



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 (PKG A2)

Volume 4.2: Structural Calculation Report (PKG A2, Bridges)

Section 4.2.2

1. CB23
2. ORB22
3. OP18a
4. OP19

July 25, 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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
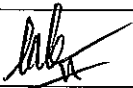
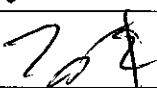
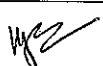

Consulting Services
for
Detailed Design for Danang - Quang Ngai Expressway Development Project
(Dịch vụ tư vấn Thiết kế kỹ thuật dự án Đường cao tốc Đà Nẵng - Quảng Ngãi)

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

Volume 4: Structural Calculation Report (PKG A2)
(Tập 4: Báo cáo tính toán kết cấu (Gói thầu A2))

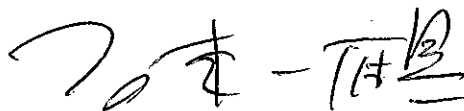
Volume 4.2: Structural Calculation Report (PKG A2, Bridges)
(Tập 4.2: Báo cáo tính toán kết cấu (Gói thầu A2, Phần cầu))

Section 4.2.2

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Date (Ngày)	July 25, 2013 (25/07/2013)	July 25, 2013 (25/07/2013)	July 25, 2013 (25/07/2013)	July 25, 2013 (25/07/2013)

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

Project Manager/Giám đốc Dự án


Ichizuru Ishimoto

Da Nang, July 25, 2013/Đà Nẵng ngày 25 tháng 07 năm 2013

1 CB23

Table of content - CB23 Bridge

A. Substructure design

1. Abutment A2L
2. Bored pile capacity

MINISTRY OF TRANSPORT

VIETNAM EXPRESSWAY CORPORATION - PROJECT MANAGEMENT UNIT NO.85

DA NANG - QUANG NGAI EXPRESSWAY DEVELOPMENT PROJECT

PACKAGE: A2

BRIDGE

CB23

CALCULATION SHEETS

SUBSTRUCTURE

CALCULATION SHEET
ABUTMENT

Table of content

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

	Da Nang Quang Ngai Expressway project	Item.	Eng.	Date:	Sign.
	CB23 BRIDGE	Design			
	DETAIL DESIGN	Check			
	ABUTMENT A2L	Revise			

LOAD COMPONENTS

Assumptions :

1. Bridge is considered to be in seismic with acceleration coeff. $A = 0.0580 g$
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 :

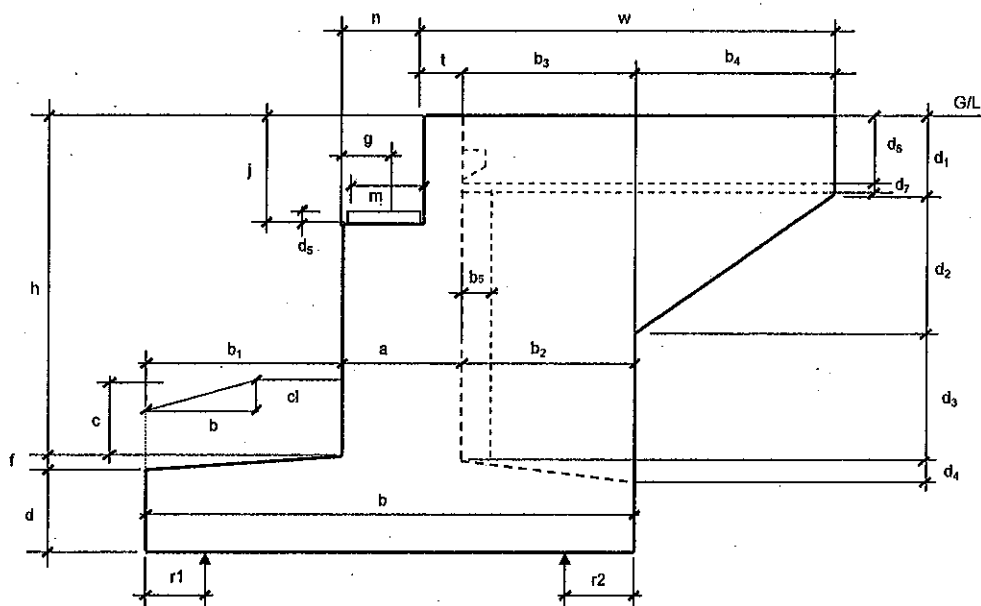
Level Table(at center of abutment)

Level of top of headwall	HTwL	19.074	m
Level of top of bearing	BTL	16.481	m
Level of top of stem abutment	HTL	16.331	m
Level of top of footing	FTL	10.500	m
Level of bottom of footing	FBL	8.500	m
Ground level	GL	11.270	m
Highest water level	HWL	12.600	m
Skew angle	α	20.00	deg

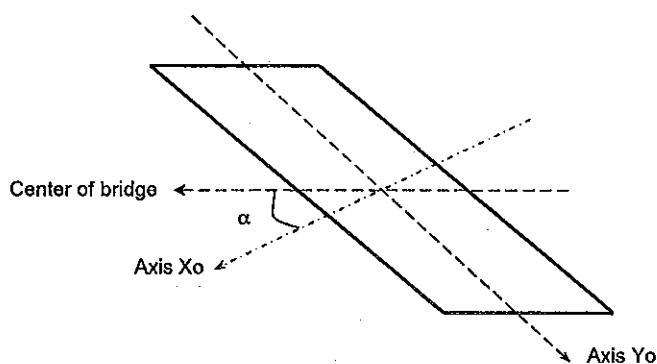
I. Loads from substructure

Abutment dimensions

VERTICAL VIEW



PLAN VIEW



Material Unit Weights

- Unit Weight of Reinf. concrete
- Unit Weight of Soil
- Unit Bouyancy Weight of Soil

$$\begin{aligned} \gamma_c &= 24.5 \text{ kN/m}^3 \\ \gamma_s &= 17.7 \text{ kN/m}^3 \\ \gamma_{sbo} &= 7.8 \text{ kN/m}^3 \end{aligned}$$

	Da Nang Quang Ngai Expressway project CB23 BRIDGE DETAIL DESIGN ABUTMENT A2L	Item.	Eng.	Date.	Sign.
		Design			
		Check			
		Revise			

ABUTMENT DIMENSIONS (IN METRES)					
Item	Symbol	Value	Item	Symbol	Value
Height of stem	h	8.574	Horizontal Dimension	b ₃	3.700
Footing Width	b	9.000	Horizontal Dimension	b ₄	3.030
Stem Width	a	1.800	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	3.030
Width of stem at bearing	n	1.300	Vertical Dimension	d ₃	3.544
Ballast Wall Height	j	2.743	Vertical Dimension	d ₄	0.000
Ballast Wall Thickness	t	0.500	Vertical Dimension	d ₅	0.150
Wingwall Length	w	7.230	Vertical Dimension	d ₆	1.200
Soil Cover at Toe	c	0.770	Vertical Dimension	d ₇	0.300
Girder Reaction	g	0.833	With of bearing pad	m	0.650
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

Slope front of abutment

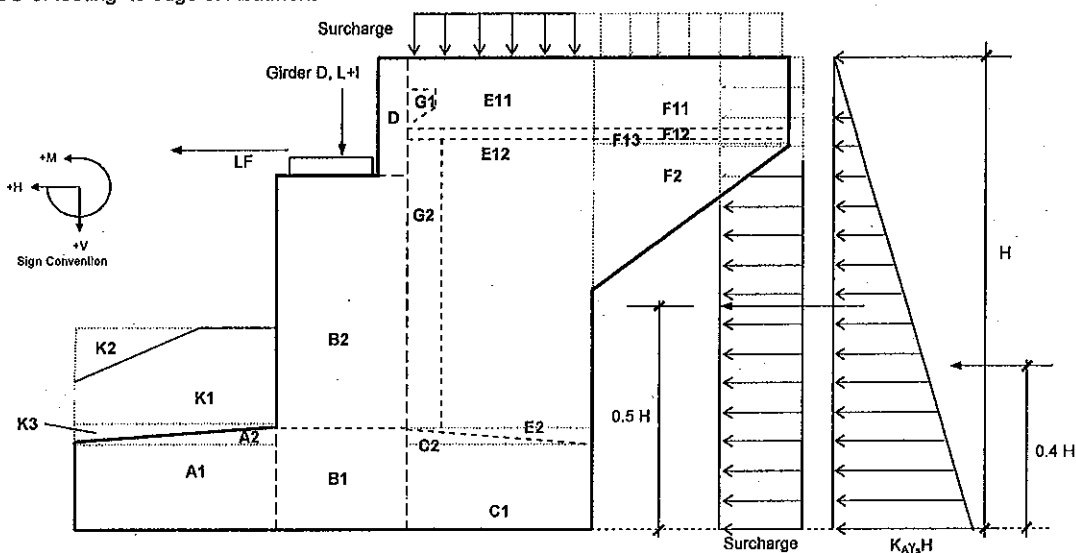
Width of Abutment

Width of abutment (inclined direction)

Height of Abutment

Distance from CG of footing to edge of Abutment

cos (α)	=	0.94
cl	=	0.00 m
bl	=	0.00 m
L	=	12.600 m
Ltr	=	13.409 m
Ht	=	10.57 m
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	13.409	2300	1.750	2.750	6324
Section A2	-	13.409	-	2.333	2.167	-
Section B1	3.600	13.409	1183	4.400	0.100	118
Section B2	10.496	13.409	3448	4.400	0.100	345
Section C1	7.400	13.409	2431	7.150	-2.650	-6442
Section C2	-	13.409	-	6.533	-2.033	-
Section D	1.372	13.409	451	5.050	-0.550	-248
Section E11	6.290	1.000	154	7.150	-2.650	-408
Section E12	24.324	1.600	953	7.150	-2.650	-2527
Part extra stem	5.287	0.740	96	8.017	-3.517	-337
Section F11	3.636	1.000	89	10.515	-6.015	-536
Section F12	1.010	1.300	32	8.665	-4.165	-134
Section F13	1.515	1.600	59	10.515	-6.015	-357
Section F2	4.590	1.600	180	10.010	-5.510	-992
Section G1	0.135	12.409	243	5.450	-0.950	-231
Section G2	0.125	14.148	43	5.550	-1.050	-45
Bearing seats (w1seat= 0.75m)	0.098	3.750	13	4.333	0.167	2
Curbs +Handrail on Abutment	0.50	7.230	96	8.415	-3.915	-375
Total SW of Abutment (DC)			11771			-5843

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.

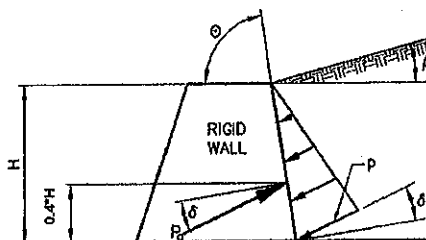
	Da Nang Quang Ngai Expressway project CB23 BRIDGE DETAIL DESIGN ABUTMENT A2L	Item.	Eng.	Date.	Sign.
		Design			
		Check			
		Revise			

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	31.72	11.809	6615	7.150	-2.650	-17530
Section E2	-	11.809	-	7.767	-3.267	-
Section E3	-	1.600	-	9.000	-4.500	-
Section K1	2.695	13.409	638	1.750	2.750	-
Section K2	-	13.409	-	-	4.500	-
Section K3	-	13.409	-	1.167	3.333	-
Total Earth on Footing			7253			-17530

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 $H = 10.57 \text{ m}$
- Width for horizontal earth pressure $W = 13.41 \text{ m}$
- Density of Soil $\gamma_s = 1800 \text{ kg/m}^3$
- Internal Friction Angle of Soil $\phi'_f = 30.0 \text{ deg}$
- Incline angle of back face wall $\theta = 90.0 \text{ deg}$
- Friction angle between fill and wall $\delta = 0.0 \text{ deg}$
- Incline angle of fill soil $\beta = 0.0 \text{ deg}$
- Gravitational acceleration $g = 9.81 \text{ m/s}^2$
- Basic earth pressure
 $p = K \cdot \gamma_s \cdot g \cdot Z \cdot 10^{-9} \text{ (Mpa, Z:mm)}$
 K : taken as K_a (assume wall move or deflect sufficiently to reach minimum active conditions)

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

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

$$\begin{aligned} \Gamma &= 2.250 \\ K_a &= 0.333 \\ p &= 0.062 \text{ Mpa} \end{aligned}$$

Horizontal earth pressure:

- $E_a = 0.5 \cdot p \cdot Z \cdot B \cdot 10^3 \text{ (kN)}$ $E_a = 4412 \text{ kN}$
- $M = E_a \cdot 0.4H$ $M = 18662 \text{ kNm}$
- Horizontal Earth Pressure act at a height of $0.4H$

<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=	10.57m	heq=	0.61 m

(Linear interpolation)

- Vertical force $E_{sv} = 534 \text{ kN}$
 $e_v = -2.65 \text{ m}$
 $M = -1416 \text{ kNm}$
 - Horizontal force $E_{sh} = 509 \text{ kN}$
 $e_h = 5.29 \text{ m}$
 $M = 2691 \text{ kNm}$
- $$\Delta p = k \gamma_s g h_{eq} \sigma^9$$

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5. Earthquake effects

Bridge is located at: Nui Thanh district - Quang Nam province

According to TCXDVN 375:2006 and 22TCN272-05, bridge is in seismic zone 1 and acceleration coefficient as below

• Peak ground acceleration coefficient $A = 0.0580 \text{ g}$

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

- Backfill slop angle $i = 0.0 \text{ deg}$
- Slope of wall to vertical $\beta' = 0.0 \text{ deg}$
- Angle of friction of soil $\phi = 30.0 \text{ deg}$
- Angle of friction between soil and abutment $\delta = 0.0 \text{ deg}$
- Horizontal acceleration coefficient $kh = 0.087$
- Vertical acceleration coefficient $k_v = 0.035$
- Angle $\theta = \arctan(k_h / (1 - k_v)) = 5.2 \text{ deg}$

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

• Seismic active lateral Earth pressure coefficient $K_{AE} = 0.39$

• $E_{AE} = 0.5 \cdot g \cdot \gamma \cdot H^2 \cdot (1 - k_v) \cdot K_{AE} \cdot 10^{-9} \text{ (kN/m)}$

• Seismic active lateral Earth pressure coefficient $E_{AE} = 4980 \text{ kN}$

$M_{AE} = E_{AS} \cdot 0.3H + (E_{AE} - E_{AS}) \cdot 0.6H$

$M_{AE} = 17598 \text{ KNm}$

<A.11.1.1.1>

E_{AS} is the static component of seismic active pressure calculated with $\theta = k_v = 0$

5.2. Earthquake effects to abutment (EQ)

Seismic force for substructures: elements above ground $F_h = C_{sm} \cdot W$; elements under ground $F_h = A \cdot S \cdot W$

- Soil profile type I
- Site Coefficients. $S = 1.0$
- Elastic Seismic Response Coefficient $2.5A = 0.145$
- $C_{sm} = 1.2 \cdot A \cdot S / T_m^{2/3} \leq 2.5 \cdot A$ $C_{sm} = 0.068$
- Period of vibration of the fundamental mode $T_m = 2 \cdot \pi \cdot \sqrt{m/k}$ $T_m = 1.039 \text{ s}$

Description	Area (m ²)	Length (m)	Force (kN)	$\chi^{(1)}$ (m)	Arm ⁽²⁾ (m)	Moment (kN·m)
Section A1	7.000	13.409	133	-	1.000	133
Section A2	-	13.409	-	-	2.000	-
Section B1	3.600	13.409	69	-	1.000	69
Section B2	10.496	13.409	234	-	4.916	1150
Section C1	7.400	13.409	141	-	1.000	141
Section C2	-	13.409	-	-	2.000	-
Section D	1.372	13.409	31	-	9.203	281
Section E11	6.290	1.000	9	-	7.974	71
Section E12	24.324	1.600	55	-	3.537	-
Section E2	5.287	0.740	6	-	2.000	11
Section F11	3.636	1.000	5	-	7.974	41
Section F12	1.010	1.300	2	-	7.224	-
Section F13	1.515	1.600	3	-	8.324	-
Section F2	4.590	1.600	10	-	7.564	79
Section G1	0.135	12.409	2	-	7.861	19
Section G2	0.125	14.148	3	-	3.537	9
Total EQ of Abutment Selfweight			703			2004

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6. Braking Force(BR)

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

- Number of lanes
- Multiple presence factor
- Take 25 % of Truck load
- BR = 25% * n * m * (2*145+35)
- Acting at 1.8m higher of road face

n	=	3 lanes	
m	=	0.85	
BR	=	104 kN	Long. Axis
e	=	12.4 m	
Mlong	=	1289 KNm	Long. Axis

7. Centrifugal Force , CE (3.6.3)

- Plan of bridge (1:"straight",2: "Curve")
- Design Speed

$$C = 4/3 * (V^2 / gR)$$

Acting at 1.8m higher of road face

$$CE = n * m * (2*145+35) * C$$

	=	1	
V	=	60 km/h	
V	=	16.7 m/s	
R	=	- m	
C	=	-	
CE	=	0.00 KN	
e	=	12.46 m	
Mtrans	=	0.00 KNm	Trans. Axis

8. Water Load (WA) :NA

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SUPERSTRUCTURE LOADS

II. Loads from superstructure

Item	Sign	Value	Unit
Span length	Lsp	40.00	m
Span between bearings	Lb	39.10	m
Skew angle	α	20.00	deg
Deck slab length	Ldeck	40.00	m
Bridge Width	Bc	12.74	m
Girder height	hgl	2.10	m
Deck slab depth	hdkslab	0.224	m
Asphalt depth	has	0.084	m
Unit weight of concrete	yc	24.50	kN/m3
Unit weight of asphalt concrete	ya	22.10	kN/m3

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

Item	Sign	Value	Unit
1.1. Girders			
Weight of 1 girder	DC	1056.20	kN
Number of girders	n	5	Girders
Sum of girders weight	DC	5280.98	kN
Precast Planks	DC	619.05	kN
Diaphragm	DC	863.97	kN
Total	DC	6763.99	kN
1.2. Deck slab			
Deck slab	DC	2540.74	kN
1.3. Pavement			
Asphalt concrete	DW	797.51	kN
1.4. Handrail			
Handrail + median	DC	948.00	kN

2. Live load (LL):

Truck	
Tandem	
Lane load	
Pedestrian	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

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

Truck	P1(kN)	P2(kN)	P3(kN)	Sum(kN)	Total(kN)
Axle load	145	145	35		
Influence value	1.000	0.890	0.780		
Reaction	145.0	129.1	27.3	301.4	960.6

Tandem	P1(kN)	P2(kN)	Sum(kN)	Total(kN)
Axle load	110	110		
Influence value	1.000	0.969		
Reaction	110	106.6	216.6	690.5

Lane load	Wl(kN/m)	Total(kN)
Value	9.3	
Influence value	19.55	
Reaction	181.8	463.6

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

3. Earthquake effects on superstructure (EQ)

Longitudinal moveable bearings at Abutment

Horizontal force from superstructure due to EQ - transverse direction

At bearing

$$H_{eq} = 375 \text{ kN}$$

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

Bearing displacement due to uniform temperature and shrinkage creep

$$H = G.A.\Delta u/h_{rt}$$

Shear modulus G

Bearing area

Height of elastomeric layers

Number of bearing

Horizontal force due to TU+SH&CR

Acting at top of bearing

$$\Delta u = 0.026 \text{ m}$$

<14.6.3.1-2>

$$G = 1 \text{ MPa}$$

$$A = 0.165 \text{ m}^2$$

$$h_{rt} = 0.065 \text{ m}$$

$$nb = 5 \text{ bears}$$

$$H(tu+sh+cr) = 330 \text{ kN}$$

5. Wind loads (Ws)

5.1. Transverse wind on superstructure (WS)

Wind zone

Basic 3 second gust wind

Correction factor

Design wind velocity

Drag coefficient

Overall width of bridge

Depth of superstructure (including solid parapet)

Windy obstructed area of superstructure

Force due to transverse wind

$$F_{hy} = \max(0.0006 \cdot V^2 \cdot A_t \cdot C_d, 1.8 \cdot A_t) \text{ (kN)}$$

Zone III

$$V_b = 53.00 \text{ m/s}$$

$$S = 1.09$$

$$V = 57.77 \text{ m/s}$$

$$C_d = 1.45$$

$$b = 12.74 \text{ m}$$

$$d = 3.39 \text{ m}$$

$$b/d = 3.76$$

$$A_t = 135.56 \text{ m}^2$$

$$F_{hy} = 392.4 \text{ kN} \quad <3.8.1>$$

5.2. Wind load on vehicles (WL)

Transverse wind on vehicles

Transverse horizontal force due to wind on live load

At 1.8m from surface

$$W_{ltran} = 1.50 \text{ kN/m}$$

$$F_{hy} = 60.00 \text{ kN}$$

6. Combinations

Loads from superstructure to Abutment

Loads at bottom of stem		Vertical		Longitudinal			Transversal		
Loads	Sign	N (kN)	x (m)	Hx (kN)	z ₁ (m)	My (kN·m)	Hy (kN)	y (m)	Mx (kN·m)
Girder + Deckslab	DC	4652	0.07			312			
Handrail	DC	474	0.07			32			
Pavement	DW	399	0.07			27			
LiveLoad	LL	1424	0.07			95		1.68	2386
Pedestrian	PL	0	0.07			0		-	-
Trans. wind on Struc.	WS						196	5.83	1144
Trans. wind on vehi.	WL						30	7.63	229
Earth quake	EQ						375	5.83	2186
TU+SH&CR	TU+SH&CR			330	5.83	1924			

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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 + Decks slab	DC	4652	0.17			777			
Handrail	DC	474	0.17			79			
Pavement	DW	399	0.17			67			
LiveLoad	LL	1424	0.17			238		1.68	2386
Pedestrian	PL	0	0.17			0		-	-
Trans. wind on Struc.	WS						196	7.83	1537
Trans. wind on vehi.	WL						30	9.63	289
Eearth quake	EQ						375	7.83	2935
TU+SH&CR	TU+SH&CR			330	7.83	2584			

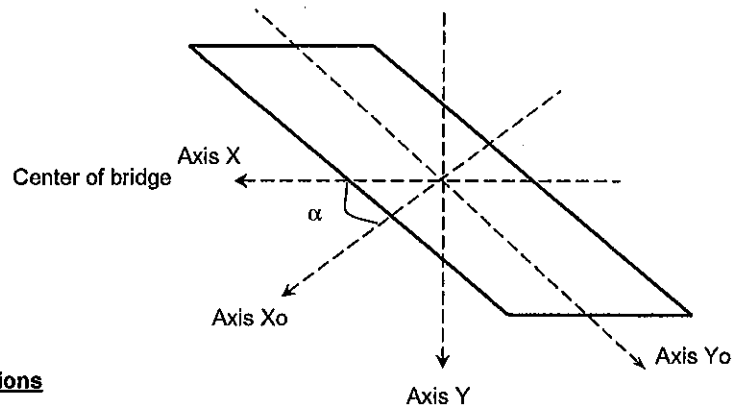
Loads	Sign	Load combinations						
		Str-IA	Str-IB	Str-IIIA	Str-IIIB	Ser-I	Ext-IA	Ext-IB
Girder + Decks slab	DC	1.25	0.90	1.25	0.90	1.00	1.25	0.90
Handrail	DC	1.25	0.90	1.25	0.90	1.00	1.25	0.90
Pavement	DW	1.50	0.65	1.50	0.65	1.00	1.50	0.65
LiveLoad	LL	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Pedestrian	PL	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					
Load combinations	N (kN)	Longitudinal		Tranversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	9498	165	1599	0	4175
Strength Str-IB	7365	165	1456	0	4175
Strength Str-IIIA	8929	165	1560	108	3907
Strength Str-IIIB	6796	165	1417	108	3907
Service Ser-I	6949	330	2390	89	2958
Extreme Ext-IA	7718	0	517	375	3378
Extreme Ext-IB	5585	0	374	375	3378

Load combinations at bottom of pilecap					
Load combinations	N (kN)	Longitudinal		Tranversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	9498	165	2878	0	4175
Strength Str-IB	7365	165	2522	0	4175
Strength Str-IIIA	8929	165	2783	108	4124
Strength Str-IIIB	6796	165	2427	108	4124
Service Ser-I	6949	330	3745	89	3135
Extreme Ext-IA	7718	0	1289	375	4128
Extreme Ext-IB	5585	0	933	375	4128

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LOAD COMBINATIONS



III. Load Combinations

1. Loads from substructure

Loads at bottom of pilecap	Sign	Vertical		Longitudinal			Tranversal		
		N (kN)	x (m)	Hx (kN)	z ₁ (m)	My (kN·m)	Hy (kN)	y (m)	Mx (kN·m)
Self weight of Abutment	DC	11771				-5843			639.342
Soils on pilecap	EV	7253				-17530			
Horizontal Earth Pressure	EH			4695		19859			
Vertical Surcharge	LSv	534				-1416			
Horizontal Surcharge	LSH			542		2864			
Braking Force	BR			104		1289			
Centrifugal Force	CE			-		-			-
Buoyancy of Abutment	WA	-2987				273			
Buoyancy of Earth on Abutment	WA	-1255				1410			
Earthquake effects to Abutment	EQ			703		2004	211		601
Earthquake effects to soil	E _{AE}			5299		18728			

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
Centrifugal 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	21199	8172	5294	0	799
Strength Str-IB	13815	5355	3312	0	575
Strength Str-IIIA	20985	7914	4199	0	799
Strength Str-IIIB	13601	5097	2217	0	575
Service Ser-I	15317	5341	908	0	639
Extreme Ext-IA	20531	6325	-7185	211	1400
Extreme Ext-IB	13147	6325	2749	211	1177

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2. Loads from superstructure

Load Combination at bottom of pilecap					
Load combinations	N (kN)	Longitudinal		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	9498	165	2878	0	4175
Strength Str-IB	7365	165	2522	0	4175
Strength Str-IIIA	8929	165	2783	108	4424
Strength Str-IIIB	6796	165	2427	108	4124
Service Ser-I	6949	330	3745	89	3135
Extreme Ext-IA	7718	0	1289	375	4128
Extreme Ext-IB	5585	0	933	375	4128

3. Total loads at bottom of pilecap

Load Combination at bottom of pilecap					
Load combinations	N (kN)	Longitudinal		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	30697	8337	8172	0	4974
Strength Str-IB	21180	5520	5834	0	4750
Strength Str-IIIA	29914	8079	6982	108	4923
Strength Str-IIIB	20397	5262	4644	108	4699
Service Ser-I	22266	5671	4652	89	3775
Extreme Ext-IA	28249	6325	-5896	586	5528
Extreme Ext-IB	18732	6325	3681	586	5305

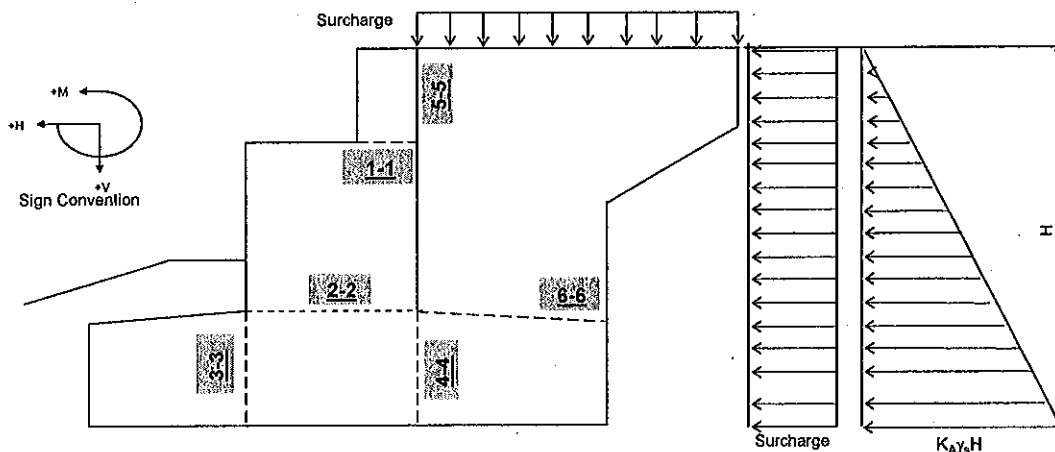
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ELEMENTS CHECKING

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 Ser-I	Strength I		Extreme I Ext-I
			Str-IA	Str-IB	
Selfweight	DC	1.00	1.25	0.90	1.25
Horizontal Earth Pressure	EH	1.00	1.50	0.90	
Surcharge (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		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Selfweight	694		-97		
Horizontal Earth Pressure		316	347		
Surcharge (horizontal)		296	406		
Horizontal Seismic Earth Pressure		357	327		
Abutment earthquake force		33	45	10	14

Load Combination at bottom of headwall

Load combinations	N (kN)	Longitudinal		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Service Ser-I	694	612	656	0	0
Strength Str-IA	867	992	1109	0	0
Strength Str-IB	624	803	935	0	0
Extreme Ext-I	867	716	617	10	14

1.2 Section 2-2

Table of Load Factors

Loads	Sign	Service I Ser-I	Strength I		Extreme I Ext-I
			Str-IA	Str-IB	
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	
Surcharge (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

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Unfactored Loads	N (kN)	Longitudinal		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Selfweight	4142		-354		
Superstructure Dead Load	4652		312		
Pavement	399		27		
Handrail+curb	474		32		
Live Load	1424		95		2386
Horizontal Earth Pressure		3087	10588		
Surcharge (Horizontal)		455	1949		
TU+SH&CR		330	1924		
Horizontal Seismic Earth Pressure		3484	9984		
Abutment earthquake force		267	921	192	949

Load Combination at bottom of stem wall					
Load combinations	N (kN)	Longitudinal		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Service Ser-I	11091	3872	14573	0	2386
Strength Str-IA	14676	5591	20449	0	4175
Strength Str-IB	11093	3739	14077	0	4175
Extreme Ext-I	12895	5721	16947	192	2142

1.3 Section 3-3

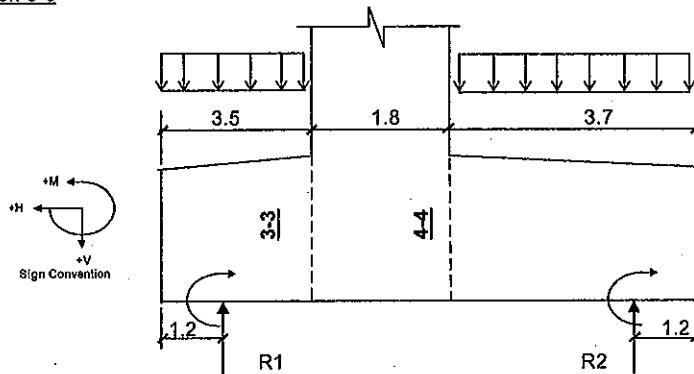


Table of Load Factors

Loads	Sign	Service I Ser-I	Strength I		Extreme I Ext-I
			Str-IA	Str-IB	
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		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Selfweight at front side	-2300		-4024		
Vertical soil on foot at front side	-638		-1117		
Reaction of piles					
Ser-I	10381	2118	30220	-9	23
Str-IA	14840	3115	43435	34	176
Str-IB	10158	2063	29522	22	124
Ext-I	10894	2360	32293	-187	-524

Load Combination at section 3-3					
Load combinations	N (kN)	Longitudinal		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Service Ser-I	7443	2118	25079	-9	23
Strength Str-IA	11104	3115	36898	34	176
Strength Str-IB	7514	2063	24895	22	124
Extreme Ext-I	7158	2360	25755	-187	-524

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1.4 Section 4-4

Table of Load Factors

Loads	Sign	Service I Ser-I	Strength I		Extreme 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		Tranversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Selfweight of behind side	-3995		-8572		
Vertical soil on foot at behind side	-6615		-12238		
Surcharge(Vertical)	-534		-989		
Reaction of piles					
	Ser-I	3824	1425	5290	-43
	Str-IA	4816	2095	5781	-29
	Str-IB	3392	1388	4338	-19
	Ext-I	6803	1592	12126	-177

Load Combination at section 4-4					
Load combinations	N (kN)	Longitudinal		Tranversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Service Ser-I	-7321	1425	-16508	-43	-98
Strength Str-IA	-10043	2095	-23185	-29	-40
Strength Str-IB	-7092	1388	-16121	-19	-22
Extreme Ext-I	-7388	1592	-15604	-177	-510

1.4 Section 5-5 & 6-6

Slope of triang pressure
Uniform horizontal pressure

tgβ = 5.89
U.p = 3.59 kN/m2

Load Combination at section 5-5					
Load combinations	N (kN)	Horizontal		Vertical	
		Hx (kN)	Mx (kN.m)	Hy (kN)	My (kN.m)
Service Ser-I		168	395		
Strength Str-IA		252	487		

Load Combination at section 6-6					
Load combinations	N (kN)	Horizontal		Vertical	
		Hx (kN)	Mx (kN.m)	Hy (kN)	My (kN.m)
Service Ser-I				340	471
Strength Str-IA				474	661

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REINFORCEMENT CHECKING - HEAD AND STEM 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				
			1-1	1-1	2-2	2-2	2-2
INTERNAL FORCES AT SECTION							
	Combination		Strength	Service	Service	Strength	Extreme
Qu	Shear	kN	992	612	3872	5591	5721
Mu	Flexural Moment	kNm	1109	656	14573	20449	16947
Nu	Axial load	kN	867	694	11091	14676	12895
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	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	6.517	6.517	6.517
Amc	Section area	m2	6.704	6.704	24.136	24.136	24.136
	Steel choice						
Aps	Tension prestressing steel	P.S type	0	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	0 T00.0
		Number	tendons	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000
As	Tension Reinforcement	Number	bars	83	78	78	78
		Diameter	mm	16	32	32	32
		Area	m2	0.01677	0.01677	0.06248	0.06248
A's	Compression Reinforcement	Number	bars	83	78	78	78
		Diameter	mm	16	16	16	16
		Area	m2	0.01677	0.01677	0.01576	0.01576
A'c	Shear reinforcement	Number	bars	21	20	20	20
		Diameter	mm	14	14	14	14
		Area	m2	0.00317	0.00317	0.00302	0.00302
φ	Resistance factors for flexure		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	0.065	0.065	0.065
	For T section behavior	m	0.000	0.000	0.065	0.065	0.065
	For rectangular section behavior	m	0.000	0.000	0.065	0.065	0.065
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	1841	1841	1841
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 - HEAD AND STEM WALL

a	Depth of equivalent stress block	m	0.000	0.000	0.055	0.055	0.055
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	2575	2575	42633	42633	42633
Mr	Factored resistance	kNm	2318	2575	42633	38370	42633
Mu	Flexural moment	kNm	1109	656	14573	20449	16947
(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*Mcrr	Cracking moment	kNm	1157	1157	15556	15556	15556
(5.7.3.3.2)	Checking $M_r \geq \min(1.2M_{crr}, 1.33M_u)$		OK	OK	OK	OK	OK
(5.7.3.4)	Control of cracking by distr. of reinf for RC member- Check?		No	Yes	Yes	No	No
	Existing condition for structure	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	m ²	0.019	0.019	0.020	0.020	0.020
fsa	Value	Mpa	292	292	283	283	283
0.6*fy		Mpa	240	240	240	240	240
	Tensile stress in reinf $\min(fsa, 0.6fy)$	Mpa	240	240	240	240	240
x	Dist. From compression fiber to centroid	m	-	0.08	0.306	-	-
J.d	Arm	m	-	0.415	1.639	-	-
Icr	Moment of inertia of the cracked section	m ⁴	-	0.018	1.034	-	-
fs	Tensile stress in reinforcement $f_s = M_{sls} / (A_s * J.d)$	Mpa	-	94	142	-	-
	Checking for control cracking $f_s < f_{sa}$		N.a	OK	OK	N.a	N.a
(5.10.8.2)	Shrinkage and temperature Reinforcement (side distribution)						
Areq	Area of required reinf	m ²	0.00045	0.00045	0.00127	0.00127	0.00127
	Distribution on sides 7 D16	m ²	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

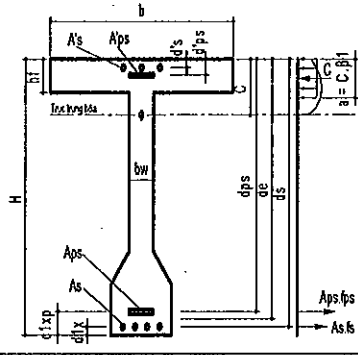
β	Factor indicating diag. cracked concr. to tension		2.0	2.0	2.0	2.0	2.0
θ	Angle of inclination of diagonal compressive	degree	34.06	29.06	29.22	32.50	31.27
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	90
bv	Effective web width as minimum web width - in dv	m	13.409	13.409	13.409	13.409	13.409
dv	Effective shear depth	m	0.442	0.442	1.714	1.714	1.714
	(de - a/2)	m	0.442	0.442	1.714	1.714	1.714
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	m ²	0.0032	0.0032	0.0030	0.0030	0.0030
θ	Assume	degree	34.00	29.00	29.22	32.50	32.00
v	Shear stress in concrete	kN/m ²	186	103	168	270	249
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
ex	Strain in tensile reinforcement		8.39E-04	5.04E-04	5.14E-04	7.19E-04	6.42E-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 f'c		0.006	0.003	0.006	0.009	0.008
β	Final value		2.0	2.0	2.0	2.0	2.0
θ	Final value	degree	34.06	29.06	29.22	32.50	31.27
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	5389	5389	20892	20892	20892
Vs	Shear resistance provided by shear reinforcement	kN	1382	1682	6168	5415	5682
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	6771	7070	27060	26307	26574
Vn2	Vn2	kN	44450	44450	172335	172335	172335
Vn	Nominal shear resistance $Vn = \min(Vn1, Vn2)$	kN	6771	7070	27060	26307	26574
Vr	Factored shear resistance	kN	6094	7070	27060	23677	26574
Vu	Shear	kN	992	612	3872	5591	5721
(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	m ²	0.0091	0.0091	0.0091	0.0091	0.0091
	Minimum shear reinforcement Checking		-	-	-	-	-
	$0.1 * f'c * bv * dv$	kN	17780	17780	68934	68934	68934
	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			
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
fpv	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 SECTION							
	Combination		Service	Strength	Extreme	Extreme	Strength
Qu	Shear	kN	7443	11104	7158	7388	10043
Mu	Flexural Moment	kNm	25079	36898	25755	15604	23185
Nu	Axial load	kN	2118	3115	2360	1592	2095
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	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	0	0	0	0	0
	Diameter	mm	28	28	28	32	32
	Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
A'c	Shear reinforcement	Number	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 resistance	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 resistance	kNm	48232	43409	48232	38990	35091
Mu	Flexural moment	kNm	25079	36898	25755	15604	23185
(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*Mcrr	Cracking moment	kNm	18309	18309	18309	18088	18088
(5.7.3.3.2)	Checking $M_r \geq \min(1.2M_{crr}, 1.33M_u)$		OK	OK	OK	OK	OK
(5.7.3.4)	Control of cracking by distr. of reinf for RC member- Check?		Yes	No	No	No	No
	Existing condition for structure	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	m ²	0.020	0.020	0.020	0.019	0.019
f _{sa}	Value	Mpa	160	160	160	163	163
0.6*f _y		Mpa	240	240	240	240	240
	Tensile stress in reinf Min(f _{sa} , 0.6f _y)	Mpa	160	160	160	163	163
x	Dist. From compression fiber to centroid	m	0.335	-	-	-	-
J.d	Arm	m	1.722	-	-	-	-
I _{cr}	Moment of inertia of the cracked section	m ⁴	1.216	-	-	-	-
f _s	Tensile stress in reinforcement $f_s = M_{sls} / (A_s * J.d)$	Mpa	216	-	-	-	-
(5.10.8.2)	Shrinkage and temperature Reinforcement (side distribution)						
A _{req}	Area of required reinf	m ²	0.00127	0.00127	0.00127	0.00127	0.00127
	Distribution on sides 7 D16	m ²	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	2.1	1.9
θ	Angle of inclination of diagonal compressive	degree	39.03	42.46	39.07	37.80	41.55
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	90
b _v	Effective web width as minimum web width - in d _v	m	12.600	12.600	12.600	12.600	12.600
d _v	Effective shear depth	m	1.792	1.792	1.792	1.884	1.884
	(d _e - 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
n _{cat}	Amount of bars in spacing S	bars	20	20	20	20	20
A _v	Shear reinf area in spacing S	m ²	0.0040	0.0040	0.0040	0.0040	0.0040
θ	Assume	degree	39.00	42.50	39.16	38.00	41.50
v	Shear stress in concrete	kN/m ²	330	546	317	33	470
f _{po}	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e _x	Strain in tensile reinforcement		1.30E-03	1.86E-03	1.31E-03	1.18E-03	1.64E-03
	if e _x < 0, multiple with reduce factor		-	-	-	-	-
	Strain checking	$\leq 2.00E-3$	Ok	Ok	Ok	Ok	Ok
v/f _c	Ratio of shear stress and f _c		0.011	0.018	0.011	0.001	0.016
β	Final value		2.1	1.8	2.1	2.1	1.9
θ	Final value	degree	39.03	42.46	39.07	37.80	41.55
V _c	Nominal shear resistance provided by tensile stresses in the concrete	kN	21151	18296	21128	22973	20364
V _s	Shear resistance provided by shear reinforcement	kN	5955	5275	5946	6540	5725
V _p	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0	0
V _{n1}	V _{n1} = V _c + V _s + V _p	kN	27106	23572	27074	29513	26089
V _{n2}	V _{n2}	kN	169355	169355	169355	178018	178018
V _n	Nominal shear resistance V _n = min(V _{n1} , V _{n2})	kN	27106	23572	27074	29513	26089
V _r	Factored shear resistance	kN	27106	21214	27074	29513	23480
V _u	Shear	kN	7443	11104	7158	7388	10043
(5.8.2.7)	Shear checking		OK	OK	OK	OK	OK

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REINFORCEMENT CHECKING - WING WALL

MATERIALS			
NORMAL CONCRETE			
f_c	Compressive Strength of concrete at 28 days	Mpa	30
E_c	Modulus of Elasticity	Mpa	27691
f_r	Modulus of Rupture	Mpa	3.5
g_c	Unit weight of concrete	kN/m3	24.5
PRESTRESSING STEEL			
f_{pu}	Tensile strength of prestressing steel	Mpa	1860
f_{py}	Yield strength of prestressing steel	Mpa	1670
E_p	Modulus of Elasticity	Mpa	195000
REINFORCEMENT			
f_y	Yield strength	Mpa	400
E_s	Modulus of Elasticity	Mpa	200000
n_c	Ratio E_s/E_c		7

Sign	Parameters	Unit	Sections				
			5-5	5-5	6-6	6-6	6-6
INTERNAL FORCES AT SECTION							
	Combination		Service	Strength	Service	Strength	Strength
Q_u	Shear	kN	168	252	340	474	474
M_u	Flexural Moment	kNm	395	487	471	661	661
N_u	Axial load	kN	0	0	0	0	0
T_u	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
d_{lx}	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
d_s	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
d_{lxp}	Dis. From tens. fiber to centroid of tension prestressing steel	m	0.000	0.000	0.000	0.000	0.000
d_{ps}	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
b_w	Web width or diameter of a circular section	m	1.000	1.000	1.000	1.000	1.000
h_f	Compression flange depth	m	0.000	0.000	0.000	0.000	0.000
I_z	Moment of inertia of section	m4	0.043	0.043	0.043	0.043	0.043
A_{mc}	Section area	m2	0.800	0.800	0.800	0.800	0.800
	Steel choice						
A_{ps}	Tension prestressing steel	P.S type	0	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	0 T00.0
		Number	tendons	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000
A_s	Tension Reinforcement	Number	bars	6	6	6	6
		Diameter	mm	25	25	25	25
		Area	m2	0.00295	0.00295	0.00295	0.00295
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	3	3	3	3
		Diameter	mm	14	14	14	14
		Area	m2	0.00045	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
ϕ_v	Resistance factors for shear		1.00	0.90	1.00	0.90	0.90
ϕ_n	Resistance factors for axial force		1.00	1.00	1.00	1.00	1.00
β_1	Stress block factor		0.836	0.836	0.836	0.836	0.836
c	Dis. Between centroid and top fiber	m	0.033	0.033	0.033	0.033	0.033
	For T section behavior	m	0.033	0.033	0.033	0.033	0.033
	For rectangular section behavior	m	0.033	0.033	0.033	0.033	0.033
f_{pe}	Effective stress in the prestressing steel after losses	Mpa	1116	1116	1116	1116	1116
f_{ps}	Aver. stress in pres. steel at the time for which the nominal resistance	Mpa	1838	1838	1838	1838	1838
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 - WING WALL							
a	Depth of equivalent stress block	m	0.027	0.027	0.027	0.027	0.027
de	Corresp. effective depth from extreme comp. fiber to centroid of tensile force in the tensile reinf.	m	0.741	0.741	0.741	0.741	0.741
Mn	Nominal resistance	kNm	836	836	836	836	836
Mr	Factored resistance	kNm	836	752	836	752	752
Mu	Flexural moment	kNm	395	487	471	661	661
(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	0.04	0.04
	Maximum reinforcement Checking	≤ 0.42	OK	OK	OK	OK	OK
r min	Minimum reinforcement		0.37%	0.37%	0.37%	0.37%	0.37%
	Minimum reinforcement Checking for RC	0.23%	OK	OK	OK	OK	OK
1.2*Mer	Cracking moment	kNm	230	230	230	230	230
(5.7.3.3.2)	Checking $M_r \geq \min(1.2M_{er}, 1.33M_u)$		OK	OK	OK	OK	OK
(5.7.3.4)	Control of cracking by distr. of reinf for RC member- Check?		Yes	No	Yes	No	No
	Existing condition for structure	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	m ²	0.020	0.020	0.020	0.020	0.020
f _{sa}	Value	Mpa	285	285	285	285	285
0.6*f _y		Mpa	240	240	240	240	240
	Tensil stress in reinf Min(f _{sa} , 0.6f _y)	Mpa	240	240	240	240	240
x	Dist. From compression fiber to centroid	m	0.155	-	0.155	-	-
J.d	Arm	m	0.689	-	0.689	-	-
I _{cr}	Moment of inertia of the cracked section	m ⁴	0.008	-	0.008	-	-
f _s	Tensile stress in reinforcement $f_s = M_{sls} / (A_s * J.d)$	Mpa	195	-	232	-	-
	Checking for control cracking $f_s < f_{sa}$		OK	N.a	OK	N.a	N.a
(5.10.8.2)	Shrinkage and temperature Reinforcement (side distribution)						
A _{req}	Area of required reinf	m ²	0.00042	0.00042	0.00042	0.00042	0.00042
	Distribution on sides 7 D16	m ²	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	2.0	1.7	1.7
θ	Angle of inclination of diagonal compressive	degree	37.09	39.92	40.38	42.90	42.90
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	90
b _v	Effective web width as minimum web width - in dv	m	1.000	1.000	1.000	1.000	1.000
d _v	Effective shear depth	m	0.727	0.727	0.727	0.727	0.727
	(d _e - a/2)	m	0.727	0.727	0.727	0.727	0.727
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	0.600
n _{eat}	Amount of bars in spacing S	bars	3	3	3	3	3
A _v	Shear reinf area in spacing S	m ²	0.0005	0.0005	0.0005	0.0005	0.0005
β	Assume		2.0	2.0	2.0	2.0	2.0
θ	Assume	degree	37.20	39.90	40.40	42.90	42.90
v	Shear stress in concrete	kN/m ²	231	385	467	724	724
f _{po}	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e _x	Strain in tensile reinforcement		1.11E-03	1.39E-03	1.44E-03	1.97E-03	1.97E-03
	if e _x < 0, multiple with reduce factor		-	-	-	-	-
	Strain checking	$\leq 2.00E-3$	Ok	Ok	Ok	Ok	Ok
v/f _c	Ratio of shear stress and f _c		0.008	0.013	0.016	0.024	0.024
β	Final value		2.2	2.0	2.0	1.7	1.7
θ	Final value	degree	37.09	39.92	40.38	42.90	42.90
V _c	Nominal shear resistance provided by tensile stresses in the concrete	kN	717	665	656	573	573
V _s	Shear resistance provided by shear reinforcement	kN	291	263	258	236	236
V _p	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0	0
V _{n1}	V _{n1} = V _c + V _s + V _p	kN	1008	927	915	809	809
V _{n2}	V _{n2}	kN	5456	5456	5456	5456	5456
V _n	Nominal shear resistance V _n = min(V _{n1} , V _{n2})	kN	1008	927	915	809	809
V _r	Factored shear resistance	kN	1008	835	915	728	728
V _u	Shear	kN	168	252	340	474	474
(5.8.2.7)	Shear checking		OK	OK	OK	OK	OK

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		Design			
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I. BORED PILE DESIGN

I. BORED PILE DATA

1. Load Combinations at top of bored pile

No	Combinations	Sign	F _v (kN)	Longitudinal		Transvesal	
				F _{Hx} (kN)	My (kN•m)	F _{Hy} (kN)	Mx (kN•m)
1	Strength Str-IA		5487	1059	-3165	-11	-59
2	Service Ser-I		3839	721	-2161	3	-8
3	Extreme Ext-IA		3714	813	-2494	88	255
4							
5							
6							

2. Bored pile Material

Normal concrete		
Compressive strength at 28 days age	f _c	30 MPa
Concrete elastic modulus	E _c	27691 MPa
Reinforcement TCVN1651-2008; CBV-400		
Yield strength	f _y	400 MPa
Reinforcement elastic modulus	E _s	200,000 MPa

3. Bored pile Section

Pile diameter	D	1.20 m
Section area	A	1.131 m ²
Moment inertia	I _x	0.102 m ⁴
	I _y	0.102 m ⁴
Radius of gyration of gross concrete section; r = sqrt(I/A)	r _x	0.300 m
	r _y	0.300 m

II. PILE DESIGN

1. Limit of Reinforcement

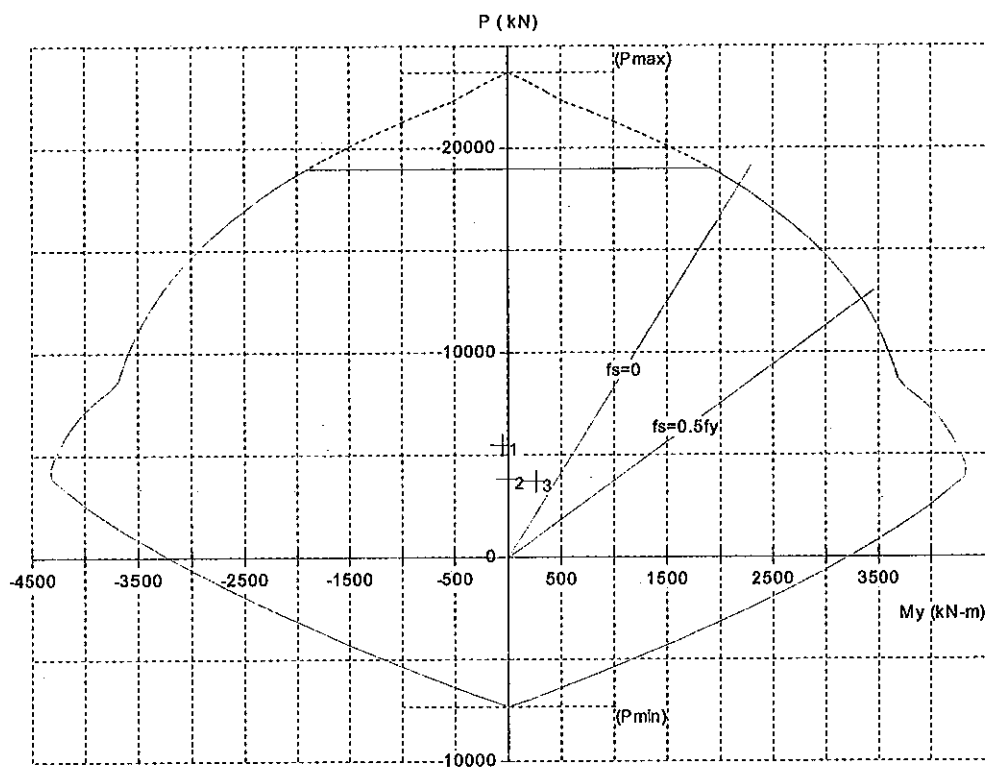
S.5.7.4.2

Minimum area of longitudinal reinforcement in column		
As.fy / (Ag . f _c) >= 0.135	As ≥	0.011 m ²
As / Ag >= 0.01	As ≥	0.011 m ²
Maximum area of longitudinal reinforcement in column		
As / Ag <= 0.08	As ≤	0.090 m ²
Trial Rebars:	Ok As	0.019 m ²
1layers x 24 = 24 bars D32 @150 As1		0.019 m ²

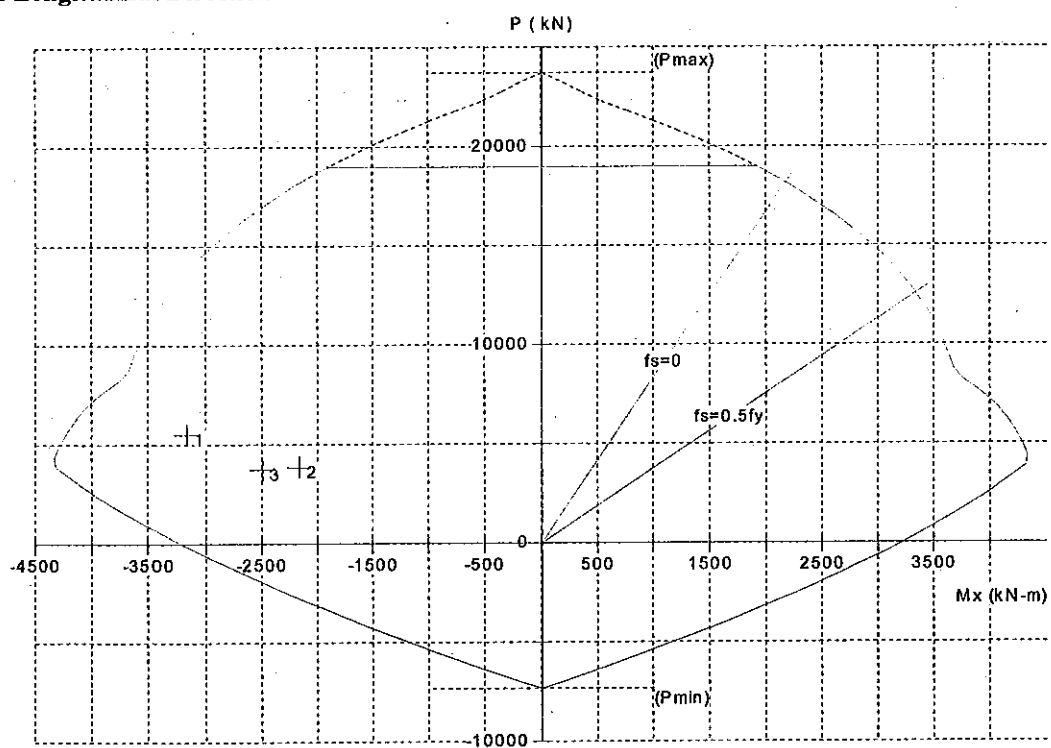
2. Interaction diagram M-P

Using Pca-Column software

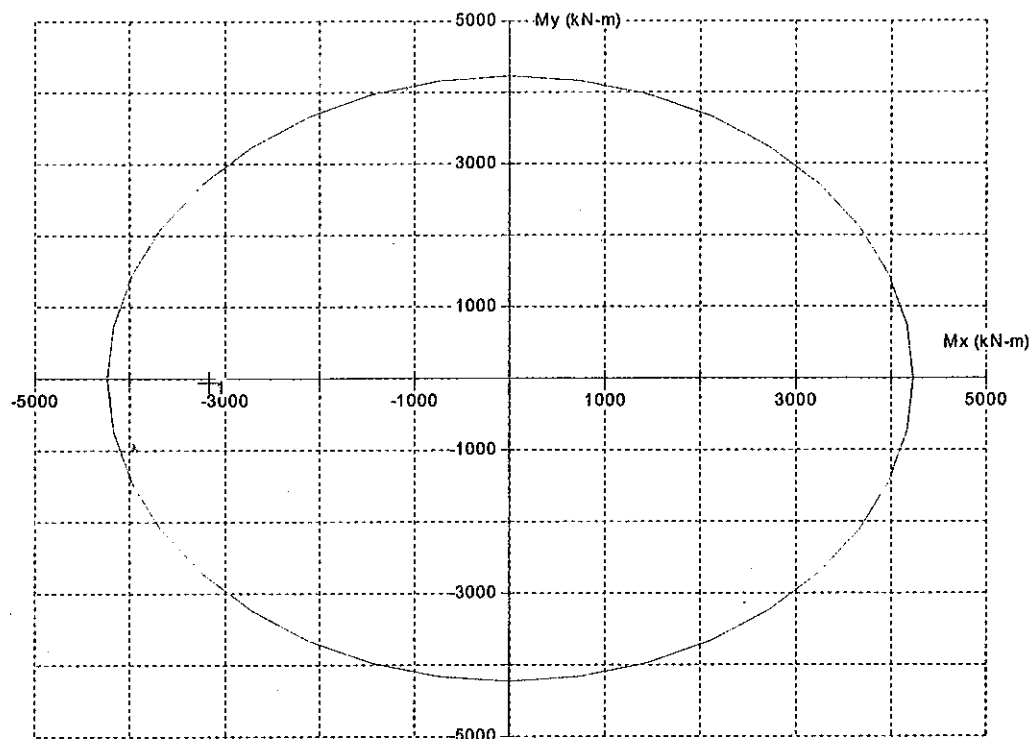
**In Transverse Direction



**In Longitudinal Direction



****In Both Direction**



3. Column ties $P = 5487 \text{ kN}$

S.5.7.4.6, S.5.10.6.3, S.5.10.11.4.1d - e

Bridge is in seismic zone	Sz	1	
Area of concrete core measured out-to-out of ties	Ac	0.933	m ²
Tie diameter	Dtie	14	mm
Cross section area of 1 tie	As-tr	0.00015	m ²
Spacing of hoops	s	75	mm
Length of reinforcement tie in 1 hoop	Ltie	3.41	m
Ratio of ties reinf. in one hoop/ volume of conc. core one pitch spacing			
$\rho_s = A_{s-tr} \cdot L_{tie} / (A_c \cdot \text{spacing})$	ρ_s	0.0074	
Ratio of spiral reinf. To total volume of concrete core shall satisfy			S.5.7.4.6
$\rho_s \geq 0.45 \cdot (A_g/A_c - 1) \cdot f_c / f_y = \text{Req1}$	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	
$\rho_s \geq 0.12 \cdot f_c / f_y = \text{Req2}$	Req2	0.0090	N/A
Length distributed spiral with pitch 75mm below pilecap	Ldis	1.80	m

4. Shear Design

Shear resistance factors	ϕ_v	1.0	
Factored shear force	Vu	1059	kN
Required shear capacity $V_n = V_u / \phi_v$	Vn	1059	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
$d_v = 0.9 \cdot d_e$ $d_e = D/2 + D_r/\pi()$			
Diameter of the circle passing through the centers of the long. reinf.	Dr	0.99	m
	de	0.92	m
	dv	0.82	m
$V_c = 0.083 \cdot \beta \cdot \sqrt{f_c} \cdot b_v \cdot d_v$	Vc	900	kN

Difference between required shear capacity and the capacity provided by concrete is the minimum required capacity for shear reinforcements			
$V_s = V_n - V_c$	V_s	160	kN
In this case $V_c > V_n$ so shear reinforcement is no need			
Stirrup diameter	D_s	14	
Number of stirrup legs / cross section	n_s	2	
Shear legs area	A_v	0.0003	m ²
Angle of inclination of shear reinf. to long. axis	α	90	deg
$V_s = A_v \cdot f_y \cdot d_v \cdot (\cot \theta + \cot \alpha) \cdot \sin \alpha / s$	$s \leq$	0.00	m
Stirrup spacing used	s	0.10	m
Check minimum shear reinforcement requirement		OK	
$A_v \geq 0.083 \cdot \sqrt{f_c} \cdot b_v \cdot s / f_y = \text{Req.}$	Req	0.0000	m ²
Check maximum shear reinforcement spacing requirement		OK	
$F = 0.1 \cdot f_c \cdot b_v \cdot d_v$	F	3609	kN
If $V_u < 0.1 \cdot f_c \cdot b_v \cdot d_v$ then $S_{\max} = 0.8 \cdot d_v \leq 600\text{mm}$			
If $V_u > 0.1 \cdot f_c \cdot b_v \cdot d_v$ then $S_{\max} = 0.4 \cdot d_v \leq 300\text{mm}$	S_{\max}	0.60	m

SPACE PILE FOUNDATION ANALYSIS PROGRM
Turbo BASIC

PROJECT: : CB23-A2L

INITIA DATA

Kn = 0.00 Ax = 9.58 By = 13.41 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 = 400 (t/m4)

LOAD COMBINATIONS

COMB.	Hx	Hy	P	Mx	My	Mz
1	850.00	0.00	3129.00	-507.00	833.00	0.00
2	563.00	0.00	2159.00	-484.00	595.00	0.00
3	824.00	11.00	3050.00	-502.00	712.00	0.00
4	536.00	11.00	2079.00	-479.00	474.00	0.00
5	578.00	9.00	2270.00	-385.00	474.00	0.00
6	645.00	60.00	2880.00	-564.00	-601.00	0.00
7	645.00	60.00	1910.00	-541.00	375.00	0.00

PROPERTIES OF PILES

PILE	Lo	H	Bpx	Bpy	A	B	Cday	Fo	Io	Po	Co	Ct
1	0.00	15.00	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	4.23	0.000	0.00
2	3.30	-1.15	0.000	0.00
3	3.30	-6.52	0.000	0.00
4	0.00	5.43	0.000	0.00
5	0.00	0.05	0.000	0.00
6	0.00	-5.32	0.000	0.00
7	-3.30	5.56	0.000	0.00
8	-3.30	-3.16	0.000	0.00

DISPLACEMENTS

COMB.	X	Y	Z	Fix	Fiy	Fiz
1	0.01335	0.00017	0.001813	-0.000050	0.000207	0.000047
2	0.00885	0.00013	0.001253	-0.000038	0.000140	0.000031
3	0.01291	0.00033	0.001771	-0.000048	0.000192	0.000048
4	0.00840	0.00029	0.001211	-0.000037	0.000124	0.000032
5	0.00904	0.00026	0.001324	-0.000035	0.000128	0.000034
6	0.00974	0.00103	0.001734	-0.000035	0.000030	0.000048
7	0.01012	0.00106	0.001097	-0.000046	0.000155	0.000048

FORCES ON PILES

PILE	COMB.	N	Q2	Q3	M1	M2	M3
------	-------	---	----	----	----	----	----

1	1	559.34	-107.96	1.14	1.216	5.969	322.651
	2	387.18	-71.51	0.75	0.806	4.201	213.591
	3	539.33	-104.74	-0.22	1.238	1.732	313.478
	4	366.86	-68.16	-0.60	0.826	-0.042	204.038
	5	391.29	-73.48	-0.31	0.875	0.777	220.254
	6	409.37	-82.37	-6.34	1.244	-17.792	252.555
	7	372.11	-82.37	-6.34	1.244	-17.263	246.369
2	1	504.25	-105.84	1.14	1.216	5.969	316.122
	2	345.15	-70.10	0.75	0.806	4.201	209.266
	3	485.53	-102.58	-0.22	1.238	1.732	306.832
	4	326.17	-66.72	-0.60	0.826	-0.042	199.605
	5	352.73	-71.96	-0.31	0.875	0.777	215.555
	6	370.18	-80.21	-6.34	1.244	-17.792	245.878
	7	321.07	-80.21	-6.34	1.244	-17.263	239.692
3	1	449.15	-103.72	1.14	1.216	5.969	309.592
	2	303.13	-68.70	0.75	0.806	4.201	204.941
	3	431.74	-100.43	-0.22	1.238	1.732	300.187
	4	285.49	-65.28	-0.60	0.826	-0.042	195.171
	5	314.17	-70.43	-0.31	0.875	0.777	210.857
	6	330.99	-78.04	-6.34	1.244	-17.792	239.200
	7	270.03	-78.04	-6.34	1.244	-17.263	233.015
4	1	430.22	-108.43	-0.16	1.216	1.959	324.110
	2	301.27	-71.82	-0.11	0.806	1.545	214.557
	3	420.30	-105.22	-1.54	1.238	-2.348	314.963
	4	291.21	-68.48	-1.49	0.826	-2.764	205.029
	5	312.56	-73.82	-1.24	0.875	-2.108	221.304
	6	397.85	-82.86	-7.67	1.244	-21.892	254.047
	7	278.17	-82.86	-7.67	1.244	-21.363	247.862
5	1	375.12	-106.31	-0.16	1.216	1.959	317.581
	2	259.24	-70.42	-0.11	0.806	1.545	210.233
	3	366.50	-103.07	-1.54	1.238	-2.348	308.317
	4	250.52	-67.04	-1.49	0.826	-2.764	200.595
	5	274.00	-72.30	-1.24	0.875	-2.108	216.605
	6	358.66	-80.69	-7.67	1.244	-21.892	247.370
	7	227.13	-80.69	-7.67	1.244	-21.363	241.184
6	1	320.02	-104.20	-0.16	1.216	1.959	311.052
	2	217.21	-69.02	-0.11	0.806	1.545	205.908
	3	312.71	-100.91	-1.54	1.238	-2.348	301.672
	4	209.83	-65.61	-1.49	0.826	-2.764	196.162
	5	235.44	-70.77	-1.24	0.875	-2.108	211.907
	6	319.47	-78.53	-7.67	1.244	-21.892	240.693
	7	176.09	-78.53	-7.67	1.244	-21.363	234.507
7	1	290.18	-108.49	-1.46	1.216	-2.050	324.277
	2	207.03	-71.86	-0.97	0.806	-1.111	214.668
	3	290.62	-105.28	-2.86	1.238	-6.429	315.132
	4	207.49	-68.52	-2.37	0.826	-5.486	205.142
	5	226.20	-73.86	-2.18	0.875	-4.993	221.424
	6	378.56	-82.91	-9.00	1.244	-25.992	254.217
	7	174.13	-82.91	-9.00	1.244	-25.463	248.032
8	1	200.72	-105.05	-1.46	1.216	-2.050	313.675
	2	138.79	-69.58	-0.97	0.806	-1.111	207.645
	3	203.27	-101.78	-2.86	1.238	-6.429	304.342
	4	141.43	-66.18	-2.37	0.826	-5.486	197.943
	5	163.59	-71.38	-2.18	0.875	-4.993	213.795
	6	314.92	-79.40	-9.00	1.244	-25.992	243.375
	7	91.25	-79.40	-9.00	1.244	-25.463	237.190

SUMMARY OF FORCES

	PILE	COMB.	N	Q2	Q3	M1	M2	M3
Nmin	8	7	91.25	-79.40	-9.00	1.244	-25.463	237.190
Nmax	1	1	559.34	-107.96	1.14	1.216	5.969	322.651
Q2max	7	1	290.18	-108.49	-1.46	1.216	-2.050	324.277
Q3max	7	6	378.56	-82.91	-9.00	1.244	-25.992	254.217
M1max	1	6	409.37	-82.37	-6.34	1.244	-17.792	252.555
M2max	7	6	378.56	-82.91	-9.00	1.244	-25.992	254.217
M3max	7	1	290.18	-108.49	-1.46	1.216	-2.050	324.277

CHECKING CALCULATI
IN COMPARISON WITH INITIA LOAD MATRIX

1	850.00	0.00	3129.00	-507.00	833.00	0.00
2	563.00	0.00	2159.00	-484.00	595.00	0.00

			CB23-A2L.OUT			
3	824.00	11.00	3050.00	-502.00	712.00	0.00
4	536.00	11.00	2079.00	-479.00	474.00	0.00
5	578.00	9.00	2270.00	-385.00	474.00	0.00
6	645.00	60.00	2880.00	-564.00	-601.00	0.00
7	645.00	60.00	1910.00	-541.00	375.00	0.00

CALCULATION SHEET

BORED PILE CAPACITY

	DANANG QUANG NGAI EXPRESSWAY	Item.	Eng.	Date.	Sign.
	CB23 BRIDGE	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	CB23-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 = 7.66	Modulus of elasticity of concret	$E_c = 27691$ MPa
Pile tip elevation	EL3 = 3.00		
Pile Length	$L = 6.00$ m	Depth of socket	$H_s = 4.66$ m
Diameter of drilled-shaft	$D_p = 1.20$ m	Diameter of socket	$D_s = 1.20$ m
Pile Cross-Sectional Perimeter	$P = 3.77$ m	Socket Cross-Sect. Perimeter	$P_{soc} = 3.77$ m
Pile Cross-Sectional Area	$A_b = 1.13$ m ²	Socket Cross-Sectional Area	$A_{soc} = 1.13$ m ²
Working normal force at pile head	$N = 5653.5$ kN		
Working normal force at top of socket	$P_i = 5576.0$ kN		
Intack rock modulus	$E_i = 25000$ MPa		Figure C10.8.3.5-2 Lrfd
Modulus modification ratio	$K_c = 0.05$		Figure C10.8.3.5-3 Lrfd
Elastic modulus of the insitu rock	$E_r = K_c * E_i = 1250.0$ MPa		
Influence coefficient	$I_p = f(H_s/D_s, E_c/E_r) = 0.30$		Figure C10.8.3.5-1 Lrfd
	$H_s/D_s = 3.88$		
	$E_c/E_r = 22.15$		
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

$$r_e = (\sum P_i) * H_s / (A_{soc} * E_c) = 0.830 \text{ mm}$$

The settlement of base of drilled shaft

$$r_{base} = (\sum P_i) * I_p / (D_s * E_r) = 1.115 \text{ mm}$$

$$r_e + r_{base} = 1.945 \text{ mm} < 10 \text{ mm}$$

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 $q_u < 1.9$ Mpa - may be taken after Carter & Kulhawy 1988 $\rightarrow q_s = 0.15 * q_u$

C10.8.3.5-4

if $q_u > 1.9$ Mpa - may be taken after Horvath & Kenney 1979 $\rightarrow q_s = 0.21 * \sqrt{q_u}$

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 $\rightarrow q_s = 0.65 * \alpha_E * p_a * (q_u/p_a)^{0.5} < 7.8 * p_a * (f_c/p_a)^{0.5}$

$$q_{s0} = 7.8 * p_a * (f_c/p_a)^{0.5}$$

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_n 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	7.66	7.00	0.66	55	42.41	1.37	3403	0.65	2212
2	7.00	6.00	1.00	64	42.41	1.37	5156	0.65	3351
3	6.00	5.00	1.00	73	42.41	1.37	5156	0.65	3351
4	5.00	4.00	1.00	71	53.63	1.54	5798	0.65	3769
5	4.00	3.00	1.00	53	53.63	1.54	5798	0.65	3769
6									
7									
8									
Sum			4.66				25309		16451

	DANANG QUANG NGAI EXPRESSWAY	Item.	Eng.	Date.	Sign.
	CB23 BRIDGE	Design			
	DETAIL DESIGN	Check			
	EMPIRICAL ESTIMATION OF PILE CAPACITY	Revise			

Case2												Type: "1: closed joints", "2: open joints"
No.	Depth (m)	RQD (%)	q_u (MPa)	E_m / E_i	α_E	Type	q_{s0} (MPa)	q_s (MPa)	$q_s - used$ (MPa)	Q_{SR} (kN)	ϕ_s	Q_R (kN)
1	0.66	54.75	42.41	0.28	0.69	1	13.58	0.92	0.92	2294	0.55	1262
2	1.00	63.75	42.41	0.53	0.81	1	13.58	1.09	1.09	4114	0.55	2263
3	1.00	72.75	42.41	0.73	0.89	1	13.58	1.20	1.20	4519	0.55	2485
4	1.00	71.25	53.63	0.71	0.89	1	13.58	1.34	1.34	5047	0.55	2776
5	1.00	52.50	53.63	0.22	0.64	1	13.58	0.97	0.97	3645	0.55	2005
6	-	-	-	-	-	-	-	-	-	-	-	-
7	-	-	-	-	-	-	-	-	-	-	-	-
8	-	-	-	-	-	-	-	-	-	-	-	-
Sum	4.66									19619		10791

Unit base resistance

$$q_p = K_b \cdot (p_1 - p_0) + \sigma_v$$

Limit pressure determined from presuremeter tests

$$p_1 = 5.89 \text{ MPa}$$

C10.8.3.5-7

At rest total horizontal stress measured at ther base elevation

$$p_0 = - \text{ MPa}$$

Coefficient that depen on diameter socket

$$K_b = 4.51$$

Table C10.8.3.5-1

Total vertical stress at the base elevation

$$\sigma_v = - \text{ MPa}$$

$$q_p = - \text{ MPa}$$

$$\phi = 0.50$$

Table 10.5.5-3

$$Q_{pR} = A_p \cdot q_p$$

$$Q_{pR} = - \text{ kN}$$

$$Q_R = \phi \cdot Q_{pR}$$

$$Q_R = - \text{ kN}$$

ESTIMATED PILE CAPACITY:

Pile Structural Capacity	$Q_T = 0.75 \cdot 0.85 \cdot 0.85 \cdot f_c \cdot A_g$	18385 kN	1874 T
Pile resistance	Q_R	10791 kN	1100 T
Deducting pile weight		-152 kN	-15 T
Estimated Pile Capacity		10639 kN	1085 T
Maximum Reaction - ULS	Ok	5487 kN	559 T

	DANANG QUANG NGAI EXPRESSWAY	Item.	Eng.	Date.	Sign.
	CB23 BRIDGE	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	CB23-A2	Pile Concrete comp. strength	$f'_c = 30.0$ MPa
Bottom of pilecap elavation	EL1 = 8.50	Concrete Unit Weight	$g_c = 24.5$ kN/m ³
Top of socket elevation	EL2 = -2.58	Modulus of elasticity of concret	$E_c = 27691$ MPa
Pile tip elevation	EL3 = -7.00		
Pile Length	$L = 15.50$ m	Depth of socket	$H_s = 4.42$ m
Diameter of drilled-shaft	$D_p = 1.20$ m	Diameter of socket	$D_s = 1.20$ m
Pile Cross-Sectional Perimeter	$P = 3.77$ m	Socket Cross-Sect. Perimeter	$P_{soc} = 3.77$ m
Pile Cross-Sectional Area	$A_b = 1.13$ m ²	Socket Cross-Sectional Area	$A_{soc} = 1.13$ m ²
Working normal force at pile head	$N = 6227.6$ kN		
Working normal force at top of socket	$P_i = 6154.1$ kN		
Intack rock modulus	$E_i = 25000$ MPa		Figure C10.8.3.5-2 Lrfd
Modulus modification ratio	$K_c = 0.05$		Figure C10.8.3.5-3 Lrfd
Elastic modulus of the insitu rock	$E_r = K_c * E_i = 1250.0$ MPa		
Influence coefficient	$I_p = f(H_s/D_s, E_c/E_r) = 0.30$		Figure C10.8.3.5-1 Lrfd
	$H_s/D_s = 3.68$		
	$E_c/E_r = 22.15$		
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

$$r_e = (\sum P_i) * H_s / (A_{soc} * E_c) = 0.869 \text{ mm}$$

The settlement of base of drilled shaft

$$r_{base} = (\sum P_i) * I_p / (D_s * E_r) = 1.231 \text{ mm}$$

$$r_e + r_{base} = 2.099 \text{ mm} < 10 \text{ mm}$$

Compute the bearing capacity based on Sharftt resistance alone

Unit side resistance

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

if $q_u < 1.9$ Mpa - may be taken after Carter & Kulhawy 1988 $\rightarrow q_s = 0.15 * q_u$

C10.8.3.5-4

if $q_u > 1.9$ Mpa - may be taken after Horvath & Kenney 1979 $\rightarrow q_s = 0.21 * \sqrt{q_u}$

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 $\rightarrow q_s = 0.65 * \alpha_E * p_a * (q_u/p_a)^{0.5} < 7.8 * p_a * (f'_c/p_a)^{0.5}$

$$q_{s0} = 7.8 * p_a * (f'_c/p_a)^{0.5}$$

10.8.3.5.4d-1-Lrfd2007

Drilling method used in construction:

Case 2

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

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	-2.58	-4.08	1.50	27	49.58	1.48	8362	0.65	5435
2	-4.08	-5.58	1.50	27	53.69	1.54	8701	0.65	5656
3	-5.50	-7.00	1.50	40	53.69	1.54	8701	0.65	5656
4									
5									
6									
7									
8									
Sum			4.50				25764		16747

	DANANG QUANG NGAI EXPRESSWAY	Item.	Eng.	Date.	Sign.
	CB23 BRIDGE	Design			
	DETAIL DESIGN	Check			
	EMPIRICAL ESTIMATION OF PILE CAPACITY	Revise			

Case2 Type: "1: closed joints", "2: open joints"												
No.	Depth (m)	RQD (%)	q _u (MPa)	E _m / E _i	α _E	Type	q _{s0} (MPa)	q _s (MPa)	q _s - used (MPa)	Q _{SR} (kN)	φ _s	Q _R (kN)
1	1.50	27.00	49.58	0.07	0.50	1	13.58	0.72	0.72	4085	0.55	2247
2	1.50	27.00	53.69	0.07	0.50	1	13.58	0.75	0.75	4251	0.55	2338
3	1.50	40.00	53.69	0.12	0.56	1	13.58	0.85	0.85	4815	0.55	2648
4	-	-	-	-	-	-	-	-	-	-	-	-
5	-	-	-	-	-	-	-	-	-	-	-	-
6	-	-	-	-	-	-	-	-	-	-	-	-
7	-	-	-	-	-	-	-	-	-	-	-	-
8	-	-	-	-	-	-	-	-	-	-	-	-
Sum	4.50									13151		7233

Unit base resistance

$$q_p = K_b \cdot (p_1 - p_0) + \sigma_v$$

Limit pressure determined from presuremeter tests

$$p_1 = 5.89 \text{ MPa}$$

C10.8.3.5-7

At rest total horizontal stress measured at ther base elevation

$$p_0 = - \text{ MPa}$$

Coefficient that depen on diameter socket

$$K_b = 4.44$$

Table C10.8.3.5-1

Total vertical stress at the base elevation

$$\sigma_v = - \text{ MPa}$$

$$q_p = - \text{ MPa}$$

$$\phi = 0.50$$

Table 10.5.5-3

$$Q_{pR} = A_p \cdot q_p$$

$$Q_{pR} = - \text{ kN}$$

$$Q_R = \phi \cdot Q_{pR}$$

$$Q_R = - \text{ kN}$$

ESTIMATED PILE CAPACITY:

Pile Structural Capacity	$Q_T = 0.75 \cdot 0.85 \cdot 0.85 \cdot f_c \cdot A_g$	18385 kN	1874 T
Pile resistance	Q_R	7233 kN	737 T
Deducting pile weight		-307 kN	-31 T
Estimated Pile Capacity		6926 kN	706 T
Maximum Reaction - ULS	Ok	5798 kN	591.00 T

2 ORB22

Table of content - ORB22 Bridge

A. Substructure design

1. Abutment A1
2. Pier P1
3. Bored pile capacity

B. Miscellaneous

1. Expansion joint

A. SUBSTRUCTURE DESIGN

1. Abutment A1

Table of content

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

Da Nang Quang Ngai Expressway project

BRIDGE
ORB22

CALCULATION SHEETS
ABUTMENT A2

LOAD COMPONENTS

Assumptions :

1. Bridge is considered to be in seismic with acceleration coeff. $A = 0.0580 \text{ g}$
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

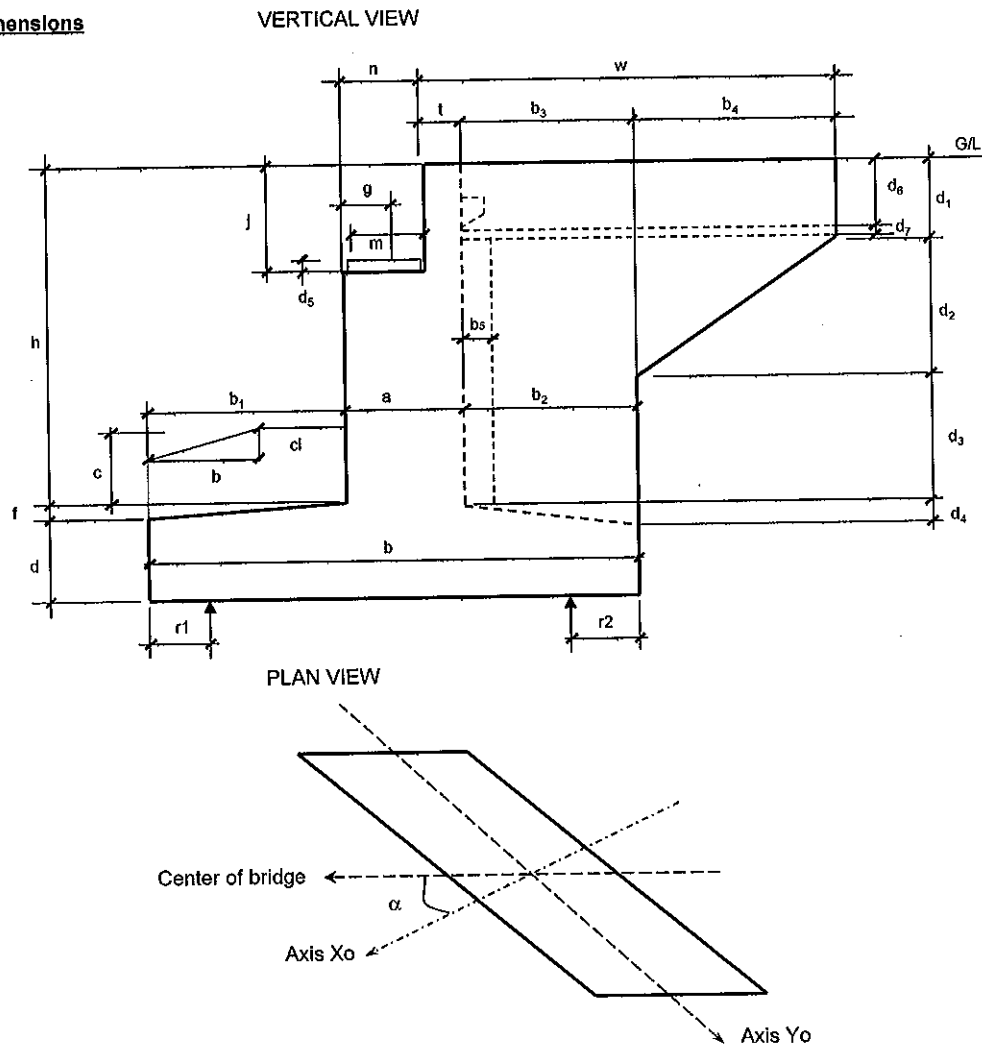
Input :

Level Table(at center of abutment)

Level of top of headwall	HTWL	13.568	m
Level of top of bearing	BTL	11.929	m
Level of top of stem abutment	HTL	11.779	m
Level of top of footing	FTL	5.500	m
Level of bottom of footing	FBL	3.500	m
Ground level	GL	6.250	m
Lowest water level	HWL	6.970	m
Skew angle	α	0.00	deg

I. Loads from substructure

Abutment dimensions



Material Unit Weights

- Unit Weight of Reinf. concrete
- Unit Weight of Soil
- Unit Bouyancy Weight of Soil

$$\begin{aligned}
 \gamma_c &= 24.5 \text{ kN/m}^3 \\
 \gamma_s &= 18.0 \text{ kN/m}^3 \\
 \gamma_{sbo} &= 8.2 \text{ kN/m}^3
 \end{aligned}$$

ABUTMENT DIMENSIONS (IN METRES)					
Item	Symbol	Value	Item	Symbol	Value
Height of stem	h	8.068	Horizontal Dimension	b ₃	3.500
Footing Width	b	7.000	Horizontal Dimension	b ₄	2.100
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	2.100
Width of stem at bearing	n	1.000	Vertical Dimension	d ₃	3.968
Ballast Wall Height	j	1.789	Vertical Dimension	d ₄	0.000
Ballast Wall Thickness	t	0.500	Vertical Dimension	d ₅	0.150
Wingwall Length	w	6.100	Vertical Dimension	d ₆	1.200
Soil Cover at Toe	c	0.750	Vertical Dimension	d ₇	0.300
Girder Reaction	g	0.600	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	3.500	Distance to cl of pile	r2	1.000

Slope front of abutment

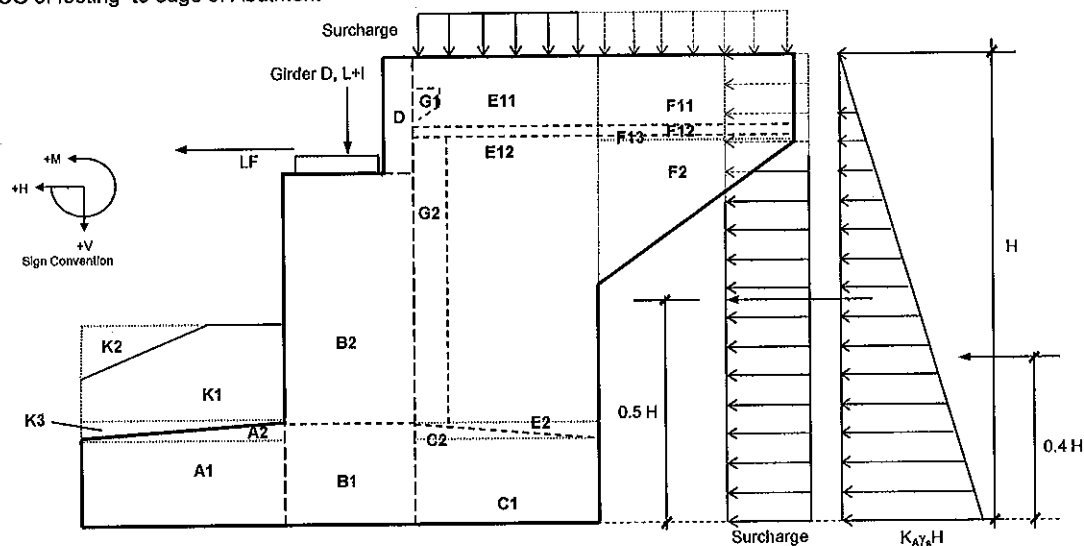
Width of Abutment

Width of abutment (inclined direction)

Height of Abutment

Distance from CG of footing to edge of Abutment

cos (α)	=	1.00
cl	=	0.00 m
bl	=	0.00 m
L	=	12.600 m
Ltr	=	12.600 m
Ht	=	10.07 m
b/2	=	3.50 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	4.000	12.600	1235	1.000	2.500	3087
Section A2	-	12.600	-	1.333	2.167	-
Section B1	3.000	12.600	926	2.750	0.750	695
Section B2	9.419	12.600	2907	2.750	0.750	2181
Section C1	7.000	12.600	2161	5.250	-1.750	-3782
Section C2	-	12.600	-	4.667	-1.167	-
Section D	0.895	12.600	276	3.250	0.250	69
Section E11	5.950	0.500	73	5.250	-1.750	-128
Section E12	21.238	0.500	260	5.250	-1.750	-455
Part extra stem	5.034	0.740	91	6.083	-2.583	-236
Section F11	2.520	0.500	31	8.050	-4.550	-140
Section F12	0.840	0.500	10	6.300	-2.800	-29
Section F13	1.050	0.500	13	8.050	-4.550	-59
Section F2	2.205	0.500	27	7.700	-4.200	-113
Section G1	0.135	12.100	283	3.650	-0.150	-42
Section G2	0.125	13.136	40	3.750	-0.250	-10
Bearing seats (w1seat= 0.65m)	0.083	3.250	10	2.600	0.900	9
Curbs +Handrail on Abutment	0.50	6.100	81	6.050	-2.550	-206
Total SW of Abutment (DC)			8425			840
Transverser moment			495		6.175	3057

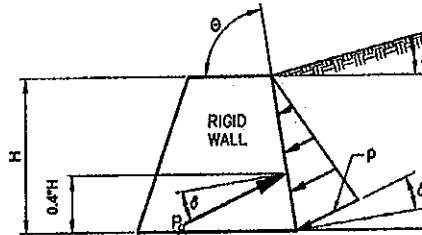
Notes: 1. Distance 'X' is measured horizontally from Toe of Retaining to CG of Section
2. Moment 'Arm' is measured from CG horizontally and from Underside of Footing Vertically.

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	28.24	12.100	6150	5.250	-1.750	-10763
Section E2	-	12.100	-	5.833	-2.333	-
Section E3	-	0.500	-	7.000	-3.500	-
Section K1	1.500	12.600	340	1.000	2.500	-
Section K2	-	12.600	-	-	3.500	-
Section K3	-	12.600	-	0.667	2.833	-
Total Earth on Footing			6490			-10763

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
- Width for horizontal earth pressure
- Density of Soil
- Internal Friction Angle of Soil
- Incline angle of back face wall
- Friction angle between fill and wall
- Incline angle of fill soil
- Gravitational acceleration
- Basic earth pressure

H	=	10.07 m
W	=	12.60 m
γ_s	=	1835 kg/m ³
ϕ'_f	=	30.0 deg
θ	=	90.0 deg
δ	=	0.0 deg
β	=	0.0 deg
g	=	9.81 m/s ²

$$p = K \cdot \gamma_s \cdot g \cdot Z \cdot 10^{-9} \text{ (Mpa, Z:mm)}$$

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

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

Γ	=	2.250
K_a	=	0.333
p	=	0.060 Mpa

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

Horizontal earth pressure:

- $E_a = 0.5 \cdot p \cdot Z \cdot B \cdot 10^3$ (kN)
- $M = E_a \cdot 0.4H$
- Horizontal Earth Pressure act at a height of 0.4 H

E_a	=	3832 kN
M	=	15431 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=	10.07m	heq=	0.61 m

(Linear interpolation)

- Vertical force

ESv	=	484 kN
ev	=	-1.75 m
M	=	-847 kNm

- Horizontal force

ESh	=	464 kN
eh	=	5.03 m
M	=	2337 kNm

$$\Delta p = k \gamma_s g h_{eq} \sigma^0$$

5. Earthquake effects

Bridge is located at: Phu Ninh district - Quang Nam province

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

• Peak ground acceleration coefficient

$$A = 0.0580 \text{ g}$$

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

- Backfill slope angle
- Slope of wall to vertical
- Angle of friction of soil
- Angle of friction between soil and abutment
- Horizontal acceleration coefficient
- Vertical acceleration coefficient
- Angle $\theta = \arctan(k_h / (1 - k_v))$

$$\begin{aligned} i &= 0.0 \text{ deg} \\ \beta' &= 0.0 \text{ deg} \\ \phi &= 30.0 \text{ deg} \\ \delta &= 0.0 \text{ deg} \\ k_h &= 0.087 \\ k_v &= 0.035 \\ \theta &= 5.2 \text{ deg} \end{aligned}$$

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

• Seismic active lateral Earth pressure coefficient

$$K_{AE} = 0.39$$

• $E_{AE} = 0.5 \cdot g \cdot \gamma \cdot H^2 \cdot (1 - k_v) \cdot K_{AE} \cdot 10^{-9} \text{ (kN/m)}$

• Seismic active lateral Earth pressure coefficient

$$E_{AE} = 4325 \text{ kN}$$

$$M_{AE} = E_{AS} \cdot 0.3H + (E_{AE} - E_{AS}) \cdot 0.6H$$

$$M_{AE} = 14551 \text{ KNm}$$

<A.11.1.1.1>

E_{AS} is the static component of seismic active pressure calculated with $\theta = k_v = 0$

5.2. Earthquake effects to abutment (EQ)

Seismic force for substructures: elements above ground $F_h = C_{sm} \cdot W$; elements under ground $F_h = A \cdot S \cdot W$

- Soil profile type
- Site Coefficients.
- Elastic Seismic Response Coefficient
- $C_{sm} = 1.2 \cdot A \cdot S / T_m^{2/3} \leq 2.5 \cdot A$
- Period of vibration of the fundamental mode
- $T_m = 2 \cdot \pi \cdot \sqrt{m/k}$

$$\begin{aligned} \text{Soil type } I & \\ S &= 1.0 \\ 2.5A &= 0.145 \\ C_{sm} &= 0.066 \\ T_m &= 1.074 \text{ s} \end{aligned}$$

Description	Area (m ²)	Length (m)	Force (kN)	$\chi^{(1)}$ (m)	Arm ⁽²⁾ (m)	Moment (kN·m)
Section A1	4.000	12.600	72	-	1.000	72
Section A2	-	12.600	-	-	2.000	-
Section B1	3.000	12.600	54	-	1.000	54
Section B2	9.419	12.600	193	-	5.140	992
Section C1	7.000	12.600	125	-	1.000	125
Section C2	-	12.600	-	-	2.000	-
Section D	0.895	12.600	18	-	9.174	168
Section E11	5.950	0.500	4	-	7.468	32
Section E12	21.238	0.500	15	-	3.284	-
Section E2	5.034	0.740	5	-	2.000	11
Section F11	2.520	0.500	2	-	7.468	13
Section F12	0.840	0.500	1	-	6.718	-
Section F13	1.050	0.500	1	-	7.818	-
Section F2	2.205	0.500	2	-	7.368	12
Section G1	0.135	12.100	2	-	7.355	17
Section G2	0.125	13.136	2	-	3.284	8
Total EQ of Abutment Selfweight			496			1502

6. Braking Force(BR)

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

- Number of lanes
 - Multiple presence factor
 - Take 25 % of Truck load
- BR = 25% * n * m * (2*145+35)
- Acting at 1.8m higher of road face

n	=	3 lanes	
m	=	0.85	
BR	=	104 kN	Long. Axis
e	=	11.9 m	
Mlong	=	1237 KNm	Long. Axis

7. Centrifugal Force, CE (3.6.3)

- Plan of bridge (1: "straight", 2: "Curve")
- Design Speed

$$C = 4/3 * (V^2 / gR)$$

Acting at 1.8m higher of road face

$$CE = n * m * (2*145+35) * C$$

	=	1	
V	=	120 km/h	
V	=	33.3 m/s	
R	=	- m	
C	=	-	
CE	=	0.00 KN	
e	=	11.95 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	21.00	m
Span between bearings	Lb	20.30	m
Skew angle	α	0.00	deg
Deck slab length	Ldeck	21.00	m
Bridge Width	Bc	12.48	m
Girder height	hgl	1.45	m
Deck slab depth	hdkslab	0.22	m
Asphalt depth	has	0.084	m
Unit weight of concrete	γ_c	24.50	kN/m ³
Unit weight of asphalt concrete	γ_a	22.10	kN/m ³

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	308.82	kN
Diaphragm	DC	197.13	kN
Total	DC	1983.29	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	497.70	kN

2. Live load (LL):

Truck	
Tandem	
Lane load	
Pedestrian	Wpd = 0.0 kN/m ²
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

$$\text{Reaction} = [(1+IM)*\text{Vehicle} + \text{Lane load}]*n*m$$

Truck	P1(kN)	P2(kN)	P3(kN)	Sum(kN)	Total(kN)
Axle load	145	145	35		
Influence value	1.000	0.788	0.576		
Reaction	145.0	114.3	20.2	279.5	890.8

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	680.5

Lane load	Wl(kN/m)	Total(kN)
Value	9.3	
Influence value	10.15	
Reaction	94.4	240.7

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

3. Earthquake effects on superstructure (EQ)

Longitudinal moveable bearings at Abutment

Horizontal force from superstructure due to EQ - transverse direction
At bearing

$$H_{eq} = 143 \text{ kN}$$

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

Bearing displacement due to uniform temperature and shrinkage creep

$$\Delta u = 0.026 \text{ m}$$

$$H = G.A.\Delta u/h_r$$

<14.6.3.1-2>

Shear modulus G

$$G = 1 \text{ MPa}$$

Bearing area

$$A = 0.165 \text{ m}^2$$

Height of elastomeric layers

$$h_r = 0.065 \text{ m}$$

Number of bearing

$$n_b = 5 \text{ bears}$$

Horizontal force due to TU+SH&CR

$$H(tu+sh+cr) = 330 \text{ kN}$$

Acting at top of bearing

5. Wind loads (Ws)

5.1. Transverse wind on superstructure (WS)

Wind zone

Zone III

Basic 3 second gust wind

$$V_b = 53.00 \text{ m/s}$$

Correction factor

$$S = 1.09$$

Design wind velocity

$$V = 57.77 \text{ m/s}$$

Drag coefficient

$$C_d = 1.39$$

Overall width of bridge

$$b = 12.48 \text{ m}$$

Depth of superstructure (including solid parapet)

$$d = 2.73 \text{ m}$$

$$b/d = 4.57$$

Windy obstructed area of superstructure

$$A_t = 57.35 \text{ m}^2$$

Force due to transverse wind

$$F_{hy} = 160.0 \text{ kN}$$

$$F_{hy} = \max(0.0006*V^2*A_t*C_d, 1.8*A_t) \text{ (kN)}$$

<3.8.1>

5.2. Wind load on vehicles (WL)

Transverse wind on vehicles

$$W_{ltran} = 1.50 \text{ kN/m}$$

Transverse horizontal force due to wind on live load

$$F_{hy} = 31.50 \text{ kN}$$

At 1.8m from surface

6. Combinations

Loads from superstructure to Abutment

Loads at bottom of stem		Vertical		Longitudinal			Transversal		
Loads	Sign	N (kN)	x (m)	Hx (kN)	z ₁ (m)	My (kN·m)	Hy (kN)	y (m)	Mx (kN·m)
Girder + Deckslab	DC	1685	0.15			253			
Handrail	DC	249	0.15			37			
Pavement	DW	224	0.15			34			
Live Load	LL	1131	0.15			170		1.38	1556
Pedestrian	PL	0	0.15			0		-	-
Trans. wind on Struc.	WS						80	6.28	502
Trans. wind on vehl.	WL						16	8.08	127
Earth quake	EQ						143	6.28	899
TU+SH&CR	TU+SH&CR			330	6.28	2072			

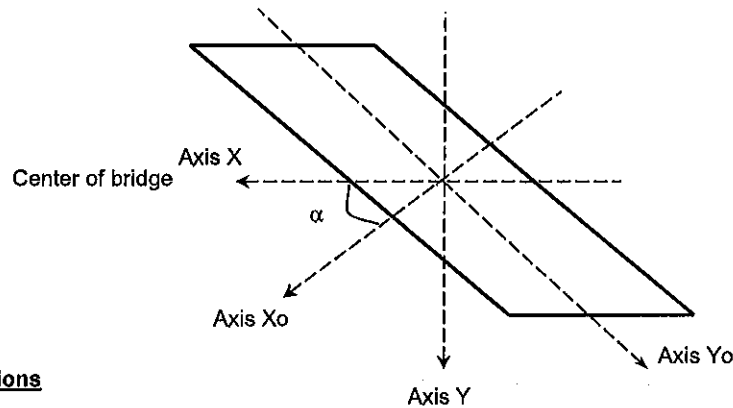
Loads at bottom of pilecap		Vertical		Longitudinal			Transversal		
Loads	Sign	N (kN)	x (m)	Hx (kN)	z ₁ (m)	My (kN.m)	Hy (kN)	y (m)	Mx (kN.m)
Girder + Decks slab	DC	1685	0.90			1517			
Handrail	DC	249	0.90			224			
Pavement	DW	224	0.90			201			
LiveLoad	LL	1131	0.90			1018		1.38	1556
Pedestrial	PL	0	0.90			0		-	-
Trans. wind on Struc.	WS						80	8.28	662
Trans. wind on vehl.	WL						16	10.08	159
Eearth quake	EQ						143	8.28	1186
TU+SH&CR	TU+SH&CR			330	8.28	2732			

Loads	Sign	Load combinations						
		Str-IA	Str-IB	Str-IIIA	Str-IIIB	Ser-I	Ext-IA	Ext-IB
Girder + Decks slab	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 vehl.	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					
Load combinations	N (kN)	Longitudinal		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	4733	165	1746	0	2723
Strength Str-IB	3866	165	1616	0	2723
Strength Str-IIIA	4281	165	1678	48	2428
Strength Str-IIIB	3414	165	1548	48	2428
Service Ser-I	3289	330	2565	40	1834
Extreme Ext-IA	3319	0	498	143	1677
Extreme Ext-IB	2452	0	368	143	1677

Load combinations at bottom of pilecap					
Load combinations	N (kN)	Longitudinal		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	4733	165	5626	0	2723
Strength Str-IB	3866	165	4846	0	2723
Strength Str-IIIA	4281	165	5219	48	2524
Strength Str-IIIB	3414	165	4438	48	2524
Service Ser-I	3289	330	5692	40	1913
Extreme Ext-IA	3319	0	2987	143	1964
Extreme Ext-IB	2452	0	2207	143	1964

LOAD COMBINATIONS



III. Load Combinations

1. Loads from substructure

Loads at bottom of pilecap	Sign	Vertical		Longitudinal			Transversal		
		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	8425				840			608.747
Soils on pilecap	EV	6490				-10763			
Horizontal Earth Pressure	EH			3832		15431			
Vertical Surcharge	L _{Sv}	484				-847			
Horizontal Surcharge	L _{Sh}			464		2337			
Braking Force	BR			104		1237			
Centrifugal Force	CE			-		-	-		-
Buoyancy of Abutment	WA	-2054				-116			
Buoyancy of Earth on Abutment	WA	-771				561			
Earthquake effects to Abutment	EQ			496		1502	149		451
Earthquake effects to soil	E _{AE}			4325		14551			

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
Centrifugal 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		Transversal	
		H _x (kN)	M _y (kN.m)	H _y (kN)	M _x (kN.m)
Strength Str-IA	17316	6741	14883	0	761
Strength Str-IB	11447	4442	10174	0	548
Strength Str-IIIA	17122	6514	13792	0	761
Strength Str-IIIB	11253	4215	9083	0	548
Service Ser-I	12575	4399	8680	0	609
Extreme Ext-IA	16711	5104	4382	149	1212
Extreme Ext-IB	10841	5104	8932	149	999

2. Loads from superstructure

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	4733	165	5626	0	2723
Strength Str-IB	3866	165	4846	0	2723
Strength Str-IIIA	4281	165	5219	48	2524
Strength Str-IIIB	3414	165	4438	48	2524
Service Ser-I	3289	330	5692	40	1913
Extreme Ext-IA	3319	0	2987	143	1964
Extreme Ext-IB	2452	0	2207	143	1964

3. Total loads at bottom of pilecap

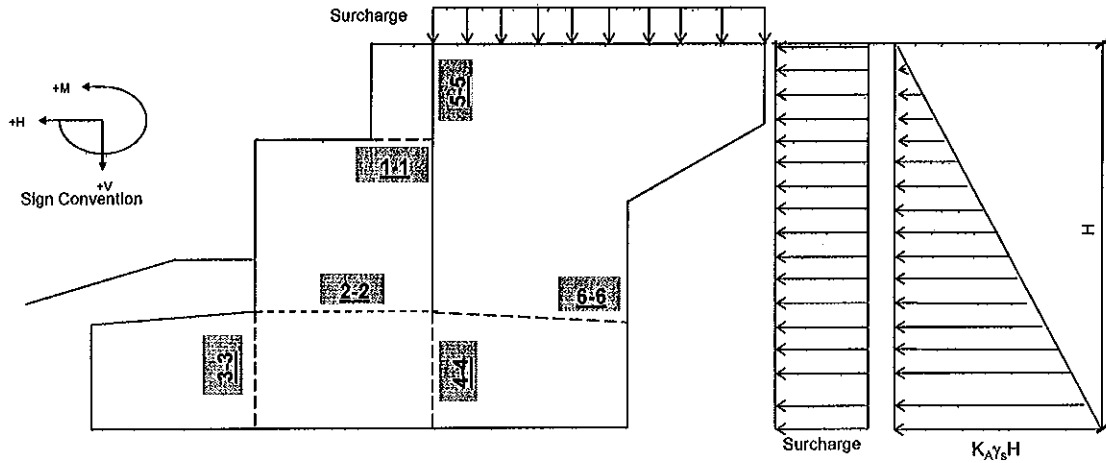
Load Combination at bottom of pilecap					
Load combinations	N (kN)	Longitudinal		Tranversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	22049	6906	20509	0	3484
Strength Str-IB	15313	4607	15019	0	3270
Strength Str-IIIA	21403	6679	19011	48	3285
Strength Str-IIIB	14666	4380	13521	48	3072
Service Ser-I	15864	4729	14372	40	2522
Extreme Ext-IA	20030	5104	7370	292	3175
Extreme Ext-IB	13293	5104	11138	292	2962

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		Extreme I Ext-I
			Str-IA	Str-IB	
Selfweight	DC	1.00	1.25	0.90	1.25
Horizontal Earth Pressure	EH	1.00	1.50	0.90	
Surcharge (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		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Selfweight	559		-113		
Horizontal Earth Pressure		121	87		
Surcharge (horizontal)		217	194		
Horizontal Seismic Earth Pressure		137	82		
Abutment earthquake force		21	18	6	6

Load Combination at bottom of headwall

Load combinations	N (kN)	Longitudinal		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Service Ser-I	559	338	168	0	0
Strength Str-IA	698	561	328	0	0
Strength Str-IB	503	488	316	0	0
Extreme Ext-I	698	334	97	6	6

1.2 Section 2-2

Table of Load Factors

Loads	Sign	Service I Ser-I	Strength I		Extreme I Ext-I
			Str-IA	Str-IB	
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	
Surcharge (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		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Selfweight	3466		-166		
Superstructure Dead Load	1685		253		
Pavement	224		34		
Handrail+curb	249		37		
Live Load	1131		170		1556
Horizontal Earth Pressure		2461	7941		
Surcharge (Horizontal)		400	1616		
TU+SH&CR		330	2072		
Horizontal Seismic Earth Pressure		2777	7488		
Abutment earthquake force		214	754	107	503

Load Combination at bottom of stem wall					
Load combinations	N (kN)	Longitudinal		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Service Ser-I	6756	3191	11955	0	1556
Strength Str-IA	9066	4557	16276	0	2723
Strength Str-IB	6986	3080	11440	0	2723
Extreme Ext-I	7652	4580	13084	107	1280

1.3 Section 3-3

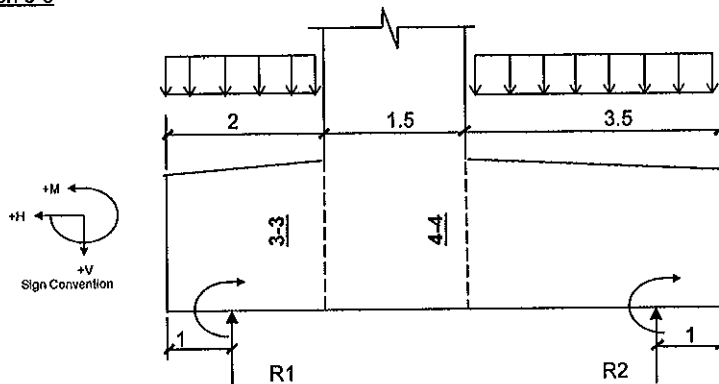


Table of Load Factors

Loads	Sign	Service I Ser-I	Strength I		Extreme I Ext-I
			Str-IA	Str-IB	
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		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Selfweight at front side	1235		1235		
Vertical soil on foot at front side	340		340		
Reaction of piles					
Ser-I	-11605	-2500	-7124	49	221
Str-IA	-9631	-3025	-4063	-129	-143
Str-IB	-17610	-3946	-10521	121	408
Ext-I	-12504	-2702	-7652	61	242

Load Combination at section 3-3					
Load combinations	N (kN)	Longitudinal		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Service Ser-I	-10030	-2500	-5549	49	221
Strength Str-IA	-7628	-3025	-2061	-129	-143
Strength Str-IB	-16193	-3946	-9104	121	408
Extreme Ext-I	-10501	-2702	-5649	61	242

1.4 Section 4-4

Table of Load Factors

Loads	Sign	Service I Ser-I	Strength I		Extreme I Ext-I
			Str-IA	Str-IB	
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		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Selfweight of behind side	2666		-4937		
Vertical soil on foot at behind side	6150		-10763		
Surcharge(Vertical)	484		-847		
Reaction of piles					
Ser-I	-3061	-1875	11013	-98	-110
Str-IA	-2812	-1347	9476	-92	-128
Str-IB	-4443	-2960	16424	-121	-125
Ext-I	-3359	-2027	12037	-100	-116

Load Combination at section 4-4					
Load combinations	N (kN)	Longitudinal		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Service Ser-I	6240	-1875	-5533	-98	-110
Strength Str-IA	9671	-1347	-12707	-92	-128
Strength Str-IB	4339	-2960	811	-121	-125
Extreme Ext-I	8519	-2027	-9088	-100	-116

1.4 Section 5-5 & 6-6

Slope of triang pressure
Uniform horizontal pressure

$\tan \beta = 6.00$
 $U.p = 3.66 \text{ kN/m}^2$

Load Combination at section 5-5					
Load combinations	N (kN)	Horizontal		Vertical	
		Hx (kN)	Mx (kN.m)	Hy (kN)	My (kN.m)
Service Ser-I		70	163		
Strength Str-IA		110	259		

Load Combination at section 6-6					
Load combinations	N (kN)	Horizontal		Vertical	
		Hx (kN)	Mx (kN.m)	Hy (kN)	My (kN.m)
Service Ser-I				193	170
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 concrete supporting the bearing device

Factored bearing resistance shall be taken

$$Pr = \phi \cdot Pn = \phi \cdot 0.85 \cdot f_c \cdot A1 \cdot m$$

Dimension of bearing plate

w0	=	0.550 m
b0	=	0.650 m
A1	=	0.358 m ²
w	=	1.000 m
b	=	1.100 m
A2	=	1.100 m ²

Area under bearing device

Distributed width and length

Notational area

Where supporting surface is wider on all sides than loaded area

$$m = \sqrt{A2/A1} \leq 2.0 \quad \text{case 1}$$

where loaded area is subjected to nonuniformly distributed bearing

$$m = 0.75 \cdot \sqrt{A2/A1} \leq 1.5 \quad \text{case 2}$$

Modification factor

Resistance factor

Factored bearing resistance

Bearing reaction of approach bridge

$$Pu = 1.25 \cdot DC + 1.5 \cdot DW + 1.75 \cdot LL$$

m	=	1.754
ϕ	=	0.700
Pr	=	11194 kN
Pu	=	3609 kN

<S.5.5.4.2>

> Pu

Ok

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

$$Pr = \phi \cdot fn \cdot Ab$$

fn take the lesser of

$$fn = 0.7 \cdot f_{ci} \cdot \sqrt{A/Ag} \text{ and}$$

$$fn = 2.25 \cdot f_{ci}$$

Maximum area of the portion of supporting surface

Gross area of bearing plate

Effective net area of bearing plate, Ag minus stud of bearing

Nominal concrete strength at time of application

Factored bearing resistance

fn	=	36.84 MPa
A	=	1.100 m ²
Ag	=	0.358 m ²
Ab	=	0.358 m ²
fci	=	30 MPa
Pr	=	9218 kN

Ok

REINFORCEMENT CHECKING - HEAD AND STEM 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				
			1-1	1-1	2-2	2-2	2-2
INTERNAL FORCES AT SECTION							
	Combination		Strength	Service	Service	Strength	Extreme
Qu	Shear	kN	561	338	3191	4557	4580
Mu	Flexural Moment	kNm	328	168	11955	16276	13084
Nu	Axial load	kN	698	559	6756	9066	7652
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	12.600	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
		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	0 T00.0
		Number	tendons	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000
As	Tension Reinforcement	Number	bars	83	77	77	77
		Diameter	mm	16	25	25	25
		Area	m2	0.01677	0.01677	0.03781	0.03781
A's	Compression Reinforcement	Number	bars	83	77	77	77
		Diameter	mm	16	16	16	16
		Area	m2	0.01677	0.01677	0.01555	0.01555
A'c	Shear reinforcement	Number	bars	20	19	19	19
		Diameter	mm	14	14	14	14
		Area	m2	0.00302	0.00302	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 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
a	Depth of equivalent stress block	m	0.000	0.000	0.028	0.028	0.028
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	21308	21308	21308
Mr	Factored resistance	kNm	2318	2575	21308	19177	21308
Mu	Flexural moment	kNm	328	168	11955	16276	13084

(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	10004	10004	10004
(5.7.3.3.2)	Checking $M_r \geq \min(1.2M_{cr}, 1.33M_u)$		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	m ²	0.018	0.018	0.019	0.019	0.019
f _{sa}	Value	Mpa	298	298	287	287	287
0.6*f _y		Mpa	240	240	240	240	240
	Tensil stress in reinf Min(f _{sa} , 0.6f _y)	Mpa	240	240	240	240	240
x	Dist. From compression fiber to centroid	m	-	0.082	0.226	-	-
J.d	Arm	m	-	0.415	1.366	-	-
I _{cr}	Moment of inertia of the cracked section	m ⁴	-	0.018	0.442	-	-
f _s	Tensile stress in reinforcement f _s = M _s / (A _s *J.d)	Mpa	-	24	232	-	-
	Checking for control cracking f _s < f _{sa}		N.a	OK	OK	N.a	N.a
(5.10.8.2)	Shrinkage and temperature Reinforcement (side distribution)						
A _{req}	Area of required reinf	m ²	0.00045	0.00045	0.00126	0.00126	0.00126
	Distribution on sides 7 D16	m ²	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		3.4	4.0	2.2	2.0	2.1
θ	Angle of inclination of diagonal compressive	degree	28.54	27.00	36.44	40.27	38.25
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	90
b _v	Effective web width as minimum web width - in d _v	m	12.600	12.600	12.600	12.600	12.600
d _v	Effective shear depth	m	0.442	0.442	1.427	1.427	1.427
	(d _e - 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
n _{cat}	Amount of bars in spacing S	bars	20	20	19	19	19
A _v	Shear reinf area in spacing S	m ²	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/m ²	112	61	177	282	255
f _{po}	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e _x	Strain in tensile reinforcement		2.69E-04	1.24E-04	1.04E-03	1.43E-03	1.23E-03
	if e _x < 0, multiple with reduce factor		-	-	-	-	-
	Strain checking	$\leq 2.00E-3$	Ok	Ok	Ok	Ok	Ok
v/fc	Ratio of shear stress and f _c		0.004	0.002	0.006	0.009	0.008
β	Final value		3.4	4.0	2.2	2.0	2.1
θ	Final value	degree	28.54	27.00	36.44	40.27	38.25
V _c	Nominal shear resistance provided by tensile stresses in the concrete	kN	8648	10125	18030	16274	17198
V _s	Shear resistance provided by shear reinforcement	kN	1636	1747	3697	3222	3462
V _p	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0	0
V _{n1}	V _{n1} = V _c + V _s + V _p	kN	10284	11871	21727	19495	20660
V _{n2}	V _{n2}	kN	41769	41769	134866	134866	134866
V _n	Nominal shear resistance V _n = min(V _{n1} , V _{n2})	kN	10284	11871	21727	19495	20660
V _r	Factored shear resistance	kN	9256	11871	21727	17546	20660
V _u	Shear	kN	561	338	3191	4557	4580
(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	m ²	0.0086	0.0086	0.0086	0.0086	0.0086
	Minimum shear reinforcement Checking		-	-	-	-	-
	0.1*f _c *b _v *d _v	kN	16708	16708	53946	53946	53946
	S _{max}	m	0.35	0.35	0.60	0.60	0.60
	Maximum spacing S _{max}		-	-	-	-	-

REINFORCEMENT CHECKING - PILECAP SECTION

MATERIALS			
NORMAL CONCRETE			
f_c	Compressive Strength of concrete at 28 days	Mpa	30
E_c	Modulus of Elasticity	Mpa	27691
f_r	Modulus of Rupture	Mpa	3.5
g_c	Unit weight of concrete	kN/m3	24.5
PRESTRESSING STEEL			
f_{pu}	Tensile strength of prestressing steel	Mpa	1860
f_{py}	Yield strength of prestressing steel	Mpa	1670
E_p	Modulus of Elasticity	Mpa	195000
REINFORCEMENT			
f_y	Yield strength	Mpa	400
E_s	Modulus of Elasticity	Mpa	200000
n_c	Ratio E_s/E_c		7

Sign	Parameters	Unit	Sections				
			3-3	3-3	3-3	4-4	4-4
INTERNAL FORCES AT SECTION							
	Combination		Service	Strength	Extreme	Extreme	Strength
Qu	Shear	kN	10030	7628	10501	8519	9671
Mu	Flexural Moment	kNm	5549	2061	5649	9088	12707
Nu	Axial load	kN	2500	3025	2702	2027	1347
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	12.600	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
		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	0 T00.0
		Number	tendons	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000
As	Tension Reinforcement	Number	bars	84	84	84	84
		Diameter	mm	22	22	22	22
		Area	m2	0.03192	0.03192	0.03192	0.03192
A's	Compression Reinforcement	Number	bars	84	84	84	84
		Diameter	mm	22	22	22	22
		Area	m2	0.03192	0.03192	0.03192	0.03192
A/c	Shear reinforcement	Number	bars	20	20	20	20
		Diameter	mm	16	16	16	16
		Area	m2	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	0.000	0.000	0.000	0.000	0.000
	For T section behavior	m	0.000	0.000	0.000	0.000	0.000
	For rectangular section behavior	m	0.000	0.000	0.000	0.000	0.000
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	1860	1860	1860
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	0.000	0.000	0.000
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	22408	22408	22408	22408	22408
Mr	Factored resistance	kNm	22408	20167	22408	22408	20167
Mu	Flexual moment	kNm	5549	2061	5649	9088	12707

(5.7.3.2)	Flexural moment Checking		OK	OK	OK	OK	OK
	Limits for reinforcement						
c/de	Maximum reinforcement		0.00	0.00	0.00	0.00	0.00
	Maximum reinforcement Checking	≤ 0.42	OK	OK	OK	OK	OK
1.2*Mc _r	Cracking moment	kNm	17391	17391	17391	17391	17391
(5.7.3.3.2)	Checking $M_r \geq \min(1.2M_{cr}, 1.33M_u)$		OK	OK	OK	OK	OK
(5.7.3.4)	Control of cracking by distr. of reinf for RC member- Check?		Yes	No	No	No	No
	Existing condition for structure	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	m ²	0.018	0.018	0.018	0.018	0.018
f _{sa}	Value	Mpa	169	169	169	169	169
0.6*f _y	Value	Mpa	240	240	240	240	240
	Tensile stress in reinf Min(f _{sa} , 0.6f _y)	Mpa	169	169	169	169	169
x	Dist. From compression fiber to centroid	m	0.238	-	-	-	-
J.d	Arm	m	1.76	-	-	-	-
I _{cr}	Moment of inertia of the cracked section	m ⁴	0.634	-	-	-	-
f _s	Tensile stress in reinforcement $f_s = M_{sls} / (A_s * J.d)$	Mpa	99	-	-	-	-
	Checking for control cracking $f_s < f_{sa}$		OK	N.a	N.a	N.a	N.a
(5.10.8.2)	Shrinkage and temperature Reinforcement (side distribution)						
A _{req}	Area of required reinf	m ²	0.00127	0.00127	0.00127	0.00127	0.00127
	Distribution on sides	m ²	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	2.4	2.1	1.9	1.7
θ	Angle of inclination of diagonal compressive	degree	38.74	30.85	38.61	41.80	43.00
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	90
b _v	Effective web width as minimum web width - in dv	m	12.600	12.600	12.600	12.600	12.600
d _v	Effective shear depth	m	1.839	1.839	1.839	1.916	1.916
	(d _e - a/2)	m	1.839	1.839	1.839	1.916	1.916
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	0.600
n _{cat}	Amount of bars in spacing S	bars	20	20	20	20	20
A _v	Shear reinf area in spacing S	m ²	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/m ²	433	366	453	33	445
f _{po}	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e _x	Strain in tensile reinforcement		1.27E-03	6.16E-04	1.26E-03	1.70E-03	2.03E-03
	if e _x < 0, multiple with reduce factor		-	-	-	-	-
	Strain checking	$\leq 2.00E-3$	Ok	Ok	Ok	Ok	N.G
v/f _c	Ratio of shear stress and f _c		0.014	0.012	0.015	0.001	0.015
β	Final value		2.1	2.4	2.1	1.9	1.7
θ	Final value	degree	38.74	30.85	38.61	41.80	43.00
V _c	Nominal shear resistance provided by tensile stresses in the concrete	kN	21872	25759	21949	20392	18877
V _s	Shear resistance provided by shear reinforcement	kN	6173	8293	6202	5772	5534
V _p	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0	0
V _{n1}	V _{n1} = V _c + V _s + V _p	kN	28045	34051	28151	26164	24411
V _{n2}	V _{n2}	kN	173786	173786	173786	181062	181062
V _n	Nominal shear resistance V _n = min(V _{n1} , V _{n2})	kN	28045	34051	28151	26164	24411
V _r	Factored shear resistance	kN	28045	30646	28151	26164	21970
V _u	Shear	kN	10030	7628	10501	8519	9671
(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
E_c	Modulus of Elasticity	Mpa	27691
f_r	Modulus of Rupture	Mpa	3.5
g_c	Unit weight of concrete	kN/m ³	24.5
PRESTRESSING STEEL			
f_{pu}	Tensile strength of prestressing steel	Mpa	1860
f_{py}	Yield strength of prestressing steel	Mpa	1670
E_p	Modulus of Elasticity	Mpa	195000
REINFORCEMENT			
f_y	Yield strength	Mpa	400
E_s	Modulus of Elasticity	Mpa	200000
n_c	Ratio E_s/E_c		7

Sign	Parameters	Unit	Sections				
			5-5	5-5	6-6	6-6	6-6
INTERNAL FORCES AT SECTION							
	Combination		Service	Strength	Service	Strength	Strength
Qu	Shear	kN	70	110	193	299	299
Mu	Flexural Moment	kNm	163	259	170	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
		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	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	3	3	3	3
		Diameter	mm	12	12	12	12
		Area	m2	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	0.013	0.013	0.013	0.013	0.013
	For T section behavior	m	0.013	0.013	0.013	0.013	0.013
	For rectangular section behavior	m	0.013	0.013	0.013	0.013	0.013
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	1847	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	0.28

a	Depth of equivalent stress block	m	0.011	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	0.441	0.441	0.441	0.441	0.441
Mn	Nominal resistance	kNm	303	303	303	303	303
Mr	Factored resistance	kNm	303	273	303	273	273
Mu	Flexural moment	kNm	163	259	170	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	Minimum reinforcement		0.38%	0.38%	0.38%	0.38%	0.38%
	Minimum reinforcement Checking for RC	0.23%	OK	OK	OK	OK	OK
1.2*Mcrr	Cracking moment	kNm	88	88	88	88	88
(5.7.3.3.2)	Checking $M_r \geq \min(1.2M_{cr}, 1.33M_u)$		OK	OK	OK	OK	OK
(5.7.3.4)	Control of cracking by distr. of reinf for RC member- Check?		Yes	No	Yes	No	No
	Existing condition for structure	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	m ²	0.020	0.020	0.020	0.020	0.020
f _{sa}	Value	Mpa	285	285	285	285	285
0.6*f _y	Tensile stress in reinf Min(f _{sa} , 0.6f _y)	Mpa	240	240	240	240	240
x	Dist. From compression fiber to centroid	m	0.095	-	0.095	-	-
J.d	Arm	m	0.409	-	0.409	-	-
I _{cr}	Moment of inertia of the cracked section	m ⁴	0.002	-	0.002	-	-
f _s	Tensile stress in reinforcement $f_s = M_{sls} / (A_s * J.d)$	Mpa	212	-	221	-	-
	Checking for control cracking $f_s < f_{sa}$		OK	N.a	OK	N.a	N.a
(5.10.8.2)	Shrinkage and temperature Reinforcement (side distribution)						
A _{req}	Area of required reinf	m ²	0.00031	0.00031	0.00031	0.00031	0.00031
	Distribution on sides 7 D16	m ²	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	1.8	2.0	1.7	1.7
θ	Angle of inclination of diagonal compressive	degree	37.23	41.98	39.60	43.00	43.00
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	90
b _v	Effective web width as minimum web width - in dv	m	1.000	1.000	1.000	1.000	1.000
d _v	Effective shear depth	m	0.436	0.436	0.436	0.436	0.436
	(de - a/2)	m	0.436	0.436	0.436	0.436	0.436
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	0.600
n _{cat}	Amount of bars in spacing S	bars	3	3	3	3	3
A _v	Shear reinf area in spacing S	m ²	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/m ²	161	281	443	763	763
f _{po}	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e _x	Strain in tensile reinforcement		1.12E-03	1.75E-03	1.36E-03	2.04E-03	2.04E-03
	if e _x <0, multiple with reduce factor		-	-	-	-	-
	Strain checking	$\leq 2.00E-3$	Ok	Ok	Ok	N.G	N.G
v/f _c	Ratio of shear stress and f _c		0.005	0.009	0.015	0.025	0.025
β	Final value		2.2	1.8	2.0	1.7	1.7
θ	Final value	degree	37.23	41.98	39.60	43.00	43.00
V _c	Nominal shear resistance provided by tensile stresses in the concrete	kN	428	364	402	341	341
V _s	Shear resistance provided by shear reinforcement	kN	130	110	119	106	106
V _p	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0	0
V _{n1}	V _{n1} =V _c +V _s +V _p	kN	558	473	521	446	446
V _{n2}	V _{n2}	kN	3268	3268	3268	3268	3268
V _n	Nominal shear resistance V _n =min(V _{n1} , V _{n2})	kN	558	473	521	446	446
V _r	Factored shear resistance	kN	558	426	521	402	402
V _u	Shear	kN	70	110	193	299	299
(5.8.2.7)	Shear checking		OK	OK	OK	OK	OK

Da Nang Quang Ngai Expressway project

BRIDGE
ORB22

CALCULATION SHEETS
PIER P1 Right

	DA NANG - QUANG NGAI EXPRESS WAY PROJECT ORB22 Right BRIDGE DETAIL DESIGN PIER P1 RIGHT DESIGN	Item.	Eng.	Date	Sign.
		Design	-		
		Check	-		
		Revise	-		

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 $A = 0.0580 g$

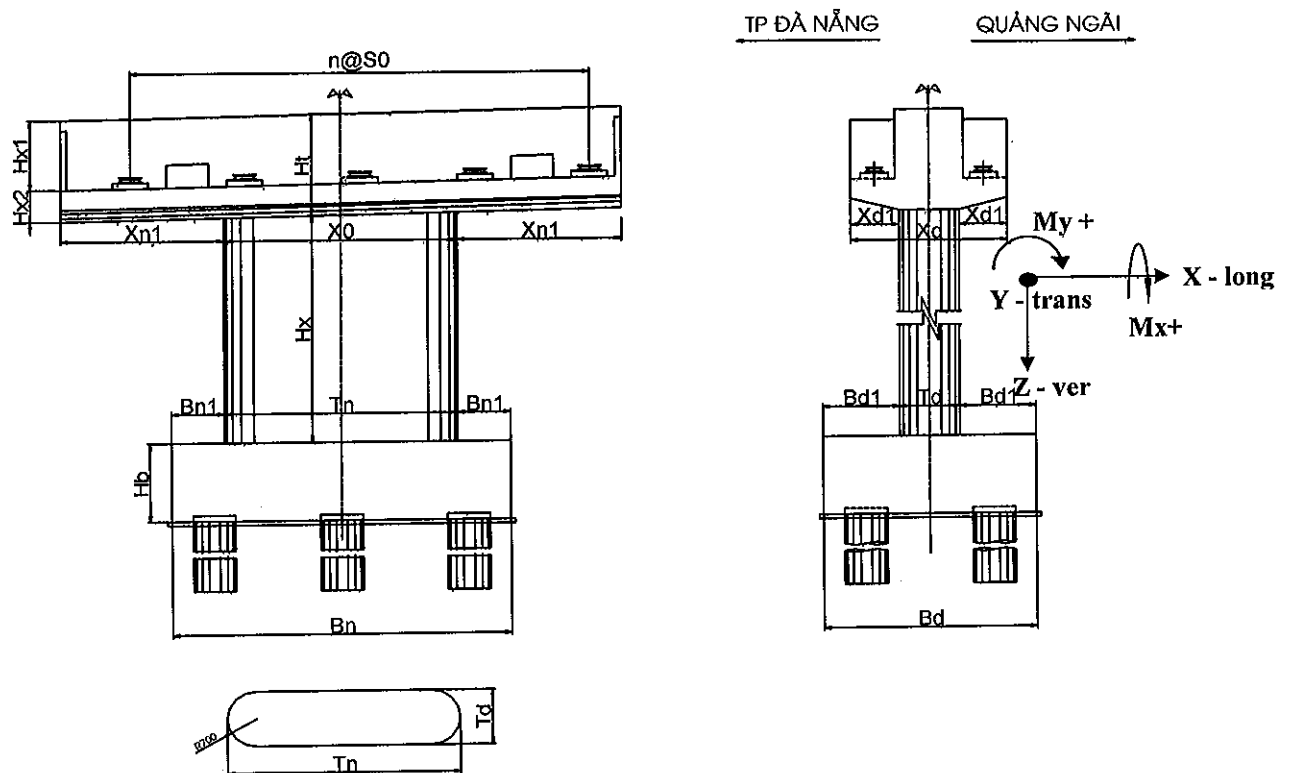
Input data:

Bridge type	<i>Simple PC I girder L=21m with link slab</i>			
Span length	Left	=	21.05	Right = 21.05 m
Girder length between bearings	Left	=	21.05	Right = 21.05 m
Bridge width	B	=	12.74	m

Level Table(at center of pier)			
Top of pier cap	ThL	12.870	m
Top of pier column	TcL	10.590	m
Bottom of upper pier column	BucL	10.590	m
Bottom of pier column	BcL	-1.000	m
Bottom of upper pilecap	BupL	-1.000	m
Bottom of pilecap	BpL	-2.500	m
Tip of pile	TpL		m
Skew angle	Ska	90.000	deg
Ground level	GL	2.980	m
Maximum water level (H1%)	HWL	6.970	m
Navigation water level (H5%)	NWL	4.000	m
Minimum water level	MWL	2.500	m
Average Annual water level	AWL	5.500	m
Local scour level (at water level H1%)	LsL	0.000	m

Material unit weight			
Structural concrete	$\gamma_c =$	2500 kg/m ³	24.50 kN/m ³
Asphalt concrete	$\gamma_a =$	2250 kg/m ³	22.10 kN/m ³
Soil - ground	$\gamma_s =$	1800 kg/m ³	17.70 kN/m ³
Saturated soil	$\gamma_{ss} =$	800 kg/m ³	7.80 kN/m ³

II. PIER DIMENSIONS



Pier Dimensions Table

Notation	Dimensions	Value (m)	Notation	Dimensions	Value (m)
	Transverse direction			Longitudinal direction	
*	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		*	Anchorage block	
	Width	0.65		Width	0.40
	Length	0.55		Length	1.00
	Height	0.15		Height	0.52
	Number	10.00		Number	4.00
*	Pier Cap				
Hx1	Haunch 1 height	1.48	xd	Pier cap width	1.40
Hx2	Haunch 2 height	0.80	GL	Left bearing to pier c.line	1.200
Hx	Pier cap height	2.28	GR	Right bearing to pier c.line	1.200
Xn1	Haunch width	3.49			
Xn0	Bottom of pier cap width	5.50	Hc	Curtain wall height	1.20
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 thickness	1.40
Htt	Pier column height	11.59	Rv	Round nose radius	0.70
tnb	Upper pier column width	0.00	tdb	Upper pier column thickness	0.00
Htb	Upper pier column height	0.00	Rvb	Upper round nose radius	0.00
Ht	Column height	11.59			

Notation	Dimensions Transverse direction	Value (m)	Notation	Dimensions Longitudinal direction	Value (m)
*	Pile Cap				
Bn	Pile cap width	8.00	Bd	Pile cap length	5.50
Hb	Pile cap depth	1.50			
Bn1	Transverse cantilever	1.25	bd1	Long. Cantilever	2.05
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 (m3)	Vertical F_V (kN)	Longitudinal			Transversal		
			F_{HX} (kN)	Arm. $_{HX}$ (m)	M_y (kN•m)	F_{HY} (kN)	Arm. $_{HY}$ (m)	M_x (kN•m)
Bearing pedestal	0.54	13.1						
Bearing devices		50.0						
Anchorage block	0.84	20.5						
Pier Cap	70.25	1721.1						
Curtain wall	0.50	12.1						
Upper pier column	0.00	0.0						
Pier Column	84.37	2067.0						
Upper pilecap	0.00	0.0						
PileCap	66.00	1617.0						
Shear key	0.00	0.0						
Total at bottom of Column		3883.9						
Total at bottom of pilecap		5500.9						

2. Soil on pilecap

Item	Volume (m3)	Vertical F_V (kN)	Longitudinal			Transversal		
			F_{HX} (kN)	Arm. $_{HX}$ (m)	M_y (kN•m)	F_{HY} (kN)	Arm. $_{HY}$ (m)	M_x (kN•m)
Soil on pile cap	146.15	2586.8						
Total at bottom of Column								
Total at bottom of pilecap		2586.8						

3. Buoyancy on pier

Case1: Maximum water level (H1%)

Item	Volume (m3)	Vertical F_V (kN)	Longitudinal			Transversal		
			F_{HX} (kN)	Arm. $_{HX}$ (m)	M_y (kN•m)	F_{HY} (kN)	Arm. $_{HY}$ (m)	M_x (kN•m)
Upper pier column	0.00	0.0						
Pier Column	58.02	-569.1						
Upper pilecap	0.00	0.0						
PileCap	66.00	-647.5						
Shear key	0.00	0.0						
Total at bottom of Column		-569.1						
Total at bottom of pilecap		-1216.6						

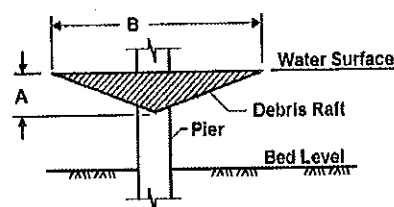
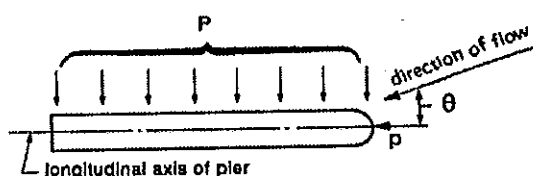
Case2: Minimum water level (Hmin)

Item	Volume (m3)	Vertical F _V (kN)	Longitudinal			Transversal		
			F _{HX} (kN)	Arm _{HX} (m)	M _y (kN•m)	F _{HV} (kN)	Arm _{HV} (m)	M _x (kN•m)
Upper pier column	0.00	0.0						
Pier Column	25.48	-249.9						
Upper pilecap	0.00	0.0						
PileCap	66.00	-647.5						
Shear key	0.00	0.0						
Total at bottom of Column		-249.9						
Total at bottom of pilecap		-897.4						

Case3: average Annual water level

Item	Volume (m3)	Vertical F _V (kN)	Longitudinal			Transversal		
			F _{HX} (kN)	Arm _{HX} (m)	M _y (kN•m)	F _{HV} (kN)	Arm _{HV} (m)	M _x (kN•m)
Upper pier column	0.00	0.0						
Pier Column	47.32	-464.2						
Upper pilecap	0.00	0.0						
PileCap	66.00	-647.5						
Shear key	0.00	0.0						
Total at bottom of Column		-464.2						
Total at bottom of pilecap		-1111.6						

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"; "3:Debris lodged against the pier "; "4:Wedged - nosed pier with nose angle 90deg or less"		1	
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	2.0	m
Width of debris raft	B	14.0	m
Area of debris raft	Area	14.0	m2
Stream pressure due to driftwood raft at H1%	$pLdebris$	0.32	kN/m2
Equivalent force	$Fhdebris$	4.4	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 (m)	Vertical F_V (kN)	Longitudinal			Transversal		
			F_{HX} (kN)	Arm _{HX} (m)	M_y (kN•m)	F_{HY} (kN)	Arm _{HY} (m)	M_x (kN•m)
Upper pier Column	0.00		0.0	11.6	0.0	0.0	11.6	0.0
Pier Column	7.97		0.0	4.0	0.0	4.9	4.0	19.7
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	4.9		19.7
Upper pier Column	0.00		0.0	13.1	0.0	0.0	13.1	0.0
Pier Column	7.97		0.0	5.5	0.0	4.9	5.5	27.1
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	4.9		27.1

Additional stream pressure due to driftwood raft

Item	Exposed height (m)	Vertical F_V (kN)	Longitudinal			Transversal		
			F_{HX} (kN)	Arm _{HX} (m)	M_y (kN•m)	F_{HY} (kN)	Arm _{HY} (m)	M_x (kN•m)
Pier Column					0.0	4.4	8.0	35.2
Total at bottom of Column					0.0	4.4		35.2
Pier Column					0.0	4.4	9.5	41.9
Total at bottom of pilecap					0.0	4.4		41.9

Case2: Minimum water level (Hmin)

Item	Exposed height (m)	Vertical F_V (kN)	Longitudinal			Transversal		
			F_{HX} (kN)	Arm _{HX} (m)	M_y (kN•m)	F_{HY} (kN)	Arm _{HY} (m)	M_x (kN•m)
Upper pier Column	0.00		0.0	11.6	0.0	0.0	11.6	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	13.1	0.0	0.0	13.1	0.0
Pier Column	3.50		0.0	3.3	0.0	0.3	3.3	0.9
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	0.3		0.9

Case3: average Annual water level

Item	Exposed height (m)	Vertical F_V (kN)	Longitudinal			Transversal		
			F_{HX} (kN)	Arm _{HX} (m)	M_y (kN•m)	F_{HY} (kN)	Arm _{HY} (m)	M_x (kN•m)
Upper pier Column	0.00		0.0	11.6	0.0	0.0	11.6	0.0
Pier Column	6.50		0.0	3.3	0.0	4.0	3.3	13.1
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	4.0		13.1
Upper pier Column	0.00		0.0	13.1	0.0	0.0	13.1	0.0
Pier Column	6.50		0.0	4.8	0.0	4.0	4.8	19.2
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	4.0		19.2

5. Wind Loads

3.000

Wind loads data

$$P_D = 0.0006 V^2 C_d \geq 1.8 \text{ (kN/m}^2\text{)}$$

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		I
"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	7.9 m
Correct coefficient for wind zone and elevation of pier	S	1.09
Design wind speed $V = S \cdot Vb$	V	57.8 m/s
Obstacle coefficient for pier	Cd	1.3
Wind pressure on pier	P _D	2.60 kN/m ²

At Maximum water level (HI%)

Item	Exposed height (m)	Vertical F _V (kN)	Longitudinal			Transversal		
			F _{HX} (kN)	Arm. _{HX} (m)	M _y (kN·m)	F _{HY} (kN)	Arm. _{HY} (m)	M _x (kN·m)
Curtain wall	1.20		0.9	14.5	13.6	4.4	14.5	63.3
Pier Cap	2.28		74.1	12.7	942.9	8.3	12.7	105.8
Upper pier Column	0.00		0.0	8.0	0.0	0.0	8.0	0.0
Pier Column	3.62		51.8	9.8	506.9	13.2	9.8	129.0
Upper pilecap	0.00		0.0	8.0	0.0	0.0	8.0	0.0
Total at bottom of Column			126.8		1463.4	25.9		298.1
Curtain wall	1.20		0.9	16.0	15.0	4.4	16.0	69.8
Pier Cap	2.28		74.1	14.2	1054.0	8.3	14.2	118.2
Upper pier Column	0.00		0.0	9.5	0.0	0.0	9.5	0.0
Pier Column	3.62		51.8	11.3	584.6	13.2	11.3	148.8
Upper pilecap	0.00		0.0	9.5	0.0	0.0	9.5	0.0
PileCap	0.00		0.0	9.5	0.0	0.0	9.5	0.0
Total at bottom of pilecap			126.8		1653.6	25.9		336.9

At Minimum water level (Hmin)

Item	Exposed height (m)	Vertical F _V (kN)	Longitudinal			Transversal		
			F _{HX} (kN)	Arm. _{HX} (m)	M _y (kN·m)	F _{HY} (kN)	Arm. _{HY} (m)	M _x (kN·m)
Curtain wall	1.20		0.9	14.5	13.6	4.4	14.5	63.3
Pier Cap	2.28		74.1	12.7	942.9	8.3	12.7	105.8
Upper pier Column	0.00		0.0	3.5	0.0	0.0	3.5	0.0
Pier Column	8.09		115.8	7.5	873.9	29.5	7.5	222.5
Upper pilecap	0.00		0.0	3.5	0.0	0.0	3.5	0.0
Total at bottom of Column			190.8		1830.4	42.2		391.5
Curtain wall	1.20		0.9	16.0	15.0	4.4	16.0	69.8
Pier Cap	2.28		74.1	14.2	1054.0	8.3	14.2	118.2
Upper pier Column	0.00		0.0	5.0	0.0	0.0	5.0	0.0
Pier Column	8.09		115.8	9.0	1047.7	29.5	9.0	266.7
Upper pilecap	0.00		0.0	5.0	0.0	0.0	5.0	0.0
PileCap	0.00		0.0	5.0	0.0	0.0	5.0	0.0
Total at bottom of pilecap			190.8		2116.7	42.2		454.8

At average Annual water level

Item	Exposed height (m)	Vertical F _V (kN)	Longitudinal			Transversal		
			F _{HX} (kN)	Arm. _{HX} (m)	M _y (kN·m)	F _{HY} (kN)	Arm. _{HY} (m)	M _x (kN·m)
Curtain wall	1.20		0.9	14.5	13.6	4.4	14.5	63.3
Pier Cap	2.28		74.1	12.7	942.9	8.3	12.7	105.8
Upper pier Column	0.00		0.0	6.5	0.0	0.0	6.5	0.0
Pier Column	5.09		72.9	9.0	659.2	18.6	9.0	167.8

Upper pilecap	0.00		0.0	6.5	0.0	0.0	6.5	0.0
Total at bottom of Column			147.9		1615.6	31.2		336.8
Curtain wall	1.20		0.9	16.0	15.0	4.4	16.0	69.8
Pier Cap	2.28		74.1	14.2	1054.0	8.3	14.2	118.2
Upper pier Column	0.00		0.0	8.0	0.0	0.0	8.0	0.0
Pier Column	5.09		72.9	10.5	768.5	18.6	10.5	195.6
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			147.9		1837.5	31.2		383.7

6.Vessel Collision

Vessel collision load data

Class of navigable waterway; (Enter: "No" incase there is no vessel)		NO	
Design vessel tonnage	Self-propelled vessel	0.0	DWT
	Towed barge	0.0	DWT
Dimensions of design vessels:			
Self-propelled vessel	Maximum length (LOA)	1.0	m
	Maximum breadth	2.0	m
	Laden draught	3.0	m
Towed barge	Maximum length (LOA)	1.0	m
	Maximum breadth	2.0	m
	Laden draught	3.0	m
Mean annual stream velocity	Vs	1.11	m/s
Design impact velocity	Self-propelled vessel	Vs+2.5	3.61 m/s
	Towed barge	Vs+1.6	2.71 m/s

Ship Collision force on pier

$$P_s = 1.2e5 * V * \sqrt{\text{DWT}}$$

Equivalent static vessel impact force	Ps	0	kN
Vessel displacement tonnage	M	40	Mg
Underkeel clearance	T	1.5	m
Hydrodynamic mass coefficient	Ch	0.00	
Vessel collision energy, $KE = 500 \cdot Ch \cdot M \cdot V^2$	KE	0.0E+00	Joule
Ship bow damage length, $a = 1.54e3 \cdot (KE/Ps)$	as	0.0	mm

Towed barge Collision force on pier

$$P_b = 6e4 \cdot ab$$

Equivalent static vessel impact force	Pb	0	kN
Vessel displacement tonnage	M	20	Mg
Underkeel clearance	T	1.5	m
Hydrodynamic mass coefficient	Ch	0.00	
Vessel collision energy, $KE = 500 \cdot Ch \cdot M \cdot V^2$	KE	0.0E+00	Joule
Barge bow damage length, $a = 3100 \cdot [\sqrt{1+1.3e-7*KE}-1]$	ab	0.0	mm

Vessel collision is applied for pier P1 or P2

The values here are taken from whole bridge model analysis (refer to annex)

Item	Height (m)	Vertical F _v (kN)	Longitudinal			Transversal		
			F _{HX} (kN)	Arm _{HX} (m)	M _y (kN*m)	F _{HY} (kN)	Arm _{HY} (m)	M _x (kN*m)
Pier column	6.50		173.0	6.5	1447.0	404.0	6.50	3310.0
Total at bottom of Column								
Pier column	8.00		173.0	8.0	1794.0	404.0	8.00	4111.0
Total at bottom of pilecap			173.0		1794.0	404.0		4111.0

For substructure design, equivalent forces shall be applied separately as follows:

- 100% of design impact force in a direction parallel to the alignment of the centerline of the navigable channel
- 50% of design impact force in a direction normal to the alignment of the centerline of the navigable channel

Item	Height (m)	Vertical F_v (kN)	Longitudinal			Transversal		
			F_{HX} (kN)	Arm. _{HX} (m)	M_y (kN·m)	F_{HY} (kN)	Arm. _{HY} (m)	M_x (kN·m)
Pier column	16.32		0.0	16.3	0.0	0.0	16.32	0.0
Total at bottom of Column			0.0		0.0	0.0		0.0
Pier column	17.82		0.0	17.8	0.0	0.0	17.82	0.0
Total at bottom of pilecap			0.0		0.0	0.0		0.0

IV. SUPERSTRUCTURE LOADS

1. Dead Loads

Left side Span

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	60.30	738.7		1.200	-886.4			
Diaphragm	7.41	90.8		1.200	-108.9			
Precast plank	12.22	149.7		1.200	-179.7			
Deck slab	53.51	655.5		1.200	-786.6			
Total at bottom of Column		1634.7			-1961.6			
Total at bottom of pilecap		1634.7			-1961.6			
Stage2 (DW)								
Pavement	20.73	229.0		1.200	-274.8			
Parapet + railing		248.9		1.200	-298.6			
Lighting post + mis.		21.0		1.200	-25.2			
Total at bottom of Column		498.9			-598.7			
Total at bottom of pilecap		498.9			-598.7			

Right side Span

Right side Span

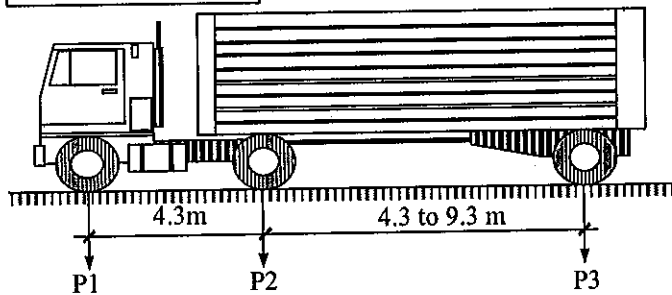
Item	Volume	Vertical F_v	Longitudinal			Transversal		
			F_{HX}	Arm. _{HX}	M_y	F_{HY}	Arm. _{HY}	M_x
	(m3)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Stage1 (DC)								
Girders	60.30	738.7		1.200	886.4			
Diaphragm	7.41	90.8		1.200	108.9			
Precast plank	12.22	149.7		1.200	179.7			
Deck slab	53.51	655.5		1.200	786.6			
Total at bottom of Column		1634.7			1961.6			
Total at bottom of pilecap		1634.7			1961.6			
Stage2 (DW)								
Pavement	20.73	229.0		1.200	274.8			
Parapet + railing		248.9		1.200	298.6			
Lighting post + mis.		21.0		1.200	25.2			
Total at bottom of Column		498.9			598.7			
Total at bottom of pilecap		498.9			598.7			

2. Live Load

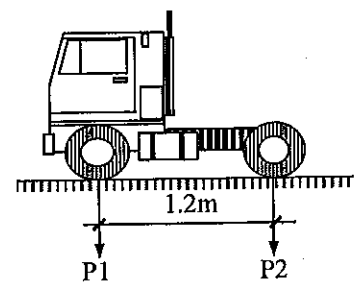
Live load data			
Design Truck	P1	35.0	kN
	P2	145.0	kN
	P3	145.0	kN

Design Tandem	P1	110.0 kN
	P2	110.0 kN
Design Lane Load	P_L	9.3 kN/m
Pedestrian Load	P_p	3.0 kN/m ²
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

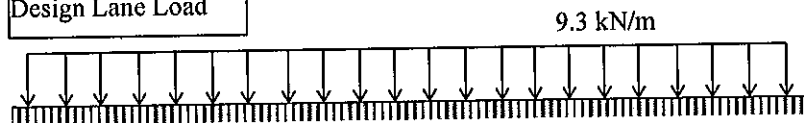
Design Truck



Design Tandem

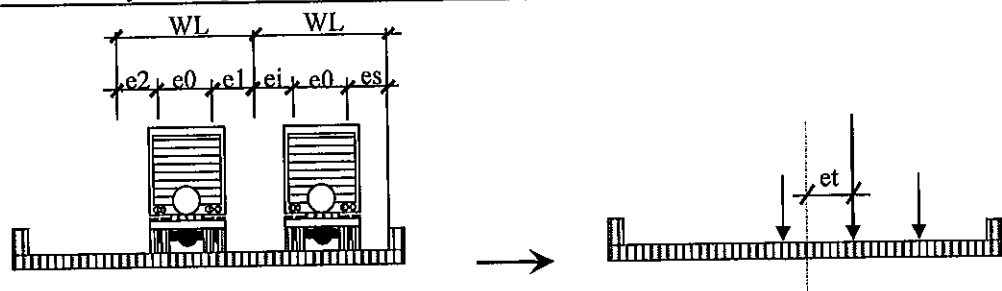


Design Lane Load



Transverse Distribution

Distance from wheel axis to inner face of curb and between 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	



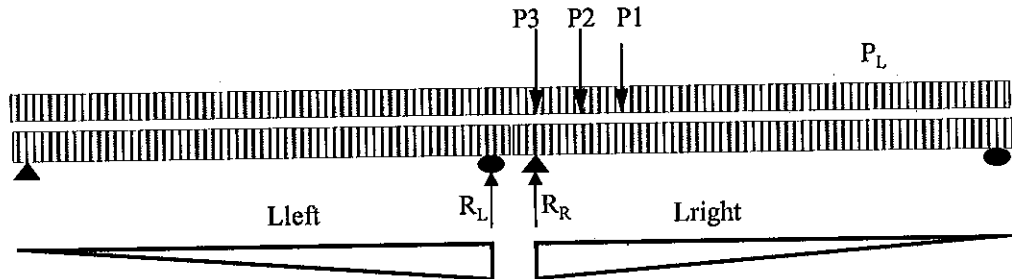
Longitudinal Distribution

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

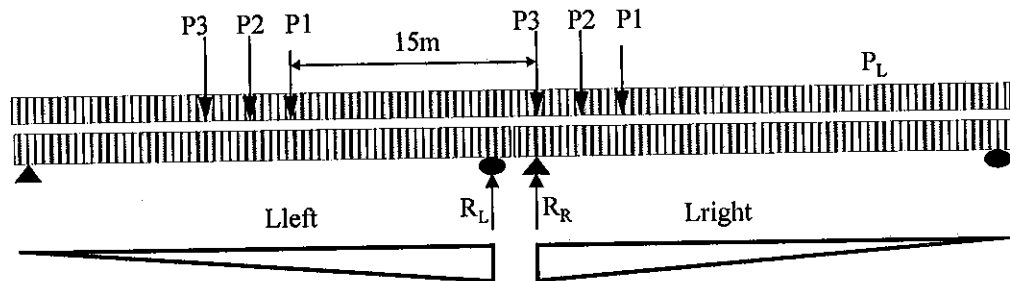
Influence line value								
Case	Left span				Right span			
	P3	P2	P1	Aleft	P3	P2	P1	Aright
Case1a:				10.53	1.00	0.80	0.59	10.53
Case1b:	0.91			10.53		1.00	0.80	10.53
Case2a:*	1.00	0.80	0.59	10.53	-0.12	-0.33	0.00	10.53
Case2b:	-0.01	0.20	0.40	10.53	1.00	0.80	0.59	10.53
Case3a:				10.53		1.00	0.94	10.53
Case3b:		1.06		10.53			1.00	10.53

* 2 Trucks in right span

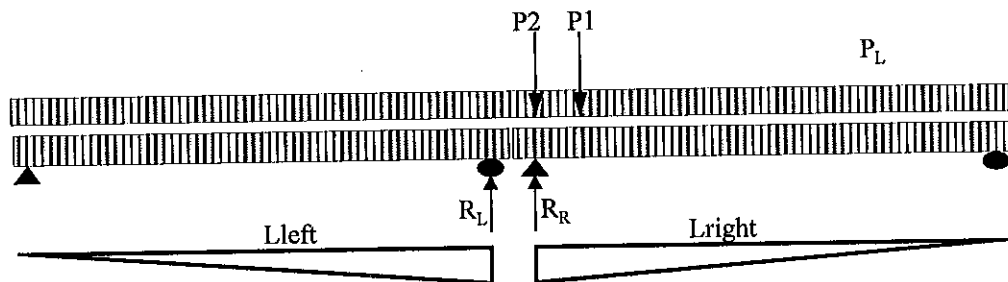
Truck + Lane load



2 Trucks + Lane load



Tandem + Lane load



For 1 truck or tandem: Reaction = $[(P_i \cdot y_i) \cdot (1 + IM) + PL \cdot A] \cdot n_{lane} \cdot m$

For 2 trucks: Reaction = $0.9 \cdot [(P_i \cdot y_i) \cdot (1 + IM) + PL \cdot A] \cdot n_{lane} \cdot m$

Item	Loaded Lane			Longitudinal			Transversal		
	Reaction	Reaction	Vertical	F _{HX}	Arm _{·HX}	M _y	F _{HV}	Arm _{·HV}	M _x
	left	right	F _v						
	(kN)	(kN)	(kN)						
Case1a:	117.5	539.1	656.5			505.9			3276.1
Case1b:	315.3	376.7	692.1			73.7			3453.4
Case2a:	105.7	397.8	503.5			350.5			2512.3
Case2b:	161.9	485.2	647.0			388.0			3228.8
Case3a:	117.5	438.1	555.5			384.7			2772.0
Case3b:	291.9	282.5	574.3			-11.3			2865.9

2	Loaded Lane			m = 1.00					
Item	Reaction	Reaction	Vertical	Longitudinal			Transversal		
	left	right	F _v	F _{HX}	Arm. _{HX}	M _y	F _{HY}	Arm. _{HY}	M _x
	(kN)	(kN)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Case1a:	195.8	898.5	1094.2			843.2			3818.9
Case1b:	525.5	627.9	1153.4			122.8			4025.5
Case2a:	176.2	662.9	839.1			584.1			2928.5
Case2b:	269.8	808.6	1078.4			646.6			3763.7
Case3a:	195.8	730.1	925.9			641.2			3231.2
Case3b:	486.4	470.8	957.2			-18.8			3340.7

3	Loaded Lane		m = 0.85						
Item	Reaction	Reaction	Vertical	Longitudinal			Transversal		
	left	right	F _v	F _{HX}	Arm _{HX}	M _y	F _{HY}	Arm _{HY}	M _x
	(kN)	(kN)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Case1a:	249.6	1145.5	1395.1			1075.1			2776.3
Case1b:	670.1	800.6	1470.6			156.6			2926.6
Case2a:	224.6	845.2	1069.9			744.7			2129.1
Case2b:	344.0	1031.0	1375.0			824.4			2736.2
Case3a:	249.6	930.9	1180.5			817.5			2349.1
Case3b:	620.2	600.2	1220.4			-24.0			2428.7

4	Loaded Lane		m = 0.65						
Item	Reaction	Reaction	Vertical F _V	Longitudinal			Transversal		
	left	right		F _{HX}	Arm. _{HX}	M _y	F _{HY}	Arm. _{HY}	M _x
	(kN)	(kN)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Case1a:	254.5	1168.0	1422.5			1096.2			697.0
Case1b:	683.2	816.3	1499.5			159.7			734.7
Case2a:	229.0	861.8	1090.9			759.3			534.5
Case2b:	350.7	1051.2	1401.9			840.6			687.0
Case3a:	254.5	949.1	1203.6			833.5			589.8
Case3b:	632.4	612.0	1244.4			-24.5			609.7

Item	Vertical	Longitudinal			Transversal		
		F _{HX}	Arm _{HX}	M _y	F _{HY}	Arm _{HY}	M _x
	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Live Load							
Total at bottom of Column	1078.4	0.0	0.0	646.6	0.0	0.0	3763.7
Total at bottom of pilecap	1078.4	0.0	0.0	646.6	0.0	0.0	3763.7

Pedestrian Load									
Item	Reaction	Reaction	Vertical	Longitudinal			Transversal		
	left	right	F _v	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	0.0	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 (m)	Vertical F_V (kN)	Longitudinal			Transversal		
			F_{HX} (kN)	Arm. $_{HX}$ (m)	M_y (kN·m)	F_{HY} (kN)	Arm. $_{HY}$ (m)	M_x (kN·m)
Centrifugal force	1.80							
Total at bottom of Column						0.0	15.981	0.0
Total at bottom of pilecap						0.0	17.481	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
$Br1 = 25\% * (\text{design truck}) * n * m$	Br1	207.19 kN
$Br2 = 5\% * (\text{design truck} + 9.3 * L_{\text{bridge}}) * n * m$	Br2	119.70 kN
$Br = \max(Br1, Br2)$	Br	207.19 kN

Item	From surface (m)	Vertical F_V (kN)	Longitudinal			Transversal		
			F_{HX} (kN)	Arm. $_{HX}$ (m)	M_y (kN·m)	F_{HY} (kN)	Arm. $_{HY}$ (m)	M_x (kN·m)
Take 50 %								
Braking force	1.80							
Total at bottom of Column			103.6	15.981	1655.5			
Total at bottom of pilecap			103.6	17.481	1810.9			

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	Δt_{\max}	20.0 deg
Minus temperature amplitude	Δt_{\min}	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: $F_{hx} = (3.E.I) \cdot \Delta s / H^3$	F_{hx}	0 kN
	E	27691 MPa
	I	1.126 m4

Item	From surface (m)	Vertical F_V (kN)	Longitudinal			Transversal		
			F_{HX} (kN)	Arm. $_{HX}$ (m)	M_y (kN·m)	F_{HY} (kN)	Arm. $_{HY}$ (m)	M_x (kN·m)
Total at bottom of Column			0.3	14.02	4.5			
Total at bottom of pilecap			0.3	15.52	5.0			

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	F_{hx}	0 kN

Item	From surface (m)	Vertical F_V (kN)	Longitudinal			Transversal		
			F_{HX} (kN)	Arm. $_{HX}$ (m)	M_y (kN·m)	F_{HY} (kN)	Arm. $_{HY}$ (m)	M_x (kN·m)
Total at bottom of Column			0.3	14.02	4.2			
Total at bottom of pilecap			0.3	15.52	4.7			

7.Wind on Structure

Wind loads data		$P_D = 0.0006 V^2 C_d \geq 1.8$ (kN/m ²)	
Average elevation of deck girder upper ground or water plane level	Hele_g	11.8	m
Correct coefficient for wind zone and elevation of pier	S	1.10	
Design wind speed $V = S \cdot V_b$	V	58.2	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	2.77	kN/m ²

Item	Exposed height (m)	Vertical F _V (kN)	Longitudinal			Transversal		
			F _{HX} (kN)	Arm. _{HX} (m)	M _y (kN·m)	F _{HY} (kN)	Arm. _{HY} (m)	M _x (kN·m)
Superstructure	3.03		44.1	15.4	678.4	176.4	15.4	2713.7
Total at bottom of Column			44.1		678.4	176.4		2713.7
Superstructure	3.03		44.1	16.9	744.6	176.4	16.9	2978.3
Total at bottom of pilecap			44.1		744.6	176.4		2978.3

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 transversal 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 (kN)	Longitudinal			Transversal		
		F _{HX} (kN)	Arm. _{HX} (m)	M _y (kN·m)	F _{HY} (kN)	Arm. _{HY} (m)	M _x (kN·m)
Superstructure		15.8	17.8	280.7	31.6	17.8	561.4
Total at bottom of Column		15.8		280.7	31.6		561.4
Superstructure		15.8	19.3	304.4	31.6	19.3	608.8
Total at bottom of pilecap		15.8		304.4	31.6		608.8

9.Earth Quake

N/A

Earth Quake data			
Acceleration coefficient	A	0.0580	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	3	other
Response Modification Factor			
Column		1.0	
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 (kN)	Longitudinal			Transversal		
		F _{HX} (kN)	Arm. _{HX} (m)	M _y (kN·m)	F _{HY} (kN)	Arm. _{HY} (m)	M _x (kN·m)
Total at bottom of Column							

Total at bottom of pilecap					
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	DA NANG - QUANG NGAI EXPRESS WAY PROJECT ORB22 Right BRIDGE DETAIL DESIGN PIER P1 RIGHT DESIGN	Item.	Eng.	Date	Sign.
		Design	-		
		Check	-		
		Revise	-		

b.LOAD COMBINATIONS

I. LOAD COMBINATIONS

Loads at Bottom of Column						
Loads	Sign	F _v (kN)	Longitudinal		Transvesal	
			F _{HX} (kN)	My (kN•m)	F _{HY} (kN)	Mx (kN•m)
Superstructure Loads						
1.Stage1 - Dead Load	DC	3269		0		
2.Stage2 - Pave.+Parapet+Railing+Mis.	DW	998		0		
3.Live Load	LL	1078		647		3764
4.Pedestrian	LL	0		0		0
5.Centrifugal force	CE				0	0
6.Braking force	BR		104	1656		
7.Uniform temperature	TU		0	5		
8.Creep and Shrinkage	CR&SH		0	4		
9.Wind pressure on superstructure	WS		44	678	176	2714
10.Wind pressure on vehicles	WL		16	281	32	561
11.Earthequake						
a - Longitudinal direction	EQ		0	0		
b - Transverse direction	EQ				0	0
Substructure Loads						
1.Pier selfweight	DC	3884				
2.Soil on pile cap	EV	0				
3.Bouyancy on pier						
a - Maximum water level	WA	-569				
b - Minimum water level	WA	-250				
c - Average annual water level	WA	-464				
4.Stream pressure						
a - Maximum water level	WA		0	0	9	55
b - Minimum water level	WA		0	0	0	0
c - Average annual water level	WA		0	0	4	13
5.Wind pressure						
a - Maximum water level	WS		127	1463	26	298
b - Minimum water level	WS		191	1830	42	392
c - Average annual water level	WS		148	1616	31	337
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		Transvesal	
			F _{HX} (kN)	My (kN•m)	F _{HY} (kN)	M _x (kN•m)
Superstructure Loads						
1.Stage1 - Dead Load	DC	3269		0		
2.Stage2 - Pave.+Parapet+Railing+Mis.	DW	998		0		
3.Live Load	LL	1078		647		3764
4.Pedestrian	LL	0		0		0
5.Centrifugal force	CE				0	0
6.Braking force	BR		104	1811		
7.Uniform temperature	TU		0	5		
8.Creep and Shrinkage	CR&SH		0	5		
9.Wind pressure on superstructure	WS		44	745	176	2978
10.Wind pressure on vehicles	WL		16	304	32	609
11.Earthequake						
a - Longitudinal direction	EQ		0	0		
b - Transverse direction	EQ				0	0
Substructure Loads						
1.Pier selfweight	DC	5501				
2.Soil on pile cap	EV	2587				
3.Bouyancy on pier						
a - Maximum water level	WA	-1217				
b - Minimum water level	WA	-897				
c - Average annual water level	WA	-1112				
4.Stream pressure						
a - Maximum water level	WA		0	0	9	62
b - Minimum water level	WA		0	0	0	1
c - Average annual water level	WA		0	0	4	19
5.Wind pressure						
a - Maximum water level	WS		127	1654	26	337
b - Minimum water level	WS		191	2117	42	455
c - Average annual water level	WS		148	1837	31	384
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	8.00	0.50	0.50	0.50	0.50	5.00
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.Earthquake							
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	-	-	-	-	-	-

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.Earthquake							
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	Ext2b	Ext2c	Ext2d	Printed:7/20/2013
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		15	16	17	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.Earthquake							
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

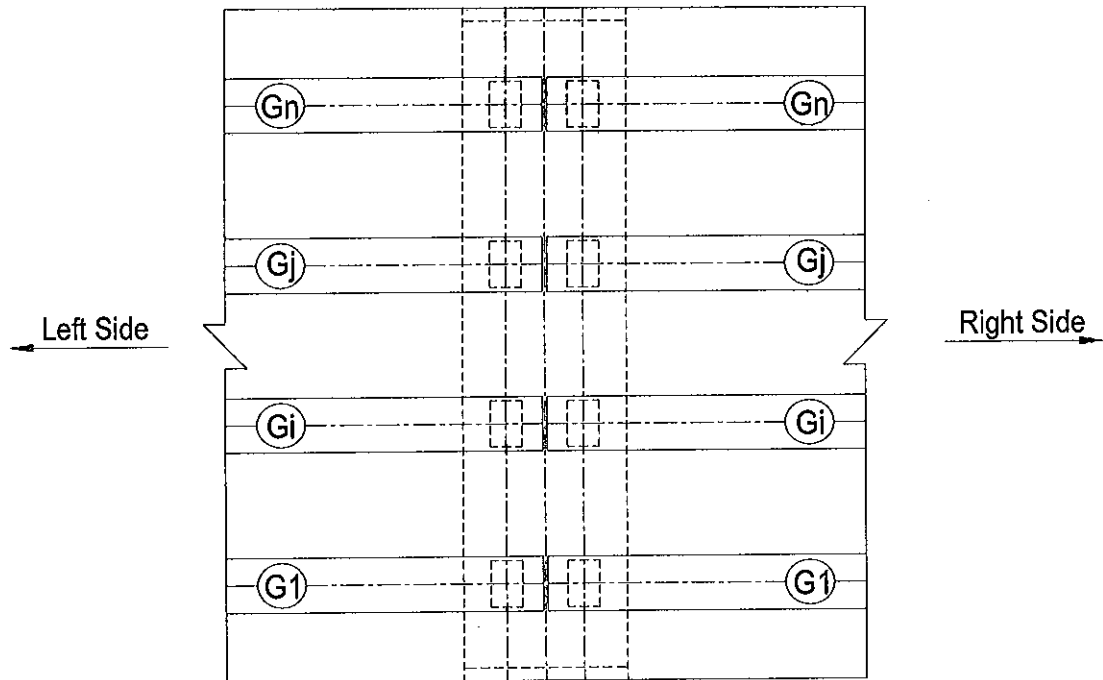
No	Combinations	3	Sign	F _v (kN)	Longitudinal		Transvesal	
					F _{HX} (kN)	My (kN•m)	F _{HY} (kN)	Mx (kN•m)
1	Strength 1a		Str1a	12076	184	4065	0	6587
2	Strength 1b		Str1b	8405	182	4033	9	6641
3	Strength 2a		Str2a	10188	329	3517	306	4348
4	Strength 2b		Str2b	6517	240	3003	293	4271
5	Strength 3a		Str3a	11644	250	4396	119	6885
6	Strength 3b		Str3b	7973	226	4269	122	6902
7	Service 1		Ser1	8980	190	3344	97	5257
8	Extreme 1a	EQL	Ext1a	10513	52	1151	4	1895
9	Extreme 1b	EQL	Ext1b	7162	52	1151	4	1895
10	Extreme 1c	EQT	Ext1c	10513	52	1151	4	1895
11	Extreme 1d	EQT	Ext1d	7162	52	1151	4	1895
12	Extreme 2a	CTL	Ext2a	10513	52	1151	4	1895
13	Extreme 2b	CTL	Ext2b	7162	52	1151	4	1895
14	Extreme 2c	CTT	Ext2c	10513	52	1151	4	1895
15	Extreme 2d	CTT	Ext2d	7162	52	1151	4	1895

Load Combinations at Bottom of PileCap

No	Combinations		Sign	F _v (kN)	Longitudinal		Transvesal	
					F _{HX} (kN)	My (kN•m)	F _{HY} (kN)	Mx (kN•m)
1	Strength 1a		Str1a	16942	184	4341	0	6587
2	Strength 1b		Str1b	11541	182	4305	9	6649
3	Strength 2a		Str2a	15054	329	4011	306	4807
4	Strength 2b		Str2b	9653	240	3362	293	4704
5	Strength 3a		Str3a	16510	250	4771	119	7064
6	Strength 3b		Str3b	11109	226	4607	122	7078
7	Service 1		Ser1	12536	190	3630	97	5403
8	Extreme 1a	EQL	Ext1a	15379	52	1229	4	1901
9	Extreme 1b	EQL	Ext1b	10298	52	1229	4	1901
10	Extreme 1c	EQT	Ext1c	15379	52	1229	4	1901
11	Extreme 1d	EQT	Ext1d	10298	52	1229	4	1901
12	Extreme 2a	CTL	Ext2a	15379	52	1229	4	1901
13	Extreme 2b	CTL	Ext2b	12471	52	1229	4	1901
14	Extreme 2c	CTT	Ext2c	15379	52	1229	4	1901
15	Extreme 2d	CTT	Ext2d	10298	52	1229	4	1901

	DA NANG - QUANG NGAI EXPRESS WAY PROJECT ORB22 Right BRIDGE DETAIL DESIGN PIER P1 RIGHT DESIGN	Item.	Eng.	Date	Sign.
		Design	-		
		Check	-		
		Revise	-		

c. PIER CAP ANALYSIS



1. DEAD LOAD

Stage1 - Dead load

Load: <i>Girders</i>	<i>G1</i>	<i>G2</i>	<i>G3</i>	<i>G4</i>	<i>G5</i>	<i>G6</i>	<i>G7</i>
Left side Reactions (kN)	147.7	147.7	147.7	147.7	147.7	147.7	
Right side Reactions (kN)	147.7	147.7	147.7	147.7	147.7	147.7	
Total reactions both side (kN)	295.5	295.5	295.5	295.5	295.5	295.5	0.0

Load: <i>Diaphragm</i>	<i>G1</i>	<i>G2</i>	<i>G3</i>	<i>G4</i>	<i>G5</i>	<i>G6</i>	<i>G7</i>
Left side Reactions (kN)	18.2	18.2	18.2	18.2	18.2	18.2	
Right side Reactions (kN)	18.2	18.2	18.2	18.2	18.2	18.2	
Total reactions both side (kN)	36.3	36.3	36.3	36.3	36.3	36.3	0.0

Load: <i>Precast plank</i>	<i>G1</i>	<i>G2</i>	<i>G3</i>	<i>G4</i>	<i>G5</i>	<i>G6</i>	<i>G7</i>
Left side Reactions (kN)	29.9	29.9	29.9	29.9	29.9	29.9	
Right side Reactions (kN)	29.9	29.9	29.9	29.9	29.9	29.9	
Total reactions both side (kN)	59.9	59.9	59.9	59.9	59.9	59.9	0.0

Load: <i>DeckSlab</i>	<i>G1</i>	<i>G2</i>	<i>G3</i>	<i>G4</i>	<i>G5</i>	<i>G6</i>	<i>G7</i>
Left side Reactions (kN)	131.1	131.1	131.1	131.1	131.1	131.1	
Right side Reactions (kN)	131.1	131.1	131.1	131.1	131.1	131.1	
Total reactions both side (kN)	262.2	262.2	262.2	262.2	262.2	262.2	

Load: <i>Stage1 (DC)</i>	<i>G1</i>	<i>G2</i>	<i>G3</i>	<i>G4</i>	<i>G5</i>	<i>G6</i>	<i>G7</i>
Total reactions both side (kN)	653.9	653.9	653.9	653.9	653.9	653.9	

Stage2 - Dead load

Load: <i>Pavement</i>	<i>G1</i>	<i>G2</i>	<i>G3</i>	<i>G4</i>	<i>G5</i>	<i>G6</i>	<i>G7</i>
Left side Reactions (kN)	45.8	45.8	45.8	45.8	45.8	45.8	

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Right side Reactions (kN)	45.8	45.8	45.8	45.8	45.8	45.8
Total reactions both side (kN)	91.6	91.6	91.6	91.6	91.6	91.6

Load: <i>Parapet+railing</i>	<i>G1</i>	<i>G2</i>	<i>G3</i>	<i>G4</i>	<i>G5</i>	<i>G6</i>	<i>G7</i>
Left side Reactions (kN)	124.4	0.0	0.0	0.0	0.0	124.4	
Right side Reactions (kN)	124.4	0.0	0.0	0.0	0.0	124.4	
Total reactions both side (kN)	248.9	0.0	0.0	0.0	0.0	248.9	

Load: <i>Lighting post.+m/s</i>	<i>G1</i>	<i>G2</i>	<i>G3</i>	<i>G4</i>	<i>G5</i>	<i>G6</i>	<i>G7</i>
Left side Reactions (kN)	4.2	4.2	4.2	4.2	4.2	4.2	
Right side Reactions (kN)	4.2	4.2	4.2	4.2	4.2	4.2	
Total reactions both side (kN)	8.4	8.4	8.4	8.4	8.4	8.4	

Load: <i>Stage2 (DC)</i>	<i>G1</i>	<i>G2</i>	<i>G3</i>	<i>G4</i>	<i>G5</i>	<i>G6</i>	<i>G7</i>
Total reactions both side (kN)	348.9	100.0	100.0	100.0	100.0	348.9	

2. LIVE LOAD

Live load reactions is calculated by level rule

Summary of Live load Reactions:

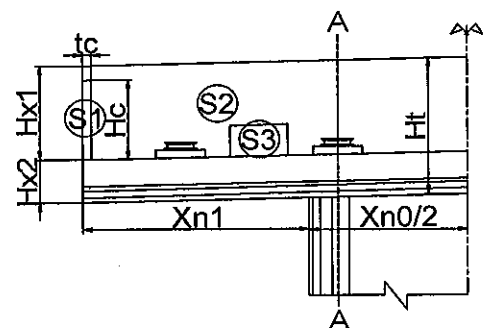
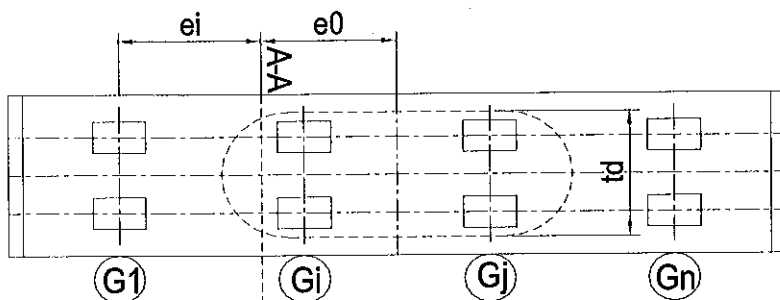
1 Loaded Lane		n =	1.0	m =	1.20	1+IM =	1.25
Live load (kN)	Factor	<i>G1</i>	<i>G2</i>	<i>G3</i>	<i>G4</i>	<i>G5</i>	<i>G6</i>
Truck + Lane load		450.4	241.6	0.0	0.0	0.0	0.0
Tandem + Lane load		373.8	200.5	0.0	0.0	0.0	0.0
0.9*(2Truck+Lane)	0.9	421.1	225.9	0.0	0.0	0.0	0.0

$$\text{Reaction} = [(1+IM)*\text{Vehicle} + \text{Laneload}] * n * m$$

2 Loaded Lanes		n =	2.0	m =	1.00	1+IM =	1.25
Live load (kN)	Factor	<i>G1</i>	<i>G2</i>	<i>G3</i>	<i>G4</i>	<i>G5</i>	<i>G6</i>
Truck + Lane load		375.4	416.4	353.0	8.7	8.7	0.0
Tandem + Lane load		311.5	345.5	292.9	7.2	7.2	0.0
0.9*(2Truck+Lane)	0.9	351.0	389.3	330.0	8.1	8.1	0.0

$$\text{Reaction} = [(1+IM)*\text{Vehicle} + \text{Laneload}] * n * m$$

3. PIER CAP DESIGN



Cantilever section (A-A)

Distance from centerline of pier to section A-A

$$e0 = 2.28 \text{ m}$$

Item	<i>G1</i>	<i>G2</i>	<i>G3</i>	<i>G4</i>	<i>G5</i>	<i>G6</i>	<i>G7</i>
Bearing is taken into account	1	1	0	0	0	0	0
Distance from bearing to section A-A							
Left side ei	3.89	1.42	-	-	-	-	-
Right side ei	3.89	1.42	-	-	-	-	-

Dead load of substructure

Notation	Dimensions	Value(m)	Notation	Dimensions	Value(m)
Hx1	Haunch 1 height	1.48	Hc	Curtain wall height	1.20
Hx2	Haunch 2 height	0.80	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 - e0	0.47
yc	Concrete unit weight (kN/m3)	24.50	e0	Dist. from CL of pier to sec.A-A	2.28

Item	Volume	Section G1			Volume	Section A-A		
		F _v	Arm. _{Fv}	M _x		F _v	Arm. _{Fv}	M _x
Component	(m3)	(kN)	(m)	(kN•m)	(m3)	(kN)	(m)	(kN•m)
S1	0.40	9.7	-0.01	-0.1	0.40	9.7	3.88	37.7
S2	6.35	155.6	0.61	94.3	28.26	692.3	1.98	1369.6
S3					0.68	16.6	1.56	26.0
Total at section G1		165.3		94.3				
Total at section A-A						718.6		1433.2

Load components at section bearing G1

Item	Vertical	Torsion Moment			Bending Moment		
		F _{Hx}	Arm. _{Hx}	M _y	F _{Hy}	Arm. _{Hy}	M _x
		(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Pier Selfweight	165.3						94.3
DC stage1	654					0.0	0.0
DW stage2	348.9					0.0	0.0
Live Load	421.1			238.0		0.0	0.0
Pedestrian	0.0					0.0	0.0

Load components at section A-A

Item	Vertical	Torsion Moment			Bending Moment		
		F _{Hx}	Arm. _{Hx}	M _y	F _{Hy}	Arm. _{Hy}	M _x
		(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Pier Selfweight	718.6						1433.2
DC stage1	1307.7					3.89	3470.9
DW stage2	448.9					3.89	1498.6
Live Load	647.0			238.0		3.89	1958.4
Pedestrian	0.0					3.89	0.0

Load factors and Load combinations

Load Combinations

Item	Ser1	Str1a	Section	Comb.	Vertical	Bending	Torsion
					F _v	M _x	M _y
Pier Selfweight	1.00	1.25	G1	Ser1	1589	94	238
DC stage1	1.00	1.25		Str1a	2284	118	416
DW stage2	1.00	1.50					
Live Load	1.00	1.75	A-A	Ser1	3122	8361	238
Pedestrian	1.00	1.75		Str1a	4339	11805	416

	DA NANG - QUANG NGAI EXPRESS WAY PROJECT ORB22 Right BRIDGE DETAIL DESIGN PIER P1 RIGHT DESIGN	Item.	Eng.	Date	Sign.
		Design	-		
		Check	-		
		Revise	-		

d.COLUMN DESIGN

I. COLUMN DATA

1.Load Combinations at Bottom of Pier Column

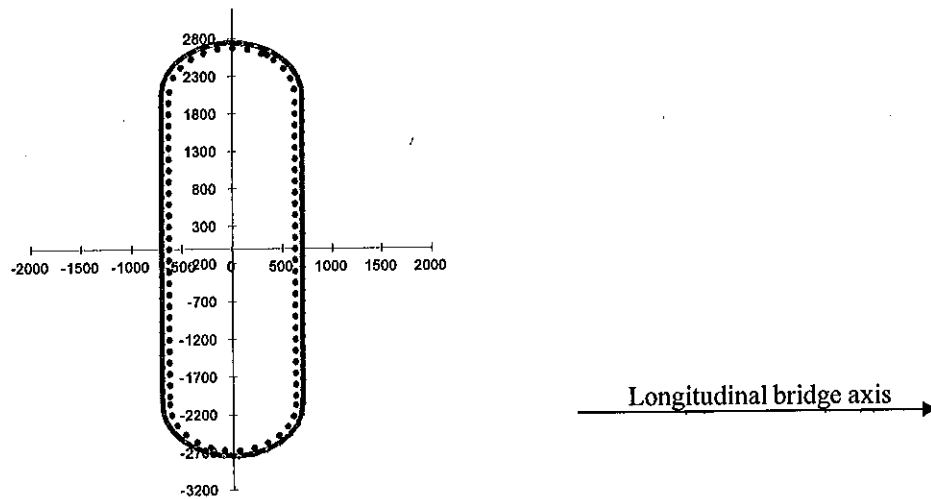
1.Load Combinations at Bottom of Pier Column								
No	Combinations		Sign	F _v (kN)	Longitudinal		Transvesal	
					F _{Hx} (kN)	My (kN•m)	F _{Ily} (kN)	Mx (kN•m)
1	Strength 1a		Str1a	12076	184	4065	0	6587
2	Strength 1b		Str1b	8405	182	4033	9	6641
3	Strength 2a		Str2a	10188	329	3517	306	4348
4	Strength 2b		Str2b	6517	240	3003	293	4271
5	Strength 3a		Str3a	11644	250	4396	119	6885
6	Strength 3b		Str3b	7973	226	4269	122	6902
7	Service 1		Ser1	8980	190	3344	97	5257
8	Extreme 1a	EQL	Ext1a	10513	52	1151	4	1895
9	Extreme 1b	EQL	Ext1b	7162	52	1151	4	1895
10	Extreme 1c	EQT	Ext1c	10513	52	1151	4	1895
11	Extreme 1d	EQT	Ext1d	7162	52	1151	4	1895
12	Extreme 2a	CTL	Ext2a	10513	52	1151	4	1895
13	Extreme 2b	CTL	Ext2b	7162	52	1151	4	1895
14	Extreme 2c	CTT	Ext2c	10513	52	1151	4	1895
15	Extreme 2d	CTT	Ext2d	7162	52	1151	4	1895

2. Pier Column Material

Normal concrete			
Compressive strength at 28 days age	f _c	30	MPa
Concrete elastic modulus	E _c	27691	MPa
Reinforcement TCVN1651-2008; CBV-400			
Yield strength	f _y	400	MPa
Reinforcement elastic modulus	E _s	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	m ²
Moment inertia	Ix	16.498	m ⁴
	Iy	1.126	m ⁴
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	13.87	m
Slenderness ratio: if $K.Lu / r > 22$ than considered	$Kt.Lu/rx$	19.3	no
	$Kl.Lu/ry$	74.1	yes
Moment inertia of longitudinal reinforcements	Is	0	m ⁴
Ratio of max factored Per. load moment to max factored total load moment	β_d	0	

P - Δ analysis

**Longitudinal Direction

P - Δ moment dertermination procedure:

Initial Determining displacement for gross cross section
 Displacement for cracked section
 Moment P-Δ
 Added lateral force
 Step: i st Determining displacement for gross cross section
 Displacement for cracked section
 Moment P-Δ
 Added lateral force

$$\begin{aligned}\Delta x_g &= F_x \cdot H^3 / (3.E.I_g) \\ \Delta x_{cr} &= F_{cr} \cdot \Delta x_g \\ M_{P-\Delta} &= \Delta x_{cr} \cdot P \\ \Delta F_x &= M_{P-\Delta} / H \\ \Delta x_{g\ i} &= (F_x + \Delta F_{x\ i-1}) \cdot H^3 / (3.E.I_g) \\ \Delta x_{cr\ i} &= F_{cr} \cdot \Delta x_{g\ i} \\ M_{P-\Delta\ i} &= \Delta x_{cr\ i} \cdot P \\ \Delta F_{x\ i} &= M_{P-\Delta\ i} / H\end{aligned}$$

Combination	Fv (kN)	My (kNm)	Initial					
			Fx (kN)	Δxg (m)	Fcr (kN)	Δxcr (m)	M P-Δ (kNm)	ΔFx (kN)
Strength 1a	12076	4065	293	0.008	2.5	0.021	252	18.2
Strength 1b	8405	4033	291	0.008	2.5	0.021	174	12.6
Strength 2a	10188	3517	254	0.007	2.5	0.018	184	13.3
Strength 2b	6517	3003	217	0.006	2.5	0.015	101	7.3
Strength 3a	11644	4396	317	0.009	2.5	0.023	263	19.0
Strength 3b	7973	4269	308	0.009	2.5	0.022	175	12.6
Service 1	8980	3344	241	0.007	2.5	0.017	154	11.1
Extreme 1a	10513	1151	83	0.002	2.5	0.006	62	4.5
Extreme 1b	7162	1151	8	0.000	2.5	0.001	4	5.0
Extreme 1c	10513	1151	83	0.002	2.5	0.006	62	4.5
Extreme 1d	7162	1151	83	0.002	2.5	0.006	42	3.1
Extreme 2a	10513	1151	83	0.002	2.5	0.006	62	4.5
Extreme 2b	7162	1151	83	0.002	2.5	0.006	42	3.1
Extreme 2c	10513	1151	83	0.002	2.5	0.006	62	4.5
Extreme 2d	7162	1151	83	0.002	2.5	0.006	42	3.1

Combination	Fv (kN)	My (kNm)	1st Trial					
			Fx (kN)	Δxg (m)	Fcr (kN)	Δxcr (m)	M P-Δ (kNm)	ΔFx (kN)
Strength 1a	12076	4065	311	0.009	2.5	0.022	268	19.3
Strength 1b	8405	4033	303	0.009	2.5	0.022	182	13.1
Strength 2a	10188	3517	267	0.008	2.5	0.019	194	14.0
Strength 2b	6517	3003	224	0.006	2.5	0.016	104	7.5
Strength 3a	11644	4396	336	0.010	2.5	0.024	279	20.1
Strength 3b	7973	4269	320	0.009	2.5	0.023	182	13.1
Service 1	8980	3344	252	0.007	2.5	0.018	162	11.6
Extreme 1a	10513	1151	87	0.002	2.5	0.006	66	4.7
Extreme 1b	7162	1151	13	0.000	2.5	0.001	7	0.5
Extreme 1c	10513	1151	87	0.002	2.5	0.006	66	4.7
Extreme 1d	7162	1151	86	0.002	2.5	0.006	44	3.2
Extreme 2a	10513	1151	87	0.002	2.5	0.006	66	4.7
Extreme 2b	7162	1151	86	0.002	2.5	0.006	44	3.2
Extreme 2c	10513	1151	87	0.002	2.5	0.006	66	4.7
Extreme 2d	7162	1151	86	0.002	2.5	0.006	44	3.2

Combination	Fv (kN)	My (kNm)	2nd Trial					
			Fx (kN)	Δxg (m)	Fcr (kN)	Δxcr (m)	M P-Δ (kNm)	ΔFx (kN)
Strength 1a	12076	4065	312	0.009	2.5	0.022	269	19.4
Strength 1b	8405	4033	304	0.009	2.5	0.022	182	13.1
Strength 2a	10188	3517	268	0.008	2.5	0.019	194	14.0
Strength 2b	6517	3003	224	0.006	2.5	0.016	104	7.5
Strength 3a	11644	4396	337	0.010	2.5	0.024	280	20.2
Strength 3b	7973	4269	321	0.009	2.5	0.023	182	13.2
Service 1	8980	3344	253	0.007	2.5	0.018	162	11.7
Extreme 1a	10513	1151	88	0.003	2.5	0.006	66	4.7
Extreme 1b	7162	1151	8	0.000	2.5	0.001	4	0.3
Extreme 1c	10513	1151	88	0.003	2.5	0.006	66	4.7
Extreme 1d	7162	1151	86	0.002	2.5	0.006	44	3.2
Extreme 2a	10513	1151	88	0.003	2.5	0.006	66	4.7
Extreme 2b	7162	1151	86	0.002	2.5	0.006	44	3.2
Extreme 2c	10513	1151	88	0.003	2.5	0.006	66	4.7
Extreme 2d	7162	1151	86	0.002	2.5	0.006	44	3.2

****Transverse Direction**

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Combination	Fv (kN)	Mx (kNm)	Initial					
			Fx (kN)	Δxg (m)	Fcr (kN)	Δxcr (m)	M P-Δ (kNm)	ΔFx (kN)
Strength 1a	12076	6587	475	0.001	2.5	0.002	28	2.0
Strength 1b	8405	6641	479	0.001	2.5	0.002	20	1.4
Strength 2a	10188	4348	313	0.001	2.5	0.002	16	1.1
Strength 2b	6517	4271	308	0.001	2.5	0.001	10	0.7
Strength 3a	11644	6885	496	0.001	2.5	0.002	28	2.0
Strength 3b	7973	6902	498	0.001	2.5	0.002	19	1.4
Service 1	8980	5257	379	0.001	2.5	0.002	17	1.2
Extreme 1a	10513	1895	137	0.000	2.5	0.001	7	0.5
Extreme 1b	7162	1895	137	0.000	2.5	0.001	5	0.3
Extreme 1c	10513	1895	137	0.000	2.5	0.001	7	0.5
Extreme 1d	7162	1895	137	0.000	2.5	0.001	5	0.3
Extreme 2a	10513	1895	137	0.000	2.5	0.001	7	0.5
Extreme 2b	7162	1895	137	0.000	2.5	0.001	5	0.3
Extreme 2c	10513	1895	137	0.000	2.5	0.001	7	0.5
Extreme 2d	7162	1895	137	0.000	2.5	0.001	5	0.3

Combination	Fv (kN)	Mx (kNm)	1st Trial					
			Fx (kN)	Δxg (m)	Fcr (kN)	Δxcr (m)	M P-Δ (kNm)	ΔFx (kN)
Strength 1a	12076	6587	477	0.001	2.5	0.002	28	2.0
Strength 1b	8405	6641	480	0.001	2.5	0.002	20	1.4
Strength 2a	10188	4348	315	0.001	2.5	0.002	16	1.1
Strength 2b	6517	4271	309	0.001	2.5	0.002	10	0.7
Strength 3a	11644	6885	498	0.001	2.5	0.002	28	2.0
Strength 3b	7973	6902	499	0.001	2.5	0.002	19	1.4
Service 1	8980	5257	380	0.001	2.5	0.002	17	1.2
Extreme 1a	10513	1895	137	0.000	2.5	0.001	7	0.5
Extreme 1b	7162	1895	137	0.000	2.5	0.001	5	0.3
Extreme 1c	10513	1895	137	0.000	2.5	0.001	7	0.5
Extreme 1d	7162	1895	137	0.000	2.5	0.001	5	0.3
Extreme 2a	10513	1895	137	0.000	2.5	0.001	7	0.5
Extreme 2b	7162	1895	137	0.000	2.5	0.001	5	0.3
Extreme 2c	10513	1895	137	0.000	2.5	0.001	7	0.5
Extreme 2d	7162	1895	137	0.000	2.5	0.001	5	0.3

Combination	Fv (kN)	Mx (kNm)	2nd Trial					
			Fx (kN)	Δxg (m)	Fcr (kN)	Δxcr (m)	M P-Δ (kNm)	ΔFx (kN)
Strength 1a	12076	6587	477	0.001	2.5	0.002	28	2.0
Strength 1b	8405	6641	480	0.001	2.5	0.002	20	1.4
Strength 2a	10188	4348	315	0.001	2.5	0.002	16	1.1
Strength 2b	6517	4271	309	0.001	2.5	0.002	10	0.7
Strength 3a	11644	6885	498	0.001	2.5	0.002	28	2.0
Strength 3b	7973	6902	499	0.001	2.5	0.002	19	1.4
Service 1	8980	5257	380	0.001	2.5	0.002	17	1.2
Extreme 1a	10513	1895	137	0.000	2.5	0.001	7	0.5
Extreme 1b	7162	1895	137	0.000	2.5	0.001	5	0.3
Extreme 1c	10513	1895	137	0.000	2.5	0.001	7	0.5
Extreme 1d	7162	1895	137	0.000	2.5	0.001	5	0.3
Extreme 2a	10513	1895	137	0.000	2.5	0.001	7	0.5
Extreme 2b	7162	1895	137	0.000	2.5	0.001	5	0.3
Extreme 2c	10513	1895	137	0.000	2.5	0.001	7	0.5
Extreme 2d	7162	1895	137	0.000	2.5	0.001	5	0.3

****Load Combinations at bottom of column considering Slenderness Effect**

Printed: 7/20/2013

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	12076	6587	28	6615	4065	269	4334
Strength 1b	8405	6641	20	6661	4033	182	4215
Strength 2a	10188	4348	16	4363	3517	194	3711
Strength 2b	6517	4271	10	4281	3003	104	3107
Strength 3a	11644	6885	28	6913	4396	280	4676
Strength 3b	7973	6902	19	6921	4269	182	4451
Service 1	8980	5257	17	5274	3344	162	3506
Extreme 1a	10513	1895	7	1902	1151	66	1217
Extreme 1b	7162	1895	5	1900	1151	4	1155
Extreme 1c	10513	1895	7	1902	1151	66	1217
Extreme 1d	7162	1895	5	1900	1151	44	1195
Extreme 2a	10513	1895	7	1902	1151	66	1217
Extreme 2b	7162	1895	5	1900	1151	44	1195
Extreme 2c	10513	1895	7	1902	1151	66	1217
Extreme 2d	7162	1895	5	1900	1151	44	1195

II. PIER COLUMN DESIGN

S.5.7.4.2

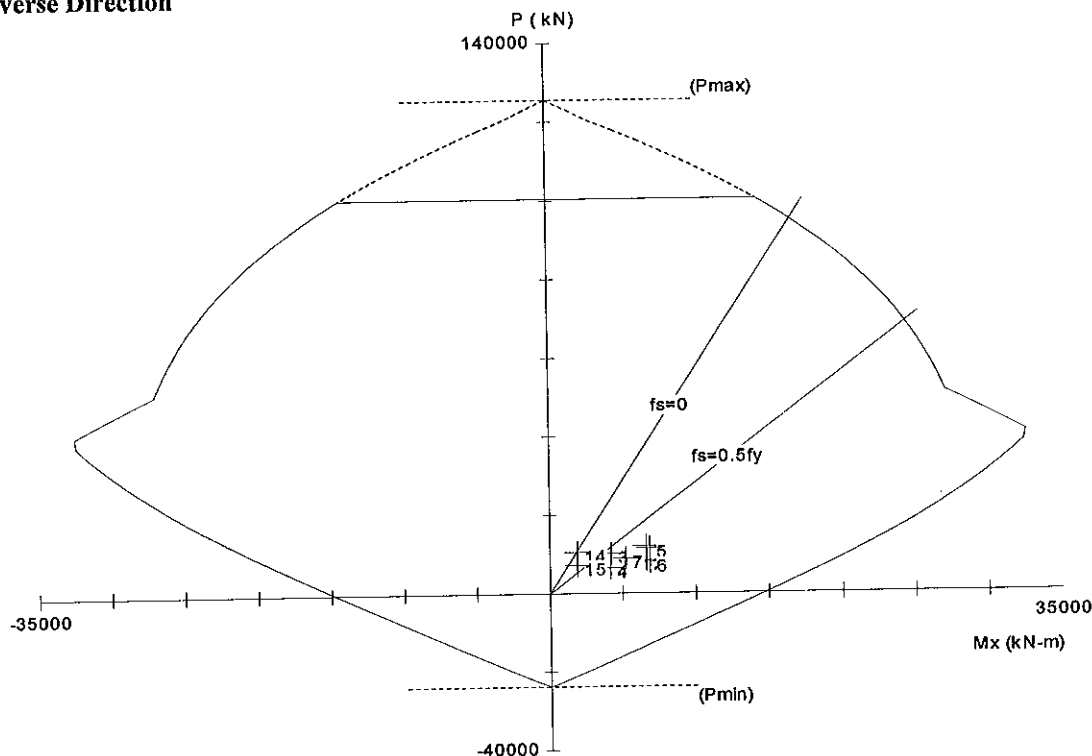
1. Limit of Reinforcement

Minimum area of longitudinal reinforcement in column						
$A_s \cdot f_y / (A_g \cdot f_c) \geq 0.135$				$A_s \geq$	0.074	m2
$A_s / A_g \geq 0.01$				$A_s \geq$	0.073	m2
Maximum area of longitudinal reinforcement in column						
$A_s / A_g \leq 0.08$				$A_s \leq$	0.582	m2
Trial Rebars:				A_s	0.066	m2
1 layers	x 82	= 82 bars	D32 @150	As1	0.066	m2
1 layers	x 82	= 0 bars	D25 @150	As2	0.000	m2

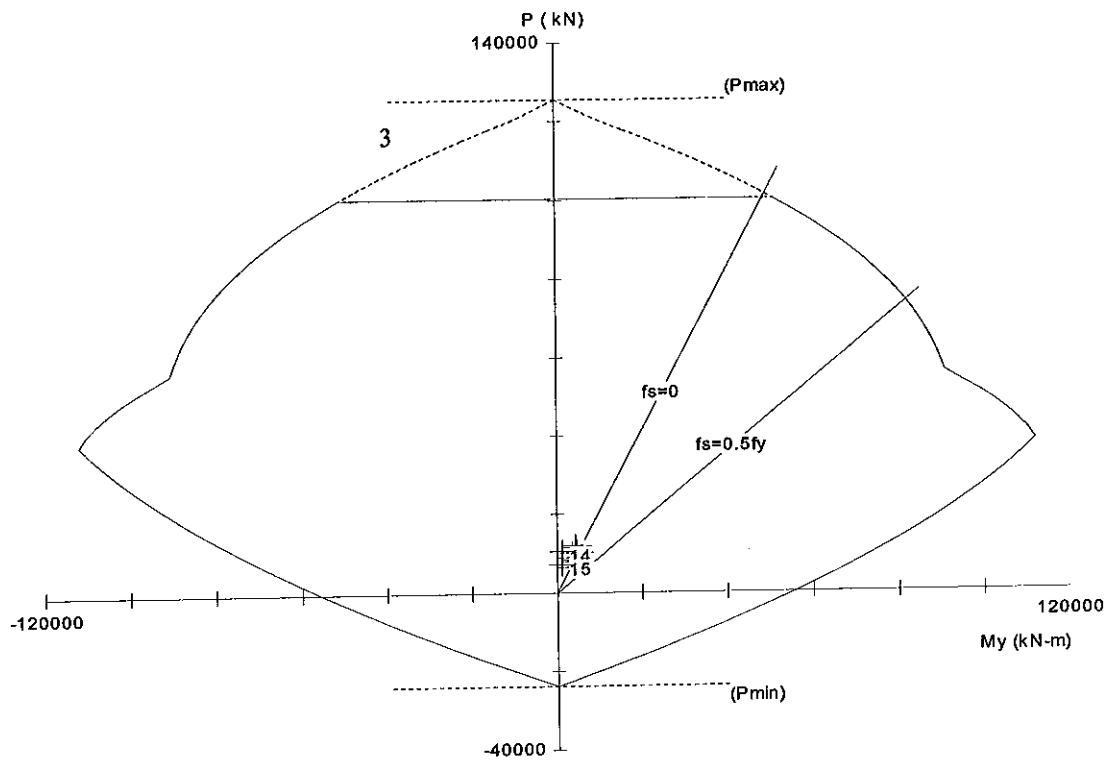
2. Iteration diagram M-P

Using Pca-Column software

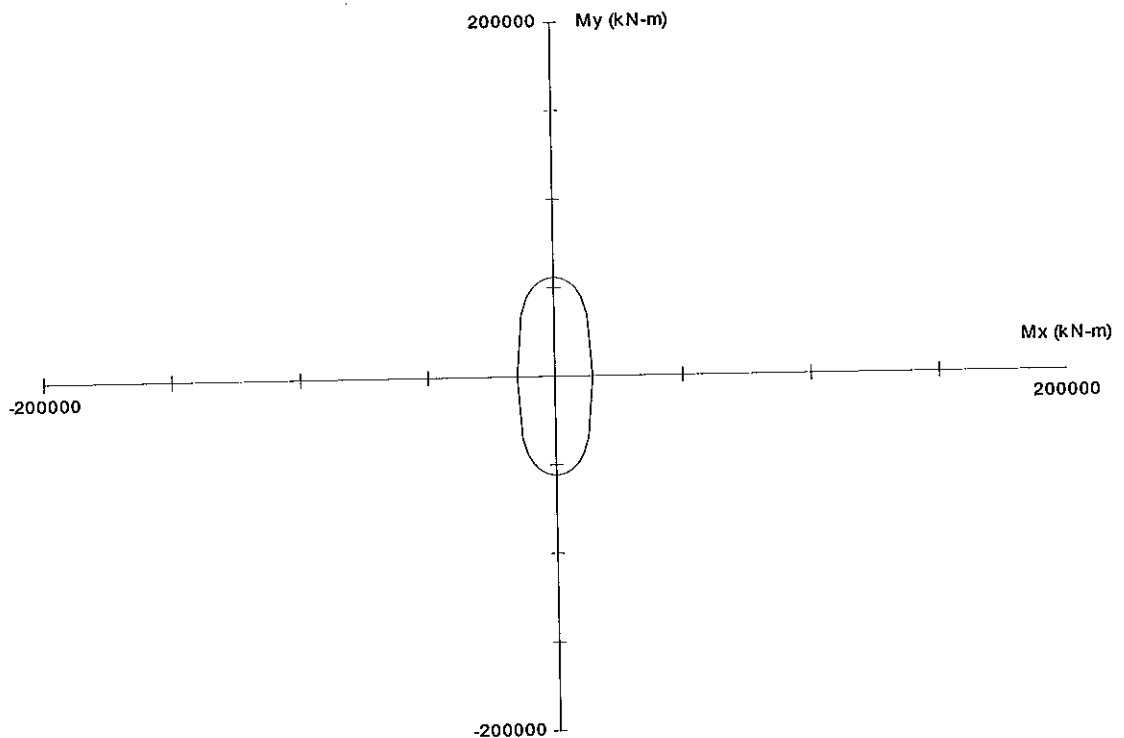
**In Transverse Direction



**In Longitudinal Direction



****In Both Direction**



P = 0 kN		S.5.7.4.6, S.5.10.6.3, S5.10.11.4.1d - e		
3. Column Ties		Sz	1	
Bridge is in seismic zone		Ac	6.719	m2
Area of concrete core measured out-to-out of ties		Dtie	16	mm2
Tie diameter		As-tr	0.0002	m2
Cross section area of 1 tie		s	150	mm
Spacing of hoops		Ltie	19.90	m
Length of reinforcement tie in 1 hoop				
Ratio of ties reinf. in one hoop/ volume of conc. core one pitch spacing		ps	0.0040	
ps = As-tr . Ltie / (Ac * spacing)				S.5.7.4.6
Ratio of spiral reinf. To total volume of concrete core shall satisfy		Req1	0.0028	N/A
ps >= 0.45 . (Ag/Ac - 1). fc/ fy = Req1				
				S.5.10.11.3

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Transverse Reinforcement for Confinement at Plastic Hinges		S.5.10.11.4.1.d	
For a circular column	"1:applied", "2:Not applied"	2	
$\rho_s \geq 0.12 \cdot f_c / f_y = \text{Req2}$	Req2	0.0090	N/A
For a rectangular column			
Rectangular hoop reinforcement shall satisfy			
Either $A_{sh} \geq 0.30 \cdot s \cdot h_c \cdot f_c / f_y$ [$A_g / A_c - 1$] = Req1			
or $A_{sh} \geq 0.12 \cdot s \cdot h_c \cdot f_c / f_y = \text{Req2}$			
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	1.93	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: $L_e = \max(L4, L5)$	Le	2.75	m

4. Shear Design

Direction		Long.- X	Trans.-Y	Unit
Shear resistance factors	ϕ_v	1.0	1.0	
Factored shear force in longitudinal	V_u	52	4	kN
Required shear capacity $V_n = V_u / \phi_v$	V_n	52	4	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	1.40	5.20	m
	width	5.20	1.40	m
$d = h - \text{cover} - d_{1x}$	d	1.31	5.11	m
$d_v = \max(0.72 \cdot h; 0.9 \cdot d)$	d_v	1.18	4.60	m
$V_c = \min(0.083 \cdot \beta \cdot \sqrt{f_c} \cdot b_v \cdot d_v, 0.25 \cdot f_c \cdot b_v \cdot d_v)$	V_c	5591	5858	kN
Difference between required shear capacity and the capacity provided by concrete is the minimum required capacity for shear reinforcements				
$V_s = V_n - V_c$	V_s	0	0	kN
In this case $V_c > V_n$ so shear reinforcement is no need				

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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
$V_s = A_v \cdot f_y \cdot d_v \cdot (\cot \theta + \cot \alpha) \cdot \sin \alpha / s$	$s \leq$	-	-	m
Stirrup spacing used	s	0.10	0.10	m
Check minimum shear reinforcement requirement		OK	OK	
$A_v \geq 0.083 \cdot \sqrt{f_c} \cdot b_v \cdot s / f_y = \text{Req}$	Req	0.0006	0.0002	m2
Check maximum shear reinforcement spacing requirement		OK	OK	
$F = 0.1 \cdot f_c \cdot b_v \cdot d_v$	F	18447	19329	kN
If $V_u < 0.1 \cdot f_c \cdot b_v \cdot d_v$ then $S_{\max} = 0.8 \cdot d_v \leq 600 \text{mm}$				
If $V_u > 0.1 \cdot f_c \cdot b_v \cdot d_v$ then $S_{\max} = 0.4 \cdot d_v \leq 300 \text{mm}$				
	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			
$V_n = c \cdot A_{cv} + \mu \cdot A_{vf} \cdot f_y$	Vn	31368	kN
$V_n \leq 0.2 \cdot f_c \cdot A_{cv}$	$V_n \leq$	43676	kN
$V_n \leq 5.5 \cdot A_{cv} \cdot (1 \text{MPa})$	$V_n \leq$	40037	kN
Normal 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	52	kN
	Conclude		OK

5. Control of cracking by distribution of reinforcement

Tensile stress in rebars should be satisfied equation: $f_s \leq f_{sa} = Z / [(d_c \cdot A)^{1/3}]$ and $f_s \leq 0.6 \cdot f_y$				
Direction		Long.-X	Trans.-Y	Unit
Existing condition for structure	1,2 or 3	1	1	
Crack width parameter	Z	30000	30000	N/mm
Flexural moment	Ms	3506	5274	kNm
Axial thrust at service limit state	Ns	8980	8980	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.				
$A = 2 \cdot d_c \cdot b / N$	A	0.0068	0.0064	m2
	f _{sa}	429	439	MPa
	0.6f _y	240	240	MPa
Min (f _{sa} , 0.6f _y) = f _{s1}	f _{s1}	240	240	MPa
$e = M_s / N_s + d - h/2$	e	1.00	3.10	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	
$i = 1 / (1 - j \cdot d/e)$	i	3.90	3.90	
Stress in rebars: $f_s = (M_s + N_s(d - h/2)) / (A_s \cdot j \cdot i \cdot d)$	f _s	34	93	MPa
	Conclude	OK	OK	
Maximum width of crack: $a_n = 0.076 \cdot \beta \cdot f_s \cdot (d_c \cdot A)^{1/3}$	a _n	0.030	0.080	mm
Where	β	0.167	0.167	

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PACKAGE: A2

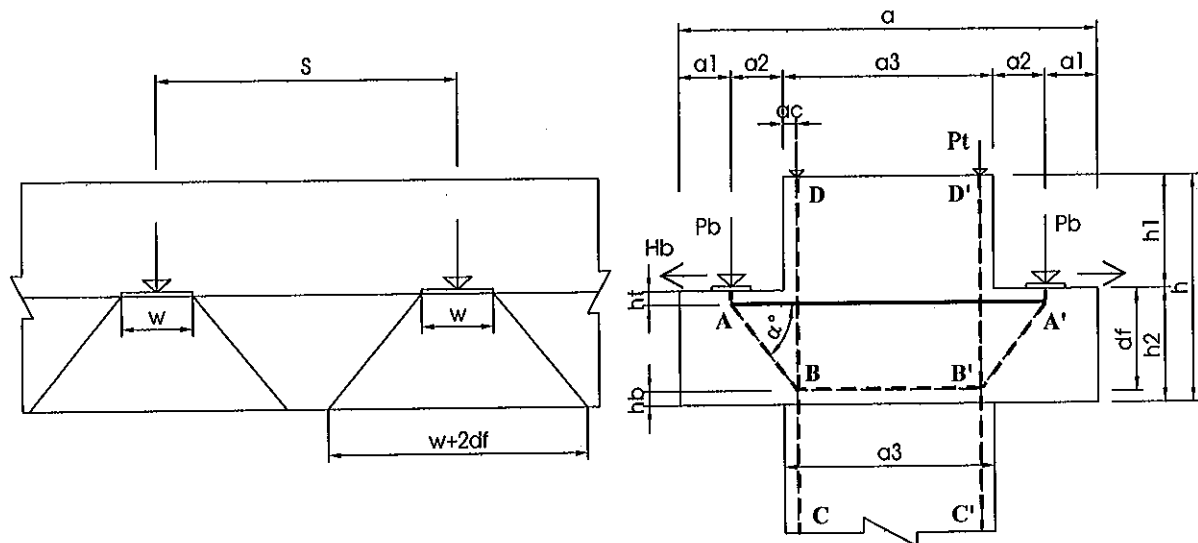
BRIDGE
ORB22

CALCULATION SHEETS
HEAD STOCK - Rebars

	DA NANG - QUANG NGAI EXPRESS WAY PROJECT ORB22 Right BRIDGE DETAIL DESIGN HEAD STOCK	Item.	Eng.	Date.	Sign.
		Design			
		Check			
		Revise			

HEAD STOCK DESIGN

1. Head Stock dimensions



Item	Sign	Unit	Value
Bearings spacing	S	m	2.55
Bearing pad dimension	W	m	0.40
Bearing pad dimension	L	m	0.50
	W+2df	m	1.87
Reactions - strength limit state	Pb	kN	1265.8
	Hb	kN	135.2
	Pt	kN	23.1
Head stock dimension	a1	m	0.55
	a2	m	0.45
	a3	m	1.60
	a	m	3.60
	h1	m	1.30
	h2	m	0.80
	h	m	2.10
Distance from top of ledge to compression rebars	df	m	0.74
Distance from concrete face to rebars	ht	m	0.06
	hb	m	0.064
	ac	m	0.113
Angle distribution	α	deg	50.13
Materials			
Reinforcement concrete unit weight	gc	kN/m3	24.50
Compressive strength of concrete	fc	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)	1192	-1649	-1057	-1289	-23

(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	$W_{in} =$	1.872 m
Effective width of tension tie for outer bearing	$W_{out} =$	1.872 m
Required capacity of tension tie		
$Tr = T / \phi_t$	$Tr =$	1325 kN
Tie consists	12 bars @ 150 D25	$At =$ 0.0059 m ²

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:

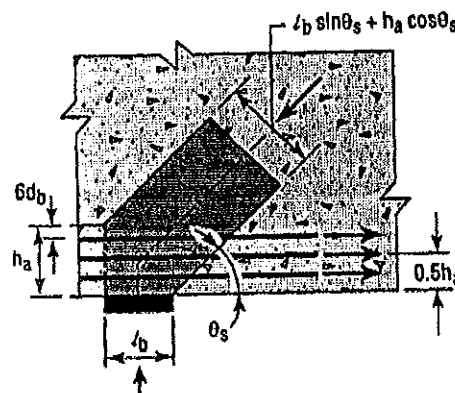
$$P_n = f_y \cdot A_t \quad P_n = 2357 \text{ kN} \quad \text{Ok}$$

b. Compression Strut - AB

S.5.6.3.3

Required capacity of compression strut		
$Cr = C / \phi_c$	$Cr =$	2356 kN

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



Width of bearing pad	$L =$	0.50 m
$W_s = L \cdot \sin \alpha$	$W_s =$	0.38 m
Effective width	$W =$	1.87 m
Area of compression strut	$Acs = W_s \cdot W =$	0.72 m ²

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	$\sigma = T / A_t$	$\sigma =$	202 MPa
Strain in tie	$\epsilon_s = \sigma / E_s$	$\epsilon_s =$	0.00101 mm/mm
Concrete strain component			
$\epsilon_l = \epsilon_s + (\epsilon_s + 0.002) \cdot \cot^2 \alpha$	$\epsilon_l =$	0.00311	
Limiting compressive stress f_{cu} , shall be taken as			
$f_{cu} = f_c / (0.8 + 170 \cdot \epsilon_l) \leq 0.85 \cdot f_c$	$f_{cu} =$	22.57 MPa	
Capacity of compression strut	$P_n = f_{cu} \cdot Acs =$	16215 kN	Ok

c. Check node region

S.5.6.3.5

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

$$\text{Limit stress} = 0.75 \cdot \phi \cdot f_c$$

$$\phi = 0.7$$

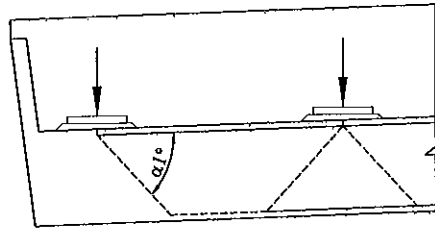
$$\text{Limit stress} = 15.8 \text{ MPa}$$

Node section on concrete strut side governs

$$\text{Compressive stress} = C / A_{cs}$$

$$\text{Compressive stress} = 2.3 \text{ MPa} \quad \text{Ok}$$

4. Longitudinal rebars



Distributed angle

$$\alpha_1 = 45.0 \text{ deg}$$

Effective width of tension tie

$$W_1 = 1.000 \text{ m}$$

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

$$T_r = T / \phi_t$$

$$T_r = 563 \text{ kN}$$

Tie consists

6 bars @ 150 D25

$$A_t = 0.0029 \text{ m}^2$$

$$P_n = f_y \cdot A_t$$

$$P_n = 1178 \text{ kN} \quad \text{Ok}$$

5. Design for Punching Shear

S.5.13.2.5.4

Nominal punching shear resistance, V_n (N), shall be taken as

- At interior pads:

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

$$V_n = 3809.2 \text{ kN}$$

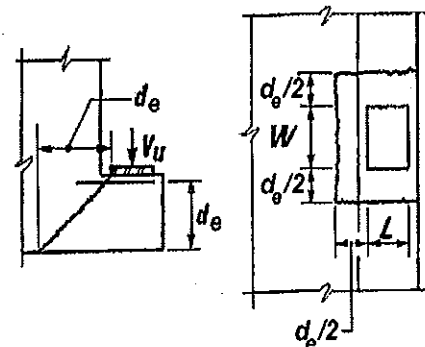
$$V_r = 3428.28 \text{ kN} \quad \text{Ok}$$

- At exterior pads:

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

$$V_n = 2169.59 \text{ kN}$$

$$V_r = 1952.63 \text{ kN} \quad \text{Ok}$$



6. Design of Hanger Reinforcement

S.5.13.2.5.5

Nominal punching shear resistance, V_n (N), shall be taken as

- For the strength limit state:

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

$$V_n = 25622.4 \text{ kN}$$

$$V_r = 23060.2 \text{ kN} \quad \text{Ok}$$

12 bars @ 150 D20

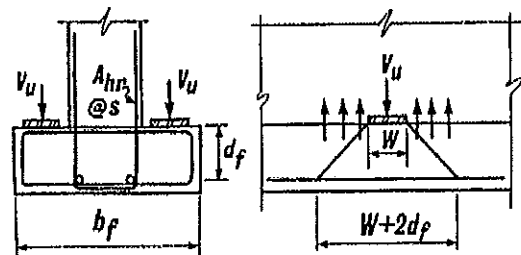
$$A_{hr} = 3768 \text{ mm}^2$$

- For inverted T-beams

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

$$V_n = 18812.3 \text{ kN}$$

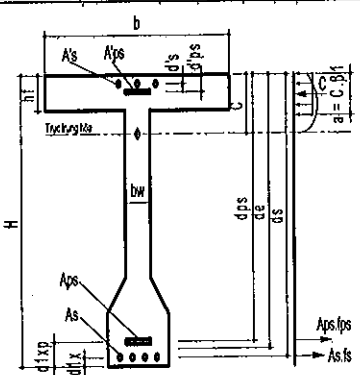
$$V_r = 16931 \text{ kN} \quad \text{Ok}$$



DA NANG - QUANG NGAI EXPRESS WAY PROJECT				Item.	Eng.	Date.	Sign.
ORB22 BRIDGE				Design			
DETAIL DESIGN				Check			
PIER CAP-PIR - CHECK STRENGTH				Revise			

22TCN272-05; AASHTO LRFD 2nd - 1998

F. REINFORCEMENT CHECKING - PIER CAP

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	Section - CANTILEVER				
			A-A	A-A	G1	G1	
INTERNAL FORCES AT SECTION							
	Combination		Strength	Service	Strength	Service	
Qu	Shear	kN	4339	3122	2284	1589	
Mu	Flexural Moment	kNm	11805	8361	118	94	
Nu	Axial load	kN	0	0	0	0	
Tu	Torsional Moment	kNm	0	17	0	284	
FLEXURAL MOMENT CHECKING							
H	Section height	m	2.724	2.724	2.724	2.724	
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	2.591	2.591	2.591	2.591	
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	2.724	2.724	2.724	2.724	
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	2.695	2.695	2.695	2.695	
Amc	Section area	m2	4.358	4.358	4.358	4.358	
	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	
		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	
		Area	m2	0.00000	0.00000	0.00000	0.00000
As	Tension Reinforcement	Number	bars	26	26	39	
		Diameter	mm	32	32	32	
		Area	m2	0.02083	0.02083	0.02083	0.03124
A's	Compression Reinforcement	Number	bars	0	0	0	
		Diameter	mm	28	28	28	32
		Area	m2	0.00000	0.00000	0.00000	0.00000
A'c	Shear reinforcement	Number	bars	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	0.366	
	For T section behavior	m	0.244	0.244	0.244	0.366	
	For rectangular section behavior	m	0.244	0.244	0.244	0.366	
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	1813	1813	1813	1789	
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	0.306
de	Corresp. effective depth from extreme comp. fiber to centroid of tensile force in the tensile reinf.	m	2.591	2.591	2.591	2.591
Mn	Nominal resistance	kNm	20734	20734	20734	30463
Mr	Factored resistance	kNm	18660	20734	18660	30463
Mu	Flexural moment	kNm	11805	8361	118	94
(5.7.3.2)	Flexural moment Checking		OK	OK	OK	OK
	Limits for reinforcement					
c/de	Maximum reinforcement		0.09	0.09	0.09	0.14
	Maximum reinforcement Checking	≤ 0.42	OK	OK	OK	OK
r min	Minimum reinforcement		0.48%	0.48%	0.48%	0.72%
	Minimum reinforcement Checking for RC	0.23%	OK	OK	OK	OK
1.2*Mcrr	Cracking moment	kNm	4500	4500	4500	4734
(5.7.3.3.2)	Checking $M_r \geq \min(1.2M_{cr}, 1.33M_u)$		OK	OK	OK	OK
(5.7.3.4)	Control of cracking by distr. of reinf for RC member- Check?		No	Yes	No	Yes
	Existing condition for structure	1,2 or 3	1	1	1	1
de	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	m ²	0.008	0.008	0.008	0.005
fsa	Value	Mpa	369	369	369	423
0.6*fy		Mpa	240	240	240	240
	Tensil stress in reinf Min(fs,0.6fy)	Mpa	240	240	240	240
x	Dist. From compression fiber to centroid	m	-	0.602	-	0.716
J.d	Arm	m	-	2.39	-	2.352
Icr	Moment of inertia of the cracked section	m ⁴	-	0.693	-	0.965
fs	Tensile stress in reinforcement $f_s = M_{sls} / (A_s * J.d)$	Mpa	-	168	-	1
	Checking for control cracking $f_s < f_{sa}$		N.a	OK	N.a	OK
(5.10.8.2)	Shrinkage and temperature Reinforcement (side distribution)					
Areq	Area of required reinf	m ²	0.00094	0.00094	0.00094	0.00094
	Distribution on sides 6 D16	m ²	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		1.9	2.1	3.3	3.7
θ	Angle of inclination of diagonal compressive	degree	41.87	38.59	28.62	27.86
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	2.489	2.489	2.489	2.438
	($d_e - a/2$)	m	2.489	2.489	2.489	2.438
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	m ²	0.0013	0.0013	0.0013	0.0013
β	Assume		2.0	2.0	2.0	2.0
θ	Assume	degree	42.03	39.62	42.52	33.75
v	Shear stress in concrete	kN/m ²	1211	784	637	407
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116
ϵ_x	Strain in tensile reinforcement		1.72E-03	1.26E-03	3.10E-04	1.97E-04
	if $\epsilon_x < 0$, multiple with reduce factor		-	-	-	-
	Strain checking	$\leq 2.00E-3$	Ok	Ok	Ok	Ok
v/fc	Ratio of shear stress and fc		0.040	0.026	0.021	0.014
β	Final value		1.9	2.1	3.3	3.7
θ	Final value	degree	41.87	38.59	28.62	27.86
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	3350	3774	5890	6568
Vs	Shear resistance provided by shear reinforcement	kN	9302	10445	15276	15449
Vp	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0
Vn1	$V_{n1} = V_c + V_s + V_p$	kN	12652	14220	21166	22017
Vn2	Vn2	kN	29867	29867	29867	29254
Vn	Nominal shear resistance $V_n = \min(V_{n1}, V_{n2})$	kN	12652	14220	21166	22017
Vr	Factored shear resistance	kN	11387	14220	19049	22017
Vu	Shear	kN	4339	3122	2284	1589
(5.8.2.7)	Shear checking		OK	OK	OK	OK
	Region requiring transverse reinf Checking		Need	Need	No need	No need
	Minimum shear reinf area	m ²	0.0003	0.0003	0.0003	0.0003
	Minimum shear reinforcement Checking		OK	OK	-	-
	$0.1 * f_c * b_v * d_v$	kN	11947	11947	11947	11702
	Smax	m	0.60	0.60	0.60	0.60
	Maximum spacing Smax		OK	OK	-	-

ϕ_t	Resistance factor for torsion	(5.5.4.2)	0.90	1.00	0.90	1.00
p_c	Outer perimeter of concrete section	m	6.000	6.000	6.000	5.400
A_{cp}	Area in outer perimeter of concrete section	m ²	4.358	4.358	4.358	4.358
f_{pc}	Comp. stress in concrete after all prestress losses at the centroid of section	Mpa	0.00	0.00	0.00	0.00
T_{cr}	Crack moment due to torsion	kNm	5688	5688	5688	6320
	$0.25 \cdot \phi \cdot T_{cr}$	kNm	1280	1422	1280	1580
T_u	Torsional moment by external forces	kNm	0	17	0	284
	Shear and Torsion combine if $T_u > 0.25 \phi T_{cr}$		No	No	No	No
A_o	Area enclosed by shear flow path	m ²	2.839	2.839	3.346	3.346
A_t	Area of one leg of closed transverse torsion reinforcement	m ²	0.0003	0.0003	0.0003	0.0003
p_h	Perimeter of the centerline of the closed transverse torsion reinf.	m	8.168	8.168	8.168	8.168
A_{oh}	Area enclosed by centerline of ext. closed transverse torsion reinf.	m ²	3.340	3.340	3.936	3.936
V_{u1}	Modified V_u in case shear and torsion combine	kN	4339	3122	2284	1619
v_1	Determine θ_t in case shear and torsion combine	kN/m ²	1211	784	637	441
θ	Assume	degree	40.61	37.29	27.00	32.05
ϵ_x	Strain in tensile reinforcement		1.72E-03	1.26E-03	3.10E-04	2.00E-04
	if $\epsilon_x < 0$, multiple with reduce factor		-	-	-	-
v_1/f_c	Ratio of shear stress and f_c		0.040	0.026	0.021	0.015
θ_t	Crack angle (S.5.8.3.4) updated modified V_u	degree	41.87	38.59	28.62	27.90
T_n	Nominal torsion resistance	kN	-	-	-	-
T_r	Factored torsional resistance	kN	-	-	-	-
(5.8.3.6.2)	Torsional checking		N.a	N.a	N.a	N.a

	DA NANG - QUANG NGAI EXPRESS WAY PROJECT ORB22 Right BRIDGE	Item.	Eng.	Date	Sign.
		Design	-		
	DETAIL DESIGN PIER P1 RIGHT DESIGN	Check	-		
		Revise	-		

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 $A = 0.0580$ g

Input data:

Bridge type	Simple PC I girder L=21m with link slab			
Span length	Left	=	21.05	Right = 21.05 m
Girder length between bearings	Left	=	21.05	Right = 21.05 m
Bridge width	B	=	12.74	m

Level Table(at center of pier)

Top of pier cap	ThL	12.870	m
Top of pier column	TcL	10.590	m
Bottom of upper pier column	BucL	10.590	m
Bottom of pier column	BcL	-1.000	m
Bottom of upper pilecap	BupL	-1.000	m
Bottom of pilecap	BpL	-2.500	m
Tip of pile	TpL		m
Skew angle	Ska	90.000	deg
Ground level	GL	2.980	m
Maximum water level (H1%)	HWL	6.970	m
Navigation water level (H5%)	NWL	4.000	m
Minimum water level	MWL	2.500	m
Average Annual water level	AWL	5.500	m
Local scour level (at water level H1%)	LsL	0.000	m

Material unit weight

Structural concrete	$\gamma_c =$	2500 kg/m ³	24.50	kN/m ³
Asphalt concrete	$\gamma_a =$	2250 kg/m ³	22.10	kN/m ³
Soil - ground	$\gamma_s =$	1800 kg/m ³	17.70	kN/m ³
Saturated soil	$\gamma_{ss} =$	800 kg/m ³	7.80	kN/m ³

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e. SHALLOW FOUNDATION CHECKING

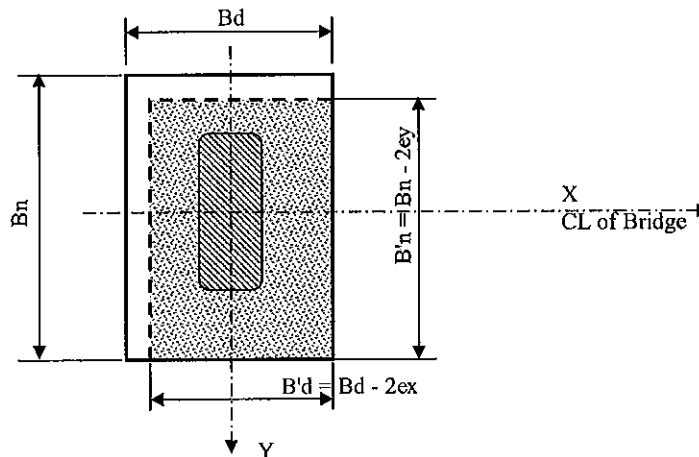
1.LOAD COMBINATIONS AT BOTTOM OF PILE CAP

Load Combinations at Bottom of PileCap

No	Combinations	Sign	F _V (kN)	Longitudinal		Transvesal	
				F _{HX} (kN)	My (kN•m)	F _{HY} (kN)	Mx (kN•m)
1	Strength 1a	Str1a	16942	184	4341	0	6587
2	Strength 1b	Str1b	11541	182	4305	9	6649
3	Strength 2a	Str2a	15054	329	4011	306	4807
4	Strength 2b	Str2b	9653	240	3362	293	4704
5	Strength 3a	Str3a	16510	250	4771	119	7064
6	Strength 3b	Str3b	11109	226	4607	122	7078
7	Service 1	Ser1	12536	190	3630	97	5403
8	Extreme 1a EQL	Ext1a	15379	52	1229	4	1901
9	Extreme 1b EQT	Ext1b	10298	52	1229	4	1901
10	Extreme 1c EQL	Ext1c	15379	52	1229	4	1901
11	Extreme 1d EQT	Ext1d	10298	52	1229	4	1901

2.CHECK BEARING RESISTANCE OF SHALLOW FOUNDATION

S.10.6.3



Pile cap properties

Longitudinal dimension	Bd	5.5	m
Transverse dimension	Bn	8.0	m
Pile cap area	A	44.0	m ²
Bending inertia moment	W _x = Bn ³ .Bd/6	W _x	58.7 m ³
	W _y = Bn.Bd ³ /6	W _y	40.3 m ³
Resistance factor for bearing capacity - SLS, shallow foundation	φ _b	0.60	
Resistance factor for bearing capacity - other limit state	φ _b	1.00	
Unaxial compression strength - saturated sample	361.8 kgf/cm ²	Q _u	5324 kN/m ²
Factored bearing resistance		Q _r	3195 kN/m ²

Compute the bearing capacity based on rock

The canadian foundation engineering manual 1992			
$Q_u = 3. [\sigma_c . K_{sp} . D]$			
$K_{sp} = [3 + C/B] / [10 . (1 + 300 . g/C)^{0.5}]$			
qu: ultimate end bearing pressure			
C: spacing of discontinuities	C=	0.25	m
B: pile width or (d pile diameter)	B=	5.5	m
g: aperture of discontinuities	g=	0.03	m
D = 1+0.4 . (L/d) <=3.4 - depth factor	D=	1.73	
L: length of the socket	L=	10	m
d: diameter of pile	d=	5.5	m
	K _{sp} =	0.050067	
σ _c : unconfined compressive strength	σ _c =	35.49258	Mpa
	Q _u =	9.2081206	Mpa
	Q _u =	9208	kN/m ²

FHWA manual 1988			
$q_u = \sigma_c . K_{sq}$			
$K_{sq} = [9 + 3 . C/B] / [10 . (1 + 300 . g/C)^{0.5}]$			
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 discontinuities	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
	K _{sq} =	0.1500	
σ _c : unconfined compressive strength	σ _c =	35.493	Mpa
	Q _u =	5.3244	Mpa
	Q _u =	5324	kN/m ²

Bearing Resistance - effective footing - AT BOTTOM OF FOOTING

Load Combination	Fv /A' (kPa)	Qr (kPa)	Check
Strength 1a	470	3195	OK
Strength 1b	355	3195	OK
Strength 2a	412	3195	OK
Strength 2b	286	3195	OK
Strength 3a	470	3195	OK
Strength 3b	354	3195	OK
Service I	357	3195	OK
Extreme 1a	371	5324	OK
Extreme 1b	256	5324	OK
Extreme 1c	371	5324	OK
Extreme 1d	256	5324	OK

3.CHECK SLIDING AT THE BASE OF FOOTING

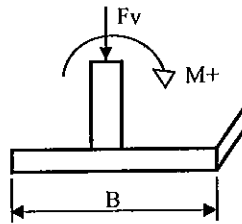
S.10.6.3.3

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 $Q_t = F_v \tan(\phi)$	Qt	
For concrete cast against soil: $\tan(\phi) = \tan(\phi_f)$	$\tan(\phi_f)$	0.55
Internal friction angle of soil	ϕ_f	29 deg
Resistance factor for shear resistance between soil and foundation	ϕ_t	0.80
Norminal passive resistance	Q_{ep}	0.00 kN
Resistance factor for passive resistance	ϕ_{ep}	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	16942	184	0	184	7513	OK
Strength 1b	0.80	11541	182	9	182	5118	OK
Strength 2a	0.80	15054	329	306	450	6676	OK
Strength 2b	0.80	9653	240	293	378	4281	OK
Strength 3a	0.80	16510	250	119	277	7321	OK
Strength 3b	0.80	11109	226	122	256	4926	OK
Service I	1.00	12536	190	97	214	6949	OK
Extreme 1a	1.00	15379	52	4	52	8525	OK
Extreme 1b	1.00	10298	52	4	52	5708	OK
Extreme 1c	1.00	15379	52	4	52	8525	OK
Extreme 1d	1.00	10298	52	4	52	5708	OK

4.CHECK OVERTURNING AT THE BASE OF FOOTING

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



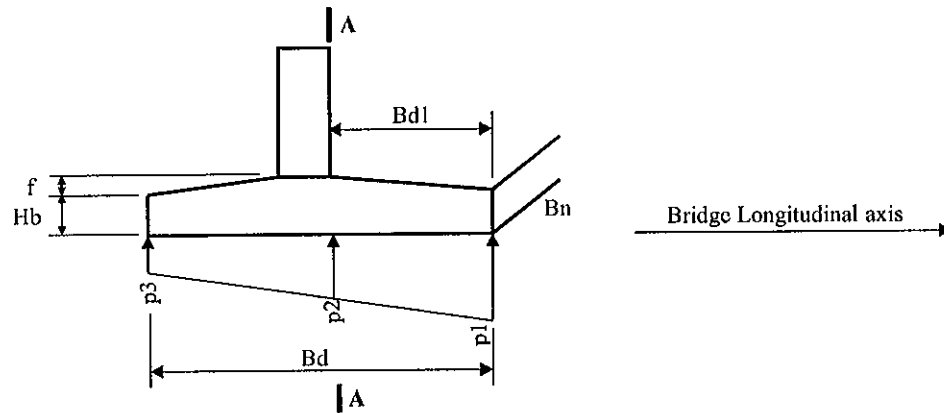
S.10.6.3.2.5

The location of the resultant of the reaction forces shall be within the middle three-fourths of the base.			
Longitudinal direction	$e_x = M_y / F_v \leq 3 \cdot B_d / 8 = [e_x]$	[ex]	2.06 m
Transverse direction	$e_y = M_x / F_v \leq 3 \cdot B_n / 8 = [e_y]$	[ey]	3.00 m
For seismic provision			S.10.6.5
According to 22TCN272-05: 1			
Longitudinal direction	$e_x = M_y / F_v \leq 0.6 B_d / 2 = [e_x]$	[ex]	1.65 m
Transverse direction	$e_y = M_x / F_v \leq 0.6 B_n / 2 = [e_y]$	[ey]	2.40 m
According to LRFD 2004: 2			
Where $\gamma_{EQ} = 0$			
Longitudinal direction	$e_x = M_y / F_v \leq 2/3 \cdot B_d / 2 = [e_x]$	[ex]	1.83 m
Transverse direction	$e_y = M_x / F_v \leq 2/3 \cdot B_n / 2 = [e_y]$	[ey]	2.67 m
Where $\gamma_{EQ} = 1$			
Longitudinal direction	$e_x = M_y / F_v \leq 8/10 \cdot B_d / 2 = [e_x]$	[ex]	2.20 m
Transverse direction	$e_y = M_x / F_v \leq 8/10 \cdot B_n / 2 = [e_y]$	[ey]	3.20 m
Where γ_{EQ} between 0 and 1, restrictions of the location can get by linear interpolation			
Choosing value for seismic: following LRFD 2004, with $\gamma_{EQ} = 0.5$			
Longitudinal direction		[ex]	2.02 m
Transverse direction		[ey]	2.93 m

Load Combination	Fv (kN)	Mx (kN·m)	My (kN·m)	Longitudinal		Transverse	
				ex (m)	Check $e_x < [e_x]$	ey (m)	Check $e_y < [e_y]$
Strength 1a	16942	6587	4341	0.26	OK	0.39	OK
Strength 1b	11541	6649	4305	0.37	OK	0.58	OK
Strength 2a	15054	4807	4011	0.27	OK	0.32	OK
Strength 2b	9653	4704	3362	0.35	OK	0.49	OK
Strength 3a	16510	7064	4771	0.29	OK	0.43	OK
Strength 3b	11109	7078	4607	0.41	OK	0.64	OK
Service I	12536	5403	3630	0.29	OK	0.43	OK
Extreme 1a	15379	1901	1229	0.08	OK	0.12	OK
Extreme 1b	10298	1901	1229	0.12	OK	0.18	OK
Extreme 1c	15379	1901	1229	0.08	OK	0.12	OK
Extreme 1d	10298	1901	1229	0.12	OK	0.18	OK

5.CHECK BENDING MOMENT AND SHEAR OF FOOTING

Transverse Section A-A



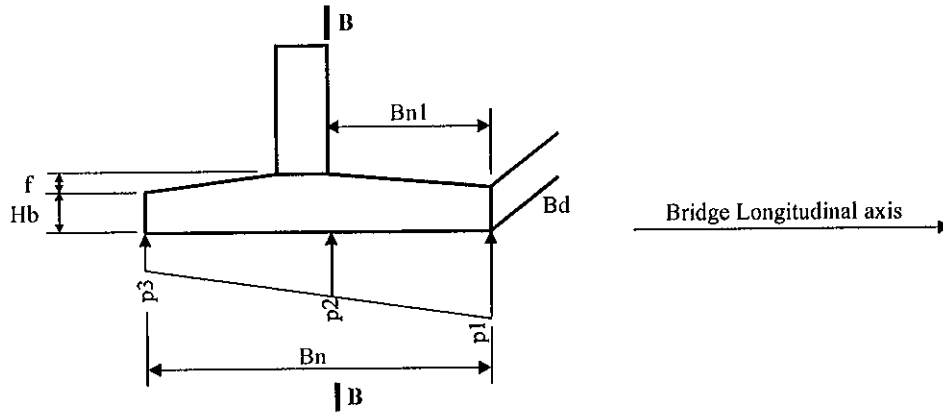
Transverse section (A-A)		
Footing dimensions		
Hb	2.00	m
f	0.00	m
Bd	5.50	m
Bd1	2.28	m
Bn	8.00	m
Hbb	0.00	m
Bdb	0.00	m
Bdb1	0.00	m
Bnb	0.00	m
Considering of bouyancy "1:yes" "0:no"	1	
Internal force at section A-A due to selfweight of pilecap		
Shear force	Qself	537 kN
Bending moment	Mself	613 kN
$p1 = Fv / A + My / Wy$ $p3 = Fv / A - My / Wy$		

Earth on toe shall be neglected.

Load Combination	Load factor	Qself (kN)	Mselft (kN•m)	p1 (kPa)	p3 (kPa)	p2 (kPa)	MA (kN•m)	QA (kN)
Strength 1a	1.25	671	766	605	277	469	10905	9138
Strength 1b	0.90	483	551	369	156	280	6528	5448
Strength 2a	1.25	671	766	442	243	359	7869	6641
Strength 2b	0.90	483	551	303	136	234	5281	4415
Strength 3a	1.25	671	766	494	257	395	8844	7447
Strength 3b	0.90	483	551	367	138	272	6437	5349
Service 1	1.00	537	613	375	195	300	6686	5629
Extreme 1a	1.25	671	766	380	319	355	6983	6039
Extreme 1b	0.90	483	551	264	204	239	4789	4117
Extreme 1c	1.25	671	766	380	319	355	6983	6039
Extreme 1d	0.90	483	551	264	204	239	4789	4117

Maximum internal force at section A-A (transverse section)		
Bending moment	MA	10905 kNm
Shear force	QA	9138 kN

Longitudinal Section B-B



Longitudinal section (B-B)		
Footing dimensions		
Hb	2.00	m
Bc	5.50	m
Bn	8.00	m
Bn1	1.49	m
Bd	5.50	m
Bn2	2.35	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	1360 kN
Bending moment	Mself	5727 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)	MA (kN•m)	QA (kN)
Strength 1a	1.25	671	1701	497	273	456	1234	3223
Strength 1b	0.90	483	1224	376	149	334	971	2415
Strength 2a	1.25	671	1701	424	260	394	812	2670
Strength 2b	0.90	483	1224	300	139	270	534	1843
Strength 3a	1.25	671	1701	496	255	451	1218	3197
Strength 3b	0.90	483	1224	373	132	328	950	2383
Service 1	1.00	537	1360	377	193	343	859	2404
Extreme 1a	1.25	671	1701	382	317	370	594	2401
Extreme 1b	0.90	483	1224	266	202	254	369	1645
Extreme 1c	1.25	671	1701	382	317	370	594	2401
Extreme 1d	0.90	483	1224	266	202	254	369	1645
								-671

Maximum internal force at section A-A (transverse section)		
Bending moment	MA	1234 kNm
Shear force	QA	3223 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 $d_v/2$ 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 d_v , we may take $d_v = 0.9 \cdot d_e$				
$d_e = H - \text{cover} - d_x1$		d_e	1.81	m
		d_v	1.63	m
Perimeter of two-way shear				
$b0 = (td+tn) \cdot 2 + 4 \cdot d_v$		b0	20.31	m
Compressive strength of pilecap concrete		f_c	30	Mpa
Yield strength of rebar		f_y	400	Mpa
Section with transverse reinforcement				
Nominal shear resistance shall be taken as				
$V_n = V_c + V_s \leq 0.504 \cdot \sqrt{f_c} \cdot b0 \cdot d_v = V_a$				
$V_c = 0.166 \cdot \sqrt{f_c} \cdot b0 \cdot d_v$				
$V_s = A_v \cdot f_y \cdot d_v / s$				
Shear resistance of concrete		V_c	30068	kN
Assumed stirrup diameter		D_s	16	mm
Number of stirrup legs / cross section		ns	0	
Shear legs area		A_v	0.0000	m2
Stirrup spacing used		s	600	mm
Shear resistance of reinforcement		V_s	0	kN
		V_a	91292	kN
		V_n	30068	kN
Maximum reaction at bottom of column		V_u	13526	kN
Resistance factor for shear		ϕ_v	0.9	
Factored shear resistance		$\phi_v \cdot V_n$	27062	kN
Punching shear check			OK	

6.SETTLEMENT OF FOOTING ON ROCK

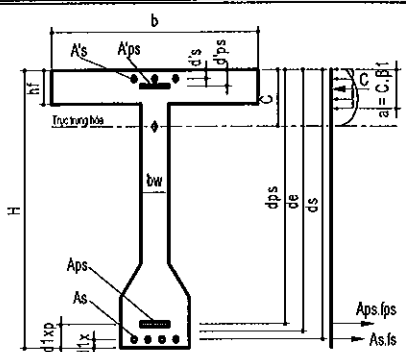
For Rectangular footing			
Elastic settlement $p = q_0 (1 - \nu^2) * B \cdot I_p / E_m$			
$I_p = (L/B)^{0.5} / \beta_z$			
Footing dimensions	Bd = B'	4.92	m
	Bn = L'	7.14	m
	L'/B'	1.45	
Ridity "1: Flexible" "2: rigid"		2	
Factor to account for footing shape and rigidity		β_z	1.07
Influence coefficient to account for rigidity and dimension of footing		I_p	1.12
Poisson's ratio		ν	0.29
Rock mass modulus $E_m = 1000 \cdot 10^{[(RMR-10)/40]}$		E_m	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 I combination		q_0	0.357 Mpa
Elastic settlement	Can be ignored	p	2.02 mm

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		Design			
		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
E _c	Modulus of Elasticity	Mpa	27691
f _r	Modulus of Rupture	Mpa	3.5
γ _c	Unit weight of concrete	kN/m3	24.5
PRESTRESSING STEEL			
f _{pu}	Tensile strength of prestressing steel	Mpa	1860
f _{py}	Yield strength of prestressing steel	Mpa	1670
E _p	Modulus of Elasticity	Mpa	195000
REINFORCEMENT			
f _y	Yield strength	Mpa	400
E _s	Modulus of Elasticity	Mpa	200000
n _c	Ratio E _s /E _c		7



Sign	Parameters	Unit	Section - PILE CAP					
			A-A	A-A	A-A	B-B	B-B	
INTERNAL FORCES AT SECTION								
	Combination		Strength	Service	Extreme	Service	Strength	
Q _u	Shear	kN	9138	5629	6039	2404	3223	
M _u	Flexural Moment	kNm	10905	6686	6983	859	1234	
N _u	Axial load	kN	0	0	0	0	0	
T _u	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	
d _{1x}	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	
d _s	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	
d _{1xp}	Dis. From tens. fiber to centroid of tension prestressing steel	m	0.000	0.000	0.000	0.000	0.000	
d _{ps}	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	5.500	5.500	
bw	Web width or diameter of a circular section	m	8.000	8.000	8.000	5.500	5.500	
h _f	Compression flange depth	m	0.000	0.000	0.000	0.000	0.000	
I _z	Moment of inertia of section	m4	16.7880	16.7880	16.7880	14.5130	14.5130	
A _{mc}	Section area	m2	16.000	16.000	16.000	11.000	11.000	
	Steel choice							
A _{ps}	Tension prestressing steel	P.S type	0 T00.0	0 T00.0	0 T00.0	0 T00.0	0 T00.0	
		Number	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	0	0	0	0	0	
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
A _s	Tension Reinforcement	Number	53	53	53	36	36	
		Diameter	mm	25	25	25	20	20
		Area	m2	0.02602	0.02602	0.02602	0.01130	0.01130
A' _s	Compression Reinforcement	Number	53	53	53	36	36	
		Diameter	mm	20	20	20	20	20
		Area	m2	0.01664	0.01664	0.01664	0.01130	0.01130
A' _c	Shear reinforcement	Number	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	
φ _v	Resistance factors for shear		0.90	1.00	1.00	1.00	0.90	
φ _n	Resistance factors for axial force		1.00	1.00	1.00	1.00	1.00	
β ₁	Stress block factor		0.836	0.836	0.836	0.836	0.836	
c	Dis. Between centroid and top fiber	m	0.022	0.022	0.022	0.000	0.000	
	For T section behavior	m	0.022	0.022	0.022	0.000	0.000	
	For rectangular section behavior	m	0.022	0.022	0.022	0.000	0.000	
f _{pe}	Effective stress in the prestressing steel after losses	Mpa	1116	1116	1116	1116	1116	
f _{ps}	Aver. stress in pres. steel at the time for which the nominal resistance	Mpa	1854	1854	1854	1860	1860	
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	0.018	0.018	0.018	0.000	0.000
dc	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	18473	18473	18473	7897	7897
Mr	Factored resistance	kNm	16626	18473	18473	7897	7107
Mu	Flexural moment	kNm	10905	6686	6983	859	1234
(5.7.3.2)	Flexural moment Checking		OK	OK	OK	OK	OK
	Limits for reinforcement						
c/de	Maximum reinforcement		0.01	0.01	0.01	0.00	0.00
	Maximum reinforcement Checking	≤ 0.42	OK	OK	OK	OK	OK
1.2*Mcrr	Cracking moment	kNm	35144	35144	35144	30048	30048
(5.7.3.3.2)	Checking $M_r \geq \min(1.2M_{cr}, 1.33M_u)$		OK	OK	OK	OK	OK
(5.7.3.4)	Control of cracking by distr. of reinf for RC member- Check?		No	Yes	No	Yes	No
	Existing condition for structure	1,2 or 3	1	1	1	1	1
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.063	0.063	0.063	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	m ²	0.019	0.019	0.019	0.018	0.018
f _{sa}	Value	Mpa	284	284	284	291	291
0.6*f _y		Mpa	240	240	240	240	240
	Tensile stress in reinf Min(f _{sa} , 0.6f _y)	Mpa	240	240	240	240	240
x	Dist. From compression fiber to centroid	m	-	0.267	-	0.216	-
J.d	Arm	m	-	1.745	-	1.762	-
I _{cr}	Moment of inertia of the cracked section	m ⁴	-	0.501	-	0.227	-
f _s	Tensile stress in reinforcement f _s = M _s / (A _s *J.d)	Mpa	-	147	-	43	-
	Checking for control cracking f _s < f _{sa}		N.a	OK	N.a	OK	N.a
(5.10.8.2)	Shrinkage and temperature Reinforcement (side distribution)						
A _{req}	Area of required reinf	m ²	0.00127	0.00127	0.00127	0.00127	0.00127
	Distribution on sides 10 D16	m ²	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	2.0	2.0	2.2	2.2
θ	Angle of inclination of diagonal compressive	degree	42.58	40.34	39.80	35.77	36.83
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	90
b _v	Effective web width as minimum web width - in dv	m	8.000	8.000	8.000	5.500	5.500
d _v	Effective shear depth	m	1.825	1.825	1.825	1.834	1.834
	(d _e - a/2)	m	1.825	1.825	1.825	1.834	1.834
s	Spacing of stirrups	m	600.000	600.000	600.000	600.000	600.000
n _{cat}	Amount of bars in spacing S	bars	8	8	8	5	5
A _v	Shear reinf area in spacing S	m ²	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	39.10	36.53	42.00	34.50	42.24
v	Shear stress in concrete	kN/m ²	695	386	414	238	355
f _{po}	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e _s	Strain in tensile reinforcement		1.89E-03	1.43E-03	1.38E-03	9.81E-04	1.08E-03
	if e _s < 0, multiple with reduce factor		-	-	-	-	-
	Strain checking	$\leq 2.00E-3$	Ok	Ok	Ok	Ok	Ok
v/f _c	Ratio of shear stress and f _c		0.023	0.013	0.014	0.008	0.012
β	Final value		1.8	2.0	2.0	2.2	2.2
θ	Final value	degree	42.58	40.34	39.80	35.77	36.83
V _c	Nominal shear resistance provided by tensile stresses in the concrete	kN	11738	13186	13389	10275	10014
V _s	Shear resistance provided by shear reinforcement	kN	2	2	2	2	2
V _p	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0	0
V _{n1}	V _{n1} = V _c + V _s + V _p	kN	11740	13189	13391	10277	10015
V _{n2}		kN	109488	109488	109488	75653	75653
V _n	Nominal shear resistance V _n = min(V _{n1} , V _{n2})	kN	11740	13189	13391	10277	10015
V _r	Factored shear resistance	kN	10566	13189	13391	10277	9014
V _u	Shear	kN	9138	5629	6039	2404	3223
(5.8.2.7)	Shear checking		OK	OK	OK	OK	OK

3. Bored pile capacity

SPACE PILE FOUNDATION ANALYSIS PROGRAM
Turbo BASIC

PROJECT: E.X Da Nang - Quang Ngai
Bridge: ORB22 - KM82+348.0

INITIAL DATA

Kn = 0.00 Ax = 7.00 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 = 2000 (t/m4)

LOAD COMBINATIONS

COMB.	Hx	Hy	P	Mx	My	Mz
1	700.00	0.00	2204.00	-295.00	2004.00	0.00
2	466.00	0.00	1517.00	-274.00	1444.00	0.00
3	678.00	5.00	2148.00	-289.00	1871.00	0.00
4	443.00	5.00	1461.00	-267.00	1311.00	0.00
5	480.00	4.00	1592.00	-223.00	1415.00	0.00
6	519.00	30.00	2029.00	-307.00	726.00	0.00
7	519.00	30.00	1343.00	-285.00	1111.00	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	-2.50	4.66	0.000	0.00
2	2.50	4.66	0.000	0.00
3	2.50	1.34	0.000	0.00
4	2.50	-1.98	0.000	0.00
5	2.50	-5.30	0.000	0.00
6	-2.50	-5.30	0.000	0.00
7	-2.50	-0.32	0.000	0.00

FORCES ON PILES

PILE	COMB.	N	Q2	Q3	M1	M2	M3
1	1	196.49	-107.11	-4.08	1.924	-4.268	193.394
	2	136.50	-71.30	-2.71	1.281	-2.463	127.870
	3	197.68	-103.80	-4.69	1.879	-5.586	188.142
	4	137.78	-67.84	-3.33	1.233	-3.773	122.325
	5	148.04	-73.49	-3.39	1.332	-3.924	132.591
	6	270.90	-79.75	-7.50	1.519	-11.189	152.640
	7	122.54	-79.75	-7.50	1.519	-12.167	146.655
2	1	484.55	-107.11	3.06	1.924	10.287	193.393
	2	338.30	-71.30	2.04	1.281	7.227	127.869
	3	468.54	-103.80	2.27	1.879	8.628	188.141

3	4	322.16	-67.84	1.25	1.233	5.555	122.324
	5	346.88	-73.49	1.54	1.332	6.149	132.591
	6	386.37	-79.75	-1.87	1.519	0.298	152.639
	7	306.61	-79.75	-1.87	1.519	-0.680	146.654
	1	453.72	-102.37	3.06	1.924	10.286	183.728
	2	314.90	-68.15	2.04	1.281	7.226	121.435
	3	438.14	-99.17	2.27	1.879	8.627	178.703
4	4	299.21	-64.80	1.25	1.233	5.554	116.131
	5	324.05	-70.21	1.54	1.332	6.149	125.902
	6	355.02	-76.01	-1.87	1.519	0.298	145.011
	7	282.70	-76.01	-1.87	1.519	-0.680	139.026
	1	422.90	-97.63	3.06	1.924	10.286	174.064
	2	291.50	-64.99	2.04	1.281	7.226	115.001
	3	407.74	-94.54	2.27	1.879	8.627	169.265
5	4	276.26	-61.77	1.24	1.233	5.554	109.937
	5	301.23	-66.93	1.54	1.332	6.148	119.213
	6	323.67	-72.27	-1.87	1.519	0.297	137.384
	7	258.79	-72.27	-1.87	1.519	-0.680	131.398
	1	392.07	-92.89	3.06	1.924	10.285	164.399
	2	268.10	-61.84	2.04	1.281	7.226	108.567
	3	377.34	-89.92	2.27	1.879	8.626	159.827
6	4	253.31	-58.73	1.24	1.233	5.554	103.744
	5	278.41	-63.65	1.54	1.332	6.148	112.524
	6	292.32	-68.53	-1.87	1.519	0.297	129.756
	7	234.88	-68.53	-1.87	1.519	-0.681	123.770
	1	104.02	-92.89	-4.08	1.924	-4.270	164.400
	2	66.30	-61.84	-2.71	1.281	-2.464	108.568
	3	106.48	-89.92	-4.70	1.879	-5.588	159.828
7	4	68.93	-58.73	-3.33	1.233	-3.774	103.745
	5	79.58	-63.65	-3.39	1.332	-3.925	112.525
	6	176.85	-68.54	-7.50	1.519	-11.191	129.757
	7	50.80	-68.54	-7.50	1.519	-12.169	123.771
	1	150.25	-100.00	-4.08	1.924	-4.269	178.897
	2	101.40	-66.57	-2.71	1.281	-2.463	118.219
	3	152.08	-96.86	-4.70	1.879	-5.587	173.985
	4	103.35	-63.29	-3.33	1.233	-3.773	113.035
	5	113.81	-68.57	-3.39	1.332	-3.925	122.558
	6	223.88	-74.14	-7.50	1.519	-11.190	141.198
	7	86.67	-74.14	-7.50	1.519	-12.168	135.213

SUMMARY OF FORCES

	FILE	COMB.	N	Q2	Q3	M1	M2	M3
Nmin	6	7	50.80	-68.54	-7.50	1.519	-12.169	123.771
Nmax	2	1	484.55	-107.11	3.06	1.924	10.287	193.393
Q2max	1	1	196.49	-107.11	-4.08	1.924	-4.268	193.394
Q3max	6	7	50.80	-68.54	-7.50	1.519	-12.169	123.771
M1max	1	1	196.49	-107.11	-4.08	1.924	-4.268	193.394
M2max	6	7	50.80	-68.54	-7.50	1.519	-12.169	123.771
M3max	1	1	196.49	-107.11	-4.08	1.924	-4.268	193.394

CHECKING CALCULATI IN COMPARISON WITH INITIA LOAD MATRIX

1	700.00	0.00	2204.00	-295.00	2004.00	0.00
2	466.00	0.00	1517.00	-274.00	1444.00	0.00
3	678.00	5.00	2148.00	-289.00	1871.00	0.00
4	443.00	5.00	1461.00	-267.00	1311.00	0.00
5	480.00	4.00	1592.00	-223.00	1415.00	0.00
6	519.00	30.00	2029.00	-307.00	726.00	0.00
7	519.00	30.00	1343.00	-285.00	1111.00	0.00

DANANG QUANG NGAI EXPRESSWAY ORB22 BRIDGE DETAIL DESIGN CHECK REINFORCEMENT OF BORED PILE	Item.	Eng.	Date.	Sign.
	Design			
	Check			
	Revise			

BORED PILE DESIGN

I. BORED PILE DATA

1. Load Combinations at top of bored pile

No	Combinations	Sign	F _v (kN)	Longitudinal		Transvesal	
				F _{Hx} (kN)	My (kN•m)	F _{Hv} (kN)	Mx (kN•m)
1	Extreme Ext-IB		488	674	-1223	74	120
2	Strength Str-IA		4890	1057	-1915	-30	-102
3	Strength Str-IA		1969	1057	-1915	40	42
4	Strength Str-IA		4890	1057	-1915	-30	-102
5	Strength Str-IA		1969	1057	-1915	40	42
6							

2. Bored pile Material

Normal concrete			
Compressive strength at 28 days age	f _c	30	MPa
Concrete elastic modulus	E _c	27691	MPa
Reinforcement			
Yield strength	f _y	420	MPa
Reinforcement elastic modulus	E _s	200,000	MPa

3. Bored pile Section

Pile diameter	D	1.00	m
Section area	A	0.785	m ²
Moment inertia	I _x	0.049	m ⁴
	I _y	0.049	m ⁴
Radius of gyration of gross concrete section; $r = \sqrt{I/A}$	r _x	0.250	m
	r _y	0.250	m

II. PILE DESIGN

1. Limit of Reinforcement

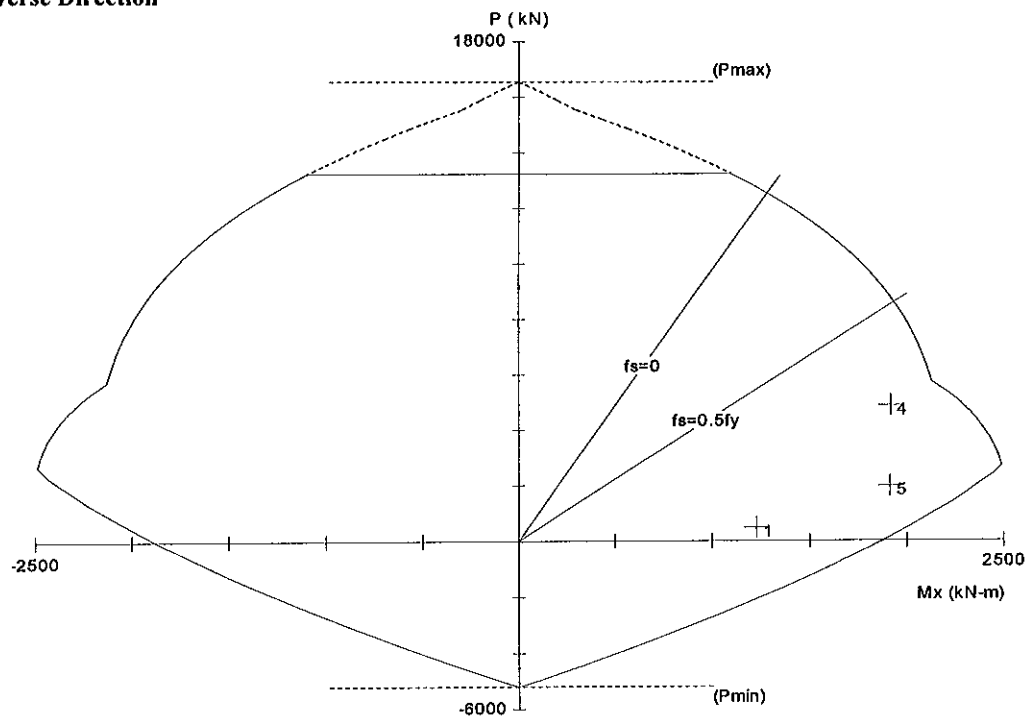
S.5.7.4.2

Minimum area of longitudinal reinforcement in column			
$A_s \cdot f_y / (A_g \cdot f_c) \geq 0.135$	$A_s \geq$	0.008	m ²
$A_s / A_g \geq 0.01$	$A_s \geq$	0.008	m ²
Maximum area of longitudinal reinforcement in column			
$A_s / A_g \leq 0.08$	$A_s \leq$	0.063	m ²
Trial Rebars:	Ok A_s	0.015	m ²
1 layers x 24 = 24 bars	D28 @150 A_{s1}	0.015	m ²

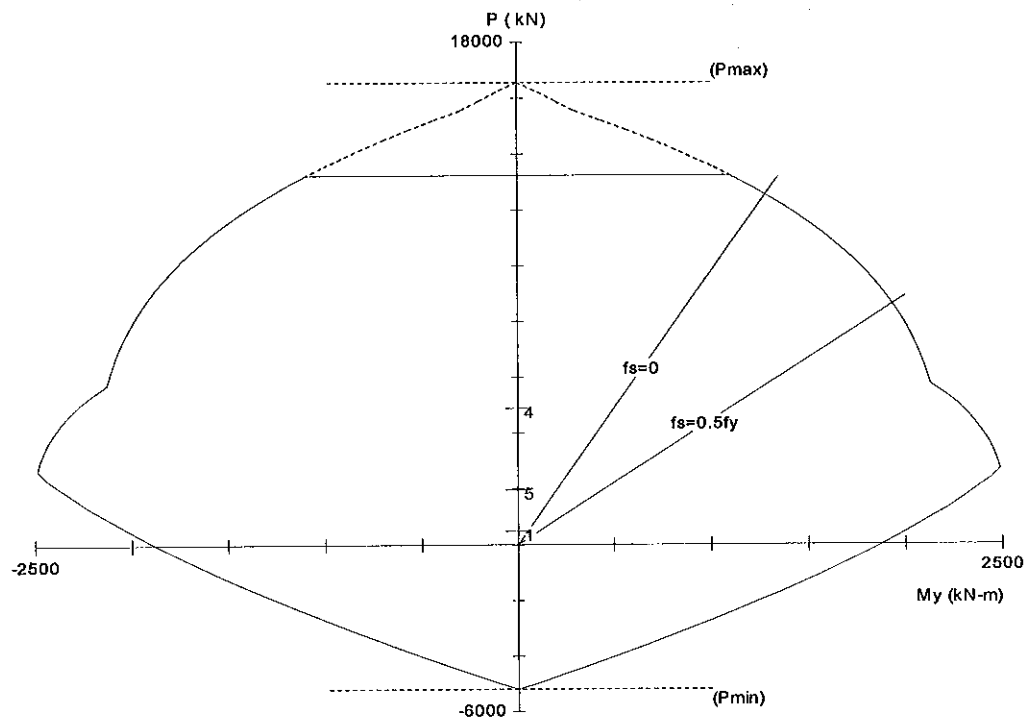
2. Interaction diagram M-P

Using Pca-Column software

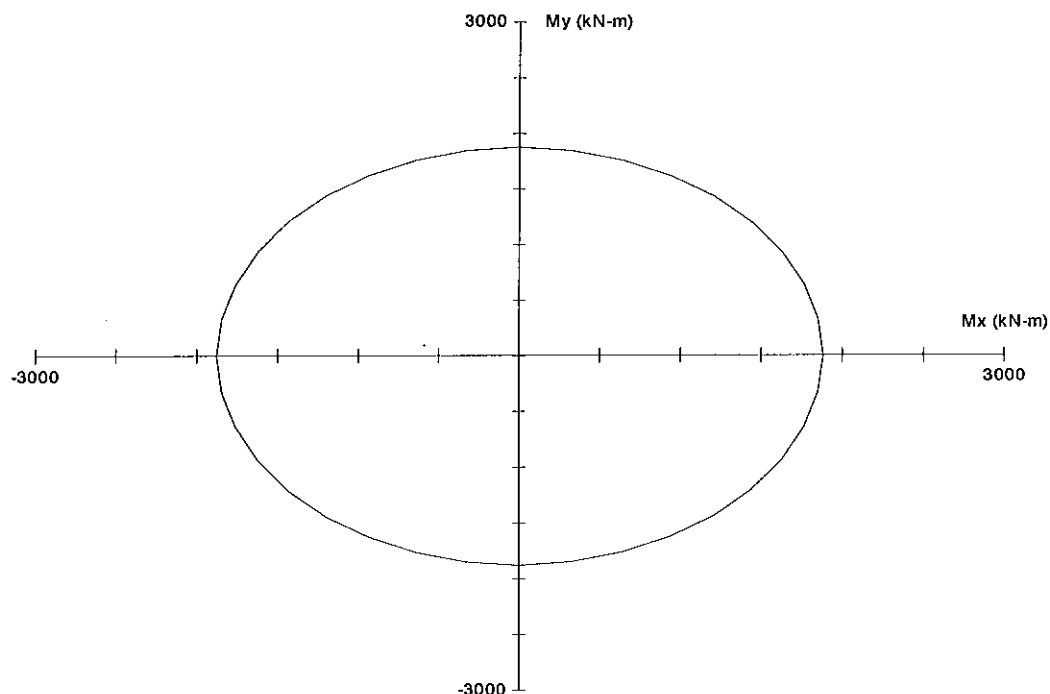
**In Transverse Direction



**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	0.622	m2
Tie diameter	Dtie	14	mm
Cross section area of 1 tie	As-tr	0.0002	m2
Spacing of hoops	s	75	mm
Length of reinforcement tie in 1 hoop	Ltie	2.78	m
Ratio of ties reinf. in one hoop/ volume of conc. core one pitch spacing			
$ps = As-tr \cdot Ltie / (Ac \cdot spacing)$	ps	0.0096	
Ratio of spiral reinf. To total volume of concrete core shall satisfy			S.5.7.4.6
$ps \geq 0.45 \cdot (Ag/Ac - 1) \cdot fc/fy = Req1$	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	
$ps \geq 0.12 \cdot fc/fy = Req2$	Req2	0.0086	N/A
Length distributed spiral with pitch 75mm below pilecap	Ldis	1.50	m

4. Shear Design

Shear resistance factors	ϕ_v	1.0	
Factored shear force	Vu	1057	kN
Required shear capacity $Vn = Vu / \phi_v$	Vn	1057	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
$dv = 0.9 \cdot de$ $de = D/2 + Dr/\pi()$			
Diameter of the circle passing through the centers of the long. reinf.	Dr	0.79	m
	de	0.75	m
	dv	0.68	m

$V_c = 0.083 \cdot \beta \cdot \sqrt{f_c} \cdot b_v \cdot d_v$	V_c	616	kN
	A_v	1963	mm ²
Angle of inclination of shear reinf. to long. axis	α	90	
$V_s = A_v \cdot f_y \cdot d_v \cdot (\cot \theta + \cot \alpha) \cdot \sin \alpha / s$	V_s	7447	kN
$V_{n1} = V_c + V_s$	V_{n1}	8063	
$V_{n2} = 0.25 f_c b_v d_v$	V_{n2}	5081	
	V_n	5081	
	Conclude		OK

DANANG QUANG NGAI EXPRESSWAY				Item.	Eng.	Date.	Sign.
ORB22 BRIDGE				Design			
DETAIL DESIGN				Check			
EMPIRICAL ESTIMATION OF PILE CAPACITY-ABUTMENT A1				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	ORB22-A1	Pile Concrete comp. strength	$f'_c =$	30.0	MPa
Bottom of pilecap elavation	EL1 = 3.50	Concrete Unit Weight	$g_c =$	24.5	kN/m ³
Top of socket elevation	EL2 = -3.22	Modulus of elasticity of concre	$E_c =$	27691	MPa
Pile tip elevation	EL3 = -8.50				
Pile Length	$L =$ 12.00 m	Depth of socket	$H_s =$	5.28	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	$A_p =$ 0.79 m ²	Socket Cross-Sectional Area	$A_{soc} =$	0.79	m ²
Working normal force at pile head	$N =$ 5121.6 kN				
Working normal force at top of socket	$P_i =$ 5060.6 kN				
Intack rock modulus	$E_i =$ 25000 MPa				
Modulus modification ratio	$K_o =$ 0.01				
Elastic modulus of the insitu rock	$E_r = K_o * E_i =$ 250.0 MPa				
Influence coefficient	$I_p = f(H_s/D_s, E_c/E_r) =$ 0.18				
	$H_s/D_s =$ 5.28				
	$E_c/E_r =$ 110.77				
					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

$$r_e = (\sum P_i) * H_s / (A_{soc} * E_c) = 1.229 \text{ mm}$$

The settlement of base of drilled shaft

$$r_{base} = (\sum P_i) * I_p / (D_s * E_r) = 3.644 \text{ mm}$$

$$r_e + r_{base} = 4.872 \text{ mm} < 10 \text{ mm}$$

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 $q_u < 1.9 \text{ Mpa}$ - may be taken after Carter & Kulhawy 1988 $\rightarrow q_s = 0.15 * q_u$

C10.8.3.5-4

if $q_u > 1.9 \text{ Mpa}$ - may be taken after Horvath & Kenney 1979 $\rightarrow q_s = 0.21 * \text{sqrt}(q_u)$

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 $\rightarrow q_s = 0.65 * \alpha_E * p_a * (q_u/p_a)^{0.5} < 7.8 * p_a * (f_c/p_a)^{0.5}$

$q_{s0} = 7.8 * p_a * (f_c/p_a)^{0.5}$

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	-3.22	-5.22	2.00	21	39.99	1.33	8344	0.65	5424
2	-5.22	-7.22	2.00	43	39.99	1.33	8344	0.65	5424
3	-7.22	-8.50	1.28	50	39.99	1.33	5340	0.65	3471
4									
5									
6									
7									
8									
Sum			5.28				22028		14318

	DANANG QUANG NGAI EXPRESSWAY	Item.	Eng.	Date.	Sign.
	ORB22 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	q _{s0} (MPa)	q _s (MPa)	q _s - used (MPa)	Q _{SR} (kN)	φ _s	Q _R (kN)
1	2.00	21.00	39.99	0.05	0.46	1	13.58	0.60	0.60	3748	0.55	2062
2	2.00	43.00	39.99	0.13	0.57	1	13.58	0.74	0.74	4678	0.55	2573
3	1.28	50.00	39.99	0.15	0.59	1	13.58	0.77	0.77	3086	0.55	1697
4	-	-	-	-	-	-	-	-	-	-	-	-
5	-	-	-	-	-	-	-	-	-	-	-	-
6	-	-	-	-	-	-	-	-	-	-	-	-
7	-	-	-	-	-	-	-	-	-	-	-	-
8	-	-	-	-	-	-	-	-	-	-	-	-
Sum	5.28									11513		6332

Unit base resistance

$$q_p = K_b \cdot (p_1 - p_0) + \sigma_v$$

Limit pressure determined from presuremeter tests

$$p_1 = \text{ - MPa}$$

C10.8.3.5-7

At rest total horizontal stress measured at ther base elevation

$$p_0 = \text{ - MPa}$$

Coefficient that depen on diameter socket

$$K_b = 5.00$$

Table C10.8.3.5-1

Total vertical stress at the base elevation

$$\sigma_v = \text{ - MPa}$$

$$q_p = \text{ - MPa}$$

$$\phi = 0.50$$

Table 10.5.5-3

$$Q_{pR} = A_p \cdot q_p$$

$$Q_{pR} = \text{ - kN}$$

$$Q_R = \phi \cdot Q_{pR}$$

$$Q_R = \text{ - kN}$$

ESTIMATED PILE CAPACITY:

Pile Structural Capacity	$Q_T = 0.75 \cdot 0.85 \cdot 0.85 \cdot f_c \cdot A_g$	12768 kN	1301 T
Pile resistance	Q_R	6332 kN	645 T
Deducting pile weight		-179 kN	-18 T
Estimated Pile Capacity		6153 kN	627 T
Maximum Reaction - ULS	Ok	4890 kN	499 T

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ORB22 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	ORB22-A2	Pile Concrete comp. strength	$f_c =$	30.0	MPa
Bottom of pilecap elavation	EL1 = 3.50	Concrete Unit Weight	$g_c =$	24.5	kN/m ³
Top of socket elevation	EL2 = -3.05	Modulus of elasticity of concre	$E_c =$	27691	MPa
Pile tip elevation	EL3 = -7.50				
Pile Length	L = 11.00 m	Depth of socket	$H_s =$	4.45	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 = 5102.4 kN				
Working normal force at top of socket	$P_i =$ 5050.9 kN				
Intack rock modulus	$E_i =$ 25000 MPa				
Modulus modification ratio	$K_o =$ 0.15				
Elastic modulus of the insitu rock	$E_r = K_o * E_i =$ 3750.0 MPa				
Influence coefficient	$I_p = f(H_s/D_s, E_c/E_r) =$ 0.30				
	$H_s/D_s =$ 4.45				
	$E_c/E_r =$ 7.38				
Rock mass modulus/ intack rock modulus	E_m / E_i				
Atmospheric pressure	$p_a =$ 0.101 MPa				
Reduction factor to account for jointing	α_E				

Figure C10.8.3.5-2 Lrfd

Figure C10.8.3.5-3 Lrfd

Figure C10.8.3.5-1 Lrfd

C.10.4.6.5-1-Lrfd 4th

10.8.3.5.4b-Lrfd 4th

The elastic shortening of the drilled shaft

$$r_e = (\sum P_i) * H_s / (A_{soc} * E_c) = 1.033 \text{ mm}$$

The settlement of base of drilled shaft

$$r_{base} = (\sum P_i) * I_p / (D_s * E_r) = 0.404 \text{ mm}$$

$$r_e + r_{base} = 1.438 \text{ mm} < 10 \text{ mm}$$

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 $q_u < 1.9 \text{ Mpa}$ - may be taken after Carter & Kulhawy 1988 $\rightarrow q_s = 0.15 * q_u$

if $q_u > 1.9 \text{ Mpa}$ - may be taken after Horvath & Kenney 1979 $\rightarrow q_s = 0.21 * \sqrt{q_u}$

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 $\rightarrow q_s = 0.65 * \alpha_E * p_a * (q_u/p_a)^{0.5} < 7.8 * p_a * (f_c/p_a)^{0.5}$

$$q_{s0} = 7.8 * p_a * (f_c/p_a)^{0.5}$$

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	-3.05	-3.16	0.11	43	39.99	1.33	459	0.65	298
2	-3.16	-4.66	1.50	23	39.99	1.33	6258	0.65	4068
3	-4.66	-7.50	2.84	55	39.99	1.33	11848	0.65	7702
4									
5									
6									
7									
8									
Sum			4.45				18565		12068

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	ORB22 BRIDGE	Design			
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Case2												Type: "1: closed joints", "2: open joints"
No.	Depth (m)	RQD (%)	q_u (MPa)	E_m / E_i	α_E	Type	q_{s0} (MPa)	q_s (MPa)	$q_s - used$ (MPa)	Q_{SR} (kN)	ϕ_s	Q_R (kN)
1	0.11	43.00	39.99	0.13	0.57	1	13.58	0.74	0.74	257	0.55	142
2	1.50	23.00	39.99	0.06	0.47	1	13.58	0.61	0.61	2893	0.55	1591
3	2.84	55.00	39.99	0.29	0.69	1	13.58	0.90	0.90	8049	0.55	4427
4	-	-	-	-	-	-	-	-	-	-	-	-
5	-	-	-	-	-	-	-	-	-	-	-	-
6	-	-	-	-	-	-	-	-	-	-	-	-
7	-	-	-	-	-	-	-	-	-	-	-	-
8	-	-	-	-	-	-	-	-	-	-	-	-
Sum	4.45									11200		6160

Unit base resistance $q_p = K_b \cdot (p_1 - p_0) + \sigma_v$

Limit pressure determined from presuremeter tests

At rest total horizontal stress measured at ther base elevation

Coefficient that depen on diameter socket

Total vertical stress at the base elevation

$$p_1 = 5.89 \text{ MPa}$$

C10.8.3.5-7

$$p_0 = - \text{ MPa}$$

$$K_b = 4.71$$

Table C10.8.3.5-1

$$\sigma_v = - \text{ MPa}$$

$$q_p = - \text{ MPa}$$

$$\phi = 0.50$$

Table 10.5.5-3

$$Q_{pR} = A_p \cdot q_p$$

$$Q_{pR} = - \text{ kN}$$

$$Q_R = \phi \cdot Q_{pR}$$

$$Q_R = - \text{ kN}$$

ESTIMATED PILE CAPACITY:

Pile Structural Capacity	$Q_T = 0.75 \cdot 0.85 \cdot 0.85 \cdot f_c \cdot A_g$	12768 kN	1301 T
Pile resistance	Q_R	6160 kN	628 T
Deducting pile weight		-161 kN	-16 T
Estimated Pile Capacity		5999 kN	611 T
Maximum Reaction - ULS	Ok	4890 kN	499 T

B. Miscellaneous

1. Expansion joint

Da Nang Quang Ngai Expressway project ORB22 BRIDGE DETAIL DESIGN EXPANSION JOINT	Item.	Eng.	Date.	Sign.
	Design.			
	Check.			
	Revise.			

EXPANSION JOINT

I. Displacement

Maximum allowable displacements in longitudinal direction at pier A1 = 50 mm
Maximum displacement 34.1 OK

Maximum allowable displacements in longitudinal direction at abutment A2= 50 mm
Maximum displacement 18.9 OK

A1

Unit (mm)						
Tải trọng	Symbol	Sign	Displacement		Service	
			Case1	Case2	a	b
TU+	TU	+	7.18	7.18	1.20	1.20
TU-	TU	-	-6.53	-6.53	1.20	1.20
Cr&Sh	CR&SH	-	-4.90	-4.90	1.20	0.50
Other loads		±	1.44	-1.44	1.00	1.00
Max Stretch			=	7.6		
Max Shrink			=	-15.2		
Maximum displacement				15.2		

A2

Unit (mm)						
Tải trọng	Ký hiệu	Dấu	Displacement		Service	
			Case1	Case2	a	b
TU+	TU	+	8.98	8.98	1.20	1.20
TU-	TU	-	-8.16	-8.16	1.20	1.20
Cr&Sh	CR&SH	-	-6.12	-6.12	1.20	0.50
Other loads		±	1.80	-1.80	1.00	1.00
Max Stretch			=	9.5		
Max Shrink			=	-18.9		
Maximum displacement				18.9		

3 OP18a

Table of content - OP18a Bridge

A. Substructure design

1. Abutment A1
2. Bored pile capacity

MINISTRY OF TRANSPORT

VIETNAM EXPRESSWAY CORPORATION - PROJECT MANAGEMENT UNIT NO.85

DA NANG - QUANG NGAI EXPRESSWAY DEVELOPMENT PROJECT

PACKAGE: A2

BRIDGE

OP18A

CALCULATION SHEETS

SUBSTRUCTURE

CALCULATION SHEET
ABUTMENT

CALCULATION SHEET
ABUTMENT A1 L

Table of content

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

	Da Nang Quang Ngai Expressway project OP18A BRIDGE DETAIL DESIGN ABUTMENT A1L	Item.	Eng.	Date.	Sign.
		Design			
		Check			
		Revise			

LOAD COMPONENTS

Assumptions :

1. Bridge is considered to be in seismic with acceleration coeff. $A = 0.0580 \text{ g}$
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 :

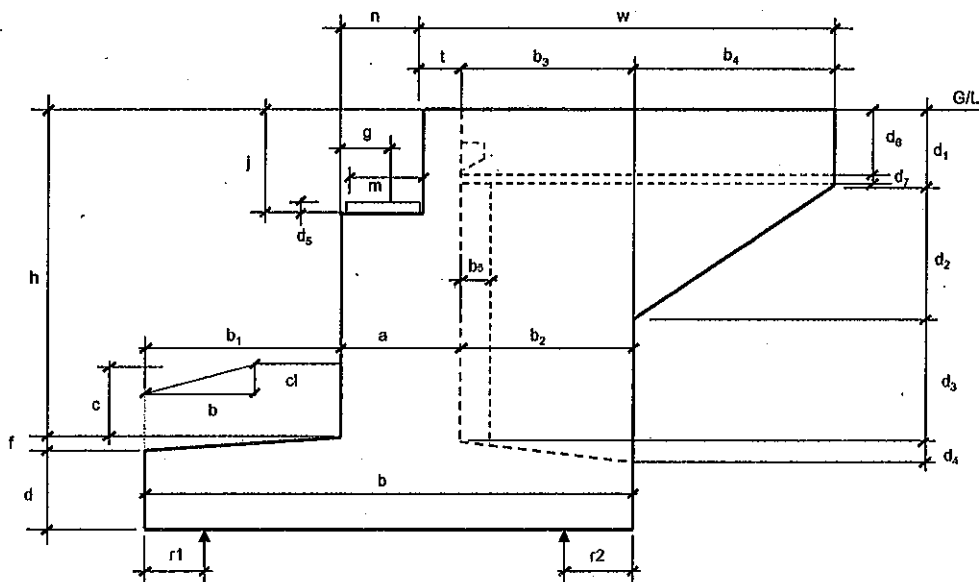
Level Table(at center of abutment)

Level of top of headwall	HTwL	15.769	m
Level of top of bearing	BTL	13.950	m
Level of top of stem abutment	HTL	13.654	m
Level of top of footing	FTL	6.500	m
Level of bottom of footing	FBL	4.500	m
Ground level	GL	7.816	m
Lowest water level	HWL	4.500	m
Skew angle	α	10.00	deg

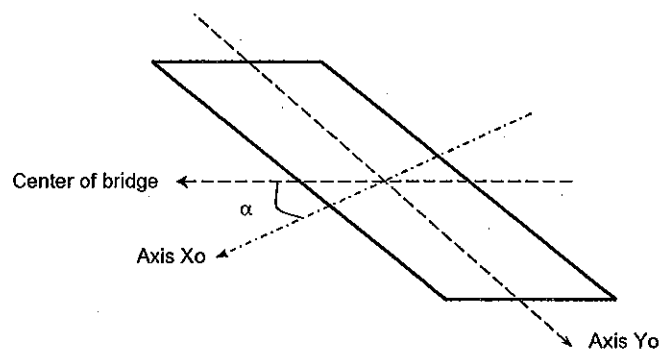
I.Loads from substructure

Abutment dimensions

VERTICAL VIEW



PLAN VIEW



Material Unit Weights

- Unit Weight of Reinf. concrete
- Unit Weight of Soil
- Unit Bouyancy Weight of Soil

$$\begin{aligned}\gamma_c &= 24.5 \text{ kN/m}^3 \\ \gamma_s &= 18.0 \text{ kN/m}^3 \\ \gamma_{sbo} &= 8.2 \text{ kN/m}^3\end{aligned}$$

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ABUTMENT A1L		Revise			

ABUTMENT DIMENSIONS (IN METRES)					
Item	Symbol	Value	Item	Symbol	Value
Height of stem	h	9.266	Horizontal Dimension	b ₃	3.700
Footing Width	b	9.000	Horizontal Dimension	b ₄	3.800
Stem Width	a	1.800	Horizontal Dimension	b ₅	0.300
Footing Depth	d	2.000	Vertical Dimension	d ₁	0.839
Footing Slope	f	0.000	Vertical Dimension	d ₂	3.800
Width of stem at bearing	n	1.300	Vertical Dimension	d ₃	4.627
Ballast Wall Height	l	2.114	Vertical Dimension	d ₄	0.000
Ballast Wall Thickness	t	0.500	Vertical Dimension	d ₅	0.296
Wingwall Length	w	8.000	Vertical Dimension	d ₆	0.900
Soil Cover at Toe	c	1.316	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

Slope front of abutment

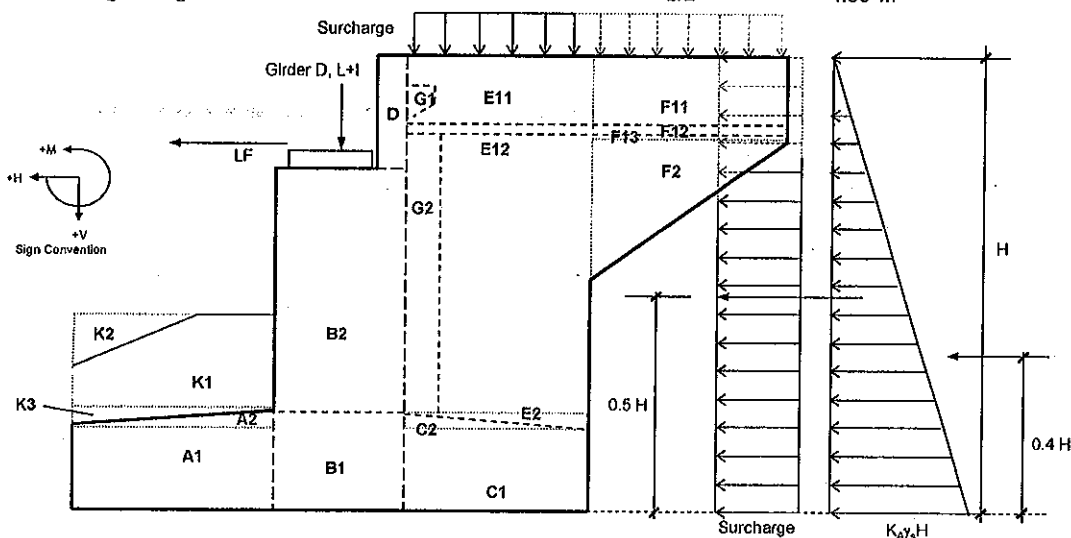
Width of Abutment

Width of abutment (Inclined direction)

Height of Abutment

Distance from CG of footing to edge of Abutment

cos (α)	=	0.98
cl	=	0.00 m
bl	=	0.00 m
L	=	12.600 m
Ltr	=	12.794 m
Ht	=	11.27 m
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	12.794	2194	1.750	2.750	6034
Section A2	-	12.794	-	2.333	2.167	-
Section B1	3.600	12.794	1128	4.400	0.100	113
Section B2	12.874	12.794	4035	4.400	0.100	404
Section C1	7.400	12.794	2320	7.150	-2.650	-6147
Section C2	-	12.794	-	6.533	-2.033	-
Section D	1.057	12.794	331	5.050	-0.550	-182
Section E11	3.330	0.500	41	7.150	-2.650	-108
Section E12	29.844	0.800	585	7.150	-2.650	-1550
Section F11	3.420	0.500	42	10.900	-6.400	-268
Section F12	1.125	0.650	18	9.050	-4.550	-82
Section F13	-	0.800	-	10.900	-6.400	-
Section F2	7.220	0.800	142	10.267	-5.767	-816
Section G1	0.135	11.794	282	5.450	-0.950	-267
Section G2	0.045	16.132	18	5.450	-0.950	-17
Bearing seats (w/seat= 0.85m)	0.266	4.250	31	4.350	0.150	5
Curbs +Handrail on Abutment	0.50	8.000	106	8.800	-4.300	-456
Total SW of Abutment (DC)			11273			-3338
Transverser moment			809		6.100	4936

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.

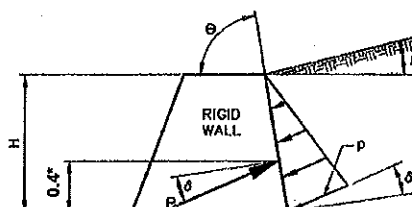
	Da Nang Quang Ngai Expressway project OP18A BRIDGE DETAIL DESIGN ABUTMENT A1L	Item.	Eng.	Date.	Sign.
		Design			
		Check			
		Revise			

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	34.28	11.194	6908	7.150	-2.650	-18307
Section E2	-	11.194	-	7.767	-3.267	-
Section E3	-	1.600	-	9.000	-4.500	-
Section K1	4.606	12.794	1081	1.750	2.750	-
Section K2	-	12.794	-	-	4.500	-
Section K3	-	12.794	-	1.167	3.333	-
Total Earth on Footing			7969			-18307

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	=	11.27 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	=	0.0 deg
• Incline angle of fill soil	β	=	0.0 deg
• Gravitational acceleration	g	=	9.81 m/s ²
• Basic earth pressure			
$p = K \cdot \gamma_s \cdot g \cdot Z \cdot 10^{-9}$ (Mpa, Z:mm)			
K: taken as K_a (assume wall move or deflect sufficiently to reach minimum active conditions)			

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

Γ	=	2.250
K_a	=	0.333
p	=	0.068 Mpa

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

Horizontal earth pressure:

• $E_a = 0.5 \cdot p \cdot Z \cdot B \cdot 10^3$ (kN)	E_a	=	4872 kN
• $M = E_a \cdot 0.4H$	M	=	21954 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=	11.27m	heq=	0.61 m

(Linear interpolation)

• Vertical force	ESv	=	520 kN
	ev	=	-2.65 m
	M	=	-1377 kNm
• Horizontal force	ESh	=	528 kN
	eh	=	5.63 m
	M	=	2972 kNm

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

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5. Earthquake effects

Bridge is located at: Tam Hiep district - Quang Ngai province

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

• Peak ground acceleration coefficient $A = 0.0580 \text{ g}$

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

- Backfill slop angle $i = 0.0 \text{ deg}$
- Slope of wall to vertical $\beta' = 0.0 \text{ deg}$
- Angle of friction of soil $\phi = 30.0 \text{ deg}$
- Angle of friction between soil and abutment $\delta = 0.0 \text{ deg}$
- Horizontal acceleration coefficient $k_h = 0.087$
- Vertical acceleration coefficient $k_v = 0.035$
- Angle $\theta = \arctan(k_h / (1 - k_v)) = 5.2 \text{ deg}$

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

• Seismic active lateral Earth pressure coefficient $K_{AE} = 0.39$

• $E_{AE} = 0.5 \cdot g \cdot \gamma \cdot H^2 \cdot (1 - k_v) \cdot K_{AE} \cdot 10^{-9} \text{ (kN/m)}$

• Seismic active lateral Earth pressure coefficient $E_{AE} = 5499 \text{ kN}$

$M_{AE} = E_{AS} \cdot 0.3H + (E_{AE} - E_{AS}) \cdot 0.6H$

$M_{AE} = 20703 \text{ KNm}$

<A.11.1.1.1>

E_{AS} is the static component of seismic active pressure calculated with $\theta = k_v = 0$

5.2. Earthquake effects to abutment (EQ)

Seismic force for substructures: elements above ground $F_h = C_{sm} \cdot W$; elements under ground $F_h = A \cdot S \cdot W$

- Soil profile type I
- Site Coefficients. $S = 1.0$
- Elastic Seismic Response Coefficient $2.5A = 0.145$
- $C_{sm} = 1.2 \cdot A \cdot S / T_m^{2/3} \leq 2.5 \cdot A$ $C_{sm} = 0.065$
- Period of vibration of the fundamental mode $T_m = 1.118 \text{ s}$

Description	Area (m ²)	Length (m)	Force (kN)	$X^{(1)}$ (m)	Arm ⁽²⁾ (m)	Moment (kN·m)
Section A1	7.000	12.794	127	-	1.000	127
Section A2	-	12.794	-	-	2.000	-
Section B1	3.600	12.794	65	-	1.000	65
Section B2	12.874	12.794	261	-	5.576	1453
Section C1	7.400	12.794	135	-	1.000	135
Section C2	-	12.794	-	-	2.000	-
Section D	1.057	12.794	21	-	10.209	218
Section E11	3.330	0.500	2	-	8.816	21
Section E12	29.844	0.800	34	-	4.033	-
Section E2	-	0.740	-	-	2.000	-
Section F11	3.420	0.500	2	-	8.816	21
Section F12	1.125	0.650	1	-	8.216	-
Section F13	-	0.800	-	-	9.447	-
Section F2	7.220	0.800	8	-	9.160	75
Section G1	0.135	11.794	2	-	8.553	19
Section G2	0.045	16.132	1	-	4.033	4
Total EQ of Abutment Selfweight			661			2140

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6. Braking Force(BR)

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

- Number of lanes
 - Multiple presence factor
 - Take 25 % of Truck load
- BR = 25% * n * m * (2*145+35)
 • Acting at 1.8m higher of road face

n	=	3 lanes	
m	=	0.85	
BR	=	104 kN	Long. Axis
e	=	13.1 m	
Mlong	=	1361 KNm	Long. Axis

7. Centrifugal Force, CE (3.6.3)

- Plan of bridge (1:"straight",2: "Curve")
- Design Speed

$C = 4/3 * (V^2 / gR)$
 Acting at 1.8m higher of road face
 $CE = n * m * (2*145+35) * C$

	=	1	
V	=	120 km/h	
V	=	33.3 m/s	
R	=	- m	
C	=	-	
CE	=	0.00 KN	
e	=	13.15 m	
Mtrans	=	0.00 KNm	Trans. Axis

8. Water Load (WA) :NA

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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	10.00	deg
Deck slab length	Ldeck	26.85	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	yc	24.50	kN/m3
Unit weight of asphalt concrete	ya	22.10	kN/m3

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

Item	Sign	Value	Unit
1.1. Girders	DC	464.77	kN
Weight of 1 girder	n	5	Girders
Number of girders	DC	2323.83	kN
Sum of girders weight	DC	394.43	kN
Precast Planks	DC	1176.38	kN
Diaphragm	DC	3894.64	kN
Total			
1.2. Deck slab	DC	1780.90	kN
Deck slab			
1.3. Pavement	DW	535.33	kN
Asphalt concrete			
1.4. Handrail	DC	639.90	kN
Handrail + median			

2. Live load (LL):

LL1:

Truck

Tandem

Lane load

Wpd = 0.0 kN/m²

Pedestrian

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

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

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

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

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

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

3. Earthquake effects on superstructure (EQ)

Longitudinal moveable bearings at Abutment
 Horizontal force from superstructure due to EQ - transverse direction
 At bearing

$$H_{eq} = 221 \text{ kN}$$

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

Bearing displacement due to uniform temperature and shrinkage creep
 $H = G.A.\Delta u/h_r$

Shear modulus G
 Bearing area
 Height of elastomeric layers
 Number of bearing
 Horizontal force due to TU+SH&CR
 Acting at top of bearing

$$\Delta u = 0.026 \text{ m} \quad <14.6.3.1-2>$$

$$G = 1 \text{ MPa}$$

$$A = 0.165 \text{ m}^2$$

$$h_r = 0.065 \text{ m}$$

$$n_b = 5 \text{ bears}$$

$$H(tu+sh+cr) = 330 \text{ kN}$$

5. Wind loads (Ws)

5.1. Transverse wind on superstructure (WS)

Wind zone
 Basic 3 second gust wind
 Correction factor
 Design wind velocity
 Drag coefficient
 Overall width of bridge
 Depth of superstructure (including solid parapet)

Windy obstructed area of superstructure
 Force due to transverse wind
 $F_{hy} = \max(0.0006*V^2*At*C_d, 1.8*At) \text{ (kN)}$

Zone III

$$V_b = 53.00 \text{ m/s}$$

$$S = 1.09$$

$$V = 57.77 \text{ m/s}$$

$$C_d = 1.39$$

$$b = 12.74 \text{ m}$$

$$d = 2.78 \text{ m}$$

$$b/d = 4.58$$

$$A_t = 75.14 \text{ m}^2$$

$$F_{hy} = 209.5 \text{ kN} \quad <3.8.1>$$

5.2. Wind load on vehicles (WL)

Transverse wind on vehicles
 Transverse horizontal force due to wind on live load
 At 1.8m from surface

$$W_{ltran} = 1.50 \text{ kN/m}$$

$$F_{hy} = 40.50 \text{ kN}$$

6. Combinations

Loads from superstructure to Abutment

Loads at bottom of stem		Vertical		Longitudinal		Transversal		
	Sign	N (kN)	x (m)	Hx (kN)	z ₁ (m)	My (kN·m)	Hy (kN)	Mx (kN·m)
Girder + Deckslab	DC	2838	0.05			142		
Handrail	DC	320	0.05			16		
Pavement	DW	268	0.05			13		
LiveLoad	LL	1233	0.05			62		1689
Pedestrian	PL	0	0.05			0		
Trans. wind on Struc.	WS						105	749
Trans. wind on vehi.	WL						20	181
Earthquake	EQ			330	7.15	2360	221	1582
TU+SH&CR	TU+SH&CR							

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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	2838	0.15			426			
Handrail	DC	320	0.15			48			
Pavement	DW	268	0.15			40			
LiveLoad	LL	1233	0.15			185		1.37	1689
Pedestrial	PL	0	0.15			0		-	-
Trans. wind on Struc.	WS						105	9.15	959
Trans. wind on vehi.	WL						20	10.95	222
Eearth quake	EQ						221	9.15	2025
TU+SH&CR	TU+SH&CR			330	9.15	3020			

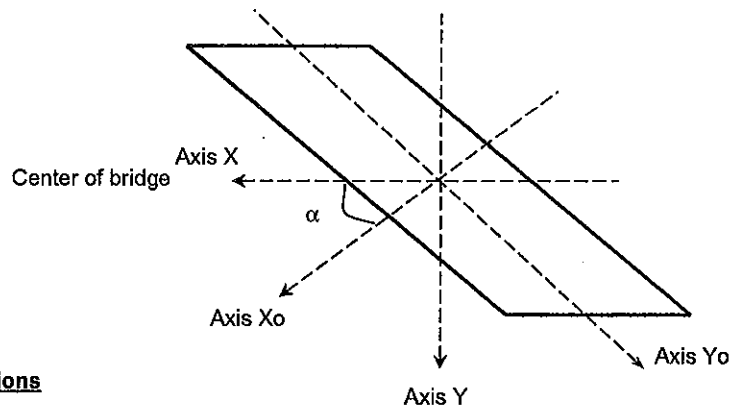
Loads	Sign	Load combinations						
		Str-IA	Str-IB	Str-IIIA	Str-IIIB	Ser-I	Ext-IA	Ext-IB
Girder + Deckslab	DC	1.25	0.90	1.25	0.90	1.00	1.25	0.90
Handrail	DC	1.25	0.90	1.25	0.90	1.00	1.25	0.90
Pavement	DW	1.50	0.65	1.50	0.65	1.00	1.50	0.65
LiveLoad	LL	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Pedestrial	PL	1.75	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					
Load combinations	N (kN)	Longitudinal		Tranversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	6506	165	1505	0	2955
Strength Str-IB	5173	165	1439	0	2955
Strength Str-IIIA	6013	165	1481	62	2760
Strength Str-IIIB	4680	165	1414	62	2760
Service Ser-I	4658	330	2593	52	2095
Extreme Ext-IA	4965	0	248	221	2427
Extreme Ext-IB	3632	0	182	221	2427

Load combinations at bottom of pilecap					
Load combinations	N (kN)	Longitudinal		Tranversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	6506	165	2486	0	2955
Strength Str-IB	5173	165	2286	0	2955
Strength Str-IIIA	6013	165	2412	62	2885
Strength Str-IIIB	4680	165	2212	62	2885
Service Ser-I	4658	330	3719	52	2198
Extreme Ext-IA	4965	0	745	221	2869
Extreme Ext-IB	3632	0	545	221	2869

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LOAD COMBINATIONS



III. Load Combinations

1. Loads from substructure

Loads at bottom of pilecap	Sign	Vertical		Longitudinal			Tranversal		
		N (kN)	x (m)	Hx (kN)	z ₁ (m)	My (kN.m)	Hy (kN)	y (m)	Mx (kN.m)
Self weight of Abutment	DC	11273				-3338			4935.79
Soils on pilecap	EV	7969				-18307			
Horizontal Earth Pressure	EH			4947		21954			
Vertical Surcharge	LSv	520				-1377			
Horizontal Surcharge	LSH			528		2972			
Braking Force	BR			104		1361			
Centrifugal Force	CE			-		-			
Buoyancy of Abutment	WA	-				-			
Buoyancy of Earth on Abutment	WA	-				-			
Earthquake effects to Abutment	EQ			661		2140	198		642
Earthquake effects to soil	E _{AE}			5583		21022			

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
Centrifugal 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	25759	8525	9215	0	6170
Strength Str-IB	18227	5557	5449	0	4442
Strength Str-IIIA	25551	8272	8033	0	6170
Strength Str-IIIB	18019	5304	4267	0	4442
Service Ser-I	19762	5578	3264	0	4936
Extreme Ext-IA	25109	6560	-4247	198	6812
Extreme Ext-IB	17578	6560	5159	198	5084

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2. Loads from superstructure

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	6506	165	2486	0	2955
Strength Str-IB	5173	165	2286	0	2955
Strength Str-IIIA	6013	165	2412	62	2885
Strength Str-IIIB	4680	165	2212	62	2885
Service Ser-I	4658	330	3719	52	2198
Extreme Ext-IA	4965	0	745	221	2869
Extreme Ext-IB	3632	0	545	221	2869

3. Total loads at bottom of pilecap

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	32264	8690	11701	0	9125
Strength Str-IB	23400	5722	7735	0	7397
Strength Str-IIIA	31563	8437	10445	62	9055
Strength Str-IIIB	22699	5469	6479	62	7327
Service Ser-I	24419	5908	6983	52	7134
Extreme Ext-IA	30074	6560	3502	419	9681
Extreme Ext-IB	21210	6560	5704	419	7954

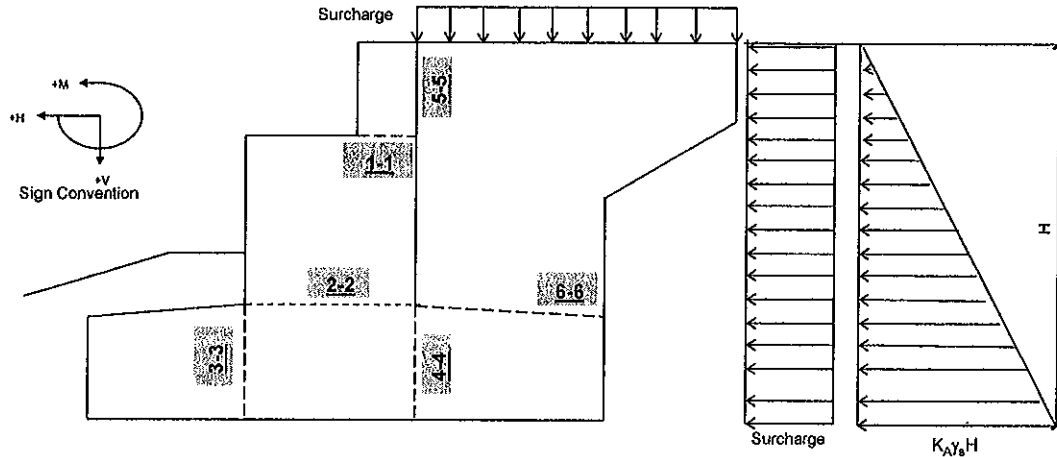
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ELEMENTS CHECKING

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 Ser-I	Strength I		Extreme I Ext-I
			Str-IA	Str-IB	
Selfweight	DC	1.00	1.25	0.90	1.25
Horizontal Earth Pressure	EH	1.00	1.50	0.90	
Surcharge (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		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Selfweight	613		-113		
Horizontal Earth Pressure		174	147		
Surcharge (horizontal)		246	260		
Horizontal Seismic Earth Pressure		197	139		
Abutment earthquake force		24	25	7	8

Load Combination at bottom of headwall					
Load combinations	N (kN)	Longitudinal		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Service Ser-I	613	421	295	0	0
Strength Str-IA	766	692	536	0	0
Strength Str-IB	552	588	487	0	0
Extreme Ext-I	766	442	223	7	8

1.2 Section 2-2

Table of Load Factors

Loads	Sign	Service I Ser-I	Strength I		Extreme I Ext-I
			Str-IA	Str-IB	
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	
Surcharge (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

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Unfactored Loads	N (kN)	Longitudinal		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Selfweight	4648		-286		
Superstructure Dead Load	2838		142		
Pavement	268		13		
Handrail+curb	320		16		
Live Load	1233		62		1689
Horizontal Earth Pressure		3346	12403		
Surcharge (Horizontal)		441	2041		
TU+SH&CR		330	2360		
Horizontal Seismic Earth Pressure		3777	11696		
Abutment earthquake force		284	1127	152	833

Load Combination at bottom of stem wall					
Load combinations	N (kN)	Longitudinal		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Service Ser-I	9306	4117	16752	0	1689
Strength Str-IA	12316	5956	23325	0	2955
Strength Str-IB	9356	3948	15916	0	2955
Extreme Ext-I	10775	6170	19583	152	1677

1.3 Section 3-3

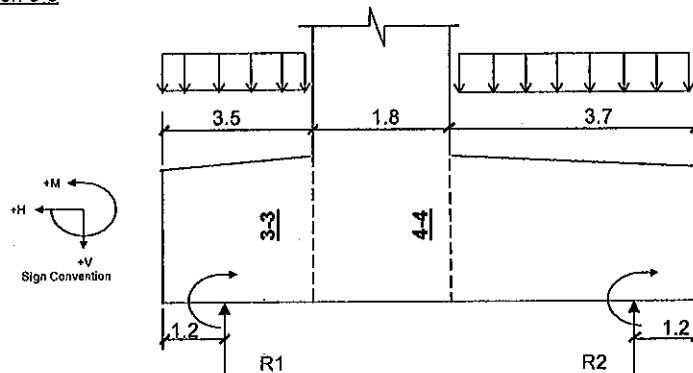


Table of Load Factors

Loads	Sign	Service I Ser-I	Strength I		Extreme I Ext-I
			Str-IA	Str-IB	
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		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Selfweight at front side	-2194		-3840		
Vertical soil on foot at front side	-1061		-1856		
Reaction of piles					
Ser-I	11852	2206	34583	-66	-230
Str-IA	16532	3247	48768	-72	-255
Str-IB	11555	2136	33658	-47	-164
Ext-I	12299	2453	36567	-206	-688

Load Combination at section 3-3					
Load combinations	N (kN)	Longitudinal		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Service Ser-I	8597	2206	28887	-66	-230
Strength Str-IA	12357	3247	41462	-72	-255
Strength Str-IB	8626	2136	28531	-47	-164
Extreme Ext-I	8124	2453	29261	-206	-688

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1.4 Section 4-4

Table of Load Factors

Loads	Sign	Service I Ser-I	Strength I		Extreme I Ext-I
			Str-IA	Str-IB	
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		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Selfweight of behind side	-3147		-6487		
Vertical soil on foot at behind side	-6908		-12780		
Surcharge(Vertical)	-520		-962		
Reaction of piles					
Ser-I	3794	1484	4558	29	95
Str-IA	4267	2184	3439	61	201
Str-IB	3465	1437	3899	40	135
Ext-I	6645	1648	11050	-65	-211

Load Combination at section 4-4					
Load combinations	N (kN)	Longitudinal		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Service Ser-I	-6781	1484	-15670	29	95
Strength Str-IA	-9902	2184	-23605	61	201
Strength Str-IB	-6494	1437	-15124	40	135
Extreme Ext-I	-6874	1648	-14793	-65	-211

1.4 Section 5-5 & 6-6

Slope of triang pressure
Uniform horizontal pressure

$tq\beta = 6.00$
 $U.p = 3.66 \text{ kN/m}^2$

Load Combination at section 5-5					
Load combinations	N (kN)	Horizontal		Vertical	
		Hx (kN)	Mx (kN.m)	Hy (kN)	My (kN.m)
Service Ser-I		102	246		
Strength Str-IA		157	392		

Load Combination at section 6-6					
Load combinations	N (kN)	Horizontal		Vertical	
		Hx (kN)	Mx (kN.m)	Hy (kN)	My (kN.m)
Service Ser-I				250	275
Strength Str-IA				387	426

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2. Elements Checking

2.1. Bearing Resistance

<S.5.7.5>

The case of absence of confinement reinforcement in the concrete supporting the bearing device

Factored bearing resistance shall be taken

$$Pr = \phi_c \cdot P_n = \phi_c \cdot 0.85 \cdot f_c \cdot A_1 \cdot m$$

Dimension of bearing plate

$$w_0 = 0.900 \text{ m}$$

$$b_0 = 0.850 \text{ m}$$

Area under bearing device

$$A_1 = 0.765 \text{ m}^2$$

Distributed width and length

$$w = 1.300 \text{ m}$$

$$b = 1.250 \text{ m}$$

Notational area

$$A_2 = 1.625 \text{ m}^2$$

Where supporting surface is wider on all sides than loaded area

$$m = \sqrt{A_2/A_1} \leq 2.0 \quad \text{case 1}$$

where loaded area is subjected to nonuniformly distributed bearing

$$m = 0.75 \cdot \sqrt{A_2/A_1} \leq 1.5 \quad \text{case 2}$$

Modification factor

$$m = 1.457 \quad \text{case 1}$$

Resistance factor

$$\phi_c = 0.700$$

Factored bearing resistance

$$Pr = 19902 \text{ kN} \quad > Pu \quad \text{S.5.5.4.2}$$

Bearing reaction of approach bridge

$$Pu = 4877 \text{ kN} \quad \text{Ok}$$

$$Pu = 1.25 \cdot DC + 1.5 \cdot DW + 1.75 \cdot LL$$

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>

$$Pr = \phi_c \cdot f_n \cdot A_b$$

f_n take the lesser of

$$f_n = 0.7 \cdot f_{ci} \cdot \sqrt{A_g/A_b} \quad \text{and} \quad f_n = 30.81 \text{ MPa}$$

$$f_n = 2.25 \cdot f_{ci}$$

Maximum area of the portion of supporting surface

$$A = 1.625 \text{ m}^2$$

Gross area of bearing plate

$$A_g = 0.765 \text{ m}^2$$

Effective net area of bearing plate, A_g minus stud of bearing

$$A_b = 0.765 \text{ m}^2$$

Nominal concrete strength at time of application

$$f_{ci} = 30 \text{ MPa}$$

Factored bearing resistance

$$Pr = 16390 \text{ kN} \quad \text{Ok}$$

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REINFORCEMENT CHECKING - HEAD AND STEM 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	197000
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 SECTION							
	Combination		Strength	Service	Service	Strength	Extreme
Qu	Shear	kN	692	421	4117	5956	6170
Mu	Flexural Moment	kNm	536	295	16752	23325	19583
Nu	Axial load	kN	766	613	9306	12316	10775
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	82	82	76	76	76
	Diameter	mm	16	16	28	28	28
	Area	m2	0.01656	0.01656	0.04682	0.04682	0.04682
A's	Compression Reinforcement	Number	82	82	76	76	76
	Diameter	mm	16	16	16	16	16
	Area	m2	0.01656	0.01656	0.01535	0.01535	0.01535
A'c	Shear reinforcement	Number	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	0.047	0.047	0.047
	For T section behavior	m	0.000	0.000	0.047	0.047	0.047
	For rectangular section behavior	m	0.000	0.000	0.047	0.047	0.047
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	1846	1846	1846
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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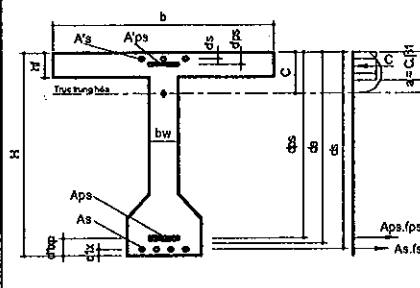
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REINFORCEMENT CHECKING - HEAD AND STEM WALL						
a	Depth of equivalent stress block	m	0.000	0.000	0.039	0.039
de	Corresp. effective depth from extreme comp. fiber to centroid of tensile force in the tensile reinf.	m	0.442	0.442	1.741	1.741
Mn	Nominal resistance	kNm	2544	2544	32000	32000
Mr	Factored resistance	kNm	2290	2544	32000	28800
Mu	Flexural moment	kNm	536	295	16752	23325
(5.7.3.2)	Flexural moment Checking		OK	OK	OK	OK
	Limits for reinforcement					
c/de	Maximum reinforcement		0.00	0.00	0.03	0.03
	Maximum reinforcement Checking	≤ 0.42	OK	OK	OK	OK
1.2*Mc	Cracking moment	kNm	1087	1087	14464	14464
(5.7.3.3.2)	Checking $M_r \geq \min(1.2M_{cr}, 1.33M_u)$		OK	OK	OK	OK
(5.7.3.4)	Control of cracking by distr. of reinf for RC member- Check?		No	Yes	Yes	No
	Existing condition for structure	1, 2 or 3	1	1	1	1
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.058	0.058	0.059	0.059
Z	Crack width parameter	N/mm	30000	30000	30000	30000
A	Area of concr. with same centroid as tens. Reinf	m ²	0.018	0.018	0.020	0.020
f _{sa}	Value	Mpa	297	297	286	286
0.6*f _y	Tensile stress in reinf Min(f _{sa} , 0.6f _y)	Mpa	240	240	240	240
x	Dist. From compression fiber to centroid	m	-	0.081	0.276	-
J.d	Arm	m	-	0.415	1.649	-
I _{cr}	Moment of inertia of the cracked section	m ⁴	-	0.017	0.796	-
f _s	Tensile stress in reinforcement $f_s = M_{sls} / (A_s * J.d)$	Mpa	-	43	217	-
	Checking for control cracking $f_s < f_{sa}$		N.a	OK	OK	N.a
(5.10.8.2)	Shrinkage and temperature Reinforcement (side distribution)					
A _{req}	Area of required reinf	m ²	0.00045	0.00045	0.00127	0.00127
	Distribution on sides	m ²	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		2.7	3.6	2.3	2.2
θ	Angle of inclination of diagonal compressive	degree	28.88	28.23	34.37	37.97
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90
b _v	Effective web width as minimum web width - in dv	m	12.600	12.600	12.600	12.600
d _v	Effective shear depth	m	0.442	0.442	1.721	1.721
	(de - a/2)	m	0.442	0.442	1.721	1.721
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600
n _{cat}	Amount of bars in spacing S	bars	20	20	19	19
A _v	Shear reinf area in spacing S	m ²	0.0030	0.0030	0.0029	0.0029
θ	Assume	degree	28.88	28.23	34.37	37.97
v	Shear stress in concrete	kN/m ²	138	76	190	305
f _{po}	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116
e _x	Strain in tensile reinforcement		4.40E-04	2.27E-04	8.64E-04	1.20E-03
	if e _x < 0, multiple with reduce factor		-	-	-	-
	Strain checking	$\leq 2.00E-3$	Ok	Ok	Ok	Ok
v/f _c	Ratio of shear stress and f _c		0.005	0.003	0.006	0.010
β	Final value		2.7	3.6	2.3	2.1
θ	Final value	degree	28.88	28.23	34.37	37.97
V _c	Nominal shear resistance provided by tensile stresses in the concrete	kN	6951	9066	22741	20901
V _s	Shear resistance provided by shear reinforcement	kN	1613	1658	4815	4219
V _p	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0
V _{n1}	$V_{n1} = V_c + V_s + V_p$	kN	8565	10724	27556	25120
V _{n2}	V _{n2}	kN	41769	41769	162674	162674
V _n	Nominal shear resistance $V_n = \min(V_{n1}, V_{n2})$	kN	8565	10724	27556	25120
V _r	Factored shear resistance	kN	7708	10724	27556	22608
V _u	Shear	kN	692	421	4117	5956
(5.8.2.7)	Shear checking		OK	OK	OK	OK
	Region requiring transverse reinf Checking		No need	No need	No need	No need
	Minimum shear reinf area	m ²	0.0086	0.0086	0.0086	0.0086
	Minimum shear reinforcement Checking		-	-	-	-
	$0.1 * f_c * b_v * d_v$	kN	16708	16708	65069	65069
	S _{max}	m	0.35	0.35	0.60	0.60
	Maximum spacing S _{max}		-	-	-	-

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REINFORCEMENT CHECKING - PILECAP SECTION

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	197000					
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 SECTION								
	Combination		Service	Strength	Extreme	Extreme	Strength	
Qu	Shear	kN	8597	12357	8124	6874	9902	
Mu	Flexural Moment	kNm	28887	41462	29261	14793	23605	
Nu	Axial load	kN	2206	3247	2453	1648	2184	
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	0.164	0.164	
d1x	Dis. From tens. fiber to centroid of tension Reinf	m	0.164	0.164	0.164	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	1.836	1.836	1.836	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	
Aps	Tension prestressing steel	P.S type	0	0	0	0	0	
		Number	tendons	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	
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
As	Tension Reinforcement	Number	168	168	168	84	84	
		Diameter	mm	28	28	28	28	28
		Area	m2	0.10349	0.10349	0.10349	0.05174	0.05174
A's	Compression Reinforcement	Number	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	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	0.154	0.154	0.154	0.077	0.077	
	For T section behavior	m	0.154	0.154	0.154	0.077	0.077	
	For rectangular section behavior	m	0.154	0.154	0.154	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 resistance	Mpa	1819	1819	1819	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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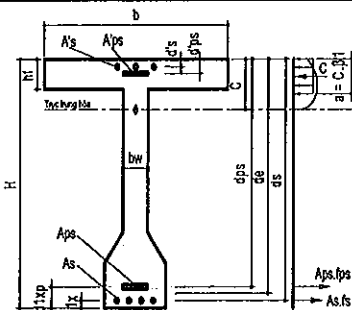
REINFORCEMENT CHECKING - PILECAP SECTION							
a	Depth of equivalent stress block	m	0.129	0.129	0.129	0.064	0.064
de	Corresp. effective depth from extreme comp. fiber to centroid of tensile force in the tensile reinf.	m	1.836	1.836	1.836	1.916	1.916
Mn	Nominal resistance	kNm	73335	73335	73335	38990	38990
Mr	Factored resistance	kNm	73335	66001	73335	38990	35091
Mu	Flexural moment	kNm	28887	41462	29261	14793	23605
(5.7.3.2)	Flexural moment Checking		OK	OK	OK	OK	OK
	Limits for reinforcement						
c/de	Maximum reinforcement		0.08	0.08	0.08	0.04	0.04
	Maximum reinforcement Checking	≤ 0.42	OK	OK	OK	OK	OK
1.2*Mcr	Cracking moment	kNm	18844	18844	18844	18088	18088
(5.7.3.3.2)	Checking $M_r \geq \min(1.2M_{cr}, 1.33M_u)$		OK	OK	OK	OK	OK
(5.7.3.4)	Control of cracking by distr. of reinf for RC member- Check?		Yes	No	No	No	No
	Existing condition for structure	1,2 or 3	3	3	3	3	3
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.064	0.064	0.064	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	m ²	0.010	0.010	0.010	0.019	0.019
f _{sa}	Value	Mpa	206	206	206	163	163
0.6*f _y	Tensile stress in reinf Min(f _{sa} , 0.6f _y)	Mpa	240	240	240	240	240
x	Dist. From compression fiber to centroid	m	0.406	-	-	-	-
J.d	Arm	m	1.701	-	-	-	-
I _{cr}	Moment of inertia of the cracked section	m ⁴	1.762	-	-	-	-
f _s	Tensile stress in reinforcement $f_s = M_{sl} / (A_s * J.d)$	Mpa	164	-	-	-	-
(5.10.8.2)	Shrinkage and temperature Reinforcement (side distribution)						
A _{req}	Area of required reinf	m ²	0.00127	0.00127	0.00127	0.00127	0.00127
	Distribution on sides	m ²	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	2.2	2.2	1.9
θ	Angle of inclination of diagonal compressive	degree	36.18	40.07	36.08	37.17	41.58
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	90
b _v	Effective web width as minimum web width - in dv	m	12.600	12.600	12.600	12.600	12.600
d _v	Effective shear depth	m	1.772	1.772	1.772	1.884	1.884
	(dc - a/2)	m	1.772	1.772	1.772	1.884	1.884
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	0.600
n _{cat}	Amount of bars in spacing S	bars	20	20	20	20	20
A _v	Shear reinf area in spacing S	m ²	0.0040	0.0040	0.0040	0.0040	0.0040
θ	Assume	degree	36.18	40.07	36.08	37.17	41.58
v	Shear stress in concrete	kN/m ²	385	615	364	33	464
f _{po}	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e _x	Strain in tensile reinforcement		1.02E-03	1.41E-03	1.01E-03	1.12E-03	1.64E-03
	if $e_x < 0$, multiple with reduce factor		-	-	-	-	-
	Strain checking	$\leq 2.00E-3$	Ok	Ok	Ok	Ok	Ok
v/f _c	Ratio of shear stress and f _c		0.013	0.021	0.012	0.001	0.015
β	Final value		2.2	2.0	2.2	2.2	1.9
θ	Final value	degree	36.18	40.07	36.08	37.17	41.58
V _c	Nominal shear resistance provided by tensile stresses in the concrete	kN	22525	20316	22584	23355	20324
V _s	Shear resistance provided by shear reinforcement	kN	6523	5672	6548	6691	5719
V _p	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0	0
V _{n1}	V _{n1} = V _c + V _s + V _p	kN	29048	25987	29131	30046	26043
V _{n2}	V _{n2}	kN	167414	167414	167414	178018	178018
V _n	Nominal shear resistance V _n = min(V _{n1} , V _{n2})	kN	29048	25987	29131	30046	26043
V _r	Factored shear resistance	kN	29048	23389	29131	30046	23439
V _u	Shear	kN	8597	12357	8124	6874	9902
(5.8.2.7)	Shear checking		OK	OK	OK	OK	OK

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	OP18A BRIDGE	Design			
	DETAIL DESIGN	Check			
	ABUTMENT A1L	Revise			

22TCN272-05; AASHTO LRFD 2nd - 1998

REINFORCEMENT CHECKING - WING WALL

MATERIALS				
NORMAL CONCRETE				
f _c	Compressive Strength of concrete at 28 days	Mpa	30	
E _c	Modulus of Elasticity	Mpa	27691	
f _r	Modulus of Rupture	Mpa	3.5	
g _c	Unit weight of concrete	kN/m ³	24.5	
PRESTRESSING STEEL				
f _{pu}	Tensile strength of prestressing steel	Mpa	1860	
f _{py}	Yield strength of prestressing steel	Mpa	1670	
E _p	Modulus of Elasticity	Mpa	197000	
REINFORCEMENT				
f _y	Yield strength	Mpa	400	
E _s	Modulus of Elasticity	Mpa	200000	
n _c	Ratio E _s /E _c		7	

Sign	Parameters	Unit	Sections				
			5-5	5-5	6-6	6-6	6-6
INTERNAL FORCES AT SECTION							
	Combination		Service	Strength	Service	Strength	Strength
Q _u	Shear	kN	102	157	250	387	387
M _u	Flexural Moment	kNm	246	392	275	426	426
N _u	Axial load	kN	0	0	0	0	0
T _u	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
d _{lx}	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
d _l x _p	Dis. From tens. fiber to centroid of tension prestressing steel	m	0.000	0.000	0.000	0.000	0.000
d _{ps}	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
b _w	Web width or diameter of a circular section	m	1.000	1.000	1.000	1.000	1.000
h _f	Compression flange depth	m	0.000	0.000	0.000	0.000	0.000
I _z	Moment of inertia of section	m ⁴	0.043	0.043	0.043	0.043	0.043
A _{mc}	Section area	m ²	0.800	0.800	0.800	0.800	0.800
	Steel choice						
A _{ps}	Tension prestressing steel	P.S type	0	0	0	0	0
		Number	tendons	0	0	0	0
		Area	m ²	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
		Area	m ²	0.00000	0.00000	0.00000	0.00000
A _s	Tension Reinforcement	Number	bars	6	6	6	6
		Diameter	mm	22	22	22	22
		Area	m ²	0.00228	0.00228	0.00228	0.00228
A' _s	Compression Reinforcement	Number	bars	6	6	6	6
		Diameter	mm	16	16	16	16
		Area	m ²	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	m ²	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
φ _v	Resistance factors for shear		1.00	0.90	1.00	0.90	0.90
φ _n	Resistance factors for axial force		1.00	1.00	1.00	1.00	1.00
β ₁	Stress block factor		0.836	0.836	0.836	0.836	0.836
c	Dis. Between centroid and top fiber	m	0.020	0.020	0.020	0.020	0.020
	For T section behavior	m	0.020	0.020	0.020	0.020	0.020
	For rectangular section behavior	m	0.020	0.020	0.020	0.020	0.020
f _{pe}	Effective stress in the prestressing steel after losses	Mpa	1116	1116	1116	1116	1116
f _{ps}	Aver. stress in pres. steel at the time for which the nominal resistance	Mpa	1847	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	0.28

Da Nang Quang Ngai Expressway project OP18A BRIDGE DETAIL DESIGN ABUTMENT AIL			Item.	Eng.	Date.	Sign.
			Design			
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REINFORCEMENT CHECKING - WING WALL							
a	Depth of equivalent stress block	m	0.017	0.017	0.017	0.017	0.017
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	644	644	644	644	644
Mr	Factored resistance	kNm	644	580	644	580	580
Mu	Flexural moment	kNm	246	392	275	426	426
(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	Minimum reinforcement		0.29%	0.29%	0.29%	0.29%	0.29%
	Minimum reinforcement Checking for RC	0.23%	OK	OK	OK	OK	OK
1.2*Mcr	Cracking moment	kNm	227	227	227	227	227
(5.7.3.3.2)	Checking $Mr \geq \min(1.2Mcr, 1.33Mu)$		OK	OK	OK	OK	OK
(5.7.3.4) Control of cracking by distr. of reinf for RC member- Check?							
	Existing condition for structure	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	m ²	0.020	0.020	0.020	0.020	0.020
f _{sa}	Value	Mpa	285	285	285	285	285
0.6*f _y		Mpa	240	240	240	240	240
	Tensile stress in reinf Min(f _{sa} , 0.6f _y)	Mpa	240	240	240	240	240
x	Dist. From compression fiber to centroid	m	0.139	-	0.139	-	-
J.d	Arm	m	0.695	-	0.695	-	-
I _{cr}	Moment of inertia of the cracked section	m ⁴	0.007	-	0.007	-	-
f _s	Tensile stress in reinforcement $f_s = M_{sls} / (A_s * J.d)$	Mpa	155	-	174	-	-
	Checking for control cracking $f_s < f_{sa}$		OK	N.a	OK	N.a	N.a
(5.10.8.2) Shrinkage and temperature Reinforcement (side distribution)							
A _{req}	Area of required reinf	m ²	0.00042	0.00042	0.00042	0.00042	0.00042
	Distribution on sides 7 D16	m ²	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	2.0	2.1	1.8	1.8
θ	Angle of inclination of diagonal compressive	degree	34.77	39.79	37.77	41.99	41.99
α	Angle of inclination of transv. reinf. to long Axis	degree	90	90	90	90	90
b _v	Effective web width as minimum web width in dv	m	1.000	1.000	1.000	1.000	1.000
d _v	Effective shear depth	m	0.733	0.733	0.733	0.733	0.733
	(d _e - a/2)	m	0.733	0.733	0.733	0.733	0.733
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	0.600
n _{cat}	Amount of bars in spacing S	bars	2	2	2	2	2
A _v	Shear reinf area in spacing S	m ²	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	34.77	39.80	37.77	41.99	41.99
v	Shear stress in concrete	kN/m ²	139	238	341	587	587
f _{po}	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e _s	Strain in tensile reinforcement		8.97E-04	1.38E-03	1.18E-03	1.75E-03	1.75E-03
	if e _x < 0, multiple with reduce factor		-	-	-	-	-
	Strain checking	$\leq 2.00E-3$	Ok	Ok	Ok	Ok	Ok
v/f _c	Ratio of shear stress and f _c		0.005	0.008	0.011	0.020	0.020
β	Final value		2.3	2.0	2.1	1.8	1.8
θ	Final value	degree	34.77	39.79	37.77	41.99	41.99
V _c	Nominal shear resistance provided by tensile stresses in the concrete	kN	762	672	710	612	612
V _s	Shear resistance provided by shear reinforcement	kN	159	133	143	123	123
V _p	Component in the direction of the applied shear of the effective P.S.	kN	0	0	0	0	0
V _{n1}	V _{n1} = V _c + V _s + V _p	kN	921	805	852	734	734
V _{n2}	V _{n2}	kN	5495	5495	5495	5495	5495
V _n	Nominal shear resistance V _n = min(V _{n1} , V _{n2})	kN	921	805	852	734	734
V _r	Factored shear resistance	kN	921	724	852	661	661
V _u	Shear	kN	102	157	250	387	387
(5.8.2.7)	Shear checking		OK	OK	OK	OK	OK

ALL_OUT

SPACE PILE FOUNDATION ANALYSIS PROGRAM
Turbo BASIC

PROJECT: : OP18a-A1L

INITIAL DATA

Kn = 0.00 Ax = 9.14 By = 12.79 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 = 400 (t/m4)

LOAD COMBINATIONS

COMB.	Hx	Hy	P	Mx	My	Mz
1	886.00	0.00	3289.00	-930.00	1193.00	0.00
2	583.00	0.00	2385.00	-754.00	789.00	0.00
3	860.00	6.00	3217.00	-923.00	1065.00	0.00
4	558.00	6.00	2314.00	-747.00	660.00	0.00
5	602.00	5.00	2489.00	-727.00	712.00	0.00
6	669.00	43.00	3066.00	-987.00	-357.00	0.00
7	669.00	43.00	2162.00	-811.00	581.00	0.00

PROPERTIES OF PILES

PILE	Lo	H	Bpx	Bpy	A	B	Cday	Fo	Io	Po	Co	Ct
1	0.00	13.50	1.057	1.057	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	5.76	0.000	0.00
2	3.30	0.73	0.000	0.00
3	3.30	-4.29	0.000	0.00
4	0.00	5.18	0.000	0.00
5	0.00	0.15	0.000	0.00
6	0.00	-4.87	0.000	0.00
7	-3.30	4.60	0.000	0.00
8	-3.30	-5.46	0.000	0.00

DISPLACEMENTS

COMB.	X	Y	Z	Fix	Fiy	Fiz
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A1L_OUT

1	0.01862	-0.00008	0.001704	0.000009	0.000229	-0.000136
2	0.01223	-0.00005	0.001244	0.000003	0.000143	-0.000089
3	0.01805	0.00004	0.001671	0.000007	0.000214	-0.000130
4	0.01167	0.00008	0.001211	0.000001	0.000128	-0.000084
5	0.01260	0.00005	0.001303	0.000003	0.000139	-0.000091
6	0.01364	0.00086	0.001652	-0.000009	0.000051	-0.000090
7	0.01400	0.00084	0.001116	-0.000004	0.000157	-0.000090

FORCES ON PILES

PILE	COMB.	N	Q2	Q3	M1	M2	M3
1	1	551.93	-106.08	-2.43	-3.180	-8.671	350.693
	2	389.71	-69.80	-1.60	-2.093	-5.567	231.100
	3	535.09	-103.03	-3.08	-3.047	-10.798	340.984
	4	373.08	-66.87	-2.25	-1.963	-7.705	221.790
	5	399.00	-72.13	-2.25	-2.128	-7.817	239.201
	6	428.76	-80.52	-7.00	-2.118	-23.387	271.754
	7	378.51	-80.52	-7.00	-2.118	-23.641	266.971
2	1	561.73	-110.32	-2.43	-3.180	-8.671	365.117
	2	392.63	-72.59	-1.60	-2.093	-5.567	240.590
	3	542.87	-107.09	-3.08	-3.047	-10.798	354.805
	4	374.00	-69.48	-2.25	-1.963	-7.705	230.695
	5	402.71	-74.96	-2.25	-2.128	-7.817	248.852
	6	417.89	-83.34	-7.00	-2.118	-23.387	281.362
	7	374.11	-83.34	-7.00	-2.118	-23.641	276.578
3	1	571.52	-114.56	-2.43	-3.180	-8.671	379.540
	2	395.56	-75.38	-1.60	-2.093	-5.567	250.081
	3	550.64	-111.15	-3.08	-3.047	-10.798	368.626
	4	374.93	-72.10	-2.25	-1.963	-7.705	239.600
	5	406.42	-77.80	-2.25	-2.128	-7.817	258.503
	6	407.03	-86.16	-7.00	-2.118	-23.387	290.970
	7	369.71	-86.16	-7.00	-2.118	-23.641	286.186
4	1	379.82	-106.57	0.35	-3.180	0.799	352.363
	2	281.70	-70.13	0.23	-2.093	0.664	232.198
	3	374.25	-103.50	-0.42	-3.047	-1.724	342.584
	4	276.22	-67.17	-0.54	-1.963	-1.858	222.821
	5	294.33	-72.46	-0.39	-2.128	-1.481	240.319
	6	389.16	-80.84	-5.14	-2.118	-17.079	272.867
	7	259.81	-80.84	-5.14	-2.118	-17.333	268.083
5	1	389.61	-110.81	0.35	-3.180	0.799	366.787
	2	284.62	-72.92	0.23	-2.093	0.664	241.689
	3	382.02	-107.56	-0.42	-3.047	-1.724	356.405
	4	277.14	-69.79	-0.54	-1.963	-1.858	231.726
	5	298.04	-75.29	-0.39	-2.128	-1.481	249.970
	6	378.30	-83.67	-5.14	-2.118	-17.079	282.474
	7	255.41	-83.67	-5.14	-2.118	-17.333	277.691
6	1	399.40	-115.05	0.35	-3.180	0.799	381.210
	2	287.55	-75.70	0.23	-2.093	0.664	251.180
	3	389.79	-111.62	-0.42	-3.047	-1.724	370.226
	4	278.06	-72.40	-0.54	-1.963	-1.858	240.631
	5	301.75	-78.13	-0.39	-2.128	-1.481	259.621
	6	367.44	-86.49	-5.14	-2.118	-17.079	292.082
	7	251.01	-86.49	-5.14	-2.118	-17.333	287.298
7	1	207.70	-107.06	3.13	-3.180	10.268	354.033
	2	173.69	-70.45	2.06	-2.093	6.895	233.297
	3	213.40	-103.97	2.25	-3.047	7.350	344.184
	4	179.36	-67.47	1.18	-1.963	3.988	223.852
	5	189.66	-72.78	1.47	-2.128	4.855	241.436
	6	349.57	-81.17	-3.29	-2.118	-10.771	273.979
	7	141.11	-81.17	-3.29	-2.118	-11.025	269.195
8	1	227.29	-115.54	3.13	-3.180	10.268	382.880

				ALL_OUT		
2	179.54	-76.03	2.06	-2.093	6.895	252.279
3	228.94	-112.09	2.25	-3.047	7.350	371.826
4	181.20	-72.71	1.18	-1.963	3.988	241.662
5	197.08	-78.45	1.47	-2.128	4.855	260.738
6	327.84	-86.82	-3.29	-2.118	-10.771	293.194
7	132.31	-86.82	-3.29	-2.118	-11.025	288.411

SUMMARY OF FORCES

	PILE	COMB.	N	Q2	Q3	M1	M2	M3
Nmin	8	7	132.31	-86.82	-3.29	-2.118	-11.025	288.411
Nmax	3	1	571.52	-114.56	-2.43	-3.180	-8.671	379.540
Q2max	8	1	227.29	-115.54	3.13	-3.180	10.268	382.880
Q3max	1	6	428.76	-80.52	-7.00	-2.118	-23.387	271.754
M1max	1	1	551.93	-106.08	-2.43	-3.180	-8.671	350.693
M2max	1	7	378.51	-80.52	-7.00	-2.118	-23.641	266.971
M3max	8	1	227.29	-115.54	3.13	-3.180	10.268	382.880

CHECKING CALCULATI IN COMPARISON WITH INITIA LOAD MATRIX

1	886.00	0.00	3289.00	-930.00	1193.00	0.00
2	583.00	0.00	2385.00	-754.00	789.00	0.00
3	860.00	6.00	3217.00	-923.00	1065.00	0.00
4	558.00	6.00	2314.00	-747.00	660.00	0.00
5	602.00	5.00	2489.00	-727.00	712.00	0.00
6	669.00	43.00	3066.00	-987.00	-357.00	0.00
7	669.00	43.00	2162.00	-811.00	581.00	0.00

CALCULATION SHEET
ABUTMENT A1R

Table of content

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

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LOAD COMPONENTS

Assumptions :

1. Bridge is considered to be in seismic with acceleration coeff. $A = 0.0580 \text{ g}$
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 :

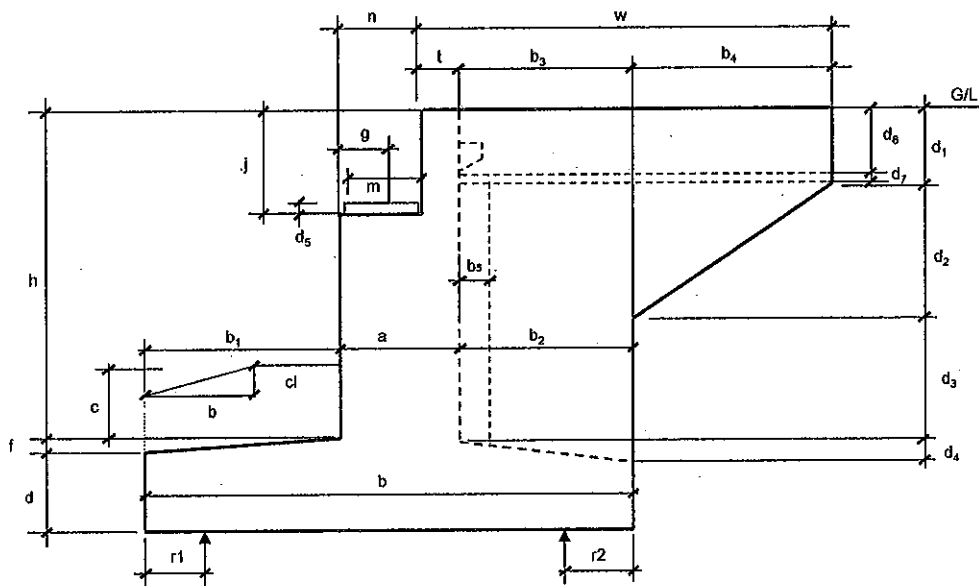
Level Table(at center of abutment)

Level of top of headwall	HTwL	16.770	m
Level of top of bearing	BTL	13.951	m
Level of top of stem abutment	HTL	13.655	m
Level of top of footing	FTL	6.500	m
Level of bottom of footing	FBL	4.500	m
Ground level	GL	7.816	m
Lowest water level	HWL	4.500	m
Skew angle	α	10.00	deg

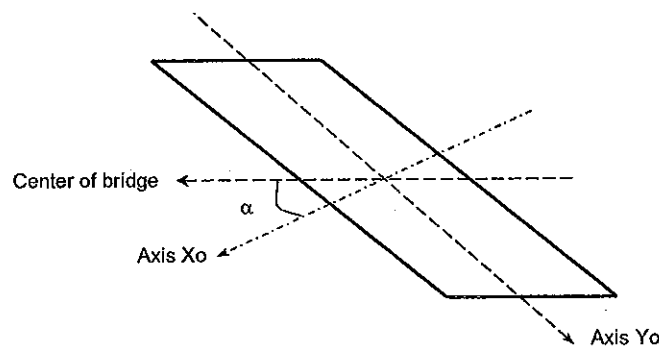
I. Loads from substructure

Abutment dimensions

VERTICAL VIEW



PLAN VIEW



Material Unit Weights

- Unit Weight of Reinf. concrete
- Unit Weight of Soil
- Unit Bouyancy Weight of Soil

γ_c	=	24.5 kN/m ³
γ_s	=	18.0 kN/m ³
γ_{sbo}	=	8.2 kN/m ³

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	ABUTMENT A1R	Revise			

ABUTMENT DIMENSIONS (IN METRES)					
Item	Symbol	Value	Item	Symbol	Value
Height of stem	h	9.266	Horizontal Dimension	b ₃	3.700
Footing Width	b	9.000	Horizontal Dimension	b ₄	3.800
Stem Width	a	1.800	Horizontal Dimension	b ₅	0.300
Footing Depth	d	2.000	Vertical Dimension	d ₁	0.839
Footing Slope	f	0.000	Vertical Dimension	d ₂	3.800
Width of stem at bearing	n	1.300	Vertical Dimension	d ₃	4.627
Ballast Wall Height	j	2.114	Vertical Dimension	d ₄	0.000
Ballast Wall Thickness	t	0.500	Vertical Dimension	d ₅	0.296
Wingwall Length	w	8.000	Vertical Dimension	d ₆	0.900
Soil Cover at Toe	c	1.316	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

Slope front of abutment

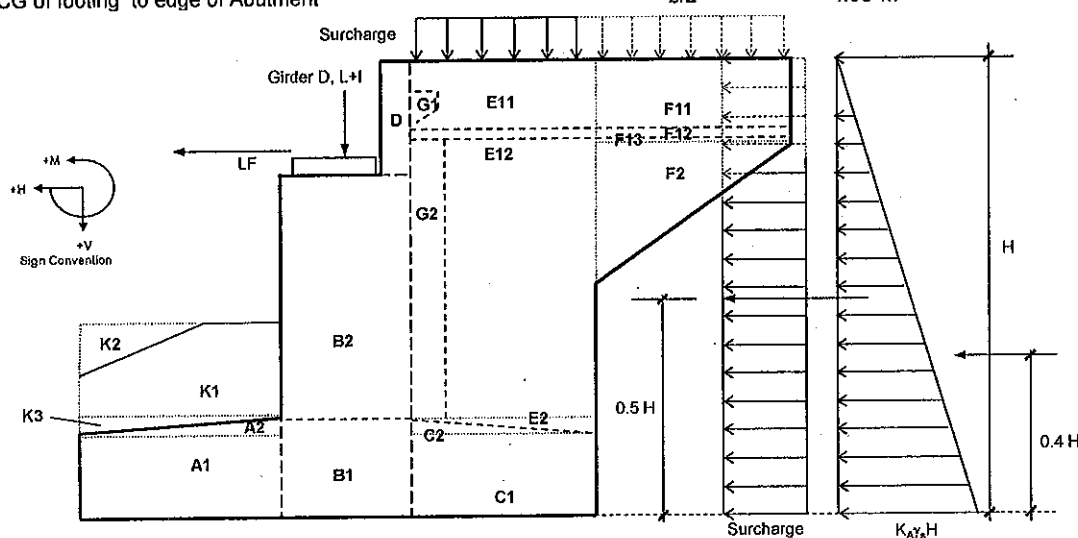
Width of Abutment

Width of abutment (inclined direction)

Height of Abutment

Distance from CG of footing to edge of Abutment

$$\begin{aligned}\cos(\alpha) &= 0.98 \\ cl &= 0.00 \text{ m} \\ bl &= 0.00 \text{ m} \\ L &= 23.280 \text{ m} \\ Ltr &= 23.639 \text{ m} \\ Hl &= 11.27 \text{ m} \\ b/2 &= 4.50 \text{ m}\end{aligned}$$



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	23.639	4054	1.750	2.750	11149
Section A2	-	23.639	-	2.333	2.167	-
Section B1	3.600	23.639	2085	4.400	0.100	208
Section B2	12.874	23.639	7456	4.400	0.100	746
Section C1	7.400	23.639	4286	7.150	-2.650	-11357
Section C2	-	23.639	-	6.533	-2.033	-
Section D	1.057	23.639	612	5.050	-0.550	-337
Section E11	3.330	0.500	41	7.150	-2.650	-108
Section E12	29.844	0.800	585	7.150	-2.650	-1550
Section F11	3.420	0.500	42	10.900	-6.400	-268
Section F12	1.125	0.650	18	9.050	-4.550	-82
Section F13	-	0.800	-	10.900	-6.400	-
Section F2	7.220	0.800	142	10.267	-5.767	-816
Section G1	0.135	22.639	317	5.450	-0.950	-302
Section G2	0.045	16.132	18	5.450	-0.950	-17
Bearing seats (w1seat= 0.85m)	0.266	4.250	31	4.350	0.150	5
Curbs + Handrail on Abutment	0.50	8.000	106	8.800	-4.300	-456
Total SW of Abutment (DC)			19793			-3185
Transverse moment			809		11.440	9257

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.

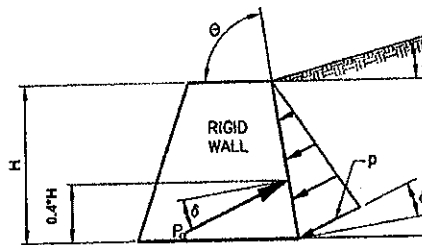
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2. Earth on Abutment (EV)

Description	Area (m ²)	Length (m)	Force (kN)	x ⁽¹⁾ (m)	Arm ⁽²⁾ (m)	Moment (kN·m)
Earth on Abutment (EV)						
Section E1	34.28	22.039	13601	7.150	-2.650	-36042
Section E2	-	22.039	-	7.767	-3.267	-
Section E3	-	1.600	-	9.000	-4.500	-
Section K1	4.606	23.639	1960	1.750	2.750	-
Section K2	-	23.639	-	-	4.500	-
Section K3	-	23.639	-	1.167	3.333	-
Total Earth on Footing			15561			-36042

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
- Width for horizontal earth pressure
- Density of Soil
- Internal Friction Angle of Soil
- Incline angle of back face wall
- Friction angle between fill and wall
- Incline angle of fill soil
- Gravitational acceleration
- Basic earth pressure

$$p = K \cdot \gamma_s \cdot Z \cdot 10^{-9} \text{ (Mpa, Z: mm)}$$

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

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

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

Horizontal earth pressure:

- $E_a = 0.5 \cdot p \cdot Z \cdot B \cdot 10^3 \text{ (kN)}$
- $M = E_a \cdot 0.4H$
- Horizontal Earth Pressure act at a height of $0.4H$

H	=	11.27 m
W	=	23.64 m
γ_s	=	1835 kg/m ³
ϕ'_f	=	30.0 deg
θ	=	90.0 deg
δ	=	0.0 deg
β	=	0.0 deg
g	=	9.81 m/s ²

Γ	=	2.250
K_a	=	0.333
p	=	0.068 Mpa

E_a	=	9001 kN
M	=	40562 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=	11.27m heq=	0.61 m

(Linear interpolation)

- Vertical force

ESv	=	960 kN
ev	=	-2.65 m
M	=	-2545 kNm

- Horizontal force

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

ESh	=	975 kN
eh	=	5.63 m
M	=	5491 kNm

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5. Earthquake effects

Bridge is located at: Tam Hiep district - Quang Ngai province
According to TCXDVN 375:2008 and 22TCN272-05, bridge is in seismic zone 2 and acceleration coefficient as below
• Peak ground acceleration coefficient $A = 0.0580 \text{ g}$

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

- Backfill slope angle $i = 0.0 \text{ deg}$
- Slope of wall to vertical $\beta' = 0.0 \text{ deg}$
- Angle of friction of soil $\phi = 30.0 \text{ deg}$
- Angle of friction between soil and abutment $\delta = 0.0 \text{ deg}$
- Horizontal acceleration coefficient $k_h = 0.087$
- Vertical acceleration coefficient $k_v = 0.035$
- Angle $\theta = \arctan(k_h / (1 - k_v)) = 5.2 \text{ deg}$

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

• Seismic active lateral Earth pressure coefficient $K_{AE} = 0.39$

$$E_{AE} = 0.5 \cdot g \cdot \gamma \cdot H^2 \cdot (1 - k_v) \cdot K_{AE} \cdot 10^{-9} \text{ (kN/m)}$$

• Seismic active lateral Earth pressure coefficient
 $M_{AE} = E_{AS} \cdot 0.3H + (E_{AE} - E_{AS}) \cdot 0.6H$

$E_{AE} = 10159 \text{ kN}$
 $M_{AE} = 38250 \text{ KNm}$

<A.11.1.1.1>

E_{AS} is the static component of seismic active pressure calculated with $\theta = k_v = 0$

5.2. Earthquake effects to abutment (EQ)

Seismic force for substructures: elements above ground $F_h = C_{sm} \cdot W$; elements under ground $F_h = A \cdot S \cdot W$

- Soil profile type
- Site Coefficients.
- Elastic Seismic Response Coefficient
 $C_{sm} = 1.2 \cdot A \cdot S / T_m^{2/3} \leq 2.5 \cdot A$
- Period of vibration of the fundamental mode
 $T_m = 2 \cdot \pi \cdot \sqrt{m/k}$

Soil type I
 $S = 1.0$
 $2.5A = 0.145$
 $C_{sm} = 0.066$
 $T_m = 1.090 \text{ s}$

Description	Area (m ²)	Length (m)	Force (kN)	X ⁽¹⁾ (m)	Arm ⁽²⁾ (m)	Moment (kN·m)
Section A1	7.000	23.639	235	-	1.000	235
Section A2	-	23.639	-	-	2.000	-
Section B1	3.600	23.639	121	-	1.000	121
Section B2	12.874	23.639	490	-	5.576	2732
Section C1	7.400	23.639	249	-	1.000	249
Section C2	-	23.639	-	-	2.000	-
Section D	1.057	23.639	40	-	10.209	411
Section E11	3.330	0.500	2	-	8.816	21
Section E12	29.844	0.800	34	-	4.033	-
Section E2	-	-	-	-	2.000	-
Section F11	3.420	0.500	2	-	8.816	21
Section F12	1.125	0.650	1	-	8.216	-
Section F13	-	0.800	-	-	9.447	-
Section F2	7.220	0.800	8	-	9.160	75
Section G1	0.135	22.639	4	-	8.553	37
Section G2	0.045	16.132	1	-	4.033	4
Total EQ of Abutment Selfweight			1188			3906

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6. Braking Force(BR)

Take

50 % Braking Force for this Abutment (Free Bearing)

- Number of lanes
- Multiple presence factor
- Take 25 % of Truck load
- BR = 25% * n * m * (2*145+35)
- Acting at 1.8m higher of road face

n	=	6 lanes	
m	=	0.65	
BR	=	158 kN	Long. Axis
e	=	13.1 m	
Mlong	=	2081 KNm	Long. Axis

7. Centrifugal Force, CE (3.6.3)

- Plan of bridge (1: "straight", 2: "Curve")
- Design Speed

$$C = 4/3 * (V^2 / gR)$$

Acting at 1.8m higher of road face

$$CE = n * m * (2*145+35) * C$$

	=	1	
V	=	120 km/h	
V	=	33.3 m/s	
R	=	- m	
C	=	-	
CE	=	0.00 KN	
e	=	13.15 m	
Mtrans	=	0.00 KNm	Trans. Axis

8. Water Load (WA)

:NA

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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	10.00	deg
Deck slab length	Ldeck	26.85	m
Bridge Width	Bc	21.58	m
Girder height	hgl	1.50	m
Deck slab depth	hdkslab	0.218	m
Asphalt depth	has	0.084	m
Unit weight of concrete	yc	24.50	kN/m3
Unit weight of asphalt concrete	ya	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	9	Girders
Sum of girders weight	DC	4182.89	kN
Precast Planks	DC	788.86	kN
Diaphragm	DC	2352.77	kN
Total	DC	7324.52	kN
1.2. Deck slab			
Deck slab	DC	3016.63	kN
1.3. Pavement			
Asphalt concrete	DW	975.96	kN
1.4. Handrail			
Handrail + median	DC	639.90	kN

2. Live load (LL):

Truck	
Tandem	
Lane load	
Pedestrian	Wpd = 0.0 kN/m2
Considerate structure as a simple span	
Reaction Influence	
Number of lanes	n = 6
Multiple presence factor	m = 0.65
Dynamic load allowance	1+IM = 1.25

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

	P1(kN)	P2(kN)	P3(kN)	Sum(kN)	Total(kN)
Truck					
Axle load	145	145	35		
Influence value	1.000	0.835	0.670		
Reaction	145.0	121.1	23.5	289.6	1411.7

	P1(kN)	P2(kN)	Sum(kN)	Total(kN)
Tandem				
Axle load	110	110		
Influence value	1.000	0.954		
Reaction	110	104.9	214.9	1047.8

Lane load	Wl(kN/m)	Total(kN)
Value	9.3	
Influence value	13.05	
Reaction	121.4	473.3

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

3. Earthquake effects on superstructure (EQ)

Longitudinal moveable bearings at Abutment

Horizontal force from superstructure due to EQ - transverse direction
At bearing

$$Heq = 393 \text{ kN}$$

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

Bearing displacement due to uniform temperature and shrinkage creep

$$H = G.A.\Delta u/h_n$$

Shear modulus G

Bearing area

Height of elastomeric layers

Number of bearing

Horizontal force due to TU+SH&CR

Acting at top of bearing

$$\Delta u = 0.026 \text{ m}$$

<14.6.3.1-2>

$$G = 1 \text{ MPa}$$

$$A = 0.158 \text{ m}^2$$

$$h_{rt} = 0.064 \text{ m}$$

$$n_b = 9 \text{ bears}$$

$$H(tu+sh+cr) = 576 \text{ kN}$$

5. Wind loads (Ws)

5.1. Transverse wind on superstructure (WS)

Wind zone

Basic 3 second gust wind

Correction factor

Design wind velocity

Drag coefficient

Overall width of bridge

Depth of superstructure (including solid parapet)

Windy obstructed area of superstructure

Force due to transverse wind

$$F_{hy} = \max(0.0006 * V^2 * A_t * C_d, 1.8 * A_t) \text{ (kN)}$$

Zone

III

$$V_b = 53.00 \text{ m/s}$$

$$S = 1.09$$

$$V = 57.77 \text{ m/s}$$

$$C_d = 1.19$$

$$b = 21.58 \text{ m}$$

$$d = 2.78 \text{ m}$$

$$b/d = 7.75$$

$$A_t = 75.14 \text{ m}^2$$

$$F_{hy} = 179.8 \text{ kN}$$

<3.8.1>

5.2. Wind load on vehicles (WL)

Transverse wind on vehicles

Transverse horizontal force due to wind on live load

At 1.8m from surface

$$W_{ltran} = 1.50 \text{ kN/m}$$

$$F_{hy} = 40.50 \text{ kN}$$

6. Combinations

Loads from superstructure to Abutment

Loads at bottom of stem		Vertical		Longitudinal			Transversal		
	Sign	N (kN)	x (m)	Hx (kN)	z ₁ (m)	My (kN·m)	Hy (kN)	y (m)	Mx (kN·m)
Girder + Decks slab	DC	5171	0.05			259			
Handrail	DC	320	0.05			16			
Pavement	DW	488	0.05			24			
Live Load	LL	1885	0.05			94		3.54	6673
Pedestrian	PL	0	0.05			0			
Trans. wind on Struc.	WS						90	7.15	643
Trans. wind on vehi.	WL						20	8.95	181
Earthquake	EQ						393	7.15	2809
TU+SH&CR	TU+SH&CR			576	7.15	4119			

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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 + Decksab	DC	5171	0.15			776			
Handrail	DC	320	0.15			48			
Pavement	DW	488	0.15			73			
LiveLoad	LL	1885	0.15			283		3.54	6673
Pedestrial	PL	0	0.15			0			
Trans. wind on Struc.	WS						90	9.15	823
Trans. wind on vehi.	WL						20	10.95	222
Eearth quake	EQ						393	9.15	3595
TU+SH&CR	TU+SH&CR			576	9.15	5270			

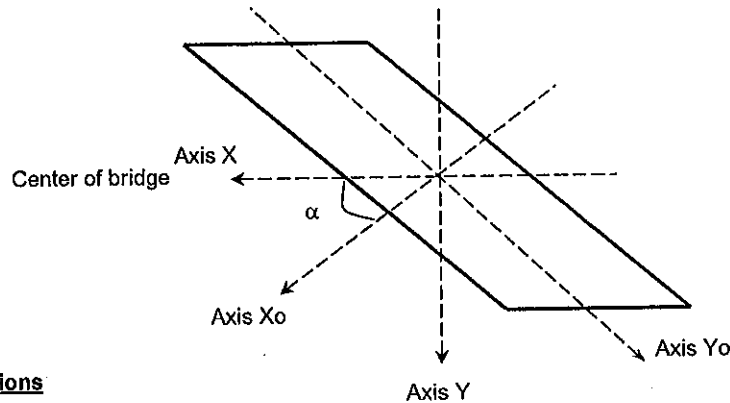
Loads		Load combinations						
Sign		Str-IA	Str-IB	Str-IIIA	Str-IIIB	Ser-I	Ext-IA	Ext-IB
Girder + Decksab	DC	1.25	0.90	1.25	0.90	1.00	1.25	0.90
Handrail	DC	1.25	0.90	1.25	0.90	1.00	1.25	0.90
Pavement	DW	1.50	0.65	1.50	0.65	1.00	1.50	0.65
LiveLoad	LL	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Pedestrial	PL	1.75	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					
Load combinations	N (kN)	Longitudinal		Tranversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	10894	288	2604	0	11678
Strength Str-IB	8557	288	2487	0	11678
Strength Str-IIIA	10140	288	2566	56	9447
Strength Str-IIIB	7803	288	2449	56	9447
Service Ser-I	7864	576	4512	47	7047
Extreme Ext-IA	8538	0	427	393	6146
Extreme Ext-IB	6201	0	310	393	6146

Load combinations at bottom of pilecap					
Load combinations	N (kN)	Longitudinal		Tranversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	10894	288	4269	0	11678
Strength Str-IB	8557	288	3919	0	11678
Strength Str-IIIA	10140	288	4156	56	9559
Strength Str-IIIB	7803	288	3806	56	9559
Service Ser-I	7864	576	6450	47	7142
Extreme Ext-IA	8538	0	1281	393	6931
Extreme Ext-IB	6201	0	930	393	6931

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LOAD COMBINATIONS



III. Load Combinations

1. Loads from substructure

Loads at bottom of pilecap	Sign	Vertical		Longitudinal			Transversal		
		N (kN)	x (m)	Hx (kN)	z ₁ (m)	My (kN.m)	Hy (kN)	y (m)	Mx (kN.m)
Self weight of Abutment	DC	19793				-3185			9256.63
Soils on pilecap	EV	15561				-36042			
Horizontal Earth Pressure	EH			9001		40562			
Vertical Surcharge	LSv	960				-2545			
Horizontal Surcharge	LSH			975		5491			
Braking Force	BR			158		2081			
Centrifugal Force	CE			-		-			-
Buoyancy of Abutment	WA	-				-			
Buoyancy of Earth on Abutment	WA	-				-			
Earthquake effects to Abutment	EQ			1188		3906	356		1172
Earthquake effects to soil	E _{AE}			10316		38840			

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
Centrifugal 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		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	47428	15485	17003	0	11571
Strength Str-IB	33498	10084	9999	0	8331
Strength Str-IIIA	47044	15031	14993	0	11571
Strength Str-IIIB	33114	9631	7989	0	8331
Service Ser-I	36313	10134	6363	0	9257
Extreme Ext-IA	46228	12071	-7378	356	12742
Extreme Ext-IB	32298	12071	9956	356	9503

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2. Loads from superstructure

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	10894	288	4269	0	11678
Strength Str-IB	8557	288	3919	0	11678
Strength Str-IIIA	10140	288	4156	56	9559
Strength Str-IIIB	7803	288	3806	56	9559
Service Ser-I	7864	576	6450	47	7142
Extreme Ext-IA	8538	0	1281	393	6931
Extreme Ext-IB	6201	0	930	393	6931

3. Total loads at bottom of pilecap

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	58322	15773	21272	0	23248
Strength Str-IB	42056	10372	13918	0	20009
Strength Str-IIIA	57184	15319	19149	56	21130
Strength Str-IIIB	40918	9919	11794	56	17890
Service Ser-I	44177	10710	12813	47	16398
Extreme Ext-IA	54765	12071	-6097	749	19674
Extreme Ext-IB	38499	12071	10886	749	16434

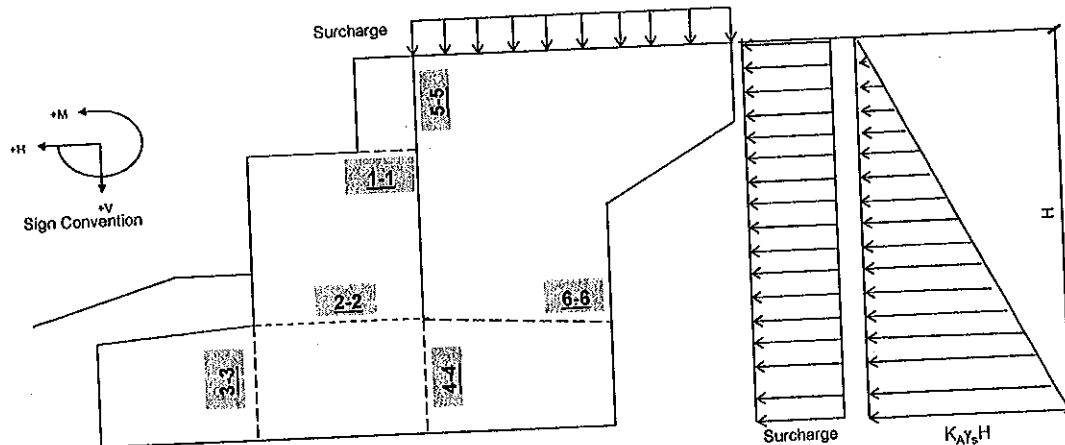
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ELEMENTS CHECKING

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 Ser-I	Strength I		Extreme 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	0.50
Surcharge (Horizontal)	LS _H	1.00	1.75	1.75	1.50
Horizontal Seismic Earth Pressure	E _{AE}				1.00
Abutment earthquake force	EQ				

Unfactored Loads	N (kN)	Longitudinal		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Selfweight	930		-127		
Horizontal Earth Pressure		322	272		
Surcharge (horizontal)		455	481		
Horizontal Seismic Earth Pressure		363	257		
Abutment earthquake force		45	47	13	14

Load Combination at bottom of headwall					
Load combinations	N (kN)	Longitudinal		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Service Ser-I	930	777	626	0	0
Strength Str-IA	1162	1279	1092	0	0
Strength Str-IB	837	1086	973	0	0
Extreme Ext-I	1162	817	514	13	14

1.2 Section 2-2

Table of Load Factors

Loads	Sign	Service I Ser-I	Strength I		Extreme 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	0.50
Surcharge (Horizontal)	LS _H	1.00	1.75	1.75	0.50
TU+SH&CR	TU+SH&CR	1.00	0.50	0.50	1.50
Horizontal Seismic Earth Pressure	E _{AE}				1.00
Abutment earthquake force	EQ				

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Unfactored Loads	N (kN)	Longitudinal		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Selfweight	8385		-477		
Superstructure Dead Load	5171		259		
Pavement	488		24		
Handrail+curb	320		16		
Live Load	1885		94		6673
Horizontal Earth Pressure		6183	22916		
Surcharge (Horizontal)		814	3772		
TU+SH&CR		576	4119		
Horizontal Seismic Earth Pressure		6978	21610		
Abutment earthquake force		534	2119	278	1513

Load Combination at bottom of stem wall					
Load combinations	N (kN)	Longitudinal		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Service Ser-I	16249	7573	30722	0	6673
Strength Str-IA	21378	10987	42981	0	11678
Strength Str-IB	18104	7277	29282	0	11678
Extreme Ext-I	19019	11409	36250	278	4850

1.3 Section 3-3

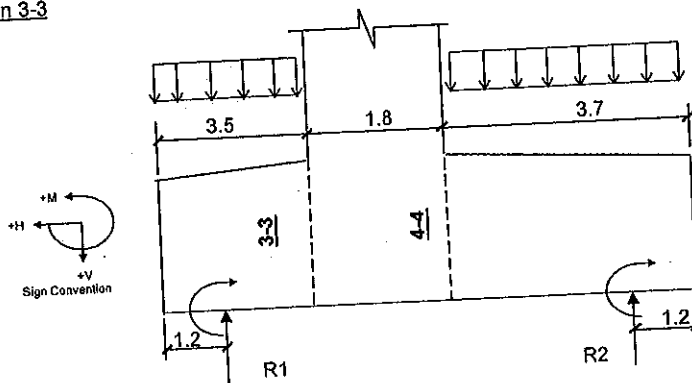


Table of Load Factors

Loads	Sign	Service I Ser-I	Strength I		Extreme 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	N (kN)	Longitudinal		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Selfweight at front side	-4054		-7095		
Vertical soil on foot at front side	-1960		-3430		
Reaction of piles	21915	4569	65705	-143	-514
Ser-I	30571	6729	92745	-181	-651
Str-IA	21245	4423	63647	-119	-423
Str-IB	22850	5149	70131	-449	-1549
Ext-I					

Load Combination at section 3-3					
Load combinations	N (kN)	Longitudinal		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Service Ser-I	15901	4569	55180	-143	-514
Strength Str-IA	22857	6729	79247	-181	-651
Strength Str-IB	15832	4423	54175	-119	-423
Extreme Ext-I	15137	5149	56632	-449	-1549

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1.4 Section 4-4

Table of Load Factors

Loads	Sign	Service I Ser-I	Strength I		Extreme I Ext-I
			Str-IA	Str-IB	
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		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Selfweight of behind side	-5113		-10124		
Vertical soil on foot at behind side	-13601		-25161		
Surcharge(Vertical)	-960		-1777		
Reaction of piles					
Ser-I	7161	2312	10159	84	272
Str-IA	8188	3405	9116	139	457
Str-IB	6523	2238	8825	92	303
Ext-I	12217	2603	21657	-60	-216

Load Combination at section 4-4					
Load combinations	N (kN)	Longitudinal		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Service Ser-I	-12513	2312	-26903	84	272
Strength Str-IA	-18244	3405	-40616	139	457
Strength Str-IB	-12000	2238	-26041	92	303
Extreme Ext-I	-13015	2603	-25854	-60	-216

1.4 Section 5-5 & 6-6

Slope of triang pressure
Uniform horizontal pressure

$$\begin{aligned} \tan \beta &= 6.00 \\ U.p &= 3.66 \text{ kN/m}^2 \end{aligned}$$

Load Combination at section 5-5					
Load combinations	N (kN)	Horizontal		Vertical	
		Hx (kN)	Mx (kN.m)	Hy (kN)	My (kN.m)
Service Ser-I		102	246		
Strength Str-IA		157	392		

Load Combination at section 6-6					
Load combinations	N (kN)	Horizontal		Vertical	
		Hx (kN)	Mx (kN.m)	Hy (kN)	My (kN.m)
Service Ser-I				250	275
Strength Str-IA				387	426

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2. Elements Checking

2.1. Bearing Resistance

<S.5.7.5>

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

$$Pr = \phi \cdot Pn = \phi \cdot 0.85 \cdot f_c \cdot A1 \cdot m$$

Dimension of bearing plate

w0	=	0.900 m
b0	=	0.850 m
A1	=	0.765 m ²
w	=	1.300 m
b	=	1.250 m
A2	=	1.625 m ²

Area under bearing device

Distributed width and length

Notational area

Where supporting surface is wider on all sides than loaded area

$$m = \sqrt{A2/A1} \leq 2.0 \quad \text{case 1}$$

where loaded area is subjected to nonuniformly distributed bearing

$$m = 0.75 \cdot \sqrt{A2/A1} \leq 1.5 \quad \text{case 2}$$

Modification factor

Resistance factor

Factored bearing resistance

Bearing reaction of approach bridge

$$Pu = 1.25 \cdot DC + 1.5 \cdot DW + 1.75 \cdot LL$$

m	=	1.457
ϕ	=	0.700
Pr	=	19902 kN
Pu	=	7430 kN

<S.5.5.4.2>

> Pu

Ok

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

$$Pr = \phi \cdot fn \cdot Ab$$

fn take the lesser of

$$fn = 0.7 \cdot fci \cdot \sqrt{A/Ag} \text{ and}$$

$$fn = 2.25 \cdot fci$$

fn	=	30.61 MPa
A	=	1.625 m ²
Ag	=	0.765 m ²
Ab	=	0.765 m ²
fci	=	30 MPa
Pr	=	16390 kN

Maximum area of the portion of supporting surface

Gross area of bearing plate

Effective net area of bearing plate, Ag minus stud of bearing

Nominal concrete strength at time of application

Factored bearing resistance

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				
E_c	Modulus of Elasticity	Mpa	27691				
f_r	Modulus of Rupture	Mpa	3.5				
g_c	Unit weight of concrete	kN/m3	24.5				
PRESTRESSING STEEL							
f_{pu}	Tensile strength of prestressing steel	Mpa	1860				
f_{py}	Yield strength of prestressing steel	Mpa	1670				
E_p	Modulus of Elasticity	Mpa	197000				
REINFORCEMENT							
f_y	Yield strength	Mpa	400				
E_s	Modulus of Elasticity	Mpa	200000				
n_c	Ratio E_s/E_c		7				
Sign	Parameters	Unit	Sections				
			1-1	1-1	2-2	2-2	2-2
INTERNAL FORCES AT SECTION							
	Combination		Strength	Service	Service	Strength	Extreme
Q_u	Shear	kN	1279	777	7573	10987	11409
M_u	Flexural Moment	kNm	1092	626	30722	42981	36250
N_u	Axial load	kN	1162	930	16249	21376	19019
T_u	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
d_s	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	23.280	23.280	23.280	23.280	23.280
bw	Web width or diameter of a circular section	m	23.280	23.280	23.280	23.280	23.280
hf	Compression flange depth	m	0.000	0.000	0.000	0.000	0.000
I_z	Moment of inertia of section	m ⁴	0.243	0.243	11.314	11.314	11.314
A_{mc}	Section area	m ²	11.640	11.640	41.904	41.904	41.904
	Steel choice						
A_{ps}	Tension prestressing steel	P.S type	0	0	0	0	0
		Number	0	0	0	0	0
		Area	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	0	0	0	0	0
		Area	0.00000	0.00000	0.00000	0.00000	0.00000
A_s	Tension Reinforcement	Number	152	152	146	146	146
		Diameter	16	16	28	28	28
		Area	0.03070	0.03070	0.08994	0.08994	0.08994
$A's$	Compression Reinforcement	Number	152	152	146	146	146
		Diameter	16	16	16	16	16
		Area	0.03070	0.03070	0.02949	0.02949	0.02949
$A'c$	Shear reinforcement	Number	37	37	36	36	36
		Diameter	14	14	14	14	14
		Area	0.00559	0.00559	0.00544	0.00544	0.00544
ϕ	Resistance factors for flexure	5.5.4.2	0.90	1.00	1.00	0.90	1.00
ϕ_v	Resistance factors for shear		0.90	1.00	1.00	0.90	1.00
ϕ_n	Resistance factors for axial force		1.00	1.00	1.00	1.00	1.00
β_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	0.049	0.049	0.049
	For T section behavior	m	0.000	0.000	0.049	0.049	0.049
	For rectangular section behavior	m	0.000	0.000	0.049	0.049	0.049
f_{pe}	Effective stress in the prestressing steel after losses	Mpa	1116	1116	1116	1116	1116
f_{ps}	Aver. stress in pres. steel at the time for which the nominal resistance	Mpa	1860	1860	1846	1846	1846
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 - HEAD AND STEM WALL

a	Depth of equivalent stress block	m	0.000	0.000	0.041	0.041	0.041
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	4716	4716	61455	61455	61455
Mr	Factored resistance	kNm	4245	4716	61455	55309	61455
Mu	Flexural moment	kNm	1092	626	30722	42981	36250
(5.7.3.2)	Flexural moment Checking		OK	OK	OK	OK	OK
	Limits for reinforcement		0.00	0.00	0.03	0.03	0.03
c/de	Maximum reinforcement	<= 0.42	OK	OK	OK	OK	OK
	Maximum reinforcement Checking		2008	2008	26752	26752	26752
1.2*Mer	Craking moment	kNm	OK	OK	OK	OK	OK
(5.7.3.3.2)	Checking $M_r \geq \min(1.2M_{er}, 1.33M_u)$		OK	OK	OK	OK	OK
(5.7.3.4)	Control of craking by distr. of reinf for RC member- Check?		No	Yes	Yes	No	No
	Existing condition for structure	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	m ²	0.018	0.018	0.019	0.019	0.019
f _{sa}	Value	Mpa	297	297	290	290	290
0.6*fy		Mpa	240	240	240	240	240
	Tensil stress in reinf Min(f _{sa} , 0.6fy)	Mpa	240	240	240	240	240
x	Dist. From compression fiber to centroid	m	-	0.082	0.281	-	-
J.d	Arm	m	-	0.415	1.647	-	-
I _{cr}	Moment of inertia of the cracked section	m ⁴	-	0.032	1.523	-	-
f _s	Tensile stress in reinforcement $f_s = M_{sls} / (A_s * J.d)$	Mpa	-	49	207	-	-
	Checking for control cracking $f_s < f_{sa}$		N.a	OK	OK	N.a	N.a
(5.10.8.2)	Shrinkage and temperature Reinforcement (side distribution)						
A _{req}	Area of required reinf	m ²	0.00046	0.00046	0.00127	0.00127	0.00127
	Distribution on sides	m ²	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		2.5	3.4	2.3	2.1	2.2
θ	Angle of inclination of diagonal compressive	degree	28.99	28.54	34.21	37.87	36.68
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	23.280	23.280	23.280	23.280	23.280
dv	Effective shear depth	m	0.442	0.442	1.721	1.721	1.721
	(de - a/2)	m	0.442	0.442	1.721	1.721	1.721
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	0.600
ncat	Amount of bars in spacing S	bars	37	37	36	36	36
Av	Shear reinf area in spacing S	m ²	0.0056	0.0056	0.0054	0.0054	0.0054
θ	Assume	degree	28.99	28.54	34.21	37.87	36.68
v	Shear stress in concrete	kN/m ²	138	76	189	305	285
f _{po}	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e _s	Strain in tensile reinforcement		4.96E-04	2.71E-04	8.51E-04	1.19E-03	1.07E-03
	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		0.005	0.003	0.006	0.010	0.009
β	Final value		2.5	3.4	2.3	2.1	2.2
θ	Final value	degree	28.99	28.54	34.21	37.87	36.68
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	11822	15933	42132	38698	39911
Vs	Shear resistance provided by shear reinforcement	kN	2971	3027	9173	8018	8371
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	14793	18959	51305	46716	48282
Vn2	Vn2	kN	77173	77173	300423	300423	300423
Vn	Nominal shear resistance Vn = min(Vn1, Vn2)	kN	14793	18959	51305	46716	48282
Vr	Factored shear resistance	kN	13314	18959	51305	42044	48282
Vu	Shear	kN	1279	777	7573	10987	11409
(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	m ²	0.0159	0.0159	0.0159	0.0159	0.0159
	Minimum shear reinforcement Checking		-	-	-	-	-
	0.1*fc*bv*dv	kN	30869	30869	120169	120169	120169
S _{max}		m	0.35	0.35	0.60	0.60	0.60
	Maximum spacing S _{max}		-	-	-	-	-

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MATERIALS

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REINFORCEMENT CHECKING - PILECAP SECTION							
a	Depth of equivalent stress block	m	0.129	0.129	0.129	0.064	0.064
de	Corresp. effective depth from extreme comp. fiber to centroid of tensile force in the tensile reinf.	m	1.836	1.836	1.836	1.916	1.916
Mn	Nominal resistance	kNm	135327	135327	135327	71947	71947
Mr	Factored resistance	kNm	135327	121794	135327	71947	64753
Mu	Flexural moment	kNm	55180	79247	56632	25854	40616
(5.7.3.2)	Flexural moment Checking		OK	OK	OK	OK	OK
	Limits for reinforcement						
c/de	Maximum reinforcement		0.08	0.08	0.08	0.04	0.04
	Maximum reinforcement Checking	≤ 0.42	OK	OK	OK	OK	OK
1.2*Mcrr	Cracking moment	kNm	34812	34812	34812	33419	33419
(5.7.3.3.2)	Checking $Mr \geq \min(1.2Mcrr, 1.33Mu)$		OK	OK	OK	OK	OK
(5.7.3.4)	Control of cracking by distr. of reinf for RC member- Check?		Yes	No	No	No	No
	Existing condition for structure	1,2 or 3	3	3	3	3	3
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.064	0.064	0.064	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	m ²	0.010	0.010	0.010	0.019	0.019
fsa	Value	Mpa	206	206	206	163	163
0.6*fy	Tensil stress in reinf Min(fs, 0.6fy)	Mpa	240	240	240	240	240
x	Dist. From compression fiber to centroid	m	0.405	-	-	-	-
J.d	Arm	m	1.701	-	-	-	-
Icr	Moment of inertia of the cracked section	m ⁴	3.253	-	-	-	-
fs	Tensile stress in reinforcement $fs = Msls / (As * J.d)$	Mpa	170	-	-	-	-
	Checking for control cracking $fs < fsa$		OK	N.a	N.a	N.a	N.a
(5.10.8.2)	Shrinkage and temperature Reinforcement (side distribution)						
Areq	Area of required reinf	m ²	0.00127	0.00127	0.00127	0.00127	0.00127
	Distribution on sides	7 D16	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	degree	2.2	2.0	2.2	2.2	1.9
θ	Angle of inclination of diagonal compressive	degree	36.38	40.35	36.39	37.02	41.33
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	23.280	23.280	23.280	23.280	23.280
dv	Effective shear depth	m	1.772	1.772	1.772	1.884	1.884
	($de - a/2$)	m	1.772	1.772	1.772	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	37	37	37	37	37
Av	Shear reinf area in spacing S	m ²	0.0075	0.0075	0.0075	0.0075	0.0075
θ	Assume	degree	36.38	40.35	36.38	37.02	41.33
v	Shear stress in concrete	kN/m ²	386	616	367	33	462
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e_s	Strain in tensile reinforcement		1.04E-03	1.44E-03	1.04E-03	1.10E-03	1.58E-03
	if $e_s < 0$, multiple with reduce factor		-	-	-	-	-
	Strain checking	$\leq 2.00E-3$	Ok	Ok	Ok	Ok	Ok
v/fc	Ratio of shear stress and fc		0.013	0.021	0.012	0.001	0.015
β	Final value		2.2	2.0	2.2	2.2	1.9
θ	Final value	degree	36.38	40.35	36.39	37.02	41.33
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	41411	37242	41408	43316	38116
Vs	Shear resistance provided by shear reinforcement	kN	11981	10390	11980	12445	10672
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	53392	47631	53388	55762	48788
Vn2	Vn2	kN	309333	309333	309333	328917	328917
Vn	Nominal shear resistance $Vn = \min(Vn1, Vn2)$	kN	53392	47631	53388	55762	48788
Vr	Factored shear resistance	kN	53392	42868	53388	55762	43909
Vu	Shear	kN	15901	22857	15137	13015	18244
(5.8.2.7)	Shear checking		OK	OK	OK	OK	OK

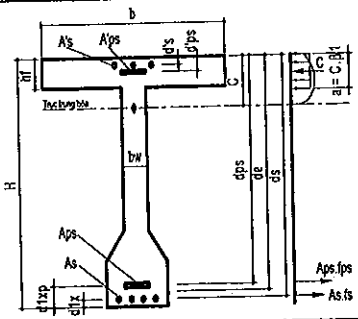
	Da Nang Quang Ngai Expressway project OP18a BRIDGE DETAIL DESIGN ABUTMENT AIR	Item.	Eng.	Date.	Sign.
		Design			
		Check			
		Revise			

22TCN272-05, AASHTO LRFD 2nd - 1998

REINFORCEMENT CHECKING - WING WALL

MATERIALS

NORMAL CONCRETE			
f _c	Compressive Strength of concrete at 28 days	Mpa	30
E _c	Modulus of Elasticity	Mpa	27691
f _r	Modulus of Rupture	Mpa	3.5
g _c	Unit weight of concrete	kN/m ³	24.5
PRESTRESSING STEEL			
f _{pu}	Tensile strength of prestressing steel	Mpa	1860
f _{py}	Yield strength of prestressing steel	Mpa	1670
E _p	Modulus of Elasticity	Mpa	197000
REINFORCEMENT			
f _y	Yield strength	Mpa	400
E _s	Modulus of Elasticity	Mpa	200000
nc	Ratio E _s /E _c		7



Sign	Parameters	Unit	Sections				
			5-5	5-5	6-6	6-6	6-6
INTERNAL FORCES AT SECTION							
	Combination		Service	Strength	Service	Strength	Strength
Qu	Shear	kN	102	157	250	387	387
Mu	Flexural Moment	kNm	246	392	275	426	426
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
A _{mc}	Section area	m2	0.800	0.800	0.800	0.800	0.800
A _{ps}	Steel choice						
	Tension prestressing steel	P.S type	0	0	0	0	0
		Number	tendons	0	0	0	0
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
		Area	m2	0.00000	0.00000	0.00000	0.00000
A _s	Tension Reinforcement	Number	6	6	6	6	6
		Diameter	mm	22	22	22	22
		Area	m2	0.00228	0.00228	0.00228	0.00228
A' _s	Compression Reinforcement	Number	6	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	2	2	2	2	2
		Diameter	mm	14	14	14	14
		Area	m2	0.00030	0.00030	0.00030	0.00030
φ	Resistance factors for flexure	5.5.4.2	1.00	0.90	1.00	0.90	0.90
φ _v	Resistance factors for shear		1.00	0.90	1.00	0.90	0.90
φ _n	Resistance factors for axial force		1.00	1.00	1.00	1.00	1.00
β ₁	Stress block factor		0.836	0.836	0.836	0.836	0.836
c	Dis. Between centroid and top fiber	m	0.020	0.020	0.020	0.020	0.020
	For T section behavior	m	0.020	0.020	0.020	0.020	0.020
	For rectangular section behavior	m	0.020	0.020	0.020	0.020	0.020
f _{pe}	Effective stress in the prestressing steel after losses	Mpa	1116	1116	1116	1116	1116
f _{ps}	Aver. stress in pres. steel at the time for which the nominal resistance	Mpa	1847	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	0.28

	Da Nang Quang Ngai Expressway project OP18a BRIDGE DETAIL DESIGN ABUTMENT A1R	Item.	Eng.	Date.	Sign.
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REINFORCEMENT CHECKING - WING WALL

a	Depth of equivalent stress block	m	0.017	0.017	0.017	0.017	0.017
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	644	644	644	644	644
Mr	Factored resistance	kNm	644	580	644	580	580
Mu	Flexural moment	kNm	246	392	275	426	426
(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	Minimum reinforcement		0.29%	0.29%	0.29%	0.29%	0.29%
	Minimum reinforcement Checking for RC	0.23%	OK	OK	OK	OK	OK
1.2*Mc	Cracking moment	kNm	227	227	227	227	227
(5.7.3.3.2)	Checking $M_r \geq \min(1.2M_c, 1.33M_u)$		OK	OK	OK	OK	OK
(5.7.3.4)	Control of cracking by distr. of reinf for RC member- Check?		Yes	No	Yes	No	No
	Existing condition for structure	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	m ²	0.020	0.020	0.020	0.020	0.020
f _{sa}	Value	Mpa	285	285	285	285	285
0.6*f _y	Tensile stress in reinf Min(f _{sa} , 0.6f _y)	Mpa	240	240	240	240	240
x	Dist. From compression fiber to centroid	m	0.139	-	0.139	-	-
J.d	Arm	m	0.695	-	0.695	-	-
I _{cr}	Moment of inertia of the cracked section	m ⁴	0.007	-	0.007	-	-
f _s	Tensile stress in reinforcement $f_s = M_{sls} / (A_s * J.d)$	Mpa	155	-	174	-	-
	Checking for control cracking $f_s < f_{sa}$		OK	N.a	OK	N.a	N.a
(5.10.8.2)	Shrinkage and temperature Reinforcement (side distribution)						
A _{req}	Area of required reinf	m ²	0.00042	0.00042	0.00042	0.00042	0.00042
	Distribution on sides	m ²	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	degree	2.3	2.0	2.1	1.8	1.8
θ	Angle of inclination of diagonal compressive	degree	34.77	39.80	37.77	41.99	42.05
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	90
b _v	Effective web width as minimum web width - in dv	m	1.000	1.000	1.000	1.000	1.000
d _v	Effective shear depth	m	0.733	0.733	0.733	0.733	0.733
	(d _e - a/2)	m	0.733	0.733	0.733	0.733	0.733
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	0.600
n _{cat}	Amount of bars in spacing S	bars	2	2	2	2	2
A _v	Shear reinf area in spacing S	m ²	0.0003	0.0003	0.0003	0.0003	0.0003
β	Assume	degree	2.0	2.0	2.0	2.0	2.0
θ	Assume	degree	34.77	39.79	37.77	41.99	41.05
v	Shear stress in concrete	kN/m ²	139	238	341	587	587
f _{po}	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e _s	Strain in tensile reinforcement		8.97E-04	1.38E-03	1.18E-03	1.75E-03	1.76E-03
	if ex<0, multiple with reduce factor		-	-	-	-	-
	Strain checking	$\leq 2.00E-3$	Ok	Ok	Ok	Ok	Ok
v/f _c	Ratio of shear stress and f _c		0.005	0.008	0.011	0.020	0.020
β	Final value	degree	2.3	2.0	2.1	1.8	1.8
θ	Final value	degree	34.77	39.80	37.77	41.99	42.05
V _c	Nominal shear resistance provided by tensile stresses in the concrete	kN	762	672	710	612	609
V _s	Shear resistance provided by shear reinforcement	kN	212	177	190	164	164
V _p	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0	0
V _{n1}	V _{n1} = V _c + V _s + V _p	kN	974	849	900	776	773
V _{n2}	V _{n2}	kN	5495	5495	5495	5495	5495
V _n	Nominal shear resistance V _n = min(V _{n1} , V _{n2})	kN	974	849	900	776	773
V _r	Factored shear resistance	kN	974	764	900	698	696
V _u	Shear	kN	102	157	250	387	387
(5.8.2.7)	Shear checking		OK	OK	OK	OK	OK

SPACE PILE FOUNDATION ANALYSIS PROGRAM
Turbo BASIC

PROJECT: : OP18a-A1R

INITIAL DATA

Kn = 0.00 Ax = 9.14 By = 23.29 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 = 400 (t/m4)

LOAD COMBINATIONS

COMB.	Hx	Hy	P	Mx	My	Mz
1	1608.00	0.00	5945.00	-2370.00	2168.00	0.00
2	1057.00	0.00	4287.00	-2040.00	1419.00	0.00
3	1562.00	6.00	5829.00	-2154.00	1952.00	0.00
4	1011.00	6.00	4171.00	-1824.00	1202.00	0.00
5	1092.00	5.00	4503.00	-1672.00	1306.00	0.00
6	1230.00	76.00	5583.00	-2005.00	-622.00	0.00
7	1230.00	76.00	3924.00	-1675.00	1110.00	0.00

PROPERTIES OF PILES

PILE	Lo	H	Bpx	Bpy	A	B	Cday	Fo	Io	Po	Co	Ct
1	0.00	13.50	1.057	1.057	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							
9					n t							
10					n t							
11					n t							
12					n t							
13					n t							
14					n t							

PILE COORD.

PILE	X	Y	Phi	Xi
1	3.30	11.01	0.000	0.00
2	3.30	7.05	0.000	0.00
3	3.30	3.09	0.000	0.00
4	3.30	-0.87	0.000	0.00
5	3.30	-4.83	0.000	0.00
6	3.30	-8.79	0.000	0.00
7	0.00	10.43	0.000	0.00
8	0.00	6.47	0.000	0.00
9	0.00	0.53	0.000	0.00
10	0.00	-5.41	0.000	0.00
11	0.00	-9.37	0.000	0.00

			AIR_OUT	
12	-3.30	9.84	0.000	0.00
13	-3.30	-0.06	0.000	0.00
14	-3.30	-9.96	0.000	0.00

DISPLACEMENTS

COMB.	X	Y	Z	Fix	Fiy	Fiz
1	0.01921	-0.00018	0.001937	0.000014	0.000179	-0.000190
2	0.01258	-0.00011	0.001414	0.000007	0.000103	-0.000125
3	0.01862	-0.00011	0.001908	0.000014	0.000164	-0.000184
4	0.01199	-0.00004	0.001384	0.000008	0.000088	-0.000119
5	0.01296	-0.00007	0.001495	0.000010	0.000096	-0.000129
6	0.01416	0.00075	0.001949	0.000009	-0.000018	-0.000136
7	0.01466	0.00075	0.001268	0.000007	0.000131	-0.000136

FORCES ON PILES

PILE	COMB.	N	Q2	Q3	M1	M2	M3
1	1	491.54	-102.60	-3.07	-4.452	-11.057	341.232
	2	346.41	-67.44	-2.02	-2.927	-7.185	224.953
	3	474.29	-99.72	-3.40	-4.307	-12.195	332.078
	4	329.14	-64.56	-2.35	-2.781	-8.323	215.801
	5	351.55	-69.72	-2.43	-3.009	-8.730	233.012
	6	370.81	-79.11	-7.62	-3.177	-26.315	270.141
	7	335.79	-79.11	-7.62	-3.177	-26.241	263.438
2	1	502.68	-107.29	-3.07	-4.452	-11.057	357.185
	2	352.22	-70.52	-2.02	-2.927	-7.185	235.439
	3	485.81	-104.25	-3.40	-4.307	-12.195	347.509
	4	335.33	-67.49	-2.35	-2.781	-8.323	225.766
	5	359.86	-72.88	-2.43	-3.009	-8.730	243.792
	6	377.77	-82.46	-7.62	-3.177	-26.315	281.524
	7	341.39	-82.46	-7.62	-3.177	-26.241	274.820
3	1	513.81	-111.97	-3.07	-4.452	-11.057	373.138
	2	358.03	-73.60	-2.02	-2.927	-7.185	245.926
	3	497.33	-108.78	-3.40	-4.307	-12.195	362.941
	4	341.52	-70.41	-2.35	-2.781	-8.323	235.732
	5	368.16	-76.05	-2.43	-3.009	-8.730	254.572
	6	384.73	-85.80	-7.62	-3.177	-26.315	292.906
	7	347.00	-85.80	-7.62	-3.177	-26.241	286.203
4	1	524.95	-116.66	-3.07	-4.452	-11.057	389.091
	2	363.84	-76.68	-2.02	-2.927	-7.185	256.412
	3	508.85	-113.31	-3.40	-4.307	-12.195	378.373
	4	347.71	-73.34	-2.35	-2.781	-8.323	245.697
	5	376.47	-79.22	-2.43	-3.009	-8.730	265.351
	6	391.69	-89.14	-7.62	-3.177	-26.315	304.289
	7	352.61	-89.14	-7.62	-3.177	-26.241	297.586
5	1	536.09	-121.35	-3.07	-4.452	-11.057	405.044
	2	369.66	-79.76	-2.02	-2.927	-7.185	266.899
	3	520.36	-117.85	-3.40	-4.307	-12.195	393.805
	4	353.90	-76.27	-2.35	-2.781	-8.323	255.662
	5	384.78	-82.38	-2.43	-3.009	-8.730	276.131
	6	398.65	-92.49	-7.62	-3.177	-26.315	315.672
	7	358.21	-92.49	-7.62	-3.177	-26.241	308.969
6	1	547.23	-126.03	-3.07	-4.452	-11.057	420.997
	2	375.47	-82.85	-2.02	-2.927	-7.185	277.385
	3	531.88	-122.38	-3.40	-4.307	-12.195	409.237
	4	360.08	-79.19	-2.35	-2.781	-8.323	265.628
	5	393.08	-85.55	-2.43	-3.009	-8.730	286.911
	6	405.61	-95.83	-7.62	-3.177	-26.315	327.055
	7	363.82	-95.83	-7.62	-3.177	-26.241	320.352
7	1	370.96	-103.29	0.84	-4.452	2.237	343.576
	2	276.76	-67.90	0.55	-2.927	1.553	226.494
	3	363.91	-100.38	0.38	-4.307	0.664	334.345
	4	269.73	-64.99	0.09	-2.781	-0.019	217.265

		AIR_OUT					
8	5	287.06	-70.18	0.21	-3.009	0.253	234.596
	6	384.27	-79.60	-4.83	-3.177	-16.830	271.813
	7	247.31	-79.60	-4.83	-3.177	-16.756	265.110
	1	382.10	-107.98	0.84	-4.452	2.237	359.529
	2	282.57	-70.98	0.55	-2.927	1.553	236.980
	3	375.43	-104.91	0.38	-4.307	0.664	349.777
	4	275.91	-67.92	0.09	-2.781	-0.019	227.230
9	5	295.36	-73.35	0.21	-3.009	0.253	245.376
	6	391.23	-82.95	-4.83	-3.177	-16.830	283.196
	7	252.92	-82.95	-4.83	-3.177	-16.756	276.493
	1	398.80	-115.00	0.84	-4.452	2.237	383.459
	2	291.29	-75.60	0.55	-2.927	1.553	252.710
	3	392.70	-111.71	0.38	-4.307	0.664	372.925
	4	285.20	-72.31	0.09	-2.781	-0.019	242.179
10	5	307.82	-78.10	0.21	-3.009	0.253	261.546
	6	401.67	-87.96	-4.83	-3.177	-16.830	300.270
	7	261.33	-87.96	-4.83	-3.177	-16.756	293.567
	1	415.51	-122.03	0.84	-4.452	2.237	407.388
	2	300.01	-80.22	0.55	-2.927	1.553	268.440
	3	409.98	-118.51	0.38	-4.307	0.664	396.072
	4	294.48	-76.70	0.09	-2.781	-0.019	257.127
11	5	320.28	-82.85	0.21	-3.009	0.253	277.715
	6	412.11	-92.98	-4.83	-3.177	-16.830	317.345
	7	269.74	-92.98	-4.83	-3.177	-16.756	310.641
	1	426.65	-126.72	0.84	-4.452	2.237	423.341
	2	305.82	-83.30	0.55	-2.927	1.553	278.926
	3	421.50	-123.05	0.38	-4.307	0.664	411.504
	4	300.67	-79.62	0.09	-2.781	-0.019	267.092
12	5	328.59	-86.02	0.21	-3.009	0.253	288.495
	6	419.07	-96.32	-4.83	-3.177	-16.830	328.727
	7	275.34	-96.32	-4.83	-3.177	-16.756	322.024
	1	250.38	-103.98	4.74	-4.452	15.531	345.920
	2	207.11	-68.35	3.12	-2.927	10.292	228.035
	3	253.53	-101.05	4.16	-4.307	13.524	336.613
	4	210.31	-65.42	2.53	-2.781	8.285	218.729
13	5	222.56	-70.65	2.85	-3.009	9.236	236.180
	6	397.73	-80.09	-2.05	-3.177	-7.345	273.486
	7	158.83	-80.09	-2.05	-3.177	-7.271	266.783
	1	278.23	-115.69	4.74	-4.452	15.531	385.803
	2	221.64	-76.05	3.12	-2.927	10.292	254.251
	3	282.32	-112.38	4.16	-4.307	13.524	375.192
	4	225.78	-72.74	2.53	-2.781	8.285	243.643
14	5	243.33	-78.57	2.85	-3.009	9.236	263.129
	6	415.13	-88.45	-2.05	-3.177	-7.345	301.943
	7	172.85	-88.45	-2.05	-3.177	-7.271	295.240
	1	306.07	-127.41	4.74	-4.452	15.531	425.682
	2	236.17	-83.75	3.12	-2.927	10.292	280.465
	3	311.11	-123.71	4.16	-4.307	13.524	413.769
	4	241.25	-80.05	2.53	-2.781	8.285	268.554
	5	264.10	-86.48	2.85	-3.009	9.236	290.077
	6	432.53	-96.81	-2.05	-3.177	-7.345	330.398
	7	186.87	-96.81	-2.05	-3.177	-7.271	323.694

SUMMARY OF FORCES

		PILE	COMB.	N	Q2	Q3	M1	M2	M3
Nmin	12	7	158.83	-80.09	-2.05	-3.177	-7.271	266.783	
Nmax	6	1	547.23	-126.03	-3.07	-4.452	-11.057	420.997	
Q2max	14	1	306.07	-127.41	4.74	-4.452	15.531	425.682	
Q3max	1	7	335.79	-79.11	-7.62	-3.177	-26.241	263.438	
M1max	1	1	491.54	-102.60	-3.07	-4.452	-11.057	341.232	
M2max	1	6	370.81	-79.11	-7.62	-3.177	-26.315	270.141	
M3max	14	1	306.07	-127.41	4.74	-4.452	15.531	425.682	

CHECKING CALCULATI
IN COMPARISON WITH INITIA LOAD MATRIX

AIR_OUT

1	1608.00	0.00	5945.00	-2370.00	2168.00	0.00
2	1057.00	0.00	4287.00	-2040.00	1419.00	0.00
3	1562.00	6.00	5829.00	-2154.00	1952.00	0.00
4	1011.00	6.00	4171.00	-1824.00	1202.00	0.00
5	1092.00	5.00	4503.00	-1672.00	1306.00	0.00
6	1230.00	76.00	5583.00	-2005.00	-622.00	0.00
7	1230.00	76.00	3924.00	-1675.00	1110.00	0.00

CALCULATION SHEET

BORED PILE CAPACITY

	Da Nang Quang Ngai Expressway project OP18A BRIDGE DETAIL DESIGN ABUTMENT A1L	Item.	Eng.	Date.	Sign.
		Design			
		Check			
		Revise			

f. BORED PILE DESIGN

I. BORED PILE DATA

1. Load Combinations at top of bored pile

No	Combinations	Sign	F _v (kN)	Longitudinal		Transvesal	
				F _{Hx} (kN)	My (kN·m)	F _{Hv} (kN)	Mx (kN·m)
1	Extreme Ext-IB		1550	783	-2609	20	72
2	Strength Str-IA		5406	1249	-4170	30	110
3	Strength Str-IA		2918	1263	-4217	-47	-154
4	Extreme Ext-IA		3626	774	-2641	75	258
5	Strength Str-IA		4851	1017	-3380	30	110

2. Bored pile Material

Normal concrete		
Compressive strength at 28 days age	f _c	30 MPa
Concrete elastic modulus	E _c	27691 MPa
Reinforcement TCVN1651-2008; CBV-400		
Yield strength	f _y	400 MPa
Reinforcement elastic modulus	E _s	200,000 MPa

3. Bored pile Section

Pile diameter	D	1.20 m
Section area	A	1.131 m ²
Moment inertia	I _x	0.102 m ⁴
	I _y	0.102 m ⁴
Radius of gyration of gross concrete section; $r = \sqrt{I/A}$	r _x	0.300 m
	r _y	0.300 m

II. PILE DESIGN

1. Limit of Reinforcement

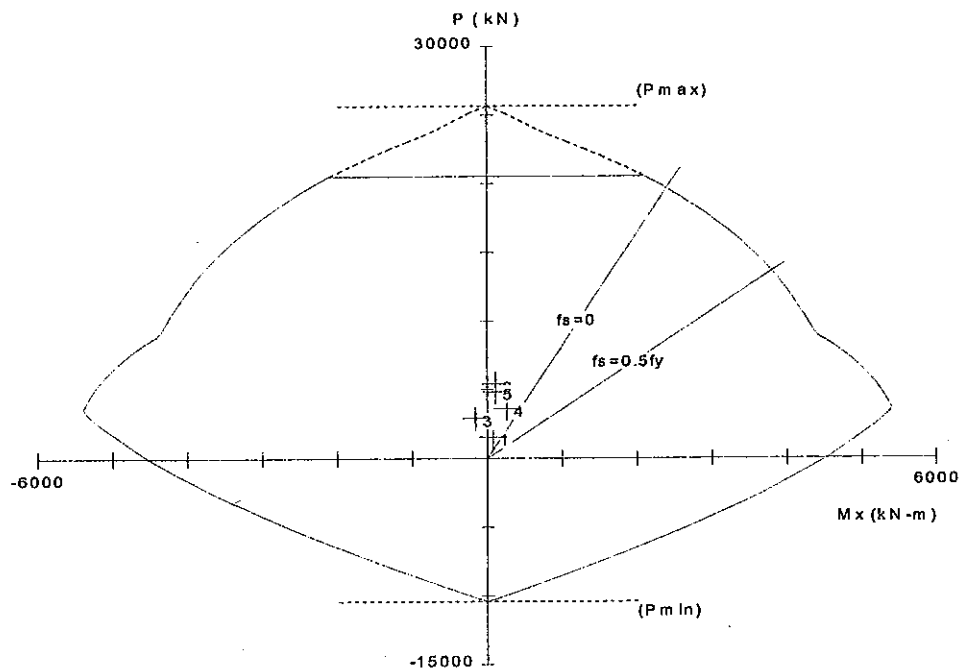
S.5.7.4.2

Minimum area of longitudinal reinforcement in column			
$A_s \cdot f_y / (A_g \cdot f_c) \geq 0.135$	$A_s \geq$	0.011	m ²
$A_s / A_g \geq 0.01$	$A_s \geq$	0.011	m ²
Maximum area of longitudinal reinforcement in column			
$A_s / A_g \leq 0.08$	$A_s \leq$	0.090	m ²
Trial Rebars:	Ok A_s	0.029	m ²
11 layers x 24 = 36 bars D32 @150 As1		0.029	m ²

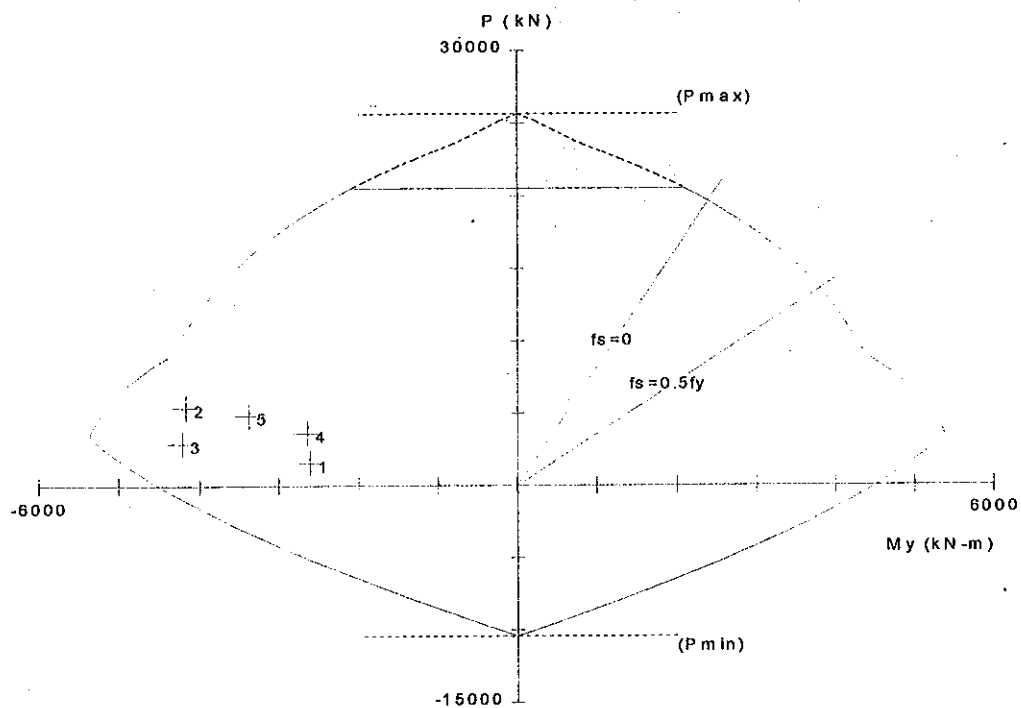
2. Interaction diagram M-P

Using Pca-Column software

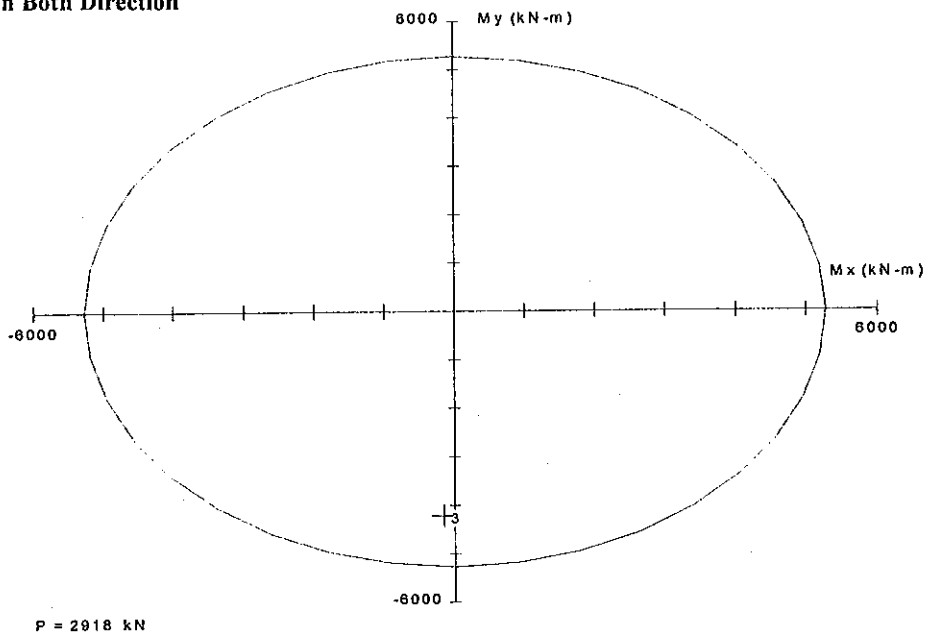
**In Transverse Direction



**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	0.933	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 reinforcement tie in 1 hoop	Ltie	3.41	m
Ratio of ties reinf. in one hoop/ volume of conc. core one pitch spacing			
$\rho_s = A_{s-tr} \cdot L_{tie} / (A_c \cdot \text{spacing})$	ρ_s	0.0074	
Ratio of spiral reinf. To total volume of concrete core shall satisfy			S.5.7.4.6
$\rho_s \geq 0.45 \cdot (A_g/A_c - 1) \cdot f_c/f_y = \text{Req1}$	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	
$\rho_s \geq 0.12 \cdot f_c/f_y = \text{Req2}$	Req2	0.0090	N/A
Length distributed spiral with pitch 75mm below pilecap	Ldis	1.80	m

4. Shear Design

Shear resistance factors	ϕ_v	1.0	
Factored shear force	Vu	1263	kN
Required shear capacity $V_n = V_u / \phi_v$	Vn	1263	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
$d_v = 0.9 \cdot d_e$ $d_e = D/2 + D_r/\pi()$			
Diameter of the circle passing through the centers of the long. reinf.	Dr	0.99	m
	de	0.92	m
	dv	0.82	m
$V_c = 0.083 \cdot \beta \cdot \sqrt{f_c} \cdot b_v \cdot d_v$	Vc	900	kN

Difference between required shear capacity and the capacity provided by concrete
is the minimum required capacity for shear reinforcements

$$V_s = V_n - V_c$$

In this case $V_c > V_n$ so shear reinforcement is no need

Stirrup diameter

Number of stirrup legs / cross section

Shear legs area

Angle of inclination of shear reinf. to long. axis

$$V_s = A_v \cdot f_y \cdot d_v \cdot (\cot \theta + \cot \alpha) \cdot \sin \alpha / s$$

Stirrup spacing used

Check minimum shear reinforcement requirement

$$A_v \geq 0.083 \cdot \sqrt{f_c} \cdot b_v \cdot s / f_y = R_{eq}$$

Check maximum shear reinforcement spacing requirement

$$F = 0.1 \cdot f_c \cdot b_v \cdot d_v$$

If $V_u < 0.1 \cdot f_c \cdot b_v \cdot d_v$ then $S_{max} = 0.8 \cdot d_v \leq 600 \text{ mm}$

If $V_u > 0.1 \cdot f_c \cdot b_v \cdot d_v$ then $S_{max} = 0.4 \cdot d_v \leq 300 \text{ mm}$

V_s	363	kN
D_s	14	
n_s	2	
A_v	0.0003	m ²
α	90	deg
$s \leq$	0.00	m
s	0.10	m
	OK	
R_{eq}	0.0000	m ²
	OK	
F	3609	kN
S_{max}	0.60	m

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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	OP18a-A1
Bottom of pilecap elavation	EL1 = 4.50
Top of socket elevation	EL2 = -2.00
Pile tip elevation	EL3 = -9.00
Pile Length	L = 13.50 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 = 6015.2 kN
Working normal force at top of socket	P _i = 5898.7 kN
Intack rock modulus	E _i = 25000 MPa
Modulus modification ratio	K _c = 0.05
Elastic modulus of the insitu rock	E _r = K _c *E _i = 1250.0 MPa
Influence coefficient	I _p = f (H _s /D _s , E _d /E _r) = 0.30
	H _s /D _s = 5.83
	E _d /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 concrete	E _c = 27691 MPa
Depth of socket	H _s = 7.00 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
Atmospheric pressure	p _a = 0.101 MPa
Reduction factor to account for jointing	α _E

C.10.4.6.5-1-Lrfd 4th

10.8.3.5.4b-Lrfd 4th

The elastic shortening of the drilled shaft

$$r_e = (\sum P_i) * H_s / (A_{soc} * E_c) = 1.318 \text{ mm}$$

The settlement of base of drilled shaft

$$r_{base} = (\sum P_i) * I_p / (D_s * E_r) = 1.180 \text{ mm}$$

$$r_e + r_{base} = 2.498 \text{ mm} < 10 \text{ mm}$$

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 q_u < 1.9 Mpa - may be taken after Carter & Kulhavy 1988 → q_s = 0.15*q_u

if q_u > 1.9 Mpa - may be taken after Horvath & Kenney 1979 → q_s = 0.21*sqrt(q_u)

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 → q_s = 0.65*α_E*p_a*(q_u/p_a)^{0.5} < 7.8*p_a*(f_c/p_a)^{0.5}

$$q_{s0} = 7.8 * p_a * (f_c / p_a)^{0.5}$$

C10.8.3.5-1

C10.8.3.5-5

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_n 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	-2.00	-3.00	1.00	12	29.90	1.15	4329	0.65	2814
2	-3.00	-4.00	1.00	12	29.90	1.15	4329	0.65	2814
3	-4.00	-5.00	1.00	10	38.01	1.29	4881	0.65	3173
4	-5.00	-6.00	1.00	10	38.01	1.29	4881	0.65	3173
5	-6.00	-9.00	3.00	16	38.01	1.29	14643	0.65	9518
6									
7									
8									
Sum			7.00				33062		21491

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	OP18a BRIDGE		Design			
	DETAIL DESIGN		Check			
	EMPIRICAL ESTIMATION OF PILE CAPACITY		Revise			

Case 2												Type: "1. closed joints", "2. open joints"
No.	Depth (m)	RQD (%)	q_u (MPa)	E_m/E_i	α_E	Type	q_{s0} (MPa)	q_s (MPa)	$q_s - used$ (MPa)	Q_{SR} (kN)	ϕ_s	Q_R (kN)
1	1.00	12.00	29.90	0.05	0.45	1	13.58	0.51	0.51	1597	0.55	878
2	1.00	12.00	29.90	0.05	0.45	1	13.58	0.51	0.51	1597	0.55	878
3	1.00	10.00	38.01	0.05	0.45	1	13.58	0.57	0.57	1800	0.55	990
4	1.00	10.00	38.01	0.05	0.45	1	13.58	0.57	0.57	1800	0.55	990
5	3.00	16.00	38.01	0.05	0.45	1	13.58	0.57	0.57	5401	0.55	2971
6	-	-	-	-	-	-	-	-	-	-	-	-
7	-	-	-	-	-	-	-	-	-	-	-	-
8	-	-	-	-	-	-	-	-	-	-	-	-
Sum	7.00									12196		6708

Unit base resistance $q_p = K_b(p_1 - p_0) + \sigma_v$

Limit pressure determined from pressuremeter tests

At rest total horizontal stress measured at their base elevation

Coefficient that depend on diameter socket

Total vertical stress at the base elevation

$$p_1 = 5.89 \text{ MPa}$$

C10.8.3.5-7

$$p_0 = - \text{MPa}$$

$$K_b = 5.60$$

Table C10.8.3.5-1

$$\sigma_v = - \text{MPa}$$

$$q_p = - \text{MPa}$$

$$\phi = 0.50$$

Table 10.5.5-3

$$Q_{pR} = A_p \cdot q_p$$

$$Q_{pR} = - \text{kN}$$

$$Q_R = \phi \cdot Q_{pR}$$

$$Q_R = - \text{kN}$$

ESTIMATED PILE CAPACITY:

Pile Structural Capacity	$Q_T = 0.75 \cdot 0.85 \cdot 0.85 \cdot f_c \cdot A_g$	12768 kN	1301 T
Pile resistance	Q_R	6708 kN	684 T
Deducting pile weight		-210 kN	-21 T
Estimated Pile Capacity		6498 kN	662 T
Maximum Reaction - ULS	Ok	5607 kN	572 T

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OP18a BRIDGE				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	OP18a-A2	Pile Concrete comp. strength	$f'_c = 30.0$ MPa
Bottom of pilecap elevation	EL1 = 4.50	Concrete Unit Weight	$g_c = 24.5$ kN/m ³
Top of socket elevation	EL2 = -3.60	Modulus of elasticity of concrete	$E_c = 27691$ MPa
Pile tip elevation	EL3 = -7.50		
Pile Length	$L = 12.00$ m	Depth of socket	$H_s = 3.90$ m
Diameter of drilled-shaft	$D_p = 1.20$ m	Diameter of socket	$D_s = 1.20$ m
Pile Cross-Sectional Perimeter	$P = 3.77$ m	Socket Cross-Sect. Perimeter	$P_{soc} = 3.77$ m
Pile Cross-Sectional Area	$A_b = 1.13$ m ²	Socket Cross-Sectional Area	$A_{soc} = 1.13$ m ²
Working normal force at pile head	$N = 5973.6$ kN		
Working normal force at top of socket	$P_i = 5908.7$ kN		
Intact rock modulus	$E_i = 25000$ MPa		
Modulus modification ratio	$K_c = 0.05$		
Elastic modulus of the insitu rock	$E_r = K_c * E_i = 1250.0$ MPa		
Influence coefficient	$I_p = f(H_s/D_s, E_c/E_r) = 0.30$		
	$H_s/D_s = 3.25$		
	$E_c/E_r = 22.15$		
Rock mass modulus/ intact rock modulus	E_m/E_i		
Atmospheric pressure	$p_a = 0.101$ MPa		
Reduction factor to account for jointing	α_g		

Figure C10.8.3.5-2 Lrfd

Figure C10.8.3.5-3 Lrfd

Figure C10.8.3.5-1 Lrfd

C.10.4.6.5-1-Lrfd 4th

10.8.3.5.4b-Lrfd 4th

The elastic shortening of the drilled shaft

$$r_e = (\sum P_i) * H_s / (A_{soc} * E_c) = 0.736 \text{ mm}$$

The settlement of base of drilled shaft

$$r_{base} = (\sum P_i) * I_p / (D_s * E_r) = 1.182 \text{ mm}$$

$$r_e + r_{base} = 1.918 \text{ mm} < 10 \text{ mm}$$

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

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

if $q_u < 1.9$ Mpa - may be taken after Carter & Kulhawy 1988 $\rightarrow q_s = 0.15 * q_u$

if $q_u > 1.9$ Mpa - may be taken after Horvath & Kenney 1979 $\rightarrow q_s = 0.21 * \sqrt{q_u}$

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 $\rightarrow q_s = 0.65 * \alpha_g * p_a * (q_u/p_a)^{0.5} < 7.8 * p_a * (f_c/p_a)^{0.5}$

$$q_{s0} = 7.8 * p_a * (f_c/p_a)^{0.5}$$

C10.8.3.5-1

C10.8.3.5-5

10.8.3.5.4d-1-Lrfd 2007

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	-3.60	-4.60	1.00	52	53.27	1.53	5778	0.65	3756
2	-4.60	-5.50	0.90	53	70.42	1.76	5979	0.65	3886
3	-5.50	-7.50	2.00	27	70.42	1.76	13287	0.65	8637
4									
5									
6									
7									
8									
Sum			3.90				25044		16279

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	OP18a BRIDGE					Design			
	DETAIL DESIGN					Check			
	EMPIRICAL ESTIMATION OF PILE CAPACITY					Revise			

Case2.										Type: "1: closed joints", "2: open joints"		
No.	Depth (m)	RQD (%)	q_u (MPa)	E_m/E_i	α_E	Type	q_{s0} (MPa)	q_s (MPa)	$q_s - used$ (MPa)	Q_{SR} (kN)	ϕ_s	Q_R (kN)
1	1.00	52.00	53.27	0.20	0.63	1	13.58	0.95	0.95	2978	0.55	1638
2	0.90	53.00	70.42	0.23	0.65	1	13.58	1.13	1.13	3183	0.55	1751
3	2.00	27.00	70.42	0.07	0.50	1	13.58	0.86	0.86	5410	0.55	2975
4	-	-	-	-	-	-	-	-	-	-	-	-
5	-	-	-	-	-	-	-	-	-	-	-	-
6	-	-	-	-	-	-	-	-	-	-	-	-
7	-	-	-	-	-	-	-	-	-	-	-	-
8	-	-	-	-	-	-	-	-	-	-	-	-
Sum	3.90									11571		6364

Unit base resistance $q_p = K_b.(p_1 - p_0) + \sigma_v$

Limit pressure determined from presuremeter tests

At rest total horizontal stress measured at ther base elevation

Coefficient that depen on diameter socket

Total vertical stress at the base elevation

$p_1 = 5.89$ MPa

C10.8.3.5-7

$p_0 = -$ MPa

$K_b = 4.52$

Table C10.8.3.5-1

$\sigma_v = -$ MPa

$q_p = -$ MPa

$\phi = 0.50$

Table 10.5.5-3

$Q_{pR} = A_p \cdot q_p$

$Q_{pR} = -$ kN

$Q_R = \phi \cdot Q_{pR}$

$Q_R = -$ kN

ESTIMATED PILE CAPACITY:

Pile Structural Capacity	$Q_T = 0.75 \cdot 0.85 \cdot 0.85 \cdot f_c \cdot A_g$	12768 kN	1301 T
Pile resistance	Q_R	6364 kN	649 T
Deducting pile weight		-169 kN	-17 T
Estimated Pile Capacity		6195 kN	632 T
Maximum Reaction - ULS	Ok	5607 kN	573 T

4 OP19

Table of content - OP19 Bridge

A. Substructure design

1. Abutment A1
2. Bored pile capacity

MINISTRY OF TRANSPORT

VIETNAM EXPRESSWAY CORPORATION - PROJECT MANAGEMENT UNIT NO.85

DA NANG - QUANG NGAI EXPRESSWAY DEVELOPMENT PROJECT

PACKAGE: A2

BRIDGE

OP19

CALCULATION SHEETS

SUBSTRUCTURE

CALCULATION SHEET

ABUTMENT

Table of content

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

	Da Nang Quang Ngai Expressway project OP19 BRIDGE DETAIL DESIGN ABUTMENT A1	Item.	Eng.	Date.	Sign.
		Design			
		Check			
		Revise			

LOAD COMPONENTS

Assumptions :

1. Bridge is considered to be in seismic with acceleration coeff. $A = 0.0580 \text{ g}$
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 :

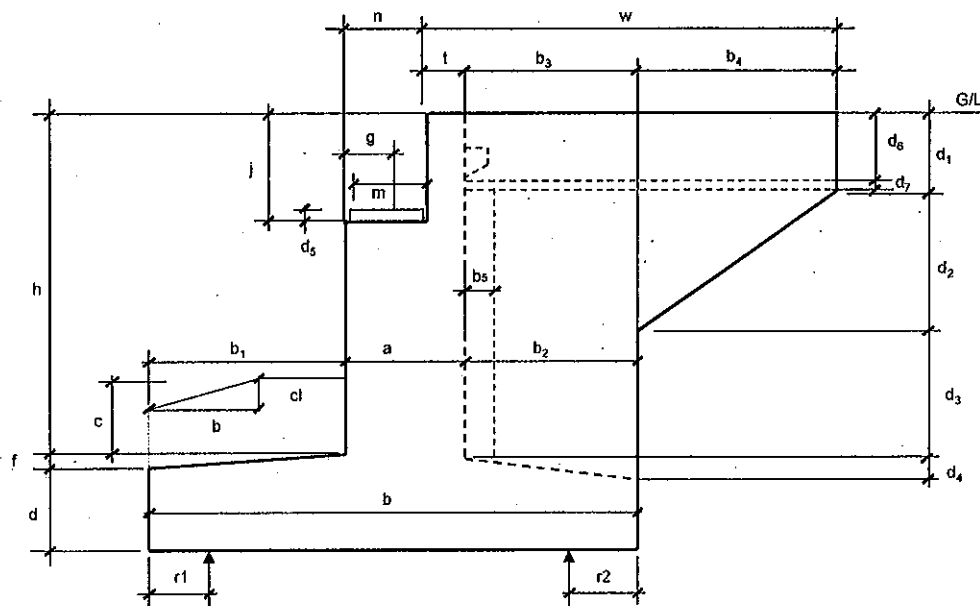
Level Table(at center of abutment)

Level of top of headwall	HTWL	17.341	m
Level of top of bearing	BTTL	15.375	m
Level of top of stem abutment	HTL	15.225	m
Level of top of footing	FTL	9.000	m
Level of bottom of footing	FBL	7.000	m
Ground level	GL	9.890	m
Highest water level	HWL	12.950	m
Skew angle	α	0.00	deg

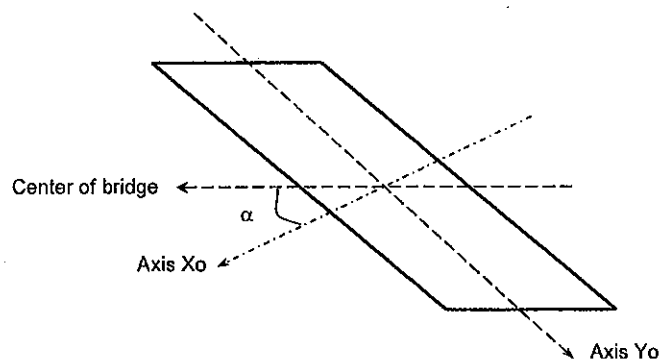
I.Loads from substructure

Abutment dimensions

VERTICAL VIEW



PLAN VIEW



Material Unit Weights

- Unit Weight of Reinf. concrete
- Unit Weight of Soil
- Unit Bouyancy Weight of Soil

$$\begin{aligned}\gamma_c &= 24.5 \text{ kN/m}^3 \\ \gamma_s &= 17.7 \text{ kN/m}^3 \\ \gamma_{sbo} &= 7.8 \text{ kN/m}^3\end{aligned}$$

	Da Nang Quang Ngai Expressway project OP19 BRIDGE DETAIL DESIGN ABUTMENT A1	Item.	Eng.	Date.	Sign.
		Design			
		Check			
		Revise			

ABUTMENT DIMENSIONS (IN METRES)					
Item	Symbol	Value	Item	Symbol	Value
Height of stem	h	8.341	Horizontal Dimension	b ₃	2.500
Footing Width	b	6.000	Horizontal Dimension	b ₄	4.000
Stem Width	a	1.500	Horizontal Dimension	b ₅	0.300
Footing Depth	d	2.000	Vertical Dimension	d ₁	2.000
Footing Slope	f	0.000	Vertical Dimension	d ₂	4.000
Width of stem at bearing	n	1.000	Vertical Dimension	d ₃	2.341
Ballast Wall Height	j	2.116	Vertical Dimension	d ₄	0.000
Ballast Wall Thickness	t	0.500	Vertical Dimension	d ₅	0.150
Wingwall Length	w	7.000	Vertical Dimension	d ₆	1.200
Soil Cover at Toe	c	0.890	Vertical Dimension	d ₇	0.300
Girder Reaction	g	0.550	With of bearing pad	m	0.510
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	2.500	Distance to cl of pile	r2	1.000

Slope front of abutment

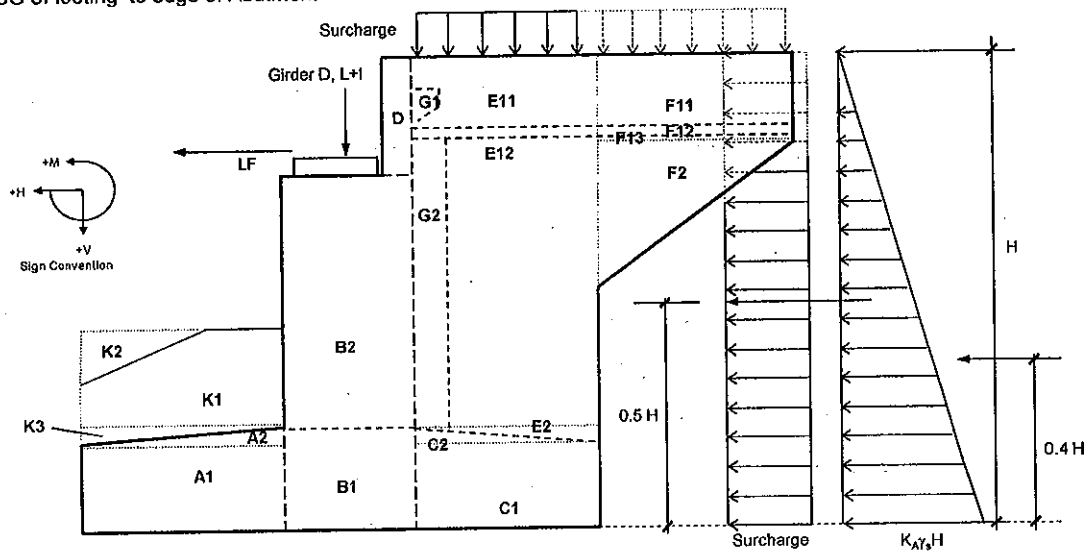
Width of Abutment

Width of abutment (Inclined direction)

Height of Abutment

Distance from CG of footing to edge of Abutment

$$\begin{aligned}
 \cos(\alpha) &= 1.00 \\
 cl &= 0.00 \text{ m} \\
 bl &= 0.00 \text{ m} \\
 L &= 12.600 \text{ m} \\
 Ltr &= 12.600 \text{ m} \\
 Ht &= 10.34 \text{ m} \\
 b/2 &= 3.00 \text{ m}
 \end{aligned}$$



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	9.338	12.600	2882	2.750	0.250	721
Section C1	5.000	12.600	1544	4.750	-1.750	-2701
Section C2	-	12.600	-	4.333	-1.333	-
Section D	1.058	12.600	327	3.250	-0.250	-82
Section E11	4.250	1.000	104	4.750	-1.750	-182
Section E12	15.853	1.000	388	4.750	-1.750	-680
Part extra stem	5.171	0.740	94	5.417	-2.417	-227
Section F11	4.800	1.000	118	8.000	-5.000	-588
Section F12	0.975	1.000	24	6.750	-3.750	-90
Section F13	2.000	1.000	49	8.000	-5.000	-245
Section F2	8.000	1.000	196	7.333	-4.333	-849
Section G1	0.135	11.600	305	3.650	-0.650	-198
Section G2	0.045	13.682	15	3.650	-0.650	-10
Bearing seats (w1seat= 0.61m)	0.077	3.050	9	2.550	0.450	4
Curbs +Handrail on Abutment	-	7.000	217	6.500	-3.500	-759
Total SW of Abutment (DC)			8431			-3184

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.

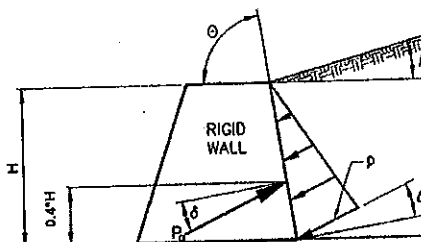
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2. Earth on Abutment (EV)

Description	Area (m ²)	Length (m)	Force (kN)	X ⁽¹⁾ (m)	Arm ⁽²⁾ (m)	Moment (kN·m)
Earth on Abutment (EV)						
Section E1	20.85	11.600	4271	4.750	-1.750	-7475
Section E2	-	11.600	-	5.167	-2.167	-
Section E3	-	1.000	-	6.000	-3.000	-
Section K1	1.780	12.600	396	1.000	2.000	-
Section K2	-	12.600	-	-	3.000	-
Section K3	-	12.600	-	0.667	2.333	-
Total Earth on Footing			4667			-7475

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
- Width for horizontal earth pressure
- Density of Soil
- Internal Friction Angle of Soil
- Incline angle of back face wall
- Friction angle between fill and wall
- Incline angle of fill soil
- Gravitational acceleration
- Basic earth pressure

$$p = K \cdot \gamma_s \cdot g \cdot Z \cdot 10^{-9} \text{ (Mpa, Z: mm)}$$

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

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

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

Horizontal earth pressure:

- $E_a = 0.5 \cdot p \cdot Z \cdot B \cdot 10^3 \text{ (kN)}$
- $M = E_a \cdot 0.4H$
- Horizontal Earth Pressure act at a height of $0.4H$

H	=	10.34 m
W	=	12.60 m
γ_s	=	1800 kg/m ³
ϕ'_f	=	30.0 deg
θ	=	90.0 deg
δ	=	0.0 deg
β	=	0.0 deg
g	=	9.81 m/s ²

Γ	=	2.250
K_a	=	0.333
p	=	0.061 Mpa

E_a	=	3965 kN
M	=	16402 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=	10.34m heq=	0.61 m

(Linear interpolation)

- Vertical force

ESv	=	339 kN
ev	=	-1.75 m
M	=	-594 kNm

- Horizontal force

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

ESh	=	468 kN
eh	=	5.17 m
M	=	2419 kNm

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5. Earthquake effects

Bridge is located at: Tam My district - Quang Nam province
According to TCXDVN 375:2006 and 22TCN272-05, bridge is in seismic zone 1 and acceleration coefficient as below

- Peak ground acceleration coefficient $A = 0.0580 g$

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

- Backfill slope angle $i = 0.0 \text{ deg}$
- Slope of wall to vertical $\beta' = 0.0 \text{ deg}$
- Angle of friction of soil $\phi = 30.0 \text{ deg}$
- Angle of friction between soil and abutment $\delta = 0.0 \text{ deg}$
- Horizontal acceleration coefficient $k_h = 0.087$
- Vertical acceleration coefficient $k_v = 0.035$
- Angle $\theta = \arctan(k_h / (1 - k_v)) = 5.2 \text{ deg}$

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

- Seismic active lateral Earth pressure coefficient $K_{AE} = 0.39$

$$E_{AE} = 0.5 \cdot g \cdot \gamma \cdot H^2 \cdot (1 - k_v) \cdot K_{AE} \cdot 10^{-9} \text{ (kN/m)}$$

- Seismic active lateral Earth pressure coefficient $E_{AE} = 4476 \text{ kN}$
 $M_{AE} = E_{AS} \cdot 0.3H^3 (E_{AE} - E_{AS}) \cdot 0.6H$
 $M_{AE} = 15468 \text{ KNm}$

<A.11.1.1.1>

E_{AS} is the static component of seismic active pressure calculated with $\theta = k_v = 0$

5.2. Earthquake effects to abutment (EQ)

Seismic force for substructures: elements above ground $F_h = C_{sm} \cdot W$; elements under ground $F_h = A \cdot S \cdot W$

- Soil profile type $S = 1.0$
- Site Coefficients $2.5A = 0.145$
- Elastic Seismic Response Coefficient $C_{sm} = 0.065$
- $C_{sm} = 1.2 \cdot A \cdot S / T_m^{2/3} \leq 2.5 \cdot A$
- Period of vibration of the fundamental mode $T_m = 1.118 \text{ s}$
- $T_m = 2 \cdot \pi \cdot \sqrt{m/k}$

Description	Area (m ²)	Length (m)	Force (kN)	X ⁽¹⁾ (m)	Arm ⁽²⁾ (m)	Moment (kN·m)
Section A1	4.000	12.600	71.62	-	1.000	72
Section A2	-	12.600	0.00	-	2.000	-
Section B1	3.000	12.600	53.71	-	1.000	54
Section B2	9.338	12.600	186.22	-	5.113	952
Section C1	5.000	12.600	89.52	-	1.000	90
Section C2	-	12.600	0.00	-	2.000	-
Section D	1.058	12.600	21.10	-	9.283	196
Section E11	4.250	1.000	6.04	-	7.741	47
Section E12	15.853	1.000	22.53	-	3.421	-
Section E2	5.171	0.740	5.44	-	2.000	11
Section F11	4.800	1.000	6.82	-	7.741	53
Section F12	0.975	1.000	1.39	-	6.991	-
Section F13	2.000	1.000	2.84	-	8.091	-
Section F2	8.000	1.000	11.37	-	7.008	80
Section G1	0.135	11.600	2.23	-	7.628	17
Section G2	0.045	13.682	0.87	-	3.421	3
Total EQ of Abutment Selfweight			482			1573

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6. Braking Force(BR)

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

- Number of lanes
- Multiple presence factor
- Take 25 % of Truck load
- BR = 25% * n * m * (2*145+35)
- Acting at 1.8m higher of road face

n	=	3 lanes	
m	=	0.85	
BR	=	104 kN	Long. Axis
e	=	12.2 m	
Mlong	=	1265 KNm	Long. Axis

7. Centrifugal Force . CE (3.6.3)

- Plan of bridge (1: "straight", 2: "Curve")
- Design Speed

$$C = 4/3 * (V^2 / gR)$$

Acting at 1.8m higher of road face

$$CE = n * m * (2*145+35) * C$$

		1	
V	=	60 km/h	
V	=	16.7 m/s	
R	=	- m	
C	=	-	
CE	=	0.00 KN	
e	=	12.23 m	
Mtrans	=	0.00 KNm	Trans. Axis

8. Water Load (WA) :NA

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SUPERSTRUCTURE LOADS

II. Loads from superstructure

Item	Sign	Value	Unit
Span length	Lsp	27.00	m
Span between bearings	Lb	26.20	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	0.22	m
Asphalt depth	has	0.084	m
Unit weight of concrete	yc	24.50	kN/m3
Unit weight of asphalt concrete	ya	22.10	kN/m3

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

Item	Sign	Value	Unit
1.1. Girders			
Weight of 1 girder	DC	437.33	kN
Number of girders	n	5	Girders
Sum of girders weight	DC	2186.63	kN
Precast Planks	DC	493.04	kN
Diaphragm	DC	268.30	kN
Total	DC	2947.97	kN
1.2. Deck slab			
Deck slab	DC	1680.00	kN
1.3. Pavement			
Asphalt concrete	DW	588.44	kN
1.4. Handrail			
Handrail + median	DC	1009.80	kN

2. Live load (LL):

Truck	<div>145 145 35 kN 4.3 4.3 m</div>
Tandem	<div>110 110 kN 1.2 m</div>
Lane load	<div>w_l 9.3 kN/m</div>
Pedestrian	$W_{pd} = 0.0 \text{ kN/m}^2$
Considerate structure as a simple span	
Reaction Influence	<div>1 26.2 m</div>
Number of lanes	$n = 3$
Multiple presence factor	$m = 0.85$
Dynamic load allowance	$1 + IM = 1.25$

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

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 superstructure (EQ)

Longitudinal moveable bearings at Abutment

Horizontal force from superstructure due to EQ - transverse direction
At bearing

$$Heq = 201 \text{ kN}$$

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

Bearing displacement due to uniform temperature and shrinkage creep

$$H = G.A.\Delta u/h_t$$

Shear modulus G

Bearing area

Height of elastomeric layers

Number of bearing

Horizontal force due to TU+SH&CR

Acting at top of bearing

$$\Delta u = 0.026 \text{ m}$$

<14.6.3.1-2>

$$G = 1 \text{ MPa}$$

$$A = 0.158 \text{ m}^2$$

$$h_{rt} = 0.066 \text{ m}$$

$$n_b = 5 \text{ bears}$$

$$H(tu+sh+cr) = 310 \text{ kN}$$

5. Wind loads (Ws)

5.1. Transverse wind on superstructure (WS)

Wind zone

Basic 3 second gust wind

Correction factor

Design wind velocity

Drag coefficient

Overall width of bridge

Depth of superstructure (including solid parapet)

Windy obstructed area of superstructure

Force due to transverse wind

$$F_{hy} = \max(0.0006 \cdot V^2 \cdot A_t \cdot C_d, 1.8 \cdot A_t) \text{ (kN)}$$

Zone IIB

$$V_b = 59.00 \text{ m/s}$$

$$S = 1.09$$

$$V = 64.31 \text{ m/s}$$

$$C_d = 1.40$$

$$b = 12.74 \text{ m}$$

$$d = 2.88 \text{ m}$$

$$b/d = 4.42$$

$$A_t = 77.76 \text{ m}^2$$

$$F_{hy} = 270.6 \text{ kN} \quad <3.8.1>$$

5.2. Wind load on vehicles (WL)

Transverse wind on vehicles

Transverse horizontal force due to wind on liveload

At 1.8m from surface

$$W_{ltran} = 1.50 \text{ kN/m}$$

$$F_{hy} = 40.50 \text{ kN}$$

6. Combinations

Loads from superstructure to Abutment

Loads at bottom of stem		Vertical		Longitudinal			Transversal		
Loads	Sign	N (kN)	x (m)	Hx (kN)	z ₁ (m)	My (kN·m)	Hy (kN)	y (m)	Mx (kN·m)
Girder + Decksab	DC	2314	0.20			463			
Handrail	DC	505	0.20			101			
Pavement	DW	294	0.20			59			
LiveLoad	LL	1234	0.20			247		1.74	2147
Pedestrian	PL	0	0.20			0			
Trans. wind on Struc.	WS						135	6.23	842
Trans. wind on vehi.	WL						20	8.03	163
Eearth quake	EQ						201	6.23	1252
TU+SH&CR	TU+SH&CR			310	6.23	1931			

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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 + Decksab	DC	2314	0.45			1041			
Handrail	DC	505	0.45			227			
Pavement	DW	294	0.45			132			
LiveLoad	LL	1234	0.45			555		1.74	2147
Pedestrial	PL	0	0.45			0		-	-
Trans. wind on Struc.	WS						135	8.23	1113
Trans. wind on vehi.	WL						20	10.03	203
Eearth quake	EQ						201	8.23	1654
TU+SH&CR	TU+SH&CR			310	8.23	2552			

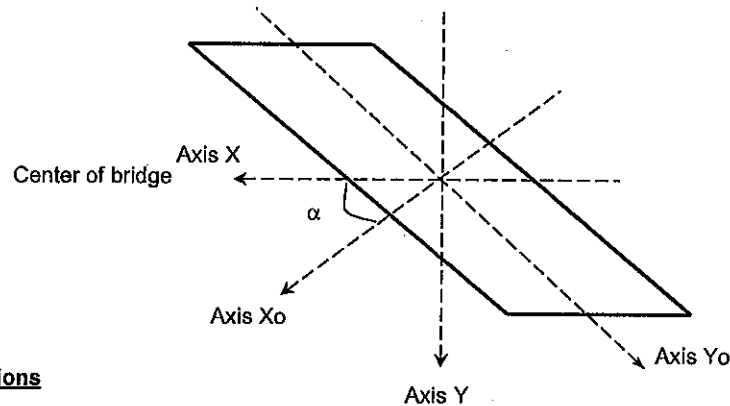
Loads	Sign	Load combinations						
		Str-IA	Str-IB	Str-IIIA	Str-IIIB	Ser-I	Ext-IA	Ext-IB
Girder + Decksab	DC	1.25	0.90	1.25	0.90	1.00	1.25	0.90
Handrail	DC	1.25	0.90	1.25	0.90	1.00	1.25	0.90
Pavement	DW	1.50	0.65	1.50	0.65	1.00	1.50	0.65
LiveLoad	LL	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Pedestrial	PL	1.75	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					
Load combinations	N (kN)	Longitudinal		Tranversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	6125	155	2191	0	3758
Strength Str-IB	4888	155	1943	0	3758
Strength Str-IIIA	5631	155	2092	74	3398
Strength Str-IIIB	4394	155	1844	74	3398
Service Ser-I	4347	310	2801	61	2563
Extreme Ext-IA	4582	0	916	201	2326
Extreme Ext-IB	3345	0	669	201	2326

Load combinations at bottom of pilecap					
Load combinations	N (kN)	Longitudinal		Tranversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	6125	155	4032	0	3758
Strength Str-IB	4888	155	3475	0	3758
Strength Str-IIIA	5631	155	3810	74	3547
Strength Str-IIIB	4394	155	3253	74	3547
Service Ser-I	4347	310	4508	61	2684
Extreme Ext-IA	4582	0	2062	201	2728
Extreme Ext-IB	3345	0	1505	201	2728

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LOAD COMBINATIONS



III. Load Combinations

1. Loads from substructure

Loads at bottom of pilecap	Sign	Vertical		Longitudinal			Tranversal		
		N (kN)	x (m)	Hx (kN)	z ₁ (m)	My (kN.m)	Hy (kN)	y (m)	Mx (kN.m)
Self weight of Abutment	DC	8431				-3184			625.254
Soils on pilecap	EV	4667				-7475			
Horizontal Earth Pressure	EH			3965		16402			
Vertical Surcharge	LSv	339				-594			
Horizontal Surcharge	LSH			468		2419			
Braking Force	BR			104		1265			
Centrifugal Force	CE			-		-			-
Buoyancy of Abutment	WA	-				-			-
Buoyancy of Earth on Abutment	WA	-				-			-
Earthquake effects to Abutment	EQ			482		1573	145		472
Earthquake effects to soil	E _{AE}			4476		15468			

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
Centrifugal 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	17434	6948	15940	0	782
Strength Str-IB	12383	4569	10577	0	563
Strength Str-IIIA	17298	6719	14704	0	782
Strength Str-IIIB	12247	4340	9341	0	563
Service Ser-I	13438	4537	8834	0	625
Extreme Ext-IA	17010	5243	4514	145	1253
Extreme Ext-IB	11959	5243	8992	145	1035

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2. Loads from superstructure

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	6125	155	4032	0	3758
Strength Str-IB	4888	155	3475	0	3758
Strength Str-IIIA	5631	155	3810	74	3547
Strength Str-IIIB	4394	155	3253	74	3547
Service Ser-I	4347	310	4508	61	2684
Extreme Ext-IA	4582	0	2062	201	2728
Extreme Ext-IB	3345	0	1505	201	2728

3. Total loads at bottom of pilecap

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	19890	7103	21530	0	4539
Strength Str-IB	13602	4724	15610	0	4321
Strength Str-IIIA	19260	6875	20072	74	4329
Strength Str-IIIB	12972	4495	14152	74	4110
Service Ser-I	14116	4847	14899	61	3310
Extreme Ext-IA	17923	5243	8134	346	3981
Extreme Ext-IB	11635	5243	12056	346	3763

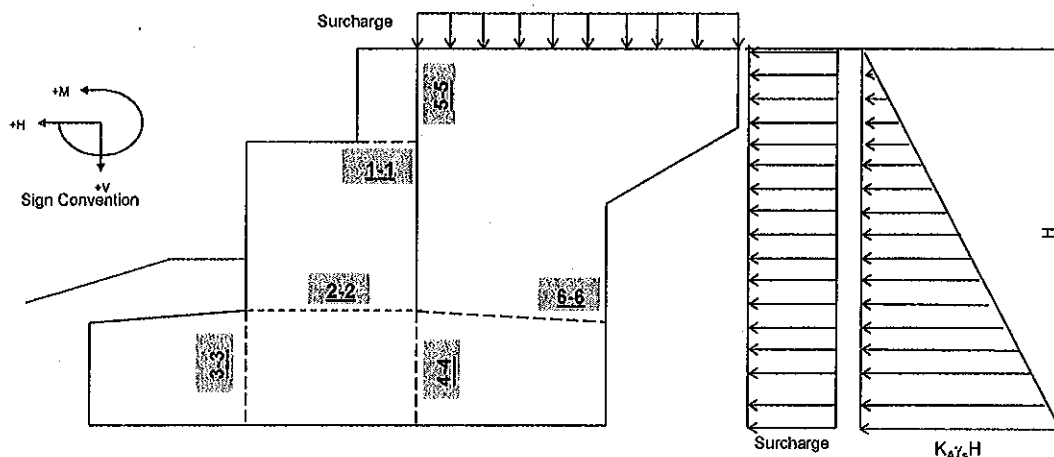
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ELEMENTS CHECKING

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		Extreme 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	
Surcharge (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		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Selfweight	631		-122		
Horizontal Earth Pressure		166	141		
Surcharge (horizontal)		235	248		
Horizontal Seismic Earth Pressure		187	133		
Abutment earthquake force		23	25	7	7

Load Combination at bottom of headwall					
Load combinations	N (kN)	Longitudinal		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Service Ser-I	631	401	267	0	0
Strength Str-IA	789	660	493	0	0
Strength Str-IB	568	560	451	0	0
Extreme Ext-I	789	422	195	7	7

1.2 Section 2-2

Table of Load Factors

Loads	Sign	Service I Ser-I	Strength I		Extreme 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	
Surcharge (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

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Unfactored Loads	N (kN)	Longitudinal		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Selfweight	3514		-194		
Superstructure Dead Load	2314		463		
Pavement	294		59		
Handrail+curb	505		101		
Live Load	1234		247		2147
Horizontal Earth Pressure		2580	8607		
Surcharge (Horizontal)		398	1659		
TU+SH&CR		310	1931		
Horizontal Seismic Earth Pressure		2912	8117		
Abutment earthquake force		210	750	123	610

Load Combination at bottom of stem wall					
Load combinations	N (kN)	Longitudinal		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Service Ser-I	7861	3288	12873	0	2147
Strength Str-IA	10517	4721	17762	0	3758
Strength Str-IB	8050	3173	12418	0	3758
Extreme Ext-I	8974	4776	14429	123	1683

1.3 Section 3-3

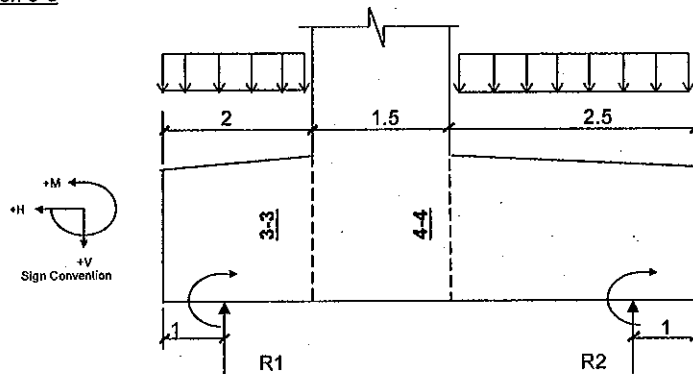


Table of Load Factors

Loads	Sign	Service I Ser-I	Strength I		Extreme I Ext-I
			Str-IA	Str-IB	
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		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Selfweight at front side	-1235		-1235		
Vertical soil on foot at front side	-396		-396		
Reaction of piles					
Ser-I	15494	3029	23660	-17	-17
Str-IA	21538	4439	33458	19	94
Str-IB	15324	2956	23261	13	73
Ext-I	16072	3275	25156	-193	-508

Load Combination at section 3-3					
Load combinations	N (kN)	Longitudinal		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Service Ser-I	13863	3029	22029	-17	-17
Strength Str-IA	19460	4439	31379	19	94
Strength Str-IB	13856	2956	21794	13	73
Extreme Ext-I	13994	3275	23078	-193	-508

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1.4 Section 4-4

Table of Load Factors

Loads	Sign	Service I Ser-I	Strength I		Extreme I Ext-I
			Str-IA	Str-IB	
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		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Selfweight of behind side	-2516		-4310		
Vertical soil on foot at behind side	-4271		-5339		
Surcharge(Vertical)	-339		-424		
Reaction of piles					
Ser-I	2291	1817	-1481	-32	-75
Str-IA	2016	2663	-4126	-19	-31
Str-IB	1951	1773	-1834	-13	-14
Ext-I	1475	1964	-3115	-150	-408

Load Combination at section 4-4					
Load combinations	N (kN)	Longitudinal		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Service Ser-I	-4835	1817	-11534	-32	-75
Strength Str-IA	-7489	2663	-17463	-19	-31
Strength Str-IB	-4751	1773	-11260	-13	-14
Extreme Ext-I	-7606	1964	-15922	-150	-408

1.4 Section 5-5 & 6-6

Slope of triang pressure
Uniform horizontal pressure

$$\begin{aligned} \tan \beta &= 5.89 \\ U.p &= 3.59 \text{ kN/m}^2 \end{aligned}$$

Load Combination at section 5-5					
Load combinations	N (kN)	Horizontal		Vertical	
		Hx (kN)	Mx (kN.m)	Hy (kN)	My (kN.m)
Service Ser-I		69	181		
Strength Str-IA		108.77	288.4		

Load Combination at section 6-6					
Load combinations	N (kN)	Horizontal		Vertical	
		Hx (kN)	Mx (kN.m)	Hy (kN)	My (kN.m)
Service Ser-I				139.3	149.6
Strength Str-IA				214.9	231.9

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2. Elements Checking

2.1. Bearing Resistance

<S.5.7.5>

The case of absence of confinement reinforcement in the concrete supporting the bearing device

Factored bearing resistance shall be taken

$$Pr = \phi \cdot Pn = \phi \cdot 0.85 \cdot f_c \cdot A1 \cdot m$$

Dimension of bearing plate

$$w0 = 0.510 \text{ m}$$

$$b0 = 0.610 \text{ m}$$

$$A1 = 0.311 \text{ m}^2$$

Area under bearing device

$$w = 1.000 \text{ m}$$

Distributed width and length

$$b = 1.100 \text{ m}$$

$$A2 = 1.100 \text{ m}^2$$

Notational area

Where supporting surface is wider on all sides than loaded area

$$m = \sqrt{A2/A1} \leq 2.0 \quad \text{case 1}$$

where loaded area is subjected to nonuniformly distributed bearing

$$m = 0.75 \cdot \sqrt{A2/A1} \leq 1.5 \quad \text{case 2}$$

Modification factor

case 1

$$m = 1.880$$

Resistance factor

$$\phi = 0.700$$

<S.5.5.4.2>

Factored bearing resistance

$$Pr = 10442 \text{ kN}$$

> Pu

Bearing reaction of approach bridge

$$Pu = 4564 \text{ kN}$$

Ok

$$Pu = 1.25 \cdot DC + 1.5 \cdot DW + 1.75 \cdot LL$$

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>

$$Pr = \phi \cdot fn \cdot Ab$$

fn take the lesser of

$$fn = 0.7 \cdot f'_{ci} \cdot \sqrt{A/A_g} \text{ and}$$

$$fn = 2.25 \cdot f'_{ci}$$

$$fn = 39.49 \text{ MPa}$$

Maximum area of the portion of supporting surface

$$A = 1.100 \text{ m}^2$$

Gross area of bearing plate

$$A_g = 0.311 \text{ m}^2$$

Effective net area of bearing plate, Ag minus stud of bearing

$$A_b = 0.311 \text{ m}^2$$

Nominal concrete strength at time of application

$$f'_{ci} = 30 \text{ MPa}$$

Factored bearing resistance

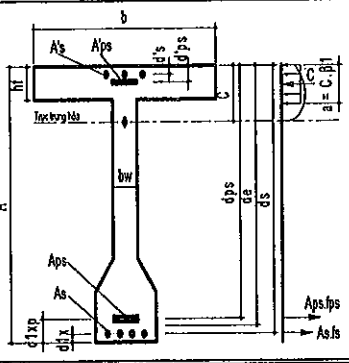
$$Pr = 8599 \text{ kN}$$

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				
E _c	Modulus of Elasticity	Mpa	27691				
f _r	Modulus of Rupture	Mpa	3.5				
g _c	Unit weight of concrete	kN/m3	24.5				
PRESTRESSING STEEL							
f _{pu}	Tensile strength of prestressing steel	Mpa	1860				
f _{py}	Yield strength of prestressing steel	Mpa	1670				
E _p	Modulus of Elasticity	Mpa	197000				
REINFORCEMENT							
f _y	Yield strength	Mpa	400				
E _s	Modulus of Elasticity	Mpa	200000				
n _c	Ratio E _s /E _c		7				
Sign	Parameters	Unit	Sections				
			1-1	1-1	2-2	2-2	2-2
INTERNAL FORCES AT SECTION							
	Combination		Strength	Service	Service	Strength	Extreme
Q _u	Shear	kN	660	401	3288	4721	4776
M _u	Flexural Moment	kNm	493	267	12873	17762	14429
N _u	Axial load	kN	789	631	7861	10517	8974
T _u	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
ds	Dis. From comp. fiber to centroid of tension Reinf	m	0.050	0.050	0.050	0.050	0.050
d's	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	12.600	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
I _z	Moment of inertia of section	m4	0.131	0.131	3.544	3.544	3.544
A _{mc}	Section area	m2	6.300	6.300	18.900	18.900	18.900
Steel choice							
A _{ps}	Tension prestressing steel	P.S type	0	0	0	0	0
		Number	0	0	0	0	0
		Area	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	0	0	0	0	0
		Area	0.00000	0.00000	0.00000	0.00000	0.00000
A _s	Tension Reinforcement	Number	99	99	94	94	94
		Diameter	16	16	25	25	25
		Area	0.02000	0.02000	0.04615	0.04615	0.04615
A's	Compression Reinforcement	Number	99	99	94	94	94
		Diameter	16	16	16	16	16
		Area	0.02000	0.02000	0.01899	0.01899	0.01899
A'c	Shear reinforcement	Number	20	20	19	19	19
		Diameter	14	14	14	14	14
		Area	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
φ _v	Resistance factors for shear		0.90	1.00	1.00	0.90	1.00
φ _n	Resistance factors for axial force		1.00	1.00	1.00	1.00	1.00
β 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	0.040	0.040	0.040
	For T section behavior	m	0.000	0.000	0.040	0.040	0.040
	For rectangular section behavior	m	0.000	0.000	0.040	0.040	0.040
f _{pe}	Effective stress in the prestressing steel after losses	Mpa	1116	1116	1116	1116	1116
f _{ps}	Aver. stress in pres. steel at the time for which the nominal resistance	Mpa	1860	1860	1846	1846	1846
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 - HEAD AND STEM WALL							
a	Depth of equivalent stress block	m	0.000	0.000	0.034	0.034	0.034
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	3072	3072	25979	25979	25979
Mr	Factored resistance	kNm	2765	3072	25979	23381	25979
Mu	Flexural moment	kNm	493	267	12873	17762	14429
(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*Mc	Cracking moment	kNm	1087	1087	10054	10054	10054
(5.7.3.3.2)	Checking $M_r \geq \min(1.2M_c, 1.33M_u)$		OK	OK	OK	OK	OK
(5.7.3.4)	Control of cracking by distr. of reinf for RC member- Check?		No	Yes	Yes	No	No
	Existing condition for structure	1,2 or 3	1	3	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	17500	30000	30000	30000
A	Area of concr. with same centroid as tens. Reinf	m ²	0.015	0.015	0.016	0.016	0.016
f _{sa}	Value	Mpa	316	184	307	307	307
0.6*f _y		Mpa	240	240	240	240	240
	Tensile stress in reinf Min(f _{sa} , 0.6f _y)	Mpa	240	184	240	240	240
x	Dist. From compression fiber to centroid	m	-	0.089	0.247	-	-
J.d	Arm	m	-	0.412	1.359	-	-
I _{cr}	Moment of inertia of the cracked section	m ⁴	-	0.021	0.528	-	-
f _s	Tensile stress in reinforcement $f_s = M_{sls} / (A_s * J.d)$	Mpa	-	32	205	-	-
	Checking for control cracking $f_s < f_{sa}$		N.a	OK	OK	N.a	N.a
(5.10.8.2)	Shrinkage and temperature Reinforcement (side distribution)						
A _{req}	Area of required reinf	m ²	0.00045	0.00045	0.00126	0.00126	0.00126
	Distribution on sides	m ²	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		3.2	3.8	2.3	2.2	2.2
θ	Angle of inclination of diagonal compressive	degree	28.66	27.52	33.83	37.19	35.67
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	90
b _v	Effective web width as minimum web width - in dv	m	12.600	12.600	12.600	12.600	12.600
d _v	Effective shear depth	m	0.442	0.442	1.424	1.424	1.424
	(d _e - a/2)	m	0.442	0.442	1.424	1.424	1.424
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	0.600
n _{cat}	Amount of bars in spacing S	bars	20	20	19	19	19
A _v	Shear reinf area in spacing S	m ²	0.0030	0.0030	0.0029	0.0029	0.0029
θ	Assume	degree	28.66	27.52	33.83	37.19	35.66
v	Shear stress in concrete	kN/m ²	132	72	183	292	266
f _{po}	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e _s	Strain in tensile reinforcement		3.31E-04	1.68E-04	8.19E-04	1.12E-03	9.72E-04
	if e _x < 0, multiple with reduce factor		-	-	-	-	-
	Strain checking	$\leq 2.00E-3$	Ok	Ok	Ok	Ok	Ok
v/f _c	Ratio of shear stress and f _c		0.004	0.002	0.006	0.010	0.009
β	Final value		3.2	3.8	2.3	2.2	2.2
θ	Final value	degree	28.66	27.52	33.83	37.19	35.67
V _c	Nominal shear resistance provided by tensile stresses in the concrete	kN	8033	9665	19017	17649	18318
V _s	Shear resistance provided by shear reinforcement	kN	1628	1708	4064	3590	3795
V _p	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0	0
V _{n1}	V _{n1} = V _c + V _s + V _p	kN	9661	11374	23081	21240	22114
V _{n2}	V _{n2}	kN	41769	41769	134577	134577	134577
V _n	Nominal shear resistance V _n = min(V _{n1} , V _{n2})	kN	9661	11374	23081	21240	22114
V _r	Factored shear resistance	kN	8695	11374	23081	19116	22114
V _u	Shear	kN	660	401	3288	4721	4776
(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	m ²	0.0086	0.0086	0.0086	0.0086	0.0086
	Minimum shear reinforcement Checking		-	-	-	-	-
	0.1*f _c *b _v *d _v	kN	16708	16708	53831	53831	53831
	S _{max}	m	0.35	0.35	0.60	0.60	0.60
	Maximum spacing S _{max}		-	-	-	-	-

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REINFORCEMENT CHECKING - PILECAP SECTION

MATERIALS				
NORMAL CONCRETE				
f _c	Compressive Strength of concrete at 28 days	Mpa	30	
E _c	Modulus of Elasticity	Mpa	27691	
f _r	Modulus of Rupture	Mpa	3.5	
g _c	Unit weight of concrete	kN/m ³	24.5	
PRESTRESSING STEEL				
f _{pu}	Tensile strength of prestressing steel	Mpa	1860	
f _{py}	Yield strength of prestressing steel	Mpa	1670	
E _p	Modulus of Elasticity	Mpa	197000	
REINFORCEMENT				
f _y	Yield strength	Mpa	400	
E _s	Modulus of Elasticity	Mpa	200000	
n _c	Ratio E _s /E _c		7	

Sign	Parameters	Unit	Sections				
			3-3	3-3	3-3	4-4	4-4
INTERNAL FORCES AT SECTION							
	Combination		Service	Strength	Extreme	Extreme	Strength
Q _u	Shear	kN	13863	19460	13994	7606	7489
M _u	Flexural Moment	kNm	22029	31379	23078	15922	17463
N _u	Axial load	kN	3029	4439	3275	1964	2663
T _u	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
I _z	Moment of inertia of section	m ⁴	8.400	8.400	8.400	8.400	8.400
A _{mc}	Section area	m ²	25.200	25.200	25.200	25.200	25.200
A _{ps}	Steel choice						
	Tension prestressing steel	P.S type	0	0	0	0	0
		Number	0	0	0	0	0
A'ps	Compression prestressing steel	P.S type	0 T00.0	0 T00.0	0 T00.0	0 T00.0	0 T00.0
		Number	0	0	0	0	0
		Area	0.00000	0.00000	0.00000	0.00000	0.00000
A _s	Tension Reinforcement	Number	100	100	100	100	100
		Diameter	32	32	32	22	22
		Area	0.08010	0.08010	0.08010	0.03800	0.03800
A's	Compression Reinforcement	Number	100	100	100	100	100
		Diameter	22	22	22	32	32
		Area	0.03800	0.03800	0.03800	0.08010	0.08010
A'c	Shear reinforcement	Number	24	24	24	24	24
		Diameter	16	16	16	16	16
		Area	0.00485	0.00485	0.00485	0.00485	0.00485
φ	Resistance factors for flexure	5.5.4.2	1.00	0.90	1.00	1.00	0.90
φ _v	Resistance factors for shear		1.00	0.90	1.00	1.00	0.90
φ _n	Resistance factors for axial force		1.00	1.00	1.00	1.00	1.00
β ₁	Stress block factor		0.836	0.836	0.836	0.836	0.836
c	Dis. Between centroid and top fiber	m	0.063	0.063	0.063	-0.063	-0.063
	For T section behavior	m	0.063	0.063	0.063	-0.063	-0.063
	For rectangular section behavior	m	0.063	0.063	0.063	-0.063	-0.063
f _{pe}	Effective stress in the prestressing steel after losses	Mpa	1116	1116	1116	1116	1116
f _{ps}	Aver. stress in pres. steel at the time for which the nominal resistance	Mpa	1843	1843	1843	1877	1877
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	0.052	0.052	0.052	-0.052	-0.052
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	57043	57043	57043	23363	23363
Mr	Factored resistance	kNm	57043	51339	57043	23363	21027
Mu	Flexural moment	kNm	22029	31379	23078	15922	17463
(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
1.2*Mer	Cracking moment	kNm	17954	17954	17954	16863	16863
(5.7.3.3.2)	Checking $Mr \geq \min(1.2Mer, 1.33Mu)$		OK	OK	OK	OK	OK
(5.7.3.4)	Control of cracking by distr. of reinf for RC member- Check?		Yes	No	No	No	No
	Existing condition for structure	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	m ²	0.017	0.017	0.017	0.015	0.015
fsa	Value	Mpa	170	170	170	179	179
0.6*fy		Mpa	240	240	240	240	240
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	170	170	170	179	179
x	Dist. From compression fiber to centroid	m	0.362	-	-	-	-
J.d	Arm	m	1.713	-	-	-	-
Icr	Moment of inertia of the cracked section	m ⁴	1.432	-	-	-	-
fs	Tensile stress in reinforcement $fs = Ms / (As * J.d)$	Mpa	161	-	-	-	-
	Checking for control cracking $fs < fsa$		OK	N.a	N.a	N.a	N.a
(5.10.8.2)	Shrinkage and temperature Reinforcement (side distribution)						
Areq	Area of required reinf	m ²	0.00127	0.00127	0.00127	0.00127	0.00127
	Distribution on sides	m ²	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	1.9	2.1	1.9	1.9
θ	Angle of inclination of diagonal compressive	degree	38.17	41.52	38.45	41.09	41.28
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	1.808	1.808	1.808	1.942	1.942
	($de - a/2$)	m	1.808	1.808	1.808	1.942	1.942
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	0.600
ncat	Amount of bars in spacing S	bars	24	24	24	24	24
Av	Shear reinf area in spacing S	m ²	0.0048	0.0048	0.0048	0.0048	0.0048
θ	Assume	degree	38.17	41.52	38.45	41.09	41.28
v	Shear stress in concrete	kN/m ²	609	949	614	33	340
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
ϵ_s	Strain in tensile reinforcement		1.22E-03	1.63E-03	1.24E-03	1.52E-03	1.57E-03
	if $\epsilon_s < 0$, multiple with reduce factor		-	-	-	-	-
	Strain checking	$\leq 2.00E-3$	Ok	Ok	Ok	Ok	Ok
v/fc	Ratio of shear stress and fc		0.020	0.032	0.020	0.001	0.011
β	Final value		2.1	1.9	2.1	1.9	1.9
θ	Final value	degree	38.17	41.52	38.45	41.09	41.28
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	21836	19569	21673	21575	21340
Vs	Shear resistance provided by shear reinforcement	kN	7434	6599	7359	7197	7151
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	29270	26167	29032	28772	28492
Vn2	Vn2	kN	170837	170837	170837	183538	183538
Vn	Nominal shear resistance $Vn = \min(Vn1, Vn2)$	kN	29270	26167	29032	28772	28492
Vr	Factored shear resistance	kN	29270	23551	29032	28772	25642
Vu	Shear	kN	13863	19460	13994	7606	7489
(5.8.2.7)	Shear checking		OK	OK	OK	OK	OK

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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	197000				
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 SECTION							
	Combination		Service	Strength	Service	Strength	Strength
Qu	Shear	kN	69.3	108.8	139	215	215
Mu	Flexural Moment	kNm	180.7	288.4	150	232	232
Nu	Axial load	kN	0.0	0.0	0	0	0
Tu	Torsional Moment	kNm	0.0	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	6	6	6	6	6
	Diameter	mm	22	22	22	22	22
	Area	m2	0.00228	0.00228	0.00228	0.00228	0.00228
A's	Compression Reinforcement	Number	6	6	6	6	6
	Diameter	mm	16	16	16	16	16
	Area	m2	0.00121	0.00121	0.00121	0.00121	0.00121
A'c	Shear reinforcement	Number	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
β1	Stress block factor		0.836	0.836	0.836	0.836	0.836
c	Dis. Between centroid and top fiber	m	0.020	0.020	0.020	0.020	0.020
	For T section behavior	m	0.020	0.020	0.020	0.020	0.020
	For rectangular section behavior	m	0.020	0.020	0.020	0.020	0.020
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	1839	1839
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 - WING WALL						
a	Depth of equivalent stress block	m	0.017	0.017	0.017	0.017
dc	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
Mn	Nominal resistance	kNm	370	370	370	370
Mr	Factored resistance	kNm	370	333	370	333
Mu	Flexural moment	kNm	181	288	150	232
(5.7.3.2)	Flexural moment Checking		OK	OK	OK	OK
	Limits for reinforcement					
c/de	Maximum reinforcement		0.05	0.05	0.05	0.05
	Maximum reinforcement Checking	≤ 0.42	OK	OK	OK	OK
r min	Minimum reinforcement		0.46%	0.46%	0.46%	0.46%
	Minimum reinforcement Checking for RC	0.23%	OK	OK	OK	OK
1.2*Mcr	Cracking moment	kNm	90	90	90	90
(5.7.3.3.2)	Checking $M_r \geq \min(1.2M_{cr}, 1.33M_u)$		OK	OK	OK	OK
(5.7.3.4)	Control of cracking by distr. of reinf for RC member- Check?		Yes	No	Yes	No
	Existing condition for structure	1, 2 or 3	1	1	1	1
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.059	0.059	0.059	0.059
Z	Crack width parameter	N/mm	30000	30000	30000	30000
A	Area of concr. with same centroid as tens. Reinf	m ²	0.020	0.020	0.020	0.020
fsa	Value	Mpa	285	285	285	285
0.6*fy	Tensile stress in reinf Min(fs, 0.6fy)	Mpa	240	240	240	240
x	Dist. From compression fiber to centroid	m	0.104	-	0.104	-
J.d	Arm	m	0.406	-	0.406	-
Icr	Moment of inertia of the cracked section	m ⁴	0.002	-	0.002	-
fs	Tensile stress in reinforcement $f_s = M_{sls} / (A_s * J.d)$	Mpa	195	-	161	-
	Checking for control cracking $f_s < f_{sa}$		OK	N.a	OK	N.a
(5.10.8.2)	Shrinkage and temperature Reinforcement (side distribution)					
Areq	Area of required reinf	m ²	0.00031	0.00031	0.00031	0.00031
	Distribution on sides	m ²	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		2.2	1.9	2.2	2.0
θ	Angle of inclination of diagonal compressive	degree	36.20	41.39	35.66	40.51
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	0.433	0.433	0.433	0.433
	(dc - a/2)	m	0.433	0.433	0.433	0.433
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	m ²	0.0002	0.0002	0.0002	0.0002
β	Assume		2.2	1.9	2.2	2.0
θ	Assume	degree	36.20	41.39	35.66	40.51
v	Shear stress in concrete	kN/m ²	160	279	322	552
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116
e _s	Strain in tensile reinforcement		1.02E-03	1.60E-03	9.71E-04	1.45E-03
	if $e_x < 0$, multiple with reduce factor		-	-	-	-
	Strain checking	$\leq 2.00E-3$	Ok	Ok	Ok	Ok
v/fc	Ratio of shear stress and fc		0.005	0.009	0.011	0.018
β	Final value		2.2	1.9	2.2	2.0
θ	Final value	degree	36.20	41.39	35.66	40.51
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	436	375	442	389
Vs	Shear resistance provided by shear reinforcement	kN	89	74	91	76
Vp	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0
Vn1	$V_{n1} = V_c + V_s + V_p$	kN	526	449	533	465
Vn2	Vn2	kN	3245	3245	3245	3245
Vn	Nominal shear resistance $V_n = \min(V_{n1}, V_{n2})$	kN	526	449	533	465
Vr	Factored shear resistance	kN	526	404	533	419
Vu	Shear	kN	69	109	139	215
(5.8.2.7)	Shear checking		OK	OK	OK	OK

SPACE PILE FOUNDATION ANALYSIS PROGRAM Turbo BASIC

PROJECT: : OP19-A1

INITIAL DATA

Kn = 0.00 Ax = 6.00 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 = 400 (t/m4)

LOAD COMBINATIONS

COMB.	Hx	Hy	Mx	My	Mz
1	724.00	0.00	2401.00	-463.00	2036.00
2	482.00	0.00	1761.00	-440.00	1432.00
3	701.00	17.00	2337.00	-432.00	1887.00
4	458.00	7.00	1696.00	-410.00	1284.00
5	494.00	5.00	1813.00	-331.00	1360.00
6	534.00	35.00	2201.00	-406.00	670.00
7	534.00	35.00	1560.00	-384.00	1070.00

PROPERTIES OF PILES

PILE	Lo	H	Bpx	Bpy	A	B	Cday	Fo	Io	Po	Co	Ct
1	0.00	15.00	1.202	1.202	1.00	0.000	0.000	0.785	0.049	0.7500000	3750000	
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	-5.30	0.000	0.00
2	-2.00	-0.13	0.000	0.00
3	-2.00	5.05	0.000	0.00
4	2.00	-5.30	0.000	0.00
5	2.00	-2.17	0.000	0.00
6	2.00	-0.13	0.000	0.00
7	2.00	2.45	0.000	0.00
8	2.00	-5.04	0.000	0.00

DISPLACEMENTS

COMB.	X	Y	Z	Fix	Fiy	Fiz
1	0.01988	0.00012	0.001763	-0.000031	0.000642	0.000052
2	0.01323	0.00010	0.001315	-0.000029	0.000426	0.000034
3	0.01919	0.00029	0.001727	-0.000031	0.000601	0.000054
4	0.01251	0.00027	0.001279	-0.000028	0.000385	0.000037

A1-8

5	0.01349	0.00021	0.001367	-0.000024	0.000412	0.000038
6	0.01400	0.00100	0.001789	-0.000036	0.000242	0.000059
7	0.01445	0.00099	0.001154	-0.000031	0.000401	0.000059

FORCES ON PILES

PILE	COMB.	N	Q2	Q3	M1	M2	M3
1	1	45.20	-89.14	-0.65	0.692	-1.056	239.091
	2	44.64	-59.35	-0.43	0.461	-0.491	159.206
	3	52.21	-86.20	-1.55	0.726	-3.670	231.699
	4	51.58	-56.28	-1.34	0.493	-3.101	151.468
	5	60.10	-60.75	-1.10	0.512	-2.555	163.578
	6	160.96	-65.21	-5.11	0.787	-13.741	180.742
	7	26.93	-65.21	-5.11	0.787	-13.859	176.649
2	1	68.51	-90.48	-0.65	0.692	-1.056	242.940
	2	66.30	-60.24	-0.43	0.461	-0.491	161.769
	3	75.06	-87.61	-1.55	0.726	-3.670	235.733
	4	72.84	-57.24	-1.34	0.493	-3.101	154.210
	5	77.86	-61.74	-1.10	0.512	-2.555	166.424
	6	187.55	-66.73	-5.11	0.787	-13.741	185.116
	7	50.11	-66.73	-5.11	0.787	-13.859	181.023
3	1	91.81	-91.83	-0.65	0.692	-1.056	246.790
	2	87.96	-61.13	-0.43	0.461	-0.491	164.331
	3	97.92	-89.01	-1.55	0.726	-3.670	239.766
	4	94.10	-58.19	-1.34	0.493	-3.101	156.952
	5	95.62	-62.73	-1.10	0.512	-2.555	169.269
	6	214.14	-68.26	-5.11	0.787	-13.741	189.490
	7	73.30	-68.26	-5.11	0.787	-13.859	185.397
4	1	415.33	-89.14	0.39	0.692	1.919	239.091
	2	290.32	-59.35	0.26	0.461	1.490	159.206
	3	399.05	-86.20	-0.47	0.726	-0.552	231.699
	4	273.82	-56.28	-0.60	0.493	-0.981	151.468
	5	297.77	-60.75	-0.34	0.512	-0.355	163.578
	6	300.55	-65.21	-3.93	0.787	-10.360	180.742
	7	258.29	-65.21	-3.93	0.787	-10.478	176.649
5	1	429.40	-89.95	0.39	0.692	1.919	241.416
	2	303.41	-59.88	0.26	0.461	1.490	160.753
	3	412.85	-87.05	-0.47	0.726	-0.552	234.135
	4	286.65	-56.86	-0.60	0.493	-0.981	153.124
	5	308.49	-61.34	-0.34	0.512	-0.355	165.296
	6	316.61	-66.13	-3.93	0.787	-10.360	183.383
	7	272.29	-66.13	-3.93	0.787	-10.478	179.291
6	1	438.61	-90.48	0.39	0.692	1.919	242.937
	2	311.97	-60.24	0.26	0.461	1.490	161.766
	3	421.89	-87.61	-0.47	0.726	-0.552	235.729
	4	295.05	-57.24	-0.60	0.493	-0.981	154.208
	5	315.51	-61.74	-0.34	0.512	-0.355	166.421
	6	327.11	-66.73	-3.93	0.787	-10.360	185.112
	7	281.45	-66.73	-3.93	0.787	-10.478	181.019
7	1	450.25	-91.15	0.39	0.692	1.919	244.859
	2	322.79	-60.68	0.26	0.461	1.490	163.046
	3	433.30	-88.31	-0.47	0.726	-0.552	237.744
	4	305.67	-57.71	-0.60	0.493	-0.981	155.577
	5	324.38	-62.23	-0.34	0.512	-0.355	167.842
	6	340.40	-67.49	-3.93	0.787	-10.360	187.296
	7	293.03	-67.49	-3.93	0.787	-10.478	183.204
8	1	461.89	-91.82	0.39	0.692	1.919	246.782
	2	333.61	-61.13	0.26	0.461	1.490	164.326
	3	444.72	-89.01	-0.47	0.726	-0.552	239.759
	4	316.29	-58.19	-0.60	0.493	-0.981	156.947
	5	333.26	-62.73	-0.34	0.512	-0.355	169.264
	6	353.68	-68.25	-3.93	0.787	-10.360	189.481
	7	304.61	-68.25	-3.93	0.787	-10.478	185.389

SUMMARY OF FORCES

PILE	COMB.	N	Q2	Q3	M1	M2	M3
------	-------	---	----	----	----	----	----

A1-8

Nmin	1	7	26.93	-65.21	-5.11	0.787	-13.859	176.649
Nmax	8	1	461.89	-91.82	-0.39	0.692	-1.919	246.782
Q2max	3	1	91.81	-91.83	-0.65	0.692	-1.056	246.790
Q3max	1	6	160.96	-65.21	-5.11	0.787	-13.741	180.742
M1max	1	6	160.96	-65.21	-5.11	0.787	-13.741	180.742
M2max	1	7	26.93	-65.21	-5.11	0.787	-13.859	176.649
M3max	3	1	91.81	-91.83	-0.65	0.692	-1.056	246.790

CHECKING CALCULATION IN COMPARISON WITH INITIAL LOAD MATRIX

1	724.00	0.00	2401.00	-463.00	2036.00	0.00
2	482.00	0.00	1761.00	-440.00	1432.00	0.00
3	701.00	7.00	2337.00	-432.00	1887.00	0.00
4	458.00	7.00	1696.00	-410.00	1284.00	0.00
5	494.00	5.00	1813.00	-331.00	1360.00	0.00
6	534.00	35.00	2201.00	-406.00	670.00	0.00
7	534.00	35.00	1560.00	-384.00	1070.00	0.00

- * Các phản lực khác nhau tại các đầu gối và các đầu cột.
- * Hiện tượng biến dạng khác nhau giữa các đầu gối và các đầu cột.

Do đó, cần phải kiểm tra lại các đầu gối và các đầu cột.

	Da Nang Quang Ngai Expressway project	Item.	Eng.	Date.	Sign.
	OP19 BRIDGE	Design			
	DETAIL DESIGN	Check			
	ABUTMENT A1	Revise			

BORED PILE DESIGN

I. BORED PILE DATA

1. Load Combinations at top of bored pile

No	Combinations	Sign	F _y (kN)	Longitudinal		Transvesal	
				F _{Hx} (kN)	My (kN·m)	F _{Hy} (kN)	Mx (kN·m)
1	Extreme Ext-IB		264	640	-1733	50	136
2	Strength Str-IA		4531	901	-2421	-4	-19
3	Strength Str-IA		901	901	-2421	6	10
4	Extreme Ext-IA		1579	640	-1773	50	135
5	Extreme Ext-IA		1579	640	-1773	50	135
6	Extreme Ext-IB		264	640	-1733	50	136

2. Bored pile Material

Normal concrete			
Compressive strength at 28 days age	f _c	30	MPa
Concrete elastic modulus	E _c	27691	MPa
Reinforcement TCVN1651-2008; CBV-400			
Yield strength	f _y	400	MPa
Reinforcement elastic modulus	E _s	200,000	MPa

3. Bored pile Section

Pile diameter	D	1.00	m
Section area	A	0.785	m ²
Moment inertia	I _x	0.049	m ⁴
	I _y	0.049	m ⁴
Radius of gyration of gross concrete section; $r = \sqrt{I/A}$	r _x	0.250	m
	r _y	0.250	m

II. PILE DESIGN

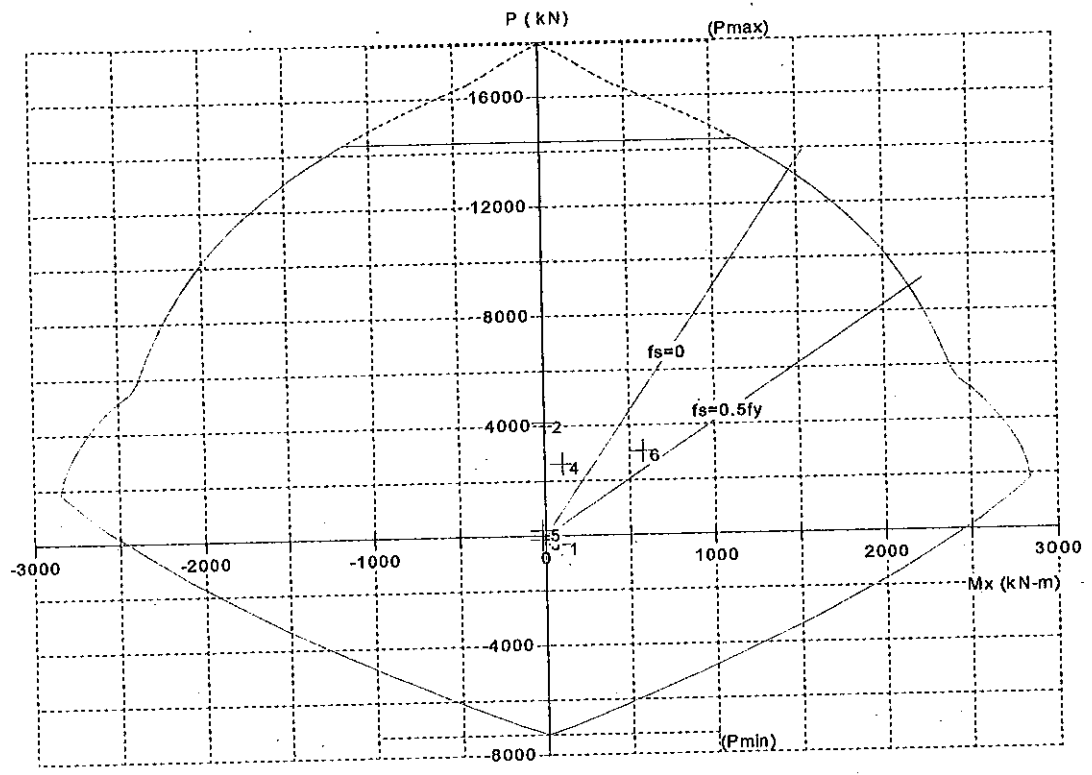
S.5.7.4.2

1. Limit of Reinforcement

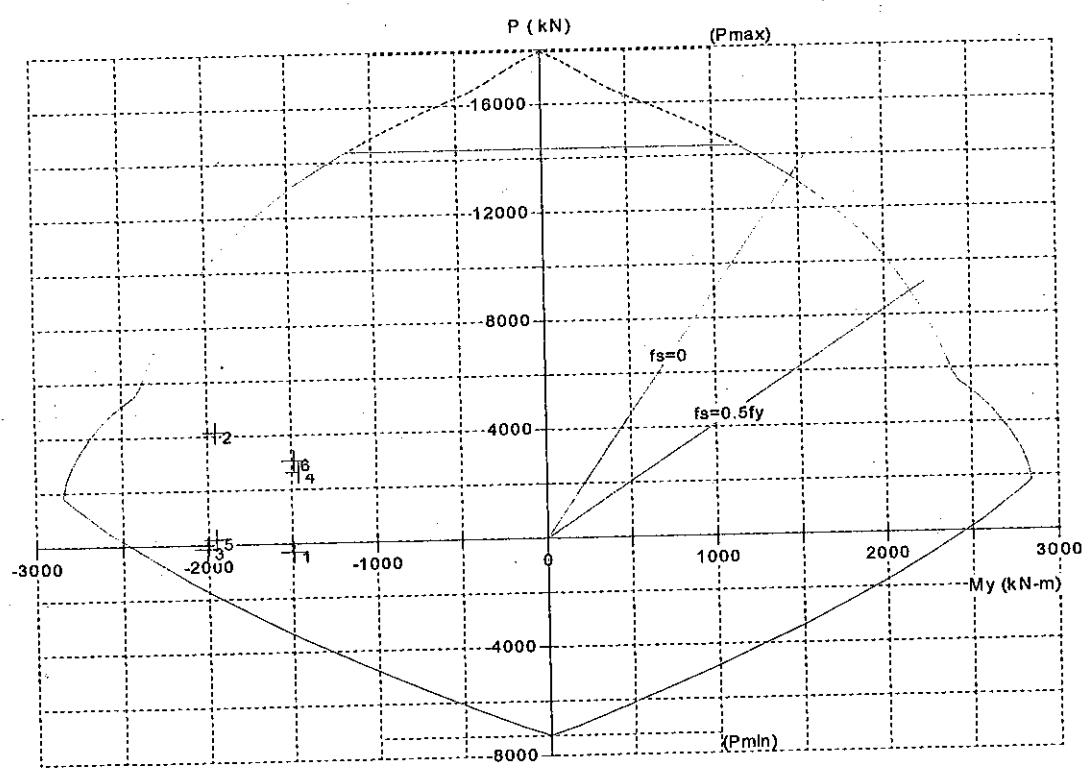
1. Limit of Reinforcement							
Minimum area of longitudinal reinforcement in column							
As.fy / (Ag . fc) >= 0.135			As ≥	0.008	m2		
As / Ag >= 0.01			As ≥	0.008	m2		
Maximum area of longitudinal reinforcement in column							
As / Ag <= 0.08			As ≤	0.063	m2		
Trial Rebars:			Ok	As	0.019	m2	
1layers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

2. Interaction diagram M-P

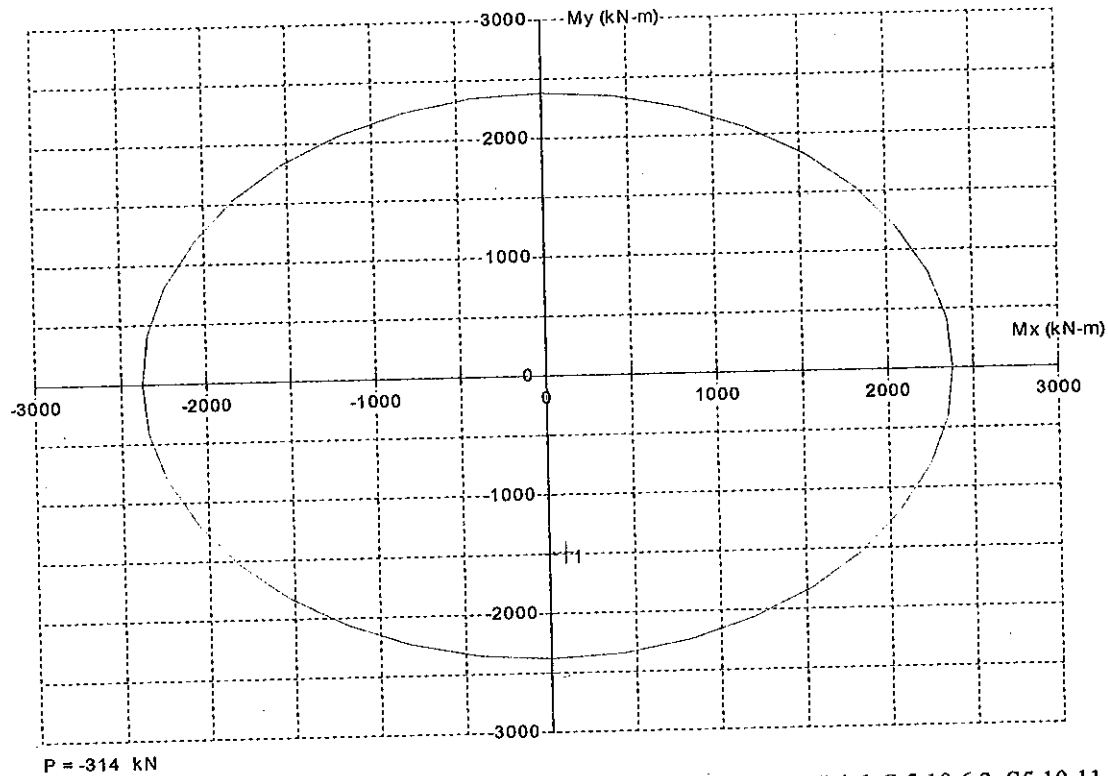
**In Transverse Direction



**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	0.622	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 reinforcement tie in 1 hoop	Ltie	2.78	m
Ratio of ties reinf. in one hoop/ volume of conc. core one pitch spacing			
$ps = As-tr \cdot Ltie / (Ac \cdot spacing)$	ps	0.0090	
Ratio of spiral reinf. To total volume of concrete core shall satisfy			S.5.7.4.6
$ps \geq 0.45 \cdot (Ag/Ac - 1) \cdot fc/fy = Req1$	Req1	0.0089	OK
			S.5.10.11.4.1.d
Transverse Reinforcement for Confinement at Plastic Hinges			
For a circular column	"1:applied", "2:Not applied"	1	
$ps \geq 0.12 \cdot fc/fy = Req2$	Req2	0.0090	N/A
Length distributed spiral with pitch 75mm below pilecap	Ldis	1.50	m

4. Shear Design

			Unit
Shear resistance factors	ϕ_v	1.0	
Factored shear force in longitudinal	V_u	901	kN
Required shear capacity $V_n = V_u / \phi_v$	V_n	901	kN
Determine concrete shear capacity			
Minimum shear reinforcement will provided in cross section			
Therefore	β	2.0	
	θ	45.0	
Cross section equivalent height	h	1.00	m
width	b	1.00	m
$d_e = h - \text{cover} - d_{lx}$	d_e	0.80	m
$d_v = \max(0.72 \cdot h; 0.9 \cdot d_e)$	d_v	0.72	m
$V_c = \min(0.083 \cdot \beta \cdot \sqrt{f_c} \cdot b_v \cdot d_v, 0.25 \cdot f_c \cdot b_v \cdot d_v)$	V_c	655	kN
Difference between required shear capacity and the capacity provided by concrete is the minimum required capacity for shear reinforcements			
$V_s = V_n - V_c$	V_s	246	kN
In this case $V_c > V_n$ so shear reinforcement is no need			
Stirrup diameter	D_s	14	
Number of stirrup legs / cross section	n_s	2	
Shear legs area	A_v	0.0003	m ²
Angle of inclination of shear reinf. to long. axis	α	90	deg
$V_s = A_v \cdot f_y \cdot d_v \cdot (\cot \theta + \cot \alpha) \cdot \sin \alpha / s$	$s \leq$	0.35	m
Stirrup spacing used	s	0.075	m
Check minimum shear reinforcement requirement		OK	
$A_v \geq 0.083 \cdot \sqrt{f_c} \cdot b_v \cdot s / f_y = \text{Req}$	Req	0.0001	m ²
Check maximum shear reinforcement spacing requirement		OK	
$F = 0.1 \cdot f_c \cdot b_v \cdot d_v$	F	2160	kN
If $V_u < 0.1 \cdot f_c \cdot b_v \cdot d_v$ then $S_{\max} = 0.8 \cdot d_v \leq 600\text{mm}$			
If $V_u > 0.1 \cdot f_c \cdot b_v \cdot d_v$ then $S_{\max} = 0.4 \cdot d_v \leq 300\text{mm}$	S_{\max}	0.58	m

DANANG QUANG NGAI EXPRESSWAY				Item.	Eng.	Date.	Sign.
OP19 BRIDGE				Design			
DETAIL DESIGN				Check			
EMPIRICAL ESTIMATION OF PILE CAPACITY-ABUTMENT A1				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	OP19	Pile Concrete comp. strength	$f_c = 30.0$ MPa
Bottom of pilecap elavation	EL1 = 7.00	Concrete Unit Weight	$\gamma_c = 24.5$ kN/m ³
Top of socket elevation	EL2 = -4.27	Modulus of elasticity of concrete	$E_c = 27691$ MPa
Pile tip elevation	EL3 = -8.00		
Pile Length	$L = 15.00$ m	Depth of socket	$H_s = 3.73$ 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	$N = 4821.1$ kN		
Working normal force at top of socket	$P_i = 4778.0$ 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	$I_p = f(H_s/D_s, E_r/E_i) = 0.30$		Figure C10.8.3.5-1 Lrfd
	$H_s/D_s = 3.73$		
	$E_r/E_i = 22.15$		
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

$$r_e = (\sum P_i) * H_s / (A_{soc} * E_c) = 0.819 \text{ mm}$$

The settlement of base of drilled shaft

$$r_{base} = (\sum P_i) * I_p / (D_s * E_r) = 1.147 \text{ mm}$$

$$r_e + r_{base} = 1.966 \text{ mm} < 10 \text{ mm}$$

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 $q_u < 1.9$ Mpa - may be taken after Carter & Kulhawy 1988 $\rightarrow q_s = 0.15 * q_u$ C10.8.3.5-4

if $q_u > 1.9$ Mpa - may be taken after Horvath & Kenney 1979 $\rightarrow q_s = 0.21 * \sqrt{q_u}$ 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 $\rightarrow q_s = 0.65 * \alpha_E * p_a * (q_u/p_a)^{0.5} < 7.8 * p_a * (f_c/p_a)^{0.5}$

$$q_{s0} = 7.8 * p_a * (f_c/p_a)^{0.5}$$

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	-4.27	-4.77	0.50	20	77.27	1.85	2900	0.65	1885
2	-4.77	-5.27	0.50	20	77.27	1.85	2900	0.65	1885
3	-5.27	-5.77	0.50	22	77.27	1.85	2900	0.65	1885
4	-5.77	-6.27	0.50	22	77.27	1.85	2900	0.65	1885
5	-6.27	-6.77	0.50	53	77.27	1.85	2900	0.65	1885
6	-6.77	-7.27	0.50	53	77.27	1.85	2900	0.65	1885
7	-7.27	-7.77	0.50	53	77.27	1.85	2900	0.65	1885
8	-7.77	-8.000	0.23	65	77.27	1.85	1334	0.65	867
Sum			3.73				21631		14060

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Case2												Type: "1: closed joints", "2: open joints"
No.	Depth (m)	RQD (%)	q_u (MPa)	E_m/E_l	α_B	Type	q_{s0} (MPa)	q_s (MPa)	$q_s - used$ (MPa)	Q_{SR} (kN)	ϕ_s	Q_R (kN)
1	0.50	20.00	77.27	0.05	0.45	1	13.58	0.82	0.82	1284	0.55	706
2	0.50	20.00	77.27	0.05	0.45	1	13.58	0.82	0.82	1284	0.55	706
3	0.50	22.00	77.27	0.06	0.46	1	13.58	0.84	0.84	1322	0.55	727
4	0.50	22.00	77.27	0.06	0.46	1	13.58	0.84	0.84	1322	0.55	727
5	0.50	53.00	77.27	0.23	0.65	1	13.58	1.18	1.18	1852	0.55	1019
6	0.50	53.00	77.27	0.23	0.65	1	13.58	1.18	1.18	1852	0.55	1019
7	0.50	53.00	77.27	0.23	0.65	1	13.58	1.18	1.18	1852	0.55	1019
8	0.23	65.00	77.27	0.56	0.83	1	13.58	1.50	1.50	1082	0.55	595
Sum	3.73									11849		6517

Unit base resistance

$$q_p = K_b \cdot (p_1 - p_0) + \sigma_v$$

Limit pressure determined from presuremeter tests

$$p_1 = \text{ - MPa}$$

C10.8.3.5-7

At rest total horizontal stress measured at ther base elevation

$$p_0 = \text{ - MPa}$$

Coefficient that depen on diameter socket

$$K_b = 4.46$$

Table C10.8.3.5-1

Total vertical stress at the base elevation

$$\sigma_v = \text{ - MPa}$$

$$q_p = \text{ - MPa}$$

$$\phi = 0.50$$

Table 10.5.5-3

$$Q_{pR} = A_p \cdot q_p$$

$$Q_{pR} = \text{ - kN}$$

$$Q_R = \phi \cdot Q_{pR}$$

$$Q_R = \text{ - kN}$$

ESTIMATED PILE CAPACITY:

Pile Structural Capacity	$Q_T = 0.75 \cdot 0.85 \cdot 0.85 \cdot f_c \cdot A_g$	12768 kN	1301 T
Pile resistance	Q_R	6517 kN	664 T
Deducting pile weight		-202 kN	-21 T
Estimated Pile Capacity		6315 kN	644 T
Maximum Reaction - ULS	Ok	4532 kN	462 T

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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	OP19	Pile Concrete comp. strength	$f'_c =$	30.0	MPa
Bottom of pilecap elevation	EL1 = 7.00	Concrete Unit Weight	$g_c =$	24.5	kN/m ³
Top of socket elevation	EL2 = -1.87	Modulus of elasticity of concrete	$E_c =$	27691	MPa
Pile tip elevation	EL3 = -8.00				
Pile Length	$L =$ 15.00 m	Depth of socket	$H_s =$	6.13	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	$N =$ 4252.2 kN				
Working normal force at top of socket	$P_i =$ 4181.3 kN				
Intack rock modulus	$E_i =$ 25000 MPa				
Modulus modification ratio	$K_o =$ 0.05				
Elastic modulus of the insitu rock	$E_r = K_o * E_i =$ 1250.0 MPa				
Influence coefficient	$I_p = f(H_s/D_s, E_c/E_r) =$ 0.30				
	$H_s/D_s =$ 6.13				
	$E_c/E_r =$ 22.15				
Rock mass modulus/ intack rock modulus	E_m/E_i				
Atmospheric pressure	$p_a =$ 0.101 MPa				
Reduction factor to account for jointing	α_E				

Figure C10.8.3.5-2 Lrfd

Figure C10.8.3.5-3 Lrfd

Figure C10.8.3.5-1 Lrfd

C.10.4.6.5-1-Lrfd 4th

10.8.3.5.4b-Lrfd 4th

The elastic shortening of the drilled shaft

$$r_e = (\sum P_i) * H_s / (A_{soc} * E_c) = 1.179 \text{ mm}$$

The settlement of base of drilled shaft

$$r_{base} = (\sum P_i) * I_p / (D_s * E_r) = 1.004 \text{ mm}$$

$$r_e + r_{base} = 2.182 \text{ mm} < 10 \text{ mm}$$

Compute the bearing capacity based on Shaft resistance alone

Unit side resistance

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

if $q_u < 1.9 \text{ Mpa}$ - may be taken after Carter & Kulhawy 1988 $\rightarrow q_s = 0.15 * q_u$

C10.8.3.5-4

if $q_u > 1.9 \text{ Mpa}$ - may be taken after Horvath & Kenney 1979 $\rightarrow q_s = 0.21 * \sqrt{q_u}$

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 $\rightarrow q_s = 0.65 * \alpha_E * p_a * (q_u/p_a)^{0.5} < 7.8 * p_a * (f'_c/p_a)^{0.5}$

$$q_{s0} = 7.8 * p_a * (f'_c/p_a)^{0.5}$$

10.8.3.5.4d-1-Lrfd 2007

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	-1.87	-2.37	0.50	0	82.83	1.91	3002	0.65	1951
2	-2.37	-3.37	1.00	15	82.83	1.91	6004	0.65	3903
3	-3.37	-4.37	1.00	12	82.83	1.91	6004	0.65	3903
4	-4.37	-5.37	1.00	0	82.83	1.91	6004	0.65	3903
5	-5.37	-6.37	1.00	0	82.83	1.91	6004	0.65	3903
6	-6.37	-7.37	1.00	13	82.83	1.91	6004	0.65	3903
7	-7.37	-8.00	0.63	67	82.83	1.91	3783	0.65	2459
8									
Sum			6.13				36807		23925

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	OP19 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	q _{s0} (MPa)	q _s (MPa)	q _s - used (MPa)	Q _{SR} (kN)	φ _s	Q _R (kN)	
1	0.50	0	82.83	0.05	0.45	1	13.58	0.85	0.85	1329	0.55	-	
2	1.00	15.00	82.83	0.05	0.45	1	13.58	0.85	0.85	2658	0.55	1462	
3	1.00	12.00	82.83	0.05	0.45	1	13.58	0.85	0.85	2658	0.55	1462	
4	1.00	0	82.83	0.05	0.45	1	13.58	0.85	0.85	2658	0.55	-	
5	1.00	0	82.83	0.05	0.45	1	13.58	0.85	0.85	2658	0.55	-	
6	1.00	13.00	82.83	0.05	0.45	1	13.58	0.85	0.85	2658	0.55	1462	
7	0.63	67.00	82.83	0.62	0.85	1	13.58	1.59	1.59	3152	0.55	1733	
8	-	-	-	-	-	-	-	-	-	-	-	-	
Sum	6.13									17770		6119	

Unit base resistance

$$q_p = K_b(p_1 - p_0) + \sigma_v$$

Limit pressure determined from presuremeter tests

$$p_1 = - \text{ MPa}$$

C10.8.3.5-7

At rest total horizontal stress measured at ther base elevation

$$p_0 = - \text{ MPa}$$

Coefficient that depen on diameter socket

$$K_b = 5.30$$

Table C10.8.3.5-1

Total vertical stress at the base elevation

$$\sigma_v = - \text{ MPa}$$

$$q_p = - \text{ MPa}$$

$$\phi = 0.50$$

Table 10.5.5-3

$$Q_{pR} = A_p \cdot q_p$$

$$Q_{pR} = - \text{ kN}$$

$$Q_R = \phi \cdot Q_{pR}$$

$$Q_R = - \text{ kN}$$

ESTIMATED PILE CAPACITY:

Pile Structural Capacity	$Q_T = 0.75 \cdot 0.85 \cdot 0.85 \cdot f_c \cdot A_g$	12768 kN	1301 T
Pile resistance	Q_R	6119 kN	624 T
Deducting pile weight		-221 kN	-22 T
Estimated Pile Capacity		5898 kN	601 T
Maximum Reaction - ULS	Ok	3963 kN	404 T