



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 4.2.3

6. ORB23
7. LRB12a
8. FO09
9. ORB25a
10. CB25

July 25, 2013

The Joint Venture of



NIPPON KOEI CO.,LTD.



NIPPON ENGINEERING CONSULTANTS CO.,LTD.



CHODAI CO.,LTD.



THAI ENGINEERING CONSULTANTS CO., LTD.

IDA Credit No. : 4779-VN

(IDA tín dụng số : 4779-VN)

Project ID No. : P106235

(Mã dự án : P106235)

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)




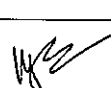

Volume 4: Structural Calculation Report (PKG A2)

(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 4.2.3

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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

Table of content - ORB23 Bridge

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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

ORB 23

CALCULATION SHEETS

SUBSTRUCTURE

Da Nang Quang Ngai Expressway project

BRIDGE
ORB23

CALCULATION SHEETS
ABUTMENT A2

Table of content

1. Structure dimensions and Loads
2. Foundation analysis
3. Elements checks
4. Bored pile check

	Da Nang Quang Ngai Expressway project ORB23 BRIDGE DETAIL DESIGN ABUTMENT A2	Item.	Eng.	Date.	Sign.
		Design			
		Check			
		Revise			

LOAD COMPONENTS

Assumptions :

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

Input :

Level Table(at center of abutment)

Level of top of headwall	HTwL	8.379	m
Level of top of bearing	BTL	6.338	m
Level of top of stem abutment	HTL	6.104	m
Level of top of footing	FTL	1.500	m
Level of bottom of footing	FBL	-0.500	m
Ground level	GL	2.050	m
Lowest water level	HWL	0.000	m
Skew angle	α	0.00	deg

Material Unit Weights

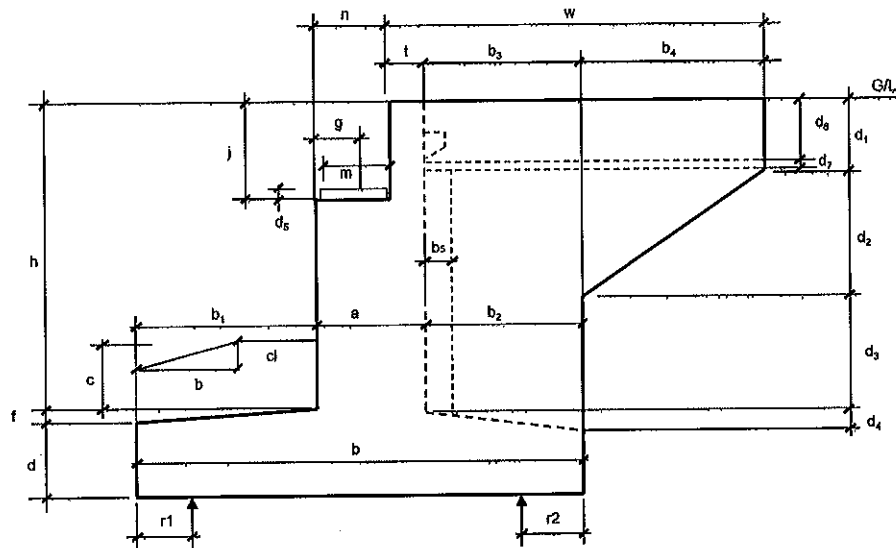
• Unit Weight of Reinf. concrete	γ_c	=	24.5 kN/m ³
• Unit Weight of Soil	γ_s	=	18.0 kN/m ³
• Unit Bouyancy Weight of Soil	γ_{sbo}	=	8.2 kN/m ³
• Unit weight of asphalt concrete	γ_a	=	22.1 kN/m ³

I. Loads from substructure

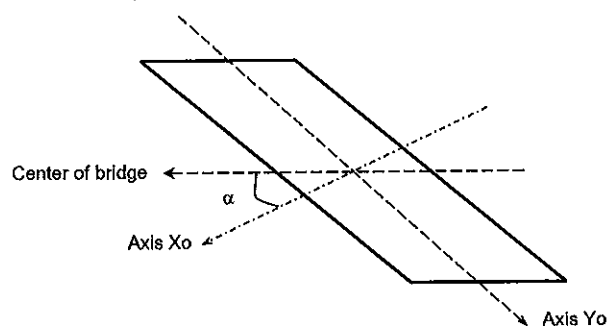
Abutment dimensions

VERTICAL VIEW

Bearing Type: **MOVE**

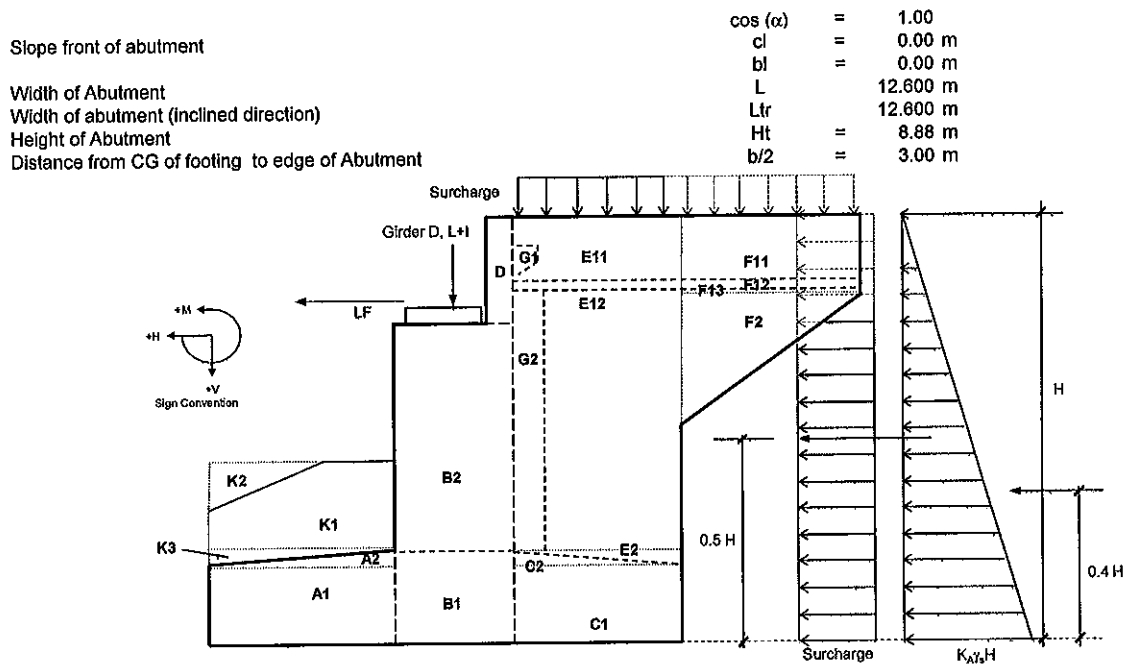


PLAN VIEW



	Da Nang Quang Ngai Expressway project ORB23 BRIDGE DETAIL DESIGN ABUTMENT A2	Item.	Eng.	Date.	Sign.
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		Revise			

ABUTMENT DIMENSIONS (IN METRES)					
Item	Symbol	Value	Item	Symbol	Value
Height of stem	h	6.879	Horizontal Dimension	b ₄	1.900
Footing Width	b	6.000	Horizontal Dimension	b ₅	0.000
Stem Width	a	1.500	Vertical Dimension	d ₁	0.930
Footing Depth	d	2.000	Vertical Dimension	d ₂	1.900
Footing Slope	f	0.000	Vertical Dimension	d ₃	4.049
Width of stem at bearing	n	1.000	Vertical Dimension	d ₄	
Ballast Wall Height	j	2.275	Vertical Dimension	d ₅	0.234
Ballast Wall Thickness	t	0.500	Vertical Dimension	d ₆	1.070
Wingwall Length	w	4.900	Vertical Dimension	d ₇	
Soil Cover at Toe	c	0.550	With of bearing pad	m	0.800
Girder Reaction	g	0.550	Wingwall Thickness	u ₁	0.500
Horizontal Dimension	b ₁	2.000	Wingwall Thickness	u ₂	0.500
Horizontal Dimension	b ₂	2.500	Distance to cl of pile	r ₁	1.000
Horizontal Dimension	b ₃	2.500	Distance to cl of pile	r ₂	1.000



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	6.906	12.600	2132	2.750	0.250	533
Section C1	5.000	12.600	1544	4.750	-1.750	-2701
Section C2	-	12.600	-	4.333	-1.333	-
Section D	1.138	12.600	351	3.250	-0.250	-88
Section E11	2.325	0.500	28	4.750	-1.750	-50
Section E12	14.873	0.500	182	4.750	-1.750	-319
Part extra stem	-	-	-	5.417	-2.417	-
Section F11	2.033	0.500	25	6.950	-3.950	-98
Section F12	-	0.500	-	5.700	-2.700	-
Section F13	-0.266	0.500	-3	6.950	-3.950	13
Section F2	1.805	0.500	22	6.633	-3.633	-80
Section G1	0.135	11.600	38	3.650	-0.650	-25
Section G2	-	6.879	-	3.500	-0.500	-
Bearing seats (w1seat= 0.65m)	0.187	3.250	15	2.550	0.450	7
Curbs +Handrail on Abutment	0.50	4.900	60	5.450	-2.450	-147
Total SW of Abutment (DC)			6555			-255
Transverser moment			44		6.175	270

Notes:

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

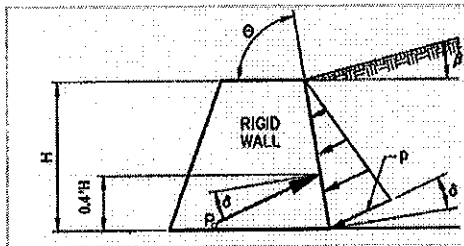
	Da Nang Quang Ngai Expressway project ORB23 BRIDGE DETAIL DESIGN ABUTMENT A2	Item.	Eng.	Date.	Sign.
		Design			
		Check			
		Revise			

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	17.20	12.100	3746	4.750	-1.750	-6555
Section E2	-	11.600	-	5.167	-2.167	-
Section E3	-	1.000	-	6.000	-3.000	-
Section K1	1.100	12.600	249	1.000	2.000	-
Section K2	-	12.600	-	-	3.000	-
Section K3	-	12.600	-	0.667	2.333	-
Total Earth on Footing			3995			-6555

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.



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

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

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

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

H	=	8.88 m
W	=	12.6 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	=	0.053 Mpa

Horizontal earth pressure:

- $E_a = 0.5 \cdot p \cdot B \cdot 10^3$ (kN)
- $M = E_a \cdot 0.4H$

E_a	=	2980 kN
M	=	10584 kNm

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

Equivalent height of soil for highway loading taken from Table 3.11.6.2.1

H=	1.50m heq=	1.7 m
H=	3.00m heq=	1.2 m
H=	6.00m heq=	0.76 m
H=	9.00m heq=	0.61 m
H=	8.88m heq=	0.62 m

(Linear interpolation)

- Vertical force

ESv	=	349 kN
ev	=	-1.75 m
M	=	-611 kNm

- Horizontal force

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

ESh	=	414 kN
eh	=	4.44 m
M	=	1836 kNm

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	ORB23 BRIDGE	Design			
	DETAIL DESIGN	Check			
	ABUTMENT A2	Revise			

5. Earthquake effects

Bridge is located at:

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

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

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 $\theta = \arctan(k_v / (1 - k_v))$ $\theta = 5.2 \text{ deg}$

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

• 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 $E_{AE} = 3363 \text{ kN}$

$M_{AE} = E_{AS} \cdot 0.3H + (E_{AE} - E_{AS}) \cdot 0.6H$ $M_{AE} = 9981 \text{ KNm}$

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

<A.11.1.1.1>

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 $2.5A = 0.145$
 $C_{sm} = 1.2 \cdot A \cdot S / T_m^{2/3} \leq 2.5 \cdot A$ $C_{sm} = 0.082$
 • Period of vibration of the fundamental mode $T_m = 0.784 \text{ s}$
 $T_m = 2 \cdot \pi \cdot \sqrt{m/k}$

Description	Area (m ²)	Length (m)	Force (kN)	X ⁽¹⁾ (m)	Arm ⁽²⁾ (m)	Moment (kN·m)
Section A1	4.000	12.600	101	-	1.000	101
Section A2	-	12.600	-	-	2.000	-
Section B1	3.000	12.600	76	-	1.000	76
Section B2	6.906	12.600	174	-	4.302	760
Section C1	5.000	12.600	126	-	1.000	126
Section C2	-	12.600	-	-	2.000	-
Section D	1.138	12.600	29	-	7.742	222
Section E11	2.325	0.500	2	-	6.344	15
Section E12	14.873	0.500	15	-	2.905	-
Section E2	-	-	-	-	2.000	-
Section F11	2.033	0.500	2	-	6.344	13
Section F12	-	0.500	-	-	5.809	-
Section F13	-0.266	0.500	-0	-	6.949	-
Section F2	1.805	0.500	2	-	7.316	13
Section G1	0.135	11.600	3	-	6.166	19
Section G2	-	6.879	-	-	2.905	-
Total EQ of Abutment Selfweight			530			1336

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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

$$BR = 25\% \cdot n \cdot m \cdot (2 \cdot 145 + 35)$$

- Acting at 1.8m higher of road face

n	=	3 lanes	
m	=	0.85	
BR	=	104 kN	Long. Axis
e	=	10.7 m	
Mlong	=	1106 KNm	Long. Axis

7. Centrifugal Force, CE (3.6.3)

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

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

Acting at 1.8m higher of road face

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

V	=	120 km/h	
V	=	33.3 m/s	
R	=	- m	
C	=	-	
CE	=	0.00 KN	
e	=	8.68 m	
Mtrans	=	0.00 KNm	Trans. Axis

8. Water Load (WA)

:NA

8.1. Buoyancy of Abutment

- Highest water Level +0.00

Description	Area (m ²)	Length (m)	Force (kN)	X ⁽¹⁾ (m)	Arm ⁽²⁾ (m)	Moment (kN·m)
Bouyancy on abutment						
Section A1	1.00	12.600	-124	1.000	2.000	-247
Section A2	-	12.600	-	-	3.000	-
Section B(B1,B2)	0.75	12.600	-93	2.750	0.250	-23
Section C1	1.25	12.600	-155	4.750	-1.750	270
Section C2	-	12.600	-	-	3.000	-
Section E2	-	1.000	-	-	3.000	-
Section E1	-	1.000	-	4.750	-1.750	-
Section F2	-	1.000	-	4.150	-1.150	-
Total Bouyancy			-371			0

8.2 Buoyancy of Earth on Abutment

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

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SUPERSTRUCTURE LOADS

II. Loads from superstructure

Item	Sign	Value	Unit
Span length	Lst	33.00	m
Span between bearings	Ls	32.20	m
Bridge Width	W	12.60	m
Number of girders	n_g	5.00	Girders
Girder height	Hg	1.65	m
Deck slab depth	Hd	0.247	m
Asphalt depth	H _{as}	0.084	m

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

Item	Sign	Value	Unit
1.1. Girders			
Sum of girders weight	DC	3347.93	kN
Precast Planks	DC	473.46	kN
Diaphragm	DC	380.73	kN
Total	DC	4202.11	kN
1.2. Deck slab			
Deck slab	DC	2464.29	kN
1.3. Pavement			
Asphalt concrete	DW	649.37	kN
1.4. Parapet			
Parapet + median	DC	889.35	kN

2. Live load (LL):

2.1. Live load

Truck	
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

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

Truck	P1(kN)	P2(kN)	P3(kN)	Sum(kN)	Total(kN)
Axle load	145	145	35		
Influence value	1.000	0.866	0.733		
Reaction	145.0	125.6	25.7	296.3	944.4
Tandem	P1(kN)	P2(kN)		Sum(kN)	Total(kN)
Axle load	110	110			
Influence value	1.000	0.963			
Reaction	110	105.9		215.9	688.2
Lane load	WL(kN/m)				Total(kN)
Value	9.3				
Influence value	16.1				
Reaction	149.7				381.8
Pedestrian	Wdb(kN)				Total(kN)
Reaction	0.0				0.0

2.1. Braking force

$$\text{BR} = 207 \text{ kN}$$

3. Earthquake effects on superstructure (EQ)

Force from superstructure due to EQ

$$\text{EQ} = 336 \text{ kN}$$

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

Bearing displacement due to uniform temperature and shrinkage creep

Shear modulus G

Bearing area

Height of elastomeric layers

Number of bearing

Horizontal force due to TU+SH&CR

Acting at top of bearing $H = G \cdot A \cdot \Delta u / h_t$

$$\begin{aligned} \Delta u &= 0.026 \text{ m} \\ G &= 1 \text{ MPa} \\ A &= 0.175 \text{ m}^2 \\ h_t &= 0.084 \text{ m} \\ nb &= 5 \text{ bears} \\ H_x &= 271 \text{ kN} \end{aligned}$$

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

Transverse wind load

$$P_D = \max(0.0006V^2, C_d A_t, 1.8A_t) =$$

Longitudinal wind load

$$F_{WSL} = 0.25P_D =$$

$$\begin{aligned} \text{Zone} &= \text{II} \\ V_b &= 45.00 \text{ m/s} \\ S &= 1.09 \\ V &= 49.05 \text{ m/s} \\ C_d &= 1.10 \\ b &= 12.60 \text{ m} \\ d &= 2.97 \text{ m} \\ b/d &= 4.25 \\ A_t &= 97.91 \text{ m}^2 \\ H_y &= 176.2 \text{ kN} \\ H_x &= 44.1 \text{ kN} \end{aligned}$$

5.2. Wind load on vehicles (WL)

Transverse wind load on vehicle

Longitudinal wind load on vehicles

(At 1.8m from surface)

$$\begin{aligned} H_y &= 24.75 \text{ kN} \\ H_x &= 24.75 \text{ kN} \end{aligned}$$

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+Deck+Parapet	DC	3778	0.20			756			
Pavement	DW	325	0.20			65			
Live Load	LL	1326	0.20			265		0.48	630
Pedestrian	PD								
Trans. wind on Struc.	WS			22	4.84		88	4.84	426
Trans. wind on vehi.	WL			12	10.95		25	10.95	271
Earth quake	EQ			168	4.84		101	4.84	487
TU+SH&CR	TU+SH&CR			135	4.84	655			

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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+Deck+Parapet	DC	3778	0.45			1700			
Pavement	DW	325	0.45			146			
LiveLoad	LL	1326	0.45			597		0.48	630
Pedestrian	PD								
Trans. wind on Struc.	WS			22	6.84	151	88	6.84	603
Trans. wind on vehi.	WL			12	12.95	160	25	12.95	321
Eearth quake	EQ			168	6.84	1148	101	6.84	689
TU+SH&CR	TU+SH&CR			135	6.84	926			

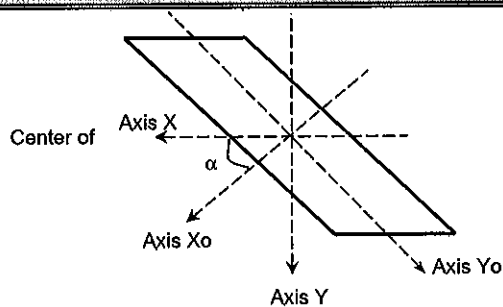
Loads	Sign	Load combinations						
		Str-IA	Str-IB	Str-IIIA	Str-IIIB	Ser-I	Ext-IA	Ext-IB
Girder+Deck+Parapet	DC	1.25	0.90	1.25	0.90	1.00	1.25	0.90
Pavement	DW	1.50	0.65	1.50	0.65	1.00	1.50	0.65
LiveLoad	LL	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Pedestrian	PD	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Trans. wind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

Load combinations at bottom of stem					
Load combinations	N (kN)	Longitudinal		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	7530	68	1834	0	1102
Strength Str-IB	5932	68	1514	0	1102
Strength Str-IIIA	7000	89	1728	60	1292
Strength Str-IIIB	5402	89	1408	60	1292
Service Ser-I	5429	154	1741	51	1029
Extreme Ext-IA	5872	168	1174	101	802
Extreme Ext-IB	4274	168	855	101	802

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	7530	68	3852	0	1102
Strength Str-IB	5932	68	3132	0	1102
Strength Str-IIIA	7000	89	3833	60	1412
Strength Str-IIIB	5402	89	3114	60	1412
Service Ser-I	5429	154	3574	51	1131
Extreme Ext-IA	5872	168	3790	101	1004
Extreme Ext-IB	4274	168	3071	101	1004

	Da Nang Quang Ngai Expressway project ORB23 BRIDGE DETAIL DESIGN ABUTMENT A2	Item.	Eng.	Date.	Sign.
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		Revise			

LOAD COMBINATIONS



III. Load Combinations

1. Loads from substructure

Loads at bottom of pilecap	Sign	Vertical	Longitudinal		Transversal	
		N (kN)	Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Self weight of Abutment	DC	6555		-255		270.1995
Soils on pilecap	EV	3995		-6555		
Horizontal Earth Pressure	EH		2980	10584		
Vertical Surcharge	LSv	349		-611		
Horizontal Surcharge	LSH		414	1836		
Braking Force	BR		104	1106		
Centrifugal Force	CE		-	-	-	-
Buoyancy of Abutment	WA	-371		0		
Buoyancy of Earth on Abutment	WA	-		-		
Earthquake effects to Abutment	EQ		530	1336	159	401
Earthquake effects to soil	E _{AE}		3363	9981		

Loads	Sign	Load factors						
		Str-IA	Str-IB	Str-IIIA	Str-IIIB	Ser-I	Ext-IA	Ext-IB
Self weight of Abutment	DC	1.25	0.90	1.25	0.90	1.00	1.25	0.90
Soils on pilecap	EV	1.35	0.90	1.35	0.90	1.00	1.35	0.90
Horizontal Earth Pressure	EH	1.50	0.90	1.50	0.90	1.00	0.00	0.00
Vertical Surcharge	LSv	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Horizontal Surcharge	LSH	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Braking Force	BR	1.75	1.75	1.35	1.35	1.00	0.50	0.50
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	13828	5375	10787	0	338
Strength Str-IB	9736	3587	7476	0	243
Strength Str-IIIA	13688	5168	9855	0	338
Strength Str-IIIB	9596	3380	6544	0	243
Service Ser-I	10529	3497	6105	0	270
Extreme Ext-IA	13391	4152	3315	159	739
Extreme Ext-IB	9299	4152	6354	159	644

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		Design			
		Check			
		Revise			

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	7530	68	3852	0	1102
Strength Str-IB	5932	68	3132	0	1102
Strength Str-IIIA	7000	89	3833	60	1412
Strength Str-IIIB	5402	89	3114	60	1412
Service Ser-I	5429	154	3574	51	1131
Extreme Ext-IA	5872	168	3790	101	1004
Extreme Ext-IB	4274	168	3071	101	1004

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	21358	5443	14839	0	1440
Strength Str-IB	15868	3655	10608	0	1346
Strength Str-IIIA	20688	5257	13689	60	1750
Strength Str-IIIB	14998	3469	9658	60	1655
Service Ser-I	15958	3652	9680	51	1402
Extreme Ext-IA	19264	4320	7106	260	1742
Extreme Ext-IB	13573	4320	9425	260	1648

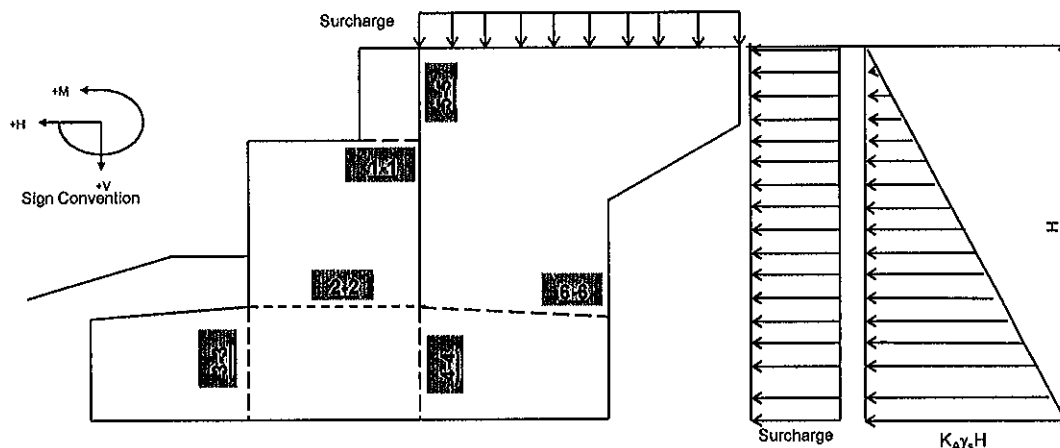
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		Check			
		Revise			

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 Selsmic 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	390		-15		
Horizontal Earth Pressure		196	178		
Surcharge (horizontal)		248	282		
Horizontal Selsmic Earth Pressure		221	168		
Abutment earthquake force		32	36	10	11

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	390	444	445	0	0
Strength Str-IA	487	727	741	0	0
Strength Str-IB	351	610	640	0	0
Extreme Ext-I	487	487	410	10	11

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
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 Selsmic Earth Pressure	E _{AE}				1.50
Abutment earthquake force	EQ				1.00

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		Revise			

Unfactored Loads	N (kN)	Longitudinal		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Selfweight	2521		-179		
Superstructure Dead Load	3778		756		
Pavement	325		65		
Live Load	1326		265		630
Horizontal Earth Pressure		1789	4922		
Surcharge (Horizontal)		372	1281		
TU+SH&CR		135	655		
Horizontal Seismic Earth Pressure		2019	4641		
Abutment earthquake force		206	586	163	663

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	7950	2297	7764	0	630
Strength Str-IA	10682	3402	11234	0	1102
Strength Str-IB	8201	2329	8024	0	1102
Extreme Ext-I	9024	3421	9138	163	978

1.3 Section 3-3

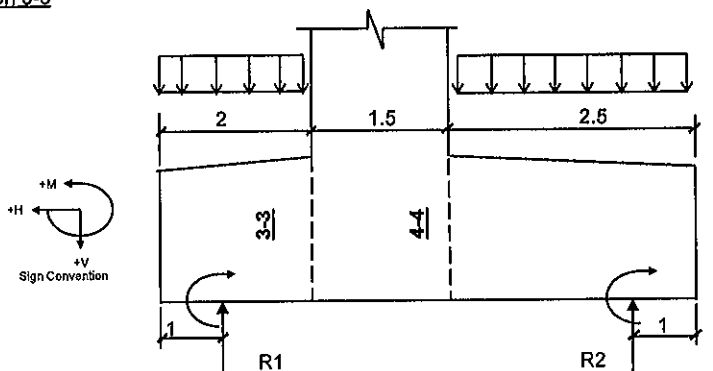


Table of Load Factors

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	Shear (kN)	Longitudinal			
		Hx (kN)	My (kN.m)		
Selfweight at front side	-1235		-1235		
Vertical soil on foot at front side	-249		-249		
Reaction of piles					
Ser-I	11810		16059		
Str-IA	16462		22753		
Str-IB	11874		16079		
Ext-I	13204		18473		

Load Combination at section 3-3					
Load combinations	N (kN)	Longitudinal			
		Hx (kN)	My (kN.m)		
Service Ser-I	10326		14574		
Strength Str-IA	14582		20873		
Strength Str-IB	10538		14744		
Extreme Ext-I	11323		16593		

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		Design			
		Check			
		Revise			

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		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Selfweight of behind side	-1798		-2337		
Vertical soil on foot at behind side	-3746		-4682		
Surcharge(Vertical)	-349		-437		
Reaction of piles					
Ser-I	3552		2142		
Str-IA	4101		1434		
Str-IB	3208		1659		
Ext-I	5328		4040		

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	-2341		-5313		
Strength Str-IA	-3814		-8572		
Strength Str-IB	-2392		-5422		
Extreme Ext-I	-2150		-5420		

1.4 Section 5-5 & 6-6

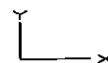
Slope of triang pressure
Uniform horizontal pressure

$$\begin{aligned} \tan \beta &= 6.00 \\ U.p &= 3.70 \text{ kN/m}^2 \end{aligned}$$

SERVICE – Element Moment X:

Combination X-Bending Moment (per unit width)
service IA

70.1164.8754.644.234.626.219.213.4.8.5.102.921.20.10.30.2
 77.4164.1553.242.933.526.418.613.118.65.373.31.70.70.20.0
 60.2164.8562.441.732.424.417.812.6.8.68.5463.62.21.20.70.4
 81.1565.351.940.731.123.216.912.0.846.674.22.92.21.60.80
 81.465.351.339.529.821.916.811.48.46.620.50.423.42.38
 81.464.750.338.228.320.414.610.784.717.16.71.584.44
 81.163.849.036.626.518.813.310.08.58.920.907.31
 80.462.547.234.624.616.911.79.06.911.1311.16
 79.4560.945.332.622.615.110.17.619.117.02
 77.9568.843.130.420.513.18.26.5755.4.84
 75.7756.240.327.818.110.9627.34110.712
 72.5852.937.224.915.58.79.413.1410.011
 68.5449.033.721.913.06.792.580.110.60.27
 63.344.429.818.810.74.931.260.810.90.25
 67.039.225.715.78.543.500.411.111.00.23
 49.633.421.412.86.752.630.161.150.90.23
 41.227.217.110.15.362.320.520.650.60.18
 32.020.813.07.884.522.611.620.820.20.2
 22.414.69.346.174.423.833.623.061.019
 13.49.12.649.510.501.686.673.651.310.0
 5.94.94.52.528.659.85510.912.170.58.9
 1.62.54.19.653.940.12.616.019.519.13.12

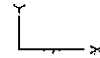


SERVICE – Element Moment Y:

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		Design			
		Check			
		Revise			

Combination Y-Bending Moment(per unit width)
service 1A

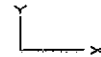
1	11.06	4.76	2.88	1.72	-1.16	-0.62	-0.22	0.14	0.30	0.60	0.72	0.72	-0.08	-0.08	-0.08
2	17.9	11.8	7.42	4.89	3.05	1.94	0.68	0.41	1.11	4.17	7.02	0.22	-2.2	-2.2	-2.12
3	19.7	14.7	10.3	6.97	4.24	2.24	0.68	0.62	0.22	0.62	0.63	3	-3.62	-3.42	-3.1
4	20.2	16.6	11.6	7.97	4.81	2.32	0.19	1.7	0.3	1.3	54.3	4.8	4.8	4.8	3.6
5	20.3	16.0	11.9	8.25	4.88	1.99	0.7	82.8	4.4	55.35	7.06	21	6.22	5.11	4.68
6	20.3	16.9	11.7	7.82	4.23	1.04	1.7	8.4	1.7	6.06	57	3157.7	36.7	36.414	
7	20.2	16.6	11.1	7.06	3.26	0.2	8.3	1.3	5.7	0.7	78.79	3.88	4	8.21	
8	20.0	16.0	10.4	6.06	2.06	1.6	9.4	8.7	7.5	10.1	11.0	3	0.816		
9	19.7	14.6	9.63	4.96	0.76	3.1	1.6	5	1.9	6	1	12	1	16.612	
10	19.3	13.8	8.61	3.84	0.6	5.4	1.8	2.6	1.1	5	15	18.21			
11	18.7	13.0	7.63	2.68	0.1	1.6	1	9.9	7.7	12.7	14	15.42			
12	17.9	12.0	6.52	1.39	3.3	5.7	3	9.10	7.1	13.2	14	16.3			
13	16.8	10.9	5.37	0.29	4.3	0.8	1	9.11	3.1	13.6	15	16.8			
14	15.6	9.74	4.28	0.7	8.6	0.7	8.5	11.4	13.5	15	15.6				
15	13.9	8.42	3.37	1.3	7.5	1	4.8	3.0	10.7	12.5	13	14.4			
16	12.1	7.04	2.54	1.4	8.4	6	5.7	0.3	9.9	10.2	11	11.63			
17	10.1	5.82	2.21	0.7	1.2	8.6	4.4	2.5	4.2	6.0	5.6	36.5	8		
18	7.90	4.78	2.50	1.18	0.42	0.21	0.42	0.92	1	1	1.62				
19	6.74	4.16	3.74	4.37	5.62	7.33	9.32	11	3	13	14.0				
20	3.84	4.22	6.19	9.30	13.3	17.7	22.1	26	2	29	32	10			
21	2.4	5.1	9.92	16.2	23.7	31.8	39.9	47	3	54	57	69			
22	1.7	6.5	14.8	25.3	37.0	49.5	62.8	76	7	89	94	42			



SERVICE – Element Shear FX:

Combination SHEAR FORCE X (per unit width)
service 1A

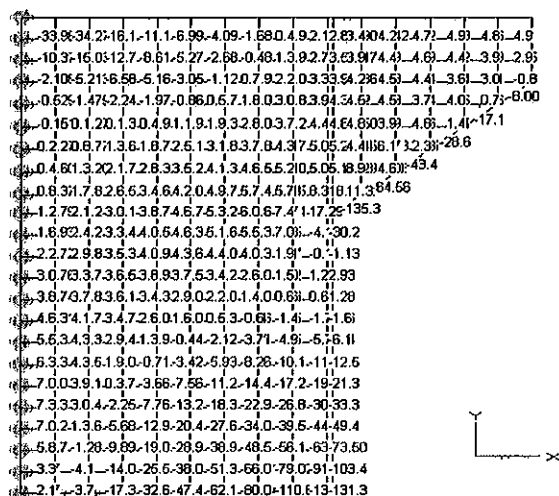
1	5.2	8.29	4.73	0.31	3.27	9.23	9.19	8.16	9.12	10.8	9.0	8.42	4.2	-2.3	-0.6	-1.8
2	56.0	33.0	32.2	30.1	26.7	22.8	18.8	14.9	11.9	8.17	5.9	-3.9	-2.2	-0.9	-0.5	
3	54.0	41.1	34.8	30.6	26.5	22.2	17.9	14.0	10.8	7.36	5.3	-3.4	-2.4	-1.1	-0.61	
4	52.4	44.7	37.4	31.7	26.6	21.7	17.0	12.8	9.7	6.2	4.3	-3.4	-1.6	-0.9	9.730	
5	52.6	46.2	39.3	32.9	26.9	21.3	16.0	11.2	7.46	4.60	3.9	-1.6	-0.8	19.01		
6	54.2	47.7	40.6	33.7	27.1	20.8	14.8	9.17	4.8	1.3	4.5	0.7	3.0	4.8	28.31	
7	56.6	49.2	41.7	34.3	27.2	20.3	13.6	6.40	0.6	5.0	7.80	7	38.41			
8	59.3	50.8	42.6	34.6	27.0	19.7	12.4	4.1	5.8	8.2	11.4	0	49.916			
9	61.7	52.2	43.1	34.7	26.7	19.2	11.6	2.7	1	15.5	17.22					
10	63.9	53.2	43.4	34.5	26.3	18.6	11.3	3.8	0.0	11.2						
11	65.7	53.8	43.3	33.9	25.6	18.0	11.3	6.1	-7	18.3						
12	66.7	53.7	42.6	32.9	24.5	17.2	11.0	6.0	-1	17.43						
13	66.7	52.8	41.1	31.3	23.0	16.0	10.0	4.6	-1	14.00						
14	65.3	50.7	38.7	29.0	21.0	14.3	8.8	3	2	7.7	16					
15	62.3	47.3	35.4	25.9	18.4	12.2	7.0	1	2	11.3	36.94					
16	57.3	42.5	31.0	22.1	15.2	9.8	5.58	1	6	52.96	36					
17	50.0	36.1	25.7	17.8	11.6	7.12	4.02	1	5	1.43	7.8					
18	40.3	28.4	19.5	12.6	7.52	4.11	2.42	2	12	1.0	9.52					
19	28.2	19.6	12.8	7.27	3.02	0.63	0.42	3	23	6	10.2					
20	14.7	10.8	6.09	1.70	1.5	3.3	3.7	2.6	1	3	62	15	6.310			
21	2.7	2.9	0.8	2.3	6.25	6.7	0.8	0.9	0.3	31	30.09					
22	2.3	3.5	6.7	8.8	3.39	3.0	10.15	11.4	10.0	15	83.72					



SERVICE – Element Shear FY:

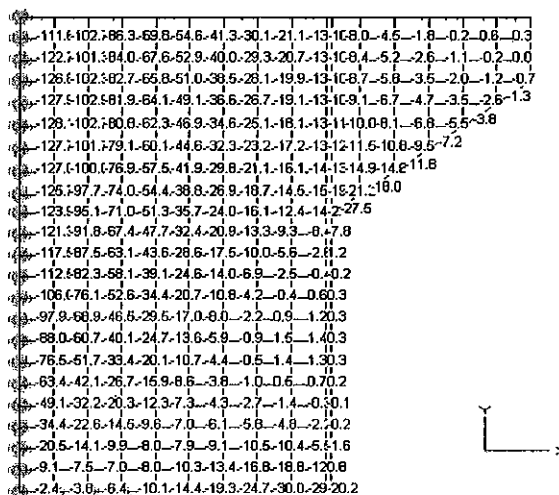
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Combination SHEAR FORCE Y (per unit width)
service 1A



STRENGTH – Element Moment X:

Combination X-Bending Moment
strong 1A



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		Design			
		Check			
		Revise			

STRENGTH – Element Moment Y:

Combination Y-Bending Moment
streng IA

1	18.6	7.9	7.4	43.2	78.1	76.0	93.0	31.0	2.1	0.0	60.9	1.15	1.21	1.22	1.2
2	28.4	16.7	11.7	7.56	4.79	2.50	0.83	0.67	1.82	2.71	3.2	3.5	3.6	3.4	3.2
3	31.2	23.3	16.3	10.8	6.70	3.45	0.71	1.5	9.3	24.14	6	5.3	5.7	5.5	5.0
4	31.8	24.9	16.3	12.5	7.71	3.66	0.27	2.7	1.5	0.6	18.8	7.5	7.7	7.3	5.6
5	32.0	25.2	16.8	12.9	7.69	3.06	1.0	1.4	3.1	7.08	49.0	9.71	9.8	8.1	7.41
6	31.9	24.9	18.4	12.3	6.76	1.73	2.6	2.8	3.2	9.31	10.11	4.12	11.10	5	10.0
7	31.6	24.4	17.5	11.1	5.29	0.84	7.1	1.8	7.1	1.11	13.14	8	13.0	12.8	
8	31.2	23.6	16.3	9.54	3.38	2.3	9.7	2.7	11.4	15.18	16.0	16.6			
9	30.8	22.7	15.0	7.93	1.47	4.5	0.9	6.1	1.4	6.1	19.2	26.6			
10	30.0	21.6	13.6	6.17	0.7	4.6	8.9	12.3	17.5	23.27	8				
11	29.0	20.3	11.9	4.26	2.8	6.9	1.5	14.6	19.2	22.23	2				
12	27.7	18.7	10.2	2.4	14.7	6.10	9.1	16.0	20.0	22.23	1				
13	26.1	17.0	8.58	0.64	6.3	7.12	23.16	9	20.5	22.23	7				
14	24.0	15.1	6.76	0.8	5.7	3	0.12	9	17.1	20.3	22.23	5			
15	21.5	13.0	5.21	1.7	7.7	6.6	12.3	16.0	18.8	20.21	6				
16	18.7	11.0	4.14	1.8	6.7	10.4	13.2	15.1	16	17.2					
17	15.6	9.01	3.57	0.8	94.0	6	3	7.8	8.6	9	19.4				
18	12.1	7.42	4.03	1.93	0.9	0.8	1.2	2.0	2	3.2					
19	8.74	6.46	5.95	6.9	8.9	11.7	14.8	17.8	20	22.1					
20	5.85	5.51	9.6	14.5	20.7	27.6	34.4	40.8	46	50.0					
21	3.7	7.9	15.3	25.1	36.6	49.1	61.6	73.1	83	89.1					
22	2.6	9.9	22.7	38.8	66.8	76.1	96.7	118.3	13	145.7					

STRENGTH – Element Shear Fx:

Combination SHEAR FORCE X (per unit width)
streng IA

1	5.7	8.47	5.52	7.49	7.44	2.37	8.31	2.25	0.19	15.14	0.10	2.6	3.5	0.9	3.0
2	89.6	52.9	51.1	47.6	42.2	36.0	29.6	23.4	18.15	12.7	9.2	6.1	3.5	1.5	0.9
3	85.7	65.4	55.2	48.4	41.0	35.0	28.2	21.9	16.15	11.6	8.3	5.5	3.8	1.8	0.9
4	82.5	70.6	58.0	50.0	41.9	34.1	26.7	20.0	14.1	11.9	7	6.9	5.4	2.5	1.4
5	82.5	72.7	61.8	51.6	42.2	33.4	25.0	17.6	11.9	7	10.6	2	2.5	1.6	30.3
6	84.6	74.6	63.6	52.8	42.4	32.6	23.1	14.2	7.2	5.35	1.0	10.35	44.9		
7	88.2	76.8	66.0	53.5	42.4	31.7	21.0	9.81	1.8	4.46	10.70	60.71			
8	92.1	79.0	66.2	53.9	42.1	30.8	19.2	6.25	14.33	21.57	6.41				
9	95.8	80.9	66.9	53.9	41.6	29.9	16.1	4.28	24.82	105.3					
10	98.7	82.2	67.2	53.5	40.8	28.0	17.6	6.15	0	18.4					
11	101.8	83.0	66.8	52.5	39.7	26.1	17.7	9.92	11	30.2					
12	102.8	82.8	65.6	50.8	38.0	26.9	17.3	9.81	2	11.0					
13	102.8	81.1	63.3	48.3	35.6	24.9	16.8	7.65	1.45	4.7					
14	100.7	77.8	63.6	44.7	32.4	22.3	13.6	5.31	3.7	10.5					
15	95.3	72.5	64.3	39.9	28.4	19.0	11.1	3.71	4.6	10.1					
16	87.5	64.9	47.5	34.0	23.5	15.3	8.71	2.7	4	19.22					
17	76.2	55.1	39.2	27.1	17.8	11.1	6.4	2.7	1	185.3					
18	61.2	43.2	29.7	19.3	11.5	6.3	3.9	3.5	2.7	1.02					
19	42.7	29.8	19.4	11.0	4.7	0.97	0.88	5	10	16.2					
20	22.1	16.2	9.0	2.65	2.37	5.1	5.4	0.4	5.78	23	10.0				
21	3.9	3.9	1.2	3.5	6.18	7.6	10.71	0.8	1.3	1.48	123.8				
22	3.5	6.0	10.3	12.8	14.3	15.71	17.6	15.6	5.24	129.2					

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STRENGTH – Element Shear F_y :

Combination SHEAR FORCE Y (per unit width)
streng IA

1	53.8	54.4	25.5	17.5	10.8	6.18	2.32	0.8	0.3	54.55	6.06	9.27	6.9	7.95	7.73	7.9
2	16.07	23.65	20.1	13.6	8.22	4.10	0.76	2.1	4.4	35.63	1.77	0.2	7.3	7.05	6.35	4.69
3	3.07	6.14	5.10	2.8	0.4	4.88	1.70	1.0	2.3	3.95	26.8	6.67	0.3	6.95	5.73	4.73
4	0.65	2.16	3.40	3.00	1.39	0.7	1.2	7.4	6.05	96.86	9.7	0.9	5.71	6.2	1.05	13.0
5	0.1	5.0	3.70	2.80	6.6	1.6	2.2	9.4	3.05	6.68	77.4	406.0	7.55	2.25	27.8	
6	0.5	6.1	4.62	1.62	9.83	7.4	4.7	6.5	6.16	5.27	78.06	39.4	13.65	46.4		
7	0.9	22.1	83.3	0.4	3.55	3.46	2.7	8.9	0.7	81.7	67.7	13.69	70.1			
8	1.4	12.6	0.4	1.25	3.96	4.17	4.88	6.2	8.65	12.28	19.1	103.6				
9	2.0	63.3	9.4	7.95	9.97	0.68	0.99	1.1	1.1	11.26	4.2	216.2				
10	2.7	73.9	2.5	1.28	2.67	0.87	8.08	3.6	10.6	6.5	47.4					
11	3.7	1.4	6.05	6.8	2.56	6.7	6.7	1.6	0.7	2.69	0.5	2.02				
12	4.7	2.6	2.85	7.65	9.85	7.05	1.93	9.32	0.9	1.74	40					
13	6.02	6.9	1.57	3.5	2.34	4.83	3.9	1.9	8.0	63.0	81.77					
14	7.30	8.4	8.5	4.6	4.0	7.2	4.0	6.3	1.06	2.3	2.26					
15	8.6	3.6	6.5	4.5	3.2	0.9	0.74	3.30	5.71	7.7	8.59					
16	9.8	5.6	7.9	3.0	5.1	1.0	5.25	9.13	12.6	15.5	17.19					
17	10.8	6.1	1.0	5.2	5.61	11.8	17.2	22.2	26.5	30.32	8					
18	11.2	4.6	2.3	3.7	11.8	20.2	26.1	35.2	41.2	46.51	2					
19	10.9	2.0	8.8	6.4	19.7	31.2	42.3	52.2	60.6	68.76	9					
20	8.6	1.1	8.2	14.9	29.0	44.1	59.6	74.3	86.1	98.113	0					
21	5.1	6.2	21.2	38.8	57.9	76.6	101.1	121.1	14.158	9						
22	3.3	5.3	25.9	49.3	72.0	94.7	122.4	170.621	202.2							

Load Combination at section 5-5					
Load combinations	N (kN)	Vertical		Horizontal	
		Hy (kN)	My (kN.m)	Hx (kN)	Mx (kN.m)
Service Ser-I		131	94		
Strength Str-IA		202	146		

Load Combination at section 6-6					
Load combinations	N (kN)	Vertical		Horizontal	
		Hy (kN)	My (kN.m)	Hx (kN)	Mx (kN.m)
Service Ser-I				84	81
Strength Str-IA				129	128

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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	=	0.800 m
b0	=	0.650 m
A1	=	0.520 m ²
w	=	1.000 m
b	=	0.850 m
A2	=	0.850 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

case 1

$$m = 1.279$$

Resistance factor

$$\phi = 0.700$$

Factored bearing resistance

$$Pr = 11867 \text{ kN}$$

<S.5.5.4.2>

Bearing reaction of approach bridge

$$Pu = 5553 \text{ kN}$$

> Pu

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

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

$$fn = 2.25 \cdot f'ci$$

$$fn = 26.85 \text{ MPa}$$

Maximum area of the portion of supporting surface

$$A = 0.850 \text{ m}^2$$

Gross area of bearing plate

$$Ag = 0.520 \text{ m}^2$$

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

$$Ab = 0.520 \text{ m}^2$$

Nominal concrete strength at time of application

$$f'ci = 30 \text{ MPa}$$

Factored bearing resistance

$$Pr = 9773 \text{ kN}$$

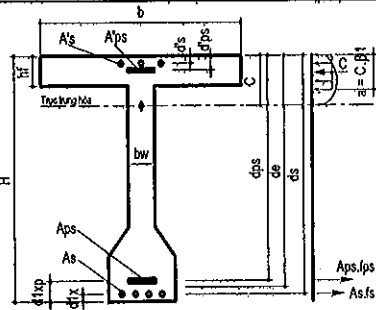
Ok

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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
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				
			1-1	1-1	2-2	2-2	2-2
INTERNAL FORCES AT SECTION							
	Combination		Strength	Service	Service	Strength	Extreme
Qu	Shear	kN	727	444	2297	3402	3421
Mu	Flexural Moment	kNm	741	445	7764	11234	9138
Nu	Axial load	kN	487	390	7950	10682	9024
Tu	Torsional Moment	kNm	0	0	0	0	0
FLEXURAL MOMENT CHECKING							
H	Section height	m	0.500	0.500	1.500	1.500	1.500
d's	Dis. From comp. fiber to centroid of comp. Reinf	m	0.058	0.058	0.058	0.058	0.058
d1x	Dis. From tens. fiber to centroid of tension Reinf	m	0.058	0.058	0.061	0.061	0.061
	Cover to reinf	m	0.050	0.050	0.050	0.050	0.050
ds	Dis. From comp. fiber to centroid of tension Reinf	m	0.442	0.442	1.439	1.439	1.439
d'ps	Dis. From comp. fiber to centroid of comp. prestressing steel	m	0.000	0.000	0.000	0.000	0.000
d1xp	Dis. From tens. fiber to centroid of tension prestressing steel	m	0.000	0.000	0.000	0.000	0.000
dps	Dis. From comp. fiber to centroid of tension prestressing steel	m	0.500	0.500	1.500	1.500	1.500
b	Width of the compression face of member	m	12.600	12.600	12.600	12.600	12.600
bw	Web width or diameter of a circular section	m	12.600	12.600	12.600	12.600	12.600
hf	Compression flange depth	m	0.000	0.000	0.000	0.000	0.000
Iz	Moment of inertia of section	m4	0.131	0.131	3.544	3.544	3.544
Amc	Section area	m2	6.300	6.300	18.900	18.900	18.900
Steel choice							
Aps	Tension prestressing steel	P.S type	0	0	0	0	0
		Number	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
As	Tension Reinforcement	Number	82	82	82	82	82
		Diameter	16	16	22	22	22
		Area	0.01656	0.01656	0.03116	0.03116	0.03116
A's	Compression Reinforcement	Number	82	82	82	82	82
		Diameter	16	16	16	16	16
		Area	0.01656	0.01656	0.01656	0.01656	0.01656
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
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	0.022	0.022	0.022
	For T section behavior	m	0.000	0.000	0.022	0.022	0.022
	For rectangular section behavior	m	0.000	0.000	0.022	0.022	0.022
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	1852	1852	1852
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	0.018	0.018	0.018
de	Corresp. effective depth from extreme comp. fiber to centroid of tensile force in the tensile reinf.	m	0.442	0.442	1.439	1.439	1.439
Mn	Nominal resistance	kNm	2544	2544	17498	17498	17498
Mr	Factored resistance	kNm	2290	2544	17498	15749	17498
Mu	Flexural moment	kNm	741	445	7764	11234	9138
(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*Mer	Cracking moment	kNm	1087	1087	9926	9926	9926
(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?		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	0.061	0.061	0.061
Z	Crack width parameter	N/mm	30000	30000	30000	30000	30000
A	Area of concr. with same centroid as tens. Reinf	m ²	0.018	0.018	0.019	0.019	0.019
f _{sa}	Value	Mpa	297	297	287	287	287
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	-	0.081	0.207	-	-
J.d	Arm	m	-	0.415	1.37	-	-
I _{cr}	Moment of inertia of the cracked section	m ⁴	-	0.017	0.371	-	-
f _s	Tensile stress in reinforcement $f_s = M_{sls} / (A_s * J.d)$	Mpa	-	65	182	-	-
	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		2.4	3.0	2.5	2.3	2.4
θ	Angle of inclination of diagonal compressive	degree	30.87	28.73	29.87	33.75	32.67
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	1.430	1.430	1.430
	(d _e - a/2)	m	0.442	0.442	1.430	1.430	1.430
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	0.600
ncat	Amount of bars in spacing S	bars	20	20	19	19	19
A _v	Shear reinf area in spacing S	m ²	0.0030	0.0030	0.0029	0.0029	0.0029
θ	Assume	degree	30.86	28.74	29.88	33.75	32.66
v	Shear stress in concrete	kN/m ²	145	53	127	210	190
f _{po}	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e _x	Strain in tensile reinforcement		6.17E-04	3.67E-04	5.54E-04	8.12E-04	7.30E-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		0.005	0.002	0.004	0.007	0.006
β	Final value		2.4	3.0	2.5	2.3	2.4
θ	Final value	degree	30.87	28.73	29.87	33.75	32.67
V _c	Nominal shear resistance provided by tensile stresses in the concrete	kN	6190	7675	20310	19127	19505
V _s	Shear resistance provided by shear reinforcement	kN	1489	1623	4763	4094	4264
V _p	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0	0
V _{n1}	$V_{n1} = V_o + V_s + V_p$	kN	7678	9298	25073	23221	23770
V _{n2}	V _{n2}	kN	41769	41769	135127	135127	135127
V _n	Nominal shear resistance $V_n = \min(V_{n1}, V_{n2})$	kN	7678	9298	25073	23221	23770
V _r	Factored shear resistance	kN	6911	9298	25073	20899	23770
V _u	Shear	kN	727	444	2297	3402	3421
(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	54051	54051	54051
	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	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	10326	14582	11323	2150	3814
Mu	Flexural Moment	kNm	14574	20873	16593	5420	8572
Nu	Axial load	kN	0	0	0	0	0
Tu	Torsional Moment	kNm	0	0	0	0	0
FLEXURAL MOMENT CHECKING							
H	Section height	m	2.000	2.000	2.000	2.000	2.000
d's	Dis. From comp. fiber to centroid of comp. Reinf	m	0.083	0.083	0.083	0.083	0.083
d1x	Dis. From tens. fiber to centroid of tension Reinf	m	0.164	0.164	0.164	0.160	0.160
ds	Dis. From comp. fiber to centroid of tension Reinf	m	1.836	1.836	1.836	1.840	1.840
d'ps	Dis. From comp. fiber to centroid of comp. prestressing steel	m	0.000	0.000	0.000	0.000	0.000
d1xp	Dis. From tens. fiber to centroid of tension prestressing steel	m	0.000	0.000	0.000	0.000	0.000
dps	Dis. From comp. fiber to centroid of tension prestressing steel	m	2.000	2.000	2.000	2.000	2.000
b	Width of the compression face of member	m	12.600	12.600	12.600	12.600	12.600
bw	Web width or diameter of a circular section	m	12.600	12.600	12.600	12.600	12.600
hf	Compression flange depth	m	0.000	0.000	0.000	0.000	0.000
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
Steel choice							
A _{ps}	Tension prestressing steel	P.S type	0	0	0	0	0
	Number	tendons	0	0	0	0	0
	Area	m ²	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	m ²	0.00000	0.00000	0.00000	0.00000	0.00000
A _s	Tension Reinforcement	Number	84	84	84	84	84
	Diameter	mm	28	28	28	20	20
	Area	m ²	0.05174	0.05174	0.05174	0.02638	0.02638
A' _s	Compression Reinforcement	Number	0	0	0	0	0
	Diameter	mm	16	16	16	16	16
	Area	m ²	0.00000	0.00000	0.00000	0.00000	0.00000
A' _c	Shear reinforcement	Number	15	15	13	13	13
	Diameter	mm	16	16	16	16	16
	Area	m ²	0.00303	0.00303	0.00263	0.00263	0.00263
φ	Resistance factors for flexure		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	0.077	0.077	0.077	0.039	0.039
	For T section behavior	m	0.077	0.077	0.077	0.039	0.039
	For rectangular section behavior	m	0.077	0.077	0.077	0.039	0.039
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	1840	1840	1840	1850	1850
k	Factor depends on type of P.S, Low relaxation strand k = 0.28		0.28	0.28	0.28	0.28	0.28

Da Nang Quang Ngai Expressway project			Item.	Eng.	Date.	Sign.	
ORB23 BRIDGE			Design				
DETAIL DESIGN			Check				
ABUTMENT A2			Revise				
22TCN272-05; AASHTO LRFD 2nd - 1998							
REINFORCEMENT CHECKING - PILECAP SECTION							
a	Depth of equivalent stress block	m	0.064	0.064	0.064	0.033	0.033
de	Corresp. effective depth from extreme comp. fiber to centroid of tensile force in the tensile reinf.	m	1.836	1.836	1.836	1.840	1.840
Mn	Nominal resistance	kNm	37334	37334	37334	19240	19240
Mr	Factored resistance	kNm	37334	33601	37334	19240	17316
Mu	Flexual moment	kNm	14574	20873	16593	5420	8572
(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	0.02	0.02
	Maximum reinforcement Checking	≤ 0.42	OK	OK	OK	OK	OK
1.2*Mer	Craking moment	kNm	18088	18088	18088	17740	17740
(5.7.3.3.2)	Checking $Mr \geq \min(1.2Mer, 1.33Mu)$		OK	OK	OK	OK	OK
(5.7.3.4)	Conctrol of craking by distr. of reinf for RC member- Check?		Yes	No	No	No	No
	Existing condition for structrure	1,2 or 3	3	3	3	3	3
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.050	0.050	0.050	0.050	0.050
Z	Crack width parameter	N/mm	17500	17500	17500	17500	17500
A	Area of concr. with same centroid as tens. Reinf	m2	0.015	0.015	0.015	0.015	0.015
fsa	Value	Mpa	193	193	193	193	193
0.6*fy		Mpa	240	240	240	240	240
	Tensil stress in reinf Min(fs,0.6fy)	Mpa	193	193	193	193	193
x	Dist. From compression fiber to centroid	m	0.297	-	-	-	-
J.d	Arm	m	1.737	-	-	-	-
Icr	Moment of inertia of the cracked section	m4	0.968	-	-	-	-
fs	Tensile stress in reinforcement $fs = Ms / (As * J.d)$	Mpa	162	-	-	-	-
	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	m2	0.00127	0.00127	0.00127	0.00127	0.00127
	Distribution on sides 7 D16	m2	0.00141	0.00141	0.00141	0.00141	0.00141
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45	0.45
	Checking		OK	OK	OK	OK	OK
SHEAR AND TORSION CHECKING							
β	Factor indicating diag. cracked concr. to tension		2.0	1.8	1.9	2.3	2.1
θ	Angle of inclination of diagonal compressive	degree	39.68	42.50	41.04	34.18	39.10
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	1.804	1.804	1.804	1.824	1.824
	(dc - a/2)	m	1.804	1.804	1.804	1.824	1.824
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	0.600
ncat	Amount of bars in spacing S	bars	15	15	13	13	13
Av	Shear reinf area in spacing S	m2	0.0030	0.0030	0.0026	0.0026	0.0026
θ	Assume	degree	40.36	42.94	41.35	35.57	40.77
v	Shear stress in concrete	kN/m2	454	713	498	33	184
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
ex	Strain in tensile reinforcement		1.37E-03	1.88E-03	1.51E-03	8.48E-04	1.31E-03
	if $ex < 0$, multiple with reduce factor		-	-	-	-	-
	Strain checking	$\leq 2.00E-3$	Ok	Ok	Ok	Ok	Ok
v/fc	Ratio of shear stress and fc		0.015	0.024	0.017	0.001	0.006
β	Final value		2.0	1.8	1.9	2.3	2.1
θ	Final value	degree	39.68	42.50	41.04	34.18	39.10
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	20913	18364	20098	24181	21479
Vs	Shear resistance provided by shear reinforcement	kN	4392	3976	3627	4701	3928
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	25305	22341	23725	28882	25406
Vn2	Vn2	kN	170458	170458	170458	172328	172328
Vn	Nominal shear resistance $Vn = \min(Vn1, Vn2)$	kN	25305	22341	23725	28882	25406
Vr	Factored shear resistance	kN	25305	20107	23725	28882	22866
Vu	Shear	kN	10326	14582	11323	2150	3814
(5.8.2.7)	Shear checking		OK	OK	OK	OK	OK

22TCN272.05: AASHTO L BED 2nd - 1998

MATERIALS

Abutment A2-ORB23.xls-Sheet:Check-WW

Da Nang Quang Ngai Expressway project				Item.	Eng.	Date.	Sign.
ORB23 BRIDGE				Design			
DETAIL DESIGN				Check			
ABUTMENT A2				Revise			
22TCN272-05: AASHTO LRFD 2nd - 1998							
REINFORCEMENT CHECKING - WING WALL							
a	Depth of equivalent stress block	m	0.006	0.006	0.006	0.006	
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	
Mn	Nominal resistance	kNm	280	280	280	280	
Mr	Factored resistance	kNm	280	252	280	252	
Mu	Flexural moment	kNm	94	146	81	128	
(5.7.3.2)	Flexural moment Checking		OK	OK	OK	OK	
	Limits for reinforcement						
c/de	Maximum reinforcement		0.02	0.02	0.02	0.02	
	Maximum reinforcement Checking	≤ 0.42	OK	OK	OK	OK	
r min	Minimum reinforcement		0.36%	0.36%	0.36%	0.36%	
	Minimum reinforcement Checking for RC	0.23%	OK	OK	OK	OK	
1.2*Mc	Cracking moment	kNm	87	87	87	87	
(5.7.3.3.2)	Checking $M_r \geq \min(1.2M_c, 1.33M_u)$		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	0.017	0.017	0.017	0.017	
f _{sa}	Value	Mpa	301	301	301	301	
0.6*f _y	Tensile stress in reinf Min(f _{sa} , 0.6*f _y)	Mpa	240	240	240	240	
x	Dist. From compression fiber to centroid	m	0.093	-	0.093	-	
J.d	Arm	m	0.41	-	0.41	-	
I _{cr}	Moment of inertia of the cracked section	m ⁴	0.002	-	0.002	-	
f _s	Tensile stress in reinforcement $f_s = M_{sl} / (A_s * J.d)$	Mpa	129	-	111	-	
	Checking for control cracking $f_s < f_{sa}$		OK	N.a	OK	N.a	
(5.10.8.2)	Shrinkage and temperature Reinforcement (side distribution)						
A _{req}	Area of required reinf	m ²	0.00031	0.00031	0.00031	0.00031	
	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	
	Checking		OK	OK	OK	OK	
SHEAR AND TORSION CHECKING							
β	Factor indicating diag. cracked concr. to tension		2.3	2.0	2.4	2.2	
θ	Angle of inclination of diagonal compressive	degree	35.18	39.71	32.70	36.97	
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 d _v	m	1.000	1.000	1.000	1.000	
d _v	Effective shear depth	m	0.438	0.438	0.438	0.438	
	(d _e - a/2)	m	0.438	0.438	0.438	0.438	
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	
n _{cat}	Amount of bars in spacing S	bars	2	2	2	2	
A _v	Shear reinf area in spacing S	m ²	0.0002	0.0002	0.0002	0.0002	
β	Assume		2.5	2.4	2.5	2.4	
θ	Assume	degree	29.28	33.12	29.14	33.44	
v	Shear stress in concrete	kN/m ²	299	513	191	328	
f _{po}	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	
e _x	Strain in tensile reinforcement		9.32E-04	1.37E-03	7.31E-04	1.10E-03	
	if e _x < 0, multiple with reduce factor		-	-	-	-	
	Strain checking	$\leq 2.00E-3$	Ok	Ok	Ok	Ok	
w/f _c	Ratio of shear stress and f _c		0.010	0.017	0.006	0.011	
β	Final value		2.3	2.0	2.4	2.2	
θ	Final value	degree	35.18	39.71	32.70	36.97	
V _c	Nominal shear resistance provided by tensile stresses in the concrete	kN	452	403	474	433	
V _s	Shear resistance provided by shear reinforcement	kN	94	80	103	88	
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	546	482	577	521	
V _{n2}	V _{n2}	kN	3286	3286	3286	3286	
V _n	Nominal shear resistance V _n = min(V _{n1} , V _{n2})	kN	546	482	577	521	
V _r	Factored shear resistance	kN	546	434	577	469	
V _u	Shear	kN	131	202	84	129	
(5.8.2.7)	Shear checking		OK	OK	OK	OK	

	Da Nang Quang Ngai Expressway project ORB23 DETAIL DESIGN ABUTMENT A2	Item.	Eng.	Date.	Sign.
		Design			
		Check			
		Revise			

BORED PILE DESIGN

I. BORED PILE DATA

1. Load Combinations at top of bored pile

STT	Comb	Axial force P (KN)	Moment (KN.m)		
			Mx	My	Mxy
STRENGTH LIMIT STATES					
1	P_min	860	28.5	950.6	951
2	P_max	4359	46.9	1654.3	1655
3	Mx_max	4359	46.9	1654.3	1655
4	My_max	4359	46.9	1654.3	1655
EXTREME EVENT LIMIT STATES					
1	P_min	500	105	1193	1198
2	P_max	3567	46	1388	1389
3	Mx_max	3567	46	1388	1389
4	My_max	3567	46	1388	1389

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

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	m2
$A_s / A_g \geq 0.01$				$A_s \geq$		0.008	m2
Maximum area of longitudinal reinforcement in column							
$A_s / A_g \leq 0.08$				$A_s \leq$		0.063	m2
Trial Rebars:				Ok	A_s	0.019	m2
1 layers	x 24	= 24 bars	D32	@150	A_{s1}	0.019	m2

2. Interaction diagram M-P

Using Pca-Column software

**Flexural check by pcaColumn

Strength and Service limit states:

Resistance factor:	Compression	ϕ_c	=	0.75 (AASHTO LRFD-2004)
	Tension	ϕ_t	=	0.90

Extreme Event limit states:

Resistance factor	Compression	ϕ_c	=	1.00
	Tension	ϕ_t	=	1.00

<Result table>

STT	Comb	Pu kN	Mux kN-m	Muy kN-m	ϕ Mnx kN-m	ϕ Mny kN-m	ϕ Mn/Mu
STRENGTH LIMIT STATES							
1	P_max	860	29	951	89.6	2754.1	2.621
2	P_min	4359	47	1654	61.6	2827.4	1.556
3	Mx_max	4359	47	1654	70.1	2826.7	1.605
4	My_max	4359	47	1654	61.6	2827.4	1.556
EXTREME EVENT LIMIT STATES							
1	P_max	500	105	1193	254.3	2933.5	2.231
2	P_min	3567	46	1388	140.8	3566.4	2.387
3	Mx_max	3567	46	1388	135.3	3554.5	2.46
4	My_max	3567	46	1388	140.8	3566.4	2.387

3. Column Ties

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	0.622	m2
Tie diameter	Dtie	16	mm
Cross section area of 1 tie	As-tr	0.00020	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			
$\rho_s = A_{s-tr} \cdot L_{tie} / (A_c \cdot \text{spacing})$	ρ_s	0.0120	
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	0.0089	OK
Transverse Reinforcement for Confinement at Plastic Hinges			
S.5.10.11.4.1.d			
For a circular column "1:applied", "2:Not applied"		1	
$\rho_s \geq 0.12 \cdot f_c/f_y = \text{Req2}$	Req2	0.0090	N/A
Length distributed spiral with pitch 75mm below pilecap	Ldis	1.50	m

4. Shear Design

Shear resistance factors	ϕ_v	1.0	
Factored shear force	V_u	740	kN
Required shear capacity $V_n = V_u / \phi_v$	V_n	740	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
$d_v = 0.9 \cdot d_e$ $d_e = D/2 + D_r/\pi()$			
Diameter of the circle passing through the centers of the long. reinf.	D_r	0.79	m
	d_e	0.75	m
	d_v	0.68	m
$V_c = 0.083 \cdot \beta \cdot \sqrt{f_c} \cdot b_v \cdot d_v$	V_c	616	kN
Diameter of Transverse Reinforcement	D_v	13	mm
Spacing of Transverse Reinforcement	s	75	mm
Area of a transverse reinforcement within distance "s"	A_v	339	mm ²
Angle of inclination of transverse reinforcement to longitudinal axis	α	90	deg.
Effective shear depth, d_v			
Alternative 1: $d_{v1} = M_n / (A_s \cdot f_y)$			
Normal flexural resistance	M_n	3555	kNm
	d_{v1}	231	mm
Alternative 2: $d_{v2} = 0.9d_e$	$d_e = D/2 + D_r/\pi$	752	mm
	d_{v1}	677	mm
Choice value of d_v 1 ("1" = d_{v1} , "2" = d_{v2})	d_v	231	mm
	$V_s = \frac{A_v f_y d_v (\cot \theta + \cot \alpha) \sin \alpha}{s}$		
Normal shear resistance of Reinforcement		418	kN
	$V_{n1} = V_c + V_s$	1034	kN
	$V_{n2} = 0.25 f_c b_v d_v$	1733	kN
	V_n	1034	kN
	Conclude		OK

SPACE PILE FOUNDATION ANALYSIS PROGRAM
Turbo BASIC

PROJECT: : Abutment A2 - ORB23

INITIA DATA

Kn = 3.34 Ax = 6.00 By = 12.60 Cz = 2.00
E v.uon = 2944008 E r.uon = 2944008 E v.nen = 2944008 E r.nen =
2944008
Mq = 150 (t/m4) Md = 150 (t/m4) m = 600 (t/m4)

LOAD COMBINATIONS

COMB.	Hx	Hy	P	Mx	My	Mz
1	555.00	0.00	2177.00	-147.00	1492.00	0.00
2	373.00	0.00	1597.00	-137.00	1081.00	0.00
3	536.00	6.00	2109.00	-178.00	1395.00	0.00
4	354.00	6.00	1529.00	-169.00	984.00	0.00
5	372.00	5.00	1627.00	-143.00	987.00	0.00
6	440.00	26.00	1964.00	-178.00	724.00	0.00
7	440.00	26.00	1384.00	-168.00	961.00	0.00

PROPERTIES OF PILES

FILE	Lo	H	Bpx	Bpy	A	B	Cday	Fo	Io	Po	Co	Ct
1	0.00	26.00	1.404	1.404	1.00	1.000	0.000	0.785	0.098	0	2906250	1453125
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	2.00	4.90	0.000	0.00
2	2.00	1.50	0.000	0.00
3	2.00	-1.90	0.000	0.00
4	2.00	-5.30	0.000	0.00
5	-2.00	-5.30	0.000	0.00
6	-2.00	-0.20	0.000	0.00
7	-2.00	4.90	0.000	0.00

DISPLACEMENTS

COMB.	X	Y	Z	Fix	Fiy	Fiz
1	0.01243	0.00018	0.003564	-0.000062	0.000890	0.000088
2	0.00840	0.00014	0.002625	-0.000048	0.000615	0.000059
3	0.01196	0.00030	0.003460	-0.000065	0.000840	0.000087
4	0.00793	0.00026	0.002521	-0.000052	0.000565	0.000058
5	0.00828	0.00024	0.002690	-0.000051	0.000573	0.000060
6	0.00936	0.00069	0.003302	-0.000068	0.000494	0.000077
7	0.00974	0.00066	0.002244	-0.000054	0.000654	0.000076

FORCES ON PILES

PILE	COMB.	N	Q2	Q3	M1	M2	M3
1	1	444.30	-75.47	1.14	1.368	4.777	168.633
	2	322.05	-50.69	0.77	0.919	3.431	112.759
	3	429.65	-72.97	0.29	1.344	2.702	163.653
	4	307.44	-48.19	-0.07	0.895	1.360	107.780
	5	321.48	-50.67	0.10	0.936	1.768	113.957
	6	363.62	-60.42	-2.59	1.188	-4.676	141.511
	7	300.49	-60.15	-2.60	1.183	-5.106	136.046
2	1	427.78	-73.32	1.14	1.368	4.777	163.089
	2	309.08	-49.25	0.77	0.919	3.431	109.036
	3	412.17	-70.86	0.29	1.344	2.702	158.206
	4	293.49	-46.79	-0.07	0.895	1.360	104.154
	5	307.81	-49.20	0.10	0.936	1.768	110.163
	6	345.53	-58.56	-2.59	1.188	-4.676	136.697
	7	285.93	-58.29	-2.60	1.183	-5.106	131.251
3	1	411.27	-71.18	1.14	1.368	4.777	157.545
	2	296.11	-47.81	0.77	0.919	3.431	105.313
	3	394.70	-68.76	0.29	1.344	2.702	152.759
	4	279.53	-45.39	-0.07	0.895	1.360	100.527
	5	294.14	-47.73	0.10	0.936	1.768	106.369
	6	327.44	-56.69	-2.59	1.188	-4.676	131.884
	7	271.38	-56.44	-2.60	1.183	-5.106	126.456
4	1	394.75	-69.03	1.14	1.368	4.777	152.001
	2	283.15	-46.37	0.77	0.919	3.431	101.589
	3	377.23	-66.65	0.29	1.344	2.702	147.312
	4	265.58	-43.98	-0.07	0.895	1.360	96.901
	5	280.47	-46.27	0.10	0.936	1.768	102.575
	6	309.35	-54.83	-2.59	1.188	-4.676	127.070
	7	256.82	-54.58	-2.60	1.183	-5.106	121.661
5	1	114.58	-69.03	-1.39	1.368	-1.746	152.001
	2	89.57	-46.37	-0.92	0.919	-0.950	101.589
	3	112.72	-66.65	-2.18	1.344	-3.706	147.312
	4	87.67	-43.98	-1.72	0.895	-2.907	96.901
	5	100.20	-46.27	-1.63	0.936	-2.696	102.575
	6	153.91	-54.83	-4.78	1.188	-10.339	127.070
	7	50.99	-54.58	-4.78	1.183	-10.747	121.661
6	1	139.36	-72.25	-1.39	1.368	-1.746	160.317
	2	109.02	-48.53	-0.92	0.919	-0.950	107.174
	3	138.93	-69.81	-2.18	1.344	-3.706	155.483
	4	108.59	-46.09	-1.72	0.895	-2.907	102.341
	5	120.70	-48.47	-1.63	0.936	-2.696	108.266
	6	181.05	-57.63	-4.78	1.188	-10.339	134.290
	7	72.82	-57.36	-4.78	1.183	-10.747	128.854
7	1	164.13	-75.47	-1.39	1.368	-1.746	168.633
	2	128.47	-50.69	-0.92	0.919	-0.950	112.759
	3	165.14	-72.97	-2.18	1.344	-3.706	163.653
	4	129.52	-48.19	-1.72	0.895	-2.907	107.780
	5	141.20	-50.67	-1.63	0.936	-2.696	113.957
	6	208.19	-60.42	-4.78	1.188	-10.339	141.511
	7	94.66	-60.15	-4.78	1.183	-10.747	136.046

SUMMARY OF FORCES

	PILE COMB.		N	Q2	Q3	M1	M2	M3
Nmin	5	7	50.99	-54.58	-4.78	1.183	-10.747	121.661
Nmax	1	1	444.30	-75.47	1.14	1.368	4.777	168.633
Q2max	1	1	444.30	-75.47	1.14	1.368	4.777	168.633
Q3max	5	7	50.99	-54.58	-4.78	1.183	-10.747	121.661
M1max	1	1	444.30	-75.47	1.14	1.368	4.777	168.633

M2max	5	7	50.99	-54.58	-4.78	1.183	-10.747	121.661
M3max	1	1	444.30	-75.47	1.14	1.368	4.777	168.633

CHECKING CALCULATI
IN COMPARISON WITH INITIA LOAD MATRIX

1	555.00	0.00	2177.00	-147.00	1492.00	0.00
2	373.00	0.00	1597.00	-137.00	1081.00	0.00
3	536.00	6.00	2109.00	-178.00	1395.00	0.00
4	354.00	6.00	1529.00	-169.00	984.00	0.00
5	372.00	5.00	1627.00	-143.00	987.00	0.00
6	440.00	26.00	1964.00	-178.00	724.00	0.00
7	440.00	26.00	1384.00	-168.00	961.00	0.00

BEARING CAPACITY OF PILE -ORB23

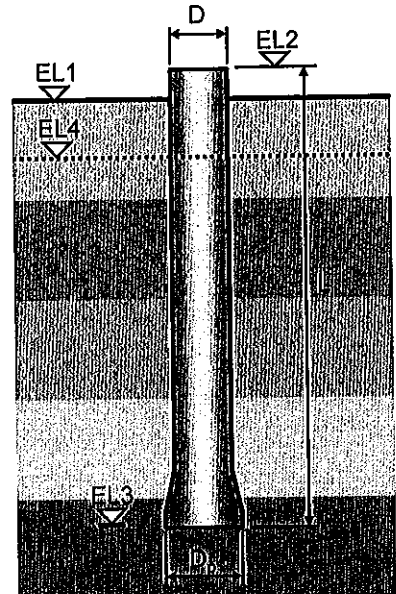
STT	Abut/pier	Boring	Ground Elev (m)	water Elev (m)	Bottom of pile cap Elev (m)	Tip of pile Elev (m)	length of pile (m)	Bearing capacity of pile		internal force of top pile		Check	
								STR	EX	STR	EX	STR	EX
1	A1	ORB23-A1	1.84	0.8	0.0	-26.0	26	469	819	437	342	OK	OK
2	A2	ORB23-A2	2.3	0.8	-0.5	-28.5	28	515	886	444	364	OK	OK

BORED PILE CAPACITY CALCULATION

Package: A2
 Bridge name: ORB23
 Abutment/ Pier: Abutment A1
 Bored hole name: ORB23-A1

1. INPUT DATA

Ground Level EL1 = 1.840 m
 Bottom of pile cap Level EL2 = 0.000 m
 Bottom of Pile Tip Level EL3 = -26.000 m
 Water Level EL4 = 0.800 m
 Pile Length L = 26.000 m
 Length of belled end: L_b = 0.000 m
 Diameter of drilled shaft D_s = 1000 mm
 Tip diameter of drilled shaft D_p = 1.00 m
 Pile Cross-Sectional Perimeter P = 3.142 m
 Pile Cross-Sectional Area A_s = 0.785 m²
 Pile Tip Cross-Sectional Area A_p = 0.785 m²
 Spacing of pile : s = 3.300 m
 Factor of group pile : η = 0.730
 Pile Concrete Strength f_c = 30 MPa
 Concrete Unit Weight γ_c = 24.5 kN/m³
 Effective Unit Weight of Soil layer over Top-Pile γ_{0-eff} = 7.0 kN/m³
 Effective Soil Overburden Pressure at bott. pilecap P_d = 12.88 kPa
 Embedment of drilled shaft D_b = 22.340 m



(22TCN272-05: Table 10.5.5.3 & Handbook)

2. CALCULATION

2.1. Resistance Factor

Resistance Factor	Strength limit state		Extreme limit state	
	For Clay	For Sand	For Clay	For Sand
Side Resistance	0.65	0.65	1.00	1.00
Base Resistance	0.55	0.50	1.00	1.00

2.2. Shaft Resistance

For Cohesive soil: (Using the α - Method)

(22TCN272-05: 10.8.3.3.1)

The normal unit side resistance: q_s = α S_u

where: S_u : Mean undrained shear strength

α : Adhesion factor

The following portion of a drilled shaft shall not be taken to contribute to the development of resistance through skin friction:

Top length of shaft: = 1.50 m

Bottom length of the shaft: = 1.00 m

For Cohesionless soil:

1

(22TCN272-05: 10.8.3.4.2)

("1" = Tuma and Reese, "2" = Meyerhof, "3" = Quiros and Reese, "4" = Reese and Wright, "5" = Reese and O'Neill)

Tuma and Reese (1974)	q _s = Kσ _v ' tanφ _r < 0.24Mpa for which K = 0.7 for D _b ≤ 7500mm K = 0.6 for 7500mm < D _b ≤ 12000mm K = 0.6 for D _b > 12000mm	Reese and Wright (1977)	for N ≤ 53 q _s = 0.0028N for 53 < N ≤ 100 q _s = 0.00021(N-53) + 0.15
Meyerhof (1976)	q _s = 0.00096N	Reese and O'Neill (1986)	q _s = βσ _v ' < 0.19Mpa for 0.26 < β ≤ 1.2 for which β = (1.6 / (1.75 + 10 ⁻⁴ z)) ^{1/2}
Quiros and Reese (1977)	q _s = 0.0025N < 0.19Mpa		

where: N : Uncorrected SPT blow count (Blows/300mm)

σ_v' : Vertical effective stress (Mpa)

φ_r : Friction angle of sand (DEG)

K : load transfer factor

D_b : Embedment of drilled shaft in sand bearing layer (mm)

b : Load transfer coefficient

z : depth below ground (mm)

Depth (m)	Layer Number	Thickness (m)	Soil Type "1"=Sand, "2"=Clay		N _{SPT}	φ _r (deg.)	S _u (kPa)	q _s (kPa)	Q _s (KN)	
									Strength	Extreme
0	1a	1.16	2	Clay	5	-	38.3	21.1	-	-
-5	2b	2.50	2	Clay	8	-	50.0	27.5	140	216
-10	4a	0.50	1	Sand	60	45.00	-	33.6	34	53
-15			-	-	-	-	-	-	-	-
-20			-	-	-	-	-	-	-	-
-25	4b	2.00	1	Sand	65	45.00	-	41.5	170	261
-30	5b	8.00	1	Sand	65	45.00	-	62.6	1023	1573
	5b	8.00	1	Sand	65	45.00	-	105.8	1728	2659
	5b	3.84	1	Sand	65	45.00	-	160.7	1260	1939
	End pile	26.000	1	Sand	65	45.00	-	-	-	-
Total Q _s =									Negative Skin Friction(DD) =	
									Positive Skin Friction Q _s =	
									4355	6701

2.3. Tip Resistance

For Cohesive soil:

The normal unit tip resistance: $q_p = N_c S_u \leq 4.0$

for which: $N_c = 6[1+0.2(Z/D)] \leq 9$ where: Z is penetration of shaft

The value of S_u shall be determined from the results of in-situ and/or laboratory testing of undisturbed sample obtained within a depth of 2.0 diameter below the tip of shaft.

If the soil within 2.0 diameter of the tip has S_u < 0.024MPa, the value of N_c shall be reduced by one-third.

With S_u > 0.096MPa with D > 1900mm, and for which shaft settlement will not be evaluated, the value of q_p shall be reduced to q_{pr}, as follows:

for which

$$q_{pr} = q_p F_r$$

$$F_r = \frac{760}{12.0aD_p + 760b} \leq 1.0$$

$$a = 0.0071 + 0.0021 \frac{Z}{D_p} \leq 0.015$$

$$b = 1.45 \sqrt{2.0S_u} \text{ with } 0.5 \leq b \leq 1.5$$

For Cohesionless soil:

2

("1" = Touma and Reese, "2" = Meyerhof, "3" = Reese and Wright, "4" = Reese and O'Neill)

Touma and Reese (1974)	Loose: q _p = 0.0	(Meyerhof (1976))	q _p = 0.013N ₆₀ D _p /D _p ≤ 0.13N ₆₀
	Medium Dense: q _p = 1.5/K		where: N ₆₀ = (0.77/g(1.92/c _u))N
	Very Dense: q _p = 3.8/K		
	K = 1 for D _p < 500mm	Reese and Wright (1977)	q _p = 0.064N for N ≤ 60
	K = 0.6 for D _p ≥ 500mm		q _p = 3.8 for N > 60
	Applicable only if D _p > 10D	Reese and O'Neill (1988)	q _p = 0.057N for N ≤ 75
			q _p = 4.3 for N > 75

Sand Consistency: 2 ("1" = Loose, "2" = Medium Dense, "3" = Very Dense)

For base diameters greater than 1270mm, q_p should be reduced as follows: q_{pr} = 1270*q_p/D_p

q _p	Q _p (KN)	
(kPa)	Strength	Extreme
6001.14	2357	4713

2.4. Bearing Capacity of Pile

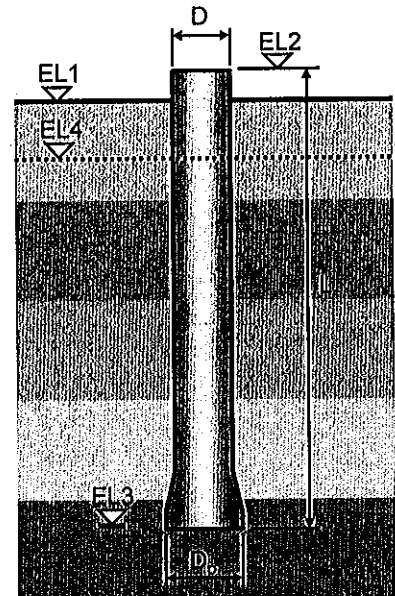
		Strength limit state		Extreme limit state	
		(KN)	(Ton)	(KN)	(Ton)
Pile Structural Capacity:	$Q_T = \phi * 0.85 * 0.85 * f'_c * A_s$	12768	1301	17024	1735
Deducting Pile Weight	PW	-300	-31	-300	-31
Soil Bearing Capacity:	$Q_r = (Q_s + Q_p) + DD + PW$	6412	654	11114	1133
Bearing resistance of Single Pile		6412	654	11114	1133
Bearing resistance of one Pile in Pile Group:		4600	469	8032	819

BORED PILE CAPACITY CALCULATION

Package: A2
 Bridge name: ORB23
 Abutment/ Pier: Abutment A2
 Bored hole name: ORB23-A2

1. INPUT DATA

Ground Level	EL1	=	2.300	m
Bottom of pile cap Level	EL2	=	-0.500	m
Bottom of Pile Tip Level	EL3	=	-28.500	m
Water Level	EL4	=	0.800	m
Pile Length	L	=	28.000	m
Length of belled end:	L_b	=	0.000	m
Diameter of drilled shaft	D_s	=	1000	mm
Tip diameter of drilled shaft	D_p	=	1.00	m
Pile Cross-Sectional Perimeter	P	=	3.142	m
Pile Cross-Sectional Area	A_s	=	0.785	m ²
Pile Tip Cross-Sectional Area	A_p	=	0.785	m ²
Spacing of pile :	s	=	3.300	m
Factor of group pile :	η	=	0.730	
Pile Concrete Strength	f'_c	=	30	MPa
Concrete Unit Weight	γ_c	=	24.5	kN/m ³
Effective Unit Weight of Soil layer over Top-Pile	γ_{0-off}	=	7.0	kN/m ³
Effective Soil Overburden Pressure at bott. pilecap	P_d	=	19.60	kPa
Embedment of drilled shaft	D_b	=	17.600	m



(22TCN272-05: Table 10.5.5.3 & Handbook)

2.1. Resistance Factor

Resistance Factor	Strength limit state		Extreme limit state	
	For Clay	For Sand	For Clay	For Sand
Side Resistance	0.65	0.65	1.00	1.00
Base Resistance	0.55	0.50	1.00	1.00

2.2. Shaft Resistance

For Cohesive soil: (Using the α - Method)

(22TCN272-05: 10.8.3.3.1)

The normal unit side resistance: $q_s = \alpha S_u$

where: S_u : Mean undrained shear strength
 α : Adhesion factor

The following portion of a drilled shaft shall not be taken to contribute to the development of resistance through skin friction:

Top length of shaft: = 1.50 m
 Bottom length of the shaft: = 1.00 m

For Cohesionless soil:

1

(22TCN272-05: 10.8.3.4.2)

("1" = Touma and Reese, "2" = Meyerhof, "3" = Quiros and Reese, "4" = Reese and Wright, "5" = Reese and O'Neill)

Touma and Reese (1974)	$q_s = K \sigma_v' \tan \phi_r \leq 0.24 \text{ Mpa}$ for which: $K = 0.7$ for $D_b \leq 7500 \text{ mm}$ $K = 0.6$ for $7500 \text{ mm} \leq D_b \leq 12000 \text{ mm}$ $K = 0.6$ for $D_b \geq 12000 \text{ mm}$	Reese and Wright (1977)	$q_s = 0.0028N$ for $53 < N \leq 100$ $q_s = 0.00021(N-53) + 0.15$
Meyerhof (1976)	$q_s = 0.00096N$	Reese and O'Neill (1988)	$q_s = \beta \sigma_v' \leq 0.19 \text{ Mpa}$ for $0.25 < \beta \leq 1.2$ for which $\beta = 1.6 - 0.76 \sqrt{z/b}$
Quiros and Reese (1977)	$q_s = 0.0025N < 0.19 \text{ Mpa}$		

where: N : Uncorrected SPT blow count (Blows/300mm)
 σ_v' : Vertical effective stress (Mpa)
 ϕ_r : Friction angle of sand (DEG)
 K : load transfer factor
 D_b : Embedment of drilled shaft in sand bearing layer (mm)
 b : Load transfer coefficient
 z : depth below ground (mm)

Depth (m)	N Value	Layer Number	Thickness (m)	Soil Type "1"=Sand, "2"=Clay	N _{SPT}	φ _r (deg.)	S _u (kPa)	q _s (kPa)	Q _s (KN)	
									Strength	Extreme
0		1b	2.40	1 Sand	5	30.83	-	12.2	60	92
-5		2c	8.00	2 Clay	6	-	25.0	13.8	225	346
-10		4a	3.30	1 Sand	65	45.00	-	89.6	604	929
-15				-		-	-	-	-	-
-20				-		-	-	-	-	-
-25				-		-	-	-	-	-
-30				-		-	-	-	-	-
		4c	4.00	1 Sand	65	45.00	-	112.6	920	1415
		4c	4.00	1 Sand	65	45.00	-	137.8	1126	1732
		4c	3.00	1 Sand	65	45.00	-	159.9	979	1507
		5a	3.30	1 Sand	65	45.00	-	179.7	1211	1863
		End pile	28.000	1 Sand	65	45.00	-	-	-	-
Total Q _s =									Negative Skin Friction(DD) =	
									Positive Skin Friction Q _s =	
									5125	7884

2.3. Tip Resistance

For Cohesive soil:

The normal unit tip resistance: $q_p = N_c S_u \leq 4.0$

for which: $N_c = 6[1+0.2(Z/D)] \leq 9$ where: Z is penetration of shaft

The value of S_u shall be determined from the results of in-situ and/or laboratory testing of undisturbed sample obtained within a depth of 2.0 diameter below the tip of shaft.

If the soil within 2.0 diameter of the tip has S_u < 0.024MPa, the value of N_c shall be reduced by one-third.

With S_u > 0.096MPa with D > 1900mm, and for which shaft settlement will not be evaluated, the value of q_p shall be reduced to q_{pr}, as follows:

$$q_{pr} = q_p F_r$$

for which

$$F_r = \frac{760}{12.0aD_p + 760b} \leq 1.0$$

$$a = 0.0071 + 0.0021 \frac{Z}{D_p} \leq 0.015$$

$$b = 1.45 \sqrt{2.0S_u} \text{ with } 0.5 \leq b \leq 1.5$$

For Cohesionless soil:

2

("1" = Touma and Reese, "2" = Meyerhof, "3" = Reese and Wright, "4" = Reese and O'Neill)

Touma and Reese (1974)	Loose: q _p = 0.0	Meyerhof (1976)	q _p = 0.013 N ₆₀ D _p /D _p < 0.13 N ₆₀
	Medium Dense: q _p = 1.5/K	Reese and Wright (1977)	q _p = 0.064 N for N ≤ 60 q _p = 3.8 for N > 60
	Very Dense: q _p = 3.8/K K = 1 for D _p < 500mm K = 0.6 for D _p ≥ 500mm Applicable only if D _p > 10D	Reese and O'Neill (1988)	q _p = 0.057 N for N ≤ 75 q _p = 4.3 for N > 75

Sand Consistency: 2 "1" = Loose, "2" = Medium Dense, "3" = Very Dense

For base diameters greater than 1270mm, q_p should be reduced as follows: q_{pr} = 1270*q_p/D_p

q _p	Q _p (KN)	
(kPa)	Strength	Extreme
5685.32	2233	4465

2.4. Bearing Capacity of Pile

		Strength limit state		Extreme limit state	
		(KN)	(Ton)	(KN)	(Ton)
Pile Structural Capacity:	$Q_T = \phi * 0.85 * 0.85 * f'_c * A_s$	12768	1301	17024	1735
Deducting Pile Weight	PW	-323	-33	-323	-33
Soil Bearing Capacity:	$Q_r = (Q_s + Q_p) + DD + PW$	7034	717	12026	1226
Bearing resistance of Single Pile:		7034	717	12026	1226
Bearing resistance of one Pile in Pile Group:		5048	515	8692	886

6 LRB12a

Table of content - Lrb12a Bridge

A. Substructure design

1. Abutment A1
2. Pier P2
3. Pier P3
4. Bored pile capacity

B. Miscellaneous

1. Expansion joint

MINISTRY OF TRANSPORT

VIETNAM EXPRESSWAY CORPORATION - PROJECT MANAGEMENT UNIT NO.85

DA NANG - QUANG NGAI EXPRESSWAY DEVELOPMENT PROJECT

PACKAGE: A2

BRIDGE

LRB 12A

CALCULATION SHEETS

SUBSTRUCTURE

MINISTRY OF TRANSPORT

VIETNAM EXPRESSWAY CORPORATION - PROJECT MANAGEMENT UNIT NO.85

DA NANG - QUANG NGAI EXPRESSWAY DEVELOPMENT PROJECT

PACKAGE: A2

BRIDGE

LRB 12a

CALCULATION SHEETS

SUPERSTRUCTURE

Da Nang Quang Ngai Expressway project

BRIDGE
LRB 12a

CALCULATION SHEETS
ABUTMENT A1

Table of content

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

	Da Nang Quang Ngai Expressway project LRB 12a 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.
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	7.867	m
Level of top of bearing	BTL	5.742	m
Level of top of stem abutment	HTL	5.592	m
Level of top of footing	FTL	1.500	m
Level of bottom of footing	FBL	-0.500	m
Ground level	GL	2.500	m
Lowest water level	HWL	-0.490	m
Skew angle	α	0.00	deg

Material Unit Weights

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

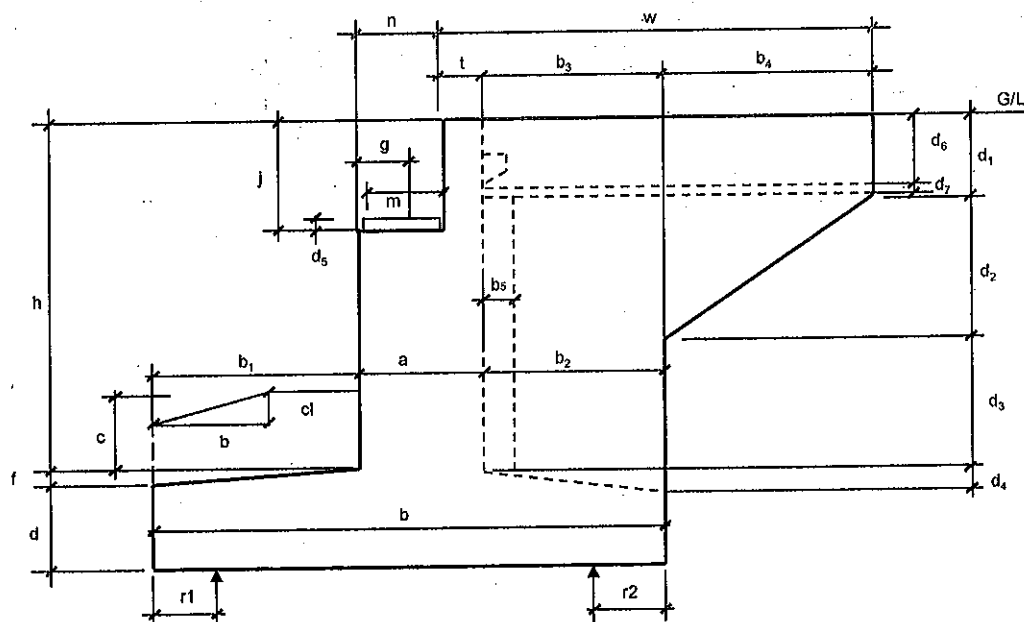
γ_c	=	24.5 kN/m ³
γ_s	=	18.0 kN/m ³
γ_{sbo}	=	8.2 kN/m ³
γ_a	=	22.1 kN/m ³

I.Loads from substructure

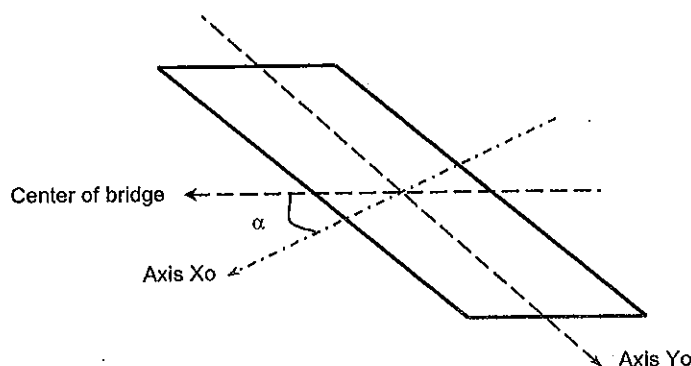
Abutment dimensions

VERTICAL VIEW

Bearing Type: **MOVE**



PLAN VIEW



Da Nang Quang Ngai Expressway project LRB 12a BRIDGE DETAIL DESIGN ABUTMENT A1	Item.	Eng.	Date.	Sign.
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ABUTMENT DIMENSIONS (IN METRES)					
Item	Symbol	Value	Item	Symbol	Value
Height of stem	h	6.300	Horizontal Dimension	b ₄	2.200
Footing Width	b	6.000	Horizontal Dimension	b ₅	0.300
Stem Width	a	1.500	Vertical Dimension	d ₁	0.930
Footing Depth	d	2.000	Vertical Dimension	d ₂	2.200
Footing Slope	f	0.000	Vertical Dimension	d ₃	3.170
Width of stem at bearing	n	1.000	Vertical Dimension	d ₄	
Ballast Wall Height	j	2.275	Vertical Dimension	d ₅	0.150
Ballast Wall Thickness	t	0.500	Vertical Dimension	d ₆	1.200
Wingwall Length	w	5.200	Vertical Dimension	d ₇	
Soil Cover at Toe	c	1.000	With of bearing pad	m	0.775
Girder Reaction	g	0.530	Wingwall Thickness	u1	0.500
Horizontal Dimension	b ₁	2.000	Wingwall Thickness	u2	0.500
Horizontal Dimension	b ₂	2.500	Distance to cl of pile	r1	1.000
Horizontal Dimension	b ₃	2.500	Distance to cl of pile	r2	1.000

Slope front of abutment

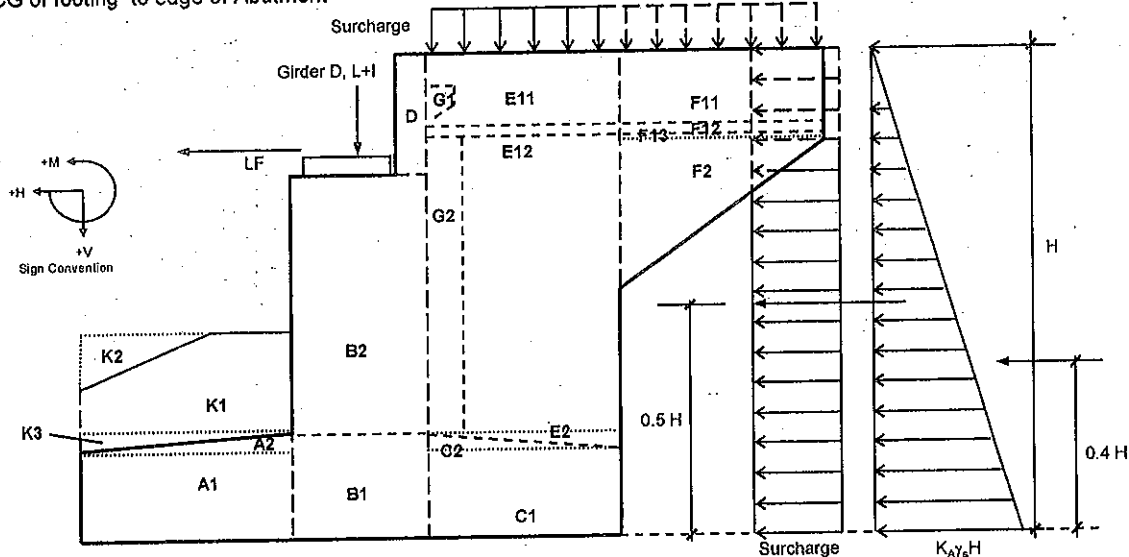
Width of Abutment

Width of abutment (inclined direction)

Height of Abutment

Distance from CG of footing to edge of Abutment

$$\begin{aligned}\cos(\alpha) &= 1.00 \\ cl &= 0.00 \text{ m} \\ bl &= 0.00 \text{ m} \\ L &= 12.600 \text{ m} \\ Ltr &= 12.600 \text{ m} \\ Ht &= 8.37 \text{ m} \\ b/2 &= 3.00 \text{ m}\end{aligned}$$



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	6.038	12.600	1864	2.750	0.250	466
Section C1	5.000	12.600	1544	4.750	-1.750	-2701
Section C2	-	12.600	-	4.333	-1.333	-
Section D	1.138	12.600	351	3.250	-0.250	-88
Section E11	2.325	0.500	28	4.750	-1.750	-50
Section E12	13.425	0.500	164	4.750	-1.750	-288
Part extra stem	-	-	-	5.417	-2.417	-
Section F11	2.640	0.500	32	7.100	-4.100	-133
Section F12	-	0.500	-	5.850	-2.850	-
Section F13	-0.594	0.500	-7	7.100	-4.100	30
Section F2	2.420	0.500	30	6.733	-3.733	-111
Section G1	0.135	12.100	40	3.650	-0.650	-26
Section G2	0.045	6.300	7	3.650	-0.650	-5
Bearing seats (w1seat= 0.80m)	0.116	4.000	11	2.530	0.470	5
Curbs +Handrail on Abutment	0.55	5.200	70	5.600	-2.600	-182
Total SW of Abutment (DC)			6295			-380
Transverser moment			55		6.175	338

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.

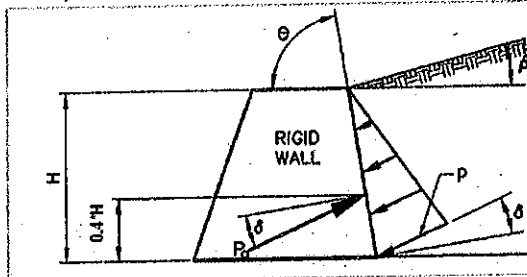
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2. Earth on Abutment (EV)

Description	Area (m ²)	Length (m)	Force (kN)	X ⁽¹⁾ (m)	Arm ⁽²⁾ (m)	Moment (kN·m)
Earth on Abutment (EV)						
Section E1	15.75	12.100	3430	4.750	-1.750	-6003
Section E2	-	12.100	-	5.167	-2.167	-
Section E3	-	0.500	-	6.000	-3.000	-
Section K1	2.000	12.600	454	1.000	2.000	-
Section K2	-	12.600	-	-	3.000	-
Section K3	-	12.600	-	0.667	2.333	-
Total Earth on Footing			3884			-6003

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.



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

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

- 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 K_a (assume wall move or deflect sufficiently to reach minimum active conditions)

H	=	8.37 m
W	=	12.6 m
γ_s	=	1835 kg/m ³
ϕ'_f	=	30.0 deg
θ	=	90.0 deg
δ	=	10.0 deg
β	=	0.0 deg
g	=	9.81 m/s ²

Γ	=	2.469
K_a	=	0.308
p	=	0.046 Mpa

Horizontal earth pressure:

- $E_a = 0.5 \cdot p \cdot Z \cdot B \cdot 10^3 \text{ (kN)}$
- $M = E_a \cdot 0.4H$

E_a	=	2449 kN
M	=	8196 kNm

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

Equivalent height of soil for highway loading taken from Table 3.11.6.2.1

H=	1.50m	heq=	1.7 m
H=	3.00m	heq=	1.2 m
H=	6.00m	heq=	0.76 m
H=	9.00m	heq=	0.61 m
H=	8.37m	heq=	0.64 m

(Linear interpolation)

- Vertical force

ESv	=	364 kN
ev	=	-1.75 m
M	=	-637 kNm

- Horizontal force

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

ESh	=	376 kN
eh	=	4.18 m
M	=	1571 kNm

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5. Earthquake effects

Bridge is located at:

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 $\delta = 10.0 \text{ deg}$
- 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 $K_{AE} = 0.37$

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

• Seismic active lateral Earth pressure coefficient

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

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

$E_{AE} = 2807 \text{ kN}$

$M_{AE} = 7946 \text{ KNm}$

<A.11.1.1.1>

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 $C_{sm} = 0.088$
- $C_{sm} = 1.2 \cdot A \cdot S / T_m^{2/3} \leq 2.5 \cdot A$
- Period of vibration of the fundamental mode $T_m = 0.703 \text{ s}$
- $T_m = 2 \cdot \pi \cdot l / \sqrt{m/k}$

Description	Area (m ²)	Length (m)	Force (kN)	$\chi^{(1)}$ (m)	Arm ⁽²⁾ (m)	Moment (kN·m)
Section A1	4.000	12.600	109	-	1.000	109
Section A2	-	12.600	-	-	2.000	-
Section B1	3.000	12.600	82	-	1.000	82
Section B2	6.038	12.600	164	-	4.013	658
Section C1	5.000	12.600	136	-	1.000	136
Section C2	-	12.600	-	-	2.000	-
Section D	1.138	12.600	31	-	7.163	221
Section E11	2.325	0.500	3	-	5.700	14
Section E12	13.425	0.500	14	-	2.550	-
Section E2	-	-	-	-	2.000	-
Section F11	2.640	0.500	3	-	5.700	16
Section F12	-	0.500	-	-	5.100	-
Section F13	-0.594	0.500	-1	-	6.435	-
Section F2	2.420	0.500	3	-	6.637	17
Section G1	0.135	12.100	4	-	5.587	20
Section G2	0.045	6.300	1	-	2.550	2
Total EQ of Abutment Selfweight			547			1275

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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
- At Top of foundation
- At Bottom of foundation

n	=	3 lanes	
m	=	0.85	
BR	=	104 kN	Long. Axis
e	=	8.1 m	
Mlong	=	839 KNm	
Mlong	=	1046 KNm	

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

V	=	120 km/h	
V	=	33.3 m/s	
R	=	- m	
C	=	-	
CE	=	0.00 KN	
e	=	8.10 m	
Mtrans	=	0.00 KNm	Trans. Axis

8. Water Load (WA) :NA

8.1. Buoyancy of Abutment

- Highest water Level

+0.49

Description	Area (m ²)	Length (m)	Force (kN)	X ⁽¹⁾ (m)	Arm ⁽²⁾ (m)	Moment (kN•m)
Bouyancy on abutment						
Section A1	0.02	12.600	-2	1.000	2.000	-5
Section A2	-	12.600	-	-	3.000	-
Section B(B1,B2)	0.02	12.600	-2	2.750	0.250	-0
Section C1	0.03	12.600	-3	4.750	-1.750	5
Section C2	-	12.600	-	-	3.000	-
Section E2	-	1.000	-	-	3.000	-
Section E1	-	1.000	-	4.750	-1.750	-
Section F2	-	1.000	-	4.280	-1.280	-
Total Bouyancy			-7			

8.2 Buoyancy of Earth on Abutment

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

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SUPERSTRUCTURE LOADS

II. Loads from superstructure

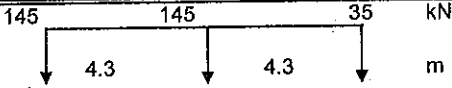
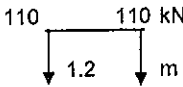
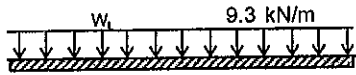
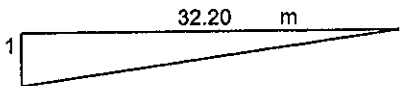
Item	Sign	Value	Unit
Span length	Lst	33.00	m
Span between bearings	Ls	32.20	m
Bridge Width	W	12.75	m
Number of girders	n_g	5.00	Girders
Girder height	Hg	1.85	m
Deck slab depth	Hd	0.227	m
Asphalt depth	H _{as}	0.08	m

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

Item	Sign	Value	Unit
1.1. Girders			
Sum of girders weight	DC	3347.93	kN
Precast Planks	DC	473.46	kN
Diaphragm	DC	380.73	kN
Total	DC	4202.11	kN
1.2. Deck slab			
Deck slab	DC	2294.12	kN
1.3. Pavement			
Asphalt concrete	DW	685.54	kN
1.4. Parapet			
Parapet + median	DC	889.35	kN

2. Live load (LL):

2.1. Live load

Truck		
Tandem		
Lane load		
Pedestrian	$W_{pd} = 0.0 \text{ kN/m}^2$	
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

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

Truck	P1(kN)	P2(kN)	P3(kN)	Sum(kN)	Total(kN)
Axle load	145	145	35		
Influence value	1.000	0.866	0.733		
Reaction	145.0	125.6	25.7	296.3	944.4
Tandem	P1(kN)	P2(kN)		Sum(kN)	Total(kN)
Axle load	110	110			
Influence value	1.000	0.963			
Reaction	110	105.9		215.9	688.2
Lane load	Wl(kN/m)				Total(kN)
Value	9.3				
Influence value	16.1				
Reaction	149.7				381.8
Pedestrian	Wdb(kN)				Total(kN)
Reaction	0.0				0.0

3. Earthquake effects on superstructure (EQ)

Force from superstructure due to EQ

$$\text{EQ} = 355 \text{ kN}$$

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

Bearing displacement due to uniform temperature and shrinkage creep

Shear modulus G

Bearing area

Height of elastomeric layers

Number of bearing

Horizontal force due to TU+SH&CR

Acting at top of bearing $H = G \cdot A \cdot \Delta u / h_r$

$$\begin{aligned} \Delta u &= 0.026 \text{ m} \\ G &= 1 \text{ MPa} \\ A &= 0.175 \text{ m}^2 \\ h_r &= 0.084 \text{ m} \\ n_b &= 5 \text{ bears} \\ H_x &= 271 \text{ kN} \end{aligned}$$

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

Transverse wind load

$$P_D = \max(0.0006V^2 \cdot C_d \cdot A_i, 1.8A_i) =$$

Longitudinal wind load

$$F_{WSL} = 0.25P_D =$$

$$\begin{aligned} \text{Zone} &= \text{III} \\ V_b &= 53.00 \text{ m/s} \\ S &= 1.09 \\ V &= 57.77 \text{ m/s} \\ C_d &= 1.40 \\ b &= 12.75 \text{ m} \\ d &= 2.94 \text{ m} \\ b/d &= 4.33 \\ A_t &= 97.09 \text{ m}^2 \\ H_y &= 272.2 \text{ kN} \\ H_x &= 68.0 \text{ kN} \end{aligned}$$

5.2. Wind load on vehicles (WL)

Transverse wind load on vehicle

Longitudinal wind load on vehicles

(At 1.8m from surface)

$$\begin{aligned} H_y &= 24.75 \text{ kN} \\ H_x &= 24.75 \text{ kN} \end{aligned}$$

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+Deck+Parapet	DC	3693	0.22			812			
Pavement	DW	343	0.22			75			
LiveLoad	LL	1326	0.22			292		0.48	630
Pedestrian	PD								
Wind on Struc.	WS			34	4.18	142	136	4.18	568
Wind on vehl.	WL			12	10.38	128	25	10.38	257
Earth quake	EQ			178	4.18	741	107	4.18	445
TU+SH&CR	TU+SH&CR			135	4.18	565			

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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+Deck+Parapet	DC	3693	0.47			1736			
Pavement	DW	343	0.47			161			
LiveLoad	LL	1326	0.47			623		0.48	630
Pedestrian	PD								
Wind on Struc.	WS			34	6.18	210	136	6.18	840
Wind on vehl.	WL			12	12.38	153	25	12.38	306
Eearth quake	EQ			178	6.18	1097	107	6.18	658
TU+SH&CR	TU+SH&CR			135	6.18	836			

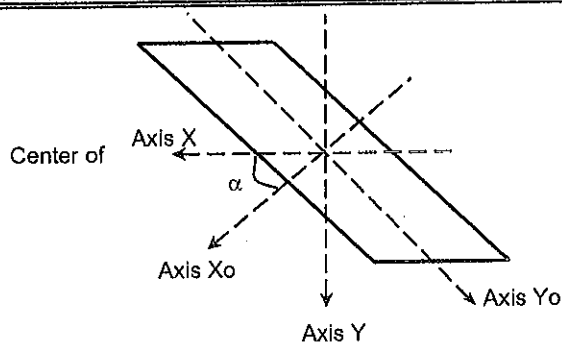
Loads	Sign	Load combinations						
		Str-IA	Str-IB	Str-III A	Str-III B	Ser-I	Ext-IA	Ext-IB
Girder+Deck+Parapet	DC	1.25	0.90	1.25	0.90	1.00	1.25	0.90
Pavement	DW	1.50	0.65	1.50	0.65	1.00	1.50	0.65
LiveLoad	LL	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Pedestrian	PD	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Trans. wind on Struc.	WS			0.40	0.40	0.30		
Trans. wind on 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	7451	68	1922	0	1102
Strength Str-IB	5867	68	1573	0	1102
Strength Str-III A	6921	94	1990	79	1334
Strength Str-III B	5337	94	1642	79	1334
Service Ser-I	5362	158	1916	66	1057
Extreme Ext-IA	5793	178	2016	107	760
Extreme Ext-IB	4209	178	1668	107	760

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	7451	68	3920	0	1102
Strength Str-IB	5867	68	3176	0	1102
Strength Str-III A	6921	94	3908	79	1493
Strength Str-III B	5337	94	3164	79	1493
Service Ser-I	5362	158	3572	66	1188
Extreme Ext-IA	5793	178	3819	107	973
Extreme Ext-IB	4209	178	3076	107	973

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LOAD COMBINATIONS



III. Load Combinations

1. Loads from substructure

Loads at bottom of pilecap	Sign	Vertical	Longitudinal		Tranversal	
		N (kN)	Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Self weight of Abutment	DC	6295		-380		337.825
Soils on pilecap	EV	3884		-6003		
Horizontal Earth Pressure	EH		2449	8196		
Vertical Surcharge	LSv	364		-637		
Horizontal Surcharge	LSH		376	1571		
Braking Force	BR		104	1046		
Centrifugal Force	CE		-	-	-	-
Buoyancy of Abutment	WA	-7		-		
Buoyancy of Earth on Abutment	WA	-		-		
Earthquake effects to Abutment	EQ		547	1275	164	382
Earthquake effects to soil	E _{AE}		2807	7946		

Loads	Sign	Load factors						
		Str-IA	Str-IB	Str-IIIA	Str-IIIB	Ser-I	Ext-IA	Ext-IB
Self weight of Abutment	DC	1.25	0.90	1.25	0.90	1.00	1.25	0.90
Soils on pilecap	EV	1.35	0.90	1.35	0.90	1.00	1.35	0.90
Horizontal Earth Pressure	EH	1.50	0.90	1.50	0.90	1.00	0.00	0.00
Vertical Surcharge	LSv	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Horizontal Surcharge	LSH	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Braking Force	BR	1.75	1.75	1.35	1.35	1.00	0.50	0.50
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	13742	4512	7181	0	422
Strength Str-IB	9791	3043	5098	0	304
Strength Str-IIIA	13596	4320	6388	0	422
Strength Str-IIIB	9645	2851	4305	0	304
Service Ser-I	10536	2928	3793	0	338
Extreme Ext-IA	13287	3594	1632	164	805
Extreme Ext-IB	9336	3594	4466	164	686

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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	7451	68	3920	0	1102
Strength Str-IB	5867	68	3176	0	1102
Strength Str-IIIA	6921	94	3908	79	1493
Strength Str-IIIB	5337	94	3164	79	1493
Service Ser-I	5362	158	3572	66	1188
Extreme Ext-IA	5793	178	3819	107	973
Extreme Ext-IB	4209	178	3075	107	973

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	21193	4580	11101	0	1525
Strength Str-IB	15658	3110	8273	0	1406
Strength Str-IIIA	20517	4414	10296	79	1915
Strength Str-IIIB	14982	2945	7469	79	1797
Service Ser-I	15898	3086	7366	66	1526
Extreme Ext-IA	19080	3771	5451	271	1778
Extreme Ext-IB	13545	3771	7541	271	1659

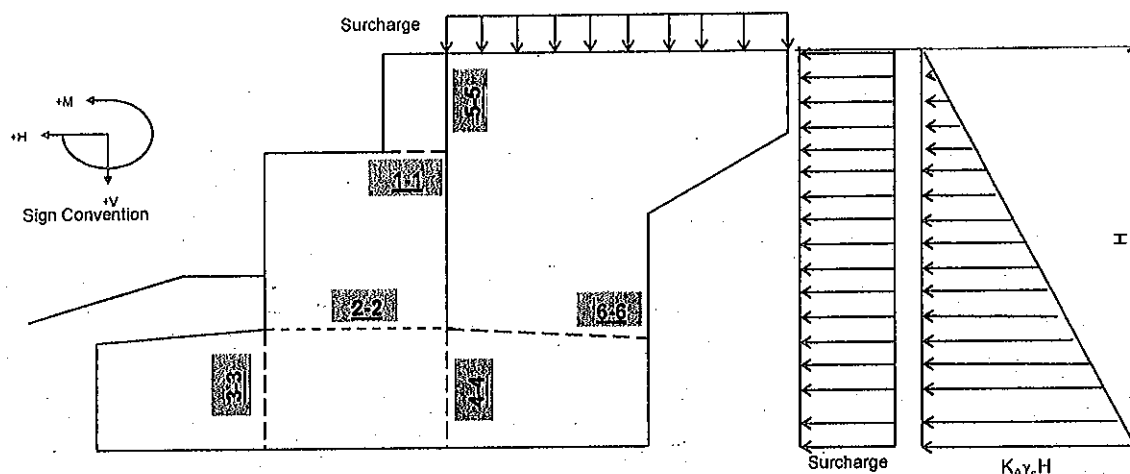
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ELEMENTS CHECKING

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

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	391		-16		
Horizontal Earth Pressure		181	165		
Surcharge (horizontal)		229	261		
Horizontal Seismic Earth Pressure		208	160		
Abutment earthquake force		34	39	10	12

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	391	410	410	0	0
Strength Str-IA	489	673	684	0	0
Strength Str-IB	352	564	591	0	0
Extreme Ext-I	489	460	389	10	12

1.2 Section 2-2

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
Live Load	LL	1.00	1.75	1.75	0.50
Braking Force	BR	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	2255		-180		
Superstructure Dead Load	3693		812		
Pavement	343		75		
Live Load	1326		292		630
Braking Force		104	839		
Horizontal Earth Pressure		1388	3499		
Surcharge (Horizontal)		328	1034		
TU+SH&CR		135	565		
Horizontal Seismic Earth Pressure		1592	3392		
Abutment earthquake force		198	509	166	598

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	7617	1956	6937	0	630
Strength Str-IA	10270	2906	10224	0	1102
Strength Str-IB	7897	2073	7839	0	1102
Extreme Ext-I	8612	2802	7584	166	913

1.3 Section 3-3

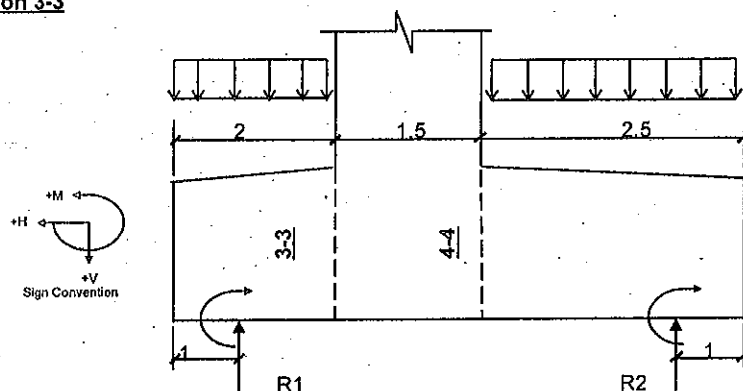


Table of Load Factors

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	Shear (kN)	Longitudinal			
		Hx (kN)	My (kN.m)		
Selfweight at front side	-1235		-1235		
Vertical soil on foot at front side	-454		-454		
Reaction of piles					
Ser-I	12003		17143		
Str-IA	16643		24245		
Str-IB	12114		17265		
Ext-I	14383		21463		

Load Combination at section 3-3					
Load combinations	N (kN)	Longitudinal			
		Hx (kN)	My (kN.m)		
Service Ser-I	10314		15455		
Strength Str-IA	14487		22089		
Strength Str-IB	10594		15745		
Extreme Ext-I	12227		19307		

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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		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Selfweight of behind side	-1791		-2357		
Vertical soil on foot at behind side	-3430		-4288		
Surcharge(Vertical)	-364		-455		
Reaction of piles					
Ser-I	3895		1987		
Str-IA	4550		1123		
Str-IB	3544		1453		
Ext-I	4698		1736		

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	-1691		-5112		
Strength Str-IA	-2957		-8407		
Strength Str-IB	-1792		-5323		
Extreme Ext-I	-2354		-7226		

1.4 Section 5-5 & 6-6

Slope of triang pressure
Uniform horizontal pressure

$$\begin{aligned} \tan \beta &= 5.55 \\ U.p &= 3.56 \text{ kN/m}^2 \end{aligned}$$

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		56	85		
Strength Str-IA		89	134		

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				131	88
Strength Str-IA				202	136

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2. Elements Checking

<S.5.7.5>

2.1. Bearing Resistance

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

w0	=	0.775 m
b0	=	0.800 m
A1	=	0.620 m ²
w	=	1.000 m
b	=	1.025 m
A2	=	1.025 m ²

Area under bearing device

Distributed width and length

Notational area

Where supporting surface is wider on all sides than loaded area

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

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

Resistance factor

Factored bearing resistance

Bearing reaction of approach bridge

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

m	=	1.286
ϕ	=	0.700
Pr	=	14230 kN
Pu	=	5486 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

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

f_n take the lesser of

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

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

f_n	=	27.00 MPa
A	=	1.025 m ²
A _g	=	0.620 m ²
A _b	=	0.620 m ²
f_{ci}	=	30 MPa
Pr	=	11719 kN

Maximum area of the portion of supporting surface

Gross area of bearing plate

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

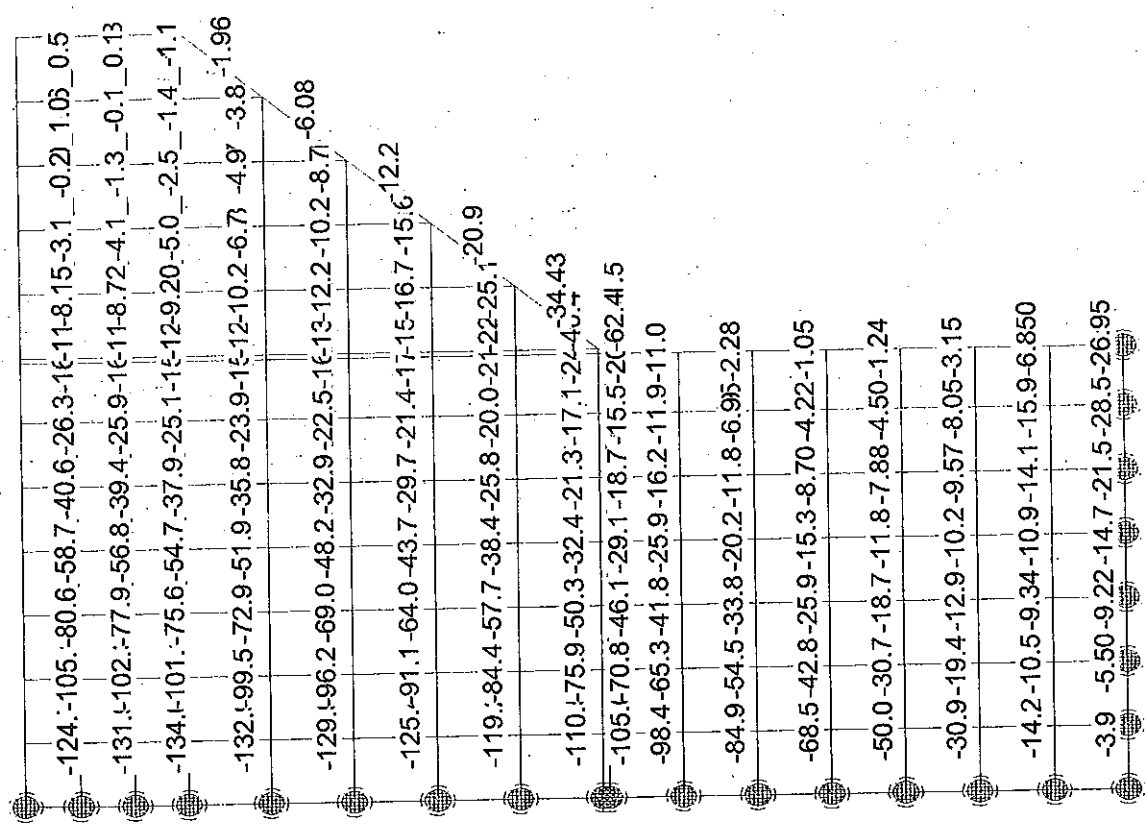
Nominal concrete strength at time of application

Factored bearing resistance

Ok

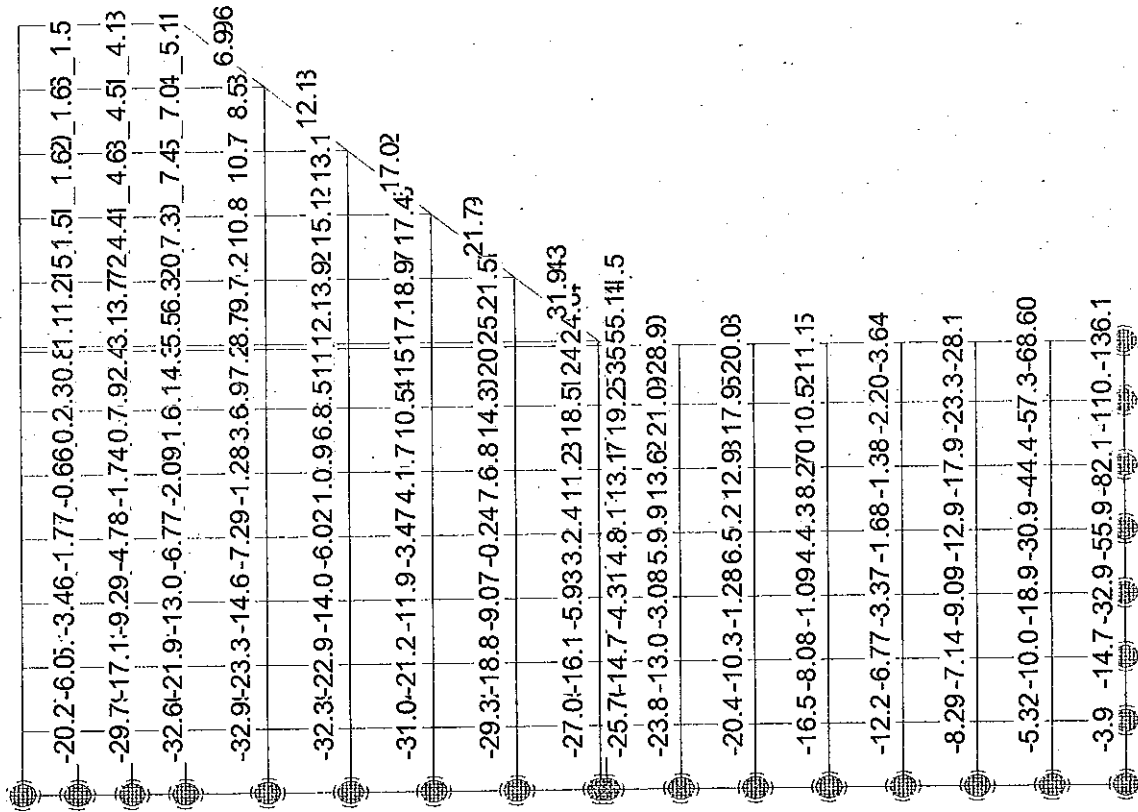
Combination X-Bending Moment(per unit width)

STR



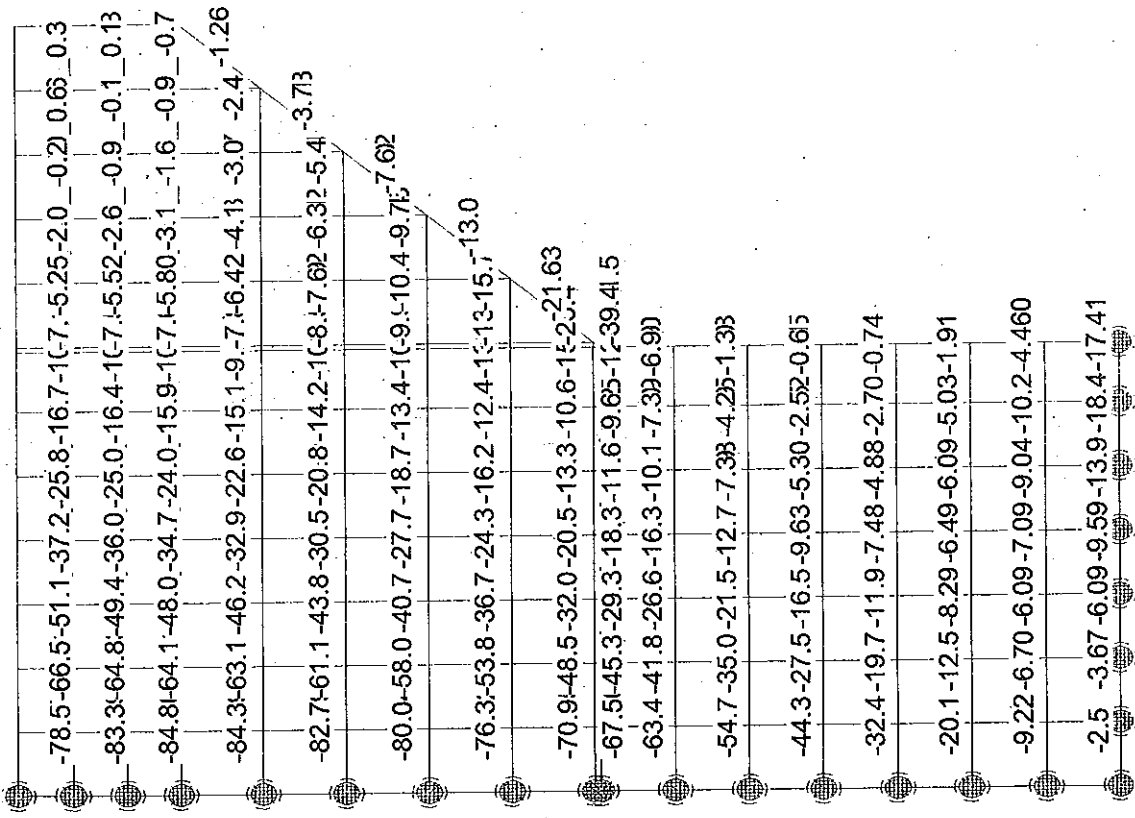
Combination Y-Bending Moment(per unit width)

STR



Combination X-Bending Moment(per unit width)

SER

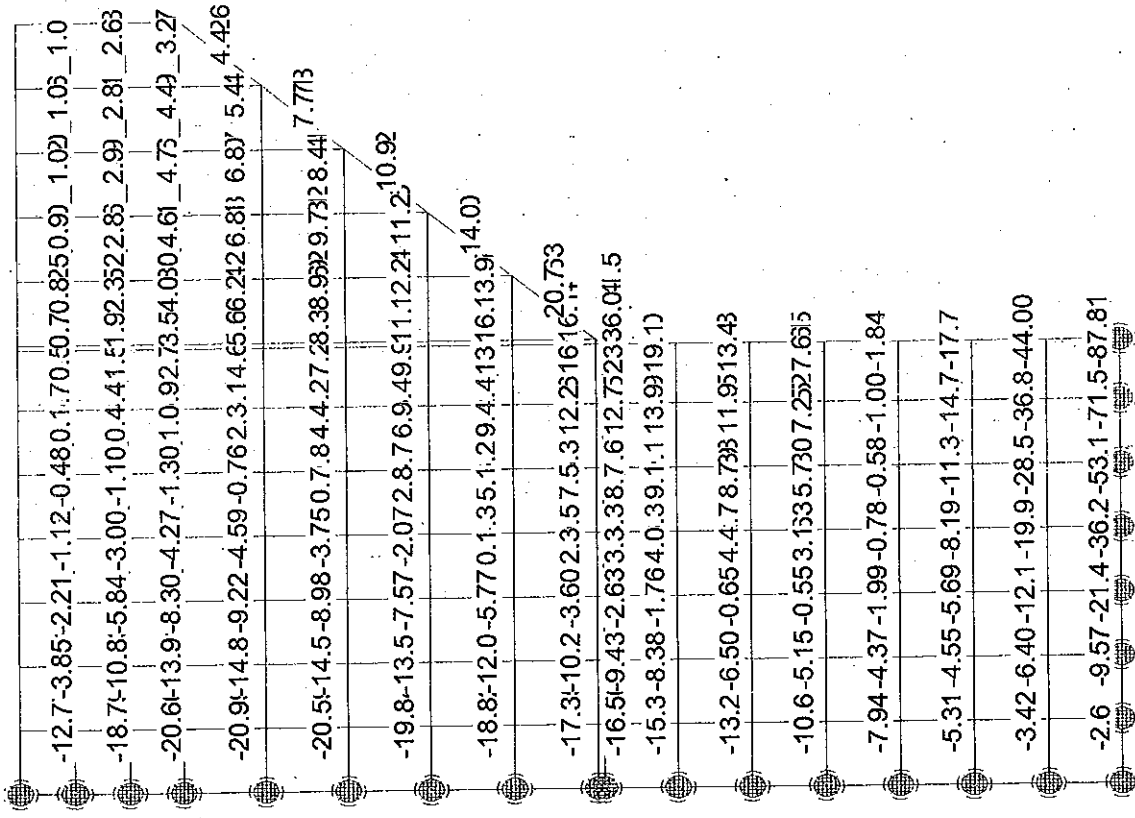


DN-QN

WWALL

Combination Y-Bending Moment(per unit width)

SER



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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	673	410	1956	2906	2802
Mu	Flexural Moment	kNm	684	410	6937	10224	7584
Nu	Axial load	kN	489	391	7617	10270	8612
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	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
As	Tension Reinforcement	Number	82	82	82	82	82
		Spacing	150	150	150	150	150
		Diameter	16	16	22	22	22
		Area	0.01656	0.01656	0.03116	0.03116	0.03116
A's	Compression Reinforcement	Number	82	82	82	82	82
		Diameter	14	14	14	14	14
		Area	0.01238	0.01238	0.01238	0.01238	0.01238
		Area	0.01238	0.01238	0.01238	0.01238	0.01238
A/c	Shear reinforcement	Number	20	20	20	20	20
		Diameter	14	14	14	14	14
		Area	0.00302	0.00302	0.00302	0.00302	0.00302
φ	Resistance factors for flexure	5.5.4.2	0.90	1.00	1.00	0.90	1.00
φv	Resistance factors for shear		0.90	1.00	1.00	0.90	1.00
φn	Resistance factors for axial force		1.00	1.00	1.00	1.00	1.00
β1	Stress block factor		0.836	0.836	0.836	0.836	0.836
c	Dis. Between centroid and top fiber	m	0.006	0.006	0.028	0.028	0.028
	For T section behavior	m	0.006	0.006	0.028	0.028	0.028
	For rectangular section behavior	m	0.006	0.006	0.028	0.028	0.028

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REINFORCEMENT CHECKING - HEAD AND STEM WALL						
f _{pe}	Effective stress in the prestressing steel after losses	Mpa	1116	1116	1116	1116
f _{ps}	Aver. stress in pres. steel at the time for which the nominal resistance	Mpa	1853	1853	1850	1850
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	0.005	0.005	0.023	0.023
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
M _n	Nominal resistance	kNm	2637	2637	17586	17586
M _r	Factored resistance	kNm	2373	2637	17586	15827
M _u	Flexural moment	kNm	684	410	6937	10224
(5.7.3.2)	Flexural moment Checking		OK	OK	OK	OK
	Limits for reinforcement					
c/de	Maximum reinforcement		0.01	0.01	0.02	0.02
	Maximum reinforcement Checking	≤ 0.42	OK	OK	OK	OK
1.2*M _{cr}	Cracking moment	kNm	1101	1101	9968	9968
(5.7.3.3.2)	Checking M _r ≥ 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	2	2	2	2
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	23000	23000	23000	23000
A	Area of concr. with same centroid as tens. Reinf	m ²	0.018	0.018	0.018	0.018
f _{sa}	Value	Mpa	227	227	225	225
0.6*f _y		Mpa	240	240	240	240
	Tensile stress in reinf Min(f _{sa} , 0.6f _y)	Mpa	227	227	225	225
x	Dist. From compression fiber to centroid	m	-	0.081	0.207	-
J.d	Arm	m	-	0.415	1.372	-
I _{cr}	Moment of inertia of the cracked section	m ⁴	-	0.017	0.371	-
f _s	Tensile stress in reinforcement f _s = M _s / (A _s *J.d)	Mpa	-	60	162	-
	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	0.00126	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
	Checking		OK	OK	OK	OK
SHEAR AND TORSION CHECKING						
β	Factor indicating diag. cracked concr. to tension		2.5	3.2	2.7	2.4
θ	Angle of inclination of diagonal compressive	degree	30.15	28.67	28.90	32.39
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
dv	Effective shear depth	m	0.439	0.439	1.429	1.429
	(de - a/2)	m	0.439	0.439	1.429	1.429
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600
n _{cat}	Amount of bars in spacing S	bars	20	20	20	20
A _v	Shear reinf area in spacing S	m ²	0.0030	0.0030	0.0030	0.0030
β	Assume		2.0	2.0	2.0	2.0
θ	Assume	degree	30.00	29.00	29.00	31.00
v	Shear stress in concrete	kN/m ²	135	53	109	179
f _{po}	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116
e _x	Strain in tensile reinforcement		5.72E-04	3.34E-04	4.51E-04	7.12E-04
	if e _x < 0, multiple with reduce factor		-	-	-	-
	Strain checking	≤ 2.00E-3	Ok	Ok	Ok	Ok
v/f _c	Ratio of shear stress and f _c		0.005	0.002	0.004	0.006
β	Final value		2.5	3.2	2.7	2.4
θ	Final value	degree	30.15	28.67	28.90	32.39
V _c	Nominal shear resistance provided by tensile stresses in the concrete	kN	6216	7953	22129	19578
V _s	Shear resistance provided by shear reinforcement	kN	1523	1618	5213	4536
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	7739	9571	27341	24115
V _{n2}	V _{n2}	kN	41523	41523	135070	135070
V _n	Nominal shear resistance V _n = min(V _{n1} , V _{n2})	kN	7739	9571	27341	24115
V _r	Factored shear resistance	kN	6965	9571	27341	21703
V _u	Shear	kN	673	410	1956	2906
(5.8.2.7)	Shear checking		OK	OK	OK	OK

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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/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	10314	14487	12227	2354	2957
Mu	Flexural Moment	kNm	15455	22089	19307	7226	8407
Nu	Axial load	kN	0	0	0	0	0
Tu	Torsional Moment	kNm	0	0	0	0	0
FLEXURAL MOMENT CHECKING							
H	Section height	m	2.000	2.000	2.000	2.000	2.000
d's	Dis. From comp. fiber to centroid of comp. Reinf	m	0.084	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
I _z	Moment of inertia of section	m4	8.400	8.400	8.400	8.400	8.400
A _{mc}	Section area	m2	25.200	25.200	25.200	25.200	25.200
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	84	84	84	84	84
		Spacing	150	150	150	150	150
		Diameter	28	28	28	20	20
		Area	0.05174	0.05174	0.05174	0.02638	0.02638
A' _s	Compression Reinforcement	Number	0	0	0	0	0
		Diameter	20	20	20	20	20
		Area	0.00000	0.00000	0.00000	0.00000	0.00000
A _{sp}	Shear reinforcement	Number	20	20	20	20	20
		Diameter	16	16	16	16	16
		Area	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
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	0.077	0.077	0.077	0.039	0.039
	For T section behavior	m	0.077	0.077	0.077	0.039	0.039
	For rectangular section behavior	m	0.077	0.077	0.077	0.039	0.039

Da Nang Quang Ngai Expressway project			Item.	Eng.	Date.	Sign.
LRB 12a BRIDGE			Design			
DETAIL DESIGN			Check			
ABUTMENT A1			Revise			
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REINFORCEMENT CHECKING - PILECAP SECTION						
fpc	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	1840	1840	1840	1850
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	0.064	0.064	0.064	0.033
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
Mn	Nominal resistance	kNm	37334	37334	37334	20041
Mr	Factored resistance	kNm	37334	33601	37334	20041
Mu	Flexural moment	kNm	15455	22089	19307	7226
(5.7.3.2)	Flexural moment Checking		OK	OK	OK	OK
	Limits for reinforcement					
c/de	Maximum reinforcement		0.04	0.04	0.04	0.02
	Maximum reinforcement Checking	≤ 0.42	OK	OK	OK	OK
1.2*Mcr	Cracking moment	kNm	18088	18088	18088	17740
(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	No	No
	Existing condition for structure	1,2 or 3	3	3	3	3
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.050	0.050	0.050	0.050
Z	Crack width parameter	N/mm	17500	17500	17500	17500
A	Area of concr. with same centroid as tens. Reinf	m ²	0.015	0.015	0.015	0.015
f _{sa}	Value	Mpa	193	193	193	193
0.6*f _y		Mpa	240	240	240	240
	Tensile stress in reinf Min(f _{sa} , 0.6f _y)	Mpa	193	193	193	193
x	Dist. From compression fiber to centroid	m	0.297	-	-	-
J.d	Arm	m	1.737	-	-	-
I _{cr}	Moment of inertia of the cracked section	m ⁴	0.968	-	-	-
f _s	Tensile stress in reinforcement $f_s = M_{sls} / (A_s * J.d)$	Mpa	172	-	-	-
	Checking for control cracking $f_s < f_{sa}$		OK	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
	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
	Checking		OK	OK	OK	OK
SHEAR AND TORSION CHECKING						
β	Factor indicating diag. cracked concr. to tension		2.0	1.8	1.9	2.3
θ	Angle of inclination of diagonal compressive	degree	39.26	42.53	41.50	35.33
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	1.804	1.804	1.804	1.900
	(de - a/2)	m	1.804	1.804	1.804	1.900
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600
ncat	Amount of bars in spacing S	bars	20	20	20	20
A _v	Shear reinf area in spacing S	m ²	0.0040	0.0040	0.0040	0.0040
θ	Assume	degree	45.00	45.00	45.00	45.00
v	Shear stress in concrete	kN/m ²	454	708	538	33
f _{po}	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116
e _x	Strain in tensile reinforcement		1.33E-03	1.88E-03	1.63E-03	9.44E-04
	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		0.015	0.024	0.018	0.001
β	Final value		2.0	1.8	1.9	2.3
θ	Final value	degree	39.26	42.53	41.50	35.33
V _c	Nominal shear resistance provided by tensile stresses in the concrete	kN	21153	18326	19554	24605
V _s	Shear resistance provided by shear reinforcement	kN	5944	5296	5491	7218
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	27097	23622	25045	31822
V _{n2}	V _{n2}	kN	170458	170458	170458	179510
V _n	Nominal shear resistance V _n = min(V _{n1} , V _{n2})	kN	27097	23622	25045	31822
V _r	Factored shear resistance	kN	27097	21260	25045	31822
V _u	Shear	kN	10314	14487	12227	2354
(5.8.2.7)	Shear checking		OK	OK	OK	OK

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LRB 12a BRIDGE			Design			
DETAIL DESIGN			Check			
ABUTMENT A1			Revise			

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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	195000
REINFORCEMENT			
fy	Yield strength	Mpa	400
Es	Modulus of Elasticity	Mpa	200000
nc	Ratio Es/Ec		7

Sign	Parameters	Unit	Sections				
			5-5	5-5	6-6	6-6	6-6
INTERNAL FORCES AT SECTION							
	Combination		Service	Strength	Service	Strength	Strength
Qu	Shear	kN	56	89	131	202	202
Mu	Flexural Moment	kNm	85	134	88	136	136
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	0	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	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000
As	Tension Reinforcement	Number	6	6	6	6	6
		Spacing	mm	150	150	150	150
		Diameter	mm	18	18	18	18
		Area	m2	0.00152	0.00152	0.00152	0.00152
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
Av	Shear reinforcement	Number	2	2	2	2	2
		Diameter	mm	12	12	12	12
		Area	m2	0.00023	0.00023	0.00023	0.00023
φ	Resistance factors for flexure	5.5.4.2	1.00	0.90	1.00	0.90	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	0.006	0.006	0.006	0.006	0.006
	For T section behavior	m	0.006	0.006	0.006	0.006	0.006
	For rectangular section behavior	m	0.006	0.006	0.006	0.006	0.006

Da Nang Quang Ngai Expressway project			Item.	Eng.	Date.	Sign.
LRB 12a BRIDGE			Design			
DETAIL DESIGN			Check			
ABUTMENT A1			Revise			
22TCN272-05; AASHTO LRFD 2nd - 1998						
REINFORCEMENT CHECKING - WING WALL						
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	1854	1854	1854	1854
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	0.005	0.005	0.005	0.005
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
Mn	Nominal resistance	kNm	240	240	240	240
Mr	Factored resistance	kNm	240	216	240	216
Mu	Flexural moment	kNm	85	134	88	136
(5.7.3.2)	Flexural moment Checking		OK	OK	OK	OK
	Limits for reinforcement					
c/de	Maximum reinforcement		0.01	0.01	0.01	0.01
	Maximum reinforcement Checking	≤ 0.42	OK	OK	OK	OK
1.2*Mcrr	Cracking moment	kNm	87	87	87	87
(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 craking 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
f _{sa}	Value	Mpa	285	285	285	285
0.6*f _y		Mpa	240	240	240	240
	Tensil stress in reinf Min($f_{sa}, 0.6f_y$)	Mpa	240	240	240	240
x	Dist. From compression fiber to centroid	m	0.087	-	0.087	-
J _d	Arm	m	0.412	-	0.412	-
I _{cr}	Moment of inertia of the cracked section	m ⁴	0.002	-	0.002	-
f _s	Tensile stress in reinforcement $f_s = M_{sl} / (A_s * J_d)$	Mpa	135	-	140	-
	Checking for control cracking $f_s < f_{sa}$		OK	N.a	OK	N.a
(5.10.8.2)	Shrinkage and temperature Reinforcement (side distribution)					
A _{req}	Area of required reinf	m ²	0.00031	0.00031	0.00031	0.00031
	Distribution on sides	7 D16	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		2.4	2.1	2.3	2.0
θ	Angle of inclination of diagonal compressive	degree	33.26	37.89	35.44	39.99
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	1.000	1.000	1.000	1.000
d _v	Effective shear depth	m	0.439	0.439	0.439	0.439
	($d_e - a/2$)	m	0.439	0.439	0.439	0.439
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600
n _{cat}	Amount of bars in spacing S	bars	2	2	2	2
A _v	Shear reinf area in spacing S	m ²	0.0002	0.0002	0.0002	0.0002
β	Assume		2.0	2.0	2.0	2.0
θ	Assume	degree	34.00	38.00	36.00	41.00
v	Shear stress in concrete	kN/m ²	128	225	299	512
f _{po}	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116
e _x	Strain in tensile reinforcement		7.72E-04	1.19E-03	9.53E-04	1.40E-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		0.004	0.008	0.010	0.017
β	Final value		2.4	2.1	2.3	2.0
θ	Final value	degree	33.26	37.89	35.44	39.99
V _c	Nominal shear resistance provided by tensile stresses in the concrete	kN	470	423	450	400
V _s	Shear resistance provided by shear reinforcement	kN	101	85	93	79
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	571	508	543	479
V _{n2}	V _{n2}	kN	3289	3289	3289	3289
V _n	Nominal shear resistance $V_n = \min(V_{n1}, V_{n2})$	kN	571	508	543	479
V _r	Factored shear resistance	kN	571	458	543	431
V _u	Shear	kN	56	89	131	202
(5.8.2.7)	Shear checking		OK	OK	OK	OK

	Da Nang Quang Ngai Expressway project LRB 12a BRIDGE DETAIL DESIGN ABUTMENT A1	Item.	Eng.	Date.	Sign.
		Design			
		Check			
		Revise			

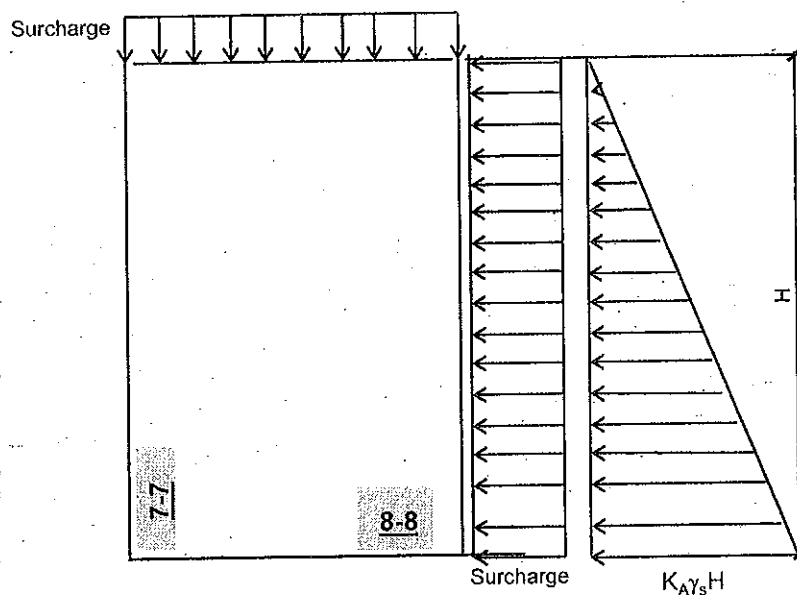
G.RETAINING WALL

1. Calculate internal force of sections

Height of retaining wall
Length of retaining wall
thickness of retaining wall

top
bottom

H	8.3
L	3.25
d1	0.51
d2	1.02



Slope of trian pressure
Uniform horizontal pressure

$\text{tg}\beta = 5.55$
 $\text{U.p} = 3.56 \text{ kN/m}^2$

Load Combination at section 7-7					
Load combinations	N (kN)	Horizontal		Vertical	
		Hx (kN)	Mx (kN.m)	Hy (kN)	My (kN.m)
Service Ser-I		125	155		
Strength Str-IA		190	237		

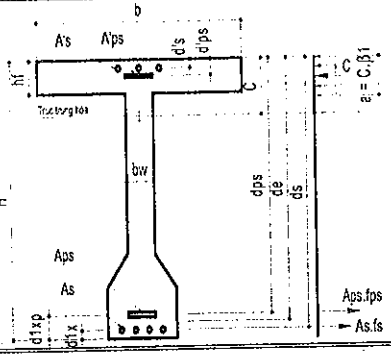
Load Combination at section 8-8					
Load combinations	N (kN)	Horizontal		Vertical	
		Hx (kN)	Mx (kN.m)	Hy (kN)	My (kN.m)
Service Ser-I				169	196
Strength Str-IA				260	302

Da Nang Quang Ngai Expressway project				Item.	Eng.	Date.	Sign.
LRB 12a BRIDGE				Design			
DETAIL DESIGN				Check			
ABUTMENT A1				Revise			

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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
γ_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



$e = C - \beta_1$

Sign	Parameters	Unit	Sections			
			7-7	7-7	6-6	6-6
INTERNAL FORCES AT SECTION						
	Combination		Service	Strength	Service	Strength
Qu	Shear	kN	125	190	169	260
Mu	Flexural Moment	kNm	155	237	196	302
Nu	Axial load	kN	0	0	0	0
Tu	Torsional Moment	kNm	0	0	0	0
FLEXURAL MOMENT CHECKING						
H	Section height	m	0.510	0.510	1.020	1.020
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	0.059	0.059	0.059	0.059
	Cover to reinf	m	0.050	0.050	0.050	0.050
ds	Dis. From comp. fiber to centroid of tension Reinf	m	0.451	0.451	0.961	0.961
d'ps	Dis. From comp. fiber to centroid of comp. prestressing steel	m	0.000	0.000	0.000	0.000
d1xp	Dis. From tens. fiber to centroid of tension prestressing steel	m	0.000	0.000	0.000	0.000
dps	Dis. From comp. fiber to centroid of tension prestressing steel	m	0.510	0.510	1.020	1.020
b	Width of the compression face of member	m	1.000	1.000	1.000	1.000
bw	Web width or diameter of a circular section	m	1.000	1.000	1.000	1.000
hf	Compression flange depth	m	0.000	0.000	0.000	0.000
Iz	Moment of inertia of section	m ⁴	0.011	0.011	0.088	0.088
Amc	Section area	m ²	0.510	0.510	1.020	1.020
Steel choice						
Aps	Tension prestressing steel	P.S type	0	0	0	0
		Number	tendons	0	0	0
		Area	m ²	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	m ²	0.00000	0.00000	0.00000
As	Tension Reinforcement	Number	bars	6	6	6
		Spacing	mm	150	150	150
		Diameter	mm	20	20	22
		Area	m ²	0.00188	0.00188	0.00228
A's	Compression Reinforcement	Number	bars	6	6	6
		Diameter	mm	16	16	16
		Area	m ²	0.00121	0.00121	0.00121
		Area	m ²	2	2	2
A'c	Shear reinforcement	Number	bars	14	14	14
		Diameter	mm	14	14	14
		Area	m ²	0.00030	0.00030	0.00030
ϕ	Resistance factors for flexure	5.5.4.2	1.00	0.90	1.00	0.90
ϕ_v	Resistance factors for shear		1.00	0.90	1.00	0.90
ϕ_n	Resistance factors for axial force		1.00	1.00	1.00	1.00
β_1	Stress block factor		0.836	0.836	0.836	0.836
c	Dis. Between centroid and top fiber	m	0.013	0.013	0.020	0.020
	For T section behavior	m	0.013	0.013	0.020	0.020
	For rectangular section behavior	m	0.013	0.013	0.020	0.020

Da Nang Quang Ngai Expressway project			Item.	Eng.	Date.	Sign.
LRB 12a BRIDGE			Design			
DETAIL DESIGN			Check			
ABUTMENT A1			Revise			
22TCN272-05; AASHTO LRFD 2nd - 1998						
REINFORCEMENT CHECKING - WING WALL						
fpe	Effective stress in the prestressing steel after losses	Mpa	1116	1116	1116	1116
fps	Aver. stress in pres. steel at the time for which the nominal resistance	Mpa	1847	1847	1850	1850
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	0.011	0.011	0.017	0.017
dc	Corresp. effective depth from extreme comp. fiber to centroid of tensile force in the tensile reinf.	m	0.451	0.451	0.961	0.961
Mn	Nominal resistance	kNm	310	310	845	845
Mr	Factored resistance	kNm	310	279	845	760
Mu	Flexural moment	kNm	155	237	196	302
(5.7.3.2)	Flexural moment Checking		OK	OK	OK	OK
	Limits for reinforcement					
c/de	Maximum reinforcement		0.03	0.03	0.02	0.02
	Maximum reinforcement Checking	≤ 0.42	OK	OK	OK	OK
1.2*Mer	Cracking moment	kNm	92	92	366	366
(5.7.3.3.2)	Checking $Mr \geq \min(1.2Mer, 1.33Mu)$		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	2	2	2	2
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	23000	23000	23000	23000
A	Area of concr. with same centroid as tens. Reinf	m ²	0.020	0.020	0.020	0.020
f _{sa}	Value	Mpa	219	219	219	219
0.6*fy		Mpa	240	240	240	240
	Tensile stress in reinf Min(f _{sa} , 0.6fy)	Mpa	219	219	219	219
x	Dist. From compression fiber to centroid	m	0.097	-	0.16	-
J.d	Arm	m	0.419	-	0.908	-
I _{cr}	Moment of inertia of the cracked section	m ⁴	0.002	-	0.012	-
fs	Tensile stress in reinforcement $fs = Msls / (As * J.d)$	Mpa	196	-	95	-
	Checking for control cracking $fs < f_{sa}$		OK	N.a	OK	N.a
(5.10.8.2)	Shrinkage and temperature Reinforcement (side distribution)					
A _{req}	Area of required reinf	m ²	0.00032	0.00032	0.00047	0.00047
	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		2.1	1.8	2.4	2.2
θ	Angle of inclination of diagonal compressive	degree	37.69	41.94	32.30	36.23
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	0.446	0.446	0.953	0.953
	(de - a/2)	m	0.446	0.446	0.953	0.953
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
A _v	Shear reinf area in spacing S	m ²	0.0003	0.0003	0.0003	0.0003
β	Assume		2.0	2.0	2.0	2.0
θ	Assume	degree	34.00	38.00	36.00	41.00
v	Shear stress in concrete	kN/m ²	280	474	177	303
f _{po}	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116
e _s	Strain in tensile reinforcement		1.17E-03	1.73E-03	7.06E-04	1.02E-03
	if $e_s < 0$, multiple with reduce factor		-	-	-	-
	Strain checking	$\leq 2.00E-3$	Ok	Ok	Ok	Ok
v/fc	Ratio of shear stress and fc		0.009	0.016	0.006	0.010
β	Final value		2.1	1.8	2.4	2.2
θ	Final value	degree	37.69	41.94	32.30	36.23
V _c	Nominal shear resistance provided by tensile stresses in the concrete	kN	433	373	1037	960
V _s	Shear resistance provided by shear reinforcement	kN	116	100	303	262
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	549	473	1340	1222
V _{n2}	Vn2	kN	3343	3343	7145	7145
V _n	Nominal shear resistance $V_n = \min(V_{n1}, V_{n2})$	kN	549	473	1340	1222
V _r	Factored shear resistance	kN	549	426	1340	1100
V _u	Shear	kN	125	190	169	260
(5.8.2.7)	Shear checking		OK	OK	OK	OK

	Da Nang Quang Ngai Expressway project LRB 12a BRIDGE DETAIL DESIGN ABUTMENT A1	Item.	Eng.	Date.	Sign.
		Design			
		Check			
		Revise			

F.BORED PILE DESIGN

I. BORED PILE DATA

1. Load Combinations at top of bored pile

STT
 Comb
 Axial force P (KN)
 Moment (KN.m)
 Mx
 My
 Mxy

STRENGTH LIMIT STATES

1	P_min	1083	34.0	1220.1	1221
2	P_max	4228	2.7	1900.3	1900
3	Mx_max	4228	2.7	1900.3	1900
4	My_max	4094	2.7	1900.4	1900

EXTREME EVENT LIMIT STATES

1	P_min	343	172	1731	1740
2	P_max	3744	164	1780	1788
3	Mx_max	343	172	1731	1740
4	My_max	3744	164	1780	1788

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

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	0.019	m ²
11ayers x 24 = 24 bars D32 @150	As1		0.019	m ²

	Da Nang Quang Ngai Expressway project LRB 12a BRIDGE DETAIL DESIGN ABUTMENT A1	Item.	Eng.	Date.	Sign.
		Design			
		Check			
		Revise			

2. Iteration diagram M-P

Using Pca-Column software

**Flexural check by pcaColumn

Strength and Service limit states:

Resistance factor:	Compression	ϕ_c	=	0.75 (AASHTO LRFD-2004)
	Tension	ϕ_t	=	0.90

Extreme Event limit states:

Resistance factor	Compression	ϕ_c	=	1.00
	Tension	ϕ_t	=	1.00

<Result table>

STT	Comb	Pu kN	Mux kN-m	Muy kN-m	ϕ Mnx kN-m	ϕ Mny kN-m	ϕ Mn/Mu
STRENGTH LIMIT STATES							
1	P_max	1082.7	34	1220	71	2559	2.1
2	P_min	4228	2.7	1900	4	2689	1.4
3	Mx_max	4228	2.7	1900	4	2689	1.4
4	My_max	4093.6	2.7	1900	4	2695	1.4
EXTREME EVENT LIMIT STATES							
1	P_max	343	172	1731	262	2636	1.5
2	P_min	3744	164	1780	301	3268	1.8
3	Mx_max	343	172	1731	262	2636	1.5
4	My_max	3744	164	1780	301	3268	1.8

3. Column Ties

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	0.622	m2
Tie diameter	Dtie	16	mm2
Cross section area of 1 tie	As-tr	0.00020	m2
Spacing of hoops	s	75	mm
Length of 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	0.0120	
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
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	0.0090	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	608	kN
Required shear capacity $V_n = Vu / \phi_v$	Vn	608	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
$V_c = 0.083 \cdot \beta \cdot \sqrt{fc} \cdot b_v \cdot dv$	Vc	616	kN
Conclude			OK

SPACE PILE FOUNDATION ANALYSIS PROGRAM
Turbo BASIC

PROJECT: : DN-QN-A1-LRB12a

INITIA DATA

Kn = 0.13 Ax = 6.00 By = 12.60 Cz = 2.00
E v.uon = 3001028 E r.uon = 3001028 E v.nen = 3001028 E r.nen = 3001028
Mq = 75 (t/m4) Md = 0 (t/m4) m = 300 (t/m4)

LOAD COMBINATIONS

COMB.	Hx	Hy	P	Mx	My	Mz
1	466.82	0.00	2160.34	-155.42	1131.58	0.00
2	317.05	0.00	1596.12	-143.37	843.37	0.00
3	449.93	8.07	2091.42	-195.22	1049.57	0.00
4	300.16	8.07	1527.20	-183.17	761.36	0.00
5	314.58	6.68	1620.54	-155.57	750.84	0.00
6	429.26	38.15	1944.99	-221.80	731.29	0.00
7	429.26	38.15	1380.77	-209.75	944.34	0.00

PROPERTIES OF PILES

PILE	Lo	H	Bpx	Bpy	A	B	Cday	Fo	Io	Po	Co	Ct
1	0.00	7.00	1.429	1.429	1.00	0.000	0.000	0.785	0.098	500	30000	15000
2						n t						
3						n t						
4						n t						
5						n t						
6						n t						
7						n t						

PILE COORD.

PILE	X	Y	Phi	Xi
1	2.00	5.30	0.000	0.00
2	2.00	1.80	0.000	0.00
3	2.00	-1.80	0.000	0.00
4	2.00	-5.30	0.000	0.00
5	-2.00	-5.30	0.000	0.00
6	-2.00	0.00	0.000	0.00
7	-2.00	5.30	0.000	0.00

DISPLACEMENTS

COMB.	X	Y	Z	Fix	Fiy	Fiz
1	0.01691	0.00003	0.002420	-0.000011	0.000563	-0.000000
2	0.01152	0.00003	0.001794	-0.000010	0.000394	-0.000000
3	0.01626	0.00032	0.002347	-0.000015	0.000531	0.000003
4	0.01087	0.00032	0.001721	-0.000015	0.000361	0.000003
5	0.01134	0.00026	0.001832	-0.000012	0.000363	0.000003
6	0.01528	0.00136	0.002200	-0.000024	0.000433	0.000014
7	0.01562	0.00136	0.001495	-0.000023	0.000540	0.000014

FORCES ON PILES

PILE	COMB.	N	Q2	Q3	M1	M2	M3
1	1	430.99	-62.02	0.01	0.000	0.272	193.714
	2	315.03	-42.11	0.00	0.000	0.251	131.268
	3	417.28	-59.86	-1.09	0.042	-3.272	187.267
	4	301.32	-39.95	-1.09	0.042	-3.293	124.821
	5	313.68	-41.87	-0.90	0.035	-2.721	131.179
	6	381.70	-57.43	-5.17	0.201	-16.689	181.487
	7	322.66	-57.33	-5.17	0.201	-16.710	178.588
2	1	426.46	-62.02	0.01	0.000	0.272	193.715
	2	310.86	-42.11	0.00	0.000	0.251	131.269
	3	410.84	-59.81	-1.09	0.042	-3.272	187.120
	4	295.24	-39.90	-1.09	0.042	-3.293	124.674
	5	308.53	-41.83	-0.90	0.035	-2.721	131.057
	6	371.68	-57.23	-5.17	0.201	-16.689	180.789
	7	312.99	-57.12	-5.17	0.201	-16.710	177.890
3	1	421.82	-62.02	0.01	0.000	0.272	193.715
	2	306.57	-42.11	0.00	0.000	0.251	131.269
	3	404.23	-59.77	-1.09	0.042	-3.272	186.969
	4	288.98	-39.86	-1.09	0.042	-3.293	124.523
	5	303.23	-41.79	-0.90	0.035	-2.721	130.932
	6	361.38	-57.01	-5.17	0.201	-16.689	180.072
	7	303.04	-56.90	-5.17	0.201	-16.710	177.173
4	1	417.29	-62.02	0.01	0.000	0.272	193.716
	2	302.40	-42.11	0.00	0.000	0.251	131.270
	3	397.79	-59.72	-1.09	0.042	-3.272	186.822
	4	282.90	-39.81	-1.09	0.042	-3.293	124.376
	5	298.08	-41.76	-0.90	0.035	-2.721	130.810
	6	351.36	-56.80	-5.17	0.201	-16.689	179.374
	7	293.37	-56.69	-5.17	0.201	-16.710	176.475
5	1	147.75	-62.02	0.01	0.000	0.272	193.716
	2	114.10	-42.11	0.01	0.000	0.251	131.270
	3	144.02	-59.72	-1.14	0.042	-3.440	186.822
	4	110.37	-39.81	-1.14	0.042	-3.461	124.376
	5	124.54	-41.76	-0.94	0.035	-2.860	130.810
	6	144.45	-56.80	-5.41	0.201	-17.487	179.374
	7	34.93	-56.69	-5.41	0.201	-17.508	176.475
6	1	154.59	-62.02	0.01	0.000	0.272	193.715
	2	120.42	-42.11	0.01	0.000	0.251	131.269
	3	153.76	-59.79	-1.14	0.042	-3.440	187.045
	4	119.58	-39.88	-1.14	0.042	-3.461	124.599
	5	132.34	-41.81	-0.94	0.035	-2.860	130.994
	6	159.62	-57.12	-5.41	0.201	-17.487	180.430
	7	49.57	-57.01	-5.41	0.201	-17.508	177.531
7	1	161.44	-62.02	0.01	0.000	0.272	193.714
	2	126.73	-42.11	0.01	0.000	0.251	131.268
	3	163.50	-59.86	-1.14	0.042	-3.440	187.267
	4	128.79	-39.95	-1.14	0.042	-3.461	124.821
	5	140.14	-41.87	-0.94	0.035	-2.860	131.179
	6	174.79	-57.43	-5.41	0.201	-17.487	181.487
	7	64.21	-57.33	-5.41	0.201	-17.508	178.588

SUMMARY OF FORCES

	PILE	COMB.	N	Q2	Q3	M1	M2	M3
Nmin	5	7	34.93	-56.69	-5.41	0.201	-17.508	176.475
Nmax	1	1	430.99	-62.02	0.01	0.000	0.272	193.714
Q2max	4	1	417.29	-62.02	0.01	0.000	0.272	193.716
Q3max	5	7	34.93	-56.69	-5.41	0.201	-17.508	176.475

M1max	1	7	322.66	-57.33	-5.17	0.201	-16.710	178.588
M2max	5	7	34.93	-56.69	-5.41	0.201	-17.508	176.475
M3max	4	1	417.29	-62.02	0.01	0.000	0.272	193.716

CHECKING CALCULATI
IN COMPARISON WITH INITIA LOAD MATRIX

1	466.82	0.00	2160.34	-155.42	1131.58	0.00
2	317.05	0.00	1596.12	-143.37	843.37	0.00
3	449.93	8.07	2091.42	-195.22	1049.57	0.00
4	300.16	8.07	1527.20	-183.17	761.36	0.00
5	314.58	6.68	1620.54	-155.57	750.84	0.00
6	429.26	38.15	1944.99	-221.80	731.29	0.00
7	429.26	38.15	1380.77	-209.75	944.34	0.00

Da Nang Quang Ngai Expressway project

Bridge LRB 12a

CALCULATION SHEETS

Pier P2 Left

Table of content

1. Structure Dimensions & Load Components
2. Load Combinations
3. Pier Cap analysis
4. Pier Column analysis
5. Foundation analysis
6. Annex

	DA NANG - QUANG NGAI EXPRESS WAY PROJECT Bridge LRB 12a DETAIL DESIGN Pier P2 Left	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 $A = 0.0580 \text{ g}$

Input data:

Bridge type	Simple PC I girder $L=33\text{m}$ with link slab			
Span length	Left	=	33.05	Right = 33.05 m
Girder length between bearings	Left	=	32.10	Right = 32.10 m
Bridge width	B	=	12.75	m

Level Table(at center of pier)

Top of pier cap	ThL	7.196	m
Top of pier column	TcL	4.261	m
Bottom of upper pier column	H_{topc}	4.260	m
Bottom of upper pilecap	H_{up}	-1.500	m
Bottom of pilecap	H_{bot}	-3.500	m
Skew angle	Ska	90.000	deg
Ground level	GL	2.500	m
Maximum water level (H1%)	H_{max}	3.980	m
Navigation water level (H5%)	H_{min}	0.000	m
Average Annual water level	H_{ave}	1.990	m
Local scour level (at water level H1%)	H_{sc}	0.000	m

Material unit weight

Concrete

Specify density	γ_c	=	24.5 kN/m ³
Compressive Strength (at 28 days)	f_c	=	30.0 MPa
Elastic Modulus	E_c	=	29440 Mpa

Reinforcement

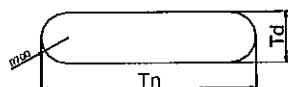
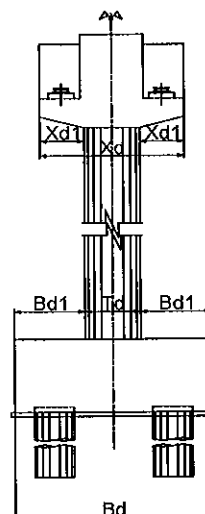
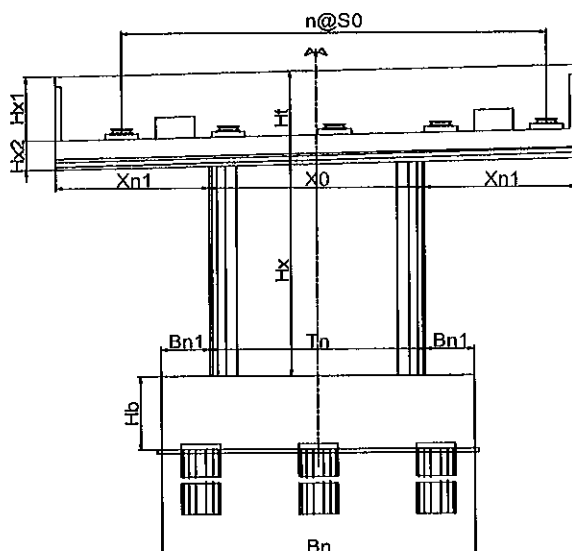
Yield strength	f_y	=	400.0 MPa
Modulus of elasticity	E_s	=	200000 Mpa
Modular ratio (steel/concrete)	$n = E_s/E_c$	=	6.7935

Asphalt concrete	γ_a	=	22.1 kN/m ³
Soil - ground	γ_s	=	17.7 kN/m ³
Saturated soil	γ_{ss}	=	7.8 kN/m ³

II. PIER DIMENSIONS

TP ĐÀ NẴNG

QUẢNG NGÃI



Pier Dimensions Table

Notation	Dimensions Transverse direction	Value (m)	Notation	Dimensions Longitudinal direction	Value (m)
*	Bearing distribution		*	Bearing pedestal	
nbear	Number of bearing	5.00		Width	0.85
nbear	Number of bearing	5.00		Length	0.65
S ₁	Bearings spacing	2.55		Height	0.15
S _r	Bearings spacing	2.55	*	Anchorage block	
b _{s1}	Total width of bridge CS	12.75		Width	0.40
b _{s2}	Carriage way width	11.76		Length	1.00
b _{s3}	Left curb width	0.50		Height	0.52
b _{s4}	Right curb width	0.49		Dist. CB's edge to exterior girder	1.27
*	Pier Cap			Dist. CB's edge to exterior girder	1.27
H _{x1}	Haunch 1 height	1.935	X _d	Pier cap width	3.60
H _{x2}	Haunch 2 height	1.00	X _{d1}	Pier cap width	1.00
H _x	Pier cap height	2.935	GL	Left bearing to pier c.line	1.270
X _{n1}	Haunch width	4.99	GR	Right bearing to pier c.line	1.270
X _{n0}	Bottom of pier cap width	2.50	H _c	Curtain wall height	1.80
X _m	Top of pier cap width	12.48	T _c	Curtain wall thickness	0.15
*	Pier Column				
T _n	Pier column width	2.50	T _d	Pier column thickness	2.50
H _n	Pier column height	5.76	Rv	Round nose radius	1.25
H	Column height	5.76			
*	Pile Cap				
B _n	Pile cap width	8.00	B _d	Pile cap length	5.00
H _b	Pile cap depth	2.00			

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	0.83	20.3						
Bearing devices		50.0						
Anchorage block	0.84	20.5						
Pier Cap	83.57	2047.4						
Curtain wall	0.50	12.1						
Upper pier column	0.00	0.0						
Pier Column	28.27	692.7						
Upper pilecap	0.00	0.0						
PileCap	80.00	1960.0						
Shear key	0.00	0.0						
Total at bottom of Column		2843.1						
Total at bottom of pilecap		4803.1						

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	140.37	2484.5						
Total at bottom of Column								
Total at bottom of pilecap		2484.5						

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	26.90	-263.9						
Upper pilecap	0.00	0.0						
PileCap	80.00	-784.8						
Shear key	0.00	0.0						
Total at bottom of Column		-263.9						
Total at bottom of pilecap		-1048.7						

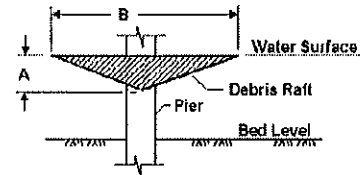
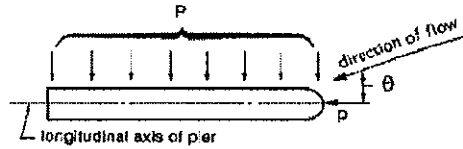
Case2: Minimum water level (Hmin)

Item	Volume (m3)	Vertical F_v (kN)	Longitudinal			Transversal		
			F_{HX} (kN)	Arm. $_{HX}$ (m)	M_y (kN•m)	F_{HY} (kN)	Arm. $_{HY}$ (m)	M_x (kN•m)
Upper pier column	0.00	0.0						
Pier Column	7.36	-72.2						
Upper pilecap	0.00	0.0						
PileCap	80.00	-784.8						
Shear key	0.00	0.0						
Total at bottom of Column		-72.2						
Total at bottom of pilecap		-857.0						

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_{HY} (kN)	Arm- HY (m)	M_x (kN•m)
Upper pier column	0.00	0.0						
Pier Column	17.13	-168.1						
Upper pilecap	0.00	0.0						
PileCap	80.00	-784.8						
Shear key	0.00	0.0						
Total at bottom of Column		-168.1						
Total at bottom of pilecap		-952.9						

4.Stream Pressure



Stream pressure data

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

Longitudinal axis of pier

$$pL = 5.14 \times 10^{-4} C_D V^2$$

Type of pier colum: "1:Semicircular - nosed pier"; "2:Square - ended pier";		1	
"3:Debris lodged against the pier "; "4:Wedged - nosed pier with nose angle 90deg or less"			
Drag coefficient	C_D	0.70	
Stream pressure at H1%	pL1%	0.44	kN/m2
Stream pressure at minimum water level	pLmin	0.05	kN/m2
Stream pressure at average annual water level	pLannual	0.44	kN/m2
Additional stream pressure due to driftwood raft lodged against pier			
Drag coefficient	C_D	0.50	
Height of debris raft	A	0.7	m
Width of debris raft	B	14.0	m
Area of debris raft	Area	5.2	m2
Stream pressure due to driftwood raft at H1%	pLdebris	0.32	kN/m2
Equivalent force	Fhdebris	1.6	kN

Lateral axis of pier

$$pT = 5.14 \times 10^{-4} C_L V^2$$

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

Case1: Maximum water level (H1%)

Item	Exposed height (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	5.8	0.0	0.0	5.8	0.0
Pier Column	5.48		0.0	2.7	0.0	6.1	2.7	16.6
Upper pilecap	0.00		0.0	0.0	0.0	0.0	0.0	0.0
Total at bottom of Column			0.0		0.0	6.1		16.6
Upper pier Column	0.00		0.0	7.8	0.0	0.0	7.8	0.0
Pier Column	5.48		0.0	4.7	0.0	6.1	4.7	28.8
Upper pilecap	0.00		0.0	2.0	0.0	0.0	2.0	0.0
PileCap	0.00		0.0	0.0	0.0	0.0	0.0	0.0
Total at bottom of pilecap			0.0		0.0	6.1		28.8

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	1.6	5.5	9.0
Total at bottom of Column					0.0	1.6		9.0
Pier Column					0.0	1.6	7.5	12.3
Total at bottom of pilecap					0.0	1.6		12.3

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	5.8	0.0	0.0	5.8	0.0
Pier Column	1.50		0.0	0.8	0.0	0.2	0.8	0.2
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	0.2		0.2
Upper pier Column	0.00		0.0	7.8	0.0	0.0	7.8	0.0
Pier Column	1.50		0.0	2.8	0.0	0.2	2.8	0.6
Upper pilecap	0.00		0.0	2.0	0.0	0.0	2.0	0.0
PileCap	0.00		0.0	0.0	0.0	0.0	0.0	0.0
Total at bottom of pilecap			0.0		0.0	0.2		0.6

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	5.8	0.0	0.0	5.8	0.0
Pier Column	3.49		0.0	1.7	0.0	3.9	1.7	6.7
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	3.9		6.7
Upper pier Column	0.00		0.0	7.8	0.0	0.0	7.8	0.0
Pier Column	3.49		0.0	3.7	0.0	3.9	3.7	14.5
Upper pilecap	0.00		0.0	2.0	0.0	0.0	2.0	0.0
PileCap	0.00		0.0	0.0	0.0	0.0	0.0	0.0
Total at bottom of pilecap			0.0		0.0	3.9		14.5

5. Wind Loads

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	3.8 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 (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)
Curtain wall	0.00		0.0	8.7	0.0	0.0	8.7	0.0
Pier Cap	2.94		0.0	7.2	0.0	27.5	7.2	198.8
Upper pier Column	0.00		0.0	5.5	0.0	0.0	5.5	0.0
Pier Column	0.28		1.8	5.6	10.2	1.8	5.6	10.2
Upper pilecap	0.00		0.0	5.5	0.0	0.0	5.5	0.0
Total at bottom of Column			1.8		10.2	29.3		209.1
Curtain wall	0.00		0.0	10.7	0.0	0.0	10.7	0.0
Pier Cap	2.94		0.0	9.2	0.0	27.5	9.2	253.8
Upper pier Column	0.00		0.0	7.5	0.0	0.0	7.5	0.0
Pier Column	0.28		1.8	7.6	13.9	1.8	7.6	13.9
Upper pilecap	0.00		0.0	7.5	0.0	0.0	7.5	0.0
PileCap	0.00		0.0	7.5	0.0	0.0	7.5	0.0
Total at bottom of pilecap			1.8		13.9	29.3		267.7

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	0.00		0.0	8.7	0.0	0.0	8.7	0.0
Pier Cap	2.94		0.0	7.2	0.0	27.5	7.2	198.8
Upper pier Column	0.00		0.0	1.5	0.0	0.0	1.5	0.0
Pier Column	4.26		27.7	3.6	100.6	27.7	3.6	100.6
Upper pilecap	0.00		0.0	1.5	0.0	0.0	1.5	0.0
Total at bottom of Column			27.7		100.6	55.2		299.5
Curtain wall	0.00		0.0	10.7	0.0	0.0	10.7	0.0
Pier Cap	2.94		0.0	9.2	0.0	27.5	9.2	253.8
Upper pier Column	0.00		0.0	3.5	0.0	0.0	3.5	0.0
Pier Column	4.26		27.7	5.6	156.1	27.7	5.6	156.1
Upper pilecap	0.00		0.0	3.5	0.0	0.0	3.5	0.0
PileCap	0.00		0.0	3.5	0.0	0.0	3.5	0.0
Total at bottom of pilecap			27.7		156.1	55.2		409.9

At average Annual water level

Item	Exposed height (m)	Vertical F_v (kN)	Longitudinal			Transversal		
			F_{HX}	Arm _{HX}	M_y	F_{HY}	Arm _{HY}	M_x
			(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Curtain wall	0.00		0.0	8.7	0.0	0.0	8.7	0.0
Pier Cap	2.94		0.0	7.2	0.0	27.5	7.2	198.8
Upper pier Column	0.00		0.0	3.5	0.0	0.0	3.5	0.0
Pier Column	2.27		14.8	4.6	68.3	14.8	4.6	68.3
Upper pilecap	0.00		0.0	3.5	0.0	0.0	3.5	0.0
Total at bottom of Column			14.8		68.3	42.3		267.1
Curtain wall	0.00		0.0	10.7	0.0	0.0	10.7	0.0
Pier Cap	2.94		0.0	9.2	0.0	27.5	9.2	253.8
Upper pier Column	0.00		0.0	5.5	0.0	0.0	5.5	0.0
Pier Column	2.27		14.8	6.6	97.9	14.8	6.6	97.9
Upper pilecap	0.00		0.0	5.5	0.0	0.0	5.5	0.0
PileCap	0.00		0.0	5.5	0.0	0.0	5.5	0.0
Total at bottom of pilecap			14.8		97.9	42.3		351.7

6. Vessel Collision

7. Vehicular Collision Force

"1:yes"; "0:no"

0

IV. SUPERSTRUCTURE LOADS

I. Dead Loads

Left side Span

Item	Volume	Vertical F _V	Longitudinal			Transversal		
			F _{HX}	Arm _{HX}	M _y	F _{HY}	Arm _{HY}	M _x
	(m3)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Stage1 (DC)								
Girders	136.65	1674.0		1.270	-2125.9			
Diaphragm	17.13	209.9		1.270	-266.5			
Precast plank	16.89	206.9		1.270	-262.7			
Deck slab	103.68	1270.0		1.270	-1612.9			
Total		3360.7			-4268.1			
Stage2 (DW)								
Pavement	35.98	397.6		1.270	-504.9			
Parapet + railing		391.1		1.270	-496.6			
Lighting post + mis.		33.0		1.270	-41.9			
Total		821.6			-1043.5			

Right side Span

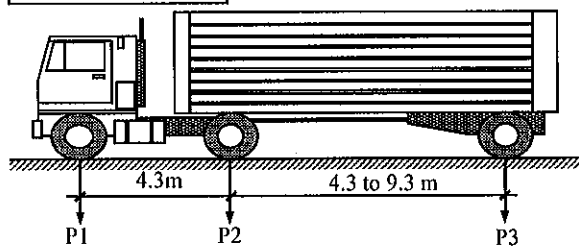
Item	Volume	Vertical F _V	Longitudinal			Transversal		
			F _{HX}	Arm _{·HX}	M _y	F _{HY}	Arm _{·HY}	M _x
	(m3)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Stage1 (DC)								
Girders	136.65	1674.0		1.270	2125.9			
Diaphragm	17.13	209.9		1.270	266.5			
Precast plank	16.89	206.9		1.270	262.7			
Deck slab	103.68	1270.0		1.270	1612.9			
Total		3360.7			4268.1			
Stage2 (DW)								
Pavement	35.98	397.6		1.270	504.9			
Parapet + railing		391.1		1.270	496.6			
Lighting post + mis.		33.0		1.270	41.9			
Total		821.6			1043.5			

2. Live Load

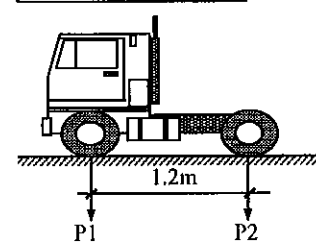
Live load data

Design Truck	P1	35.0kN	V1	4.3 m
	P2	145.0kN	V2	4.3 m
	P3	145.0kN		
Design Tandem	P1	110.0kN	V3	1.2 m
	P2	110.0kN		
Design Lane Load	P _L	9.3 kN/m		
Pedestrian Load	P _p	0.0		
Sidewalk width - both 2 sides	sw	0.0		
Maximum number of design lane	n _{lanes}	3.0		
Multiple presence factor	m	0.85		
Dynamic load allowance		(1+IM)		
Deck joint - all limit states		1.75		
Other structure - all limit states (except fatigue)		1.25		

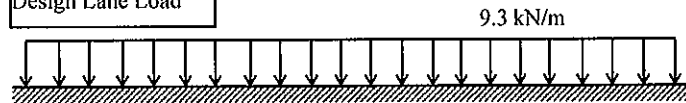
Design Truck



Design Tandem

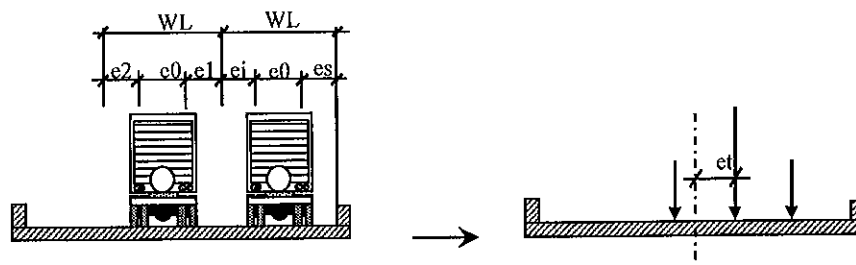


Design Lane Load



Transverse Distribution

Distance from wheel axis to inner face of curb and between wheel axis				
In general case	e _i	1.20 m	e _s	0.60 m
For deck overhang design	e _i	1.30 m	e _s	0.50 m
Distance between wheels			e ₀	1.80 m
Design lane width			WL	3.60 m
			e ₁	0.00 m
			e ₂	1.80 m
Curb width			w _c	0.50 m
Transverse eccentricity of design vehicle 1 - general case			e _{x1}	4.38
Transverse eccentricity of design vehicle 2			e _{x2}	2.58
Transverse eccentricity of design vehicle 3			e _{x3}	0.78
Transverse eccentricity of design vehicle 4			e _{x4}	-1.03
Transverse Eccentricity of design vehicle			e _t	1.68 m



Longitudinal Distribution

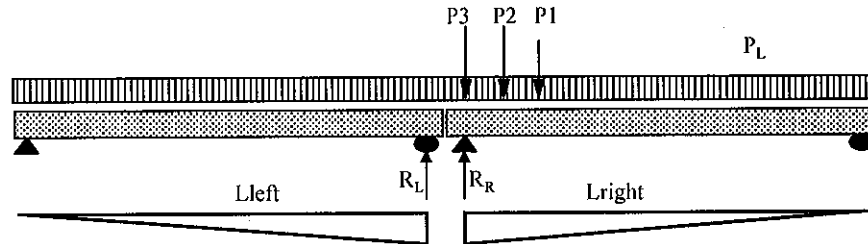
Case1a: 1Truck + Lane load on 1 span
Case1b: 1Truck + Lane load on 2 spans
Case2a: 2Trucks + Lane load on 1 span
Case2b: 2Trucks + Lane load on 2 spans
Case3a: 1Tandem + Lane load on 1 span
Case3b: 1Tandem + Lane load on 2 spans

Influence line value

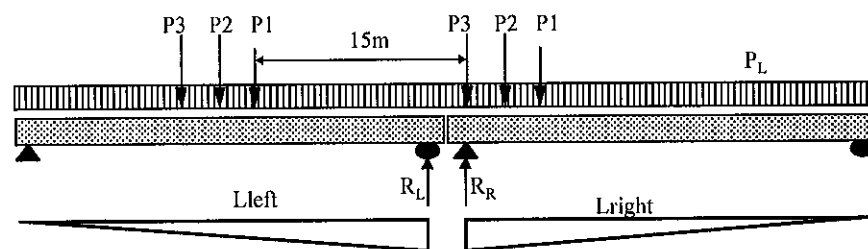
Case	Left span				Right span			
	P3	P2	P1	Aleft	P3	P2	P1	Aright
Case1a:				16.05	1.00	0.87	0.73	16.05
Case1b:	0.95			16.05		1.00	0.87	16.05
Case2a:*	1.00	0.87	0.73	16.05	0.26	0.13	0.00	16.05
Case2b:	0.34	0.48	0.61	16.05	1.00	0.87	0.73	16.05
Case3a:				16.05		1.00	0.96	16.05
Case3b:		1.04		16.05			1.00	16.05

* 2 Trucks in right span

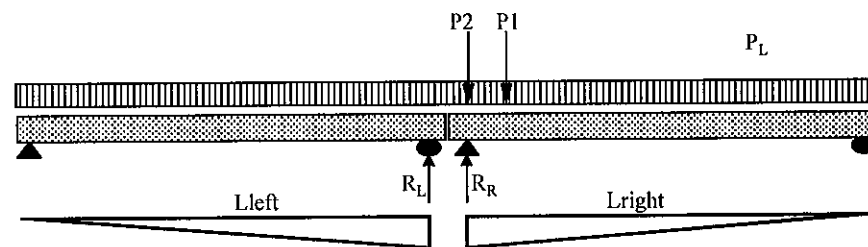
Truck + Lane load



2 Trucks + Lane load



Tandem + Lane load



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

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

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)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Case1a:	179.1	623.4	802.5			564.3			3507.1
Case1b:	384.7	442.1	826.8			72.9			3613.0
Case2a:	161.2	638.5	799.7			606.2			3494.8
Case2b:	351.0	561.1	912.1			266.8			3985.7
Case3a:	179.1	502.9	682.1			411.3			2980.6
Case3b:	351.0	344.1	695.1			-8.7			3037.7

2 Loaded Lane			m = 1.00						
Item	Reaction	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:	298.5	1039.0	1337.6			940.4			3838.8
Case1b:	641.2	736.8	1378.0			121.5			3954.8
Case2a:	268.7	1064.2	1332.9			1010.3			3825.4
Case2b:	585.0	935.1	1520.1			444.7			4362.7
Case3a:	298.5	838.2	1136.8			685.4			3262.6
Case3b:	585.0	573.5	1158.5			-14.6			3325.0

3 Loaded Lane			m = 0.85						
Item	Reaction	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:	380.6	1324.8	1705.4			1199.1			2336.4
Case1b:	817.5	939.4	1756.9			154.9			2407.0
Case2a:	342.6	1356.9	1699.4			1288.2			2328.2
Case2b:	745.8	1192.3	1938.1			567.0			2655.2
Case3a:	380.6	1068.8	1449.4			873.9			1985.7
Case3b:	745.9	731.3	1477.1			-18.6			2023.7

4 Loaded Lane			m = 0.65						
Item	Reaction	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:	388.1	1350.7	1738.8			1222.6			54.3
Case1b:	833.5	957.9	1791.4			157.9			56.0
Case2a:	349.3	1383.5	1732.7			1313.4			54.1
Case2b:	760.5	1215.7	1976.1			578.1			61.8
Case3a:	388.1	1089.7	1477.8			891.1			46.2
Case3b:	760.5	745.6	1506.1			-19.0			47.1

Item Live Load	Vertical F _V (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	1520.1	0.0	0.0	444.7	0.0	0.0	4362.7
Total at bottom of pilecap	1520.1	0.0	0.0	444.7	0.0	0.0	4362.7

Pedestrian Load									
Item	Reaction	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	10.807	0.0
Total at bottom of pilecap						0.0	12.807	0.0

4.Braking Force

Braking force data			
Axle weights of design Truck	P	325.0	kN
Number of loaded lanes	n	3.0	lanes
	m	0.85	
Br1 = 25%*(design truck)*n*m	Br1	207.19	kN
Br2 = 5%*(design truck + 9.3*Lbridge)*n*m	Br2	119.70	kN
Br = max(Br1, Br2)	Br	207.19	kN

Item	From surface (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	10.807	1119.5			
Total at bottom of pilecap			103.6	12.807	1326.7			

5.Uniform Temperature

Uniform temperature data			
Installing temperature	t0	27.0	deg
Maximum temperature	tmax	47.0	deg
Minimum temperature	tmin	10.00	deg
Plus temperature amplitude	Δtmax	20.0	deg
Minus temperature amplitude	Δtmin	17.0	deg
Coefficient of Thermal Expansion	α	1.08E-05	
Strain due to minus temperature	ε _T	1.84E-04	

Item	From surface (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	8.85	1.7			
Total at bottom of pilecap			0.3	10.85	2.2			

6.Creep & Shrinkage

Creep & shrinkage data

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			0.4	8.85	2.9			
Total at bottom of pilecap			0.4	10.85	3.8			

7. Wind on Structure

Wind loads data		$P_D = 0.0006 V^2 C_d \geq 1.8 \text{ (kN/m}^2\text{)}$	
Average elevation of deck girder upper ground or water plane level	Hele_g	6.8	m
Correct coefficient for wind zone and elevation of pier	S	1.09	
Design wind speed $V = S \cdot V_b$	V	57.8	m/s
Overall width between handrails	b	12.8	m
Superstructure height including solid parapet	d	3.03	m
	b/d	4.21	
Obstacle coefficient for pier	Cd	1.36	
Wind pressure on pier	P _D	2.72	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		68.1	10.2	695.4	272.4	10.2	2781.5
Total at bottom of Column			68.1		695.4	272.4		2781.5
Superstructure	3.03		68.1	12.2	831.6	272.4	12.2	3326.4
Total at bottom of pilecap			68.1		831.6	272.4		3326.4

8. Wind on Vehicle

For strength combination III, wind load on vehicular and on structure have to simultaneously consider (wind speed 25m/s). Wind load on vehicular in 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		24.8	12.6	312.5	49.6	12.6	625.0
Total at bottom of Column		24.8		312.5	49.6		625.0
Superstructure		24.8	14.6	362.1	49.6	14.6	724.1
Total at bottom of pilecap		24.8		362.1	49.6		724.1

9. Earth Quake

Earth Quake data		
Acceleration coefficient	A	0.0580 g
Seismic zone	Sz	1
Soil profile type: according to geological data survey		1
Coeffient site	S	1.00
Bridge importance category: "1:critical"; "2:essential"; "3:other"	IC	2 essential
Response Modification Factor		
Column		2.0
Connection		1.0
Foundation		1.0

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

Item	Vertical F _V (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		604		4060	846		5910
Total at bottom of pilecap		604		5268	846		7603

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		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 _{Hv} (kN)	Mx (kN•m)
Superstructure Loads						
1.Stage1 - Dead Load	DC	6721		0		
2.Stage2 - Pave.+Parapet+Railing+Mis.	DW	1643		0		
3.Live Load	LL	1520		445		4363
4.Pedestrian	LL	0		0		0
5.Centrifugal force	CE				0	0
6.Braking force	BR		104	1120		
7.Uniform temperature	TU		0	2		
8.Creep and Shrinkage	CR&SH		0	3		
9.Wind pressure on superstructure	WS		68	695	272	2782
10.Wind pressure on vehicles	WL		25	312	50	625
11.Earthquake						
a - Longitudinal direction	EQ		302	2030		
b - Transverse direction	EQ				423	2955
Substructure Loads						
1.Pier selfweight	DC	2843				
2.Soil on pile cap	EV	0				
3.Bouyancy on pier						
a - Maximum water level	WA	-264				
b - Minimum water level	WA	-72				
c - Average annual water level	WA	-168				
4.Stream pressure						
a - Maximum water level	WA		0	0	8	26
b - Minimum water level	WA		0	0	0	0
c - Average annual water level	WA		0	0	4	7
5.Wind pressure						
a - Maximum water level	WS		2	10	29	209
b - Minimum water level	WS		28	101	55	299
c - Average annual water level	WS		15	68	42	267
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

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	Bridge LRB 12a	Design	-		
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Loads at Bottom of Pilecap

Loads	Sign	F _v (kN)	Longitudinal		Transvesal	
			F _{lx} (kN)	My (kN•m)	F _{ly} (kN)	Mx (kN•m)
Superstructure Loads						
1.Stage1 - Dead Load	DC	6721		0		
2.Stage2 - Pave.+Parapet+Railing+Mis.	DW	1643		0		
3.Live Load	LL	1520		445		4363
4.Pedestrian	LL	0		0		0
5.Centrifugal force	CE				0	0
6.Braking force	BR		104	1327		
7.Uniform temperature	TU		0	2		
8.Creep and Shrinkage	CR&SH		0	4		
9.Wind pressure on superstructure	WS		68	832	272	3326
10.Wind pressure on vehicles	WL		25	362	50	724
11.Earthquake						
a - Longitudinal direction	EQ		604	5268		
b - Transverse direction	EQ				846	7603
Substructure Loads						
1.Pier selfweight	DC	4803				
2.Soil on pile cap	EV	2484				
3.Bouyancy on pier						
a - Maximum water level	WA	-1049				
b - Minimum water level	WA	-857				
c - Average annual water level	WA	-953				
4.Stream pressure						
a - Maximum water level	WA		0	0	8	38
b - Minimum water level	WA		0	0	0	1
c - Average annual water level	WA		0	0	4	14
5.Wind pressure						
a - Maximum water level	WS		2	14	29	268
b - Minimum water level	WS		28	156	55	410
c - Average annual water level	WS		15	98	42	352
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

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		Check	-		
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Load Factors and Load Combinations

Loads	Sign	Str1a 2	Str1b 3	Str2a 4	Str2b 5	Str3a 6	Str3b 7
Superstructure Loads							
1.Stage1 - Dead Load	DC	1.25	0.90	1.25	0.90	1.25	0.90
2.Stage2 - Pave.+Mis.	DW	1.50	0.65	1.50	0.65	1.50	0.65
3.Live Load	LL	1.75	1.75	-	-	1.35	1.35
4.Pedestrian	LL	1.75	1.75	-	-	1.35	1.35
5.Centrifugal force	CE	1.75	1.75	-	-	1.35	1.35
6.Braking force	BR	1.75	1.75	-	-	1.35	1.35
7.Uniform temperature	TU	0.50	0.50	0.50	0.50	0.50	0.50
8.Creep and Shrinkage	CR&SH	0.50	0.50	0.50	0.50	0.50	0.50
9.Wind pressure on superst.	WS	-	-	1.40	1.40	0.40	0.40
10.Wind pressure on vehicles	WL	-	-	-	-	1.00	1.00
11.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	-	-	-	-	-	-

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		Design	-		
	DETAIL DESIGN	Check	-		
		Revise	-		

Load Factors and Load Combinations

Loads	Sign	Ser1 10	Ext1a 11	Ext1b 12	Ext1c 13	Ext1d 14	
Superstructure Loads							
1.Stage1 - Dead Load	DC	1.00	1.25	0.90	1.25	0.90	
2.Stage2 - Pave.+Mis.	DW	1.00	1.50	0.65	1.50	0.65	
3.Live Load	LL	1.00	0.50	0.50	0.50	0.50	
4.Pedestrian	LL	1.00	0.50	0.50	0.50	0.50	
5.Centrifugal force	CE	1.00	0.50	0.50	0.50	0.50	
6.Braking force	BR	1.00	0.50	0.50	0.50	0.50	
7.Uniform temperature	TU	1.00	-	-	-	-	
8.Creep and Shrinkage	CR&SH	1.00	-	-	-	-	
9.Wind pressure on superst.	WS	0.30	-	-	-	-	
10.Wind pressure on vehicles	WL	1.00	-	-	-	-	
11.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	-	-	-	-	-	

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		Design	-		
		Check	-		
		Revise	-		

Load Factors and Load Combinations

Loads	Sign	Ext2a 15	Ext2b 16	Ext2c 17	Ext2d 18		
Superstructure Loads							
1.Stage1 - Dead Load	DC	1.25	0.90	1.25	0.90		
2.Stage2 - Pave.+Mis.	DW	1.50	0.65	1.50	0.65		
3.Live Load	LL	0.50	0.50	0.50	0.50		
4.Pedestrian	LL	0.50	0.50	0.50	0.50		
5.Centrifugal force	CE	0.50	0.50	0.50	0.50		
6.Braking force	BR	0.50	0.50	0.50	0.50		
7.Uniform temperature	TU	-	-	-	-		
8.Creep and Shrinkage	CR&SH	-	-	-	-		
9.Wind pressure on superst.	WS	-	-	-	-		
10.Wind pressure on vehicles	WL	-	-	-	-		
11.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		

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		Design	-		
		Check	-		
		Revise	-		

II. LOAD COMBINATIONS RESULT

Load Combinations at Bottom of PierColumn

No	Combinations	Sign	F _V (kN)	Longitudinal		Transvesal	
				F _{HX} (kN)	My (kN•m)	F _{HY} (kN)	Mx (kN•m)
1	Strength 1a	Str1a	17009	182	2740	0	7635
2	Strength 1b	Str1b	12072	182	2740	8	7660
3	Strength 2a	Str2a	14348	135	1117	459	4314
4	Strength 2b	Str2b	9412	98	990	430	4212
5	Strength 3a	Str3a	16400	203	2745	181	7747
6	Strength 3b	Str3b	11464	193	2709	178	7736
7	Service 1	Ser1	12656	158	2120	148	5912
8	Extreme 1a EQL	Ext1a	15013	354	2812	131	3075
9	Extreme 1b EQL	Ext1b	10268	354	2812	131	3075
10	Extreme 1c EQT	Ext1c	15013	142	1391	427	5143
11	Extreme 1d EQT	Ext1d	10268	142	1391	427	5143

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	22028	182	3103	0	7635
2	Strength 1b	Str1b	15288	182	3103	8	7672
3	Strength 2a	Str2a	19368	135	1386	459	5231
4	Strength 2b	Str2b	12628	98	1187	430	5070
5	Strength 3a	Str3a	21420	203	3152	181	8109
6	Strength 3b	Str3b	14680	193	3095	178	8089
7	Service 1	Ser1	16315	158	2436	148	6208
8	Extreme 1a EQL	Ext1a	20032	656	6153	258	4477
9	Extreme 1b EQL	Ext1b	13483	656	6153	258	4477
10	Extreme 1c EQT	Ext1c	20032	233	2466	850	9799
11	Extreme 1d EQT	Ext1d	13483	233	2466	850	9799

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		Design	-		
		Check	-		
		Revise	-		

D.COLUMN DESIGN

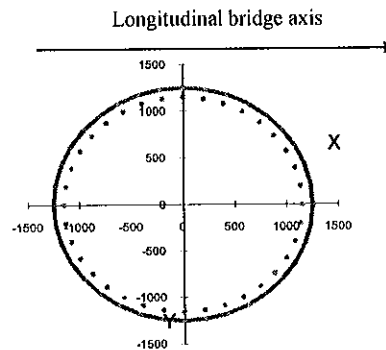
I. COLUMN DATA

1.Load Combinations at Bottom of Pier Column

No	Combinations	Sign	F _v (kN)	Longitudinal		Transvesal		
				F _{Hx} (kN)	My (kN•m)	F _{Hv} (kN)	Mx (kN•m)	
1	Strength 1a	Str1a	17009	182	2740	0	7635	
2	Strength 1b	Str1b	12072	182	2740	8	7660	
3	Strength 2a	Str2a	14348	135	1117	459	4314	
4	Strength 2b	Str2b	9412	98	990	430	4212	
5	Strength 3a	Str3a	16400	203	2745	181	7747	
6	Strength 3b	Str3b	11464	193	2709	178	7736	
7	Service I	SerI	12656	158	2120	148	5912	
8	Extreme 1a	EQL	Ext1a	15013	354	2812	131	3075
9	Extreme 1b	EQL	Ext1b	10268	354	2812	131	3075
10	Extreme 1c	EQT	Ext1c	15013	142	1391	427	5143
11	Extreme 1d	EQT	Ext1d	10268	142	1391	427	5143

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



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	Bridge LRB 12a	Design	-		
	DETAIL DESIGN	Check	-		
	Pier P2 Left	Revise	-		

3. Pier Column Section

Pier column thickness - longitudinal dimension	td	2.50	m
Pier column width - transverse dimension	tn	2.50	m
Section area	A	4.909	m ²
Moment inertia	Ix	1.896	m ⁴
	Iy	1.917	m ⁴
Radius of gyration of gross concrete section; $r = \sqrt{I/A}$	rx	0.621	m
	ry	0.625	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	8.70	m
Slenderness ratio: if $K.Lu / r > 22$ than considered	$Kt.Lu/rx$	29.4	yes
	$Kl.Lu/ry$	29.2	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 determination procedure:

Initial	Determining displacement for gross cross section	$\Delta x_g = F_x \cdot H^3 / (3.E.I_g)$
	Displacement for cracked section	$\Delta x_{cr} = F_{cr} \cdot \Delta x_g$
	Moment P-Δ	$M_{P-\Delta} = \Delta x_{cr} \cdot P$
	Added lateral force	$\Delta F_x = M_{P-\Delta} / H$
Step: i st	Determining displacement for gross cross section	$\Delta x_{g_i} = (F_x + \Delta F_{x_{i-1}}) \cdot H^3 / (3.E.I_g)$
	Displacement for cracked section	$\Delta x_{cr_i} = F_{cr} \cdot \Delta x_{g_i}$
	Moment P-Δ	$M_{P-\Delta_i} = \Delta x_{cr_i} \cdot P$
	Added lateral force	$\Delta F_{x_i} = M_{P-\Delta_i} / H$

Combination	Fv (kN)	My (kNm)	Initial					
			Fx (kN)	Δx_g (m)	Fcr (kN)	Δx_{cr} (m)	M P-Δ (kNm)	ΔF_x (kN)
Strength 1a	17009	2740	315	0.001	2.5	0.003	55	6.4
Strength 1b	12072	2740	315	0.001	2.5	0.003	39	4.5
Strength 2a	14348	1117	128	0.001	2.5	0.001	19	2.2
Strength 2b	9412	990	114	0.000	2.5	0.001	11	1.3
Strength 3a	16400	2745	316	0.001	2.5	0.003	53	6.1
Strength 3b	11464	2709	311	0.001	2.5	0.003	37	4.2
Service I	12656	2120	244	0.001	2.5	0.003	32	3.7
Extreme 1a	15013	2812	323	0.001	2.5	0.003	50	5.8
Extreme 1b	10268	2812	323	0.001	2.5	0.003	34	3.9
Extreme 1c	15013	1391	160	0.001	2.5	0.002	25	2.9
Extreme 1d	10268	1391	160	0.001	2.5	0.002	17	1.9

Combination	Fv (kN)	My (kNm)	1st Trial					
			Fx (kN)	Δx_g (m)	Fcr (kN)	Δx_{cr} (m)	M P-Δ (kNm)	ΔF_x (kN)
Strength 1a	17009	2740	321	0.001	2.5	0.003	56	6.5
Strength 1b	12072	2740	320	0.001	2.5	0.003	40	4.6
Strength 2a	14348	1117	131	0.001	2.5	0.001	19	2.2
Strength 2b	9412	990	115	0.000	2.5	0.001	11	1.3
Strength 3a	16400	2745	322	0.001	2.5	0.003	54	6.3
Strength 3b	11464	2709	316	0.001	2.5	0.003	37	4.3
Service I	12656	2120	247	0.001	2.5	0.003	32	3.7
Extreme 1a	15013	2812	329	0.001	2.5	0.003	51	5.9
Extreme 1b	10268	2812	327	0.001	2.5	0.003	35	4.0
Extreme 1c	15013	1391	163	0.001	2.5	0.002	25	2.9
Extreme 1d	10268	1391	162	0.001	2.5	0.002	17	2.0

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Combination	Fv (kN)	My (kNm)	2nd Trial					
			Fx (kN)	Δxg (m)	Fcr (kN)	Δxcr (m)	M P- Δ (kNm)	ΔFx (kN)
Strength 1a	17009	2740	322	0.001	2.5	0.003	56	6.5
Strength 1b	12072	2740	320	0.001	2.5	0.003	40	4.6
Strength 2a	14348	1117	131	0.001	2.5	0.001	19	2.2
Strength 2b	9412	990	115	0.000	2.5	0.001	11	1.3
Strength 3a	16400	2745	322	0.001	2.5	0.003	54	6.3
Strength 3b	11464	2709	316	0.001	2.5	0.003	37	4.3
Service I	12656	2120	248	0.001	2.5	0.003	32	3.7
Extreme 1a	15013	2812	329	0.001	2.5	0.003	51	5.9
Extreme 1b	10268	2812	327	0.001	2.5	0.003	35	4.0
Extreme 1c	15013	1391	163	0.001	2.5	0.002	25	2.9
Extreme 1d	10268	1391	162	0.001	2.5	0.002	17	2.0

****Transverse Direction**

Combination	Fv (kN)	Mx (kNm)	Initial					
			Fx (kN)	Δxg (m)	Fcr (kN)	Δxcr (m)	M P- Δ (kNm)	ΔFx (kN)
Strength 1a	17009	7635	878	0.004	2.5	0.009	156	17.9
Strength 1b	12072	7660	881	0.004	2.5	0.009	111	12.8
Strength 2a	14348	4314	496	0.002	2.5	0.005	74	8.5
Strength 2b	9412	4212	484	0.002	2.5	0.005	48	5.5
Strength 3a	16400	7747	891	0.004	2.5	0.009	153	17.5
Strength 3b	11464	7736	890	0.004	2.5	0.009	106	12.2
Service I	12656	5912	680	0.003	2.5	0.007	90	10.3
Extreme 1a	15013	3075	354	0.001	2.5	0.004	55	6.4
Extreme 1b	10268	3075	354	0.001	2.5	0.004	38	4.4
Extreme 1c	15013	5143	591	0.002	2.5	0.006	93	10.7
Extreme 1d	10268	5143	591	0.002	2.5	0.006	63	7.3

Combination	Fv (kN)	Mx (kNm)	1st Trial					
			Fx (kN)	Δxg (m)	Fcr (kN)	Δxcr (m)	M P- Δ (kNm)	ΔFx (kN)
Strength 1a	17009	7635	896	0.004	2.5	0.009	159	18.3
Strength 1b	12072	7660	894	0.004	2.5	0.009	113	13.0
Strength 2a	14348	4314	505	0.002	2.5	0.005	76	8.7
Strength 2b	9412	4212	490	0.002	2.5	0.005	48	5.5
Strength 3a	16400	7747	908	0.004	2.5	0.009	156	17.9
Strength 3b	11464	7736	902	0.004	2.5	0.009	108	12.4
Service I	12656	5912	690	0.003	2.5	0.007	91	10.5
Extreme 1a	15013	3075	360	0.002	2.5	0.004	56	6.5
Extreme 1b	10268	3075	358	0.001	2.5	0.004	38	4.4
Extreme 1c	15013	5143	602	0.003	2.5	0.006	94	10.9
Extreme 1d	10268	5143	599	0.003	2.5	0.006	64	7.4

Combination	Fv (kN)	Mx (kNm)	2nd Trial					
			Fx (kN)	Δxg (m)	Fcr (kN)	Δxcr (m)	M P- Δ (kNm)	ΔFx (kN)
Strength 1a	17009	7635	896	0.004	2.5	0.009	159	18.3
Strength 1b	12072	7660	894	0.004	2.5	0.009	113	13.0
Strength 2a	14348	4314	505	0.002	2.5	0.005	76	8.7
Strength 2b	9412	4212	490	0.002	2.5	0.005	48	5.5
Strength 3a	16400	7747	909	0.004	2.5	0.009	156	17.9
Strength 3b	11464	7736	902	0.004	2.5	0.009	108	12.4
Service I	12656	5912	690	0.003	2.5	0.007	91	10.5
Extreme 1a	15013	3075	360	0.002	2.5	0.004	56	6.5
Extreme 1b	10268	3075	358	0.001	2.5	0.004	38	4.4
Extreme 1c	15013	5143	602	0.003	2.5	0.006	94	10.9
Extreme 1d	10268	5143	599	0.003	2.5	0.006	64	7.4

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

Combination	Fv Vert. (kN)	Mx Trans. (kNm)	Mx P-Δ (kNm)	Mx Total (kNm)	My Long. (kNm)	My P-Δ (kNm)	My Total (kNm)
Strength 1a	17009	7635	159	7794	2740	56	2796
Strength 1b	12072	7660	113	7773	2740	40	2779
Strength 2a	14348	4314	76	4389	1117	19	1136
Strength 2b	9412	4212	48	4261	990	11	1001
Strength 3a	16400	7747	156	7903	2745	54	2799
Strength 3b	11464	7736	108	7844	2709	37	2746
Service 1	12656	5912	91	6003	2120	32	2152
Extreme 1a	15013	3075	56	3131	2812	51	2863
Extreme 1b	10268	3075	38	3113	2812	35	2847
Extreme 1c	15013	5143	94	5238	1391	25	1416
Extreme 1d	10268	5143	64	5207	1391	17	1408

II. PIER COLUMN DESIGN

1. Limit of Reinforcement

S.5.7.4.2

Minimum area of longitudinal reinforcement in column					
As.fy / (Ag . fc) >= 0.135			As ≥	0.050	m2
As / Ag >= 0.01			As ≥	0.049	m2
Maximum area of longitudinal reinforcement in column					
As / Ag <= 0.08			As ≤	0.393	m2
Trial Rebars:				As	0.031 m2
1layers	x 50	= 50 bars	D28	@150 As1	0.031 m2
1layers	x 0	= 0 bars	D25	@150 As2	0.000 m2

2. Interaction diagram M-P

Using Pca-Column software

****In Both Direction**

Strength and Service limit states:

Resistance factor:	Compression	$\phi_c = 0.75$ (AASHTO LRFD-2004)
	Tension	$\phi_t = 0.90$

Extreme Event limit states:

Resistance factor	Compression	$\phi_c = 1.00$ (AASHTO LRFD-2004)
	Tension	$\phi_t = 1.00$

No.	COMBINATION	Pu kN	Mux kN-m	Muy kN-m	ϕ Mnx kN-m	ϕ Mny kN-m	ϕ Mn/Mu
1	Strength 1a	17009	2796	7794	7142.3	19906.4	2.56
2	Strength 1b	12072	2779	7773	6617.8	18509.9	2.38
3	Strength 2a	14348	1136	4389	5184.4	20038.9	4.57
4	Strength 2b	9412	1001	4261	4133.3	17586.2	4.13
5	Strength 3a	16400	2799	7903	7089.9	20012.9	2.53
6	Strength 3b	11464	2746	7844	6381.5	18228.5	2.33
7	Service 1	12656	2152	6003	6732.8	18783.6	3.13
8	Extreme 1a	15013	2863	3131	15240.7	16662.5	5.33
9	Extreme 1b	10268	2847	3113	13490.9	14751.9	4.74
10	Extreme 1c	15013	1416	5238	5890.3	21776.5	4.16
11	Extreme 1d	10268	1408	5207	5214	19275	3.70

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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	4.531	m2
Tie diameter	Dtie	16	mm
Cross section area of 1 tie	As-tr	0.0002	m2
Spacing of hoops	s	150	mm
Length of reinforcement tie in 1 hoop	Ltie	7.23	m
Ratio of ties reinf. in one hoop/ volume of conc. core one pitch spacing			
$ps = As-tr \cdot Ltie / (Ac \cdot spacing)$	ps	0.0021	
Ratio of spiral reinf. To total volume of concrete core shall satisfy			S.5.7.4.6
$ps \geq 0.45 \cdot (Ag/Ac - 1) \cdot f_c / f_y = Req1$	Req1	0.0028	N/A
			S.5.10.11.3
Transverse Reinforcement for Confinement at Plastic Hinges			S.5.10.11.4.1.d
For a circular column "1:applied", "2:Not applied"		2	
$ps \geq 0.12 \cdot f_c / f_y = Req2$	Req2	0.0090	N/A
For a rectangular column			
Rectangular hoop reinforcement shall satisfy			
Either $A_{sh} \geq 0.30 \cdot s \cdot hc \cdot f_c / f_y$ [$Ag/Ac - 1$] = Req1			
or $A_{sh} \geq 0.12 \cdot s \cdot hc \cdot f_c / f_y = Req2$			
In longitudinal direction "1:applied", "2:Not applied"		2	
Number of cross tie	nt_x	4	ties
Total cross-sectional area of tie reinf.	Ash_x	0.0008	m2
Core dimension of tied column	hc_x	2.40	m
Rectangular hoop reinforcement shall satisfy	Req1_x	0.0007	m2
	Req2_x	0.0032	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	2.40	m
Rectangular hoop reinforcement shall satisfy	Req1_y	0.0007	m2
	Req2_y	0.0032	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	2.50	m
1/6 of clear height of column	L2	0.96	m
or 450mm	L3	0.45	m
Chosen value: $L = \max(L1, L2, L3)$	L	2.50	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	1.25	m
or 380mm	L5	0.38	m
Chosen value: $Le = \max(L4, L5)$	Le	1.25	m

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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	354	459	kN
Required shear capacity $V_n = V_u / \phi_v$	V_n	354	459	kN
Determine concrete shear capacity				
Minimum shear reinforcement will provided in cross section				
Therefore	β	2.0	2.0	
	θ	45.0	45.0	
Cross section equivalent height	h	2.50	2.50	m
width	b	1.96	1.96	m
$d = h - \text{cover} - d_{1x}$	d	2.41	2.41	m
$d_v = \max(0.72 \cdot h; 0.9 \cdot d)$	d_v	2.17	2.17	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	3879	3879	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				
Stirrup diameter	D_s	16	16	
Number of stirrup legs / cross section	n_s	2	2	
Shear legs area	A_v	0.0004	0.0004	m ²
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	0.0002	0.0002	m ²
Check maximum shear reinforcement spacing requirement		OK	OK	
$F = 0.1 \cdot f_c \cdot b_v \cdot d_v$	F	12798	12798	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	0.60	m

Interface shear transfer

S.5.8.4

Area of concrete engaged in shear transfer	A_{cv}	4.909	m ²
Area of shear reinforcement crossing the shear plane	A_{vf}	0.031	m ²
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			
$V_n = c \cdot A_{cv} + \mu \cdot A_{vf} \cdot f_y$	V_n	15756	kN
$V_n \leq 0.2 \cdot f_c \cdot A_{cv}$	$V_n \leq$	29452	kN
$V_n \leq 5.5 \cdot A_{cv} \cdot (1\text{MPa})$	$V_n \leq$	26998	kN
Nominal shear resistance	V_n	15756	kN
Factor for shear friction		1.0	
Factored shear resistance	V_r	15756	kN
Horizontal force at bottom of pier column	V_u	52	kN
	Conclude		OK

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5. Control of cracking by distribution of reinforcement

or Control of Cracking by distribution of reinforcement				
Tensile stress in rebars should be satisfied equation: $f_s \leq f_{sa} = Z/[(dc.A)^{1/3}]$ and $f_s \leq 0.6.f_y$				
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	2152	6003	kNm
Axial thrust at service limit state	Ns	12656	12656	kN
Cross section equivalent	height	h	2.50	m
	width	b	1.96	m
Concrete thickness from tension fiber to tension reinf.	dc	0.05	0.05	m
Concrete thickness from compression fiber to tension reinf.	d	2.41	2.41	kN
Number of rebars	N	25	25	bars
Area of rebars	As	0.0154	0.0154	m2
Area of concrete assumed to participate with reinf.				
$A = 2 \cdot dc \cdot b / N$	A	0.0079	0.0079	m2
	f _{sa}	410	410	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	1.33	1.64	m
$e/d > 1.15$	e/d	1.15	1.15	
$j = 0.74 + 0.1(e/d) \leq 0.9$	j	0.86	0.86	
$i = 1/(1-j.d/e)$	i	3.90	3.90	
Stress in rebars: $f_s = (M_s + N_s(d - h/2))/(A_s.j.i.d)$	f _s	136	167	MPa
	Conclude	OK	OK	
Maximum width of crack: $a_n = 0.076.\beta.f_s.(dc.A)^{1/3}$	a _n	0.126	0.155	mm
Where	β	0.167	0.167	

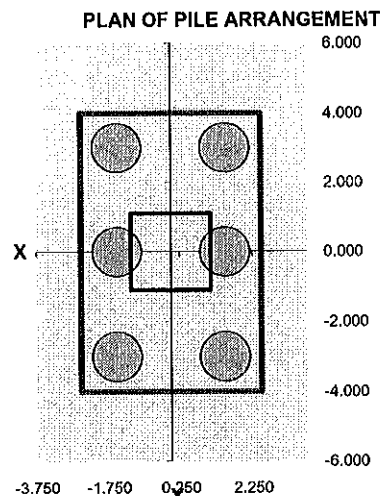
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E. PILE CAP DESIGN

1. PILES AND PILECAP DATA

Piles Data			
Number of piles: n_p			6
Co-ordinates of Piles			
Number	Diameter	X_i (m)	Y_i (m)
1	1.000	1.500	3.000
2	1.000	1.500	0.000
3	1.000	1.500	-3.000
4	1.000	-1.500	-3.000
5	1.000	-1.500	0.000
6	1.000	-1.500	3.000
-			
-			
-			
-			
-			
$\sum X_i^2, \sum Y_i^2$		13.500	36.000

Pilecap Data	
X_i	Y_i
2.500	4.000
2.500	-4.000
-2.500	-4.000
-2.500	4.000
2.500	4.000
Column Data	
X_i	Y_i
1.108	1.108
1.108	-1.108
-1.108	-1.108
-1.108	1.108
1.108	1.108

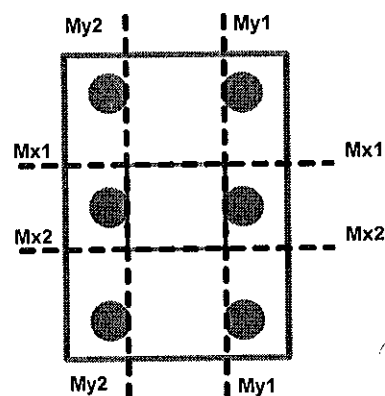


2. CRITICAL SECTIONS

Pile cap Length	L	=	8.000 m
Pile cap Width	W	=	5.000 m
Pile cap thickness	H	=	2.000 m
Column Width	w_c	=	2.500 m
Column Thickness	t_c	=	2.500 m
Round nose radius	c_1	=	1.250 m

Column Area	A_c	=	4.909 m ²
Column block equivalent:			
Width	w_{ce}	=	2.216 m
Thickness	t_{ce}	=	2.216 m

Distance from Pile to Critical Sections - Arm (m)				
Pile No.	Section			
	Mx1	Mx2	My1	My2
1	1.892	-	0.392	-
2	-	-	0.392	-
3	-	1.892	0.392	-
4	-	1.892	-	0.392
5	-	-	-	0.392
6	1.892	-	-	0.392
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-



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3. INTERNAL FORCES CALCULATION

3.1. Pile Reaction (from Pile Foundation analysis)

AXIAL FORCE (KN)											
Pile	STR I-A	STR I-B	STR II-A	STR II-B	STR III-A	STR III-B	SER	EXT I-A	EXT I-B	EXT I-C	EXT I-D
1	4523.9	3463.4	3742.3	2655.1	4491.8	3423.8	3419.8	4442.5	3372.2	4522.4	3452.0
2	3915.8	2845.5	3259.1	2188.7	3816.0	2745.6	2899.9	4026.6	2956.2	3550.3	2480.0
3	3307.6	2227.5	2775.8	1722.3	3140.1	2067.5	2380.1	3610.8	2540.4	2578.3	1508.0
4	2599.9	1519.7	2494.8	1441.3	2429.3	1356.7	1830.0	2047.9	977.6	1968.1	897.7
5	3208.1	2137.7	2978.0	1907.8	3105.2	2034.9	2349.8	2463.8	1393.5	2940.1	1869.8
6	3816.1	2755.7	3461.3	2374.1	3781.1	2713.0	2869.7	2879.7	1809.4	3912.1	2841.8
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-

BENDING MOMENT - Mx (KNm)											
Pile	STR I-A	STR I-B	STR II-A	STR II-B	STR III-A	STR III-B	SER	EXT I-A	EXT I-B	EXT I-C	EXT I-D
1	-56.0	-51.3	149.3	142.9	16.5	18.0	17.1	84.4	84.4	312.6	312.6
2	56.0	51.3	-149.3	-142.9	-16.5	-18.0	-17.1	-84.4	-84.4	-312.6	-312.6
3	56.0	51.3	-149.3	-142.9	-16.5	-18.0	-17.1	-84.4	-84.4	-312.6	-312.6
4	56.0	51.3	-149.3	-142.9	-16.5	-18.0	-17.1	-84.4	-84.4	-312.6	-312.6
5	56.0	51.3	-149.3	-142.9	-16.5	-18.0	-17.1	-84.4	-84.4	-312.6	-312.6
6	-56.0	-51.3	149.3	142.9	16.5	18.0	17.1	84.4	84.4	312.6	312.6
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-

BENDING MOMENT - My (KNm)											
Pile	STR I-A	STR I-B	STR II-A	STR II-B	STR III-A	STR III-B	SER	EXT I-A	EXT I-B	EXT I-C	EXT I-D
1	16.2	16.2	17.4	17.4	20.9	20.9	16.5	153.7	153.7	49.4	49.4
2	16.2	16.2	17.4	17.4	20.9	20.9	16.5	153.7	153.7	49.4	49.4
3	16.2	16.2	17.4	17.4	20.9	20.9	16.5	153.7	153.7	49.4	49.4
4	-16.2	-16.2	-17.4	-17.4	-20.9	-20.9	-16.5	-153.7	-153.7	-49.4	-49.4
5	-16.2	-16.2	-17.4	-17.4	-20.9	-20.9	-16.5	-153.7	-153.7	-49.4	-49.4
6	-16.2	-16.2	-17.4	-17.4	-20.9	-20.9	-16.5	-153.7	-153.7	-49.4	-49.4
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-

3.2. Pilecap weight

Section	V (m3)	Ni (KN)	Arm (m)	Mx (KNm)	My (KNm)
Mx1	28.9	-419.4	1.446	-606.5	
Mx2	28.9	-419.4	1.446	-606.5	
My1	22.3	-323.0	0.696		-224.8
My2	22.3	-323.0	0.696		-224.8

LOAD FACTOR FOR DEAD LOAD OF PILE CAP										
STR I-A	STR I-B	STR II-A	STR II-B	STR III-A	STR III-B	SER	EXT I-A	EXT I-B	EXT I-C	EXT I-D
1.25	0.90	1.25	0.90	1.25	0.90	1.00	1.25	0.90	1.25	0.90

3.3. Internal Forces at Critical Sections

INTERNAL FORCE AT SECTION Mx1 - Unit KN, KNm											
	STR I-A	STR I-B	STR II-A	STR II-B	STR III-A	STR III-B	SER	EXT I-A	EXT I-B	EXT I-C	EXT I-D
Shear	7815.8	5841.7	6679.4	4651.8	7748.7	5759.3	5870.1	6798.0	4804.1	7910.3	5916.4
Moment	14910.9	11119.6	13171.3	9256.4	14928.9	11102.3	11328.8	13265.8	9427.5	15827.2	11988.7

INTERNAL FORCE AT SECTION Mx2 - Unit KN, KNm											
	STR I-A	STR I-B	STR II-A	STR II-B	STR III-A	STR III-B	SER	EXT I-A	EXT I-B	EXT I-C	EXT I-D
Shear	5383.4	3369.7	4746.4	2786.2	5045.2	3046.7	3790.7	5134.5	3140.5	4022.1	2028.3
Moment	10532.4	6647.1	8916.5	5154.6	9747.6	5897.4	7325.7	9780.7	5942.2	7219.4	3381.1

INTERNAL FORCE AT SECTION My1 - Unit KN, KNm											
	STR I-A	STR I-B	STR II-A	STR II-B	STR III-A	STR III-B	SER	EXT I-A	EXT I-B	EXT I-C	EXT I-D
Shear	11343.5	8245.7	9373.5	6275.4	11044.1	7946.2	8376.8	11676.1	8578.1	10247.3	7149.3
Moment	4375.2	3194.5	3605.9	2425.1	4271.6	3090.9	3236.8	4918.1	3737.3	4044.5	2863.8

INTERNAL FORCE AT SECTION My2 - Unit KN, KNm											
	STR I-A	STR I-B	STR II-A	STR II-B	STR III-A	STR III-B	SER	EXT I-A	EXT I-B	EXT I-C	EXT I-D
Shear	9220.4	6122.4	8530.3	5432.5	8911.8	5813.9	6726.5	6987.7	3889.7	8416.5	5318.6
Moment	3444.9	2264.2	3170.9	1990.2	3310.1	2129.4	2490.6	2156.8	976.1	3030.3	1849.6

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4. One-way Shear capacity Check

S.5.8

Critical shear section for one-way shear is located at distance d_v from face of equivalent square column.					
Estimated distance between internal flexural force components d_v , we may take $d_v = 0.9 \cdot d_e$					
$d_e = H - \text{cover} - d_x$				d_e	1.81 m
				d_v	1.63 m
Direction			Long.-X	Trans.-Y	Unit
Shear resistance factors		ϕ_v	0.9	0.9	
Factored shear force in longitudinal		V_u	7910	11676	kN
Required shear capacity $V_n = V_u / \phi_v$		V_n	8789	12973	kN
Determine concrete shear capacity					
Minimum shear reinforcement will provided in cross section					
Therefore		β	2.0	2.0	
		θ	45.0	45.0	
Cross section	height	h	2.00	2.00	m
	width	b	5.00	8.00	m
$d = h - \text{cover} - d_x$		d	1.81	1.81	m
$d_v = \max(0.72 \cdot h; 0.9 \cdot d)$		d_v	1.63	1.63	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	7422	11875	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	1367	1098	kN
In this case $V_c > V_n$ so shear reinforcement is no need					
Stirrup diameter		D_s	16	16	
Number of stirrup legs / cross section		n_s	20	36	
Shear legs area		A_v	0.0040	0.0073	m ²
Angle of inclination of shear reinf. to long. axis		α	90	90	deg
$V_s = A_v \cdot f_y \cdot d_v \cdot (\cot \theta + \cot \alpha) \cdot \sin \alpha / s$		$s \leq$	0.14	0.32	m
Stirrup spacing used		s	0.30	0.30	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	0.0017	0.0027	m ²
Check maximum shear reinforcement spacing requirement			OK	OK	
$F = 0.1 \cdot f_c \cdot b_v \cdot d_v$		F	24489	39182	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	0.60	m

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5. Two-way Shear capacity Design

S.5.13.3.6.3

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

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

The same dimension $d_v/2$ is used to check two-way shear for a corner pile.

Column v.s Pilecap

Pier Column dimensions	Longitudinal axis	td	2.50	m
	Transverse axis	tn	2.50	m
Perimeter of two-way shear				
$b_0 = (td+tn)*2 + 4*d_v$		b0	17.33	m
Section with transverse reinforcement				
Nominal shear resistance shall be taken as				
$V_n = V_c + V_s \leq 0.504 \cdot \sqrt{f_c} \cdot b_0 \cdot d_v = V_a$				
$V_c = 0.166 \cdot \sqrt{f_c} \cdot b_0 \cdot d_v$				
$V_s = A_v \cdot f_y \cdot d_v / s$				
Shear resistance of concrete		Vc	25719	kN
Assumed stirrup diameter		Ds	16	mm
Number of stirrup legs / cross section		ns	28	
Shear legs area		Av	0.0057	m ²
Stirrup spacing used		s	600	mm
Shear resistance of reinforcement		Vs	6156	kN
		Va	78088	kN
		Vn	31875	kN
Maximum reaction at bottom of column		Vu	17009	kN
Resistance factor for shear		ϕ_v	0.9	
Factored shear resistance		$\phi_v \cdot V_n$	28688	kN
Punching shear check			OK	

Conner pile v.s Pilecap

Conner pile vs Pilecap			
Pile diameter	D	1.00	m
Radius of critical section for two-way shear $R_{co} = D/2 + d_v/2$	Rco	1.32	m
Distance from pile center of conner pile to edge of pilecap	a1	1.00	m
Perimeter of two-way shear			
$b_0 = 2*a_1 + 1/4*2*pi()*R_{co}$	b0	4.07	m
Section with transverse reinforcement			
Norminal shear resistance shall be taken as			
$V_n = V_c + V_s \leq 0.504.\sqrt{f_c}. b_0 . d_v = V_a$			
$V_c = 0.166 . \sqrt{f_c}. b_0 . d_v$			
$V_s = A_v. f_y . d_v / s$			
Shear resistance of concrete	Vc	6038	kN
Assumed stirrup diameter	Ds	16	mm
Number of stirrup legs / cross section	ns	13	
Shear legs area	Av	0.0026	m2
Stirrup spacing used	s	300	mm
Shear resistance of reinforcement	Vs	5716	kN
	Va	18332	kN
	Vn	11754	kN
Maximum reaction of conner pile	Vu	4524	kN
Resistance factor for shear	ϕ_v	0.9	
Factored shear resistance	ϕ_v*V_n	10579	kN
Punching shear check		OK	

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REINFORCEMENT CHECKING - PILE 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			
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		7.00			
Sign	Parameters	Unit	Section			
			Mx	Mx	My	My
INTERNAL FORCES AT SECTION						
	Combination		Strength	Service	Strength	Service
Qu	Shear	kN	7910	5916	11676	7149
Mu	Flexural Moment	kNm	15827	11329	4918	3237
Nu	Axial load	kN				
Tu	Torsional Moment	kNm				
FLEXURAL MOMENT CHECKING						
H	Section height	m	2.000	2.000	2.000	2.000
d's	Dis. From comp. fiber to centroid of comp. Reinf	m	0.084	0.084	0.084	0.084
d1x	Dis. From tens. fiber to centroid of tension Reinf	m	0.163	0.163	0.160	0.160
	Cover to reinf	m	0.050	0.050	0.050	0.050
ds	Dis. From comp. fiber to centroid of tension Reinf	m	1.838	1.838	1.840	1.840
d'ps	Dis. From comp. fiber to centroid of comp. prestressing steel	m	0.000	0.000	0.000	0.000
d1xp	Dis. From tens. fiber to centroid of tension prestressing steel	m	0.000	0.000	0.000	0.000
dps	Dis. From comp. fiber to centroid of tension prestressing steel	m	2.000	2.000	2.000	2.000
b	Width of the compression face of member	m	5.000	5.000	8.000	8.000
bw	Web width or diameter of a circular section	m	5.000	5.000	8.000	8.000
hf	Compression flange depth	m	0.000	0.000	0.000	0.000
Iz	Moment of inertia of section	m4	3.333	3.333	5.333	5.333
Amc	Section area	m2	10.000	10.000	16.000	16.000
	Steel choice					
Aps	Tension prestressing steel	P.S type	0 T00.0	0 T00.0	0 T00.0	0 T00.0
	Number	tendons	0	0	0	0
	Area	m2	0.00000	0.00000	0.00000	0.00000
A'ps	Compression prestressing steel	P.S type	0 T00.0	0 T00.0	0 T00.0	0 T00.0
	Number	tendons	0	0	0	0
	Area	m2	0.00000	0.00000	0.00000	0.00000
As	Tension Reinforcement	Number	66	66	53	53
	Diameter	mm	25	25	20	20
	Area	m2	0.03241	0.03241	0.01664	0.01664
A's	Compression Reinforcement	Number	33	33	53	53
	Diameter	mm	18	18	18	18
	Area	m2	0.00838	0.00838	0.01346	0.01346
A'c	Shear reinforcement	Number	16	16	26	26
	Diameter	mm	16	16	16	16
	Area	m2	0.00323	0.00323	0.00525	0.00525

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ϕ	Resistance factors for flexure	5.5.4.2	0.90	1.00	0.90	1.00
ϕ_v	Resistance factors for shear		0.90	1.00	0.90	1.00
ϕ_n	Resistance factors for axial force		1.00	1.00	1.00	1.00
β_1	Stress block factor		0.836	0.836	0.836	0.836
c	Dis. Between centroid and top fiber	m	0.090	0.090	0.007	0.007
	For T section behavior	m	0.090	0.090	0.007	0.007
	For rectangular section behavior	m	0.090	0.090	0.007	0.007
f _{pe}	Effective stress in the prestressing steel after losses	Mpa	1116	1116	1116	1116
f _{ps}	Aver. stress in pres. steel at the time for which the nominal resistance	Mpa	1836	1836	1858	1858
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	0.075	0.075	0.006	0.006
d _e	Corresp. effective depth from extreme comp. fiber to centroid of tensile force in the tensile reinf.	m	1.838	1.838	1.840	1.840
M _n	Nominal resistance	kNm	23175	23175	11792	11792
M _r	Factored resistance	kNm	20857	23175	10613	11792
M _u	Flexual moment	kNm	15827	11329	4918	3237
(5.7.3.2)	Flexural moment Checking		OK	OK	OK	OK
	Limits for reinforcement					
c/d _e	Maximum reinforcement		0.05	0.05	0.00	0.00
	Maximum reinforcement Checking	≤ 0.42	OK	OK	OK	OK
1.2*M _{cr}	Craking moment	kNm	7227	7227	11083	11083
(5.7.3.3.2)	Checking $M_r \geq \min(1.2M_{cr}, 1.33M_u)$		OK	OK	OK	OK
(5.7.3.4)	Conetrol of craking by distr. of reinf for RC member- Check?		No	Yes	No	Yes
	Existing condition for structure	1,2 or 3	3	3	3	3
d _c	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.063	0.063	0.060	0.060
Z	Crack width parameter	N/mm	17500	17500	17500	17500
A	Area of concr. with same centroid as tens. Reinf	m ²	0.009	0.009	0.018	0.018
f _{sa}	Value	Mpa	208	208	170	170
0.6*f _y		Mpa	240	240	240	240
	Tensil stress in reinf Min(f _{sa} , 0.6f _y)	Mpa	208	208	170	170
x	Dist. From compression fiber to centroid	m	-	0.365	-	0.217
J.d	Arm	m	-	1.716	-	1.768
I _{cr}	Moment of inertia of the cracked section	m ⁴	-	0.577	-	0.336
f _s	Tensile stress in reinforcement f _s = M _s / (A _s *J.d)	Mpa	-	204	-	110
	Checking for control cracking f _s < f _{sa}		N.a	OK	N.a	OK
(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
	Distribution on sides 10 D16	m ²	0.00202	0.00202	0.00202	0.00202
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45
	Checking		OK	OK	OK	OK

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SHEAR AND TORSION CHECKING						
β	Factor indicating diag. cracked concr. to tension		1.7	2.0	1.8	2.1
θ	Angle of inclination of diagonal compressive	degree	42.87	40.28	42.73	38.11
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90
bv	Effective web width as minimum web width - in dv	m	5.000	5.000	8.000	8.000
dv	Effective shear depth	m	1.800	1.800	1.837	1.837
	(dc - a/2)	m	1.800	1.800	1.837	1.837
s	Spacing of stirrups	m	0.300	0.300	0.300	0.300
ncat	Amount of bars in spacing S	bars	16	16	26	26
Av	Shear reinf area in spacing S	m2	0.0032	0.0032	0.0053	0.0053
β	Assume		2.0	2.0	2.0	2.0
θ	Assume	degree	45.00	45.00	45.00	45.00
v	Shear stress in concrete	kN/m2	977	657	883	487
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116
ϵ_x	Strain in tensile reinforcement		1.97E-03	1.43E-03	1.93E-03	1.21E-03
	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		0.033	0.022	0.029	0.016
β	Final value		1.7	2.0	1.8	2.1
θ	Final value	degree	42.87	40.28	42.73	38.11
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	7099	8143	11696	14107
Vs	Shear resistance provided by shear reinforcement	kN	8356	9153	13924	16396
Vp	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0
Vn1	$Vn1 = Vc + Vs + Vp$	kN	15454	17297	25620	30503
Vn2	Vn2	kN	67493	67493	110213	110213
Vn	Nominal shear resistance $Vn = \min(Vn1, Vn2)$	kN	15454	17297	25620	30503
Vr	Factored shear resistance	kN	13909	17297	23058	30503
Vu	Shear	kN	7910	5916	11676	7149
(5.8.2.7)	Shear checking		OK	OK	OK	OK

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F.BORED PILE DESIGN

I. BORED PILE DATA

1. Load Combinations at top of bored pile

STT	Comb	Axial force P (KN)	Moment (KN.m)		
			Mx	My	Mxy
STRENGTH LIMIT STATES					
1	P_min	1357	18	21	28
2	P_max	4524	56	16	58
3	Mx_max	3742	149	17	150
4	My_max	4492	16	21	27
EXTREME EVENT LIMIT STATES					
1	P_min	898	313	49	316
2	P_max	4522	313	49	316
3	Mx_max	4522	313	49	316
4	My_max	4442	84	154	175

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

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	0.010 m ²
1 layers x 20 = 20 bars D25 @150 As1		0.010 m ²

2. Interaction diagram M-P

Using Pca-Column software

**Flexural check by pcaColumn

Strength and Service limit states:

Resistance factor:	Compression	ϕ_c	=	0.75 (AASHTO LRFD-2004)
	Tension	ϕ_t	=	0.90

Extreme Event limit states:

Resistance factor	Compression	ϕ_c	=	1.00
	Tension	ϕ_t	=	1.00

	DA NANG - QUANG NGAI EXPRESS WAY PROJECT Bridge LRB 12a DETAIL DESIGN Pier P2 Left	Item.	Eng.	Date	Sign.
		Design	-		
		Check	-		
		Revise	-		

<Result table>

STT	Comb	Pu kN	Mux kN-m	Muy kN-m	ϕ Mnx kN-m	ϕ Mny kN-m	ϕ Mn/Mu
STRENGTH LIMIT STATES							
1	P_max	1356.7	18	21	1109	1287	61.6
2	P_min	4523.9	56	16	2014	583	36.0
3	Mx_max	3742.3	149.3	17	2076	242	13.9
4	My_max	4491.8	16.5	21	1302	1649	78.9
EXTREME EVENT LIMIT STATES							
1	P_max	898	313	49	1729	271	5.5
2	P_min	4522	313	49	2518	394	8.0
3	Mx_max	4522	313	49	2518	394	8.0
4	My_max	4442	84	154	1214	2226	14.5

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	0.00015	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			
$\rho_s = A_{s-tr} \cdot L_{tie} / (A_c \cdot \text{spacing})$	ρ_s	0.0090	
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	0.0089	OK
Transverse Reinforcement for Confinement at Plastic Hinges			S.5.10.11.4.1.d
For a circular column "1:applied", "2:Not applied"		1	
$\rho_s \geq 0.12 \cdot f_c / f_y = \text{Req2}$	Req2	0.0090	N/A
Length distributed spiral with pitch 75mm below pilecap	Ldis	1.50	m

4. Shear Design

Shear resistance factors	ϕ_v	1.0	
Factored shear force	Vu	140	kN
Required shear capacity $V_n = V_u / \phi_v$	Vn	140	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
$d_v = 0.9 \cdot d_e$ $d_e = D/2 + D_r/\pi()$			
Diameter of the circle passing through the centers of the long. reinf.	Dr	0.79	m
	d_e	0.75	m
	d_v	0.68	m
$V_c = 0.083 \cdot \beta \cdot \sqrt{f_c} \cdot b_v \cdot d_v$	Vc	616	kN
	Conclude		OK

SPACE PILE FOUNDATION ANALYSIS PROGRM
Turbo BASIC

PROJECT: : DN-QN-P2-LRB12a

INITIA DATA

Kn = 0.13 Ax = 5.00 By = 8.00 Cz = 2.00
E v.uon = 3001028 E r.uon = 3001028 E v.nen = 3001028 E r.nen =
3001028
Mq = 100 (t/m4) Md = 0 (t/m4) m = 350 (t/m4)

LOAD COMBINATIONS

COMB.	Hx	Hy	P	Mx	My	Mz
1	18.52	0.08	2178.54	-778.65	316.30	0.00
2	18.52	1.31	1523.89	-787.89	316.30	0.00
3	9.76	41.90	1907.37	-501.13	118.98	0.00
4	9.76	40.19	1252.72	-484.35	118.98	0.00
5	19.60	17.08	2116.56	-817.64	314.89	0.00
6	19.60	17.47	1461.91	-819.45	314.89	0.00
7	15.24	14.09	1605.43	-626.22	243.52	0.00
8	66.83	26.54	1984.84	-458.04	627.25	0.00
9	66.83	26.54	1330.20	-458.04	627.25	0.00
10	23.75	86.93	1984.84	-1000.57	251.38	0.00
11	23.75	86.93	1330.20	-1000.57	251.38	0.00

PROPERTIES OF PILES

PILE	Lo	H	Bpx	Bpy	A	B	Cday	Fo	Io	Po	Co	Ct
1	0.00	10.00	1.350	1.350	1.00	0.000	0.000	0.785	0.098	500	30000	15000
2						n t						
3						n t						
4						n t						
5						n t						
6						n t						

PILE COORD.

PILE	X	Y	Phi	Xi
1	1.50	3.00	0.000	0.00
2	1.50	0.00	0.000	0.00
3	1.50	-3.00	0.000	0.00
4	-1.50	-3.00	0.000	0.00
5	-1.50	0.00	0.000	0.00
6	-1.50	3.00	0.000	0.00

DISPLACEMENTS

COMB.	X	Y	Z	Fix	Fiy	Fiz
1	0.00125	0.00057	0.003545	-0.000202	0.000235	-0.000000
2	0.00125	0.00062	0.002480	-0.000205	0.000235	-0.000000

3	0.00057	0.00184	0.003104	-0.000160	0.000093	-0.000000
4	0.00057	0.00177	0.002038	-0.000155	0.000093	-0.000000
5	0.00128	0.00120	0.003444	-0.000224	0.000236	-0.000000
6	0.00128	0.00121	0.002379	-0.000225	0.000236	-0.000000
7	0.00100	0.00095	0.002612	-0.000172	0.000183	-0.000000
8	0.00359	0.00127	0.003230	-0.000138	0.000518	-0.000000
9	0.00359	0.00127	0.002165	-0.000138	0.000518	-0.000000
10	0.00133	0.00379	0.003230	-0.000322	0.000202	-0.000000
11	0.00133	0.00379	0.002165	-0.000322	0.000202	-0.000000

FORCES ON PILES

PILE	COMB.	N	Q2	Q3	M1	M2	M3
1	1	461.15	-2.71	0.10	0.000	5.712	1.656
	2	353.05	-2.71	-0.09	0.000	5.226	1.656
	3	381.48	-1.46	-6.66	0.000	-15.219	1.771
	4	270.65	-1.46	-6.39	0.000	-14.571	1.771
	5	457.88	-2.88	-2.62	0.000	-1.677	2.127
	6	349.01	-2.88	-2.68	0.000	-1.839	2.127
	7	348.60	-2.24	-2.17	0.000	-1.741	1.681
	8	452.85	-10.09	-4.20	0.000	-8.600	15.671
	9	343.75	-10.09	-4.20	0.000	-8.600	15.671
	10	461.00	-3.57	-13.82	0.000	-31.868	5.031
	11	351.89	-3.57	-13.82	0.000	-31.868	5.031
2	1	399.16	-2.71	0.10	0.000	5.712	1.656
	2	290.06	-2.71	-0.09	0.000	5.226	1.656
	3	332.22	-1.46	-6.66	0.000	-15.219	1.771
	4	223.11	-1.46	-6.39	0.000	-14.571	1.771
	5	388.99	-2.88	-2.62	0.000	-1.677	2.127
	6	279.88	-2.88	-2.68	0.000	-1.839	2.127
	7	295.61	-2.24	-2.17	0.000	-1.741	1.681
	8	410.46	-10.09	-4.20	0.000	-8.600	15.671
	9	301.35	-10.09	-4.20	0.000	-8.600	15.671
	10	361.91	-3.57	-13.82	0.000	-31.868	5.031
	11	252.80	-3.57	-13.82	0.000	-31.868	5.031
3	1	337.17	-2.71	0.10	0.000	5.712	1.656
	2	227.06	-2.71	-0.09	0.000	5.226	1.656
	3	282.96	-1.46	-6.66	0.000	-15.219	1.771
	4	175.57	-1.46	-6.39	0.000	-14.571	1.771
	5	320.09	-2.88	-2.62	0.000	-1.677	2.127
	6	210.75	-2.88	-2.68	0.000	-1.839	2.127
	7	242.62	-2.24	-2.17	0.000	-1.741	1.681
	8	368.07	-10.09	-4.20	0.000	-8.600	15.671
	9	258.96	-10.09	-4.20	0.000	-8.600	15.671
	10	262.82	-3.57	-13.82	0.000	-31.868	5.031
	11	153.72	-3.57	-13.82	0.000	-31.868	5.031
4	1	265.03	-2.71	0.10	0.000	5.712	1.656
	2	154.91	-2.71	-0.09	0.000	5.226	1.656
	3	254.31	-1.46	-6.66	0.000	-15.219	1.771
	4	146.92	-1.46	-6.39	0.000	-14.571	1.771
	5	247.64	-2.88	-2.62	0.000	-1.677	2.127
	6	138.30	-2.88	-2.68	0.000	-1.839	2.127
	7	186.54	-2.24	-2.17	0.000	-1.741	1.681
	8	208.76	-10.09	-4.20	0.000	-8.600	15.671
	9	99.65	-10.09	-4.20	0.000	-8.600	15.671
	10	200.62	-3.57	-13.82	0.000	-31.868	5.031
	11	91.51	-3.57	-13.82	0.000	-31.868	5.031
5	1	327.02	-2.71	0.10	0.000	5.712	1.656
	2	217.91	-2.71	-0.09	0.000	5.226	1.656
	3	303.57	-1.46	-6.66	0.000	-15.219	1.771

	4	194.47	-1.46	-6.39	0.000	-14.571	1.771
	5	316.53	-2.88	-2.62	0.000	-1.677	2.127
	6	207.43	-2.88	-2.68	0.000	-1.839	2.127
	7	239.53	-2.24	-2.17	0.000	-1.741	1.681
	8	251.15	-10.09	-4.20	0.000	-8.600	15.671
	9	142.05	-10.09	-4.20	0.000	-8.600	15.671
	10	299.70	-3.57	-13.82	0.000	-31.868	5.031
	11	190.60	-3.57	-13.82	0.000	-31.868	5.031
6	1	389.00	-2.71	0.10	0.000	5.712	1.656
	2	280.91	-2.71	-0.09	0.000	5.226	1.656
	3	352.83	-1.46	-6.66	0.000	-15.219	1.771
	4	242.01	-1.46	-6.39	0.000	-14.571	1.771
	5	385.43	-2.88	-2.62	0.000	-1.677	2.127
	6	276.55	-2.88	-2.68	0.000	-1.839	2.127
	7	292.53	-2.24	-2.17	0.000	-1.741	1.681
	8	293.55	-10.09	-4.20	0.000	-8.600	15.671
	9	184.44	-10.09	-4.20	0.000	-8.600	15.671
	10	398.79	-3.57	-13.82	0.000	-31.868	5.031
	11	289.68	-3.57	-13.82	0.000	-31.868	5.031

SUMMARY OF FORCES

	PILE	COMB.	N	Q2	Q3	M1	M2	M3
Nmin	4	11	91.51	-3.57	-13.82	0.000	-31.868	5.031
Nmax	1	1	461.15	-2.71	0.10	0.000	5.712	1.656
Q2max	1	8	452.85	-10.09	-4.20	0.000	-8.600	15.671
Q3max	1	10	461.00	-3.57	-13.82	0.000	-31.868	5.031
M1max	1	1	461.15	-2.71	0.10	0.000	5.712	1.656
M2max	1	10	461.00	-3.57	-13.82	0.000	-31.868	5.031
M3max	1	8	452.85	-10.09	-4.20	0.000	-8.600	15.671

CHECKING CALCULATI IN COMPARISON WITH INITIA LOAD MATRIX

1	18.52	0.08	2178.54	-778.65	316.30	0.00
2	18.52	1.31	1523.89	-787.89	316.31	0.00
3	9.76	41.90	1907.37	-501.13	118.98	0.00
4	9.76	40.19	1252.72	-484.35	118.98	0.00
5	19.60	17.08	2116.56	-817.64	314.89	0.00
6	19.60	17.47	1461.91	-819.45	314.89	0.00
7	15.24	14.09	1605.43	-626.22	243.52	0.00
8	66.83	26.54	1984.84	-458.04	627.25	0.00
9	66.83	26.54	1330.20	-458.04	627.25	0.00
10	23.75	86.93	1984.84	-1000.56	251.38	0.00
11	23.75	86.93	1330.20	-1000.57	251.38	0.00

Da Nang Quang Ngai Expressway project

Bridge LRB 12a

CALCULATION SHEETS

Pier P3 Left

Table of content

1. Structure Dimensions & Load Components
2. Load Combinations
3. Pier Cap analysis
4. Pier Column analysis
5. Foundation analysis
6. Annex

	DA NANG - QUANG NGAI EXPRESS WAY PROJECT Bridge LRB 12a DETAIL DESIGN Pier P3 Left	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	Simple PC I girder $L=33m$ with link slab			
Span length	Left	=	33.05	Right = 33.05 m
Girder length between bearings	Left	=	32.10	Right = 32.10 m
Bridge width	B	=	12.75	m

Level Table(at center of pier)

Top of pier cap	ThL	7.136	m
Top of pier column	TcL	4.201	m
Bottom of upper pier column	H _{topc}	4.200	m
Bottom of upper pilecap	H _{up}	-0.500	m
Bottom of pilecap	H _{bot}	-2.500	m
Skew angle	Ska	90.000	deg
Ground level	GL	2.000	m
Maximum water level (H1%)	H _{max}	3.980	m
Navigation water level (H5%)	H _{min}	0.000	m
Average Annual water level	H _{ave}	1.990	m
Local scour level (at water level H1%)	H _{sc}	0.000	m

Material unit weight

Concrete

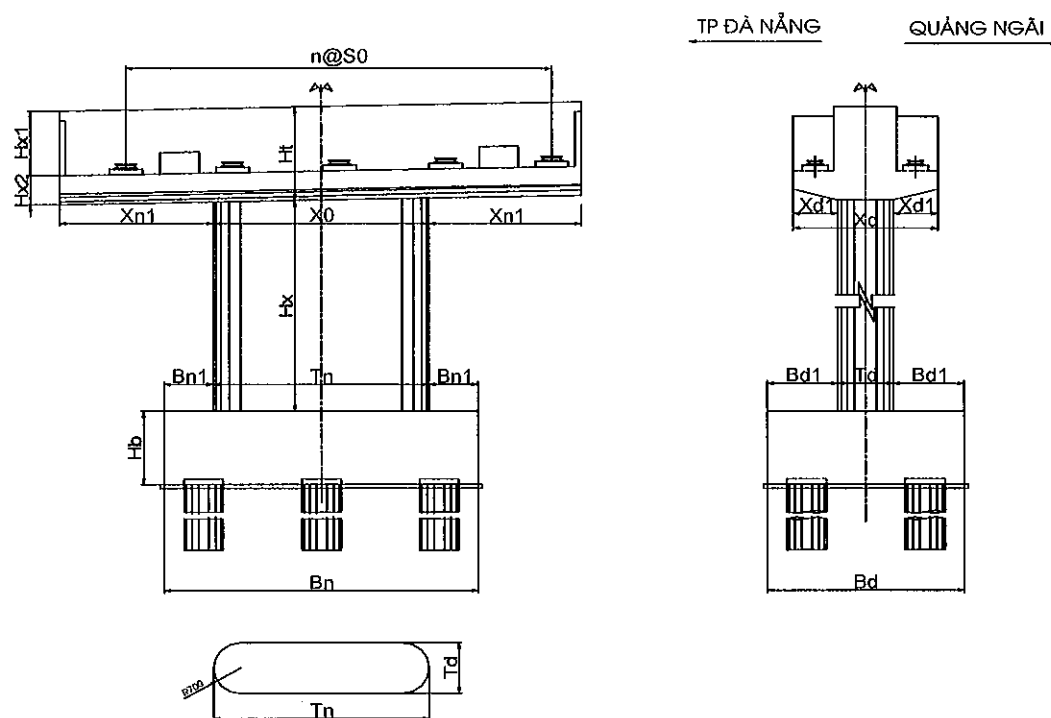
Specify density	γ_c	=	24.5 kN/m ³
Compressive Strength (at 28 days)	f_c	=	30.0 MPa
Elastic Modulus	E_c	=	29440 Mpa

Reinforcement

Yield strength	f_y	=	400.0 MPa
Modulus of elasticity	E_s	=	200000 Mpa
Modular ratio (steel/concrete)	$n = E_s/E_c$	=	6.7935

Asphalt concrete	γ_a	=	22.1 kN/m ³
Soil - ground	γ_s	=	17.7 kN/m ³
Saturated soil	γ_{ss}	=	7.8 kN/m ³

II. PIER DIMENSIONS



Pier Dimensions Table

Notation	Dimensions Transverse direction	Value (m)	Notation	Dimensions Longitudinal direction	Value (m)
*	Bearing distribution		*	Bearing pedestal Width	0.85
nbear	Number of bearing	5.00		Length	0.65
nbear	Number of bearing	5.00		Height	0.15
S _l	Bearings spacing	2.55	*	Anchorage block Width	0.40
S _r	Bearings spacing	2.55		Length	1.00
b _{s1}	Total width of bridge CS	12.75		Height	0.52
b _{s2}	Carriage way width	11.76		Dist. CB's edge to exterior girder	1.27
b _{s3}	Left curb width	0.50		Dist. CB's edge to exterior girder	1.27
b _{s4}	Right curb width	0.49			
*	Pier Cap				
H _{x1}	Haunch 1 height	1.935	X _d	Pier cap width	3.60
H _{x2}	Haunch 2 height	1.00	X _{d1}	Pier cap width	1.00
H _x	Pier cap height	2.935	GL	Left bearing to pier c.line	1.270
X _{n1}	Haunch width	4.99	GR	Right bearing to pier c.line	1.270
X _{n0}	Bottom of pier cap width	2.50	H _c	Curtain wall height	1.80
X _{nt}	Top of pier cap width	12.48	T _c	Curtain wall thickness	0.15
*	Pier Column				
T _n	Pier column width	2.50	T _d	Pier column thickness	2.50
H _{tt}	Pier column height	4.70	Rv	Round nose radius	1.25
H	Column height	4.70			
*	Pile Cap				
B _n	Pile cap width	8.00	B _d	Pile cap length	5.00
H _b	Pile cap depth	2.00			

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	0.83	20.3						
Bearing devices		50.0						
Anchorage block	0.84	20.5						
Pier Cap	83.57	2047.4						
Curtain wall	0.50	12.1						
Upper pier column	0.00	0.0						
Pier Column	23.07	565.2						
Upper pilecap	0.00	0.0						
PileCap	80.00	1960.0						
Shear key	0.00	0.0						
Total at bottom of Column		2715.6						
Total at bottom of pilecap		4675.6						

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	87.73	1552.8						
Total at bottom of Column								
Total at bottom of pilecap		1552.8						

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	21.99	-215.7						
Upper pilecap	0.00	0.0						
PileCap	80.00	-784.8						
Shear key	0.00	0.0						
Total at bottom of Column		-215.7						
Total at bottom of pilecap		-1000.5						

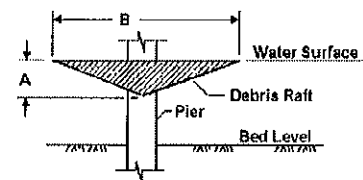
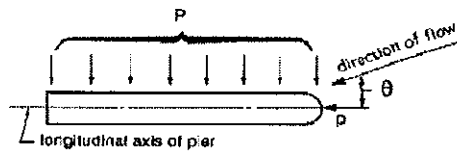
Case2: Minimum water level (Hmin)

Item	Volume (m3)	Vertical F_v (kN)	Longitudinal			Transversal		
			F_{HX} (kN)	Arm. $_{HX}$ (m)	M_y (kN•m)	F_{HY} (kN)	Arm. $_{HY}$ (m)	M_x (kN•m)
Upper pier column	0.00	0.0						
Pier Column	2.45	-24.1						
Upper pilecap	0.00	0.0						
PileCap	80.00	-784.8						
Shear key	0.00	0.0						
Total at bottom of Column		-24.1						
Total at bottom of pilecap		-808.9						

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_{HY} (kN)	Arm- HY (m)	M_x (kN•m)
Upper pier column	0.00	0.0						
Pier Column	12.22	-119.9						
Upper pilecap	0.00	0.0						
PileCap	80.00	-784.8						
Shear key	0.00	0.0						
Total at bottom of Column		-119.9						
Total at bottom of pilecap		-904.7						

4.Stream Pressure



Stream pressure data

Angle between direction of flow and long. axis of pier	θ	0.0	deg
Design velocity of water at H1%	$V_{1\%}$	1.11	m/s
Design velocity of water at minimum water level	V_{min}	0.39	m/s
Design velocity of water at average annual water level	V_{annual}	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%	$pL_{1\%}$	0.44	kN/m2
Stream pressure at minimum water level	pL_{min}	0.05	kN/m2
Stream pressure at average annual water level	pL_{annual}	0.44	kN/m2
Additional stream pressure due to driftwood raft lodged against pier			
Drag coefficient	C_D	0.50	
Height of debris raft	A	1.0	m
Width of debris raft	B	14.0	m
Area of debris raft	Area	6.9	m2
Stream pressure due to driftwood raft at H1%	pL_{debris}	0.32	kN/m2
Equivalent force	$F_{hdebris}$	2.2	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%	$pT_{1\%}$	0.00	kN/m2
Stream pressure at minimum water level	pT_{min}	0.00	kN/m2
Stream pressure at average annual water level	pT_{annual}	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	4.7	0.0	0.0	4.7	0.0
Pier Column	4.48		0.0	2.2	0.0	5.0	2.2	11.1
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	5.0		11.1
Upper pier Column	0.00		0.0	6.7	0.0	0.0	6.7	0.0
Pier Column	4.48		0.0	4.2	0.0	5.0	4.2	21.1
Upper pilecap	0.00		0.0	2.0	0.0	0.0	2.0	0.0
PileCap	0.00		0.0	0.0	0.0	0.0	0.0	0.0
Total at bottom of pilecap			0.0		0.0	5.0		21.1

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	2.2	4.5	9.8
Total at bottom of Column					0.0	2.2		9.8
Pier Column					0.0	2.2	6.5	14.2
Total at bottom of pilecap					0.0	2.2		14.2

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	4.7	0.0	0.0	4.7	0.0
Pier Column	0.50		0.0	0.3	0.0	0.1	0.3	0.0
Upper pilecap	0.00		0.0	0.0	0.0	0.0	0.0	0.0
Total at bottom of Column			0.0		0.0	0.1		0.0
Upper pier Column	0.00		0.0	6.7	0.0	0.0	6.7	0.0
Pier Column	0.50		0.0	2.3	0.0	0.1	2.3	0.2
Upper pilecap	0.00		0.0	2.0	0.0	0.0	2.0	0.0
PileCap	0.00		0.0	0.0	0.0	0.0	0.0	0.0
Total at bottom of pilecap			0.0		0.0	0.1		0.2

Case3: average Annual water level

Item	Exposed height (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	4.7	0.0	0.0	4.7	0.0
Pier Column	2.49		0.0	1.2	0.0	2.8	1.2	3.4
Upper pilecap	0.00		0.0	0.0	0.0	0.0	0.0	0.0
Total at bottom of Column			0.0		0.0	2.8		3.4
Upper pier Column	0.00		0.0	6.7	0.0	0.0	6.7	0.0
Pier Column	2.49		0.0	3.2	0.0	2.8	3.2	9.0
Upper pilecap	0.00		0.0	2.0	0.0	0.0	2.0	0.0
PileCap	0.00		0.0	0.0	0.0	0.0	0.0	0.0
Total at bottom of pilecap			0.0		0.0	2.8		9.0

5. Wind Loads

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	4.1 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 (H1%)

Item	Exposed height (m)	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)
Curtain wall	0.00		0.0	7.6	0.0	0.0	7.6	0.0
Pier Cap	2.94		0.0	6.2	0.0	27.5	6.2	169.7
Upper pier Column	0.00		0.0	4.5	0.0	0.0	4.5	0.0
Pier Column	0.22		1.4	4.6	6.6	1.4	4.6	6.6
Upper pilecap	0.00		0.0	4.5	0.0	0.0	4.5	0.0
Total at bottom of Column			1.4		6.6	28.9		176.2
Curtain wall	0.00		0.0	9.6	0.0	0.0	9.6	0.0
Pier Cap	2.94		0.0	8.2	0.0	27.5	8.2	224.7
Upper pier Column	0.00		0.0	6.5	0.0	0.0	6.5	0.0
Pier Column	0.22		1.4	6.6	9.4	1.4	6.6	9.4
Upper pilecap	0.00		0.0	6.5	0.0	0.0	6.5	0.0
PileCap	0.00		0.0	6.5	0.0	0.0	6.5	0.0
Total at bottom of pilecap			1.4		9.4	28.9		234.1

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 _{Hv} (kN)	Arm _{Hv} (m)	M _x (kN·m)
Curtain wall	0.00		0.0	7.6	0.0	0.0	7.6	0.0
Pier Cap	2.94		0.0	6.2	0.0	27.5	6.2	169.7
Upper pier Column	0.00		0.0	0.5	0.0	0.0	0.5	0.0
Pier Column	4.20		27.3	2.6	71.1	27.3	2.6	71.1
Upper pilecap	0.00		0.0	0.5	0.0	0.0	0.5	0.0
Total at bottom of Column			27.3		71.1	54.8		240.7
Curtain wall	0.00		0.0	9.6	0.0	0.0	9.6	0.0
Pier Cap	2.94		0.0	8.2	0.0	27.5	8.2	224.7
Upper pier Column	0.00		0.0	2.5	0.0	0.0	2.5	0.0
Pier Column	4.20		27.3	4.6	125.7	27.3	4.6	125.7
Upper pilecap	0.00		0.0	2.5	0.0	0.0	2.5	0.0
PileCap	0.00		0.0	2.5	0.0	0.0	2.5	0.0
Total at bottom of pilecap			27.3		125.7	54.8		350.4

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	0.00		0.0	7.6	0.0	0.0	7.6	0.0
Pier Cap	2.94		0.0	6.2	0.0	27.5	6.2	169.7
Upper pier Column	0.00		0.0	2.5	0.0	0.0	2.5	0.0
Pier Column	2.21		14.4	3.6	51.7	14.4	3.6	51.7
Upper pilecap	0.00		0.0	2.5	0.0	0.0	2.5	0.0
Total at bottom of Column			14.4		51.7	41.9		221.4
Curtain wall	0.00		0.0	9.6	0.0	0.0	9.6	0.0
Pier Cap	2.94		0.0	8.2	0.0	27.5	8.2	224.7
Upper pier Column	0.00		0.0	4.5	0.0	0.0	4.5	0.0
Pier Column	2.21		14.4	5.6	80.5	14.4	5.6	80.5
Upper pilecap	0.00		0.0	4.5	0.0	0.0	4.5	0.0
PileCap	0.00		0.0	4.5	0.0	0.0	4.5	0.0
Total at bottom of pilecap			14.4		80.5	41.9		305.1

6.Vessel Collision

7.Vehicular Collision Force

"1:yes", "0:no"

0

IV. SUPERSTRUCTURE LOADS

1. Dead Loads

Left side Span

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)
Stage1 (DC)								
Girders	136.65	1674.0		1.270	-2125.9			
Diaphragm	17.13	209.9		1.270	-266.5			
Precast plank	16.89	206.9		1.270	-262.7			
Deck slab	103.68	1270.0		1.270	-1612.9			
Total		3360.7			-4268.1			
Stage2 (DW)								
Pavement	35.98	397.6		1.270	-504.9			
Parapet + railing		391.1		1.270	-496.6			
Lighting post + mis.		33.0		1.270	-41.9			
Total		821.6			-1043.5			

Right side Span

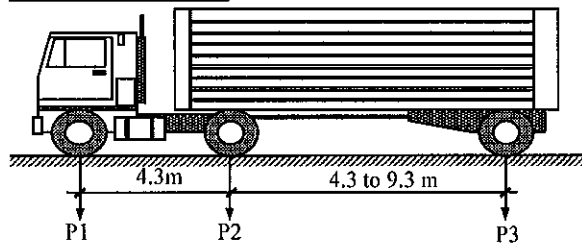
Item	Volume (m3)	Vertical F_v (kN)	Longitudinal			Transversal		
			F_{HX} (kN)	Arm. _{HX} (m)	M_y (kN•m)	F_{HY} (kN)	Arm. _{HY} (m)	M_x (kN•m)
Stage1 (DC)								
Girders	136.65	1674.0		1.270	2125.9			
Diaphragm	17.13	209.9		1.270	266.5			
Precast plank	16.89	206.9		1.270	262.7			
Deck slab	103.68	1270.0		1.270	1612.9			
Total		3360.7			4268.1			
Stage2 (DW)								
Pavement	35.98	397.6		1.270	504.9			
Parapet + railing		391.1		1.270	496.6			
Lighting post + mis.		33.0		1.270	41.9			
Total		821.6			1043.5			

2. Live Load

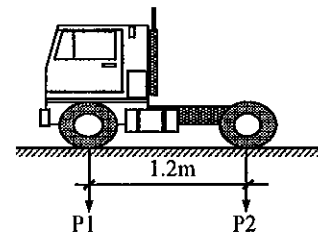
Live load data

Design Truck	P1	35.0KN	V1	4.3 m
	P2	145.0KN	V2	4.3 m
	P3	145.0KN		
Design Tandem	P1	110.0KN	V3	1.2 m
	P2	110.0KN		
Design Lane Load	P_L	9.3 kN/m		
Pedestrian Load	P_p	0.0		
Sidewalk width - both 2 sides	sw	0.0		
Maximum number of design lane	n_{lanes}	3.0		
Multiple presence factor	m	0.85		
Dynamic load allowance		(1+IM)		
Deck joint - all limit states		1.75		
Other structure - all limit states (except fatigue)		1.25		

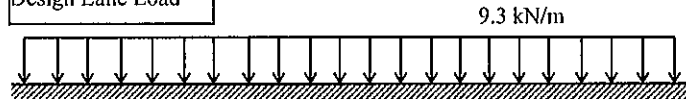
Design Truck



Design Tandem

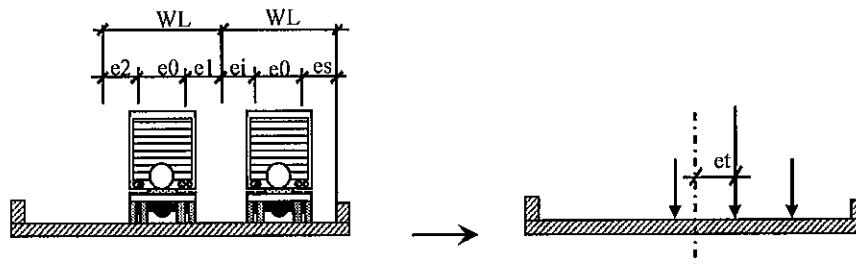


Design Lane Load



Transverse Distribution

Distance from wheel axis to inner face of curb and between wheel axis					
In general case	e_i	1.20 m	e_s	0.60	m
For deck overhang design	e_i	1.30 m	e_s	0.50	m
Distance between wheels			e_0	1.80	m
Design lane width			WL	3.60	m
			e_l	0.00	m
			e_2	1.80	m
Curb width			wc	0.50	m
Transverse excentricity of design vehicle 1 - general case			e_{x1}	4.38	
Transverse excentricity of design vehicle 2			e_{x2}	2.58	
Transverse excentricity of design vehicle 3			e_{x3}	0.78	
Transverse excentricity of design vehicle 4			e_{x4}	-1.03	
Transverse Excentricity of design vehicle			e_t	1.68	m



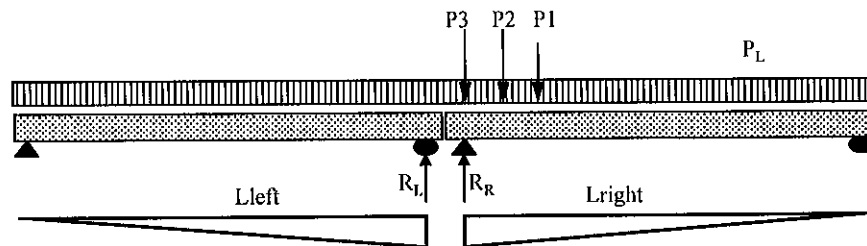
Longitudinal Distribution

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

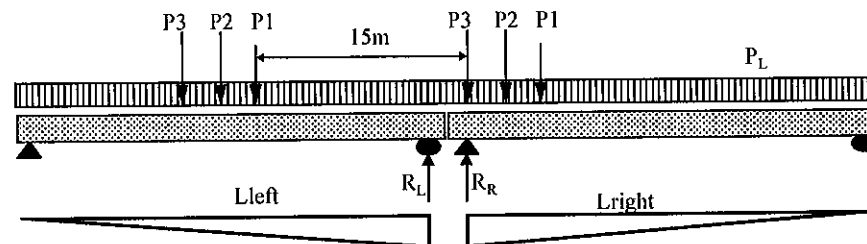
Influence line value								
Case	Left span				Right span			
	P3	P2	P1	Aleft	P3	P2	P1	Aright
Case1a:				16.05	1.00	0.87	0.73	16.05
Case1b:	0.95			16.05		1.00	0.87	16.05
Case2a:*	1.00	0.87	0.73	16.05	0.26	0.13	0.00	16.05
Case2b:	0.34	0.48	0.61	16.05	1.00	0.87	0.73	16.05
Case3a:				16.05		1.00	0.96	16.05
Case3b:		1.04		16.05			1.00	16.05

* 2 Trucks in right span

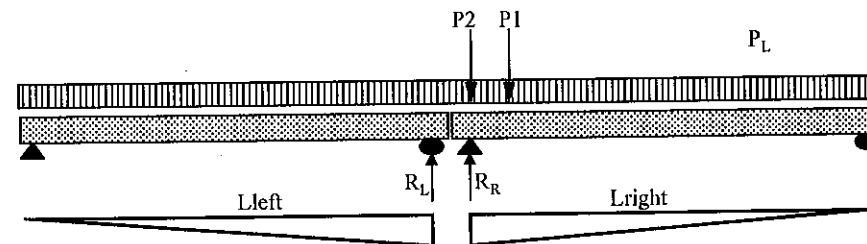
Truck + Lane load



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 _{HY}	Arm _{-HY}	M _x
	left	right	F _V						
	(kN)	(kN)	(kN)	(kN)	(m)	(kN·m)	(kN)	(m)	(kN·m)
Case1a:	179.1	623.4	802.5			564.3			3507.1
Case1b:	384.7	442.1	826.8			72.9			3613.0
Case2a:	161.2	638.5	799.7			606.2			3494.8
Case2b:	351.0	561.1	912.1			266.8			3985.7
Case3a:	179.1	502.9	682.1			411.3			2980.6
Case3b:	351.0	344.1	695.1			-8.7			3037.7

2		Loaded Lane		m = 1.00					
Item	Reaction	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:	298.5	1039.0	1337.6			940.4			3838.8
Case1b:	641.2	736.8	1378.0			121.5			3954.8
Case2a:	268.7	1064.2	1332.9			1010.3			3825.4
Case2b:	585.0	935.1	1520.1			444.7			4362.7
Case3a:	298.5	838.2	1136.8			685.4			3262.6
Case3b:	585.0	573.5	1158.5			-14.6			3325.0

3		Loaded Lane		m = 0.85					
Item	Reaction	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:	380.6	1324.8	1705.4			1199.1			2336.4
Case1b:	817.5	939.4	1756.9			154.9			2407.0
Case2a:	342.6	1356.9	1699.4			1288.2			2328.2
Case2b:	745.8	1192.3	1938.1			567.0			2655.2
Case3a:	380.6	1068.8	1449.4			873.9			1985.7
Case3b:	745.9	731.3	1477.1			-18.6			2023.7

4		Loaded Lane		m = 0.65					
Item	Reaction	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:	388.1	1350.7	1738.8			1222.6			54.3
Case1b:	833.5	957.9	1791.4			157.9			56.0
Case2a:	349.3	1383.5	1732.7			1313.4			54.1
Case2b:	760.5	1215.7	1976.1			578.1			61.8
Case3a:	388.1	1089.7	1477.8			891.1			46.2
Case3b:	760.5	745.6	1506.1			-19.0			47.1

Item Live Load	Vertical F _V (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	1520.1	0.0	0.0	444.7	0.0	0.0	4362.7
Total at bottom of pilecap	1520.1	0.0	0.0	444.7	0.0	0.0	4362.7

Pedestrian Load									
Item	Reaction	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	9.747	0.0
Total at bottom of pilecap						0.0	11.747	0.0

4.Braking Force

Braking force data			
Axle weights of design Truck	P	325.0	kN
Number of loaded lanes	n	3.0	lanes
	m	0.85	
Br1 = 25%*(design truck)*n*m	Br1	207.19	kN
Br2 = 5%*(design truck + 9.3*Lbridge)*n*m	Br2	119.70	kN
Br = max(Br1, Br2)	Br	207.19	kN

Item	From surface (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 %	1.80							
Braking force								
Total at bottom of Column			103.6	9.747	1009.7			
Total at bottom of pilecap			103.6	11.747	1216.9			

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	Δtmax	20.0	deg
Minus temperature amplitude	Δtmin	17.0	deg
Coefficient of Thermal Expansion	α	1.08E-05	
Strain due to minus temperature	ε _T	1.84E-04	

Item	From surface (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			116.9	7.79	639.3			
Total at bottom of pilecap			116.9	9.79	873.0			

6.Creep & Shrinkage

Creep & shrinkage data

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			197.8	7.79	1081.0			
Total at bottom of pilecap			197.8	9.79	1476.5			

7. Wind on Structure

Wind loads data		$P_D = 0.0006 V^2 C_d \geq 1.8 \text{ (kN/m}^2\text{)}$	
Average elevation of deck girder upper ground or water plane level	Hele_g	7.2	m
Correct coefficient for wind zone and elevation of pier	S	1.09	
Design wind speed $V = S \cdot V_b$	V	57.8	m/s
Overall width between handrails	b	12.8	m
Superstructure height including solid parapet	d	3.03	m
	b/d	4.21	
Obstacle coefficient for pier	Cd	1.36	
Wind pressure on pier	P _D	2.72	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		68.1	9.1	623.2	272.4	9.1	2492.7
Total at bottom of Column			68.1		623.2	272.4		2492.7
Superstructure	3.03		68.1	11.1	759.4	272.4	11.1	3037.6
Total at bottom of pilecap			68.1		759.4	272.4		3037.6

8. Wind on Vehicle

For strength combination III, wind load on vehicular and on structure have to simultaneously consider (wind speed 25m/s). Wind load on vehicular in 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		24.8	11.5	286.2	49.6	11.5	572.4
Total at bottom of Column		24.8		286.2	49.6		572.4
Superstructure		24.8	13.5	335.8	49.6	13.5	671.6
Total at bottom of pilecap		24.8		335.8	49.6		671.6

9. Earth Quake

Earth Quake data		
Acceleration coefficient	A	0.0580 g
Seismic zone	Sz	1
Soil profile type: according to geological data survey		1
Coefficient site	S	1.00
Bridge importance category: "1:critical"; "2:essential"; "3:other"	IC	2 essential
Response Modification Factor		
Column		2.0
Connection		1.0
Foundation		1.0

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

Item	Vertical F _V	Longitudinal			Transversal		
		F _{HX}	Arm. _{HX}	M _y	F _{HY}	Arm. _{HY}	M _x
	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Total at bottom of Column		693		3746	608		3404
Total at bottom of pilecap		693		5132	608		4619

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		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	6721		0		
2.Stage2 - Pavc.+Parapet+Railing+Mis.	DW	1643		0		
3.Live Load	LL	1520		445		4363
4.Pedestrian	LL	0		0		0
5.Centrifugal force	CE				0	0
6.Braking force	BR		104	1010		
7.Uniform temperature	TU		117	639		
8.Creep and Shrinkage	CR&SH		198	1081		
9.Wind pressure on superstructure	WS		68	623	272	2493
10.Wind pressure on vehicles	WL		25	286	50	572
11.Earthquake						
a - Longitudinal direction	EQ		346	1873		
b - Transverse direction	EQ				304	1702
Substructure Loads						
1.Pier selfweight	DC	2716				
2.Soil on pile cap	EV	0				
3.Bouyancy on pier						
a - Maximum water level	WA	-216				
b - Minimum water level	WA	-24				
c - Average annual water level	WA	-120				
4.Stream pressure						
a - Maximum water level	WA		0	0	7	21
b - Minimum water level	WA		0	0	0	0
c - Average annual water level	WA		0	0	3	3
5.Wind pressure						
a - Maximum water level	WS		1	7	29	176
b - Minimum water level	WS		27	71	55	241
c - Average annual water level	WS		14	52	42	221
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

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Loads at Bottom of Pilecap

Loads	Sign	F _V (kN)	Longitudinal		Transvesal	
			F _{Hx} (kN)	My (kN•m)	F _{Hv} (kN)	Mx (kN•m)
Superstructure Loads						
1.Stage1 - Dead Load	DC	6721		0		
2.Stage2 - Pave.+Parapet+Railing+Mis.	DW	1643		0		
3.Live Load	LL	1520		445		4363
4.Pedestrian	LL	0		0		0
5.Centrifugal force	CE				0	0
6.Braking force	BR		104	1217		
7.Uniform temperature	TU		117	873		
8.Creep and Shrinkage	CR&SH		198	1476		
9.Wind pressure on superstructure	WS		68	759	272	3038
10.Wind pressure on vehicles	WL		25	336	50	672
11.Earthquake						
a - Longitudinal direction	EQ		693	5132		
b - Transverse direction	EQ				608	4619
Substructure Loads						
1.Pier selfweight	DC	4676				
2.Soil on pile cap	EV	1553				
3.Bouyancy on pier						
a - Maximum water level	WA	-1001				
b - Minimum water level	WA	-809				
c - Average annual water level	WA	-905				
4.Stream pressure						
a - Maximum water level	WA		0	0	7	31
b - Minimum water level	WA		0	0	0	0
c - Average annual water level	WA		0	0	3	9
5.Wind pressure						
a - Maximum water level	WS		1	9	29	234
b - Minimum water level	WS		27	126	55	350
c - Average annual water level	WS		14	80	42	305
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

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Load Factors and Load Combinations

Loads	Sign	Str1a 2	Str1b 3	Str2a 4	Str2b 5	Str3a 6	Str3b 7
Superstructure Loads							
1.Stage1 - Dead Load	DC	1.25	0.90	1.25	0.90	1.25	0.90
2.Stage2 - Pave.+Mis.	DW	1.50	0.65	1.50	0.65	1.50	0.65
3.Live Load	LL	1.75	1.75	-	-	1.35	1.35
4.Pedestrian	LL	1.75	1.75	-	-	1.35	1.35
5.Centrifugal force	CE	1.75	1.75	-	-	1.35	1.35
6.Braking force	BR	1.75	1.75	-	-	1.35	1.35
7.Uniform temperature	TU	0.50	0.50	0.50	0.50	0.50	0.50
8.Creep and Shrinkage	CR&SH	0.50	0.50	0.50	0.50	0.50	0.50
9.Wind pressure on superst.	WS	-	-	1.40	1.40	0.40	0.40
10.Wind pressure on vehicles	WL	-	-	-	-	1.00	1.00
11.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	-	-	-	-	-	-

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		Check	-		
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Load Factors and Load Combinations

Loads	Sign	Ser1 10	Ext1a 11	Ext1b 12	Ext1c 13	Ext1d 14	
Superstructure Loads							
1.Stage1 - Dead Load	DC	1.00	1.25	0.90	1.25	0.90	
2.Stage2 - Pavc.+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	-	-	-	-	-	

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Load Factors and Load Combinations

Loads	Sign	Ext2a 15	Ext2b 16	Ext2c 17	Ext2d 18		
Superstructure Loads							
1.Stage1 - Dead Load	DC	1.25	0.90	1.25	0.90		
2.Stage2 - Pave.+Mis.	DW	1.50	0.65	1.50	0.65		
3.Live Load	LL	0.50	0.50	0.50	0.50		
4.Pedestrian	LL	0.50	0.50	0.50	0.50		
5.Centrifugal force	CE	0.50	0.50	0.50	0.50		
6.Braking force	BR	0.50	0.50	0.50	0.50		
7.Uniform temperature	TU	-	-	-	-		
8.Creep and Shrinkage	CR&SH	-	-	-	-		
9.Wind pressure on superst.	WS	-	-	-	-		
10.Wind pressure on vehicles	WL	-	-	-	-		
11.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		

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II. LOAD COMBINATIONS RESULT

Load Combinations at Bottom of PierColumn

No	Combinations	Sign	F _V (kN)	Longitudinal		Transvesal	
				F _{HX} (kN)	My (kN•m)	F _{HY} (kN)	Mx (kN•m)
1	Strength 1a	Str1a	16897	339	3405	0	7635
2	Strength 1b	Str1b	12006	339	3405	7	7656
3	Strength 2a	Str2a	14237	291	1832	458	3827
4	Strength 2b	Str2b	9346	255	1742	429	3758
5	Strength 3a	Str3a	16289	360	3388	181	7555
6	Strength 3b	Str3b	11398	350	3362	177	7551
7	Service 1	Ser1	12576	472	3669	148	5755
8	Extreme 1a EQL	Ext1a	14901	398	2600	94	2695
9	Extreme 1b EQL	Ext1b	10202	398	2600	94	2695
10	Extreme 1c EQT	Ext1c	14901	156	1289	307	3887
11	Extreme 1d EQT	Ext1d	10202	156	1289	307	3887

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	20659	339	4083	0	7635
2	Strength 1b	Str1b	14383	339	4083	7	7666
3	Strength 2a	Str2a	17999	291	2414	458	4743
4	Strength 2b	Str2b	11722	255	2251	429	4611
5	Strength 3a	Str3a	20051	360	4108	181	7917
6	Strength 3b	Str3b	13775	350	4061	177	7901
7	Service 1	Ser1	15304	472	4612	148	6051
8	Extreme 1a EQL	Ext1a	18663	745	5963	185	3576
9	Extreme 1b EQL	Ext1b	12578	745	5963	185	3576
10	Extreme 1c EQT	Ext1c	18663	260	2370	611	6810
11	Extreme 1d EQT	Ext1d	12578	260	2370	611	6810

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D.COLUMN DESIGN

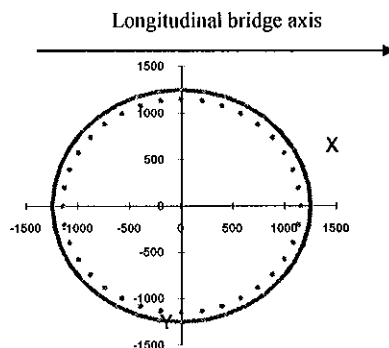
I. COLUMN DATA

1. Load Combinations at Bottom of Pier Column

No	Combinations	Sign	F _V (kN)	Longitudinal		Transvesal	
				F _{HX} (kN)	M _y (kN•m)	F _{HY} (kN)	M _x (kN•m)
1	Strength 1a	Str1a	16897	339	3405	0	7635
2	Strength 1b	Str1b	12006	339	3405	7	7656
3	Strength 2a	Str2a	14237	291	1832	458	3827
4	Strength 2b	Str2b	9346	255	1742	429	3758
5	Strength 3a	Str3a	16289	360	3388	181	7555
6	Strength 3b	Str3b	11398	350	3362	177	7551
7	Service I	SerI	12576	472	3669	148	5755
8	Extreme 1a EQL	Ext1a	14901	398	2600	94	2695
9	Extreme 1b EQL	Ext1b	10202	398	2600	94	2695
10	Extreme 1c EQT	Ext1c	14901	156	1289	307	3887
11	Extreme 1d EQT	Ext1d	10202	156	1289	307	3887

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



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3. Pier Column Section

Pier column thickness - longitudinal dimension	td	2.50	m
Pier column width - transverse dimension	tn	2.50	m
Section area	A	4.909	m ²
Moment inertia	Ix	1.896	m ⁴
	Iy	1.917	m ⁴
Radius of gyration of gross concrete section; $r = \sqrt{I/A}$	rx	0.621	m
	ry	0.625	m

4. Slenderness Effect

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

Transverse direction: Fixed at bottom; translation free, rotation free at top	Kt	2.10	
Longitudinal direction: Fixed at bottom; translation free, rotation free at top	Kl	2.10	
Unsupported length from top to bottom of column	Lu	7.64	m
Slenderness ratio: if $K.Lu / r > 22$ than considered	Kt.Lu/rx	25.8	yes
	Kl.Lu/ry	25.7	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 determination procedure:

Initial	Determining displacement for gross cross section	$\Delta_{xg} = F_x \cdot H^3 / (3.E.I_g)$
	Displacement for cracked section	$\Delta_{xcr} = F_{cr} \cdot \Delta_{xg}$
	Moment P- Δ	$M_{P-\Delta} = \Delta_{xcr} \cdot P$
	Added lateral force	$\Delta F_x = M_{P-\Delta} / H$
Step: i st	Determining displacement for gross cross section	$\Delta_{xg\ i} = (F_x + \Delta F_x\ i-1) \cdot H^3 / (3.E.I_g)$
	Displacement for cracked section	$\Delta_{xcr\ i} = F_{cr} \cdot \Delta_{xg\ i}$
	Moment P- Δ	$M_{P-\Delta\ i} = \Delta_{xcr\ i} \cdot P$
	Added lateral force	$\Delta F_x\ i = M_{P-\Delta\ i} / H$

Combination	Fv (kN)	My (kNm)	Initial					
			Fx (kN)	Δ_{xg} (m)	Fcr (kN)	Δ_{xcr} (m)	M P- Δ (kNm)	ΔF_x (kN)
Strength 1a	16897	3405	446	0.001	2.5	0.003	53	6.9
Strength 1b	12006	3405	446	0.001	2.5	0.003	37	4.9
Strength 2a	14237	1832	240	0.001	2.5	0.002	24	3.1
Strength 2b	9346	1742	228	0.001	2.5	0.002	15	2.0
Strength 3a	16289	3388	444	0.001	2.5	0.003	50	6.6
Strength 3b	11398	3362	440	0.001	2.5	0.003	35	4.6
Service I	12576	3669	481	0.001	2.5	0.003	42	5.5
Extreme 1a	14901	2600	341	0.001	2.5	0.002	35	4.6
Extreme 1b	10202	2600	341	0.001	2.5	0.002	24	3.2
Extreme 1c	14901	1289	169	0.000	2.5	0.001	18	2.3
Extreme 1d	10202	1289	169	0.000	2.5	0.001	12	1.6

Combination	Fv (kN)	My (kNm)	1st Trial					
			Fx (kN)	Δ_{xg} (m)	Fcr (kN)	Δ_{xcr} (m)	M P- Δ (kNm)	ΔF_x (kN)
Strength 1a	16897	3405	453	0.001	2.5	0.003	53	7.0
Strength 1b	12006	3405	451	0.001	2.5	0.003	38	5.0
Strength 2a	14237	1832	243	0.001	2.5	0.002	24	3.2
Strength 2b	9346	1742	230	0.001	2.5	0.002	15	2.0
Strength 3a	16289	3388	450	0.001	2.5	0.003	51	6.7
Strength 3b	11398	3362	445	0.001	2.5	0.003	35	4.6
Service I	12576	3669	486	0.001	2.5	0.003	43	5.6
Extreme 1a	14901	2600	345	0.001	2.5	0.002	36	4.7
Extreme 1b	10202	2600	344	0.001	2.5	0.002	25	3.2
Extreme 1c	14901	1289	171	0.000	2.5	0.001	18	2.3
Extreme 1d	10202	1289	170	0.000	2.5	0.001	12	1.6

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Combination	Fv (kN)	My (kNm)	2nd Trial					
			Fx (kN)	Δxg (m)	Fcr (kN)	Δxcr (m)	M P-Δ (kNm)	ΔFx (kN)
Strength 1a	16897	3405	453	0.001	2.5	0.003	53	7.0
Strength 1b	12006	3405	451	0.001	2.5	0.003	38	5.0
Strength 2a	14237	1832	243	0.001	2.5	0.002	24	3.2
Strength 2b	9346	1742	230	0.001	2.5	0.002	15	2.0
Strength 3a	16289	3388	450	0.001	2.5	0.003	51	6.7
Strength 3b	11398	3362	445	0.001	2.5	0.003	35	4.6
Service I	12576	3669	486	0.001	2.5	0.003	43	5.6
Extreme 1a	14901	2600	345	0.001	2.5	0.002	36	4.7
Extreme 1b	10202	2600	344	0.001	2.5	0.002	25	3.2
Extreme 1c	14901	1289	171	0.000	2.5	0.001	18	2.3
Extreme 1d	10202	1289	170	0.000	2.5	0.001	12	1.6

****Transverse Direction**

Combination	Fv (kN)	Mx (kNm)	Initial					
			Fx (kN)	Δxg (m)	Fcr (kN)	Δxcr (m)	M P-Δ (kNm)	ΔFx (kN)
Strength 1a	16897	7635	1000	0.003	2.5	0.007	119	15.6
Strength 1b	12006	7656	1003	0.003	2.5	0.007	85	11.1
Strength 2a	14237	3827	501	0.001	2.5	0.004	50	6.6
Strength 2b	9346	3758	492	0.001	2.5	0.003	33	4.3
Strength 3a	16289	7555	989	0.003	2.5	0.007	114	14.9
Strength 3b	11398	7551	989	0.003	2.5	0.007	80	10.4
Service I	12576	5755	754	0.002	2.5	0.005	67	8.8
Extreme 1a	14901	2695	353	0.001	2.5	0.002	37	4.9
Extreme 1b	10202	2695	353	0.001	2.5	0.002	25	3.3
Extreme 1c	14901	3887	509	0.001	2.5	0.004	54	7.0
Extreme 1d	10202	3887	509	0.001	2.5	0.004	37	4.8

Combination	Fv (kN)	Mx (kNm)	1st Trial					
			Fx (kN)	Δxg (m)	Fcr (kN)	Δxcr (m)	M P-Δ (kNm)	ΔFx (kN)
Strength 1a	16897	7635	1015	0.003	2.5	0.007	121	15.9
Strength 1b	12006	7656	1014	0.003	2.5	0.007	86	11.3
Strength 2a	14237	3827	508	0.001	2.5	0.004	51	6.7
Strength 2b	9346	3758	496	0.001	2.5	0.004	33	4.3
Strength 3a	16289	7555	1004	0.003	2.5	0.007	116	15.1
Strength 3b	11398	7551	999	0.003	2.5	0.007	81	10.5
Service I	12576	5755	762	0.002	2.5	0.005	68	8.9
Extreme 1a	14901	2695	358	0.001	2.5	0.003	38	4.9
Extreme 1b	10202	2695	356	0.001	2.5	0.003	26	3.4
Extreme 1c	14901	3887	516	0.001	2.5	0.004	54	7.1
Extreme 1d	10202	3887	514	0.001	2.5	0.004	37	4.9

Combination	Fv (kN)	Mx (kNm)	2nd Trial					
			Fx (kN)	Δxg (m)	Fcr (kN)	Δxcr (m)	M P-Δ (kNm)	ΔFx (kN)
Strength 1a	16897	7635	1016	0.003	2.5	0.007	121	15.9
Strength 1b	12006	7656	1014	0.003	2.5	0.007	86	11.3
Strength 2a	14237	3827	508	0.001	2.5	0.004	51	6.7
Strength 2b	9346	3758	496	0.001	2.5	0.004	33	4.3
Strength 3a	16289	7555	1005	0.003	2.5	0.007	116	15.1
Strength 3b	11398	7551	999	0.003	2.5	0.007	81	10.5
Service I	12576	5755	763	0.002	2.5	0.005	68	8.9
Extreme 1a	14901	2695	358	0.001	2.5	0.003	38	4.9
Extreme 1b	10202	2695	356	0.001	2.5	0.003	26	3.4
Extreme 1c	14901	3887	516	0.001	2.5	0.004	54	7.1
Extreme 1d	10202	3887	514	0.001	2.5	0.004	37	4.9

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

Combination	Fv Vert. (kN)	Mx Trans. (kNm)	Mx P-Δ (kNm)	Mx Total (kNm)	My Long. (kNm)	My P-Δ (kNm)	My Total (kNm)
Strength 1a	16897	7635	121	7756	3405	53	3459
Strength 1b	12006	7656	86	7742	3405	38	3443
Strength 2a	14237	3827	51	3878	1832	24	1856
Strength 2b	9346	3758	33	3790	1742	15	1757
Strength 3a	16289	7555	116	7671	3388	51	3439
Strength 3b	11398	7551	81	7631	3362	35	3397
Service I	12576	5755	68	5823	3669	43	3712
Extreme 1a	14901	2695	38	2733	2600	36	2636
Extreme 1b	10202	2695	26	2721	2600	25	2625
Extreme 1c	14901	3887	54	3941	1289	18	1307
Extreme 1d	10202	3887	37	3924	1289	12	1301

II. PIER COLUMN DESIGN

1. Limit of Reinforcement

S.5.7.4.2

Minimum area of longitudinal reinforcement in column					
As.fy / (Ag . fc) >= 0.135			As ≥	0.050	m2
As / Ag >= 0.01			As ≥	0.049	m2
Maximum area of longitudinal reinforcement in column					
As / Ag <= 0.08			As ≤	0.393	m2
Trial Rebars:				As	0.031 m2
Layers	x 50	= 50 bars	D28 @150 As1	0.031	m2
Layers	x 0	= 0 bars	D25 @150 As2	0.000	m2

2. Interaction diagram M-P

Using Pca-Column software

****In Both Direction**

Strength and Service limit states:

Resistance factor:	Compression	$\phi_c = 0.75$ (AASHTO LRFD-2004)
	Tension	$\phi_t = 0.90$

Extreme Event limit states:

Resistance factor	Compression	$\phi_c = 1.00$ (AASHTO LRFD-2004)
	Tension	$\phi_t = 1.00$

No.	COMBINATION	Pu kN	Mux kN-m	Muy kN-m	ϕ Mnx kN-m	ϕ Mny kN-m	ϕ Mn/Mu
1	Strength 1a	16897	3459	7756	8591	19267.1	2.49
2	Strength 1b	12006	3443	7742	7947.7	17869.5	2.31
3	Strength 2a	14237	1856	3878	8912.6	18622.8	4.81
4	Strength 2b	9346	1757	3790	7575.1	16345.1	4.31
5	Strength 3a	16289	3439	7671	8663	19327.3	2.52
6	Strength 3b	11398	3397	7631	7817.2	17561.1	2.30
7	Service I	12576	3712	5823	10676.8	16747.5	2.88
8	Extreme 1a	14901	2636	2733	15625.1	16200.3	5.93
9	Extreme 1b	10202	2625	2721	13838.2	14344.5	5.27
10	Extreme 1c	14901	1307	3941	7094.2	21396.7	5.43
11	Extreme 1d	10202	1301	3924	6284.5	18945.2	4.83

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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	4.531	m2
Tie diameter	Dtie	16	mm2
Cross section area of 1 tie	As-tr	0.0002	m2
Spacing of hoops	s	150	mm
Length of reinforcement tie in 1 hoop	Ltie	7.23	m
Ratio of ties reinf. in one hoop/ volume of conc. core one pitch spacing			
$ps = As-tr \cdot Ltie / (Ac \cdot spacing)$	ps	0.0021	
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	0.0028	N/A
			S.5.10.11.3
Transverse Reinforcement for Confinement at Plastic Hinges			S.5.10.11.4.1.d
For a circular column "1:applied", "2:Not applied"		2	
$ps \geq 0.12 \cdot fc / fy = Req2$	Req2	0.0090	N/A
For a rectangular column			
Rectangular hoop reinforcement shall satisfy			
Either $As_h \geq 0.30 \cdot s \cdot hc \cdot fc / fy$, $[Ag/Ac - 1] = Req1$			
or $As_h \geq 0.12 \cdot s \cdot hc \cdot fc / fy = Req2$			
In longitudinal direction "1:applied", "2:Not applied"		2	
Number of cross tie	nt_x	4	ties
Total cross-sectional area of tie reinf.	As_h_x	0.0008	m2
Core dimension of tied column	hc_x	2.40	m
Rectangular hoop reinforcement shall satisfy	Req1_x	0.0007	m2
	Req2_x	0.0032	m2
	Conclude		N/A
In transverse direction			
Number of cross tie	nt_y	4	ties
Total cross-sectional area of tie reinf.	As_h_y	0.0008	m2
Core dimension of tied column	hc_y	2.40	m
Rectangular hoop reinforcement shall satisfy	Req1_y	0.0007	m2
	Req2_y	0.0032	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	2.50	m
1/6 of clear height of column	L2	0.78	m
or 450mm	L3	0.45	m
Chosen value: $L = \max(L1, L2, L3)$	L	2.50	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	1.25	m
or 380mm	L5	0.38	m
Chosen value: $Le = \max(L4, L5)$	Le	1.25	m

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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	472	458	kN
Required shear capacity $V_n = V_u / \phi_v$	V_n	472	458	kN
Determine concrete shear capacity				
Minimum shear reinforcement will provided in cross section				
Therefore	β	2.0	2.0	
	θ	45.0	45.0	
Cross section equivalent height	h	2.50	2.50	m
width	b	1.96	1.96	m
$d = h - \text{cover} - d_{lx}$	d	2.41	2.41	m
$d_v = \max(0.72 \cdot h; 0.9 \cdot d)$	d_v	2.17	2.17	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	3879	3879	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				
Stirrup diameter	D_s	16	16	
Number of stirrup legs / cross section	n_s	2	2	
Shear legs area	A_v	0.0004	0.0004	m ²
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	0.0002	0.0002	m ²
Check maximum shear reinforcement spacing requirement		OK	OK	
$F = 0.1 \cdot f_c \cdot b_v \cdot d_v$	F	12798	12798	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	0.60	m

Interface shear transfer

S.5.8.4

Area of concrete engaged in shear transfer	A_{cv}	4.909	m ²
Area of shear reinforcement crossing the shear plane	A_{vf}	0.031	m ²
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			
$V_n = c \cdot A_{cv} + \mu \cdot A_{vf} \cdot f_y$	V_n	15756	kN
$V_n \leq 0.2 \cdot f_c \cdot A_{cv}$	$V_n \leq$	29452	kN
$V_n \leq 5.5 \cdot A_{cv} \cdot (1\text{MPa})$	$V_n \leq$	26998	kN
Normal shear resistance	V_n	15756	kN
Factor for shear friction		1.0	
Factored shear resistance	V_r	15756	kN
Horizontal force at bottom of pier column	V_u	52	kN
	Conclude		OK

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5. Control of cracking by distribution of reinforcement

Tensile stress in rebars should be satisfied equation: $f_s \leq f_{sa} = Z/[I/(dc.A)^{1/3}]$ and $f_s \leq 0.6.f_y$				
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	3712	5823	kNm
Axial thrust at service limit state	Ns	12576	12576	kN
Cross section equivalent	height	h	2.50	m
	width	b	1.96	m
Concrete thickness from tension fiber to tension reinf.	dc	0.05	0.05	m
Concrete thickness from compression fiber to tension reinf.	d	2.41	2.41	kN
Number of rebars	N	25	25	bars
Area of rebars	As	0.0154	0.0154	m2
Area of concrete assumed to participate with reinf.				
$A = 2 \cdot dc \cdot b / N$	A	0.0079	0.0079	m2
	f _{sa}	410	410	MPa
	0.6f _y	240	240	MPa
Min (f _{sa} , 0.6f _y) = f _{sl}	f _{sl}	240	240	MPa
$e = Ms/Ns+d-h/2$	e	1.46	1.63	m
$e/d > 1.15$	e/d	1.15	1.15	
$j = 0.74 + 0.1(e/d) \leq 0.9$	j	0.86	0.86	
$i = 1/(1-j.d/e)$	i	3.90	3.90	
Stress in rebars: $f_s = (Ms+Ns(d-h/2))/(As.j.i.d)$	f _s	148	165	MPa
	Conclude	OK	OK	
Maximum width of crack: $a_n = 0.076 \cdot \beta .f_s.(dc.A)^{1/3}$	a _n	0.137	0.153	mm
Where	β	0.167	0.167	

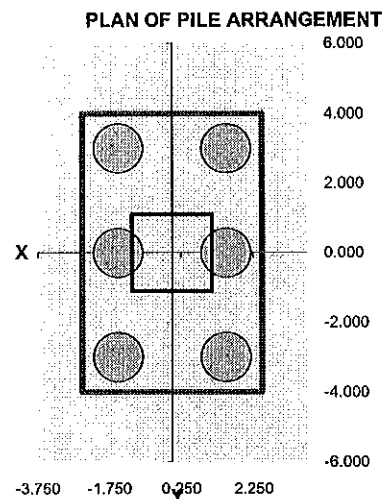
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E. PILE CAP DESIGN

1. PILES AND PILECAP DATA

Piles Data			
Number of piles: n_p			6
Co-ordinates of Piles			
Number	Diameter	X_i (m)	Y_i (m)
1	1.000	1.500	3.000
2	1.000	1.500	0.000
3	1.000	1.500	-3.000
4	1.000	-1.500	-3.000
5	1.000	-1.500	0.000
6	1.000	-1.500	3.000
-			
-			
-			
-			
-			
-			
$\sum X_i^2, \sum Y_i^2$		13.500	36.000

Pilecap Data	
X_i	Y_i
2.500	4.000
2.500	-4.000
-2.500	-4.000
-2.500	4.000
2.500	4.000
Column Data	
X_i	Y_i
1.108	1.108
1.108	-1.108
-1.108	-1.108
-1.108	1.108
1.108	1.108

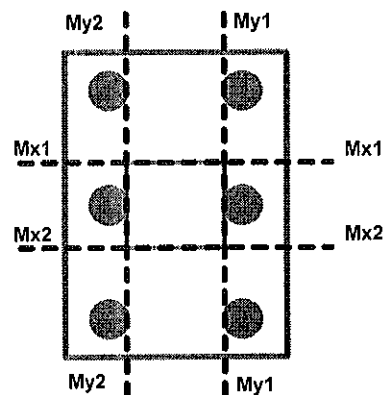


2. CRITICAL SECTIONS

Pile cap Length	L	=	8.000 m
Pile cap Width	W	=	5.000 m
Pile cap thickness	H	=	2.000 m
Column Width	w_c	=	2.500 m
Column Thickness	t_c	=	2.500 m
Round nose radius	c_1	=	1.250 m

Column Area	A_c	=	4.909 m ²
Column block equivalent:			
Width	w_{cb}	=	2.216 m
Thickness	t_{cb}	=	2.216 m

Distance from Pile to Critical Sections - Arm (m)				
Pile No.	Section			
	Mx1	Mx2	My1	My2
1	1.892	-	0.392	-
2	-	-	0.392	-
3	-	1.892	0.392	-
4	-	1.892	-	0.392
5	-	-	-	0.392
6	1.892	-	-	0.392
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-
-	-	-	-	-



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3. INTERNAL FORCES CALCULATION

3.1. Pile Reaction (from Pile Foundation analysis)

AXIAL FORCE (KN)											
Pile	STR I-A	STR I-B	STR II-A	STR II-B	STR III-A	STR III-B	SER	EXT I-A	EXT I-B	EXT II-A	EXT II-B
1	4468.1	3447.1	3720.2	2652.5	4437.7	3403.5	3585.7	4148.1	3155.0	4016.8	3023.8
2	3860.4	2835.4	3240.2	2189.5	3767.3	2735.0	3070.9	3822.0	2829.0	3338.2	2345.2
3	3252.7	2223.7	2760.1	1726.5	3096.9	2066.5	2556.3	3496.0	2502.9	2659.6	1666.6
4	2267.9	1238.8	2128.9	1146.7	2095.4	1079.8	1395.6	1922.6	929.5	2053.7	1060.8
5	2875.5	1850.6	2609.0	1609.7	2765.8	1748.2	1910.2	2248.6	1255.5	2732.4	1739.3
6	3483.1	2462.2	3089.0	2072.8	3436.3	2416.7	2424.8	2574.6	1581.6	3410.9	2417.9
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-

BENDING MOMENT - Mx (KNm)											
Pile	STR I-A	STR I-B	STR II-A	STR II-B	STR III-A	STR III-B	SER	EXT I-A	EXT I-B	EXT II-A	EXT II-B
1	-56.3	-53.4	171.9	159.7	23.1	21.7	22.1	57.2	57.2	225.5	225.5
2	56.3	53.4	-171.9	-159.7	-23.1	-21.7	-22.1	-57.2	-57.2	-225.5	-225.5
3	56.3	53.4	-171.9	-159.7	-23.1	-21.7	-22.1	-57.2	-57.2	-225.5	-225.5
4	56.3	53.4	-171.9	-159.7	-23.1	-21.7	-22.1	-57.2	-57.2	-225.5	-225.5
5	56.3	53.4	-171.9	-159.7	-23.1	-21.7	-22.1	-57.2	-57.2	-225.5	-225.5
6	-56.3	-53.4	171.9	159.7	23.1	21.7	22.1	57.2	57.2	225.5	225.5
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-

BENDING MOMENT - My (KNm)											
Pile	STR I-A	STR I-B	STR II-A	STR II-B	STR III-A	STR III-B	SER	EXT I-A	EXT I-B	EXT II-A	EXT II-B
1	62.3	62.3	74.2	62.3	70.7	67.3	107.1	194.0	194.0	62.1	62.1
2	62.3	62.3	74.2	62.3	70.7	67.3	107.1	194.0	194.0	62.1	62.1
3	62.3	62.3	74.2	62.3	70.7	67.3	107.1	194.0	194.0	62.1	62.1
4	-62.3	-62.3	-74.2	-62.3	-70.7	-67.3	-107.1	-194.0	-194.0	-62.1	-62.1
5	-62.3	-62.3	-74.2	-62.3	-70.7	-67.3	-107.1	-194.0	-194.0	-62.1	-62.1
6	-62.3	-62.3	-74.2	-62.3	-70.7	-67.3	-107.1	-194.0	-194.0	-62.1	-62.1
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-

3.2. Pilecap weight

Section	V (m3)	Ni (KN)	Arm (m)	Mx (KNm)	My (KNm)
Mx1	28.9	-419.4	1.446	-606.5	
Mx2	28.9	-419.4	1.446	-606.5	
My1	22.3	-323.0	0.696		-224.8
My2	22.3	-323.0	0.696		-224.8

LOAD FACTOR FOR DEAD LOAD OF PILE CAP										
STR I-A	STR I-B	STR II-A	STR II-B	STR III-A	STR III-B	SER	EXT I-A	EXT I-B	EXT I-C	EXT I-D
1.25	0.90	1.25	0.90	1.25	0.90	1.00	1.25	0.90	1.25	0.90

3.3. Internal Forces at Critical Sections

INTERNAL FORCE AT SECTION Mx1 - Unit KN, KNm											
	STR I-A	STR I-B	STR II-A	STR II-B	STR III-A	STR III-B	SER	EXT I-A	EXT I-B	EXT I-C	EXT I-D
Shear	7427.0	5531.9	6285.0	4347.8	7349.9	5442.7	5591.1	6198.5	4359.1	6903.5	5084.3
Moment	14174.7	10529.2	12470.3	8714.9	14187.6	10510.7	10810.9	12077.1	8531.2	13747.7	10202.0

INTERNAL FORCE AT SECTION Mx2 - Unit KN, KNm											
	STR I-A	STR I-B	STR II-A	STR II-B	STR III-A	STR III-B	SER	EXT I-A	EXT I-B	EXT I-C	EXT I-D
Shear	4996.4	3085.1	4364.8	2495.7	4668.1	2768.8	3532.5	4894.3	3055.0	4189.1	2349.9
Moment	9800.7	6112.8	8149.2	4571.4	9020.8	5364.1	6827.2	9380.6	5834.7	7709.6	4164.1

INTERNAL FORCE AT SECTION My1 - Unit KN, KNm											
	STR I-A	STR I-B	STR II-A	STR II-B	STR III-A	STR III-B	SER	EXT I-A	EXT I-B	EXT I-C	EXT I-D
Shear	11177.5	8215.6	9316.9	6277.8	10898.3	7914.3	8889.9	11062.3	8196.2	9610.9	6744.9
Moment	4448.3	3321.0	3754.1	2560.9	4364.0	3217.8	3709.9	4798.1	3708.3	3833.3	2743.6

INTERNAL FORCE AT SECTION My2 - Unit KN, KNm											
	STR I-A	STR I-B	STR II-A	STR II-B	STR III-A	STR III-B	SER	EXT I-A	EXT I-B	EXT I-C	EXT I-D
Shear	8222.8	5260.9	7423.1	4538.5	7893.8	4954.0	5407.6	6342.0	3475.9	7793.3	4927.2
Moment	2915.4	1788.0	2566.2	1504.7	2761.2	1652.7	1701.5	1782.8	693.0	2747.5	1657.8

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		Design	-		
		Check	-		
		Revise	-		

4. One-way Shear capacity Check

S.5.8

Critical shear section for one-way shear is located at distance d_v from face of equivalent square column.				
Estimated distance between internal flexural force components d_v , we may take $d_v = 0.9 \cdot d_e$				
$d_e = H - \text{cover} - d_x$			d_e	1.81 m
			d_v	1.63 m
Direction		Long.- X	Trans.-Y	Unit
Shear resistance factors	ϕ_v	0.9	0.9	
Factored shear force in longitudinal	V_u	7427	11177	kN
Required shear capacity $V_n = V_u / \phi_v$	V_n	8252	12419	kN
Determine concrete shear capacity				
Minimum shear reinforcement will provided in cross section				
Therefore	β	2.0	2.0	
	θ	45.0	45.0	
Cross section	height	h	2.00	2.00 m
	width	b	5.00	8.00 m
$d = h - \text{cover} - d_{lx}$	d	1.81	1.81	m
$d_v = \max(0.72 \cdot h; 0.9 \cdot d)$	d_v	1.63	1.63	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	7422	11875	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	830	544	kN
In this case $V_c > V_n$ so shear reinforcement is no need				
Stirrup diameter	D_s	16	16	
Number of stirrup legs / cross section	n_s	20	36	
Shear legs area	A_v	0.0040	0.0073	m ²
Angle of inclination of shear reinf. to long. axis	α	90	90	deg
$V_s = A_v \cdot f_y \cdot d_v \cdot (\cot \theta + \cot \alpha) \cdot \sin \alpha / s$	$s \leq$	0.24	0.65	m
Stirrup spacing used	s	0.30	0.30	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	0.0017	0.0027	m ²
Check maximum shear reinforcement spacing requirement		OK	OK	
$F = 0.1 \cdot f_c \cdot b_v \cdot d_v$	F	24489	39182	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	0.60	m

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5. Two-way Shear capacity Design

S.5.13.3.6.3

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

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

The same dimension $dv/2$ is used to check two-way shear for a corner pile.

Column v.s Pilecap

Pier Column dimensions	Longitudinal axis	td	2.50	m
	Transverse axis	tn	2.50	m
Perimeter of two-way shear				
$b0 = (td+tn)*2 + 4*dv$		b0	17.33	m
Section with transverse reinforcement				
Nominal shear resistance shall be taken as				
$V_n = V_c + V_s \leq 0.504 \cdot \sqrt{f_c} \cdot b0 \cdot dv = V_a$				
$V_c = 0.166 \cdot \sqrt{f_c} \cdot b0 \cdot dv$				
$V_s = A_v \cdot f_y \cdot dv / s$				
Shear resistance of concrete		V_c	25719	kN
Assumed stirrup diameter		D_s	16	mm
Number of stirrup legs / cross section		ns	28	
Shear legs area		A_v	0.0057	m ²
Stirrup spacing used		s	600	mm
Shear resistance of reinforcement		V_s	6156	kN
		V_a	78088	kN
		V_n	31875	kN
Maximum reaction at bottom of column		V_u	16897	kN
Resistance factor for shear		ϕ_v	0.9	
Factored shear resistance		$\phi_v \cdot V_n$	28688	kN
Punching shear check			OK	

Conner pile v.s Pilecap

Pile diameter	D	1.00	m
Radius of critical section for two-way shear $R_{co} = D/2 + dv/2$	R_{co}	1.32	m
Distance from pile center of conner pile to edge of pilecap	a1	1.00	m
Perimeter of two-way shear			
$b0 = 2*a1 + 1/4*2*pi()*R_{co}$	b0	4.07	m
Section with transverse reinforcement			
Nominal shear resistance shall be taken as			
$V_n = V_c + V_s \leq 0.504.\sqrt{f'c} . b0 . dv = V_a$			
$V_c = 0.166 . \sqrt{f'c} . b0 . dv$			
$V_s = A_v . f_y . dv / s$			
Shear resistance of concrete	V_c	6038	kN
Assumed stirrup diameter	D_s	16	mm
Number of stirrup legs / cross section	ns	13	
Shear legs area	A_v	0.0026	m2
Stirrup spacing used	s	300	mm
Shear resistance of reinforcement	V_s	5716	kN
	V_a	18332	kN
	V_n	11754	kN
Maximum reaction of conner pile	V_u	4468	kN
Resistance factor for shear	ϕ_v	0.9	
Factored shear resistance	ϕ_v*V_n	10579	kN
Punching shear check		OK	

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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	
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		7.00	

Sign	Parameters	Unit	Section			
			M _x	M _x	M _y	M _y
INTERNAL FORCES AT SECTION						
	Combination		Strength	Service	Strength	Service
Qu	Shear	kN	7427	5591	11177	8890
Mu	Flexural Moment	kNm	14138	10811	4798	3710
Nu	Axial load	kN				
Tu	Torsional Moment	kNm				
FLEXURAL MOMENT CHECKING						
H	Section height	m	2.000	2.000	2.000	2.000
d's	Dis. From comp. fiber to centroid of comp. Reinf	m	0.084	0.084	0.084	0.084
d1x	Dis. From tens. fiber to centroid of tension Reinf	m	0.163	0.163	0.160	0.160
	Cover to reinf	m	0.050	0.050	0.050	0.050
ds	Dis. From comp. fiber to centroid of tension Reinf	m	1.838	1.838	1.840	1.840
d'ps	Dis. From comp. fiber to centroid of comp. prestressing steel	m	0.000	0.000	0.000	0.000
d1xp	Dis. From tens. fiber to centroid of tension prestressing steel	m	0.000	0.000	0.000	0.000
dps	Dis. From comp. fiber to centroid of tension prestressing steel	m	2.000	2.000	2.000	2.000
b	Width of the compression face of member	m	5.000	5.000	8.000	8.000
bw	Web width or diameter of a circular section	m	5.000	5.000	8.000	8.000
hf	Compression flange depth	m	0.000	0.000	0.000	0.000
I _z	Moment of inertia of section	m ⁴	3.333	3.333	5.333	5.333
A _{mc}	Section area	m ²	10.000	10.000	16.000	16.000
	Steel choice					
A _{ps}	Tension prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0
		Number	tendons	0	0	0
		Area	m ²	0.00000	0.00000	0.00000
A' _{ps}	Compression prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0
		Number	tendons	0	0	0
		Area	m ²	0.00000	0.00000	0.00000
A _s	Tension Reinforcement	Number	bars	66	66	53
		Diameter	mm	25	25	20
		Area	m ²	0.03241	0.03241	0.01664
A' _s	Compression Reinforcement	Number	bars	66	66	53
		Diameter	mm	18	18	18
		Area	m ²	0.01676	0.01676	0.01346
A' _c	Shear reinforcement	Number	bars	16	16	26
		Diameter	mm	16	16	16
		Area	m ²	0.00323	0.00323	0.00525

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ϕ	Resistance factors for flexure	5.5.4.2	0.90	1.00	0.90	1.00
ϕ_v	Resistance factors for shear		0.90	1.00	0.90	1.00
ϕ_n	Resistance factors for axial force		1.00	1.00	1.00	1.00
β_1	Stress block factor		0.836	0.836	0.836	0.836
c	Dis. Between centroid and top fiber	m	0.059	0.059	0.007	0.007
	For T section behavior	m	0.059	0.059	0.007	0.007
	For rectangular section behavior	m	0.059	0.059	0.007	0.007
f _{pe}	Effective stress in the prestressing steel after losses	Mpa	1116	1116	1116	1116
f _{ps}	Aver. stress in pres. steel at the time for which the nominal resistance	Mpa	1844	1844	1858	1858
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	0.049	0.049	0.006	0.006
de	Corresp. effective depth from extreme comp. fiber to centroid of tensile force in the tensile reinf.	m	1.838	1.838	1.840	1.840
M _n	Nominal resistance	kNm	23102	23102	11792	11792
M _r	Factored resistance	kNm	20791	23102	10613	11792
M _u	Flexural moment	kNm	14188	10811	4798	3710
(5.7.3.2)	Flexural moment Checking		OK	OK	OK	OK
	Limits for reinforcement					
c/de	Maximum reinforcement		0.03	0.03	0.00	0.00
	Maximum reinforcement Checking	≤ 0.42	OK	OK	OK	OK
1.2*M _{cr}	Cracking moment	kNm	7110	7110	11083	11083
(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	3	3	3	3
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.063	0.063	0.060	0.060
Z	Crack width parameter	N/mm	17500	17500	17500	17500
A	Area of concr. with same centroid as tens. Reinf	m ²	0.009	0.009	0.018	0.018
f _{sa}	Value	Mpa	208	208	170	170
0.6*f _y		Mpa	240	240	240	240
	Tensile stress in reinf Min(f _{sa} , 0.6f _y)	Mpa	208	208	170	170
x	Dist. From compression fiber to centroid	m	-	0.365	-	0.217
J.d	Arm	m	-	1.716	-	1.768
I _{cr}	Moment of inertia of the cracked section	m ⁴	-	0.581	-	0.336
f _s	Tensile stress in reinforcement f _s = M _s / (A _s *J.d)	Mpa	-	194	-	126
	Checking for control cracking f _s < f _{sa}		N.a	OK	N.a	OK
(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
	Distribution on sides 10 D16	m ²	0.00202	0.00202	0.00202	0.00202
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45
	Checking		OK	OK	OK	OK

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SHEAR AND TORSION CHECKING							
β	Factor indicating diag. cracked concr. to tension		1.8	2.0	1.8	2.0	
θ	Angle of inclination of diagonal compressive	degree	42.12	39.51	42.45	40.67	
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	
bv	Effective web width as minimum web width - in dv	m	5.000	5.000	8.000	8.000	
dv	Effective shear depth	m	1.813	1.813	1.837	1.837	
	(de - a/2)	m	1.813	1.813	1.837	1.837	
s	Spacing of stirrups	m	0.300	0.300	0.300	0.300	
ncat	Amount of bars in spacing S	bars	16	16	26	26	
Av	Shear reinf area in spacing S	m ²	0.0032	0.0032	0.0053	0.0053	
β	Assume		2.0	2.0	2.0	2.0	
θ	Assume	degree	45.00	45.00	45.00	45.00	
v	Shear stress in concrete	kN/m ²	910	617	845	605	
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	
e _x	Strain in tensile reinforcement		1.78E-03	1.35E-03	1.86E-03	1.47E-03	
	if ex<0, multiple with reduce factor		-	-	-	-	
	Strain checking	$\leq 2.00E-3$	Ok	Ok	Ok	Ok	
v/fc	Ratio of shear stress and fc		0.030	0.021	0.028	0.020	
β	Final value		1.8	2.0	1.8	2.0	
θ	Final value	degree	42.12	39.51	42.45	40.67	
Vc	Nominal shear resistance provided by tensile stresses in the concret	kN	7504	8379	11916	13149	
Vs	Shear resistance provided by shear reinforcement	kN	8640	9473	14064	14968	
Vp	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0	
Vn1	Vn1=Vc+Vs+Vp	kN	16144	17852	25980	28117	
Vn2	Vn2	kN	67986	67986	110213	110213	
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	16144	17852	25980	28117	
Vr	Factored shear resistance	kN	14530	17852	23382	28117	
Vu	Shear	kN	7427	5591	11177	8890	
(5.8.2.7)	Shear checking		OK	OK	OK	OK	

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		Design	-		
		Check	-		
		Revise	-		

<Result table>

STT	Comb	Pu kN	Mux kN-m	Muy kN-m	ϕ Mnx kN-m	ϕ Mny kN-m	ϕ Mn/Mu
STRENGTH LIMIT STATES							
1	P_max	1079.8	21.7	67	498	1545	23.0
2	P_min	4468.1	56.3	62	1408	1559	25.0
3	Mx_max	3720.2	171.9	74	1921	829	11.2
4	My_max	3585.7	22.1	107	422	2043	19.1
EXTREME EVENT LIMIT STATES							
1	P_max	929	57	194	495	1684	8.7
2	P_min	4148	57	194	701	2385	12.3
3	Mx_max	4017	225	62	2373	654	10.5
4	My_max	4148	57	194	701	2385	12.3

3. Column Ties

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

Bridge is in seismic zone	Sz	1	
Area of concrete core measured out-to-out of ties	Ac	0.622	m2
Tie diameter	Dtie	14	mm2
Cross section area of 1 tie	As-tr	0.00015	m2
Spacing of hoops	s	75	mm
Length of reinforcement tie in 1 hoop	Ltie	2.78	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	0.0090	
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	0.0089	OK
Transverse Reinforcement for Confinement at Plastic Hinges			
S.5.10.11.4.1.d			
For a circular column "1:applied", "2:Not applied"		1	
$\rho_s \geq 0.12 \cdot f_c/f_y = \text{Req2}$	Req2	0.0090	N/A
Length distributed spiral with pitch 75mm below pilecap	Ldis	1.50	m

4. Shear Design

Shear resistance factors	ϕ_v	1.0	
Factored shear force	Vu	12	kN
Required shear capacity $V_n = V_u / \phi_v$	Vn	12	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
$d_v = 0.9 \cdot d_e$ $d_e = D/2 + D_r/\pi()$			
Diameter of the circle passing through the centers of the long. reinf.	Dr	0.79	m
	d_e	0.75	m
	d_v	0.68	m
$V_c = 0.083 \cdot \beta \cdot \sqrt{f_c} \cdot b_v \cdot d_v$	Vc	616	kN
	Conclude		OK

SPACE PILE FOUNDATION ANALYSIS PROGRAM
Turbo BASIC

PROJECT: : DN-QN-P3L-LRB12a

INITIA DATA

Kn = 0.13 Ax = 5.00 By = 8.00 Cz = 2.00
E v.uon = 3001028 E r.uon = 3001028 E v.nen = 3001028 E r.nen = 3001028
Mq = 100 (t/m4) Md = 0 (t/m4) m = 350 (t/m4)

LOAD COMBINATIONS

COMB.	Hx	Hy	P	Mx	My	Mz
1	34.52	0.01	2059.90	-778.27	416.16	0.00
2	34.52	0.73	1433.01	-781.40	416.16	0.00
3	29.66	46.71	1788.73	-483.53	246.07	0.00
4	25.96	43.74	1161.84	-470.06	229.47	0.00
5	36.71	18.41	1997.92	-806.99	418.73	0.00
6	35.65	18.07	1371.03	-805.38	413.99	0.00
7	48.08	15.07	1523.28	-616.80	470.17	0.00
8	75.92	18.87	1856.44	-364.53	607.84	0.00
9	75.92	18.87	1249.08	-364.53	607.84	0.00
10	26.47	62.25	1856.44	-694.15	241.64	0.00
11	26.47	62.25	1249.08	-694.15	241.64	0.00

PROPERTIES OF PILES

PILE	Lo	H	Bpx	Bpy	A	B	Cday	Fo	Io	Po	Co	Ct
1	0.00	10.00	1.350	1.350	1.00	0.000	0.000	0.785	0.098	500	30000	15000
2						n t						
3						n t						
4						n t						
5						n t						
6						n t						

PILE COORD.

PILE	X	Y	Phi	Xi
1	1.50	3.00	0.000	0.00
2	1.50	0.00	0.000	0.00
3	1.50	-3.00	0.000	0.00
4	-1.50	-3.00	0.000	0.00
5	-1.50	0.00	0.000	0.00
6	-1.50	3.00	0.000	0.00

DISPLACEMENTS

COMB.	X	Y	Z	Fix	Fiy	Fiz
1	0.00202	0.00057	0.003352	-0.000202	0.000327	-0.000000
2	0.00202	0.00059	0.002332	-0.000203	0.000327	-0.000000

3	0.00154	0.00200	0.002911	-0.000159	0.000209	-0.000000
4	0.00137	0.00188	0.001891	-0.000154	0.000192	-0.000000
5	0.00210	0.00124	0.003251	-0.000222	0.000332	-0.000000
6	0.00206	0.00122	0.002231	-0.000222	0.000327	-0.000000
7	0.00262	0.00098	0.002479	-0.000171	0.000385	-0.000000
8	0.00390	0.00093	0.003021	-0.000108	0.000522	-0.000000
9	0.00390	0.00093	0.002033	-0.000108	0.000522	-0.000000
10	0.00141	0.00270	0.003021	-0.000225	0.000201	-0.000000
11	0.00141	0.00270	0.002033	-0.000225	0.000201	-0.000000

FORCES ON PILES

PILE	COMB.	N	Q2	Q3	M1	M2	M3
1	1	455.46	-5.16	0.12	0.000	5.742	6.355
	2	351.39	-5.16	0.00	0.000	5.439	6.355
	3	379.23	-4.50	-7.44	0.000	-17.524	7.562
	4	270.39	-3.93	-6.96	0.000	-16.280	6.354
	5	452.37	-5.50	-2.84	0.000	-2.353	7.210
	6	346.94	-5.34	-2.78	0.000	-2.215	6.865
	7	365.51	-7.25	-2.33	0.000	-2.251	10.916
	8	422.84	-11.52	-2.98	0.000	-5.830	19.774
	9	321.61	-11.52	-2.98	0.000	-5.830	19.774
	10	409.46	-4.00	-9.90	0.000	-22.982	6.335
	11	308.24	-4.00	-9.90	0.000	-22.982	6.335
2	1	393.52	-5.16	0.12	0.000	5.742	6.355
	2	289.03	-5.16	0.00	0.000	5.439	6.355
	3	330.30	-4.50	-7.44	0.000	-17.524	7.562
	4	223.19	-3.93	-6.96	0.000	-16.280	6.354
	5	384.03	-5.50	-2.84	0.000	-2.353	7.210
	6	278.80	-5.34	-2.78	0.000	-2.215	6.865
	7	313.04	-7.25	-2.33	0.000	-2.251	10.916
	8	389.60	-11.52	-2.98	0.000	-5.830	19.774
	9	288.38	-11.52	-2.98	0.000	-5.830	19.774
	10	340.29	-4.00	-9.90	0.000	-22.982	6.335
	11	239.06	-4.00	-9.90	0.000	-22.982	6.335
3	1	331.57	-5.16	0.12	0.000	5.742	6.355
	2	226.68	-5.16	0.00	0.000	5.439	6.355
	3	281.36	-4.50	-7.44	0.000	-17.524	7.562
	4	175.99	-3.93	-6.96	0.000	-16.280	6.354
	5	315.69	-5.50	-2.84	0.000	-2.353	7.210
	6	210.65	-5.34	-2.78	0.000	-2.215	6.865
	7	260.58	-7.25	-2.33	0.000	-2.251	10.916
	8	356.37	-11.52	-2.98	0.000	-5.830	19.774
	9	255.14	-11.52	-2.98	0.000	-5.830	19.774
	10	271.11	-4.00	-9.90	0.000	-22.982	6.335
	11	169.89	-4.00	-9.90	0.000	-22.982	6.335
4	1	231.18	-5.16	0.12	0.000	5.742	6.355
	2	126.28	-5.16	0.00	0.000	5.439	6.355
	3	217.01	-4.50	-7.44	0.000	-17.524	7.562
	4	116.89	-3.93	-6.96	0.000	-16.280	6.354
	5	213.60	-5.50	-2.84	0.000	-2.353	7.210
	6	110.07	-5.34	-2.78	0.000	-2.215	6.865
	7	142.26	-7.25	-2.33	0.000	-2.251	10.916
	8	195.98	-11.52	-2.98	0.000	-5.830	19.774
	9	94.75	-11.52	-2.98	0.000	-5.830	19.774
	10	209.35	-4.00	-9.90	0.000	-22.982	6.335
	11	108.13	-4.00	-9.90	0.000	-22.982	6.335
5	1	293.12	-5.16	0.12	0.000	5.742	6.355
	2	188.64	-5.16	0.00	0.000	5.439	6.355
	3	265.95	-4.50	-7.44	0.000	-17.524	7.562

	4	164.09	-3.93	-6.96	0.000	-16.280	6.354
	5	281.94	-5.50	-2.84	0.000	-2.353	7.210
	6	178.21	-5.34	-2.78	0.000	-2.215	6.865
	7	194.72	-7.25	-2.33	0.000	-2.251	10.916
	8	229.21	-11.52	-2.98	0.000	-5.830	19.774
	9	127.98	-11.52	-2.98	0.000	-5.830	19.774
	10	278.53	-4.00	-9.90	0.000	-22.982	6.335
	11	177.30	-4.00	-9.90	0.000	-22.982	6.335
6	1	355.06	-5.16	0.12	0.000	5.742	6.355
	2	250.99	-5.16	0.00	0.000	5.439	6.355
	3	314.88	-4.50	-7.44	0.000	-17.524	7.562
	4	211.29	-3.93	-6.96	0.000	-16.280	6.354
	5	350.29	-5.50	-2.84	0.000	-2.353	7.210
	6	246.35	-5.34	-2.78	0.000	-2.215	6.865
	7	247.18	-7.25	-2.33	0.000	-2.251	10.916
	8	262.45	-11.52	-2.98	0.000	-5.830	19.774
	9	161.22	-11.52	-2.98	0.000	-5.830	19.774
	10	347.70	-4.00	-9.90	0.000	-22.982	6.335
	11	246.47	-4.00	-9.90	0.000	-22.982	6.335

SUMMARY OF FORCES

	FILE	COMB.	N	Q2	Q3	M1	M2	M3
Nmin	4	9	94.75	-11.52	-2.98	0.000	-5.830	19.774
Nmax	1	1	455.46	-5.16	0.12	0.000	5.742	6.355
Q2max	1	8	422.84	-11.52	-2.98	0.000	-5.830	19.774
Q3max	1	10	409.46	-4.00	-9.90	0.000	-22.982	6.335
M1max	1	1	455.46	-5.16	0.12	0.000	5.742	6.355
M2max	1	10	409.46	-4.00	-9.90	0.000	-22.982	6.335
M3max	1	8	422.84	-11.52	-2.98	0.000	-5.830	19.774

CHECKING CALCULATI IN COMPARISON WITH INITIA LOAD MATRIX

1	34.52	0.01	2059.90	-778.27	416.16	0.00
2	34.52	0.73	1433.01	-781.40	416.16	0.00
3	29.66	46.71	1788.73	-483.53	246.07	0.00
4	25.96	43.74	1161.84	-470.06	229.47	0.00
5	36.71	18.41	1997.92	-806.99	418.73	0.00
6	35.65	18.07	1371.03	-805.38	413.99	0.00
7	48.08	15.07	1523.28	-616.80	470.17	0.00
8	75.92	18.87	1856.44	-364.53	607.84	0.00
9	75.92	18.87	1249.08	-364.53	607.84	0.00
10	26.47	62.25	1856.44	-694.15	241.64	0.00
11	26.47	62.25	1249.08	-694.15	241.64	0.00

CROSS BEAM OF PIERS DESIGN

Package: PKGA2
 Bridge name: LRB12a
 Girder Type: Cross Beam of Pier for P.C.I

1. INITIAL DATA

1.1. GENERAL CONDITIONS

1.2. MATERIAL

1.2.1. Prestressing Steel

Prestress classification: ASTM A416-90a 'Uncoated Seven Wires Stress Relieved Strand for Pre-stressed Concrete'

Prestressing Cable Type ("1"=Low Relaxation, "2"=Stress-relieved) 1

Tensile strength	f_{pu}	=	1860 MPa
Yield strength	f_{py}	=	1674 MPa
Modulus of elasticity	E_p	=	195000 MPa

Stress limits for Posttensioning Tendons

Prior to seating - short-term f_s may be allowed	$0.90 \cdot f_{py}$	=	1507 MPa
At anchorages immediately after anchor set	$0.70 \cdot f_{pu}$	=	1302 MPa
At end of the seating loss zone immediately after anchor set	$0.74 \cdot f_{pu}$	=	1376 MPa
At service limit state after losses	$0.80 \cdot f_{py}$	=	1339 MPa

Tendon Properties

Nominal diameter of Strand	D_{tr}	=	12.7 mm
Nominal Area of Strand	A_{str}	=	98.7 mm ²
Number of strands	n_{str}	=	12 strands
Nominal Area of Tendon	A_{ps}	=	1184 mm ²
Nominal diameter of Tendon	D_{ps}	=	38.8 mm
Duct Diameter	D_{duct}	=	80 mm
Number of Tendon	n_{ten}	=	4 Tendons
Coefficient of friction	μ	=	0.20
Wobble friction coefficient	K	=	0.00066 m ⁻¹
Anchor Set	Set	=	0.0060 m
Stress in the prestressing steel at jacking	f_{pj}	=	1395 MPa
Jacking Force	P_j	=	1652 KN

CROSS BEAM OF PIERS DESIGN

1.2.2. Reinforcing Steel

Reinf. Standart: (Enter "0" for TCVN 1651-2008, "1" for ASTM A615)

Modulus of elasticity

E_s

= 200000 Mpa

Yield strength for deform bar

f_y

= 400 Mpa

Yield strength for round bar

f_{yr}

= 250 Mpa

Reinforcing bar Area									
Diameter	12	14	16	18	20	22	25	28	32
(mm2)	113	151	202	255	314	380	491	616	804

1.2.3. Concrete

Density of concrete

γ_c

= 25.0 kN/m3

Average ambient relative humidity

H

= 85 %

Specified compressive strength of concrete at 28 days

f_c

= 40 MPa

Compressive strength of concrete at time of initial prestress

f_{ci}

= 34 MPa

Modulus of elasticity

E_c

= 34987 MPa

E_{ci}

= 32256 MPa

Temporary Stress Limit before Losses

Compressive stress

$f_{pe} = 0.60 \cdot f_{ci}$

= 20.40 MPa

Tensile stress of concrete at time of initial prestress

$f_{ctbl} = 0.58 \sqrt{f_{ci}}$

= -3.38 MPa

Limits of compressive stress of concrete at service limit state after losses

Due to the sum of effective prestress and permanent loads

$0.45 f_c$

= 18.00 MPa

Due to live load and 1/2 the sum of eff. prestress and permanent loads

$0.40 f_c$

= 16.00 MPa

Limits of tensile stress of concrete at limit state after losses

Moderate corrosion conditions

(enter "1")

$0.5 \sqrt{f_c}$

= -3.16 MPa

Severe corrosive conditions

(enter "2")

$0.25 \sqrt{f_c}$

= -1.58 MPa

f_{ctal}

= -3.16 MPa

Mat. Exchange Coefficient between Concrete and Prestressing Steel

$n = E_p / E_c$

= 5.57

Stress Block Factor

β_1

= 0.76

Modulus of Rupture

$f_r = 0.63 \sqrt{f_c}$

= 3.98 MPa

TCVN 1651 : 1985

Diameter	12	14	16	18	20	22	25	28	32
(mm2)	113	151	202	255	314	380	491	616	804

ASTM A 615/A615 M -00

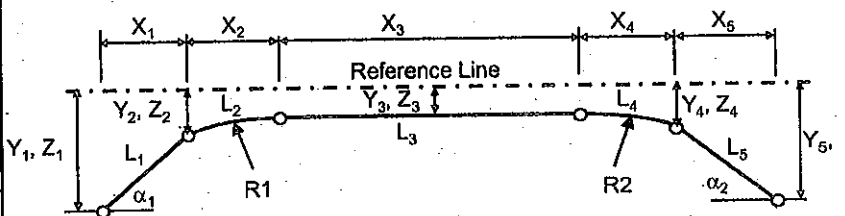
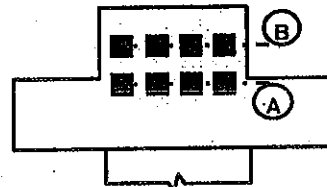
Diameter	10	13	16	19	22	25	29	32	36
(mm2)	71	129	199	284	387	510	645	819	1006

3. CABLE ARRANGEMENT

3.1. Cable Data

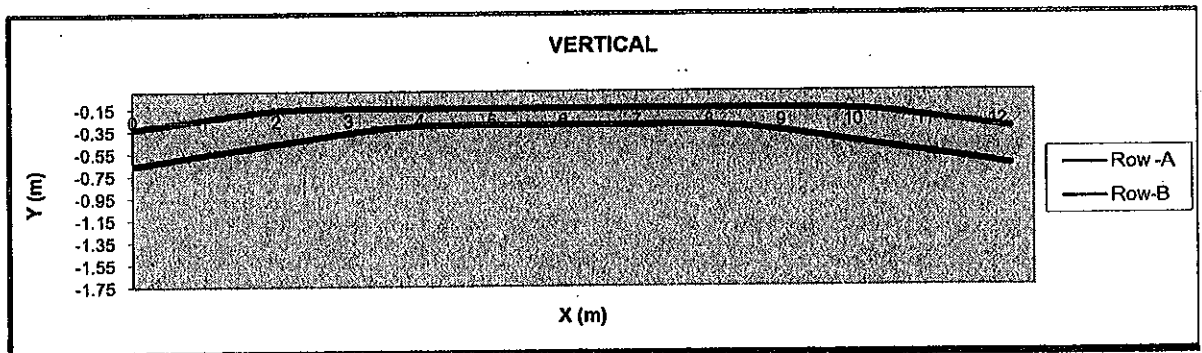
Vertical Data			Horizontal Data			
Row	A	B	Cable	1, 3	2, 4	
Z ₁ (m)	0.660	0.330	Y ₁ (m)	0.360	-0.360	
Z ₅ (m)	0.660	0.330	Y ₂ (m)	0.360	-0.360	
Z ₃ (m)	0.300	0.150	Y ₅ (m)	0.360	-0.360	
α_{v1} (°)	5.5	5.0	α_{h1} (°)	0.0	0.0	
α_{v2} (°)	5.5	5.0	α_{h2} (°)	0.0	0.0	
R _{v1} (m)	15.0	15.0	R _{h1} (m)	0.0	0.0	
R _{v2} (m)	15.0	15.0	R _{h2} (m)	0.0	0.0	

Vertical Data		
ΣX	12.180	12.180
α_1	5.500	5.000
α_2	5.500	5.000
R ₁	15.000	15.000
R ₂	15.000	15.000
X ₁	3.022	1.405
X ₂	1.438	1.307
X ₃	3.260	6.756
X ₄	1.438	1.307
X ₅	3.022	1.405
Z ₁	0.660	0.330
Z ₂	0.369	0.207
Z ₃	0.300	0.150
Z ₄	0.369	0.207
Z ₅	0.660	0.330
L ₁	3.036	1.410
L ₂	1.440	1.309
L ₃	3.260	6.756
L ₄	1.440	1.309
L ₅	3.036	1.410



3.2. Cable Coordinate

X (m)	Z (m)		ΔX (m)	ΔZ (m)		ΔL (m)		$\Delta\alpha$ (radian)	
	A	B		A	B	A	B	A	B
0.0000	0.6600	0.3300							
0.4950	0.6123	0.2867	0.4950	-0.0477	-0.0433	0.4973	0.4969	0.0000	0.0000
0.9900	0.5647	0.2434	0.4950	-0.0477	-0.0433	0.4973	0.4969	0.0000	0.0004
1.5000	0.5156	0.1990	0.5100	-0.0491	-0.0444	0.5124	0.5119	0.0000	0.0232
2.0100	0.4665	0.1664	0.5100	-0.0491	-0.0325	0.5124	0.5110	0.0000	0.0339
2.5200	0.4174	0.1512	0.5100	-0.0491	-0.0152	0.5124	0.5102	-0.0002	0.0274
3.0300	0.3682	0.1500	0.5100	-0.0492	-0.0012	0.5124	0.5100	0.0180	0.0024
3.5400	0.3282	0.1500	0.5100	-0.0400	0.0000	0.5116	0.5100	0.0339	0.0000
4.0500	0.3056	0.1500	0.5100	-0.0226	0.0000	0.5105	0.5100	0.0333	0.0000
4.5600	0.3000	0.1500	0.5100	-0.0056	0.0000	0.5100	0.5100	0.0110	0.0000
5.0700	0.3000	0.1500	0.5100	0.0000	0.0000	0.5100	0.5100	0.0000	0.0000
5.5800	0.3000	0.1500	0.5100	0.0000	0.0000	0.5100	0.5100	0.0000	0.0000
6.0900	0.3000	0.1500	0.5100	0.0000	0.0000	0.5100	0.5100	0.0000	0.0000
6.6000	0.3000	0.1500	0.5100	0.0000	0.0000	0.5100	0.5100	0.0000	0.0000
7.1100	0.3000	0.1500	0.5100	0.0000	0.0000	0.5100	0.5100	0.0000	0.0000
7.6200	0.3000	0.1500	0.5100	0.0000	0.0000	0.5100	0.5100	0.0000	0.0000
8.1300	0.3056	0.1500	0.5100	0.0056	0.0000	0.5100	0.5100	0.0333	0.0000
8.6400	0.3282	0.1500	0.5100	0.0226	0.0000	0.5105	0.5100	0.0339	0.0000
9.1500	0.3682	0.1500	0.5100	0.0400	0.0000	0.5116	0.5100	0.0180	0.0024
9.6600	0.4174	0.1512	0.5100	0.0492	0.0012	0.5124	0.5100	-0.0002	0.0274
10.1700	0.4665	0.1664	0.5100	0.0491	0.0152	0.5124	0.5102	0.0000	0.0339
10.6800	0.5156	0.1990	0.5100	0.0491	0.0325	0.5124	0.5110	0.0000	0.0232
11.1900	0.5647	0.2434	0.5100	0.0491	0.0444	0.5124	0.5119	0.0000	0.0004
11.6850	0.6123	0.2867	0.4950	0.0477	0.0433	0.4973	0.4969	0.0000	0.0000
12.1800	0.6600	0.3300	0.4950	0.0477	0.0433	0.4973	0.4969		
						12.2122	12.1940		



4. LOADS

4.1. Reaction

Loads	LEFT SPAN			RIGHT SPAN		
	R1L	R2L	R3L	R1R	R2R	R3R
	(KN)	(KN)	(KN)	(KN)	(KN)	(KN)
Selfweight of Girder	336.0	336.0	-	336.0	336.0	-
Selfweight of Precast Plank	30.7	61.4	-	30.7	61.4	-
Selfweight of Cast-in-place Slab	225.0	225.0	-	225.0	225.0	-
Selfweight of Diaphragms	30.6	61.2	-	30.6	61.2	-
Dead load of Curb	177.9	52.6	-	177.9	52.6	-
Dead load of Wearing Surface	73.3	73.3	-	73.3	73.3	-
Dead load of Railing	-	-	-	-	-	-
Dead load of Utilities	-	-	-	-	-	-
Total Stage 2 DC	366.7	397.5	-	366.7	397.5	-
Total Stage 4 DC	433.5	338.8	-	433.5	338.8	-
Total Stage 4 DW	73.3	73.3	-	73.3	73.3	-

Loads	REACTION DUE TO LIVE LOAD					
	R1L	R2L	R3L	R1R	R2R	R3R
	(KN)	(KN)	(KN)	(KN)	(KN)	(KN)
Sum LL+IM	236.9	276.5	-	271.8	317.3	-

5. Result and checking

5.1 Cross section

5.2 Result of Internal force

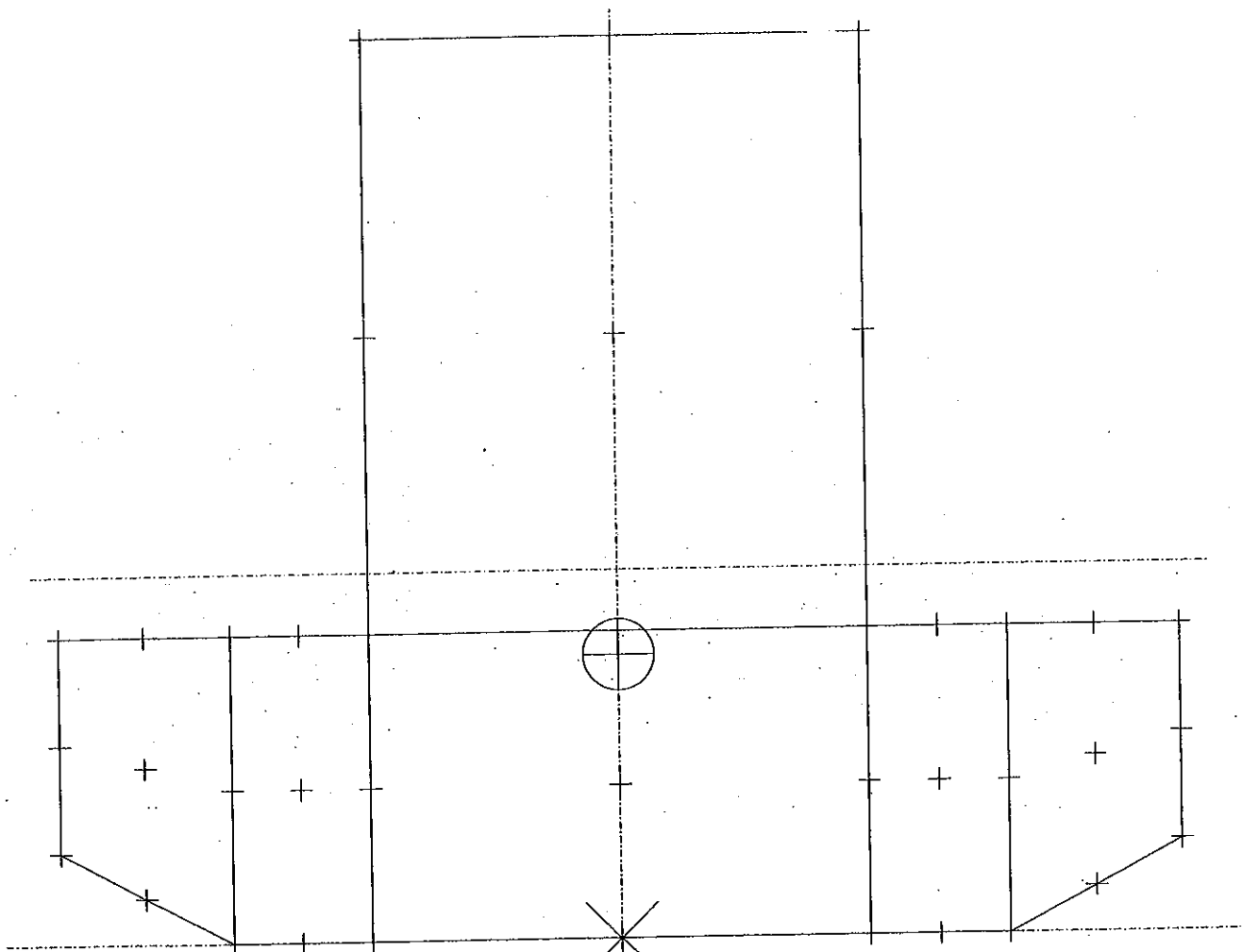
5.3 Checking for construction stage

5.4 Checking for final stage

Cross-section : 1Headstock1:001

Part : 1 Variant : 1

Description : 1Headstock1



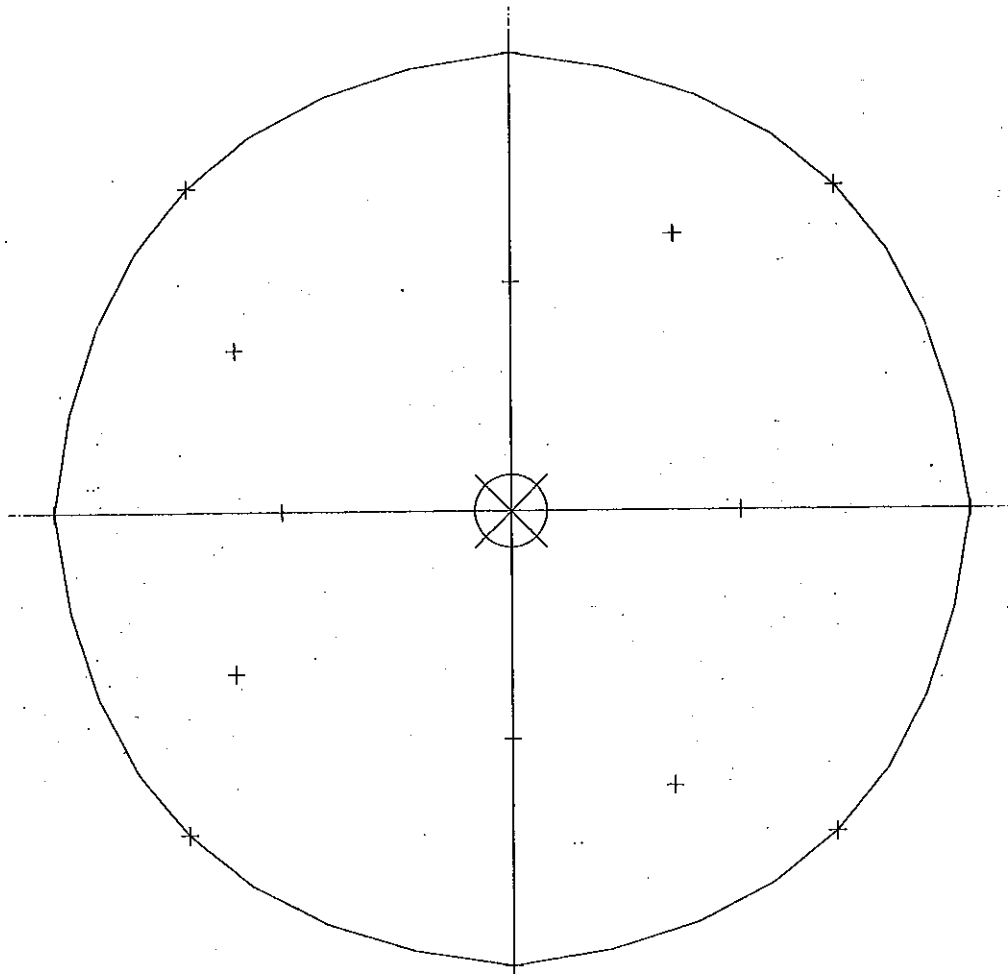
CROSS-SECTION DATA

Cross-section area	0.64990E+01	m2
Shear area - Bending about Z-axis	0.39420E+01	m2
Shear area - Bending about Y-axis	0.46347E+01	m2
Torsional moment of inertia I	0.38215E+01	m4
Moment of inertia about Y-axis	0.41076E+01	m4
Eccentricity Z (Shear lag)	0.00000E+00	m
Shear lag factor Y	0.10000E+01	
Moment of inertia about Y-axis (Shear lag)	0.41076E+01	m4
Moment of inertia about Z-axis	0.45580E+01	m4
Eccentricity Y (Shear lag)	0.00000E+00	m
Shear lag factor Z	0.10000E+01	
Moment of inertia about Z-axis (Shear lag)	0.45580E+01	m4
Warping moment of inertia	0.74924E+00	m6
Bending axis origin - Eccentricity ey	1.19730	m
Bending axis origin - Eccentricity ez	0.00000	m
Main axis angle	0.00000	Deg
Shear axis origin - Eccentricity ey	0.92253	m
Shear axis origin - Eccentricity ez	0.00000	m
Main axis angle - Shear	0.00000	Deg
Y-below : Gravity centre - minY	1.19730	m
Y-above : Gravity centre - maxY	1.71770	m
Z-left : Gravity centre - minZ	1.80000	m
Z-right : Gravity centre - maxZ	1.80000	m
Perimeter exposed to drying (outside)	12.58300	m
Perimeter (inside)	0.00000	m

Cross-section : 2Body.001

Part : 1 Variant : 1

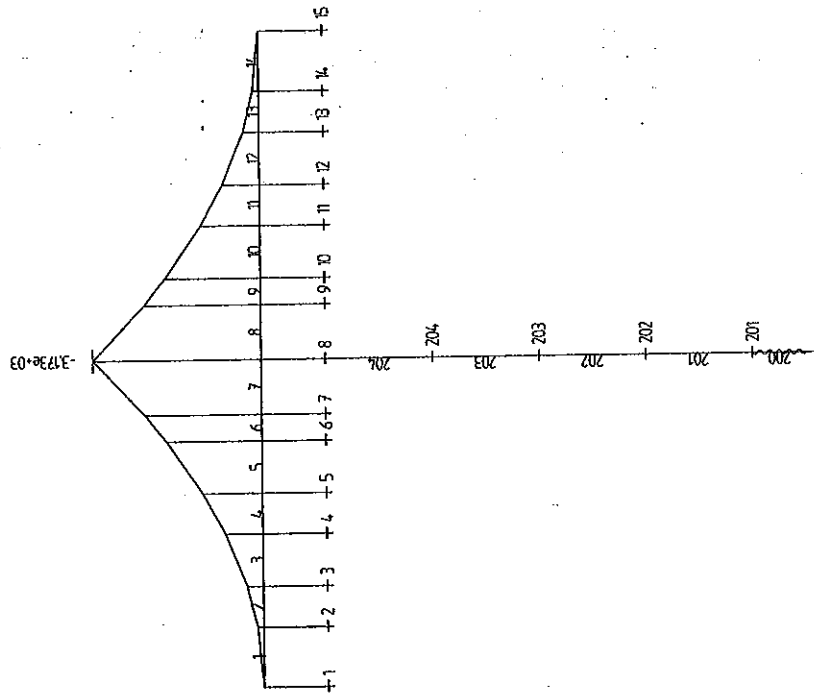
Description : 2Body



CROSS - SECTION DATA

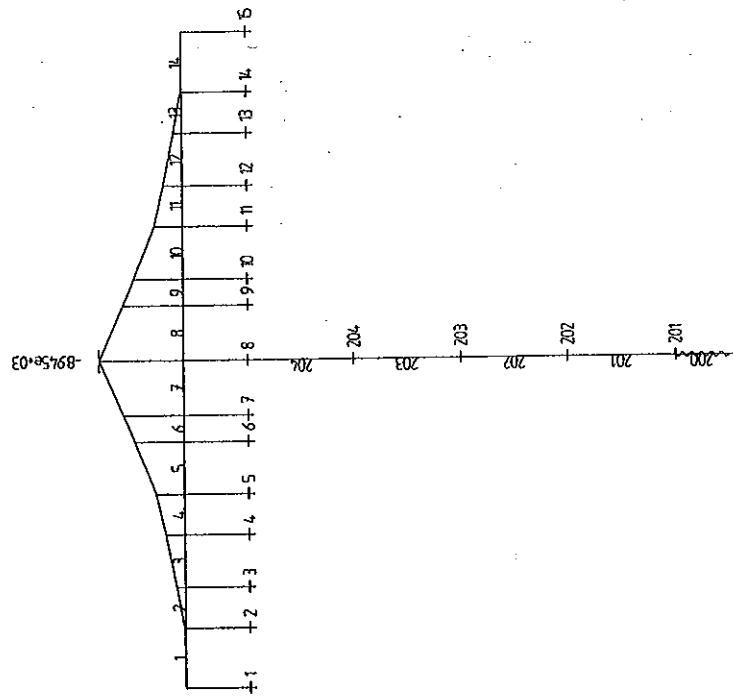
Cross-section area	0.48509E+01	m2
Shear area - Bending about Z-axis	0.41901E+01	m2
Shear area - Bending about Y-axis	0.41891E+01	m2
Torsional moment of inertia I	0.37456E+01	m4
Moment of inertia about Y-axis	0.18728E+01	m4
Eccentricity Z (Shear lag)	0.00000E+00	m
Shear lag factor Y	0.10000E+01	
Moment of inertia about Y-axis (Shear lag)	0.18728E+01	m4
Moment of inertia about Z-axis	0.18728E+01	m4
Eccentricity Y (Shear lag)	0.00000E+00	m
Shear lag factor Z	0.10000E+01	
Moment of inertia about Z-axis (Shear lag)	0.18728E+01	m4
Warping moment of inertia	0.15531E-32	m6
Bending axis origin - Eccentricity ey	0.00000	m
Bending axis origin - Eccentricity ez	0.00000	m
Main axis angle	0.00000	Deg
Shear axis origin - Eccentricity ey	0.00000	m
Shear axis origin - Eccentricity ez	0.00000	m
Main axis angle - Shear	0.00000	Deg
Y-below : Gravity centre - minY	1.25000	m
Y-above : Gravity centre - maxY	1.25000	m
Z-left : Gravity centre - minZ	1.25000	m
Z-right : Gravity centre - maxZ	1.25000	m
Perimeter exposed to drying (outside)	7.83825	m
Perimeter (inside)	0.00000	m

Construction stage 1
Mz - bending moment



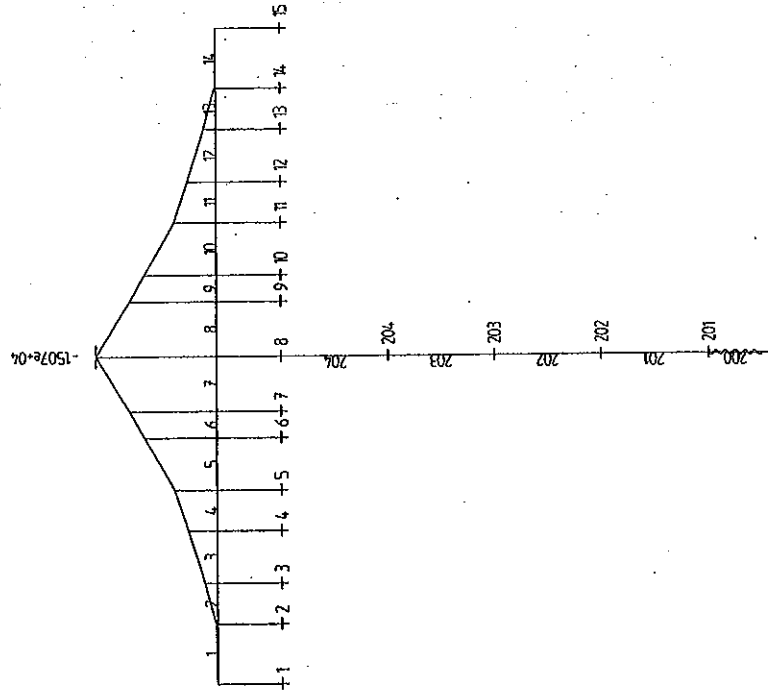
PIER

PIER

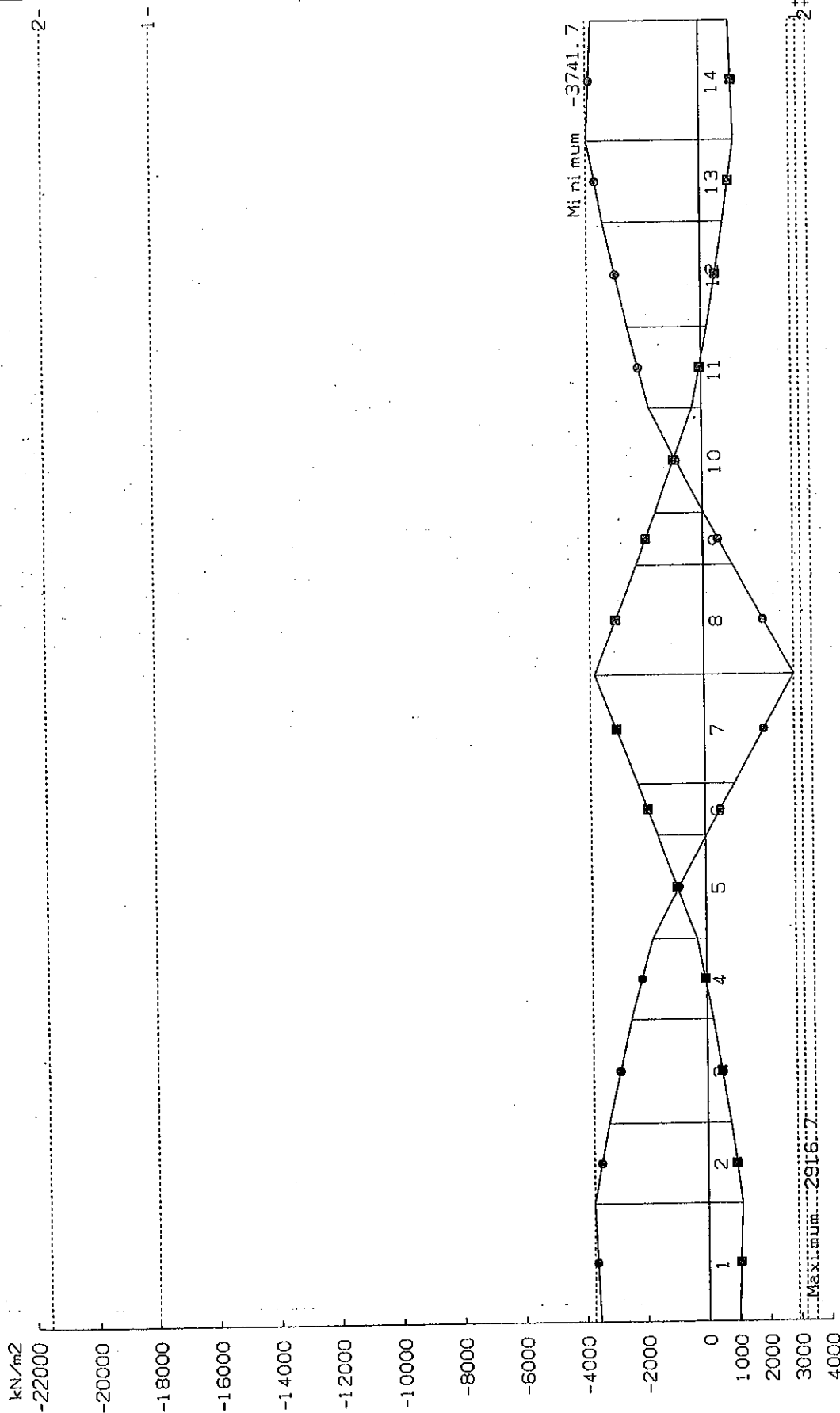


PIER

Construction stage 3
Mz - bending moment



SER1AF. sup MAX Stress-Top: S-T (-3742 , 2917)
SER1AF. sup MIN Stress-Top: S-T (-3742 , 2917)
SER1AF. sup MAX Stress-Bot: S-B (-3605 , 1098)
SER1AF. sup MIN Stress-Bot: S-B (-3605 , 1098)



15/01/2013
15:33

Project:

RmSet: Ser1A. Stage: 7
Check Fib

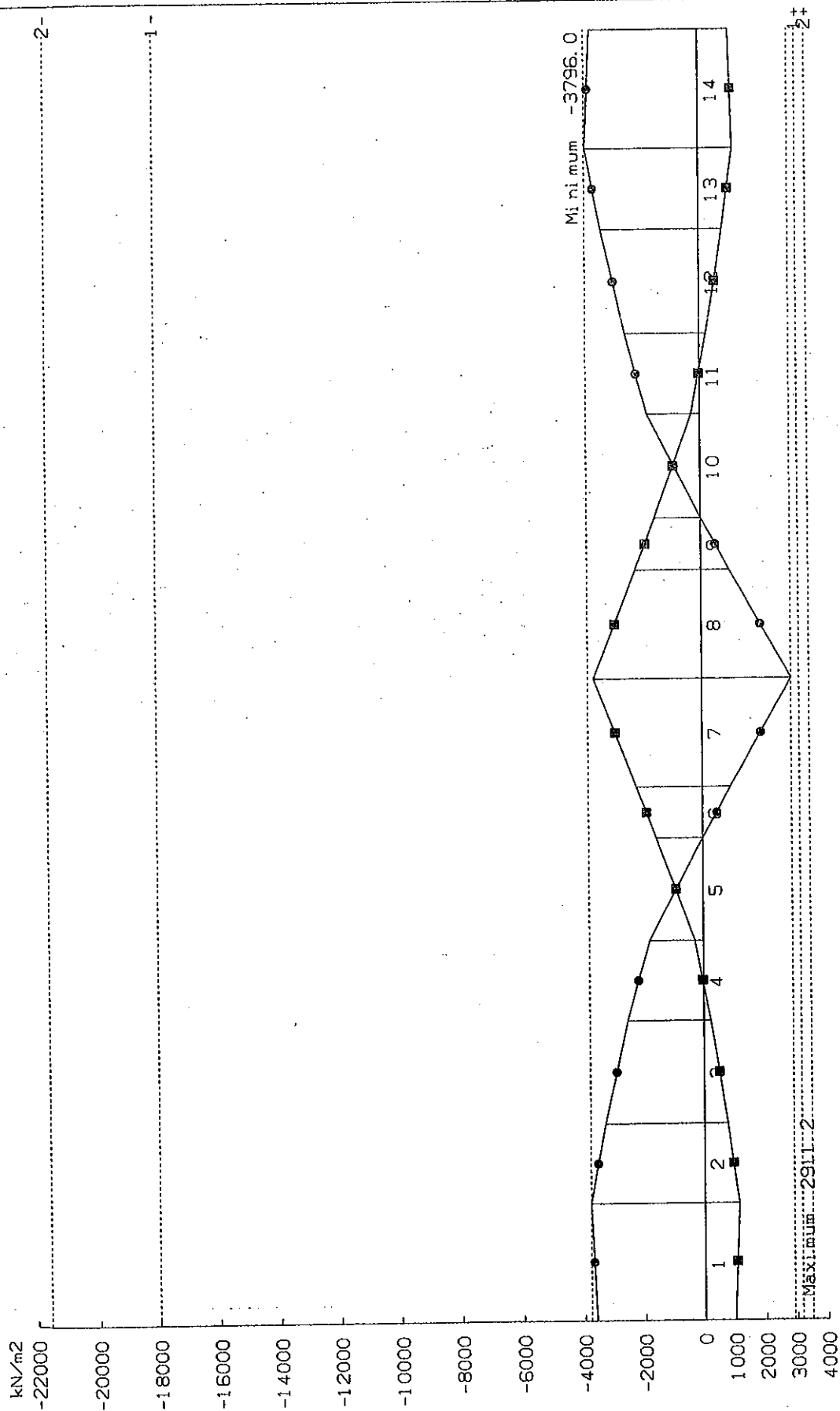
1 cm Plot = 1986.5 kN/m2

0 1986.5 3973.1 5959.6 7946.1 9932.7

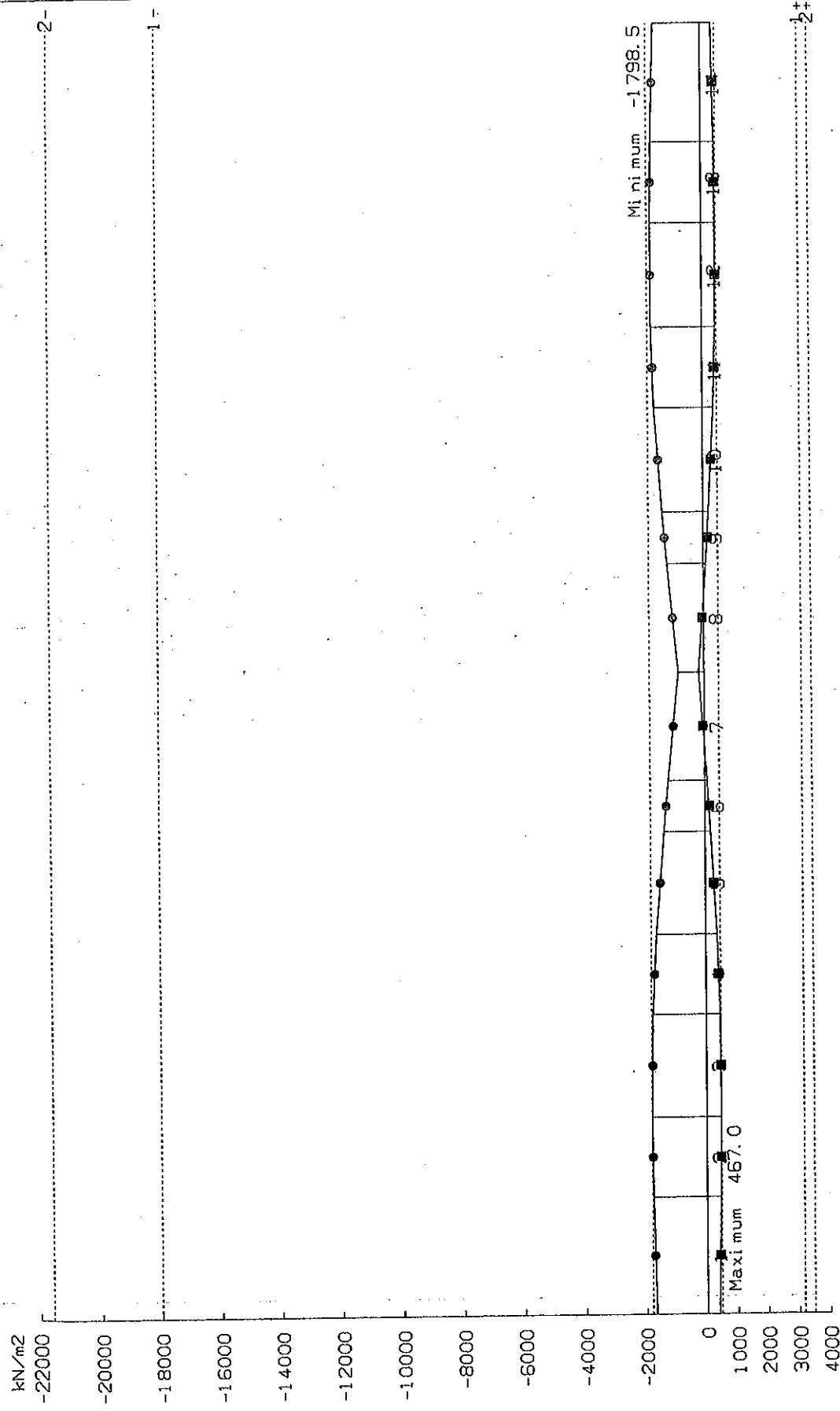


RM2004

SER1BE. sup MAX Stress-Top: S-T (-3796 , 2911)
SER1BE. sup MIN Stress-Top: S-T (-3796 , 2911)
SER1BE. sup MAX Stress-Bot: S-B (-3604 , 1114)
SER1BE. sup MIN Stress-Bot: S-B (-3604 , 1114)



fibl401. sup MAX Stress-Top: S-T (-1799 , 0)	fibl401. sup MIN Stress-Top: S-T (-1799 , 0)	fibl401. sup MAX Stress-Bot: S-B (-186 , 467)	fibl401. sup MIN Stress-Bot: S-B (-186 , 467)
---	---	--	--



16/01/2013
15:33

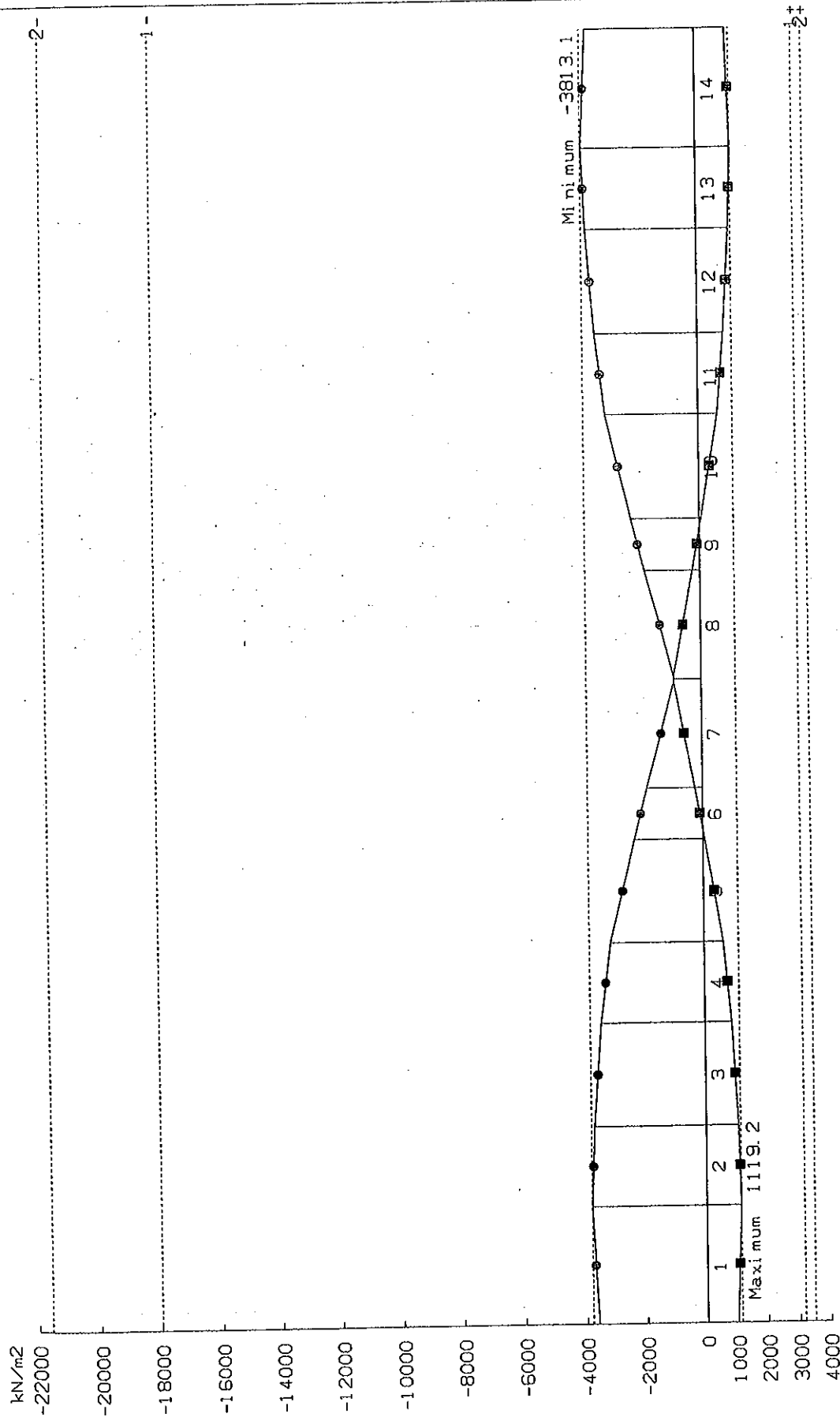
Project:

RmSet: Stress1401, Stage: 1
Self weigh dcl & Tensioning

1 cm Plot = 1986.5 kN/m2



RM2004



fibl402. sup
MAX
Stress-Top: S-T
(-3813 , 0)

fibl402. sup
MIN
Stress-Top: S-T
(-3813 , 0)

fibl402. sup
MAX
Stress-Bot: S-B
(-932 , 1119)

fibl402. sup
MIN
Stress-Bot: S-B
(-932 , 1119)

Project:

RmSet: Stress1402, Stage: 2
Deaolad Girder-DC2-Tensioning

1 cm Plot = 1986.5 kN/m2

0 1986.5 3973.1 5959.6 7946.1 9932.7

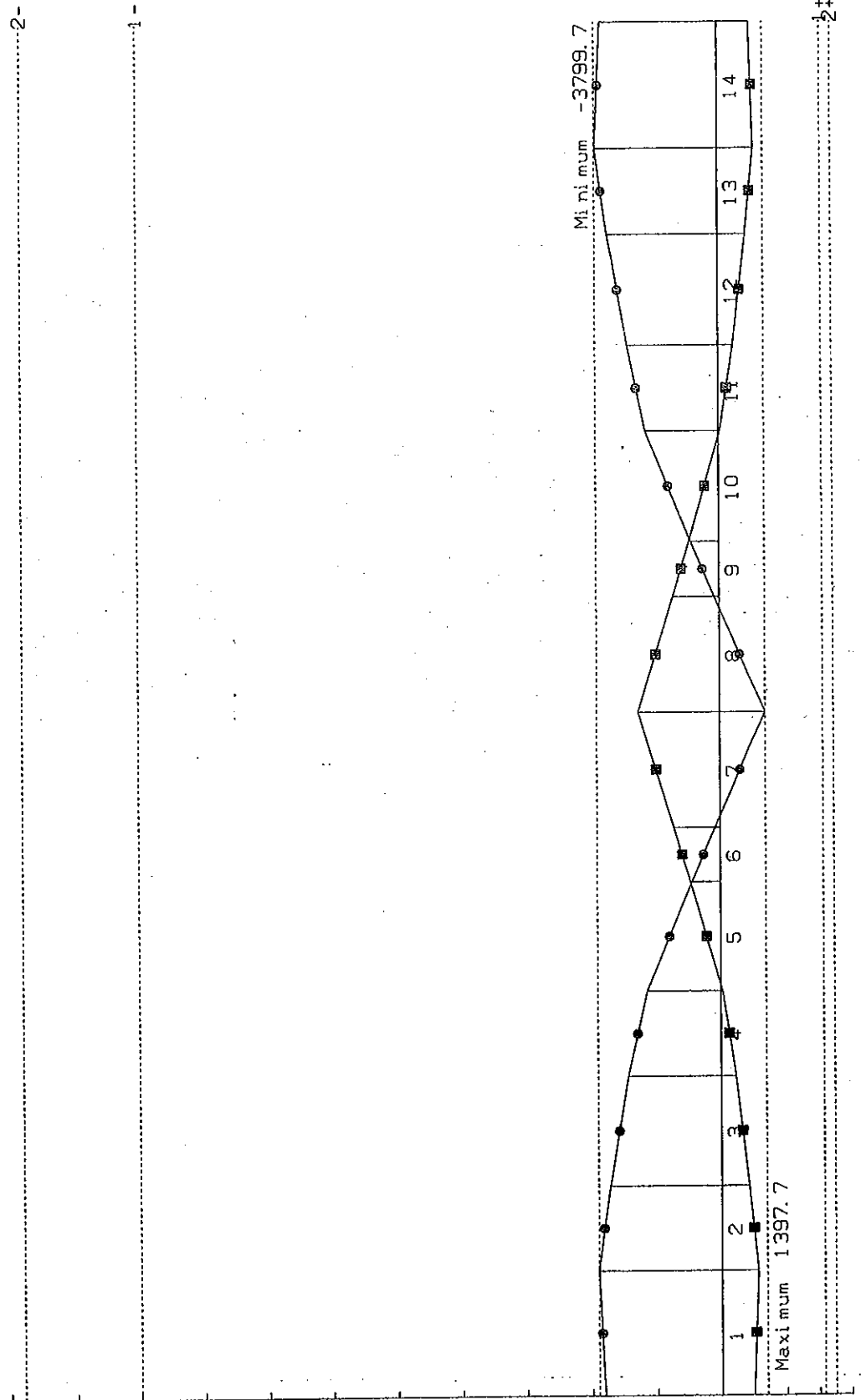


RM2004

16/01/2013
15:33

kN/m²

-22000
-20000
-18000
-16000
-14000
-12000
-10000
-8000
-6000
-4000
-2000
0
1000
2000
3000
4000



fibl403. sup
MAX
Stress-Top: S-T
(-3800, 1398)

fibl403. sup
MIN
Stress-Top: S-T
(-3800, 1398)

fibl403. sup
MAX
Stress-Bot: S-B
(-2536, 1115)

fibl403. sup
MIN
Stress-Bot: S-B
(-2536, 1115)

TDV

RM2004

Project:

RmSet: Stress1403, Stage: 3
DSLAb+Parapet

1 cm Plot = 1986.5 kN/m²

0 1986.5 3973.1 5959.6 7946.1 9932.7

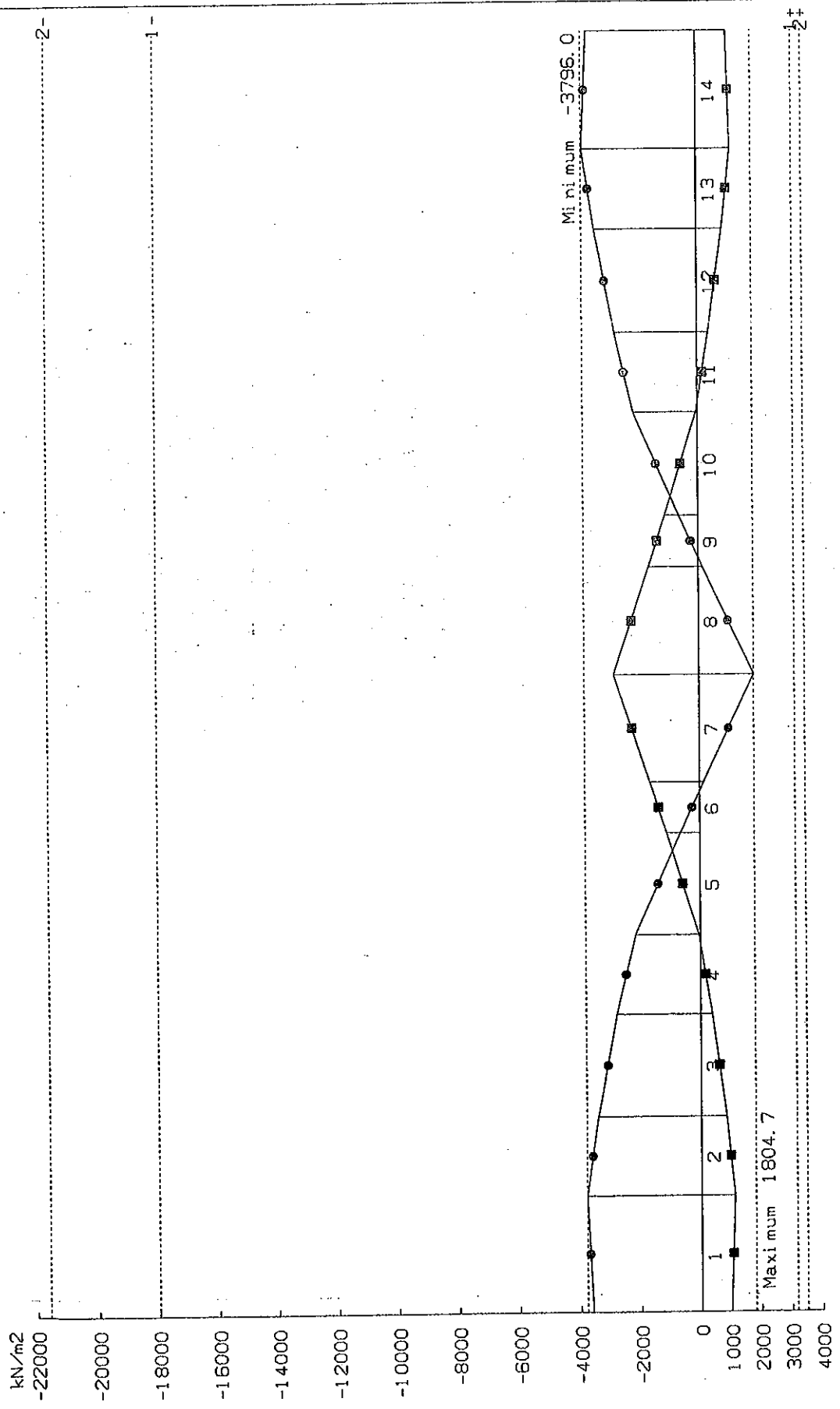
16/01/201
15:33

fibl404. sup
MAX
Stress-Top: S-T
(-3796 , 1805)

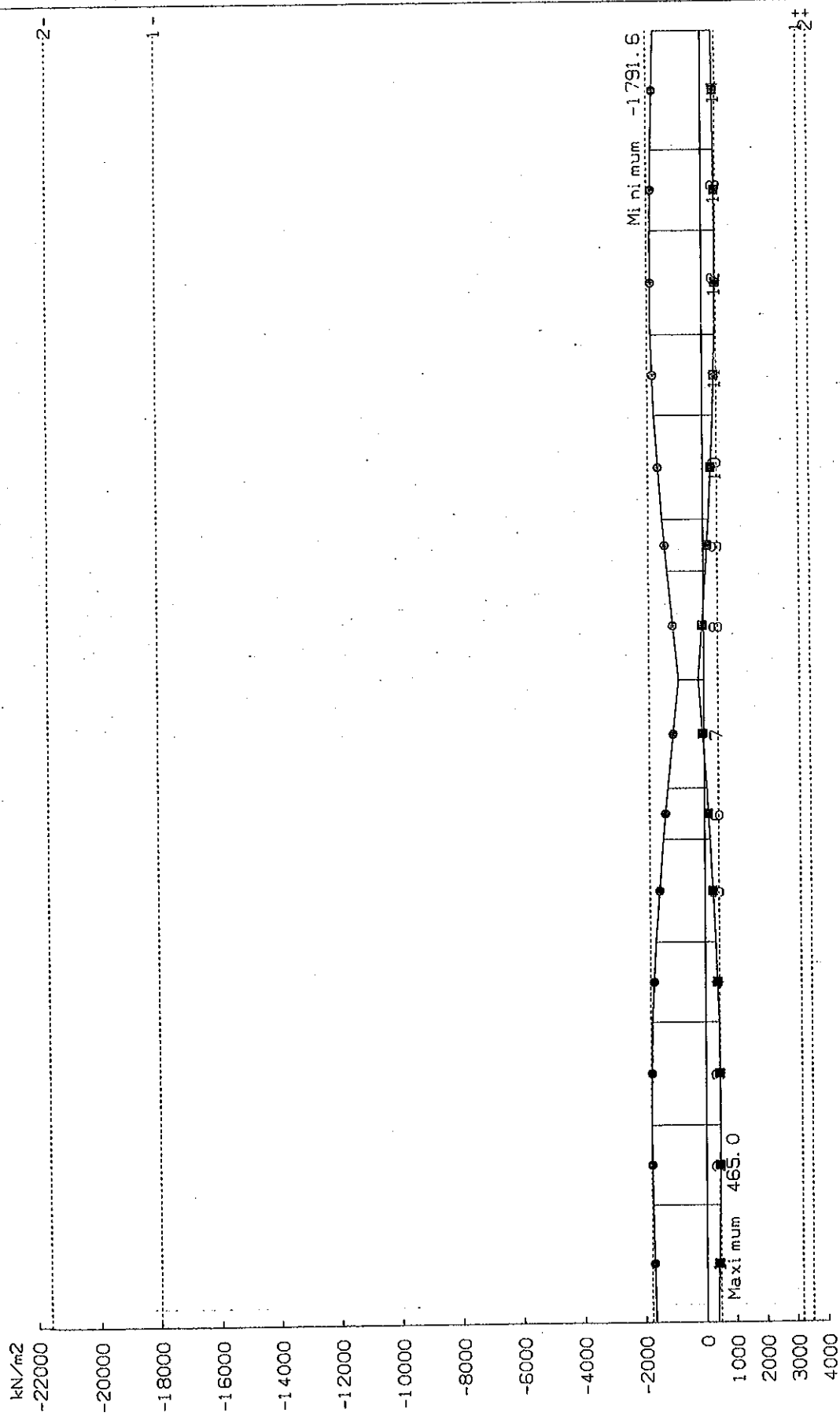
fibl404. sup
MIN
Stress-Top: S-T
(-3796 , 1805)

fibl404. sup
MAX
Stress-Bot: S-B
(-2825 , 1114)

fibl404. sup
MIN
Stress-Bot: S-B
(-2825 , 1114)



fib1501. sup MAX Stress-Top: S-T (-1792 , 0)
fib1501. sup MIN Stress-Top: S-T (-1792 , 0)
fib1501. sup MAX Stress-Bot: S-B (-187 , 465)
fib1501. sup MIN Stress-Bot: S-B (-187 , 465)



16/01/201
15:33

RmSet: Stress1501. Stage: 1
Self weigh dcl: & Tensioning

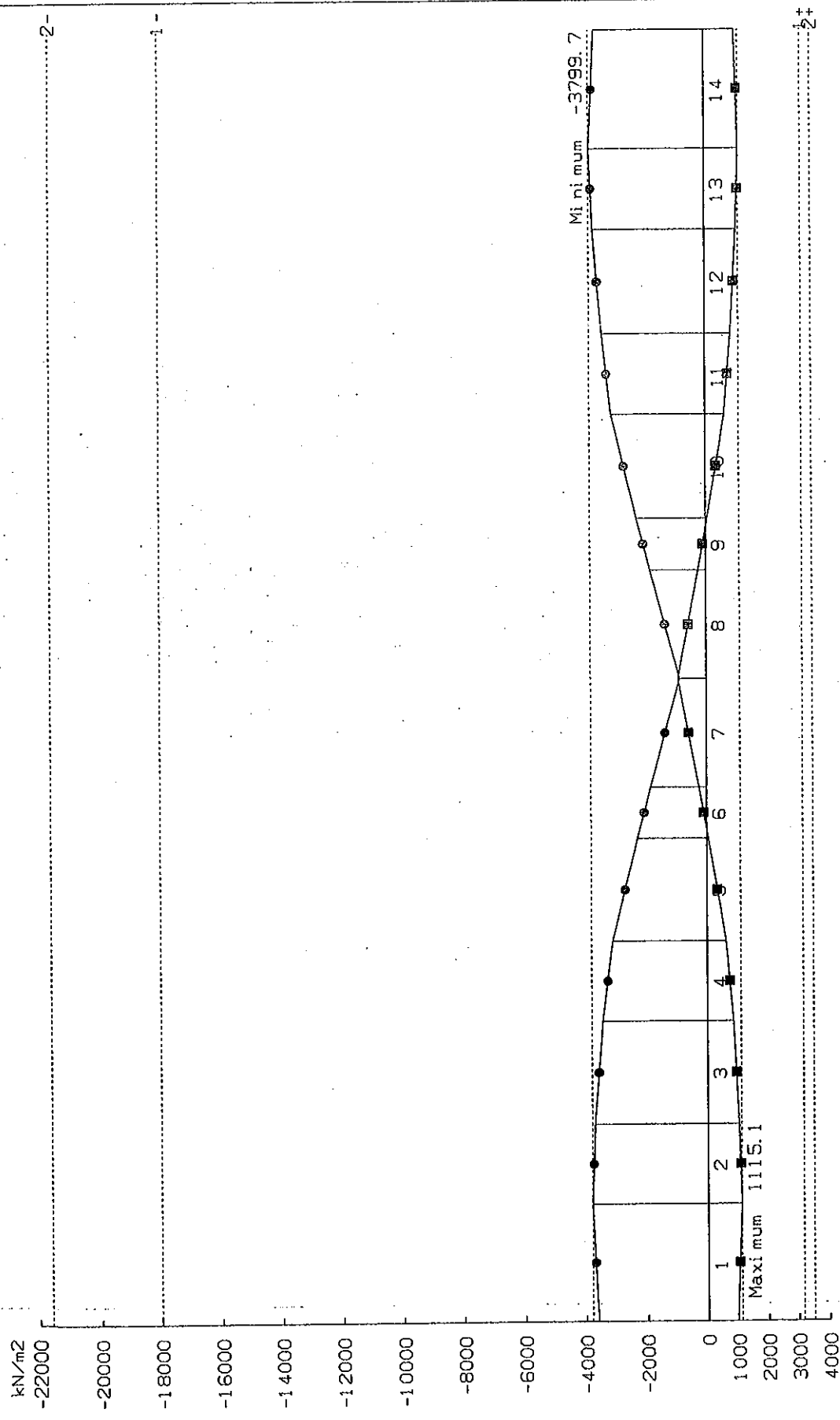
1 cm Plot = 1986.5 kN/m2

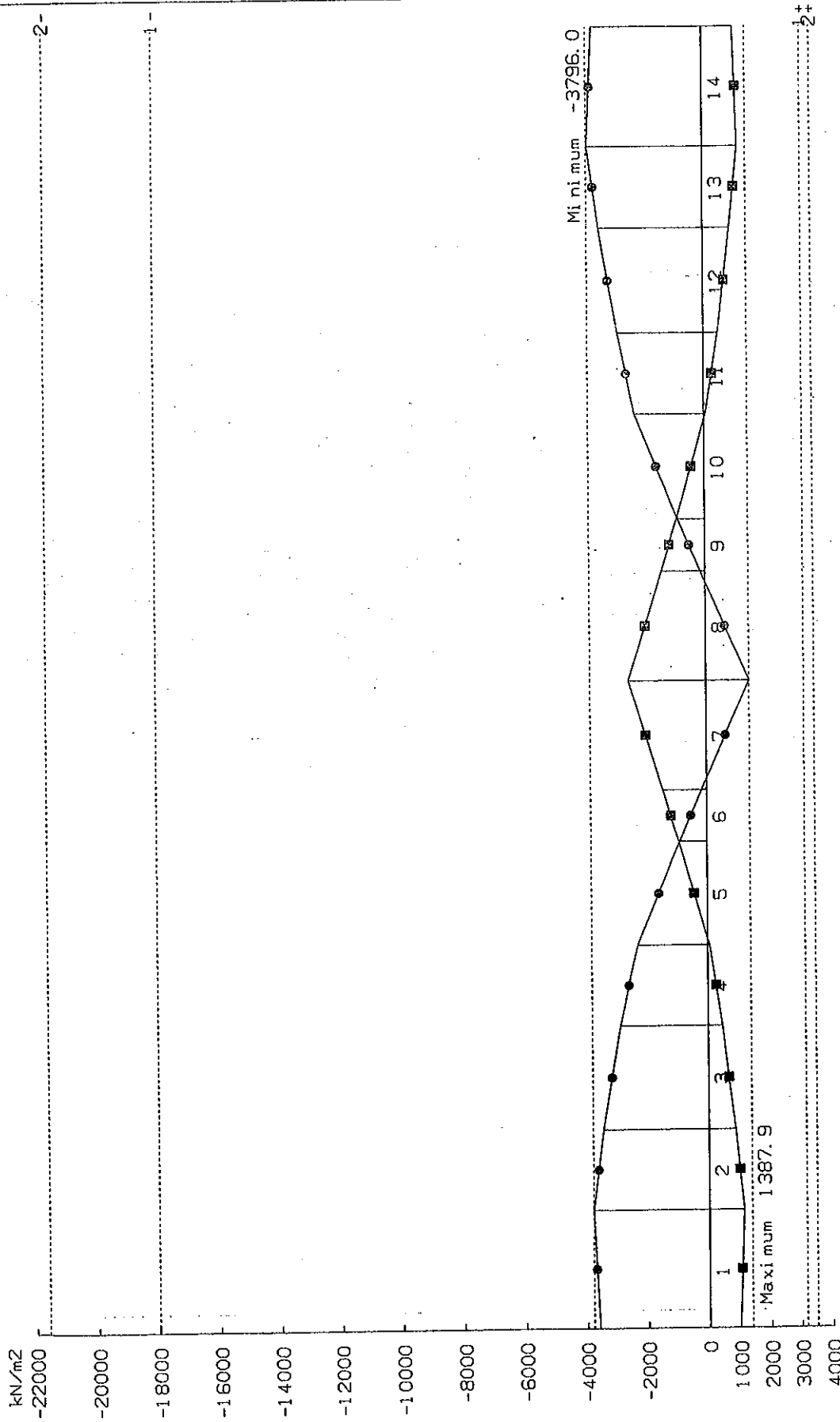
0 1986.5 3973.1 5959.6 7946.1 9932.7



RM2004

fi bl 502. sup MAX Stress-Top: S-I (-3800 , 0)
fi bl 502. sup MIN Stress-Top: S-I (-3800 , 0)
fi bl 502. sup MAX Stress-Bot: S-B (-933 , 1115)
fi bl 502. sup MIN Stress-Bot: S-B (-933 , 1115)





fib1503. sup	MAX	Stress-Top: S-T	(-3796 , 1388)
fib1503. sup	MIN	Stress-Top: S-T	(-3796 , 1388)
fib1503. sup	MAX	Stress-Bot: S-B	(-2532 , 1114)
fib1503. sup	MIN	Stress-Bot: S-B	(-2532 , 1114)

16/01/2015
15:33

Project:

RmSet: Stress1503, Stage: 3
DSlab+Parapet

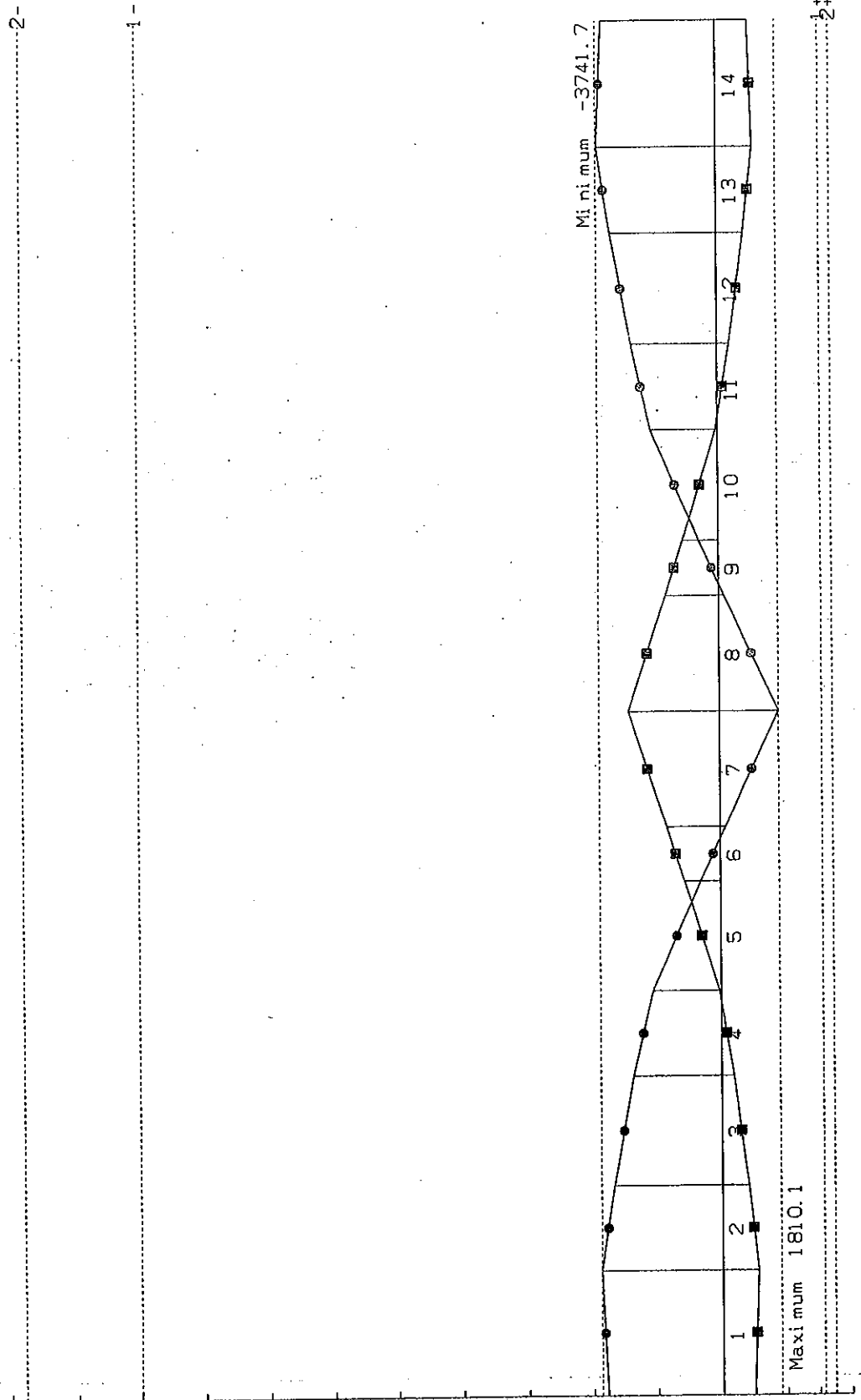
1 cm Plot = 1986.5 kN/m2

0 1986.5 3973.1 5959.6 7946.1 9932.7



RM2004

kN/m2
-22000
-20000
-18000
-16000
-14000
-12000
-10000
-8000
-6000
-4000
-2000
0
1000
2000
3000
4000



fib1504. sup
MAX
Stress-Top: S-I
(-3742 , 1810)

fib1504. sup
MIN
Stress-Top: S-I
(-3742 , 1810)

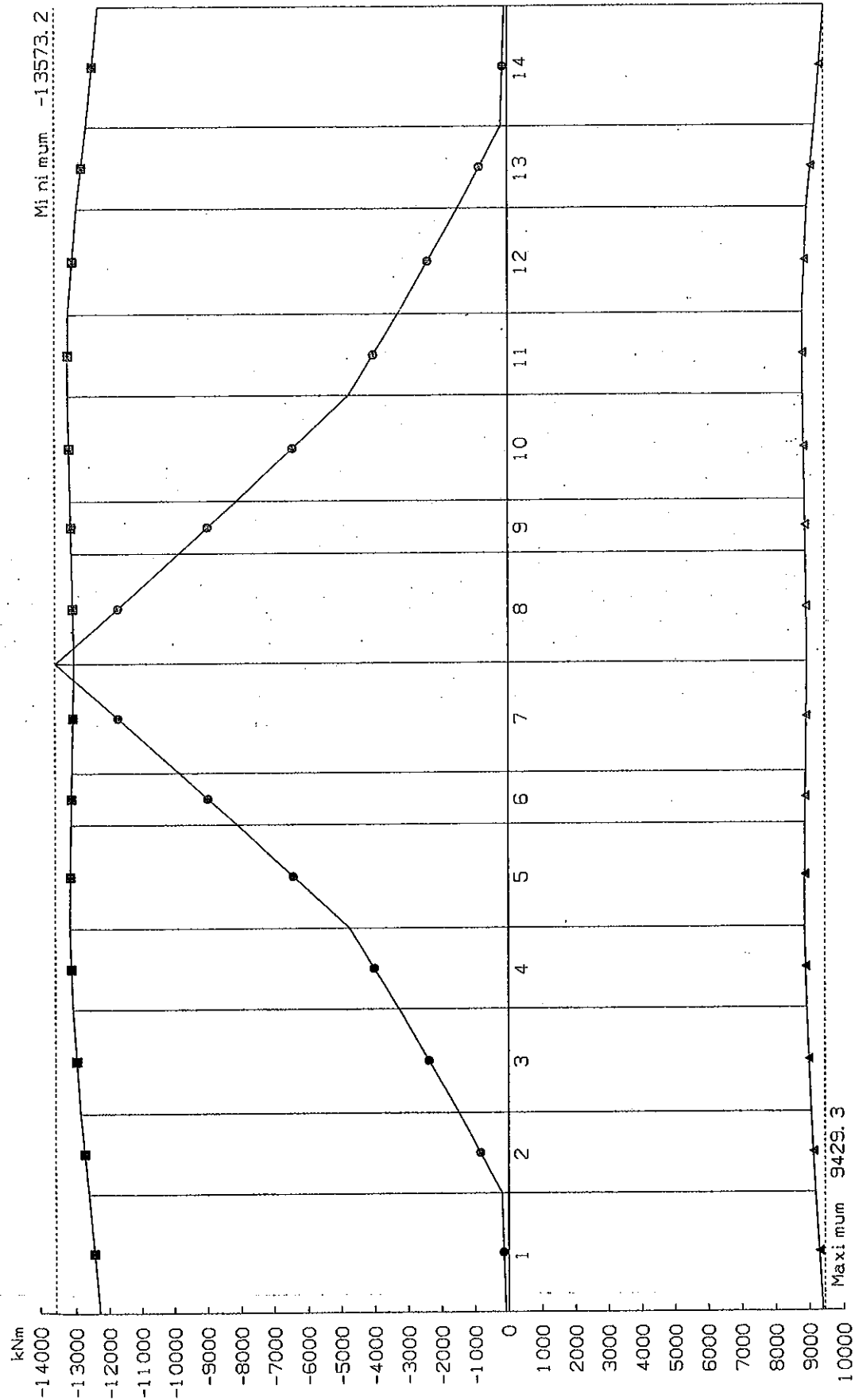
fib1504. sup
MAX
Stress-Bot: S-B
(-2827 , 1098)

fib1504. sup
MIN
Stress-Bot: S-B
(-2827 , 1098)

Project:

RmSet: Stress1504, Stage: 4
DW-Stage End

1 cm Plot = 1886.5 kN/m2
0 1886.5 3873.1 5859.6 7946.1 9932.7



STR2. sup
MAXMz: Mz
total: local: joined
(-13573 , 0)

STR2. sup
MINMz: Mz
total: local: joined
(-13573 , 0)

ult-STR2. sup
MINMz: Mz
total: local: joined
(-13190 , 0)

ult-STR2. sup
MAXMz: Mz
total: local: joined
(0 , 9429)

18/01/20
15:33

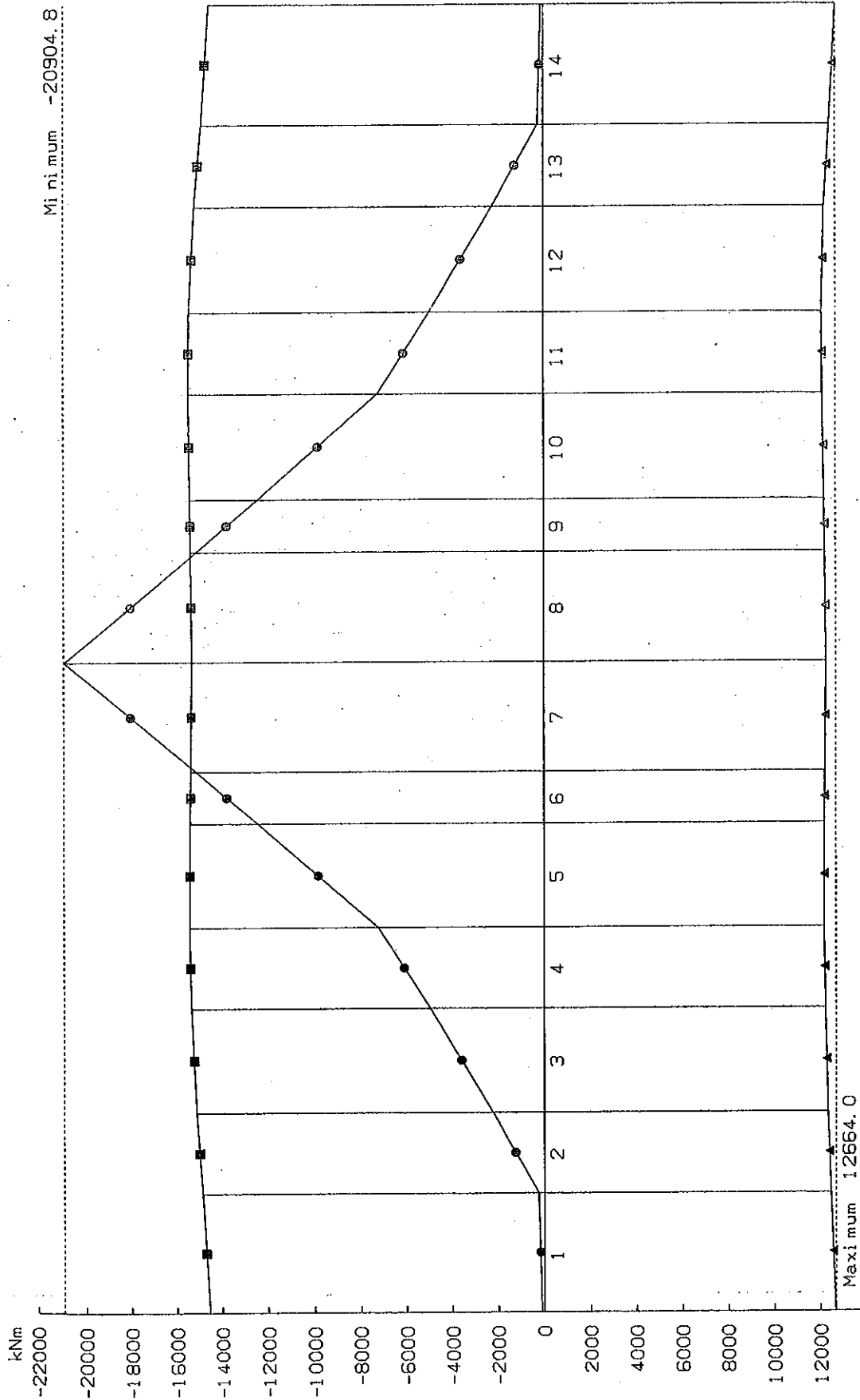
RmSet: UL1-Str2, Stage: 8
UL-Check

Project:



RM2004

1 cm Plot = 1822.0 kNm
0 1822.0 3644.0 5466.0 7287.9 9109.9



STR3. sup MAXMz: Mz total: local: joined (-20905 , 0)	STR3. sup MINMz: Mz total: local: joined (-20905 , 0)	Ult-STR3. sup MINMz: Mz total: local: joined (-15448 , 0)	Ult-STR3. sup MAXMz: Mz total: local: joined (0 , 12664)
--	--	--	---

16/01/2011
15:33

Project:

RmSet: ULT-Str3. Stage: 8
UL-Check

1 cm Plot = 2658.9 kNm

0 2658.9 5317.8 7976.7 10635.6 13294.5



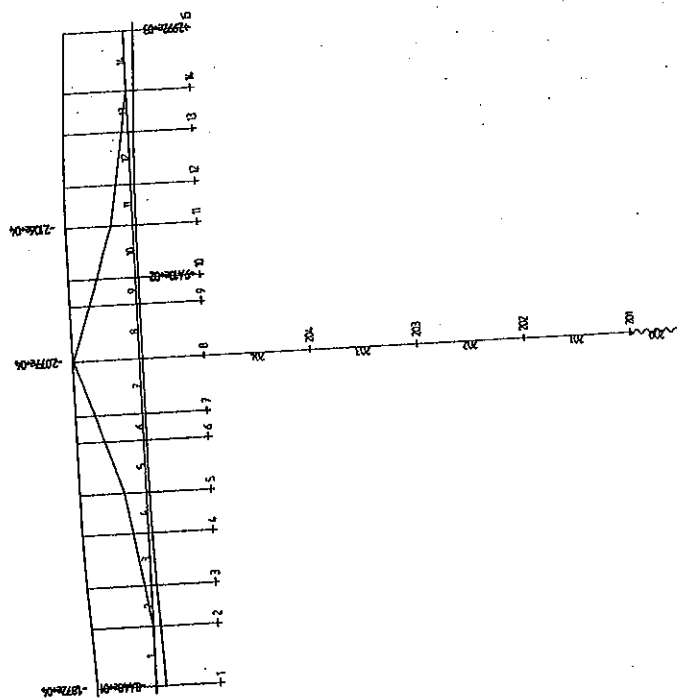
RM2004

Ultimate Moment Check - STR1a

m_{\max} M_Z - U_{lit} M_Z

min Mz - Lit Mz

Ultimate Moment:



OFFICE	PROGRAM
<div style="text-align: center; font-size: 2em; font-weight: bold;">FBI</div>	

Part	Qty	Unit	Estimate
990.021			10.000000

DATE: - January 2008 - 12/1 / 2008 / 12/1

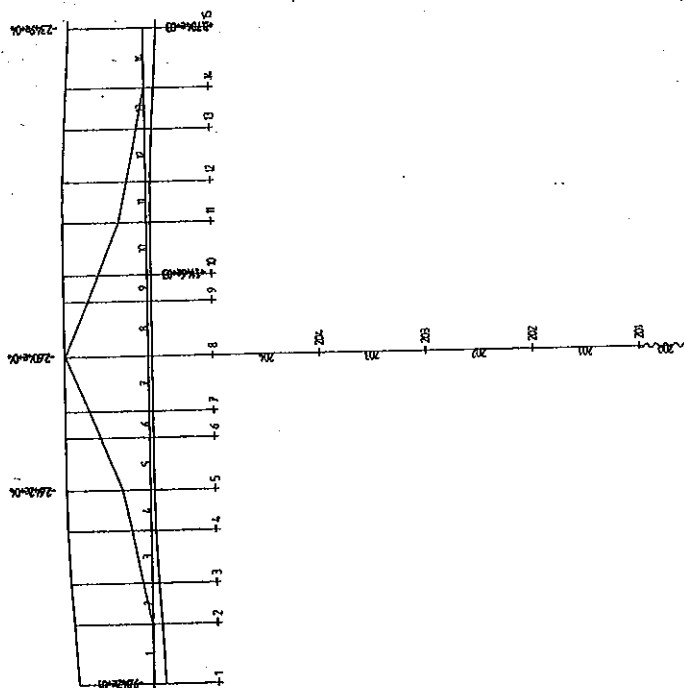
9/ER

Ultimate Moment Check - STR1

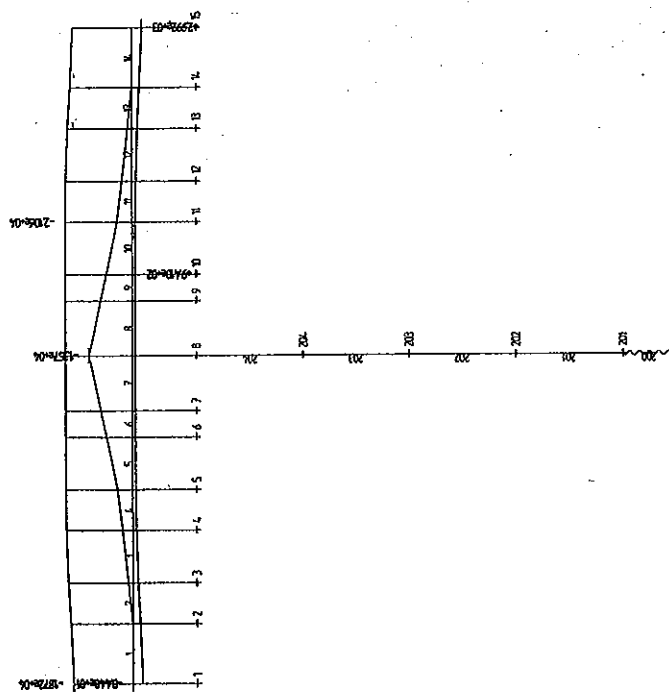
 $\max MZ - \text{Uff MZ}$

min MZ - UH MZ

Ultimate Moment

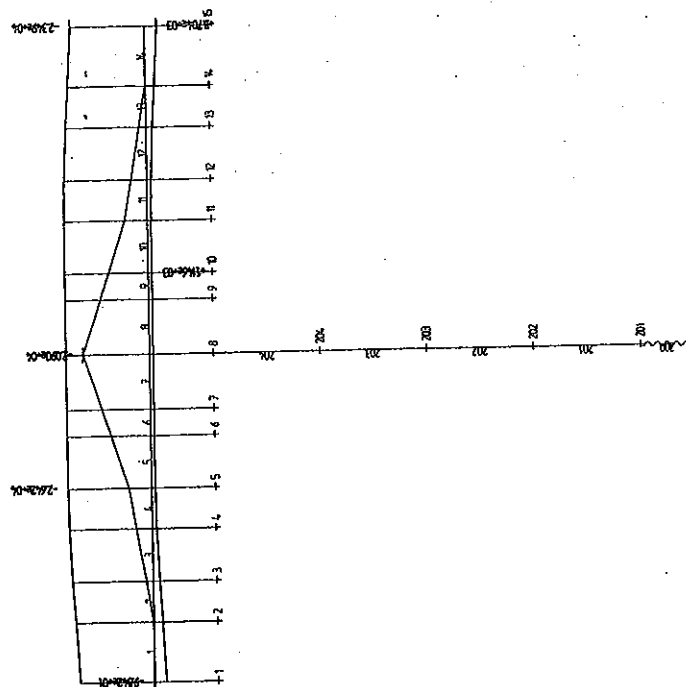


Ultimate Moment



PIER

Ultimate Moment



PIER

AD

EDUC

OFFICE

123006

PROJECT	PART	FILE NAME	DATE-CREATION
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DATE 5/1/07

NOT

Checking pier cap (Summary Design)

General Properties

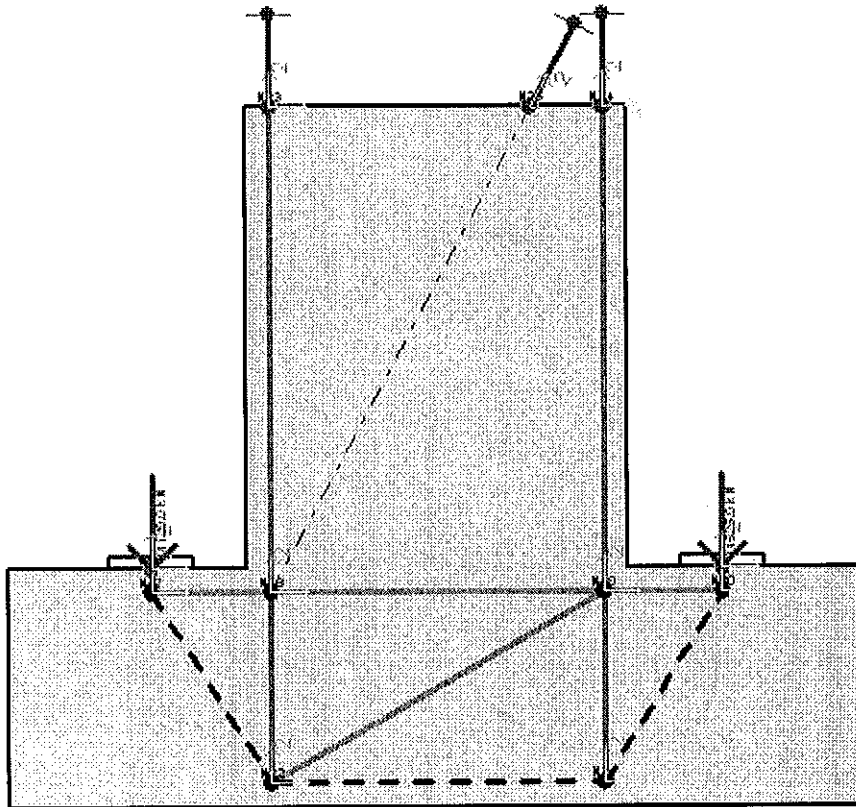
D-Region Thickness = 2200.0 mm

Concrete Cylinder Strength = 40.00 MPa

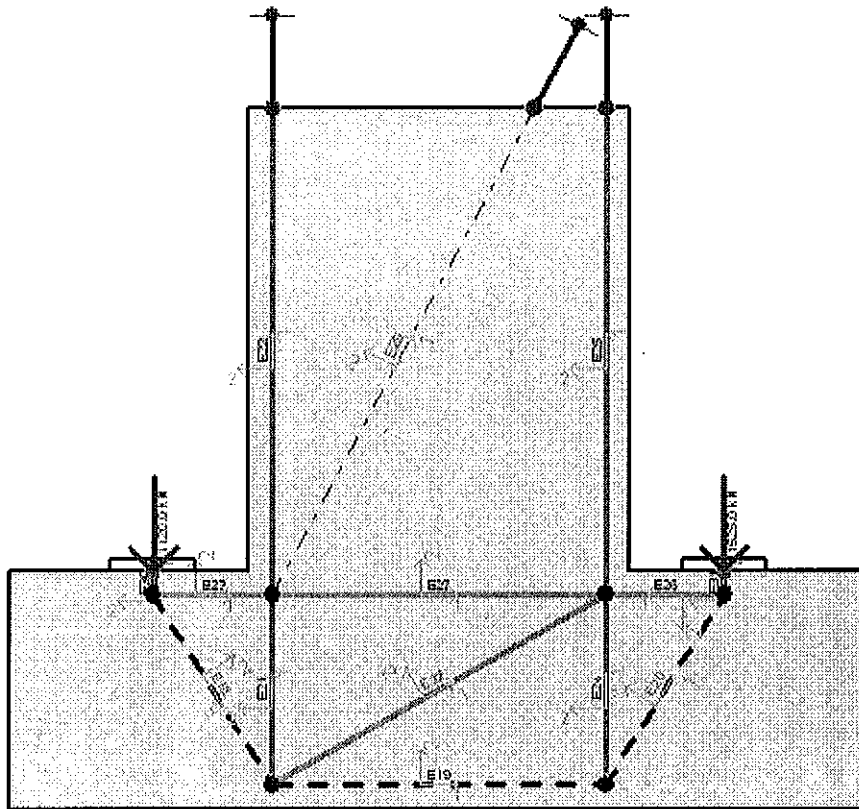
Non-Prestressed Reinforcement Yield Strength = 420.00 MPa

Load Condition: LC1

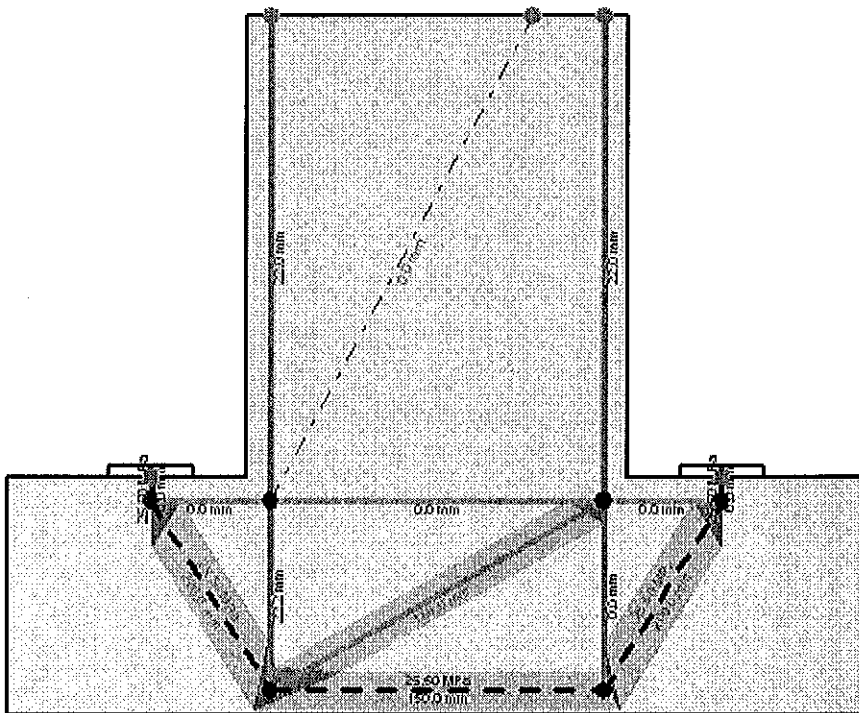
▪ Strut-and-Tie Node IDs and Axes



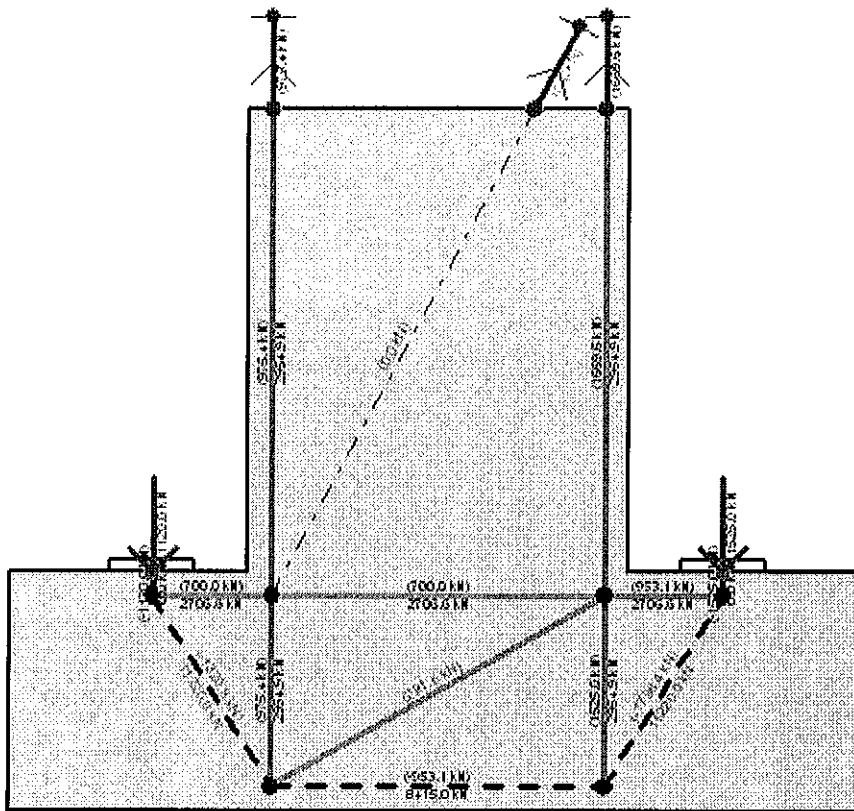
- Strut-and-Tie Element IDs and Axes



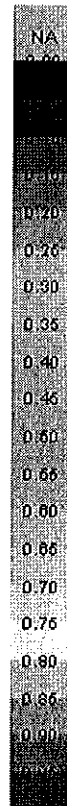
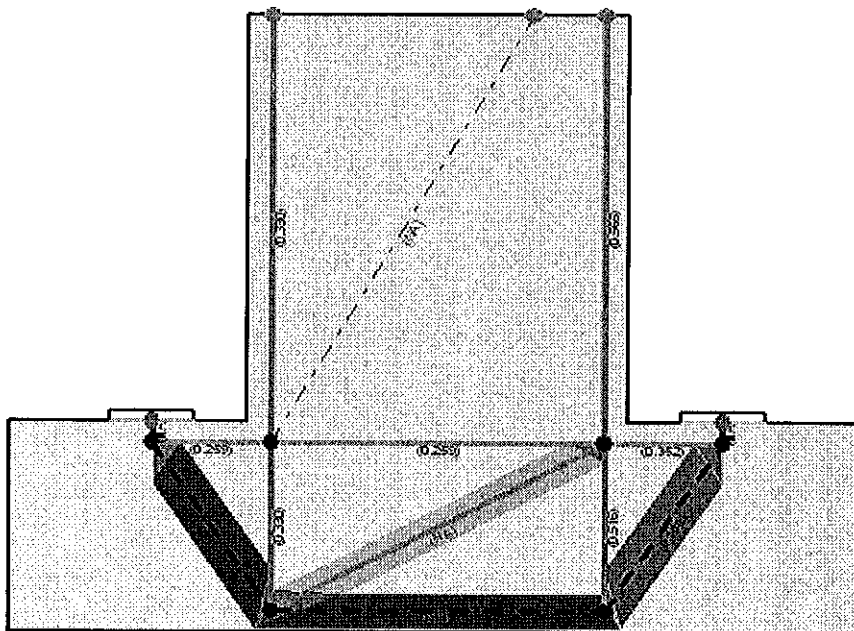
- Truss Member Stress Limits and Effective Widths



- Factored Forces and Design Strengths



- Stress Ratios



Summary of the Design

• Struts

Strut ID	F_u (kN)	β_s	ϕ	$\phi f_c =$ $\phi(0.85)\beta_s f'_c$ (MPa)	Effective Width (mm)	Effective Thickness Scale Factor (mm)	ϕF_{ns} (kN)
E18	-1320.8	1.000	0.750	25.50	200.0	1.000	11220.0
E19	-953.1	1.000	0.750	25.50	150.0	1.000	8415.0
E20	-1798.4	1.000	0.750	25.50	200.0	1.000	11220.0
E30	-1120.0	1.000	0.750	25.50	0.0	1.000	0.0
E31	-1525.0	1.000	0.750	25.50	0.0	1.000	0.0

• Ties

Tie ID	F_u (kN)	Required A_s (mm ²)	Provided A_s (mm ²)	ϕ	ϕF_{ns} (kN)
E21	975.4	3096.4	7504.4	0.750	2363.9
E22	975.4	3096.4	7504.4	0.750	2363.9
E23	700.0	2222.2	6874.0	0.750	2165.3
E24	1525.0	4841.3	7504.4	0.750	2363.9
E25	1669.6	5300.5	7504.4	0.750	2363.9
E26	953.1	3025.8	6874.0	0.750	2165.3
E27	700.0	2222.2	6874.0	0.750	2165.3
E32	291.5	NA	NA	NA	NA

$$E21 = E22 = E24 = E25 = 14 \times 314 \times 1 + 14 \times 314 \times \cos(45) = 7504.4 \text{ mm}^2$$

$$E23 = E26 = E27 = 14 \times 491 \times 1 = 6874 \text{ mm}^2$$

PILE CAPACITY OF BRIDGE LRB12A-PKGA2

STT	Boring	Abu/ Pier	Water level		Bottom of pile cap		Top of rock layer		Bottom of pile Tip		Pile length		Bearing capacity (T)		Internal axil force (T)		Checking
			m		m		m		m		m		Strength	Extreme	Strength	Extreme	
1	LRB12A-A1R	A1R	5.00		-0.50		-8.24		-12.50		12.00		568		431.0	382.0	OK
2	LRB12A-A2R	A2R	5.00		-0.50		-9.23		-15.00		14.50		532		431.0	382.0	OK
3	LRB12A-P1R	P1R	5.00		-2.00		-9.06		-12.50		10.50		568		455.5	423.5	OK
4	LRB12A-P2R	P2R	5.00		-3.00		-14.37		-18.00		15.00		583		461.2	461.0	OK
5	LRB12A-P3R	P3R	5.00		-1.00		-9.10		-12.50		11.50		509		455.5	423.5	OK
6	LRB12A-A1L	A1L	5.00		-0.50		-9.64		-12.50		12.00		555		431.0	382.0	OK
7	LRB12A-A2L	A2L	5.00		-0.50		-11.13		-14.50		14.00		540		431.0	382.0	OK
8	LRB12A-P1L	P1L	5.00		-2.00		-11.17		-14.00		12.00		563		455.5	423.5	OK
9	LRB12A-P2L	P2L	5.00		-3.50		-12.97		-17.00		13.50		512		461.2	461.0	OK
10	LRB12A-P3L	P3L	5.00		-2.50		-9.80		-14.00		11.50		554		455.5	423.5	OK

DANANG QUANG NGAI EXPRESSWAY		Item.	Eng.	Date.	Sign.
LRB11 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	LRB12A-A1R	Pile Concrete comp. strength	$f'_c =$	30.0	MPa
Bottom of pilecap elevation	EL1 = -0.50	Concrete Unit Weight	$g_c =$	24.5	kN/m ³
Top of socket elevation	EL2 = -8.24	Modulus of elasticity of concrete	$E_c =$	27691	MPa
Pile tip elevation	EL3 = -12.50				
Pile Length	L = 12.00 m	Depth of socket	$H_s =$	4.26	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 = 4459.3 kN				
Working normal force at top of socket	$P_1 =$ 4410.0 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	$I_p = f(H_s/D_s, E_c/E_r) =$ 0.30				Figure C10.8.3.5-1 Lrfd
	$H_s/D_s =$ 4.26				
	$E_c/E_r =$ 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

$$r_e = (\sum P_i) * H_s / (A_{soc} * E_c) = 0.864 \text{ mm}$$

The settlement of base of drilled shaft

$$r_{base} = (\sum P_i) * I_p / (D_s * E_r) = 1.058 \text{ mm}$$

$$r_e + r_{base} = 1.922 \text{ mm} < 10 \text{ mm}$$

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-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	-8.24	-8.74	0.50	30	63.88	1.68	2636	0.65	1714
2	-8.74	-9.74	1.00	1	-	-	-	-	-
3	-9.74	-10.74	1.00	57	63.88	1.68	5273	0.65	3427
4	-10.74	-12.50	1.76	50	63.88	1.68	9280	0.65	6032
5									
6									
7									
8									
Sum			4.26				17190		11173

Printed: 7/22/2013

Page: 1/2

	DANANG QUANG NGAI EXPRESSWAY	Item.	Eng.	Date.	Sign.
	LRB11 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 ₀ (MPa)	E _m / E _i	α _E	Type	qs0 (MPa)	q _s (MPa)	q _s - used (MPa)	Q _{SR} (kN)	φ _s	Q _R (kN)
1	0.50	30.00	63.88	0.08	0.52	1	13.58	0.85	0.85	1340	0.55	737
2	1.00	1.00	-	0.05	0.45	1	13.58	-	-	-	0.55	-
3	1.00	57.00	63.88	0.34	0.72	1	13.58	1.19	1.19	3741	0.55	2058
4	1.76	50.00	63.88	0.15	0.59	1	13.58	0.97	0.97	5363	0.55	2950
5	-	-	-	-	-	-	-	-	-	-	-	-
6	-	-	-	-	-	-	-	-	-	-	-	-
7	-	-	-	-	-	-	-	-	-	-	-	-
8	-	-	-	-	-	-	-	-	-	-	-	-
Sum	4.26									10444		5744

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

$$K_b = 4.64$$

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	5744 kN	586 T
Deducting pile weight		-172 kN	-17 T
Estimated Pile Capacity		5573 kN	568 T
Maximum Reaction - ULS	Ok	4228 kN	431 T

	DANANG QUANG NGAI EXPRESSWAY	Item.	Eng.	Date.	Sign.
	LRB11 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	LRB12A-A2R	Pile Concrete comp. strength	$f'_c = 30.0$ MPa
Bottom of pilecap elevation	EL1 = -0.50	Concrete Unit Weight	$g_c = 24.5$ kN/m ³
Top of socket elevation	EL2 = -9.23	Modulus of elasticity of concrete	$E_c = 27691$ MPa
Pile tip elevation	EL3 = -15.00		
Pile Length	$L = 14.50$ m	Depth of socket	$H_s = 5.77$ 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	$N = 4507.4$ kN		
Working normal force at top of socket	$P_i = 4440.7$ kN		
Intact 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	$I_p = f(H_s/D_s, E_i/E_r) = 0.30$		Figure C10.8.3.5-1 Lrfd
	$H_s/D_s = 5.77$		
	$E_i/E_r = 22.15$		
Rock mass modulus/ intact 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

$$r_e = (\sum P_i) * H_s / (A_{soc} * E_c) = 1.178 \text{ mm}$$

The settlement of base of drilled shaft

$$r_{base} = (\sum P_i) * I_p / (D_s * E_r) = 1.066 \text{ mm}$$

$$r_e + r_{base} = 2.244 \text{ mm} < 10 \text{ mm}$$

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

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

ϕ_s is the resistance factor - table 10.5.5-3 LRFD

q_u is the uniaxial compressive strength of the rock

Case 1									
No.	EL _T	EL _B	Depth (m)	RQD (%)	q_u (MPa)	q_s (MPa)	Q_{SR} (kN)	ϕ_s	Q_R (kN)
1	-9.23	-10.23	1.00	20	50.93	1.50	4708	0.65	3060
2	-10.23	-11.23	1.00	30	50.93	1.50	4708	0.65	3060
3	-11.23	-12.23	1.00	1	-	-	-	-	-
4	-12.23	-13.23	1.00	69	72.76	1.79	5628	0.65	3658
5	-13.23	-14.23	1.00	1	-	-	-	-	-
6	-14.23	-15.00	0.77	49	84.79	1.93	4678	0.65	3041
7									
8									
Sum			5.77				19722		12819

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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	qs0 (MPa)	q _s (MPa)	q _s - used (MPa)	Q _{SR} (kN)	φ _s	Q _R (kN)
1	1.00	20.00	50.93	0.05	0.45	2	13.58	0.66	0.66	2084	0.55	1146
2	1.00	30.00	50.93	0.07	0.48	2	13.58	0.71	0.71	2239	0.55	1231
3	1.00	1.00	-	0.05	0.45	2	13.58	-	-	-	0.55	-
4	1.00	69.00	72.76	0.10	0.55	2	13.58	0.97	0.97	3045	0.55	1675
5	1.00	1.00	-	0.05	0.45	2	13.58	-	-	-	0.55	-
6	0.77	49.00	84.79	0.10	0.55	2	13.58	1.04	1.04	2515	0.55	1383
7	-	-	-	-	-	-	-	-	-	-	-	-
8	-	-	-	-	-	-	-	-	-	-	-	-
Sum	5.77									9883		5436

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

$$K_b = 5.17$$

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	5436 kN	554 T
Deducting pile weight		-212 kN	-22 T
Estimated Pile Capacity		5223 kN	532 T
Maximum Reaction - ULS	Ok	4228 kN	431 T

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Case2									Type: "1: closed joints", "2: open joints"			
No.	Depth (m)	RQD (%)	q ₀ (MPa)	E _m / E _i	α _E	Type	qs0 (MPa)	q _s (MPa)	q _s - used (MPa)	Q _{SR} (kN)	φ _s	Q _R (kN)
1	1.00	26.00	43.08	0.07	0.49	1	13.58	0.66	0.66	2087	0.55	1148
2	1.00	28.00	43.08	0.08	0.50	1	13.58	0.68	0.68	2144	0.55	1179
3	1.00	90.00	61.54	0.90	0.96	1	13.58	1.56	1.56	4887	0.55	2688
4	0.44	45.00	61.54	0.13	0.58	1	13.58	0.93	0.93	1288	0.55	708
5	-	-	-	-	-	-	-	-	-	-	-	-
6	-	-	-	-	-	-	-	-	-	-	-	-
7	-	-	-	-	-	-	-	-	-	-	-	-
8	-	-	-	-	-	-	-	-	-	-	-	-
Sum	3.44									10406		5724

Unit base resistance

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

Limit pressure determined from pressuremeter tests

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

C10.8.3.5-7

At rest total horizontal stress measured at the base elevation

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

Coefficient that depend on diameter socket

$$K_b = 4.35$$

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	5724 kN	583 T
Deducting pile weight		-148 kN	-15 T
Estimated Pile Capacity		5576 kN	568 T
Maximum Reaction - ULS	Ok	4468 kN	456 T

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	EMPIRICAL ESTIMATION OF PILE CAPACITY-PIER P2	Revise			

Case2								Type: "1: closed joints" "2: open joints"				
No.	Depth (m)	RQD (%)	q _a (MPa)	E _m / E _i	α _E	Type	qs0 (MPa)	q _s (MPa)	q _s - used (MPa)	Q _{SR} (kN)	φ _s	Q _R (kN)
1	1.00	45.00	40.36	0.13	0.58	1	13.58	0.75	0.75	2371	0.55	1304
2	1.00	38.00	40.36	0.11	0.56	1	13.58	0.73	0.73	2298	0.55	1264
3	1.00	59.00	72.89	0.40	0.75	1	13.58	1.32	1.32	4149	0.55	2282
4	0.63	38.00	72.89	0.11	0.56	1	13.58	0.98	0.98	1946	0.55	1070
5	-	-	-	-	-	-	-	-	-	-	-	-
6	-	-	-	-	-	-	-	-	-	-	-	-
7	-	-	-	-	-	-	-	-	-	-	-	-
8	-	-	-	-	-	-	-	-	-	-	-	-
Sum	3.63									10764		5920

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

$$K_b = 4.42$$

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	5920 kN	603 T
Deducting pile weight		-201 kN	-21 T
Estimated Pile Capacity		5719 kN	583 T
Maximum Reaction - ULS	Ok	4524 kN	461 T

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	EMPIRICAL ESTIMATION OF PILE CAPACITY-PIER P3	Revise			

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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	LRB12A-P3R	Pile Concrete comp. strength	$f'_c =$	30.0	MPa
Bottom of pilecap elevation	EL1 = -1.00	Concrete Unit Weight	$g_c =$	24.5	kN/m ³
Top of socket elevation	EL2 = -9.10	Modulus of elasticity of concrete	$E_c =$	27691	MPa
Pile tip elevation	EL3 = -12.50				
Pile Length	L = 11.50 m	Depth of socket	$H_s =$	3.40	m
Diameter of drilled-shaft	$D_p =$ 1.00 m	Diameter of socket	$D_s =$	1.00	m
Pile Cross-Sectional Perimeter	P = 3.14 m	Socket Cross-Sect. Perimeter	$P_{soc} =$	3.14	m
Pile Cross-Sectional Area	$A_b =$ 0.79 m ²	Socket Cross-Sectional Area	$A_{soc} =$	0.79	m ²
Working normal force at pile head	N = 4690.0 kN				
Working normal force at top of socket	$P_i =$ 4650.7 kN				
Intack rock modulus	$E_i =$ 25000 MPa				
Modulus modification ratio	$K_e =$ 0.05				
Elastic modulus of the insitu rock	$E_r = K_e * E_i =$ 1250.0 MPa				
Influence coefficient	$I_p = f(H_s/D_s, E_c/E_r) =$ 0.30				
	$H_s/D_s =$ 3.40				
	$E_c/E_r =$ 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

$$r_e = (\sum P_i) * H_s / (A_{soc} * E_c) = 0.727 \text{ mm}$$

The settlement of base of drilled shaft

$$r_{base} = (\sum P_i) * I_p / (D_s * E_r) = 1.116 \text{ mm}$$

$$r_e + r_{base} = 1.843 \text{ mm} < 10 \text{ mm}$$

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 $\rightarrow q_s = 0.21 * \sqrt{q_u}$

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	-9.10	-10.10	1.00	20	42.24	1.36	4288	0.65	2787
2	-10.10	-11.10	1.00	30	84.48	1.93	6064	0.65	3941
3	-11.10	-12.50	1.40	60	84.48	1.93	8489	0.65	5518
4									
5									
6									
7									
8									
Sum			3.40				18841		12247

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Case2												Type: "1: closed joints"/"2: open joints"
No.	Depth (m)	RQD (%)	q _b (MPa)	E _m / E _i	α _B	Type	q _{s0} (MPa)	q _s (MPa)	q _s - used (MPa)	Q _{SR} (kN)	φ _s	Q _R (kN)
1	1.00	20.00	42.24	0.05	0.45	2	13.58	0.60	0.60	1898	0.55	1044
2	1.00	30.00	84.48	0.07	0.48	2	13.58	0.92	0.92	2883	0.55	1586
3	1.40	60.00	84.48	0.10	0.55	2	13.58	1.04	1.04	4593	0.55	2526
4	-	-	-	-	-	-	-	-	-	-	-	-
5	-	-	-	-	-	-	-	-	-	-	-	-
6	-	-	-	-	-	-	-	-	-	-	-	-
7	-	-	-	-	-	-	-	-	-	-	-	-
8	-	-	-	-	-	-	-	-	-	-	-	-
Sum	3.40									9374		5156

Unit base resistance

$$q_p = K_b.(p_l - p_0) + \sigma_v$$

Limit pressure determined from presuremeter tests

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

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

$$K_b = 4.34$$

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	5156 kN	526 T
Deducting pile weight		-159 kN	-16 T
Estimated Pile Capacity		4997 kN	509 T
Maximum Reaction - ULS	Ok	4468 kN	456 T

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EMPIRICAL ESTIMATION OF PILE CAPACITY-A1				Revise			

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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	LRB12A-A1L	Pile Concrete comp. strength	$f'_c =$	30.0	MPa
Bottom of pilecap elevation	EL1 = -0.50	Concrete Unit Weight	$g_c =$	24.5	kN/m ³
Top of socket elevation	EL2 = -9.64	Modulus of elasticity of concrete	$E_c =$	27691	MPa
Pile tip elevation	EL3 = -12.50				
Pile Length	L = 12.00 m	Depth of socket	$H_s =$	2.86	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 = 4459.3 kN				
Working normal force at top of socket	$P_i =$ 4426.2 kN				
Intack rock modulus	$E_i =$ 25000 MPa				Figure C10.8.3.5-2 Lrfd
Modulus modification ratio	$K_o =$ 0.05				Figure C10.8.3.5-3 Lrfd
Elastic modulus of the insitu rock	$E_r = K_o * E_i =$ 1250.0 MPa				
Influence coefficient	$I_p = f(H_s/D_s, E_o/E_r) =$ 0.30				Figure C10.8.3.5-1 Lrfd
	$H_s/D_s =$ 2.86				
	$E_o/E_r =$ 22.15				
Rock mass modulus/ intack rock modulus	E_{m1} / 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

$$r_e = (\sum P_i) * H_s / (A_{soc} * E_c) = 0.582 \text{ mm}$$

The settlement of base of drilled shaft

$$r_{base} = (\sum P_i) * I_p / (D_s * E_r) = 1.062 \text{ mm}$$

$$r_e + r_{base} = 1.644 \text{ mm} < 10 \text{ mm}$$

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 $\rightarrow q_s = 0.21 * \sqrt{q_u}$

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	-9.64	-10.64	1.00	73	59.17	1.62	5075	0.65	3299
2	-10.64	-11.64	1.00	30	59.17	1.62	5075	0.65	3299
3	-11.64	-12.50	0.86	58	59.17	1.62	4364	0.65	2837
4									
5									
6									
7									
8									
Sum			2.86				14514		9434

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	EMPIRICAL ESTIMATION OF PILE CAPACITY-A1	Revise			

Case2							Type: "1: closed joints", "2: open joints"					
No.	Depth (m)	RQD (%)	q _u (MPa)	E _m / E _i	α _B	Type	qs0 (MPa)	q _s (MPa)	q _s - used (MPa)	Q _{SR} (kN)	φ _s	Q _R (kN)
1	1.00	73.00	59.17	0.73	0.89	1	13.58	1.42	1.42	4453	0.55	2449
2	1.00	30.00	59.17	0.08	0.52	1	13.58	0.82	0.82	2579	0.55	1419
3	0.86	58.00	59.17	0.37	0.73	1	13.58	1.17	1.17	3155	0.55	1735
4	-	-	-	-	-	-	-	-	-	-	-	-
5	-	-	-	-	-	-	-	-	-	-	-	-
6	-	-	-	-	-	-	-	-	-	-	-	-
7	-	-	-	-	-	-	-	-	-	-	-	-
8	-	-	-	-	-	-	-	-	-	-	-	-
Sum	2.86									10188		5603

Unit base resistance

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

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

$$K_b = 4.12$$

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	5603 kN	571 T
Deducting pile weight		-161 kN	-16 T
Estimated Pile Capacity		5442 kN	555 T
Maximum Reaction - ULS	Ok	4228 kN	431 T

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DETAIL DESIGN		Check			
EMPIRICAL ESTIMATION OF PILE CAPACITY-A2		Revise			

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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	LRB12A-A2L	Pile Concrete comp. strength	$f'_c =$ 30.0 MPa
Bottom of pilecap elevation	EL1 = -0.50	Concrete Unit Weight	$\gamma_c =$ 24.5 kN/m ³
Top of socket elevation	EL2 = -11.13	Modulus of elasticity of concrete	$E_c =$ 27691 MPa
Pile tip elevation	EL3 = -14.50		
Pile Length	L = 14.00 m	Depth of socket	$H_s =$ 3.37 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 = 4497.8 kN		
Working normal force at top of socket	$P_i =$ 4458.8 kN		
Intact 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	$I_p = f(H_s/D_s, E_d/E_r) =$ 0.30		
	$H_s/D_s =$ 3.37		
	$E_d/E_r =$ 22.15		

Figure C10.8.3.5-2 Lrfd

Figure C10.8.3.5-3 Lrfd

Figure C10.8.3.5-1 Lrfd

Rock mass modulus/ intact 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

$$r_e = (\sum P_i) * H_s / (A_{soc} * E_c) = 0.691 \text{ mm}$$

The settlement of base of drilled shaft

$$r_{base} = (\sum P_i) * I_p / (D_s * E_r) = 1.070 \text{ mm}$$

$$r_e + r_{base} = 1.761 \text{ mm} < 10 \text{ mm}$$

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-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	-11.13	-12.130	1.00	30	49.38	1.48	4636	0.65	3013
2	-12.130	-13.130	1.00	50	49.38	1.48	4636	0.65	3013
3	-13.130	-14.500	1.37	60	52.90	1.53	6574	0.65	4273
4									
5									
6									
7									
8									
Sum			3.37				15846		10300

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Case2												Type: "1: closed joints", "2: open joints"
No.	Depth (m)	RQD (%)	q_u (MPa)	E_m / E_i	α_B	Type	q_{s0} (MPa)	q_s (MPa)	$q_s - used$ (MPa)	Q_{SR} (kN)	ϕ_s	Q_R (kN)
1	1.00	30.00	49.38	0.08	0.52	1	13.58	0.75	0.75	2356	0.55	1296
2	1.00	50.00	49.38	0.15	0.59	1	13.58	0.85	0.85	2679	0.55	1474
3	1.37	60.00	52.90	0.42	0.76	1	13.58	1.15	1.15	4931	0.55	2712
4	-	-	-	-	-	-	-	-	-	-	-	-
5	-	-	-	-	-	-	-	-	-	-	-	-
6	-	-	-	-	-	-	-	-	-	-	-	-
7	-	-	-	-	-	-	-	-	-	-	-	-
8	-	-	-	-	-	-	-	-	-	-	-	-
Sum	3.37									9966		5481

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

$$K_b = 4.33$$

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	5481 kN	559 T
Deducting pile weight		-188 kN	-19 T
Estimated Pile Capacity		5294 kN	540 T
Maximum Reaction - ULS	Ok	4228 kN	431 T

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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	LRB12A-P1L	Pile Concrete comp. strength	$f'_c = 30.0$ MPa
Bottom of pilecap elavation	EL1 = -2.00	Concrete Unit Weight	$\gamma_c = 24.5$ kN/m ³
Top of socket elevation	EL2 = -11.17	Modulus of elasticity of concrete	$E_c = 27691$ MPa
Pile tip elevation	EL3 = -14.00		
Pile Length	$L = 12.00$ m	Depth of socket	$H_s = 2.83$ 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	$N = 4699.6$ kN		
Working normal force at top of socket	$P_i = 4666.9$ 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	$I_p = f(H_s/D_s, E_c/E_r) = 0.30$		
	$H_s/D_s = 2.83$		
	$E_c/E_r = 22.15$		
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

$$r_e = (\sum P_i) * H_s / (A_{soc} * E_c) = 0.607 \text{ mm}$$

The settlement of base of drilled shaft

$$r_{base} = (\sum P_i) * I_p / (D_s * E_r) = 1.120 \text{ mm}$$

$$r_e + r_{base} = 1.727 \text{ mm} < 10 \text{ mm}$$

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

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

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	-11.17	-11.17	-	1	-	-	-	-	-
2	-11.17	-12.17	1.00	25	68.19	1.73	5448	0.65	3541
3	-12.17	-13.17	1.00	62	68.19	1.73	5448	0.65	3541
4	-13.17	-14.00	0.83	62	68.19	1.73	4522	0.65	2939
5									
6									
7									
8									
Sum			2.83				15418		10021

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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_c (MPa)	q_s - used (MPa)	Q_{SR} (kN)	ϕ_s	Q_R (kN)
1	-	1.00	-	-	-	1	-	-	-	-	-	-
2	1.00	25.00	68.19	0.07	0.48	1	13.58	0.82	0.82	2590	0.55	1425
3	1.00	62.00	68.19	0.48	0.79	1	13.58	1.35	1.35	4234	0.55	2328
4	0.83	62.00	68.19	0.48	0.79	1	13.58	1.35	1.35	3514	0.55	1933
5	-	-	-	-	-	-	-	-	-	-	-	-
6	-	-	-	-	-	-	-	-	-	-	-	-
7	-	-	-	-	-	-	-	-	-	-	-	-
8	-	-	-	-	-	-	-	-	-	-	-	-
Sum	2.83									10338		5686

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

$$K_b = 4.10$$

Table C10.8.3.5-1

Total vertical stress at the base elvation

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

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

$$\phi = 0.50$$

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	5686 kN	580 T
Deducting pile weight		-160 kN	-16 T
Estimated Pile Capacity		5525 kN	563 T
Maximum Reaction - ULS	Ok	4468 kN	456 T

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EMPIRICAL ESTIMATION OF PILE CAPACITY-PIER P2				Revise			

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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	LRB12A-P2L		Pile Concrete comp. strength	$f'_c =$	30.0	MPa
Bottom of pilecap elevation	EL1 =	-3.50	Concrete Unit Weight	$g_c =$	24.5	kN/m ³
Top of socket elevation	EL2 =	-12.97	Modulus of elasticity of concrete	$E_c =$	27691	MPa
Pile tip elevation	EL3 =	-17.00				
Pile Length	L =	13.50 m	Depth of socket	$H_s =$	4.03	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 =	4784.4 kN				
Working normal force at top of socket	$P_i =$	4737.8 kN				
Intact 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	$I_p = f(H_s/D_s, E_i/E_r) =$	0.30				
	$H_s/D_s =$	4.03				
	$E_i/E_r =$	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_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

$$r_e = (\sum P_i) * H_s / (A_{soc} * E_c) = 0.878 \text{ mm}$$

The settlement of base of drilled shaft

$$r_{base} = (\sum P_i) * I_p / (D_s * E_r) = 1.137 \text{ mm}$$

$$r_e + r_{base} = 2.015 \text{ mm} < 10 \text{ mm}$$

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 $\rightarrow q_s = 0.15 * q_u$

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

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

ϕ_s is the resistance factor - table 10.5.5-3 LRFD

q_u is the uniaxial compressive strength of the rock

Case 1									
No.	EL _T	EL _B	Depth (m)	RQD (%)	q_u (MPa)	q_s (MPa)	Q_{SR} (kN)	ϕ_s	Q_R (kN)
1	-12.97	-13.97	1.00	22	39.97	1.33	4171	0.65	2711
2	-13.97	-14.97	1.00	61	39.97	1.33	4171	0.65	2711
3	-14.97	-15.97	1.00	22	39.97	1.33	4171	0.65	2711
4	-15.97	-17.00	1.03	50	39.97	1.33	4296	0.65	2792
5									
6									
7									
8									
Sum			4.03				16809		10926

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Case2												Type: "1: closed joints", "2: open joints"
No.	Depth (m)	RQD (%)	q _u (MPa)	E _m / E _i	α _B	Type	qs0 (MPa)	q _s (MPa)	q _s - used (MPa)	Q _{sr} (kN)	φ _s	Q _R (kN)
1	1.00	22.00	39.97	0.06	0.46	1	13.58	0.61	0.61	1901	0.55	1046
2	1.00	61.00	39.97	0.45	0.78	1	13.58	1.01	1.01	3185	0.55	1752
3	1.00	22.00	39.97	0.06	0.46	1	13.58	0.61	0.61	1901	0.55	1046
4	1.03	50.00	39.97	0.15	0.59	1	13.58	0.77	0.77	2483	0.55	1366
5	-	-	-	-	-	-	-	-	-	-	-	-
6	-	-	-	-	-	-	-	-	-	-	-	-
7	-	-	-	-	-	-	-	-	-	-	-	-
8	-	-	-	-	-	-	-	-	-	-	-	-
Sum	4.03									9470		5208

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

$$K_b = 4.56$$

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	5208 kN	531 T
Deducting pile weight		-187 kN	-19 T
Estimated Pile Capacity		5021 kN	512 T
Maximum Reaction - ULS	Ok	4524 kN	461 T

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EMPIRICAL ESTIMATION OF PILE CAPACITY-PIER P3		Revise			

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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	LRB12A-P3L	Pile Concrete comp. strength	$f'_c =$	30.0 MPa
Bottom of pilecap elevation	EL1 = -2.50	Concrete Unit Weight	$g_c =$	24.5 kN/m ³
Top of socket elevation	EL2 = -9.80	Modulus of elasticity of concrete	$E_c =$	27691 MPa
Pile tip elevation	EL3 = -14.00			
Pile Length	L = 11.50 m	Depth of socket	$H_s =$	4.20 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 = 4690.0 kN			
Working normal force at top of socket	$P_i =$ 4641.4 kN			
Intact rock modulus	$E_i =$ 5000 MPa			Figure C10.8.3.5-2 Lrfd
Modulus modification ratio	$K_o =$ 0.05			Figure C10.8.3.5-3 Lrfd
Elastic modulus of the insitu rock	$E_r = K_o * E_i =$ 250.0 MPa			
Influence coefficient	$I_p = f(H_s/D_s, E_d/E_r) =$ 0.30			Figure C10.8.3.5-1 Lrfd
	$H_s/D_s =$ 4.20			
	$E_d/E_r =$ 110.77			
Rock mass modulus/ intact 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_B			10.8.3.5.4b-Lrfd 4th

The elastic shortening of the drilled shaft

$$r_e = (\sum P_i) * H_s / (A_{soc} * E_c) = 0.896 \text{ mm}$$

The settlement of base of drilled shaft

$$r_{base} = (\sum P_i) * I_p / (D_s * E_r) = 5.570 \text{ mm}$$

$$r_e + r_{base} = 6.466 \text{ mm} < 10 \text{ mm}$$

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

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

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

Case 1									
No.	EL _T	EL _B	Depth (m)	RQD (%)	q_b (MPa)	q_s (MPa)	Q_{SR} (kN)	ϕ_s	Q_R (kN)
1	-9.80	-10.80	1.00	30	66.09	1.71	5363	0.65	3486
2	-10.80	-11.80	1.00	20	66.09	1.71	5363	0.65	3486
3	-11.80	-13.80	2.00	20	66.09	1.71	10727	0.65	6972
4	-13.80	-14.00	0.20	30	66.09	1.71	1073	0.65	697
5									
6									
7									
8									
Sum			4.20				22526		14642

	DANANG QUANG NGAI EXPRESSWAY	Item.	Eng.	Date.	Sign.
	LRB11 BRIDGE	Design			
	DETAIL DESIGN	Check			
	EMPIRICAL ESTIMATION OF PILE CAPACITY-PIER P3	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	30.00	66.09	0.07	0.48	2	13.58	0.81	0.81	2550	0.55	1402
2	1.00	20.00	66.09	0.05	0.45	2	13.58	0.76	0.76	2374	0.55	1306
3	2.00	20.00	66.09	0.05	0.45	2	13.58	0.76	0.76	4748	0.55	2612
4	0.20	30.00	66.09	0.07	0.48	2	13.58	0.81	0.81	510	0.55	280
5	-	-	-	-	-	-	-	-	-	-	-	-
6	-	-	-	-	-	-	-	-	-	-	-	-
7	-	-	-	-	-	-	-	-	-	-	-	-
8	-	-	-	-	-	-	-	-	-	-	-	-
Sum	4.20									10182		5600

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

$$K_b = 4.62$$

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	$Q_f = 0.75 \cdot 0.85 \cdot 0.85 \cdot f_c \cdot A_g$	12768 kN	1301 T
Pile resistance	Q_R	5600 kN	571 T
Deducting pile weight		-165 kN	-17 T
Estimated Pile Capacity		5435 kN	554 T
Maximum Reaction - ULS	Ok	4468 kN	456 T

MINISTRY OF TRANSPORT

VIETNAM EXPRESSWAY CORPORATION - PROJECT MANAGEMENT UNIT NO.85

DA NANG - QUANG NGAI EXPRESSWAY DEVELOPMENT PROJECT

PACKAGE: A2

BRIDGE

LRB 12a

CALCULATION SHEETS

MISCELLANEOUS

	DA NANG - QUANG NGAI EXPRESSWAY PROJECT					
	LRB12a BRIDGE					
	DETAIL DESIGN					
	STEEL DOWEL AND EXPANSION JOINT					

STEEL DOWEL AND EXPANSION JOINT

I. Displacement

Maximum allowable displacements in longitudinal direction = 40 mm
Maximum displacement 25.4 OK
A1

Unit (mm)

Tải trọng	Symbol	Sign	Displacement	Service
			Case1	a
TU+	TU	+	7.00	1.20
TU-	TU	-	-7.00	1.20
Cr&Sh	CR&SH	-	13.00	1.20
Other loads		±	1.40	1.00

Max Stretch = 25.4
Max Shrink = 8.6
Maximum displacement 25.4

II. Force in Pier

Galvanised steel dowel
D = 32.0 mm
Number = 12.0 bar
 f_u = 380 Mpa
Resistance force $R_n = 0.48 \cdot A \cdot f_u$ = 1760.3 kN OK S-6.13.2.7

Load	Symbol	Force (kN)	a	b
Cr&Sh	CR&SH	198	-	0.50
TU	TU	117	-	0.50
EQ		846.4	1.00	-

Maximale shear force $Q = \max(CR\&SH+TU, EQ)/0.8 = 1058.0 \text{ kN}$

7 FO09

Table of content - F009 Bridge

A. Superstructure design

1. Voided slab girder
2. Fibre stress check of voided slab girder
3. Flexure, Shear and torsion of voided slab girder
4. Deck Slab of voided slab girder
5. Cross beam for voided slab girder

B. Substructure design

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

C. Miscellaneous

1. Bearings
2. Expansion Joint
3. Steel Dowels
4. Parapet

MINISTRY OF TRANSPORT

VIET NAM EXPRESSWAY CORPORATION - PROJECT MANAGEMENT UNIT NO.85

DA NANG - QUANG NGAI EXPRESS WAY PROJECT
PACKAGE: A2

BRIDGE
F009

CALCULATION SHEETS
SUPERSTRUCTURE

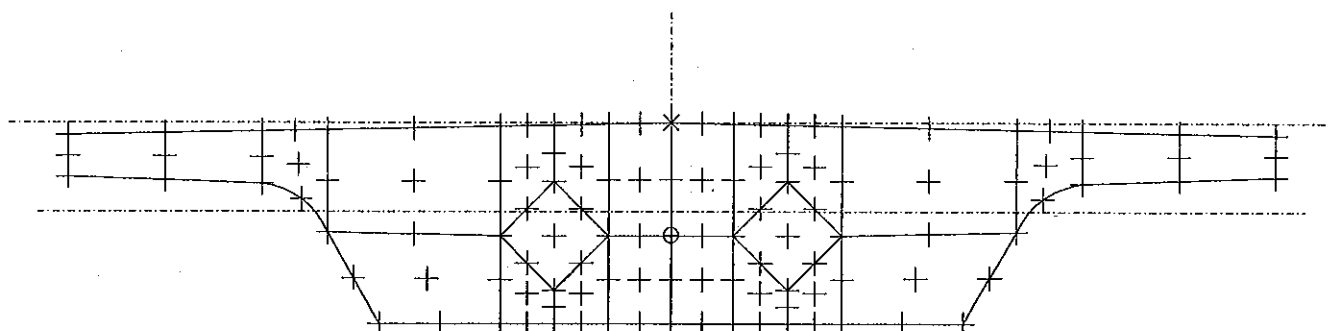
CALCULATION SHEET

VOIDED SLAB GIRDER

Cross-section : Dac001

Part : 1 Variant : 1

Description : Dac



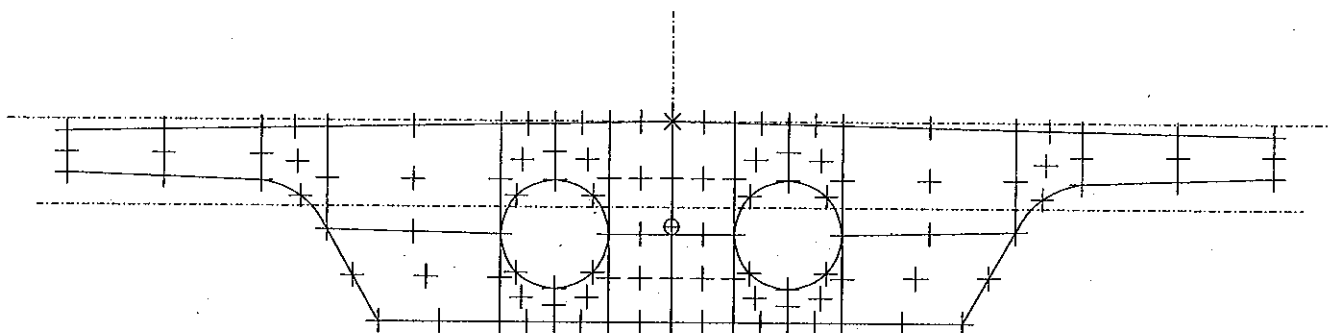
CROSS-SECTION DATA

Cross-section area	0.56755E+01	m2
Shear area - Bending about Z-axis	0.20571E+01	m2
Shear area - Bending about Y-axis	0.51696E+01	m2
Torsional moment of inertia I	0.18909E+01	m4
Moment of inertia about Y-axis	0.13853E+02	m4
Eccentricity Z (Shear lag)	0.00000E+00	m
Shear lag factor Y	0.10000E+01	
Moment of inertia about Y-axis (Shear lag)	0.13853E+02	m4
Moment of inertia about Z-axis	0.65683E+00	m4
Eccentricity Y (Shear lag)	0.00000E+00	m
Shear lag factor Z	0.10000E+01	
Moment of inertia about Z-axis (Shear lag)	0.65683E+00	m4
Warping moment of inertia	0.60581E+00	m6
Bending axis origin - Eccentricity ey	-0.53081	m
Bending axis origin - Eccentricity ez	0.00000	m
Main axis angle	0.00000	Deg
Shear axis origin - Eccentricity ey	-0.67189	m
Shear axis origin - Eccentricity ez	0.00000	m
Main axis angle - Shear	0.00000	Deg
Y-below : Gravity centre - minY	0.66919	m
Y-above : Gravity centre - maxY	0.53081	m
Z-left : Gravity centre - minZ	3.62000	m
Z-right : Gravity centre - maxZ	3.62000	m
Perimeter exposed to drying (outside)	15.84916	m
Perimeter (inside)	0.00000	m

Cross-section : Rong:001

Part : 1 Variant : 1

Description : Rong



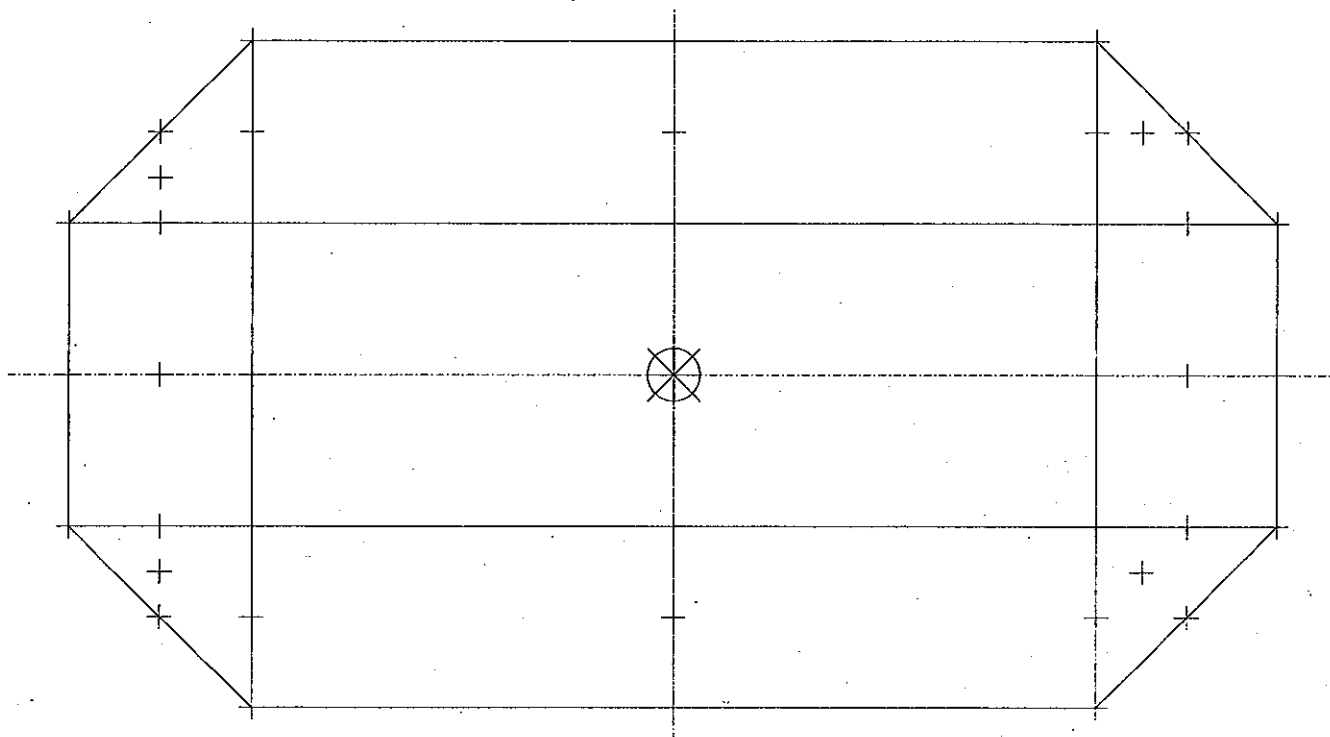
CROSS-SECTION DATA

Cross-section area	0.50197E+01	m2
Shear area - Bending about Z-axis	0.17864E+01	m2
Shear area - Bending about Y-axis	0.38377E+01	m2
Torsional moment of inertia I	0.17724E+01	m4
Moment of inertia about Y-axis	0.13514E+02	m4
Eccentricity Z (Shear lag)	0.00000E+00	m
Shear lag factor Y	0.10000E+01	
Moment of inertia about Y-axis (Shear lag)	0.13514E+02	m4
Moment of inertia about Z-axis	0.62430E+00	m4
Eccentricity Y (Shear lag)	0.00000E+00	m
Shear lag factor Z	0.10000E+01	
Moment of inertia about Z-axis (Shear lag)	0.62430E+00	m4
Warping moment of inertia	0.62083E+00	m6
Bending axis origin - Eccentricity ey	-0.51197	m
Bending axis origin - Eccentricity ez	0.00000	m
Main axis angle	0.00000	Deg
Shear axis origin - Eccentricity ey	-0.62911	m
Shear axis origin - Eccentricity ez	0.00000	m
Main axis angle - Shear	0.00000	Deg
Y-below : Gravity centre - minY	0.68803	m
Y-above : Gravity centre - maxY	0.51197	m
Z-left : Gravity centre - minZ	3.62000	m
Z-right : Gravity centre - maxZ	3.62000	m
Perimeter exposed to drying (outside)	15.84916	m
Perimeter (inside)	4.07589	m

Cross-section : PCL-P1:001

Part : 1 Variant : 1

Description : PCL-P1



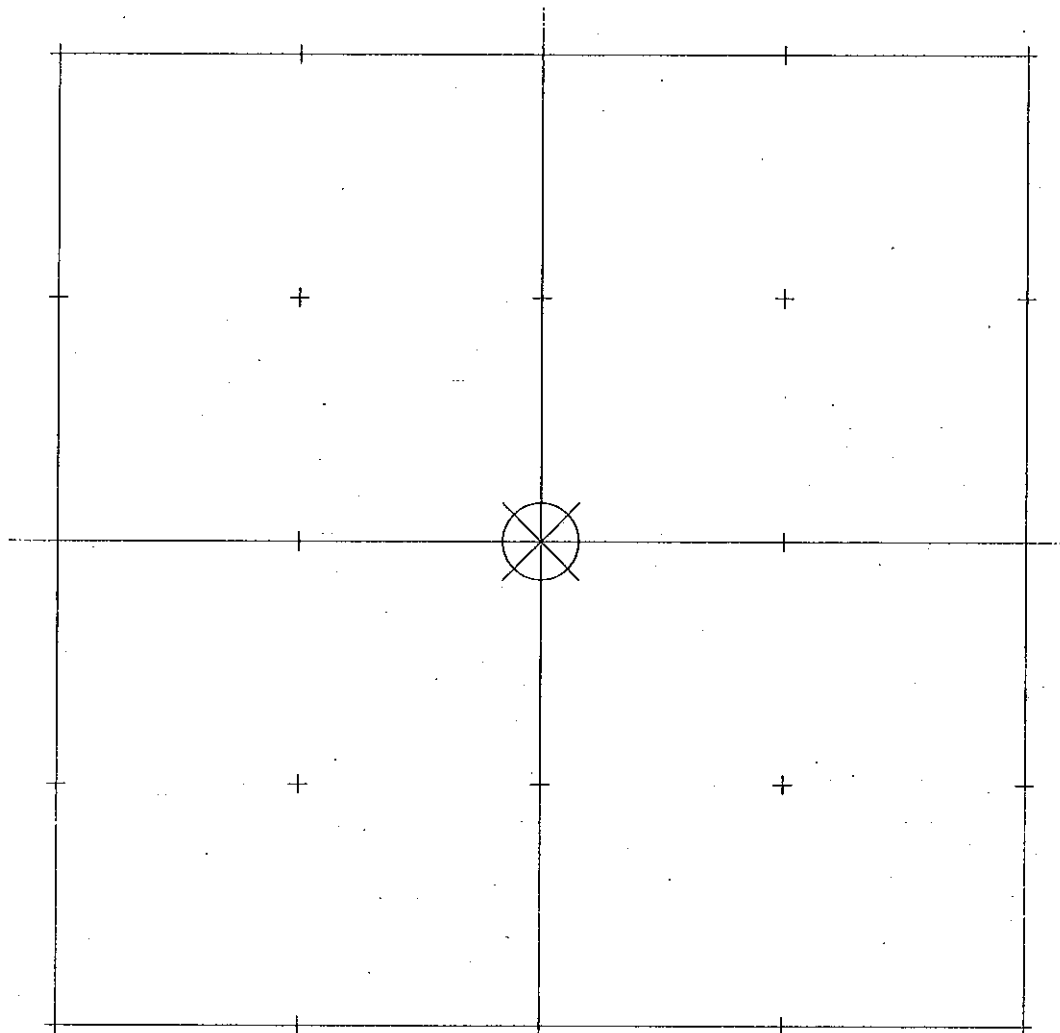
CROSS-SECTION DATA

Cross-section area	0.20200E+01	m2
Shear area - Bending about Z-axis	0.17053E+01	m2
Shear area - Bending about Y-axis	0.17949E+01	m2
Torsional moment of inertia I	0.54765E+00	m4
Moment of inertia about Y-axis	0.58663E+00	m4
Eccentricity Z (Shear lag)	0.00000E+00	m
Shear lag factor Y	0.10000E+01	
Moment of inertia about Y-axis (Shear lag)	0.58663E+00	m4
Moment of inertia about Z-axis	0.18448E+00	m4
Eccentricity Y (Shear lag)	0.00000E+00	m
Shear lag factor Z	0.10000E+01	
Moment of inertia about Z-axis (Shear lag)	0.18448E+00	m4
Warping moment of inertia	0.12541E-01	m6
Bending axis origin - Eccentricity ey	0.00000	m
Bending axis origin - Eccentricity ez	0.00000	m
Main axis angle	0.00000	Deg
Shear axis origin - Eccentricity ey	0.00000	m
Shear axis origin - Eccentricity ez	0.00000	m
Main axis angle - Shear	0.00000	Deg
Y-below : Gravity centre - minY	0.55000	m
Y-above : Gravity centre - maxY	0.55000	m
Z-left : Gravity centre - minZ	1.00000	m
Z-right : Gravity centre - maxZ	1.00000	m
Perimeter exposed to drying (outside)	5.49706	m
Perimeter (inside)	0.00000	m

Cross-section : PCAP-P1.001

Part : 1 Variant : 1

Description : PCAP-P1



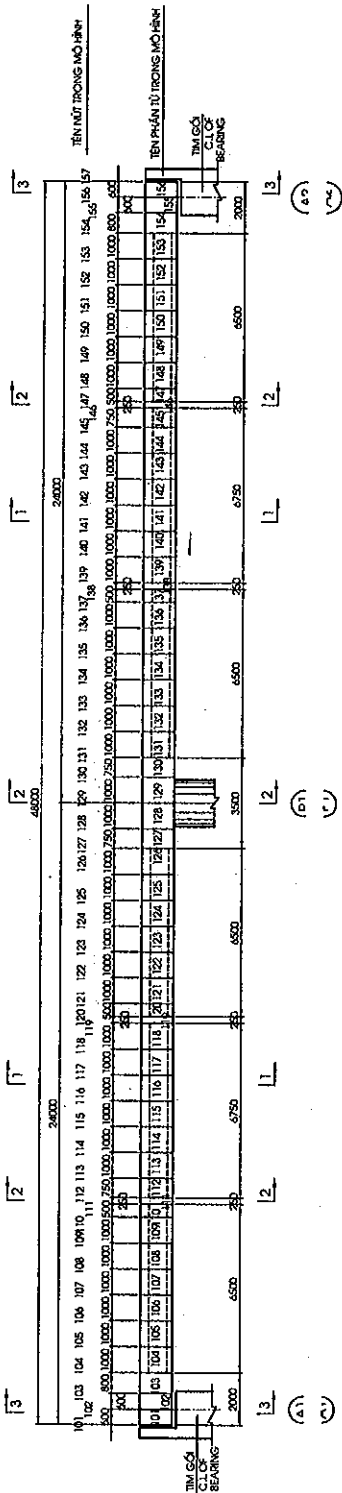
CROSS-SECTION DATA

Cross-section area	0.25000E+02 m2
Shear area - Bending about Z-axis	0.21053E+02 m2
Shear area - Bending about Y-axis	0.21053E+02 m2
Torsional moment of inertia I	0.89143E+02 m4
Moment of inertia about Y-axis	0.52083E+02 m4
Eccentricity Z (Shear lag)	0.00000E+00 m
Shear lag factor Y	0.10000E+01
Moment of inertia about Y-axis (Shear lag)	0.52083E+02 m4
Moment of inertia about Z-axis	0.52083E+02 m4
Eccentricity Y (Shear lag)	0.00000E+00 m
Shear lag factor Z	0.10000E+01
Moment of inertia about Z-axis (Shear lag)	0.52083E+02 m4
Warping moment of inertia	0.27087E+01 m6
Bending axis origin - Eccentricity ey	0.00000 m
Bending axis origin - Eccentricity ez	0.00000 m
Main axis angle	0.00000 Deg
Shear axis origin - Eccentricity ey	0.00000 m
Shear axis origin - Eccentricity ez	0.00000 m
Main axis angle - Shear	0.00000 Deg
Y-below : Gravity centre - minY	2.50000 m
Y-above : Gravity centre - maxY	2.50000 m
Z-left : Gravity centre - minZ	2.50000 m
Z-right : Gravity centre - maxZ	2.50000 m
Perimeter exposed to drying (outside)	20.00000 m
Perimeter (inside)	0.00000 m

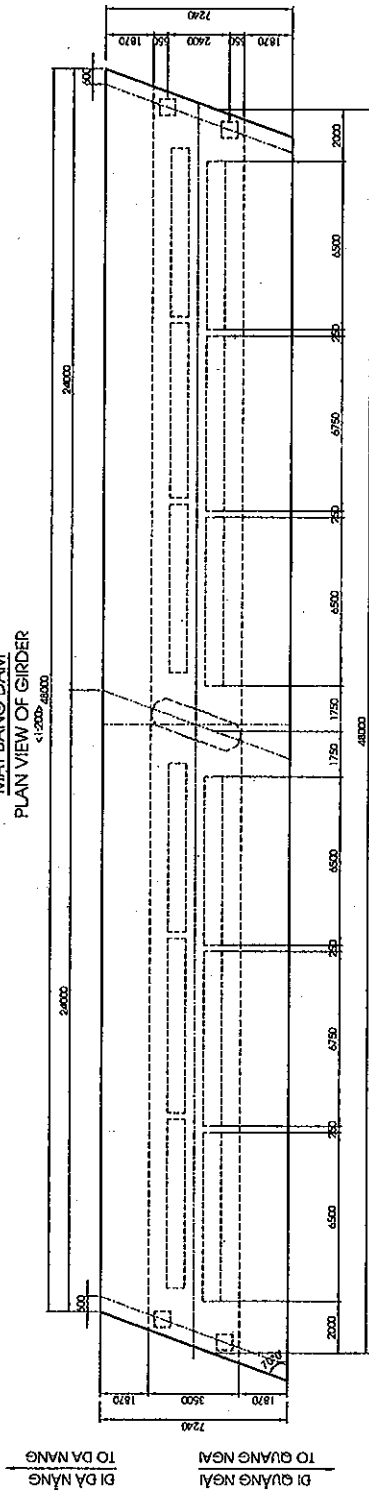
BỐ TRÍ CHUNG DẦM GENERAL VIEW OF GIRDER

NH1

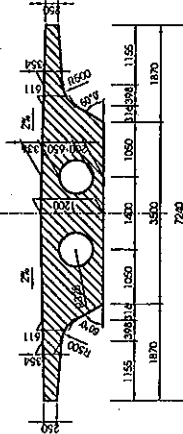
DONG NHON LAKE

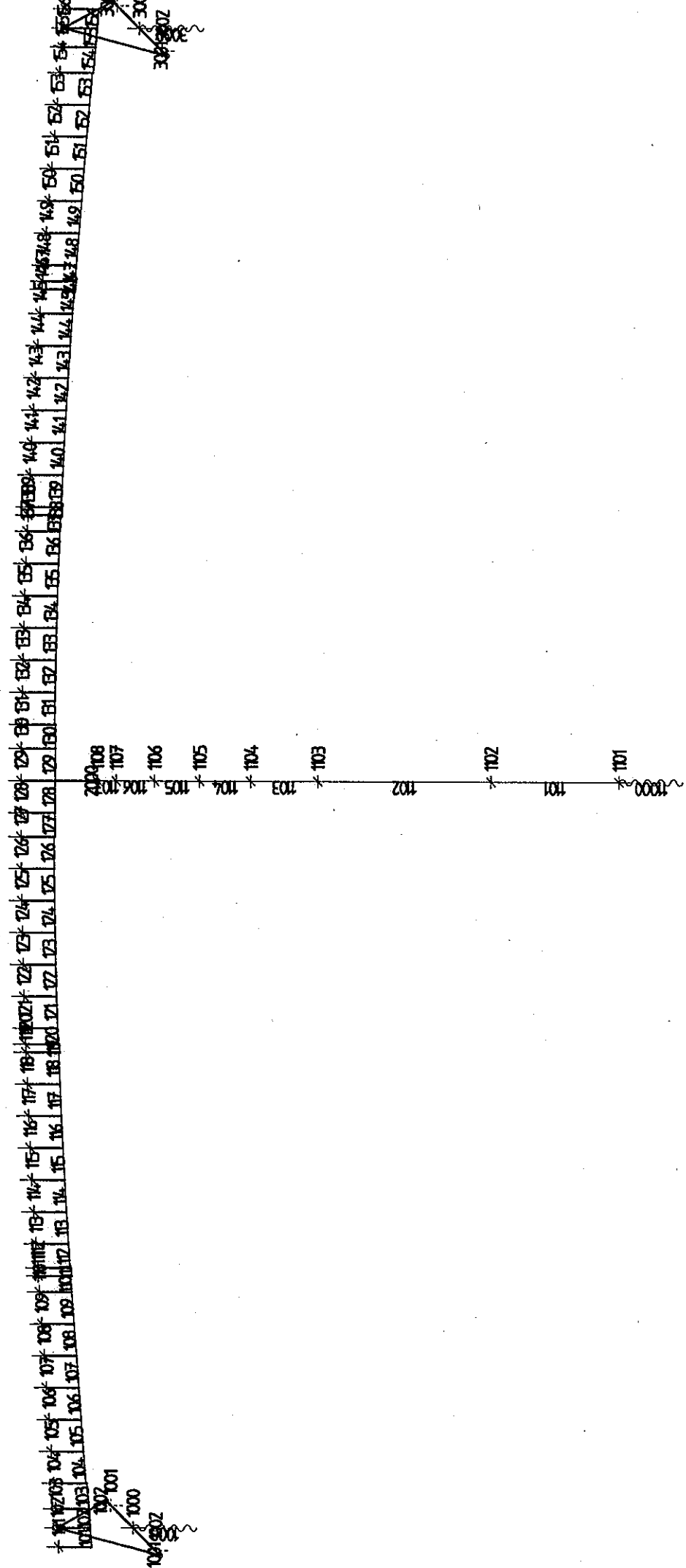


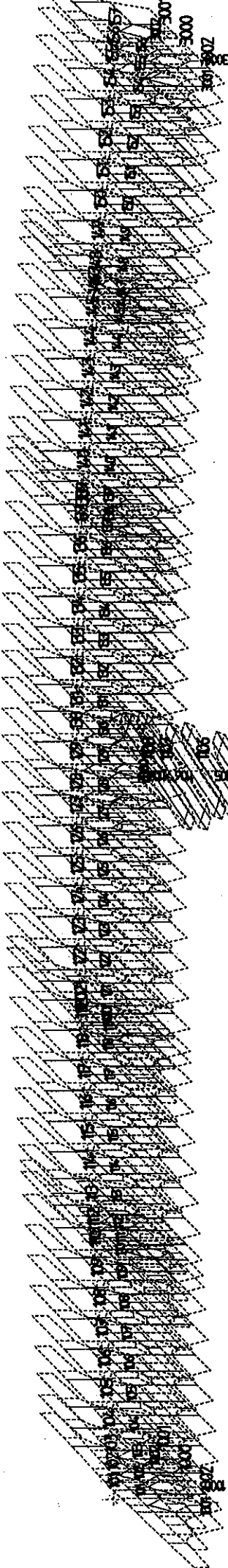
MẶT BẰNG DẦM PLAN VIEW OF GIRDER



MẶT CẮT 1-1 SECTION 1-1



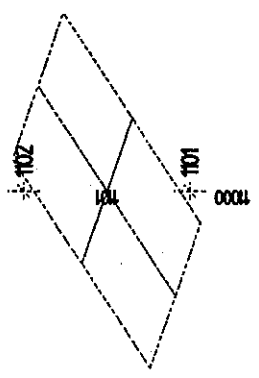




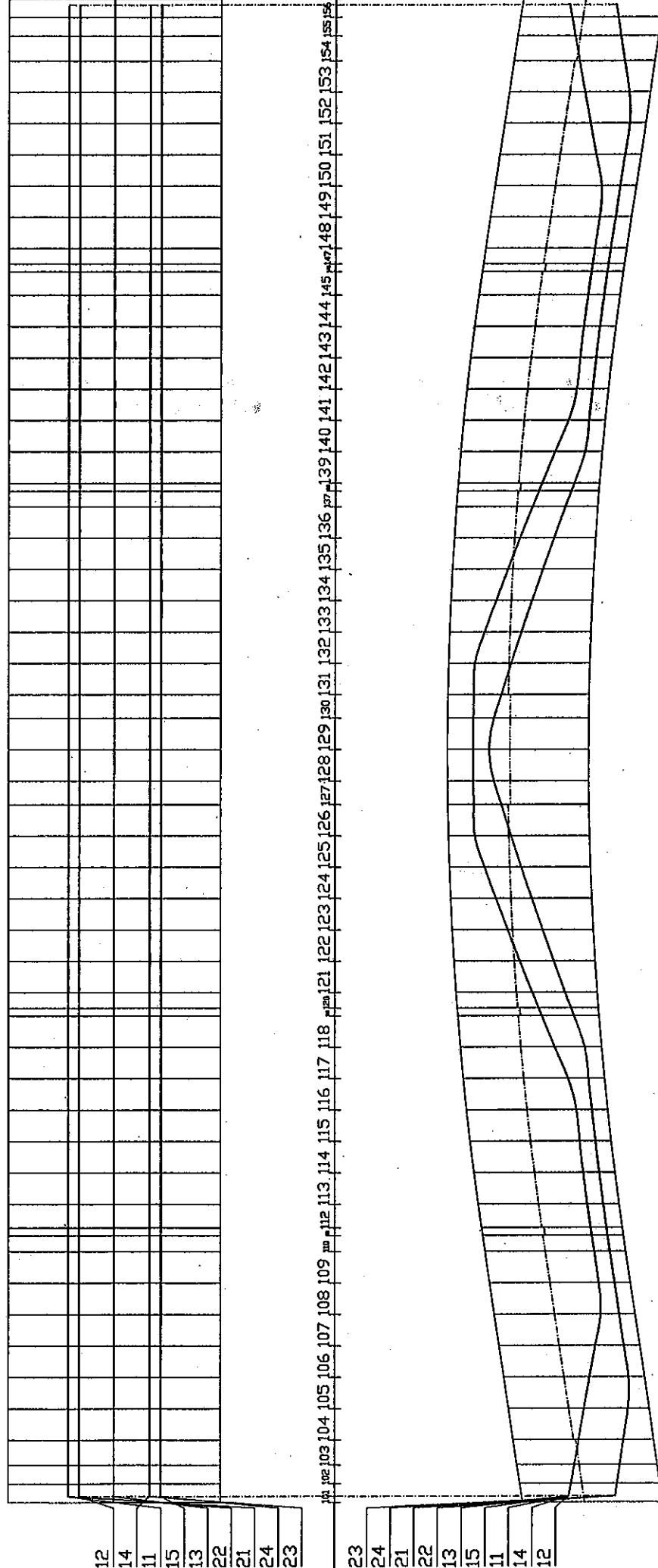
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Plan



Elevation

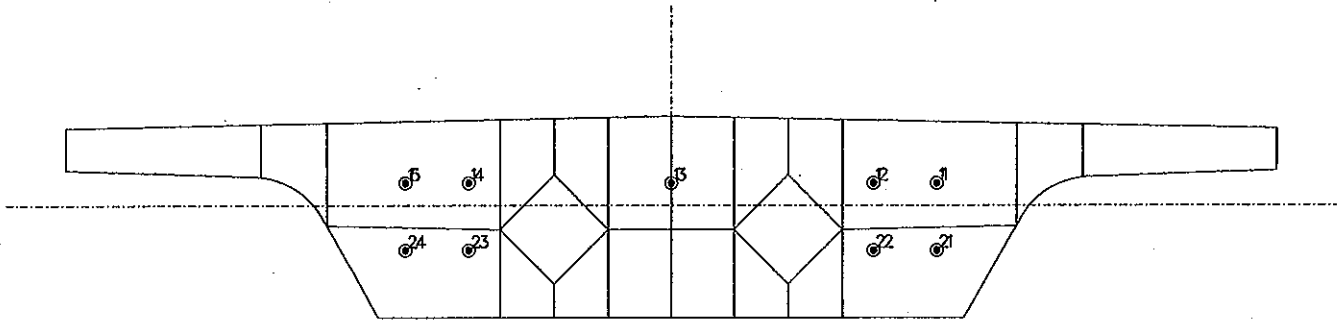
Project: FD09
DU AN DUONG CAO TDC DA NANG - QUANG NGAI

Tendons 11 12 13 14 15 21 22 23 24
Schematic figure

31/01/2013
16:56

Element:101 - Cross-section : 001 - begin

Description : Dac



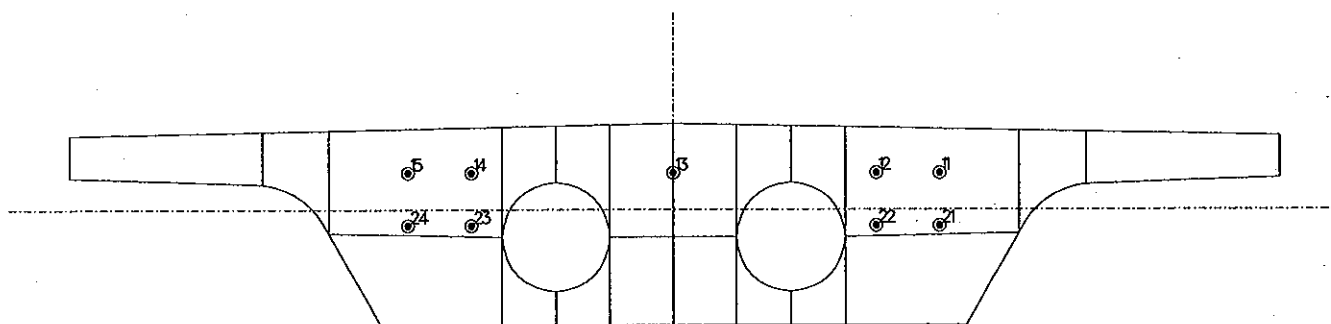
C R O S S - S E C T I O N D A T A

Y-below	:	Gravity centre - minY	0.66919	m
Y-above	:	Gravity centre - maxY	0.53081	m
Z-left	:	Gravity centre - minZ	3.62000	m
Z-right	:	Gravity centre - maxZ	3.62000	m

[illegible]

[illegible]

Description : Rong

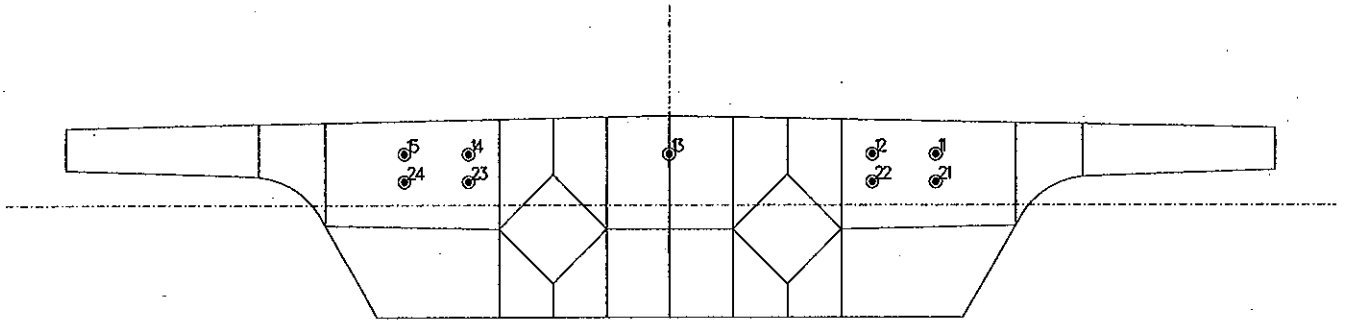


Y-below	:	Gravity centre - minY	0.68803	m
Y-above	:	Gravity centre - maxY	0.51197	m
Z-left	:	Gravity centre - minZ	3.62000	m
Z-right	:	Gravity centre - maxZ	3.62000	m

[illegible]

Element:128 - Cross-section : 001 - begin

Description : Dac



CROSS - SECTION DATA

Y-below : Gravity centre - minY

0.66919 m

Y-above : Gravity centre - maxY

0.53081 m

Z-left : Gravity centre - min Z

3.62000 m

Z-right : Gravity centre - maxZ

3.62000 m

[illegible]

kNm

Minimum -8459.0

-8000

-7000

-6000

-5000

-4000

-3000

-2000

-1000

0

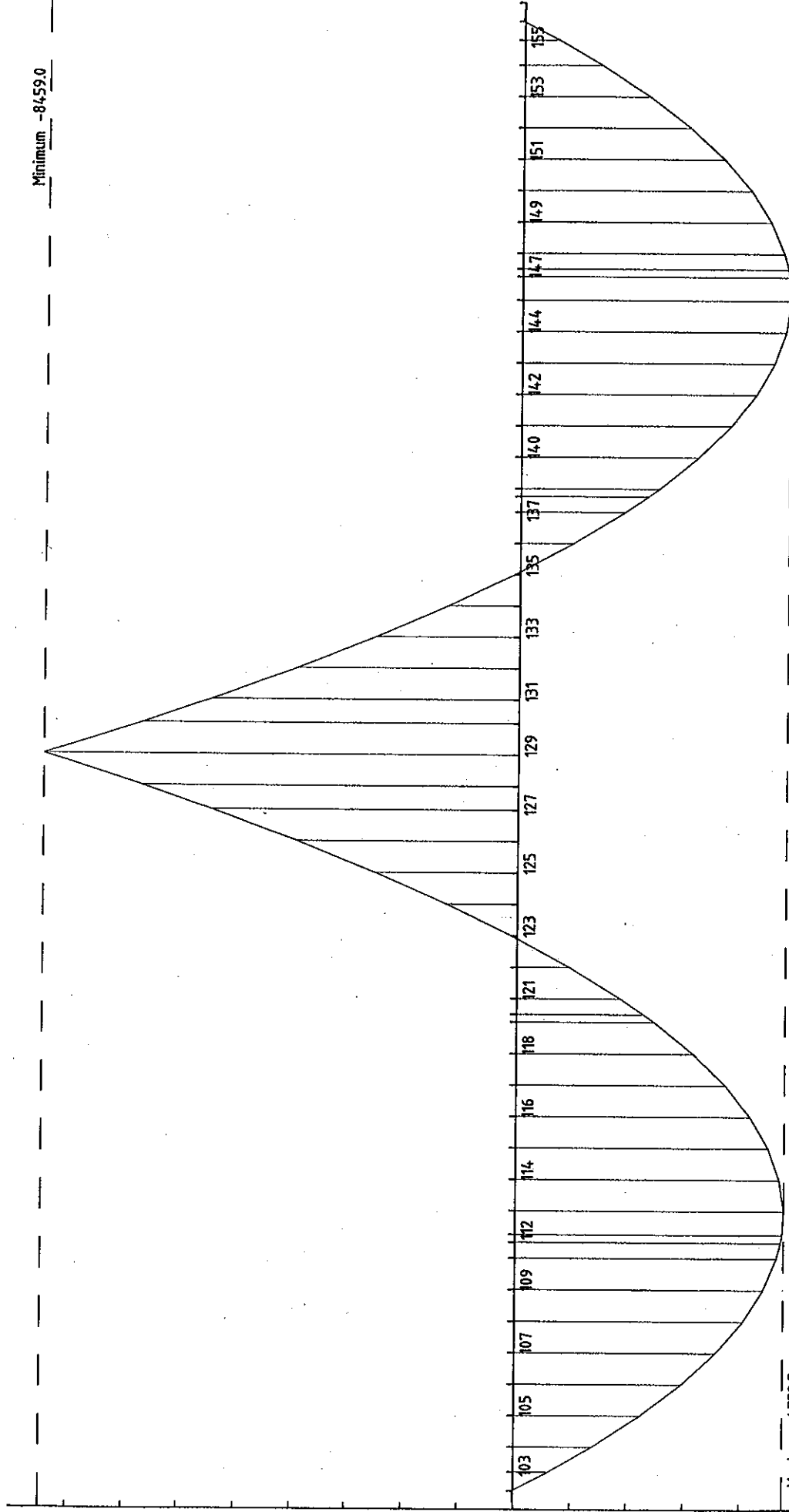
1000

2000

3000

4000

Maximum 4758.5



MO MEN DO TAI TRONG BAN THAN
Results: LC:Mz,total local joined
RnSet: SW-Mz, Load case: SW-SUM

04/01/2013
13:26

Project: F009
DÙ AN DUONG CAO TỐC DA NANG - QUANG NGAI

RM2004

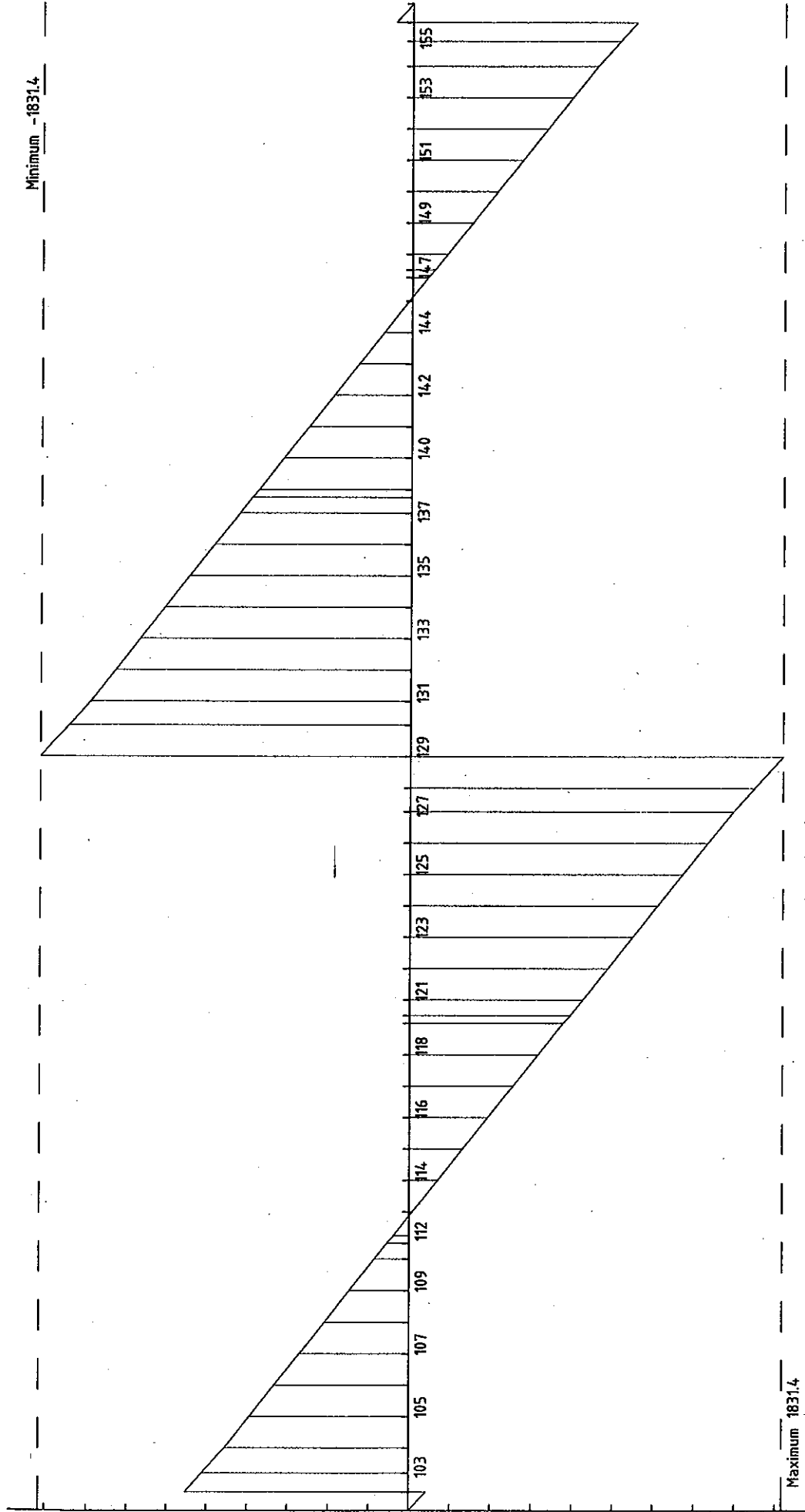
1 cm Plot = 1046.9 kNm

0 104.9 209.9 314.8 420.7 526.6

kN

-1800
-1600
-1400
-1200
-1000
-800
-600
-400
-200
0
200
400
600
800
1000
1200
1400
1600
1800

Minimum -1831.4



Maximum 1831.4

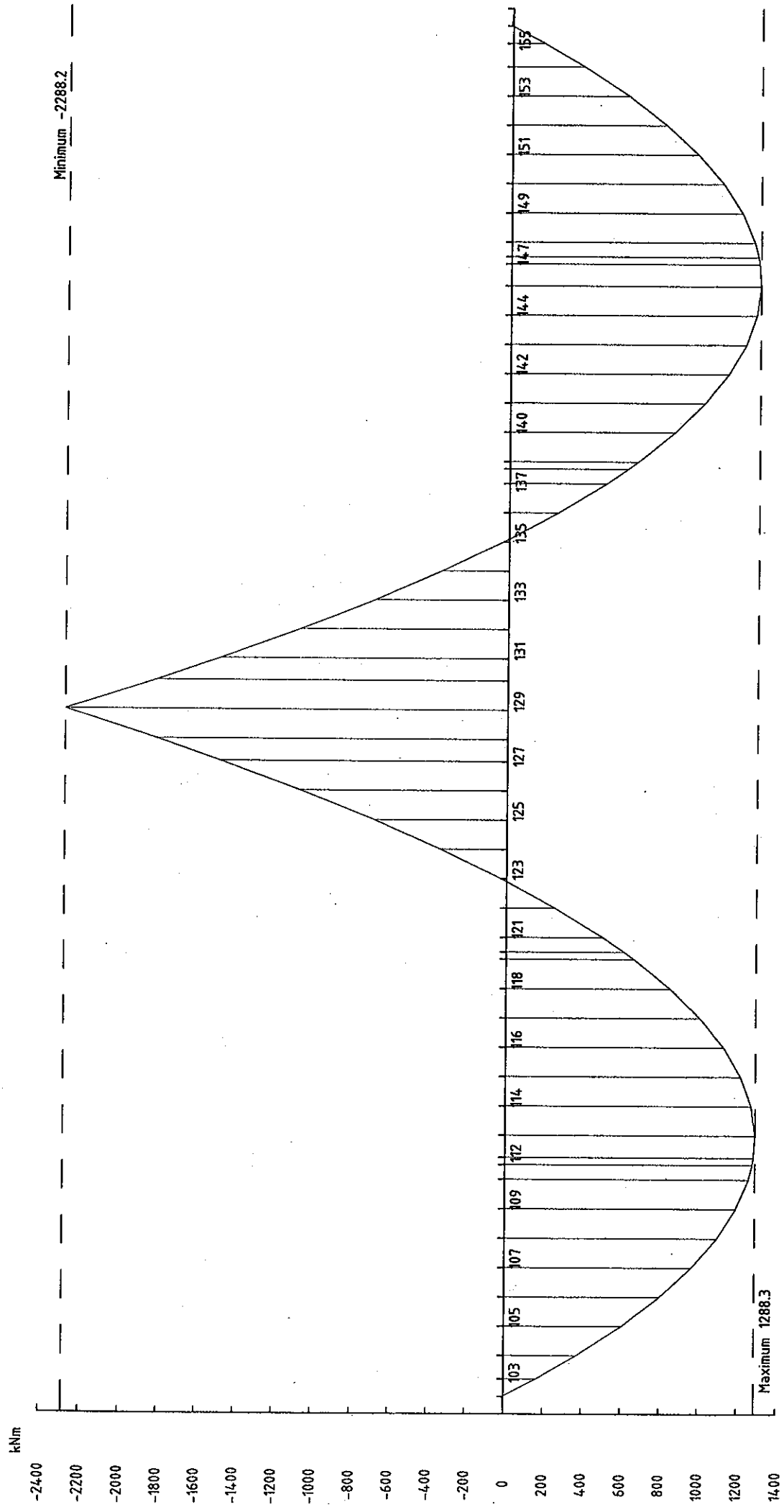
LUC CAT DO TAI TRONG BAN THAN
Results: LC-Qy, total local joined
RmSet: SW-Qy, Load case: SW-SUM

Project: F009
DU AN DUONG CAO TOC DA NANG - QUANG NGAI

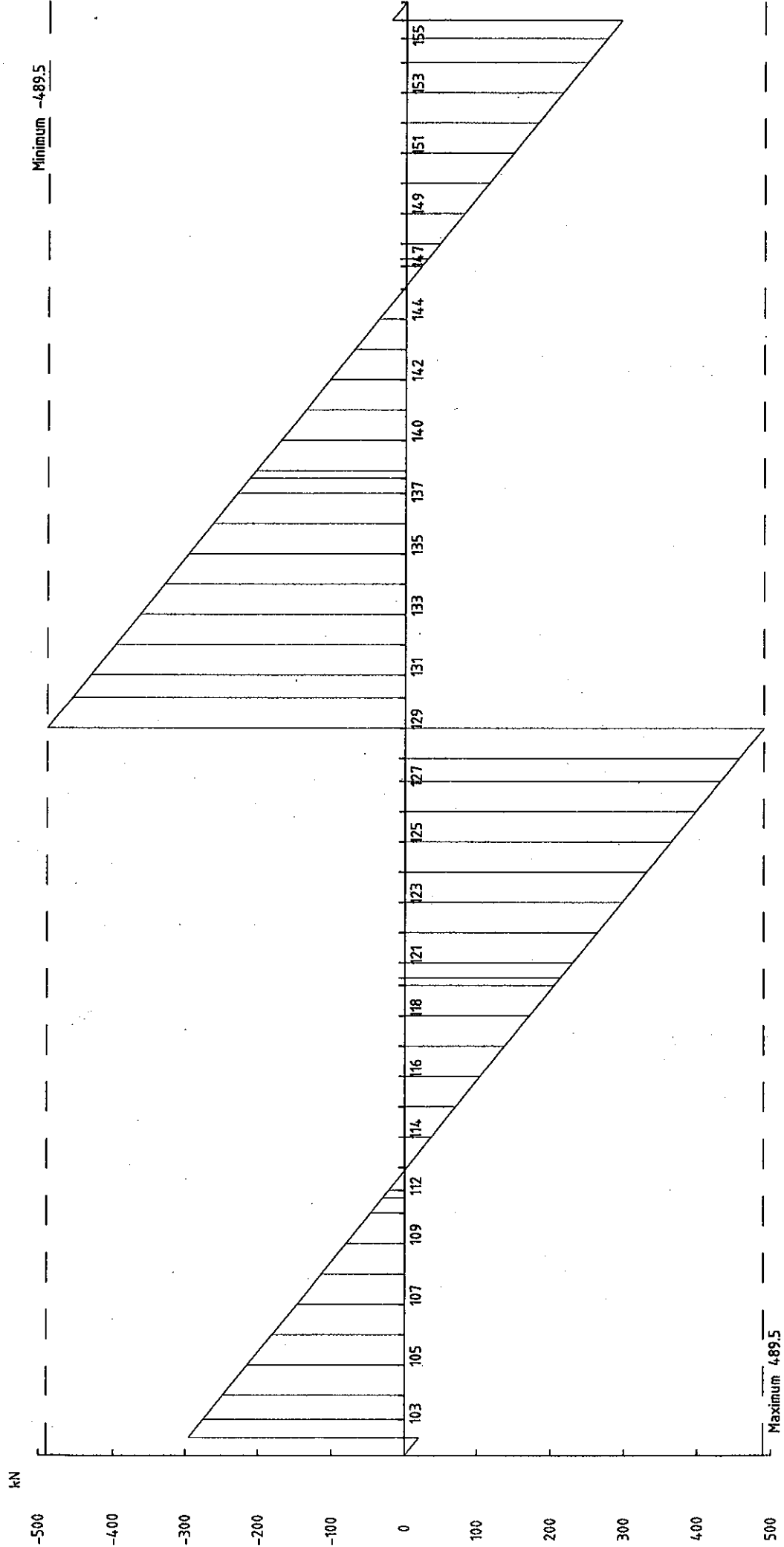
04/01/2013
13:26

1 cm Plot = 290.1 kN

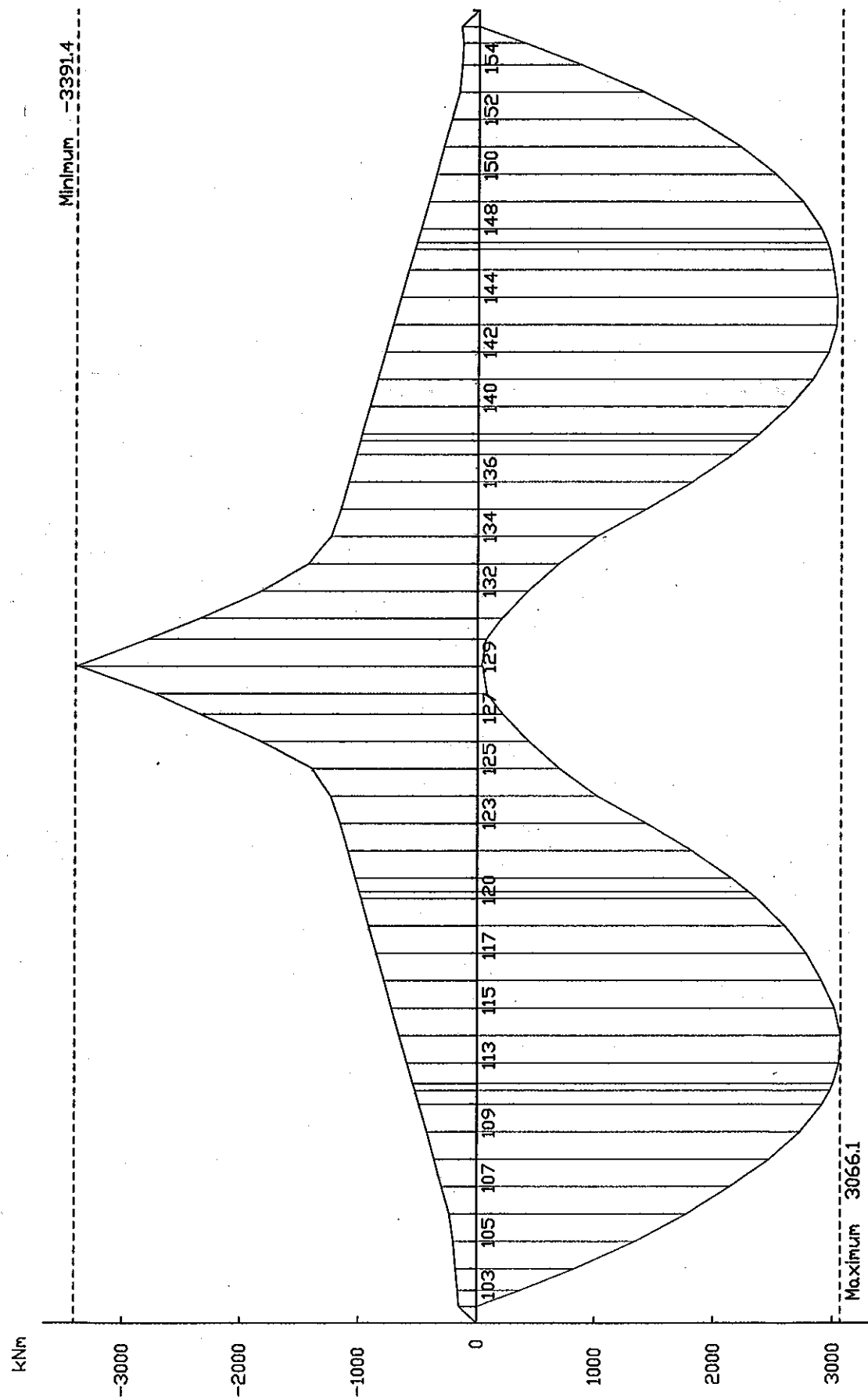
NSA



<div>04/01/2013</div> <div>13:26</div>	<div>MO MEN DO TINH TAI PHAN 2</div> <div>Results: LC:Mz,total local joined</div> <div>RmSet: SDL-Mz, Load case: SDL-SUM</div>	<div>Project: F009</div> <div>DU AN DUONG CAO TOC DA NANG - QUANG NGAI</div>
<div>1 cm Plot = 283.3 kNm</div> <div>283.3 546.6 864.9 1081.2 1363.5</div>		



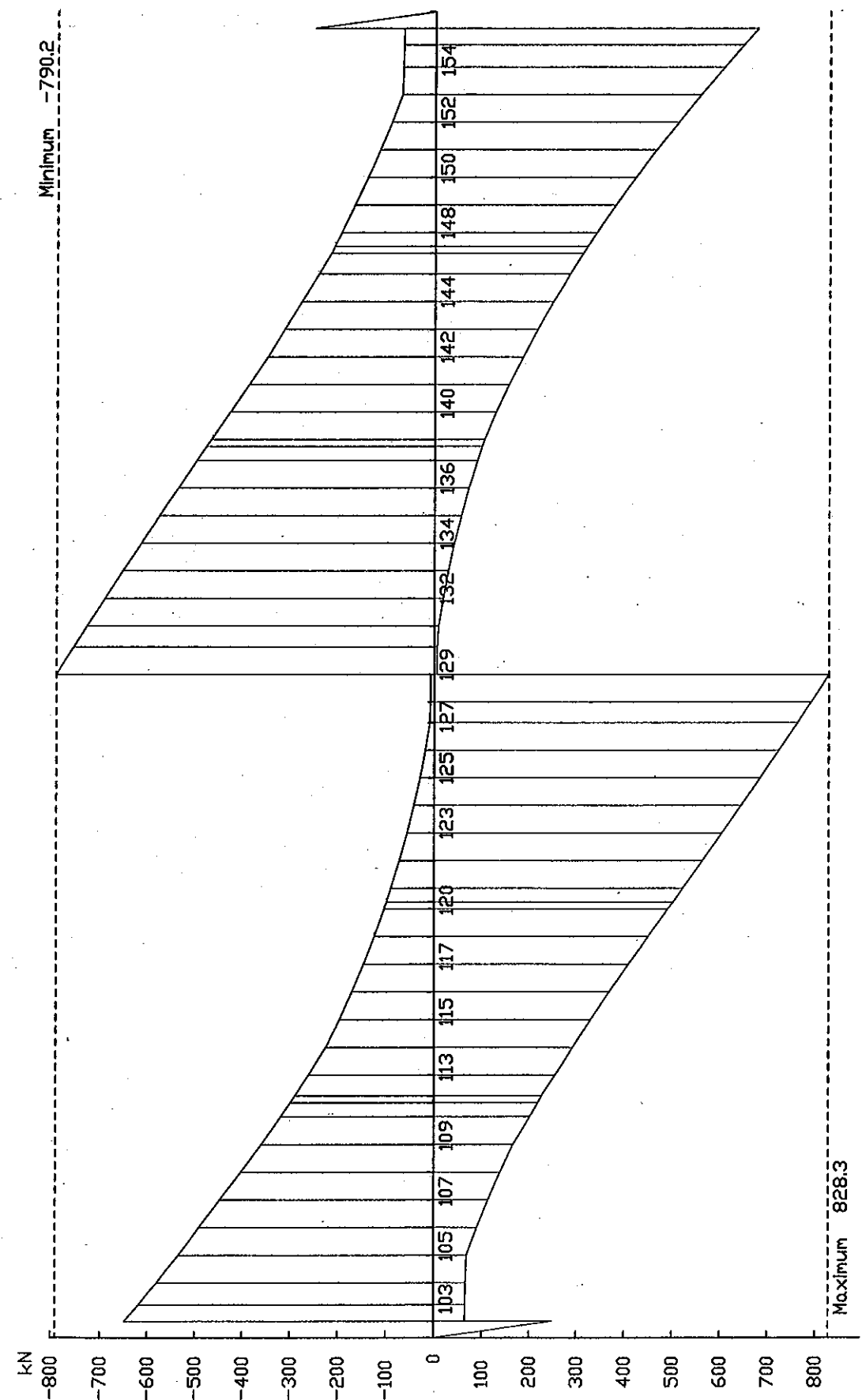
04/01/2013 13:27	
<div>LUC CAT DO TINH TAI PHAN 2</div> <div>Results: LC-Qy, total local joined</div> <div>RnSet: SDL-Qy, Load case: SDL-SUM</div>	<div>1 cm Plot = 77.54 kN</div> <div>0 77.54 155.08 232.63 309.17 385.72</div>
<div>Project: F009</div> <div>DU AN DUONG CAO TOC DA NANG - QUANG NGAI</div>	



MO MEN DO HOAT TAI

01/02/2013
9:45

1 cm Plot = 511.5 kNm

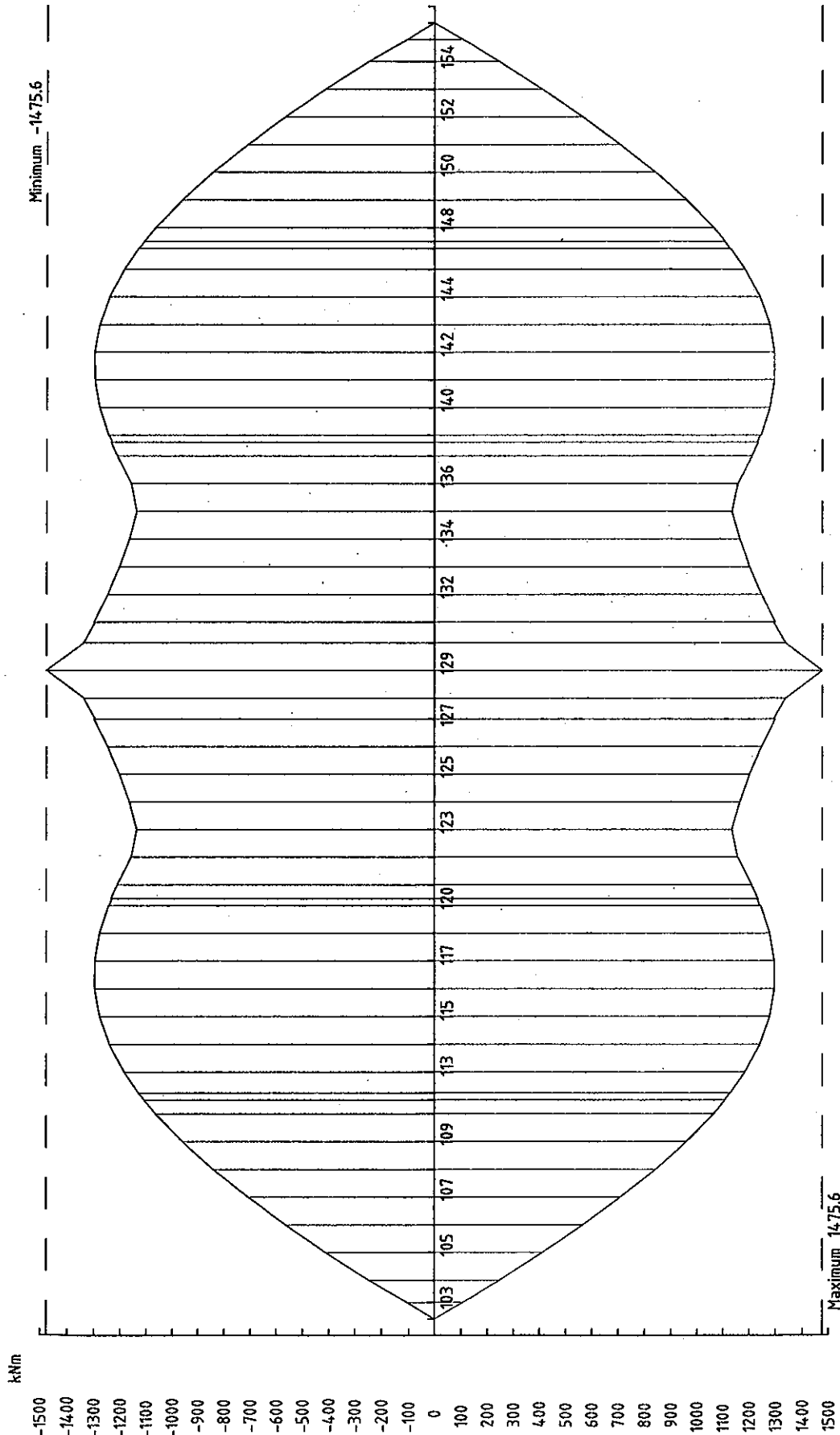


LiveLoad.sup
MAXQyQy
totallocal,joined
(0 , 828)

LiveLoad.sup
MINQyQy
totallocal,joined
(-790 , 0)

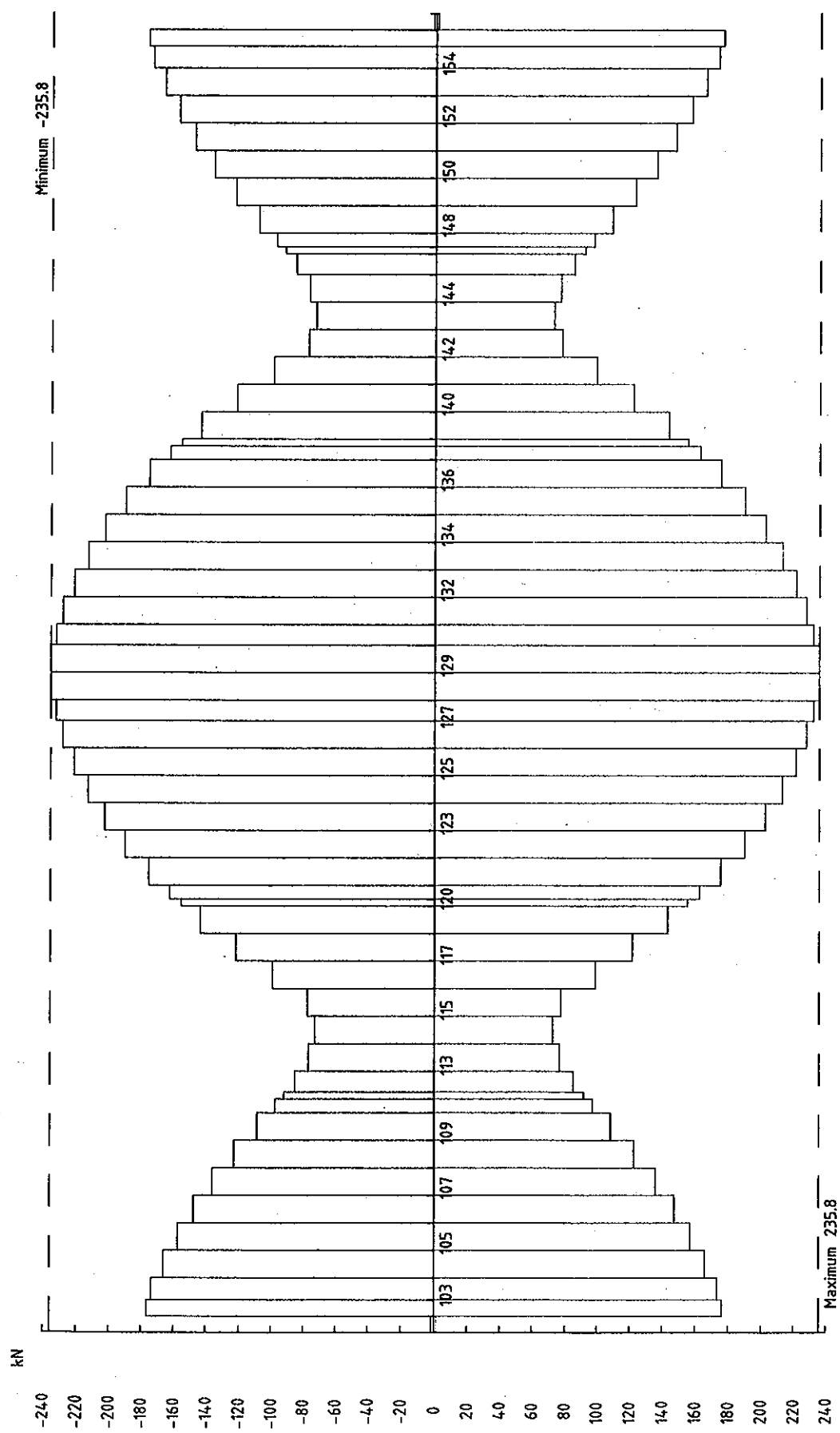
EQ sup
MAXMz:Mz
total:local:joined
(0 , 14.76)

EQ sup
MINMz:Mz
total:local:joined
(-14.76 , 0)

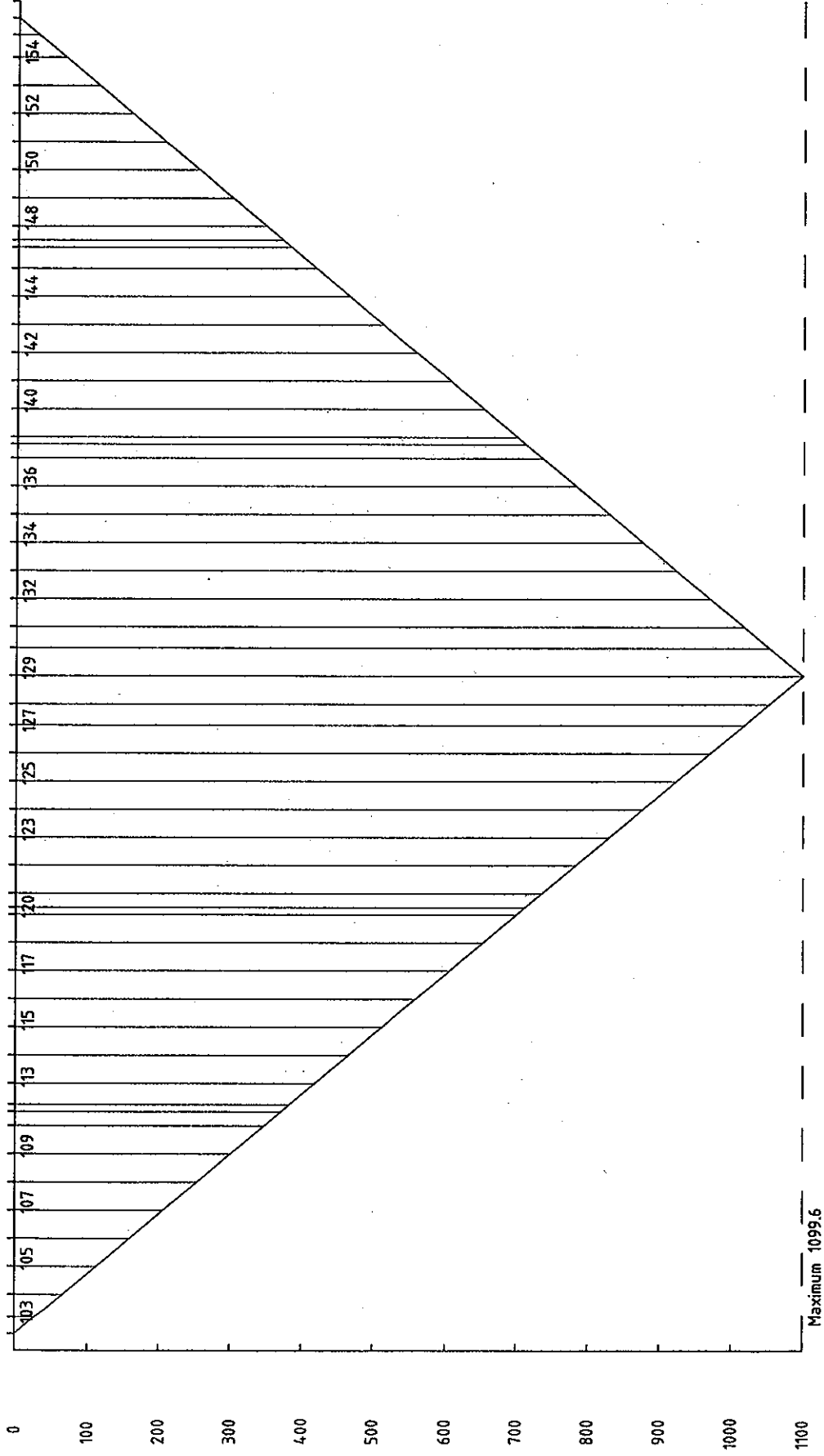


EQ.sup
MAXQy:Qy
totallocal:joined
(0 , 236)

EQ.sup
MINQy:Qy
totallocal:joined
(-236 , 0)



kNm



SE sup
MAXMz:Mz
totallocal:joined
(0 , 1100)

SE sup
MINMz:Mz
totallocal:joined
(0 , 1100)

MO MEN DO GOI LUN

Project: F009
DU AN DUONG CAO TOC DA NANG - QUANG NGAI

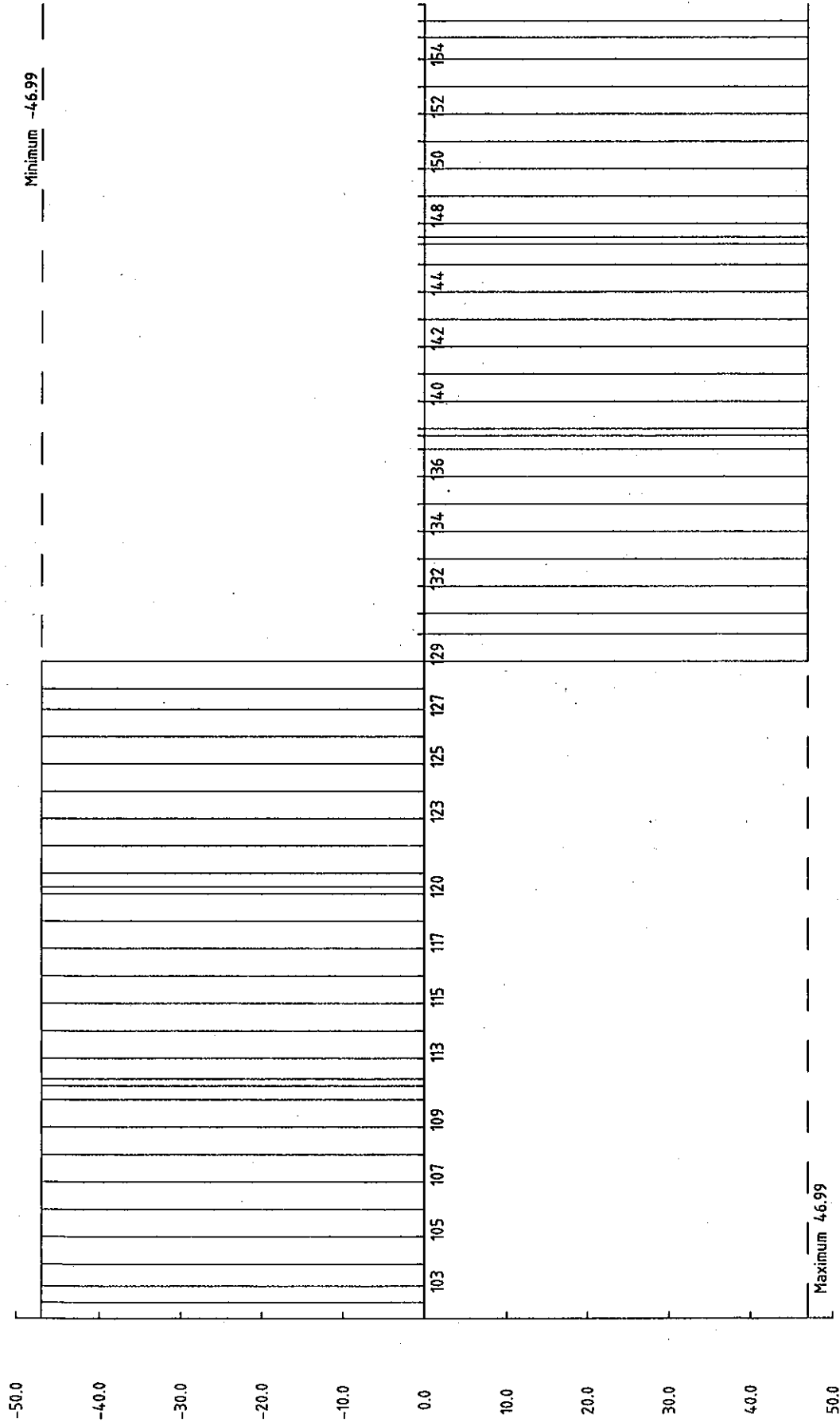
RmSet: SE-Mz

1 cm Plot = 87.10 kNm

0 67.18 114.20 261.39 344.48 435.10

04/10/2013
13:39

kN



SE:sup
MAXQy:Qy
totallocal:joined
(-47.0 , 47.0)

SE:sup
MINQy:Qy
totallocal:joined
(-47.0 , 47.0)

LUC CAT DO GOI LUN

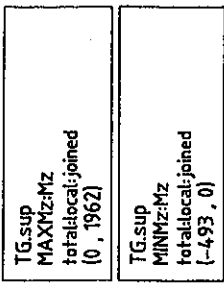
Project: F009
DU AN DUONG CAO TOC DA NANG - QUANG NGAI

RmSet: SE-Qy

1 cm Plot = 7.444 kN

04/01/2013
13:39

22.350 29.177 37.223



04/01/2013
13:38

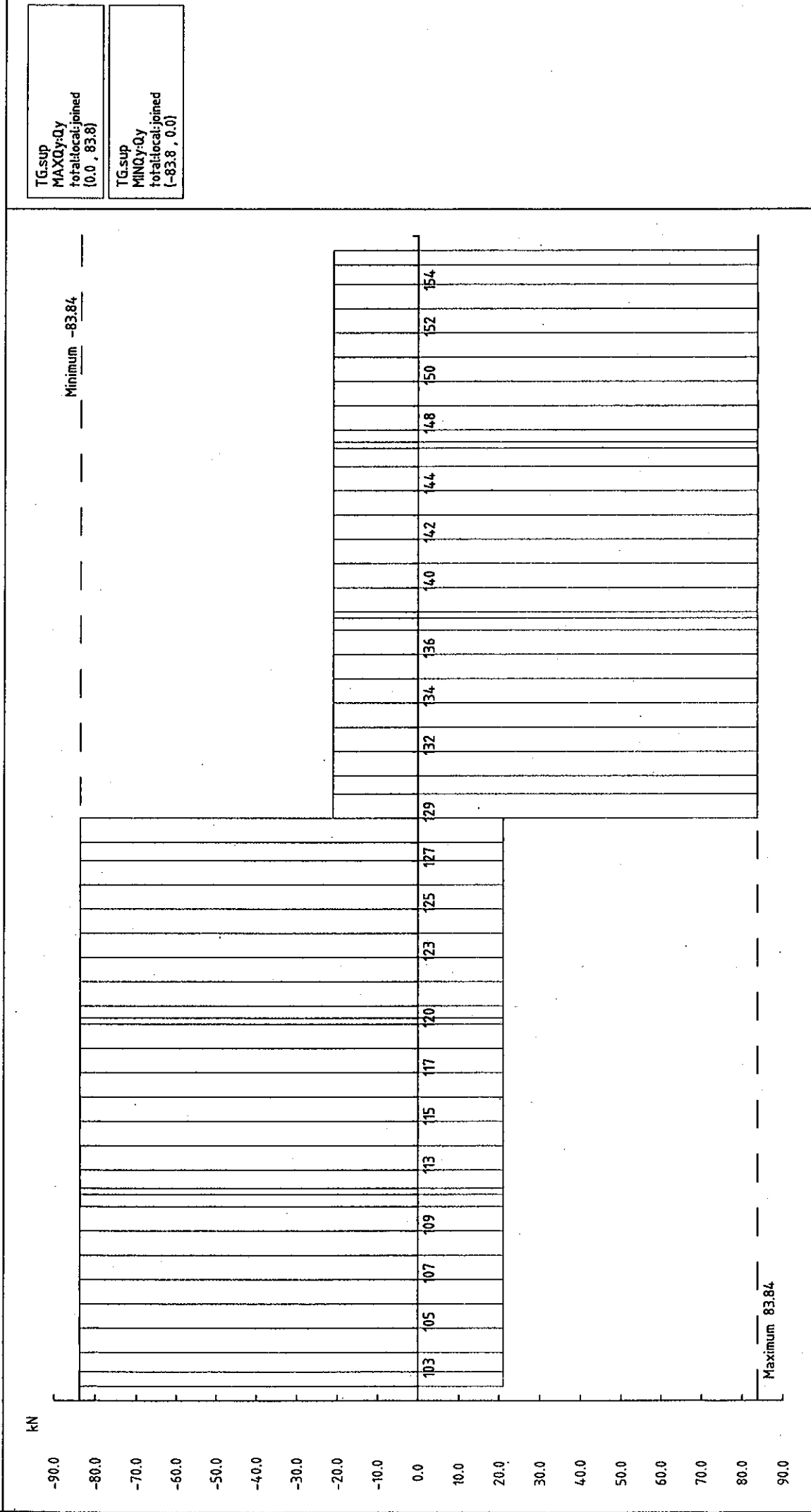
MO MEN DO GRADIEN NHiet DO

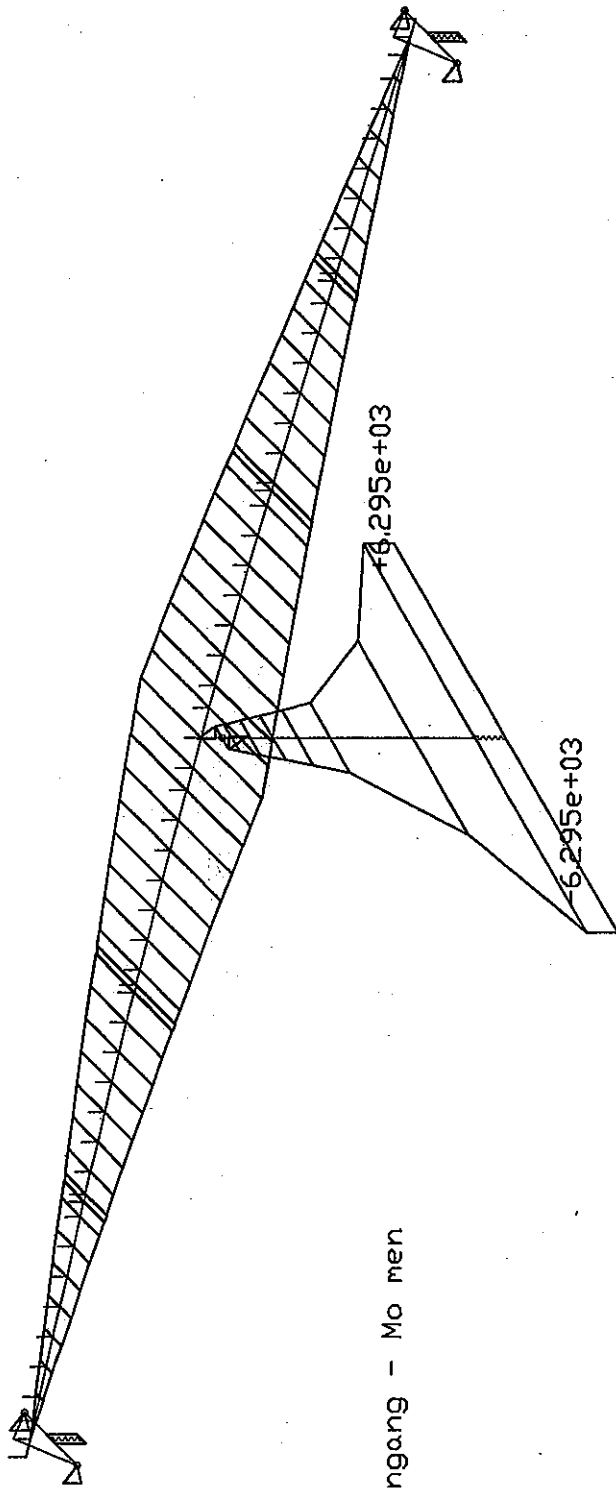
Project: F009
DU AN DUONG CAO TOC DA NANG - QUANG NGAI

RmSet: TG-My

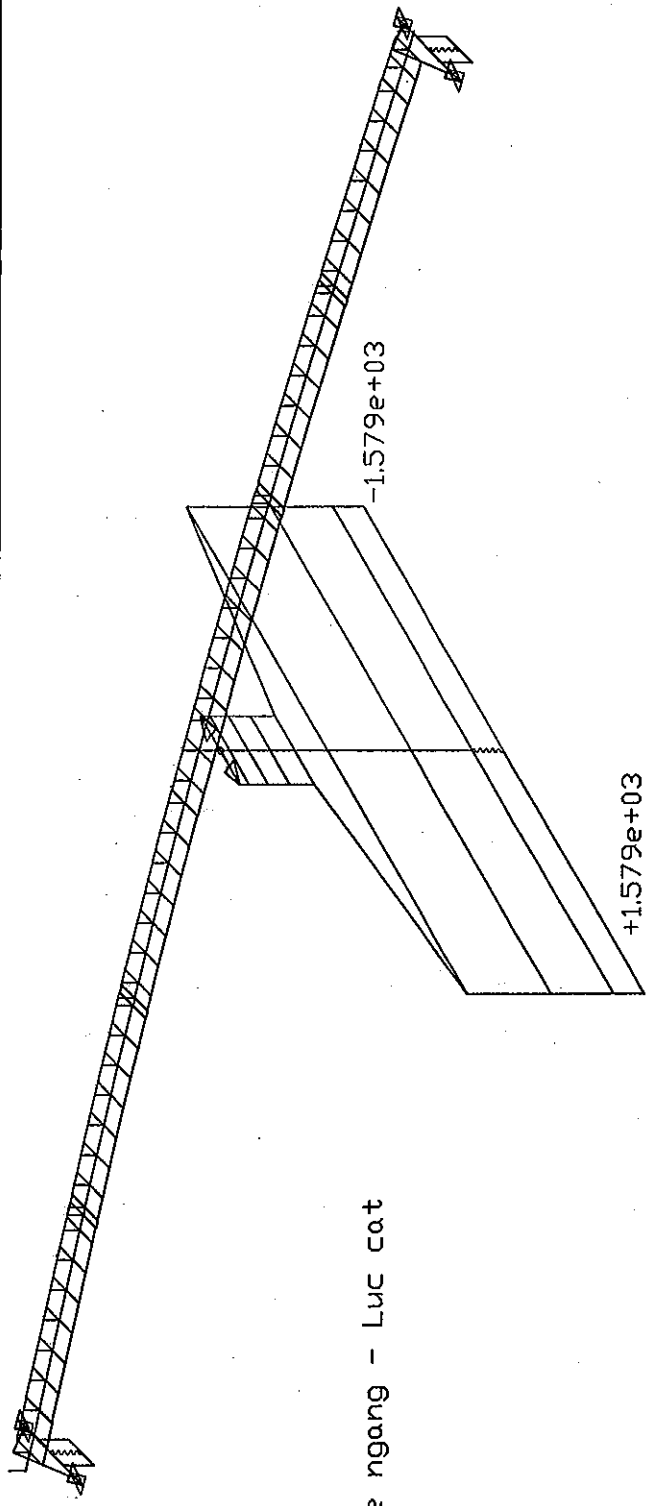
1 cm Plot = 194.4 kNm

0	196.6	398.9	503.3	777.7	972.2
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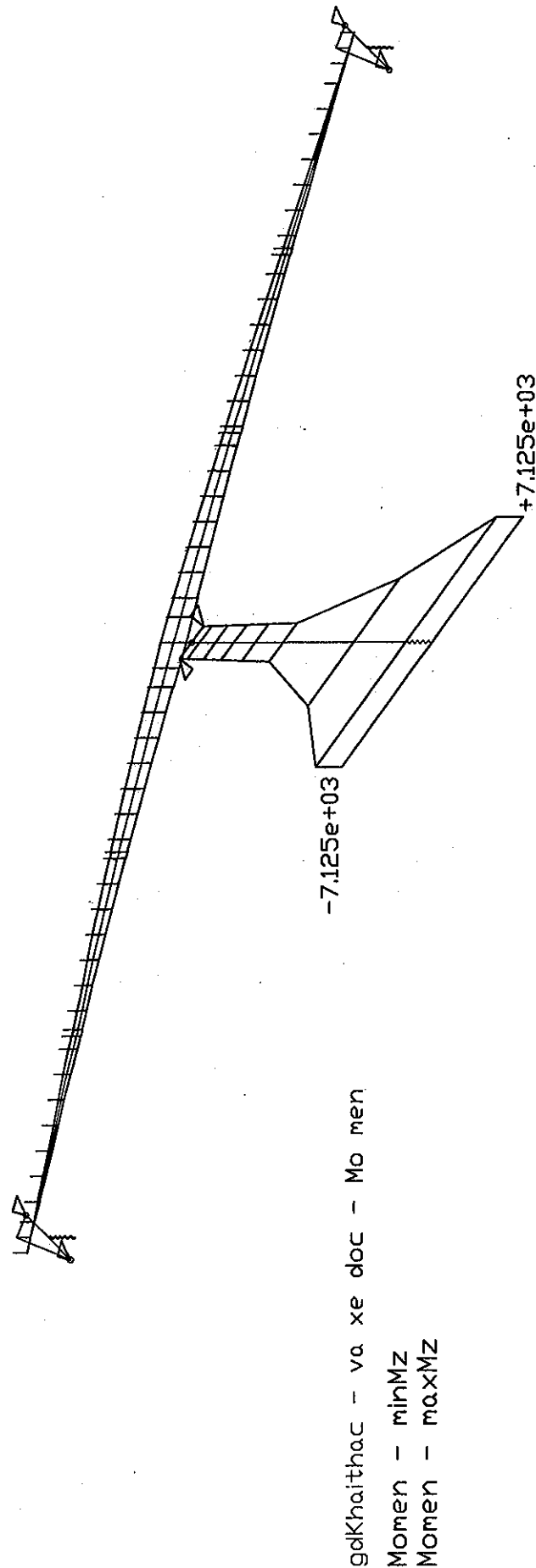


gdKhaithac - va xe ngang - Mo men
 Momen - minMy
 Momen - maxMy

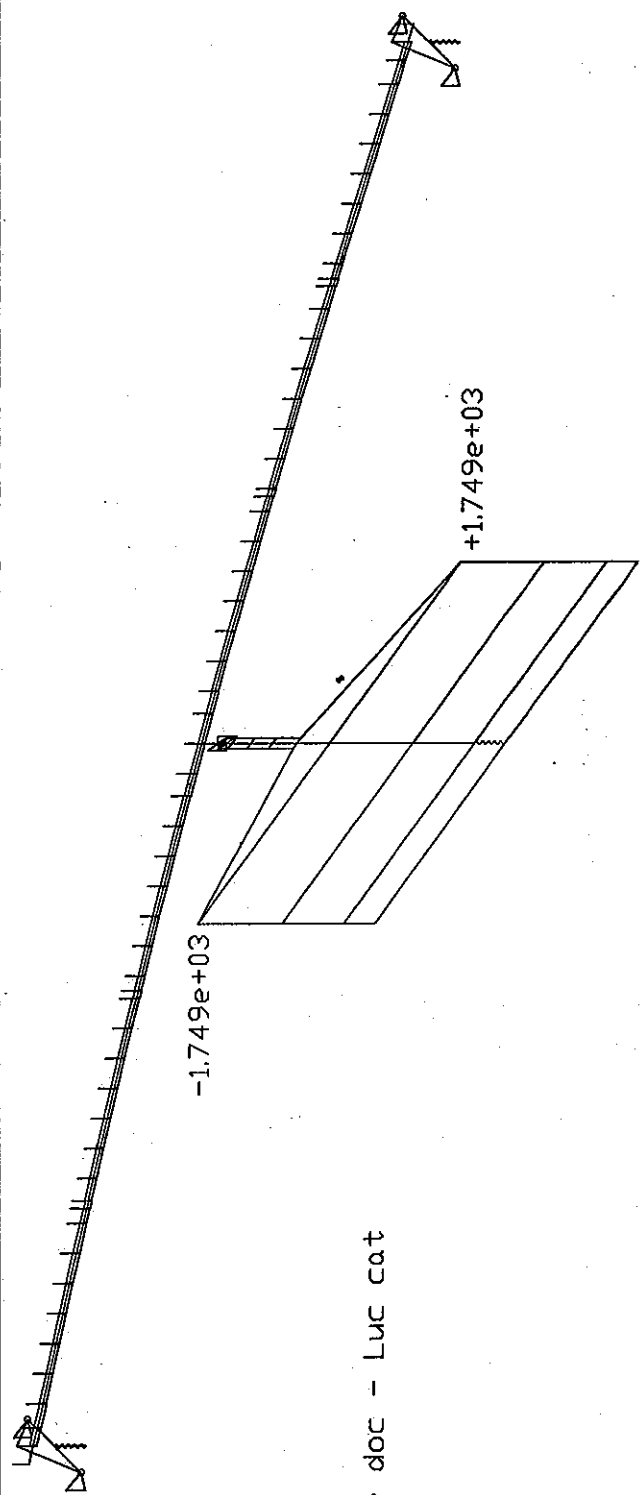


gdKhaithac - va xe ngang - Luc cat
 Luc Cat - minQz
 Luc cat - maxQz

	Project: F009 DU AN DUONG CAO TOC DA NANG - QUANG NGAI	gdKhaithac - va xe ngang	31/01/2013 1656
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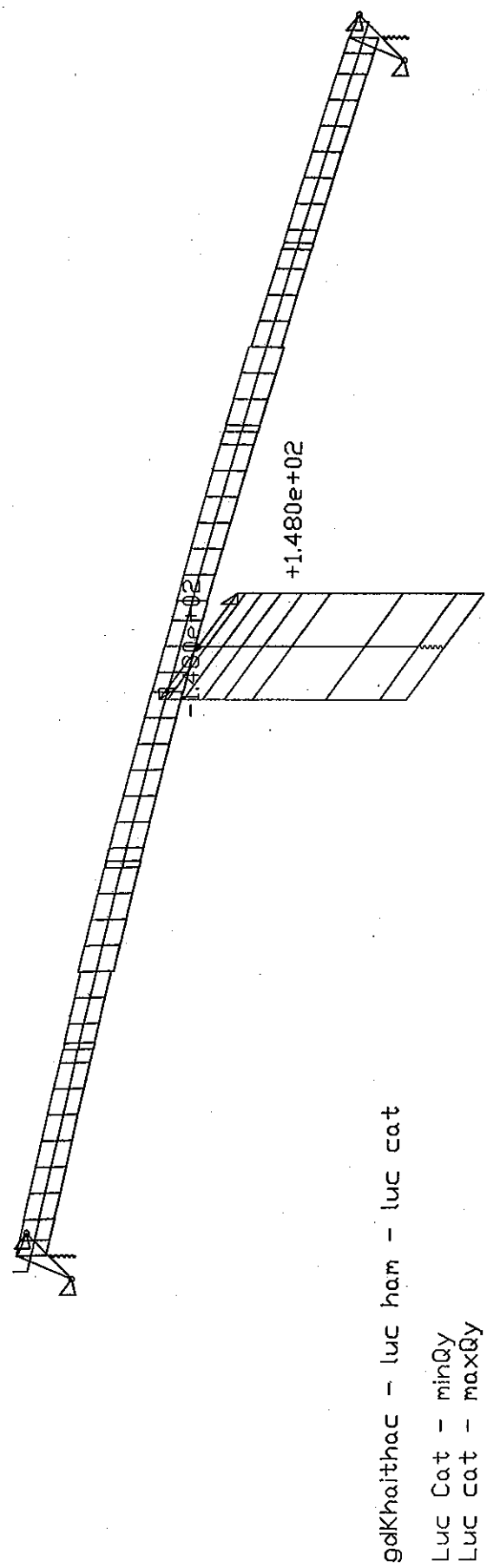
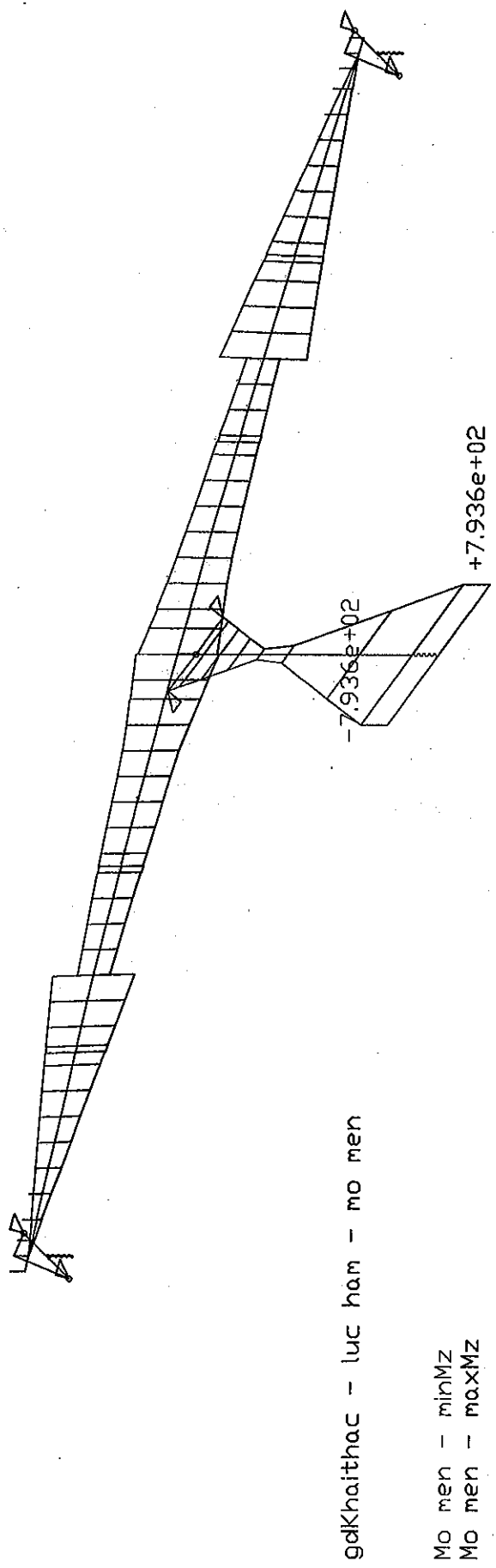


gdKhaithac - va xe doc - Mo men
 Momen - minMz
 Momen - maxMz

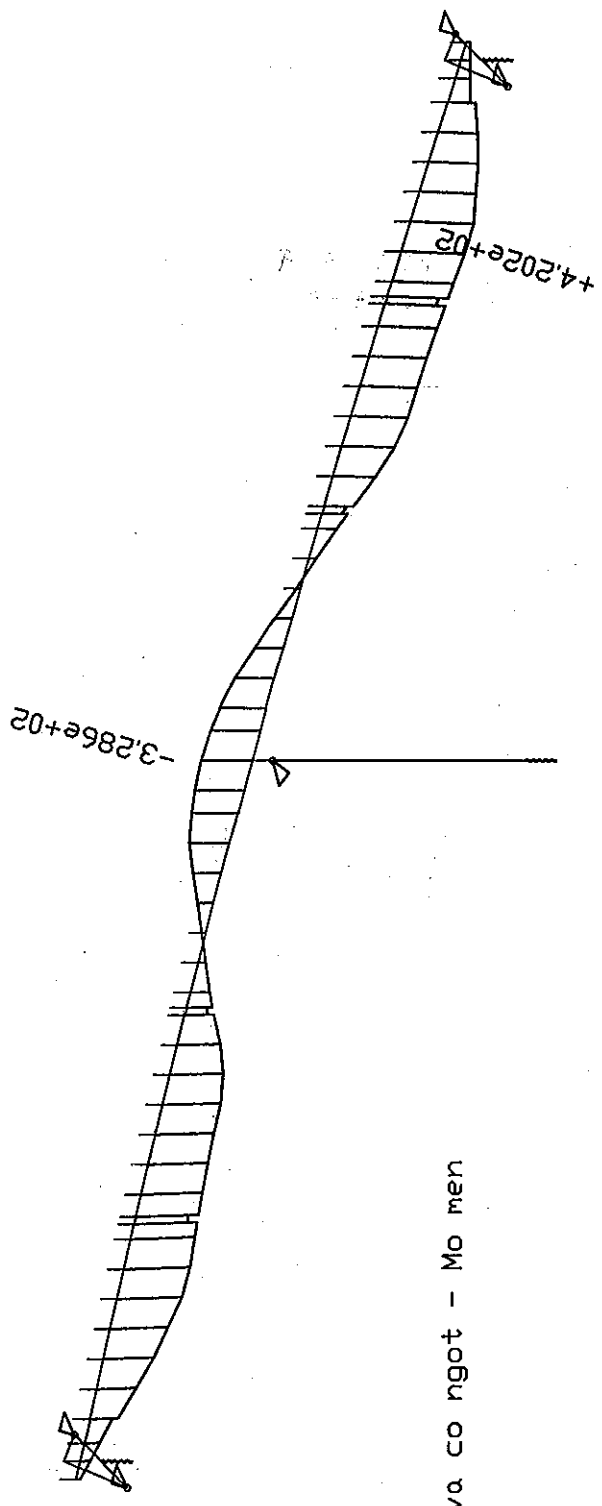


gdKhaithac - va xe doc - Luc cat
 Luc Cat - minQy
 Luc cat - maxQy

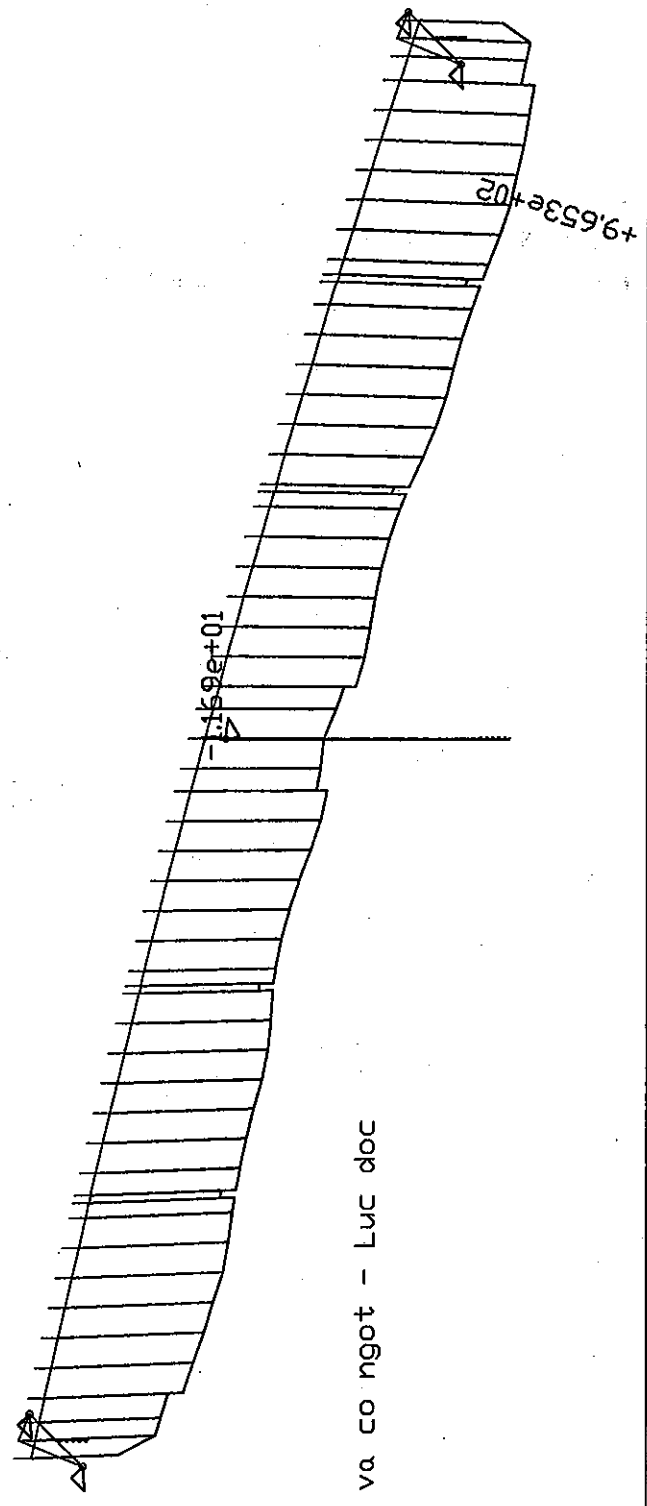
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<div data-bbox="1340 11 1403 143" data-label="Page-Header"> <p>31/01/2013 16:56</p> </div> <div data-bbox="1372 663 1419 1028" data-label="Text"> <p>gdKhaithac - luc ham</p> </div>	<div data-bbox="1403 1205 1481 1979" data-label="Text"> <p>Project: F009 DU AN DUONG CAO TOC DA NANG - QUANG NGAI</p> </div>
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gdKhaithac - tu bien va co ngot - Mo men



gdKhaithac - tu bien va co ngot - Luc doc

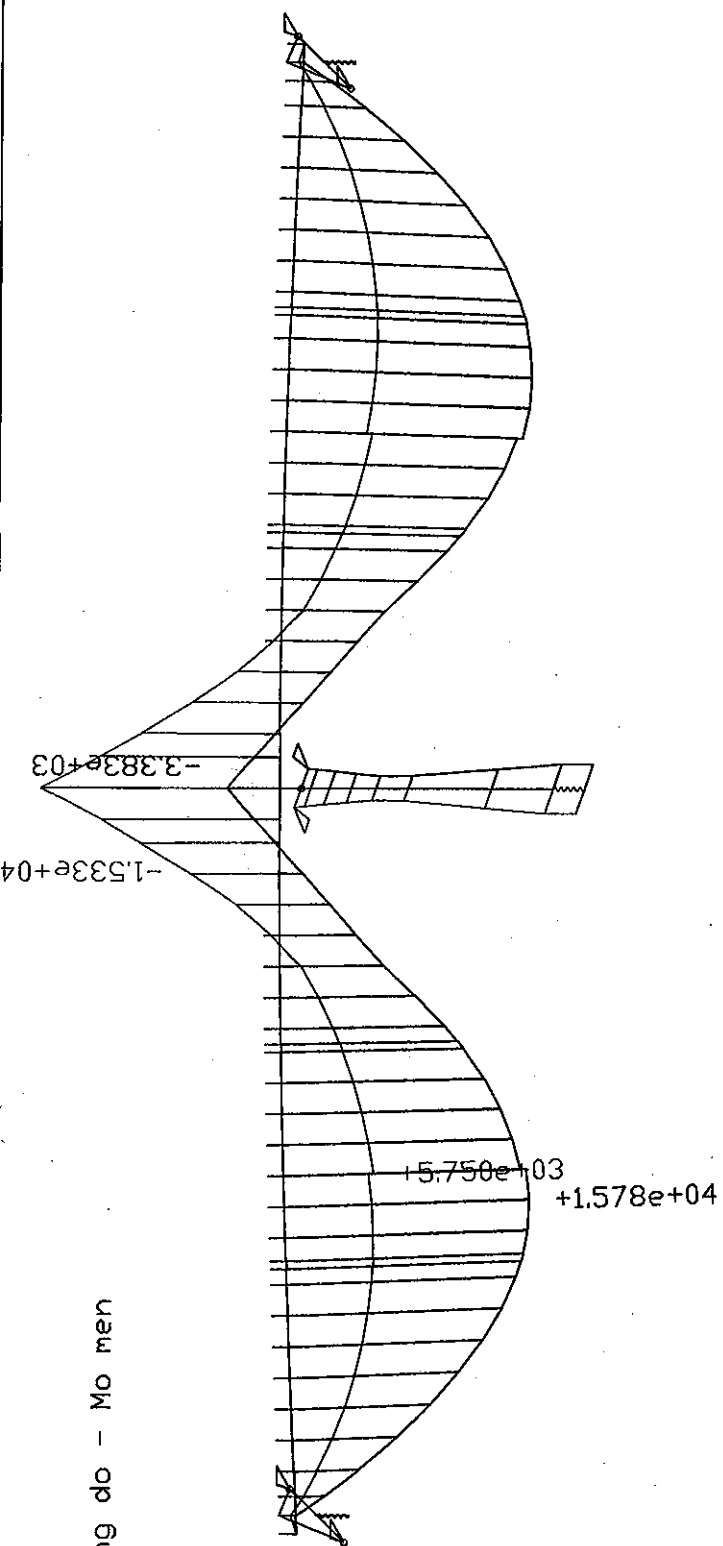
gdKhaithac - tu bien va co ngot

Project: F009
DU AN DUONG CAO TOC DA NANG - QUANG NGAI

gdKhaithac - Cuong do - Mo men

Momen - minMy

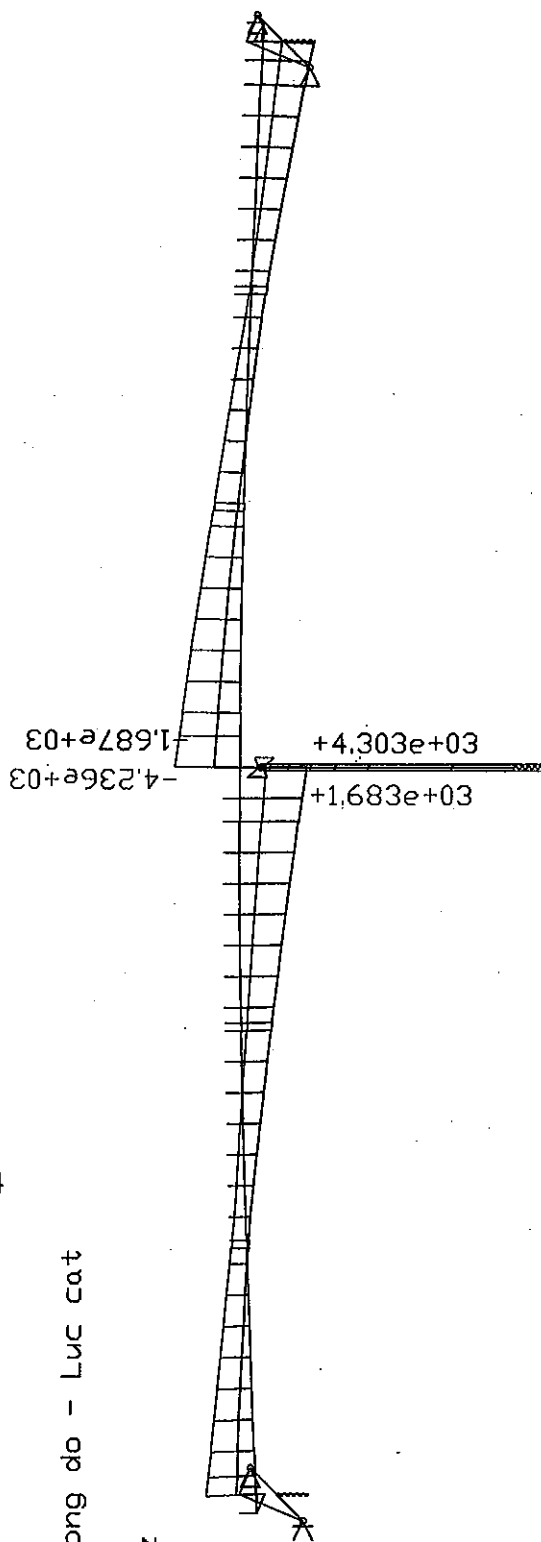
Momen - maxMy



gdKhaithac - cuong do - Luc cat

Luc Cat - minQz

Luc cat - maxQz



-

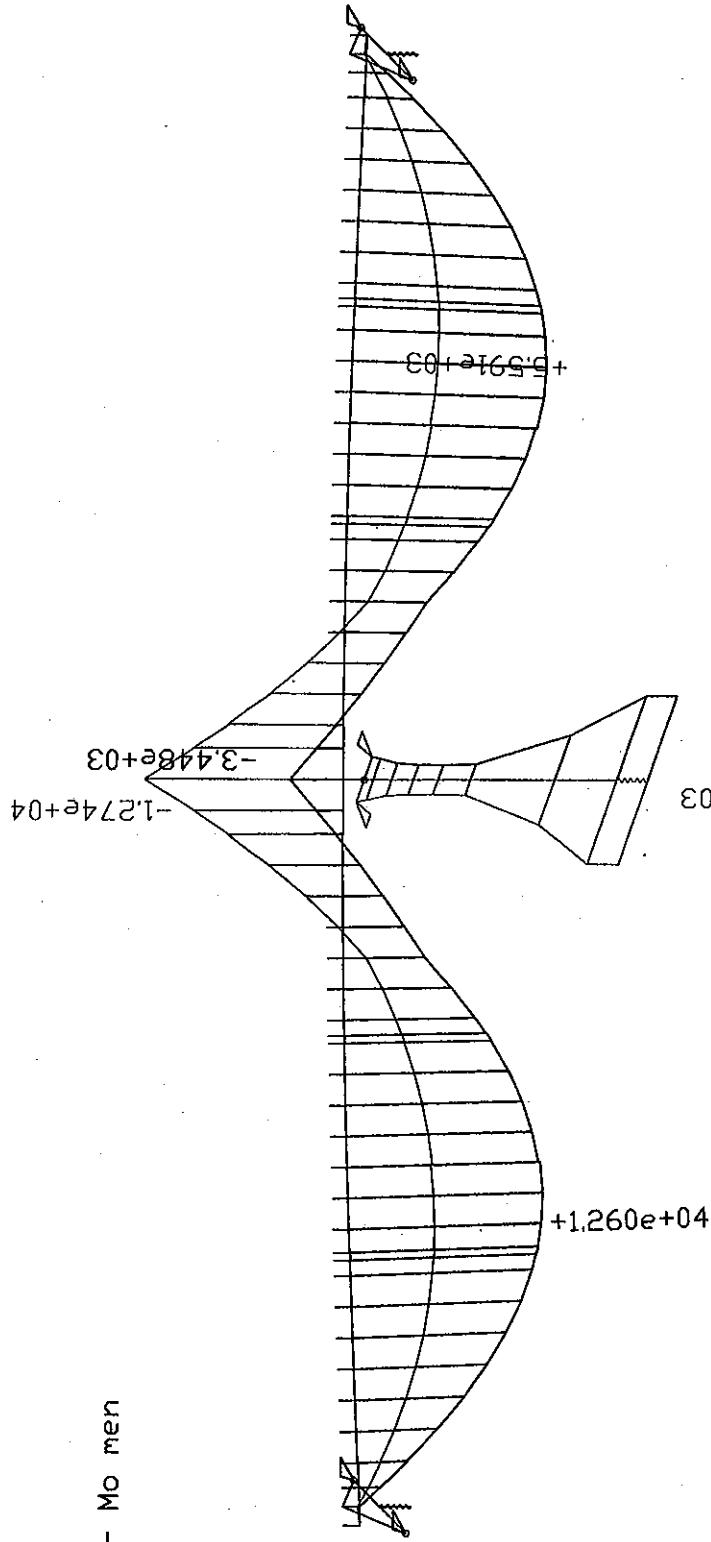
Project: F009
DU AN DUONG CAO TOC DA NANG - QUANG NGAI

gdKhaithac - To hop cuong do - Strength

gdKhaithac - dac biet - Mo men

Momen - minMz

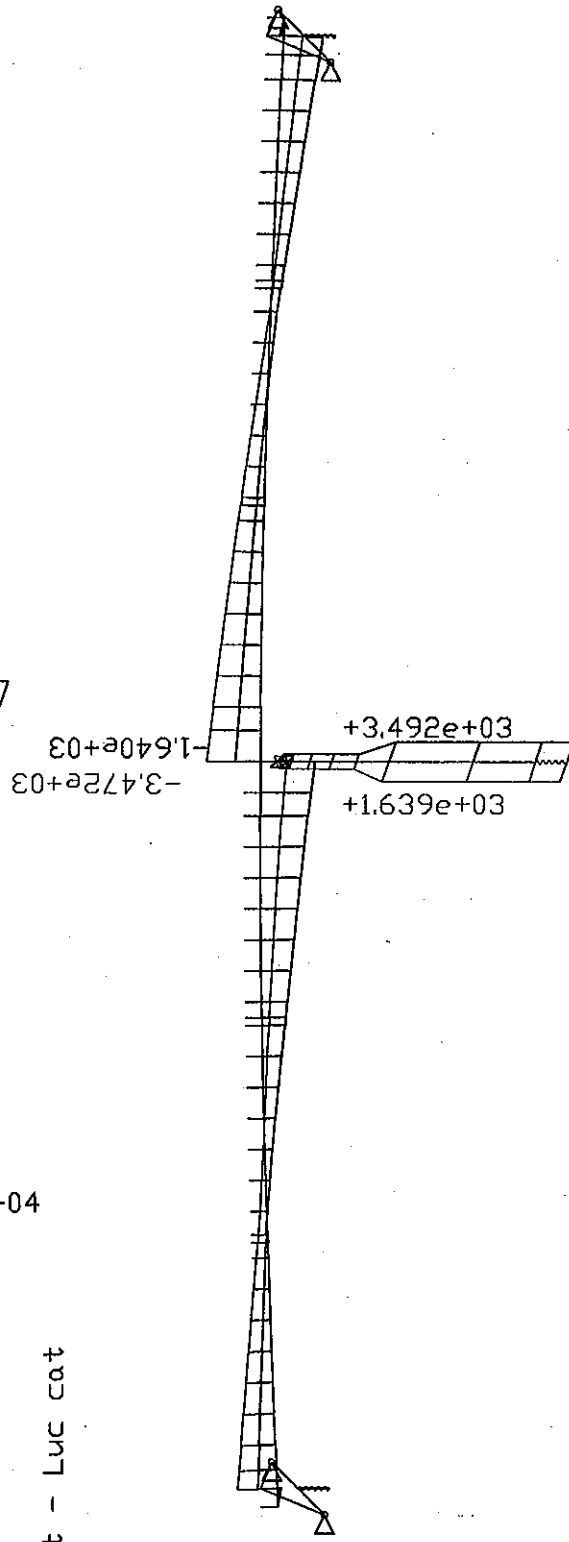
Momen - maxMz



gdKhaithac - dac biet - Luc cat

Luc Cat - minQy

Luc cat - maxQy



Project: F009

DU AN DUONG CAO TOC DA NANG - QUANG NGAI

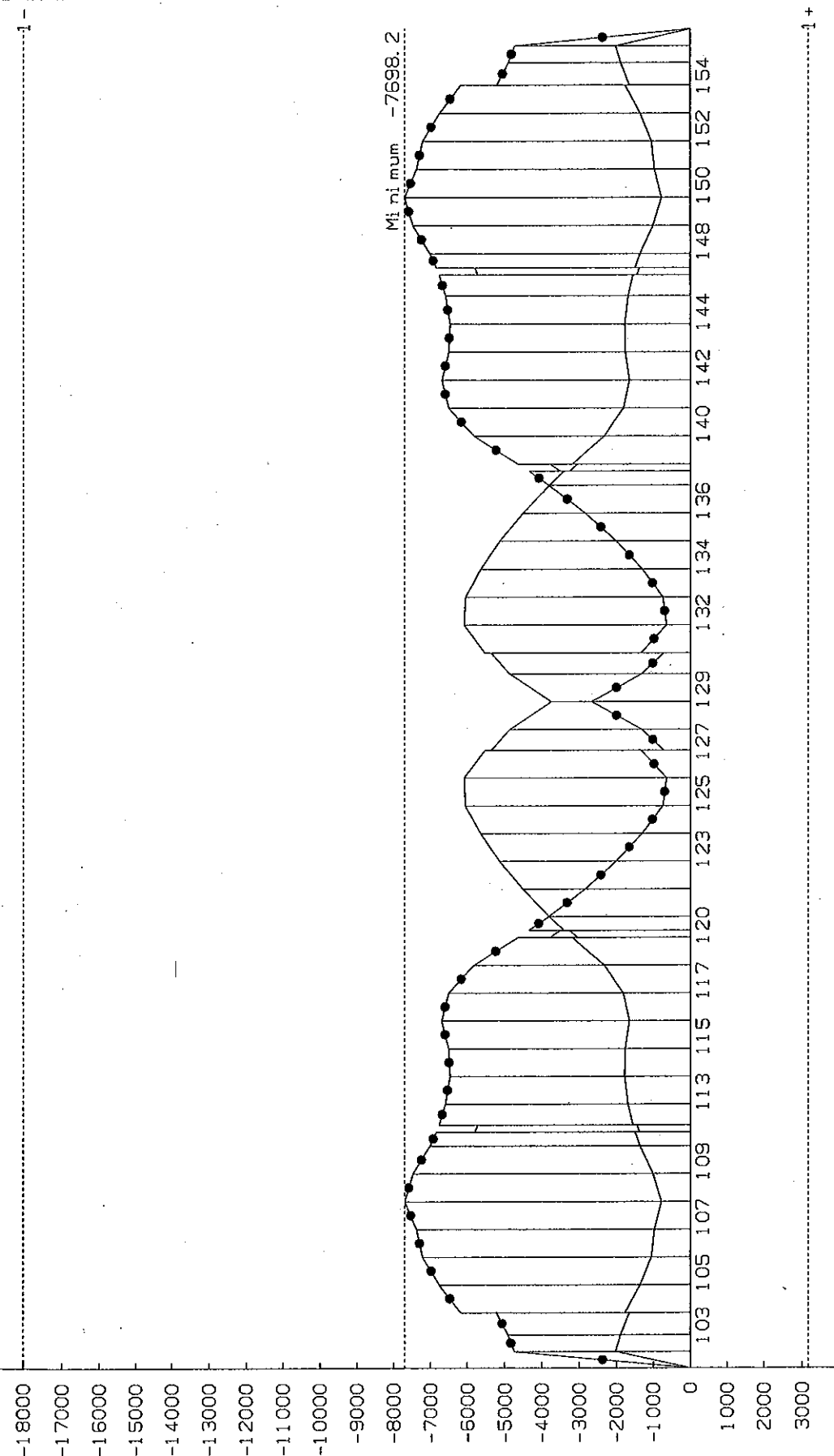
gdKhaithac - To hop dac biet - Extreme

31/01/2013
16:56

CALCULATION SHEET

***FIBRE STRESS CHECK OF VOIDED
SLAB GIRDER***

kN/m2



STG-1. SUP
MAXMz
STRESS÷MG: FIT
(-6087 , 0)

STG-1. SUP
MAXMz
STRESS÷MG: FIB
(-7698 , 0)



Project: F009
DU AN DUONG CAO TOC DA NANG - QUANG NGAI

KIEM TRA UNG SUAT GIAI DOAN THI CONG

RmSet: STG-1

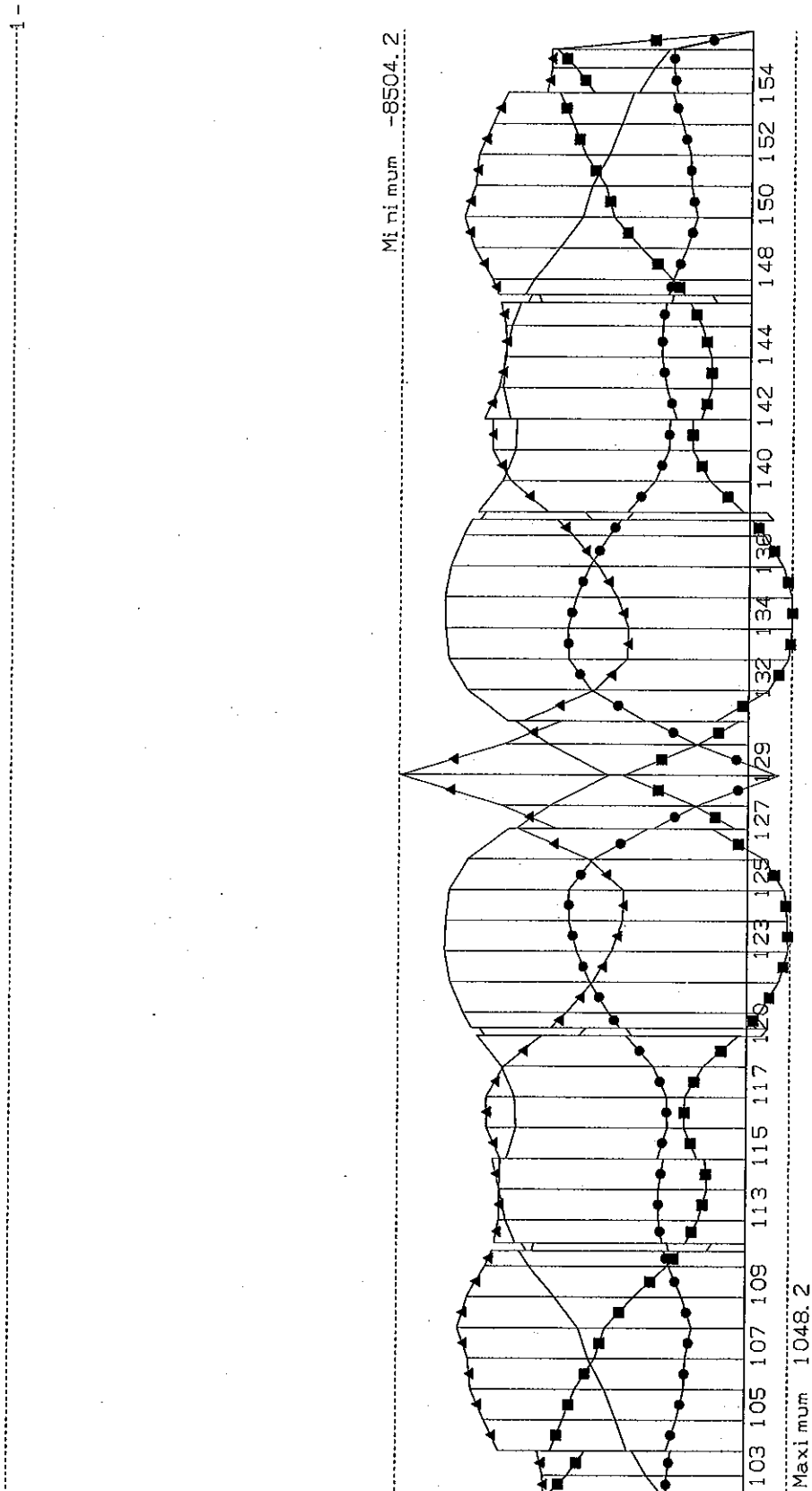
1 cm Plot = 1676.0 kN/m2



04/02/2013
14:08

kN/m²

-18000
-17000
-16000
-15000
-14000
-13000
-12000
-11000
-10000
-9000
-8000
-7000
-6000
-5000
-4000
-3000
-2000
-1000
0
1000
2000
3000



Service1. sup
MAXMz
STRESS=MG: FIT
(-7409 , 0)

Service1. sup
MINMz
STRESS=MG: FIT
(-4418 , 768)

Service1. sup
MAXMz
STRESS=MG: FIB
(-4758 , 1048)

Service1. sup
MINMz
STRESS=MG: FIB
(-8504 , 0)



RM2004

Project: F009
DU AN DUONG CAO TOC DA NANG - QUANG NGAI

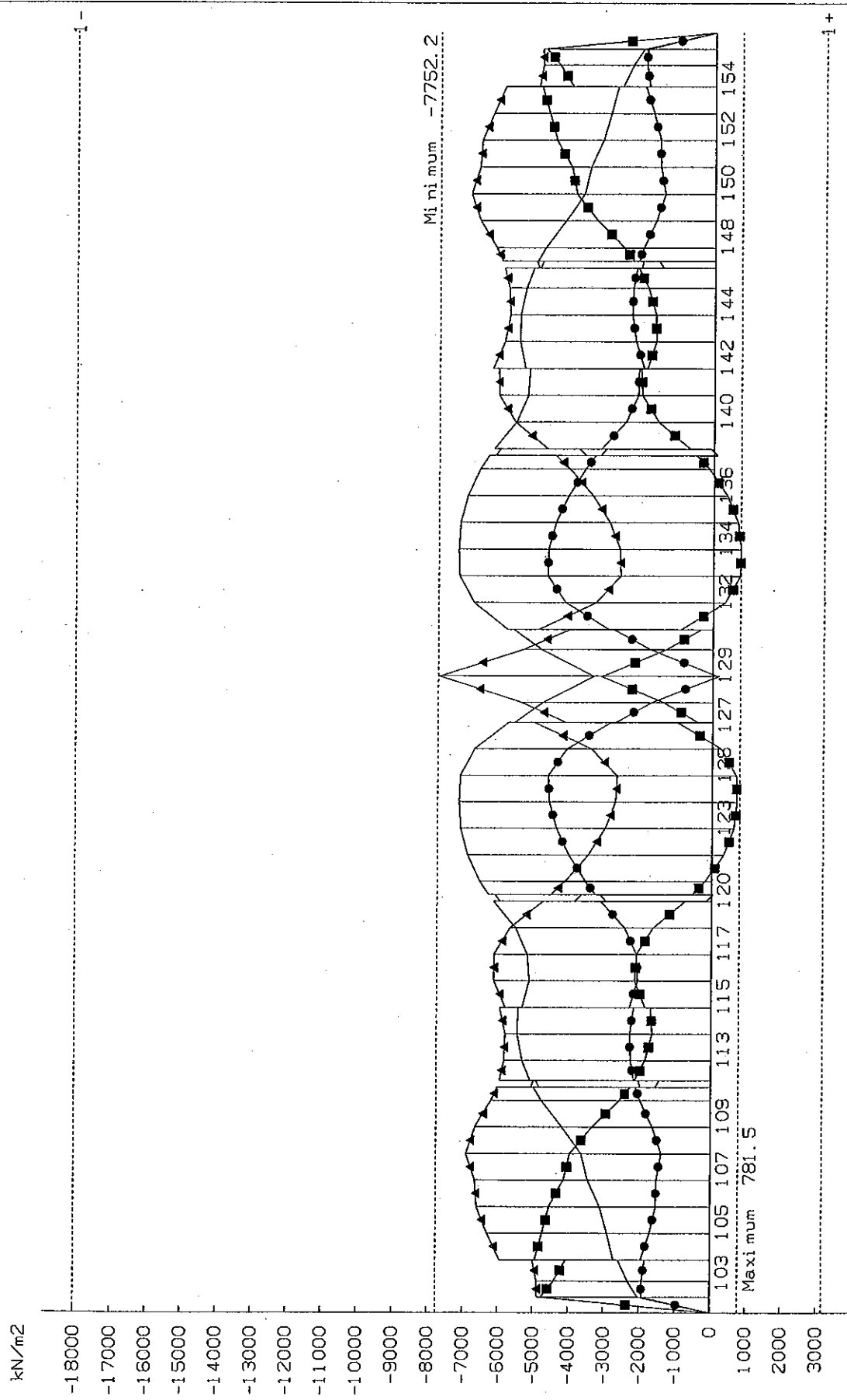
KIEM TRA UNG SUAT TO HOP SERVICE-1

RmSet: SERVICE-1

1 cm Plot = 1676.0 kN/m²

0 1676.0 3352.1 5028.1 6704.2 8380.2

04/02/2013
14:09



04/02/2013
14:09

KIEM TRA UNG SUAT TO HOP SERVICE-3

RmSet: SERVICE-3

Project: F009
DU AN DUONG CAO TOC DA NANG - QUANG NGAI



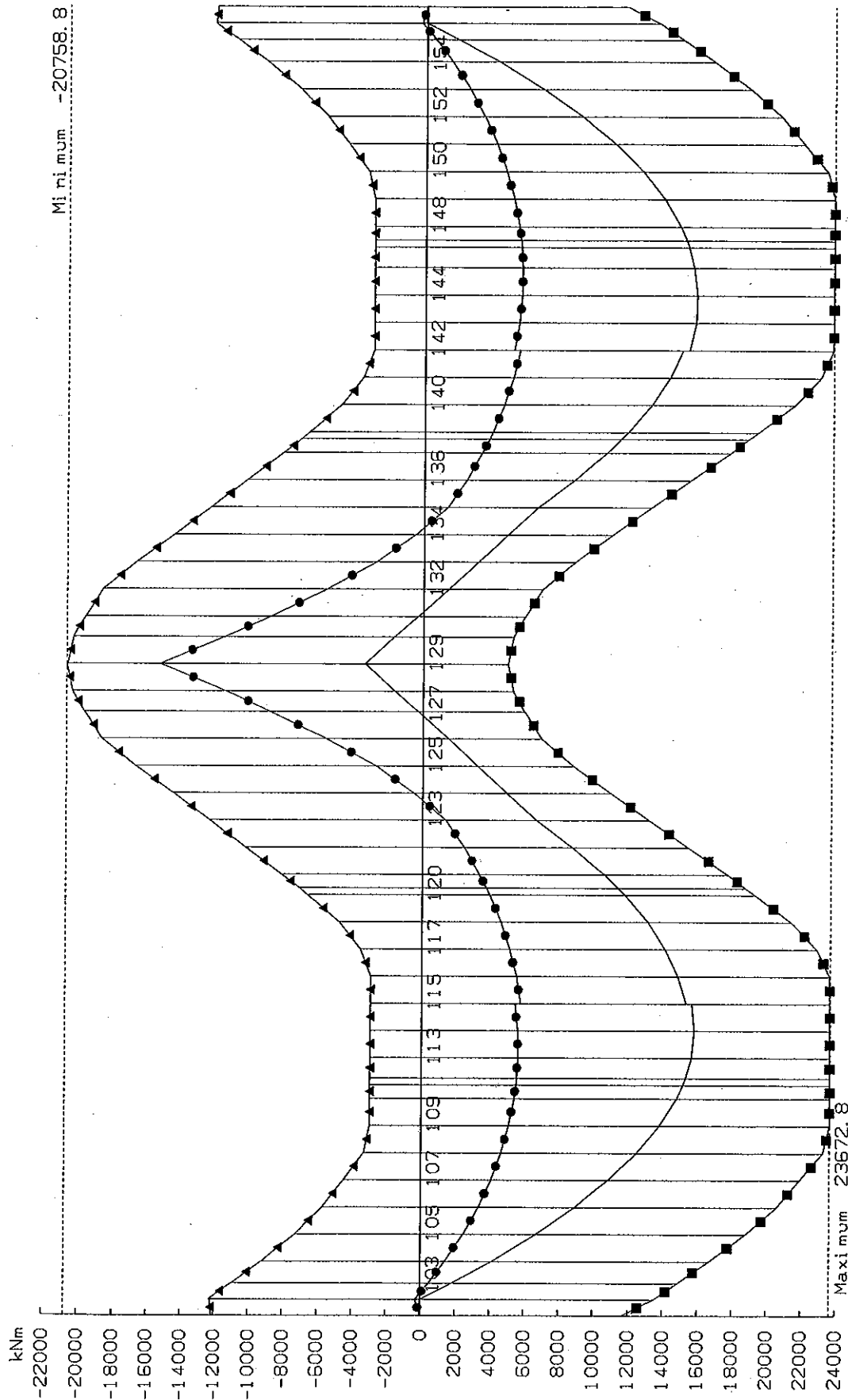
RN2004

1 cm Plot = 1676.0 kN/m²



CALCULATION SHEET

***FLEXURE, SHEAR AND TORSION OF
VOIDED SLAB GIRDER***



BIEU DO BAO VAT LIEU - TO HOP CUONG DO

RmSet: STRENG-ALL

Project: F009
DU AN DUONG CAO TOC DA NANG - QUANG NGAI

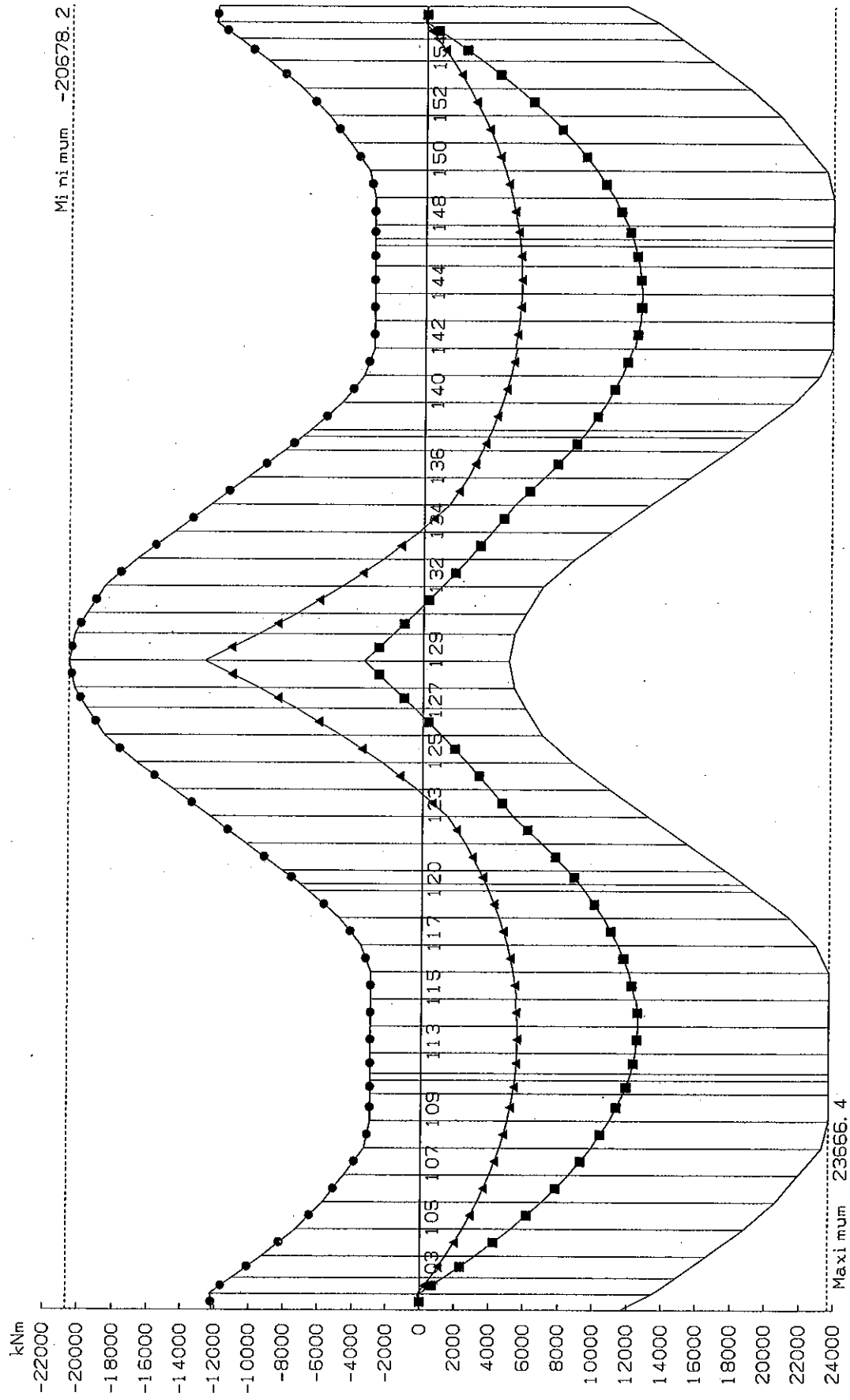


RM2004

1 cm Plot = 3519.3 kNm

0 3519.3 7038.7 10558.0 14077.4 17596.7

04/02/2013
14:10



Ult-extreme. sup MAXMz: Mz secondary: local: joined (0 , 23666)	Ult-extreme. sup MINMz: Mz secondary: local: joined (-20678 , 0)	Extreme. sup MAXMz: Mz secondary: local: joined (-3448 , 12605)	Extreme. sup MINMz: Mz secondary: local: joined (-12741 , 5591)
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22TCN272-05: AASHTO LRFD 2nd - 1998

REINFORCEMENT CHECKING

[illegible]

Sign	Parameters	Unit	Top of pier	Middle of span	

INTERNAL FORCES AT SECTION

	Combination		Strength	Extreme	Strength	Extreme
Qu	Shear	kN	3975	3162	118	14
Mu	Flexural Moment	kNm	15239	12741	15781	12605
Nu	Axial load	kN	287	83	4	1
Tu	Torsional Moment	kNm				

FLEXURAL MOMENT CHECKING

H	Section height		m	1.200	1.200	1.200	1.200
d's	Dis. From comp. fiber to centroid of comp. Reinf		m	0.057	0.057	0.057	0.057
d1x	Dis. From tens. fiber to centroid of tension Reinf		m	0.057	0.057	0.057	0.057
	Cover to reinf		m	0.050	0.050	0.050	0.050
ds	Dis. From comp. fiber to centroid of tension Reinf		m	1.143	1.143	1.143	1.143
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	0.288	0.288	0.203	0.203
dps	Dis. From comp. fiber to centroid of tension prestressing steel		m	0.913	0.913	0.998	0.998
b	Width of the compression face of member		m	3.500	3.500	7.240	7.240
bw	Web width or diameter of a circular section		m	3.500	3.500	2.200	2.200
hf	Compression flange depth		m	0.000	0.000	0.250	0.250
Iz	Moment of inertia of section		m4	0.504	0.504	1.043	1.043
Amc	Section area		m2	4.200	4.200	8.688	8.688
	Steel choice						
Aps	Tension prestressing steel	P.S type		12 T15.2	12 T15.2	12 T15.2	12 T15.2
		Number	tendons	9	9	9	9
		Area	m2	0.01512	0.01512	0.01512	0.01512
A'ps	Compression prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0
		Number	tendons	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000
As	Tension Reinforcement	Number	bars	63	63	19	19
		Diameter	mm	14	14	14	14
		Area	m2	0.00951	0.00951	0.00287	0.00287
A's	Compression Reinforcement	Number	bars	19	19	63	63
		Diameter	mm	14	14	14	14
		Area	m2	0.00287	0.00287	0.00951	0.00951
A'c	Shear reinforcement	Number	bars	10	10	6	6
		Diameter	mm	20	20	16	16
		Area	m2	0.00314	0.00314	0.00121	0.00121
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	2.00
β1	Stress block factor			0.764	0.764	0.764	0.764
c	Dis. Between centroid and top fiber		m	0.309	0.309	0.130	0.130
	For T section behavior		m	0.309	0.309	-0.112	-0.112
	For rectangular section behavior		m	0.309	0.309	0.130	0.130
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	1681	1681	1791	1791
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	0.236	0.236	0.099	0.099
de	Corresp. effective depth from extreme comp. fiber to centroid of tensile force in the tensile reinf.	m	0.943	0.943	1.003	1.003
Mn	Nominal resistance	kNm	24166	24166	24430	24430
Mr	Factored resistance	kNm	21750	24166	21987	24430
Mu	Flexural moment	kNm	15239	12741	15781	12605
(5.7.3.2)	Flexural moment Checking		OK	OK	OK	OK
	Limits for reinforcement					
r min	Minimum reinforcement		0.23%	0.23%	0.03%	0.03%
	Minimum reinforcement Checking for RC	0.30%	N.a	N.a	N.a	N.a
1.2*Mer	Cracking moment	kNm	2704	2704	4658	4658
(5.7.3.3.2)	Checking $M_r \geq \min(1.2M_{er}, 1.33M_u)$		OK	OK	OK	OK
(5.7.3.4)	Control of cracking by distr. of reinf for RC member- Check?		No	No	No	No
	Existing condition for structure	1,2 or 3	1	1	1	1
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.057	0.057	0.057	0.057
Z	Crack width parameter	N/mm	30000	30000	30000	30000
A	Area of concr. with same centroid as tens. Reinf	m ²	0.006	0.006	0.043	0.043
fsa	Value	Mpa	421	421	222	222
0.6*fy		Mpa	240	240	240	240
	Tensile stress in reinf $\min(f_{sa}, 0.6f_y)$	Mpa	240	240	222	222
x	Dist. From compression fiber to centroid	m	-	-	-	-
J.d	Arm	m	-	-	-	-
Icr	Moment of inertia of the cracked section	m ⁴	-	-	-	-
fs	Tensile stress in reinforcement $f_s = M_{sls} / (A_s * J.d)$	Mpa	-	-	-	-
	Checking for control cracking $f_s < f_{sa}$		N.a	N.a	N.a	N.a
SHEAR AND TORSION CHECKING						
β	Factor indicating diag. cracked concr. to tension		2.5	4.4	4.9	5.5
θ	Angle of inclination of diagonal compressive	degree	29.69	27.00	27.00	27.00
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	3.500	3.500	2.200	2.200
dv	Effective shear depth	m	0.864	0.864	0.954	0.954
	$(d_c - a/2)$	m	0.825	0.825	0.954	0.954
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600
ncat	Amount of bars in spacing S	Bar	10	10	6	6
Av	Shear reinf area in spacing S	m ²	0.0031	0.0031	0.0012	0.0012
θ	Assume	degree	39.58	28.61	27.00	27.00
v	Shear stress in concrete	kN/m ²	1461	1046	62	7
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1141	1141	1128	1128
ϵ_x	Strain in tensile reinforcement		5.43E-04	7.28E-05	-1.11E-04	-1.08E-03
	if $\epsilon_x < 0$, multiple with reduce factor		-	-	-8.65E-06	-8.36E-05
	Strain checking	$\leq 2.00E-3$	Ok	Ok	Ok	Ok
v/fc	Ratio of shear stress and fc		0.037	0.026	0.002	0.000
β	Final value		2.5	4.4	4.9	5.5
θ	Final value	degree	29.69	27.00	27.00	27.00
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	3946	6924	5447	6066
Vs	Shear resistance provided by shear reinforcement	kN	3173	3550	1513	1513
Vp	Component in the direction of the applied shear of the effective P.S	kN	25250	25250	25250	25250
Vn1	$V_{n1} = V_c + V_s + V_p$	kN	32369	35724	32210	32829
Vn2	Vn2	kN	55490	55490	46234	46234
Vn	Nominal shear resistance $V_n = \min(V_{n1}, V_{n2})$	kN	32369	35724	32210	32829
Vr	Factored shear resistance	kN	29132	35724	28989	32829
Vu	Shear	kN	3975	3162	118	14
(5.8.2.7)	Shear checking		OK	OK	OK	OK

CALCULATION SHEET

DECK SLAB OF VOIDED SLAB GIRDER

Design tandem : Uniform loads	q1	= 0.0	kN/m2
	q2	= 762.4	kN/m2
Design truck : Uniform loads	q1	= 0.0	kN/m2
	q2	= 455.7	kN/m2
Using loads	q1	= 0.0	kN/m2
	q2	= 762.4	kN/m2
*)Lane loads:	q	= 3.7	kN/m2
*)Handrail loads:	Plc	= 11.45	kN/m
*)Median strip loads:	P	= 0.00	kN/m
*)Pavement loads:	q	= 1.67	kN/m2
*)Self weight of cantilever:	Area	A	= 0.35 m2
	Force	Pw	= 8.55 kN/m

***)Vehicle collision on Handrail:**

Test level	L2		
Tranverse vehicle impact force	Ft	= 120.00	kN
Vertical force of vehicle	Fv	= 20.00	kN
Longitudinal length of distribution of Ft	Lt	= 1.22	m
Longitudinal length of distribution of Fv	Lv	= 5.50	m
Height of wall	h	= 0.51	m

Item	Load combinations		
	Service	Strength	Extreme
Self weight	1.00	1.25	1.25
Pavement	1.00	1.50	1.50
Handrail	1.00	1.50	1.50
Lane load	1.00	1.75	0.50
Vehicle load	1.00	1.75	0.50
Hozirontal vehicle collision			1.00
Vertical vehicle collision			1.00

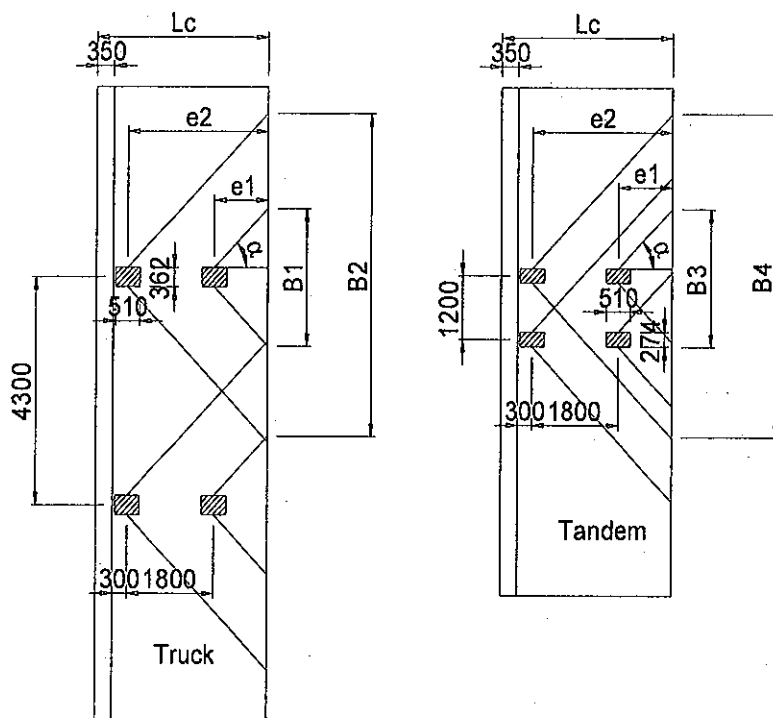
Item	Load combinations		
	N	Q	M
	kN	kN	kNm
Self weight	-	8.5	4.9
Pavement	-	1.9	1.1
Handrail	-	11.5	13.2
Lane load	-	4.3	2.5
Vehicle load	-	110.0	97.2
Hozirontal vehicle collision	3.2	-	1.7
Vertical vehicle collision	-	63.6	73.5

Result of internal forces of section A-A

Load combination	N	Q	M
	(kN/m)	(kN/m)	(kNm/m)
Service	0.0	136.2	119.0
Strength	0.0	230.8	202.1
Extreme	3.2	151.5	152.6

2. Section B-B

*)Distribution of wheel loads



Lcantilever	= 1.553	m
e1	= 0.00	m
e2	= 0.90	m
B1	= 0.36	m
B2	= 1.40	m
B3	= 0.27	m
B4	= 1.32	m
α	= 30.00	deg

Design tandem : Uniform loads

q1 = 0.0 kN/m²

q2 = 277.5 kN/m²

Design truck : Uniform loads

q1 = 0.0 kN/m²

q2 = 171.5 kN/m²

Using loads

q1 = 0.0 kN/m²

q2 = 277.5 kN/m²

*)Lane loads:

q = 3.7 kN/m²

*)Handrail loads:

Plc = 11.45 kN/m

*)Median strip loads:

P = 0.00 kN/m

*)Pavement loads:

q = 1.89 kN/m²

*)Self weight of cantilever:

Area A = 0.67 m²

Force Pw = 16.38 kN/m

*)Vehicle collision on Handrail:

Test level L2

Transverse vehicle impact force Ft = 120.00 kN

Vertical force of vehicle Fv = 20.00 kN

Longitudinal length of distribution of Ft Lt = 1.22 m

Longitudinal length of distribution of Fv Lv = 5.50 m

Height of wall h = 0.51 m

Item	Load combinations		
	Service	Strength	Extreme
Self weight	1.00	1.25	1.25
Pavement	1.00	1.50	1.50
Handrail	1.00	1.50	1.50
Lane load	1.00	1.75	0.50
Vehicle load	1.00	1.75	0.50
Hozirontal vehicle collision			1.00
Vertical vehicle collision			1.00

Item	Load combinations		
	N	Q	M
	kN	kN	kNm
Self weight	-	16.4	12.7
Pavement	-	2.2	1.3
Handrail	-	11.5	13.2
Lane load	-	4.3	2.5
Vehicle load	-	110.0	113.2
Hozirontal vehicle collision	3.1	-	1.6
Vertical vehicle collision	-	56.7	88.0

Result of internal forces of section B-B

Load combination	N	Q	M
	(kN/m)	(kN/m)	(kNm/m)
Service	0.0	144.3	142.8
Strength	0.0	240.9	240.0
Extreme	3.1	154.8	185.1

	DA NANG - QUANG NGAI EXPRESSWAY PROJECT			Item.	Eng.	Date.	Sign.
	FO09 BRIDGE			Design			
	DETAIL DESIGN			Check			
	CHECK SECTION A-A			Revise			

22TCN272-05; AASHTO LRFD 2nd - 1998

REINFORCEMENT CHECKING - DECK SLAB

MATERIALS				
NORMAL CONCRETE				
fc	Compressive Strength of concrete at 28 days	Mpa	40	
Ec	Modulus of Elasticity	Mpa	31975	
fr	Modulus of Rupture	Mpa	4.0	
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	197000	
REINFORCEMENT				
fy	Yield strength	Mpa	400	
Es	Modulus of Elasticity	Mpa	200000	
nc	Ratio Es/Ec		6	

Sign	Parameters	Unit	Section A-A					
INTERNAL FORCES AT SECTION								
	Combination		Service	Strength	Extreme			
Qu	Shear	kN	136	231	151			
Mu	Flexural Moment	kNm	119	202	153			
Nu	Axial load	kN	0	0	0			
Tu	Torsional Moment	kNm						
FLEXURAL MOMENT CHECKING								
H	Section height	m	0.354	0.354	0.354			
d's	Dis. From comp. fiber to centroid of comp. Reinf	m	0.058	0.058	0.058			
d1x	Dis. From tens. fiber to centroid of tension Reinf	m	0.060	0.060	0.060			
	Cover to reinf	m	0.050	0.050	0.050			
ds	Dis. From comp. fiber to centroid of tension Reinf	m	0.294	0.294	0.294			
d'ps	Dis. From comp. fiber to centroid of comp. prestressing steel	m	0.150	0.150	0.150			
d1xp	Dis. From tens. fiber to centroid of tension prestressing steel	m	0.100	0.100	0.100			
dps	Dis. From comp. fiber to centroid of tension prestressing steel	m	0.254	0.254	0.254			
b	Width of the compression face of member	m	1.000	1.000	1.000			
bw	Web width or diameter of a circular section	m	1.000	1.000	1.000			
hf	Compression flange depth	m	0.000	0.000	0.000			
Iz	Moment of inertia of section	m4	0.004	0.004	0.004			
Amc	Section area	m2	0.354	0.354	0.354			
	Steel choice							
Aps	Tension prestressing steel	P.S type	3 T15.2	3 T15.2	3 T15.2			
		Number	tendons	1	1			
		Area	m2	0.00042	0.00042	0.00042		
A'ps	Compression prestressing steel	P.S type	0 T00.0	0 T00.0	0 T00.0			
		Number	tendons	0	0			
		Area	m2	0.00000	0.00000	0.00000		
As	Tension Reinforcement	Number	bars	6	6			
		Diameter	mm	20	20			
		Area	m2	0.00188	0.00188	0.00188		
A's	Compression Reinforcement	Number	bars	6	6			
		Diameter	mm	16	16			
		Area	m2	0.00121	0.00121	0.00121		
A'c	Shear reinforcement	Number	bars	2	2			
		Diameter	mm	14	14			
		Area	m2	0.00030	0.00030	0.00030		
φ	Resistance factors for flexure	5.5.4.2	1.00	0.90	1.00			
φv	Resistance factors for shear		1.00	0.90	1.00			
φn	Resistance factors for axial force		1.00	1.00	1.00			
β1	Stress block factor		0.764	0.764	0.764			
c	Dis. Between centroid and top fiber	m	0.039	0.039	0.039			
	For T section behavior	m	0.039	0.039	0.039			
	For rectangular section behavior	m	0.039	0.039	0.039			
fpe	Effective stress in the prestressing steel after losses	Mpa	1116	1116	1116			
fps	Aver. stress in pres. steel at the time for which the nominal resistance	Mpa	1779	1779	1779			
k	Factor depends on type of P.S, Low relaxation strand k = 0.28		0.28	0.28	0.28			

a	Depth of equivalent stress block	m	0.030	0.030	0.030		
de	Corresp. effective depth from extreme comp. fiber to centroid of tensile force in the tensile reinf.	m	0.274	0.274	0.274		
Mn	Nominal resistance	kNm	368	368	368		
Mr	Factored resistance	kNm	368	331	368		
Mu	Flexural moment	kNm	119	202	153		
(5.7.3.2)	Flexural moment Checking		OK	OK	OK		
	Limits for reinforcement						
r min	Minimum reinforcement		0.53%	0.53%	0.53%		
	Minimum reinforcement Checking for RC	0.30%	N.a	N.a	N.a		
1.2*Mcrr	Cracking moment	kNm	56	56	56		
(5.7.3.3.2)	Checking $M_r \geq \min(1.2M_{cr}, 1.33M_u)$		OK	OK	OK		
(5.7.3.4)	Control of cracking by distr. of reinf for RC member- Check?		Yes	No	No		
	Existing condition for structure	1,2 or 3	1	1	1		
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.060	0.060	0.060		
Z	Crack width parameter	N/mm	30000	30000	30000		
A	Area of concr. with same centroid as tens. Reinf	m ²	0.020	0.020	0.020		
fsa	Value	Mpa	282	282	282		
0.6*fy	Tensile stress in reinf Min(fs,0.6fy)	Mpa	240	240	240		
x	Dist. From compression fiber to centroid	m					
J.d	Arm	m					
Icr	Moment of inertia of the cracked section	m ⁴					
fs	Tensile stress in reinforcement $f_s = M_{sls} / (A_s * J.d)$	Mpa					
	Checking for control cracking $f_s < f_{sa}$		N.a	N.a	N.a		
SHEAR AND TORSION CHECKING							
β	Factor indicating diag. cracked concr. to tension		3.7	2.2	2.6		
θ	Angle of inclination of diagonal compressive	degree	27.86	35.82	28.94		
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90		
bv	Effective web width as minimum web width - in dv	m	1.000	1.000	1.000		
dv	Effective shear depth	m	0.259	0.259	0.259		
	($d_e - a/2$)	m	0.259	0.259	0.259		
s	Spacing of stirrups	m	0.600	0.600	0.600		
ncat	Amount of bars in spacing S	Bar	2	2	2		
Av	Shear reinf area in spacing S	m ²	0.0003	0.0003	0.0003		
θ	Assume	degree	33.37	38.58	37.19		
v	Shear stress in concrete	kN/m ²	526	989	585		
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1124	1124	1124		
e_s	Strain in tensile reinforcement		1.97E-04	9.85E-04	4.71E-04		
	if $e_s < 0$, multiple with reduce factor		-	-	-		
	Strain checking	$\leq 2.00E-3$	Ok	Ok	Ok		
v/fc	Ratio of shear stress and fc		0.013	0.025	0.015		
β	Final value		3.7	2.2	2.6		
θ	Final value	degree	27.86	35.82	28.94		
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	504	305	357		
Vs	Shear resistance provided by shear reinforcement	kN	99	72	94		
Vp	Component in the direction of the applied shear of the effective P.S	kN	0	0	0		
Vn1	$V_{n1} = V_c + V_s + V_p$	kN	603	377	451		
Vn2	Vn2	kN	2591	2591	2591		
Vn	Nominal shear resistance $V_n = \min(V_{n1}, V_{n2})$	kN	603	377	451		
Vr	Factored shear resistance	kN	603	339	451		
Vu	Shear	kN	136	231	151		
(5.8.2.7)	Shear checking		OK	OK	OK		

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FO09 BRIDGE			Design			
DETAIL DESIGN			Check			
CHECK SECTION B-B			Revise			

22TCN272-05; AASHTO LRFD 2nd - 1998

REINFORCEMENT CHECKING - DECK SLAB

MATERIALS						
NORMAL CONCRETE						
f_c	Compressive Strength of concrete at 28 days	Mpa	40			
E_c	Modulus of Elasticity	Mpa	31975			
f_r	Modulus of Rupture	Mpa	4.0			
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		6			
Sign	Parameters	Unit	Section B-B			
INTERNAL FORCES AT SECTION						
	Combination		Service	Strength	Extreme	
Qu	Shear	kN	144	241	155	
Mu	Flexural Moment	kNm	143	240	185	
Nu	Axial load	kN	0	0	0	
Tu	Torsional Moment	kNm				
FLEXURAL MOMENT CHECKING						
H	Section height	m	0.611	0.611	0.611	
d's	Dis. From comp. fiber to centroid of comp. Reinf	m	0.058	0.058	0.058	
d1x	Dis. From tens. fiber to centroid of tension Reinf	m	0.060	0.060	0.060	
	Cover to reinf	m	0.050	0.050	0.050	
ds	Dis. From comp. fiber to centroid of tension Reinf	m	0.551	0.551	0.551	
d'ps	Dis. From comp. fiber to centroid of comp. prestressing steel	m	0.150	0.150	0.150	
d1xp	Dis. From tens. fiber to centroid of tension prestressing steel	m	0.100	0.100	0.100	
dps	Dis. From comp. fiber to centroid of tension prestressing steel	m	0.511	0.511	0.511	
b	Width of the compression face of member	m	1.000	1.000	1.000	
bw	Web width or diameter of a circular section	m	1.000	1.000	1.000	
hf	Compression flange depth	m	0.000	0.000	0.000	
Iz	Moment of inertia of section	m4	0.019	0.019	0.019	
Amc	Section area	m2	0.611	0.611	0.611	
	Steel choice					
Aps	Tension prestressing steel	P.S type	3 T15.2	3 T15.2	3 T15.2	
		Number	tendons	1	1	
		Area	m2	0.00042	0.00042	
A'ps	Compression prestressing steel	P.S type	0 T00.0	0 T00.0	0 T00.0	
		Number	tendons	0	0	
		Area	m2	0.00000	0.00000	
As	Tension Reinforcement	Number	bars	6	6	
		Diameter	mm	20	20	
		Area	m2	0.00188	0.00188	
A's	Compression Reinforcement	Number	bars	6	6	
		Diameter	mm	16	16	
		Area	m2	0.00121	0.00121	
A'c	Shear reinforcement	Number	bars	2	2	
		Diameter	mm	14	14	
		Area	m2	0.00030	0.00030	
ϕ	Resistance factors for flexure	5.5.4.2	1.00	0.90	1.00	
ϕ_v	Resistance factors for shear		1.00	0.90	1.00	
ϕ_n	Resistance factors for axial force		1.00	1.00	1.00	
β_1	Stress block factor		0.764	0.764	0.764	
c	Dis. Between centroid and top fiber	m	0.040	0.040	0.040	
	For T section behavior	m	0.040	0.040	0.040	
	For rectangular section behavior	m	0.040	0.040	0.040	
fpe	Effective stress in the prestressing steel after losses	Mpa	1116	1116	1116	
fps	Aver. stress in pres. steel at the time for which the nominal resistance	Mpa	1819	1819	1819	
k	Factor depends on type of P.S. Low relaxation strand k = 0.28		0.28	0.28	0.28	

a	Depth of equivalent stress block	m	0.030	0.030	0.030
de	Corresp. effective depth from extreme comp. fiber to centroid of tensile force in the tensile reinf.	m	0.531	0.531	0.531
Mn	Nominal resistance	kNm	762	762	762
Mr	Factored resistance	kNm	762	686	762
Mu	Flexural moment	kNm	143	240	185
(5.7.3.2)	Flexural moment Checking		OK	OK	OK
	Limits for reinforcement				
r min	Minimum reinforcement		0.31%	0.31%	0.31%
	Minimum reinforcement Checking for RC	0.30%	N.a	N.a	N.a
1.2*Mcr	Cracking moment	kNm	159	159	159
(5.7.3.3.2)	Checking $M_r \geq \min(1.2M_{cr}, 1.33M_u)$		OK	OK	OK
(5.7.3.4)	Control of cracking by distr. of reinf for RC member- Check?		Yes	No	No
	Existing condition for structure	1,2 or 3	1	1	1
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.060	0.060	0.060
Z	Crack width parameter	N/mm	30000	30000	30000
A	Area of concr. with same centroid as tens. Reinf	m ²	0.020	0.020	0.020
fsa	Value	Mpa	282	282	282
0.6*fy	Tensile stress in reinf Min(fs,0.6fy)	Mpa	240	240	240
x	Dist. From compression fiber to centroid	m			
J.d	Arm	m			
Icr	Moment of inertia of the cracked section	m ⁴			
fs	Tensile stress in reinforcement $f_s = M_{sl} / (A_s * J.d)$	Mpa			
	Checking for control cracking $f_s < f_{sa}$		N.a	N.a	N.a
SHEAR AND TORSION CHECKING					
β	Factor indicating diag. cracked concr. to tension		4.9	3.1	4.4
θ	Angle of inclination of diagonal compressive	degre	27.00	28.71	27.00
α	Angle of inclination of transv. reinf. to long. Axis	degre	90	90	90
bv	Effective web width as minimum web width - in dv	m	1.000	1.000	1.000
dv	Effective shear depth	m	0.516	0.516	0.516
	(de - a/2)	m	0.516	0.516	0.516
s	Spacing of stirrups	m	0.600	0.600	0.600
ncat	Amount of bars in spacing S	Bar	2	2	2
Av	Shear reinf area in spacing S	m ²	0.0003	0.0003	0.0003
θ	Assume	degre	28.59	35.52	28.78
v	Shear stress in concrete	kN/m ²	280	519	300
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1121	1121	1121
ex	Strain in tensile reinforcement		-1.33E-04	3.56E-04	6.33E-05
	if $e_x < 0$, multiple with reduce factor		-5.99E-06	-	-
	Strain checking	$\leq 2.00E-3$	Ok	Ok	Ok
v/fc	Ratio of shear stress and fc		0.007	0.013	0.008
β	Final value		4.9	3.1	4.4
θ	Final value	degre	27.00	28.71	27.00
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	1333	833	1199
Vs	Shear resistance provided by shear reinforcement	kN	204	190	204
Vp	Component in the direction of the applied shear of the effective P.S	kN	0	0	0
Vn1	$V_{n1} = V_c + V_s + V_p$	kN	1537	1022	1403
Vn2	Vn2	kN	5157	5157	5157
Vn	Nominal shear resistance $V_n = \min(V_{n1}, V_{n2})$	kN	1537	1022	1403
Vr	Factored shear resistance	kN	1537	920	1403
Vu	Shear	kN	144	241	155
(5.8.2.7)	Shear checking		OK	OK	OK

CALCULATION SHEET

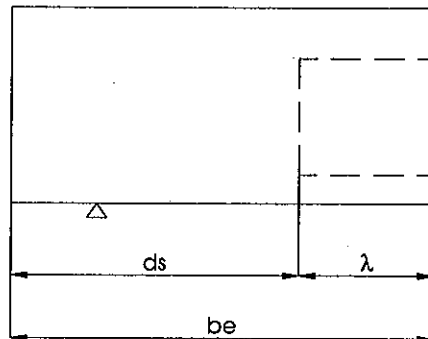
***CROSS BEAM FOR VOIDED SLAB
GIRDER***

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FO09 BRIDGE		Design			
DETAIL DESIGN		Check			
CHECK CROSS BEAM		Revise			

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I. Input data

Height of girder	1.2 m
Width of girder	7.24 m
Bottom slab width	3.5 m



1. Effective width beam

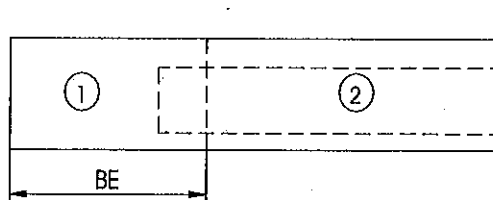
$$be = ds + k \cdot \lambda$$

with:

$\lambda =$	$n \cdot db / 6$	$\lambda =$	0.4333 m
number of void	n		2
Spacing of void	db		1.3 m
	k		1
Effective width beam	$be =$		2.433 m
Area of end of girder	$A1$		5.745 m ²

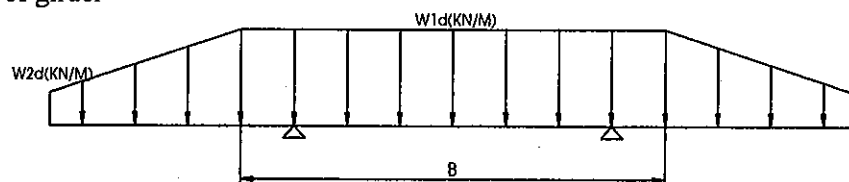
2. Loads

2.1. Dead load use midas software



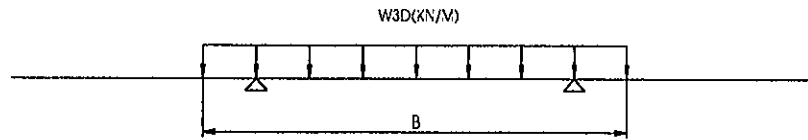
-Dead load for selfweight girder

Past "1" : end of girder



W1d	Selfweight of middle girder	71.54 KN/m
W2d	Selfweight of flag girder	14.904 KN/m

Past "2" : middle of girder



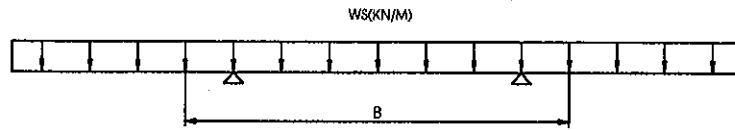
$$W3d = (\sum R_d - \sum W_d) / B$$

$\sum R_d$ Reaction force of selfweight girder 2420 KN

W_d Selfweight end girder 412.04 KN

$W3d = 573.7 \text{ KN/m}$

-Surface

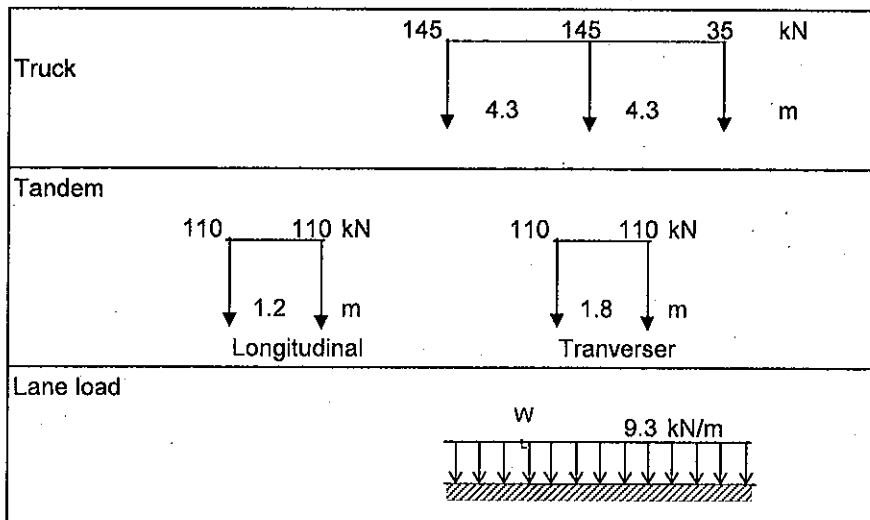


W_s Wearing surface 3.9795 KN/m

-Handrail Load for handrail 57.67 KN

2.2 Live load use Midas software

Caculation Cross beam is used last axle of truck or tandem vehicular



3. Load combination

Loads	Sign	Load combinations				
		Str-IA	Str-IB	Str-III A	Str-III B	Ser-I
Girder	DC	1.25	0.90	1.25	0.90	1.00
Handrail	DC	1.25	0.90	1.25	0.90	1.00
Pavement	DW	1.50	0.65	1.50	0.65	1.00
LiveLoad	LL	1.75	1.75	1.35	1.35	1.00

Load Combination				
Load combinations	Top		Bottom	
	Hx (kN)	My (kN.m)	Hx (kN)	My (kN.m)
Strength Str-IA	1037	949	375	622
Strength Str-IB	881	854	375	571
Service Ser-I	716	616	215	392

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FO09 BRIDGE			Design			
DETAIL DESIGN			Check			
CHECK GIRDER SECTION			Revise			

22TCN272-05; AASHTO LRFD 2nd - 1998

REINFORCEMENT CHECKING

MATERIALS							
NORMAL CONCRETE							
f _c	Compressive Strength of concrete at 28 days	Mpa	40				
E _c	Modulus of Elasticity	Mpa	31975				
f _r	Modulus of Rupture	Mpa	4.0				
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		6				
Sign	Parameters	Unit	Top	Bot			
INTERNAL FORCES AT SECTION							
	Combination		Strength	Service	Strength	Service	
Qu	Shear	kN	1037	716	375	215	
Mu	Flexural Moment	kNm	949	616	622	392	
Nu	Axial load	kN	0	0	0	0	
Tu	Torsional Moment	kNm					
FLEXURAL MOMENT CHECKING							
H	Section height	m	0.600	0.600	1.200	1.200	
d's	Dis. From comp. fiber to centroid of comp. Reinf	m	0.058	0.058	0.058	0.058	
d1x	Dis. From tens. fiber to centroid of tension Reinf	m	0.060	0.060	0.060	0.060	
	Cover to reinf	m	0.050	0.050	0.050	0.050	
ds	Dis. From comp. fiber to centroid of tension Reinf	m	0.540	0.540	1.140	1.140	
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	0.100	0.100	0.000	0.000	
dps	Dis. From comp. fiber to centroid of tension prestressing steel	m	0.500	0.500	1.200	1.200	
b	Width of the compression face of member	m	2.433	2.433	2.750	2.750	
bw	Web width or diameter of a circular section	m	2.433	2.433	2.750	2.750	
hf	Compression flange depth	m	0.000	0.000	0.000	0.000	
I _z	Moment of inertia of section	m ⁴	0.044	1.913	1.913	1.913	
A _{mc}	Section area	m ²	1.460	1.460	1.460	1.460	
Steel choice							
A _{ps}	Tension prestressing steel	P.S type	3 T15.2	3 T15.2	3 T15.2	3 T15.2	
		Number	tendons	3	3	0	0
		Area	m ²	0.00126	0.00126	0.00000	0.00000
A' _{ps}	Compression prestressing steel	P.S type	0 T00.0	0 T00.0	0 T00.0	0 T00.0	
		Number	tendons	0	0	0	0
		Area	m ²	0.00000	0.00000	0.00000	0.00000
A _s	Tension Reinforcement	Number	bars	16	16	16	16
		Diameter	mm	20	20	16	16
		Area	m ²	0.00509	0.00509	0.00328	0.00328
A' _s	Compression Reinforcement	Number	bars	16	16	0	0
		Diameter	mm	16	16	20	20
		Area	m ²	0.00328	0.00328	0.00000	0.00000
A' _c	Shear reinforcement	Number	bars	6	6	6	6
		Diameter	mm	20	20	20	20
		Area	m ²	0.00188	0.00188	0.00188	0.00188
φ	Resistance factors for flexure		5.5.4.2	0.90	1.00	0.90	1.00
φ _v	Resistance factors for shear			0.90	1.00	0.90	1.00
φ _n	Resistance factors for axial force			1.00	1.00	1.00	2.00
β ₁	Stress block factor			0.764	0.764	0.764	0.764
c	Dis. Between centroid and top fiber	m		0.048	0.048	0.018	0.018
	For T section behavior	m		0.048	0.048	0.018	0.018
	For rectangular section behavior	m		0.048	0.048	0.018	0.018
f _{pe}	Effective stress in the prestressing steel after losses	Mpa		1116	1116	1116	1116
f _{ps}	Aver. stress in pres. steel at the time for which the nominal resistance	Mpa		1810	1810	1852	1852
k	Factor depends on type of P.S, Low relaxation strand k = 0.28			0.28	0.28	0.28	0.28

a	Depth of equivalent stress block	m	0.036	0.036	0.014	0.014
de	Corresp. effective depth from extreme comp. fiber to centroid of tensile force in the tensile reinf.	m	0.519	0.519	1.140	1.140
Mn	Nominal resistance	kNm	2110	2110	1485	1485
Mr	Factored resistance	kNm	1899	2110	1337	1485
Mu	Flexural moment	kNm	949	616	622	392
(5.7.3.2)	Flexural moment Checking		OK	OK	OK	OK
	Limits for reinforcement					
r min	Minimum reinforcement		0.35%	0.35%	0.22%	0.22%
	Minimum reinforcement Checking for RC	0.30%	N.a	N.a	N.G	N.G
1.2*Mcr	Cracking moment	kNm	379	16557	7741	7741
(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
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.060	0.060	0.058	0.058
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
fsa	Value	Mpa	292	292	287	287
0.6*fy	Tensile stress in reinf Min(fs,0.6fy)	Mpa	240	240	240	240
x	Dist. From compression fiber to centroid	m	-	0.102	-	0.121
J.d	Arm	m	-	0.485	-	1.10
Icr	Moment of inertia of the cracked section	m ⁴	-	0.007	-	0.022
fs	Tensile stress in reinforcement $f_s = M_{sls} / (A_s * J.d)$	Mpa	-	249	-	109
	Checking for control cracking $f_s < f_{sa}$		N.a	N.a	N.a	OK
SHEAR AND TORSION CHECKING						
β	Factor indicating diag. cracked concr. to tension		2.3	3.0	2.1	2.4
θ	Angle of inclination of diagonal compressive	deg	35.45	28.74	38.03	33.33
α	Angle of inclination of transv. reinf. to long. Axis	deg	90	90	90	90
bv	Effective web width as minimum web width - in dv	m	2.433	2.433	2.750	2.750
dv	Effective shear depth	m	0.501	0.501	1.133	1.133
	($d_e - a/2$)	m	0.501	0.501	1.133	1.133
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600
ncat	Amount of bars in spacing S	Bar	6	6	6	6
Av	Shear reinf area in spacing S	m ²	0.0019	0.0019	0.0019	0.0019
θ	Assume	deg	35.47	28.75	38.04	33.33
v	Shear stress in concrete	kN/m ²	946	588	134	69
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1122	1122	1116	1116
ϵ_s	Strain in tensile reinforcement		9.54E-04	3.70E-04	1.20E-03	7.77E-04
	if $\epsilon_s < 0$, multiple with reduce factor		-	-	-	-
	Strain checking	$\leq 2.00E-3$	Ok	Ok	Ok	Ok
v/fc	Ratio of shear stress and fc		0.024	0.015	0.003	0.002
β	Final value		2.3	3.0	2.1	2.4
θ	Final value	deg	35.45	28.74	38.03	33.33
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	1443	1932	3461	3851
Vs	Shear resistance provided by shear reinforcement	kN	883	1147	1819	2164
Vp	Component in the direction of the applied shear of the effective P.S	kN	2104	2104	0	0
Vn1	$V_{n1} = V_c + V_s + V_p$	kN	4430	5183	5280	6016
Vn2	Vn2	kN	14288	14288	31157	31157
Vn	Nominal shear resistance $V_n = \min(V_{n1}, V_{n2})$	kN	4430	5183	5280	6016
Vr	Factored shear resistance	kN	3987	5183	4752	6016
Vu	Shear	kN	1037	716	375	215
(5.8.2.7)	Shear checking		OK	OK	OK	OK

MINISTRY OF TRANSPORT

VIET NAM EXPRESSWAY CORPORATION - PROJECT MANAGEMENT UNIT NO.85

DA NANG - QUANG NGAI EXPRESS WAY PROJECT
PACKAGE: A2

BRIDGE
F009

CALCULATION SHEETS
SUBSTRUCTURE

CALCULATION SHEET

PIER P1

	DA NANG - QUANG NGAI EXPRESSWAY PROJECT FO09 - Bridge DETAIL DESIGN Pier Design - P1	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 $A = 0.0580 \text{ g}$

Input data:

Bridge width $B = 7.50 \text{ m}$

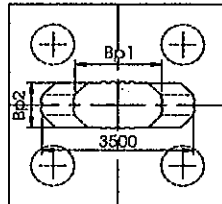
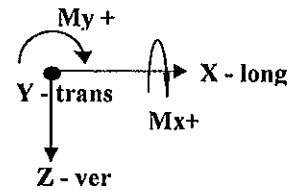
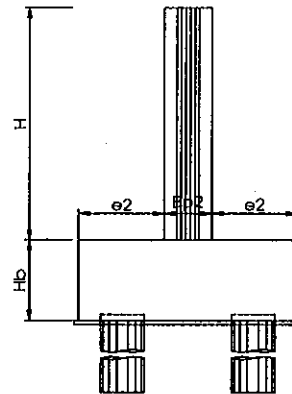
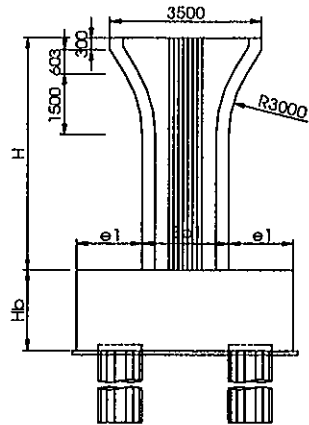
Level Table(at center of pier)

Top of pier cap	ThL	16.115	m
Top of pier column	TcL	16.115	m
Bottom of upper pier column	BucL	16.115	m
Bottom of pier column	BcL	10.000	m
Bottom of upper pilecap	BupL	10.000	m
Bottom of pilecap	BpL	8.000	m
Tip of pile	TpL	4.000	m
Skew angle	Ska	0.000	deg
Ground level	GL	11.098	m
Maximum water level (H1%)	HWL	0.000	m
Navigation water level (H5%)	NWL	0.000	m
Minimum water level	MWL	0.000	m
Average Annual water level	AWL	0.000	m
Local scour level (at water level H1%)	LsL	0.000	m

Material unit weight

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

II. PIER DIMENSIONS



Pier Dimensions Table

Notation	Dimensions Transverse direction	Value (m)	Notation	Dimensions Longitudinal direction	Value (m)
*	Pier Column				
Bp2	Pier column width	2.00	D	Diameter of pier column	-
Htt	Pier column height	6.11	Lp	Pier column thickness	1.10
Bp1	Upper pier column width	2.00			
Htb	Upper pier column height	0.00			
Ht	Column height	6.11			

Notation	Dimensions Transverse direction	Value (m)	Notation	Dimensions Longitudinal direction	Value (m)
*	Pile Cap				
Bb	Pile cap width	5.00	Lb	Pile cap length	5.00
Hb	Pile cap depth	2.00			
e1	Transverse cantilever	1.50	e2	Long. Cantilever	1.95
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 Calculated by software

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	25.23	446.6						
Total at bottom of Column								
Total at bottom of pilecap		446.6						

3.Buoyancy on pier N/A

4.Stream Pressure N/A

5.Wind Loads

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	V_b	53.0 m/s
Type of terrain at bridge location		1
"1:exposed area"; "2: forest, houses, ... with height 10m"; "3:houses area..with height>10m"		
Average elevation of pier upper ground or water plane level	Hele_p	5.0 m
Correct coefficient for wind zone and elevation of pier	S	1.09
Design wind speed $V = S \cdot V_b$	V	57.8 m/s
Obstacle coefficient for pier	C_d	1.21
Wind pressure on pier	P_D	2.42 kN/m2

6.Vehicular Collision Force "1:yes"; "0:no"

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	7.44		1800.0			1800.0		
Total at bottom of Column			1800.0			1800.0		
Pier column	9.44		1800.0			1800.0		
Total at bottom of pilecap			1800.0			1800.0		

Vehicular collision force is calculated by software

7.Settlement Loads

Settlement load caused by non-uniform settlement of piers and abutments for continuous spans, settlement value is assumed 10mm for each piers or abutments. Some combinations of non-uniform settlement shall be considered for the unfavourable case.

IV. SUPERSTRUCTURE LOADS

1.Dead Loads

1.1 Selfweight : Calculated by software

1.2 Dead loads stage 2:

a) Wearing surface weight :

Traffic lanes:

7.0 cm asphalt + 0.4 cm water proof membrane: 2250 kg/m3

10.6 kN/m3

b) Curb and rail at side:
Concrete median + steel rail for 2 side:

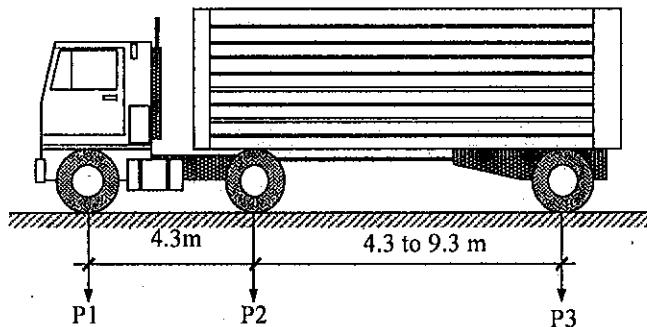
22.9 kN/m3

2.Live Load

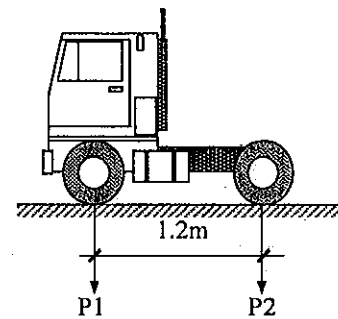
Live load data

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	2.0 lanes
Multiple presence factor	m	1.00
Dynamic load allowance (1+IM)		
Deck joint - all limit states		1.75
Other structure - all limit states (except fatigue)		1.25

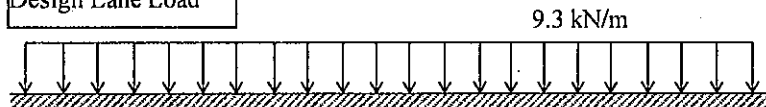
Design Truck



Design Tandem



Design Lane Load



3.Centrifugal Force

Centrifugal force data

$$CE = n * m * (\text{Axle weights}) * C$$

Axle weights of design Truck	P	325.0 kN
Number of loaded lanes	n	2.0 lanes
	m	1.00
Factor, $C = (4/3) * V^2 / (g * R)$	C	0.0 kN
Highway design speed	V	13.9 m/s
Radius of curvature of traffic lane	R	1000000.0 m
Centrifugal force	CE	0.0 kN

4.Braking Force

Braking force data

Axle weights of design Truck	P	325.0 kN
Number of loaded lanes	n	2.0 lanes
	m	1.00
$Br1 = 25\% * (\text{design truck}) * n * m$	Br1	162.50 kN
$Br2 = 5\% * (\text{design truck} + 9.3 * L_{\text{bridge}}) * n * m$	Br2	155.26 kN
$Br = \max(Br1, Br2)$	Br	162.50 kN

5.Uniform Temperature

Uniform temperature data

Installing temperature	t0	25.0	deg
Maximum temperature	tmax	47.0	deg
Minimum temperature	tmin	10.00	deg
Plus temperature amplitude	Δt_{max}	22.0	deg
Minus temperature amplitude	Δt_{min}	15.0	deg
Coefficient of Thermal Expansion	α	1.08E-05	

6.Creep & Shrinkage

(CEB-FIB MODEL CODE 1990)

7.Wind on Structure

Wind loads data

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

Average elevation of deck girder upper ground or water plane level	Hele_g	6.3	m
Correct coefficient for wind zone and elevation of pier	S	1.09	
Design wind speed $V = S \cdot V_b$	V	57.8	m/s
Overall width between handrails	b	7.5	m
Superstructure height including solid parapet	d	1.89	m
	b/d	3.98	
Obstacle coefficient for pier	Cd	1.21	
Wind pressure on pier	P _D	2.42	kN/m2

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.

9.Earth Quake

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	2	essential
Response Modification Factor			
Column		2.0	
Connection		1.0	
Foundation		1.0	

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			Revise	-		

B.LOAD COMBINATIONS

I. LOAD COMBINATIONS

Item		Sign	Strength I-A	Strength I-B	Strength II-A	Strength II-B	Strength III-A	Strength III-B
Dead load from superstr.	Struct.	DC	1.25	0.90	1.25	0.90	1.25	0.90
	Wear.	DW	1.50	0.65	1.50	0.65	1.50	0.65
Prestr. of cable or EL second. force		PS	1.00	1.00	1.00	1.00	1.00	1.00
Live load (Vehi.+IM+Lane+Ped.)	Max	LL	1.75				1.35	
	Min	LL		1.75				1.35
Braking force		BR	1.75	1.75			1.35	1.35
Wind load on superstr.	Trans.	WS			1.40	1.40	0.40	0.40
	Long.	WS			1.40	1.40	0.40	0.40
Wind load on vehicular	Trans.	WL					1.00	1.00
	Long.	WL					1.00	1.00
Temperature uniform	Uniform	TU	0.50	0.50	0.50	0.50	0.50	0.50
	Gradient	TG						
Creep & shrinkage		Cr&Sh	0.50	0.50	0.50	0.50	0.50	0.50
Settlement Load		SE	0.50	0.50	0.50	0.50	0.50	0.50
Dead load from substr.		DC	1.25	0.90	1.25	0.90	1.25	0.90
Soil on pilecap		EV	1.35	0.90	1.35	0.90	1.35	0.90
Wind load on substr.		WS			1.40	1.40	0.40	0.40
Water buoyancy	Max	WA	1.00		1.00		1.00	
	Min	WA		1.00		1.00		1.00
	Annual	WA						
Horizontal water pressure	Max	WA	1.00		1.00		1.00	
	Min	WA		1.00		1.00		1.00
	Annual	WA						
Earthquake	Trans.	EQ						
	Long.	EQ						
Collision force	Trans.	CT						
	Long.	CT						

Item		Sign	Service I	Service III	Extreme I-A	Extreme I-B	Extreme I-C	Extreme I-D
Dead load from superstr.	Struct.	DC	1.00	1.00	1.25	0.90	1.25	0.90
	Wear.	DW	1.00	1.00	1.50	0.65	1.50	0.65
Prestr. of cable or EL second. force		PS	1.00	1.00	1.00	1.00	1.00	1.00
Live load (Vehi.+IM+Lane+Ped.)	Max	LL	1.00	0.80	0.50		0.50	
	Min	LL	1.00	0.80		0.50		0.50
Braking force		BR	1.00	0.80	0.50	0.50	0.50	0.50
Wind load on superstr.	Trans.	WS	0.30	0.30				
	Long.	WS	0.30	0.30				
Wind load on vehicular	Trans.	WL	1.00	1.00				
	Long.	WL	1.00	1.00				
Temperature uniform	Uniform	TU	1.00	1.00				
	Gradient	TG	0.50	0.50				
Creep & shrinkage		Cr&Sh	1.00	1.00				
Settlement Load		SE						
Dead load from substr.		DC	1.00	1.00	1.25	0.90	1.25	0.90
Soil on pilecap		EV	1.00	1.00	1.35	0.90	1.35	0.90
Wind load on substr.		WS	0.30	0.30				
Water buoyancy	Max	WA	1.00	1.00				
	Min	WA						
	Annual	WA			1.00	1.00	1.00	1.00
Horizontal water pressure	Max	WA	1.00	1.00				
	Min	WA						
	Annual	WA			1.00	1.00	1.00	1.00
Earthquake	Trans.	EQ			1.00	1.00	0.30	0.30
	Long.	EQ			0.30	0.30	1.00	1.00
Collision force	Trans.	CT						
	Long.	CT						

Item		Sign	Extreme II-A	Extreme II-B	Extreme II-C	Extreme II-D
Dead load from superstr.	Struct.	DC	1.25	0.90	1.25	0.90
	Wear.	DW	1.50	0.65	1.50	0.65
Prestr. of cable or EL second. force		PS	1.00	1.00	1.00	1.00
Live load (Vehi.+IM+Lane+Ped.)	Max	LL	0.50		0.50	
	Min	LL		0.50		0.50
Braking force		BR	0.50	0.50	0.50	0.50
Wind load on superstr.	Trans.	WS				
	Long.	WS				
Wind load on vehicular	Trans.	WL				
	Long.	WL				
Temperature uniform	Uniform	TU				
	Gradient	TG				
Creep & shrinkage		Cr&Sh				
Settlement Load		SE				
Dead load from substr.		DC	1.25	0.90	1.25	0.90
Soil on pilecap		EV	1.35	0.90	1.35	0.90
Wind load on substr.		WS				
Water buoyancy	Max	WA				
	Min	WA				
	Annual	WA	1.00	1.00	1.00	1.00
Horizontal water pressure	Max	WA				
	Min	WA				
	Annual	WA	1.00	1.00	1.00	1.00
Earthquake	Trans.	EQ				
	Long.	EQ				
Collision force	Trans.	CT	1.00	1.00	0.00	0.00
	Long.	CT	0.00	0.00	1.00	1.00

II. LOAD COMBINATIONS RESULT

Load Combinations at Bottom of PierColumn

No	Combinations		Sign	F _V (kN)	Longitudinal		Transvesal	
					F _{HX} (kN)	My (kN•m)	F _{HY} (kN)	Mx (kN•m)
1	Strength I-A		Str1a	8088	-268	-974	97	-698
2	Strength I-B		Str1b	4964	-248	-1812	152	-1499
3	Strength II-A		Str2a	5951	42	154	115	-587
4	Strength II-B		Str2b	3742	42	154	115	-587
5	Strength III-A		Str3a	6889	-212	-1415	167	-1530
6	Strength III-B		Str3b	5390	-222	-791	139	-897
7	Service I		Ser1	5091	-160	-1086	143	-1173
8	Service III		Ser3	5125	90	772	-29	309
9	Extreme 1a	EQT	Ext1a	6362	68	512	-51	2209
10	Extreme 1b	EQT	Ext1b	4381	-73	-509	38	-2248
11	Extreme 1c	EQL	Ext1c	6348	-71	-2706	43	-428
12	Extreme 1d	EQL	Ext1d	4426	67	2702	-53	390
13	Extreme 3a	CTT	Ext3a	6362	149	765	-1630	3533
14	Extreme 3b	CTT	Ext3b	4381	-153	-762	1617	-3571
15	Extreme 3c	CTL	Ext3c	6348	-1820	-4145	184	-987
16	Extreme 3d	CTL	Ext3d	4426	1816	4141	-194	950

Load Combinations at Top of PierColumn

No	Combinations		Sign	F _V (kN)	Longitudinal		Transvesal	
					F _{HX} (kN)	My (kN•m)	F _{HY} (kN)	Mx (kN•m)
1	Strength I-A		Str1a	7803	-268	246	97	-256
2	Strength I-B		Str1b	4727	273	-1305	-85	-869
3	Strength II-A		Str2a	5666	36	-22	98	-103
4	Strength II-B		Str2b	3537	36	-22	98	-103
5	Strength III-A		Str3a	6626	172	-676	-66	-970
6	Strength III-B		Str3b	5186	-224	223	134	-276
7	Service I		Ser1	4846	136	-711	4	-568
8	Service III		Ser3	4902	-150	655	74	191
9	Extreme 1a	EQT	Ext1a	6023	-71	232	42	621
10	Extreme 1b	EQT	Ext1b	4185	70	-263	-46	-700
11	Extreme 1c	EQL	Ext1c	6054	78	-1029	-24	-248
12	Extreme 1d	EQL	Ext1d	4224	-82	1043	14	164
13	Extreme 3a	CTT	Ext3a	6023	-152	346	-179	968
14	Extreme 3b	CTT	Ext3b	4185	150	-377	175	-1046
15	Extreme 3c	CTL	Ext3c	6054	27	-1373	-165	-329
16	Extreme 3d	CTL	Ext3d	4224	-30	1387	155	244

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 I-A	Str1a	10222	-268	-1510	97	-893
2	Strength I-B	Str1b	6451	-250	-2306	147	-1800
3	Strength II-A	Str2a	8085	44	240	122	-824
4	Strength II-B	Str2b	5246	44	240	122	-824
5	Strength III-A	Str3a	9014	-207	-1847	180	-1876
6	Strength III-B	Str3b	6895	-221	-1234	141	-1177
7	Service I	Ser1	7094	-160	-1403	141	-1457
8	Service III	Ser3	7119	91	952	-26	366
9	Extreme 1a EQT	Ext1a	8523	68	649	-52	3020
10	Extreme 1b EQT	Ext1b	5883	-71	-659	42	-3037
11	Extreme 1c EQL	Ext1c	8477	-71	-4070	42	-514
12	Extreme 1d EQL	Ext1d	5929	68	4060	-52	497
13	Extreme 3a CTT	Ext3a	8523	148	1062	-1631	6792
14	Extreme 3b CTT	Ext3b	5883	-152	-1072	1621	-6809
15	Extreme 3c CTL	Ext3c	8477	-1820	-7784	183	-1354
16	Extreme 3d CTL	Ext3d	5929	1817	7774	-193	1337

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		Design	-		
		Check	-		
		Revise	-		

C.1.COLUMN DESIGN

I. COLUMN DATA

1.Load Combinations at Bottom of Pier Column

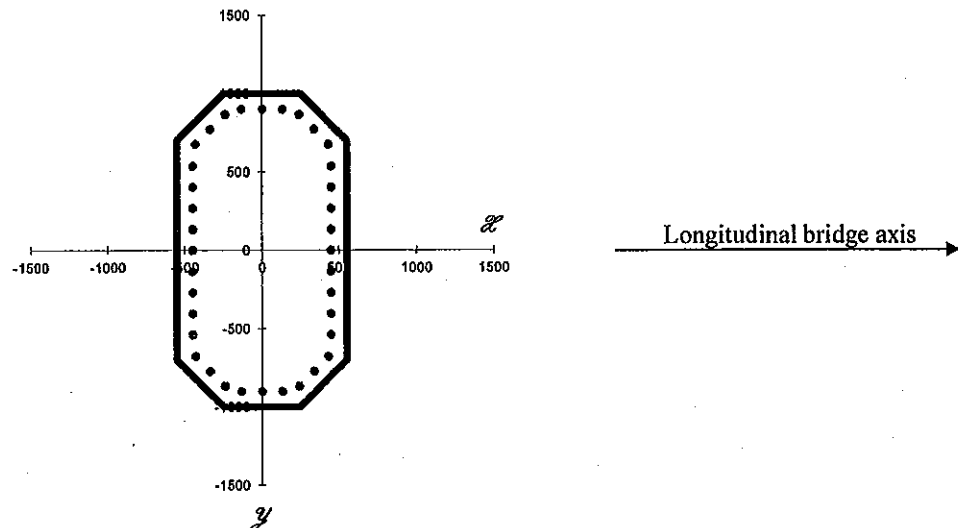
No	Combinations	Sign	F _v (kN)	Longitudinal		Transvesal	
				F _{Hx} (kN)	My (kN•m)	F _{Hy} (kN)	Mx (kN•m)
1	Strength I-A	Str1a	8088	-268	-974	97	-698
2	Strength I-B	Str1b	4964	-248	-1812	152	-1499
3	Strength II-A	Str2a	5951	42	154	115	-587
4	Strength II-B	Str2b	3742	42	154	115	-587
5	Strength III-A	Str3a	6889	-212	-1415	167	-1530
6	Strength III-B	Str3b	5390	-222	-791	139	-897
7	Service I	Ser1	5091	-160	-1086	143	-1173
8	Service III	Ser3	5125	90	772	-29	309
9	Extreme 1a EQT	Ext1a	6362	68	512	-51	2209
10	Extreme 1b EQT	Ext1b	4381	-73	-509	38	-2248
11	Extreme 1c EQL	Ext1c	6348	-71	-2706	43	-428
12	Extreme 1d EQL	Ext1d	4426	67	2702	-53	390
13	Extreme 3a CTT	Ext3a	6362	149	765	-1630	3533
14	Extreme 3b CTT	Ext3b	4381	-153	-762	1617	-3571
15	Extreme 3c CTL	Ext3c	6348	-1820	-4145	184	-987
16	Extreme 3d CTL	Ext3d	4426	1816	4141	-194	950

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	1.10	m
Pier column width - transverse dimension	tn	2.00	m
Section area	A	2.020	m ²
Moment inertia	I _x	0.587	m ⁴
	I _y	0.185	m ⁴
Radius of gyration of gross concrete section; $r = \sqrt{I/A}$	r _x	0.539	m
	r _y	0.302	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	K _t	0.65	
Longitudinal direction: Fixed at bottom; translation free, rotation free at top	K _l	0.65	
Unsupported length from top to bottom of column	L _u	6.12	m
Slenderness ratio: if $K \cdot L_u / r > 22$ than considered	$K_t \cdot L_u / r_x$	7.4	no
	$K_l \cdot L_u / r_y$	13.1	no
Moment inertia of longitudinal reinforcements	I _s	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 determination 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

$$\Delta x_g = F_x \cdot H^3 / (3 \cdot E \cdot I_g)$$

$$\Delta x_{cr} = F_{cr} \cdot \Delta x_g$$

$$M_{P-\Delta} = \Delta x_{cr} \cdot P$$

$$\Delta F_x = M_{P-\Delta} / H$$

$$\Delta x_{g\ i} = (F_x + \Delta F_{x\ i-1}) \cdot H^3 / (3 \cdot E \cdot I_g)$$

$$\Delta x_{cr\ i} = F_{cr} \cdot \Delta x_{g\ i}$$

$$M_{P-\Delta\ i} = \Delta x_{cr\ i} \cdot P$$

$$\Delta F_{x\ i} = M_{P-\Delta\ i} / H$$

Combination	Fv (kN)	My (kNm)	Initial					
			Fx (kN)	Δxg (m)	Fcr (kN)	Δxcr (m)	M P- Δ (kNm)	ΔFx (kN)
Strength I-A	8088	-974	-159	-0.002	2.5	-0.006	-48	-7.9
Strength I-B	4964	-1812	-296	-0.004	2.5	-0.011	-55	-9.0
Strength II-A	5951	154	25	0.000	2.5	0.001	6	0.9
Strength II-B	3742	154	25	0.000	2.5	0.001	4	0.6
Strength III-A	6889	-1415	-231	-0.003	2.5	-0.009	-59	-9.7
Strength III-B	5390	-791	-129	-0.002	2.5	-0.005	-26	-4.2
Service I	5091	-1086	-178	-0.003	2.5	-0.007	-34	-5.5
Service III	5125	772	126	0.002	2.5	0.005	24	3.9
Extreme 1a	6362	512	84	0.001	2.5	0.003	20	3.2
Extreme 1b	4381	-509	-83	-0.001	2.5	-0.003	-14	-2.2
Extreme 1c	6348	-2706	-443	-0.007	2.5	-0.016	-105	-17.1
Extreme 1d	4426	2702	442	0.007	2.5	0.016	73	11.9
Extreme 3a	6362	765	125	0.002	2.5	0.005	30	4.8
Extreme 3b	4381	-762	-125	-0.002	2.5	-0.005	-20	-3.3
Extreme 3c	6348	-4145	-678	-0.010	2.5	-0.025	-160	-26.2
Extreme 3d	4426	4141	677	0.010	2.5	0.025	112	18.3

Combination	Fv (kN)	My (kNm)	1st Trial					
			Fx (kN)	Δxg (m)	Fcr (kN)	Δxcr (m)	M P- Δ (kNm)	ΔFx (kN)
Strength I-A	8088	-974	-167	-0.002	2.5	-0.006	-50	-8.2
Strength I-B	4964	-1812	-305	-0.005	2.5	-0.011	-56	-9.2
Strength II-A	5951	154	26	0.000	2.5	0.001	6	0.9
Strength II-B	3742	154	26	0.000	2.5	0.001	4	0.6
Strength III-A	6889	-1415	-241	-0.004	2.5	-0.009	-62	-10.1
Strength III-B	5390	-791	-134	-0.002	2.5	-0.005	-27	-4.4
Service I	5091	-1086	-183	-0.003	2.5	-0.007	-35	-5.7
Service III	5125	772	130	0.002	2.5	0.005	25	4.1
Extreme 1a	6362	512	87	0.001	2.5	0.003	21	3.4
Extreme 1b	4381	-509	-85	-0.001	2.5	-0.003	-14	-2.3
Extreme 1c	6348	-2706	-460	-0.007	2.5	-0.017	-109	-17.8
Extreme 1d	4426	2702	454	0.007	2.5	0.017	75	12.2
Extreme 3a	6362	765	130	0.002	2.5	0.005	31	5.0
Extreme 3b	4381	-762	-128	-0.002	2.5	-0.005	-21	-3.4
Extreme 3c	6348	-4145	-704	-0.010	2.5	-0.026	-167	-27.2
Extreme 3d	4426	4141	695	0.010	2.5	0.026	115	18.8

Combination	Fv (kN)	My (kNm)	2nd Trial					
			Fx (kN)	Δxg (m)	Fcr (kN)	Δxcr (m)	M P- Δ (kNm)	ΔFx (kN)
Strength I-A	8088	-974	-168	-0.002	2.5	-0.006	-50	-8.3
Strength I-B	4964	-1812	-306	-0.005	2.5	-0.011	-57	-9.2
Strength II-A	5951	154	26	0.000	2.5	0.001	6	0.9
Strength II-B	3742	154	26	0.000	2.5	0.001	4	0.6
Strength III-A	6889	-1415	-242	-0.004	2.5	-0.009	-62	-10.1
Strength III-B	5390	-791	-134	-0.002	2.5	-0.005	-27	-4.4
Service I	5091	-1086	-183	-0.003	2.5	-0.007	-35	-5.7
Service III	5125	772	130	0.002	2.5	0.005	25	4.1
Extreme 1a	6362	512	87	0.001	2.5	0.003	21	3.4
Extreme 1b	4381	-509	-86	-0.001	2.5	-0.003	-14	-2.3
Extreme 1c	6348	-2706	-460	-0.007	2.5	-0.017	-109	-17.8
Extreme 1d	4426	2702	454	0.007	2.5	0.017	75	12.2
Extreme 3a	6362	765	130	0.002	2.5	0.005	31	5.0
Extreme 3b	4381	-762	-128	-0.002	2.5	-0.005	-21	-3.4
Extreme 3c	6348	-4145	-705	-0.011	2.5	-0.026	-167	-27.3
Extreme 3d	4426	4141	696	0.010	2.5	0.026	115	18.8

****Transverse Direction**

Combination	Fv (kN)	Mx (kNm)	Initial					
			Fx (kN)	Δxg (m)	Fcr (kN)	Δxcr (m)	M P-Δ (kNm)	ΔFx (kN)
Strength I-A	8088	-698	-114	-0.001	2.5	-0.001	-11	-1.8
Strength I-B	4964	-1499	-245	-0.001	2.5	-0.003	-14	-2.3
Strength II-A	5951	-587	-96	0.000	2.5	-0.001	-7	-1.1
Strength II-B	3742	-587	-96	0.000	2.5	-0.001	-4	-0.7
Strength III-A	6889	-1530	-250	-0.001	2.5	-0.003	-20	-3.3
Strength III-B	5390	-897	-147	-0.001	2.5	-0.002	-9	-1.5
Service I	5091	-1173	-192	-0.001	2.5	-0.002	-11	-1.9
Service III	5125	309	51	0.000	2.5	0.001	3	0.5
Extreme 1a	6362	2209	361	0.002	2.5	0.004	27	4.4
Extreme 1b	4381	-2248	-368	-0.002	2.5	-0.004	-19	-3.1
Extreme 1c	6348	-428	-70	0.000	2.5	-0.001	-5	-0.9
Extreme 1d	4426	390	64	0.000	2.5	0.001	3	0.5
Extreme 3a	6362	3533	578	0.003	2.5	0.007	43	7.1
Extreme 3b	4381	-3571	-584	-0.003	2.5	-0.007	-30	-4.9
Extreme 3c	6348	-987	-161	-0.001	2.5	-0.002	-12	-2.0
Extreme 3d	4426	950	155	0.001	2.5	0.002	8	1.3

Combination	Fv (kN)	Mx (kNm)	1st Trial					
			Fx (kN)	Δxg (m)	Fcr (kN)	Δxcr (m)	M P-Δ (kNm)	ΔFx (kN)
Strength I-A	8088	-698	-116	-0.001	2.5	-0.001	-11	-1.8
Strength I-B	4964	-1499	-247	-0.001	2.5	-0.003	-14	-2.4
Strength II-A	5951	-587	-97	0.000	2.5	-0.001	-7	-1.1
Strength II-B	3742	-587	-97	0.000	2.5	-0.001	-4	-0.7
Strength III-A	6889	-1530	-254	-0.001	2.5	-0.003	-20	-3.3
Strength III-B	5390	-897	-148	-0.001	2.5	-0.002	-9	-1.5
Service I	5091	-1173	-194	-0.001	2.5	-0.002	-12	-1.9
Service III	5125	309	51	0.000	2.5	0.001	3	0.5
Extreme 1a	6362	2209	366	0.002	2.5	0.004	27	4.5
Extreme 1b	4381	-2248	-371	-0.002	2.5	-0.004	-19	-3.1
Extreme 1c	6348	-428	-71	0.000	2.5	-0.001	-5	-0.9
Extreme 1d	4426	390	64	0.000	2.5	0.001	3	0.5
Extreme 3a	6362	3533	585	0.003	2.5	0.007	44	7.1
Extreme 3b	4381	-3571	-589	-0.003	2.5	-0.007	-30	-4.9
Extreme 3c	6348	-987	-163	-0.001	2.5	-0.002	-12	-2.0
Extreme 3d	4426	950	157	0.001	2.5	0.002	8	1.3

Combination	Fv (kN)	Mx (kNm)	2nd Trial					
			Fx (kN)	Δxg (m)	Fcr (kN)	Δxcr (m)	M P-Δ (kNm)	ΔFx (kN)
Strength I-A	8088	-698	-116	-0.001	2.5	-0.001	-11	-1.8
Strength I-B	4964	-1499	-247	-0.001	2.5	-0.003	-14	-2.4
Strength II-A	5951	-587	-97	0.000	2.5	-0.001	-7	-1.1
Strength II-B	3742	-587	-97	0.000	2.5	-0.001	-4	-0.7
Strength III-A	6889	-1530	-254	-0.001	2.5	-0.003	-20	-3.4
Strength III-B	5390	-897	-148	-0.001	2.5	-0.002	-9	-1.5
Service I	5091	-1173	-194	-0.001	2.5	-0.002	-12	-1.9
Service III	5125	309	51	0.000	2.5	0.001	3	0.5
Extreme 1a	6362	2209	366	0.002	2.5	0.004	27	4.5
Extreme 1b	4381	-2248	-371	-0.002	2.5	-0.004	-19	-3.1
Extreme 1c	6348	-428	-71	0.000	2.5	-0.001	-5	-0.9
Extreme 1d	4426	390	64	0.000	2.5	0.001	3	0.5
Extreme 3a	6362	3533	585	0.003	2.5	0.007	44	7.1
Extreme 3b	4381	-3571	-589	-0.003	2.5	-0.007	-30	-4.9
Extreme 3c	6348	-987	-163	-0.001	2.5	-0.002	-12	-2.0
Extreme 3d	4426	950	157	0.001	2.5	0.002	8	1.3

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

Combination	Fv Vert. (kN)	Mx Trans. (kNm)	Mx P-Δ (kNm)	Mx Total (kNm)	My Long. (kNm)	My P-Δ (kNm)	My Total (kNm)
Strength I-A	8088	-698	-11	-709	-974	-50	-1024
Strength I-B	4964	-1499	-14	-1513	-1812	-57	-1869
Strength II-A	5951	-587	-7	-594	154	6	160
Strength II-B	3742	-587	-4	-591	154	4	158
Strength III-A	6889	-1530	-20	-1550	-1415	-62	-1477
Strength III-B	5390	-897	-9	-906	-791	-27	-818
Service I	5091	-1173	-12	-1185	-1086	-35	-1121
Service III	5125	309	3	312	772	25	797
Extreme 1a	6362	2209	27	2236	512	21	533
Extreme 1b	4381	-2248	-19	-2267	-509	-14	-523
Extreme 1c	6348	-428	-5	-433	-2706	-109	-2815
Extreme 1d	4426	390	3	393	2702	75	2777
Extreme 3a	6362	3533	44	3577	765	31	796
Extreme 3b	4381	-3571	-30	-3601	-762	-21	-783
Extreme 3c	6348	-987	-12	-999	-4145	-167	-4312
Extreme 3d	4426	950	8	958	4141	115	4256

II. PIER COLUMN DESIGN

1. Limit of Reinforcement

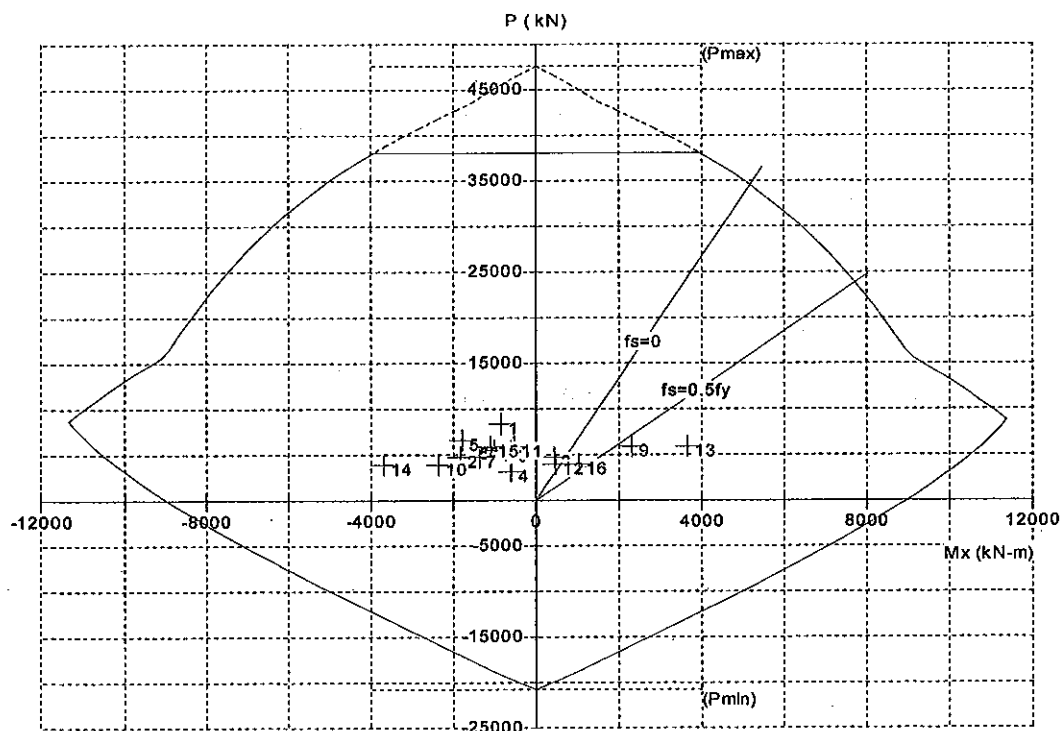
S.5.7.4.2

Minimum area of longitudinal reinforcement in column					
$A_s \cdot f_y / (A_g \cdot f_c) \geq 0.135$		$A_s \geq$		0.020	m2
$A_s / A_g \geq 0.01$		$A_s \geq$		0.020	m2
Maximum area of longitudinal reinforcement in column					
$A_s / A_g \leq 0.08$		$A_s \leq$		0.162	m2
Trial Rebars:		Ok	A_s	0.058	m2
1layers	x 72	= 72 bars	D32 @150	As1	0.058 m2
1layers	x 0	= 0 bars	D25 @150	As2	0.000 m2

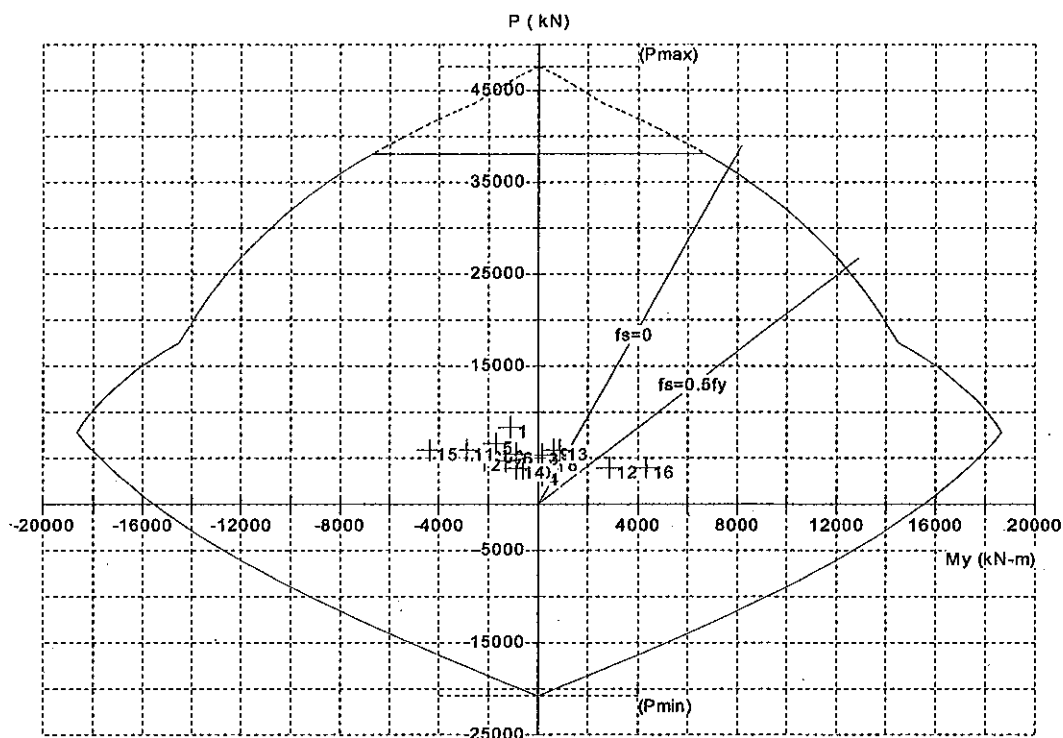
2. Iteration diagram M-P

Using Pca-Column software

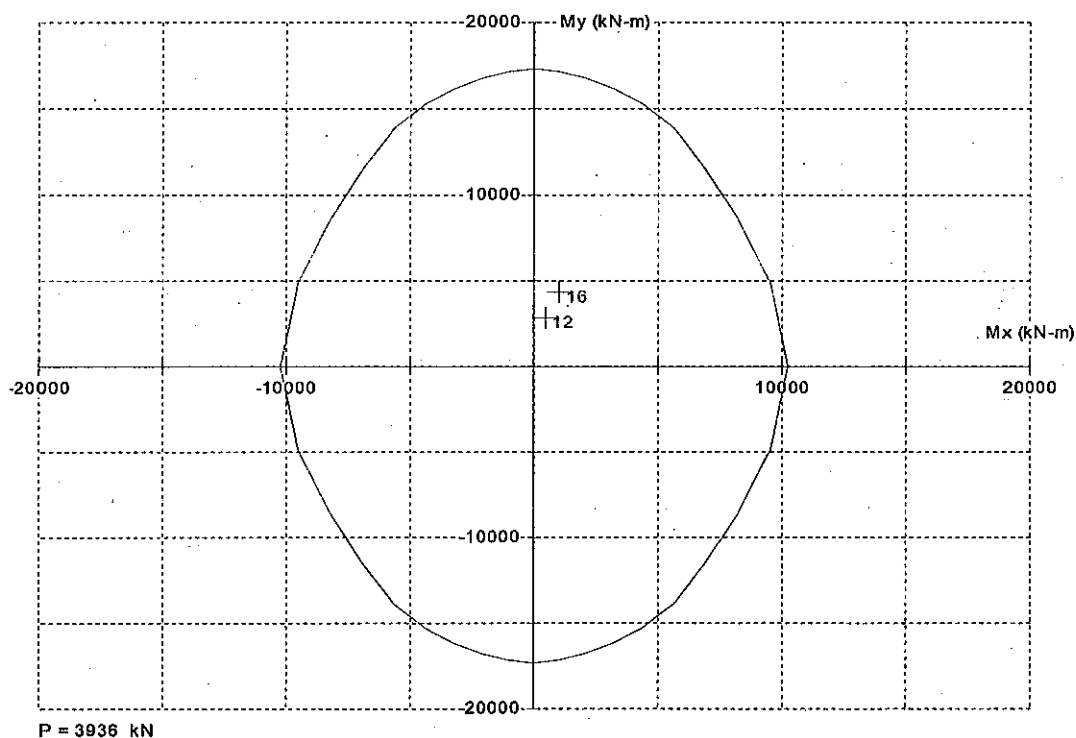
****In Transverse Direction**



****In Longitudinal Direction**



****In Both Direction**



3. Column Ties

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	1.818	m2
Tie diameter	Dtie	16	mm2
Cross section area of 1 tie	As-tr	0.0002	m2
Spacing of hoops	s	100	mm
Length of reinforcement tie in 1 hoop	Ltie	5.00	m
Ratio of ties reinf. in one hoop/ volume of conc. core one pitch spacing			
$ps = As-tr \cdot Ltie / (Ac \cdot spacing)$	ps	0.0056	
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	0.0038	OK
			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 $A_{sh} \geq 0.30 \cdot s \cdot h_c \cdot f_c / f_y \cdot [A_g / A_c - 1] = \text{Req1}$			
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"	1	
Number of cross tie	nt_x	2	ties
Total cross-sectional area of tie reinf.	Ash_x	0.0004	m2
Core dimension of tied column	hc_x	1.00	m
Rectangular hoop reinforcement shall satisfy	Req1_x	0.0003	m2
	Req2_x	0.0009	m2
	Conclude		OK
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	1.90	m
Rectangular hoop reinforcement shall satisfy	Req1_y	0.0005	m2
	Req2_y	0.0017	m2
	Conclude		OK
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	2.00	m
1/6 of clear height of column	L2	1.02	m
or 450mm	L3	0.45	m
Chosen value: $L = \max(L1, L2, L3)$	L	2.00	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	1.00	m
or 380mm	L5	0.38	m
Chosen value: $L_e = \max(L4, L5)$	Le	1.00	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	1820	1630	kN
Required shear capacity $V_n = V_u / \phi_v$	V_n	1820	1630	kN
Determine concrete shear capacity				
Minimum shear reinforcement will provided in cross section				
Therefore	β	2.0	2.0	
	θ	45.0	45.0	
Cross section equivalent height	h	0.88	1.60	m
width	b	1.60	0.88	m
$d = h - \text{cover} - d_{1x}$	d	0.79	1.51	m
$d_v = \max(0.72 \cdot h; 0.9 \cdot d)$	d_v	0.71	1.36	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	1040	1090	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	780	540	kN
In this case $V_c > V_n$ so shear reinforcement is no need				
Stirrup diameter	D_s	16	16	
Number of stirrup legs / cross section	ns	8	8	

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$V_s = V_n - V_c$	V_s	780	540	kN
In this case $V_c > V_n$ so shear reinforcement is no need				
Stirrup diameter	D_s	16	16	
Number of stirrup legs / cross section	ns	8	8	
Shear legs area	A_v	0.0016	0.0016	m ²
Angle of inclination of shear reinf. to long. axis	α	90	90	deg
$V_s = A_v \cdot f_y \cdot d_v \cdot (\cot \theta + \cot \alpha) \cdot \sin \alpha / s$	$s \leq$	0.59	1.63	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	0.0002	0.0001	m ²
Check maximum shear reinforcement spacing requirement		OK	OK	
$F = 0.1 \cdot f_c \cdot b_v \cdot d_v$	F	3430	3597	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}	0.57	0.60	m
Interface shear transfer				S.5.8.4
Area of concrete engaged in shear transfer	A_{cv}	2.020		m ²
Area of shear reinforcement crossing the shear plane	A_{vf}	0.058		m ²
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				
$V_n = c \cdot A_{cv} + \mu \cdot A_{vf} \cdot f_y$	V_n	24483		kN
$V_n \leq 0.2 \cdot f_c \cdot A_{cv}$	$V_n \leq$	12120		kN
$V_n \leq 5.5 \cdot A_{cv} \cdot (1\text{MPa})$	$V_n \leq$	11110		kN
Norminal shear resistance	V_n	11110		kN
Factor for shear friction		1.0		
Factored shear resistance	V_r	11110		kN
Horizontal force at bottom of pier column	V_u	1829		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	3	3	
Crack width parameter	Z	17500	17500	N/mm
Flexural moment	M_s	-1121	-1185	kNm
Axial thrust at service limit state	N_s	5091	5091	kN
Cross section equivalent	height	0.88	1.60	m
	width	1.60	0.88	m
Concrete thickness from tension fiber to tension reinf.	d_c	0.05	0.05	m
Concrete thickness from compression fiber to tension reinf.	d	0.79	1.51	kN
Number of rebars	N	12	9	bars
Area of rebars	A_s	0.0096	0.0072	m ²
Area of concrete assumed to participate with reinf.				
$A = 2 \cdot d_c \cdot b / N$	A	0.0133	0.0098	m ²
	f_{sa}	200	222	MPa
	$0.6 \cdot f_y$	240	240	MPa
Min ($f_{sa}, 0.6 \cdot f_y$) = f_{s1}	f_{s1}	200	222	MPa
$e = M_s / N_s + d - h/2$	e	0.13	0.48	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	27	67	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	0.031	0.070	mm
Where	β	0.174	0.174	

	DA NANG - QUANG NGAI EXPRESSWAY PROJECT	Item.	Eng.	Date	Sign.
	FO09 - Bridge	Design	-		
	DETAIL DESIGN	Check	-		
	Pier Design - P1	Revise	-		

D.PILECAP DESIGN

I. PILECAP DATA

1.Load Combinations at Bottom of Pilecap

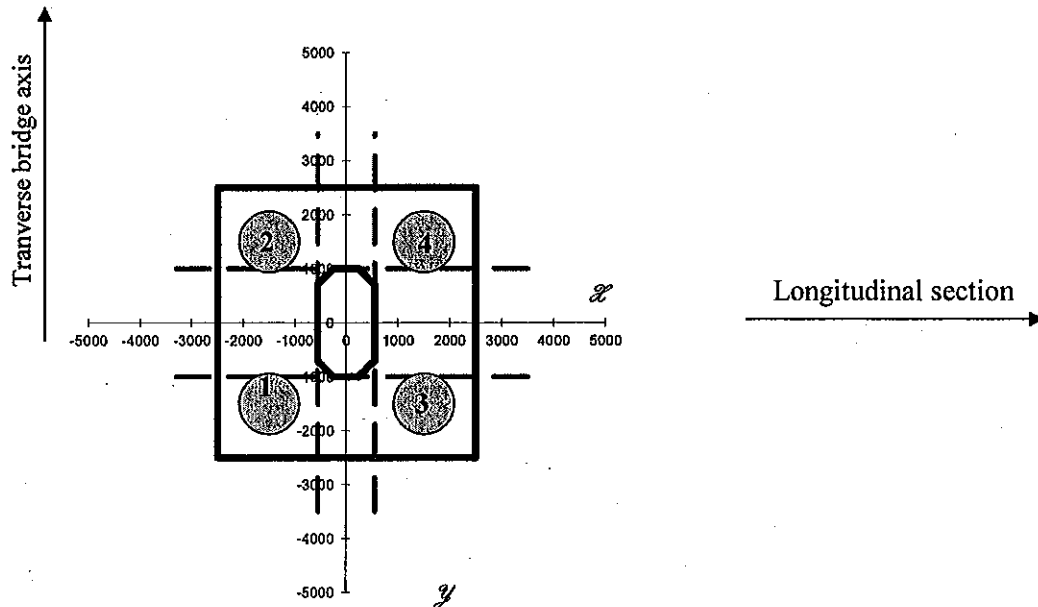
No	Combinations	Sign	F _v (kN)	Longitudinal		Transvesal	
				F _{Hx} (kN)	My (kN•m)	F _{Hy} (kN)	Mx (kN•m)
1	Strength I-A	Str1a	10222	-268	-1510	97	-893
2	Strength I-B	Str1b	6451	-250	-2306	147	-1800
3	Strength II-A	Str2a	8085	44	240	122	-824
4	Strength II-B	Str2b	5246	44	240	122	-824
5	Strength III-A	Str3a	9014	-207	-1847	180	-1876
6	Strength III-B	Str3b	6895	-221	-1234	141	-1177
7	Service I	Ser1	7094	-160	-1403	141	-1457
8	Service III	Ser3	7119	91	952	-26	366
9	Extreme 1a	EQT	8523	68	649	-52	3020
10	Extreme 1b	EQT	5883	-71	-659	42	-3037
11	Extreme 1c	EQL	8477	-71	-4070	42	-514
12	Extreme 1d	EQL	5929	68	4060	-52	497
13	Extreme 3a	CTT	8523	148	1062	-1631	6792
14	Extreme 3b	CTT	5883	-152	-1072	1621	-6809
15	Extreme 3c	CTL	8477	-1820	-7784	183	-1354
16	Extreme 3d	CTL	5929	1817	7774	-193	1337

2. PileCap 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. Pilecap dimensions - piles arrangement

Pilecap dimensions	Longitudinal	Bd	5.00	m	
	Transverse	Bn	5.00	m	
	Height	Hb	2.00	m	
Distance from edge to	transverse section	ex	1.50	m	
	longitudinal section	ey	1.95	m	
Soil on top of pilecap	"1:consider", "0:not consider"	1	hsoil	1.10	m



3. Piles Reactions refer to annex

II. PILECAP DESIGN

1. One-way Shear capacity Check

S.5.8

Critical shear section for one-way shear is located at distance d_v from face of equivalent square column.			
Estimated distance between internal flexural force components d_v , we may take $d_v = 0.9 \cdot d_e$			
$d_e = H - \text{cover} - dx1$	d_e	1.81	m
	d_v	1.63	m

Shear force at critical section; soil on pilecap can be ignored

Considering of bouyancy "1:yes" "0:no"

Transverse section	Left side			Right side			
Reactions of piles	Comb	Pile No	Reaction	Comb	Pile No	Reaction	
	1	1	2925	1	3	2349	kN
	Strength I-A	2	2762	Strength I-A	4	2186	kN
Selfweight of pilecap							
Load factor	1.25			1.25			
bouyancy	1		24			24	kN
Total shear force at section			5710			4559	

Longitudinal section	Upper side			Lower side			
Reactions of piles	Comb	Pile No	Reaction	Comb	Pile No	Reaction	
	1	1	2925	9	2	2397	kN
	Strength I-A	3	2349	Extreme 1a	4	2613	kN
Selfweight of pilecap							
Load factor	1.25			1.25			
bouyancy	1		-59			-59	kN
Total shear force at section			5214			4950	

One-way Shear Design

Direction		Long.- X	Trans.-Y	Unit
Shear resistance factors	ϕ_v	0.9	0.9	
Factored shear force in longitudinal	V_u	5214	5710	kN
Required shear capacity $V_n = V_u / \phi_v$	V_n	5794	6345	kN
Determine concrete shear capacity				
Minimum shear reinforcement will provided in cross section				
Therefore	β	2.0	2.0	
	θ	45.0	45.0	
Cross section	height	h	2.00	2.00 m
	width	b	5.00	5.00 m
$d = h - \text{cover} - d_{lx}$	d	1.81	1.81	m
$d_v = \max(0.72 \cdot h; 0.9 \cdot d)$	d_v	1.63	1.63	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	7422	7422	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				
Stirrup diameter	D_s	16	16	
Number of stirrup legs / cross section	ns	9	9	
Shear legs area	A_v	0.0018	0.0018	m ²
Angle of inclination of shear reinf. to long. axis	α	90	90	deg
Stirrup spacing used	s	0.60	0.60	m
Check maximum shear reinforcement spacing requirement		OK	OK	
$F = 0.1 \cdot f_c \cdot b_v \cdot d_v$	F	24489	24489	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	0.60	m

2.Two-way Shear capacity Design

S.5.13.3.6.3

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

Two-way shear is evaluated on a perimeter located $dv/2$ away from the face of the actual pier column.The same dimension $dv/2$ is used to check two-way shear for a corner pile.**Column v.s Pilecap**

Pier Column dimensions	Longitudinal axis	td	1.10	m
	Transverse axis	tn	2.00	m
Perimeter of two-way shear				
$b0 = (td+tn)*2 + 4*dv$		b0	11.44	m
Compressive strength of pilecap concrete		fc	30	Mpa
Yield strength of rebar		fy	400	Mpa
Section with transverse reinforcement				
Nominal shear resistance shall be taken as				
$Vn = Vc + Vs \leq 0.504.\sqrt{fc}. b0 . dv = Va$				
$Vc = 0.166 . \sqrt{fc}. b0 . dv$				
$Vs = Av . fy . dv / s$				
Shear resistance of concrete		Vc	16983	kN
Assumed stirrup diameter		Ds	16	mm
Number of stirrup legs / cross section		ns	9	
Shear legs area		Av	0.0018	m2
Stirrup spacing used		s	600	mm
Shear resistance of reinforcement		Vs	1979	kN
		Va	51563	kN
		Vn	18962	kN
Maximum reaction at bottom of column		Vu	8088	kN
Resistance factor for shear		ϕ_v	0.9	
Factored shear resistance		$\phi_v * Vn$	17066	kN
Punching shear check			OK	

Conner pile v.s Pilecap

Pile diameter	D	1.00	m
Radius of critical section for two-way shear $Rco = D/2 + dv/2$	Rco	1.32	m
Distance from pile center of conner pile to edge of pilecap	a1	1.00	m
Perimeter of two-way shear			
$b0 = 2*a1 + 1/4*2*pi()*Rco$	b0	4.07	m
Compressive strength of pilecap concrete	fc	30	Mpa
Yield strength of rebar	fy	400	Mpa
Section with transverse reinforcement			
Nominal shear resistance shall be taken as			
$Vn = Vc + Vs \leq 0.504.\sqrt{fc}. b0 . dv = Va$			
$Vc = 0.166 . \sqrt{fc}. b0 . dv$			
$Vs = Av . fy . dv / s$			
Shear resistance of concrete	Vc	6038	kN
Assumed stirrup diameter	Ds	16	mm
Number of stirrup legs / cross section	ns	8	
Shear legs area	Av	0.0016	m2
Stirrup spacing used	s	600	mm
Shear resistance of reinforcement	Vs	1759	kN
	Va	18332	kN
	Vn	7797	kN
Maximum reaction at bottom of column	Vu	3873	kN
Resistance factor for shear	ϕ_v	0.9	
Factored shear resistance	$\phi_v * Vn$	7017	kN
Punching shear check			OK

2.Bending Moment design - Longitudinal section

e0: distance from piles to edge									
e: distance from reaction to section									
$M^* = N.e - M2 - Q3.Hb/2$									
Section	Long.	Comb.	1	N	Q3	M2	e0	e=ey-e0	M*
	Lower		Strength I-A	(kN)	(kN)	(kNm)	(m)	(m)	(kN.m)
Pile number			1	2925	-25	-101	1.0	0.95	2904
			3	2349	-25	-101	1.0	0.95	2357
Distance - ey			1.95						
Self. of pilecap		1.25	1	-286		-279			-279
Soil on pilecap		1.30	1	-84		-82			-82
Reaction from column									
				Q	H				M
Sum				4902	-49				4900
Section	Long.	Comb.	7	N	Q3	M2	e0	e=ey-e0	M*
	Lower		Service I	(kN)	(kN)	(kNm)	(m)	(m)	(kN.m)
Pile number			1	2150	-34	-154	1.0	0.95	2231
			3	1678	-34	-154	1.0	0.95	1782
Distance - ey			1.95						
Self. of pilecap		1.00	1	-286		-279			-279
Soil on pilecap		1.00	1	-84		-82			-82
Reaction from column									
				Q	H				M
Sum				3457	-69				3652
Section	Long.	Comb.	15	N	Q3	M2	e0	e=ey-e0	M*
	Upper		Extreme 3c	(kN)	(kN)	(kNm)	(m)	(m)	(kN.m)
Pile number			1	3873	-47	-174	1.0	0.95	3899
			3	585	-47	-174	1.0	0.95	776
Distance - ey			1.95						
Self. of pilecap		1.25	1	-286		-279			-279
Soil on pilecap		1.30	1	-84		-82			-82
Reaction from column									
				Q	H				M
Sum				4087	-93				4314

M* = N.e + M2 + Q3.Hb/2									
Section	Long.	Comb.	1	N	Q3	M2	e0	e=ey-e0	M*
	Lower		Strength I-A	(kN)	(kN)	(kNm)	(m)	(m)	(kN.m)
Pile number			2	2762	-25	-101	1.0	0.95	2498
			4	2186	-25	-101	1.0	0.95	1951
Distance - ey			1.95						
Self. of pilecap		1.25	1	-286		-279			-279
Soil on pilecap		1.30	1	-84		-82			-82
Reaction from column									
				Q	H				M
Sum				4578	-49				4088
Section	Long.	Comb.	7	N	Q3	M2	e0	e=ey-e0	M*
	Lower		Service I	(kN)	(kN)	(kNm)	(m)	(m)	(kN.m)
Pile number			2	1868	-34	-154	1.0	0.95	1587
			4	1396	-34	-154	1.0	0.95	1138
Distance - ey			1.95						
Self. of pilecap		1.00	1	-286		-279			-279
Soil on pilecap		1.00	1	-84		-82			-82
Reaction from column									
				Q	H				M
Sum				2893	-69				2363
Section	Long.	Comb.	16	N	Q3	M2	e0	e=ey-e0	M*
	Upper		Extreme 3d	(kN)	(kN)	(kNm)	(m)	(m)	(kN.m)
Pile number			2	-54	49	178	1.0	0.95	175
			4	3224	49	178	1.0	0.95	3290
Distance - ey			1.95						
Self. of pilecap		0.90	1	-286		-279			-279
Soil on pilecap		0.90	1	-84		-82			-82
Reaction from column									
				Q	H				M
Sum				2799	98				3104

3.Bending Moment design - Transversal section

e0: distance from piles to edge									
e: distance from reaction to section									
$M^* = N.e - M3 + Q2.Hb/2$									
Section	Trans.	Comb.	1	N	Q2	M3	e0	e=ey-e0	M*
	Lower		Strength I-A	(kN)	(kN)	(kNm)	(m)	(m)	(kN.m)
Pile number			1	2925	66	-54	1.0	0.50	1583
			2	2762	66	-54	1.0	0.50	1502
Distance - ey			1.50						
Self. of pilecap		1.25	1	-286		-215			-215
Soil on pilecap		1.30	1	-84		-63			-63
Reaction from column									
				Q	H				M
Sum				5316	132				2806
Section	Trans.	Comb.	7	N	Q2	M3	e0	e=ey-e0	M*
	Lower		Service I	(kN)	(kN)	(kNm)	(m)	(m)	(kN.m)
Pile number			1	2150	39	-4	1.0	0.50	1118
			2	1868	39	-4	1.0	0.50	977
Distance - ey			1.50						
Self. of pilecap		1.00	1	-286		-215			-215
Soil on pilecap		1.00	1	-84		-63			-63
Reaction from column									
				Q	H				M
Sum				3648	78				1817
Section	Trans.	Comb.	15	N	Q2	M3	e0	e=ey-e0	M*
	Upper		Extreme 3c	(kN)	(kN)	(kNm)	(m)	(m)	(kN.m)
Pile number			1	3873	456	-521	1.0	0.50	2914
			2	3653	456	-521	1.0	0.50	2804
Distance - ey			1.50						
Self. of pilecap		1.25	1	-286		-215			-215
Soil on pilecap		1.30	1	-84		-63			-63
Reaction from column									
				Q	H				M
Sum				7155	912				5439
$M^* = N.e + M3 - Q2.Hb/2$									

Section	Trans.	Comb.	1	N	Q2	M3	e0	e=ey-e0	M*
	Lower		Strength I-A	(kN)	(kN)	(kNm)	(m)	(m)	(kN.m)
Pile number			3	2349	66	-54	1.0	0.50	1054
			4	2186	66	-54	1.0	0.50	973
Distance - ey			1.50						
Self. of pilecap	1.25		1	-286		-215			-215
Soil on pilecap	1.30		1	-84		-63			-63
Reaction from column									
				Q	H				M
Sum				4164	132				1748

Section	Trans.	Comb.	7	N	Q2	M3	e0	e=ey-e0	M*
	Lower		Service I	(kN)	(kN)	(kNm)	(m)	(m)	(kN.m)
Pile number			3	1678	39	-4	1.0	0.50	796
			4	1396	39	-4	1.0	0.50	655
Distance - ey			1.50						
Self. of pilecap	1.00		1	-286		-215			-215
Soil on pilecap	1.00		1	-84		-63			-63
Reaction from column									
				Q	H				M
Sum				2703	78				1173

Section	Trans.	Comb.	16	N	Q2	M3	e0	e=ey-e0	M*
	Upper		Extreme 3d	(kN)	(kN)	(kNm)	(m)	(m)	(kN.m)
Pile number			3	3017	-454	516	1.0	0.50	2478
			4	3224	-454	516	1.0	0.50	2582
Distance - ey			1.50						
Self. of pilecap	0.90		1	-286		-215			-215
Soil on pilecap	0.90		1	-84		-63			-63
Reaction from column									
				Q	H				M
Sum				5870	-907				4783

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INTERNAL FORCES AT SECTION

FLEXURAL MOMENT CHECKING

File:EX A2 - F009 - Pier P1.xls,xls-Sheet:PileCap-LongX

a	Depth of equivalent stress block	m	0.018	0.018	0.018		
dc	Corresp. effective depth from extreme comp. fiber to centroid of tensile force in the tensile reinf.	m	1.788	1.788	1.915		
Mn	Nominal resistance	kNm	11211	11211	11509		
Mr	Factored resistance	kNm	10090	11211	11509		
Mu	Flexural moment	kNm	4900	3652	4314		
(5.7.3.2)	Flexural moment Checking		OK	OK	OK		
	Limits for reinforcement						
c/de	Maximum reinforcement		0.01	0.01	0.01		
	Maximum reinforcement Checking	≤ 0.42	OK	OK	OK		
1.2*Mcrr	Cracking moment	kNm	6978	6978	6978		
(5.7.3.3.2)	Checking $M_r \geq \min(1.2M_{cr}, 1.33M_u)$		OK	OK	OK		
(5.7.3.4)	Control of cracking by distr. of reinf for RC member- Check?		No	Yes	No		
	Existing condition for structure	1,2 or 3	3	3	3		
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.063	0.063	0.063		
Z	Crack width parameter	N/mm	17500	17500	17500		
A	Area of concr. with same centroid as tens. Reinf	m ²	0.019	0.019	0.019		
f _{sa}	Value	Mpa	165	165	165		
0.6*f _y		Mpa	240	240	240		
	Tensile stress in reinf Min(f _{sa} , 0.6f _y)	Mpa	165	165	165		
x	Dist. From compression fiber to centroid	m	-	0.263	-		
J _d	Arm	m	-	1.70	-		
I _{cr}	Moment of inertia of the cracked section	m ⁴	-	0.296	-		
f _s	Tensile stress in reinforcement $f_s = M_{sls} / (A_s * J.d)$	Mpa	-	133	-		
	Checking for control cracking $f_s \leq f_{sa}$		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		
	Distribution on sides 7 D16	m ²	0.00141	0.00141	0.00141		
	Required Spacing not larger than	m	0.45	0.45	0.45		
	Checking		OK	OK	OK		
SHEAR AND TORSION CHECKING							
β	Factor indicating diag. cracked concr. to tension		1.9	2.1	2.0		
θ	Angle of inclination of diagonal compressive	degree	41.79	38.94	40.40		
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90		
b _v	Effective web width as minimum web width - in dv	m	5.000	5.000	5.000		
d _v	Effective shear depth	m	1.778	1.778	1.906		
	(dc - a/2)	m	1.778	1.778	1.906		
s	Spacing of stirrups	m	0.600	0.600	0.600		
n _{cat}	Amount of bars in spacing S	bars	9	9	9		
A _v	Shear reinf area in spacing S	m ²	0.0018	0.0018	0.0018		
θ	Assume	degree	41.79	38.95	40.39		
v	Shear stress in concrete	kN/m ²	613	389	429		
f _{po}	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116		
e _s	Strain in tensile reinforcement		1.70E-03	1.29E-03	1.44E-03		
	if $e_x < 0$, multiple with reduce factor		-	-	-		
	Strain checking	$\leq 2.00E-3$	Ok	Ok	Ok		
v/f _c	Ratio of shear stress and f _c		0.020	0.013	0.014		
β	Final value		1.9	2.1	2.0		
θ	Final value	degree	41.79	38.94	40.40		
V _c	Nominal shear resistance provided by tensile stresses in the concrete	kN	7517	8350	8594		
V _s	Shear resistance provided by shear reinforcement	kN	2412	2668	2714		
V _p	Component in the direction of the applied shear of the effective P.S	kN	0	0	0		
V _{n1}	$V_{n1} = V_c + V_s + V_p$	kN	9929	11017	11309		
V _{n2}	V _{n2}	kN	66688	66688	71469		
V _n	Nominal shear resistance $V_n = \min(V_{n1}, V_{n2})$	kN	9929	11017	11309		
V _r	Factored shear resistance	kN	8936	11017	11309		
V _u	Shear	kN	4902	3457	4087		
(5.8.2.7)	Shear checking		OK	OK	OK		
	Region requiring transverse reinf Checking		Need	No need	No need		
	$0.1 * f_c * b_v * d_v$	kN	26675	26675	28588		
	S _{max}	m	0.60	0.60	0.60		
	Maximum spacing S _{max}		OK	-	-		

DA NANG - QUANG NGAI EXPRESSWAY PROJECT			Item.	Eng.	Date.	Sign.
FO09 - Bridge			Design			
DETAIL DESIGN			Check			
CHECK PILE CAP - TRANSVERSAL DIRECTION			Revise			

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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	
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	

The diagram illustrates the cross-section of a pile cap. Key dimensions and areas are labeled: H is the total height; b is the top width; b_w is the web width; $d's$ is the effective depth to the center of compression reinforcement; d_{1x} is the distance to the center of tension reinforcement; d_s is the distance to the center of tension reinforcement; d'_{ps} is the effective depth to the center of compression prestressing steel; d_{1xp} is the distance to the center of tension prestressing steel; d_{ps} is the distance to the center of tension prestressing steel. Reinforcement areas are denoted as $A's$ (compression), A'_{ps} (compression prestressing), A_s (tension), and A_{ps} (tension prestressing). The ratio A_s/f_s is also indicated.

Sign	Parameters	Unit	Section - Transverse section			
			Lower	Lower	Lower	
INTERNAL FORCES AT SECTION						
	Combination		Strength	Service	Extreme	
Qu	Shear	kN	5316	3648	7155	
Mu	Flexural Moment	kNm	2806	1817	5439	
Nu	Axial load	kN	0	0	0	
Tu	Torsional Moment	kNm	0	0	0	
FLEXURAL MOMENT CHECKING						
H	Section height	m	2.000	2.000	2.000	
d's	Dis. From comp. fiber to centroid of comp. Reinf	m	0.085	0.085	0.214	
d1x	Dis. From tens. fiber to centroid of tension Reinf	m	0.214	0.214	0.085	
	Cover to reinf	m	0.050	0.050	0.050	
ds	Dis. From comp. fiber to centroid of tension Reinf	m	1.786	1.786	1.915	
d'ps	Dis. From comp. fiber to centroid of comp. prestressing steel	m	0.000	0.000	0.000	
d1xp	Dis. From tens. fiber to centroid of tension prestressing steel	m	0.000	0.000	0.000	
dps	Dis. From comp. fiber to centroid of tension prestressing steel	m	2.000	2.000	2.000	
b	Width of the compression face of member	m	5.000	5.000	5.000	
bw	Web width or diameter of a circular section	m	5.000	5.000	5.000	
hf	Compression flange depth	m	0.000	0.000	0.000	
Iz	Moment of inertia of section	m4	3.333	3.333	3.333	
A _{mc}	Section area	m2	10.000	10.000	10.000	
	Steel choice					
A _s	Tension Reinforcement	Number	bars	33	33	33
		Diameter	mm	28	28	28
		Area	m2	0.02033	0.02033	0.02033
A' _s	Compression Reinforcement	Number	bars	33	33	33
		Diameter	mm	20	20	20
		Area	m2	0.01036	0.01036	0.01036
A' _c	Shear reinforcement	Number	bars	9	9	9
		Diameter	mm	16	16	16
		Area	m2	0.00182	0.00182	0.00182
φ	Resistance factors for flexure	5.5.4.2	0.90	1.00	1.00	
φ _v	Resistance factors for shear		0.90	1.00	1.00	
φ _n	Resistance factors for axial force		1.00	1.00	1.00	
β ₁	Stress block factor		0.836	0.836	0.836	
c	Dis. Between centroid and top fiber	m	0.037	0.037	0.037	
	For T section behavior	m	0.037	0.037	0.037	
	For rectangular section behavior	m	0.037	0.037	0.037	
f _{pe}	Effective stress in the prestressing steel after losses	Mpa	1116	1116	1116	
f _{ps}	Aver. stress in pres. steel at the time for which the nominal resistance	Mpa	1850	1850	1850	
k	Factor depends on type of P.S, Low relaxation strand k = 0.28		0.28	0.28	0.28	

a	Depth of equivalent stress block	m	0.031	0.031	0.031		
de	Corresp. effective depth from extreme comp. fiber to centroid of tensile force in the tensile reinf.	m	1.786	1.786	1.915		
Mn	Nominal resistance	kNm	14108	14108	14622		
Mr	Factored resistance	kNm	12697	14108	14622		
Mu	Flexural moment	kNm	2806	1817	5439		
(5.7.3.2)	Flexural moment Checking		OK	OK	OK		
	Limits for reinforcement						
c/de	Maximum reinforcement		0.02	0.02	0.02		
	Maximum reinforcement Checking	≤ 0.42	OK	OK	OK		
1.2*Mer	Craking moment	kNm	7033	7033	7033		
(5.7.3.3.2)	Checking $M_r \geq \min(1.2M_{er}, 1.33M_u)$		OK	OK	OK		
(5.7.3.4)	Control of cracking by distr. of reinf for RC member- Check?		No	Yes	No		
	Existing condition for structure	1,2 or 3	3	3	3		
de	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.064	0.064	0.064		
Z	Crack width parameter	N/mm	17500	17500	17500		
A	Area of concr. with same centroid as tens. Reinf	m ²	0.019	0.019	0.019		
f _{sa}	Value	Mpa	163	163	163		
0.6*f _y	Tensil stress in reinf Min(f _{sa} ,0.6f _y)	Mpa	240	240	240		
x	Dist. From compression fiber to centroid	m	-	0.292	-		
J.d	Arm	m	-	1.689	-		
I _{cr}	Moment of inertia of the cracked section	m ⁴	-	0.362	-		
f _s	Tensile stress in reinforcement $f_s = M_{sls} / (A_s * J.d)$	Mpa	-	53	-		
	Checking for control cracking $f_s \leq f_{sa}$		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		
	Distribution on sides 7 D16	m ²	0.00141	0.00141	0.00141		
	Required Spacing not larger than	m	0.45	0.45	0.45		
	Checking		OK	OK	OK		
SHEAR AND TORSION CHECKING							
β	Factor indicating diag. cracked concr. to tension		2.1	2.3	1.9		
θ	Angle of inclination of diagonal compressive	degree	38.21	34.78	41.76		
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90		
b _v	Effective web width as minimum web width - in dv	m	5.000	5.000	5.000		
d _v	Effective shear depth	m	1.770	1.770	1.899		
	(d _e - a/2)	m	1.770	1.770	1.899		
s	Spacing of stirrups	m	0.600	0.600	0.600		
n _{cat}	Amount of bars in spacing S	bars	9	9	9		
A _v	Shear reinf area in spacing S	m ²	0.0018	0.0018	0.0018		
θ	Assume	degree	38.20	34.79	41.76		
v	Shear stress in concrete	kN/m ²	667	412	753		
f _{po}	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116		
e _s	Strain in tensile reinforcement		1.22E-03	8.98E-04	1.69E-03		
	if e _s <0, multiple with reduce factor		-	-	-		
	Strain checking	$\leq 2.00E-3$	Ok	Ok	Ok		
v/f _c	Ratio of shear stress and f _c		0.022	0.014	0.025		
β	Final value		2.1	2.3	1.9		
θ	Final value	degree	38.21	34.78	41.76		
V _c	Nominal shear resistance provided by tensile stresses in the concrete	kN	8477	9203	8042		
V _s	Shear resistance provided by shear reinforcement	kN	2726	3090	2578		
V _p	Component in the direction of the applied shear of the effective P.S	kN	0	0	0		
V _{n1}	$V_{n1} = V_c + V_s + V_p$	kN	11203	12293	10620		
V _{n2}	V _{n2}	kN	66389	66389	71226		
V _n	Nominal shear resistance $V_n = \min(V_{n1}, V_{n2})$	kN	11203	12293	10620		
V _r	Factored shear resistance	kN	10082	12293	10620		
V _u	Shear	kN	5316	3648	7155		
(5.8.2.7)	Shear checking		OK	OK	OK		
	Region requiring transverse reinf Checking		Need	No need	Need		
	$0.1 * f_c * b_v * d_v$	kN	26556	26556	28491		
	S _{max}	m	0.60	0.60	0.60		
	Maximum spacing S _{max}		OK	-	OK		

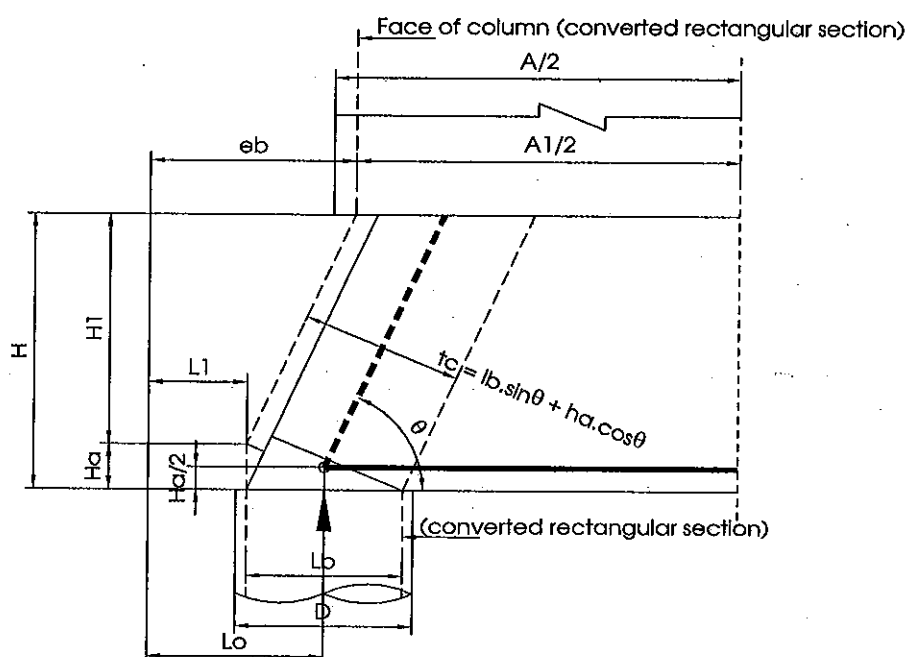
	DA NANG - QUANG NGAI EXPRESS WAY PROJECT	Item.	Eng.	Date	Sign.
	FO09 - Bridge	Design	-		
	DETAIL DESIGN	Check	-		
	Pier Design - P1	Revise	-		

e1.PILECAP DESIGN - STRUT AND TIE MODEL

I. PILECAP DATA

1. PileCap Material

Normal concrete			
Compressive strength at 28 days age	f_c	30	MPa
Concrete elastic modulus	E_c	27691	MPa
Reinforcement TCVN1651-2008; CBV-400			
Yield strength	f_y	400	MPa
Reinforcement elastic modulus	E_s	200,000	MPa



2. Strut and tie - Checking

Direction		Long.- X	Trans.-Y	Unit
Height of pilecap	H_b	2.00	2.00	m
Pilecap width in direction	W_{pc}	5.00	5.00	m
Pile diameter	D	1.00	1.00	m
Distance from C.L of outer pile to face of pilecap	L_0	1.00	1.00	m
Column width in direction	W	2.00	1.10	m
Distance from design section to face of pilecap	e_0	1.50	1.95	m
Distance from C.L of outer pile to design section	Arm	0.50	0.95	m
Pilecap should be design as strut and tie model? Yes or No		Yes	Yes	
Converted rectangular of pile	L_b	0.89	0.89	m
Column section area	A_{col}	2.02	2.02	m ²
Converted rectangular of column in direction	W_b	1.84	1.01	m
Dist. from converted rect. of column to face of pilecap	eb	1.58	2.00	m
Dist. from converted rect. of pile to face of pilecap	L_1	0.56	0.56	m

Dist. from C.L of tension rebars to bottom of pilecap	Ha/2	0.167	0.183	m
Distance from top of tension tie to to of pilecap	H1	1.67	1.63	m
Inclined angle of compression strut	θ	58.40	48.65	deg
Reaction of outer piles row	R_EXT	4087	7155	kN
	R_ULS	4902	5316	kN
	R_SLS	3457	3648	kN
Compression strut Checking				
Strut dimension Thickness	tc	0.93	0.91	m
Width: Wstr = np . Lb	Wcs	1.77	1.77	m
Number of pile in outer row	np	2.00	2.00	piles
Area of strut section: Acs = tc.Wcs	Acs	1.65	1.61	m ²
Resistance factor ULS	ϕ	1.00	1.00	
Resistance factor SLS	ϕ	0.70	0.70	
Compression force in strut - C = T / cos θ EXT		5756	9531	kN
Compression force in strut - C = T / cos θ ULS		4798	7082	kN
Strain in tension tie	ϵ_s	0.0009	0.0015	
$\epsilon_l = \epsilon_s + (\epsilon_s + 0.002) \cdot \cot\theta$	ϵ_l	0.0027	0.0047	
Limiting compressive stress	f _{cu}	23.72	18.82	MPa
Rebars area of compressive strut	A _{ss}	0	0	m ²
Resistance of compressive strut: Cr = $\phi \cdot [f_{cu} \cdot A_{cs} + f_y \cdot A_{ss}]$ EXT	Cr	39094	30253	kN
Resistance of compressive strut: Cr = $\phi \cdot [f_{cu} \cdot A_{cs} + f_y \cdot A_{ss}]$ ULS	Cr	27366	21177	kN
Conclusion		Ok	Ok	
Tension tie Checking				
Tension tie force - T = R / tan θ	T_EXT	2514	6297	kN
	T_ULS	3016	4679	kN
Resistance factor EXT	ϕ	1.00	1.00	
Resistance factor ULS	ϕ	0.90	0.90	
Resistance of tension Tie: Pr = $\phi \cdot f_y \cdot A_s$				
Area of required rebars for tie EXT	A _s \geq	7540	15743	mm ²
Area of required rebars for tie ULS	A _s \geq	6984	12996	mm ²
Triad number of rebars				
Long. - X 33 bars D25 @150 A _s		16203		mm ²
Trans. - Y 33 bars D28 @150 A _s			20328	mm ²
Conclusion		Ok	Ok	
Node region Checking				
Limiting compressive stress	f _{cu}	16	16	MPa
Compressive stress at node	P/A _{cs}	2.97	4.45	kN
Conclusion		Ok	Ok	

SPACE PILE FOUNDATION ANALYSIS PROGRAM
Turbo BASIC

PROJECT: : FO09-P1

INITIA DATA

Kn = 0.00 Ax = 5.00 By = 5.00 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	-27.00	10.00	1042.00	91.00	-154.00	0.00
2	-25.00	15.00	658.00	183.00	-235.00	0.00
3	4.00	12.00	824.00	84.00	24.00	0.00
4	4.00	12.00	535.00	84.00	24.00	0.00
5	-21.00	18.00	919.00	191.00	-188.00	0.00
6	-23.00	14.00	703.00	120.00	-126.00	0.00
7	-16.00	14.00	723.00	149.00	-143.00	0.00
8	9.00	-3.00	726.00	-37.00	97.00	0.00
9	7.00	-5.00	869.00	-308.00	66.00	0.00
10	-7.00	4.00	600.00	310.00	-67.00	0.00
11	-7.00	4.00	864.00	52.00	-415.00	0.00
12	7.00	-5.00	604.00	-51.00	414.00	0.00
13	15.00	-166.00	869.00	-692.00	108.00	0.00
14	-15.00	165.00	600.00	694.00	-109.00	0.00
15	-186.00	19.00	864.00	138.00	-793.00	0.00
16	185.00	-20.00	604.00	-136.00	792.00	0.00

PROPERTIES OF PILES

PILE	Lo	H	Bpx	Bpy	A	B	Cday	Fo	Io	Po	Co	Ct
1	0.00	25.00	1.620	1.620	1.00	0.000	0.000	0.785	0.049	0	100000	50000
2						n t						
3						n t						
4						n t						

PILE COORD.

PILE	X	Y	Phi	Xi
1	-1.50	-1.50	0.000	0.00
2	-1.50	1.50	0.000	0.00
3	1.50	-1.50	0.000	0.00
4	1.50	1.50	0.000	0.00

DISPLACEMENTS

COMB.	X	Y	Z	Fix	Fiy	Fiz
1	-0.00240	0.00006	0.006254	0.000132	-0.000470	0.000000
2	-0.00274	-0.00017	0.003949	0.000295	-0.000626	0.000000
3	0.00036	0.00021	0.004946	0.000107	0.000072	0.000000
4	0.00036	0.00021	0.003211	0.000107	0.000072	0.000000
5	-0.00225	-0.00004	0.005516	0.000295	-0.000507	0.000000
6	-0.00201	0.00013	0.004220	0.000170	-0.000389	0.000000

7	-0.00171	-0.00004	0.004340	0.000230	-0.000385	0.000000
8	0.00106	0.00004	0.004358	-0.000060	0.000251	0.000000
9	0.00077	0.00144	0.005216	-0.000609	0.000176	0.000000
10	-0.00077	-0.00151	0.003601	0.000619	-0.000178	0.000000
11	-0.00272	-0.00006	0.005186	0.000085	-0.000898	0.000000
12	0.00272	0.00000	0.003625	-0.000078	0.000896	0.000000
13	0.00146	-0.00558	0.005216	-0.000503	0.000307	0.000000
14	-0.00146	0.00551	0.003601	0.000513	-0.000310	0.000000
15	-0.01503	0.00031	0.005186	0.000179	-0.002682	0.000000
16	0.01497	-0.00038	0.003625	-0.000170	0.002674	0.000000

FORCES ON PILES

PILE	COMB.	N	Q2	Q3	M1	M2	M3
1	1	298.12	6.75	-2.50	0.000	-10.346	-5.530
	2	222.04	6.25	-3.75	0.000	-18.126	0.060
	3	208.16	-1.00	-3.00	0.000	-11.003	0.753
	4	135.91	-1.00	-3.00	0.000	-11.003	0.753
	5	279.80	5.25	-4.50	0.000	-20.148	-0.477
	6	210.70	5.75	-3.50	0.000	-14.070	-5.001
	7	219.20	4.00	-3.50	0.000	-15.696	-0.376
	8	162.08	-2.25	0.75	0.000	3.648	-0.717
	9	168.19	-1.75	1.25	0.000	19.888	-0.028
	10	199.79	1.75	-1.00	0.000	-19.475	0.084
	11	277.43	1.75	-1.00	0.000	-5.013	19.591
	12	90.18	-1.75	1.25	0.000	5.481	-19.535
	13	166.62	-3.75	41.50	0.000	125.862	1.814
	14	201.37	3.75	-41.25	0.000	-125.450	-1.758
	15	394.77	46.50	-4.75	0.000	-17.702	-53.110
	16	-26.68	-46.25	5.00	0.000	18.114	52.641
2	1	281.58	6.75	-2.50	0.000	-10.346	-5.530
	2	185.21	6.25	-3.75	0.000	-18.126	0.060
	3	194.83	-1.00	-3.00	0.000	-11.003	0.753
	4	122.58	-1.00	-3.00	0.000	-11.003	0.753
	5	243.00	5.25	-4.50	0.000	-20.148	-0.477
	6	189.46	5.75	-3.50	0.000	-14.070	-5.001
	7	190.46	4.00	-3.50	0.000	-15.696	-0.376
	8	169.55	-2.25	0.75	0.000	3.648	-0.717
	9	244.34	-1.75	1.25	0.000	19.888	-0.028
	10	122.43	1.75	-1.00	0.000	-19.475	0.084
	11	266.78	1.75	-1.00	0.000	-5.013	19.591
	12	99.87	-1.75	1.25	0.000	5.481	-19.535
	13	229.47	-3.75	41.50	0.000	125.862	1.814
	14	137.31	3.75	-41.25	0.000	-125.450	-1.758
	15	372.37	46.50	-4.75	0.000	-17.702	-53.110
	16	-5.50	-46.25	5.00	0.000	18.114	52.641
3	1	239.42	6.75	-2.50	0.000	-10.346	-5.530
	2	143.79	6.25	-3.75	0.000	-18.126	0.060
	3	217.17	-1.00	-3.00	0.000	-11.003	0.753
	4	144.92	-1.00	-3.00	0.000	-11.003	0.753
	5	216.50	5.25	-4.50	0.000	-20.148	-0.477
	6	162.04	5.75	-3.50	0.000	-14.070	-5.001
	7	171.04	4.00	-3.50	0.000	-15.696	-0.376
	8	193.45	-2.25	0.75	0.000	3.648	-0.717
	9	190.16	-1.75	1.25	0.000	19.888	-0.028
	10	177.57	1.75	-1.00	0.000	-19.475	0.084
	11	165.22	1.75	-1.00	0.000	-5.013	19.591
	12	202.13	-1.75	1.25	0.000	5.481	-19.535
	13	205.03	-3.75	41.50	0.000	125.862	1.814
	14	162.69	3.75	-41.25	0.000	-125.450	-1.758
	15	59.63	46.50	-4.75	0.000	-17.702	-53.110
	16	307.50	-46.25	5.00	0.000	18.114	52.641
4	1	222.88	6.75	-2.50	0.000	-10.346	-5.530
	2	106.96	6.25	-3.75	0.000	-18.126	0.060

3	203.84	-1.00	-3.00	0.000	-11.003	0.753
4	131.59	-1.00	-3.00	0.000	-11.003	0.753
5	179.70	5.25	-4.50	0.000	-20.148	-0.477
6	140.80	5.75	-3.50	0.000	-14.070	-5.001
7	142.30	4.00	-3.50	0.000	-15.696	-0.376
8	200.92	-2.25	0.75	0.000	3.648	-0.717
9	266.31	-1.75	1.25	0.000	19.888	-0.028
10	100.21	1.75	-1.00	0.000	-19.475	0.084
11	154.57	1.75	-1.00	0.000	-5.013	19.591
12	211.82	-1.75	1.25	0.000	5.481	-19.535
13	267.88	-3.75	41.50	0.000	125.862	1.814
14	98.63	3.75	-41.25	0.000	-125.450	-1.758
15	37.23	46.50	-4.75	0.000	-17.702	-53.110
16	328.68	-46.25	5.00	0.000	18.114	52.641

SUMMARY OF FORCES

	PILE	COMB.	N	Q2	Q3	M1	M2	M3
Nmin	1	16	-26.68	-46.25	5.00	0.000	18.114	52.641
Nmax	1	15	394.77	46.50	-4.75	0.000	-17.702	-53.110
Q2max	1	15	394.77	46.50	-4.75	0.000	-17.702	-53.110
Q3max	1	13	166.62	-3.75	41.50	0.000	125.862	1.814
M1max	1	1	298.12	6.75	-2.50	0.000	-10.346	-5.530
M2max	1	13	166.62	-3.75	41.50	0.000	125.862	1.814
M3max	1	15	394.77	46.50	-4.75	0.000	-17.702	-53.110

CHECKING CALCULATI IN COMPARISON WITH INITIA LOAD MATRIX

1	-27.00	10.00	1042.00	91.00	-154.00	0.00
2	-25.00	15.00	658.00	183.00	-235.00	0.00
3	4.00	12.00	824.00	84.00	24.00	0.00
4	4.00	12.00	535.00	84.00	24.00	0.00
5	-21.00	18.00	919.00	191.00	-188.00	0.00
6	-23.00	14.00	703.00	120.00	-126.00	0.00
7	-16.00	14.00	723.00	149.00	-143.00	0.00
8	9.00	-3.00	726.00	-37.00	97.00	0.00
9	7.00	-5.00	869.00	-308.00	66.00	0.00
10	-7.00	4.00	600.00	310.00	-67.00	0.00
11	-7.00	4.00	864.00	52.00	-415.00	0.00
12	7.00	-5.00	604.00	-51.00	414.00	0.00
13	15.00	-166.00	869.00	-692.00	108.00	0.00
14	-15.00	165.00	600.00	694.00	-109.00	0.00
15	-186.00	19.00	864.00	138.00	-793.00	0.00
16	185.00	-20.00	604.00	-136.00	792.00	0.00

	DA NANG - QUANG NGAI EXPRESSWAY PROJECT FO09 - Bridge DETAIL DESIGN Pier Design - P1	Item.	Eng.	Date	Sign.
		Design	-		
		Check	-		
		Revise	-		

E.BORED PILE DESIGN

I. BORED PILE DATA

1. Load Combinations at top of bored pile

No	Combinations	Sign	F _V (kN)	Longitudinal		Transvesal	
				F _{IX} (kN)	My (kN•m)	F _{HY} (kN)	Mx (kN•m)
1	Strength1a		2925	-66	54	25	101
2	Service1		2150	-39	4	34	154
3	Extreme2c		3873	-456	521	47	174
4	Extreme2d		-262	454	-516	-49	-178
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	1.00	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

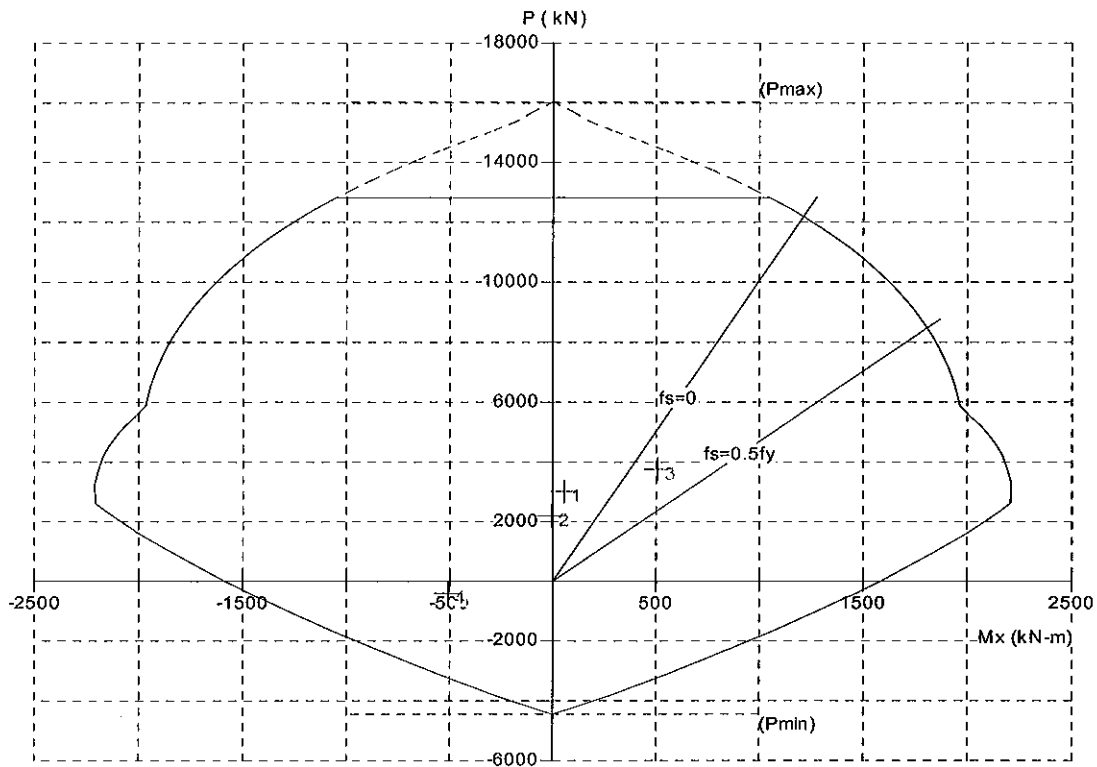
II. PILE DESIGN

1. Limit of Reinforcement

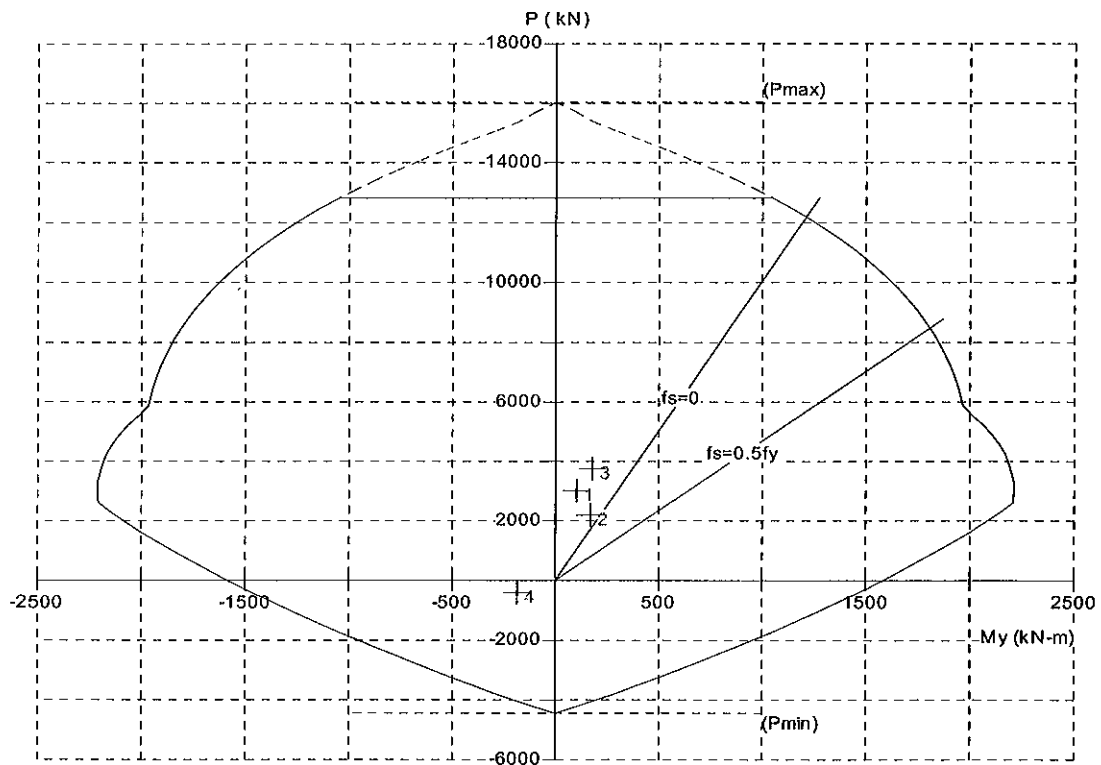
S.5.7.4.2

Minimum area of longitudinal reinforcement in column			
As.fy / (Ag . f _c) >= 0.135	As ≥	0.008	m ²
As / Ag >= 0.01	As ≥	0.008	m ²
Maximum area of longitudinal reinforcement in column			
As / Ag <= 0.08	As ≤	0.063	m ²
Trial Rebars:	Ok As	0.012	m ²
11ayers x 20 = 20 bars	D28 @150 As1	0.012	m ²

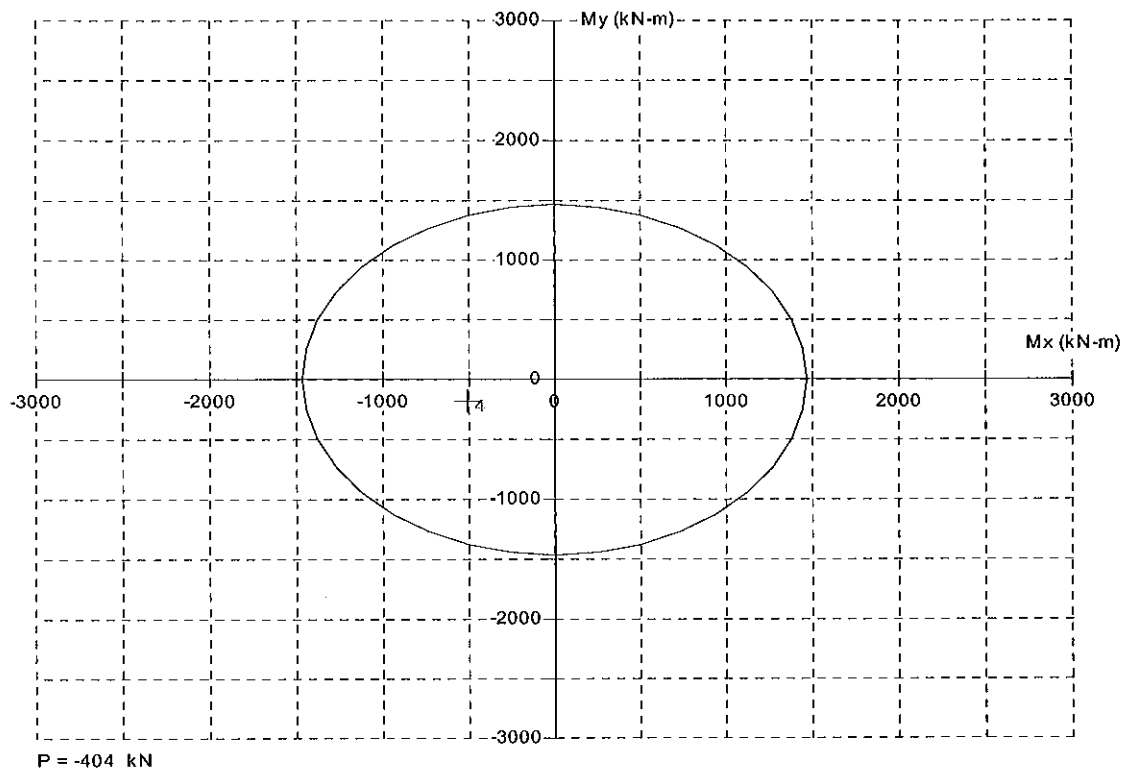
**In Transverse Direction



**In Longitudinal Direction



****In Both Direction**



3. Column Ties

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

Bridge is in seismic zone	Sz	1	
Area of concrete core measured out-to-out of ties	Ac	0.622	m2
Tie diameter	Dtie	14	mm2
Cross section area of 1 tie	As-tr	0.0002	m2
Spacing of hoops	s	75	mm
Length of reinforcement tie in 1 hoop	Ltie	2.78	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	0.0090	
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	0.0089	OK
Transverse Reinforcement for Confinement at Plastic Hinges			S.5.10.11.4.1.d
For a circular column "1:applied", "2:Not applied"		1	
$\rho_s \geq 0.12 \cdot f_c / f_y = \text{Req2}$	Req2	0.0090	OK
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	456	kN
Required shear capacity $V_n = V_u / \phi_v$	Vn	456	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
$d_v = 0.9 \cdot d_e$ $d_e = D/2 + D_r/\pi()$			
Diameter of the circle passing through the centers of the long. reinf.	Dr	0.79	m
	de	0.75	m
	dv	0.68	m
$V_c = 0.083 \cdot \beta \cdot \sqrt{f_c} \cdot b_v \cdot d_v$	Vc	616	kN

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CALCULATION SHEET
ABUTMENT A1

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	ABUTMENT A1	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

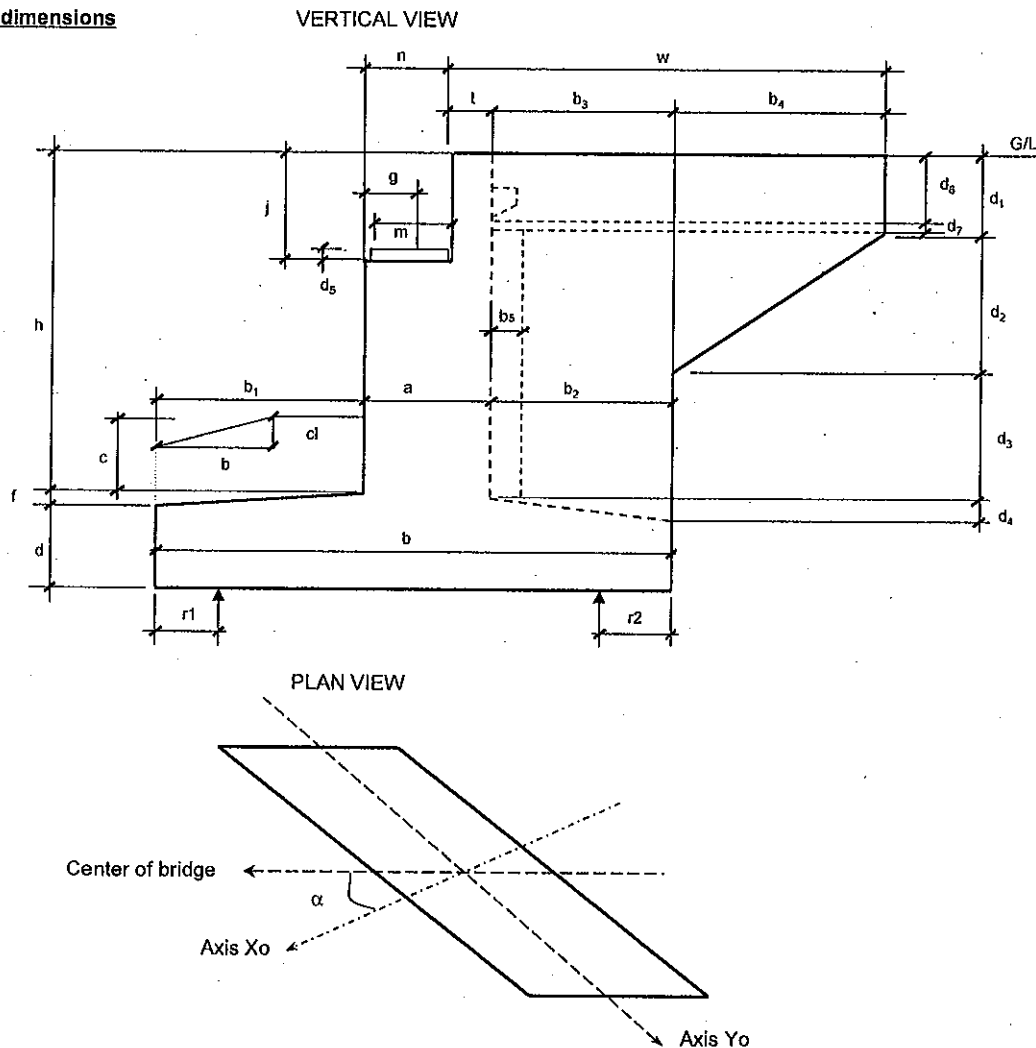
Input :

Level Table(at center of abutment)

Level of top of headwall	HTwL	17.218	m
Level of top of bearing	BTL	15.951	m
Level of top of stem abutment	HTL	15.686	m
Level of top of footing	FTL	11.000	m
Level of bottom of footing	FBL	9.000	m
Ground level	GL	12.150	m
Highest water level	HWL		m
Skew angle	α	20.00	deg

I.Loads from substructure

Abutment dimensions



Material Unit Weights

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

γ_c	=	24.5 kN/m ³
γ_s	=	17.7 kN/m ³
γ_{sbo}	=	7.8 kN/m ³

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ABUTMENT DIMENSIONS (IN METRES)					
Item	Symbol	Value	Item	Symbol	Value
Height of stem	h	6.218	Horizontal Dimension	b ₃	1.941
Footing Width	b	5.000	Horizontal Dimension	b ₄	1.859
Stem Width	a	1.600	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	1.859
Width of stem at bearing	n	1.200	Vertical Dimension	d ₃	2.359
Ballast Wall Height	j	1.532	Vertical Dimension	d ₄	0.000
Ballast Wall Thickness	t	0.400	Vertical Dimension	d ₅	0.265
Wingwall Length	w	4.200	Vertical Dimension	d ₆	0.000
Soil Cover at Toe	c	1.150	Vertical Dimension	d ₇	0.000
Girder Reaction	g	0.650	With of bearing pad	m	0.970
Distance to cl of pile	r1	1.000	Wingwall Thickness	u1	0.500
Horizontal Dimension	b ₁	1.600	Wingwall Thickness	u2	0.500
Horizontal Dimension	b ₂	1.800	Distance to cl of pile	r2	1.000

Slope front of abutment

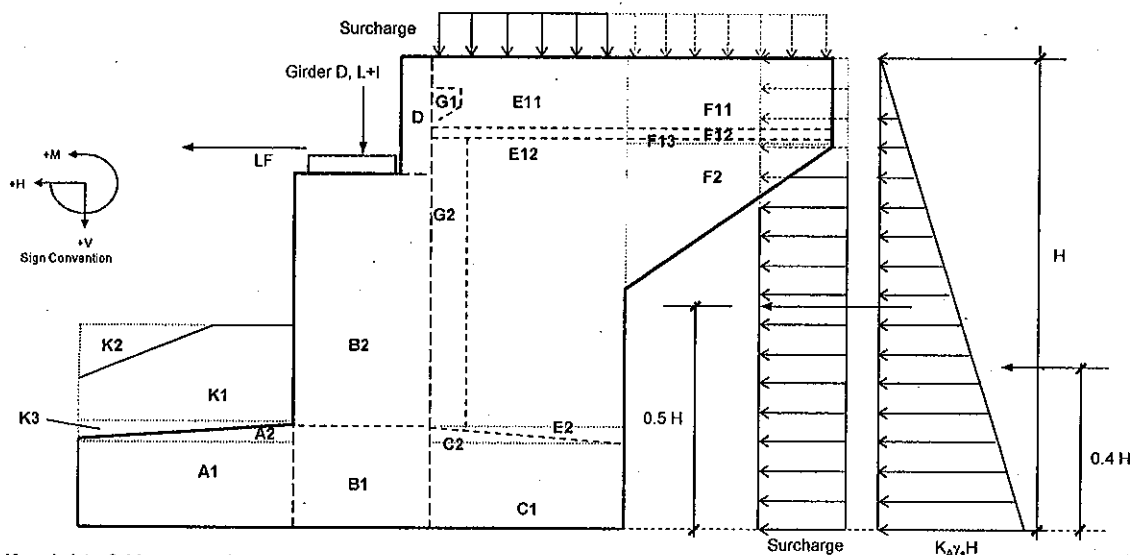
Width of Abutment

Width of abutment (inclined direction)

Height of Abutment

Distance from CG of footing to edge of Abutment

cos (α)	=	0.94
cl	=	0.00 m
bl	=	0.00 m
L	=	7.240 m
Ltr	=	7.705 m
Ht	=	8.22 m
b/2	=	2.50 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	3.200	7.705	604	0.800	1.700	1027
Section A2	-	7.705	-	1.067	1.433	-
Section B1	3.200	7.705	604	2.400	0.100	60
Section B2	7.498	6.707	1232	2.400	0.100	123
Section C1	3.600	7.705	680	4.100	-1.600	-1087
Section C2	-	7.705	-	3.800	-1.300	-
Section D	0.613	6.707	101	3.000	-0.500	-50
Section E11	3.882	1.000	95	4.171	-1.671	-159
Section E12	8.187	1.000	201	4.171	-1.671	-335
Section E2	-	1.000	-	4.494	-1.994	-
Section F11	-	1.000	-	6.071	-3.571	-
Section F12	-	1.000	-	5.100	-2.600	-
Section F13	3.718	1.000	91	6.071	-3.571	-325
Section F2	1.728	1.000	42	5.761	-3.261	-138
Section G1	0.135	6.705	168	3.350	-0.850	-143
Section G2	0.125	12.436	38	3.450	-0.950	-36
Bearing seats (w1seat= 0.97m)	0.257	1.940	38	2.250	0.250	9
Curbs +Handrail on Abutment	0.50	4.200	56	4.900	-2.400	-134
Total SW of Abutment (DC)			3948			-1187

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.

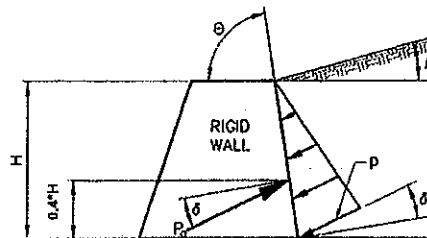
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		Revise			

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	11.19	5.707	1128	4.100	-1.600	-1805
Section E2	-	5.707	-	4.400	-1.900	-
Section E3	-0.33	1.000	-6	5.071	-2.571	15
Section K1	1.840	7.705	250	0.800	1.700	426
Section K2	-	7.705	-	-	2.500	-
Section K3	-	7.705	-	0.533	1.967	-
Total Earth on Footing			1372			-1364

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 Ka (assume wall move or deflect sufficiently to reach minimum active conditions)

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

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

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 $0.4 H$

H	=	8.22 m
W	=	7.24 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	=	0.048 Mpa

E_a	=	1439 kN
M	=	4730 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=	8.22m	heq=	0.65 m

(Linear interpolation)

- Vertical force

ESv	=	159 kN
ev	=	-1.60 m
M	=	-254 kNm

- Horizontal force

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

ESh	=	227 kN
eh	=	4.11 m
M	=	934 kNm

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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 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 $\theta = \arctan(k_h / (1 - k_v))$ $\theta = 5.2 \text{ deg}$

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

• 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 $E_{AE} = 1624 \text{ kN}$

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

$M_{AE} = 4461 \text{ KNm}$

<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 Soil type II
- Site Coefficients. $S = 1.2$
- Elastic Seismic Response Coefficient $2.5A = 0.145$
- $C_{sm} = 1.2 \cdot A \cdot S / T_m^{2/3} \leq 2.5 \cdot A$ $C_{sm} = 0.111$
- Period of vibration of the fundamental mode $T_m = 2 \cdot \pi \cdot \sqrt{m/k}$ $T_m = 0.649 \text{ s}$

Description	Area (m ²)	Length (m)	Force (kN)	X ⁽¹⁾ (m)	Arm ⁽²⁾ (m)	Moment (kN·m)
Section A1	3.200	7.705	42	-	1.000	42
Section A2	-	7.705	-	-	2.000	-
Section B1	3.200	7.705	42	-	1.000	42
Section B2	7.498	6.707	137	-	4.343	596
Section C1	3.600	7.705	47	-	1.000	47
Section C2	-	7.705	-	-	2.000	-
Section D	0.613	6.707	11	-	7.452	84
Section E11	3.882	1.000	7	-	6.218	41
Section E12	8.187	1.000	14	-	3.109	-
Section E2	-	1.000	-	-	2.000	-
Section F11	-	1.000	-	-	6.218	-
Section F12	-	1.000	-	-	6.218	-
Section F13	3.718	1.000	6	-	5.218	-
Section F2	1.728	1.000	3	-	5.598	16
Section G1	0.135	6.705	2	-	5.505	8
Section G2	0.125	12.436	3	-	3.109	8
Total EQ of Abutment Selfweight			314			885

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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	=	2 lanes	
m	=	1.00	
BR	=	81 kN	Long. Axis
e	=	10.1 m	
Mlong	=	820 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

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

V	=	60 km/h	
V	=	16.7 m/s	
R	=	- m	
C	=	-	
CE	=	0.00 kN	
e	=	10.10 m	
Mtrans	=	0.00 KNm	Trans. Axis

8. Water Load (WA)

8.1. Buoyancy of Abutment

- Highest water Level

Description	Area (m ²)	Length (m)	Force (kN)	X ⁽¹⁾ (m)	Arm ⁽²⁾ (m)	Moment (kN·m)
Bouyancy on abutment						
Section A1	-	7.705	-	0.800	1.700	-
Section A2	-	7.705	-	-	2.500	-
Section B(B1,B2)	-	7.705	-	2.400	0.100	-
Section C1	-	7.705	-	4.100	-1.600	-
Section C2	-	7.705	-	-	2.500	-
Section E2	-	1.000	-	-	2.500	-
Section E1	-	1.000	-	4.171	-1.671	-
Section F2	-	1.000	-	0.021	2.479	-
Total Bouyancy						

8.2 Buoyancy of Earth on Abutment

Description	Area (m ²)	Length (m)	Force (kN)	X ⁽¹⁾ (m)	Arm ⁽²⁾ (m)	Moment (kN·m)
Bouyancy of earth on abutment						
Section E2	-	5.707	-	-	2.500	-
Section E1	-	5.707	-	4.100	-1.600	-
Section K2	-	7.705	-	-	2.500	-
Section K1	-	7.705	-	0.800	1.700	-
- Section K3	-	7.705	-	-	2.500	-
Total Bouyancy						

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SUPERSTRUCTURE LOADS

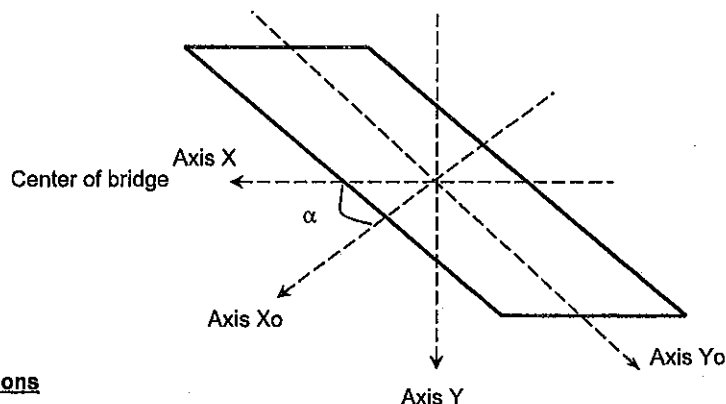
Bearing	Load combinations					
	Service-I		Strength-I		Extreme	
	N	H	N	H	N	H
Expansion bearing 1	1415.0	53.0	1927.0	69.0	1581.0	128.0
Expansion bearing 2	1415.0	53.0	1927.0	69.0	1581.0	128.0
Total	2830.0	106.0	3854.0	138.0	3162.0	256.0
Arm of level of horizontal force						
At top pilecap	4.95					
At bottom pilecap	6.95					
Arm of level of vertical force						
At centre of stem	0.15					
At bottom pilecap	0.250					

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	3854	0	578	138	683
Strength Str-IB	3854	0	578	138	683
Strength Str-IIIA	3854	0	578	138	683
Strength Str-IIIB	3854	0	578	138	683
Service Ser-I	2830	0	425	106	525
Extreme Ext-IA	3162	0	474	256	1267
Extreme Ext-IB	3162	0	474	256	1267

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	3854	0	964	138	959
Strength Str-IB	3854	0	964	138	959
Strength Str-IIIA	3854	0	964	138	959
Strength Str-IIIB	3854	0	964	138	959
Service Ser-I	2830	0	708	106	737
Extreme Ext-IA	3162	0	791	256	1779
Extreme Ext-IB	3162	0	791	256	1779

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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	4435				-491			
Soils on pilecap	EV	1989				-2207			
Horizontal Earth Pressure	EH			1531		5034			
Vertical Surcharge	LSv	247				-396			
Horizontal Surcharge	LSH			242		994			
Braking Force	BR			81		820			
Centrifugal Force	CE			-		-	-		-
Buoyancy of Abutment	WA	-				-			
Buoyancy of Earth on Abutment	WA	-				-			
Earthquake effects to Abutment	EQ			342		900	103		270
Earthquake effects to soil	E _{AE}			1728		4747			

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	8661	2863	6439	0	0
Strength Str-IB	6214	1944	4584	0	0
Strength Str-IIIA	8563	2733	5872	0	0
Strength Str-IIIB	6115	1814	4016	0	0
Service Ser-I	6671	2232	3754	0	0
Extreme Ext-IA	8352	2232	2762	103	270
Extreme Ext-IB	5905	2232	3927	103	270

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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	4842	0	3632	138	959
Strength Str-IB	4842	0	3632	138	959
Strength Str-IIIA	4842	0	3632	138	959
Strength Str-IIIB	4842	0	3632	138	959
Service Ser-I	3522	0	2642	106	737
Extreme Ext-IA	3660	0	2745	256	1779
Extreme Ext-IB	3660	0	2745	256	1779

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	13503	2863	10070	138	959
Strength Str-IB	11056	1944	8215	138	959
Strength Str-IIIA	13405	2733	9503	138	959
Strength Str-IIIB	10957	1814	7648	138	959
Service Ser-I	10193	2232	6395	106	737
Extreme Ext-IA	12012	2232	5507	359	2049
Extreme Ext-IB	9565	2232	6672	359	2049

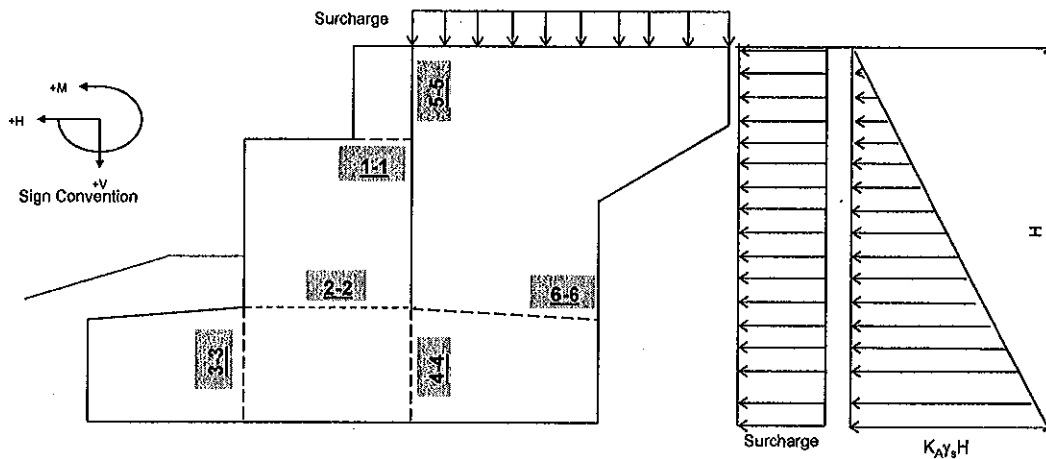
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ELEMENTS CHECKING

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 _n	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	268		-59		
Horizontal Earth Pressure		53	33		
Surcharge (horizontal)		117	90		
Horizontal Seismic Earth Pressure		60	31		
Abutment earthquake force		13	10	4	3

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	268	171	64	0	0
Strength Str-IA	335	285	133	0	0
Strength Str-IB	242	253	134	0	0
Extreme Ext-I	335	162	27	4	3

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
Horizontal Earth Pressure	EH	1.00	1.50	0.90	
Surcharge (Horizontal)	LS _n	1.00	1.75	1.75	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	1500		-102		
Horizontal Earth Pressure		877	2180		
Surcharge (Horizontal)		211	657		
Horizontal Selsmic Earth Pressure		989	2056		
Abutment earthquake force		150	391	45	117

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	4330	1088	3159	108	525
Strength Str-IA	5729	1685	4870	138	683
Strength Str-IB	5204	1159	3598	138	683
Extreme Ext-I	5037	1740	4150	301	1385

1.3 Section 3-3

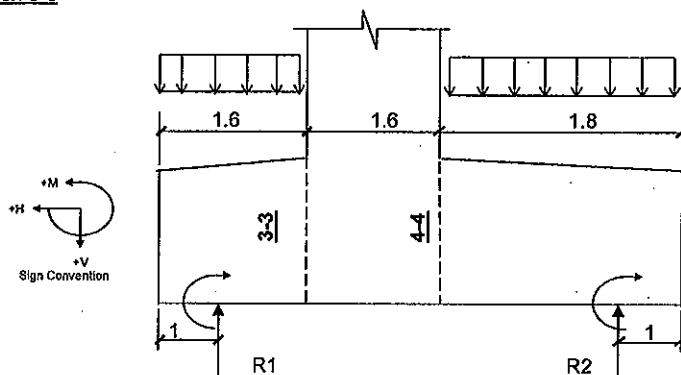


Table of Load Factors

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	-604		-483		
Vertical soil on foot at front side	-250		-200		
Reaction of piles					
Ser-I	5560	1	3018	-62	-206
Str-IA	9721	1711	8734	-102	-503
Str-IB	7529	1161	6475	-95	-396
Ext-I	7788	1321	7334	-222	-610

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	4705	1	2335	-62	-206
Strength Str-IA	8628	1711	7859	-102	-503
Strength Str-IB	6760	1161	5860	-95	-396
Extreme Ext-I	6695	1321	6459	-222	-610

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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	-1109		-1229		
Vertical soil on foot at behind side	-1122		-1010		
Surcharge(Vertical)	-159		-143		
Reaction of piles					
Ser-I	2750	-1	2415	-46	-148
Str-IA	1198	1153	-1009	-35	-244
Str-IB	1389	782	-215	-42	-206
Ext-I	2238	887	0	-131	-361

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	360	-1	34	-46	-148
Strength Str-IA	-1981	1153	-4159	-35	-244
Strength Str-IB	-897	782	-2481	-42	-206
Extreme Ext-I	-742	887	-2971	-131	-361

1.4 Section 5-5 & 6-6

Slope of triang pressure
Uniform horizontal pressure

$\tan \beta = 5.89$
 $U.p = 3.82 \text{ kN/m}^2$

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		74	193		
Strength Str-IA		116	303		

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				207	178
Strength Str-IA				320	278

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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
fpv	Yield strength of prestressing steel	Mpa	1670
Ep	Modulus of Elasticity	Mpa	197000
Cốt thép thường			
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	285	171	1088	1685	1740	
Mu	Flexural Moment	kNm	133	64	3159	4870	4150	
Nu	Axial load	kN	335	268	4330	5729	5037	
Tu	Torsional Moment	kNm	0	0	0	0	0	
FLEXURAL MOMENT CHECKING								
H	Section height	m	0.400	0.400	1.600	1.600	1.600	
d's	Dis. From comp. fiber to centroid of comp. Reinf	m	0.074	0.074	0.079	0.079	0.079	
d1x	Dis. From tens. fiber to centroid of tension Reinf	m	0.074	0.074	0.079	0.079	0.079	
	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	0.326	0.326	1.522	1.522	1.522	
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	0.400	0.400	1.600	1.600	1.600	
b	Width of the compression face of member	m	7.240	7.240	7.240	7.240	7.240	
bw	Web width or diameter of a circular section	m	7.240	7.240	7.240	7.240	7.240	
hf	Compression flange depth	m	0.000	0.000	0.000	0.000	0.000	
Iz	Moment of inertia of section	m4	0.039	0.039	2.471	2.471	2.471	
Amc	Section area	m2	2.896	2.896	11.584	11.584	11.584	
	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	0.00000
A'ps	Compression 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
As	Tension Reinforcement	Number	bars	48	42	42	42	
		Diameter	mm	16	16	20	20	20
		Area	m2	0.00970	0.00970	0.01319	0.01319	0.01319
A's	Compression Reinforcement	Number	bars	48	42	42	42	
		Diameter	mm	16	16	16	16	16
		Area	m2	0.00970	0.00970	0.00848	0.00848	0.00848
A'c	Shear reinforcement	Number	bars	12	12	12	12	
		Diameter	mm	14	14	14	14	14
		Area	m2	0.00181	0.00181	0.00181	0.00181	0.00181
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	0.012	0.012	0.012	
	For T section behavior	m	0.000	0.000	0.012	0.012	0.012	
	For rectangular section behavior	m	0.000	0.000	0.012	0.012	0.012	
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	1856	1856	1856	
k	Factor depends on type of P.S, Low relaxation strand k = 0.28		0.28	0.28	0.28	0.28	0.28	

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REINFORCEMENT CHECKING - HEAD AND STEM WALL							
a	Depth of equivalent stress block	m	0.000	0.000	0.010	0.010	0.010
de	Corresp. effective depth from extreme comp. fiber to centroid of tensile force in the tensile reinf.	m	0.326	0.326	1.522	1.522	1.522
Mn	Nominal resistance	kNm	977	977	7750	7750	7750
Mr	Factored resistance	kNm	880	977	7750	6975	7750
Mu	Flexural moment	kNm	133	64	3159	4870	4150
(5.7.3.2)	Flexural moment Checking		OK	OK	OK	OK	OK
	Limits for reinforcement						
c/de	Maximum reinforcement		0.00	0.00	0.01	0.01	0.01
	Maximum reinforcement Checking	<= 0.42	OK	OK	OK	OK	OK
1.2*Mc	Cracking moment	kNm	400	400	6445	6445	6445
(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	1	1	1	1
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.058	0.058	0.060	0.060	0.060
Z	Crack width parameter	N/mm	30000	30000	30000	30000	30000
A	Area of concr. with same centroid as tens. Reinf	m ²	0.017	0.017	0.021	0.021	0.021
fsa	Value	Mpa	299	299	279	279	279
0.6*fy		Mpa	240	240	240	240	240
	Tensile stress in reinf Min(fs, 0.6fy)	Mpa	240	240	240	240	240
x	Dist. From compression fiber to centroid	m	-	0.069	0.185	-	-
J.d	Arm	m	-	0.303	1.46	-	-
Icr	Moment of inertia of the cracked section	m ⁴	-	0.005	0.181	-	-
fs	Tensile stress in reinforcement $f_s = M_{sls} / (A_s * J.d)$	Mpa	-	22	164	-	-
	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	0.00036	0.00036	0.00123	0.00123	0.00123
	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

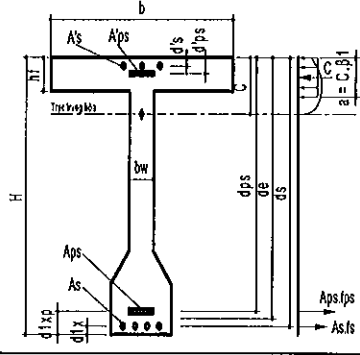
SHEAR AND TORSION CHECKING							
β	Factor indicating diag. cracked concr. to tension		3.5	4.0	3.1	2.4	2.4
θ	Angle of inclination of diagonal compressive	degree	28.52	27.00	28.69	31.46	31.08
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	90
bv	Effective web width as minimum web width - in dv	m	7.240	7.240	7.240	7.240	7.240
dv	Effective shear depth	m	0.326	0.326	1.516	1.516	1.516
	(de - a/2)	m	0.326	0.326	1.516	1.516	1.516
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	0.600
ncat	Amount of bars in spacing S	bars	12	12	12	12	12
Av	Shear reinf area in spacing S	m ²	0.0018	0.0018	0.0018	0.0018	0.0018
θ	Assume	degree	28.52	27.00	28.69	31.45	31.08
v	Shear stress in concrete	kN/m ²	134	72	99	170	158
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e _s	Strain in tensile reinforcement		2.59E-04	1.18E-04	3.46E-04	6.54E-04	6.30E-04
	if $e_s < 0$, multiple with reduce factor		-	-	-	-	-
	Strain checking	<= 2.00E-3	Ok	Ok	Ok	Ok	Ok
v/fc	Ratio of shear stress and fc		0.004	0.002	0.003	0.006	0.005
β	Final value		3.5	4.0	3.1	2.4	2.4
θ	Final value	degree	28.52	27.00	28.69	31.46	31.08
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	3707	4334	15543	12098	12165
Vs	Shear resistance provided by shear reinforcement	kN	725	773	3347	2994	3039
Vp	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0	0
Vn1	$V_{n1} = V_c + V_s + V_p$	kN	4432	5107	18890	15092	15204
Vn2	Vn2	kN	17702	17702	82341	82341	82341
Vn	Nominal shear resistance $V_n = \min(V_{n1}, V_{n2})$	kN	4432	5107	18890	15092	15204
Vr	Factored shear resistance	kN	3988	5107	18890	13583	15204
Vu	Shear	kN	285	171	1088	1685	1740
(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	0.0049	0.0049	0.0049	0.0049	0.0049
	Minimum shear reinforcement Checking		-	-	-	-	-
	$0.1 * f_c * b_v * d_v$	kN	7081	7081	32936	32936	32936
	Smax	m	0.26	0.26	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
fpy	Yield strength of prestressing steel	Mpa	1670
Ep	Modulus of Elasticity	Mpa	197000
Cốt thép thường			
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	4705	8628	6695	742	1981	
Mu	Flexural Moment	kNm	2335	7859	6459	2971	4159	
Nu	Axial load	kN	1	1711	1321	887	1153	
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	0.074	0.074	0.074	0.074	0.074	
d1x	Dis. From tens. fiber to centroid of tension Reinf	m	0.179	0.179	0.179	0.179	0.179	
	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	1.822	1.822	1.822	1.822	1.822	
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	7.240	7.240	7.240	7.240	7.240	
bw	Web width or diameter of a circular section	m	7.240	7.240	7.240	7.240	7.240	
h1f	Compression flange depth	m	0.000	0.000	0.000	0.000	0.000	
Iz	Moment of inertia of section	m4	4.827	4.827	4.827	4.827	4.827	
Amc	Section area	m2	14.480	14.480	14.480	14.480	14.480	
	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	0.00000
A'ps	Compression 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
As	Tension Reinforcement	Number	bars	48	48	48	48	
		Diameter	mm	25	25	25	20	20
		Area	m2	0.02357	0.02357	0.02357	0.01507	0.01507
A's	Compression Reinforcement	Number	bars	48	48	48	48	
		Diameter	mm	20	20	20	25	25
		Area	m2	0.01507	0.01507	0.01507	0.02357	0.02357
A'c	Shear reinforcement	Number	bars	12	12	12	12	
		Diameter	mm	16	16	16	16	16
		Area	m2	0.00242	0.00242	0.00242	0.00242	0.00242
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	0.022	0.022	0.022	-0.022	-0.022	
	For T section behavior	m	0.022	0.022	0.022	-0.022	-0.022	
	For rectangular section behavior	m	0.022	0.022	0.022	-0.022	-0.022	
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	1854	1854	1854	1866	1866	
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 FO09 BRIDGE DETAIL DESIGN ABUTMENT A1	Item.	Eng.	Date.	Sign.
		Design			
		Check			
		Revise			

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REINFORCEMENT CHECKING - PILECAP SECTION							
a	Depth of equivalent stress block	m	0.018	0.018	0.018	-0.018	-0.018
de	Corresp. effective depth from extreme comp. fiber to centroid of tensile force in the tensile reinf.	m	1.822	1.822	1.822	1.822	1.822
Mn	Nominal resistance	kNm	16694	16694	16694	10253	10253
Mr	Factored resistance	kNm	16694	15025	16694	10253	9227
Mu	Flexural moment	kNm	2335	7859	6459	2971	4159
(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	-0.01	-0.01
	Maximum reinforcement Checking	≤ 0.42	OK	OK	OK	OK	OK
1.2*Mcr	Craking moment	kNm	10104	10104	10104	9884	9884
(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	0.063	0.063	0.063	0.060	0.060
Z	Crack width parameter	N/mm	17500	17500	17500	17500	17500
A	Area of concr. with same centroid as tens. Reinf	m ²	0.019	0.019	0.019	0.018	0.018
f _{sa}	Value	Mpa	166	166	166	170	170
0.6*f _y		Mpa	240	240	240	240	240
	Tensile stress in reinf Min(f _{sa} , 0.6f _y)	Mpa	166	166	166	170	170
x	Dist. From compression fiber to centroid	m	0.266	-	-	-	-
J.d	Arm	m	1.733	-	-	-	-
I _{cr}	Moment of inertia of the cracked section	m ⁴	0.448	-	-	-	-
f _s	Tensile stress in reinforcement $f_s = M_{sl} / (A_s * J.d)$	Mpa	57	-	-	-	-
	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 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	2.5	2.2
θ	Angle of inclination of diagonal compressive	degree	35.63	42.02	40.48	30.59	36.13
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	7.240	7.240	7.240	7.240	7.240
d _v	Effective shear depth	m	1.812	1.812	1.812	1.831	1.831
	(d _c - a/2)	m	1.812	1.812	1.812	1.831	1.831
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	0.600
n _{cat}	Amount of bars in spacing S	bars	12	12	12	12	12
A _v	Shear reinf area in spacing S	m ²	0.0024	0.0024	0.0024	0.0024	0.0024
θ	Assume	degree	35.63	42.00	40.47	30.60	36.12
v	Shear stress in concrete	kN/m ²	359	731	510	33	166
f _{po}	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e _s	Strain in tensile reinforcement		9.70E-04	1.75E-03	1.45E-03	6.00E-04	1.01E-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		0.012	0.024	0.017	0.001	0.006
β	Final value		2.2	1.8	2.0	2.5	2.2
θ	Final value	degree	35.63	42.02	40.48	30.59	36.13
V _c	Nominal shear resistance provided by tensile stresses in the concrete	kN	13403	10932	11804	14788	13394
V _s	Shear resistance provided by shear reinforcement	kN	4085	3250	3431	5004	4053
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	17489	14183	15235	19792	17447
V _{n2}	V _{n2}	kN	98408	98408	98408	99407	99407
V _n	Nominal shear resistance V _n = min(V _{n1} , V _{n2})	kN	17489	14183	15235	19792	17447
V _r	Factored shear resistance	kN	17489	12764	15235	19792	15703
V _u	Shear	kN	4705	8628	6695	742	1981
(5.8.2.7)	Shear checking		OK	OK	OK	OK	OK

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SECRET

[illegible]

INTERNAL FORCES AT SECTION

FLEXURAL MOMENT CHECKING

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Da Nang Quang Ngai Expressway project			Item.	Eng.	Date.	Sign.
FO09 BRIDGE			Design			
DETAIL DESIGN			Check			
ABUTMENT A1			Revise			

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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	0.426	0.426	0.422	0.422	0.422
Mn	Nominal resistance	kNm	349	349	345	345	345
Mr	Factored resistance	kNm	349	314	345	310	310
Mu	Flexural moment	kNm	193	303	178	278	278
(5.7.3.2)	Flexural moment Checking		OK	OK	OK	OK	OK
	Limits for reinforcement						
c/de	Maximum reinforcement		0.05	0.05	0.05	0.05	0.05
	Maximum reinforcement Checking	≤ 0.42	OK	OK	OK	OK	OK
r min	Minimum reinforcement		0.46%	0.46%	0.46%	0.46%	0.46%
	Minimum reinforcement Checking for RC	0.23%	OK	OK	OK	OK	OK
1.2*Mcr	Cracking moment	kNm	90	90	90	90	90
(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	0.061	0.061	0.061	0.061	0.061
Z	Crack width parameter	N/mm	30000	30000	30000	30000	30000
A	Area of concr. with same centroid as tens. Reinf	m ²	0.020	0.020	0.020	0.020	0.020
f _{sa}	Value	Mpa	279	279	279	279	279
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	0.102	-	0.101	-	-
J.d	Arm	m	0.392	-	0.388	-	-
I _{cr}	Moment of inertia of the cracked section	m ⁴	0.002	-	0.002	-	-
f _s	Tensile stress in reinforcement f _s = M _s / (A _s *J.d)	Mpa	216	-	202	-	-
	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	2.1	1.8	1.8
θ	Angle of inclination of diagonal compressive	degree	37.20	41.93	38.33	42.44	42.44
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
dv	Effective shear depth	m	0.418	0.418	0.413	0.413	0.413
	(d _e - a/2)	m	0.418	0.418	0.413	0.413	0.413
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	0.0004	0.0004	0.0004	0.0004	0.0004
β	Assume		2.0	2.0	2.0	2.0	2.0
θ	Assume	degree	37.19	41.93	38.32	42.44	42.44
v	Shear stress in concrete	kN/m ²	177	308	501	862	862
f _{po}	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e _s	Strain in tensile reinforcement		1.12E-03	1.73E-03	1.23E-03	1.86E-03	1.86E-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		0.006	0.010	0.017	0.029	0.029
β	Final value		2.2	1.8	2.1	1.8	1.8
θ	Final value	degree	37.20	41.93	38.33	42.44	42.44
V _c	Nominal shear resistance provided by tensile stresses in the concrete	kN	411	350	394	335	335
V _s	Shear resistance provided by shear reinforcement	kN	148	125	141	122	122
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	559	475	535	457	457
V _{n2}	V _{n2}	kN	3132	3132	3098	3098	3098
V _n	Nominal shear resistance V _n = min(V _{n1} , V _{n2})	kN	559	475	535	457	457
V _r	Factored shear resistance	kN	559	428	535	411	411
V _u	Shear	kN	74	116	207	320	320
(5.8.2.7)	Shear checking		OK	OK	OK	OK	OK

SPACE PILE FOUNDATION ANALYSIS PROGRAM
Turbo BASIC

PROJECT: : FO09-A1

INITIA DATA

Kn = 0.00 Ax = 5.00 By = 7.70 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	292.00	14.00	1113.00	-98.00	807.00	0.00
2	198.00	14.00	909.00	-98.00	604.00	0.00
3	279.00	14.00	1107.00	-98.00	744.00	0.00
4	185.00	14.00	903.00	-98.00	541.00	0.00
5	0.00	11.00	847.00	-75.00	484.00	0.00
6	225.00	36.00	1022.00	-208.00	395.00	0.00
7	225.00	36.00	819.00	-208.00	500.00	0.00

PROPERTIES OF PILES

PILE	Lo	H	Bpx	Bpy	A	B	Cday	Fo	Io	Po	Co	Ct
1	0.00	27.00	1.535	1.535	1.00	0.000	0.000	0.785	0.049	0	108000	54000
2						n t						
3						n t						
4						n t						
5						n t						

PILE COORD.

PILE	X	Y	Phi	Xi
1	1.50	3.33	0.000	0.00
2	1.50	0.56	0.000	0.00
3	1.50	-2.24	0.000	0.00
4	-1.50	1.71	0.000	0.00
5	-1.50	-2.80	0.000	0.00

DISPLACEMENTS

COMB.	X	Y	Z	Fix	Fiy	Fiz
1	0.01626	-0.00032	0.004690	0.000283	0.002254	-0.000098
2	0.01106	-0.00001	0.003914	0.000172	0.001541	-0.000062
3	0.01534	-0.00026	0.004711	0.000260	0.002083	-0.000093
4	0.01014	0.00006	0.003935	0.000150	0.001369	-0.000057
5	0.00111	0.00026	0.003943	0.000044	0.000406	0.000011
6	0.01107	0.00122	0.004538	0.000004	0.001205	-0.000049
7	0.01186	0.00116	0.003480	0.000024	0.001494	-0.000049

FORCES ON PILES

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

1	1	297.43	-56.57	-3.48	-1.373	-17.098	94.255
	2	235.78	-38.45	-3.23	-0.865	-13.449	63.816
	3	290.69	-54.07	-3.45	-1.303	-16.391	92.009
	4	229.04	-35.94	-3.19	-0.795	-12.743	61.569
	5	183.76	-0.21	-2.12	0.160	-6.986	-10.291
	6	264.12	-44.08	-7.54	-0.692	-20.742	88.226
	7	235.36	-44.08	-7.54	-0.692	-21.268	80.474
2	1	330.23	-58.15	-3.48	-1.373	-17.098	98.562
	2	255.77	-39.44	-3.23	-0.865	-13.449	66.531
	3	320.84	-55.56	-3.45	-1.303	-16.391	96.096
	4	246.38	-36.85	-3.19	-0.795	-12.743	64.064
	5	188.89	-0.03	-2.12	0.160	-6.986	-10.792
	6	264.63	-44.87	-7.54	-0.692	-20.742	90.398
	7	238.16	-44.87	-7.54	-0.692	-21.268	82.645
3	1	363.26	-59.73	-3.48	-1.373	-17.098	102.900
	2	275.90	-40.44	-3.23	-0.865	-13.449	69.266
	3	351.21	-57.07	-3.45	-1.303	-16.391	100.212
	4	263.85	-37.77	-3.19	-0.795	-12.743	66.578
	5	194.07	0.16	-2.12	0.160	-6.986	-11.296
	6	265.16	-45.67	-7.54	-0.692	-20.742	92.584
	7	240.97	-45.67	-7.54	-0.692	-21.268	84.832
4	1	34.42	-57.49	-1.78	-1.373	-12.447	96.773
	2	54.55	-39.03	-2.16	-0.865	-10.517	65.403
	3	47.65	-54.94	-1.83	-1.303	-11.978	94.398
	4	67.79	-36.47	-2.21	-0.795	-10.048	63.028
	5	135.97	-0.11	-2.32	0.160	-7.527	-10.584
	6	113.63	-44.54	-6.69	-0.692	-18.397	89.496
	7	49.99	-44.54	-6.69	-0.692	-18.924	81.743
5	1	87.66	-60.05	-1.78	-1.373	-12.447	103.765
	2	87.01	-40.64	-2.16	-0.865	-10.517	69.812
	3	96.60	-57.37	-1.83	-1.303	-11.978	101.033
	4	95.94	-37.96	-2.21	-0.795	-10.048	67.079
	5	144.31	0.19	-2.32	0.160	-7.527	-11.397
	6	114.47	-45.83	-6.69	-0.692	-18.397	93.021
	7	54.53	-45.83	-6.69	-0.692	-18.924	85.268

SUMMARY OF FORCES

	PILE	COMB.	N	Q2	Q3	M1	M2	M3
Nmin	4	1	34.42	-57.49	-1.78	-1.373	-12.447	96.773
Nmax	3	1	363.26	-59.73	-3.48	-1.373	-17.098	102.900
Q2max	5	1	87.66	-60.05	-1.78	-1.373	-12.447	103.765
Q3max	1	6	264.12	-44.08	-7.54	-0.692	-20.742	88.226
M1max	1	1	297.43	-56.57	-3.48	-1.373	-17.098	94.255
M2max	1	7	235.36	-44.08	-7.54	-0.692	-21.268	80.474
M3max	5	1	87.66	-60.05	-1.78	-1.373	-12.447	103.765

CHECKING CALCULATI IN COMPARISON WITH INITIA LOAD MATRIX

1	292.00	14.00	1113.00	-98.00	807.00	0.00
2	198.00	14.00	909.00	-98.00	604.00	0.00
3	279.00	14.00	1107.00	-98.00	744.00	0.00
4	185.00	14.00	903.00	-98.00	541.00	0.00
5	0.00	11.00	847.00	-75.00	484.00	0.00
6	225.00	36.00	1022.00	-208.00	395.00	0.00
7	225.00	36.00	819.00	-208.00	500.00	0.00

	DA NANG - QUANG NGAI EXPRESS WAY PROJECT FO09 BRIDGE DETAIL DESIGN ABUTMENT A1	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)	M _y (kN•m)	F _{Hy} (kN)	M _x (kN•m)
1	Strength 1a		3564	586	-1009	34	168
2	Service Ser-I		1904	-2	111	21	69
3	Extreme Ext-IA		2601	448	-908	74	203
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	1.00 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

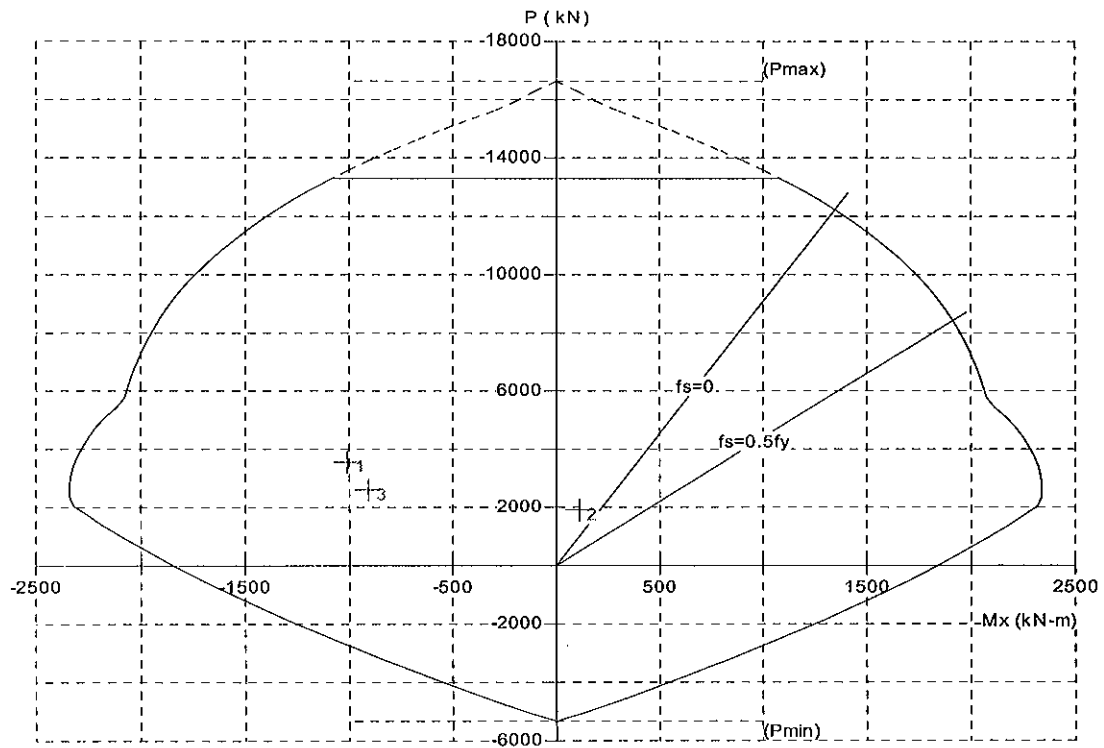
S.5.7.4.2

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

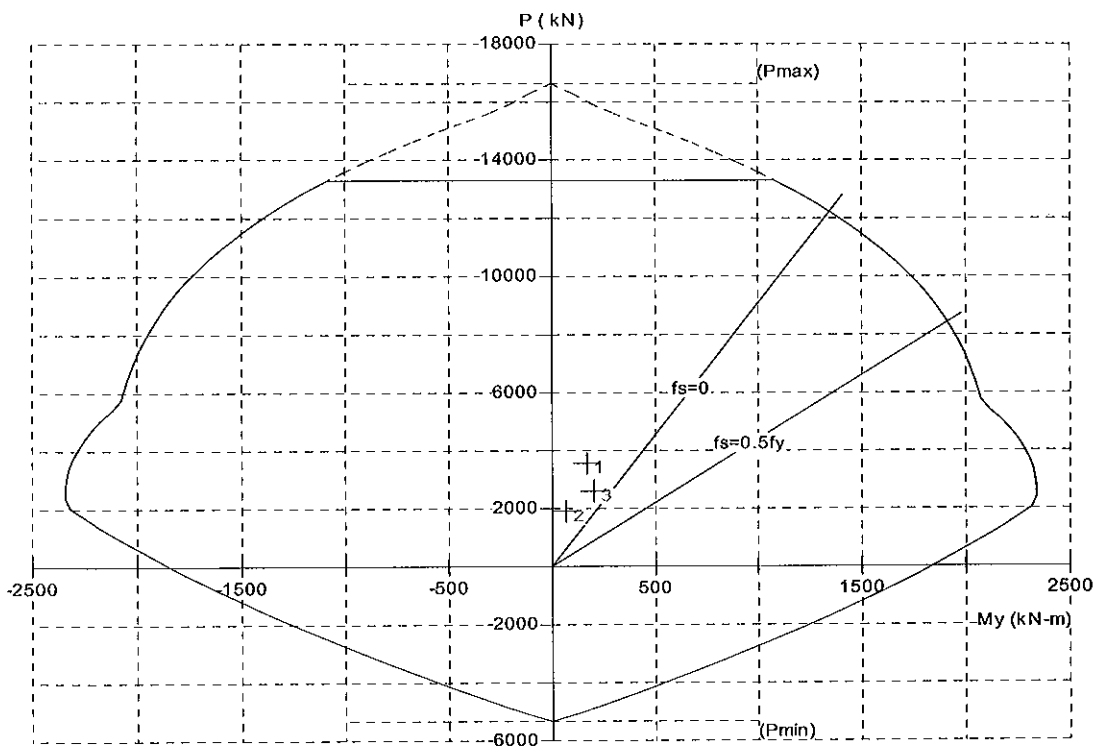
2. Iteration diagram M-P

Using Pca-Column software

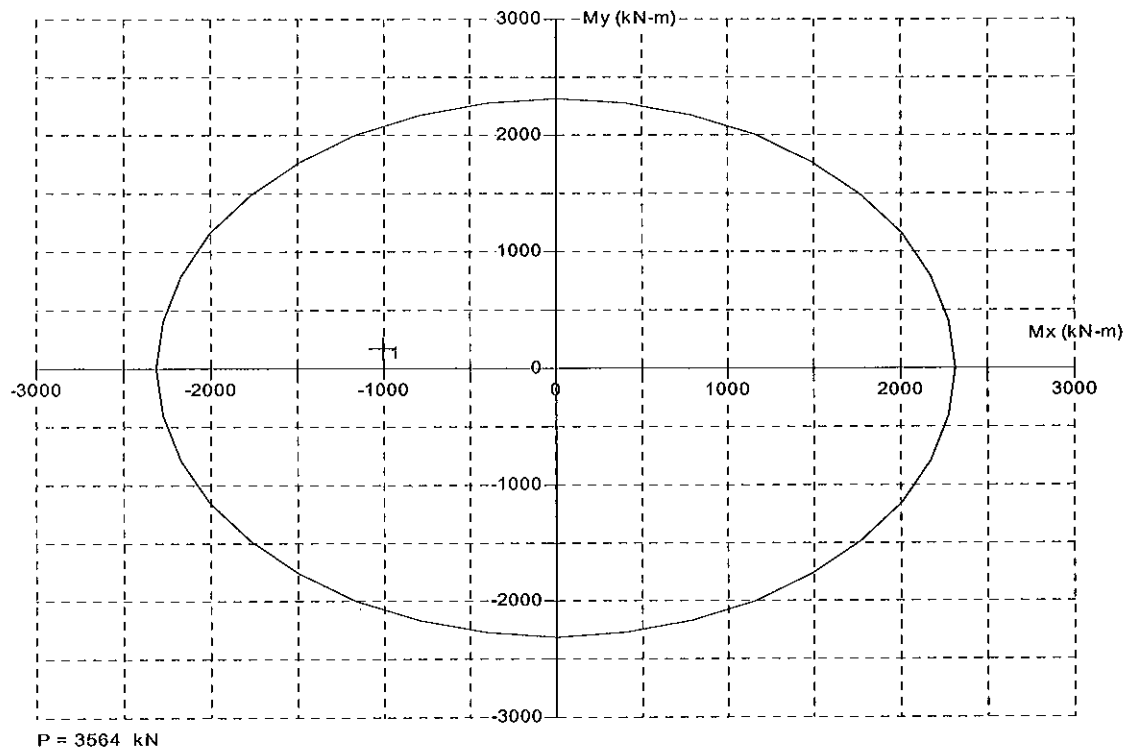
**In Transverse Direction



****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	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	2.78	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	0.0090	
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	0.0089	OK
Transverse Reinforcement for Confinement at Plastic Hinges			S.5.10.11.4.1.d
For a circular column "1:applied", "2:Not applied"		1	
$\rho_s \geq 0.12 \cdot f_c/f_y = \text{Req2}$	Req2	0.0090	N/A
Length distributed spiral with pitch 75mm below pilecap	Ldis	1.50	m

4. Shear Design

Shear resistance factors	ϕ_v	1.0	
Factored shear force	Vu	586	kN
Required shear capacity $V_n = V_u / \phi_v$	Vn	586	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
$d_v = 0.9 \cdot d_e$ $d_e = D/2 + D_r/\pi()$			
Diameter of the circle passing through the centers of the long. reinf.	Dr	0.79	m
	d_e	0.75	m
	d_v	0.68	m

$V_c = 0.083 \cdot \beta \cdot \sqrt{f_c} \cdot b_v \cdot d_v$	V_c	616	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	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
$V_s = A_v \cdot f_y \cdot d_v \cdot (\cot \theta + \cot \alpha) \cdot \sin \alpha / s$	$s \leq$	-	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	2468	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

DA NANG - QUANG NGAI EXPRESSWAY PROJECT				
FO09 - Bridge				
DETAIL DESIGN				
EMPIRICAL ESTIMATION OF BEARING CAPACITY OF PILE- ABUTMENT A1				
Item.	Eng.	Date.	Sign.	
Design				
Check				
Revise				

General Data

AASHTO - LRFD 3rd 2004; 22TCN 272-05

Bored hole name	FO09-A1	Ground water level	GWL = 15.6
Ground level	EL1 = 15.6	Local scour level	LSL = 11
Bottom of pilecap elevation	EL2 = 11.00		
Pile tip elevation	EL3 = -16.00		
Diameter of drilled-shaft	D = 1.0 m	Diameter of drilled-shaft at tip	D _{tip} = 1.0 m
Pile Length	L = 27.0 m	Embedment of drilled-shaft	D _b = 15.40 m
Pile Cross-Sectional Perimeter	P = 3.14 m	Center to center of piles	S = 3 m
Pile Cross-Sectional Area	A _b = 0.79 m2	Factor of group piles	n = 0.70
Pile Concrete Strength	f' _c = 30 Mpa	Resistance factor for soils refer to Table.10.5.5.3	
Concrete Unit Weight	γ _c = 24.5 kN/m³		
Non-contributed length in calculation - in Clay soils			
Top pile length	L _{nt} = 1.5 m		
Bottom pile length	L _{nb} = 1.0 m		
Factored bearing Resistance		In which case	1 "1": Strength ; "2": Extreme
Q_R = φqp . qp . Ap + φqs . qs . As			
< S.10.7.3.2			

Shaft Resistance			
In cohesive soils			
(1) • α - Method	< S.10.8.3.3.1	In cohesionless soils	Used
		(1) • Tourma and Reese (1974)	< S.10.8.3.4.2
		(2) • Meyerhof (1976)	
		(3) • Quiros and Reese (1977)	
		(4) • Reese and Wright (1977)	
		(5) • Reese and O'Neill (1988)	←
Tip Resistance			
In cohesive soils			
(1) • Reese and O'Neill (1988)	< S.10.8.3.3.2	In cohesionless soils	Used
		(1) • Tourma and Reese (1974)	< S.10.8.3.4.3
		(2) • Meyerhof (1976)	
		(3) • Reese and Wright (1977)	
		(4) • Reese and O'Neill (1988)	←
Notes:	"1": elevation of soil that taken into account		
	"2": elevation of tip pile		
	"3": top elevation of good layer for bearing		

DA NANG - QUANG NGAI EXPRESSWAY PROJECT				
FO09 - Bridge				
DETAIL DESIGN				
EMPIRICAL ESTIMATION OF BEARING CAPACITY OF PILE- ABUTMENT A2				
Item.	Eng.	Date.	Sign.	
Design				
Check				
Revise				

General Data

AASHTO - LRFD 3rd 2004; 22TCN 272-05

Bored hole name	FO09-A2	Ground water level	GWL = 16.3
Ground level	EL1 = 16.3	Local scour level	LSL = 11
Bottom of pilecap elevation	EL2 = 11.00		
Pile tip elevation	EL3 = -16.00		
Diameter of drilled-shaft	D = 1.0 m	Diameter of drilled-shaft at tip	D _{tip} = 1.0 m
Pile Length	L = 27.0 m	Embedment of drilled-shaft	D _b = 12.10 m
Pile Cross-Sectional Perimeter	P = 3.14 m	Center to center of piles	S = 3 m
Pile Cross-Sectional Area	A _b = 0.79 m2	Factor of group piles	n = 0.70
Pile Concrete Strength	f' _c = 30 Mpa	Resistance factor for soils refer to Table.10.5.5.3	
Concrete Unit Weight	γ _c = 24.5 kN/m ³		
Non-contributed length in calculation - in Clay soils			
Top pile length	L _{nt} = 1.5 m		
Bottom pile length	L _{nb} = 1.0 m	In which case	1 "1": Strength ; "2": Extreme
Factored bearing Resistance			Q_R = φqp . qp . Ap + φqs . qs . As < S.10.7.3.2
Shaft Resistance			
In cohesive soils			
(1) • α - Method	< S.10.8.3.3.1	In cohesionless soils	Used
		(1) • Tourma and Reese (1974)	
		(2) • Meyerhof (1976)	
		(3) • Quiros and Reese (1977)	
		(4) • Reese and Wright (1977)	
		(5) • Reese and O'Neill (1988)	←
Tip Resistance			
In cohesionless soils			Used
(1) • Reese and O'Neill (1988)	< S.10.8.3.3.2	In cohesionless soils	
		(1) • Tourma and Reese (1974)	
		(2) • Meyerhof (1976)	
		(3) • Reese and Wright (1977)	
		(4) • Reese and O'Neill (1988)	←
Notes:			
"1": elevation of soil that taken into account			
"2": elevation of tip pile			
"3": top elevation of good layer for bearing			

DA NANG - QUANG NGAI EXPRESSWAY PROJECT				
FO09 - Bridge				
DETAIL DESIGN				
EMPIRICAL ESTIMATION OF BEARING CAPACITY OF PILE- PIER P1				
Item.	Eng.	Date.	Sign.	
Design				
Check				
Revise				

General Data

AASHTO - LRFD 3rd 2004; 22TCN 272-05

Bored hole name	FO09-P1	Ground water level	GWL = 16
Ground level	EL1 = 16	Local scour level	LSL = 8
Bottom of pilecap elevation	EL2 = 8.00		
Pile tip elevation	EL3 = -17.00		
Diameter of drilled-shaft	D = 1.0 m	Diameter of drilled-shaft at tip	D _{tip} = 1.0 m
Pile Length	L = 25.0 m	Embedment of drilled-shaft	D _b = 16.80 m
Pile Cross-Sectional Perimeter	P = 3.14 m	Center to center of piles	S = 3 m
Pile Cross-Sectional Area	A _b = 0.79 m2	Factor of group piles	n = 0.70
Pile Concrete Strength	f' _c = 30 Mpa	Resistance factor for soils refer to Table.10.5.5.3	
Concrete Unit Weight	γ _c = 24.5 kN/m ³		
Non-contributed length in calculation - in Clay soils			
Top pile length	L _{nt} = 1.5 m		
Bottom pile length	L _{nb} = 1.0 m		
Factored bearing Resistance		In which case	1 "1": Strength ; "2": Extreme
		Q_R = φ_{qp} . qp . Ap + φ_{qs} . qs . As	
		< S.10.7.3.2	
Shaft Resistance		Used	
In cohesive soils			
(1) • α - Method			
Tip Resistance		Used	
In cohesive soils			
(1) • Reese and O'Neill (1988)			
Notes:	"1": elevation of soil that taken into account		
	"2": elevation of tip pile		
	"3": top elevation of good layer for bearing		

MINISTRY OF TRANSPORT
VIET NAM EXPRESSWAY CORPORATION - PROJECT MANAGEMENT UNIT NO.85

DA NANG - QUANG NGAI EXPRESS WAY PROJECT
PACKAGE: A2

BRIDGE
F009

CALCULATION SHEETS
MISCELLANEOUS

CALCULATION SHEET
POT BEARINGS

	DA NANG - QUANG NGAI EXPRESSWAY PROJECT FO09 BRIDGE DETAIL DESIGN LOADS	Item.	Eng.	Date.	Sign.
		Design			
		Check			
		Revise			

POT BEARING - design checking

AASHTO LRFD 4th 2007

I. POT BEARING CHARACTERISTICS

ELASTOMERIC DISC				
Description	Sign	Specification	Unit	Require.
Hardness, type D durometer - Shore A scale	IRHD	ASTM D2240		60 ± 5
Tensile stress at 100% elongation		ASTM D412	MPa	15.8
Tensile stress at 200% elongation		ASTM D412	MPa	27.6
Tensile strength		ASTM D412	MPa	41.4
Ultimate elongation		ASTM D412	%	220
Compression set at 70°C in 22 hours		ASTM D395	%	40

PTFE SHEET				
Description	Sign	Specification	Unit	Require.
Tensile strength - minimum	fu	ASTM D1457	MPa	19.3
Elongation - minimum		ASTM D1457	%	200
Specific gravity - minimum		ASTM D792		2.16 ± 0.03
Melting point		ASTM D1457	°C	328 ± 2

STAINLESS STEEL - ASTM A240M, type 304				
Description	Sign	Specification	Unit	Require.
Tensile strength - minimum	fu	ASTM A240M	MPa	515
Yield strength - minimum	fy	ASTM A240M	MPa	205
Elongation in 50mm - minimum			%	40

STRUCTURAL STEEL - ASTM A709M, grade 250				
Description	Sign	Specification	Unit	Require.
Tensile strength	fu	ASTM A709M	MPa	400-500
Yield strength - minimum	fy	ASTM A709M	MPa	250
Elongation in 50mm - minimum			%	23

BOLT STEEL - ASTM A307, grade A				
Description	Sign	Specification	Unit	Require.
Tensile strength - minimum	fu	ASTM A307	MPa	414

BRASS SEALING RING				
Description	Sign	Specification	Unit	Require.
Tensile strength	fu	ASTM B36M	MPa	350-420

II. POT BEARING DESIGN CHECK

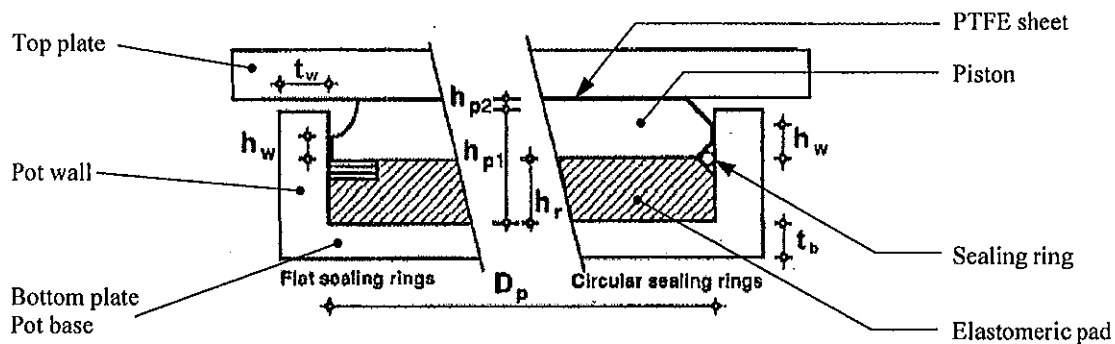
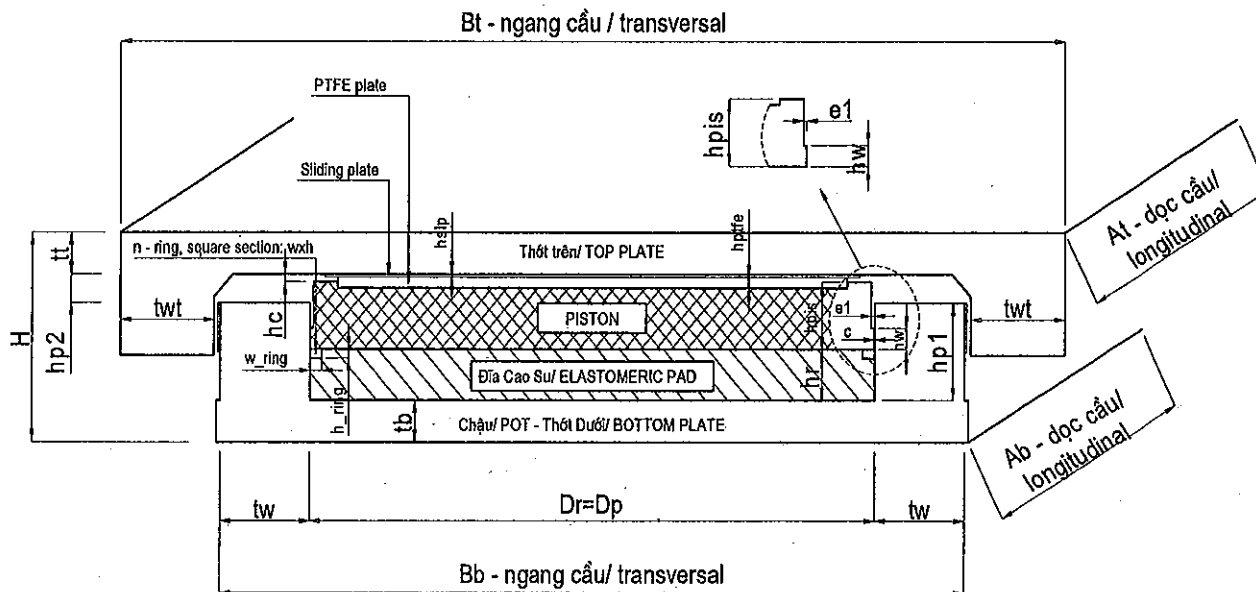


Figure C14.7.4.3-1 Pot Bearing—Critical Dimensions for Clearances.



POT BEARING Parameters				
Description		Sign	Unit	Value
Service limit state	Vertical max	$F_{z,se}$	kN	1700
	Vertical min	$F_{z,se \min}$	kN	750
	Horizontal	$H_{xy,se}$	kN	60
	Rotational	θ_{se}	rad	0
	Longitudinal movement	ΔL	mm	± 50
Ultimate limit state	Vertical	$F_{z,ul}$	kN	2350
	Horizontal	$H_{xy,ul}$	kN	130
	Rotational	θ_{ul}	rad	0.012
Top steel plate	Longitudinal dimension (longitudinal axis)	At	mm	485
	Transverse dimension	Bt	mm	565
	Thickness	tt	mm	25
Bottom steel plate	Longitudinal dimension (longitudinal axis)	Ab	mm	485
	Transverse dimension	Bb	mm	485
	Thickness	tb	mm	30
Inside diameter of pot		Dp	mm	385
Outside diameter of pot		Do	mm	485
Thickness of pot wall		tw	mm	50
Thickness of baffle		twt	mm	35

Pot cavity depth	hp1	mm	45	
	hp1 >	mm	40	
Clearance between top of piston and top of pot wall	hp2	mm	10	
Diameter of elastomeric pad	dr	mm	385	
Thickness of elastomeric pad	hr	mm	20	
Thickness of piston	hpis	mm	30	
Height from top of rim to underside of piston	hw	mm	20	
	hp1-hr > hw	mm	25	
Clearance between piston and internal diameter of pot	c	mm	0.5	
Diameter of PTFE sheet	dp	mm	385	
Thickness of PTFE sheet	hptfe	mm	7	
Top sliding plate	hslp	mm	2	
Clearance between top of piston and bottom of top plate	hc	mm	5	
Diameter of bolt	db	mm	40	
Total height of pot bearing	H	mm	110	
Bearing height check	Ok	Hcheck	mm	110

1. Thickness of Elastomeric pad Check

<14.7.4.3>

$$hr \geq 3.33 * Dp * \theta u$$

Ok

Thickness of elastomeric pad	hr	≥	15 mm
Internal diameter of pot	Dp	=	385 mm
Required rotation at ultimate limit state	θu	=	0.012 rad
Thickness of elastomeric pad, choose	hr0	=	20 mm

2. Pot cavity depth Check

<14.7.4.3>

$$hp1 \geq 0.5 * Dp * \theta u + hr + hw$$

Ok

Pot cavity depth	hp1	≥	42 mm
Internal diameter of pot	Dp	=	385 mm
Required rotation at ultimate limit state	θu	=	0.012 rad
Thickness of elastomeric pad	hr	=	20 mm
Height from top of rim to underside of piston	hw	=	20 mm
Pot cavity depth, choose	hp1	=	45 mm

3. Vertical clearance Check

<14.7.4.3>

$$hp2 \geq R0 * \theta u + 2 * \delta u + 3$$

Ok

Clearance between top of piston and top of pot wall	hp2	≥	7 mm
Required rotation at ultimate limit state	θu	=	0.012 rad
Vertical deflection due to factored load	δu	=	0.74 mm
Shear modulus of elastomeric pad	G	=	0.93 MPa
Elastic modulus of elastomeric pad	Ec	=	803.5 MPa
Stress in elastomeric pad	σep	=	14.6 MPa
Radial distance from center of pot to object in question	R0	=	192.5 mm
Clearance between top of piston and top of pot wall, choose	hp2	=	10 mm

4. Compressive stress of Elastomeric pad Check

<14.7.4.4>

$$F/Sr = F / (\pi * (dr/2)^2) \leq q_0$$

Ok

Vertical load at service limit state	F	=	1700 kN
Area of elastomeric pad	Sr	=	
Diameter of elastomeric pad	dr	≥	294 mm
Average compressive stress of elastomeric at service state should not exceed this value	q0	=	25 MPa
Diameter of elastomeric pad, choose	dr0	=	385 mm

5. Sealing Rings

<14.7.4.5>

case: *Rings with rectangular cross-sections*

n

Three rectangular rings shall be used. Each ring shall be circular in plan but shall be cut at one point around its circumference. The faces of the cut shall be on a plane at 45° to the vertical and to the tangent of the circumference.

The rings shall be oriented so that the cuts on each of the three rings are equally spaced around the circumference of the pot.

Width of each seal ring

Width of each ring shall not be less than either

$$0.02 \cdot D_p = 7.7 \text{ mm}$$

$$\text{or } 6.0 \text{ mm} = 6 \text{ mm}$$

And shall not exceed 19mm

$$\leq 19 \text{ mm}$$

Width of each seal ring, choose

$$W_{sr} = 9 \text{ mm}$$

Depth of each seal ring

Depth of each shall not be less than 0.2times its width

$$0.2 \cdot W_{sr} = 1.8 \text{ mm}$$

Depth of each seal ring, choose

$$d_{sr} = 3 \text{ mm}$$

case: *Rings with circular cross-sections*

y

One circular closed ring shall be used with an outside diameter of D_p .

Cross-sectional diameter

Ok

Cross-sectional diameter not less than either 0.0175 D_p or 4 mm.

$$0.0175 \cdot D_p = 6.7 \text{ mm}$$

$$\text{or } 4.0 \text{ mm} = 4 \text{ mm}$$

Cross-sectional diameter, choose

$$D_{sr} = 10 \text{ mm}$$

6. Thickness of pot wall, pot base Check

<14.7.4.6>

case: *Base bearing directly against concrete or grout*

y

Ok

$t_b \geq 20 \text{ mm}$

$$t_b \geq 20 \text{ mm}$$

$t_b \geq 0.06 \cdot D_p$

$$t_b \geq 23 \text{ mm}$$

Thickness of pot base, choose

$$t_{b0} = 30 \text{ mm}$$

case: *Base bearing directly against steel girders or distribution plate*

n

$t_b \geq 12.5 \text{ mm}$

$$t_b \geq 12.5 \text{ mm}$$

$t_b \geq 0.04 \cdot D_p$

$$t_b \geq 15 \text{ mm}$$

Thickness of pot base, choose

$$t_{b0} = 30 \text{ mm}$$

Minimum pot wall thickness for an unguided sliding pot bearing

Ok

$t_w \geq D_p \cdot \sigma_s / (1.25 \cdot F_y)$

$$t_w \geq 18.0 \text{ mm}$$

$$\sigma_s = 15 \text{ MPa}$$

$t_w \geq 20 \text{ mm}$

$$t_w \geq 20 \text{ mm}$$

Thickness of pot wall, choose

$$t_{w0} = 50 \text{ mm}$$

7. Thickness of pot wall, pot base Check

<14.7.4.7>

$$t_w, t_b \geq \sqrt{25 \cdot H_{xy,ul} \cdot \theta_u / F_y}$$

Ok

Pot bearing subject lateral loads

Thickness of pot wall and pot base

$$t_w \geq 12.5 \text{ mm}$$

Required rotation at ultimate limit state

$$\theta_u = 0.012 \text{ rad}$$

Required horizontal force at ultimate limit state

$$H_{xy,ul} = 130 \text{ kN}$$

Yield strength of pot steel

$$F_y = 250 \text{ MPa}$$

Thickness of pot wall, choose

$$t_{w0} = 50 \text{ mm}$$

Thickness of pot base, choose

$$t_{b0} = 30 \text{ mm}$$

8. Height of top of rim to underside of piston Check <14.7.4.7>

Pot bearing that transfers load through the piston

hw ≥ 3mm

hw ≥ 0.03*Dp

Height from top of rim to underside of piston

Internal diameter of pot

Yield strength of Piston steel

Required horizontal force at ultimate limit state

Height from top of rim to underside of piston, choose

Thickness of piston

Thickness of piston should not be less than 6%Dp, except at rim

Thickness of piston, choose

$$hw \geq 1.5 \cdot H_{xy,ul} / (D_p \cdot F_y)$$

Ok

hw	≥	3 mm
hw	≥	11.6 mm
hw	≥	2.0 mm
Dp	=	385 mm
Fy	=	250 MPa
Hxy,ul	=	130 kN
hw0	=	20 mm

Ok

6%.Dp	=	23.1 mm
hpis	=	30 mm

9. Clearance bet. piston and int. diameter of pot Check <14.7.4.7>

If surface of piston rim is cyclindrical

Clearance between piston and internal diameter of pot

Required rotation at ultimate limit state

Internal diameter of pot

Height from top of rim to underside of piston

Clearance between piston and internal diameter of pot, choose

$$c \geq \max(0u \cdot (hw - D_p \cdot 0u/2), 0.5)$$

Ok

c	≥	0.5 mm
0u	=	0.012 rad
Dp	=	385 mm
hw	=	20 mm
c	=	0.5 mm

10. Compressive stress for PTFE sheet Check <14.7.2.4-1>

Compressive stress for PTFE sheet

Diameter of PTFE sheet

Average compressive stress for PTFE at service limit state should not exceed the value

Vertical load at service limit state

Diameter of PTFE sheet, choose

$$\sigma_p = F / (\pi \cdot (d_p/2)^2) \leq q_p$$

Ok

σp	=	MPa
dp	≥	264 mm
qp	=	31 MPa
Fz,se	=	1700 kN
dp	=	385 mm

11. Shear resistance of bolt Check <6.13.2.7>

Required horizontal force at ultimate limit state

Bolt diameter

Number of bolt

Tensile strength of bolt steel

Number of shear planes per bolt

Shear resistance of bolts

Friction force between concrete and steel pot

$$R_{n1} = \phi \cdot (f_{ms} \cdot V_{min})$$

Friction coefficient steel/concrete

Resistance factor

Shear resistance of bolts and friction

$$H_u \leq (R_{n1} = 0.48 A_b \cdot F_{ub} \cdot N_s + R_{n2})$$

Ok

Hxy,ul	=	130 kN
db	=	40 mm
Nbolt	=	4 bolts
Fu	=	414 MPa
Ns	=	1
Rn1	=	999 kN
Rn2	=	184 kN
fms	=	0.35
φ	=	0.70
Rn	=	1183 kN

12. Compressive stress for concrete at top and bottom plate Check

Vertical load at service limit state

Thickness of top steel plate

Thickness of bottom steel plate

Internal diameter of pot

Concrete stress at underside of bottom plate

Concrete stress at uperside of top plate

Allowable compressive stress

$$\sigma = F / (\pi \cdot (\sqrt{3} \cdot t_t + b + D_p/2)^2)$$

Ok

Fz,se	=	1700 kN
tt	=	25 mm
tb	=	30 mm
Dp	=	385 mm
σb	=	9.1 MPa
σt	=	9.7 MPa
[σ]	=	30.0 MPa

CALCULATION SHEET

STEEL DOWELS&EXPANSION JOINT

	DA NANG - QUANG NGAI EXPRESSWAY PROJECT			
	FO09 BRIDGE			
	DETAIL DESIGN			
	LOADS			

LOADS

I. Displacement

Maximum allowable displacements in longitudinal direction =

40 mm

Maximum displacement

15.6

OK

A1

Tải trọng	Symbol	Sign	Unit (mm)	
			Displacement	Service
			Case1	a
TU+	TU	+	6.00	1.20
TU-	TU	-	-6.00	1.20
Cr&Sh	CR&SH	-	-8.00	1.20
Other loads		±	1.20	1.00
Max Stretch			=	-1.2
Max Shrink			=	-15.6
Maximum displacement				15.6

II. Force

Galvanised steel dowel

D

=

32.0 mm

Number

=

4.0 bar

fu

=

380 Mpa

Resistance force

$R_n = 0.48 \cdot A \cdot f_u$

=

586.8 kN

OK

S-6.13.2.7

Load	Symbol	Force (kN)	a	b
Cr&Sh	CR&SH	0	-	0.50
TU	TU	0	-	0.50
EQ	EQ	256	1.00	-

Maximal shear force $Q = \max(CR\&SH+TU, EQ)/0.8$

=

320.0 kN

CALCULATION SHEET

PARAPET

CALCULATION SHEET FOR OUTSIDE BARRIER OF FO09 BRIDGES

- Bridge Design Standard 22 TCN - 272 - 05 (considered with AASHTO LRFD 2007)

A. GENERAL DATA:

1. Design live load

Design vehicle load	HL93	
Number of lanes	2.00	(lanes)

2. Bridge width

Width of carriageway	$B_{CAR} =$	6.50	(m)
Width of barrier wall	$B_{lc} =$	0.50	(m)
Bridge width	$B =$	7.50	(m)

3. Material properties:

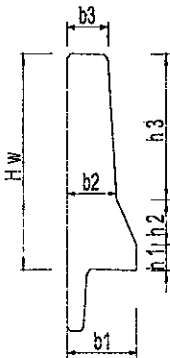
Concrete

Compressive strength of cylindrical at 28 days age	$f_c =$	25.00	MPa
Concrete density	$g =$	24.50	KN/m ³
Elastic modulus	$E_c =$	25278.73	MPa
Tensile strength of concrete	$f_r =$	3.15	MPa

Steel CB-400-V

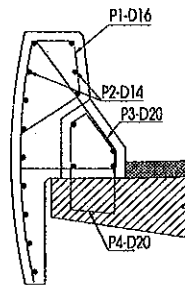
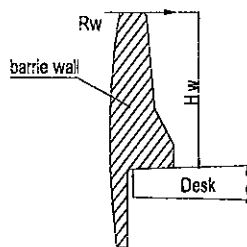
Modulus of elasticity	$E_s =$	200000	MPa
Yield strength of steel bar	$f_y =$	400	MPa

4. Dimensions of RC barrier wall:



b1	500	(mm)
b2	325	(mm)
b3	290	(mm)
h1	159	(mm)
h2	255	(mm)
h3	355	(mm)

5. Diagram of Calculation



6. Railing shall be proportioned such that:

$$R \geq F_t$$

(13.7.3.3-1)

In which:

- R - Total resistance of the barrier wall
 F_t - Transverse vehicle impact force

8. General value:

- Diameter of longitudinal steel bar 14 (mm)

- Diameter of stirrup 20 (mm)
- Reinf. Spacing of stirrup 150 (mm)
- Φ Bending resistance factor 1

8.1 Choose Design force for barrier wall :

- Barrier wall containment level:

Ft	120 (KN)	(AASHTO2007 Table 13.2-1)
He(min)	810	

8.2 Total capacity of Barrier wall:

8.2.1. Resistance of concrete wall for vertical axial (Mw.H)

+ Mw for out-face

Segment	Width of Segment $b' = h$	Number of bars n	Effective Depth $d(+)$	Area of bars A_s	$a = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b}$	$\Phi \cdot Mn(+)$ $\Phi \cdot A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right)$
	(mm)	(Bar)	(mm)	(mm ²)	(mm)	(KNmm)
1	159	1	223	154	18	13170.19
2	255	1	258	154	11	15536.56
3	355	4	433	616	33	102627.45

+ Mw for Int-face

Segment	Width of Segment $b' = h$	Number of bars n	Effective Depth $d(+)$	Area of bars A_s	$a = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b}$	$\Phi \cdot Mn(-)$ $\Phi \cdot A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right)$
	(mm)	(Bar)	(mm)	(mm ²)	(mm)	(KNmm)
1	159	1	223	154	18	13170.19
2	255	1	248	154	11	14920.80
3	355	4	423	616	33	100164.45

+ Resistance of concrete wall for vertical axial (Mw.H)

Segment	Width of Segment $b' = h$	$\Phi \cdot Mn(+)$ Out-face	$\Phi \cdot Mn(-)$ Int-face	$\Phi \cdot Mni$ Average of two face	Mw.H $\sum \Phi \cdot Mni$
	(mm)	(KNmm)	(KNmm)	(KNmm)	(KNmm)
1	159	13170.19	13170.19	13170.19	129794.82
2	255	15536.56	14920.80	15228.68	
3	355	102627.45	100164.45	101395.95	

Where:

- d - Average distance from compression face to centroid of tension reinforcement (mm)
- a - Thickness of the equivalent stress block (mm)
- A_s - Area of tension reinforcement (mm²)

8.2.2. Transverse Ultimate resistance of wall (Mc)

+ Transverse resistance of RC barrier wall (Mc)

Shear contact area: (mm²/mm)

$$A_s = \frac{\pi \cdot \Phi^2}{4 \cdot D}$$

(with D is Reinf. Spacing of shear)
and b = 1 m

Segment	Hight of Segment h	Shear contact area A_s	Effective Depth d	$a = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b}$	Mci $\Phi \cdot A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right)$	Mc $\frac{\sum \Phi \cdot M \cdot hi}{\sum hi}$
	(mm)	(mm ² /mm)	(mm)	(mm)	(KNmm)	(KNmm)
1	159	2.094	310	39.42	243.191	

2	255	2.094	139.5	39.42	100.353	195.826
3	355	2.094	310	39.42	243.191	

+ Total ultimate resistance of RC barrier wall:

- For impacts within a wall segment :

$$R_w = \left(\frac{2}{2.L_c - L_t} \right) \left(8.M_b + 8.M_w.H + \frac{M_c.L_c^2}{H_w} \right) \quad (\text{TCN 13.7.3.4-1})$$

In which :

- Rw - Total transverse resistance of the RC barrier wall (N)
- Lc - Critical length of yield line failure pattern (mm)
- Lt - Longitudinal length of distribution of impact force Ft (mm)
- Mw - Flexural resistance of a wall (KNmm/mm)
- Mc - Transverse flexural resistance of wall (KNmm/mm)
- Mb - Additional flexural resistance of beam in addition to Mw, if any, at top of wall (KNmm/mm)
- Hw - Height of barrier wall Hw (mm)

- Critical length of yield line failure pattern Lc :

$$L_c = \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2} \right)^2 + \frac{8.H_w.(M_b + M_w.H)}{M_c}} \quad (\text{TCN 13.7.3.4-2})$$

Lt (mm)	Hw (mm)	Mb (KN.mm)	Mc (KN.mm/mm)	Mw.H (KN.mm)	Lc (mm)	Rw (KN)
2440	769	0	195.83	129794.82	3579	1822.91

- For impacts at end of wall or at joint :

$$R_w = \left(\frac{2}{2.L_c - L} \right) \left(M_b + M_w.H + \frac{M_c.L_c^2}{H_w} \right) \quad (\text{TCN 13.7.3.4-1})$$

- Critical length of yield line failure pattern Lc :

$$L_c = \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2} \right)^2 + \frac{H_w.(M_b + M_w.H)}{M_c}}$$

Lt (mm)	Hw (mm)	Mb (KN.mm)	Mc (KN.mm/mm)	Mw.H (KN.mm)	Lc (mm)	Rw (KN)
2440	769	0	195.83	129794.82	2634	1341.26

9. RESISTANCE CHECK FOR RC BARRIER WALL

- Condition 1

$$R = R_w \geq F_t$$

With : Ft = 120

(KN)

+ Resistance Check for RC barrier wall accordant Condition 1

Combination	Resistance barrier Wall Rw (KN)	Ft (KN)	Check Condition (1)
1. Impact at end of wall or joint	1341.26	120	OK
2. Impact at a wall segment	1822.91	120	OK

10. SEFT WEIGHT OF RC BARRIER WALL (DC_{lc})

+ Seft weight of concrete	γ_c	24.5	(KN/m ³)
+ Seft weight of steel	γ_s	78.5	(KN/m ³)
+ seft weight of Asphalt concrete	γ_a	22.1	(KN/m ³)

- Seft weight of concrete wall

+ Area of concrete wall	$A_c =$	0.529	(m ²)
+ Load due to weight of wall	$DC_c = \gamma_c \cdot A_c$		
	$DC_{lc} =$	12.96	(KN/m)

11. CHECK SHEAR-RESISTANCE OF RC AT BASE OF THE WALL JOINT WITH DECK

- Arrangement of stirrup D20 attach in overhang

Assuming that R_w spreads out at a 1:1 slope from L_c

- The tensile force per unit of length in the overhang, is given by:

$$T = \frac{R_{\max}}{L_c + 2 \cdot H_w}$$

- Height of barrier	$H_w =$	769	(mm)
- Maximum of load impact on barrier wall	$R_{\max} =$	1822.91	(KN)
	$L_c =$	3579	(mm)
	$T_1 =$	356.23	(N/mm)
- For Impact at end of barrier wall	$R_{\max} =$	1341.26	(KN)
	$L_c =$	2634	(mm)
	$T_2 =$	321.53	(N/mm)
- Shear load for calculate	$T = \text{Max}(T_1, T_2)$		
	$T =$	356.23	(N/mm)

- The nominal shear resistance V_n of the interface plane following:

$$V_n = c \cdot A_{cv} + \mu \cdot (A_{vf} \cdot f_y + P_c)$$

Which shall not exceed $0.2f_c$ or $5.5A_{cv}$

Where:

- Shear contact area:	$A_{cv} = b_1 \cdot 1 \text{ mm}$		
	$A_{cv} =$	500	(mm ² /mm)
- Dowel area across shear plane:	$A_{vf} = \frac{\pi \cdot \Phi^2}{4 \cdot D}$		(Determined in 9.2.2)
	$A_{vf} =$	2.094	(mm ² /mm)
- Yield strenght of reinforcement	$f_y =$	400	(MPa)
- Permanent compressive force:	$P_c = DC_{lc} \cdot 1 \text{ mm}$		
	$P_c =$	12.96	(N/mm)
- Strength of weaker concrete	$f_c =$	25	(MPa)
- Cohesion factor	$c =$	0.52	[5.8.4.2 - 22TCN 272-05]
- Friction factor	$\mu =$	0.60	[A5.8.4.2 - 22TCN 272-05]
	$V_n =$	770.43	(N/mm)
	$0.2f_c \cdot A_{cv} =$	2500	(N/mm)
	$5.5A_{cv} =$	2750	(N/mm)
- Nominal shear resistance:	$V_n = \text{Min}(V_n, 0.2f_c \cdot A_{cv}, 5.5A_{cv})$		
	$V_n =$	770.43	(N/mm)
		$> T = 356.23$	(N/mm) : OK

+ The minimum cross-sectional area of dowels across the shear plane:

$$A'_{vf} = 0.35 \frac{b_1 \cdot s}{f_y} \quad [5.8.4.1 - 22TCN 272-05]$$

$$A'_{vf} = 0.55 \frac{f_y}{f_c}$$

$$A'_{vf} = 65.63 \text{ (mm}^2\text{)}$$

$$n = 2$$

- Number of stirrup input deck

- Cross-sectional area of stirrup input deck

$$A_s = n \cdot A_{vf} \cdot s$$

$$A_s = 628.32 \text{ (mm}^2\text{)}$$

> A'_{vf} : OK

- The development length l_n shall not less than 3 values then:

$$\frac{100 \cdot \Phi}{\sqrt{30}} = 365 \text{ (mm)}$$

With $\Phi = 20 \text{ (mm)}$

$$8 \cdot \Phi = 160 \text{ (mm)}$$

$$\text{And } 150 \text{ (mm)}$$

$$l_n = 365 \text{ (mm)}$$

(The required modify)

- The development length:

- Modification factor for adequate cover:

$$k_1 = 0.7$$

$$l'_n = k_1 \cdot l_n$$

$$l'_n = 256 \text{ (mm)}$$

$$l_n = 256 \text{ (mm)}$$

- The development length after modify:

- The Available development length:

$$l_c = hf - as(+)$$

$$l_c = 160 \text{ mm (which is not adequate)}$$

- Unless the required area is reduced to

$$A_{vf}(hc) = A_{vf} \cdot l_c / l_n$$

$$A_{vf}(hc) = 1.311 \text{ (mm}^2\text{)}$$

By using this area to recalculate M_c , L_c , R_w (The determined following 5.2.2)

Segment	Height of Segment h (mm)	Shear contact area As (mm ² /mm)	Effective Depth d (mm)	$a = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b}$ (mm)	$M_{ci} = \Phi \cdot A_s \cdot f_y \cdot \left(d - \frac{a}{2} \right)$ (KNmm)	$M_c = \frac{\sum \Phi \cdot M \cdot h_i}{\sum h_i}$ (KNmm)
1	159	1.311	310	24.68	156.096	126.447
2	255	1.311	139.5	24.68	66.684	
3	355	1.311	310	24.68	156.096	

- For impacts within a wall segment :

Lt (mm)	Hw (mm)	Mb (KN.mm)	Mc (KN.mm/mm)	Mw.H (KN.mm)	Lc (mm)	Rw (KN)
2440	769	0	126.45	129794.82	4013	1319.87

- For impacts at end of wall or joint :

Lt (mm)	Hw (mm)	Mb (KN.mm)	Mc (KN.mm/mm)	Mw.H (KN.mm)	Lc (mm)	Rw (KN)
2440	769	0	126.45	129794.82	2729	897.54

+ Check railing following Condition 1

Combination	Resistance of Wall		Check
	Rw (KN)	R (KN)	Condition (1)
1. Impact at end of wall or joint	897.54	120.00	OK
2. Impact at a wall segment	1319.87	120.00	OK

8 ORB25a

A. SUBSTRUCTURE DESIGN

1 .Abutment A1

Table of content

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

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

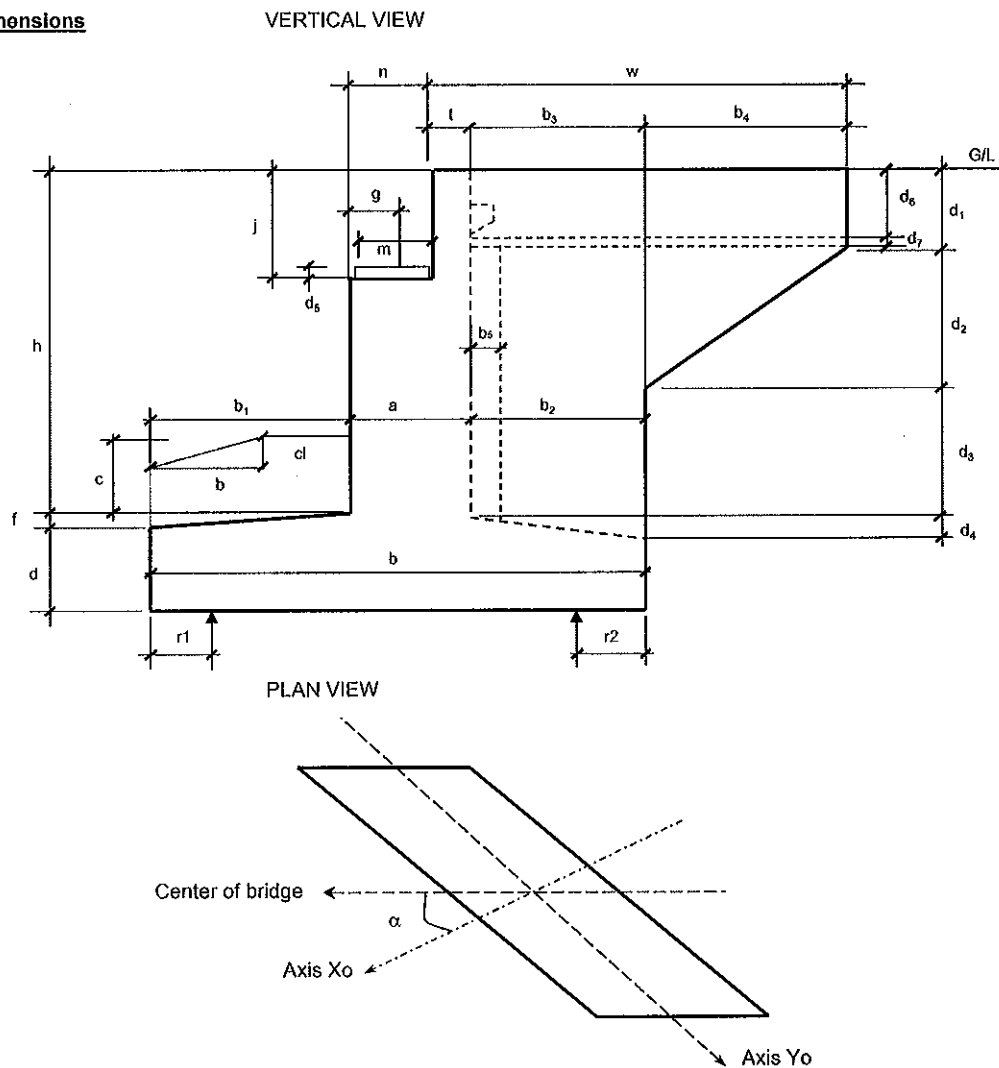
Input :

Level Table(at center of abutment)

Level of top of headwall	HTwL	5.934	m
Level of top of bearing	BTL	4.295	m
Level of top of stem abutment	HTL	4.145	m
Level of top of footing	FTL	-0.500	m
Level of bottom of footing	FBL	-2.500	m
Ground level	GL	5.895	m
Lowest water level	HWL	3.270	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	6.434	Horizontal Dimension	b ₃	2.000
Footing Width	b	5.500	Horizontal Dimension	b ₄	2.400
Stem Width	a	1.500	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	2.400
Width of stem at bearing	n	1.000	Vertical Dimension	d ₃	3.195
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	4.900	Vertical Dimension	d ₆	1.200
Soil Cover at Toe	c	6.395	Vertical Dimension	d ₇	0.000
Girder Reaction	g	0.600	Width 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	2.000	Distance to cl of pile	r2	1.000

Slope front of abutment

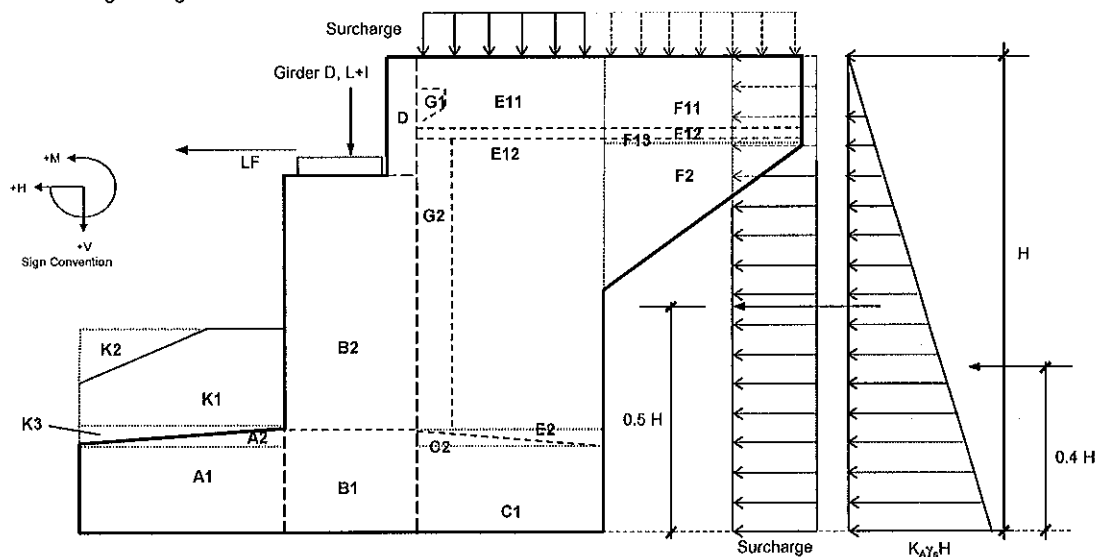
Width of Abutment

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	=	8.43 m
b/2	=	2.75 m



1. Self weight of Abutment (DC)

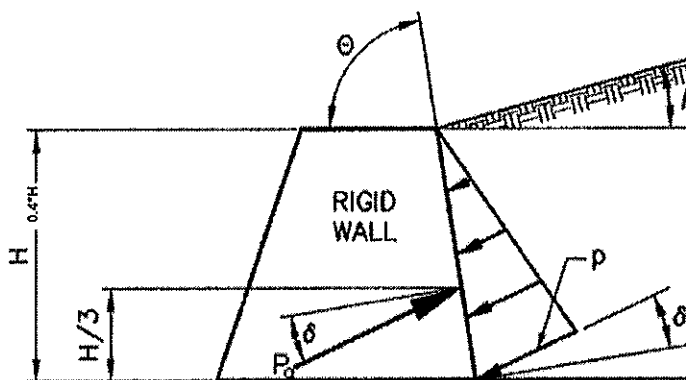
Description	Area (m ²)	Length (m)	Force (kN)	X ⁽¹⁾ (m)	Arm ⁽²⁾ (m)	Moment (kN·m)
SW of Abutment (DC)						
Section A1	4.000	12.600	1235	1.000	1.750	2161
Section A2	-	12.600	-	1.333	1.417	-
Section B1	3.000	12.600	926	2.750	-	-
Section B2	6.968	12.600	2151	2.750	-	-
Section C1	4.000	12.600	1235	4.500	-1.750	-2161
Section C2	-	12.600	-	4.167	-1.417	-
Section D	0.895	12.600	276	3.250	-0.500	-138
Section E11	1.678	0.500	21	4.500	-1.750	-36
Section E12	11.190	0.500	137	4.500	-1.750	-240
Part extra stem	4.217	0.740	76	5.083	-2.333	-178
Section F11	2.880	0.500	35	6.700	-3.950	-139
Section F12	-	0.500	-	5.700	-2.950	-
Section F13	-0.866	0.500	-11	6.700	-3.950	42
Section F2	2.880	0.500	35	6.300	-3.550	-125
Section G1	0.135	12.100	283	3.650	-0.900	-254
Section G2	0.045	10.468	12	3.650	-0.900	-10
Bearing seats (w1seat= 0.65m)	0.083	3.250	10	2.600	0.150	2
Curbs +Handrail on Abutment	0.50	4.900	65	5.450	-2.700	-175
Total SW of Abutment (DC)			6486			-1253
Transverser moment			294		6.175	1816
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	12.87	12.100	2803	4.500	-1.750	-4905
Section E2	-	12.100	-	4.833	-2.083	-
Section E3	-	0.500	-	5.500	-2.750	-
Section K1	12.790	12.600	2901	1.000	1.750	-
Section K2	-	12.600	-	-	2.750	-
Section K3	-	12.600	-	0.667	2.083	-
Total Earth on Footing			5703			-4905

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

$$\begin{aligned}\theta &= 90.0 \text{ deg} \\ \delta &= 20.0 \text{ deg} \\ \beta &= 0.0 \text{ deg} \\ g &= 9.81 \text{ m/s}^2\end{aligned}$$

- 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)

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

$$\begin{aligned}\Gamma &= 2.684 \\ Ka &= 0.297 \\ p &= 0.045 \text{ Mpa}\end{aligned}$$

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

Horizontal earth pressure:

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

$$\begin{aligned}Ea &= 2398 \text{ kN} \\ M &= 8091 \text{ kNm}\end{aligned}$$

<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=	8.43m heq=	0.64 m

(Linear interpolation)

- Vertical force

$$\begin{aligned}ESv &= 290 \text{ kN} \\ ev &= -1.75 \text{ m} \\ M &= -507 \text{ kNm}\end{aligned}$$

- Horizontal force

$$\begin{aligned}ESh &= 363 \text{ kN} \\ eh &= 4.22 \text{ m} \\ M &= 1531 \text{ kNm}\end{aligned}$$

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

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 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 $\delta = 20.0 \text{ deg}$
 • Horizontal acceleration coefficient $k_h = 0.087$
 • Vertical acceleration coefficient $k_v = 0.035$
 • Angle $\theta = \arctan(k_h / (1 - k_v))$ $\theta = 5.2 \text{ deg}$

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

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

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

• Seismic active lateral Earth pressure coefficient $E_{AE} = 2790 \text{ kN}$

$M_{AE} = E_{AS} \cdot 0.3H + (E_{AE} - E_{AS}) \cdot 0.6H$ $M_{AE} = 8053 \text{ KNm}$

<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 $2.5A = 0.145$
 $C_{sm} = 1.2 \cdot A \cdot S / T_m^{2/3} \leq 2.5 \cdot A$ $C_{sm} = 0.086$
 • Period of vibration of the fundamental mode $T_m = 0.722 \text{ s}$
 $T_m = 2 \cdot \pi \cdot (\sum m_i \cdot h_i^2 / \sum k_i)^{1/2}$

Decription	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	6.968	12.600	186	-	4.323	804
Section C1	4.000	12.600	72	-	1.000	72
Section C2	-	12.600	-	-	2.000	-
Section D	0.895	12.600	24	-	7.540	180
Section E11	1.678	0.500	1	-	5.834	7
Section E12	11.190	0.500	8	-	2.617	-
Section E2	4.217	0.740	4	-	2.000	9
Section F11	2.880	0.500	2	-	5.834	12
Section F12	-	0.500	-	-	5.234	-
Section F13	-0.866	0.500	-1	-	6.615	-
Section F2	2.880	0.500	2	-	6.795	14
Section G1	0.135	12.100	2	-	5.721	13
Section G2	0.045	10.468	1	-	2.617	2
Total EQ of Abutment Selfweight			427			1237

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	=	10.3 m	
Mlong	=	1067 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	=	10.32 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	12.74	m
Girder height	hgi	1.20	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	279.89	kN
Diaphragm	DC	142.39	kN
Total	DC	1899.63	kN
1.2. Deck slab			
Deck slab	DC	1415.82	kN
1.3. Pavement			
Asphalt concrete	DW	457.68	kN
1.4. Handrail			
Handrail + median	DC	543.83	kN

2. Live load (LL):

Truck		145	145	35	kN
Tandem		110	110	kN	m
Lane load		w_L	9.3	kN/m	
Pedestrian	Wpd=	0.0	kN/m ²		
Considerate structure as a simple span					
Reaction Influence		20.3	m		
Number of lanes	n	3			
Multiple presence factor	m	0.85			
Dynamic load allowance	1+IM	1.25			

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

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

$$H_{eq} = 187 \text{ kN}$$

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

Bearing displacement due to uniform temperature and shrinkage creep

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

$$H = G \cdot A \cdot \Delta u / h_t$$

<14.8.3.1-2>

Shear modulus G

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

Bearing area

$$A = 0.120 \text{ m}^2$$

Height of elastomeric layers

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

Number of bearing

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

Horizontal force due to TU+SH&CR

$$H(tu+sh+cr) = 240 \text{ kN}$$

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

$$S = 1.08$$

Design wind velocity

$$V = 57.24 \text{ m/s}$$

Drag coefficient

$$C_d = 1.37$$

Overall width of bridge

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

Depth of superstructure (including solid parapet)

$$d = 2.58 \text{ m}$$

$$b/d = 4.94$$

$$A_t = 54.12 \text{ m}^2$$

Windy obstructed area of superstructure

Force due to transverse wind

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

$$F_{hy} = 145.6 \text{ kN}$$

<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

At 1.8m from surface

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

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 + Deckslab	DC	1658	0.15			249			
Handrail	DC	272	0.15			41			
Pavement	DW	229	0.15			34			
Live Load	LL	1131	0.15			170		1.38	1556
Pedestrian	PL	0	0.15			0		-	-
Trans. wind on Struc.	WS						73	4.65	338
Trans. wind on vehi.	WL						16	6.45	102
Earth quake	EQ						187	4.65	867
TU+SH&CR	TU+SH&CR			240	4.65	1115			

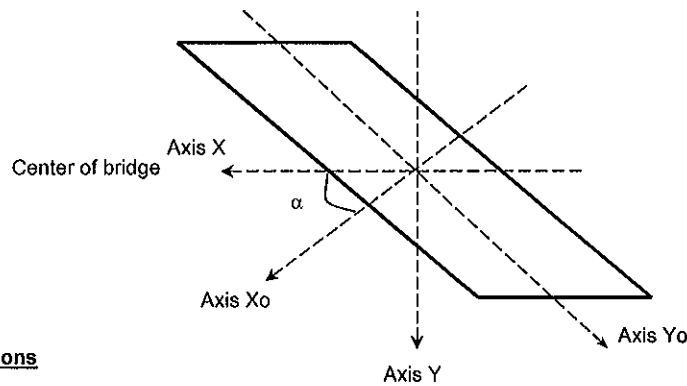
Loads at bottom of pilecap		Vertical		Longitudinal			Tranversal		
Loads	Sign	N (kN)	x (m)	Hx (kN)	z ₁ (m)	My (kN.m)	Hy (kN)	y (m)	Mx (kN.m)
Girder + Deckslab	DC	1658	0.15			249			
Handrail	DC	272	0.15			41			
Pavement	DW	229	0.15			34			
LiveLoad	LL	1131	0.15			170		1.38	1556
Pedestrial	PL	0	0.15			0		-	-
Trans. wind on Struc.	WS						73	6.65	484
Trans. wind on vehi.	WL						16	8.45	133
Eearth quake	EQ						187	6.65	1240
TU+SH&CR	TU+SH&CR			240	6.65	1595			

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	4735	120	1268	0	2723
Strength Str-IB	3866	120	1137	0	2723
Strength Str-IIIA	4283	120	1200	45	2337
Strength Str-IIIB	3413	120	1069	45	2337
Service Ser-I	3290	240	1608	38	1759
Extreme Ext-IA	3321	0	498	187	1645
Extreme Ext-IB	2451	0	368	187	1645

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	4735	120	1508	0	2723
Strength Str-IB	3866	120	1377	0	2723
Strength Str-IIIA	4283	120	1440	45	2427
Strength Str-IIIB	3413	120	1309	45	2427
Service Ser-I	3290	240	2088	38	1834
Extreme Ext-IA	3321	0	498	187	2018
Extreme Ext-IB	2451	0	368	187	2018

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	6486				-1253			509.95
Soils on pilecap	EV	5703				-4905			
Horizontal Earth Pressure	EH			2398		8091			
Vertical Surcharge	L _{sv}	290				-507			
Horizontal Surcharge	L _{sh}			363		1531			
Braking Force	BR			104		1067			
Centrifugal Force	CE			-		-	-		-
Buoyancy of Abutment	WA	-2134				134			
Buoyancy of Earth on Abutment	WA	-1790				-129			
Earthquake effects to Abutment	EQ			427		1237	128		371
Earthquake effects to soil	E _{AE}			2790		8053			

Table of load factors

Loads	Sign	Load combinations						
		Str-IA	Str-IB	Str-III-A	Str-III-B	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	12390	4414	7613	0	637
Strength Str-IB	7553	2975	5404	0	459
Strength Str-III-A	12274	4227	6776	0	637
Strength Str-III-B	7437	2788	4568	0	459
Service Ser-I	8555	2865	4029	0	510
Extreme Ext-IA	12028	3451	2152	128	1009
Extreme Ext-IB	7191	3451	4798	128	830

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	4735	120	1508	0	2723
Strength Str-IB	3866	120	1377	0	2723
Strength Str-IIIA	4283	120	1440	45	2427
Strength Str-IIIB	3413	120	1309	45	2427
Service Ser-I	3290	240	2088	38	1834
Extreme Ext-IA	3321	0	498	187	2018
Extreme Ext-IB	2451	0	368	187	2018

3. Total loads at bottom of pilecap

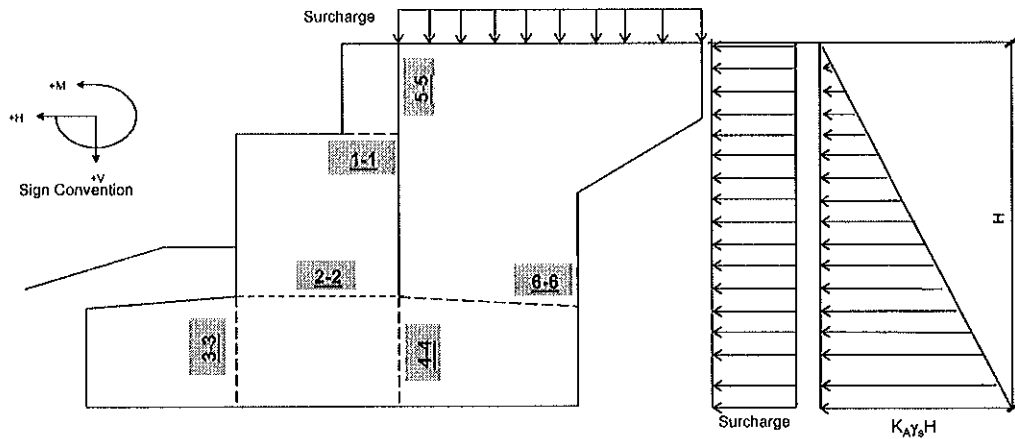
Load Combination at bottom of pilecap					
Load combinations	N (kN)	Longitudinal		Transversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	17125	4534	9121	0	3360
Strength Str-IB	11418	3095	6782	0	3182
Strength Str-IIIA	16557	4347	8216	45	3064
Strength Str-IIIB	10850	2908	5877	45	2886
Service Ser-I	11845	3105	6117	38	2344
Extreme Ext-IA	15349	3451	2651	315	3027
Extreme Ext-IB	9642	3451	5166	315	2848

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		108	77		
Surcharge (horizontal)		193	173		
Horizontal Seismic Earth Pressure		126	77		
Abutment earthquake force		26	23	8	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	559	301	137	0	0
Strength Str-IA	698	500	277	0	0
Strength Str-IB	503	436	271	0	0
Extreme Ext-I	698	311	84	8	7

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

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	1529		-1672		
Vertical soil on foot at behind side	2803		-2803		
Surcharge(Vertical)	290		-290		
Reaction of piles	Ser-I	-2903	-1540	5054	-43
	Str-IA	-4089	-2246	7217	-34
	Str-IB	-2490	-1530	4588	-24
	Ext-I	-5341	-1722	8056	-183

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	1718	-1540	290	-43	-17
Strength Str-IA	2112	-2246	836	-34	27
Strength Str-IB	1915	-1530	54	-24	28
Extreme Ext-I	499	-1722	2037	-183	-264

1.4 Section 5-5 & 6-6

Slope of triang pressure
Uniform horizontal pressure

$$\begin{aligned} \text{tg}\beta &= 5.35 \\ \text{U.p} &= 3.42 \text{ kN/m}^2 \end{aligned}$$

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		74	172		
Strength Str-IA		116	273		

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				205	181
Strength Str-IA				315	278

2. Elements Checking

2.1. Bearing Resistance

<S.5.7.5>

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

Factored bearing resistance shall be taken

$$Pr = \phi \cdot P_n = \phi \cdot 0.85 \cdot f'c \cdot A1 \cdot m$$

Dimension of bearing plate

$$w0 = 0.550 \text{ m}$$

$$b0 = 0.650 \text{ m}$$

Area under bearing device

$$A1 = 0.358 \text{ m}^2$$

Distributed width and length

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

$$b = 1.100 \text{ m}$$

Notational area

$$A2 = 1.100 \text{ m}^2$$

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

$$m = 1.754$$

Resistance factor

$$\phi = 0.700$$

<S.5.5.4.2>

Factored bearing resistance

$$Pr = 11194 \text{ kN}$$

> Pu

Bearing reaction of approach bridge

$$Pu = 3610 \text{ kN}$$

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

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

$$fn = 2.25 \cdot f'ci$$

$$fn = 36.84 \text{ MPa}$$

Maximum area of the portion of supporting surface

$$A = 1.100 \text{ m}^2$$

Gross area of bearing plate

$$Ag = 0.358 \text{ m}^2$$

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

$$Ab = 0.358 \text{ m}^2$$

Nominal concrete strength at time of application

$$f'ci = 30 \text{ MPa}$$

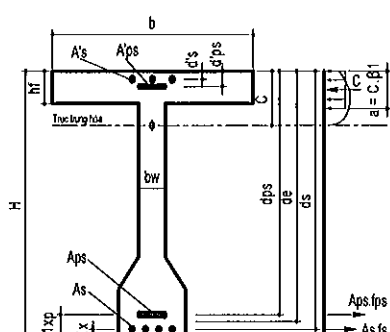
Factored bearing resistance

$$Pr = 9218 \text{ kN}$$

Ok

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/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				
			1-1	1-1	2-2	2-2	2-2
INTERNAL FORCES AT SECTION							
	Combination		Strength	Service	Service	Strength	Extreme
Qu	Shear	kN	500	301	1956	2774	2808
Mu	Flexural Moment	kNm	277	137	6064	8251	6745
Nu	Axial load	kN	698	559	6000	8122	6708
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	16	22	22
		Area	m2	0.01677	0.01677	0.02926	0.02926
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	0.020	0.020	0.020
	For T section behavior	m	0.000	0.000	0.020	0.020	0.020
	For rectangular section behavior	m	0.000	0.000	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	1860	1860	1853	1853	1853
k	Factor depends on type of P.S, Low relaxation strand k = 0.28		0.28	0.28	0.28	0.28	0.28
a	Depth of equivalent stress block	m	0.000	0.000	0.017	0.017	0.017
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	16458	16458	16458
Mr	Factored resistance	kNm	2318	2575	16458	14812	16458
Mu	Flexural moment	kNm	277	137	6064	8251	6745

(5.7.3.2)	Flexural moment Checking		OK	OK	OK	OK	OK
c/de	Limits for reinforcement						
	Maximum reinforcement		0.00	0.00	0.01	0.01	0.01
	Maximum reinforcement Checking	≤ 0.42	OK	OK	OK	OK	OK
1.2*Mc _r	Cracking moment	kNm	1087	1087	9918	9918	9918
(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	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	298	298	287	287	287
0.6*f _y	Tensit 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	0.201	-	-
J _d	Arm	m	-	0.415	1.374	-	-
I _{cr}	Moment of inertia of the cracked section	m ⁴	-	0.018	0.351	-	-
f _s	Tensile stress in reinforcement $f_s = M_{sls} / (A_s * J_d)$	Mpa	-	20	151	-	-
	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	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		3.6	4.2	2.5	2.4	2.4
θ	Angle of inclination of diagonal compressive	degree	28.12	27.00	29.22	32.17	31.28
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	1.432	1.432	1.432
	(dc - a/2)	m	0.442	0.442	1.432	1.432	1.432
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	100	54	108	171	156
f _{po}	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e _s	Strain in tensile reinforcement		2.18E-04	9.31E-05	5.14E-04	6.98E-04	6.43E-04
	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		0.003	0.002	0.004	0.006	0.005
β	Final value		3.6	4.2	2.5	2.4	2.4
θ	Final value	degree	28.12	27.00	29.22	32.17	31.28
V _c	Nominal shear resistance provided by tensile stresses in the concrete	kN	9157	10676	20532	19685	19940
V _s	Shear resistance provided by shear reinforcement	kN	1665	1747	4898	4356	4509
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	10822	12423	25430	24040	24450
V _{n2}	V _{n2}	kN	41769	41769	135368	135368	135368
V _n	Nominal shear resistance $V_n = \min(V_{n1}, V_{n2})$	kN	10822	12423	25430	24040	24450
V _r	Factored shear resistance	kN	9740	12423	25430	21636	24450
V _u	Shear	kN	500	301	1956	2774	2808
(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	54147	54147	54147
	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	4567	7148	4425	499	2112	
Mu	Flexural Moment	kNm	2416	4020	1710	2037	836	
Nu	Axial load	kN	1540	2246	1722	1722	2246	
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	0.163	0.163	
d1x	Dis. From tens. fiber to centroid of tension Reinf	m	0.163	0.163	0.163	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	1.838	1.838	1.838	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	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	bars	84	84	84	84	
		Diameter	mm	25	25	25	20	20
		Area	m2	0.04124	0.04124	0.04124	0.02638	0.02638
A's	Compression Reinforcement	Number	bars	84	84	84	84	
		Diameter	mm	20	20	20	25	25
		Area	m2	0.02638	0.02638	0.02638	0.04124	0.04124
A'c	Shear reinforcement	Number	bars	20	20	20	20	20
		Diameter	mm	16	16	16	16	16
		Area	m2	0.00404	0.00404	0.00404	0.00404	0.00404
φ	Resistance factors for flexure	5.5.4.2	1.00	0.90	1.00	1.00	0.90	
φv	Resistance factors for shear		1.00	0.90	1.00	1.00	0.90	
φn	Resistance factors for axial force		1.00	1.00	1.00	1.00	1.00	
β1	Stress block factor		0.836	0.836	0.836	0.836	0.836	
c	Dis. Between centroid and top fiber	m	0.022	0.022	0.022	-0.022	-0.022	
	For T section behavior	m	0.022	0.022	0.022	-0.022	-0.022	
	For rectangular section behavior	m	0.022	0.022	0.022	-0.022	-0.022	
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	1854	1854	1854	1866	1866	
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	0.019	0.019	0.019	-0.019	-0.019	
de	Conesp. effective depth from extreme comp. fiber to centroid of tensile force in the tensile reinf.	m	1.838	1.838	1.838	1.916	1.916	
Mn	Nominal resistance	kNm	29373	29373	29373	17479	17479	
Mr	Factored resistance	kNm	29373	26436	29373	17479	15731	
Mu	Flexual moment	kNm	2416	4020	1710	2037	836	

(5.7.3.2)	Flexural moment Checking		OK	OK	OK	OK	OK
c/de	Limits for reinforcement		0.01	0.01	0.01	-0.01	-0.01
1.2*Mer	Maximum reinforcement	<= 0.42	OK	OK	OK	OK	OK
(5.7.3.3.2)	Cracking moment	kNm	17586	17586	17586	17201	17201
	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
dc	Existing condition for structure	1,2 or 3	3	3	3	3	3
Z	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.063	0.063	0.063	0.060	0.060
A	Crack width parameter	N/mm	17500	17500	17500	17500	17500
fsa	Area of concr. with same centroid as tens. Reinf	m ²	0.019	0.019	0.019	0.018	0.018
0.6*fy	Value	Mpa	166	166	166	171	171
x	Tensile stress in reinf Min(fs,0.6fy)	Mpa	240	240	240	240	240
J.d	Dist. From compression fiber to centroid	m	0.268	-	-	-	-
Icr	Arm	m	1.748	-	-	-	-
fs	Moment of inertia of the cracked section	m ⁴	0.797	-	-	-	-
	Tensile stress in reinforcement $f_s = M_{sls} / (A_s * J.d)$	Mpa	34	-	-	-	-
	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)						
Areq	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		2.8	2.4	3.2	4.1	3.9
θ	Angle of inclination of diagonal compressive	degree	28.84	30.94	28.66	27.00	27.42
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	1.828	1.828	1.828	1.925	1.925
	($d_e - a/2$)	m	1.828	1.828	1.828	1.925	1.925
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	0.600
ncat	Amount of bars in spacing S	bars	20	20	20	20	20
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	198	345	192	33	97
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e_s	Strain in tensile reinforcement		4.18E-04	6.21E-04	3.32E-04	1.16E-04	1.60E-04
	if $e_s < 0$, multiple with reduce factor		-	-	-	-	-
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok	Ok
v/fc	Ratio of shear stress and f'c		0.007	0.011	0.006	0.001	0.003
β	Final value		2.8	2.4	3.2	4.1	3.9
θ	Final value	degree	28.84	30.94	28.66	27.00	27.42
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	29638	25574	33165	44673	42463
Vs	Shear resistance provided by shear reinforcement	kN	8943	8214	9007	10177	9996
Vp	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0	0
Vn1	$V_{n1} = V_c + V_s + V_p$	kN	38581	33788	42172	54849	52459
Vn2	Vn2	kN	172769	172769	172769	181937	181937
Vn	Nominal shear resistance $V_n = \min(V_{n1}, V_{n2})$	kN	38581	33788	42172	54849	52459
Vr	Factored shear resistance	kN	38581	30409	42172	54849	47213
Vu	Shear	kN	4567	7148	4425	499	2112
(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	
γ _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	
nc	Ratio E _s /E _c		7	

The diagram illustrates the cross-section of a wing wall. Key dimensions include the total height H, top width b, and web width bw. Reinforcement details show tension reinforcement (A's, A's) and prestressing steel (A'ps, Aps) at various depths (d's, d1x, ds, d1xp, dps). A triangular stress distribution is shown on the right with a resultant at a distance a = c/β1 from the top fiber.

Sign	Parameters	Unit	Sections				
			5-5	5-5	6-6	6-6	6-6
INTERNAL FORCES AT SECTION							
Qu	Combination		Service	Strength	Service	Strength	Strength
Qu	Shear	kN	74	116	205	315	315
Mu	Flexural Moment	kNm	172	273	181	278	278
Nu	Axial load	kN	0	0	0	0	0
Tu	Torsional Moment	kNm	0	0	0	0	0
FLEXURAL MOMENT CHECKING							
H	Section height	m	0.500	0.500	0.500	0.500	0.500
d's	Dis. From comp. fiber to centroid of comp. Reinf	m	0.058	0.058	0.058	0.058	0.058
d1x	Dis. From tens. fiber to centroid of tension Reinf	m	0.059	0.059	0.059	0.059	0.059
	Cover to reinf	m	0.050	0.050	0.050	0.050	0.050
ds	Dis. From comp. fiber to centroid of tension Reinf	m	0.441	0.441	0.441	0.441	0.441
d'ps	Dis. From comp. fiber to centroid of comp. prestressing steel	m	0.000	0.000	0.000	0.000	0.000
d1xp	Dis. From tens. fiber to centroid of tension prestressing steel	m	0.000	0.000	0.000	0.000	0.000
dps	Dis. From comp. fiber to centroid of tension prestressing steel	m	0.500	0.500	0.500	0.500	0.500
b	Width of the compression face of member	m	1.000	1.000	1.000	1.000	1.000
bw	Web width or diameter of a circular section	m	1.000	1.000	1.000	1.000	1.000
hf	Compression flange depth	m	0.000	0.000	0.000	0.000	0.000
Iz	Moment of inertia of section	m4	0.010	0.010	0.010	0.010	0.010
Amc	Section area	m2	0.500	0.500	0.500	0.500	0.500
	Steel choice						
Aps	Tension prestressing steel	P.S type	0	0	0	0	0
		Number	tendons	0	0	0	0
		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	22	22	22	22
		Area	m2	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	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	0.020	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

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	0.441	0.441	0.441	0.441	0.441
Min	Nominal resistance	kNm	370	370	370	370	370
Mr	Factored resistance	kNm	370	333	370	333	333
Mu	Flexural moment	kNm	172	273	181	278	278
(5.7.3.2)	Flexural moment Checking		OK	OK	OK	OK	OK
	Limits for reinforcement						
c/de	Maximum reinforcement		0.05	0.05	0.05	0.05	0.05
	Maximum reinforcement Checking	≤ 0.42	OK	OK	OK	OK	OK
r min	Minimum reinforcement		0.46%	0.46%	0.46%	0.46%	0.46%
	Minimum reinforcement Checking for RC	0.23%	OK	OK	OK	OK	OK
1.2*Mc	Cracking moment	kNm	90	90	90	90	90
(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
de	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
fsa	Value	Mpa	285	285	285	285	285
0.6*fy		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	0.104	-	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	186	-	195	-	-
	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)						
Areq	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	1.9	2.1	1.8	1.8
θ	Angle of inclination of diagonal compressive	degree	35.80	41.11	38.01	42.15	42.15
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	90
bv	Effective web width as minimum web width - in dv	m	1.000	1.000	1.000	1.000	1.000
dv	Effective shear depth	m	0.433	0.433	0.433	0.433	0.433
	($d_e - a/2$)	m	0.433	0.433	0.433	0.433	0.433
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	0.600
ncat	Amount of bars in spacing S	bars	3	3	3	3	3
Av	Shear reinf area in spacing S	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	171	298	474	809	809
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
ϵ_s	Strain in tensile reinforcement		9.83E-04	1.53E-03	1.20E-03	1.79E-03	1.79E-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 f_c		0.006	0.010	0.016	0.027	0.027
β	Final value		2.2	1.9	2.1	1.8	1.8
θ	Final value	degree	35.80	41.11	38.01	42.15	42.15
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	440	381	416	357	357
Vs	Shear resistance provided by shear reinforcement	kN	136	112	125	108	108
Vp	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0	0
Vn1	$V_{n1} = V_c + V_s + V_p$	kN	576	493	542	466	466
Vn2	V_{n2}	kN	3245	3245	3245	3245	3245
Vu	Nominal shear resistance $V_u = \min(V_{n1}, V_{n2})$	kN	576	493	542	466	466
Vr	Factored shear resistance	kN	576	444	542	419	419
Vu	Shear	kN	74	116	205	315	315
(5.8.2.7)	Shear checking		OK	OK	OK	OK	OK

DANANG QUANG NGAI EXPRESSWAY ORB25a 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 _{HY} (kN)	Mx (kN•m)
1	Strength Str-IA		0	0	0	0	0
2	Strength Str-IA		4625	787	-1106	-12	-59
3	Strength Str-IA		1697	787	-1106	12	-12
4	Strength Str-IA		4625	787	-1106	-12	-59
5	Strength Str-IA		1697	787	-1106	12	-12
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

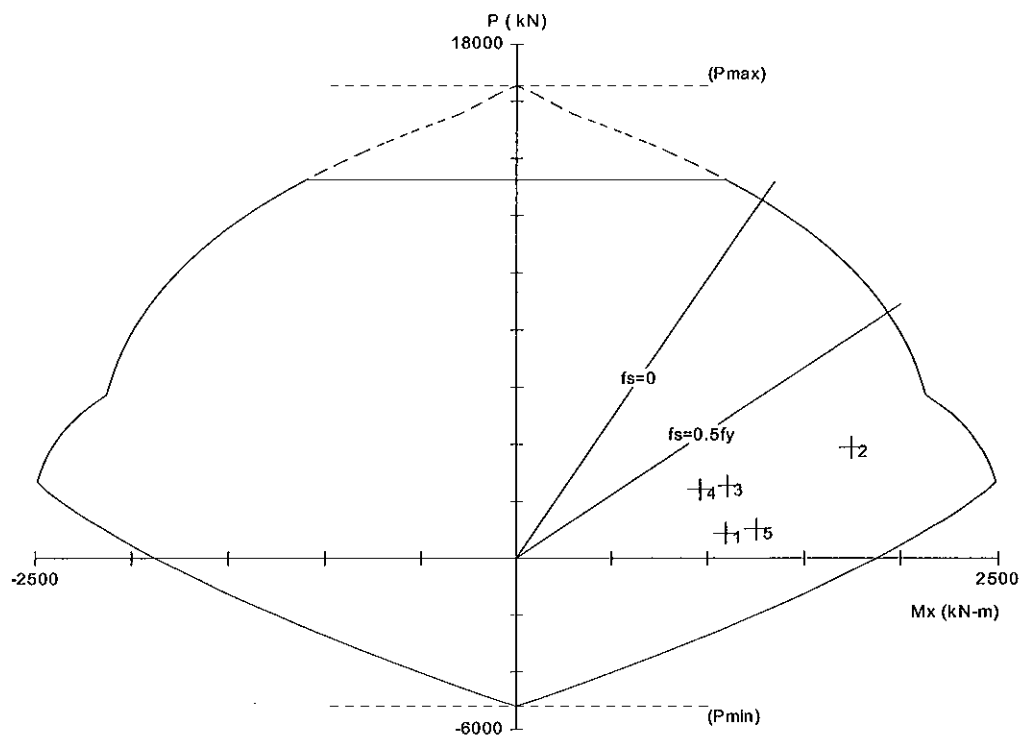
S.5.7.4.2

Minimum area of longitudinal reinforcement in column					
As.fy / (Ag . f _c) >= 0.135	As ≥	0.008	m ²		
As / Ag >= 0.01	As ≥	0.008	m ²		
Maximum area of longitudinal reinforcement in column					
As / Ag <= 0.08	As ≤	0.063	m ²		
Trial Rebars:	Ok As	0.015	m ²		
11 layers x 24 = 24 bars	D28 @150	As1	0.015	m ²	

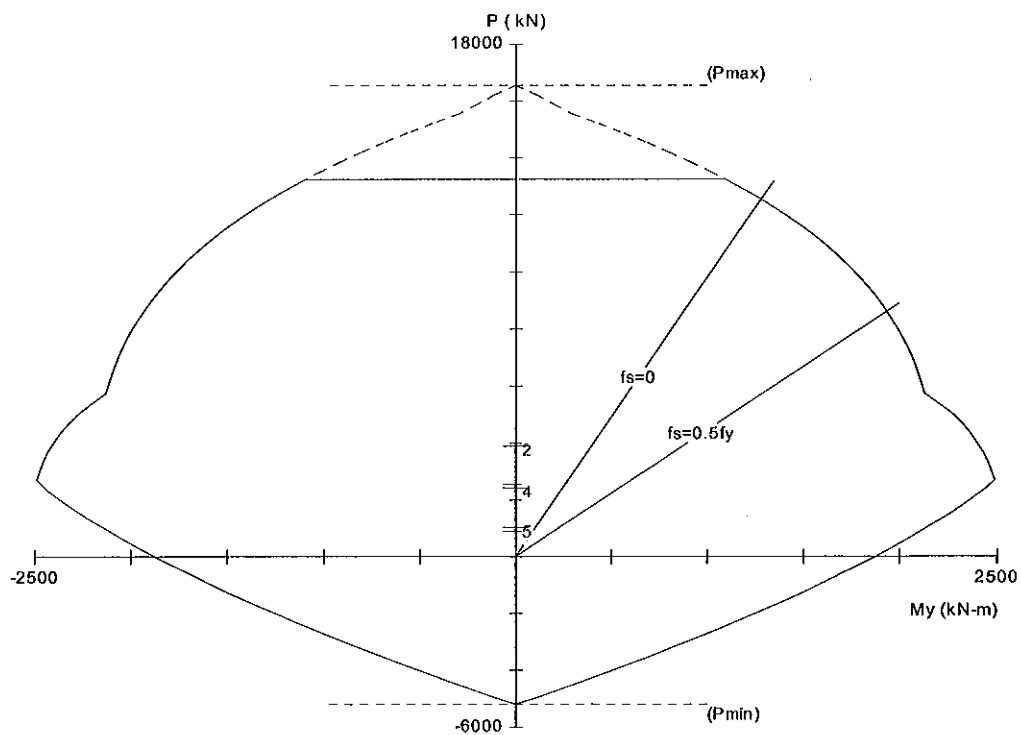
2. Interaction diagram M-P

Using Pca-Column software

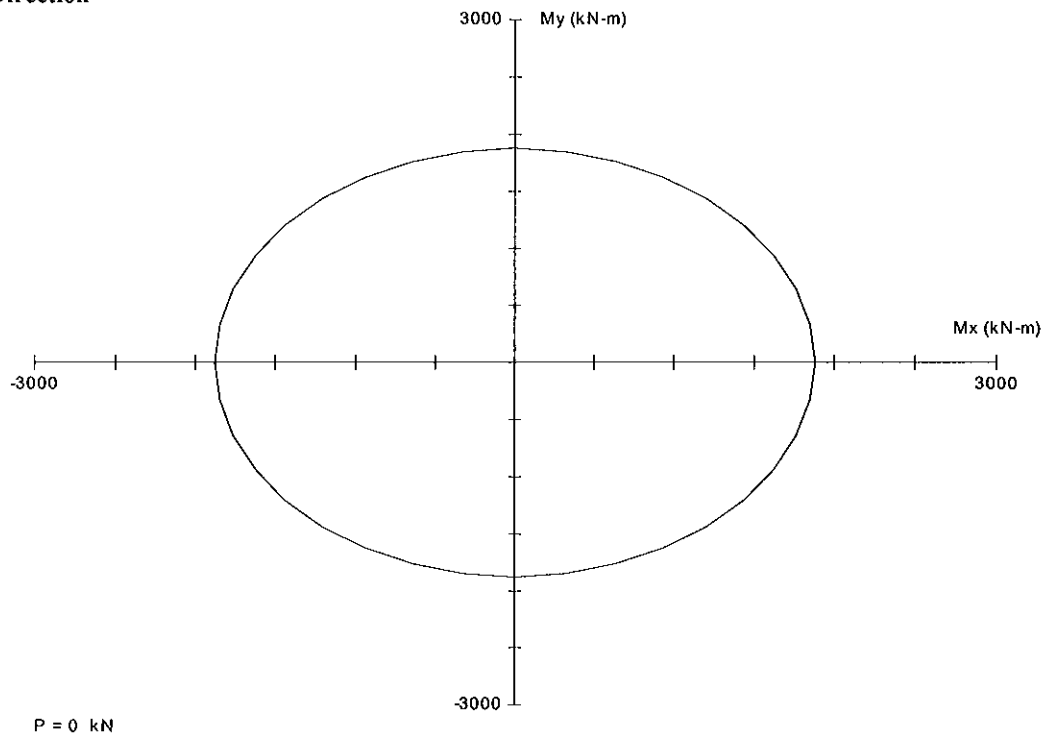
**In Transverse Direction



**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			
$\rho_s = A_{s-tr} \cdot L_{tie} / (A_c \cdot \text{spacing})$	ρ_s	0.0096	
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	0.0084	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	
$\rho_s \geq 0.12 \cdot f_c/f_y = \text{Req2}$	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	787	kN
Required shear capacity $V_n = V_u / \phi_v$	Vn	787	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
$d_v = 0.9 \cdot d_e$	$d_e = D/2 + D_r/\pi()$		

Diameter of the circle passing through the centers of the long. reinf.	Dr	0.79	m
	de	0.75	m
	dv	0.68	m
$V_c = 0.083 \cdot \beta \cdot \sqrt{f_c} \cdot b_v \cdot d_v$	Vc	616	kN
	A _v	1963	mm ²
Angle of inclination of shear reinf. to long. axis	α	90	
$V_s = A_v \cdot f_y \cdot d_v \cdot (\cot \theta + \cot \alpha) \cdot \sin \alpha / s$	Vs	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

SPACE PILE FOUNDATION ANALYSIS PROGRM
Turbo BASIC

PROJECT: : Cau ORB25a - KM91+140.0

INITIA DATA

Kn = 0.00 Ax = 5.50 By = 12.60 Cz = 2.00
 E v.uon = 2944008 E r.uon = 2944008 E v.nen = 2944008 E r.nen =
 2944008
 Mq = 0 (t/m4) Md = 0 (t/m4) m = 2000 (t/m4)

LOAD COMBINATIONS

COMB.	Hx	Hy	P	Mx	My	Mz
1	462.00	0.00	1746.00	-343.00	930.00	0.00
2	315.00	0.00	1164.00	-324.00	691.00	0.00
3	443.00	5.00	1688.00	-312.00	838.00	0.00
4	296.00	5.00	1106.00	-294.00	599.00	0.00
5	316.00	4.00	1207.00	-239.00	624.00	0.00
6	352.00	32.00	1565.00	-309.00	270.00	0.00
7	352.00	32.00	983.00	-290.00	527.00	0.00

PROPERTIES OF PILES

PILE	Lo	H	Bpx	Bpy	A	B	Cday	Fo	Io	Po	Co	Ct
1	0.00	18.50	1.530	1.530	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						

PILE COORD.

PILE	X	Y	Phi	Xi
1	-1.75	4.66	0.000	0.00
2	1.75	4.66	0.000	0.00
3	1.75	0.09	0.000	0.00
4	1.75	-5.30	0.000	0.00
5	-1.75	-5.30	0.000	0.00
6	-1.75	0.09	0.000	0.00

DISPLACEMENTS

COMB.	X	Y	Z	Fix	Fiy	Fiz
1	0.00750	0.00019	0.004286	-0.000095	0.001251	0.000044
2	0.00519	0.00015	0.002860	-0.000077	0.000891	0.000030
3	0.00712	0.00023	0.004143	-0.000090	0.001163	0.000042
4	0.00481	0.00020	0.002717	-0.000072	0.000804	0.000028
5	0.00512	0.00018	0.002963	-0.000067	0.000847	0.000030
6	0.00512	0.00053	0.003843	-0.000094	0.000657	0.000033
7	0.00547	0.00050	0.002417	-0.000076	0.000831	0.000033

FORCES ON PILES

PILE	COMB.	N	Q2	Q3	M1	M2	M3
1	1	173.00	-80.27	-1.18	0.874	1.257	112.703
	2	112.99	-54.73	-0.80	0.596	1.322	75.375
	3	172.31	-76.97	-1.97	0.838	-0.485	109.454
	4	112.34	-51.43	-1.59	0.560	-0.414	72.126
	5	122.13	-54.90	-1.47	0.598	-0.393	77.399
	6	213.53	-61.16	-6.23	0.666	-8.865	97.181
	7	89.91	-61.16	-6.23	0.666	-9.550	90.552
2	1	471.48	-80.27	1.18	0.874	5.978	112.703
	2	325.66	-54.73	0.80	0.596	4.541	75.375
	3	449.86	-76.97	0.30	0.838	4.042	109.454
	4	304.08	-51.43	-0.08	0.560	2.610	72.126
	5	324.33	-54.90	0.14	0.598	2.836	77.399
	6	370.33	-61.16	-4.43	0.666	-5.267	97.181
	7	288.09	-61.16	-4.43	0.666	-5.953	90.552
3	1	442.00	-77.18	1.18	0.874	5.978	106.538
	2	301.76	-52.63	0.80	0.596	4.541	71.172
	3	421.79	-74.01	0.30	0.838	4.042	103.543
	4	281.55	-49.45	-0.08	0.560	2.610	68.177
	5	303.51	-52.79	0.14	0.598	2.836	73.183
	6	340.99	-58.81	-4.43	0.666	-5.267	92.484
	7	264.34	-58.81	-4.43	0.666	-5.953	85.855
4	1	407.23	-73.55	1.18	0.874	5.978	99.268
	2	273.58	-50.15	0.80	0.596	4.541	66.215
	3	388.68	-70.52	0.30	0.838	4.042	96.572
	4	254.98	-47.12	-0.08	0.560	2.610	63.519
	5	278.96	-50.31	0.14	0.598	2.836	68.210
	6	306.39	-56.04	-4.43	0.666	-5.267	86.945
	7	236.33	-56.04	-4.43	0.666	-5.953	80.316
5	1	108.75	-73.55	-1.18	0.874	1.257	99.268
	2	60.91	-50.15	-0.80	0.596	1.322	66.215
	3	111.13	-70.52	-1.97	0.838	-0.485	96.572
	4	63.24	-47.12	-1.59	0.560	-0.414	63.519
	5	76.75	-50.31	-1.47	0.598	-0.393	68.210
	6	149.58	-56.04	-6.23	0.666	-8.865	86.945
	7	38.15	-56.04	-6.23	0.666	-9.550	80.316
6	1	143.52	-77.18	-1.18	0.874	1.257	106.538
	2	89.09	-52.63	-0.80	0.596	1.322	71.172

3	144.24	-74.01	-1.97	0.838	-0.485	103.543
4	89.81	-49.45	-1.59	0.560	-0.414	68.177
5	101.31	-52.79	-1.47	0.598	-0.393	73.183
6	184.19	-58.81	-6.23	0.666	-8.865	92.484
7	66.16	-58.81	-6.23	0.666	-9.550	85.855

SUMMARY OF FORCES

	PILE	COMB.	N	Q2	Q3	M1	M2	M3
Nmin	5	7	38.15	-56.04	-6.23	0.666	-9.550	80.316
Nmax	2	1	471.48	-80.27	1.18	0.874	5.978	112.703
Q2max	1	1	173.00	-80.27	-1.18	0.874	1.257	112.703
Q3max	1	7	89.91	-61.16	-6.23	0.666	-9.550	90.552
M1max	1	1	173.00	-80.27	-1.18	0.874	1.257	112.703
M2max	1	7	89.91	-61.16	-6.23	0.666	-9.550	90.552
M3max	1	1	173.00	-80.27	-1.18	0.874	1.257	112.703

CHECKING CALCULATI IN COMPARISON WITH INITIA LOAD MATRIX

1	462.00	0.00	1746.00	-343.00	930.00	0.00
2	315.00	0.00	1164.00	-324.00	691.00	0.00
3	443.00	5.00	1688.00	-312.00	838.00	0.00
4	296.00	5.00	1106.00	-294.00	599.00	0.00
5	316.00	4.00	1207.00	-239.00	624.00	0.00
6	352.00	32.00	1565.00	-309.00	270.00	0.00
7	352.00	32.00	983.00	-290.00	527.00	0.00

PILE CAPACITY AND SETTLEMENT OF GROUP PILE

1. DATA

- Code of boreholes	A1
- Pile diameter :	1000 mm
- Elevation of the underground water	EL ₃ = 0.00 m
- Ground elevation after scour	EL ₄ = 0.000 m
- Base bottom elevation	EL ₁ = -2.500 m
- Expected pipe tip elevation	EL ₂ = -25.000 m
- Pipe length	L = 22.5 m
- Perimeter of cross section	P = 3.14 m
- Area of the pile cross section	A _b = 0.79 m ²
- Concrete weight	γ _c = 24.50 kN/m ³
Soil layer at Tip of pile:	3

('1' = cohesionless soil, '2' = cohesive soil, '3' = intermediate material, '4' = rock)

* Reference document

- + Standard 22 TCN 272-05
- + Standard AASHTO LRFD 2007

Legends

* Because the pile foundation of the bridge crosses the soil area with strong weather rock and IGM, so 22TCN272-05 can not meet requirement to base for the calculation of the pile load bearing capacity, then AASHTO 2007 is referred to calculate the process items which standard 22TCN272-05 has not yet mentioned

2. CALCULATION FOR THE BEARING FORCE OF BORED PILES

2.1. SIDEWALL FRICTION q_s:

2.1.1. For cohesionless soil

+ Using Method β : O'Neil and Reese (1999)

$$q_s = \beta \sigma'_v \leq 0.19 \text{ MPa}; \quad \text{with } 0.25 < \beta < 1.2$$

+ For sandy soil:

$$\beta = 1.5 - 7.7 \times 10^{-3} (z)^{0.5} \quad \text{for } N_{60} \geq 15$$

$$\beta = N_{60}/15 [1.5 - 7.7 \times 10^{-3} (z)^{0.5}] \quad \text{for } N_{60} < 15$$

+ For Gravelly sand and gravels and SPT values are more than 50 blows/30cm:

$$\beta = 2 - 0.00082 (z)^{0.5} \quad \text{with } 0.25 < \beta < 1.8$$

With : + z : The depth of the subsoil layer at the middle location of the soil layer

+ N : The average number of hammers SPT (not yet calibrated) along the pile body (hammer/300)

2.1.2. As for cohesive soil, side wall friction is calculated according to the method α

$$q_s = \alpha S_u \quad (10.8.3.3.1-1)$$

In which:

- + The cohesive factor α is calculated according to the method API (10.7.3.3.2a-1)
- + S_u : medium non- drain shear resistance strengthen

* Calculation method S_u:

$$S_u = 0.06 N_{60} (\text{bar}) = 0.006 N_{60} (\text{MPa}) \quad (\text{Terzaghi \& Peck})$$

* Calculation method α: (10.8.3.5.1b AASHTO LRFD 2007)

$$\alpha = 0.55 \quad \text{with } S_u/0.101 \leq 1.5$$

$$\alpha = 0.55 - 0.1(S_u/0.101 - 1.5) \quad \text{with } 1.5 \leq S_u/0.101 \leq 2.5$$

* Calculation method of effective vertical stress s'_v at the center of each soil layer:

$$\sigma'_z = \sigma_{vz} - u_z$$

At the soil layer over the underground water level : $\sigma_v = \gamma \cdot z_i$; $u_z = 0$

At the soil layer under the ground water level : $\sigma_v = \gamma_{\text{sat}} \cdot z_i$; $u_z = z_i$

Suppose: clay layer and water- penetration weathered rock layer

Soil layer placed under the ground water level and obtain the penetration, use the saturated weight

In which

- + σ_v : total stress at the depth z (kN/m²)
- + σ'_v : effective stress at the depth z (kN/m²)
- + u : Pore water pressure (kN/m²)
- + z_i : Depth to the middle point of the i layer counted from the surface (m)
- + l_i : thickness of the i soil layer (m)

+ γ : Natural density of the soil layer above the ground water level (kN/m³)

+ γ_{sat} : the saturated density of the soil layer under the ground water (kN/m³)

$$\gamma_{sat} = \gamma_d(1 - \gamma_w/\gamma_s) + \gamma_w$$

+ γ_d : Dry density (kN/m³)

+ γ_s : Unit weight (kN/m³)

+ γ_w : water density $\gamma_w = 10$ (kN/m³)

+ With: Energy efficiency $E_h = 50$

Name of layer	Layer thickness	Elevation of the layer bottom	depth	Unit weight	Dry density	saturated density	Total stress	Pore water pressure	Effective stress
	li		Z_i	γ_s	γ_d	γ_{sat}	σ_{vz}	u	σ'_z
	(m)	(m)	(m)	(kN/m ³)	(kN/m ³)	(kN/m ³)	(kN/m ²)	(kN/m ²)	(kN/m ²)
2	3.71	-2.50	1.86	18.70	13.90	0.00	0.00	0.00	0.00
3	2.00	-6.21	4.71	18.70	13.90	16.47	77.56	47.10	30.46
3	2.00	-10.21	6.71	18.70	13.90	16.47	110.49	67.10	43.39
3	3.20	-13.41	9.31	18.70	13.90	16.47	153.31	93.10	60.21
4	1.80	-15.21	11.81	18.70	13.90	16.47	194.47	118.10	76.37
5	2.10	-17.31	13.76	18.70	13.90	16.47	226.58	137.60	88.98
6	7.69	-25.00	18.66	25.40	15.00	19.09	356.21	186.55	169.66

Name of layer	Soil type	σ'_z (N/mm ²)	SPT N (Blow/30cm)	$N_{60} =$ $N \cdot E_h/60$	S_u (N/mm ²)	α	β	q_s (N/mm ²)	Resistance factor ϕ_s	value $\phi_s \cdot q_s$ (N/mm)
2	1	0.000	3	3	-	-			0.55	0.00
3	1	0.030	12	10	-	-	0.777	0.024	0.55	26.04
3	3	0.043	15	13	-	-	1.800	0.078	0.60	93.73
3	3	0.060	19	16	-	-	1.800	0.108	0.60	208.07
4	3	0.076	40	33	-	-	1.800	0.137	0.60	148.47
5	3	0.089	50	42	-	-	1.800	0.160	0.60	201.82
6	3	0.170	100	83	-	-	1.800	0.190	0.60	876.66
Total										1554.79

Total resistance of the side wall: $Q_s = \phi_s \phi_l A_s = \Sigma \phi_s \phi_l P = 4884.5$ (Kn)

ϕ_s : Shaft resistance factor of the pile in the ground (according to the table 10.5.5.2.4-1 AASHTO 2007)

2.2. PILE TIP RESISTANCE q_p :

Soil layer at Tip of pile: IGM

2.2.1. Pile tip resistance: Method Reese and O'Neill (1999)

- Nominal Tip resistance follow:

$$q_p = 0.057N \quad \text{with } N \leq 50$$

$$q_p = 0.59\sigma'_v((N60(Pa/\sigma'_v))^0.8 \quad \text{with } N > 50$$

In which

$N = 100$ (hammer /300mm)-Number of hammer SPT-N has not been calibrated

$D = 1000$ (mm) Diameter of bore pile

$D_p = 1000$ (mm) diameter of bore pipe

$\sigma'_v = 0.17$ (Mpa) effective vertical prestress

=> Nominal tip resistance $q_p = 2.632$ (Mpa)

- Tip resistance of pile:

$$Q_b = \phi \cdot q_p \cdot A_b = 1136.78 \text{ (kN)}$$

With tip resistance factor : $\phi = 0.55$ (Table 10.5.5.2.4-1 AASHTO2007)

2.3. Pile capacity:

- Design resistance of the single pile :

$$Q_T = (Q_s + Q_p) = 6021.3 \text{ kN}$$

- Resistance of the resisting pile according to the pile group:

$$Q_R = \eta \cdot Q_T = 5065.91 \text{ kN}$$

In which

$$+ \eta = 0.84 \quad \text{Effective factor due to the pile working by group} \quad (\text{AASHTO 2007 - 10.8.3.6.3})$$

$$+ d = 3.32 \text{ (m)} \quad \text{The distance from the center to center of pile.}$$

2.4. The Uplift resistance of pile :

Name of layer	Soil type	σ'_z (N/mm ²)	SPT N (Blow/30cm)	$N_{60} =$ $N \cdot E_p / 60$	S_u (N/mm ²)	α	β	q_s (N/mm ²)	Resistance factor ϕ_s	value $\phi_s \cdot q_s$ (N/mm)
2	1	0.00	3	3	0.00	0.00	0.00	0.00	1.00	0.00
3	1	0.03	12	10	0.00	0.00	0.78	0.02	1.00	-47.35
3.00	3	0.04	15	13	0.00	0.00	1.80	0.08	1.00	-156.21
3.00	3	0.06	19	16	0.00	0.00	1.80	0.11	1.00	-346.79
4.00	3	0.08	40	33	0.00	0.00	1.80	0.14	1.00	-247.45
5.00	3	0.09	50	42	0.00	0.00	1.80	0.16	1.00	-336.36
6.00	3	0.17	100	83	0.00	0.00	1.80	0.19	1.00	-1461.10
Total										-2595.26

Total uplift resistance of the side wall:

$$Q_s = \phi_s \phi_l A_s = \Sigma \phi_s \phi_l P = -8153.2 \text{ (Kn)}$$

ϕ_s : Shaft resistance factor of the pile in the ground (according to the table 10.5.5.2.4-1 AASHTO 2007)

- Design resistance of the single pile :

$$Q_U = Q_s = -8153.2 \text{ kN}$$

- Uplift Resistance of the resisting pile according to the pile group:

$$Q_R = \eta \cdot Q_T = -6859.60 \text{ kN}$$

In which

$$+ \eta = 0.84 \quad \text{Effective factor due to the pile working by group} \quad (\text{AASHTO 2007 - 10.8.3.6.3})$$

$$+ d = 3.32 \text{ (m)} \quad \text{The distance from the center to center of pile.}$$

2.4. Conclusion

Internal force of pile		Seftweight of Pile W	Factored force of pile		Pile capacity		Check	
PMax	PMin		QMax	QMin	Nominal resistance	Uplift resistance		
(KN)	(KN)	(KN)	(KN)	(KN)	Qr (KN)	Qu (KN)	Qmax<Qr	Qmin>Qu
4714.8	381.5	256	4971	638	5065.91	-6859.60	OK	OK

3. THE SETTLEMENT OF PILE GROUP:

- The Settlement of pile groups in cohesionless soils using SPT value:

$$\rho = \frac{30qI\sqrt{B}}{N_{60}} \quad (\text{mm}) \quad (10.7.2.3.3-1)$$

In which: - q: net foundation pressure applid at $2D_b/3$, this pressure is equal to the applied load at

the top of group divided by the area of the equivalent footing and does not include the weight of the piles or soil between the piers (Mpa).

- B : Width or smallest dimensions of pile group (mm).

- I : Influence factor of the effective group embedment (dim)

$$I = 1 - 0.125D'/B$$

- D' : Effective depth taken as $2D_b/3$, (mm)

- D_b : depth of embedment of piles in layer that provides support, (mm)

Nmax	Equivalent area	B	D_b	D'	I	N_{60}	q	ρ	Check
(Kn)	(mm ²)	(mm)	(mm)	(mm)	(dim)	blow	(Mpa)	(mm)	$\rho < 25.4$
47577.86	2.53E+08	5800	11590	7726.67	0.83	83	0.19	4.30	Ok

PILE CAPACITY AND SETTLEMENT OF GROUP PILE

1. DATA

- Code of boreholes	A2
- Pile diameter :	1000 mm
- Elevation of the underground water	$EL_3 = 0.00$ m
- Ground elevation after scour	$EL_4 = 0.000$ m
- Base bottom elevation	$EL_1 = -2.500$ m
- Expected pipe tip elevation	$EL_2 = -25.000$ m
- Pipe length	$L = 22.5$ m
- Perimeter of cross section	$P = 3.14$ m
- Area of the pile cross section	$A_b = 0.79$ m ²
- Concrete weight	$\gamma_c = 24.50$ kN/m ³
Soil layer at Tip of pile:	3
('1' = cohesionless soil, '2' = cohesive soil, '3' = intermediate material, '4' = rock)	

* Reference document

- + Standard 22 TCN 272-05
- + Standard AASHTO LRFD 2007

Legends

* Because the pile foundation of the bridge crosses the soil area with strong weather rock and IGM, so 22TCN272-05 can not meet requirement to base for the calculation of the pipe load bearing capacity, then AASHTO 2007 is referred to calculate the process items which standard 22TCN272-05 has not yet mentioned

2. CALCULATION FOR THE BEARING FORCE OF BORED PILES

2.1. SIDEWALL FRICTION q_s :

2.1.1. For cohesionless soil

+ Using Method β : O'Neil and Reese (1999)

$$q_s = \beta \sigma'_v \leq 0.19 \text{ MPa}; \quad \text{with } 0.25 < \beta < 1.2$$

+ For sandy soil:

$$\beta = 1.5 - 7.7 \times 10^{-3} (z)^{0.5} \quad \text{for } N_{60} \geq 15$$

$$\beta = N_{60} / 15 [1.5 - 7.7 \times 10^{-3} (z)^{0.5}] \quad \text{for } N_{60} < 15$$

+ For Gravelly sand and gravels and SPT values are more than 50 blows/30cm:

$$\beta = 2 - 0.00082 (z)^{0.5} \quad \text{with } 0.25 < \beta < 1.8$$

With : + z : The depth of the subsoil layer at the middle location of the soil layer

+ N : The average number of hammers SPT (not yet calibrated) along the pile body (hammer/300)

2.1.2. As for cohesive soil, side wall friction is calculated according to the method α

$$q_s = \alpha S_u \quad (10.8.3.3.1-1)$$

In which:

- + The cohesive factor α is calculated according to the method API (10.7.3.3.2a-1)
- + S_u : medium non- drain shear resistance strengthen

* Calculation method S_u :

$$S_u = 0.06 * N_{60} \text{ (bar)} = 0.006 * N_{60} \text{ (MPa)} \quad (\text{Terzaghi \& Peck})$$

* Calculation method α : (10.8.3.5.1b AASHTO LRFD 2007)

$$\alpha = 0.55 \quad \text{with } S_u / 0.101 \leq 1.5$$

$$\alpha = 0.55 - 0.1 (S_u / 0.101 - 1.5) \quad \text{with } 1.5 \leq S_u / 0.101 \leq 2.5$$

* Calculation method of effective vertical stress σ'_v at the center of each soil layer:

$$\sigma'_z = \sigma_{vz} - u_z$$

At the soil layer over the underground water level : $\sigma_v = \gamma \cdot z_i$; $u_z = 0$

At the soil layer under the ground water level : $\sigma_v = \gamma_{sat} \cdot z_i$; $u_z = z_i$

Suppose: clay layer and water- penetration weathered rock layer

Soil layer placed under the ground water level and obtain the penetration, use the saturated weight

In which

+ σ_v : total stress at the depth z (kN/m²)

+ σ'_v : effective stress at the depth z (kN/m²)

+ u : Pore water pressure (kN/m²)

+ z_i : Depth to the middle point of the i layer counted from the surface (m)

+ l_i : thickness of the i soil layer (m)

+ γ : Natural density of the soil layer above the ground water level (kN/m³)

+ γ_{sat} : the saturated density of the soil layer under the ground water (kN/m³)

$$\gamma_{sat} = \gamma_d (1 - \gamma_w / \gamma_s) + \gamma_w$$

γ_d : Dry density (kN/m³)
 γ_s : Unit weight (kN/m³)
 γ_w : water density (kN/m³)
 $\gamma_w = 10$
 + With: Energy efficiency $E_h = 50$

Name of layer	Layer thickness	Elevation of the layer bottom	depth	Unit weight	Dry density	saturated density	Total stress	Pore water pressure	Effective stress
	li		Z_i	γ_s	γ_d	γ_{sat}	σ_{vz}	u	σ'_z
	(m)	(m)	(m)	(kN/m ³)	(kN/m ³)	(kN/m ³)	(kN/m ²)	(kN/m ²)	(kN/m ²)
2	0.88	-2.50							
3	2.00	-3.38	0.44	18.70	13.90	0.00	0.00	0.00	0.00
4	2.60	-5.38	1.88	18.70	13.90	16.47	30.96	18.80	12.16
4	3.00	-7.98	4.18	18.70	13.90	16.47	68.83	41.80	27.03
5	5.00	-10.98	6.98	18.70	13.90	16.47	114.94	69.80	45.14
5	4.60	-15.98	10.98	18.70	13.90	16.47	180.81	109.80	71.01
5	4.60	-20.58	15.78	18.70	13.90	16.47	259.85	157.80	102.05
6	4.42	-25.00	20.29	25.40	15.00	19.09	387.43	202.90	184.53

Name of layer	Soil type	σ'_z (N/mm ²)	SPT N (Blow/30cm)	$N_{60} =$ $N \cdot E_h / 60$	S_u (N/mm ²)	α	β	q_s (N/mm ²)	Resistance factor ϕ_s	value $\phi_s \cdot q_s$ (N/mm)
2	1	0.000	5	4	-	-			0.55	0.00
3	1	0.012	10	8	-	-	0.777	0.009	0.55	10.40
4	3	0.027	17	14	-	-	1.800	0.049	0.60	75.90
4	3	0.045	20	17	-	-	1.800	0.081	0.60	146.25
5	3	0.071	27	23	-	-	1.800	0.128	0.60	383.43
5	3	0.102	50	42	-	-	1.800	0.184	0.60	506.97
6	3	0.185	100	83	-	-	1.800	0.190	0.60	503.88
Total										1626.83

Total resistance of the side wall: $Q_s = \phi_s \phi_r A_s = \sum \phi_s \phi_r P = 5110.8$ (Kn)

ϕ_s : Shaft resistance factor of the pile in the ground (according to the table 10.5.5.2.4-1 AASHTO 2007)

2.2. PILE TIP RESISTANCE q_p :

Soil layer at Tip of pile: IGM

2.2.1. Pile tip resistance: Method Reese and O'Neill (1999)

- Nominal Tip resistance follow:

$$q_p = 0.057N \quad \text{with } N \leq 50$$

$$q_p = 0.59\sigma'_v \left((N60(Pa/\sigma'_v))^{0.8} \right) \quad \text{with } N > 50$$

In which

$N = 100$ (hammer /300mm)-Number of hammer SPT-N has not been calibrated

$D = 1000$ (mm) Diameter of bore pile

$D_p = 1000$ (mm) diameter of bore pipe

$\sigma'_v = 0.18$ (Mpa) effective vertical prestress

=> Nominal tip resistance $q_p = 2.676$ (Mpa)

- Tip resistance of pile:

$$Q_b = \phi \cdot q_p \cdot A_b = 1156.04 \text{ (kN)}$$

With tip resistance factor : $\phi = 0.55$ (Table 10.5.5.2.4-1 AASHTO2007)

2.3. Pile capacity:

- Design resistance of the single pile :

$$Q_T = (Q_s + Q_p) = 6266.9 \text{ kN}$$

- Resistance of the resisting pile according to the pile group:

$$Q_R = \eta \cdot Q_T = 5272.54 \text{ kN}$$

In which

+ $\eta = 0.84$ Effective factor due to the pile working by group (AASHTO 2007 - 10.8.3.6.3)

+ $d = 3.32$ (m) The distance from the center to center of pile.

2.4. The Uplift resistance of pile :

Name of layer	Soil type	σ'_z (N/mm ²)	SPT N (Blow/30cm)	$N_{60} =$ $N \cdot E_n/60$	S_u (N/mm ²)	α	β	q_s (N/mm ²)	Resistance factor ϕ_s	value $\phi_s \cdot q_s$ (N/mm)
2	1	0.00	5	4	0.00	0.00	0.00	0.00	1.00	0.00
3	1	0.01	10	8	0.00	0.00	0.78	0.01	1.00	-18.90
4.00	3	0.03	17	14	0.00	0.00	1.80	0.05	1.00	-126.51
4.00	3	0.05	20	17	0.00	0.00	1.80	0.08	1.00	-243.75
5.00	3	0.07	27	23	0.00	0.00	1.80	0.13	1.00	-639.05
5.00	3	0.10	50	42	0.00	0.00	1.80	0.18	1.00	-844.95
6.00	3	0.18	100	83	0.00	0.00	1.80	0.19	1.00	-839.80
Total										-2712.96

Total uplift resistance of the side wall: $Q_s = \phi_s \phi_i A_s = \Sigma \phi_s \phi_i P = -8523.0$ (Kn)

ϕ_s : Shaft resistance factor of the pile in the ground (according to the table 10.5.5.2.4-1 AASHTO 2007)

- **Design resistance of the single pile :**

$$Q_U = Q_s = -8523.0 \text{ kN}$$

- **Uplift Resistance of the resisting pile according to the pile group:**

$$Q_R = \eta \cdot Q_T = -7170.70 \text{ kN}$$

In which

- + $\eta = 0.84$ Effective factor due to the pile working by group (AASHTO 2007 - 10.8.3.6.3)
+ $d = 3.32$ (m) The distance from the center to center of pile.

2.4. Conclusion

Internal force of pile		Seftweight of Pile W (KN)	Factored force of pile		Pile capacity		Check	
PMax (KN)	PMin (KN)		QMax (KN)	QMin (KN)	Nominal resistance Qr (KN)	Uplift resistance Qu (KN)		
4714.8	381.5	256	4971	638	5272.54	-7170.70	OK	OK

3. THE SETTLEMENT OF PILE GROUP:

- The Settlement of pile groups in cohesionless soils using SPT value:

$$p = (30qI\sqrt{B})/N_{60} \quad (\text{mm}) \quad (10.7.2.3.3-1)$$

In which: - q : net foundation pressure applid at $2D_b/3$, this pressure is equal to the applied load at the top of group divided by the area of the equivalent footing and does not include the weight of the piles or soil between the piers (Mpa).

- B : Width or smallest dimensions of pile group (mm).
- I : Influence factor of the effective group embedment (dim)

$$I = 1 - 0,125D'/B$$

- D' : Effective depth taken as $2D_b/3$, (mm)
- D_b : depth of embedment of piles in layer that provides support, (mm)

Nmax (Kn)	Equivalent area (mm ²)	B (mm)	D_b (mm)	D' (mm)	I (dim)	N_{60} blow	q (Mpa)	p (mm)	Check $p < 25.4$
47577.86	2.87E+08	5800	14020	9346.67	0.80	83	0.17	3.63	Ok

DANANG QUANG NGAI EXPRESSWAY ORB25a 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	Strength Str-IA		0	0	0	0	0
2	Strength Str-IA		4478	781	-1106	-11	-55
3	Strength Str-IA		1638	781	-1106	11	-9
4	Strength Str-IA		4478	781	-1106	-11	-55
5	Strength Str-IA		1638	781	-1106	11	-9
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

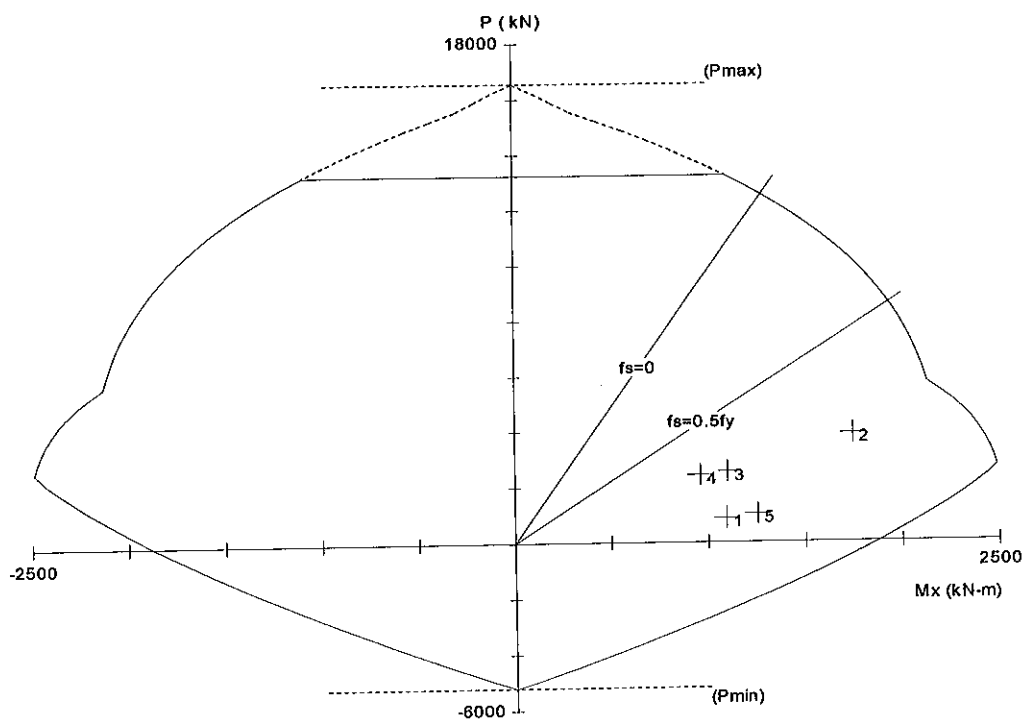
1. Limit of Reinforcement

S.5.7.4.2

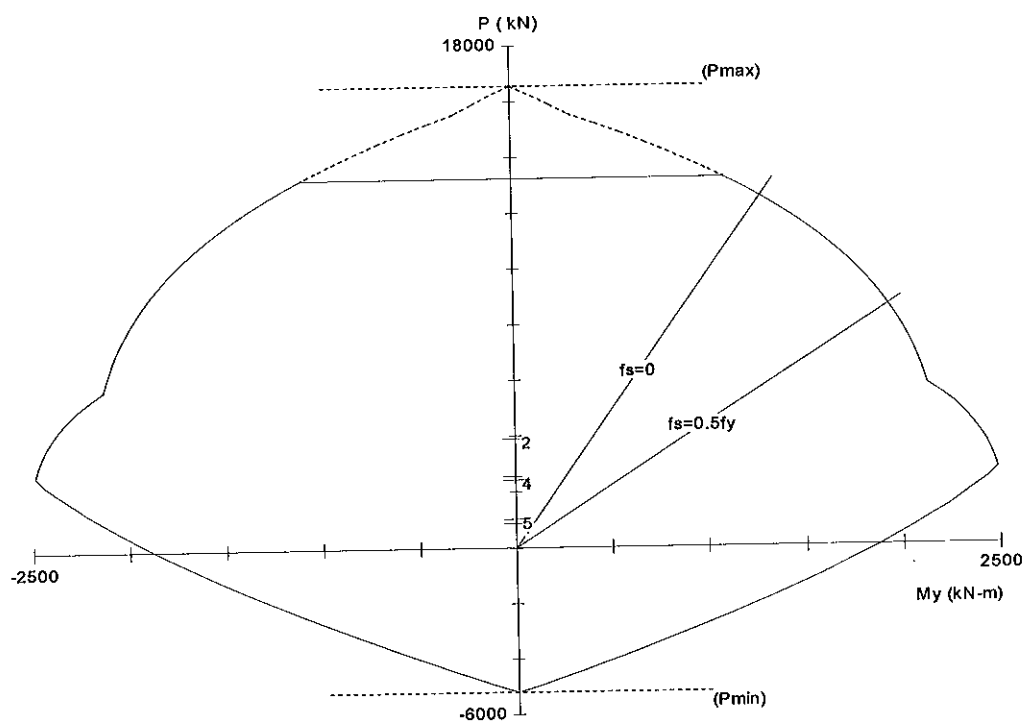
Minimum area of longitudinal reinforcement in column		
As.fy / (Ag . f _c) >= 0.135	As ≥	0.008 m ²
As / Ag >= 0.01	As ≥	0.008 m ²
Maximum area of longitudinal reinforcement in column		
As / Ag <= 0.08	As ≤	0.063 m ²
Trial Rebars:	Ok As	0.015 m ²
11 layers x 24 = 24 bars D28 @150 As1		0.015 m ²

2. Iteration diagram M-P

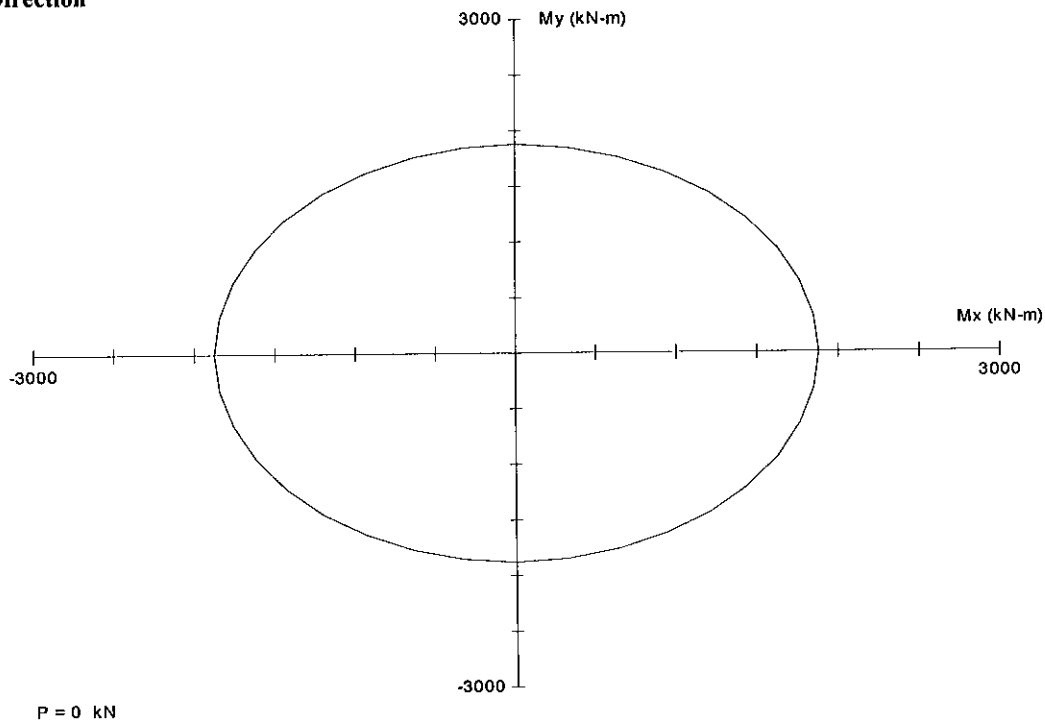
****In Transverse Direction**



****In Longitudinal Direction**



****In Both Direction**



3. Column Ties

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

Bridge is in seismic zone	Sz	1	
Area of concrete core measured out-to-out of ties	Ac	0.622	m ²
Tie diameter	Dtie	14	mm
Cross section area of 1 tie	As-tr	0.0002	m ²
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
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	0.0086	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	781	kN
Required shear capacity $Vn = Vu / \phi_v$	Vn	781	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

9 CB25

Da Nang Quang Ngai Expressway project

BRIDGE
CB25

CALCULATION SHEETS
ABUTMENT A2

Table of content

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

	Da Nang Quang Ngai Expressway project CB25 BRIDGE DETAIL DESIGN ABUTMENT A2	Item.	Eng.	Date.	Sign.
		Design			
		Check			
		Revise			

LOAD COMPONENTS

Assumptions :

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

Input :

Level Table(at center of abutment)

Level of top of headwall	HTwL	7.271	m
Level of top of bearing	BTL	5.146	m
Level of top of stem abutment	HTL	4.996	m
Level of top of footing	FTL	1.500	m
Level of bottom of footing	FBL	-0.500	m
Ground level	GL	2.20	m
Lowest water level	HWL	-0.500	m
Skew angle	α	0.00	deg

Material Unit Weights

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

γ_c	=	24.5 kN/m ³
γ_s	=	18.0 kN/m ³
γ_{sbo}	=	8.2 kN/m ³
γ_a	=	22.1 kN/m ³

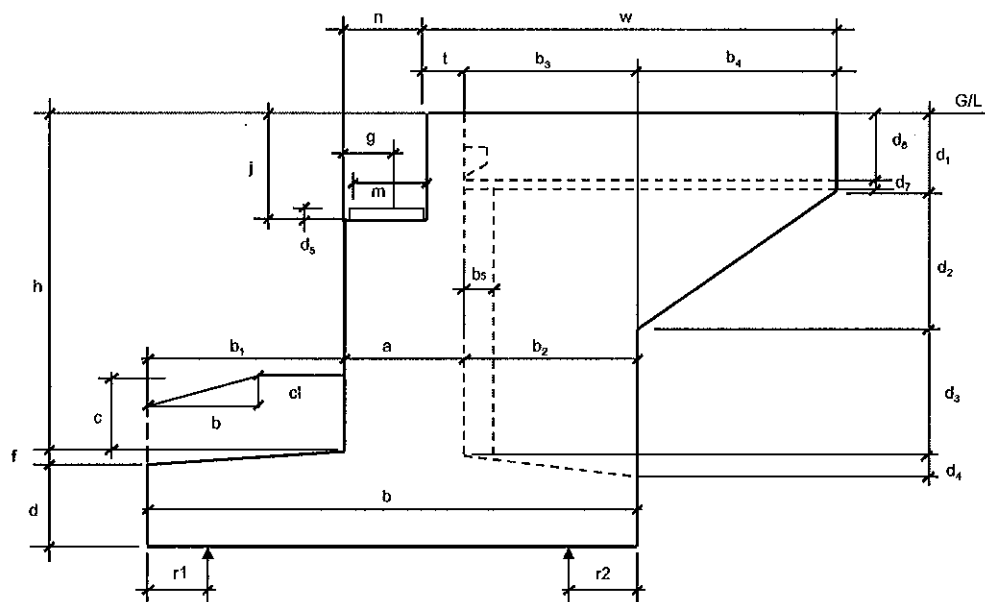
I.Loads from substructure

Abutment dimensions

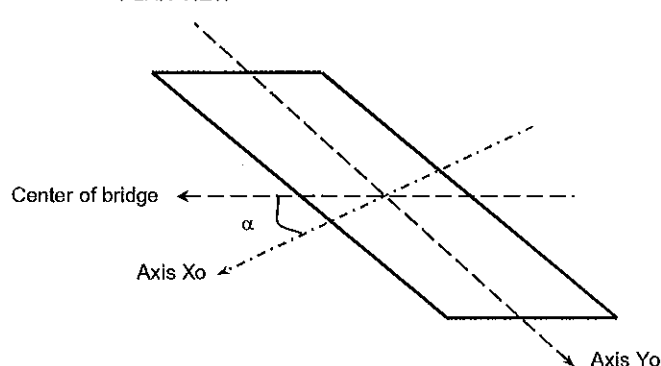
VERTICAL VIEW

Bearing Type:

FIX



PLAN VIEW



	Da Nang Quang Ngai Expressway project CB25 BRIDGE DETAIL DESIGN ABUTMENT A2	Item.	Eng.	Date.	Sign.
		Design			
		Check			
		Revise			

ABUTMENT DIMENSIONS (IN METRES)					
Item	Symbol	Value	Item	Symbol	Value
Height of stem	h	5.771	Horizontal Dimension	b ₄	1.800
Footling Width	b	5.500	Horizontal Dimension	b ₅	0.300
Stem Width	a	1.500	Vertical Dimension	d ₁	0.930
Footling Depth	d	2.000	Vertical Dimension	d ₂	1.800
Footling Slope	f	0.000	Vertical Dimension	d ₃	3.041
Width of stem at bearing	n	1.000	Vertical Dimension	d ₄	
Ballast Wall Height	j	2.275	Vertical Dimension	d ₅	0.150
Ballast Wall Thickness	t	0.500	Vertical Dimension	d ₆	1.070
Wingwall Length	w	4.500	Vertical Dimension	d ₇	
Soil Cover at Toe	c	0.700	With of bearing pad	m	0.800
Girder Reaction	g	0.550	Wingwall Thickness	u1	0.500
Horizontal Dimension	b ₁	1.800	Wingwall Thickness	u2	0.500
Horizontal Dimension	b ₂	2.200	Distance to cl of pile	r1	1.000
Horizontal Dimension	b ₃	2.200	Distance to cl of pile	r2	1.000

Slope front of abutment

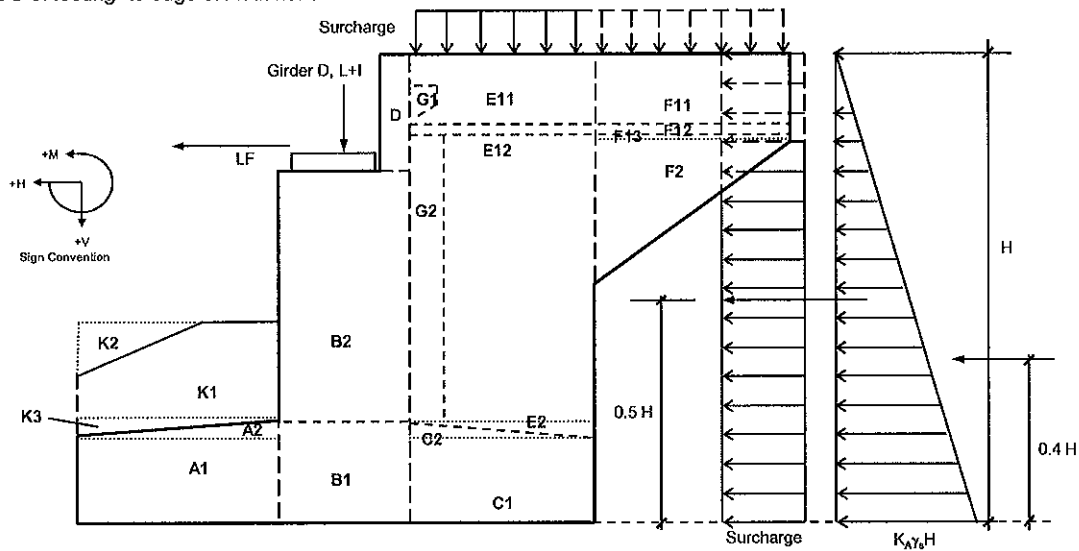
Width of Abutment

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	=	7.77 m
b/2	=	2.75 m



1. Self weight of Abutment (DC)

Description	Area (m ²)	Length (m)	Force (kN)	X ⁽¹⁾ (m)	Arm ⁽²⁾ (m)	Moment (kN•m)
SW of Abutment (DC)						
Section A1	3.600	12.600	1111	0.900	1.850	2056
Section A2	-	12.600	-	1.200	1.550	-
Section B1	3.000	12.600	926	2.550	0.200	185
Section B2	5.244	12.600	1619	2.550	0.200	324
Section C1	4.400	12.600	1358	4.400	-1.650	-2241
Section C2	-	12.600	-	4.033	-1.283	-
Section D	1.138	12.600	351	3.050	-0.300	-105
Section E11	2.046	0.500	25	4.400	-1.650	-41
Section E12	10.650	0.500	130	4.400	-1.650	-215
Part extra stem	-	-	-	5.017	-2.267	-
Section F11	1.926	0.500	24	6.400	-3.650	-86
Section F12	-	0.500	-	5.300	-2.550	-
Section F13	-0.252	0.500	-3	6.400	-3.650	11
Section F2	1.620	0.500	20	6.100	-3.350	-66
Section G1	0.135	11.600	38	3.450	-0.700	-27
Section G2	0.045	5.771	6	3.450	-0.700	-4
Bearing seats (w1seat= 0.65m)	0.120	3.250	10	2.350	0.400	4
Curbs +Handrail on Abutment	0.50	4.500	55	5.050	-2.300	-127
Total SW of Abutment (DC)			5671			-334
Transverse moment			40		6.175	249

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.

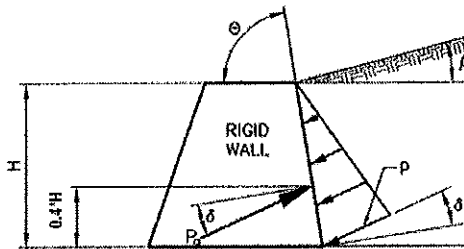
	Da Nang Quang Ngai Expressway project	Item.	Eng.	Date.	Sign.
	CB25 BRIDGE	Design			
	DETAIL DESIGN	Check			
	ABUTMENT A2	Revise			

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	12.70	12.100	2765	4.400	-1.650	-4563
Section E2	-	11.600	-	4.767	-2.017	-
Section E3	-	1.000	-	5.500	-2.750	-
Section K1	1.260	12.600	286	0.900	1.850	-
Section K2	-	12.600	-	-	2.750	-
Section K3	-	12.600	-	0.600	2.150	-
Total Earth on Footing			3051			-4563

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.



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

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

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

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

H	=	7.77 m
W	=	12.6 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	=	0.047 Mpa

Horizontal earth pressure:

- $E_a = 0.5 \cdot p \cdot Z \cdot B \cdot 10^3$ (kN)
- $M = E_a \cdot 0.4H$

E_a	=	2283 kN
M	=	7095 kNm

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

Equivalent height of soil for highway loading taken from Table 3.11.6.2.1

H=	1.50m	heq=	1.7 m
H=	3.00m	heq=	1.2 m
H=	6.00m	heq=	0.76 m
H=	9.00m	heq=	0.61 m
H=	7.77m	heq=	0.67 m

(Linear interpolation)

• Vertical force

ESv	=	335 kN
ev	=	-1.65 m
M	=	-553 kNm

• Horizontal force

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

ESh	=	394 kN
eh	=	3.89 m
M	=	1533 kNm

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5. Earthquake effects

Bridge is located at:

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

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

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 $kv = 0.035$
- Angle $\theta = \arctan(k_v / (1 - k_v)) = 5.2 \text{ deg}$

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

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

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

• Seismic active lateral Earth pressure coefficient

$E_{AE} = 2576 \text{ kN}$

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

$M_{AE} = 6691 \text{ KNm}$

<A.11.1.1.1>

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

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 $2.5A = 0.145$
- $C_{sm} = 1.2 \cdot A \cdot S / T_m^{2/3} \leq 2.5 \cdot A$ $C_{sm} = 0.098$
- Period of vibration of the fundamental mode $T_m = 2 \cdot \pi \cdot () \cdot \sqrt{m/k}$ $T_m = 0.597 \text{ s}$

Decription	Area (m ²)	Length (m)	Force (kN)	$X^{(1)}$ (m)	Arm ⁽²⁾ (m)	Moment (kN•m)
Section A1	3.600	12.600	109	-	1.000	109
Section A2	-	12.600	-	-	2.000	-
Section B1	3.000	12.600	91	-	1.000	91
Section B2	5.244	12.600	159	-	3.748	595
Section C1	4.400	12.600	133	-	1.000	133
Section C2	-	12.600	-	-	2.000	-
Section D	1.138	12.600	34	-	6.634	229
Section E11	2.046	0.500	2	-	5.236	13
Section E12	10.650	0.500	13	-	2.351	-
Section E2	-	-	-	-	2.000	-
Section F11	1.926	0.500	2	-	5.236	12
Section F12	-	0.500	-	-	4.701	-
Section F13	-0.252	0.500	-0	-	5.841	-
Section F2	1.620	0.500	2	-	6.241	12
Section G1	0.135	11.600	4	-	5.058	19
Section G2	0.045	5.771	1	-	2.351	1
Total EQ of Abutment Selfweight			550			1215

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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	=	9.6 m	
Mlong	=	991 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

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

V	=	120 km/h	
V	=	33.3 m/s	
R	=	10500.000 m	
C	=	0.014	
CE	=	11.92 KN	
e	=	7.57 m	
Mtrans	=	90.24 KNm	Trans. Axis

8. Water Load (WA) :NA

8.1. Buoyancy of Abutment

- Highest water Level +0.50

Description	Area (m ²)	Length (m)	Force (kN)	X ⁽¹⁾ (m)	Arm ⁽²⁾ (m)	Moment (kN•m)
Bouyancy on abutment						
Section A1	-	12.600	-	0.900	1.850	-
Section A2	-	12.600	-	-	2.750	-
Section B(B1,B2)	-	12.600	-	2.550	0.200	-
Section C1	-	12.600	-	4.400	-1.650	-
Section C2	-	12.600	-	-	2.750	-
Section E2	-	1.000	-	-	2.750	-
Section E1	-	1.000	-	4.400	-1.650	-
Section F2	-	1.000	-	3.820	-1.070	-
Total Bouyancy			-			-

8.2 Buoyancy of Earth on Abutment

Description	Area (m ²)	Length (m)	Force (kN)	X ⁽¹⁾ (m)	Arm ⁽²⁾ (m)	Moment (kN•m)
Bouyancy of earth on abutment						
Section E2	-	11.600	-	-	2.750	-
Section E1	-	11.600	-	4.400	-1.650	-
Section K2	-	12.600	-	-	2.750	-
Section K1	-	12.600	-	0.900	1.850	-
- Section K3	-	12.600	-	-	2.750	-
Total Bouyancy			-			-

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SUPERSTRUCTURE LOADS

II. Loads from superstructure

Item	Sign	Value	Unit
Span length	Lst	33.00	m
Span between bearings	Ls	32.20	m
Bridge Width	W	12.60	m
Number of girders	n_g	5.00	Girders
Girder height	Hg	1.60	m
Deck slab depth	Hd	0.247	m
Asphalt depth	H α σ	0.084	m

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

Item	Sign	Value	Unit
1.1. Girders			
Sum of girders weight	DC	3347.93	kN
Precast Planks	DC	473.46	kN
Diaphragm	DC	380.73	kN
Total	DC	4202.1	kN
1.2. Deck slab			
Deck slab	DC	2464.3	kN
1.3. Pavement			
Asphalt concrete	DW	649.4	kN
1.4. Parapet			
Parapet + median	DC	889.4	kN

2. Live load (LL):

2.1. Live load

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

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

Truck	P1(kN)	P2(kN)	P3(kN)	Sum(kN)	Total(kN)
Axle load	145	145	35		
Influence value	1.000	0.866	0.733		
Reaction	145.0	125.6	25.7	296.3	944.4
Tandem	P1(kN)	P2(kN)		Sum(kN)	Total(kN)
Axle load	110	110			
Influence value	1.000	0.963			
Reaction	110	105.9		215.9	688.2
Lane load	Wl(kN/m)				Total(kN)
Value	9.3				
Influence value	16.1				
Reaction	149.7				381.8
Pedestrian	Wdb(kN)				Total(kN)
Reaction	0.0				0.0

3. Earthquake effects on superstructure (EQ)

Force from superstructure due to EQ

$$\text{EQ} = 403 \text{ kN}$$

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

Bearing displacement due to uniform temperature and shrinkage creep

Shear modulus G

Bearing area

Height of elastomeric layers

Number of bearing

Horizontal force due to TU+SH&CR

Acting at top of bearing $H = G \cdot A \cdot \Delta u / h_n$

$$\begin{aligned} \Delta u &= 0.026 \text{ m} \\ G &= 1 \text{ MPa} \\ A &= 0.175 \text{ m}^2 \\ h_n &= 0.084 \text{ m} \\ n_b &= 5 \text{ bears} \\ H_x &= 271 \text{ kN} \end{aligned}$$

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

Transverse wind load

$$P_D = \max(0.0006V^2 \cdot C_d \cdot A_{f1} \cdot 1.8A_{f1}) =$$

Longitudinal wind load

$$F_{WSL} = 0.25P_D =$$

$$\begin{aligned} \text{Zone} &= \text{III} \\ V_b &= 53.00 \text{ m/s} \\ S &= 1.09 \\ V &= 57.77 \text{ m/s} \\ C_d &= 1.10 \\ b &= 12.60 \text{ m} \\ d &= 2.92 \text{ m} \\ b/d &= 4.32 \\ A_t &= 96.26 \text{ m}^2 \\ H_y &= 212.0 \text{ kN} \\ H_x &= 53.0 \text{ kN} \end{aligned}$$

5.2. Wind load on vehicles (WL)

Transverse wind load on vehicle

Longitudinal wind load on vehicles

(At 1.8m from surface)

$$\begin{aligned} H_y &= 24.75 \text{ kN} \\ H_x &= 24.75 \text{ kN} \end{aligned}$$

6. Combinations

Loads from superstructure to Abutment

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Loads at bottom of stem		Vertical		Longitudinal			Tranversal		
Loads	Sign	N (kN)	x (m)	Hx (kN)	z ₁ (m)	My (kN•m)	Hy (kN)	y (m)	Mx (kN•m)
Girder+Deck+Parapet	DC	3778	0.20			756			
Pavement	DW	325	0.20			65			
LiveLoad	LL	1326	0.20			265		0.48	630
Pedestrian	PD							-	-
Trans. wind on Struc.	WS			53	3.65		106	3.65	387
Trans. wind on vehi.	WL			25	9.85		25	9.85	244
Eearth quake	EQ			403	3.65		121	3.65	440
TU+SH&CR	TU+SH&CR			271	3.65	987			

Loads at bottom of pilecap		Vertical		Longitudinal			Tranversal		
Loads	Sign	N (kN)	x (m)	Hx (kN)	z ₁ (m)	My (kN•m)	Hy (kN)	y (m)	Mx (kN•m)
Girder+Deck+Parapet	DC	3778	0.40			1511			
Pavement	DW	325	0.40			130			
LiveLoad	LL	1326	0.40			530		0.48	630
Pedestrian	PD							-	-
Trans. wind on Struc.	WS			53	5.65	299	106	5.65	599
Trans. wind on vehi.	WL			25	11.85	293	25	11.85	293
Eearth quake	EQ			403	5.65	2273	121	5.65	682
TU+SH&CR	TU+SH&CR			271	5.65	1529			

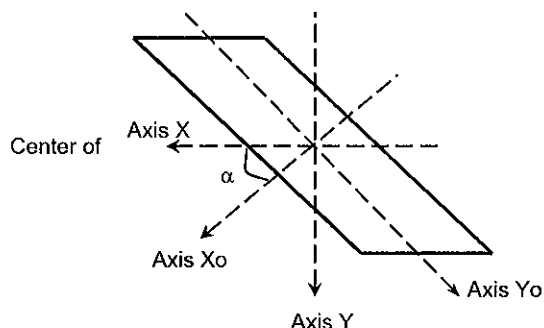
Loads	Sign	Load combinations						
		Str-IA	Str-IB	Str-IIIA	Str-IIIB	Ser-I	Ext-IA	Ext-IB
Girder+Deck+Parapet	DC	1.25	0.90	1.25	0.90	1.00	1.25	0.90
Pavement	DW	1.50	0.65	1.50	0.65	1.00	1.50	0.65
LiveLoad	LL	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Pedestrian	PD	1.75	1.75	1.35	1.35	1.00	0.50	0.50
Trans. wind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

Load combinations at bottom of stem					
Load combinations	N (kN)	Longitudinal		Tranversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	7530	135	2000	0	1102
Strength Str-IB	5932	135	1680	0	1102
Strength Str-IIIA	7000	181	1894	67	1249
Strength Str-IIIB	5402	181	1574	67	1249
Service Ser-I	5429	311	2073	57	990
Extreme Ext-IA	5872	403	1174	121	755
Extreme Ext-IB	4274	403	855	121	755

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	7530	135	3777	0	1102
Strength Str-IB	5932	135	3137	0	1102
Strength Str-IIIA	7000	181	3977	67	1383
Strength Str-IIIB	5402	181	3338	67	1383
Service Ser-I	5429	311	4084	57	1103
Extreme Ext-IA	5872	403	4622	121	997
Extreme Ext-IB	4274	403	3983	121	997

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LOAD COMBINATIONS



III. Load Combinations

1. Loads from substructure

Loads at bottom of pilecap	Sign	Vertical	Longitudinal		Tranversal	
		N (kN)	Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Self weight of Abutment	DC	5671		-334		249.1705
Soils on pilecap	EV	3051		-4563		
Horizontal Earth Pressure	EH		2283	7095		
Vertical Surcharge	LSv	335		-553		
Horizontal Surcharge	LSH		394	1533		
Braking Force	BR		104	991		
Centrifugal Force	CE		-	-	12	90
Buoyancy of Abutment	WA	-		-		
Buoyancy of Earth on Abutment	WA	-		-		
Earthquake effects to Abutment	EQ		550	1215	165	364
Earthquake effects to soil	E _{AE}		2576	6691		

Loads	Sign	Load factors						
		Str-IA	Str-IB	Str-III A	Str-III B	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	11794	4296	7516	21	469
Strength Str-IB	8436	2926	5429	21	382
Strength Str-III A	11660	4096	6728	16	433
Strength Str-III B	8302	2727	4641	16	346
Service Ser-I	9057	2781	4170	12	339
Extreme Ext-IA	11375	3376	2315	171	721
Extreme Ext-IB	8017	3376	4485	171	634

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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	7530	135	3777	0	1102
Strength Str-IB	5932	135	3137	0	1102
Strength Str-IIIA	7000	181	3977	67	1383
Strength Str-IIIB	5402	181	3338	67	1383
Service Ser-I	5429	311	4084	57	1103
Extreme Ext-IA	5872	403	4622	121	997
Extreme Ext-IB	4274	403	3983	121	997

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	19324	4431	11293	21	1572
Strength Str-IB	14368	3061	8566	21	1485
Strength Str-IIIA	18660	4278	10705	83	1816
Strength Str-IIIB	13704	2908	7979	83	1729
Service Ser-I	14486	3092	8254	68	1442
Extreme Ext-IA	17248	3778	6937	292	1718
Extreme Ext-IB	12292	3778	8467	292	1631

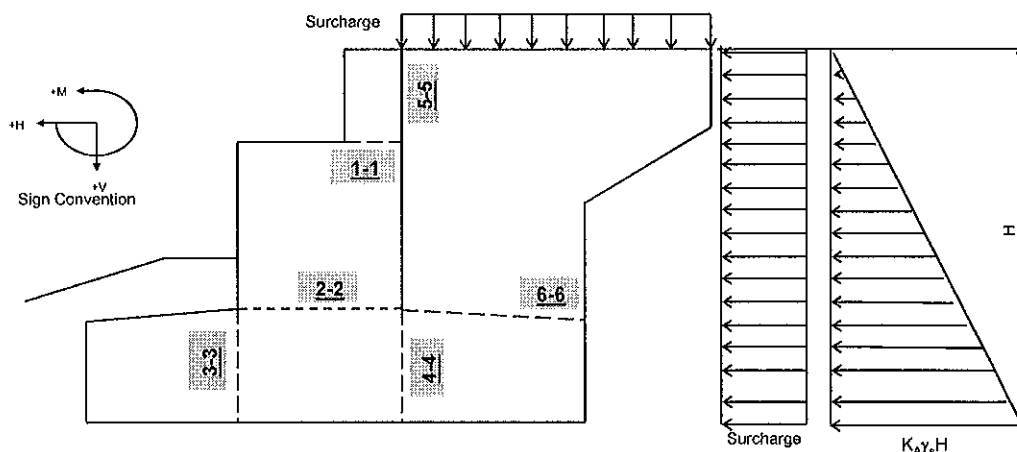
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ELEMENTS CHECKING

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	390		-15		
Horizontal Earth Pressure		196	178		
Surcharge (horizontal)		248	282		
Horizontal Seismic Earth Pressure		221	168		
Abutment earthquake force		38	43	11	13

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	390	444	445	0	0
Strength Str-IA	487	727	741	0	0
Strength Str-IB	351	610	640	0	0
Extreme Ext-I	487	493	417	11	13

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
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	2008		-179		
Superstructure Dead Load	3778		756		
Pavement	325		65		
Live Load	1326		265		630
Horizontal Earth Pressure		1259	2906		
Surcharge (Horizontal)		337	971		
TU+SH&CR		271	987		
Horizontal Seismic Earth Pressure		1421	2740		
Abutment earthquake force		197	456	180	577

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	7437	1866	5771	0	630
Strength Str-IA	10041	2613	7834	0	1102
Strength Str-IB	7740	1857	5834	0	1102
Extreme Ext-I	8383	2497	6003	180	892

1.3 Section 3-3

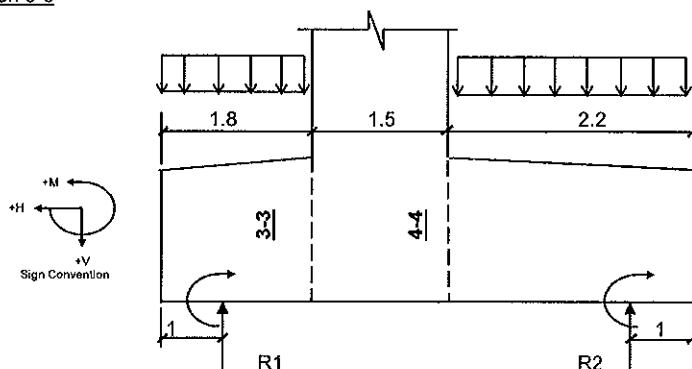


Table of Load Factors

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	Shear (kN)	Longitudinal			
		Hx (kN)	My (kN.m)		
Selfweight at front side	-1111		-1000		
Vertical soil on foot at front side	-214		-193		
Reaction of piles					
Ser-I	11276		12848		
Str-IA	15312		17746		
Str-IB	11274		12780		
Ext-I	12779		15116		

Load Combination at section 3-3					
Load combinations	N (kN)	Longitudinal			
		Hx (kN)	My (kN.m)		
Service Ser-I	9951		11655		
Strength Str-IA	13633		16236		
Strength Str-IB	10081		11707		
Extreme Ext-I	11101		13606		

	Da Nang Quang Ngai Expressway project CB25 BRIDGE DETAIL DESIGN ABUTMENT A2	Item.	Eng.	Date.	Sign.
		Design			
		Check			
		Revise			

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	-1554		-1784		
Vertical soil on foot at behind side	-2765		-3042		
Surcharge(Vertical)	-335		-369		
Reaction of piles					
Ser-I	2922		636		
Str-IA	3631		234		
Str-IB	2809		550		
Ext-I	4122		1277		

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	-1733		-4559		
Strength Str-IA	-2631		-6748		
Strength Str-IB	-1664		-4438		
Extreme Ext-I	-1721		-5244		

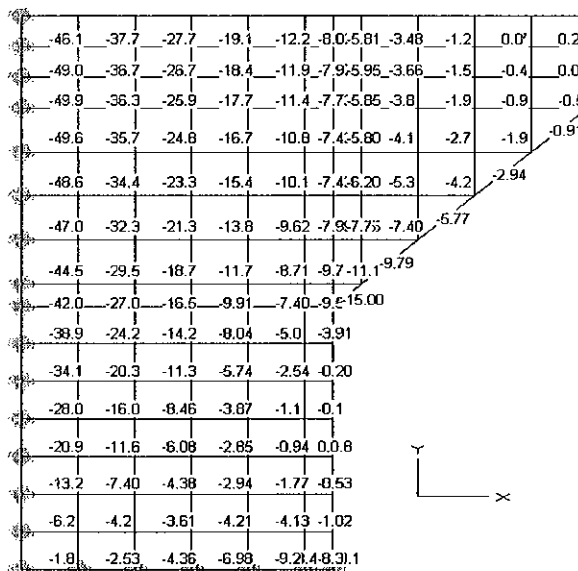
1.4 Section 5-5 & 6-6

Slope of triang pressure
Uniform horizontal pressure

$\tan \beta = 6.00$
 $U.p = 4.03 \text{ kN/m}^2$

SERVICE – Element Moment X:

Combination X-Bending Moment (per unit width)
service IA

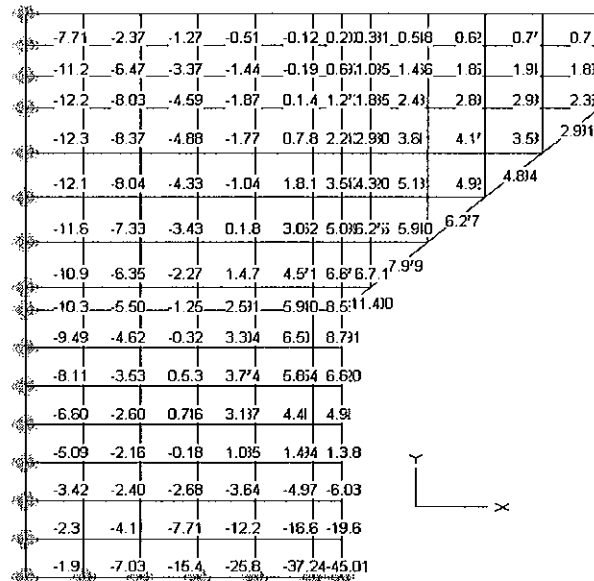


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	Da Nang Quang Ngai Expressway project CB25 BRIDGE DETAIL DESIGN ABUTMENT A2	Item.	Eng.	Date.	Sign.
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SERVICE – Element Moment Y:

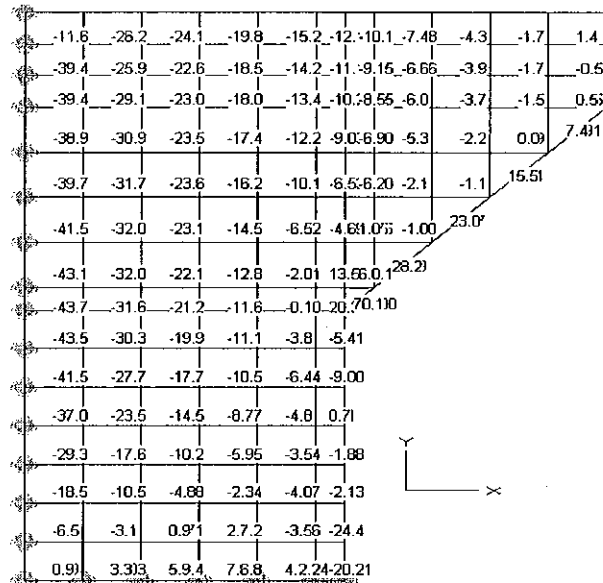
Combination Y-Bending Moment(per unit width)
service 1A



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SERVICE – Element Shear FX:

Combination SHEAR FORCE X (per unit width)
service 1A

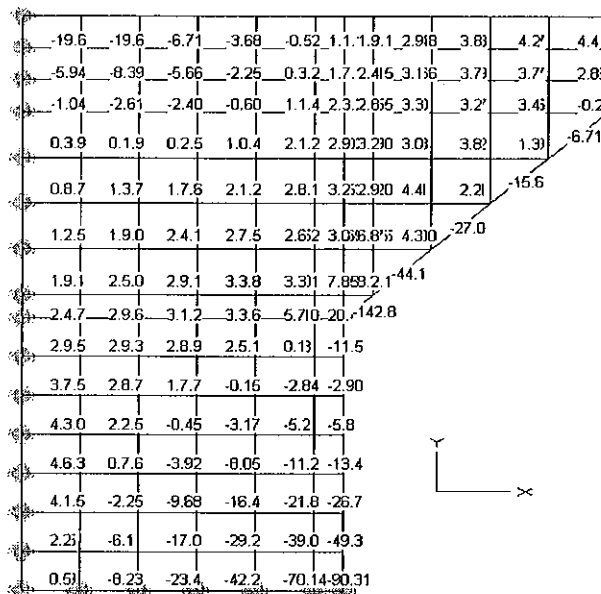


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SERVICE – Element Shear FY:

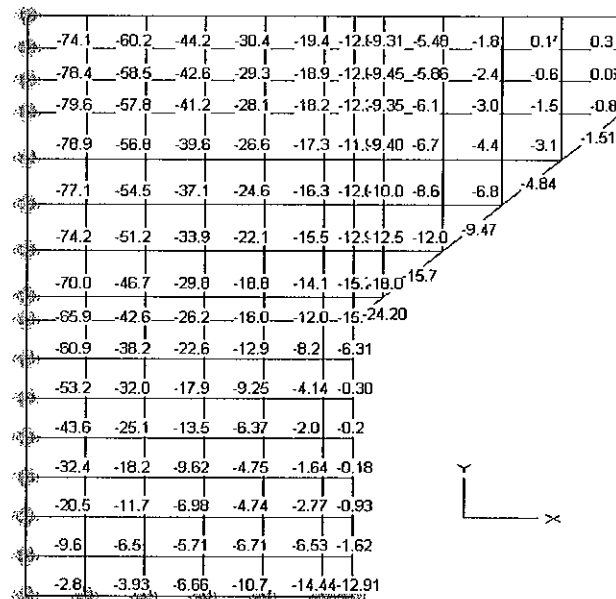
Combination SHEAR FORCE Y (per unit width)
service 1A



DN-QN F:\Hai\Da Nang-Quang Ngai\Hai\PKG4\ORB06\cal\aces\A2\TUONG CANH HO A2-CB09.ACE

STRENGTH – Element Moment X:

Combination X-Bending Moment (per unit width)
strong 1A

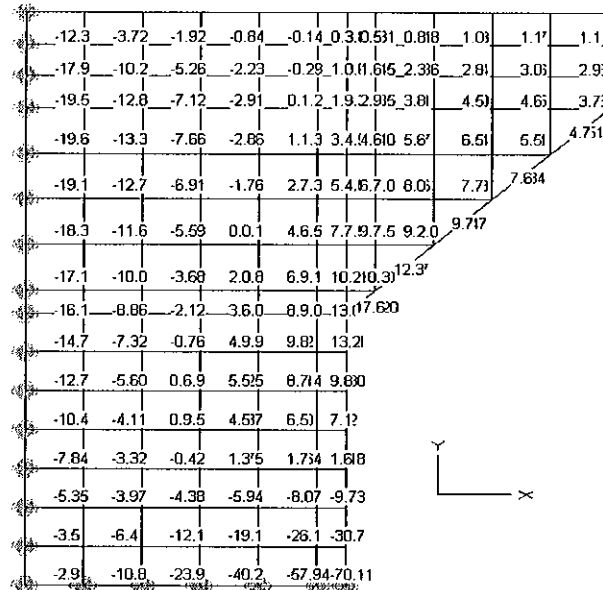


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	Da Nang Quang Ngai Expressway project CB25 BRIDGE DETAIL DESIGN ABUTMENT A2	Item.	Eng.	Date.	Sign.
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		Check			
		Revise			

STRENGTH – Element Moment Y:

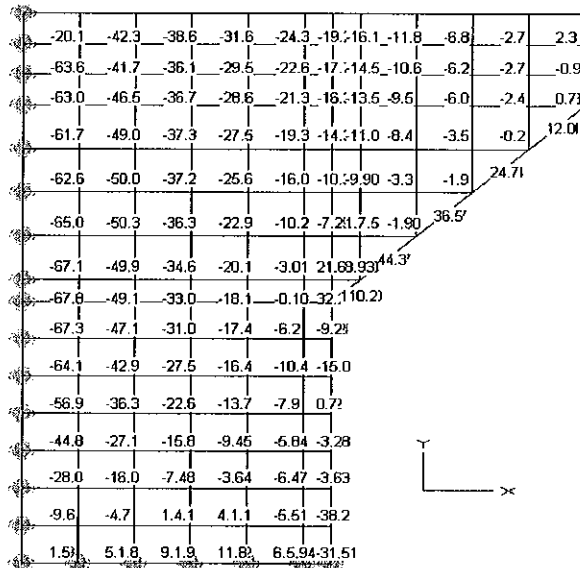
Combination Y-Bending Moment(per unit width)
 streng IAK



DN-QN F:\Hai\Da Nang-Quang Ngai\Hai\PKG4\OR06\cal\aces\A2\TUONG CANH HO A2-CB09.ACE

STRENGTH – Element Shear Fx:

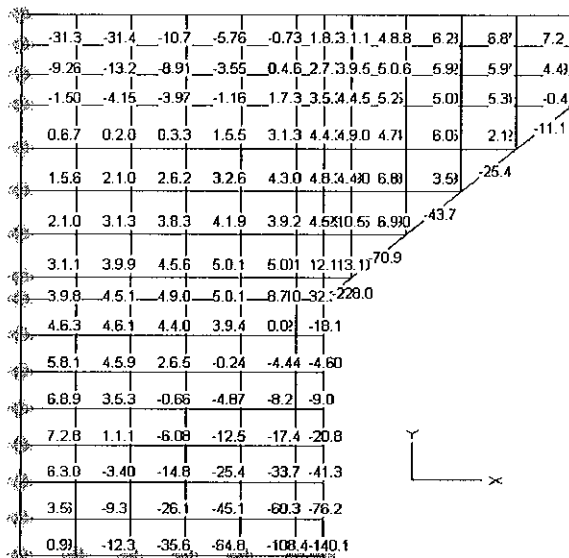
Combination SHEAR FORCE X (per unit width)
 streng IAK



DN-QN F:\Hai\Da Nang-Quang Ngai\Hai\PKG4\OR06\cal\aces\A2\TUONG CANH HO A2-CB09.ACE

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



DN-QN F:\Hai\Da Nang-Quang Ngai\Hai\PKG4\ORB06\cal\aces\A2\TUONG CANH IIO A2-CB09.ACE

Load Combination at section 5-5					
Load combinations	N (kN)	Vertical		Horizontal	
		Hy (kN)	My (kN.m)	Hx (kN)	Mx (kN.m)
Service Ser-I		90	45		
Strength Str-IA		140	70		

Load Combination at section 6-6					
Load combinations	N (kN)	Vertical		Horizontal	
		Hy (kN)	My (kN.m)	Hx (kN)	Mx (kN.m)
Service Ser-I				44	50
Strength Str-IA				68	80

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		Design			
		Check			
		Revise			

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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	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				
			1-1	1-1	2-2	2-2	2-2
INTERNAL FORCES AT SECTION							
	Combination		Strength	Service	Service	Strength	Extreme
Q_u	Shear	kN	727	444	1866	2613	2497
M_u	Flexural Moment	kNm	741	445	5771	7834	6003
N_u	Axial load	kN	487	390	7437	10041	8383
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
d_{lx}	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	1.441	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
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	0.500	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	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	82	82	82	82
		Diameter	mm	16	16	18	18
		Area	m2	0.01656	0.01656	0.02083	0.02083
A'_s	Compression Reinforcement	Number	bars	82	82	82	82
		Diameter	mm	16	16	16	16
		Area	m2	0.01656	0.01656	0.01656	0.01656
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
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	0.006	0.006	0.006
	For T section behavior	m	0.000	0.000	0.006	0.006	0.006
	For rectangular section behavior	m	0.000	0.000	0.006	0.006	0.006
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	1858	1858	1858
k	Factor depends on type of P.S, Low relaxation strand $k = 0.28$		0.28	0.28	0.28	0.28	0.28

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REINFORCEMENT CHECKING - HEAD AND STEM WALL							
a	Depth of equivalent stress block	m	0.000	0.000	0.005	0.005	0.005
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	2544	2544	11616	11616	11616
Mr	Factored resistance	kNm	2290	2544	11616	10455	11616
Mu	Flexural moment	kNm	741	445	5771	7834	6003
(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*Mcrr	Craking moment	kNm	1087	1087	9824	9824	9824
(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 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	0.018	0.018	0.018
fsa	Value	Mpa	297	297	293	293	293
0.6*fy		Mpa	240	240	240	240	240
	Tensil stress in reinf $\text{Min}(f_{sa}, 0.6f_y)$	Mpa	240	240	240	240	240
x	Dist. From compression fiber to centroid	m	-	0.081	0.171	-	-
J.d	Arm	m	-	0.415	1.384	-	-
Icr	Moment of inertia of the cracked section	m ⁴	-	0.017	0.257	-	-
fs	Tensile stress in reinforcement $f_s = M_{sls} / (A_s * J.d)$	Mpa	-	65	200	-	-
	Checking for control cracking $f_s < f_{sa}$		N.a	OK	OK	N.a	N.a
SHEAR AND TORSION CHECKING							
β	Factor indicating diag. cracked concr. to tension		2.4	3.0	2.5	2.4	2.5
θ	Angle of inclination of diagonal compressive	degree	30.91	28.73	28.99	31.88	30.07
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	90
bv	Effective web width as minimum web width - in dv	m	12.600	12.600	12.600	12.600	12.600
dv	Effective shear depth	m	0.442	0.442	1.438	1.438	1.438
	($d_e - a/2$)	m	0.442	0.442	1.438	1.438	1.438
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	0.600
ncat	Amount of bars in spacing S	bars	20	20	19	19	19
Av	Shear reinf area in spacing S	m ²	0.0030	0.0030	0.0029	0.0029	0.0029
θ	Assume	degree	30.46	28.70	27.86	28.51	27.67
v	Shear stress in concrete	kN/m ²	145	53	103	160	138
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e_x	Strain in tensile reinforcement		6.20E-04	3.67E-04	4.94E-04	6.80E-04	5.67E-04
	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 fc		0.005	0.002	0.003	0.005	0.005
β	Final value		2.4	3.0	2.5	2.4	2.5
θ	Final value	degree	30.91	28.73	28.99	31.88	30.07
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	6185	7673	20863	19851	20370
Vs	Shear resistance provided by shear reinforcement	kN	1486	1623	4965	4424	4751
Vp	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0	0
Vn1	$V_{n1} = V_c + V_s + V_p$	kN	7672	9296	25829	24275	25120
Vn2	Vn2	kN	41769	41769	135924	135924	135924
Vn	Nominal shear resistance $V_n = \min(V_{n1}, V_{n2})$	kN	7672	9296	25829	24275	25120
Vr	Factored shear resistance	kN	6904	9296	25829	21847	25120
Vu	Shear	kN	727	444	1866	2613	2497
(5.8.2.7)	Shear checking		OK	OK	OK	OK	OK

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REINFORCEMENT CHECKING - PILECAP SECTION								
MATERIALS								
NORMAL CONCRETE								
f'c	Compressive Strength of concrete at 28 days	Mpa	30					
Ec	Modulus of Elasticity	Mpa	27691					
fr	Modulus of Rupture	Mpa	3.5					
gc	Unit weight of concrete	kN/m3	24.5					
PRESTRESSING STEEL								
fpu	Tensile strength of prestressing steel	Mpa	1860					
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	9951	13633	11101	1721	2631	
Mu	Flexural Moment	kNm	11655	16236	13606	5244	6748	
Nu	Axial load	kN	0	0	0	0	0	
Tu	Torsional Moment	kNm	0	0	0	0	0	
FLEXURAL MOMENT CHECKING								
H	Section height	m	2.000	2.000	2.000	2.000	2.000	
d's	Dis. From comp. fiber to centroid of comp. Reinf	m	0.084	0.084	0.084	0.084	0.084	
d1x	Dis. From tens. fiber to centroid of tension Reinf	m	0.163	0.163	0.163	0.159	0.159	
	Cover to reinf	m	0.075	0.075	0.075	0.075	0.075	
ds	Dis. From comp. fiber to centroid of tension Reinf	m	1.838	1.838	1.838	1.841	1.841	
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	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	84	84	84	84	84	
		Diameter	mm	25	25	25	18	18
		Area	m2	0.04124	0.04124	0.04124	0.02134	0.02134
A's	Compression Reinforcement	Number	0	0	0	0	0	
		Diameter	mm	18	18	18	18	18
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
A'c	Shear reinforcement	Number	19	19	19	19	19	
		Diameter	mm	16	16	16	16	16
		Area	m2	0.00384	0.00384	0.00384	0.00384	0.00384
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	0.061	0.061	0.061	0.032	0.032	
	For T section behavior	m	0.061	0.061	0.061	0.032	0.032	
	For rectangular section behavior	m	0.061	0.061	0.061	0.032	0.032	
fpe	Effective stress in the prestressing steel after losses	Mpa	1116	1116	1116	1116	1116	
fps	Aver. stress in pres. steel at the time for which the nominal resistance	Mpa	1844	1844	1844	1852	1852	
k	Factor depends on type of P.S, Low relaxation strand k = 0.28		0.28	0.28	0.28	0.28	0.28	

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			Revise				
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REINFORCEMENT CHECKING - PILECAP SECTION							
a	Depth of equivalent stress block	m	0.051	0.051	0.051	0.027	0.027
de	Corresp. effective depth from extreme comp. fiber to centroid of tensile force in the tensile reinf.	m	1.838	1.838	1.838	1.841	1.841
Mn	Nominal resistance	kNm	29891	29891	29891	15598	15598
Mr	Factored resistance	kNm	29891	26902	29891	15598	14039
Mu	Flexual moment	kNm	11655	16236	13606	5244	6748
(5.7.3.2)	Flexural moment Checking		OK	OK	OK	OK	OK
	Limits for reinforcement						
c/de	Maximum reinforcement		0.03	0.03	0.03	0.02	0.02
	Maximum reinforcement Checking	<= 0.42	OK	OK	OK	OK	OK
1.2*Mer	Craking moment	kNm	17942	17942	17942	17672	17672
(5.7.3.3.2)	Checking Mr>=min(1.2Mer,1.33Mu)		OK	OK	OK	OK	OK
(5.7.3.4)	Conctrol of craking by distr. of reinf for RC member- Check?		Yes	No	No	No	No
	Existing condition for structure	1,2 or 3	3	3	3	3	3
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.050	0.050	0.050	0.050	0.050
Z	Crack width parameter	N/mm	17500	17500	17500	17500	17500
A	Area of concr. with same centroid as tens. Reinf	m2	0.015	0.015	0.015	0.015	0.015
f _{sa}	Value	Mpa	193	193	193	193	193
0.6*f _y		Mpa	240	240	240	240	240
	Tensil stress in reinf Min(f _{sa} ,0.6f _y)	Mpa	193	193	193	193	193
x	Dist. From compression fiber to centroid	m	0.268	-	-	-	-
J.d	Arm	m	1.748	-	-	-	-
I _{cr}	Moment of inertia of the cracked section	m4	0.792	-	-	-	-
f _s	Tensile stress in reinforcement f _s = Msls / (As*J.d)	Mpa	162	-	-	-	-
	Checking for control cracking f _s <f _{sa}		OK	N.a	N.a	N.a	N.a
SHEAR AND TORSION CHECKING							
β	Factor indicating diag. cracked concr. to tension		1.9	1.7	1.9	2.2	2.0
θ	Angle of inclination of diagonal compressive	degree	41.00	42.96	41.73	36.31	39.24
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	1.812	1.812	1.812	1.828	1.828
	(d _e - a/2)	m	1.812	1.812	1.812	1.828	1.828
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	0.600
n _{cat}	Amount of bars in spacing S	bars	19	19	19	19	19
A _v	Shear reinf area in spacing S	m2	0.0038	0.0038	0.0038	0.0038	0.0038
θ	Assume	degree	39.91	42.41	41.07	29.33	33.89
v	Shear stress in concrete	kN/m2	436	664	486	33	127
f _{po}	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e _s	Strain in tensile reinforcement		1.50E-03	1.99E-03	1.68E-03	1.03E-03	1.32E-03
	if e _x <0, multiple with reduce factor		-	-	-	-	-
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok	Ok
v/f _c	Ratio of shear stress and f _c		0.015	0.022	0.016	0.001	0.004
β	Final value		1.9	1.7	1.9	2.2	2.0
θ	Final value	degree	41.00	42.96	41.73	36.31	39.24
V _c	Nominal shear resistance provided by tensile stresses in the concrete	kN	20233	17894	19366	23163	21446
V _s	Shear resistance provided by shear reinforcement	kN	5332	4978	5198	6363	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	25565	22871	24564	29526	27171
V _{n2}	V _{n2}	kN	171218	171218	171218	172719	172719
V _n	Nominal shear resistance V _n =min(V _{n1} ,V _{n2})	kN	25565	22871	24564	29526	27171
V _r	Factored shear resistance	kN	25565	20584	24564	29526	24454
V _u	Shear	kN	9951	13633	11101	1721	2631
(5.8.2.7)	Shear checking		OK	OK	OK	OK	OK

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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

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REINFORCEMENT CHECKING - WING WALL

a	Depth of equivalent stress block	m	0.006	0.006	0.006	0.006
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
Mn	Nominal resistance	kNm	280	280	280	280
Mr	Factored resistance	kNm	280	252	280	252
Mu	Flexural moment	kNm	45	70	50	80
(5.7.3.2)	Flexural moment Checking		OK	OK	OK	OK
	Limits for reinforcement					
c/de	Maximum reinforcement		0.02	0.02	0.02	0.02
	Maximum reinforcement Checking	≤ 0.42	OK	OK	OK	OK
r min	Minimum reinforcement		0.36%	0.36%	0.36%	0.36%
	Minimum reinforcement Checking for RC	0.23%	OK	OK	OK	OK
1.2*Mer	Cracking moment	kNm	87	87	87	87
(5.7.3.3.2)	Checking $Mr \geq \min(1.2Mer, 1.33Mu)$		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	0.017	0.017	0.017	0.017
fsa	Value	Mpa	301	301	301	301
0.6*fy	Tensile stress in reinf Min(fs,0.6fy)	Mpa	240	240	240	240
x	Dist. From compression fiber to centroid	m	0.093	-	0.093	-
J.d	Arm	m	0.41	-	0.41	-
Icr	Moment of inertia of the cracked section	m ⁴	0.002	-	0.002	-
fs	Tensile stress in reinforcement $fs = Ms / (As * J.d)$	Mpa	62	-	69	-
	Checking for control cracking $fs < fsa$		OK	N.a	OK	N.a

SHEAR AND TORSION CHECKING

β	Factor indicating diag. cracked concr. to tension		2.5	2.4	2.8	2.4
θ	Angle of inclination of diagonal compressive	degree	29.24	33.02	28.87	31.69
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	0.438	0.438	0.438	0.438
	($de - a/2$)	m	0.438	0.438	0.438	0.438
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600
neat	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		2.0	2.2	2.0	2.0
θ	Assume	degree	29.24	33.03	28.87	31.69
v	Shear stress in concrete	kN/m ²	205	355	100	172
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116
ϵ_s	Strain in tensile reinforcement		5.15E-04	7.52E-04	4.33E-04	6.68E-04
	if $\epsilon_s < 0$, multiple with reduce factor		-	-	-	-
	Strain checking	$\leq 2.00E-3$	Ok	Ok	Ok	Ok
v/fc	Ratio of shear stress and fc		0.007	0.012	0.003	0.006
β	Final value		2.5	2.4	2.8	2.4
θ	Final value	degree	29.24	33.02	28.87	31.69
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	498	472	552	481
Vs	Shear resistance provided by shear reinforcement	kN	118	102	120	107
Vp	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0
Vn1	$Vn1 = Vc + Vs + Vp$	kN	616	573	672	588
Vn2	$Vn2$	kN	3286	3286	3286	3286
Vn	Nominal shear resistance $Vn = \min(Vn1, Vn2)$	kN	616	573	672	588
Vr	Factored shear resistance	kN	616	516	672	529
Vu	Shear	kN	90	140	44	68
(5.8.2.7)	Shear checking		OK	OK	OK	OK

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BORED PILE DESIGN

I. BORED PILE DATA

1. Load Combinations at top of bored pile

STT	Comb	Axial force P (KN)	Moment (KN.m)		
			Mx	My	Mxy
STRENGTH LIMIT STATES					
1	P_min	705	36.6	843.4	844
2	P_max	4069	34.6	1452.5	1453
3	Mx_max	4069	34.6	1452.5	1453
4	My_max	4069	34.6	1452.5	1453
EXTREME EVENT LIMIT STATES					
1	P_min	243	126	1101	1108
2	P_max	3457	72	1296	1298
3	Mx_max	3457	72	1296	1298
4	My_max	3457	72	1296	1298

2. Bored pile Material

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

3. Bored pile Section

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

II. PILE DESIGN

1. Limit of Reinforcement

S.5.7.4.2

Minimum area of longitudinal reinforcement in column							
$As.fy / (Ag \cdot f_c) \geq 0.135$				$As \geq$	0.008	m2	
$As / Ag \geq 0.01$				$As \geq$	0.008	m2	
Maximum area of longitudinal reinforcement in column							
$As / Ag \leq 0.08$				$As \leq$	0.063	m2	
Trial Rebars:				Ok	As	0.019	m2
11ayers	x 24	= 24 bars	D32	@150	As1	0.019	m2

2. Iteration diagram M-P

Using Pca-Column software

**Flexural check by pcaColumn

Strength and Service limit states:

Resistance factor:	Compression	ϕ_c	=	0.75 (AASHTO LRFD-2004)
	Tension	ϕ_t	=	0.90

Extreme Event limit states:

Resistance factor	Compression	ϕ_c	=	1.00
	Tension	ϕ_t	=	1.00

<Result table>

STT	Comb	Pu kN	Mux kN-m	Muy kN-m	ϕ Mnx kN-m	ϕ Mny kN-m	ϕ Mn/Mu
STRENGTH LIMIT STATES							
1	P_max	971.3	33.9	872.2	106.6	2742.3	3.144
2	P_min	3382.8	37.1	1471.5	74.3	2946.7	2.003
3	Mx_max	3382.8	37.1	1471.5	74.3	2946.7	2.003
4	My_max	3382.8	37.1	1471.5	74.3	2946.7	2.003
EXTREME EVENT LIMIT STATES							
1	P_max	615	105	1067	260.2	2643.9	2.478
2	P_min	2818	52	1249	124.5	2990.7	2.394
3	Mx_max	2818	52	1249	124.5	2990.7	2.394
4	My_max	2818	52	1249	124.5	2990.7	2.394

3. Column Ties

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

Bridge is in seismic zone	Sz	1	
Area of concrete core measured out-to-out of ties	Ac	0.622	m2
Tie diameter	Dtie	16	mm2
Cross section area of 1 tie	As-tr	0.00020	m2
Spacing of hoops	s	75	mm
Length of 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	0.0120	
Ratio of spiral reinf. To total volume of concrete core shall satisfy			S.5.7.4.6
$ps \geq 0.45 \cdot (Ag/Ac - 1) \cdot f_c / fy = Req1$	Req1	0.0089	OK
Transverse Reinforcement for Confinement at Plastic Hinges			
S.5.10.11.4.1.d			
For a circular column "1:applied", "2:Not applied"		1	
$ps \geq 0.12 \cdot f_c / fy = Req2$	Req2	0.0090	N/A
Length distributed spiral with pitch 75mm below pilecap	Ldis	1.50	m

4. Shear Design

Shear resistance factors	ϕ_v	1.0	
Factored shear force	V_u	622	kN
Required shear capacity $V_n = V_u / \phi_v$	V_n	622	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
$d_v = 0.9 \cdot d_e$ $d_e = D/2 + D_r/\pi()$			
Diameter of the circle passing through the centers of the long. reinf.	D_r	0.79	m
	d_e	0.75	m
	d_v	0.68	m
$V_c = 0.083 \cdot \beta \cdot \sqrt{f_c} \cdot b_v \cdot d_v$	V_c	616	kN
Diameter of Transverse Reinforcement	D_v	14	mm
Spacing of Transverse Reinforcement	s	75	mm
Area of a transverse reinforcement within distance "s"	A_v	453.00	mm ²
Angle of inclination of transverse reinforcement to longitudinal axis	α	90	deg.
Effetive shear depth, d_v			
Alternative 1: $d_{v1} = M_n / (A_s \cdot f_y)$			
Normal flexural resistance	M_n	2991	KNm
	d_{v1}	194	mm
Alternative 2: $d_{v2} = 0.9d_e$	$d_e = D/2 + D_r/\pi$	752	mm
	d_{v1}	677	mm
Choice value of d <input type="text" value="1"/> ("1" = d_{v1} , "2" = d_{v2})	d_v	194	mm
	$V_s = A_v f_y d_v (\cot \theta + \cot \alpha) \sin \alpha$		
Norminal shear resistance of Reinforcement	s	470	kN
	$V_{n1} = V_c + V_s$	1086	kN
	$V_{n2} = 0.25 f'_c b_v d_v$	1458	kN
	V_n	1086	kN
	Conclude		OK

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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	CB25-A1	Pile Concrete comp. strength	$f_c = 30.0$ MPa
Bottom of pilecap elavation	EL1 = -0.50	Concrete Unit Weight	$g_c = 24.5$ kN/m ³
Top of socket elevation	EL2 = -11.77	Modulus of elasticity of concre	$E_c = 27691$ MPa
Pile tip elevation	EL3 = -14.50		
Pile Length	$L = 14.00$ m	Depth of socket	$H_s = 2.73$ 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	$N = 4380.1$ kN		
Working normal force at top of socket	$P_i = 4348.5$ kN		
Intack rock modulus	$E_i = 400000$ MPa		Figure C10.8.3.5-2 Lrfd
Modulus modification ratio	$K_e = 0.12$		Figure C10.8.3.5-3 Lrfd
Elastic modulus of the insitu rock	$E_r = K_e * E_i = 48000$ MPa		
Influence coefficient	$I_p = f(H_s/D_s, E_c/E_r) = 0.35$		Figure C10.8.3.5-1 Lrfd
	$H_s/D_s = 2.73$		
	$E_c/E_r = 0.58$		
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_B		10.8.3.5.4b-Lrfd 4th

The elastic shortening of the drilled shaft

$$r_e = (\sum P_i) * H_s / (A_{soc} * E_c) = 0.546 \text{ mm}$$

The settlement of base of drilled shaft

$$r_{base} = (\sum P_i) * I_p / (D_s * E_r) = 0.032 \text{ mm}$$

$$r_e + r_{base} = 0.578 \text{ mm} < 10 \text{ mm}$$

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

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

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	-11.77	-12.27	0.50	60	64.49	1.69	2649	0.65	1722
2	-12.27	-12.77	0.50	78	64.49	1.69	2649	0.65	1722
3	-12.77	-14.50	1.73	75	64.49	1.69	9165	0.65	5958
4									
5									
6									
7									
8									
Sum			2.73				14463		9401

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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	0.50	60.00	64.49	0.42	0.76	1	13.58	1.26	1.26	1987	0.55	1093
2	0.50	78.00	64.49	0.78	0.91	1	13.58	1.51	1.51	2376	0.55	1307
3	1.73	75.00	64.49	0.75	0.90	1	13.58	1.49	1.49	8114	0.55	4463
4	-	-	-	-	-	-	-	-	-	-	-	-
5	-	-	-	-	-	-	-	-	-	-	-	-
6	-	-	-	-	-	-	-	-	-	-	-	-
7	-	-	-	-	-	-	-	-	-	-	-	-
8	-	-	-	-	-	-	-	-	-	-	-	-
Sum	2.73									12478		6863

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

$$K_b = 4.04$$

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:

File Structural Capacity	$Q_T = 0.75 \cdot 0.85 \cdot 0.85 \cdot f_c \cdot A_g$	12768 kN	1301 T
File resistance	Q_R	6863 kN	700 T
Deducting pile weight		-183 kN	-19 T
Estimated Pile Capacity		6680 kN	681 T
Maximum Reaction - ULS	Ok	4110 kN	419 T

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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	CB25-A2	Pile Concrete comp. strength	$f_c = 30.0$ MPa
Bottom of pilecap elavation	EL1 = -0.50	Concrete Unit Weight	$g_c = 24.5$ kN/m ³
Top of socket elevation	EL2 = -13.86	Modulus of elasticity of concre	$E_c = 27691$ MPa
Pile tip elevation	EL3 = -16.50		
Pile Length	L = 16.00 m	Depth of socket	$H_s = 2.64$ 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 = 4418.6 kN		
Working normal force at top of socket	$P_i = 4388.1$ kN		
Intack rock modulus	$E_i = 400000$ MPa		Figure C10.8.3.5-2 Lrfd
Modulus modification ratio	$K_c = 0.12$		Figure C10.8.3.5-3 Lrfd
Elastic modulus of the insitu rock	$E_r = K_c * E_i = 48000$ MPa		
Influence coefficient	$I_p = f(H_s/D_s, E_c/E_r) = 0.35$		Figure C10.8.3.5-1 Lrfd
	$H_s/D_s = 2.64$		
	$E_c/E_r = 0.58$		
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

$$r_e = (\sum P_i) * H_s / (A_{soc} * E_c) = 0.533 \text{ mm}$$

The settlement of base of drilled shaft

$$r_{base} = (\sum P_i) * I_p / (D_s * E_r) = 0.032 \text{ mm}$$

$$r_e + r_{base} = 0.565 \text{ mm} < 10 \text{ mm}$$

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	-13.86	-14.86	1.00	100	59.40	1.62	5085	0.65	3305
2	-14.86	-15.86	1.00	75	59.40	1.62	5085	0.65	3305
3	-15.86	-16.50	0.64	61	59.40	1.62	3254	0.65	2115
4									
5									
6									
7									
8									
Sum			2.64				13423		8725

	DANANG QUANG NGAI EXPRESSWAY					Item.	Eng.	Date.	Sign.
	CB25 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	1.00	100.00	59.40	1.00	1.00	1	13.58	1.59	1.59	5002	0.55	2751
2	1.00	75.00	59.40	0.75	0.90	1	13.58	1.43	1.43	4502	0.55	2476
3	0.64	61.00	59.40	0.45	0.78	1	13.58	1.24	1.24	2485	0.55	1367
4	-	-	-	-	-	-	-	-	-	-	-	-
5	-	-	-	-	-	-	-	-	-	-	-	-
6	-	-	-	-	-	-	-	-	-	-	-	-
7	-	-	-	-	-	-	-	-	-	-	-	-
8	-	-	-	-	-	-	-	-	-	-	-	-
Sum	2.64									11988		6593

Unit base resistance

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

Limit pressure determined from pressuremeter 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

$$K_b = 3.98$$

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	6593 kN	672 T
Deducting pile weight		-205 kN	-21 T
Estimated Pile Capacity		6388 kN	651 T
Maximum Reaction - ULS	Ok	4110 kN	419 T

SPACE PILE FOUNDATION ANALYSIS PROGRAM
Turbo BASIC

PROJECT: : DN-QN-A2-CB25

INITIA DATA

Kn = 0.19 Ax = 5.50 By = 12.60 Cz = 2.00
E v.uon = 3001028 E r.uon = 3001028 E v.nen = 3001028 E r.nen = 3001028
Mq = 75 (t/m4) Md = 75 (t/m4) m = 400 (t/m4)

LOAD COMBINATIONS

COMB.	Hx	Hy	P	Mx	My	Mz
1	451.69	2.13	1969.84	-160.23	1151.18	0.00
2	312.07	2.13	1464.64	-151.34	873.24	0.00
3	436.06	8.49	1902.10	-185.15	1091.25	0.00
4	296.45	8.49	1396.90	-176.26	813.32	0.00
5	315.21	6.98	1476.63	-147.01	841.39	0.00
6	385.13	29.74	1758.16	-175.11	707.09	0.00
7	385.13	29.74	1252.96	-166.22	863.13	0.00

PROPERTIES OF PILES

PILE	Lo	H	Bpx	Bpy	A	B	Cday	Fo	Io	Po	Co	Ct
1	0.00	15.00	1.404	1.404	1.00	0.000	0.000	0.785	0.098	500	30000	
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	1.75	4.90	0.000	0.00
2	1.75	1.50	0.000	0.00
3	1.75	-1.90	0.000	0.00
4	1.75	-5.30	0.000	0.00
5	-1.75	-5.30	0.000	0.00
6	-1.75	-0.20	0.000	0.00
7	-1.75	4.90	0.000	0.00

DISPLACEMENTS

COMB.	X	Y	Z	Fix	Fiy	Fiz
1	0.01395	0.00027	0.003744	-0.000070	0.001107	0.000099
2	0.00972	0.00022	0.002791	-0.000056	0.000796	0.000069
3	0.01344	0.00044	0.003619	-0.000074	0.001057	0.000097

4	0.00921	0.00039	0.002667	-0.000060	0.000746	0.000067
5	0.00976	0.00035	0.002820	-0.000058	0.000781	0.000071
6	0.01144	0.00097	0.003398	-0.000076	0.000770	0.000093
7	0.01188	0.00094	0.002328	-0.000062	0.000936	0.000092

FORCES ON FILES

FILE	COMB.	N	Q2	Q3	M1	M2	M3
1	1	414.74	-63.39	0.57	1.429	3.525	148.060
	2	307.06	-43.77	0.30	0.989	2.408	101.368
	3	401.35	-61.26	-0.34	1.406	1.086	143.448
	4	293.67	-41.64	-0.60	0.966	-0.032	96.756
	5	307.61	-44.27	-0.36	1.019	0.585	103.215
	6	352.39	-54.44	-3.35	1.336	-7.338	132.147
	7	294.12	-54.28	-3.36	1.333	-7.736	127.132
2	1	398.39	-61.50	0.57	1.429	3.525	142.745
	2	293.89	-42.47	0.30	0.989	2.408	97.688
	3	384.13	-59.40	-0.34	1.406	1.086	138.217
	4	279.64	-40.37	-0.60	0.966	-0.032	93.161
	5	294.12	-42.93	-0.36	1.019	0.585	99.423
	6	334.57	-52.67	-3.35	1.336	-7.338	127.175
	7	279.50	-52.52	-3.36	1.333	-7.736	122.172
3	1	382.03	-59.62	0.57	1.429	3.525	137.429
	2	280.73	-41.16	0.30	0.989	2.408	94.009
	3	366.91	-57.55	-0.34	1.406	1.086	132.986
	4	265.62	-39.09	-0.60	0.966	-0.032	89.566
	5	280.62	-41.58	-0.36	1.019	0.585	95.630
	6	316.76	-50.91	-3.35	1.336	-7.338	122.204
	7	264.88	-50.76	-3.36	1.333	-7.736	117.213
4	1	365.67	-57.73	0.57	1.429	3.525	132.113
	2	267.57	-39.85	0.30	0.989	2.408	90.329
	3	349.69	-55.69	-0.34	1.406	1.086	127.756
	4	251.60	-37.82	-0.60	0.966	-0.032	85.971
	5	267.12	-40.24	-0.36	1.019	0.585	91.837
	6	298.95	-49.15	-3.35	1.336	-7.338	117.232
	7	250.25	-49.00	-3.36	1.333	-7.736	112.254
5	1	98.83	-57.73	-1.38	1.429	-1.947	132.113
	2	75.71	-39.85	-1.04	0.989	-1.380	90.329
	3	94.96	-55.69	-2.25	1.406	-4.299	127.756
	4	71.85	-37.82	-1.91	0.966	-3.732	85.971
	5	79.04	-40.24	-1.74	1.019	-3.319	91.837
	6	113.34	-49.15	-5.17	1.336	-12.456	117.232
	7	24.73	-49.00	-5.17	1.333	-12.841	112.254
6	1	123.36	-60.56	-1.38	1.429	-1.947	140.087
	2	95.46	-41.81	-1.04	0.989	-1.380	95.848
	3	120.79	-58.48	-2.25	1.406	-4.299	135.602
	4	92.89	-39.73	-1.91	0.966	-3.732	91.363
	5	99.28	-42.25	-1.74	1.019	-3.319	97.526
	6	140.06	-51.79	-5.17	1.336	-12.456	124.690
	7	46.67	-51.64	-5.17	1.333	-12.841	119.693
7	1	147.90	-63.39	-1.38	1.429	-1.947	148.060
	2	115.20	-43.77	-1.04	0.989	-1.380	101.368
	3	146.62	-61.26	-2.25	1.406	-4.299	143.448
	4	113.92	-41.64	-1.91	0.966	-3.732	96.756
	5	119.53	-44.27	-1.74	1.019	-3.319	103.215
	6	166.78	-54.44	-5.17	1.336	-12.456	132.147
	7	68.60	-54.28	-5.17	1.333	-12.841	127.132

SUMMARY OF FORCES

PILE COMB.			N	Q2	Q3	M1	M2	M3
Nmin	5	7	24.73	-49.00	-5.17	1.333	-12.841	112.254
Nmax	1	1	414.74	-63.39	0.57	1.429	3.525	148.060
Q2max	1	1	414.74	-63.39	0.57	1.429	3.525	148.060
Q3max	5	7	24.73	-49.00	-5.17	1.333	-12.841	112.254
M1max	1	1	414.74	-63.39	0.57	1.429	3.525	148.060
M2max	5	7	24.73	-49.00	-5.17	1.333	-12.841	112.254
M3max	1	1	414.74	-63.39	0.57	1.429	3.525	148.060

CHECKING CALCULATI
IN COMPARISON WITH INITIA LOAD MATRIX

1	451.69	2.13	1969.84	-160.23	1151.18	0.00
2	312.07	2.13	1464.64	-151.34	873.24	0.00
3	436.06	8.49	1902.10	-185.15	1091.25	0.00
4	296.45	8.49	1396.90	-176.26	813.32	0.00
5	315.21	6.98	1476.63	-147.01	841.39	0.00
6	385.13	29.74	1758.16	-175.11	707.09	0.00
7	385.13	29.74	1252.96	-166.22	863.13	0.00
