Ministry of Transport
Vietnam Expressway Corporation
Project Management Unit No. 85

THE WORLD BANK

IDA Credit No. : 4779-VN
Project ID No. : P106235

Consulting Services
for
Detailed Design for Danang - Quang Ngai Expressway Development Project

Detailed Engineering Design Report (Final)

Volume 4: Structural Calculation Report (PKG6)

Volume 4.2: Structural Calculation Report (PKG6, Bridges)

Section 4.2.2

4. ORB12 BRIDGE
5. CB12 BRIDGE
6. ORB13 BRIDGE
7. OP11A BRIDGE
8. LRB09 BRIDGE
9. CB13 BRIDGE

July 15, 2013

The Joint Venture of
NIPPON KOEI CO.,LTD.
NIPPON ENGINEERING CONSULTANTS CO.,LTD.
CHODAI CO.,LTD.
THAI ENGINEERING CONSULTANTS CO., LTD.

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(Dịch vụ tư vấn Thiết kế kỹ thuật dự án Đường cao tốc Đà Nẵng - Quãng Ngãi)

Detailed Engineering Design Report (Final)
(Báo cáo thiết kế kỹ thuật chi tiết)

Volume 4: Structural Calculation Report (PKG6)
(Tập 4: Báo cáo tính toán kết cấu (Gói thầu 6))

Volume 4.2: Structural Calculation Report (PKG6, Bridges)
(Tập 4.2: Báo cáo tính toán kết cấu (Gói thầu 6, Phần cầu))

Section 4.2.2

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Date (Ngày)	July 15, 2013 (15/07/2013)	July 15, 2013 (15/07/2013)	July 15, 2013 (15/07/2013)	July 15, 2013 (15/07/2013)

THE JOINT VENTURE OF NK-NE-CHODAI-TEC/LIÊN DANH TƯ VẤN

Project Manager/Giám đốc Dự án

長井崇泰

For ***Ichizuru Ishimoto***

Da Nang, July 15, 2013/Đà Nẵng ngày 15 tháng 07 năm 2013

MINISTRY OF TRANSPORT

VIETNAM EXPRESSWAY CORPORATION - PROJECT MANAGEMENT UNIT NO.85

DA NANG - QUANG NGAI EXPRESSWAY DEVELOPMENT PROJECT

PACKAGE: 6

BRIDGE

ORB12

CALCULATION SHEETS

HA NOI, 2012

Table of content - ORB12 Bridge

A. Substructure design

1. Abutment A2
2. Bored pile capacity

Da Nang Quang Ngai Expressway project

BRIDGE

ORB12

CALCULATION SHEETS

ABUTMENT A2

2013

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1.Structure dimensions and Loads
2. Foundation analysis
3. Elements checks
4.Pile Design

	Da Nang Quang Ngai Expressway project	Item.	Eng.	Date.	Sign.
	ORB12 BRIDGE	Design			
	DETAIL DESIGN	Check			
	ABUTMENT A2	Revise			

LOAD COMPONENTS

Assumptions :

1. Bridge is considered to be in seismic with acceleration coeff.
2. The Design of the Abutment accords with Specification for bridge design 22-TCN-272-05 and AASHTO LRFD 2004 for reference
3. Design live load: HL-93 and lane loading 9.3 KN/m

Input :

Level Table(at center of abutment)

Level of top of headwall	HTwL	**18.722**	m
Level of top of bearing	BTL	**16.736**	m
Level of top of stem abutment	HTL	**16.502**	m
Level of top of footing	FTL	**11.000**	m
Level of bottom of footing	FBL	**9.000**	m
Ground level	GL	**11.500**	m
Lowest water level	HWL	**8.000**	m
Skew angle	α	**0.00**	deg

Material Unit Weights

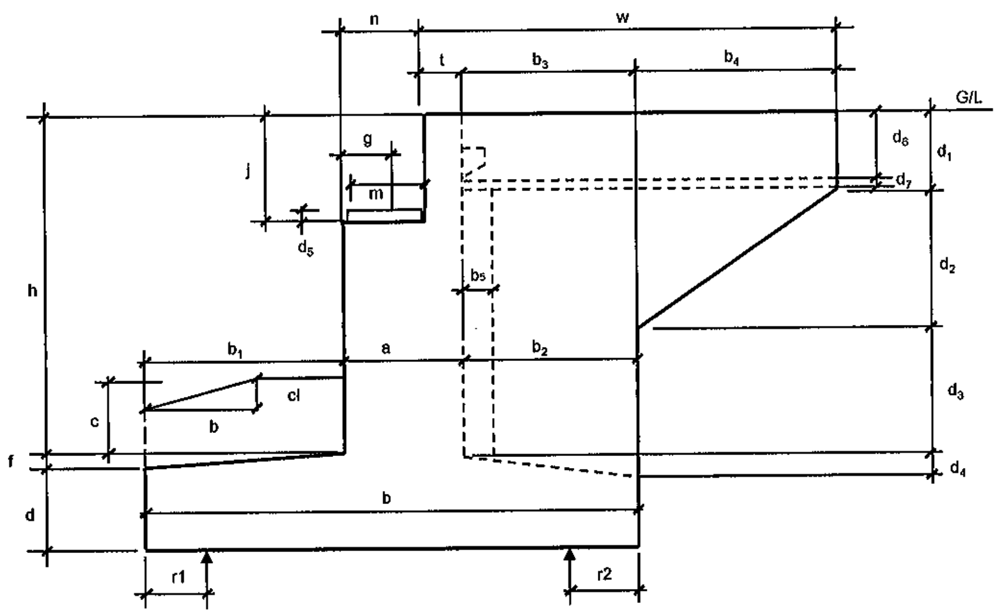
- Unit Weight of Reinf. concrete γ_c = 24.5 kN/m^3
- Unit Weight of Soil γ_s = 18.0 kN/m^3
- Unit Bouyancy Weight of Soil γ_{sbo} = 8.2 kN/m^3
- Unit weight of asphalt concrete γ_a = 22.1 kN/m^3

I.Loads from substructure

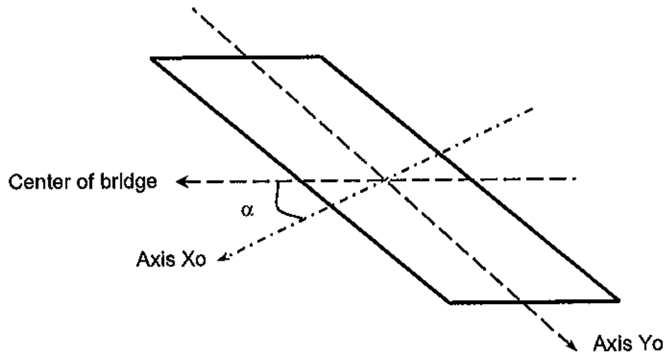
Abutment dimensions

VERTICAL VIEW

Bearing Type:	MOVE

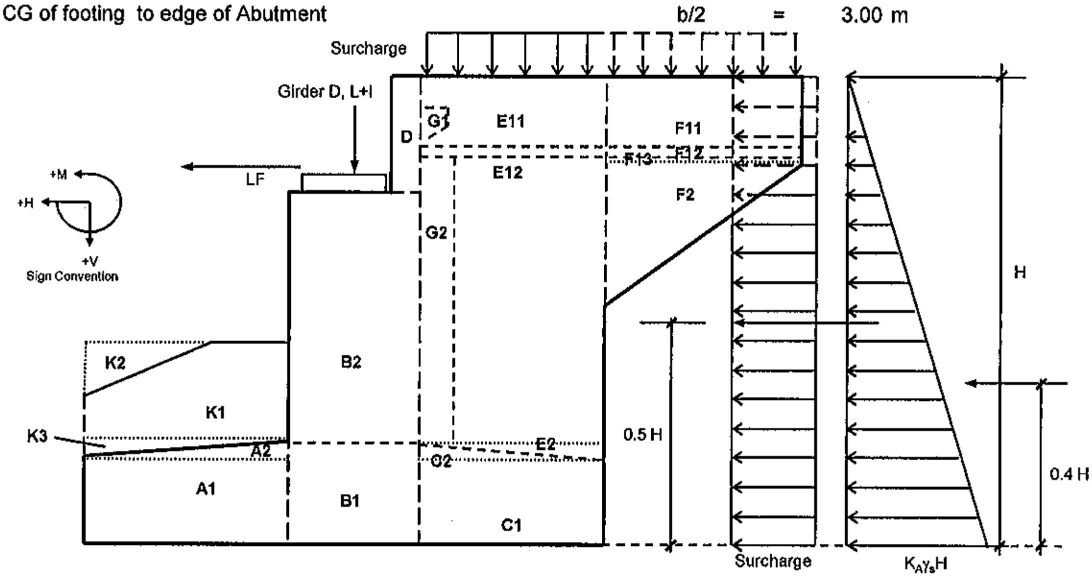


PLAN VIEW



ABUTMENT DIMENSIONS (IN METRES)					
Item	Symbol	Value	Item	Symbol	Value
Height of stem	h	7.722	Horizontal Dimension	b_4	4.000
Footing Width	b	6.000	Horizontal Dimension	b_5	0.300
Stem Width	a	1.500	Vertical Dimension	d_1	0.930
Footing Depth	d	2.000	Vertical Dimension	d_2	4.000
Footing Slope	f	0.000	Vertical Dimension	d_3	2.792
Width of stem at bearing	n	1.000	Vertical Dimension	d_4	
Ballast Wall Height	j	2.220	Vertical Dimension	d_5	0.234
Ballast Wall Thickness	t	0.500	Vertical Dimension	d_6	1.070
Wingwall Length	w	7.000	Vertical Dimension	d_7	
Soil Cover at Toe	c	0.500	With of bearing pad	m	0.800
Girder Reaction	g	0.550	Wingwall Thickness	u1	0.500
Horizontal Dimension	b_1	2.000	Wingwall Thickness	u2	0.500
Horizontal Dimension	b_2	2.500	Distance to cl of pile	r1	1.000
Horizontal Dimension	b_3	2.500	Distance to cl of pile	r2	1.000

	cos (α)	=	1.00
Slope front of abutment	cl	=	0.00 m
	bl	=	0.00 m
Width of Abutment	L		12.600 m
Width of abutment (inclined direction)	Ltr		12.600 m
Height of Abutment	Ht	=	9.72 m
Distance from CG of footing to edge of Abutment	b/2	=	3.00 m



1. Self weight of Abutment (DC)

Description	Area (m^2)	Length (m)	Force (kN)	X[(1)] (m)	Arm[(2)] (m)	Moment (kN•m)
SW of Abutment (DC)						
Section A1	4.000	12.600	1235	1.000	2.000	2470
Section A2	-	12.600	-	1.333	1.667	-
Section B1	3.000	12.600	926	2.750	0.250	232
Section B2	8.253	12.600	2548	2.750	0.250	637
Section C1	5.000	12.600	1544	4.750	-1.750	-2701
Section C2	-	12.600	-	4.333	-1.333	-
Section D	1.110	12.600	343	3.250	-0.250	-86
Section E11	2.325	0.500	28	4.750	-1.750	-50
Section E12	16.980	0.500	208	4.750	-1.750	-364
Part extra stem	-	-	-	5.417	-2.417	-
Section F11	4.280	0.500	52	8.000	-5.000	-262
Section F12	-	0.500	-	6.750	-3.750	-
Section F13	-0.560	0.500	-7	8.000	-5.000	34
Section F2	8.000	0.500	98	7.333	-4.333	-425
Section G1	0.135	11.600	38	3.650	-0.650	-25
Section G2	0.045	7.722	9	3.650	-0.650	-6
Bearing seats (w1seat= 0.65m)	0.187	3.250	15	2.550	0.450	7
Curbs +Handrail on Abutment	0.50	7.000	86	6.500	-3.500	-300
Total SW of Abutment (DC)			***7122***			***-839***
Tranverser moment			*144*		*6.175*	*887*

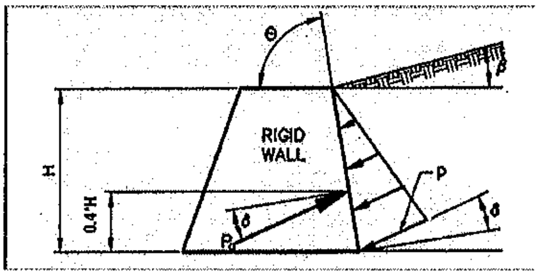
Notes: 1. Distance 'X' is measured horizontally from Toe of Retaining to CG of Section
2. Moment 'Arm' is measured from CG horizontally and from Underside of Footing Vertically.

2. Earth on Abutment (EV)

Description	Area (m^2)	Length (m)	Force (kN)	X[1] (m)	Arm[2] (m)	Moment (kN•m)
Earth on Abutment (EV)						
Section E1	19.31	12.100	4205	4.750	-1.750	-7358
Section E2	-	11.600	-	5.167	-2.167	-
Section E3	-	1.000	-	6.000	-3.000	-
Section K1	1.000	12.600	227	1.000	2.000	-
Section K2	-	12.600	-	-	3.000	-
Section K3	-	12.600	-	0.667	2.333	-
Total Earth on Footing			*4431*			*-7358*

3. Horizontal Earth Pressure on Abutment (EH)

To be safe, horizontal earth pressure at front face of abutment may be neglected.
Horizontal earth pressure at behind face of abutment shall be considered.



$$Ka = \frac{\sin^2(\theta + \phi'_f)}{\Gamma.[\sin^2\theta.\sin(\theta - \delta)]}$$

$$\Gamma = \left[1 + \sqrt{\frac{\sin(\phi'_f+\delta).\sin(\phi'_f-\beta)}{\sin(\theta-\delta).\sin(\theta+\beta)}}\right]^{-2}$$

• Height for horizontal earth pressure	H	=	9.72 m
• Width for horizontal earth pressure	W	=	12.6 m
• Density of Soil	γ_s	=	1835 kg/m^3
• Internal Friction Angle of Soil	ϕ'_f	=	30.0 deg
• Incline angle of back face wall	θ	=	90.0 deg
• Friction angle between fill and wall	δ	=	0.0 deg
• Incline angle of fill soil	β	=	0.0 deg
• Gravitational acceleration	g	=	9.81 m/s^2

• Basic earth pressure

p = K.γs. g. Z .10^-9 (Mpa, Z:mm)

K: taken as Ka (assume wall move or deflect sufficiently to reach minimum active conditions)

	Γ	=	2.250
	Ka	=	0.333
	p	=	0.058 Mpa
Horizontal earth pressure:			
• Ea = 0,5. p. Z . B . 10^3 (kN)	Ea	=	3573 kN
• M= Ea . 0,4H	M	=	13894 kNm

4. Earth Pressure on Abutment due to Surcharge (ES)

Equivalent height of soil for highway loading taken from Table 3.11.6.2.1

H=	1.50m	heq=	1.7 m	
H=	3.00m	heq=	1.2 m	
H=	6.00m	heq=	0.76 m	
H=	9.00m	heq=	0.61 m	
H=	9.72m	heq=	0.61 m	(Linear interpolation)

• Vertical force	ESv	=	346 kN
	ev	=	-1.75 m
	M	=	-605 kNm
• Horizontal force	ESh	=	448 kN
$\Delta p = k\gamma_s.g.h_{eq}x10^9$	eh	=	4.86 m
	M	=	2179 kNm

5. Earthquake effects

Bridge is located at:
According to TCXDVN 375:2006 and 22TCN272-05, bridge is in seismic zone 2 and acceleration coefficient as below
• Peak ground acceleration coefficient A = 0.0310 g

5.1. Seismic active lateral Earth pressure (E_{AE})

• Backfill slop angle i = 0.0 deg
• Slope of wall to vertical β' = 0.0 deg
• Angle of friction of soil ϕ = 30.0 deg
• Angle of friction between soil and abutment δ = 0.0 deg
• Horizontal acceleration coefficient kh = 0.047
• Vertical acceleration coefficient kv = 0.019
• Angle $\theta = \arctan(k_h/(1-k_v))$ θ = 2.7 deg

$$K_{AE} = \frac{\cos^2(\phi-\theta-\beta)}{\cos\theta.\cos^2\beta\cos(\delta+\beta+\theta)} x \left[1+\sqrt{\frac{\sin(\phi+\delta).\sin(\phi-\theta-i)}{\cos(\delta+\beta+\theta).\cos(i-\beta)}}\right]^{-2}$$

• Seismic active lateral Earth pressure coefficient K_{AE} = 0.36

• E_{AE} = 0,5. g. γ. H^2 . (1-kv) . K_{AE}. 10^-9 (kN/m)

• Seismic active lateral Earth pressure coefficient E_{AE} = 3806 kN
M_{AE}= E_{AS}*0.3H+(E_{AE} - E_{AS})*0.6H M_{AE} = 11783 KNm <A.11.1.1.1>
E_{AS} is the static component of seismic active pressure calculated with θ = kv = 0

5.2. Earthquake effects to abutment (EQ)

Seismic force for substructures: elements above ground Fh = Csm* W; elements under ground Fh = A*S*W
• Soil profile type Soil type I
• Site Coefficients. S = 1.0
• Elastic Seismic Response Coefficient 2.5A = 0.078
Csm = 1.2*A*S/ Tm^2/3 ≤ 2.5*A Csm = 0.039
• Period of vibration of the fundamental mode
Tm = 2*pi()*sqrt(m/k) Tm = 0.937 s

Decription	Area (m^2)	Length (m)	Force (kN)	X[1] (m)	Arm[2] (m)	Moment (kN•m)
Section A1	4.000	12.600	48	-	1.000	48
Section A2	-	12.600	-	-	2.000	-
Section B1	3.000	12.600	36	-	1.000	36
Section B2	8.253	12.600	99	-	4.751	470
Section C1	5.000	12.600	60	-	1.000	60
Section C2	-	12.600	-	-	2.000	-
Section D	1.110	12.600	13	-	8.612	115
Section E11	2.325	0.500	1	-	7.187	8
Section E12	16.980	0.500	8	-	3.326	
Section E2	-	-	-	-	2.000	-
Section F11	4.280	0.500	2	-	7.187	15
Section F12	-	0.500	-	-	6.652	
Section F13	-0.560	0.500	-0	-	7.792	
Section F2	8.000	0.500	4	-	7.459	28
Section G1	0.135	11.600	1	-	7.009	10
Section G2	0.045	7.722	0	-	3.326	1
Total EQ of Abutment Selfweight			***273***			***791***

6. Braking Force(BR)

Take 50 % Braking Force for this Abutment (Free Bearing)

• Number of lanes	n	=	3 lanes	
• Multiple presence factor	m	=	0.85	
• Take 25 % of Truck load				
BR = 25% * n * m * (2*145+35)	BR	=	104 kN	Long. Axis
• Acting at 1.8m higher of road face	e	=	11.5 m	
	Mlong	=	1194 KNm	Long. Axis

7. Centrifucal Force , CE (3.6.3)

• Plan of bridge (1:"straight",2: "Curve")			1	
• Design Speed	V	=	120 km/h	
	V	=	33.3 m/s	
	R		- m	
$C = 4/3*(V^2/gR)$	C		-	
Acting at 1.8m higher of road face				
CE = n * m * (2*145+35) * C	CE	=	0.00 KN	
	e	=	9.52 m	
	Mtrans	=	0.00 KNm	Trans. Axis

8. Water Load (WA) *:NA*

8.1. Buoyancy of Abutment

• Heightest water Level +8.00

Description	Area (m^2)	Length (m)	Force (kN)	$X^{(1)}$ (m)	$Arm^{(2)}$ (m)	Moment (kN•m)
Bouyancy on abutment						
Section A1	-	12.600	-	1.000	2.000	-
Section A2	-	12.600	-	-	3.000	-
Section B(B1,B2)	-	12.600	-	2.750	0.250	-
Section C1	-	12.600	-	4.750	-1.750	-
Section C2	-	12.600	-	-	3.000	-
Section E2	-	1.000	-	-	3.000	-
Section E1	-	1.000	-	4.750	-1.750	-
Section F2	-	1.000	-	4.069	-1.069	-
Total Bouyancy			-			-

8.2 Buoyancy of Earth on Abutment

Description	Area (m^2)	Length (m)	Force (kN)	$X^{(1)}$ (m)	$Arm^{(2)}$ (m)	Moment (kN•m)
Bouyancy of earth on abutement						
Section E2	-	11.600	-	-	3.000	-
Section E1	-	11.600	-	4.750	-1.750	-
Section K2	-	12.600	-	-	3.000	-
Section K1	-	12.600	-	1.000	2.000	-
- Section K3	-	12.600	-	-	3.000	-
Total Bouyancy			-			-

SUPERSTRUCTURE LOADS

II.Loads from superstructure

Item	Sign	Value	Unit
Span length	Lst	30.00	m
Span between bearings	Ls	29.10	m
Bridge Width	W	12.60	m
Number of girders	n_g	5.00	Girders
Girder height	Hg	1.60	m
Deck slab depth	Hd	0.242	m
Asphalt depth	$H\alpha\sigma$	0.084	m

1.Dead loads (DC): One span at abutment

Item	Sign	Value	Unit
1.1.Girders			
Sum of girders weight	DC	3278.10	kN
Precast Planks	DC	430.42	kN
Diaphragm	DC	380.73	kN
Total	DC	4089.25	kN
1.2.Deck slab			
Deck slab	DC	2194.92	kN
1.3.Pavement			
Asphalt concrete	DW	590.34	kN
1.4.Parapet			
Parapet + median	DC	808.50	kN

2.Live load (LL):
2.1. Live load

Truck	145 — 4.3 m — 145 — 4.3 m — 35 kN		
Tandem	110 — 1.2 m — 110 kN		
Lane load	w_L 9.3 kN/m		
Pedestrial	Wpd= 0.0 kN/m2		
Considerate structure as a simple span			
Reaction Influence	1 — 29.10 m		
Number of lanes	n	3	
Multiple presence factor	m	0.85	
Dynamic load allowance	1+IM	1.25	

*Reaction = [(1+IM)*Vehicle + Laneload]*n*m*

Truck	P1(kN)	P2(kN)	P3(kN)	Sum(kN)	Total(kN)
Axle load	145	145	35		
Influence value	1.000	0.852	0.704		
Reaction	145.0	123.6	24.7	293.2	934.7
Tandem	P1(kN)	P2(kN)		Sum(kN)	Total(kN)
Axle load	110	110			
Influence value	1.000	0.959			
Reaction	110	105.5		215.5	686.8
Lane load	Wl(kN/m)				Total(kN)
Value	9.3				
Influence value	14.55				
Reaction	135.3				345.1
Pedestrian	Wdb(kN)				Total(kN)
Reaction	0.0				0.0

3.Earthquake effects on superstructrue (EQ)

Force from superstructure due to EQ — EQ = 149 kN

4.Uniform Temperature, Srinkage& Creep(TU+SH&CR)

Bearing displacement due to uniform temperature and shrinkage creep	Δu	=	0.026 m
Shear modulus G	G	=	1 MPa
Bearing area	A	=	0.175 m2
Height of elastomeric layers	hrt		0.084 m
Number of bearing	nb	=	5 bears
Horizontal force due to TU+SH&CR			
Acting at top of bearing $H = G.A.\Delta u/h_{rt}$	Hx	=	271 kN

5.Wind loads (Ws)

5.1.Tranverse wind on superstructure (WS)

Wind zone	Zone		II
Basic 3 second gust wind	Vb	=	45.00 m/s
Correction factor	S	=	1.09
Design wind velocity	V	=	49.05 m/s
Drag coefficient	Cd	=	1.10
Overall width of bridge	b	=	12.60 m
Depth of superstructure(including solid parapet)	d	=	2.91 m
	b/d	=	4.33
Windy obstructed area of superstructure	At	=	87.36 m2

Transverse wind load

$P_D = \max(0.0006V^2.C_d.A_t, 1.8A_t) =$ — Hy = 157.2 kN

Longitudinal wind load

$F_{WSL} = 0.25P_D =$ — Hx = 39.3 kN

5.2.Wind load on vehicles (WL)

Transverse wind load on vehicle	Hy	=	22.50 kN
Longitudinal wind load on vehicles	Hx	=	22.50 kN
(At 1.8m from surface)			

6.Combinations

Loads from superstructure to Abutment

Loads at bottom of stem		Vertical		Longitudinal			Tranversal		
Loads	Sign	N (kN)	x (m)	Hx (kN)	z_1 (m)	My (kN•m)	Hy (kN)	y (m)	Mx (kN•m)
Girder+Deck+Parapet	DC	3546	0.20			709			
Pavement	DW	295	0.20			59			
LiveLoad	LL	1280	0.20			256		0.48	608
Pedestrian	PD							-	-
Trans. wind on Struc.	WS			20	5.74		79	5.74	451
Trans. wind on vehi.	WL			11	11.74		23	11.74	264
Eearth quake	EQ			75	5.74		45	5.74	257
TU+SH&CR	TU+SH&CR			135	5.74	777			

	Da Nang Quang Ngai Expressway project ORB12 BRIDGE DETAIL DESIGN ABUTMENT A2	Item.	Eng.	Date.	Sign.
		Design			
		Check			
		Revise			

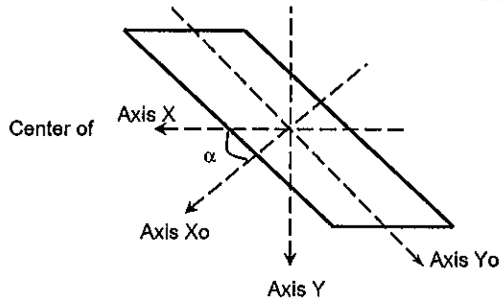
Loads at bottom of pilecap		Vertical		Longitudinal			Tranversal		
Loads	Sign	N (kN)	x (m)	Hx (kN)	z_1 (m)	My (kN•m)	Hy (kN)	y (m)	Mx (kN•m)
Girder+Deck+Parapet	DC	3546	0.45			1596			
Pavement	DW	295	0.45			133			
LiveLoad	LL	1280	0.45			576		0.48	608
Pedestrian	PD			-	-	-		-	-
Trans. wind on Struc.	WS			20	7.74	152	79	7.74	608
Trans. wind on vehi.	WL			11	13.74	155	23	13.74	309
Eearth quake	EQ			75	7.74	577	45	7.74	346
TU+SH&CR	TU+SH&CR			135	7.74	1048			

Loads		Load combinations						
	Sign	Str-IA	Str-IB	Str-IIIA	Str-IIIB	Ser-I	Ext-IA	Ext-IB
Girder+Deck+Parapet	DC	1.25	0.90	1.25	0.90	1.00	1.25	0.90
Pavement	DW	1.50	0.65	1.50	0.65	1.00	1.50	0.65
LiveLoad	LL	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Pedestrian	PD	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Trans. wind on Struc.	WS			0.40	0.40	0.30		
Trans. wind on vehi.	WL			1.00	1.00	1.00		
Eearth quake	EQ						1.00	1.00
TU+SH&CR	TU+SH&CR	0.50	0.50	0.50	0.50	1.00		

Load combinations at bottom of stem		Longitudinal		Tranversal	
Load combinations	N (kN)	Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	7115	68	1811	0	1064
Strength Str-IB	5623	68	1513	0	1064
Strength Str-IIIA	6603	87	1709	54	1265
Strength Str-IIIB	5111	87	1411	54	1265
Service Ser-I	5121	153	1801	46	1007
Extreme Ext-IA	5516	75	1103	45	561
Extreme Ext-IB	4023	75	805	45	561

Load combinations at bottom of pilecap		Longitudinal		Tranversal	
Load combinations	N (kN)	Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	7115	68	3726	0	1064
Strength Str-IB	5623	68	3054	0	1064
Strength Str-IIIA	6603	87	3711	54	1373
Strength Str-IIIB	5111	87	3039	54	1373
Service Ser-I	5121	153	3552	46	1100
Extreme Ext-IA	5516	75	3059	45	650
Extreme Ext-IB	4023	75	2388	45	650

LOAD COMBINATIONS



III.Load Combinations

1.Loads from substructure

Loads at bottom of pilecap	Sign	Vertical	Longitudinal		Tranversal	
		N (kN)	Hx (kN)	My (kN•m)	Hy (kN)	Mx (kN•m)
Self weight of Abutment	DC	7122		-839		886.5448
Soils on pilecap	EV	4431		-7358		
Horizontal Earth Pressure	EH		3573	13894		
Vertical Surcharge	LSv	346		-605		
Horizontal Surcharge	LSh		448	2179		
Braking Force	BR		104	1194		
Centrifucal Force	CE		-	-	-	-
Buoyancy of Abutment	WA	-		-		
Buoyancy of Earth on Abutment	WA	-		-		
Earthquake effects to Abutment	EQ		273	791	82	237
Earthquake effects to soil	E_{AE}		3806	11783		

Loads	Sign	Load factors						
		Str-IA	Str-IB	Str-IIIA	Str-IIIB	Ser-I	Ext-IA	Ext-IB
Self weight of Abutment	DC	1.25	0.90	1.25	0.90	1.00	1.25	0.90
Soils on pilecap	EV	1.35	0.90	1.35	0.90	1.00	1.35	0.90
Horizontal Earth Pressure	EH	1.50	0.90	1.50	0.90	1.00	0.00	0.00
Vertical Surcharge	LSv	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Horizontal Surcharge	LSh	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Braking Force	BR	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Centrifucal Force	CE	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Buoyancy of Abutment	WA	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Buoyancy of Earth on Abutment	WA	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Earthquake effects to Abutment	EQ						1.00	1.00
Earthquake effects to soil	E_{AE}						1.00	1.00

Load Combination at bottom of pilecap					
Load combinations	N (kN)	Longitudinal Hx (kN)	Longitudinal My (kN.m)	Tranversal Hy (kN)	Tranversal Mx (kN.m)
Strength Str-IA	15491	6325	14702	0	1108
Strength Str-IB	11004	4181	9970	0	798
Strength Str-IIIA	15352	6104	13595	0	1108
Strength Str-IIIB	10865	3961	8863	0	798
Service Ser-I	11900	4125	8464	0	887
Extreme Ext-IA	15058	4355	2976	82	1346
Extreme Ext-IB	10571	4355	6581	82	1035

2. Loads from superstructure

Load Combination at bottom of pilecap					
		Longitudinal		Tranversal	
Load combinations	N (kN)	Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	7115	68	3726	0	1064
Strength Str-IB	5623	68	3054	0	1064
Strength Str-IIIA	6603	87	3711	54	1373
Strength Str-IIIB	5111	87	3039	54	1373
Service Ser-I	5121	153	3552	46	1100
Extreme Ext-IA	5516	75	3059	45	650
Extreme Ext-IB	4023	75	2388	45	650

3. Total loads at bottom of pilecap

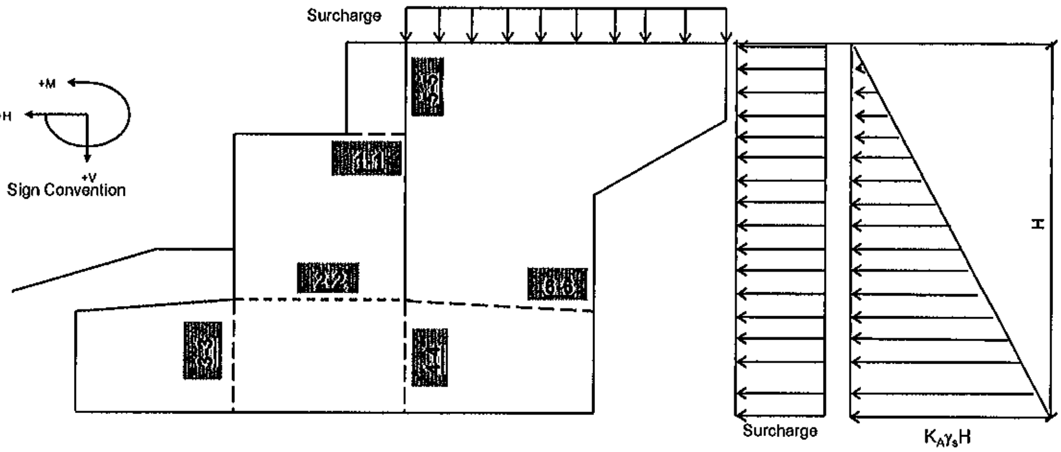
Load Combination at bottom of pilecap					
		Longitudinal		Tranversal	
Load combinations	N (kN)	Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	22606	6393	18428	0	2172
Strength Str-IB	16627	4249	13025	0	1862
Strength Str-IIIA	21956	6191	17306	54	2481
Strength Str-IIIB	15977	4047	11903	54	2171
Service Ser-I	17021	4277	12017	46	1986
Extreme Ext-IA	20574	4430	6035	127	1996
Extreme Ext-IB	14595	4430	8969	127	1686

ELEMENTS CHECKING

IV.Elements checking

The abutment walls shall be checked at sections 1-1, 2-2, 3-3, 4-4, 5-5

1. Calculate internal force of sections



1.1 Section 1-1

Table of Load Factors

Loads	Sign	Service I Ser-I	Strength I Str-IA	 Str-IB	Extreme I Ext-I
Selfweight	DC	1.00	1.25	0.90	1.25
Horizontal Earth Pressure	EH	1.00	1.50	0.90	
Surchage (Horizontal)	LS_h	1.00	1.75	1.75	0.50
Horizontal Seismic Earth Pressure	E_{AE}				1.50
Abutment earthquake force	EQ				1.00

Unfactored Loads	N (kN)	Longitudinal Hx (kN)	 My (kN.m)	Tranversal Hy (kN)	 Mx (kN.m)
Selfweight	381		-15		
Horizontal Earth Pressure		186	165		
Surcharge (horizontal)		245	272		
Horizontal Seismic Earth Pressure		198	140		
Abutment earthquake force		15	16	4	5

Load Combination at bottom of headwall

Load combinations	N (kN)	Longitudinal Hx (kN)	 My (kN.m)	Tranversal Hy (kN)	 Mx (kN.m)
Service Ser-I	381	431	422	0	0
Strength Str-IA	476	708	705	0	0
Strength Str-IB	343	596	611	0	0
Extreme Ext-I	476	435	344	4	5

1.2 Section 2-2

Table of Load Factors

Loads	Sign	Service I Ser-I	Strength I Str-IA	 Str-IB	Extreme I Ext-I
Selfweight	DC	1.00	1.25	0.90	1.25
Superstructure Dead Load	DC	1.00	1.25	0.90	1.25
Pavement	DW	1.00	1.50	0.65	1.50
Live Load	LL	1.00	1.75	1.75	0.50
Horizontal Earth Pressure	EH	1.00	1.50	0.90	
Surchage (Horizontal)	LS_h	1.00	1.75	1.75	0.50
TU+SH&CR	TU+SH&CR	1.00	0.50	0.50	
Horizontal Seismic Earth Pressure	E_{AE}				1.50
Abutment earthquake force	EQ				1.00

Unfactored Loads	N (kN)	Longitudinal Hx (kN)	Longitudinal My (kN.m)	Tranversal Hy (kN)	Tranversal Mx (kN.m)
Selfweight	2929		-175		
Superstructure Dead Load	3546		709		
Pavement	295		59		
Live Load	1280		256		608
Horizontal Earth Pressure		2254	6962		
Surchage (Horizontal)		393	1519		
TU+SH&CR		135	777		
Horizontal Seismic Earth Pressure		2401	5904		
Abutment earthquake force		114	371	79	368

Load Combination at bottom of stem wall

Load combinations	N (kN)	Longitudinal Hx (kN)	Longitudinal My (kN.m)	Tranversal Hy (kN)	Tranversal Mx (kN.m)
Service Ser-I	8050	2783	10107	0	608
Strength Str-IA	10776	4137	14694	0	1064
Strength Str-IB	8259	2785	10279	0	1064
Extreme Ext-I	9176	3913	10871	79	672

1.3 Section 3-3

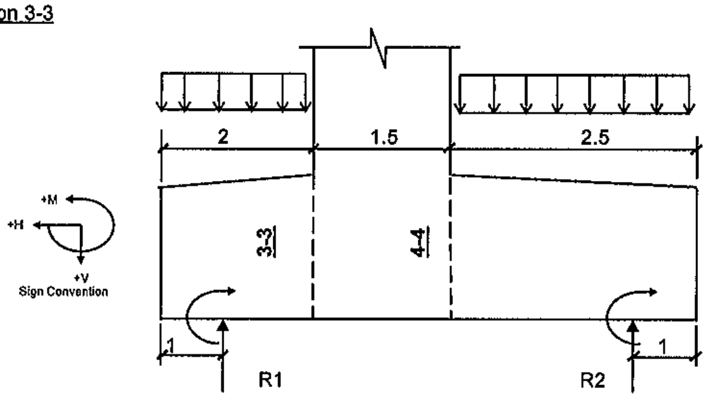


Table of Load Factors

Loads	Sign	Service I Ser-I	Strength I Str-IA	Strength I Str-IB	Extreme I Ext-I
Selfweight at front side	DC	1.00	1.25	0.90	1.25
Vertical soil on foot at front side	DC	1.00	1.35	0.90	1.35
Reaction of piles	RE	1.00	1.00	1.00	1.00

Unfactored Loads		Shear (kN)	Longitudinal Hx (kN)	Longitudinal My (kN.m)		
Selfweight at front side		-1235		-1235		
Vertical soil on foot at front side		-227		-227		
Reaction of piles	Ser-I	13891		21294		
	Str-IA	19419		30201		
	Str-IB	13903		21234		
	Ext-I	14435		22428		

Load Combination at section 3-3

Load combinations	N (kN)	Longitudinal Hx (kN)	Longitudinal My (kN.m)		
Service Ser-I	12430		19833		
Strength Str-IA	17570		28351		
Strength Str-IB	12587		19919		
Extreme Ext-I	12586		20578		

1.4 Section 4-4

Table of Load Factors

Loads	Sign	Service I	Strength I		Extreme I
		Ser-I	Str-IA	Str-IB	Ext-I
Selfweight at behind side	DC	1.00	1.25	0.90	1.25
Vertical soil on foot at behind side	DC	1.00	1.35	0.90	1.35
Surcharge(Vertical)	EV	1.00	1.75	1.75	0.50
Reaction of piles	RE	1.00	1.00	1.00	1.00

Unfactored Loads		N	Longitudinal		Tranversal	
			Hx	My	Hy	Mx
		(kN)	(kN)	(kN.m)	(kN)	(kN.m)
Selfweight of behind side		-1924		-2806		
Vertical soil on foot at behind side		-4205		-5256		
Surcharge(Vertical)		-346		-432		
Reaction of piles	Ser-I	3130		1084		
	Str-IA	3186		-553		
	Str-IB	2724		540		
	Ext-I	6138		5206		

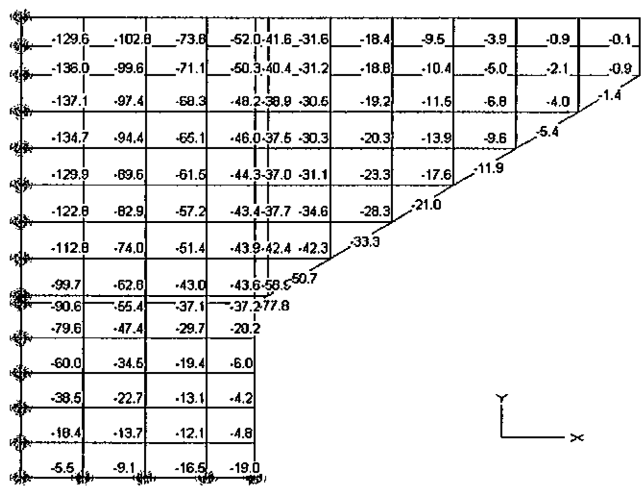
Load Combination at section 4-4					
Load combinations	N	Longitudinal		Tranversal	
		Hx	My	Hy	Mx
	(kN)	(kN)	(kN.m)	(kN)	(kN.m)
Service Ser-I	-3344		-7409		
Strength Str-IA	-5500		-11912		
Strength Str-IB	-3397		-7472		
Extreme Ext-I	-2115		-5612		

1.4 Section 5-5 & 6-6

Slope of triang pressure	tgβ	=	6.00	
Uniform horizontal pressure	U.p	=	3.66	kN/m2

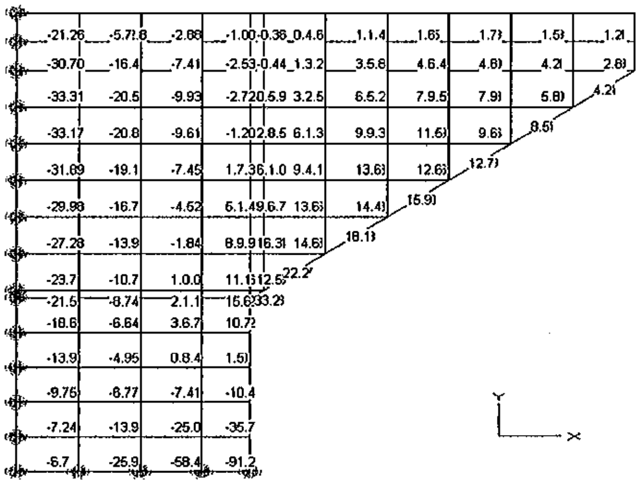
SERVICE – Element Moment X:

Combination X-Bending Moment
service IA



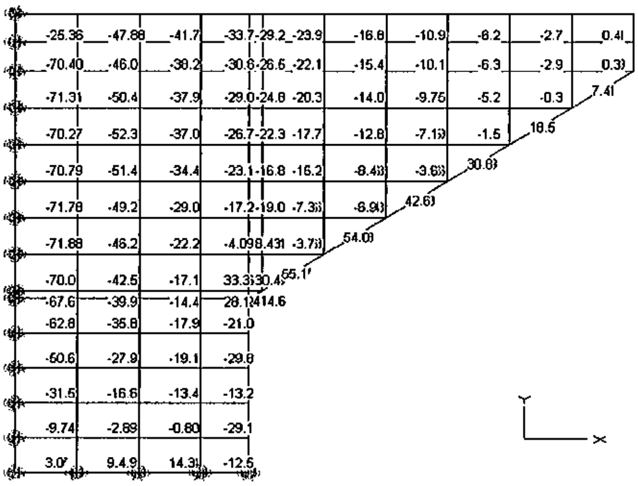
SERVICE – Element Moment Y:

Combination Y-Bending Moment
service IA



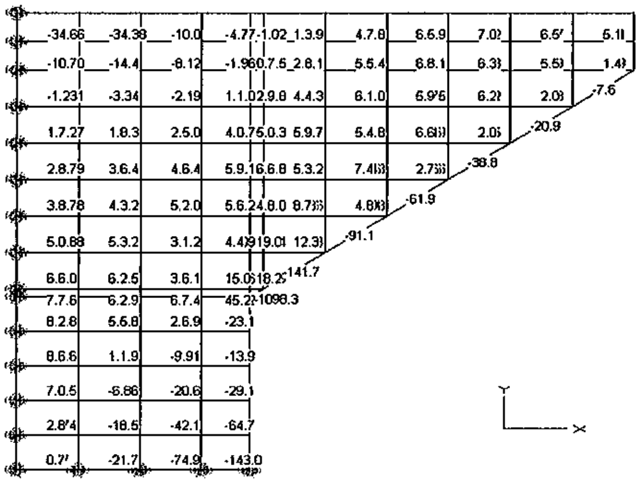
SERVICE – Element Shear FX:

Combination SHEAR FORCE X (per unit width)
service IA



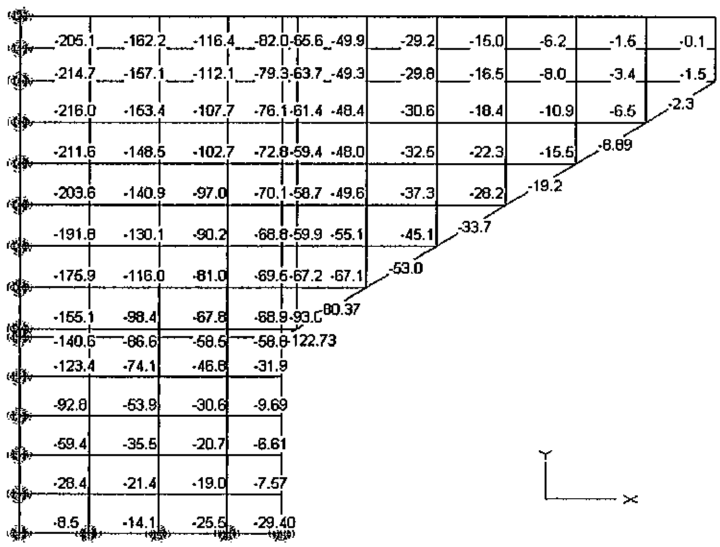
SERVICE – Element Shear FY:

Combination SHEAR FORCE Y (per unit width)
service IA



STRENGTH – Element Moment X:

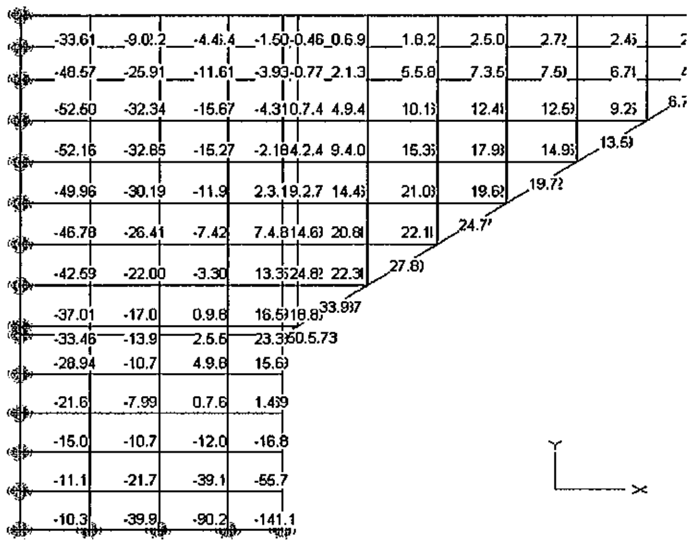
Combination X-Bending Moment (per unit width)
streng IA\



STRENGTH – Element Moment Y:

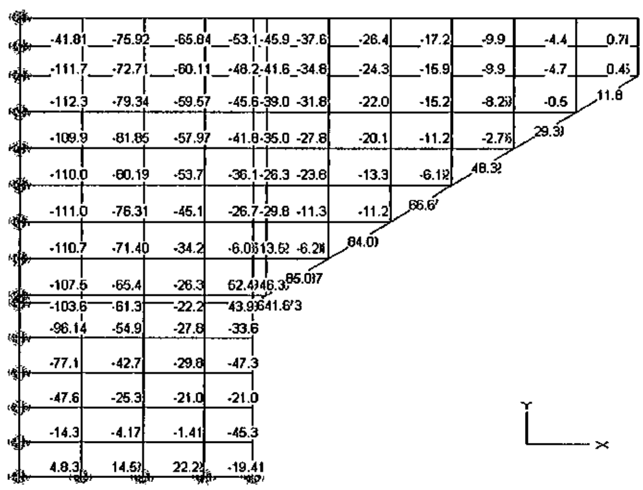
	Da Nang Quang Ngai Expressway project	Item.	Eng.	Date.	Sign.
	ORB12 BRIDGE	Design			
	DETAIL DESIGN	Check			
	ABUTMENT A2	Revise			

Combination Y-Bending Moment(per unit width)
streng IA\



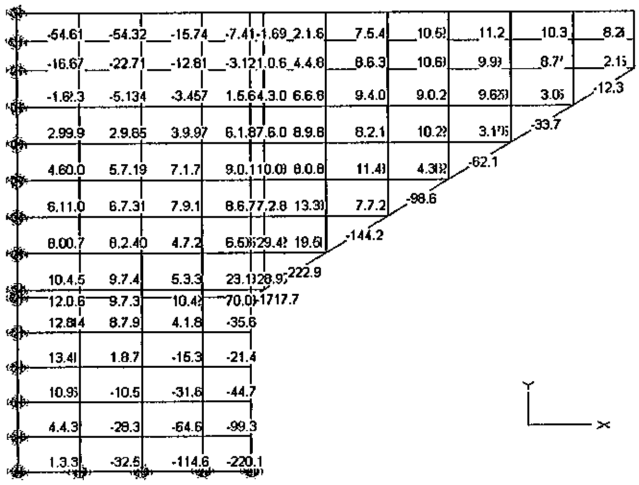
STRENGTH – Element Shear Fx:

Combination SHEAR FORCE X (per unit width)
streng IA\



STRENGTH – Element Shear Fy:

Combination SHEAR FORCE Y (per unit width)
streng IA\



Load Combination at section 5-5					
Load combinations	N (kN)	Vertical Hy (kN)	Vertical My (kN.m)	Horizontal Hx (kN)	Horizontal Mx (kN.m)
Service Ser-I		143	91		
Strength Str-IA		220	141		

Load Combination at section 6-6					
Load combinations	N (kN)	Vertical Hy (kN)	Vertical My (kN.m)	Horizontal Hx (kN)	Horizontal Mx (kN.m)
Service Ser-I				71	137
Strength Str-IA				112	216

2. Elements Checking

2.1. Bearing Resistance

<S.5.7.5>

The case of absence of confinement reinforcement in the conrete supporting the bearing device
Factored bearing resistance shall be taken

$Pr = \phi. Pn = \phi. 0.85. f'c. A1. m$

Dimension of bearing plate	w0	=	0.800 m	
	b0	=	0.650 m	
Area under bearing device	A1	=	0.520 m2	
Distributed width and length	w	=	1.000 m	
	b	=	0.850 m	
Notational area	A2	=	0.850 m2	

Where supporting surface is wider on all sides than loades area

$m = sqrt(A2/A1) \le 2.0$ case 1

where loaded area is subjected to nouniformly distributed bearing

$m = 0.75*sqrt(A2/A1) \le 1.5$ case 2

Modification factor case 1	m	=	1.279	
Resistance factor	ϕ	=	0.700	<S.5.5.4.2>
Factored bearing resistance	Pr	=	11867 kN	> Pu
Bearing reaction of approach bridge	Pu	=	5291 kN	Ok

$Pu = 1.25*DC + 1.5*DW + 1.75*LL$

In case factored applied load exceeds the factored resistance,
provision shall be made to resist the bursting and spalling force in article 5.10.9

Factored bearing resistance shall be taken <S.5.10.9.7.2>

$Pr = \phi. fn. Ab$

fn take the lesser of

$fn = 0.7. f'ci. sqrt(A/Ag)$ and

$fn = 2.25. f'ci$

	fn	=	26.85 MPa	
Maximum area of the portion of supporting surface	A	=	0.850 m2	
Gross area of bearing plate	Ag	=	0.520 m2	
Effective net area of bearing plate, Ag minus stud of bearing	Ab	=	0.520 m2	
Nominal concrete strength at time of application	f'ci	=	30 MPa	
Factored bearing resistance	Pr	=	9773 kN	Ok

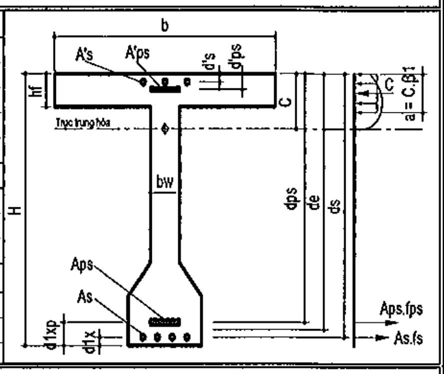
	Da Nang Quang Ngai Expressway project ORB12 BRIDGE	Item.	Eng.	Date.	Sign.
		Design			
	DETAIL DESIGN	Check			
	ABUTMENT A2	Revise			

22TCN272-05; AASHTO LRFD 2nd - 1998

REINFORCEMENT CHECKING - HEAD AND STEM WALL

MATERIALS

	NORMAL CONCRETE		
f'c	Compressive Strength of concrete at 28 days	Mpa	30
Ec	Modulus of Elasticity	Mpa	27691
fr	Modulus of Rupture	Mpa	3.5
gc	Unit weight of concrete	kN/m3	24.5
	PRESTRESSING STEEL		
fpu	Tensile strength of prestressing steel	Mpa	1860
fpy	Yield strength of prestressing steel	Mpa	1670
Ep	Modulus of Elasticity	Mpa	195000
	REINFORCEMENT		
fy	Yield strength	Mpa	400
Es	Modulus of Elasticity	Mpa	200000
nc	Ratio Es/Ec		7



Sign	Parameters		Unit	Sections				
				1-1	1-1	2-2	2-2	2-2
	INTERNAL FORCES AT SECTIOIN							
	Combination			Strength	Service	Service	Strength	Extreme
Qu	Shear		kN	708	431	2783	4137	3913
Mu	Flexural Moment		kNm	705	422	10107	14694	10871
Nu	Axial load		kN	476	381	8050	10776	9176
Tu	Torsional Moment		kNm	0	0	0	0	0
	FLEXURAL MOMENT CHECKING							
H	Section height		m	0.500	0.500	1.500	1.500	1.500
d's	Dis. From comp. fiber to centroid of comp. Reinf		m	0.058	0.058	0.058	0.058	0.058
d1x	Dis. From tens. fiber to centroid of tension Reinf		m	0.058	0.058	0.063	0.063	0.063
	Cover to reinf		m	0.050	0.050	0.050	0.050	0.050
ds	Dis. From comp. fiber to centroid of tension Reinf		m	0.442	0.442	1.438	1.438	1.438
d'ps	Dis. From comp. fiber to centroid of comp. prestressing steel		m	0.000	0.000	0.000	0.000	0.000
d1xp	Dis. From tens. fiber to centroid of tension prestressing steel		m	0.000	0.000	0.000	0.000	0.000
dps	Dis. From comp. fiber to centroid of tension prestressing steel		m	0.500	0.500	1.500	1.500	1.500
b	Width of the compression face of member		m	12.600	12.600	12.600	12.600	12.600
bw	Web width or diameter of a circular section		m	12.600	12.600	12.600	12.600	12.600
hf	Compression flange depth		m	0.000	0.000	0.000	0.000	0.000
Iz	Moment of inertia of section		m4	0.131	0.131	3.544	3.544	3.544
Amc	Section area		m2	6.300	6.300	18.900	18.900	18.900
	Steel choice							
Aps	Tension prestressing steel	P.S type		0	0	0	0	0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
A'ps	Compression prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	0 T00.0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
As	Tension Reinforcement	Number	bars	82	82	82	82	82
		Diameter	mm	16	16	25	25	25
		Area	m2	0.01656	0.01656	0.04026	0.04026	0.04026
A's	Compression Reinforcement	Number	bars	82	82	82	82	82
		Diameter	mm	16	16	16	16	16
		Area	m2	0.01656	0.01656	0.01656	0.01656	0.01656
A'c	Shear reinforcement	Number	bars	20	20	19	19	19
		Diameter	mm	14	14	14	14	14
		Area	m2	0.00302	0.00302	0.00287	0.00287	0.00287
ϕ	Resistance factors for flexure		5.5.4.2	0.90	1.00	1.00	0.90	1.00
ϕv	Resistance factors for shear			0.90	1.00	1.00	0.90	1.00
ϕn	Resistance factors for axial force			1.00	1.00	1.00	1.00	1.00
β1	Stress block factor			0.836	0.836	0.836	0.836	0.836
c	Dis. Between centroid and top fiber		m	0.000	0.000	0.035	0.035	0.035
	For T section behavior		m	0.000	0.000	0.035	0.035	0.035
	For rectangular section behavior		m	0.000	0.000	0.035	0.035	0.035
fpe	Effective stress in the prestressing steel after losses		Mpa	1116	1116	1116	1116	1116
fps	Aver. stress in pres. steel at the time for which the nominal resistance		Mpa	1860	1860	1848	1848	1848
k	Factor depends on type of P.S, Low relaxation strand k = 0.28			0.28	0.28	0.28	0.28	0.28

REINFORCEMENT CHECKING - HEAD AND STEM WALL

a	Depth of equivalent stress block	m	0.000	0.000	0.030	0.030	0.030
de	Corresp. effective depth from extreme comp. fiber						
	to centroid of tensile force in the tensile reinf.	m	0.442	0.442	1.438	1.438	1.438
Mn	Nominal resistance	kNm	2544	2544	22627	22627	22627
Mr	Factored reistance	kNm	**2290**	**2544**	**22627**	**20364**	**22627**
Mu	Flexual moment	kNm	705	422	10107	14694	10871
(5.7.3.2)	**Flexural moment Checking**		**OK**	**OK**	**OK**	**OK**	**OK**
	Limits for reinforcement						
c/de	Maximum reinforcement		0.00	0.00	0.02	0.02	0.02
	Maximum reinforcement Checking	<= 0.42	**OK**	**OK**	**OK**	**OK**	**OK**
1.2*Mcr	Craking moment	kNm	1087	1087	10018	10018	10018
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**		**OK**	**OK**	**OK**	**OK**	**OK**
(5.7.3.4)	**Conctrol of craking by distr. of reinf for RC member- Check?**		No	Yes	Yes	No	No
	Existing condition for structrure	1,2 or 3	1	1	1	1	1
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.058	0.058	0.063	0.063	0.063
Z	Crack width parameter	N/mm	30000	30000	30000	30000	30000
A	Area of concr. with same centroid as tens. Reinf	m2	0.018	0.018	0.019	0.019	0.019
fsa	Value	Mpa	297	297	282	282	282
0.6*fy		Mpa	240	240	240	240	240
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	240	240	240	240	240
x	Dist. From compression fiber to centroid	m	-	0.081	0.232	-	-
J.d	Arm	m	-	0.415	1.36	-	-
Icr	Moment of inertia of the cracked section	m4	-	0.017	0.465	-	-
fs	Tensile stress in reinforcement fs = Msls / (As*J.d)	Mpa	-	61	185	-	-
	Checking for control cracking fs<fsa		**N.a**	**OK**	**OK**	**N.a**	**N.a**
(5.10.8.2)	**Shrinkage and temperature Reinfocement (side distribution)**						
Areq	Area of required reinf	m2	0.00045	0.00045	0.00126	0.00126	0.00126
	Distribution on sides 7 D16	m2	0.00141	0.00141	0.00141	0.00141	
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45	0.45
	Checking		**OK**	**OK**	**OK**	**OK**	**OK**

SHEAR AND TORSION CHECKING

β	Factor indicating diag. cracked concr. to tension		2.5	3.1	2.4	2.3	2.4
θ	Angle of inclination of diagonal compressive	degree	30.48	28.70	31.30	35.35	32.66
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	90
bv	Effective web width as minimum web width - in dv	m	12.600	12.600	12.600	12.600	12.600
dv	Effective shear depth	m	0.442	0.442	1.423	1.423	1.423
	(de - a/2)	m	0.442	0.442	1.423	1.423	1.423
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	0.600
ncat	Amount of bars in spacing S	bars	20	20	19	19	19
Av	Shear reinf area in spacing S	m2	0.0030	0.0030	0.0029	0.0029	0.0029
θ	Assume	degree	30.33	28.80	33.44	37.69	34.80
v	Shear stress in concrete	kN/m2	141	53	155	256	218
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e_x	Strain in tensile reinforcement		5.92E-04	3.49E-04	6.44E-04	9.46E-04	7.29E-04
	if ex<0, multiple with reduce factor		-	-	-	-	-
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok	Ok
v/f'c	Ratio of shear stress and f'c		0.005	0.002	0.005	0.009	0.007
β	Final value		2.5	3.1	2.4	2.3	2.4
θ	Final value	degree	30.48	28.70	31.30	35.35	32.66
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	6224	7852	19798	18420	19412
Vs	Shear resistance provided by shear reinforcement	kN	1512	1626	4475	3836	4246
Vp	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0	0
Vn1	Vn1=Vc+Vs+Vp	kN	7736	9477	24273	22256	23658
Vn2	Vn2	kN	41769	41769	134450	134450	134450
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	7736	9477	24273	22256	23658
Vr	Factored shear resistance	kN	**6963**	**9477**	**24273**	**20031**	**23658**
Vu	Shear	kN	708	431	2783	4137	3913
(5.8.2.7)	**Shear checking**		**OK**	**OK**	**OK**	**OK**	**OK**
	Region requiring transverse reinf Checking		No need	No need	No need	No need	No need
	Minimum shear reinf area	m2	0.0086	0.0086	0.0086	0.0086	0.0086
	Mimimum shear reinforcement Checking		-	-	-	-	-
	0.1*f'c*bv*dv	kN	16708	16708	53780	53780	53780
	Smax	m	0.35	0.35	0.60	0.60	0.60
	Maximum spacing Smax		-	-	-	-	-

	Da Nang Quang Ngai Expressway project ORB12 BRIDGE	Item.	Eng.	Date.	Sign.
		Design			
	DETAIL DESIGN	Check			
	ABUTMENT A2	Revise			

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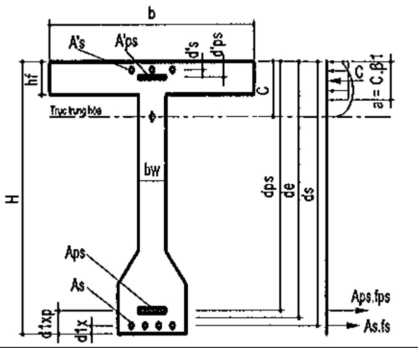
REINFORCEMENT CHECKING - PILECAP SECTION

MATERIALS



	NORMAL CONCRETE		
f'c	Compressive Strength of concrete at 28 days	Mpa	30
Ec	Modulus of Elasticity	Mpa	27691
fr	Modulus of Rupture	Mpa	3.5
gc	Unit weight of concrete	kN/m3	24.5
	PRESTRESSING STEEL		
fpu	Tensile strength of prestressing steel	Mpa	1860
fpy	Yield strength of prestressing steel	Mpa	1670
Ep	Modulus of Elasticity	Mpa	195000
	REINFORCEMENT		
fy	Yield strength	Mpa	400
Es	Modulus of Elasticity	Mpa	200000
nc	Ratio Es/Ec		7

Sign	Parameters		Unit	Sections				
				3-3	3-3	3-3	4-4	4-4
	INTERNAL FORCES AT SECTIOIN							
	Combination			Service	Strength	Extreme	Extreme	Strength
Qu	Shear		kN	12430	17570	12586	2115	5500
Mu	Flexural Moment		kNm	19833	28351	20578	5612	11912
Nu	Axial load		kN	0	0	0	0	0
Tu	Torsional Moment		kNm	0	0	0	0	0
	FLEXURAL MOMENT CHECKING							
H	Section height		m	2.000	2.000	2.000	2.000	2.000
d's	Dis. From comp. fiber to centroid of comp. Reinf		m	0.083	0.083	0.083	0.083	0.083
d1x	Dis. From tens. fiber to centroid of tension Reinf		m	0.166	0.166	0.166	0.160	0.160
	Cover to reinf		m	0.075	0.075	0.075	0.075	0.075
ds	Dis. From comp. fiber to centroid of tension Reinf		m	1.834	1.834	1.834	1.840	1.840
d'ps	Dis. From comp. fiber to centroid of comp. prestressing steel		m	0.000	0.000	0.000	0.000	0.000
d1xp	Dis. From tens. fiber to centroid of tension prestressing steel		m	0.000	0.000	0.000	0.000	0.000
dps	Dis. From comp. fiber to centroid of tension prestressing steel		m	2.000	2.000	2.000	2.000	2.000
b	Width of the compression face of member		m	12.600	12.600	12.600	12.600	12.600
bw	Web width or diameter of a circular section		m	12.600	12.600	12.600	12.600	12.600
hf	Compression flange depth		m	0.000	0.000	0.000	0.000	0.000
Iz	Moment of inertia of section		m4	8.400	8.400	8.400	8.400	8.400
Amc	Section area		m2	25.200	25.200	25.200	25.200	25.200
	Steel choice							
Aps	Tension prestressing steel	P.S type		0	0	0	0	0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
A'ps	Compression prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	0 T00.0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
As	Tension Reinforcement	Number	bars	84	84	84	84	84
		Diameter	mm	32	32	32	20	20
		Area	m2	0.06728	0.06728	0.06728	0.02638	0.02638
A's	Compression Reinforcement	Number	bars	0	0	0	0	0
		Diameter	mm	16	16	16	16	16
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
A'c	Shear reinforcement	Number	bars	15	15	13	13	13
		Diameter	mm	16	16	16	16	16
		Area	m2	0.00303	0.00303	0.00263	0.00263	0.00263
ϕ	Resistance factors for flexure		5.5.4.2	1.00	0.90	1.00	1.00	0.90
ϕv	Resistance factors for shear			1.00	0.90	1.00	1.00	0.90
ϕn	Resistance factors for axial force			1.00	1.00	1.00	1.00	1.00
$\beta 1$	Stress block factor			0.836	0.836	0.836	0.836	0.836
c	Dis. Between centroid and top fiber		m	0.100	0.100	0.100	0.039	0.039
	For T section behavior		m	0.100	0.100	0.100	0.039	0.039
	For rectangular section behavior		m	0.100	0.100	0.100	0.039	0.039
fpe	Effective stress in the prestressing steel after losses		Mpa	1116	1116	1116	1116	1116
fps	Aver. stress in pres. steel at the time for which the nominal resistance		Mpa	1833	1833	1833	1850	1850
k	Factor depends on type of P.S, Low relaxation strand k = 0.28			0.28	0.28	0.28	0.28	0.28

REINFORCEMENT CHECKING - PILECAP SECTION

a	Depth of equivalent stress block	m	0.084	0.084	0.084	0.033	0.033
de	Corresp. effective depth from extreme comp. fiber						
	to centroid of tensile force in the tensile reinf.	m	1.834	1.834	1.834	1.840	1.840
Mn	Nominal resistance	kNm	48232	48232	48232	19240	19240
Mr	Factored reistance	kNm	**48232**	**43409**	**48232**	**19240**	**17316**
Mu	Flexual moment	kNm	19833	28351	20578	5612	11912
(5.7.3.2)	**Flexural moment Checking**		**OK**	**OK**	**OK**	**OK**	**OK**
	Limits for reinforcement						
c/de	Maximum reinforcement		0.05	0.05	0.05	0.02	0.02
	Maximum reinforcement Checking	<= 0.42	**OK**	**OK**	**OK**	**OK**	**OK**
1.2*Mcr	Craking moment	kNm	18309	18309	18309	17740	17740
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**		**OK**	**OK**	**OK**	**OK**	**OK**
(5.7.3.4)	**Conctrol of craking by distr. of reinf for RC member- Check?**		Yes	No	No	No	No
	Existing condition for structrure	1,2 or 3	3	3	3	3	3
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.050	0.050	0.050	0.050	0.050
Z	Crack width parameter	N/mm	17500	17500	17500	17500	17500
A	Area of concr. with same centroid as tens. Reinf	m2	0.015	0.015	0.015	0.015	0.015
fsa	Value	Mpa	193	193	193	193	193
0.6*fy		Mpa	240	240	240	240	240
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	193	193	193	193	193
x	Dist. From compression fiber to centroid	m	0.335	-	-	-	-
J.d	Arm	m	1.722	-	-	-	-
Icr	Moment of inertia of the cracked section	m4	1.216	-	-	-	-
fs	Tensile stress in reinforcement fs = Msls / (As*J.d)	Mpa	171	-	-	-	-
	Checking for control cracking fs<fsa		**OK**	**N.a**	**N.a**	**N.a**	**N.a**
(5.10.8.2)	**Shrinkage and temperature Reinfocement (side distribution)**						
Areq	Area of required reinf	m2	0.00127	0.00127	0.00127	0.00127	0.00127
	Distribution on sides 7 D16	m2	0.00141	0.00141	0.00141	0.00141	0.00141
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45	0.45
	Checking		**OK**	**OK**	**OK**	**OK**	**OK**
	SHEAR AND TORSION CHECKING						
β	Factor indicating diag. cracked concr. to tension		2.0	1.8	2.0	2.3	1.8
θ	Angle of inclination of diagonal compressive	degree	39.88	42.57	40.10	34.27	42.24
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	90
bv	Effective web width as minimum web width - in dv	m	12.600	12.600	12.600	12.600	12.600
dv	Effective shear depth	m	1.792	1.792	1.792	1.824	1.824
	(de - a/2)	m	1.792	1.792	1.792	1.824	1.824
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	0.600
ncat	Amount of bars in spacing S	bars	15	15	13	13	13
Av	Shear reinf area in spacing S	m2	0.0030	0.0030	0.0026	0.0026	0.0026
θ	Assume	degree	39.22	42.29	40.05	36.36	42.41
v	Shear stress in concrete	kN/m2	550	865	557	33	266
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e_x	Strain in tensile reinforcement		1.39E-03	1.89E-03	1.41E-03	8.56E-04	1.81E-03
	if ex<0, multiple with reduce factor		-	-	-	-	-
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok	Ok
v/f'c	Ratio of shear stress and f'c		0.018	0.029	0.019	0.001	0.009
β	Final value		2.0	1.8	2.0	2.3	1.8
θ	Final value	degree	39.88	42.57	40.10	34.27	42.24
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	20660	18160	20537	24138	18884
Vs	Shear resistance provided by shear reinforcement	kN	4332	3941	3726	4685	3516
Vp	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0	0
Vn1	Vn1=Vc+Vs+Vp	kN	24992	22101	24263	28823	22401
Vn2	Vn2	kN	169355	169355	169355	172328	172328
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	24992	22101	24263	28823	22401
Vr	Factored shear resistance	kN	**24992**	**19891**	**24263**	**28823**	**20161**
Vu	Shear	kN	12430	17570	12586	2115	5500
(5.8.2.7)	**Shear checking**		**OK**	**OK**	**OK**	**OK**	**OK**

	Da Nang Quang Ngai Expressway project ORB12 BRIDGE DETAIL DESIGN ABUTMENT A2	Item.	Eng.	Date.	Sign.
		Design			
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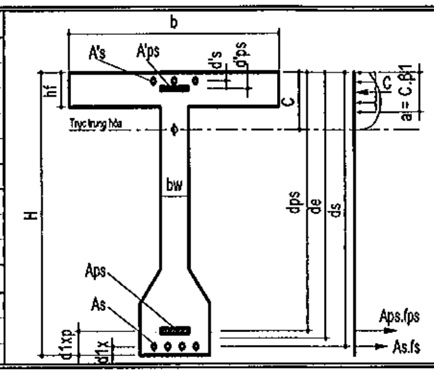
REINFORCEMENT CHECKING - WING WALL

MATERIALS



	NORMAL CONCRETE		
f'c	Compressive Strength of concrete at 28 days	Mpa	30
Ec	Modulus of Elasticity	Mpa	27691
fr	Modulus of Rupture	Mpa	3.5
gc	Unit weight of concrete	kN/m3	24.5
	PRESTRESSING STEEL		
fpu	Tensile strength of prestressing steel	Mpa	1860
fpy	Yield strength of prestressing steel	Mpa	1670
Ep	Modulus of Elasticity	Mpa	195000
	REINFORCEMENT		
fy	Yield strength	Mpa	400
Es	Modulus of Elasticity	Mpa	200000
nc	Ratio Es/Ec		7

Sign	Parameters		Unit	Sections				
				5-5	5-5	6-6	6-6	
	INTERNAL FORCES AT SECTIOIN							
	Combination			Service	Strength	Service	Strength	
Qu	Shear		kN	143	220	71	112	
Mu	Flexural Moment		kNm	91	141	137	216	
Nu	Axial load		kN	0	0	0	0	
Tu	Torsional Moment		kNm	0	0	0	0	
	FLEXURAL MOMENT CHECKING							
H	Section height		m	0.500	0.500	0.500	0.500	
d's	Dis. From comp. fiber to centroid of comp. Reinf		m	0.058	0.058	0.058	0.058	
d1x	Dis. From tens. fiber to centroid of tension Reinf		m	0.060	0.060	0.060	0.060	
	Cover to reinf		m	0.050	0.050	0.050	0.050	
ds	Dis. From comp. fiber to centroid of tension Reinf		m	0.440	0.440	0.440	0.440	
d'ps	Dis. From comp. fiber to centroid of comp. prestressing steel		m	0.000	0.000	0.000	0.000	
d1xp	Dis. From tens. fiber to centroid of tension prestressing steel		m	0.000	0.000	0.000	0.000	
dps	Dis. From comp. fiber to centroid of tension prestressing steel		m	0.500	0.500	0.500	0.500	
b	Width of the compression face of member		m	1.000	1.000	1.000	1.000	
bw	Web width or diameter of a circular section		m	1.000	1.000	1.000	1.000	
hf	Compression flange depth		m	0.000	0.000	0.000	0.000	
Iz	Moment of inertia of section		m4	0.010	0.010	0.010	0.010	
Amc	Section area		m2	0.500	0.500	0.500	0.500	
	Steel choice							
Aps	Tension prestressing steel	P.S type		0	0	0	0	
		Number	tendons	0	0	0	0	
		Area	m2	0.00000	0.00000	0.00000	0.00000	
A'ps	Compression prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	
		Number	tendons	0	0	0	0	
		Area	m2	0.00000	0.00000	0.00000	0.00000	
As	Tension Reinforcement	Number	bars	6	6	6	6	
		Diameter	mm	20	20	20	20	
		Area	m2	0.00188	0.00188	0.00188	0.00188	
A's	Compression Reinforcement	Number	bars	6	6	6	6	
		Diameter	mm	16	16	16	16	
		Area	m2	0.00121	0.00121	0.00121	0.00121	
A'c	Shear reinforcement	Number	bars	2	2	2	2	
		Diameter	mm	12	12	12	12	
		Area	m2	0.00023	0.00023	0.00023	0.00023	
ϕ	Resistance factors for flexure		5.5.4.2	1.00	0.90	1.00	0.90	
ϕv	Resistance factors for shear			1.00	0.90	1.00	0.90	
ϕn	Resistance factors for axial force			1.00	1.00	1.00	1.00	
β1	Stress block factor			0.836	0.836	0.836	0.836	
c	Dis. Between centroid and top fiber		m	0.013	0.013	0.013	0.013	
	For T section behavior		m	0.013	0.013	0.013	0.013	
	For rectangular section behavior		m	0.013	0.013	0.013	0.013	
fpe	Effective stress in the prestressing steel after losses		Mpa	1116	1116	1116	1116	
fps	Aver. stress in pres. steel at the time for which the nominal resistance		Mpa	1847	1847	1847	1847	
k	Factor depends on type of P.S, Low relaxation strand k = 0.28			0.28	0.28	0.28	0.28	

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		Design			
		Check			
		Revise			

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REINFORCEMENT CHECKING - WING WALL

a	Depth of equivalent stress block	m	0.011	0.011	0.011	0.011	
de	Corresp. effective depth from extreme comp. fiber						
	to centroid of tensile force in the tensile reinf.	m	0.440	0.440	0.440	0.440	
Mn	Nominal resistance	kNm	302	302	302	302	
Mr	Factored reistance	kNm	**302**	**272**	**302**	**272**	
Mu	Flexual moment	kNm	91	**141**	**137**	**216**	
(5.7.3.2)	**Flexural moment Checking**		**OK**	**OK**	**OK**	**OK**	
	Limits for reinforcement						
c/de	Maximum reinforcement		0.03	0.03	0.03	0.03	
	Maximum reinforcement Checking	<= 0.42	**OK**	**OK**	**OK**	**OK**	
r min	Mimimum reinforcement		0.38%	0.38%	0.38%	0.38%	
	Mimimum reinforcement Checking for RC	0.23%	**OK**	**OK**	**OK**	**OK**	
1.2*Mcr	Craking moment	kNm	88	88	88	88	
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**		**OK**	**OK**	**OK**	**OK**	
(5.7.3.4)	**Conctrol of craking by distr. of reinf for RC member- Check?**		Yes	No	Yes	No	
	Existing condition for structrure	1,2 or 3	1	1	1	1	
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.060	0.060	0.060	0.060	
Z	Crack width parameter	N/mm	30000	30000	30000	30000	
A	Area of concr. with same centroid as tens. Reinf	m2	0.020	0.020	0.020	0.020	
fsa	Value	Mpa	282	282	282	282	
0.6*fy		Mpa	240	240	240	240	
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	240	240	240	240	
x	Dist. From compression fiber to centroid	m	0.095	-	0.095	-	
J.d	Arm	m	0.408	-	0.408	-	
Icr	Moment of inertia of the cracked section	m4	0.002	-	0.002	-	
fs	Tensile stress in reinforcement fs = Msls / (As*J.d)	Mpa	119	-	178	-	
	Checking for control cracking fs<fsa		**OK**	**N.a**	**OK**	**N.a**	
(5.10.8.2)	**Shrinkage and temperature Reinfocement (side distribution)**						
Areq	Area of required reinf	m2	0.00031	0.00031	0.00031	0.00031	
	Distribution on sides 7 D16	m2	0.00141	0.00141	0.00141	0.00141	
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45	
	Checking		**OK**	**OK**	**OK**	**OK**	

SHEAR AND TORSION CHECKING

β	Factor indicating diag. cracked concr. to tension		2.3	2.1	2.2	2.0	
θ	Angle of inclination of diagonal compressive	degree	33.99	38.23	35.60	40.89	
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	
bv	Effective web width as minimum web width - in dv	m	1.000	1.000	1.000	1.000	
dv	Effective shear depth	m	0.435	0.435	0.435	0.435	
	(de - a/2)	m	0.435	0.435	0.435	0.435	
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	
ncat	Amount of bars in spacing S	bars	2	2	2	2	
Av	Shear reinf area in spacing S	m2	0.0002	0.0002	0.0002	0.0002	
β	Assume		2.0	2.2	2.0	2.0	
θ	Assume	degree	34.57	38.94	36.11	41.24	
v	Shear stress in concrete	kN/m2	329	563	164	287	
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	
e_x	Strain in tensile reinforcement		8.32E-04	1.22E-03	9.67E-04	1.49E-03	
	if ex<0, multiple with reduce factor		-	-	-	-	
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok	
v/f'c	Ratio of shear stress and f'c		0.011	0.019	0.005	0.010	
β	Final value		2.3	2.1	2.2	2.0	
θ	Final value	degree	33.99	38.23	35.60	40.89	
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	459	416	444	387	
Vs	Shear resistance provided by shear reinforcement	kN	97	83	92	76	
Vp	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0	
Vn1	Vn1=Vc+Vs+Vp	kN	557	499	536	462	
Vn2	Vn2	kN	3260	3260	3260	3260	
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	557	499	536	462	
Vr	Factored shear resistance	kN	**557**	449	**536**	416	
Vu	Shear	kN	143	220	71	112	
(5.8.2.7)	**Shear checking**		**OK**	**OK**	**OK**	**OK**	

BORED PILE DESIGN

I. BORED PILE DATA

1.Load Combinations at top of bored pile

STT	Comb	Axial force P (KN)	Moment (KN.m)		
			Mx	My	Mxy
STRENGTH LIMIT STATES					
1	P_min	651	21.8	1100.9	1101
2	P_max	4219	59.8	1899.3	1900
3	Mx_max	4219	59.8	1899.3	1900
4	My_max	4219	59.8	1899.3	1900
EXTREME EVENT LIMIT STATES					
1	P_min	608	68	1179	1181
2	P_max	3206	8	1423	1423
3	Mx_max	3206	8	1423	1423
4	My_max	3206	8	1423	1423

2. Bored pile Material

Normal concrete			
Compressive strength at 28 days age	f'c	30	MPa
Concrete elastic modulus	Ec	27691	MPa
Reinforcement TCVN1651-2008; CBV-400			
Yield strength	fy	400	MPa
Reinforcement elastic modulus	Es	200,000	MPa

3. Bored pile Section

Pile diameter	D	1.00	m
Section area	A	0.785	m2
Moment inertia	Ix	0.049	m4
	Iy	0.049	m4
Radius of gyration of gross concrete section; r = sqrt(I/A)	rx	0.250	m
	ry	0.250	m

II. PILE DESIGN

1. Limit of Reinforcement S.5.7.4.2

Minimum area of longitudinal reinforcement in column				
As.fy / (Ag . f'c) >= 0.135		As ≥	0.008	m2
As / Ag >= 0.01		As ≥	0.008	m2
Maximum area of longitudinal reinforcement in column				
As / Ag <= 0.08		As ≤	0.063	m2
Trial Rebars:	Ok	As	**0.019**	m2
1layers x 24 = 24 bars D32	@150	As1	0.019	m2

2. Interation diagram M-P Using Pca-Column software

****Flexural check by pcaColumn**

Strength and Service limit states:

Resistance factor: Compression φ_c = 0.75 *(AASHTO LRFD-2004)*

Tension φ_t = 0.90

Extreme Event limit states:

Resistance factor Compression φ_c = 1.00

Tension φ_t = 1.00

<Result table>

STT	Comb	Pu kN	Mux kN-m	Muy kN-m	φ Mnx kN-m	φ Mny kN-m	φ Mn/Mu
STRENGTH LIMIT STATES							
1	P_max	650.5	21.8	1100.9	50	2526.9	2.295
2	P_min	4218.6	59.8	1899.3	90.6	2876.9	1.515
3	Mx_max	4218.6	59.8	1899.3	90.6	2876.9	1.515
4	My_max	4218.6	59.8	1899.3	90.6	2876.9	1.515
EXTREME EVENT LIMIT STATES							
1	P_max	608	68	1179	156.2	2707.8	2.297
2	P_min	3206	8	1423	18.1	3215.1	2.259
3	Mx_max	3206	8	1423	18.1	3215.1	2.259
4	My_max	3206	8	1423	18.1	3215.1	2.259

3. Column Ties S.5.7.4.6, S.5.10.6.3, S5.10.11.4.1d - e

Bridge is in seismic zone	Sz	1	
Area of concrete core measured out-to-out of ties	Ac	0.622	m2
Tie diameter	Dtie	16	mm2
Cross section area of 1 tie	As-tr	0.00020	m2
Spacing of hoops	s	75	mm
Length of reinfocement tie in 1 hoop	Ltie	2.78	m
Ratio of ties reinf. in one hoop/ volume of conc. core one pitch spacing			
ρs = As-tr . Ltie / (Ac * spacing)	ρs	0.0120	
Ratio of spiral reinf. To total volume of concrete core shall satisfy			S.5.7.4.6
ρs >= 0.45 . (Ag/Ac - 1). f'c/ fy = Req1	Req1	0.0089	**OK**
Transverse Reinforcement for Confinement at Plastic Hinges		S.5.10.11.4.1.d	
For a circular column "1:applied","2:Not applied"		1	
ρs >= 0.12 . f'c/ fy = Req2	Req2	0.0090	**N/A**

4. Shear Design

Shear resistance factors	ϕv	1.0	
Factored shear force	Vu	812	kN
Required shear capacity $Vn = Vu/\phi v$	Vn	812	kN
Determine concrete shear capacity			
Minimum shear reinforcement will provided in cross section			
Therefore	β	2.0	
	θ	45.0	deg
Diameter of bored pile	D	1.00	m
Width of cross section	b	1.00	m
dv = 0.9*de de = D/2 + Dr/pi()			
Diameter of the circle passing through the centers of the long. reinf.	Dr	0.79	m
	de	0.75	m
	dv	0.68	m
Vc = 0.083 . β . sqrt(f'c) . bv. dv	Vc	616	kN
Diameter of Transverse Reinforcement	D_v	13	mm
Spacing of Transverse Reinforcement	s	75	mm
Area of a transverse reinforcement within distance "s"	A_v	339.30	mm2
Angle of inclination of transverse reinforcement to longitudinal axis	α	90	deg.
Effetive shear depth, d_v			
Alternative 1: $d_{v1} = M_n/(A_s{*}f_y)$			
Normal flexural resistance	M_n	3215	KNm
	d_{v1}	**209**	mm
Alternative 2: $d_{v2} = 0.9d_e$	$d_e = D/2 + D_r/\pi$	752	mm
	d_{v1}	677	mm
Choice value of d 1 *("1" = d_{v1}, "2" = d_{v2})*	d_v	**209**	mm
Norminal shear resistance of Reinforcement	$V_s = \dfrac{A_v f_y d_v (\cot g\theta + \cot g\alpha)\sin\alpha}{s}$	**378**	kN
	$V_{n1} = V_c + V_s$	**994**	kN
	$V_{n2} = 0.25f'_c b_v d_v$	**1568**	kN
	V_n	**994**	kN
	Conclude		**OK**

SPACE PILE FOUNDATION ANALYSIS PROGRM
Turbo BASIC

PROJECT: : DN-QN-A12-ORB12

INITIA DATA

Kn = 0.14 Ax = 6.00 By = 12.60 Cz = 2.00
E v.uon = 3001028 E r.uon = 3001028 E v.nen = 3001028 E r.nen = 3001028
Mq = 75 (t/m4) Md = 0 (t/m4) m = 300 (t/m4)

LOAD COMBINATIONS

COMB.	Hx	Hy	P	Mx	My	Mz
1	541.46	0.00	2468.87	0.00	942.76	0.00
2	335.60	0.00	1767.79	0.00	605.05	0.00
3	537.83	0.00	2403.34	0.00	914.98	0.00
4	324.57	0.00	1656.55	0.00	500.70	0.00
5	362.75	0.00	1833.87	0.00	581.67	0.00
6	474.45	26.01	2207.33	-123.23	894.49	0.00
7	474.45	26.01	1581.59	-123.23	1155.71	0.00

PROPERTIES OF PILES

PILE	Lo	H	Bpx	Bpy	A	B	Cday	Fo	Io	Po	Co	Ct
1	0.00	9.00	1.181	1.181	1.00	0.000	0.000	0.785	0.098	500	30000	15000
2					n	t						
3					n	t						
4					n	t						
5					n	t						
6					n	t						
7					n	t						
8					n	t						

PILE COORD.

PILE	X	Y	Phi	Xi
1	2.00	4.80	0.000	0.00
2	2.00	2.31	0.000	0.00
3	2.00	-0.25	0.000	0.00
4	2.00	-2.81	0.000	0.00
5	2.00	-5.30	0.000	0.00
6	-2.00	-5.30	0.000	0.00
7	-2.00	-0.25	0.000	0.00
8	-2.00	4.80	0.000	0.00

DISPLACEMENTS

COMB.	X	Y	Z	Fix	Fiy	Fiz
1	0.01765	0.00024	0.002768	-0.000050	0.000392	0.000182

2	0.01085	0.00017	0.002015	-0.000036	0.000214	0.000113
3	0.01753	0.00024	0.002690	-0.000049	0.000390	0.000181
4	0.01044	0.00016	0.001894	-0.000034	0.000189	0.000109
5	0.01170	0.00017	0.002091	-0.000037	0.000221	0.000122
6	0.01551	0.00109	0.002475	-0.000062	0.000357	0.000178
7	0.01602	0.00105	0.001639	-0.000049	0.000527	0.000178

FORCES ON PILES

PILE	COMB.	N	Q2	Q3	M1	M2	M3
1	1	396.61	-67.03	1.09	2.360	4.754	204.284
	2	273.51	-41.57	0.68	1.464	3.075	127.431
	3	387.44	-66.58	1.09	2.345	4.695	202.888
	4	254.60	-40.22	0.66	1.416	2.947	123.726
	5	283.61	-44.94	0.73	1.583	3.283	138.024
	6	364.43	-59.08	-2.09	2.302	-5.125	179.749
	7	306.23	-58.92	-2.10	2.297	-5.469	174.974
2	1	383.47	-65.26	1.09	2.360	4.754	198.630
	2	264.09	-40.47	0.68	1.464	3.075	123.925
	3	374.65	-64.82	1.09	2.345	4.695	197.272
	4	245.78	-39.16	0.66	1.416	2.947	120.334
	5	273.84	-43.76	0.73	1.583	3.283	134.233
	6	348.34	-57.35	-2.09	2.302	-5.125	174.234
	7	293.48	-57.20	-2.10	2.297	-5.469	169.472
3	1	370.03	-63.45	1.09	2.360	4.754	192.853
	2	254.47	-39.35	0.68	1.464	3.075	120.342
	3	361.57	-63.03	1.09	2.345	4.695	191.533
	4	236.77	-38.07	0.66	1.416	2.947	116.867
	5	263.86	-42.55	0.73	1.583	3.283	130.360
	6	331.90	-55.59	-2.09	2.302	-5.125	168.598
	7	280.44	-55.44	-2.10	2.297	-5.469	163.848
4	1	356.59	-61.64	1.09	2.360	4.754	187.075
	2	244.85	-38.23	0.68	1.464	3.075	116.759
	3	348.49	-61.23	1.09	2.345	4.695	185.795
	4	227.75	-36.99	0.66	1.416	2.947	113.401
	5	253.88	-41.33	0.73	1.583	3.283	126.486
	6	315.45	-53.82	-2.09	2.302	-5.125	162.963
	7	267.40	-53.68	-2.10	2.297	-5.469	158.225
5	1	343.45	-59.87	1.09	2.360	4.754	181.422
	2	235.44	-37.13	0.68	1.464	3.075	113.253
	3	335.69	-59.47	1.09	2.345	4.695	180.179
	4	218.93	-35.93	0.66	1.416	2.947	110.009
	5	244.12	-40.15	0.73	1.583	3.283	122.696
	6	299.36	-52.10	-2.09	2.302	-5.125	157.448
	7	254.65	-51.96	-2.10	2.297	-5.469	152.722
6	1	179.66	-59.87	-1.74	2.360	-4.292	181.422
	2	146.11	-37.13	-1.08	1.464	-2.534	113.253
	3	172.62	-59.47	-1.73	2.345	-4.289	180.179
	4	139.73	-35.93	-1.04	1.416	-2.481	110.009
	5	151.77	-40.15	-1.16	1.583	-2.781	122.696
	6	150.08	-52.10	-4.85	2.302	-13.948	157.448
	7	34.00	-51.96	-4.85	2.297	-14.273	152.722
7	1	206.24	-63.45	-1.74	2.360	-4.292	192.853
	2	165.14	-39.35	-1.08	1.464	-2.534	120.342
	3	198.50	-63.03	-1.73	2.345	-4.289	191.533
	4	157.57	-38.07	-1.04	1.416	-2.481	116.867
	5	171.52	-42.55	-1.16	1.583	-2.781	130.360
	6	182.61	-55.59	-4.85	2.302	-13.948	168.598
	7	59.79	-55.44	-4.85	2.297	-14.273	163.848
8	1	232.82	-67.03	-1.74	2.360	-4.292	204.284

2	184.18	-41.57	-1.08	1.464	-2.534	127.431
3	224.38	-66.58	-1.73	2.345	-4.289	202.888
4	175.41	-40.22	-1.04	1.416	-2.481	123.726
5	191.26	-44.94	-1.16	1.583	-2.781	138.024
6	215.15	-59.08	-4.85	2.302	-13.948	179.749
7	85.59	-58.92	-4.85	2.297	-14.273	174.974

SUMMARY OF FORCES

	PILE	COMB.	N	Q2	Q3	M1	M2	M3
Nmin	6	7	34.00	-51.96	-4.85	2.297	-14.273	152.722
Nmax	1	1	396.61	-67.03	1.09	2.360	4.754	204.284
Q2max	1	1	396.61	-67.03	1.09	2.360	4.754	204.284
Q3max	6	7	34.00	-51.96	-4.85	2.297	-14.273	152.722
M1max	1	1	396.61	-67.03	1.09	2.360	4.754	204.284
M2max	6	7	34.00	-51.96	-4.85	2.297	-14.273	152.722
M3max	1	1	396.61	-67.03	1.09	2.360	4.754	204.284

CHECKING CALCULATI
IN COMPARISON WITH INITIA LOAD MATRIX

1	541.46	0.00	2468.87	0.00	942.76	0.00
2	335.60	0.00	1767.79	0.00	605.05	0.00
3	537.83	0.00	2403.34	0.00	914.98	0.00
4	324.57	0.00	1656.55	0.00	500.70	0.00
5	362.75	0.00	1833.87	0.00	581.67	0.00
6	474.45	26.01	2207.33	-123.23	894.49	0.00
7	474.45	26.01	1581.59	-123.23	1155.71	0.00

BEARING CAPACITY OF PILE -ORB12 BRIDGE

STT	Abut/pier	Boring	Water level	Bottom of pile	Top of rock	Bottom of pile Tip	Pile length	Bearing capacity of pile (T)		interal force of top pile (T)		Check
			m	m	m	m	m	STR	EX	STR	EX	
1	A1	ORB12-A1	2.4	9.00	2.40	-0.50	9.50	555	1020	430	327	OK
2	A2	ORB12-A2	3.7	9.00	5.40	3.00	6.00	532	975	430	327	OK

ASSUMPTION:

- The rock is reasonably sound
- The rock strength measured during site investigation will not deteriorate during construction when water or drilling fluids are used
- The bottom of the socket is properly cleaned out.

DATA & CALCULATION:

Bored hole name		**ORB12-A1**		Pile Concrete comp. strength	$f'_c =$	30.0	MPa	
Bottom of pilecap elevation	EL1 =	9.00		Concrete Unit Weight	$g_c =$	24.5	kN/m^3	
Top of socket elevation	EL2 =	2.40		Modulus of elasticity of concre	$E_c =$	27691	MPa	
Pile tip elevation	EL3 =	-0.50						
Pile Length	L =	9.50	m	Depth of socket	$H_s =$	2.90	m	
Diameter of drilled-shaft	$D_p =$	1.00	m	Diameter of socket	$D_s =$	1.00	m	
Pile Cross-Sectional Perimeter	P =	3.14	m	Socket Cross-Sect. Perimeter	$P_{soc} =$	3.14	m	
Pile Cross-Sectional Area	$A_b =$	0.79	m^2	Socket Cross-Sectional Area	$A_{soc} =$	0.79	m^2	
Working normal force at pile head	N =	6579.1	kN					
Working normal force at top of socket	$P_i =$	6545.6	kN					
Intack rock modulus	$E_i =$	50000	MPa					*Figure C10.8.3.5-2 Lrfd*
Modulus modification ratio	$K_e =$	0.05						*Figure C10.8.3.5-3 Lrfd*
Elastic modulus of the insitu rock	$E_r = K_e * E_i =$	2500.0	MPa					
Influence coefficient	$I_p = f(H_s/D_s, E_c/E_r) =$	0.38						*Figure C10.8.3.5-1 Lrfd*
	$H_s/D_s =$	2.90						
	$E_c/E_r =$	11.08						

Rock mass modulus/ intack rock modulus	E_m / E_i			*C.10.4.6.5-1-Lrfd 4th*
Atmospheric pressure	$p_a =$	0.101	MPa	
Reduction factor to account for jointing	α_E			*10.8.3.5.4b-Lrfd 4th*

The elastic shortening of the drilled shaft

$r_e = (\Sigma P_i) * H_s / (A_{soc} * E_c) =$ 0.873 mm

The settlement of base of drilled shaft

$r_{base} = (\Sigma P_i) * I_p / (D_s * E_r) =$ 0.995 mm

$r_e + r_{base} =$ **1.868** mm < 10 mm

Compute the bearing capacity based on Sharft resistance alone

Unit side resistance

Case 1: The drilling fluid used will not form a lubricated film on the sides of the socket

if qu < 1.9 Mpa - may be taken after Carter & Kulhawy 1988 → qs = 0.15*qu *C10.8.3.5-4*

if qu > 1.9 Mpa - may be taken after Horvath & Kenney 1979 → qs = 0.21*sqrt(qu) *C10.8.3.5-5*

Case 2: Side of rock socket is considered to be smooth or where the rock is drilled using a drilling slurry

May be taken after Horvath & Kenney 1979 → qs = 0.65*α_E*pa*(qu/pa)^0.5<7.8*pa*(f'c/pa)^0.5

qs0 = 7.8*pa*(f'c/pa)^0.5 *10.8.3.5.4d-1-Lrfd2007*

Drilling method used in construction: $\boxed{\textit{Case 2}}$

$Q_R = \phi * Q_n = \phi_s * Q_{SR}$

ϕ_s is the resistance factor - table 10.5.5-3 LRFD

q_u is the uniaxial compressive strength of the rock

Case1

No.	EL_T	EL_B	Depth (m)	RQD (%)	q_u (MPa)	q_s (MPa)	Q_{SR} (kN)	ϕ_s	Q_R (kN)
1	2.40	1.20	1.20	83	77.29	1.85	6960	0.65	4524
2	1.20	-0.10	1.30	27	15.46	0.83	3372	0.65	2192
3	-0.10	-0.50	0.40	80	78.20	1.86	2334	0.65	1517
4			-	-	-	-	-	-	-
5									
6									
7									
8									
Sum			**2.90**				**12666**		**8233**

Case2 ... Type: 1-closed joints; 2-open joints

No.	Depth (m)	RQD (%)	q_u (MPa)	E_m/E_i	α_E	Type	qs0 (MPa)	q_s (MPa)	*q_s - used* (MPa)	Q_{SR} (kN)	ϕ_s	Q_R (kN)
1	1.20	83.00	77.29	0.83	0.93	1	13.58	1.69	*1.69*	6381	0.55	3510
2	1.30	27.00	15.46	0.07	0.50	1	13.58	0.40	*0.40*	1647	0.55	906
3	0.40	80.00	78.20	0.80	0.92	1	13.58	1.68	*1.68*	2112	0.55	1162
4	-	-	-	-	-	-	-	-	-	-	-	-
5	-	-	-	-	-	-	-	-	-	-	-	-
6	-	-	-	-	-	-	-	-	-	-	-	-
7	-	-	-	-	-	-	-	-	-	-	-	-
8	-	-	-	-	-	-	-	-	-	-	-	-
Sum	**2.90**									**10140**		**5577**

Unit base resistance $q_p = Kb.(p1-p0) + \sigma v$

Limit pressure determined from presuremeter tests	$p_l =$	-	MPa	*C10.8.3.5-7*
At rest total horizontal stress measured at ther base elevation	$p_0 =$	-	MPa	
Coefficient that depen on diameter socket	$K_b =$	4.14		*Table C10.8.3.5-1*
Total vertical stress at the base elevation	$\sigma v =$	-	MPa	
	$q_p =$	-	MPa	
	$\phi =$	0.50		*Table 10.5.5-3*
$Q_{pR} = Ap . qp$	$Q_{pR} =$	-	kN	
$Q_R = \phi.Q_{pR}$	$Q_R =$	-	kN	

ESTIMATED PILE CAPACITY:

Pile Structural Capacity $Q_T = 0.75*0.85*0.85*f'c*Ag$	*12768*	kN	*1301*	T
Pile resistance Q_R	*5577*	kN	*569*	T
Deducting pile weight	*-132*	kN	*-13*	T
Estimated Pile Capacity (STR)	*5445*	kN	*555*	T
Estimated Pile Capacity (EXT)	*10008*	kN	*1020*	T

ASSUMPTION:

- The rock is reasonably sound
- The rock strength measured during site investigation will not deteriorate during construction when water or drilling fluids are used
- The bottom of the socket is properly cleaned out.

DATA & CALCULATION:

Bored hole name		ORB12-A2		Pile Concrete comp. strength	f'_c =	30.0	MPa
Bottom of pilecap elevation	EL1 =	9.00		Concrete Unit Weight	g_c =	24.5	kN/m^3
Top of socket elevation	EL2 =	5.40		Modulus of elasticity of concre	E_c =	27691	MPa
Pile tip elevation	EL3 =	3.00					
Pile Length	L =	6.00	m	Depth of socket	H_s =	2.40	m
Diameter of drilled-shaft	D_p =	1.00	m	Diameter of socket	D_s =	1.00	m
Pile Cross-Sectional Perimeter	P =	3.14	m	Socket Cross-Sect. Perimeter	P_{soc} =	3.14	m
Pile Cross-Sectional Area	A_b =	0.79	m^2	Socket Cross-Sectional Area	A_{soc} =	0.79	m^2
Working normal force at pile head	N =	6276.3	kN				
Working normal force at top of socket	P_i =	6248.5	kN				
Intack rock modulus	E_i =	50000	MPa	*Figure C10.8.3.5-2 Lrfd*			
Modulus modification ratio	K_e =	0.05		*Figure C10.8.3.5-3 Lrfd*			
Elastic modulus of the insitu rock	$E_r = K_e{*}E_i$ =	2500.0	MPa				
Influence coefficient	$I_p = f(H_s/D_s, E_c/E_r)$ =	0.42		*Figure C10.8.3.5-1 Lrfd*			
	H_s/D_s =	2.40					
	E_c/E_r =	11.08					

Rock mass modulus/ intack rock modulus	E_m / E_i			*C.10.4.6.5-1-Lrfd 4th*
Atmospheric pressure	p_a =	0.101	MPa	
Reduction factor to account for jointing	α_E			*10.8.3.5.4b-Lrfd 4th*

The elastic shortening of the drilled shaft

$$r_e = (\Sigma P_i){*}H_s/(A_{soc}{*}E_c) = 0.690 \text{ mm}$$

The settlement of base of drilled shaft

$$r_{base} = (\Sigma P_i){*}I_p/(D_s{*}E_r) = 1.050 \text{ mm}$$

$$r_e + r_{base} = \mathbf{1.739} \text{ mm} < 10 \text{ mm}$$

Compute the bearing capacity based on Sharft resistance alone

Unit side resistance

Case 1: The drilling fluid used will not form a lubricated film on the sides of the socket

if qu < 1.9 Mpa - may be taken after Carter & Kulhawy 1988 → qs = 0.15*qu — *C10.8.3.5-4*

if qu > 1.9 Mpa - may be taken after Horvath & Kenney 1979 → qs = 0.21*sqrt(qu) — *C10.8.3.5-5*

Case 2: Side of rock socket is considered to be smooth or where the rock is drilled using a drilling slurry

May be taken after Horvath & Kenney 1979 → $qs = 0.65{*}\alpha_E{*}pa{*}(qu/pa)^{0.5} < 7.8{*}pa{*}(f'c/pa)^{0.5}$

$qs0 = 7.8{*}pa{*}(f'c/pa)^{0.5}$ — *10.8.3.5.4d-1-Lrfd2007*

Drilling method used in construction: *Case 2*

$Q_R = \phi{*}Q_n = \phi_s{*}Q_{SR}$

ϕ_s is the resistance factor - table 10.5.5-3 LRFD

q_u is the uniaxial compressive strength of the rock

Case1

No.	EL_T	EL_B	Depth (m)	RQD (%)	q_u (MPa)	q_s (MPa)	Q_{SR} (kN)	ϕ_s	Q_R (kN)
1	5.40	4.40	1.00	50	78.34	1.86	5839	0.65	3796
2	4.40	3.40	1.00	60	75.35	1.82	5727	0.65	3722
3	3.40	3.00	0.40	70	75.35	1.82	2291	0.65	1489
4									
5									
6									
7									
8									
Sum			**2.40**				**13857**		**9007**

	DANANG QUANG NGAI EXPRESSWAY	Item.	Eng.	Date.	Sign.
	ORB12 BRIDGE	Design			
	DETAIL DESIGN	Check			
	EMPIRICAL ESTIMATION OF PILE CAPACITY-ABUTMENT A2	Revise			

Case2: [illegible] Type: 1 closed joints ; 2 open joints

No.	Depth (m)	RQD (%)	q_u (MPa)	E_m / E_i	α_E	Type	qs0 (MPa)	q_s (MPa)	q_s - used (MPa)	Q_{SR} (kN)	ϕ_s	Q_R (kN)
1	1.00	50.00	78.34	0.15	0.59	1	13.58	1.07	*1.07*	3375	0.55	1856
2	1.00	60.00	75.35	0.42	0.76	1	13.58	1.37	*1.37*	4295	0.55	2362
3	0.40	70.00	75.35	0.70	0.88	1	13.58	1.58	*1.58*	1983	0.55	1091
4	-	-	-	-	-	-	-	-	-	-	-	-
5	-	-	-	-	-	-	-	-	-	-	-	-
6	-	-	-	-	-	-	-	-	-	-	-	-
7	-	-	-	-	-	-	-	-	-	-	-	-
8	-	-	-	-	-	-	-	-	-	-	-	-
Sum	**2.40**									**9653**		**5309**

Unit base resistance $q_p = Kb.(p1-p0) + \sigma v$

Limit pressure determined from presuremeter tests	$p_l =$	5.89	MPa	*C10.8.3.5-7*
At rest total horizontal stress measured at ther base elevation	$p_0 =$	-	MPa	
Coefficient that depen on diameter socket	$K_b =$	3.84		*Table C10.8.3.5-1*
Total vertical stress at the base elevation	$\sigma v =$	-	MPa	
	$q_p =$	-	MPa	
	$\phi =$	0.50		*Table 10.5.5-3*
$Q_{pR} = Ap . qp$	$Q_{pR} =$	-	kN	
$Q_R = \phi.Q_{pR}$	$Q_R =$	-	kN	

ESTIMATED PILE CAPACITY:

Pile Structural Capacity $Q_T = 0.75*0.85*0.85*fc*Ag$	*12768*	kN	*1301*	T
Pile resistance Q_R	*5309*	kN	*541*	T
Deducting pile weight	*-88*	kN	*-9*	T
Estimated Pile Capacity (STR)	*5221*	kN	*532*	T
Estimated Pile Capacity (EXT)	*9565*	kN	*975*	T

MINISTRY OF TRANSPORT

VIETNAM EXPRESSWAY CORPORATION - PROJECT MANAGEMENT UNIT NO.85

DA NANG - QUANG NGAI EXPRESSWAY DEVELOPMENT PROJECT

PACKAGE: 6

BRIDGE

CB12

CALCULATION SHEETS

HA NOI, 2012

Da Nang Quang Ngai Expressway project

BRIDGE

CB12

Km45+540.00

CALCULATION SHEETS

ABUTMENT A2

Da Nang, 2013

Assumptions :

1. Bridge is considered to be in seismic with acceleration coeff. A = 0.0310 g
2. The Design of the Abutment accords with Specification for bridge design 22-TCN-272-05 and AASHTO LRFD 2004 for reference
3. Design live load: HL-93 and lane loading 9.3 KN/m

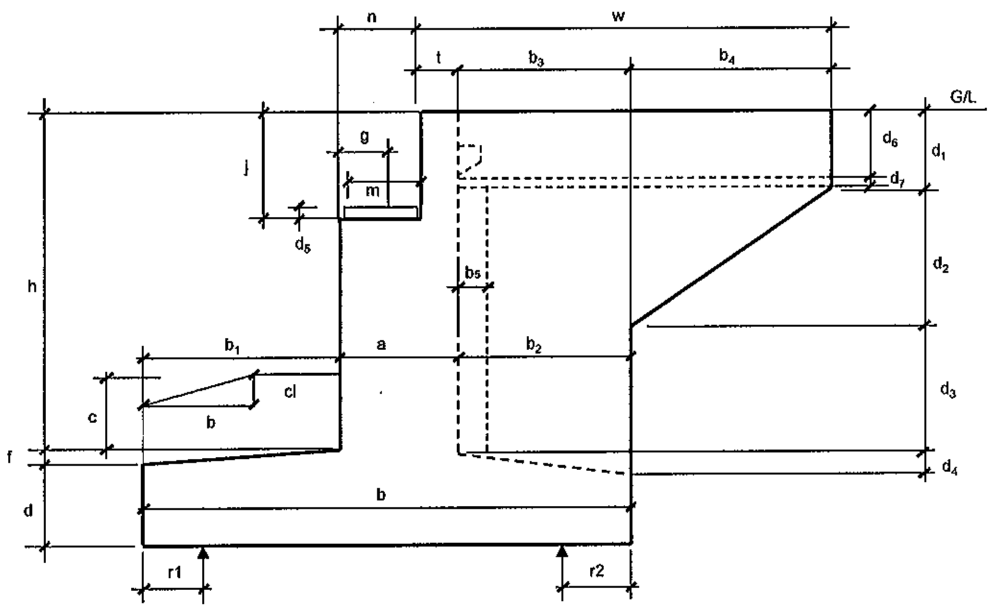
Input :

Level Table(at center of abutment)

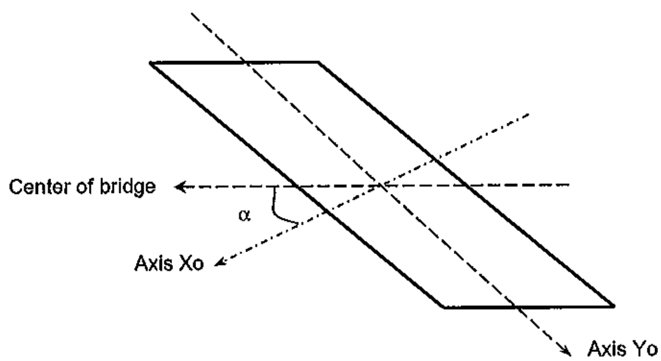
Level of top of headwall	HTwL	**18.978**	m
Level of top of bearing	BTL	**17.014**	m
Level of top of stem abutment	HTL	**16.864**	m
Level of top of footing	FTL	**11.000**	m
Level of bottom of footing	FBL	**9.000**	m
Ground level	GL	**11.700**	m
Lowest water level	HWL	**11.700**	m
Skew angle	α	**20.00**	deg

I.Loads from substructure

Abutment dimensions VERTICAL VIEW



PLAN VIEW

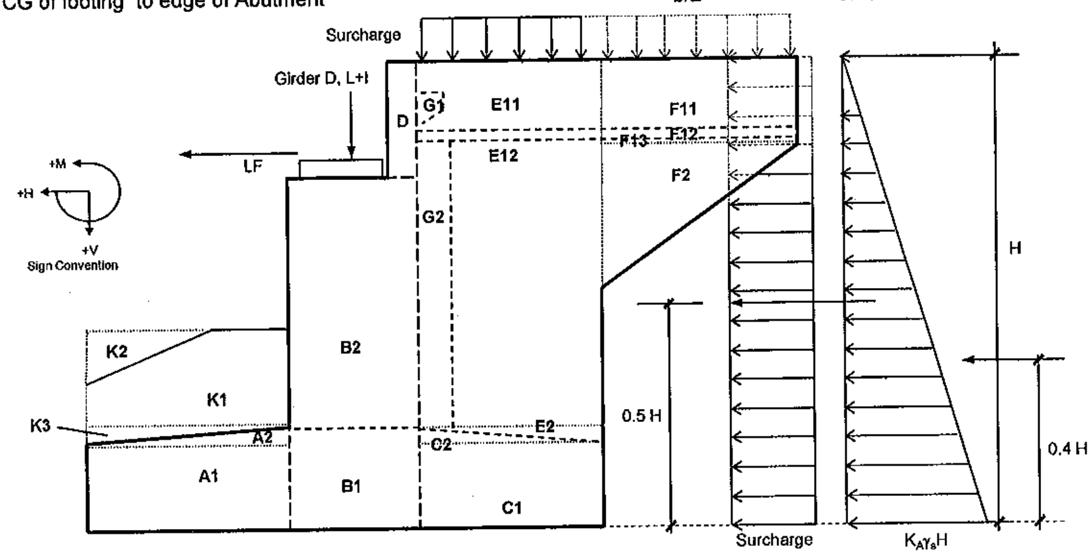


Material Unit Weights

- Unit Weight of Reinf. concrete γ_c = 24.5 kN/m^3
- Unit Weight of Soil γ_s = 18.0 kN/m^3
- Unit Bouyancy Weight of Soil γ_{sbo} = 8.2 kN/m^3

ABUTMENT DIMENSIONS (IN METRES)					
Item	Symbol	Value	Item	Symbol	Value
Height of stem	h	7.978	Horizontal Dimension	b_3	3.000
Footing Width	b	6.500	Horizontal Dimension	b_4	2.275
Stem Width	a	1.500	Horizontal Dimension	b_5	0.500
Footing Depth	d	2.000	Vertical Dimension	d_1	2.000
Footing Slope	f	0.000	Vertical Dimension	d_2	2.275
Width of stem at bearing	n	1.000	Vertical Dimension	d_3	3.703
Ballast Wall Height	j	2.114	Vertical Dimension	d_4	0.000
Ballast Wall Thickness	t	0.500	Vertical Dimension	d_5	0.150
Wingwall Length	w	6.000	Vertical Dimension	d_6	1.161
Soil Cover at Toe	c	0.700	Vertical Dimension	d_7	0.300
Girder Reaction	g	0.550	With of bearing pad	m	0.600
Distance to cl of pile	r1	1.000	Wingwall Thickness	u1	0.500
Horizontal Dimension	b_1	2.000	Wingwall Thickness	u2	0.500
Horizontal Dimension	b_2	3.000	Distance to cl of pile	r2	1.000

	cos (α)	=	0.94	
Slope front of abutment	cl	=	0.00	m
	bl	=	0.00	m
Width of Abutment	L		12.600	m
Width of abutment (inclined direction)	Ltr		13.409	m
Height of Abutment	Ht	=	9.98	m
Distance from CG of footing to edge of Abutment	b/2	=	3.25	m



1. Self weight of Abutment (DC)

Description	Area (m^2)	Length (m)	Force (kN)	X[1] (m)	Arm[2] (m)	Moment (kN•m)
SW of Abutment (DC)						
Section A1	4.000	13.409	1314	1.000	2.250	2957
Section A2	-	13.409	-	1.333	1.917	-
Section B1	3.000	13.409	986	2.750	0.500	493
Section B2	8.796	13.409	2890	2.750	0.500	1445
Section C1	6.000	13.409	1971	5.000	-1.750	-3449
Section C2	-	13.409	-	4.500	-1.250	-
Section D	1.057	13.409	347	3.250	-	-
Section E11	5.100	0.500	62	5.000	-1.750	-109
Section E12	17.934	0.500	220	5.000	-1.750	-384
Part extra stem	4.989	0.740	90	5.750	-2.500	-226
Section F11	2.641	0.500	32	7.638	-4.388	-142
Section F12	0.791	0.500	10	6.138	-2.888	-28
Section F13	1.226	0.500	15	7.638	-4.388	-66
Section F2	2.588	0.500	32	7.258	-4.008	-127
Section G1	0.135	12.909	285	3.650	-0.400	-114
Section G2	0.125	13.034	40	3.750	-0.500	-20
Bearing seats (w1seat= 0.70m)	0.090	3.500	10	2.550	0.700	7
Curbs +Handrail on Abutment	0.50	6.000	79	6.000	-2.750	-219
Total SW of Abutment (DC)			*8383*			*16*
Tranverser moment			*452*		*6.175*	*2789*

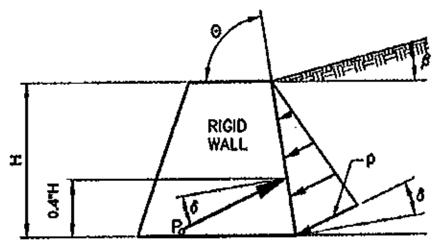
Notes: 1. Distance 'X' is measured horizontally from Toe of Retaining to CG of Section
2. Moment 'Arm' is measured from CG horizontally and from Underside of Footing Vertically.

2. Earth on Abutment (EV)

Description	Area (m^2)	Length (m)	Force (kN)	$X^{(1)}$ (m)	$Arm^{(2)}$ (m)	Moment (kN•m)
Earth on Abutment (EV)						
Section E1	23.93	12.909	5561	5.000	-1.750	-9732
Section E2	-	12.909	-	5.500	-2.250	-
Section E3	-	0.500	-	6.500	-3.250	-
Section K1	1.400	13.409	338	1.000	2.250	-
Section K2	-	13.409	-	-	3.250	-
Section K3	-	13.409	-	0.667	2.583	-
Total Earth on Footing			*5899*			*-9732*

3. Horizontal Earth Pressure on Abutment (EH)

To be safe, horizontal earth pressure at front face of abutment may be neglected.
Horizontal earth pressure at behind face of abutment shall be considered.



- Height for horizontal earth pressure — H = 9.98 m
- Width for horizontal earth pressure — W 13.41 m
- Density of Soil — γ_s = 1835 kg/m^3
- Internal Friction Angle of Soil — ϕ'_f = 30.0 deg
- Incline angle of back face wall — θ = 90.0 deg
- Friction angle between fill and wall — δ = 30.0 deg
- Incline angle of fill soil — β = 0.0 deg
- Gravitational acceleration — g = 9.81 m/s^2
- Basic earth pressure

p = K.γs. g. Z .10^-9 (Mpa, Z:mm)
K: taken as Ka (assume wall move or deflect sufficiently to reach minimum active conditions)

$$Ka = \frac{\sin^2(\theta + \phi'_f)}{\Gamma.[\sin^2 \theta . \sin(\theta - \delta)]}$$

Γ = 2.914
Ka = 0.297
p = 0.053 Mpa

$$\Gamma = \left[1 + \sqrt{\frac{\sin(\phi'_f + \delta).\sin(\phi'_f - \beta)}{\sin(\theta - \delta).\sin(\theta + \beta)}}\right]^{-2}$$

Horizontal earth pressure:
- Ea = 0,5. p. Z . B . 10^3 (kN) — Ea = 3570 kN
- M= Ea . 0,4H — M = 14250 kNm
- Horizontal Earth Pressure act at a height of 0.4 H

<S 3.11.5.1>

4. Earth Pressure on Abutment due to Surcharge (ES)

Equivalent height of soil for highway loading taken from Table 3.11.6.2.1

H=	1.50m heq=	1.7 m	
H=	3.00m heq=	1.2 m	
H=	6.00m heq=	0.76 m	
H=	9.00m heq=	0.61 m	
H=	9.98m heq=	0.61 m	(Linear interpolation)

- **Vertical force**

ESv = 442 kN
ev = -1.75 m
M = -773 kNm

- **Horizontal force**

$$\Delta p = k\gamma_s g h_{eq} x 10^{-9}$$

ESh = 437 kN
eh = 4.99 m
M = 2178 kNm

5. Earthquake effects

Bridge is located at: Thang Binh district - Quang Nam province

According to TCXDVN 375:2006 and 22TCN272-05, bridge is in seismic zone 2and acceleration coefficient as below

• Peak ground acceleration coefficient	A	=	0.0310 g

5.1. Seismic active lateral Earth pressure (E_{AE})

• Backfill slop angle	i	=	0.0 deg
• Slope of wall to vertical	β'	=	0.0 deg
• Angle of friction of soil	ϕ	=	30.0 deg
• Angle of friction between soil and abutment	δ	=	30.0 deg
• Horizontal acceleration coefficient	kh	=	0.047
• Vertical acceleration coefficient	kv	=	0.019
• Angle $\theta = \arctan(k_h/(1-k_v))$	θ	=	2.7 deg

$$K_{AE} = \frac{\cos^2(\phi-\theta-\beta)}{\cos\theta.\cos^2\beta.\cos(\delta+\beta+\theta)} x \left[1+\sqrt{\frac{\sin(\phi+\delta).\sin(\phi-\theta-i)}{\cos(\delta+\beta+\theta).\cos(i-\beta)}}\right]^{-2}$$

• Seismic active lateral Earth pressure coefficient	K_{AE}	=	0.33

• $E_{AE} = 0,5.\ g.\ \gamma.\ H^2\ .\ (1\text{-}kv)\ .\ K_{AE}.\ 10^{\wedge}\text{-}9$ (kN/m)

• Seismic active lateral Earth pressure coefficient	E_{AE}	=	3894 kN	
$M_{AE} = E_{AS}*0.3H+(E_{AE} - E_{AS})*0.6H$	M_{AE}	=	12624 KNm	<A.11.1.1.1>

E_{AS} is the static component of seismic active pressure calculated with θ = kv = 0

5.2. Earthquake effects to abutment (EQ)

Seismic force for substructures: elements above ground Fh = Csm* W; elements under ground Fh = A*S*W

• Soil profile type	Soil type		I
• Site Coefficients.	S	=	1.0
• Elastic Seismic Response Coefficient	2.5A	=	0.078
Csm = 1.2*A*S/ Tm^2/3 ≤ 2.5*A	Csm	=	0.036
• Period of vibration of the fundamental mode			
Tm = 2*pi()*sqrt(m/k)	Tm	=	1.057 s

Decription	Area (m^2)	Length (m)	Force (kN)	X[1] (m)	Arm[2] (m)	Moment (kN•m)
Section A1	4.000	13.409	41	-	1.000	41
Section A2	-	13.409	-	-	2.000	-
Section B1	3.000	13.409	31	-	1.000	31
Section B2	8.796	13.409	104	-	4.932	511
Section C1	6.000	13.409	61	-	1.000	61
Section C2	-	13.409	-	-	2.000	-
Section D	1.057	13.409	12	-	8.921	111
Section E11	5.100	0.500	2	-	7.398	14
Section E12	17.934	0.500	7	-	3.259	
Section E2	4.989	0.740	3	-	2.000	6
Section F11	2.641	0.500	1	-	7.398	7
Section F12	0.791	0.500	0	-	6.667	
Section F13	1.226	0.500	0	-	7.709	
Section F2	2.588	0.500	1	-	7.220	7
Section G1	0.135	12.909	1	-	7.265	10
Section G2	0.125	13.034	1	-	3.259	4
Total EQ of Abutment Selfweight			***265***			***803***

6. Braking Force(BR)

Take 50 % Braking Force for this Abutment (Free Bearing)					
• Number of lanes	n	=	3	lanes	
• Multiple presence factor	m	=	0.85		
• Take 25 % of Truck load					
BR = 25% * n * m * (2*145+35)	BR	=	104	kN	Long. Axis
• Acting at 1.8m higher of road face	e	=	11.8	m	
	Mlong	=	1227	KNm	Long. Axis

7. Centrifugal Force , CE (3.6.3)

• Plan of bridge (1:"straight",2: "Curve")			1		
• Design Speed	V	=	120	km/h	
	V	=	33.3	m/s	
	R		-	m	
$C = 4/3*(V^2/gR)$	C		-		
Acting at 1.8m higher of road face					
CE = n * m * (2*145+35) * C	CE	=	0.00	KN	
	e	=	11.86	m	
	Mtrans	=	0.00	KNm	Trans. Axis

8. Water Load (WA) ***:NA***

II.Loads from superstructure

Item	Sign	Value	Unit
Span length	Lsp	27.00	m
Span between bearings	Lb	26.20	m
Skew angle	α	20.00	deg
Deck slab length	Ldeck	27.00	m
Bridge Width	Bc	12.48	m
Girder height	hgi	1.50	m
Deck slab depth	hdkslab	0.26	m
Asphalt depth	has	0.084	m
Unit weight of concrete	γc	24.50	kN/m3
Unit weight of asphalt concrete	γa	22.10	kN/m3

1.Dead loads (DC): One span at abutment

Item	Sign	Value	Unit
1.1.Girders			
Weight of 1 girder	DC	464.77	kN
Number of girders	n	5	Girders
Sum of girders weight	DC	2323.83	kN
Precast Planks	DC	397.49	kN
Diaphragm	DC	145.48	kN
Total	DC	2866.79	kN
1.2.Deck slab			
Deck slab	DC	2196.32	kN
1.3.Pavement			
Asphalt concrete	DW	575.51	kN
1.4.Handrail			
Handrail + median	DC	639.90	kN

2.Live load (LL):

Truck	145 145 35 kN 4.3 4.3 m		
Tandem	110 110 kN 1.2 m		
Lane load	w_l 9.3 kN/m		
Pedestrial	Wpd= 0.0 kN/m2		
Considerate structure as a simple span Reaction Influence	26.2 m 1		
Number of lanes	n	3	
Multiple presence factor	m	0.85	
Dynamic load allowance	1+IM	1.25	

*Reaction = [(1+IM)*Vehicle + Laneload]*n*m*

Truck	P1(kN)	P2(kN)	P3(kN)	Sum(kN)	Total(kN)
Axle load	145	145	35		
Influence value	1.000	0.836	0.672		
Reaction	145.0	121.2	23.5	289.7	923.5

Tandem	P1(kN)	P2(kN)	Sum(kN)	Total(kN)
Axle load	110	110		
Influence value	1.000	0.954		
Reaction	110	105.0	215.0	685.2

Lane load	Wl(kN/m)	Total(kN)
Value	9.3	
Influence value	13.1	
Reaction	121.8	310.7

Pedestrian	Wdb(kN)	Total(kN)
Reaction	0.0	0.0

3.Earthquake effects on superstructrue (EQ)

Longitudinal moveable bearings at Abutment

Horizontal force from superstructure due to EQ - transverse direction

At bearing — Heq = 113 kN

4.Uniform Temperature, Srinkage& Creep(TU+SH&CR)

Bearing displacement due to uniform temperature and shrinkage creep — Δu = 0.026 m

$H = G.A.\Delta u/h_{rt}$ <14.6.3.1-2>

Shear modulus G — G = 1 MPa

Bearing area — A = 0.158 m2

Height of elastomeric layers — hrt 0.064 m

Number of bearing — nb = 5 bears

Horizontal force due to TU+SH&CR

Acting at top of bearing — H(tu+sh+cr) = 320 kN

5.Wind loads (Ws)

5.1.Tranverse wind on superstructure (WS)

Wind zone	Zone		III
Basic 3 second gust wind	Vb	=	53.00 m/s
Correction factor	S	=	1.09
Design wind velocity	V	=	57.77 m/s
Drag coefficient	Cd	=	1.40
Overall width of bridge	b	=	12.48 m
Depth of superstructure(including solid parapet)	d	=	2.82 m
	b/d	=	4.42
Windy obstructed area of superstructure	At	=	76.17 m2

Force due to tranverse wind

Fhy=max(0.0006*V2*At*Cd,1.8*At) (kN) — Fhy = 213.9 kN <3.8.1>

5.2.Wind load on vehicles (WL)

Transverse wind on vehicles — Wltran = 1.50 kN/m

Transverse horizontal force due to wind on liveload

At 1.8m from surface — Fhy = 40.50 kN

6.Combinations

Loads from superstructure to Abutment

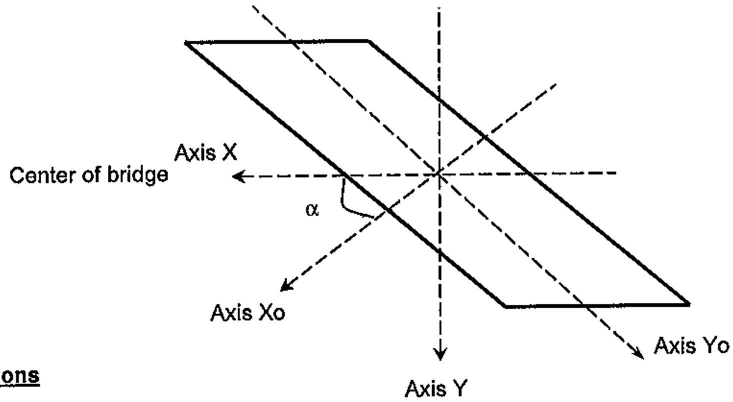
Loads at bottom of stem		Vertical		Longitudinal			Tranversal		
Loads	Sign	N (kN)	x (m)	Hx (kN)	z_1 (m)	My (kN•m)	Hy (kN)	y (m)	Mx (kN•m)
Girder + Deckslab	DC	2532	0.20			506			
Handrail	DC	320	0.20			64			
Pavement	DW	288	0.20			58			
LiveLoad	LL	1234	0.20			247		1.38	1697
Pedestrial	PL	0	0.20			0		-	-
Trans. wind on Struc.	WS						107	5.86	627
Trans. wind on vehl.	WL						20	7.66	155
Eearth quake	EQ						113	5.86	660
TU+SH&CR	TU+SH&CR			320	5.86	1876			

Loads at bottom of pilecap		Vertical		Longitudinal			Tranversal		
Loads	Sign	N (kN)	x (m)	Hx (kN)	z_1 (m)	My (kN•m)	Hy (kN)	y (m)	Mx (kN•m)
Girder + Deckslab	DC	2532	0.70			1772			
Handrail	DC	320	0.70			224			
Pavement	DW	288	0.70			201			
LiveLoad	LL	1234	0.70			864		1.38	1697
Pedestrial	PL	0	0.70			0		-	-
Trans. wind on Struc.	WS						107	7.86	841
Trans. wind on vehi.	WL						20	9.66	196
Eearth quake	EQ						113	7.86	885
TU+SH&CR	TU+SH&CR			320	7.86	2516			

Loads		Load combinations						
	Sign	Str-IA	Str-IB	Str-IIIA	Str-IIIB	Ser-I	Ext-IA	Ext-IB
Girder + Deckslab	DC	1.25	0.90	1.25	0.90	1.00	1.25	0.90
Handrail	DC	1.25	0.90	1.25	0.90	1.00	1.25	0.90
Pavement	DW	1.50	0.65	1.50	0.65	1.00	1.50	0.65
LiveLoad	LL	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Pedestrial	PL	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Trans. wind on Struc.	WS			0.40	0.40	0.30		
Trans. wind on vehi.	WL			1.00	1.00	1.00		
Eearth quake	EQ						1.00	1.00
TU+SH&CR	TU+SH&CR	0.50	0.50	0.50	0.50	1.00		

Load combinations at bottom of stem					
		Longitudinal		Tranversal	
Load combinations	N (kN)	Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	6156	160	2169	0	2970
Strength Str-IB	4913	160	1921	0	2970
Strength Str-IIIA	5662	160	2070	63	2697
Strength Str-IIIB	4419	160	1822	63	2697
Service Ser-I	4373	320	2751	52	2040
Extreme Ext-IA	4613	0	923	113	1509
Extreme Ext-IB	3370	0	674	113	1509

Load combinations at bottom of pilecap					
		Longitudinal		Tranversal	
Load combinations	N (kN)	Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	6156	160	5567	0	2970
Strength Str-IB	4913	160	4697	0	2970
Strength Str-IIIA	5662	160	5221	63	2823
Strength Str-IIIB	4419	160	4352	63	2823
Service Ser-I	4373	320	5577	52	2145
Extreme Ext-IA	4613	0	3229	113	1734
Extreme Ext-IB	3370	0	2359	113	1734



III.Load Combinations

1.Loads from substructure

Loads at bottom of pilecap	Sign	Vertical		Longitudinal			Tranversal		
		N (kN)	x (m)	H_x (kN)	z_1 (m)	M_y (kN•m)	H_y (kN)	y (m)	M_x (kN•m)
Self weight of Abutment	DC	8383				16			603.305
Soils on pilecap	EV	5899				-9732			
Horizontal Earth Pressure	EH			3355		13391			
Vertical Surcharge	L_{Sv}	442				-773			
Horizontal Surcharge	L_{Sh}			410		2047			
Braking Force	BR			104		1227			
Centrifucal Force	CE			-		-	-		-
Buoyancy of Abutment	WA	-1869				-33			
Buoyancy of Earth on Abutment	WA	-440				33			
Earthquake effects to Abutment	EQ			265		803	80		241
Earthquake effects to soil	E_{AE}			3659		11863			

Table of load factors

Loads	Sign	Load combinations						
		Str-IA	Str-IB	Str-IIIA	Str-IIIB	Ser-I	Ext-IA	Ext-IB
Self weight of Abutment	DC	1.25	0.90	1.25	0.90	1.00	1.25	0.90
Soils on pilecap	EV	1.35	0.90	1.35	0.90	1.00	1.35	0.90
Horizontal Earth Pressure	EH	1.50	0.90	1.50	0.90	1.00	0.00	0.00
Vertical Surcharge	L_{Sv}	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Horizontal Surcharge	L_{Sh}	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Braking Force	BR	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Centrifucal Force	CE	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Buoyancy of Abutment	WA	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Buoyancy of Earth on Abutment	WA	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Earthquake effects to Abutment	EQ						1.00	1.00
Earthquake effects to soil	E_{AE}						1.00	1.00

Load Combination at bottom of pilecap					
Load combinations	N (kN)	Longitudinal		Tranversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	16907	5932	11345	0	754
Strength Str-IB	11319	3919	7684	0	543
Strength Str-IIIA	16731	5726	10345	0	754
Strength Str-IIIB	11142	3713	6684	0	543
Service Ser-I	12416	3869	6176	0	603
Extreme Ext-IA	16355	4181	798	80	995
Extreme Ext-IB	10766	4181	5172	80	784

2. Loads from superstructure

Load Combination at bottom of pilecap						
		Longitudinal		Tranversal	
Load combinations	N (kN)	Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	6156	160	5567	0	2970
Strength Str-IB	4913	160	4697	0	2970
Strength Str-IIIA	5662	160	5221	63	2823
Strength Str-IIIB	4419	160	4352	63	2823
Service Ser-I	4373	320	5577	52	2145
Extreme Ext-IA	4613	0	3229	113	1734
Extreme Ext-IB	3370	0	2359	113	1734

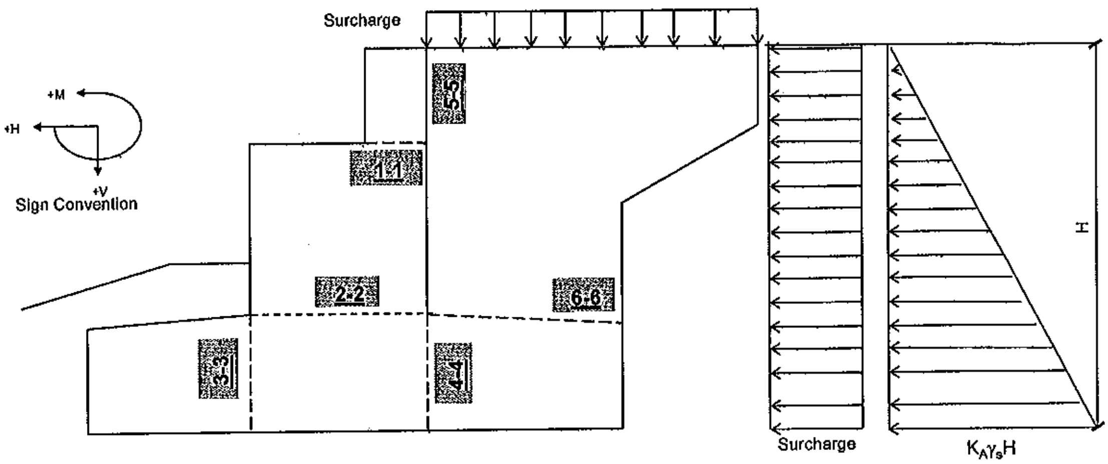
3. Total loads at bottom of pilecap

Load Combination at bottom of pilecap					
		Longitudinal		Tranversal	
Load combinations	N (kN)	Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	23063	6092	16912	0	3724
Strength Str-IB	16232	4079	12382	0	3513
Strength Str-IIIA	22393	5886	15566	63	3577
Strength Str-IIIB	15561	3873	11036	63	3366
Service Ser-I	16789	4189	11753	52	2748
Extreme Ext-IA	20968	4181	4027	192	2729
Extreme Ext-IB	14137	4181	7531	192	2517

IV.Elements checking

The abutment walls shall be checked at sections 1-1, 2-2, 3-3, 4-4, 5-5

1. Calculate internal force of sections



1.1 Section 1-1
Table of Load Factors

Loads	Sign	Service I Ser-I	Strength I Str-IA	Strength I Str-IB	Extreme I Ext-I
Selfweight	DC	1.00	1.25	0.90	1.25
Horizontal Earth Pressure	EH	1.00	1.50	0.90	
Surchage (Horizontal)	LS_h	1.00	1.75	1.75	0.50
Horizontal Seismic Earth Pressure	E_{AE}				1.50
Abutment earthquake force	EQ				1.00

Unfactored Loads	N (kN)	Longitudinal Hx (kN)	Longitudinal My (kN.m)	Tranversal Hy (kN)	Tranversal Mx (kN.m)
Selfweight	632		-114		
Horizontal Earth Pressure		171	144		
Surcharge (horizontal)		241	255		
Horizontal Seismic Earth Pressure		186	128		
Abutment earthquake force		14	15	4	4

Load Combination at bottom of headwall

Load combinations	N (kN)	Longitudinal Hx (kN)	Longitudinal My (kN.m)	Tranversal Hy (kN)	Tranversal Mx (kN.m)
Service Ser-I	632	412	285	0	0
Strength Str-IA	791	678	520	0	0
Strength Str-IB	569	576	473	0	0
Extreme Ext-I	791	413	191	4	4

1.2 Section 2-2
Table of Load Factors

Loads	Sign	Service I Ser-I	Strength I Str-IA	Strength I Str-IB	Extreme I Ext-I
Selfweight	DC	1.00	1.25	0.90	1.25
Superstructure Dead Load	DC	1.00	1.25	0.90	1.25
Pavement	DW	1.00	1.50	0.65	1.50
Handrail+curb	DC	1.00	1.25	0.90	1.25
Live Load	LL	1.00	1.75	1.75	0.50
Horizontal Earth Pressure	EH	1.00	1.50	0.90	
Surchage (Horizontal)	LS_h	1.00	1.75	1.75	0.50
TU+SH&CR	TU+SH&CR	1.00	0.50	0.50	
Horizontal Seismic Earth Pressure	E_{AE}				1.50
Abutment earthquake force	EQ				1.00

Unfactored Loads	N (kN)	Longitudinal Hx (kN)	Longitudinal My (kN.m)	Tranversal Hy (kN)	Tranversal Mx (kN.m)
Selfweight	3522		-202		
Superstructure Dead Load	2532		506		
Pavement	288		58		
Handrail+curb	320		64		
Live Load	1234		247		1697
Horizontal Earth Pressure		2429	7752		
Surchage (Horizontal)		403	1606		
TU+SH&CR		320	1876		
Horizontal Seismic Earth Pressure		2649	6867		
Abutment earthquake force		117	400	69	323

Load Combination at bottom of stem wall

Load combinations	N (kN)	Longitudinal Hx (kN)	Longitudinal My (kN.m)	Tranversal Hy (kN)	Tranversal Mx (kN.m)
Service Ser-I	7895	3152	11906	0	1697
Strength Str-IA	10558	4508	16354	0	2970
Strength Str-IB	8083	3051	11525	0	2970
Extreme Ext-I	9016	4292	12173	69	1171

1.3 Section 3-3

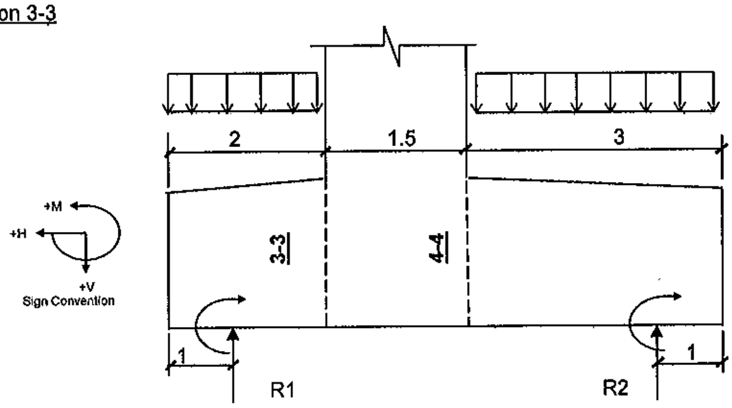


Table of Load Factors

Loads	Sign	Service I Ser-I	Strength I Str-IA	Strength I Str-IB	Extreme I Ext-I
Selfweight at front side	DC	1.00	1.25	0.90	1.25
Vertical soil on foot at front side	DC	1.00	1.35	0.90	1.35
Reaction of piles	RE	1.00	1.00	1.00	1.00

Unfactored Loads		N (kN)	Longitudinal Hx (kN)	Longitudinal My (kN.m)	Tranversal Hy (kN)	Tranversal Mx (kN.m)
Selfweight at front side		1314		1314		
Vertical soil on foot at front side		338		338		
Reaction of piles	Ser-I	-12313	-2394	-7685	38	21
	Str-IA	-17221	-3481	-10504	95	99
	Str-IB	-12067	-2332	-7583	64	78
	Ext-I	-12897	-2388	-7961	-45	-20

Load Combination at section 3-3

Load combinations	N (kN)	Longitudinal Hx (kN)	Longitudinal My (kN.m)	Tranversal Hy (kN)	Tranversal Mx (kN.m)
Service Ser-I	-10661	-2394	-6033	38	21
Strength Str-IA	-15122	-3481	-8406	95	99
Strength Str-IB	-10581	-2332	-6096	64	78
Extreme Ext-I	-10798	-2388	-5862	-45	-20

1.4 Section 4-4

Table of Load Factors

Loads	Sign	Service I	Strength I		Extreme I
		Ser-I	Str-IA	Str-IB	Ext-I
Selfweight at behind side	DC	1.00	1.25	0.90	1.25
Vertical soil on foot at behind side	DC	1.00	1.35	0.90	1.35
Surcharge(Vertical)	EV	1.00	1.75	1.75	0.50
Reaction of piles	RE	1.00	1.00	1.00	1.00

Unfactored Loads		N	Longitudinal		Tranversal	
			Hx	My	Hy	Mx
		(kN)	(kN)	(kN.m)	(kN)	(kN.m)
Selfweight of behind side		2432		-3916		
Vertical soil on foot at behind side		5561		-8342		
Surcharge(Vertical)		442		-663		
Reaction of piles	Ser-I	-4472	-1795	12414	-87	-230
	Str-IA	-5843	-2611	16723	-95	-281
	Str-IB	-4168	-1749	11700	-64	-179
	Ext-I	-8067	-1791	19837	-151	-266

Load Combination at section 4-4					
		Longitudinal		Tranversal	
Load combinations	N	Hx	My	Hy	Mx
	(kN)	(kN)	(kN.m)	(kN)	(kN.m)
Service Ser-I	3964	-1795	-506	-87	-230
Strength Str-IA	5478	-2611	-593	-95	-281
Strength Str-IB	3799	-1749	-492	-64	-179
Extreme Ext-I	2702	-1791	3349	-151	-266

1.4 Section 5-5 & 6-6

Slope of triang pressure	tgβ	=	5.35
Uniform horizontal pressure	U.p	=	3.26 kN/m2

Load Combination at section 5-5					
		Horizontal		Vertical	
Load combinations	N	Hx	Mx	Hy	My
	(kN)	(kN)	(kN.m)	(kN)	(kN.m)
Service Ser-I		74	163		
Strength Str-IA		110	259		

Load Combination at section 6-6					
		Horizontal		Vertical	
Load combinations	N	Hx	Mx	Hy	My
	(kN)	(kN)	(kN.m)	(kN)	(kN.m)
Service Ser-I				203	179
Strength Str-IA				299	264

2. Elements Checking

2.1. Bearing Resistance

<S.5.7.5>

The case of absence of confinement reinforcement In the conrete supporting the bearing device
Factored bearing resistance shall be taken

Pr = ϕ. Pn = ϕ. 0.85. f'c. A1. m

Dimension of bearing plate	w0	=	0.600 m	
	b0	=	0.700 m	
Area under bearing device	A1	=	0.420 m2	
Distributed width and length	w	=	1.000 m	
	b	=	1.100 m	
Notational area	A2	=	1.100 m2	

Where supporting surface is wider on all sides than loades area

m = sqrt(A2/A1) ≤ 2.0 case 1

where loaded area is subjected to nouniformly distributed bearing

m = 0.75*sqrt(A2/A1) ≤ 1.5 case 2

Modification factor case 1	m	=	1.618	
Resistance factor	ϕ	=	0.700	<S.5.5.4.2>
Factored bearing resistance	Pr	=	12133 kN	> Pu
Bearing reaction of approach bridge	Pu	=	4591 kN	Ok

Pu = 1.25*DC + 1.5*DW + 1.75*LL

In case factored applied load exceeds the factored resistance,
provision shall be made to resist the bursting and spalling force in article 5.10.9

<S.5.10.9.7.2>

Factored bearing resistance shall be taken

Pr = ϕ. fn. Ab

fn take the lesser of

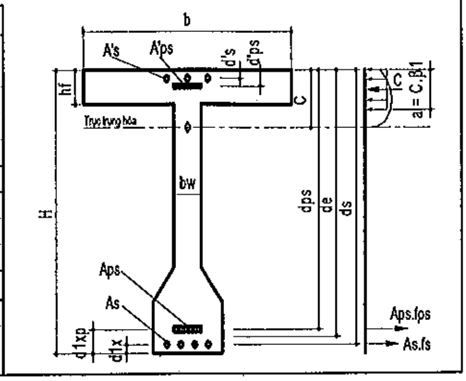
fn = 0.7. f'cl. sqrt(A/Ag) and

fn = 2.25. f'ci

	fn	=	33.99 MPa	
Maximum area of the portion of supporting surface	A	=	1.100 m2	
Gross area of bearing plate	Ag	=	0.420 m2	
Effective net area of bearing plate, Ag minus stud of bearing	Ab	=	0.420 m2	
Nominal concrete strength at time of application	f'ci	=	30 MPa	
Factored bearing resistance	Pr	=	9992 kN	Ok

REINFORCEMENT CHECKING - HEAD AND STEM WALL

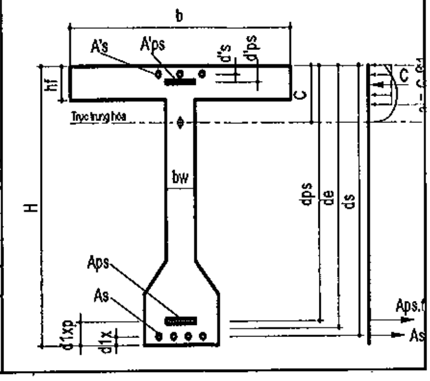
MATERIALS			
NORMAL CONCRETE			
f'c	Compressive Strength of concrete at 28 days	Mpa	30
Ec	Modulus of Elasticity	Mpa	27691
fr	Modulus of Rupture	Mpa	3.5
gc	Unit weight of concrete	kN/m3	24.5
PRESTRESSING STEEL			
fpu	Tensile strength of prestressing steel	Mpa	1860
fpy	Yield strength of prestressing steel	Mpa	1670
Ep	Modulus of Elasticity	Mpa	195000
REINFORCEMENT			
fy	Yield strength	Mpa	400
Es	Modulus of Elasticity	Mpa	200000
nc	Ratio Es/Ec		7



Sign	Parameters		Unit	Sections 1-1	1-1	2-2	2-2	2-2
INTERNAL FORCES AT SECTIOIN								
	Combination			Strength	Service	Service	Strength	Extreme
Qu	Shear		kN	678	412	3152	4508	4292
Mu	Flexural Moment		kNm	520	285	11906	16354	12173
Nu	Axial load		kN	791	632	7895	10558	9016
Tu	Torsional Moment		kNm	0	0	0	0	0
FLEXURAL MOMENT CHECKING								
H	Section height		m	0.500	0.500	1.500	1.500	1.500
d's	Dis. From comp. fiber to centroid of comp. Reinf		m	0.058	0.058	0.058	0.058	0.058
d1x	Dis. From tens. fiber to centroid of tension Reinf		m	0.058	0.058	0.059	0.059	0.059
	Cover to reinf		m	0.050	0.050	0.050	0.050	0.050
ds	Dis. From comp. fiber to centroid of tension Reinf		m	0.442	0.442	1.441	1.441	1.441
d'ps	Dis. From comp. fiber to centroid of comp. prestressing steel		m	0.000	0.000	0.000	0.000	0.000
d1xp	Dis. From tens. fiber to centroid of tension prestressing steel		m	0.000	0.000	0.000	0.000	0.000
dps	Dis. From comp. fiber to centroid of tension prestressing steel		m	0.500	0.500	1.500	1.500	1.500
b	Width of the compression face of member		m	13.409	13.409	13.409	13.409	13.409
bw	Web width or diameter of a circular section		m	13.409	13.409	13.409	13.409	13.409
hf	Compression flange depth		m	0.000	0.000	0.000	0.000	0.000
Iz	Moment of inertia of section		m4	0.140	0.140	3.771	3.771	3.771
Amc	Section area		m2	6.704	6.704	20.113	20.113	20.113
	Steel choice							
Aps	Tension prestressing steel	P.S type		0	0	0	0	0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
A'ps	Compression prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	0 T00.0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
As	Tension Reinforcement	Number	bars	83	83	154	154	154
		Diameter	mm	16	16	20	20	20
		Area	m2	0.01677	0.01677	0.04836	0.04836	0.04836
A's	Compression Reinforcement	Number	bars	83	83	77	77	77
		Diameter	mm	16	16	16	16	16
		Area	m2	0.01677	0.01677	0.01555	0.01555	0.01555
A'c	Shear reinforcement	Number	bars	21	21	20	20	20
		Diameter	mm	14	14	14	14	14
		Area	m2	0.00317	0.00317	0.00302	0.00302	0.00302
φ	Resistance factors for flexure		5.5.4.2	0.90	1.00	1.00	0.90	1.00
φv	Resistance factors for shear			0.90	1.00	1.00	0.90	1.00
φn	Resistance factors for axial force			1.00	1.00	1.00	1.00	1.00
β1	Stress block factor			0.836	0.836	0.836	0.836	0.836
c	Dis. Between centroid and top fiber		m	0.000	0.000	0.046	0.046	0.046
	For T section behavior		m	0.000	0.000	0.046	0.046	0.046
	For rectangular section behavior		m	0.000	0.000	0.046	0.046	0.046
fpe	Effective stress in the prestressing steel after losses		Mpa	1116	1116	1116	1116	1116
fps	Aver. stress in pres. steel at the time for which the nominal resistanc		Mpa	1860	1860	1844	1844	1844
k	Factor depends on type of P.S, Low relaxation strand k = 0.28			0.28	0.28	0.28	0.28	0.28
a	Depth of equivalent stress block		m	0.000	0.000	0.038	0.038	0.038
de	Corresp. effective depth from extreme comp. fiber							
	to centroid of tensile force in the tensile reinf.		m	0.442	0.442	1.441	1.441	1.441
Mn	Nominal resistance		kNm	2575	2575	27260	27260	27260
Mr	Factored reistance		kNm	**2318**	**2575**	**27260**	**24534**	**27260**
Mu	Flexual moment		kNm	520	285	11906	16354	12173

(5.7.3.2)	**Flexural moment Checking**		OK	OK	OK	OK	OK
	Limits for reinforcement						
c/de	Maximum reinforcement		0.00	0.00	0.03	0.03	0.03
	Maximum reinforcement Checking	<= 0.42	OK	OK	OK	OK	OK
1.2*Mcr	Craking moment	kNm	1157	1157	10739	10739	10739
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**		OK	OK	OK	OK	OK
(5.7.3.4)	**Conctrol of craking by distr. of reinf for RC member- Check?**		No	Yes	Yes	No	No
	Existing condition for structrure	1,2 or 3	1	1	1	1	1
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.058	0.058	0.059	0.059	0.059
Z	Crack width parameter	N/mm	30000	30000	30000	30000	30000
A	Area of concr. with same centroid as tens. Reinf	m2	0.019	0.019	0.010	0.010	0.010
fsa	Value	Mpa	292	292	354	354	354
0.6*fy		Mpa	240	240	240	240	240
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	240	240	240	240	240
x	Dist. From compression fiber to centroid	m	-	0.08	0.246	-	-
J.d	Arm	m	-	0.415	1.359	-	-
Icr	Moment of inertia of the cracked section	m4	-	0.018	0.553	-	-
fs	Tensile stress in reinforcement fs = Msls / (As*J.d)	Mpa	-	41	181	-	-
	Checking for control cracking fs<fsa		N.a	OK	OK	N.a	N.a
(5.10.8.2)	**Shrinkage and temperature Reinfocement (side distribution)**						
Areq	Area of required reinf.	m2	0.00045	0.00045	0.00126	0.00126	0.00126
	Distribution on sides 7 D16	m2	0.00141	0.00141	0.00141	0.00141	
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45	0.45
	Checking		OK	OK	OK	OK	OK
	SHEAR AND TORSION CHECKING						
β	Factor indicating diag. cracked concr. to tension		2.8	3.6	2.4	2.2	2.3
θ	Angle of inclination of diagonal compressive	degree	28.83	28.05	33.04	36.45	33.59
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	90
bv	Effective web width as minimum web width - in dv	m	13.409	13.409	13.409	13.409	13.409
dv	Effective shear depth	m	0.442	0.442	1.422	1.422	1.422
	(de - a/2)	m	0.442	0.442	1.422	1.422	1.422
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	0.600
ncat	Amount of bars in spacing S	bars	21	21	20	20	20
Av	Shear reinf area in spacing S	m2	0.0032	0.0032	0.0030	0.0030	0.0030
θ	Assume	degree	28.87	28.19	28.88	30.16	30.27
v	Shear stress in concrete	kN/m2	127	69	165	263	225
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e_x	Strain in tensile reinforcement		4.16E-04	2.13E-04	7.53E-04	1.04E-03	7.99E-04
	if ex<0, multiple with reduce factor		-	-	-	-	-
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok	Ok
v/fc	Ratio of shear stress and fc		0.004	0.002	0.006	0.009	0.008
β	Final value		2.8	3.6	2.4	2.2	2.3
θ	Final value	degree	28.83	28.05	33.04	36.45	33.59
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	7646	9805	20526	19111	20301
Vs	Shear resistance provided by shear reinforcement	kN	1697	1753	4402	3876	4310
Vp	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0	0
Vn1	Vn1=Vc+Vs+Vp	kN	9343	11559	24928	22987	24611
Vn2	Vn2	kN	44450	44450	142984	142984	142984
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	9343	11559	24928	22987	24611
Vr	Factored shear resistance	kN	**8409**	**11559**	**24928**	**20689**	**24611**
Vu	Shear	kN	678	412	3152	4508	4292
(5.8.2.7)	**Shear checking**		OK	OK	OK	OK	OK
	Region requiring transverse reinf Checking		No need	No need	No need	No need	No need
	Minimum shear reinf area	m2	0.0091	0.0091	0.0091	0.0091	0.0091
	Mimimum shear reinforcement Checking		-	-	-	-	-
	0.1*fc*bv*dv	kN	17780	17780	57194	57194	57194
	Smax	m	0.35	0.35	0.60	0.60	0.60
	Maximum spacing Smax		-	-	-	-	-

MATERIALS



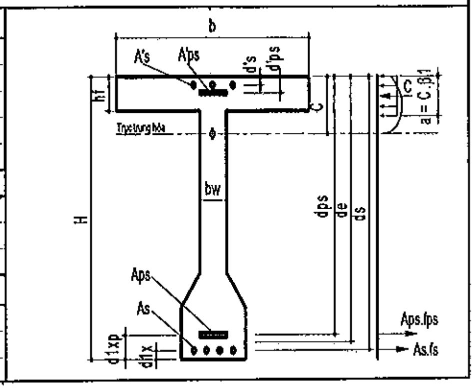
	NORMAL CONCRETE		
fc	Compressive Strength of concrete at 28 days	Mpa	30
Ec	Modulus of Elasticity	Mpa	27691
fr	Modulus of Rupture	Mpa	3.5
gc	Unit weight of concrete	kN/m3	24.5
	PRESTRESSING STEEL		
fpu	Tensile strength of prestressing steel	Mpa	1860
fpy	Yield strength of prestressing steel	Mpa	1670
Ep	Modulus of Elasticity	Mpa	195000
	REINFORCEMENT		
fy	Yield strength	Mpa	400
Es	Modulus of Elasticity	Mpa	200000
nc	Ratio Es/Ec		7

Sign	Parameters		Unit	Sections 3-3	3-3	3-3	4-4	4-4
	INTERNAL FORCES AT SECTIOIN							
	Combination			Service	Strength	Extreme	Extreme	Strength
Qu	Shear		kN	10661	15122	10798	2702	5478
Mu	Flexural Moment		kNm	6033	8406	5862	3349	593
Nu	Axial load		kN	2394	3481	2388	1791	2611
Tu	Torsional Moment		kNm	0	0	0	0	0
	FLEXURAL MOMENT CHECKING							
H	Section height		m	2.000	2.000	2.000	2.000	2.000
d's	Dis. From comp. fiber to centroid of comp. Reinf		m	0.084	0.084	0.084	0.161	0.161
d1x	Dis. From tens. fiber to centroid of tension Reinf		m	0.161	0.161	0.161	0.084	0.084
	Cover to reinf		m	0.050	0.050	0.050	0.050	0.050
ds	Dis. From comp. fiber to centroid of tension Reinf		m	1.839	1.839	1.839	1.916	1.916
d'ps	Dis. From comp. fiber to centroid of comp. prestressing steel		m	0.000	0.000	0.000	0.000	0.000
d1xp	Dis. From tens. fiber to centroid of tension prestressing steel		m	0.000	0.000	0.000	0.000	0.000
dps	Dis. From comp. fiber to centroid of tension prestressing steel		m	2.000	2.000	2.000	2.000	2.000
b	Width of the compression face of member		m	13.409	13.409	13.409	13.409	13.409
bw	Web width or diameter of a circular section		m	13.409	13.409	13.409	13.409	13.409
hf	Compression flange depth		m	0.000	0.000	0.000	0.000	0.000
Iz	Moment of inertia of section		m4	8.939	8.939	8.939	8.939	8.939
Amc	Section area		m2	26.817	26.817	26.817	26.817	26.817
	Steel choice							
Aps	Tension prestressing steel	P.S type		0	0	0	0	0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
A'ps	Compression prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	0 T00.0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
As	Tension Reinforcement	Number	bars	84	84	84	84	84
		Diameter	mm	22	22	22	20	20
		Area	m2	0.03192	0.03192	0.03192	0.02638	0.02638
A's	Compression Reinforcement	Number	bars	0	0	0	0	0
		Diameter	mm	20	20	20	22	22
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
A'c	Shear reinforcement	Number	bars	21	21	21	21	21
		Diameter	mm	16	16	16	16	16
		Area	m2	0.00424	0.00424	0.00424	0.00424	0.00424
ϕ	Resistance factors for flexure		5.5.4.2	1.00	0.90	1.00	1.00	0.90
ϕv	Resistance factors for shear			1.00	0.90	1.00	1.00	0.90
ϕn	Resistance factors for axial force			1.00	1.00	1.00	1.00	1.00
β1	Stress block factor			0.836	0.836	0.836	0.836	0.836
c	Dis. Between centroid and top fiber		m	0.045	0.045	0.045	0.037	0.037
	For T section behavior		m	0.045	0.045	0.045	0.037	0.037
	For rectangular section behavior		m	0.045	0.045	0.045	0.037	0.037
fpe	Effective stress in the prestressing steel after losses		Mpa	1116	1116	1116	1116	1116
fps	Aver. stress in pres. steel at the time for which the nominal resistance		Mpa	1848	1848	1848	1850	1850
k	Factor depends on type of P.S, Low relaxation strand k = 0.28			0.28	0.28	0.28	0.28	0.28
a	Depth of equivalent stress block		m	0.037	0.037	0.037	0.031	0.031
de	Corresp. effective depth from extreme comp. fiber							
	to centroid of tensile force in the tensile reinf.		m	1.839	1.839	1.839	1.916	1.916
Mn	Nominal resistance		kNm	23242	23242	23242	20052	20052
Mr	Factored reistance		kNm	**23242**	**20918**	**23242**	**20052**	**18047**
Mu	Flexual moment		kNm	6033	8406	5862	3349	593

(5.7.3.2)	**Flexural moment Checking**		OK	OK	OK	OK	OK
	Limits for reinforcement						
c/de	Maximum reinforcement		0.02	0.02	0.02	0.02	0.02
	Maximum reinforcement Checking	<= 0.42	OK	OK	OK	OK	OK
1.2*Mcr	Craking moment	kNm	18930	18930	18930	18856	18856
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**		OK	OK	OK	OK	OK
(5.7.3.4)	**Conctrol of craking by distr. of reinf for RC member- Check?**		Yes	No	No	No	No
	Existing condition for structrure	1,2 or 3	3	3	3	3	3
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.061	0.061	0.061	0.060	0.060
Z	Crack width parameter	N/mm	17500	17500	17500	17500	17500
A	Area of concr. with same centroid as tens. Reinf	m2	0.019	0.019	0.019	0.019	0.019
fsa	Value	Mpa	165	165	165	167	167
0.6*fy		Mpa	240	240	240	240	240
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	165	165	165	167	167
x	Dist. From compression fiber to centroid	m	0.231	-	-	-	-
J.d	Arm	m	1.762	-	-	-	-
Icr	Moment of inertia of the cracked section	m4	0.633	-	-	-	-
fs	Tensile stress in reinforcement fs = Msls / (As*J.d)	Mpa	107	-	-	-	-
	Checking for control cracking fs<fsa		OK	N.a	N.a	N.a	N.a
(5.10.8.2)	**Shrinkage and temperature Reinfocement (side distribution)**						
Areq	Area of required reinf	m2	0.00127	0.00127	0.00127	0.00127	0.00127
	Distribution on sides 7 D16	m2	0.00141	0.00141	0.00141	0.00141	0.00141
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45	0.45
	Checking		OK	OK	OK	OK	OK
	SHEAR AND TORSION CHECKING						
β	Factor indicating diag. cracked concr. to tension		2.0	1.8	2.0	2.5	2.5
θ	Angle of inclination of diagonal compressive	degree	39.92	42.17	39.37	30.48	30.04
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	90
bv	Effective web width as minimum web width - in dv	m	13.409	13.409	13.409	13.409	13.409
dv	Effective shear depth	m	1.820	1.820	1.820	1.901	1.901
	(de - a/2)	m	1.820	1.820	1.820	1.901	1.901
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	0.600
ncat	Amount of bars in spacing S	bars	21	21	21	21	21
Av	Shear reinf area in spacing S	m2	0.0042	0.0042	0.0042	0.0042	0.0042
θ	Assume	degree	38.22	41.43	39.67	30.88	34.58
v	Shear stress in concrete	kN/m2	437	688	442	33	239
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e_x	Strain in tensile reinforcement		1.39E-03	1.79E-03	1.34E-03	5.92E-04	5.65E-04
	if ex<0, multiple with reduce factor		-	-	-	-	-
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok	Ok
v/f'c	Ratio of shear stress and f'c		0.015	0.023	0.015	0.001	0.008
β	Final value		2.0	1.8	2.0	2.5	2.5
θ	Final value	degree	39.92	42.17	39.37	30.48	30.04
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	22309	20144	22650	28479	28658
Vs	Shear resistance provided by shear reinforcement	kN	6152	5683	6274	9132	9295
Vp	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0	0
Vn1	Vn1=Vc+Vs+Vp	kN	28461	25827	28923	37611	37952
Vn2	Vn2	kN	183061	183061	183061	191131	191131
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	28461	25827	28923	37611	37952
Vr	Factored shear resistance	kN	**28461**	**23244**	**28923**	**37611**	**34157**
Vu	Shear	kN	10661	15122	10798	2702	5478
(5.8.2.7)	**Shear checking**		OK	OK	OK	OK	OK

MATERIALS

Sign	Parameter	Unit	Value
	NORMAL CONCRETE		
fc	Compressive Strength of concrete at 28 days	Mpa	30
Ec	Modulus of Elasticity	Mpa	27691
fr	Modulus of Rupture	Mpa	3.5
gc	Unit weight of concrete	kN/m3	24.5
	PRESTRESSING STEEL		
fpu	Tensile strength of prestressing steel	Mpa	1860
fpy	Yield strength of prestressing steel	Mpa	1670
Ep	Modulus of Elasticity	Mpa	195000
	REINFORCEMENT		
fy	Yield strength	Mpa	400
Es	Modulus of Elasticity	Mpa	200000
nc	Ratio Es/Ec		7



Sign	Parameters		Unit	Sections 5-5	5-5	6-6	6-6	6-6
	INTERNAL FORCES AT SECTIOIN							
	Combination			Service	Strength	Service	Strength	Strength
Qu	Shear		kN	74	110	203	299	299
Mu	Flexural Moment		kNm	163	259	179	264	264
Nu	Axial load		kN	0	0	0	0	0
Tu	Torsional Moment		kNm	0	0	0	0	0
	FLEXURAL MOMENT CHECKING							
H	Section height		m	0.500	0.500	0.500	0.500	0.500
d's	Dis. From comp. fiber to centroid of comp. Reinf		m	0.058	0.058	0.058	0.058	0.058
d1x	Dis. From tens. fiber to centroid of tension Reinf		m	0.059	0.059	0.059	0.059	0.059
	Cover to reinf		m	0.050	0.050	0.050	0.050	0.050
ds	Dis. From comp. fiber to centroid of tension Reinf		m	0.441	0.441	0.441	0.441	0.441
d'ps	Dis. From comp. fiber to centroid of comp. prestressing steel		m	0.000	0.000	0.000	0.000	0.000
d1xp	Dis. From tens. fiber to centroid of tension prestressing steel		m	0.000	0.000	0.000	0.000	0.000
dps	Dis. From comp. fiber to centroid of tension prestressing steel		m	0.500	0.500	0.500	0.500	0.500
b	Width of the compression face of member		m	1.000	1.000	1.000	1.000	1.000
bw	Web width or diameter of a circular section		m	1.000	1.000	1.000	1.000	1.000
hf	Compression flange depth		m	0.000	0.000	0.000	0.000	0.000
Iz	Moment of inertia of section		m4	0.010	0.010	0.010	0.010	0.010
Amc	Section area		m2	0.500	0.500	0.500	0.500	0.500
	Steel choice							
Aps	Tension prestressing steel	P.S type		0	0	0	0	0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
A'ps	Compression prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	0 T00.0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
As	Tension Reinforcement	Number	bars	7	7	7	7	7
		Diameter	mm	20	20	20	20	20
		Area	m2	0.00220	0.00220	0.00220	0.00220	0.00220
A's	Compression Reinforcement	Number	bars	7	7	7	7	7
		Diameter	mm	16	16	16	16	16
		Area	m2	0.00141	0.00141	0.00141	0.00141	0.00141
A'c	Shear reinforcement	Number	bars	3	3	3	3	3
		Diameter	mm	12	12	12	12	12
		Area	m2	0.00034	0.00034	0.00034	0.00034	0.00034
ϕ	Resistance factors for flexure		5.5.4.2	1.00	0.90	1.00	0.90	0.90
ϕv	Resistance factors for shear			1.00	0.90	1.00	0.90	0.90
ϕn	Resistance factors for axial force			1.00	1.00	1.00	1.00	1.00
$\beta 1$	Stress block factor			0.836	0.836	0.836	0.836	0.836
c	Dis. Between centroid and top fiber		m	0.015	0.015	0.015	0.015	0.015
	For T section behavior		m	0.015	0.015	0.015	0.015	0.015
	For rectangular section behavior		m	0.015	0.015	0.015	0.015	0.015
fpe	Effective stress in the prestressing steel after losses		Mpa	1116	1116	1116	1116	1116
fps	Aver. stress in pres. steel at the time for which the nominal resistance		Mpa	1844	1844	1844	1844	1844
k	Factor depends on type of P.S, Low relaxation strand k = 0.28			0.28	0.28	0.28	0.28	0.28

a	Depth of equivalent stress block	m	0.012	0.012	0.012	0.012	0.012
de	Corresp. effective depth from extreme comp. fiber						
	to centroid of tensile force in the tensile reinf.	m	0.441	0.441	0.441	0.441	0.441
Mn	Nominal resistance	kNm	353	353	353	353	353
Mr	Factored reistance	kNm	**353**	**318**	**353**	**318**	**318**
Mu	Flexual moment	kNm	163	259	179	264	264
(5.7.3.2)	**Flexural moment Checking**		**OK**	**OK**	**OK**	**OK**	**OK**
	Limits for reinforcement						
c/de	Maximum reinforcement		0.03	0.03	0.03	0.03	0.03
	Maximum reinforcement Checking	<= 0.42	OK	OK	OK	OK	OK
r min	Mimimum reinforcement		0.44%	0.44%	0.44%	0.44%	0.44%
	Mimimum reinforcement Checking for RC	0.23%	OK	OK	OK	OK	OK
1.2*Mcr	Craking moment	kNm	89	89	89	89	89
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**		OK	OK	OK	OK	OK
(5.7.3.4)	**Conctrol of craking by distr. of reinf for RC member- Check?**		Yes	No	Yes	No	No
	Existing condition for structrure	1,2 or 3	1	1	1	1	1
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.059	0.059	0.059	0.059	0.059
Z	Crack width parameter	N/mm	30000	30000	30000	30000	30000
A	Area of concr. with same centroid as tens. Reinf	m2	0.017	0.017	0.017	0.017	0.017
fsa	Value	Mpa	301	301	301	301	301
0.6*fy		Mpa	240	240	240	240	240
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	240	240	240	240	240
x	Dist. From compression fiber to centroid	m	0.102	-	0.102	-	-
J.d	Arm	m	0.407	-	0.407	-	-
Icr	Moment of inertia of the cracked section	m4	0.002	-	0.002	-	-
fs	Tensile stress in reinforcement fs = Msls / (As*J.d)	Mpa	183	-	200	-	-
	Checking for control cracking fs<fsa		**OK**	**N.a**	**OK**	**N.a**	**N.a**
(5.10.8.2)	**Shrinkage and temperature Reinfocement (side distribution)**						
Areq	Area of required reinf	m2	0.00031	0.00031	0.00031	0.00031	0.00031
	Distribution on sides 7 D16	m2	0.00141	0.00141	0.00141	0.00141	0.00141
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45	0.45
	Checking		**OK**	**OK**	**OK**	**OK**	**OK**
	SHEAR AND TORSION CHECKING						
β	Factor indicating diag. cracked concr. to tension		2.2	2.0	2.1	1.8	1.8
θ	Angle of inclination of diagonal compressive	degree	35.64	40.99	38.29	42.02	42.02
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	90
bv	Effective web width as minimum web width - in dv	m	1.000	1.000	1.000	1.000	1.000
dv	Effective shear depth	m	0.435	0.435	0.435	0.435	0.435
	(de - a/2)	m	0.435	0.435	0.435	0.435	0.435
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	0.600
ncat	Amount of bars in spacing S	bars	3	3	3	3	3
Av	Shear reinf area in spacing S	m2	0.0003	0.0003	0.0003	0.0003	0.0003
β	Assume		2.0	2.0	2.0	2.0	2.0
θ	Assume	degree	36.15	41.31	38.30	42.39	42.39
v	Shear stress in concrete	kN/m2	170	282	467	765	765
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e_x	Strain in tensile reinforcement		9.70E-04	1.50E-03	1.23E-03	1.75E-03	1.75E-03
	if ex<0, multiple with reduce factor		-	-	-	-	-
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok	Ok
v/f'c	Ratio of shear stress and f'c		0.006	0.009	0.016	0.025	0.025
β	Final value		2.2	2.0	2.1	1.8	1.8
θ	Final value	degree	35.64	40.99	38.29	42.02	42.02
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	444	386	416	362	362
Vs	Shear resistance provided by shear reinforcement	kN	137	113	125	109	109
Vp	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0	0
Vn1	Vn1=Vc+Vs+Vp	kN	581	499	540	472	472
Vn2	Vn2	kN	3261	3261	3261	3261	3261
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	581	499	540	472	472
Vr	Factored shear resistance	kN	**581**	449	**540**	424	424
Vu	Shear	kN	74	110	203	299	299
(5.8.2.7)	**Shear checking**		**OK**	**OK**	**OK**	**OK**	**OK**

SPACE PILE FOUNDATION ANALYSIS PROGRM
Turbo BASIÇ

PROJECT: E.X Da Nang - Quang Ngai
Bridge: CB12 - KM45+540.00

INITIA DATA

Kn = 0.00 Ax = 6.50 By = 12.60 Cz = 2.00
E v.uon = 2944008 E r.uon = 2944008 E v.nen = 2944008 E r.nen = 2944008
Mq = 0 (t/m4) Md = 0 (t/m4) m = 1529.052001953125 (t/m4)

LOAD COMBINATIONS

COMB.	Hx	Hy	P	Mx	My	Mz
1	621.00	0.00	2351.00	-380.00	1724.00	0.00
2	416.00	0.00	1655.00	-358.00	1262.00	0.00
3	600.00	6.00	2283.00	-365.00	1587.00	0.00
4	395.00	6.00	1586.00	-343.00	1125.00	0.00
5	427.00	5.00	1711.00	-280.00	1198.00	0.00
6	426.00	20.00	2137.00	-278.00	410.00	0.00
7	426.00	20.00	1441.00	-257.00	768.00	0.00

PROPERTIES OF PILES

PILE	Lo	H	Bpx	Bpy	A	B	Cday	Fo	Io	Po	Co	Ct
1	0.00	8.00	1.422	1.422	1.00	1.000	0.000	1.000	0.098	0	150000	75000
2					n	t						
3					n	t						
4					n	t						
5					n	t						
6					n	t						
7					n	t						

PILE COORD.

PILE	X	Y	Phi	Xi
1	4.09	4.66	0.000	0.00
2	2.88	1.34	0.000	0.00
3	1.67	-1.98	0.000	0.00
4	0.47	-5.30	0.000	0.00
5	-4.32	-5.30	0.000	0.00
6	-2.51	-0.32	0.000	0.00
7	-0.70	4.66	0.000	0.00

FORCES ON PILES

PILE	COMB.	N	Q2	Q3	M1	M2	M3
1	1	493.59	-94.59	4.56	1.758	7.097	183.711
	2	350.23	-63.37	3.06	1.177	5.060	122.694
	3	474.91	-91.43	3.58	1.710	5.407	178.329
	4	331.41	-60.21	2.08	1.130	3.367	117.308
	5	354.16	-65.08	2.45	1.219	3.704	126.649
	6	375.79	-65.02	0.38	1.246	2.737	134.683
	7	294.93	-65.02	0.38	1.246	0.069	129.499
2	1	457.10	-90.67	3.14	1.758	4.051	175.343
	2	321.76	-60.74	2.10	1.177	3.019	117.088
	3	439.27	-87.62	2.19	1.710	2.443	170.186
	4	303.80	-57.69	1.16	1.130	1.409	111.928

	5	327.25	-62.36	1.46	1.219	1.592	120.847
	6	344.37	-62.25	-0.63	1.246	0.579	128.753
	7	270.64	-62.25	-0.63	1.246	-2.089	123.569
3	1	420.61	-86.75	1.71	1.758	1.006	166.975
	2	293.29	-58.12	1.15	1.177	0.979	111.482
	3	403.63	-83.81	0.81	1.710	-0.520	162.044
	4	276.19	-55.17	0.24	1.130	-0.549	106.549
	5	300.33	-59.64	0.47	1.219	-0.519	115.046
	6	312.95	-59.47	-1.65	1.246	-1.580	122.822
	7	246.36	-59.47	-1.65	1.246	-4.248	117.638
4	1	384.12	-82.83	0.28	1.758	-2.040	158.606
	2	264.82	-55.49	0.19	1.177	-1.061	105.876
	3	367.99	-79.99	-0.58	1.710	-3.484	153.902
	4	248.59	-52.65	-0.68	1.130	-2.507	101.169
	5	273.42	-56.92	-0.52	1.219	-2.631	109.244
	6	281.53	-56.69	-2.66	1.246	-3.738	116.892
	7	222.07	-56.69	-2.66	1.246	-6.406	111.708
5	1	143.80	-82.83	-5.37	1.758	-14.111	158.606
	2	98.93	-55.49	-3.60	1.177	-9.147	105.876
	3	145.61	-79.99	-6.08	1.710	-15.228	153.902
	4	100.59	-52.65	-4.31	1.130	-10.267	101.169
	5	111.57	-56.92	-4.44	1.219	-10.999	109.244
	6	226.99	-56.69	-6.66	1.246	-12.292	116.892
	7	99.24	-56.69	-6.66	1.246	-14.960	111.708
6	1	198.53	-88.71	-3.23	1.758	-9.542	171.159
	2	141.63	-59.43	-2.16	1.177	-6.087	114.285
	3	199.06	-85.71	-4.00	1.710	-10.783	166.115
	4	142.00	-56.43	-2.93	1.130	-7.330	109.238
	5	151.94	-61.00	-2.95	1.219	-7.832	117.947
	6	274.12	-60.86	-5.15	1.246	-9.055	125.787
	7	135.67	-60.86	-5.15	1.246	-11.723	120.603
7	1	253.26	-94.59	-1.09	1.758	-4.973	183.711
	2	184.34	-63.37	-0.73	1.177	-3.026	122.694
	3	252.52	-91.43	-1.92	1.710	-6.338	178.329
	4	183.42	-60.21	-1.56	1.130	-4.393	117.308
	5	192.31	-65.08	-1.47	1.219	-4.664	126.649
	6	321.25	-65.02	-3.63	1.246	-5.817	134.683
	7	172.10	-65.02	-3.63	1.246	-8.485	129.499

SUMMARY OF FORCES

	PILE	COMB.	N	Q2	Q3	M1	M2	M3
Nmin	5	2	98.93	-55.49	-3.60	1.177	-9.147	105.876
Nmax	1	1	493.59	-94.59	4.56	1.758	7.097	183.711
Q2max	1	1	493.59	-94.59	4.56	1.758	7.097	183.711
Q3max	5	6	226.99	-56.69	-6.66	1.246	-12.292	116.892
M1max	1	1	493.59	-94.59	4.56	1.758	7.097	183.711
M2max	5	3	145.61	-79.99	-6.08	1.710	-15.228	153.902
M3max	1	1	493.59	-94.59	4.56	1.758	7.097	183.711

CHECKING CALCULATI
IN COMPARISON WITH INITIA LOAD MATRIX

1	621.00	0.00	2351.00	-380.00	1724.00	0.00
2	416.00	0.00	1655.00	-358.00	1262.00	0.00
3	600.00	6.00	2283.00	-365.00	1587.00	0.00
4	395.00	6.00	1586.00	-343.00	1125.00	0.00
5	427.00	5.00	1711.00	-280.00	1198.00	0.00
6	426.00	20.00	2137.00	-278.00	410.00	0.00
7	426.00	20.00	1441.00	-257.00	768.00	0.00

f.BORED PILE DESIGN

I. BORED PILE DATA

1.Load Combinations at top of bored pile

No	Combinations	Sign	F_V (kN)	Longitudinal F_{HX} (kN)	Longitudinal My (kN•m)	Transvesal F_{HY} (kN)	Transvesal Mx (kN•m)
1	Strength Str-IB		971	544	-1039	35	90
2	Strength Str-IA		4842	928	-1802	-45	-70
3	Strength Str-IA		4842	928	-1802	-45	-70
4	Strength Str-IA		4842	928	-1802	-45	-70
5	Strength Str-IA		4842	928	-1802	-45	-70
6							

2. Bored pile Material

Normal concrete			
Compressive strength at 28 days age	f'c	30	MPa
Concrete elastic modulus	Ec	27691	MPa
Reinforcement			
Yield strength	fy	400	MPa
Reinforcement elastic modulus	Es	200,000	MPa

3. Bored pile Section

Pile diameter	D	1.20	m
Section area	A	1.131	m2
Moment inertia	Ix	0.102	m4
	Iy	0.102	m4
Radius of gyration of gross concrete section; r = sqrt(I/A)	rx	0.300	m
	ry	0.300	m

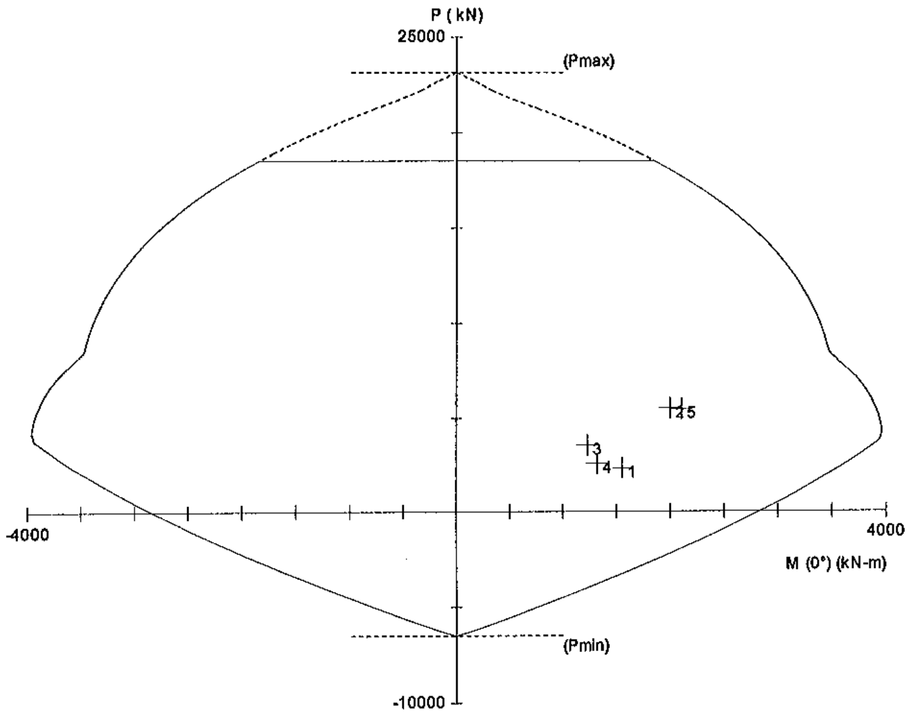
II. PILE DESIGN

1. Limit of Reinforcement

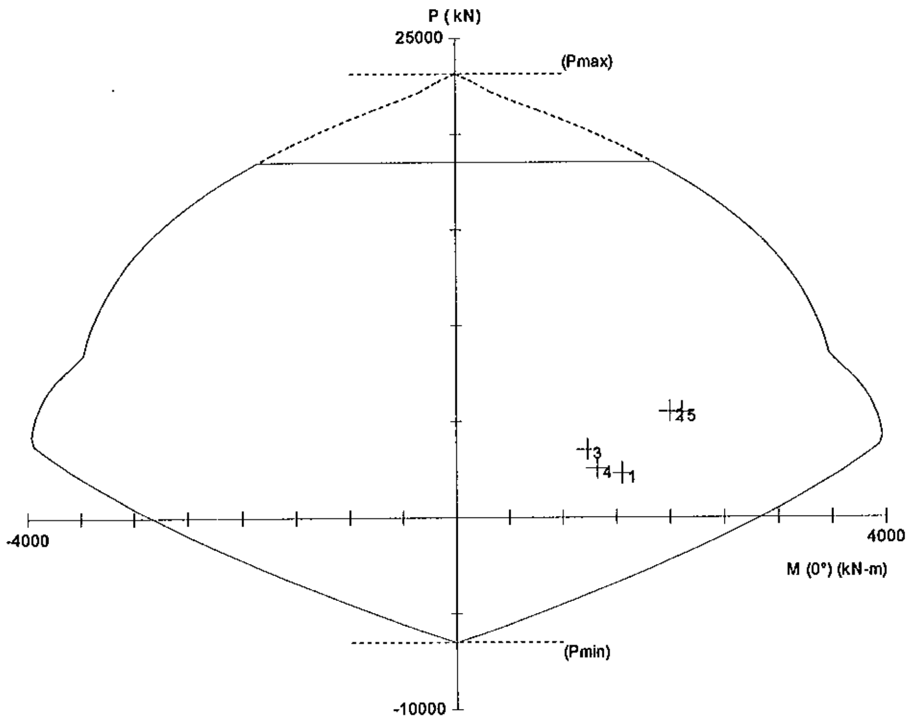
S.5.7.4.2

Minimum area of longitudinal reinforcement in column			
As.fy / (Ag . f'c) >= 0.135	As ≥	0.011	m2
As / Ag >= 0.01	As ≥	0.011	m2
Maximum area of longitudinal reinforcement in column			
As / Ag <= 0.08	As ≤	0.090	m2
Trial Rebars: Ok	As	**0.015**	m2
1 layers x 24 = 24 bars D28 @150	As1	0.015	m2

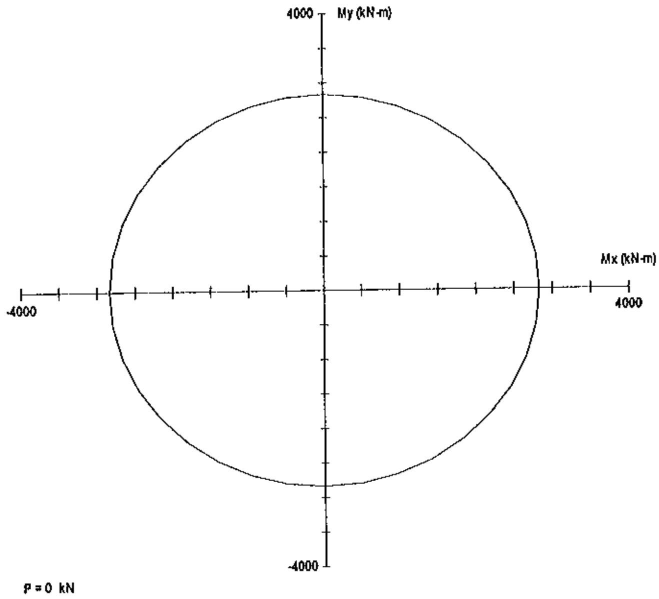
**In Transverse Direction



**In Longitudinal Direction



**In Both Direction



S.5.7.4.6, S.5.10.6.3, S5.10.11.4.1d - e

3. Column Ties

Bridge is in seismic zone	Sz	1	
Area of concrete core measured out-to-out of ties	Ac	0.933	m2
Tie diameter	Dtie	14	mm2
Cross section area of 1 tie	As-tr	0.0002	m2
Spacing of hoops	s	75	mm
Length of reinfocement tie in 1 hoop	Ltie	3.41	m
Ratio of ties reinf. in one hoop/ volume of conc. core one pitch spacing			
ρs = As-tr . Ltie / (Ac * spacing)	ρs	0.0078	
Ratio of spiral reinf. To total volume of concrete core shall satisfy			S.5.7.4.6
ρs >= 0.45 . (Ag/Ac - 1). f'c/ fy = Req1	Req1	0.0072	**OK**
Transverse Reinforcement for Confinement at Plastic Hinges		S.5.10.11.4.1.d	
For a circular column "1:applied","2:Not applied"		1	
ρs >= 0.12 . f'c/ fy = Req2	Req2	0.0090	**N/A**
Length distributed spiral with pitch 75mm below pilecap	Ldis	1.80	m

4. Shear Design

Shear resistance factors	ϕv	1.0	
Factored shear force	Vu	928	kN
Required shear capacity Vn = Vu/ ϕv	Vn	928	kN
Determine concrete shear capacity			
Minimum shear reinforcement will provided in cross section			
Therefore	β	2.0	
	θ	45.0	deg
Diameter of bored pile	D	1.20	m
Width of cross section	b	1.20	m
dv = 0.9*de de = D/2 + Dr/pi()			
Diameter of the circle passing through the centers of the long. reinf.	Dr	0.99	m
	de	0.92	m
	dv	0.82	m
Vc = 0.083 . β . sqrt(f'c) . bv. dv	Vc	900	kN
	A_v	1963	mm2
Angle of inclination of shear reinf. to long. axis	α	90	
Vs=Av*fy*dv*(cotgθ+cotgα)*sinα/s	Vs	8635	KN
Vn1 = Vc + Vs	V_{n1}	9535	
Vn2 = 0.25f'cbvdv	V_{n2}	7423	
	V_n	7423	
	Conclude		**OK**

ASSUMPTION:

- The rock is reasonably sound
- The rock strength measured during site investigation will not deteriorate during construction when water or drilling fluids are used
- The bottom of the socket is properly cleaned out.

DATA & CALCULATION:

Bored hole name		CB12-A1	
Bottom of pilecap elevation	EL1 =	11.00	
Top of socket elevation	EL2 =	3.75	
Pile tip elevation	EL3 =	1.00	
Pile Length	L =	10.00	m
Diameter of drilled-shaft	D_p =	1.00	m
Pile Cross-Sectional Perimeter	P =	3.14	m
Pile Cross-Sectional Area	A_b =	0.79	m^2
Working normal force at pile head	N =	5034.7	kN
Working normal force at top of socket	P_i =	5003.0	kN
Intack rock modulus	E_i =	25000	MPa
Modulus modification ratio	K_e =	0.50	
Elastic modulus of the insitu rock	$E_r = K_e*E_i$ =	12500.0	MPa
Influence coefficient	$I_p = f(H_s/D_s, E_c/E_r)$ =	0.30	
	H_s/D_s =	2.75	
	E_c/E_r =	2.22	

Pile Concrete comp. strength	f'_c =	30.0	MPa
Concrete Unit Weight	g_c =	24.5	kN/m^3
Modulus of elasticity of concre	E_c =	27691	MPa
Depth of socket	H_s =	2.75	m
Diameter of socket	D_s =	1.00	m
Socket Cross-Sect. Perimeter	P_{soc} =	3.14	m
Socket Cross-Sectional Area	A_{soc} =	0.79	m^2

Figure C10.8.3.5-2 Lrfd
Figure C10.8.3.5-3 Lrfd
Figure C10.8.3.5-1 Lrfd

Rock mass modulus/ intack rock modulus	E_m / E_i		*C.10.4.6.5-1-Lrfd 4th*
Atmospheric pressure	p_a =	0.101 MPa	
Reduction factor to account for jointing	α_E		*10.8.3.5.4b-Lrfd 4th*

The elastic shorterning of the drilled shaft

$r_e = (\Sigma P_i)*H_s/(A_{soc}*E_c) =$ 0.633 mm

The settlement of base of drilled shaft

$r_{base} = (\Sigma P_i)*I_p/(D_s*E_r) =$ 0.120 mm

$r_e + r_{base} =$ **0.753** mm < 10 mm

Compute the bearing capacity based on Sharft resistance alone

Unit side resistance

Case 1: The drilling fluid used will not form a lubricated film on the sides of the socket

if qu < 1.9 Mpa - may be taken after Carter & Kulhawy 1988 → qs = 0.15*qu *C10.8.3.5-4*

if qu > 1.9 Mpa - may be taken after Horvath & Kenney 1979 → qs = 0.21*sqrt(qu) *C10.8.3.5-5*

Case 2: Side of rock socket is considered to be smooth or where the rock is drilled using a drilling slurry

May be taken after Horvath & Kenney 1979 → qs = 0.65*α_E*pa*(qu/pa)^0.5<7.8*pa*(f'c/pa)^0.5

qs0 = 7.8*pa*(f'c/pa)^0.5 *10.8.3.5.4d-1-Lrfd2007*

Drilling method used in construction: *Case 2*

$Q_R = \phi*Q_n = \phi_s*Q_{SR}$

ϕ_s is the resistance factor - table 10.5.5-3 LRFD

q_u is the uniaxial compressive strength of the rock

Case1

No.	EL_T	EL_B	Depth (m)	RQD (%)	q_u (MPa)	q_s (MPa)	Q_{SR} (kN)	ϕ_s	Q_R (kN)
1	3.75	2.75	1.00	80	76.02	1.83	5752	0.65	3739
2	2.75	1.75	1.00	80	76.02	1.83	5752	0.65	3739
3	1.75	1.00	0.75	90	76.27	1.83	4321	0.65	2809
4									
5									
6									
7									
8									
Sum			2.75				15825		10286

	DANANG QUANG NGAI EXPRESSWAY	Item.	Eng.	Date.	Sign.
	CB12 BRIDGE	Design			
	DETAIL DESIGN	Check			
	EMPIRICAL ESTIMATION OF PILE CAPACITY-ABUTMENT A1	Revise			

Case2 — Type: "1: closed joints"; "2: open joints"

No.	Depth (m)	RQD (%)	q_u (MPa)	E_m/E_i	α_E	Type	qs0 (MPa)	q_s (MPa)	q_s - used (MPa)	Q_{SR} (kN)	ϕ_s	Q_R (kN)
1	1.00	80.00	76.02	0.80	0.92	1	13.58	1.66	*1.66*	5206	0.55	2863
2	1.00	80.00	76.02	0.80	0.92	1	13.58	1.66	*1.66*	5206	0.55	2863
3	0.75	90.00	76.27	0.90	0.96	1	13.58	1.73	*1.73*	4081	0.55	2244
4	-	-	-	-	-	-	-	-	-	-	-	-
5	-	-	-	-	-	-	-	-	-	-	-	-
6	-	-	-	-	-	-	-	-	-	-	-	-
7	-	-	-	-	-	-	-	-	-	-	-	-
8	-	-	-	-	-	-	-	-	-	-	-	-
Sum	2.75									14492		7970

Unit base resistance $q_p = Kb.(p1-p0) + \sigma v$

Limit pressure determined from presuremeter tests $p_l =$ - MPa *C10.8.3.5-7*

At rest total horizontal stress measured at ther base elevation $p_0 =$ - MPa

Coefficient that depen on diameter socket $K_b =$ 4.05 *Table C10.8.3.5-1*

Total vertical stress at the base elevation $\sigma v =$ - MPa

$q_p =$ - MPa

$\phi =$ 0.50 *Table 10.5.5-3*

$Q_{pR} = Ap . qp$ $Q_{pR} =$ - kN

$Q_R = \phi.Q_{pR}$ $Q_R =$ - kN

ESTIMATED PILE CAPACITY:

Pile Structural Capacity $Q_T = 0.75*0.85*0.85*fc*Ag$	*12768*	kN	*1301*	T
Pile resistance Q_R	*7970*	kN	*812*	T
Deducting pile weight	*-137*	kN	*-14*	T
Estimated Pile Capacity	*7834*	kN	*799*	T
Maximum Reaction - ULS **Ok**	*4842*	kN	*494*	T

ASSUMPTION:

- The rock is reasonably sound
- The rock strength measured during site investigation will not deteriorate during construction when water or drilling fluids are used
- The bottom of the socket is properly cleaned out.

DATA & CALCULATION:

Bored hole name		CB12-A2	
Bottom of pilecap elavation	EL1 =	12.00	
Top of socket elevation	EL2 =	2.10	
Pile tip elevation	EL3 =	-1.00	
Pile Length	L =	13.00	m
Diameter of drilled-shaft	D_p =	1.00	m
Pile Cross-Sectional Perimeter	P =	3.14	m
Pile Cross-Sectional Area	A_b =	0.79	m^2
Working normal force at pile head	N =	5092.5	kN
Working normal force at top of socket	P_i =	5056.7	kN
Intack rock modulus	E_i =	25000	MPa
Modulus modification ratio	K_e =	0.03	
Elastic modulus of the insitu rock	$E_r = K_e*E_i$ =	750.0	MPa
Influence coefficient	$I_p = f(H_s/D_s, E_c/E_r)$ =	0.30	
	H_s/D_s =	3.10	
	E_c/E_r =	36.92	

Pile Concrete comp. strength	f'_c =	30.0	MPa
Concrete Unit Weight	g_c =	24.5	kN/m^3
Modulus of elasticity of concre	E_c =	27691	MPa
Depth of socket	H_s =	3.10	m
Diameter of socket	D_s =	1.00	m
Socket Cross-Sect. Perimeter	P_{soc} =	3.14	m
Socket Cross-Sectional Area	A_{soc} =	0.79	m^2

Figure C10.8.3.5-2 Lrfd

Figure C10.8.3.5-3 Lrfd

Figure C10.8.3.5-1 Lrfd

Rock mass modulus/ intack rock modulus E_m / E_i — *C.10.4.6.5-1-Lrfd 4th*

Atmospheric pressure p_a = 0.101 MPa

Reduction factor to account for jointing α_E — *10.8.3.5.4b-Lrfd 4th*

The elastic shorterning of the drilled shaft

$r_e = (\Sigma P_i)*H_s/(A_{soc}*E_c) =$ 0.721 mm

The settlement of base of drilled shaft

$r_{base} = (\Sigma P_i)*I_p/(D_s*E_r) =$ 2.023 mm

$r_e + r_{base} =$ **2.743** mm < 10 mm

Compute the bearing capacity based on Sharft resistance alone

Unit side resistance

Case 1: The drilling fluid used will not form a lubricated film on the sides of the socket

if qu < 1.9 Mpa - may be taken after Carter & Kulhawy 1988 → qs = 0.15*qu — *C10.8.3.5-4*

if qu > 1.9 Mpa - may be taken after Horvath & Kenney 1979 → qs = 0.21*sqrt(qu) — *C10.8.3.5-5*

Case 2: Side of rock socket is considered to be smooth or where the rock is drilled using a drilling slurry

May be taken after Horvath & Kenney 1979 → qs = 0.65*α_E*pa*(qu/pa)^0.5<7.8*pa*(f'c/pa)^0.5

qs0 = 7.8*pa*(f'c/pa)^0.5 — *10.8.3.5.4d-1-Lrfd2007*

Drilling method used in construction: *Case 2*

$Q_R = \phi*Q_n = \phi_s*Q_{SR}$

ϕ_s is the resistance factor - table 10.5.5-3 LRFD

q_u is the uniaxial compressive strength of the rock

Case1

No.	EL_T	EL_B	Depth (m)	RQD (%)	q_u (MPa)	q_s (MPa)	Q_{SR} (kN)	ϕ_s	Q_R (kN)
1	2.10	1.10	1.00	30	92.02	2.01	6329	0.65	4114
2	1.10	0.10	1.00	60	79.58	1.87	5885	0.65	3825
3	0.10	-1.00	1.10	60	79.58	1.87	6474	0.65	4208
4									
5									
6									
7									
8									
Sum			**3.10**				**18688**		**12147**

	DANANG QUANG NGAI EXPRESSWAY	Item.	Eng.	Date.	Sign.
	CB12 BRIDGE	Design			
	DETAIL DESIGN	Check			
	EMPIRICAL ESTIMATION OF PILE CAPACITY-ABUTMENT A2	Revise			

Case2 *Type: "1: closed joints", "2: open joints"*

No.	Depth (m)	RQD (%)	q_u (MPa)	E_m / E_i	α_E	Type	qs0 (MPa)	q_s (MPa)	q_s*-used* (MPa)	Q_{SR} (kN)	ϕ_s	Q_R (kN)
1	1.00	30.00	92.02	0.08	0.52	1	13.58	1.02	*1.02*	3216	0.55	1769
2	1.00	60.00	79.58	0.42	0.76	1	13.58	1.41	*1.41*	4414	0.55	2428
3	1.10	60.00	79.58	0.42	0.76	1	13.58	1.41	*1.41*	4856	0.55	2671
4	-	-	-	-	-	-	-	-	-	-	-	-
5	-	-	-	-	-	-	-	-	-	-	-	-
6	-	-	-	-	-	-	-	-	-	-	-	-
7	-	-	-	-	-	-	-	-	-	-	-	-
8	-	-	-	-	-	-	-	-	-	-	-	-
Sum	**3.10**									**12486**		**6868**

Unit base resistance $q_p = Kb.(p1\text{-}p0) + \sigma v$

Limit pressure determined from presuremeter tests	$p_l =$	5.89	MPa	*C10.8.3.5-7*
At rest total horizontal stress measured at ther base elevation	$p_0 =$	-	MPa	
Coefficient that depen on diameter socket	$K_b =$	4.24		*Table C10.8.3.5-1*
Total vertical stress at the base elevation	$\sigma v =$	-	MPa	
	$q_p =$	-	MPa	
	$\phi =$	0.50		*Table 10.5.5-3*
$Q_{pR} = Ap . qp$	$Q_{pR} =$	-	kN	
$Q_R = \phi.Q_{pR}$	$Q_R =$	-	kN	

ESTIMATED PILE CAPACITY:

Pile Structural Capacity $Q_T = 0.75*0.85*0.85*f'c*Ag$	*12768*	kN	*1301*	T
Pile resistance Q_R	*6868*	kN	*700*	T
Deducting pile weight	*-174*	kN	*-18*	T
Estimated Pile Capacity	*6693*	kN	*682*	T
Maximum Reaction - ULS **Ok**	*4842*	kN	*494*	T

MINISTRY OF TRANSPORT

VIETNAM EXPRESSWAY CORPORATION - PROJECT MANAGEMENT UNIT NO.85

DA NANG - QUANG NGAI EXPRESSWAY DEVELOPMENT PROJECT

PACKAGE: A6

BRIDGE

ORB13

CALCULATION SHEETS

HA NOI, 2012

MINISTRY OF TRANSPORT

VIETNAM EXPRESSWAY CORPORATION - PROJECT MANAGEMENT UNIT NO.85

DA NANG - QUANG NGAI EXPRESSWAY DEVELOPMENT PROJECT

PACKAGE: A6

BRIDGE

ORB 13

CALCULATION SHEETS

SUBSTRUCTURE

HA NOI, 2012

CALCULATION SHEET

ABUTMENT A1

LOAD COMPONENTS

Assumptions :

1. Bridge is considered to be in seismic with acceleration coeff. A = 0.0301 g
2. The Design of the Abutment accords with Specification for bridge design 22-TCN-272-05 and AASHTO LRFD 2004 for reference
3. Design live load: HL-93 and lane loading 9.3 KN/m

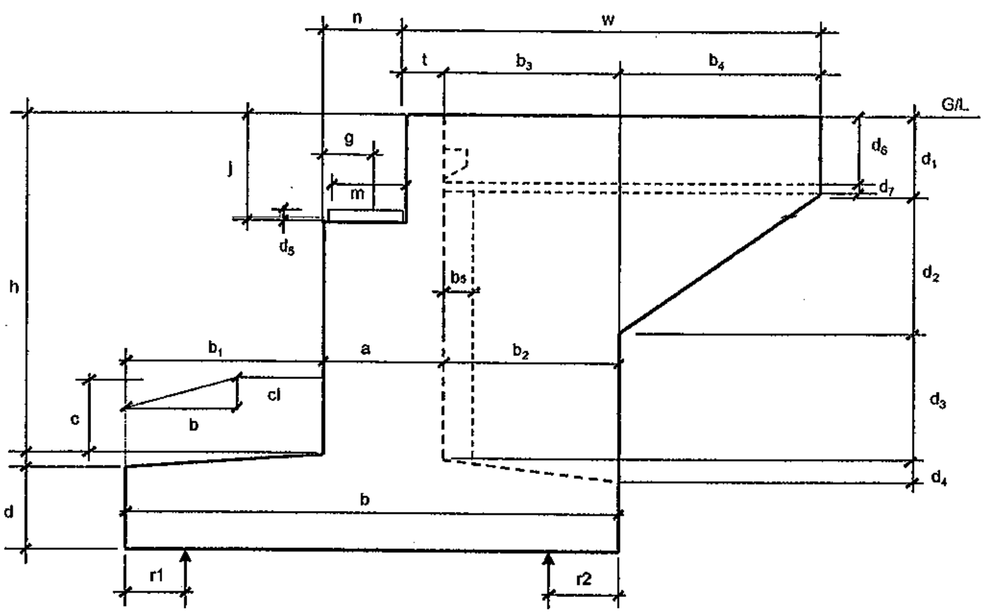
Input :

Level Table(at center of abutment)

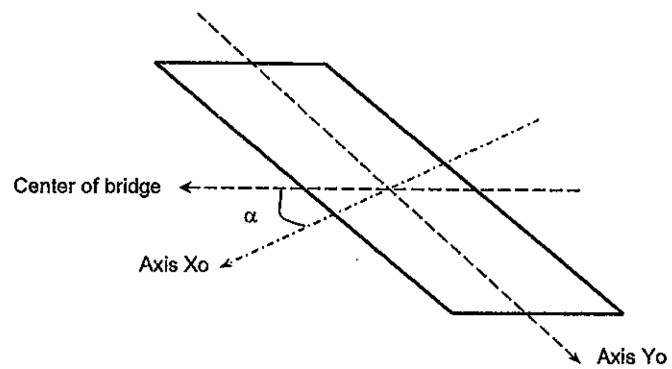
Level of top of headwall	HTwL	20.018	m
Level of top of bearing	BTL	18.038	m
Level of top of stem abutment	HTL	17.888	m
Level of top of footing	FTL	11.000	m
Level of bottom of footing	FBL	9.000	m
Ground level	GL	11.000	m
Lowest water level	HWL	9.000	m
Skew angle	α	0.00	deg

I.Loads from substructure

Abutment dimensions VERTICAL VIEW



PLAN VIEW



Material Unit Weights

• Unit Weight of Reinf. concrete	γ_c	=	24.5 kN/m^3
• Unit Weight of Soil	γ_s	=	18.0 kN/m^3
• Unit Bouyancy Weight of Soil	γ_{sbo}	=	8.2 kN/m^3

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	DETAIL DESIGN	Check			
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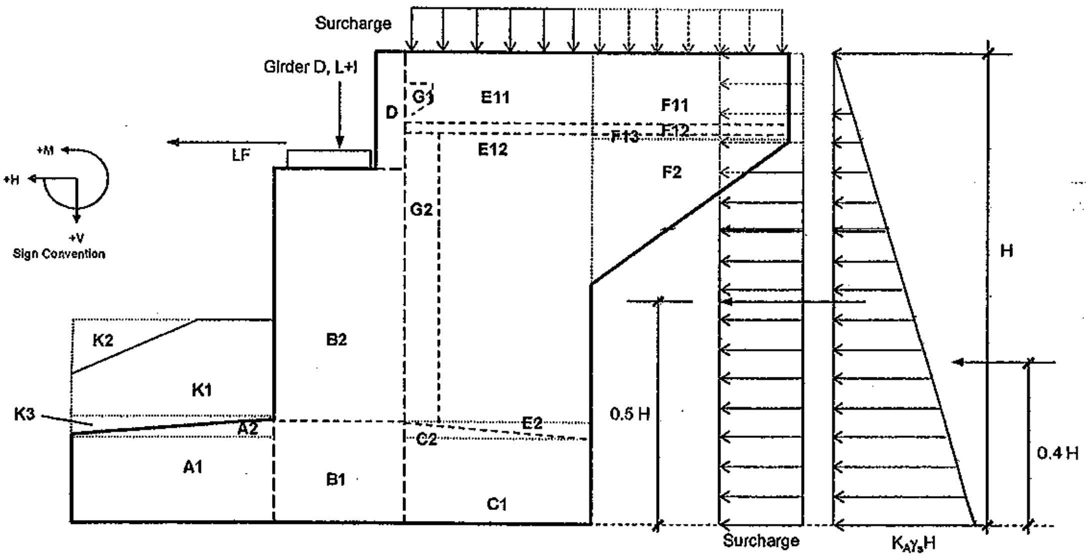
ABUTMENT DIMENSIONS (IN METRES)					
Item	Symbol	Value	Item	Symbol	Value
Height of stem	h	9.018	Horizontal Dimension	b_3	3.700
Footing Width	b	9.000	Horizontal Dimension	b_4	5.800
Stem Width	a	1.800	Horizontal Dimension	b_5	0.300
Footing Depth	d	2.000	Vertical Dimension	d_1	0.930
Footing Slope	f	0.000	Vertical Dimension	d_2	5.800
Width of stem at bearing	n	1.300	Vertical Dimension	d_3	2.288
Ballast Wall Height	j	2.130	Vertical Dimension	d_4	0.000
Ballast Wall Thickness	t	0.500	Vertical Dimension	d_5	0.150
Wingwall Length	w	10.000	Vertical Dimension	d_6	0.930
Soil Cover at Toe	c	0.000	Vertical Dimension	d_7	0.300
Girder Reaction	g	0.850	With of bearing pad	m	0.900
Distance to cl of pile	r1	1.200	Wingwall Thickness	u1	0.500
Horizontal Dimension	b_1	3.500	Wingwall Thickness	u2	0.800
Horizontal Dimension	b_2	3.700	Distance to cl of pile	r2	1.200

	$\cos(\alpha)$	=	1.00
Slope front of abutment	cl	=	0.00 m
	bl	=	0.00 m
Width of Abutment	L		12.600 m
Width of abutment (inclined direction)	Ltr		12.600 m
Height of Abutment	Ht	=	11.02 m
Distance from CG of footing to edge of Abutment	b/2	=	4.50 m



1. Self weight of Abutment (DC)

Description	Area (m^2)	Length (m)	Force (kN)	X[1] (m)	Arm[2] (m)	Moment (kN•m)
SW of Abutment (DC)						
Section A1	7.000	12.600	2161	1.750	2.750	5942
Section A2	-	12.600	-	2.333	2.167	-
Section B1	3.600	12.600	1111	4.400	0.100	111
Section B2	12.398	12.600	3827	4.400	0.100	383
Section C1	7.400	12.600	2284	7.150	-2.650	-6054
Section C2	-	12.600	-	6.533	-2.033	-
Section D	1.065	12.600	329	5.050	-0.550	-181
Section E11	2.331	0.500	29	7.150	-2.650	-76
Section E12	29.926	0.800	587	7.150	-2.650	-1554
Section F11	5.394	0.500	66	11.900	-7.400	-489
Section F12	1.425	0.650	23	10.050	-5.550	-126
Section F13	-1.740	0.800	-34	11.900	-7.400	252
Section F2	16.820	0.800	330	10.933	-6.433	-2121
Section G1	0.135	11.600	281	5.450	-0.950	-267
Section G2	0.045	15.576	17	5.450	-0.950	-16
Bearing seats (w1seat= 0.85m)	0.135	4.250	18	4.350	0.150	3
Curbs +Handrail on Abutment	0.50	10.000	133	9.800	-5.300	-702
Total SW of Abutment (DC)			***11160***			***-4894***
Tranverser moment			*977*		*6.100*	*5958*

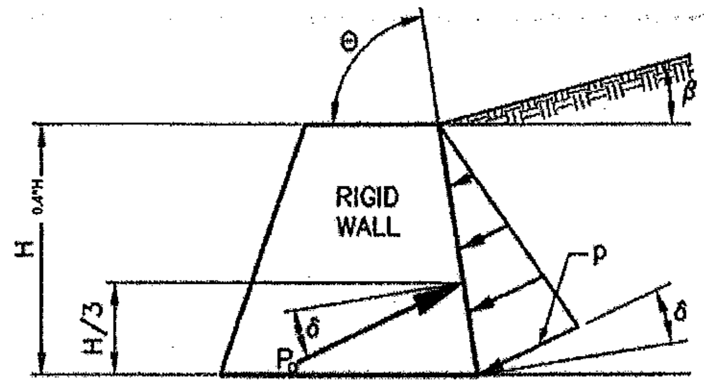
Notes: 1. Distance 'X' is measured horizontally from Toe of Retaining to CG of Section
2. Moment 'Arm' is measured from CG horizontally and from Underside of Footing Vertically.

2. Earth on Abutment (EV)

Description	Area (m^2)	Length (m)	Force (kN)	X[1] (m)	Arm[2] (m)	Moment (kN•m)
Earth on Abutment (EV)						
Section E1	33.37	11.000	6607	7.150	-2.650	-17507
Section E2	-	11.000	-	7.767	-3.267	-
Section E3	-	1.600	-	9.000	-4.500	-
Section K1	-	12.600	-	1.750	2.750	-
Section K2	-	12.600	-	-	4.500	-
Section K3	-	12.600	-	1.167	3.333	-
Total Earth on Footing			*6607*			*-17507*

3. Horizontal Earth Pressure on Abutment (EH)

To be safe, horizontal earth pressure at front face of abutment may be neglected.
Horizontal earth pressure at behind face of abutment shall be considered.



- Height for horizontal ear
- Width for horizontal eart
- Density of Soil
- Internal Friction Angle of
- Incline angle of back face wall — θ = 90.0 deg
- Friction angle between fill and wall — δ = 0.0 deg
- Incline angle of fill soil — β = 0.0 deg
- Gravitational acceleration — g = 9.81 m/s^2
- Basic earth pressure

p = K.γs. g. Z .10^-9 (Mpa, Z:mm)

K: taken as Ka (assume wall move or deflect sufficiently to reach minimum active conditions)

$$Ka = \frac{\sin^2(\theta + \phi'_f)}{\Gamma.[\sin^2\theta.\sin(\theta - \delta)]}$$

$$\Gamma = \left[1 + \sqrt{\frac{\sin(\phi'_f+\delta).\sin(\phi'_f-\beta)}{\sin(\theta-\delta).\sin(\theta+\beta)}}\right]^{-2}$$

Γ = 2.250
Ka = 0.333
p = 0.066 Mpa

Horizontal earth pressure:
- Ea = 0,5. p. Z . B . 10^3 (kN) — Ea = 4589 kN
- M= Ea . 0,4H — M = 20224 kNm
- Horizontal Earth Pressure act at a height of 0.4 H — <S 3.11.5.1>

4. Earth Pressure on Abutment due to Surcharge (ES)

Equivalent height of soil for highway loading taken from Table 3.11.6.2.1

H=	**1.50m**	heq=	**1.7** m	
H=	**3.00m**	heq=	**1.2** m	
H=	**6.00m**	heq=	**0.76** m	
H=	**9.00m**	heq=	**0.61** m	
H=	11.02m	heq=	0.61 m	(Linear interpolation)

- **Vertical force**

ESv = 512 kN
ev = -2.65 m
M = -1357 kNm

- **Horizontal force**

$$\Delta p = k\gamma_s g h_{eq} x 10^{-9}$$

ESh = 508 kN
eh = 5.51 m
M = 2799 kNm

5. Earthquake effects

Bridge is located at: Son Tinh district - Quang Ngai province
According to TCXDVN 375:2006 and 22TCN272-05, bridge is in seismic zone 2and acceleration coefficient as below

- Peak ground acceleration coefficient A = 0.0301 g

5.1. Seismic active lateral Earth pressure (E_{AE})

- Backfill slop angle i = 0.0 deg
- Slope of wall to vertical β' = 0.0 deg
- Angle of friction of soil ϕ = 30.0 deg
- Angle of friction between soil and abutment δ = 0.0 deg
- Horizontal acceleration coefficient kh = 0.045
- Vertical acceleration coefficient kv = 0.018
- Angle $\theta = \arctan(k_h/(1-k_v))$ θ = 2.6 deg

$$K_{AE} = \frac{\cos^2(\phi-\theta-\beta)}{\cos\theta.\cos^2\beta\cos(\delta+\beta+\theta)} x \left[1+\sqrt{\frac{\sin(\phi+\delta).\sin(\phi-\theta-i)}{\cos(\delta+\beta+\theta).\cos(i-\beta)}}\right]^{-2}$$

- Seismic active lateral Earth pressure coefficient K_{AE} = 0.36

- $E_{AE} = 0,5.\ g.\ \gamma.\ H^2.\ (1-kv).\ K_{AE}.\ 10$^-9 (kN/m)

- Seismic active lateral Earth pressure coefficient E_{AE} = 4880 kN

$M_{AE} = E_{AS}*0.3H+(E_{AE} - E_{AS})*0.6H$ M_{AE} = 17091 KNm <A.11.1.1.1>

E_{AS} is the static component of seismic active pressure calculated with θ = kv = 0

5.2. Earthquake effects to abutment (EQ)

Seismic force for substructures: elements above ground Fh = Csm* W; elements under ground Fh = A*S*W

- Soil profile type Soil type I
- Site Coefficients. S = 1.0
- Elastic Seismic Response Coefficient 2.5A = 0.075

Csm = 1.2*A*S/ Tm^2/3 ≤ 2.5*A Csm = 0.034

- Period of vibration of the fundamental mode

Tm = 2*pi()*sqrt(m/k) Tm = 1.076 s

Decription	Area (m^2)	Length (m)	Force (kN)	$X^{(1)}$ (m)	$Arm^{(2)}$ (m)	Moment (kN•m)
Section A1	7.000	12.600	65	-	1.000	65
Section A2	-	12.600	-	-	2.000	-
Section B1	3.600	12.600	33	-	1.000	33
Section B2	12.398	12.600	132	-	5.444	717
Section C1	7.400	12.600	69	-	1.000	69
Section C2	-	12.600	-	-	2.000	-
Section D	1.065	12.600	11	-	9.953	113
Section E11	2.331	0.500	1	-	8.553	7
Section E12	29.926	0.800	18	-	3.894	
Section E2	-	0.740	-	-	2.000	-
Section F11	5.394	0.500	2	-	8.553	17
Section F12	1.425	0.650	1	-	7.938	
Section F13	-1.740	0.800	-1	-	9.168	
Section F2	16.820	0.800	10	-	8.155	81
Section G1	0.135	11.600	1	-	8.305	10
Section G2	0.045	15.576	1	-	3.894	2
Total EQ of Abutment Selfweight			***342***			***1113***

6. Braking Force(BR)

Take 50 *% Braking Force for this Abutment (Free Bearing)*

• Number of lanes	n	=	3 lanes	
• Multiple presence factor	m	=	0.85	
• Take 25 % of Truck load				
BR = 25% * n * m * (2*145+35)	BR	=	104 kN	Long. Axis
• Acting at 1.8m higher of road face	e	=	12.9 m	
	Mlong	=	1335 KNm	Long. Axis

7. Centrifucal Force , CE (3.6.3)

• Plan of bridge (1:"straight",2: "Curve")			1	
• Design Speed	V	=	120 km/h	
	V	=	33.3 m/s	
	R		- m	
$C = 4/3*(V^2/gR)$	C		-	
Acting at 1.8m higher of road face				
CE = n * m * (2*145+35) * C	CE	=	0.00 KN	
	e	=	12.90 m	
	Mtrans	=	0.00 KNm	Trans. Axis

8. Water Load (WA) ***:NA***

SUPERSTRUCTURE LOADS

II.Loads from superstructure

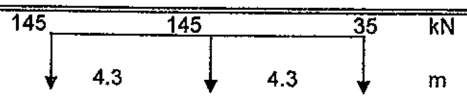
Item	Sign	Value	Unit
Span length	Lsp	27.00	m
Span between bearings	Lb	26.10	m
Skew angle	α	0.00	deg
Deck slab length	Ldeck	27.00	m
Bridge Width	Bc	12.74	m
Girder height	hgi	1.50	m
Deck slab depth	hdkslab	0.218	m
Asphalt depth	has	0.084	m
Unit weight of concrete	γc	24.50	kN/m3
Unit weight of asphalt concrete	γa	22.10	kN/m3

1.Dead loads (DC): One span at abutment

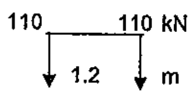
Item	Sign	Value	Unit
1.1.Girders			
Weight of 1 girder	DC	464.77	kN
Number of girders	n	5	Girders
Sum of girders weight	DC	2323.83	kN
Precast Planks	DC	394.43	kN
Diaphragm	DC	1176.38	kN
Total	DC	3894.64	kN
1.2.Deck slab			
Deck slab	DC	1790.85	kN
1.3.Pavement			
Asphalt concrete	DW	538.32	kN
1.4.Handrail			
Handrail + median	DC	639.90	kN

2.Live load (LL):

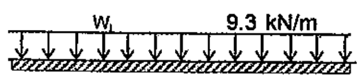
Truck



Tandem



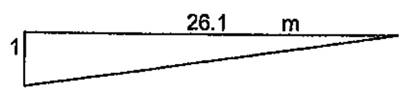
Lane load



Pedestrial Wpd= 0.0 kN/m2

Considerate structure as a simple span

Reaction Influence



Number of lanes	n	3	
Multiple presence factor	m	0.85	
Dynamic load allowance	1+IM	1.25	

*Reaction = [(1+IM)*Vehicle + Laneload]*n*m*

Truck	P1(kN)	P2(kN)	P3(kN)	Sum(kN)	Total(kN)
Axle load	145	145	35		
Influence value	1.000	0.835	0.670		
Reaction	145.0	121.1	23.5	289.6	923.0

Tandem	P1(kN)	P2(kN)	Sum(kN)	Total(kN)
Axle load	110	110		
Influence value	1.000	0.954		
Reaction	110	104.9	214.9	685.1

Lane load	Wl(kN/m)	Total(kN)
Value	9.3	
Influence value	13.05	
Reaction	121.4	309.5

Pedestrian	Wdb(kN)	Total(kN)
Reaction	0.0	0.0

3.Earthquake effects on superstructrue (EQ)

Longitudinal moveable bearings at Abutment

Horizontal force from superstructure due to EQ - transverse direction

At bearing — Heq = 118 kN

4.Uniform Temperature, Srinkage& Creep(TU+SH&CR)

Bearing displacement due to uniform temperature and shrinkage creep — Δu = 0.026 m

$H = G.A.\Delta u/h_{rt}$ <14.6.3.1-2>

Shear modulus G	G	=	1 MPa
Bearing area	A	=	0.165 m2
Height of elastomeric layers	hrt		0.065 m
Number of bearing	nb	=	5 bears
Horizontal force due to TU+SH&CR			
Acting at top of bearing	H(tu+sh+cr)	=	330 kN

5.Wind loads (Ws)

5.1.Tranverse wind on superstructure (WS)

Wind zone	Zone		III
Basic 3 second gust wind	Vb	=	53.00 m/s
Correction factor	S	=	1.09
Design wind velocity	V	=	57.77 m/s
Drag coefficient	Cd	=	1.39
Overall width of bridge	b	=	12.74 m
Depth of superstructure(including solid parapet)	d	=	2.78 m
	b/d	=	4.58
Windy obstructed area of superstructure	At	=	75.14 m2
Force due to tranverse wind			
Fhy=max(0.0006*V2*At*Cd,1.8*At) (kN)	Fhy	=	209.5 kN <3.8.1>

5.2.Wind load on vehicles (WL)

Transverse wind on vehicles	Wltran	=	1.50 kN/m
Transverse horizontal force due to wind on liveload			
At 1.8m from surface	Fhy	=	40.50 kN

6.Combinations

Loads from superstructure to Abutment

Loads at bottom of stem		Vertical		Longitudinal			Tranversal		
Loads	Sign	N (kN)	x (m)	Hx (kN)	z_1 (m)	My (kN•m)	Hy (kN)	y (m)	Mx (kN•m)
Girder + Deckslab	DC	2843	0.05			142			
Handrail	DC	320	0.05			16			
Pavement	DW	269	0.05			13			
LiveLoad	LL	1233	0.05			62		1.37	1689
Pedestrial	PL	0	0.05			0		-	-
Trans. wind on Struc.	WS						105	6.89	722
Trans. wind on vehi.	WL						20	8.69	176
Eearth quake	EQ						118	6.89	813
TU+SH&CR	TU+SH&CR			330	6.89	2273			

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		Design			
		Check			
		Revise			

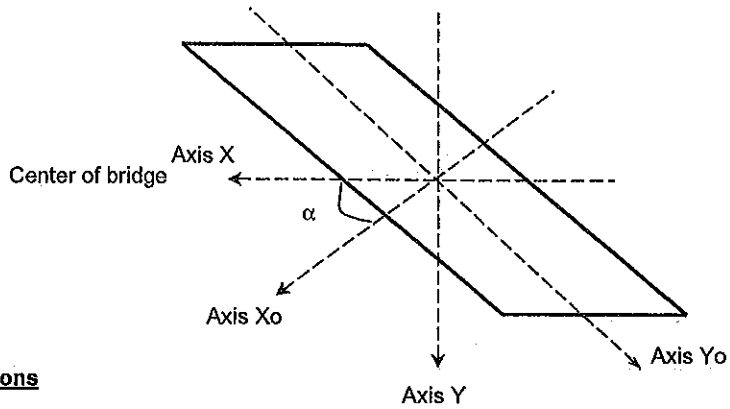
Loads at bottom of pilecap		Vertical		Longitudinal			Tranversal		
Loads	Sign	N (kN)	x (m)	Hx (kN)	z_1 (m)	My (kN•m)	Hy (kN)	y (m)	Mx (kN•m)
Girder + Deckslab	DC	2843	0.15			426			
Handrail	DC	320	0.15			48			
Pavement	DW	269	0.15			40			
LiveLoad	LL	1233	0.15			185		1.37	1689
Pedestrial	PL	0	0.15			0			
Trans. wind on Struc.	WS						105	8.89	931
Trans. wind on vehi.	WL						20	10.69	216
Eearth quake	EQ						118	8.89	1049
TU+SH&CR	TU+SH&CR			330	8.89	2933			

Loads		Load combinations						
	Sign	Str-IA	Str-IB	Str-IIIA	Str-IIIB	Ser-I	Ext-IA	Ext-IB
Girder + Deckslab	DC	1.25	0.90	1.25	0.90	1.00	1.25	0.90
Handrail	DC	1.25	0.90	1.25	0.90	1.00	1.25	0.90
Pavement	DW	1.50	0.65	1.50	0.65	1.00	1.50	0.65
LiveLoad	LL	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Pedestrial	PL	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Trans. wind on Struc.	WS			0.40	0.40	0.30		
Trans. wind on vehi.	WL			1.00	1.00	1.00		
Eearth quake	EQ						1.00	1.00
TU+SH&CR	TU+SH&CR	0.50	0.50	0.50	0.50	1.00		

Load combinations at bottom of stem					
		Longitudinal		Tranversal	
Load combinations	N (kN)	Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	6514	165	1462	0	2955
Strength Str-IB	5178	165	1395	0	2955
Strength Str-IIIA	6021	165	1438	62	2744
Strength Str-IIIB	4685	165	1371	62	2744
Service Ser-I	4664	330	2506	52	2081
Extreme Ext-IA	4973	0	249	118	1657
Extreme Ext-IB	3638	0	182	118	1657

Load combinations at bottom of pilecap					
		Longitudinal		Tranversal	
Load combinations	N (kN)	Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	6514	165	2444	0	2955
Strength Str-IB	5178	165	2243	0	2955
Strength Str-IIIA	6021	165	2370	62	2868
Strength Str-IIIB	4685	165	2169	62	2868
Service Ser-I	4664	330	3633	52	2184
Extreme Ext-IA	4973	0	746	118	1893
Extreme Ext-IB	3638	0	546	118	1893

LOAD COMBINATIONS



III.Load Combinations

1.Loads from substructure

Loads at bottom of pilecap	Sign	Vertical N (kN)	x (m)	Longitudinal Hx (kN)	z_1 (m)	My (kN•m)	Tranversal Hy (kN)	y (m)	Mx (kN•m)
Self weight of Abutment	DC	11160				-4894			5958.12
Soils on pilecap	EV	6607				-17507			
Horizontal Earth Pressure	EH			4589		20224			
Vertical Surcharge	LSv	512				-1357			
Horizontal Surcharge	LSh			508		2799			
Braking Force	BR			104		1335			
Centrifucal Force	CE			-		-	-		-
Buoyancy of Abutment	WA	-				-			
Buoyancy of Earth on Abutment	WA	-				-			
Earthquake effects to Abutment	EQ			342		1113	103		334
Earthquake effects to soil	E_{AE}			4880		17091			

Table of load factors

Loads	Sign	Load combinations Str-IA	Str-IB	Str-IIIA	Str-IIIB	Ser-I	Ext-IA	Ext-IB
Self weight of Abutment	DC	1.25	0.90	1.25	0.90	1.00	1.25	0.90
Soils on pilecap	EV	1.35	0.90	1.35	0.90	1.00	1.35	0.90
Horizontal Earth Pressure	EH	1.50	0.90	1.50	0.90	1.00	0.00	0.00
Vertical Surcharge	LSv	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Horizontal Surcharge	LSh	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Braking Force	BR	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Centrifucal Force	CE	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Buoyancy of Abutment	WA	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Buoyancy of Earth on Abutment	WA	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Earthquake effects to Abutment	EQ						1.00	1.00
Earthquake effects to soil	E_{AE}						1.00	1.00

Load Combination at bottom of pilecap					
Load combinations	N (kN)	Longitudinal Hx (kN)	My (kN.m)	Tranversal Hy (kN)	Mx (kN.m)
Strength Str-IA	23765	7954	5444	0	7448
Strength Str-IB	16886	5200	2901	0	5362
Strength Str-IIIA	23561	7709	4333	0	7448
Strength Str-IIIB	16681	4956	1790	0	5362
Service Ser-I	18279	5200	600	0	5958
Extreme Ext-IA	23125	5527	-10160	103	7782
Extreme Ext-IB	16246	5527	-569	103	5696

2. Loads from superstructure

Load Combination at bottom of pilecap					
Load combinations	N	Longitudinal		Tranversal	
		Hx	My	Hy	Mx
	(kN)	(kN)	(kN.m)	(kN)	(kN.m)
Strength Str-IA	6514	165	2444	0	2955
Strength Str-IB	5178	165	2243	0	2955
Strength Str-IIIA	6021	165	2370	62	2868
Strength Str-IIIB	4685	165	2169	62	2868
Service Ser-I	4664	330	3633	52	2184
Extreme Ext-IA	4973	0	746	118	1893
Extreme Ext-IB	3638	0	546	118	1893

3. Total loads at bottom of pilecap

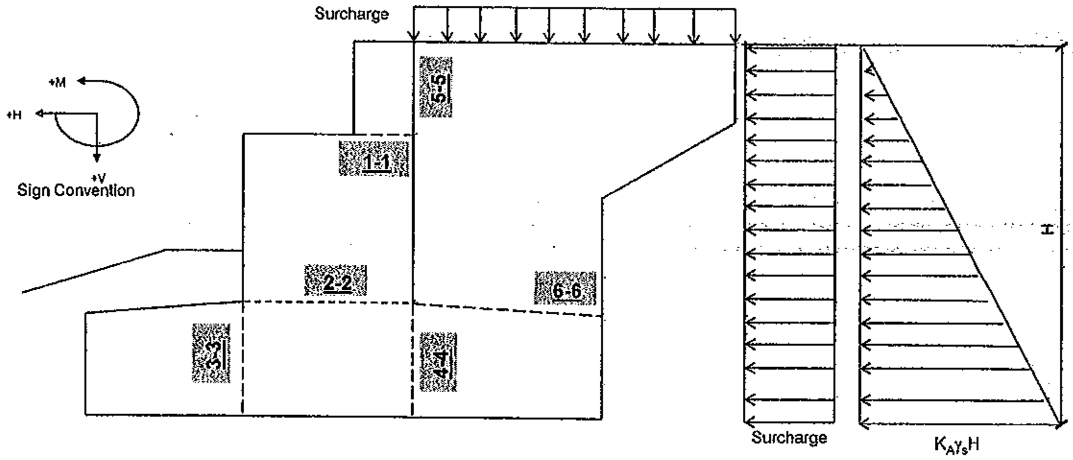
Load Combination at bottom of pilecap					
Load combinations	N	Longitudinal		Tranversal	
		Hx	My	Hy	Mx
	(kN)	(kN)	(kN.m)	(kN)	(kN.m)
Strength Str-IA	30279	8119	7887	0	10403
Strength Str-IB	22064	5365	5144	0	8317
Strength Str-IIIA	29582	7874	6702	62	10316
Strength Str-IIIB	21367	5121	3959	62	8231
Service Ser-I	22943	5530	4232	52	8142
Extreme Ext-IA	28099	5527	-9414	221	9675
Extreme Ext-IB	19884	5527	-23	221	7590

ELEMENTS CHECKING

IV.Elements checking

The abutment walls shall be checked at sections 1-1, 2-2, 3-3, 4-4, 5-5&6-6

1. Calculate internal force of sections



1.1 Section 1-1

Table of Load Factors

Loads	Sign	Service I	Strength I		Extreme I
		Ser-I	Str-IA	Str-IB	Ext-I
Selfweight	DC	1.00	1.25	0.90	1.25
Horizontal Earth Pressure	EH	1.00	1.50	0.90	
Surchage (Horizontal)	LS_h	1.00	1.75	1.75	0.50
Horizontal Seismic Earth Pressure	E_{AE}				1.50
Abutment earthquake force	EQ				1.00

Unfactored Loads	N	Longitudinal		Tranversal	
		Hx	My	Hy	Mx
	(kN)	(kN)	(kN.m)	(kN)	(kN.m)
Selfweight	610		-112		
Horizontal Earth Pressure		171	146		
Surcharge (horizontal)		240	256		
Horizontal Seismic Earth Pressure		182	123		
Abutment earthquake force		12	13	4	4

Load Combination at bottom of headwall					
Load combinations	N	Longitudinal		Tranversal	
		Hx	My	Hy	Mx
	(kN)	(kN)	(kN.m)	(kN)	(kN.m)
Service Ser-I	610	411	289	0	0
Strength Str-IA	762	677	526	0	0
Strength Str-IB	549	574	478	0	0
Extreme Ext-I	762	406	186	4	4

1.2 Section 2-2

Table of Load Factors

Loads	Sign	Service I	Strength I		Extreme I
		Ser-I	Str-IA	Str-IB	Ext-I
Selfweight	DC	1.00	1.25	0.90	1.25
Superstructure Dead Load	DC	1.00	1.25	0.90	1.25
Pavement	DW	1.00	1.50	0.65	1.50
Handrail+curb	DC	1.00	1.25	0.90	1.25
Live Load	LL	1.00	1.75	1.75	0.50
Horizontal Earth Pressure	EH	1.00	1.50	0.90	
Surchage (Horizontal)	LS_h	1.00	1.75	1.75	0.50
TU+SH&CR	TU+SH&CR	1.00	0.50	0.50	
Horizontal Seismic Earth Pressure	E_{AE}				1.50
Abutment earthquake force	EQ				1.00

Unfactored Loads	N (kN)	Longitudinal Hx (kN)	Longitudinal My (kN.m)	Tranversal Hy (kN)	Tranversal Mx (kN.m)
Selfweight	4437		-284		
Superstructure Dead Load	2843		142		
Pavement	269		13		
Handrail+curb	320		16		
Live Load	1233		62		1689
Horizontal Earth Pressure		3074	11089		
Surchage (Horizontal)		416	1875		
TU+SH&CR		330	2273		
Horizontal Seismic Earth Pressure		3269	9371		
Abutment earthquake force		144	553	79	415

Load Combination at bottom of stem wall

Load combinations	N (kN)	Longitudinal Hx (kN)	Longitudinal My (kN.m)	Tranversal Hy (kN)	Tranversal Mx (kN.m)
Service Ser-I	9101	3820	15186	0	1689
Strength Str-IA	12060	5504	21022	0	2955
Strength Str-IB	9172	3659	14401	0	2955
Extreme Ext-I	10520	5255	15440	79	1259

1.3 Section 3-3

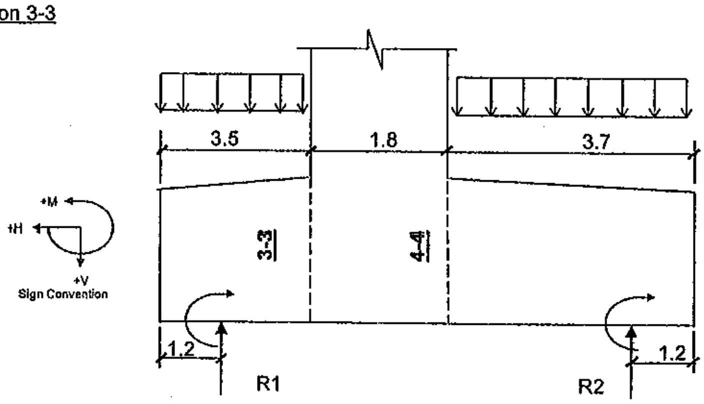


Table of Load Factors

Loads	Sign	Service I Ser-I	Strength I Str-IA	Strength I Str-IB	Extreme I Ext-I
Selfweight at front side	DC	1.00	1.25	0.90	1.25
Vertical soil on foot at front side	DC	1.00	1.35	0.90	1.35
Reaction of piles	RE	1.00	1.00	1.00	1.00

Unfactored Loads		N (kN)	Longitudinal Hx (kN)	Longitudinal My (kN.m)	Tranversal Hy (kN)	Tranversal Mx (kN.m)
Selfweight at front side		-2161		-3782		
Vertical soil on foot at front side		0		0		
Reaction of piles	Ser-I	9885	2149	29092	-15	-26
	Str-IA	13803	3169	41092	0	24
	Str-IB	9651	2077	28332	0	20
	Ext-I	9363	2162	28046	-64	-166

Load Combination at section 3-3

Load combinations	N (kN)	Longitudinal Hx (kN)	Longitudinal My (kN.m)	Tranversal Hy (kN)	Tranversal Mx (kN.m)
Service Ser-I	7724	2149	25310	-15	-26
Strength Str-IA	11102	3169	36365	0	24
Strength Str-IB	7707	2077	24929	0	20
Extreme Ext-I	6662	2162	23319	-64	-166

1.4 Section 4-4

Table of Load Factors

Loads	Sign	Service I	Strength I		Extreme I
		Ser-I	Str-IA	Str-IB	Ext-I
Selfweight at behind side	DC	1.00	1.25	0.90	1.25
Vertical soil on foot at behind side	DC	1.00	1.35	0.90	1.35
Surcharge(Vertical)	EV	1.00	1.75	1.75	0.50
Reaction of piles	RE	1.00	1.00	1.00	1.00

Unfactored Loads		N	Longitudinal		Tranversal	
			Hx	My	Hy	Mx
		(kN)	(kN)	(kN.m)	(kN)	(kN.m)
Selfweight of behind side		-3284		-7582		
Vertical soil on foot at behind side		-6607		-12222		
Surcharge(Vertical)		-512		-947		
Reaction of piles	Ser-I	4319	1432	6563	-12	-22
	Str-IA	5222	2112	6825	0	16
	Str-IB	4040	1384	6011	0	13
	Ext-I	7036	1438	13258	-49	-130

Load Combination at section 4-4					
Load combinations	N	Longitudinal		Tranversal	
		Hx	My	Hy	Mx
	(kN)	(kN)	(kN.m)	(kN)	(kN.m)
Service Ser-I	-6083	1432	-14188	-12	-22
Strength Str-IA	-8697	2112	-20809	0	16
Strength Str-IB	-5757	1384	-13470	0	13
Extreme Ext-I	-6244	1438	-13193	-49	-130

1.4 Section 5-5 & 6-6

Slope of triang pressure	tgβ	=	6.00
Uniform horizontal pressure	U.p	=	3.66 kN/m2

Load Combination at section 5-5					
Load combinations	N	Horizontal		Vertical	
		Hx	Mx	Hy	My
	(kN)	(kN)	(kN.m)	(kN)	(kN.m)
Service Ser-I		128	362		
Strength Str-IA		196	565		

Load Combination at section 6-6					
Load combinations	N	Horizontal		Vertical	
		Hx	Mx	Hy	My
	(kN)	(kN)	(kN.m)	(kN)	(kN.m)
Service Ser-I				326	398
Strength Str-IA				500	611

2. Elements Checking

2.1. Bearing Resistance <S.5.7.5>

The case of absence of confinement reinforcement in the conrete supporting the bearing device
Factored bearing resistance shall be taken

$Pr = \phi.\ Pn = \phi.\ 0.85.\ f'c.\ A1.\ m$

Dimension of bearing plate	w0	=	0.900 m	
	b0	=	0.850 m	
Area under bearing device	A1	=	0.765 m2	
Distributed width and length	w	=	1.300 m	
	b	=	1.250 m	
Notational area	A2	=	1.625 m2	

Where supporting surface is wider on all sides than loades area

$m = sqrt(A2/A1) \le 2.0$ case 1

where loaded area is subjected to nouniformly distributed bearing

$m = 0.75*sqrt(A2/A1) \le 1.5$ case 2

Modification factor	case 1	m	=	1.457	
Resistance factor		ϕ	=	0.700	<S.5.5.4.2>
Factored bearing resistance		Pr	=	19902 kN	> Pu
Bearing reaction of approach bridge		Pu	=	4883 kN	Ok

$Pu = 1.25*DC + 1.5*DW + 1.75*LL$

In case factored applied load exceeds the factored resistance,
provision shall be made to resist the bursting and spalling force in article 5.10.9

Factored bearing resistance shall be taken <S.5.10.9.7.2>

$Pr = \phi.\ fn.\ Ab$

fn take the lesser of

$fn = 0.7.\ f'ci.\ sqrt(A/Ag)$ and

$fn = 2.25.\ f'ci$

	fn	=	30.61 MPa	
Maximum area of the portion of supporting surface	A	=	1.625 m2	
Gross area of bearing plate	Ag	=	0.765 m2	
Effective net area of bearing plate, Ag minus stud of bearing	Ab	=	0.765 m2	
Nominal concrete strength at time of application	f'ci	=	30 MPa	
Factored bearing resistance	Pr	=	16390 kN	Ok

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	ORB13 BRIDGE	Design			
	DETAIL DESIGN	Check			
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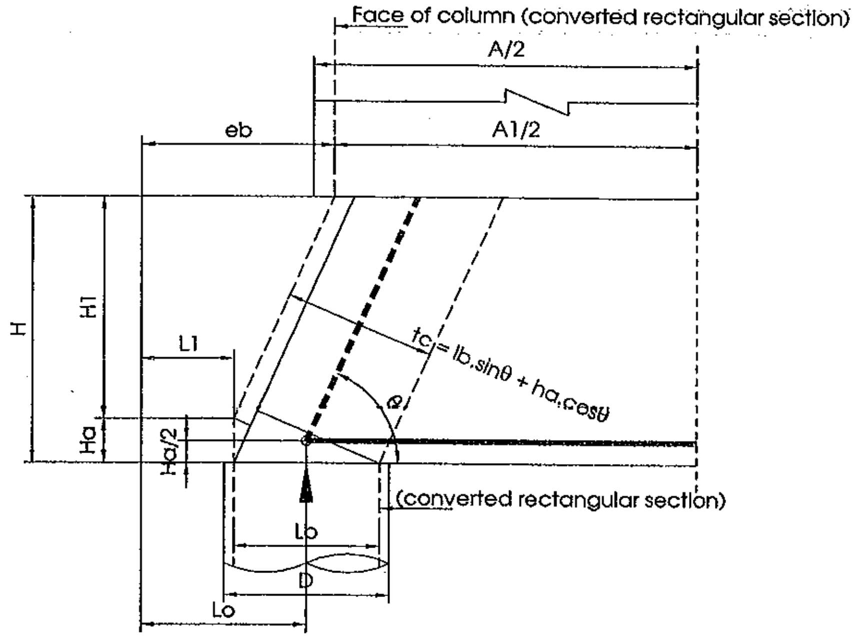
PILECAP DESIGN - STRUT AND TIE MODEL

I. PILECAP DATA

1. PileCap Material

Normal concrete			
Compressive strength at 28 days age	f'c	30	MPa
Concrete elastic modulus	Ec	27691	MPa
Reinforcement TCVN1651-2008; CBV-400			
Yield strength	fy	400	MPa
Reinforcement elastic modulus	Es	200,000	MPa



2. Strut and tie - Checking

Side		Beside	Front	Unit
Height of pilecap	Hb	2.00	2.00	m
Pilecap width in direction	Wpc	12.60	12.60	m
Pile diameter	D	1.20	1.20	m
Distance from C.L of outer pile to face of pilecap	L0	1.20	1.20	m
Column width in direction	W	1.80	1.80	m
Distance from design section to face of pilecap	e0	3.70	3.50	m
Distance from C.L of outer pile to design section	Arm	2.50	2.30	m
Pilecap should be design as strut and tie model? Yes or No		No	No	
Converted rectangular of pile	Lb	1.20	1.20	m
Column section area	Acol	0.00	0.00	m2
Converted rectangular of colum in direction	Wb	0.00	0.00	m
Dist. from converted rect. of column to face of pilecap	eb	3.70	3.50	m
Dist. from converted rect. of pile to face of pilecap	L1	0.60	0.60	m
Dist. from C.L of tension rebars to bottom of pilecap	Ha/2	0.166	0.166	m
Distance from top of tenision tie to to of pilecap	H1	1.67	1.67	m
Inclined angle of compression strut	θ	28.28	29.91	deg
Reaction of outer piles row	R_ULS	5848	16922	kN
	R_SLS	4364	12174	kN

Compression strut Checking				
Strut dimension — Thickness	tc	0.86	0.89	m
Width: Wcs = np . Lb	Wcs	9.60	8.40	m
Number of pile in outer row	np	8.00	7.00	piles
Area of strut section: Acs = tc.Wcs	Acs	8.27	7.44	m2
Resitance factor	ϕ	0.70	0.70	
Compression force in strut - C = T / cosθ		12342	33939	kN
Strain in tension tie	εs	0.0005	0.0014	
ε1 = εs + (εs + 0.002). Cotgθ	ε1	0.0052	0.0074	
Limiting compressive stress	fcu	17.78	14.61	MPa
Rebar area of compressive strut	Ass	0	0	m2
Resistance of compressive strut: Cr = ϕ.[fcu.Acs + fy. Ass]	Cr	102882	76140	kN
Conclusion		**Ok**	**Ok**	
Tension tie Checking				
Tension tie force - T = R / tanθ	T_ULS	10868	29420	kN
	T_SLS	8110	21166	kN
Resitance factor	ϕ	0.90	0.90	
Resistance of tension Tie: Pr = ϕ.fy.As				
Area of required rebars for tie	As ≥	30190	81723	mm2
Trial number of rebars				
Bottom 168 bars D28 @150	As	103488		mm2
Bottom 168 bars D28 @150	As		103488	mm2
Conclusion		**Ok**	**Ok**	
Node region Checking				
Limiting compressive stress	fcu	16	16	MPa
Compressive stress at node	P/Acs	0.71	2.27	kN/m2
Conclusion		**Ok**	**Ok**	

3. Control of cracking by distribution of reinforcement for Tension tie

Tensile stress in rebars should be satisfied equation: fs <= fsa = Z/[(dc.A)^1/3] and fs <= 0.6.fy				
Direction		Beside	Front	Unit
Existing condition for structrure	1,2 or 3	3	3	
Crack width parameter	Z	17500	17500	N/mm
Cross section equivalent — height	h	2.00	2.00	m
width	b	12.60	12.60	m
Concrete thickness from tension fiber to tension reinf.	dc	0.05	0.05	m
Concrete thickness from compression fiber to tension reinf.	d	1.83	1.80	m
Number of rebars	N	168	168	bars
Area of rebars	As	0.1035	0.1035	m2
Area of concrete assumed to participate with reinf.				
A = 2 . dc . b / N	A	0.0075	0.0075	m2
	fsa	243	243	MPa
	0.6fy	240	240	MPa
Min (fsa, 0.6fy) = fs1	fs1	240	240	MPa
Stress in rebars: fs = T_SLS / (As)	fs	78	205	MPa
	Conclude	**OK**	**OK**	
Maximum width of crack: an = 0.076.β.fs.(dc.A)^(1/3)	an	0.075	0.195	mm
Where	β	1.200	1.200	

	Da Nang Quang Ngai Expressway project ORB13 BRIDGE	Item.	Eng.	Date.	Sign.
		Design			
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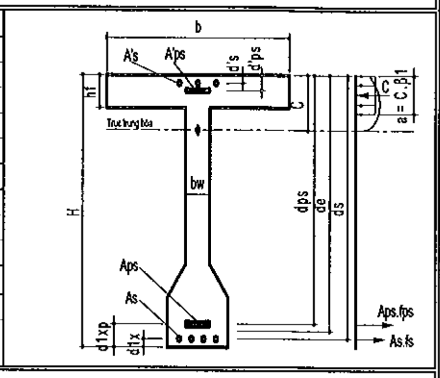
	ABUTMENT A1	Revise			

22TCN272-05; AASHTO LRFD 2nd - 1998

REINFORCEMENT CHECKING - HEAD AND STEM WALL

MATERIALS

	NORMAL CONCRETE		
f'c	Compressive Strength of concrete at 28 days	Mpa	30
Ec	Modulus of Elasticity	Mpa	27691
fr	Modulus of Rupture	Mpa	3.5
gc	Unit weight of concrete	kN/m3	24.5
	PRESTRESSING STEEL		
fpu	Tensile strength of prestressing steel	Mpa	1860
fpy	Yield strength of prestressing steel	Mpa	1670
Ep	Modulus of Elasticity	Mpa	195000
	REINFORCEMENT		
fy	Yield strength	Mpa	400
Es	Modulus of Elasticity	Mpa	200000
nc	Ratio Es/Ec		7



Sign	Parameters		Unit	Sections 1-1	1-1	2-2	2-2	2-2
	INTERNAL FORCES AT SECTIOIN							
	Combination			Strength	Service	Service	Strength	Extreme
Qu	Shear		kN	677	411	3820	5504	5255
Mu	Flexural Moment		kNm	526	289	15186	21022	15440
Nu	Axial load		kN	762	610	9101	12060	10520
Tu	Torsional Moment		kNm	0	0	0	0	0
	FLEXURAL MOMENT CHECKING							
H	Section height		m	0.500	0.500	1.800	1.800	1.800
d's	Dis. From comp. fiber to centroid of comp. Reinf		m	0.058	0.058	0.058	0.058	0.058
d1x	Dis. From tens. fiber to centroid of tension Reinf		m	0.058	0.058	0.059	0.059	0.059
	Cover to reinf		m	0.050	0.050	0.050	0.050	0.050
ds	Dis. From comp. fiber to centroid of tension Reinf		m	0.442	0.442	1.741	1.741	1.741
d'ps	Dis. From comp. fiber to centroid of comp. prestressing steel		m	0.000	0.000	0.000	0.000	0.000
d1xp	Dis. From tens. fiber to centroid of tension prestressing steel		m	0.000	0.000	0.000	0.000	0.000
dps	Dis. From comp. fiber to centroid of tension prestressing steel		m	0.500	0.500	1.800	1.800	1.800
b	Width of the compression face of member		m	12.600	12.600	12.600	12.600	12.600
bw	Web width or diameter of a circular section		m	12.600	12.600	12.600	12.600	12.600
hf	Compression flange depth		m	0.000	0.000	0.000	0.000	0.000
Iz	Moment of inertia of section		m4	0.131	0.131	6.124	6.124	6.124
Amc	Section area		m2	6.300	6.300	22.680	22.680	22.680
	Steel choice							
Aps	Tension prestressing steel	P.S type		0	0	0	0	0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
A'ps	Compression prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	0 T00.0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
As	Tension Reinforcement	Number	bars	84	84	78	78	78
		Diameter	mm	16	16	32	32	32
		Area	m2	0.01697	0.01697	0.06248	0.06248	0.06248
A's	Compression Reinforcement	Number	bars	84	84	78	78	78
		Diameter	mm	16	16	16	16	16
		Area	m2	0.01697	0.01697	0.01576	0.01576	0.01576
A'c	Shear reinforcement	Number	bars	20	20	19	19	19
		Diameter	mm	14	14	14	14	14
		Area	m2	0.00302	0.00302	0.00287	0.00287	0.00287
ϕ	Resistance factors for flexure		5.5.4.2	0.90	1.00	1.00	0.90	1.00
ϕv	Resistance factors for shear			0.90	1.00	1.00	0.90	1.00
ϕn	Resistance factors for axial force			1.00	1.00	1.00	1.00	1.00
$\beta 1$	Stress block factor			0.836	0.836	0.836	0.836	0.836
c	Dis. Between centroid and top fiber		m	0.000	0.000	0.070	0.070	0.070
	For T section behavior		m	0.000	0.000	0.070	0.070	0.070
	For rectangular section behavior		m	0.000	0.000	0.070	0.070	0.070
fpe	Effective stress in the prestressing steel after losses		Mpa	1116	1116	1116	1116	1116
fps	Aver. stress in pres. steel at the time for which the nominal resistance		Mpa	1860	1860	1840	1840	1840
k	Factor depends on type of P.S, Low relaxation strand k = 0.28			0.28	0.28	0.28	0.28	0.28

REINFORCEMENT CHECKING - HEAD AND STEM WALL

a	Depth of equivalent stress block	m	0.000	0.000	0.058	0.058	0.058
de	Corresp. effective depth from extreme comp. fiber						
	to centroid of tensile force in the tensile reinf.	m	0.442	0.442	1.741	1.741	1.741
Mn	Nominal resistance	kNm	2606	2606	42601	42601	42601
Mr	Factored reistance	kNm	**2346**	**2606**	**42601**	**38341**	**42601**
Mu	Flexual moment	kNm	526	289	15186	21022	15440
(5.7.3.2)	**Flexural moment Checking**		**OK**	**OK**	**OK**	**OK**	**OK**
	Limits for reinforcement						
c/de	Maximum reinforcement		0.00	0.00	0.04	0.04	0.04
	Maximum reinforcement Checking	<= 0.42	**OK**	**OK**	**OK**	**OK**	**OK**
1.2*Mcr	Craking moment	kNm	1087	1087	14654	14654	14654
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**		**OK**	**OK**	**OK**	**OK**	**OK**
(5.7.3.4)	**Conctrol of craking by distr. of reinf for RC member- Check?**		No	Yes	Yes	No	No
	Existing condition for structrure	1,2 or 3	1	1	1	1	1
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.058	0.058	0.059	0.059	0.059
Z	Crack width parameter	N/mm	30000	30000	30000	30000	30000
A	Area of concr. with same centroid as tens. Reinf	m2	0.017	0.017	0.019	0.019	0.019
fsa	Value	Mpa	299	299	288	288	288
0.6*fy		Mpa	240	240	240	240	240
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	240	240	240	240	240
x	Dist. From compression fiber to centroid	m	-	0.082	0.315	-	-
J.d	Arm	m	-	0.415	1.636	-	-
Icr	Moment of inertia of the cracked section	m4	-	0.018	1.027	-	-
fs	Tensile stress in reinforcement fs = Msls / (As*J.d)	Mpa	-	41	149	-	-
	Checking for control cracking fs<fsa		**N.a**	**OK**	**OK**	**N.a**	**N.a**
(5.10.8.2)	**Shrinkage and temperature Reinfocement (side distribution)**						
Areq	Area of required reinf	m2	0.00045	0.00045	0.00127	0.00127	0.00127
	Distribution on sides 7 D16	m2	0.00141	0.00141	0.00141	0.00141	
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45	0.45
	Checking		**OK**	**OK**	**OK**	**OK**	**OK**

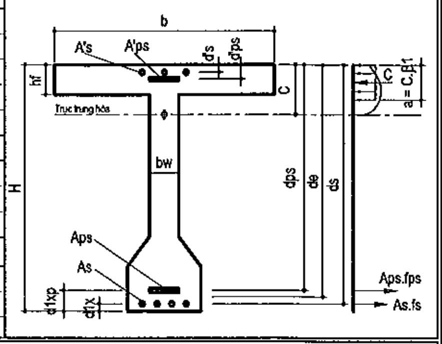
SHEAR AND TORSION CHECKING

β	Factor indicating diag. cracked concr. to tension		2.8	3.6	2.5	2.4	2.5
θ	Angle of inclination of diagonal compressive	degree	28.84	28.08	29.94	33.19	29.79
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	90
bv	Effective web width as minimum web width - in dv	m	12.600	12.600	12.600	12.600	12.600
dv	Effective shear depth	m	0.442	0.442	1.712	1.712	1.712
	(de - a/2)	m	0.442	0.442	1.712	1.712	1.712
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	0.600
ncat	Amount of bars in spacing S	bars	20	20	19	19	19
Av	Shear reinf area in spacing S	m2	0.0030	0.0030	0.0029	0.0029	0.0029
θ	Assume	degree	28.93	28.40	35.67	39.68	40.23
v	Shear stress in concrete	kN/m2	135	74	177	284	244
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e_x	Strain in tensile reinforcement		4.19E-04	2.15E-04	5.59E-04	7.66E-04	5.49E-04
	if ex<0, multiple with reduce factor		-	-	-	-	-
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok	Ok
v/f'c	Ratio of shear stress and f'c		0.005	0.002	0.006	0.009	0.008
β	Final value		2.8	3.6	2.5	2.4	2.5
θ	Final value	degree	28.84	28.08	29.94	33.19	29.79
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	7161	9189	24291	23155	24342
Vs	Shear resistance provided by shear reinforcement	kN	1616	1668	5685	5006	5719
Vp	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0	0
Vn1	Vn1=Vc+Vs+Vp	kN	8777	10857	29976	28161	30061
Vn2	Vn2	kN	41769	41769	161776	161776	161776
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	8777	10857	29976	28161	30061
Vr	Factored shear resistance	kN	**7899**	**10857**	**29976**	**25345**	**30061**
Vu	Shear	kN	677	411	3820	5504	5255
(5.8.2.7)	**Shear checking**		**OK**	**OK**	**OK**	**OK**	**OK**
	Region requiring transverse reinf Checking		No need	No need	No need	No need	No need
	Minimum shear reinf area	m2	0.0086	0.0086	0.0086	0.0086	0.0086
	Mimimum shear reinforcement Checking		-	-	-	-	-
	0.1*f'c*bv*dv	kN	16708	16708	64710	64710	64710
	Smax	m	0.35	0.35	0.60	0.60	0.60
	Maximum spacing Smax		-	-	-	-	-

REINFORCEMENT CHECKING - PILECAP SECTION

MATERIALS

	NORMAL CONCRETE		
f'c	Compressive Strength of concrete at 28 days	Mpa	30
Ec	Modulus of Elasticity	Mpa	27691
fr	Modulus of Rupture	Mpa	3.5
gc	Unit weight of concrete	kN/m3	24.5
	PRESTRESSING STEEL		
fpu	Tensile strength of prestressing steel	Mpa	1860
fpy	Yield strength of prestressing steel	Mpa	1670
Ep	Modulus of Elasticity	Mpa	195000
	REINFORCEMENT		
fy	Yield strength	Mpa	400
Es	Modulus of Elasticity	Mpa	200000
nc	Ratio Es/Ec		7



Sign	Parameters		Unit	3-3	3-3	3-3	4-4	4-4
				Sections				
	INTERNAL FORCES AT SECTIOIN							
	Combination			Service	Strength	Extreme	Extreme	Strength
Qu	Shear		kN	7724	11102	6662	6244	8697
Mu	Flexural Moment		kNm	25310	36365	23319	13193	20809
Nu	Axial load		kN	2149	3169	2162	1438	2112
Tu	Torsional Moment		kNm	0	0	0	0	0
	FLEXURAL MOMENT CHECKING							
H	Section height		m	2.000	2.000	2.000	2.000	2.000
d's	Dis. From comp. fiber to centroid of comp. Reinf		m	0.084	0.084	0.084	0.166	0.166
d1x	Dis. From tens. fiber to centroid of tension Reinf		m	0.166	0.166	0.166	0.084	0.084
	Cover to reinf		m	0.050	0.050	0.050	0.050	0.050
ds	Dis. From comp. fiber to centroid of tension Reinf		m	1.834	1.834	1.834	1.916	1.916
d'ps	Dis. From comp. fiber to centroid of comp. prestressing steel		m	0.000	0.000	0.000	0.000	0.000
d1xp	Dis. From tens. fiber to centroid of tension prestressing steel		m	0.000	0.000	0.000	0.000	0.000
dps	Dis. From comp. fiber to centroid of tension prestressing steel		m	2.000	2.000	2.000	2.000	2.000
b	Width of the compression face of member		m	12.600	12.600	12.600	12.600	12.600
bw	Web width or diameter of a circular section		m	12.600	12.600	12.600	12.600	12.600
hf	Compression flange depth		m	0.000	0.000	0.000	0.000	0.000
Iz	Moment of inertia of section		m4	8.400	8.400	8.400	8.400	8.400
Amc	Section area		m2	25.200	25.200	25.200	25.200	25.200
	Steel choice							
Aps	Tension prestressing steel	P.S type		0	0	0	0	0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
A'ps	Compression prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	0 T00.0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
As	Tension Reinforcement	Number	bars	84	84	84	84	84
		Diameter	mm	32	32	32	28	28
		Area	m2	0.06728	0.06728	0.06728	0.05174	0.05174
A's	Compression Reinforcement	Number	bars	0	0	0	0	0
		Diameter	mm	28	28	28	28	28
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
A'c	Shear reinforcement	Number	bars	20	20	20	20	20
		Diameter	mm	16	16	16	16	16
		Area	m2	0.00404	0.00404	0.00404	0.00404	0.00404
ϕ	Resistance factors for flexure		5.5.4.2	1.00	0.90	1.00	1.00	0.90
ϕv	Resistance factors for shear			1.00	0.90	1.00	1.00	0.90
ϕn	Resistance factors for axial force			1.00	1.00	1.00	1.00	1.00
β1	Stress block factor			0.836	0.836	0.836	0.836	0.836
c	Dis. Between centroid and top fiber		m	0.100	0.100	0.100	0.077	0.077
	For T section behavior		m	0.100	0.100	0.100	0.077	0.077
	For rectangular section behavior		m	0.100	0.100	0.100	0.077	0.077
fpe	Effective stress in the prestressing steel after losses		Mpa	1116	1116	1116	1116	1116
fps	Aver. stress in pres. steel at the time for which the nominal resistanc		Mpa	1833	1833	1833	1840	1840
k	Factor depends on type of P.S, Low relaxation strand k = 0.28			0.28	0.28	0.28	0.28	0.28

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REINFORCEMENT CHECKING - PILECAP SECTION

a	Depth of equivalent stress block	m	0.084	0.084	0.084	0.064	0.064
de	Corresp. effective depth from extreme comp. fiber						
	to centroid of tensile force in the tensile reinf.	m	1.834	1.834	1.834	1.916	1.916
Mn	Nominal resistance	kNm	48232	48232	48232	38990	38990
Mr	Factored reistance	kNm	**48232**	**43409**	**48232**	**38990**	**35091**
Mu	Flexual moment	kNm	25310	36365	23319	13193	20809
(5.7.3.2)	**Flexural moment Checking**		**OK**	**OK**	**OK**	**OK**	**OK**
	Limits for reinforcement						
c/de	Maximum reinforcement		0.05	0.05	0.05	0.04	0.04
	Maximum reinforcement Checking	<= 0.42	**OK**	**OK**	**OK**	**OK**	**OK**
1.2*Mcr	Craking moment	kNm	18309	18309	18309	18088	18088
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**		**OK**	**OK**	**OK**	**OK**	**OK**
(5.7.3.4)	**Conctrol of craking by distr. of reinf for RC member- Check?**		Yes	No	No	No	No
	Existing condition for structrure	1,2 or 3	3	3	3	3	3
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.066	0.066	0.066	0.064	0.064
Z	Crack width parameter	N/mm	17500	17500	17500	17500	17500
A	Area of concr. with same centroid as tens. Reinf	m2	0.020	0.020	0.020	0.019	0.019
fsa	Value	Mpa	160	160	160	163	163
0.6*fy		Mpa	240	240	240	240	240
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	160	160	160	163	163
x	Dist. From compression fiber to centroid	m	0.335	-	-	-	-
J.d	Arm	m	1.722	-	-	-	-
Icr	Moment of inertia of the cracked section	m4	1.216	-	-	-	-
fs	Tensile stress in reinforcement fs = Msls / (As*J.d)	Mpa	218	-	-	-	-
(5.10.8.2)	**Shrinkage and temperature Reinfocement (side distribution)**						
Areq	Area of required reinf	m2	0.00127	0.00127	0.00127	0.00127	0.00127
	Distribution on sides 7 D16	m2	0.00141	0.00141	0.00141	0.00141	0.00141
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45	0.45
	Checking		**OK**	**OK**	**OK**	**OK**	**OK**

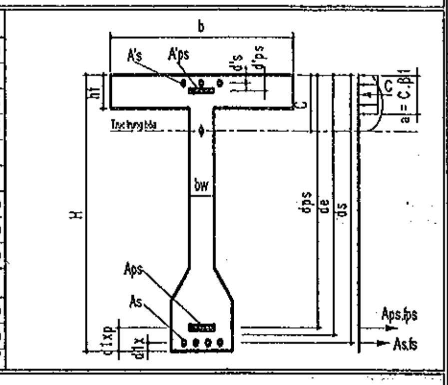
SHEAR AND TORSION CHECKING

β	Factor indicating diag. cracked concr. to tension		2.1	1.8	2.1	2.2	2.0
θ	Angle of inclination of diagonal compressive	degree	39.21	42.37	38.03	36.21	40.59
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	90
bv	Effective web width as minimum web width - in dv	m	12.600	12.600	12.600	12.600	12.600
dv	Effective shear depth	m	1.792	1.792	1.792	1.884	1.884
	(de - a/2)	m	1.792	1.792	1.792	1.884	1.884
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	0.600
ncat	Amount of bars in spacing S	bars	20	20	20	20	20
Av	Shear reinf area in spacing S	m2	0.0040	0.0040	0.0040	0.0040	0.0040
θ	Assume	degree	39.22	42.37	38.03	36.09	40.41
v	Shear stress in concrete	kN/m2	342	546	295	33	407
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e_x	Strain in tensile reinforcement		1.32E-03	1.84E-03	1.20E-03	1.02E-03	1.46E-03
	if ex<0, multiple with reduce factor		-	-	-	-	-
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok	Ok
v/f'c	Ratio of shear stress and f'c		0.011	0.018	0.010	0.001	0.014
β	Final value		2.1	1.8	2.1	2.2	2.0
θ	Final value	degree	39.21	42.37	38.03	36.21	40.59
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	21045	18401	21724	23936	21290
Vs	Shear resistance provided by shear reinforcement	kN	5915	5292	6171	6930	5922
Vp	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0	0
Vn1	Vn1=Vc+Vs+Vp	kN	26960	23692	27895	30865	27212
Vn2	Vn2	kN	169355	169355	169355	178018	178018
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	26960	23692	27895	30865	27212
Vr	Factored shear resistance	kN	**26960**	**21323**	**27895**	**30865**	**24490**
Vu	Shear	kN	7724	11102	6662	6244	8697
(5.8.2.7)	**Shear checking**		**OK**	**OK**	**OK**	**OK**	**OK**

REINFORCEMENT CHECKING - WING WALL

MATERIALS

	NORMAL CONCRETE		
f'c	Compressive Strength of concrete at 28 days	Mpa	30
Ec	Modulus of Elasticity	Mpa	27691
fr	Modulus of Rupture	Mpa	3.5
gc	Unit weight of concrete	kN/m3	24.5
	PRESTRESSING STEEL		
fpu	Tensile strength of prestressing steel	Mpa	1860
fpy	Yield strength of prestressing steel	Mpa	1670
Ep	Modulus of Elasticity	Mpa	195000
	REINFORCEMENT		
fy	Yield strength	Mpa	400
Es	Modulus of Elasticity	Mpa	200000
nc	Ratio Es/Ec		7



Sign	Parameters		Unit	Sections				
				5-5	5-5	6-6	6-6	6-6
	INTERNAL FORCES AT SECTIOIN							
	Combination			Service	Strength	Service	Strength	Strength
Qu	Shear		kN	128	196	326	500	500
Mu	Flexural Moment		kNm	362	565	398	611	611
Nu	Axial load		kN	0	0	0	0	0
Tu	Torsional Moment		kNm	0	0	0	0	0
	FLEXURAL MOMENT CHECKING							
H	Section height		m	0.800	0.800	0.800	0.800	0.800
d's	Dis. From comp. fiber to centroid of comp. Reinf		m	0.058	0.058	0.058	0.058	0.058
d1x	Dis. From tens. fiber to centroid of tension Reinf		m	0.059	0.059	0.059	0.059	0.059
	Cover to reinf		m	0.050	0.050	0.050	0.050	0.050
ds	Dis. From comp. fiber to centroid of tension Reinf		m	0.741	0.741	0.741	0.741	0.741
d'ps	Dis. From comp. fiber to centroid of comp. prestressing steel		m	0.000	0.000	0.000	0.000	0.000
d1xp	Dis. From tens. fiber to centroid of tension prestressing steel		m	0.000	0.000	0.000	0.000	0.000
dps	Dis. From comp. fiber to centroid of tension prestressing steel		m	0.800	0.800	0.800	0.800	0.800
b	Width of the compression face of member		m	1.000	1.000	1.000	1.000	1.000
bw	Web width or diameter of a circular section		m	1.000	1.000	1.000	1.000	1.000
hf	Compression flange depth		m	0.000	0.000	0.000	0.000	0.000
Iz	Moment of inertia of section		m4	0.043	0.043	0.043	0.043	0.043
Amc	Section area		m2	0.800	0.800	0.800	0.800	0.800
	Steel choice							
Aps	Tension prestressing steel	P.S type		0	0	0	0	0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
A'ps	Compression prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	0 T00.0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
As	Tension Reinforcement	Number	bars	7	7	7	7	7
		Diameter	mm	25	25	25	25	25
		Area	m2	0.00344	0.00344	0.00344	0.00344	0.00344
A's	Compression Reinforcement	Number	bars	7	7	7	7	7
		Diameter	mm	16	16	16	16	16
		Area	m2	0.00141	0.00141	0.00141	0.00141	0.00141
A'c	Shear reinforcement	Number	bars	2	2	2	2	2
		Diameter	mm	12	12	12	12	12
		Area	m2	0.00023	0.00023	0.00023	0.00023	0.00023
ϕ	Resistance factors for flexure		5.5.4.2	1.00	0.90	1.00	0.90	0.90
ϕv	Resistance factors for shear			1.00	0.90	1.00	0.90	0.90
ϕn	Resistance factors for axial force			1.00	1.00	1.00	1.00	1.00
$\beta 1$	Stress block factor			0.836	0.836	0.836	0.836	0.836
c	Dis. Between centroid and top fiber		m	0.038	0.038	0.038	0.038	0.038
	For T section behavior		m	0.038	0.038	0.038	0.038	0.038
	For rectangular section behavior		m	0.038	0.038	0.038	0.038	0.038
fpe	Effective stress in the prestressing steel after losses		Mpa	1116	1116	1116	1116	1116
fps	Aver. stress in pres. steel at the time for which the nominal resistance		Mpa	1835	1835	1835	1835	1835
k	Factor depends on type of P.S, Low relaxation strand k = 0.28			0.28	0.28	0.28	0.28	0.28

	Da Nang Quang Ngai Expressway project ORB13 BRIDGE DETAIL DESIGN ABUTMENT A1	Item.	Eng.	Date.	Sign.
		Design			
		Check			
		Revise			

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	REINFORCEMENT CHECKING - WING WALL						
a	Depth of equivalent stress block	m	0.032	0.032	0.032	0.032	0.032
de	Corresp. effective depth from extreme comp. fiber						
	to centroid of tensile force in the tensile reinf.	m	0.741	0.741	0.741	0.741	0.741
Mn	Nominal resistance	kNm	973	973	973	973	973
Mr	Factored reistance	kNm	973	876	973	876	876
Mu	Flexual moment	kNm	362	565	398	611	611
(5.7.3.2)	**Flexural moment Checking**		**OK**	**OK**	**OK**	**OK**	**OK**
	Limits for reinforcement						
c/de	Maximum reinforcement		0.05	0.05	0.05	0.05	0.05
	Maximum reinforcement Checking	<= 0.42	**OK**	**OK**	**OK**	**OK**	**OK**
r min	Mimimum reinforcement		0.43%	0.43%	0.43%	0.43%	0.43%
	Minimum reinforcement Checking for RC	0.23%	**OK**	**OK**	**OK**	**OK**	**OK**
1.2*Mcr	Craking moment	kNm	232	232	232	232	232
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**		**OK**	**OK**	**OK**	**OK**	**OK**
(5.7.3.4)	**Conctrol of craking by distr. of reinf for RC member- Check?**		Yes	No	Yes	No	No
	Existing condition for structrure	1,2 or 3	1	1	1	1	1
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.059	0.059	0.059	0.059	0.059
Z	Crack width parameter	N/mm	30000	30000	30000	30000	30000
A	Area of concr. with same centroid as tens. Reinf	m2	0.017	0.017	0.017	0.017	0.017
fsa	Value	Mpa	301	301	301	301	301
0.6*fy		Mpa	240	240	240	240	240
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	240	240	240	240	240
x	Dist. From compression fiber to centroid	m	0.166	-	0.166	-	-
J.d	Arm	m	0.686	-	0.686	-	-
Icr	Moment of inertia of the cracked section	m4	0.01	-	0.01	-	-
fs	Tensile stress in reinforcement fs = Msls / (As*J.d)	Mpa	154	-	169	-	-
	Checking for control cracking fs<fsa		**OK**	**N.a**	**OK**	**N.a**	**N.a**
(5.10.8.2)	**Shrinkage and temperature Reinfocement (side distribution)**						
Areq	Area of required reinf	m2	0.00042	0.00042	0.00042	0.00042	0.00042
	Distribution on sides 7 D16	m2	0.00141	0.00141	0.00141	0.00141	0.00141
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45	0.45
	Checking		**OK**	**OK**	**OK**	**OK**	**OK**
	SHEAR AND TORSION CHECKING						
β	Factor indicating diag. cracked concr. to tension		2.3	2.1	2.2	1.9	1.9
θ	Angle of inclination of diagonal compressive	degree	34.30	39.04	37.05	41.53	41.53
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	90
bv	Effective web width as minimum web width - in dv	m	1.000	1.000	1.000	1.000	1.000
dv	Effective shear depth	m	0.725	0.725	0.725	0.725	0.725
	(de - a/2)	m	0.725	0.725	0.725	0.725	0.725
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	0.600
ncat	Amount of bars in spacing S	bars	2	2	2	2	2
Av	Shear reinf area in spacing S	m2	0.0002	0.0002	0.0002	0.0002	0.0002
β	Assume		2.0	2.0	2.0	2.0	2.0
θ	Assume	degree	35.13	39.92	37.72	41.87	41.86
v	Shear stress in concrete	kN/m2	177	300	450	766	766
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e_x	Strain in tensile reinforcement		8.59E-04	1.30E-03	1.11E-03	1.63E-03	1.63E-03
	if ex<0, multiple with reduce factor		-	-	-	-	-
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok	Ok
v/f'c	Ratio of shear stress and f'c		0.006	0.010	0.015	0.026	0.026
β	Final value		2.3	2.1	2.2	1.9	1.9
θ	Final value	degree	34.30	39.04	37.05	41.53	41.53
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	761	679	716	623	623
Vs	Shear resistance provided by shear reinforcement	kN	160	135	145	123	123
Vp	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0	0
Vn1	Vn1=Vc+Vs+Vp	kN	922	814	861	746	746
Vn2	Vn2	kN	5439	5439	5439	5439	5439
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	922	814	861	746	746
Vr	Factored shear resistance	kN	922	732	861	672	672
Vu	Shear	kN	128	196	326	500	500
(5.8.2.7)	**Shear checking**		**OK**	**OK**	**OK**	**OK**	**OK**

f.BORED PILE DESIGN

I. BORED PILE DATA

1.Load Combinations at top of bored pile

No	Combinations	Sign	F_V (kN)	Longitudinal F_{HX} (kN)	Longitudinal My (kN•m)	Transvesal F_{HY} (kN)	Transvesal Mx (kN•m)
1	Strength 1a		5221	1015	-2994	0	-9
2	Service Ser-I		3779	692	-2046	6	10
3	Extreme Ext-IA		3655	692	-2083	26	68
4							
5							
6							

2. Bored pile Material

Normal concrete			
Compressive strength at 28 days age	f'c	30	MPa
Concrete elastic modulus	Ec	27691	MPa
Reinforcement TCVN1651-2008; CBV-400			
Yield strength	fy	400	MPa
Reinforcement elastic modulus	Es	200,000	MPa

3. Bored pile Section

Pile diameter	D	1.20	m
Section area	A	1.131	m2
Moment inertia	Ix	0.102	m4
	Iy	0.102	m4
Radius of gyration of gross concrete section; r = sqrt(I/A)	rx	0.300	m
	ry	0.300	m

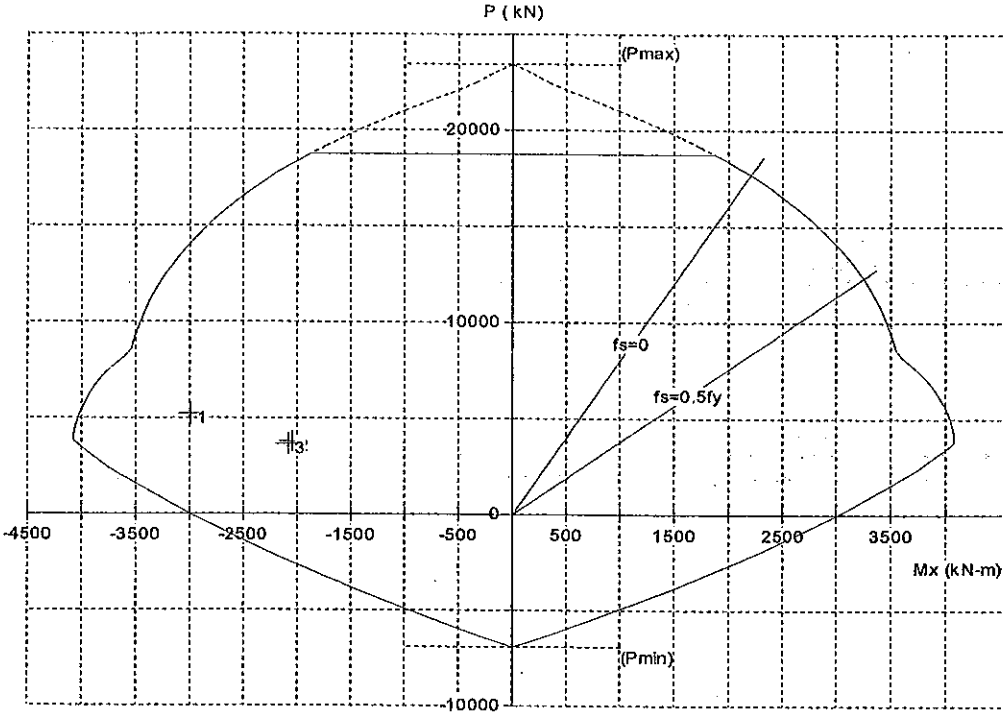
II. PILE DESIGN

1. Limit of Reinforcement S.5.7.4.2

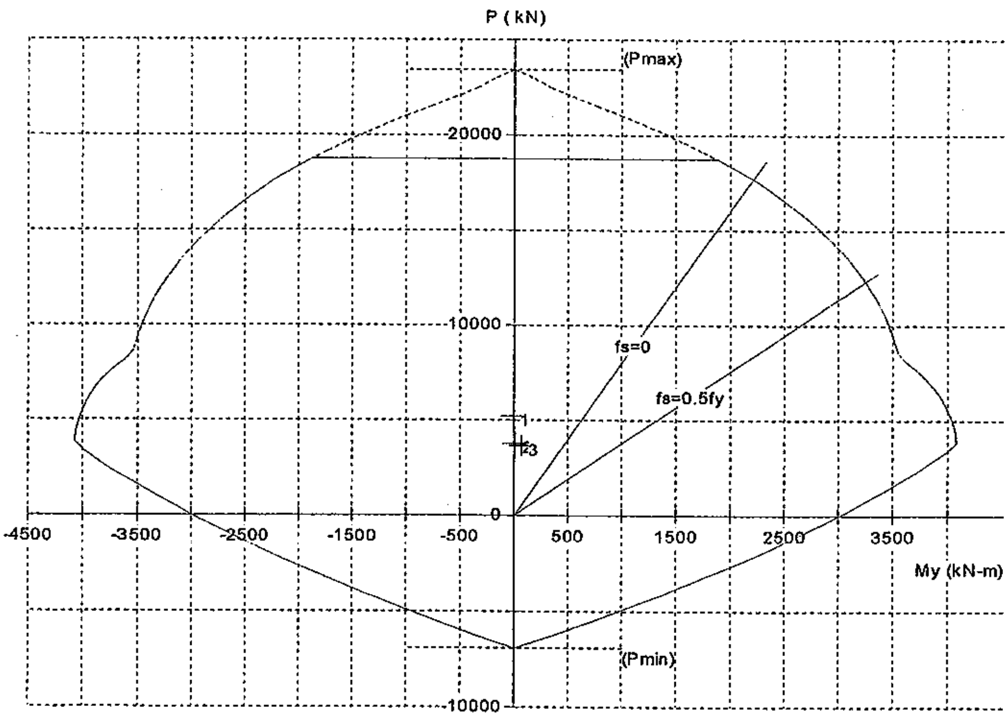
Minimum area of longitudinal reinforcement in column			
As.fy / (Ag . f'c) >= 0.135	As ≥	0.011	m2
As / Ag >= 0.01	As ≥	0.011	m2
Maximum area of longitudinal reinforcement in column			
As / Ag <= 0.08	As ≤	0.090	m2
Trial Rebars: **Ok**	As	**0.019**	m2
1layers x 24 = 24 bars D32 @150	As1	0.019	m2

2. Interation diagram M-P Using Pca-Column software

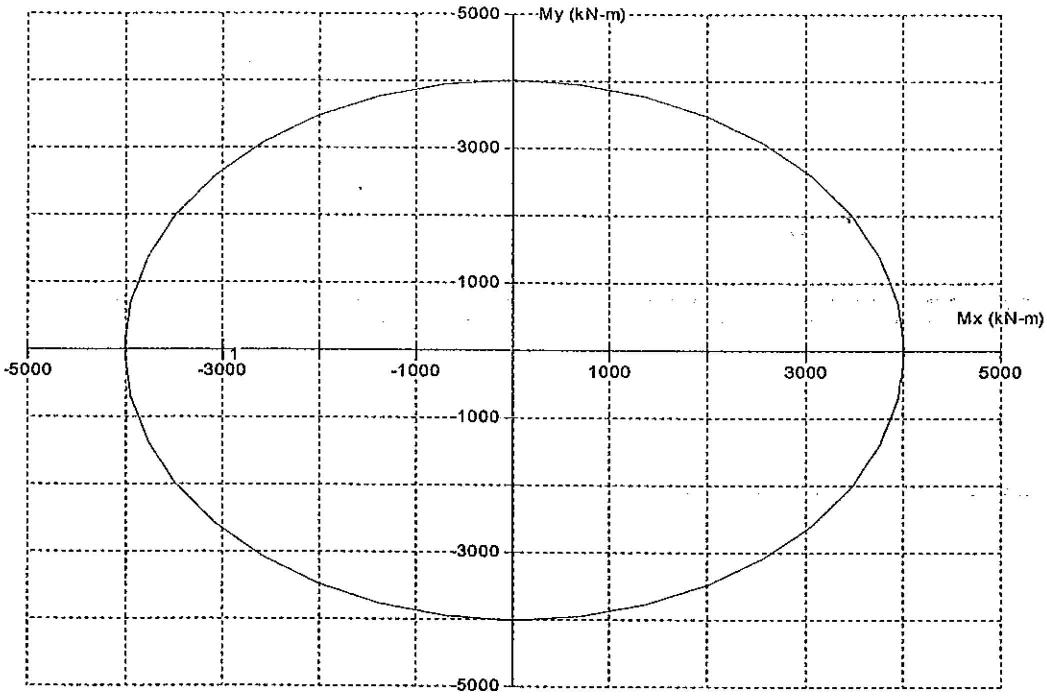
****In Transverse Direction**



****In Longitudinal Direction**



In Both Direction



P = 5224 kN

3. Column Ties S.5.7.4.6, S.5.10.6.3, S5.10.11.4.1d - e

Bridge is in seismic zone	Sz	1	
Area of concrete core measured out-to-out of ties	Ac	0.933	m2
Tie diameter	Dtie	14	mm2
Cross seotion area of 1 tie	As-tr	0.00015	m2
Spacing of hoops	s	75	mm
Length of reinfocement tie in 1 hoop	Ltie	3.41	m
Ratio of ties reinf. in one hoop/ volume of conc. core one pitch spacing			
ρs = As-tr . Ltie / (Ac * spacing)	ρs	0.0074	
Ratio of spiral reinf. To total volume of concrete core shall satisfy			S.5.7.4.6
ρs >= 0.45 . (Ag/Ac - 1). f'c/ fy = Req1	Req1	0.0072	**OK**
Transverse Reinforcement for Confinement at Plastic Hinges		S.5.10.11.4.1.d	
For a circular column "1:applied","2:Not applied"		1	
ρs >= 0.12 . f'c/ fy = Req2	Req2	0.0090	**N/A**
Length distributed spiral with pitch 75mm below pilecap	Ldis	1.80	m

4. Shear Design

Shear resistance factors	ϕv	1.0	
Factored shear force	Vu	1015	kN
Required shear capacity Vn = Vu/ ϕv	Vn	1015	kN
Determine concrete shear capacity			
Minimum shear reinforcement will provided in cross section			
Therefore	β	2.0	
	θ	45.0	deg
Diameter of bored pile	D	1.20	m
Width of cross section	b	1.20	m
dv = 0.9*de de = D/2 + Dr/pi()			
Diameter of the circle passing through the centers of the long. reinf.	Dr	0.99	m
	de	0.92	m
	dv	0.82	m
Vc = 0.083 . β . sqrt(f'c) . bv. dv	Vc	900	kN

Difference between required shear capacity and the capacity provided by concrete			
is the minimum required capacity for shear reinforcements			
Vs = Vn - Vc	Vs	116	kN
In this case Vc > Vn so shear reinforcement is no need			
Stirrup diameter	Ds	14	
Number of stirrup legs / cross section	ns	2	
Shear legs area	Av	0.0003	m2
Angle of inclination of shear reinf. to long. axis	α	90	deg
Vs = Av. fy. dv. (cot θ + cot α). sin α / s	s <=	0.00	m
Stirrup spacing used	s	0.10	m
Check minimum shear reinforcement requirement		**OK**	
Av >= 0.083. sqrt(f'c). bv. s/ fy = Req	Req	0.0000	m2
Check maximum shear reinforcement spacing requirement		**OK**	
F = 0.1*f'c*bv*dv	F	3609	kN
If Vu < 0.1*f'c*bv*dv then Smax=0.8*dv <= 600mm			
If Vu > 0.1*f'c*bv*dv then Smax=0.4*dv <= 300mm	Smax	0.60	m

SPACE PILE FOUNDATION ANALYSIS PROGRM
Turbo BASIC

PROJECT: : ORB13-A1

INITIA DATA

Kn = 0.00 Ax = 9.00 By = 12.60 Cz = 2.00
E v.uon = 2822779 E r.uon = 2822779 E v.nen = 2822779 E r.nen = 2822779
Mq = 0 (t/m4) Md = 0 (t/m4) m = 500 (t/m4)

LOAD COMBINATIONS

COMB.	Hx	Hy	P	Mx	My	Mz
1	828.00	0.00	3088.00	-1060.00	804.00	0.00
2	547.00	0.00	2250.00	-848.00	525.00	0.00
3	803.00	6.00	3017.00	-1052.00	683.00	0.00
4	522.00	6.00	2179.00	-839.00	404.00	0.00
5	564.00	5.00	2340.00	-830.00	432.00	0.00
6	563.00	22.00	2866.00	-986.00	-959.00	0.00
7	563.00	22.00	2028.00	-774.00	-2.00	0.00

PROPERTIES OF PILES

PILE	Lo	H	Bpx	Bpy	A	B	Cday	Fo	Io	Po	Co	Ct
1	0.00	7.00	1.732	1.732	1.20	0.000	0.000	1.131	0.102	0	6250000	3125000
2					n	t						
3					n	t						
4					n	t						
5					n	t						
6					n	t						
7					n	t						
8					n	t						

PILE COORD.

PILE	X	Y	Phi	Xi
1	3.30	5.10	0.000	0.00
2	3.30	0.00	0.000	0.00
3	3.30	-5.10	0.000	0.00
4	0.00	5.10	0.000	0.00
5	0.00	0.00	0.000	0.00
6	0.00	-5.10	0.000	0.00
7	-3.30	4.10	0.000	0.00
8	-3.30	-4.10	0.000	0.00

FORCES ON PILES

PILE	COMB.	N	Q2	Q3	M1	M2	M3
1	1	532.23	-103.50	0.00	0.000	0.952	305.182
	2	378.33	-68.38	0.00	0.000	0.762	201.826
	3	514.83	-100.43	-0.72	0.031	-1.183	296.390
	4	360.90	-65.31	-0.72	0.031	-1.374	193.033
	5	385.24	-70.55	-0.60	0.026	-1.028	208.517

	6	372.59	-70.59	-2.63	0.114	-6.917	212.358
	7	329.59	-70.59	-2.63	0.114	-7.107	209.358
2	1	493.24	-103.50	0.00	0.000	0.952	305.182
	2	347.14	-68.38	0.00	0.000	0.762	201.826
	3	475.47	-100.38	-0.72	0.031	-1.183	296.212
	4	329.38	-65.25	-0.72	0.031	-1.374	192.855
	5	354.16	-70.50	-0.60	0.026	-1.028	208.369
	6	333.91	-70.38	-2.63	0.114	-6.917	211.704
	7	298.70	-70.38	-2.63	0.114	-7.107	208.705
3	1	454.25	-103.50	0.00	0.000	0.952	305.182
	2	315.95	-68.38	0.00	0.000	0.762	201.826
	3	436.12	-100.32	-0.72	0.031	-1.183	296.034
	4	297.86	-65.19	-0.72	0.031	-1.374	192.677
	5	323.08	-70.45	-0.60	0.026	-1.028	208.220
	6	295.22	-70.16	-2.63	0.114	-6.917	211.051
	7	267.81	-70.16	-2.63	0.114	-7.107	208.051
4	1	409.67	-103.50	0.00	0.000	0.952	305.182
	2	303.03	-68.38	0.00	0.000	0.762	201.826
	3	402.43	-100.43	-0.75	0.031	-1.298	296.390
	4	295.75	-65.31	-0.75	0.031	-1.490	193.033
	5	314.77	-70.55	-0.63	0.026	-1.124	208.517
	6	400.42	-70.59	-2.77	0.114	-7.340	212.358
	7	277.93	-70.59	-2.77	0.114	-7.530	209.358
5	1	370.68	-103.50	0.00	0.000	0.952	305.182
	2	271.84	-68.38	0.00	0.000	0.762	201.826
	3	363.08	-100.38	-0.75	0.031	-1.298	296.212
	4	264.23	-65.25	-0.75	0.031	-1.490	192.855
	5	283.69	-70.50	-0.63	0.026	-1.124	208.369
	6	361.73	-70.38	-2.77	0.114	-7.340	211.704
	7	247.04	-70.38	-2.77	0.114	-7.530	208.705
6	1	331.69	-103.50	0.00	0.000	0.952	305.182
	2	240.65	-68.38	0.00	0.000	0.762	201.826
	3	323.72	-100.32	-0.75	0.031	-1.298	296.034
	4	232.71	-65.19	-0.75	0.031	-1.490	192.677
	5	252.61	-70.45	-0.63	0.026	-1.124	208.220
	6	323.04	-70.16	-2.77	0.114	-7.340	211.051
	7	216.15	-70.16	-2.77	0.114	-7.530	208.051
7	1	279.47	-103.50	0.00	0.000	0.952	305.182
	2	221.61	-68.38	0.00	0.000	0.762	201.826
	3	282.32	-100.42	-0.79	0.031	-1.414	296.355
	4	224.43	-65.30	-0.79	0.031	-1.605	192.998
	5	238.21	-70.54	-0.66	0.026	-1.220	208.488
	6	420.65	-70.55	-2.91	0.114	-7.763	212.230
	7	220.22	-70.55	-2.91	0.114	-7.953	209.230
8	1	216.78	-103.50	0.00	0.000	0.952	305.182
	2	171.46	-68.38	0.00	0.000	0.762	201.826
	3	219.04	-100.33	-0.79	0.031	-1.414	296.069
	4	173.75	-65.20	-0.79	0.031	-1.605	192.712
	5	188.23	-70.46	-0.66	0.026	-1.220	208.249
	6	358.45	-70.20	-2.91	0.114	-7.763	211.179
	7	170.55	-70.20	-2.91	0.114	-7.953	208.179

SUMMARY OF FORCES

	PILE	COMB.	N	Q2	Q3	M1	M2	M3
Nmin	8	7	170.55	-70.20	-2.91	0.114	-7.953	208.179
Nmax	1	1	532.23	-103.50	0.00	0.000	0.952	305.182
Q2max	1	1	532.23	-103.50	0.00	0.000	0.952	305.182
Q3max	7	6	420.65	-70.55	-2.91	0.114	-7.763	212.230
M1max	1	6	372.59	-70.59	-2.63	0.114	-6.917	212.358
M2max	7	7	220.22	-70.55	-2.91	0.114	-7.953	209.230
M3max	1	1	532.23	-103.50	0.00	0.000	0.952	305.182

CHECKING CALCULATI

IN COMPARISON WITH INITIA LOAD MATRIX

1	828.00	0.00	3088.00	-1060.00	804.00	0.00
2	547.00	0.00	2250.00	-848.00	525.00	0.00
3	803.00	6.00	3017.00	-1052.00	683.00	0.00
4	522.00	6.00	2179.00	-839.00	404.00	0.00
5	564.00	5.00	2340.00	-830.00	432.00	0.00
6	563.00	22.00	2866.00	-986.00	-959.00	0.00
7	563.00	22.00	2028.00	-774.00	-2.00	0.00

CALCULATION SHEET

PIER P1

a.STRUCTURE DIMENSIONS & LOAD COMPONENTS

I. GENERAL DATA

Assumptions :

1.The Design of the Pier conforms to "Specification for bridge design 22-TCN-272-05" and AASHTO LRFD 2004, JIS,... for reference.
2.Design live load: HL-93 and lane loading 9.3 KN/m
3.Bridge is considered to be in seismic with acceleration coefficient A = 0.0301 g

Input data:

Bridge type	***Simple PC I girder L=30m with link slab***						
Span length	Left	=	27.00	Right	=	27.00	m
Girder length between bearings	Left	=	26.10	Right	=	26.10	m
Bridge width	B	=	12.74	m			

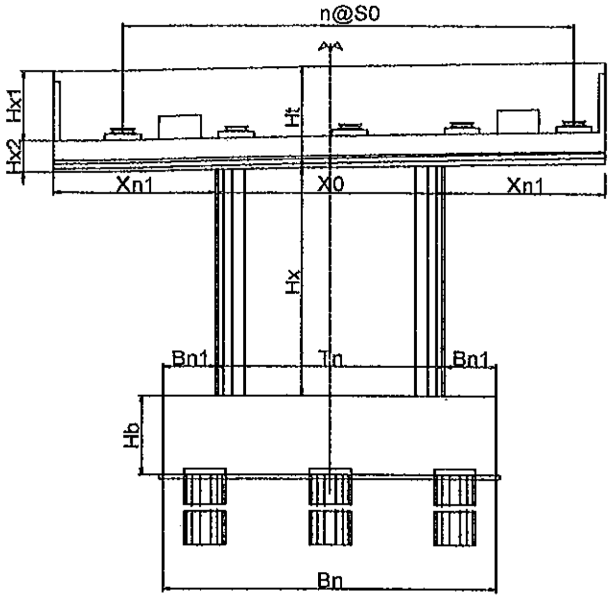
Level Table(at center of pier)

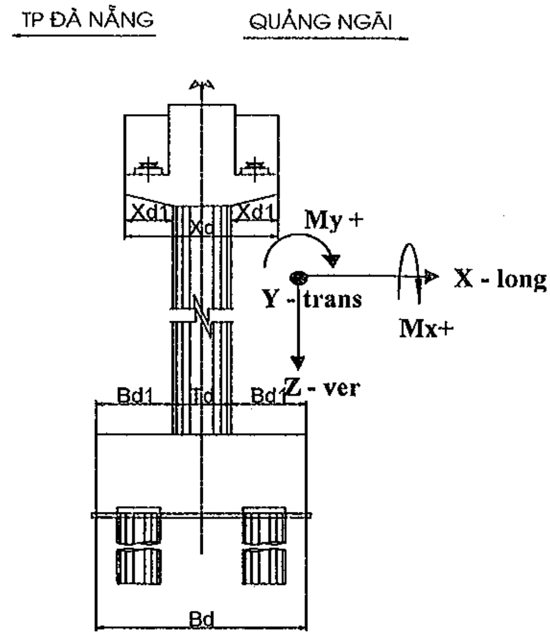
Top of pier cap	ThL	19.558	m
Top of pier column	TcL	16.973	m
Bottom of upper pier column	BucL	16.973	m
Bottom of pier column	BcL	6.000	m
Bottom of upper pilecap	BupL	6.000	m
Bottom of pilecap	BpL	4.000	m
Tip of pile	TpL	-6.000	m
Skew angle	Ska	90.000	deg
Ground level	GL	7.500	m
Maximum water level (H1%)	HWL	13.300	m
Navigation water level (H5%)	NWL	9.500	m
Minimum water level	MWL	9.500	m
Average Annual water level	AWL	9.500	m
Local scour level (at water level H1%)	LsL	9.500	m

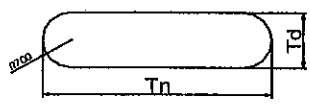
Material unit weight

Structural concrete	γc =	2500	kg/m3	24.50	kN/m3
Asphalt concrete	γa =	2250	kg/m3	22.10	kN/m3
Soil - ground	γ_s =	1800	kg/m3	17.70	kN/m3
Saturated soil	γ_{ss} =	800	kg/m3	7.80	kN/m3

II. PIER DIMENSIONS







Pier Dimensions Table

Notation	Dimensions Transverse direction	Value (m)	Notation	Dimensions Longitudinal direction	Value (m)
*	***Bearing distribution***			Dist. from bearing to pier c.line	
nbear	Number of bearing	5.00	e1	Girder 1	5.10
S0	Bearings spacing	2.55	e2	Girder 2	2.55
			e3	Girder 3	0.00
			e4	Girder 4	-2.55
			e5	Girder 5	-5.10
*	***Bearing pedestal*** Width	0.75	*	***Anchorage block*** Width	0.40
	Length	0.65		Length	1.00
	Height	0.15		Height	0.52
	Number	10.00		Number	4.00
*	***Pier Cap***				
Hx1	Haunch 1 height	1.59	xd	Pier cap width	1.40
Hx2	Haunch 2 height	1.00	GL	Left bearing to pier c.line	1.300
Hx	Pier cap height	2.59	GR	Right bearing to pier c.line	1.300
Xn1	Haunch width	3.49			
Xn0	Bottom of pier cap width	5.50	Hc	Curtain wall height	1.50
Xnt	Top of pier cap width	12.48	tc	Curtain wall thickness	0.15
*	***Pier Column***				
tn	Pier column width	5.50	td	Pier column thichness	1.40
Htt	Pier column height	10.97	Rv	Round nose radius	0.70
tnb	Upper pier column width	0.00	tdb	Upper pier column thichness	0.00
Htb	Upper pier column height	0.00	Rvb	Upper round nose radius	0.00
Ht	Column height	10.97			

Notation	Dimensions Transverse direction	Value (m)	Notation	Dimensions Longitudinal direction	Value (m)
*	***Pile Cap***				
Bn	Pile cap width	8.00	Bd	Pile cap length	6.00
Hb	Pile cap depth	2.00			
Bn1	Transverse cantilever	1.25	bd1	Long. Cantilever	2.30
Bnb	Upper pile cap width	0.00	Bdb	Upper pile cap length	0.00
Hbb	Upper pile cap depth	0.00			

III. SUBSTRUCTURE LOADS

1.Pier Selfweight

Item	Volume	Vertical	Longitudinal			Transversal		
		F_V	F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m3)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Bearing pedestal	0.73	17.9						
Bearing devices		50.0						
Anchorage block	0.84	20.5						
Pier Cap	67.57	1655.5						
Curtain wall	0.45	11.0						
Upper pier column	0.00	0.0						
Pier Column	79.88	1957.0						
Upper pilecap	0.00	0.0						
PileCap	96.00	2352.0						
Shear key	0.00	0.0						
Total at bottom of Column		***3712.0***						
Total at bottom of pilecap		***6064.0***						

2.Soil on pilecap

Item	Volume	Vertical	Longitudinal			Transversal		
		F_V	F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m3)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Soil on pile cap	61.08	1081.1						
Total at bottom of Column								
Total at bottom of pilecap		***1081.1***						

3.Buoyancy on pier

Case1: Maximum water level (H1%)

Item	Volume	Vertical	Longitudinal			Transversal		
		F_V	F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m3)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Upper pier column	0.00	0.0						
Pier Column	53.14	-521.3						
Upper pilecap	0.00	0.0						
PileCap	96.00	-941.8						
Shear key	0.00	0.0						
Total at bottom of Column		***-521.3***						
Total at bottom of pilecap		***-1463.1***						

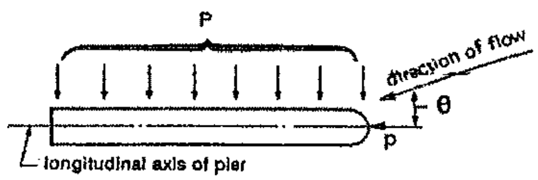
Case2: Minimum water level (Hmin)

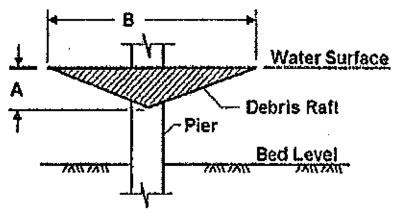
Item	Volume	Vertical	Longitudinal			Transversal		
		F_V	F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m3)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Upper pier column	0.00	0.0						
Pier Column	25.48	-249.9						
Upper pilecap	0.00	0.0						
PileCap	96.00	-941.8						
Shear key	0.00	0.0						
Total at bottom of Column		***-249.9***						
Total at bottom of pilecap		***-1191.7***						

Case3: average Annual water level

Item	Volume	Vertical	Longitudinal			Transversal		
		F_V	F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m3)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Upper pier column	0.00	0.0						
Pier Column	25.48	-249.9						
Upper pilecap	0.00	0.0						
PileCap	96.00	-941.8						
Shear key	0.00	0.0						
Total at bottom of Column		***-249.9***						
Total at bottom of pilecap		***-1191.7***						

4.Stream Pressure





Stream pressure data

Angle between direction of flow and long. axis of pier	θ	0.0	deg
Design velocity of water at H1%	V1%	1.11	m/s
Design velocity of water at minimum water level	Vmin	0.39	m/s
Design velocity of water at average annual water level	Vannual	1.11	m/s

Longitudinal axis of pier $pL = 5.14 \times 10^{-4} C_D V^2$

Type of pier colum: "1:Semicircular - nosed pier"; "2:Square - ended pier";		1	
"3:Debris lodged against the pier "; "4:Wedged - nosed pier with nose angle 90deg or less"			
Drag coefficient	C_D	0.70	
Stream pressure at H1%	pL1%	0.44	kN/m2
Stream pressure at minimum water level	pLmin	0.05	kN/m2
Stream pressure at average annual water level	pLannual	0.44	kN/m2
Additional stream pressure due to driftwood raft lodged against pier			
Drag coefficient	C_D	0.50	
Height of debris raft	A	2.9	m
Width of debris raft	B	14.0	m
Area of debris raft	Area	20.3	m2
Stream pressure due to driftwood raft at H1%	pLdebris	0.32	kN/m2
Equivalent force	Fhdebris	6.4	kN

Lateral axis of pier $pT = 5.14 \times 10^{-4} C_L V^2$

Drag coefficient	C_L	0.00	
Stream pressure at H1%	pT1%	0.00	kN/m2
Stream pressure at minimum water level	pTmin	0.00	kN/m2
Stream pressure at average annual water level	pTannual	0.00	kN/m2

Case1: Maximum water level (H1%)

Item	Exposed height	Vertical F_V	Longitudinal			Transversal		
			F_{HX}	Arm.$_{HX}$	M_y	F_{HY}	Arm.$_{HY}$	M_x
	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Upper pier Column	0.00		0.0	11.0	0.0	0.0	11.0	0.0
Pier Column	7.30		0.0	3.7	0.0	4.5	3.7	16.5
Upper pilecap	0.00		0.0	0.0	0.0	0.0	0.0	0.0
Total at bottom of Column			***0.0***		***0.0***	***4.5***		***16.5***
Upper pier Column	0.00		0.0	13.0	0.0	0.0	13.0	0.0
Pier Column	7.30		0.0	5.7	0.0	4.5	5.7	25.6
Upper pilecap	0.00		0.0	2.0	0.0	0.0	2.0	0.0
PileCap	0.00		0.0	0.0	0.0	0.0	0.0	0.0
Total at bottom of pilecap			***0.0***		***0.0***	***4.5***		***25.6***

Additional stream pressure due to driftwood raft

Item	Exposed height	Vertical F_V	Longitudinal			Transversal		
			F_{HX}	Arm.$_{HX}$	M_y	F_{HY}	Arm.$_{HY}$	M_x
	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Pier Column					0.0	6.4	7.3	46.9
Total at bottom of Column					***0.0***	***6.4***		***46.9***
Pier Column					0.0	6.4	9.3	59.8
Total at bottom of pilecap					***0.0***	***6.4***		***59.8***

Case2: Minimum water level (Hmin)

Item	Exposed height	Vertical F_V	Longitudinal			Transversal		
			F_{HX}	Arm.$_{HX}$	M_y	F_{HY}	Arm.$_{HY}$	M_x
	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Upper pier Column	0.00		0.0	11.0	0.0	0.0	11.0	0.0
Pier Column	3.50		0.0	1.8	0.0	0.3	1.8	0.5
Upper pilecap	0.00		0.0	0.0	0.0	0.0	0.0	0.0
Total at bottom of Column			***0.0***		***0.0***	***0.3***		***0.5***
Upper pier Column	0.00		0.0	13.0	0.0	0.0	13.0	0.0
Pier Column	3.50		0.0	3.8	0.0	0.3	3.8	1.0
Upper pilecap	0.00		0.0	2.0	0.0	0.0	2.0	0.0
PileCap	0.00		0.0	0.0	0.0	0.0	0.0	0.0
Total at bottom of pilecap			***0.0***		***0.0***	***0.3***		***1.0***

Case3: average Annual water level

Item	Exposed height	Vertical F_V	Longitudinal			Transversal		
			F_{HX}	Arm.$_{HX}$	M_y	F_{HY}	Arm.$_{HY}$	M_x
	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Upper pier Column	0.00		0.0	11.0	0.0	0.0	11.0	0.0
Pier Column	3.50		0.0	1.8	0.0	2.2	1.8	3.8
Upper pilecap	0.00		0.0	0.0	0.0	0.0	0.0	0.0
Total at bottom of Column			***0.0***		***0.0***	***2.2***		***3.8***
Upper pier Column	0.00		0.0	13.0	0.0	0.0	13.0	0.0
Pier Column	3.50		0.0	3.8	0.0	2.2	3.8	8.1
Upper pilecap	0.00		0.0	2.0	0.0	0.0	2.0	0.0
PileCap	0.00		0.0	0.0	0.0	0.0	0.0	0.0
Total at bottom of pilecap			***0.0***		***0.0***	***2.2***		***8.1***

5.Wind Loads

Wind loads data		$P_D = 0.0006\ V^2\ C_d \geq 1.8$ (kN/m2)		
Reference standard "Wind region TCVN 2737-1995"				
Wind region (I, II, III, IV) at bridge location			**III**	
3 second gust wind velocity with 100 years return period		Vb	53.0	m/s
Type of terrain at bridge location			1	
"1:exposed area"; "2: forest, houses,...with height 10m"; "3:houses area..with height>10m"				
Average elevation of pier upper ground or water plane level		Hele_p	9.6	m
Correct coefficient for wind zone and elevation of pier		S	1.09	
Design wind speed V = S*Vb		V	57.8	m/s
Obstacle coefficient for pier		Cd	1.3	
Wind pressure on pier		P_D	2.60	kN/m2

At Maximum water level (H1%)

Item	Exposed height	Vertical F_V	Longitudinal			Transversal		
			F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Curtain wall	1.50		1.2	14.3	16.8	5.5	14.3	78.2
Pier Cap	2.59		84.0	12.3	1030.1	9.4	12.3	115.6
Upper pier Column	0.00		0.0	7.3	0.0	0.0	7.3	0.0
Pier Column	3.67		52.6	9.1	480.5	13.4	9.1	122.3
Upper pilecap	0.00		0.0	7.3	0.0	0.0	7.3	0.0
Total at bottom of Column			***137.7***		***1527.3***	***28.3***		***316.1***
Curtain wall	1.50		1.2	16.3	19.1	5.5	16.3	89.1
Pier Cap	2.59		84.0	14.3	1198.0	9.4	14.3	134.4
Upper pier Column	0.00		0.0	9.3	0.0	0.0	9.3	0.0
Pier Column	3.67		52.6	11.1	585.6	13.4	11.1	149.1
Upper pilecap	0.00		0.0	9.3	0.0	0.0	9.3	0.0
PileCap	0.00		0.0	9.3	0.0	0.0	9.3	0.0
Total at bottom of pilecap			***137.7***		***1802.8***	***28.3***		***372.6***

At Minimum water level (Hmin)

Item	Exposed height	Vertical F_V	Longitudinal			Transversal		
			F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Curtain wall	1.50		1.2	14.3	16.8	5.5	14.3	78.2
Pier Cap	2.59		84.0	12.3	1030.1	9.4	12.3	115.6
Upper pier Column	0.00		0.0	3.5	0.0	0.0	3.5	0.0
Pier Column	7.47		107.0	7.2	774.3	27.2	7.2	197.1
Upper pilecap	0.00		0.0	3.5	0.0	0.0	3.5	0.0
Total at bottom of Column			***192.1***		***1821.1***	***42.1***		***390.9***
Curtain wall	1.50		1.2	16.3	19.1	5.5	16.3	89.1
Pier Cap	2.59		84.0	14.3	1198.0	9.4	14.3	134.4
Upper pier Column	0.00		0.0	5.5	0.0	0.0	5.5	0.0
Pier Column	7.47		107.0	9.2	988.2	27.2	9.2	251.6
Upper pilecap	0.00		0.0	5.5	0.0	0.0	5.5	0.0
PileCap	0.00		0.0	5.5	0.0	0.0	5.5	0.0
Total at bottom of pilecap			***192.1***		***2205.4***	***42.1***		***475.1***

At average Annual water level

Item	Exposed height	Vertical F_V	Longitudinal			Transversal		
			F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Curtain wall	1.50		1.2	14.3	16.8	5.5	14.3	78.2
Pier Cap	2.59		84.0	12.3	1030.1	9.4	12.3	115.6
Upper pier Column	0.00		0.0	3.5	0.0	0.0	3.5	0.0
Pier Column	7.47		107.0	7.2	774.3	27.2	7.2	197.1
Upper pilecap	0.00		0.0	3.5	0.0	0.0	3.5	0.0
Total at bottom of Column			***192.1***		***1821.1***	***42.1***		***390.9***
Curtain wall	1.50		1.2	16.3	19.1	5.5	16.3	89.1
Pier Cap	2.59		84.0	14.3	1198.0	9.4	14.3	134.4
Upper pier Column	0.00		0.0	5.5	0.0	0.0	5.5	0.0
Pier Column	7.47		107.0	9.2	988.2	27.2	9.2	251.6
Upper pilecap	0.00		0.0	5.5	0.0	0.0	5.5	0.0
PileCap	0.00		0.0	5.5	0.0	0.0	5.5	0.0
Total at bottom of pilecap			***192.1***		***2205.4***	***42.1***		***475.1***

IV. SUPERSTRUCTURE LOADS

1.Dead Loads

Left side Span

Item	Volume	Vertical F_V	Longitudinal			Transversal		
			F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m3)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Stage1 (DC)								
Girders	94.85	1161.9		1.300	-1510.5			
Diaphragm	11.28	138.2		1.300	-179.6			
Precast plank	15.69	192.2		1.300	-249.8			
Deck slab	79.86	978.3		1.300	-1271.8			
Total at bottom of Column		***2470.5***			***-3211.7***			
Total at bottom of pilecap		***2470.5***			***-3211.7***			
Stage2 (DW)								
Pavement	26.65	294.5		1.300	-382.8			
Parapet + railing		320.0		1.300	-415.9			
Lighting post + mis.		27.0		1.300	-35.1			
Total at bottom of Column		***641.4***			***-833.8***			
Total at bottom of pilecap		***641.4***			***-833.8***			

Right side Span

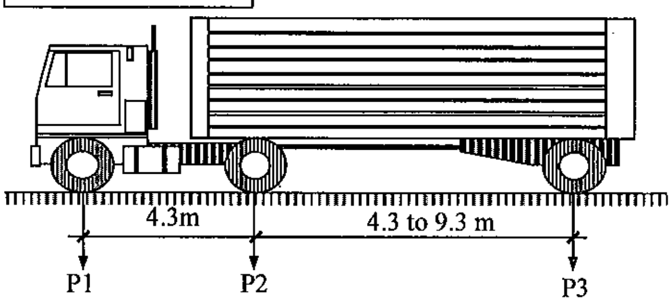
Item	Volume	Vertical	Longitudinal			Transversal		
		F_V	F_{HX}	Arm.$_{HX}$	M_y	F_{HY}	Arm.$_{HY}$	M_x
	(m3)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Stage1 (DC)								
Girders	94.85	1161.9		1.300	1510.5			
Diaphragm	11.28	138.2		1.300	179.6			
Precast plank	15.69	192.2		1.300	249.8			
Deck slab	79.86	978.3		1.300	1271.8			
Total at bottom of Column		***2470.5***			***3211.7***			
Total at bottom of pilecap		***2470.5***			***3211.7***			
Stage2 (DW)								
Pavement	26.65	294.5		1.300	382.8			
Parapet + railing		320.0		1.300	415.9			
Lighting post + mis.		27.0		1.300	35.1			
Total at bottom of Column		***641.4***			***833.8***			
Total at bottom of pilecap		***641.4***			***833.8***			

2.Live Load

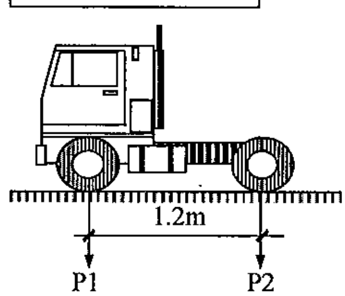
Live load data

Design Truck	P1	35.0	kN
	P2	145.0	kN
	P3	145.0	kN
Design Tandem	P1	110.0	kN
	P2	110.0	kN
Design Lane Load	P_L	9.3	kN/m
Pedestrian Load	Pp	3.0	kN/m2
Sidewalk width - both 2 sides	sw	0.0	m
Maximum number of design lane	nlanes	3.0	lanes
Multiple presence factor	m	0.85	
Dynamic load allowance (1+IM)			
Deck joint - all limit states		1.75	
Other structure - all limit states (except fatigue)		1.25	

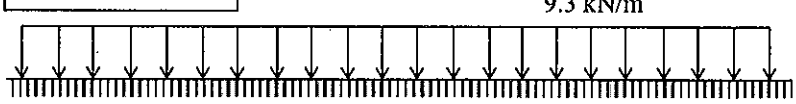
Design Truck



Design Tandem

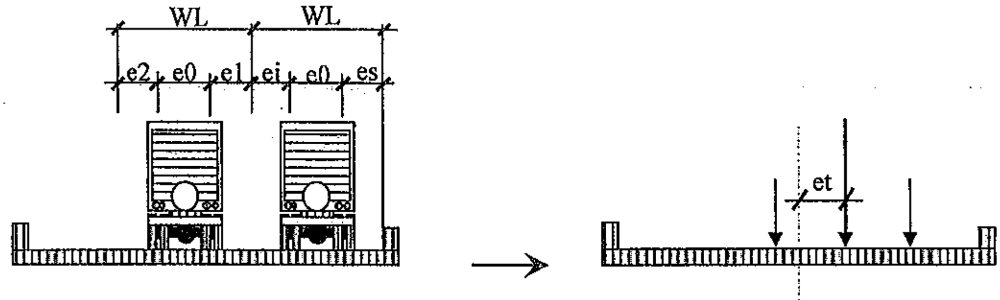


Design Lane Load



Transverse Distribution

Distance from wheel axis to inner face of curb and bewteen wheel axis						
In general case	ei	1.20	m	es	0.60	m
For deck overhang design	ei	1.50	m	es	0.30	m
Distance between wheels				e0	1.80	m
Design lane width				WL	3.60	m
				e1	0.00	m
				e2	1.80	m
Curb width				wc	0.50	m
Transverse excentricity of design vehicle 1 - general case				ex1	4.37	
Transverse excentricity of design vehicle 2				ex2	1.37	
Transverse excentricity of design vehicle 3				ex3	-1.63	
Transverse excentricity of design vehicle 4				ex4	-4.63	
Transverse Excentricity of design vehicle				et	-0.13	m



Longitudinal Distribution

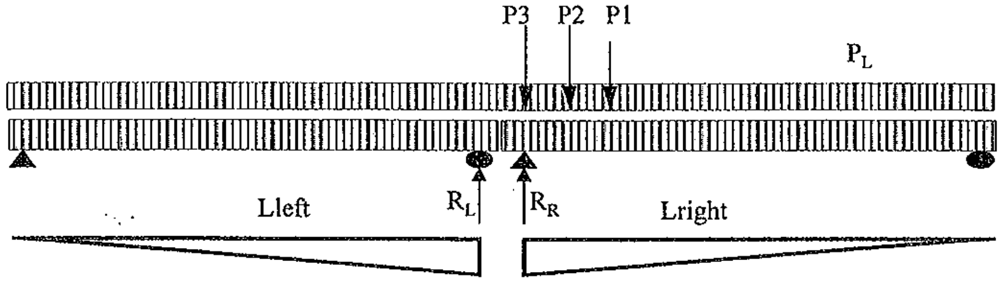
Case1a: 1Truck + Lane load on 1 span
Case1b: 1Truck + Lane load on 2 spans
Case2a: 2Trucks + Lane load on 1 span
Case2b: 2Trucks + Lane load on 2 spans
Case3a: 1Tandem + Lane load on 1 span
Case3b: 1Tandem + Lane load on 2 spans

Influence line value

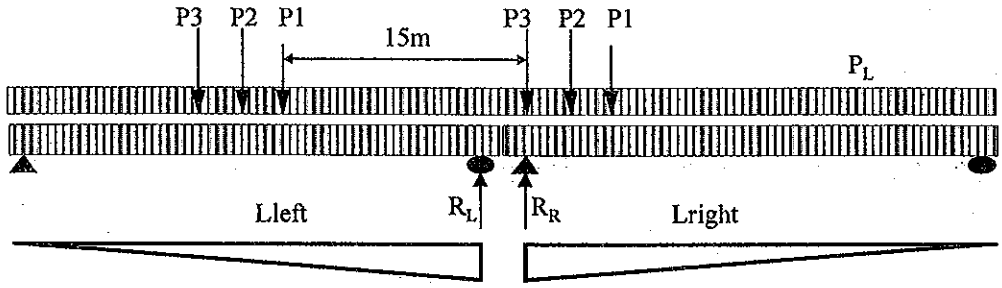
Case	Left span				Right span			
	P3	P2	P1	Aleft	P3	P2	P1	Aright
Case1a:				13.05	1.00	0.84	0.67	13.05
Case1b:	0.93			13.05		1.00	0.84	13.05
Case2a:*	1.00	0.84	0.67	13.05	0.10	-0.07	0.00	13.05
Case2b:	0.20	0.36	0.52	13.05	1.00	0.84	0.67	13.05
Case3a:				13.05		1.00	0.95	13.05
Case3b:		1.05		13.05			1.00	13.05

* 2 Trucks in right span

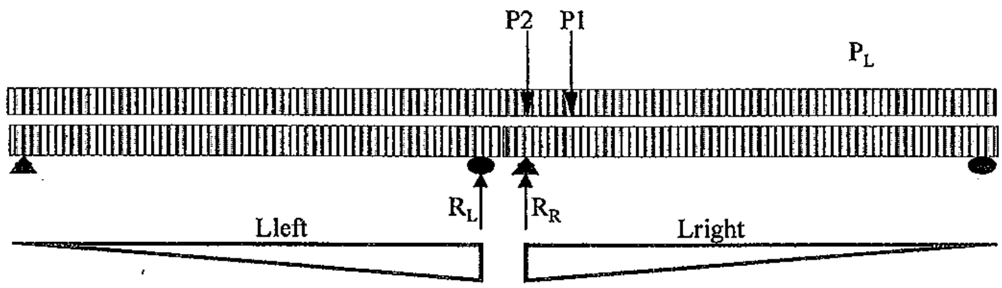
Truck + Lane load



2Trucks + Lane load



Tandem + Lane load



For 1 truck or tandem: Reaction = [(Pi* yi)*(1+IM) + PL*A]*nlane*m

For 2 trucks: Reaction = 0.9* [(Pi* yi)*(1+IM) + PL*A]*nlane*m

1 Loaded Lane m = 1.20

Item	Reaction left	Reaction right	Vertical F_V	Longitudinal F_{HX}	Longitudinal $Arm._{HX}$	Longitudinal M_y	Transversal F_{HY}	Transversal $Arm._{HY}$	Transversal M_x
	(kN)	(kN)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Case1a:	145.6	580.0	725.6			564.7			3171.1
Case1b:	349.0	407.0	756.0			75.4			3303.5
Case2a:	131.1	527.3	658.3			515.0			2876.9
Case2b:	264.6	522.0	786.6			334.6			3437.6
Case3a:	145.6	468.1	613.7			419.1			2681.8
Case3b:	319.5	310.6	630.1			-11.5			2753.7

2 Loaded Lane m = 1.00

Item	Reaction left	Reaction right	Vertical F_V	Longitudinal F_{HX}	Longitudinal $Arm._{HX}$	Longitudinal M_y	Transversal F_{HY}	Transversal $Arm._{HY}$	Transversal M_x
	(kN)	(kN)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Case1a:	242.7	966.7	1209.4			941.1			3471.0
Case1b:	581.6	678.3	1259.9			125.7			3616.0
Case2a:	218.5	878.8	1097.2			858.4			3149.0
Case2b:	441.0	870.0	1311.1			557.7			3762.7
Case3a:	242.7	780.1	1022.8			698.6			2935.5
Case3b:	532.5	517.7	1050.2			-19.2			3014.1

3 Loaded Lane m = 0.85

Item	Reaction left	Reaction right	Vertical F_V	Longitudinal F_{HX}	Longitudinal $Arm._{HX}$	Longitudinal M_y	Transversal F_{HY}	Transversal $Arm._{HY}$	Transversal M_x
	(kN)	(kN)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Case1a:	309.5	1232.5	1542.0			1199.9			2112.5
Case1b:	741.6	864.9	1606.4			160.3			2200.8
Case2a:	278.5	1120.4	1399.0			1094.5			1916.6
Case2b:	562.3	1109.3	1671.6			711.0			2290.1
Case3a:	309.5	994.6	1304.1			890.7			1786.6
Case3b:	678.9	660.1	1339.0			-24.4			1834.5

4 Loaded Lane m = 0.65

Item	Reaction left	Reaction right	Vertical F_V	Longitudinal F_{HX}	Longitudinal $Arm._{HX}$	Longitudinal M_y	Transversal F_{HY}	Transversal $Arm._{HY}$	Transversal M_x
	(kN)	(kN)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Case1a:	315.5	1256.7	1572.2			1223.5			1115.5
Case1b:	756.1	881.8	1637.9			163.4			1162.2
Case2a:	284.0	1142.4	1426.4			1115.9			1012.1
Case2b:	573.4	1131.0	1704.4			725.0			1209.3
Case3a:	315.5	1014.1	1329.7			908.1			943.4
Case3b:	692.2	673.0	1365.3			-24.9			968.7

Item Live Load	Vertical F_V	Longitudinal F_{HX}	Longitudinal $Arm._{HX}$	Longitudinal M_y	Transversal F_{HY}	Transversal $Arm._{HY}$	Transversal M_x
	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Total at bottom of Column	*1311.1*	*0.0*	*0.0*	*557.7*	*0.0*	*0.0*	*3762.7*
Total at bottom of pilecap	*1311.1*	*0.0*	*0.0*	*557.7*	*0.0*	*0.0*	*3762.7*

Pedestrian Load

Item	Reaction left	Reaction right	Vertical F_V	Longitudinal F_{HX}	Longitudinal $Arm._{HX}$	Longitudinal M_y	Transversal F_{HY}	Transversal $Arm._{HY}$	Transversal M_x
	(kN)	(kN)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
1side	0.0	0.0	0.0			0.0		0.0	
2side	0.0	0.0	0.0			0.0		0.0	

3.Centrifugal Force

Centrifugal force data CE = n * m * (Axle weights) * C

Axle weights of design Truck	P	325.0	kN
Number of loaded lanes	n	4.0	lanes
	m	0.65	
Factor, C = (4/3)* V^2/ (g*R)	C	0.0	kN
Highway design speed	V	33.3	m/s
Radius of curvature of traffic lane	R	-	m
Centrifugal force	CE	0.0	kN

Item	From surface	Vertical F_V	Longitudinal F_{HX}	Longitudinal $Arm._{HX}$	Longitudinal M_y	Transversal F_{HY}	Transversal $Arm._{HY}$	Transversal M_x
	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Centrifugal force	1.80							
Total at bottom of Column						*0.0*	15.669	*0.0*
Total at bottom of pilecap						*0.0*	17.669	*0.0*

4.Braking Force

Braking force data

Axle weights of design Truck	P	325.0	kN
Number of loaded lanes	n	4.0	lanes
	m	0.65	
Br1 = 25%*(design truck)*n*m	Br1	211.25	kN
Br2 = 5%*(design truck + 9.3*Lbridge)*n*m	Br2	107.54	kN
Br = max(Br1, Br2)	Br	211.25	kN

Item	From surface	Vertical F_V	Longitudinal F_{HX}	Longitudinal $Arm._{HX}$	Longitudinal M_y	Transversal F_{HY}	Transversal $Arm._{HY}$	Transversal M_x
Take 50 %	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Braking force	1.80							
Total at bottom of Column			***105.6***	15.669	***1655.0***			
Total at bottom of pilecap			***105.6***	17.669	***1866.3***			

5.Uniform Temperature

Uniform temperature data

Installing temperature	t0	27.0	deg
Maximum temperature	tmax	47.0	deg
Minimum temperature	tmin	10.00	deg
Plus temperature amplitude	Δtmax	20.0	deg
Minus temperature amplitude	Δtmin	17.0	deg
Coefficient of Thermal Expansion	α	1.08E-05	
Strain due to minus temperature	ε_T	1.84E-04	
Span length from longitudinal center of inter-span	Lsp	0.1	m
Displacement along girder	Δs	9.18E-06	m
Horizontal force applies to top of pier: Fhx =(3.E.I) . Δs / H^3	Fhx	0	kN
	E	27691	MPa
	I	1.126	m4

Item	From surface	Vertical F_V	Longitudinal F_{HX}	Longitudinal $Arm._{HX}$	Longitudinal M_y	Transversal F_{HY}	Transversal $Arm._{HY}$	Transversal M_x
	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Total at bottom of Column			***0.3***	13.71	***4.7***			
Total at bottom of pilecap			***0.3***	15.71	***5.4***			

6.Creep & Shrinkage

Creep & shrinkage data

Strain due to creep & shrinkage	ε_{CS}	1.73E-04	
Displacement along girder	Δs	8.64E-06	m
Horizontal force applies to top of pier	Fhx	0	kN

Item	From surface	Vertical F_V	Longitudinal F_{HX}	Longitudinal $Arm._{HX}$	Longitudinal M_y	Transversal F_{HY}	Transversal $Arm._{HY}$	Transversal M_x
	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Total at bottom of Column			***0.3***	13.71	***4.4***			
Total at bottom of pilecap			***0.3***	15.71	***5.1***			

7.Wind on Structure

Wind loads data	$P_D = 0.0006\ V^2\ C_d \geq 1.8$ (kN/m2)		
Average elevation of deck girder upper ground or water plane level	Hele_g	14.2	m
Correct coefficient for wind zone and elevation of pier	S	1.11	
Design wind speed V = S*Vb	V	58.9	m/s
Overall width between handrails	b	12.7	m
Superstructure height including solid parapet	d	3.03	m
	b/d	4.21	
Obstacle coefficient for pier	Cd	1.36	
Wind pressure on pier	P_D	2.83	kN/m2

Item	**Exposed**	**Vertical**	**Longitudinal**			**Transversal**		
	height	F_V	F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Superstructure	3.03		57.8	15.1	871.0	231.2	15.1	3484.0
Total at bottom of Column			***57.8***		***871.0***	***231.2***		***3484.0***
Superstructure	3.03		57.8	17.1	986.6	231.2	17.1	3946.4
Total at bottom of pilecap			***57.8***		***986.6***	***231.2***		***3946.4***

8.Wind on Vehicle

For strength combination III, wind load on vehicular and on structure have to simultaneously consider (wind speed 25m/s). Wind load on vehicular in tranversal direction, is 1.5 kN/m at 1.8m height from asphalt surface. Wind load on vehicular in longitudinal direction is 0.75 kN/m at 1.8m height from asphalt surface.

Item	**Vertical**	**Longitudinal**			**Transversal**		
	F_V	F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Superstructure		20.3	17.5	353.7	40.5	17.5	707.5
Total at bottom of Column		***20.3***		***353.7***	***40.5***		***707.5***
Superstructure		20.3	19.5	394.2	40.5	19.5	788.5
Total at bottom of pilecap		***20.3***		***394.2***	***40.5***		***788.5***

9.Earth Quake

Earth Quake data			
Acceleration coefficient	A	0.0301	g
Seismic zone	Sz	1	
Soil profile type: according to geological data survey		I	
Coeffient site	S	1.00	
Bridge importance category: "1:critical"; "2:essential"; "3:other"	IC	2	essential
Response Modification Factor			
Column		2.0	
Connection		1.0	
Foundation		1.0	

Response Spectrum - Single mode method is used for EQ analysis.
Result of pier internal force is showed here. For detail refer to annex.

Item	**Vertical**	**Longitudinal**			**Transversal**		
	F_V	F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Total at bottom of Column		***685***		***6624***	***281***		***2713***
Total at bottom of pilecap		***685***		***7994***	***281***		***3275***

b.LOAD COMBINATIONS

I. LOAD COMBINATIONS

Loads at Bottom of Column						
Loads	Sign	F_V (kN)	Longitudinal F_{HX} (kN)	Longitudinal My (kN•m)	Transvesal F_{HY} (kN)	Transvesal Mx (kN•m)
Superstructure Loads						
1.Stage1 - Dead Load	DC	4941		0		
2.Stage2 - Pave.+Parapet+Railing+Mis.	DW	1283		0		
3.Live Load	LL	1311		558		3763
4.Pedestrian	LL	0		0		0
5.Centrifugal force	CE				0	0
6.Braking force	BR		106	1655		
7.Uniform temperature	TU		0	5		
8.Creep and Shrinkage	CR&SH		0	4		
9.Wind pressure on superstructure	WS		58	871	231	3484
10.Wind pressure on vehicles	WL		20	354	41	707
11.Earthequake						
a - Longitudinal direction	EQ		343	3312		
b - Transverse direction	EQ				141	1356
Substructure Loads						
1.Pier selfweight	DC	3712				
2.Soil on pile cap	EV	0				
3.Bouyancy on pier						
a - Maximum water level	WA	-521				
b - Minimum water level	WA	-250				
c - Average annual water level	WA	-250				
4.Stream pressure						
a - Maximum water level	WA		0	0	11	63
b - Minimum water level	WA		0	0	0	0
c - Average annual water level	WA		0	0	2	4
5.Wind pressure						
a - Maximum water level	WS		138	1527	28	316
b - Minimum water level	WS		192	1821	42	391
c - Average annual water level	WS		192	1821	42	391
6.Vessel collision force						
a - Longitudinal direction	CV		0	0		
b - Transverse direction	CV				0	0
7.Vehicular collision force						
a - Longitudinal direction	CT		1800	8172		
b - Transverse direction	CT				1800	8172

Loads at Bottom of Pilecap

Loads	Sign	F_V (kN)	Longitudinal F_{HX} (kN)	Longitudinal My (kN•m)	Transvesal F_{HY} (kN)	Transvesal Mx (kN•m)
Superstructure Loads						
1.Stage1 - Dead Load	DC	4941		0		
2.Stage2 - Pave.+Parapet+Railing+Mis.	DW	1283		0		
3.Live Load	LL	1311		558		3763
4.Pedestrian	LL	0		0		0
5.Centrifugal force	CE				0	0
6.Braking force	BR		106	1866		
7.Uniform temperature	TU		0	5		
8.Creep and Shrinkage	CR&SH		0	5		
9.Wind pressure on superstructure	WS		58	987	231	3946
10.Wind pressure on vehicles	WL		20	394	41	788
11.Earthequake						
a - Longitudinal direction	EQ		685	7994		
b - Transverse direction	EQ				281	3275
Substructure Loads						
1.Pier selfweight	DC	6064				
2.Soil on pile cap	EV	1081				
3.Bouyancy on pier						
a - Maximum water level	WA	-1463				
b - Minimum water level	WA	-1192				
c - Average annual water level	WA	-1192				
4.Stream pressure						
a - Maximum water level	WA		0	0	11	73
b - Minimum water level	WA		0	0	0	1
c - Average annual water level	WA		0	0	2	8
5.Wind pressure						
a - Maximum water level	WS		138	1803	28	373
b - Minimum water level	WS		192	2205	42	475
c - Average annual water level	WS		192	2205	42	475
6.Vessel collision force						
a - Longitudinal direction	CV		173	1794		
b - Transverse direction	CV				404	4111
7.Vehicular collision force						
a - Longitudinal direction	CT		1800	11772		
b - Transverse direction	CT				1800	11772

Load Factors and Load Combinations

Loads	Sign	Str1a 2	Str1b 3	Str2a 4	Str2b 5	Str3a 6	Str3b 7
Superstructure Loads							
1.Stage1 - Dead Load	DC	1.25	0.90	1.25	0.90	1.25	0.90
2.Stage2 - Pave.+Mis.	DW	1.50	0.65	1.50	0.65	1.50	0.65
3.Live Load	LL	1.75	1.75	-	-	1.35	1.35
4.Pedestrian	LL	1.75	1.75	-	-	1.35	1.35
5.Centrifugal force	CE	1.75	1.75	-	-	1.35	1.35
6.Braking force	BR	1.75	1.75	-	-	1.35	1.35
7.Uniform temperature	TU	0.50	0.50	0.50	0.50	0.50	0.50
8.Creep and Shrinkage	CR&SH	0.50	0.50	0.50	0.50	0.50	0.50
9.Wind pressure on superst.	WS	-	-	1.40	1.40	0.40	0.40
10.Wind pressure on vehicles	WL	-	-	-	-	1.00	1.00
11.Earthequake							
a - Longitudinal direction	EQL	-	-	-	-	-	-
b - Transverse direction	EQT	-	-	-	-	-	-
Substructure Loads							
1.Pier selfweight	DC	1.25	0.90	1.25	0.90	1.25	0.90
2.Soil on pile cap	EV	1.35	0.90	1.35	0.90	1.35	0.90
3.Bouyancy on pier							
a - Maximum water level	WA		1.00		1.00		1.00
b - Minimum water level	WA	1.00		1.00		1.00	
c - Average annual WL	WA						
4.Stream pressure							
a - Maximum water level	WA		1.00		1.00		1.00
b - Minimum water level	WA	1.00		1.00		1.00	
c - Average annual WL	WA						
5.Wind pressure							
a - Maximum water level	WS	-	-		1.40		0.40
b - Minimum water level	WS	-	-	1.40		0.40	
c - Average annual WL	WS	-	-				
6.Vessel collision force							
a - Longitudinal direction	CV	-	-	-	-	-	-
b - Transverse direction	CV	-	-	-	-	-	-
7.Vehicular collision force							
a - Longitudinal direction	CT	-	-	-	-	-	-
b - Transverse direction	CT	-	-	-	-	-	-

Load Factors and Load Combinations

Loads	Sign	Ser1 10	Ext1a 11	Ext1b 12	Ext1c 13	Ext1d 14	
Superstructure Loads							
1.Stage1 - Dead Load	DC	1.00	1.25	0.90	1.25	0.90	
2.Stage2 - Pave.+Mis.	DW	1.00	1.50	0.65	1.50	0.65	
3.Live Load	LL	1.00	0.50	0.50	0.50	0.50	
4.Pedestrian	LL	1.00	0.50	0.50	0.50	0.50	
5.Centrifugal force	CE	1.00	0.50	0.50	0.50	0.50	
6.Braking force	BR	1.00	0.50	0.50	0.50	0.50	
7.Uniform temperature	TU	1.00	-	-	-	-	
8.Creep and Shrinkage	CR&SH	1.00	-	-	-	-	
9.Wind pressure on superst.	WS	0.30	-	-	-	-	
10.Wind pressure on vehicles	WL	1.00	-	-	-	-	
11.Earthequake							
a - Longitudinal direction	EQL	-	1.00	1.00	0.30	0.30	
b - Transverse direction	EQT	-	0.30	0.30	1.00	1.00	
Substructure Loads							
1.Pier selfweight	DC	1.00	1.25	0.90	1.25	0.90	
2.Soil on pile cap	EV	1.00	1.35	0.90	1.35	0.90	
3.Bouyancy on pier							
a - Maximum water level	WA						
b - Minimum water level	WA	1.00					
c - Average annual WL	WA		1.00	1.00	1.00	1.00	
4.Stream pressure							
a - Maximum water level	WA						
b - Minimum water level	WA	1.00					
c - Average annual WL	WA		1.00	1.00	1.00	1.00	
5.Wind pressure							
a - Maximum water level	WS		-	-	-	-	
b - Minimum water level	WS	0.30	-	-	-	-	
c - Average annual WL	WS		-	-	-	-	
6.Vessel collision force							
a - Longitudinal direction	CV	-	-	-	-	-	
b - Transverse direction	CV	-	-	-	-	-	
7.Vehicular collision force							
a - Longitudinal direction	CT	-	-	-	-	-	
b - Transverse direction	CT	-	-	-	-	-	

Load Factors and Load Combinations

Loads	Sign	Ext2a 15	Ext2b 16	Ext2c 17	Ext2d 18		
Superstructure Loads							
1.Stage1 - Dead Load	DC	1.25	0.90	1.25	0.90		
2.Stage2 - Pave.+Mis.	DW	1.50	0.65	1.50	0.65		
3.Live Load	LL	0.50	0.50	0.50	0.50		
4.Pedestrian	LL	0.50	0.50	0.50	0.50		
5.Centrifugal force	CE	0.50	0.50	0.50	0.50		
6.Braking force	BR	0.50	0.50	0.50	0.50		
7.Uniform temperature	TU	-	-	-	-		
8.Creep and Shrinkage	CR&SH	-	-	-	-		
9.Wind pressure on superst.	WS	-	-	-	-		
10.Wind pressure on vehicles	WL	-	-	-	-		
11.Earthequake							
a - Longitudinal direction	EQL	-	-	-	-		
b - Transverse direction	EQT	-	-	-	-		
Substructure Loads							
1.Pier selfweight	DC	1.25	0.90	1.25	0.90		
2.Soil on pile cap	EV	1.35	0.90	1.35	0.90		
3.Bouyancy on pier							
a - Maximum water level	WA						
b - Minimum water level	WA						
c - Average annual WL	WA	1.00	1.00	1.00	1.00		
4.Stream pressure							
a - Maximum water level	WA						
b - Minimum water level	WA						
c - Average annual WL	WA	1.00	1.00	1.00	1.00		
5.Wind pressure							
a - Maximum water level	WS	-	-	-	-		
b - Minimum water level	WS	-	-	-	-		
c - Average annual WL	WS	-	-	-	-		
6.Vessel collision force							
a - Longitudinal direction	CV						
b - Transverse direction	CV						
7.Vehicular collision force							
a - Longitudinal direction	CT	1.00	1.00	1.00	1.00		
b - Transverse direction	CT	1.00	1.00	1.00	1.00		

II. LOAD COMBINATIONS RESULT

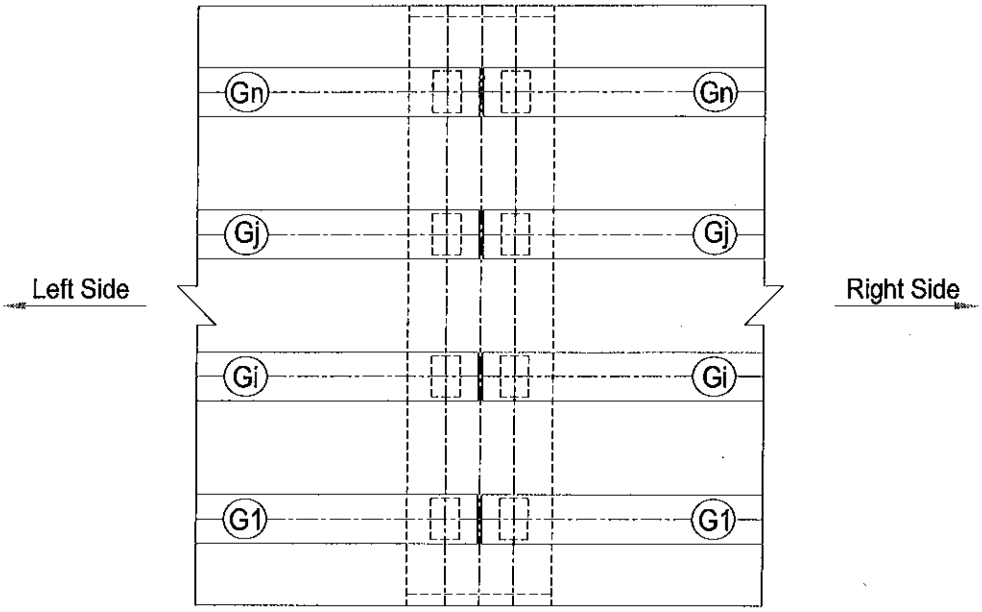
Load Combinations at Bottom of PierColumn

No	Combinations		Sign	F_V (kN)	Longitudinal F_{HX} (kN)	Longitudinal My (kN•m)	Transvesal F_{HY} (kN)	Transvesal Mx (kN•m)
1	Strength 1a		Str1a	14785	185	3877	0	6585
2	Strength 1b		Str1b	10395	185	3877	11	6648
3	Strength 2a		Str2a	12491	350	3773	383	5425
4	Strength 2b		Str2b	8100	274	3362	374	5384
5	Strength 3a		Str3a	14261	263	4422	150	7338
6	Strength 3b		Str3b	9870	241	4305	155	7371
7	Service 1		Ser1	10997	202	3383	123	5633
8	Extreme 1a	EQL	Ext1a	13146	395	4418	44	2292
9	Extreme 1b	EQL	Ext1b	9027	395	4418	44	2292
10	Extreme 1c	EQT	Ext1c	13146	156	2100	143	3241
11	Extreme 1d	EQT	Ext1d	9027	156	2100	143	3241

Load Combinations at Bottom of PileCap

No	Combinations		Sign	F_V (kN)	Longitudinal F_{HX} (kN)	Longitudinal My (kN•m)	Transvesal F_{HY} (kN)	Transvesal Mx (kN•m)
1	Strength 1a		Str1a	18243	185	4247	0	6586
2	Strength 1b		Str1b	12543	185	4247	11	6657
3	Strength 2a		Str2a	15948	350	4474	383	6191
4	Strength 2b		Str2b	10248	274	3910	374	6119
5	Strength 3a		Str3a	17718	263	4949	150	7638
6	Strength 3b		Str3b	12018	241	4788	155	7668
7	Service 1		Ser1	13488	202	3786	123	5879
8	Extreme 1a	EQL	Ext1a	16604	738	9206	86	2872
9	Extreme 1b	EQL	Ext1b	11175	738	9206	86	2872
10	Extreme 1c	EQT	Ext1c	16604	258	3610	283	5165
11	Extreme 1d	EQT	Ext1d	11175	258	3610	283	5165

c.PIER CAP ANALYSIS



1. DEAD LOAD

Stage1 - Dead load

Load: ***Girders***	***G1***	***G2***	***G3***	***G4***	***G5***	***G6***	***G7***
Left side Reactions (kN)	232.4	232.4	232.4	232.4	232.4		
Right side Reactions (kN)	232.4	232.4	232.4	232.4	232.4		
Total reactions both side (kN)	***464.8***	***464.8***	***464.8***	***464.8***	***464.8***		***0.0***

Load: ***Diaphragm***	***G1***	***G2***	***G3***	***G4***	***G5***	***G6***	***G7***
Left side Reactions (kN)	27.6	27.6	27.6	27.6	27.6		
Right side Reactions (kN)	27.6	27.6	27.6	27.6	27.6		
Total reactions both side (kN)	***55.3***	***55.3***	***55.3***	***55.3***	***55.3***		***0.0***

Load: ***Precast plank***	***G1***	***G2***	***G3***	***G4***	***G5***	***G6***	***G7***
Left side Reactions (kN)	38.4	38.4	38.4	38.4	38.4		
Right side Reactions (kN)	38.4	38.4	38.4	38.4	38.4		
Total reactions both side (kN)	***76.9***	***76.9***	***76.9***	***76.9***	***76.9***		***0.0***

Load: ***DeckSlab***	***G1***	***G2***	***G3***	***G4***	***G5***	***G6***	***G7***
Left side Reactions (kN)	195.7	195.7	195.7	195.7	195.7		
Right side Reactions (kN)	195.7	195.7	195.7	195.7	195.7		
Total reactions both side (kN)	***391.3***	***391.3***	***391.3***	***391.3***	***391.3***		

Load: ***Stage1 (DC)***	***G1***	***G2***	***G3***	***G4***	***G5***	***G6***	***G7***
Total reactions both side (kN)	***988.2***	***988.2***	***988.2***	***988.2***	***988.2***		

Stage2 - Dead load

Load: ***Pavement***	***G1***	***G2***	***G3***	***G4***	***G5***	***G6***	***G7***
Left side Reactions (kN)	58.9	58.9	58.9	58.9	58.9		
Right side Reactions (kN)	58.9	58.9	58.9	58.9	58.9		
Total reactions both side (kN)	***117.8***	***117.8***	***117.8***	***117.8***	***117.8***		

Load: *Parapet+railing*	*G1*	*G2*	*G3*	*G4*	*G5*	*G6*	*G7*
Left side Reactions (kN)	160.0	0.0	0.0	0.0	160.0		
Right side Reactions (kN)	160.0	0.0	0.0	0.0	160.0		
Total reactions both side (kN)	***320.0***	***0.0***	***0.0***	***0.0***	***320.0***		

Load: *Lighting post.+mis*	*G1*	*G2*	*G3*	*G4*	*G5*	*G6*	*G7*
Left side Reactions (kN)	5.4	5.4	5.4	5.4	5.4		
Right side Reactions (kN)	5.4	5.4	5.4	5.4	5.4		
Total reactions both side (kN)	***10.8***	***10.8***	***10.8***	***10.8***	***10.8***		

Load: *Stage2 (DC)*	*G1*	*G2*	*G3*	*G4*	*G5*	*G6*	*G7*
Total reactions both side (kN)	***448.5***	***128.6***	***128.6***	***128.6***	***448.5***		

2. LIVE LOAD

Live load reactions is calculated by level rule

Summary of Live load Reactions:

1 Loaded Lane n = 1.0 m = 1.20 1+IM = 1.25

Live load (kN)	Factor	*G1*	*G2*	*G3*	*G4*	*G5*	*G6*
Truck + Lane load		539.5	216.4	0.0	0.0	0.0	0.0
Tandem + Lane load		449.7	180.4	0.0	0.0	0.0	0.0
0.9*(2Truck+Lane)	0.9	561.4	225.2	0.0	0.0	0.0	0.0

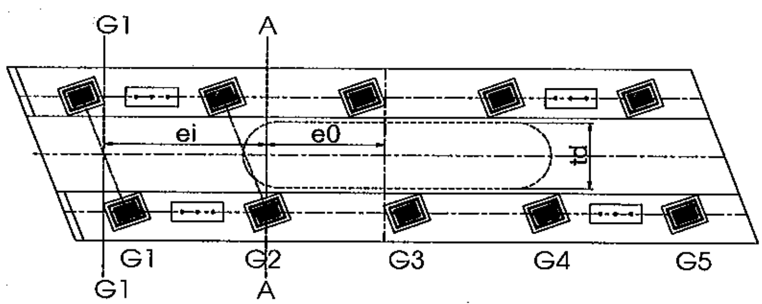
Reaction = [(1+IM)*Vehicle + Laneload]*n*m

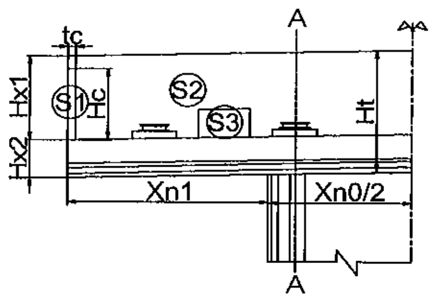
2 Loaded Lanes n = 2.0 m = 1.00 1+IM = 1.25

Live load (kN)	Factor	*G1*	*G2*	*G3*	*G4*	*G5*	*G6*
Truck + Lane load		449.6	518.8	291.5	0.0	0.0	0.0
Tandem + Lane load		374.8	432.4	243.0	0.0	0.0	0.0
0.9*(2Truck+Lane)	0.9	467.9	539.8	303.3	0.0	0.0	0.0

Reaction = [(1+IM)*Vehicle + Laneload]*n*m

3. PIER CAP DESIGN





Cantilever section (A-A)

Distance from centerline of pier to section A-A c0 = 2.28 m

Item		*G1*	*G2*	*G3*	*G4*	*G5*	*G6*	*G7*
Bearing is taken into account		1	1	0	0	0	0	0
Distance from bearing to section A-A								
Left side	ei	2.82	0.27	-	-	-	-	-
Right side	ei	2.82	0.27	-	-	-	-	-

Dead load of substructure

Notation	Dimensions	Value(m)	Notation	Dimensions	Value(m)
Hx1	Haunch 1 height	1.59	Hc	Curtain wall height	1.50
Hx2	Haunch 2 height	1.00	tc	Curtain wall thickness	0.15
Xn1	Haunch width	3.49	xd	Pier cap width	1.40
Xn0	Bottom of pier cap width	5.50		Xn0/2 - c0	0.47
γc	Concrete unit weight (kN/m3)	24.50	c0	Dist. from CL of pier to sec.A-A	2.28

Item	Volume	Section G1			Volume	Section A-A		
		Fv	Arm.$_{Fv}$	M_x		Fv	Arm.$_{Fv}$	M_x
Component	(m3)	(kN)	(m)	(kN•m)	(m3)	(kN)	(m)	(kN•m)
S1	0.50	12.1	1.07	12.9	0.50	12.1	3.88	47.1
S2	6.17	151.2	0.61	91.7	21.44	525.3	1.98	1039.2
S3					0.68	16.6	1.56	26.0
Total at section G1		***163.4***		***104.6***				
Total at section A-A						***554.0***		***1112.3***

Load components at section bearing G1

Item	Vertical	Torsion Moment			Bending Moment		
	F_V	F_{HX}	Arm.$_{HX}$	M_y	F_{HY}	Arm.$_{HY}$	M_x
	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Pier Selfweight	163.4						104.6
DC stage1	988					0.0	0.0
DW stage2	448.5					0.0	0.0
Live Load	561.4			257.8		0.0	0.0
Pedestrian	0.0					0.0	0.0

Load components at section A-A

Item	Vertical	Torsion Moment			Bending Moment		
	F_V	F_{HX}	Arm.$_{HX}$	M_y	F_{HY}	Arm.$_{HY}$	M_x
	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Pier Selfweight	554.0						1112.3
DC stage1	1976.4					2.82	3047.0
DW stage2	577.1					2.82	1297.7
Live Load	786.6			257.8		2.82	1641.4
Pedestrian	0.0					2.82	0.0

Load factors and Load combinations

Item	Ser1	Str1a
Pier Selfweight	1.00	1.25
DC stage1	1.00	1.25
DW stage2	1.00	1.50
Live Load	1.00	1.75
Pedestrian	1.00	1.75

Load Combinations

Section	Comb.	Vertical	Bending	Torsion
		Fv	Mx	My
G1	Ser1	***2162***	***105***	***258***
	Str1a	***3095***	***131***	***451***
A-A	Ser1	***3894***	***7098***	***258***
	Str1a	***5405***	***10018***	***451***

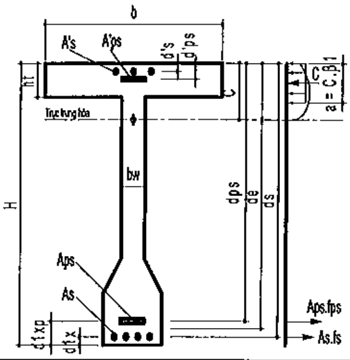
	DA NANG - QUANG NGAI EXPRESS WAY PROJECT ORB13 BRIDGE DETAIL DESIGN PIER CAP-P1 - CHECK STRENGTH	Item.	Eng.	Date.	Sign.
		Design			
		Check			
		Revise			

22TCN272-05; AASHTO LRFD 2nd - 1998

REINFORCEMENT CHECKING - PIER CAP

MATERIALS

	NORMAL CONCRETE		
f'c	Compressive Strength of concrete at 28 days	Mpa	30
Ec	Modulus of Elasticity	Mpa	27691
fr	Modulus of Rupture	Mpa	3.5
gc	Unit weight of concrete	kN/m3	24.5
	PRESTRESSING STEEL		
fpu	Tensile strength of prestressing steel	Mpa	1860
fpy	Yield strength of prestressing steel	Mpa	1670
Ep	Modulus of Elasticity	Mpa	195000
	REINFORCEMENT		
fy	Yield strength	Mpa	400
Es	Modulus of Elasticity	Mpa	200000
nc	Ratio Es/Ec		7



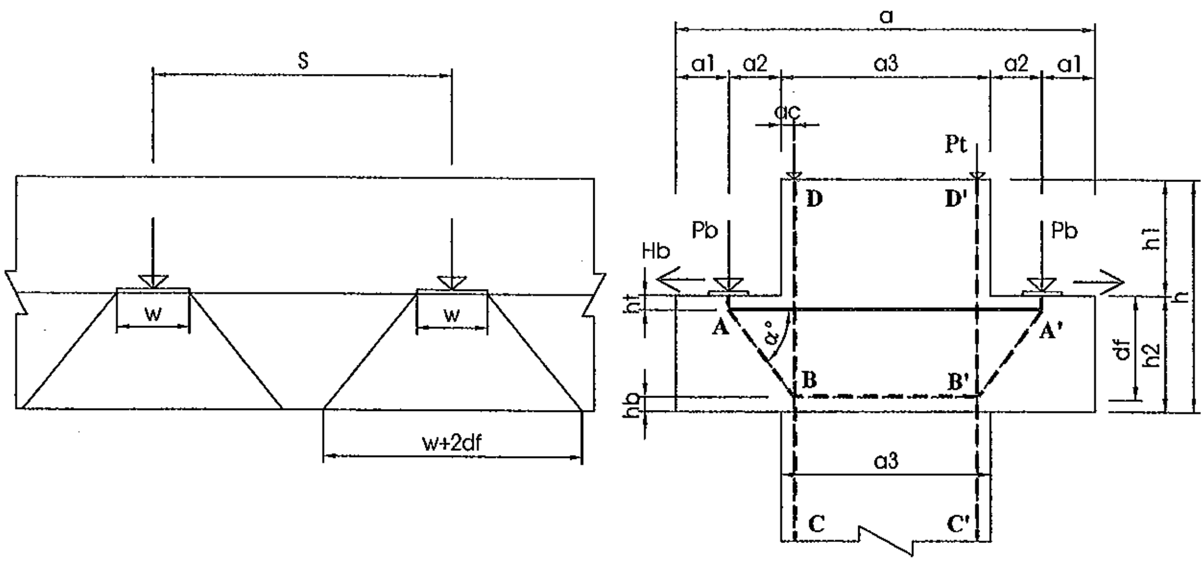
Sign	Parameters		Unit	Section - CANTILEVER				
				A-A	A-A	G1	G1	
	INTERNAL FORCES AT SECTIOIN							
	Combination			Strength	Service	Strength	Service	
Qu	Shear		kN	5405	3894	3095	2162	
Mu	Flexural Moment		kNm	10018	7098	131	105	
Nu	Axial load		kN	0	0	0	0	
Tu	Torsional Moment		kNm	0	17	0	284	
	FLEXURAL MOMENT CHECKING							
H	Section height		m	2.585	2.585	2.585	2.585	
d's	Dis. From comp. fiber to centroid of comp. Reinf		m	0.058	0.058	0.058	0.058	
d1x	Dis. From tens. fiber to centroid of tension Reinf		m	0.133	0.133	0.133	0.133	
	Cover to reinf		m	0.050	0.050	0.050	0.050	
ds	Dis. From comp. fiber to centroid of tension Reinf		m	2.452	2.452	2.452	2.452	
d'ps	Dis. From comp. fiber to centroid of comp. prestressing steel		m	0.000	0.000	0.000	0.000	
d1xp	Dis. From tens. fiber to centroid of tension prestressing steel		m	0.000	0.000	0.000	0.000	
dps	Dis. From comp. fiber to centroid of tension prestressing steel		m	2.585	2.585	2.585	2.585	
b	Width of the compression face of member		m	1.600	1.600	1.600	1.600	
bw	Web width or diameter of a circular section		m	1.600	1.600	1.600	1.600	
hf	Compression flange depth		m	1.000	1.000	1.000	1.000	
Iz	Moment of inertia of section		m4	2.303	2.303	2.303	2.303	
Amc	Section area		m2	4.136	4.136	4.136	4.136	
	Steel choice							
Aps	Tension prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	
		Number	tendons	0	0	0	0	
		Area	m2	0.00000	0.00000	0.00000	0.00000	
A'ps	Compression prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	
		Number	tendons	0	0	0	0	
		Area	m2	0.00000	0.00000	0.00000	0.00000	
As	Tension Reinforcement	Number	bars	26	26	26	26	
		Diameter	mm	32	32	32	32	
		Area	m2	0.02083	0.02083	0.02083	0.02083	
A's	Compression Reinforcement	Number	bars	0	0	0	0	
		Diameter	mm	16	16	16	16	
		Area	m2	0.00000	0.00000	0.00000	0.00000	
A'c	Shear reinforcement	Number	bars	4	4	4	4	
		Diameter	mm	20	20	20	20	
		Area	m2	0.00126	0.00126	0.00126	0.00126	
ϕ	Resistance factors for flexure		5.5.4.2	0.90	1.00	0.90	1.00	
ϕv	Resistance factors for shear			0.90	1.00	0.90	1.00	
ϕn	Resistance factors for axial force			1.00	1.00	1.00	1.00	
β1	Stress block factor			0.836	0.836	0.836	0.836	
c	Dis. Between centroid and top fiber		m	0.244	0.244	0.244	0.244	
	For T section behavior		m	0.244	0.244	0.244	0.244	
	For rectangular section behavior		m	0.244	0.244	0.244	0.244	
fpe	Effective stress in the prestressing steel after losses		Mpa	1116	1116	1116	1116	
fps	Aver. stress in pres. steel at the time for which the nominal resistance		Mpa	1810	1810	1810	1810	
k	Factor depends on type of P.S, Low relaxation strand k = 0.28			0.28	0.28	0.28	0.28	

a	Depth of equivalent stress block	m	0.204	0.204	0.204	0.204
de	Corresp. effective depth from extreme comp. fiber					
	to centroid of tensile force in the tensile reinf.	m	2.452	2.452	2.452	2.452
Mn	Nominal resistance	kNm	19576	19576	19576	19576
Mr	Factored reistance	kNm	**17618**	**19576**	**17618**	**19576**
Mu	Flexual moment	kNm	10018	7098	131	105
(5.7.3.2)	**Flexural moment Checking**		**OK**	**OK**	**OK**	**OK**
	Limits for reinforcement					
c/de	Maximum reinforcement		0.10	0.10	0.10	0.10
	Maximum reinforcement Checking	<= 0.42	**OK**	**OK**	**OK**	**OK**
r min	Minimum reinforcement		0.50%	0.50%	0.50%	0.50%
	Minimum reinforcement Checking for RC	0.23%	**OK**	**OK**	**OK**	**OK**
1.2*Mcr	Craking moment	kNm	4074	4074	4074	4074
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**		**OK**	**OK**	**OK**	**OK**
(5.7.3.4)	**Conetrol of craking by distr. of reinf for RC member- Check?**		No	Yes	No	Yes
	Existing condition for structrure	1,2 or 3	1	1	1	1
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.066	0.066	0.066	0.066
Z	Crack width parameter	N/mm	30000	30000	30000	30000
A	Area of concr. with same centroid as tens. Reinf	m2	0.008	0.008	0.008	0.008
fsa	Value	Mpa	369	369	369	369
0.6*fy		Mpa	240	240	240	240
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	240	240	240	240
x	Dist. From compression fiber to centroid	m	-	0.584	-	0.584
J.d	Arm	m	-	2.257	-	2.257
Icr	Moment of inertia of the cracked section	m4	-	0.615	-	0.615
fs	Tensile stress in reinforcement fs = Msls / (As*J.d)	Mpa	-	151	-	2
	Checking for control cracking fs<fsa		**N.a**	**OK**	**N.a**	**OK**
(5.10.8.2)	**Shrinkage and temperature Reinforcement (side distribution)**					
Areq	Area of required reinf	m2	0.00093	0.00093	0.00093	0.00093
	Distribution on sides 6 D16	m2	0.00121	0.00121	0.00121	0.00121
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45
	Checking		**OK**	**OK**	**OK**	**OK**
	SHEAR AND TORSION CHECKING					
β	Factor indicating diag. cracked concr. to tension		1.8	2.1	2.4	2.6
θ	Angle of inclination of diagonal compressive	degree	41.84	38.97	31.74	28.97
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90
bv	Effective web width as minimum web width - in dv	m	1.600	1.600	1.600	1.600
dv	Effective shear depth	m	2.350	2.350	2.350	2.350
	(de - a/2)	m	2.350	2.350	2.350	2.350
s	Spacing of stirrups	m	0.150	0.150	0.150	0.150
ncat	Amount of bars in spacing S	bars	4	4	4	4
Av	Shear reinf area in spacing S	m2	0.0013	0.0013	0.0013	0.0013
β	Assume		2.0	2.0	2.0	2.0
θ	Assume	degree	42.08	39.28	29.45	28.77
v	Shear stress in concrete	kN/m2	1597	1036	915	575
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116
e_x	Strain in tensile reinforcement		1.74E-03	1.30E-03	6.71E-04	4.83E-04
	if ex<0, multiple with reduce factor		-	-	-	-
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok
v/f'c	Ratio of shear stress and f'c		0.053	0.035	0.030	0.019
β	Final value		1.8	2.1	2.4	2.6
θ	Final value	degree	41.84	38.97	31.74	28.97
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	3129	3528	4126	4402
Vs	Shear resistance provided by shear reinforcement	kN	8791	9731	12723	14219
Vp	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0
Vn1	Vn1=Vc+Vs+Vp	kN	11920	13258	16849	18621
Vn2	Vn2	kN	28199	28199	28199	28199
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	11920	13258	16849	18621
Vr	Factored shear resistance	kN	**10728**	**13258**	**15164**	**18621**
Vu	Shear	kN	5405	3894	3095	2162
(5.8.2.7)	**Shear checking**		**OK**	**OK**	**OK**	**OK**
	Region requiring transverse reinf Checking		Need	Need	Need	No need
	Minimum shear reinf area	m2	0.0003	0.0003	0.0003	0.0003
	Minimum shear reinforcement Checking		**OK**	**OK**	**OK**	-
	0.1*f'c*bv*dv	kN	11280	11280	11280	11280
	Smax	m	0.60	0.60	0.60	0.60
	Maximum spacing Smax		**OK**	**OK**	**OK**	-

φt	Resistance factor for torsion	(5.5.4.2)	0.90	1.00	0.90	1.00	
pc	Outer perimeter of concrete section	m	6.000	6.000	6.000	5.400	
Acp	Area in outer perimeter of concrete section	m2	4.136	4.136	4.136	4.136	
fpc	Comp. stress in concrete after all prestress losses at the centroid of se	Mpa	0.00	0.00	0.00	0.00	
Tcr	Crack moment due to torsion	kNm	5122	5122	5122	5691	
	0.25*φ*Tcr	kNm	1152	1281	1152	1423	
Tu	Torsional moment by external forces	kNm	0	17	0	284	
	Shear and Torsion combine if Tu>0.25φTcr		No	No	No	No	
Ao	Area enclosed by shear flow path	m2	2.679	2.679	3.168	3.168	
At	Area of one leg of closed transverse torsion reinforcement	m2	0.0003	0.0003	0.0003	0.0003	
ph	Perimeter of the centerline of the closed transverse torsion reinf.	m	7.890	7.890	7.890	7.890	
Aoh	Area enclosed by centerline of ext. closed transverse torsion reinf.	m2	3.152	3.152	3.728	3.728	
Vu1	Modified Vu in case shear and torsion combine	kN	5405	3894	3095	2185	
v1	Determine θt in case shear and torsion combine	kN/m2	1597	1036	915	603	
θ	Assume	degree	40.61	37.29	27.00	32.05	
ε_x	Strain in tensile reinforcement		1.74E-03	1.30E-03	6.71E-04	4.88E-04	
	if εx<0, multiple with reduce factor		-	-	-	-	
v1/f'c	Ratio of shear stress and fc		0.053	0.035	0.030	0.020	
θt	Crack angle (5.5.8.3.4) upadated modified Vu	degree	41.84	38.97	31.74	28.98	
Tn	Nominal torsion resistance	kN	-	-	-	-	
Tr	Factored torsional resistance	kN	-	-	-	-	
(5.8.3.6.2)	**Torsional checking**		N.a	N.a	N.a	N.a	

HEAD STOCK DESIGN

1. Head Stock dimensions



Item	Sign	Unit	Value
Bearings spacing	S	m	2.55
Bearing pad dimension	W	m	0.45
Bearing pad dimension	L	m	0.35
	W+2df	m	1.65
Reactions - strength limit state	Pb	kN	1490.0
	Hb	kN	130.0
	Pt	kN	125.0
Head stock dimension	a1	m	0.55
	a2	m	0.45
	a3	m	1.60
	a	m	3.60
	h1	m	1.79
	h2	m	0.66
	h	m	2.45
Distance from top of ledge to compression rebars	df	m	0.60
Distance from concrete face to rebars	ht	m	0.06
	hb	m	0.058
	ac	m	0.110
Angle distribution	α	deg	43.93
Materials			
Reinforcement concrete unit weight	gc	kN/m3	24.50
Compressive strength of concrete	f'c	MPa	30.0
Yield strength of rebars	fy	MPa	400.0
Young modulus	Es	MPa	200000

Check the capacity of the cantilever using strut and tie method. Begin by determining the reaction applied to the ledge. Assumed simple model with Tie and Strut as shown in figure.

2. Truss force

Member	AA'	AB	BB'	BC	BD
Truss force (kN)	**1677**	**-2148**	**-1547**	**-1615**	**-125**

(Minus value is mean compression member - Strut, plus value is tension member - Tie)

3. Truss Check

Resistance factor for tension Tie	$\phi t =$	0.90
Resistance factor for compression Strut	$\phi c =$	0.70

a. Tension Tie - AA' S.5.6.3.4.1

Effective width of tension tie for inner bearing		W_in =	1.654 m
Effective width of tension tie for outer bearing		W_out =	1.654 m
Required capacity of tension tie			
$Tr = T / \phi t$		Tr =	1863 kN
Tie consists	11 bars @ 150 D25	At =	0.0054 m2

Tie with standard hooks at the ends, the bars are developed by the time they reach the intersection of the tie and strut. Capacity of tie:

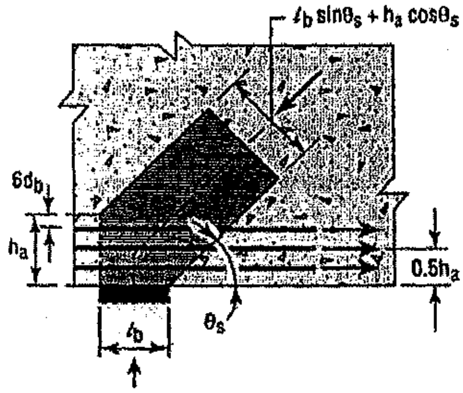
$Pn = fy \cdot At$	Pn =	2160 kN	Ok

b. Compression Strut - AB S.5.6.3.3

Required capacity of compression strut

$Cr = C / \phi c$ Cr = 3068 kN

The width of compression strut is based on width of bearing pad, for safety and simply w as below



Width of bearing pad	L =	0.35 m
$Ws = L \cdot \sin \alpha$	Ws =	0.24 m
Effective width	W =	1.65 m
Area of compression strut	Acs = Ws .W =	0.40 m2

The allowable compressive stress in the strut is dependent on the strain in the tension ties crossing the strut. The strain in the tension tie is found assuming a cracked cross section.

Stress in tie	$\sigma = T / At$	$\sigma =$	310 MPa	
Strain in tie	$\varepsilon s = \sigma / Es$	$\varepsilon s =$	0.00155 mm/mm	
Concrete strain component				
$\varepsilon 1 = \varepsilon s + (\varepsilon s + 0.002) \cdot \cot^2\alpha$		$\varepsilon 1 =$	0.00538	
Limiting compressive stress fcu, shall be taken as				
$fcu = f'c / (0.8 + 170 \cdot \varepsilon 1) <= 0.85 \cdot f'c$		fcu =	17.50 MPa	
Capacity of compression strut		Pn = fcu . Acs =	7028 kN	Ok

c. Check node region S.5.6.3.5

Limit on compressive stresses for node regions anchoring a one-direction tension tie

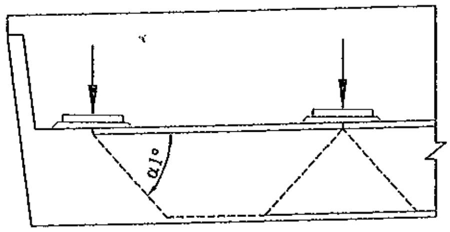
Limit stress = 0.75 . ϕ . f'c ϕ = 0.7

Limit stress = 15.8 MPa

Node section on concrete strut side governs

Compressive stress = C / Acs Compressive stress = 5.3 MPa Ok

4. Longitudinal rebars



Distributed angle α1 = 45.0 deg

Effective width of tension tie W1 = 1.000 m

Required capacity of tension tie, considering 40% of bearing reaction in calculation

Tr = T / ϕt Tr = 662 kN

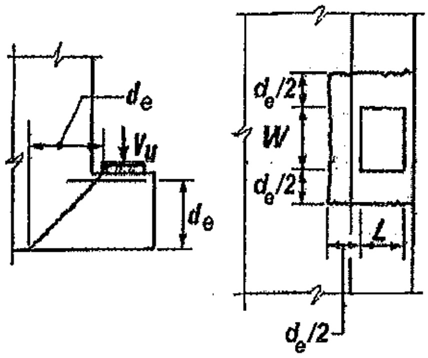
Tie consists 6 bars @ 150 D25 At = 0.0029 m2

Pn = fy·At Pn = 1178 kN Ok

5. Design for Punching Shear S.5.13.2.5.4

Nominal punching shear resistance, Vn (N), shall be taken a

- At interior pads:



$$V_n = 0,328\sqrt{f_c'}(W + 2L + 2d_e)d_e$$

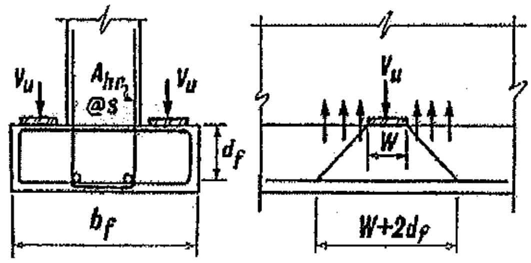
Vn = 2517.19 kN Ok

- At exterior pads:

$$V_n = 0,328\sqrt{f_c'}(W + L + d_e)d_e$$

Vn = 1500.11 kN Ok

6. Design of Hanger Reinforcement S.5.13.2.5.5



Nominal punching shear resistance, Vn (N), shall t

- For the strength limit state:

$$V_n = \frac{A_{hr} f_y}{s} S$$

Vn = 23487.2 kN Ok

11 bars @ 150 D20 Ahr = 3454 mm2

- For inverted T-beams

$$V_n = (0,165\sqrt{f_c'} b_f d_f) + \frac{A_{hr} f_y}{s}(W + 2d_f)$$

Vn = 15236.4 kN Ok

d.COLUMN DESIGN

I. COLUMN DATA

1.Load Combinations at Bottom of Pier Column

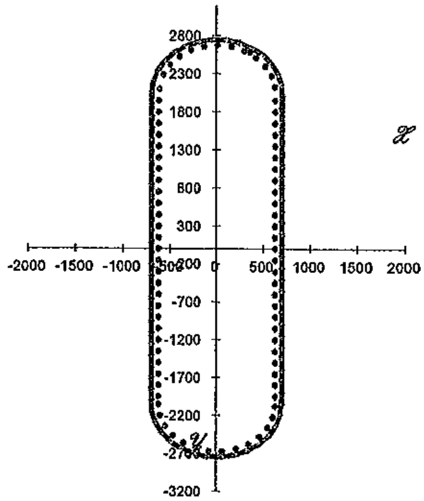
No	Combinations		Sign	F_V	Longitudinal		Transvesal	
				(kN)	F_{HX} (kN)	My (kN•m)	F_{HY} (kN)	Mx (kN•m)
1	Strength 1a		Str1a	14785	185	3877	0	6585
2	Strength 1b		Str1b	10395	185	3877	11	6648
3	Strength 2a		Str2a	12491	350	3773	383	5425
4	Strength 2b		Str2b	8100	274	3362	374	5384
5	Strength 3a		Str3a	14261	263	4422	150	7338
6	Strength 3b		Str3b	9870	241	4305	155	7371
7	Service 1		Ser1	10997	202	3383	123	5633
8	Extreme 1a	EQL	Ext1a	13146	395	4418	44	2292
9	Extreme 1b	EQL	Ext1b	9027	395	4418	44	2292
10	Extreme 1c	EQT	Ext1c	13146	156	2100	143	3241
11	Extreme 1d	EQT	Ext1d	9027	156	2100	143	3241

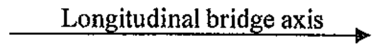
2. Pier Column Material

Normal concrete			
Compressive strength at 28 days age	f'c	30	MPa
Concrete elastic modulus	Ec	27691	MPa
Reinforcement TCVN1651-2008; CBV-400			
Yield strength	fy	400	MPa
Reinforcement elastic modulus	Es	200,000	MPa

3. Pier Column Section

Pier column thickness - longitudinal dimension	td	1.40	m
Pier column width - transverse dimension	tn	5.50	m
Section area	A	7.279	m2
Moment inertia	Ix	16.498	m4
	Iy	1.126	m4
Radius of gyration of gross concrete section; r = sqrt(I/A)	rx	1.505	m
	ry	0.393	m





4. Slenderness Effect

S.5.7.4.3, S.4.5.3.2.2b, S.4.6.2.5

Transverse direction: Fixed at bottom; translation free, rotation free at top	Kt	2.10	
Longitudinal direction: Fixed at bottom; translation free, rotation free at top	Kl	2.10	
Unsupported length from top to bottom of column	Lu	13.56	m
Slenderness ratio: if K.Lu / r > 22 than considered	Kt.Lu/rx	18.9	no
	Kl.Lu/ry	72.4	yes
Moment inertia of longitudinal reinforcements	Is	0	m4
Ratio of max factored Per. load moment to max factored total load moment	βd	0	

P - Δ analysis

****Longitudinal Direction**

P - Δ moment dertermination procedure:

Initial	Determining displacement for gross cross section		Δxg = Fx . H^3 / (3.E.Ig)
	Displacement for cracked section		Δxcr = Fcr . Δxg
	Moment P-Δ		M P-Δ = Δxcr . P
	Added lateral force		ΔFx = M P-Δ / H
Step: i st	Determining displacement for gross cross section		Δxg i = (Fx+ΔFx i-1) . H^3 / (3.E.Ig)
	Displacement for cracked section		Δxcr i = Fr . Δxg i
	Moment P-Δ		M P-Δ i = Δxcr i . P
	Added lateral force		ΔFx i = M P-Δ i / H

Combination	Fv	My	Initial					
			Fx	Δxg	Fcr	Δxcr	M P-Δ	ΔFx
	(kN)	(kNm)	(kN)	(m)	(kN)	(m)	(kNm)	(kN)
Strength 1a	14785	3877	286	0.008	2.5	0.019	282	20.8
Strength 1b	10395	3877	286	0.008	2.5	0.019	198	14.6
Strength 2a	12491	3773	278	0.007	2.5	0.019	232	17.1
Strength 2b	8100	3362	248	0.007	2.5	0.017	134	9.9
Strength 3a	14261	4422	326	0.009	2.5	0.022	310	22.8
Strength 3b	9870	4305	318	0.008	2.5	0.021	209	15.4
Service 1	10997	3383	250	0.007	2.5	0.017	183	13.5
Extreme 1a	13146	4418	326	0.009	2.5	0.022	285	21.0
Extreme 1b	9027	4418	326	0.009	2.5	0.022	196	14.5
Extreme 1c	13146	2100	155	0.004	2.5	0.010	136	10.0
Extreme 1d	9027	2100	155	0.004	2.5	0.010	93	6.9

Combination	Fv	My	1st Trial					
			Fx	Δxg	Fcr	Δxcr	M P-Δ	ΔFx
	(kN)	(kNm)	(kN)	(m)	(kN)	(m)	(kNm)	(kN)
Strength 1a	14785	3877	307	0.008	2.5	0.020	302	22.3
Strength 1b	10395	3877	301	0.008	2.5	0.020	208	15.3
Strength 2a	12491	3773	295	0.008	2.5	0.020	246	18.1
Strength 2b	8100	3362	258	0.007	2.5	0.017	139	10.3
Strength 3a	14261	4422	349	0.009	2.5	0.023	331	24.4
Strength 3b	9870	4305	333	0.009	2.5	0.022	219	16.1
Service 1	10997	3383	263	0.007	2.5	0.018	193	14.2
Extreme 1a	13146	4418	347	0.009	2.5	0.023	304	22.4
Extreme 1b	9027	4418	340	0.009	2.5	0.023	205	15.1
Extreme 1c	13146	2100	165	0.004	2.5	0.011	144	10.6
Extreme 1d	9027	2100	162	0.004	2.5	0.011	97	7.2

Combination	Fv	My	2nd Trial					
			Fx	Δxg	Fcr	Δxcr	M P-Δ	ΔFx
	(kN)	(kNm)	(kN)	(m)	(kN)	(m)	(kNm)	(kN)
Strength 1a	14785	3877	308	0.008	2.5	0.021	303	22.4
Strength 1b	10395	3877	301	0.008	2.5	0.020	209	15.4
Strength 2a	12491	3773	296	0.008	2.5	0.020	247	18.2
Strength 2b	8100	3362	258	0.007	2.5	0.017	139	10.3
Strength 3a	14261	4422	351	0.009	2.5	0.023	333	24.6
Strength 3b	9870	4305	334	0.009	2.5	0.022	219	16.2
Service 1	10997	3383	264	0.007	2.5	0.018	193	14.2
Extreme 1a	13146	4418	348	0.009	2.5	0.023	305	22.5
Extreme 1b	9027	4418	341	0.009	2.5	0.023	205	15.1
Extreme 1c	13146	2100	166	0.004	2.5	0.011	145	10.7
Extreme 1d	9027	2100	162	0.004	2.5	0.011	97	7.2

**Transverse Direction

Combination	Fv	Mx	Initial						
			Fx	Δxg	Fcr	Δxcr	M P-Δ	ΔFx
	(kN)	(kNm)	(kN)	(m)	(kN)	(m)	(kNm)	(kN)
Strength 1a	14785	6585	486	0.001	2.5	0.002	33	2.4
Strength 1b	10395	6648	490	0.001	2.5	0.002	23	1.7
Strength 2a	12491	5425	400	0.001	2.5	0.002	23	1.7
Strength 2b	8100	5384	397	0.001	2.5	0.002	15	1.1
Strength 3a	14261	7338	541	0.001	2.5	0.002	35	2.6
Strength 3b	9870	7371	544	0.001	2.5	0.002	24	1.8
Service 1	10997	5633	415	0.001	2.5	0.002	21	1.5
Extreme 1a	13146	2292	169	0.000	2.5	0.001	10	0.7
Extreme 1b	9027	2292	169	0.000	2.5	0.001	7	0.5
Extreme 1c	13146	3241	239	0.000	2.5	0.001	14	1.1
Extreme 1d	9027	3241	239	0.000	2.5	0.001	10	0.7

Combination	Fv	Mx	1st Trial						
			Fx	Δxg	Fcr	Δxcr	M P-Δ	ΔFx
	(kN)	(kNm)	(kN)	(m)	(kN)	(m)	(kNm)	(kN)
Strength 1a	14785	6585	488	0.001	2.5	0.002	33	2.4
Strength 1b	10395	6648	492	0.001	2.5	0.002	23	1.7
Strength 2a	12491	5425	402	0.001	2.5	0.002	23	1.7
Strength 2b	8100	5384	398	0.001	2.5	0.002	15	1.1
Strength 3a	14261	7338	544	0.001	2.5	0.002	35	2.6
Strength 3b	9870	7371	545	0.001	2.5	0.002	24	1.8
Service 1	10997	5633	417	0.001	2.5	0.002	21	1.5
Extreme 1a	13146	2292	170	0.000	2.5	0.001	10	0.7
Extreme 1b	9027	2292	170	0.000	2.5	0.001	7	0.5
Extreme 1c	13146	3241	240	0.000	2.5	0.001	14	1.1
Extreme 1d	9027	3241	240	0.000	2.5	0.001	10	0.7

Combination	Fv	Mx	2nd Trial						
			Fx	Δxg	Fcr	Δxcr	M P-Δ	ΔFx
	(kN)	(kNm)	(kN)	(m)	(kN)	(m)	(kNm)	(kN)
Strength 1a	14785	6585	488	0.001	2.5	0.002	33	2.4
Strength 1b	10395	6648	492	0.001	2.5	0.002	23	1.7
Strength 2a	12491	5425	402	0.001	2.5	0.002	23	1.7
Strength 2b	8100	5384	398	0.001	2.5	0.002	15	1.1
Strength 3a	14261	7338	544	0.001	2.5	0.002	35	2.6
Strength 3b	9870	7371	545	0.001	2.5	0.002	24	1.8
Service 1	10997	5633	417	0.001	2.5	0.002	21	1.5
Extreme 1a	13146	2292	170	0.000	2.5	0.001	10	0.7
Extreme 1b	9027	2292	170	0.000	2.5	0.001	7	0.5
Extreme 1c	13146	3241	240	0.000	2.5	0.001	14	1.1
Extreme 1d	9027	3241	240	0.000	2.5	0.001	10	0.7

**Load Combinations at bottom of column considering Slenderness Effect

Combination	Fv Vert. (kN)	Mx Trans. (kNm)	Mx P-Δ (kNm)	Mx Total (kNm)	My Long. (kNm)	My P-Δ (kNm)	My Total (kNm)
Strength 1a	14785	6585	33	6618	3877	303	4180
Strength 1b	10395	6648	23	6671	3877	209	4085
Strength 2a	12491	5425	23	5448	3773	247	4020
Strength 2b	8100	5384	15	5398	3362	139	3502
Strength 3a	14261	7338	35	7373	4422	333	4755
Strength 3b	9870	7371	24	7395	4305	219	4524
Service 1	10997	5633	21	5654	3383	193	3576
Extreme 1a	13146	2292	10	2302	4418	305	4723
Extreme 1b	9027	2292	7	2299	4418	205	4623
Extreme 1c	13146	3241	14	3256	2100	145	2245
Extreme 1d	9027	3241	10	3251	2100	97	2197

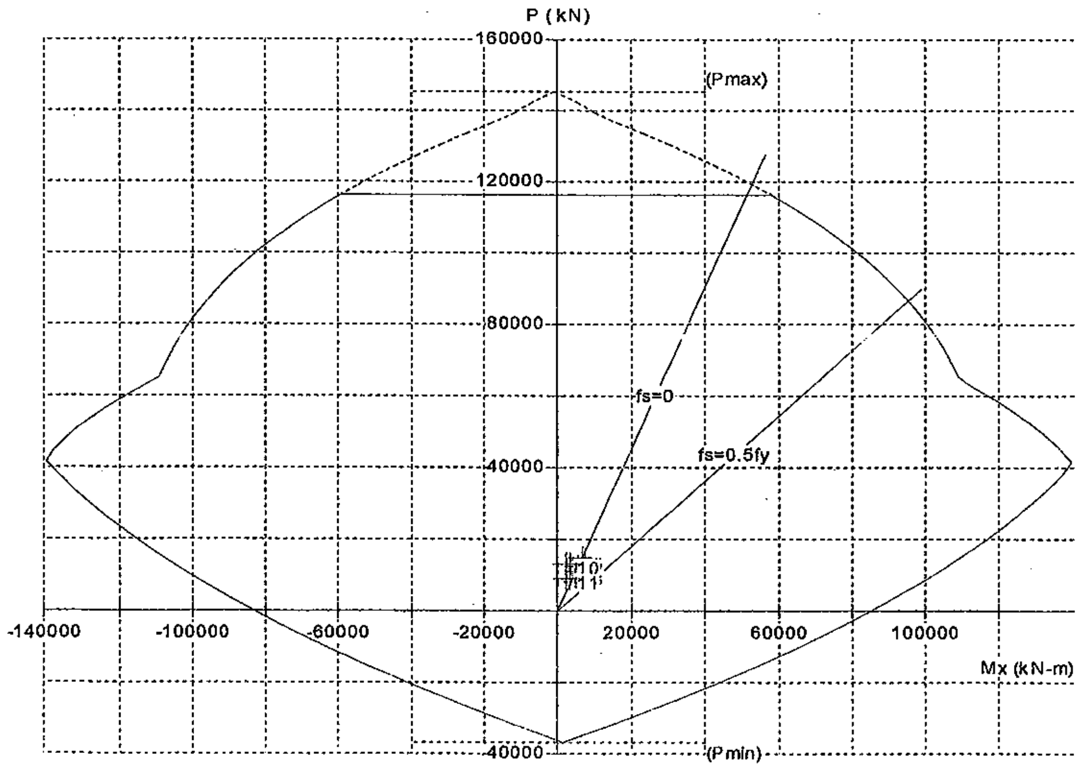
II. PIER COLUMN DESIGN

1. Limit of Reinforcement S.5.7.4.2

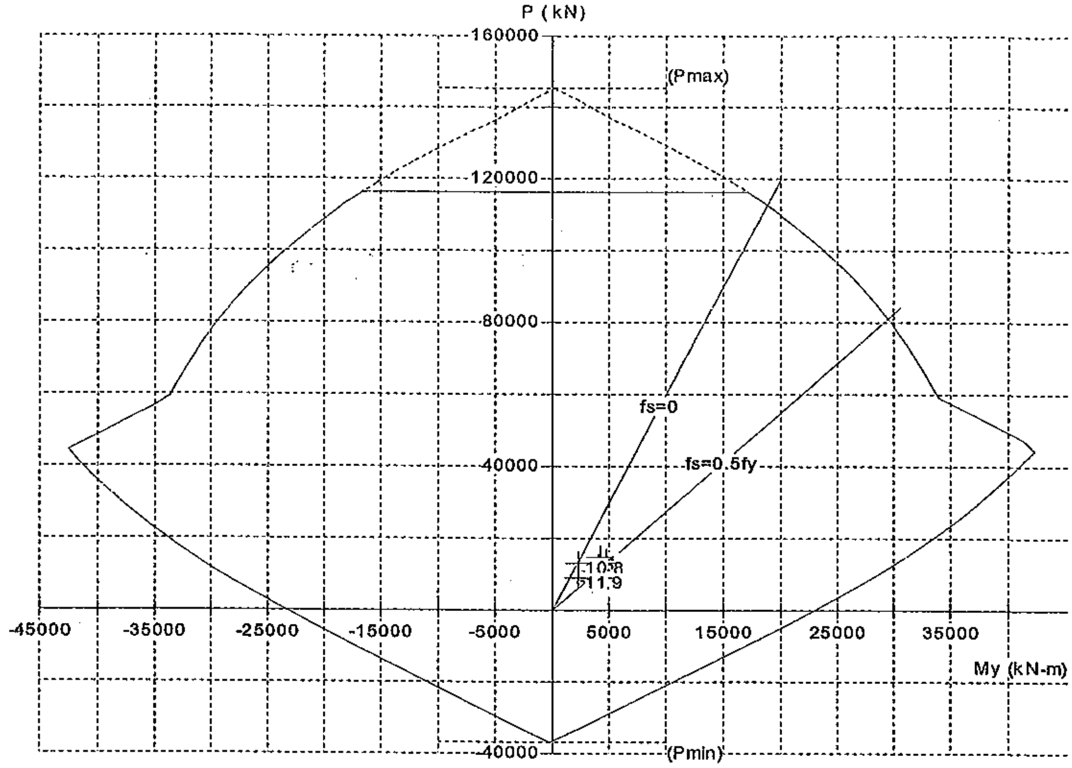
Minimum area of longitudinal reinforcement in column			
As.fy / (Ag . f'c) >= 0.135	As ≥	0.074	m2
As / Ag >= 0.01	As ≥	0.073	m2
Maximum area of longitudinal reinforcement in column			
As / Ag <= 0.08	As ≤	0.582	m2
Trial Rebars:	As	**0.066**	m2
1layers x 82 = 82 bars D32 @150	As1	0.066	m2
1layers x 0 = 0 bars D25 @150	As2	0.000	m2

2. Interation diagram M-P Using Pca-Column software

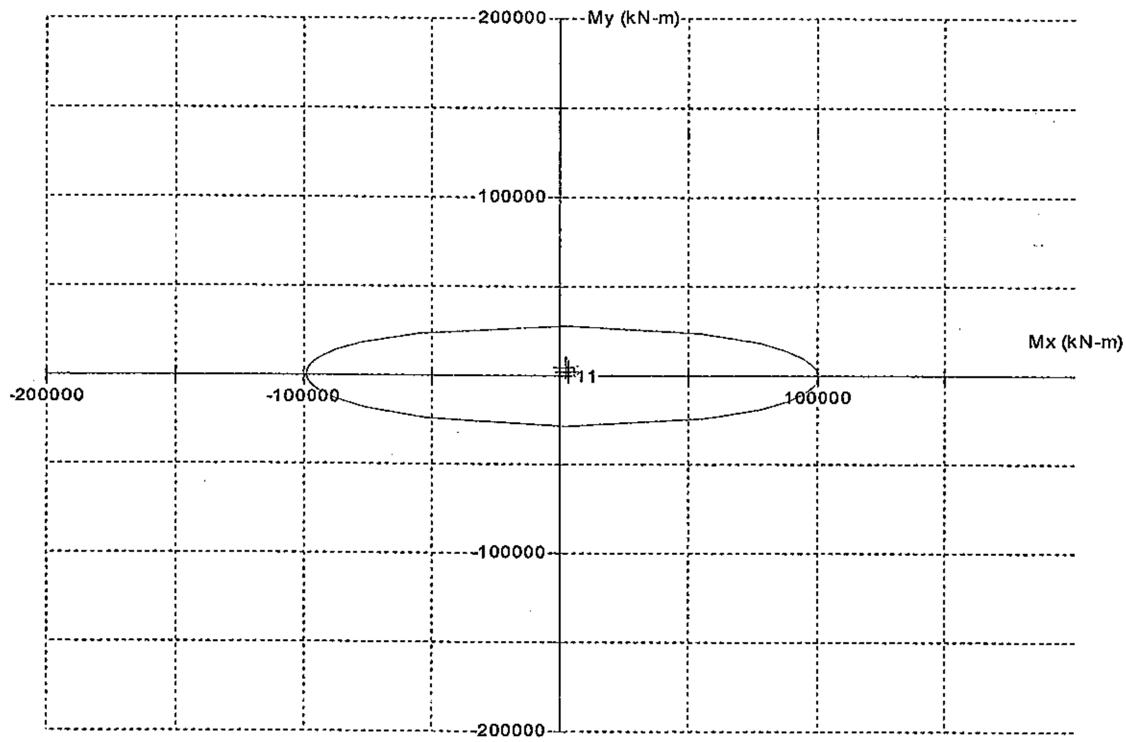
**In Transverse Direction



****In Longitudinal Direction**



****In Both Direction**



3. Column Ties S.5.7.4.6, S.5.10.6.3, S5.10.11.4.1d - e

Bridge is in seismic zone	Sz	1	
Area of concrete core measured out-to-out of ties	Ac	6.719	m2
Tie diameter	Dtie	16	mm2
Cross section area of 1 tie	As-tr	0.0002	m2
Spacing of hoops	s	150	mm
Length of reinfocement tie in 1 hoop	Ltie	19.90	m
Ratio of ties reinf. in one hoop/ volume of conc. core one pitch spacing			
ρs = As-tr . Ltie / (Ac * spacing)	ρs	0.0040	
Ratio of spiral reinf. To total volume of concrete core shall satisfy			S.5.7.4.6
ρs >= 0.45 . (Ag/Ac - 1). f'c/ fy = Req1	Req1	0.0028	N/A

			S.5.10.11.3
Transverse Reinforcement for Confinement at Plastic Hinges		S.5.10.11.4.1.d	
For a circular column "1:applied","2:Not applied"		2	
ρs >= 0.12 . f'c/ fy = Req2	Req2	0.0090	N/A
For a rectangular column			
Rectangular hoop reinforcement shall satisfy			
Either Ash >= 0.30 . s.hc.f'c/ fy. [Ag/Ac -1] = Req1			
or Ash >= 0.12 . s.hc.f'c/ fy = Req2			
In longitudinal direction "1:applied","2:Not applied"		2	
Number of cross tie	nt_x	8	ties
Total cross-sectional area of tie reinf.	Ash_x	0.0016	m2
Core dimension of tied column	hc_x	1.30	m
Rectangular hoop reinforcement shall satisfy	Req1_x	0.0004	m2
	Req2_x	0.0018	m2
	Conclude		N/A
In transverse direction			
Number of cross tie	nt_y	4	ties
Total cross-sectional area of tie reinf.	Ash_y	0.0008	m2
Core dimension of tied column	hc_y	5.00	m
Rectangular hoop reinforcement shall satisfy	Req1_y	0.0014	m2
	Req2_y	0.0068	m2
	Conclude		N/A
Spacing of Transverse Reinforcement for Confinement		S.5.10.11.4.1.e	
Transverse reinforcement for confinement shall be:			
* Provided at the top and bottom of the column over a length not less than the greatest of the			
maximum cross-sectional column dimensions,one-sixth of the clear height of the column, or 450 mm;			
Maximum cross-sectional column dimensions	L1	5.50	m
1/6 of clear height of column	L2	1.83	m
or 450mm	L3	0.45	m
Chosen value: L = max(L1,L2,L3)	L	5.50	m
* Spaced not to exceed one-quarter of the minimum member dimension or 100 mm center-to-center.			
	Spacing	0.10	m
Column connections		S.5.10.11.4.3	
* Development length for all longitudinal steel shall be 1.25 times that required in S.5.11			
* Column transverse reinforcement, as specified in Article 5.10.11.4.1d, shall be continued for a distance			
not less than one-half the maximum column dimension or 380 mm from the face of the column connection into			
the adjoining member.			
1/2 maximum column dimension	L4	2.75	m
or 380mm	L5	0.38	m
Chosen value: Le = max(L4,L5)	Le	2.75	m

4. Shear Design

Direction			Long.- X	Trans.-Y	Unit
Shear resistance factors		ϕv	1.0	1.0	
Factored shear force in longitudinal		Vu	1853	1802	kN
Required shear capacity Vn = Vu/ ϕv		Vn	1853	1802	kN
Determine concrete shear capacity					
Minimum shear reinforcement will provided in cross section					
Therefore		β	2.0	2.0	
		θ	45.0	45.0	
Cross section equivalent	height	h	1.40	5.20	m
	width	b	5.20	1.40	m
d = h - cover - d1x		d	1.31	5.11	m
dv = max(0.72*h; 0.9*d)		dv	1.18	4.60	m
Vc = min(0.083 . β . sqrt(f'c) . bv. dv, 0.25.f'c.bv.dv)		Vc	5591	5858	kN
Difference between required shear capacity and the capacity provided by concrete					
is the minimum required capacity for shear reinforcements					
Vs = Vn - Vc		Vs	0	0	kN

			S.5.10.11.3
Transverse Reinforcement for Confinement at Plastic Hinges		S.5.10.11.4.1.d	
For a circular column "1:applied","2:Not applied"		2	
ρs >= 0.12 . f'c/ fy = Req2	Req2	0.0090	**N/A**
For a rectangular column			
Rectangular hoop reinforcement shall satisfy			
Either Ash >= 0.30 . s.hc.f'c/ fy. [Ag/Ac -1] = Req1			
or Ash >= 0.12 . s.hc.f'c/ fy = Req2			
In longitudinal direction "1:applied","2:Not applied"		2	
Number of cross tie	nt_x	8	ties
Total cross-sectional area of tie reinf.	Ash_x	0.0016	m2
Core dimension of tied column	hc_x	1.30	m
Rectangular hoop reinforcement shall satisfy	Req1_x	0.0004	m2
	Req2_x	0.0018	m2
	Conclude		**N/A**
In transverse direction			
Number of cross tie	nt_y	4	ties
Total cross-sectional area of tie reinf.	Ash_y	0.0008	m2
Core dimension of tied column	hc_y	5.00	m
Rectangular hoop reinforcement shall satisfy	Req1_y	0.0014	m2
	Req2_y	0.0068	m2
	Conclude		**N/A**
Spacing of Transverse Reinforcement for Confinement		S.5.10.11.4.1.e	
Transverse reinforcement for confinement shall be:			
* Provided at the top and bottom of the column over a length not less than the greatest of the maximum cross-sectional column dimensions,one-sixth of the clear height of the column, or 450 mm;			
Maximum cross-sectional column dimensions	L1	5.50	m
1/6 of clear height of column	L2	1.83	m
or 450mm	L3	0.45	m
Chosen value: L = max(L1,L2,L3)	L	5.50	m
* Spaced not to exceed one-quarter of the minimum member dimension or 100 mm center-to-center.			
	Spacing	0.10	m
Column connections		S.5.10.11.4.3	
* Development length for all longitudinal steel shall be 1.25 times that required in S.5.11			
* Column transverse reinforcement, as specified in Article 5.10.11.4.1d, shall be continued for a distance not less than one-half the maximum column dimension or 380 mm from the face of the column connection into the adjoining member.			
1/2 maximum column dimension	L4	2.75	m
or 380mm	L5	0.38	m
Chosen value: Le = max(L4,L5)	Le	2.75	m

4. Shear Design

Direction		Long.- X	Trans.-Y	Unit
Shear resistance factors	ϕv	1.0	1.0	
Factored shear force in longitudinal	Vu	1853	1802	kN
Required shear capacity Vn = Vu/ ϕv	Vn	1853	1802	kN
Determine concrete shear capacity				
Minimum shear reinforcement will provided in cross section				
Therefore	β	2.0	2.0	
	θ	45.0	45.0	
Cross section equivalent height	h	1.40	5.20	m
width	b	5.20	1.40	m
d = h - cover - d1x	d	1.31	5.11	m
dv = max(0.72*h; 0.9*d)	dv	1.18	4.60	m
Vc = min(0.083 . β . sqrt(f'c) . bv. dv, 0.25.f'c.bv.dv)	Vc	5591	5858	kN
Difference between required shear capacity and the capacity provided by concrete is the minimum required capacity for shear reinforcements				
Vs = Vn - Vc	Vs	0	0	kN

In this case Vc > Vn so shear reinforcement is no need				
Stirrup diameter	Ds	16	16	
Number of stirrup legs / cross section	ns	6	2	
Shear legs area	Av	0.0012	0.0004	m2
Angle of inclination of shear reinf. to long. axis	α	90	90	deg
Vs = Av. fy. dv. (cot θ + cot α). sin α / s	s <=	-	-	m
Stirrup spacing used	s	0.10	0.10	m
Check minimum shear reinforcement requirement		**OK**	**OK**	
Av >= 0.083. sqrt(f'c). bv. s/ fy = Req	Req	0.0006	0.0002	m2
Check maximum shear reinforcement spacing requirement		**OK**	**OK**	
F = 0.1*f'c*bv*dv	F	18447	19329	kN
If Vu < 0.1*f'c*bv*dv then Smax=0.8*dv <= 600mm				
If Vu > 0.1*f'c*bv*dv then Smax=0.4*dv <= 300mm	Smax	0.60	0.60	m

Interface shear transfer S.5.8.4

Area of concrete engaged in shear transfer	Acv	7.279	m2
Area of shear reinforcement crossing the shear plane	Avf	0.066	m2
For concrete placed against clean, hardened concrete with surface roughened			
Cohesion factor specified in Article 5.8.4.2	c	0.7	MPa
Friction factor	μ	1	
For normal density concrete	λ	1	
Nominal shear resistance of the interface plane shall be taken as			
Vn = c. Acv + μ. Avf. fy	Vn	31368	kN
Vn <= 0.2 . f'c . Acv	Vn <=	43676	kN
Vn <= 5.5 . Acv . (1MPa)	Vn <=	40037	kN
Norminal shear resistance	Vn	31368	kN
Factor for shear friction		1.0	
Factored shear resistance	Vr	31368	kN
Horizontal force at bottom of pier column	Vu	2585	kN
	Conclude		**OK**

5. Control of cracking by distribution of reinforcement

Tensile stress in rebars should be satisfied equation: fs <= fsa = Z/[(dc.A)^1/3] and fs <= 0.6.fy				
Direction		Long.- X	Trans.-Y	Unit
Existing condition for structrure	1,2 or 3	1	1	
Crack width parameter	Z	30000	30000	N/mm
Flexural moment	Ms	3576	5654	kNm
Axial thrust at service limit state	Ns	10997	10997	kN
Cross section equivalent height	h	1.40	5.20	m
width	b	5.20	1.40	m
Concrete thickness from tension fiber to tension reinf.	dc	0.05	0.05	m
Concrete thickness from compression fiber to tension reinf.	d	1.31	5.11	kN
Number of rebars	N	76	22	bars
Area of rebars	As	0.0609	0.0176	m2
Area of concrete assumed to participate with reinf.				
A = 2 . dc . b / N	A	0.0068	0.0064	m2
	fsa	429	439	MPa
	0.6fy	240	240	MPa
Min (fsa, 0.6fy) = fs1	fs1	240	240	MPa
e = Ms/Ns+d-h/2	e	0.94	3.03	m
e/d > 1.15	e/d	1.15	1.15	
j = 0.74 + 0.1(e/d) $\leq$ 0.9	j	0.86	0.86	
i = 1/(1-j.d/e)	i	3.90	3.90	
Stress in rebars: fs = (Ms+Ns(d-h/2))/(As.j.i.d)	fs	39	111	MPa
	Conclude	**OK**	**OK**	
Maximum width of crack: an = 0.076.β.fs.(dc.A)^(1/3)	an	0.034	0.096	mm
Where	β	0.167	0.167	

e. SHALLOW FOUNDATION CHECKING

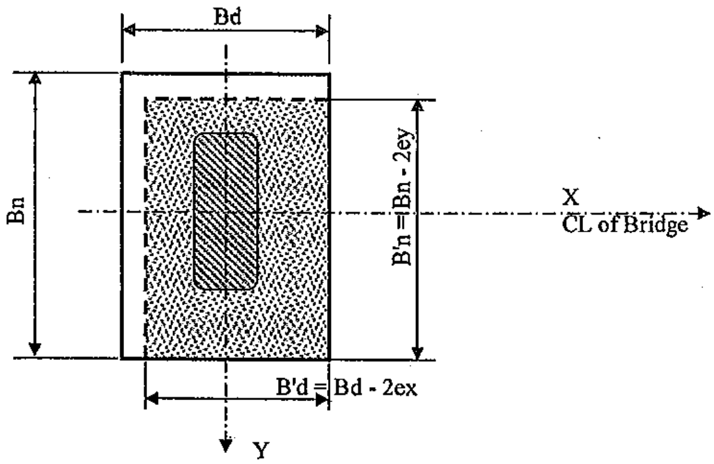
1.LOAD COMBINATIONS AT BOTTOM OF PILE CAP

Load Combinations at Bottom of PileCap

No	Combinations		Sign	F_V (kN)	Longitudinal F_{HX} (kN)	Longitudinal My (kN•m)	Transvesal F_{HY} (kN)	Transvesal Mx (kN•m)
1	Strength 1a		Str1a	18243	185	4248	0	6586
2	Strength 1b		Str1b	12543	185	4248	11	6657
3	Strength 2a		Str2a	15949	350	4475	383	6192
4	Strength 2b		Str2b	10249	274	3911	374	6120
5	Strength 3a		Str3a	17719	263	4949	150	7638
6	Strength 3b		Str3b	12019	241	4788	155	7669
7	Service 1		Ser1	13489	202	3787	123	5879
8	Extreme 1a	EQL	Ext1a	16604	738	9206	86	2872
9	Extreme 1b	EQT	Ext1b	11176	738	9206	86	2872
10	Extreme 1c	EQL	Ext1c	16604	258	3610	283	5165
11	Extreme 1d	EQT	Ext1d	11176	258	3610	283	5165

2.CHECK BEARING RESISTANCE OF SHALLOW FOUNDATION

S.10.6.3



Pile cap properties

Longitudinal dimension		Bd	6.0	m
Transverse dimension		Bn	8.0	m
Pile cap area		A	48.0	m2
Bending inertia moment	Wx = Bn^3.Bd/6	Wx	64.0	m3
	Wy = Bn.Bd^3/6	Wy	48.0	m3
Resistance factor for bearing capacity - SLS, shallow foundation		ϕb	0.60	
Resistance factor for bearing capacity - other limit state		ϕb	1.00	
Unaxial compression strength - saturated sample	361.8 kgf/cm2	Qu	5324	kN/m2
Factored bearing resistance		Qr	3195	kN/m2

Compute the bearing capacity based on rock

The canadian foundation engineering manual 1992		
Qu = 3.[σc.Ksp.D]		
Ksp = [3+C/B]/[10.(1+300.g/C)^0.5]		
qu: ultimate end bearing pressure		
C: spacing of discontinuities	C=	0.25 m
B: pile width or (d pile diameter)	B=	6 m
g: aperture of discontinuite	g=	0.03 m
D = 1+0.4. (L/d) <=3.4 - depth factor	D=	1.67
L: length of the socket	L=	10 m
d: diameter of pile	d=	6 m
	Ksp=	0.050005
sc: unconfined compressive strength	sc =	35.49258 Mpa
	Qu=	8.873978 Mpa
	Qu=	8874 kN/m2

FHWA manual 1988		
qu = σc.Ksq		
Ksq = [9+3.C/B]/[10.(1+300.g/C)^0.5]		
qu: ultimate end bearing pressure		
C: spacing of discontinuities	C=	0.25 m
B: pile width or (d pile diameter)	B=	6 m
g: aperture of discontinuite	g=	0.03 m
D = 1+0.4. (L/d) <=3.4 - depth factor	D=	1.667
L: length of the socket	L=	10 m
d: diameter of pile	d=	6 m
	Ksq=	0.1500
σc: unconfined compressive strength	σc =	35.493 Mpa
	Qu=	5.3244 Mpa
	Qu=	5324 kN/m2

Stress at corner points

σ max= Fv/A+Mx/Wx+My/Wy

σ min= Fv/A-Mx/Wx-My/Wy

Bearing Resistance AT BOTTOM OF FOOTING

Load Combination	**Fv /A** (kPa)	**Mx/Wx** (kPa)	**My/Wy** (kPa)	**σmax** (kPa)	**σmin** (kPa)	**Qr** (kPa)	**Check**
Strength 1a	380	103	88	571	189	3195	**OK**
Strength 1b	261	104	88	454	69	3195	**OK**
Strength 2a	332	97	93	522	142	3195	**OK**
Strength 2b	214	96	81	391	36	3195	**OK**
Strength 3a	369	119	103	592	147	3195	**OK**
Strength 3b	250	120	100	470	31	3195	**OK**
Service 1	281	92	79	452	110	3195	**OK**
Extreme 1a	346	45	192	583	109	5324	**OK**
Extreme 1b	233	45	192	469	-4	5324	**OK**
Extreme 1c	346	81	75	502	190	5324	**OK**
Extreme 1d	233	81	75	389	77	5324	**OK**

Effective Footing

Ecentricity	ex = Mx/ Fv
	ey = My/ Fv
Effective footing dimensions	B'd = Bd - 2ex
	B'n = Bn - 2ey
Effective footing area	A' = B'd . B'n

Effective Footing

Load Combination	**ex** (m)	**ey** (m)	**Bd'** (m)	**Bn'** (m)	**A'** (m2)	**W'x** (m3)	**W'y** (m3)
Strength 1a	0.23	0.36	5.5	7.3	40.3	48.9	37.2
Strength 1b	0.34	0.53	5.3	6.9	36.9	42.7	32.8
Strength 2a	0.28	0.39	5.4	7.2	39.3	47.3	35.6
Strength 2b	0.38	0.60	5.2	6.8	35.6	40.4	31.1
Strength 3a	0.28	0.43	5.4	7.1	38.8	46.2	35.2
Strength 3b	0.40	0.64	5.2	6.7	35.0	39.2	30.3
Service 1	0.28	0.44	5.4	7.1	38.8	46.1	35.1
Extreme 1a	0.55	0.17	4.9	7.7	37.4	47.8	30.5
Extreme 1b	0.82	0.26	4.4	7.5	32.6	40.7	23.6
Extreme 1c	0.22	0.31	5.6	7.4	41.1	50.5	38.1
Extreme 1d	0.32	0.46	5.4	7.1	37.9	44.7	33.8

Bearing Resistance - effective footing - AT BOTTOM OF FOOTING			
Load Combination	Fv /A' (kPa)	Qr (kPa)	Check
Strength 1a	453	3195	**OK**
Strength 1b	340	3195	**OK**
Strength 2a	406	3195	**OK**
Strength 2b	288	3195	**OK**
Strength 3a	456	3195	**OK**
Strength 3b	344	3195	**OK**
Service 1	348	3195	**OK**
Extreme 1a	444	5324	**OK**
Extreme 1b	343	5324	**OK**
Extreme 1c	404	5324	**OK**
Extreme 1d	295	5324	**OK**

3.CHECK SLIDING AT THE BASE OF FOOTING

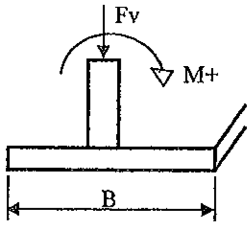
S.10.6.3.3

Horizontal force H = $(F_{HX}^2 + F_{HY}^2)^{0.5}$			
Factored resistance against failure by sliding			
$Q_r = \phi\, Q_n = \phi_t Q_t + \phi_{ep}\, Q_{ep}$			
Norminal shear resistance between soil and foundation Qt = Fv.tan(ϕ)	Qt		
For concrete cast against soil: tan (ϕ) = tan (ϕf)	tan(ϕf)	0.55	
Internal friction angle of soil	ϕf	29	deg
Resistance factor for shear reistance between soil and foundation	ϕt	0.80	
Norminal passive resistance	Qep	0.00	kN
Resistance factor for passive reistance	ϕep	0.50	

Load Combination	Resist. Factor ϕ	Fv (kN)	F_{HX} (kN)	F_{HY} (kN)	H (kN)	Qr (kN)	Check H < Qr
Strength 1a	0.80	18243	185	0	185	8090	**OK**
Strength 1b	0.80	12543	185	11	186	5562	**OK**
Strength 2a	0.80	15949	350	383	519	7072	**OK**
Strength 2b	0.80	10249	274	374	464	4545	**OK**
Strength 3a	0.80	17719	263	150	303	7857	**OK**
Strength 3b	0.80	12019	241	155	287	5330	**OK**
Service 1	1.00	13489	202	123	236	7477	**OK**
Extreme 1a	1.00	16604	738	86	743	9204	**OK**
Extreme 1b	1.00	11176	738	86	743	6195	**OK**
Extreme 1c	1.00	16604	258	283	383	9204	**OK**
Extreme 1d	1.00	11176	258	283	383	6195	**OK**

4.CHECK OVERTURNNING AT THE BASE OF FOOTING

S.11.6.3.3, S.11.6.3.7
S.10.6.4.2-lrfd2007



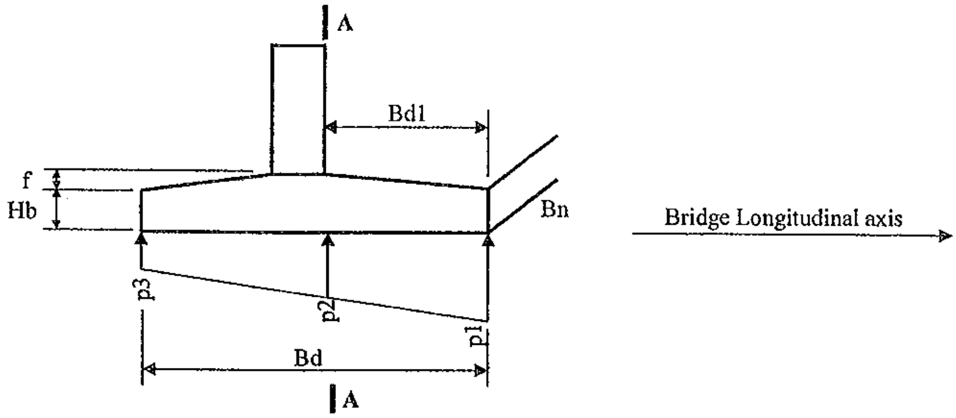
S.10.6.3.2.5

The location of the resultant of the reaction forces shall be within the middle three-fourths of the base.				
Longitudinal direction	ex =My / Fv <= 3.Bd / 8 = [ex]	[ex]	2.25	m
Transverse direction	ey =Mx / Fv <= 3.Bn / 8 = [ey]	[ey]	3.00	m
For seismic provision				S.10.6.5
According to 22TCN272-05: 1				
Longitudinal direction	ex =My / Fv <= 0,6 Bd / 2 = [ex]	[ex]	1.80	m
Transverse direction	ey =Mx / Fv <= 0.6 Bn / 2 = [ey]	[ey]	2.40	m
According to LRFD 2004: 2				
Where γEQ = 0				
Longitudinal direction	ex =My / Fv <= 2/3. Bd / 2 = [ex]	[ex]	2.00	m
Transverse direction	ey =Mx / Fv <= 2/3. Bn / 2 = [ey]	[ey]	2.67	m
Where γEQ = 1				
Longitudinal direction	ex =My / Fv <= 8/10. Bd / 2 = [ex]	[ex]	2.40	m
Transverse direction	ey =Mx / Fv <= 8/10. Bn / 2 = [ey]	[ey]	3.20	m
Where γEQ between 0 and 1, restrictions of the location can get by linear interpolation				
Choosing value for seismic: following LRFD 2004, with γEQ = 0.5				
Longitudinal direction		[ex]	2.20	m
Transverse direction		[ey]	2.93	m

Load Combination	**Fv** (kN)	**Mx** (kN•m)	**My** (kN•m)	**Longitudinal** ex (m)	**Check** ex <[ex]	**Transverse** ey (m)	**Check** ey <[ey]
Strength 1a	18243	6586	4248	0.23	**OK**	0.36	**OK**
Strength 1b	12543	6657	4248	0.34	**OK**	0.53	**OK**
Strength 2a	15949	6192	4475	0.28	**OK**	0.39	**OK**
Strength 2b	10249	6120	3911	0.38	**OK**	0.60	**OK**
Strength 3a	17719	7638	4949	0.28	**OK**	0.43	**OK**
Strength 3b	12019	7669	4788	0.40	**OK**	0.64	**OK**
Service 1	13489	5879	3787	0.28	**OK**	0.44	**OK**
Extreme 1a	16604	2872	9206	0.55	**OK**	0.17	**OK**
Extreme 1b	11176	2872	9206	0.82	**OK**	0.26	**OK**
Extreme 1c	16604	5165	3610	0.22	**OK**	0.31	**OK**
Extreme 1d	11176	5165	3610	0.32	**OK**	0.46	**OK**

5.CHECK BENDING MOMENT AND SHEAR OF FOOTING

Transverse Section A-A



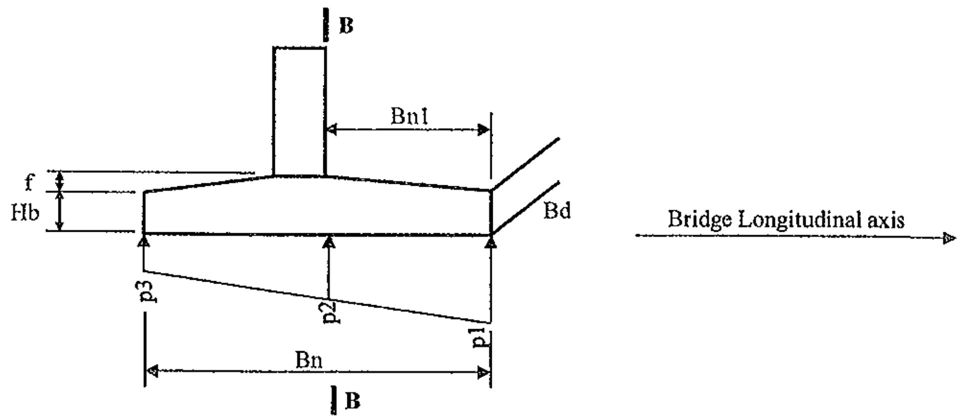
Transverse section (A-A)			
Footing dimensions	Hb	2.00	m
	f	0.00	m
	Bd	6.00	m
	Bd1	2.53	m
	Bn	8.00	m
	Hbb	0.00	m
	Bdb	0.00	m
	Bdb1	0.00	m
	Bnb	0.00	m
Considering of bouyancy "1:yes" "0:no"		1	
Internal force at section A-A due to selfweight of pilecap			
Shear force	Qself	595	kN
Bending moment	Mself	754	kN
p1 = Fv /A + My/ Wy			
p3 = Fv /A - My/ Wy			

Earth on toe shall be neglected.

Load Combination	Load factor	Qself (kN)	Mselft (kN•m)	p1 (kPa)	p3 (kPa)	p2 (kPa)	M_A (kN•m)	Q_A (kN)
Strength 1a	1.25	744	943	571	292	453	**12716**	**9640**
Strength 1b	0.90	536	679	350	173	275	7662	5796
Strength 2a	1.25	744	943	425	239	347	9306	7081
Strength 2b	0.90	536	679	295	132	226	6305	4745
Strength 3a	1.25	744	943	472	266	385	10435	7944
Strength 3b	0.90	536	679	350	151	266	7589	5707
Service 1	1.00	595	754	360	202	293	7915	6024
Extreme 1a	1.25	744	943	538	154	376	11475	8512
Extreme 1b	0.90	536	679	425	41	263	8836	6428
Extreme 1c	1.25	744	943	421	271	358	9325	7147
Extreme 1d	0.90	536	679	308	158	245	6685	5063

Maximum internal force at section A-A (transverse section)			
Bending moment	M_A	**12716**	kNm
Shear force	Q_A	**9640**	kN

Longitudinal Section B-B



Longtidinal section (B-B)			
Footing dimensions	Hb	2.00	m
	Bc	5.50	m
	Bn	8.00	m
	Bn1	1.49	m
	Bd	6.00	m
	Bn2	2.30	m
	Bnb	0.00	m
	Bnb1	-2.75	m
	Bdb	0.00	m
Considering of bouyancy "1:yes" "0:no"		1	
Internal force at section B-B due to selfweight of pilecap			
Shear force	Qself	1475	kN
Bending moment	Mself	6174	kN
p1 = Fv /A + Mx/ Wx			
p3 = Fv /A - Mx/ Wx			

Earth on toe shall be neglected.

Load Combination	Load factor	**Qself** (kN)	**Mselft** (kN•m)	**p1** (kPa)	**p3** (kPa)	**p2** (kPa)	**M_A** (kN•m)	**Q_A** (kN)
Strength 1a	1.25	744	1844	483	277	445	1270	3391
Strength 1b	0.90	536	1328	365	157	327	1006	2549
Strength 2a	1.25	744	1844	429	236	393	918	2920
Strength 2b	0.90	536	1328	309	118	274	641	2062
Strength 3a	1.25	744	1844	488	250	444	1293	3413
Strength 3b	0.90	536	1328	370	131	326	1026	2566
Service 1	1.00	595	1475	373	189	339	919	2577
Extreme 1a	1.25	744	1844	391	301	374	707	2665
Extreme 1b	0.90	536	1328	278	188	261	475	1865
Extreme 1c	1.25	744	1844	427	265	397	915	2925
Extreme 1d	0.90	536	1328	314	152	284	682	2125
								-744

Maximum internal force at section A-A (transverse section)			
Bending moment	M_A	**1293**	kNm
Shear force	Q_A	**3413**	kN

5.PUNCHING SHEAR CHECK (TWO WAY SHEAR) S.5.13.3.6.3

Assume the entire column vertical load needs to be carried at the perimeter.

Two-way shear is evaluated on a perimeter located dv/2 away from the face of the actual pier column.

Pier Column dimensions	Longitudinal axis	td	1.40	m
	Transverse axis	tn	5.50	m
Estimated distance between internal flexural force components dv, we may take dv = 0.9*de				
de = H - cover - dx1		de	1.81	m
		dv	1.63	m
Perimeter of two-way shear				
b0 = (td+tn)*2 + 4*dv		b0	20.31	m
Compressive strength of pilecap concrete		f'c	30	Mpa
Yield strength of rebar		fy	400	Mpa
Section with transverse reinforcement				
Norminal shear resistance shall be taken as				
Vn = Vc + Vs <= 0.504.sqrt(f'c). b0 . dv = Va				
Vc = 0.166 . sqrt(f'c). b0 . dv				
Vs = Av. fy . dv / s				
Shear resistance of concrete		Vc	30068	kN
Assumed stirrup diameter		Ds	16	mm
Number of stirrup legs / cross section		ns	0	
Shear legs area		Av	0.0000	m2
Stirrup spacing used		s	600	mm
Shear resistance of reinforcement		Vs	0	kN
		Va	91292	kN
		Vn	30068	kN
Maximum reaction at bottom of column		Vu	13526	kN
Resistance factor for shear		ϕv	0.9	
Factored shear resistance		ϕv*Vn	27062	kN
Punching shear check			**OK**	

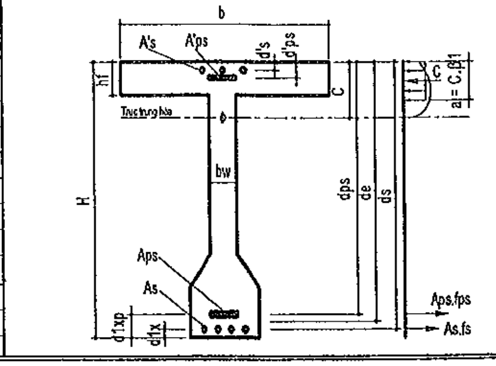
6.SETTLEMENT OF FOOTING ON ROCK

For Rectangular footing				
Elastic settlement p = q0 (1 - v^2) * B.Ip / Em				
Ip = (L/B)^0.5 / βz				
Footing dimensions	Bd =	B'	5.44	m
	Bn =	L'	7.13	m
		L'/B'	1.31	
Ridity "1: Flexible" "2: rigid"			2	
Factor to account for footing shape and rigidity		βz	1.09	
Influence coefficient to account for rigidity and dimension of footing		Ip	1.05	
Poisson's ratio		v	0.29	
Rock mass modulus Em = 1000*10^[(RMR-10)/40]		Em	891	Mpa
Rock mass rating a.10.4.6.4, Table 10.6.4.4-1,2		RMR	8	
Applied vertical stress at base of loaded area - Service 1 combination		q0	0.348	Mpa
Elastic settlement	**Can be ignored**	p	2.05	mm

REINFORCEMENT CHECKING - PILE CAP

MATERIALS

	NORMAL CONCRETE		
f'c	Compressive Strength of concrete at 28 days	Mpa	30
Ec	Modulus of Elasticity	Mpa	27691
fr	Modulus of Rupture	Mpa	3.5
gc	Unit weight of concrete	kN/m3	24.5
	PRESTRESSING STEEL		
fpu	Tensile strength of prestressing steel	Mpa	1860
fpy	Yield strength of prestressing steel	Mpa	1670
Ep	Modulus of Elasticity	Mpa	195000
	REINFORCEMENT		
fy	Yield strength	Mpa	400
Es	Modulus of Elasticity	Mpa	200000
nc	Ratio Es/Ec		7



Sign	Parameters		Unit	Section - PILE CAP A-A	A-A	A-A	B-B	B-B
	INTERNAL FORCES AT SECTIOIN							
	Combination			Strength	Service	Extreme	Service	Strength
Qu	Shear		kN	9640	6024	8512	2577	3391
Mu	Flexural Moment		kNm	12716	7915	11475	919	1270
Nu	Axial load		kN	0	0	0	0	0
Tu	Torsional Moment		kNm	0	0	0	0	0
	FLEXURAL MOMENT CHECKING							
H	Section height		m	2.000	2.000	2.000	2.000	2.000
d's	Dis. From comp. fiber to centroid of comp. Reinf		m	0.088	0.088	0.088	0.088	0.088
d1x	Dis. From tens. fiber to centroid of tension Reinf		m	0.166	0.166	0.166	0.166	0.166
	Cover to reinf		m	0.075	0.075	0.075	0.075	0.075
ds	Dis. From comp. fiber to centroid of tension Reinf		m	1.834	1.834	1.834	1.834	1.834
d'ps	Dis. From comp. fiber to centroid of comp. prestressing steel		m	0.000	0.000	0.000	0.000	0.000
d1xp	Dis. From tens. fiber to centroid of tension prestressing steel		m	0.000	0.000	0.000	0.000	0.000
dps	Dis. From comp. fiber to centroid of tension prestressing steel		m	2.000	2.000	2.000	2.000	2.000
b	Width of the compression face of member		m	8.000	8.000	8.000	6.000	6.000
bw	Web width or diameter of a circular section		m	8.000	8.000	8.000	6.000	6.000
hf	Compression flange depth		m	0.000	0.000	0.000	0.000	0.000
Iz	Moment of inertia of section		m4	16.7880	16.7880	16.7880	14.5130	14.5130
Amc	Section area		m2	16.000	16.000	16.000	12.000	12.000
	Steel choice							
Aps	Tension prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	0 T00.0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
A'ps	Compression prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	0 T00.0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
As	Tension Reinforcement	Number	bars	53	53	53	40	40
		Diameter	mm	28	28	28	22	22
		Area	m2	0.03265	0.03265	0.03265	0.01520	0.01520
A's	Compression Reinforcement	Number	bars	53	53	53	40	40
		Diameter	mm	20	20	20	20	20
		Area	m2	0.01664	0.01664	0.01664	0.01256	0.01256
A'c	Shear reinforcement	Number	bars	8	8	8	5	5
		Diameter	mm	16	16	16	16	16
		Area	m2	0.00162	0.00162	0.00162	0.00101	0.00101
ϕ	Resistance factors for flexure		5.5.4.2	0.90	1.00	1.00	1.00	0.90
ϕv	Resistance factors for shear			0.90	1.00	1.00	1.00	0.90
ϕn	Resistance factors for axial force			1.00	1.00	1.00	1.00	1.00
β1	Stress block factor			0.836	0.836	0.836	0.836	0.836
c	Dis. Between centroid and top fiber		m	0.038	0.038	0.038	0.008	0.008
	For T section behavior		m	0.038	0.038	0.038	0.008	0.008
	For rectangular section behavior		m	0.038	0.038	0.038	0.008	0.008
fpe	Effective stress in the prestressing steel after losses		Mpa	1116	1116	1116	1116	1116
fps	Aver. stress in pres. steel at the time for which the nominal resistance		Mpa	1850	1850	1850	1858	1858
k	Factor depends on type of P.S, Low relaxation strand k = 0.28			0.28	0.28	0.28	0.28	0.28

a	Depth of equivalent stress block	m	0.031	0.031	0.031	0.007	0.007
de	Corresp. effective depth from extreme comp. fiber						
	to centroid of tensile force in the tensile reinf.	m	1.834	1.834	1.834	1.834	1.834
Mn	Nominal resistance	kNm	23268	23268	23268	10707	10707
Mr	Factored resistance	kNm	**20941**	**23268**	**23268**	**10707**	9637
Mu	Flexual moment	kNm	12716	7915	11475	919	1270
(5.7.3.2)	**Flexural moment Checking**		**OK**	**OK**	**OK**	**OK**	**OK**
	Limits for reinforcement						
c/de	Maximum reinforcement		0.02	0.02	0.02	0.00	0.00
	Maximum reinforcement Checking	<= 0.42	**OK**	**OK**	**OK**	**OK**	**OK**
1.2*Mcr	Craking moment	kNm	35423	35423	35423	30172	30172
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**		**OK**	**OK**	**OK**	**OK**	**OK**
(5.7.3.4)	**Conctrol of craking by distr. of reinf for RC member- Check?**		No	Yes	No	Yes	No
	Existing condition for structrure	1,2 or 3	1	1	1	1	1
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.064	0.064	0.064	0.061	0.061
Z	Crack width parameter	N/mm	30000	30000	30000	30000	30000
A	Area of concr. with same centroid as tens. Reinf	m2	0.019	0.019	0.019	0.018	0.018
fsa	Value	Mpa	280	280	280	289	289
0.6*fy		Mpa	240	240	240	240	240
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	240	240	240	240	240
x	Dist. From compression fiber to centroid	m	-	0.296	-	0.238	-
J.d	Arm	m	-	1.735	-	1.755	-
Icr	Moment of inertia of the cracked section	m4	-	0.614	-	0.30	-
fs	Tensile stress in reinforcement fs = Msls / (As*J.d)	Mpa	-	140	-	34	-
	Checking for control cracking fs<fsa		**N.a**	**OK**	**N.a**	**OK**	**N.a**
(5.10.8.2)	**Shrinkage and temperature Reinfocement (side distribution)**						
Areq	Area of required reinf	m2	0.00127	0.00127	0.00127	0.00127	0.00127
	Distribution on sides 10 D16	m2	0.00202	0.00202	0.00202	0.00202	0.00202
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45	0.45
	Checking		**OK**	**OK**	**OK**	**OK**	**OK**
	SHEAR AND TORSION CHECKING						
β	Factor indicating diag. cracked concr. to tension		1.8	2.1	1.9	2.3	2.2
θ	Angle of inclination of diagonal compressive	degree	42.51	38.47	41.78	33.63	35.96
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	90
bv	Effective web width as minimum web width - in dv	m	8.000	8.000	8.000	6.000	6.000
dv	Effective shear depth	m	1.818	1.818	1.818	1.831	1.831
	(de - a/2)	m	1.818	1.818	1.818	1.831	1.831
s	Spacing of stirrups	m	600.000	600.000	600.000	600.000	600.000
ncat	Amount of bars in spacing S	bars	8	8	8	5	5
Av	Shear reinf area in spacing S	m2	0.0016	0.0016	0.0016	0.0010	0.0010
β	Assume		2.0	2.0	2.0	2.0	2.0
θ	Assume	degree	42.51	38.47	41.78	33.63	35.97
v	Shear stress in concrete	kN/m2	736	414	585	235	343
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e_x	Strain in tensile reinforcement		1.88E-03	1.25E-03	1.70E-03	8.02E-04	9.97E-04
	if ex<0, multiple with reduce factor		-	-	-	-	-
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok	Ok
v/f'c	Ratio of shear stress and f'c		0.025	0.014	0.020	0.008	0.011
β	Final value		1.8	2.1	1.9	2.3	2.2
θ	Final value	degree	42.51	38.47	41.78	33.63	35.96
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	11751	13832	12299	11688	11144
Vs	Shear resistance provided by shear reinforcement	kN	2	2	2	2	2
Vp	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0	0
Vn1	Vn1=Vc+Vs+Vp	kN	11753	13834	12301	11689	11145
Vn2	Vn2	kN	109098	109098	109098	82375	82375
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	11753	13834	12301	11689	11145
Vr	Factored shear resistance	kN	**10577**	**13834**	**12301**	**11689**	**10031**
Vu	Shear	kN	9640	6024	8512	2577	3391
(5.8.2.7)	**Shear checking**		**OK**	**OK**	**OK**	**OK**	**OK**

CALCULATION SHEET

BORED PILE CAPACITY

		Item.	Eng.	Date.	Sign.
	DANANG QUANG NGAI EXPRESSWAY	Design			
	DETAIL DESIGN	Check			
	EMPIRICAL ESTIMATION OF PILE CAPACITY	Revise			

AASHTO - LRFD 3rd 2004 & 4th 2007; 22TCN-272-05

ASSUMPTION:

- The rock is reasonably sound
- The rock strength measured during site investigation will not deteriorate during construction when water or drilling fluids are used
- The bottom of the socket is properly cleaned out.

DATA & CALCULATION:

Bored hole name		**ORB13-A1**	
Bottom of pilecap elevation	EL1 =	9.00	
Top of socket elevation	EL2 =	4.29	
Pile tip elevation	EL3 =	1.00	
Pile Length	L =	8.00	m
Diameter of drilled-shaft	D_p =	1.20	m
Pile Cross-Sectional Perimeter	P =	3.77	m
Pile Cross-Sectional Area	A_b =	1.13	m^2
Working normal force at pile head	N =	5440.8	kN
Working normal force at top of socket	P_i =	5386.1	kN
Intack rock modulus	E_i =	25000	MPa
Modulus modification ratio	K_e =	0.05	
Elastic modulus of the insitu rock	$E_r = K_e*E_i$ =	1250.0	MPa
Influence coefficient	$I_p = f(H_s/D_s, E_c/E_r)$ =	0.30	
	H_s/D_s =	2.74	
	E_c/E_r =	22.15	

Pile Concrete comp. strength	f'_c =	30.0	MPa
Concrete Unit Weight	g_c =	24.5	kN/m^3
Modulus of elasticity of concre	E_c =	27691	MPa
Depth of socket	H_s =	3.29	m
Diameter of socket	D_s =	1.20	m
Socket Cross-Sect. Perimeter	P_{soc} =	3.77	m
Socket Cross-Sectional Area	A_{soc} =	1.13	m^2

Figure C10.8.3.5-2 Lrfd (Intack rock modulus)
Figure C10.8.3.5-3 Lrfd (Modulus modification ratio)
Figure C10.8.3.5-1 Lrfd (Influence coefficient)

Rock mass modulus/ intack rock modulus E_m / E_i — *C.10.4.6.5-1-Lrfd 4th*
Atmospheric pressure p_a = 0.101 MPa
Reduction factor to account for jointing α_E — *10.8.3.5.4b-Lrfd 4th*

The elastic shortening of the drilled shaft

$r_e = (\Sigma P_i)*H_s/(A_{soc}*E_c)$ = 0.566 mm

The settlement of base of drilled shaft

$r_{base} = (\Sigma P_i)*I_p/(D_s*E_r)$ = 1.077 mm

$r_e + r_{base}$ = **1.643** mm < 10 mm

Compute the bearing capacity based on Sharft resistance alone
Unit side resistance

Case 1: The drilling fluid used will not form a lubricated film on the sides of the socket

if qu < 1.9 Mpa - may be taken after Carter & Kulhawy 1988 → qs = 0.15*qu — *C10.8.3.5-4*
if qu > 1.9 Mpa - may be taken after Horvath & Kenney 1979 → qs = 0.21*sqrt(qu) — *C10.8.3.5-5*

Case 2: Side of rock socket is considered to be smooth or where the rock is drilled using a drilling slurry

May be taken after Horvath & Kenney 1979 → qs = 0.65*α_E*pa*(qu/pa)^0.5<7.8*pa*(f'c/pa)^0.5
qs0 = 7.8*pa*(f'c/pa)^0.5 — *10.8.3.5.4d-1-Lrfd2007*

Drilling method used in construction: *Case 2*

$Q_R = \phi*Q_n = \phi_s*Q_{SR}$

ϕ_s is the resistance factor - table 10.5.5-3 LRFD

q_u is the uniaxial compressive strength of the rock

Case1

No.	EL_T	EL_B	Depth (m)	RQD (%)	q_u (MPa)	q_s (MPa)	Q_{SR} (kN)	ϕ_s	Q_R (kN)
1	4.29	3.29	1.00	50	48.94	1.47	5538	0.65	3600
2	3.29	2.29	1.00	50	81.57	1.90	7150	0.65	4648
3	2.29	1.00	1.29	50	81.57	1.90	9224	0.65	5995
4									-
5									
6									
7									
8									
Sum			**3.29**				**21912**		**14243**

Case2 *Type: "1: closed joints", "2: open joints"*

No.	Depth (m)	RQD (%)	q_u (MPa)	E_m / E_i	α_E	Type	qs0 (MPa)	q_s (MPa)	q_s - used (MPa)	Q_{SR} (kN)	ϕ_s	Q_R (kN)
1	1.00	50.00	48.94	0.15	0.59	1	13.58	0.85	*0.85*	3201	0.55	1760
2	1.00	50.00	81.57	0.15	0.59	1	13.58	1.10	*1.10*	4132	0.55	2273
3	1.29	50.00	81.57	0.15	0.59	1	13.58	1.10	*1.10*	5331	0.55	2932
4	-	-	-	-	-	-	-	-	-	-	-	-
5	-	-	-	-	-	-	-	-	-	-	-	-
6	-	-	-	-	-	-	-	-	-	-	-	-
7	-	-	-	-	-	-	-	-	-	-	-	-
8	-	-	-	-	-	-	-	-	-	-	-	-
Sum	**3.29**									**12663**		**6965**

Unit base resistance	$q_p = Kb.(p1-p0) + \sigma v$				
Limit pressure determined from presuremeter tests		$p_l =$	5.89	MPa	*C10.8.3.5-7*
At rest total horizontal stress measured at ther base elevation		$p_0 =$	-	MPa	
Coefficient that depen on diameter socket		$K_b =$	4.05		*Table C10.8.3.5-1*
Total vertical stress at the base elevation		$\sigma v =$	-	MPa	
		$q_p =$	-	MPa	
		$\phi =$	0.50		*Table 10.5.5-3*
	$Q_{pR} = Ap . qp$	$Q_{pR} =$	-	kN	
	$Q_R = \phi.Q_{pR}$	$Q_R =$	-	kN	

ESTIMATED PILE CAPACITY:

Pile Structural Capacity	$Q_T = 0.75*0.85*0.85*f'c*Ag$	*18385*	kN	*1874*	T
Pile resistance	Q_R	*6965*	kN	*710*	T
Deducting pile weight		*-170*	kN	*-17*	T
Estimated Pile Capacity		*6795*	kN	*693*	T
Maximum Reaction - ULS	Ok	*5219*	kN	*532*	T

ASSUMPTION:

- The rock is reasonably sound
- The rock strength measured during site investigation will not deteriorate during construction when water or drilling fluids are used
- The bottom of the socket is properly cleaned out.

DATA & CALCULATION:

Bored hole name		**ORB13-A2**	
Bottom of pilecap elevation	$EL1 =$	9.00	
Top of socket elevation	$EL2 =$	2.14	
Pile tip elevation	$EL3 =$	-0.50	
Pile Length	$L =$	9.50	m
Diameter of drilled-shaft	$D_p =$	1.20	m
Pile Cross-Sectional Perimeter	$P =$	3.77	m
Pile Cross-Sectional Area	$A_b =$	1.13	m^2
Working normal force at pile head	$N =$	5482.4	kN
Working normal force at top of socket	$P_i =$	5438.5	kN
Intack rock modulus	$E_i =$	25000	MPa
Modulus modification ratio	$K_e =$	0.05	
Elastic modulus of the insitu rock	$E_r = K_e * E_i =$	1250.0	MPa
Influence coefficient	$I_p = f(H_s/D_s, E_c/E_r) =$	0.30	
	$H_s/D_s =$	2.20	
	$E_c/E_r =$	22.15	

Pile Concrete comp. strength	$f'_c =$	30.0	MPa
Concrete Unit Weight	$g_c =$	24.5	kN/m^3
Modulus of elasticity of concre	$E_c =$	27691	MPa
Depth of socket	$H_s =$	2.64	m
Diameter of socket	$D_s =$	1.20	m
Socket Cross-Sect. Perimeter	$P_{soc} =$	3.77	m
Socket Cross-Sectional Area	$A_{soc} =$	1.13	m^2

Figure C10.8.3.5-2 Lrfd
Figure C10.8.3.5-3 Lrfd
Figure C10.8.3.5-1 Lrfd

Rock mass modulus/ intack rock modulus E_m / E_i — *C.10.4.6.5-1-Lrfd 4th*

Atmospheric pressure $p_a =$ 0.101 MPa

Reduction factor to account for jointing α_E — *10.8.3.5.4b-Lrfd 4th*

The elastic shortening of the drilled shaft

$r_e = (\Sigma P_i)*H_s/(A_{soc}*E_c) =$ 0.458 mm

The settlement of base of drilled shaft

$r_{base} = (\Sigma P_i)*I_p/(D_s*E_r) =$ 1.088 mm

$r_e + r_{base} =$ **1.546** mm < 10 mm

Compute the bearing capacity based on Sharft resistance alone

Unit side resistance

Case 1: The drilling fluid used will not form a lubricated film on the sides of the socket

if qu < 1.9 Mpa - may be taken after Carter & Kulhawy 1988 → qs = 0.15*qu — *C10.8.3.5-4*

if qu > 1.9 Mpa - may be taken after Horvath & Kenney 1979 → qs = 0.21*sqrt(qu) — *C10.8.3.5-5*

Case 2: Side of rock socket is considered to be smooth or where the rock is drilled using a drilling slurry

May be taken after Horvath & Kenney 1979 → qs = 0.65*α_E*pa*(qu/pa)^0.5<7.8*pa*(f'c/pa)^0.5

qs0 = 7.8*pa*(f'c/pa)^0.5 — *10.8.3.5.4d-1-Lrfd2007*

Drilling method used in construction: $\boxed{\textit{Case 2}}$

$Q_R = \phi * Q_n = \phi_s * Q_{SR}$

ϕ_s is the resistance factor - table 10.5.5-3 LRFD

q_u is the uniaxial compressive strength of the rock

Case1

No.	EL_T	EL_B	Depth (m)	RQD (%)	q_u (MPa)	q_s (MPa)	Q_{SR} (kN)	ϕ_s	Q_R (kN)
1	2.14	1.14	1.00	63	47.71	1.45	5468	0.65	3554
2	1.14	0.14	1.00	63	47.71	1.45	5468	0.65	3554
3	0.14	-0.5	0.64	63	67.04	1.72	4149	0.65	2697
Sum			**2.64**				**15085**		**9805**

	DANANG QUANG NGAI EXPRESSWAY DETAIL DESIGN EMPIRICAL ESTIMATION OF PILE CAPACITY	Item.	Eng.	Date.	Sign.
		Design			
		Check			
		Revise			

Case2 — Type: "1: closed joints", "2: open joints"

No.	Depth (m)	RQD (%)	q_u (MPa)	E_m/E_i	α_E	Type	qs0 (MPa)	q_s (MPa)	q_s - used (MPa)	Q_{SR} (kN)	ϕ_s	Q_R (kN)
1	1.00	63.00	47.71	0.51	0.80	1	13.58	1.15	*1.15*	4319	0.55	2376
2	1.00	63.00	47.71	0.51	0.80	1	13.58	1.15	*1.15*	4319	0.55	2376
3	0.64	63.00	67.04	0.51	0.80	1	13.58	1.36	*1.36*	3277	0.55	1802
0	-	-	-	-	-	-	-	-	-	-	-	-
0	-	-	-	-	-	-	-	-	-	-	-	-
0	-	-	-	-	-	-	-	-	-	-	-	-
0	-	-	-	-	-	-	-	-	-	-	-	-
0	-	-	-	-	-	-	-	-	-	-	-	-
Sum	**2.64**									**11916**		**6554**

Unit base resistance	$q_p = Kb.(p1\text{-}p0) + \sigma v$				
Limit pressure determined from presuremeter tests		$p_l =$	5.89	MPa	*C10.8.3.5-7*
At rest total horizontal stress measured at ther base elevation		$p_0 =$	-	MPa	
Coefficient that depen on diameter socket		$K_b =$	3.72		*Table C10.8.3.5-1*
Total vertical stress at the base elevation		$\sigma v =$	-	MPa	
		$q_p =$	-	MPa	
		$\phi =$	0.50		*Table 10.5.5-3*
	$Q_{pR} = Ap \,.\, qp$	$Q_{pR} =$	-	kN	
	$Q_R = \phi.Q_{pR}$	$Q_R =$	-	kN	

ESTIMATED PILE CAPACITY:

Pile Structural Capacity	$Q_T = 0.75*0.85*0.85*f'c*Ag$	*18385*	kN	*1874*	T
Pile resistance	Q_R	*6554*	kN	*668*	T
Deducting pile weight		*-187*	kN	*-19*	T
Estimated Pile Capacity		*6366*	kN	*649*	T
Maximum Reaction - ULS	**Ok**	*5219*	kN	*532*	T

MINISTRY OF TRANSPORT

VIETNAM EXPRESSWAY CORPORATION - PROJECT MANAGEMENT UNIT NO.85

DA NANG - QUANG NGAI EXPRESSWAY DEVELOPMENT PROJECT

PACKAGE: A6

BRIDGE

ORB13

CALCULATION SHEETS

MISCELLANEOUS

HA NOI, 2012

EXPANSION JOINT

I. Displacement

Maximum allowable displacements in longitudinal direction = 40 mm

Maximum displacement 19.4 OK

A1

Unit (mm)

Tải trọng	Symbol	Sign	Displacement	Service
			Case1	a
TU+	TU	+	11.00	1.20
TU-	TU	-	-9.00	1.20
Cr&Sh	CR&SH	-	-9.00	1.20
Other loads		±	2.20	1.00

Max Stretch = 4.6

Max Shrink = -19.4

Maximum displacement 19.4

II. Force in Pier

Galvanised steel dowel

D = 32.0 mm

Number = 12.0 bar

fu = 380 Mpa

Resistance force Rn=0.48*A*fu = 1760.3 kN OK S-6.13.2.7

Load	Symbol	Force (kN)	a	b
Cr&Sh	CR&SH	0	-	0.50
TU	TU	0	-	0.50
EQ		856	1.00	-

Maximale shear force Q = max(CR&SH+TU,EQ)/0.8 = 1070.0 kN

MINISTRY OF TRANSPORT

VIETNAM EXPRESSWAY CORPORATION - PROJECT MANAGEMENT UNIT NO.85

DA NANG - QUANG NGAI EXPRESSWAY DEVELOPMENT PROJECT

PACKAGE: 6

BRIDGE

OP11A

CALCULATION SHEETS

HA NOI, 2012

Table of content - OP11A Bridge

A. Substructure design

1. Abutment A2
2. Bored pile capacity

Da Nang Quang Ngai Expressway project

BRIDGE

OP11a

CALCULATION SHEETS

ABUTMENT A2

HA NOI, 2013

Table of content

1.Structure dimensions and Loads
2. Foundation analysis
3. Elements checks

Assumptions :

1. Bridge is considered to be in seismic with acceleration coeff. A = 0.0310 g
2. The Design of the Abutment accords with Specification for bridge design 22-TCN-272-05 and AASHTO LRFD 2004 for reference
3. Design live load: HL-93 and lane loading 9.3 KN/m

Input :

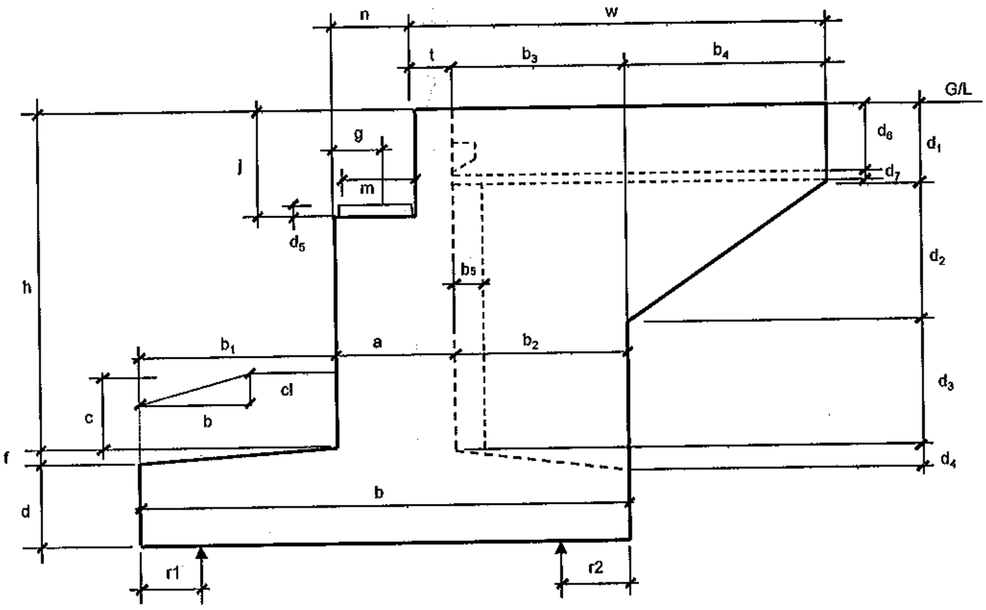
Level Table(at center of abutment)

Level of top of headwall	HTwL	18.692	m
Level of top of bearing	BTL	16.728	m
Level of top of stem abutment	HTL	16.578	m
Level of top of footing	FTL	13.000	m
Level of bottom of footing	FBL	11.000	m
Ground level	GL	12.000	m
Lowest water level	HWL	12.890	m
Skew angle	α	20.00	deg

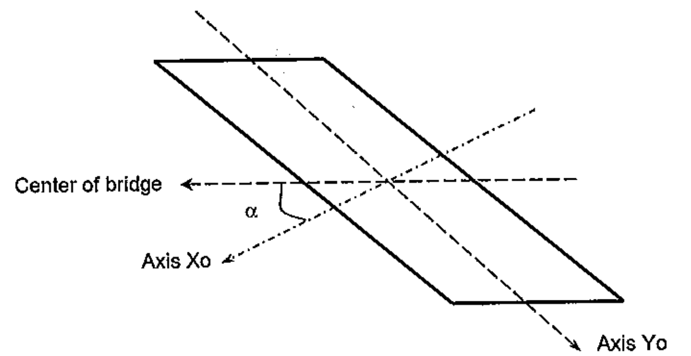
I.Loads from substructure

Abutment dimensions

VERTICAL VIEW



PLAN VIEW



22 22

0

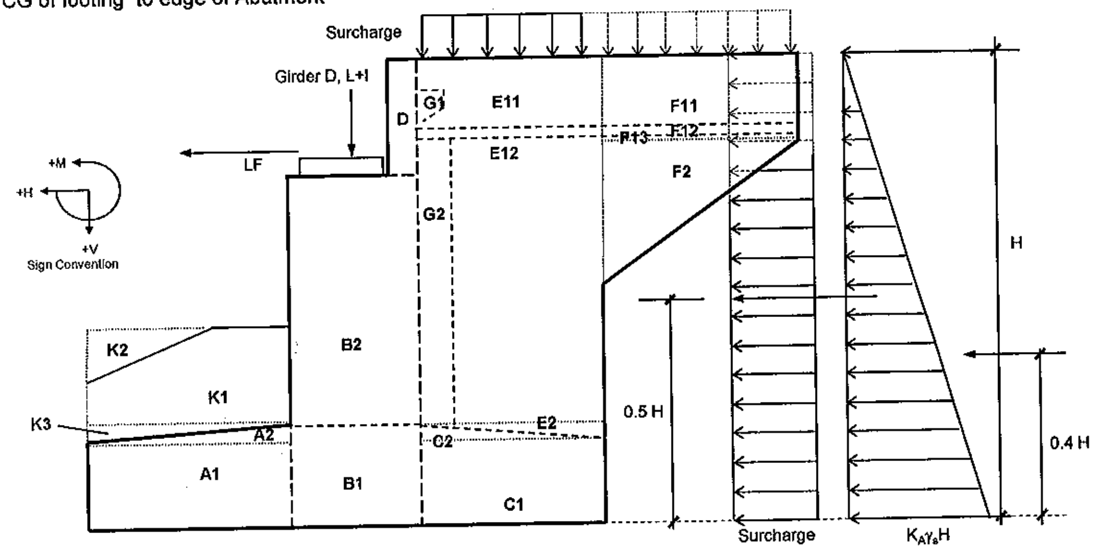
14

Material Unit Weights

• Unit Weight of Reinf. concrete	γ_c	=	24.5 kN/m^3
• Unit Weight of Soil	γ_s	=	18.0 kN/m^3
• Unit Bouyancy Weight of Soil	γ_{sbo}	=	8.2 kN/m^3

ABUTMENT DIMENSIONS (IN METRES)					
Item	Symbol	Value	Item	Symbol	Value
Height of stem	h	5.692	Horizontal Dimension	b_3	3.000
Footing Width	b	6.500	Horizontal Dimension	b_4	2.975
Stem Width	a	1.500	Horizontal Dimension	b_5	0.500
Footing Depth	d	2.000	Vertical Dimension	d_1	0.930
Footing Slope	f	0.000	Vertical Dimension	d_2	2.975
Width of stem at bearing	n	1.000	Vertical Dimension	d_3	1.787
Ballast Wall Height	j	2.114	Vertical Dimension	d_4	0.000
Ballast Wall Thickness	t	0.500	Vertical Dimension	d_5	0.150
Wingwall Length	w	6.700	Vertical Dimension	d_6	1.200
Soil Cover at Toe	c	0.000	Vertical Dimension	d_7	0.300
Girder Reaction	g	0.500	With of bearing pad	m	0.550
Distance to cl of pile	r1	1.000	Wingwall Thickness	u1	0.500
Horizontal Dimension	b_1	2.000	Wingwall Thickness	u2	0.500
Horizontal Dimension	b_2	3.000	Distance to cl of pile	r2	1.000

	cos (α)	=	0.94
Slope front of abutment	cl	=	0.00 m
	bl	=	0.00 m
Width of Abutment	L		12.600 m
Width of abutment (inclined direction)	Ltr		13.409 m
Height of Abutment	Ht	=	7.69 m
Distance from CG of footing to edge of Abutment	b/2	=	3.25 m



1. Self weight of Abutment (DC)

Description	Area (m^2)	Length (m)	Force (kN)	X[1] (m)	Arm[2] (m)	Moment (kN•m)
SW of Abutment (DC)						
Section A1	4.000	13.409	1314	1.000	2.250	2957
Section A2	-	13.409	-	1.333	1.917	-
Section B1	3.000	13.409	986	2.750	0.500	493
Section B2	5.367	13.409	1763	2.750	0.500	882
Section C1	6.000	13.409	1971	5.000	-1.750	-3449
Section C2	-	13.409	-	4.500	-1.250	-
Section D	1.057	13.409	347	3.250	-	-
Section E11	1.890	0.500	23	5.000	-1.750	-41
Section E12	14.286	0.500	175	5.000	-1.750	-306
Part extra stem	3.846	0.740	70	5.750	-2.500	-174
Section F11	3.570	0.500	44	7.988	-4.738	-207
Section F12	0.896	0.500	11	6.488	-3.238	-36
Section F13	-1.696	0.500	-21	7.988	-4.738	98
Section F2	4.425	0.500	54	7.492	-4.242	-230
Section G1	0.135	12.909	255	3.650	-0.400	-102
Section G2	0.125	8.384	26	3.750	-0.500	-13
Bearing seats (w1seat= 0.60m)	0.083	3.000	9	2.500	0.750	7
Curbs +Handrail on Abutment	0.50	6.700	89	6.350	-3.100	-275
Total SW of Abutment (DC)			***7116***			***-397***
Tranverser moment			*345*		*6.175*	*2131*

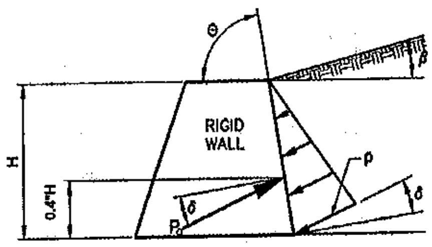
Notes: 1. Distance 'X' is measured horizontally from Toe of Retaining to CG of Section
2. Moment 'Arm' is measured from CG horizontally and from Underside of Footing Vertically.

2. Earth on Abutment (EV)

Description	Area (m^2)	Length (m)	Force (kN)	X[1] (m)	Arm[2] (m)	Moment (kN•m)
Earth on Abutment (EV)						
Section E1	17.08	12.909	3968	5.000	-1.750	-6943
Section E2	-	12.909	-	5.500	-2.250	-
Section E3	-	0.500	-	6.500	-3.250	-
Section K1	-	13.409	-	1.000	2.250	-
Section K2	-	13.409	-	-	3.250	-
Section K3	-	13.409	-	0.667	2.583	-
Total Earth on Footing			*3968*			*-6943*

3. Horizontal Earth Pressure on Abutment (EH)

To be safe, horizontal earth pressure at front face of abutment may be neglected.
Horizontal earth pressure at behind face of abutment shall be considered.



- Height for horizontal earth pressure — H = 7.69 m
- Width for horizontal earth pressure — W 13.41 m
- Density of Soil — γ_s = 1835 kg/m^3
- Internal Friction Angle of Soil — ϕ'_f = 30.0 deg
- Incline angle of back face wall — θ = 90.0 deg
- Friction angle between fill and wall — δ = 30.0 deg
- Incline angle of fill soil — β = 0.0 deg
- Gravitational acceleration — g = 9.81 m/s^2
- Basic earth pressure

p = K.γs. g. Z .10^-9 (Mpa, Z:mm)
K: taken as Ka (assume wall move or deflect sufficiently to reach minimum active conditions)

$$Ka = \frac{\sin^2(\theta + \phi'_f)}{\Gamma.[\sin^2\theta.\sin(\theta - \delta)]}$$

Γ = 2.914
Ka = 0.297
p = 0.041 Mpa

$$\Gamma = \left[1 + \sqrt{\frac{\sin(\phi'_f + \delta).\sin(\phi'_f - \beta)}{\sin(\theta - \delta).\sin(\theta + \beta)}}\right]^{-2}$$

Horizontal earth pressure:
- Ea = 0,5. p. Z . B . 10^3 (kN)
- M= Ea . 0,4H
- Horizontal Earth Pressure act at a height of 0.4 H

Ea = 2122 kN
M = 6529 kNm

<S 3.11.5.1>

4. Earth Pressure on Abutment due to Surcharge (ES)

Equivalent height of soil for highway loading taken from Table 3.11.6.2.1

H=	1.50m heq=	1.7 m
H=	3.00m heq=	1.2 m
H=	6.00m heq=	0.76 m
H=	9.00m heq=	0.61 m
H=	7.69m heq=	0.68 m (Linear interpolation)

- **Vertical force**

ESv = 489 kN
ev = -1.75 m
M = -856 kNm

- **Horizontal force**

$$\Delta p = k\gamma_s g h_{eq} 10^{-9}$$

ESh = 373 kN
eh = 3.85 m
M = 1433 kNm

5. Earthquake effects

Bridge is located at: Thang Binh district - Quang Nam province

According to TCXDVN 375:2006 and 22TCN272-05, bridge is in seismic zone 2and acceleration coefficient as below

- Peak ground acceleration coefficient   A = 0.0310 g

5.1. Seismic active lateral Earth pressure (E_{AE})

• Backfill slop angle	i	=	0.0 deg
• Slope of wall to vertical	β'	=	0.0 deg
• Angle of friction of soil	φ	=	30.0 deg
• Angle of friction between soil and abutment	δ	=	30.0 deg
• Horizontal acceleration coefficient	kh	=	0.047
• Vertical acceleration coefficient	kv	=	0.019
• Angle $\theta = \arctan(k_h/(1-k_v))$	θ	=	2.7 deg

$$K_{AE} = \frac{\cos^2(\phi-\theta-\beta)}{\cos\theta.\cos^2\beta.\cos(\delta+\beta+\theta)} x \left[1+\sqrt{\frac{\sin(\phi+\delta).\sin(\phi-\theta-i)}{\cos(\delta+\beta+\theta).\cos(i-\beta)}}\right]^{-2}$$

- Seismic active lateral Earth pressure coefficient K_{AE} = 0.33

- $E_{AE} = 0,5.\ g.\ \gamma.\ H^2\ .\ (1\text{-}kv)\ .\ K_{AE}.\ 10^{\wedge}\text{-}9$ (kN/m)

- Seismic active lateral Earth pressure coefficient E_{AE} = 2314 kN

$M_{AE} = E_{AS}*0.3H+(E_{AE} - E_{AS})*0.6H$ M_{AE} = 5783 KNm   <A.11.1.1.1>

E_{AS} is the static component of seismic active pressure calculated with θ = kv = 0

5.2. Earthquake effects to abutment (EQ)

Seismic force for substructures: elements above ground Fh = Csm* W; elements under ground Fh = A*S*W

• Soil profile type	Soil type		I
• Site Coefficients.	S	=	1.0
• Elastic Seismic Response Coefficient	2.5A	=	0.078
Csm = 1.2*A*S/ Tm^2/3 ≤ 2.5*A	Csm	=	0.049
• Period of vibration of the fundamental mode			
Tm = 2*pi()*sqrt(m/k)	Tm	=	0.659 s

Decription	Area (m^2)	Length (m)	Force (kN)	X[1] (m)	Arm[2] (m)	Moment (kN•m)
Section A1	4.000	13.409	41	-	1.000	41
Section A2	-	13.409	-	-	2.000	-
Section B1	3.000	13.409	31	-	1.000	31
Section B2	5.367	13.409	87	-	3.789	328
Section C1	6.000	13.409	61	-	1.000	61
Section C2	-	13.409	-	-	2.000	-
Section D	1.057	13.409	17	-	6.635	113
Section E11	1.890	0.500	1	-	5.092	4
Section E12	14.286	0.500	5	-	2.096	
Section E2	3.846	0.740	2	-	2.000	4
Section F11	3.570	0.500	1	-	5.092	7
Section F12	0.896	0.500	0	-	4.342	
Section F13	-1.696	0.500	-1	-	5.977	
Section F2	4.425	0.500	2	-	5.770	10
Section G1	0.135	12.909	1	-	4.979	7
Section G2	0.125	8.384	1	-	2.096	2
Total EQ of Abutment Selfweight			*249*			*607*

6. Braking Force(BR)

Take 50 *% Braking Force for this Abutment (Free Bearing)*

• Number of lanes	n		[redacted] lanes	
• Multiple presence factor	m	=	0.85	
• Take 25 % of Truck load				
BR = 25% * n * m * (2*145+35)	BR	=	104 kN	Long. Axis
• Acting at 1.8m higher of road face	e	=	9.6 m	
	Mlong	=	991 KNm	Long. Axis

7. Centrifucal Force , CE (3.6.3)

• Plan of bridge (1:"straight",2: "Curve")			1	
• Design Speed	V	=	120 km/h	
	V	=	33.3 m/s	
	R		- m	
$C = 4/3*(V^2/gR)$	C		-	
Acting at 1.8m higher of road face				
CE = n * m * (2*145+35) * C	CE	=	0.00 KN	
	e	=	9.58 m	
	Mtrans	=	0.00 KNm	Trans. Axis

8. Water Load (WA) *:NA*

II.Loads from superstructure

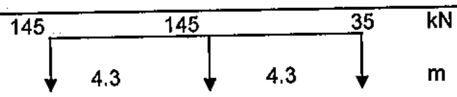
Item	Sign	Value	Unit
Span length	Lsp	21.00	m
Span between bearings	Lb	20.20	m
Skew angle	α	20.00	deg
Deck slab length	Ldeck	21.00	m
Bridge Width	Bc	12.48	m
Girder height	hgi	1.20	m
Deck slab depth	hdkslab	0.22	m
Asphalt depth	has	0.084	m
Unit weight of concrete	γc	24.50	kN/m3
Unit weight of asphalt concrete	γa	22.10	kN/m3

1.Dead loads (DC): One span at abutment

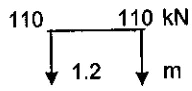
Item	Sign	Value	Unit
1.1.Girders			
Weight of 1 girder	DC	295.47	kN
Number of girders	n	5	Girders
Sum of girders weight	DC	1477.35	kN
Precast Planks	DC	298.66	kN
Diaphragm	DC	193.18	kN
Total	DC	1969.19	kN
1.2.Deck slab			
Deck slab	DC	1387.15	kN
1.3.Pavement			
Asphalt concrete	DW	447.62	kN
1.4.Handrail			
Handrail + median	DC	568.80	kN

2.Live load (LL):

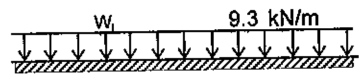
Truck



Tandem



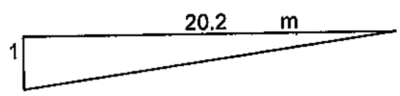
Lane load



Pedestrial Wpd= 0.0 kN/m2 22 22 22

Considerate structure as a simple span

Reaction Influence 0 0



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Number of lanes	n	3
Multiple presence factor	m	0.85
Dynamic load allowance	1+IM	1.25

*Reaction = [(1+IM)*Vehicle + Laneload]*n*m*

Truck	P1(kN)	P2(kN)	P3(kN)	Sum(kN)	Total(kN)
Axle load	145	145	35		
Influence value	1.000	0.787	0.574		
Reaction	145.0	114.1	20.1	279.2	890.1

Tandem	P1(kN)	P2(kN)	Sum(kN)	Total(kN)
Axle load	110	110		
Influence value	1.000	0.941		
Reaction	110	103.5	213.5	680.4

Lane load	Wl(kN/m)	Total(kN)
Value	9.3	
Influence value	10.1	
Reaction	93.9	239.5

Pedestrian	Wdb(kN)	Total(kN)
Reaction	0.0	0.0

3.Earthquake effects on superstructrue (EQ)

Longitudinal moveable bearings at Abutment

Horizontal force from superstructure due to EQ - transverse direction

At bearing — Heq = 107 kN

4.Uniform Temperature, Srinkage& Creep(TU+SH&CR)

Bearing displacement due to uniform temperature and shrinkage creep — Δu = 0.026 m

$H = G.A.\Delta u/h_{rt}$ <14.6.3.1-2>

Shear modulus G — G = 1 MPa

Bearing area — A = 0.165 m2

Height of elastomeric layers — hrt 0.065 m

Number of bearing — nb = 5 bears

Horizontal force due to TU+SH&CR

Acting at top of bearing — H(tu+sh+cr) = 330 kN

5.Wind loads (Ws)

5.1.Tranverse wind on superstructure (WS)

Wind zone — Zone III

Basic 3 second gust wind — Vb = 53.00 m/s

Correction factor — S = 1.09

Design wind velocity — V = 57.77 m/s

Drag coefficient — Cd = 1.36

Overall width of bridge — b = 12.48 m

Depth of superstructure(including solid parapet) — d = 2.48 m

b/d = 5.03

Windy obstructed area of superstructure — At = 52.10 m2

Force due to tranverse wind

Fhy=max(0.0006*V2*At*Cd,1.8*At) (kN) — Fhy = 142.2 kN <3.8.1>

5.2.Wind load on vehicles (WL)

Transverse wind on vehicles — Wltran = 1.50 kN/m

Transverse horizontal force due to wind on liveload

At 1.8m from surface — Fhy = 31.50 kN

6.Combinations

Loads from superstructure to Abutment

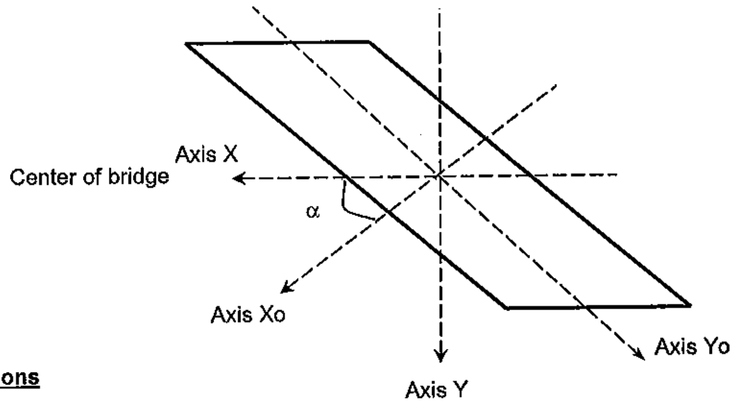
Loads at bottom of stem		Vertical		Longitudinal			Tranversal		
Loads	Sign	N (kN)	x (m)	Hx (kN)	z_1 (m)	My (kN•m)	Hy (kN)	y (m)	Mx (kN•m)
Girder + Deckslab	DC	1678	0.25			420			
Handrail	DC	284	0.25			71			
Pavement	DW	224	0.25			56			
LiveLoad	LL	1130	0.25			282		1.38	1553
Pedestrial	PL	0	0.25			0		-	-
Trans. wind on Struc.	WS						71	3.58	254
Trans. wind on vehi.	WL						16	5.38	85
Eearth quake	EQ						107	3.58	384
TU+SH&CR	TU+SH&CR			330	3.58	1181			

Loads at bottom of pilecap		Vertical		Longitudinal			Tranversal		
Loads	Sign	N (kN)	x (m)	Hx (kN)	z_1 (m)	My (kN•m)	Hy (kN)	y (m)	Mx (kN•m)
Girder + Deckslab	DC	1678	0.75			1259			
Handrail	DC	284	0.75			213			
Pavement	DW	224	0.75			168			
LiveLoad	LL	1130	0.75			847		1.38	1553
Pedestrial	PL	0	0.75			0		-	-
Trans. wind on Struc.	WS						71	5.58	397
Trans. wind on vehi.	WL						16	7.38	116
Eearth quake	EQ						107	5.58	599
TU+SH&CR	TU+SH&CR			330	5.58	1841			

Loads		Load combinations						
	Sign	Str-IA	Str-IB	Str-IIIA	Str-IIIB	Ser-I	Ext-IA	Ext-IB
Girder + Deckslab	DC	1.25	0.90	1.25	0.90	1.00	1.25	0.90
Handrail	DC	1.25	0.90	1.25	0.90	1.00	1.25	0.90
Pavement	DW	1.50	0.65	1.50	0.65	1.00	1.50	0.65
LiveLoad	LL	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Pedestrial	PL	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Trans. wind on Struc.	WS			0.40	0.40	0.30		
Trans. wind on vehi.	WL			1.00	1.00	1.00		
Eearth quake	EQ						1.00	1.00
TU+SH&CR	TU+SH&CR	0.50	0.50	0.50	0.50	1.00		

Load combinations at bottom of stem					
		Longitudinal		Tranversal	
Load combinations	N (kN)	Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	4766	165	1782	0	2718
Strength Str-IB	3889	165	1563	0	2718
Strength Str-IIIA	4314	165	1669	44	2283
Strength Str-IIIB	3437	165	1450	44	2283
Service Ser-I	3316	330	2010	37	1714
Extreme Ext-IA	3354	0	838	107	1161
Extreme Ext-IB	2477	0	619	107	1161

Load combinations at bottom of pilecap					
		Longitudinal		Tranversal	
Load combinations	N (kN)	Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	4766	165	4495	0	2718
Strength Str-IB	3889	165	3837	0	2718
Strength Str-IIIA	4314	165	4156	44	2372
Strength Str-IIIB	3437	165	3498	44	2372
Service Ser-I	3316	330	4328	37	1788
Extreme Ext-IA	3354	0	2515	107	1376
Extreme Ext-IB	2477	0	1857	107	1376



III.Load Combinations

1.Loads from substructure

Loads at bottom of pilecap	Sign	Vertical		Longitudinal			Tranversal		
		N (kN)	x (m)	H_x (kN)	z_1 (m)	M_y (kN•m)	H_y (kN)	y (m)	M_x (kN•m)
Self weight of Abutment	DC	7116				-397			465.086
Soils on pilecap	EV	3968				-6943			
Horizontal Earth Pressure	EH			1994		6135			
Vertical Surcharge	L_{Sv}	489				-856			
Horizontal Surcharge	L_{Sh}			350		1347			
Braking Force	BR			104		991			
Centrifucal Force	CE			-		-	-		-
Buoyancy of Abutment	WA	-1616				0			
Buoyancy of Earth on Abutment	WA	-				-			
Earthquake effects to Abutment	EQ			249		607	75		182
Earthquake effects to soil	E_{AE}			2175		5435			

Table of load factors

Loads	Sign	Load combinations						
		Str-IA	Str-IB	Str-IIIA	Str-IIIB	Ser-I	Ext-IA	Ext-IB
Self weight of Abutment	DC	1.25	0.90	1.25	0.90	1.00	1.25	0.90
Soils on pilecap	EV	1.35	0.90	1.35	0.90	1.00	1.35	0.90
Horizontal Earth Pressure	EH	1.50	0.90	1.50	0.90	1.00	0.00	0.00
Vertical Surcharge	L_{Sv}	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Horizontal Surcharge	L_{Sh}	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Braking Force	BR	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Centrifucal Force	CE	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Buoyancy of Abutment	WA	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Buoyancy of Earth on Abutment	WA	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Earthquake effects to Abutment	EQ						1.00	1.00
Earthquake effects to soil	E_{AE}						1.00	1.00

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Load Combination at bottom of pilecap					
Load combinations	N (kN)	Longitudinal Hx (kN)	Longitudinal My (kN.m)	Tranversal Hy (kN)	Tranversal Mx (kN.m)
Strength Str-IA	13491	3785	1924	0	581
Strength Str-IB	9215	2589	1507	0	419
Strength Str-IIIA	13296	3603	1332	0	581
Strength Str-IIIB	9019	2407	915	0	419
Service Ser-I	9957	2448	275	0	465
Extreme Ext IA	12880	2651	-3088	75	763
Extreme Ext IB	8604	2651	175	75	601

2. Loads from superstructure

Load Combination at bottom of pilecap					
		Longitudinal		Tranversal	
Load combinations	N (kN)	Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	4766	165	4495	0	2718
Strength Str-IB	3889	165	3837	0	2718
Strength Str-IIIA	4314	165	4156	44	2372
Strength Str-IIIB	3437	165	3498	44	2372
Service Ser-I	3316	330	4328	37	1788
Extreme Ext-IA	3354	0	2515	107	1376
Extreme Ext-IB	2477	0	1857	107	1376

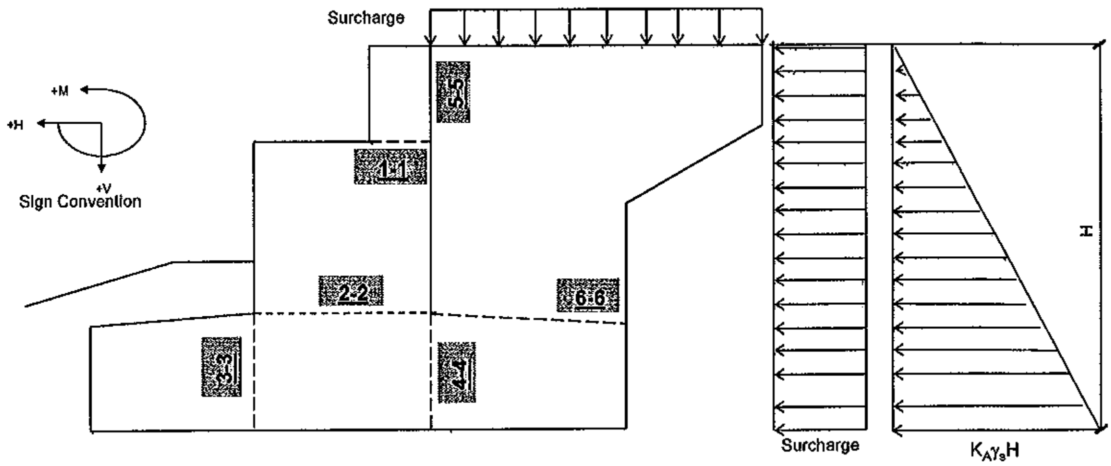
3. Total loads at bottom of pilecap

Load Combination at bottom of pilecap					
		Longitudinal		Tranversal	
Load combinations	N (kN)	Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	18257	3950	6419	0	3299
Strength Str-IB	13104	2754	5344	0	3137
Strength Str-IIIA	17609	3768	5488	44	2953
Strength Str-IIIB	12456	2572	4412	44	2790
Service Ser-I	13273	2778	4603	37	2253
Extreme Ext-IA	16234	2651	-573	182	2139
Extreme Ext-IB	11080	2651	2033	182	1976

IV.Elements checking

The abutment walls shall be checked at sections 1-1, 2-2, 3-3, 4-4, 5-5

1. Calculate internal force of sections



1.1 Section 1-1
Table of Load Factors

Loads	Sign	Service I	Strength I		Extreme I
		Ser-I	Str-IA	Str-IB	Ext-I
Selfweight	DC	1.00	1.25	0.90	1.25
Horizontal Earth Pressure	EH	1.00	1.50	0.90	
Surchage (Horizontal)	LS_h	1.00	1.75	1.75	0.50
Horizontal Seismic Earth Pressure	E_{AE}				1.50
Abutment earthquake force	EQ				1.00

Unfactored Loads	N	Longitudinal		Tranversal	
		Hx	My	Hy	Mx
	(kN)	(kN)	(kN.m)	(kN)	(kN.m)
Selfweight	603		-102		
Horizontal Earth Pressure		171	144		
Surcharge (horizontal)		241	255		
Horizontal Seismic Earth Pressure		186	128		
Abutment earthquake force		18	19	6	6

Load Combination at bottom of headwall					
Load combinations	N	Longitudinal		Tranversal	
		Hx	My	Hy	Mx
	(kN)	(kN)	(kN.m)	(kN)	22
Service Ser-I	603	412	297	0	0
Strength Str-IA	753	678	535	0	0
Strength Str-IB	542	576	484	0	0
Extreme Ext-I	753	418	211	6	6

22.0

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1.2 Section 2-2 14
Table of Load Factors

Loads	Sign	Service I	Strength I		Extreme I
		Ser-I	Str-IA	Str-IB	Ext-I
Selfweight	DC	1.00	1.25	0.90	1.25
Superstructure Dead Load	DC	1.00	1.25	0.90	1.25
Pavement	DW	1.00	1.50	0.65	1.50
Handrail+curb	DC	1.00	1.25	0.90	1.25
Live Load	LL	1.00	1.75	1.75	0.50
Horizontal Earth Pressure	EH	1.00	1.50	0.90	
Surchage (Horizontal)	LS_h	1.00	1.75	1.75	0.50
TU+SH&CR	TU+SH&CR	1.00	0.50	0.50	
Horizontal Seismic Earth Pressure	E_{AE}				1.50
Abutment earthquake force	EQ				1.00

Unfactored Loads	N (kN)	Longitudinal Hx (kN)	Longitudinal My (kN.m)	Tranversal Hy (kN)	Tranversal Mx (kN.m)
Selfweight	2366		-199		
Superstructure Dead Load	1678		420		
Pavement	224		56		
Handrail+curb	284		71		
Live Load	1130		282		1553
Horizontal Earth Pressure		1236	2815		
Surchage (Horizontal)		337	959		
TU+SH&CR		330	1181		
Horizontal Seismic Earth Pressure		1348	2494		
Abutment earthquake force		105	241	64	192

Load Combination at bottom of stem wall

Load combinations	N (kN)	Longitudinal Hx (kN)	Longitudinal My (kN.m)	Tranversal Hy (kN)	Tranversal Mx (kN.m)
Service Ser-I	5682	1903	5584	0	1553
Strength Str-IA	7723	2609	7433	0	2718
Strength Str-IB	6018	1867	5595	0	2718
Extreme Ext-I	6311	2296	5050	64	969

1.3 Section 3-3

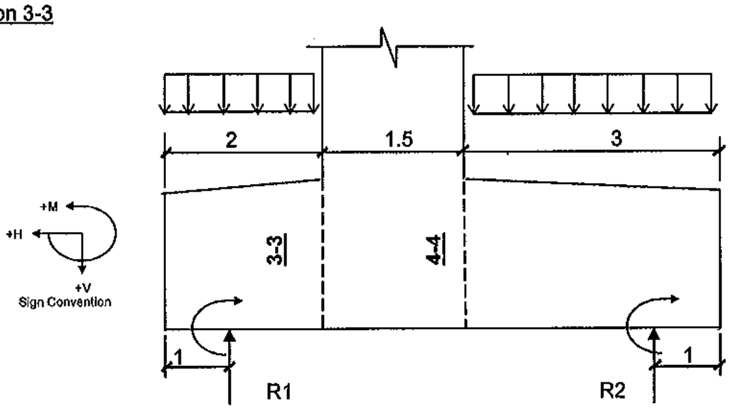


Table of Load Factors

Loads	Sign	Service I Ser-I	Strength I Str-IA	Strength I Str-IB	Extreme I Ext-I
Selfweight at front side	DC	1.00	1.25	0.90	1.25
Vertical soil on foot at front side	DC	1.00	1.35	0.90	1.35
Reaction of piles	RE	1.00	1.00	1.00	1.00

Unfactored Loads		N (kN)	Longitudinal Hx (kN)	Longitudinal My (kN.m)	Tranversal Hy (kN)	Tranversal Mx (kN.m)
Selfweight at front side		1314		1314		
Vertical soil on foot at front side		0		0		
Reaction of piles	Ser-I	-8441	-1575	-5154	21	55
	Str-IA	-11614	-2237	-6947	61	142
	Str-IB	-8298	-1553	-5063	42	101
	Ext-I	-8906	-1508	-5658	-63	-79

Load Combination at section 3-3

Load combinations	N (kN)	Longitudinal Hx (kN)	Longitudinal My (kN.m)	Tranversal Hy (kN)	Tranversal Mx (kN.m)
Service Ser-I	-7127	-1575	-3840	21	55
Strength Str-IA	-9972	-2237	-5304	61	142
Strength Str-IB	-7115	-1553	-3881	42	101
Extreme Ext-I	-7264	-1508	-4016	-63	-79

1.4 Section 4-4

Table of Load Factors

Loads	Sign	Service I	Strength I		Extreme I
		Ser-I	Str-IA	Str-IB	Ext-I
Selfweight at behind side	DC	1.00	1.25	0.90	1.25
Vertical soil on foot at behind side	DC	1.00	1.35	0.90	1.35
Surcharge(Vertical)	EV	1.00	1.75	1.75	0.50
Reaction of piles	RE	1.00	1.00	1.00	1.00

Unfactored Loads			Longitudinal		Tranversal	
		N (kN)	Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Selfweight of behind side		2327		-3762		
Vertical soil on foot at behind side		3968		-5952		
Surcharge(Vertical)		489		-734		
Reaction of piles	Ser-I	-4586	-1181	11638	-60	-122
	Str-IA	-6220	-1678	15941	-61	-122
	Str-IB	-4377	-1165	11179	#VALUE!	-83
	Ext-I	-7202	-1131	16839	-123	-221

Load Combination at section 4-4					
		Longitudinal		Tranversal	
Load combinations	N (kN)	Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Service Ser-I	2197	-1181	1191	-60	-122
Strength Str-IA	2901	-1678	1921	-61	-122
Strength Str-IB	2145	-1165	1153	#VALUE!	-83
Extreme Ext-I	1308	-1131	3735	-123	-221

1.4 Section 5-5 & 6-6

Slope of triang pressure	tgß	=	5.35	
Uniform horizontal pressure	U.p	=	3.61	kN/m2

Load Combination at section 5-5					
		Horizontal		Vertical	
Load combinations	N (kN)	Hx (kN)	Mx (kN.m)	Hy (kN)	My (kN.m)
Service Ser-I		74	172		
Strength Str-IA		116	273		

Load Combination at section 6-6					
		Horizontal		Vertical	
Load combinations	N (kN)	Hx (kN)	Mx (kN.m)	Hy (kN)	My (kN.m)
Service Ser-I				203	179
Strength Str-IA				315	278

2. Elements Checking

2.1. Bearing Resistance

<S.5.7.5>

The case of absence of confinement reinforcement in the conrete supporting the bearing device
Factored bearing resistance shall be taken

$Pr = \phi. Pn = \phi. 0.85. f'c. A1. m$

Dimension of bearing plate	w0	=	0.550 m	
	b0	=	0.600 m	
Area under bearing device	A1	=	0.330 m2	
Distributed width and length	w	=	1.000 m	
	b	=	1.050 m	
Notational area	A2	=	1.050 m2	

Where supporting surface is wider on all sides than loades area

$m = sqrt(A2/A1) \leq 2.0$ case 1

where loaded area is subjected to nouniformly distributed bearing

$m = 0.75*sqrt(A2/A1) \leq 1.5$ case 2

Modification factor	case 1	m	=	1.784	
Resistance factor		ϕ	=	0.700	<S.5.5.4.2>
Factored bearing resistance		Pr	=	10507 kN	> Pu
Bearing reaction of approach bridge		Pu	=	3638 kN	Ok

$Pu = 1.25*DC + 1.5*DW + 1.75*LL$

In case factored applied load exceeds the factored resistance,
provision shall be made to resist the bursting and spalling force in article 5.10.9

<S.5.10.9.7.2>

Factored bearing resistance shall be taken

$Pr = \phi. fn. Ab$

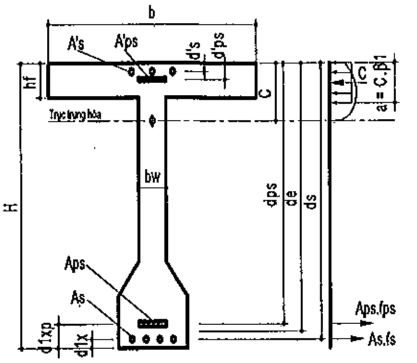
fn take the lesser of

$fn = 0.7. f'ci. sqrt(A/Ag)$ and

$fn = 2.25. f'ci$

	fn	=	37.46 MPa	
Maximum area of the portion of supporting surface	A	=	1.050 m2	
Gross area of bearing plate	Ag	=	0.330 m2	
Effective net area of bearing plate, Ag minus stud of bearing	Ab	=	0.330 m2	
Nominal concrete strength at time of application	f'ci	=	30 MPa	
Factored bearing resistance	Pr	=	8653 kN	Ok

MATERIALS			
NORMAL CONCRETE			
fc	Compressive Strength of concrete at 28 days	Mpa	30
Ec	Modulus of Elasticity	Mpa	27691
fr	Modulus of Rupture	Mpa	3.5
gc	Unit weight of concrete	kN/m3	24.5
PRESTRESSING STEEL			
fpu	Tensile strength of prestressing steel	Mpa	1860
fpy	Yield strength of prestressing steel	Mpa	1670
Ep	Modulus of Elasticity	Mpa	195000
REINFORCEMENT			
fy	Yield strength	Mpa	400
Es	Modulus of Elasticity	Mpa	200000
nc	Ratio Es/Ec		7

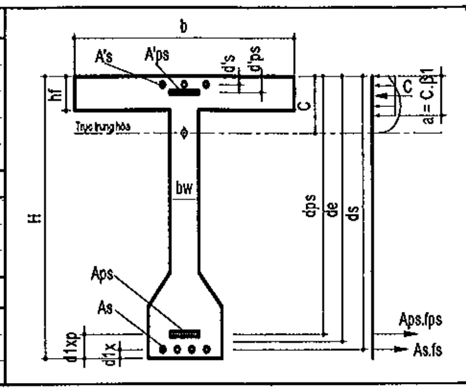


Sign	Parameters		Unit	Sections 1-1	1-1	2-2	2-2	2-2
INTERNAL FORCES AT SECTIOIN								
	Combination			Strength	Service	Service	Strength	Extreme
Qu	Shear		kN	678	412	1903	2609	2296
Mu	Flexural Moment		kNm	535	297	5584	7433	5050
Nu	Axial load		kN	753	603	5682	7723	6311
Tu	Torsional Moment		kNm	0	0	0	0	0
FLEXURAL MOMENT CHECKING								
H	Section height		m	0.500	0.500	1.500	1.500	1.500
d's	Dis. From comp. fiber to centroid of comp. Reinf		m	0.058	0.058	0.058	0.058	0.058
d1x	Dis. From tens. fiber to centroid of tension Reinf		m	0.058	0.058	0.059	0.059	0.059
	Cover to reinf		m	0.050	0.050	0.050	0.050	0.050
ds	Dis. From comp. fiber to centroid of tension Reinf		m	0.442	0.442	1.441	1.441	1.441
d'ps	Dis. From comp. fiber to centroid of comp. prestressing steel		m	0.000	0.000	0.000	0.000	0.000
d1xp	Dis. From tens. fiber to centroid of tension prestressing steel		m	0.000	0.000	0.000	0.000	0.000
dps	Dis. From comp. fiber to centroid of tension prestressing steel		m	0.500	0.500	1.500	1.500	1.500
b	Width of the compression face of member		m	13.409	13.409	13.409	13.409	13.409
bw	Web width or diameter of a circular section		m	13.409	13.409	13.409	13.409	13.409
hf	Compression flange depth		m	0.000	0.000	0.000	0.000	0.000
Iz	Moment of inertia of section		m4	0.140	0.140	3.771	3.771	3.771
Amc	Section area		m2	6.704	6.704	20.113	20.113	20.113
	Steel choice							
Aps	Tension prestressing steel	P.S type		0	0	0	0	0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
A'ps	Compression prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	0 T00.0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
As	Tension Reinforcement	Number	bars	83	83	77	77	77
		Diameter	mm	16	16	22	22	22
		Area	m2	0.01677	0.01677	0.02926	0.02926	0.02926
A's	Compression Reinforcement	Number	bars	83	83	77	77	77
		Diameter	mm	16	16	16	16	16
		Area	m2	0.01677	0.01677	0.01555	0.01555	0.01555
A'c	Shear reinforcement	Number	bars	21	21	20	20	20
		Diameter	mm	14	14	14	14	14
		Area	m2	0.00317	0.00317	0.00302	0.00302	0.00302
ϕ	Resistance factors for flexure		5.5.4.2	0.90	1.00	1.00	0.90	1.00
ϕv	Resistance factors for shear			0.90	1.00	1.00	0.90	1.00
ϕn	Resistance factors for axial force			1.00	1.00	1.00	1.00	1.00
β1	Stress block factor			0.836	0.836	0.836	0.836	0.836
c	Dis. Between centroid and top fiber		m	0.000	0.000	0.019	0.019	0.019
	For T section behavior		m	0.000	0.000	0.019	0.019	0.019
	For rectangular section behavior		m	0.000	0.000	0.019	0.019	0.019
fpe	Effective stress in the prestressing steel after losses		Mpa	1116	1116	1116	1116	1116
fps	Aver. stress in pres. steel at the time for which the nominal resistanc		Mpa	1860	1860	1853	1853	1853
k	Factor depends on type of P.S, Low relaxation strand k = 0.28			0.28	0.28	0.28	0.28	0.28
a	Depth of equivalent stress block		m	0.000	0.000	0.016	0.016	0.016
de	Corresp. effective depth from extreme comp. fiber to centroid of tensile force in the tensile reinf.		m	0.442	0.442	1.441	1.441	1.441
Mn	Nominal resistance		kNm	2575	2575	16461	16461	16461
Mr	Factored reistance		kNm	**2318**	**2575**	**16461**	**14815**	**16461**
Mu	Flexual moment		kNm	535	297	5584	7433	5050

(5.7.3.2)	**Flexural moment Checking**		**OK**	**OK**	**OK**	**OK**	**OK**
	Limits for reinforcement						
c/de	Maximum reinforcement		0.00	0.00	0.01	0.01	0.01
	Maximum reinforcement Checking	<= 0.42	**OK**	**OK**	**OK**	**OK**	**OK**
1.2*Mcr	Craking moment	kNm	1157	1157	10545	10545	10545
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**		**OK**	**OK**	**OK**	**OK**	**OK**
(5.7.3.4)	**Conctrol of craking by distr. of reinf for RC member- Check?**		No	Yes	Yes	No	No
	Existing condition for structrure	1,2 or 3	1	1	1	1	1
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.058	0.058	0.059	0.059	0.059
Z	Crack width parameter	N/mm	30000	30000	30000	30000	30000
A	Area of concr. with same centroid as tens. Reinf	m2	0.019	0.019	0.021	0.021	0.021
fsa	Value	Mpa	292	292	281	281	281
0.6*fy		Mpa	240	240	240	240	240
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	240	240	240	240	240
x	Dist. From compression fiber to centroid	m	-	0.08	0.195	-	-
J.d	Arm	m	-	0.415	1.376	-	-
Icr	Moment of inertia of the cracked section	m4	-	0.018	0.353	-	-
fs	Tensile stress in reinforcement fs = Msls / (As*J.d)	Mpa	-	43	139	-	-
	Checking for control cracking fs<fsa		**N.a**	**OK**	**OK**	**N.a**	**N.a**
(5.10.8.2)	**Shrinkage and temperature Reinfocement (side distribution)**						
Areq	Area of required reinf	m2	0.00045	0.00045	0.00126	0.00126	0.00126
	Distribution on sides 7 D16	m2	0.00141	0.00141	0.00141	0.00141	
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45	0.45
	Checking		**OK**	**OK**	**OK**	**OK**	**OK**
	SHEAR AND TORSION CHECKING						
β	Factor indicating diag. cracked concr. to tension		2.8	3.6	2.6	2.4	2.9
θ	Angle of inclination of diagonal compressive	degree	28.86	28.20	28.95	30.76	28.80
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	90
bv	Effective web width as minimum web width - in dv	m	13.409	13.409	13.409	13.409	13.409
dv	Effective shear depth	m	0.442	0.442	1.433	1.433	1.433
	(de - a/2)	m	0.442	0.442	1.433	1.433	1.433
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	0.600
ncat	Amount of bars in spacing S	bars	21	21	20	20	20
Av	Shear reinf area in spacing S	m2	0.0032	0.0032	0.0030	0.0030	0.0030
θ	Assume	degree	28.87	28.19	28.88	30.16	30.27
v	Shear stress in concrete	kN/m2	127	69	99	151	120
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e_x	Strain in tensile reinforcement		4.32E-04	2.25E-04	4.75E-04	6.10E-04	3.99E-04
	if ex<0, multiple with reduce factor		-	-	-	-	-
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok	Ok
v/f'c	Ratio of shear stress and f'c		0.004	0.002	0.003	0.005	0.004
β	Final value		2.8	3.6	2.6	2.4	2.9
θ	Final value	degree	28.86	28.20	28.95	30.76	28.80
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	7481	9671	22770	21386	25378
Vs	Shear resistance provided by shear reinforcement	kN	1695	1743	5215	4847	5248
Vp	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0	0
Vn1	Vn1=Vc+Vs+Vp	kN	9177	11414	27985	26233	30626
Vn2	Vn2	kN	44450	44450	144108	144108	144108
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	9177	11414	27985	26233	30626
Vr	Factored shear resistance	kN	**8259**	**11414**	**27985**	**23609**	**30626**
Vu	Shear	kN	678	412	1903	2609	2296
(5.8.2.7)	**Shear checking**		**OK**	**OK**	**OK**	**OK**	**OK**
	Region requiring transverse reinf Checking		No need	No need	No need	No need	No need
	Minimum shear reinf area	m2	0.0091	0.0091	0.0091	0.0091	0.0091
	Mimimum shear reinforcement Checking		-	-	-	-	-
	0.1*f'c*bv*dv	kN	17780	17780	57643	57643	57643
	Smax	m	0.35	0.35	0.60	0.60	0.60
	Maximum spacing Smax		-	-	-	-	-

MATERIALS

	NORMAL CONCRETE		
f'c	Compressive Strength of concrete at 28 days	Mpa	30
Ec	Modulus of Elasticity	Mpa	27691
fr	Modulus of Rupture	Mpa	3.5
gc	Unit weight of concrete	kN/m3	24.5
	PRESTRESSING STEEL		
fpu	Tensile strength of prestressing steel	Mpa	1860
fpy	Yield strength of prestressing steel	Mpa	1670
Ep	Modulus of Elasticity	Mpa	195000
	REINFORCEMENT		
fy	Yield strength	Mpa	400
Es	Modulus of Elasticity	Mpa	200000
nc	Ratio Es/Ec		7

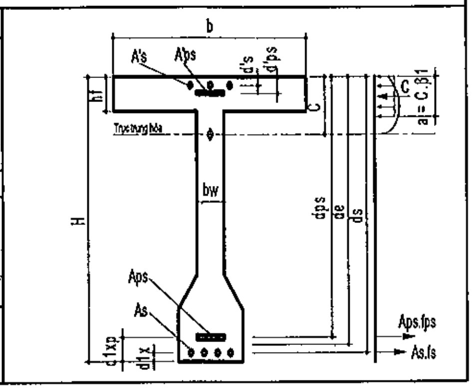


Sign	Parameters		Unit	Sections 3-3	3-3	3-3	4-4	4-4
	INTERNAL FORCES AT SECTIOIN							
	Combination			Service	Strength	Extreme	Extreme	Strength
Qu	Shear		kN	7127	9972	7264	1308	2901
Mu	Flexural Moment		kNm	3840	5304	4016	3735	1921
Nu	Axial load		kN	1575	2237	1508	1131	1678
Tu	Torsional Moment		kNm	0	0	0	0	0
	FLEXURAL MOMENT CHECKING							
H	Section height		m	2.000	2.000	2.000	2.000	2.000
d's	Dis. From comp. fiber to centroid of comp. Reinf		m	0.084	0.084	0.084	0.161	0.161
d1x	Dis. From tens. fiber to centroid of tension Reinf		m	0.161	0.161	0.161	0.084	0.084
	Cover to reinf		m	0.050	0.050	0.050	0.050	0.050
ds	Dis. From comp. fiber to centroid of tension Reinf		m	1.839	1.839	1.839	1.916	1.916
d'ps	Dis. From comp. fiber to centroid of comp. prestressing steel		m	0.000	0.000	0.000	0.000	0.000
d1xp	Dis. From tens. fiber to centroid of tension prestressing steel		m	0.000	0.000	0.000	0.000	0.000
dps	Dis. From comp. fiber to centroid of tension prestressing steel		m	2.000	2.000	2.000	2.000	2.000
b	Width of the compression face of member		m	13.409	13.409	13.409	13.409	13.409
bw	Web width or diameter of a circular section		m	13.409	13.409	13.409	13.409	13.409
hf	Compression flange depth		m	0.000	0.000	0.000	0.000	0.000
Iz	Moment of inertia of section		m4	8.939	8.939	8.939	8.939	8.939
Amc	Section area		m2	26.817	26.817	26.817	26.817	26.817
	Steel choice							
Aps	Tension prestressing steel	P.S type		0	0	0	0	0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
A'ps	Compression prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	0 T00.0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
As	Tension Reinforcement	Number	bars	84	84	84	84	84
		Diameter	mm	22	22	22	22	22
		Area	m2	0.03192	0.03192	0.03192	0.03192	0.03192
A's	Compression Reinforcement	Number	bars	0	0	0	0	0
		Diameter	mm	20	20	20	20	20
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
A'c	Shear reinforcement	Number	bars	21	21	21	21	21
		Diameter	mm	16	16	16	16	16
		Area	m2	0.00424	0.00424	0.00424	0.00424	0.00424
ϕ	Resistance factors for flexure		5.5.4.2	1.00	0.90	1.00	1.00	0.90
ϕv	Resistance factors for shear			1.00	0.90	1.00	1.00	0.90
ϕn	Resistance factors for axial force			1.00	1.00	1.00	1.00	1.00
β1	Stress block factor			0.836	0.836	0.836	0.836	0.836
c	Dis. Between centroid and top fiber		m	0.045	0.045	0.045	0.045	0.045
	For T section behavior		m	0.045	0.045	0.045	0.045	0.045
	For rectangular section behavior		m	0.045	0.045	0.045	0.045	0.045
fpe	Effective stress in the prestressing steel after losses		Mpa	1116	1116	1116	1116	1116
fps	Aver. stress in pres. steel at the time for which the nominal resistanc		Mpa	1848	1848	1848	1848	1848
k	Factor depends on type of P.S, Low relaxation strand k = 0.28			0.28	0.28	0.28	0.28	0.28
a	Depth of equivalent stress block		m	0.037	0.037	0.037	0.037	0.037
de	Corresp. effective depth from extreme comp. fiber							
	to centroid of tensile force in the tensile reinf.		m	1.839	1.839	1.839	1.916	1.916
Mn	Nominal resistance		kNm	23242	23242	23242	24225	24225
Mr	Factored reistance		kNm	**23242**	**20918**	**23242**	**24225**	**21803**
Mu	Flexual moment		kNm	3840	5304	4016	3735	1921

(5.7.3.2)	**Flexural moment Checking**		OK	OK	OK	OK	OK
	Limits for reinforcement						
c/de	Maximum reinforcement		0.02	0.02	0.02	0.02	0.02
	Maximum reinforcement Checking	<= 0.42	OK	OK	OK	OK	OK
1.2*Mcr	Craking moment	kNm	18930	18930	18930	18930	18930
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**		OK	OK	OK	OK	OK
(5.7.3.4)	**Conctrol of craking by distr. of reinf for RC member- Check?**		Yes	No	No	No	No
	Existing condition for structrure	1,2 or 3	3	3	3	3	3
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.061	0.061	0.061	0.061	0.061
Z	Crack width parameter	N/mm	17500	17500	17500	17500	17500
A	Area of concr. with same centroid as tens. Reinf	m2	0.019	0.019	0.019	0.019	0.019
fsa	Value	Mpa	165	165	165	165	165
0.6*fy		Mpa	240	240	240	240	240
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	165	165	165	165	165
x	Dist. From compression fiber to centroid	m	0.231	-	-	-	-
J.d	Arm	m	1.762	-	-	-	-
Icr	Moment of inertia of the cracked section	m4	0.633	-	-	-	-
fs	Tensile stress in reinforcement fs = Msls / (As*J.d)	Mpa	68	-	-	-	-
	Checking for control cracking fs<fsa		OK	N.a	N.a	N.a	N.a
(5.10.8.2)	**Shrinkage and temperature Reinfocement (side distribution)**						
Areq	Area of required reinf	m2	0.00127	0.00127	0.00127	0.00127	0.00127
	Distribution on sides 7 D16	m2	0.00141	0.00141	0.00141	0.00141	0.00141
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45	0.45
	Checking		OK	OK	OK	OK	OK
	SHEAR AND TORSION CHECKING						
β	Factor indicating diag. cracked concr. to tension		2.3	2.1	2.3	2.9	3.1
θ	Angle of inclination of diagonal compressive	degree	34.99	37.66	34.96	28.78	28.71
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	90
bv	Effective web width as minimum web width - in dv	m	13.409	13.409	13.409	13.409	13.409
dv	Effective shear depth	m	1.820	1.820	1.820	1.897	1.897
	(de - a/2)	m	1.820	1.820	1.820	1.897	1.897
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	0.600
ncat	Amount of bars in spacing S	bars	21	21	21	21	21
Av	Shear reinf area in spacing S	m2	0.0042	0.0042	0.0042	0.0042	0.0042
θ	Assume	degree	38.22	41.43	39.67	30.88	34.58
v	Shear stress in concrete	kN/m2	292	454	298	33	127
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e_x	Strain in tensile reinforcement		9.16E-04	1.17E-03	9.13E-04	3.91E-04	3.57E-04
	if ex<0, multiple with reduce factor		-	-	-	-	-
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok	Ok
v/f'c	Ratio of shear stress and f'c		0.010	0.015	0.010	0.001	0.004
β	Final value		2.3	2.1	2.3	2.9	3.1
θ	Final value	degree	34.99	37.66	34.96	28.78	28.71
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	25267	23712	25282	33964	35525
Vs	Shear resistance provided by shear reinforcement	kN	7354	6670	7363	9767	9795
Vp	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0	0
Vn1	Vn1=Vc+Vs+Vp	kN	32622	30382	32645	43731	45320
Vn2	Vn2	kN	183061	183061	183061	190805	190805
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	32622	30382	32645	43731	45320
Vr	Factored shear resistance	kN	**32622**	**27343**	**32645**	**43731**	40788
Vu	Shear	kN	7127	9972	7264	1308	2901
(5.8.2.7)	**Shear checking**		OK	OK	OK	OK	OK

REINFORCEMENT CHECKING - WING WALL

	MATERIALS		
	NORMAL CONCRETE		
fc	Compressive Strength of concrete at 28 days	Mpa	30
Ec	Modulus of Elasticity	Mpa	27691
fr	Modulus of Rupture	Mpa	3.5
gc	Unit weight of concrete	kN/m3	24.5
	PRESTRESSING STEEL		
fpu	Tensile strength of prestressing steel	Mpa	1860
fpy	Yield strength of prestressing steel	Mpa	1670
Ep	Modulus of Elasticity	Mpa	195000
	REINFORCEMENT		
fy	Yield strength	Mpa	400
Es	Modulus of Elasticity	Mpa	200000
nc	Ratio Es/Ec		7



Sign	Parameters		Unit	Sections 5-5	5-5	6-6	6-6	6-6
	INTERNAL FORCES AT SECTIOIN							
	Combination			Service	Strength	Service	Strength	Strength
Qu	Shear		kN	74	116	203	315	315
Mu	Flexural Moment		kNm	172	273	179	278	278
Nu	Axial load		kN	0	0	0	0	0
Tu	Torsional Moment		kNm	0	0	0	0	0
	FLEXURAL MOMENT CHECKING							
H	Section height		m	0.500	0.500	0.500	0.500	0.500
d's	Dis. From comp. fiber to centroid of comp. Reinf		m	0.058	0.058	0.058	0.058	0.058
d1x	Dis. From tens. fiber to centroid of tension Reinf		m	0.059	0.059	0.059	0.059	0.059
	Cover to reinf		m	0.050	0.050	0.050	0.050	0.050
ds	Dis. From comp. fiber to centroid of tension Reinf		m	0.441	0.441	0.441	0.441	0.441
d'ps	Dis. From comp. fiber to centroid of comp. prestressing steel		m	0.000	0.000	0.000	0.000	0.000
d1xp	Dis. From tens. fiber to centroid of tension prestressing steel		m	0.000	0.000	0.000	0.000	0.000
dps	Dis. From comp. fiber to centroid of tension prestressing steel		m	0.500	0.500	0.500	0.500	0.500
b	Width of the compression face of member		m	1.000	1.000	1.000	1.000	1.000
bw	Web width or diameter of a circular section		m	1.000	1.000	1.000	1.000	1.000
hf	Compression flange depth		m	0.000	0.000	0.000	0.000	0.000
Iz	Moment of inertia of section		m4	0.010	0.010	0.010	0.010	0.010
Amc	Section area		m2	0.500	0.500	0.500	0.500	0.500
	Steel choice							
Aps	Tension prestressing steel	P.S type		0	0	0	0	0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
A'ps	Compression prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	0 T00.0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
As	Tension Reinforcement	Number	bars	7	7	7	7	7
		Diameter	mm	20	20	22	22	22
		Area	m2	0.00220	0.00220	0.00266	0.00266	0.00266
A's	Compression Reinforcement	Number	bars	7	7	7	7	7
		Diameter	mm	16	16	16	16	16
		Area	m2	0.00141	0.00141	0.00141	0.00141	0.00141
A'c	Shear reinforcement	Number	bars	3	3	3	3	3
		Diameter	mm	12	12	14	14	14
		Area	m2	0.00034	0.00034	0.00045	0.00045	0.00045
φ	Resistance factors for flexure		5.5.4.2	1.00	0.90	1.00	0.90	0.90
φv	Resistance factors for shear			1.00	0.90	1.00	0.90	0.90
φn	Resistance factors for axial force			1.00	1.00	1.00	1.00	1.00
β1	Stress block factor			0.836	0.836	0.836	0.836	0.836
c	Dis. Between centroid and top fiber		m	0.015	0.015	0.023	0.023	0.023
	For T section behavior		m	0.015	0.015	0.023	0.023	0.023
	For rectangular section behavior		m	0.015	0.015	0.023	0.023	0.023
fpe	Effective stress in the prestressing steel after losses		Mpa	1116	1116	1116	1116	1116
fps	Aver. stress in pres. steel at the time for which the nominal resistance		Mpa	1844	1844	1835	1835	1835
k	Factor depends on type of P.S, Low relaxation strand k = 0.28			0.28	0.28	0.28	0.28	0.28

a	Depth of equivalent stress block	m	0.012	0.012	0.020	0.020	0.020
de	Corresp. effective depth from extreme comp. fiber						
	to centroid of tensile force in the tensile reinf.	m	0.441	0.441	0.441	0.441	0.441
Mn	Nominal resistance	kNm	353	353	432	432	432
Mr	Factored reistance	kNm	**353**	**318**	**432**	**388**	**388**
Mu	Flexual moment	kNm	172	273	179	278	278
(5.7.3.2)	**Flexural moment Checking**		**OK**	**OK**	**OK**	**OK**	**OK**
	Limits for reinforcement						
c/de	Maximum reinforcement		0.03	0.03	0.05	0.05	0.05
	Maximum reinforcement Checking	<= 0.42	**OK**	**OK**	**OK**	**OK**	**OK**
r min	Mimimum reinforcement		0.44%	0.44%	0.53%	0.53%	0.53%
	Mimimum reinforcement Checking for RC	0.23%	**OK**	**OK**	**OK**	**OK**	**OK**
1.2*Mcr	Craking moment	kNm	89	89	90	90	90
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**		**OK**	**OK**	**OK**	**OK**	**OK**
(5.7.3.4)	**Conctrol of craking by distr. of reinf for RC member- Check?**		Yes	No	Yes	No	No
	Existing condition for structrure	1,2 or 3	1	1	1	1	1
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.059	0.059	0.059	0.059	0.059
Z	Crack width parameter	N/mm	30000	30000	30000	30000	30000
A	Area of concr. with same centroid as tens. Reinf	m2	0.017	0.017	0.017	0.017	0.017
fsa	Value	Mpa	301	301	301	301	301
0.6*fy		Mpa	240	240	240	240	240
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	240	240	240	240	240
x	Dist. From compression fiber to centroid	m	0.102	-	0.111	-	-
J.d	Arm	m	0.407	-	0.404	-	-
Icr	Moment of inertia of the cracked section	m4	0.002	-	0.003	-	-
fs	Tensile stress in reinforcement fs = Msls / (As*J.d)	Mpa	192	-	167	-	-
	Checking for control cracking fs<fsa		**OK**	**N.a**	**OK**	**N.a**	**N.a**
(5.10.8.2)	**Shrinkage and temperature Reinfocement (side distribution)**						
Areq	Area of required reinf	m2	0.00031	0.00031	0.00031	0.00031	0.00031
	Distribution on sides 7 D16	m2	0.00141	0.00141	0.00141	0.00141	0.00141
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45	0.45
	Checking		**OK**	**OK**	**OK**	**OK**	**OK**
	SHEAR AND TORSION CHECKING						
β	Factor indicating diag. cracked concr. to tension		2.2	1.9	2.2	1.9	1.9
θ	Angle of inclination of diagonal compressive	degree	36.15	41.31	36.22	41.14	41.14
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	90
bv	Effective web width as minimum web width - in dv	m	1.000	1.000	1.000	1.000	1.000
dv	Effective shear depth	m	0.435	0.435	0.431	0.431	0.431
	(de - a/2)	m	0.435	0.435	0.431	0.431	0.431
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	0.600
ncat	Amount of bars in spacing S	bars	3	3	3	3	3
Av	Shear reinf area in spacing S	m2	0.0003	0.0003	0.0005	0.0005	0.0005
β	Assume		2.0	2.0	2.0	2.0	2.0
θ	Assume	degree	36.15	41.31	38.30	42.39	42.39
v	Shear stress in concrete	kN/m2	170	296	471	812	812
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e_x	Strain in tensile reinforcement		1.01E-03	1.58E-03	1.02E-03	1.54E-03	1.54E-03
	if ex<0, multiple with reduce factor		-	-	-	-	-
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok	Ok
v/f'c	Ratio of shear stress and f'c		0.006	0.010	0.016	0.027	0.027
β	Final value		2.2	1.9	2.2	1.9	1.9
θ	Final value	degree	36.15	41.31	36.22	41.14	41.14
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	439	378	435	379	379
Vs	Shear resistance provided by shear reinforcement	kN	135	112	178	149	149
Vp	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0	0
Vn1	Vn1=Vc+Vs+Vp	kN	574	490	613	528	528
Vn2	Vn2	kN	3261	3261	3234	3234	3234
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	574	490	613	528	528
Vr	Factored shear resistance	kN	**574**	441	**613**	**475**	475
Vu	Shear	kN	74	116	203	315	315
(5.8.2.7)	**Shear checking**		**OK**	**OK**	**OK**	**OK**	**OK**

SPACE PILE FOUNDATION ANALYSIS PROGRM
Turbo BASIÇ

PROJECT: : Cau OP11a

INITIA DATA

Kn = 0.00 Ax = 6.50 By = 12.60 Cz = 2.00
E v.uon = 2944008 E r.uon = 2944008 E v.nen = 2944008 E r.nen = 2944008
Mq = 0 (t/m4) Md = 0 (t/m4) m = 20000 (t/m4)

LOAD COMBINATIONS

COMB.	Hx	Hy	P	Mx	My	Mz
1	403.00	0.00	1861.00	-336.00	654.00	0.00
2	281.00	0.00	1336.00	-320.00	545.00	0.00
3	384.00	5.00	1795.00	-301.00	559.00	0.00
4	262.00	5.00	1270.00	-284.00	450.00	0.00
5	283.00	4.00	1353.00	-230.00	469.00	0.00
6	270.00	19.00	1655.00	-218.00	-58.00	0.00
7	270.00	19.00	1129.00	-201.00	207.00	0.00

PROPERTIES OF PILES

PILE	Lo	H	Bpx	Bpy	A	B	Cday	Fo	Io	Po	Co	Ct
1	0.00	8.00	1.384	1.384	1.00	1.000	0.000	1.000	0.098	0	150000	75000
2					n	t						
3					n	t						
4					n	t						
5					n	t						
6					n	t						
7					n	t						

PILE COORD.

PILE	X	Y	Phi	Xi
1	-4.32	5.30	0.000	0.00
2	-2.51	0.32	0.000	0.00
3	-0.70	-4.66	0.000	0.00
4	4.09	-4.66	0.000	0.00
5	2.88	-1.34	0.000	0.00
6	1.67	1.98	0.000	0.00
7	0.47	5.30	0.000	0.00

DISPLACEMENTS

COMB.	X	Y	Z	Fix	Fiy	Fiz
1	0.00120	0.00003	0.002649	-0.000029	0.000152	-0.000014
2	0.00087	0.00004	0.001892	-0.000036	0.000131	-0.000010
3	0.00112	0.00004	0.002562	-0.000020	0.000128	-0.000013
4	0.00079	0.00004	0.001805	-0.000026	0.000106	-0.000009
5	0.00084	0.00003	0.001928	-0.000019	0.000105	-0.000010
6	0.00062	-0.00001	0.002417	0.000042	-0.000042	-0.000009
7	0.00075	0.00006	0.001618	-0.000008	0.000058	-0.000009

FORCES ON PILES

PILE	COMB.	N	Q2	Q3	M1	M2	M3
1	1	211.81	-53.60	3.63	-0.429	6.404	60.008
	2	149.78	-37.37	2.53	-0.299	5.377	40.395
	3	208.87	-51.11	2.71	-0.405	4.670	58.260
	4	146.81	-34.88	1.61	-0.275	3.635	38.652
	5	155.01	-37.67	1.95	-0.299	3.613	42.259
	6	235.04	-36.04	-0.40	-0.273	-2.985	48.942
	7	139.00	-36.04	-0.40	-0.273	-0.064	42.983
2	1	224.72	-57.57	2.18	-0.429	4.542	65.125
	2	155.59	-40.14	1.52	-0.299	4.079	43.963
	3	221.95	-54.86	1.35	-0.405	2.912	63.091
	4	152.84	-37.43	0.69	-0.275	2.441	41.934
	5	164.78	-40.43	0.95	-0.299	2.319	45.816
	6	247.87	-38.57	-1.32	-0.273	-4.170	52.200
	7	145.60	-38.57	-1.32	-0.273	-1.250	46.242
3	1	237.64	-61.54	0.74	-0.429	2.679	70.242
	2	161.41	-42.91	0.51	-0.299	2.780	47.531
	3	235.04	-58.61	-0.02	-0.405	1.153	67.922
	4	158.87	-39.98	-0.24	-0.275	1.246	45.216
	5	174.55	-43.19	-0.06	-0.299	1.023	49.374
	6	260.71	-41.10	-2.25	-0.273	-5.356	55.458
	7	152.22	-41.10	-2.25	-0.273	-2.436	49.500
4	1	309.63	-61.54	-3.08	-0.429	-2.242	70.242
	2	223.13	-42.91	-2.15	-0.299	-0.652	47.531
	3	295.38	-58.61	-3.63	-0.405	-3.493	67.922
	4	208.91	-39.98	-2.69	-0.275	-1.910	45.216
	5	224.44	-43.19	-2.72	-0.299	-2.398	49.374
	6	240.68	-41.10	-4.68	-0.273	-8.490	55.458
	7	179.66	-41.10	-4.68	-0.273	-5.569	49.500
5	1	301.01	-58.90	-2.12	-0.429	-1.000	66.831
	2	219.24	-41.07	-1.48	-0.299	0.215	45.152
	3	286.65	-56.11	-2.71	-0.405	-2.320	64.701
	4	204.88	-38.28	-2.07	-0.275	-1.113	43.028
	5	217.92	-41.35	-2.04	-0.299	-1.534	47.002
	6	232.13	-39.41	-4.06	-0.273	-7.699	53.286
	7	175.25	-39.41	-4.06	-0.273	-4.778	47.328
6	1	292.41	-56.25	-1.16	-0.429	0.242	63.419
	2	215.37	-39.22	-0.81	-0.299	1.080	42.774

	3	277.93	-53.61	-1.80	-0.405	-1.148	61.480
	4	200.86	-36.58	-1.46	-0.275	-0.317	40.840
	5	211.41	-39.51	-1.37	-0.299	-0.671	44.630
	6	223.57	-37.73	-3.45	-0.273	-6.908	51.114
	7	170.84	-37.73	-3.45	-0.273	-3.988	45.155
7	1	283.79	-53.60	-0.19	-0.429	1.484	60.008
	2	211.49	-37.37	-0.13	-0.299	1.946	40.395
	3	269.20	-51.11	-0.89	-0.405	0.025	58.260
	4	196.83	-34.88	-0.84	-0.275	0.480	38.652
	5	204.89	-37.67	-0.70	-0.299	0.193	42.259
	6	215.01	-36.04	-2.84	-0.273	-6.117	48.942
	7	166.43	-36.04	-2.84	-0.273	-3.197	42.983

SUMMARY OF FORCES

	PILE	COMB.	N	Q2	Q3	M1	M2	M3
Nmin	1	7	139.00	-36.04	-0.40	-0.273	-0.064	42.983
Nmax	4	1	309.63	-61.54	-3.08	-0.429	-2.242	70.242
Q2max	3	1	237.64	-61.54	0.74	-0.429	2.679	70.242
Q3max	4	6	240.68	-41.10	-4.68	-0.273	-8.490	55.458
M1max	1	1	211.81	-53.60	3.63	-0.429	6.404	60.008
M2max	4	6	240.68	-41.10	-4.68	-0.273	-8.490	55.458
M3max	3	1	237.64	-61.54	0.74	-0.429	2.679	70.242

CHECKING CALCULATI
IN COMPARISON WITH INITIA LOAD MATRIX

1	403.00	0.00	1861.00	-336.00	654.00	0.00
2	281.00	0.00	1336.00	-320.00	545.00	0.00
3	384.00	5.00	1795.00	-301.00	559.00	0.00
4	262.00	5.00	1270.00	-284.00	450.00	0.00
5	283.00	4.00	1353.00	-230.00	469.00	0.00
6	270.00	19.00	1655.00	-218.00	-58.00	0.00
7	270.00	19.00	1129.00	-201.00	207.00	0.00

DANANG QUANG NGAI EXPRESSWAY OP11A BRIDGE	Item.	Eng.	Date.	Sign.
	Design			
DETAIL DESIGN CHECK REINFORCEMENT OF BORED PILE	Check			
	Revise			

BORED PILE DESIGN

I. BORED PILE DATA

1.Load Combinations at top of bored pile

No	Combinations	Sign	F_V (kN)	Longitudinal F_{HX} (kN)	Longitudinal My (kN•m)	Transvesal F_{HY} (kN)	Transvesal Mx (kN•m)
1	Strength Str-IA		0	0	0	0	0
2	Strength Str-IA		3495	629	-1074	15	38
3	Strength Str-IA		0	0	0	0	0
4	Strength Str-IA		89	542	-890	-15	-26
5	Strength Str-IA		1220	629	-1074	-15	-26
6							

2. Bored pile Material

Normal concrete			
Compressive strength at 28 days age	f'c	30	MPa
Concrete elastic modulus	Ec	27691	MPa
Reinforcement			
Yield strength	fy	420	MPa
Reinforcement elastic modulus	Es	200,000	MPa

3. Bored pile Section

Pile diameter	D	1.00	m
Section area	A	0.785	m2
Moment inertia	Ix	0.049	m4
	Iy	0.049	m4
Radius of gyration of gross concrete section; r = sqrt(I/A)	rx	0.250	m
	ry	0.250	m

II. PILE DESIGN

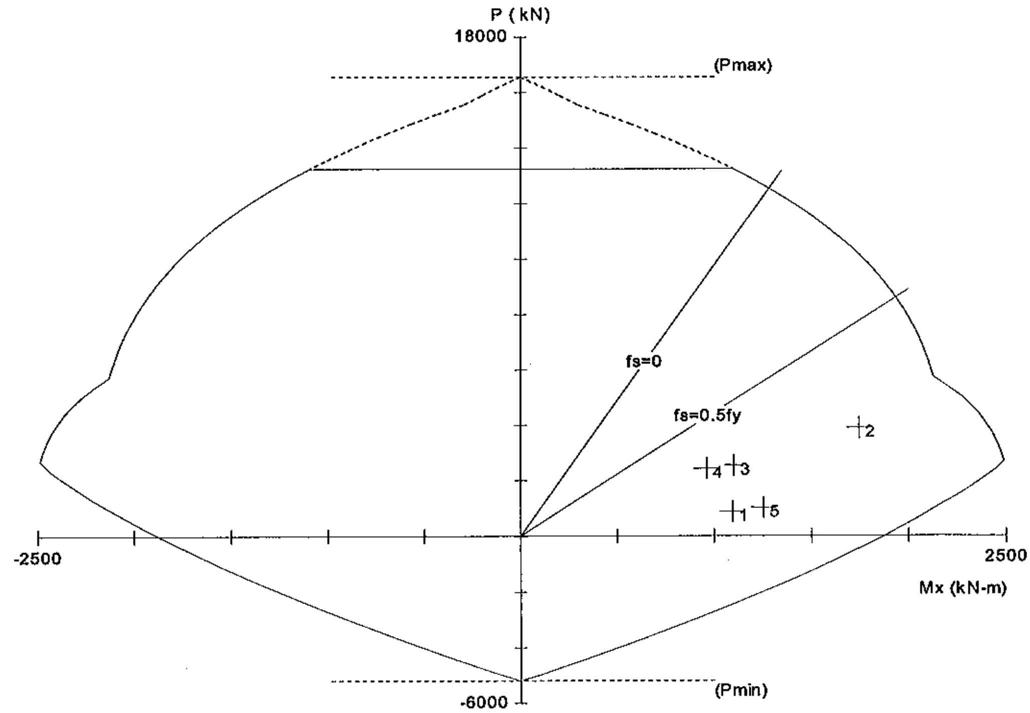
1. Limit of Reinforcement

S.5.7.4.2

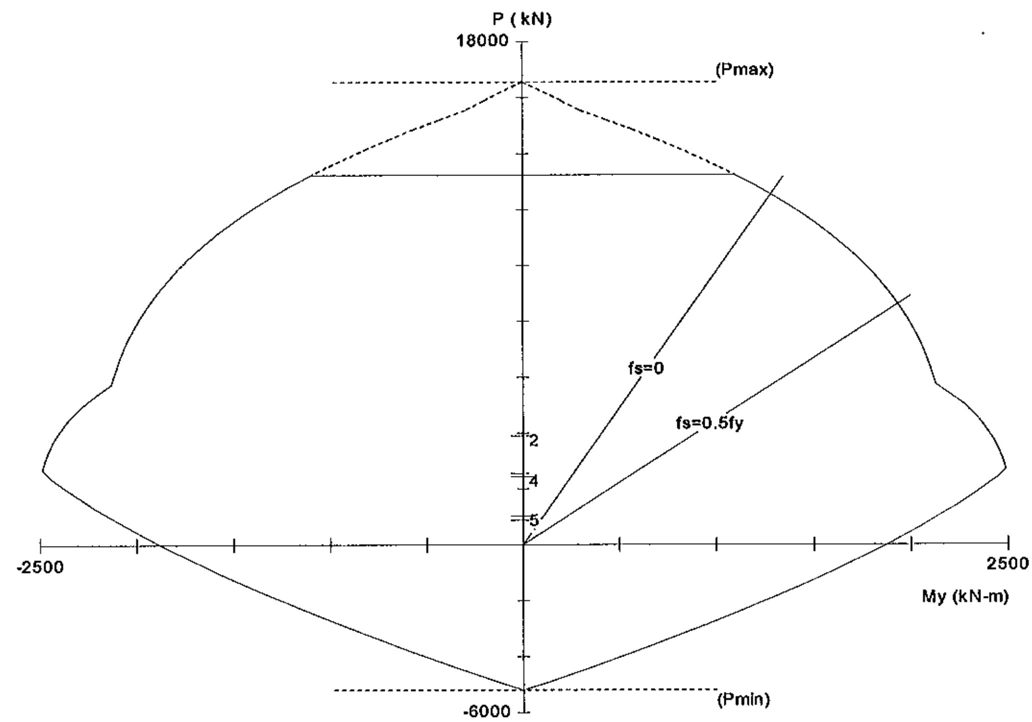
Minimum area of longitudinal reinforcement in column			
As.fy / (Ag . f'c) >= 0.135	As ≥	0.008	m2
As / Ag >= 0.01	As ≥	0.008	m2
Maximum area of longitudinal reinforcement in column			
As / Ag <= 0.08	As ≤	0.063	m2
Trial Rebars: Ok	As	**0.015**	m2
1layers x 24 = 24 bars D28 @150	As1	0.015	m2

2. Interation diagram M-P

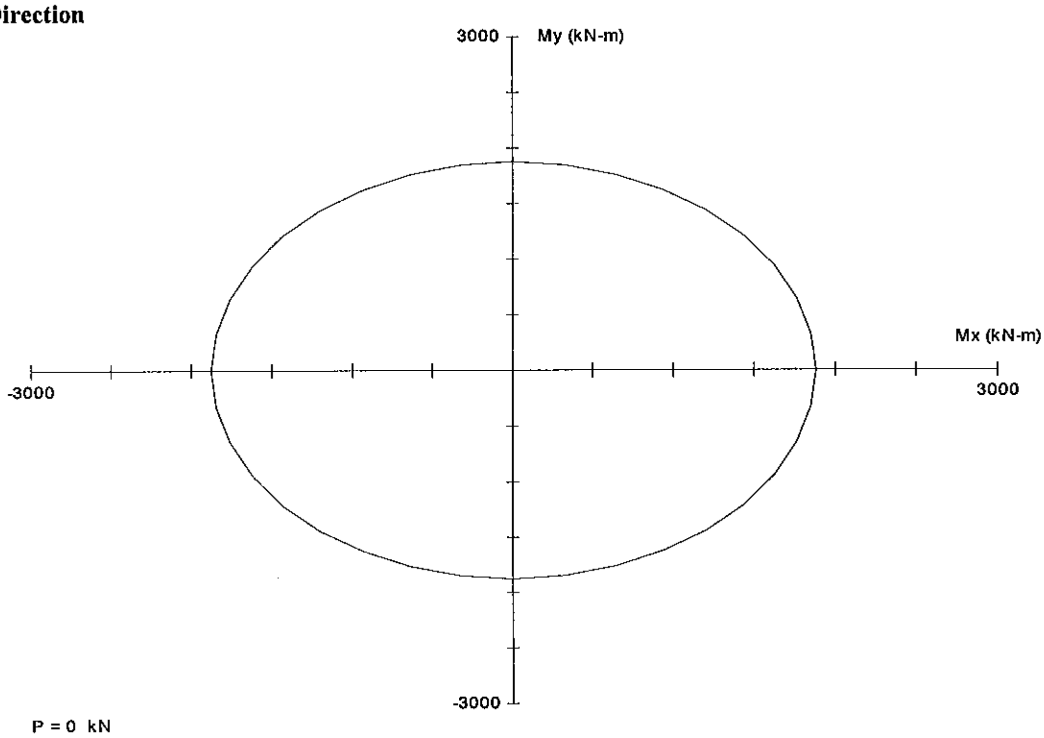
****In Transverse Direction**



****In Longitudinal Direction**



****In Both Direction**



3. Column Ties S.5.7.4.6, S.5.10.6.3, S5.10.11.4.1d - e

Bridge is in seismic zone	Sz	1	
Area of concrete core measured out-to-out of ties	Ac	0.622	m2
Tie diameter	Dtie	14	mm2
Cross section area of 1 tie	As-tr	0.0002	m2
Spacing of hoops	s	75	mm
Length of reinfocement tie in 1 hoop	Ltie	2.78	m
Ratio of ties reinf. in one hoop/ volume of conc. core one pitch spacing			
ρs = As-tr . Ltie / (Ac * spacing)	ρs	0.0096	
Ratio of spiral reinf. To total volume of concrete core shall satisfy			S.5.7.4.6
ρs >= 0.45 . (Ag/Ac - 1). f'c/ fy = Req1	Req1	0.0084	**OK**
Transverse Reinforcement for Confinement at Plastic Hinges		S.5.10.11.4.1.d	
For a circular column "1:applied","2:Not applied"		1	
ρs >= 0.12 . f'c/ fy = Req2	Req2	0.0086	**N/A**
Length distributed spiral with pitch 75mm below pilecap	Ldis	1.50	m

4. Shear Design

Shear resistance factors	ϕv	1.0	
Factored shear force	Vu	629	kN
Required shear capacity Vn = Vu/ ϕv	Vn	629	kN
Determine concrete shear capacity			
Minimum shear reinforcement will provided in cross section			
Therefore	β	2.0	
	θ	45.0	deg
Diameter of bored pile	D	1.00	m
Width of cross section	b	1.00	m
dv = 0.9*de de = D/2 + Dr/pi()			
Diameter of the circle passing through the centers of the long. reinf.	Dr	0.79	m
	de	0.75	m
	dv	0.68	m

Vc = 0.083 . β . sqrt(f'c) . bv. dv	Vc	616	kN
	A_v	1963	mm2
Angle of inclination of shear reinf. to long. axis	α	90	
Vs=Av*fy*dv*(cotgθ+cotgα)*sinα/s	Vs	7447	KN
Vn1 = Vc + Vs	V_{n1}	8063	
Vn2 = 0.25f'cbvdv	V_{n2}	5081	
	V_n	5081	
	Conclude		**OK**

AASHTO - LRFD 3rd 2004 & 4th 2007; 22TCN-272-05

ASSUMPTION:

- The rock is reasonably sound
- The rock strength measured during site investigation will not deteriorate during construction when water or drilling fluids are used
- The bottom of the socket is properly cleaned out.

DATA & CALCULATION:

Bored hole name		OP11a-A1		
Bottom of pilecap elavation	EL1 =	9.00		
Top of socket elevation	EL2 =	4.38		
Pile tip elevation	EL3 =	1.00		
Pile Length	L =	8.00	m	
Diameter of drilled-shaft	D_p =	1.00	m	
Pile Cross-Sectional Perimeter	P =	3.14	m	
Pile Cross-Sectional Area	A_b =	0.79	m^2	
Working normal force at pile head	N =	3191.6	kN	
Working normal force at top of socket	P_i =	3152.5	kN	
Intack rock modulus	E_i =	25000	MPa	*Figure C10.8.3.5-2 Lrfd*
Modulus modification ratio	K_e =	0.05		*Figure C10.8.3.5-3 Lrfd*
Elastic modulus of the insitu rock	$E_r = K_e{*}E_i$ =	1250.0	MPa	
Influence coefficient	$I_p = f(H_s/D_s, E_c/E_r)$ =	0.30		*Figure C10.8.3.5-1 Lrfd*
	H_s/D_s =	3.38		
	E_c/E_r =	22.15		

Pile Concrete comp. strength	f'_c =	30.0	MPa
Concrete Unit Weight	g_c =	24.5	kN/m^3
Modulus of elasticity of concre	E_c =	27691	MPa
Depth of socket	H_s =	3.38	m
Diameter of socket	D_s =	1.00	m
Socket Cross-Sect. Perimeter	P_{soc} =	3.14	m
Socket Cross-Sectional Area	A_{soc} =	0.79	m^2

Rock mass modulus/ intack rock modulus	E_m / E_i			*C.10.4.6.5-1-Lrfd 4th*
Atmospheric pressure	p_a =	0.101	MPa	
Reduction factor to account for jointing	α_E			*10.8.3.5.4b-Lrfd 4th*

The elastic shortening of the drilled shaft

$r_c = (\Sigma P_i){*}H_s/(A_{soc}{*}E_c) =$ 0.490 mm

The settlement of base of drilled shaft

$r_{base} = (\Sigma P_i){*}I_p/(D_s{*}E_r) =$ 0.757 mm

$r_c + r_{base} =$ **1.247** mm < 10 mm

Compute the bearing capacity based on Sharft resistance alone

Unit side resistance

Case 1: The drilling fluid used will not form a lubricated film on the sides of the socket

if qu < 1.9 Mpa - may be taken after Carter & Kulhawy 1988 → qs = 0.15*qu — C10.8.3.5-4

if qu > 1.9 Mpa - may be taken after Horvath & Kenney 1979 → qs = 0.21*sqrt(qu) — C10.8.3.5-5

Case 2: Side of rock socket is considered to be smooth or where the rock is drilled using a drilling slurry

May be taken after Horvath & Kenney 1979 → qs = 0.65*α_E*pa*(qu/pa)^0.5<7.8*pa*(f'c/pa)^0.5

qs0 = 7.8*pa*(f'c/pa)^0.5 — *10.8.3.5.4d-1-Lrfd2007*

Drilling method used in construction: *Case 2*

$Q_R = \phi{*}Q_n = \phi_s{*}Q_{SR}$

ϕ_s is the resistance factor - table 10.5.5-3 LRFD

q_u is the uniaxial compressive strength of the rock

Case1

No.	EL_T	EL_B	Depth (m)	RQD (%)	q_u (MPa)	q_s (MPa)	Q_{SR} (kN)	ϕ_s	Q_R (kN)
1	4.38	3.38	1.00	75	42.72	1.37	4312	0.65	2803
2	3.38	2.38	1.00	75	42.72	1.37	4312	0.65	2803
3	2.38	1.00	1.38	95	42.72	1.37	5951	0.65	3868
4						-	-	-	-
5									
6									
7									
8									
Sum			**3.38**				**14575**		**9474**

	DANANG QUANG NGAI EXPRESSWAY OP11a BRIDGE DETAIL DESIGN EMPIRICAL ESTIMATION OF PILE CAPACITY-ABUTMENT A1	Item.	Eng.	Date.	Sign.
		Design			
		Check			
		Revise			

Case2 — *Type: "1: closed joints", "2: open joints"*

No.	Depth (m)	RQD (%)	q_u (MPa)	E_m / E_i	α_E	Type	qs0 (MPa)	q_s (MPa)	q_s - *used* (MPa)	Q_{SR} (kN)	ϕ_s	Q_R (kN)
1	1.00	75.00	42.72	0.75	0.90	1	13.58	1.22	*1.22*	3818	0.55	2100
2	1.00	75.00	42.72	0.75	0.90	1	13.58	1.22	*1.22*	3818	0.55	2100
3	1.38	95.00	42.72	0.95	0.98	1	13.58	1.32	*1.32*	5736	0.55	3155
4	-	-	-	-	-	-	-	-	-	-	-	-
5	-	-	-	-	-	-	-	-	-	-	-	-
6	-	-	-	-	-	-	-	-	-	-	-	-
7	-	-	-	-	-	-	-	-	-	-	-	-
8	-	-	-	-	-	-	-	-	-	-	-	-
Sum	**3.38**									**13372**		**7354**

Unit base resistance $q_p = Kb.(p1\text{-}p0) + \sigma v$

Limit pressure determined from presuremeter tests	p_l =	5.89	MPa	*C10.8.3.5-7*
At rest total horizontal stress measured at ther base elevation	p_0 =	-	MPa	
Coefficient that depen on diameter socket	K_b =	4.33		*Table C10.8.3.5-1*
Total vertical stress at the base elevation	σv =	-	MPa	
	q_p =	-	MPa	
	ϕ =	0.50		*Table 10.5.5-3*
$Q_{pR} = Ap . qp$	Q_{pR} =	-	kN	
$Q_R = \phi . Q_{pR}$	Q_R =	-	kN	

ESTIMATED PILE CAPACITY:

Pile Structural Capacity $Q_T = 0.75*0.85*0.85*f'c*Ag$	*12768*	kN	*1301*	T
Pile resistance Q_R	*7354*	kN	*750*	T
Deducting pile weight	*-118*	kN	*-12*	T
Estimated Pile Capacity	*7236*	kN	*738*	T
Maximum Reaction - ULS **Ok**	*3037*	kN	*310*	T

ASSUMPTION:

- The rock is reasonably sound
- The rock strength measured during site investigation will not deteriorate during construction when water or drilling fluids are used
- The bottom of the socket is properly cleaned out.

DATA & CALCULATION:

Bored hole name		**OP11a-A2**	
Bottom of pilecap elavation	EL1 =	9.00	
Top of socket elevation	EL2 =	3.19	
Pile tip elevation	EL3 =	0.00	
Pile Length	L =	9.00	m
Diameter of drilled-shaft	D_p =	1.00	m
Pile Cross-Sectional Perimeter	P =	3.14	m
Pile Cross-Sectional Area	A_b =	0.79	m^2
Working normal force at pile head	N =	3210.8	kN
Working normal force at top of socket	P_i =	3174.0	kN
Intack rock modulus	E_i =	25000	MPa
Modulus modification ratio	K_e =	0.05	
Elastic modulus of the insitu rock	$E_r = K_e*E_i$ =	1250.0	MPa
Influence coefficient	$I_p = f(H_s/D_s, E_c/E_r)$ =	0.30	
	H_s/D_s =	3.19	
	E_c/E_r =	22.15	

Pile Concrete comp. strength	f'_c =	30.0	MPa
Concrete Unit Weight	g_c =	24.5	kN/m^3
Modulus of elasticity of concre	E_c =	27691	MPa
Depth of socket	H_s =	3.19	m
Diameter of socket	D_s =	1.00	m
Socket Cross-Sect. Perimeter	P_{soc} =	3.14	m
Socket Cross-Sectional Area	A_{soc} =	0.79	m^2

Figure C10.8.3.5-2 Lrfd (Intack rock modulus)
Figure C10.8.3.5-3 Lrfd (Modulus modification ratio)
Figure C10.8.3.5-1 Lrfd (Influence coefficient)

Rock mass modulus/ intack rock modulus	E_m / E_i			*C.10.4.6.5-1-Lrfd 4th*
Atmospheric pressure	p_a =	0.101	MPa	
Reduction factor to account for jointing	α_E			*10.8.3.5.4b-Lrfd 4th*

The elastic shorterning of the drilled shaft

$r_e = (\Sigma P_i)*H_s/(A_{soc}*E_c) =$ 0.466 mm

The settlement of base of drilled shaft

$r_{base} = (\Sigma P_i)*I_p/(D_s*E_r) =$ 0.762 mm

$r_e + r_{base} =$ **1.227** mm < 10 mm

Compute the bearing capacity based on Sharft resistance alone
Unit side resistance

Case 1: The drilling fluid used will not form a lubricated film on the sides of the socket

if qu < 1.9 Mpa - may be taken after Carter & Kulhawy 1988 → qs = 0.15*qu *C10.8.3.5-4*

if qu > 1.9 Mpa - may be taken after Horvath & Kenney 1979 → qs = 0.21*sqrt(qu) *C10.8.3.5-5*

Case 2: Side of rock socket is considered to be smooth or where the rock is drilled using a drilling slurry

May be taken after Horvath & Kenney 1979 → qs = 0.65*α_E*pa*(qu/pa)^0.5<7.8*pa*(f'c/pa)^0.5

qs0 = 7.8*pa*(f'c/pa)^0.5 *10.8.3.5.4d-1-Lrfd2007*

Drilling method used in construction: $\boxed{\textit{Case 2}}$

$Q_R = \phi*Q_n = \phi_s*Q_{SR}$

ϕ_s is the resistance factor - table 10.5.5-3 LRFD

q_u is the uniaxial compressive strength of the rock

Case1

No.	EL_T	EL_B	Depth (m)	RQD (%)	q_u (MPa)	q_s (MPa)	Q_{SR} (kN)	ϕ_s	Q_R (kN)
1	3.19	2.19	1.00	75	42.72	1.37	4312	0.65	2803
2	2.19	1.19	1.00	75	42.72	1.37	4312	0.65	2803
3	1.19	-	1.19	95	42.72	1.37	5131	0.65	3335
4						-	-	-	-
5									
6									
7									
8									
Sum			**3.19**				**13755**		**8941**

Case2 *Type: "1: closed joints"; "2: open joints"*

No.	Depth (m)	RQD (%)	q_u (MPa)	E_m/E_i	α_E	Type	qs0 (MPa)	q_s (MPa)	*q_s - used* (MPa)	Q_{SR} (kN)	ϕ_s	Q_R (kN)
1	1.00	75.00	42.72	0.75	0.90	1	13.58	1.22	*1.22*	3818	0.55	2100
2	1.00	75.00	42.72	0.75	0.90	1	13.58	1.22	*1.22*	3818	0.55	2100
3	1.19	95.00	42.72	0.95	0.98	1	13.58	1.32	*1.32*	4947	0.55	2721
4	-	-	-	-	-	-	-	-	-	-	-	-
5	-	-	-	-	-	-	-	-	-	-	-	-
6	-	-	-	-	-	-	-	-	-	-	-	-
7	-	-	-	-	-	-	-	-	-	-	-	-
8	-	-	-	-	-	-	-	-	-	-	-	-
Sum	**3.19**									**12582**		**6920**

Unit base resistance	$q_p = Kb.(p1\text{-}p0) + \sigma v$				
Limit pressure determined from presuremeter tests		$p_l =$	5.89	MPa	*C10.8.3.5-7*
At rest total horizontal stress measured at ther base elevation		$p_0 =$	-	MPa	
Coefficient that depen on diameter socket		$K_b =$	4.27		*Table C10.8.3.5-1*
Total vertical stress at the base elevation		$\sigma v =$	-	MPa	
		$q_p =$	-	MPa	
		$\phi =$	0.50		*Table 10.5.5-3*
	$Q_{pR} = Ap . qp$	$Q_{pR} =$	-	kN	
	$Q_R = \phi.Q_{pR}$	$Q_R =$	-	kN	

ESTIMATED PILE CAPACITY:

Pile Structural Capacity	$Q_T = 0.75{*}0.85{*}0.85{*}fc{*}Ag$	*12768*	kN	*1301*	T
Pile resistance	Q_R	*6920*	kN	*705*	T
Deducting pile weight		*-129*	kN	*-13*	T
Estimated Pile Capacity		*6791*	kN	*692*	T
Maximum Reaction - ULS	**Ok**	*3037*	kN	*310*	T

8. LRB09

Table of content - LRB09 Bridge

A. Substructure design

1. Abutment A1
2. Abutment A2
3. Pier P1 (P2)
4. Bored pile capacity

B. Miscellaneous

1. Expansion joint

Da Nang Quang Ngai Expressway project

BRIDGE

LRB09

CALCULATION SHEETS

ABUTMENT A1

2013

Table of content

1.Structure dimensions and Loads
2. Foundation analysis
3. Elements checks
4.Pile Design

LOAD COMPONENTS

Assumptions :

1. Bridge is considered to be in seismic with acceleration coeff.
2. The Design of the Abutment accords with Specification for bridge design 22-TCN-272-05 and AASHTO LRFD 2004 for reference
3. Design live load: HL-93 and lane loading 9.3 KN/m

Input :

Level Table(at center of abutment)

Level of top of headwall	HTwL	16.608	m
Level of top of bearing	BTL	14.567	m
Level of top of stem abutment	HTL	14.333	m
Level of top of footing	FTL	11.500	m
Level of bottom of footing	FBL	9.500	m
Ground level	GL	12.000	m
Lowest water level	HWL	0.000	m
Skew angle	α	20.00	deg

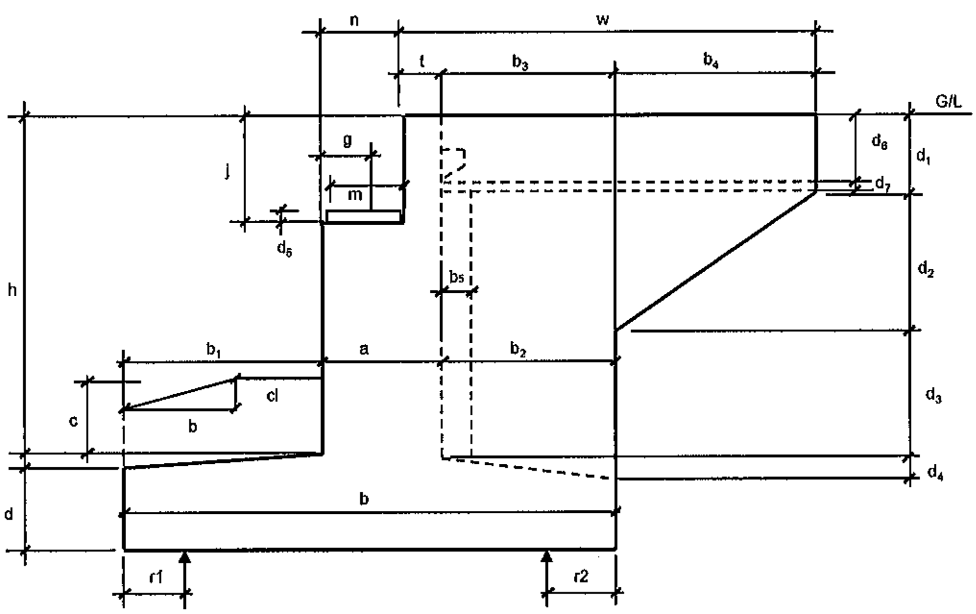
Material Unit Weights

- Unit Weight of Reinf. concrete γ_c = 24.5 kN/m^3
- Unit Weight of Soil γ_s = 18.0 kN/m^3
- Unit Bouyancy Weight of Soil γ_{sbo} = 8.2 kN/m^3
- Unit weight of asphalt concrete γ_a = 22.1 kN/m^3

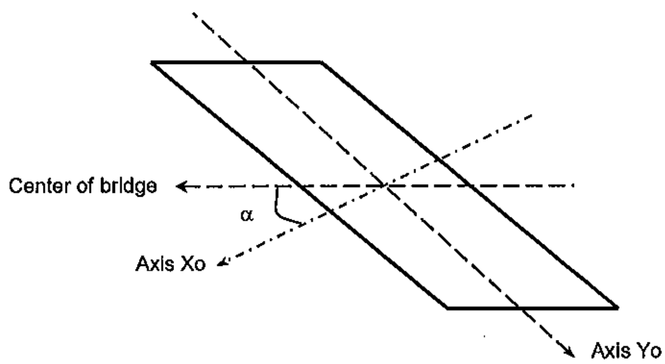
I.Loads from substructure

Abutment dimensions VERTICAL VIEW

Bearing Type:	MOVE

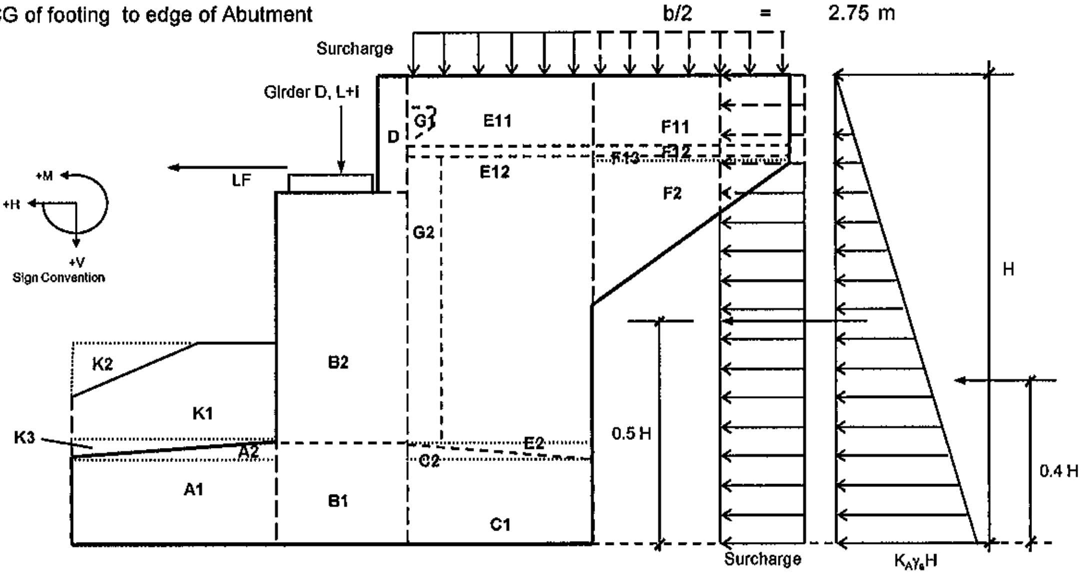


PLAN VIEW



ABUTMENT DIMENSIONS (IN METRES)					
Item	Symbol	Value	Item	Symbol	Value
Height of stem	h	5.108	Horizontal Dimension	b_4	**1.127**
Footing Width	b	**5.500**	Horizontal Dimension	b_5	**0.300**
Stem Width	a	**1.500**	Vertical Dimension	d_1	**0.930**
Footing Depth	d	**2.000**	Vertical Dimension	d_2	**1.127**
Footing Slope	f	**0.000**	Vertical Dimension	d_3	**3.051**
Width of stem at bearing	n	**1.000**	Vertical Dimension	d_4	
Ballast Wall Height	j	**2.275**	Vertical Dimension	d_5	**0.234**
Ballast Wall Thickness	t	**0.500**	Vertical Dimension	d_6	**1.070**
Wingwall Length	w	**4.000**	Vertical Dimension	d_7	
Soil Cover at Toe	c	0.500	With of bearing pad	m	**0.800**
Girder Reaction	g	**0.550**	Wingwall Thickness	u1	**0.500**
Horizontal Dimension	b_1	**1.800**	Wingwall Thickness	u2	**0.500**
Horizontal Dimension	b_2	**2.200**	Distance to cl of pile	r1	**1.000**
Horizontal Dimension	b_3	**2.341**	Distance to cl of pile	r2	**1.000**

	cos (α)	=	0.94
Slope front of abutment	cl	=	0.00 m
	bl	=	0.00 m
Width of Abutment	L		12.600 m
Width of abutment (inclined direction)	Ltr		13.409 m
Height of Abutment	Ht	=	7.11 m
Distance from CG of footing to edge of Abutment	b/2	=	2.75 m



1. Self weight of Abutment (DC)

Description	Area (m^2)	Length (m)	Force (kN)	X[(1)] (m)	Arm[(2)] (m)	Moment (kN•m)
SW of Abutment (DC)						
Section A1	3.600	13.409	1183	0.900	1.850	2188
Section A2	-	13.409	-	1.200	1.550	-
Section B1	3.000	13.409	986	2.550	0.200	197
Section B2	4.250	13.409	1396	2.550	0.200	279
Section C1	4.400	13.409	1445	4.400	-1.650	-2385
Section C2	-	13.409	-	4.033	-1.283	-
Section D	1.138	13.409	374	3.050	-0.300	-112
Section E11	2.177	0.500	27	4.471	-1.721	-46
Section E12	9.781	0.500	120	4.471	-1.721	-206
Part extra stem	-	-	-	5.111	-2.361	-
Section F11	1.206	0.500	15	6.205	-3.455	-51
Section F12	-	0.500	-	5.034	-2.284	-
Section F13	-0.158	0.500	-2	6.205	-3.455	7
Section F2	0.635	0.500	8	6.017	-3.267	-25
Section G1	0.135	12.409	41	3.450	-0.700	-29
Section G2	0.045	5.108	6	3.450	-0.700	-4
Bearing seats (w1seat= 0.65m)	0.187	3.250	15	2.350	0.400	6
Curbs +Handrail on Abutment	0.50	4.000	49	4.800	-2.050	-100
Total SW of Abutment (DC)			***5661***			***-282***
Tranverser moment			*21*		*6.175*	*127*

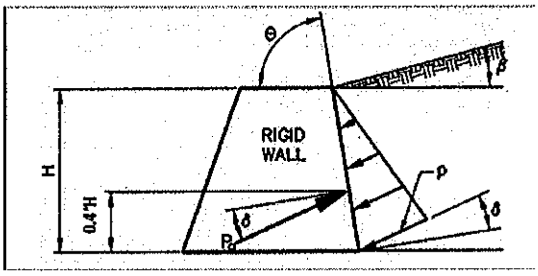
Notes:
1. Distance 'X' is measured horizontally from Toe of Retaining to CG of Section
2. Moment 'Arm' is measured from CG horizontally and from Underside of Footing Vertically.

2. Earth on Abutment (EV)

Description	Area (m^2)	Length (m)	Force (kN)	X[(1)] (m)	Arm[(2)] (m)	Moment (kN•m)
Earth on Abutment (EV)						
Section E1	11.24	12.909	2611	4.400	-1.650	-4308
Section E2	-	12.409	-	4.767	-2.017	-
Section E3	-0.43	1.000	-8	5.571	-2.821	22
Section K1	0.900	13.409	217	0.900	1.850	-
Section K2	-	13.409	-	-	2.750	-
Section K3	-	13.409	-	0.600	2.150	-
Total Earth on Footing			*2821*			*-4286*

3. Horizontal Earth Pressure on Abutment (EH)

To be safe, horizontal earth pressure at front face of abutment may be neglected.
Horizontal earth pressure at behind face of abutment shall be considered.



$$Ka = \frac{\sin^2(\theta + \phi'_f)}{\Gamma.[\sin^2\theta.\sin(\theta - \delta)]}$$

$$\Gamma = \left[1 + \sqrt{\frac{\sin(\phi'_f+\delta).\sin(\phi'_f-\beta)}{\sin(\theta-\delta).\sin(\theta+\beta)}}\right]^{-2}$$

• Height for horizontal earth pressure	H	=	7.11 m
• Width for horizontal earth pressure	W	=	13.4 m
• Density of Soil	γ_s	=	1835 kg/m^3
• Internal Friction Angle of Soil	ϕ'_f	=	30.0 deg
• Incline angle of back face wall	θ	=	90.0 deg
• Friction angle between fill and wall	δ	=	0.0 deg
• Incline angle of fill soil	β	=	0.0 deg
• Gravitational acceleration	g	=	9.81 m/s^2

• Basic earth pressure

p = K.γs. g. Z .10^-9 (Mpa, Z:mm)

K: taken as Ka (assume wall move or deflect sufficiently to reach minimum active conditions)

Γ	=	2.250
Ka	=	0.333
p	=	0.043 Mpa

Horizontal earth pressure:

• Ea = 0,5. p. Z . B . 10^3 (kN)	Ea	=	2032 kN
• M= Ea . 0,4H	M	=	5778 kNm

4. Earth Pressure on Abutment due to Surcharge (ES)

Equivalent height of soil for highway loading taken from Table 3.11.6.2.1

H=	1.50m	heq=	1.7 m	
H=	3.00m	heq=	1.2 m	
H=	6.00m	heq=	0.76 m	
H=	9.00m	heq=	0.61 m	
H=	7.11m	heq=	0.70 m	(Linear Interpolation)

• Vertical force

ESv	=	374 kN
ev	=	-1.65 m
M	=	-617 kNm

• Horizontal force

$$\Delta p = k.\gamma_s.g.h_{eq} x 10^9$$

ESh	=	403 kN
eh	=	3.55 m
M	=	1432 kNm

5. Earthquake effects

Bridge is located at:
According to TCXDVN 375:2006 and 22TCN272-05, bridge is in seismic zone 2 and acceleration coefficient as below
- Peak ground acceleration coefficient A = 0.0310 g

5.1. Seismic active lateral Earth pressure (E_{AE})

• Backfill slop angle	i	=	0.0 deg
• Slope of wall to vertical	β'	=	0.0 deg
• Angle of friction of soil	φ	=	30.0 deg
• Angle of friction between soil and abutment	δ	=	0.0 deg
• Horizontal acceleration coefficient	kh	=	0.047
• Vertical acceleration coefficient	kv	=	0.019
• Angle $\theta = \arctan(k_h/(1-k_v))$	θ	=	2.7 deg

$$K_{AE} = \frac{\cos^2(\phi-\theta-\beta)}{\cos\theta.\cos^2\beta\cos(\delta+\beta+\theta)} x \left[1+\sqrt{\frac{\sin(\phi+\delta).\sin(\phi-\theta-i)}{\cos(\delta+\beta+\theta).\cos(i-\beta)}}\right]^{-2}$$

- Seismic active lateral Earth pressure coefficient K_{AE} = 0.36

- $E_{AE} = 0,5.\ g.\ \gamma.\ H^2.\ (1\text{-}kv).\ K_{AE}.\ 10^{\wedge}\text{-}9$ (kN/m)

- Seismic active lateral Earth pressure coefficient E_{AE} = 2165 kN

$M_{AE}= E_{AS}*0.3H+(E_{AE} - E_{AS})*0.6H$ M_{AE} = 4901 KNm <A.11.1.1.1>

E_{AS} is the static component of seismic active pressure calculated with θ = kv = 0

5.2. Earthquake effects to abutment (EQ)

Seismic force for substructures: elements above ground Fh = Csm* W; elements under ground Fh = A*S*W

• Soil profile type	Soil type		I
• Site Coefficients.	S	=	1.0
• Elastic Seismic Response Coefficient	2.5A	=	0.078
Csm = 1.2*A*S/ Tm^2/3 ≤ 2.5*A	Csm	=	0.057
• Period of vibration of the fundamental mode			
Tm = 2*pi()*sqrt(m/k)	Tm	=	0.522 s

Decription	Area (m^2)	Length (m)	Force (kN)	X(1) (m)	Arm(2) (m)	Moment (kN•m)
Section A1	3.600	13.409	68	-	1.000	68
Section A2	-	13.409	-	-	2.000	-
Section B1	3.000	13.409	57	-	1.000	57
Section B2	4.250	13.409	80	-	3.417	274
Section C1	4.400	13.409	83	-	1.000	83
Section C2	-	13.409	-	-	2.000	-
Section D	1.138	13.409	21	-	5.971	128
Section E11	2.177	0.500	2	-	4.573	7
Section E12	9.781	0.500	7	-	2.019	
Section E2	-	-	-	-	2.000	-
Section F11	1.206	0.500	1	-	4.573	4
Section F12	-	0.500	-	-	4.038	
Section F13	-0.158	0.500	-0	-	5.178	
Section F2	0.635	0.500	0	-	5.802	3
Section G1	0.135	12.409	2	-	4.395	10
Section G2	0.045	5.108	0	-	2.019	1
Total EQ of Abutment Selfweight			***321***			***633***

6. Braking Force(BR)

Take 50 *% Braking Force for this Abutment (Free Bearing)*

• Number of lanes	n	=	3 lanes	
• Multiple presence factor	m	=	0.85	
• Take 25 % of Truck load				
BR = 25% * n * m * (2*145+35)	BR	=	104 kN	Long. Axis
• Acting at 1.8m higher of road face	e	=	8.9 m	
	Mlong	=	923 KNm	Long. Axis

7. Centrifucal Force , CE (3.6.3)

• Plan of bridge (1:"straight",2: "Curve")			1	
• Design Speed	V	=	120 km/h	
	V	=	33.3 m/s	
	R		- m	
$C = 4/3*(V^2/gR)$	C		-	
Acting at 1.8m higher of road face				
CE = n * m * (2*145+35) * C	CE	=	0.00 KN	
	e	=	6.91 m	
	Mtrans	=	0.00 KNm	Trans. Axis

8. Water Load (WA) ***:NA***

8.1. Buoyancy of Abutment

• Heightest water Level +0.00

Description	Area (m^2)	Length (m)	Force (kN)	$X^{(1)}$ (m)	$Arm^{(2)}$ (m)	Moment (kN•m)
Bouyancy on abutment						
Section A1	-	13.409	-	0.900	1.850	-
Section A2	-	13.409	-	-	2.750	-
Section B(B1,B2)	-	13.409	-	2.550	0.200	-
Section C1	-	13.409	-	4.400	-1.650	-
Section C2	-	13.409	-	-	2.750	-
Section E2	-	1.000	-	-	2.750	-
Section E1	-	1.000	-	4.471	-1.721	-
Section F2	-	1.000	-	0.791	1.959	-
Total Bouyancy			-			-

8.2 Buoyancy of Earth on Abutment

Description	Area (m^2)	Length (m)	Force (kN)	$X^{(1)}$ (m)	$Arm^{(2)}$ (m)	Moment (kN•m)
Bouyancy of earth on abutement						
Section E2	-	12.409	-	-	2.750	-
Section E1	-	12.409	-	4.400	-1.650	-
Section K2	-	13.409	-	-	2.750	-
Section K1	-	13.409	-	0.900	1.850	-
- Section K3	-	13.409	-	-	2.750	-
Total Bouyancy			-			-

SUPERSTRUCTURE LOADS

II.Loads from superstructure

Item	Sign	Value	Unit
Span length	Lst	33.00	m
Span between bearings	Ls	32.10	m
Bridge Width	W	12.60	m
Number of girders	n_g	5.00	Girders
Girder height	Hg	1.65	m
Deck slab depth	Hd	0.247	m
Asphalt depth	Hασ	0.084	m

1.Dead loads (DC): One span at abutment

Item	Sign	Value	Unit
1.1.Girders			
Sum of girders weight	DC	3347.93	kN
Precast Planks	DC	473.46	kN
Diaphragm	DC	405.16	kN
Total	DC	4226.55	kN
1.2.Deck slab			
Deck slab	DC	2464.29	kN
1.3.Pavement			
Asphalt concrete	DW	649.37	kN
1.4.Parapet			
Parapet + median	DC	889.35	kN

2.Live load (LL):

2.1. Live load

Truck	145 — 4.3 m — 145 — 4.3 m — 35 kN		
Tandem	110 — 1.2 m — 110 kN		
Lane load	w_l 9.3 kN/m		
Pedestrial	Wpd= 0.0 kN/m2		
Considerate structure as a simple span			
Reaction Influence	1; 32.10 m		
Number of lanes	n	3	
Multiple presence factor	m	0.85	
Dynamic load allowance	1+IM	1.25	

*Reaction = [(1+IM)*Vehicle + Laneload]*n*m*

Truck	P1(kN)	P2(kN)	P3(kN)	Sum(kN)	Total(kN)
Axle load	145	145	35		
Influence value	1.000	0.866	0.732		
Reaction	145.0	125.6	25.6	296.2	944.1
Tandem	P1(kN)	P2(kN)		Sum(kN)	Total(kN)
Axle load	110	110			
Influence value	1.000	0.963			
Reaction	110	105.9		215.9	688.1
Lane load	Wl(kN/m)				Total(kN)
Value	9.3				
Influence value	16.05				
Reaction	149.3				380.6
Pedestrian	Wdb(kN)				Total(kN)
Reaction	0.0				0.0

3.Earthquake effects on superstructrue (EQ)

Force from superstructure due to EQ	EQ	=	236 kN

4.Uniform Temperature, Srinkage& Creep(TU+SH&CR)

Bearing displacement due to uniform temperature and shrinkage creep	Δu	=	0.026 m
Shear modulus G	G	=	1 MPa
Bearing area	A	=	0.175 m2
Height of elastomeric layers	hrt		0.084 m
Number of bearing	nb	=	5 bears
Horizontal force due to TU+SH&CR			
Acting at top of bearing $H = G.A.\Delta u/h_{rt}$	Hx	=	271 kN

5.Wind loads (Ws)

5.1.Tranverse wind on superstructure (WS)

Wind zone	Zone		III
Basic 3 second gust wind	Vb	=	53.00 m/s
Correction factor	S	=	1.09
Design wind velocity	V	=	57.77 m/s
Drag coefficient	Cd	=	1.40
Overall width of bridge	b	=	12.60 m
Depth of superstructure(including solid parapet)	d	=	2.97 m
	b/d	=	4.25
Windy obstructed area of superstructure	At	=	97.91 m2
Transverse wind load			
$P_D = \max(0.0006V^2.C_d.A_t, 1.8A_t)$ =	Hy	=	274.5 kN
Longitudinal wind load			
$F_{WSL} = 0.25P_D$ =	Hx	=	68.6 kN

5.2.Wind load on vehicles (WL)

Transverse wind load on vehicle	Hy	=	24.75 kN
Longitudinal wind load on vehicles	Hx	=	24.75 kN
(At 1.8m from surface)			

6.Combinations

Loads from superstructure to Abutment

Loads at bottom of stem		Vertical		Longitudinal			Tranversal		
Loads	Sign	N (kN)	x (m)	Hx (kN)	z_1 (m)	My (kN•m)	Hy (kN)	y (m)	Mx (kN•m)
Girder+Deck+Parapet	DC	3790	0.20			758			
Pavement	DW	325	0.20			65			
LiveLoad	LL	1325	0.20			265		0.48	629
Pedestrian	PD							-	-
Trans. wind on Struc.	WS			34	3.07		137	3.07	421
Trans. wind on vehi.	WL			12	9.18		25	9.18	227
Eearth quake	EQ			118	3.07		71	3.07	217
TU+SH&CR	TU+SH&CR			135	3.07	415			

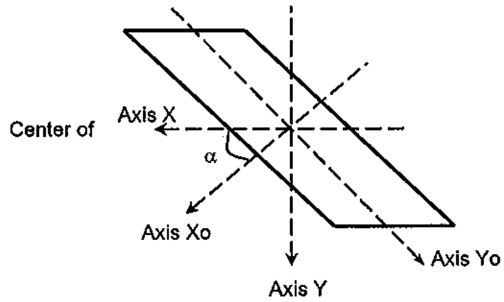
Loads at bottom of pilecap		Vertical		Longitudinal			Tranversal		
Loads	Sign	N (kN)	x (m)	Hx (kN)	z_1 (m)	My (kN•m)	Hy (kN)	y (m)	Mx (kN•m)
Girder+Deck+Parapet	DC	3790	0.40			1516			
Pavement	DW	325	0.40			130			
LiveLoad	LL	1325	0.40			530		0.48	629
Pedestrian	PD			-	-	-		-	-
Trans. wind on Struc.	WS			34	5.07	174	137	5.07	695
Trans. wind on vehi.	WL			12	11.18	138	25	11.18	277
Eearth quake	EQ			118	5.07	598	71	5.07	359
TU+SH&CR	TU+SH&CR			135	5.07	686			

Loads		Load combinations						
	Sign	Str-IA	Str-IB	Str-IIIA	Str-IIIB	Ser-I	Ext-IA	Ext-IB
Girder+Deck+Parapet	DC	1.25	0.90	1.25	0.90	1.00	1.25	0.90
Pavement	DW	1.50	0.65	1.50	0.65	1.00	1.50	0.65
LiveLoad	LL	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Pedestrian	PD	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Trans. wind on Struc.	WS			0.40	0.40	0.30		
Trans. wind on vehi.	WL			1.00	1.00	1.00		
Eearth quake	EQ						1.00	1.00
TU+SH&CR	TU+SH&CR	0.50	0.50	0.50	0.50	1.00		

Load combinations at bottom of stem		Longitudinal		Tranversal	
Load combinations	N (kN)	Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	7543	68	1716	0	1101
Strength Str-IB	5940	68	1396	0	1101
Strength Str-IIIA	7013	94	1610	80	1245
Strength Str-IIIB	5411	94	1290	80	1245
Service Ser-I	5440	158	1503	66	983
Extreme Ext-IA	5887	118	1177	71	532
Extreme Ext-IB	4285	118	857	71	532

Load combinations at bottom of pilecap		Longitudinal		Tranversal	
Load combinations	N (kN)	Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	7543	68	3360	0	1101
Strength Str-IB	5940	68	2719	0	1101
Strength Str-IIIA	7013	94	3356	80	1404
Strength Str-IIIB	5411	94	2715	80	1404
Service Ser-I	5440	158	3053	66	1115
Extreme Ext-IA	5887	118	2953	71	673
Extreme Ext-IB	4285	118	2312	71	673

LOAD COMBINATIONS



III.Load Combinations

1.Loads from substructure

Loads at bottom of pilecap	Sign	Vertical N (kN)	Longitudinal Hx (kN)	Longitudinal My (kN•m)	Tranversal Hy (kN)	Tranversal Mx (kN•m)
Self weight of Abutment	DC	5661		-282		127.2781
Soils on pilecap	EV	2821		-4286		
Horizontal Earth Pressure	EH		1910	5430		
Vertical Surcharge	LSv	374		-617		
Horizontal Surcharge	LSh		379	1346		
Braking Force	BR		104	923		
Centrifucal Force	CE		-	-	-	-
Buoyancy of Abutment	WA	-		-		
Buoyancy of Earth on Abutment	WA	-		-		
Earthquake effects to Abutment	EQ		321	633	96	190
Earthquake effects to soil	E_{AE}		2035	4605		

Loads	Sign	Load factors Str-IA	Str-IB	Str-IIIA	Str-IIIB	Ser-I	Ext-IA	Ext-IB
Self weight of Abutment	DC	1.25	0.90	1.25	0.90	1.00	1.25	0.90
Soils on pilecap	EV	1.35	0.90	1.35	0.90	1.00	1.35	0.90
Horizontal Earth Pressure	EH	1.50	0.90	1.50	0.90	1.00	0.00	0.00
Vertical Surcharge	LSv	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Horizontal Surcharge	LSh	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Braking Force	BR	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Centrifucal Force	CE	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Buoyancy of Abutment	WA	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Buoyancy of Earth on Abutment	WA	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Earthquake effects to Abutment	EQ						1.00	1.00
Earthquake effects to soil	E_{AE}						1.00	1.00

Load Combination at bottom of pilecap					
Load combinations	N (kN)	Longitudinal Hx (kN)	Longitudinal My (kN.m)	Tranversal Hy (kN)	Tranversal Mx (kN.m)
Strength Str-IA	11539	3709	4895	0	159
Strength Str-IB	8288	2563	3665	0	115
Strength Str-IIIA	11389	3516	4235	0	159
Strength Str-IIIB	8139	2370	3004	0	115
Service Ser-I	8856	2392	2513	0	127
Extreme Ext-IA	11071	2597	-75	96	349
Extreme Ext-IB	7820	2597	1952	96	305

2. Loads from superstructure

Load Combination at bottom of pilecap					
		Longitudinal		Tranversal	
Load combinations	N (kN)	Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	7543	68	3360	0	1101
Strength Str-IB	5940	68	2719	0	1101
Strength Str-IIIA	7013	94	3356	80	1404
Strength Str-IIIB	5411	94	2715	80	1404
Service Ser-I	5440	158	3053	66	1115
Extreme Ext-IA	5887	118	2953	71	673
Extreme Ext-IB	4285	118	2312	71	673

3. Total loads at bottom of pilecap

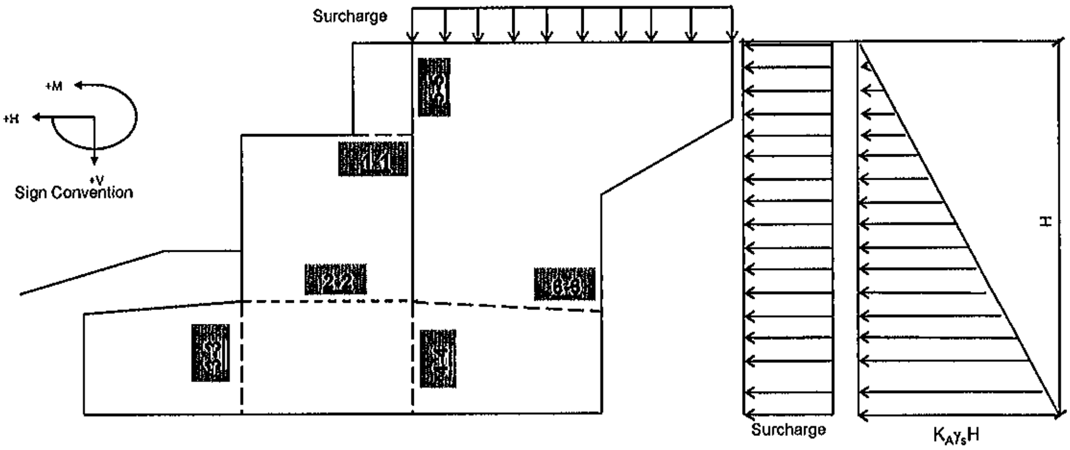
Load Combination at bottom of pilecap					
		Longitudinal		Tranversal	
Load combinations	N (kN)	Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	19082	3776	8256	0	1260
Strength Str-IB	14229	2630	6384	0	1216
Strength Str-IIIA	18402	3609	7591	80	1564
Strength Str-IIIB	13549	2464	5720	80	1519
Service Ser-I	14295	2550	5565	66	1242
Extreme Ext-IA	16958	2715	2878	167	1023
Extreme Ext-IB	12105	2715	4264	167	978

ELEMENTS CHECKING

IV.Elements checking

The abutment walls shall be checked at sections 1-1, 2-2, 3-3, 4-4, 5-5

1. Calculate internal force of sections



1.1 Section 1-1

Table of Load Factors

Loads	Sign	Service I	Strength I		Extreme I
		Ser-I	Str-IA	Str-IB	Ext-I
Selfweight	DC	1.00	1.25	0.90	1.25
Horizontal Earth Pressure	EH	1.00	1.50	0.90	
Surchage (Horizontal)	LS_h	1.00	1.75	1.75	0.50
Horizontal Seismic Earth Pressure	E_{AE}				1.50
Abutment earthquake force	EQ				1.00

Unfactored Loads	N	Longitudinal		Tranversal	
		Hx	My	Hy	Mx
	(kN)	(kN)	(kN.m)	(kN)	(kN.m)
Selfweight	415		-16		
Horizontal Earth Pressure		222	202		
Surcharge (horizontal)		281	319		
Horizontal Seismic Earth Pressure		236	171		
Abutment earthquake force		24	27	7	8

Load Combination at bottom of headwall					
Load combinations	N	Longitudinal		Tranversal	
		Hx	My	Hy	Mx
	(kN)	(kN)	(kN.m)	(kN)	(kN.m)
Service Ser-I	415	502	505	0	0
Strength Str-IA	518	824	841	0	0
Strength Str-IB	373	691	726	0	0
Extreme Ext-I	518	518	423	7	8

1.2 Section 2-2

Table of Load Factors

Loads	Sign	Service I	Strength I		Extreme I
		Ser-I	Str-IA	Str-IB	Ext-I
Selfweight	DC	1.00	1.25	0.90	1.25
Superstructure Dead Load	DC	1.00	1.25	0.90	1.25
Pavement	DW	1.00	1.50	0.65	1.50
Live Load	LL	1.00	1.75	1.75	0.50
Horizontal Earth Pressure	EH	1.00	1.50	0.90	
Surchage (Horizontal)	LS_h	1.00	1.75	1.75	0.50
TU+SH&CR	TU+SH&CR	1.00	0.50	0.50	
Horizontal Seismic Earth Pressure	E_{AE}				1.50
Abutment earthquake force	EQ				1.00

Unfactored Loads	N	Longitudinal		Tranversal	
		Hx	My	Hy	Mx
	(kN)	(kN)	(kN.m)	(kN)	(kN.m)
Selfweight	1811		-191		
Superstructure Dead Load	3790		758		
Pavement	325		65		
Live Load	1325		265		629
Horizontal Earth Pressure		1050	2144		
Surchage (Horizontal)		311	794		
TU+SH&CR		135	415		
Horizontal Seismic Earth Pressure		1190	2039		
Abutment earthquake force		104	209	102	280

Load Combination at bottom of stem wall

Load combinations	N	Longitudinal		Tranversal	
		Hx	My	Hy	Mx
	(kN)	(kN)	(kN.m)	(kN)	(kN.m)
Service Ser-I	7250	1496	4250	0	629
Strength Str-IA	9806	2186	6083	0	1101
Strength Str-IB	7570	1556	4543	0	1101
Extreme Ext-I	8150	2044	4602	102	594

1.3 Section 3-3

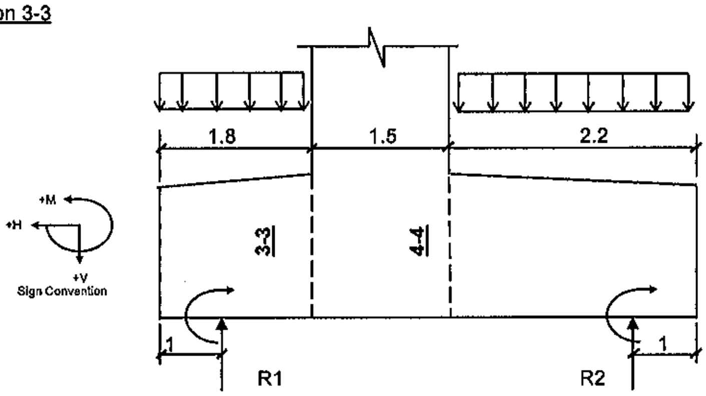


Table of Load Factors

Loads	Sign	Service I	Strength I		Extreme I
		Ser-I	Str-IA	Str-IB	Ext-I
Selfweight at front side	DC	1.00	1.25	0.90	1.25
Vertical soil on foot at front side	DC	1.00	1.35	0.90	1.35
Reaction of piles	RE	1.00	1.00	1.00	1.00

Unfactored Loads		Shear	Longitudinal			
			Hx	My		
		(kN)	(kN)	(kN.m)		
Selfweight at front side		-1183		-1064		
Vertical soil on foot at front side		-217		-195		
Reaction of piles	Ser-I	10454		10592		
	Str-IA	14440		14849		
	Str-IB	10706		10816		
	Ext-I	11137		11509		

Load Combination at section 3-3

Load combinations	N	Longitudinal			
		Hx	My		
	(kN)	(kN)	(kN.m)		
Service Ser-I	9054		9332		
Strength Str-IA	12668		13254		
Strength Str-IB	9447		9682		
Extreme Ext-I	9366		9915		

1.4 Section 4-4

Table of Load Factors

Loads	Sign	Service I	Strength I		Extreme I
		Ser-I	Str-IA	Str-IB	Ext-I
Selfweight at behind side	DC	1.00	1.25	0.90	1.25
Vertical soil on foot at behind side	DC	1.00	1.35	0.90	1.35
Surcharge(Vertical)	EV	1.00	1.75	1.75	0.50
Reaction of piles	RE	1.00	1.00	1.00	1.00

Unfactored Loads		N (kN)	Longitudinal Hx (kN)	My (kN.m)	Tranversal Hy (kN)	Mx (kN.m)
Selfweight of behind side		-1613		-1807		
Vertical soil on foot at behind side		-2603		-2864		
Surcharge(Vertical)		-374		-412		
Reaction of piles	Ser-I	3841		2381		
	Str-IA	4642		2273		
	Str-IB	3522		1976		
	Ext-I	5821		4386		

Load Combination at section 4-4					
Load combinations	N (kN)	Longitudinal Hx (kN)	My (kN.m)	Tranversal Hy (kN)	Mx (kN.m)
Service Ser-I	-749		-2701		
Strength Str-IA	-1543		-4571		
Strength Str-IB	-927		-2947		
Extreme Ext-I	103		-1945		

1.4 Section 5-5 & 6-6

Slope of triang pressure	tgβ	=	6.00
Uniform horizontal pressure	U.p	=	4.23 kN/m2

Load Combination at section 5-5

Load combinations	N (kN)	Vertical Hy (kN)	Vertical My (kN.m)	Horizontal Hx (kN)	Horizontal Mx (kN.m)
Service Ser-I		143	135		
Strength Str-IA		285	202		

Load Combination at section 6-6

Load combinations	N (kN)	Vertical Hy (kN)	Vertical My (kN.m)	Horizontal Hx (kN)	Horizontal Mx (kN.m)
Service Ser-I				71	136
Strength Str-IA				108	204

2. Elements Checking

2.1. Bearing Resistance <S.5.7.5>

The case of absence of confinement reinforcement in the conrete supporting the bearing device
Factored bearing resistance shall be taken

Pr = ϕ. Pn = ϕ. 0.85. f'c. A1. m

Dimension of bearing plate	w0	=	0.800 m	
	b0	=	0.650 m	
Area under bearing device	A1	=	0.520 m2	
Distributed width and length	w	=	1.000 m	
	b	=	0.850 m	
Notational area	A2	=	0.850 m2	

Where supporting surface is wider on all sides than loades area
m = sqrt(A2/A1) ≤ 2.0 case 1
where loaded area is subjected to nouniformly distributed bearing
m = 0.75*sqrt(A2/A1) ≤ 1.5 case 2

Modification factor	case 1	m	=	1.279	
Resistance factor		ϕ	=	0.700	<S.5.5.4.2>
Factored bearing resistance		Pr	=	11867 kN	> Pu
Bearing reaction of approach bridge		Pu	=	5565 kN	Ok

Pu = 1.25*DC + 1.5*DW + 1.75*LL

In case factored applied load exceeds the factored resistance,
provision shall be made to resist the bursting and spalling force in article 5.10.9

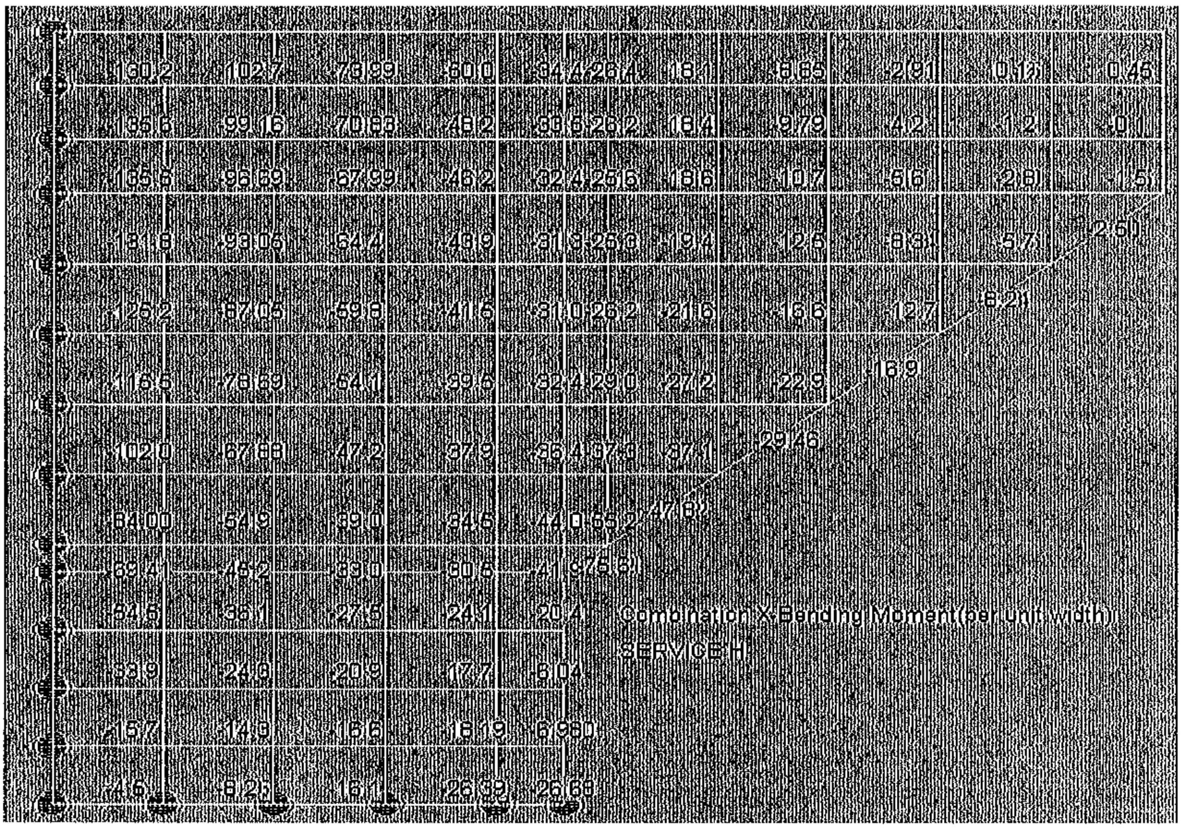
Factored bearing resistance shall be taken <S.5.10.9.7.2>

Pr = ϕ. fn. Ab
fn take the lesser of
fn = 0.7. f'ci. sqrt(A/Ag) and — fn = 26.85 MPa
fn = 2.25. f'ci

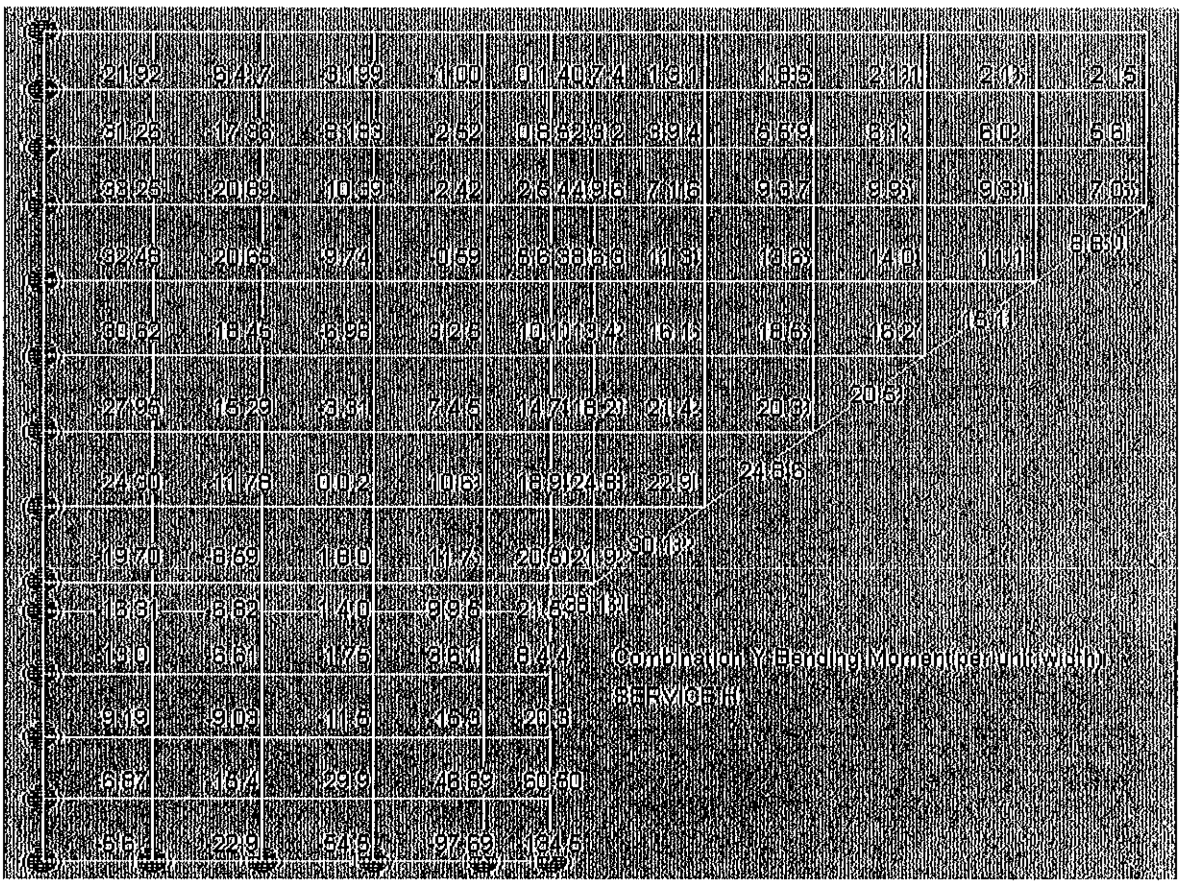
Maximum area of the portion of supporting surface	A	=	0.850 m2	
Gross area of bearing plate	Ag	=	0.520 m2	
Effective net area of bearing plate, Ag minus stud of bearing	Ab	=	0.520 m2	
Nominal concrete strength at time of application	f'ci	=	30 MPa	
Factored bearing resistance	Pr	=	9773 kN	Ok

LL Surcharge

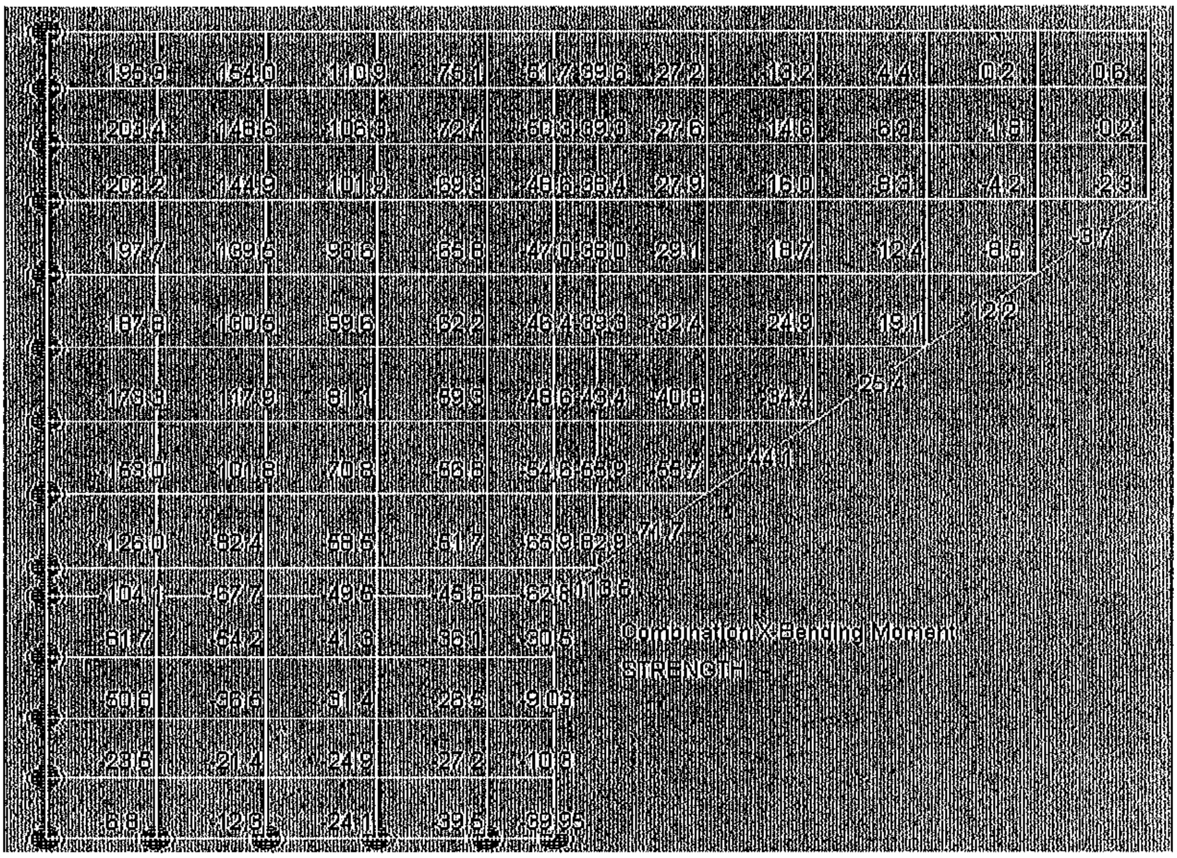
SERVICE – Element Moment X:



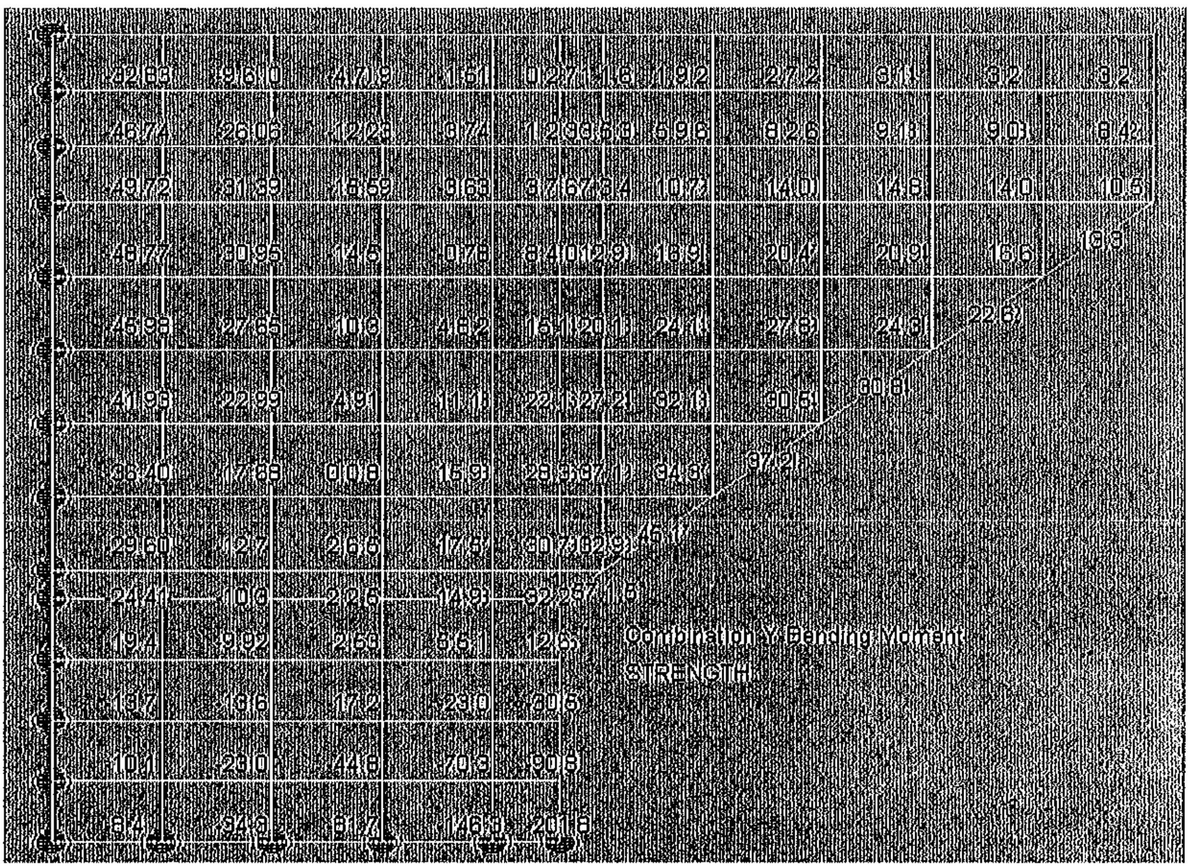
SERVICE – Element Moment Y:



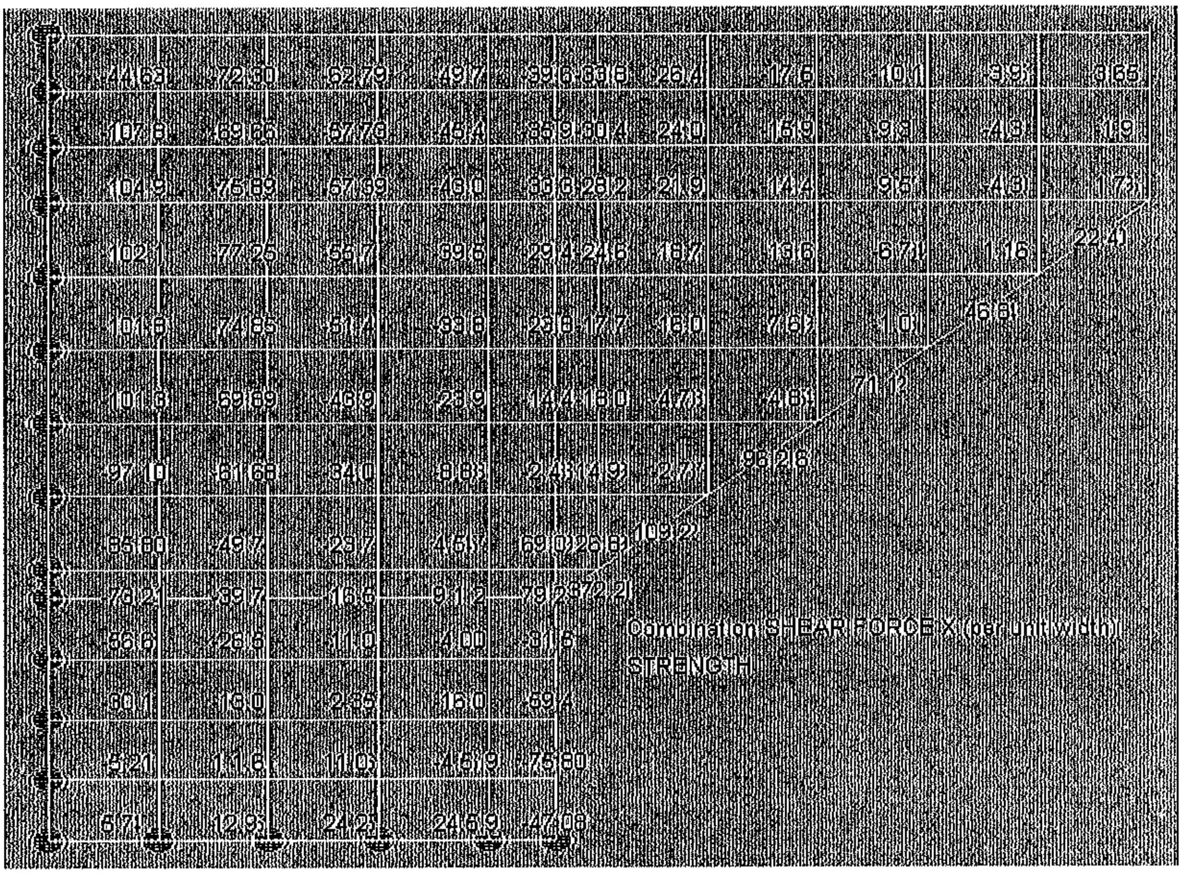
STRENGTH – Element Moment X:



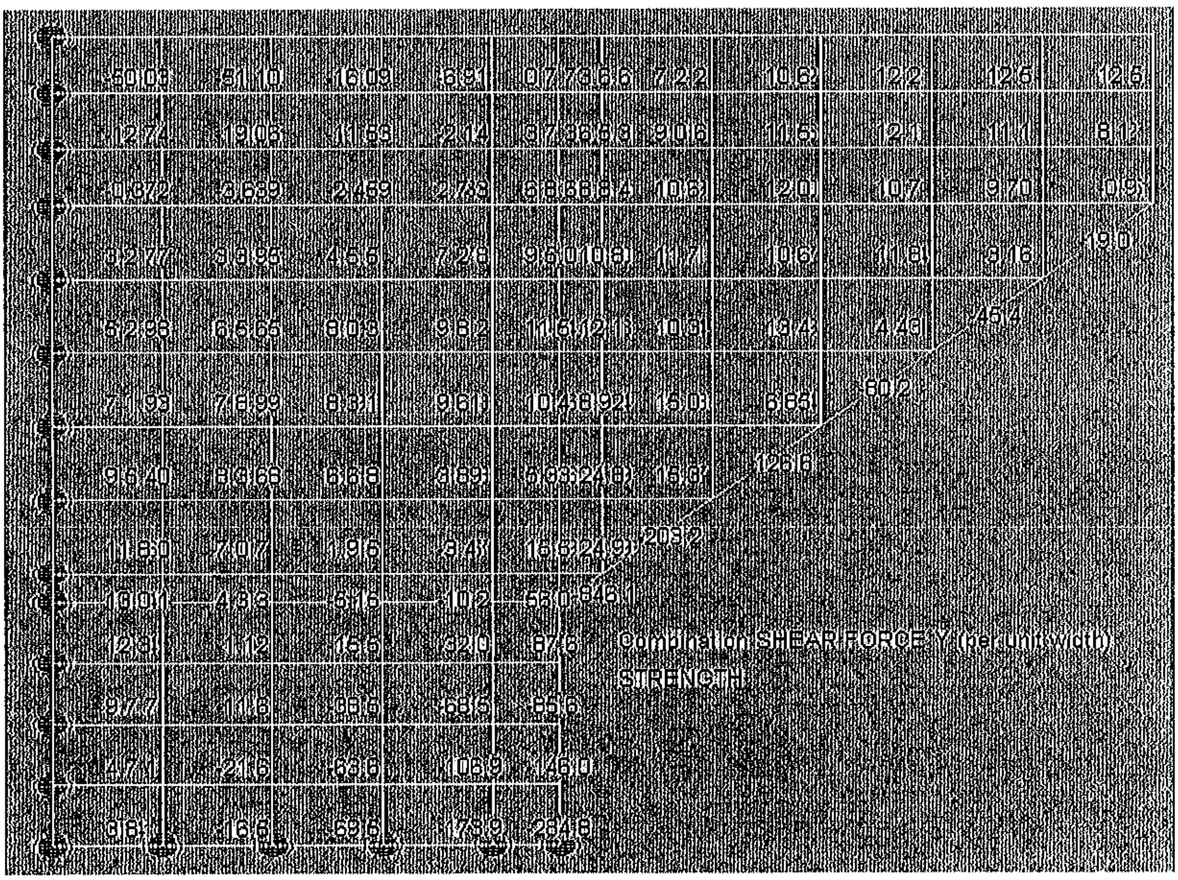
STRENGTH – Element Moment Y:



STRENGTH – Element Shear X:



STRENGTH – Element Shear Y:



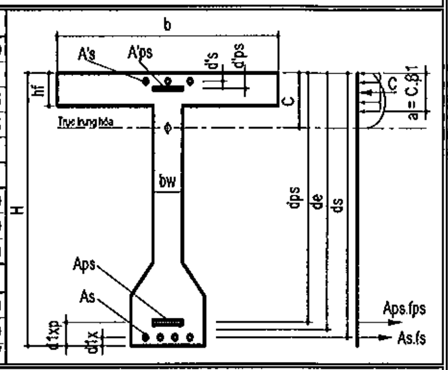
	Da Nang Quang Ngai Expressway project LRB09 BRIDGE	Item.	Eng.	Date.	Sign.
		Design			
	DETAIL DESIGN ABUTMENT A1	Check			
		Revise			

22TCN272-05; AASHTO LRFD 2nd - 1998

REINFORCEMENT CHECKING - HEAD AND STEM WALL

MATERIALS

	NORMAL CONCRETE		
f'c	Compressive Strength of concrete at 28 days	Mpa	30
Ec	Modulus of Elasticity	Mpa	27691
fr	Modulus of Rupture	Mpa	3.5
gc	Unit weight of concrete	kN/m3	24.5
	PRESTRESSING STEEL		
fpu	Tensile strength of prestressing steel	Mpa	1860
fpy	Yield strength of prestressing steel	Mpa	1670
Ep	Modulus of Elasticity	Mpa	195000
	REINFORCEMENT		
fy	Yield strength	Mpa	400
Es	Modulus of Elasticity	Mpa	200000
nc	Ratio Es/Ec		7



Sign	Parameters		Unit	Sections 1-1	1-1	2-2	2-2	2-2
	INTERNAL FORCES AT SECTIOIN							
	Combination			Strength	Service	Service	Strength	Extreme
Qu	Shear		kN	824	502	1496	2186	2044
Mu	Flexural Moment		kNm	841	505	4250	6083	4602
Nu	Axial load		kN	518	415	7250	9806	8150
Tu	Torsional Moment		kNm	0	0	0	0	0
	FLEXURAL MOMENT CHECKING							
H	Section height		m	0.500	0.500	1.500	1.500	1.500
d's	Dis. From comp. fiber to centroid of comp. Reinf		m	0.058	0.058	0.058	0.058	0.058
d1x	Dis. From tens. fiber to centroid of tension Reinf		m	0.058	0.058	0.060	0.060	0.060
	Cover to reinf		m	0.050	0.050	0.050	0.050	0.050
ds	Dis. From comp. fiber to centroid of tension Reinf		m	0.442	0.442	1.440	1.440	1.440
d'ps	Dis. From comp. fiber to centroid of comp. prestressing steel		m	0.000	0.000	0.000	0.000	0.000
d1xp	Dis. From tens. fiber to centroid of tension prestressing steel		m	0.000	0.000	0.000	0.000	0.000
dps	Dis. From comp. fiber to centroid of tension prestressing steel		m	0.500	0.500	1.500	1.500	1.500
b	Width of the compression face of member		m	12.600	12.600	12.600	12.600	12.600
bw	Web width or diameter of a circular section		m	12.600	12.600	12.600	12.600	12.600
hf	Compression flange depth		m	0.000	0.000	0.000	0.000	0.000
Iz	Moment of inertia of section		m4	0.131	0.131	3.544	3.544	3.544
Amc	Section area		m2	6.300	6.300	18.900	18.900	18.900
	Steel choice							
Aps	Tension prestressing steel	P.S type		0	0	0	0	0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
A'ps	Compression prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	0 T00.0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
As	Tension Reinforcement	Number	bars	**82**	**82**	**82**	**82**	**82**
		Diameter	mm	**16**	**16**	**20**	**20**	**20**
		Area	m2	0.01557	0.01557	0.02420	0.02420	0.02420
A's	Compression Reinforcement	Number	bars	**82**	**82**	**82**	**82**	**82**
		Diameter	mm	**16**	**16**	**16**	**16**	**16**
		Area	m2	0.01656	0.01656	0.01656	0.01656	0.01656
A'c	Shear reinforcement	Number	bars	**20**	**20**	**19**	**19**	**19**
		Diameter	mm	**14**	**14**	**14**	**14**	**14**
		Area	m2	0.00302	0.00302	0.00287	0.00287	0.00287
ϕ	Resistance factors for flexure		5.5.4.2	0.90	1.00	1.00	0.90	1.00
ϕv	Resistance factors for shear			0.90	1.00	1.00	0.90	1.00
ϕn	Resistance factors for axial force			1.00	1.00	1.00	1.00	1.00
β1	Stress block factor			0.836	0.836	0.836	0.836	0.836
c	Dis. Between centroid and top fiber		m	-0.001	-0.001	0.011	0.011	0.011
	For T section behavior		m	-0.001	-0.001	0.011	0.011	0.011
	For rectangular section behavior		m	-0.001	-0.001	0.011	0.011	0.011
fpe	Effective stress in the prestressing steel after losses		Mpa	1116	1116	1116	1116	1116
fps	Aver. stress in pres. steel at the time for which the nominal resistance		Mpa	1862	1862	1856	1856	1856
k	Factor depends on type of P.S, Low relaxation strand k = 0.28			0.28	0.28	0.28	0.28	0.28

	Da Nang Quang Ngai Expressway project	Item.	Eng.	Date.	Sign.
	LRB09 BRIDGE	Design			
	DETAIL DESIGN	Check			
	ABUTMENT A1	Revise			

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REINFORCEMENT CHECKING - HEAD AND STEM WALL

a	Depth of equivalent stress block	m	-0.001	-0.001	0.010	0.010	0.010
de	Corresp. effective depth from extreme comp. fiber						
	to centroid of tensile force in the tensile reinf.	m	0.442	0.442	1.440	1.440	1.440
Mn	Nominal resistance	kNm	2367	2367	13538	13538	13538
Mr	Factored reistance	kNm	**2131**	**2367**	**13538**	**12184**	**13538**
Mu	Flexual moment	kNm	841	505	4250	6083	4602
(5.7.3.2)	**Flexural moment Checking**		**OK**	**OK**	**OK**	**OK**	**OK**
	Limits for reinforcement						
c/de	Maximum reinforcement		0.00	0.00	0.01	0.01	0.01
	Maximum reinforcement Checking	<= 0.42	**OK**	**OK**	**OK**	**OK**	**OK**
1.2*Mcr	Craking moment	kNm	1084	1084	9857	9857	9857
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**		**OK**	**OK**	**OK**	**OK**	**OK**
(5.7.3.4)	**Conctrol of craking by distr. of reinf for RC member- Check?**		No	Yes	Yes	No	No
	Existing condition for structrure	1,2 or 3	1	1	1	1	1
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.058	0.058	0.060	0.060	0.060
Z	Crack width parameter	N/mm	30000	30000	30000	30000	30000
A	Area of concr. with same centroid as tens. Reinf	m2	0.018	0.018	0.018	0.018	0.018
fsa	Value	Mpa	297	297	290	290	290
0.6*fy		Mpa	240	240	240	240	240
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	240	240	240	240	240
x	Dist. From compression fiber to centroid	m	-	0.079	0.184	-	-
J.d	Arm	m	-	0.416	1.379	-	-
Icr	Moment of inertia of the cracked section	m4	-	0.016	0.295	-	-
fs	Tensile stress in reinforcement fs = Msls / (As*J.d)	Mpa	-	78	127	-	-
	Checking for control cracking fs<fsa		**N.a**	**OK**	**OK**	**N.a**	**N.a**
(5.10.8.2)	**Shrinkage and temperature Reinfocement (side distribution)**						
Areq	Area of required reinf	m2	0.00045	0.00045	0.00126	0.00126	0.00126
	Distribution on sides 7 D16	m2	0.00141	0.00141	0.00141	0.00141	
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45	0.45
	Checking		**OK**	**OK**	**OK**	**OK**	**OK**

SHEAR AND TORSION CHECKING

β	Factor indicating diag. cracked concr. to tension		2.4	2.7	4.2	3.9	4.1
θ	Angle of inclination of diagonal compressive	degree	32.87	28.89	27.00	27.40	27.00
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	90
bv	Effective web width as minimum web width - in dv	m	12.600	12.600	12.600	12.600	12.600
dv	Effective shear depth	m	0.443	0.443	1.435	1.435	1.435
	(de - a/2)	m	0.443	0.443	1.435	1.435	1.435
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	0.600
ncat	Amount of bars in spacing S	bars	20	20	19	19	19
Av	Shear reinf area in spacing S	m2	0.0030	0.0030	0.0029	0.0029	0.0029
θ	Assume	degree	31.64	28.80	33.96	37.37	35.68
v	Shear stress in concrete	kN/m2	164	53	83	134	113
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e_x	Strain in tensile reinforcement		7.42E-04	4.46E-04	9.23E-05	1.58E-04	1.15E-04
	if ex<0, multiple with reduce factor		-	-	-	-	-
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok	Ok
v/f'c	Ratio of shear stress and f'c		0.005	0.002	0.003	0.004	0.004
β	Final value		2.4	2.7	4.2	3.9	4.1
θ	Final value	degree	32.87	28.89	27.00	27.40	27.00
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	6021	6897	34717	31707	33409
Vs	Shear resistance provided by shear reinforcement	kN	1379	1615	5388	5296	5388
Vp	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0	0
Vn1	Vn1=Vc+Vs+Vp	kN	7400	8512	40105	37003	38796
Vn2	Vn2	kN	41828	41828	135631	135631	135631
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	7400	8512	40105	37003	38796
Vr	Factored shear resistance	kN	**6660**	**8512**	**40105**	**33303**	**38796**
Vu	Shear	kN	824	502	1496	2186	2044
(5.8.2.7)	**Shear checking**		**OK**	**OK**	**OK**	**OK**	**OK**
	Region requiring transverse reinf Checking		No need	No need	No need	No need	No need
	Minimum shear reinf area	m2	0.0086	0.0086	0.0086	0.0086	0.0086
	Mimimum shear reinforcement Checking		-	-	-	-	-
	0.1*f'c*bv*dv	kN	16731	16731	54252	54252	54252
	Smax	m	0.35	0.35	0.60	0.60	0.60
	Maximum spacing Smax		-	-	-	-	-

	Da Nang Quang Ngai Expressway project LRB09 BRIDGE DETAIL DESIGN ABUTMENT A1	Item.	Eng.	Date.	Sign.
		Design			
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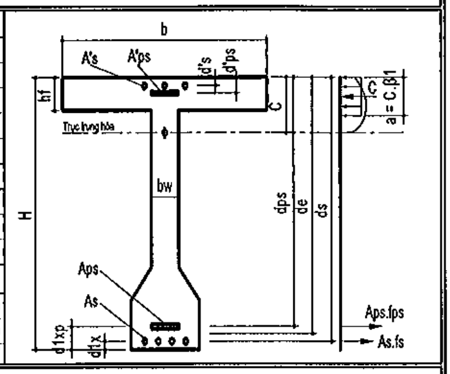
REINFORCEMENT CHECKING - PILECAP SECTION

MATERIALS

	NORMAL CONCRETE		
f'c	Compressive Strength of concrete at 28 days	Mpa	30
Ec	Modulus of Elasticity	Mpa	27691
fr	Modulus of Rupture	Mpa	3.5
gc	Unit weight of concrete	kN/m3	24.5
	PRESTRESSING STEEL		
fpu	Tensile strength of prestressing steel	Mpa	1860
fpy	Yield strength of prestressing steel	Mpa	1670
Ep	Modulus of Elasticity	Mpa	195000
	REINFORCEMENT		
fy	Yield strength	Mpa	400
Es	Modulus of Elasticity	Mpa	200000
nc	Ratio Es/Ec		7



Sign	Parameters		Unit	Sections				
				3-3	3-3	3-3	4-4	4-4
	INTERNAL FORCES AT SECTIOIN							
	Combination			Service	Strength	Extreme	Extreme	Strength
Qu	Shear		kN	9054	12668	9366	103	1543
Mu	Flexural Moment		kNm	9332	13254	9915	1945	4571
Nu	Axial load		kN	0	0	0	0	0
Tu	Torsional Moment		kNm	0	0	0	0	0
	FLEXURAL MOMENT CHECKING							
H	Section height		m	2.000	2.000	2.000	2.000	2.000
d's	Dis. From comp. fiber to centroid of comp. Reinf		m	0.083	0.083	0.083	0.083	0.083
d1x	Dis. From tens. fiber to centroid of tension Reinf		m	0.163	0.163	0.163	0.160	0.160
	Cover to reinf		m	0.075	0.075	0.075	0.075	0.075
ds	Dis. From comp. fiber to centroid of tension Reinf		m	1.838	1.838	1.838	1.840	1.840
d'ps	Dis. From comp. fiber to centroid of comp. prestressing steel		m	0.000	0.000	0.000	0.000	0.000
d1xp	Dis. From tens. fiber to centroid of tension prestressing steel		m	0.000	0.000	0.000	0.000	0.000
dps	Dis. From comp. fiber to centroid of tension prestressing steel		m	2.000	2.000	2.000	2.000	2.000
b	Width of the compression face of member		m	12.600	12.600	12.600	12.600	12.600
bw	Web width or diameter of a circular section		m	12.600	12.600	12.600	12.600	12.600
hf	Compression flange depth		m	0.000	0.000	0.000	0.000	0.000
Iz	Moment of inertia of section		m4	8.400	8.400	8.400	8.400	8.400
Amc	Section area		m2	25.200	25.200	25.200	25.200	25.200
	Steel choice							
Aps	Tension prestressing steel	P.S type		0	0	0	0	0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
A'ps	Compression prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	0 T00.0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
As	Tension Reinforcement	Number	bars	**84**	**84**	**84**	**84**	**84**
		Diameter	mm	**25**	**25**	**25**	**20**	**20**
		Area	m2	0.03876	0.03876	0.03876	0.02479	0.02479
A's	Compression Reinforcement	Number	bars	**0**	**0**	**0**	**0**	**0**
		Diameter	mm	16	16	16	16	16
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
A'c	Shear reinforcement	Number	bars	**15**	**15**	**13**	**13**	**13**
		Diameter	mm	**16**	**16**	**16**	**16**	**16**
		Area	m2	0.00303	0.00303	0.00263	0.00263	0.00263
ϕ	Resistance factors for flexure		5.5.4.2	1.00	0.90	1.00	1.00	0.90
ϕv	Resistance factors for shear			1.00	0.90	1.00	1.00	0.90
ϕn	Resistance factors for axial force			1.00	1.00	1.00	1.00	1.00
$\beta 1$	Stress block factor			0.836	0.836	0.836	0.836	0.836
c	Dis. Between centroid and top fiber		m	0.058	0.058	0.058	0.037	0.037
	For T section behavior		m	0.058	0.058	0.058	0.037	0.037
	For rectangular section behavior		m	0.058	0.058	0.058	0.037	0.037
fpe	Effective stress in the prestressing steel after losses		Mpa	1116	1116	1116	1116	1116
fps	Aver. stress in pres. steel at the time for which the nominal resistanc		Mpa	1845	1845	1845	1850	1850
k	Factor depends on type of P.S, Low relaxation strand k = 0.28			0.28	0.28	0.28	0.28	0.28

	Da Nang Quang Ngai Expressway project	Item.	Eng.	Date.	Sign.
	LRB09 BRIDGE	Design			
	DETAIL DESIGN	Check			
	ABUTMENT A1	Revise			

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REINFORCEMENT CHECKING - PILECAP SECTION

a	Depth of equivalent stress block	m	0.048	0.048	0.048	0.031	0.031
de	Corresp. effective depth from extreme comp. fiber						
	to centroid of tensile force in the tensile reinf.	m	1.838	1.838	1.838	1.840	1.840
Mn	Nominal resistance	kNm	28112	28112	28112	18089	18089
Mr	Factored reistance	kNm	**28112**	**25301**	**28112**	**18089**	**16280**
Mu	Flexual moment	kNm	9332	13254	9915	1945	4571
(5.7.3.2)	**Flexural moment Checking**		**OK**	**OK**	**OK**	**OK**	**OK**
	Limits for reinforcement						
c/de	Maximum reinforcement		0.03	0.03	0.03	0.02	0.02
	Maximum reinforcement Checking	<= 0.42	**OK**	**OK**	**OK**	**OK**	**OK**
1.2*Mcr	Craking moment	kNm	17908	17908	17908	17718	17718
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**		**OK**	**OK**	**OK**	**OK**	**OK**
(5.7.3.4)	**Conctrol of craking by distr. of reinf for RC member- Check?**		Yes	No	No	No	No
	Existing condition for structrure	1,2 or 3	3	3	3	3	3
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.050	0.050	0.050	0.050	0.050
Z	Crack width parameter	N/mm	17500	17500	17500	17500	17500
A	Area of concr. with same centroid as tens. Reinf	m2	0.015	0.015	0.015	0.015	0.015
fsa	Value	Mpa	193	193	193	193	193
0.6*fy		Mpa	240	240	240	240	240
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	193	193	193	193	193
x	Dist. From compression fiber to centroid	m	0.261	-	-	-	-
J.d	Arm	m	1.751	-	-	-	-
Icr	Moment of inertia of the cracked section	m4	0.749	-	-	-	-
fs	Tensile stress in reinforcement fs = Msls / (As*J.d)	Mpa	138	-	-	-	-
	Checking for control cracking fs<fsa		**OK**	**N.a**	**N.a**	**N.a**	**N.a**
(5.10.8.2)	**Shrinkage and temperature Reinfocement (side distribution)**						
Areq	Area of required reinf	m2	0.00127	0.00127	0.00127	0.00127	0.00127
	Distribution on sides 7 D16	m2	0.00141	0.00141	0.00141	0.00141	0.00141
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45	0.45
	Checking		**OK**	**OK**	**OK**	**OK**	**OK**

SHEAR AND TORSION CHECKING

β	Factor indicating diag. cracked concr. to tension		2.0	1.8	2.0	3.6	2.4
θ	Angle of inclination of diagonal compressive	degree	39.45	42.29	39.96	28.30	32.40
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	90
bv	Effective web width as minimum web width - in dv	m	12.600	12.600	12.600	12.600	12.600
dv	Effective shear depth	m	1.813	1.813	1.813	1.825	1.825
	(de - a/2)	m	1.813	1.813	1.813	1.825	1.825
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	0.600
ncat	Amount of bars in spacing S	bars	15	15	13	13	13
Av	Shear reinf area in spacing S	m2	0.0030	0.0030	0.0026	0.0026	0.0026
θ	Assume	degree	40.60	42.90	41.16	29.97	36.91
v	Shear stress in concrete	kN/m2	396	616	410	33	75
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e_x	Strain in tensile reinforcement		1.35E-03	1.82E-03	1.40E-03	2.33E-04	7.13E-04
	if ex<0, multiple with reduce factor		-	-	-	-	-
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok	Ok
v/f'c	Ratio of shear stress and f'c		0.013	0.021	0.014	0.001	0.002
β	Final value		2.0	1.8	2.0	3.6	2.4
θ	Final value	degree	39.45	42.29	39.96	28.30	32.40
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	21155	18715	20857	37183	24989
Vs	Shear resistance provided by shear reinforcement	kN	4451	4027	3788	5933	5033
Vp	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0	0
Vn1	Vn1=Vc+Vs+Vp	kN	25606	22742	24645	43116	30022
Vn2	Vn2	kN	171364	171364	171364	172422	172422
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	25606	22742	24645	43116	30022
Vr	Factored shear resistance	kN	**25606**	**20468**	**24645**	**43116**	**27019**
Vu	Shear	kN	9054	12668	9366	103	1543
(5.8.2.7)	**Shear checking**		**OK**	**OK**	**OK**	**OK**	**OK**

	Da Nang Quang Ngai Expressway project LRB09 BRIDGE	Item.	Eng.	Date.	Sign.
		Design			
	DETAIL DESIGN ABUTMENT A1	Check			
		Revise			

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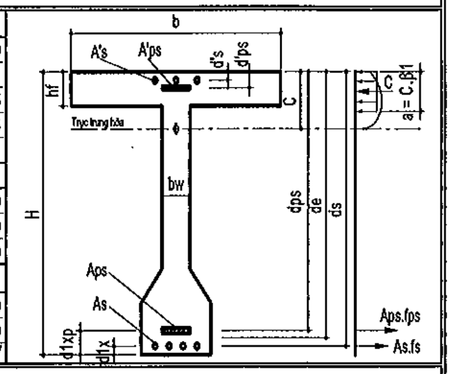
REINFORCEMENT CHECKING - WING WALL

MATERIALS

	NORMAL CONCRETE		
f'c	Compressive Strength of concrete at 28 days	Mpa	30
Ec	Modulus of Elasticity	Mpa	27691
fr	Modulus of Rupture	Mpa	3.5
gc	Unit weight of concrete	kN/m3	24.5
	PRESTRESSING STEEL		
fpu	Tensile strength of prestressing steel	Mpa	1860
fpy	Yield strength of prestressing steel	Mpa	1670
Ep	Modulus of Elasticity	Mpa	195000
	REINFORCEMENT		
fy	Yield strength	Mpa	400
Es	Modulus of Elasticity	Mpa	200000
nc	Ratio Es/Ec		7



Sign	Parameters		Unit	Sections				
				5-5	5-5	6-6	6-6	
	INTERNAL FORCES AT SECTIOIN							
	Combination			Service	Strength	Service	Strength	
Qu	Shear		kN	143	285	71	108	
Mu	Flexural Moment		kNm	135	202	136	204	
Nu	Axial load		kN	0	0	0	0	
Tu	Torsional Moment		kNm	0	0	0	0	
	FLEXURAL MOMENT CHECKING							
H	Section height		m	0.500	0.500	0.500	0.500	
d's	Dis. From comp. fiber to centroid of comp. Reinf		m	0.058	0.058	0.058	0.058	
d1x	Dis. From tens. fiber to centroid of tension Reinf		m	0.060	0.060	0.060	0.060	
	Cover to reinf		m	0.050	0.050	0.050	0.050	
ds	Dis. From comp. fiber to centroid of tension Reinf		m	0.440	0.440	0.440	0.440	
d'ps	Dis. From comp. fiber to centroid of comp. prestressing steel		m	0.000	0.000	0.000	0.000	
d1xp	Dis. From tens. fiber to centroid of tension prestressing steel		m	0.000	0.000	0.000	0.000	
dps	Dis. From comp. fiber to centroid of tension prestressing steel		m	0.500	0.500	0.500	0.500	
b	Width of the compression face of member		m	1.000	1.000	1.000	1.000	
bw	Web width or diameter of a circular section		m	1.000	1.000	1.000	1.000	
hf	Compression flange depth		m	0.000	0.000	0.000	0.000	
Iz	Moment of inertia of section		m4	0.010	0.010	0.010	0.010	
Amc	Section area		m2	0.500	0.500	0.500	0.500	
	Steel choice							
Aps	Tension prestressing steel	P.S type		0	0	0	0	
		Number	tendons	0	0	0	0	
		Area	m2	0.00000	0.00000	0.00000	0.00000	
A'ps	Compression prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	
		Number	tendons	0	0	0	0	
		Area	m2	0.00000	0.00000	0.00000	0.00000	
As	Tension Reinforcement	Number	bars	7	7	7	7	
		Diameter	mm	20	20	20	20	
		Area	m2	0.00220	0.00220	0.00220	0.00220	
A's	Compression Reinforcement	Number	bars	7	7	7	7	
		Diameter	mm	16	16	16	16	
		Area	m2	0.00141	0.00141	0.00141	0.00141	
A'c	Shear reinforcement	Number	bars	2	2	2	2	
		Diameter	mm	12	12	12	12	
		Area	m2	0.00023	0.00023	0.00023	0.00023	
ϕ	Resistance factors for flexure		5.5.4.2	1.00	0.90	1.00	0.90	
ϕv	Resistance factors for shear			1.00	0.90	1.00	0.90	
ϕn	Resistance factors for axial force			1.00	1.00	1.00	1.00	
β1	Stress block factor			0.836	0.836	0.836	0.836	
c	Dis. Between centroid and top fiber		m	0.015	0.015	0.015	0.015	
	For T section behavior		m	0.015	0.015	0.015	0.015	
	For rectangular section behavior		m	0.015	0.015	0.015	0.015	
fpe	Effective stress in the prestressing steel after losses		Mpa	1116	1116	1116	1116	
fps	Aver. stress in pres. steel at the time for which the nominal resistanc		Mpa	1844	1844	1844	1844	
k	Factor depends on type of P.S, Low relaxation strand k = 0.28			0.28	0.28	0.28	0.28	

REINFORCEMENT CHECKING - WING WALL

a	Depth of equivalent stress block	m	0.012	0.012	0.012	0.012
de	Corresp. effective depth from extreme comp. fiber					
	to centroid of tensile force in the tensile reinf.	m	0.440	0.440	0.440	0.440
Mn	Nominal resistance	kNm	352	352	352	352
Mr	Factored reistance	kNm	**352**	**317**	**352**	**317**
Mu	Flexual moment	kNm	**135**	**202**	136	**204**
(5.7.3.2)	**Flexural moment Checking**		**OK**	**OK**	**OK**	**OK**
	Limits for reinforcement					
c/de	Maximum reinforcement		0.03	0.03	0.03	0.03
	Maximum reinforcement Checking	<= 0.42	**OK**	**OK**	**OK**	**OK**
r min	Mimimum reinforcement		0.44%	0.44%	0.44%	0.44%
	Mimimum reinforcement Checking for RC	0.23%	**OK**	**OK**	**OK**	**OK**
1.2*Mcr	Craking moment	kNm	89	89	89	89
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**		**OK**	**OK**	**OK**	**OK**
(5.7.3.4)	**Conctrol of craking by distr. of reinf for RC member- Check?**		Yes	No	Yes	No
	Existing condition for structrure	1,2 or 3	1	1	1	1
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.060	0.060	0.060	0.060
Z	Crack width parameter	N/mm	30000	30000	30000	30000
A	Area of concr. with same centroid as tens. Reinf	m2	0.017	0.017	0.017	0.017
fsa	Value	Mpa	297	297	297	297
0.6*fy		Mpa	240	240	240	240
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	240	240	240	240
x	Dist. From compression fiber to centroid	m	0.102	-	0.102	-
J.d	Arm	m	0.406	-	0.406	-
Icr	Moment of inertia of the cracked section	m4	0.002	-	0.002	-
fs	Tensile stress in reinforcement fs = Msls / (As*J.d)	Mpa	151	-	152	-
	Checking for control cracking fs<fsa		**OK**	**N.a**	**OK**	**N.a**
(5.10.8.2)	**Shrinkage and temperature Reinfocement (side distribution)**					
Areq	Area of required reinf	m2	0.00031	0.00031	0.00031	0.00031
	Distribution on sides 7 D16	m2	0.00141	0.00141	0.00141	0.00141
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45
	Checking		**OK**	**OK**	**OK**	**OK**

SHEAR AND TORSION CHECKING

β	Factor indicating diag. cracked concr. to tension		2.3	2.0	2.3	2.1
θ	Angle of inclination of diagonal compressive	degree	35.33	40.60	33.89	38.10
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90
bv	Effective web width as minimum web width - in dv	m	1.000	1.000	1.000	1.000
dv	Effective shear depth	m	0.434	0.434	0.434	0.434
	(de - a/2)	m	0.434	0.434	0.434	0.434
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600
ncat	Amount of bars in spacing S	bars	2	2	2	2
Av	Shear reinf area in spacing S	m2	0.0002	0.0002	0.0002	0.0002
β	Assume		2.0	2.2	2.0	2.0
θ	Assume	degree	34.57	38.94	36.11	41.24
v	Shear stress in concrete	kN/m2	330	730	164	277
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116
e_x	Strain in tensile reinforcement		9.44E-04	1.46E-03	8.24E-04	1.21E-03
	if ex<0, multiple with reduce factor		-	-	-	-
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok
v/f'c	Ratio of shear stress and f'c		0.011	0.024	0.005	0.009
β	Final value		2.3	2.0	2.3	2.1
θ	Final value	degree	35.33	40.60	33.89	38.10
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	446	389	459	417
Vs	Shear resistance provided by shear reinforcement	kN	92	76	97	83
Vp	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0
Vn1	Vn1=Vc+Vs+Vp	kN	538	465	557	500
Vn2	Vn2	kN	3254	3254	3254	3254
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	538	465	557	500
Vr	Factored shear resistance	kN	**538**	**419**	**557**	**450**
Vu	Shear	kN	143	285	71	108
(5.8.2.7)	**Shear checking**		**OK**	**OK**	**OK**	**OK**

BORED PILE DESIGN

I. BORED PILE DATA

1.Load Combinations at top of bored pile

STT	Comb	Axial force P (KN)	Moment (KN.m)		
			Mx	My	Mxy
STRENGTH LIMIT STATES					
1	P_min	1057	7.7	750.2	750
2	P_max	4990	11.7	1099.1	1099
3	Mx_max	3452	7.7	750.2	750
4	My_max	4636	11.7	1099.1	1099
EXTREME EVENT LIMIT STATES					
1	P_min	917	75	832	835
2	P_max	3776	73	866	870
3	Mx_max	3649	73	866	870
4	My_max	3649	73	866	870

2. Bored pile Material

Normal concrete			
Compressive strength at 28 days age	f'c	30	MPa
Concrete elastic modulus	Ec	27691	MPa
Reinforcement TCVN1651-2008; CBV-400			
Yield strength	fy	400	MPa
Reinforcement elastic modulus	Es	200,000	MPa

3. Bored pile Section

Pile diameter	D	1.00	m
Section area	A	0.785	m2
Moment inertia	Ix	0.049	m4
	Iy	0.049	m4
Radius of gyration of gross concrete section; r = sqrt(I/A)	rx	0.250	m
	ry	0.250	m

II. PILE DESIGN

1. Limit of Reinforcement

S.5.7.4.2

Minimum area of longitudinal reinforcement in column			
As.fy / (Ag . f'c) >= 0.135	As ≥	0.008	m2
As / Ag >= 0.01	As ≥	0.008	m2
Maximum area of longitudinal reinforcement in column			
As / Ag <= 0.08	As ≤	0.063	m2
Trial Rebars: **Ok**	As	**0.019**	m2
1layers x 24 = 24 bars D32 @150	As1	0.019	m2

2. Interation diagram M-P

Using Pca-Column software

****Flexural check by pcaColumn**

Strength and Service limit states:

Resistance factor: Compression φ_c = 0.75 *(AASHTO LRFD-2004)*

Tension φ_t = 0.90

Extreme Event limit states:

Resistance factor Compression φ_c = 1.00

Tension φ_t = 1.00

<Result table>

STT	Comb	Pu kN	Mux kN-m	Muy kN-m	φ Mnx kN-m	φ Mny kN-m	φ Mn/Mu
STRENGTH LIMIT STATES							
1	P_max	1057	7.7	750.2	26.3	2557.7	3.409
2	P_min	4990.2	11.7	1099.1	28.1	2642.6	2.404
3	Mx_max	3451.8	7.7	750.2	27.8	2711.7	3.615
4	My_max	4636.3	11.7	1099.1	28.4	2667.1	2.427
EXTREME EVENT LIMIT STATES							
1	P_max	917	75	832	255.4	2833.5	3.406
2	P_min	3776	73	866	282	3345.1	3.863
3	Mx_max	3649	73	866	280.5	3328	3.843
4	My_max	3649	73	866	280.5	3328	3.843

3. Column Ties

S.5.7.4.6, S.5.10.6.3, S5.10.11.4.1d - e

Bridge is in seismic zone	Sz	1	
Area of concrete core measured out-to-out of ties	Ac	0.622	m2
Tie diameter	Dtie	16	mm2
Cross section area of 1 tie	As-tr	0.00020	m2
Spacing of hoops	s	75	mm
Length of reinfocement tie in 1 hoop	Ltie	2.78	m
Ratio of ties reinf. in one hoop/ volume of conc. core one pitch spacing			
ρs = As-tr . Ltie / (Ac * spacing)	ρs	0.0120	
Ratio of spiral reinf. To total volume of concrete core shall satisfy			S.5.7.4.6
ρs >= 0.45 . (Ag/Ac - 1). f'c/ fy = Req1	Req1	0.0089	**OK**
Transverse Reinforcement for Confinement at Plastic Hinges		S.5.10.11.4.1.d	
For a circular column "1:applied","2:Not applied"		1	
ρs >= 0.12 . f'c/ fy = Req2	Req2	0.0090	**N/A**

4. Shear Design

Shear resistance factors	ϕv	1.0	
Factored shear force	Vu	429	kN
Required shear capacity Vn = Vu/ ϕv	Vn	429	kN
Determine concrete shear capacity			
Minimum shear reinforcement will provided in cross section			
Therefore	β	2.0	
	θ	45.0	deg
Diameter of bored pile	D	1.00	m
Width of cross section	b	1.00	m
dv = 0.9*de de = D/2 + Dr/pi()			
Diameter of the circle passing through the centers of the long. reinf.	Dr	0.79	m
	de	0.75	m
	dv	0.68	m
Vc = 0.083 . β . sqrt(f'c) . bv. dv	Vc	616	kN
Diameter of Transverse Reinforcement	D_v	13	mm
Spacing of Transverse Reinforcement	s	75	mm
Area of a transverse reinforcement within distance "s"	A_v	339.30	mm2
Angle of inclination of transverse reinforcement to longitudinal axis	α	90	deg.
Effetive shear depth, d_v			
Alternative 1: $d_{v1} = M_n/(A_s{*}f_y)$			
Normal flexural resistance	M_n	3328	KNm
	d_{v1}	**216**	mm
Alternative 2: $d_{v2} = 0.9d_e$	$d_e = D/2 + D_r/\pi$	752	mm
	d_{v1}	677	mm
Choice value of d 1 *("1" = d_{v1}, "2" = d_{v2})*	d_v	**216**	mm
	$V_s = \dfrac{A_v f_y d_v(\cot g\theta + \cot g\alpha)\sin\alpha}{s}$		
Norminal shear resistance of Reinforcement		**392**	kN
	$V_{n1} = V_c + V_s$	**1007**	kN
	$V_{n2} = 0.25f'_c b_v d_v$	**1623**	kN
	V_n	**1007**	kN
	Conclude		**OK**

SPACE PILE FOUNDATION ANALYSIS PROGRM
Turbo BASIC

PROJECT: : DN-QN-A1-LRB09

INITIA DATA

Kn = 0.18 Ax = 5.50 By = 12.60 Cz = 2.00
E v.uon = 3001028 E r.uon = 3001028 E v.nen = 3001028 E r.nen = 3001028
Mq = 75 (t/m4) Md = 0 (t/m4) m = 600 (t/m4)

LOAD COMBINATIONS

COMB.	Hx	Hy	P	Mx	My	Mz
1	384.94	0.00	1945.13	-128.47	841.55	0.00
2	268.14	0.00	1450.42	-123.93	650.78	0.00
3	367.94	8.12	1875.86	-159.38	773.81	0.00
4	251.13	8.12	1381.15	-154.84	583.05	0.00
5	259.95	6.72	1457.21	-126.60	567.30	0.00
6	276.75	17.04	1728.66	-104.24	293.35	0.00
7	276.75	17.04	1233.95	-99.70	434.70	0.00

PROPERTIES OF PILES

PILE	Lo	H	Bpx	Bpy	A	B	Cday	Fo	Io	Po	Co	Ct
1	0.00	10.00	1.620	1.620	1.00	0.000	0.000	0.785	0.098	500	30000	15000
2					n	t						
3					n	t						
4					n	t						
5					n	t						
6					n	t						

PILE COORD.

PILE	X	Y	Phi	Xi
1	1.50	6.18	0.000	0.00
2	1.50	0.55	0.000	0.00
3	1.50	-5.10	0.000	0.00
4	-1.50	-6.18	0.000	0.00
5	-1.50	-0.55	0.000	0.00
6	-1.50	5.10	0.000	0.00

DISPLACEMENTS

COMB.	X	Y	Z	Fix	Fiy	Fiz
1	0.01115	-0.00009	0.003787	0.000037	0.001310	0.000000
2	0.00788	-0.00006	0.002824	0.000025	0.000960	0.000000
3	0.01060	0.00010	0.003652	0.000030	0.001228	-0.000000
4	0.00733	0.00013	0.002689	0.000017	0.000877	-0.000000

5	0.00752	0.00009	0.002837	0.000020	0.000882	-0.000000
6	0.00746	0.00033	0.003366	0.000013	0.000708	-0.000000
7	0.00771	0.00032	0.002403	0.000018	0.000814	-0.000000

FORCES ON PILES

PILE	COMB.	N	Q2	Q3	M1	M2	M3
1	1	472.61	-60.37	-0.02	0.000	-1.194	112.037
	2	351.87	-42.01	-0.01	0.000	-0.790	76.472
	3	454.54	-57.73	-1.34	0.000	-4.309	107.898
	4	333.79	-39.36	-1.34	0.000	-3.904	72.333
	5	345.51	-40.77	-1.11	0.000	-3.422	75.734
	6	371.99	-43.63	-2.80	0.000	-7.473	88.320
	7	300.77	-43.53	-2.80	0.000	-7.612	84.814
2	1	490.62	-60.37	-0.02	0.000	-1.194	112.037
	2	363.78	-42.01	-0.01	0.000	-0.790	76.472
	3	468.88	-57.73	-1.34	0.000	-4.309	107.898
	4	342.04	-39.36	-1.34	0.000	-3.904	72.333
	5	355.20	-40.77	-1.11	0.000	-3.422	75.734
	6	378.43	-43.63	-2.80	0.000	-7.473	88.320
	7	309.30	-43.53	-2.80	0.000	-7.612	84.814
3	1	508.69	-60.37	-0.02	0.000	-1.194	112.037
	2	375.73	-42.01	-0.01	0.000	-0.790	76.472
	3	483.28	-57.73	-1.34	0.000	-4.309	107.898
	4	350.32	-39.36	-1.34	0.000	-3.904	72.333
	5	364.94	-40.77	-1.11	0.000	-3.422	75.734
	6	384.90	-43.63	-2.80	0.000	-7.473	88.320
	7	317.85	-43.53	-2.80	0.000	-7.612	84.814
4	1	175.76	-60.37	-0.02	0.000	-1.194	112.037
	2	131.61	-42.01	-0.01	0.000	-0.790	76.472
	3	170.75	-57.73	-1.34	0.000	-4.309	107.898
	4	126.59	-39.36	-1.34	0.000	-3.904	72.333
	5	140.23	-40.77	-1.11	0.000	-3.422	75.734
	6	204.23	-43.63	-2.80	0.000	-7.473	88.320
	7	110.54	-43.53	-2.80	0.000	-7.612	84.814
5	1	157.76	-60.37	-0.02	0.000	-1.194	112.037
	2	119.70	-42.01	-0.01	0.000	-0.790	76.472
	3	156.40	-57.73	-1.34	0.000	-4.309	107.898
	4	118.34	-39.36	-1.34	0.000	-3.904	72.333
	5	130.53	-40.77	-1.11	0.000	-3.422	75.734
	6	197.79	-43.63	-2.80	0.000	-7.473	88.320
	7	102.02	-43.53	-2.80	0.000	-7.612	84.814
6	1	139.69	-60.37	-0.02	0.000	-1.194	112.037
	2	107.75	-42.01	-0.01	0.000	-0.790	76.472
	3	142.01	-57.73	-1.34	0.000	-4.309	107.898
	4	110.07	-39.36	-1.34	0.000	-3.904	72.333
	5	120.80	-40.77	-1.11	0.000	-3.422	75.734
	6	191.32	-43.63	-2.80	0.000	-7.473	88.320
	7	93.46	-43.53	-2.80	0.000	-7.612	84.814

SUMMARY OF FORCES

	PILE	COMB.	N	Q2	Q3	M1	M2	M3
Nmin	6	7	93.46	-43.53	-2.80	0.000	-7.612	84.814
Nmax	3	1	508.69	-60.37	-0.02	0.000	-1.194	112.037
Q2max	1	1	472.61	-60.37	-0.02	0.000	-1.194	112.037
Q3max	1	7	300.77	-43.53	-2.80	0.000	-7.612	84.814
M1max	1	1	472.61	-60.37	-0.02	0.000	-1.194	112.037

M2max	1	7	300.77	-43.53	-2.80	0.000	-7.612	84.814
M3max	1	1	472.61	-60.37	-0.02	0.000	-1.194	112.037

CHECKING CALCULATI
IN COMPARISON WITH INITIA LOAD MATRIX

1	384.94	0.00	1945.13	-128.47	841.55	0.00
2	268.14	0.00	1450.42	-123.93	650.78	0.00
3	367.94	8.12	1875.86	-159.38	773.81	0.00
4	251.13	8.12	1381.15	-154.84	583.05	0.00
5	259.95	6.72	1457.21	-126.60	567.30	0.00
6	276.75	17.04	1728.66	-104.24	293.35	0.00
7	276.75	17.04	1233.95	-99.70	434.70	0.00

Da Nang Quang Ngai Expressway project

BRIDGE

LRB09

CALCULATION SHEETS

ABUTMENT A2

2013

Table of content

1.Structure dimensions and Loads
2. Foundation analysis
3. Elements checks
4.Pile Design

LOAD COMPONENTS

Assumptions :

1. Bridge is considered to be in seismic with acceleration coeff.
2. The Design of the Abutment accords with Specification for bridge design 22-TCN-272-05 and AASHTO LRFD 2004 for reference
3. Design live load: HL-93 and lane loading 9.3 KN/m

Input :

Level Table(at center of abutment)

Level of top of headwall	HTwL	17.448	m
Level of top of bearing	BTL	15.407	m
Level of top of stem abutment	HTL	15.173	m
Level of top of footing	FTL	10.500	m
Level of bottom of footing	FBL	8.500	m
Ground level	GL	11.300	m
Lowest water level	HWL	8.000	m
Skew angle	α	20.00	deg

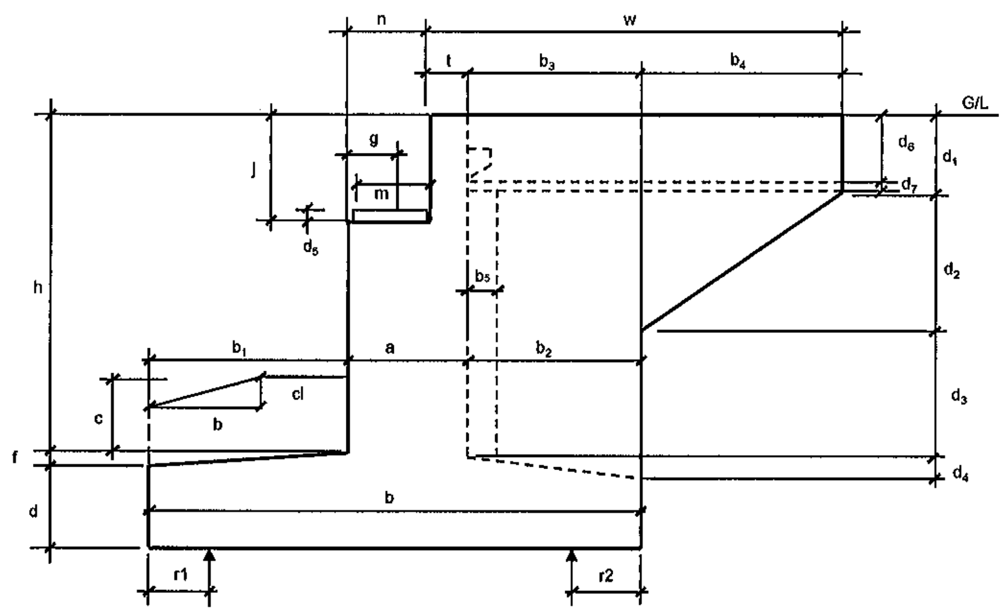
Material Unit Weights

- Unit Weight of Reinf. concrete γ_c = 24.5 kN/m^3
- Unit Weight of Soil γ_s = 18.0 kN/m^3
- Unit Bouyancy Weight of Soil γ_{sbo} = 8.2 kN/m^3
- Unit weight of asphalt concrete γ_a = 22.1 kN/m^3

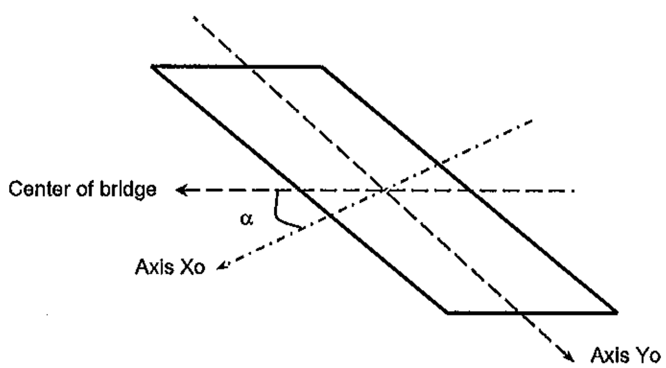
I.Loads from substructure

Abutment dimensions VERTICAL VIEW

Bearing Type:	MOVE

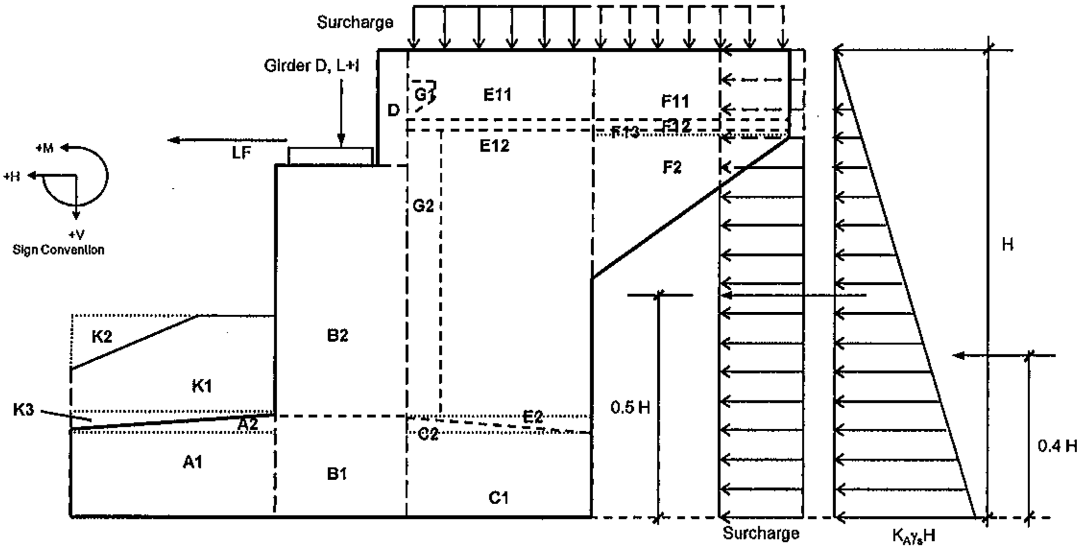


PLAN VIEW



ABUTMENT DIMENSIONS (IN METRES)					
Item	Symbol	Value	Item	Symbol	Value
Height of stem	h	6.948	Horizontal Dimension	b_4	2.307
Footing Width	b	6.000	Horizontal Dimension	b_5	0.300
Stem Width	a	1.500	Vertical Dimension	d_1	0.930
Footing Depth	d	2.000	Vertical Dimension	d_2	2.307
Footing Slope	f	0.000	Vertical Dimension	d_3	3.711
Width of stem at bearing	n	1.000	Vertical Dimension	d_4	
Ballast Wall Height	j	2.275	Vertical Dimension	d_5	0.234
Ballast Wall Thickness	t	0.500	Vertical Dimension	d_6	1.070
Wingwall Length	w	5.500	Vertical Dimension	d_7	
Soil Cover at Toe	c	0.800	With of bearing pad	m	0.800
Girder Reaction	g	0.530	Wingwall Thickness	u1	0.500
Horizontal Dimension	b_1	2.000	Wingwall Thickness	u2	0.500
Horizontal Dimension	b_2	2.500	Distance to cl of pile	r1	1.000
Horizontal Dimension	b_3	2.660	Distance to cl of pile	r2	1.000

	cos (α)	=	0.94
Slope front of abutment	cl	=	0.00 m
	bl	=	0.00 m
Width of Abutment	L		12.600 m
Width of abutment (inclined direction)	Ltr		13.409 m
Height of Abutment	Ht	=	8.95 m
Distance from CG of footing to edge of Abutment	b/2	=	3.00 m



1. Self weight of Abutment (DC)

Description	Area (m^2)	Length (m)	Force (kN)	X[1] (m)	Arm[2] (m)	Moment (kN•m)
SW of Abutment (DC)						
Section A1	4.000	13.409	1314	1.000	2.000	2628
Section A2	-	13.409	-	1.333	1.667	-
Section B1	3.000	13.409	986	2.750	0.250	246
Section B2	7.010	13.409	2303	2.750	0.250	576
Section C1	5.000	13.409	1643	4.750	-1.750	-2874
Section C2	-	13.409	-	4.333	-1.333	-
Section D	1.138	13.409	374	3.250	-0.250	-93
Section E11	2.474	0.500	30	4.830	-1.830	-55
Section E12	16.011	0.500	196	4.830	-1.830	-359
Part extra stem	-	-	-	5.524	-2.524	-
Section F11	2.469	0.500	30	7.314	-4.314	-130
Section F12	-	0.500	-	5.984	-2.984	-
Section F13	-0.323	0.500	-4	7.314	-4.314	17
Section F2	2.662	0.500	33	6.930	-3.930	-128
Section G1	0.135	12.409	41	3.650	-0.650	-27
Section G2	0.045	6.948	8	3.650	-0.650	-5
Bearing seats (w1seat= 0.65m)	0.187	3.250	15	2.530	0.470	7
Curbs +Handrail on Abutment	0.50	5.500	67	5.750	-2.750	-185
Total SW of Abutment (DC)			***7035***			***-384***
Tranverser moment			*59*		*6.175*	*364*

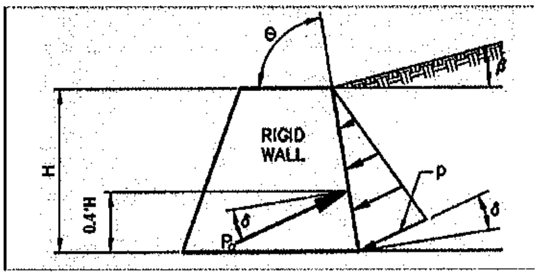
Notes:
1. Distance 'X' is measured horizontally from Toe of Retaining to CG of Section
2. Moment 'Arm' is measured from CG horizontally and from Underside of Footing Vertically.

2. Earth on Abutment (EV)

Description	Area (m^2)	Length (m)	Force (kN)	X(1) (m)	Arm(2) (m)	Moment (kN•m)
Earth on Abutment (EV)						
Section E1	17.37	12.909	4036	4.750	-1.750	-7063
Section E2	-	12.409	-	5.167	-2.167	-
Section E3	-0.60	1.000	-11	6.080	-3.080	33
Section K1	1.600	13.409	386	1.000	2.000	-
Section K2	-	13.409	-	-	3.000	-
Section K3	-	13.409	-	0.667	2.333	-
Total Earth on Footing			*4411*			*-7030*

3. Horizontal Earth Pressure on Abutment (EH)

To be safe, horizontal earth pressure at front face of abutment may be neglected.
Horizontal earth pressure at behind face of abutment shall be considered.



$$Ka = \frac{\sin^2(\theta + \phi'_f)}{\Gamma.[\sin^2\theta.\sin(\theta - \delta)]}$$

$$\Gamma = \left[1 + \sqrt{\frac{\sin(\phi'_f+\delta).\sin(\phi'_f-\beta)}{\sin(\theta-\delta).\sin(\theta+\beta)}}\right]^{-2}$$

• Height for horizontal earth pressure	H	=	8.95 m
• Width for horizontal earth pressure	W	=	13.4 m
• Density of Soil	γ_s	=	1835 kg/m^3
• Internal Friction Angle of Soil	ϕ'_f	=	30.0 deg
• Incline angle of back face wall	θ	=	90.0 deg
• Friction angle between fill and wall	δ	=	0.0 deg
• Incline angle of fill soil	β	=	0.0 deg
• Gravitational acceleration	g	=	9.81 m/s^2

• Basic earth pressure

p = K.γs. g. Z .10^-9 (Mpa, Z:mm)

K: taken as Ka (assume wall move or deflect sufficiently to reach minimum active conditions)

	Γ	=	2.250
	Ka	=	0.333
	p	=	0.054 Mpa
Horizontal earth pressure:			
• Ea = 0,5. p. Z . B . 10^3 (kN)	Ea	=	3221 kN
• M= Ea . 0,4H	M	=	11528 kNm

4. Earth Pressure on Abutment due to Surcharge (ES)

Equivalent height of soil for highway loading taken from Table 3.11.6.2.1

H=	**1.50m**	**heq=**	**1.7** m	
H=	**3.00m**	**heq=**	**1.2** m	
H=	**6.00m**	**heq=**	**0.76** m	
H=	**9.00m**	**heq=**	**0.61** m	
H=	8.95m	heq=	0.61 m	(Linear Interpolation)

• Vertical force	ESv	=	370 kN
	ev	=	-1.75 m
	M	=	-647 kNm
• Horizontal force	ESh	=	441 kN
$\Delta p = k.\gamma_s.g.h_{eq}x10^9$	eh	=	4.47 m
	M	=	1973 kNm

5. Earthquake effects

Bridge is located at:
According to TCXDVN 375:2006 and 22TCN272-05, bridge is in seismic zone 2 and acceleration coefficient as below
• Peak ground acceleration coefficient A = 0.0310 g

5.1. Seismic active lateral Earth pressure (E_{AE})

• Backfill slop angle i = 0.0 deg
• Slope of wall to vertical β' = 0.0 deg
• Angle of friction of soil φ = 30.0 deg
• Angle of friction between soil and abutment δ = 0.0 deg
• Horizontal acceleration coefficient kh = 0.047
• Vertical acceleration coefficient kv = 0.019
• Angle θ = arctan(k_h/ (1-k_v)) θ = 2.7 deg

$$K_{AE} = \frac{\cos^2(\phi-\theta-\beta)}{\cos\theta.\cos^2\beta\cos(\delta+\beta+\theta)} x \left[1+\sqrt{\frac{\sin(\phi+\delta).\sin(\phi-\theta-i)}{\cos(\delta+\beta+\theta).\cos(i-\beta)}}\right]^{-2}$$

• Seismic active lateral Earth pressure coefficient K_{AE} = 0.36

• E_{AE} = 0,5. g. γ. H^2 . (1-kv) . K_{AE}. 10^-9 (kN/m)

• Seismic active lateral Earth pressure coefficient E_{AE} = 3431 kN
M_{AE}= E_{AS}*0.3H+(E_{AE} - E_{AS})*0.6H M_{AE} = 9776 KNm <A.11.1.1.1>
E_{AS} is the static component of seismic active pressure calculated with θ = kv = 0

5.2. Earthquake effects to abutment (EQ)

Seismic force for substructures: elements above ground Fh = Csm* W; elements under ground Fh = A*S*W
• Soil profile type Soil type I
• Site Coefficients. S = 1.0
• Elastic Seismic Response Coefficient 2.5A = 0.078
Csm = 1.2*A*S/ Tm^2/3 ≤ 2.5*A Csm = 0.042
• Period of vibration of the fundamental mode
Tm = 2*pi()*sqrt(m/k) Tm = 0.822 s

Decription	Area (m^2)	Length (m)	Force (kN)	X[(1)] (m)	Arm[(2)] (m)	Moment (kN•m)
Section A1	4.000	13.409	56	-	1.000	56
Section A2	-	13.409	-	-	2.000	-
Section B1	3.000	13.409	42	-	1.000	42
Section B2	7.010	13.409	98	-	4.337	423
Section C1	5.000	13.409	70	-	1.000	70
Section C2	-	13.409	-	-	2.000	-
Section D	1.138	13.409	16	-	7.811	124
Section E11	2.474	0.500	1	-	6.413	8
Section E12	16.011	0.500	8	-	2.939	
Section E2	-	-	-	-	2.000	-
Section F11	2.469	0.500	1	-	6.413	8
Section F12	-	0.500	-	-	5.878	
Section F13	-0.323	0.500	-0	-	7.018	
Section F2	2.662	0.500	1	-	7.249	10
Section G1	0.135	12.409	2	-	6.235	11
Section G2	0.045	6.948	0	-	2.939	1
Total EQ of Abutment Selfweight			***295***			***752***

6. Braking Force(BR)

Take 50 % Braking Force for this Abutment (Free Bearing)

• Number of lanes	n	=	3 lanes	
• Multiple presence factor	m	=	0.85	
• Take 25 % of Truck load				
BR = 25% * n * m * (2*145+35)	BR	=	104 kN	Long. Axis
• Acting at 1.8m higher of road face	e	=	10.7 m	
	Mlong	=	1113 KNm	Long. Axis

7. Centrifucal Force , CE (3.6.3)

• Plan of bridge (1:"straight",2: "Curve")			1	
• Design Speed	V	=	120 km/h	
	V	=	33.3 m/s	
	R		- m	
$C = 4/3*(V^2/gR)$	C		-	
Acting at 1.8m higher of road face				
CE = n * m * (2*145+35) * C	CE	=	0.00 KN	
	e	=	8.75 m	
	Mtrans	=	0.00 KNm	Trans. Axis

8. Water Load (WA) ***:NA***

8.1. Buoyancy of Abutment

• Heightest water Level +8.00

Description	Area (m^2)	Length (m)	Force (kN)	X[1] (m)	Arm[2] (m)	Moment (kN•m)
Bouyancy on abutment						
Section A1	-	13.409	-	1.000	2.000	-
Section A2	-	13.409	-	-	3.000	-
Section B(B1,B2)	-	13.409	-	2.750	0.250	-
Section C1	-	13.409	-	4.750	-1.750	-
Section C2	-	13.409	-	-	3.000	-
Section E2	-	1.000	-	-	3.000	-
Section E1	-	1.000	-	4.830	-1.830	-
Section F2	-	1.000	-	4.090	-1.090	-
Total Bouyancy			-			-

8.2 Buoyancy of Earth on Abutment

Description	Area (m^2)	Length (m)	Force (kN)	X[1] (m)	Arm[2] (m)	Moment (kN•m)
Bouyancy of earth on abutement						
Section E2	-	12.409	-	-	3.000	-
Section E1	-	12.409	-	4.750	-1.750	-
Section K2	-	13.409	-	-	3.000	-
Section K1	-	13.409	-	1.000	2.000	-
- Section K3	-	13.409	-	-	3.000	-
Total Bouyancy			-			-

SUPERSTRUCTURE LOADS

II.Loads from superstructure

Item	Sign	Value	Unit
Span length	Lst	33.00	m
Span between bearings	Ls	32.10	m
Bridge Width	W	12.60	m
Number of girders	n_g	5.00	Girders
Girder height	Hg	1.65	m
Deck slab depth	Hd	0.247	m
Asphalt depth	Hασ	0.084	m

1.Dead loads (DC): One span at abutment

Item	Sign	Value	Unit
1.1.Girders			
Sum of girders weight	DC	3347.93	kN
Precast Planks	DC	473.46	kN
Diaphragm	DC	405.16	kN
Total	DC	4226.55	kN
1.2.Deck slab			
Deck slab	DC	2464.29	kN
1.3.Pavement			
Asphalt concrete	DW	649.37	kN
1.4.Parapet			
Parapet + median	DC	889.35	kN

2.Live load (LL):
2.1. Live load

Truck

145 145 35 kN

4.3 4.3 m

Tandem

110 110 kN

1.2 m

Lane load

W_L 9.3 kN/m

Pedestrial Wpd= 0.0 kN/m2

Considerate structure as a simple span

Reaction Influence

32.10 m

1

Number of lanes	n	3	
Multiple presence factor	m	0.85	
Dynamic load allowance	1+IM	1.25	

*Reaction = [(1+IM)*Vehicle + Laneload]*n*m*

Truck	P1(kN)	P2(kN)	P3(kN)	Sum(kN)	Total(kN)
Axle load	145	145	35		
Influence value	1.000	0.866	0.732		
Reaction	145.0	125.6	25.6	296.2	944.1
Tandem	P1(kN)	P2(kN)		Sum(kN)	Total(kN)
Axle load	110	110			
Influence value	1.000	0.963			
Reaction	110	105.9		215.9	688.1
Lane load	Wl(kN/m)				Total(kN)
Value	9.3				
Influence value	16.05				
Reaction	149.3				380.6
Pedestrian	Wdb(kN)				Total(kN)
Reaction	0.0				0.0

3.Earthquake effects on superstructrue (EQ)

Force from superstructure due to EQ	EQ	=	174 kN

4.Uniform Temperature, Srinkage& Creep(TU+SH&CR)

Bearing displacement due to uniform temperature and shrinkage creep	Δu	=	0.026 m
Shear modulus G	G	=	1 MPa
Bearing area	A	=	0.175 m2
Height of elastomeric layers	hrt		0.084 m
Number of bearing	nb	=	5 bears
Horizontal force due to TU+SH&CR			
Acting at top of bearing $H = G.A.\Delta u/h_{rt}$	Hx	=	271 kN

5.Wind loads (Ws)

5.1.Tranverse wind on superstructure (WS)

Wind zone	Zone		III
Basic 3 second gust wind	Vb	=	53.00 m/s
Correction factor	S	=	1.09
Design wind velocity	V	=	57.77 m/s
Drag coefficient	Cd	=	1.40
Overall width of bridge	b	=	12.60 m
Depth of superstructure(including solid parapet)	d	=	2.97 m
	b/d	=	4.25
Windy obstructed area of superstructure	At	=	97.91 m2
Transverse wind load			
$P_D = \max(0.0006V^2.C_d.A_t, 1.8A_t)$ =	Hy	=	274.5 kN
Longitudinal wind load			
$F_{WSL} = 0.25P_D$ =	Hx	=	68.6 kN

5.2.Wind load on vehicles (WL)

Transverse wind load on vehicle	Hy	=	24.75 kN
Longitudinal wind load on vehicles	Hx	=	24.75 kN
(At 1.8m from surface)			

6.Combinations

Loads from superstructure to Abutment

Loads at bottom of stem		Vertical		Longitudinal			Tranversal		
Loads	Sign	N (kN)	x (m)	Hx (kN)	z_1 (m)	My (kN•m)	Hy (kN)	y (m)	Mx (kN•m)
Girder+Deck+Parapet	DC	3790	0.22			834			
Pavement	DW	325	0.22			71			
LiveLoad	LL	1325	0.22			291		0.48	629
Pedestrian	PD							-	-
Trans. wind on Struc.	WS			34	4.91		137	4.91	673
Trans. wind on vehi.	WL			12	11.02		25	11.02	273
Eearth quake	EQ			87	4.91		52	4.91	257
TU+SH&CR	TU+SH&CR			135	4.91	664			

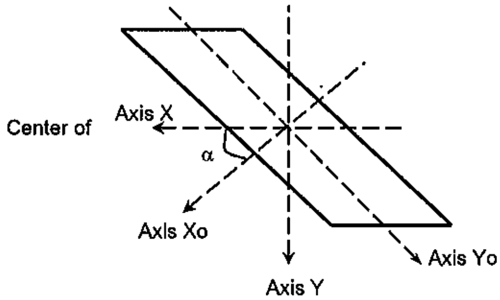
Loads at bottom of pilecap		Vertical		Longitudinal			Tranversal		
Loads	Sign	N (kN)	x (m)	Hx (kN)	z_1 (m)	My (kN•m)	Hy (kN)	y (m)	Mx (kN•m)
Girder+Deck+Parapet	DC	3790	0.47			1781			
Pavement	DW	325	0.47			153			
LiveLoad	LL	1325	0.47			623		0.48	629
Pedestrian	PD			-	-	-		-	-
Trans. wind on Struc.	WS			34	6.91	237	137	6.91	948
Trans. wind on vehi.	WL			12	13.02	161	25	13.02	322
Eearth quake	EQ			87	6.91	602	52	6.91	361
TU+SH&CR	TU+SH&CR			135	6.91	935			

Loads		Load combinations						
	Sign	Str-IA	Str-IB	Str-IIIA	Str-IIIB	Ser-I	Ext-IA	Ext-IB
Girder+Deck+Parapet	DC	1.25	0.90	1.25	0.90	1.00	1.25	0.90
Pavement	DW	1.50	0.65	1.50	0.65	1.00	1.50	0.65
LiveLoad	LL	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Pedestrian	PD	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Trans. wind on Struc.	WS			0.40	0.40	0.30		
Trans. wind on vehi.	WL			1.00	1.00	1.00		
Eearth quake	EQ						1.00	1.00
TU+SH&CR	TU+SH&CR	0.50	0.50	0.50	0.50	1.00		

Load combinations at bottom of stem		Longitudinal		Tranversal	
Load combinations	N (kN)	Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	7543	68	1992	0	1101
Strength Str-IB	5940	68	1639	0	1101
Strength Str-IIIA	7013	94	1875	80	1392
Strength Str-IIIB	5411	94	1523	80	1392
Service Ser-I	5440	158	1861	66	1104
Extreme Ext-IA	5887	87	1295	52	571
Extreme Ext-IB	4285	87	943	52	571

Load combinations at bottom of pilecap		Longitudinal		Tranversal	
Load combinations	N (kN)	Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	7543	68	4013	0	1101
Strength Str-IB	5940	68	3260	0	1101
Strength Str-IIIA	7013	94	4020	80	1551
Strength Str-IIIB	5411	94	3267	80	1551
Service Ser-I	5440	158	3724	66	1236
Extreme Ext-IA	5887	87	3369	52	676
Extreme Ext-IB	4285	87	2616	52	676

LOAD COMBINATIONS



III.Load Combinations

1.Loads from substructure

Loads at bottom of pilecap	Sign	Vertical N (kN)	Longitudinal Hx (kN)	My (kN•m)	Tranversal Hy (kN)	Mx (kN•m)
Self weight of Abutment	DC	7035		-384		363.7061
Soils on pilecap	EV	4411		-7030		
Horizontal Earth Pressure	EH		3027	10833		
Vertical Surcharge	LSv	370		-647		
Horizontal Surcharge	LSh		414	1854		
Braking Force	BR		104	1113		
Centrifucal Force	CE		-	-	-	-
Buoyancy of Abutment	WA	-		-		
Buoyancy of Earth on Abutment	WA	-		-		
Earthquake effects to Abutment	EQ		295	752	88	226
Earthquake effects to soil	E_{AE}		3224	9187		

Loads	Sign	Load factors						
		Str-IA	Str-IB	Str-IIIA	Str-IIIB	Ser-I	Ext-IA	Ext-IB
Self weight of Abutment	DC	1.25	0.90	1.25	0.90	1.00	1.25	0.90
Soils on pilecap	EV	1.35	0.90	1.35	0.90	1.00	1.35	0.90
Horizontal Earth Pressure	EH	1.50	0.90	1.50	0.90	1.00	0.00	0.00
Vertical Surcharge	LSv	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Horizontal Surcharge	LSh	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Braking Force	BR	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Centrifucal Force	CE	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Buoyancy of Abutment	WA	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Buoyancy of Earth on Abutment	WA	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Earthquake effects to Abutment	EQ						1.00	1.00
Earthquake effects to soil	E_{AE}						1.00	1.00

Load Combination at bottom of pilecap					
Load combinations	N (kN)	Longitudinal Hx (kN)	My (kN.m)	Tranversal Hy (kN)	Mx (kN.m)
Strength Str-IA	15396	5446	10340	0	455
Strength Str-IB	10949	3630	7138	0	327
Strength Str-IIIA	15248	5239	9411	0	455
Strength Str-IIIB	10801	3423	6210	0	327
Service Ser-I	11816	3545	5739	0	364
Extreme Ext-IA	14934	3778	1129	88	680
Extreme Ext-IB	10487	3778	4427	88	553

2. Loads from superstructure

Load Combination at bottom of pilecap					
		Longitudinal		Tranversal	
Load combinations	N (kN)	Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	7543	68	4013	0	1101
Strength Str-IB	5940	68	3260	0	1101
Strength Str-IIIA	7013	94	4020	80	1551
Strength Str-IIIB	5411	94	3267	80	1551
Service Ser-I	5440	158	3724	66	1236
Extreme Ext-IA	5887	87	3369	52	676
Extreme Ext-IB	4285	87	2616	52	676

3. Total loads at bottom of pilecap

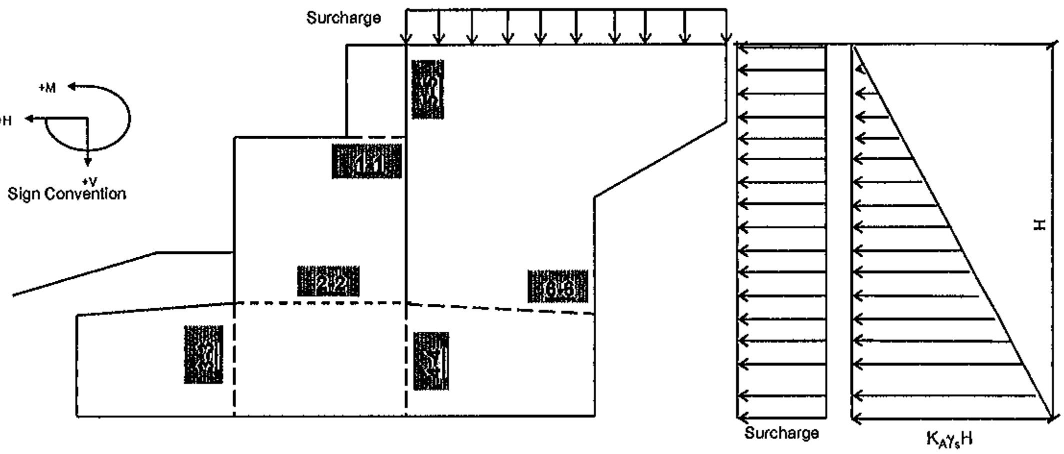
Load Combination at bottom of pilecap					
		Longitudinal		Tranversal	
Load combinations	N (kN)	Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	22939	5514	14353	0	1556
Strength Str-IB	16889	3698	10398	0	1429
Strength Str-IIIA	22261	5333	13431	80	2006
Strength Str-IIIB	16211	3517	9476	80	1878
Service Ser-I	17255	3703	9464	66	1600
Extreme Ext-IA	20821	3865	4499	141	1356
Extreme Ext-IB	14771	3865	7043	141	1229

ELEMENTS CHECKING

IV.Elements checking

The abutment walls shall be checked at sections 1-1, 2-2, 3-3, 4-4, 5-5

1. Calculate internal force of sections



1.1 Section 1-1
Table of Load Factors

Loads	Sign	Service I	Strength I		Extreme I
		Ser-I	Str-IA	Str-IB	Ext-I
Selfweight	DC	1.00	1.25	0.90	1.25
Horizontal Earth Pressure	EH	1.00	1.50	0.90	
Surchage (Horizontal)	LS_h	1.00	1.75	1.75	0.50
Horizontal Seismic Earth Pressure	E_{AE}				1.50
Abutment earthquake force	EQ				1.00

Unfactored Loads	N	Longitudinal		Tranversal	
		Hx	My	Hy	Mx
	(kN)	(kN)	(kN.m)	(kN)	(kN.m)
Selfweight	415		-16		
Horizontal Earth Pressure		222	202		
Surcharge (horizontal)		281	319		
Horizontal Seismic Earth Pressure		236	171		
Abutment earthquake force		18	20	5	6

Load Combination at bottom of headwall					
Load combinations	N	Longitudinal		Tranversal	
		Hx	My	Hy	Mx
	(kN)	(kN)	(kN.m)	(kN)	(kN.m)
Service Ser-I	415	502	505	0	0
Strength Str-IA	518	824	841	0	0
Strength Str-IB	373	691	726	0	0
Extreme Ext-I	518	512	416	5	6

1.2 Section 2-2
Table of Load Factors

Loads	Sign	Service I	Strength I		Extreme I
		Ser-I	Str-IA	Str-IB	Ext-I
Selfweight	DC	1.00	1.25	0.90	1.25
Superstructure Dead Load	DC	1.00	1.25	0.90	1.25
Pavement	DW	1.00	1.50	0.65	1.50
Live Load	LL	1.00	1.75	1.75	0.50
Horizontal Earth Pressure	EH	1.00	1.50	0.90	
Surchage (Horizontal)	LS_h	1.00	1.75	1.75	0.50
TU+SH&CR	TU+SH&CR	1.00	0.50	0.50	
Horizontal Seismic Earth Pressure	E_{AE}				1.50
Abutment earthquake force	EQ				1.00

Unfactored Loads	N	Longitudinal		Tranversal	
		Hx	My	Hy	Mx
	(kN)	(kN)	(kN.m)	(kN)	(kN.m)
Selfweight	2717		-191		
Superstructure Dead Load	3790		834		
Pavement	325		71		
Live Load	1325		291		629
Horizontal Earth Pressure		1942	5397		
Surchage (Horizontal)		374	1300		
TU+SH&CR		135	664		
Horizontal Seismic Earth Pressure		2202	5131		
Abutment earthquake force		115	331	87	356

Load Combination at bottom of stem wall

Load combinations	N	Longitudinal		Tranversal	
		Hx	My	Hy	Mx
	(kN)	(kN)	(kN.m)	(kN)	(kN.m)
Service Ser-I	8157	2452	8367	0	629
Strength Str-IA	10940	3636	12124	0	1101
Strength Str-IB	8386	2470	8600	0	1101
Extreme Ext-I	9284	3605	9733	87	671

1.3 Section 3-3

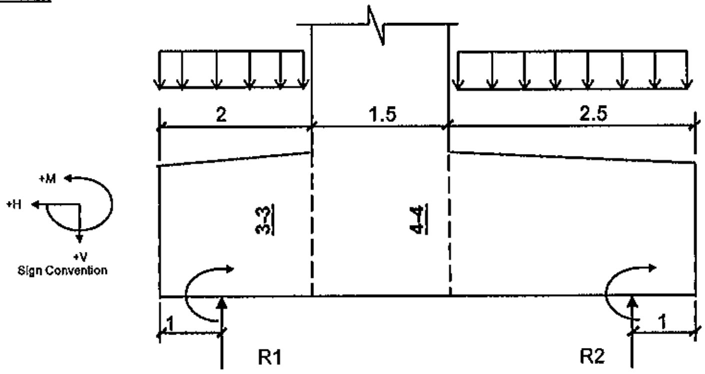


Table of Load Factors

Loads	Sign	Service I	Strength I		Extreme I
		Ser-I	Str-IA	Str-IB	Ext-I
Selfweight at front side	DC	1.00	1.25	0.90	1.25
Vertical soil on foot at front side	DC	1.00	1.35	0.90	1.35
Reaction of piles	RE	1.00	1.00	1.00	1.00

Unfactored Loads		Shear	Longitudinal			
			Hx	My		
		(kN)	(kN)	(kN.m)		
Selfweight at front side		-1314		-1314		
Vertical soil on foot at front side		-386		-386		
Reaction of piles	Ser-I	13214		18835		
	Str-IA	18331		26617		
	Str-IB	13239		18795		
	Ext-I	14087		20380		

Load Combination at section 3-3

Load combinations	N	Longitudinal			
		Hx	My		
	(kN)	(kN)	(kN.m)		
Service Ser-I	11514		17135		
Strength Str-IA	16167		24454		
Strength Str-IB	11708		17265		
Extreme Ext-I	11923		18216		

1.4 Section 4-4

Table of Load Factors

Loads	Sign	Service I	Strength I		Extreme I
		Ser-I	Str-IA	Str-IB	Ext-I
Selfweight at behind side	DC	1.00	1.25	0.90	1.25
Vertical soil on foot at behind side	DC	1.00	1.35	0.90	1.35
Surcharge(Vertical)	EV	1.00	1.75	1.75	0.50
Reaction of piles	RE	1.00	1.00	1.00	1.00

Unfactored Loads		N	Longitudinal		Tranversal	
			Hx	My	Hy	Mx
		(kN)	(kN)	(kN.m)	(kN)	(kN.m)
Selfweight of behind side		-1928		-2539		
Vertical soil on foot at behind side		-4025		-5032		
Surcharge(Vertical)		-370		-462		
Reaction of piles	Ser-I	4041		2689		
	Str-IA	4608		1940		
	Str-IB	3650		2142		
	Ext-I	6733		6325		

Load Combination at section 4-4					
Load combinations	N	Longitudinal		Tranversal	
		Hx	My	Hy	Mx
	(kN)	(kN)	(kN.m)	(kN)	(kN.m)
Service Ser-I	-2282		-5343		
Strength Str-IA	-3883		-8835		
Strength Str-IB	-2354		-5480		
Extreme Ext-I	-1295		-3873		

1.4 Section 5-5 & 6-6

Slope of triang pressure	tgβ	=	6.00
Uniform horizontal pressure	U.p	=	3.68 kN/m2

Load Combination at section 5-5					
Load combinations	N	Vertical		Horizontal	
		Hy	My	Hx	Mx
	(kN)	(kN)	(kN.m)	(kN)	(kN.m)
Service Ser-I		143	135		
Strength Str-IA		285	202		

Load Combination at section 6-6					
Load combinations	N	Vertical		Horizontal	
		Hy	My	Hx	Mx
	(kN)	(kN)	(kN.m)	(kN)	(kN.m)
Service Ser-I				71	137
Strength Str-IA				112	216

2. Elements Checking

2.1. Bearing Resistance <S.5.7.5>

The case of absence of confinement reinforcement in the conrete supporting the bearing device
Factored bearing resistance shall be taken

$Pr = \phi.\ Pn = \phi.\ 0.85.\ f'c.\ A1.\ m$

Dimension of bearing plate	w0	=	0.800 m	
	b0	=	0.650 m	
Area under bearing device	A1	=	0.520 m2	
Distributed width and length	w	=	1.000 m	
	b	=	0.850 m	
Notational area	A2	=	0.850 m2	

Where supporting surface is wider on all sides than loades area
$m = sqrt(A2/A1) \le 2.0$ case 1
where loaded area is subjected to nouniformly distributed bearing
$m = 0.75*sqrt(A2/A1) \le 1.5$ case 2

Modification factor case 1	m	=	1.279	
Resistance factor	ϕ	=	0.700	<S.5.5.4.2>
Factored bearing resistance	Pr	=	11867 kN	> Pu
Bearing reaction of approach bridge	Pu	=	5565 kN	Ok

$Pu = 1.25*DC + 1.5*DW + 1.75*LL$

In case factored applied load exceeds the factored resistance,
provision shall be made to resist the bursting and spalling force in article 5.10.9

Factored bearing resistance shall be taken <S.5.10.9.7.2>

$Pr = \phi.\ fn.\ Ab$

fn take the lesser of

$fn = 0.7.\ f'ci.\ sqrt(A/Ag)$ and

$fn = 2.25.\ f'ci$

	fn	=	26.85 MPa	
Maximum area of the portion of supporting surface	A	=	0.850 m2	
Gross area of bearing plate	Ag	=	0.520 m2	
Effective net area of bearing plate, Ag minus stud of bearing	Ab	=	0.520 m2	
Nominal concrete strength at time of application	f'ci	=	30 MPa	
Factored bearing resistance	Pr	=	9773 kN	Ok

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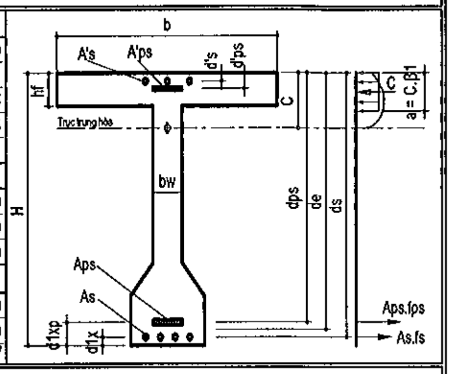
REINFORCEMENT CHECKING - HEAD AND STEM WALL

MATERIALS

	NORMAL CONCRETE		
f'c	Compressive Strength of concrete at 28 days	Mpa	30
Ec	Modulus of Elasticity	Mpa	27691
fr	Modulus of Rupture	Mpa	3.5
gc	Unit weight of concrete	kN/m3	24.5
	PRESTRESSING STEEL		
fpu	Tensile strength of prestressing steel	Mpa	1860
fpy	Yield strength of prestressing steel	Mpa	1670
Ep	Modulus of Elasticity	Mpa	195000
	REINFORCEMENT		
fy	Yield strength	Mpa	400
Es	Modulus of Elasticity	Mpa	200000
nc	Ratio Es/Ec		7



Sign	Parameters		Unit	Sections				
				1-1	1-1	2-2	2-2	2-2
	INTERNAL FORCES AT SECTIOIN							
	Combination			Strength	Service	Service	Strength	Extreme
Qu	Shear		kN	824	502	2452	3636	3605
Mu	Flexural Moment		kNm	841	505	8367	12124	9733
Nu	Axial load		kN	518	415	8157	10940	9284
Tu	Torsional Moment		kNm	0	0	0	0	0
	FLEXURAL MOMENT CHECKING							
H	Section height		m	0.500	0.500	1.500	1.500	1.500
d's	Dis. From comp. fiber to centroid of comp. Reinf		m	0.058	0.058	0.058	0.058	0.058
d1x	Dis. From tens. fiber to centroid of tension Reinf		m	0.058	0.058	0.061	0.061	0.061
	Cover to reinf		m	0.050	0.050	0.050	0.050	0.050
ds	Dis. From comp. fiber to centroid of tension Reinf		m	0.442	0.442	1.439	1.439	1.439
d'ps	Dis. From comp. fiber to centroid of comp. prestressing steel		m	0.000	0.000	0.000	0.000	0.000
d1xp	Dis. From tens. fiber to centroid of tension prestressing steel		m	0.000	0.000	0.000	0.000	0.000
dps	Dis. From comp. fiber to centroid of tension prestressing steel		m	0.500	0.500	1.500	1.500	1.500
b	Width of the compression face of member		m	12.600	12.600	12.600	12.600	12.600
bw	Web width or diameter of a circular section		m	12.600	12.600	12.600	12.600	12.600
hf	Compression flange depth		m	0.000	0.000	0.000	0.000	0.000
Iz	Moment of inertia of section		m4	0.131	0.131	3.544	3.544	3.544
Amc	Section area		m2	6.300	6.300	18.900	18.900	18.900
	Steel choice							
Aps	Tension prestressing steel	P.S type		0	0	0	0	0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
A'ps	Compression prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	0 T00.0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
As	Tension Reinforcement	Number	bars	82	82	82	82	82
		Diameter	mm	16	16	22	22	22
		Area	m2	0.01557	0.01557	0.02928	0.02928	0.02928
A's	Compression Reinforcement	Number	bars	82	82	82	82	82
		Diameter	mm	16	16	16	16	16
		Area	m2	0.01656	0.01656	0.01656	0.01656	0.01656
A'c	Shear reinforcement	Number	bars	20	20	19	19	19
		Diameter	mm	14	14	14	14	14
		Area	m2	0.00302	0.00302	0.00287	0.00287	0.00287
ϕ	Resistance factors for flexure		5.5.4.2	0.90	1.00	1.00	0.90	1.00
ϕv	Resistance factors for shear			0.90	1.00	1.00	0.90	1.00
ϕn	Resistance factors for axial force			1.00	1.00	1.00	1.00	1.00
β1	Stress block factor			0.836	0.836	0.836	0.836	0.836
c	Dis. Between centroid and top fiber		m	-0.001	-0.001	0.019	0.019	0.019
	For T section behavior		m	-0.001	-0.001	0.019	0.019	0.019
	For rectangular section behavior		m	-0.001	-0.001	0.019	0.019	0.019
fpe	Effective stress in the prestressing steel after losses		Mpa	1116	1116	1116	1116	1116
fps	Aver. stress in pres. steel at the time for which the nominal resistance		Mpa	1862	1862	1853	1853	1853
k	Factor depends on type of P.S, Low relaxation strand k = 0.28			0.28	0.28	0.28	0.28	0.28

REINFORCEMENT CHECKING - HEAD AND STEM WALL

a	Depth of equivalent stress block	m	-0.001	-0.001	0.016	0.016	0.016
de	Corresp. effective depth from extreme comp. fiber						
	to centroid of tensile force in the tensile reinf.	m	0.442	0.442	1.439	1.439	1.439
Mn	Nominal resistance	kNm	2367	2367	16429	16429	16429
Mr	Factored reistance	kNm	**2131**	**2367**	**16429**	**14787**	**16429**
Mu	Flexual moment	kNm	841	505	8367	12124	9733
(5.7.3.2)	**Flexural moment Checking**		**OK**	**OK**	**OK**	**OK**	**OK**
	Limits for reinforcement						
c/de	Maximum reinforcement		0.00	0.00	0.01	0.01	0.01
	Maximum reinforcement Checking	<= 0.42	**OK**	**OK**	**OK**	**OK**	**OK**
1.2*Mcr	Craking moment	kNm	1084	1084	9908	9908	9908
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**		**OK**	**OK**	**OK**	**OK**	**OK**
(5.7.3.4)	**Conctrol of craking by distr. of reinf for RC member- Check?**		No	Yes	Yes	No	No
	Existing condition for structrure	1,2 or 3	1	1	1	1	1
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.058	0.058	0.061	0.061	0.061
Z	Crack width parameter	N/mm	30000	30000	30000	30000	30000
A	Area of concr. with same centroid as tens. Reinf	m2	0.018	0.018	0.019	0.019	0.019
fsa	Value	Mpa	297	297	287	287	287
0.6*fy		Mpa	240	240	240	240	240
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	240	240	240	240	240
x	Dist. From compression fiber to centroid	m	-	0.079	0.201	-	-
J.d	Arm	m	-	0.416	1.372	-	-
Icr	Moment of inertia of the cracked section	m4	-	0.016	0.35	-	-
fs	Tensile stress in reinforcement fs = Msls / (As*J.d)	Mpa	-	78	208	-	-
	Checking for control cracking fs<fsa		**N.a**	**OK**	**OK**	**N.a**	**N.a**
(5.10.8.2)	**Shrinkage and temperature Reinforcement (side distribution)**						
Areq	Area of required reinf	m2	0.00045	0.00045	0.00126	0.00126	0.00126
	Distribution on sides 7 D16	m2	0.00141	0.00141	0.00141	0.00141	
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45	0.45
	Checking		**OK**	**OK**	**OK**	**OK**	**OK**

SHEAR AND TORSION CHECKING

β	Factor indicating diag. cracked concr. to tension		2.4	2.7	2.4	2.3	2.3
θ	Angle of inclination of diagonal compressive	degree	32.87	28.89	30.80	35.03	33.57
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	90
bv	Effective web width as minimum web width - in dv	m	12.600	12.600	12.600	12.600	12.600
dv	Effective shear depth	m	0.443	0.443	1.431	1.431	1.431
	(de - a/2)	m	0.443	0.443	1.431	1.431	1.431
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	0.600
ncat	Amount of bars in spacing S	bars	20	20	19	19	19
Av	Shear reinf area in spacing S	m2	0.0030	0.0030	0.0029	0.0029	0.0029
θ	Assume	degree	31.64	28.80	33.96	37.37	35.68
v	Shear stress in concrete	kN/m2	164	53	136	224	200
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e_x	Strain in tensile reinforcement		7.42E-04	4.46E-04	6.13E-04	9.19E-04	7.97E-04
	if ex<0, multiple with reduce factor		-	-	-	-	-
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok	Ok
v/f'c	Ratio of shear stress and f'c		0.005	0.002	0.005	0.007	0.007
β	Final value		2.4	2.7	2.4	2.3	2.3
θ	Final value	degree	32.87	28.89	30.80	35.03	33.57
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	6021	6897	20058	18652	19210
Vs	Shear resistance provided by shear reinforcement	kN	1379	1615	4591	3905	4125
Vp	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0	0
Vn1	Vn1=Vc+Vs+Vp	kN	7400	8512	24649	22557	23335
Vn2	Vn2	kN	41828	41828	135237	135237	135237
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	7400	8512	24649	22557	23335
Vr	Factored shear resistance	kN	**6660**	**8512**	**24649**	**20301**	**23335**
Vu	Shear	kN	824	502	2452	3636	3605
(5.8.2.7)	**Shear checking**		**OK**	**OK**	**OK**	**OK**	**OK**
	Region requiring transverse reinf Checking		No need	No need	No need	No need	No need
	Minimum shear reinf area	m2	0.0086	0.0086	0.0086	0.0086	0.0086
	Mimimum shear reinforcement Checking		-	-	-	-	-
	0.1*f'c*bv*dv	kN	16731	16731	54095	54095	54095
	Smax	m	0.35	0.35	0.60	0.60	0.60
	Maximum spacing Smax		-	-	-	-	-

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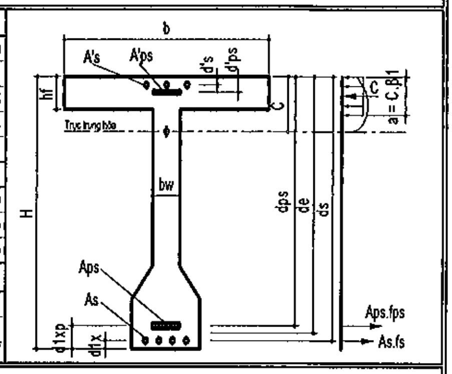
REINFORCEMENT CHECKING - PILECAP SECTION

MATERIALS

	NORMAL CONCRETE		
f'c	Compressive Strength of concrete at 28 days	Mpa	30
Ec	Modulus of Elasticity	Mpa	27691
fr	Modulus of Rupture	Mpa	3.5
gc	Unit weight of concrete	kN/m3	24.5
	PRESTRESSING STEEL		
fpu	Tensile strength of prestressing steel	Mpa	1860
fpy	Yield strength of prestressing steel	Mpa	1670
Ep	Modulus of Elasticity	Mpa	195000
	REINFORCEMENT		
fy	Yield strength	Mpa	400
Es	Modulus of Elasticity	Mpa	200000
nc	Ratio Es/Ec		7



Sign	Parameters		Unit	Sections				
				3-3	3-3	3-3	4-4	4-4
	INTERNAL FORCES AT SECTIOIN							
	Combination			Service	Strength	Extreme	Extreme	Strength
Qu	Shear		kN	11514	16167	11923	1295	3883
Mu	Flexural Moment		kNm	17135	24454	18216	3873	8835
Nu	Axial load		kN	0	0	0	0	0
Tu	Torsional Moment		kNm	0	0	0	0	0
	FLEXURAL MOMENT CHECKING							
H	Section height		m	2.000	2.000	2.000	2.000	2.000
d's	Dis. From comp. fiber to centroid of comp. Reinf		m	0.083	0.083	0.083	0.083	0.083
d1x	Dis. From tens. fiber to centroid of tension Reinf		m	0.166	0.166	0.166	0.160	0.160
	Cover to reinf		m	0.075	0.075	0.075	0.075	0.075
ds	Dis. From comp. fiber to centroid of tension Reinf		m	1.834	1.834	1.834	1.840	1.840
d'ps	Dis. From comp. fiber to centroid of comp. prestressing steel		m	0.000	0.000	0.000	0.000	0.000
d1xp	Dis. From tens. fiber to centroid of tension prestressing steel		m	0.000	0.000	0.000	0.000	0.000
dps	Dis. From comp. fiber to centroid of tension prestressing steel		m	2.000	2.000	2.000	2.000	2.000
b	Width of the compression face of member		m	12.600	12.600	12.600	12.600	12.600
bw	Web width or diameter of a circular section		m	12.600	12.600	12.600	12.600	12.600
hf	Compression flange depth		m	0.000	0.000	0.000	0.000	0.000
Iz	Moment of inertia of section		m4	8.400	8.400	8.400	8.400	8.400
Amc	Section area		m2	25.200	25.200	25.200	25.200	25.200
	Steel choice							
Aps	Tension prestressing steel	P.S type		0	0	0	0	0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
A'ps	Compression prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	0 T00.0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
As	Tension Reinforcement	Number	bars	84	84	84	84	84
		Diameter	mm	32	32	32	20	20
		Area	m2	0.06323	0.06323	0.06323	0.02479	0.02479
A's	Compression Reinforcement	Number	bars	0	0	0	0	0
		Diameter	mm	16	16	16	16	16
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
A'c	Shear reinforcement	Number	bars	15	15	13	13	13
		Diameter	mm	16	16	16	16	16
		Area	m2	0.00303	0.00303	0.00263	0.00263	0.00263
ϕ	Resistance factors for flexure		5.5.4.2	1.00	0.90	1.00	1.00	0.90
ϕv	Resistance factors for shear			1.00	0.90	1.00	1.00	0.90
ϕn	Resistance factors for axial force			1.00	1.00	1.00	1.00	1.00
$\beta 1$	Stress block factor			0.836	0.836	0.836	0.836	0.836
c	Dis. Between centroid and top fiber		m	0.094	0.094	0.094	0.037	0.037
	For T section behavior		m	0.094	0.094	0.094	0.037	0.037
	For rectangular section behavior		m	0.094	0.094	0.094	0.037	0.037
fpe	Effective stress in the prestressing steel after losses		Mpa	1116	1116	1116	1116	1116
fps	Aver. stress in pres. steel at the time for which the nominal resistanc		Mpa	1835	1835	1835	1850	1850
k	Factor depends on type of P.S, Low relaxation strand k = 0.28			0.28	0.28	0.28	0.28	0.28

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			Revise			

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REINFORCEMENT CHECKING - PILECAP SECTION

a	Depth of equivalent stress block	m	0.079	0.079	0.079	0.031	0.031
de	Corresp. effective depth from extreme comp. fiber						
	to centroid of tensile force in the tensile reinf.	m	1.834	1.834	1.834	1.840	1.840
Mn	Nominal resistance	kNm	45387	45387	45387	18089	18089
Mr	Factored resistance	kNm	**45387**	**40849**	**45387**	**18089**	**16280**
Mu	Flexual moment	kNm	17135	24454	18216	3873	8835
(5.7.3.2)	**Flexural moment Checking**		**OK**	**OK**	**OK**	**OK**	**OK**
	Limits for reinforcement						
c/de	Maximum reinforcement		0.05	0.05	0.05	0.02	0.02
	Maximum reinforcement Checking	<= 0.42	**OK**	**OK**	**OK**	**OK**	**OK**
1.2*Mcr	Craking moment	kNm	18251	18251	18251	17718	17718
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**		**OK**	**OK**	**OK**	**OK**	**OK**
(5.7.3.4)	**Control of craking by distr. of reinf for RC member- Check?**		Yes	No	No	No	No
	Existing condition for structrure	1,2 or 3	3	3	3	3	3
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.050	0.050	0.050	0.050	0.050
Z	Crack width parameter	N/mm	17500	17500	17500	17500	17500
A	Area of concr. with same centroid as tens. Reinf	m2	0.015	0.015	0.015	0.015	0.015
fsa	Value	Mpa	193	193	193	193	193
0.6*fy		Mpa	240	240	240	240	240
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	193	193	193	193	193
x	Dist. From compression fiber to centroid	m	0.326	-	-	-	-
J.d	Arm	m	1.725	-	-	-	-
Icr	Moment of inertia of the cracked section	m4	1.152	-	-	-	-
fs	Tensile stress in reinforcement fs = Msls / (As*J.d)	Mpa	157	-	-	-	-
	Checking for control cracking fs<fsa		**OK**	**N.a**	**N.a**	**N.a**	**N.a**
(5.10.8.2)	**Shrinkage and temperature Reinfocement (side distribution)**						
Areq	Area of required reinf	m2	0.00127	0.00127	0.00127	0.00127	0.00127
	Distribution on sides 7 D16	m2	0.00141	0.00141	0.00141	0.00141	0.00141
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45	0.45
	Checking		**OK**	**OK**	**OK**	**OK**	**OK**

SHEAR AND TORSION CHECKING

β	Factor indicating diag. cracked concr. to tension		2.1	1.8	2.0	2.4	2.0
θ	Angle of inclination of diagonal compressive	degree	39.17	42.16	39.74	30.74	40.14
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	90
bv	Effective web width as minimum web width - in dv	m	12.600	12.600	12.600	12.600	12.600
dv	Effective shear depth	m	1.795	1.795	1.795	1.825	1.825
	(de - a/2)	m	1.795	1.795	1.795	1.825	1.825
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	0.600
ncat	Amount of bars in spacing S	bars	15	15	13	13	13
Av	Shear reinf area in spacing S	m2	0.0030	0.0030	0.0026	0.0026	0.0026
θ	Assume	degree	39.00	41.92	39.52	35.89	41.85
v	Shear stress in concrete	kN/m2	509	794	527	33	188
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e_x	Strain in tensile reinforcement		1.32E-03	1.79E-03	1.37E-03	6.09E-04	1.41E-03
	if ex<0, multiple with reduce factor		-	-	-	-	-
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok	Ok
v/f'c	Ratio of shear stress and f'c		0.017	0.026	0.018	0.001	0.006
β	Final value		2.1	1.8	2.0	2.4	2.0
θ	Final value	degree	39.17	42.16	39.74	30.74	40.14
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	21098	18677	20770	25596	20883
Vs	Shear resistance provided by shear reinforcement	kN	4449	4004	3779	5371	3788
Vp	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0	0
Vn1	Vn1=Vc+Vs+Vp	kN	25547	22681	24549	30968	24671
Vn2	Vn2	kN	169594	169594	169594	172422	172422
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	25547	22681	24549	30968	24671
Vr	Factored shear resistance	kN	**25547**	**20413**	**24549**	**30968**	**22204**
Vu	Shear	kN	11514	16167	11923	1295	3883
(5.8.2.7)	**Shear checking**		**OK**	**OK**	**OK**	**OK**	**OK**

	Da Nang Quang Ngai Expressway project LRB09 BRIDGE	Item.	Eng.	Date.	Sign.
		Design			
	DETAIL DESIGN	Check			
	ABUTMENT A2	Revise			

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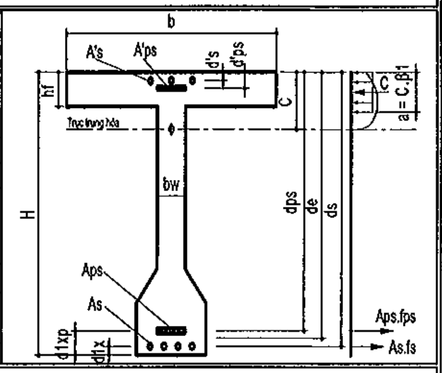
REINFORCEMENT CHECKING - WING WALL

MATERIALS

	NORMAL CONCRETE		
f'c	Compressive Strength of concrete at 28 days	Mpa	30
Ec	Modulus of Elasticity	Mpa	27691
fr	Modulus of Rupture	Mpa	3.5
gc	Unit weight of concrete	kN/m3	24.5
	PRESTRESSING STEEL		
fpu	Tensile strength of prestressing steel	Mpa	1860
fpy	Yield strength of prestressing steel	Mpa	1670
Ep	Modulus of Elasticity	Mpa	195000
	REINFORCEMENT		
fy	Yield strength	Mpa	400
Es	Modulus of Elasticity	Mpa	200000
nc	Ratio Es/Ec		7



Sign	Parameters		Unit	Sections				
				5-5	5-5	6-6	6-6	
	INTERNAL FORCES AT SECTIOIN							
	Combination			Service	Strength	Service	Strength	
Qu	Shear		kN	143	285	71	112	
Mu	Flexural Moment		kNm	135	202	137	216	
Nu	Axial load		kN	0	0	0	0	
Tu	Torsional Moment		kNm	0	0	0	0	
	FLEXURAL MOMENT CHECKING							
H	Section height		m	0.500	0.500	0.500	0.500	
d's	Dis. From comp. fiber to centroid of comp. Reinf		m	0.058	0.058	0.058	0.058	
d1x	Dis. From tens. fiber to centroid of tension Reinf		m	0.060	0.060	0.060	0.060	
	Cover to reinf		m	0.050	0.050	0.050	0.050	
ds	Dis. From comp. fiber to centroid of tension Reinf		m	0.440	0.440	0.440	0.440	
d'ps	Dis. From comp. fiber to centroid of comp. prestressing steel		m	0.000	0.000	0.000	0.000	
d1xp	Dis. From tens. fiber to centroid of tension prestressing steel		m	0.000	0.000	0.000	0.000	
dps	Dis. From comp. fiber to centroid of tension prestressing steel		m	0.500	0.500	0.500	0.500	
b	Width of the compression face of member		m	1.000	1.000	1.000	1.000	
bw	Web width or diameter of a circular section		m	1.000	1.000	1.000	1.000	
hf	Compression flange depth		m	0.000	0.000	0.000	0.000	
Iz	Moment of inertia of section		m4	0.010	0.010	0.010	0.010	
Amc	Section area		m2	0.500	0.500	0.500	0.500	
	Steel choice							
Aps	Tension prestressing steel	P.S type		0	0	0	0	
		Number	tendons	0	0	0	0	
		Area	m2	0.00000	0.00000	0.00000	0.00000	
A'ps	Compression prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	
		Number	tendons	0	0	0	0	
		Area	m2	0.00000	0.00000	0.00000	0.00000	
As	Tension Reinforcement	Number	bars	7	7	7	7	
		Diameter	mm	20	20	20	20	
		Area	m2	0.00220	0.00220	0.00220	0.00220	
A's	Compression Reinforcement	Number	bars	7	7	7	7	
		Diameter	mm	16	16	16	16	
		Area	m2	0.00141	0.00141	0.00141	0.00141	
A'c	Shear reinforcement	Number	bars	2	2	2	2	
		Diameter	mm	12	12	12	12	
		Area	m2	0.00023	0.00023	0.00023	0.00023	
ϕ	Resistance factors for flexure		5.5.4.2	1.00	0.90	1.00	0.90	
ϕv	Resistance factors for shear			1.00	0.90	1.00	0.90	
ϕn	Resistance factors for axial force			1.00	1.00	1.00	1.00	
β1	Stress block factor			0.836	0.836	0.836	0.836	
c	Dis. Between centroid and top fiber		m	0.015	0.015	0.015	0.015	
	For T section behavior		m	0.015	0.015	0.015	0.015	
	For rectangular section behavior		m	0.015	0.015	0.015	0.015	
fpe	Effective stress in the prestressing steel after losses		Mpa	1116	1116	1116	1116	
fps	Aver. stress in pres. steel at the time for which the nominal resistance		Mpa	1844	1844	1844	1844	
k	Factor depends on type of P.S, Low relaxation strand k = 0.28			0.28	0.28	0.28	0.28	

REINFORCEMENT CHECKING - WING WALL

a	Depth of equivalent stress block	m	0.012	0.012	0.012	0.012	
de	Corresp. effective depth from extreme comp. fiber						
	to centroid of tensile force in the tensile reinf.	m	0.440	0.440	0.440	0.440	
Mn	Nominal resistance	kNm	352	352	352	352	
Mr	Factored reistance	kNm	**352**	**317**	**352**	**317**	
Mu	Flexual moment	kNm	**135**	**202**	**137**	**216**	
(5.7.3.2)	**Flexural moment Checking**		**OK**	**OK**	**OK**	**OK**	
	Limits for reinforcement						
c/de	Maximum reinforcement		0.03	0.03	0.03	0.03	
	Maximum reinforcement Checking	<= 0.42	**OK**	**OK**	**OK**	**OK**	
r min	Mimimum reinforcement		0.44%	0.44%	0.44%	0.44%	
	Mimimum reinforcement Checking for RC	0.23%	**OK**	**OK**	**OK**	**OK**	
1.2*Mcr	Craking moment	kNm	89	89	89	89	
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**		**OK**	**OK**	**OK**	**OK**	
(5.7.3.4)	**Conctrol of craking by distr. of reinf for RC member- Check?**		Yes	No	Yes	No	
	Existing condition for structrure	1,2 or 3	1	1	1	1	
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.060	0.060	0.060	0.060	
Z	Crack width parameter	N/mm	30000	30000	30000	30000	
A	Area of concr. with same centroid as tens. Reinf	m2	0.017	0.017	0.017	0.017	
fsa	Value	Mpa	297	297	297	297	
0.6*fy		Mpa	240	240	240	240	
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	240	240	240	240	
x	Dist. From compression fiber to centroid	m	0.102	-	0.102	-	
J.d	Arm	m	0.406	-	0.406	-	
Icr	Moment of inertia of the cracked section	m4	0.002	-	0.002	-	
fs	Tensile stress in reinforcement fs = Msls / (As*J.d)	Mpa	151	-	154	-	
	Checking for control cracking fs<fsa		**OK**	**N.a**	**OK**	**N.a**	
(5.10.8.2)	**Shrinkage and temperature Reinfocement (side distribution)**						
Areq	Area of required reinf	m2	0.00031	0.00031	0.00031	0.00031	
	Distribution on sides 7 D16	m2	0.00141	0.00141	0.00141	0.00141	
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45	
	Checking		**OK**	**OK**	**OK**	**OK**	

SHEAR AND TORSION CHECKING

β	Factor indicating diag. cracked concr. to tension		2.3	2.0	2.3	2.1	
θ	Angle of inclination of diagonal compressive	degree	35.33	40.60	33.96	38.78	
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	
bv	Effective web width as minimum web width - in dv	m	1.000	1.000	1.000	1.000	
dv	Effective shear depth	m	0.434	0.434	0.434	0.434	
	(de - a/2)	m	0.434	0.434	0.434	0.434	
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	
ncat	Amount of bars in spacing S	bars	2	2	2	2	
Av	Shear reinf area in spacing S	m2	0.0002	0.0002	0.0002	0.0002	
β	Assume		2.0	2.2	2.0	2.0	
θ	Assume	degree	34.57	38.94	36.11	41.24	
v	Shear stress in concrete	kN/m2	330	730	164	288	
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	
e_x	Strain in tensile reinforcement		9.44E-04	1.46E-03	8.30E-04	1.28E-03	
	if ex<0, multiple with reduce factor		-	-	-	-	
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok	
v/f'c	Ratio of shear stress and f'c		0.011	0.024	0.005	0.010	
β	Final value		2.3	2.0	2.3	2.1	
θ	Final value	degree	35.33	40.60	33.96	38.78	
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	446	389	459	409	
Vs	Shear resistance provided by shear reinforcement	kN	92	76	97	81	
Vp	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0	
Vn1	Vn1=Vc+Vs+Vp	kN	538	465	556	491	
Vn2	Vn2	kN	3254	3254	3254	3254	
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	538	465	556	491	
Vr	Factored shear resistance	kN	**538**	**419**	**556**	**441**	
Vu	Shear	kN	143	285	71	112	
(5.8.2.7)	**Shear checking**		**OK**	**OK**	**OK**	**OK**	

BORED PILE DESIGN

I. BORED PILE DATA

1.Load Combinations at top of bored pile

STT	Comb	Axial force P (KN)	Moment (KN.m)		
			Mx	My	Mxy
STRENGTH LIMIT STATES					
1	P_min	950	45.9	989.5	991
2	P_max	3979	52.3	1765.8	1767
3	Mx_max	3979	52.3	1765.8	1767
4	My_max	3979	52.3	1765.8	1767
EXTREME EVENT LIMIT STATES					
1	P_min	831	79	1118	1121
2	P_max	3119	8	1342	1342
3	Mx_max	3119	8	1342	1342
4	My_max	3119	8	1342	1342

2. Bored pile Material

Normal concrete			
Compressive strength at 28 days age	f'c	30	MPa
Concrete elastic modulus	Ec	27691	MPa
Reinforcement TCVN1651-2008; CBV-400			
Yield strength	fy	400	MPa
Reinforcement elastic modulus	Es	200,000	MPa

3. Bored pile Section

Pile diameter	D	1.00	m
Section area	A	0.785	m2
Moment inertia	Ix	0.049	m4
	Iy	0.049	m4
Radius of gyration of gross concrete section; r = sqrt(I/A)	rx	0.250	m
	ry	0.250	m

II. PILE DESIGN

1. Limit of Reinforcement

S.5.7.4.2

Minimum area of longitudinal reinforcement in column			
As.fy / (Ag . f'c) >= 0.135	As ≥	0.008	m2
As / Ag >= 0.01	As ≥	0.008	m2
Maximum area of longitudinal reinforcement in column			
As / Ag <= 0.08	As ≤	0.063	m2
Trial Rebars: Ok	As	**0.019**	m2
1layers x 24 = 24 bars D32 @150	As1	0.019	m2

2. Interation diagram M-P

Using Pca-Column software

****Flexural check by pcaColumn**

Strength and Service limit states:

Resistance factor: Compression φ_c = 0.75 *(AASHTO LRFD-2004)*

Tension φ_t = 0.90

Extreme Event limit states:

Resistance factor Compression φ_c = 1.00

Tension φ_t = 1.00

<Result table>

STT	Comb	Pu kN	Mux kN-m	Muy kN-m	φ Mnx kN-m	φ Mny kN-m	φ Mn/Mu
STRENGTH LIMIT STATES							
1	P_max	949.5	45.9	989.5	117.2	2527.1	2.554
2	P_min	3978.7	52.3	1765.8	79.8	2693.6	1.525
3	Mx_max	3978.7	52.3	1765.8	79.8	2693.6	1.525
4	My_max	3978.7	52.3	1765.8	79.8	2693.6	1.525
EXTREME EVENT LIMIT STATES							
1	P_max	831	79	1118	199.2	2819.2	2.522
2	P_min	3119	8	1342	19.5	3275.7	2.441
3	Mx_max	3119	8	1342	19.5	3275.7	2.441
4	My_max	3119	8	1342	19.5	3275.7	2.441

3. Column Ties

S.5.7.4.6, S.5.10.6.3, S5.10.11.4.1d - e

Bridge is in seismic zone	Sz	1	
Area of concrete core measured out-to-out of ties	Ac	0.622	m2
Tie diameter	Dtie	16	mm2
Cross section area of 1 tie	As-tr	0.00020	m2
Spacing of hoops	s	75	mm
Length of reinfocement tie in 1 hoop	Ltie	2.78	m
Ratio of ties reinf. in one hoop/ volume of conc. core one pitch spacing			
ρs = As-tr . Ltie / (Ac * spacing)	ρs	0.0120	
Ratio of spiral reinf. To total volume of concrete core shall satisfy			S.5.7.4.6
ρs >= 0.45 . (Ag/Ac - 1). f'c/ fy = Req1	Req1	0.0089	**OK**
Transverse Reinforcement for Confinement at Plastic Hinges		S.5.10.11.4.1.d	
For a circular column "1:applied","2:Not applied"		1	
ρs >= 0.12 . f'c/ fy = Req2	Req2	0.0090	**N/A**

4. Shear Design

Shear resistance factors	ϕv	1.0	
Factored shear force	Vu	698	kN
Required shear capacity Vn = Vu/ ϕv	Vn	698	kN
Determine concrete shear capacity			
Minimum shear reinforcement will provided in cross section			
Therefore	β	2.0	
	θ	45.0	deg
Diameter of bored pile	D	1.00	m
Width of cross section	b	1.00	m
dv = 0.9*de de = D/2 + Dr/pi()			
Diameter of the circle passing through the centers of the long. reinf.	Dr	0.79	m
	de	0.75	m
	dv	0.68	m
Vc = 0.083 . β . sqrt(f'c) . bv. dv	Vc	616	kN
Diameter of Transverse Reinforcement	D_v	13	mm
Spacing of Transverse Reinforcement	s	75	mm
Area of a transverse reinforcement within distance "s"	A_v	339.30	mm2
Angle of inclination of transverse reinforcement to longitudinal axis	α	90	deg.
Effetive shear depth, d_v			
Alternative 1: $d_{v1} = M_n/(A_s{*}f_y)$			
Normal flexural resistance	M_n	3276	KNm
	d_{v1}	**213**	mm
Alternative 2: $d_{v2} = 0.9d_e$ $d_e = D/2 + D_r/\pi$		752	mm
	d_{v1}	677	mm
Choice value of d $\boxed{1}$ *("1" = d_{v1}, "2" = d_{v2})*	d_v	**213**	mm
Norminal shear resistance of Reinforcement $V_s = \dfrac{A_v f_y d_v(\cot g\theta + \cot g\alpha)\sin\alpha}{s}$		**385**	kN
$V_{n1} = V_c + V_s$		**1001**	kN
$V_{n2} = 0.25f'_c b_v d_v$		**1597**	kN
V_n		**1001**	kN
Conclude			**OK**

SPACE PILE FOUNDATION ANALYSIS PROGRM
Turbo BASIC

PROJECT: : DN-QN-A2-LRB09

INITIA DATA

Kn = 0.18 Ax = 6.00 By = 12.60 Cz = 2.00
E v.uon = 3001028 E r.uon = 3001028 E v.nen = 3001028 E r.nen = 3001028
Mq = 75 (t/m4) Md = 0 (t/m4) m = 600 (t/m4)

LOAD COMBINATIONS

COMB.	Hx	Hy	P	Mx	My	Mz
1	562.08	0.00	2338.32	-158.60	1463.06	0.00
2	376.97	0.00	1721.61	-145.62	1059.91	0.00
3	543.62	8.12	2269.23	-204.45	1369.14	0.00
4	358.51	8.12	1652.52	-191.47	965.99	0.00
5	377.43	6.72	1758.97	-163.06	964.69	0.00
6	400.06	15.78	2122.41	-144.25	483.90	0.00
7	400.06	15.78	1505.71	-131.28	743.29	0.00

PROPERTIES OF PILES

PILE	Lo	H	Bpx	Bpy	A	B	Cday	Fo	Io	Po	Co	Ct
1	0.00	8.00	1.204	1.204	1.00	0.000	0.000	0.785	0.098	500	30000	15000
2					n	t						
3					n	t						
4					n	t						
5					n	t						
6					n	t						
7					n	t						
8					n	t						

PILE COORD.

PILE	X	Y	Phi	Xi
1	2.00	4.80	0.000	0.00
2	2.00	2.31	0.000	0.00
3	2.00	-0.25	0.000	0.00
4	2.00	-2.81	0.000	0.00
5	2.00	-5.30	0.000	0.00
6	-2.00	-5.30	0.000	0.00
7	-2.00	-0.25	0.000	0.00
8	-2.00	4.80	0.000	0.00

DISPLACEMENTS

COMB.	X	Y	Z	Fix	Fiy	Fiz
1	0.01291	0.00025	0.002890	-0.000068	0.000588	0.000132
2	0.00866	0.00019	0.002147	-0.000053	0.000395	0.000089

3	0.01245	0.00043	0.002815	-0.000073	0.000552	0.000132
4	0.00820	0.00037	0.002073	-0.000058	0.000359	0.000088
5	0.00857	0.00034	0.002217	-0.000057	0.000358	0.000092
6	0.00851	0.00055	0.002812	-0.000066	0.000158	0.000102
7	0.00900	0.00051	0.001879	-0.000051	0.000346	0.000102

FORCES ON PILES

PILE	COMB.	N	Q2	Q3	M1	M2	M3
1	1	405.58	-71.11	1.22	1.970	5.331	179.995
	2	294.60	-47.69	0.82	1.321	3.788	120.696
	3	394.15	-68.90	0.22	1.964	2.702	174.920
	4	283.18	-45.48	-0.18	1.315	1.158	115.621
	5	296.03	-47.88	0.03	1.372	1.713	122.286
	6	317.97	-51.05	-0.99	1.522	-0.851	136.818
	7	259.74	-50.90	-1.00	1.518	-1.315	130.992
2	1	389.82	-69.13	1.22	1.970	5.331	174.531
	2	282.38	-46.37	0.82	1.321	3.788	117.032
	3	377.32	-66.94	0.22	1.964	2.702	169.472
	4	269.89	-44.17	-0.18	1.315	1.158	111.974
	5	282.86	-46.50	0.03	1.372	1.713	118.481
	6	302.75	-49.53	-0.99	1.522	-0.851	132.598
	7	248.07	-49.38	-1.00	1.518	-1.315	126.783
3	1	373.71	-67.12	1.22	1.970	5.331	168.946
	2	269.90	-45.01	0.82	1.321	3.788	113.287
	3	360.12	-64.92	0.22	1.964	2.702	163.905
	4	256.31	-42.82	-0.18	1.315	1.158	108.246
	5	269.41	-45.10	0.03	1.372	1.713	114.592
	6	287.20	-47.97	-0.99	1.522	-0.851	128.285
	7	236.14	-47.83	-1.00	1.518	-1.315	122.482
4	1	357.61	-65.10	1.22	1.970	5.331	163.362
	2	257.42	-43.66	0.82	1.321	3.788	109.542
	3	342.92	-62.91	0.22	1.964	2.702	158.338
	4	242.73	-41.47	-0.18	1.315	1.158	104.518
	5	255.95	-43.69	0.03	1.372	1.713	110.703
	6	271.65	-46.42	-0.99	1.522	-0.851	123.972
	7	224.21	-46.27	-1.00	1.518	-1.315	118.180
5	1	341.85	-63.13	1.22	1.970	5.331	157.898
	2	245.20	-42.34	0.82	1.321	3.788	105.878
	3	326.09	-60.95	0.22	1.964	2.702	152.890
	4	229.45	-40.16	-0.18	1.315	1.158	100.871
	5	242.78	-42.32	0.03	1.372	1.713	106.898
	6	256.44	-44.89	-0.99	1.522	-0.851	119.752
	7	212.54	-44.75	-1.00	1.518	-1.315	113.971
6	1	124.72	-63.13	-1.94	1.970	-3.411	157.898
	2	99.34	-42.34	-1.30	1.321	-2.075	105.878
	3	122.17	-60.95	-2.93	1.964	-6.014	152.890
	4	96.79	-40.16	-2.29	1.315	-4.678	100.871
	5	110.69	-42.32	-2.17	1.372	-4.375	106.898
	6	198.03	-44.89	-3.43	1.522	-7.603	119.752
	7	84.74	-44.75	-3.43	1.518	-8.049	113.971
7	1	156.58	-67.12	-1.94	1.970	-3.411	168.946
	2	124.04	-45.01	-1.30	1.321	-2.075	113.287
	3	156.20	-64.92	-2.93	1.964	-6.014	163.905
	4	123.66	-42.82	-2.29	1.315	-4.678	108.246
	5	137.31	-45.10	-2.17	1.372	-4.375	114.592
	6	228.80	-47.97	-3.43	1.522	-7.603	128.285
	7	108.34	-47.83	-3.43	1.518	-8.049	122.482
8	1	188.45	-71.11	-1.94	1.970	-3.411	179.995
	2	148.73	-47.69	-1.30	1.321	-2.075	120.696

3	190.23	-68.90	-2.93	1.964	-6.014	174.920
4	150.52	-45.48	-2.29	1.315	-4.678	115.621
5	163.94	-47.88	-2.17	1.372	-4.375	122.286
6	259.56	-51.05	-3.43	1.522	-7.603	136.818
7	131.94	-50.90	-3.43	1.518	-8.049	130.992

SUMMARY OF FORCES

	PILE	COMB.	N	Q2	Q3	M1	M2	M3
Nmin	6	7	84.74	-44.75	-3.43	1.518	-8.049	113.971
Nmax	1	1	405.58	-71.11	1.22	1.970	5.331	179.995
Q2max	1	1	405.58	-71.11	1.22	1.970	5.331	179.995
Q3max	6	7	84.74	-44.75	-3.43	1.518	-8.049	113.971
M1max	1	1	405.58	-71.11	1.22	1.970	5.331	179.995
M2max	6	7	84.74	-44.75	-3.43	1.518	-8.049	113.971
M3max	1	1	405.58	-71.11	1.22	1.970	5.331	179.995

CHECKING CALCULATI
IN COMPARISON WITH INITIA LOAD MATRIX

1	562.08	0.00	2338.32	-158.60	1463.06	0.00
2	376.97	0.00	1721.61	-145.62	1059.91	0.00
3	543.62	8.12	2269.23	-204.45	1369.14	0.00
4	358.51	8.12	1652.52	-191.47	965.99	0.00
5	377.43	6.72	1758.97	-163.06	964.69	0.00
6	400.06	15.78	2122.41	-144.25	483.90	0.00
7	400.06	15.78	1505.71	-131.28	743.29	0.00

Da Nang Quang Ngai Expressway project

Bridge LRB09

CALCULATION SHEETS

Pier P2

2013

Table of content

1.Structure Dimensions & Load Components
2. Load Combinations
3. Pier Cap analysis
4. Pier Column analysis
5. Foundation analysis
6. Annex

A.STRUCTURE DIMENSIONS & LOAD COMPONENTS

I.GENERAL DATA

Assumptions :

1.The Design of the Pier conforms to "Specification for bridge design 22-TCN-272-05" and AASHTO LRFD 2004, JIS,... for reference.
2.Design live load: HL-93 and lane loading 9.3 KN/m
3.Bridge is considered to be in seismic with acceleration coefficient A = 0.0310 g

Input data:

Bridge type	*Simple PC I girder L=30m with link slab*					
Span length	Left	=	33.00	Right	=	33.00 m
Girder length between bearings	Left	=	32.10	Right	=	32.10 m
Bridge width	B	=	12.75 m			

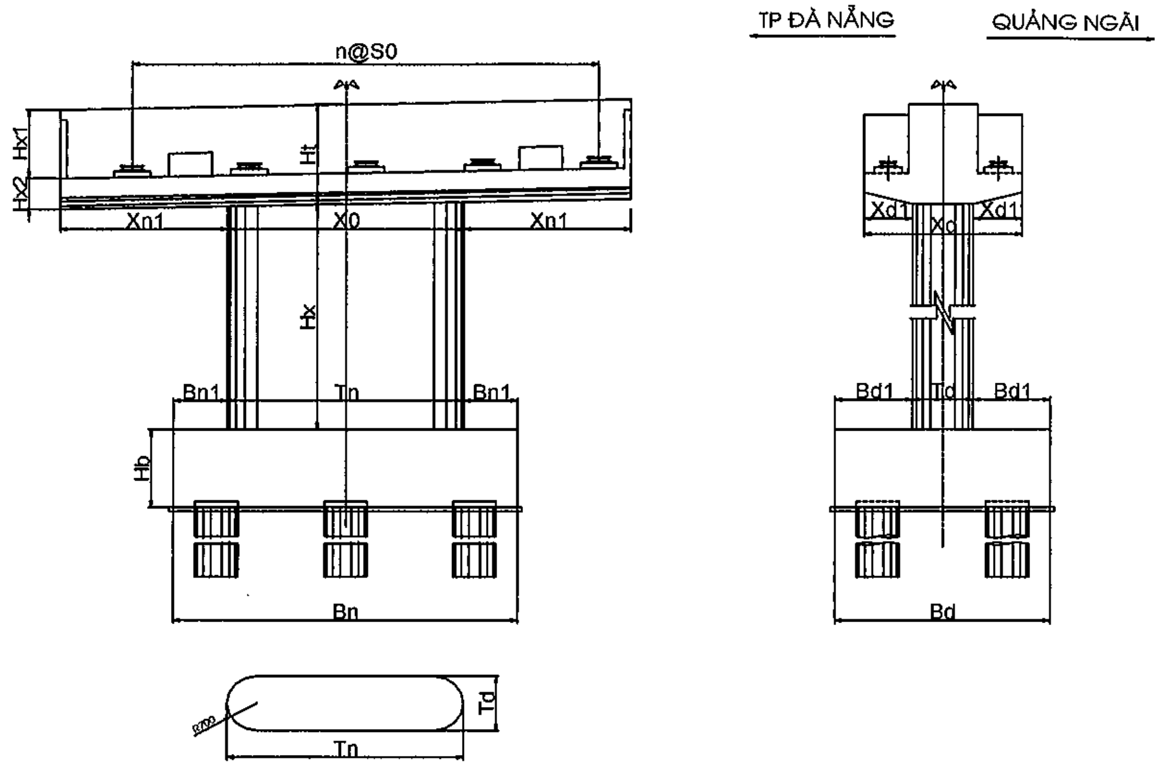
Level Table(at center of pier)

Top of pier cap	ThL	17.173	m
Top of pier column	TcL	13.801	m
Bottom of upper pier column	H_{topc}	13.800	m
Bottom of pier column	BcL	9.500	m
Bottom of upper pilecap	H_{up}	9.500	m
Bottom of pilecap	H_{bot}	7.500	m
Skew angle	Ska	70.000	deg
Ground level	GL	10.500	m
Maximum water level (H1%)	H_{max}	13.010	m
Navigation water level (H5%)	H_{min}	10.500	m
Average Annual water level	H_{ave}	11.755	m
Local scour level (at water level H1%)	H_{sc}	9.500	m

Material unit weight

Concrete

Specify density	γ_c	=	24.5 kN/m3
Compressive Strength (at 28 days)	f'_c	=	30.0 MPa
Elastic Modulus	E_c	=	29440 Mpa
Reinforcement			
Yield strength	f_y	=	400.0 MPa
Modulus of elasticity	E_s	=	200000 Mpa
Modular ratio (steel/concrete)	$n = E_s/E_c$	=	6.7935
Asphalt concrete	γa	=	22.1 kN/m3
Soil - ground	γ_s	=	17.7 kN/m3
Saturated soil	γ_{ss}	=	7.8 kN/m3



Pier Dimensions Table

Notation	Dimensions Transverse direction	Value (m)	Notation	Dimensions Longitudinal direction	Value (m)
*	***Bearing distribution***		*	***Bearing pedestal*** Width	0.85
nbear	Number of bearing	5.00		Length	0.65
nbear	Number of bearing	5.00		Height	0.15
S_l	Bearings spacing	2.55	*	***Anchorage block*** Width	0.40
S_r	Bearings spacing	2.55		Length	1.00
b_{s1}	Total width of bridge CS	12.75		Height	0.52
b_{s2}	Carriage way width	11.76		Dist. CB's edge to exterior girder	1.27
b_{s3}	Left curb width	0.50		Dist. CB's edge to exterior girder	1.27
b_{s4}	Right curb width	0.49			
*	***Pier Cap***				
H_{x1}	Haunch 1 height	1.935	X_d	Pier cap width	3.60
H_{x2}	Haunch 2 height	1.00	X_{d1}	Pier cap width	1.00
H_x	Pier cap height	2.94	GL	Left bearing to pier c.line	1.250
X_{n1}	Haunch width	3.89	GR	Right bearing to pier c.line	1.250
X_{n0}	Bottom of pier cap width	5.50	H_c	Curtain wall height	1.80
X_{nt}	Top of pier cap width	13.28	T_c	Curtain wall thickness	0.15
*	***Pier Column***				
T_n	Pier column width	5.50	T_d	Pier column thichness	1.40
H_{tt}	Pier column height	4.30	Rv	Round nose radius	0.70
T_{nb}	Upper pier column width	0.00	T_{db}	Upper pier column thichness	0.00
H_{tb}	Upper pier column height	0.00	R_{vb}	Upper round nose radius	0.00
H	Column height	4.30			
*	***Pile Cap***				
B_n	Pile cap width	8.00	B_d	Pile cap length	5.00
H_b	Pile cap depth	2.00			
B_{nb}	Upper pile cap width	0.00	B_{db}	Upper pile cap length	0.00
H_{bb}	Upper pile cap depth	0.00			

III. SUBSTRUCTURE LOADS

1. Pier Selfweight

Item	Volume	Vertical	Longitudinal			Transversal		
		F_V	F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m3)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Bearing pedestal	0.83	20.3						
Bearing devices		50.0						
Anchorage block	0.84	20.5						
Pier Cap	88.92	2178.6						
Curtain wall	0.50	12.1						
Upper pier column	0.00	0.0						
Pier Column	31.30	766.9						
Upper pilecap	0.00	0.0						
PileCap	80.00	1960.0						
Shear key	0.00	0.0						
Total at bottom of Column		***3048.5***						
Total at bottom of pilecap		***5008.5***						

2. Soil on pilecap

Item	Volume	Vertical	Longitudinal			Transversal		
		F_V	F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m3)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Soil on pile cap	32.72	579.2						
Total at bottom of Column								
Total at bottom of pilecap		***579.2***						

3. Buoyancy on pier

Case1: Maximum water level (H1%)

Item	Volume	Vertical	Longitudinal			Transversal		
		F_V	F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m3)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Upper pier column	0.00	0.0						
Pier Column	25.55	-250.7						
Upper pilecap	0.00	0.0						
PileCap	80.00	-784.8						
Shear key	0.00	0.0						
Total at bottom of Column		***-250.7***						
Total at bottom of pilecap		***-1035.5***						

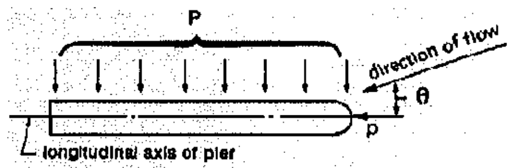
Case2: Minimum water level (Hmin)

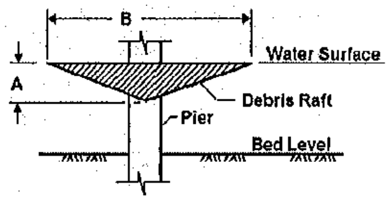
Item	Volume	Vertical	Longitudinal			Transversal		
		F_V	F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m3)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Upper pier column	0.00	0.0						
Pier Column	7.28	-71.4						
Upper pilecap	0.00	0.0						
PileCap	80.00	-784.8						
Shear key	0.00	0.0						
Total at bottom of Column		***-71.4***						
Total at bottom of pilecap		***-856.2***						

Case3: average Annual water level

Item	Volume	Vertical F_V	Longitudinal			Transversal		
			F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m3)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Upper pier column	0.00	0.0						
Pier Column	16.41	-161.0						
Upper pilecap	0.00	0.0						
PileCap	80.00	-784.8						
Shear key	0.00	0.0						
Total at bottom of Column		***-161.0***						
Total at bottom of pilecap		***-945.8***						

4.Stream Pressure





Stream pressure data

Angle between direction of flow and long. axis of pier	θ	0.0	deg
Design velocity of water at H1%	V1%	1.11	m/s
Design velocity of water at minimum water level	Vmin	0.39	m/s
Design velocity of water at average annual water level	Vannual	1.11	m/s

Longitudinal axis of pier $pL = 5.14 \times 10^{-4} C_D V^2$

Type of pier colum: "1:Semicircular - nosed pier"; "2:Square - ended pier";		1	
"3:Debris lodged against the pier "; "4:Wedged - nosed pier with nose angle 90deg or less"			
Drag coefficient	C_D	0.70	
Stream pressure at H1%	pL1%	0.44	kN/m2
Stream pressure at minimum water level	pLmin	0.05	kN/m2
Stream pressure at average annual water level	pLannual	0.44	kN/m2
Additional stream pressure due to driftwood raft lodged against pier			
Drag coefficient	C_D	0.50	
Height of debris raft	A	1.3	m
Width of debris raft	B	14.0	m
Area of debris raft	Area	8.8	m2
Stream pressure due to driftwood raft at H1%	pLdebris	0.32	kN/m2
Equivalent force	Fhdebris	2.8	kN

Lateral axis of pier $pT = 5.14 \times 10^{-4} C_L V^2$

Drag coefficient	C_L	0.00	
Stream pressure at H1%	pT1%	0.00	kN/m2
Stream pressure at minimum water level	pTmin	0.00	kN/m2
Stream pressure at average annual water level	pTannual	0.00	kN/m2

Case1: Maximum water level (H1%)

Item	Exposed height	Vertical F_V	Longitudinal			Transversal		
			F_{HX}	Arm.$_{HX}$	M_y	F_{HY}	Arm.$_{HY}$	M_x
	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Upper pier Column	0.00		0.0	4.3	0.0	0.0	4.3	0.0
Pier Column	3.51		0.0	1.8	0.0	2.2	1.8	3.8
Upper pilecap	0.00		0.0	0.0	0.0	0.0	0.0	0.0
Total at bottom of Column			***0.0***		***0.0***	***2.2***		***3.8***
Upper pier Column	0.00		0.0	6.3	0.0	0.0	6.3	0.0
Pier Column	3.51		0.0	3.8	0.0	2.2	3.8	8.2
Upper pilecap	0.00		0.0	2.0	0.0	0.0	2.0	0.0
PileCap	0.00		0.0	0.0	0.0	0.0	0.0	0.0
Total at bottom of pilecap			***0.0***		***0.0***	***2.2***		***8.2***

Additional stream pressure due to driftwood raft

Item	Exposed height	Vertical F_V	Longitudinal			Transversal		
			F_{HX}	Arm.$_{HX}$	M_y	F_{HY}	Arm.$_{HY}$	M_x
	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Pier Column					0.0	2.8	3.5	9.8
Total at bottom of Column					***0.0***	***2.8***		***9.8***
Pier Column					0.0	2.8	5.5	15.3
Total at bottom of pilecap					***0.0***	***2.8***		***15.3***

Case2: Minimum water level (Hmin)

Item	Exposed height	Vertical F_V	Longitudinal			Transversal		
			F_{HX}	Arm.$_{HX}$	M_y	F_{HY}	Arm.$_{HY}$	M_x
	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Upper pier Column	0.00		0.0	4.3	0.0	0.0	4.3	0.0
Pier Column	1.00		0.0	0.5	0.0	0.1	0.5	0.0
Upper pilecap	0.00		0.0	0.0	0.0	0.0	0.0	0.0
Total at bottom of Column			***0.0***		***0.0***	***0.1***		***0.0***
Upper pier Column	0.00		0.0	6.3	0.0	0.0	6.3	0.0
Pier Column	1.00		0.0	2.5	0.0	0.1	2.5	0.2
Upper pilecap	0.00		0.0	2.0	0.0	0.0	2.0	0.0
PileCap	0.00		0.0	0.0	0.0	0.0	0.0	0.0
Total at bottom of pilecap			***0.0***		***0.0***	***0.1***		***0.2***

Case3: average Annual water level

Item	Exposed height	Vertical F_V	Longitudinal			Transversal		
			F_{HX}	Arm.$_{HX}$	M_y	F_{HY}	Arm.$_{HY}$	M_x
	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Upper pier Column	0.00		0.0	4.3	0.0	0.0	4.3	0.0
Pier Column	2.25		0.0	1.1	0.0	1.4	1.1	1.6
Upper pilecap	0.00		0.0	0.0	0.0	0.0	0.0	0.0
Total at bottom of Column			***0.0***		***0.0***	***1.4***		***1.6***
Upper pier Column	0.00		0.0	6.3	0.0	0.0	6.3	0.0
Pier Column	2.25		0.0	3.1	0.0	1.4	3.1	4.4
Upper pilecap	0.00		0.0	2.0	0.0	0.0	2.0	0.0
PileCap	0.00		0.0	0.0	0.0	0.0	0.0	0.0
Total at bottom of pilecap			***0.0***		***0.0***	***1.4***		***4.4***

5. Wind Loads

Wind loads data $P_D = 0.0006\ V^2\ C_d \geq 1.8$ (kN/m2)

Reference standard "Wind region TCVN 2737-1995"			
Wind region (I, II, III, IV) at bridge location		**III**	
3 second gust wind velocity with 100 years return period	Vb	53.0	m/s
Type of terrain at bridge location		**1**	
"1:exposed area"; "2: forest, houses,... with height 10m"; "3:houses area..with height>10m"			
Average elevation of pier upper ground or water plane level	Hele_p	5.3	m
Correct coefficient for wind zone and elevation of pier	S	1.09	
Design wind speed V = S*Vb	V	57.8	m/s
Obstacle coefficient for pier	Cd	1.3	
Wind pressure on pier	P_D	2.60	kN/m2

At Maximum water level (H1%)

Item	**Exposed height**	**Vertical F_V**	**Longitudinal**			**Transversal**		
			F_{HX}	Arm.$_{HX}$	M_y	F_{HY}	Arm.$_{HY}$	M_x
	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Curtain wall	0.00		0.0	7.7	0.0	0.0	7.7	0.0
Pier Cap	2.94		0.0	5.8	0.0	27.5	5.8	158.7
Upper pier Column	0.00		0.0	3.5	0.0	0.0	3.5	0.0
Pier Column	0.79		11.3	3.9	44.2	2.9	3.9	11.2
Upper pilecap	0.00		0.0	3.5	0.0	0.0	3.5	0.0
Total at bottom of Column			***11.3***		***44.2***	***30.4***		***169.9***
Curtain wall	0.00		0.0	9.7	0.0	0.0	9.7	0.0
Pier Cap	2.94		0.0	7.8	0.0	27.5	7.8	213.7
Upper pier Column	0.00		0.0	5.5	0.0	0.0	5.5	0.0
Pier Column	0.79		11.3	5.9	66.8	2.9	5.9	17.0
Upper pilecap	0.00		0.0	5.5	0.0	0.0	5.5	0.0
PileCap	0.00		0.0	5.5	0.0	0.0	5.5	0.0
Total at bottom of pilecap			***11.3***		***66.8***	***30.4***		***230.7***

At Minimum water level (Hmin)

Item	**Exposed height**	**Vertical F_V**	**Longitudinal**			**Transversal**		
			F_{HX}	Arm.$_{HX}$	M_y	F_{HY}	Arm.$_{HY}$	M_x
	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Curtain wall	0.00		0.0	7.7	0.0	0.0	7.7	0.0
Pier Cap	2.94		0.0	5.8	0.0	27.5	5.8	158.7
Upper pier Column	0.00		0.0	1.0	0.0	0.0	1.0	0.0
Pier Column	3.30		47.2	2.7	125.2	12.0	2.7	31.9
Upper pilecap	0.00		0.0	1.0	0.0	0.0	1.0	0.0
Total at bottom of Column			***47.2***		***125.2***	***39.5***		***190.5***
Curtain wall	0.00		0.0	9.7	0.0	0.0	9.7	0.0
Pier Cap	2.94		0.0	7.8	0.0	27.5	7.8	213.7
Upper pier Column	0.00		0.0	3.0	0.0	0.0	3.0	0.0
Pier Column	3.30		47.2	4.7	219.7	12.0	4.7	55.9
Upper pilecap	0.00		0.0	3.0	0.0	0.0	3.0	0.0
PileCap	0.00		0.0	3.0	0.0	0.0	3.0	0.0
Total at bottom of pilecap			***47.2***		***219.7***	***39.5***		***269.6***

At average Annual water level

Item	Exposed height	Vertical F_V	Longitudinal			Transversal		
			F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Curtain wall	0.00		0.0	7.7	0.0	0.0	7.7	0.0
Pier Cap	2.94		0.0	5.8	0.0	27.5	5.8	158.7
Upper pier Column	0.00		0.0	2.3	0.0	0.0	2.3	0.0
Pier Column	2.05		29.3	3.3	96.0	7.5	3.3	24.4
Upper pilecap	0.00		0.0	2.3	0.0	0.0	2.3	0.0
Total at bottom of Column			***29.3***		***96.0***	***35.0***		***183.1***
Curtain wall	0.00		0.0	9.7	0.0	0.0	9.7	0.0
Pier Cap	2.94		0.0	7.8	0.0	27.5	7.8	213.7
Upper pier Column	0.00		0.0	4.3	0.0	0.0	4.3	0.0
Pier Column	2.05		29.3	5.3	154.5	7.5	5.3	39.3
Upper pilecap	0.00		0.0	4.3	0.0	0.0	4.3	0.0
PileCap	0.00		0.0	4.3	0.0	0.0	4.3	0.0
Total at bottom of pilecap			***29.3***		***154.5***	***35.0***		***253.0***

6.Vessel Collision

7.Vehicular Collision Force '1'yes '0' no 0

1. Dead Loads

Left side Span

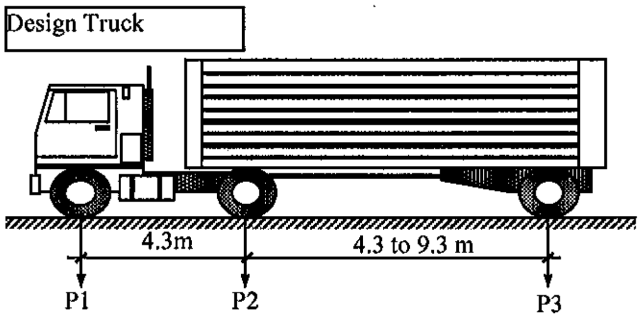
Item	Volume	Vertical	Longitudinal			Transversal		
		F_V	F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m3)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Stage1 (DC)								
Girders	136.65	1674.0		1.250	-2092.5			
Diaphragm	18.23	223.4		1.250	-279.2			
Precast plank	15.28	187.2		1.250	-234.0			
Deck slab	102.55	1256.3		1.250	-1570.4			
Total		***3340.8***			***-4176.0***			
Stage2 (DW)								
Pavement	32.71	361.4		1.250	-451.8			
Parapet + railing		444.7		1.250	-555.8			
Lighting post + mis.		30.0		1.250	-37.5			
Total		***836.1***			***-1045.1***			

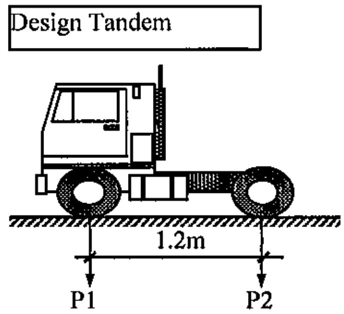
Right side Span

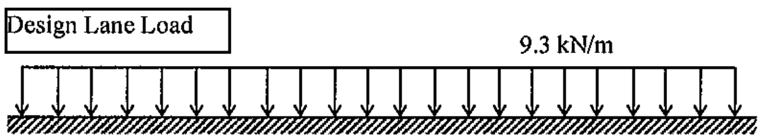
Item	Volume	Vertical	Longitudinal			Transversal		
		F_V	F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m3)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Stage1 (DC)								
Girders	136.65	1674.0		1.250	2092.5			
Diaphragm	18.23	223.4		1.250	279.2			
Precast plank	15.28	187.2		1.250	234.0			
Deck slab	102.55	1256.3		1.250	1570.4			
Total		***3340.8***			***4176.0***			
Stage2 (DW)								
Pavement	32.71	361.4		1.250	451.8			
Parapet + railing		444.7		1.250	555.8			
Lighting post + mis.		30.0		1.250	37.5			
Total		***836.1***			***1045.1***			

Live load data

Design Truck	P1	35.0KN	V1	4.3 m
	P2	145.0KN	V2	4.3 m
	P3	145.0KN		
Design Tandem	P1	110.0KN	V3	1.2 m
	P2	110.0KN		
Design Lane Load	P_L	9.3 KN/m		
Pedestrian Load	Pp	0.0		
Sidewalk width - both 2 sides	sw	0.0		
Maximum number of design lane	nlanes	3.0		
Multiple presence factor	m	0.85		
Dynamic load allowance		(1+IM)		
Deck joint - all limit states		1.75		
Other structure - all limit states (except fatigue)		1.25		

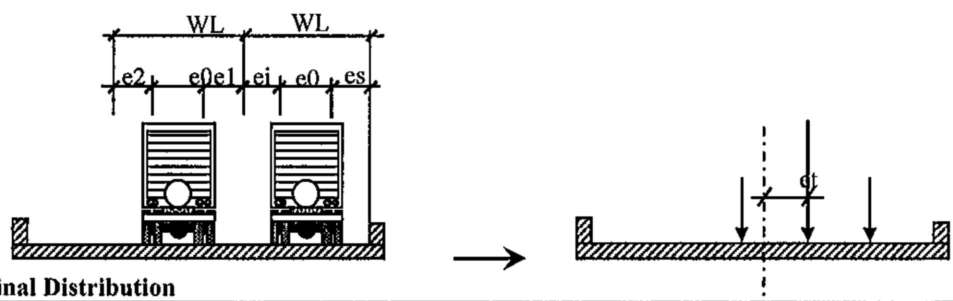






Transverse Distribution

Distance from wheel axis to inner face of curb and bewteen wheel axis					
In general case	ei	1.20 m	es	0.60	m
For deck overhang design	ei	1.30 m	es	0.50	m
Distance between wheels			e0	1.80	m
Design lane width			WL	3.60	m
			e1	0.00	m
			e2	1.80	m
Curb width			wc	0.50	m
Transverse excentricity of design vehicle 1 - general case			ex1	4.38	
Transverse excentricity of design vehicle 2			ex2	2.58	
Transverse excentricity of design vehicle 3			ex3	0.78	
Transverse excentricity of design vehicle 4			ex4	-1.03	
Transverse Excentricity of design vehicle			et	1.68	m



Longitudinal Distribution

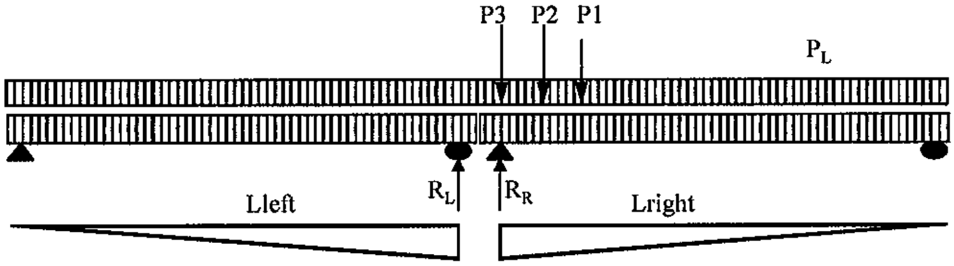
Case1a: 1Truck + Lane load on 1 span
Case1b: 1Truck + Lane load on 2 spans
Case2a: 2Trucks + Lane load on 1 span
Case2b: 2Trucks + Lane load on 2 spans
Case3a: 1Tandem + Lane load on 1 span
Case3b: 1Tandem + Lane load on 2 spans

Influence line value

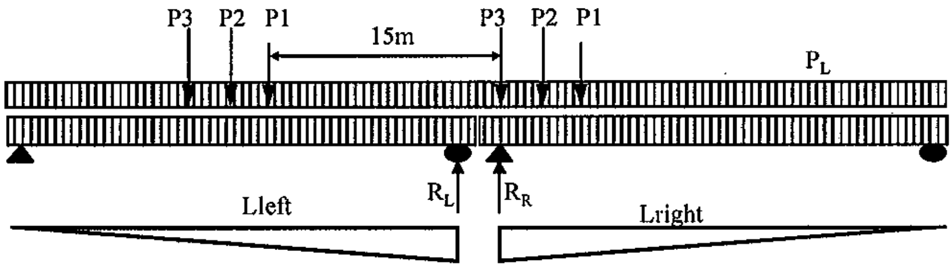
Case	Left span				Right span			
	P3	P2	P1	Aleft	P3	P2	P1	Aright
Case1a:				16.05	1.00	0.87	0.73	16.05
Case1b:	0.94			16.05		1.00	0.87	16.05
Case2a:*	1.00	0.87	0.73	16.05	0.26	0.13	0.00	16.05
Case2b:	0.34	0.48	0.61	16.05	1.00	0.87	0.73	16.05
Case3a:				16.05		1.00	0.96	16.05
Case3b:		1.04		16.05			1.00	16.05

* 2 Trucks in right span

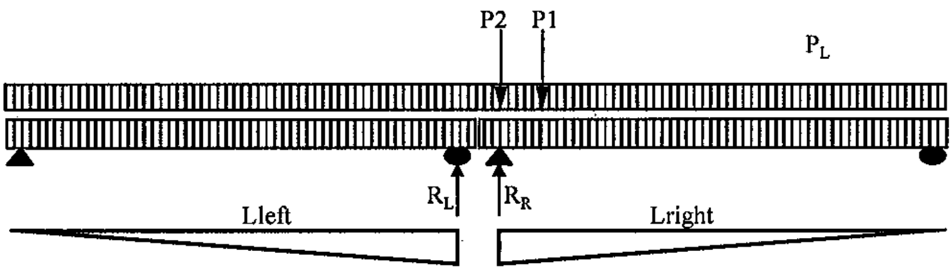
Truck + Lane load



2Trucks + Lane load



Tandem + Lane load



For 1 truck or tandem: Reaction = [(Pi* yi)*(1+IM) + PL*A]*nlane*m

For 2 trucks: Reaction = 0.9* [(Pi* yi)*(1+IM) + PL*A]*nlane*m

2 Loaded Lane m = 1.20

Item	Reaction left	Reaction right	Vertical F_V	Longitudinal			Transversal		
				F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(kN)	(kN)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Case1a:	179.1	623.4	802.5			555.4			3507.1
Case1b:	384.4	442.1	826.5			72.1			3611.8
Case2a:	161.2	638.5	799.7			596.6			3494.8
Case2b:	350.4	561.1	911.5			263.3			3983.3
Case3a:	179.1	502.9	682.1			404.8			2980.6
Case3b:	350.8	344.1	694.9			-8.4			3036.8

2 Loaded Lane m = 1.00

Item	Reaction left	Reaction right	Vertical F_V	Longitudinal			Transversal		
				F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(kN)	(kN)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Case1a:	298.5	1039.0	1337.6			925.6			3838.8
Case1b:	640.7	736.8	1377.5			120.1			3953.5
Case2a:	268.7	1064.2	1332.9			994.4			3825.4
Case2b:	584.1	935.1	1519.2			438.8			4360.1
Case3a:	298.5	838.2	1136.8			674.6			3262.6
Case3b:	584.7	573.5	1158.2			-13.9			3324.0

3 Loaded Lane m = 0.85

Item	Reaction left	Reaction right	Vertical F_V	Longitudinal			Transversal		
				F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(kN)	(kN)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Case1a:	380.6	1324.8	1705.4			1180.2			2336.4
Case1b:	816.9	939.4	1756.3			153.2			2406.2
Case2a:	342.6	1356.9	1699.4			1267.9			2328.2
Case2b:	744.7	1192.3	1937.0			559.5			2653.6
Case3a:	380.6	1068.8	1449.4			860.2			1985.7
Case3b:	745.5	731.3	1476.7			-17.7			2023.1

4 Loaded Lane m = 0.65

Item	Reaction left	Reaction right	Vertical F_V	Longitudinal			Transversal		
				F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(kN)	(kN)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Case1a:	388.1	1350.7	1738.8			1203.3			54.3
Case1b:	832.9	957.9	1790.8			156.2			56.0
Case2a:	349.3	1383.5	1732.7			1292.7			54.1
Case2b:	759.3	1215.7	1974.9			570.5			61.7
Case3a:	388.1	1089.7	1477.8			877.0			46.2
Case3b:	760.1	745.6	1505.7			-18.1			47.1

Item Live Load	Vertical F_V	Longitudinal			Transversal		
		F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Total at bottom of Column	***1519.2***	***0.0***	***0.0***	***438.8***	***0.0***	***0.0***	***4360.1***
Total at bottom of pilecap	***1519.2***	***0.0***	***0.0***	***438.8***	***0.0***	***0.0***	***4360.1***

Pedestrian Load

Item	Reaction left	Reaction right	Vertical F_V	Longitudinal			Transversal		
				F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(kN)	(kN)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
1side	0.0	0.0	0.0			0.0		0.0	
2side	0.0	0.0	0.0			0.0		0.0	

3.Centrifugal Force

Centrifugal force data	CE = n * m * (Axle weights) * C		
Axle weights of design Truck	P	325.0	kN
Number of loaded lanes	n	3.0	lanes
	m	0.85	
Factor, C = (4/3)* V^2/ (g*R)	C	0.000	kN
Highway design speed	V	33.3	m/s
Radius of curvature of traffic lane	R	0.0	m
Centrifugal force	CE	0.0	kN

Item	From surface	Vertical F_V	Longitudinal			Transversal		
			F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Centrifugal force	1.80							
Total at bottom of Column						*0.0*	9.784	*0.0*
Total at bottom of pilecap						*0.0*	11.784	*0.0*

4.Braking Force

Braking force data			
Axle weights of design Truck	P	325.0	kN
Number of loaded lanes	n	3.0	lanes
	m	0.85	
Br1 = 25%*(design truck)*n*m	Br1	207.19	kN
Br2 = 5%*(design truck + 9.3*Lbridge)*n*m	Br2	119.70	kN
Br = max(Br1, Br2)	Br	207.19	kN

Item	From surface	Vertical F_V	Longitudinal			Transversal		
			F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
Take 100 %	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Braking force	1.80							
Total at bottom of Column			*207.2*	9.784	*2027.1*			
Total at bottom of pilecap			*207.2*	11.784	*2441.5*			

5.Uniform Temperature

Uniform temperature data			
Installing temperature	t0	27.0	deg
Maximum temperature	tmax	47.0	deg
Minimum temperature	tmin	10.00	deg
Plus temperature amplitude	Δtmax	20.0	deg
Minus temperature amplitude	Δtmin	17.0	deg
Coefficient of Thermal Expansion	α	1.08E-05	
Strain due to minus temperature	ε_T	1.84E-04	

Item	From surface	Vertical F_V	Longitudinal			Transversal		
			F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Total at bottom of Column			*54.2*	7.82	*214.9*			
Total at bottom of pilecap			*54.2*	9.82	*323.4*			

6.Creep & Shrinkage

Creep & shrinkage data

Item	From surface	Vertical F_V	Longitudinal			Transversal		
			F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Total at bottom of Column			*90.9*	7.82	*346.8*			
Total at bottom of pilecap			*90.9*	9.82	*528.6*			

7.Wind on Structure

Wind loads data	$P_D = 0.0006\ V^2\ C_d \geq 1.8$ (kN/m2)		
Average elevation of deck girder upper ground or water plane level	Hele_g	8.8	m
Correct coefficient for wind zone and elevation of pier	S	1.09	
Design wind speed V = S*Vb	V	57.8	m/s
Overall width between handrails	b	12.8	m
Superstructure height including solid parapet	d	2.98	m
	b/d	4.28	
Obstacle coefficient for pier	Cd	1.36	
Wind pressure on pier	P_D	2.72	kN/m2

Item	Exposed height	Vertical F_V	Longitudinal			Transversal		
			F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Superstructure	2.98		66.9	9.2	612.8	267.5	9.2	2451.1
Total at bottom of Column			***66.9***		***612.8***	***267.5***		***2451.1***
Superstructure	2.98		66.9	11.2	746.5	267.5	11.2	2986.1
Total at bottom of pilecap			***66.9***		***746.5***	***267.5***		***2986.1***

8.Wind on Vehicle

For strength combination III, wind load on vehicular and on structure have to simultaneously consider (wind speed 25m/s). Wind load on vehicular in tranversal direction, is 1.5 kN/m at 1.8m height from asphalt surface. Wind load on vehicular in longitudinal direction is 0.75 kN/m at 1.8m height from asphalt surface.

Item	Vertical F_V	Longitudinal			Transversal		
		F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Superstructure		24.8	11.5	285.5	49.5	11.5	570.9
Total at bottom of Column		***24.8***		***285.5***	***49.5***		***570.9***
Superstructure		24.8	13.5	335.0	49.5	13.5	669.9
Total at bottom of pilecap		***24.8***		***335.0***	***49.5***		***669.9***

9.Earth Quake

Earth Quake data			
Acceleration coefficient	A	0.0310	g
Seismic zone	Sz	1	
Soil profile type: according to geological data survey		I	
Coeffient site	S	1.00	
Bridge importance category: "1:critical"; "2:essential"; "3:other"	IC	2	essential
Response Modification Factor			
Column		2.0	
Connection		1.0	
Foundation		1.0	

Response Spectrum - Single mode method is used for EQ analysis.
Result of pier internal force is showed here. For detail refer to annex.

Item	Vertical F_V	Longitudinal			Transversal		
		F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Total at bottom of Column		*366*		*1414*	*338*		*1355*
Total at bottom of pilecap		*366*		*2147*	*338*		*2030*

B.LOAD COMBINATIONS

I.LOAD COMBINATIONS

Loads at Bottom of Column						
Loads	Sign	F_V (kN)	Longitudinal F_{HX} (kN)	Longitudinal My (kN•m)	Transvesal F_{HY} (kN)	Transvesal Mx (kN•m)
Superstructure Loads						
1.Stage1 - Dead Load	DC	6682		0		
2.Stage2 - Pave.+Parapet+Railing+Mis.	DW	1672		0		
3.Live Load	LL	1519		439		4360
4.Pedestrian	LL	0		0		0
5.Centrifugal force	CE				0	0
6.Braking force	BR		207	2027		
7.Uniform temperature	TU		54	215		
8.Creep and Shrinkage	CR&SH		91	347		
9.Wind pressure on superstructure	WS		67	613	268	2451
10.Wind pressure on vehicles	WL		25	285	50	571
11.Earthequake						
a - Longitudinal direction	EQ		183	707		
b - Transverse direction	EQ				169	677
Substructure Loads						
1.Pier selfweight	DC	3048				
2.Soil on pile cap	EV	0				
3.Bouyancy on pier						
a - Maximum water level	WA	-251				
b - Minimum water level	WA	-71				
c - Average annual water level	WA	-161				
4.Stream pressure						
a - Maximum water level	WA		0	0	5	14
b - Minimum water level	WA		0	0	0	0
c - Average annual water level	WA		0	0	1	2
5.Wind pressure						
a - Maximum water level	WS		11	44	30	170
b - Minimum water level	WS		47	125	40	191
c - Average annual water level	WS		29	96	35	183

Loads at Bottom of Pilecap

Loads	Sign	F_V (kN)	Longitudinal F_{HX} (kN)	Longitudinal My (kN•m)	Transvesal F_{HY} (kN)	Transvesal Mx (kN•m)
Superstructure Loads						
1.Stage1 - Dead Load	DC	6682		0		
2.Stage2 - Pave.+Parapet+Railing+Mis.	DW	1672		0		
3.Live Load	LL	1519		439		4360
4.Pedestrian	LL	0		0		0
5.Centrifugal force	CE				0	0
6.Braking force	BR		207	2441		
7.Uniform temperature	TU		54	323		
8.Creep and Shrinkage	CR&SH		91	529		
9.Wind pressure on superstructure	WS		67	747	268	2986
10.Wind pressure on vehicles	WL		25	335	50	670
11.Earthequake						
a - Longitudinal direction	EQ		366	2147		
b - Transverse direction	EQ				338	2030
Substructure Loads						
1.Pier selfweight	DC	5008				
2.Soil on pile cap	EV	579				
3.Bouyancy on pier						
a - Maximum water level	WA	-1035				
b - Minimum water level	WA	-856				
c - Average annual water level	WA	-946				
4.Stream pressure						
a - Maximum water level	WA		0	0	5	18
b - Minimum water level	WA		0	0	0	0
c - Average annual water level	WA		0	0	1	4
5.Wind pressure						
a - Maximum water level	WS		11	67	30	231
b - Minimum water level	WS		47	220	40	270
c - Average annual water level	WS		29	155	35	253

Load Factors and Load Combinations

Loads	Sign	Str1a 2	Str1b 3	Str2a 4	Str2b 5	Str3a 6	Str3b 7
Superstructure Loads							
1.Stage1 - Dead Load	DC	1.25	0.90	1.25	0.90	1.25	0.90
2.Stage2 - Pave.+Mis.	DW	1.50	0.65	1.50	0.65	1.50	0.65
3.Live Load	LL	1.75	1.75	-	-	1.35	1.35
4.Pedestrian	LL	1.75	1.75	-	-	1.35	1.35
5.Centrifugal force	CE	1.75	1.75	-	-	1.35	1.35
6.Braking force	BR	1.75	1.75	-	-	1.35	1.35
7.Uniform temperature	TU	0.50	0.50	0.50	0.50	0.50	0.50
8.Creep and Shrinkage	CR&SH	0.50	0.50	0.50	0.50	0.50	0.50
9.Wind pressure on superst.	WS	-	-	1.40	1.40	0.40	0.40
10.Wind pressure on vehicles	WL	-	-	-	-	1.00	1.00
11.Earthequake							
a - Longitudinal direction	EQL	-	-	-	-	-	-
b - Transverse direction	EQT	-	-	-	-	-	-
Substructure Loads							
1.Pier selfweight	DC	1.25	0.90	1.25	0.90	1.25	0.90
2.Soil on pile cap	EV	1.35	0.90	1.35	0.90	1.35	0.90
3.Bouyancy on pier							
a - Maximum water level	WA		1.00		1.00		1.00
b - Minimum water level	WA	1.00		1.00		1.00	
c - Average annual WL	WA						
4.Stream pressure							
a - Maximum water level	WA		1.00		1.00		1.00
b - Minimum water level	WA	1.00		1.00		1.00	
c - Average annual WL	WA						
5.Wind pressure							
a - Maximum water level	WS	-	-		1.40		0.40
b - Minimum water level	WS	-	-	1.40		0.40	
c - Average annual WL	WS	-	-				

Load Factors and Load Combinations

Loads	Sign	Ser1 10	Ext1a 11	Ext1b 12	Ext1c 13	Ext1d 14	
Superstructure Loads							
1.Stage1 - Dead Load	DC	1.00	1.25	0.90	1.25	0.90	
2.Stage2 - Pave.+Mis.	DW	1.00	1.50	0.65	1.50	0.65	
3.Live Load	LL	1.00	0.50	0.50	0.50	0.50	
4.Pedestrian	LL	1.00	0.50	0.50	0.50	0.50	
5.Centrifugal force	CE	1.00	0.50	0.50	0.50	0.50	
6.Braking force	BR	1.00	0.50	0.50	0.50	0.50	
7.Uniform temperature	TU	1.00	-	-	-	-	
8.Creep and Shrinkage	CR&SH	1.00	-	-	-	-	
9.Wind pressure on superst.	WS	0.30	-	-	-	-	
10.Wind pressure on vehicles	WL	1.00	-	-	-	-	
11.Earthequake							
a - Longitudinal direction	EQL	-	1.00	1.00	0.30	0.30	
b - Transverse direction	EQT	-	0.30	0.30	1.00	1.00	
Substructure Loads							
1.Pier selfweight	DC	1.00	1.25	0.90	1.25	0.90	
2.Soil on pile cap	EV	1.00	1.35	0.90	1.35	0.90	
3.Bouyancy on pier							
a - Maximum water level	WA						
b - Minimum water level	WA	1.00					
c - Average annual WL	WA		1.00	1.00	1.00	1.00	
4.Stream pressure							
a - Maximum water level	WA						
b - Minimum water level	WA	1.00					
c - Average annual WL	WA		1.00	1.00	1.00	1.00	
5.Wind pressure							
a - Maximum water level	WS		-	-	-	-	
b - Minimum water level	WS	0.30	-	-	-	-	
c - Average annual WL	WS		-	-	-	-	

II. LOAD COMBINATIONS RESULT

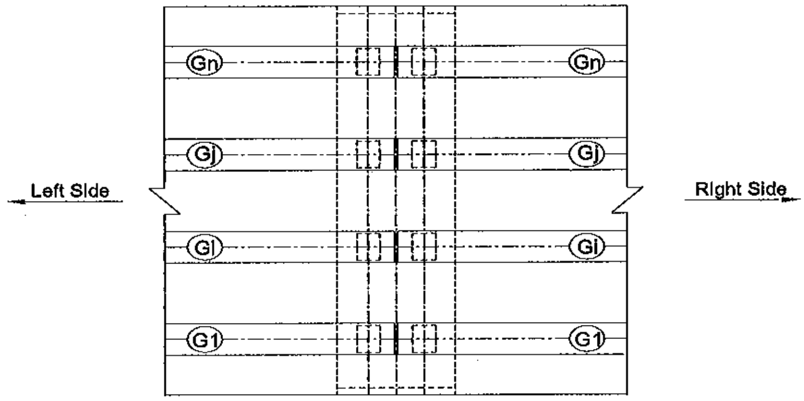
Load Combinations at Bottom of PierColumn

No	Combinations		Sign	F_V (kN)	Longitudinal F_{HX} (kN)	Longitudinal My (kN•m)	Transvesal F_{HY} (kN)	Transvesal Mx (kN•m)
1	Strength 1a		Str1a	17258	435	4596	0	7630
2	Strength 1b		Str1b	12252	435	4596	5	7644
3	Strength 2a		Str2a	14600	232	1314	430	3698
4	Strength 2b		Str2b	9593	182	1201	422	3683
5	Strength 3a		Str3a	16651	423	4191	172	7514
6	Strength 3b		Str3b	11644	408	4158	174	7519
7	Service 1		Ser1	12850	411	3535	142	5724
8	Extreme 1a	EQL	Ext1a	15270	287	1940	52	2385
9	Extreme 1b	EQL	Ext1b	10443	287	1940	52	2385
10	Extreme 1c	EQT	Ext1c	15270	159	1445	170	2859
11	Extreme 1d	EQT	Ext1d	10443	159	1445	170	2859

Load Combinations at Bottom of PileCap

No	Combinations		Sign	F_V (kN)	Longitudinal F_{HX} (kN)	Longitudinal My (kN•m)	Transvesal F_{HY} (kN)	Transvesal Mx (kN•m)
1	Strength 1a		Str1a	19705	435	5467	0	7630
2	Strength 1b		Str1b	13752	435	5467	5	7648
3	Strength 2a		Str2a	17047	232	1779	430	4558
4	Strength 2b		Str2b	11094	182	1565	422	4521
5	Strength 3a		Str3a	19098	423	5036	172	7859
6	Strength 3b		Str3b	13145	408	4975	174	7861
7	Service 1		Ser1	14604	411	4357	142	6007
8	Extreme 1a	EQL	Ext1a	17717	470	3587	103	2793
9	Extreme 1b	EQL	Ext1b	11943	470	3587	103	2793
10	Extreme 1c	EQT	Ext1c	17717	214	2084	339	4215
11	Extreme 1d	EQT	Ext1d	11943	214	2084	339	4215

C.PIER CAP ANALYSIS



1. DEAD LOAD

Left side Reactions (kN)								
STT	Term	*G1*	*G2*	*G3*	*G4*	*G5*	*G6*	*G7*
1	Girders	334.8	334.8	334.8	334.8	334.8		
2	Diaphragm	44.7	44.7	44.7	44.7	44.7		
3	Precast plank	37.4	37.4	37.4	37.4	37.4		
4	DeckSlab	230.9	230.9	230.9	230.9	230.9		
5	Pavement	72.3	72.3	72.3	72.3	72.3		
6	Parapet+railing	222.3	222.3	222.3	222.3	222.3		
Right side Reactions (kN)								
STT	Term	*G1*	*G2*	*G3*	*G4*	*G5*	*G6*	*G7*
1	Girders	334.8	334.8	334.8	334.8	334.8		
2	Diaphragm	44.7	44.7	44.7	44.7	44.7		
3	Precast plank	37.4	37.4	37.4	37.4	37.4		
4	DeckSlab	230.9	230.9	230.9	230.9	230.9		
5	Pavement	72.3	72.3	72.3	72.3	72.3		
6	Parapet+railing	222.3	222.3	222.3	222.3	222.3		
Total reactions (kN)- DC		***1295.6***	***1295.6***	***1295.6***	***1295.6***	***1295.6***		
Total reactions (kN) -DW		***589.3***	***589.3***	***589.3***	***589.3***	***589.3***		

2. LIVE LOAD
Distribution of Live Load for beam

Distribution load factor: design truck					
Exterior girder		Interior girder 1		Interior girder 2	
X	Y	X	Y	X	Y
-0.18	1.07	-2.73	-	-5.28	-
1.63	0.36	-0.93	0.64	-3.48	-
3.43	-	0.88	0.66	-1.68	0.34
5.23	-	2.68	-	0.13	0.95
DF	1.43		1.29		1.29

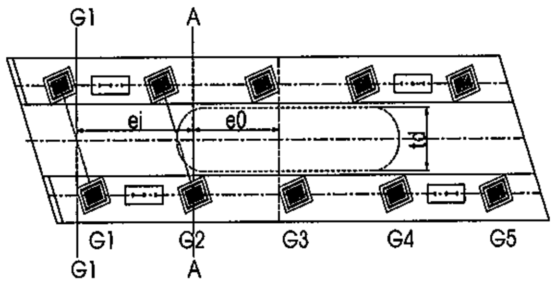
Distribution load factor: lane load					
Exterior girder		Interior girder 1		Interior girder 2	
X	Y	X	Y	X	Y
-0.775	1.30	-3.33	-	-5.88	-
2.825	-	0.275	0.89	-2.275	0.11
DF	1.30		0.89		0.11

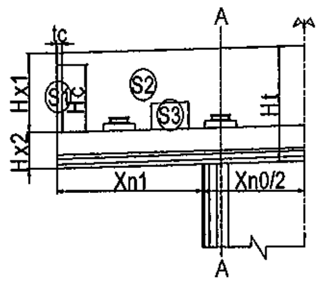
Live load reactions is calculated by level rule

HL93		Rl (kN)	Rr (kN)	Reations at left (KN)			Reations at right (KN)		
				$\Sigma R*DF_e*IM$	$\Sigma R*DF_I*IM$	$\Sigma R*DF_{II}*IM$	$\Sigma R*DF_e*IM$	$\Sigma R*DF_I*IM$	$\Sigma R*DF_{II}*IM$
Truck	P1	-	28	-	-	-	51	46	46
	P2	-	137	-	-	-	245	221	221
	P3	145	-	259	235	235	-	-	-
	Total			**259**	**235**	**235**	**296**	**267**	**267**
	P1	26	19	46	41	41	34	31	31
	P2	126	60	225	203	203	108	97	97
	P3	145	41	259	235	235	74	67	67
	Total			**530**	**479**	**479**	**216**	**195**	**195**
Tedan	P4	110	-	197	178	178	-	-	-
	P5	-	110	-	-	-	197	178	178
	Total			197	178	178	197	178	178
Lane	Wl	149	149	195	133	133	195	133	16

Live load reactions							
Reaction	*G1*	*G2*	*G3*	*G4*	*G5*	*G6*	*G7*
Left side	672	564	564				
Right side	490	400	283				

3. PIER CAP DESIGN





Cantilever section (A-A)

Distance from centerline of pier to section A-A Lc = **4.03** m

Item	*G1*	*G2*	*G3*	*G4*	*G5*	*G6*	*G7*
Bearing is taken into account	1	1	0	0	0	0	0
Distance from bearing to A-A							
Left side ei	2.76	0.21	-	-	-	-	-
Right side ei	2.76	0.21	-	-	-	-	-

Dead load of substructure

Notation	Dimensions	Value(m)	Notation	Dimensions	Value(m)
Hx1	Haunch 1 height	1.94	Hc	Curtain wall height	1.80
Hx2	Haunch 2 height	1.00	tc	Curtain wall thickness	0.15
Xn1	Haunch width	3.89	xd	Pier cap width	3.60
Xn0	Bottom of pier cap width	5.50	xd1	Pier cap width	1.00

Item	**Volume**	**Section A-A**		
		Fv	**Arm.$_{Fv}$**	**M_x**
	(m3)	(kN)	(m)	(kN•m)
S1	0.54	13.2	4.03	53.3
S2	25.65	628.5	2.02	1266.5
S3	0.42	10.4	1.49	15.4
Total		***652.2***		***1335.3***

Load components at section A-A

Item	**Vertical**	**Moment (kN•m)**		**Load Factor**	
	Fv-(kN)	Mx	Mz	**Ser1**	**Str1a**
Pier Selfweight	652.2	1335.3	0.0	1.00	1.25
DC stage1	2591.3	3848.1	0.0	1.00	1.25
DW stage2	1178.5	1750.1	0.0	1.00	1.50
Live Load	2126.7	3409.3	431.7	1.00	1.75
Pedestrian	0.0	0.0	0.0	1.00	1.75

Comb.	**Vertical**	**Bending**	
	Fv	**Mx**	**My**
Ser1	***6549***	***10343***	***432***
Str1a	***9544***	***15071***	***755***

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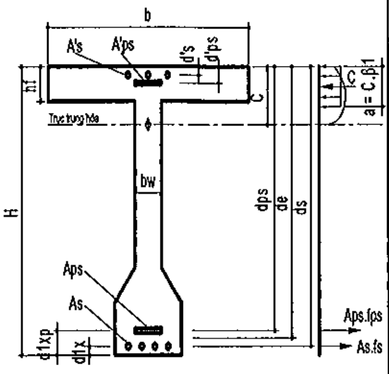
	Bridge LRB09	Design			
	DETAIL DESIGN	Check			
	Pier P2	Revise			

22TCN272-05; AASHTO LRFD 2nd - 1998

REINFORCEMENT CHECKING - PIER CAP

MATERIALS

	NORMAL CONCRETE		
f'c	Compressive Strength of concrete at 28 days	Mpa	30
Ec	Modulus of Elasticity	Mpa	27691
fr	Modulus of Rupture	Mpa	3.5
gc	Unit weight of concrete	kN/m3	24.5
	PRESTRESSING STEEL		
fpu	Tensile strength of prestressing steel	Mpa	1860
fpy	Yield strength of prestressing steel	Mpa	1670
Ep	Modulus of Elasticity	Mpa	195000
	REINFORCEMENT		
fy	Yield strength	Mpa	400
Es	Modulus of Elasticity	Mpa	200000
nc	Ratio Es/Ec		7.00



Sign	Parameters		Unit	Section - A-A				
				Mx	Mx			
	INTERNAL FORCES AT SECTIOIN							
	Combination			Strength	Service			
Qu	Shear		kN	9544	6549			
Mu	Flexural Moment		kNm	15071	10343			
Nu	Axial load		kN					
Tu	Torsional Moment		kNm					
	FLEXURAL MOMENT CHECKING							
H	Section height		m	2.935	2.935			
d's	Dis. From comp. fiber to centroid of comp. Reinf		m	0.058	0.058			
d1x	Dis. From tens. fiber to centroid of tension Reinf		m	0.116	0.116			
	Cover to reinf		m	0.050	0.050			
ds	Dis. From comp. fiber to centroid of tension Reinf		m	2.819	2.819			
d'ps	Dis. From comp. fiber to centroid of comp. prestressing steel		m	0.000	0.000			
d1xp	Dis. From tens. fiber to centroid of tension prestressing steel		m	0.000	0.000			
dps	Dis. From comp. fiber to centroid of tension prestressing steel		m	2.935	2.935			
b	Width of the compression face of member		m	1.600	1.600			
bw	Web width or diameter of a circular section		m	3.600	3.600			
hf	Compression flange depth		m	0.000	0.000			
Iz	Moment of inertia of section		m4	3.371	3.371			
Amc	Section area		m2	4.696	4.696			
	Steel choice							
Aps	Tension prestressing steel	P.S type		0 T00.0	0 T00.0			
		Number	tendons	0	0			
		Area	m2	0.00000	0.00000			
A'ps	Compression prestressing steel	P.S type		0 T00.0	0 T00.0			
		Number	tendons	0	0			
		Area	m2	0.00000	0.00000			
As	Tension Reinforcement	Number	bars	26	26			
		Diameter	mm	32	32			
		Area	m2	0.02083	0.02083			
A's	Compression Reinforcement	Number	bars	30	30			
		Diameter	mm	16	16			
		Area	m2	0.00606	0.00606			
A'c	Shear reinforcement	Number	bars	4	4			
		Diameter	mm	20	20			
		Area	m2	0.00126	0.00126			

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ϕ	Resistance factors for flexure	5.5.4.2	0.90	1.00			
ϕv	Resistance factors for shear		0.90	1.00			
ϕn	Resistance factors for axial force		1.00	1.00			
β1	Stress block factor		0.836	0.836			
c	Dis. Between centroid and top fiber	m	0.077	0.077			
	For T section behavior	m	0.077	0.077			
	For rectangular section behavior	m	0.173	0.173			
fpe	Effective stress in the prestressing steel after losses	Mpa	1116	1116			
fps	Aver. stress in pres. steel at the time for which the nominal resistanc	Mpa	1846	1846			
k	Factor depends on type of P.S, Low relaxation strand k = 0.28		0.28	0.28			
a	Depth of equivalent stress block	m	0.064	0.064			
de	Corresp. effective depth from extreme comp. fiber						
	to centroid of tensile force in the tensile reinf.	m	2.819	2.819			
Mn	Nominal resistance	kNm	23153	23153			
Mr	Factored reistance	kNm	**20838**	**23153**			
Mu	Flexual moment	kNm	**15071**	**10343**			
(5.7.3.2)	**Flexural moment Checking**		**OK**	**OK**			
	Limits for reinforcement						
c/de	Maximum reinforcement		0.03	0.03			
	Maximum reinforcement Checking	<= 0.42	**OK**	**OK**			
1.2*Mcr	Craking moment	kNm	4884	4884			
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**		**OK**	**OK**			
(5.7.3.4)	**Conctrol of craking by distr. of reinf for RC member- Check?**		No	Yes			
	Existing condition for structrure	1,2 or 3	2	2			
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.066	0.066			
Z	Crack width parameter	N/mm	23000	23000			
A	Area of concr. with same centroid as tens. Reinf	m2	0.008	0.008			
fsa	Value	Mpa	283	283			
0.6*fy		Mpa	240	240			
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	240	240			
x	Dist. From compression fiber to centroid	m	-	0.631			
J.d	Arm	m	-	2.609			
Icr	Moment of inertia of the cracked section	m4	-	0.844			
fs	Tensile stress in reinforcement fs = Msls / (As*J.d)	Mpa	-	190			
	Checking for control cracking fs<fsa		**N.a**	**OK**			

SHEAR AND TORSION CHECKING

β	Factor indicating diag. cracked concr. to tension		1.7	1.9			
θ	Angle of inclination of diagonal compressive	degree	42.97	41.67			
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90			
bv	Effective web width as minimum web width - in dv	m	3.600	3.600			
dv	Effective shear depth	m	2.787	2.787			
	(de - a/2)	m	2.787	2.787			
s	Spacing of stirrups	m	0.150	0.150			
ncat	Amount of bars in spacing S	bars	4	4			
Av	Shear reinf area in spacing S	m2	0.0013	0.0013			
β	Assume		2.0	2.0			
θ	Assume	degree	41.99	36.33			
v	Shear stress in concrete	kN/m2	1057	653			
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116			
e_x	Strain in tensile reinforcement		1.99E-03	1.67E-03			
	if ex<0, multiple with reduce factor		-	-			
	Strain checking	<=2.00E-3	Ok	Ok			
v/f'c	Ratio of shear stress and f'c		0.035	0.022			
β	Final value		1.7	1.9			
θ	Final value	degree	42.97	41.67			
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	7862	8540			
Vs	Shear resistance provided by shear reinforcement	kN	10021	10486			
Vp	Component in the direction of the applied shear of the effective P.S	kN	0	0			
Vn1	Vn1=Vc+Vs+Vp	kN	17884	19025			
Vn2	Vn2	kN	75244	75244			
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	17884	19025			
Vr	Factored shear resistance	kN	**16095**	**19025**			
Vu	Shear	kN	9544	6549			
(5.8.2.7)	**Shear checking**		**OK**	**OK**			

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		Design	-		
	DETAIL DESIGN Pier P2	Check	-		
		Revise	-		

D. COLUMN DESIGN

I. COLUMN DATA

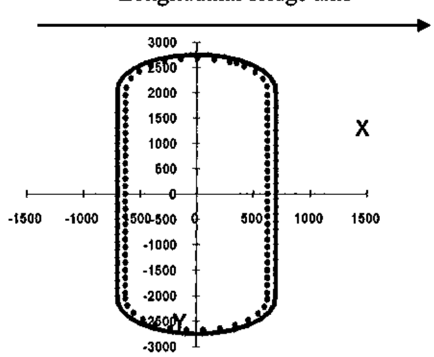
1.Load Combinations at Bottom of Pier Column

No	Combinations		Sign	F_V (kN)	Longitudinal F_{HX} (kN)	Longitudinal My (kN•m)	Transvesal F_{HY} (kN)	Transvesal Mx (kN•m)
1	Strength 1a		Str1a	17258	435	4596	0	7630
2	Strength 1b		Str1b	12252	435	4596	5	7644
3	Strength 2a		Str2a	14600	232	1314	430	3698
4	Strength 2b		Str2b	9593	182	1201	422	3683
5	Strength 3a		Str3a	16651	423	4191	172	7514
6	Strength 3b		Str3b	11644	408	4158	174	7519
7	Service 1		Ser1	12850	411	3535	142	5724
8	Extreme 1a	EQL	Ext1a	15270	287	1940	52	2385
9	Extreme 1b	EQL	Ext1b	10443	287	1940	52	2385
10	Extreme 1c	EQT	Ext1c	15270	159	1445	170	2859
11	Extreme 1d	EQT	Ext1d	10443	159	1445	170	2859

2. Pier Column Material

Normal concrete			
Compressive strength at 28 days age	f'c	30	MPa
Concrete elastic modulus	Ec	27691	MPa
Reinforcement TCVN1651-2008; CBV-400			
Yield strength	fy	400	MPa
Reinforcement elastic modulus	Es	200,000	MPa

Longitudinal bridge axis



3. Pier Column Section

Pier column thickness - longitudinal dimension	td	1.40	m
Pier column width - transverse dimension	tn	5.50	m
Section area	A	7.279	m2
Moment inertia	Ix	16.498	m4
	Iy	1.126	m4
Radius of gyration of gross concrete section; r = sqrt(I/A)	rx	1.505	m
	ry	0.393	m

4. Slenderness Effect S.5.7.4.3, S.4.5.3.2.2b, S.4.6.2.5

Transverse direction: Fixed at bottom; translation free, rotation free at top	Kt	2.10	
Longitudinal direction: Fixed at bottom; translation free, rotation free at top	Kl	2.10	
Unsupported length from top to bottom of column	Lu	7.67	m
Slenderness ratio: if K.Lu / r > 22 than considered	Kt.Lu/rx	10.7	no
	Kl.Lu/ry	41.0	yes
Moment inertia of longitudinal reinforcements	Is	0	m4
Ratio of max factored Per. load moment to max factored total load moment	βd	0	

P - Δ analysis

****Longitudinal Direction**

P - Δ moment dertermination procedure:

Initial	Determining displacement for gross cross section	Δxg = Fx . H^3 / (3.E.Ig)
	Displacement for cracked section	Δxcr = Fcr . Δxg
	Moment P-Δ	M P-Δ = Δxcr . P
	Added lateral force	ΔFx = M P-Δ / H
Step: i st	Determining displacement for gross cross section	Δxg i = (Fx+ΔFx i-1) . H^3 / (3.E.Ig)
	Displacement for cracked section	Δxcr i = Fr . Δxg i
	Moment P-Δ	M P-Δ i = Δxcr i . P
	Added lateral force	ΔFx i = M P-Δ i / H

Combination	Fv	My	Initial					
			Fx	Δxg	Fcr	Δxcr	M P-Δ	ΔFx
	(kN)	(kNm)	(kN)	(m)	(kN)	(m)	(kNm)	(kN)
Strength 1a	17258	4596	599	0.003	2.5	0.007	125	16.3
Strength 1b	12252	4596	599	0.003	2.5	0.007	89	11.5
Strength 2a	14600	1314	171	0.001	2.5	0.002	30	3.9
Strength 2b	9593	1201	156	0.001	2.5	0.002	18	2.4
Strength 3a	16651	4191	546	0.003	2.5	0.007	110	14.3
Strength 3b	11644	4158	542	0.003	2.5	0.007	76	9.9
Service 1	12850	3535	461	0.002	2.5	0.006	71	9.3
Extreme 1a	15270	1940	253	0.001	2.5	0.003	47	6.1
Extreme 1b	10443	1940	253	0.001	2.5	0.003	32	4.2
Extreme 1c	15270	1445	188	0.001	2.5	0.002	35	4.5
Extreme 1d	10443	1445	188	0.001	2.5	0.002	24	3.1

Combination	Fv	My	1st Trial					
			Fx	Δxg	Fcr	Δxcr	M P-Δ	ΔFx
	(kN)	(kNm)	(kN)	(m)	(kN)	(m)	(kNm)	(kN)
Strength 1a	17258	4596	615	0.003	2.5	0.007	128	16.7
Strength 1b	12252	4596	611	0.003	2.5	0.007	90	11.8
Strength 2a	14600	1314	175	0.001	2.5	0.002	31	4.0
Strength 2b	9593	1201	159	0.001	2.5	0.002	18	2.4
Strength 3a	16651	4191	560	0.003	2.5	0.007	113	14.7
Strength 3b	11644	4158	552	0.003	2.5	0.007	78	10.1
Service 1	12850	3535	470	0.002	2.5	0.006	73	9.5
Extreme 1a	15270	1940	259	0.001	2.5	0.003	48	6.2
Extreme 1b	10443	1940	257	0.001	2.5	0.003	32	4.2
Extreme 1c	15270	1445	193	0.001	2.5	0.002	36	4.6
Extreme 1d	10443	1445	191	0.001	2.5	0.002	24	3.1

Combination	Fv	My	2nd Trial					
			Fx	Δxg	Fcr	Δxcr	M P-Δ	ΔFx
	(kN)	(kNm)	(kN)	(m)	(kN)	(m)	(kNm)	(kN)
Strength 1a	17258	4596	616	0.003	2.5	0.007	128	16.7
Strength 1b	12252	4596	611	0.003	2.5	0.007	90	11.8
Strength 2a	14600	1314	175	0.001	2.5	0.002	31	4.0
Strength 2b	9593	1201	159	0.001	2.5	0.002	18	2.4
Strength 3a	16651	4191	561	0.003	2.5	0.007	113	14.7
Strength 3b	11644	4158	552	0.003	2.5	0.007	78	10.1
Service 1	12850	3535	470	0.002	2.5	0.006	73	9.5
Extreme 1a	15270	1940	259	0.001	2.5	0.003	48	6.2
Extreme 1b	10443	1940	257	0.001	2.5	0.003	32	4.2
Extreme 1c	15270	1445	193	0.001	2.5	0.002	36	4.6
Extreme 1d	10443	1445	191	0.001	2.5	0.002	24	3.1

****Transverse Direction**

Combination	Fv	Mx	Initial					
			Fx	Δxg	Fcr	Δxcr	M P-Δ	ΔFx
	(kN)	(kNm)	(kN)	(m)	(kN)	(m)	(kNm)	(kN)
Strength 1a	17258	7630	994	0.000	2.5	0.001	14	1.8
Strength 1b	12252	7644	996	0.000	2.5	0.001	10	1.3
Strength 2a	14600	3698	482	0.000	2.5	0.000	6	0.8
Strength 2b	9593	3683	480	0.000	2.5	0.000	4	0.5
Strength 3a	16651	7514	979	0.000	2.5	0.001	13	1.8
Strength 3b	11644	7519	980	0.000	2.5	0.001	9	1.2
Service 1	12850	5724	746	0.000	2.5	0.001	8	1.0
Extreme 1a	15270	2385	311	0.000	2.5	0.000	4	0.5
Extreme 1b	10443	2385	311	0.000	2.5	0.000	3	0.3
Extreme 1c	15270	2859	373	0.000	2.5	0.000	5	0.6
Extreme 1d	10443	2859	373	0.000	2.5	0.000	3	0.4

Combination	Fv	Mx	1st Trial					
			Fx	Δxg	Fcr	Δxcr	M P-Δ	ΔFx
	(kN)	(kNm)	(kN)	(m)	(kN)	(m)	(kNm)	(kN)
Strength 1a	17258	7630	996	0.000	2.5	0.001	14	1.8
Strength 1b	12252	7644	997	0.000	2.5	0.001	10	1.3
Strength 2a	14600	3698	483	0.000	2.5	0.000	6	0.8
Strength 2b	9593	3683	480	0.000	2.5	0.000	4	0.5
Strength 3a	16651	7514	981	0.000	2.5	0.001	13	1.8
Strength 3b	11644	7519	981	0.000	2.5	0.001	9	1.2
Service 1	12850	5724	747	0.000	2.5	0.001	8	1.0
Extreme 1a	15270	2385	311	0.000	2.5	0.000	4	0.5
Extreme 1b	10443	2385	311	0.000	2.5	0.000	3	0.3
Extreme 1c	15270	2859	373	0.000	2.5	0.000	5	0.6
Extreme 1d	10443	2859	373	0.000	2.5	0.000	3	0.4

Combination	Fv	Mx	2nd Trial					
			Fx	Δxg	Fcr	Δxcr	M P-Δ	ΔFx
	(kN)	(kNm)	(kN)	(m)	(kN)	(m)	(kNm)	(kN)
Strength 1a	17258	7630	996	0.000	2.5	0.001	14	1.8
Strength 1b	12252	7644	997	0.000	2.5	0.001	10	1.3
Strength 2a	14600	3698	483	0.000	2.5	0.000	6	0.8
Strength 2b	9593	3683	480	0.000	2.5	0.000	4	0.5
Strength 3a	16651	7514	981	0.000	2.5	0.001	13	1.8
Strength 3b	11644	7519	981	0.000	2.5	0.001	9	1.2
Service 1	12850	5724	747	0.000	2.5	0.001	8	1.0
Extreme 1a	15270	2385	311	0.000	2.5	0.000	4	0.5
Extreme 1b	10443	2385	311	0.000	2.5	0.000	3	0.3
Extreme 1c	15270	2859	373	0.000	2.5	0.000	5	0.6
Extreme 1d	10443	2859	373	0.000	2.5	0.000	3	0.4

****Load Combinations at bottom of column considering Slenderness Effect**

Combination	Fv Vert. (kN)	Mx Trans. (kNm)	Mx P-Δ (kNm)	Mx Total (kNm)	My Long. (kNm)	My P-Δ (kNm)	My Total (kNm)
Strength 1a	17258	7630	14	7644	4596	128	4725
Strength 1b	12252	7644	10	7654	4596	90	4687
Strength 2a	14600	3698	6	3704	1314	31	1345
Strength 2b	9593	3683	4	3687	1201	18	1219
Strength 3a	16651	7514	13	7527	4191	113	4303
Strength 3b	11644	7519	9	7528	4158	78	4236
Service 1	12850	5724	8	5731	3535	73	3608
Extreme 1a	15270	2385	4	2389	1940	48	1988
Extreme 1b	10443	2385	3	2387	1940	32	1973
Extreme 1c	15270	2859	5	2864	1445	36	1481
Extreme 1d	10443	2859	3	2862	1445	24	1469

II. PIER COLUMN DESIGN

1. Limit of Reinforcement S.5.7.4.2

Minimum area of longitudinal reinforcement in column			
As.fy / (Ag . f'c) >= 0.135	As ≥	0.074	m2
As / Ag >= 0.01	As ≥	0.073	m2
Maximum area of longitudinal reinforcement in column			
As / Ag <= 0.08	As ≤	0.582	m2
Trial Rebars: NG	As	**0.051**	m2
1layers x 82 = 82 bars D28 @150	As1	0.051	m2
1layers x 0 = 0 bars D28 @150	As2	0.000	m2

2. Interation diagram M-P Using Pca-Column software

****In Both Direction**

Strength and Service limit states:

Resistance factor: Compression φ_c = 0.75 *(AASHTO LRFD-2004)*

Tension φ_t = 0.90

Extreme Event limit states:

Resistance factor Compression φ_c = 1.00 *(AASHTO LRFD-2004)*

Tension φ_t = 1.00

No.	COMBINATION	Pu kN	Mux kN-m	Muy kN-m	φ Mnx kN-m	φ Mny kN-m	φ Mn/Mu
1	Strength 1a	17258	4725	7644	18006	29130	3.81
2	Strength 1b	12252	4687	7654	16132	26343	3.44
3	Strength 2a	14600	1345	3704	15046	41436	11.19
4	Strength 2b	9593	1219	3687	12979	39258	10.65
5	Strength 3a	16651	4303	7527	17613	30809	4.09
6	Strength 3b	11644	4236	7528	15723	27941	3.71
7	Service 1	12850	3608	5731	16419	26081	4.55
8	Extreme 1a	15270	1988	2389	19195	23067	9.66
9	Extreme 1b	*10443*	1973	2387	17147	20745	8.69
10	Extreme 1c	15270	1481	2864	18148	35096	12.25
11	Extreme 1d	10443	1469	2862	16265	31689	11.07

3. Column Ties	S.5.7.4.6, S.5.10.6.3, S5.10.11.4.1d - e		
Bridge is in seismic zone	Sz	1	
Area of concrete core measured out-to-out of ties	Ac	6.719	m2
Tie diameter	Dtie	16	mm2
Cross section area of 1 tie	As-tr	0.0002	m2
Spacing of hoops	s	150	mm
Length of reinfocement tie in 1 hoop	Ltie	8.93	m
Ratio of ties reinf. in one hoop/ volume of conc. core one pitch spacing			
ρs = As-tr . Ltie / (Ac * spacing)	ρs	0.0018	
Ratio of spiral reinf. To total volume of concrete core shall satisfy			S.5.7.4.6
ρs >= 0.45 . (Ag/Ac - 1). f'c/ fy = Req1	Req1	0.0028	N/A
			S.5.10.11.3
Transverse Reinforcement for Confinement at Plastic Hinges			S.5.10.11.4.1.d
For a circular column "1:applied","2:Not applied"		2	
ρs >= 0.12 . f'c/ fy = Req2	Req2	0.0090	N/A
For a rectangular column			
Rectangular hoop reinforcement shall satisfy			
Either Ash >= 0.30 . s.hc.f'c/ fy. [Ag/Ac -1] = Req1			
or Ash >= 0.12 . s.hc.f'c/ fy = Req2			
In longitudinal direction "1:applied","2:Not applied"		2	
Number of cross tie	nt_x	4	ties
Total cross-sectional area of tie reinf.	Ash_x	0.0008	m2
Core dimension of tied column	hc_x	1.30	m
Rectangular hoop reinforcement shall satisfy	Req1_x	0.0004	m2
	Req2_x	0.0018	m2
	Conclude		N/A
In transverse direction			
Number of cross tie	nt_y	4	ties
Total cross-sectional area of tie reinf.	Ash_y	0.0008	m2
Core dimension of tied column	hc_y	5.40	m
Rectangular hoop reinforcement shall satisfy	Req1_y	0.0015	m2
	Req2_y	0.0073	m2
	Conclude		N/A
Spacing of Transverse Reinforcement for Confinement			S.5.10.11.4.1.e
Transverse reinforcement for confinement shall be:			
* Provided at the top and bottom of the column over a length not less than the greatest of the			
maximum cross-sectional column dimensions,one-sixth of the clear height of the column, or 450 mm;			
Maximum cross-sectional column dimensions	L1	5.50	m
1/6 of clear height of column	L2	0.72	m
or 450mm	L3	0.45	m
Chosen value: L = max(L1,L2,L3)	L	5.50	m
* Spaced not to exceed one-quarter of the minimum member dimension or 100 mm center-to-center.			
	Spacing	0.10	m
Column connections			S.5.10.11.4.3
* Development length for all longitudinal steel shall be 1.25 times that required in S.5.11			
* Column transverse reinforcement, as specified in Article 5.10.11.4.1d, shall be continued for a distance			
not less than one-half the maximum column dimension or 380 mm from the face of the column connection into			
the adjoining member.			
1/2 maximum column dimension	L4	2.75	m
or 380mm	L5	0.38	m
Chosen value: Le = max(L4,L5)	Le	2.75	m

4. Shear Design

Direction			Long.- X	Trans.-Y	Unit
Shear resistance factors		ϕv	1.0	1.0	
Factored shear force in longitudinal		Vu	435	430	kN
Required shear capacity Vn = Vu/ ϕv		Vn	435	430	kN
Determine concrete shear capacity					
Minimum shear reinforcement will provided in cross section					
Therefore		β	2.0	2.0	
		θ	45.0	45.0	
Cross section equivalent	height	h	1.40	5.50	m
	width	b	5.20	5.20	m
d = h - cover - d1x		d	1.31	5.41	m
dv = max(0.72*h; 0.9*d)		dv	1.18	4.87	m
Vc = min(0.083 . β . sqrt(f'c) . bv. dv, 0.25.f'c.bv.dv)		Vc	5591	23035	kN
Difference between required shear capacity and the capacity provided by concrete					
is the minimum required capacity for shear reinforcements					
Vs = Vn - Vc		Vs	0	0	kN
In this case Vc > Vn so shear reinforcement is no need					
Stirrup diameter		Ds	16	16	
Number of stirrup legs / cross section		ns	6	3	
Shear legs area		Av	0.0012	0.0006	m2
Angle of inclination of shear reinf. to long. axis		α	90	90	deg
Vs = Av. fy. dv. (cot θ + cot α). sin α / s		s <=	-	-	m
Stirrup spacing used		s	0.10	0.10	m
Check minimum shear reinforcement requirement			**OK**	**OK**	
Av >= 0.083. sqrt(f'c). bv. s/ fy = Req		Req	0.0006	0.0006	m2
Check maximum shear reinforcement spacing requirement			**OK**	**OK**	
F = 0.1*f'c*bv*dv		F	18447	76006	kN
If Vu < 0.1*f'c*bv*dv then Smax=0.8*dv <= 600mm					
If Vu > 0.1*f'c*bv*dv then Smax=0.4*dv <= 300mm		Smax	0.60	0.60	m

Interface shear transfer S.5.8.4

Area of concrete engaged in shear transfer	Acv	7.279	m2
Area of shear reinforcement crossing the shear plane	Avf	0.051	m2
For concrete placed against clean, hardened concrete with surface roughened			
Cohesion factor specified in Article 5.8.4.2	c	0.7	MPa
Friction factor	μ	1	
For normal density concrete	λ	1	
Nominal shear resistance of the interface plane shall be taken as			
Vn = c. Acv + μ. Avf. fy	Vn	25300	kN
Vn <= 0.2 . f'c . Acv	Vn <=	43676	kN
Vn <= 5.5 . Acv . (1MPa)	Vn <=	40037	kN
Norminal shear resistance	Vn	25300	kN
Factor for shear friction		1.0	
Factored shear resistance	Vr	25300	kN
Horizontal force at bottom of pier column	Vu	104	kN
	Conclude		**OK**

5. Control of cracking by distribution of reinforcement

Tensile stress in rebars should be satisfied equation: fs <= fsa = Z/[(dc.A)^1/3] and fs <= 0.6.fy				
Direction		Long.- X	Trans.-Y	Unit
Existing condition for structrure	1,2 or 3	1	1	
Crack width parameter	Z	30000	30000	N/mm
Flexural moment	Ms	3608	5731	kNm
Axial thrust at service limit state	Ns	12850	12850	kN
Cross section equivalent height	h	1.40	5.50	m
width	b	5.20	5.20	m
Concrete thickness from tension fiber to tension reinf.	dc	0.05	0.05	m
Concrete thickness from compression fiber to tension reinf.	d	1.31	5.41	kN
Number of rebars	N	41	23	bars
Area of rebars	As	0.0253	0.0142	m2
Area of concrete assumed to participate with reinf.				
A = 2 . dc . b / N	A	0.0127	0.0226	m2
	fsa	349	288	MPa
	0.6fy	240	240	MPa
Min (fsa, 0.6fy) = fs1	fs1	240	240	MPa
e = Ms/Ns+d-h/2	e	0.89	3.11	m
e/d > 1.15	e/d	1.15	1.15	
j = 0.74 + 0.1(e/d) ≤ 0.9	j	0.86	0.86	
i = 1/(1-j.d/e)	i	3.90	3.90	
Stress in rebars: fs = (Ms+Ns(d-h/2))/(As.j.i.d)	fs	104	156	MPa
	Conclude	**OK**	**OK**	
Maximum width of crack: an = 0.076.β.fs.(dc.A)^(1/3)	an	0.113	0.206	mm
Where	β	0.167	0.167	

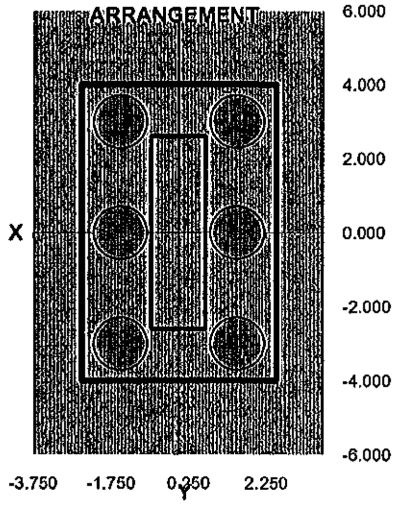
E. PILE CAP DESIGN

1. PILES AND PILECAP DATA

Piles Data			
Number of piles: n_p			6
Co-ordinates of Piles			
Number	Diameter	X_i (m)	Y_i (m)
1	1.000	1.500	3.000
2	1.000	1.500	0.000
3	1.000	1.500	-3.000
4	1.000	-1.500	-3.000
5	1.000	-1.500	0.000
6	1.000	-1.500	3.000
-			
-			
-			
-			
-			
-			
SX_i^2, SY_i^2		13.500	36.000

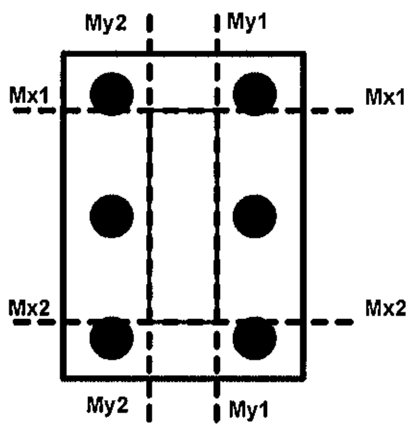
Pilecap Data	
X_i	Y_i
2.500	4.000
2.500	-4.000
-2.500	-4.000
-2.500	4.000
2.500	4.000
Column Data	
X_i	Y_i
0.700	2.600
0.700	-2.600
-0.700	-2.600
-0.700	2.600
0.700	2.600

PLAN OF PILE ARRANGEMENT



2. CRITICAL SECTIONS

Pile cap Length	L	=	8.000 m
Pile cap Width	W	=	5.000 m
Pile cap thickness	H	=	2.000 m
Column Width	w_c	=	5.500 m
Column Thickness	t_c	=	1.400 m
Round nose radius	c_i	=	0.700 m
Column Area	A_c	=	7.279 m2
Column block equivalent:			
Width	w_{ce}	=	5.200 m
Thickness	t_{ce}	=	1.400 m



Distance from Pile to Critical Sections - Arm (m)				
Pile No.	Section			
	Mx1	Mx2	My1	My2
1	0.400	-	0.800	-
2	-	-	0.800	-
3	-	0.400	0.800	-
4	-	0.400	-	0.800
5	-	-	-	0.800
6	0.400	-	-	0.800
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-

3. INTERNAL FORCES CALCULATION

3.1. Pile Reaction (from Pile Foundation analysis)

AXIAL FORCE (KN)											
Pile	STR I-A	STR I-B	STR II-A	STR II-B	STR III-A	STR III-B	SER	EXT I-A	EXT I-B	EXT II-A	EXT II-B
1	4516.1	3526.4	3515.6	2486.1	4423.7	3422.6	3454.9	3653.1	2690.9	3606.4	2644.1
2	3915.2	2923.0	3068.7	2043.7	3769.5	2768.0	2952.8	3412.1	2449.9	3205.0	2242.8
3	3314.2	2319.7	2621.7	1601.3	3115.4	2113.4	2450.7	3171.1	2208.8	2803.7	1841.4
4	2052.4	1057.8	2166.5	1211.8	1942.2	959.0	1413.3	2252.4	1290.1	2299.2	1336.9
5	2653.3	1661.1	2613.6	1654.3	2596.3	1613.6	1915.4	2493.4	1531.1	2700.5	1738.2
6	3254.3	2264.5	3060.5	2096.7	3250.5	2268.2	2417.5	2734.4	1772.2	3101.8	2139.6
-	-	-	-	-	-	-	-	-	-	-	-
-											

BENDING MOMENT - Mx (KNm)											
Pile	STR I-A	STR I-B	STR II-A	STR II-B	STR III-A	STR III-B	SER	EXT I-A	EXT I-B	EXT II-A	EXT II-B
1	-69.1	-67.3	135.6	132.7	-0.3	0.2	3.9	16.9	16.9	101.3	101.3
2	69.1	67.3	-135.6	-132.7	0.3	-0.2	-3.9	-16.9	-16.9	-101.3	-101.3
3	69.1	67.3	-135.6	-132.7	0.3	-0.2	-3.9	-16.9	-16.9	-101.3	-101.3
4	69.1	67.3	-135.6	-132.7	0.3	-0.2	-3.9	-16.9	-16.9	-101.3	-101.3
5	69.1	67.3	-135.6	-132.7	0.3	-0.2	-3.9	-16.9	-16.9	-101.3	-101.3
6	-69.1	-67.3	135.6	132.7	-0.3	0.2	3.9	16.9	16.9	101.3	101.3
-	-	-	-	-	-	-	-	-	-	-	-
-											

BENDING MOMENT - My (KNm)											
Pile	STR I-A	STR I-B	STR II-A	STR II-B	STR III-A	STR III-B	SER	EXT I-A	EXT I-B	EXT II-A	EXT II-B
1	39.0	39.0	46.5	32.6	44.0	40.0	55.1	94.3	94.3	32.6	32.6
2	39.0	39.0	46.5	32.6	44.0	40.0	55.1	94.3	94.3	32.6	32.6
3	39.0	39.0	46.5	32.6	44.0	40.0	55.1	94.3	94.3	32.6	32.6
4	-39.0	-39.0	-46.5	-32.6	-44.0	-40.0	-55.1	-94.3	-94.3	-32.6	-32.6
5	-39.0	-39.0	-46.5	-32.6	-44.0	-40.0	-55.1	-94.3	-94.3	-32.6	-32.6
6	-39.0	-39.0	-46.5	-32.6	-44.0	-40.0	-55.1	-94.3	-94.3	-32.6	-32.6
-	-	-	-	-	-	-	-	-	-	-	-
-											

3.2. Pilecap weight

Section	V (m3)	Ni (KN)	Arm (m)	Mx (KNm)	My (KNm)
Mx1	14.0	-203.0	0.700	-142.1	
Mx2	14.0	-203.0	0.700	-142.1	
My1	28.8	-417.6	0.900		-375.8
My2	28.8	-417.6	0.900		-375.8

LOAD FACTOR FOR DEAD LOAD OF PILE CAP										
STR I-A	STR I-B	STR II-A	STR II-B	STR III-A	STR III-B	SER	EXT I-A	EXT I-B	EXT II-A	EXT II-B
1.25	0.90	1.25	0.90	1.25	0.90	1.00	1.25	0.90	1.25	0.90

3.3. Internal Forces at Critical Sections

INTERNAL FORCE AT SECTION Mx1 - Unit KN, KNm											
	STR I-A	STR I-B	STR II-A	STR II-B	STR III-A	STR III-B	SER	EXT I-A	EXT I-B	EXT II-A	EXT II-B
Shear	7516.6	5608.2	6322.3	4400.1	7420.5	5508.1	5669.3	6133.8	4280.3	6454.4	4600.9
Moment	2794.0	2055.2	2725.4	1971.5	2893.1	2150.0	2215.8	2412.7	1692.2	2709.7	1989.2

INTERNAL FORCE AT SECTION Mx2 - Unit KN, KNm											
	STR I-A	STR I-B	STR II-A	STR II-B	STR III-A	STR III-B	SER	EXT I-A	EXT I-B	EXT II-A	EXT II-B
Shear	5112.8	3194.8	4534.5	2630.4	4803.8	2889.7	3661.0	5169.7	3316.2	4849.1	2995.6
Moment	2108.4	1358.4	1467.5	732.6	1847.0	1101.3	1396.6	1959.0	1238.5	1661.9	941.5

INTERNAL FORCE AT SECTION My1 - Unit KN, KNm											
	STR I-A	STR I-B	STR II-A	STR II-B	STR III-A	STR III-B	SER	EXT I-A	EXT I-B	EXT II-A	EXT II-B
Shear	11223.5	8393.2	8684.0	5755.3	10786.6	7928.1	8440.8	9714.3	6973.7	9093.1	6352.4
Moment	9043.5	6793.9	7034.4	4664.4	8709.1	6425.1	6876.1	8002.1	5824.2	7320.1	5142.2

INTERNAL FORCE AT SECTION My2 - Unit KN, KNm											
	STR I-A	STR I-B	STR II-A	STR II-B	STR III-A	STR III-B	SER	EXT I-A	EXT I-B	EXT II-A	EXT II-B
Shear	7437.9	4607.6	7318.6	4586.9	7267.0	4465.0	5328.6	6958.2	4217.6	7579.5	4838.9
Moment	5781.2	3531.6	5663.3	3534.3	5629.4	3414.3	4055.9	5231.5	3053.6	5913.6	3735.7

4. One-way Shear capacity Check

S.5.8

Critical shear section for one-way shear is located at distance dv from face of equivalent square column.			
Estimated distance between internal flexural force components dv, we may take dv = 0.9*de			
de = H - cover - dx1	de	1.81	m
	dv	1.63	m

Direction			Long.- X	Trans.-Y	Unit
Shear resistance factors		ϕv	0.9	0.9	
Factored shear force in longitudinal		Vu	7517	11224	kN
Required shear capacity Vn = Vu/ ϕv		Vn	8352	12471	kN
Determine concrete shear capacity					
Minimum shear reinforcement will provided in cross section					
Therefore		β	2.0	2.0	
		θ	45.0	45.0	
Cross section	height	h	2.00	2.00	m
	width	b	5.00	8.00	m
d = h - cover - d1x		d	1.81	1.81	m
dv = max(0.72*h; 0.9*d)		dv	1.63	1.63	m
Vc = min(0.083 . β . sqrt(f'c) . bv. dv, 0.25.f'c.bv.dv)		Vc	7422	11875	kN
Difference between required shear capacity and the capacity provided by concrete					
is the minimum required capacity for shear reinforcements					
Vs = Vn - Vc		Vs	930	595	kN
In this case Vc > Vn so shear reinforcement is no need					
Stirrup diameter		Ds	16	16	
Number of stirrup legs / cross section		ns	20	36	
Shear legs area		Av	0.0040	0.0073	m2
Angle of inclination of shear reinf. to long. axis		α	90	90	deg
Vs = Av. fy. dv. (cot θ + cot α). sin α / s		s <=	0.21	0.60	m
Stirrup spacing used		s	0.30	0.30	m
Check minimum shear reinforcement requirement			**OK**	**OK**	
Av >= 0.083. sqrt(f'c). bv. s/ fy = Req		Req	0.0017	0.0027	m2
Check maximum shear reinforcement spacing requirement			**OK**	**OK**	
F = 0.1*f'c*bv*dv		F	24489	39182	kN
If Vu < 0.1*f'c*bv*dv then Smax=0.8*dv <= 600mm					
If Vu > 0.1*f'c*bv*dv then Smax=0.4*dv <= 300mm		Smax	0.60	0.60	m

5. Two-way Shear capacity Design S.5.13.3.6.3

Assume the entire column vertical load needs to be carried at the perimeter.
Two-way shear is evaluated on a perimeter located dv/2 away from the face of the actual pier column.
The same dimension dv/2 is used to check two-way shear for a corner pile.

Column v.s Pilecap

Pier Column dimensions	Longitudinal axis	td	5.50	m
	Transverse axis	tn	1.40	m
Perimeter of two-way shear				
b0 = (td+tn)*2 + 4*dv		b0	18.28	m
Section with transverse reinforcement				
Norminal shear resistance shall be taken as				
Vn = Vc + Vs <= 0.504.sqrt(f'c). b0 . dv = Va				
Vc = 0.166 . sqrt(f'c). b0 . dv				
Vs = Av. fy . dv / s				
Shear resistance of concrete		Vc	27136	kN
Assumed stirrup diameter		Ds	16	mm
Number of stirrup legs / cross section		ns	30	
Shear legs area		Av	0.0061	m2
Stirrup spacing used		s	600	mm
Shear resistance of reinforcement		Vs	6596	kN
		Va	82390	kN
		Vn	33732	kN
Maximum reaction at bottom of column		Vu	17258	kN
Resistance factor for shear		ϕv	0.9	
Factored shear resistance		ϕv*Vn	30359	kN
Punching shear check			**OK**	

Conner pile v.s Pilecap

Pile diameter	D	1.00	m
Radius of critical section for two-way shear Rco = D/2 + dv/2	Rco	1.32	m
Distance from pile center of conner pile to edge of pilecap	a1	1.00	m
Perimeter of two-way shear			
b0 = 2*a1 + 1/4*2*pi()*Rco	b0	4.07	m
Section with transverse reinforcement			
Norminal shear resistance shall be taken as			
Vn = Vc + Vs <= 0.504.sqrt(f'c). b0 . dv = Va			
Vc = 0.166 . sqrt(f'c). b0 . dv			
Vs = Av. fy . dv / s			
Shear resistance of concrete	Vc	6038	kN
Assumed stirrup diameter	Ds	16	mm
Number of stirrup legs / cross section	ns	13	
Shear legs area	Av	0.0026	m2
Stirrup spacing used	s	300	mm
Shear resistance of reinforcement	Vs	5716	kN
	Va	18332	kN
	Vn	11754	kN
Maximum reaction of conner pile	Vu	4516	kN
Resistance factor for shear	ϕv	0.9	
Factored shear resistance	ϕv*Vn	10579	kN
Punching shear check		**OK**	

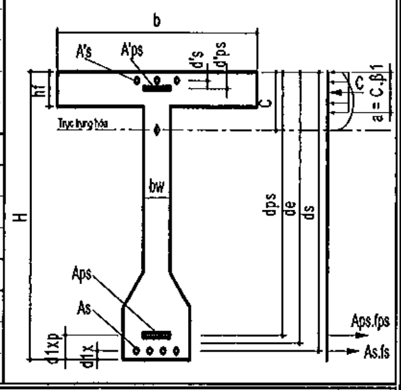
	DA NANG - QUANG NGAI EXPRESS WAY PROJECT Bridge LRB09	Item.	Eng.	Date.	Sign.
		Design			
	DETAIL DESIGN Pier P2	Check			
		Revise			

22TCN272-05; AASHTO LRFD 2nd - 1998

REINFORCEMENT CHECKING - PILE CAP

MATERIALS

	NORMAL CONCRETE		
f'c	Compressive Strength of concrete at 28 days	Mpa	30
Ec	Modulus of Elasticity	Mpa	27691
fr	Modulus of Rupture	Mpa	3.5
gc	Unit weight of concrete	kN/m3	24.5
	PRESTRESSING STEEL		
fpu	Tensile strength of prestressing steel	Mpa	1860
fpy	Yield strength of prestressing steel	Mpa	1670
Ep	Modulus of Elasticity	Mpa	195000
	REINFORCEMENT		
fy	Yield strength	Mpa	400
Es	Modulus of Elasticity	Mpa	200000
nc	Ratio Es/Ec		7.00



Sign	Parameters		Unit	Section - Transverse section				
				Mx	Mx	My	My	
	INTERNAL FORCES AT SECTIOIN							
	Combination			Strength	Service	Strength	Service	
Qu	Shear		kN	7517	5669	11224	8441	
Mu	Flexural Moment		kNm	2893	2216	9044	6876	
Nu	Axial load		kN					
Tu	Torsional Moment		kNm					
	FLEXURAL MOMENT CHECKING							
H	Section height		m	2.000	2.000	2.000	2.000	
d's	Dis. From comp. fiber to centroid of comp. Reinf		m	0.084	0.084	0.084	0.084	
d1x	Dis. From tens. fiber to centroid of tension Reinf		m	0.160	0.160	0.163	0.163	
	Cover to reinf		m	0.050	0.050	0.050	0.050	
ds	Dis. From comp. fiber to centroid of tension Reinf		m	1.840	1.840	1.838	1.838	
d'ps	Dis. From comp. fiber to centroid of comp. prestressing steel		m	0.000	0.000	0.000	0.000	
d1xp	Dis. From tens. fiber to centroid of tension prestressing steel		m	0.000	0.000	0.000	0.000	
dps	Dis. From comp. fiber to centroid of tension prestressing steel		m	2.000	2.000	2.000	2.000	
b	Width of the compression face of member		m	5.000	5.000	8.000	8.000	
bw	Web width or diameter of a circular section		m	5.000	5.000	8.000	8.000	
hf	Compression flange depth		m	0.000	0.000	0.000	0.000	
Iz	Moment of inertia of section		m4	3.333	3.333	5.333	5.333	
Amc	Section area		m2	10.000	10.000	16.000	16.000	
	Steel choice							
Aps	Tension prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	
		Number	tendons	0	0	0	0	
		Area	m2	0.00000	0.00000	0.00000	0.00000	
A'ps	Compression prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	
		Number	tendons	0	0	0	0	
		Area	m2	0.00000	0.00000	0.00000	0.00000	
As	Tension Reinforcement	Number	bars	33	33	53	53	
		Diameter	mm	20	20	25	25	
		Area	m2	0.01036	0.01036	0.02602	0.02602	
A's	Compression Reinforcement	Number	bars	33	33	53	53	
		Diameter	mm	18	18	18	18	
		Area	m2	0.00838	0.00838	0.01346	0.01346	
A'c	Shear reinforcement	Number	bars	8	8	13	13	
		Diameter	mm	16	16	16	16	
		Area	m2	0.00162	0.00162	0.00263	0.00263	

ϕ	Resistance factors for flexure	5.5.4.2	0.90	1.00	0.90	1.00	
ϕv	Resistance factors for shear		0.90	1.00	0.90	1.00	
ϕn	Resistance factors for axial force		1.00	1.00	1.00	1.00	
β1	Stress block factor		0.836	0.836	0.836	0.836	
c	Dis. Between centroid and top fiber	m	0.007	0.007	0.029	0.029	
	For T section behavior	m	0.007	0.007	0.029	0.029	
	For rectangular section behavior	m	0.007	0.007	0.029	0.029	
fpe	Effective stress in the prestressing steel after losses	Mpa	1116	1116	1116	1116	
fps	Aver. stress in pres. steel at the time for which the nominal resistanc	Mpa	1858	1858	1852	1852	
k	Factor depends on type of P.S, Low relaxation strand k = 0.28		0.28	0.28	0.28	0.28	
a	Depth of equivalent stress block	m	0.006	0.006	0.025	0.025	
de	Corresp. effective depth from extreme comp. fiber						
	to centroid of tensile force in the tensile reinf.	m	1.840	1.840	1.838	1.838	
Mn	Nominal resistance	kNm	7342	7342	18613	18613	
Mr	Factored reistance	kNm	**6608**	**7342**	**16751**	**18613**	
Mu	Flexual moment	kNm	**2893**	**2216**	**9044**	**6876**	
(5.7.3.2)	**Flexural moment Checking**		**OK**	**OK**	**OK**	**OK**	
	Limits for reinforcement						
c/de	Maximum reinforcement		0.00	0.00	0.02	0.02	
	Maximum reinforcement Checking	<= 0.42	**OK**	**OK**	**OK**	**OK**	
1.2*Mcr	Craking moment	kNm	6927	6927	11207	11207	
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**		**OK**	**OK**	**OK**	**OK**	
(5.7.3.4)	**Conctrol of craking by distr. of reinf for RC member- Check?**		No	Yes	No	Yes	
	Existing condition for structrure	1,2 or 3	3	3	3	3	
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.060	0.060	0.063	0.063	
Z	Crack width parameter	N/mm	17500	17500	17500	17500	
A	Area of concr. with same centroid as tens. Reinf	m2	0.018	0.018	0.019	0.019	
fsa	Value	Mpa	170	170	166	166	
0.6*fy		Mpa	240	240	240	240	
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	170	170	166	166	
x	Dist. From compression fiber to centroid	m	-	0.217	-	0.267	
J.d	Arm	m	-	1.768	-	1.748	
Icr	Moment of inertia of the cracked section	m4	-	0.209	-	0.503	
fs	Tensile stress in reinforcement fs = Msls / (As*J.d)	Mpa	-	121	-	151	
	Checking for control cracking fs<fsa		**N.a**	**OK**	**N.a**	**OK**	
(5.10.8.2)	**Shrinkage and temperature Reinfocement (side distribution)**						
Areq	Area of required reinf	m2	0.00127	0.00127	0.00127	0.00127	
	Distribution on sides 10 D16	m2	0.00202	0.00202	0.00202	0.00202	
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45	
	Checking		**OK**	**OK**	**OK**	**OK**	

SHEAR AND TORSION CHECKING

β	Factor indicating diag. cracked concr. to tension		1.8	1.9	1.9	2.0	
θ	Angle of inclination of diagonal compressive	degree	42.46	41.17	41.69	39.81	
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	
bv	Effective web width as minimum web width - in dv	m	5.000	5.000	8.000	8.000	
dv	Effective shear depth	m	1.837	1.837	1.825	1.825	
	(de - a/2)	m	1.837	1.837	1.825	1.825	
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	
ncat	Amount of bars in spacing S	bars	8	8	13	13	
Av	Shear reinf area in spacing S	m2	0.0016	0.0016	0.0026	0.0026	
β	Assume		2.0	2.0	2.0	2.0	
θ	Assume	degree	42.04	38.67	41.01	36.75	
v	Shear stress in concrete	kN/m2	909	617	854	578	
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	
e_x	Strain in tensile reinforcement		1.87E-03	1.54E-03	1.67E-03	1.38E-03	
	if ex<0, multiple with reduce factor		-	-	-	-	
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok	
v/f'c	Ratio of shear stress and f'c		0.030	0.021	0.028	0.019	
β	Final value		1.8	1.9	1.9	2.0	
θ	Final value	degree	42.46	41.17	41.69	39.81	
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	7439	8060	12417	13386	
Vs	Shear resistance provided by shear reinforcement	kN	2162	2263	3587	3834	
Vp	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0	
Vn1	Vn1=Vc+Vs+Vp	kN	9602	10323	16004	17220	
Vn2	Vn2	kN	68884	68884	109511	109511	
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	9602	10323	16004	17220	
Vr	Factored shear resistance	kN	**8642**	**10323**	**14404**	**17220**	
Vu	Shear	kN	7517	5669	11224	8441	
(5.8.2.7)	**Shear checking**		**OK**	**OK**	**OK**	**OK**	

F.BORED PILE DESIGN

I.BORED PILE DATA

1.Load Combinations at top of bored pile

STT	Comb	Axial force P (KN)	Moment (KN.m)		
			Mx	My	Mxy
STRENGTH LIMIT STATES					
1	P_min	959	0	40	40
2	P_max	4516	69	39	79
3	Mx_max	3516	136	46	143
4	My_max	3455	4	55	55
EXTREME EVENT LIMIT STATES					
1	P_min	1290	17	94	96
2	P_max	3653	17	94	96
3	Mx_max	3606	101	33	106
4	My_max	3653	17	94	96

2. Bored pile Material

Normal concrete			
Compressive strength at 28 days age	f'c	30	MPa
Concrete elastic modulus	Ec	27691	MPa
Reinforcement TCVN1651-2008; CBV-400			
Yield strength	fy	400	MPa
Reinforcement elastic modulus	Es	200,000	MPa

3. Bored pile Section

Pile diameter	D	1.00	m
Section area	A	0.785	m2
Moment inertia	Ix	0.049	m4
	Iy	0.049	m4
Radius of gyration of gross concrete section; r = sqrt(I/A)	rx	0.250	m
	ry	0.250	m

II.PILE DESIGN

1. Limit of Reinforcement S.5.7.4.2

Minimum area of longitudinal reinforcement in column				
As.fy / (Ag . f'c) >= 0.135		As ≥	0.008	m2
As / Ag >= 0.01		As ≥	0.008	m2
Maximum area of longitudinal reinforcement in column				
As / Ag <= 0.08		As ≤	0.063	m2
Trial Rebars:	Ok	As	**0.010**	m2
1layers x 20 = 20 bars	**D25 @150**	**As1**	0.010	m2

2. Interation diagram M-P Using Pca-Column software

****Flexural check by pcaColumn**

Strength and Service limit states:

Resistance factor: Compression φ_c = 0.75 *(AASHTO LRFD-2004)*

Tension φ_t = 0.90

Extreme Event limit states:

Resistance factor Compression φ_c = 1.00

Tension φ_t = 1.00

<Result table>

STT	Comb	Pu kN	Mux kN-m	Muy kN-m	φ Mnx kN-m	φ Mny kN-m	φ Mn/Mu
STRENGTH LIMIT STATES							
1	P_max	959	0.2	40	8	1593.4	39.836
2	P_min	4516.1	69.1	39	1830.1	1032.9	26.485
3	Mx_max	3515.6	135.6	46.5	1977.6	678.2	14.584
4	My_max	3454.9	3.9	55.1	146.9	2076	37.678
EXTREME EVENT LIMIT STATES							
1	P_max	1290	17	94	331.6	1833.8	19.508
2	P_min	3653	17	94	428.1	2366.9	25.18
3	Mx_max	3606	101	33	2271.3	742.1	22.488
4	My_max	3653	17	94	428.1	2366.9	25.18

3. Column Ties S.5.7.4.6, S.5.10.6.3, S5.10.11.4.1d - e

Bridge is in seismic zone	Sz	1	
Area of concrete core measured out-to-out of ties	Ac	0.622	m2
Tie diameter	Dtie	14	mm2
Cross section area of 1 tie	As-tr	0.00015	m2
Spacing of hoops	s	75	mm
Length of reinfocement tie in 1 hoop	Ltie	2.78	m
Ratio of ties reinf. in one hoop/ volume of conc. core one pitch spacing			
ρs = As-tr . Ltie / (Ac * spacing)	ρs	0.0090	
Ratio of spiral reinf. To total volume of concrete core shall satisfy			S.5.7.4.6
ρs >= 0.45 . (Ag/Ac - 1). f'c/ fy = Req1	Req1	0.0089	**OK**
Transverse Reinforcement for Confinement at Plastic Hinges		S.5.10.11.4.1.d	
For a circular column "1:applied","2:Not applied"		1	
ρs >= 0.12 . f'c/ fy = Req2	Req2	0.0090	N/A
Length distributed spiral with pitch 75mm below pilecap	Ldis	1.50	m

4. Shear Design

Shear resistance factors	ϕv	1.0	
Factored shear force	Vu	8	kN
Required shear capacity Vn = Vu/ ϕv	Vn	8	kN
Determine concrete shear capacity			
Minimum shear reinforcement will provided in cross section			
Therefore	β	2.0	
	θ	45.0	deg
Diameter of bored pile	D	1.00	m
Width of cross section	b	1.00	m
dv = 0.9*de de = D/2 + Dr/pi()			
Diameter of the circle passing through the centers of the long. reinf.	Dr	0.79	m
	de	0.75	m
	dv	0.68	m
Vc = 0.083 . β . sqrt(f'c) . bv. dv	Vc	616	kN
	Conclude		**OK**

Da Nang Quang Ngai Expressway project

Bridge LRB09

CALCULATION SHEETS

Pier P1

2013

A.STRUCTURE DIMENSIONS & LOAD COMPONENTS

I. GENERAL DATA

Assumptions :

1.The Design of the Pier conforms to "Specification for bridge design 22-TCN-272-05" and AASHTO LRFD 2004, JIS,... for reference.
2.Design live load: HL-93 and lane loading 9.3 KN/m
3.Bridge is considered to be in seismic with acceleration coefficient A = 0.0310 g

Input data:

Bridge type	*Simple PC I girder L=33m with link slab*					
Span length	Left	=	33.05	Right	=	33.05 m
Girder length between bearings	Left	=	32.10	Right	=	32.10 m
Bridge width	B	=	12.75 m			

Level Table(at center of pier)

Top of pier cap	ThL	16.886	m
Top of pier column	TcL	11.501	m
Bottom of upper pier column	H_{topc}	11.500	m
Bottom of upper pilecap	H_{up}	2.000	m
Bottom of pilecap	H_{bot}	0.500	m
Skew angle	Ska	70.000	deg
Ground level	GL	6.000	m
Maximum water level (H1%)	H_{max}	13.010	m
Navigation water level (H5%)	H_{min}	8.500	m
Average Annual water level	H_{ave}	10.755	m
Local scour level (at water level H1%)	H_{sc}	5.500	m

Material unit weight

Concrete

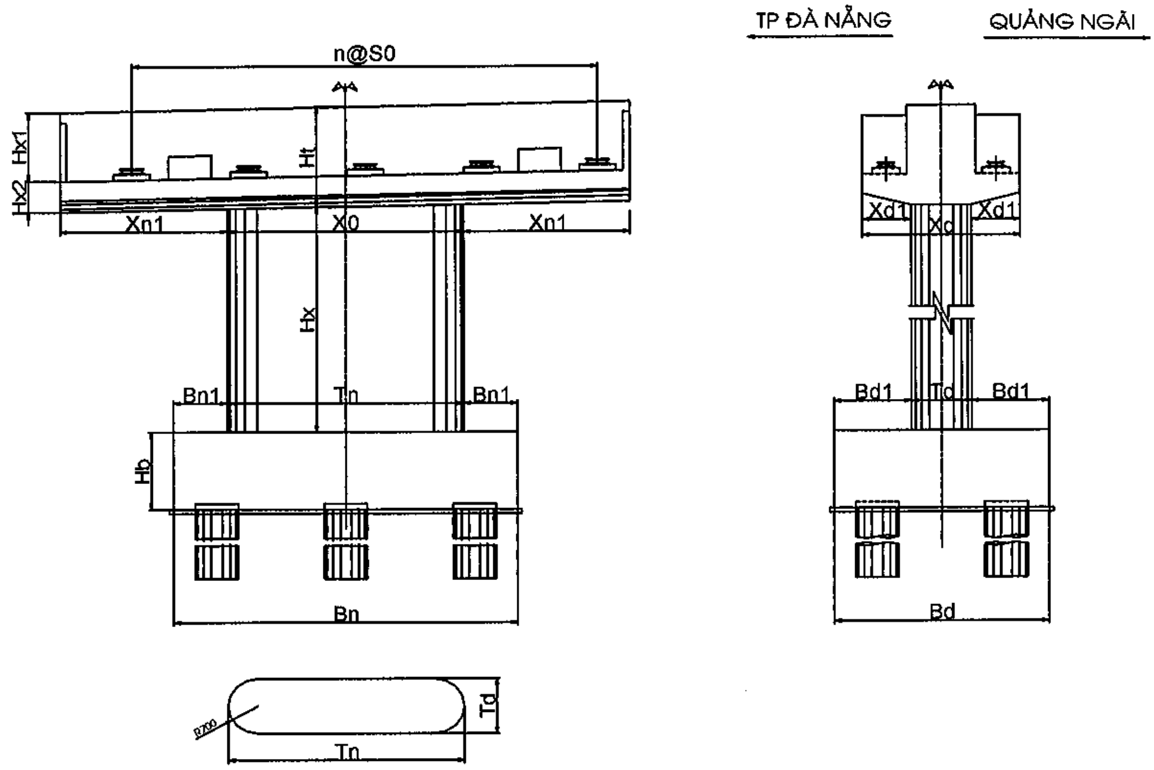
Specify density	γ_c	=	24.5 kN/m3
Compressive Strength (at 28 days)	f'_c	=	30.0 MPa
Elastic Modulus	E_c	=	29440 Mpa

Reinforcement

Yield strength	f_y	=	400.0 MPa
Modulus of elasticity	E_s	=	200000 Mpa
Modular ratio (steel/concrete)	$n = E_s/E_c$	=	6.7935

Asphalt concrete	γa	=	22.1 kN/m3
Soil - ground	γ_s	=	17.7 kN/m3
Saturated soil	γ_{ss}	=	7.8 kN/m3

II. PIER DIMENSIONS



Pier Dimensions Table

Notation	Dimensions Transverse direction	Value (m)	Notation	Dimensions Longitudinal direction	Value (m)
*	***Bearing distribution***		*	***Bearing pedestal*** Width	0.85
nbear	Number of bearing	5.00		Length	0.65
nbear	Number of bearing	5.00		Height	0.15
S_l	Bearings spacing	2.55	*	***Anchorage block*** Width	0.40
S_r	Bearings spacing	2.55		Length	1.00
b_{s1}	Total width of bridge CS	12.75		Height	0.52
b_{s2}	Carriage way width	11.76		Dist. CB's edge to exterior girder	1.27
b_{s3}	Left curb width	0.50		Dist. CB's edge to exterior girder	1.27
b_{s4}	Right curb width	0.49			
*	***Pier Cap***				
H_{x1}	Haunch 1 height	1.935	X_d	Pier cap width	3.60
H_{x2}	Haunch 2 height	1.00	X_{d1}	Pier cap width	1.00
H_x	Pier cap height	2.935	GL	Left bearing to pier c.line	1.270
X_{n1}	Haunch width	3.89	GR	Right bearing to pier c.line	1.270
X_{n0}	Bottom of pier cap width	5.50	H_c	Curtain wall height	1.80
X_{nt}	Top of pier cap width	13.28	T_c	Curtain wall thickness	0.15
*	***Pier Column***				
T_n	Pier column width	5.50	T_d	Pier column thichness	1.40
H_{tt}	Pier column height	9.50	Rv	Round nose radius	0.70
T_{nb}	Upper pier column width	0.00	T_{db}	Upper pier column thichness	0.00
H_{tb}	Upper pier column height	0.00	R_{vb}	Upper round nose radius	0.00
H	Column height	9.50			
*	***Pile Cap***				
B_n	Pile cap width	8.00	B_d	Pile cap length	6.00
H_b	Pile cap depth	1.50			
B_{nb}	Upper pile cap width	0.00	B_{db}	Upper pile cap length	0.00
H_{bb}	Upper pile cap depth	0.00			

III. SUBSTRUCTURE LOADS

1. Pier Selfweight

Item	Volume	Vertical	Longitudinal			Transversal		
		F_V	F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m3)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Bearing pedestal	0.83	20.3						
Bearing devices		50.0						
Anchorage block	0.84	20.5						
Pier Cap	88.92	2178.6						
Curtain wall	0.50	12.1						
Upper pier column	0.00	0.0						
Pier Column	69.15	1694.3						
Upper pilecap	0.00	0.0						
PileCap	72.00	1764.0						
Shear key	0.00	0.0						
Total at bottom of Column		***3975.9***						
Total at bottom of pilecap		***5739.9***						

2. Soil on pilecap

Item	Volume	Vertical	Longitudinal			Transversal		
		F_V	F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m3)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Soil on pile cap	162.88	1270.5						
Total at bottom of Column								
Total at bottom of pilecap		***1270.5***						

3. Buoyancy on pier

Case1: Maximum water level (H1%)

Item	Volume	Vertical	Longitudinal			Transversal		
		F_V	F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m3)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Upper pier column	0.00	0.0						
Pier Column	69.15	-678.4						
Upper pilecap	0.00	0.0						
PileCap	72.00	-706.3						
Shear key	0.00	0.0						
Total at bottom of Column		***-678.4***						
Total at bottom of pilecap		***-1384.7***						

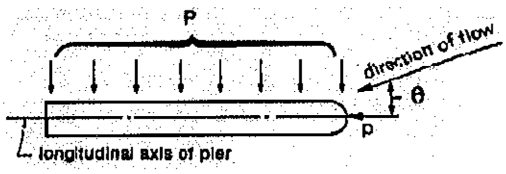
Case2: Minimum water level (Hmin)

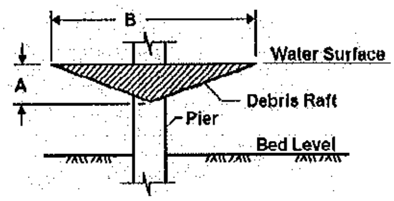
Item	Volume	Vertical	Longitudinal			Transversal		
		F_V	F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m3)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Upper pier column	0.00	0.0						
Pier Column	47.32	-464.2						
Upper pilecap	0.00	0.0						
PileCap	72.00	-706.3						
Shear key	0.00	0.0						
Total at bottom of Column		***-464.2***						
Total at bottom of pilecap		***-1170.5***						

Case3: average Annual water level

Item	Volume	Vertical F_V	Longitudinal F_{HX}	Longitudinal $Arm._{HX}$	Longitudinal M_y	Transversal F_{HY}	Transversal $Arm._{HY}$	Transversal M_x
	(m3)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Upper pier column	0.00	0.0						
Pier Column	63.73	-625.2						
Upper pilecap	0.00	0.0						
PileCap	72.00	-706.3						
Shear key	0.00	0.0						
Total at bottom of Column		***-625.2***						
Total at bottom of pilecap		***-1331.5***						

4.Stream Pressure





Stream pressure data

Angle between direction of flow and long. axis of pier	θ	0.0	deg
Design velocity of water at H1%	V1%	1.11	m/s
Design velocity of water at minimum water level	Vmin	0.39	m/s
Design velocity of water at average annual water level	Vannual	1.11	m/s

Longitudinal axis of pier $pL = 5.14 \times 10^{-4} C_D V^2$

Type of pier colum: "1:Semicircular - nosed pier"; "2:Square - ended pier";		1	
"3:Debris lodged against the pier "; "4:Wedged - nosed pier with nose angle 90deg or less"			
Drag coefficient	C_D	0.70	
Stream pressure at H1%	pL1%	0.44	kN/m2
Stream pressure at minimum water level	pLmin	0.05	kN/m2
Stream pressure at average annual water level	pLannual	0.44	kN/m2
Additional stream pressure due to driftwood raft lodged against pier			
Drag coefficient	C_D	0.50	
Height of debris raft	A	3.0	m
Width of debris raft	B	14.0	m
Area of debris raft	Area	21.0	m2
Stream pressure due to driftwood raft at H1%	pLdebris	0.32	kN/m2
Equivalent force	Fhdebris	6.6	kN

Lateral axis of pier $pT = 5.14 \times 10^{-4} C_L V^2$

Drag coefficient	C_L	0.00	
Stream pressure at H1%	pT1%	0.00	kN/m2
Stream pressure at minimum water level	pTmin	0.00	kN/m2
Stream pressure at average annual water level	pTannual	0.00	kN/m2

Case1: Maximum water level (H1%)

Item	Exposed height	Vertical F_V	Longitudinal			Transversal		
			F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Upper pier Column	1.51		0.0	10.3	0.0	0.0	10.3	0.0
Pier Column	9.50		0.0	4.8	0.0	5.9	4.8	28.0
Upper pilecap	0.00		0.0	0.0	0.0	0.0	0.0	0.0
Total at bottom of Column			***0.0***		***0.0***	***5.9***		***28.0***
Upper pier Column	1.51		0.0	11.8	0.0	0.0	11.8	0.0
Pier Column	9.50		0.0	6.3	0.0	5.9	6.3	36.9
Upper pilecap	0.00		0.0	1.5	0.0	0.0	1.5	0.0
PileCap	0.00		0.0	0.0	0.0	0.0	0.0	0.0
Total at bottom of pilecap			***0.0***		***0.0***	***5.9***		***36.9***

Additional stream pressure due to driftwood raft

Item	Exposed height	Vertical F_V	Longitudinal			Transversal		
			F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Pier Column					0.0	6.6	11.0	73.2
Total at bottom of Column					***0.0***	***6.6***		***73.2***
Pier Column					0.0	6.6	12.5	83.2
Total at bottom of pilecap					***0.0***	***6.6***		***83.2***

Case2: Minimum water level (Hmin)

Item	Exposed height	Vertical F_V	Longitudinal			Transversal		
			F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Upper pier Column	0.00		0.0	9.5	0.0	0.0	9.5	0.0
Pier Column	6.50		0.0	3.3	0.0	0.5	3.3	1.6
Upper pilecap	0.00		0.0	0.0	0.0	0.0	0.0	0.0
Total at bottom of Column			***0.0***		***0.0***	***0.5***		***1.6***
Upper pier Column	0.00		0.0	11.0	0.0	0.0	11.0	0.0
Pier Column	6.50		0.0	4.8	0.0	0.5	4.8	2.3
Upper pilecap	0.00		0.0	1.5	0.0	0.0	1.5	0.0
PileCap	0.00		0.0	0.0	0.0	0.0	0.0	0.0
Total at bottom of pilecap			***0.0***		***0.0***	***0.5***		***2.3***

Case3: average Annual water level

Item	Exposed height	Vertical F_V	Longitudinal			Transversal		
			F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Upper pier Column	0.00		0.0	9.5	0.0	0.0	9.5	0.0
Pier Column	8.75		0.0	4.4	0.0	5.4	4.4	23.8
Upper pilecap	0.00		0.0	0.0	0.0	0.0	0.0	0.0
Total at bottom of Column			***0.0***		***0.0***	***5.4***		***23.8***
Upper pier Column	0.00		0.0	11.0	0.0	0.0	11.0	0.0
Pier Column	8.75		0.0	5.9	0.0	5.4	5.9	31.9
Upper pilecap	0.00		0.0	1.5	0.0	0.0	1.5	0.0
PileCap	0.00		0.0	0.0	0.0	0.0	0.0	0.0
Total at bottom of pilecap			***0.0***		***0.0***	***5.4***		***31.9***

Wind loads data	$P_D = 0.0006\ V^2\ C_d \geq 1.8$ (kN/m2)		
Reference standard "Wind region TCVN 2737-1995"			
Wind region (I, II, III, IV) at bridge location		III	
3 second gust wind velocity with 100 years return period	Vb	53.0	m/s
Type of terrain at bridge location		1	
"1:exposed area"; "2: forest, houses,... with height 10m"; "3:houses area..with height>10m"			
Average elevation of pier upper ground or water plane level	Hele_p	8.7	m
Correct coefficient for wind zone and elevation of pier	S	1.09	
Design wind speed V = S*Vb	V	57.8	m/s
Obstacle coefficient for pier	Cd	1.4	
Wind pressure on pier	P_D	2.80	kN/m2

At Maximum water level (H1%)

Item	Exposed height	Vertical	Longitudinal			Transversal		
		F_V	F_{HX}	Arm.$_{HX}$	M_y	F_{HY}	Arm.$_{HY}$	M_x
	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Curtain wall	0.00		0.0	14.9	0.0	0.0	14.9	0.0
Pier Cap	2.94		0.0	11.0	0.0	29.6	11.0	324.9
Upper pier Column	0.00		0.0	11.0	0.0	0.0	11.0	0.0
Pier Column	0.00		0.0	11.0	0.0	0.0	11.0	0.0
Upper pilecap	0.00		0.0	11.0	0.0	0.0	11.0	0.0
Total at bottom of Column			***0.0***		***0.0***	***29.6***		***324.9***
Curtain wall	0.00		0.0	16.4	0.0	0.0	16.4	0.0
Pier Cap	2.94		0.0	12.5	0.0	29.6	12.5	369.3
Upper pier Column	0.00		0.0	12.5	0.0	0.0	12.5	0.0
Pier Column	0.00		0.0	12.5	0.0	0.0	12.5	0.0
Upper pilecap	0.00		0.0	12.5	0.0	0.0	12.5	0.0
PileCap	0.00		0.0	12.5	0.0	0.0	12.5	0.0
Total at bottom of pilecap			***0.0***		***0.0***	***29.6***		***369.3***

At Minimum water level (Hmin)

Item	Exposed height	Vertical	Longitudinal			Transversal		
		F_V	F_{HX}	Arm.$_{HX}$	M_y	F_{HY}	Arm.$_{HY}$	M_x
	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Curtain wall	0.00		0.0	14.9	0.0	0.0	14.9	0.0
Pier Cap	2.94		0.0	11.0	0.0	29.6	11.0	324.9
Upper pier Column	0.00		0.0	6.5	0.0	0.0	6.5	0.0
Pier Column	3.00		46.3	8.0	370.0	11.8	8.0	94.2
Upper pilecap	0.00		0.0	6.5	0.0	0.0	6.5	0.0
Total at bottom of Column			***46.3***		***370.0***	***41.4***		***419.1***
Curtain wall	0.00		0.0	16.4	0.0	0.0	16.4	0.0
Pier Cap	2.94		0.0	12.5	0.0	29.6	12.5	369.3
Upper pier Column	0.00		0.0	8.0	0.0	0.0	8.0	0.0
Pier Column	3.00		46.3	9.5	439.4	11.8	9.5	111.9
Upper pilecap	0.00		0.0	8.0	0.0	0.0	8.0	0.0
PileCap	0.00		0.0	8.0	0.0	0.0	8.0	0.0
Total at bottom of pilecap			***46.3***		***439.4***	***41.4***		***481.2***

At average Annual water level

Item	Exposed height	Vertical F_V	Longitudinal			Transversal		
			F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Curtain wall	0.00		0.0	14.9	0.0	0.0	14.9	0.0
Pier Cap	2.94		0.0	11.0	0.0	29.6	11.0	324.9
Upper pier Column	0.00		0.0	8.8	0.0	0.0	8.8	0.0
Pier Column	0.75		11.5	9.1	104.8	2.9	9.1	26.7
Upper pilecap	0.00		0.0	8.8	0.0	0.0	8.8	0.0
Total at bottom of Column			***11.5***		***104.8***	***32.5***		***351.6***
Curtain wall	0.00		0.0	16.4	0.0	0.0	16.4	0.0
Pier Cap	2.94		0.0	12.5	0.0	29.6	12.5	369.3
Upper pier Column	0.00		0.0	10.3	0.0	0.0	10.3	0.0
Pier Column	0.75		11.5	10.6	122.1	2.9	10.6	31.1
Upper pilecap	0.00		0.0	10.3	0.0	0.0	10.3	0.0
PileCap	0.00		0.0	10.3	0.0	0.0	10.3	0.0
Total at bottom of pilecap			***11.5***		***122.1***	***32.5***		***400.4***

6.Vessel Collision

7.Vehicular Collision Force "1 yes", "0 no" 0

IV. SUPERSTRUCTURE LOADS

1. Dead Loads

Left side Span

Item	Volume	Vertical	Longitudinal			Transversal		
		F_V	F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m3)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Stage1 (DC)								
Girders	136.65	1674.0		1.270	-2125.9			
Diaphragm	18.23	223.4		1.270	-283.7			
Precast plank	16.89	206.9		1.270	-262.7			
Deck slab	103.68	1270.0		1.270	-1612.9			
Total		***3374.2***			***-4285.2***			
Stage2 (DW)								
Pavement	35.98	397.6		1.270	-504.9			
Parapet + railing		391.1		1.270	-496.6			
Lighting post + mis.		33.0		1.270	-41.9			
Total		***821.6***			***-1043.5***			

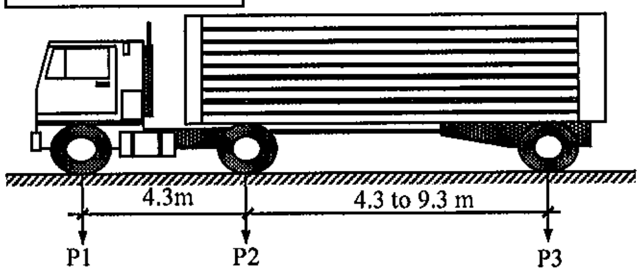
Right side Span

Item	Volume	Vertical	Longitudinal			Transversal		
		F_V	F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m3)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Stage1 (DC)								
Girders	136.65	1674.0		1.270	2125.9			
Diaphragm	18.23	223.4		1.270	283.7			
Precast plank	16.89	206.9		1.270	262.7			
Deck slab	103.68	1270.0		1.270	1612.9			
Total		***3374.2***			***4285.2***			
Stage2 (DW)								
Pavement	35.98	397.6		1.270	504.9			
Parapet + railing		391.1		1.270	496.6			
Lighting post + mis.		33.0		1.270	41.9			
Total		***821.6***			***1043.5***			

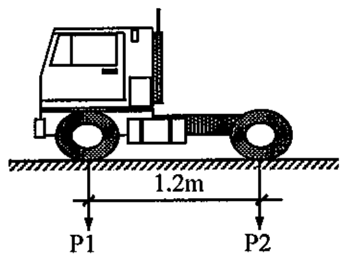
Live load data

Design Truck	P1	35.0KN	V1	4.3 m
	P2	145.0KN	V2	4.3 m
	P3	145.0KN		
Design Tandem	P1	110.0KN	V3	1.2 m
	P2	110.0KN		
Design Lane Load	P_L	9.3 KN/m		
Pedestrian Load	Pp	0.0		
Sidewalk width - both 2 sides	sw	0.0		
Maximum number of design lane	nlanes	3.0		
Multiple presence factor	m	0.85		
Dynamic load allowance		(1+IM)		
Deck joint - all limit states		1.75		
Other structure - all limit states (except fatigue)		1.25		

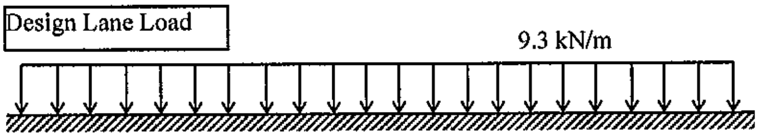
Design Truck



Design Tandem

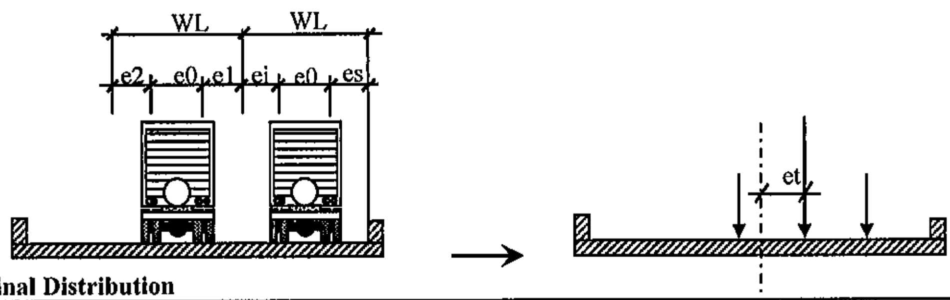


Design Lane Load



Transverse Distribution

Distance from wheel axis to inner face of curb and bewteen wheel axis						
In general case	ei	1.20	m	es	0.60	m
For deck overhang design	ei	1.30	m	es	0.50	m
Distance between wheels				e0	1.80	m
Design lane width				WL	3.60	m
				e1	0.00	m
				e2	1.80	m
Curb width				wc	0.50	m
Transverse excentricity of design vehicle 1 - general case				ex1	4.38	
Transverse excentricity of design vehicle 2				ex2	2.58	
Transverse excentricity of design vehicle 3				ex3	0.78	
Transverse excentricity of design vehicle 4				ex4	-1.03	
Transverse Excentricity of design vehicle				et	1.68	m



Longitudinal Distribution

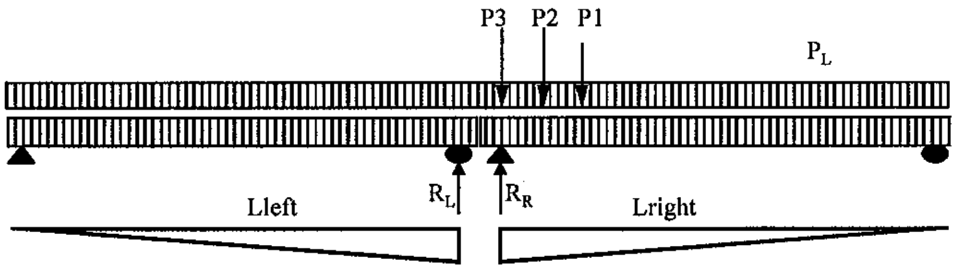
Case1a: 1Truck + Lane load on 1 span
Case1b: 1Truck + Lane load on 2 spans
Case2a: 2Trucks + Lane load on 1 span
Case2b: 2Trucks + Lane load on 2 spans
Case3a: 1Tandem + Lane load on 1 span
Case3b: 1Tandem + Lane load on 2 spans

Influence line value

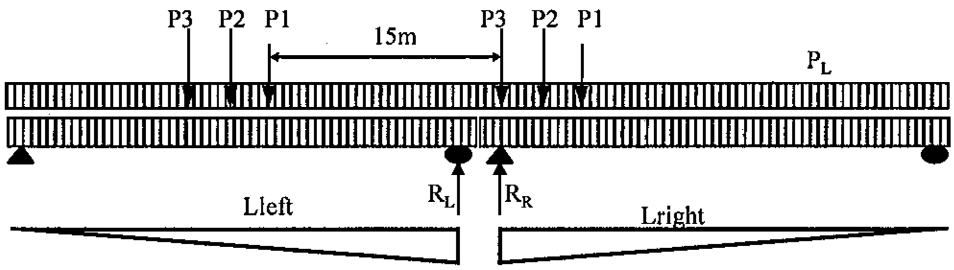
Case	Left span				Right span			
	P3	P2	P1	Aleft	P3	P2	P1	Aright
Case1a:				16.05	1.00	0.87	0.73	16.05
Case1b:	0.95			16.05		1.00	0.87	16.05
Case2a:*	1.00	0.87	0.73	16.05	0.26	0.13	0.00	16.05
Case2b:	0.34	0.48	0.61	16.05	1.00	0.87	0.73	16.05
Case3a:				16.05		1.00	0.96	16.05
Case3b:		1.04		16.05			1.00	16.05

* 2 Trucks in right span

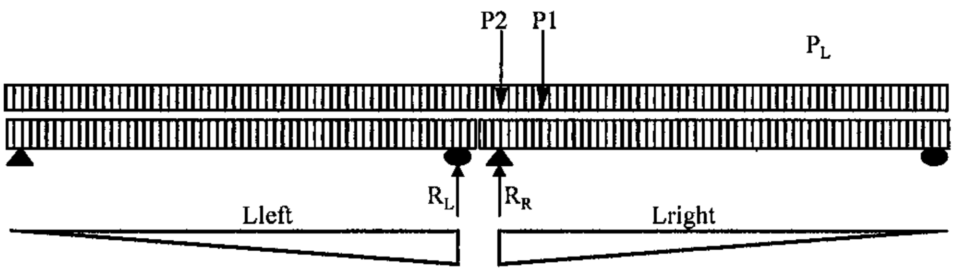
Truck + Lane load



2Trucks + Lane load



Tandem + Lane load



For 1 truck or tandem: Reaction = [(Pi* yi)*(1+IM) + PL*A]*nlane*m

For 2 trucks: Reaction = 0.9* [(Pi* yi)*(1+IM) + PL*A]*nlane*m

7 Loaded Lane m = 1.20

Item	Reaction left	Reaction right	Vertical F_V	Longitudinal			Transversal		
				F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(kN)	(kN)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Case1a:	179.1	623.4	802.5			564.3			3507.1
Case1b:	384.7	442.1	826.8			72.9			3613.0
Case2a:	161.2	638.5	799.7			606.2			3494.8
Case2b:	351.0	561.1	912.1			266.8			3985.7
Case3a:	179.1	502.9	682.1			411.3			2980.6
Case3b:	351.0	344.1	695.1			-8.7			3037.7

2 Loaded Lane m = 1.00

Item	Reaction left	Reaction right	Vertical F_V	Longitudinal F_{HX}	Longitudinal $Arm._{HX}$	Longitudinal M_y	Transversal F_{HY}	Transversal $Arm._{HY}$	Transversal M_x
	(kN)	(kN)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Case1a:	298.5	1039.0	1337.6			940.4			3838.8
Case1b:	641.2	736.8	1378.0			121.5			3954.8
Case2a:	268.7	1064.2	1332.9			1010.3			3825.4
Case2b:	585.0	935.1	1520.1			444.7			4362.7
Case3a:	298.5	838.2	1136.8			685.4			3262.6
Case3b:	585.0	573.5	1158.5			-14.6			3325.0

3 Loaded Lane m = 0.85

Item	Reaction left	Reaction right	Vertical F_V	Longitudinal F_{HX}	Longitudinal $Arm._{HX}$	Longitudinal M_y	Transversal F_{HY}	Transversal $Arm._{HY}$	Transversal M_x
	(kN)	(kN)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Case1a:	380.6	1324.8	1705.4			1199.1			2336.4
Case1b:	817.5	939.4	1756.9			154.9			2407.0
Case2a:	342.6	1356.9	1699.4			1288.2			2328.2
Case2b:	745.8	1192.3	1938.1			567.0			2655.2
Case3a:	380.6	1068.8	1449.4			873.9			1985.7
Case3b:	745.9	731.3	1477.1			-18.6			2023.7

4 Loaded Lane m = 0.65

Item	Reaction left	Reaction right	Vertical F_V	Longitudinal F_{HX}	Longitudinal $Arm._{HX}$	Longitudinal M_y	Transversal F_{HY}	Transversal $Arm._{HY}$	Transversal M_x
	(kN)	(kN)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Case1a:	388.1	1350.7	1738.8			1222.6			54.3
Case1b:	833.5	957.9	1791.4			157.9			56.0
Case2a:	349.3	1383.5	1732.7			1313.4			54.1
Case2b:	760.5	1215.7	1976.1			578.1			61.8
Case3a:	388.1	1089.7	1477.8			891.1			46.2
Case3b:	760.5	745.6	1506.1			-19.0			47.1

Item Live Load	Vertical F_V	Longitudinal F_{HX}	Longitudinal $Arm._{HX}$	Longitudinal M_y	Transversal F_{HY}	Transversal $Arm._{HY}$	Transversal M_x
	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Total at bottom of Column	*1520.1*	*0.0*	*0.0*	*444.7*	*0.0*	*0.0*	*4362.7*
Total at bottom of pilecap	*1520.1*	*0.0*	*0.0*	*444.7*	*0.0*	*0.0*	*4362.7*

Pedestrian Load

Item	Reaction left	Reaction right	Vertical F_V	Longitudinal F_{HX}	Longitudinal $Arm._{HX}$	Longitudinal M_y	Transversal F_{HY}	Transversal $Arm._{HY}$	Transversal M_x
	(kN)	(kN)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
1side	0.0	0.0	0.0			0.0		0.0	
2side	0.0	0.0	0.0			0.0		0.0	

3.Centrifugal Force

Centrifugal force data	CE = n * m * (Axle weights) * C		
Axle weights of design Truck	P	325.0	kN
Number of loaded lanes	n	3.0	lanes
	m	0.85	
Factor, C = (4/3)* V^2/ (g*R)	C	0.0	kN
Highway design speed	V	11.1	m/s
Radius of curvature of traffic lane	R	-	m
Centrifugal force	CE	0.0	kN

Item	From surface	Vertical	Longitudinal			Transversal		
		F_V	F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Centrifugal force	1.80							
Total at bottom of Column						***0.0***	16.997	***0.0***
Total at bottom of pilecap						***0.0***	18.497	***0.0***

4.Braking Force

Braking force data			
Axle weights of design Truck	P	325.0	kN
Number of loaded lanes	n	3.0	lanes
	m	0.85	
Br1 = 25%*(design truck)*n*m	Br1	207.19	kN
Br2 = 5%*(design truck + 9.3*Lbridge)*n*m	Br2	119.70	kN
Br = max(Br1, Br2)	Br	207.19	kN

Item	From surface	Vertical	Longitudinal			Transversal		
		F_V	F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
Take 50 %	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Braking force	1.80							
Total at bottom of Column			***103.6***	16.997	***1760.8***			
Total at bottom of pilecap			***103.6***	18.497	***1916.2***			

5.Uniform Temperature

Uniform temperature data			
Installing temperature	t0	27.0	deg
Maximum temperature	tmax	47.0	deg
Minimum temperature	tmin	10.00	deg
Plus temperature amplitude	Δtmax	20.0	deg
Minus temperature amplitude	Δtmin	17.0	deg
Coefficient of Thermal Expansion	α	1.08E-05	
Strain due to minus temperature	ε_T	1.84E-04	

Item	From surface	Vertical	Longitudinal			Transversal		
		F_V	F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Total at bottom of Column			***45.1***	15.04	***395.8***			
Total at bottom of pilecap			***45.1***	16.54	***485.9***			

6.Creep & Shrinkage

Creep & shrinkage data

Item	From surface	Vertical	Longitudinal			Transversal		
		F_V	F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Total at bottom of Column			***75.1***	15.04	***664.9***			
Total at bottom of pilecap			***75.1***	16.54	***815.1***			

7.Wind on Structure

Wind loads data	$P_D = 0.0006\ V^2\ C_d \geq 1.8$ (kN/m2)		
Average elevation of deck girder upper ground or water plane level	Hele_g	13.0	m
Correct coefficient for wind zone and elevation of pier	S	1.10	
Design wind speed V = S*Vb	V	58.6	m/s
Overall width between handrails	b	12.8	m
Superstructure height including solid parapet	d	3.03	m
	b/d	4.21	
Obstacle coefficient for pier	Cd	1.36	
Wind pressure on pier	P_D	2.80	kN/m2

Item	Exposed height	Vertical F_V	Longitudinal			Transversal		
			F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Superstructure	3.03		70.0	16.4	1147.9	280.0	16.4	4591.7
Total at bottom of Column			***70.0***		***1147.9***	***280.0***		***4591.7***
Superstructure	3.03		70.0	17.9	1252.9	280.0	17.9	5011.6
Total at bottom of pilecap			***70.0***		***1252.9***	***280.0***		***5011.6***

8.Wind on Vehicle

For strength combination III, wind load on vehicular and on structure have to simultaneously consider (wind speed 25m/s). Wind load on vehicular in tranversal direction, is 1.5 kN/m at 1.8m height from asphalt surface. Wind load on vehicular in longitudinal direction is 0.75 kN/m at 1.8m height from asphalt surface.

Item	Vertical F_V	Longitudinal			Transversal		
		F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Superstructure		24.8	18.8	465.9	49.6	18.8	931.9
Total at bottom of Column		***24.8***		***465.9***	***49.6***		***931.9***
Superstructure		24.8	20.3	503.1	49.6	20.3	1006.2
Total at bottom of pilecap		***24.8***		***503.1***	***49.6***		***1006.2***

9.Earth Quake

Earth Quake data			
Acceleration coefficient	A	0.0310	g
Seismic zone	Sz	1	
Soil profile type: according to geological data survey		I	
Coeffient site	S	1.00	
Bridge importance category: "1:critical"; "2:essential"; "3:other"	IC	2	essential
Response Modification Factor			
Column		2.0	
Connection		1.0	
Foundation		1.0	

Response Spectrum - Single mode method is used for EQ analysis.
Result of pier internal force is showed here. For detail refer to annex.

Item	Vertical F_V	Longitudinal			Transversal		
		F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Total at bottom of Column		***236***		***2057***	***365***		***3277***
Total at bottom of pilecap		***236***		***2529***	***365***		***4007***

B.LOAD COMBINATIONS

I. LOAD COMBINATIONS

Loads at Bottom of Column						
			Longitudinal		Transvesal	
Loads	Sign	F_V (kN)	F_{HX} (kN)	My (kN•m)	F_{HY} (kN)	Mx (kN•m)
Superstructure Loads						
1.Stage1 - Dead Load	DC	6748		0		
2.Stage2 - Pave.+Parapet+Railing+Mis.	DW	1643		0		
3.Live Load	LL	1520		445		4363
4.Pedestrian	LL	0		0		0
5.Centrifugal force	CE				0	0
6.Braking force	BR		104	1761		
7.Uniform temperature	TU		45	396		
8.Creep and Shrinkage	CR&SH		75	665		
9.Wind pressure on superstructure	WS		70	1148	280	4592
10.Wind pressure on vehicles	WL		25	466	50	932
11.Earthequake						
a - Longitudinal direction	EQ		118	1028		
b - Transverse direction	EQ				183	1639
Substructure Loads						
1.Pier selfweight	DC	3976				
2.Soil on pile cap	EV	0				
3.Bouyancy on pier						
a - Maximum water level	WA	-678				
b - Minimum water level	WA	-464				
c - Average annual water level	WA	-625				
4.Stream pressure						
a - Maximum water level	WA		0	0	13	101
b - Minimum water level	WA		0	0	0	2
c - Average annual water level	WA		0	0	5	24
5.Wind pressure						
a - Maximum water level	WS		0	0	30	325
b - Minimum water level	WS		46	370	41	419
c - Average annual water level	WS		11	105	33	352
6.Vessel collision force						
a - Longitudinal direction	CV		0	0		
b - Transverse direction	CV				0	0
7.Vehicular collision force						
a - Longitudinal direction	CT		0	0		
b - Transverse direction	CT				0	0

Loads at Bottom of Pilecap

Loads	Sign	F_V (kN)	Longitudinal F_{HX} (kN)	Longitudinal My (kN•m)	Transvesal F_{HY} (kN)	Transvesal Mx (kN•m)
Superstructure Loads						
1.Stage1 - Dead Load	DC	6748		0		
2.Stage2 - Pave.+Parapet+Railing+Mis.	DW	1643		0		
3.Live Load	LL	1520		445		4363
4.Pedestrian	LL	0		0		0
5.Centrifugal force	CE				0	0
6.Braking force	BR		104	1916		
7.Uniform temperature	TU		45	486		
8.Creep and Shrinkage	CR&SH		75	815		
9.Wind pressure on superstructure	WS		70	1253	280	5012
10.Wind pressure on vehicles	WL		25	503	50	1006
11.Earthequake						
a - Longitudinal direction	EQ		236	2529		
b - Transverse direction	EQ				365	4007
Substructure Loads						
1.Pier selfweight	DC	5740				
2.Soil on pile cap	EV	1270				
3.Bouyancy on pier						
a - Maximum water level	WA	-1385				
b - Minimum water level	WA	-1170				
c - Average annual water level	WA	-1332				
4.Stream pressure						
a - Maximum water level	WA		0	0	13	110
b - Minimum water level	WA		0	0	0	2
c - Average annual water level	WA		0	0	5	32
5.Wind pressure						
a - Maximum water level	WS		0	0	30	369
b - Minimum water level	WS		46	439	41	481
c - Average annual water level	WS		11	122	33	400
6.Vessel collision force						
a - Longitudinal direction	CV		173	1794		
b - Transverse direction	CV				404	4111
7.Vehicular collision force						
a - Longitudinal direction	CT		0	0		
b - Transverse direction	CT				0	0

Load Factors and Load Combinations

Loads	Sign	Str1a 2	Str1b 3	Str2a 4	Str2b 5	Str3a 6	Str3b 7
Superstructure Loads							
1.Stage1 - Dead Load	DC	1.25	0.90	1.25	0.90	1.25	0.90
2.Stage2 - Pave.+Mis.	DW	1.50	0.65	1.50	0.65	1.50	0.65
3.Live Load	LL	1.75	1.75	-	-	1.35	1.35
4.Pedestrian	LL	1.75	1.75	-	-	1.35	1.35
5.Centrifugal force	CE	1.75	1.75	-	-	1.35	1.35
6.Braking force	BR	1.75	1.75	-	-	1.35	1.35
7.Uniform temperature	TU	0.50	0.50	0.50	0.50	0.50	0.50
8.Creep and Shrinkage	CR&SH	0.50	0.50	0.50	0.50	0.50	0.50
9.Wind pressure on superst.	WS	-	-	1.40	1.40	0.40	0.40
10.Wind pressure on vehicles	WL	-	-	-	-	1.00	1.00
11.Earthequake							
a - Longitudinal direction	EQL	-	-	-	-	-	-
b - Transverse direction	EQT	-	-	-	-	-	-
Substructure Loads							
1.Pier selfweight	DC	1.25	0.90	1.25	0.90	1.25	0.90
2.Soil on pile cap	EV	1.35	0.90	1.35	0.90	1.35	0.90
3.Bouyancy on pier							
a - Maximum water level	WA		1.00		1.00		1.00
b - Minimum water level	WA	1.00		1.00		1.00	
c - Average annual WL	WA						
4.Stream pressure							
a - Maximum water level	WA		1.00		1.00		1.00
b - Minimum water level	WA	1.00		1.00		1.00	
c - Average annual WL	WA						
5.Wind pressure							
a - Maximum water level	WS	-	-		1.40		0.40
b - Minimum water level	WS	-	-	1.40		0.40	
c - Average annual WL	WS	-	-				
6.Vessel collision force							
a - Longitudinal direction	CV	-	-	-	-	-	-
b - Transverse direction	CV	-	-	-	-	-	-
7.Vehicular collision force							
a - Longitudinal direction	CT	-	-	-	-	-	-
b - Transverse direction	CT	-	-	-	-	-	-

Load Factors and Load Combinations

Loads	**Sign**	**Ser1** 10	**Ext1a** 11	**Ext1b** 12	**Ext1c** 13	**Ext1d** 14	
Superstructure Loads							
1.Stage1 - Dead Load	DC	1.00	1.25	0.90	1.25	0.90	
2.Stage2 - Pave.+Mis.	DW	1.00	1.50	0.65	1.50	0.65	
3.Live Load	LL	1.00	0.50	0.50	0.50	0.50	
4.Pedestrian	LL	1.00	0.50	0.50	0.50	0.50	
5.Centrifugal force	CE	1.00	0.50	0.50	0.50	0.50	
6.Braking force	BR	1.00	0.50	0.50	0.50	0.50	
7.Uniform temperature	TU	1.00	-	-	-	-	
8.Creep and Shrinkage	CR&SH	1.00	-	-	-	-	
9.Wind pressure on superst.	WS	0.30	-	-	-	-	
10.Wind pressure on vehicles	WL	1.00	-	-	-	-	
11.Earthequake							
a - Longitudinal direction	EQL	-	1.00	1.00	0.30	0.30	
b - Transverse direction	EQT	-	0.30	0.30	1.00	1.00	
Substructure Loads							
1.Pier selfweight	DC	1.00	1.25	0.90	1.25	0.90	
2.Soil on pile cap	EV	1.00	1.35	0.90	1.35	0.90	
3.Bouyancy on pier							
a - Maximum water level	WA						
b - Minimum water level	WA	1.00					
c - Average annual WL	WA		1.00	1.00	1.00	1.00	
4.Stream pressure							
a - Maximum water level	WA						
b - Minimum water level	WA	1.00					
c - Average annual WL	WA		1.00	1.00	1.00	1.00	
5.Wind pressure							
a - Maximum water level	WS		-	-	-	-	
b - Minimum water level	WS	0.30	-	-	-	-	
c - Average annual WL	WS		-	-	-	-	
6.Vessel collision force							
a - Longitudinal direction	CV	-	-	-	-	-	
b - Transverse direction	CV	-	-	-	-	-	
7.Vehicular collision force							
a - Longitudinal direction	CT	-	-	-	-	-	
b - Transverse direction	CT	-	-	-	-	-	

Load Factors and Load Combinations

Loads	Sign	Ext2a 15	Ext2b 16	Ext2c 17	Ext2d 18		
Superstructure Loads							
1.Stage1 - Dead Load	DC	1.25	0.90	1.25	0.90		
2.Stage2 - Pave.+Mis.	DW	1.50	0.65	1.50	0.65		
3.Live Load	LL	0.50	0.50	0.50	0.50		
4.Pedestrian	LL	0.50	0.50	0.50	0.50		
5.Centrifugal force	CE	0.50	0.50	0.50	0.50		
6.Braking force	BR	0.50	0.50	0.50	0.50		
7.Uniform temperature	TU	-	-	-	-		
8.Creep and Shrinkage	CR&SH	-	-	-	-		
9.Wind pressure on superst.	WS	-	-	-	-		
10.Wind pressure on vehicles	WL	-	-	-	-		
11.Earthequake							
a - Longitudinal direction	EQL	-	-	-	-		
b - Transverse direction	EQT	-	-	-	-		
Substructure Loads							
1.Pier selfweight	DC	1.25	0.90	1.25	0.90		
2.Soil on pile cap	EV	1.35	0.90	1.35	0.90		
3.Bouyancy on pier							
a - Maximum water level	WA						
b - Minimum water level	WA						
c - Average annual WL	WA	1.00	1.00	1.00	1.00		
4.Stream pressure							
a - Maximum water level	WA						
b - Minimum water level	WA						
c - Average annual WL	WA	1.00	1.00	1.00	1.00		
5.Wind pressure							
a - Maximum water level	WS	-	-	-	-		
b - Minimum water level	WS	-	-	-	-		
c - Average annual WL	WS	-	-	-	-		
6.Vessel collision force							
a - Longitudinal direction	CV						
b - Transverse direction	CV						
7.Vehicular collision force							
a - Longitudinal direction	CT	1.00	1.00	1.00	1.00		
b - Transverse direction	CT	1.00	1.00	1.00	1.00		

II. LOAD COMBINATIONS RESULT

Load Combinations at Bottom of PierColumn

No	Combinations		Sign	F_V (kN)	Longitudinal F_{HX} (kN)	Longitudinal My (kN•m)	Transvesal F_{HY} (kN)	Transvesal Mx (kN•m)
1	Strength 1a		Str1a	18066	241	4390	0	7636
2	Strength 1b		Str1b	12702	241	4390	13	7736
3	Strength 2a		Str2a	15406	223	2655	450	7017
4	Strength 2b		Str2b	10042	158	2137	446	6984
5	Strength 3a		Str3a	17458	271	4581	179	8827
6	Strength 3b		Str3b	12094	253	4433	186	8889
7	Service 1		Ser1	13423	283	4187	146	6799
8	Extreme 1a	EQL	Ext1a	16005	170	2131	60	2697
9	Extreme 1b	EQL	Ext1b	10855	170	2131	60	2697
10	Extreme 1c	EQT	Ext1c	16005	87	1411	188	3844
11	Extreme 1d	EQT	Ext1d	10855	87	1411	188	3844

Load Combinations at Bottom of PileCap

No	Combinations		Sign	F_V (kN)	Longitudinal F_{HX} (kN)	Longitudinal My (kN•m)	Transvesal F_{HY} (kN)	Transvesal Mx (kN•m)
1	Strength 1a		Str1a	21280	241	4782	0	7637
2	Strength 1b		Str1b	14726	241	4782	13	7745
3	Strength 2a		Str2a	18620	223	3020	450	7692
4	Strength 2b		Str2b	12066	158	2405	446	7643
5	Strength 3a		Str3a	20672	271	5018	179	9095
6	Strength 3b		Str3b	14118	253	4842	186	9158
7	Service 1		Ser1	15752	283	4673	146	7019
8	Extreme 1a	EQL	Ext1a	19219	288	3709	115	3415
9	Extreme 1b	EQL	Ext1b	12880	288	3709	115	3415
10	Extreme 1c	EQT	Ext1c	19219	123	1939	370	6221
11	Extreme 1d	EQT	Ext1d	12880	123	1939	370	6221

D.COLUMN DESIGN

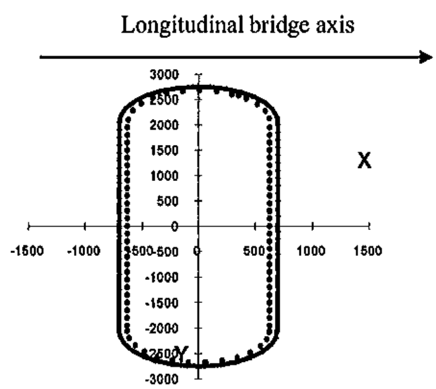
I. COLUMN DATA

1.Load Combinations at Bottom of Pier Column

No	Combinations		Sign	F_V (kN)	Longitudinal F_{HX} (kN)	Longitudinal My (kN•m)	Transvesal F_{HY} (kN)	Transvesal Mx (kN•m)
1	Strength 1a		Str1a	18066	241	4390	0	7636
2	Strength 1b		Str1b	12702	241	4390	13	7736
3	Strength 2a		Str2a	15406	223	2655	450	7017
4	Strength 2b		Str2b	10042	158	2137	446	6984
5	Strength 3a		Str3a	17458	271	4581	179	8827
6	Strength 3b		Str3b	12094	253	4433	186	8889
7	Service 1		Ser1	13423	283	4187	146	6799
8	Extreme 1a	EQL	Ext1a	16005	170	2131	60	2697
9	Extreme 1b	EQL	Ext1b	10855	170	2131	60	2697
10	Extreme 1c	EQT	Ext1c	16005	87	1411	188	3844
11	Extreme 1d	EQT	Ext1d	10855	87	1411	188	3844

2. Pier Column Material

Normal concrete			
Compressive strength at 28 days age	f'c	30	MPa
Concrete elastic modulus	Ec	27691	MPa
Reinforcement TCVN1651-2008; CBV-400			
Yield strength	fy	400	MPa
Reinforcement elastic modulus	Es	200,000	MPa



3. Pier Column Section

Pier column thickness - longitudinal dimension	td	1.40	m
Pier column width - transverse dimension	tn	5.50	m
Section area	A	7.279	m2
Moment inertia	Ix	16.498	m4
	Iy	1.126	m4
Radius of gyration of gross concrete section; r = sqrt(I/A)	rx	1.505	m
	ry	0.393	m

4. Slenderness Effect S.5.7.4.3, S.4.5.3.2.2b, S.4.6.2.5

Transverse direction: Fixed at bottom; translation free, rotation free at top	Kt	2.10	
Longitudinal direction: Fixed at bottom; translation free, rotation free at top	Kl	2.10	
Unsupported length from top to bottom of column	Lu	14.89	m
Slenderness ratio: if K.Lu / r > 22 than considered	Kt.Lu/rx	20.8	no
	Kl.Lu/ry	79.5	yes
Moment inertia of longitudinal reinforcements	Is	0	m4
Ratio of max factored Per. load moment to max factored total load moment	βd	0	

P - Δ analysis

****Longitudinal Direction**

P - Δ moment dertermination procedure:

Initial	Determining displacement for gross cross section	Δxg = Fx . H^3 / (3.E.Ig)
	Displacement for cracked section	Δxcr = Fcr . Δxg
	Moment P-Δ	M P-Δ = Δxcr . P
	Added lateral force	ΔFx = M P-Δ / H
Step: i st	Determining displacement for gross cross section	Δxg i = (Fx+ΔFx i-1) . H^3 / (3.E.Ig)
	Displacement for cracked section	Δxcr i = Fr . Δxg i
	Moment P-Δ	M P-Δ i = Δxcr i . P
	Added lateral force	ΔFx i = M P-Δ i / H

Combination	Fv	My	Initial					
	(kN)	(kNm)	Fx (kN)	Δxg (m)	Fcr (kN)	Δxcr (m)	M P-Δ (kNm)	ΔFx (kN)
Strength 1a	18066	4390	295	0.010	2.5	0.026	470	31.5
Strength 1b	12702	4390	295	0.010	2.5	0.026	330	22.2
Strength 2a	15406	2655	178	0.006	2.5	0.016	242	16.3
Strength 2b	10042	2137	144	0.005	2.5	0.013	127	8.5
Strength 3a	17458	4581	308	0.011	2.5	0.027	474	31.8
Strength 3b	12094	4433	298	0.010	2.5	0.026	317	21.3
Service 1	13423	4187	281	0.010	2.5	0.025	333	22.4
Extreme 1a	16005	2131	143	0.005	2.5	0.013	202	13.6
Extreme 1b	10855	2131	143	0.005	2.5	0.013	137	9.2
Extreme 1c	16005	1411	95	0.003	2.5	0.008	134	9.0
Extreme 1d	10855	1411	95	0.003	2.5	0.008	91	6.1

Combination	Fv	My	1st Trial					
	(kN)	(kNm)	Fx (kN)	Δxg (m)	Fcr (kN)	Δxcr (m)	M P-Δ (kNm)	ΔFx (kN)
Strength 1a	18066	4390	326	0.012	2.5	0.029	520	34.9
Strength 1b	12702	4390	317	0.011	2.5	0.028	355	23.8
Strength 2a	15406	2655	195	0.007	2.5	0.017	264	17.8
Strength 2b	10042	2137	152	0.005	2.5	0.013	135	9.0
Strength 3a	17458	4581	340	0.012	2.5	0.030	523	35.1
Strength 3b	12094	4433	319	0.011	2.5	0.028	340	22.9
Service 1	13423	4187	304	0.011	2.5	0.027	359	24.1
Extreme 1a	16005	2131	157	0.006	2.5	0.014	221	14.9
Extreme 1b	10855	2131	152	0.005	2.5	0.013	146	9.8
Extreme 1c	16005	1411	104	0.004	2.5	0.009	146	9.8
Extreme 1d	10855	1411	101	0.004	2.5	0.009	97	6.5

Combination	Fv	My	2nd Trial					
			Fx	Δxg	Fcr	Δxcr	M P-Δ	ΔFx
	(kN)	(kNm)	(kN)	(m)	(kN)	(m)	(kNm)	(kN)
Strength 1a	18066	4390	330	0.012	2.5	0.029	525	35.3
Strength 1b	12702	4390	319	0.011	2.5	0.028	357	24.0
Strength 2a	15406	2655	196	0.007	2.5	0.017	266	17.9
Strength 2b	10042	2137	153	0.005	2.5	0.013	135	9.1
Strength 3a	17458	4581	343	0.012	2.5	0.030	528	35.4
Strength 3b	12094	4433	321	0.011	2.5	0.028	342	23.0
Service 1	13423	4187	305	0.011	2.5	0.027	361	24.3
Extreme 1a	16005	2131	158	0.006	2.5	0.014	223	15.0
Extreme 1b	10855	2131	153	0.005	2.5	0.013	146	9.8
Extreme 1c	16005	1411	105	0.004	2.5	0.009	148	9.9
Extreme 1d	10855	1411	101	0.004	2.5	0.009	97	6.5

Transverse Direction

Combination	Fv	Mx	Initial					
			Fx	Δxg	Fcr	Δxcr	M P-Δ	ΔFx
	(kN)	(kNm)	(kN)	(m)	(kN)	(m)	(kNm)	(kN)
Strength 1a	18066	7636	513	0.001	2.5	0.003	56	3.7
Strength 1b	12702	7736	520	0.001	2.5	0.003	40	2.7
Strength 2a	15406	7017	471	0.001	2.5	0.003	44	2.9
Strength 2b	10042	6984	469	0.001	2.5	0.003	28	1.9
Strength 3a	17458	8827	593	0.001	2.5	0.004	62	4.2
Strength 3b	12094	8889	597	0.001	2.5	0.004	43	2.9
Service 1	13423	6799	457	0.001	2.5	0.003	37	2.5
Extreme 1a	16005	2697	181	0.000	2.5	0.001	17	1.2
Extreme 1b	10855	2697	181	0.000	2.5	0.001	12	0.8
Extreme 1c	16005	3844	258	0.001	2.5	0.002	25	1.7
Extreme 1d	10855	3844	258	0.001	2.5	0.002	17	1.1

Combination	Fv	Mx	1st Trial					
			Fx	Δxg	Fcr	Δxcr	M P-Δ	ΔFx
	(kN)	(kNm)	(kN)	(m)	(kN)	(m)	(kNm)	(kN)
Strength 1a	18066	7636	517	0.001	2.5	0.003	56	3.8
Strength 1b	12702	7736	522	0.001	2.5	0.003	40	2.7
Strength 2a	15406	7017	474	0.001	2.5	0.003	44	3.0
Strength 2b	10042	6984	471	0.001	2.5	0.003	28	1.9
Strength 3a	17458	8827	597	0.001	2.5	0.004	63	4.2
Strength 3b	12094	8889	600	0.001	2.5	0.004	44	2.9
Service 1	13423	6799	459	0.001	2.5	0.003	37	2.5
Extreme 1a	16005	2697	182	0.000	2.5	0.001	18	1.2
Extreme 1b	10855	2697	182	0.000	2.5	0.001	12	0.8
Extreme 1c	16005	3844	260	0.001	2.5	0.002	25	1.7
Extreme 1d	10855	3844	259	0.001	2.5	0.002	17	1.1

Combination	Fv	Mx	2nd Trial					
			Fx	Δxg	Fcr	Δxcr	M P-Δ	ΔFx
	(kN)	(kNm)	(kN)	(m)	(kN)	(m)	(kNm)	(kN)
Strength 1a	18066	7636	517	0.001	2.5	0.003	56	3.8
Strength 1b	12702	7736	522	0.001	2.5	0.003	40	2.7
Strength 2a	15406	7017	474	0.001	2.5	0.003	44	3.0
Strength 2b	10042	6984	471	0.001	2.5	0.003	28	1.9
Strength 3a	17458	8827	597	0.001	2.5	0.004	63	4.2
Strength 3b	12094	8889	600	0.001	2.5	0.004	44	2.9
Service 1	13423	6799	459	0.001	2.5	0.003	37	2.5
Extreme 1a	16005	2697	182	0.000	2.5	0.001	18	1.2
Extreme 1b	10855	2697	182	0.000	2.5	0.001	12	0.8
Extreme 1c	16005	3844	260	0.001	2.5	0.002	25	1.7
Extreme 1d	10855	3844	259	0.001	2.5	0.002	17	1.1

****Load Combinations at bottom of column considering Slenderness Effect**

Combination	Fv Vert. (kN)	Mx Trans. (kNm)	Mx P-Δ (kNm)	Mx Total (kNm)	My Long. (kNm)	My P-Δ (kNm)	My Total (kNm)
Strength 1a	18066	7636	56	7692	4390	525	4915
Strength 1b	12702	7736	40	7776	4390	357	4747
Strength 2a	15406	7017	44	7061	2655	266	2922
Strength 2b	10042	6984	28	7013	2137	135	2273
Strength 3a	17458	8827	63	8890	4581	528	5108
Strength 3b	12094	8889	44	8933	4433	342	4775
Service 1	13423	6799	37	6836	4187	361	4549
Extreme 1a	16005	2697	18	2714	2131	223	2354
Extreme 1b	10855	2697	12	2709	2131	146	2278
Extreme 1c	16005	3844	25	3869	1411	148	1559
Extreme 1d	10855	3844	17	3861	1411	97	1508

II. PIER COLUMN DESIGN

1. Limit of Reinforcement S.5.7.4.2

Minimum area of longitudinal reinforcement in column							
As.fy / (Ag . f'c) >= 0.135					As ≥	0.074	m2
As / Ag >= 0.01					As ≥	0.073	m2
Maximum area of longitudinal reinforcement in column							
As / Ag <= 0.08					As ≤	0.582	m2
Trial Rebars:					As	**0.040**	m2
1layers	x 82	= 82 bars	D25	@150	As1	0.040	m2
1layers	x 0	= 0 bars	D25	@150	As2	0.000	m2

2. Interation diagram M-P Using Pca-Column software

****In Both Direction**

Strength and Service limit states:

Resistance factor: Compression φ_c = 0.75 *(AASHTO LRFD-2004)*

Tension φ_t = 0.90

Extreme Event limit states:

Resistance factor Compression φ_c = 1.00 *(AASHTO LRFD-2004)*

Tension φ_t = 1.00

No.	COMBINATION	Pu kN	Mux kN-m	Muy kN-m	φ Mnx kN-m	φ Mny kN-m	φ Mn/Mu
1	Strength 1a	18066	4915	7692	18360.4	28734.2	3.74
2	Strength 1b	12702	4747	7776	16300.7	26702	3.43
3	Strength 2a	15406	2922	7061	16091.7	38885.4	5.51
4	Strength 2b	10042	2273	7013	13012.2	40147.1	5.73
5	Strength 3a	17458	5108	8890	17900.1	31153.4	3.50
6	Strength 3b	12094	4775	8933	15790.3	29540.3	3.31
7	Service I	13423	4549	6836	16752	25174	3.68
8	Extreme 1a	16005	2354	2714	19564.3	22556.3	8.31
9	Extreme 1b	10855	2278	2709	17356.5	20640.4	7.62
10	Extreme 1c	16005	1559	3869	17354.9	43070.1	11.13
11	Extremo 1d	10855	1508	3861	15472	39614	10.26

3. Column Ties S.5.7.4.6, S.5.10.6.3, S5.10.11.4.1d - e

Bridge is in seismic zone	Sz	1	
Area of concrete core measured out-to-out of ties	Ac	6.719	m2
Tie diameter	Dtie	16	mm2
Cross section area of 1 tie	As-tr	0.0002	m2
Spacing of hoops	s	150	mm
Length of reinfocement tie in 1 hoop	Ltie	8.93	m
Ratio of ties reinf. in one hoop/ volume of conc. core one pitch spacing			
ρs = As-tr . Ltie / (Ac * spacing)	ρs	0.0018	
Ratio of spiral reinf. To total volume of concrete core shall satisfy			S.5.7.4.6
ρs >= 0.45 . (Ag/Ac - 1). f'c/ fy = Req1	Req1	0.0028	N/A
			S.5.10.11.3
Transverse Reinforcement for Confinement at Plastic Hinges			S.5.10.11.4.1.d
For a circular column "1:applied","2:Not applied"		2	
ρs >= 0.12 . f'c/ fy = Req2	Req2	0.0090	N/A
For a rectangular column			
Rectangular hoop reinforcement shall satisfy			
Either Ash >= 0.30 . s.hc.f'c/ fy. [Ag/Ac -1] = Req1			
or Ash >= 0.12 . s.hc.f'c/ fy = Req2			
In longitudinal direction "1:applied","2:Not applied"		2	
Number of cross tie	nt_x	4	ties
Total cross-sectional area of tie reinf.	Ash_x	0.0008	m2
Core dimension of tied column	hc_x	1.30	m
Rectangular hoop reinforcement shall satisfy	Req1_x	0.0004	m2
	Req2_x	0.0018	m2
	Conclude		N/A
In transverse direction			
Number of cross tie	nt_y	4	ties
Total cross-sectional area of tie reinf.	Ash_y	0.0008	m2
Core dimension of tied column	hc_y	5.40	m
Rectangular hoop reinforcement shall satisfy	Req1_y	0.0015	m2
	Req2_y	0.0073	m2
	Conclude		N/A
Spacing of Transverse Reinforcement for Confinement			S.5.10.11.4.1.e
Transverse reinforcement for confinement shall be:			
* Provided at the top and bottom of the column over a length not less than the greatest of the maximum cross-sectional column dimensions,one-sixth of the clear height of the column, or 450 mm;			
Maximum cross-sectional column dimensions	L1	5.50	m
1/6 of clear height of column	L2	1.58	m
or 450mm	L3	0.45	m
Chosen value: L = max(L1,L2,L3)	L	5.50	m
* Spaced not to exceed one-quarter of the minimum member dimension or 100 mm center-to-center.			
	Spacing	0.10	m
Column connections			S.5.10.11.4.3
* Development length for all longitudinal steel shall be 1.25 times that required in S.5.11			
* Column transverse reinforcement, as specified in Article 5.10.11.4.1d, shall be continued for a distance not less than one-half the maximum column dimension or 380 mm from the face of the column connection into the adjoining member.			
1/2 maximum column dimension	L4	2.75	m
or 380mm	L5	0.38	m
Chosen value: Le = max(L4,L5)	Le	2.75	m

4. Shear Design

Direction		Long.- X	Trans.-Y	Unit
Shear resistance factors	ϕv	1.0	1.0	
Factored shear force in longitudinal	Vu	283	450	kN
Required shear capacity Vn = Vu/ ϕv	Vn	283	450	kN
Determine concrete shear capacity				
Minimum shear reinforcement will provided in cross section				
Therefore	β	2.0	2.0	
	θ	45.0	45.0	
Cross section equivalent height	h	1.40	5.50	m
width	b	5.20	5.20	m
d = h - cover - d1x	d	1.31	5.41	m
dv = max(0.72*h; 0.9*d)	dv	1.18	4.87	m
Vc = min(0.083 . β . sqrt(f'c) . bv. dv, 0.25.f'c.bv.dv)	Vc	5591	23035	kN
Difference between required shear capacity and the capacity provided by concrete				
is the minimum required capacity for shear reinforcements				
Vs = Vn - Vc	Vs	0	0	kN
In this case Vc > Vn so shear reinforcement is no need				
Stirrup diameter	Ds	16	16	
Number of stirrup legs / cross section	ns	6	3	
Shear legs area	Av	0.0012	0.0006	m2
Angle of inclination of shear reinf. to long. axis	α	90	90	deg
Vs = Av. fy. dv. (cot θ + cot α). sin α / s	s <=	-	-	m
Stirrup spacing used	s	0.10	0.10	m
Check minimum shear reinforcement requirement		**OK**	**OK**	
Av >= 0.083. sqrt(f'c). bv. s/ fy = Req	Req	0.0006	0.0006	m2
Check maximum shear reinforcement spacing requirement		**OK**	**OK**	
F = 0.1*f'c*bv*dv	F	18447	76006	kN
If Vu < 0.1*f'c*bv*dv then Smax=0.8*dv <= 600mm				
If Vu > 0.1*f'c*bv*dv then Smax=0.4*dv <= 300mm	Smax	0.60	0.60	m

Interface shear transfer S.5.8.4

Area of concrete engaged in shear transfer	Acv	7.279	m2
Area of shear reinforcement crossing the shear plane	Avf	0.040	m2
For concrete placed against clean, hardened concrete with surface roughened			
Cohesion factor specified in Article 5.8.4.2	c	0.7	MPa
Friction factor	μ	1	
For normal density concrete	λ	1	
Nominal shear resistance of the interface plane shall be taken as			
Vn = c. Acv + μ. Avf. fy	Vn	21200	kN
Vn <= 0.2 . f'c . Acv	Vn <=	43676	kN
Vn <= 5.5 . Acv . (1MPa)	Vn <=	40037	kN
Norminal shear resistance	Vn	21200	kN
Factor for shear friction		1.0	
Factored shear resistance	Vr	21200	kN
Horizontal force at bottom of pier column	Vu	52	kN
	Conclude		**OK**

5. Control of cracking by distribution of reinforcement

Tensile stress in rebars should be satisfied equation: fs <= fsa = Z/[(dc.A)^1/3] and fs <= 0.6.fy				
Direction		Long.- X	Trans.-Y	Unit
Existing condition for structrure	1,2 or 3	1	1	
Crack width parameter	Z	30000	30000	N/mm
Flexural moment	Ms	4549	6836	kNm
Axial thrust at service limit state	Ns	13423	13423	kN
Cross section equivalent height	h	1.40	5.50	m
width	b	5.20	5.20	m
Concrete thickness from tension fiber to tension reinf.	dc	0.05	0.05	m
Concrete thickness from compression fiber to tension reinf.	d	1.31	5.41	kN
Number of rebars	N	41	25	bars
Area of rebars	As	0.0201	0.0123	m2
Area of concrete assumed to participate with reinf.				
A = 2 . dc . b / N	A	0.0127	0.0208	m2
	fsa	349	296	MPa
	0.6fy	240	240	MPa
Min (fsa, 0.6fy) = fs1	fs1	240	240	MPa
e = Ms/Ns+d-h/2	e	0.95	3.17	m
e/d > 1.15	e/d	1.15	1.15	
j = 0.74 + 0.1(e/d) ≤ 0.9	j	0.86	0.86	
i = 1/(1-j.d/e)	i	3.90	3.90	
Stress in rebars: fs = (Ms+Ns(d-h/2))/(As.j.i.d)	fs	145	192	MPa
	Conclude	**OK**	**OK**	
Maximum width of crack: an = 0.076.β.fs.(dc.A)^(1/3)	an	0.158	0.247	mm
Where	β	0.167	0.167	

e. SHALLOW FOUNDATION CHECKING

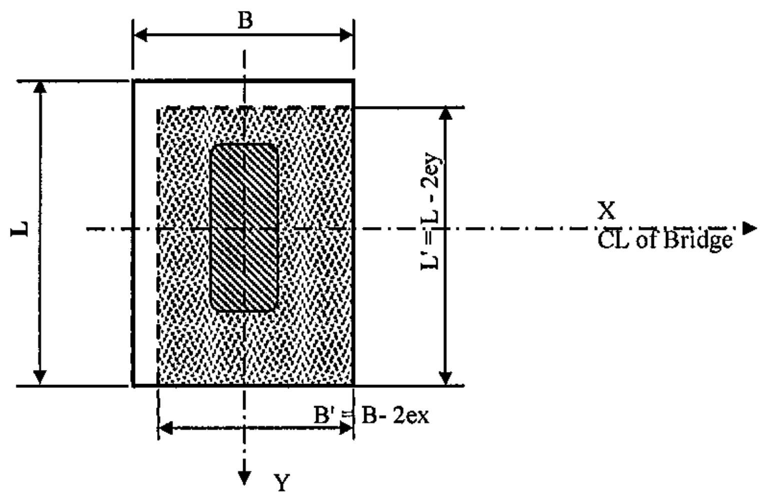
1.LOAD COMBINATIONS AT BOTTOM OF PILE CAP

Load Combinations at Bottom of PileCap

No	Combinations		Sign	F_V (kN)	Longitudinal F_{HX} (kN)	Longitudinal My (kN•m)	Transvesal F_{HY} (kN)	Transvesal Mx (kN•m)
1	Strength 1a		Str1a	21280	241	4782	0	7637
2	Strength 1b		Str1b	14726	241	4782	13	7745
3	Strength 2a		Str2a	18620	223	3020	450	7692
4	Strength 2b		Str2b	12066	158	2405	446	7643
5	Strength 3a		Str3a	20672	271	5018	179	9095
6	Strength 3b		Str3b	14118	253	4842	186	9158
7	Service 1		Ser1	15752	283	4673	146	7019
8	Extreme 1a	EQL	Ext1a	19219	288	3709	115	3415
9	Extreme 1b	EQT	Ext1b	12880	288	3709	115	3415
10	Extreme 1c	EQL	Ext1c	19219	123	1939	370	6221
11	Extreme 1d	EQT	Ext1d	12880	123	1939	370	6221

2.CHECK BEARING RESISTANCE OF SHALLOW FOUNDATION

S.10.6.3



Pile cap properties

Longitudinal dimension		B	**6.0**	m
Transverse dimension		L	**8.0**	m
Hight of foundation		H	**1.5**	m
Pile cap area		A	48.0	m2
		B/6	1.0	
		L/6	1.3	
Bending inertia moment	Wx = Bn^3.Bd/6	Wx	64.0	m3
	Wy = Bn.Bd^3/6	Wy	48.0	m3
Resistance factor for bearing capacity - SLS, shallow foundation		ϕb	0.60	
Resistance factor for bearing capacity - other limit state		ϕb	1.00	
Unaxial compression strength - saturated sample	180.9 kgf/cm2	Qu	2662	kN/m2
Factored bearing resistance		Qr	1597	kN/m2

Compute the bearing capacity based on rock

FHWA manual 1988			
qu = σc.Ksq			
Ksq = [9+3.C/B]/[10.(1+300.g/C)^0.5]			
qu: ultimate end bearing pressure			
C: spacing of discontinuities	C=	0.25	m
B: pile width or (d pile diameter)	B=	6	m
g: aperture of discontinuite	g=	0.03	m
D = 1+0.4. (L/d) <=3.4 - depth factor	D=	1.667	
L: length of the socket	L=	10	m
d: diameter of pile	d=	6	m
	Ksq=	0.1500	
σc: unconfined compressive strength	σc =	17.746	Mpa
	Qu=	2.6622	Mpa
	Qu=	2662	kN/m2

Stress at corner points	
Effective Footing	
Ecentricity	ex = Mx/ Fv
	ey = My/ Fv
Effective footing dimensions	B'd = Bd - 2ex
	B'n = Bn - 2ey
Effective footing area	A' = B'd . B'n

Effective Footing							
Load Combination	**ex** (m)	**ey** (m)	**B'** (m)	**L'** (m)	**A'** (m2)	**W'x** (m3)	**W'y** (m3)
Strength 1a	0.22	0.36	5.6	7.3	40.4	49.1	37.4
Strength 1b	0.32	0.53	5.4	6.9	37.2	43.1	33.2
Strength 2a	0.16	0.41	5.7	7.2	40.7	48.7	38.5
Strength 2b	0.20	0.63	5.6	6.7	37.7	42.3	35.2
Strength 3a	0.24	0.44	5.5	7.1	39.3	46.6	36.1
Strength 3b	0.34	0.65	5.3	6.7	35.6	39.8	31.5
Service 1	0.30	0.45	5.4	7.1	38.4	45.5	34.6
Extreme 1a	0.19	0.18	5.6	7.6	42.9	54.7	40.2
Extreme 1b	0.29	0.27	5.4	7.5	40.5	50.4	36.6
Extreme 1c	0.10	0.32	5.8	7.4	42.6	52.2	41.2
Extreme 1d	0.15	0.48	5.7	7.0	40.1	47.0	38.1

Bearing Resistance - effective footing - AT BOTTOM OF FOOTING							
Load Combination	**Fv /A'** (kPa)	**Mx/Wx** (kPa)	**My/Wy** (kPa)	**σmax** (kPa)	**σmin** (kPa)	**Qr** (kPa)	**Check**
Strength 1a	526	156	128	810	243	1597	**OK**
Strength 1b	396	180	144	720	72	1597	**OK**
Strength 2a	457	158	78	694	221	1597	**OK**
Strength 2b	320	181	68	569	71	1597	**OK**
Strength 3a	526	195	139	861	192	1597	**OK**
Strength 3b	396	230	153	780	13	1597	**OK**
Service 1	410	154	135	699	121	1597	**OK**
Extreme 1a	448	62	92	603	293	2662	**OK**
Extreme 1b	318	68	101	487	149	2662	**OK**
Extreme 1c	451	119	47	617	285	2662	**OK**
Extreme 1d	321	132	51	505	138	2662	**OK**

3.CHECK SLIDING AT THE BASE OF FOOTING

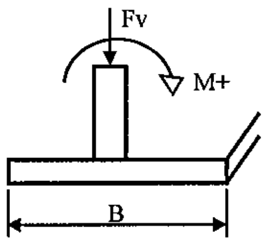
S.10.6.3.3

Horizontal force $H = (F_{HX}\text{^}2 + F_{HY}\text{^}2)\text{^}0.5$			
Factored resistance against failure by sliding			
$Q_r = \phi\, Q_n = \phi_t Q_t + \phi_{ep}\, Q_{ep}$			
Norminal shear resistance between soil and foundation Qt = Fv.tan(ϕ)	Qt		
For concrete cast against soil: tan (ϕ) = tan (ϕf)	tan(ϕf)	0.27	
Internal friction angle of soil	ϕf	15	deg
Resistance factor for shear reistance between soil and foundation	ϕt	0.80	
Norminal passive resistance	Qep	0.00	kN
Resistance factor for passive reistance	ϕep	0.50	

Load Combination	**Resist. Factor** ϕ	**Fv** (kN)	$\mathbf{F_{HX}}$ (kN)	$\mathbf{F_{HY}}$ (kN)	**H** (kN)	**Qr** (kN)	**Check** H < Qr
Strength 1a	0.80	21280	241	0	241	4562	**OK**
Strength 1b	0.80	14726	241	13	242	3157	**OK**
Strength 2a	0.80	18620	223	450	503	3991	**OK**
Strength 2b	0.80	12066	158	446	473	2587	**OK**
Strength 3a	0.80	20672	271	179	325	4431	**OK**
Strength 3b	0.80	14118	253	186	314	3026	**OK**
Service 1	1.00	15752	283	146	319	4221	**OK**
Extreme 1a	1.00	19219	288	115	310	5150	**OK**
Extreme 1b	1.00	12880	288	115	310	3451	**OK**
Extreme 1c	1.00	19219	123	370	390	5150	**OK**
Extreme 1d	1.00	12880	123	370	390	3451	**OK**

4.CHECK OVERTURNNING AT THE BASE OF FOOTING

S.11.6.3.3, S.11.6.3.7
S.10.6.4.2-lrfd2007



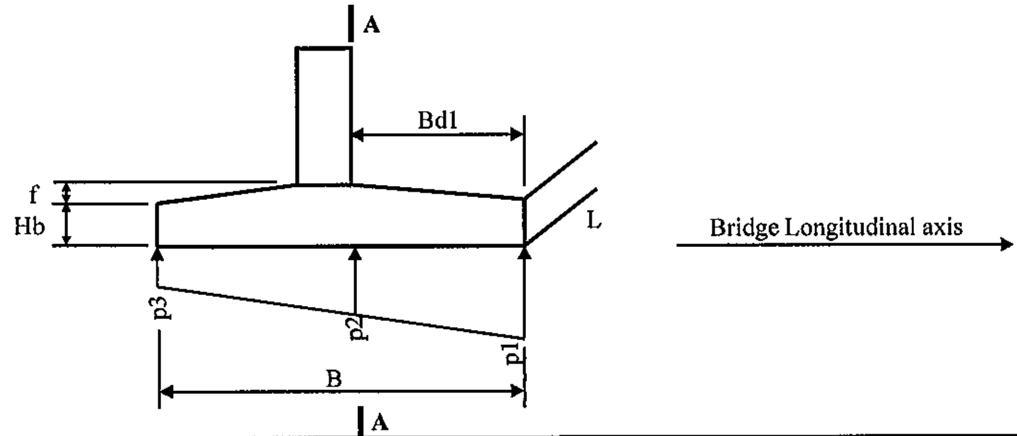
S.10.6.3.2.5

The location of the resultant of the reaction forces shall be within the middle three-fourths of the base.				
Longitudinal direction	ex =My / Fv <= 3.Bd / 8 = [ex]	[ex]	2.25	m
Transverse direction	ey =Mx / Fv <= 3.Bn / 8 = [ey]	[ey]	3.00	m
For seismic provision				S.10.6.5
According to 22TCN272-05: **1**				
Longitudinal direction	ex =My / Fv <= 0.6 Bd / 2 = [ex]	[ex]	1.80	m
Transverse direction	ey =Mx / Fv <= 0.6 Bn / 2 = [ey]	[ey]	2.40	m
According to LRFD 2004: **2**				
Where γEQ = 0				
Longitudinal direction	ex =My / Fv <= 2/3. Bd / 2 = [ex]	[ex]	2.00	m
Transverse direction	ey =Mx / Fv <= 2/3. Bn / 2 = [ey]	[ey]	2.67	m
Where γEQ = 1				
Longitudinal direction	ex =My / Fv <= 8/10. Bd / 2 = [ex]	[ex]	2.40	m
Transverse direction	ey =Mx / Fv <= 8/10. Bn / 2 = [ey]	[ey]	3.20	m
Where γEQ between 0 and 1, restrictions of the location can get by linear interpolation				
Choosing value for seismic: following LRFD 2004, with γEQ = 0.5				
Longitudinal direction		[ex]	2.20	m
Transverse direction		[ey]	2.93	m

Load Combination	**Fv** (kN)	**Mx** (kN•m)	**My** (kN•m)	**Longitudinal** ex (m)	**Longitudinal** Check ex <[ex]	**Transverse** ey (m)	**Transverse** Check ey <[ey]
Strength 1a	21280	7637	4782	0.22	**OK**	0.36	**OK**
Strength 1b	14726	7745	4782	0.32	**OK**	0.53	**OK**
Strength 2a	18620	7692	3020	0.16	**OK**	0.41	**OK**
Strength 2b	12066	7643	2405	0.20	**OK**	0.63	**OK**
Strength 3a	20672	9095	5018	0.24	**OK**	0.44	**OK**
Strength 3b	14118	9158	4842	0.34	**OK**	0.65	**OK**
Service 1	15752	7019	4673	0.30	**OK**	0.45	**OK**
Extreme 1a	19219	3415	3709	0.19	**OK**	0.18	**OK**
Extreme 1b	12880	3415	3709	0.29	**OK**	0.27	**OK**
Extreme 1c	19219	6221	1939	0.10	**OK**	0.32	**OK**
Extreme 1d	12880	6221	1939	0.15	**OK**	0.48	**OK**

5.CHECK BENDING MOMENT AND SHEAR OF FOOTING

Transverse Section A-A



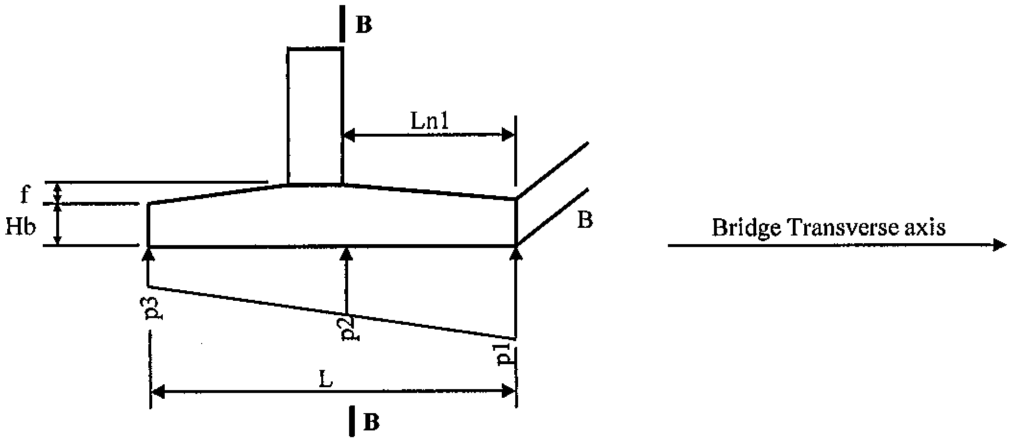
Transverse section (A-A)			
Footing dimensions	H	1.50	m
	f	0.00	m
	B	6.00	m
	Bd1	2.40	m
	L	8.00	m
Considering of bouyancy **"1:yes" "0:no"**		1	
Internal force at section A-A due to selfweight of pilecap			
Shear force	Qself	423	kN
Bending moment	Mself	508	kN
p1 = Fv /A + My/ Wy			
p3 = Fv /A - My/ Wy			

Earth on toe shall be neglected.

Load Combination	Load factor	**Qself** (kN)	**Mselft** (kN•m)	**p1** (kPa)	**p3** (kPa)	**p2** (kPa)	**M_A** (kN•m)	**Q_A** (kN)
Strength 1a	1.25	529	635	543	344	463	**11263**	**8999**
Strength 1b	0.90	381	457	406	207	327	**8295**	**6696**
Strength 2a	1.25	529	635	451	325	400	**9366**	**7512**
Strength 2b	0.90	381	457	301	201	261	**6181**	**5061**
Strength 3a	1.25	529	635	535	326	452	**11054**	**8812**
Strength 3b	0.90	381	457	395	193	314	**8024**	**6467**
Service 1	1.00	423	508	426	231	348	**8698**	**6999**
Extreme 1a	1.25	529	635	478	323	416	**9896**	**7917**
Extreme 1b	0.90	381	457	346	191	284	**7031**	**5699**
Extreme 1c	1.25	529	635	441	360	408	**9273**	**7492**
Extreme 1d	0.90	381	457	309	228	276	**6408**	**5275**

Maximum internal force at section A-A (transverse section)			
Bending moment	M_A	**11263**	kNm
Shear force	Q_A	**8999**	kN

Longitudinal Section B-B



Longtidinal section (B-B)			
Footing dimensions	H	1.50	m
	L	8.00	m
	Ln1	1.48	m
	B	6.00	m
Considering of bouyancy **"1:yes" "0:no"**		1	
Internal force at section B-B due to selfweight of pilecap			
Shear force	Qself	196	kN
Bending moment	Mself	145	kN
p1 = Fv /A + Mx/ Wx			
p3 = Fv /A - Mx/ Wx			

Earth on toe shall be neglected.

Load Combination	Load factor	**Qself** (kN)	**Mselft** (kN•m)	**p1** (kPa)	**p3** (kPa)	**p2** (kPa)	**M_A** (kN•m)	**Q_A** (kN)
Strength 1a	1.25	245	182	563	324	518	**3435**	**4566**
Strength 1b	0.90	177	131	428	186	383	**2594**	**3431**
Strength 2a	1.25	245	182	508	268	464	**3074**	**4079**
Strength 2b	0.90	177	131	371	132	327	**2219**	**2927**
Strength 3a	1.25	245	182	573	289	520	**3483**	**4618**
Strength 3b	0.90	177	131	437	151	384	**2638**	**3479**
Service 1	1.00	196	145	438	218	397	**2655**	**3520**
Extreme 1a	1.25	245	182	454	347	434	**2770**	**3705**
Extreme 1b	0.90	177	131	322	215	302	**1949**	**2598**
Extreme 1c	1.25	245	182	498	303	462	**3023**	**4023**
Extreme 1d	0.90	177	131	366	171	329	**2203**	**2916**

Maximum internal force at section B-B (transverse section)			
Bending moment	M_A	**3483**	kNm
Shear force	Q_A	**4618**	kN

5.PUNCHING SHEAR CHECK (TWO WAY SHEAR) S.5.13.3.6.3

Assume the entire column vertical load needs to be carried at the perimeter.
Two-way shear is evaluated on a perimeter located dv/2 away from the face of the actual pier column.

Pier Column dimensions	Longitudinal axis	td	**1.40**	m
	Transverse axis	tn	**5.50**	m
Estimated distance between internal flexural force components dv, we may take dv = 0.9*de				
de = H - cover - dx1		de	1.31	m
		dv	1.18	m
Perimeter of two-way shear				
b0 = (td+tn)*2 + 4*dv		b0	18.51	m
Compressive strength of pilecap concrete		f'c	**30**	Mpa
Yield strength of rebar		fy	**400**	Mpa
Section with transverse reinforcement				
Norminal shear resistance shall be taken as				
Vn = Vc + Vs <= 0.504.sqrt(f'c). b0 . dv = Va				
Vc = 0.166 . sqrt(f'c). b0 . dv				
Vs = Av. fy . dv / s				
Shear resistance of concrete		Vc	19830	kN
Assumed stirrup diameter		Ds	16	mm
Number of stirrup legs / cross section		ns	0	
Shear legs area		Av	0.0000	m2
Stirrup spacing used		s	600	mm
Shear resistance of reinforcement		Vs	0	kN
		Va	60205	kN
		Vn	19830	kN
Maximum reaction at bottom of column		Vu	13526	kN
Resistance factor for shear		ϕv	0.9	
Factored shear resistance		ϕv*Vn	17847	kN
Punching shear check			**OK**	

6.SETTLEMENT OF FOOTING ON ROCK

For Rectangular footing				
Elastic settlement	p = q0 (1 - v^2) * B.Ip / Em			
	Ip = (L/B)^0.5 / βz			
Footing dimensions	Bd =	B'	5.41	m
	Bn =	L'	7.11	m
		L'/B'	1.31	
Ridity "1: Flexible" "2: rigid"			2	
Factor to account for footing shape and rigidity		βz	1.09	
Influence coefficient to account for rigidity and dimension of footing		Ip	1.06	
Poisson's ratio		v	0.29	
Rock mass modulus Em = 1000*10^[(RMR-10)/40]		Em	891	Mpa
Rock mass rating a.10.4.6.4, Table 10.6.4.4-1,2		RMR	8	
Applied vertical stress at base of loaded area - Service 1 combination		q0	0.410	Mpa
Elastic settlement	**Can be ignored**	p	2.40	mm

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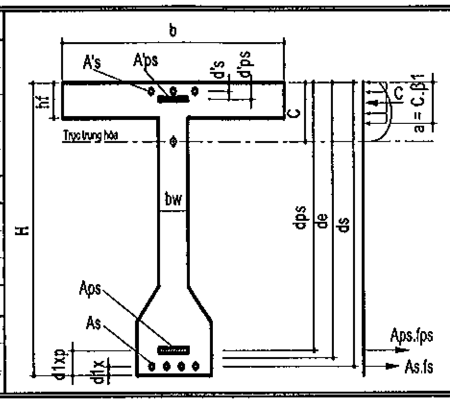
REINFORCEMENT CHECKING - PILE CAP

MATERIALS

	NORMAL CONCRETE		
f'c	Compressive Strength of concrete at 28 days	Mpa	30
Ec	Modulus of Elasticity	Mpa	29440
fr	Modulus of Rupture	Mpa	3.5
gc	Unit weight of concrete	kN/m3	24.5
	PRESTRESSING STEEL		
fpu	Tensile strength of prestressing steel	Mpa	1860
fpy	Yield strength of prestressing steel	Mpa	1670
Ep	Modulus of Elasticity	Mpa	195000
	REINFORCEMENT		
fy	Yield strength	Mpa	400
Es	Modulus of Elasticity	Mpa	200000
nc	Ratio Es/Ec		7



Sign	Parameters		Unit	Section - PILE CAP				
				A-A	A-A	A-A	B-B	B-B
	INTERNAL FORCES AT SECTIOIN							
	Combination			Strength	Service	Extreme	Service	Strength
Qu	Shear		kN	8999	6999	7917	3520	3483
Mu	Flexural Moment		kNm	11263	8698	9896	2655	4618
Nu	Axial load		kN	0	0	0	0	0
Tu	Torsional Moment		kNm	0	0	0	0	0
	FLEXURAL MOMENT CHECKING							
H	Section height		m	1.500	1.500	1.500	1.500	1.500
d's	Dis. From comp. fiber to centroid of comp. Reinf		m	0.083	0.083	0.083	0.083	0.083
d1x	Dis. From tens. fiber to centroid of tension Reinf		m	0.091	0.091	0.091	0.086	0.086
	Cover to reinf		m	0.075	0.075	0.075	0.075	0.075
ds	Dis. From comp. fiber to centroid of tension Reinf		m	1.409	1.409	1.409	1.414	1.414
d'ps	Dis. From comp. fiber to centroid of comp. prestressing steel		m	0.000	0.000	0.000	0.000	0.000
d1xp	Dis. From tens. fiber to centroid of tension prestressing steel		m	0.000	0.000	0.000	0.000	0.000
dps	Dis. From comp. fiber to centroid of tension prestressing steel		m	1.500	1.500	1.500	1.500	1.500
b	Width of the compression face of member		m	8.000	8.000	8.000	6.000	6.000
bw	Web width or diameter of a circular section		m	8.000	8.000	8.000	6.000	6.000
hf	Compression flange depth		m	0.000	0.000	0.000	0.000	0.000
Iz	Moment of inertia of section		m4	2.2500	2.2500	2.2500	1.6875	1.6875
Amc	Section area		m2	12.000	12.000	12.000	9.000	9.000
	Steel choice							
Aps	Tension prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	0 T00.0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
A'ps	Compression prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	0 T00.0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
As	Tension Reinforcement	Number	bars	53	53	53	40	40
		Diameter	mm	32	32	32	22	22
		Area	m2	0.04245	0.04245	0.04245	0.01520	0.01520
A's	Compression Reinforcement	Number	bars	53	53	53	40	40
		Diameter	mm	20	20	20	20	20
		Area	m2	0.01664	0.01664	0.01664	0.01256	0.01256
A'c	Shear reinforcement	Number	bars	27	27	27	20	20
		Diameter	mm	16	16	16	16	16
		Area	m2	0.00545	0.00545	0.00545	0.00404	0.00404
ϕ	Resistance factors for flexure		5.5.4.2	0.90	1.00	1.00	1.00	0.90
ϕv	Resistance factors for shear			0.90	1.00	1.00	1.00	0.90
ϕn	Resistance factors for axial force			1.00	1.00	1.00	1.00	1.00
β1	Stress block factor			0.836	0.836	0.836	0.836	0.836
c	Dis. Between centroid and top fiber		m	0.061	0.061	0.061	0.008	0.008
	For T section behavior		m	0.061	0.061	0.061	0.008	0.008
	For rectangular section behavior		m	0.061	0.061	0.061	0.008	0.008
fpe	Effective stress in the prestressing steel after losses		Mpa	1116	1116	1116	1116	1116
fps	Aver. stress in pres. steel at the time for which the nominal resistance		Mpa	1839	1839	1839	1857	1857
k	Factor depends on type of P.S, Low relaxation strand k = 0.28			0.28	0.28	0.28	0.28	0.28

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a	Depth of equivalent stress block	m	0.051	0.051	0.051	0.007	0.007
de	Corresp. effective depth from extreme comp. fiber						
	to centroid of tensile force in the tensile reinf.	m	1.409	1.409	1.409	1.414	1.414
Mn	Nominal resistance	kNm	23113	23113	23113	8176	8176
Mr	Factored reistance	kNm	**20801**	**23113**	**23113**	**8176**	**7359**
Mu	Flexual moment	kNm	11263	8698	9896	2655	4618
(5.7.3.2)	**Flexural moment Checking**		**OK**	**OK**	**OK**	**OK**	**OK**
	Limits for reinforcement						
c/de	Maximum reinforcement		0.04	0.04	0.04	0.01	0.01
	Maximum reinforcement Checking	<= 0.42	**OK**	**OK**	**OK**	**OK**	**OK**
1.2*Mcr	Craking moment	kNm	6472	6472	6472	4684	4684
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**		**OK**	**OK**	**OK**	**OK**	**OK**
(5.7.3.4)	**Conctrol of craking by distr. of reinf for RC member- Check?**		No	Yes	No	Yes	No
	Existing condition for structrure	1,2 or 3	3	3	3	3	3
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.066	0.066	0.066	0.061	0.061
Z	Crack width parameter	N/mm	17500	17500	17500	17500	17500
A	Area of concr. with same centroid as tens. Reinf	m2	0.020	0.020	0.020	0.018	0.018
fsa	Value	Mpa	160	160	160	169	169
0.6*fy		Mpa	240	240	240	240	240
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	160	160	160	169	169
x	Dist. From compression fiber to centroid	m	-	0.289	-	0.207	-
J.d	Arm	m	-	1.313	-	1.345	-
Icr	Moment of inertia of the cracked section	m4	-	0.441	-	0.174	-
fs	Tensile stress in reinforcement fs = Msls / (As*J.d)	Mpa	-	156	-	130	-
	Checking for control cracking fs<fsa		**N.a**	**OK**	**N.a**	**OK**	**N.a**
(5.10.8.2)	**Shrinkage and temperature Reinfocement (side distribution)**						
Areq	Area of required reinf	m2	0.00118	0.00118	0.00118	0.00113	0.00113
	Distribution on sides 10 D16	m2	0.00202	0.00202	0.00202	0.00202	0.00202
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45	0.45
	Checking		**OK**	**OK**	**OK**	**OK**	**OK**
	SHEAR AND TORSION CHECKING						
β	Factor indicating diag. cracked concr. to tension		2.0	2.1	2.1	2.1	2.0
θ	Angle of inclination of diagonal compressive	degree	40.89	37.53	39.09	37.98	41.60
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	90
bv	Effective web width as minimum web width - in dv	m	8.000	8.000	8.000	6.000	6.000
dv	Effective shear depth	m	1.384	1.384	1.384	1.411	1.411
	(de - a/2)	m	1.384	1.384	1.384	1.411	1.411
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	0.600
ncat	Amount of bars in spacing S	bars	27	27	27	20	20
Av	Shear reinf area in spacing S	m2	0.0055	0.0055	0.0055	0.0040	0.0040
β	Assume		2.0	2.0	2.0	2.0	2.0
θ	Assume	degree	45.00	45.00	45.00	45.00	45.00
v	Shear stress in concrete	kN/m2	903	632	715	416	457
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e_x	Strain in tensile reinforcement		1.49E-03	1.15E-03	1.31E-03	1.20E-03	1.65E-03
	if ex<0, multiple with reduce factor		-	-	-	-	-
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok	Ok
v/f'c	Ratio of shear stress and f'c		0.030	0.021	0.024	0.014	0.015
β	Final value		2.0	2.1	2.1	2.1	2.0
θ	Final value	degree	40.89	37.53	39.09	37.98	41.60
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	9845	10792	10353	8153	7695
Vs	Shear resistance provided by shear reinforcement	kN	5811	6551	6194	4866	4279
Vp	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0	0
Vn1	Vn1=Vc+Vs+Vp	kN	15656	17343	16547	13019	11974
Vn2	Vn2	kN	83022	83022	83022	63475	63475
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	15656	17343	16547	13019	11974
Vr	Factored shear resistance	kN	**14090**	**17343**	**16547**	**13019**	**10777**
Vu	Shear	kN	8999	6999	7917	3520	3483
(5.8.2.7)	**Shear checking**		**OK**	**OK**	**OK**	**OK**	**OK**

BEARING CAPACITY OF PILE -LRB09 BRIDGE

STT	Abut/pier	Boring	Bottom of pile	Top of rock	Bottom of pile Tip	Pile length	Bearing capacity of pile (T)		interal force of top pile (T)		Check
			m	m	m	m	STR	EX	STR	EX	
1	A1	LRB09-A1	9.50	2.68	0.00	9.50	655	1201	509	385	OK
2	A2	LRB09-A2	8.50	4.90	2.50	6.00	630	1153	405	318	OK
3	P2	LRB09-P2	7.50	3.30	-1.00	8.50	561	1032	460	372	OK

ASSUMPTION:

- The rock is reasonably sound
- The rock strength measured during site investigation will not deteriorate during construction when water or drilling fluids are used
- The bottom of the socket is properly cleaned out.

DATA & CALCULATION:

Bored hole name		**LRB09-A1**	
Bottom of pilecap elavation	$EL1 =$	9.50	
Top of socket elevation	$EL2 =$	2.68	
Pile tip elevation	$EL3 =$	0.00	
Pile Length	$L =$	9.50	m
Diameter of drilled-shaft	$D_p =$	1.00	m
Pile Cross-Sectional Perimeter	$P =$	3.14	m
Pile Cross-Sectional Area	$A_b =$	0.79	m^2
Working normal force at pile head	$N =$	6579.1	kN
Working normal force at top of socket	$P_i =$	6548.1	kN
Intack rock modulus	$E_i =$	50000	MPa
Modulus modification ratio	$K_e =$	0.05	
Elastic modulus of the insitu rock	$E_r = K_e{*}E_i =$	2500.0	MPa
Influence coefficient	$I_p = f(H_s/D_s, E_c/E_r) =$	0.38	
	$H_s/D_s =$	2.68	
	$E_c/E_r =$	11.08	

Pile Concrete comp. strength	$f'_c =$	30.0	MPa
Concrete Unit Weight	$g_c =$	24.5	kN/m^3
Modulus of elasticity of concre	$E_c =$	27691	MPa
Depth of socket	$H_s =$	2.68	m
Diameter of socket	$D_s =$	1.00	m
Socket Cross-Sect. Perimeter	$P_{soc} =$	3.14	m
Socket Cross-Sectional Area	$A_{soc} =$	0.79	m^2

Figure C10.8.3.5-2 Lrfd
Figure C10.8.3.5-3 Lrfd
Figure C10.8.3.5-1 Lrfd

Rock mass modulus/ intack rock modulus	E_m / E_i			*C.10.4.6.5-1-Lrfd 4th*
Atmospheric pressure	$p_a =$	0.101	MPa	
Reduction factor to account for jointing	α_E			*10.8.3.5.4b-Lrfd 4th*

The elastic shortening of the drilled shaft

$r_e = (\Sigma P_i){*}H_s/(A_{soc}{*}E_c) =$ 0.807 mm

The settlement of base of drilled shaft

$r_{base} = (\Sigma P_i){*}I_p/(D_s{*}E_r) =$ 0.995 mm

$r_e + r_{base} =$ **1.802** mm < 10 mm

Compute the bearing capacity based on Sharft resistance alone

Unit side resistance

Case 1: The drilling fluid used will not form a lubricated film on the sides of the socket

if qu < 1.9 Mpa - may be taken after Carter & Kulhawy 1988 → qs = 0.15*qu — *C10.8.3.5-4*

if qu > 1.9 Mpa - may be taken after Horvath & Kenney 1979 → qs = 0.21*sqrt(qu) — *C10.8.3.5-5*

Case 2: Side of rock socket is considered to be smooth or where the rock is drilled using a drilling slurry

May be taken after Horvath & Kenney 1979 → qs = 0.65*α_E*pa*(qu/pa)^0.5<7.8*pa*(f'c/pa)^0.5

qs0 = 7.8*pa*(f'c/pa)^0.5 — *10.8.3.5.4d-1-Lrfd2007*

Drilling method used in construction: $\boxed{\textit{Case 2}}$

$Q_R = \phi{*}Q_n = \phi_s{*}Q_{SR}$

ϕ_s is the resistance factor - table 10.5.5-3 LRFD

q_u is the uniaxial compressive strength of the rock

Case1

No.	EL_T	EL_B	Depth (m)	RQD (%)	q_u (MPa)	q_s (MPa)	Q_{SR} (kN)	ϕ_s	Q_R (kN)
1	2.68	1.18	1.50	65	60.00	1.63	7665	0.65	4983
2	1.18	0.68	0.50	90	60.00	1.63	2555	0.65	1661
3	0.68	0.00	0.68	90	60.00	1.63	3475	0.65	2259
4			-	-	-	-	-	-	-
5									
6									
7									
8									
Sum			**2.68**				**13696**		**8902**

Case2 [illegible] *Type: 1) closed joints ; 2) open joints*

No.	Depth (m)	RQD (%)	q_u (MPa)	E_m / E_i	α_E	Type	qs0 (MPa)	q_s (MPa)	*q_s - used* (MPa)	Q_{SR} (kN)	ϕ_s	Q_R (kN)
1	1.50	65.00	60.00	0.56	0.83	1	13.58	1.32	*1.32*	6221	0.55	3421
2	0.50	90.00	60.00	0.90	0.96	1	13.58	1.54	*1.54*	2413	0.55	1327
3	0.68	90.00	60.00	0.90	0.96	1	13.58	1.54	*1.54*	3282	0.55	1805
4	-	-	-	-	-	-	-	-	-	-	-	-
5	-	-	-	-	-	-	-	-	-	-	-	-
6	-	-	-	-	-	-	-	-	-	-	-	-
7	-	-	-	-	-	-	-	-	-	-	-	-
8	-	-	-	-	-	-	-	-	-	-	-	-
Sum	**2.68**									**11915**		**6553**

Unit base resistance $q_p = Kb.(p1\text{-}p0) + \sigma v$

Limit pressure determined from presuremeter tests	$p_l =$	-	MPa	*C10.8.3.5-7*
At rest total horizontal stress measured at ther base elevation	$p_0 =$	-	MPa	
Coefficient that depen on diameter socket	$K_b =$	4.01		*Table C10.8.3.5-1*
Total vertical stress at the base elevation	$\sigma v =$	-	MPa	
	$q_p =$	-	MPa	
	$\phi =$	0.50		*Table 10.5.5-3*
$Q_{pR} = Ap \cdot qp$	$Q_{pR} =$	-	kN	
$Q_R = \phi . Q_{pR}$	$Q_R =$	-	kN	

ESTIMATED PILE CAPACITY:

Pile Structural Capacity $Q_T = 0.75*0.85*0.85*fc*Ag$	*12768*	kN	*1301*	T
Pile resistance Q_R	*6553*	kN	*668*	T
Deducting pile weight	*-130*	kN	*-13*	T
Estimated Pile Capacity (STR)	*6423*	kN	*655*	T
Estimated Pile Capacity (EXT)	*11785*	kN	*1201*	T

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	LRB09 BRIDGE	Design			
	DETAIL DESIGN	Check			
	EMPIRICAL ESTIMATION OF PILE CAPACITY-ABUTMENT A2	Revise			

AASHTO - LRFD 3rd 2004 & 4th 2007; 22TCN-272-05

ASSUMPTION:

- The rock is reasonably sound
- The rock strength measured during site investigation will not deteriorate during construction when water or drilling fluids are used
- The bottom of the socket is properly cleaned out.

DATA & CALCULATION:

Bored hole name		**LRB09-A2**	
Bottom of pilecap elavation	$EL1 =$	8.50	
Top of socket elevation	$EL2 =$	4.90	
Pile tip elevation	$EL3 =$	2.50	
Pile Length	$L =$	6.00	m
Diameter of drilled-shaft	$D_p =$	1.00	m
Pile Cross-Sectional Perimeter	$P =$	3.14	m
Pile Cross-Sectional Area	$A_b =$	0.79	m^2
Working normal force at pile head	$N =$	6276.3	kN
Working normal force at top of socket	$P_i =$	6248.5	kN
Intack rock modulus	$E_i =$	50000	MPa
Modulus modification ratio	$K_e =$	0.05	
Elastic modulus of the insitu rock	$E_r = K_e{*}E_i =$	2500.0	MPa
Influence coefficient	$I_p = f(H_s/D_s, E_c/E_r) =$	0.42	
	$H_s/D_s =$	2.40	
	$E_c/E_r =$	11.08	

Pile Concrete comp. strength	$f'_c =$	30.0	MPa
Concrete Unit Weight	$g_c =$	24.5	kN/m^3
Modulus of elasticity of concre	$E_c =$	27691	MPa
Depth of socket	$H_s =$	2.40	m
Diameter of socket	$D_s =$	1.00	m
Socket Cross-Sect. Perimeter	$P_{soc} =$	3.14	m
Socket Cross-Sectional Area	$A_{soc} =$	0.79	m^2

Figure C10.8.3.5-2 Lrfd
Figure C10.8.3.5-3 Lrfd
Figure C10.8.3.5-1 Lrfd

Rock mass modulus/ intack rock modulus E_m / E_i — *C.10.4.6.5-1-Lrfd 4th*

Atmospheric pressure $p_a =$ 0.101 MPa

Reduction factor to account for jointing α_E — *10.8.3.5.4b-Lrfd 4th*

The elastic shortening of the drilled shaft

$r_e = (\Sigma P_i){*}H_s/(A_{soc}{*}E_c) =$ 0.690 mm

The settlement of base of drilled shaft

$r_{base} = (\Sigma P_i){*}I_p/(D_s{*}E_r) =$ 1.050 mm

$r_e + r_{base} =$ **1.739** mm < 10 mm

Compute the bearing capacity based on Sharft resistance alone

Unit side resistance

Case 1: The drilling fluid used will not form a lubricated film on the sides of the socket

if qu < 1.9 Mpa - may be taken after Carter & Kulhawy 1988 → qs = 0.15*qu — *C10.8.3.5-4*

if qu > 1.9 Mpa - may be taken after Horvath & Kenney 1979 → qs = 0.21*sqrt(qu) — *C10.8.3.5-5*

Case 2: Side of rock socket is considered to be smooth or where the rock is drilled using a drilling slurry

May be taken after Horvath & Kenney 1979 → qs = 0.65*α_E*pa*(qu/pa)^0.5<7.8*pa*(f'c/pa)^0.5

qs0 = 7.8*pa*(f'c/pa)^0.5 — *10.8.3.5.4d-1-Lrfd2007*

Drilling method used in construction: *Case 2*

$Q_R = \phi{*}Q_n = \phi_s{*}Q_{SR}$

ϕ_s is the resistance factor - table 10.5.5-3 LRFD

q_u is the uniaxial compressive strength of the rock

Case1

No.	EL_T	EL_B	Depth (m)	RQD (%)	q_u (MPa)	q_s (MPa)	Q_{SR} (kN)	ϕ_s	Q_R (kN)
1	4.90	3.90	1.00	85	60.20	1.63	5119	0.65	3327
2	3.90	2.90	1.00	85	60.20	1.63	5119	0.65	3327
3	2.90	2.50	0.40	90	60.20	1.63	2048	0.65	1331
4									
5									
6									
7									
8									
Sum			**2.40**				**12285**		**7985**

		Item.	Eng.	Date.	Sign.
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	LRB09 BRIDGE	Design			
	DETAIL DESIGN	Check			
	EMPIRICAL ESTIMATION OF PILE CAPACITY-ABUTMENT A2	Revise			

Case2 — *Type: 1: closed joints; 2: open joints*

No.	Depth (m)	RQD (%)	q_u (MPa)	E_m / E_i	α_E	Type	qs0 (MPa)	q_s (MPa)	q_s - used (MPa)	Q_{SR} (kN)	ϕ_s	Q_R (kN)
1	1.00	85.00	60.20	0.85	0.94	1	13.58	1.51	*1.51*	4733	0.55	2603
2	1.00	85.00	60.20	0.85	0.94	1	13.58	1.51	*1.51*	4733	0.55	2603
3	0.40	90.00	60.20	0.90	0.96	1	13.58	1.54	*1.54*	1934	0.55	1063
4	-	-	-	-	-	-	-	-	-	-	-	-
5	-	-	-	-	-	-	-	-	-	-	-	-
6	-	-	-	-	-	-	-	-	-	-	-	-
7	-	-	-	-	-	-	-	-	-	-	-	-
8	-	-	-	-	-	-	-	-	-	-	-	-
Sum	**2.40**									**11400**		**6270**

Unit base resistance $q_p = Kb.(p1-p0) + \sigma v$

Limit pressure determined from presuremeter tests	$p_l =$	5.89	MPa	*C10.8.3.5-7*
At rest total horizontal stress measured at ther base elevation	$p_0 =$	-	MPa	
Coefficient that depen on diameter socket	$K_b =$	3.84		*Table C10.8.3.5-1*
Total vertical stress at the base elevation	$\sigma v =$	-	MPa	
	$q_p =$	-	MPa	
	$\phi =$	0.50		*Table 10.5.5-3*
$Q_{pR} = Ap \,.\, qp$	$Q_{pR} =$	-	kN	
$Q_R = \phi . Q_{pR}$	$Q_R =$	-	kN	

ESTIMATED PILE CAPACITY:

Pile Structural Capacity $Q_T = 0.75*0.85*0.85*fc*Ag$	*12768*	kN	*1301*	T
Pile resistance Q_R	*6270*	kN	*639*	T
Deducting pile weight	*-88*	kN	*-9*	T
Estimated Pile Capacity (STR)	*6182*	kN	*630*	T
Estimated Pile Capacity (EXT)	*11312*	kN	*1153*	T

ASSUMPTION:

- The rock is reasonably sound
- The rock strength measured during site investigation will not deteriorate during construction when water or drilling fluids are used
- The bottom of the socket is properly cleaned out.

DATA & CALCULATION:

Bored hole name		LRB09-P2	
Bottom of pilecap elavation	$EL1 =$	7.50	
Top of socket elevation	$EL2 =$	3.30	
Pile tip elevation	$EL3 =$	-1.00	
Pile Length	$L =$	8.50	m
Diameter of drilled-shaft	$D_p =$	1.00	m
Pile Cross-Sectional Perimeter	$P =$	3.14	m
Pile Cross-Sectional Area	$A_b =$	0.79	m^2
Working normal force at pile head	$N =$	6324.4	kN
Working normal force at top of socket	$P_i =$	6274.7	kN
Intack rock modulus	$E_i =$	50000	MPa
Modulus modification ratio	$K_e =$	0.05	
Elastic modulus of the insitu rock	$E_r = K_e*E_i =$	2500.0	MPa
Influence coefficient	$I_p = f(H_s/D_s, E_c/E_r) =$	0.42	
	$H_s/D_s =$	4.30	
	$E_c/E_r =$	11.08	

Pile Concrete comp. strength	$f'_c =$	30.0	MPa
Concrete Unit Weight	$g_c =$	24.5	kN/m^3
Modulus of elasticity of concre	$E_c =$	27691	MPa
Depth of socket	$H_s =$	4.30	m
Diameter of socket	$D_s =$	1.00	m
Socket Cross-Sect. Perimeter	$P_{soc} =$	3.14	m
Socket Cross-Sectional Area	$A_{soc} =$	0.79	m^2

Figure C10.8.3.5-2 Lrfd
Figure C10.8.3.5-3 Lrfd
Figure C10.8.3.5-1 Lrfd

Rock mass modulus/ intack rock modulus E_m / E_i — *C.10.4.6.5-1-Lrfd 4th*

Atmospheric pressure $p_a =$ 0.101 MPa

Reduction factor to account for jointing α_E — *10.8.3.5.4b-Lrfd 4th*

The elastic shortening of the drilled shaft

$r_e = (\Sigma P_i)*H_s/(A_{soc}*E_c) =$ 1.241 mm

The settlement of base of drilled shaft

$r_{base} = (\Sigma P_i)*I_p/(D_s*E_r) =$ 1.054 mm

$r_e + r_{base} =$ **2.295** mm < 10 mm

Compute the bearing capacity based on Sharft resistance alone

Unit side resistance

Case 1: The drilling fluid used will not form a lubricated film on the sides of the socket

if qu < 1.9 Mpa - may be taken after Carter & Kulhawy 1988 → qs = 0.15*qu — *C10.8.3.5-4*

if qu > 1.9 Mpa - may be taken after Horvath & Kenney 1979 → qs = 0.21*sqrt(qu) — *C10.8.3.5-5*

Case 2: Side of rock socket is considered to be smooth or where the rock is drilled using a drilling slurry

May be taken after Horvath & Kenney 1979 → qs = 0.65*α_E*pa*(qu/pa)^0.5<7.8*pa*(f'c/pa)^0.5

qs0 = 7.8*pa*(f'c/pa)^0.5 — *10.8.3.5.4d-1-Lrfd2007*

Drilling method used in construction: $\boxed{\text{Case 2}}$

$Q_R = \phi*Q_n = \phi_s*Q_{SR}$

ϕ_s is the resistance factor - table 10.5.5-3 LRFD

q_u is the uniaxial compressive strength of the rock

Case1

No.	EL_T	EL_B	Depth (m)	RQD (%)	q_u (MPa)	q_s (MPa)	Q_{SR} (kN)	ϕ_s	Q_R (kN)
1	3.30	1.30	2.00	25	56.96	1.58	9958	0.65	6473
2	1.30	-0.70	2.00	35	56.96	1.58	9958	0.65	6473
3	-0.70	-1.00	0.30	45	56.96	1.58	1494	0.65	971
4									
5									
6									
7									
8									
Sum			**4.30**				**21410**		**13917**

	DANANG QUANG NGAI EXPRESSWAY	Item.	Eng.	Date.	Sign.
	LRB09 BRIDGE	Design			
	DETAIL DESIGN	Check			
	EMPIRICAL ESTIMATION OF PILE CAPACITY-ABUTMENT A2	Revise			

Case2										*Type: 1 closed joints; 2 open joints*		
No.	Depth (m)	RQD (%)	q_u (MPa)	E_m / E_i	α_E	Type	qs0 (MPa)	q_s (MPa)	*q_s - used* (MPa)	Q_{SR} (kN)	ϕ_s	Q_R (kN)
1	2.00	25.00	56.96	0.06	0.47	2	13.58	0.73	*0.73*	4571	0.55	2514
2	2.00	35.00	56.96	0.08	0.50	2	13.58	0.78	*0.78*	4898	0.55	2694
3	0.30	45.00	56.96	0.09	0.53	2	13.58	0.83	*0.83*	784	0.55	431
4	-	-	-	-	-	-	-	-	-	-	-	-
5	-	-	-	-	-	-	-	-	-	-	-	-
6	-	-	-	-	-	-	-	-	-	-	-	-
7	-	-	-	-	-	-	-	-	-	-	-	-
8	-	-	-	-	-	-	-	-	-	-	-	-
Sum	**4.30**									**10253**		**5639**

Unit base resistance $q_p = Kb.(p1-p0) + \sigma v$

Limit pressure determined from presuremeter tests	$p_l =$	5.89	MPa	*C10.8.3.5-7*
At rest total horizontal stress measured at ther base elevation	$p_0 =$	-	MPa	
Coefficient that depen on diameter socket	$K_b =$	4.66		*Table C10.8.3.5-1*
Total vertical stress at the base elevation	$\sigma v =$	-	MPa	
	$q_p =$	-	MPa	
	$\phi =$	0.50		*Table 10.5.5-3*
$Q_{pR} = Ap . qp$	$Q_{pR} =$	-	kN	
$Q_R = \phi . Q_{pR}$	$Q_R =$	-	kN	

ESTIMATED PILE CAPACITY:

Pile Structural Capacity $Q_T = 0.75*0.85*0.85*fc*Ag$	*12768*	kN	*1301*	T
Pile resistance Q_R	*5639*	kN	*575*	T
Deducting pile weight	*-131*	kN	*-13*	T
Estimated Pile Capacity (STR)	*5508*	kN	*561*	T
Estimated Pile Capacity (EXT)	*10122*	kN	*1032*	T

CALCULATION SHEET

EXPANSION JOINT

EXPANSION JOINT

I. Displacement

Maximum allowable displacements in longitudinal direction = 50.0 mm

Maximum displacement 15.4 mm OK

A1

Unit (mm)

Tải trọng	Symbol	Sign	Displacement	Service
			Case1	a
TU+	TU	+	11.05	1.20
TU-	TU	-	-10.05	1.20
Cr&Sh	CR&SH	-	-4.65	1.20
Other loads		±	2.21	1.00

Max Stretch	=	9.9
Max Shrink	=	-15.4
Maximum displacement		15.4

MINISTRY OF TRANSPORT

VIETNAM EXPRESSWAY CORPORATION - PROJECT MANAGEMENT UNIT NO.85

DA NANG - QUANG NGAI EXPRESSWAY DEVELOPMENT PROJECT

PACKAGE: 6

BRIDGE

CB13

CALCULATION SHEETS

HA NOI, 2012

Table of content - CB13 Bridge

A. Substructure design

1. Abutment A2
2. Bored pile capacity

Da Nang Quang Ngai Expressway project

BRIDGE

CB13

Km48+390.00

CALCULATION SHEETS

ABUTMENT A2

Da Nang, 2013

Table of content

1. Structure dimensions and Loads
2. Foundation analysis
3. Elements checks

Assumptions :

1. Bridge is considered to be in seismic with acceleration coeff. A = 0.0310 g
2. The Design of the Abutment accords with Specification for bridge design 22-TCN-272-05 and AASHTO LRFD 2004 for reference
3. Design live load: HL-93 and lane loading 9.3 KN/m

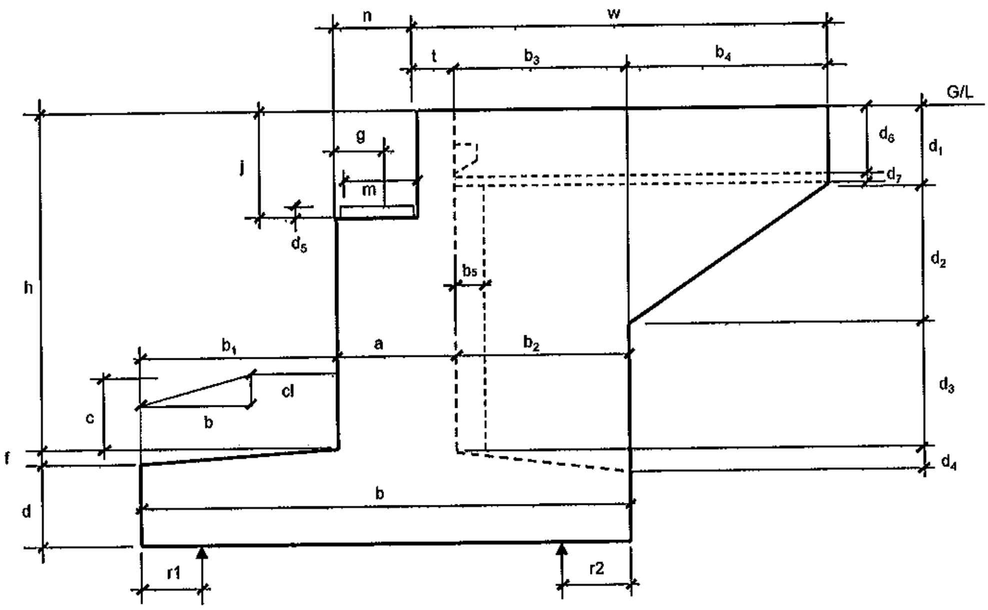
Input :

Level Table(at center of abutment)

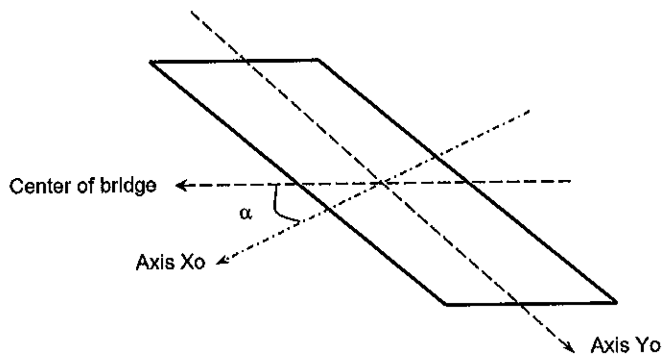
Level of top of headwall	HTwL	20.591	m
Level of top of bearing	BTL	18.627	m
Level of top of stem abutment	HTL	18.477	m
Level of top of footing	FTL	12.500	m
Level of bottom of footing	FBL	10.500	m
Ground level	GL	12.820	m
Lowest water level	HWL	13.900	m
Skew angle	α	10.00	deg

I.Loads from substructure

Abutment dimensions VERTICAL VIEW



PLAN VIEW

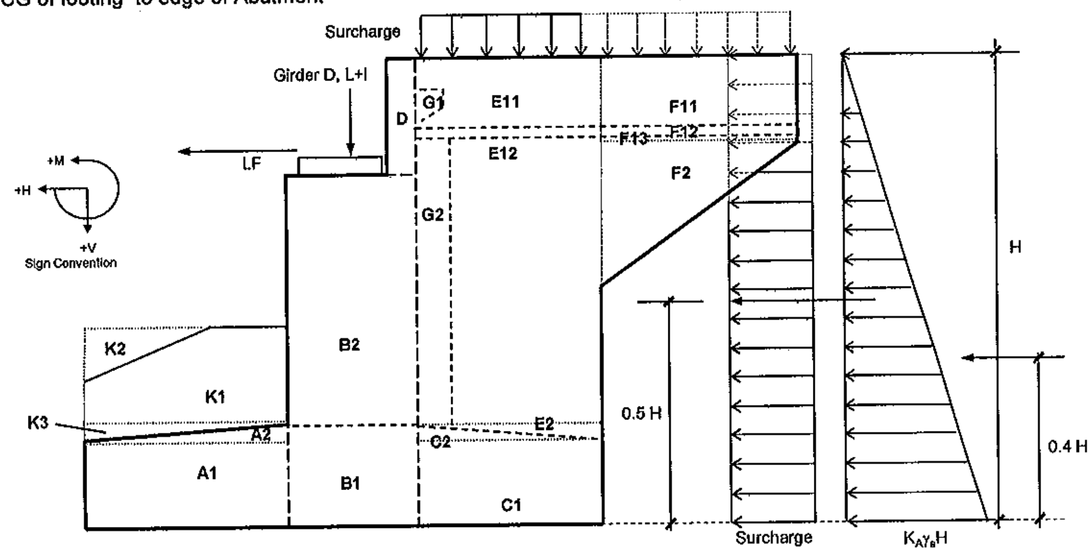


Material Unit Weights

- Unit Weight of Reinf. concrete γ_c = 24.5 kN/m^3
- Unit Weight of Soil γ_s = 18.0 kN/m^3
- Unit Bouyancy Weight of Soil γ_{sbo} = 8.2 kN/m^3

ABUTMENT DIMENSIONS (IN METRES)					
Item	Symbol	Value	Item	Symbol	Value
Height of stem	h	8.091	Horizontal Dimension	b_3	3.000
Footing Width	b	6.500	Horizontal Dimension	b_4	5.346
Stem Width	a	1.500	Horizontal Dimension	b_5	0.500
Footing Depth	d	2.000	Vertical Dimension	d_1	2.000
Footing Slope	f	0.000	Vertical Dimension	d_2	5.346
Width of stem at bearing	n	1.000	Vertical Dimension	d_3	0.745
Ballast Wall Height	j	2.114	Vertical Dimension	d_4	0.000
Ballast Wall Thickness	t	0.500	Vertical Dimension	d_5	0.150
Wingwall Length	w	8.900	Vertical Dimension	d_6	1.200
Soil Cover at Toe	c	0.320	Vertical Dimension	d_7	0.300
Girder Reaction	g	0.550	With of bearing pad	m	0.600
Distance to cl of pile	r1	1.000	Wingwall Thickness	u1	0.800
Horizontal Dimension	b_1	2.000	Wingwall Thickness	u2	0.800
Horizontal Dimension	b_2	3.000	Distance to cl of pile	r2	1.000

	cos (α)	=	0.98
Slope front of abutment	cl	=	0.00 m
	bl	=	0.00 m
Width of Abutment	L		12.600 m
Width of abutment (inclined direction)	Ltr		12.794 m
Height of Abutment	Ht	=	10.09 m
Distance from CG of footing to edge of Abutment	b/2	=	3.25 m



1. Self weight of Abutment (DC)

Description	Area (m^2)	Length (m)	Force (kN)	X[(1)] (m)	Arm[(2)] (m)	Moment (kN·m)
SW of Abutment (DC)						
Section A1	4.000	12.794	1254	1.000	2.250	2821
Section A2	-	12.794	-	1.333	1.917	-
Section B1	3.000	12.794	940	2.750	0.500	470
Section B2	8.966	12.794	2810	2.750	0.500	1405
Section C1	6.000	12.794	1881	5.000	-1.750	-3291
Section C2	-	12.794	-	4.500	-1.250	-
Section D	1.057	12.794	331	3.250	-	-
Section E11	5.100	0.800	100	5.000	-1.750	-175
Section E12	18.273	0.800	358	5.000	-1.750	-627
Part extra stem	5.046	0.740	91	5.750	-2.500	-229
Section F11	6.415	0.800	126	9.173	-5.923	-745
Section F12	1.252	0.800	25	7.673	-4.423	-109
Section F13	2.673	0.800	52	9.173	-5.923	-310
Section F2	14.290	0.800	280	8.282	-5.032	-1409
Section G1	0.135	11.994	282	3.650	-0.400	-113
Section G2	0.125	13.182	40	3.750	-0.500	-20
Bearing seats (w1seat= 0.70m)	0.090	3.500	10	2.550	0.700	7
Curbs +Handrail on Abutment	0.50	8.900	118	7.450	-4.200	-495
Total SW of Abutment (DC)			*8699*			*-2820*
Tranverser moment			*1008*		*6.100*	*6148*

Notes: 1. Distance 'X' is measured horizontally from Toe of Retaining to CG of Section
2. Moment 'Arm' is measured from CG horizontally and from Underside of Footing Vertically.

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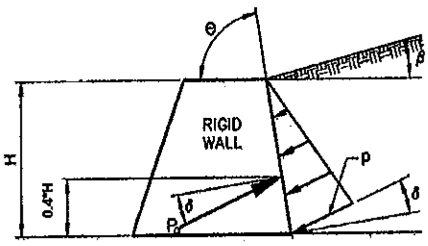
2. Earth on Abutment (EV)

Description	Area (m^2)	Length (m)	Force (kN)	X[(1)] (m)	Arm[(2)] (m)	Moment (kN•m)
Earth on Abutment (EV)						
Section E1	24.27	11.994	5241	5.000	-1.750	-9171
Section E2	-	11.994	-	5.500	-2.250	-
Section E3	-	0.800	-	6.500	-3.250	-
Section K1	0.640	12.794	147	1.000	2.250	-
Section K2	-	12.794	-	-	3.250	-
Section K3	-	12.794	-	0.667	2.583	-
Total Earth on Footing			*5388*			*-9171*

3. Horizontal Earth Pressure on Abutment (EH)

To be safe, horizontal earth pressure at front face of abutment may be neglected.
Horizontal earth pressure at behind face of abutment shall be considered.



- Height for horizontal earth pressure — H = 10.09 m
- Width for horizontal earth pressure — W 12.79 m
- Density of Soil — γ_s = 1835 kg/m^3
- Internal Friction Angle of Soil — ϕ'_f = 30.0 deg
- Incline angle of back face wall — θ = 90.0 deg
- Friction angle between fill and wall — δ = 30.0 deg
- Incline angle of fill soil — β = 0.0 deg
- Gravitational acceleration — g = 9.81 m/s^2
- Basic earth pressure

p = K.γs. g. Z .10^-9 (Mpa, Z:mm)
K: taken as Ka (assume wall move or deflect sufficiently to reach minimum active conditions)

$$Ka = \frac{\sin^2(\theta + \phi'_f)}{\Gamma.[\sin^2\theta.\sin(\theta - \delta)]}$$

Γ = 2.914
Ka = 0.297
p = 0.054 Mpa

$$\Gamma = \left[1 + \sqrt{\frac{\sin(\phi'_f + \delta).\sin(\phi'_f - \beta)}{\sin(\theta - \delta).\sin(\theta + \beta)}}\right]^{-2}$$

Horizontal earth pressure:

- Ea = 0,5. p. Z . B . 10^3 (kN) — Ea = 3484 kN
- M= Ea . 0,4H — M = 14065 kNm
- Horizontal Earth Pressure act at a height of 0.4 H — <S 3.11.5.1>

4. Earth Pressure on Abutment due to Surcharge (ES)

Equivalent height of soil for highway loading taken from Table 3.11.6.2.1

H=	1.50m heq=	1.7 m	
H=	3.00m heq=	1.2 m	
H=	6.00m heq=	0.76 m	
H=	9.00m heq=	0.61 m	
H=	10.09m heq=	0.61 m	(Linear interpolation)

• Vertical force

ESv = 421 kN
ev = -1.75 m
M = -738 kNm

• Horizontal force

$$\Delta p = k\gamma_s g h_{eq} x 10^{-9}$$

ESh = 421 kN
eh = 5.05 m
M = 2126 kNm

5. Earthquake effects

Bridge is located at: Thang Binh district - Quang Nam province

According to TCXDVN 375:2006 and 22TCN272-05, bridge is in seismic zone 2and acceleration coefficient as below

- Peak ground acceleration coefficient — A = 0.0310 g

5.1. Seismic active lateral Earth pressure (E_{AE})

• Backfill slop angle	i	=	0.0 deg
• Slope of wall to vertical	β'	=	0.0 deg
• Angle of friction of soil	ϕ	=	30.0 deg
• Angle of friction between soil and abutment	δ	=	30.0 deg
• Horizontal acceleration coefficient	kh	=	0.047
• Vertical acceleration coefficient	kv	=	0.019
• Angle $\theta = \arctan(k_h/(1-k_v))$	θ	=	2.7 deg

$$K_{AE} = \frac{\cos^2(\phi-\theta-\beta)}{\cos\theta.\cos^2\beta\cos(\delta+\beta+\theta)} x\left[1+\sqrt{\frac{\sin(\phi+\delta).\sin(\phi-\theta-i)}{\cos(\delta+\beta+\theta).\cos(i-\beta)}}\right]^{-2}$$

- Seismic active lateral Earth pressure coefficient — K_{AE} = 0.33

- $E_{AE} = 0{,}5.\ g.\ \gamma.\ H^2.\ (1-kv).\ K_{AE}.\ 10^{\wedge}-9$ (kN/m)

- Seismic active lateral Earth pressure coefficient — E_{AE} = 3800 kN

$M_{AE} = E_{AS}*0.3H+(E_{AE}-E_{AS})*0.6H$ — M_{AE} = 12460 KNm <A.11.1.1.1>

E_{AS} is the static component of seismic active pressure calculated with θ = kv = 0

5.2. Earthquake effects to abutment (EQ)

Seismic force for substructures: elements above ground Fh = Csm* W; elements under ground Fh = A*S*W

• Soil profile type	Soil type		I
• Site Coefficients.	S	=	1.0
• Elastic Seismic Response Coefficient	2.5A	=	0.078
Csm = 1.2*A*S/ Tm^2/3 ≤ 2.5*A	Csm	=	0.035
• Period of vibration of the fundamental mode			
Tm = 2*pi()*sqrt(m/k)	Tm	=	1.095 s

Decription	Area (m^2)	Length (m)	Force (kN)	X[1] (m)	Arm[2] (m)	Moment (kN•m)
Section A1	4.000	12.794	39	-	1.000	39
Section A2	-	12.794	-	-	2.000	-
Section B1	3.000	12.794	29	-	1.000	29
Section B2	8.966	12.794	98	-	4.989	491
Section C1	6.000	12.794	58	-	1.000	58
Section C2	-	12.794	-	-	2.000	-
Section D	1.057	12.794	12	-	9.034	105
Section E11	5.100	0.800	3	-	7.491	23
Section E12	18.273	0.800	11	-	3.296	
Section E2	5.046	0.740	3	-	2.000	6
Section F11	6.415	0.800	4	-	7.491	29
Section F12	1.252	0.800	1	-	6.741	
Section F13	2.673	0.800	2	-	7.841	
Section F2	14.290	0.800	9	-	6.309	55
Section G1	0.135	11.994	1	-	7.378	9
Section G2	0.125	13.182	1	-	3.296	4
Total EQ of Abutment Selfweight			***271***			***848***

6. Braking Force(BR)

Take 50 *% Braking Force for this Abutment (Free Bearing)*

• Number of lanes	n	=	3 lanes	
• Multiple presence factor	m	=	0.85	
• Take 25 % of Truck load				
BR = 25% * n * m * (2*145+35)	BR	=	104 kN	Long. Axis
• Acting at 1.8m higher of road face	e	=	12.0 m	
	Mlong	=	1239 KNm	Long. Axis

7. Centrifucal Force , CE (3.6.3)

• Plan of bridge (1:"straight",2: "Curve")			1	
• Design Speed	V	=	120 km/h	
	V	=	33.3 m/s	
	R		- m	
$C = 4/3*(V^2/gR)$	Ç		-	
Acting at 1.8m higher of road face				
CE = n * m * (2*145+35) * C	CE	=	0.00 KN	
	e	=	11.98 m	
	Mtrans	=	0.00 KNm	Trans. Axis

8. Water Load (WA) ***:NA***

II.Loads from superstructure

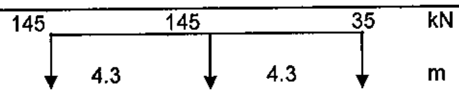
Item	Sign	Value	Unit
Span length	Lsp	27.00	m
Span between bearings	Lb	26.20	m
Skew angle	α	10.00	deg
Deck slab length	Ldeck	27.00	m
Bridge Width	Bc	12.48	m
Girder height	hgi	1.50	m
Deck slab depth	hdkslab	0.22	m
Asphalt depth	has	0.084	m
Unit weight of concrete	γc	24.50	kN/m3
Unit weight of asphalt concrete	γa	22.10	kN/m3

1.Dead loads (DC): One span at abutment

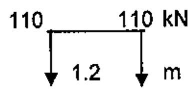
Item	Sign	Value	Unit
1.1.Girders			
Weight of 1 girder	DC	464.77	kN
Number of girders	n	5	Girders
Sum of girders weight	DC	2323.83	kN
Precast Planks	DC	394.43	kN
Diaphragm	DC	145.48	kN
Total	DC	2863.74	kN
1.2.Deck slab			
Deck slab	DC	1827.24	kN
1.3.Pavement			
Asphalt concrete	DW	575.51	kN
1.4.Handrail			
Handrail + median	DC	639.90	kN

2.Live load (LL):

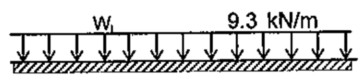
Truck



Tandem



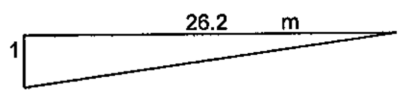
Lane load



Pedestrial Wpd= 0.0 kN/m2

Considerate structure as a simple span

Reaction Influence



Number of lanes	n	3	
Multiple presence factor	m	0.85	
Dynamic load allowance	1+IM	1.25	

*Reaction = [(1+IM)*Vehicle + Laneload]*n*m*

Truck	P1(kN)	P2(kN)	P3(kN)	Sum(kN)	Total(kN)
Axle load	145	145	35		
Influence value	1.000	0.836	0.672		
Reaction	145.0	121.2	23.5	289.7	923.5

Tandem	P1(kN)	P2(kN)	Sum(kN)	Total(kN)
Axle load	110	110		
Influence value	1.000	0.954		
Reaction	110	105.0	215.0	685.2

Lane load	Wl(kN/m)	Total(kN)
Value	9.3	
Influence value	13.1	
Reaction	121.8	310.7

Pedestrian	Wdb(kN)	Total(kN)
Reaction	0.0	0.0

3.Earthquake effects on superstructrue (EQ)

Longitudinal moveable bearings at Abutment

Horizontal force from superstructure due to EQ - transverse direction

At bearing Heq = 103 kN

4.Uniform Temperature, Srinkage& Creep(TU+SH&CR)

Bearing displacement due to uniform temperature and shrinkage creep Δu = 0.026 m

$H = G.A.\Delta u/h_{rt}$ <14.6.3.1-2>

Shear modulus G — G = 1 MPa

Bearing area — A = 0.158 m2

Height of elastomeric layers — hrt 0.064 m

Number of bearing — nb = 5 bears

Horizontal force due to TU+SH&CR

Acting at top of bearing — H(tu+sh+cr) = 320 kN

5.Wind loads (Ws)

5.1.Tranverse wind on superstructure (WS)

Wind zone	Zone		III
Basic 3 second gust wind	Vb	=	53.00 m/s
Correction factor	S	=	1.09
Design wind velocity	V	=	57.77 m/s
Drag coefficient	Cd	=	1.40
Overall width of bridge	b	=	12.48 m
Depth of superstructure(including solid parapet)	d	=	2.79 m
	b/d	=	4.48
Windy obstructed area of superstructure	At	=	75.22 m2

Force due to tranverse wind

Fhy=max(0.0006*V2*At*Cd,1.8*At) (kN) — Fhy = 210.7 kN <3.8.1>

5.2.Wind load on vehicles (WL)

Transverse wind on vehicles — Wltran = 1.50 kN/m

Transverse horizontal force due to wind on liveload

At 1.8m from surface — Fhy = 40.50 kN

6.Combinations

Loads from superstructure to Abutment

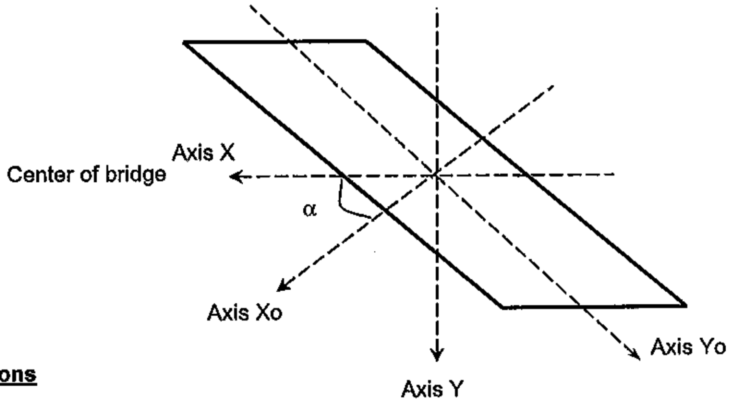
Loads at bottom of stem		Vertical		Longitudinal			Tranversal		
Loads	Sign	N (kN)	x (m)	Hx (kN)	z_1 (m)	My (kN•m)	Hy (kN)	y (m)	Mx (kN•m)
Girder + Deckslab	DC	2345	0.20			469			
Handrail	DC	320	0.20			64			
Pavement	DW	288	0.20			58			
LiveLoad	LL	1234	0.20			247		1.38	1697
Pedestrial	PL	0	0.20			0		-	-
Trans. wind on Struc.	WS						105	5.98	630
Trans. wind on vehi.	WL						20	7.78	157
Eearth quake	EQ						103	5.98	618
TU+SH&CR	TU+SH&CR			320	5.98	1912			

Loads at bottom of pilecap		Vertical		Longitudinal			Tranversal		
Loads	Sign	N (kN)	x (m)	Hx (kN)	z_1 (m)	My (kN•m)	Hy (kN)	y (m)	Mx (kN•m)
Girder + Deckslab	DC	2345	0.70			1642			
Handrail	DC	320	0.70			224			
Pavement	DW	288	0.70			201			
LiveLoad	LL	1234	0.70			864		1.38	1697
Pedestrial	PL	0	0.70			0		-	-
Trans. wind on Struc.	WS						105	7.98	840
Trans. wind on vehi.	WL						20	9.78	198
Eearth quake	EQ						103	7.98	825
TU+SH&CR	TU+SH&CR			320	7.98	2552			

Loads		Load combinations						
	Sign	Str-IA	Str-IB	Str-IIIA	Str-IIIB	Ser-I	Ext-IA	Ext-IB
Girder + Deckslab	DC	1.25	0.90	1.25	0.90	1.00	1.25	0.90
Handrail	DC	1.25	0.90	1.25	0.90	1.00	1.25	0.90
Pavement	DW	1.50	0.65	1.50	0.65	1.00	1.50	0.65
LiveLoad	LL	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Pedestrial	PL	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Trans. wind on Struc.	WS			0.40	0.40	0.30		
Trans. wind on vehi.	WL			1.00	1.00	1.00		
Eearth quake	EQ						1.00	1.00
TU+SH&CR	TU+SH&CR	0.50	0.50	0.50	0.50	1.00		

Load combinations at bottom of stem		Longitudinal		Tranversal	
Load combinations	N (kN)	Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	5923	160	2141	0	2970
Strength Str-IB	4746	160	1905	0	2970
Strength Str-IIIA	5430	160	2042	62	2700
Strength Str-IIIB	4252	160	1806	62	2700
Service Ser-I	4187	320	2750	52	2043
Extreme Ext-IA	4380	0	876	103	1467
Extreme Ext-IB	3203	0	641	103	1467

Load combinations at bottom of pilecap		Longitudinal		Tranversal	
Load combinations	N (kN)	Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	5923	160	5422	0	2970
Strength Str-IB	4746	160	4598	0	2970
Strength Str-IIIA	5430	160	5077	62	2825
Strength Str-IIIB	4252	160	4252	62	2825
Service Ser-I	4187	320	5483	52	2147
Extreme Ext-IA	4380	0	3066	103	1673
Extreme Ext-IB	3203	0	2242	103	1673



III.Load Combinations

1.Loads from substructure

Loads at bottom of pilecap	Sign	Vertical		Longitudinal			Tranversal		
		N (kN)	x (m)	H_x (kN)	z_1 (m)	M_y (kN•m)	H_y (kN)	y (m)	M_x (kN•m)
Self weight of Abutment	DC	8699				-2820			610.138
Soils on pilecap	EV	5388				-9171			
Horizontal Earth Pressure	EH			3432		13851			
Vertical Surcharge	L_{Sv}	421				-738			
Horizontal Surcharge	L_{Sh}			415		2093			
Braking Force	BR			104		1239			
Centrifucal Force	CE			-		-	-		-
Buoyancy of Abutment	WA	-1965				-5			
Buoyancy of Earth on Abutment	WA	-542				626			
Earthquake effects to Abutment	EQ			271		848	81		254
Earthquake effects to soil	E_{AE}			3742		12270			

Table of load factors

Loads	Sign	Load combinations						
		Str-IA	Str-IB	Str-IIIA	Str-IIIB	Ser-I	Ext-IA	Ext-IB
Self weight of Abutment	DC	1.25	0.90	1.25	0.90	1.00	1.25	0.90
Soils on pilecap	EV	1.35	0.90	1.35	0.90	1.00	1.35	0.90
Horizontal Earth Pressure	EH	1.50	0.90	1.50	0.90	1.00	0.00	0.00
Vertical Surcharge	L_{Sv}	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Horizontal Surcharge	L_{Sh}	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Braking Force	BR	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Centrifucal Force	CE	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Buoyancy of Abutment	WA	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Buoyancy of Earth on Abutment	WA	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Earthquake effects to Abutment	EQ						1.00	1.00
Earthquake effects to soil	E_{AE}						1.00	1.00

Load Combination at bottom of pilecap					
Load combinations		Longitudinal		Tranversal	
	N (kN)	Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	16379	6055	10034	0	763
Strength Str-IB	10910	3996	6837	0	549
Strength Str-IIIA	16211	5847	8996	0	763
Strength Str-IIIB	10741	3788	5799	0	549
Service Ser-I	12003	3950	5077	0	610
Extreme Ext-IA	15852	4272	-868	81	1017
Extreme Ext-IB	10383	4272	4246	81	804

2. Loads from superstructure

Load Combination at bottom of pilecap					
		Longitudinal		Tranversal	
Load combinations	N (kN)	Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	5923	160	5422	0	2970
Strength Str-IB	4746	160	4598	0	2970
Strength Str-IIIA	5430	160	5077	62	2825
Strength Str-IIIB	4252	160	4252	62	2825
Service Ser-I	4187	320	5483	52	2147
Extreme Ext-IA	4380	0	3066	103	1673
Extreme Ext-IB	3203	0	2242	103	1673

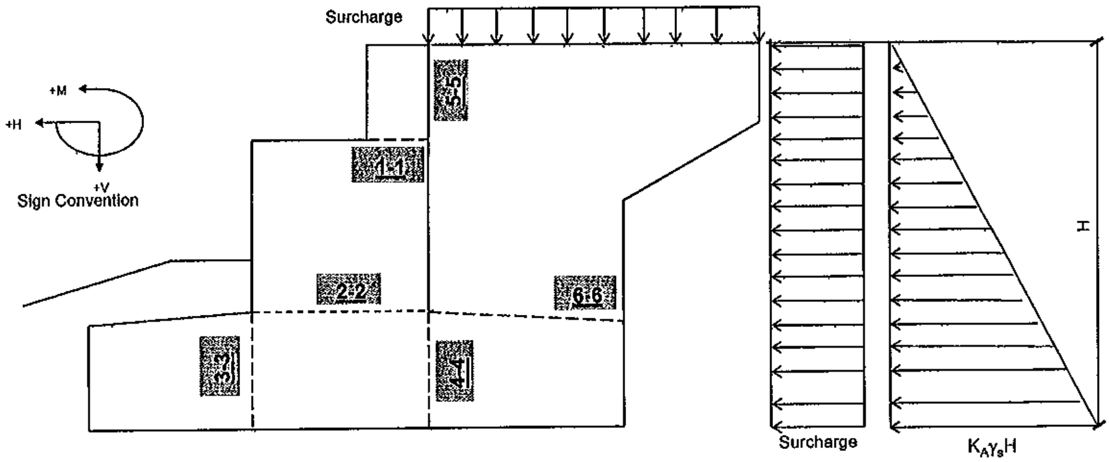
3. Total loads at bottom of pilecap

Load Combination at bottom of pilecap					
		Longitudinal		Tranversal	
Load combinations	N (kN)	Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	22302	6215	15456	0	3732
Strength Str-IB	15656	4156	11435	0	3519
Strength Str-IIIA	21640	6007	14073	62	3588
Strength Str-IIIB	14993	3948	10052	62	3374
Service Ser-I	16190	4270	10560	52	2757
Extreme Ext-IA	20233	4272	2199	185	2691
Extreme Ext-IB	13586	4272	6488	185	2477

IV.Elements checking

The abutment walls shall be checked at sections 1-1, 2-2, 3-3, 4-4, 5-5&6-6

1. Calculate internal force of sections



1.1 Section 1-1
Table of Load Factors

Loads	Sign	Service I Ser-I	Strength I Str-IA	Strength I Str-IB	Extreme I Ext-I
Selfweight	DC	1.00	1.25	0.90	1.25
Horizontal Earth Pressure	EH	1.00	1.50	0.90	
Surchage (Horizontal)	LS_h	1.00	1.75	1.75	0.50
Horizontal Seismic Earth Pressure	E_{AE}				1.50
Abutment earthquake force	EQ				1.00

Unfactored Loads	N (kN)	Longitudinal Hx (kN)	Longitudinal My (kN.m)	Tranversal Hy (kN)	Tranversal Mx (kN.m)
Selfweight	614		-113		
Horizontal Earth Pressure		155	131		
Surcharge (horizontal)		220	232		
Horizontal Seismic Earth Pressure		169	116		
Abutment earthquake force		13	14	4	4

Load Combination at bottom of headwall

Load combinations	N (kN)	Longitudinal Hx (kN)	Longitudinal My (kN.m)	Tranversal Hy (kN)	Tranversal Mx (kN.m)
Service Ser-I	614	375	251	0	0
Strength Str-IA	767	617	462	0	0
Strength Str-IB	552	524	423	0	0
Extreme Ext-I	767	377	163	4	4

1.2 Section 2-2
Table of Load Factors

Loads	Sign	Service I Ser-I	Strength I Str-IA	Strength I Str-IB	Extreme I Ext-I
Selfweight	DC	1.00	1.25	0.90	1.25
Superstructure Dead Load	DC	1.00	1.25	0.90	1.25
Pavement	DW	1.00	1.50	0.65	1.50
Handrail+curb	DC	1.00	1.25	0.90	1.25
Live Load	LL	1.00	1.75	1.75	0.50
Horizontal Earth Pressure	EH	1.00	1.50	0.90	
Surchage (Horizontal)	LS_h	1.00	1.75	1.75	0.50
TU+SH&CR	TU+SH&CR	1.00	0.50	0.50	
Horizontal Seismic Earth Pressure	E_{AE}				1.50
Abutment earthquake force	EQ				1.00

Unfactored Loads	N (kN)	Longitudinal Hx (kN)	Longitudinal My (kN.m)	Tranversal Hy (kN)	Tranversal Mx (kN.m)
Selfweight	3424		-194		
Superstructure Dead Load	2345		469		
Pavement	288		58		
Handrall+curb	320		64		
Live Load	1234		247		1697
Horizontal Earth Pressure		2275	7362		
Surchage (Horizontal)		369	1491		
TU+SH&CR		320	1912		
Horizontal Selsmic Earth Pressure		2481	6522		
Abutment earthquake force		111	385	64	306

Load Combination at bottom of stem wall

Load combinations	N (kN)	Longitudinal Hx (kN)	Longitudinal My (kN.m)	Tranversal Hy (kN)	Tranversal Mx (kN.m)
Service Ser-I	7611	2963	11409	0	1697
Strength Str-IA	10203	4217	15550	0	2970
Strength Str-IB	7827	2852	10966	0	2970
Extreme Ext-I	8660	4017	11547	64	1154

1.3 Section 3-3

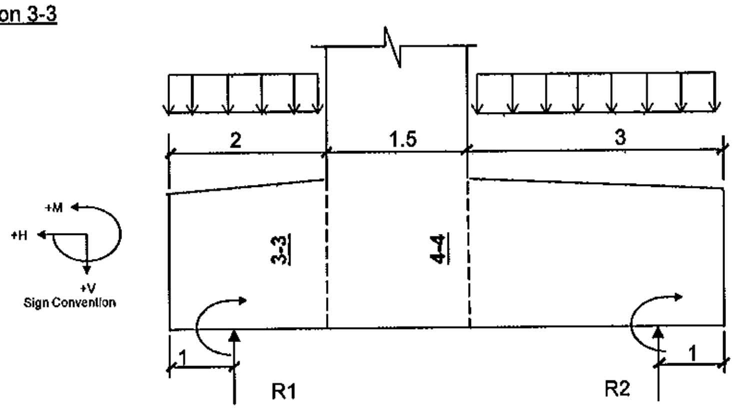


Table of Load Factors

Loads	Sign	Service I Ser-I	Strength I Str-IA	Strength I Str-IB	Extreme I Ext-I
Selfweight at front side	DC	1.00	1.25	0.90	1.25
Vertical soil on foot at front side	DC	1.00	1.35	0.90	1.35
Reaction of piles	RE	1.00	1.00	1.00	1.00

Unfactored Loads		N (kN)	Longitudinal Hx (kN)	Longitudinal My (kN.m)	Tranversal Hy (kN)	Tranversal Mx (kN.m)
Selfweight at front side		1254		1254		
Vertical soil on foot at front side		147		147		
Reaction of piles	Ser-I	-11235	-2438	-8794	45	142
	Str-IA	-15740	-3548	-12228	105	232
	Str-IB	-11085	-2377	-8778	71	179
	Ext-I	-11686	-2444	-8583	-31	189

Load Combination at section 3-3

Load combinations	N (kN)	Longitudinal Hx (kN)	Longitudinal My (kN.m)	Tranversal Hy (kN)	Tranversal Mx (kN.m)
Service Ser-I	-9834	-2438	-7393	45	142
Strength Str-IA	-13974	-3548	-10461	105	232
Strength Str-IB	-9824	-2377	-7517	71	179
Extreme Ext-I	-9920	-2444	-6816	-31	189

1.4 Section 4-4

Table of Load Factors

Loads	Sign	Service I	Strength I		Extreme I
		Ser-I	Str-IA	Str-IB	Ext-I
Selfweight at behind side	DC	1.00	1.25	0.90	1.25
Vertical soil on foot at behind side	DC	1.00	1.35	0.90	1.35
Surcharge(Vertical)	EV	1.00	1.75	1.75	0.50
Reaction of piles	RE	1.00	1.00	1.00	1.00

Unfactored Loads		N (kN)	Longitudinal Hx (kN)	My (kN.m)	Tranversal Hy (kN)	Mx (kN.m)
Selfweight of behind side		2913		-6180		
Vertical soil on foot at behind side		5241		-7861		
Surcharge(Vertical)		421		-632		
Reaction of piles	Ser-I	-4952	-1829	11734	-94	-59
	Str-IA	-6558	-2661	15750	-106	-64
	Str-IB	-4572	-1783	10874	-71	-25
	Ext-I	-8542	-1833	19412	-155	-28

Load Combination at section 4-4

Load combinations	N (kN)	Longitudinal Hx (kN)	My (kN.m)	Tranversal Hy (kN)	Mx (kN.m)
Service Ser-I	3623	-1829	-2940	-94	-59
Strength Str-IA	4896	-2661	-3694	-106	-64
Strength Str-IB	3504	-1783	-2870	-71	-25
Extreme Ext-I	2385	-1833	758	-155	-28

1.4 Section 5-5 & 6-6

Slope of triang pressure	tgβ	=	5.35
Uniform horizontal pressure	U.p	=	3.26 kN/m2

Load Combination at section 5-5

Load combinations	N (kN)	Horizontal Hx (kN)	Mx (kN.m)	Vertical Hy (kN)	My (kN.m)
Service Ser-I		118	275		
Strength Str-IA		186	437		

Load Combination at section 6-6

Load combinations	N (kN)	Horizontal Hx (kN)	Mx (kN.m)	Vertical Hy (kN)	My (kN.m)
Service Ser-I				305	269
Strength Str-IA				504	417

2. Elements Checking

2.1. Bearing Resistance

<S.5.7.5>

The case of absence of confinement reinforcement in the conrete supporting the bearing device
Factored bearing resistance shall be taken

$Pr = \phi.\ Pn = \phi.\ 0.85.\ f'c.\ A1.\ m$

Dimension of bearing plate	w0	=	0.600 m	
	b0	=	0.700 m	
Area under bearing device	A1	=	0.420 m2	
Distributed width and length	w	=	1.000 m	
	b	=	1.100 m	
Notational area	A2	=	1.100 m2	

Where supporting surface is wider on all sides than loades area

$m = \text{sqrt}(A2/A1) \leq 2.0$ case 1

where loaded area is subjected to nouniformly distributed bearing

$m = 0.75^*\text{sqrt}(A2/A1) \leq 1.5$ case 2

Modification factor	case 1	m	=	1.618	
Resistance factor		ϕ	=	0.700	<S.5.5.4.2>
Factored bearing resistance		Pr	=	12133 kN	> Pu
Bearing reaction of approach bridge		Pu	=	4404 kN	Ok

$Pu = 1.25^*DC + 1.5^*DW + 1.75^*LL$

In case factored applied load exceeds the factored resistance,
provision shall be made to resist the bursting and spalling force in article 5.10.9

Factored bearing resistance shall be taken <S.5.10.9.7.2>

$Pr = \phi.\ fn.\ Ab$

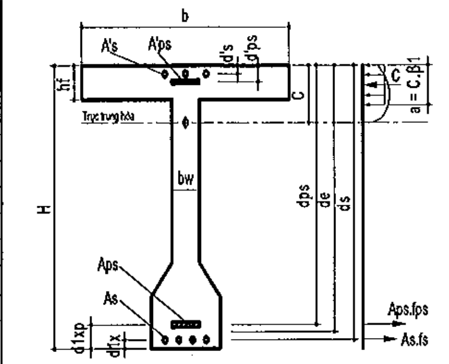
fn take the lesser of

$fn = 0.7.\ f'ci.\ \text{sqrt}(A/Ag)$ and

$fn = 2.25.\ f'ci$

	fn	=	33.99 MPa	
Maximum area of the portion of supporting surface	A	=	1.100 m2	
Gross area of bearing plate	Ag	=	0.420 m2	
Effective net area of bearing plate, Ag minus stud of bearing	Ab	=	0.420 m2	
Nominal concrete strength at time of application	f'ci	=	30 MPa	
Factored bearing resistance	Pr	=	9992 kN	Ok

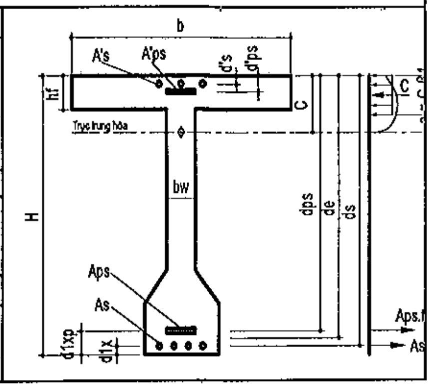
MATERIALS			
NORMAL CONCRETE			
f'c	Compressive Strength of concrete at 28 days	Mpa	30
Ec	Modulus of Elasticity	Mpa	27691
fr	Modulus of Rupture	Mpa	3.5
gc	Unit weight of concrete	kN/m3	24.5
PRESTRESSING STEEL			
fpu	Tensile strength of prestressing steel	Mpa	1860
fpy	Yield strength of prestressing steel	Mpa	1670
Ep	Modulus of Elasticity	Mpa	195000
REINFORCEMENT			
fy	Yield strength	Mpa	400
Es	Modulus of Elasticity	Mpa	200000
nc	Ratio Es/Ec		7



Sign	Parameters		Unit	Sections 1-1	1-1	2-2	2-2	2-2
INTERNAL FORCES AT SECTIOIN								
	Combination			Strength	Service	Service	Strength	Extreme
Qu	Shear		kN	617	375	2963	4217	4017
Mu	Flexural Moment		kNm	462	251	11409	15550	11547
Nu	Axial load		kN	767	614	7611	10203	8660
Tu	Torsional Moment		kNm	0	0	0	0	0
FLEXURAL MOMENT CHECKING								
H	Section height		m	0.500	0.500	1.500	1.500	1.500
d's	Dis. From comp. fiber to centroid of comp. Reinf		m	0.058	0.058	0.058	0.058	0.058
d1x	Dis. From tens. fiber to centroid of tension Reinf		m	0.058	0.058	0.059	0.059	0.059
	Cover to reinf		m	0.050	0.050	0.050	0.050	0.050
ds	Dis. From comp. fiber to centroid of tension Reinf		m	0.442	0.442	1.441	1.441	1.441
d'ps	Dis. From comp. fiber to centroid of comp. prestressing steel		m	0.000	0.000	0.000	0.000	0.000
d1xp	Dis. From tens. fiber to centroid of tension prestressing steel		m	0.000	0.000	0.000	0.000	0.000
dps	Dis. From comp. fiber to centroid of tension prestressing steel		m	0.500	0.500	1.500	1.500	1.500
b	Width of the compression face of member		m	12.794	12.794	12.794	12.794	12.794
bw	Web width or diameter of a circular section		m	12.794	12.794	12.794	12.794	12.794
hf	Compression flange depth		m	0.000	0.000	0.000	0.000	0.000
Iz	Moment of inertia of section		m4	0.133	0.133	3.598	3.598	3.598
Amc	Section area		m2	6.397	6.397	19.192	19.192	19.192
	Steel choice							
Aps	Tension prestressing steel	P.S type		0	0	0	0	0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
A'ps	Compression prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	0 T00.0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
As	Tension Reinforcement	Number	bars	83	83	77	77	77
		Diameter	mm	16	16	25	25	25
		Area	m2	0.01677	0.01677	0.03781	0.03781	0.03781
A's	Compression Reinforcement	Number	bars	83	83	77	77	77
		Diameter	mm	16	16	16	16	16
		Area	m2	0.01677	0.01677	0.01555	0.01555	0.01555
A'c	Shear reinforcement	Number	bars	20	20	19	19	19
		Diameter	mm	14	14	14	14	14
		Area	m2	0.00302	0.00302	0.00287	0.00287	0.00287
ϕ	Resistance factors for flexure		5.5.4.2	0.90	1.00	1.00	0.90	1.00
ϕv	Resistance factors for shear			0.90	1.00	1.00	0.90	1.00
ϕn	Resistance factors for axial force			1.00	1.00	1.00	1.00	1.00
β1	Stress block factor			0.836	0.836	0.836	0.836	0.836
c	Dis. Between centroid and top fiber		m	0.000	0.000	0.033	0.033	0.033
	For T section behavior		m	0.000	0.000	0.033	0.033	0.033
	For rectangular section behavior		m	0.000	0.000	0.033	0.033	0.033
fpe	Effective stress in the prestressing steel after losses		Mpa	1116	1116	1116	1116	1116
fps	Aver. stress in pres. steel at the time for which the nominal resistanc		Mpa	1860	1860	1848	1848	1848
k	Factor depends on type of P.S, Low relaxation strand k = 0.28			0.28	0.28	0.28	0.28	0.28
a	Depth of equivalent stress block		m	0.000	0.000	0.027	0.027	0.027
de	Corresp. effective depth from extreme comp. fiber							
	to centroid of tensile force in the tensile reinf.		m	0.442	0.442	1.441	1.441	1.441
Mn	Nominal resistance		kNm	2575	2575	21310	21310	21310
Mr	Factored reistance		kNm	**2318**	**2575**	**21310**	19179	**21310**
Mu	Flexual moment		kNm	462	251	11409	15550	11547

(5.7.3.2)	**Flexural moment Checking**		**OK**	**OK**	**OK**	**OK**	**OK**
	Limits for reinforcement						
c/de	Maximum reinforcement		0.00	0.00	0.02	0.02	0.02
	Maximum reinforcement Checking	<= 0.42	**OK**	**OK**	**OK**	**OK**	**OK**
1.2*Mcr	Craking moment	kNm	1104	1104	10155	10155	10155
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**		**OK**	**OK**	**OK**	**OK**	**OK**
(5.7.3.4)	**Conctrol of craking by distr. of reinf for RC member- Check?**		No	Yes	Yes	No	No
	Existing condition for structrure	1,2 or 3	1	1	1	1	1
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.058	0.058	0.059	0.059	0.059
Z	Crack width parameter	N/mm	30000	30000	30000	30000	30000
A	Area of concr. with same centroid as tens. Reinf	m2	0.018	0.018	0.020	0.020	0.020
fsa	Value	Mpa	296	296	286	286	286
0.6*fy		Mpa	240	240	240	240	240
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	240	240	240	240	240
x	Dist. From compression fiber to centroid	m	-	0.081	0.224	-	-
J.d	Arm	m	-	0.415	1.366	-	-
Icr	Moment of inertia of the cracked section	m4	-	0.018	0.442	-	-
fs	Tensile stress in reinforcement fs = Msls / (As*J.d)	Mpa	-	36	221	-	-
	Checking for control cracking fs<fsa		**N.a**	**OK**	**OK**	**N.a**	**N.a**
(5.10.8.2)	**Shrinkage and temperature Reinfocement (side distribution)**						
Areq	Area of required reinf	m2	0.00045	0.00045	0.00126	0.00126	0.00126
	Distribution on sides 7 D16	m2	0.00141	0.00141	0.00141	0.00141	
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45	0.45
	Checking		**OK**	**OK**	**OK**	**OK**	**OK**
	SHEAR AND TORSION CHECKING						
β	Factor indicating diag. cracked concr. to tension		3.0	3.8	2.3	2.1	2.3
θ	Angle of inclination of diagonal compressive	degree	28.73	27.68	34.91	38.46	35.43
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	90
bv	Effective web width as minimum web width - in dv	m	12.794	12.794	12.794	12.794	12.794
dv	Effective shear depth	m	0.442	0.442	1.427	1.427	1.427
	(de - a/2)	m	0.442	0.442	1.427	1.427	1.427
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	0.600
ncat	Amount of bars in spacing S	bars	20	20	19	19	19
Av	Shear reinf area in spacing S	m2	0.0030	0.0030	0.0029	0.0029	0.0029
θ	Assume	degree	28.87	28.19	28.88	30.16	30.27
v	Shear stress in concrete	kN/m2	121	66	162	257	220
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e_x	Strain in tensile reinforcement		3.64E-04	1.82E-04	9.09E-04	1.25E-03	9.52E-04
	if ex<0, multiple with reduce factor		-	-	-	-	-
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok	Ok
v/f'c	Ratio of shear stress and f'c		0.004	0.002	0.005	0.009	0.007
β	Final value		3.0	3.8	2.3	2.1	2.3
θ	Final value	degree	28.73	27.68	34.91	38.46	35.43
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	7819	9672	18937	17370	18736
Vs	Shear resistance provided by shear reinforcement	kN	1623	1696	3912	3437	3838
Vp	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0	0
Vn1	Vn1=Vc+Vs+Vp	kN	9442	11369	22849	20807	22574
Vn2	Vn2	kN	42413	42413	136966	136966	136966
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	9442	11369	22849	20807	22574
Vr	Factored shear resistance	kN	**8498**	**11369**	**22849**	**18726**	**22574**
Vu	Shear	kN	617	375	2963	4217	4017
(5.8.2.7)	**Shear checking**		**OK**	**OK**	**OK**	**OK**	**OK**
	Region requiring transverse reinf Checking		No need	No need	No need	No need	No need
	Minimum shear reinf area	m2	0.0087	0.0087	0.0087	0.0087	0.0087
	Mimimum shear reinforcement Checking		-	-	-	-	-
	0.1*f'c*bv*dv	kN	16965	16965	54786	54786	54786
	Smax	m	0.35	0.35	0.60	0.60	0.60
	Maximum spacing Smax		-	-	-	-	-

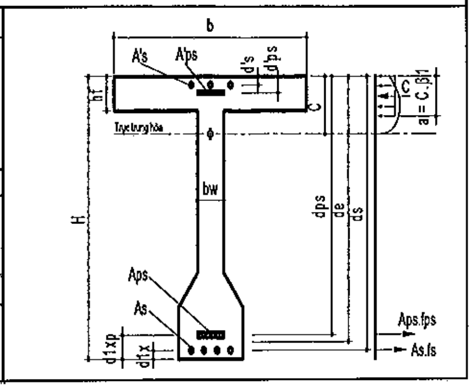
MATERIALS			
NORMAL CONCRETE			
f'c	Compressive Strength of concrete at 28 days	Mpa	30
Ec	Modulus of Elasticity	Mpa	27691
fr	Modulus of Rupture	Mpa	3.5
gc	Unit weight of concrete	kN/m3	24.5
PRESTRESSING STEEL			
fpu	Tensile strength of prestressing steel	Mpa	1860
fpy	Yield strength of prestressing steel	Mpa	1670
Ep	Modulus of Elasticity	Mpa	195000
REINFORCEMENT			
fy	Yield strength	Mpa	400
Es	Modulus of Elasticity	Mpa	200000
nc	Ratio Es/Ec		7



Sign	Parameters		Unit	Sections 3-3	3-3	3-3	4-4	4-4
INTERNAL FORCES AT SECTIOIN								
	Combination			Service	Strength	Extreme	Extreme	Strength
Qu	Shear		kN	9834	13974	9920	2385	4896
Mu	Flexural Moment		kNm	7393	10461	6816	758	3694
Nu	Axial load		kN	2438	3548	2444	1833	2661
Tu	Torsional Moment		kNm	0	0	0	0	0
FLEXURAL MOMENT CHECKING								
H	Section height		m	2.000	2.000	2.000	2.000	2.000
d's	Dis. From comp. fiber to centroid of comp. Reinf		m	0.084	0.084	0.084	0.161	0.161
d1x	Dis. From tens. fiber to centroid of tension Reinf		m	0.161	0.161	0.161	0.084	0.084
	Cover to reinf		m	0.050	0.050	0.050	0.050	0.050
ds	Dis. From comp. fiber to centroid of tension Reinf		m	1.839	1.839	1.839	1.916	1.916
d'ps	Dis. From comp. fiber to centroid of comp. prestressing steel		m	0.000	0.000	0.000	0.000	0.000
d1xp	Dis. From tens. fiber to centroid of tension prestressing steel		m	0.000	0.000	0.000	0.000	0.000
dps	Dis. From comp. fiber to centroid of tension prestressing steel		m	2.000	2.000	2.000	2.000	2.000
b	Width of the compression face of member		m	12.794	12.794	12.794	12.794	12.794
bw	Web width or diameter of a circular section		m	12.794	12.794	12.794	12.794	12.794
hf	Compression flange depth		m	0.000	0.000	0.000	0.000	0.000
Iz	Moment of inertia of section		m4	8.530	8.530	8.530	8.530	8.530
Amc	Section area		m2	25.589	25.589	25.589	25.589	25.589
	Steel choice							
Aps	Tension prestressing steel	P.S type		0	0	0	0	0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
A'ps	Compression prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	0 T00.0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
As	Tension Reinforcement	Number	bars	84	84	84	84	84
		Diameter	mm	22	22	22	20	20
		Area	m2	0.03192	0.03192	0.03192	0.02638	0.02638
A's	Compression Reinforcement	Number	bars	0	0	0	0	0
		Diameter	mm	20	20	20	22	22
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
A'c	Shear reinforcement	Number	bars	20	20	20	20	20
		Diameter	mm	16	16	16	16	16
		Area	m2	0.00404	0.00404	0.00404	0.00404	0.00404
ϕ	Resistance factors for flexure		5.5.4.2	1.00	0.90	1.00	1.00	0.90
ϕv	Resistance factors for shear			1.00	0.90	1.00	1.00	0.90
ϕn	Resistance factors for axial force			1.00	1.00	1.00	1.00	1.00
$\beta 1$	Stress block factor			0.836	0.836	0.836	0.836	0.836
c	Dis. Between centroid and top fiber		m	0.047	0.047	0.047	0.039	0.039
	For T section behavior		m	0.047	0.047	0.047	0.039	0.039
	For rectangular section behavior		m	0.047	0.047	0.047	0.039	0.039
fpe	Effective stress in the prestressing steel after losses		Mpa	1116	1116	1116	1116	1116
fps	Aver. stress in pres. steel at the time for which the nominal resistance		Mpa	1848	1848	1848	1850	1850
k	Factor depends on type of P.S, Low relaxation strand k = 0.28			0.28	0.28	0.28	0.28	0.28
a	Depth of equivalent stress block		m	0.039	0.039	0.039	0.032	0.032
de	Corresp. effective depth from extreme comp. fiber to centroid of tensile force in the tensile reinf.		m	1.839	1.839	1.839	1.916	1.916
Mn	Nominal resistance		kNm	23231	23231	23231	20044	20044
Mr	Factored reistance		kNm	**23231**	**20907**	**23231**	**20044**	**18040**
Mu	Flexual moment		kNm	7393	10461	6816	758	3694

(5.7.3.2)	**Flexural moment Checking**		**OK**	**OK**	**OK**	**OK**	**OK**
	Limits for reinforcement						
c/de	Maximum reinforcement		0.03	0.03	0.03	0.02	0.02
	Maximum reinforcement Checking	<= 0.42	**OK**	**OK**	**OK**	**OK**	**OK**
1.2*Mcr	Craking moment	kNm	18083	18083	18083	18008	18008
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**		**OK**	**OK**	**OK**	**OK**	**OK**
(5.7.3.4)	**Conctrol of craking by distr. of reinf for RC member- Check?**		Yes	No	No	No	No
	Existing condition for structrure	1,2 or 3	3	3	3	3	3
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.061	0.061	0.061	0.060	0.060
Z	Crack width parameter	N/mm	17500	17500	17500	17500	17500
A	Area of concr. with same centroid as tens. Reinf	m2	0.019	0.019	0.019	0.018	0.018
fsa	Value	Mpa	168	168	168	170	170
0.6*fy		Mpa	240	240	240	240	240
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	168	168	168	170	170
x	Dist. From compression fiber to centroid	m	0.237	-	-	-	-
J.d	Arm	m	1.76	-	-	-	-
Icr	Moment of inertia of the cracked section	m4	0.63	-	-	-	-
fs	Tensile stress in reinforcement fs = Msls / (As*J.d)	Mpa	132	-	-	-	-
	Checking for control cracking fs<fsa		OK	N.a	N.a	N.a	N.a
(5.10.8.2)	**Shrinkage and temperature Reinfocement (side distribution)**						
Areq	Area of required reinf	m2	0.00127	0.00127	0.00127	0.00127	0.00127
	Distribution on sides 7 D16	m2	0.00141	0.00141	0.00141	0.00141	0.00141
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45	0.45
	Checking		**OK**	**OK**	**OK**	**OK**	**OK**
	SHEAR AND TORSION CHECKING						
β	Factor indicating diag. cracked concr. to tension		2.0	1.8	2.0	3.4	2.3
θ	Angle of inclination of diagonal compressive	degree	40.24	42.45	39.32	28.56	33.47
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	90
bv	Effective web width as minimum web width - in dv	m	12.794	12.794	12.794	12.794	12.794
dv	Effective shear depth	m	1.819	1.819	1.819	1.900	1.900
	(de - a/2)	m	1.819	1.819	1.819	1.900	1.900
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	0.600
ncat	Amount of bars in spacing S	bars	20	20	20	20	20
Av	Shear reinf area in spacing S	m2	0.0040	0.0040	0.0040	0.0040	0.0040
θ	Assume	degree	38.22	41.43	39.67	30.88	34.58
v	Shear stress in concrete	kN/m2	422	667	426	33	224
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e_x	Strain in tensile reinforcement		1.42E-03	1.86E-03	1.33E-03	2.80E-04	7.89E-04
	if ex<0, multiple with reduce factor		-	-	-	-	-
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok	Ok
v/f'c	Ratio of shear stress and f'c		0.014	0.022	0.014	0.001	0.007
β	Final value		2.0	1.8	2.0	3.4	2.3
θ	Final value	degree	40.24	42.45	39.32	28.56	33.47
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	21089	18870	21630	37271	25945
Vs	Shear resistance provided by shear reinforcement	kN	5792	5357	5982	9401	7738
Vp	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0	0
Vn1	Vn1=Vc+Vs+Vp	kN	26881	24227	27613	46672	33683
Vn2	Vn2	kN	174589	174589	174589	182304	182304
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	26881	24227	27613	46672	33683
Vr	Factored shear resistance	kN	**26881**	**21804**	**27613**	**46672**	**30315**
Vu	Shear	kN	9834	13974	9920	2385	4896
(5.8.2.7)	**Shear checking**		**OK**	**OK**	**OK**	**OK**	**OK**

MATERIALS			
NORMAL CONCRETE			
f'c	Compressive Strength of concrete at 28 days	Mpa	30
Ec	Modulus of Elasticity	Mpa	27691
fr	Modulus of Rupture	Mpa	3.5
gc	Unit weight of concrete	kN/m3	24.5
PRESTRESSING STEEL			
fpu	Tensile strength of prestressing steel	Mpa	1860
fpy	Yield strength of prestressing steel	Mpa	1670
Ep	Modulus of Elasticity	Mpa	195000
REINFORCEMENT			
fy	Yield strength	Mpa	400
Es	Modulus of Elasticity	Mpa	200000
nc	Ratio Es/Ec		7



Sign	Parameters		Unit	Sections 5-5	5-5	6-6	6-6	6-6
INTERNAL FORCES AT SECTIOIN								
	Combination			Service	Strength	Service	Strength	Strength
Qu	Shear		kN	118	186	305	504	504
Mu	Flexural Moment		kNm	275	437	269	417	417
Nu	Axial load		kN	0	0	0	0	0
Tu	Torsional Moment		kNm	0	0	0	0	0
FLEXURAL MOMENT CHECKING								
H	Section height		m	0.800	0.800	0.800	0.800	0.800
d's	Dis. From comp. fiber to centroid of comp. Reinf		m	0.058	0.058	0.058	0.058	0.058
d1x	Dis. From tens. fiber to centroid of tension Reinf		m	0.059	0.059	0.059	0.059	0.059
	Cover to reinf		m	0.050	0.050	0.050	0.050	0.050
ds	Dis. From comp. fiber to centroid of tension Reinf		m	0.741	0.741	0.741	0.741	0.741
d'ps	Dis. From comp. fiber to centroid of comp. prestressing steel		m	0.000	0.000	0.000	0.000	0.000
d1xp	Dis. From tens. fiber to centroid of tension prestressing steel		m	0.000	0.000	0.000	0.000	0.000
dps	Dis. From comp. fiber to centroid of tension prestressing steel		m	0.800	0.800	0.800	0.800	0.800
b	Width of the compression face of member		m	1.000	1.000	1.000	1.000	1.000
bw	Web width or diameter of a circular section		m	1.000	1.000	1.000	1.000	1.000
hf	Compression flange depth		m	0.000	0.000	0.000	0.000	0.000
Iz	Moment of inertia of section		m4	0.043	0.043	0.043	0.043	0.043
Amc	Section area		m2	0.800	0.800	0.800	0.800	0.800
	Steel choice							
Aps	Tension prestressing steel	P.S type		0	0	0	0	0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
A'ps	Compression prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	0 T00.0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
As	Tension Reinforcement	Number	bars	7	7	7	7	7
		Diameter	mm	22	22	22	22	22
		Area	m2	0.00266	0.00266	0.00266	0.00266	0.00266
A's	Compression Reinforcement	Number	bars	7	7	7	7	7
		Diameter	mm	16	16	16	16	16
		Area	m2	0.00141	0.00141	0.00141	0.00141	0.00141
A'c	Shear reinforcement	Number	bars	3	3	3	3	3
		Diameter	mm	12	12	12	12	12
		Area	m2	0.00034	0.00034	0.00034	0.00034	0.00034
ϕ	Resistance factors for flexure		5.5.4.2	1.00	0.90	1.00	0.90	0.90
ϕv	Resistance factors for shear			1.00	0.90	1.00	0.90	0.90
ϕn	Resistance factors for axial force			1.00	1.00	1.00	1.00	1.00
β1	Stress block factor			0.836	0.836	0.836	0.836	0.836
c	Dis. Between centroid and top fiber		m	0.023	0.023	0.023	0.023	0.023
	For T section behavior		m	0.023	0.023	0.023	0.023	0.023
	For rectangular section behavior		m	0.023	0.023	0.023	0.023	0.023
fpe	Effective stress in the prestressing steel after losses		Mpa	1116	1116	1116	1116	1116
fps	Aver. stress in pres. steel at the time for which the nominal resistance		Mpa	1845	1845	1845	1845	1845
k	Factor depends on type of P.S, Low relaxation strand k = 0.28			0.28	0.28	0.28	0.28	0.28

a	Depth of equivalent stress block	m	0.020	0.020	0.020	0.020	0.020
de	Corresp. effective depth from extreme comp. fiber						
	to centroid of tensile force in the tensile reinf.	m	0.741	0.741	0.741	0.741	0.741
Mn	Nominal resistance	kNm	751	751	751	751	751
Mr	Factored reistance	kNm	751	676	751	676	676
Mu	Flexual moment	kNm	275	437	269	417	417
(5.7.3.2)	**Flexural moment Checking**		**OK**	**OK**	**OK**	**OK**	**OK**
	Limits for reinforcement						
c/de	Maximum reinforcement		0.03	0.03	0.03	0.03	0.03
	Maximum reinforcement Checking	<= 0.42	**OK**	**OK**	**OK**	**OK**	**OK**
r min	Mimimum reinforcement		0.33%	0.33%	0.33%	0.33%	0.33%
	Minimum reinforcement Checking for RC	0.23%	**OK**	**OK**	**OK**	**OK**	**OK**
1.2*Mcr	Craking moment	kNm	227	227	227	227	227
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**		**OK**	**OK**	**OK**	**OK**	**OK**
(5.7.3.4)	**Conctrol of craking by distr. of reinf for RC member- Check?**		Yes	No	Yes	No	No
	Existing condition for structrure	1,2 or 3	1	1	1	1	1
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.059	0.059	0.059	0.059	0.059
Z	Crack width parameter	N/mm	30000	30000	30000	30000	30000
A	Area of concr. with same centroid as tens. Reinf	m2	0.017	0.017	0.017	0.017	0.017
fsa	Value	Mpa	301	301	301	301	301
0.6*fy		Mpa	240	240	240	240	240
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	240	240	240	240	240
x	Dist. From compression fiber to centroid	m	0.149	-	0.149	-	-
J.d	Arm	m	0.691	-	0.691	-	-
Icr	Moment of inertia of the cracked section	m4	0.008	-	0.008	-	-
fs	Tensile stress in reinforcement fs = Msls / (As*J.d)	Mpa	150	-	146	-	-
	Checking for control cracking fs<fsa		**OK**	**N.a**	**OK**	**N.a**	**N.a**
(5.10.8.2)	**Shrinkage and temperature Reinfocement (side distribution)**						
Areq	Area of required reinf	m2	0.00042	0.00042	0.00042	0.00042	0.00042
	Distribution on sides 7 D16	m2	0.00141	0.00141	0.00141	0.00141	0.00141
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45	0.45
	Checking		**OK**	**OK**	**OK**	**OK**	**OK**
	SHEAR AND TORSION CHECKING						
β	Factor indicating diag. cracked concr. to tension		2.3	2.1	2.2	1.9	1.9
θ	Angle of inclination of diagonal compressive	degree	34.32	39.21	36.53	41.36	41.36
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	90
bv	Effective web width as minimum web width - in dv	m	1.000	1.000	1.000	1.000	1.000
dv	Effective shear depth	m	0.731	0.731	0.731	0.731	0.731
	(de - a/2)	m	0.731	0.731	0.731	0.731	0.731
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	0.600
ncat	Amount of bars in spacing S	bars	3	3	3	3	3
Av	Shear reinf area in spacing S	m2	0.0003	0.0003	0.0003	0.0003	0.0003
β	Assume		2.0	2.0	2.0	2.0	2.0
θ	Assume	degree	36.15	41.31	38.30	42.39	42.39
v	Shear stress in concrete	kN/m2	162	282	416	766	766
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e_x	Strain in tensile reinforcement		8.60E-04	1.32E-03	1.05E-03	1.59E-03	1.59E-03
	if ex<0, multiple with reduce factor		-	-	-	-	-
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok	Ok
v/f'c	Ratio of shear stress and f'c		0.005	0.009	0.014	0.026	0.026
β	Final value		2.3	2.1	2.2	1.9	1.9
θ	Final value	degree	34.32	39.21	36.53	41.36	41.36
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	767	681	732	634	634
Vs	Shear resistance provided by shear reinforcement	kN	242	203	223	188	188
Vp	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0	0
Vn1	Vn1=Vc+Vs+Vp	kN	1010	884	955	822	822
Vn2	Vn2	kN	5484	5484	5484	5484	5484
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	1010	884	955	822	822
Vr	Factored shear resistance	kN	1010	796	955	740	740
Vu	Shear	kN	118	186	305	504	504
(5.8.2.7)	**Shear checking**		**OK**	**OK**	**OK**	**OK**	**OK**

SPACE PILE FOUNDATION ANALYSIS PROGRM
Turbo BASIC

PROJECT: E.X Da Nang - Quang Ngai
Bridge : CB13 - KM48+390.3

INITIA DATA

Kn = 0.00   Ax = 6.50   By = 12.60   Cz = 2.00
E v.uon = 2944008   E r.uon = 2944008   E v.nen = 2944008
E r.nen = 2944008
Mq = 0 (t/m4)   Md = 0 (t/m4)   m = 20000 (t/m4)

LOAD COMBINATIONS

COMB.	Hx	Hy	P	Mx	My	Mz
1	633.00	0.00	2273.00	-380.00	1576.00	0.00
2	424.00	0.00	1596.00	-359.00	1166.00	0.00
3	612.00	6.00	2206.00	-366.00	1435.00	0.00
4	402.00	6.00	1528.00	-344.00	1025.00	0.00
5	435.00	5.00	1650.00	-281.00	1076.00	0.00
6	436.00	19.00	2062.00	-274.00	224.00	0.00
7	436.00	19.00	1385.00	-252.00	661.00	0.00

PROPERTIES OF PILES

PILE	Lo	H	Bpx	Bpy	A	B	Cday	Fo	Io	Po	Co	Ct
1	0.00	11.50	1.384	1.384	1.00	1.000	0.000	1.000	0.098	0	150000	75000
2					n	t						
3					n	t						
4					n	t						
5					n	t						
6					n	t						
7					n	t						

PILE COORD.

PILE	X	Y	Phi	Xi
1	3.11	4.66	0.000	0.00
2	2.52	1.34	0.000	0.00
3	1.94	-1.98	0.000	0.00
4	1.35	-5.30	0.000	0.00
5	-3.22	-5.30	0.000	0.00
6	-2.34	-0.32	0.000	0.00
7	-1.46	4.66	0.000	0.00

FORCES ON PILES

PILE	COMB.	N	Q2	Q3	M1	M2	M3
1	1	452.98	-97.27	3.89	0.739	7.457	98.331
	2	323.11	-65.15	2.61	0.495	5.605	64.696
	3	435.04	-94.09	2.94	0.720	6.467	96.707
	4	304.92	-61.83	1.65	0.475	4.601	62.888
	5	324.38	-66.89	1.99	0.513	4.695	68.309
	6	341.44	-67.17	0.07	0.528	5.938	85.381
	7	266.20	-67.17	0.07	0.528	2.548	74.197
2	1	418.42	-92.71	3.09	0.739	6.421	92.459
	2	296.04	-62.10	2.07	0.495	4.912	60.763
	3	401.40	-89.65	2.16	0.720	5.457	90.983

	4	278.82	-58.89	1.13	0.475	3.935	59.112
	5	299.00	-63.72	1.43	0.513	3.977	64.235
	6	312.35	-63.91	-0.51	0.528	5.199	81.188
	7	243.93	-63.91	-0.51	0.528	1.808	70.003
3	1	383.85	-88.15	2.29	0.739	5.386	86.587
	2	268.96	-59.04	1.53	0.495	4.218	56.830
	3	367.76	-85.21	1.37	0.720	4.448	85.259
	4	252.72	-55.96	0.61	0.475	3.269	55.336
	5	273.62	-60.56	0.87	0.513	3.258	60.161
	6	283.27	-60.66	-1.08	0.528	4.459	76.995
	7	221.66	-60.66	-1.08	0.528	1.069	65.810
4	1	349.28	-83.59	1.48	0.739	4.351	80.715
	2	241.88	-55.99	0.99	0.495	3.524	52.897
	3	334.13	-80.76	0.59	0.720	3.439	79.535
	4	226.61	-53.03	0.10	0.475	2.603	51.560
	5	248.24	-57.40	0.31	0.513	2.540	56.086
	6	254.18	-57.40	-1.66	0.528	3.720	72.802
	7	199.39	-57.40	-1.66	0.528	0.330	61.617
5	1	170.97	-83.59	-4.79	0.739	-3.731	80.715
	2	114.72	-55.99	-3.21	0.495	-1.889	52.897
	3	172.10	-80.76	-5.53	0.720	-4.440	79.535
	4	115.82	-53.03	-3.94	0.475	-2.594	51.560
	5	130.18	-57.40	-4.04	0.513	-3.068	56.086
	6	246.63	-57.40	-6.14	0.528	-2.051	72.802
	7	117.87	-57.40	-6.14	0.528	-5.442	61.617
6	1	222.82	-90.43	-3.59	0.739	-2.178	89.523
	2	155.34	-60.57	-2.40	0.495	-0.849	58.797
	3	222.56	-87.43	-4.35	0.720	-2.926	88.121
	4	154.98	-57.43	-3.16	0.475	-1.595	57.224
	5	168.25	-62.14	-3.20	0.513	-1.990	62.198
	6	290.25	-62.29	-5.27	0.528	-0.942	79.092
	7	151.27	-62.29	-5.27	0.528	-4.333	67.907
7	1	274.68	-97.27	-2.38	0.739	-0.625	98.331
	2	195.95	-65.15	-1.59	0.495	0.192	64.696
	3	273.01	-94.09	-3.18	0.720	-1.412	96.707
	4	194.13	-61.83	-2.39	0.475	-0.596	62.888
	5	206.32	-66.89	-2.37	0.513	-0.912	68.309
	6	333.88	-67.17	-4.41	0.528	0.167	85.381
	7	184.68	-67.17	-4.41	0.528	-3.224	74.197

SUMMARY OF FORCES

	PILE	COMB.	N	Q2	Q3	M1	M2	M3
Nmin	5	2	114.72	-55.99	-3.21	0.495	-1.889	52.897
Nmax	1	1	452.98	-97.27	3.89	0.739	7.457	98.331
Q2max	1	1	452.98	-97.27	3.89	0.739	7.457	98.331
Q3max	5	6	246.63	-57.40	-6.14	0.528	-2.051	72.802
M1max	1	1	452.98	-97.27	3.89	0.739	7.457	98.331
M2max	1	1	452.98	-97.27	3.89	0.739	7.457	98.331
M3max	1	1	452.98	-97.27	3.89	0.739	7.457	98.331

CHECKING CALCULATI
IN COMPARISON WITH INITIA LOAD MATRIX

1	633.00	0.00	2273.00	-380.00	1576.00	0.00
2	424.00	0.00	1596.00	-359.00	1166.00	0.00
3	612.00	6.00	2206.00	-366.00	1435.00	0.00
4	402.00	6.00	1528.00	-344.00	1025.00	0.00
5	435.00	5.00	1650.00	-281.00	1076.00	0.00
6	436.00	19.00	2062.00	-274.00	224.00	0.00
7	436.00	19.00	1385.00	-252.00	661.00	0.00

DANANG QUANG NGAI EXPRESSWAY CB13 BRIDGE	Item.	Eng.	Date.	Sign.
	Design			
DETAIL DESIGN CHECK REINFORCEMENT OF BORED PILE	Check			
	Revise			

I.BORED PILE DESIGN

I. BORED PILE DATA

1.Load Combinations at top of bored pile

No	Combinations	Sign	F_V (kN)	Longitudinal F_{HX} (kN)	Longitudinal My (kN•m)	Transvesal F_{HY} (kN)	Transvesal Mx (kN•m)
1	Strength Str-IB		1125	549	-519	31	19
2	Strength Str-IA		4444	954	-965	-38	-73
3	Strength Str-IA		4444	954	-965	-38	-73
4	Strength Str-IA		4444	954	-965	-38	-73
5	Strength Str-IA		4444	954	-965	-38	-73
6							

2. Bored pile Material

Normal concrete			
Compressive strength at 28 days age	f'c	30	MPa
Concrete elastic modulus	Ec	27691	MPa
Reinforcement			
Yield strength	fy	400	MPa
Reinforcement elastic modulus	Es	200,000	MPa

3. Bored pile Section

Pile diameter	D	1.20	m
Section area	A	1.131	m2
Moment inertia	Ix	0.102	m4
	Iy	0.102	m4
Radius of gyration of gross concrete section; r = sqrt(I/A)	rx	0.300	m
	ry	0.300	m

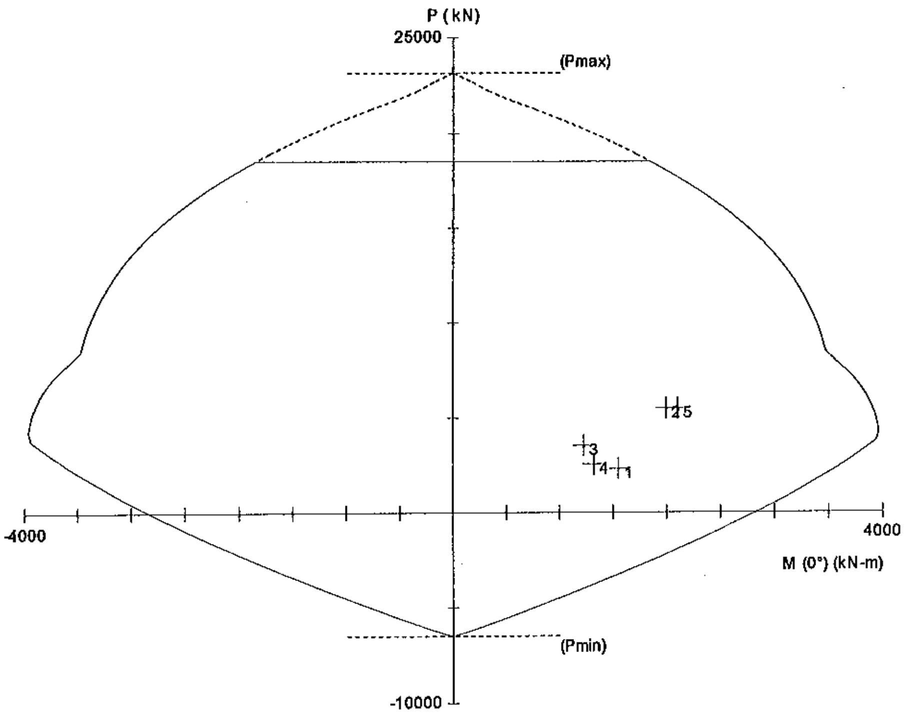
II. PILE DESIGN

1. Limit of Reinforcement

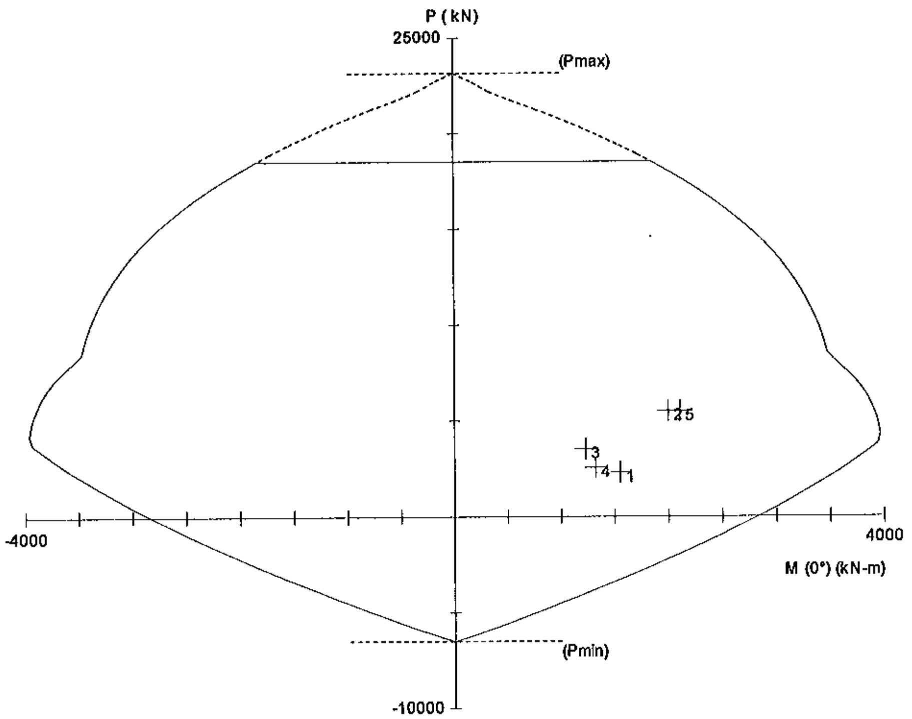
S.5.7.4.2

Minimum area of longitudinal reinforcement in column				
As.fy / (Ag . f'c) >= 0.135		As ≥	0.011	m2
As / Ag >= 0.01		As ≥	0.011	m2
Maximum area of longitudinal reinforcement in column				
As / Ag <= 0.08		As ≤	0.090	m2
Trial Rebars:	**Ok**	As	**0.015**	m2
1 layers x 24 = 24 bars	D28 @150	As1	0.015	m2

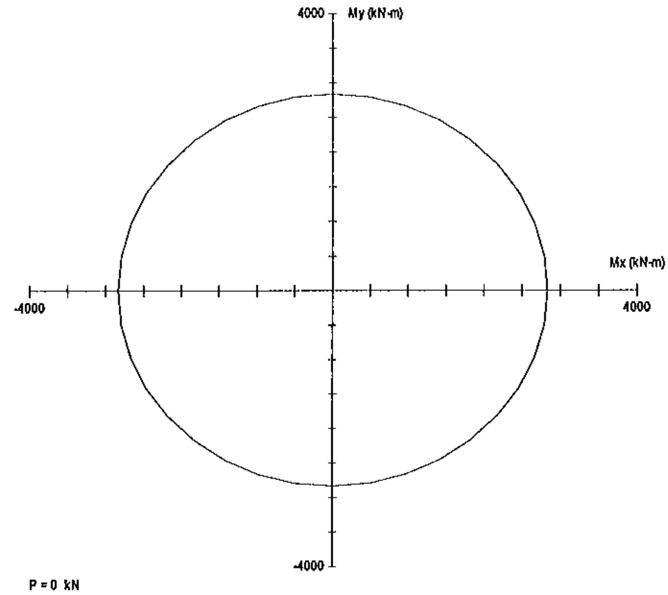
**In Transverse Direction



**In Longitudinal Direction



****In Both Direction**



3. Column Ties S.5.7.4.6, S.5.10.6.3, S5.10.11.4.1d - e

Bridge is in seismic zone	Sz	1	
Area of concrete core measured out-to-out of ties	Ac	0.933	m2
Tie diameter	Dtie	14	mm2
Cross section area of 1 tie	As-tr	0.0002	m2
Spacing of hoops	s	75	mm
Length of reinfocement tie in 1 hoop	Ltie	3.41	m
Ratio of ties reinf. in one hoop/ volume of conc. core one pitch spacing			
ρs = As-tr . Ltie / (Ac * spacing)	ρs	0.0078	
Ratio of spiral reinf. To total volume of concrete core shall satisfy			S.5.7.4.6
ρs >= 0.45 . (Ag/Ac - 1). f'c/ fy = Req1	Req1	0.0072	**OK**
Transverse Reinforcement for Confinement at Plastic Hinges		S.5.10.11.4.1.d	
For a circular column "1:applied","2:Not applied"		1	
ρs >= 0.12 . f'c/ fy = Req2	Req2	0.0090	**N/A**
Length distributed spiral with pitch 75mm below pilecap	Ldis	1.80	m

4. Shear Design

Shear resistance factors	ϕv	1.0	
Factored shear force	Vu	954	kN
Required shear capacity Vn = Vu/ ϕv	Vn	954	kN
Determine concrete shear capacity			
Minimum shear reinforcement will provided in cross section			
Therefore	β	2.0	
	θ	45.0	deg
Diameter of bored pile	D	1.20	m
Width of cross section	b	1.20	m
dv = 0.9*de de = D/2 + Dr/pi()			
Diameter of the circle passing through the centers of the long. reinf.	Dr	0.99	m
	de	0.92	m
	dv	0.82	m
Vc = 0.083 . β . sqrt(f'c) . bv. dv	Vc	900	kN
	A_v	1963	mm2
Angle of inclination of shear reinf. to long. axis	α	90	
Vs=Av*fy*dv*(cotgθ+cotgα)*sinα/s	Vs	8635	KN
Vn1 = Vc + Vs	V_{n1}	9535	
Vn2 = 0.25f'cbvdv	V_{n2}	7423	
	V_n	7423	
	Conclude		**OK**

ASSUMPTION:

- The rock is reasonably sound
- The rock strength measured during site investigation will not deteriorate during construction when water or drilling fluids are used
- The bottom of the socket is properly cleaned out.

DATA & CALCULATION:

Bored hole name **CB13-A1**

Bottom of pilecap elevation EL1 = 10.50

Top of socket elevation EL2 = 2.20

Pile tip elevation EL3 = -1.00

Pile Length L = 11.50 m

Diameter of drilled-shaft D_p = 1.00 m

Pile Cross-Sectional Perimeter P = 3.14 m

Pile Cross-Sectional Area A_b = 0.79 m^2

Working normal force at pile head N = 4665.2 kN

Working normal force at top of socket P_i = 4628.3 kN

Intack rock modulus E_i = 25000 MPa — *Figure C10.8.3.5-2 Lrfd*

Modulus modification ratio K_o = 0.50 — *Figure C10.8.3.5-3 Lrfd*

Elastic modulus of the insitu rock $E_r = K_e*E_i$ = 12500.0 MPa

Influence coefficient $I_p = f(H_s/D_s, E_c/E_r)$ = 0.30 — *Figure C10.8.3.5-1 Lrfd*

H_s/D_s = 3.20

E_c/E_r = 2.22

Pile Concrete comp. strength f'_c = 30.0 MPa

Concrete Unit Weight g_c = 24.5 kN/m^3

Modulus of elasticity of concre E_c = 27691 MPa

Depth of socket H_s = 3.20 m

Diameter of socket D_s = 1.00 m

Socket Cross-Sect. Perimeter P_{soc} = 3.14 m

Socket Cross-Sectional Area A_{soc} = 0.79 m^2

Rock mass modulus/ intack rock modulus E_m / E_i — *C.10.4.6.5-1-Lrfd 4th*

Atmospheric pressure p_a = 0.101 MPa

Reduction factor to account for jointing α_E — *10.8.3.5.4b-Lrfd 4th*

The elastic shorterning of the drilled shaft

$r_e = (\Sigma P_i)*H_s/(A_{soc}*E_c)$ = 0.681 mm

The settlement of base of drilled shaft

$r_{base} = (\Sigma P_i)*I_p/(D_s*E_r)$ = 0.111 mm

$r_e + r_{base}$ = **0.792** mm < 10 mm

Compute the bearing capacity based on Sharft resistance alone

Unit side resistance

Case 1: The drilling fluid used will not form a lubricated film on the sides of the socket

if qu < 1.9 Mpa - may be taken after Carter & Kulhawy 1988 → qs = 0.15*qu — *C10.8.3.5-4*

if qu > 1.9 Mpa - may be taken after Horvath & Kenney 1979 → qs = 0.21*sqrt(qu) — *C10.8.3.5-5*

Case 2: Side of rock socket is considered to be smooth or where the rock is drilled using a drilling slurry

May be taken after Horvath & Kenney 1979 → qs = 0.65*α_E*pa*(qu/pa)^0.5<7.8*pa*(f'c/pa)^0.5

qs0 = 7.8*pa*(f'c/pa)^0.5 — *10.8.3.5.4d-1-Lrfd2007*

Drilling method used in construction: *Case 2*

$Q_R = \phi*Q_n = \phi_s*Q_{SR}$

ϕ_s is the resistance factor - table 10.5.5-3 LRFD

q_u is the uniaxial compressive strength of the rock

Case1

No.	EL_T	EL_B	Depth (m)	RQD (%)	q_u (MPa)	q_s (MPa)	Q_{SR} (kN)	ϕ_s	Q_R (kN)
1	2.20	1.20	1.00	40	44.50	1.40	4401	0.65	2861
2	1.20	0.20	1.00	60	44.50	1.40	4401	0.65	2861
3	0.20	-1.00	1.20	80	49.78	1.48	5586	0.65	3631
4									
5									
6									
7									
8									
Sum			**3.20**				**14388**		**9352**

	DANANG QUANG NGAI EXPRESSWAY CB13 BRIDGE DETAIL DESIGN EMPIRICAL ESTIMATION OF PILE CAPACITY-ABUTMENT A1	Item.	Eng.	Date.	Sign.
		Design			
		Check			
		Revise			

Case2 *Type: "1: closed joints", "2: open joints"*

No.	Depth (m)	RQD (%)	q_u (MPa)	E_m / E_i	α_E	Type	qs0 (MPa)	q_s (MPa)	q_s - used (MPa)	Q_{SR} (kN)	ϕ_s	Q_R (kN)
1	1.00	40.00	44.50	0.12	0.56	1	13.58	0.78	*0.78*	2435	0.55	1339
2	1.00	60.00	44.50	0.42	0.76	1	13.58	1.05	*1.05*	3301	0.55	1816
3	1.20	80.00	49.78	0.80	0.92	1	13.58	1.34	*1.34*	5055	0.55	2780
4	-	-	-	-	-	-	-	-	-	-	-	-
5	-	-	-	-	-	-	-	-	-	-	-	-
6	-	-	-	-	-	-	-	-	-	-	-	-
7	-	-	-	-	-	-	-	-	-	-	-	-
8	-	-	-	-	-	-	-	-	-	-	-	-
Sum	**3.20**									**10791**		**5935**

Unit base resistance $q_p = Kb.(p1\text{-}p0) + \sigma v$

Limit pressure determined from presuremeter tests	p_l =	-	MPa	*C10.8.3.5-7*
At rest total horizontal stress measured at ther base elevation	p_0 =	-	MPa	
Coefficient that depen on diameter socket	K_b =	4.27		*Table C10.8.3.5-1*
Total vertical stress at the base elevation	σv =	-	MPa	
	q_p =	-	MPa	
	ϕ =	0.50		*Table 10.5.5-3*
$Q_{pR} = Ap \,.\, qp$	Q_{pR} =	-	kN	
$Q_R = \phi.Q_{pR}$	Q_R =	-	kN	

ESTIMATED PILE CAPACITY:

Pile Structural Capacity $Q_T = 0.75*0.85*0.85*fc*Ag$	*12768*	kN	*1301*	T
Pile resistance Q_R	*5935*	kN	*605*	T
Deducting pile weight	*-158*	kN	*-16*	T
Estimated Pile Capacity	*5778*	kN	*589*	T
Maximum Reaction - ULS Ok	*4444*	kN	*453*	T

ASSUMPTION:

- The rock is reasonably sound
- The rock strength measured during site investigation will not deteriorate during construction when water or drilling fluids are used
- The bottom of the socket is properly cleaned out.

DATA & CALCULATION:

Bored hole name		CB13-A2	
Bottom of pilecap elavation	$EL1 =$	10.50	
Top of socket elevation	$EL2 =$	1.57	
Pile tip elevation	$EL3 =$	-1.00	
Pile Length	$L =$	11.50	m
Diameter of drilled-shaft	$D_p =$	1.00	m
Pile Cross-Sectional Perimeter	$P =$	3.14	m
Pile Cross-Sectional Area	$A_b =$	0.79	m^2
Working normal force at pile head	$N =$	4665.2	kN
Working normal force at top of socket	$P_i =$	4635.5	kN
Intack rock modulus	$E_i =$	25000	MPa
Modulus modification ratio	$K_o =$	0.03	
Elastic modulus of the insitu rock	$E_r = K_e{*}E_i =$	750.0	MPa
Influence coefficient	$I_p = f(H_s/D_s, E_c/E_r) =$	0.30	
	$H_s/D_s =$	2.57	
	$E_c/E_r =$	36.92	

Pile Concrete comp. strength	$f'_c =$	30.0	MPa
Concrete Unit Weight	$g_c =$	24.5	kN/m^3
Modulus of elasticity of concre	$E_c =$	27691	MPa
Depth of socket	$H_s =$	2.57	m
Diameter of socket	$D_s =$	1.00	m
Socket Cross-Sect. Perimeter	$P_{soc} =$	3.14	m
Socket Cross-Sectional Area	$A_{soc} =$	0.79	m^2

Figure C10.8.3.5-2 Lrfd

Figure C10.8.3.5-3 Lrfd

Figure C10.8.3.5-1 Lrfd

Rock mass modulus/ intack rock modulus	E_m / E_i			*C.10.4.6.5-1-Lrfd 4th*
Atmospheric pressure	$p_a =$	0.101	MPa	
Reduction factor to account for jointing	α_E			*10.8.3.5.4b-Lrfd 4th*

The elastic shortening of the drilled shaft

$r_e = (\Sigma P_i){*}H_s/(A_{soc}{*}E_c) =$ 0.548 mm

The settlement of base of drilled shaft

$r_{base} = (\Sigma P_i){*}I_p/(D_s{*}E_r) =$ 1.854 mm

$r_e + r_{base} =$ **2.402** mm < 10 mm

Compute the bearing capacity based on Sharft resistance alone

Unit side resistance

Case 1: The drilling fluid used will not form a lubricated film on the sides of the socket

if qu < 1.9 Mpa - may be taken after Carter & Kulhawy 1988 → qs = 0.15*qu — *C10.8.3.5-4*

if qu > 1.9 Mpa - may be taken after Horvath & Kenney 1979 → qs = 0.21*sqrt(qu) — *C10.8.3.5-5*

Case 2: Side of rock socket is considered to be smooth or where the rock is drilled using a drilling slurry

May be taken after Horvath & Kenney 1979 → qs = 0.65*α_E*pa*(qu/pa)^0.5<7.8*pa*(f'c/pa)^0.5

qs0 = 7.8*pa*(f'c/pa)^0.5 — *10.8.3.5.4d-1-Lrfd2007*

Drilling method used in construction: Case 2

$Q_R = \phi{*}Q_n = \phi_s{*}Q_{SR}$

ϕ_s is the resistance factor - table 10.5.5-3 LRFD

q_u is the uniaxial compressive strength of the rock

Case1:

No.	EL_T	EL_B	Depth (m)	RQD (%)	q_u (MPa)	q_s (MPa)	Q_{SR} (kN)	ϕ_s	Q_R (kN)
1	1.57	0.07	1.50	95	76.52	1.84	8656	0.65	5627
2	0.07	-1.00	1.07	95	68.52	1.74	5843	0.65	3798
3									
4									
5									
6									
7									
8									
Sum			**2.57**				**14500**		**9425**

	DANANG QUANG NGAI EXPRESSWAY CB13 BRIDGE DETAIL DESIGN EMPIRICAL ESTIMATION OF PILE CAPACITY-ABUTMENT A2	Item.	Eng.	Date.	Sign.
		Design			
		Check			
		Revise			

Case2 — *Type: "1: closed joints", "2: open joints"*

No.	Depth (m)	RQD (%)	q_u (MPa)	E_m/E_i	α_E	Type	qs0 (MPa)	q_s (MPa)	q_s-used (MPa)	Q_{SR} (kN)	ϕ_s	Q_R (kN)
1	1.50	95.00	76.52	0.95	0.98	1	13.58	1.77	1.77	8345	0.55	4590
2	1.07	95.00	68.52	0.95	0.98	1	13.58	1.68	1.68	5633	0.55	3098
3	-	-	-	-	-	-	-	-	-	-	-	-
4	-	-	-	-	-	-	-	-	-	-	-	-
5	-	-	-	-	-	-	-	-	-	-	-	-
6	-	-	-	-	-	-	-	-	-	-	-	-
7	-	-	-	-	-	-	-	-	-	-	-	-
8	-	-	-	-	-	-	-	-	-	-	-	-
Sum	2.57									13978		7688

Unit base resistance $q_p = K_b.(p1\text{-}p0) + \sigma v$

Limit pressure determined from presuremeter tests	p_l =	5.89	MPa	*C10.8.3.5-7*
At rest total horizontal stress measured at ther base elevation	p_0 =	-	MPa	
Coefficient that depen on diameter socket	K_b =	3.94		*Table C10.8.3.5-1*
Total vertical stress at the base elevation	σv =	-	MPa	
	q_p =	-	MPa	
	ϕ =	0.50		*Table 10.5.5-3*
Q_{pR} = Ap . qp	Q_{pR} =	-	kN	
Q_R = $\phi.Q_{pR}$	Q_R =	-	kN	

ESTIMATED PILE CAPACITY:

Pile Structural Capacity Q_T = 0.75*0.85*0.85*fc*Ag	*12768*	kN	*1301*	T
Pile resistance Q_R	*7688*	kN	*784*	T
Deducting pile weight	*-153*	kN	*-16*	T
Estimated Pile Capacity	*7535*	kN	*768*	T
Maximum Reaction - ULS **Ok**	*4444*	kN	*453*	T