x	1.00	x	0.60	1.440
C4	2.40	x	0.50	x	1.00	1.200
C5	0.50	x	2.40	x	0.50	0.120
C7	0.50	x	2.40	x	0.50	0.120
Pq,l						3.640
Pq,r						6.004
Rv,l						36.092
Rv,r						54.979
	sum					95.774
						10.876
						1.600
						1.500
						1.600
						1.500
						0.871
						1.031
						18.733

Braking forces L/s

Braking forces R/s

Load	Description									
	Y	x	H	x	W	x	L	=	Weight	
C13	=	2.40	x	1.10	x	1.50	x	1.00	=	3.960
C14	=	0.50	x	2.40	x	1.00	x	1.00	=	1.800
Rv										90.94
Rservice										7.500
	sum									
										104.200
										9.495

Section 7 - 7

H1 = 1.00 m W1 = 1.50 m

H2 = 1.00 m W2 = 1.50 m

Back fill depth = 0.00 m

Load	Description									
	Y	x	H	x	W	x	L	=	Weight	
C15	=	0.50	x	2.40	x	1.00	x	1.50	=	1.800
C16	=	2.40	x	1.30	x	1.50			=	4.680
Ewet.	=	2.40	x	3.80	x	1.50			=	13.680
Est.	=								=	0.000
Rp row1									=	(49.03)
Rp row2									=	(49.03)
	sum									
										(77.900)
										(19.651)

Arch - 2

[illegible]

Load	Description										Vo	Ho	Xo	Yo	M = Vo Xo
	Y		x		H		W		Weight	ton					
	m		m		m		t/m ³								
C5r1	0.50	x	2.40	x	0.30	x	1.85		=	0.666	0.666	0.617	m		0.411
C5r2		=	2.40	x	1.00	x	1.85		=	4.440	4.440	0.925			4.107
E5w1		=	1.85	x	0.00	x	1.85		=	0.000	0.000	0.925			0.000
E5a1r		=	2.00	x	3.80	x	1.85		=	14.060	14.060	0.925			13.006
Ro.4									=	(29.34)	(29.340)	1.225			(35.942)
Ro.3									=	(38.34)	(38.340)	0.625			(23.963)
									=	(38.34)	(48.514)				(42.381)
									sum						

[illegible]

Section 5 - 6			
H1	=	1.00 m	W1 = 1.50 m
H2	=	1.35 m	W2 = 1.50 m
Length	=	1.80 m	Pv tot. = 91.07 ton
pav-1			

[illegible]

1-400

Project : Medan Flood Control		Japan International Cooperation Agency (JICA)		Calculation sheet							
Channel : Percut River				Date	Nos of sheet						
Structur : Br. P-2				Prepared by							
Station : P+84 + 28											
III.B.8.4 Earthquake condition CAsse - IV, Low water level width of foot = 10 m											
Total weight											
Description	Dead load	Line load	Uniform Pedestrian load	total load	ton						
L/s span = 31.00											
Total weight	485.37	27.27	190.60	18.60	721.84						
Reaction to pier	242.69	13.64	95.30	9.30	360.92						
Load per m width	24.27	1.36	9.53	0.93	36.09						
R/s span = 40.00											
Total weight	800.51	27.27	247.79	24.00	1,099.57						
Reaction to pier	400.26	13.64	123.90	12.00	549.79						
Load per m width	40.03	1.36	12.39	1.20	54.98						
Total load to pier	64.29	2.73	21.92	2.13	91.07						
Braking forces L/s = 5% x "D" load (SNI No.1725-1989-F).Dep.PU Braking forces L/s = 0.05 x 10.893 = 0.5448 ton Braking forces R/s = 0.05 x 13.753 = 0.6876 ton											
Section 1 - 1											
Load	Yc	x	H	x	W						
	m		m		t/m3						
C3	2.40	x	1.00	x	0.60						
C7	2.40	x	0.50	x	0.20						
Pq											
Rv											
sum											
				Vo	ton						
				Ho	ton						
				Yo	m						
				Xo	m						
					M						
					t.m						
					0.432						
					0.067						
					5.984						
					36.100						
					37.780						
					5.984						
					11.278						
Section 2 - 2											
H1	=	0.25 m	W1	=	0.40 m						
H2	=	1.10 m	W2	=	0.60 m						
H3	=	1.00 m	W3	=	0.60 m						
Rv,l	=	36.09 ton	Pq,r	=	54.98 ton						
Pq,l	=	3.84 ton	H2+H6	=	1.60 m						
			H1+H7	=	1.500 m						
Weight ton											
Load	Y	x	H	x	W						
	m		m		t/m3						
C1	2.40	x	0.25	x	0.40						
C2	2.40	x	1.10	x	0.60						
C3	2.40	x	1.00	x	0.60						
C4	2.40	x	0.50	x	1.00						
C6	0.50	x	2.40	x	0.50						
C7	0.50	x	2.40	x	0.50						
Pq,l											
Pq,r											
Rv,l											
Braking forces L/s Braking forces R/s											
				Vo	ton						
				Ho	ton						
				Yo	m						
				Xo	m						
					M						
					t.m						
					0.240						
					1.584						
					1.440						
					1.200						
					0.120						
					0.120						
					3.840						
					6.004						
					36.092						
					54.979						
					0.545						
					0.688						
					95.774						
					10.878						
					16.723						

Section 3 - 3

H1	=	0.25 m	W1	=	0.40 m	H6	=	0.50 m	W6	=	0.200 m
H2	=	1.10 m	W2	=	0.60 m	H7	=	0.50 m	W7	=	0.200 m
H3	=	1.00 m	W3	=	0.60 m	H8	=	0.50 m	W8	=	0.200 m
H4	=	1.00 m	W4	=	0.60 m	H9	=	0.50 m	W9	=	0.200 m
Pq,l	=	3.84 ton									
Pq,r	=	6.00 ton									

Load	Description			Vo	Ho	Yo	M
	Y	X	W				
C1	2.40	0.25	0.40	0.240			
C2	2.40	1.10	0.60	1.584			
C3	2.40	1.00	0.60	1.440			
C4	2.40	0.50	1.00	1.200			
C8	0.50	2.40	0.50	0.120			
C9	0.50	2.40	0.50	0.120			
Pq,l				3.840			21.842
Pq,r				6.004			35.423
Rv,l				36.092			6.000
Rv,r				54.979			5.900
	sum			95.774	10.876		64.589

Section 4 - 4

H1	=	0.30 m	W1	=	1.85 m
H2	=	1.00 m	W2	=	1.85 m
H wet	=	0.00 m			
H sat	=	3.80 m			

Load	Description			Vo	Ho	Yo	M
	Y	X	W				
C5-1	0.50	2.40	0.30	0.15	0.617		0.411
C5-2	2.40	1.00	1.85	4.440	0.925		4.107
Ewet,r	1.85	0.00	1.85	0.000	0.925		0.000
Esat,r	2.00	3.80	1.85	14.060	0.925		13.008
Pq-3				(44.77)			(54.843)
Pq-4				(47.68)			(29.800)
	sum			73.284			(67.120)

Section 5 - 5

H1	=	0.30 m	W1	=	1.85 m
H2	=	1.00 m	W2	=	1.85 m
H wet	=	0.00 m			
H sat	=	3.80 m			

Load	Description			Vo	Ho	Yo	M
	Y	X	W				
C5-1	0.50	2.40	0.30	0.15	0.617		0.411
C5-2	2.40	1.00	1.85	4.440	0.925		4.107
Ewet,l	1.85	0.00	1.85	0.000	0.925		0.000
Esat,l	2.00	3.80	1.85	14.060	0.925		13.008
Pq-1				(58.78)			(72.006)
Pq-2				(56.80)			(54.482)
	sum			(96.414)			(108.965)

Section 6 - 6

H1	=	1.00 m	W1	=	1.50 m
H2	=	1.35 m	W2	=	1.50 m
Length	=	1.80 m	Rv tot.	=	91.07 ton

Load	Description										Vo	Ho	Xo	Yo	M = Vo.Xo
	Y	x	H	x	W	x	L	=	Weight						
C13	=	2.40	x	1.10	x	1.50	x	1.00	=	3.960	ton				2.970
C14	=	0.50	x	2.40	x	1.00	x	1.50	x	1.800	ton				0.900
Rv										91.07					0.000
Reserve										7.500	7.500			0.750	5.625
								sum			104.331				9.495

Section 7 - 7

H1	=	1.00	m	W1	=	1.50	m
H2	=	1.00	m	W2	=	1.50	m
				Back fill depth	=	3.80	m

Load	Description										Vo	Ho	Xo	Yo	M = Vo.Xo
	Y	x	H	x	W	x	L	=	Weight						
C15	=	0.50	x	2.40	x	1.00	x	1.50	=	1.800	ton	1.800		0.900	t.m
C16	=	2.40	x	1.30	x	1.50	x	1.50	=	4.680	4.680			3.510	
Ewet	=	2.40	x	0.00	x	1.50	x	1.50	=	0.000	0.000			0.750	0.000
Esat	=	2.40	x	3.80	x	1.50	x	1.50	=	13.680	13.680			0.750	10.260
Rp row1										(58.78)	(58.780)			0.700	(41.146)
Rp row2											(58.800)				(26.476)
								sum			(95.420)				

Project : Medan Flood Control	Japan International Cooperation Agency (JICA)	Calculation sheet
Channel : Perut River		Date
Structure : Br - P 2		Nos of sheet
Station : Ps 84 + 28		Prepared by

III.B.9 Resume forces on Pier Section

Case - I, Normal Condition (HWL)

Case	Forces	Unit	Section						
			1	2	3	4	5	6	7
Case - I	Moment	t.m	11.278	1.903	7.325	65.127	80.177	9.495	18.251
		kg.cm	1,127,800	190,300	732,500	6,512,700	8,017,700	949,500	1,825,100
	Shear	ton	0.0	1.232	1.232	0.0	0.0	0.0	0.0
		kg	0.0	1,232	1,232.0	0.0	0.0	0.0	0.0
Normal	ton	37.780	95.644	70.584	70.584	70.584	73.894	104.200	73.900
	kg	37,780	95,644	70,584	70,584	73,894	104,200	73,900	

Case - II, Normal Condition (LWL)

Case	Forces	Unit	Section						
			1	2	3	4	5	6	7
Case - II	Moment	t.m	11.278	1.903	7.325	83.420	87.028	9.495	19.651
		kg.cm	1,127,800	190,300	732,500	8,342,000	8,702,800	949,500	1,965,100
	Shear	ton	0.0	1.232	1.232	0.0	0.0	0.0	0.0
		kg	0.0	1,232	1,232.0	0.0	0.0	0.0	0.0
Normal	ton	37.780	95.644	70.584	82.328	82.328	82.328	104.200	77.900
	kg	37,780	95,644	70,584	82,328	82,328	82,328	104,200	77,900

Case - III, Earthquake Condition (HWL)

Case	Forces	Unit	Section						
			1	2	3	4	5	6	7
Case - III	Moment	t.m	11.128	16.703	64.473	67.120	108.965	9.495	28.681
		kg.cm	1,112,800	1,670,300	6,447,300	6,712,000	10,896,500	949,500	2,868,100
	Shear	ton	5.984	10.857	10.857	0.0	0.0	0.0	0.0
		kg	5,984	10,857	10,857	0.0	0.0	0.0	0.0
Normal	ton	37.780	95.643	95.643	73.284	73.284	96.414	104.331	95.520
	kg	37,780	95,643	95,643	73,284	73,284	96,414	104,331	95,520

Case - IV, Earthquake Condition (LWL)

Case	Forces	Unit	Section						
			1	2	3	4	5	6	7
Case - IV	Moment	t.m	11.278	16.703	64.473	67.120	108.965	9.495	28.476
		kg.cm	1,127,800	1,670,300	6,447,300	6,712,000	10,896,500	949,500	2,647,500
	Shear	ton	5.984	10.857	10.857	0.0	0.0	0.0	0.0
		kg	5,984	10,857	10,857	0.0	0.0	0.0	0.0
Normal	ton	37.780	95.643	95.643	73.284	73.284	96.414	104.331	95.420
	kg	37,780	95,643	95,643	73,284	73,284	96,414	104,331	95,420

mmh

Project : Medan Flood Control Channel : Percut River Structure : Br - P 2 Station : Pe 84 + 28			Japan International Cooperation Agency (JICA)		Calculation sheet		
III.B 10 Reinforcing steel bar of Pier section			Section - 3			Date Nos of sheet Prepared by	
Item	Description	unit	Normal condition Calcu- lation	Earth quake condition Calcu- lation			
Allowance stresses							
concrete compressive steel tensile concrete shear transformend Moment	$f_c = K - 225$	kg/cm2	75	75	75	75	
	f_s	kg/cm2	1400	1400	1400	1400	
	tau without r.bar	kg/cm2	8	8	8	8	
	tau with r.bar	kg/cm2	20	20	20	20	
Moment	$n = E_s/E_c$	kg/cm2	21	21	21	21	
	M	kg.cm	7.325	7.325	64.473	64.473	
Shear	S	ton	732.500	732.500	6.447.300	6.447.300	
	N	kg	1.232	1.232	10.857	10.857	
Normal	N	ton	70.854	70.854	95.643	95.643	
	N	kg	70.854	70.854	95.643	95.643	
dimension	b	cm	100	100	100	100	
	ht	cm	140	140	140	140	
	d'	cm	10	10	10	10	
	h	cm	130	130	130	130	
	b.h	cm2	13.000	13.000	13.000	13.000	
	$eo1 = M/N$	cm	10.34	10.34	67.41	67.41	
	$eo2 = 1/30.M, > 2\text{ cm}$	cm	4.67	4.67	2.00	2.00	
	used eo2	cm	2.00	2.00	69.41	69.41	
	$eo = eo1 + eo2$	cm	12.34	12.34	0.50	0.50	
	eo/ht (see table)	cm	0.09	0.09	4.40	4.40	
Calculation for reinfor- cement bar with steel full working	C	cm	4.40	4.40	4.40	4.40	
	lk	cm	440	440	440	440	
	lk/100.lk	cm	14.000	14.000	14.000	14.000	
	$(lk/100.lk)^2$	cm	0.0314	0.0314	0.0010	0.0010	
	$e1 = C.lk/(100.lk)^2$	cm	0.0010	0.0010	0.608	0.608	
	$e2 = 0.15.lk$	cm	21.00	21.00	33.95	33.95	
	$e = eo + e1 + e2$	cm	93.95	93.95	151.02	151.02	
	$ea = e + 1/2.M - d'$	cm	6.656.494	6.656.494	14.443.864	14.443.864	
	$M' = N.ea$	kg.cm	139.786.366	139.786.366	303.321.137	303.321.137	
	$n.M'/(b.f_s)$	kg/cm	140.000	140.000	2.166.58	2.166.58	
$Ca = h / \sqrt{n.M'/(b.f_s)}$	kg/cm	998.47	998.47	31.60	31.60		
n/c	kg/cm	4.114	4.114	2.793	2.793		
$eo = f_s / (n.f_c)$	kg/cm	1.575	1.575	0.8889	0.8889		
delta	kg/cm	0.8889	0.8889	0.60	0.60		
from table fined Ca	kg/cm	2.785	2.785	2.785	2.785		
fine a > eo	kg/cm	1.564	1.564	2.103	2.103		
a'	kg/cm	2.103	2.103	0.390	0.390		
ksi(i)	kg/cm	0.390	0.390	0.873	0.873		
zeta	kg/cm	0.873	0.873	0.688	0.688		
100 nw	kg/cm	0.688	0.688	0.00688	0.00688		
nw	kg/cm	0.00688	0.00688	0.00033	0.00033		
w	kg/cm	0.00033	0.00033	0.72	0.72		
ea/h	kg/cm	0.72	0.72	4.260	4.260		
from table fined i	kg/cm	4.260	4.260	4.26	4.26		
i.A = w.bh	kg/cm	4.26	4.26	1.00	1.00		
A	cm2	1.00	1.00	2.555	2.555		
A' = delta.i	cm2	2.555	2.555				

= 1.327		A' - used bar dia. 25 mm @ 12.5 cm,		Area @ bar = 4.900		
= 10		nos of bar = 10		total area = 10 x 4.900		
= 13.27 cm >		steel bar full working		total area = 10 x 4.900 cm2 >		
tension of bar compressive concrete tensile of bar	n.e fs fc' = fs / (n.e) fs' = fs / e'	kg/cm2	32.84		1,400.00	
			42.63		42.63	
			665.72		665.72	
			11,375.00		11,375.00	
shear concrete	Tca = S / (7/8.b.h)	kg/cm2	0.11		0.11	
Section -- 5						
Item	Description	unit	Normal condition		Earth quake condition	
			Calcu- lation	Calcu- lation	Calcu- lation	Calcu- lation
Allowance stresses						
concrete compressive steel tensile concrete shear	fc = K - 225	kg/cm2	75	75	75	75
	fs	kg/cm2	1400	1400	1400	1400
	ts without r.bar	kg/cm2	8	8	8	8
	ts with r.bar	kg/cm2	20	20	20	20
transformend	n = Es/EC	kg/cm2	21	21	21	21
Moment	M	tm	87,028	87,028	108,965	108,965
Shear	S	kg cm	8,702,800	8,702,800	10,896,500	10,896,500
		ton	82,328	82,328	96,414	96,414
dimension		kg	82328	82328	96414	96414
	b	cm	100	100	100	100
	ht	cm	150	150	150	150
	d'	cm	10	10	10	10
	h	cm	140	140	140	140
	b.h	cm2	14,000	14,000	14,000	14,000
	M	kg cm	8,702,800	8,702,800	10,896,500	10,896,500
Calculation for reinfor - cement bar with steel full working	n.M	kg cm	182,758,800	182,758,800	228,826,500	228,826,500
	b.fs	kg/cm	140,000	140,000	140,000	140,000
	n.M / (b.fs)	kg/cm	1,305.42	1,305.42	1,634.48	1,634.48
	@sqrt(n.M / (b.fs))	kg/cm	36.13	36.13	40.43	40.43
	Ca=h / (@sqrt(n.M/b.fs) see PBI 1971 N.I-2	kg/cm	3,875	3,875	3,463	3,463
	n/c	kg/cm	1,575	1,575	1,575	1,575
	eo = fs / (n.fc)	kg/cm	0.8889	0.8889	0.8889	0.8889
	delta	kg/cm	0.60	0.60	0.60	0.60
	from table fined Ca	kg/cm	3.962	3.962	3.962	3.962
	fine eo > eo	kg/cm	2.448	2.448	2.448	2.448
Calculation for reinfor - cement bar with steel full working	e'	kg/cm	3.865	3.865	3.865	3.865
	ksi(j)	kg/cm	0.285	0.285	0.285	0.285
	zeta	kg/cm	0.904	0.904	0.904	0.904
	100 nw	kg/cm	6.724	6.724	6.724	6.724
	nw	kg/cm	0.06724	0.06724	0.06724	0.06724
	w	kg/cm	0.00320	0.00320	0.00320	0.00320
	A	cm2	44.83	44.83	44.83	44.83
	A' = delta.i.A	cm2	26.90	26.90	26.90	26.90
	A' - used bar dia. 25 mm @ 12.5 cm,	cm2	4.900	4.900	4.900	4.900
	Area @ bar = 4.900	cm2	10	10	10	10
nos of bar = 10	cm2	10	10	10	10	
total area = 10 x 4.900 cm2 >	cm2	49.00	49.00	49.00	49.00	
steel bar full working						
tension of bar compressive concrete tensile of bar	n.e fs fc' = fs / (n.e) fs' = fs / e'	kg/cm2	51.41		1,400.00	
			27.23		27.23	
			362.23		362.23	
			12,250.00		12,250.00	
shear concrete	Tca = S / (7/8.b.h)	kg/cm2	6.72		6.72	
bar						