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

THE WORLD BANK

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

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

Detailed Engineering Design Report (Final)

Volume 4: Structural Calculation Report (PKG6)

Volume 4.2: Structural Calculation Report (PKG6, Bridges)

Section 4.2.1

0. Typical calculation sheet
1. OP11 BRIDGE
2. CB11 BRIDGE
3. ORB11 BRIDGE

July 15, 2013

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

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

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

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

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

Section 4.2.1

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

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

Project Manager/Giám đốc Dự án

長井崇泰

For ***Ichizuru Ishimoto***

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

MINISTRY OF TRANSPORT

VIETNAM EXPRESSWAY CORPORATION - PROJECT MANAGEMENT UNIT NO.85

DA NANG - QUANG NGAI EXPRESSWAY DEVELOPMENT PROJECT

PACKAGE: 6

TYPICAL CALCULATION SHEETS

HA NOI, 2012

Table of content - Typical calculation

DA NANG - QUANG NGAI EXPRESSWAY DEVELOPMENT PROJECT

PACKAGE: 6

CALCULATION SHEETS

GENERAL INPUT DATA

HA NOI, 2012

TABLE CONTENT

1. GENARAL

1.1 Location

Package 6 from Km42+100 to Km52+000, at Quang Nam province.

No	Bridge	Station	District	Commune
1	OP11	KM42+723	Thang Binh	Binh Quy
2	CB11	KM43+655		Binh Quy
3	ORB11	KM44+440		Binh Quy
4	ORB12	KM45+438		Binh Chanh
5	CB12	KM45+540		Binh Chanh
6	ORB13	KM45+884.5		Binh Chanh
7	OP11A	KM47+135.8		Binh Chanh
8	LRB09	KM47+910.75		Binh Chanh
9	CB13	KM48+390.3		Binh Chanh

1.2 Structure

Package 6

No	Bridge	Type of girder	Arrange
1	OP11	I Girder	1@21
2	CB11	I Girder	1@27
3	ORB11	I Girder	17@40
4	ORB12	I Girder	7@40
5	CB12	I Girder	1@27
6	ORB13	I Girder	1@33
7	OP11A	I Girder	1@40
8	LRB09	I Girder	3@40
9	CB13	I Girder	1@40

2. SPECIFICATIONS AND MATERIALS

2.1. Specifications

[1]. Specification for bridge design *22TCN-272-05*
References:
[1]. CEB – FIP model code 1990 *(comite euro – international du beton)*

[2]. Loads and effects design standard *TCVN 2737-1995*
[3]. Seismic standard *TCXDVN375-2006*
[4]. AASHTO LRFD 4th edition, 2007 specification for bridge design
[5]. AASHTO LRFD 1rt edition, 1998 specification for bridge construction
[6]. Transportation works in seismic zone - *22TCN221-95*

2.2. Materials characteristics

a. Concrete:

Items	Unit	Super Tee girder	I girder	Pier	Abutment	Bored pile
Density	kg/m3	2400	2400	2400	2400	2400
Compressive strength at 28 age days, f'c, cylinder specimen	MPa	50	40	30	30	30
Elastic modulus $Ec=0.043\gamma_c^{1.5}\sqrt{f'c}$	MPa	35750	31975	27691	27691	27691
Coefficient of Thermal Expansion α	/ °C	1.08e-5	1.08e-5	1.08e-5	1.08e-5	1.08e-5
Limited compressive stress - 0.45f'c At service stage	MPa	22.5	18.00			
Limited tensile stress - $0.5\sqrt{f'c}$ At service stage	MPa	-3.54	-3.16			

b. Reinforcement:

Reinforcement type	Grade	Elastic Modulus E(MPa)	fy (MPa)	fu (Mpa)
TCVN 1651 - 08				
Plain round bar	CB240-T	200 000	240	380
Deformed bar	CB400-V	200 000	400	570

TCVN 1651-08. Dimensions and Mass of Rebar

Designation	Nominal Diameter (mm)	Section Area (mm^2)	Nominal mass (kg/m)
D6	6	28.3	0.222
D8	8	50.3	0.395
D10	10	78.5	0.617
D12	12	113.0	0.888
D14	14	154.0	1.210
D16	16	201.0	1.580
D18	18	254.5	2.000
D20	20	314.0	2.470
D22	22	380.1	2.980

D25	25	491.0	3.850
D28	28	616.0	4.840
D32	32	804.0	6.310
D36	36	1017.9	7.990
D40	40	1257.0	9,860
D50	50	1964.0	15,420

c. Prestressing steel:

Uncoated strand with low-relaxation used in box girder shall conform to ASTM A416/ A416M-99, grade 270. Corresponding prestressing steel and anchorage systems have characteristics as below.

Type	Unit	Strand 15.2mm	Strand 12.7mm
Area of 1 strand	mm2	140	98.7
Ultimate strength f_{pu}	MPa	1860	1860
Yield strength f_{py}	MPa	1670	1670
Elastic modulus	MPa	195000	195000
Friction coeff./ unit length	m^{-1}	6.6e-4	6.6e-4
Angle friction coeff.	Rad^{-1}	0.25	0.25
Wedge slip	mm	6	6
Relaxation		2.5% (low relaxation) after 1000 hours, at 20^{O}C and 0.7P_M	
Prestressing force beforce wedge installing	kN	195	137
Nominal mass	kg/m	1.101	0.774

Notes: friction coefficients - angle and unit length, wedge slip are assumed. Final values are in accordance with prestressing steel and anchorage testing result.

d. Steel structure:

Welding shall be in accordance with AWS D1.5 "Bridge Welding Code".

Structural steel used in design, its mechanical property and the limitation of thickness are indicated in tables as below. The modulus of elasticity will be taken as 200 GPa.

ASTM designation	A790M grade 250	A790M grade 345	A790M grade 345W	A790M grade HPS 345W	A790M grades 690/690W
Thickness of plates, mm	Up to 100 incl.	Up to 100 incl.	Up to 100 incl.	Up to 100 incl.	Over 65 to 100 incl.
Shapes	All Groups	All Groups	All Groups	Not applicable	Not applicable

Minimum Strength, *Fu*, MPa	400	450	485	485	690
Minimum yield strength, F_y, MPa	250	345	345	345	620

* *Fy* : Specified minimum yield point or specified minimum yield strength.

e. Others Steel structure:

Nuts and bolts of steel guardrail shall conform to ASTM A307, steel tubes shall conform to ASTM A500, grade B. Galvanizing of rail elements shall conform to ASTM A123, nuts and bolts shall conform to ASTM A153.

ASTM designation	A500 grade B	A307 grade A
Minimum Strength, *Fu*, MPa		414
Minimum yield strength, F_y, MPa	290	

3. LOADS

3.1. Selfweight

Unit weight of reinforcement concrete	**24.50** kN/m^3
Unit weight of structural steel	**77.01** kN/m^3
Unit weight of Asphalt concrete	**22.1** kN/m^3

3.2. Dead loads stage 2

a) Wearing surface weight:

Traffic lanes:

+ *For Thruway bridges:*

8.0 cm asphalt + 0.4 cm water proof membrane:

+ *For Flyover bridges:*

7.0 cm asphalt + 0.4 cm water proof membrane:

b) Curb and rail at side:

+ *For Thruway bridges(for one bridge):*

Parapet + Median strip:

$(0.56+0.408)m^2 \times 24.5kN/m^3$ = ***23.7 kN/m***

+ *For Flyover bridges:*

Parapet + steel rail for 2 side:

$(0.416)m^2 \times 2 \times 24.5kN/m^3 + 1.25x2\ kN/m$ = ***22.9 kN/m***

c) Miscellaneous dead loads:

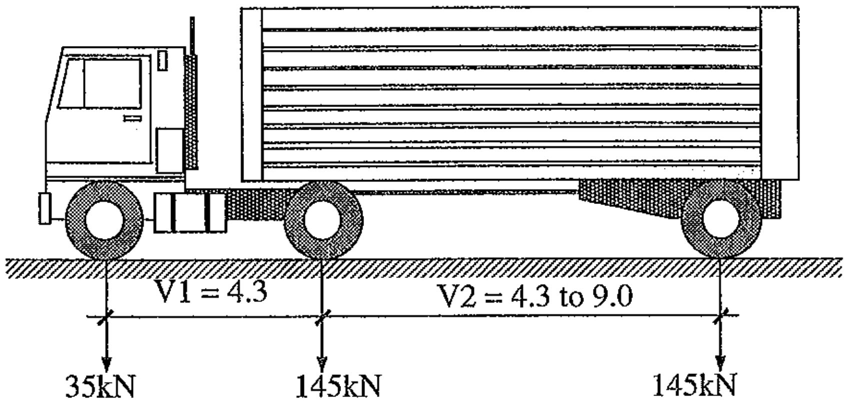
(traffic lights, water drainage …) = ***2.00 kN/m*** assumed

3.3. Live Loads

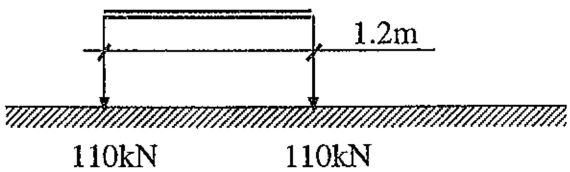
Vehicular live loading on the roadways of bridges or incidental structures, designated HL-93, shall consist of a combination of the:

- Design Truck or design Tandem, and
- Design lane load

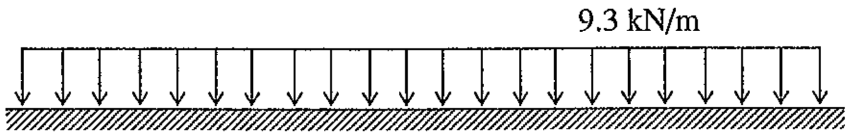
Design Truck



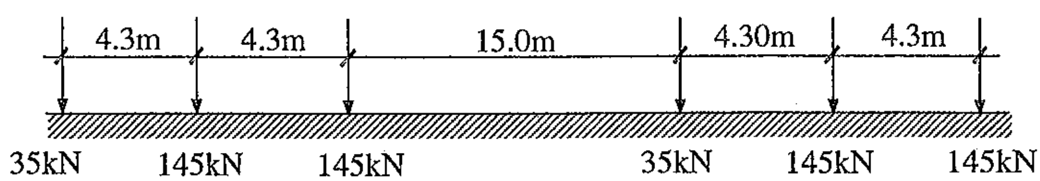
Design Tandem



Design Lane



Design 2 Trucks - 15m



Dynamic load allowance IM of vehicular takes as below table.

Components	IM
Deck Joints - all limit states	75%
All other components	
• Fatigue and fracture limit state	15%
• All other limit states	25%

Dynamic load allowance need not be applied to foundation components that are entirely below ground level.

Unless otherwise specified, the extreme force effect shall be taken as the larger of the following:

- The effect of the design tandem combined with the effect of the design lane load, or
- The effect of one design truck with the variable axle combined with the effect of the design lane load, and
- For both negative moment between points of contraflexure under a uniform load on all spans, and reaction at interior piers only, 90 percent of the effect of two design trucks spaced a minimum of 15 000 mm between the lead axle of one truck and the rear axle of the other truck, combined with 90 percent of the effect of the design lane load. The distance between the 145 000-N axles of each truck shall be taken as 4300 mm.

Multiple presence factors "m" for live load.

Number of loaded lanes	Multiple presence factors "m"
1	1.20
2	1.00
3	0.85
>3	0.65

3.4. Braking force

The braking force shall be taken as the greater of:

- 25 percent of the axle weights of the design truck or design tandem or
- 5 percent of the design truck plus lane load or 5 percent of the design tandem plus lane load.

This braking force shall be placed in all design lanes which are considered to be loaded in accordance with Article 3.6.1.1.1 and which are carrying traffic headed in the same direction. These forces shall be assumed to act horizontally at a distance of 1800 mm above the roadway surface in either longitudinal direction to cause extreme force effects. All design lanes shall be simultaneously loaded for bridges likely to become one-directional in the future. The multiple presence factors specified in Article 3.6.1.1.2 shall apply.

3.5. Temperature load

3.5.1. Uniform temperature:

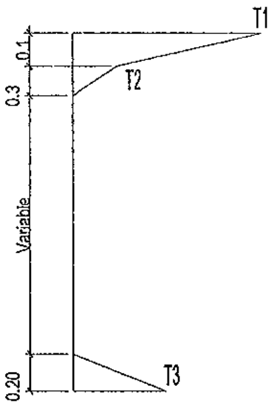
Annual average Temp.	Maximum Temp.	Minimum Temp.
+25^{0}C	+47^{0}C	10^{0}C

- Reference temperature: +25^{0}C.
- Uniform temperature:

Structure (^{0}C)	Increase	Decrease
Reinforcement concrete	***+22.0***	***-15.0***

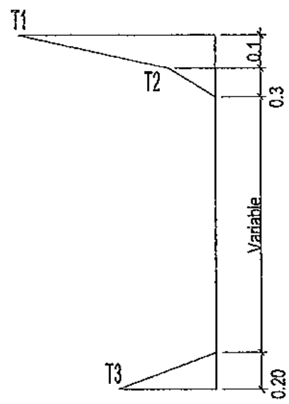
3.5.2. Gradient temperature:

Positive gradient



$T1 = 23^0 C$
$T2 = 6^0 \ C$
$T3 = 3^0 \ C$

Negative gradient



$T1 = - 7^0 C$
$T2 = - 1^0 C$
$T3 = 0^0 C$

3.6. Wind load

Static wind load:

Bridges of package A4 locates in Quang Ngai province., According to *TCVN 2737-1995* - Appendix E, bridge is in wind region III. Following *22TCN-272-05*, sec. 3.8.1:

Design wind speed $V = S.V_B$ where $V_B = $ ***53.0 m/s***

V_B – 3 second gust wind velocity with 100 years return period can take from table as below:

Wind region TCVN 2737-1995	V_B (m/s)
I	38
II	45
III	53
IV	59

S – correct coefficient for wind zone and elevation of deck slab can takes from table as below:

Elevation of deckslab upper ground area or water plane	Exposed area	Forest, houses area with maximum trees, houses height 10m	houses area with houses height > 10m
10	1.09	1.00	0.81
20	1.14	1.06	0.89
30	1.17	1.10	0.94
40	1.20	1.13	0.98
50	1.21	1.16	1.01

Elevation of deckslab upper ground area - exposed area.

Horizontal wind pressure $P_D = 0.0006\ V^2\ C_d \geq 1.8$ (kN/m2)

C_d – obstacle coefficient depends on ratio b/d

b – overall width between handrails

d – superstructure height including solid parapet

Wind load on vehicular:

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

3.7. Earth Quake

a. Acceleration coefficient, site coefficient and response spectrum:

As for earth quake distribution map of Viet Nam in *22TCN-221-95*, bridge locates in seismic zone with grade 7 following MSK-64 scale. According to *22TCN-272-05* sec 3.10.4, Package A4 is in seismic zone No.2.

Acceleration coefficient	Seismic zone	MSK-64 scale
$A \leq 0.09$	1	grade ≤ 6.5
$0.09 < A \leq 0.19$	2	$6.5 <$ grade ≤ 7.5
$0.19 < A < 0.29$	3	$7.5 <$ grade ≤ 8

According to "Seismic standard *TCXDVN375-2006*", peak ground acceleration coefficient:

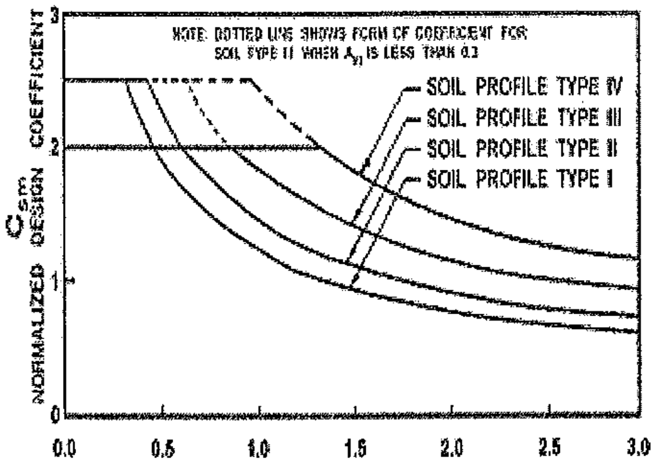
No	Bridge	A
1	OP11	0.0310
2	CB11	0.0310
3	ORB11	0.0310
4	ORB12	0.0310
5	CB12	0.0310
6	ORB13	0.0310
7	OP11A	0.0310
8	LRB09	0.0310
9	CB13	0.0310

Coefficient site S: according to geological data survey, at Package 6, ***soil profile type is I***.

Site coefficient	Soil profile type			
	I	II	III	IV
S	1.0	1.2	1.5	2.0

Seismic design response spectrum for soil profile type I for Package A4 brige is stipulated in specification for bridge design 22TCN-272-05.

Response spectrum:



b. Response modification factor:

Because of important of briges in package A4 - ***essential category***, response modification factors show in table as below are proposed in design.

Response Modification Factor

Components	R
Single column	2.0
Multiple column bents	3.5
Connection: columns, piers, or pile bents to cap beam or superstructure; columns or	1.0

piers to foundations	
Foundations	1.0

c. Analysis:

Seismic demands shall be determined by elastic response spectrum analysis. The number of modes included in the analysis shall be sufficient to get a participating mass of approximately 85-90%. The seismic response spectrum is defined according to Section 3.10.6 of 22TCN-272-05.
Response Spectrum - Single mode method is simultaneously used.
Combination of seismic force effects in different directions is used as follows: 100% for one of the perpendicular directions combined with 30% for the other perpendicular direction.

3.8. Creep and Shrinkage

Creep and shrinkage effect are followed CEB-FIB model 1990 , base on construction schedule, material characteristic and structural dimensions. See attached construction schedule .
Average annual humidity is ***80.0%.***

4. LOAD MODIFIER FACTORS AND LOAD COMBINATIONS

4.1. Load modifier factors

Each component and connection shall satisfy equation as below for each limit state, unless otherwise specified. For service and extreme event limit states, resistance factors shall be taken as 1.0, except for bolts, and for concrete columns in Seismic Zones 3 and 4. All limit states shall be considered of equal importance.

$$\Sigma \eta i.\ \gamma i.\ Qi \leq \phi.\ Rn = Rr$$

Where:

- ηi: load modifier factors
- γi: load factors
- ϕ: resistance factors
- Qi: forces effect
- Rn: norminal resistance
- Rr: factored reistance

For loads for which a maximum value of γi is appropriate: $\eta i = \eta_D \times \eta_R \times \eta_I \geq 0.95$
For loads for which a minimum value of γi is appropriate: $\eta i = 1/(\eta_D \times \eta_R \times \eta_I) \leq 1.0$

For this bridge:

Factors	Sign	Limit states	Value
Factor relating to ductility	η_D	Strength	1.0
		Other	1.0
Factor relating to redundancy	η_R	Strength	1.0
		Other	1.0
Factor relating to operational importance	η_I	Strength	1.0
		Other	1.0

Load modifier factors used in design:

Load modifier factors for loads for which	ηi
A maximum value of load factors is appropriate	***1.0***
A minimum value of load factors is appropriate	***1.0***

4.2. Load combinations

Design load combinations:

Load Combinations / Limit states	DC	DW	LL IM CE BR PL LS EL	WA	WS	WL	TU CR SH	TG	SE	EQ	CT
Strength – IA	1.25	1.50	1.75	1.00	-	-	0.50	-	0.50	-	-
Strength – IB	0.90	0.65	1.75	1.00	-	-	0.50	-	0.50	-	-
Strength – IIA	1.25	1.50	-	1.00	1.40	-	0.50	-	0.50	-	-
Strength – IIB	0.90	0.65	-	1.00	1.40	-	0.50	-	0.50	-	-
Strength – IIIA	1.25	1.50	1.35	1.00	0.40	1.00	0.50	-	0.50	-	-
Strength – IIIB	0.90	0.65	1.35	1.00	0.40	1.00	0.50	-	0.50	-	-
Service – I	1.00	1.00	1.00	1.00	0.30	1.00	1.00	0.50	0.50	-	-
Service – III	1.00	1.00	0.80	1.00	0.30	1.00	1.00	0.50	0.50	-	-
ExtremeT – IA	1.25	1.50	0.50	1.00	-	-	-	-	-	1.00	-
ExtremeT – IB	0.90	0.65	0.50	1.00	-	-	-	-	-	1.00	-
ExtremeL – IC	1.25	1.50	0.50	1.00	-	-	-	-	-	1.00	-
ExtremeL – ID	0.90	0.65	0.50	1.00	-	-	-	-	-	1.00	-
ExtremeT - IIA	1.25	1.50	0.50	1.00	-	-	-	-	-	-	1.00
ExtremeT - IIB	0.90	0.65	0.50	1.00	-	-	-	-	-	-	1.00
ExtremeL - IIC	1.25	1.50	0.50	1.00	-	-	-	-	-	-	1.00
ExtremeL - IID	0.90	0.65	0.50	1.00	-	-	-	-	-	-	1.00

Where:

DC	Dead load
DW	Pavement dead load
BR	Braking force
IM	Impact load
LL	Live load
PL	Pedestrian load
SE	Settlement
CR	Creep
SH	Shrinkage
TG	Gradient temperature
TU	Uniform temperature
WA	Water pressure
WL	Wind load on vehicular
WS	Wind load on structure
EQ	Earth quake
CT	Vehicular collision force
EL	Accumulated locked-in force

Other loads, such as: horizontal earth pressure (EH), earth surcharge load (ES), vertical pressure from dead load of earth fill (EV) are used for substructure design, load factors of these conform to bridge design specification *22TCN 272-05*.

CALCULATION SHEET

I21 GIRDER

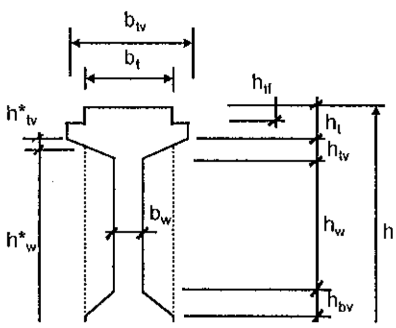
CONTENT

1. INPUT DATA:

1.1. General Data

Specification for bridge design:	TCN 272-05		
Live load	HL93		
Impact (or dynamic) of the live load	IM =	0.25	
Pedestrian	PL =	0.00	(kN/m^2)
Length of Girder	L_d =	21.00	(m)
Span between support	L_{tt} =	20.30	(m)
Carriageway width in bridge	w =	11.75	(m)
Parapet width	c =	0.50	(m)
Bridge width	B =	12.75	(m)
Number of girder	N_d =	5.00	girder
Space betweem 2 girders	S =	2.55	(m)
Distance from inside of parapet to exterior girder center	d_e =	0.78	(m)
Width of bridge deck	b_{ds} =	12.48	(m)
Length of the overhang (cantilever arm length)	L_h =	1.28	(m)
Thickness of bridge deck	t_s =	0.22	(m)
Precast plank width	b_p =	1.95	(m)
Precast plank thick	h_p =	0.08	(m)
Pavement thick	h_{pa} =	0.084	(m)

1.2. Girder dimension:



Width of over part		b_{tv} =	800.00	(mm)
		b_t =	600.00	(mm)
Width of under part		b_b =	600.00	(mm)
Girder high		h =	1200.00	(mm)
		h_{tf} =	80.00	(mm)
		h_t =	200.00	(mm)
Cross section at end			*at the midle*	
b_w =	600.00		200.00	(mm)
h^*_{tv} =	35.00		110.00	(mm)
h^*_w =	965.00		440.00	(mm)
h_b =	0.00		250.00	(mm)
h_{bv} =	0.00		200.00	(mm)

1.3. MATERIAL PROPERTIES:

1.3.1 Concrete:

Girder concrete

Girder concrete strength at the 28 age days	f'_c =	45.00	MPa	
Unit weight of Concrete	γ_c =	2400.00	kG/m^3	
Modulus of elasticity	$E_c = 0.043\ \gamma_c^{1.5}\ sqrt(f'_c)$ =	33914.98	MPa	(5.4.2.4-1)

Deck concrete

Deck concrete strength at the 28 age days	f'_c =	35.00	MPa	
Unit weight of concrete	γ_c =	2400.00	kG/m^3	
Modulus of elasticity	$E_c = 0.043\ \gamma_c^{1.5}\ sqrt(f'_c)$ =	29910.20	MPa	(5.4.2.4-1)

1.3.2 Prestressing steel

Diameter of one strand	D =	12.70	mm
Area of one strand	$A_s^{12.7}$ =	98.70	mm^2
Ultimate Tendon strength	f_{pu} =	1860.00	MPa
Yield strength of prestressing steel	$f_{py} = 0.9\ f_{pu}$ =	1674.00	MPa
Modulus of strand	E_p =	197000.00	MPa
Wobble friction coefficient (mm-1)	K =	6.60E-07	mm^{-1}
Coeficient of friction (1/RAD)	μ =	0.25	
Number of Strands in one Tendon	n =	9.00	Strands
Area of one Tendon	A_s =	888.30	mm^2
Stress in the prestressing steel at jacking	$f_{pj} = 0.7\ f_{pu}$ =	1302.00	MPa
Jacking force for one tendon	P_j =	1156.57	kN
Anchorage set	ΔL =	6.00	mm
Area of one duck	A_g =	3318.31	mm^2
Number of Tendons	N =	4.00	Tendons

1.3.3 Reinforcing Steel:

Yield strength (deformed bar)	f_{py} =	400.00	(MPa)
Modulus of steel	E_s =	200000.00	(MPa)

2. INTERNAL FORCE:

2.1. Dead Load:

2.1.1 Load:

Interior Beam:

Bridge deck	DC_d =	12.94 (kN/m)
Precat plank & cross beam	DC_{pl} =	4.55 (kN/m)
Parapet	DC_{pa} =	4.74 (kN/m)
Pavement	DW_p =	4.44 (kN/m)

Exterior Beam:

Bridge deck	DC_d =	12.94 (kN/m)
Precat plank & cross beam	DC_{pl} =	2.27 (kN/m)
Parapet	DC_{pa} =	4.80 (kN/m)
Pavement	DW_p =	4.44 (kN/m)

2.1.2 Internal Force due to dead load:

Formula :

$M = 0.5\, q.X_i\, (L\text{-}X_i)$

$Q = q.(0.5.L\text{-}X_i)$

L_{tt} = 20.30 (m)

		INTERIOR GIRDER									
		Girder (DC)		Concrete Deck (DC)		Plank & cr.beam (DC)		Parapet (DC)		Pavement (DW)	
Section	X_i (m)	M (kNm)	Q (kN)	M (kNm)	Q (kN)	M (kNm)	Q (kN)	M (kNm)	Q (kN)	M (kNm)	Q (kN)
Support	0.00	0.00	149.57	0.00	131.33	0.00	46.14	0.00	48.11	0.00	45.08
L/8	2.54	332.08	112.17	291.60	98.50	102.46	34.61	106.82	36.08	100.09	33.81
L/4	5.08	569.29	74.78	499.89	65.67	175.64	23.07	183.12	24.06	171.59	22.54
3L/8	7.61	711.61	37.39	624.86	32.83	219.55	11.54	228.90	12.03	214.49	11.27
L/2	10.15	759.05	0.00	666.52	0.00	234.18	0.00	244.16	0.00	228.79	0.00
		EXTERIOR GIRDER									
Gối	0.00	0.00	149.57	0.00	131.33	0.00	46.14	0.00	48.67	0.00	45.08
L/8	2.54	332.08	112.17	291.60	98.50	51.23	34.61	108.06	36.50	100.09	33.81
L/4	5.08	569.29	74.78	499.89	65.67	87.82	23.07	185.25	24.33	171.59	22.54
3L/8	7.61	711.61	37.39	624.86	32.83	109.77	11.54	231.56	12.17	214.49	11.27
L/2	10.15	759.05	0.00	666.52	0.00	117.09	0.00	247.00	0.00	228.79	0.00

2.2 Live Load:

2.2.1. Distribution factors for Live load:

Modular Ratio: Girder Concrete/Deck Concrete	$n = E_g / E_d$ =	1.13
Distance from girder centroid to bridge deck centroid	$e^I g$ =	699.42 (mm)
	$e^E g$ =	699.42 (mm)
Longgitudinal stiffness parameter	$K^I g = n(Ig + A\, e^2_g)$ =	7.6E+11
	$K^E g = n(Ig + A\, e^2_g)$ =	7.6E+11
Ration	$K^I g / (L\, t^3_s)$ =	3.74
	$K^E g / (L\, t^3_s)$ =	3.74
	S / L =	0.13

(a) Distribution Factor for Moment: g(M)

Interior Beam:

For one lane $$0.06+\left(\frac{S}{4300}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{Lt_s^3}\right)^{0.1} = \quad \mathbf{0.557}$$

Two or more lanes $$0.075+\left(\frac{S}{2900}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{Lt_s^3}\right)^{0.1} = \quad \mathbf{0.772}$$

Exterior Beam:

For one lane, follow the lever rule

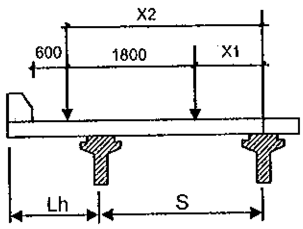
X_1 = 925.000

X_2 = 2725.00

Y_1 = 0.363

Y_2 = 1.069

=> $g(M) = 0.5^*\Sigma yi = \boxed{0.716}$



Two or more lanes

$$e = 0.77 + \frac{d_e}{2800} = \quad 1.047 < (=) 1$$

Choice e = 1.047 IF(e>1,1,e)

=> $g(M) = e^* g_{trong} = \boxed{0.809}$

(b) Distribution Factor for Shear force: g(Q)

Interior Beam:

For one lane $$0.36 + \frac{S}{7600} = \quad \mathbf{0.696}$$

Two or more lanes $$0.2 + \frac{S}{3600} - \left(\frac{S}{10700}\right)^2 = \quad \mathbf{0.852}$$

Exterior Beam:
For one lane, follow the lever rule

$g(Q) = 0.5^*\Sigma yi = \boxed{0.716}$

Two or more lanes

$$e = 0.6 + \frac{de}{3000} = \quad 0.858$$

$=> g(Q) = e^* g_{trong} = \boxed{0.731}$

(c) Correction factor for skew bridge:

* Correction factor of distribution factor for moment (Table 4.6.2.2.2d-1)

Skew angle	θ =	20	Degree.	
Factor	c1 =	0.000		
Correction factor	CF(M) =	1.000		

Area of applications
300<= θ <= 600
1100<= S <= 4900
6000 <= L <= 73000
Nb >= 4

$$CF(M) = 1.0 - c1.(\tan\theta)^{1.5}$$
$$c1 = 0.25\left(\frac{Kg}{L.ts^3}\right)^{0.25}\left(\frac{S}{L}\right)^{0.5}$$

* Regulation factor of distribution factor for shear force (Table 4.6.2.2.3c-1)

Correction Factor CF(Q)= 1.049

Area of applications
00<= θ <= 600
1100<= S <= 4900
6000 <= L <= 73000
Nb >= 4

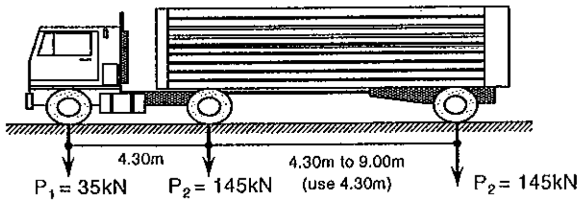
$$CF(Q) = 1.0 + 0.20\left(\frac{L.ts^3}{Kg}\right)^{0.3}.\tan\theta$$

(d) Table of Distribution factors for Live load:

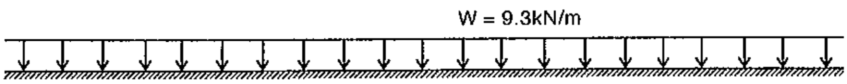
Interior Beam	g(M)	g(Q)	m	m*g(M)	m*g(Q)	m*g(M)*CF(M)	m*g(Q)*CF(Q)
1 lane	0.557	0.696	1.20	0.668	0.835	0.668	0.876
2 or more lanes	0.772	0.852	1.00	0.772	0.852	**0.772**	**0.893**
Exterior Beam							
1 lane	0.716	0.716	1.20	0.859	0.859	**0.859**	**0.901**
2 or more lanes	0.809	0.731	1.00	0.809	0.731	0.809	0.767

2.2.2 Live Load:

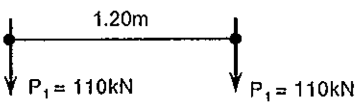
Design Truck



Design Lane Load



Design Tendem



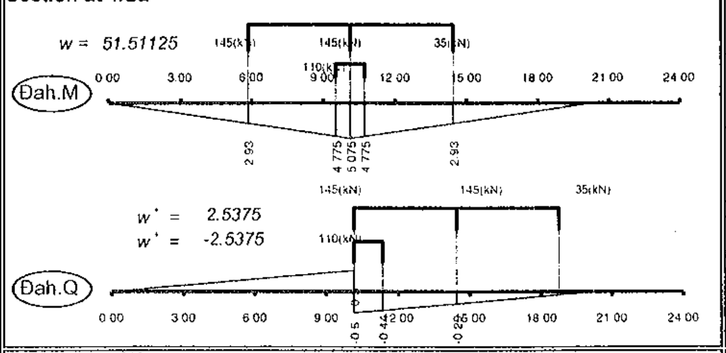
- Truck	P1 =	35.00 (kN)
	P2 =	145.00 (kN)
- Lane load	W =	9.30 (kN)
- Tandem	P1 =	110.00 (kN)
- Pedestrian	PL =	0.00 kN/m^2
- Dynammic load	IM =	0.25

2.2.3 Internal Force due to Live load:

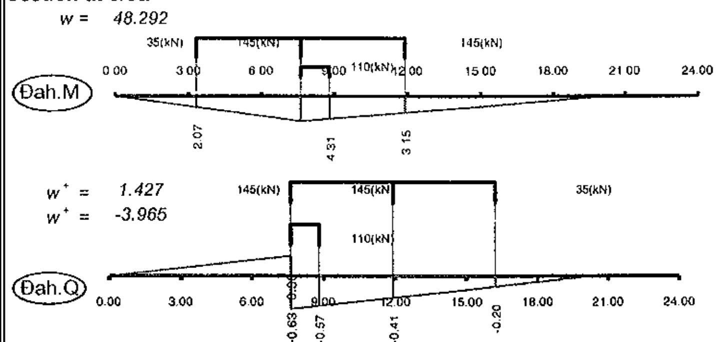
Design truck or Tendem	Momen	$M_{TR(Ta)} = \Sigma Piyi$	(kNm)
	Shear force	$Q_{TR(Ta)} = \Sigma Piyi$	(kN)
Lane load	Momen	$M_{Ln} = W.F_M$	(kNm)
	Shear force	$Q_{Ln} = W.F_Q$	(kN)
Pedestrian	Momen	$M_{PL} = PL.F_M$	(kNm)
	Shear force	$Q_{PL} = PL.F_Q$	(kN)

Influence line for Momen & Shear force	Load	Momen (KN.m)	Shear
Section at 1/2L	Truck	1262.38	116.96
	Lane	479.05	23.60
	Tandem	1050.50	103.50
	Design	**1262.38**	**116.96**
	Pedestrian	0.00	0.00
Section at 3/8L	Truck	1218.41	157.58
	Lane	449.11	36.87
	Tandem	997.22	131.00
	Design	**1218.41**	**157.58**
	Pedestrian	0.00	0.00
Section at 1/4L	Truck	1005.91	198.21
	Lane	359.29	53.10
	Tandem	804.38	158.50
	Design	**1005.91**	**198.21**
	Pedestrian	0.00	0.00
Section at 1/8L	Truck	606.04	238.83
	Lan	209.59	72.27
	Tandem	471.97	186.00
	Design	**606.04**	**238.83**
	Pedestrian	0.00	0.00
At support	Truck	0.00	279.46
	Lane	0.00	94.40
	Tandem	0.00	213.50
	Design	**0.00**	**279.46**
	Pedestrian	0.00	0.00

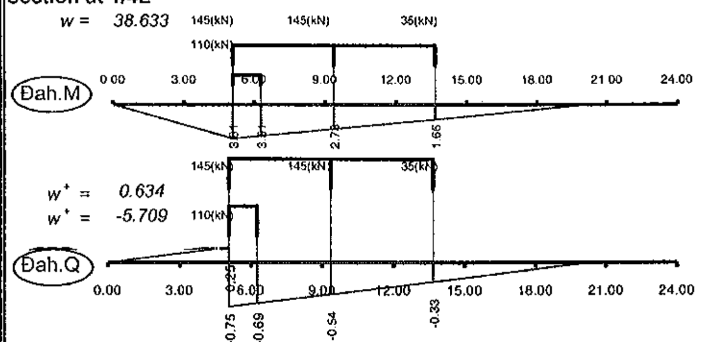
Section at 1/2L



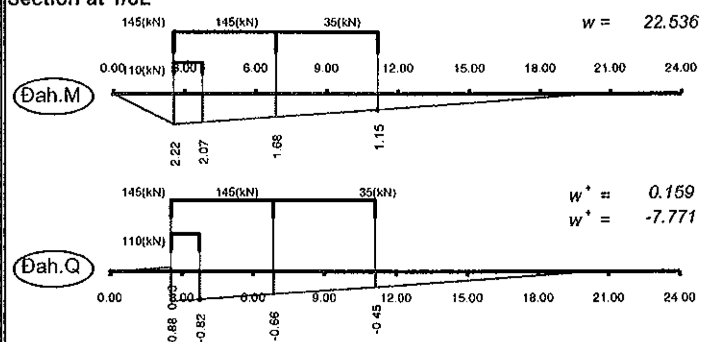
Section at 3/8L



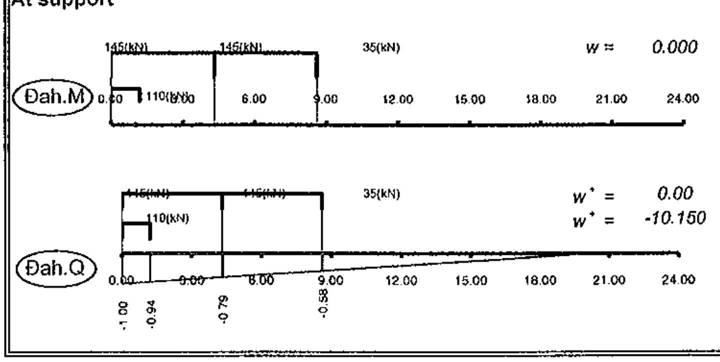
Section at 1/4L



Section at 1/8L



At support



Internal Force due to Live load : $M_{(LL+IM)} = m*g(M) * [max\{M_{TR}, M_{Ta}\}*(1+IM) + M_{Ln}]$

$Q_{(LL+IM)} = m*g(Q) * [max\{Q_{TR}, Q_{Ta}\}*(1+IM) + Q_{Ln}]$

Internal Force due to pedestrian :

$M = g(M) * M_{PL}$

$Q = g(Q) * Q_{PL}$

In which:

$M_{TR(Ta)}$ moment due to truck or Tandem

$Q_{TR(Ta)}$ Shear force due to truck or Tandem

yi Value of influence line F Area of influence line

m Lane factor g Distribution factor

	m*g(M)	m*g(Q)
Interior	0.772	0.893
Exterior	0.859	0.901

TABLE OF INTERNAL FORCE DUE TO LIVE LOAD

Setion	Xi	Interior Girder		Exterior Girder	
		M	Q	M	Q
	(m)	(kNm)	(kN)	(kNm)	(kN)
Support	0.00	0.00	396.37	0.00	399.76
L/8	2.54	747.09	331.24	830.60	334.07
L/4	5.08	1248.84	268.75	1388.44	271.05
3L/8	7.61	1523.42	208.90	1693.71	210.68
L/2	10.15	1589.00	151.68	1766.62	152.97

2.3 Load combination:

Strength limit state: $U = \eta\ [1.25\ DC + 1.50\ DW + 1.75\ (LL+IM)]$

Service limit state: $U = \eta\ [1.00\ DC + 1.00\ DW + 1.00\ (LL+IM)]$

Fatigue state: $U = 0.75\ (LL+IM)]$

The modify load factort $\eta = \eta_D\ \eta_R\ \eta_I$

STATE	Modify Load Factor			
	η_D	η_R	η_I	$\eta = \eta_D\ \eta_R\ \eta_I$
Strength	1.00	1.00	1.00	1.00
Service	1.00	1.00	1.00	1.00

2.3.1 Load combination - - Interior Girder:

STATE	Strength										
Load	Load Factor	Section									
		Support		L/8		L/4		3L/8		L/2	
	γ	M (kNm)	Q (kN)	M (kNm)	Q (kN)	M (kNm)	Q (kN)	M (kNm)	Q (kN)	M (kNm)	Q (kN)
DC	1.25	0.00	468.94	1041.20	351.71	1784.92	234.47	2231.15	117.24	2379.89	0.00
DW	1.50	0.00	67.62	150.14	50.72	257.39	33.81	321.73	16.91	343.18	0.00
LL+IM	1.75	0.00	693.64	1307.40	579.67	2185.47	470.31	2665.98	365.57	2780.74	265.43
Total		**0.00**	**1230.21**	**2498.75**	**982.10**	**4227.77**	**738.60**	**5218.87**	**499.71**	**5503.82**	**265.43**

STATE	Service										
Load	Load Factor	Section									
		Support		L/8		L/4		3L/8		L/2	
	γ	M (kNm)	Q (kN)	M (kNm)	Q (kN)	M (kNm)	Q (kN)	M (kNm)	Q (kN)	M (kNm)	Q (kN)
DC	1.00	0.00	375.16	832.96	281.37	1427.94	187.58	1784.92	93.79	1903.92	0.00
DW	1.00	0.00	45.08	100.09	33.81	171.59	22.54	214.49	11.27	228.79	0.00
LL+IM	1.00	0.00	396.37	747.09	331.24	1248.84	268.75	1523.42	208.90	1589.00	151.68
Total		**0.00**	**816.60**	**1680.14**	**646.42**	**2848.36**	**478.87**	**3522.83**	**313.96**	**3721.70**	**151.68**

2.3.2 Load combination - Exterior Girder:

STATE	Strength										
Load	Load factor	Section									
		Supprt		L/8		L/4		3L/8		L/2	
	γ	M (kNm)	Q (kN)	M (kNm)	Q (kN)	M (kNm)	Q (kN)	M (kNm)	Q (kN)	M (kNm)	Q (kN)
DC	1.25	0.00	469.64	978.72	352.23	1677.80	234.82	2097.25	117.41	2237.07	0.00
DW	1.50	0.00	67.62	150.14	50.72	257.39	33.81	321.73	16.91	343.18	0.00
LL+IM	1.75	0.00	699.57	1453.55	584.63	2429.77	474.34	2964.00	368.70	3091.59	267.70
Total		**0.00**	**1236.84**	**2582.41**	**987.58**	**4364.95**	**742.97**	**5382.98**	**503.01**	**5671.84**	**267.70**

STATE	Service										
Load	load factor	Section									
		Support		L/8		L/4		3L/8		L/2	
	γ	M (kNm)	Q (kN)	M (kNm)	Q (kN)	M (kNm)	Q (kN)	M (kNm)	Q (kN)	M (kNm)	Q (kN)
DC	1.00	0.00	375.71	782.97	281.79	1342.24	187.86	1677.80	93.93	1789.66	0.00
DW	1.00	0.00	45.08	100.09	33.81	171.59	22.54	214.49	11.27	228.79	0.00
LL+IM	1.00	0.00	399.76	830.60	334.07	1388.44	271.05	1693.71	210.68	1766.62	152.97

Total			0.00	820.55	1713.67	649.67	2902.27	481.45	3586.00	315.88	3785.06	152.97

3. TENDON PROFILE AND PROPERTY OF GIRDER CROSS SECTION

3.1. Tendon profile:

Tendon profile follow Parabol equation:

$$y_i = f - \frac{4.(f - c).x.(l - x)}{l^2}$$

in which: Origin of coordinates in left edge of the Girder bottom (0.0)

- f — Maximum deflection at mid span of tendon
- c — Distance from maximum deflection point to girder bottom
- (x_i, y_i) — Coordination of point under consider i = 1,2...
- L — actual distance between cable ends (X-axis)
- $L_p = X_2 - X_1$ — Distance between 2 point under consider

angle of rotation of tendon for X_i-axis $\quad \mathrm{Tan}(\alpha) = (4.f\,(1-2.Xi / L)) / L$

$\alpha = 2\,f / 0.5\,L - \tan(\alpha)$

L_{span} =	21000	(mm)
$L_{su.}$ =	20300	(mm)
L_{cap} =	20700	(mm)

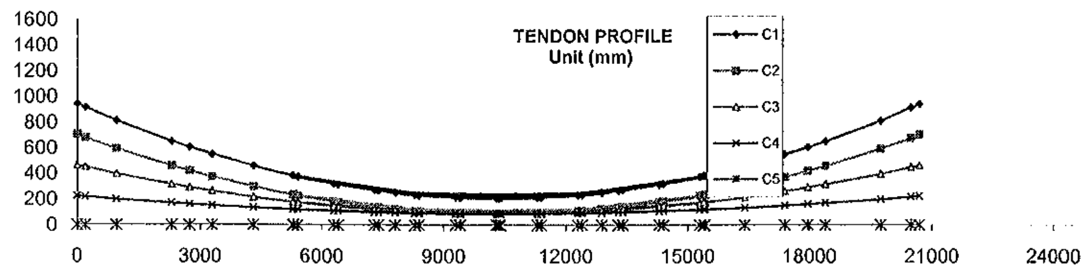
	f =	945	(mm)	Lcáp =	20700	(mm)	C =	210	(mm)
	Section	Xi	Yi	Lp	ΣLcáp	Tan(αi)	(α)	Σα	Section
		(mm)	(mm)	(mm)	(mm)		(rad)	(rad)	
TENDON No 1	Anchorage	0.00	945.00	0.00	0.00	0.1826	0.0000	0.0000	Anchorage
	Support	200.00	916.87	200.00	200.00	0.1791	0.0035	0.0035	Support
	L/8	2737.50	607.61	2537.50	2737.50	0.1343	0.0483	0.0518	L/8
	L/4	5275.00	386.72	2537.50	5275.00	0.0895	0.0931	0.1449	L/4
	3L/8	7812.50	254.18	2537.50	7812.50	0.0448	0.1378	0.2827	3L/8
	L/2	10350.00	210.00	2537.50	10350.00	0.0000	0.1826	0.4653	L/2

	f =	705	(mm)	Lcáp =	20700	(mm)	C =	90	(mm)
	Section	Xi	Yi	Lp	ΣLcáp	Tan(α)	(α)	Σα	Section
		(mm)	(mm)	(mm)	(mm)		(rad)	(rad)	
TENDON No 2	Anchorage	0.00	705.00	0.00	0.00	0.1362	0.0000	0.0000	Anchorage
	Support	200.00	681.46	200.00	200.00	0.1336	0.0026	0.0026	Support
	L/8	2737.50	422.70	2537.50	2737.50	0.1002	0.0360	0.0387	L/8
	L/4	5275.00	237.87	2537.50	5275.00	0.0668	0.0694	0.1081	L/4
	3L/8	7812.50	126.97	2537.50	7812.50	0.0334	0.1028	0.2109	3L/8
	L/2	10350.00	90.00	2537.50	10350.00	0.0000	0.1362	0.3472	L/2

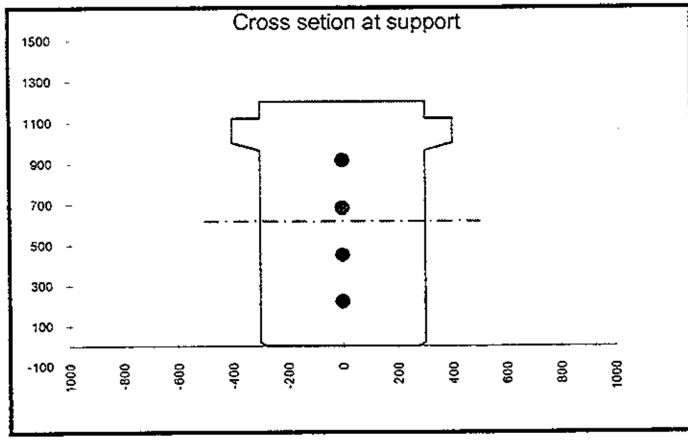
	f =	465	(mm)	Lcáp =	20700	(mm)	C =	90	(mm)
	Section	Xi	Yi	Lp	ΣLcáp	Tan(α)	(α)	Σα	Section
		(mm)	(mm)	(mm)	(mm)		(rad)	(rad)	
TENDON No 3	Anchorage	0.00	465.00	0.00	0.00	0.0899	0.0000	0.0000	Anchorage
	Support	200.00	450.65	200.00	200.00	0.0881	0.0017	0.0017	Support
	L/8	2737.50	292.86	2537.50	2737.50	0.0661	0.0238	0.0255	L/8
	L/4	5275.00	180.16	2537.50	5275.00	0.0441	0.0458	0.0713	L/4
	3L/8	7812.50	112.54	2537.50	7812.50	0.0220	0.0678	0.1391	3L/8
	L/2	10350.00	90.00	2537.50	10350.00	0.0000	0.0899	0.2290	L/2

	f =	225	(mm)	Lcáp =	20700	(mm)	C =	90	(mm)
	Section	Xi	Yi	Lp	ΣLcáp	Tan(α)	(α)	Σα	Section
		(mm)	(mm)	(mm)	(mm)		(rad)	(rad)	
TENDON No 4	Anchorage	0.00	225.00	0.00	0.0	0.0435	0.0000	0.0000	Anchorage
	Support	200.00	219.83	200.00	200.0	0.0426	0.0008	0.0008	Support
	L/8	2737.50	163.03	2537.50	2737.5	0.0320	0.0115	0.0123	L/8
	L/4	5275.00	122.46	2537.50	5275.0	0.0213	0.0222	0.0345	L/4
	3L/8	7812.50	98.11	2537.50	7812.5	0.0107	0.0328	0.0673	3L/8
	L/2	10350.00	90.00	2537.50	10350.0	0.0000	0.0435	0.1108	L/2

Section	TENDON No 1		TENDON No 2		TENDON No 3		TENDON No 4	
	X_i (mm)	Y_i (mm)	X_i (mm)	Y_i (mm)	X_i (mm)	Y_i (mm)	X_i (mm)	Y_i (mm)
Anchorage	0.00	945.00	0.0	705.00	0.0	465.00	0.00	225.00
Support	**200.00**	**916.87**	**200.0**	**681.46**	**200.0**	**450.65**	**200.00**	**219.83**
1	950.00	816.26	950.0	597.28	950.0	399.32	950.00	201.35
2	2300.00	654.63	2300.0	462.04	2300.0	316.85	2300.00	171.67
3	3300.00	551.02	3300.0	375.35	3300.0	263.99	3300.00	152.64
L/8	**2737.50**	**607.61**	**2737.5**	**422.70**	**2737.5**	**292.86**	**2737.50**	**163.03**
4	4300.00	461.14	4300.0	300.14	4300.0	218.13	4300.00	136.13
5	5300.00	384.98	5300.0	236.41	5300.0	179.28	5300.00	122.14
6	6300.00	322.54	6300.0	184.17	6300.0	147.42	6300.00	110.67
7	7300.00	273.83	7300.0	143.41	7300.0	122.56	7300.00	101.72
L/4	**5275.00**	**386.72**	**5275.0**	**237.87**	**5275.0**	**180.16**	**5275.00**	**122.46**
8	8300.00	238.83	8300.0	114.13	8300.0	104.71	8300.00	95.30
9	9300.00	217.56	9300.0	96.33	9300.0	93.86	9300.00	91.39
10	10300.00	210.02	10300.0	90.01	10300.0	90.01	10300.00	90.00
11	11300.00	216.19	11300.0	95.18	11300.0	93.16	11300.00	91.14
3L/8	**7812.50**	**254.18**	**7812.5**	**126.97**	**7812.5**	**112.54**	**7812.50**	**98.11**
12	12300.00	236.09	12300.0	111.83	12300.0	103.31	12300.00	94.79
13	13300.00	269.71	13300.0	139.96	13300.0	120.46	13300.00	100.97
14	14300.00	317.05	14300.0	179.58	14300.0	144.62	14300.00	109.66
15	15300.00	378.12	15300.0	230.67	15300.0	175.78	15300.00	120.88
L/2	**10350.00**	**210.00**	**10350.0**	**90.00**	**10350.0**	**90.00**	**10350.00**	**90.00**
2	5400.00	378.12	5400.0	230.67	5400.0	175.78	5400.00	120.88
3	6400.00	317.05	6400.0	179.58	6400.0	144.62	6400.00	109.66
4	7400.00	269.71	7400.0	139.96	7400.0	120.46	7400.00	100.97
5	8400.00	236.09	8400.0	111.83	8400.0	103.31	8400.00	94.79
-	**12887.50**	**254.18**	**12887.5**	**126.97**	**12887.5**	**112.54**	**12887.50**	**98.11**
6	9400.00	216.19	9400.0	95.18	9400.0	93.16	9400.00	91.14
7	10400.00	210.02	10400.0	90.01	10400.0	90.01	10400.00	90.00
8	11400.00	217.56	11400.0	96.33	11400.0	93.86	11400.00	91.39
9	12400.00	238.83	12400.0	114.13	12400.0	104.71	12400.00	95.30
-	**15425.00**	**386.72**	**15425.0**	**237.87**	**15425.0**	**180.16**	**15425.00**	**122.46**
10	13400.00	273.83	13400.0	143.41	13400.0	122.56	13400.00	101.72
11	14400.00	322.54	14400.0	184.17	14400.0	147.42	14400.00	110.67
12	15400.00	384.98	15400.0	236.41	15400.0	179.28	15400.00	122.14
13	16400.00	461.14	16400.0	300.14	16400.0	218.13	16400.00	136.13
-	**17962.50**	**607.61**	**17962.5**	**422.70**	**17962.5**	**292.86**	**17962.50**	**163.03**
14	17400.00	551.02	17400.0	375.35	17400.0	263.99	17400.00	152.64
14	18400.00	654.63	18400.0	462.04	18400.0	316.85	18400.00	171.67
16	19750.00	816.26	19750.0	597.28	19750.0	399.32	19750.00	201.35
Support	20500.00	916.87	20500.0	681.46	20500.0	450.65	20500.00	219.83
Anchorage	20700.00	945.00	20700.0	705.00	20700.0	465.00	20700.00	225.00



3.2 Property of Girder Cross section at transfer (Stage I: net cross section):

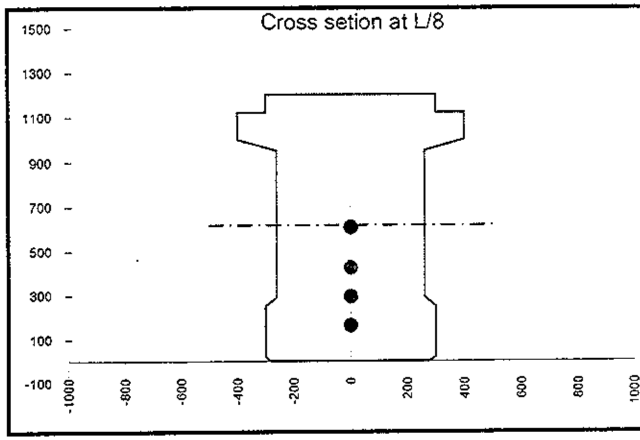


A	0.7471	m^2
e_x	0.0000	m
e_y	0.6169	m
I_x	0.0917	m^4
I_y	0.0249	m^4
I_{xy}	0.0000	m^4
y_b	0.6169	m
y_t	0.5831	m
S_b	0.1486	m^3
S_t	0.1572	m^3
$E_{cáp\,(Y)}$	0.0497	m
q =	17.9304	(KN/m)

ĐẶC TR NG HÌNH HỌC - MẶT CẮT L/4

Âiãøm goïc	x (mm)	y (mm)	A (m^2)	Q_X' (m^3)	Q_Y' (m^3)	I_X' (m^4)	I_Y' (m^4)	$I_X'Y'$ (m^4)
Đ òng bao								
-	0.00	0.00	-	-	-	-	-	-
1	280.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	300.00	20.00	0.01	0.00	0.00	0.00	0.00	0.00
3	300.00	250.00	0.07	0.02	0.04	0.00	0.02	0.01
4	100.00	450.00	0.11	0.08	0.04	0.04	0.01	0.02
5	100.00	890.00	0.04	0.06	0.01	0.06	0.00	0.01
6	400.00	1000.00	-0.26	-0.48	-0.13	-0.69	-0.05	-0.18
7	400.00	1120.00	0.05	0.10	0.04	0.16	0.02	0.06
8	300.00	1120.00	0.11	0.25	0.08	0.42	0.04	0.13
9	300.00	1200.00	0.02	0.06	0.01	0.10	0.01	0.03
10	-300.00	1200.00	0.72	1.73	0.00	3.11	0.06	0.00
11	-300.00	1120.00	0.02	0.06	-0.01	0.10	0.01	-0.03
12	-400.00	1120.00	0.11	0.25	-0.08	0.42	0.04	-0.13
13	-400.00	1000.00	0.05	0.10	-0.04	0.16	0.02	-0.06
14	-100.00	890.00	-0.26	-0.48	0.13	-0.69	-0.05	0.18
15	-100.00	450.00	0.04	0.06	-0.01	0.06	0.00	-0.01
16	-300.00	250.00	0.11	0.08	-0.04	0.04	0.01	-0.02
17	-300.00	20.00	0.07	0.02	-0.04	0.00	0.02	-0.01
18	-280.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00
19	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
20								
21								
Tổng cộng			0.5166	0.3144	0.0000	0.2761	0.0142	0.0000

Cáp	x	y			
1	0.00	386.72	Diện tích 1 bó cáp	As =	888.30 (mm2)
2	0.00	237.87	Số bó cáp	n=	4.00 bó
3	47.00	180.16		Σ As Yi =	823634.09 (mm3)
4	-50.00	122.46		Σ As =	3553.20 (mm2)



A	0.6936	m^2
e_x	0.0000	m
e_y	0.6173	m
I_x	0.0896	m^4
I_y	0.0207	m^4
I_{xy}	0.0000	m^4
y_b	0.6173	m
y_t	0.5827	m
S_b	0.1451	m^3
S_t	0.1537	m^3
$E_{cáp\,(Y)}$	0.2457	m
q =	16.6457	(KN/m)

ĐẶC TR NG HÌNH HỌC - MẶT CẮT 3L/8

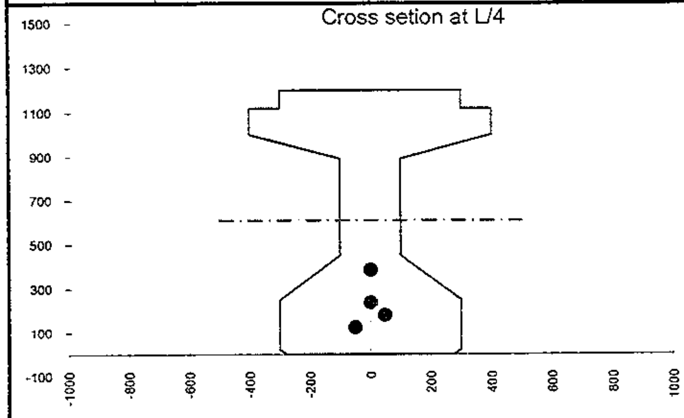
Âiãøm goïc	x (mm)	y (mm)	A (m^2)	Q_X' (m^3)	Q_Y' (m^3)	I_X' (m^4)	I_Y' (m^4)	$I_X'Y'$ (m^4)
Đ òng bao								
-	0.00	0.00	-	-	-	-	-	-
1	280.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	300.00	20.00	0.01	0.00	0.00	0.00	0.00	0.00
3	300.00	250.00	0.07	0.02	0.04	0.00	0.02	0.01
4	100.00	450.00	0.11	0.08	0.04	0.04	0.01	0.02
5	100.00	890.00	0.04	0.06	0.01	0.06	0.00	0.01
6	400.00	1000.00	-0.26	-0.48	-0.13	-0.69	-0.05	-0.18
7	400.00	1120.00	0.05	0.10	0.04	0.16	0.02	0.06
8	300.00	1120.00	0.11	0.25	0.08	0.42	0.04	0.13
9	300.00	1200.00	0.02	0.06	0.01	0.10	0.01	0.03
10	-300.00	1200.00	0.72	1.73	0.00	3.11	0.06	0.00
11	-300.00	1120.00	0.02	0.06	-0.01	0.10	0.01	-0.03
12	-400.00	1120.00	0.11	0.25	-0.08	0.42	0.04	-0.13
13	-400.00	1000.00	0.05	0.10	-0.04	0.16	0.02	-0.06
14	-100.00	890.00	-0.26	-0.48	0.13	-0.69	-0.05	0.18
15	-100.00	450.00	0.04	0.06	-0.01	0.06	0.00	-0.01
16	-300.00	250.00	0.11	0.08	-0.04	0.04	0.01	-0.02
17	-300.00	20.00	0.07	0.02	-0.04	0.00	0.02	-0.01
18	-280.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00
19	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
20								
21								
Tổng cộng			0.5166	0.3144	0.0000	0.2761	0.0142	0.0000

Cáp	x	y			
1	0.00	254.18	Diện tích 1 bó cáp	As =	888.30 (mm2)
2	0.00	126.97	Số bó cáp	n=	4.00 bó
3	150.00	112.54		Σ As Yi =	525696.52 (mm3)
4	-150.00	98.11		Σ As =	3553.20 (mm2)

5		
6		
7		

K. cách từ trọng tâm bó cáp đến TTH = 376.78 (mm)

Cross setion at L/4

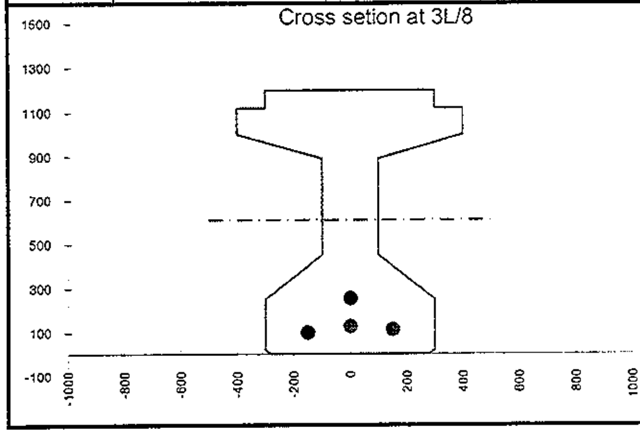


A	0.5166	m^2
e_x	0.0000	m
e_y	0.6086	m
I_x	0.0848	m^4
I_y	0.0142	m^4
I_{xy}	0.0000	m^4
y_b	0.6086	m
y_t	0.5914	m
S_b	0.1393	m^3
S_t	0.1433	m^3
$E_{cáp\,(Y)}$	0.3768	m
q =	12.3984	(KN/m)

5		
6		
7		

K. cách từ trọng tâm bó cáp đến TTH = 460.63 (mm)

Cross setion at 3L/8



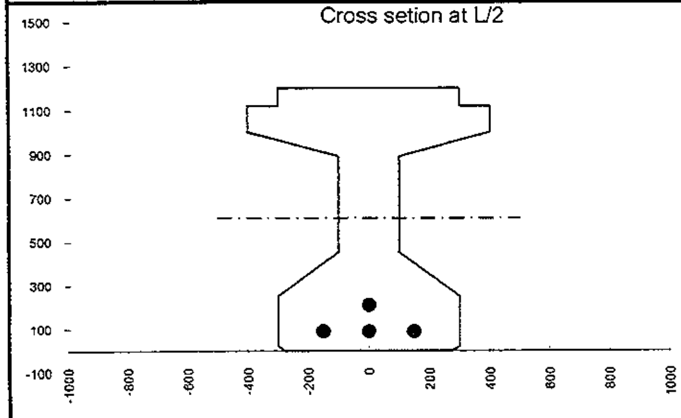
A	0.5166	m^2
e_x	0.0000	m
e_y	0.6086	m
I_x	0.0848	m^4
I_y	0.0142	m^4
I_{xy}	0.0000	m^4
y_b	0.6086	m
y_t	0.5914	m
S_b	0.1393	m^3
S_t	0.1433	m^3
$E_{cáp\,(Y)}$	0.4606	m
q =	12.3984	(KN/m)

ĐẶC TR NG HÌNH HỌC - MẶT CÁT L/2

Âiãøm goïc	x (mm)	y (mm)	A (m^2)	Q_X' (m^3)	Q_Y' (m^3)	I_X' (m^4)	I_Y' (m^4)	$I_{X'Y'}$ (m^4)
Đ ờng bao								
-	0.00	0.00	-	-	-	-	-	-
1	280.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	300.00	20.00	0.01	0.00	0.00	0.00	0.00	0.00
3	300.00	250.00	0.07	0.02	0.04	0.00	0.02	0.01
4	100.00	450.00	0.11	0.08	0.04	0.04	0.01	0.02
5	100.00	890.00	0.04	0.06	0.01	0.06	0.00	0.01
6	400.00	1000.00	-0.26	-0.48	-0.13	-0.69	-0.05	-0.18
7	400.00	1120.00	0.05	0.10	0.04	0.16	0.02	0.06
8	300.00	1120.00	0.11	0.25	0.08	0.42	0.04	0.13
9	300.00	1200.00	0.02	0.06	0.01	0.10	0.01	0.03
10	-300.00	1200.00	0.72	1.73	0.00	3.11	0.06	0.00
11	-300.00	1120.00	0.02	0.06	-0.01	0.10	0.01	-0.03
12	-400.00	1120.00	0.11	0.25	-0.08	0.42	0.04	-0.13
13	-400.00	1000.00	0.05	0.10	-0.04	0.16	0.02	-0.06
14	-100.00	890.00	-0.26	-0.48	0.13	-0.69	-0.05	0.18
15	-100.00	450.00	0.04	0.06	-0.01	0.06	0.00	-0.01
16	-300.00	250.00	0.11	0.08	-0.04	0.04	0.01	-0.02
17	-300.00	20.00	0.07	0.02	-0.04	0.00	0.02	-0.01
18	-280.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00
19	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
20								
21								
22								
23								
Tổng cộng			0.5166	0.3144	0.0000	0.2761	0.0142	0.0000
Cáp								
1	0.00	210.00	Diện tích 1 bó cáp			As =	888.30 (mm2)	

2	0.00	90.00
3	150.00	90.00
4	-150.00	90.00
5		
6		
7		

Số bó cáp n=	4.00	bó
Σ As Yi =	426384.00	(mm3)
Σ As =	3553.20	(mm2)
K. cách từ trọng tâm bó cáp đến TTH =	488.58	(mm)



A	0.5166	m^2
e_x	0.0000	m
e_y	0.6086	m
I_x	0.0848	m^4
I_y	0.0142	m^4
I_{xy}	0.0000	m^4
y_b	0.6086	m
y_t	0.5914	m
S_b	0.1393	m^3
S_t	0.1433	m^3
$E_{cap\,(Y)}$	0.4886	m
q =	12.3984	(KN/m)

Uniform load due to sefl weigh of Girder in Stage 1: **Q = 14.74 (KN/m)**

3.3. Property of Girder cross section in service stage (stage II: Composite cross section) :

3.3.1. Effective flange width (4.6.2.6)

Modular Ratio: Deck Concrete/Girder Concrete n = Eb / Ed = 0.88

For Interior Girder:

$$b_I = \min\begin{cases}1/4L_{tt}\\ 12h_f + \max(0.5b, b_w)\\ S\end{cases} \quad \Rightarrow n^* b_I = 2248.88861 \text{ (mm)}$$

For Exterior Girder:

$$b_E = 0.5b_I + \min\begin{cases}1/8L_{tt}\\ 6h_f + \max(0.5b_w, 0.25b)\\ L_h\end{cases} \quad \Rightarrow n^* b_E = 2248.88861 \text{ (mm)}$$

3.3.2. Property of Girder cross section in stage II (service stage):

Interior Girder:

Âiãøm goïc	x (mm)	y (mm)	A (m^2)	Q_X' (m^3)	Q_Y' (m^3)	I_X' (m^4)	I_Y' (m^4)	$I_X'Y'$ (m^4)
Đ ờng bao								
-	0.00	0.00	-	-	-	-	-	-
1	280.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	300.00	20.00	0.01	0.00	0.00	0.00	0.00	0.00
3	300.00	965.00	0.28	0.28	0.17	0.27	0.08	0.13
4	400.00	1000.00	-0.09	-0.17	-0.06	-0.25	-0.03	-0.09
5	400.00	1120.00	0.05	0.10	0.04	0.16	0.02	0.06
6	300.00	1120.00	0.11	0.25	0.08	0.42	0.04	0.13
7	300.00	1200.00	0.02	0.06	0.01	0.10	0.01	0.03
8	-300.00	1200.00	0.72	1.73	0.00	3.11	0.06	0.00
9	-300.00	1120.00	0.02	0.06	-0.01	0.10	0.01	-0.03
10	-400.00	1120.00	0.11	0.25	-0.08	0.42	0.04	-0.13
11	-400.00	1000.00	0.05	0.10	-0.04	0.16	0.02	-0.06
12	-300.00	965.00	-0.09	-0.17	0.06	-0.25	-0.03	0.09
13	-300.00	20.00	0.28	0.28	-0.17	0.27	0.08	-0.13
14	-280.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00
15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
16								
17								
18								
19								
20								
21								
22								
23								
1	0.00	1200.00	0.00	0.00	0.00	0.00	0.00	0.00
2	1124.44	1200.00	-1.35	-3.24	-1.52	-5.83	-1.71	-2.73
3	1124.44	1400.00	0.22	0.58	0.51	1.14	0.85	0.99
4	-1124.44	1400.00	3.15	8.82	0.00	18.51	3.98	0.00

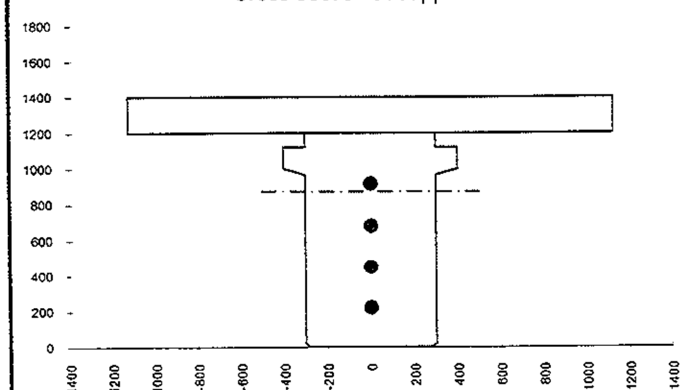
Exterior Girder:

Âiãøm goïc	x (mm)	y (mm)	A (m^2)	Q_X' (m^3)	Q_Y' (m^3)	I_X' (m^4)	I_Y' (m^4)	$I_X'Y'$ (m^4)
Đ ờng bao								
-	0.00	0.00	-	-	-	-	-	-
1	280.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	300.00	20.00	0.01	0.00	0.00	0.00	0.00	0.00
3	300.00	965.00	0.28	0.28	0.17	0.27	0.08	0.13
4	400.00	1000.00	-0.09	-0.17	-0.06	-0.25	-0.03	-0.09
5	400.00	1120.00	0.05	0.10	0.04	0.16	0.02	0.06
6	300.00	1120.00	0.11	0.25	0.08	0.42	0.04	0.13
7	300.00	1200.00	0.02	0.06	0.01	0.10	0.01	0.03
8	-300.00	1200.00	0.72	1.73	0.00	3.11	0.06	0.00
9	-300.00	1120.00	0.02	0.06	-0.01	0.10	0.01	-0.03
10	-400.00	1120.00	0.11	0.25	-0.08	0.42	0.04	-0.13
11	-400.00	1000.00	0.05	0.10	-0.04	0.16	0.02	-0.06
12	-300.00	965.00	-0.09	-0.17	0.06	-0.25	-0.03	0.09
13	-300.00	20.00	0.28	0.28	-0.17	0.27	0.08	-0.13
14	-280.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00
15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
16								
17								
18								
19								
20								
21								
22								
23								
1	0.00	1200.00	0.00	0.00	0.00	0.00	0.00	0.00
2	1124.44	1200.00	-1.35	-3.24	-1.52	-5.83	-1.71	-2.73
3	1124.44	1400.00	0.22	0.58	0.51	1.14	0.85	0.99
4	-1124.44	1400.00	3.15	8.82	0.00	18.51	3.98	0.00

5	-1124.44	1200.00	0.22	0.58	-0.51	1.14	0.85	-0.99
6	0.00	1200.00	-1.35	-3.24	1.52	-5.83	-1.71	2.73
7			0.00	0.00	0.00	0.00	0.00	0.00
8			0.00	0.00	0.00	0.00	0.00	0.00
Tổng cộng			1.1969	1.0456	0.0000	1.1376	0.2145	0.0000

Cáp					
1	0.00	916.87	Diện tích 1 bó cáp	As =	888.30 (mm2)
2	0.00	681.46	Số bó cáp	n=	4.00 bó
3	0.00	450.65		Σ As Yi =	2015384.34 (mm3)
4	0.00	219.83		Σ As =	3553.20 (mm2)
5			K. cách từ trọng tâm bó cáp đến TTH =		306.41 (mm)
6					
7					

Cross section at support



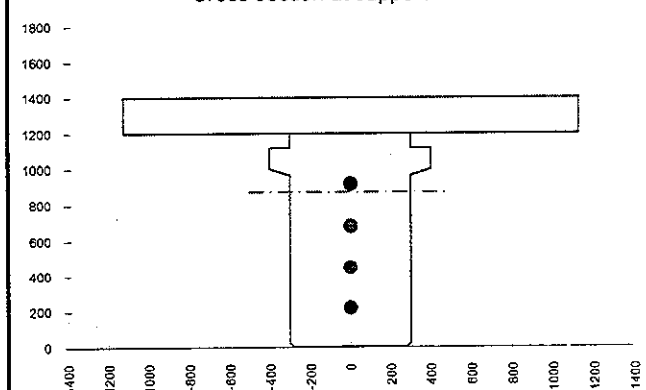
A	1.1969	m^2
e_x	0.0000	m
e_y	0.8736	m
I_x	0.2242	m^4
I_y	0.2145	m^4
I_{xy}	0.0000	m^4
y_b	0.8736	m
y_t	0.5264	m
S_b	0.2566	m^3
S_t	0.4259	m^3
$E_{cáp (Y)}$	0.3064	m
q =	28.7251	(KN/m)

Âiãøm goïc	x (mm)	y (mm)	A (m^2)	Q_X' (m^3)	Q_Y' (m^3)	I_X' (m^4)	I_Y' (m^4)	$I_X'Y'$ (m^4)
Đ òng bao								
-	0.00	0.00	-	-	-	-	-	-
1	280.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	300.00	20.00	0.01	0.00	0.00	0.00	0.00	0.00
3	300.00	250.00	0.07	0.02	0.04	0.00	0.02	0.01
4	261.00	289.00	0.02	0.01	0.01	0.00	0.01	0.00
5	261.00	950.00	0.17	0.21	0.09	0.22	0.04	0.08
6	400.00	1000.00	-0.12	-0.23	-0.08	-0.34	-0.04	-0.12
7	400.00	1120.00	0.05	0.10	0.04	0.16	0.02	0.06
8	300.00	1120.00	0.11	0.25	0.08	0.42	0.04	0.13
9	300.00	1200.00	0.02	0.06	0.01	0.10	0.01	0.03
10	-300.00	1200.00	0.72	1.73	0.00	3.11	0.06	0.00
11	-300.00	1120.00	0.02	0.06	-0.01	0.10	0.01	-0.03
12	-400.00	1120.00	0.11	0.25	-0.08	0.42	0.04	-0.13
13	-400.00	1000.00	0.05	0.10	-0.04	0.16	0.02	-0.06
14	-261.00	950.00	-0.12	-0.23	0.08	-0.34	-0.04	0.12
15	-261.00	289.00	0.17	0.21	-0.09	0.22	0.04	-0.08
16	-300.00	250.00	0.02	0.01	-0.01	0.00	0.01	0.00
17	-300.00	20.00	0.07	0.02	-0.04	0.00	0.02	-0.01
18	-280.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00

5	-1124.44	1200.00	0.22	0.58	-0.51	1.14	0.85	-0.99
6	0.00	1200.00	-1.35	-3.24	1.52	-5.83	-1.71	2.73
7			0.00	0.00	0.00	0.00	0.00	0.00
8			0.00	0.00	0.00	0.00	0.00	0.00
Tổng cộng			1.1969	1.0456	0.0000	1.1376	0.2145	0.0000

Cáp					
1	0.00	916.87	Diện tích 1 bó cáp	As =	888.30 (mm2)
2	0.00	681.46	Số bó cáp	n=	4.00 bó
3	0.00	450.65		Σ As Yi =	2015384.34 (mm3)
4	0.00	219.83		Σ As =	3553.20 (mm2)
5			K. cách từ trọng tâm bó cáp đến TTH =		306.41 (mm)
6					
7					

Cross section at support



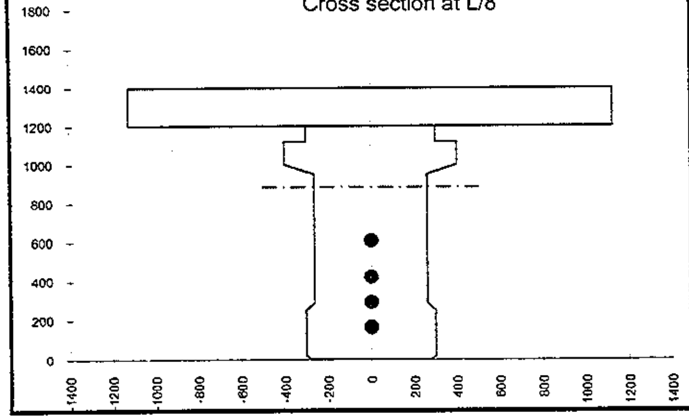
A	1.1969	m^2
e_x	0.0000	m
e_y	0.8736	m
I_x	0.2242	m^4
I_y	0.2145	m^4
I_{xy}	0.0000	m^4
y_b	0.8736	m
y_t	0.5264	m
S_b	0.2566	m^3
S_t	0.4259	m^3
$E_{cáp (Y)}$	0.3064	m
q =	29.3235	(KN/m)

Âiãøm goïc	x (mm)	y (mm)	A (m^2)	Q_X' (m^3)	Q_Y' (m^3)	I_X' (m^4)	I_Y' (m^4)	$I_X'Y'$ (m^4)
Đ òng bao								
-	0.00	0.00	-	-	-	-	-	-
1	280.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	300.00	20.00	0.01	0.00	0.00	0.00	0.00	0.00
3	300.00	250.00	0.07	0.02	0.04	0.00	0.02	0.01
4	261.00	289.00	0.02	0.01	0.01	0.00	0.01	0.00
5	261.00	950.00	0.17	0.21	0.09	0.22	0.04	0.08
6	400.00	1000.00	-0.12	-0.23	-0.08	-0.34	-0.04	-0.12
7	400.00	1120.00	0.05	0.10	0.04	0.16	0.02	0.06
8	300.00	1120.00	0.11	0.25	0.08	0.42	0.04	0.13
9	300.00	1200.00	0.02	0.06	0.01	0.10	0.01	0.03
10	-300.00	1200.00	0.72	1.73	0.00	3.11	0.06	0.00
11	-300.00	1120.00	0.02	0.06	-0.01	0.10	0.01	-0.03
12	-400.00	1120.00	0.11	0.25	-0.08	0.42	0.04	-0.13
13	-400.00	1000.00	0.05	0.10	-0.04	0.16	0.02	-0.06
14	-261.00	950.00	-0.12	-0.23	0.08	-0.34	-0.04	0.12
15	-261.00	289.00	0.17	0.21	-0.09	0.22	0.04	-0.08
16	-300.00	250.00	0.02	0.01	-0.01	0.00	0.01	0.00
17	-300.00	20.00	0.07	0.02	-0.04	0.00	0.02	-0.01
18	-280.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00

19	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
20								
21								
22								
23								
24								
25								
26								
1	0.00	1200.00	0.00	0.00	0.00	0.00	0.00	0.00
2	1124.44	1200.00	-1.35	-3.24	-1.52	-5.83	-1.71	-2.73
3	1124.44	1400.00	0.22	0.58	0.51	1.14	0.85	0.99
4	-1124.44	1400.00	3.15	8.82	0.00	18.51	3.98	0.00
5	-1124.44	1200.00	0.22	0.58	-0.51	1.14	0.85	-0.99
6	0.00	1200.00	-1.35	-3.24	1.52	-5.83	-1.71	2.73
7								
8								
Tổng cộng			1.1433	1.0128	0.0000	1.1155	0.2103	0.0000
Cáp								
1	0.00	607.61						
2	0.00	422.70						
3	0.00	292.86						
4	0.00	163.03						
5								
6								
7								

Diện tích 1 bó cáp	As =	888.30	(mm2)
Số bó cáp	n=	4.00	bó
	Σ As Yi =	1320196.69	(mm3)
	Σ As =	3553.20	(mm2)
K. cách từ trọng tâm bó cáp đến TTH =		514.29	(mm)

Cross section at L/8

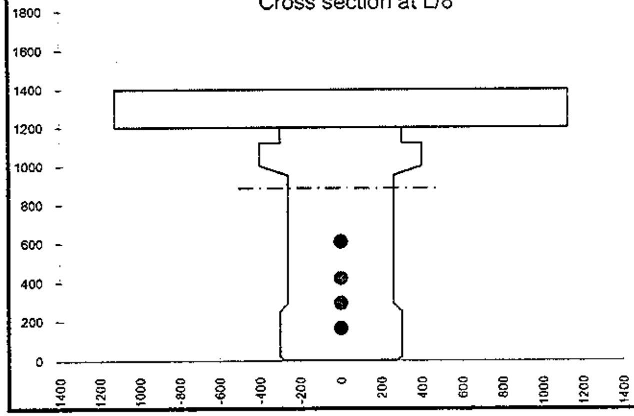


A	1.1433	m^2
e_x	0.0000	m
e_y	0.8858	m
I_x	0.2183	m^4
I_y	0.2103	m^4
I_{xy}	0.0000	m^4
y_b	0.8858	m
y_t	0.5142	m
S_b	0.2464	m^3
S_t	0.4245	m^3
$E_{cáp\ (Y)}$	0.5143	m
q =	27.4404	(KN/m)

19	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
20								
21								
22								
23								
24								
25								
26								
1	0.00	1200.00	0.00	0.00	0.00	0.00	0.00	0.00
2	1124.44	1200.00	-1.35	-3.24	-1.52	-5.83	-1.71	-2.73
3	1124.44	1400.00	0.22	0.58	0.51	1.14	0.85	0.99
4	-1124.44	1400.00	3.15	8.82	0.00	18.51	3.98	0.00
5	-1124.44	1200.00	0.22	0.58	-0.51	1.14	0.85	-0.99
6	0.00	1200.00	-1.35	-3.24	1.52	-5.83	-1.71	2.73
7								
8								
Tổng cộng			1.1433	1.0128	0.0000	1.1155	0.2103	0.0000
Cáp								
1	0.00	607.61						
2	0.00	422.70						
3	0.00	292.86						
4	0.00	163.03						
5								
6								
7								

Diện tích 1 bó cáp	As =	888.30	(mm2)
Số bó cáp	n=	4.00	bó
	Σ As Yi =	1320196.69	(mm3)
	Σ As =	3553.20	(mm2)
K. cách từ trọng tâm bó cáp đến TTH =		514.29	(mm)

Cross section at L/8



A	1.1433	m^2
e_x	0.0000	m
e_y	0.8858	m
I_x	0.2183	m^4
I_y	0.2103	m^4
I_{xy}	0.0000	m^4
y_b	0.8858	m
y_t	0.5142	m
S_b	0.2464	m^3
S_t	0.4245	m^3
$E_{cáp\ (Y)}$	0.5143	m
q =	28.0120	(KN/m)

Interior Girder:

ĐẶC TR   NG HÌNH HỌC - MẶT CẮT L/4

Âiãøm goïc	x (mm)	y (mm)	A (m^2)	Q_X' (m^3)	Q_Y' (m^3)	I_X' (m^4)	I_Y' (m^4)	$I_X'Y'$ (m^4)
Đ ờng bao								
-	0.00	0.00	-	-	-	-	-	-
1	280.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	300.00	20.00	0.01	0.00	0.00	0.00	0.00	0.00
3	300.00	250.00	0.07	0.02	0.04	0.00	0.02	0.01
4	100.00	450.00	0.11	0.08	0.04	0.04	0.01	0.02
5	100.00	890.00	0.04	0.06	0.01	0.06	0.00	0.01
6	400.00	1000.00	-0.26	-0.48	-0.13	-0.69	-0.05	-0.18
7	400.00	1120.00	0.05	0.10	0.04	0.16	0.02	0.06
8	300.00	1120.00	0.11	0.25	0.08	0.42	0.04	0.13
9	300.00	1200.00	0.02	0.06	0.01	0.10	0.01	0.03
10	-300.00	1200.00	0.72	1.73	0.00	3.11	0.06	0.00
11	-300.00	1120.00	0.02	0.06	-0.01	0.10	0.01	-0.03
12	-400.00	1120.00	0.11	0.25	-0.08	0.42	0.04	-0.13
13	-400.00	1000.00	0.05	0.10	-0.04	0.16	0.02	-0.06
14	-100.00	890.00	-0.26	-0.48	0.13	-0.69	-0.05	0.18
15	-100.00	450.00	0.04	0.06	-0.01	0.06	0.00	-0.01
16	-300.00	250.00	0.11	0.08	-0.04	0.04	0.01	-0.02
17	-300.00	20.00	0.07	0.02	-0.04	0.00	0.02	-0.01
18	-280.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00
19	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
20								
21								
22								
23								
24								
25								
26								
27								
28								
29								
30								
31								
32								
1	0.00	1200.00	0.00	0.00	0.00	0.00	0.00	0.00
2	1124.44	1200.00	-1.35	-3.24	-1.52	-5.83	-1.71	-2.73
3	1124.44	1400.00	0.22	0.58	0.51	1.14	0.85	0.99
4	-1124.44	1400.00	3.15	8.82	0.00	18.51	3.98	0.00
5	-1124.44	1200.00	0.22	0.58	-0.51	1.14	0.85	-0.99
6	0.00	1200.00	-1.35	-3.24	1.52	-5.83	-1.71	2.73
7								
8								
Tổng cộng			0.9664	0.8991	0.0000	1.0377	0.2038	0.0000
Cáp								
1	0.00	386.72	Diện tích 1 bó cáp			As =	888.30 (mm2)	

Exterior Girder:

ĐẶC TR   NG HÌNH HỌC - MẶT CẮT L/4

Âiãøm goïc	x (mm)	y (mm)	A (m^2)	Q_X' (m^3)	Q_Y' (m^3)	I_X' (m^4)	I_Y' (m^4)	$I_X'Y'$ (m^4)
Đ ờng bao								
-	0.00	0.00	-	-	-	-	-	-
1	280.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	300.00	20.00	0.01	0.00	0.00	0.00	0.00	0.00
3	300.00	250.00	0.07	0.02	0.04	0.00	0.02	0.01
4	100.00	450.00	0.11	0.08	0.04	0.04	0.01	0.02
5	100.00	890.00	0.04	0.06	0.01	0.06	0.00	0.01
6	400.00	1000.00	-0.26	-0.48	-0.13	-0.69	-0.05	-0.18
7	400.00	1120.00	0.05	0.10	0.04	0.16	0.02	0.06
8	300.00	1120.00	0.11	0.25	0.08	0.42	0.04	0.13
9	300.00	1200.00	0.02	0.06	0.01	0.10	0.01	0.03
10	-300.00	1200.00	0.72	1.73	0.00	3.11	0.06	0.00
11	-300.00	1120.00	0.02	0.06	-0.01	0.10	0.01	-0.03
12	-400.00	1120.00	0.11	0.25	-0.08	0.42	0.04	-0.13
13	-400.00	1000.00	0.05	0.10	-0.04	0.16	0.02	-0.06
14	-100.00	890.00	-0.26	-0.48	0.13	-0.69	-0.05	0.18
15	-100.00	450.00	0.04	0.06	-0.01	0.06	0.00	-0.01
16	-300.00	250.00	0.11	0.08	-0.04	0.04	0.01	-0.02
17	-300.00	20.00	0.07	0.02	-0.04	0.00	0.02	-0.01
18	-280.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00
19	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
20								
21								
22								
23								
24								
25								
26								
27								
28								
29								
30								
31								
32								
1	0.00	1200.00	0.00	0.00	0.00	0.00	0.00	0.00
2	1124.44	1200.00	-1.35	-3.24	-1.52	-5.83	-1.71	-2.73
3	1124.44	1400.00	0.22	0.58	0.51	1.14	0.85	0.99
4	-1124.44	1400.00	3.15	8.82	0.00	18.51	3.98	0.00
5	-1124.44	1200.00	0.22	0.58	-0.51	1.14	0.85	-0.99
6	0.00	1200.00	-1.35	-3.24	1.52	-5.83	-1.71	2.73
7								
8								
Tổng cộng			0.9664	0.8991	0.0000	1.0377	0.2038	0.0000
Cáp								
1	0.00	386.72	Diện tích 1 bó cáp			As =	888.30 (mm2)	

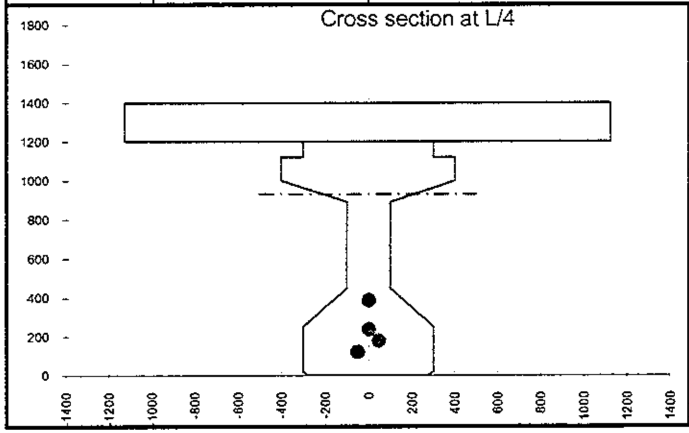
2	0.00	237.87
3	47.00	180.16
4	-50.00	122.46
5		
6		
7		

Số bó cáp n= 4.00 bó

Σ As Yi = 823634.09 (mm3)

Σ As = 3553.20 (mm2)

K. cách từ trọng tâm bó cáp đến TTH = 698.59 (mm)



A	0.9664	m^2
e_x	0.0000	m
e_y	0.9304	m
I_x	0.2012	m^4
I_y	0.2038	m^4
I_{xy}	0.0000	m^4
y_b	0.9304	m
y_t	0.4696	m
S_b	0.2163	m^3
S_t	0.4284	m^3
$E_{cáp\,(Y)}$	0.6986	m
q =	23.1931	(KN/m)

ĐẶC TR NG HÌNH HỌC - MẶT CẮT 3L/8

Âiãøm goïc	x (mm)	y (mm)	A (m^2)	Q_X' (m^3)	Q_Y' (m^3)	I_X' (m^4)	I_Y' (m^4)	$I_X'Y'$ (m^4)
Đ òng bao								
-	0.00	0.00	-	-	-	-	-	-
1	280.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	300.00	20.00	0.01	0.00	0.00	0.00	0.00	0.00
3	300.00	250.00	0.07	0.02	0.04	0.00	0.02	0.01
4	100.00	450.00	0.11	0.08	0.04	0.04	0.01	0.02
5	100.00	890.00	0.04	0.06	0.01	0.06	0.00	0.01
6	400.00	1000.00	-0.26	-0.48	-0.13	-0.69	-0.05	-0.18
7	400.00	1120.00	0.05	0.10	0.04	0.16	0.02	0.06
8	300.00	1120.00	0.11	0.25	0.08	0.42	0.04	0.13
9	300.00	1200.00	0.02	0.06	0.01	0.10	0.01	0.03
10	-300.00	1200.00	0.72	1.73	0.00	3.11	0.06	0.00
11	-300.00	1120.00	0.02	0.06	-0.01	0.10	0.01	-0.03
12	-400.00	1120.00	0.11	0.25	-0.08	0.42	0.04	-0.13
13	-400.00	1000.00	0.05	0.10	-0.04	0.16	0.02	-0.06
14	-100.00	890.00	-0.26	-0.48	0.13	-0.69	-0.05	0.18
15	-100.00	450.00	0.04	0.06	-0.01	0.06	0.00	-0.01
16	-300.00	250.00	0.11	0.08	-0.04	0.04	0.01	-0.02
17	-300.00	20.00	0.07	0.02	-0.04	0.00	0.02	-0.01
18	-280.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00
19	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
20								
21								
22								
23								

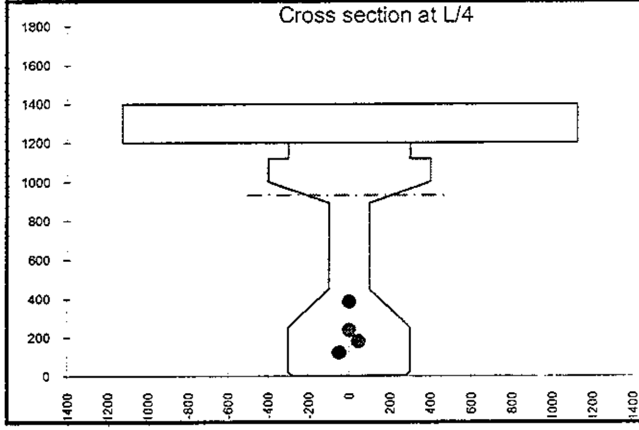
2	0.00	237.87
3	47.00	180.16
4	-50.00	122.46
5		
6		
7		

Số bó cáp n= 4.00 bó

Σ As Yi = 823634.09 (mm3)

Σ As = 3553.20 (mm2)

K. cách từ trọng tâm bó cáp đến TTH = 698.59 (mm)



A	0.9664	m^2
e_x	0.0000	m
e_y	0.9304	m
I_x	0.2012	m^4
I_y	0.2038	m^4
I_{xy}	0.0000	m^4
y_b	0.9304	m
y_t	0.4696	m
S_b	0.2163	m^3
S_t	0.4284	m^3
$E_{cáp\,(Y)}$	0.6986	m
q =	23.6763	(KN/m)

ĐẶC TR NG HÌNH HỌC - MẶT CẮT 3L/8

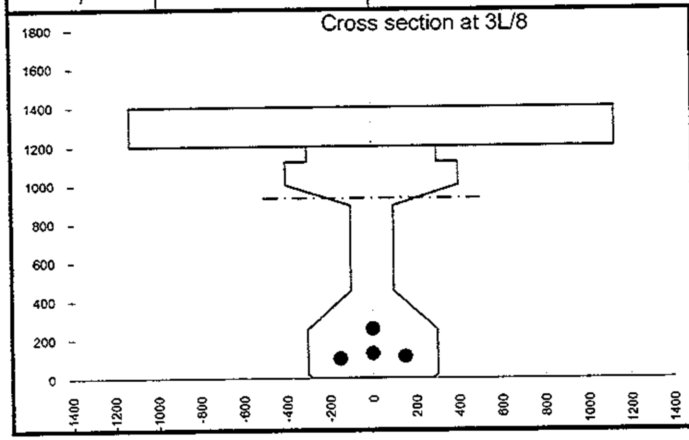
Âiãøm goïc	x (mm)	y (mm)	A (m^2)	Q_X' (m^3)	Q_Y' (m^3)	I_X' (m^4)	I_Y' (m^4)	$I_X'Y'$ (m^4)
Đ òng bao								
-	0.00	0.00	-	-	-	-	-	-
1	280.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	300.00	20.00	0.01	0.00	0.00	0.00	0.00	0.00
3	300.00	250.00	0.07	0.02	0.04	0.00	0.02	0.01
4	100.00	450.00	0.11	0.08	0.04	0.04	0.01	0.02
5	100.00	890.00	0.04	0.06	0.01	0.06	0.00	0.01
6	400.00	1000.00	-0.26	-0.48	-0.13	-0.69	-0.05	-0.18
7	400.00	1120.00	0.05	0.10	0.04	0.16	0.02	0.06
8	300.00	1120.00	0.11	0.25	0.08	0.42	0.04	0.13
9	300.00	1200.00	0.02	0.06	0.01	0.10	0.01	0.03
10	-300.00	1200.00	0.72	1.73	0.00	3.11	0.06	0.00
11	-300.00	1120.00	0.02	0.06	-0.01	0.10	0.01	-0.03
12	-400.00	1120.00	0.11	0.25	-0.08	0.42	0.04	-0.13
13	-400.00	1000.00	0.05	0.10	-0.04	0.16	0.02	-0.06
14	-100.00	890.00	-0.26	-0.48	0.13	-0.69	-0.05	0.18
15	-100.00	450.00	0.04	0.06	-0.01	0.06	0.00	-0.01
16	-300.00	250.00	0.11	0.08	-0.04	0.04	0.01	-0.02
17	-300.00	20.00	0.07	0.02	-0.04	0.00	0.02	-0.01
18	-280.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00
19	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
20								
21								
22								
23								

24									
25									
26									
27									
28									
29									
30									
31									
1	0.00	1200.00	0.00	0.00	0.00	0.00	0.00	0.00	
2	1124.44	1200.00	-1.35	-3.24	-1.52	-5.83	-1.71	-2.73	
3	1124.44	1400.00	0.22	0.58	0.51	1.14	0.85	0.99	
4	-1124.44	1400.00	3.15	8.82	0.00	18.51	3.98	0.00	
5	-1124.44	1200.00	0.22	0.58	-0.51	1.14	0.85	-0.99	
6	0.00	1200.00	-1.35	-3.24	1.52	-5.83	-1.71	2.73	
7			0.00	0.00	0.00	0.00	0.00	0.00	
8			0.00	0.00	0.00	0.00	0.00	0.00	
Tổng cộng			0.9664	0.8991	0.0000	1.0377	0.2038	0.0000	

Cáp		
1	0.00	254.18
2	0.00	126.97
3	150.00	112.54
4	-150.00	98.11
5		
6		
7		

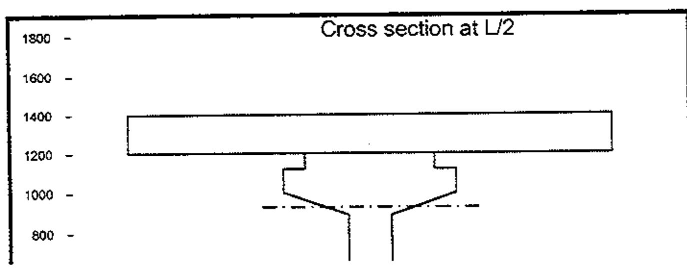
Diện tích 1 bó cáp	As =	888.30	(mm2)
Số bó cáp	n=	4.00	bó
	Σ As Yi =	525696.52	(mm3)
	Σ As =	3553.20	(mm2)
K. cách từ trọng tâm bó cáp đến TTH =		782.44	(mm)

Cross section at 3L/8



A	0.9664	m^2
e_x	0.0000	m
e_y	0.9304	m
I_x	0.2012	m^4
I_y	0.2038	m^4
I_{xy}	0.0000	m^4
y_b	0.9304	m
y_t	0.4696	m
S_b	0.2163	m^3
S_t	0.4284	m^3
$E_{cáp\,(Y)}$	0.7824	m
q =	23.1931	(KN/m)

Cross section at L/2



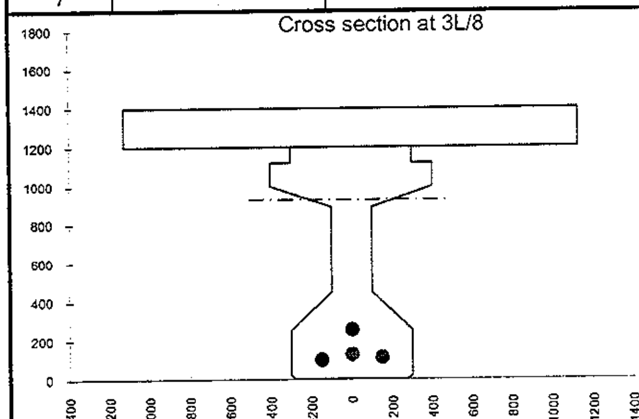
A	0.9664	m^2
e_x	0.0000	m
e_y	0.9304	m
I_x	0.2012	m^4
I_y	0.2038	m^4
I_{xy}	0.0000	m^4
y_b	0.9304	m

24								
25								
26								
27								
28								
29								
30								
31								
1	0.00	1200.00	0.00	0.00	0.00	0.00	0.00	0.00
2	1124.44	1200.00	-1.35	-3.24	-1.52	-5.83	-1.71	-2.73
3	1124.44	1400.00	0.22	0.58	0.51	1.14	0.85	0.99
4	-1124.44	1400.00	3.15	8.82	0.00	18.51	3.98	0.00
5	-1124.44	1200.00	0.22	0.58	-0.51	1.14	0.85	-0.99
6	0.00	1200.00	-1.35	-3.24	1.52	-5.83	-1.71	2.73
7			0.00	0.00	0.00	0.00	0.00	0.00
8			0.00	0.00	0.00	0.00	0.00	0.00
Tổng cộng			0.9664	0.8991	0.0000	1.0377	0.2038	0.0000

Cáp		
1	0.00	254.18
2	0.00	126.97
3	150.00	112.54
4	-150.00	98.11
5		
6		
7		

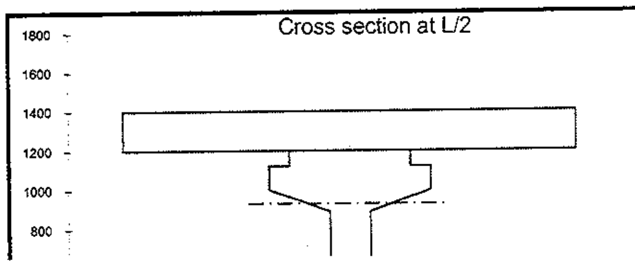
Diện tích 1 bó cáp	As =	888.30	(mm2)
Số bó cáp	n=	4.00	bó
	Σ As Yi =	525696.52	(mm3)
	Σ As =	3553.20	(mm2)
K. cách từ trọng tâm bó cáp đến TTH =		782.44	(mm)

Cross section at 3L/8

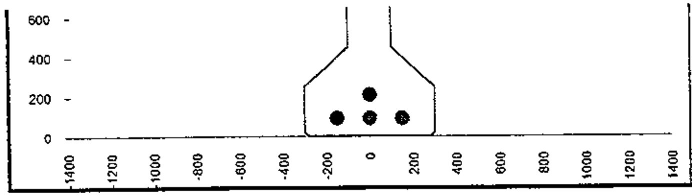


A	0.9664	m^2
e_x	0.0000	m
e_y	0.9304	m
I_x	0.2012	m^4
I_y	0.2038	m^4
I_{xy}	0.0000	m^4
y_b	0.9304	m
y_t	0.4696	m
S_b	0.2163	m^3
S_t	0.4284	m^3
$E_{cáp\,(Y)}$	0.7824	m
q =	23.6763	(KN/m)

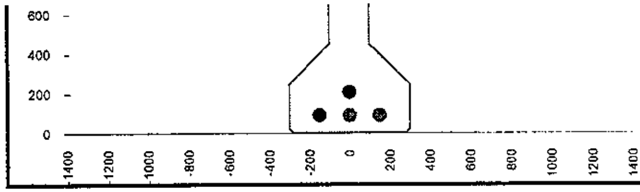
Cross section at L/2



A	0.9664	m^2
e_x	0.0000	m
e_y	0.9304	m
I_x	0.2012	m^4
I_y	0.2038	m^4
I_{xy}	0.0000	m^4
y_b	0.9304	m



y_t	0.4696	m
S_b	0.2163	m^3
S_t	0.4284	m^3
$E_{cáp\,(Y)}$	0.8104	m
q =	23.1931	(KN/m)



y_t	0.4696	m
S_b	0.2163	m^3
S_t	0.4284	m^3
$E_{cáp\,(Y)}$	0.8104	m
q =	0.0000	(KN/m)

4. LOSS OF PRESTRESS

4.1 Loss of prestressing force immediately (Instantaneous losses):

4.1.1 Friction between Prestressing Tendon and Duck:

Formula: $\Delta f_{pF} = f_{pj}\,(1-e^{-(Kx+\mu\alpha)})$ (5.9.5.2.2)

Xi: Length of tendon from the jacking end to any point under consideration

Section		Tendon no. 1		Tendon no. 2		Tendon no. 3		Tendon no. 4				SUM/4
	X_i (mm)	$\Sigma\alpha$ (rad)	Δf_{pF} (MPa)	$\Sigma\alpha$ (rad)	Δf_{pF} (MPa)	$\Sigma\alpha$ (rad)	Δf_{pF} (MPa)	$\Sigma\alpha$ (rad)	Δf_{pF} (MPa)			$\Sigma\Delta f_{pF}$ (MPa)
Ancho.	0.00	0.0000	**0.00**	0.0000	**0.00**	0.0000	**0.00**	0.0000	**0.00**			**0.00**
Support	200.00	0.0035	**1.32**	0.0026	**1.03**	0.0017	**0.74**	0.0008	**0.45**			**0.93**
L/8	2737.50	0.0518	**19.08**	0.0387	**14.85**	0.0255	**10.61**	0.0123	**6.35**			**13.31**
L/4	5275.00	0.1449	**50.68**	0.1081	**39.12**	0.0713	**27.45**	0.0345	**15.67**			**34.36**
3L/8	7812.50	0.2827	**95.09**	0.2109	**73.23**	0.1391	**50.97**	0.0673	**28.31**			**63.58**
L/2	10350.00	0.4653	**150.88**	0.3472	**116.36**	0.2290	**80.81**	0.1108	**44.19**			**100.28**

4.1.2 Anchorage seating or Set:

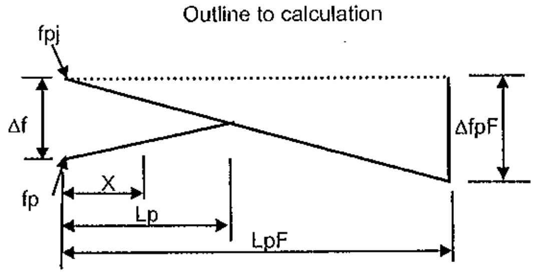
Formula

$$L_{pA} = \sqrt{\frac{E(\Delta L)L_{pF}}{\Delta f_{pF}}}$$

$$\Delta f = \frac{2\Delta f_{pF} L_{pA}}{L_{pF}}$$

$$\Delta f_{pA} = \Delta f\left(1 - \frac{X}{L_{pA}}\right)$$

Outline to calculation



Trong đó:

L_{pA}	Effective length due to anchorage set
E	Cable modulus of elasticity
ΔL	Setting length
L_{pF}	The length from anchorage to point that loss stress due to friction was known
Δf_{pF}	The loss stress value at the point that the leng from anchorage tl it is L_{pF}
Δf	The loss stress value at Anchorage

Choice the length from anchorage to point that loss stress due to friction was known (LpF) and calculation follow:

Tendon no.1		Xi (mm)	Δf_{pA} (MPa)
LpF =	10350	0	**262.53**
Δf_{pF} =	150.88	200	**256.70**
LpA =	9004.5	2738	**182.72**
Δf =	262.53	5275	**108.74**
		7813	**34.76**
		10350	**0.00**

Tendon no.2		Xi (mm)	Δf_{pA} (MPa)
LpF =	10350	0	**230.56**
Δf_{pF} =	116.36	200	**226.06**
LpA =	10253.5	2738	**169.00**
Δf =	230.56	5275	**111.94**
		7813	**54.89**
		10350	**0.00**

Tendon no.3		Xi (mm)	Δf_{pA} (MPa)
LpF =	10350	0	**161.62**
Δf_{pF} =	80.81	200	**158.50**
LpA =	10350.0	2738	**118.87**
Δf =	161.62	5275	**79.25**
		7813	**39.62**
		10350	**0.00**

Tendon no.4		Xi (mm)	Δf_{pA} (MPa)
LpF =	10350	0	**88.38**
Δf_{pF} =	44.19	200	**86.67**
LpA =	10350.0	2738	**65.01**
Δf =	88.38	5275	**43.34**
		7813	**21.67**
		10350	**0.00**

4.1.3 Elastic deformation of concrete:

Formula

$$\Delta f_{ES} = \frac{N-1}{2N}\frac{E_p}{E_{ci}} f_{cgp} \quad (5.9.5.2.3b\text{-}1)$$

In which:

Number of tendon	N =	4.00	(Tendon)
Cable modulus of elasticity	Ep =	197000.0	MPa
Concrete strength at transfer	f'_{ci} =	40.50	MPa
Unit weight of concrete	γ_c =	2450.00	kg/m3
Concrete modulus of elasticity at transfer	E_{ci} =	33185.3	MPa

Total stress of concrete in the Tendon centroid (f_{cgp}) due to prestressing force and self weigh of girder

$$f_{cgp} = \frac{F_j}{A} + \frac{F_j e^2}{I_X} - \frac{M_{DC} e}{I_X}$$

Compression force due to pretressing consider loss stress: $F_j = N^* f_{pj}{}^* A_s - A_s{}^*\Sigma(\Delta f_{pFi} + \Delta f_{pAi})$

A	Area of girder cross section
Ix	Inertia Moment of Girder cross section
e	Distance from tendon centroid to neutral line of girder section
M_{DC}	Maximum moment due to self weigh of girder at jacking

Total loss stress due to friction and Anchorage:

Section	Xi	Tendon1 ΔfpF+ΔfpA	Tendon2 ΔfpF+ΔfpA	Tendon3 ΔfpF+ΔfpA	Tendon4 ΔfpF+ΔfpA		SUM Σ(ΔfpF+ΔfpA)	ΣFj
	(mm)	(MPa)	(MPa)	(MPa)	(MPa)		(MPa)	(kN)
Anchorage	0	262.53	230.56	161.62	88.38		743.09	3966.18
Support	200	258.02	227.09	159.23	87.12		731.46	3976.51
L/8	2738	201.80	183.85	129.48	71.36		586.50	4105.28
L/4	5275	159.42	151.06	106.70	59.00		476.18	4203.27
3L/8	7813	129.85	128.12	90.60	49.98		398.54	4272.24
L/2	10350	150.88	116.36	80.81	44.19		392.24	4277.84

Loss stress due to Elastic deformation of concrete

Section	Xi (mm)	Fj (kN)	A (mm2)	Ix (mm4)	e (mm)	M_{DC} kNm	f_{cgp} (MPa)	Δf_{ES} (MPa)
Anchorage	0	3966.18	7.5E+05	9.2E+10	49.71	0.00	5.42	**12.06**
Support	200	3976.51	7.5E+05	9.2E+10	49.71	0.00	5.43	**12.09**
L/8	2738	4105.28	6.9E+05	9.0E+10	245.70	332.08	7.77	**17.31**
L/4	5275	4203.27	5.2E+05	8.5E+10	376.78	569.29	12.65	**28.15**
3L/8	7813	4272.24	5.2E+05	8.5E+10	460.63	711.61	15.10	**33.61**
L/2	10350	4277.84	5.2E+05	8.5E+10	488.58	759.05	15.95	**35.51**

Total loss of prestressing force immediately - Remaining prestressing force:

Tendon1 Section	Xi (mm)	Δf_{pF} (MPa)	Δf_{pA} (MPa)	Δf_{ES} (MPa)	ΣΔ (MPa)	F_j^1 (kN)	(α) (rad)	F_j^1*Cos(α) (kN)	F_j^1*Sin(α) (kN)
anchorage	0	0.00	262.53	12.06	274.59	912.65	0.1806	897.80	163.95
Support	200	1.32	256.70	12.09	270.11	916.63	0.1772	902.27	161.58
L/8	2738	19.08	182.72	17.31	219.11	961.93	0.1335	953.37	128.05
L/4	5275	50.68	108.74	28.15	187.57	989.95	0.0893	986.00	88.29
3L/8	7813	95.09	34.76	33.61	163.46	1011.37	0.0447	1010.36	45.23
L/2	10350	150.88	0.00	35.51	186.40	990.99	0.0000	990.99	0.00

Tendon2 Section	Xi (mm)	Δf_{pF} (MPa)	Δf_{pA} (MPa)	Δf_{ES} (MPa)	ΣΔ (MPa)	F_j^2 (kN)	(α) (rad)	F_j^2*Cos(α) (kN)	F_j^2*Sin(α) (kN)
anchorage	0	0.00	230.56	12.06	242.61	941.05	0.1354	932.44	127.03
Support	200	1.03	226.06	12.09	239.17	944.11	0.1328	935.79	125.02
L/8	2738	14.85	169.00	17.31	201.16	977.87	0.0999	973.00	97.49
L/4	5275	39.12	111.94	28.15	179.21	997.37	0.0667	995.15	66.48
3L/8	7813	73.23	54.89	33.61	161.73	1012.90	0.0334	1012.34	33.81
L/2	10350	116.36	0.00	35.51	151.88	1021.65	0.0000	1021.65	0.00

Tendon3 Section	Xi (mm)	Δf_{pF} (MPa)	Δf_{pA} (MPa)	Δf_{ES} (MPa)	ΣΔ (MPa)	F_j^3 (kN)	(α) (rad)	F_j^3*Cos(α) (kN)	F_j^3*Sin(α) (kN)
anchorage	0	0.00	161.62	10.72	172.34	1003.48	0.0896	999.45	89.81
Support	200	0.74	158.50	10.74	169.98	1005.58	0.0879	1001.69	88.27
L/8	2738	10.61	118.87	15.38	144.87	1027.88	0.0660	1025.64	67.78
L/4	5275	27.45	79.25	25.02	131.72	1039.56	0.0440	1038.55	45.76
3L/8	7813	50.97	39.62	29.88	120.47	1049.55	0.0220	1049.30	23.12
L/2	10350	80.81	0.00	31.57	112.38	1056.74	0.0000	1056.74	0.00

Tendon4 Section	Xi (mm)	Δf_{pF} (MPa)	Δf_{pA} (MPa)	Δf_{ES} (MPa)	ΣΔ (MPa)	F_j^4 (kN)	(α) (rad)	F_j^4*Cos(α) (kN)	F_j^4*Sin(α) (kN)
anchorage	0	0.00	88.38	8.04	96.42	1070.92	0.0435	1069.91	46.52
Support	200	0.45	86.67	8.06	95.18	1072.02	0.0426	1071.05	45.67
L/8	2738	6.35	65.01	11.54	82.90	1082.93	0.0320	1082.38	34.61
L/4	5275	15.67	43.34	18.77	77.77	1087.48	0.0213	1087.23	23.18
3L/8	7813	28.31	21.67	22.41	72.39	1092.26	0.0107	1092.20	11.64

L/2	10350	44.19	0.00	23.68	67.87	1096.28	0.0000	1096.28	0.00

SUM 1to4 Section	Xi (mm)	ΣF_j (kN)	Fj*Cos(α) (kN)	Fj*Sin(α) (kN)	$e_{cáp}$ (mm)	$M_i = \Sigma F_j Cos(\alpha)^* e_{cap}$ (kNm)
anchorage	0	3928.10	3899.60	427.30	49.71	193.85
Support	0	3938.33	3910.81	420.54	49.71	194.41
L/8	0	4050.62	4034.39	327.94	245.70	991.26
L/4	0	4114.36	4106.94	223.70	376.78	1547.42
3L/8	0	4166.09	4164.19	113.80	460.63	1918.16
L/2	0	4165.67	4165.67	0.00	488.58	2035.27

4.2. Loss of prestressing force at service stage (time - dependent losses):

4.2.1 Loss of prestress due to Shrinkage:

Formula: $\Delta fp_{SH} = (93 - 0.85^*H)$

Relative humidity of environment H = 80.00 %

Δfp_{SH} = **25.00 (MPa)**

4.2.2 Loss of prestress due to Creep:

Formula $\Delta fp_{CR} = 12.0\ f_{cgp} - 7.0 * \Delta f_{cdp}$

In which:

f_{cgp} Stress in concrete at tendons centroid (f_{cgp}) due to prestressing tendon and self weigh of girder

Δf_{cdp} Stess at tendons centroid changes due to permanent load, except dead load action at transfer

Section	Xi		Interior Girder		Exterior Girder	
		f_{cgp}	Δf_{cdp}	Δfp_{CR}	Δf_{cdp}	Δfp_{CR}
	(m)	(MPa)	(MPa)	**(MPa)**	(MPa)	**(MPa)**
Support	0.00	5.43	0.00	**65.16**	0.00	**65.16**
L/8	2.54	7.77	0.80	**87.70**	1.43	**83.28**
L/4	5.08	12.65	4.23	**122.11**	3.85	**124.79**
3L/8	7.61	15.10	4.59	**149.05**	5.73	**141.08**
L/2	10.15	15.95	7.10	**141.76**	6.43	**146.41**

4.2.3 Loss of prestress due to Relaxation:

(a) At transfer:

Formula: $$\Delta f_{pR1} = \frac{\log(24t)}{40}\left[\frac{f_{pj}}{f_{py}} - 0{,}55\right] f_{pj}$$

In which:

t : time estimated in days from stressing to transfer t = 0.00 days

fpy : Specified yeild strength of prestressing steel f_{py} = 1674.00 (MPa)

fpj : Initial stress in the tendon at the end of stressing

Section	Xi (m)	fpj (MPa)	Δfp_{R1} **(MPa)**
Support	0.00	1289.91	**0.00**
L/8	2.54	1284.69	**0.00**
L/4	5.08	1273.85	**0.00**
3L/8	7.61	1268.39	**0.00**
L/2	10.15	1266.49	**0.00**

(b) After Transfer:

Formula: $\Delta fp_{R2} = 30\%^*(138 - 0.3\ \Delta fp_F - 0.4\ \Delta f_{ES} - 0.2\ (\Delta fp_{SH} + \Delta fp_{CR}))$

Interior Girder						
Section	Xi (m)	Δf_{pF} (MPa)	Δf_{pES} (MPa)	Δfp_{SH} (MPa)	Δfp_{CR} (MPa)	Δfp_{R2} **(MPa)**
Support	0.00	0.93	12.09	25.00	16.29	**37.39**
L/8	2.54	13.31	17.31	25.00	21.92	**35.31**
L/4	5.08	34.36	28.15	25.00	30.53	**31.60**
3L/8	7.61	63.58	33.61	25.00	37.26	**27.91**
L/2	10.15	100.28	35.51	25.00	35.44	**24.49**

Exterior Girder						
Section	Xi (m)	Δf_{pF} (MPa)	Δf_{pES} (MPa)	Δfp_{SH} (MPa)	Δfp_{CR} (MPa)	**Δfp_{R2} (MPa)**
Support	0.00	0.93	12.09	25.00	16.29	**37.39**
L/8	2.54	13.31	17.31	25.00	20.82	**35.38**
L/4	5.08	34.36	28.15	25.00	31.20	**31.56**
3L/8	7.61	63.58	33.61	25.00	35.27	**28.03**
L/2	10.15	100.28	35.51	25.00	36.60	**24.42**

TOTAL LOSS STRESS AT SERVICE STAGE

Interior Girder						
Setion	Xi (m)	Δfp_{SH} (MPa)	Δfp_{CR} (MPa)	Δfp_{R1} (MPa)	Δfp_{R2} (MPa)	**Sum (MPa)**
Support	0.00	25.00	65.16	0.00	37.39	**127.55**
L/8	2.54	25.00	87.70	0.00	35.31	**148.01**
L/4	5.08	25.00	122.11	0.00	31.60	**178.71**
3L/8	7.61	25.00	149.05	0.00	27.91	**201.96**
L/2	10.15	25.00	141.76	0.00	24.49	**191.25**

Exterior Girder						
Section	Xi (m)	Δfp_{SH} (MPa)	Δfp_{CR} (MPa)	Δfp_{R1} (MPa)	Δfp_{R2} (MPa)	**Sum (MPa)**
Support	0.00	25.00	65.16	0.00	37.39	**127.55**
L/8	2.54	25.00	83.28	0.00	35.38	**143.66**
L/4	5.08	25.00	124.79	0.00	31.56	**181.35**
3L/8	7.61	25.00	141.08	0.00	28.03	**194.11**
L/2	10.15	25.00	146.41	0.00	24.42	**195.82**

4.3. Total Prestressing force consider loss in the service stage:
Interior Girder

Tendon1	Xi	$\Sigma\Delta_{pT}$	F_j^1	(α)	$Fj^1 \cdot Cos(\alpha)$	$Fj^1 \cdot Sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	397.66	803.33	0.1772	790.75	141.61
L/8	2.54	367.12	830.46	0.1335	823.06	110.55
L/4	5.08	366.28	831.20	0.0893	827.89	74.13
3L/8	7.61	365.41	831.97	0.0447	831.14	37.21
L/2	10.15	377.64	821.10	0.0000	821.10	0.00

Tendon2	Xi	$\Sigma\Delta_{pT}$	F_j^2	(α)	$Fj^2 \cdot Cos(\alpha)$	$Fj^2 \cdot Sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	366.72	830.81	0.1354	823.20	112.15
L/8	2.54	349.17	846.40	0.1328	838.94	112.08
L/4	5.08	357.92	838.62	0.0999	834.45	83.61
3L/8	7.61	363.69	833.50	0.0667	831.65	55.55
L/2	10.15	343.13	851.77	0.0334	851.29	28.43

Tendon3	Xi	$\Sigma\Delta_{pT}$	F_j^3	(α)	$Fj^3 \cdot Cos(\alpha)$	$Fj^3 \cdot Sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	299.88	890.18	0.0879	886.74	78.14
L/8	2.54	317.99	874.10	0.0660	872.20	57.64
L/4	5.08	323.57	869.14	0.0440	868.29	38.26
3L/8	7.61	333.68	860.16	0.0220	859.95	18.94
L/2	10.15	311.72	879.66	0.0000	879.66	0.00

Tendon4	Xi	$\Sigma\Delta_{pT}$	F_j^4	(α)	$Fj^4 \cdot Cos(\alpha)$	$Fj^4 \cdot Sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	223.97	957.62	0.0426	956.75	40.79
L/8	2.54	243.19	940.54	0.0320	940.06	30.06
L/4	5.08	261.60	924.18	0.0213	923.97	19.70
3L/8	7.61	279.73	908.08	0.0107	908.03	9.68
L/2	10.15	263.64	922.38	0.0000	922.38	0.00

SUM 1to4	Xi	ΣF_j	Fj*Cos(α)	Vp=Fj*Sin(α)	e_{cable}	$M_j = \Sigma F_j Cos(\alpha) * e_{cap}$
Section	(mm)	(kN)	(kN)	(kN)	(m)	(kNm)
Support	0.00	3481.94	3457.45	372.69	0.05	171.9
L/8	0.00	3491.50	3474.27	310.33	0.25	853.6
L/4	0.00	3463.14	3454.60	215.69	0.38	1301.6
3L/8	0.00	3433.72	3430.77	121.39	0.46	1580.3
L/2	0.00	3474.91	3474.44	28.43	0.49	1697.6

Exterior Girder

Tendon1	Xi	$\Sigma\Delta_{pT}$	F_j^1	(α)	Fj^1*Cos(α)	Fj^1*Sin(α)
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	397.66	803.33	0.1772	790.75	141.61
L/8	0.00	362.77	834.32	0.1335	826.89	111.06
L/4	0.00	368.92	828.85	0.0893	825.55	73.92
3L/8	0.00	357.57	838.94	0.0447	838.10	37.52
L/2	0.00	382.22	817.04	0.0000	817.04	0.00

Tendon2	Xi	$\Sigma\Delta_{pT}$	F_j^2	(α)	Fj^2*Cos(α)	Fj^2*Sin(α)
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	366.72	830.81	0.1354	823.20	112.15
L/8	0.00	344.82	850.26	0.1328	842.77	112.59
L/4	0.00	360.56	836.28	0.0999	832.11	83.38
3L/8	0.00	355.84	840.47	0.0667	838.61	56.02
L/2	0.00	347.70	847.70	0.0334	847.23	28.30

Tendon3	Xi	$\Sigma\Delta_{pT}$	F_j^3	(α)	Fj^3*Cos(α)	Fj^3*Sin(α)
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	299.88	890.18	0.0879	886.74	78.14
L/8	0.00	313.64	877.96	0.0660	876.05	57.90
L/4	0.00	326.22	866.79	0.0440	865.95	38.15
3L/8	0.00	325.83	867.13	0.0220	866.92	19.10
L/2	0.00	316.30	875.60	0.0000	875.60	0.00

Tendon4	Xi	$\Sigma\Delta_{pT}$	F_j^4	(α)	Fj^4*Cos(α)	Fj^4*Sin(α)
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	223.97	957.62	0.0426	956.75	40.79
L/8	0.00	238.84	944.41	0.0320	943.93	30.19
L/4	0.00	264.25	921.84	0.0213	921.63	19.65
3L/8	0.00	271.88	915.05	0.0107	915.00	9.75
L/2	0.00	268.21	918.31	0.0000	918.31	0.00

SUM 1to4	Xi	ΣF_j	Fj*Cos(α)	Vp=Fj*Sin(α)	e_{caple}	$M_j = \Sigma F_j Cos(\alpha) * e_{cap}$
Section	(mm)	(kN)	(kN)	(kN)	(m)	(kNm)
Support	0.00	3481.94	3457.45	372.69	0.05	171.9
L/8	0.00	3506.95	3489.65	311.74	0.25	857.4
L/4	0.00	3453.76	3445.24	215.10	0.38	1298.1
3L/8	0.00	3461.60	3458.63	122.39	0.46	1593.2
L/2	0.00	3458.66	3458.18	28.30	0.49	1689.6

5. FIBRE STRESS CHECK:

Formula:

Top fibre: $f_{ti} = \frac{F_i}{A} - \frac{F_i e}{S_t} + \frac{M_{DC}}{S_t}$ Bottom fibre $f_{bi} = \frac{F_i}{A} + \frac{F_i e}{S_b} - \frac{M_{DC}}{S_b}$

Note (+) : Compression tresses ; (-) Tension stresses

Concrete strength at transfer	fci' = 0.9 fc =	40.50 MPa	
Comprresion stress Limit at transfer	0.6 fci' =	24.30 MPa	
Tension stress Limit at transfer	0.25 SQRT(fci') < 1.38 =	-1.38 MPa	(5.9.4.1.2-1)

Setion	Xi	A	St	Sb	Fj'*Cos(α)	e	M_{DC}	f_{ti}	f_{bi}	Kiểm tra	
	(mm)	(mm2)	(mm3)	(mm3)	(kN)	(mm)	(kNm)	(MPa)	(MPa)	f_{ti}	f_{bi}
girder end	0	7.47E+05	1.57E+08	1.49E+08	3899.60	49.71	0.00	3.99	6.52	OK	OK
Support	200	7.47E+05	1.57E+08	1.49E+08	3910.81	49.71	0.00	4.00	6.54	OK	OK
L/8	2738	6.94E+05	1.54E+08	1.45E+08	4034.39	245.70	332.08	1.53	10.36	OK	OK
L/4	5275	5.17E+05	1.43E+08	1.39E+08	4106.94	376.78	569.29	1.12	14.97	OK	OK
3L/8	7813	5.17E+05	1.43E+08	1.39E+08	4164.19	460.63	711.61	-0.36	16.72	OK	OK
L/2	10350	5.17E+05	1.43E+08	1.39E+08	4165.67	488.58	759.05	-0.84	17.23	OK	OK

5.2 Stress check during contruction the deck:

5.2.1 Increase load:

Exterior Diaphragms beam	DC_{dn1} =	35.57 (kN)
Interior Diaphragms beam	DC_{dn1} =	16.29 (kN)
Precast plank	DC_{VK} =	3.74 (kN/m)
Wet concrete of deck	DC_{mc} =	12.94 (kN/m)

5.2.2 Stress check:

Compression strength of concrete	fc =	45.00 MPa	
Compression stress limit	0.45 fc =	20.25 MPa	(5.9.4.2.1-1)
Tension stress limit	0.5 SQRT(fc) =	-3.35 MPa	(5.9.4.2.2-1)

Setion	Xi	A	St	Sb	Fi	e	M_{DC}	f_{ti}	f_{bi}	Kiểm tra	
	(mm)	(mm2)	(mm3)	(mm3)	(kN)	(mm)	(kNm)	(MPa)	(MPa)	f_{ti}	f_{bi}
Girder end	0	7.47E+05	1.57E+08	1.49E+08	3899.60	49.71	0.00	3.99	6.52	OK	OK
Support	200	7.47E+05	1.57E+08	1.49E+08	3910.81	49.71	0.00	4.00	6.54	OK	OK
L/8	2738	6.94E+05	1.54E+08	1.45E+08	4034.39	245.70	726.14	4.09	7.64	OK	OK
L/4	5275	5.17E+05	1.43E+08	1.39E+08	4106.94	376.78	1244.81	5.84	10.12	OK	OK
3L/8	7813	5.17E+05	1.43E+08	1.39E+08	4164.19	460.63	1556.02	5.53	10.66	OK	OK
L/2	10350	5.17E+05	1.43E+08	1.39E+08	4165.67	488.58	1659.75	5.44	10.76	OK	OK

5.3 Stress check at the top fibre of Girder - Service state :

5.3.1 Due to prestressing tendon and self weigh of girder - Service limit stage I:

Compression Stress Limit:	0.45 fc =	20.25 MPa	(5.9.4.2.1-1)
Tension Stress Limit:	- 0.5*SQRT(fc) =	-3.35 MPa	

$$f_f = \frac{P_{pe}}{A} - \frac{P_{pe}e_c}{S_t} + \frac{M_g + M_s}{S_t} + \frac{M_{SDL}}{S_{tg}}$$

Interior Girder

Setion	Xi	A	S_t	S_{tg}	P_{pe}	P_{pe}*e_c	M_g + M_s	M_{SDL}	f_t	Check
	(mm)	(mm2)	(mm3)	(mm3)	(kN)	(kNm)	(kNm)	(kNm)	(MPa)	ft
Support	200	7.47E+05	1.57E+08	7.22E+08	3457.45	171.87	0.00	0.00	3.535	OK
L/8	2738	6.94E+05	1.54E+08	7.32E+08	3474.27	853.63	726.14	206.92	4.462	OK
L/4	5275	5.17E+05	1.43E+08	7.93E+08	3454.60	1301.63	1244.81	354.71	6.738	OK
3L/8	7813	5.17E+05	1.43E+08	7.93E+08	3430.77	1580.33	1556.02	443.39	7.030	OK
L/2	10350	5.17E+05	1.43E+08	7.93E+08	3474.44	1697.55	1659.75	472.95	7.058	OK

Exterior Girder

Setion	Xi	A	S_t	S_{tg}	P_{pe}	P_{pe}*e_c	M_g + M_s	M_{SDL}	f_t	Check
	(mm)	(mm2)	(mm3)	(mm3)	(kN)	(kNm)	(kNm)	(kNm)	(MPa)	f_t
Support	200	7.47E+05	1.57E+08	7.22E+08	3457.45	171.87	0.00	0.00	3.535	OK
L/8	2738	6.94E+05	1.54E+08	7.32E+08	3489.65	857.41	674.91	208.16	4.129	OK
L/4	5275	5.17E+05	1.43E+08	7.93E+08	3445.24	1298.11	1156.99	356.84	6.134	OK
3L/8	7813	5.17E+05	1.43E+08	7.93E+08	3458.63	1593.16	1446.24	446.05	6.232	OK
L/2	10350	5.17E+05	1.43E+08	7.93E+08	3458.18	1689.61	1542.66	475.78	6.268	OK

5.3.2 Due to 1/2 (Prestressing tendon + self weigh of girder) and Live load - Service limit stage I:

Compression Stress Limit: 0.40 fc = 18.00 MPa (5.9.4.2.1-1)

Tension Stress Limit: - 0.5*SQRT(fc) = -3.35 MPa

$$f_t = 0.5\left(\frac{P_{pe}}{A} - \frac{P_{pe}e_c}{S_t} + \frac{M_g + M_s}{S_t} + \frac{M_{SDL}}{S_{tg}}\right) + \frac{M_{LL}}{S_{tg}}$$

Interior Girder

Setion	Xi (mm)	A (mm2)	S_t (mm3)	S_{tg} (mm3)	P_{pe} (kN)	$P_{pe}*e_c$ (kNm)	$M_g + M_s$ (kNm)	M_{SDL} (kNm)	M_{LL} (kNm)	f_t (MPa)	Check f_t
Support	200	7.47E+05	1.57E+08	7.22E+08	3457.45	171.87	0.00	0.00	0.00	1.767	OK
L/8	2738	6.94E+05	1.54E+08	7.32E+08	3474.27	853.63	726.14	206.92	747.09	3.252	OK
L/4	5275	5.17E+05	1.43E+08	7.93E+08	3454.60	1301.63	1244.81	354.71	1248.84	4.943	OK
3L/8	7813	5.17E+05	1.43E+08	7.93E+08	3430.77	1580.33	1556.02	443.39	1523.42	5.435	OK
L/2	10350	5.17E+05	1.43E+08	7.93E+08	3474.44	1697.55	1659.75	472.95	1589.00	5.532	OK

Exterior Girder

Setion	Xi (mm)	A (mm2)	S_t (mm3)	S_{tg} (mm3)	P_{pe} (kN)	$P_{pe}*e_c$ (kNm)	$M_g + M_s$ (kNm)	M_{SDL} (kNm)	M_{LL} (kNm)	f_t (MPa)	Check f_t
Support	200	747100.0	1.57E+08	7.22E+08	3457.45	171.87	0.00	0.00	0.00	1.767	OK
L/8	2738	693571.0	1.54E+08	7.32E+08	3489.65	857.41	674.91	208.16	830.60	3.199	OK
L/4	5275	516600.0	1.43E+08	7.93E+08	3445.24	1298.11	1156.99	356.84	1388.44	4.817	OK
3L/8	7813	516600.0	1.43E+08	7.93E+08	3458.63	1593.16	1446.24	446.05	1693.71	5.251	OK
L/2	10350	516600.0	1.43E+08	7.93E+08	3458.18	1689.61	1542.66	475.78	1766.62	5.361	OK

5.3.3 Due to prestressing tendon + self weigh of girder + live load - Service limit stage I:

Compression Stress Limit: 0.60 fc = 27.00 MPa (5.9.4.2.1-1)

Tension Stress Limit: - 0.5*SQRT(fc) = -3.35 MPa

$$f_t = \left(\frac{P_{pe}}{A} - \frac{P_{pe}e_c}{S_t} + \frac{M_g + M_s}{S_t} + \frac{M_{SDL}}{S_{tg}}\right) + \frac{M_{LL}}{S_{tg}}$$

Interior Girder

Setion	Xi (mm)	A (mm2)	S_t (mm3)	S_{tg} (mm3)	P_{pe} (kN)	$P_{pe}*e_c$ (kNm)	$M_g + M_s$ (kNm)	M_{SDL} (kNm)	M_{LL} (kNm)	f_t (MPa)	Check f_t
Support	200	7.47E+05	1.57E+08	7.22E+08	3457.45	171.87	0.00	0.00	0.00	3.535	OK
L/8	2738	6.94E+05	1.54E+08	7.32E+08	3474.27	853.63	726.14	206.92	747.09	5.483	OK
L/4	5275	5.17E+05	1.43E+08	7.93E+08	3454.60	1301.63	1244.81	354.71	1248.84	8.312	OK
3L/8	7813	5.17E+05	1.43E+08	7.93E+08	3430.77	1580.33	1556.02	443.39	1523.42	8.951	OK
L/2	10350	5.17E+05	1.43E+08	7.93E+08	3474.44	1697.55	1659.75	472.95	1589.00	9.061	OK

Exterior Girder

Setion	Xi (mm)	A (mm2)	S_t (mm3)	S_{tg} (mm3)	P_{pe} (kN)	$P_{pe}*e_c$ (kNm)	$M_g + M_s$ (kNm)	M_{SDL} (kNm)	M_{LL} (kNm)	f_t (MPa)	Check f_t
Support	200	7.47E+05	1.57E+08	7.22E+08	3457.45	171.87	0.00	0.00	0.00	3.535	OK
L/8	2738	6.94E+05	1.54E+08	7.32E+08	3489.65	857.41	674.91	208.16	830.60	5.263	OK
L/4	5275	5.17E+05	1.43E+08	7.93E+08	3445.24	1298.11	1156.99	356.84	1388.44	7.884	OK
3L/8	7813	5.17E+05	1.43E+08	7.93E+08	3458.63	1593.16	1446.24	446.05	1693.71	8.367	OK
L/2	10350	5.17E+05	1.43E+08	7.93E+08	3458.18	1689.61	1542.66	475.78	1766.62	8.495	OK

5.4 Stress check at the top fibre of deck - Service state:

5.4.1 Due to additional load (dead load part 2) - Service limit stage I:

Compression Stress Limit: 0.45 fc = 15.75 MPa (5.9.4.2.1-1)

$$f_t = \frac{M_{SDL}}{S_{tc}}$$

Setion	Xi	MSDL (kNm)		S_{tc} (mm3)		f_t (MPa)		Check	
	(mm)	in.Girder	Ex.Girder	in.Girder	Ex.Girder	in.Girder	Ex.Girder	in.Girder	Ex.Girder
Support	200.00	0.00	0.00	3.8E+08	3.76E+08	0.000	0.000	OK	OK
L/8	2737.50	206.92	208.16	3.7E+08	3.74E+08	0.553	0.556	OK	OK
L/4	5275.00	354.71	356.84	3.8E+08	3.78E+08	0.939	0.944	OK	OK
3L/8	7812.50	443.39	446.05	3.8E+08	3.78E+08	1.173	1.180	OK	OK
L/2	10350.00	472.95	475.78	3.8E+08	3.78E+08	1.252	1.259	OK	OK

5.4.2 **Due to additional load (dead load part 2) and live load - Service limit stage I:**

Compression Stress Limit: 0.6 fc = 21.00 MPa (5.9.4.2.1-1)

$$f_{tc} = \frac{M_{SDL} + M_{LL}}{S_{tc}}$$

Setion	Xi	MSDL + MLL (kNm)		S_{tc} (mm3)		f_t (MPa)		Check	
	(mm)	in.Girder	Ex.Girder	in.Girder	Ex.Girder	in.Girder	Ex.Girder	in.Girder	Ex.Girder
Support	200.00	0.00	0.00	3.8E+08	3.76E+08	0.000	0.000	OK	OK
L/8	2737.50	954.00	1038.75	3.7E+08	3.74E+08	2.548	2.775	OK	OK
L/4	5275.00	1603.55	1745.28	3.8E+08	3.78E+08	4.244	4.619	OK	OK
3L/8	7812.50	1966.81	2139.76	3.8E+08	3.78E+08	5.205	5.663	OK	OK
L/2	10350.00	2061.95	2242.40	3.8E+08	3.78E+08	5.457	5.935	OK	OK

5.5 Stress check at the bottom fibre of girder - Service III (Stage III):

Compression Stress Limit: 0.45 fc = 27.00 MPa (5.9.4.2.1-1)

Tension Stress Limit: - 0.5*SQRT(fc) = -3.35 MPa (5.9.4.2.1-1)

$$f_b = \frac{P_{pe}}{A} + \frac{P_{pe}e_c}{S_b} - \frac{M_g + M_s}{S_b} - \frac{M_{SDL} + 0.8M_{LL}}{S_{bc}}$$

Interior Girder

Setion	Xi (mm)	A (mm2)	S_b (mm3)	S_{bc} (mm3)	P_{pe} (kN)	P_{pe}*e_c (kNm)	$M_g + M_s$ (kNm)	M_{SDL} (kNm)	M_{LL} (kNm)	f_b (MPa)	Check f_b
Support	200	7.47E+05	1.49E+08	2.57E+08	3457.45	171.87	0.00	0.00	0.00	5.784	OK
L/8	2738	6.94E+05	1.45E+08	2.46E+08	3474.27	853.63	726.14	206.92	747.09	2.622	OK
L/4	5275	5.17E+05	1.39E+08	2.16E+08	3454.60	1301.63	1244.81	354.71	1248.84	0.835	OK
3L/8	7813	5.17E+05	1.39E+08	2.16E+08	3430.77	1580.33	1556.02	443.39	1523.42	-0.870	OK
L/2	10350	5.17E+05	1.39E+08	2.16E+08	3474.44	1697.55	1659.75	472.95	1589.00	-1.068	OK

Exterior Girder

Setion	Xi (mm)	A (mm2)	S_b (mm3)	S_{bc} (mm3)	P_{pe} (kN)	P_{pe}*e_c (kNm)	$M_g + M_s$ (kNm)	M_{SDL} (kNm)	M_{LL} (kNm)	f_b (MPa)	Check f_b
Support	200	7.47E+05	1.49E+08	2.57E+08	3457.45	171.87	0.00	0.00	0.00	5.784	OK
L/8	2738	6.94E+05	1.45E+08	2.46E+08	3489.65	857.41	674.91	208.16	830.60	2.747	OK
L/4	5275	5.17E+05	1.39E+08	2.16E+08	3445.24	1298.11	1156.99	356.84	1388.44	0.896	OK
3L/8	7813	5.17E+05	1.39E+08	2.16E+08	3458.63	1593.16	1446.24	446.05	1693.71	-0.578	OK
L/2	10350	5.17E+05	1.39E+08	2.16E+08	3458.18	1689.61	1542.66	475.78	1766.62	-0.986	OK

5.6 Stress check at the bottom fibre of girder - Service I (Stage III):

Compression Stress Limit: 0.45 fc = 27.00 MPa (5.9.4.2.1-1)

Tension Stress Limit: - 0.5*SQRT(fc) = -3.35 MPa (5.9.4.2.1-1)

$$f_b = \frac{P_{pe}}{A} + \frac{P_{pe}e_c}{S_b} - \frac{M_g + M_s}{S_b} - \frac{M_{SDL} + M_{LL}}{S_{bc}}$$

Interior Girder

Setion	Xi (mm)	A (mm2)	S_b (mm3)	S_{bc} (mm3)	P_{pe} (kN)	P_{pe}*e_c (kNm)	$M_g + M_s$ (kNm)	M_{SDL} (kNm)	M_{LL} (kNm)	f_b (MPa)	Check f_b
Support	200	7.47E+05	1.49E+08	2.57E+08	3457.45	171.87	0.00	0.00	0.00	5.784	OK
L/8	2738	6.94E+05	1.45E+08	2.46E+08	3474.27	853.63	726.14	206.92	747.09	2.016	OK
L/4	5275	5.17E+05	1.39E+08	2.16E+08	3454.60	1301.63	1244.81	354.71	1248.84	-0.320	OK
3L/8	7813	5.17E+05	1.39E+08	2.16E+08	3430.77	1580.33	1556.02	443.39	1523.42	-2.279	OK
L/2	10350	5.17E+05	1.39E+08	2.16E+08	3474.44	1697.55	1659.75	472.95	1589.00	-2.538	OK

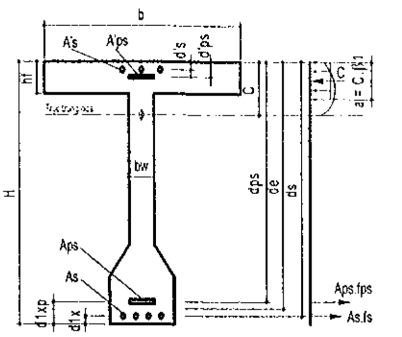
Exterior Girder

Setion	Xi (mm)	A (mm2)	S_b (mm3)	S_{bc} (mm3)	P_{pe} (kN)	P_{pe}*e_c (kNm)	$M_g + M_s$ (kNm)	M_{SDL} (kNm)	M_{LL} (kNm)	f_b (MPa)	Check f_b
Support	200	7.47E+05	1.49E+08	2.57E+08	3457.45	171.87	0.00	0.00	0.00	5.784	OK
L/8	2738	6.94E+05	1.45E+08	2.46E+08	3489.65	857.41	674.91	208.16	830.60	2.073	OK
L/4	5275	5.17E+05	1.39E+08	2.16E+08	3445.24	1298.11	1156.99	356.84	1388.44	-0.388	OK
3L/8	7813	5.17E+05	1.39E+08	2.16E+08	3458.63	1593.16	1446.24	446.05	1693.71	-2.145	OK
L/2	10350	5.17E+05	1.39E+08	2.16E+08	3458.18	1689.61	1542.66	475.78	1766.62	-2.620	OK

REINFORCEMENT OF GIRDER CHECKING - STRENGTH LOAD COMBINATION

MATERIALS

	NORMAL CONCRETE		
f'c	Compressive Strength of concrete at 28 days	Mpa	45
Ec	Modulus of Elasticity	Mpa	33915
fr	Modulus of Rupture	Mpa	4.2
gc	Unit weight of concrete	kN/m3	24.0
	PRESTRESSING STEEL		
fpu	Tensile strength of prestressing steel	Mpa	1860
fpy	Yield strength of prestressing steel	Mpa	1674
Ep	Modulus of Elasticity	Mpa	197000
	REINFORCEMENT		
fy	Yield strength	Mpa	400
Es	Modulus of Elasticity	Mpa	200000
nc	Ratio Es/Ec		6

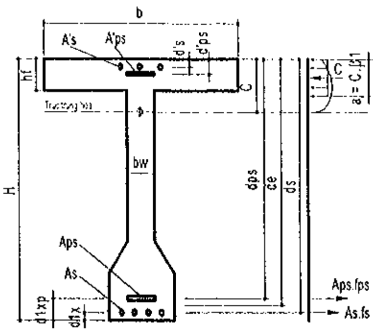


Sign	Parameters		Unit	Support	L/8	L/4	3L/8	L/2
				Section				
	INTERNAL FORCES AT SECTION							
	Combination			Strength	Strength	Strength	Strength	Strength
Qu	Shear		kN	1237	988	743	503	268
Mu	Flexural Moment		kNm	0	2582	4365	5383	5672
Nu	Axial load		kN					
Tu	Torsional Moment		kNm					
	6.1 FLEXURAL MOMENT CHECKING							
H	Section height		m	1.400	1.400	1.400	1.400	1.400
d's	Dis. From comp. fiber to centroid of comp. Reinf		m	0.062	0.062	0.062	0.062	0.062
d1x	Dis. From tens. fiber to centroid of tension Reinf		m	0.061	0.061	0.061	0.061	0.061
	Cover to reinf		m	0.040	0.040	0.040	0.040	0.040
ds	Dis. From comp. fiber to centroid of tension Reinf		m	1.339	1.339	1.339	1.339	1.339
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	0.454	0.297	0.185	0.118	0.096
dps	Dis. From comp. fiber to centroid of tension prestressing steel		m	0.946	1.103	1.215	1.282	1.304
b	Width of the compression face of member		m	2.249	2.249	2.249	2.249	2.249
bw	Web width or diameter of a circular section		m	0.600	0.522	0.200	0.200	0.200
hf	Compression flange depth		m	0.200	0.200	0.200	0.200	0.200
Iz	Moment of inertia of section		m4	0.224	0.218	0.201	0.201	0.201
Amc	Section area		m2	1.197	1.143	0.966	0.966	0.966
	Steel choice							
	Tension prestressing steel	P.S type		9 T12.7	9 T12.7	9 T12.7	9 T12.7	9 T12.7
Aps		Number	tendons	4	4	4	4	4
		Area	m2	0.00355	0.00355	0.00355	0.00355	0.00355
	Compression prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	0 T00.0
A'ps		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
	Tension Reinforcement	Number	bars	6	6	6	6	6
As		Diameter	mm	16	16	16	16	16
		Area	m2	0.00121	0.00121	0.00121	0.00121	0.00121
	Compression Reinforcement	Number	bars	4	4	4	4	4
A's		Diameter	mm	12	12	12	12	12
		Area	m2	0.00045	0.00045	0.00045	0.00045	0.00045
	Shear reinforcement	Number	bars	2	2	2	2	2
A'c		Diameter	mm	14	14	14	14	14
		Area	m2	0.00030	0.00030	0.00030	0.00030	0.00030
f	Resistance factors for flexure		5.5.4.2	0.90	0.90	0.90	0.90	0.90
fv	Resistance factors for shear			0.90	0.90	0.90	0.90	0.90
fn	Resistance factors for axial force			1.00	1.00	1.00	1.00	1.00
b1	Stress block factor			0.729	0.729	0.729	0.729	0.729
c	Dis. Between centroid and top fiber		m	0.107	0.107	0.108	0.108	0.108
	For T section behavior		m	-0.122	-0.167	-0.635	-0.642	-0.645
	For rectangular section behavior		m	0.107	0.107	0.108	0.108	0.108
fpe	Effective stress in the prestressing steel after losses		Mpa	803	830	831	832	821
fps	Aver. stress in pres. steel at the time for which the nominal resis		Mpa	1801	1809	1814	1816	1817
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	0.078	0.078	0.078	0.079	0.079
de	Corresp. effective depth from extreme comp. fiber to centroid of tensile force in the tensile reinf.	m	0.974	1.119	1.223	1.286	1.306
Mn	Nominal resistance	kNm	5872	6878	7507	7950	8098
Mr	Factored resistance	kNm	**5284**	**6190**	**6756**	**7155**	**7288**
Mu	Flexual moment	kNm	0	2582	4365	5383	5672
(5.7.3.2)	**Flexural moment Checking**		OK	OK	OK	OK	OK
	Limits for reinforcement						
c/de	Maximum reinforcement		0.11	0.10	0.09	0.08	0.08
	Maximum reinforcement Checking	<= 0.42	OK	OK	OK	OK	OK
r min	Mimimum reinforcement		0.10%	0.11%	0.13%	0.13%	0.13%
	Mimimum reinforcement Checking for RC	0.34%	N.a	N.a	N.a	N.a	N.a
1.2*Mcr	Craking moment	kNm	879	856	790	790	790
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**		OK	OK	OK	OK	OK
(5.8.3.5)	Tensile force in steel should be satisfied - F_yc	kN	1349	3656	4763	5111	5139
	Checking As.fy+Aps.fps >= F_yc		Ok	Ok	Ok	Ok	Ok
(5.7.3.4)	**Conctrol of craking by distr. of reinf for RC member- Check?**		No	No	No	No	No
	Existing condition for structrure	1,2 or 3	1	1	1	1	1
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.058	0.058	0.058	0.058	0.058
Z	Crack width parameter	N/mm	30000	30000	30000	30000	30000
A	Area of concr. with same centroid as tens. Reinf	m2	0.043	0.043	0.043	0.043	0.043
fsa	Value	Mpa	220	220	220	220	220
0.6*fy		Mpa	240	240	240	240	240
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	220	220	220	220	220
x	Dist. From compression fiber to centroid	m	-	-	-	-	-
J.d	Arm	m	-	-	-	-	-
Icr	Moment of inertia of the cracked section	m4	-	-	-	-	-
fs	Tensile stress in reinforcement fs = Msls / (As*J.d)	Mpa	-	-	-	-	-
	Checking for control crackIng fs<fsa		N.a	N.a	N.a	N.a	N.a
(5.10.8.2)	**Shrinkage and temperature Reinfocement (side distribution)**						
Areq	Area of required reinf	m2	0.00031	0.00029	0.00025	0.00025	0.00025
	Distribution on sides 9 D12	m2	0.00102	0.00102	0.00102	0.00102	0.00102
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45	0.45
	Checking		OK	OK	OK	OK	OK
	6.2 SHEAR FORCE CHECKING						
β	Factor indicating diag. cracked concr. to tension		5.8	3.0	2.0	1.9	1.8
θ	Angle of inclination of diagonal compressive	degree	27.00	28.76	37.96	41.80	42.01
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	0.600	0.522	0.200	0.200	0.200
dv	Effective shear depth	m	1.008	1.080	1.184	1.246	1.267
	(de - a/2)	m	0.935	1.080	1.184	1.246	1.267
s	Spacing of stirrups	m	0.150	0.150	0.300	0.300	0.300
ncat	Amount of bars in spacing S	bars	2	2	2	2	2
Av	Shear reinf area in spacing S	m2	0.0003	0.0003	0.0003	0.0003	0.0003
β	Assume		5.9	3.9	2.2	2.0	2.0
θ	Assume	degree	27.00	27.00	35.79	39.67	40.92
v	Shear stress in concrete	kN/m2	2272	1946	3486	2242	1174
fpo	Parameter taken as modulus of elasticity of prestressing tendon	Mpa	817	845	849	850	839
e_x	Strain in tensile reinforcement		-1.79E-03	3.78E-04	1.26E-03	1.70E-03	1.75E-03
	if ex<0, multiple with reduce factor		-1.11E-04	-	-	-	-
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok	Ok
v/f'c	Ratio of shear stress and f'c		0.050	0.043	0.077	0.050	0.026
β	Final value		5.8	3.0	2.0	1.9	1.8
θ	Final value	degree	27.00	28.76	37.96	41.80	42.01
Vc	Nominal shear resistance provided by tensile stresses in the co	kN	1937	939	265	258	259
Vs	Shear resistance provided by shear reinforcement	kN	1593	1585	611	561	567
Vp	Component in the direction of the applied shear of the effective	kN	0	0	0	0	0
Vn1	Vn1=Vc+Vs+Vp	kN	3530	2524	877	819	825
Vn2	Vn2	kN	6804	6343	2664	2804	2851
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	3530	2524	877	819	825
Vr	Factored shear resistance	kN	**3177**	**2272**	**789**	**737**	**743**
Vu	Shear	kN	1237	988	743	503	268
(5.8.2.7)	**Shear checking**		OK	OK	OK	OK	OK
	Region requiring transverse reinf Checking		Need	Need	Need	Need	Need
	Minimum shear reinf area	m2	0.0001	0.0001	0.0001	0.0001	0.0001
	Mimimum shear reinforcement Checking		OK	OK	OK	OK	OK
	0.1*f'c*bv*dv	kN	2722	2537	1066	1122	1140
	Smax	m	0.60	0.60	0.60	0.60	0.60
	Maximum spacing Smax		OK	OK	OK	OK	OK

REINFORCEMENT OF GIRDER CHECKING - SERVICE LOAD COMBINATION

MATERIALS



	NORMAL CONCRETE		
f'c	Compressive Strength of concrete at 28 days	Mpa	45
Ec	Modulus of Elasticity	Mpa	33915
fr	Modulus of Rupture	Mpa	4.2
gc	Unit weight of concrete	kN/m3	24.0
	PRESTRESSING STEEL		
fpu	Tensile strength of prestressing steel	Mpa	1860
fpy	Yield strength of prestressing steel	Mpa	1674
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				
				Support	L/8	L/4	3L/8	L/2
	INTERNAL FORCES AT SECTION							
	Combination			Service	Service	Service	Service	Service
Qu	Shear		kN	821	650	481	316	153
Mu	Flexural Moment		kNm	0	1714	2902	3586	3785
Nu	Axial load		kN					
Tu	Torsional Moment		kNm					
	6.1 FLEXURAL MOMENT CHECKING							
H	Section height		m	1.416	1.416	1.416	1.416	1.416
d's	Dis. From comp. fiber to centroid of comp. Reinf		m	0.062	0.062	0.062	0.062	0.062
d1x	Dis. From tens. fiber to centroid of tension Reinf		m	0.061	0.061	0.061	0.061	0.061
	Cover to reinf		m	0.040	0.040	0.040	0.040	0.040
ds	Dis. From comp. fiber to centroid of tension Reinf		m	1.355	1.355	1.355	1.355	1.355
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	0.454	0.297	0.185	0.118	0.096
dps	Dis. From comp. fiber to centroid of tension prestressing steel		m	0.962	1.119	1.231	1.298	1.320
b	Width of the compression face of member		m	2.550	2.550	2.550	2.550	2.550
bw	Web width or diameter of a circular section		m	0.600	0.522	0.200	0.200	0.200
hf	Compression flange depth		m	0.216	0.216	0.216	0.216	0.216
Iz	Moment of inertia of section		m4	0.224	0.218	0.201	0.201	0.201
Amc	Section area		m2	1.197	1.143	0.966	0.966	0.966
	Steel choice							
Aps	Tension prestressing steel	P.S type		9 T12.7	9 T12.7	9 T12.7	9 T12.7	9 T12.7
		Number	tendons	4	4	4	4	4
		Area	m2	0.00355	0.00355	0.00355	0.00355	0.00355
A'ps	Compression prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	0 T00.0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
As	Tension Reinforcement	Number	bars	6	6	6	6	6
		Diameter	mm	16	16	16	16	16
		Area	m2	0.00121	0.00121	0.00121	0.00121	0.00121
A's	Compression Reinforcement	Number	bars	4	4	4	4	4
		Diameter	mm	12	12	12	12	12
		Area	m2	0.00045	0.00045	0.00045	0.00045	0.00045
A'c	Shear reinforcement	Number	bars	2	2	2	2	2
		Diameter	mm	14	14	14	14	14
		Area	m2	0.00030	0.00030	0.00030	0.00030	0.00030
f	Resistance factors for flexure		5.5.4.2	1.00	1.00	1.00	1.00	1.00
fv	Resistance factors for shear			1.00	1.00	1.00	1.00	1.00
fn	Resistance factors for axial force			1.00	1.00	1.00	1.00	1.00
b1	Stress block factor			0.729	0.729	0.729	0.729	0.729
c	Dis. Between centroid and top fiber		m	0.095	0.095	0.095	0.095	0.095
	For T section behavior		m	-0.259	-0.327	-1.022	-1.033	-1.037
	For rectangular section behavior		m	0.095	0.095	0.095	0.095	0.095
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	1809	1816	1820	1822	1822
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	0.069	0.069	0.069	0.069	0.070
de	Corresp. effective depth from extreme comp. fiber to centroid of tensile force in the tensile reinf.	m	0.990	1.135	1.239	1.302	1.322
Mn	Nominal resistance	kNm	5735	6734	7330	7774	7921
Mr	Factored reistance	kNm	5735	6734	7330	7774	7921
Mu	Flexual moment	kNm	0	1714	2902	3586	3785
(5.7.3.2)	**Flexural moment Checking**		OK	OK	OK	OK	OK
	Limits for reinforcement						
c/de	Maximum reinforcement		0.10	0.08	0.08	0.07	0.07
	Maximum reinforcement Checking	<= 0.42	OK	OK	OK	OK	OK
r min	Mimimum reinforcement		0.10%	0.11%	0.13%	0.13%	0.13%
	Mimimum reinforcement Checking for RC	0.34%	N.a	N.a	N.a	N.a	N.a
1.2*Mcr	Craking moment	kNm	860	838	773	773	773
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**		OK	OK	OK	OK	OK
(5.8.3.5)	Tensile force in steel should be satisfied - F_yc	kN	805	2194	2882	3141	3090
	Checking As.fy+Aps.fps >= F_yc		Ok	Ok	Ok	Ok	Ok
(5.7.3.4)	**Conctrol of craking by distr. of reinf for RC member- Check?**		Yes	Yes	Yes	Yes	Yes
	Existing condition for structrure	1,2 or 3	1	1	1	1	1
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.058	0.058	0.058	0.058	0.058
Z	Crack width parameter	N/mm	30000	30000	30000	30000	30000
A	Area of concr. with same centroid as tens. Reinf	m2	0.049	0.049	0.049	0.049	0.049
fsa	Value	Mpa	211	211	211	211	211
0.6*fy		Mpa	240	240	240	240	240
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	211	211	211	211	211
x	Dist. From compression fiber to centroid	m	0.072	0.078	0.081	0.083	0.084
J.d	Arm	m	0.966	1.109	1.212	1.274	1.294
Icr	Moment of inertia of the cracked section	m4	0.012	0.012	0.012	0.012	0.012
fs	Tensile stress in reinforcement fs = Msls / (As*J.d)	Mpa	-	1275	1976	2323	2413
	Checking for control cracking fs<fsa		N.a	N.a	N.a	N.a	N.a
(5.10.8.2)	**Shrinkage and temperature Reinfocement (side distribution)**						
Areq	Area of required reinf	m2	0.00028	0.00027	0.00023	0.00023	0.00023
	Distribution on sides 18 D12	m2	0.00204	0.00204	0.00204	0.00204	0.00204
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45	0.45
	Checking		OK	OK	OK	OK	OK

6.2 SHEAR FORCE CHECKING

β	Factor indicating diag. cracked concr. to tension		6.8	6.3	6.8	6.6	6.5
θ	Angle of inclination of diagonal compressive	degree	27.00	27.00	27.00	27.00	27.00
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	90
bv	Effective web width as minimum web width - in dv	m	0.600	0.522	0.200	0.200	0.200
dv	Effective shear depth	m	1.020	1.101	1.205	1.267	1.288
	(de - a/2)	m	0.955	1.101	1.205	1.267	1.288
s	Spacing of stirrups	m	0.150	0.150	0.150	0.150	0.150
ncat	Amount of bars in spacing S	bars	2	2	2	2	2
Av	Shear reinf area in spacing S	m2	0.0003	0.0003	0.0003	0.0003	0.0003
β	Assume		2.0	2.0	2.0	2.0	2.0
θ	Assume	degree	45.00	45.00	45.00	45.00	45.00
v	Shear stress in concrete	kN/m2	1341	1131	1998	1247	594
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1135	1136	1140	1140	1140
e_x	Strain in tensile reinforcement		-3.85E-03	-2.29E-03	-1.49E-03	-1.13E-03	-1.10E-03
	if ex<0, multiple with reduce factor		-2.36E-04	-1.60E-04	-2.44E-04	-1.85E-04	-1.80E-04
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok	Ok
v/f'c	Ratio of shear stress and f'c		0.030	0.025	0.044	0.028	0.013
β	Final value		6.8	6.3	6.8	6.6	6.5
θ	Final value	degree	27.00	27.00	27.00	27.00	27.00
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	2309	2012	909	930	937
Vs	Shear resistance provided by shear reinforcement	kN	1611	1740	1904	2002	2035
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	3921	3752	2813	2933	2972
Vn2	Vn2	kN	6882	6464	2710	2851	2897
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	3921	3752	2710	2851	2897
Vr	Factored shear resistance	kN	3921	3752	2710	2851	2897
Vu	Shear	kN	821	650	481	316	153
(5.8.2.7)	**Shear checking**		OK	OK	OK	OK	OK
	Region requiring transverse reinf Checking		No need	No need	Need	No need	No need
	Minimum shear reinf area	m2	0.0001	0.0001	0.0000	0.0000	0.0000
	Mimimum shear reinforcement Checking		-	-	OK	-	-
	0.1*f'c*bv*dv	kN	2753	2585	1084	1140	1159
	Smax	m	0.60	0.60	0.60	0.60	0.60
	Maximum spacing Smax		-	-	OK	-	-

CALCULATION SHEET

I24 GIRDER

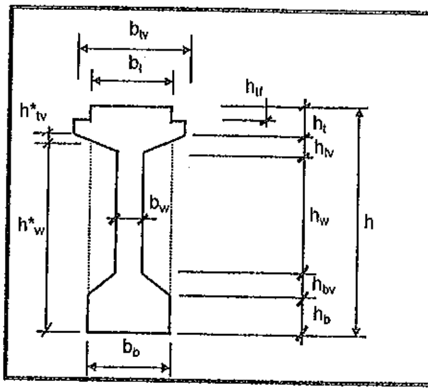
CONTENT

1. INPUT DATA:

1.1. General Data

Specification for bridge design:	TCN 272-05		
Live load	HL93		
Impact (or dynamic) of the live load	IM =	0.25	
Pedestrian	PL =	0.00	(kN/m^2)
Length of Girder	L_d =	24.00	(m)
Span between support	L_{tt} =	23.20	(m)
Carriageway width in bridge	w =	11.75	(m)
Parapet width	c =	0.50	(m)
Bridge width	B =	12.75	(m)
Number of girder	N_d =	5.00	girder
Space betweem 2 girders	S =	2.55	(m)
Distance from inside of parapet to exterior girder center	d_e =	0.78	(m)
Width of bridge deck	b_{ds} =	12.48	(m)
Length of the overhang (cantilever arm length)	L_h =	1.28	(m)
Thickness of bridge deck	t_s =	0.22	(m)
Precast plank width	b_p =	1.95	(m)
Precast plank thick	h_p =	0.08	(m)
Pavement thick	h_{pa} =	0.084	(m)

1.2. Girder dimension:



Width of over part		b_{tv} =	800.00	(mm)
		b_t =	600.00	(mm)
Width of under part		b_b =	600.00	(mm)
Girder high		h =	1450.00	(mm)
		h_{tf} =	80.00	(mm)
		h_t =	200.00	(mm)
Cross section at end		*at the midle*		
b_w =	600.00		200.00	(mm)
h^*_{tv} =	37.00		110.00	(mm)
h^*_w =	1213.00		690.00	(mm)
h_b =	0.00		250.00	(mm)
h_{bv} =	0.00		200.00	(mm)

1.3. MATERIAL PROPERTIES:

1.3.1 Concrete:

Girder concrete				
Girder concrete strength at the 28 age days	f'_c =	45.00	MPa	
Unit weight of Concrete	γ_c =	2400.00	kG/m^3	
Modulus of elasticity	$E_c = 0.043\, \gamma_c^{1.5}\, sqrt(f'_c)$ =	33914.98	MPa	(5.4.2.4-1)
Deck concrete				
Deck concrete strength at the 28 age days	f'_c =	35.00	MPa	
Unit weight of concrete	γ_c =	2400.00	kG/m^3	
Modulus of elasticity	$E_c = 0.043\, \gamma_c^{1.5}\, sqrt(f'_c)$ =	29910.20	MPa	(5.4.2.4-1)

1.3.2 Prestressing steel

Diameter of one strand	D =	12.70	mm
Area of one strand	$A_s^{12.7}$ =	98.70	mm^2
Ultimate Tendon strength	f_{pu} =	1860.00	MPa
Yield strength of prestressing steel	$f_{py} = 0.9\, f_{pu}$ =	1674.00	MPa
Modulus of strand	E_p =	197000.00	MPa
Wobble friction coefficient (mm-1)	K =	6.60E-07	mm^{-1}
Coeficient of friction (1/RAD)	μ =	0.25	
Number of Strands in one Tendon	n =	9.00	Strands
Area of one Tendon	A_s =	888.30	mm^2
Stress in the prestressing steel at jacking	$f_{pj} = 0.7\, f_{pu}$ =	1302.00	MPa
Jacking force for one tendon	P_j =	1156.57	kN
Anchorage set	ΔL =	6.00	mm
Area of one duck	A_g =	3318.31	mm^2
Number of Tendons	N =	5.00	Tendons

1.3.3 Reinforcing Steel:

Yield strength (deformed bar)	f_{py} =	400.00	(MPa)
Modulus of steel	E_s =	200000.00	(MPa)

2. INTERNAL FORCE:

2.1. Dead Load:

2.1.1 Load:

Interior Beam:

Bridge deck	DC_d =	12.94 (kN/m)
Precat plank & cross beam	DC_{pl} =	4.65 (kN/m)
Parapet	DC_{pa} =	4.74 (kN/m)
Pavement	DW_p =	4.44 (kN/m)

Exterior Beam:

Bridge deck	DC_d =	12.94 (kN/m)
Precat plank & cross beam	DC_{pl} =	2.32 (kN/m)
Parapet	DC_{pa} =	4.80 (kN/m)
Pavement	DW_p =	4.44 (kN/m)

2.1.2 Internal Force due to dead load:

Formula :

$M = 0.5\ q.X_i\ (L-X_i)$

$Q = q.(0.5.L-X_i)$

L_{tt} = 23.20 (m)

Section	X_i (m)	Girder (DC) M (kNm)	Girder (DC) Q (kN)	Concrete Deck (DC) M (kNm)	Concrete Deck (DC) Q (kN)	Plank & cr.beam (DC) M (kNm)	Plank & cr.beam (DC) Q (kN)	Parapet (DC) M (kNm)	Parapet (DC) Q (kN)	Pavement (DW) M (kNm)	Pavement (DW) Q (kN)
INTERIOR GIRDER											
Support	0.00	0.00	191.50	0.00	150.10	0.00	53.91	0.00	54.98	0.00	51.52
L/8	2.90	485.93	143.62	380.87	112.57	136.81	40.44	139.52	41.24	130.74	38.64
L/4	5.80	833.02	95.75	652.92	75.05	234.52	26.96	239.18	27.49	224.12	25.76
3L/8	8.70	1041.28	47.87	816.14	37.52	293.16	13.48	298.98	13.75	280.15	12.88
L/2	11.60	1110.70	0.00	870.55	0.00	312.70	0.00	318.91	0.00	298.82	0.00
EXTERIOR GIRDER											
Gối	0.00	0.00	191.50	0.00	150.10	0.00	53.91	0.00	55.62	0.00	51.52
L/8	2.90	485.93	143.62	380.87	112.57	68.40	40.44	141.14	41.72	130.74	38.64
L/4	5.80	833.02	95.75	652.92	75.05	117.26	26.96	241.96	27.81	224.12	25.76
3L/8	8.70	1041.28	47.87	816.14	37.52	146.58	13.48	302.44	13.91	280.15	12.88
L/2	11.60	1110.70	0.00	870.55	0.00	156.35	0.00	322.61	0.00	298.82	0.00

2.2 Live Load:

2.2.1. Distribution factors for Live load:

Modular Ratio: Girder Concrete/Deck Concrete	$n = E_g / E_d$ =	1.13
Distance from girder centroid to bridge deck centroid	$e^I g$ =	825.75 (mm)
	$e^E g$ =	825.75 (mm)
Longgitudinal stiffness parameter	$K^I g = n(Ig + A\ e^2_g)$ =	1.1E+12
	$K^E g = n(Ig + A\ e^2_g)$ =	1.1E+12
Ration	$K^I g / (L\ t^3_s)$ =	4.87
	$K^E g / (L\ t^3_s)$ =	4.87
	S / L =	0.11

(a) Distribution Factor for Moment: g(M)

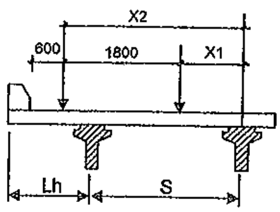
Interior Beam:

For one lane $$0.06+\left(\frac{S}{4300}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{Lt_s^3}\right)^{0.1} = \quad \mathbf{0.550}$$

Two or more lanes $$0.075+\left(\frac{S}{2900}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{Lt_s^3}\right)^{0.1} = \quad \mathbf{0.772}$$

Exterior Beam:

For one lane, follow the lever rule



X_1 = 925.000

X_2 = 2725.00

Y_1 = 0.363

Y_2 = 1.069

=> $g(M) = 0.5*\Sigma yi = \boxed{0.716}$

Two or more lanes

$$e = 0.77 + \frac{d_e}{2800} = \quad 1.047 < (=)\ 1$$

Choice e = 1.047 IF(e>1,1,e)

=> $g(M) = e*g_{trong} = \boxed{0.808}$

(b) Distribution Factor for Shear force: g(Q)

Interior Beam:

For one lane $$0.36 + \frac{S}{7600} = \quad \mathbf{0.696}$$

Two or more lanes $$0.2 + \frac{S}{3600} - \left(\frac{S}{10700}\right)^2 = \quad \mathbf{0.852}$$

Exterior Beam:
For one lane, follow the lever rule

$g(Q) = 0.5*\Sigma yi =$ $\boxed{0.716}$

Two or more lanes

$$e = 0.6 + \frac{de}{3000} = \quad 0.858$$

$\Rightarrow g(Q) = e*g_{trong} =$ $\boxed{0.731}$

(c) Correction factor for skew bridge:

* Correction factor of distribution factor for moment (Table 4.6.2.2.2d-1)

Skew angle	θ =	20	Degree.
Factor	c1 =	0.000	
Correction factor	CF(M) =	1.000	

Area of applications
300<= θ <= 600
1100<= S <= 4900
6000 <= L <= 73000
Nb >= 4

$$CF(M) = 1.0 - c1.(\tan\theta)^{1.5}$$
$$c1 = 0.25\left(\frac{Kg}{Lts^3}\right)^{0.25}\left(\frac{S}{L}\right)^{0.5}$$

* Regulation factor of distribution factor for shear force (Table 4.6.2.2.3c-1)

Correction Factor CF(Q)= 1.045

Area of applications
00<= θ <= 600
1100<= S <= 4900
6000 <= L <= 73000
Nb >= 4

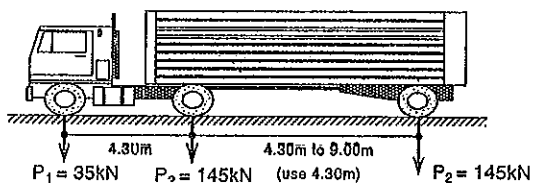
$$CF(Q) = 1.0 + 0.20\left(\frac{Lts^3}{Kg}\right)^{0.3}.\tan\theta$$

(d) Table of Distribution factors for Live load:

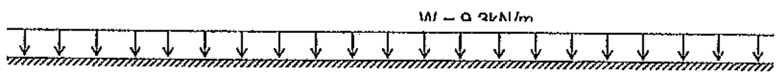
Interior Beam	g(M)	g(Q)	m	m*g(M)	m*g(Q)	m*g(M)*CF(M)	m*g(Q)*CF(Q)
1 lane	0.550	0.696	1.20	0.660	0.835	0.660	0.872
2 or more lanes	0.772	0.852	1.00	0.772	0.852	0.772	0.890
Exterior Beam							
1 lane	0.716	0.716	1.20	0.859	0.859	0.859	0.898
2 or more lanes	0.808	0.731	1.00	0.808	0.731	0.808	0.764

2.2.2 Live Load:

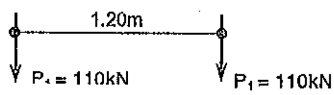
Design Truck



Design Lane Load



Design Tendem



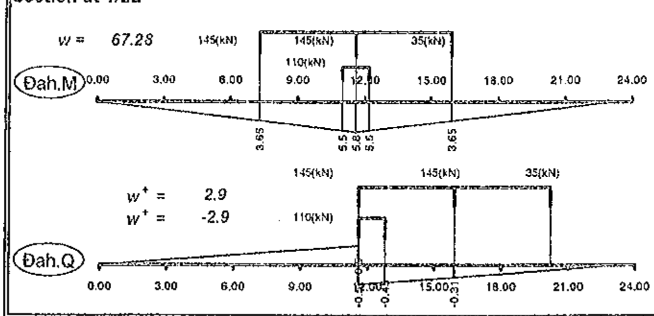
- Truck	P1 =	35.00	(kN)
	P2 =	145.00	(kN)
- Lane load	W =	9.30	(kN)
- Tandem	P1 =	110.00	(kN)
- Pedestrian	PL =	0.00	kN/m^2
- Dynammic load	IM =	0.25	

2.2.3 Internal Force due to Live load:

Design truck or Tendem	Momen	$M_{TR(Ta)} = \Sigma Piyi$	(kNm)
	Shear force	$Q_{TR(Ta)} = \Sigma Piyi$	(kN)
Lane load	Momen	$M_{Ln} = W.F_M$	(kNm)
	Shear force	$Q_{Ln} = W.F_Q$	(kN)
Pedestrian	Momen	$M_{PL} = PL.F_M$	(kNm)
	Shear force	$Q_{PL} = PL.F_Q$	(kN)

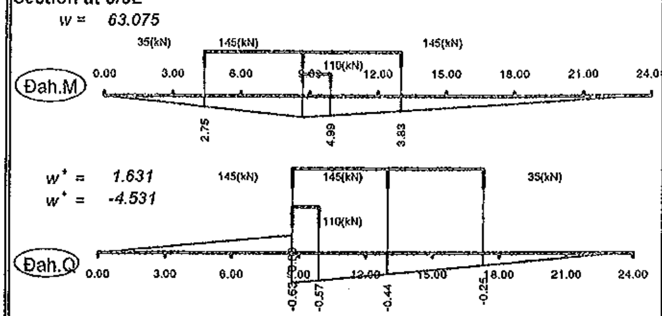
Influence line for Momen & Shear force

Section at 1/2L



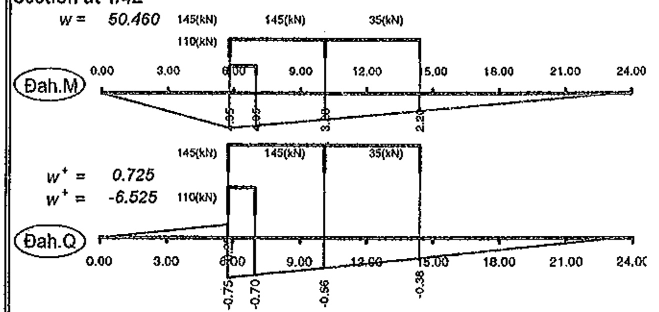
Load	Momen (KN.m)	Shear
Truck	1498.00	122.65
Lane	625.70	26.97
Tandem	1210.00	104.31
Design	**1498.00**	**122.65**
Pedestrian	0.00	0.00

Section at 3/8L



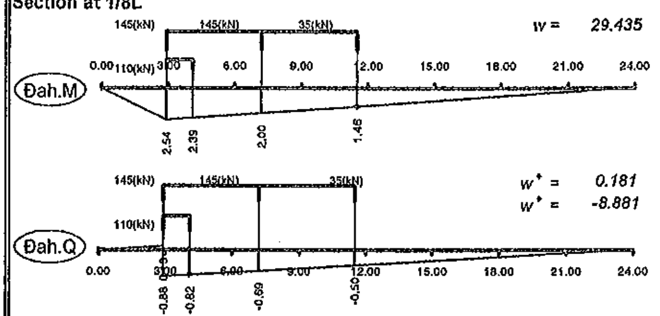
Load	Momen (KN.m)	Shear
Truck	1439.31	163.28
Lane	586.60	42.14
Tandem	1146.75	131.81
Design	**1439.31**	**163.28**
Pedestrian	0.00	0.00

Section at 1/4L



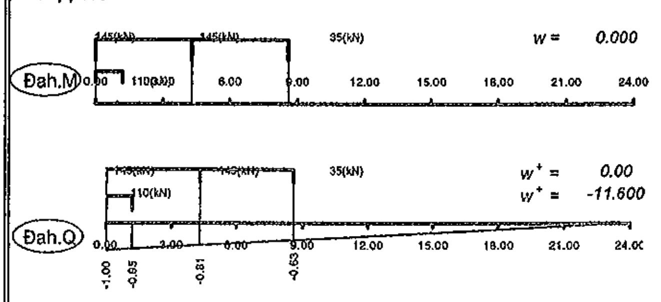
Load	Momen (KN.m)	Shear
Truck	1182.63	203.90
Lane	469.28	60.68
Tandem	924.00	159.31
Design	**1182.63**	**203.90**
Pedestrian	0.00	0.00

Section at 1/8L



Load	Momen (KN.m)	Shear
Truck	709.13	244.53
Lan	273.75	82.60
Tandem	541.75	186.81
Design	**709.13**	**244.53**
Pedestrian	0.00	0.00

At support



Load	Momen (KN.m)	Shear
Truck	0.00	285.15
Lane	0.00	107.88
Tandem	0.00	214.31
Design	**0.00**	**285.15**
Pedestrian	0.00	0.00

Internal Force due to Live load : $M_{(LL+IM)} = m*g(M) * [max(M_{TR}, M_{Ta})*(1+IM) + M_{Ln}]$

$Q_{(LL+IM)} = m*g(Q) * [max(Q_{TR}, Q_{Ta})*(1+IM) + Q_{Ln}]$

Internal Force due to pedestrian :

$M = g(M) * M_{PL}$

$Q = g(Q) * Q_{PL}$

In which:

$M_{TR(Ta)}$ moment due to truck or Tandem

$Q_{TR(Ta)}$ Shear force due to truck or Tandem

yi Value of influence line F Area of influence line

m Lane factor g Distribution factor

Interior	m*g(M)	m*g(Q)
	0.772	0.890
Exterior		
	0.859	0.898

TABLE OF INTERNAL FORCE DUE TO LIVE LOAD

Setion	Xi	Interior Girder		Exterior Girder	
		M	Q	M	Q
	(m)	(kNm)	(kN)	(kNm)	(kN)
Support	0.00	0.00	413.29	0.00	416.82
L/8	2.90	896.00	345.58	996.37	348.54
L/4	5.80	1504.13	280.88	1672.61	283.28
3L/8	8.70	1842.54	219.17	2048.93	221.05
L/2	11.60	1929.40	160.47	2145.52	161.84

2.3 Load combination:

Strength limit state: $U = \eta\ [1.25\ DC + 1.50\ DW + 1.75\ (LL+IM)]$

Service limit state: $U = \eta\ [1.00\ DC + 1.00\ DW + 1.00\ (LL+IM)]$

Fatigue state: $U = 0.75\ (LL+IM)]$

The modify load factort $\eta = \eta_D\ \eta_R\ \eta_I$

STATE	Modify Load Factor			
	η_D	η_R	η_I	$\eta = \eta_D\ \eta_R\ \eta_I$
Strength	1.00	1.00	1.00	1.00
Service	1.00	1.00	1.00	1.00

2.3.1 Load combination - - Interior Girder:

STATE	Strength										
Load	Load Factor	Section									
		Support		L/8		L/4		3L/8		L/2	
	γ	M	Q	M	Q	M	Q	M	Q	M	Q
		(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)
DC	1.25	0.00	563.12	1428.91	422.34	2449.55	281.56	3061.94	140.78	3266.07	0.00
DW	1.50	0.00	77.28	196.10	57.96	336.18	38.64	420.22	19.32	448.24	0.00
LL+IM	1.75	0.00	723.25	1568.00	604.77	2632.22	491.53	3224.44	383.55	3376.45	280.82
Total		**0.00**	**1363.65**	**3193.01**	**1085.07**	**5417.95**	**811.73**	**6706.61**	**543.65**	**7090.75**	**280.82**

STATE	Service										
Load	Load Factor	Section									
		Support		L/8		L/4		3L/8		L/2	
	γ	M	Q	M	Q	M	Q	M	Q	M	Q
		(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)
DC	1.00	0.00	450.49	1143.12	337.87	1959.64	225.25	2449.55	112.62	2612.86	0.00
DW	1.00	0.00	51.52	130.74	38.64	224.12	25.76	280.15	12.88	298.82	0.00
LL+IM	1.00	0.00	413.29	896.00	345.58	1504.13	280.88	1842.54	219.17	1929.40	160.47
Total		**0.00**	**915.30**	**2169.86**	**722.09**	**3687.89**	**531.88**	**4572.24**	**344.68**	**4841.08**	**160.47**

2.3.2 Load combination - Exterior Girder:

STATE	Strength										
Load	Load factor	Section									
		Supprt		L/8		L/4		3L/8		L/2	
	γ	M	Q	M	Q	M	Q	M	Q	M	Q
		(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)
DC	1.25	0.00	563.91	1345.43	422.93	2306.44	281.96	2883.05	140.98	3075.26	0.00
DW	1.50	0.00	77.28	196.10	57.96	336.18	38.64	420.22	19.32	448.24	0.00
LL+IM	1.75	0.00	729.44	1743.64	609.94	2927.07	495.74	3585.62	386.83	3754.65	283.22
Total		**0.00**	**1370.64**	**3285.17**	**1090.84**	**5569.69**	**816.34**	**6888.90**	**547.13**	**7278.15**	**283.22**

STATE	Service										
Load	load factor	Section									
		Support		L/8		L/4		3L/8		L/2	
	γ	M	Q	M	Q	M	Q	M	Q	M	Q
		(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)
DC	1.00	0.00	451.13	1076.34	338.35	1845.15	225.57	2306.44	112.78	2460.21	0.00
DW	1.00	0.00	51.52	130.74	38.64	224.12	25.76	280.15	12.88	298.82	0.00
LL+IM	1.00	0.00	416.82	996.37	348.54	1672.61	283.28	2048.93	221.05	2145.52	161.84
Total		**0.00**	**919.48**	**2203.44**	**725.53**	**3741.88**	**534.61**	**4635.52**	**346.71**	**4904.55**	**161.84**

3. TENDON PROFILE AND PROPERTY OF GIRDER CROSS SECTION

3.1. Tendon profile:

Tendon profile follow Parabol equation:

$$y_i = f - \frac{4.(f-c).x.(l-x)}{l^2}$$

in which: Origin of coordinates in left edge of the Girder bottom (0.0)

- f — Maximum deflection at mid span of tendon
- c — Distance from maximum deflection point to girder bottom
- (x_i, y_i) — Coordination of point under consider i = 1,2...
- L — actual distance between cable ends (X-axis)
- $L_p = X_2 - X_1$ — Distance between 2 point under consider

angle of rotation of tendon for X_i-axis Tan(α) = (4.f (1-2.Xi / L)) / L

α = 2 f / 0.5 L - tan(α)

L_{span} =	24000	(mm)
$L_{su.}$ =	23200	(mm)
L_{cap} =	23700	(mm)

	f =	1210	(mm)	Lcáp =	23700	(mm)	C =	330	(mm)
	Section	Xi	Yi	Lp	ΣLcáp	Tan(α)	(α)	Σα	Section
		(mm)	(mm)	(mm)	(mm)		(rad)	(rad)	
TENDON No 1	Anchorage	0.00	1210.00	0.00	0.00	0.2042	0.0000	0.0000	Anchorage
	Support	250.00	1173.26	250.00	250.00	0.1999	0.0043	0.0043	Support
	L/8	3150.00	804.33	2900.00	3150.00	0.1499	0.0543	0.0586	L/8
	L/4	6050.00	540.82	2900.00	6050.00	0.1000	0.1043	0.1629	L/4
	3L/8	8950.00	382.70	2900.00	8950.00	0.0500	0.1542	0.3171	3L/8
	L/2	11850.00	330.00	2900.00	11850.00	0.0000	0.2042	0.5213	L/2

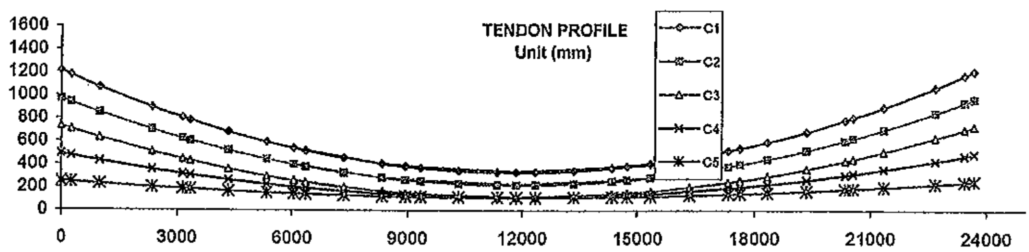
	f =	970	(mm)	Lcáp =	23700	(mm)	C =	220	(mm)
	Section	Xi	Yi	Lp	ΣLcáp	Tan(α)	(α)	Σα	Section
		(mm)	(mm)	(mm)	(mm)		(rad)	(rad)	
TENDON No 2	Anchorage	0.00	970.00	0.00	0.00	0.1637	0.0000	0.0000	Anchorage
	Support	250.00	938.69	250.00	250.00	0.1603	0.0035	0.0035	Support
	L/8	3150.00	624.26	2900.00	3150.00	0.1202	0.0435	0.0470	L/8
	L/4	6050.00	399.67	2900.00	6050.00	0.0801	0.0836	0.1306	L/4
	3L/8	8950.00	264.92	2900.00	8950.00	0.0401	0.1236	0.2542	3L/8
	L/2	11850.00	220.00	2900.00	11850.00	0.0000	0.1637	0.4179	L/2

	f =	730	(mm)	Lcáp =	23700	(mm)	C =	110	(mm)
	Section	Xi	Yi	Lp	ΣLcáp	Tan(α)	(α)	Σα	Section
		(mm)	(mm)	(mm)	(mm)		(rad)	(rad)	
TENDON No 3	Anchorage	0.00	730.00	0.00	0.00	0.1232	0.0000	0.0000	Anchorage
	Support	250.00	704.12	250.00	250.00	0.1206	0.0026	0.0026	Support
	L/8	3150.00	444.19	2900.00	3150.00	0.0905	0.0328	0.0354	L/8
	L/4	6050.00	258.53	2900.00	6050.00	0.0603	0.0629	0.0983	L/4
	3L/8	8950.00	147.13	2900.00	8950.00	0.0302	0.0931	0.1913	3L/8
	L/2	11850.00	110.00	2900.00	11850.00	0.0000	0.1232	0.3145	L/2

	f =	490	(mm)	Lcáp =	23700	(mm)	C =	110	(mm)
	Section	Xi	Yi	Lp	ΣLcáp	Tan(α)	(α)	Σα	Section
		(mm)	(mm)	(mm)	(mm)		(rad)	(rad)	
TENDON No 4	Anchorage	0.00	490.00	0.00	0.0	0.0827	0.0000	0.0000	Anchorage
	Support	250.00	474.14	250.00	250.0	0.0810	0.0017	0.0017	Support
	L/8	3150.00	314.83	2900.00	3150.0	0.0607	0.0220	0.0237	L/8
	L/4	6050.00	201.03	2900.00	6050.0	0.0405	0.0422	0.0660	L/4
	3L/8	8950.00	132.76	2900.00	8950.0	0.0202	0.0625	0.1284	3L/8
	L/2	11850.00	110.00	2900.00	11850.0	0.0000	0.0827	0.2111	L/2

	f =	250	(mm)	Lcáp =	23700	(mm)	C =	110	(mm)
	Section	Xi	Yi	Lp	ΣLcáp	Tan(α)	(α)	Σα	Section
		(mm)	(mm)	(mm)	(mm)		(rad)	(rad)	
TENDON No 5	Anchorage	0.00	250.00	0.00	0.0	0.0422	0.0000	0.0000	Anchorage
	Support	250.00	244.16	250.00	250.0	0.0413	0.0009	0.0009	Support
	L/8	3150.00	185.46	2900.00	3150.0	0.0310	0.0112	0.0121	L/8
	L/4	6050.00	143.54	2900.00	6050.0	0.0207	0.0215	0.0336	L/4
	3L/8	8950.00	118.38	2900.00	8950.0	0.0103	0.0319	0.0655	3L/8
	L/2	11850.00	110.00	2900.00	11850.0	0.0000	0.0422	0.1077	L/2

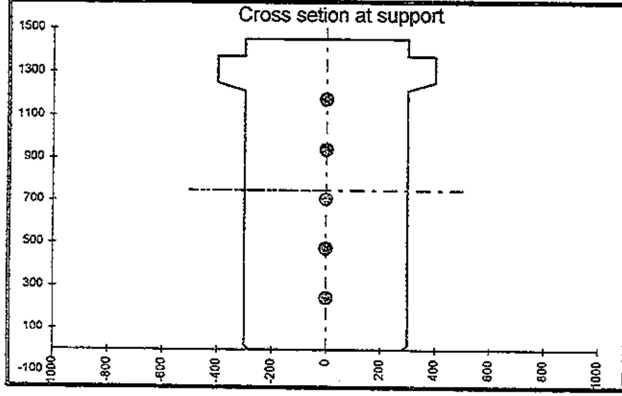
	TENDON No 1		TENDON No 2		TENDON No 3		TENDON No 4		TENDON No 5	
Section	X_i	Y_i	X_i	Y_i	X_i	Y_i	X_i	Y_i	X_i	Y_i
	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)
Anchorage	0.00	1210.00	0.0	970.00	0.0	730.00	0.00	490.00	0.00	250.00
Support	**250.00**	**1173.26**	**250.0**	**938.69**	**250.0**	**704.12**	**250.00**	**474.14**	**250.00**	**244.16**
1	1000.00	1067.74	1000.0	848.76	1000.0	629.77	1000.00	428.57	1000.00	227.37
2	2350.00	895.58	2350.0	702.03	2350.0	508.48	2350.00	354.23	2350.00	199.98
3	3350.00	782.78	3350.0	605.89	3350.0	429.00	3350.00	305.52	3350.00	182.03
L/8	**3150.00**	**804.33**	**3150.0**	**624.26**	**3150.0**	**444.19**	**3150.00**	**314.83**	**3150.00**	**185.46**
4	4350.00	682.51	4350.0	520.43	4350.0	358.36	4350.00	262.22	4350.00	166.08
5	5350.00	594.77	5350.0	445.66	5350.0	296.54	5350.00	224.33	5350.00	152.12
6	6350.00	519.57	6350.0	381.57	6350.0	243.56	6350.00	191.86	6350.00	140.16
7	7350.00	456.90	7350.0	328.16	7350.0	199.41	7350.00	164.80	7350.00	130.19
L/4	**6050.00**	**540.82**	**6050.0**	**399.67**	**6050.0**	**258.53**	**6050.00**	**201.03**	**6050.00**	**143.54**
8	8350.00	406.77	8350.0	285.43	8350.0	164.09	8350.00	143.15	8350.00	122.21
9	9350.00	369.17	9350.0	253.38	9350.0	137.60	9350.00	126.91	9350.00	116.23
10	10350.00	344.10	10350.0	232.02	10350.0	119.93	10350.00	116.09	10350.00	112.24
11	11350.00	331.57	11350.0	221.34	11350.0	111.10	11350.00	110.68	11350.00	110.25
3L/8	**8950.00**	**382.70**	**8950.0**	**264.92**	**8950.0**	**147.13**	**8950.00**	**132.76**	**8950.00**	**118.38**
12	12350.00	331.57	12350.0	221.34	12350.0	111.10	12350.00	110.68	12350.00	110.25
13	13350.00	344.10	13350.0	232.02	13350.0	119.93	13350.00	116.09	13350.00	112.24
14	14350.00	369.17	14350.0	253.38	14350.0	137.60	14350.00	126.91	14350.00	116.23
15	15350.00	406.77	15350.0	285.43	15350.0	164.09	15350.00	143.15	15350.00	122.21
L/2	**11850.00**	**330.00**	**11850.0**	**220.00**	**11850.0**	**110.00**	**11850.00**	**110.00**	**11850.00**	**110.00**
2	8350.00	406.77	8350.0	285.43	8350.0	164.09	8350.00	143.15	8350.00	122.21
3	9350.00	369.17	9350.0	253.38	9350.0	137.60	9350.00	126.91	9350.00	116.23
4	10350.00	344.10	10350.0	232.02	10350.0	119.93	10350.00	116.09	10350.00	112.24
5	11350.00	331.57	11350.0	221.34	11350.0	111.10	11350.00	110.68	11350.00	110.25
-	**14750.00**	**382.70**	**14750.0**	**264.92**	**14750.0**	**147.13**	**14750.00**	**132.76**	**14750.00**	**118.38**
6	12350.00	331.57	12350.0	221.34	12350.0	111.10	12350.00	110.68	12350.00	110.25
7	13350.00	344.10	13350.0	232.02	13350.0	119.93	13350.00	116.09	13350.00	112.24
8	14350.00	369.17	14350.0	253.38	14350.0	137.60	14350.00	126.91	14350.00	116.23
9	15350.00	406.77	15350.0	285.43	15350.0	164.09	15350.00	143.15	15350.00	122.21
-	**17650.00**	**540.82**	**17650.0**	**399.67**	**17650.0**	**258.53**	**17650.00**	**201.03**	**17650.00**	**143.54**
10	16350.00	456.90	16350.0	328.16	16350.0	199.41	16350.00	164.80	16350.00	130.19
11	17350.00	519.57	17350.0	381.57	17350.0	243.56	17350.00	191.86	17350.00	140.16
12	18350.00	594.77	18350.0	445.66	18350.0	296.54	18350.00	224.33	18350.00	152.12
13	19350.00	682.51	19350.0	520.43	19350.0	358.36	19350.00	262.22	19350.00	166.08
-	**20550.00**	**804.33**	**20550.0**	**624.26**	**20550.0**	**444.19**	**20550.00**	**314.83**	**20550.00**	**185.46**
14	20350.00	782.78	20350.0	605.89	20350.0	429.00	20350.00	305.52	20350.00	182.03
14	21350.00	895.58	21350.0	702.03	21350.0	508.48	21350.00	354.23	21350.00	199.98
16	22700.00	1067.74	22700.0	848.76	22700.0	629.77	22700.00	428.57	22700.00	227.37
Support	23450.00	1173.26	23450.0	938.69	23450.0	704.12	23450.00	474.14	23450.00	244.16
Anchorage	23700.00	1210.00	23700.0	970.00	23700.0	730.00	23700.00	490.00	23700.00	250.00



3.2 Property of Girder Cross section at transfer (Stage I: net cross section):

Điểm góc	x (mm)	y (mm)	A (m^2)	Q_X' (m^3)	Q_Y' (m^3)	I_X' (m^4)	I_Y' (m^4)	$I_X'Y'$ (m^4)
Đường bao								
-	0.00	0.00	-	-	-	-	-	-
1	280.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	300.00	20.00	0.01	0.00	0.00	0.00	0.00	0.00
3	300.00	1213.33	0.36	0.44	0.21	0.54	0.10	0.20
4	400.00	1250.00	-0.11	-0.27	-0.08	-0.50	-0.04	-0.14
5	400.00	1370.00	0.05	0.13	0.04	0.25	0.02	0.08
6	300.00	1370.00	0.14	0.38	0.10	0.77	0.05	0.20
7	300.00	1450.00	0.02	0.07	0.01	0.14	0.01	0.03
8	-300.00	1450.00	0.87	2.52	0.00	5.49	0.08	0.00
9	-300.00	1370.00	0.02	0.07	-0.01	0.14	0.01	-0.03
10	-400.00	1370.00	0.14	0.38	-0.10	0.77	0.05	-0.20
11	-400.00	1250.00	0.05	0.13	-0.04	0.25	0.02	-0.08
12	-300.00	1213.33	-0.11	-0.27	0.08	-0.50	-0.04	0.14
13	-300.00	20.00	0.36	0.44	-0.21	0.54	0.10	-0.20
14	-280.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00
15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
16								
17								
18								
19								
20								
21								
Tổng cộng			0.8973	0.6667	0.0000	0.6566	0.0294	0.0000
Cáp								
1	0.00	1173.26						
2	0.00	938.69						
3	0.00	704.12						
4	0.00	474.14						
5	0.00	244.16						
6								
7								

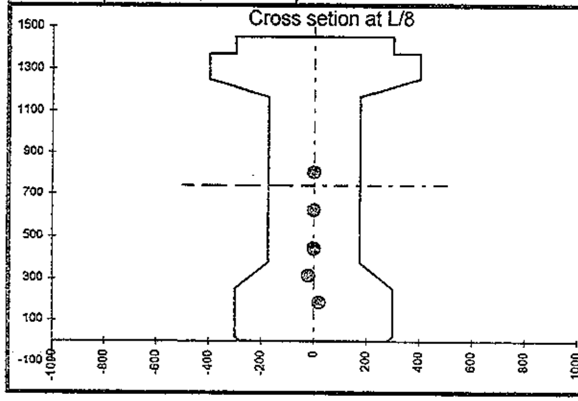
Diện tích 1 bó cáp	As =	888.30 (mm2)
Số bó cáp	n=	5.00 bó
	Σ As Yi =	3139567.77 (mm3)
	Σ As =	4441.50 (mm2)
K. cách từ trọng tâm bó cáp đến TTH =		36.19 (mm)



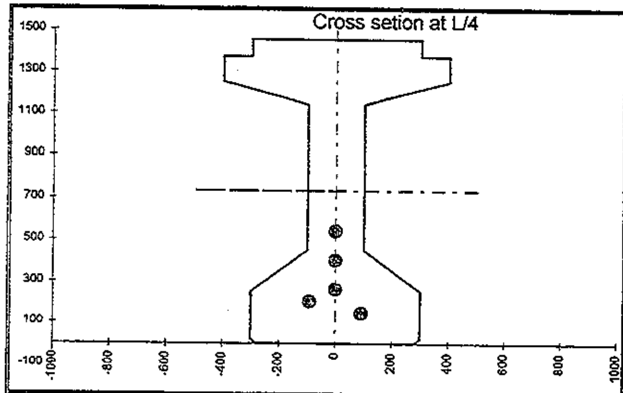
A	0.8973	m^2
e_x	0.0000	m
e_y	0.7431	m
I_x	0.1611	m^4
I_y	0.0294	m^4
I_{xy}	0.0000	m^4
y_b	0.7431	m
y_t	0.7069	m
S_b	0.2169	m^3
S_t	0.2279	m^3
$E_{cáp (Y)}$	0.0362	m
q =	21.5344	(KN/m)

Điểm góc	x (mm)	y (mm)	A (m^2)	Q_X' (m^3)	Q_Y' (m^3)	I_X' (m^4)	I_Y' (m^4)	$I_X'Y'$ (m^4)
Đường bao								
-	0.00	0.00	-	-	-	-	-	-
1	280.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	300.00	20.00	0.01	0.00	0.00	0.00	0.00	0.00
3	300.00	250.00	0.07	0.02	0.04	0.00	0.02	0.01
4	173.64	376.36	0.07	0.04	0.03	0.02	0.01	0.02
5	173.64	1167.00	0.14	0.21	0.05	0.27	0.01	0.06
6	400.00	1250.00	-0.25	-0.60	-0.14	-1.09	-0.06	-0.26
7	400.00	1370.00	0.05	0.13	0.04	0.25	0.02	0.08
8	300.00	1370.00	0.14	0.38	0.10	0.77	0.05	0.20
9	300.00	1450.00	0.02	0.07	0.01	0.14	0.01	0.03
10	-300.00	1450.00	0.87	2.52	0.00	5.49	0.08	0.00
11	-300.00	1370.00	0.02	0.07	-0.01	0.14	0.01	-0.03
12	-400.00	1370.00	0.14	0.38	-0.10	0.77	0.05	-0.20
13	-400.00	1250.00	0.05	0.13	-0.04	0.25	0.02	-0.08
14	-173.64	1167.00	-0.25	-0.60	0.14	-1.09	-0.06	0.26
15	-173.64	376.36	0.14	0.21	-0.05	0.27	0.01	-0.06
16	-300.00	250.00	0.07	0.04	-0.03	0.02	0.01	-0.02
17	-300.00	20.00	0.07	0.02	-0.04	0.00	0.02	-0.01
18	-280.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00
19	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
20								
21								
Tổng cộng			0.6756	0.5003	0.0000	0.5172	0.0165	0.0000
Cáp								
1	0.00	804.33						
2	0.00	624.26						
3	0.00	444.19						
4	-20.00	314.83						
5	20.00	185.46						
6								
7								

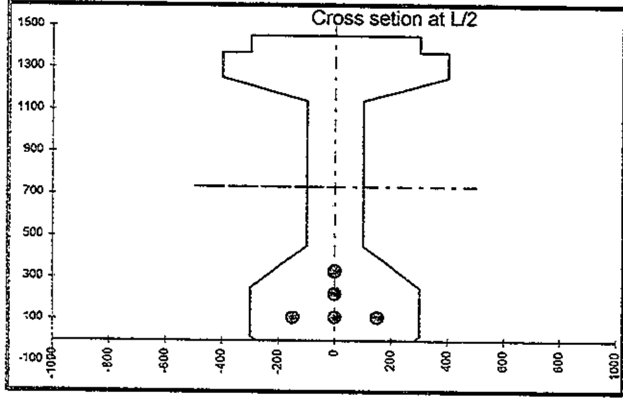
Diện tích 1 bó cáp	As =	888.30 (mm2)
Số bó cáp	n=	5.00 bó
	Σ As Yi =	2108002.37 (mm3)
	Σ As =	4441.50 (mm2)
K. cách từ trọng tâm bó cáp đến TTH =		265.84 (mm)



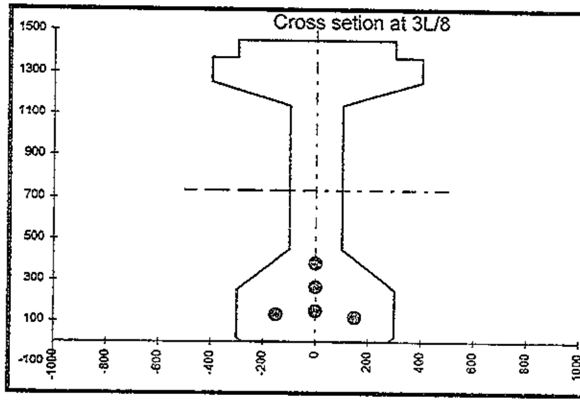
A	0.6756	m^2
e_x	0.0000	m
e_y	0.7405	m
I_x	0.1467	m^4
I_y	0.0165	m^4
I_{xy}	0.0000	m^4
y_b	0.7405	m
y_t	0.7095	m
S_b	0.1982	m^3
S_t	0.2068	m^3
$E_{cáp (Y)}$	0.2658	m
q =	16.2151	(KN/m)



A	0.5666	m^2
e_x	0.0000	m
e_y	0.7323	m
I_x	0.1415	m^4
I_y	0.0144	m^4
I_{xy}	0.0000	m^4
y_b	0.7323	m
y_t	0.7177	m
S_b	0.1932	m^3
S_t	0.1971	m^3
$E_{cáp\ (Y)}$	0.4235	m
q =	13.5984	(KN/m)



A	0.5666	m^2
e_x	0.0000	m
e_y	0.7323	m
I_x	0.1415	m^4
I_y	0.0144	m^4
I_{xy}	0.0000	m^4
y_b	0.7323	m
y_t	0.7177	m
S_b	0.1932	m^3
S_t	0.1971	m^3
$E_{cap\ (Y)}$	0.5563	m
q =	13.5984	(KN/m)



A	0.5666	m^2
e_x	0.0000	m
e_y	0.7323	m
I_x	0.1415	m^4
I_y	0.0144	m^4
I_{xy}	0.0000	m^4
y_b	0.7323	m
y_t	0.7177	m
S_b	0.1932	m^3
S_t	0.1971	m^3
$E_{cáp\ (Y)}$	0.5231	m
q =	13.5984	(KN/m)

Uniform load due to sefl weigh of Girder in Stage 1: Q = 16.51 (KN/m)

3.3. Property of Girder cross section in service stage (stage II: Composite cross section) :

3.3.1. Effective flange width (4.6.2.6)

Modular Ratio: Deck Concrete/Girder Concrete n = Eb / Ed = 0.88

For Interior Girder:

$$b_I = \min\begin{cases}1/4L_{tt}\\ 12h_f + \max(0.5b, b_w)\\ S\end{cases} \quad \Rightarrow n^* b_I = 2248.88861 \text{ (mm)}$$

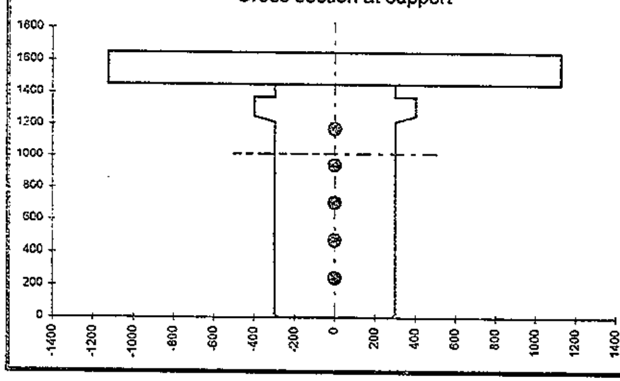
For Exterior Girder:

$$b_E = 0.5b_I + \min\begin{cases}1/8L_{tt}\\ 6h_f + \max(0.5b_w, 0.25b)\\ L_h\end{cases} \quad \Rightarrow n^* b_E = 2248.88861 \text{ (mm)}$$

3.3.2. Property of Girder cross section in stage II (service stage):

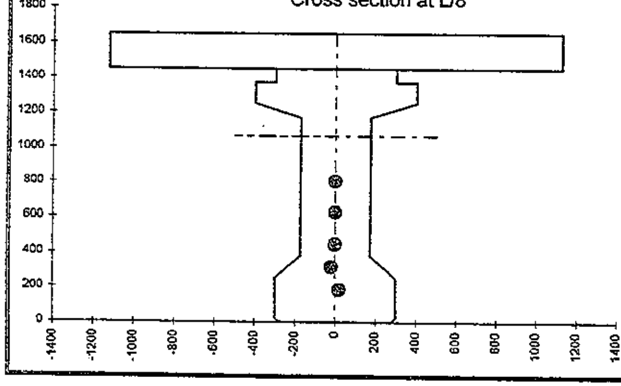
Interior Girder:

Cross section at support



A	1.3470	m^2
e_x	0.0000	m
e_y	1.0125	m
I_x	0.3577	m^4
I_y	0.2190	m^4
I_{xy}	0.0000	m^4
y_b	1.0125	m
y_t	0.6375	m
S_b	0.3533	m^3
S_t	0.5611	m^3
$E_{cáp\,(Y)}$	0.3056	m
q =	32.3291	(KN/m)

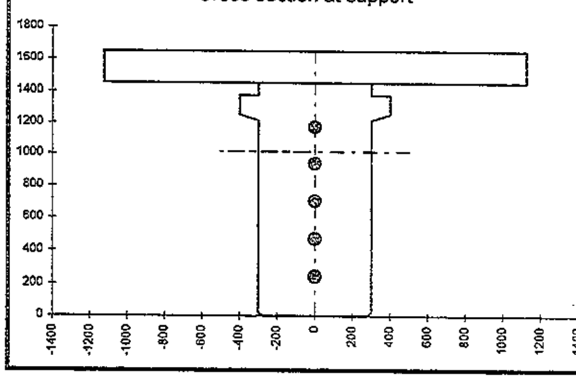
Cross section at L/8



A	1.1254	m^2
e_x	0.0000	m
e_y	1.0640	m
I_x	0.3252	m^4
I_y	0.2061	m^4
I_{xy}	0.0000	m^4
y_b	1.0640	m
y_t	0.5860	m
S_b	0.3056	m^3
S_t	0.5550	m^3
$E_{cáp\,(Y)}$	0.5894	m
q =	27.0098	(KN/m)

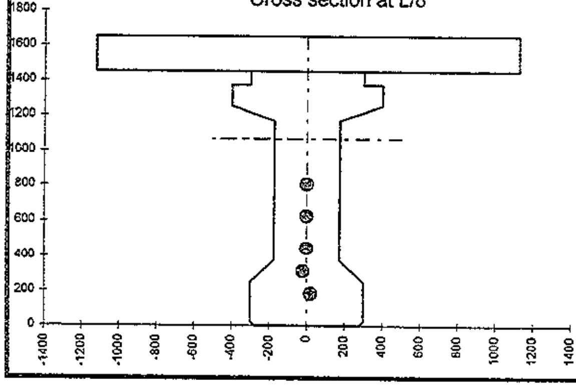
Exterior Girder:

Cross section at support



A	1.3470	m^2
e_x	0.0000	m
e_y	1.0125	m
I_x	0.3577	m^4
I_y	0.2190	m^4
I_{xy}	0.0000	m^4
y_b	1.0125	m
y_t	0.6375	m
S_b	0.3533	m^3
S_t	0.5611	m^3
$E_{cáp\,(Y)}$	0.3056	m
q =	33.0026	(KN/m)

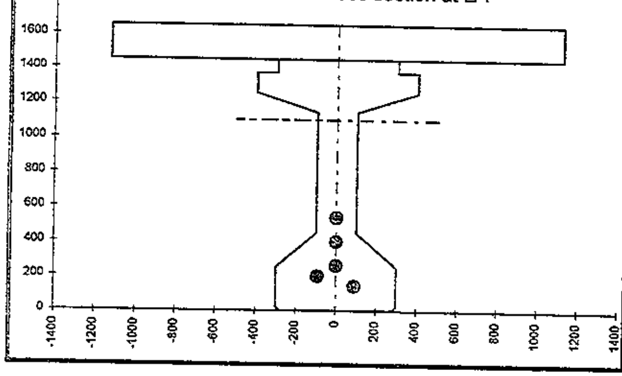
Cross section at L/8



A	1.1254	m^2
e_x	0.0000	m
e_y	1.0640	m
I_x	0.3252	m^4
I_y	0.2061	m^4
I_{xy}	0.0000	m^4
y_b	1.0640	m
y_t	0.5860	m
S_b	0.3056	m^3
S_t	0.5550	m^3
$E_{cáp\,(Y)}$	0.5894	m
q =	27.5725	(KN/m)

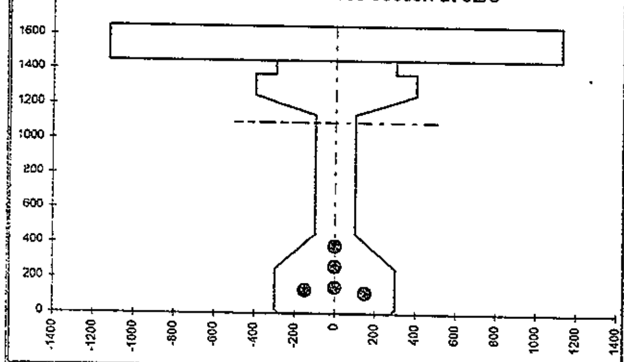
Interior Girder:

Cross section at L/4



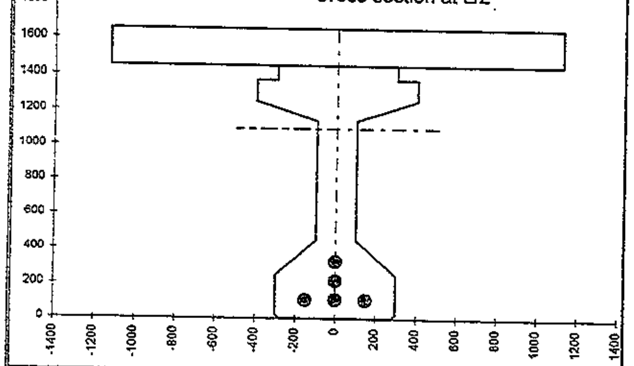
A	1.0164	m^2
e_x	0.0000	m
e_y	1.0941	m
I_x	0.3107	m^4
I_y	0.2039	m^4
I_{xy}	0.0000	m^4
y_b	1.0941	m
y_t	0.5559	m
S_b	0.2839	m^3
S_t	0.5589	m^3
$E_{cáp\ (Y)}$	0.7854	m
q =	24.3931	(KN/m)

Cross section at 3L/8



A	1.0164	m^2
e_x	0.0000	m
e_y	1.0941	m
I_x	0.3107	m^4
I_y	0.2039	m^4
I_{xy}	0.0000	m^4
y_b	1.0941	m
y_t	0.5559	m
S_b	0.2839	m^3
S_t	0.5589	m^3
$E_{cáp\ (Y)}$	0.8850	m
q =	24.3931	(KN/m)

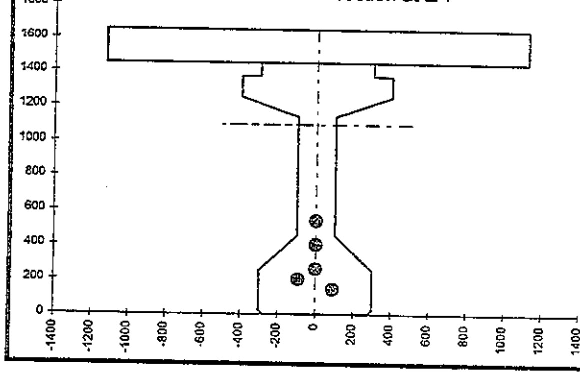
Cross section at L/2



A	1.0164	m^2
e_x	0.0000	m
e_y	1.0941	m
I_x	0.3107	m^4
I_y	0.2039	m^4
I_{xy}	0.0000	m^4
y_b	1.0941	m
y_t	0.5559	m
S_b	0.2839	m^3
S_t	0.5589	m^3
$E_{cáp\ (Y)}$	0.9181	m
q =	24.3931	(KN/m)

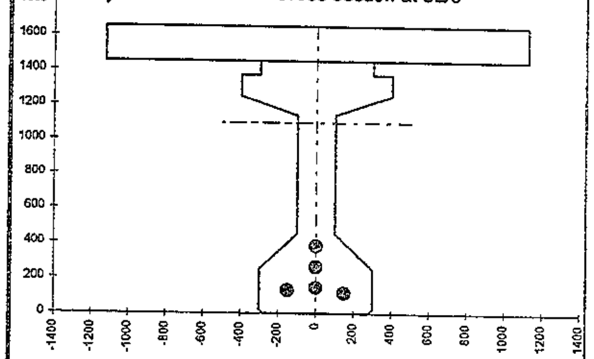
Exterior Girder:

Cross section at L/4



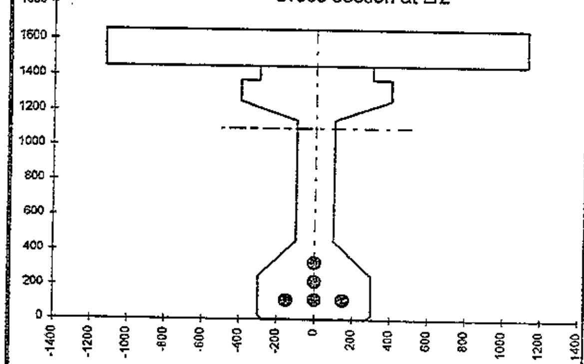
A	1.0164	m^2
e_x	0.0000	m
e_y	1.0941	m
I_x	0.3107	m^4
I_y	0.2039	m^4
I_{xy}	0.0000	m^4
y_b	1.0941	m
y_t	0.5559	m
S_b	0.2839	m^3
S_t	0.5589	m^3
$E_{cáp\ (Y)}$	0.7854	m
q =	24.9013	(KN/m)

Cross section at 3L/8



A	1.0164	m^2
e_x	0.0000	m
e_y	1.0941	m
I_x	0.3107	m^4
I_y	0.2039	m^4
I_{xy}	0.0000	m^4
y_b	1.0941	m
y_t	0.5559	m
S_b	0.2839	m^3
S_t	0.5589	m^3
$E_{cáp\ (Y)}$	0.8850	m
q =	24.9013	(KN/m)

Cross section at L/2



A	1.0164	m^2
e_x	0.0000	m
e_y	1.0941	m
I_x	0.3107	m^4
I_y	0.2039	m^4
I_{xy}	0.0000	m^4
y_b	1.0941	m
y_t	0.5559	m
S_b	0.2839	m^3
S_t	0.5589	m^3
$E_{cáp\ (Y)}$	0.9181	m
q =	0.0000	(KN/m)

prestressing force immediately (Instantaneous losses):

4.1.1 Friction between Prestressing Tendon and Duck:

Formula:

$$\Delta f_{pF} = f_{pj}\,(1-e^{-(Kx+\mu\alpha)}) \qquad (5.9.5.2.2)$$

Xi: Length of tendon from the jacking end to any point under consideration

Section		Tendon no. 1		Tendon no. 2		Tendon no. 3		Tendon no. 4		Tendon no. 5		SUM/5
	X_i	$\Sigma\alpha$	Δf_{pF}	$\Sigma\alpha$	Δf_{pF}	$\Sigma\alpha$	Δf_{pF}	$\Sigma\alpha$	Δf_{pF}	$\Sigma\alpha$	Δf_{pF}	$\Sigma\Delta f_{pF}$
	(mm)	(rad)	(MPa)	(rad)	(MPa)	(rad)	(MPa)	(rad)	(MPa)	(rad)	(MPa)	(MPa)
Ancho.	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.00
Support	250.00	0.0043	1.62	0.0035	1.34	0.0026	1.06	0.0017	0.78	0.0009	0.50	1.06
L/8	3150.00	0.0586	21.60	0.0470	17.87	0.0354	14.14	0.0237	10.39	0.0121	6.63	14.13
L/4	6050.00	0.1629	56.93	0.1306	46.83	0.0983	36.65	0.0660	26.39	0.0336	16.05	36.57
3L/8	8950.00	0.3171	106.31	0.2542	87.37	0.1913	68.12	0.1284	48.56	0.0655	28.70	67.81
L/2	11850.00	0.5213	168.00	0.4179	138.30	0.3145	107.83	0.2111	76.56	0.1077	44.47	107.03

4.1.2 Anchorage seating or Set:

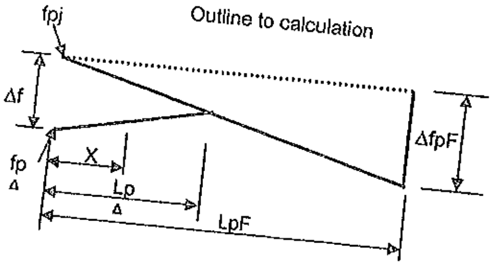
Formula

$$L_{pA} = \sqrt{\frac{E(\Delta L)L_{pF}}{\Delta f_{pF}}}$$

$$\Delta f = \frac{2\Delta f_{pF} L_{pA}}{L_{pF}}$$

$$\Delta f_{pA} = \Delta f\left(1 - \frac{X}{L_{pA}}\right)$$

Outline to calculation



Trong đó:

- L_{pA} Effective length due to anchorage set
- E Cable modulus of elasticity
- ΔL Setting length
- L_{pF} The length from anchorage to point that loss stress due to friction was known
- Δf_{pF} The loss stress value at the point that the leng from anchorage ti it is L_{pF}
- Δf The loss stress value at Anchorage

Choice the length from anchorage to point that loss stress due to friction was known (LpF) and calculation follow:

Tendon no.1		Xi (mm)	Δf_{pA} (MPa)
LpF =	11850	0	258.90
Δf_{pF} =	168.00	250	251.81
LpA =	9130.9	3150	169.58
Δf =	258.90	6050	87.36
		8950	5.13
		11850	0.00

Tendon no.2		Xi (mm)	Δf_{pA} (MPa)
LpF =	11850	0	234.91
Δf_{pF} =	138.30	250	229.07
LpA =	10063.5	3150	161.38
Δf =	234.91	6050	93.69
		8950	25.99
		11850	0.00

Tendon no.3		Xi (mm)	Δf_{pA} (MPa)
LpF =	11850	0	207.42
Δf_{pF} =	107.83	250	202.87
LpA =	11397.2	3150	150.09
Δf =	207.42	6050	97.31
		8950	44.54
		11850	0.00

Tendon no.4		Xi (mm)	Δf_{pA} (MPa)
LpF =	11850	0	153.11
Δf_{pF} =	76.56	250	149.88
LpA =	11850.0	3150	112.41
Δf =	153.11	6050	74.94
		8950	37.47
		11850	0.00

Tendon no.5		Xi (mm)	Δf_{pA} (MPa)
LpF =	11850	0	88.93
Δf_{pF} =	44.47	250	87.06
LpA =	11850.0	3150	65.29
Δf =	88.93	6050	43.53
		8950	21.76
		11850	0.00

$$\Delta f_{ES} = \frac{N-1}{2N}\frac{E_p}{E_{ci}} f_{cgp} \quad (5.9.5.2.3b\text{-}1)$$

Number of tendon	N =	5.00 (Tendon)
Cable modulus of elasticity	Ep =	197000.0 MPa
Concrete strength at transfer	f'_{ci} =	40.50 MPa
Unit weight of concrete	γ_c =	2450.00 kg/m3
Concrete modulus of elasticity at transfer	E_{ci} =	33185.3 MPa

Total stress of concrete in the Tendon centroid (f_{cgp}) due to prestressing force and self weigh of girder

$$f_{cgp} = \frac{F_j}{A} + \frac{F_j e^2}{I_X} - \frac{M_{DC} e}{I_X}$$

Compression force due to pretressing consider loss stress:

$F_j = N^*f_{pj}{}^*A_s - A_s{}^*\Sigma(\Delta f_{pFi} + \Delta f_{pAi})$

- A — Area of girder cross section
- Ix — Inertia Moment of Girder cross section
- e — Distance from tendon centroid to neutral line of girder section
- M_{DC} — Maximum moment due to self weigh of girder at jacking

Total loss stress due to friction and Anchorage:

Section	Xi	Tendon1	Tendon2	Tendon3	Tendon4	Tendon5	SUM	ΣFj
		ΔfpF+ΔfpA	ΔfpF+ΔfpA	ΔfpF+ΔfpA	ΔfpF+ΔfpA	ΔfpF+ΔfpA	(ΔfpF+ΔfpA)	
	(mm)	(MPa)	(MPa)	(MPa)	(MPa)	(MPa)	(MPa)	(kN)
Anchorage	0	258.90	234.91	207.42	153.11	88.93	943.27	4944.92
Support	250	253.43	230.41	203.93	150.67	87.56	926.00	4960.27
L/8	3150	191.18	179.25	164.23	122.80	71.92	729.39	5134.92
L/4	6050	144.28	140.52	133.97	101.34	59.58	579.69	5267.90
3L/8	8950	111.44	113.36	112.65	86.03	50.46	473.94	5361.83
L/2	11850	168.00	138.30	107.83	76.56	44.47	535.16	5307.45

Loss stress due to Elastic deformation of concrete

Section	Xi	Fj	A	Ix	e	M_{DC}	f_{cgp}	Δf_{ES}
	(mm)	(kN)	(mm2)	(mm4)	(mm)	kNm	(MPa)	(MPa)
Anchorage	0	4944.92	9.0E+05	1.6E+11	36.19	0.00	5.55	**13.18**
Support	250	4960.27	9.0E+05	1.6E+11	36.19	0.00	5.57	**13.22**
L/8	3150	5134.92	6.8E+05	1.5E+11	265.84	485.93	9.19	**21.83**
L/4	6050	5267.90	5.7E+05	1.4E+11	423.53	833.02	13.48	**32.01**
3L/8	8950	5361.83	5.7E+05	1.4E+11	523.07	1041.28	15.98	**37.95**
L/2	11850	5307.45	5.7E+05	1.4E+11	556.25	1110.70	16.61	**39.43**

Total loss of prestressing force immediately - Remaining prestressing force:

Tendon1 Section	Xi	Δf_{pF}	Δf_{pA}	Δf_{ES}	ΣΔ	F_j^1	(α)	F_j^1*Cos(α)	F_j^1*Sin(α)
	(mm)	(MPa)	(MPa)	(MPa)	(MPa)	(kN)	(rad)	(kN)	(kN)
anchorage	0	0.00	258.90	13.18	272.08	914.87	0.2014	896.37	183.06
Support	250	1.62	251.81	13.22	266.65	919.70	0.1973	901.86	180.29
L/8	3150	21.60	169.58	21.83	213.01	967.35	0.1488	956.66	143.43
L/4	6050	56.93	87.36	32.01	176.30	999.96	0.0996	995.00	99.46
3L/8	8950	106.31	5.13	37.95	149.39	1023.86	0.0499	1022.59	51.11
L/2	11850	168.00	0.00	39.43	207.43	972.30	0.0000	972.30	0.00

Tendon2 Section	Xi	Δf_{pF}	Δf_{pA}	Δf_{ES}	ΣΔ	F_j^2	(α)	F_j^2*Cos(α)	F_j^2*Sin(α)
	(mm)	(MPa)	(MPa)	(MPa)	(MPa)	(kN)	(rad)	(kN)	(kN)
anchorage	0	0.00	234.91	12.36	247.27	936.92	0.1623	924.61	151.37
Support	250	1.34	229.07	12.40	242.81	940.88	0.1589	929.03	148.89
L/8	3150	17.87	161.38	20.46	199.72	979.16	0.1196	972.16	116.85
L/4	6050	46.83	93.69	30.01	170.53	1005.08	0.0800	1001.87	80.28
3L/8	8950	87.37	25.99	35.58	148.93	1024.27	0.0400	1023.45	41.00
L/2	11850	138.30	0.00	36.97	175.27	1000.87	0.0000	1000.87	0.00

Tendon3 Section	Xi	Δf_{pF}	Δf_{pA}	Δf_{ES}	ΣΔ	F_j^3	(α)	F_j^3*Cos(α)	F_j^3*Sin(α)
	(mm)	(MPa)	(MPa)	(MPa)	(MPa)	(kN)	(rad)	(kN)	(kN)
anchorage	0	0.00	207.42	10.98	218.40	962.56	0.1226	955.33	117.70
Support	250	1.06	202.87	11.02	214.95	965.63	0.1200	958.68	115.62
L/8	3150	14.14	150.09	18.19	182.42	994.52	0.0902	990.48	89.59
L/4	6050	36.65	97.31	26.68	160.65	1013.86	0.0602	1012.03	61.03
3L/8	8950	68.12	44.54	31.62	144.28	1028.41	0.0301	1027.94	30.99
L/2	11850	107.83	0.00	32.86	140.69	1031.59	0.0000	1031.59	0.00

Tendon4 Section	Xi	Δf_{pF}	Δf_{pA}	Δf_{ES}	ΣΔ	F_j^4	(α)	F_j^4*Cos(α)	F_j^4*Sin(α)
	(mm)	(MPa)	(MPa)	(MPa)	(MPa)	(kN)	(rad)	(kN)	(kN)
anchorage	0	0.00	153.11	8.24	161.35	1013.24	0.0825	1009.79	83.51
Support	250	0.78	149.88	8.26	158.93	1015.39	0.0808	1012.08	81.93
L/8	3150	10.39	112.41	13.64	136.44	1035.36	0.0606	1033.46	62.75
L/4	6050	26.39	74.94	20.01	121.35	1048.78	0.0405	1047.92	42.42
3L/8	8950	48.56	37.47	23.72	109.75	1059.08	0.0202	1058.86	21.43
L/2	11850	76.56	0.00	24.65	101.20	1066.67	0.0000	1066.67	0.00

Tendon5 Section	Xi (mm)	Δf_{pF} (MPa)	Δf_{pA} (MPa)	Δf_{ES} (MPa)	$\Sigma\Delta$ (MPa)	F_j^5 (kN)	(α) (rad)	$F_j^5 \cdot Cos(\alpha)$ (kN)	$F_j^5 \cdot Sin(\alpha)$ (kN)
anchorage	0	0.00	88.93	0.00	88.93	1077.57	0.0422	1076.61	45.43
Support	250	0.50	87.06	0.00	87.56	1078.79	0.0413	1077.87	44.52
L/8	3150	6.63	65.29	0.00	71.92	1092.68	0.0310	1092.15	33.83
L/4	6050	16.05	43.53	0.00	59.58	1103.64	0.0206	1103.41	22.79
3L/8	8950	28.70	21.76	0.00	50.46	1111.74	0.0103	1111.68	11.48
L/2	11850	44.47	0.00	0.00	44.47	1117.07	0.0000	1117.07	0.00

SUM 1to5 Section	Xi (mm)	ΣF_j (kN)	$Fj \cdot Cos(\alpha)$ (kN)	$Fj \cdot Sin(\alpha)$ (kN)	$e_{cáp}$ (mm)	$M_l = \Sigma F_j Cos(\alpha) \cdot e_{cáp}$ (kNm)
anchorage	0	4905.16	4862.72	581.07	36.19	175.99
Support	250	4920.38	4879.51	571.25	36.19	176.60
L/8	3150	5069.07	5044.91	446.46	265.84	1341.13
L/4	6050	5171.33	5160.23	305.97	423.53	2185.53
3L/8	8950	5247.36	5244.51	156.01	523.07	2743.26
L/2	11850	5188.50	5188.50	0.00	556.25	2886.12

4.2. Loss of prestressing force at service stage (time - dependent losses):

4.2.1 Loss of prestress due to Shrinkage:

Formula:

$\Delta fp_{SH} = (93 - 0.85 \cdot H)$

Relative humidity of environment $H =$ 80.00 %

$\boxed{\Delta fp_{SH} = \textbf{25.00 (MPa)}}$

4.2.2 Loss of prestress due to Creep:

Formula

In which:

$\Delta fp_{CR} = 12.0\, f_{cgp} - 7.0 \cdot \Delta f_{cdp}$

f_{cgp} Stress in concrete at tendons centroid (f_{cgp}) due to prestressing tendon and self weigh of girder

Δf_{cdp} Stess at tendons centroid changes due to permanent load, except dead load action at transfer

Section	Xi	f_{cgp}	Interior Girder		Exterior Girder	
			Δf_{cdp}	Δfp_{CR}	Δf_{cdp}	Δfp_{CR}
	(m)	(MPa)	(MPa)	**(MPa)**	(MPa)	**(MPa)**
Support	0.00	5.57	0.00	**66.82**	0.00	**66.82**
L/8	2.90	9.19	0.69	**105.48**	1.31	**101.17**
L/4	5.80	13.48	3.83	**134.99**	3.48	**137.40**
3L/8	8.70	15.98	4.10	**163.07**	5.22	**155.25**
L/2	11.60	16.61	6.48	**153.94**	5.87	**158.16**

4.2.3 Loss of prestress due to Relaxation:

(a) At transfer:

Formula:

$$\Delta f_{pR1} = \frac{\log(24t)}{40}\left[\frac{f_{pj}}{f_{py}} - 0{,}55\right] f_{pj}$$

In which:

t : time estimated in days from stressing to transfer $t =$ 0.00 days

fpy : Specified yeild strength of prestressing steel $f_{py} =$ 1674.00 (MPa)

fpj : Initial stress in the tendon at the end of stressing

Section	Xi (m)	fpj (MPa)	Δfp_{R1} **(MPa)**
Support	0.00	1288.78	**0.00**
L/8	2.90	1280.17	**0.00**
L/4	5.80	1269.99	**0.00**
3L/8	8.70	1264.05	**0.00**
L/2	11.60	1262.57	**0.00**

(b) After Transfer:

Formula:

$$\Delta fp_{R2} = 30\% \cdot (138 - 0.3\, \Delta fp_F - 0.4\, \Delta f_{ES} - 0.2\, (\Delta fp_{SH} + \Delta fp_{CR}))$$

Section	Interior Girder					
	Xi (m)	Δf_{pF} (MPa)	Δf_{pES} (MPa)	Δfp_{SH} (MPa)	Δfp_{CR} (MPa)	Δfp_{R2} **(MPa)**
Support	0.00	1.06	13.22	25.00	13.36	**37.42**
L/8	2.90	14.13	21.83	25.00	21.10	**34.74**
L/4	5.80	36.57	32.01	25.00	27.00	**31.15**
3L/8	8.70	67.81	37.95	25.00	32.61	**27.29**
L/2	11.60	107.03	39.43	25.00	30.79	**23.69**

Section	Xi	Δf_{pF}	Δf_{pES}	Δfp_{SH}	Δfp_{CR}	Δfp_{R2}
	(m)	(MPa)	(MPa)	(MPa)	(MPa)	(MPa)
Support	0.00	1.06	13.22	25.00	13.36	**37.42**
L/8	2.90	14.13	21.83	25.00	20.23	**34.80**
L/4	5.80	36.57	32.01	25.00	27.48	**31.12**
3L/8	8.70	67.81	37.95	25.00	31.05	**27.38**
L/2	11.60	107.03	39.43	25.00	31.63	**23.64**

TOTAL LOSS STRESS AT SERVICE STAGE

Setion	Interior Girder					
	Xi	Δfp_{SH}	Δfp_{CR}	Δfp_{R1}	Δfp_{R2}	Sum
	(m)	(MPa)	(MPa)	(MPa)	(MPa)	(MPa)
Support	0.00	25.00	66.82	0.00	37.42	**129.24**
L/8	2.90	25.00	105.48	0.00	34.74	**165.23**
L/4	5.80	25.00	134.99	0.00	31.15	**191.14**
3L/8	8.70	25.00	163.07	0.00	27.29	**215.36**
L/2	11.60	25.00	153.94	0.00	23.69	**202.63**

Section	Exterior Girder					
	Xi	Δfp_{SH}	Δfp_{CR}	Δfp_{R1}	Δfp_{R2}	Sum
	(m)	(MPa)	(MPa)	(MPa)	(MPa)	(MPa)
Support	0.00	25.00	66.82	0.00	37.42	**129.24**
L/8	2.90	25.00	101.17	0.00	34.80	**160.96**
L/4	5.80	25.00	137.40	0.00	31.12	**193.52**
3L/8	8.70	25.00	155.25	0.00	27.38	**207.63**
L/2	11.60	25.00	158.16	0.00	23.64	**206.80**

4.3. Total Prestressing force consider loss in the service stage:
Interior Girder

Tendon1 Section	Xi	$\Sigma\Delta_{pT}$	F_j^1	(α)	$Fj^1 * Cos(\alpha)$	$Fj^1 * Sin(\alpha)$
	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	395.89	804.90	0.1973	789.28	157.79
L/8	2.90	378.24	820.58	0.1488	811.51	121.67
L/4	5.80	367.44	830.17	0.0996	826.06	82.57
3L/8	8.70	364.75	832.56	0.0499	831.52	41.56
L/2	11.60	410.06	792.31	0.0000	792.31	0.00

Tendon2 Section	Xi	$\Sigma\Delta_{pT}$	F_j^2	(α)	$Fj^2 * Cos(\alpha)$	$Fj^2 * Sin(\alpha)$
	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	372.04	826.08	0.1623	815.23	133.46
L/8	2.90	364.94	832.39	0.1589	821.90	131.72
L/4	5.80	361.67	835.30	0.1196	829.33	99.68
3L/8	8.70	364.29	832.96	0.0800	830.30	66.53
L/2	11.60	377.90	820.88	0.0400	820.22	32.86

Tendon3 Section	Xi	$\Sigma\Delta_{pT}$	F_j^3	(α)	$Fj^3 * Cos(\alpha)$	$Fj^3 * Sin(\alpha)$
	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	347.64	847.76	0.1200	841.66	101.51
L/8	2.90	380.18	818.86	0.0902	815.53	73.77
L/4	5.80	373.56	824.73	0.0602	823.24	49.64
3L/8	8.70	376.01	822.56	0.0301	822.19	24.79
L/2	11.60	346.90	848.41	0.0000	848.41	0.00

Tendon4 Section	XI	$\Sigma\Delta_{pT}$	F_j^4	(α)	$Fj^4 * Cos(\alpha)$	$Fj^4 * Sin(\alpha)$
	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	290.59	898.43	0.0808	895.50	72.50
L/8	2.90	324.16	868.62	0.0606	867.02	52.64
L/4	5.80	327.58	865.57	0.0405	864.87	35.01
3L/8	8.70	336.70	857.47	0.0202	857.30	17.35
L/2	11.60	312.38	879.08	0.0000	879.08	0.00

Tendon5 Section	Xi (mm)	$\Sigma\Delta_{pT}$ (MPa)	F_j^5 (kN)	(α) (rad)	$F_j^5 \cdot Cos(\alpha)$ (kN)	$F_j^5 \cdot Sin(\alpha)$ (kN)
Support	0.00	218.17	962.77	0.0413	961.95	39.73
L/8	2.90	252.79	932.02	0.0310	931.57	28.86
L/4	5.80	263.06	922.89	0.0206	922.69	19.06
3L/8	8.70	274.94	912.34	0.0103	912.29	9.42
L/2	11.60	253.09	931.75	0.0000	931.75	0.00

SUM 1to5 Section	Xi (mm)	ΣF_j (kN)	$F_j \cdot Cos(\alpha)$ (kN)	$V_p = F_j \cdot Sin(\alpha)$ (kN)	e_{cable} (m)	$M_j = \Sigma F_j Cos(\alpha) \cdot e_{cdp}$ (kNm)
Support	0.00	4339.93	4303.61	504.99	0.04	155.8
L/8	2.90	4272.45	4247.52	408.66	0.27	1129.2
L/4	5.80	4278.66	4266.18	285.96	0.42	1806.9
3L/8	8.70	4257.89	4253.59	159.65	0.52	2224.9
L/2	11.60	4272.43	4271.78	32.86	0.56	2376.2

Exterior Girder

Tendon1 Section	Xi (mm)	$\Sigma\Delta_{pT}$ (MPa)	F_j^1 (kN)	(α) (rad)	$F_j^1 \cdot Cos(\alpha)$ (kN)	$F_j^1 \cdot Sin(\alpha)$ (kN)
Support	0.00	395.89	804.90	0.1973	789.28	157.79
L/8	2.90	373.97	824.37	0.1488	815.25	122.23
L/4	5.80	369.82	828.06	0.0996	823.95	82.36
3L/8	8.70	357.02	839.42	0.0499	838.38	41.90
L/2	11.60	414.24	788.60	0.0000	788.60	0.00

Tendon2 Section	Xi (mm)	$\Sigma\Delta_{pT}$ (MPa)	F_j^2 (kN)	(α) (rad)	$F_j^2 \cdot Cos(\alpha)$ (kN)	$F_j^2 \cdot Sin(\alpha)$ (kN)
Support	0.00	372.04	826.08	0.1623	815.23	133.46
L/8	2.90	360.68	836.18	0.1589	825.64	132.32
L/4	5.80	364.05	833.18	0.1196	827.23	99.43
3L/8	8.70	356.56	839.83	0.0800	837.15	67.08
L/2	11.60	382.07	817.17	0.0400	816.51	32.71

Tendon3 Section	Xi (mm)	$\Sigma\Delta_{pT}$ (MPa)	F_j^3 (kN)	(α) (rad)	$F_j^3 \cdot Cos(\alpha)$ (kN)	$F_j^3 \cdot Sin(\alpha)$ (kN)
Support	0.00	347.64	847.76	0.1200	841.66	101.51
L/8	2.90	375.91	822.64	0.0902	819.30	74.11
L/4	5.80	375.94	822.62	0.0602	821.13	49.52
3L/8	8.70	368.28	829.43	0.0301	829.05	25.00
L/2	11.60	351.08	844.70	0.0000	844.70	0.00

Tendon4 Section	Xi (mm)	$\Sigma\Delta_{pT}$ (MPa)	F_j^4 (kN)	(α) (rad)	$F_j^4 \cdot Cos(\alpha)$ (kN)	$F_j^4 \cdot Sin(\alpha)$ (kN)
Support	0.00	290.59	898.43	0.0808	895.50	72.50
L/8	2.90	319.89	872.41	0.0606	870.80	52.87
L/4	5.80	329.96	863.46	0.0405	862.75	34.92
3L/8	8.70	328.98	864.34	0.0202	864.16	17.49
L/2	11.60	316.55	875.37	0.0000	875.37	0.00

Tendon5 Section	Xi (mm)	$\Sigma\Delta_{pT}$ (MPa)	F_j^5 (kN)	(α) (rad)	$F_j^5 \cdot Cos(\alpha)$ (kN)	$F_j^5 \cdot Sin(\alpha)$ (kN)
Support	0.00	218.17	962.77	0.0413	961.95	39.73
L/8	2.90	248.52	935.80	0.0310	935.36	28.98
L/4	5.80	265.44	920.78	0.0206	920.58	19.01
3L/8	8.70	267.21	919.20	0.0103	919.16	9.49
L/2	11.60	257.26	928.04	0.0000	928.04	0.00

SUM 1to5 Section	Xi (mm)	ΣF_j (kN)	$F_j \cdot Cos(\alpha)$ (kN)	$V_p = F_j \cdot Sin(\alpha)$ (kN)	e_{cable} (m)	$M_j = \Sigma F_j Cos(\alpha) \cdot e_{cdp}$ (kNm)
Support	0.00	4339.93	4303.61	504.99	0.04	155.8
L/8	2.90	4291.40	4266.35	410.51	0.27	1134.2
L/4	5.80	4268.10	4255.64	285.24	0.42	1802.4
3L/8	8.70	4292.22	4287.89	160.96	0.52	2242.9
L/2	11.60	4253.89	4253.23	32.71	0.56	2365.9

5. FIBRE STRESS CHECK:

Formula:

Top fibre: $f_{ti} = \dfrac{F_i}{A} - \dfrac{F_i e}{S_t} + \dfrac{M_{DC}}{S_t}$ Bottom fibre $f_{bi} = \dfrac{F_i}{A} + \dfrac{F_i e}{S_b} - \dfrac{M_{DC}}{S_b}$

Note (+) : Compression tresses ; (-) Tension stresses

Concrete strength at transfer	fci' = 0.9 fc =	40.50 MPa
Comprresion stress Limit at transfer	0.6 fci' =	24.30 MPa
Tension stress Limit at transfer	0.25 SQRT(fci') < 1.38 =	-1.38 MPa (5.9.4.1.2-1)

Setion	Xi	A	St	Sb	Fj'*Cos(α)	e	M_{DC}	f_{ti}	f_{bi}	Kiểm tra	
	(mm)	(mm2)	(mm3)	(mm3)	(kN)	(mm)	(kNm)	(MPa)	(MPa)	f_{ti}	f_{bi}
irder en	0	8.97E+05	2.28E+08	2.17E+08	4862.72	36.19	0.00	4.65	6.23	OK	OK
Support	250	8.97E+05	2.28E+08	2.17E+08	4879.51	36.19	0.00	4.66	6.25	OK	OK
L/8	3150	6.76E+05	2.07E+08	1.98E+08	5044.91	265.84	485.93	3.33	11.78	OK	OK
L/4	6050	5.67E+05	1.97E+08	1.93E+08	5160.23	423.53	833.02	2.25	16.11	OK	OK
3L/8	8950	5.67E+05	1.97E+08	1.93E+08	5244.51	523.07	1041.28	0.62	18.06	OK	OK
L/2	11850	5.67E+05	1.97E+08	1.93E+08	5188.50	556.25	1110.70	0.15	18.34	OK	OK

5.2 Stress check during contruction the deck:

5.2.1 Increase load:

Exterior Diaphragms beam	DC_{dn1} =	44.93 (kN)
Interior Diaphragms beam	DC_{dn1} =	20.97 (kN)
Precast plank	DC_{VK} =	3.74 (kN/m)
Wet concrete of deck	DC_{mc} =	12.94 (kN/m)

5.2.2 Stress check:

Compression strength of concrete	fc =	45.00 MPa	
Compression stress limit	0.45 fc =	20.25 MPa	(5.9.4.2.1-1)
Tension stress limit	0.5 SQRT(fc) =	-3.35 MPa	(5.9.4.2.2-1)

Setion	Xi	A	St	Sb	Fi	e	M_{DC}	f_{ti}	f_{bi}	Kiểm tra	
	(mm)	(mm2)	(mm3)	(mm3)	(kN)	(mm)	(kNm)	(MPa)	(MPa)	f_{ti}	f_{bi}
irder en	0	8.97E+05	2.28E+08	2.17E+08	4862.72	36.19	0.00	4.65	6.23	OK	OK
Support	250	8.97E+05	2.28E+08	2.17E+08	4879.51	36.19	0.00	4.66	6.25	OK	OK
L/8	3150	6.76E+05	2.07E+08	1.98E+08	5044.91	265.84	1003.60	5.83	9.17	OK	OK
L/4	6050	5.67E+05	1.97E+08	1.93E+08	5160.23	423.53	1720.46	6.75	11.51	OK	OK
3L/8	8950	5.67E+05	1.97E+08	1.93E+08	5244.51	523.07	2150.58	6.25	12.32	OK	OK
L/2	11850	5.67E+05	1.97E+08	1.03E+08	5180.50	556.25	2293.95	6.15	12.22	OK	OK

5.3 Stress check at the top fibre of Girder - Service state :

5.3.1 Due to prestressing tendon and self weigh of girder - Service limit stage I:

Compression Stress Limit:	0.45 fc =	20.25 MPa	(5.9.4.2.1-1)
Tension Stress Limit:	- 0.5*SQRT(fc) =	-3.35 MPa	

$$f_f = \frac{P_{pe}}{A} - \frac{P_{pe} e_c}{S_t} + \frac{M_g + M_s}{S_t} + \frac{M_{SDL}}{S_{tg}}$$

Interior Girder

Setion	Xi	A	S_t	S_{tg}	P_{pe}	P_{pe}*e_c	$M_g + M_s$	M_{SDL}	f_t	Check
	(mm)	(mm2)	(mm3)	(mm3)	(kN)	(kNm)	(kNm)	(kNm)	(MPa)	ft
Support	250	8.97E+05	2.28E+08	8.49E+08	4303.61	155.76	0.00	0.00	4.113	OK
L/8	3150	6.76E+05	2.07E+08	8.79E+08	4247.52	1129.16	1003.60	270.26	5.987	OK
L/4	6050	5.67E+05	1.97E+08	9.14E+08	4266.18	1806.87	1720.46	463.30	7.598	OK
3L/8	8950	5.67E+05	1.97E+08	9.14E+08	4253.59	2224.94	2150.58	579.12	7.764	OK
L/2	11850	5.67E+05	1.97E+08	9.14E+08	4271.78	2376.18	2293.95	617.73	7.798	OK

Exterior Girder

Setion	Xi	A	S_t	S_{tg}	P_{pe}	P_{pe}*e_c	$M_g + M_s$	M_{SDL}	f_t	Check
	(mm)	(mm2)	(mm3)	(mm3)	(kN)	(kNm)	(kNm)	(kNm)	(MPa)	f_t
Support	250	8.97E+05	2.28E+08	8.49E+08	4303.61	155.76	0.00	0.00	4.113	OK
L/8	3150	6.76E+05	2.07E+08	8.79E+08	4266.35	1134.16	935.20	271.88	5.662	OK
L/4	6050	5.67E+05	1.97E+08	9.14E+08	4255.64	1802.41	1603.20	466.07	7.010	OK
3L/8	8950	5.67E+05	1.97E+08	9.14E+08	4287.89	2242.88	2004.00	582.59	6.993	OK
L/2	11850	5.67E+05	1.97E+08	9.14E+08	4253.23	2365.87	2137.60	621.43	7.029	OK

5.3.2 Due to 1/2 (Prestressing tendon + self weigh of girder) and Live load - Service limit stage I:

Compression Stress Limit: 0.40 fc = 18.00 MPa (5.9.4.2.1-1)

Tension Stress Limit: - 0.5*SQRT(fc) = -3.35 MPa

$$f_t = 0.5\left(\frac{P_{pe}}{A} - \frac{P_{pe}e_c}{S_t} + \frac{M_g + M_s}{S_t} + \frac{M_{SDL}}{S_{tg}}\right) + \frac{M_{LL}}{S_{tg}}$$

Interior Girder

Setion	Xi	A	S_t	S_{tg}	P_{pe}	P_{pe}*e_c	M_g + M_s	M_{SDL}	M_{LL}	f_t	Check
	(mm)	(mm2)	(mm3)	(mm3)	(kN)	(kNm)	(kNm)	(kNm)	(kNm)	(MPa)	f_t
Support	250	8.97E+05	2.28E+08	8.49E+08	4303.61	155.76	0.00	0.00	0.00	2.057	OK
L/8	3150	6.76E+05	2.07E+08	8.79E+08	4247.52	1129.16	1003.60	270.26	896.00	4.013	OK
L/4	6050	5.67E+05	1.97E+08	9.14E+08	4266.18	1806.87	1720.46	463.30	1504.13	5.444	OK
3L/8	8950	5.67E+05	1.97E+08	9.14E+08	4253.59	2224.94	2150.58	579.12	1842.54	5.897	OK
L/2	11850	5.67E+05	1.97E+08	9.14E+08	4271.78	2376.18	2293.95	617.73	1929.40	6.010	OK

Exterior Girder

Setion	Xi	A	S_t	S_{tg}	P_{pe}	P_{pe}*e_c	M_g + M_s	M_{SDL}	M_{LL}	f_t	Check
	(mm)	(mm2)	(mm3)	(mm3)	(kN)	(kNm)	(kNm)	(kNm)	(kNm)	(MPa)	f_t
Support	250	897266.7	2.28E+08	8.49E+08	4303.61	155.76	0.00	0.00	0.00	2.057	OK
L/8	3150	675628.7	2.07E+08	8.79E+08	4266.35	1134.16	935.20	271.88	996.37	3.965	OK
L/4	6050	566600.0	1.97E+08	9.14E+08	4255.64	1802.41	1603.20	466.07	1672.61	5.335	OK
3L/8	8950	566600.0	1.97E+08	9.14E+08	4287.89	2242.88	2004.00	582.59	2048.93	5.738	OK
L/2	11850	566600.0	1.97E+08	9.14E+08	4253.23	2365.87	2137.60	621.43	2145.52	5.861	OK

5.3.3 Due to prestressing tendon + self weigh of girder + live load - Service limit stage I:

Compression Stress Limit: 0.60 fc = 27.00 MPa (5.9.4.2.1-1)

Tension Stress Limit: - 0.5*SQRT(fc) = -3.35 MPa

$$f_t = \left(\frac{P_{pe}}{A} - \frac{P_{pe}e_c}{S_t} + \frac{M_g + M_s}{S_t} + \frac{M_{SDL}}{S_{tg}}\right) + \frac{M_{LL}}{S_{tg}}$$

Interior Girder

Setion	Xi	A	S_t	S_{tg}	P_{pe}	P_{pe}*e_c	M_g + M_s	M_{SDL}	M_{LL}	f_t	Check
	(mm)	(mm2)	(mm3)	(mm3)	(kN)	(kNm)	(kNm)	(kNm)	(kNm)	(MPa)	f_t
Support	250	8.97E+05	2.28E+08	8.49E+08	4303.61	155.76	0.00	0.00	0.00	4.113	OK
L/8	3150	6.76E+05	2.07E+08	8.79E+08	4247.52	1129.16	1003.60	270.26	896.00	7.007	OK
L/4	6050	5.67E+05	1.97E+08	9.14E+08	4266.18	1806.87	1720.46	463.30	1504.13	9.243	OK
3L/8	8950	5.67E+05	1.97E+08	9.14E+08	4253.59	2224.94	2150.58	579.12	1842.54	9.779	OK
L/2	11850	5.67E+05	1.97E+08	9.14E+08	4271.78	2376.18	2293.95	617.73	1929.40	9.909	OK

Exterior Girder

Setion	Xi	A	S_t	S_{tg}	P_{pe}	P_{pe}*e_c	M_g + M_s	M_{SDL}	M_{LL}	f_t	Check
	(mm)	(mm2)	(mm3)	(mm3)	(kN)	(kNm)	(kNm)	(kNm)	(kNm)	(MPa)	f_t
Support	250	8.97E+05	2.28E+08	8.49E+08	4303.61	155.76	0.00	0.00	0.00	4.113	OK
L/8	3150	6.76E+05	2.07E+08	8.79E+08	4266.35	1134.16	935.20	271.88	996.37	6.796	OK
L/4	6050	5.67E+05	1.97E+08	9.14E+08	4255.64	1802.41	1603.20	466.07	1672.61	8.840	OK
3L/8	8950	5.67E+05	1.97E+08	9.14E+08	4287.89	2242.88	2004.00	582.59	2048.93	9.235	OK
L/2	11850	5.67E+05	1.97E+08	9.14E+08	4253.23	2365.87	2137.60	621.43	2145.52	9.376	OK

5.4 Stress check at the top fibre of deck - Service state:

5.4.1 Due to additional load (dead load part 2) - Service limit stage I:

Compression Stress Limit: 0.45 fc = 15.75 MPa (5.9.4.2.1-1)

$$f_t = \frac{M_{SDL}}{S_{tc}}$$

Setion	Xi	MSDL (kNm)		S_{tc} (mm3)		f_t (MPa)		Check	
	(mm)	in.Girder	Ex.Girder	in.Girder	Ex.Girder	in.Girder	Ex.Girder	in.Girder	Ex.Girder
Support	250.00	0.00	0.00	4.9E+08	4.95E+08	0.000	0.000	OK	OK
L/8	3150.00	270.26	271.88	4.9E+08	4.89E+08	0.552	0.555	OK	OK
L/4	6050.00	463.30	466.07	4.9E+08	4.93E+08	0.940	0.946	OK	OK
3L/8	8950.00	579.12	582.59	4.9E+08	4.93E+08	1.175	1.182	OK	OK
L/2	11850.00	617.73	621.43	4.9E+08	4.93E+08	1.253	1.261	OK	OK

5.4.2 Due to additional load (dead load part 2) and live load - Service limit stage I:

Compression Stress Limit: 0.6 fc = 21.00 MPa (5.9.4.2.1-1)

$$f_{tc} = \frac{M_{SDL} + M_{LL}}{S_{tc}}$$

Setion	Xi	MSDL + MLL (kNm)		S_{tc} (mm3)		f_t (MPa)		Check	
	(mm)	in.Girder	Ex.Girder	in.Girder	Ex.Girder	in.Girder	Ex.Girder	in.Girder	Ex.Girder
Support	250.00	0.00	0.00	4.9E+08	4.95E+08	0.000	0.000	OK	OK
L/8	3150.00	1166.26	1268.24	4.9E+08	4.89E+08	2.383	2.591	OK	OK
L/4	6050.00	1967.43	2138.68	4.9E+08	4.93E+08	3.992	4.339	OK	OK
3L/8	8950.00	2421.66	2631.52	4.9E+08	4.93E+08	4.913	5.339	OK	OK
L/2	11850.00	2547.13	2766.95	4.9E+08	4.93E+08	5.168	5.614	OK	OK

5.5 Stress check at the bottom fibre of girder - Service III (Stage III):

Compression Stress Limit: 0.45 fc = 27.00 MPa (5.9.4.2.1-1)
Tension Stress Limit: - 0.5*SQRT(fc) = -3.35 MPa (5.9.4.2.1-1)

$$f_b = \frac{P_{pe}}{A} + \frac{P_{pe}e_c}{S_b} - \frac{M_g + M_s}{S_b} - \frac{M_{SDL} + 0.8M_{LL}}{S_{bc}}$$

Interior Girder

Setion	Xi	A	S_b	S_{bc}	P_{pe}	P_{pe}*e_c	$M_g + M_s$	M_{SDL}	M_{LL}	f_b	Check
	(mm)	(mm2)	(mm3)	(mm3)	(kN)	(kNm)	(kNm)	(kNm)	(kNm)	(MPa)	f_b
Support	250	8.97E+05	2.17E+08	3.53E+08	4303.61	155.76	0.00	0.00	0.00	5.515	OK
L/8	3150	6.76E+05	1.98E+08	3.06E+08	4247.52	1129.16	1003.60	270.26	896.00	3.691	OK
L/4	6050	5.67E+05	1.93E+08	2.84E+08	4266.18	1806.87	1720.46	463.30	1504.13	2.107	OK
3L/8	8950	5.67E+05	1.93E+08	2.84E+08	4253.59	2224.94	2150.58	579.12	1842.54	0.661	OK
L/2	11850	5.67E+05	1.93E+08	2.84E+08	4271.78	2376.18	2293.95	617.73	1929.40	0.353	OK

Exterior Girder

Setion	Xi	A	S_b	S_{bc}	P_{pe}	P_{pe}*e_c	$M_g + M_s$	M_{SDL}	M_{LL}	f_b	Check
	(mm)	(mm2)	(mm3)	(mm3)	(kN)	(kNm)	(kNm)	(kNm)	(kNm)	(MPa)	f_b
Support	250	8.97E+05	2.17E+08	3.53E+08	4303.61	155.76	0.00	0.00	0.00	5.515	OK
L/8	3150	6.76E+05	1.98E+08	3.06E+08	4266.35	1134.16	935.20	271.88	996.37	3.821	OK
L/4	6050	5.67E+05	1.93E+08	2.84E+08	4255.64	1802.41	1603.20	466.07	1672.61	2.188	OK
3L/8	8950	5.67E+05	1.93E+08	2.84E+08	4287.89	2242.88	2004.00	582.59	2048.93	0.979	OK
L/2	11850	5.67E+05	1.93E+08	2.84E+08	4253.23	2365.87	2137.60	621.43	2145.52	0.454	OK

5.6 Stress check at the bottom fibre of girder - Service I (Stage III):

Compression Stress Limit: 0.45 fc = 27.00 MPa (5.9.4.2.1-1)
Tension Stress Limit: - 0.5*SQRT(fc) = -3.35 MPa (5.9.4.2.1-1)

$$f_b = \frac{P_{pe}}{A} + \frac{P_{pe}e_c}{S_b} - \frac{M_g + M_s}{S_b} - \frac{M_{SDL} + M_{LL}}{S_{bc}}$$

Interior Girder

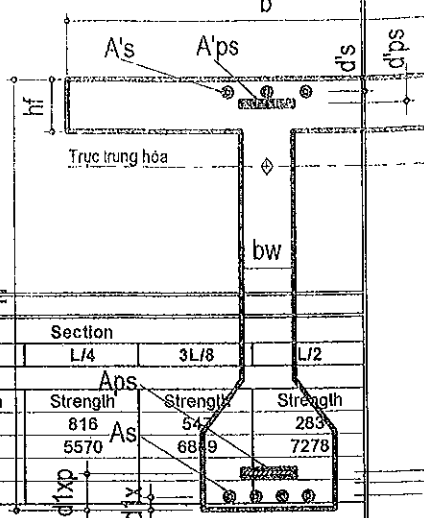
Setion	Xi	A	S_b	S_{bc}	P_{pe}	P_{pe}*e_c	$M_g + M_s$	M_{SDL}	M_{LL}	f_b	Check
	(mm)	(mm2)	(mm3)	(mm3)	(kN)	(kNm)	(kNm)	(kNm)	(kNm)	(MPa)	f_b
Support	250	8.97E+05	2.17E+08	3.53E+08	4303.61	155.76	0.00	0.00	0.00	5.515	OK
L/8	3150	6.76E+05	1.98E+08	3.06E+08	4247.52	1129.16	1003.60	270.26	896.00	3.105	OK
L/4	6050	5.67E+05	1.93E+08	2.84E+08	4266.18	1806.87	1720.46	463.30	1504.13	1.048	OK
3L/8	8950	5.67E+05	1.93E+08	2.84E+08	4253.59	2224.94	2150.58	579.12	1842.54	-0.637	OK
L/2	11850	5.67E+05	1.93E+08	2.84E+08	4271.78	2376.18	2293.95	617.73	1929.40	-1.006	OK

Exterior Girder

Setion	Xi	A	S_b	S_{bc}	P_{pe}	P_{pe}*e_c	$M_g + M_s$	M_{SDL}	M_{LL}	f_b	Check
	(mm)	(mm2)	(mm3)	(mm3)	(kN)	(kNm)	(kNm)	(kNm)	(kNm)	(MPa)	f_b
Support	250	8.97E+05	2.17E+08	3.53E+08	4303.61	155.76	0.00	0.00	0.00	5.515	OK
L/8	3150	6.76E+05	1.98E+08	3.06E+08	4266.35	1134.16	935.20	271.88	996.37	3.169	OK
L/4	6050	5.67E+05	1.93E+08	2.84E+08	4255.64	1802.41	1603.20	466.07	1672.61	1.010	OK
3L/8	8950	5.67E+05	1.93E+08	2.84E+08	4287.89	2242.88	2004.00	582.59	2048.93	-0.464	OK
L/2	11850	5.67E+05	1.93E+08	2.84E+08	4253.23	2365.87	2137.60	621.43	2145.52	-1.057	OK

REINFORCEMENT OF GIRDER CHECKING - STRENGTH LOAD COMBINATION

	MATERIALS		
	NORMAL CONCRETE		
f'c	Compressive Strength of concrete at 28 days	Mpa	45
Ec	Modulus of Elasticity	Mpa	33915
fr	Modulus of Rupture	Mpa	4.2
gc	Unit weight of concrete	kN/m3	24.0
	PRESTRESSING STEEL		
fpu	Tensile strength of prestressing steel	Mpa	1860
fpy	Yield strength of prestressing steel	Mpa	1674
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: Support	L/8	L/4	3L/8	L/2
	INTERNAL FORCES AT SECTION							
	Combination			Strength	Strength	Strength	Strength	Strength
Qu	Shear		kN	1371	1091	816	547	283
Mu	Flexural Moment		kNm	0	3285	5570	68[illegible]9	7278
Nu	Axial load		kN					
Tu	Torsional Moment		kNm					
	6.1 FLEXURAL MOMENT CHECKING							
H	Section height		m	1.650	1.650	1.650	1.650	1.650
d's	Dis. From comp. fiber to centroid of comp. Reinf		m	0.062	0.062	0.062	0.062	0.062
d1x	Dis. From tens. fiber to centroid of tension Reinf		m	0.061	0.061	0.061	0.061	0.061
	Cover to reinf		m	0.040	0.040	0.040	0.040	0.040
ds	Dis. From comp. fiber to centroid of tension Reinf		m	1.589	1.589	1.589	1.589	1.589
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	0.707	0.475	0.309	0.209	0.176
dps	Dis. From comp. fiber to centroid of tension prestressing steel		m	0.943	1.175	1.341	1.441	1.474
b	Width of the compression face of member		m	2.249	2.249	2.249	2.249	2.249
bw	Web width or diameter of a circular section		m	0.600	0.347	0.200	0.200	0.200
hf	Compression flange depth		m	0.200	0.200	0.200	0.200	0.200
Iz	Moment of inertia of section		m4	0.358	0.325	0.311	0.311	0.311
Amc	Section area		m2	1.347	1.125	1.016	1.016	1.016
	Steel choice							
Aps	Tension prestressing steel	P.S type		9 T12.7	9 T12.7	9 T12.7	9 T12.7	9 T12.7
		Number	tendons	5	5	5	5	5
		Area	m2	0.00444	0.00444	0.00444	0.00444	0.00444
A'ps	Compression prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	0 T00.0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
As	Tension Reinforcement	Number	bars	6	6	6	6	6
		Diameter	mm	16	16	16	16	16
		Area	m2	0.00121	0.00121	0.00121	0.00121	0.00121
A's	Compression Reinforcement	Number	bars	4	4	4	4	4
		Diameter	mm	12	12	12	12	12
		Area	m2	0.00045	0.00045	0.00045	0.00045	0.00045
A'c	Shear reinforcement	Number	bars	2	2	2	2	2
		Diameter	mm	14	14	14	14	14
		Area	m2	0.00030	0.00030	0.00030	0.00030	0.00030
f	Resistance factors for flexure		5.5.4.2	0.90	0.90	0.90	0.90	0.90
fv	Resistance factors for shear			0.90	0.90	0.90	0.90	0.90
fn	Resistance factors for axial force			1.00	1.00	1.00	1.00	1.00
b1	Stress block factor			0.729	0.729	0.729	0.729	0.729
c	Dis. Between centroid and top fiber		m	0.132	0.133	0.133	0.133	0.133
	For T section behavior		m	-0.033	-0.175	-0.391	-0.398	-0.400
	For rectangular section behavior		m	0.132	0.133	0.133	0.133	0.133
fpe	Effective stress in the prestressing steel after losses		Mpa	805	819	825	823	792
fps	Aver. stress in pres. steel at the time for which the nominal resistance		Mpa	1787	1801	1808	1812	1813
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	0.096	0.097	0.097	0.097	0.097
de	Corresp. effective depth from extreme comp. fiber to centroid of tensile force in the tensile reinf.	m	0.980	1.199	1.355	1.449	1.481
Mn	Nominal resistance	kNm	7373	9214	10540	11361	11635
Mr	Factored reistance	kNm	6635	8292	9486	10225	10471
Mu	Flexual moment	kNm	0	3285	5570	6889	7278
(5.7.3.2)	**Flexural moment Checking**		OK	OK	OK	OK	OK
	Limits for reinforcement						
c/de	Maximum reinforcement		0.13	0.11	0.10	0.09	0.09
	Maximum reinforcement Checking	<= 0.42	OK	OK	OK	OK	OK
r min	Mimimum reinforcement		0.09%	0.11%	0.12%	0.12%	0.12%
	Mimimum reinforcement Checking for RC	0.34%	N.a	N.a	N.a	N.a	N.a
1.2*Mcr	Craking moment	kNm	1195	1087	1039	1039	1039
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**		OK	OK	OK	OK	OK
(5.8.3.5)	Tensile force in steel should be satisfied - F_yc	kN	1494	4262	5489	5834	5831
	Checking As.fy+Aps.fps >= F_yc		Ok	Ok	Ok	Ok	Ok
(5.7.3.4)	**Conctrol of craking by distr. of reinf for RC member- Check?**		No	No	No	No	No
	Existing condition for structrure	1,2 or 3	1	1	1	1	1
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.058	0.058	0.058	0.058	0.058
Z	Crack width parameter	N/mm	30000	30000	30000	30000	30000
A	Area of concr. with same centroid as tens. Reinf	m2	0.043	0.043	0.043	0.043	0.043
fsa	Value	Mpa	220	220	220	220	220
0.6*fy		Mpa	240	240	240	240	240
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	220	220	220	220	220
x	Dist. From compression fiber to centroid	m	-	-	-	-	-
J.d	Arm	m	-	-	-	-	-
Icr	Moment of inertia of the cracked section	m4	-	-	-	-	-
fs	Tensile stress in reinforcement fs = Msls / (As*J.d)	Mpa	-	-	-	-	-
	Checking for control cracking fs<fsa		N.a	N.a	N.a	N.a	N.a
(5.10.8.2)	**Shrinkage and temperature Reinfocement (side distribution)**						
Areq	Area of required reinf	m2	0.00032	0.00027	0.00024	0.00024	0.00024
	Distribution on sides 9 D12	m2	0.00102	0.00102	0.00102	0.00102	0.00102
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45	0.45
	Checking		OK	OK	OK	OK	OK
	6.2 SHEAR FORCE CHECKING						
β	Factor indicating diag. cracked concr. to tension		5.9	4.0	2.2	2.0	2.0
θ	Angle of inclination of diagonal compressive	degree	27.00	27.00	35.60	39.44	40.66
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	90
bv	Effective web width as minimum web width - in dv	m	0.600	0.347	0.200	0.200	0.200
dv	Effective shear depth	m	1.188	1.188	1.307	1.401	1.432
	(de - a/2)	m	0.932	1.151	1.307	1.401	1.432
s	Spacing of stirrups	m	0.150	0.150	0.300	0.300	0.300
ncal	Amount of bars in spacing S	bars	2	2	2	2	2
Av	Shear reinf area in spacing S	m2	0.0003	0.0003	0.0003	0.0003	0.0003
β	Assume		5.9	3.9	2.2	2.0	2.0
θ	Assume	degree	27.00	27.00	35.79	39.67	40.92
v	Shear stress in concrete	kN/m2	2137	2938	3470	2170	1099
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	820	838	846	843	812
e_x	Strain in tensile reinforcement		-2.06E-03	1.03E-04	9.59E-04	1.34E-03	1.47E-03
	if ex<0, multiple with reduce factor		-1.28E-04	-	-	-	-
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok	Ok
v/f'c	Ratio of shear stress and f'c		0.047	0.065	0.077	0.048	0.024
β	Final value		5.9	4.0	2.2	2.0	2.0
θ	Final value	degree	27.00	27.00	35.60	39.44	40.66
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	2356	913	318	318	314
Vs	Shear resistance provided by shear reinforcement	kN	1878	1878	735	686	671
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	4234	2790	1053	1003	985
Vn2	Vn2	kN	8019	4641	2941	3152	3222
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	4234	2790	1053	1003	985
Vr	Factored shear resistance	kN	3810	2511	947	903	887
Vu	Shear	kN	1371	1091	816	547	283
(5.8.2.7)	**Shear checking**		OK	OK	OK	OK	OK
	Region requiring transverse reinf Checking		Need	Need	Need	Need	Need
	Minimum shear reinf area	m2	0.0001	0.0001	0.0001	0.0001	0.0001
	Mimimum shear reinforcement Checking		OK	OK	OK	OK	OK
	0.1*f'c*bv*dv	kN	3208	1857	1176	1261	1289
	Smax	m	0.60	0.60	0.60	0.60	0.60
	Maximum spacing Smax		OK	OK	OK	OK	OK

CALCULATION SHEET

I27 GIRDER

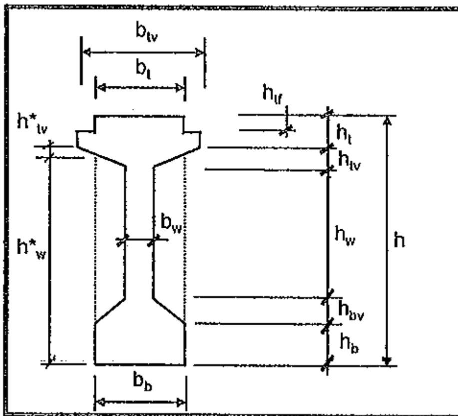
CONTENT

1. INPUT DATA:

1.1. General Data

Specification for bridge design:	TCN 272-05		
Live load	**HL93**		
Impact (or dynamic) of the live load	$IM =$	0.25	
Pedestrian	$PL =$	0.00	(kN/m^2)
Length of Girder	$L_d =$	27.00	(m)
Span between support	$L_{tt} =$	26.10	(m)
Carriageway width in bridge	$w =$	11.75	(m)
Parapet width	$c =$	0.50	(m)
Bridge width	$B =$	12.75	(m)
Number of girder	$N_d =$	5.00	girder
Space betweem 2 girders	$S =$	2.55	(m)
Distance from inside of parapet to exterior girder center	$d_e =$	0.77	(m)
Width of bridge deck	$b_{ds} =$	12.48	(m)
Length of the overhang (cantilever arm length)	$L_h =$	1.27	(m)
Thickness of bridge deck	$t_s =$	0.22	(m)
Precast plank width	$b_p =$	1.95	(m)
Precast plank thick	$h_p =$	0.08	(m)
Pavement thick	$h_{pa} =$	0.084	(m)

1.2. Girder dimension:



Width of over part		$b_{tv} =$	800.00	(mm)
		$b_t =$	600.00	(mm)
Width of under part		$b_b =$	600.00	(mm)
Girder high		$h =$	1500.00	(mm)
		$h_{tf} =$	80.00	(mm)
		$h_t =$	200.00	(mm)
Cross section at end		*at the midle*		
$b_w =$	600.00		200.00	(mm)
$h^*_{tv} =$	34.00		110.00	(mm)
$h^*_w =$	1266.00		740.00	(mm)
$h_b =$	0.00		250.00	(mm)
$h_{bv} =$	0.00		200.00	(mm)

1.3. MATERIAL PROPERTIES:

1.3.1 Concrete:

Girder concrete

Girder concrete strength at the 28 age days	$f'_c =$	45.00	MPa	
Unit weight of Concrete	$\gamma_c =$	2400.00	kG/m^3	
Modulus of elasticity	$E_c = 0.043\,\gamma_c^{1.5}\,\mathrm{sqrt}(f'_c) =$	33914.98	MPa	(5.4.2.4-1)

Deck concrete

Deck concrete strength at the 28 age days	$f'_c =$	35.00	MPa	
Unit weight of concrete	$\gamma_c =$	2400.00	kG/m^3	
Modulus of elasticity	$E_c = 0.043\,\gamma_c^{1.5}\,\mathrm{sqrt}(f'_c) =$	29910.20	MPa	(5.4.2.4-1)

1.3.2 Prestressing steel

Diameter of one strand	$D =$	12.70	mm
Area of one strand	$A_s^{12.7} =$	98.70	mm^2
Ultimate Tendon strength	$f_{pu} =$	1860.00	MPa
Yield strength of prestressing steel	$f_{py} = 0.9\,f_{pu} =$	1674.00	MPa
Modulus of strand	$E_p =$	197000.00	MPa
Wobble friction coefficient (mm-1)	$K =$	6.60E-07	mm^{-1}
Coeficient of friction (1/RAD)	$\mu =$	0.25	
Number of Strands in one Tendon	$n =$	12.00	Strands
Area of one Tendon	$A_s =$	1184.40	mm^2
Stress in the prestressing steel at jacking	$f_{pj} = 0.75\,f_{pu} =$	1395.00	MPa
Jacking force for one tendon	$P_j =$	1652.24	kN
Anchorage set	$\Delta L =$	6.00	mm
Area of one duck	$A_g =$	3318.31	mm^2
Number of Tendons	$N =$	4.00	Tendons

1.3.3 Reinforcing Steel:

Yield strength (deformed bar)	$f_{py} =$	400.00	(MPa)
Modulus of steel	$E_s =$	200000.00	(MPa)

2. INTERNAL FORCE:

2.1. Dead Load:

2.1.1 Load:

Interior Beam:

Bridge deck	DC_d =	13.30 (kN/m)
Precat plank & cross beam	DC_{pl} =	4.58 (kN/m)
Parapet	DC_{pa} =	4.74 (kN/m)
Pavement	DW_p =	4.44 (kN/m)

Exterior Beam:

Bridge deck	DC_d =	13.30 (kN/m)
Precat plank & cross beam	DC_{pl} =	2.29 (kN/m)
Parapet	DC_{pa} =	4.74 (kN/m)
Pavement	DW_p =	4.44 (kN/m)

2.1.2 Internal Force due to dead load:

Formula : $M = 0.5\, q.X_i\, (L-X_i)$

$Q = q.(0.5.L-X_i)$

L_{tt} = 26.10 (m)

Section	X_i (m)	Girder (DC) M (kNm)	Girder (DC) Q (kN)	Concrete Deck (DC) M (kNm)	Concrete Deck (DC) Q (kN)	Plank & cr.beam (DC) M (kNm)	Plank & cr.beam (DC) Q (kN)	Parapet (DC) M (kNm)	Parapet (DC) Q (kN)	Pavement (DW) M (kNm)	Pavement (DW) Q (kN)
INTERIOR GIRDER											
Support	0.00	0.00	216.70	0.00	173.55	0.00	59.81	0.00	61.86	0.00	57.96
L/8	3.26	618.62	162.53	495.42	130.16	170.74	44.86	176.58	46.39	165.46	43.47
L/4	6.53	1060.48	108.35	849.30	86.77	292.70	29.91	302.71	30.93	283.65	28.98
3L/8	9.79	1325.61	54.18	1061.62	43.39	365.87	14.95	378.39	15.46	354.56	14.49
L/2	13.05	1413.98	0.00	1132.40	0.00	390.26	0.00	403.62	0.00	378.20	0.00
EXTERIOR GIRDER											
Gối	0.00	0.00	216.70	0.00	173.55	0.00	59.81	0.00	61.86	0.00	57.96
L/8	3.26	618.62	162.53	495.42	130.16	85.37	44.86	176.58	46.39	165.46	43.47
L/4	6.53	1060.48	108.35	849.30	86.77	146.35	29.91	302.71	30.93	283.65	28.98
3L/8	9.79	1325.61	54.18	1061.62	43.39	182.94	14.95	378.39	15.46	354.56	14.49
L/2	13.05	1413.98	0.00	1132.40	0.00	195.13	0.00	403.62	0.00	378.20	0.00

2.2 Live Load:

2.2.1. Distribution factors for Live load:

Modular Ratio: Girder Concrete/Deck Concrete	$n = E_g / E_d$ =	1.13
Distance from girder centroid to bridge deck centroid	$e^I g$ =	853.99 (mm)
	$e^E g$ =	853.99 (mm)
Longgitudinal stiffness parameter	$K^I g = n(Ig + A\, e^2_g)$ =	1.2E+12
	$K^E g = n(Ig + A\, e^2_g)$ =	1.2E+12
Ration	$K^I g / (L\, t^3_s)$ =	4.37
	$K^E g / (L\, t^3_s)$ =	4.37
	S / L =	0.10

(a) Distribution Factor for Moment: g(M)

Interior Beam:

For one lane $$0.06+\left(\frac{S}{4300}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{Lt_s^3}\right)^{0.1} = \mathbf{0.528}$$

Two or more lanes $$0.075+\left(\frac{S}{2900}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{Lt_s^3}\right)^{0.1} = \mathbf{0.749}$$

Exterior Beam:

For one lane, follow the lever rule

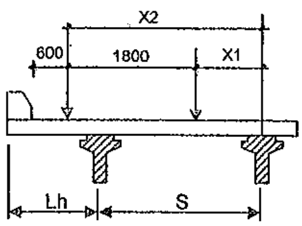
X_1 = 920.000

X_2 = 2720.00

Y_1 = 0.361

Y_2 = 1.067

=> $g(M) = 0.5*\Sigma yi = \boxed{0.714}$



Two or more lanes $$e = 0.77 + \frac{d_e}{2800} = \quad 1.045 < (=) 1$$

Choice e = 1.045 IF(e>1,1,e)

=> $g(M) = e*g_{trong} = \boxed{0.782}$

(b) Distribution Factor for Shear force: g(Q)

Interior Beam:

For one lane $$0.36 + \frac{S}{7600} = \mathbf{0.696}$$

Two or more lanes $$0.2 + \frac{S}{3600} - \left(\frac{S}{10700}\right)^2 = \mathbf{0.852}$$

Exterior Beam:

For one lane, follow the lever rule

$g(Q) = 0.5*\Sigma yi =$ $\boxed{0.714}$

Two or more lanes

$$e = 0.6 + \frac{de}{3000} = \quad 0.857$$

$\Rightarrow g(Q) = e*g_{trong} =$ $\boxed{0.729}$

(c) Correction factor for skew bridge:

* Correction factor of distribution factor for moment (Table 4.6.2.2.2d-1)

Skew angle	θ =	0	**Degree.**
Factor	c1 =	0.000	
Correction factor	CF(M) =	1.000	

Area of applications
300<= θ <= 600
1100<= S <= 4900
6000 <= L <= 73000
Nb >= 4

$$\boxed{\begin{aligned} &CF(M) = 1.0 - c1.(\tan\theta)^{1.5} \\ &c1 = 0.25\left(\frac{Kg}{L.ts^3}\right)^{0.25}\left(\frac{S}{L}\right)^{0.5} \end{aligned}}$$

* Regulation factor of distribution factor for shear force (Table 4.6.2.2.3c-1)

Correction Factor CF(Q)= 1.000

Area of applications
00<= θ <= 600
1100<= S <= 4900
6000 <= L <= 73000
Nb >= 4

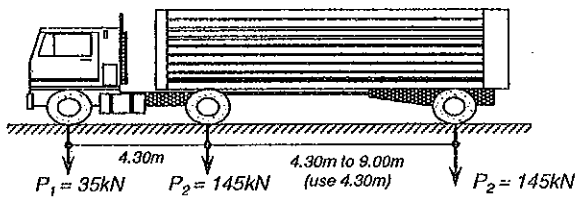
$$\boxed{CF(Q) = 1.0 + 0.20\left(\frac{L.ts^3}{Kg}\right)^{0.3}.\tan\theta}$$

(d) Table of Distribution factors for Live load:

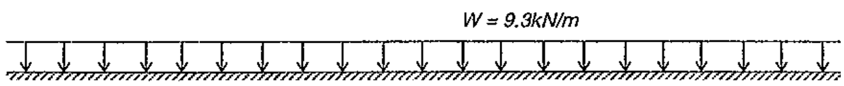
Interior Beam	g(M)	g(Q)	m	m*g(M)	m*g(Q)	m*g(M)*CF(M)	m*g(Q)*CF(Q)
1 lane	0.528	0.696	1.20	0.634	0.835	0.634	0.835
2 or more lanes	0.749	0.852	1.00	0.749	0.852	**0.749**	**0.852**
Exterior Beam							
1 lane	0.714	0.714	1.20	0.856	0.856	**0.856**	**0.856**
2 or more lanes	0.782	0.729	1.00	0.782	0.729	0.782	0.729

2.2.2 Live Load:

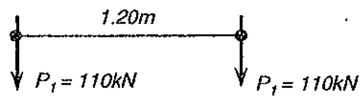
Design Truck



Design Lane Load



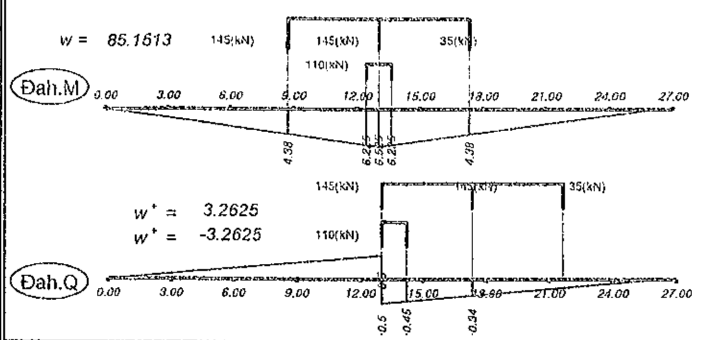
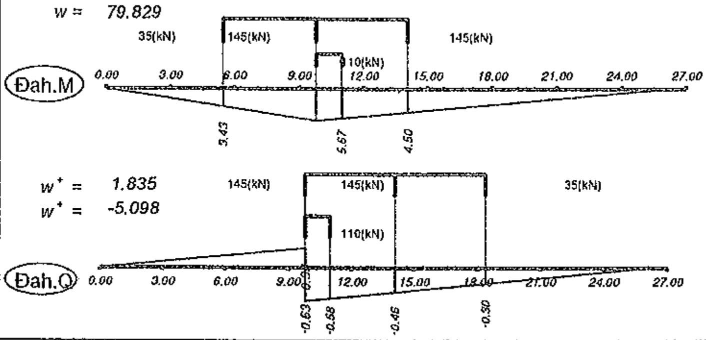
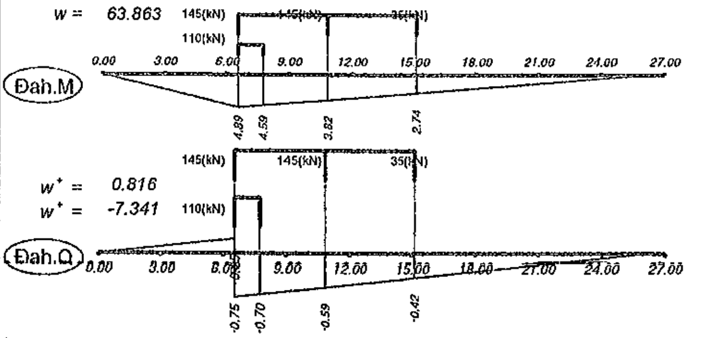
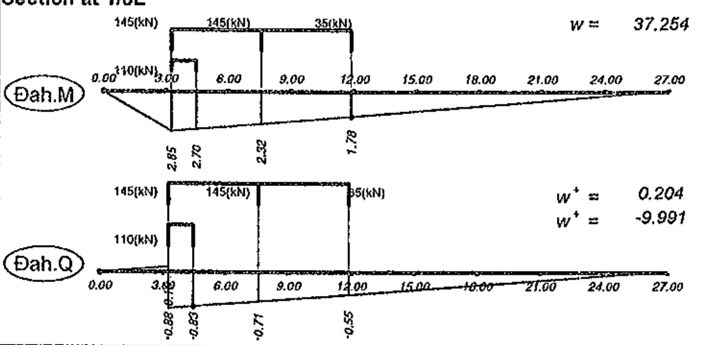
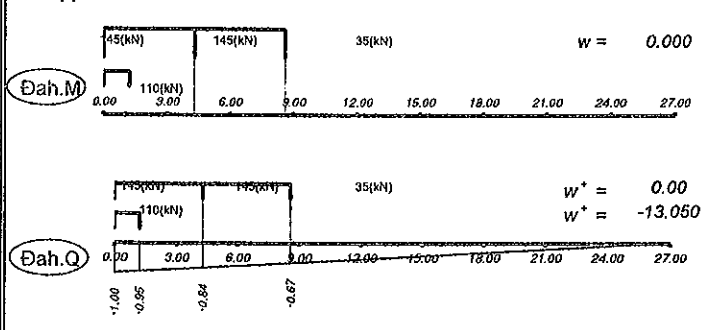
Design Tendem



- Truck	P1 =	35.00	(kN)
	P2 =	145.00	(kN)
- Lane load	W =	9.30	(kN)
- Tandem	P1 =	110.00	(kN)
- Pedestrian	PL =	0.00	kN/m^2
- Dynammic load	IM =	0.25	

2.2.3 Internal Force due to Live load:

Design truck or Tendem	Momen	$M_{TR(Ta)} = \Sigma Piyi$	(kNm)
	Shear force	$Q_{TR(Ta)} = \Sigma Piyi$	(kN)
Lane load	Momen	$M_{Ln} = W.F_M$	(kNm)
	Shear force	$Q_{Ln} = W.F_Q$	(kN)
Pedestrian	Momen	$M_{PL} = PL.F_M$	(kNm)
	Shear force	$Q_{PL} = PL.F_Q$	(kN)

Influence line for Momen & Shear force	Load	Momen (KN.m)	Shear
Section at 1/2L 	Truck	1733.63	127.08
	Lane	791.91	30.34
	Tandem	1369.50	104.94
	Design	**1733.63**	**127.08**
	Pedestrian	0.00	0.00
Section at 3/8L 	Truck	1660.21	167.70
	Lane	742.41	47.41
	Tandem	1296.28	132.44
	Design	**1660.21**	**167.70**
	Pedestrian	0.00	0.00
Section at 1/4L 	Truck	1359.34	208.33
	Lane	593.93	68.27
	Tandem	1043.63	159.94
	Design	**1359.34**	**208.33**
	Pedestrian	0.00	0.00
Section at 1/8L 	Truck	812.21	248.95
	Lan	346.46	92.92
	Tandem	611.53	187.44
	Design	**812.21**	**248.95**
	Pedestrian	0.00	0.00
At support 	Truck	0.00	289.58
	Lane	0.00	121.37
	Tandem	0.00	214.94
	Design	**0.00**	**289.58**
	Pedestrian	0.00	0.00

Internal Force due to Live load : $M_{(LL+IM)} = m^*g(M) * [max\{M_{TR}, M_{Ta}\}^*(1+IM) + M_{Ln}]$

$Q_{(LL+IM)} = m^*g(Q) * [max\{Q_{TR}, Q_{Ta}\}^*(1+IM) + Q_{Ln}]$

Internal Force due to pedestrian :

$M = g(M) * M_{PL}$

$Q = g(Q) * Q_{PL}$

In which:

$M_{TR(Ta)}$ moment due to truck or Tandem

$Q_{TR(Ta)}$ Shear force due to truck or Tandem

yi Value of influence line F Area of influence line

m Lane factor g Distribution factor

Interior	m*g(M)	m*g(Q)
	0.749	0.852
Exterior		
	0.856	0.856

TABLE OF INTERNAL FORCE DUE TO LIVE LOAD

Setion	Xi	Interior Girder		Exterior Girder	
		M	Q	M	Q
	(m)	(kNm)	(kN)	(kNm)	(kN)
Support	0.00	0.00	411.58	0.00	413.96
L/8	3.26	1019.60	344.12	1166.28	346.11
L/4	6.53	1716.98	279.88	1963.98	281.50
3L/8	9.79	2109.75	218.88	2413.26	220.15
L/2	13.05	2215.53	161.10	2534.24	162.04

2.3 Load combination:

Strength limit state: $U = \eta\ [1.25\ DC + 1.50\ DW + 1.75\ (LL+IM)]$

Service limit state: $U = \eta\ [1.00\ DC + 1.00\ DW + 1.00\ (LL+IM)]$

Fatigue state: $U = 0.75\ (LL+IM)]$

The modify load factort $\eta = \eta_D\ \eta_R\ \eta_I$

STATE	Modify Load Factor			
	η_D	η_R	η_I	$\eta = \eta_D\ \eta_R\ \eta_I$
Strength	1.00	1.00	1.00	1.00
Service	1.00	1.00	1.00	1.00

2.3.1 Load combination - - Interior Girder:

STATE	Strength										
Load	Load Factor	Section									
		Support		L/8		L/4		3L/8		L/2	
	γ	M	Q	M	Q	M	Q	M	Q	M	Q
		(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)
DC	1.25	0.00	639.90	1826.70	479.92	3131.49	319.95	3914.37	159.97	4175.32	0.00
DW	1.50	0.00	86.94	248.19	65.21	425.47	43.47	531.84	21.74	567.30	0.00
LL+IM	1.75	0.00	720.27	1784.30	602.20	3004.72	489.79	3692.07	383.04	3877.17	281.93
Total		**0.00**	**1447.11**	**3859.20**	**1147.33**	**6561.69**	**853.21**	**8138.28**	**564.74**	**8619.79**	**281.93**

STATE	Service										
Load	Load Factor	Section									
		Support		L/8		L/4		3L/8		L/2	
	γ	M	Q	M	Q	M	Q	M	Q	M	Q
		(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)
DC	1.00	0.00	511.92	1461.36	383.94	2505.19	255.96	3131.49	127.98	3340.26	0.00
DW	1.00	0.00	57.96	165.46	43.47	283.65	28.98	354.56	14.49	378.20	0.00
LL+IM	1.00	0.00	411.58	1019.60	344.12	1716.98	279.88	2109.75	218.88	2215.53	161.10
Total		**0.00**	**981.46**	**2646.42**	**771.53**	**4505.83**	**564.82**	**5595.81**	**361.35**	**5933.98**	**161.10**

2.3.2 Load combination - Exterior Girder:

STATE	Strength										
Load	Load factor	Section									
		Supprt		L/8		L/4		3L/8		L/2	
	γ	M	Q	M	Q	M	Q	M	Q	M	Q
		(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)
DC	1.25	0.00	639.90	1719.99	479.92	2948.56	319.95	3685.70	159.97	3931.41	0.00
DW	1.50	0.00	86.94	248.19	65.21	425.47	43.47	531.84	21.74	567.30	0.00
LL+IM	1.75	0.00	724.44	2040.98	605.69	3436.97	492.63	4223.20	385.25	4434.93	283.56
Total		**0.00**	**1451.28**	**4009.17**	**1150.82**	**6811.00**	**856.05**	**8440.74**	**566.96**	**8933.63**	**283.56**

STATE	Service										
Load	load factor	Section									
		Support		L/8		L/4		3L/8		L/2	
	γ	M	Q	M	Q	M	Q	M	Q	M	Q
		(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)
DC	1.00	0.00	511.92	1375.99	383.94	2358.85	255.96	2948.56	127.98	3145.13	0.00
DW	1.00	0.00	57.96	165.46	43.47	283.65	28.98	354.56	14.49	378.20	0.00
LL+IM	1.00	0.00	413.96	1166.28	346.11	1963.98	281.50	2413.26	220.15	2534.24	162.04
Total		**0.00**	**983.84**	**2707.73**	**773.52**	**4606.48**	**566.44**	**5716.38**	**362.61**	**6057.57**	**162.04**

3. TENDON PROFILE AND PROPERTY OF GIRDER CROSS SECTION

3.1. Tendon profile:

Tendon profile follow Parabol equation:

$$y_i = f - \frac{4.(f-c).x.(l-x)}{l^2}$$

in which: Origin of coordinates in left edge of the Girder bottom (0.0)

- f — Maximum deflection at mid span of tendon
- c — Distance from maximum deflection point to girder bottom
- (x_i, y_i) — Coordination of point under consider i = 1,2...
- L — actual distance between cable ends (X-axis)
- $L_p = X_2 - X_1$ — Distance between 2 point under consider

angle of rotation of tendon for X_i-axis — $\tan(\alpha) = (4.f\,(1-2.Xi\,/\,L))\,/\,L$

$\alpha = 2\,f\,/\,0.5\,L - \tan(\alpha)$

L_{span} =	27000	(mm)
$L_{su.}$ =	26100	(mm)
L_{cap} =	26700	(mm)

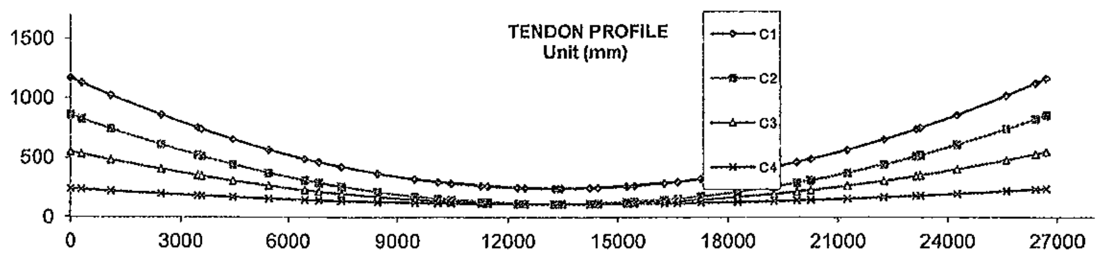
	f =	1170	(mm)	Lcáp =	26700	(mm)	C =	240	(mm)
	Section	**Xi**	**Yi**	**Lp**	**ΣLcáp**	**Tan(αi)**	(α)	Σα	**Section**
		(mm)	(mm)	(mm)	(mm)		(rad)	(rad)	
TENDON No 1	**Anchorage**	0.00	1170.00	0.00	0.00	0.1753	0.0000	0.0000	**Anchorage**
	Support	300.00	1128.67	300.00	300.00	0.1713	0.0039	0.0039	**Support**
	L/8	3562.50	739.88	3262.50	3562.50	0.1285	0.0468	0.0507	**L/8**
	L/4	6825.00	462.17	3262.50	6825.00	0.0857	0.0896	0.1403	**L/4**
	3L/8	10087.50	295.54	3262.50	10087.50	0.0428	0.1324	0.2728	**3L/8**
	L/2	13350.00	240.00	3262.50	13350.00	0.0000	0.1753	0.4480	**L/2**

	f =	860	(mm)	Lcáp =	26700	(mm)	C =	110	(mm)
	Section	**Xi**	**Yi**	**Lp**	**ΣLcáp**	**Tan(α)**	(α)	Σα	**Section**
		(mm)	(mm)	(mm)	(mm)		(rad)	(rad)	
TENDON No 2	**Anchorage**	0.00	860.00	0.00	0.00	0.1288	0.0000	0.0000	**Anchorage**
	Support	300.00	826.67	300.00	300.00	0.1259	0.0029	0.0029	**Support**
	L/8	3562.50	513.13	3262.50	3562.50	0.0945	0.0344	0.0373	**L/8**
	L/4	6825.00	289.17	3262.50	6825.00	0.0630	0.0659	0.1031	**L/4**
	3L/8	10087.50	154.79	3262.50	10087.50	0.0315	0.0974	0.2005	**3L/8**
	L/2	13350.00	110.00	3262.50	13350.00	0.0000	0.1288	0.3293	**L/2**

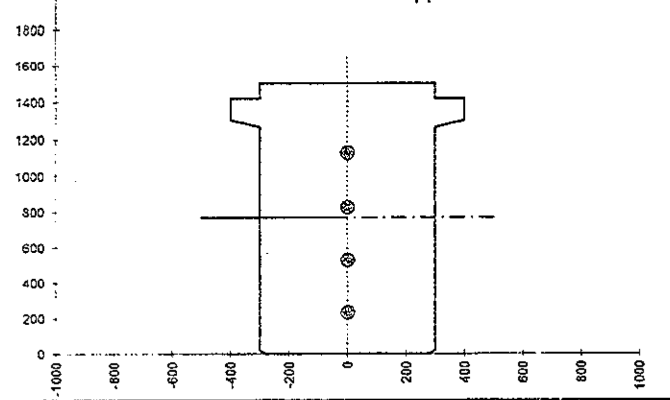
	f =	550	(mm)	Lcáp =	26700	(mm)	C =	110	(mm)
	Section	**Xi**	**Yi**	**Lp**	**ΣLcáp**	**Tan(α)**	(α)	Σα	**Section**
		(mm)	(mm)	(mm)	(mm)		(rad)	(rad)	
TENDON No 3	**Anchorage**	0.00	550.00	0.00	0.00	0.0824	0.0000	0.0000	**Anchorage**
	Support	300.00	530.45	300.00	300.00	0.0805	0.0019	0.0019	**Support**
	L/8	3562.50	346.50	3262.50	3562.50	0.0604	0.0220	0.0238	**L/8**
	L/4	6825.00	215.11	3262.50	6825.00	0.0403	0.0421	0.0660	**L/4**
	3L/8	10087.50	136.28	3262.50	10087.50	0.0201	0.0623	0.1282	**3L/8**
	L/2	13350.00	110.00	3262.50	13350.00	0.0000	0.0824	0.2106	**L/2**

	f =	240	(mm)	Lcáp =	26700	(mm)	C =	110	(mm)
	Section	**Xi**	**Yi**	**Lp**	**ΣLcáp**	**Tan(α)**	(α)	Σα	**Section**
		(mm)	(mm)	(mm)	(mm)		(rad)	(rad)	
TENDON No 4	**Anchorage**	0.00	240.00	0.00	0.0	0.0360	0.0000	0.0000	**Anchorage**
	Support	300.00	234.22	300.00	300.0	0.0351	0.0008	0.0008	**Support**
	L/8	3562.50	179.88	3262.50	3562.5	0.0264	0.0096	0.0104	**L/8**
	L/4	6825.00	141.06	3262.50	6825.0	0.0176	0.0184	0.0288	**L/4**
	3L/8	10087.50	117.76	3262.50	10087.5	0.0088	0.0272	0.0560	**3L/8**
	L/2	13350.00	110.00	3262.50	13350.0	0.0000	0.0360	0.0919	**L/2**

	TENDON No 1		TENDON No 2		TENDON No 3		TENDON No 4	
Section	X_i	Y_i	X_i	Y_i	X_i	Y_i	X_i	Y_i
	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)
Anchorage	0.00	1170.00	0.0	860.00	0.0	550.00	0.00	240.00
Support	**300.00**	**1128.67**	**300.0**	**826.67**	**300.0**	**530.45**	**300.00**	**234.22**
1	1100.00	1023.06	1100.0	741.50	1100.0	480.48	1100.00	219.46
2	2450.00	859.97	2450.0	609.98	2450.0	403.32	2450.00	196.66
3	3450.00	751.44	3450.0	522.45	3450.0	351.97	3450.00	181.49
L/8	**3562.50**	**739.88**	**3562.5**	**513.13**	**3562.5**	**346.50**	**3562.50**	**179.88**
4	4450.00	653.33	4450.0	443.33	4450.0	305.56	4450.00	167.78
5	5450.00	565.67	5450.0	372.64	5450.0	264.08	5450.00	155.52
6	6450.00	488.44	6450.0	310.35	6450.0	227.54	6450.00	144.73
7	7450.00	421.65	7450.0	256.49	7450.0	195.94	7450.00	135.39
L/4	**6825.00**	**462.17**	**6825.0**	**289.17**	**6825.0**	**215.11**	**6825.00**	**141.06**
8	8450.00	365.29	8450.0	211.04	8450.0	169.28	8450.00	127.51
9	9450.00	319.37	9450.0	174.01	9450.0	147.55	9450.00	121.09
10	10450.00	283.89	10450.0	145.39	10450.0	130.76	10450.00	116.13
11	11450.00	258.84	11450.0	125.19	11450.0	118.91	11450.00	112.63
3L/8	**10087.50**	**295.54**	**10087.5**	**154.79**	**10087.5**	**136.28**	**10087.50**	**117.76**
12	12450.00	244.23	12450.0	113.41	12450.0	112.00	12450.00	110.59
13	13450.00	240.05	13450.0	110.04	13450.0	110.02	13450.00	110.01
14	14450.00	246.31	14450.0	115.09	14450.0	112.99	14450.00	110.88
15	15450.00	263.01	15450.0	128.56	15450.0	120.89	15450.00	113.22
L/2	**13350.00**	**240.00**	**13350.0**	**110.00**	**13350.0**	**110.00**	**13350.00**	**110.00**
2	11250.00	263.01	11250.0	128.56	11250.0	120.89	11250.00	113.22
3	12250.00	246.31	12250.0	115.09	12250.0	112.99	12250.00	110.88
4	13250.00	240.05	13250.0	110.04	13250.0	110.02	13250.00	110.01
5	14250.00	244.23	14250.0	113.41	14250.0	112.00	14250.00	110.59
-	**16612.50**	**295.54**	**16612.5**	**154.79**	**16612.5**	**136.28**	**16612.50**	**117.76**
6	15250.00	258.84	15250.0	125.19	15250.0	118.91	15250.00	112.63
7	16250.00	283.89	16250.0	145.39	16250.0	130.76	16250.00	116.13
8	17250.00	319.37	17250.0	174.01	17250.0	147.55	17250.00	121.09
9	18250.00	365.29	18250.0	211.04	18250.0	169.28	18250.00	127.51
-	**19875.00**	**462.17**	**19875.0**	**289.17**	**19875.0**	**215.11**	**19875.00**	**141.06**
10	19250.00	421.65	19250.0	256.49	19250.0	195.94	19250.00	135.39
11	20250.00	488.44	20250.0	310.35	20250.0	227.54	20250.00	144.73
12	21250.00	565.67	21250.0	372.64	21250.0	264.08	21250.00	155.52
13	22250.00	653.33	22250.0	443.33	22250.0	305.56	22250.00	167.78
-	**23137.50**	**739.88**	**23137.5**	**513.13**	**23137.5**	**346.50**	**23137.50**	**179.88**
14	23250.00	751.44	23250.0	522.45	23250.0	351.97	23250.00	181.49
14	24250.00	859.97	24250.0	609.98	24250.0	403.32	24250.00	196.66
16	25600.00	1023.06	25600.0	741.50	25600.0	480.48	25600.00	219.46
Support	26400.00	1128.67	26400.0	826.67	26400.0	530.45	26400.00	234.22
Anchorage	26700.00	1170.00	26700.0	860.00	26700.0	550.00	26700.00	240.00

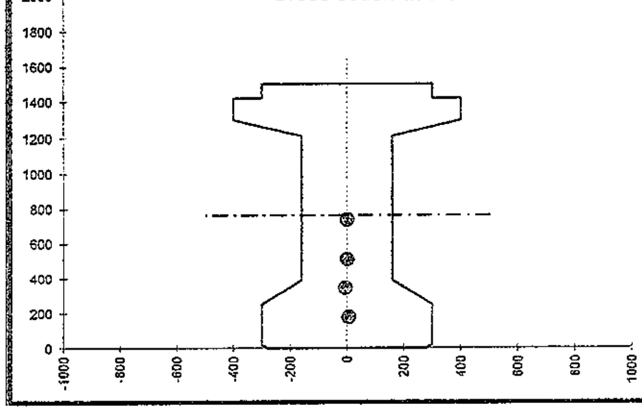


3.2 Property of Girder Cross section at transfer (Stage I: net cross section):



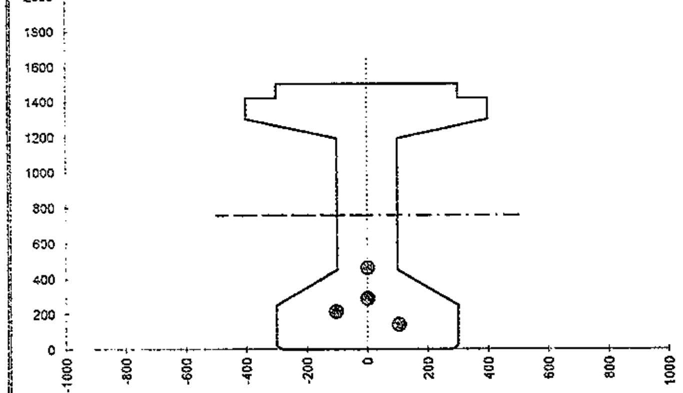
Cross setion at support

A	0.9273	m^2
e_x	0.0000	m
e_y	0.7682	m
I_x	0.1782	m^4
I_y	0.0303	m^4
I_{xy}	0.0000	m^4
y_b	0.7682	m
y_t	0.7318	m
S_b	0.2320	m^3
S_t	0.2436	m^3
$E_{cáp\ (Y)}$	0.0882	m
q =	22.2544	(KN/m)



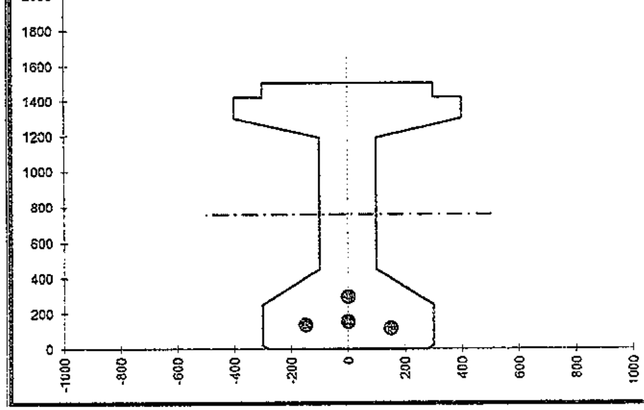
Cross setion at L/8

A	0.6703	m^2
e_x	0.0000	m
e_y	0.7645	m
I_x	0.1599	m^4
I_y	0.0160	m^4
I_{xy}	0.0000	m^4
y_b	0.7645	m
y_t	0.7355	m
S_b	0.2092	m^3
S_t	0.2174	m^3
$E_{cáp\ (Y)}$	0.3196	m
q =	16.0877	(KN/m)



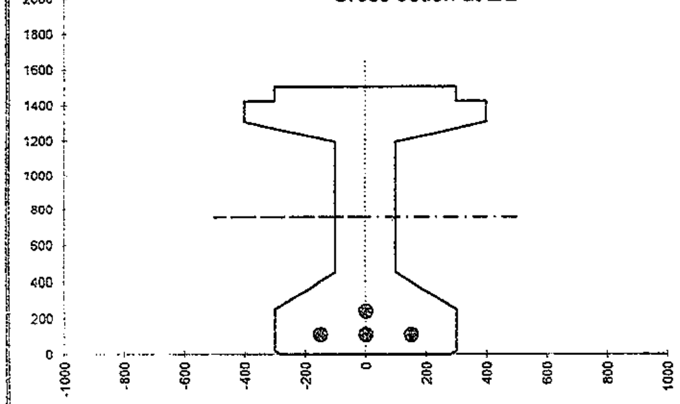
Cross setion at L/4

A	0.5766	m^2
e_x	0.0000	m
e_y	0.7570	m
I_x	0.1549	m^4
I_y	0.0144	m^4
I_{xy}	0.0000	m^4
y_b	0.7570	m
y_t	0.7430	m
S_b	0.2047	m^3
S_t	0.2085	m^3
$E_{cáp\ (Y)}$	0.4801	m
q =	13.8384	(KN/m)



Cross setion at 3L/8

A	0.5766	m^2
e_x	0.0000	m
e_y	0.7570	m
I_x	0.1549	m^4
I_y	0.0144	m^4
I_{xy}	0.0000	m^4
y_b	0.7570	m
y_t	0.7430	m
S_b	0.2047	m^3
S_t	0.2085	m^3
$E_{cáp\ (Y)}$	0.5809	m
q =	13.8384	(KN/m)



Cross setion at L/2

A	0.5766	m^2
e_x	0.0000	m
e_y	0.7570	m
I_x	0.1549	m^4
I_y	0.0144	m^4
I_{xy}	0.0000	m^4
y_b	0.7570	m
y_t	0.7430	m
S_b	0.2047	m^3
S_t	0.2085	m^3
$E_{cap\ (Y)}$	0.6145	m
q =	13.8384	(KN/m)

Uniform load due to sefl weigh of Girder in Stage 1: Q = 16.61 (KN/m)

3.3. Property of Girder cross section in service stage (stage II: Composite cross section) :

3.3.1. Effective flange width (4.6.2.6)

Modular Ratio: Deck Concrete/Girder Concrete n = Eb / Ed = 0.88

For Interior Girder:

$$b_I = \min\begin{cases}1/4L_u \\ 12h_f + \max(0.5b, b_w) \\ S\end{cases} \quad \Rightarrow n^* b_I = 2248.88861 \text{ (mm)}$$

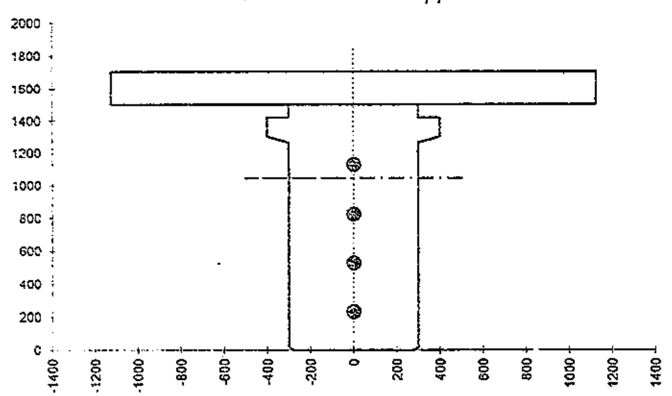
For Exterior Girder:

$$b_E = 0.5b_I + \min\begin{cases}1/8L_u \\ 6h_f + \max(0.5b_w, 0.25b) \\ L_h\end{cases} \quad \Rightarrow n^* b_E = 2244.47903 \text{ (mm)}$$

3.3.2. Property of Girder cross section in stage II (service stage):

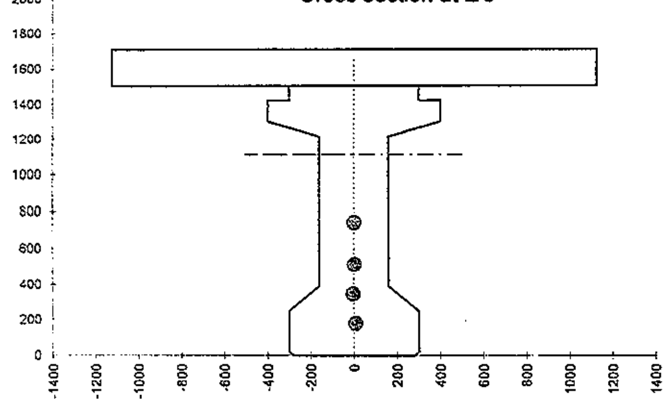
Interior Girder:

Cross section at support



A	1.3928	m^2
e_x	0.0000	m
e_y	1.0474	m
I_x	0.3961	m^4
I_y	0.2265	m^4
I_{xy}	0.0000	m^4
y_b	1.0474	m
y_t	0.6596	m
S_b	0.3782	m^3
S_t	0.6006	m^3
$E_{cáp\ (Y)}$	0.3674	m
q =	33.4269	(KN/m)

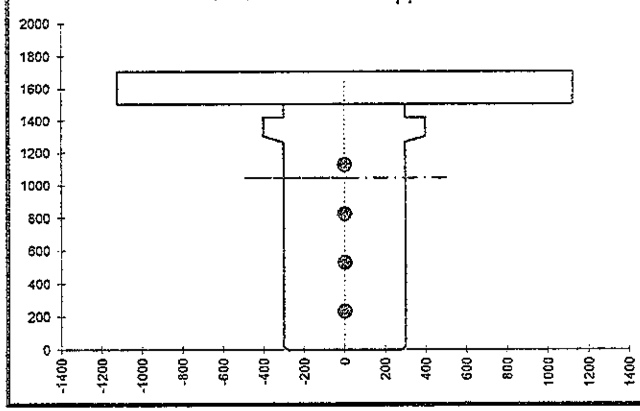
Cross section at L/8



A	1.1358	m^2
e_x	0.0000	m
e_y	1.1083	m
I_x	0.3550	m^4
I_y	0.2122	m^4
I_{xy}	0.0000	m^4
y_b	1.1083	m
y_t	0.5987	m
S_b	0.3203	m^3
S_t	0.5930	m^3
$E_{cáp\ (Y)}$	0.6635	m
q =	27.2602	(KN/m)

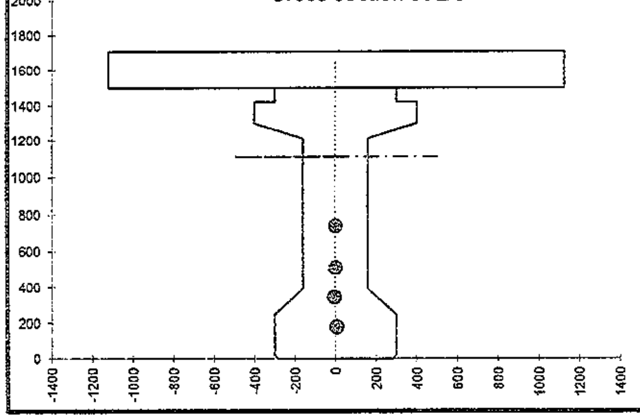
Exterior Girder:

Cross section at support



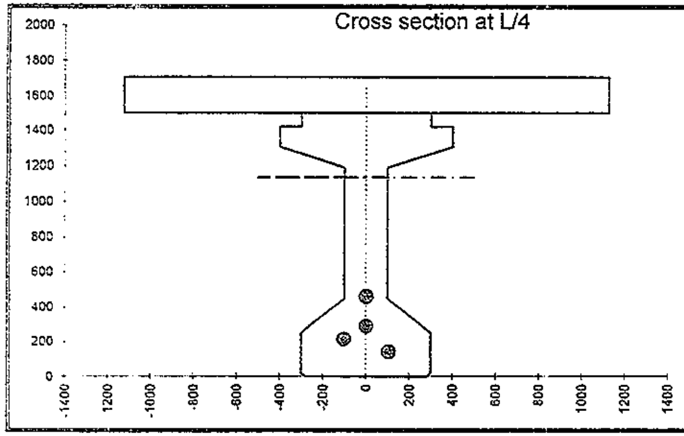
A	1.3919	m^2
e_x	0.0000	m
e_y	1.0470	m
I_x	0.3958	m^4
I_y	0.2254	m^4
I_{xy}	0.0000	m^4
y_b	1.0470	m
y_t	0.6600	m
S_b	0.3781	m^3
S_t	0.5998	m^3
$E_{cáp\ (Y)}$	0.3670	m
q =	34.1009	(KN/m)

Cross section at L/8

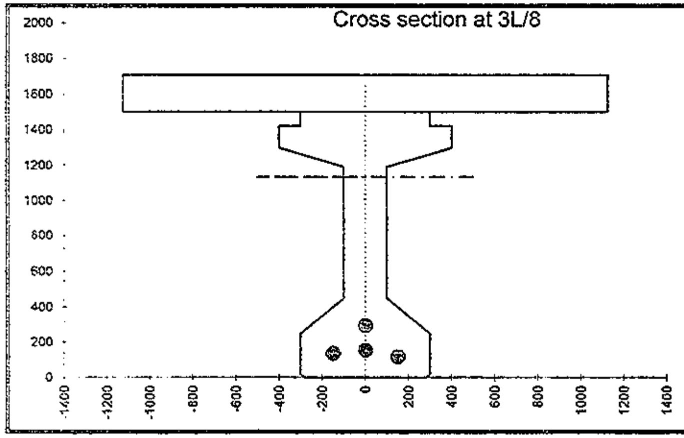


A	1.1349	m^2
e_x	0.0000	m
e_y	1.1079	m
I_x	0.3548	m^4
I_y	0.2111	m^4
I_{xy}	0.0000	m^4
y_b	1.1079	m
y_t	0.5991	m
S_b	0.3202	m^3
S_t	0.5922	m^3
$E_{cáp\ (Y)}$	0.6631	m
q =	27.8057	(KN/m)

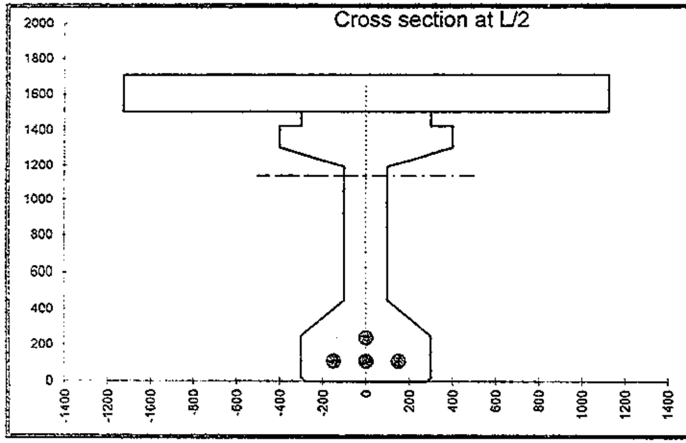
Interior Girder:


Cross section at L/4

A	1.0421	m^2
e_x	0.0000	m
e_y	1.1351	m
I_x	0.3411	m^4
I_y	0.2106	m^4
I_{xy}	0.0000	m^4
y_b	1.1351	m
y_t	0.5719	m
S_b	0.3005	m^3
S_t	0.5966	m^3
$E_{cáp\ (Y)}$	0.8583	m
q =	25.0109	(KN/m)

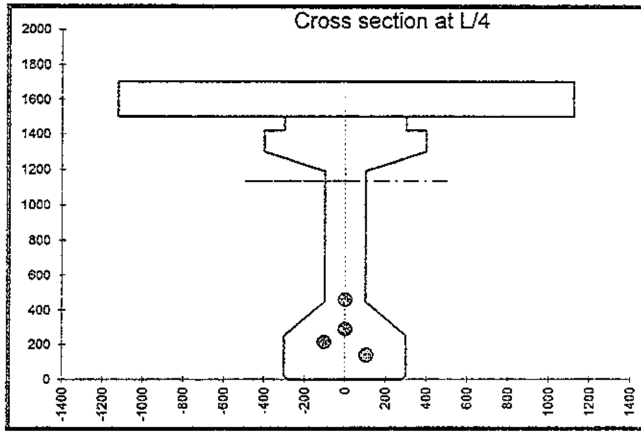

Cross section at 3L/8

A	1.0421	m^2
e_x	0.0000	m
e_y	1.1351	m
I_x	0.3411	m^4
I_y	0.2106	m^4
I_{xy}	0.0000	m^4
y_b	1.1351	m
y_t	0.5719	m
S_b	0.3005	m^3
S_t	0.5966	m^3
$E_{cáp\ (Y)}$	0.9590	m
q =	25.0109	(KN/m)

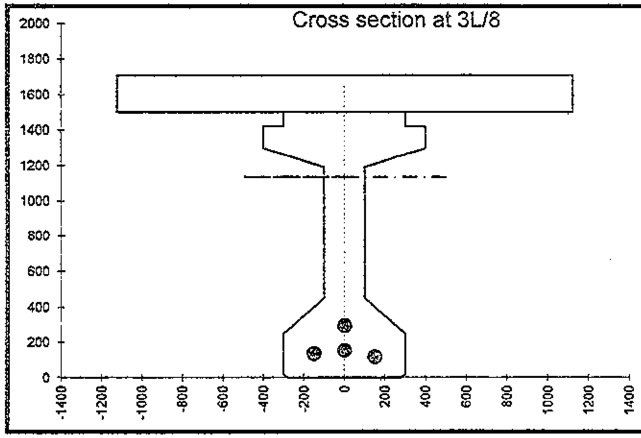

Cross section at L/2

A	1.0421	m^2
e_x	0.0000	m
e_y	1.1351	m
I_x	0.3411	m^4
I_y	0.2106	m^4
I_{xy}	0.0000	m^4
y_b	1.1351	m
y_t	0.5719	m
S_b	0.3005	m^3
S_t	0.5966	m^3
$E_{cáp\ (Y)}$	0.9926	m
q =	25.0109	(KN/m)

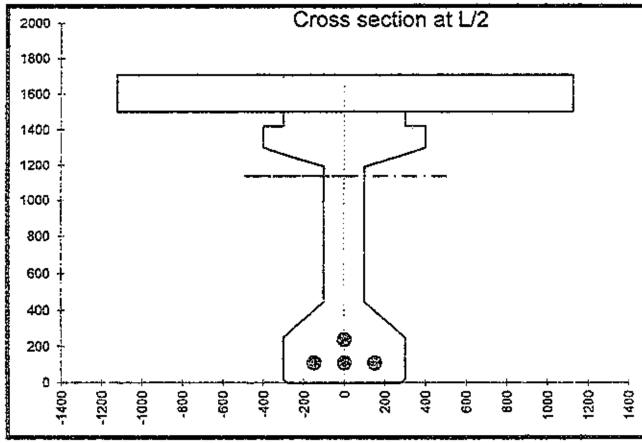
Exterior Girder:


Cross section at L/4

A	1.0412	m^2
e_x	0.0000	m
e_y	1.1347	m
I_x	0.3409	m^4
I_y	0.2095	m^4
I_{xy}	0.0000	m^4
y_b	1.1347	m
y_t	0.5723	m
S_b	0.3005	m^3
S_t	0.5958	m^3
$E_{cáp\ (Y)}$	0.8579	m
q =	25.5096	(KN/m)


Cross section at 3L/8

A	1.0412	m^2
e_x	0.0000	m
e_y	1.1347	m
I_x	0.3409	m^4
I_y	0.2095	m^4
I_{xy}	0.0000	m^4
y_b	1.1347	m
y_t	0.5723	m
S_b	0.3005	m^3
S_t	0.5958	m^3
$E_{cáp\ (Y)}$	0.9586	m
q =	25.5096	(KN/m)


Cross section at L/2

A	1.0412	m^2
e_x	0.0000	m
e_y	1.1347	m
I_x	0.3409	m^4
I_y	0.2095	m^4
I_{xy}	0.0000	m^4
y_b	1.1347	m
y_t	0.5723	m
S_b	0.3005	m^3
S_t	0.5958	m^3
$E_{cáp\ (Y)}$	0.9922	m
q =	0.0000	(KN/m)

4. LOSS OF PRESTRESS

4.1 Loss of prestressing force immediately (Instantaneous losses):

4.1.1 Friction between Prestressing Tendon and Duck:

Formula: $\Delta f_{pF} = f_{pj}\,(1-e^{-(Kx+\mu\alpha)})$ (5.9.5.2.2)

Xi: Length of tendon from the jacking end to any point under consideration

Section		Tendon no. 1		Tendon no. 2		Tendon no. 3		Tendon no. 4		SUM/4
	X_i	$\Sigma\alpha$	Δf_{pF}	$\Sigma\alpha$	Δf_{pF}	$\Sigma\alpha$	Δf_{pF}	$\Sigma\alpha$	Δf_{pF}	$\Sigma\Delta f_{pF}$
	(mm)	(rad)	(MPa)	(rad)	(MPa)	(rad)	(MPa)	(rad)	(MPa)	(MPa)
Ancho.	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.00
Support	300.00	0.0039	1.65	0.0029	1.29	0.0019	0.92	0.0008	0.56	1.10
L/8	3562.50	0.0507	20.81	0.0373	16.19	0.0238	11.55	0.0104	6.89	13.86
L/4	6825.00	0.1403	54.14	0.1031	41.62	0.0660	28.98	0.0288	16.23	35.24
3L/8	10087.50	0.2728	100.60	0.2005	77.00	0.1282	52.97	0.0560	28.51	64.77
L/2	13350.00	0.4480	158.76	0.3293	121.52	0.2106	83.16	0.0919	43.65	101.77

4.1.2 Anchorage seating or Set:

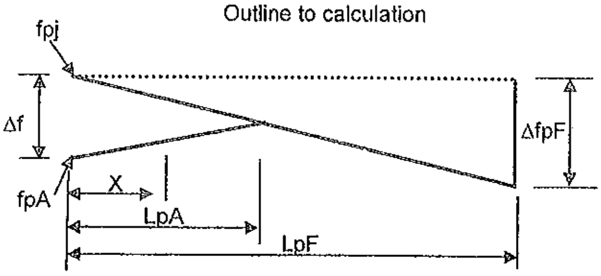
Formula

$$L_{pA} = \sqrt{\frac{E(\Delta L)L_{pF}}{\Delta f_{pF}}}$$

$$\Delta f = \frac{2\Delta f_{pF} L_{pA}}{L_{pF}}$$

$$\Delta f_{pA} = \Delta f\left(1 - \frac{X}{L_{pA}}\right)$$

Outline to calculation



Trong đó:

L_{pA}	Effective length due to anchorage set
E	Cable modulus of elasticity
ΔL	Setting length
L_{pF}	The length from anchorage to point that loss stress due to friction was known
Δf_{pF}	The loss stress value at the point that the leng from anchorage ti it is L_{pF}
Δf	The loss stress value at Anchorage

Choice the length from anchorage to point that loss stress due to friction was known (LpF) and calculation follow:

Tendon no.1		Xi (mm)	Δf_{pA} (MPa)
LpF =	13350	0	237.12
Δf_{pF} =	158.76	300	229.99
LpA =	9969.5	3563	152.39
Δf =	237.12	6825	74.79
		10088	0.00
		13350	0.00

Tendon no.2		Xi (mm)	Δf_{pA} (MPa)
LpF =	13350	0	207.46
Δf_{pF} =	121.52	300	202.00
LpA =	11395.1	3563	142.60
Δf =	207.46	6825	83.20
		10088	23.81
		13350	0.00

Tendon no.3		Xi (mm)	Δf_{pA} (MPa)
LpF =	13350	0	166.33
Δf_{pF} =	83.16	300	162.59
LpA =	13350.0	3563	121.94
Δf =	166.33	6825	81.29
		10088	40.65
		13350	0.00

Tendon no.4		Xi (mm)	Δf_{pA} (MPa)
LpF =	13350	0	87.29
Δf_{pF} =	43.65	300	85.33
LpA =	13350.0	3563	64.00
Δf =	87.29	6825	42.67
		10088	21.33
		13350	0.00

4.1.3 Elastic deformation of concrete:

Formula

$$\Delta f_{ES} = \frac{N-1}{2N}\frac{E_p}{E_{ci}} f_{cgp} \quad (5.9.5.2.3b\text{-}1)$$

In which:

Number of tendon	N =	4.00 (Tendon)
Cable modulus of elasticity	Ep =	197000.0 MPa
Concrete strength at transfer	f'_{ci} =	40.50 MPa
Unit weight of concrete	γ_c =	2450.00 kg/m3
Concrete modulus of elasticity at transfer	E_{ci} =	33185.3 MPa

Total stress of concrete in the Tendon centroid (f_{cgp}) due to prestressing force and self weigh of girder

$$f_{cgp} = \frac{F_j}{A} + \frac{F_j e^2}{I_X} - \frac{M_{DC} e}{I_X}$$

Compression force due to pretressing consider loss stress: $F_j = N^* f_{pj}{}^* A_s - A_s{}^*\Sigma(\Delta f_{pFi} + \Delta f_{pAi})$

A	Area of girder cross section
Ix	Inertia Moment of Girder cross section
e	Distance from tendon centroid to neutral line of girder section
M_{DC}	Maximum moment due to self weigh of girder at jacking

Total loss stress due to friction and Anchorage:

Section	Xi	Tendon1	Tendon2	Tendon3	Tendon4	SUM	ΣFj
		$\Delta fpF+\Delta f_{pA}$	$\Delta fpF+\Delta f_{pA}$	$\Delta fpF+\Delta f_{pA}$	$\Delta fpF+\Delta f_{pA}$	$\Sigma(\Delta fpF+\Delta f_{pA})$	
	(mm)	(MPa)	(MPa)	(MPa)	(MPa)	(MPa)	(kN)
Anchorage	0	237.12	207.46	166.33	87.29	698.20	5782.00
Support	300	231.64	203.28	163.51	85.89	684.32	5798.44
L/8	3563	173.20	158.78	133.49	70.89	536.36	5973.68
L/4	6825	128.93	124.82	110.28	58.89	422.93	6108.03
3L/8	10088	100.60	100.81	93.62	49.84	344.87	6200.48
L/2	13350	158.76	121.52	83.16	43.65	407.10	6126.78

Loss stress due to Elastic deformation of concrete

Section	Xi	Fj	A	Ix	e	M_{DC}	f_{cgp}	Δf_{ES}
	(mm)	(kN)	(mm2)	(mm4)	(mm)	kNm	(MPa)	(MPa)
Anchorage	0	5782.00	9.3E+05	1.8E+11	88.23	0.00	6.49	**14.44**
Support	300	5798.44	9.3E+05	1.8E+11	88.23	0.00	6.51	**14.48**
L/8	3563	5973.68	6.7E+05	1.6E+11	319.62	618.62	11.49	**25.58**
L/4	6825	6108.03	5.8E+05	1.5E+11	480.14	1060.48	16.40	**36.50**
3L/8	10088	6200.48	5.8E+05	1.5E+11	580.92	1325.61	19.29	**42.94**
L/2	13350	6126.78	5.8E+05	1.5E+11	614.51	1413.98	19.95	**44.41**

Total loss of prestressing force immediately - Remaining prestressing force:

Tendon1	Xi	Δf_{pF}	Δf_{pA}	Δf_{ES}	ΣΔ	F_j^1	(α)	$Fj^1*Cos(\alpha)$	$Fj^1*Sin(\alpha)$
Section	(mm)	(MPa)	(MPa)	(MPa)	(MPa)	(kN)	(rad)	(kN)	(kN)
anchorage	0	0.00	237.12	14.44	251.57	1354.28	0.1735	1333.95	233.82
Support	300	1.65	229.99	14.48	246.12	1360.73	0.1697	1341.19	229.80
L/8	3563	20.81	152.39	25.58	198.78	1416.80	0.1278	1405.25	180.58
L/4	6825	54.14	74.79	36.50	165.43	1456.30	0.0855	1450.98	124.31
3L/8	10088	100.60	0.00	42.94	143.54	1482.22	0.0428	1480.87	63.43
L/2	13350	158.76	0.00	44.41	203.18	1411.59	0.0000	1411.59	0.00

Tendon2	Xi	Δf_{pF}	Δf_{pA}	Δf_{ES}	ΣΔ	F_j^2	(α)	$Fj^2*Cos(\alpha)$	$Fj^2*Sin(\alpha)$
Section	(mm)	(MPa)	(MPa)	(MPa)	(MPa)	(kN)	(rad)	(kN)	(kN)
anchorage	0	0.00	207.46	14.44	221.90	1389.42	0.1281	1378.03	177.54
Support	300	1.29	202.00	14.48	217.77	1394.32	0.1253	1383.39	174.23
L/8	3563	16.19	142.60	25.58	184.37	1433.88	0.0942	1427.52	134.84
L/4	6825	41.62	83.20	36.50	161.32	1461.17	0.0629	1458.28	91.83
3L/8	10088	77.00	23.81	42.94	143.75	1481.98	0.0315	1481.25	46.64
L/2	13350	121.52	0.00	44.41	165.94	1455.70	0.0000	1455.70	0.00

Tendon3	Xi	Δf_{pF}	Δf_{pA}	Δf_{ES}	ΣΔ	F_j^3	(α)	$Fj^3*Cos(\alpha)$	$Fj^3*Sin(\alpha)$
Section	(mm)	(MPa)	(MPa)	(MPa)	(MPa)	(kN)	(rad)	(kN)	(kN)
anchorage	0	0.00	166.33	12.84	179.17	1440.03	0.0822	1435.17	118.25
Support	300	0.92	162.59	12.88	176.39	1443.33	0.0804	1438.67	115.88
L/8	3563	11.55	121.94	22.74	156.23	1467.20	0.0603	1464.53	88.47
L/4	6825	28.98	81.29	32.44	142.72	1483.20	0.0403	1482.00	59.68
3L/8	10088	52.97	40.65	38.17	131.79	1496.15	0.0201	1495.84	30.12
L/2	13350	83.16	0.00	39.48	122.64	1506.98	0.0000	1506.98	0.00

Tendon4	Xi	Δf_{pF}	Δf_{pA}	Δf_{ES}	ΣΔ	F_j^4	(α)	$Fj^4*Cos(\alpha)$	$Fj^4*Sin(\alpha)$
Section	(mm)	(MPa)	(MPa)	(MPa)	(MPa)	(kN)	(rad)	(kN)	(kN)
anchorage	0	0.00	87.29	9.63	96.92	1537.44	0.0359	1536.45	55.24
Support	300	0.56	85.33	9.66	95.55	1539.07	0.0351	1538.12	54.06
L/8	3563	6.89	64.00	17.05	87.94	1548.08	0.0264	1547.54	40.79
L/4	6825	16.23	42.67	24.33	83.23	1553.67	0.0176	1553.43	27.30
3L/8	10088	28.51	21.33	28.63	78.47	1559.30	0.0088	1559.24	13.70
L/2	13350	43.65	0.00	29.61	73.26	1565.47	0.0000	1565.47	0.00

SUM 1to4	Xi	ΣF_j	Fj*Cos(α)	Fj*Sin(α)	$e_{cáp}$	$M_j = \Sigma F_j Cos(\alpha)*e_{cáp}$
Section	(mm)	(kN)	(kN)	(kN)	(mm)	(kNm)
anchorage	0	5721.18	5683.59	584.86	88.23	501.48
Support	0	5737.45	5701.37	573.97	88.23	503.04
L/8	0	5865.96	5844.84	444.69	319.62	1868.15
L/4	0	5954.33	5944.69	303.12	480.14	2854.27
3L/8	0	6019.66	6017.20	153.89	580.92	3495.51
L/2	0	5939.75	5939.75	0.00	614.51	3650.06

4.2. Loss of prestressing force at service stage (time - dependent losses):

4.2.1 Loss of prestress due to Shrinkage:

Formula: $\Delta fp_{SH} = (93 - 0.85*H)$

Relative humidity of environment H = 80.00 %

$\Delta fp_{SH} =$ **25.00 (MPa)**

4.2.2 Loss of prestress due to Creep:

Formula $\Delta fp_{CR} = 12.0\ f_{cgp} - 7.0 * \Delta f_{cdp}$

In which:

f_{cgp} Stress in concrete at tendons centroid (f_{cgp}) due to prestressing tendon and self weigh of girder

Δf_{cdp} Stess at tendons centroid changes due to permanent load, except dead load action at transfer

Section	Xi		Interior Girder		Exterior Girder	
		f_{cgp}	Δf_{cdp}	Δfp_{CR}	Δf_{cdp}	Δfp_{CR}
	(m)	(MPa)	(MPa)	(MPa)	(MPa)	(MPa)
Support	0.00	6.51	0.00	**78.08**	0.00	**78.08**
L/8	3.26	11.49	0.99	**130.96**	1.80	**125.29**
L/4	6.53	16.40	5.01	**161.64**	4.56	**164.82**
3L/8	9.79	19.29	5.35	**194.00**	6.73	**184.38**
L/2	13.05	19.95	8.31	**181.21**	7.54	**186.63**

4.2.3 Loss of prestress due to Relaxation:

(a) At transfer:

Formula: $$\Delta f_{pR1} = \frac{\log(24t)}{40}\left[\frac{f_{pj}}{f_{py}} - 0.55\right]f_{pj}$$

In which:

t : time estimated in days from stressing to transfer t = 0.00 days

fpy : Specified yeild strength of prestressing steel f_{py} = 1674.00 (MPa)

fpj : Initial stress in the tendon at the end of stressing

Section	Xi	fpj	Δfp_{R1}
	(m)	(MPa)	(MPa)
Support	0.00	1380.52	**0.00**
L/8	3.26	1369.42	**0.00**
L/4	6.53	1358.50	**0.00**
3L/8	9.79	1352.06	**0.00**
L/2	13.05	1350.59	**0.00**

(b) After Transfer:

Formula: $\Delta fp_{R2} = 30\%*(138 - 0.3\ \Delta fp_F - 0.4\ \Delta f_{ES} - 0.2\ (\Delta fp_{SH} + \Delta fp_{CR}))$

Interior Girder						
Section	Xi	Δf_{pF}	Δf_{pES}	Δfp_{SH}	Δfp_{CR}	Δfp_{R2}
	(m)	(MPa)	(MPa)	(MPa)	(MPa)	(MPa)
Support	0.00	1.10	14.48	25.00	19.52	**36.89**
L/8	3.26	13.86	25.58	25.00	32.74	**33.62**
L/4	6.53	35.24	36.50	25.00	40.41	**29.92**
3L/8	9.79	64.77	42.94	25.00	48.50	**26.01**
L/2	13.05	101.77	44.41	25.00	45.30	**22.69**

Exterior Girder						
Section	Xi	Δf_{pF}	Δf_{pES}	Δfp_{SH}	Δfp_{CR}	Δfp_{R2}
	(m)	(MPa)	(MPa)	(MPa)	(MPa)	(MPa)
Support	0.00	1.10	14.48	25.00	19.52	**36.89**
L/8	3.26	13.86	25.58	25.00	31.32	**33.70**
L/4	6.53	35.24	36.50	25.00	41.20	**29.88**
3L/8	9.79	64.77	42.94	25.00	46.09	**26.15**
L/2	13.05	101.77	44.41	25.00	46.66	**22.61**

TOTAL LOSS STRESS AT SERVICE STAGE

Interior Girder						
Setion	Xi	Δfp_{SH}	Δfp_{CR}	Δfp_{R1}	Δfp_{R2}	Sum
	(m)	(MPa)	(MPa)	(MPa)	(MPa)	(MPa)
Support	0.00	25.00	78.08	0.00	36.89	**139.97**
L/8	3.26	25.00	130.96	0.00	33.62	**189.58**
L/4	6.53	25.00	161.64	0.00	29.92	**216.57**
3L/8	9.79	25.00	194.00	0.00	26.01	**245.01**
L/2	13.05	25.00	181.21	0.00	22.69	**228.90**

Exterior Girder						
Section	Xi	Δfp_{SH}	Δfp_{CR}	Δfp_{R1}	Δfp_{R2}	Sum
	(m)	(MPa)	(MPa)	(MPa)	(MPa)	(MPa)
Support	0.00	25.00	78.08	0.00	36.89	**139.97**
L/8	3.26	25.00	125.29	0.00	33.70	**184.00**
L/4	6.53	25.00	164.82	0.00	29.88	**219.69**
3L/8	9.79	25.00	184.38	0.00	26.15	**235.53**
L/2	13.05	25.00	186.63	0.00	22.61	**234.24**

4.3. Total Prestressing force consider loss in the service stage:
Interior Girder

Tendon1	Xi	$\Sigma\Delta_{pT}$	F_j^1	(α)	$Fj^1*Cos(\alpha)$	$Fj^1*Sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	386.09	1194.95	0.1697	1177.79	201.80
L/8	3.26	388.36	1192.26	0.1278	1182.54	151.96
L/4	6.53	382.00	1199.80	0.0855	1195.42	102.41
3L/8	9.79	388.55	1192.04	0.0428	1190.94	51.01
L/2	13.05	432.08	1140.48	0.0000	1140.48	0.00

Tendon2	Xi	$\Sigma\Delta_{pT}$	F_j^2	(α)	$Fj^2*Cos(\alpha)$	$Fj^2*Sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	357.74	1228.54	0.1281	1218.46	156.99
L/8	3.26	373.95	1209.34	0.1253	1199.86	151.11
L/4	6.53	377.89	1204.66	0.0942	1199.33	113.29
3L/8	9.79	388.76	1191.79	0.0629	1189.44	74.90
L/2	13.05	394.84	1184.59	0.0315	1184.00	37.28

Tendon3	Xi	$\Sigma\Delta_{pT}$	F_j^3	(α)	$Fj^3*Cos(\alpha)$	$Fj^3*Sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	319.14	1274.25	0.0804	1270.14	102.30
L/8	3.26	365.97	1218.79	0.0603	1216.57	73.49
L/4	6.53	372.79	1210.70	0.0403	1209.72	48.72
3L/8	9.79	387.73	1193.01	0.0201	1192.77	24.02
L/2	13.05	360.69	1225.03	0.0000	1225.03	0.00

Tendon4	Xi	$\Sigma\Delta_{pT}$	F_j^4	(α)	$Fj^4*Cos(\alpha)$	$Fj^4*Sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	236.89	1371.66	0.0351	1370.82	48.18
L/8	3.26	285.13	1314.53	0.0264	1314.08	34.64
L/4	6.53	304.51	1291.57	0.0176	1291.38	22.69
3L/8	9.79	328.23	1263.48	0.0088	1263.43	11.10
L/2	13.05	307.37	1288.19	0.0000	1288.19	0.00

SUM 1to4	Xi	ΣF_j	$Fj*Cos(\alpha)$	$Vp=Fj*Sin(\alpha)$	e_{cable}	$M_j = \Sigma F_j Cos(\alpha)*e_{cáp}$
Section	(mm)	(kN)	(kN)	(kN)	(m)	(kNm)
Support	0.00	5069.40	5037.21	509.27	0.09	444.4
L/8	0.00	4934.92	4913.04	411.21	0.32	1570.3
L/4	0.00	4906.74	4895.84	287.11	0.48	2350.7
3L/8	0.00	4840.32	4836.58	161.04	0.58	2809.7
L/2	0.00	4838.30	4837.71	37.28	0.61	2972.8

Exterior Girder

Tendon1	Xi	$\Sigma\Delta_{pT}$	F_j^1	(α)	$Fj^1 \cdot Cos(\alpha)$	$Fj^1 \cdot Sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	386.09	1194.95	0.1697	1177.79	201.80
L/8	0.00	382.78	1198.88	0.1278	1189.10	152.81
L/4	0.00	385.13	1196.09	0.0855	1191.73	102.10
3L/8	0.00	379.07	1203.26	0.0428	1202.16	51.50
L/2	0.00	437.42	1134.16	0.0000	1134.16	0.00

Tendon2	Xi	$\Sigma\Delta_{pT}$	F_j^2	(α)	$Fj^2 \cdot Cos(\alpha)$	$Fj^2 \cdot Sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	357.74	1228.54	0.1281	1218.46	156.99
L/8	0.00	368.36	1215.95	0.1253	1206.42	151.94
L/4	0.00	381.02	1200.96	0.0942	1195.64	112.94
3L/8	0.00	379.28	1203.02	0.0629	1200.64	75.61
L/2	0.00	400.18	1178.27	0.0315	1177.68	37.08

Tendon3	Xi	$\Sigma\Delta_{pT}$	F_j^3	(α)	$Fj^3 \cdot Cos(\alpha)$	$Fj^3 \cdot Sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	319.14	1274.25	0.0804	1270.14	102.30
L/8	0.00	360.38	1225.40	0.0603	1223.17	73.89
L/4	0.00	375.92	1207.00	0.0403	1206.02	48.57
3L/8	0.00	378.25	1204.24	0.0201	1203.99	24.24
L/2	0.00	366.03	1218.71	0.0000	1218.71	0.00

Tendon4	Xi	$\Sigma\Delta_{pT}$	F_j^4	(α)	$Fj^4 \cdot Cos(\alpha)$	$Fj^4 \cdot Sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	236.89	1371.66	0.0351	1370.82	48.18
L/8	0.00	279.54	1321.15	0.0264	1320.69	34.81
L/4	0.00	307.64	1287.87	0.0176	1287.67	22.63
3L/8	0.00	318.76	1274.70	0.0088	1274.65	11.20
L/2	0.00	312.70	1281.87	0.0000	1281.87	0.00

SUM 1to4	Xi	ΣF_j	$Fj \cdot Cos(\alpha)$	$Vp=Fj \cdot Sin(\alpha)$	e_{caple}	$M_j = \Sigma F_j Cos(\alpha) \cdot e_{cap}$
Section	(mm)	(kN)	(kN)	(kN)	(m)	(kNm)
Support	0.00	5069.40	5037.21	509.27	0.09	444.4
L/8	0.00	4961.37	4939.38	413.45	0.32	1578.7
L/4	0.00	4891.92	4881.06	286.23	0.48	2343.6
3L/8	0.00	4885.22	4881.45	162.55	0.58	2835.7
L/2	0.00	4813.01	4812.43	37.08	0.61	2957.3

5. FIBRE STRESS CHECK:

Formula:

Top fibre: $f_{ti} = \frac{F_i}{A} - \frac{F_i e}{S_t} + \frac{M_{DC}}{S_t}$ Bottom fibre $f_{bi} = \frac{F_i}{A} + \frac{F_i e}{S_b} - \frac{M_{DC}}{S_b}$

Note (+) : Compression tresses ; (-) Tension stresses

Concrete strength at transfer fci' = 0.9 fc = 40.50 MPa

Comprresion stress Limit at transfer 0.6 fci' = 24.30 MPa

Tension stress Limit at transfer 0.25 SQRT(fci') < 1.38 = -1.38 MPa (5.9.4.1.2-1)

Setion	Xi	A	St	Sb	Fj'*Cos(α)	e	M_{DC}	f_{ti}	f_{bi}	Kiểm tra	
	(mm)	(mm2)	(mm3)	(mm3)	(kN)	(mm)	(kNm)	(MPa)	(MPa)	f_{ti}	f_{bi}
irder en	0	9.27E+05	2.44E+08	2.32E+08	**5683.59**	88.23	0.00	4.07	8.29	OK	OK
Support	300	9.27E+05	2.44E+08	2.32E+08	**5701.37**	88.23	0.00	4.08	8.32	OK	OK
L/8	3563	6.70E+05	2.17E+08	2.09E+08	**5844.84**	319.62	618.62	2.97	14.69	OK	OK
L/4	6825	5.77E+05	2.09E+08	2.05E+08	**5944.69**	480.14	1060.48	1.71	19.07	OK	OK
3L/8	10088	5.77E+05	2.09E+08	2.05E+08	**6017.20**	580.92	1325.61	0.03	21.04	OK	OK
L/2	13350	5.77E+05	2.09E+08	2.05E+08	**5939.75**	614.51	1413.98	-0.42	21.23	OK	OK

5.2 Stress check during contruction the deck:

5.2.1 Increase load:

Exterior Diaphragms beam DC_{dn1} = 46.80 (kN)

Interior Diaphragms beam DC_{dn1} = 21.90 (kN)

Precast plank DC_{VK} = 3.74 (kN/m)

Wet concrete of deck DC_{mc} = 13.30 (kN/m)

5.2.2 Stress check:

Compression strength of concrete fc = 45.00 MPa

Compression stress limit 0.45 fc = 20.25 MPa (5.9.4.2.1-1)

Tension stress limit 0.5 SQRT(fc) = -3.35 MPa (5.9.4.2.2-1)

Setion	Xi	A	St	Sb	Fi	e	M_{DC}	f_{ti}	f_{bi}	Kiểm tra	
	(mm)	(mm2)	(mm3)	(mm3)	(kN)	(mm)	(kNm)	(MPa)	(MPa)	f_{ti}	f_{bi}
irder en	0	9.27E+05	2.44E+08	2.32E+08	5683.59	88.23	0.00	4.07	8.29	OK	OK
Support	300	9.27E+05	2.44E+08	2.32E+08	5701.37	88.23	0.00	4.08	8.32	OK	OK
L/8	3563	6.70E+05	2.17E+08	2.09E+08	5844.84	319.62	1780.21	8.32	9.14	OK	OK
L/4	6825	5.77E+05	2.09E+08	2.05E+08	5944.69	480.14	2202.48	7.18	13.49	OK	OK
3L/8	10088	5.77E+05	2.09E+08	2.05E+08	6017.20	580.92	2753.10	6.88	14.06	OK	OK
L/2	13350	5.77E+05	2.09E+08	2.05E+08	5939.75	614.51	2936.64	6.88	13.79	OK	OK

5.3 Stress check at the top fibre of Girder - Service stage :

5.3.1 Due to prestressing tendon and self weigh of girder - Service limit stage I:

Compression Stress Limit: 0.45 fc = 20.25 MPa (5.9.4.2.1-1)

Tension Stress Limit: - 0.5*SQRT(fc) = -3.35 MPa

$$f_f = \frac{P_{pe}}{A} - \frac{P_{pe}e_c}{S_t} + \frac{M_g + M_s}{S_t} + \frac{M_{SDL}}{S_{tg}}$$

Interior Girder

Setion	Xi	A	S_t	S_{tg}	P_{pe}	P_{pe}*e_c	$M_g + M_s$	M_{SDL}	f_t	Check
	(mm)	(mm2)	(mm3)	(mm3)	(kN)	(kNm)	(kNm)	(kNm)	(MPa)	f_t
Support	300	9.27E+05	2.44E+08	9.05E+08	5037.21	444.44	0.00	0.00	3.608	OK
L/8	3563	6.70E+05	2.17E+08	9.43E+08	4913.04	1570.32	1284.78	342.04	6.379	OK
L/4	6825	5.77E+05	2.09E+08	9.75E+08	4895.84	2350.68	2202.48	586.36	8.382	OK
3L/8	10088	5.77E+05	2.09E+08	9.75E+08	4836.58	2809.66	2753.10	732.95	8.869	OK
L/2	13350	5.77E+05	2.09E+08	9.75E+08	4837.71	2972.84	2936.64	781.82	9.018	OK

Exterior Girder

Setion	Xi	A	S_t	S_{tg}	P_{pe}	P_{pe}*e_c	$M_g + M_s$	M_{SDL}	f_t	Check
	(mm)	(mm2)	(mm3)	(mm3)	(kN)	(kNm)	(kNm)	(kNm)	(MPa)	f_t
Support	300	9.27E+05	2.44E+08	9.04E+08	5037.21	444.44	0.00	0.00	3.608	OK
L/8	3563	6.70E+05	2.17E+08	9.41E+08	4939.38	1578.74	1199.41	342.04	5.988	OK
L/4	6825	5.77E+05	2.09E+08	9.73E+08	4881.06	2343.58	2056.13	586.36	7.689	OK
3L/8	10088	5.77E+05	2.09E+08	9.73E+08	4881.45	2835.73	2570.17	732.95	7.945	OK
L/2	13350	5.77E+05	2.09E+08	9.73E+08	4812.43	2957.30	2741.51	781.82	8.115	OK

5.3.2 Due to 1/2 (Prestressing tendon + self weigh of girder) and Live load - Service limit stage I:

Compression Stress Limit: 0.40 fc = 18.00 MPa (5.9.4.2.1-1)

Tension Stress Limit: - 0.5*SQRT(fc) = -3.35 MPa

$$f_t = 0.5\left(\frac{P_{pe}}{A} - \frac{P_{pe}e_c}{S_t} + \frac{M_g + M_s}{S_t} + \frac{M_{SDL}}{S_{tg}}\right) + \frac{M_{LL}}{S_{tg}}$$

Interior Girder

Setion	Xi (mm)	A (mm2)	S_t (mm3)	S_{tg} (mm3)	P_{pe} (kN)	$P_{pe}*e_c$ (kNm)	$M_g + M_s$ (kNm)	M_{SDL} (kNm)	M_{LL} (kNm)	f_t (MPa)	Check f_t
Support	300	9.27E+05	2.44E+08	9.05E+08	5037.21	444.44	0.00	0.00	0.00	1.804	OK
L/8	3563	6.70E+05	2.17E+08	9.43E+08	4913.04	1570.32	1284.78	342.04	1019.60	4.271	OK
L/4	6825	5.77E+05	2.09E+08	9.75E+08	4895.84	2350.68	2202.48	586.36	1716.98	5.952	OK
3L/8	10088	5.77E+05	2.09E+08	9.75E+08	4836.58	2809.66	2753.10	732.95	2109.75	6.598	OK
L/2	13350	5.77E+05	2.09E+08	9.75E+08	4837.71	2972.84	2936.64	781.82	2215.53	6.781	OK

Exterior Girder

Setion	Xi (mm)	A (mm2)	S_t (mm3)	S_{tg} (mm3)	P_{pe} (kN)	$P_{pe}*e_c$ (kNm)	$M_g + M_s$ (kNm)	M_{SDL} (kNm)	M_{LL} (kNm)	f_t (MPa)	Check f_t
Support	300	927266.7	2.44E+08	9.04E+08	5037.21	444.44	0.00	0.00	0.00	1.804	OK
L/8	3563	670320.0	2.17E+08	9.41E+08	4939.38	1578.74	1199.41	342.04	1166.28	4.233	OK
L/4	6825	576600.0	2.09E+08	9.73E+08	4881.06	2343.58	2056.13	586.36	1963.98	5.862	OK
3L/8	10088	576600.0	2.09E+08	9.73E+08	4881.45	2835.73	2570.17	732.95	2413.26	6.452	OK
L/2	13350	576600.0	2.09E+08	9.73E+08	4812.43	2957.30	2741.51	781.82	2534.24	6.661	OK

5.3.3 Due to prestressing tendon + self weigh of girder + live load - Service limit stage I:

Compression Stress Limit: 0.60 fc = 27.00 MPa (5.9.4.2.1-1)

Tension Stress Limit: - 0.5*SQRT(fc) = -3.35 MPa

$$f_t = \left(\frac{P_{pe}}{A} - \frac{P_{pe}e_c}{S_t} + \frac{M_g + M_s}{S_t} + \frac{M_{SDL}}{S_{tg}}\right) + \frac{M_{LL}}{S_{tg}}$$

Interior Girder

Setion	Xi (mm)	A (mm2)	S_t (mm3)	S_{tg} (mm3)	P_{pe} (kN)	$P_{pe}*e_c$ (kNm)	$M_g + M_s$ (kNm)	M_{SDL} (kNm)	M_{LL} (kNm)	f_t (MPa)	Check f_t
Support	300	9.27E+05	2.44E+08	9.05E+08	5037.21	444.44	0.00	0.00	0.00	3.608	OK
L/8	3563	6.70E+05	2.17E+08	9.43E+08	4913.04	1570.32	1284.78	342.04	1019.60	7.461	OK
L/4	6825	5.77E+05	2.09E+08	9.75E+08	4895.84	2350.68	2202.48	586.36	1716.98	10.142	OK
3L/8	10088	5.77E+05	2.09E+08	9.75E+08	4836.58	2809.66	2753.10	732.95	2109.75	11.032	OK
L/2	13350	5.77E+05	2.09E+08	9.75E+08	4837.71	2972.84	2936.64	781.82	2215.53	11.290	OK

Exterior Girder

Setion	Xi (mm)	A (mm2)	S_t (mm3)	S_{tg} (mm3)	P_{pe} (kN)	$P_{pe}*e_c$ (kNm)	$M_g + M_s$ (kNm)	M_{SDL} (kNm)	M_{LL} (kNm)	f_t (MPa)	Check f_t
Support	300	9.27E+05	2.44E+08	9.04E+08	5037.21	444.44	0.00	0.00	0.00	3.608	OK
L/8	3563	6.70E+05	2.17E+08	9.41E+08	4939.38	1578.74	1199.41	342.04	1166.28	7.227	OK
L/4	6825	5.77E+05	2.09E+08	9.73E+08	4881.06	2343.58	2056.13	586.36	1963.98	9.707	OK
3L/8	10088	5.77E+05	2.09E+08	9.73E+08	4881.45	2835.73	2570.17	732.95	2413.26	10.425	OK
L/2	13350	5.77E+05	2.09E+08	9.73E+08	4812.43	2957.30	2741.51	781.82	2534.24	10.718	OK

5.4 Stress check at the top fibre of deck - Service stage:

5.4.1 Due to additional load (dead load part 2) - Service limit stage I:

Compression Stress Limit: 0.45 fc = 15.75 MPa (5.9.4.2.1-1)

$$f_t = \frac{M_{SDL}}{S_{tc}}$$

Setion	Xi	MSDL (kNm)		S_{tc} (mm3)		f_t (MPa)		Check	
	(mm)	in.Girder	Ex.Girder	in.Girder	Ex.Girder	in.Girder	Ex.Girder	in.Girder	Ex.Girder
Support	300.00	0.00	0.00	5.3E+08	5.29E+08	0.000	0.000	OK	OK
L/8	3562.50	342.04	342.04	5.2E+08	5.22E+08	0.654	0.655	OK	OK
L/4	6825.00	586.36	586.36	5.3E+08	5.25E+08	1.115	1.116	OK	OK
3L/8	10087.50	732.95	732.95	5.3E+08	5.25E+08	1.393	1.395	OK	OK
L/2	13350.00	781.82	781.82	5.3E+08	5.25E+08	1.486	1.488	OK	OK

5.4.2 Due to additional load (dead load part 2) and live load - Service limit stage I:

Compression Stress Limit: 0.6 fc = 21.00 MPa (5.9.4.2.1-1)

$$f_{tc} = \frac{M_{SDL} + M_{LL}}{S_{tc}}$$

Setion	Xi	MSDL + MLL (kNm)		S_{tc} (mm3)		f_t (MPa)		Check	
	(mm)	in.Girder	Ex.Girder	in.Girder	Ex.Girder	in.Girder	Ex.Girder	in.Girder	Ex.Girder
Support	300.00	0.00	0.00	5.3E+08	5.29E+08	0.000	0.000	OK	OK
L/8	3562.50	1361.64	1508.32	5.2E+08	5.22E+08	2.604	2.888	OK	OK
L/4	6825.00	2303.34	2550.34	5.3E+08	5.25E+08	4.378	4.854	OK	OK
3L/8	10087.50	2842.71	3146.21	5.3E+08	5.25E+08	5.403	5.988	OK	OK
L/2	13350.00	2997.34	3316.06	5.3E+08	5.25E+08	5.697	6.311	OK	OK

5.5 Stress check at the bottom fibre of girder - Service III (stage III):

Compression Stress Limit: 0.45 fc = 20.25 MPa (5.9.4.2.1-1)

Tension Stress Limit: - 0.5*SQRT(fc) = -3.35 MPa (5.9.4.2.1-1)

$$f_b = \frac{P_{pe}}{A} + \frac{P_{pe}e_c}{S_b} - \frac{M_g + M_s}{S_b} - \frac{M_{SDL} + 0.8M_{LL}}{S_{bc}}$$

Interior Girder

Setion	Xi	A	S_b	S_{bc}	P_{pe}	P_{pe}*e_c	$M_g + M_s$	M_{SDL}	M_{LL}	f_b	Check
	(mm)	(mm2)	(mm3)	(mm3)	(kN)	(kNm)	(kNm)	(kNm)	(kNm)	(MPa)	f_b
Support	300	9.27E+05	2.32E+08	3.78E+08	5037.21	444.44	0.00	0.00	0.00	7.348	OK
L/8	3563	6.70E+05	2.09E+08	3.20E+08	4913.04	1570.32	1284.78	342.04	1019.60	5.080	OK
L/4	6825	5.77E+05	2.05E+08	3.01E+08	4895.84	2350.68	2202.48	586.36	1716.98	2.693	OK
3L/8	10088	5.77E+05	2.05E+08	3.01E+08	4836.58	2809.66	2753.10	732.95	2109.75	0.610	OK
L/2	13350	5.77E+05	2.05E+08	3.01E+08	4837.71	2972.84	2936.64	781.82	2215.53	0.068	OK

Exterior Girder

Setion	Xi	A	S_b	S_{bc}	P_{pe}	P_{pe}*e_c	$M_g + M_s$	M_{SDL}	M_{LL}	f_b	Check
	(mm)	(mm2)	(mm3)	(mm3)	(kN)	(kNm)	(kNm)	(kNm)	(kNm)	(MPa)	f_b
Support	300	9.27E+05	2.32E+08	3.78E+08	5037.21	444.44	0.00	0.00	0.00	7.348	OK
L/8	3563	6.70E+05	2.09E+08	3.20E+08	4939.38	1578.74	1199.41	342.04	1166.28	5.200	OK
L/4	6825	5.77E+05	2.05E+08	3.00E+08	4881.06	2343.58	2056.13	586.36	1963.98	2.689	OK
3L/8	10088	5.77E+05	2.05E+08	3.00E+08	4881.45	2835.73	2570.17	732.95	2413.26	0.899	OK
L/2	13350	5.77E+05	2.05E+08	3.00E+08	4812.43	2957.30	2741.51	781.82	2534.24	0.051	OK

5.6 Stress check at the bottom fibre of girder - Service I (stage III):

Compression Stress Limit: 0.45 fc = 20.25 MPa (5.9.4.2.1-1)

Tension Stress Limit: - 0.5*SQRT(fc) = -3.35 MPa (5.9.4.2.1-1)

$$f_b = \frac{P_{pe}}{A} + \frac{P_{pe}e_c}{S_b} - \frac{M_g + M_s}{S_b} - \frac{M_{SDL} + M_{LL}}{S_{bc}}$$

Interior Girder

Setion	Xi	A	S_b	S_{bc}	P_{pe}	P_{pe}*e_c	$M_g + M_s$	M_{SDL}	M_{LL}	f_b	Check
	(mm)	(mm2)	(mm3)	(mm3)	(kN)	(kNm)	(kNm)	(kNm)	(kNm)	(MPa)	f_b
Support	300	9.27E+05	2.32E+08	3.78E+08	5037.21	444.44	0.00	0.00	0.00	7.348	OK
L/8	3563	6.70E+05	2.09E+08	3.20E+08	4913.04	1570.32	1284.78	342.04	1019.60	4.443	OK
L/4	6825	5.77E+05	2.05E+08	3.01E+08	4895.84	2350.68	2202.48	586.36	1716.98	1.551	OK
3L/8	10088	5.77E+05	2.05E+08	3.01E+08	4836.58	2809.66	2753.10	732.95	2109.75	-0.794	OK
L/2	13350	5.77E+05	2.05E+08	3.01E+08	4837.71	2972.84	2936.64	781.82	2215.53	-1.406	OK

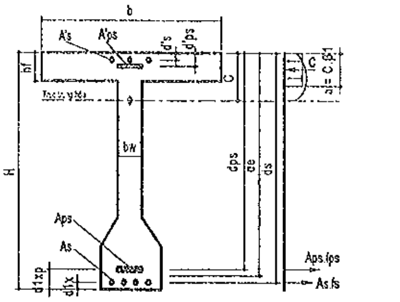
Exterior Girder

Setion	Xi	A	S_b	S_{bc}	P_{pe}	P_{pe}*e_c	$M_g + M_s$	M_{SDL}	M_{LL}	f_b	Check
	(mm)	(mm2)	(mm3)	(mm3)	(kN)	(kNm)	(kNm)	(kNm)	(kNm)	(MPa)	f_b
Support	300	9.27E+05	2.32E+08	3.78E+08	5037.21	444.44	0.00	0.00	0.00	7.348	OK
L/8	3563	6.70E+05	2.09E+08	3.20E+08	4939.38	1578.74	1199.41	342.04	1166.28	4.471	OK
L/4	6825	5.77E+05	2.05E+08	3.00E+08	4881.06	2343.58	2056.13	586.36	1963.98	1.382	OK
3L/8	10088	5.77E+05	2.05E+08	3.00E+08	4881.45	2835.73	2570.17	732.95	2413.26	-0.708	OK
L/2	13350	5.77E+05	2.05E+08	3.00E+08	4812.43	2957.30	2741.51	781.82	2534.24	-1.636	OK

REINFORCEMENT OF GIRDER CHECKING - STRENGTH LOAD COMBINATION

MATERIALS

Sign	Parameters	Unit	Value
	NORMAL CONCRETE		
f'c	Compressive Strength of concrete at 28 days	Mpa	45
Ec	Modulus of Elasticity	Mpa	33915
fr	Modulus of Rupture	Mpa	4.2
gc	Unit weight of concrete	kN/m3	24.0
	PRESTRESSING STEEL		
fpu	Tensile strength of prestressing steel	Mpa	1860
fpy	Yield strength of prestressing steel	Mpa	1674
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	Support	L/8	L/4	3L/8	L/2
				Section				
	INTERNAL FORCES AT SECTION							
	Combination			Strength	Strength	Strength	Strength	Strength
Qu	Shear		kN	1451	1151	856	567	284
Mu	Flexural Moment		kNm	0	4009	6811	8441	8934
Nu	Axial load		kN					
Tu	Torsional Moment		kNm					
	6.1 FLEXURAL MOMENT CHECKING							
H	Section height		m	1.700	1.700	1.700	1.700	1.700
d's	Dis. From comp. fiber to centroid of comp. Reinf		m	0.062	0.062	0.062	0.062	0.062
d1x	Dis. From tens. fiber to centroid of tension Reinf		m	0.061	0.061	0.061	0.061	0.061
	Cover to reinf		m	0.040	0.040	0.040	0.040	0.040
ds	Dis. From comp. fiber to centroid of tension Reinf		m	1.639	1.639	1.639	1.639	1.639
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	0.680	0.445	0.277	0.176	0.143
dps	Dis. From comp. fiber to centroid of tension prestressing steel		m	1.020	1.255	1.423	1.524	1.558
b	Width of the compression face of member		m	2.249	2.249	2.249	2.249	2.249
bw	Web width or diameter of a circular section		m	0.600	0.320	0.200	0.200	0.200
hf	Compression flange depth		m	0.200	0.200	0.200	0.200	0.200
Iz	Moment of inertia of section		m4	0.396	0.355	0.341	0.341	0.341
Amc	Section area		m2	1.393	1.136	1.042	1.042	1.042
	Steel choice							
Aps	Tension prestressing steel	P.S type		12 T12.7	12 T12.7	12 T12.7	12 T12.7	12 T12.7
		Number	tendons	4	4	4	4	4
		Area	m2	0.00474	0.00474	0.00474	0.00474	0.00474
A'ps	Compression prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	0 T00.0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
As	Tension Reinforcement	Number	bars	6	6	6	6	6
		Diameter	mm	16	16	16	16	16
		Area	m2	0.00114	0.00114	0.00114	0.00114	0.00114
A's	Compression Reinforcement	Number	bars	4	4	4	4	4
		Diameter	mm	12	12	12	12	12
		Area	m2	0.00045	0.00045	0.00045	0.00045	0.00045
A'c	Shear reinforcement	Number	bars	2	2	2	2	2
		Diameter	mm	14	14	14	14	14
		Area	m2	0.00030	0.00030	0.00030	0.00030	0.00030
f	Resistance factors for flexure		5.5.4.2	0.90	0.90	0.90	0.90	0.90
fv	Resistance factors for shear			0.90	0.90	0.90	0.90	0.90
fn	Resistance factors for axial force			1.00	1.00	1.00	1.00	1.00
b1	Stress block factor			0.729	0.729	0.729	0.729	0.729
c	Dis. Between centroid and top fiber		m	0.140	0.141	0.141	0.141	0.141
	For T section behavior		m	-0.005	-0.153	-0.319	-0.324	-0.326
	For rectangular section behavior		m	0.140	0.141	0.141	0.141	0.141
fpe	Effective stress in the prestressing steel after losses		Mpa	1194.95	1192.26	1199.80	1191.79	1140.48
fps	Aver. stress in pres. steel at the time for which the nominal resistance		Mpa	1789	1802	1808	1812	1813
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	0.102	0.102	0.103	0.103	0.103
de	Corresp. effective depth from extreme comp. fiber						
	to centroid of tensile force in the tensile reinf.	m	1.052	1.275	1.434	1.530	1.582
Mn	Nominal resistance	kNm	8483	10473	11918	12805	13101
Mr	Factored reistance	kNm	7634	9426	10727	11525	11791
Mu	Flexual moment	kNm	0	4009	6811	8441	8934
(5.7.3.2)	**Flexural moment Checking**		OK	OK	OK	OK	OK
	Limits for reinforcement						
c/de	Maximum reinforcement		0.13	0.11	0.10	0.09	0.09
	Maximum reinforcement Checking	<= 0.42	OK	OK	OK	OK	OK
r min	Mimimum reinforcement		0.08%	0.10%	0.11%	0.11%	0.11%
	Mimimum reinforcement Checking for RC	0.34%	N.a	N.a	N.a	N.a	N.a
1.2*Mcr	Craking moment	kNm	1287	1154	1110	1110	1110
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**		OK	OK	OK	OK	OK
(5.8.3.5)	Tensile force in steel should be satisfied - F_yc	kN	1582	4894	6415	6919	6852
	Checking As.fy+Aps.fps >= F_yc		Ok	Ok	Ok	Ok	Ok
(5.7.3.4)	**Conctrol of craking by distr. of reinf for RC member- Check?**		No	No	No	No	No
	Existing condition for structrure	1,2 or 3	1	1	1	1	1
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.058	0.058	0.058	0.058	0.058
Z	Crack width parameter	N/mm	30000	30000	30000	30000	30000
A	Area of concr. with same centroid as tens. Reinf	m2	0.043	0.043	0.043	0.043	0.043
fsa	Value	Mpa	220	220	220	220	220
0.6*fy		Mpa	240	240	240	240	240
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	220	220	220	220	220
x	Dist. From compression fiber to centroid	m	-	-	-	-	-
J.d	Arm	m	-	-	-	-	-
Icr	Moment of inertia of the cracked section	m4	-	-	-	-	-
fs	Tensile stress in reinforcement fs = Msls / (As*J.d)	Mpa	-	-	-	-	-
	Checking for control cracking fs<fsa		N.a	N.a	N.a	N.a	N.a
(5.10.8.2)	**Shrinkage and temperature Reinfocement (side distribution)**						
Areq	Area of required reinf	m2	0.00033	0.00027	0.00025	0.00025	0.00025
	Distribution on sides 7 D12	m2	0.00079	0.00079	0.00079	0.00079	0.00079
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45	0.45
	Checking		OK	OK	OK	OK	OK
	6.2 SHEAR FORCE CHECKING						
β	Factor indicating diag. cracked concr. to tension		6.8	6.0	4.8	3.0	2.5
θ	Angle of inclination of diagonal compressive	degree	27.00	27.00	26.80	28.74	29.51
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	0.600	0.320	0.200	0.200	0.200
dv	Effective shear depth	m	1.224	1.224	1.383	1.478	1.510
	(de - a/2)	m	1.001	1.223	1.383	1.478	1.510
s	Spacing of stirrups	m	0.150	0.150	0.300	0.300	0.300
ncat	Amount of bars in spacing S	bars	2	2	2	2	2
Av	Shear reinf area in spacing S	m2	0.0003	0.0003	0.0003	0.0003	0.0003
β	Assume		6.8	5.9	4.7	2.9	2.5
θ	Assume	degree	27.00	27.00	26.49	28.79	29.79
v	Shear stress in concrete	kN/m2	2196	3265	3440	2131	1043
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1219	1221	1231	1223	1171
e_x	Strain in tensile reinforcement		-3.75E-03	-1.19E-03	-4.24E-05	3.71E-04	5.32E-04
	if ex<0, multiple with reduce factor		-2.36E-04	-1.33E-04	-7.11E-06	-	-
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok	Ok
v/f'c	Ratio of shear stress and f'c		0.049	0.073	0.076	0.047	0.023
β	Final value		6.8	6.0	4.8	3.0	2.5
θ	Final value	degree	27.00	27.00	26.80	28.74	29.51
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	2772	1305	746	497	419
Vs	Shear resistance provided by shear reinforcement	kN	1935	1935	1102	1085	1074
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	4707	3240	1848	1582	1493
Vn2	Vn2	kN	8262	4406	3111	3326	3398
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	4707	3240	1848	1582	1493
Vr	Factored shear resistance	kN	**4236**	**2916**	**1663**	1424	1344
Vu	Shear	kN	1451	1151	856	567	284
(5.8.2.7)	**Shear checking**		OK	OK	OK	OK	OK
	Region requiring transverse reinf Checking		Need	Need	Need	Need	Need
	Minimum shear reinf area	m2	0.0001	0.0001	0.0001	0.0001	0.0001
	Mimimum shear reinforcement Checking		OK	OK	OK	OK	OK
	0.1*f'c*bv*dv	kN	3305	1763	1244	1330	1359
	Smax	m	0.60	0.60	0.60	0.60	0.60
	Maximum spacing Smax		OK	OK	OK	OK	OK

CALCULATION SHEET

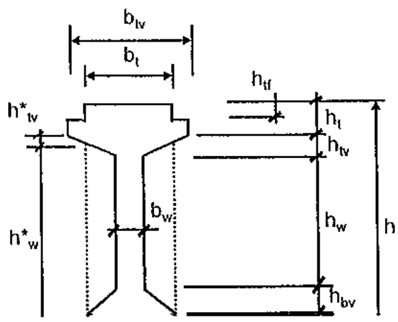
I30 GIRDER

1. INPUT DATA:

1.1. General Data

Specification for bridge design:	TCN 272-05	
Live load	**HL93**	
Impact (or dynamic) of the live load	IM =	0.25
Pedestrian	PL =	0.00 (kN/m^2)
Length of Girder	L_d =	30.00 (m)
Span between support	L_{tt} =	29.20 (m)
Carriageway width in bridge	w =	11.75 (m)
Parapet width	c =	0.50 (m)
Bridge width	B =	12.74 (m)
Number of girder	N_d =	5.00 (girder)
Space betweem 2 girders	S =	2.55 (m)
Distance from inside of parapet to exterior girder center	d_e =	0.77 (m)
Width of bridge deck	b_{ds} =	12.47 (m)
Length of the overhang (cantilever arm length)	L_h =	1.27 (m)
Thickness of bridge deck	t_s =	0.223 (m)
Precast plank width	b_p =	1.85 (m)
Precast plank thick	h_p =	0.08 (m)
Pavement thick	h_{pa} =	0.084 (m)

1.2. Girder dimension:



Width of over part	b_{tv} =		900.00 (mm)
	b_t =		700.00 (mm)
Width of under part	b_b =		700.00 (mm)
Girder high	h =		1600.00 (mm)
	h_{tf} =		80.00 (mm)
	h_t =		80.00 (mm)
	Cross section at end		*at the midle*
	b_w =	700.00	250.00 (mm)
	h^*_{tv} =	34.00	110.00 (mm)
	h^*_w =	1486.00	960.00 (mm)
	h_b =	0.00	250.00 (mm)
	h_{bv} =	0.00	200.00 (mm)

1.3. MATERIAL PROPERTIES:

1.3.1 Concrete:

Girder concrete

Girder concrete strength at the 28 age days	f'_c =	45.00 MPa	
Unit weight of Concrete	γ_c =	2400.00 kG/m^3	
Modulus of elasticity	$E_c = 0.043\ \gamma_c^{1.5}\ sqrt(f'_c)$ =	33914.98 MPa	(5.4.2.4-1)

Deck concrete

Deck concrete strength at the 28 age days	f'_c =	35.00 MPa	
Unit weight of concrete	γ_c =	2400.00 kG/m^3	
Modulus of elasticity	$E_c = 0.043\ \gamma_c^{1.5}\ sqrt(f'_c)$ =	29910.20 MPa	(5.4.2.4-1)

1.3.2 Prestressing steel

Diameter of one strand	D =	12.70 mm
Area of one strand	$A_s^{12.7}$ =	98.70 mm^2
Ultimate Tendon strength	f_{pu} =	1860.00 MPa
Yield strength of prestressing steel	$f_{py} = 0.9\ f_{pu}$ =	1674.00 MPa
Modulus of strand	E_p =	197000.00 MPa
Wobble friction coefficient (mm-1)	K =	6.60E-07 mm^{-1}
Coeficient of friction (1/RAD)	μ =	0.25
Number of Strands in one Tendon	n =	15.00 Strands
Area of one Tendon	A_s =	1480.50 mm^2
Stress in the prestressing steel at jacking	$f_{pj} = 0.7\ f_{pu}$ =	1302.00 MPa
Jacking force for one tendon	P_j =	1927.61 kN
Anchorage set	ΔL =	6.00 mm
Area of one duck	A_g =	5026.55 mm^2
Number of Tendons	N =	4.00 Tendons

1.3.3 Reinforcing Steel:

Yield strength (deformed bar)	f_{py} =	400.00 (MPa)
Modulus of steel	E_s =	200000.00 (MPa)

2. INTERNAL FORCE:

2.1. Dead Load:

2.1.1 Load:

Interior Beam:

Bridge deck	DC_d =	13.32 (kN/m)
Precat plank & cross beam	DC_{pl} =	4.71 (kN/m)
Parapet	DC_{pa} =	4.74 (kN/m)
Pavement	DW_p =	4.44 (kN/m)

Exterior Beam:

Bridge deck	DC_d =	13.32 (kN/m)
Precat plank & cross beam	DC_{pl} =	2.36 (kN/m)
Parapet	DC_{pa} =	4.80 (kN/m)
Pavement	DW_p =	4.44 (kN/m)

2.1.2 Internal Force due to dead load:

Formula : $M = 0.5\, q.X_i\,(L-X_i)$

$Q = q.(0.5.L-X_i)$

L_{tt} = 29.20 (m)

INTERIOR GIRDER											
		Girder (DC)		Concrete Deck (DC)		Plank & cr.beam (DC)		Parapet (DC)		Pavement (DW)	
Section	X_i (m)	M (kNm)	Q (kN)	M (kNm)	Q (kN)	M (kNm)	Q (kN)	M (kNm)	Q (kN)	M (kNm)	Q (kN)
Support	0.00	0.00	291.53	0.00	194.44	0.00	68.78	0.00	69.20	0.00	64.85
L/8	3.65	931.08	218.65	621.00	145.83	219.65	51.58	221.02	51.90	207.10	48.63
L/4	7.30	1596.14	145.77	1064.57	97.22	376.55	34.39	378.89	34.60	355.03	32.42
3L/8	10.95	1995.17	72.88	1330.71	48.61	470.68	17.19	473.61	17.30	443.79	16.21
L/2	14.60	2128.18	0.00	1419.43	0.00	502.06	0.00	505.19	0.00	473.38	0.00
EXTERIOR GIRDER											
Gối	0.00	0.00	291.53	0.00	194.44	0.00	68.78	0.00	70.01	0.00	64.85
L/8	3.65	931.08	218.65	621.00	145.83	109.83	51.58	223.58	52.51	207.10	48.63
L/4	7.30	1596.14	145.77	1064.57	97.22	188.27	34.39	383.29	35.00	355.03	32.42
3L/8	10.95	1995.17	72.88	1330.71	48.61	235.34	17.19	479.11	17.50	443.79	16.21
L/2	14.60	2128.18	0.00	1419.43	0.00	251.03	0.00	511.05	0.00	473.38	0.00

2.2 Live Load:

2.2.1. Distribution factors for Live load:

Modular Ratio: Girder Concrete/Deck Concrete	$n = E_g / E_d$ =	1.13
Distance from girder centroid to bridge deck centroid	$e^I g$ =	908.97 (mm)
	$e^E g$ =	908.97 (mm)
Longgitudinal stiffness parameter	$K^I g = n(Ig + A\, e^2_g)$ =	1.6E+12
	$K^E g = n(Ig + A\, e^2_g)$ =	1.6E+12
Ration	$K^I g / (L\, t^3_s)$ =	4.98
	$K^E g / (L\, t^3_s)$ =	4.97
	S / L =	0.09

(a) Distribution Factor for Moment: g(M)

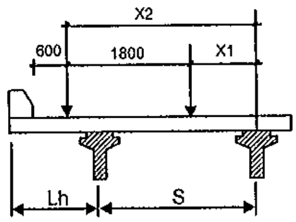
Interior Beam:

For one lane $$0.06+\left(\frac{S}{4300}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{Lt_s^3}\right)^{0.1} =$$ **0.518**

Two or more lanes $$0.075+\left(\frac{S}{2900}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{Lt_s^3}\right)^{0.1} =$$ **0.742**

Exterior Beam:

For one lane, follow the lever rule



X_1 = 920.000

X_2 = 2720.00

Y_1 = 0.361

Y_2 = 1.067

=> $g(M) = 0.5*\Sigma yi$ = $\boxed{0.714}$

Two or more lanes $$e = 0.77 + \frac{d_e}{2800} =$$ 1.045 < (=) 1

Choice e = 1.045 IF(e>1,1,e)

=> $g(M) = e*g_{trong}$ = $\boxed{0.776}$

(b) Distribution Factor for Shear force: g(Q)

Interior Beam:

For one lane $$0.36 + \frac{S}{7600} =$$ **0.696**

Two or more lanes $$0.2 + \frac{S}{3600} - \left(\frac{S}{10700}\right)^2 =$$ **0.852**

Exterior Beam:
For one lane, follow the lever rule

g(Q) = 0.5*Σyi = $\boxed{0.714}$

Two or more lanes

$$e = 0.6 + \frac{de}{3000} = \quad 0.857$$

=> g(Q) =e*g_{trong} = $\boxed{0.729}$

(c) Correction factor for skew bridge:

* Correction factor of distribution factor for moment (Table 4.6.2.2.2d-1)

Skew angle	θ =	0	Degree.	Area of applications 300<= θ <= 600 1100<= S <= 4900 6000 <= L <= 73000 Nb >= 4
Factor	c1 =	0.000		
Correction factor	CF(M) =	1.000		

$$CF(M) = 1.0 - c1.(\tan\theta)^{1.5}$$
$$c1 = 0.25.\left(\frac{Kg}{L.ts^3}\right)^{0.25}.\left(\frac{S}{L}\right)^{0.5}$$

* Regulation factor of distribution factor for shear force (Table 4.6.2.2.3c-1)

Correction Factor CF(Q)= 1.000

Area of applications
00<= θ <= 600
1100<= S <= 4900
6000 <= L <= 73000
Nb >= 4

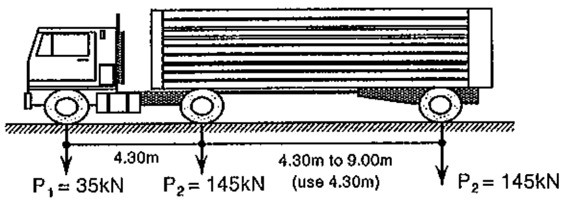
$$CF(Q) = 1.0 + 0.20\left(\frac{L.ts^3}{Kg}\right)^{0.3}.\tan\theta$$

(d) Table of Distribution factors for Live load:

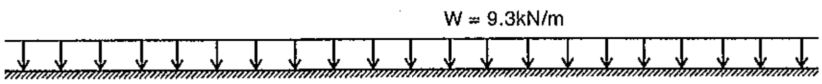
Interior Beam	g(M)	g(Q)	m	m*g(M)	m*g(Q)	m*g(M)*CF(M)	m*g(Q)*CF(Q)
1 lane	0.518	0.696	1.20	0.622	0.835	0.622	0.835
2 or more lanes	0.742	0.852	1.00	0.742	0.852	**0.742**	**0.852**
Exterior Beam							
1 lane	0.714	0.714	1.20	0.856	0.856	**0.856**	**0.856**
2 or more lanes	0.776	0.729	1.00	0.776	0.729	0.776	0.729

2.2.2 Live Load:

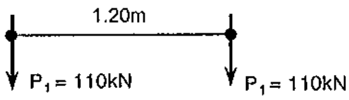
Design Truck



Design Lane Load



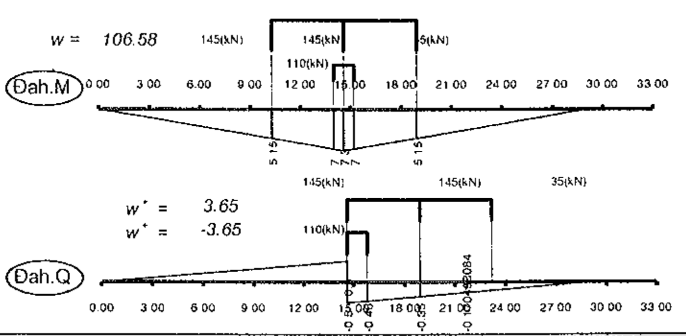
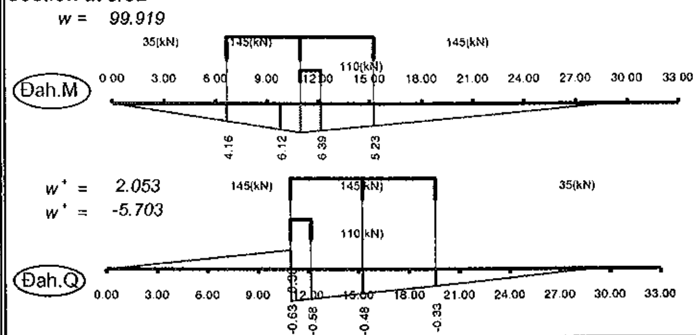
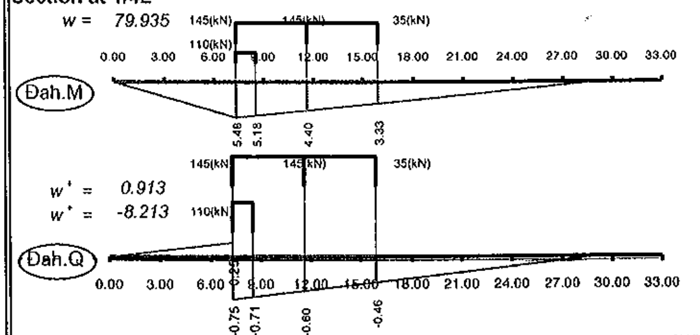
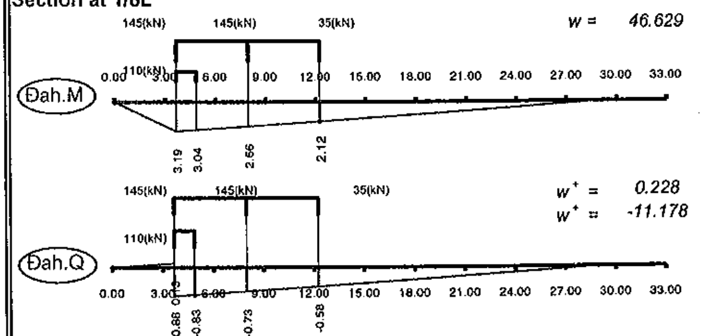
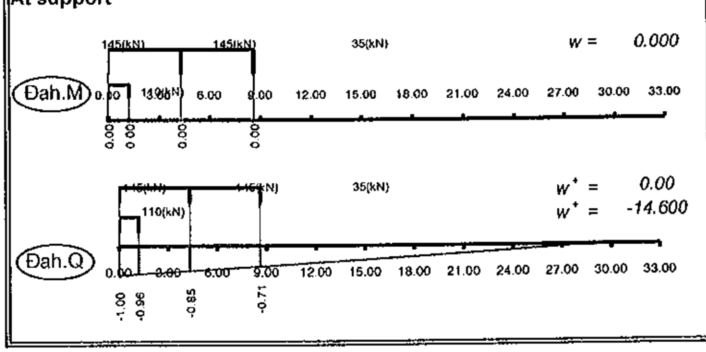
Design Tendem



- Truck	P1 =	35.00 (kN)
	P2 =	145.00 (kN)
- Lane load	W =	9.30 (kN)
- Tandem	P1 =	110.00 (kN)
- Pedestrian	PL =	0.00 kN/m^2
- Dynammic load	IM =	0.25

2.2.3 Internal Force due to Live load:

Design truck or Tendem	Momen	$M_{TR(Ta)} = \Sigma P_i y_i$	(kNm)
	Shear force	$Q_{TR(Ta)} = \Sigma P_i y_i$	(kN)
Lane load	Momen	$M_{Ln} = W.F_M$	(kNm)
	Shear force	$Q_{Ln} = W.F_Q$	(kN)
Pedestrian	Momen	$M_{PL} = PL.F_M$	(kNm)
	Shear force	$Q_{PL} = PL.F_Q$	(kN)

Influence line for Momen & Shear force	Load	Momen (KN.m)	Shear
Section at 1/2L 	Truck	1985.50	130.84
	Lane	991.19	33.95
	Tandem	1540.00	105.48
	Design	**1985.50**	**130.84**
	Pedestrian	0.00	0.00
Section at 3/8L 	Truck	1896.34	171.46
	Lane	929.24	53.04
	Tandem	1456.13	132.98
	Design	**1896.34**	**171.46**
	Pedestrian	0.00	0.00
Section at 1/4L 	Truck	1548.25	212.09
	Lane	743.40	76.38
	Tandem	1171.50	160.48
	Design	**1548.25**	**212.09**
	Pedestrian	0.00	0.00
Section at 1/8L 	Truck	922.41	252.71
	Lan	433.65	103.96
	Tandem	686.13	187.98
	Design	**922.41**	**252.71**
	Pedestrian	0.00	0.00
At support 	Truck	0.00	293.34
	Lane	0.00	135.78
	Tandem	0.00	215.48
	Design	**0.00**	**293.34**
	Pedestrian	0.00	0.00

Internal Force due to Live load : $M_{(LL+IM)} = m*g(M) * [max\{M_{TR}, M_{Ta}\}*(1+IM) + M_{Ln}]$

$Q_{(LL+IM)} = m*g(Q) * [max\{Q_{TR}, Q_{Ta}\}*(1+IM) + Q_{Ln}]$

Internal Force due to pedestrian :

$M = g(M) * M_{PL}$

$Q = g(Q) * Q_{PL}$

In which:

$M_{TR(Ta)}$ moment due to truck or Tandem

$Q_{TR(Ta)}$ Shear force due to truck or Tandem

yi Value of influence line   F Area of influence line

m Lane factor   g Distribution factor

Interior	m*g(M)	m*g(Q)
	0.742	0.852
Exterior		
	0.856	0.856

TABLE OF INTERNAL FORCE DUE TO LIVE LOAD

Setion	Xi	Interior Girder		Exterior Girder	
		M	Q	M	Q
	(m)	(kNm)	(kN)	(kNm)	(kN)
Support	0.00	0.00	427.86	0.00	430.34
L/8	3.65	1177.94	357.52	1358.92	359.59
L/4	7.30	1988.69	290.79	2294.23	292.47
3L/8	10.95	2449.70	227.67	2826.07	228.99
L/2	14.60	2578.43	168.17	2974.58	169.15

2.3 Load combination:

Strength limit state: $U = \eta\,[1.25\ DC + 1.50\ DW + 1.75\ (LL+IM)]$

Service limit state: $U = \eta\,[1.00\ DC + 1.00\ DW + 1.00\ (LL+IM)]$

Fatigue state: $U = 0.75\ (LL+IM)]$

The modify load factort $\eta = \eta_D\,\eta_R\,\eta_I$

STATE	Modify Load Factor			
	η_D	η_R	η_I	$\eta = \eta_D\,\eta_R\,\eta_I$
Strength	1.00	1.00	1.00	1.00
Service	1.00	1.00	1.00	1.00

2.3.1 Load combination - - Interior Girder:

STATE	Strength										
Load	Load Factor	Section									
		Support		L/8		L/4		3L/8		L/2	
	γ	M (kNm)	Q (kN)	M (kNm)	Q (kN)	M (kNm)	Q (kN)	M (kNm)	Q (kN)	M (kNm)	Q (kN)
DC	1.25	0.00	779.94	2490.94	584.96	4270.18	389.97	5337.73	194.99	5693.58	0.00
DW	1.50	0.00	97.27	310.65	72.95	532.55	48.63	665.68	24.32	710.06	0.00
LL+IM	1.75	0.00	748.75	2061.40	625.66	3480.21	508.88	4286.97	398.43	4512.25	294.30
Total		**0.00**	**1625.96**	**4862.99**	**1283.56**	**8282.94**	**947.49**	**10290.39**	**617.73**	**10915.89**	**294.30**

STATE	Service										
Load	Load Factor	Section									
		Support		L/8		L/4		3L/8		L/2	
	γ	M (kNm)	Q (kN)	M (kNm)	Q (kN)	M (kNm)	Q (kN)	M (kNm)	Q (kN)	M (kNm)	Q (kN)
DC	1.00	0.00	623.95	1992.75	467.97	3416.15	311.98	4270.18	155.99	4554.86	0.00
DW	1.00	0.00	64.85	207.10	48.63	355.03	32.42	443.79	16.21	473.38	0.00
LL+IM	1.00	0.00	427.86	1177.94	357.52	1988.69	290.79	2449.70	227.67	2578.43	168.17
Total		**0.00**	**1116.66**	**3377.80**	**874.12**	**5759.87**	**635.19**	**7163.67**	**399.87**	**7606.66**	**168.17**

2.3.2 Load combination - Exterior Girder:

STATE	Strength										
Load	Load factor	Section									
		Supprt		L/8		L/4		3L/8		L/2	
	γ	M (kNm)	Q (kN)	M (kNm)	Q (kN)	M (kNm)	Q (kN)	M (kNm)	Q (kN)	M (kNm)	Q (kN)
DC	1.25	0.00	780.95	2356.86	585.71	4040.34	390.47	5050.42	195.24	5387.11	0.00
DW	1.50	0.00	97.27	310.65	72.95	532.55	48.63	665.68	24.32	710.06	0.00
LL+IM	1.75	0.00	753.09	2378.12	629.28	4014.91	511.83	4945.63	400.74	5205.52	296.01
Total		**0.00**	**1631.30**	**5045.63**	**1287.94**	**8587.79**	**950.94**	**10661.73**	**620.29**	**11302.69**	**296.01**

STATE	Service										
Load	load factor	Section									
		Support		L/8		L/4		3L/8		L/2	
	γ	M (kNm)	Q (kN)	M (kNm)	Q (kN)	M (kNm)	Q (kN)	M (kNm)	Q (kN)	M (kNm)	Q (kN)
DC	1.00	0.00	624.76	1885.49	468.57	3232.27	312.38	4040.34	156.19	4309.69	0.00
DW	1.00	0.00	64.85	207.10	48.63	355.03	32.42	443.79	16.21	473.38	0.00
LL+IM	1.00	0.00	430.34	1358.92	359.59	2294.23	292.47	2826.07	228.99	2974.58	169.15
Total		**0.00**	**1119.94**	**3451.51**	**876.79**	**5881.53**	**637.28**	**7310.20**	**401.39**	**7757.65**	**169.15**

3. TENDON PROFILE AND PROPERTY OF GIRDER CROSS SECTION

3.1. Tendon profile:

Tendon profile follow Parabol equation:

$$y_i = f - \frac{4.(f - c).x.(l - x)}{l^2}$$

in which: Origin of coordinates in left edge of the Girder bottom (0.0)

- f — Maximum deflection at mid span of tendon
- c — Distance from maximum deflection point to girder bottom
- (x_i, y_i) — Coordination of point under consider i = 1,2...
- L — actual distance between cable ends (X-axis)
- $L_p = X_2 - X_1$ — Distance between 2 point under consider

angle of rotation of tendon for X_i-axis — Tan(α) = (4.f (1-2.Xi / L)) / L

α = 2 f / 0.5 L - tan(α)

L_{span} =	30000	(mm)
$L_{su.}$ =	29200	(mm)
L_{cap} =	29700	(mm)

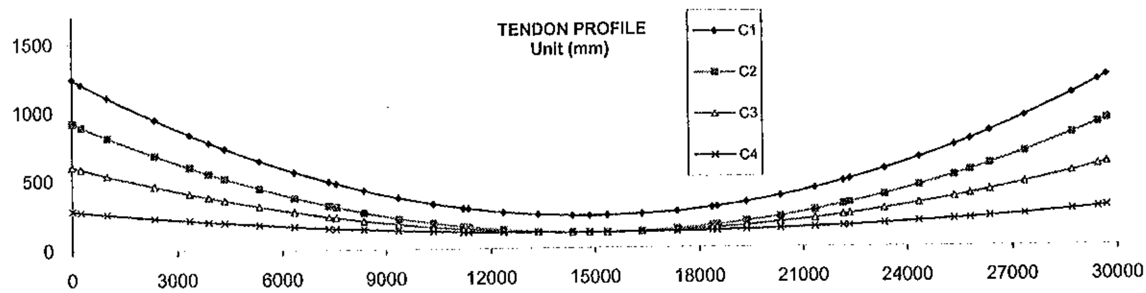
	f =	1240	(mm)	Lcáp =	29700	(mm)	C =	230	(mm)
	Section	**Xi**	**Yi**	**Lp**	**ΣLcáp**	**Tan(αi)**	(α)	Σα	**Section**
		(mm)	(mm)	(mm)	(mm)		(rad)	(rad)	
TENDON No 1	**Anchorage**	0.00	1240.00	0.00	0.00	0.1670	0.0000	0.0000	**Anchorage**
	Support	250.00	1206.28	250.00	250.00	0.1642	0.0028	0.0028	**Support**
	L/8	3900.00	779.16	3650.00	3900.00	0.1231	0.0439	0.0467	**L/8**
	L/4	7550.00	474.07	3650.00	7550.00	0.0821	0.0849	0.1316	**L/4**
	3L/8	11200.00	291.02	3650.00	11200.00	0.0410	0.1260	0.2575	**3L/8**
	L/2	14850.00	230.00	3650.00	14850.00	0.0000	0.1670	0.4245	**L/2**

	f =	920	(mm)	Lcáp =	29700	(mm)	C =	110	(mm)
	Section	**Xi**	**Yi**	**Lp**	**ΣLcáp**	**Tan(α)**	(α)	Σα	**Section**
		(mm)	(mm)	(mm)	(mm)		(rad)	(rad)	
TENDON No 2	**Anchorage**	0.00	920.00	0.00	0.00	0.1239	0.0000	0.0000	**Anchorage**
	Support	250.00	892.96	250.00	250.00	0.1218	0.0021	0.0021	**Support**
	L/8	3900.00	550.41	3650.00	3900.00	0.0914	0.0325	0.0346	**L/8**
	L/4	7550.00	305.74	3650.00	7550.00	0.0609	0.0630	0.0976	**L/4**
	3L/8	11200.00	158.93	3650.00	11200.00	0.0305	0.0935	0.1911	**3L/8**
	L/2	14850.00	110.00	3650.00	14850.00	0.0000	0.1239	0.3150	**L/2**

	f =	600	(mm)	Lcáp =	29700	(mm)	C =	110	(mm)
	Section	**Xi**	**Yi**	**Lp**	**ΣLcáp**	**Tan(α)**	(α)	Σα	**Section**
		(mm)	(mm)	(mm)	(mm)		(rad)	(rad)	
TENDON No 3	**Anchorage**	0.00	600.00	0.00	0.00	0.0808	0.0000	0.0000	**Anchorage**
	Support	250.00	583.64	250.00	250.00	0.0794	0.0014	0.0014	**Support**
	L/8	3900.00	376.42	3650.00	3900.00	0.0596	0.0212	0.0226	**L/8**
	L/4	7550.00	228.41	3650.00	7550.00	0.0397	0.0411	0.0637	**L/4**
	3L/8	11200.00	139.60	3650.00	11200.00	0.0199	0.0609	0.1246	**3L/8**
	L/2	14850.00	110.00	3650.00	14850.00	0.0000	0.0808	0.2054	**L/2**

	f =	280	(mm)	Lcáp =	29700	(mm)	C =	110	(mm)
	Section	**Xi**	**Yi**	**Lp**	**ΣLcáp**	**Tan(α)**	(α)	Σα	**Section**
		(mm)	(mm)	(mm)	(mm)		(rad)	(rad)	
TENDON No 4	**Anchorage**	0.00	280.00	0.00	0.0	0.0377	0.0000	0.0000	**Anchorage**
	Support	250.00	274.32	250.00	250.0	0.0371	0.0006	0.0006	**Support**
	L/8	3900.00	202.43	3650.00	3900.0	0.0278	0.0099	0.0105	**L/8**
	L/4	7550.00	151.08	3650.00	7550.0	0.0185	0.0192	0.0297	**L/4**
	3L/8	11200.00	120.27	3650.00	11200.0	0.0093	0.0284	0.0582	**3L/8**
	L/2	14850.00	110.00	3650.00	14850.0	0.0000	0.0377	0.0959	**L/2**

Section	TENDON No 1		TENDON No 2		TENDON No 3		TENDON No 4	
	X_i (mm)	Y_i (mm)	X_i (mm)	Y_i (mm)	X_i (mm)	Y_i (mm)	X_i (mm)	Y_i (mm)
Anchorage	0.00	1240.00	0.0	920.00	0.0	600.00	0.00	280.00
Support	**250.00**	**1206.28**	**250.0**	**892.96**	**250.0**	**583.64**	**250.00**	**274.32**
1	1000.00	1108.55	1000.0	814.58	1000.0	536.23	1000.00	257.88
2	2350.00	945.63	2350.0	683.92	2350.0	457.19	2350.00	230.45
3	3350.00	835.71	3350.0	595.77	3350.0	403.86	3350.00	211.95
L/8	**3900.00**	**779.16**	**3900.0**	**550.41**	**3900.0**	**376.42**	**3900.00**	**202.43**
4	4350.00	734.95	4350.0	514.96	4350.0	354.98	4350.00	194.99
5	5350.00	643.35	5350.0	441.50	5350.0	310.54	5350.00	179.57
6	6350.00	560.91	6350.0	375.38	6350.0	270.54	6350.00	165.70
7	7350.00	487.63	7350.0	316.61	7350.0	234.99	7350.00	153.36
L/4	**7550.00**	**474.07**	**7550.0**	**305.74**	**7550.0**	**228.41**	**7550.00**	**151.08**
8	8350.00	423.51	8350.0	265.19	8350.0	203.88	8350.00	142.57
9	9350.00	368.55	9350.0	221.11	9350.0	177.22	9350.00	133.32
10	10350.00	322.75	10350.0	184.38	10350.0	155.00	10350.00	125.61
11	11350.00	286.11	11350.0	155.00	11350.0	137.22	11350.00	119.44
3L/8	**11200.00**	**291.02**	**11200.0**	**158.93**	**11200.0**	**139.60**	**11200.00**	**120.27**
12	12350.00	258.63	12350.0	132.96	12350.0	123.89	12350.00	114.82
13	13350.00	240.31	13350.0	118.26	13350.0	115.00	13350.00	111.73
14	14350.00	231.15	14350.0	110.92	14350.0	110.56	14350.00	110.19
15	15350.00	231.15	15350.0	110.92	15350.0	110.56	15350.00	110.19
L/2	**14850.00**	**230.00**	**14850.0**	**110.00**	**14850.0**	**110.00**	**14850.00**	**110.00**
2	14350.00	231.15	14350.0	110.92	14350.0	110.56	14350.00	110.19
3	15350.00	231.15	15350.0	110.92	15350.0	110.56	15350.00	110.19
4	16350.00	240.31	16350.0	118.26	16350.0	115.00	16350.00	111.73
5	17350.00	258.63	17350.0	132.96	17350.0	123.89	17350.00	114.82
-	**18500.00**	**291.02**	**18500.0**	**158.93**	**18500.0**	**139.60**	**18500.00**	**120.27**
6	18350.00	286.11	18350.0	155.00	18350.0	137.22	18350.00	119.44
7	19350.00	322.75	19350.0	184.38	19350.0	155.00	19350.00	125.61
8	20350.00	368.55	20350.0	221.11	20350.0	177.22	20350.00	133.32
9	21350.00	423.51	21350.0	265.19	21350.0	203.88	21350.00	142.57
-	**22150.00**	**474.07**	**22150.0**	**305.74**	**22150.0**	**228.41**	**22150.00**	**151.08**
10	22350.00	487.63	22350.0	316.61	22350.0	234.99	22350.00	153.36
11	23350.00	560.91	23350.0	375.38	23350.0	270.54	23350.00	165.70
12	24350.00	643.35	24350.0	441.50	24350.0	310.54	24350.00	179.57
13	25350.00	734.95	25350.0	514.96	25350.0	354.98	25350.00	194.99
-	**25800.00**	**779.16**	**25800.0**	**550.41**	**25800.0**	**376.42**	**25800.00**	**202.43**
14	26350.00	835.71	26350.0	595.77	26350.0	403.86	26350.00	211.95
14	27350.00	945.63	27350.0	683.92	27350.0	457.19	27350.00	230.45
16	28700.00	1108.55	28700.0	814.58	28700.0	536.23	28700.00	257.88
Support	29450.00	1206.28	29450.0	892.96	29450.0	583.64	29450.00	274.32
Anchorage	29700.00	1240.00	29700.0	920.00	29700.0	600.00	29700.00	280.00



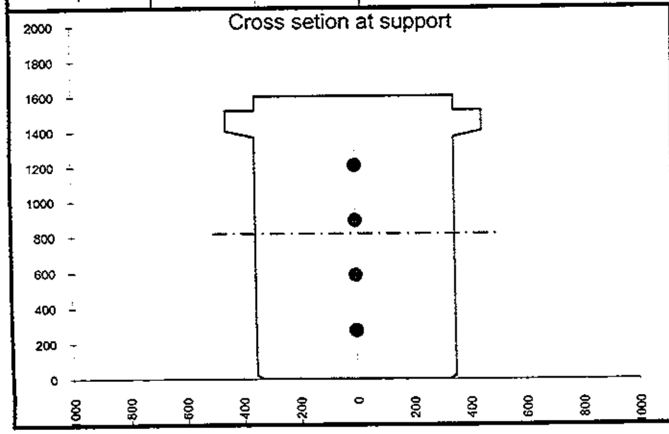
3.2 Property of Girder Cross section at transfer (Stage I: net cross section):

Ăiăøm goïc	x (mm)	y (mm)	A (m^2)	Q_X' (m^3)	Q_Y' (m^3)	I_X' (m^4)	I_Y' (m^4)	$I_{X'Y'}$ (m^4)
Đ ờng bao								
-	0.00	0.00	-	-	-	-	-	-
1	330.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	350.00	20.00	0.01	0.00	0.00	0.00	0.00	0.00
3	350.00	1366.00	0.47	0.65	0.33	0.89	0.17	0.34
4	450.00	1400.00	-0.12	-0.34	-0.10	-0.72	-0.06	-0.21
5	450.00	1520.00	0.05	0.16	0.05	0.35	0.03	0.11
6	350.00	1520.00	0.15	0.46	0.12	1.05	0.07	0.28
7	350.00	1600.00	0.03	0.09	0.02	0.20	0.01	0.05
8	-350.00	1600.00	1.12	3.58	0.00	8.60	0.14	0.00
9	-350.00	1520.00	0.03	0.09	-0.02	0.20	0.01	-0.05
10	-450.00	1520.00	0.15	0.46	-0.12	1.05	0.07	-0.28
11	-450.00	1400.00	0.05	0.16	-0.05	0.35	0.03	-0.11
12	-350.00	1366.00	-0.12	-0.34	0.10	-0.72	-0.06	0.21
13	-350.00	20.00	0.47	0.65	-0.33	0.89	0.17	-0.34
14	-330.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00
15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
16								
17								
18								
19								
20								
21								
Tổng cộng			1.1470	0.9358	0.0000	1.0135	0.0500	0.0000

Cáp	x	y
1	0.00	1206.28
2	0.00	892.96
3	0.00	583.64
4	0.00	274.32
5		
6		
7		

Diện tích 1 bó cáp As = 1480.50 (mm2)
Số bó cáp n= 4.00 bó
Σ As Yi = 4378136.37 (mm3)
Σ As = 5922.00 (mm2)
K. cách từ trọng tâm bó cáp đến TTH = 76.53 (mm)

Cross setion at support



A	1.1470	m^2
e_x	0.0000	m
e_y	0.8158	m
I_x	0.2501	m^4
I_y	0.0500	m^4
I_{xy}	0.0000	m^4
y_b	0.8158	m
y_t	0.7842	m
S_b	0.3065	m^3
S_t	0.3189	m^3
$E_{cáp\,(Y)}$	0.0765	m
q =	27.5280	(KN/m)

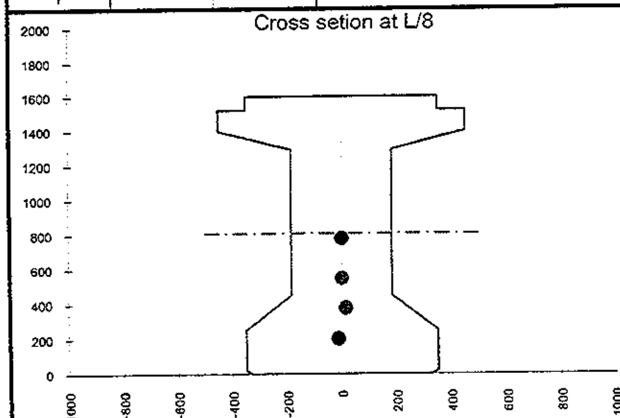
ĐẶC TR NG HÌNH HỌC - MẶT CẮT L/4

Ăiăøm goïc	x (mm)	y (mm)	A (m^2)	Q_X' (m^3)	Q_Y' (m^3)	I_X' (m^4)	I_Y' (m^4)	$I_{X'Y'}$ (m^4)
Đ ờng bao								
-	0.00	0.00	-	-	-	-	-	-
1	330.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	350.00	20.00	0.01	0.00	0.00	0.00	0.00	0.00
3	350.00	250.00	0.08	0.02	0.06	0.01	0.03	0.01
4	183.50	450.00	0.11	0.08	0.06	0.04	0.02	0.03
5	183.50	1290.00	0.15	0.27	0.06	0.38	0.02	0.07
6	450.00	1400.00	-0.32	-0.87	-0.21	-1.76	-0.10	-0.42
7	450.00	1520.00	0.05	0.16	0.05	0.35	0.03	0.11
8	350.00	1520.00	0.15	0.46	0.12	1.05	0.07	0.28
9	350.00	1600.00	0.03	0.09	0.02	0.20	0.01	0.05
10	-350.00	1600.00	1.12	3.58	0.00	8.60	0.14	0.00
11	-350.00	1520.00	0.03	0.09	-0.02	0.20	0.01	-0.05
12	-450.00	1520.00	0.15	0.46	-0.12	1.05	0.07	-0.28
13	-450.00	1400.00	0.05	0.16	-0.05	0.35	0.03	-0.11
14	-183.50	1290.00	-0.32	-0.87	0.21	-1.76	-0.10	0.42
15	-183.50	450.00	0.15	0.27	-0.06	0.38	0.02	-0.07
16	-350.00	250.00	0.11	0.08	-0.06	0.04	0.02	-0.03
17	-350.00	20.00	0.08	0.02	-0.06	0.01	0.03	-0.01
18	-330.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00
19	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
20								
21								
Tổng cộng			0.8233	0.6656	0.0000	0.7620	0.0257	0.0000

Cáp	x	y
1	0.00	779.16
2	0.00	550.41
3	14.00	376.42
4	-14.00	202.43
5		
6		
7		

Diện tích 1 bó cáp As = 1480.50 (mm2)
Số bó cáp n= 4.00 bó
Σ As Yi = 2825424.21 (mm3)
Σ As = 5922.00 (mm2)
K. cách từ trọng tâm bó cáp đến TTH = 331.40 (mm)

Cross setion at L/8

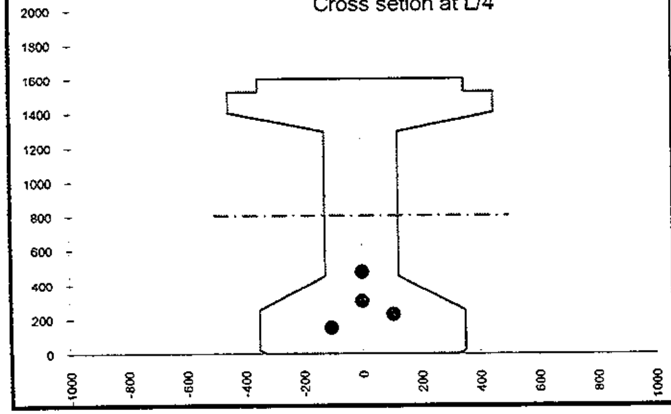


A	0.8233	m^2
e_x	0.0000	m
e_y	0.8085	m
I_x	0.2238	m^4
I_y	0.0257	m^4
I_{xy}	0.0000	m^4
y_b	0.8085	m
y_t	0.7915	m
S_b	0.2769	m^3
S_t	0.2828	m^3
$E_{cáp\,(Y)}$	0.3314	m
q =	19.7584	(KN/m)

ĐẶC TR NG HÌNH HỌC - MẶT CẮT 3L/8

Âiãøm goïc	x (mm)	y (mm)	A (m^2)	Q_X' (m^3)	Q_Y' (m^3)	I_X' (m^4)	I_Y' (m^4)	$I_X'Y'$ (m^4)
Đ ờng bao								
-	0.00	0.00	-	-	-	-	-	-
1	330.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	350.00	20.00	0.01	0.00	0.00	0.00	0.00	0.00
3	350.00	250.00	0.08	0.02	0.06	0.01	0.03	0.01
4	125.00	450.00	0.13	0.09	0.06	0.05	0.02	0.03
5	125.00	1290.00	0.11	0.18	0.03	0.26	0.00	0.03
6	450.00	1400.00	-0.41	-1.09	-0.23	-2.20	-0.11	-0.47
7	450.00	1520.00	0.05	0.16	0.05	0.35	0.03	0.11
8	350.00	1520.00	0.15	0.46	0.12	1.05	0.07	0.28
9	350.00	1600.00	0.03	0.09	0.02	0.20	0.01	0.05
10	-350.00	1600.00	1.12	3.58	0.00	8.60	0.14	0.00
11	-350.00	1520.00	0.03	0.09	-0.02	0.20	0.01	-0.05
12	-450.00	1520.00	0.15	0.46	-0.12	1.05	0.07	-0.28
13	-450.00	1400.00	0.05	0.16	-0.05	0.35	0.03	-0.11
14	-125.00	1290.00	-0.41	-1.09	0.23	-2.20	-0.11	0.47
15	-125.00	450.00	0.11	0.18	-0.03	0.26	0.00	-0.03
16	-350.00	250.00	0.13	0.09	-0.06	0.05	0.02	-0.03
17	-350.00	20.00	0.08	0.02	-0.06	0.01	0.03	-0.01
18	-330.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00
19	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
20								
21								
Tổng cộng			0.7069	0.5671	0.0000	0.6687	0.0223	0.0000
Cáp								
1	0.00	474.07	Diện tích 1 bó cáp			As =	1480.50	(mm2)
2	0.00	305.74	Số bó cáp			n=	4.00	bó
3	105.00	228.41				Σ As Yi =	1716344.09	(mm3)
4	-105.00	151.08				Σ As =	5922.00	(mm2)
5			K. cách từ trọng tâm bó cáp đến TTH =				512.45	(mm)
6								
7								

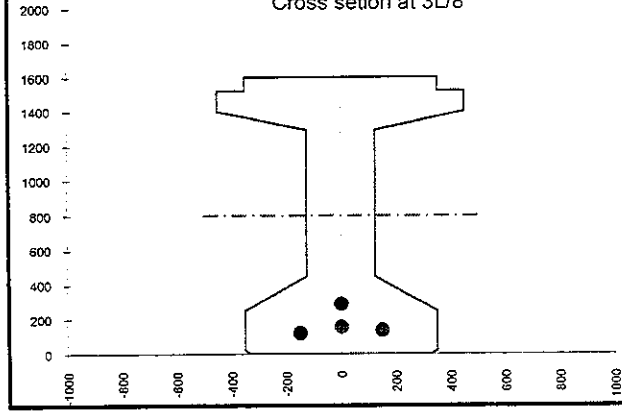
Cross setion at L/4



A	0.7069	m^2
e_x	0.0000	m
e_y	0.8023	m
I_x	0.2138	m^4
I_y	0.0223	m^4
I_{xy}	0.0000	m^4
y_b	0.8023	m
y_t	0.7977	m
S_b	0.2665	m^3
S_t	0.2680	m^3
$E_{cáp\ (Y)}$	0.5125	m
q =	16.9644	(KN/m)

Âiãøm goïc	x (mm)	y (mm)	A (m^2)	Q_X' (m^3)	Q_Y' (m^3)	I_X' (m^4)	I_Y' (m^4)	$I_X'Y'$ (m^4)
Đ ờng bao								
-	0.00	0.00	-	-	-	-	-	-
1	330.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	350.00	20.00	0.01	0.00	0.00	0.00	0.00	0.00
3	350.00	250.00	0.08	0.02	0.06	0.01	0.03	0.01
4	125.00	450.00	0.13	0.09	0.06	0.05	0.02	0.03
5	125.00	1290.00	0.11	0.18	0.03	0.26	0.00	0.03
6	450.00	1400.00	-0.41	-1.09	-0.23	-2.20	-0.11	-0.47
7	450.00	1520.00	0.05	0.16	0.05	0.35	0.03	0.11
8	350.00	1520.00	0.15	0.46	0.12	1.05	0.07	0.28
9	350.00	1600.00	0.03	0.09	0.02	0.20	0.01	0.05
10	-350.00	1600.00	1.12	3.58	0.00	8.60	0.14	0.00
11	-350.00	1520.00	0.03	0.09	-0.02	0.20	0.01	-0.05
12	-450.00	1520.00	0.15	0.46	-0.12	1.05	0.07	-0.28
13	-450.00	1400.00	0.05	0.16	-0.05	0.35	0.03	-0.11
14	-125.00	1290.00	-0.41	-1.09	0.23	-2.20	-0.11	0.47
15	-125.00	450.00	0.11	0.18	-0.03	0.26	0.00	-0.03
16	-350.00	250.00	0.13	0.09	-0.06	0.05	0.02	-0.03
17	-350.00	20.00	0.08	0.02	-0.06	0.01	0.03	-0.01
18	-330.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00
19	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
20								
21								
Tổng cộng			0.7069	0.5671	0.0000	0.6687	0.0223	0.0000
Cáp								
1	0.00	291.02	Diện tích 1 bó cáp			As =	1480.50	(mm2)
2	0.00	158.93	Số bó cáp			n=	4.00	bó
3	150.00	139.60				Σ As Yi =	1050896.02	(mm3)
4	-150.00	120.27				Σ As =	5922.00	(mm2)
5			K. cách từ trọng tâm bó cáp đến TTH =				624.82	(mm)
6								
7								

Cross setion at 3L/8



A	0.7069	m^2
e_x	0.0000	m
e_y	0.8023	m
I_x	0.2138	m^4
I_y	0.0223	m^4
I_{xy}	0.0000	m^4
y_b	0.8023	m
y_t	0.7977	m
S_b	0.2665	m^3
S_t	0.2680	m^3
$E_{cáp\ (Y)}$	0.6248	m
q =	16.9644	(KN/m)

ĐẶC TR NG HÌNH HỌC - MẶT CẮT L/2

Âiãøm goïc	x (mm)	y (mm)	A (m^2)	Q_X' (m^3)	Q_Y' (m^3)	I_X' (m^4)	I_Y' (m^4)	$I_X'Y'$ (m^4)
Đ ờng bao								
-	0.00	0.00	-	-	-	-	-	-
1	330.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	350.00	20.00	0.01	0.00	0.00	0.00	0.00	0.00
3	350.00	250.00	0.08	0.02	0.06	0.01	0.03	0.01
4	125.00	450.00	0.13	0.09	0.06	0.05	0.02	0.03
5	125.00	1290.00	0.11	0.18	0.03	0.26	0.00	0.03
6	450.00	1400.00	-0.41	-1.09	-0.23	-2.20	-0.11	-0.47
7	450.00	1520.00	0.05	0.16	0.05	0.35	0.03	0.11
8	350.00	1520.00	0.15	0.46	0.12	1.05	0.07	0.28
9	350.00	1600.00	0.03	0.09	0.02	0.20	0.01	0.05
10	-350.00	1600.00	1.12	3.58	0.00	8.60	0.14	0.00
11	-350.00	1520.00	0.03	0.09	-0.02	0.20	0.01	-0.05
12	-450.00	1520.00	0.15	0.46	-0.12	1.05	0.07	-0.28
13	-450.00	1400.00	0.05	0.16	-0.05	0.35	0.03	-0.11
14	-125.00	1290.00	-0.41	-1.09	0.23	-2.20	-0.11	0.47
15	-125.00	450.00	0.11	0.18	-0.03	0.26	0.00	-0.03
16	-350.00	250.00	0.13	0.09	-0.06	0.05	0.02	-0.03
17	-350.00	20.00	0.08	0.02	-0.06	0.01	0.03	-0.01
18	-330.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00
19	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
20								
21								
Tổng cộng			0.7069	0.5671	0.0000	0.6687	0.0223	0.0000

Cáp	x	y
1	0.00	230.00
2	0.00	110.00
3	150.00	110.00
4	-150.00	110.00
5		
6		
7		

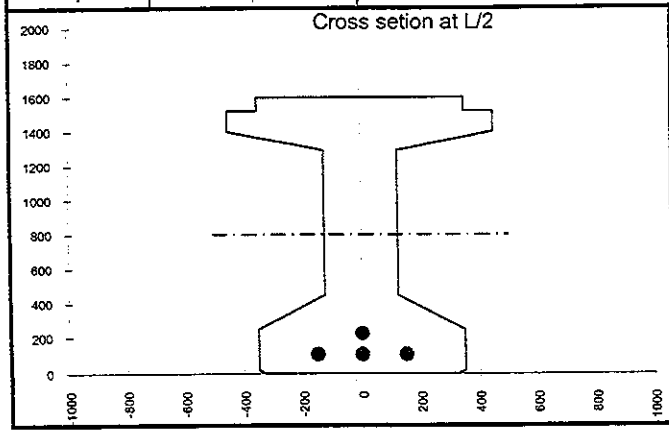
Diện tích 1 bó cáp As = 1480.50 (mm2)

Số bó cáp n= 4.00 bó

Σ As Yi = 829080.00 (mm3)

Σ As = 5922.00 (mm2)

K. cách từ trọng tâm bó cáp đến TTH = 662.28 (mm)



A	0.7069	m^2
e_x	0.0000	m
e_y	0.8023	m
I_x	0.2138	m^4
I_y	0.0223	m^4
I_{xy}	0.0000	m^4
y_b	0.8023	m
y_t	0.7977	m
S_b	0.2665	m^3
S_t	0.2680	m^3
$E_{cap (Y)}$	0.6623	m
q =	16.9644	(KN/m)

Uniform load due to sefl weigh of Girder in Stage 1: Q = 19.97 (KN/m)

3.3. Property of Girder cross section in service stage (stage II: Composite cross section) :

3.3.1. Effective flange width (4.6.2.6)

Modular Ratio: Deck Concrete/Girder Concrete n = Eb / Ed = 0.88

For Interior Girder:

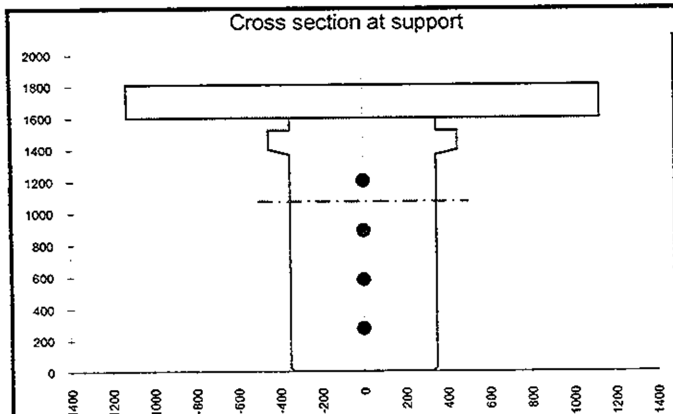
$$b_I = \min\begin{cases} 1/4L_{tt} \\ 12h_f + \max(0.5b, b_w) \\ S \end{cases} \quad \Rightarrow n^* b_I = 2248.88861 \text{ (mm)}$$

For Exterior Girder:

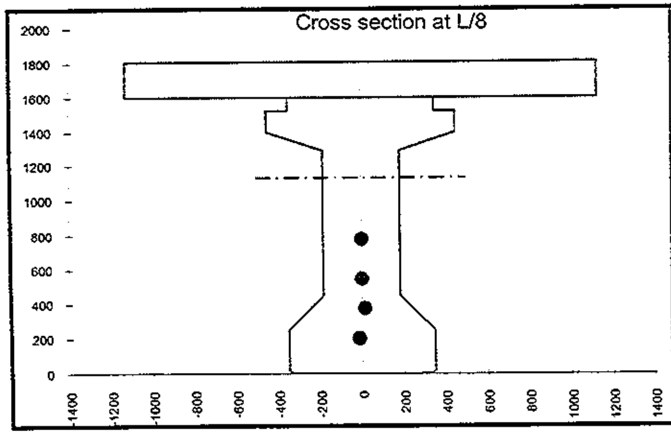
$$b_E = 0.5b_I + \min\begin{cases} 1/8L_{tt} \\ 6h_f + \max(0.5b_w, 0.25b) \\ L_h \end{cases} \quad \Rightarrow n^* b_E = 2244.47903 \text{ (mm)}$$

3.3.2. Property of Girder cross section in stage II (service stage):

Interior Girder:

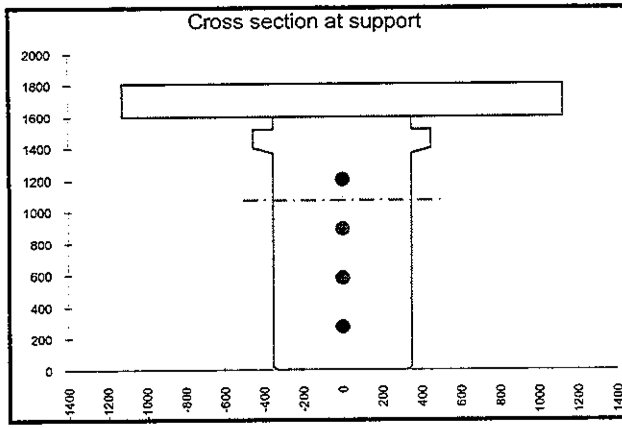


A	1.6125	m^2
e_x	0.0000	m
e_y	1.0721	m
I_x	0.5126	m^4
I_y	0.2462	m^4
I_{xy}	0.0000	m^4
y_b	1.0721	m
y_t	0.7349	m
S_b	0.4782	m^3
S_t	0.6975	m^3
$E_{cáp\,(Y)}$	0.3328	m
q =	38.7005	(KN/m)

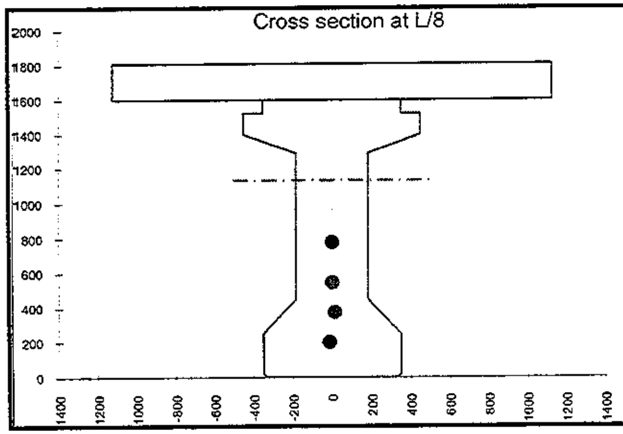


A	1.2888	m^2
e_x	0.0000	m
e_y	1.1318	m
I_x	0.4637	m^4
I_y	0.2219	m^4
I_{xy}	0.0000	m^4
y_b	1.1318	m
y_t	0.6752	m
S_b	0.4097	m^3
S_t	0.6867	m^3
$E_{cáp\,(Y)}$	0.6547	m
q =	30.9308	(KN/m)

Exterior Girder:



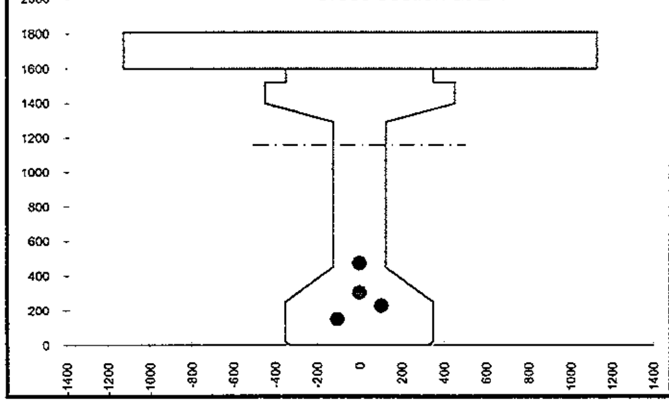
A	1.6116	m^2
e_x	0.0000	m
e_y	1.0717	m
I_x	0.5123	m^4
I_y	0.2451	m^4
I_{xy}	0.0000	m^4
y_b	1.0717	m
y_t	0.7353	m
S_b	0.4780	m^3
S_t	0.6967	m^3
$E_{cáp\,(Y)}$	0.3324	m
q =	39.4844	(KN/m)



A	1.2879	m^2
e_x	0.0000	m
e_y	1.1314	m
I_x	0.4634	m^4
I_y	0.2207	m^4
I_{xy}	0.0000	m^4
y_b	1.1314	m
y_t	0.6756	m
S_b	0.4096	m^3
S_t	0.6859	m^3
$E_{cáp\,(Y)}$	0.6543	m
q =	31.5529	(KN/m)

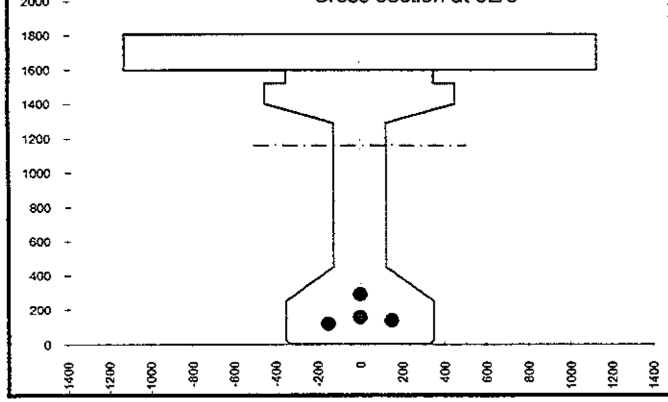
Interior Girder:

Cross section at L/4



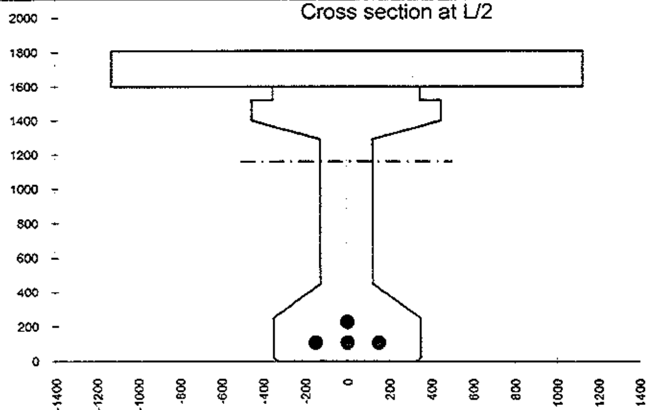
A	1.1724	m^2
e_x	0.0000	m
e_y	1.1601	m
I_x	0.4434	m^4
I_y	0.2185	m^4
I_{xy}	0.0000	m^4
y_b	1.1601	m
y_t	0.6469	m
S_b	0.3822	m^3
S_t	0.6855	m^3
$E_{cáp\ (Y)}$	0.8703	m
q =	28.1369	(KN/m)

Cross section at 3L/8



A	1.1724	m^2
e_x	0.0000	m
e_y	1.1601	m
I_x	0.4434	m^4
I_y	0.2185	m^4
I_{xy}	0.0000	m^4
y_b	1.1601	m
y_t	0.6469	m
S_b	0.3822	m^3
S_t	0.6855	m^3
$E_{cáp\ (Y)}$	0.9827	m
q =	28.1369	(KN/m)

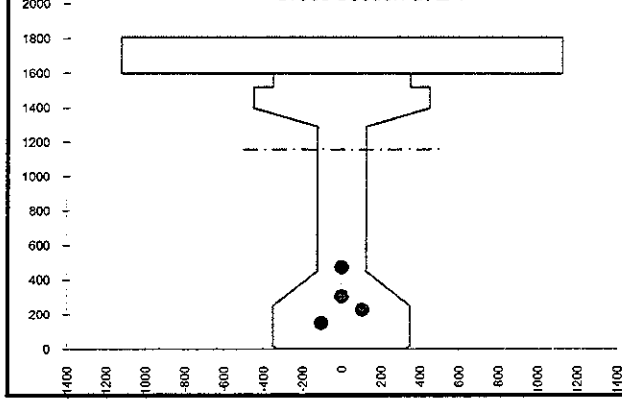
Cross section at L/2



A	1.1724	m^2
e_x	0.0000	m
e_y	1.1601	m
I_x	0.4434	m^4
I_y	0.2185	m^4
I_{xy}	0.0000	m^4
y_b	1.1601	m
y_t	0.6469	m
S_b	0.3822	m^3
S_t	0.6855	m^3
$E_{cáp\ (Y)}$	1.0201	m
q =	28.1369	(KN/m)

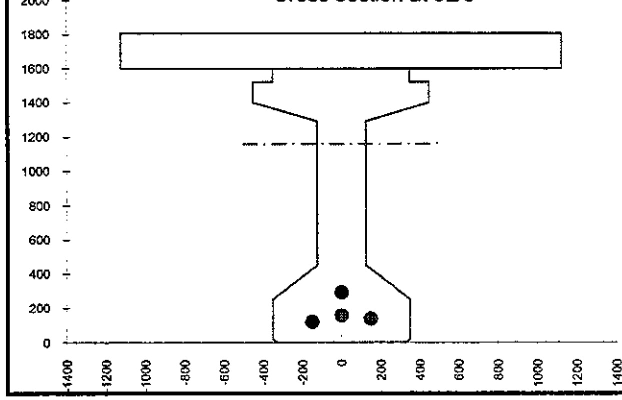
Exterior Girder:

Cross section at L/4



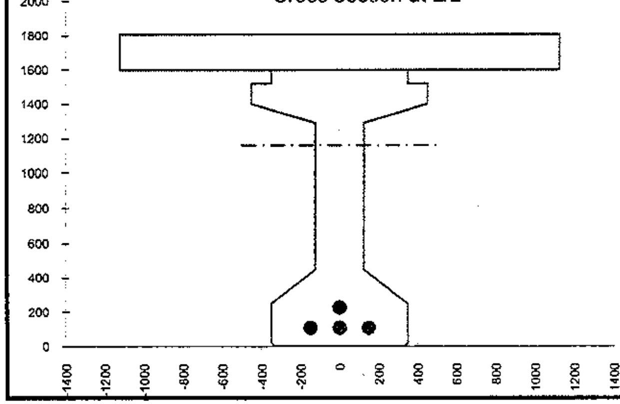
A	1.1715	m^2
e_x	0.0000	m
e_y	1.1597	m
I_x	0.4431	m^4
I_y	0.2173	m^4
I_{xy}	0.0000	m^4
y_b	1.1597	m
y_t	0.6473	m
S_b	0.3821	m^3
S_t	0.6846	m^3
$E_{cáp\ (Y)}$	0.8699	m
q =	28.7007	(KN/m)

Cross section at 3L/8



A	1.1715	m^2
e_x	0.0000	m
e_y	1.1597	m
I_x	0.4431	m^4
I_y	0.2173	m^4
I_{xy}	0.0000	m^4
y_b	1.1597	m
y_t	0.6473	m
S_b	0.3821	m^3
S_t	0.6846	m^3
$E_{cáp\ (Y)}$	0.9823	m
q =	28.7007	(KN/m)

Cross section at L/2



A	1.1715	m^2
e_x	0.0000	m
e_y	1.1597	m
I_x	0.4431	m^4
I_y	0.2173	m^4
I_{xy}	0.0000	m^4
y_b	1.1597	m
y_t	0.6473	m
S_b	0.3821	m^3
S_t	0.6846	m^3
$E_{cáp\ (Y)}$	1.0197	m
q =	28.7007	(KN/m)

4. LOSS OF PRESTRESS

4.1 Loss of prestressing force immediately (Instantaneous losses):

4.1.1 Friction between Prestressing Tendon and Duck:

Formula: $\Delta f_{pF} = f_{pj}\,(1-e^{-(Kx+\mu\alpha)})$ (5.9.5.2.2)

Xi: Length of tendon from the jacking end to any point under consideration

Section		Tendon no. 1		Tendon no. 2		Tendon no. 3		Tendon no. 4	
	X_i (mm)	$\Sigma\alpha$ (rad)	**Δf_{pF} (MPa)**	$\Sigma\alpha$ (rad)	**Δf_{pF} (MPa)**	$\Sigma\alpha$ (rad)	**Δf_{pF} (MPa)**	$\Sigma\alpha$ (rad)	**Δf_{pF} (MPa)**
Ancho.	0.00	0.0000	**0.00**	0.0000	**0.00**	0.0000	**0.00**	0.0000	**0.00**
Support	250.00	0.0028	**1.13**	0.0021	**0.89**	0.0014	**0.66**	0.0006	**0.42**
L/8	3900.00	0.0467	**18.41**	0.0346	**14.54**	0.0226	**10.66**	0.0105	**6.76**
L/4	7550.00	0.1316	**48.39**	0.0976	**37.71**	0.0637	**26.93**	0.0297	**16.06**
3L/8	11200.00	0.2575	**90.18**	0.1911	**69.87**	0.1246	**49.23**	0.0582	**28.24**
L/2	14850.00	0.4245	**142.53**	0.3150	**110.33**	0.2054	**77.24**	0.0959	**43.23**

4.1.2 Anchorage seating or Set:

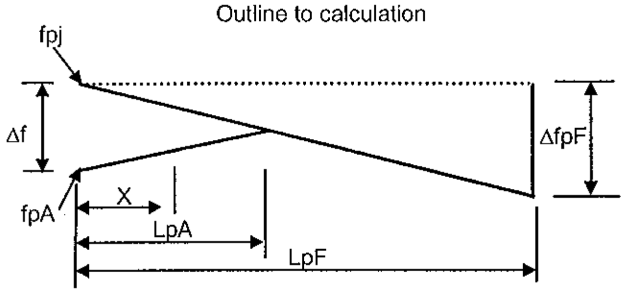
Formula

$$L_{pA} = \sqrt{\frac{E(\Delta L)L_{pF}}{\Delta f_{pF}}}$$

$$\Delta f = \frac{2\Delta f_{pF} L_{pA}}{L_{pF}}$$

$$\Delta f_{pA} = \Delta f\left(1 - \frac{X}{L_{pA}}\right)$$

Outline to calculation



Trong đó:

- L_{pA} Effective length due to anchorage set
- E Cable modulus of elasticity
- ΔL Setting length
- L_{pF} The length from anchorage to point that loss stress due to friction was known
- Δf_{pF} The loss stress value at the point that the leng from anchorage ti it is L_{pF}
- Δf The loss stress value at Anchorage

Choice the length from anchorage to point that loss stress due to friction was known (LpF) and calculation follow:

Tendon no.1		Xi (mm)	Δf_{pA} (MPa)
LpF =	14850	0	**213.02**
Δf_{pF} =	142.53	250	**208.22**
LpA =	11097.5	3900	**138.16**
Δf =	213.02	7550	**68.10**
		11200	**0.00**
		14850	**0.00**

Tendon no.2		Xi (mm)	Δf_{pA} (MPa)
LpF =	14850	0	**187.42**
Δf_{pF} =	110.33	250	**183.71**
LpA =	12613.2	3900	**129.47**
Δf =	187.42	7550	**75.24**
		11200	**21.00**
		14850	**0.00**

Tendon no.3		Xi (mm)	Δf_{pA} (MPa)
LpF =	14850	0	**154.48**
Δf_{pF} =	77.24	250	**151.88**
LpA =	14850.0	3900	**113.91**
Δf =	154.48	7550	**75.94**
		11200	**37.97**
		14850	**0.00**

Tendon no.4		Xi (mm)	Δf_{pA} (MPa)
LpF =	14850	0	**86.46**
Δf_{pF} =	43.23	250	**85.01**
LpA =	14850.0	3900	**63.75**
Δf =	86.46	7550	**42.50**
		11200	**21.25**
		14850	**0.00**

4.1.3 Elastic deformation of concrete:

Formula

$$\Delta f_{ES} = \frac{N-1}{2N}\frac{E_p}{E_{ci}} f_{cgp} \quad (5.9.5.2.3b\text{-}1)$$

In which:

Number of tendon	$N =$	4.00 (Tendon)
Cable modulus of elasticity	$Ep =$	197000.0 MPa
Concrete strength at transfer	$f'_{ci} =$	40.50 MPa
Unit weight of concrete	$\gamma_c =$	2450.00 kg/m3
Concrete modulus of elasticity at transfer	$E_{ci} =$	33185.3 MPa

Total stress of concrete in the Tendon centroid (f_{cgp}) due to prestressing force and self weigh of girder

$$f_{cgp} = \frac{F_j}{A} + \frac{F_j e^2}{I_X} - \frac{M_{DC} e}{I_X}$$

Compression force due to pretressing consider loss stress: $F_j = N{*}f_{pj}{*}A_s - A_s{*}\Sigma(\Delta f_{pFi} + \Delta f_{pAi})$

- A — Area of girder cross section
- Ix — Inertia Moment of Girder cross section
- e — Distance from tendon centroid to neutral line of girder section
- M_{DC} — Maximum moment due to self weigh of girder at jacking

Total loss stress due to friction and Anchorage:

Section	Xi	Tendon1 $\Delta fpF+\Delta f_{pA}$	Tendon2 $\Delta fpF+\Delta f_{pA}$	Tendon3 $\Delta fpF+\Delta f_{pA}$	Tendon4 $\Delta fpF+\Delta f_{pA}$	SUM $\Sigma(\Delta fpF+\Delta f_{pA})$	ΣFj
	(mm)	(MPa)	(MPa)	(MPa)	(MPa)	(MPa)	(kN)
Anchorage	0	213.02	187.42	154.48	86.46	641.38	6760.88
Support	250	209.35	184.60	152.54	85.43	631.92	6774.89
L/8	3900	156.57	144.01	124.57	70.52	495.67	6976.61
L/4	7550	116.49	112.94	102.87	58.56	390.86	7131.77
3L/8	11200	90.18	90.87	87.20	49.49	317.74	7240.02
L/2	14850	142.53	110.33	77.24	43.23	373.33	7157.73

Loss stress due to Elastic deformation of concrete

Section	Xi (mm)	Fj (kN)	A (mm2)	Ix (mm4)	e (mm)	M_{DC} kNm	f_{cgp} (MPa)	Δf_{ES} (MPa)
Anchorage	0	6760.88	1.1E+06	2.5E+11	76.53	0.00	6.05	**13.47**
Support	250	6774.89	1.1E+06	2.5E+11	76.53	0.00	6.07	**13.50**
L/8	3900	6976.61	8.2E+05	2.2E+11	331.40	931.08	10.52	**23.42**
L/4	7550	7131.77	7.1E+05	2.1E+11	512.45	1596.14	15.02	**33.45**
3L/8	11200	7240.02	7.1E+05	2.1E+11	624.82	1995.17	17.63	**39.25**
L/2	14850	7157.73	7.1E+05	2.1E+11	662.28	2128.18	18.22	**40.56**

Total loss of prestressing force immediately - Remaining prestressing force:

Tendon1 Section	Xi (mm)	Δf_{pF} (MPa)	Δf_{pA} (MPa)	Δf_{ES} (MPa)	ΣΔ (MPa)	F_j^1 (kN)	(α) (rad)	$Fj^1{*}Cos(\alpha)$ (kN)	$Fj^1{*}Sin(\alpha)$ (kN)
anchorage	0	0.00	213.02	13.47	226.50	1592.28	0.1655	1570.53	262.28
Support	250	1.13	208.22	13.50	222.85	1597.68	0.1627	1576.57	258.86
L/8	3900	18.41	138.16	23.42	179.99	1661.14	0.1225	1648.69	203.03
L/4	7550	48.39	68.10	33.45	149.94	1705.63	0.0819	1699.91	139.56
3L/8	11200	90.18	0.00	39.25	129.43	1735.99	0.0410	1734.53	71.20
L/2	14850	142.53	0.00	40.56	183.08	1656.56	0.0000	1656.56	0.00

Tendon2 Section	Xi (mm)	Δf_{pF} (MPa)	Δf_{pA} (MPa)	Δf_{ES} (MPa)	ΣΔ (MPa)	F_j^2 (kN)	(α) (rad)	$Fj^2{*}Cos(\alpha)$ (kN)	$Fj^2{*}Sin(\alpha)$ (kN)
anchorage	0	0.00	187.42	13.47	200.90	1630.18	0.1233	1617.81	200.46
Support	250	0.89	183.71	13.50	198.10	1634.32	0.1212	1622.33	197.63
L/8	3900	14.54	129.47	23.42	167.43	1679.73	0.0911	1672.77	152.83
L/4	7550	37.71	75.24	33.45	146.39	1710.88	0.0608	1707.72	104.02
3L/8	11200	69.87	21.00	39.25	130.13	1734.96	0.0304	1734.16	52.81
L/2	14850	110.33	0.00	40.56	150.89	1704.22	0.0000	1704.22	0.00

Tendon3 Section	Xi (mm)	Δf_{pF} (MPa)	Δf_{pA} (MPa)	Δf_{ES} (MPa)	ΣΔ (MPa)	F_j^3 (kN)	(α) (rad)	$Fj^3{*}Cos(\alpha)$ (kN)	$Fj^3{*}Sin(\alpha)$ (kN)
anchorage	0	0.00	154.48	11.98	166.46	1681.17	0.0806	1675.71	135.41
Support	250	0.66	151.88	12.00	164.54	1684.01	0.0793	1678.72	133.37
L/8	3900	10.66	113.91	20.81	145.38	1712.37	0.0595	1709.34	101.85
L/4	7550	26.93	75.94	29.73	132.60	1731.30	0.0397	1729.94	68.72
3L/8	11200	49.23	37.97	34.89	122.09	1746.85	0.0199	1746.51	34.69
L/2	14850	77.24	0.00	36.05	113.29	1759.88	0.0000	1759.88	0.00

Tendon4 Section	Xi (mm)	Δf_{pF} (MPa)	Δf_{pA} (MPa)	Δf_{ES} (MPa)	$\Sigma\Delta$ (MPa)	F_j^4 (kN)	(α) (rad)	$Fj^4*Cos(\alpha)$ (kN)	$Fj^4*Sin(\alpha)$ (kN)
anchorage	0	0.00	86.46	8.98	95.44	1786.31	0.0377	1785.04	67.31
Support	250	0.42	85.01	9.00	94.43	1787.81	0.0371	1786.58	66.24
L/8	3900	6.76	63.75	15.61	86.13	1800.10	0.0278	1799.40	50.04
L/4	7550	16.06	42.50	22.30	80.86	1807.90	0.0185	1807.59	33.51
3L/8	11200	28.24	21.25	26.17	75.66	1815.59	0.0093	1815.52	16.83
L/2	14850	43.23	0.00	27.04	70.27	1823.58	0.0000	1823.58	0.00

SUM 1to4 Section	Xi (mm)	ΣF_j (kN)	$Fj*Cos(\alpha)$ (kN)	$Fj*Sin(\alpha)$ (kN)	$e_{cáp}$ (mm)	$M_i = \Sigma F_i Cos(\alpha)*e_{cáp}$ (kNm)
anchorage	0	6689.95	6649.09	665.47	76.53	508.86
Support	250	6703.82	6664.20	656.10	76.53	510.02
L/8	3900	6853.34	6830.20	507.75	331.40	2263.52
L/4	7550	6955.72	6945.16	345.80	512.45	3559.05
3L/8	11200	7033.40	7030.71	175.53	624.82	4392.93
L/2	14850	6944.24	6944.24	0.00	662.28	4599.01

4.2. Loss of prestressing force at service stage (time - dependent losses):

4.2.1 Loss of prestress due to Shrinkage:

Formula: $\Delta fp_{SH} = (93 - 0.85*H)$

Relative humidity of environment H = 80.00 %

Δfp_{SH} = 25.00 (MPa)

4.2.2 Loss of prestress due to Creep:

Formula $\Delta fp_{CR} = 12.0\, f_{cgp} - 7.0 * \Delta f_{cdp}$

In which:

f_{cgp} Stress in concrete at tendons centroid (f_{cgp}) due to prestressing tendon and self weigh of girder

Δf_{cdp} Stess at tendons centroid changes due to permanent load, except dead load action at transfer

Section	Xi		Interior Girder		Exterior Girder	
	(m)	f_{cgp} (MPa)	Δf_{cdp} (MPa)	Δfp_{CR} (MPa)	Δf_{cdp} (MPa)	Δfp_{CR} (MPa)
Support	0.00	6.07	0.00	**72.78**	0.00	**72.78**
L/8	3.65	10.52	0.92	**119.79**	1.69	**114.40**
L/4	7.30	15.02	4.89	**146.02**	4.45	**149.12**
3L/8	10.95	17.63	5.26	**174.74**	6.62	**165.24**
L/2	14.60	18.22	8.20	**161.20**	7.44	**166.54**

4.2.3 Loss of prestress due to Relaxation:

(a) At transfer:

Formula:
$$\Delta f_{pR1} = \frac{\log(24t)}{40}\left[\frac{f_{pj}}{f_{py}} - 0,55\right]f_{pj}$$

In which:

t : time estimated in days from stressing to transfer t = 0.00 days

fpy : Specified yeild strength of prestressing steel f_{py} = 1674.00 (MPa)

fpj : Initial stress in the tendon at the end of stressing

Section	Xi (m)	fpj (MPa)	Δfp_{R1} (MPa)
Support	0.00	1288.50	**0.00**
L/8	3.65	1278.58	**0.00**
L/4	7.30	1268.55	**0.00**
3L/8	10.95	1262.75	**0.00**
L/2	14.60	1261.44	**0.00**

(b) After Transfer:

Formula: $\Delta fp_{R2} = 30\%*(138 - 0.3\,\Delta fp_F - 0.4\,\Delta f_{ES} - 0.2\,(\Delta fp_{SH} + \Delta fp_{CR}))$

Interior Girder						
Section	Xi (m)	Δf_{pF} (MPa)	Δf_{pES} (MPa)	Δfp_{SH} (MPa)	Δfp_{CR} (MPa)	**Δfp_{R2} (MPa)**
Support	0.00	0.00	13.50	25.00	18.20	**37.19**
L/8	3.65	0.00	23.42	25.00	29.95	**35.29**
L/4	7.30	0.00	33.45	25.00	36.51	**33.70**
3L/8	10.95	0.00	39.25	25.00	43.68	**32.57**
L/2	14.60	0.00	40.56	25.00	40.30	**32.62**

Exterior Girder						
Section	Xi (m)	Δf_{pF} (MPa)	Δf_{pES} (MPa)	Δfp_{SH} (MPa)	Δfp_{CR} (MPa)	**Δfp_{R2} (MPa)**
Support	0.00	0.00	13.50	25.00	18.20	**37.19**
L/8	3.65	0.00	23.42	25.00	28.60	**35.37**
L/4	7.30	0.00	33.45	25.00	37.28	**33.65**
3L/8	10.95	0.00	39.25	25.00	41.31	**32.71**
L/2	14.60	0.00	40.56	25.00	41.64	**32.54**

TOTAL LOSS STRESS AT SERVICE STAGE

Interior Girder						
Setion	Xi (m)	Δfp_{SH} (MPa)	Δfp_{CR} (MPa)	Δfp_{R1} (MPa)	Δfp_{R2} (MPa)	**Sum (MPa)**
Support	0.00	25.00	72.78	0.00	37.19	**134.97**
L/8	3.65	25.00	119.79	0.00	35.29	**180.08**
L/4	7.30	25.00	146.02	0.00	33.70	**204.72**
3L/8	10.95	25.00	174.74	0.00	32.57	**232.31**
L/2	14.60	25.00	161.20	0.00	32.62	**218.81**

Exterior Girder						
Section	Xi (m)	Δfp_{SH} (MPa)	Δfp_{CR} (MPa)	Δfp_{R1} (MPa)	Δfp_{R2} (MPa)	**Sum (MPa)**
Support	0.00	25.00	72.78	0.00	37.19	**134.97**
L/8	3.65	25.00	114.40	0.00	35.37	**174.77**
L/4	7.30	25.00	149.12	0.00	33.65	**207.77**
3L/8	10.95	25.00	165.24	0.00	32.71	**222.95**
L/2	14.60	25.00	166.54	0.00	32.54	**224.08**

4.3. Total Prestressing force consider loss in the service stage:

Interior Girder

Tendon1	Xi	$\Sigma\Delta_{pT}$	F_j^1	(α)	$F_j^1*\cos(\alpha)$	$F_j^1*\sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	357.83	1397.85	0.1627	1379.38	226.48
L/8	3.65	360.07	1394.53	0.1225	1384.07	170.44
L/4	7.30	354.65	1402.55	0.0819	1397.84	114.76
3L/8	10.95	361.74	1392.06	0.0410	1390.89	57.09
L/2	14.60	401.89	1332.61	0.0000	1332.61	0.00

Tendon2	Xi	$\Sigma\Delta_{pT}$	F_j^2	(α)	$F_j^2*\cos(\alpha)$	$F_j^2*\sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	333.08	1434.49	0.1233	1423.61	176.39
L/8	3.65	347.51	1413.12	0.1212	1402.75	170.88
L/4	7.30	351.11	1407.80	0.0911	1401.96	128.09
3L/8	10.95	362.43	1391.03	0.0608	1388.46	84.57
L/2	14.60	369.70	1380.27	0.0304	1379.63	42.02

Tendon3	Xi	$\Sigma\Delta_{pT}$	F_j^3	(α)	$F_j^3*\cos(\alpha)$	$F_j^3*\sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	301.43	1481.35	0.0793	1476.69	117.32
L/8	3.65	344.62	1417.40	0.0595	1414.89	84.31
L/4	7.30	350.10	1409.29	0.0397	1408.18	55.94
3L/8	10.95	364.90	1387.37	0.0199	1387.10	27.55
L/2	14.60	340.90	1422.90	0.0000	1422.90	0.00

Tendon4	Xi	$\Sigma\Delta_{pT}$	F_j^4	(α)	Fj^4*Cos(α)	Fj^4*Sin(α)
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	230.42	1586.48	0.0371	1585.39	58.78
L/8	3.65	274.51	1521.20	0.0278	1520.61	42.28
L/4	7.30	290.85	1497.01	0.0185	1496.75	27.75
3L/8	10.95	313.17	1463.97	0.0093	1463.91	13.57
L/2	14.60	294.47	1491.64	0.0000	1491.64	0.00

SUM 1to4	Xi	ΣF_j	Fj*Cos(α)	Vp=Fj*Sin(α)	e_{cable}	$M_i = \Sigma F_j Cos(\alpha) * e_{cáp}$
Section	(mm)	(kN)	(kN)	(kN)	(m)	(kNm)
Support	0.00	5900.17	5865.07	578.98	0.08	448.9
L/8	3.65	5746.24	5722.32	467.91	0.33	1896.4
L/4	7.30	5716.64	5704.73	326.53	0.51	2923.4
3L/8	10.95	5634.43	5630.35	182.78	0.62	3518.0
L/2	14.60	5627.43	5626.79	42.02	0.66	3726.5

Exterior Girder

Tendon1	Xi	$\Sigma\Delta_{pT}$	F_j^1	(α)	Fj^1*Cos(α)	Fj^1*Sin(α)
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	357.83	1397.85	0.1627	1379.38	226.48
L/8	3.65	354.76	1402.39	0.1225	1391.88	171.40
L/4	7.30	357.71	1398.03	0.0819	1393.34	114.39
3L/8	10.95	352.38	1405.92	0.0410	1404.73	57.66
L/2	14.60	407.16	1324.81	0.0000	1324.81	0.00

Tendon2	Xi	$\Sigma\Delta_{pT}$	F_j^2	(α)	Fj^2*Cos(α)	Fj^2*Sin(α)
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	333.08	1434.49	0.1233	1423.61	176.39
L/8	3.65	342.20	1420.99	0.1212	1410.56	171.83
L/4	7.30	354.16	1403.28	0.0911	1397.46	127.68
3L/8	10.95	353.07	1404.89	0.0608	1402.29	85.41
L/2	14.60	374.97	1372.47	0.0304	1371.84	41.78

Tendon3	Xi	$\Sigma\Delta_{pT}$	F_j^3	(α)	Fj^3*Cos(α)	Fj^3*Sin(α)
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	301.43	1481.35	0.0793	1476.69	117.32
L/8	3.65	339.31	1425.27	0.0595	1422.74	84.78
L/4	7.30	353.15	1404.77	0.0397	1403.66	55.76
3L/8	10.95	355.54	1401.23	0.0199	1400.95	27.83
L/2	14.60	346.17	1415.10	0.0000	1415.10	0.00

Tendon4	Xi	$\Sigma\Delta_{pT}$	F_j^4	(α)	Fj^4*Cos(α)	Fj^4*Sin(α)
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	230.42	1586.48	0.0371	1585.39	58.78
L/8	3.65	269.20	1529.06	0.0278	1528.47	42.50
L/4	7.30	293.90	1492.49	0.0185	1492.24	27.66
3L/8	10.95	303.81	1477.83	0.0093	1477.76	13.70
L/2	14.60	299.74	1483.84	0.0000	1483.84	0.00

SUM 1to4	Xi	ΣF_j	Fj*Cos(α)	Vp=Fj*Sin(α)	e_{caple}	$M_i = \Sigma F_j Cos(\alpha) * e_{cáp}$
Section	(mm)	(kN)	(kN)	(kN)	(m)	(kNm)
Support	0.00	5900.17	5865.07	578.98	0.08	448.9
L/8	3.65	5777.71	5753.65	470.51	0.33	1906.8
L/4	7.30	5698.57	5686.69	325.49	0.51	2914.2
3L/8	10.95	5689.86	5685.74	184.60	0.62	3552.6
L/2	14.60	5596.23	5595.59	41.78	0.66	3705.8

5. FIBRE STRESS CHECK:

Formula:

Top fibre: $f_{ti} = \frac{F_i}{A} - \frac{F_i e}{S_t} + \frac{M_{DC}}{S_t}$ Bottom fibre $f_{bi} = \frac{F_i}{A} + \frac{F_i e}{S_b} - \frac{M_{DC}}{S_b}$

Note (+) : Compression tresses ; (-) Tension stresses

Concrete strength at transfer fci' = 0.9 fc = 40.50 MPa

Comprresion stress Limit at transfer 0.6 fci' = 24.30 MPa

Tension stress Limit at transfer 0.25 SQRT(fci') < 1.38 = -1.38 MPa (5.9.4.1.2-1)

Setion	Xi	A	St	Sb	Fj'*Cos(α)	e	M_{DC}	f_{ti}	f_{bi}	Kiểm tra	
	(mm)	(mm2)	(mm3)	(mm3)	(kN)	(mm)	(kNm)	(MPa)	(MPa)	f_{ti}	f_{bi}
G. end	0	1.15E+06	3.19E+08	3.07E+08	**6649.09**	76.53	0.00	4.20	7.46	OK	OK
Support	250	1.15E+06	3.19E+08	3.07E+08	**6664.20**	76.53	0.00	4.21	7.47	OK	OK
L/8	3900	8.23E+05	2.83E+08	2.77E+08	**6830.20**	331.40	931.08	3.58	13.11	OK	OK
L/4	7550	7.07E+05	2.68E+08	2.66E+08	**6945.16**	512.45	1596.14	2.50	17.19	OK	OK
3L/8	11200	7.07E+05	2.68E+08	2.66E+08	**7030.71**	624.82	1995.17	1.00	18.94	OK	OK
L/2	14850	7.07E+05	2.68E+08	2.66E+08	**6944.24**	662.28	2128.18	0.60	19.10	OK	OK

5.2 Stress check during contruction the deck:

5.2.1 Increase load:

Exterior Diaphragms beam DC_{dn1} = 47.95 (kN)

Interior Diaphragms beam DC_{dn1} = 33.83 (kN)

Precast plank DC_{VK} = 3.55 (kN/m)

Wet concrete of deck DC_{mc} = 13.32 (kN/m)

5.2.2 Stress check:

Compression strength of concrete fc = 45.00 MPa

Compression stress limit 0.45 fc = 20.25 MPa (5.9.4.2.1-1)

Tension stress limit 0.5 SQRT(fc) = -3.35 MPa (5.9.4.2.2-1)

Setion	Xi	A	St	Sb	Fi	e	M_{DC}	f_{ti}	f_{bi}	Kiểm tra	
	(mm)	(mm2)	(mm3)	(mm3)	(kN)	(mm)	(kNm)	(MPa)	(MPa)	f_{ti}	f_{bi}
G. end	0	1.15E+06	3.19E+08	3.07E+08	6649.09	76.53	0.00	4.20	7.46	OK	OK
Support	250	1.15E+06	3.19E+08	3.07E+08	6664.20	76.53	0.00	4.21	7.47	OK	OK
L/8	3900	8.23E+05	2.83E+08	2.77E+08	6830.20	331.40	2392.73	8.75	7.83	OK	OK
L/4	7550	7.07E+05	2.68E+08	2.66E+08	6945.16	512.45	3037.25	7.88	11.78	OK	OK
3L/8	11200	7.07E+05	2.68E+08	2.66E+08	7030.71	624.82	3796.57	7.72	12.18	OK	OK
L/2	14850	7.07E+05	2.68E+08	2.66E+08	6944.24	662.28	4049.67	7.77	11.89	OK	OK

5.3 Stress check at the top fibre of Girder - Service state :

5.3.1 Due to prestressing tendon and self weigh of girder - Service limit state I:

Compression Stress Limit: 0.45 fc = 20.25 MPa (5.9.4.2.1-1)

Tensioh Stress Limit: - 0.5*SQRT(fc) = -3.35 MPa

$$f_f = \frac{P_{pe}}{A} - \frac{P_{pe}e_c}{S_t} + \frac{M_g + M_s}{S_t} + \frac{M_{SDL}}{S_{tg}}$$

Interior Girder

Setion	Xi	A	S_t	S_{tg}	P_{pe}	P_{pe}*e_c	$M_g + M_s$	M_{SDL}	f_t	Check
	(mm)	(mm2)	(mm3)	(mm3)	(kN)	(kNm)	(kNm)	(kNm)	(MPa)	f_t
Support	250	1.15E+06	3.19E+08	1.00E+09	5865.07	448.86	0.00	0.00	3.706	OK
L/8	3900	8.23E+05	2.83E+08	1.02E+09	5722.32	1896.37	1771.73	428.12	6.928	OK
L/4	7550	7.07E+05	2.68E+08	1.04E+09	5704.73	2923.40	3037.25	733.92	9.198	OK
3L/8	11200	7.07E+05	2.68E+08	1.04E+09	5630.35	3517.95	3796.57	917.40	9.883	OK
L/2	14850	7.07E+05	2.68E+08	1.04E+09	5626.79	3726.49	4049.67	978.56	10.103	OK

Exterior Girder

Setion	Xi	A	S_t	S_{tg}	P_{pe}	P_{pe}*e_c	$M_g + M_s$	M_{SDL}	f_t	Check
	(mm)	(mm2)	(mm3)	(mm3)	(kN)	(kNm)	(kNm)	(kNm)	(MPa)	f_t
Support	250	1.15E+06	3.19E+08	9.99E+08	5865.07	448.86	0.00	0.00	3.706	OK
L/8	3900	8.23E+05	2.83E+08	1.02E+09	5753.65	1906.76	1661.91	430.69	6.544	OK
L/4	7550	7.07E+05	2.68E+08	1.04E+09	5686.69	2914.15	2848.98	738.32	8.510	OK
3L/8	11200	7.07E+05	2.68E+08	1.04E+09	5685.74	3552.56	3561.23	922.90	8.961	OK
L/2	14850	7.07E+05	2.68E+08	1.04E+09	5595.59	3705.83	3798.64	984.43	9.206	OK

5.3.2 Due to 1/2 (Prestressing tendon + self weigh of girder) and Live load - Service limit state I:

Compression Stress Limit: 0.40 fc = 18.00 MPa (5.9.4.2.1-1)

Tension Stress Limit: - 0.5*SQRT(fc) = -3.35 MPa

$$f_t = 0.5\left(\frac{P_{pe}}{A} - \frac{P_{pe}e_c}{S_t} + \frac{M_g + M_s}{S_t} + \frac{M_{SDL}}{S_{tg}}\right) + \frac{M_{LL}}{S_{tg}}$$

Interior Girder

Setion	Xi (mm)	A (mm2)	S_t (mm3)	S_{tg} (mm3)	P_{pe} (kN)	P_{pe}*e_c (kNm)	M_g + M_s (kNm)	M_{SDL} (kNm)	M_{LL} (kNm)	f_t (MPa)	Check f_t
Support	250	1.15E+06	3.19E+08	1.00E+09	5865.07	448.86	0.00	0.00	0.00	1.853	OK
L/8	3900	8.23E+05	2.83E+08	1.02E+09	5722.32	1896.37	1771.73	428.12	1177.94	4.614	OK
L/4	7550	7.07E+05	2.68E+08	1.04E+09	5704.73	2923.40	3037.25	733.92	1988.69	6.502	OK
3L/8	11200	7.07E+05	2.68E+08	1.04E+09	5630.35	3517.95	3796.57	917.40	2449.70	7.286	OK
L/2	14850	7.07E+05	2.68E+08	1.04E+09	5626.79	3726.49	4049.67	978.56	2578.43	7.519	OK

Exterior Girder

Setion	Xi (mm)	A (mm2)	S_t (mm3)	S_{tg} (mm3)	P_{pe} (kN)	P_{pe}*e_c (kNm)	M_g + M_s (kNm)	M_{SDL} (kNm)	M_{LL} (kNm)	f_t (MPa)	Check f_t
Support	250	1147000.0	3.19E+08	9.99E+08	5865.07	448.86	0.00	0.00	0.00	1.853	OK
L/8	3900	823265.0	2.83E+08	1.02E+09	5753.65	1906.76	1661.91	430.69	1358.92	4.601	OK
L/4	7550	706850.0	2.68E+08	1.04E+09	5686.69	2914.15	2848.98	738.32	2294.23	6.454	OK
3L/8	11200	706850.0	2.68E+08	1.04E+09	5685.74	3552.56	3561.23	922.90	2826.07	7.189	OK
L/2	14850	706850.0	2.68E+08	1.04E+09	5595.59	3705.83	3798.64	984.43	2974.58	7.455	OK

5.3.3 Due to prestressing tendon + self weigh of girder + live load - Service limit state I:

Compression Stress Limit: 0.60 fc = 27.00 MPa (5.9.4.2.1-1)

Tension Stress Limit: - 0.5*SQRT(fc) = -3.35 MPa

$$f_t = \left(\frac{P_{pe}}{A} - \frac{P_{pe}e_c}{S_t} + \frac{M_g + M_s}{S_t} + \frac{M_{SDL}}{S_{tg}}\right) + \frac{M_{LL}}{S_{tg}}$$

Interior Girder

Setion	Xi (mm)	A (mm2)	S_t (mm3)	S_{tg} (mm3)	P_{pe} (kN)	P_{pe}*e_c (kNm)	M_g + M_s (kNm)	M_{SDL} (kNm)	M_{LL} (kNm)	f_t (MPa)	Check f_t
Support	250	1.15E+06	3.19E+08	1.00E+09	5865.07	448.86	0.00	0.00	0.00	3.706	OK
L/8	3900	8.23E+05	2.83E+08	1.02E+09	5722.32	1896.37	1771.73	428.12	1177.94	8.078	OK
L/4	7550	7.07E+05	2.68E+08	1.04E+09	5704.73	2923.40	3037.25	733.92	1988.69	11.101	OK
3L/8	11200	7.07E+05	2.68E+08	1.04E+09	5630.35	3517.95	3796.57	917.40	2449.70	12.228	OK
L/2	14850	7.07E+05	2.68E+08	1.04E+09	5626.79	3726.49	4049.67	978.56	2578.43	12.571	OK

Exterior Girder

Setion	Xi (mm)	A (mm2)	S_t (mm3)	S_{tg} (mm3)	P_{pe} (kN)	P_{pe}*e_c (kNm)	M_g + M_s (kNm)	M_{SDL} (kNm)	M_{LL} (kNm)	f_t (MPa)	Check f_t
Support	250	1.15E+06	3.19E+08	9.99E+08	5865.07	448.86	0.00	0.00	0.00	3.706	OK
L/8	3900	8.23E+05	2.83E+08	1.02E+09	5753.65	1906.76	1661.91	430.69	1358.92	7.873	OK
L/4	7550	7.07E+05	2.68E+08	1.04E+09	5686.69	2914.15	2848.98	738.32	2294.23	10.709	OK
3L/8	11200	7.07E+05	2.68E+08	1.04E+09	5685.74	3552.56	3561.23	922.90	2826.07	11.670	OK
L/2	14850	7.07E+05	2.68E+08	1.04E+09	5595.59	3705.83	3798.64	984.43	2974.58	12.058	OK

5.4 Stress check at the top fibre of deck - Service state:

5.4.1 Due to additional load (dead load part 2) - Service limit state I:

Compression Stress Limit: 0.45 fc = 15.75 MPa (5.9.4.2.1-1)

$$f_t = \frac{M_{SDL}}{S_{tc}}$$

Setion	Xi	MSDL (kNm)		S_{tc} (mm3)		f_t (MPa)		Check	
	(mm)	in.Girder	Ex.Girder	in.Girder	Ex.Girder	in.Girder	Ex.Girder	in.Girder	Ex.Girder
Support	250.00	0.00	0.00	6.2E+08	6.14E+08	0.000	0.000	OK	OK
L/8	3900.00	428.12	430.69	6.1E+08	6.05E+08	0.707	0.712	OK	OK
L/4	7550.00	733.92	738.32	6E+08	6.04E+08	1.214	1.223	OK	OK
3L/8	11200.00	917.40	922.90	6E+08	6.04E+08	1.518	1.529	OK	OK
L/2	14850.00	978.56	984.43	6E+08	6.04E+08	1.619	1.630	OK	OK

5.4.2 Due to additional load (dead load part 2) and live load - Service limit state I:

Compression Stress Limit: 0.6 fc = 21.00 MPa (5.9.4.2.1-1)

$$f_{tc} = \frac{M_{SDL} + M_{LL}}{S_{tc}}$$

Setion	Xi	MSDL + MLL (kNm)		S_{tc} (mm3)		f_t (MPa)		Check	
	(mm)	in.Girder	Ex.Girder	in.Girder	Ex.Girder	in.Girder	Ex.Girder	in.Girder	Ex.Girder
Support	250.00	0.00	0.00	6.2E+08	6.14E+08	0.000	0.000	OK	OK
L/8	3900.00	1606.07	1789.61	6.1E+08	6.05E+08	2.652	2.959	OK	OK
L/4	7550.00	2722.61	3032.55	6E+08	6.04E+08	4.504	5.023	OK	OK
3L/8	11200.00	3367.10	3748.97	6E+08	6.04E+08	5.570	6.209	OK	OK
L/2	14850.00	3556.99	3959.01	6E+08	6.04E+08	5.884	6.557	OK	OK

5.5 Stress check at the bottom fibre of girder - Service III (stage III):

Compression Stress Limit: 0.45 fc = 20.25 MPa (5.9.4.2.1-1)

Tension Stress Limit: - 0.5*SQRT(fc) = -3.35 MPa (5.9.4.2.1-1)

$$f_b = \frac{P_{pe}}{A} + \frac{P_{pe}e_c}{S_b} - \frac{M_g + M_s}{S_b} - \frac{M_{SDL} + M_{LL}}{S_{bc}}$$

Interior Girder

Setion	Xi (mm)	A (mm2)	S_b (mm3)	S_{bc} (mm3)	P_{pe} (kN)	P_{pe}*e_c (kNm)	M_g + M_s (kNm)	M_{SDL} (kNm)	M_{LL} (kNm)	f_b (MPa)	Check f_b
Support	250	1.15E+06	3.07E+08	4.78E+08	5865.07	448.86	0.00	0.00	0.00	6.578	OK
L/8	3900	8.23E+05	2.77E+08	4.10E+08	5722.32	1896.37	1771.73	428.12	1177.94	4.056	OK
L/4	7550	7.07E+05	2.66E+08	3.82E+08	5704.73	2923.40	3037.25	733.92	1988.69	1.561	OK
3L/8	11200	7.07E+05	2.66E+08	3.82E+08	5630.35	3517.95	3796.57	917.40	2449.70	-0.608	OK
L/2	14850	7.07E+05	2.66E+08	3.82E+08	5626.79	3726.49	4049.67	978.56	2578.43	-1.210	OK

Exterior Girder

Setion	Xi (mm)	A (mm2)	S_b (mm3)	S_{bc} (mm3)	P_{pe} (kN)	P_{pe}*e_c (kNm)	M_g + M_s (kNm)	M_{SDL} (kNm)	M_{LL} (kNm)	f_b (MPa)	Check f_b
Support	250	1.15E+06	3.07E+08	4.78E+08	5865.07	448.86	0.00	0.00	0.00	6.578	OK
L/8	3900	8.23E+05	2.77E+08	4.10E+08	5753.65	1906.76	1661.91	430.69	1358.92	4.167	OK
L/4	7550	7.07E+05	2.66E+08	3.82E+08	5686.69	2914.15	2848.98	738.32	2294.23	1.554	OK
3L/8	11200	7.07E+05	2.66E+08	3.82E+08	5685.74	3552.56	3561.23	922.90	2826.07	-0.321	OK
L/2	14850	7.07E+05	2.66E+08	3.82E+08	5595.59	3705.83	3798.64	984.43	2974.58	-1.236	OK

5.6 Stress check at the bottom fibre of girder - Service I (Stage III):

Compression Stress Limit: 0.45 fc = 20.25 MPa (5.9.4.2.1-1)

Tension Stress Limit: - 0.5*SQRT(fc) = -3.35 MPa (5.9.4.2.1-1)

$$f_b = \frac{P_{pe}}{A} + \frac{P_{pe}e_c}{S_b} - \frac{M_g + M_s}{S_b} - \frac{M_{SDL} + M_{LL}}{S_{bc}}$$

Interior Girder

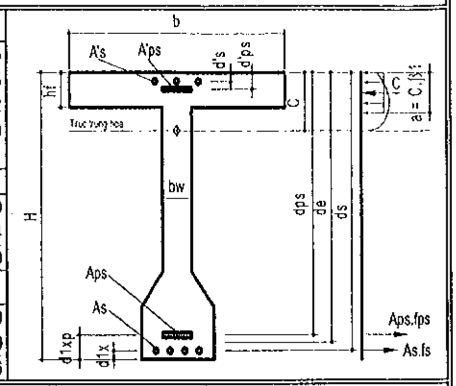
Setion	Xi (mm)	A (mm2)	S_b (mm3)	S_{bc} (mm3)	P_{pe} (kN)	P_{pe}*e_c (kNm)	M_g + M_s (kNm)	M_{SDL} (kNm)	M_{LL} (kNm)	f_b (MPa)	Check f_b
Support	250	1.15E+06	3.07E+08	4.78E+08	5865.07	448.86	0.00	0.00	0.00	6.58	OK
L/8	3900	8.23E+05	2.77E+08	4.10E+08	5722.32	1896.37	1771.73	428.12	1177.94	3.48	OK
L/4	7550	7.07E+05	2.66E+08	3.82E+08	5704.73	2923.40	3037.25	733.92	1988.69	0.52	OK
3L/8	11200	7.07E+05	2.66E+08	3.82E+08	5630.35	3517.95	3796.57	917.40	2449.70	-1.89	OK
L/2	14850	7.07E+05	2.66E+08	3.82E+08	5626.79	3726.49	4049.67	978.56	2578.43	-2.56	OK

Exterior Girder

Setion	Xi (mm)	A (mm2)	S_b (mm3)	S_{bc} (mm3)	P_{pe} (kN)	P_{pe}*e_c (kNm)	M_g + M_s (kNm)	M_{SDL} (kNm)	M_{LL} (kNm)	f_b (MPa)	Check f_b
Support	250	1.15E+06	3.07E+08	4.78E+08	5865.07	448.86	0.00	0.00	0.00	6.578	OK
L/8	3900	8.23E+05	2.77E+08	4.10E+08	5753.65	1906.76	1661.91	430.69	1358.92	3.504	OK
L/4	7550	7.07E+05	2.66E+08	3.82E+08	5686.69	2914.15	2848.98	738.32	2294.23	0.353	OK
3L/8	11200	7.07E+05	2.66E+08	3.82E+08	5685.74	3552.56	3561.23	922.90	2826.07	-1.800	OK
L/2	14850	7.07E+05	2.66E+08	3.82E+08	5595.59	3705.83	3798.64	984.43	2974.58	-2.793	OK

REINFORCEMENT OF GIRDER CHECKING - STRENGTH LOAD COMBINATION

MATERIALS



Sign	Parameter	Unit	Value
	NORMAL CONCRETE		
f'c	Compressive Strength of concrete at 28 days	Mpa	45
Ec	Modulus of Elasticity	Mpa	33915
fr	Modulus of Rupture	Mpa	4.2
gc	Unit weight of concrete	kN/m3	24.0
	PRESTRESSING STEEL		
fpu	Tensile strength of prestressing steel	Mpa	1860
fpy	Yield strength of prestressing steel	Mpa	1674
Ep	Modulus of Elasticity	Mpa	197000
	REINFORCEMENT		
fy	Yield strength	Mpa	400
Es	Modulus of Elasticity	Mpa	200000
nc	Ratio Es/Ec		6

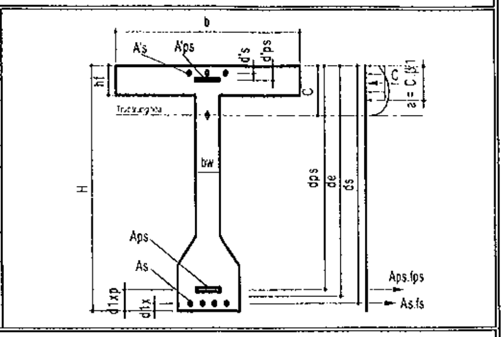
Sign	Parameters		Unit	Section: Support	L/8	L/4	3L/8	L/2
	INTERNAL FORCES AT SECTION							
	Combination			Strength	Strength	Strength	Strength	Strength
Qu	Shear		kN	1631	1288	951	620	296
Mu	Flexural Moment		kNm	0	5046	8588	10662	11303
Nu	Axial load		kN					
Tu	Torsional Moment		kNm					
	6.1 FLEXURAL MOMENT CHECKING							
H	Section height		m	1.800	1.800	1.800	1.800	1.800
d's	Dis. From comp. fiber to centroid of comp. Reinf		m	0.062	0.062	0.062	0.062	0.062
d1x	Dis. From tens. fiber to centroid of tension Reinf		m	0.062	0.062	0.062	0.062	0.062
	Cover to reinf		m	0.040	0.040	0.040	0.040	0.040
ds	Dis. From comp. fiber to centroid of tension Reinf		m	1.738	1.738	1.738	1.738	1.738
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	0.739	0.477	0.290	0.177	0.140
dps	Dis. From comp. fiber to centroid of tension prestressing steel		m	1.061	1.323	1.510	1.623	1.660
b	Width of the compression face of member		m	2.249	2.249	2.249	2.249	2.249
bw	Web width or diameter of a circular section		m	0.700	0.700	0.250	0.250	0.250
hf	Compression flange depth		m	0.200	0.200	0.200	0.200	0.200
Iz	Moment of inertia of section		m4	0.513	0.464	0.443	0.443	0.443
Amc	Section area		m2	1.613	1.289	1.172	1.172	1.172
	Steel choice							
	Tension prestressing steel	P.S type		15 T12.7	15 T12.7	15 T12.7	15 T12.7	15 T12.7
Aps		Number	tendons	5	5	5	5	5
		Area	m2	0.00740	0.00740	0.00740	0.00740	0.00740
	Compression prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	0 T00.0
A'ps		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
	Tension Reinforcement	Number	bars	6	6	6	6	6
As		Diameter	mm	16	16	16	16	16
		Area	m2	0.00121	0.00121	0.00121	0.00121	0.00121
	Compression Reinforcement	Number	bars	4	4	4	4	4
A's		Diameter	mm	12	12	12	12	12
		Area	m2	0.00045	0.00045	0.00045	0.00045	0.00045
	Shear reinforcement	Number	bars	2	2	2	2	2
A'c		Diameter	mm	16	16	16	16	16
		Area	m2	0.00040	0.00040	0.00040	0.00040	0.00040
f	Resistance factors for flexure		5.5.4.2	0.90	0.90	0.90	0.90	0.90
fv	Resistance factors for shear			0.90	0.90	0.90	0.90	0.90
fn	Resistance factors for axial force			1.00	1.00	1.00	1.00	1.00
b1	Stress block factor			0.729	0.729	0.729	0.729	0.729
c	Dis. Between centroid and top fiber		m	0.235	0.243	0.308	0.314	0.316
	For T section behavior		m	0.235	0.243	0.308	0.314	0.316
	For rectangular section behavior		m	0.212	0.215	0.216	0.216	0.217
fpe	Effective stress in the prestressing steel after losses		Mpa	1398	1395	1403	1387	1333
fps	Aver. stress in pres. steel at the time for which the nominal resistance		Mpa	1745	1764	1754	1759	1761
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	0.171	0.177	0.224	0.229	0.230
de	Corresp. effective depth from extreme comp. fiber to centroid of tensile force in the tensile reinf.		m	1.085	1.338	1.518	1.627	1.663
Mn	Nominal resistance		kNm	13274	16829	19082	20598	21104
Mr	Factored reistance		kNm	**11946**	**15146**	**17174**	**18538**	**18993**
Mu	Flexual moment		kNm	0	5046	8588	10662	11303
(5.7.3.2)	**Flexural moment Checking**			**OK**	**OK**	**OK**	**OK**	**OK**

	Limits for reinforcement						
c/de	Maximum reinforcement		0.22	0.18	0.20	0.19	0.19
	Maximum reinforcement Checking	<= 0.42	OK	OK	OK	OK	OK
r min	Mimimum reinforcement		0.08%	0.09%	0.10%	0.10%	0.10%
	Mimimum reinforcement Checking for RC	0.34%	N.a	N.a	N.a	N.a	N.a
1.2*Mcr	Craking moment	kNm	1661	1510	1507	1513	1515
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**		OK	OK	OK	OK	OK
(5.8.3.5)	Tensile force in steel should be satisfied - F_yc	kN	1779	5730	7823	8509	8436
	Checking As.fy+Aps.fps >= F_yc		Ok	Ok	Ok	Ok	Ok
(5.7.3.4)	**Conctrol of craking by distr. of reinf for RC member- Check?**		No	No	No	No	No
	Existing condition for structrure	1,2 or 3	1	1	1	1	1
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.058	0.058	0.058	0.058	0.058
Z	Crack width parameter	N/mm	30000	30000	30000	30000	30000
A	Area of concr. with same centroid as tens. Reinf	m2	0.043	0.043	0.043	0.043	0.043
fsa	Value	Mpa	220	220	220	220	220
0.6*fy		Mpa	240	240	240	240	240
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	220	220	220	220	220
x	Dist. From compression fiber to centroid	m	-	-	-	-	-
J.d	Arm	m	-	-	-	-	-
Icr	Moment of inertia of the cracked section	m4	-	-	-	-	-
fs	Tensile stress in reinforcement fs = Msls / (As*J.d)	Mpa	-	-	-	-	-
	Checking for control cracking fs<fsa		**N.a**	**N.a**	**N.a**	**N.a**	**N.a**
(5.10.8.2)	**Shrinkage and temperature Reinfocement (side distribution)**						
Areq	Area of required reinf	m2	0.00037	0.00030	0.00027	0.00027	0.00027
	Distribution on sides 8 D12	m2	0.00090	0.00090	0.00090	0.00090	0.00090
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45	0.45
	Checking		**OK**	**OK**	**OK**	**OK**	**OK**
	6.2 SHEAR FORCE CHECKING						
β	Factor indicating diag. cracked concr. to tension		6.8	6.8	6.8	6.8	6.8
θ	Angle of inclination of diagonal compressive	degree	27.00	27.00	27.00	27.00	27.00
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	90
bv	Effective web width as minimum web width - in dv	m	0.700	0.700	0.250	0.250	0.250
dv	Effective shear depth	m	1.296	1.296	1.406	1.512	1.548
	(de - a/2)	m	1.000	1.249	1.406	1.512	1.548
s	Spacing of stirrups	m	0.150	0.150	0.300	0.300	0.300
ncat	Amount of bars in spacing S	bars	2	2	2	2	2
Av	Shear reinf area in spacing S	m2	0.0004	0.0004	0.0004	0.0004	0.0004
β	Assume		6.8	6.7	6.8	5.9	5.5
θ	Assume	degree	27.00	27.00	27.00	27.00	27.00
v	Shear stress in concrete	kN/m2	1998	1577	3006	1823	850
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1435	1441	1454	1438	1381
e_x	Strain in tensile reinforcement		-5.31E-03	-3.24E-03	-2.19E-03	-1.76E-03	-1.55E-03
	if ex<0, multiple with reduce factor		-3.91E-04	-2.39E-04	-3.99E-04	-3.20E-04	-2.82E-04
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok	Ok
v/f'c	Ratio of shear stress and f'c		0.044	0.035	0.067	0.041	0.019
β	Final value		6.8	6.8	6.8	6.8	6.8
θ	Final value	degree	27.00	27.00	27.00	27.00	27.00
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	3425	3425	1327	1427	1461
Vs	Shear resistance provided by shear reinforcement	kN	2740	2740	1487	1599	1636
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	6165	6165	2814	3026	3097
Vn2	Vn2	kN	10206	10206	3955	4254	4353
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	6165	6165	2814	3026	3097
Vr	Factored shear resistance	kN	**5548**	**5548**	**2532**	**2724**	**2787**
Vu	Shear	kN	1631	1288	951	620	296
(5.8.2.7)	**Shear checking**		**OK**	**OK**	**OK**	**OK**	**OK**
	Region requiring transverse reinf Checking		Need	No need	Need	No need	No need
	Minimum shear reinf area	m2	0.0001	0.0001	0.0001	0.0001	0.0001
	Mimimum shear reinforcement Checking		OK	-	OK	-	-
	0.1*f'c*bv*dv	kN	4082	4082	1582	1701	1741
	Smax	m	0.60	0.60	0.60	0.60	0.60
	Maximum spacing Smax		OK	-	OK	-	-

REINFORCEMENT OF GIRDER CHECKING - SERVICE LOAD COMBINATION

MATERIALS

	NORMAL CONCRETE		
f'c	Compressive Strength of concrete at 28 days	Mpa	45
Ec	Modulus of Elasticity	Mpa	33915
fr	Modulus of Rupture	Mpa	4.2
gc	Unit weight of concrete	kN/m3	24.0
	PRESTRESSING STEEL		
fpu	Tensile strength of prestressing steel	Mpa	1860
fpy	Yield strength of prestressing steel	Mpa	1674
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	Support	L/8	L/4	3L/8	L/2
	INTERNAL FORCES AT SECTION							
	Combination			Service	Service	Service	Service	Service
Qu	Shear		kN	1120	877	637	401	169
Mu	Flexural Moment		kNm	0	3452	5882	7310	7758
Nu	Axial load		kN					
Tu	Torsional Moment		kNm					
	6.1 FLEXURAL MOMENT CHECKING							
H	Section height		m	1.823	1.823	1.823	1.823	1.823
d's	Dis. From comp. fiber to centroid of comp. Reinf		m	0.062	0.062	0.062	0.062	0.062
d1x	Dis. From tens. fiber to centroid of tension Reinf		m	0.062	0.062	0.062	0.062	0.062
	Cover to reinf		m	0.040	0.040	0.040	0.040	0.040
ds	Dis. From comp. fiber to centroid of tension Reinf		m	1.761	1.761	1.761	1.761	1.761
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	0.739	0.477	0.290	0.177	0.140
dps	Dis. From comp. fiber to centroid of tension prestressing steel		m	1.083	1.345	1.533	1.645	1.683
b	Width of the compression face of member		m	2.550	2.550	2.550	2.550	2.550
bw	Web width or diameter of a circular section		m	0.700	0.700	0.250	0.250	0.250
hf	Compression flange depth		m	0.223	0.223	0.223	0.223	0.223
Iz	Moment of inertia of section		m4	0.513	0.464	0.443	0.443	0.443
Amc	Section area		m2	1.613	1.289	1.172	1.172	1.172
	Steel choice							
	Tension prestressing steel	P.S type		15 T12.7	15 T12.7	15 T12.7	15 T12.7	15 T12.7
Aps		Number	tendons	5	5	5	5	5
		Area	m2	0.00740	0.00740	0.00740	0.00740	0.00740
	Compression prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	0 T00.0
A'ps		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
	Tension Reinforcement	Number	bars	6	6	6	6	6
As		Diameter	mm	16	16	16	16	16
		Area	m2	0.00121	0.00121	0.00121	0.00121	0.00121
	Compression Reinforcement	Number	bars	4	4	4	4	4
A's		Diameter	mm	12	12	12	12	12
		Area	m2	0.00045	0.00045	0.00045	0.00045	0.00045
	Shear reinforcement	Number	bars	2	2	2	2	2
A'c		Diameter	mm	16	16	16	16	16
		Area	m2	0.00040	0.00040	0.00040	0.00040	0.00040
f	Resistance factors for flexure		5.5.4.2	1.00	1.00	1.00	1.00	1.00
fv	Resistance factors for shear			1.00	1.00	1.00	1.00	1.00
fn	Resistance factors for axial force			1.00	1.00	1.00	1.00	1.00
b1	Stress block factor			0.729	0.729	0.729	0.729	0.729
c	Dis. Between centroid and top fiber		m	0.189	0.190	0.191	0.192	0.192
	For T section behavior		m	0.113	0.116	-0.020	-0.020	-0.020
	For rectangular section behavior		m	0.189	0.190	0.191	0.192	0.192
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	1769	1786	1795	1799	1801
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	0.137	0.139	0.139	0.140	0.140
de	Corresp. effective depth from extreme comp. fiber to centroid of tensile force in the tensile reinf.		m	1.107	1.360	1.541	1.649	1.685
Mn	Nominal resistance		kNm	13621	17214	19668	21211	21726
Mr	Factored reistance		kNm	13621	17214	19668	21211	21726
Mu	Flexual moment		kNm	0	3452	5882	7310	7758
(5.7.3.2)	**Flexural moment Checking**			OK	OK	OK	OK	OK
	Limits for reinforcement							
c/de	Maximum reinforcement			0.17	0.14	0.12	0.12	0.11
	Maximum reinforcement Checking		<= 0.42	OK	OK	OK	OK	OK
r min	Mimimum reinforcement			0.08%	0.09%	0.10%	0.10%	0.10%
	Mimimum reinforcement Checking for RC		0.34%	N.a	N.a	N.a	N.a	N.a
1.2*Mcr	Craking moment		kNm	1591	1441	1379	1379	1379
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**			OK	OK	OK	OK	OK
(5.8.3.5)	Tensile force in steel should be satisfied - F_yc		kN	1099	3491	4624	5023	4968
	Checking As.fy+Aps.fps >= F_yc			Ok	Ok	Ok	Ok	Ok

(5.7.3.4)	Conctrol of craking by distr. of reinf for RC member- Check?		Yes	Yes	Yes	Yes	Yes
	Existing condition for structrure	1,2 or 3	1	1	1	1	1
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.058	0.058	0.058	0.058	0.058
Z	Crack width parameter	N/mm	30000	30000	30000	30000	30000
A	Area of concr. with same centroid as tens. Reinf	m2	0.049	0.049	0.049	0.049	0.049
fsa	Value	Mpa	211	211	211	211	211
0.6*fy		Mpa	240	240	240	240	240
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	211	211	211	211	211
x	Dist. From compression fiber to centroid	m	0.077	0.085	0.091	0.094	0.095
J.d	Arm	m	1.082	1.332	1.51	1.618	1.653
Icr	Moment of inertia of the cracked section	m4	0.021	0.021	0.021	0.021	0.021
fs	Tensile stress in reinforcement fs = Msls / (As*J.d)	Mpa	-	2139	3213	3728	3871
	Checking for control cracking fs<fsa		N.a	N.a	N.a	N.a	N.a
(5.10.8.2)	Shrinkage and temperature Reinfocement (side distribution)						
Areq	Area of required reinf	m2	0.00035	0.00028	0.00025	0.00025	0.00025
	Distribution on sides 16 D12	m2	0.00181	0.00181	0.00181	0.00181	0.00181
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45	0.45
	Checking		OK	OK	OK	OK	OK
	6.2 SHEAR FORCE CHECKING						
β	Factor indicating diag. cracked concr. to tension		6.8	6.8	6.8	6.8	6.8
θ	Angle of inclination of diagonal compressive	degree	27.00	27.00	27.00	27.00	27.00
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	90
bv	Effective web width as minimum web width - in dv	m	0.700	0.700	0.250	0.250	0.250
dv	Effective shear depth	m	1.312	1.312	1.471	1.579	1.615
	(de - a/2)	m	1.039	1.291	1.471	1.579	1.615
s	Spacing of stirrups	m	0.150	0.150	0.150	0.150	0.150
ncat	Amount of bars in spacing S	bars	2	2	2	2	2
Av	Shear reinf area in spacing S	m2	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	45.00	45.00	45.00	45.00	45.00
v	Shear stress in concrete	kN/m2	1219	955	1733	1017	419
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1146	1153	1157	1157	1157
e_x	Strain in tensile reinforcement		-4.66E-03	-3.22E-03	-2.50E-03	-2.20E-03	-2.16E-03
	if ex<0, multiple with reduce factor		-3.39E-04	-2.34E-04	-4.51E-04	-3.96E-04	-3.90E-04
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok	Ok
v/f'c	Ratio of shear stress and f'c		0.027	0.021	0.039	0.023	0.009
β	Final value		6.8	6.8	6.8	6.8	6.8
θ	Final value	degree	27.00	27.00	27.00	27.00	27.00
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	3467	3467	1388	1490	1524
Vs	Shear resistance provided by shear reinforcement	kN	2774	2774	3110	3339	3415
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	6242	6242	4499	4830	4940
Vn2	Vn2	kN	10334	10334	4137	4442	4543
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	6242	6242	4137	4442	4543
Vr	Factored shear resistance	kN	6242	6242	4137	4442	4543
Vu	Shear	kN	1120	877	637	401	169
(5.8.2.7)	Shear checking		OK	OK	OK	OK	OK
	Region requiring transverse reinf Checking		No need	No need	No need	No need	No need
	Minimum shear reinf area	m2	0.0001	0.0001	0.0001	0.0001	0.0001
	Mimimum shear reinforcement Checking		-	-	-	-	-
	0.1*f'c*bv*dv	kN	4133	4133	1655	1777	1817
	Smax	m	0.60	0.60	0.60	0.60	0.60
	Maximum spacing Smax		-	-	-	-	-

CALCULATION SHEET

I33 GIRDER

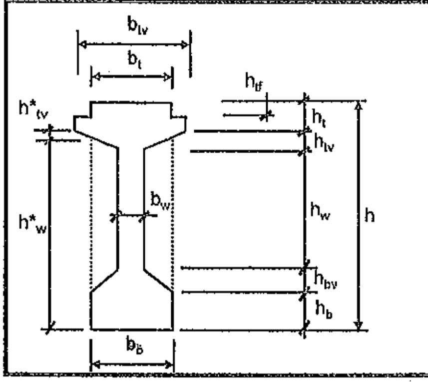
CONTENT

1. INPUT DATA:

1.1. General Data

Specification for bridge design:	TCN 272-05	
Live load	HL93	
Impact (or dynamic) of the live load	IM =	0.25
Pedestrian	PL =	0.00 (kN/m^2)
Length of Girder	L_d =	30.00 (m)
Span between support	L_{tt} =	29.20 (m)
Carriageway width in bridge	w =	11.75 (m)
Parapet width	c =	0.50 (m)
Bridge width	B =	12.74 (m)
Number of girder	N_d =	5.00 (girder)
Space betweem 2 girders	S =	2.55 (m)
Distance from inside of parapet to exterior girder center	d_e =	0.77 (m)
Width of bridge deck	b_{ds} =	12.47 (m)
Length of the overhang (cantilever arm length)	L_h =	1.27 (m)
Thickness of bridge deck	t_s =	0.223 (m)
Precast plank width	b_p =	1.85 (m)
Precast plank thick	h_p =	0.08 (m)
Pavement thick	h_{pa} =	0.084 (m)

1.2. Girder dimension:



Width of over part		b_{tv} =	900.00	(mm)
		b_t =	700.00	(mm)
Width of under part		b_b =	700.00	(mm)
Girder high		h =	1600.00	(mm)
		h_{tf} =	80.00	(mm)
		h_t =	80.00	(mm)
Cross section at end		*at the midle*		
b_w =	700.00		250.00	(mm)
h^*_{tv} =	34.00		110.00	(mm)
h^*_w =	1486.00		960.00	(mm)
h_b =	0.00		250.00	(mm)
h_{bv} =	0.00		200.00	(mm)

1.3. MATERIAL PROPERTIES:

1.3.1 Concrete:

Girder concrete

Girder concrete strength at the 28 age days	f'_c =	45.00 MPa	
Unit weight of Concrete	γ_c =	2400.00 kG/m^3	
Modulus of elasticity	$E_c = 0.043\,\gamma_c^{1.5}\,\mathrm{sqrt}(f'_c)$ =	33914.98 MPa	(5.4.2.4-1)

Deck concrete

Deck concrete strength at the 28 age days	f'_c =	35.00 MPa	
Unit weight of concrete	γ_c =	2400.00 kG/m^3	
Modulus of elasticity	$E_c = 0.043\,\gamma_c^{1.5}\,\mathrm{sqrt}(f'_c)$ =	29910.20 MPa	(5.4.2.4-1)

1.3.2 Prestressing steel

Diameter of one strand	D =	12.70 mm
Area of one strand	$A_s^{12.7}$ =	98.70 mm^2
Ultimate Tendon strength	f_{pu} =	1860.00 MPa
Yield strength of prestressing steel	$f_{py} = 0.9\,f_{pu}$ =	1674.00 MPa
Modulus of strand	E_p =	197000.00 MPa
Wobble friction coefficient (mm-1)	K =	6.60E-07 mm^{-1}
Coeficient of friction (1/RAD)	μ =	0.25
Number of Strands in one Tendon	n =	15.00 Strands
Area of one Tendon	A_s =	1480.50 mm^2
Stress in the prestressing steel at jacking	$f_{pj} = 0.7\,f_{pu}$ =	1302.00 MPa
Jacking force for one tendon	P_j =	1927.61 kN
Anchorage set	ΔL =	6.00 mm
Area of one duck	A_g =	5026.55 mm^2
Number of Tendons	N =	4.00 Tendons

1.3.3 Reinforcing Steel:

Yield strength (deformed bar)	f_{py} =	400.00 (MPa)
Modulus of steel	E_s =	200000.00 (MPa)

3. TENDON PROFILE AND PROPERTY OF GIRDER CROSS SECTION

3.1. Tendon profile:

Tendon profile follow Parabol equation:

$$y_i = f - \frac{4.(f-c).x.(l-x)}{l^2}$$

in which: Origin of coordinates in left edge of the Girder bottom (0.0)

f	Maximum deflection at mid span of tendon
c	Distance from maximum deflection point to girder bottom
(x_i, y_i)	Coordination of point under consider i = 1,2...
L	actual distance between cable ends (X-axis)
$L_p = X_2 - X_1$	Distance between 2 point under consider

angle of rotation of tendon for X_i-axis: $\tan(\alpha) = (4.f\,(1-2.Xi/L))/L$

$\alpha = 2f/0.5L - \tan(\alpha)$

L_{span} =	30000	(mm)
$L_{su.}$ =	29200	(mm)
L_{cap} =	29700	(mm)

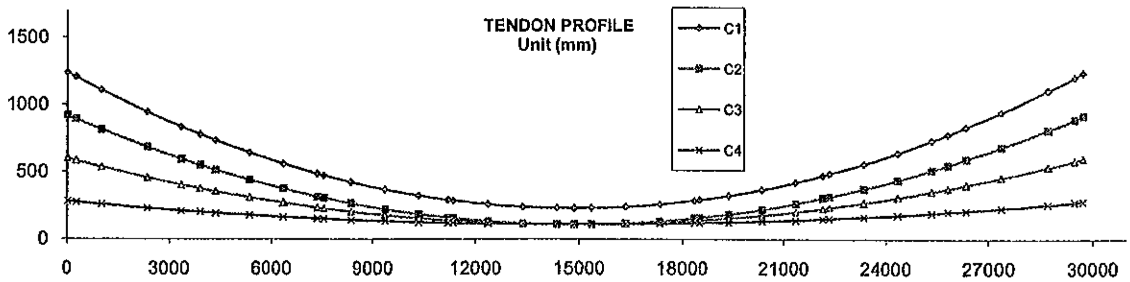
	f =	1240	(mm)	Lcáp =	29700	(mm)	C =	230	(mm)
	Section	Xi	Yi	Lp	ΣLcáp	Tan(αi)	(α)	Σα	Section
		(mm)	(mm)	(mm)	(mm)		(rad)	(rad)	
TENDON No 1	Anchorage	0.00	1240.00	0.00	0.00	0.1670	0.0000	0.0000	Anchorage
	Support	250.00	1206.28	250.00	250.00	0.1642	0.0028	0.0028	Support
	L/8	3900.00	779.16	3650.00	3900.00	0.1231	0.0439	0.0467	L/8
	L/4	7550.00	474.07	3650.00	7550.00	0.0821	0.0849	0.1316	L/4
	3L/8	11200.00	291.02	3650.00	11200.00	0.0410	0.1260	0.2575	3L/8
	L/2	14850.00	230.00	3650.00	14850.00	0.0000	0.1670	0.4245	L/2

	f =	920	(mm)	Lcáp =	29700	(mm)	C =	110	(mm)
	Section	Xi	Yi	Lp	ΣLcáp	Tan(α)	(α)	Σα	Section
		(mm)	(mm)	(mm)	(mm)		(rad)	(rad)	
TENDON No 2	Anchorage	0.00	920.00	0.00	0.00	0.1239	0.0000	0.0000	Anchorage
	Support	250.00	892.96	250.00	250.00	0.1218	0.0021	0.0021	Support
	L/8	3900.00	550.41	3650.00	3900.00	0.0914	0.0325	0.0346	L/8
	L/4	7550.00	305.74	3650.00	7550.00	0.0609	0.0630	0.0976	L/4
	3L/8	11200.00	158.93	3650.00	11200.00	0.0305	0.0935	0.1911	3L/8
	L/2	14850.00	110.00	3650.00	14850.00	0.0000	0.1239	0.3150	L/2

	f =	600	(mm)	Lcáp =	29700	(mm)	C =	110	(mm)
	Section	Xi	Yi	Lp	ΣLcáp	Tan(α)	(α)	Σα	Section
		(mm)	(mm)	(mm)	(mm)		(rad)	(rad)	
TENDON No 3	Anchorage	0.00	600.00	0.00	0.00	0.0808	0.0000	0.0000	Anchorage
	Support	250.00	583.64	250.00	250.00	0.0794	0.0014	0.0014	Support
	L/8	3900.00	376.42	3650.00	3900.00	0.0596	0.0212	0.0226	L/8
	L/4	7550.00	228.41	3650.00	7550.00	0.0397	0.0411	0.0637	L/4
	3L/8	11200.00	139.60	3650.00	11200.00	0.0199	0.0609	0.1246	3L/8
	L/2	14850.00	110.00	3650.00	14850.00	0.0000	0.0808	0.2054	L/2

	f =	280	(mm)	Lcáp =	29700	(mm)	C =	110	(mm)
	Section	Xi	Yi	Lp	ΣLcáp	Tan(α)	(α)	Σα	Section
		(mm)	(mm)	(mm)	(mm)		(rad)	(rad)	
TENDON No 4	Anchorage	0.00	280.00	0.00	0.0	0.0377	0.0000	0.0000	Anchorage
	Support	250.00	274.32	250.00	250.0	0.0371	0.0006	0.0006	Support
	L/8	3900.00	202.43	3650.00	3900.0	0.0278	0.0099	0.0105	L/8
	L/4	7550.00	151.08	3650.00	7550.0	0.0185	0.0192	0.0297	L/4
	3L/8	11200.00	120.27	3650.00	11200.0	0.0093	0.0284	0.0582	3L/8
	L/2	14850.00	110.00	3650.00	14850.0	0.0000	0.0377	0.0959	L/2

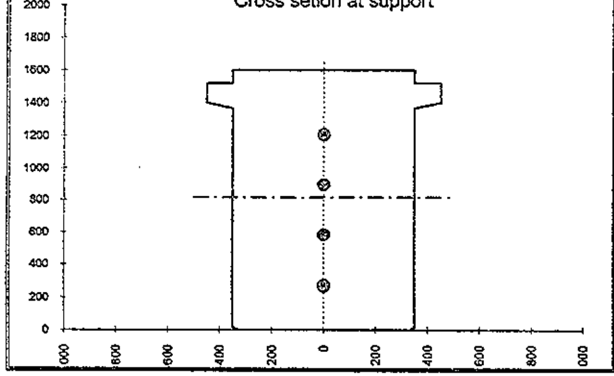
	TENDON No 1		TENDON No 2		TENDON No 3		TENDON No 4	
Section	X_i	Y_i	X_i	Y_i	X_i	Y_i	X_i	Y_i
	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)
Anchorage	0.00	1240.00	0.0	920.00	0.0	600.00	0.00	280.00
Support	**250.00**	**1206.28**	**250.0**	**892.96**	**250.0**	**583.64**	**250.00**	**274.32**
1	1000.00	1108.55	1000.0	814.58	1000.0	536.23	1000.00	257.88
2	2350.00	945.63	2350.0	683.92	2350.0	457.19	2350.00	230.45
3	3350.00	835.71	3350.0	595.77	3350.0	403.86	3350.00	211.95
L/8	**3900.00**	**779.16**	**3900.0**	**550.41**	**3900.0**	**376.42**	**3900.00**	**202.43**
4	4350.00	734.95	4350.0	514.96	4350.0	354.98	4350.00	194.99
5	5350.00	643.35	5350.0	441.50	5350.0	310.54	5350.00	179.57
6	6350.00	560.91	6350.0	375.38	6350.0	270.54	6350.00	165.70
7	7350.00	487.63	7350.0	316.61	7350.0	234.99	7350.00	153.36
L/4	**7550.00**	**474.07**	**7550.0**	**305.74**	**7550.0**	**228.41**	**7550.00**	**151.08**
8	8350.00	423.51	8350.0	265.19	8350.0	203.88	8350.00	142.57
9	9350.00	368.55	9350.0	221.11	9350.0	177.22	9350.00	133.32
10	10350.00	322.75	10350.0	184.38	10350.0	155.00	10350.00	125.61
11	11350.00	286.11	11350.0	155.00	11350.0	137.22	11350.00	119.44
3L/8	**11200.00**	**291.02**	**11200.0**	**158.93**	**11200.0**	**139.60**	**11200.00**	**120.27**
12	12350.00	258.63	12350.0	132.96	12350.0	123.89	12350.00	114.82
13	13350.00	240.31	13350.0	118.26	13350.0	115.00	13350.00	111.73
14	14350.00	231.15	14350.0	110.92	14350.0	110.56	14350.00	110.19
15	15350.00	231.15	15350.0	110.92	15350.0	110.56	15350.00	110.19
L/2	**14850.00**	**230.00**	**14850.0**	**110.00**	**14850.0**	**110.00**	**14850.00**	**110.00**
2	14350.00	231.15	14350.0	110.92	14350.0	110.56	14350.00	110.19
3	15350.00	231.15	15350.0	110.92	15350.0	110.56	15350.00	110.19
4	16350.00	240.31	16350.0	118.26	16350.0	115.00	16350.00	111.73
5	17350.00	258.63	17350.0	132.96	17350.0	123.89	17350.00	114.82
-	**18500.00**	**291.02**	**18500.0**	**158.93**	**18500.0**	**139.60**	**18500.00**	**120.27**
6	18350.00	286.11	18350.0	155.00	18350.0	137.22	18350.00	119.44
7	19350.00	322.75	19350.0	184.38	19350.0	155.00	19350.00	125.61
8	20350.00	368.55	20350.0	221.11	20350.0	177.22	20350.00	133.32
9	21350.00	423.51	21350.0	265.19	21350.0	203.88	21350.00	142.57
-	**22150.00**	**474.07**	**22150.0**	**305.74**	**22150.0**	**228.41**	**22150.00**	**151.08**
10	22350.00	487.63	22350.0	316.61	22350.0	234.99	22350.00	153.36
11	23350.00	560.91	23350.0	375.38	23350.0	270.54	23350.00	165.70
12	24350.00	643.35	24350.0	441.50	24350.0	310.54	24350.00	179.57
13	25350.00	734.95	25350.0	514.96	25350.0	354.98	25350.00	194.99
-	**25800.00**	**779.16**	**25800.0**	**550.41**	**25800.0**	**376.42**	**25800.00**	**202.43**
14	26350.00	835.71	26350.0	595.77	26350.0	403.86	26350.00	211.95
14	27350.00	945.63	27350.0	683.92	27350.0	457.19	27350.00	230.45
16	28700.00	1108.55	28700.0	814.58	28700.0	536.23	28700.00	257.88
Support	29450.00	1206.28	29450.0	892.96	29450.0	583.64	29450.00	274.32
Anchorage	29700.00	1240.00	29700.0	920.00	29700.0	600.00	29700.00	280.00



3.2 Property of Girder Cross section at transfer (Stage I: net cross section):

Âiãøm goïc	x (mm)	y (mm)	A (m^2)	Q_X' (m^3)	Q_Y' (m^3)	I_X' (m^4)	I_Y' (m^4)	$I_X'Y'$ (m^4)
Đường bao								
-	0.00	0.00	-	-	-	-	-	-
1	330.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	350.00	20.00	0.01	0.00	0.00	0.00	0.00	0.00
3	350.00	1366.00	0.47	0.65	0.33	0.89	0.17	0.34
4	450.00	1400.00	-0.12	-0.34	-0.10	-0.72	-0.06	-0.21
5	450.00	1520.00	0.05	0.16	0.05	0.35	0.03	0.11
6	350.00	1520.00	0.15	0.46	0.12	1.05	0.07	0.28
7	350.00	1600.00	0.03	0.09	0.02	0.20	0.01	0.05
8	-350.00	1600.00	1.12	3.58	0.00	8.60	0.14	0.00
9	-350.00	1520.00	0.03	0.09	-0.02	0.20	0.01	-0.05
10	-450.00	1520.00	0.15	0.46	-0.12	1.05	0.07	-0.28
11	-450.00	1400.00	0.05	0.16	-0.05	0.35	0.03	-0.11
12	-350.00	1366.00	-0.12	-0.34	0.10	-0.72	-0.06	0.21
13	-350.00	20.00	0.47	0.65	-0.33	0.89	0.17	-0.34
14	-330.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00
15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
16								
17								
18								
19								
20								
21								
Tổng cộng			1.1470	0.9358	0.0000	1.0135	0.0500	0.0000
Cáp								
1	0.00	1206.28	Diện tích 1 bó cáp		As =	1480.50 (mm2)		
2	0.00	892.96	Số bó cáp		n=	4.00 bó		
3	0.00	583.64			Σ As Yi =	4378136.37 (mm3)		
4	0.00	274.32			Σ As =	5922.00 (mm2)		
5			K. cách từ trọng tâm bó cáp đến TTH =			76.53 (mm)		
6								
7								

Cross setion at support

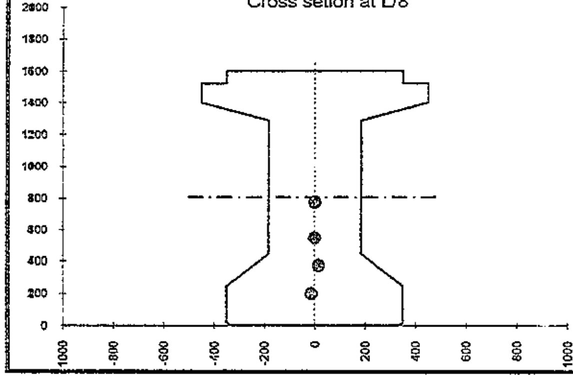


A	1.1470	m^2
e_x	0.0000	m
e_y	0.8158	m
I_x	0.2501	m^4
I_y	0.0500	m^4
I_{xy}	0.0000	m^4
y_b	0.8158	m
y_t	0.7842	m
S_b	0.3065	m^3
S_t	0.3189	m^3
$E_{cáp\,(Y)}$	0.0765	m
q =	27.5280	(KN/m)

ĐẶC TRƯNG HÌNH HỌC - MẶT CẮT L/4

Âiãøm goïc	x (mm)	y (mm)	A (m^2)	Q_X' (m^3)	Q_Y' (m^3)	I_X' (m^4)	I_Y' (m^4)	$I_X'Y'$ (m^4)
Đường bao								
-	0.00	0.00	-	-	-	-	-	-
1	330.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	350.00	20.00	0.01	0.00	0.00	0.00	0.00	0.00
3	350.00	250.00	0.08	0.02	0.06	0.01	0.03	0.01
4	183.50	450.00	0.11	0.08	0.06	0.04	0.02	0.03
5	183.50	1290.00	0.15	0.27	0.06	0.38	0.02	0.07
6	450.00	1400.00	-0.32	-0.87	-0.21	-1.76	-0.10	-0.42
7	450.00	1520.00	0.05	0.16	0.05	0.35	0.03	0.11
8	350.00	1520.00	0.15	0.46	0.12	1.05	0.07	0.28
9	350.00	1600.00	0.03	0.09	0.02	0.20	0.01	0.05
10	-350.00	1600.00	1.12	3.58	0.00	8.60	0.14	0.00
11	-350.00	1520.00	0.03	0.09	-0.02	0.20	0.01	-0.05
12	-450.00	1520.00	0.15	0.46	-0.12	1.05	0.07	-0.28
13	-450.00	1400.00	0.05	0.16	-0.05	0.35	0.03	-0.11
14	-183.50	1290.00	-0.32	-0.87	0.21	-1.76	-0.10	0.42
15	-183.50	450.00	0.15	0.27	-0.06	0.38	0.02	-0.07
16	-350.00	250.00	0.11	0.08	-0.06	0.04	0.02	-0.03
17	-350.00	20.00	0.08	0.02	-0.06	0.01	0.03	-0.01
18	-330.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00
19	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
20								
21								
Tổng cộng			0.8233	0.6656	0.0000	0.7620	0.0257	0.0000
Cáp								
1	0.00	779.16	Diện tích 1 bó cáp		As =	1480.50 (mm2)		
2	0.00	550.41	Số bó cáp		n=	4.00 bó		
3	14.00	376.42			Σ As Yi =	2825424.21 (mm3)		
4	-14.00	202.43			Σ As =	5922.00 (mm2)		
5			K. cách từ trọng tâm bó cáp đến TTH =			331.40 (mm)		
6								
7								

Cross setion at L/8



A	0.8233	m^2
e_x	0.0000	m
e_y	0.8085	m
I_x	0.2238	m^4
I_y	0.0257	m^4
I_{xy}	0.0000	m^4
y_b	0.8085	m
y_t	0.7915	m
S_b	0.2769	m^3
S_t	0.2828	m^3
$E_{cáp\,(Y)}$	0.3314	m
q =	19.7584	(KN/m)

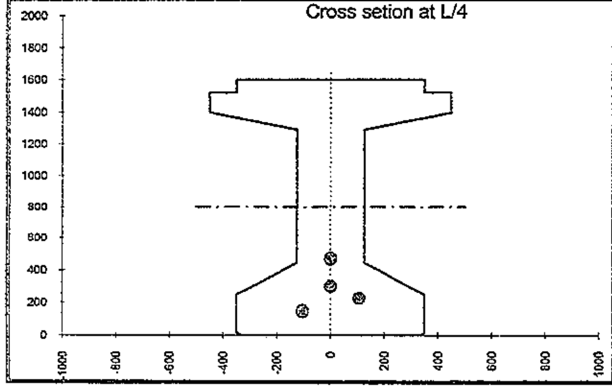
ĐẶC TRƯNG HÌNH HỌC - MẶT CẮT 3L/8

Âiãøm goïc	x (mm)	y (mm)	A (m^2)	Q_X' (m^3)	Q_Y' (m^3)	I_X' (m^4)	I_Y' (m^4)	$I_X'{}_Y'$ (m^4)
Đường bao								
-	0.00	0.00	-	-	-	-	-	-
1	330.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	350.00	20.00	0.01	0.00	0.00	0.00	0.00	0.00
3	350.00	250.00	0.08	0.02	0.06	0.01	0.03	0.01
4	125.00	450.00	0.13	0.09	0.06	0.05	0.02	0.03
5	125.00	1290.00	0.11	0.18	0.03	0.26	0.00	0.03
6	450.00	1400.00	-0.41	-1.09	-0.23	-2.20	-0.11	-0.47
7	450.00	1520.00	0.05	0.16	0.05	0.35	0.03	0.11
8	350.00	1520.00	0.15	0.46	0.12	1.05	0.07	0.28
9	350.00	1600.00	0.03	0.09	0.02	0.20	0.01	0.05
10	-350.00	1600.00	1.12	3.58	0.00	8.60	0.14	0.00
11	-350.00	1520.00	0.03	0.09	-0.02	0.20	0.01	-0.05
12	-450.00	1520.00	0.15	0.46	-0.12	1.05	0.07	-0.28
13	-450.00	1400.00	0.05	0.16	-0.05	0.35	0.03	-0.11
14	-125.00	1290.00	-0.41	-1.09	0.23	-2.20	-0.11	0.47
15	-125.00	450.00	0.11	0.18	-0.03	0.26	0.00	-0.03
16	-350.00	250.00	0.13	0.09	-0.06	0.05	0.02	-0.03
17	-350.00	20.00	0.08	0.02	-0.06	0.01	0.03	-0.01
18	-330.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00
19	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
20								
21								
Tổng cộng			0.7069	0.5671	0.0000	0.6687	0.0223	0.0000

Cáp	x (mm)	y (mm)
1	0.00	474.07
2	0.00	305.74
3	105.00	228.41
4	-105.00	151.08
5		
6		
7		

Diện tích 1 bó cáp	As =	1480.50	(mm2)
Số bó cáp	n=	4.00	bó
	Σ As Yi =	1716344.09	(mm3)
	Σ As =	5922.00	(mm2)
K. cách từ trọng tâm bó cáp đến TTH =		512.45	(mm)

Cross setion at L/4



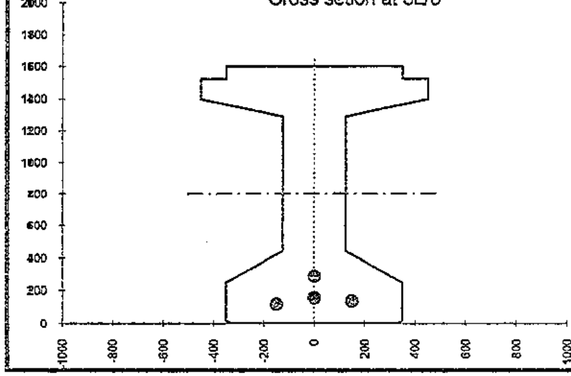
A	0.7069	m^2
e_x	0.0000	m
e_y	0.8023	m
I_x	0.2138	m^4
I_y	0.0223	m^4
I_{xy}	0.0000	m^4
y_b	0.8023	m
y_t	0.7977	m
S_b	0.2665	m^3
S_t	0.2680	m^3
$E_{cáp\,(Y)}$	0.5125	m
q =	16.9644	(KN/m)

Âiãøm goïc	x (mm)	y (mm)	A (m^2)	Q_X' (m^3)	Q_Y' (m^3)	I_X' (m^4)	I_Y' (m^4)	$I_X'{}_Y'$ (m^4)
Đường bao								
-	0.00	0.00	-	-	-	-	-	-
1	330.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	350.00	20.00	0.01	0.00	0.00	0.00	0.00	0.00
3	350.00	250.00	0.08	0.02	0.06	0.01	0.03	0.01
4	125.00	450.00	0.13	0.09	0.06	0.05	0.02	0.03
5	125.00	1290.00	0.11	0.18	0.03	0.26	0.00	0.03
6	450.00	1400.00	-0.41	-1.09	-0.23	-2.20	-0.11	-0.47
7	450.00	1520.00	0.05	0.16	0.05	0.35	0.03	0.11
8	350.00	1520.00	0.15	0.46	0.12	1.05	0.07	0.28
9	350.00	1600.00	0.03	0.09	0.02	0.20	0.01	0.05
10	-350.00	1600.00	1.12	3.58	0.00	8.60	0.14	0.00
11	-350.00	1520.00	0.03	0.09	-0.02	0.20	0.01	-0.05
12	-450.00	1520.00	0.15	0.46	-0.12	1.05	0.07	-0.28
13	-450.00	1400.00	0.05	0.16	-0.05	0.35	0.03	-0.11
14	-125.00	1290.00	-0.41	-1.09	0.23	-2.20	-0.11	0.47
15	-125.00	450.00	0.11	0.18	-0.03	0.26	0.00	-0.03
16	-350.00	250.00	0.13	0.09	-0.06	0.05	0.02	-0.03
17	-350.00	20.00	0.08	0.02	-0.06	0.01	0.03	-0.01
18	-330.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00
19	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
20								
21								
Tổng cộng			0.7069	0.5671	0.0000	0.6687	0.0223	0.0000

Cáp	x (mm)	y (mm)
1	0.00	291.02
2	0.00	158.93
3	150.00	139.60
4	-150.00	120.27
5		
6		
7		

Diện tích 1 bó cáp	As =	1480.50	(mm2)
Số bó cáp	n=	4.00	bó
	Σ As Yi =	1050896.02	(mm3)
	Σ As =	5922.00	(mm2)
K. cách từ trọng tâm bó cáp đến TTH =		624.82	(mm)

Cross setion at 3L/8

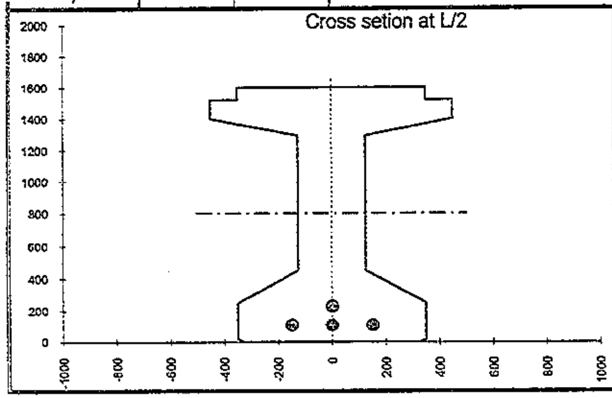


A	0.7069	m^2
e_x	0.0000	m
e_y	0.8023	m
I_x	0.2138	m^4
I_y	0.0223	m^4
I_{xy}	0.0000	m^4
y_b	0.8023	m
y_t	0.7977	m
S_b	0.2665	m^3
S_t	0.2680	m^3
$E_{cáp\,(Y)}$	0.6248	m
q =	16.9644	(KN/m)

ĐẶC TRƯNG HÌNH HỌC - MẶT CẮT L/2

Âiãøm goïc	x (mm)	y (mm)	A (m^2)	Q_X' (m^3)	Q_Y' (m^3)	I_X' (m^4)	I_Y' (m^4)	$I_X'Y'$ (m^4)
Đường bao								
-	0.00	0.00	-	-	-	-	-	-
1	330.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	350.00	20.00	0.01	0.00	0.00	0.00	0.00	0.00
3	350.00	250.00	0.08	0.02	0.06	0.01	0.03	0.01
4	125.00	450.00	0.13	0.09	0.06	0.05	0.02	0.03
5	125.00	1290.00	0.11	0.18	0.03	0.26	0.00	0.03
6	450.00	1400.00	-0.41	-1.09	-0.23	-2.20	-0.11	-0.47
7	450.00	1520.00	0.05	0.16	0.05	0.35	0.03	0.11
8	350.00	1520.00	0.15	0.46	0.12	1.05	0.07	0.28
9	350.00	1600.00	0.03	0.09	0.02	0.20	0.01	0.05
10	-350.00	1600.00	1.12	3.58	0.00	8.60	0.14	0.00
11	-350.00	1520.00	0.03	0.09	-0.02	0.20	0.01	-0.05
12	-450.00	1520.00	0.15	0.46	-0.12	1.05	0.07	-0.28
13	-450.00	1400.00	0.05	0.16	-0.05	0.35	0.03	-0.11
14	-125.00	1290.00	-0.41	-1.09	0.23	-2.20	-0.11	0.47
15	-125.00	450.00	0.11	0.18	-0.03	0.26	0.00	-0.03
16	-350.00	250.00	0.13	0.09	-0.06	0.05	0.02	-0.03
17	-350.00	20.00	0.08	0.02	-0.06	0.01	0.03	-0.01
18	-330.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00
19	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
20								
21								
Tổng cộng			0.7069	0.5671	0.0000	0.6687	0.0223	0.0000
Cáp								
1	0.00	230.00						
2	0.00	110.00						
3	150.00	110.00						
4	-150.00	110.00						
5								
6								
7								

Diện tích 1 bó cáp	As =	1480.50	(mm2)
Số bó cáp	n=	4.00	bó
	Σ As Yi =	829080.00	(mm3)
	Σ As =	5922.00	(mm2)
K. cách từ trọng tâm bó cáp đến TTH =		662.28	(mm)



A	0.7069	m^2
e_x	0.0000	m
e_y	0.8023	m
I_x	0.2138	m^4
I_y	0.0223	m^4
I_{xy}	0.0000	m^4
y_b	0.8023	m
y_t	0.7977	m
S_b	0.2665	m^3
S_t	0.2680	m^3
$E_{cap\,(Y)}$	0.6623	m
q =	16.9644	(KN/m)

Uniform load due to sefl weigh of Girder in Stage 1: Q = 19.97 (KN/m)

3.3. Property of Girder cross section in service stage (stage II: Composite cross section) :

3.3.1. Effective flange width (4.6.2.6)

Modular Ratio: Deck Concrete/Girder Concrete n = Eb / Ed = 0.88

For Interior Girder:

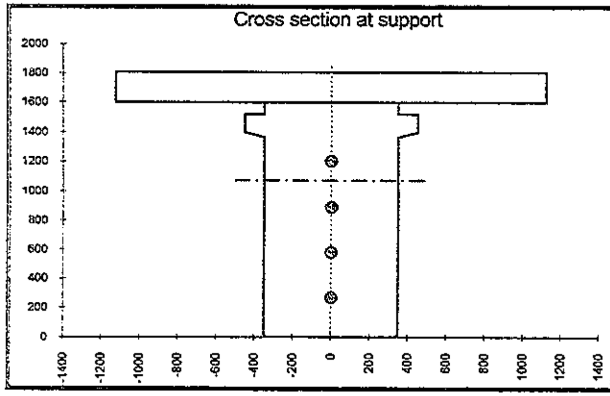
$$b_I = \min\begin{cases}1/4L_{tt}\\12h_f + \max(0.5b, b_w)\\S\end{cases} \quad \Rightarrow n^* b_I = 2248.88861 \text{ (mm)}$$

For Exterior Girder:

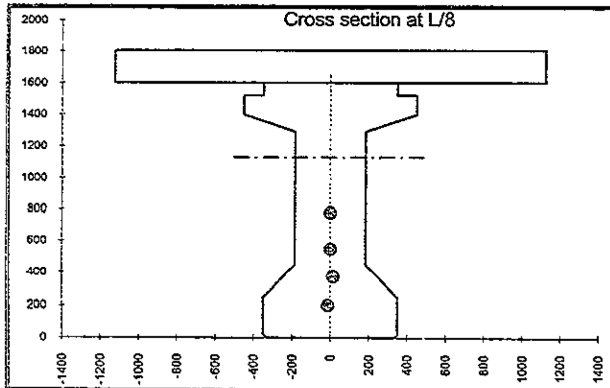
$$b_E = 0.5b_I + \min\begin{cases}1/8L_{tt}\\6h_f + \max(0.5b_w, 0.25b)\\L_h\end{cases} \quad \Rightarrow n^* b_E = 2244.47903 \text{ (mm)}$$

3.3.2. Property of Girder cross section in stage II (service stage):

Interior Girder:

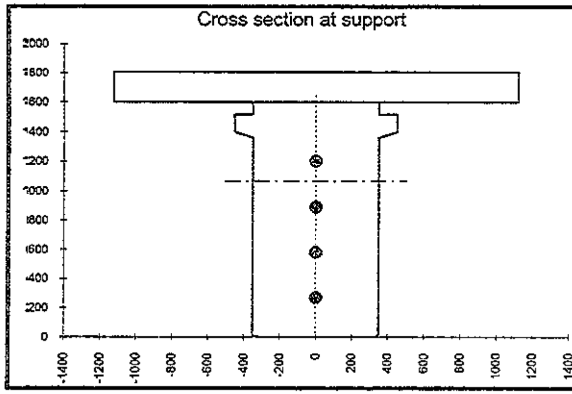


A	1.6125	m^2
e_x	0.0000	m
e_y	1.0721	m
I_x	0.5126	m^4
I_y	0.2462	m^4
I_{xy}	0.0000	m^4
y_b	1.0721	m
y_t	0.7349	m
S_b	0.4782	m^3
S_t	0.6975	m^3
$E_{cáp\ (Y)}$	0.3328	m
q =	38.7005	(KN/m)

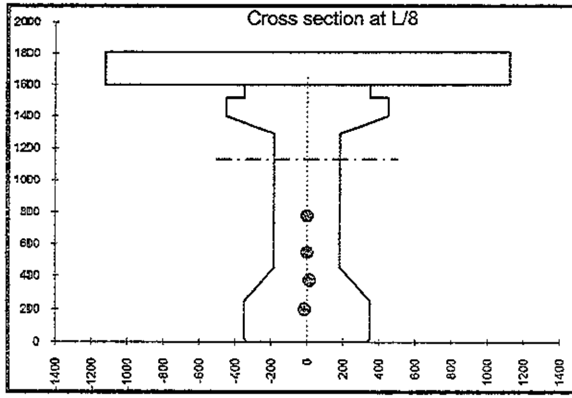


A	1.2888	m^2
e_x	0.0000	m
e_y	1.1318	m
I_x	0.4637	m^4
I_y	0.2219	m^4
I_{xy}	0.0000	m^4
y_b	1.1318	m
y_t	0.6752	m
S_b	0.4097	m^3
S_t	0.6867	m^3
$E_{cáp\ (Y)}$	0.6547	m
q =	30.9308	(KN/m)

Exterior Girder:

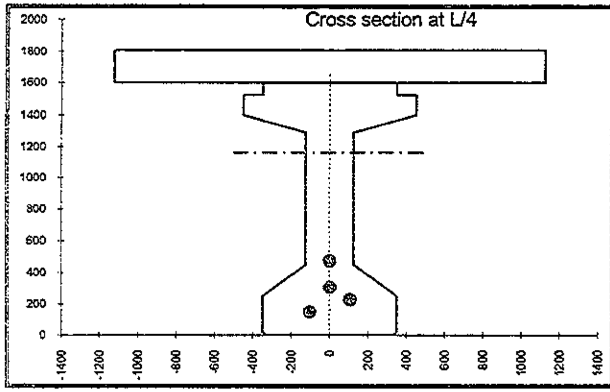


A	1.6116	m^2
e_x	0.0000	m
e_y	1.0717	m
I_x	0.5123	m^4
I_y	0.2451	m^4
I_{xy}	0.0000	m^4
y_b	1.0717	m
y_t	0.7353	m
S_b	0.4780	m^3
S_t	0.6967	m^3
$E_{cáp\ (Y)}$	0.3324	m
q =	39.4844	(KN/m)

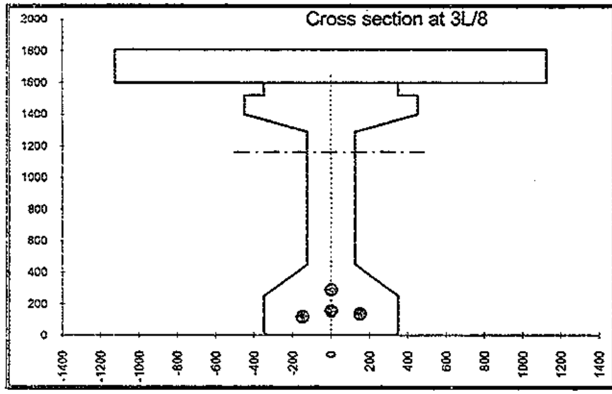


A	1.2879	m^2
e_x	0.0000	m
e_y	1.1314	m
I_x	0.4634	m^4
I_y	0.2207	m^4
I_{xy}	0.0000	m^4
y_b	1.1314	m
y_t	0.6756	m
S_b	0.4096	m^3
S_t	0.6859	m^3
$E_{cáp\ (Y)}$	0.6543	m
q =	31.5529	(KN/m)

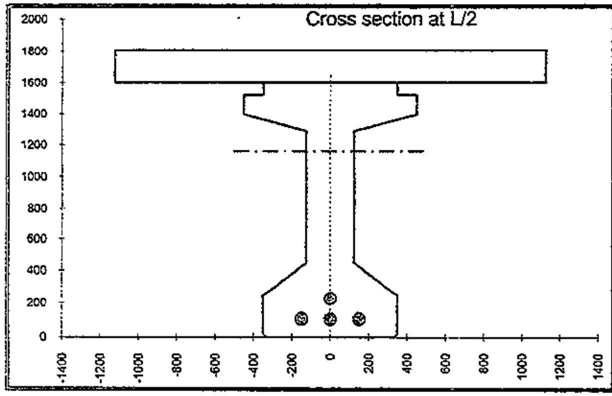
Interior Girder:



A	1.1724	m^2
e_x	0.0000	m
e_y	1.1601	m
I_x	0.4434	m^4
I_y	0.2185	m^4
I_{xy}	0.0000	m^4
y_b	1.1601	m
y_t	0.6469	m
S_b	0.3822	m^3
S_t	0.6855	m^3
$E_{cáp\ (Y)}$	0.8703	m
q =	28.1369	(KN/m)

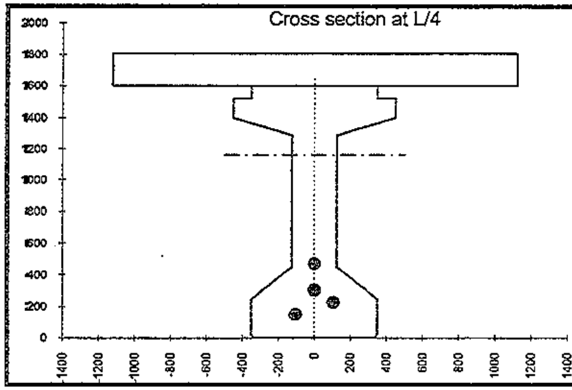


A	1.1724	m^2
e_x	0.0000	m
e_y	1.1601	m
I_x	0.4434	m^4
I_y	0.2185	m^4
I_{xy}	0.0000	m^4
y_b	1.1601	m
y_t	0.6469	m
S_b	0.3822	m^3
S_t	0.6855	m^3
$E_{cáp\ (Y)}$	0.9827	m
q =	28.1369	(KN/m)

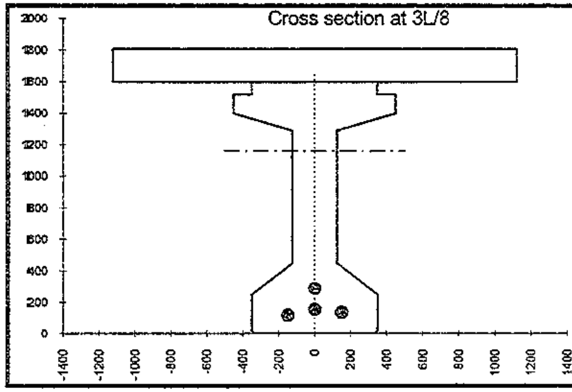


A	1.1724	m^2
e_x	0.0000	m
e_y	1.1601	m
I_x	0.4434	m^4
I_y	0.2185	m^4
I_{xy}	0.0000	m^4
y_b	1.1601	m
y_t	0.6469	m
S_b	0.3822	m^3
S_t	0.6855	m^3
$E_{cáp\ (Y)}$	1.0201	m
q =	28.1369	(KN/m)

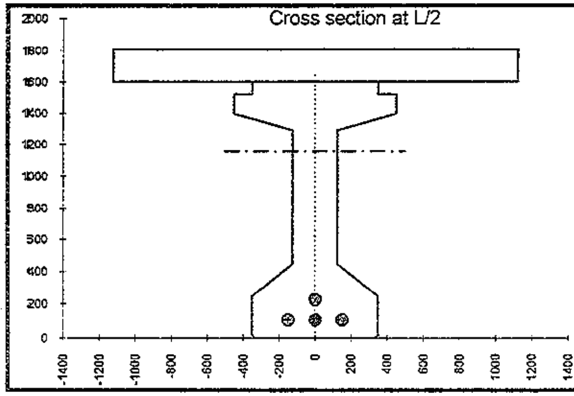
Exterior Girder:



A	1.1715	m^2
e_x	0.0000	m
e_y	1.1597	m
I_x	0.4431	m^4
I_y	0.2173	m^4
I_{xy}	0.0000	m^4
y_b	1.1597	m
y_t	0.6473	m
S_b	0.3821	m^3
S_t	0.6846	m^3
$E_{cáp\ (Y)}$	0.8699	m
q =	28.7007	(KN/m)



A	1.1715	m^2
e_x	0.0000	m
e_y	1.1597	m
I_x	0.4431	m^4
I_y	0.2173	m^4
I_{xy}	0.0000	m^4
y_b	1.1597	m
y_t	0.6473	m
S_b	0.3821	m^3
S_t	0.6846	m^3
$E_{cáp\ (Y)}$	0.9823	m
q =	28.7007	(KN/m)



A	1.1715	m^2
e_x	0.0000	m
e_y	1.1597	m
I_x	0.4431	m^4
I_y	0.2173	m^4
I_{xy}	0.0000	m^4
y_b	1.1597	m
y_t	0.6473	m
S_b	0.3821	m^3
S_t	0.6846	m^3
$E_{cáp\ (Y)}$	1.0197	m
q =	28.7007	(KN/m)

2. INTERNAL FORCE:
2.1. Dead Load:
2.1.1 Load:

Interior Beam:

Bridge deck	DC_d =	13.32 (kN/m)
Precat plank & cross beam	DC_{pl} =	4.71 (kN/m)
Parapet	DC_{pa} =	4.74 (kN/m)
Pavement	DW_p =	4.44 (kN/m)

Exterior Beam:

Bridge deck	DC_d =	13.32 (kN/m)
Precat plank & cross beam	DC_{pl} =	2.36 (kN/m)
Parapet	DC_{pa} =	4.80 (kN/m)
Pavement	DW_p =	4.44 (kN/m)

2.1.2 Internal Force due to dead load:

Formula : $M = 0.5\, q.X_i\,(L-X_i)$

$Q = q.(0.5.L-X_i)$

L_{tt} = 29.20 (m)

INTERIOR GIRDER												
		Girder (DC)		Concrete Deck (DC)		Plank & cr.beam (DC)		Parapet (DC)		Pavement (DW)	
Section	X_i	M	Q	M	Q	M	Q	M	Q	M	Q
	(m)	(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)
Support	0.00	0.00	291.53	0.00	194.44	0.00	68.78	0.00	69.20	0.00	64.85
L/8	3.65	931.08	218.65	621.00	145.83	219.65	51.58	221.02	51.90	207.10	48.63
L/4	7.30	1596.14	145.77	1064.57	97.22	376.55	34.39	378.89	34.60	355.03	32.42
3L/8	10.95	1995.17	72.88	1330.71	48.61	470.68	17.19	473.61	17.30	443.79	16.21
L/2	14.60	2128.18	0.00	1419.43	0.00	502.06	0.00	505.19	0.00	473.38	0.00
EXTERIOR GIRDER											
Gối	0.00	0.00	291.53	0.00	194.44	0.00	68.78	0.00	70.01	0.00	64.85
L/8	3.65	931.08	218.65	621.00	145.83	109.83	51.58	223.58	52.51	207.10	48.63
L/4	7.30	1596.14	145.77	1064.57	97.22	188.27	34.39	383.29	35.00	355.03	32.42
3L/8	10.95	1995.17	72.88	1330.71	48.61	235.34	17.19	479.11	17.50	443.79	16.21
L/2	14.60	2128.18	0.00	1419.43	0.00	251.03	0.00	511.05	0.00	473.38	0.00

2.2 Live Load:
2.2.1. Distribution factors for Live load:

Modular Ratio: Girder Concrete/Deck Concrete	$n = E_g / E_d$ =	1.13
Distance from girder centroid to bridge deck centroid	$e^I g$ =	908.97 (mm)
	$e^E g$ =	908.97 (mm)
Longgitudinal stiffness parameter	$K^I g = n(Ig + A\, e^2_g)$ =	1.6E+12
	$K^E g = n(Ig + A\, e^2_g)$ =	1.6E+12
Ration	$K^I g / (L\, t^3_s)$ =	4.98
	$K^E g / (L\, t^3_s)$ =	4.97
	S / L =	0.09

(a) Distribution Factor for Moment: g(M)

Interior Beam:

For one lane $$0.06+\left(\frac{S}{4300}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{Lt_s^3}\right)^{0.1} = \mathbf{0.518}$$

Two or more lanes $$0.075+\left(\frac{S}{2900}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{Lt_s^3}\right)^{0.1} = \mathbf{0.742}$$

Exterior Beam:

For one lane, follow the lever rule

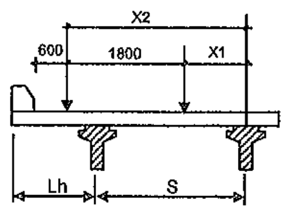
X_1 = 920.000

X_2 = 2720.00

Y_1 = 0.361

Y_2 = 1.067

=> $g(M) = 0.5^*\Sigma yi = \boxed{0.714}$



Two or more lanes $$e = 0.77 + \frac{d_e}{2800} = \quad 1.045 < (=) 1$$

Choice e = 1.045 IF(e>1,1,e)

=> $g(M) = e^* g_{trong} = \boxed{0.776}$

(b) Distribution Factor for Shear force: g(Q)

Interior Beam:

For one lane $$0.36 + \frac{S}{7600} = \mathbf{0.696}$$

Two or more lanes $$0.2 + \frac{S}{3600} - \left(\frac{S}{10700}\right)^2 = \mathbf{0.852}$$

Exterior Beam:

For one lane, follow the lever rule

$g(Q) = 0.5 * \Sigma yi = \boxed{0.714}$

Two or more lanes

$$e = 0.6 + \frac{de}{3000} = \quad 0.857$$

$=> g(Q) = e * g_{trong} = \boxed{0.729}$

(c) Correction factor for skew bridge:

* Correction factor of distribution factor for moment (Table 4.6.2.2.2d-1)

Skew angle	θ =	0	Degree.	
Factor	c1 =	0.000		
Correction factor	CF(M) =	1.000		

Area of applications
300<= θ <= 600
1100<= S <= 4900
6000 <= L <= 73000
Nb >= 4

$$CF(M) = 1.0 - c1.(\tan\theta)^{1.5}$$
$$c1 = 0.25\left(\frac{Kg}{Lts^3}\right)^{0.25}\left(\frac{S}{L}\right)^{0.5}$$

* Regulation factor of distribution factor for shear force (Table 4.6.2.2.3c-1)

Correction Factor CF(Q)= 1.000

Area of applications
00<= θ <= 600
1100<= S <= 4900
6000 <= L <= 73000
Nb >= 4

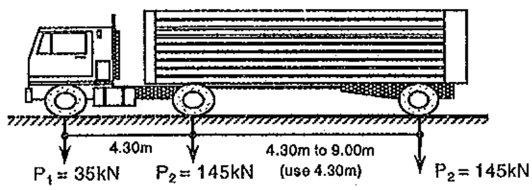
$$CF(Q) = 1.0 + 0.20\left(\frac{Lts^3}{Kg}\right)^{0.3}.\tan\theta$$

(d) Table of Distribution factors for Live load:

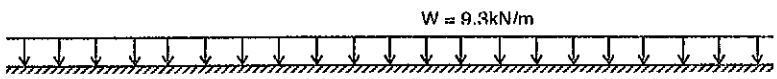
Interior Beam	g(M)	g(Q)	m	m*g(M)	m*g(Q)	m*g(M)*CF(M)	m*g(Q)*CF(Q)
1 lane	0.518	0.696	1.20	0.622	0.835	0.622	0.835
2 or more lanes	0.742	0.852	1.00	0.742	0.852	**0.742**	**0.852**
Exterior Beam							
1 lane	0.714	0.714	1.20	0.856	0.856	**0.856**	**0.856**
2 or more lanes	0.776	0.729	1.00	0.776	0.729	0.776	0.729

2.2.2 Live Load:

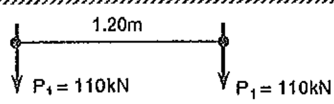
Design Truck



Design Lane Load



Design Tendem



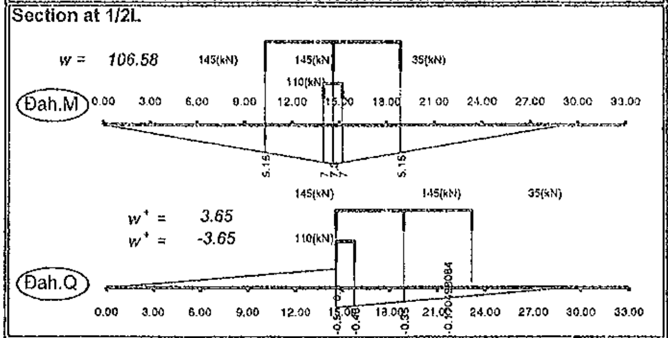
- Truck	P1 =	35.00 (kN)
	P2 =	145.00 (kN)
- Lane load	W =	9.30 (kN)
- Tandem	P1 =	110.00 (kN)
- Pedestrian	PL =	0.00 kN/m^2
- Dynammic load	IM =	0.25

2.2.3 Internal Force due to Live load:

Design truck or Tendem	Momen	$M_{TR(Ta)} = \Sigma Piyi$	(kNm)
	Shear force	$Q_{TR(Ta)} = \Sigma Piyi$	(kN)
Lane load	Momen	$M_{Ln} = W.F_M$	(kNm)
	Shear force	$Q_{Ln} = W.F_Q$	(kN)
Pedestrian	Momen	$M_{PL} = PL.F_M$	(kNm)
	Shear force	$Q_{PL} = PL.F_Q$	(kN)

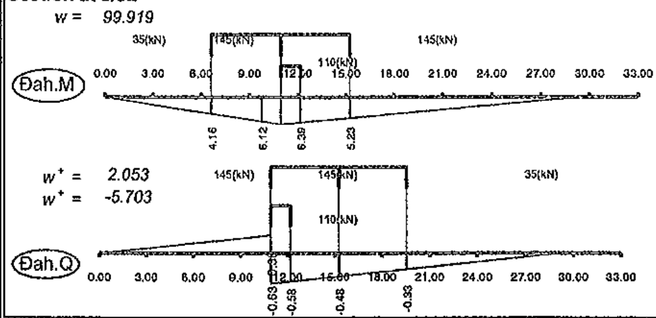
Influence line for Momen & Shear force	Load	Momen (KN.m)	Shear

Section at 1/2L



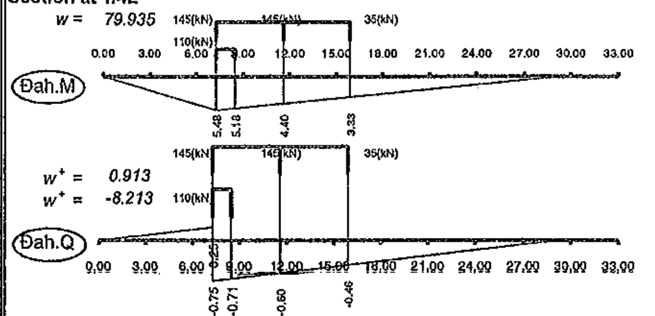
Load	Momen (KN.m)	Shear
Truck	1985.50	130.84
Lane	991.19	33.95
Tandem	1540.00	105.48
Design	**1985.50**	**130.84**
Pedestrian	0.00	0.00

Section at 3/8L



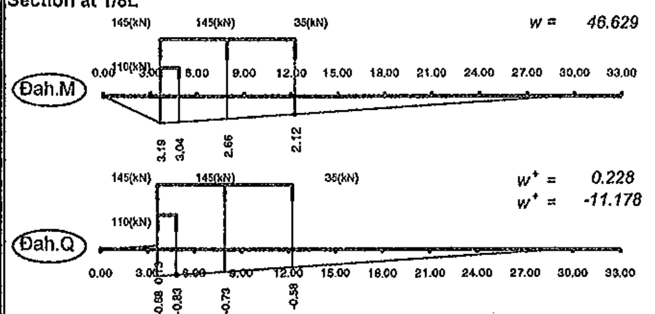
Load	Momen (KN.m)	Shear
Truck	1896.34	171.46
Lane	929.24	53.04
Tandem	1456.13	132.98
Design	**1896.34**	**171.46**
Pedestrian	0.00	0.00

Section at 1/4L



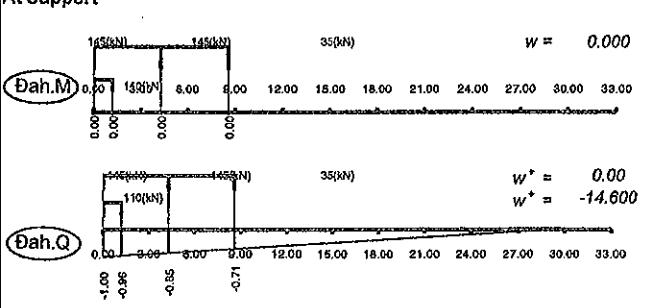
Load	Momen (KN.m)	Shear
Truck	1548.25	212.09
Lane	743.40	76.38
Tandem	1171.50	160.48
Design	**1548.25**	**212.09**
Pedestrian	0.00	0.00

Section at 1/8L



Load	Momen (KN.m)	Shear
Truck	922.41	252.71
Lan	433.65	103.96
Tandem	686.13	187.98
Design	**922.41**	**252.71**
Pedestrian	0.00	0.00

At support



Load	Momen (KN.m)	Shear
Truck	0.00	293.34
Lane	0.00	135.78
Tandem	0.00	215.48
Design	**0.00**	**293.34**
Pedestrian	0.00	0.00

Internal Force due to Live load : $M_{(LL+IM)} = m^{*}g(M)^{*}[\max(M_{TR}, M_{Ta})^{*}(1+IM) + M_{Ln}]$

$Q_{(LL+IM)} = m^{*}g(Q)^{*}[\max(Q_{TR}, Q_{Ta})^{*}(1+IM) + Q_{Ln}]$

Internal Force due to pedestrian :

$M = g(M)^{*} M_{PL}$

$Q = g(Q)^{*} Q_{PL}$

In which:

$M_{TR(Ta)}$ moment due to truck or Tandem

$Q_{TR(Ta)}$ Shear force due to truck or Tandem

yi Value of influence line F Area of influence line

m Lane factor g Distribution factor

Interior	m*g(M)	m*g(Q)
	0.742	0.852
Exterior		
	0.856	0.856

TABLE OF INTERNAL FORCE DUE TO LIVE LOAD

Setion	Xi	Interior Girder		Exterior Girder	
		M	Q	M	Q
	(m)	(kNm)	(kN)	(kNm)	(kN)
Support	0.00	0.00	427.86	0.00	430.34
L/8	3.65	1177.94	357.52	1358.92	359.59
L/4	7.30	1988.69	290.79	2294.23	292.47
3L/8	10.95	2449.70	227.67	2826.07	228.99
L/2	14.60	2578.43	168.17	2974.58	169.15

2.3 Load combination:

Strength limit state: $U = \eta\,[1.25\,DC + 1.50\,DW + 1.75\,(LL+IM)]$

Service limit state: $U = \eta\,[1.00\,DC + 1.00\,DW + 1.00\,(LL+IM)]$

Fatigue state: $U = 0.75\,(LL+IM)]$

The modify load factort $\eta = \eta_D\,\eta_R\,\eta_I$

STATE	Modify Load Factor			
	η_D	η_R	η_I	$\eta = \eta_D\,\eta_R\,\eta_I$
Strength	1.00	1.00	1.00	1.00
Service	1.00	1.00	1.00	1.00

2.3.1 Load combination - - Interior Girder:

STATE	Strength										
Load	Load Factor	Section									
		Support		L/8		L/4		3L/8		L/2	
	γ	M	Q	M	Q	M	Q	M	Q	M	Q
		(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)
DC	1.25	0.00	779.94	2490.94	584.96	4270.18	389.97	5337.73	194.99	5693.58	0.00
DW	1.50	0.00	97.27	310.65	72.95	532.55	48.63	665.68	24.32	710.06	0.00
LL+IM	1.75	0.00	748.75	2061.40	625.66	3480.21	508.88	4286.97	398.43	4512.25	294.30
Total		**0.00**	**1625.96**	**4862.99**	**1283.56**	**8282.94**	**947.49**	**10290.39**	**617.73**	**10915.89**	**294.30**

STATE	Service										
Load	Load Factor	Section									
		Support		L/8		L/4		3L/8		L/2	
	γ	M	Q	M	Q	M	Q	M	Q	M	Q
		(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)
DC	1.00	0.00	623.95	1992.75	467.97	3416.15	311.98	4270.18	155.99	4554.86	0.00
DW	1.00	0.00	64.85	207.10	48.63	355.03	32.42	443.79	16.21	473.38	0.00
LL+IM	1.00	0.00	427.86	1177.94	357.52	1988.69	290.79	2449.70	227.67	2578.43	168.17
Total		**0.00**	**1116.66**	**3377.80**	**874.12**	**5759.87**	**635.19**	**7163.67**	**399.87**	**7606.66**	**168.17**

2.3.2 Load combination - Exterior Girder:

STATE	Strength										
Load	Load factor	Section									
		Supprt		L/8		L/4		3L/8		L/2	
	γ	M	Q	M	Q	M	Q	M	Q	M	Q
		(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)
DC	1.25	0.00	780.95	2356.86	585.71	4040.34	390.47	5050.42	195.24	5387.11	0.00
DW	1.50	0.00	97.27	310.65	72.95	532.55	48.63	665.68	24.32	710.06	0.00
LL+IM	1.75	0.00	753.09	2378.12	629.28	4014.91	511.83	4945.63	400.74	5205.52	296.01
Total		**0.00**	**1631.30**	**5045.63**	**1287.94**	**8587.79**	**950.94**	**10661.73**	**620.29**	**11302.69**	**296.01**

STATE	Service										
Load	load factor	Section									
		Support		L/8		L/4		3L/8		L/2	
	γ	M	Q	M	Q	M	Q	M	Q	M	Q
		(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)
DC	1.00	0.00	624.76	1885.49	468.57	3232.27	312.38	4040.34	156.19	4309.69	0.00
DW	1.00	0.00	64.85	207.10	48.63	355.03	32.42	443.79	16.21	473.38	0.00
LL+IM	1.00	0.00	430.34	1358.92	359.59	2294.23	292.47	2826.07	228.99	2974.58	169.15
Total		**0.00**	**1119.94**	**3451.51**	**876.79**	**5881.53**	**637.28**	**7310.20**	**401.39**	**7757.65**	**169.15**

4. LOSS OF PRESTRESS

4.1 Loss of prestressing force immediately (Instantaneous losses):

4.1.1 Friction between Prestressing Tendon and Duck:

Formula: $\Delta f_{pF} = f_{pj}\,(1-e^{-(Kx+\mu\alpha)})$ (5.9.5.2.2)

Xi: Length of tendon from the jacking end to any point under consideration

Section		Tendon no. 1		Tendon no. 2		Tendon no. 3		Tendon no. 4	
	X_i	$\Sigma\alpha$	Δf_{pF}	$\Sigma\alpha$	Δf_{pF}	$\Sigma\alpha$	Δf_{pF}	$\Sigma\alpha$	Δf_{pF}
	(mm)	(rad)	(MPa)	(rad)	(MPa)	(rad)	(MPa)	(rad)	(MPa)
Ancho.	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00	0.0000	0.00
Support	250.00	0.0028	1.13	0.0021	0.89	0.0014	0.66	0.0006	0.42
L/8	3900.00	0.0467	18.41	0.0346	14.54	0.0226	10.66	0.0105	6.76
L/4	7550.00	0.1316	48.39	0.0976	37.71	0.0637	26.93	0.0297	16.06
3L/8	11200.00	0.2575	90.18	0.1911	69.87	0.1246	49.23	0.0582	28.24
L/2	14850.00	0.4245	142.53	0.3150	110.33	0.2054	77.24	0.0959	43.23

4.1.2 Anchorage seating or Set:

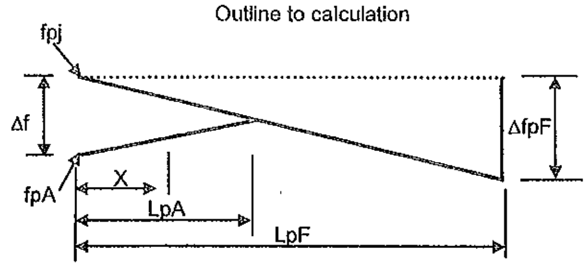
Formula

$$L_{pA} = \sqrt{\frac{E(\Delta L)L_{pF}}{\Delta f_{pF}}}$$

$$\Delta f = \frac{2\Delta f_{pF}L_{pA}}{L_{pF}}$$

$$\Delta f_{pA} = \Delta f\left(1-\frac{X}{L_{pA}}\right)$$

Outline to calculation



Trong đó:

- L_{pA} Effective length due to anchorage set
- E Cable modulus of elasticity
- ΔL Setting length
- L_{pF} The length from anchorage to point that loss stress due to friction was known
- Δf_{pF} The loss stress value at the point that the leng from anchorage ti it is L_{pF}
- Δf The loss stress value at Anchorage

Choice the length from anchorage to point that loss stress due to friction was known (LpF) and calculation follow:

Tendon no.1		Xi (mm)	Δf_{pA} (MPa)
LpF =	14850	0	213.02
Δf_{pF} =	142.53	250	208.22
LpA =	11097.5	3900	138.16
Δf =	213.02	7550	68.10
		11200	0.00
		14850	0.00

Tendon no.2		Xi (mm)	Δf_{pA} (MPa)
LpF =	14850	0	187.42
Δf_{pF} =	110.33	250	183.71
LpA =	12613.2	3900	129.47
Δf =	187.42	7550	75.24
		11200	21.00
		14850	0.00

Tendon no.3		Xi (mm)	Δf_{pA} (MPa)
LpF =	14850	0	154.48
Δf_{pF} =	77.24	250	151.88
LpA =	14850.0	3900	113.91
Δf =	154.48	7550	75.94
		11200	37.97
		14850	0.00

Tendon no.4		Xi (mm)	Δf_{pA} (MPa)
LpF =	14850	0	86.46
Δf_{pF} =	43.23	250	85.01
LpA =	14850.0	3900	63.75
Δf =	86.46	7550	42.50
		11200	21.25
		14850	0.00

4.1.3 Elastic deformation of concrete:

Formula

$$\Delta f_{ES} = \frac{N-1}{2N}\frac{E_p}{E_{ci}} f_{cgp} \quad (5.9.5.2.3b\text{-}1)$$

In which:

Number of tendon	N =	4.00 (Tendon)
Cable modulus of elasticity	Ep =	197000.0 MPa
Concrete strength at transfer	f'_{ci} =	40.50 MPa
Unit weight of concrete	γ_c =	2450.00 kg/m3
Concrete modulus of elasticity at transfer	E_{ci} =	33185.3 MPa

Total stress of concrete in the Tendon centroid (f_{cgp}) due to prestressing force and self weigh of girder

$$f_{cgp} = \frac{F_j}{A} + \frac{F_j e^2}{I_X} - \frac{M_{DC} e}{I_X}$$

Compression force due to prestressing consider loss stress: $F_j = N^* f_{pj}{}^* A_s - A_s{}^* \Sigma(\Delta f_{pFi} + \Delta f_{pAi})$

A Area of girder cross section

Ix Inertia Moment of Girder cross section

e Distance from tendon centroid to neutral line of girder section

M_{DC} Maximum moment due to self weigh of girder at jacking

Total loss stress due to friction and Anchorage:

Section	Xi	Tendon1	Tendon2	Tendon3	Tendon4	SUM	ΣFj
		ΔfpF+Δf$_{pA}$	ΔfpF+Δf$_{pA}$	ΔfpF+Δf$_{pA}$	ΔfpF+Δf$_{pA}$	Σ(ΔfpF+Δf$_{pA}$)	
	(mm)	(MPa)	(MPa)	(MPa)	(MPa)	(MPa)	(kN)
Anchorage	0	213.02	187.42	154.48	86.46	641.38	6760.88
Support	250	209.35	184.60	152.54	85.43	631.92	6774.89
L/8	3900	156.57	144.01	124.57	70.52	495.67	6976.61
L/4	7550	116.49	112.94	102.87	58.56	390.86	7131.77
3L/8	11200	90.18	90.87	87.20	49.49	317.74	7240.02
L/2	14850	142.53	110.33	77.24	43.23	373.33	7157.73

Loss stress due to Elastic deformation of concrete

Section	Xi	Fj	A	Ix	e	M_{DC}	f_{cgp}	Δf_{ES}
	(mm)	(kN)	(mm2)	(mm4)	(mm)	kNm	(MPa)	**(MPa)**
Anchorage	0	6760.88	1.1E+06	2.5E+11	76.53	0.00	6.05	**13.47**
Support	250	6774.89	1.1E+06	2.5E+11	76.53	0.00	6.07	**13.50**
L/8	3900	6976.61	8.2E+05	2.2E+11	331.40	931.08	10.52	**23.42**
L/4	7550	7131.77	7.1E+05	2.1E+11	512.45	1596.14	15.02	**33.45**
3L/8	11200	7240.02	7.1E+05	2.1E+11	624.82	1995.17	17.63	**39.25**
L/2	14850	7157.73	7.1E+05	2.1E+11	662.28	2128.18	18.22	**40.56**

Total loss of prestressing force immediately - Remaining prestressing force:

Tendon1	Xi	Δf_{pF}	Δf_{pA}	Δf_{ES}	ΣΔ	F_j^1	(α)	Fj¹*Cos(α)	Fj¹*Sin(α)
Section	(mm)	(MPa)	(MPa)	(MPa)	(MPa)	(kN)	(rad)	(kN)	(kN)
anchorage	0	0.00	213.02	13.47	226.50	1592.28	0.1655	1570.53	262.28
Support	250	1.13	208.22	13.50	222.85	1597.68	0.1627	1576.57	258.86
L/8	3900	18.41	138.16	23.42	179.99	1661.14	0.1225	1648.69	203.03
L/4	7550	48.39	68.10	33.45	149.94	1705.63	0.0819	1699.91	139.56
3L/8	11200	90.18	0.00	39.25	129.43	1735.99	0.0410	1734.53	71.20
L/2	14850	142.53	0.00	40.56	183.08	1656.56	0.0000	1656.56	0.00

Tendon2	Xi	Δf_{pF}	Δf_{pA}	Δf_{ES}	ΣΔ	F_j^2	(α)	Fj²*Cos(α)	Fj²*Sin(α)
Section	(mm)	(MPa)	(MPa)	(MPa)	(MPa)	(kN)	(rad)	(kN)	(kN)
anchorage	0	0.00	187.42	13.47	200.90	1630.18	0.1233	1617.81	200.46
Support	250	0.89	183.71	13.50	198.10	1634.32	0.1212	1622.33	197.63
L/8	3900	14.54	129.47	23.42	167.43	1679.73	0.0911	1672.77	152.83
L/4	7550	37.71	75.24	33.45	146.39	1710.88	0.0608	1707.72	104.02
3L/8	11200	69.87	21.00	39.25	130.13	1734.96	0.0304	1734.16	52.81
L/2	14850	110.33	0.00	40.56	150.89	1704.22	0.0000	1704.22	0.00

Tendon3	Xi	Δf_{pF}	Δf_{pA}	Δf_{ES}	ΣΔ	F_j^3	(α)	Fj³*Cos(α)	Fj³*Sin(α)
Section	(mm)	(MPa)	(MPa)	(MPa)	(MPa)	(kN)	(rad)	(kN)	(kN)
anchorage	0	0.00	154.48	11.98	166.46	1681.17	0.0806	1675.71	135.41
Support	250	0.66	151.88	12.00	164.54	1684.01	0.0793	1678.72	133.37
L/8	3900	10.66	113.91	20.81	145.38	1712.37	0.0595	1709.34	101.85
L/4	7550	26.93	75.94	29.73	132.60	1731.30	0.0397	1729.94	68.72
3L/8	11200	49.23	37.97	34.89	122.09	1746.85	0.0199	1746.51	34.69
L/2	14850	77.24	0.00	36.05	113.29	1759.88	0.0000	1759.88	0.00

Tendon4	Xi	Δf_{pF}	Δf_{pA}	Δf_{ES}	$\Sigma\Delta$	F_j^4	(α)	$Fj^4 \cdot Cos(\alpha)$	$Fj^4 \cdot Sin(\alpha)$
Section	(mm)	(MPa)	(MPa)	(MPa)	(MPa)	(kN)	(rad)	(kN)	(kN)
anchorage	0	0.00	86.46	8.98	95.44	1786.31	0.0377	1785.04	67.31
Support	250	0.42	85.01	9.00	94.43	1787.81	0.0371	1786.58	66.24
L/8	3900	6.76	63.75	15.61	86.13	1800.10	0.0278	1799.40	50.04
L/4	7550	16.06	42.50	22.30	80.86	1807.90	0.0185	1807.59	33.51
3L/8	11200	28.24	21.25	26.17	75.66	1815.59	0.0093	1815.52	16.83
L/2	14850	43.23	0.00	27.04	70.27	1823.58	0.0000	1823.58	0.00

SUM 1to4	Xi	ΣF_j	$Fj^* Cos(\alpha)$	$Fj^* Sin(\alpha)$	$e_{cáp}$	$M_j = \Sigma F_j Cos(\alpha)^* e_{cáp}$
Section	(mm)	(kN)	(kN)	(kN)	(mm)	(kNm)
anchorage	0	6689.95	6649.09	665.47	76.53	508.86
Support	250	6703.82	6664.20	656.10	76.53	510.02
L/8	3900	6853.34	6830.20	507.75	331.40	2263.52
L/4	7550	6955.72	6945.16	345.80	512.45	3559.05
3L/8	11200	7033.40	7030.71	175.53	624.82	4392.93
L/2	14850	6944.24	6944.24	0.00	662.28	4599.01

4.2. Loss of prestressing force at service stage (time - dependent losses):

4.2.1 Loss of prestress due to Shrinkage:

Formula: $\Delta fp_{SH} = (93 - 0.85*H)$

Relative humidity of environment $H =$ 80.00 %

$\boxed{\Delta fp_{SH} = \textbf{25.00 (MPa)}}$

4.2.2 Loss of prestress due to Creep:

Formula $\Delta fp_{CR} = 12.0\, f_{cgp} - 7.0 * \Delta f_{cdp}$

In which:

f_{cgp} Stress in concrete at tendons centroid (f_{cgp}) due to prestressing tendon and self weigh of girder

Δf_{cdp} Stess at tendons centroid changes due to permanent load, except dead load action at transfer

Section	Xi		Interior Girder		Exterior Girder	
		f_{cgp}	Δf_{cdp}	Δfp_{CR}	Δf_{cdp}	Δfp_{CR}
	(m)	(MPa)	(MPa)	(MPa)	(MPa)	(MPa)
Support	0.00	6.07	0.00	**72.78**	0.00	**72.78**
L/8	3.65	10.52	0.92	**119.79**	1.69	**114.40**
L/4	7.30	15.02	4.89	**146.02**	4.45	**149.12**
3L/8	10.95	17.63	5.26	**174.74**	6.62	**165.24**
L/2	14.60	18.22	8.20	**161.20**	7.44	**166.54**

4.2.3 Loss of prestress due to Relaxation:

(a) At transfer:

Formula:

$$\Delta f_{pR1} = \frac{\log(24t)}{40}\left[\frac{f_{pj}}{f_{py}} - 0,55\right]f_{pj}$$

In which:

t : time estimated in days from stressing to transfer $t =$ 0.00 days

fpy : Specified yeild strength of prestressing steel $f_{py} =$ 1674.00 (MPa)

fpj : Initial stress in the tendon at the end of stressing

Section	Xi	fpj	Δfp_{R1}
	(m)	(MPa)	(MPa)
Support	0.00	1288.50	**0.00**
L/8	3.65	1278.58	**0.00**
L/4	7.30	1268.55	**0.00**
3L/8	10.95	1262.75	**0.00**
L/2	14.60	1261.44	**0.00**

(b) After Transfer:

Formula: $\Delta fp_{R2} = 30\%*(138 - 0.3\,\Delta fp_F - 0.4\,\Delta f_{ES} - 0.2\,(\Delta fp_{SH} + \Delta fp_{CR}))$

Interior Girder						
Section	Xi	Δf_{pF}	Δf_{pES}	Δfp_{SH}	Δfp_{CR}	Δfp_{R2}
	(m)	(MPa)	(MPa)	(MPa)	(MPa)	(MPa)
Support	0.00	0.00	13.50	25.00	18.20	**37.19**
L/8	3.65	0.00	23.42	25.00	29.95	**35.29**
L/4	7.30	0.00	33.45	25.00	36.51	**33.70**
3L/8	10.95	0.00	39.25	25.00	43.68	**32.57**
L/2	14.60	0.00	40.56	25.00	40.30	**32.62**

Exterior Girder						
Section	Xi	Δf_{pF}	Δf_{pES}	Δfp_{SH}	Δfp_{CR}	Δfp_{R2}
	(m)	(MPa)	(MPa)	(MPa)	(MPa)	(MPa)
Support	0.00	0.00	13.50	25.00	18.20	**37.19**
L/8	3.65	0.00	23.42	25.00	28.60	**35.37**
L/4	7.30	0.00	33.45	25.00	37.28	**33.65**
3L/8	10.95	0.00	39.25	25.00	41.31	**32.71**
L/2	14.60	0.00	40.56	25.00	41.64	**32.54**

TOTAL LOSS STRESS AT SERVICE STAGE

Interior Girder						
Setion	Xi	Δfp_{SH}	Δfp_{CR}	Δfp_{R1}	Δfp_{R2}	**Sum**
	(m)	(MPa)	(MPa)	(MPa)	(MPa)	**(MPa)**
Support	0.00	25.00	72.78	0.00	37.19	**134.97**
L/8	3.65	25.00	119.79	0.00	35.29	**180.08**
L/4	7.30	25.00	146.02	0.00	33.70	**204.72**
3L/8	10.95	25.00	174.74	0.00	32.57	**232.31**
L/2	14.60	25.00	161.20	0.00	32.62	**218.81**

Exterior Girder						
Section	Xi	Δfp_{SH}	Δfp_{CR}	Δfp_{R1}	Δfp_{R2}	**Sum**
	(m)	(MPa)	(MPa)	(MPa)	(MPa)	**(MPa)**
Support	0.00	25.00	72.78	0.00	37.19	**134.97**
L/8	3.65	25.00	114.40	0.00	35.37	**174.77**
L/4	7.30	25.00	149.12	0.00	33.65	**207.77**
3L/8	10.95	25.00	165.24	0.00	32.71	**222.95**
L/2	14.60	25.00	166.54	0.00	32.54	**224.08**

4.3. Total Prestressing force consider loss in the service stage:

Interior Girder

Tendon1	Xi	$\Sigma\Delta_{pT}$	F_j^1	(α)	$Fj^1*Cos(\alpha)$	$Fj^1*Sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	357.83	1397.85	0.1627	1379.38	226.48
L/8	3.65	360.07	1394.53	0.1225	1384.07	170.44
L/4	7.30	354.65	1402.55	0.0819	1397.84	114.76
3L/8	10.95	361.74	1392.06	0.0410	1390.89	57.09
L/2	14.60	401.89	1332.61	0.0000	1332.61	0.00

Tendon2	Xi	$\Sigma\Delta_{pT}$	F_j^2	(α)	$Fj^2*Cos(\alpha)$	$Fj^2*Sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	333.08	1434.49	0.1233	1423.61	176.39
L/8	3.65	347.51	1413.12	0.1212	1402.75	170.88
L/4	7.30	351.11	1407.80	0.0911	1401.96	128.09
3L/8	10.95	362.43	1391.03	0.0608	1388.46	84.57
L/2	14.60	369.70	1380.27	0.0304	1379.63	42.02

Tendon3	Xi	$\Sigma\Delta_{pT}$	F_j^3	(α)	$Fj^3*Cos(\alpha)$	$Fj^3*Sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	301.43	1481.35	0.0793	1476.69	117.32
L/8	3.65	344.62	1417.40	0.0595	1414.89	84.31
L/4	7.30	350.10	1409.29	0.0397	1408.18	55.94
3L/8	10.95	364.90	1387.37	0.0199	1387.10	27.55
L/2	14.60	340.90	1422.90	0.0000	1422.90	0.00

Tendon4	Xi	$\Sigma\Delta_{pT}$	F_j^4	(α)	$Fj^4*Cos(\alpha)$	$Fj^4*Sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	230.42	1586.48	0.0371	1585.39	58.78
L/8	3.65	274.51	1521.20	0.0278	1520.61	42.28
L/4	7.30	290.85	1497.01	0.0185	1496.75	27.75
3L/8	10.95	313.17	1463.97	0.0093	1463.91	13.57
L/2	14.60	294.47	1491.64	0.0000	1491.64	0.00

SUM 1to4	Xi	ΣF_j	$Fj*Cos(\alpha)$	$Vp=Fj*Sin(\alpha)$	e_{cable}	$M_j = \Sigma F_j Cos(\alpha)*e_{cáp}$
Section	(mm)	(kN)	(kN)	(kN)	(m)	(kNm)
Support	0.00	5900.17	5865.07	578.98	0.08	448.9
L/8	3.65	5746.24	5722.32	467.91	0.33	1896.4
L/4	7.30	5716.64	5704.73	326.53	0.51	2923.4
3L/8	10.95	5634.43	5630.35	182.78	0.62	3518.0
L/2	14.60	5627.43	5626.79	42.02	0.66	3726.5

Exterior Girder

Tendon1	Xi	$\Sigma\Delta_{pT}$	F_j^1	(α)	$Fj^1*Cos(\alpha)$	$Fj^1*Sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	357.83	1397.85	0.1627	1379.38	226.48
L/8	3.65	354.76	1402.39	0.1225	1391.88	171.40
L/4	7.30	357.71	1398.03	0.0819	1393.34	114.39
3L/8	10.95	352.38	1405.92	0.0410	1404.73	57.66
L/2	14.60	407.16	1324.81	0.0000	1324.81	0.00

Tendon2	Xi	$\Sigma\Delta_{pT}$	F_j^2	(α)	$Fj^2*Cos(\alpha)$	$Fj^2*Sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	333.08	1434.49	0.1233	1423.61	176.39
L/8	3.65	342.20	1420.99	0.1212	1410.56	171.83
L/4	7.30	354.16	1403.28	0.0911	1397.46	127.68
3L/8	10.95	353.07	1404.89	0.0608	1402.29	85.41
L/2	14.60	374.97	1372.47	0.0304	1371.84	41.78

Tendon3	Xi	$\Sigma\Delta_{pT}$	F_j^3	(α)	$Fj^3*Cos(\alpha)$	$Fj^3*Sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	301.43	1481.35	0.0793	1476.69	117.32
L/8	3.65	339.31	1425.27	0.0595	1422.74	84.78
L/4	7.30	353.15	1404.77	0.0397	1403.66	55.76
3L/8	10.95	355.54	1401.23	0.0199	1400.95	27.83
L/2	14.60	346.17	1415.10	0.0000	1415.10	0.00

Tendon4	Xi	$\Sigma\Delta_{pT}$	F_j^4	(α)	$Fj^4*Cos(\alpha)$	$Fj^4*Sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	230.42	1586.48	0.0371	1585.39	58.78
L/8	3.65	269.20	1529.06	0.0278	1528.47	42.50
L/4	7.30	293.90	1492.49	0.0185	1492.24	27.66
3L/8	10.95	303.81	1477.83	0.0093	1477.76	13.70
L/2	14.60	299.74	1483.84	0.0000	1483.84	0.00

SUM 1to4	Xi	ΣF_j	$Fj*Cos(\alpha)$	$Vp=Fj*Sin(\alpha)$	e_{caple}	$M_j = \Sigma F_j Cos(\alpha)*e_{cáp}$
Section	(mm)	(kN)	(kN)	(kN)	(m)	(kNm)
Support	0.00	5900.17	5865.07	578.98	0.08	448.9
L/8	3.65	5777.71	5753.65	470.51	0.33	1906.8
L/4	7.30	5698.57	5686.69	325.49	0.51	2914.2
3L/8	10.95	5689.86	5685.74	184.60	0.62	3552.6
L/2	14.60	5596.23	5595.59	41.78	0.66	3705.8

5. FIBRE STRESS CHECK:

Formula:

Top fibre: $f_{ti} = \frac{F_i}{A} - \frac{F_i e}{S_t} + \frac{M_{DC}}{S_t}$ Bottom fibre $f_{bi} = \frac{F_i}{A} + \frac{F_i e}{S_b} - \frac{M_{DC}}{S_b}$

Note (+) : Compression tresses ; (-) Tension stresses
Concrete strength at transfer fci' = 0.9 fc = 40.50 MPa
Comprresion stress Limit at transfer 0.6 fci' = 24.30 MPa
Tension stress Limit at transfer 0.25 SQRT(fci') < 1.38 = -1.38 MPa (5.9.4.1.2-1)

Setion	Xi	A	St	Sb	Fj'*Cos(α)	e	M_{DC}	f_{ti}	f_{bi}	Kiểm tra	
	(mm)	(mm2)	(mm3)	(mm3)	(kN)	(mm)	(kNm)	(MPa)	(MPa)	f_{ti}	f_{bi}
G. end	0	1.15E+06	3.19E+08	3.07E+08	6649.09	76.53	0.00	4.20	7.46	OK	OK
Support	250	1.15E+06	3.19E+08	3.07E+08	6664.20	76.53	0.00	4.21	7.47	OK	OK
L/8	3900	8.23E+05	2.83E+08	2.77E+08	6830.20	331.40	931.08	3.58	13.11	OK	OK
L/4	7550	7.07E+05	2.68E+08	2.66E+08	6945.16	512.45	1596.14	2.50	17.19	OK	OK
3L/8	11200	7.07E+05	2.68E+08	2.66E+08	7030.71	624.82	1995.17	1.00	18.94	OK	OK
L/2	14850	7.07E+05	2.68E+08	2.66E+08	6944.24	662.28	2128.18	0.60	19.10	OK	OK

5.2 Stress check during contruction the deck:

5.2.1 Increase load:

Exterior Diaphragms beam DC_{dn1} = 47.95 (kN)
Interior Diaphragms beam DC_{dn1} = 33.83 (kN)
Precast plank DC_{VK} = 3.55 (kN/m)
Wet concrete of deck DC_{mc} = 13.32 (kN/m)

5.2.2 Stress check:

Compression strength of concrete fc = 45.00 MPa
Compression stress limit 0.45 fc = 20.25 MPa (5.9.4.2.1-1)
Tension stress limit 0.5 SQRT(fc) = -3.35 MPa (5.9.4.2.2-1)

Setion	Xi	A	St	Sb	Fi	e	M_{DC}	f_{ti}	f_{bi}	Kiểm tra	
	(mm)	(mm2)	(mm3)	(mm3)	(kN)	(mm)	(kNm)	(MPa)	(MPa)	f_{ti}	f_{bi}
G. end	0	1.15E+06	3.19E+08	3.07E+08	6649.09	76.53	0.00	4.20	7.46	OK	OK
Support	250	1.15E+06	3.19E+08	3.07E+08	6664.20	76.53	0.00	4.21	7.47	OK	OK
L/8	3900	8.23E+05	2.83E+08	2.77E+08	6830.20	331.40	2392.73	8.75	7.83	OK	OK
L/4	7550	7.07E+05	2.68E+08	2.66E+08	6945.16	512.45	3037.25	7.88	11.78	OK	OK
3L/8	11200	7.07E+05	2.68E+08	2.66E+08	7030.71	624.82	3796.57	7.72	12.18	OK	OK
L/2	14850	7.07E+05	2.68E+08	2.66E+08	6944.24	662.28	4049.67	7.77	11.89	OK	OK

5.3 Stress check at the top fibre of Girder - Service state :

5.3.1 Due to prestressing tendon and self weigh of girder - Service limit state I:

Compression Stress Limit: 0.45 fc = 20.25 MPa (5.9.4.2.1-1)
Tension Stress Limit: - 0.5*SQRT(fc) = -3.35 MPa

$$f_f = \frac{P_{pe}}{A} - \frac{P_{pe}e_c}{S_t} + \frac{M_g + M_s}{S_t} + \frac{M_{SDL}}{S_{tg}}$$

Interior Girder

Setion	Xi	A	S_t	S_{tg}	P_{pe}	$P_{pe}*e_c$	$M_g + M_s$	M_{SDL}	f_t	Check
	(mm)	(mm2)	(mm3)	(mm3)	(kN)	(kNm)	(kNm)	(kNm)	(MPa)	f_t
Support	250	1.15E+06	3.19E+08	1.00E+09	5865.07	448.86	0.00	0.00	3.706	OK
L/8	3900	8.23E+05	2.83E+08	1.02E+09	5722.32	1896.37	1771.73	428.12	6.928	OK
L/4	7550	7.07E+05	2.68E+08	1.04E+09	5704.73	2923.40	3037.25	733.92	9.198	OK
3L/8	11200	7.07E+05	2.68E+08	1.04E+09	5630.35	3517.95	3796.57	917.40	9.883	OK
L/2	14850	7.07E+05	2.68E+08	1.04E+09	5626.79	3726.49	4049.67	978.56	10.103	OK

Exterior Girder

Setion	Xi	A	S_t	S_{tg}	P_{pe}	$P_{pe}*e_c$	$M_g + M_s$	M_{SDL}	f_t	Check
	(mm)	(mm2)	(mm3)	(mm3)	(kN)	(kNm)	(kNm)	(kNm)	(MPa)	f_t
Support	250	1.15E+06	3.19E+08	9.99E+08	5865.07	448.86	0.00	0.00	3.706	OK
L/8	3900	8.23E+05	2.83E+08	1.02E+09	5753.65	1906.76	1661.91	430.69	6.544	OK
L/4	7550	7.07E+05	2.68E+08	1.04E+09	5686.69	2914.15	2848.98	738.32	8.510	OK
3L/8	11200	7.07E+05	2.68E+08	1.04E+09	5685.74	3552.56	3561.23	922.90	8.961	OK
L/2	14850	7.07E+05	2.68E+08	1.04E+09	5595.59	3705.83	3798.64	984.43	9.206	OK

5.3.2 Due to 1/2 (Prestressing tendon + self weigh of girder) and Live load - Service limit state I:

Compression Stress Limit: 0.40 fc = 18.00 MPa (5.9.4.2.1-1)

Tension Stress Limit: - 0.5*SQRT(fc) = -3.35 MPa

$$f_t = 0.5\left(\frac{P_{pe}}{A} - \frac{P_{pe}e_c}{S_t} + \frac{M_g + M_s}{S_t} + \frac{M_{SDL}}{S_{tg}}\right) + \frac{M_{LL}}{S_{tg}}$$

Interior Girder

Setion	Xi	A	S_t	S_{tg}	P_{pe}	$P_{pe}*e_c$	$M_g + M_s$	M_{SDL}	M_{LL}	f_t	Check
	(mm)	(mm2)	(mm3)	(mm3)	(kN)	(kNm)	(kNm)	(kNm)	(kNm)	(MPa)	f_t
Support	250	1.15E+06	3.19E+08	1.00E+09	5865.07	448.86	0.00	0.00	0.00	1.853	OK
L/8	3900	8.23E+05	2.83E+08	1.02E+09	5722.32	1896.37	1771.73	428.12	1177.94	4.614	OK
L/4	7550	7.07E+05	2.68E+08	1.04E+09	5704.73	2923.40	3037.25	733.92	1988.69	6.502	OK
3L/8	11200	7.07E+05	2.68E+08	1.04E+09	5630.35	3517.95	3796.57	917.40	2449.70	7.286	OK
L/2	14850	7.07E+05	2.68E+08	1.04E+09	5626.79	3726.49	4049.67	978.56	2578.43	7.519	OK

Exterior Girder

Setion	Xi	A	S_t	S_{tg}	P_{pe}	$P_{pe}*e_c$	$M_g + M_s$	M_{SDL}	M_{LL}	f_t	Check
	(mm)	(mm2)	(mm3)	(mm3)	(kN)	(kNm)	(kNm)	(kNm)	(kNm)	(MPa)	f_t
Support	250	1147000.0	3.19E+08	9.99E+08	5865.07	448.86	0.00	0.00	0.00	1.853	OK
L/8	3900	823265.0	2.83E+08	1.02E+09	5753.65	1906.76	1661.91	430.69	1358.92	4.601	OK
L/4	7550	706850.0	2.68E+08	1.04E+09	5686.69	2914.15	2848.98	738.32	2294.23	6.454	OK
3L/8	11200	706850.0	2.68E+08	1.04E+09	5685.74	3552.56	3561.23	922.90	2826.07	7.189	OK
L/2	14850	706850.0	2.68E+08	1.04E+09	5595.59	3705.83	3798.64	984.43	2974.58	7.455	OK

5.3.3 Due to prestressing tendon + self weigh of girder + live load - Service limit state I:

Compression Stress Limit: 0.60 fc = 27.00 MPa (5.9.4.2.1-1)

Tension Stress Limit: - 0.5*SQRT(fc) = -3.35 MPa

$$f_t = \left(\frac{P_{pe}}{A} - \frac{P_{pe}e_c}{S_t} + \frac{M_g + M_s}{S_t} + \frac{M_{SDL}}{S_{tg}}\right) + \frac{M_{LL}}{S_{tg}}$$

Interior Girder

Setion	Xi	A	S_t	S_{tg}	P_{pe}	$P_{pe}*e_c$	$M_g + M_s$	M_{SDL}	M_{LL}	f_t	Check
	(mm)	(mm2)	(mm3)	(mm3)	(kN)	(kNm)	(kNm)	(kNm)	(kNm)	(MPa)	f_t
Support	250	1.15E+06	3.19E+08	1.00E+09	5865.07	448.86	0.00	0.00	0.00	3.706	OK
L/8	3900	8.23E+05	2.83E+08	1.02E+09	5722.32	1896.37	1771.73	428.12	1177.94	8.078	OK
L/4	7550	7.07E+05	2.68E+08	1.04E+09	5704.73	2923.40	3037.25	733.92	1988.69	11.101	OK
3L/8	11200	7.07E+05	2.68E+08	1.04E+09	5630.35	3517.95	3796.57	917.40	2449.70	12.228	OK
L/2	14850	7.07E+05	2.68E+08	1.04E+09	5626.79	3726.49	4049.67	978.56	2578.43	12.571	OK

Exterior Girder

Setion	Xi	A	S_t	S_{tg}	P_{pe}	$P_{pe}*e_c$	$M_g + M_s$	M_{SDL}	M_{LL}	f_t	Check
	(mm)	(mm2)	(mm3)	(mm3)	(kN)	(kNm)	(kNm)	(kNm)	(kNm)	(MPa)	f_t
Support	250	1.15E+06	3.19E+08	9.99E+08	5865.07	448.86	0.00	0.00	0.00	3.706	OK
L/8	3900	8.23E+05	2.83E+08	1.02E+09	5753.65	1906.76	1661.91	430.69	1358.92	7.873	OK
L/4	7550	7.07E+05	2.68E+08	1.04E+09	5686.69	2914.15	2848.98	738.32	2294.23	10.709	OK
3L/8	11200	7.07E+05	2.68E+08	1.04E+09	5685.74	3552.56	3561.23	922.90	2826.07	11.670	OK
L/2	14850	7.07E+05	2.68E+08	1.04E+09	5595.59	3705.83	3798.64	984.43	2974.58	12.058	OK

5.4 Stress check at the top fibre of deck - Service state:

5.4.1 Due to additional load (dead load part 2) - Service limit state I:

Compression Stress Limit: 0.45 fc = 15.75 MPa (5.9.4.2.1-1)

$$f_t = \frac{M_{SDL}}{S_{tc}}$$

Setion	Xi	MSDL (kNm)		S_{tc} (mm3)		f_t (MPa)		Check	
	(mm)	In.Girder	Ex.Girder	In.Girder	Ex.Girder	In.Girder	Ex.Girder	In.Girder	Ex.Girder
Support	250.00	0.00	0.00	6.2E+08	6.14E+08	0.000	0.000	OK	OK
L/8	3900.00	428.12	430.69	6.1E+08	6.05E+08	0.707	0.712	OK	OK
L/4	7550.00	733.92	738.32	6E+08	6.04E+08	1.214	1.223	OK	OK
3L/8	11200.00	917.40	922.90	6E+08	6.04E+08	1.518	1.529	OK	OK
L/2	14850.00	978.56	984.43	6E+08	6.04E+08	1.619	1.630	OK	OK

5.4.2 Due to additional load (dead load part 2) and live load - Service limit state I:

Compression Stress Limit: 0.6 fc = 21.00 MPa (5.9.4.2.1-1)

$$f_{tc} = \frac{M_{SDL} + M_{LL}}{S_{tc}}$$

Setion	Xi	MSDL + MLL (kNm)		S_{tc}(mm3)		f_t(MPa)		Check	
	(mm)	in.Girder	Ex.Girder	in.Girder	Ex.Girder	in.Girder	Ex.Girder	in.Girder	Ex.Girder
Support	250.00	0.00	0.00	6.2E+08	6.14E+08	0.000	0.000	OK	OK
L/8	3900.00	1606.07	1789.61	6.1E+08	6.05E+08	2.652	2.959	OK	OK
L/4	7550.00	2722.61	3032.55	6E+08	6.04E+08	4.504	5.023	OK	OK
3L/8	11200.00	3367.10	3748.97	6E+08	6.04E+08	5.570	6.209	OK	OK
L/2	14850.00	3556.99	3959.01	6E+08	6.04E+08	5.884	6.557	OK	OK

5.5 Stress check at the bottom fibre of girder - Service III (stage III):

Compression Stress Limit: 0.45 fc = 20.25 MPa (5.9.4.2.1-1)

Tension Stress Limit: - 0.5*SQRT(fc) = -3.35 MPa (5.9.4.2.1-1)

$$f_b = \frac{P_{pe}}{A} + \frac{P_{pe}e_c}{S_b} - \frac{M_g + M_s}{S_b} - \frac{M_{SDL} + M_{LL}}{S_{bc}}$$

Interior Girder

Setion	Xi	A	S_b	S_{bc}	P_{pe}	P_{pe}*e_c	$M_g + M_s$	M_{SDL}	M_{LL}	f_b	Check
	(mm)	(mm2)	(mm3)	(mm3)	(kN)	(kNm)	(kNm)	(kNm)	(kNm)	(MPa)	f_b
Support	250	1.15E+06	3.07E+08	4.78E+08	5865.07	448.86	0.00	0.00	0.00	6.578	OK
L/8	3900	8.23E+05	2.77E+08	4.10E+08	5722.32	1896.37	1771.73	428.12	1177.94	4.056	OK
L/4	7550	7.07E+05	2.66E+08	3.82E+08	5704.73	2923.40	3037.25	733.92	1988.69	1.561	OK
3L/8	11200	7.07E+05	2.66E+08	3.82E+08	5630.35	3517.95	3796.57	917.40	2449.70	-0.608	OK
L/2	14850	7.07E+05	2.66E+08	3.82E+08	5626.79	3726.49	4049.67	978.56	2578.43	-1.210	OK

Exterior Girder

Setion	Xi	A	S_b	S_{bc}	P_{pe}	P_{pe}*e_c	$M_g + M_s$	M_{SDL}	M_{LL}	f_b	Check
	(mm)	(mm2)	(mm3)	(mm3)	(kN)	(kNm)	(kNm)	(kNm)	(kNm)	(MPa)	f_b
Support	250	1.15E+06	3.07E+08	4.78E+08	5865.07	448.86	0.00	0.00	0.00	6.578	OK
L/8	3900	8.23E+05	2.77E+08	4.10E+08	5753.65	1906.76	1661.91	430.69	1358.92	4.167	OK
L/4	7550	7.07E+05	2.66E+08	3.82E+08	5686.69	2914.15	2848.98	738.32	2294.23	1.554	OK
3L/8	11200	7.07E+05	2.66E+08	3.82E+08	5685.74	3552.56	3561.23	922.90	2826.07	-0.321	OK
L/2	14850	7.07E+05	2.66E+08	3.82E+08	5595.59	3705.83	3798.64	984.43	2974.58	-1.236	OK

5.6 Stress check at the bottom fibre of girder - Service I (Stage III):

Compression Stress Limit: 0.45 fc = 20.25 MPa (5.9.4.2.1-1)

Tension Stress Limit: - 0.5*SQRT(fc) = -3.35 MPa (5.9.4.2.1-1)

$$f_b = \frac{P_{pe}}{A} + \frac{P_{pe}e_c}{S_b} - \frac{M_g + M_s}{S_b} - \frac{M_{SDL} + M_{LL}}{S_{bc}}$$

Interior Girder

Setion	Xi	A	S_b	S_{bc}	P_{pe}	P_{pe}*e_c	$M_g + M_s$	M_{SDL}	M_{LL}	f_b	Check
	(mm)	(mm2)	(mm3)	(mm3)	(kN)	(kNm)	(kNm)	(kNm)	(kNm)	(MPa)	f_b
Support	250	1.15E+06	3.07E+08	4.78E+08	5865.07	448.86	0.00	0.00	0.00	6.58	OK
L/8	3900	8.23E+05	2.77E+08	4.10E+08	5722.32	1896.37	1771.73	428.12	1177.94	3.48	OK
L/4	7550	7.07E+05	2.66E+08	3.82E+08	5704.73	2923.40	3037.25	733.92	1988.69	0.52	OK
3L/8	11200	7.07E+05	2.66E+08	3.82E+08	5630.35	3517.95	3796.57	917.40	2449.70	-1.89	OK
L/2	14850	7.07E+05	2.66E+08	3.82E+08	5626.79	3726.49	4049.67	978.56	2578.43	-2.56	OK

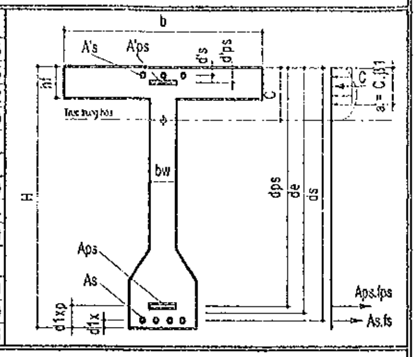
Exterior Girder

Setion	Xi	A	S_b	S_{bc}	P_{pe}	P_{pe}*e_c	$M_g + M_s$	M_{SDL}	M_{LL}	f_b	Check
	(mm)	(mm2)	(mm3)	(mm3)	(kN)	(kNm)	(kNm)	(kNm)	(kNm)	(MPa)	f_b
Support	250	1.15E+06	3.07E+08	4.78E+08	5865.07	448.86	0.00	0.00	0.00	6.578	OK
L/8	3900	8.23E+05	2.77E+08	4.10E+08	5753.65	1906.76	1661.91	430.69	1358.92	3.504	OK
L/4	7550	7.07E+05	2.66E+08	3.82E+08	5686.69	2914.15	2848.98	738.32	2294.23	0.353	OK
3L/8	11200	7.07E+05	2.66E+08	3.82E+08	5685.74	3552.56	3561.23	922.90	2826.07	-1.800	OK
L/2	14850	7.07E+05	2.66E+08	3.82E+08	5595.59	3705.83	3798.64	984.43	2974.58	-2.793	OK

REINFORCEMENT OF GIRDER CHECKING - STRENGTH LOAD COMBINATION

MATERIALS

	NORMAL CONCRETE		
f'c	Compressive Strength of concrete at 28 days	Mpa	45
Ec	Modulus of Elasticity	Mpa	33915
fr	Modulus of Rupture	Mpa	4.2
gc	Unit weight of concrete	kN/m3	24.0
	PRESTRESSING STEEL		
fpu	Tensile strength of prestressing steel	Mpa	1860
fpy	Yield strength of prestressing steel	Mpa	1674
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: Support	L/8	L/4	3L/8	L/2
	INTERNAL FORCES AT SECTION							
	Combination			Strength	Strength	Strength	Strength	Strength
Qu	Shear		kN	1631	1288	951	620	296
Mu	Flexural Moment		kNm	0	5046	8588	10662	11303
Nu	Axial load		kN					
Tu	Torsional Moment		kNm					
	6.1 FLEXURAL MOMENT CHECKING							
H	Section height		m	1.800	1.800	1.800	1.800	1.800
d's	Dis. From comp. fiber to centroid of comp. Reinf		m	0.062	0.062	0.062	0.062	0.062
d1x	Dis. From tens. fiber to centroid of tension Reinf		m	0.062	0.062	0.062	0.062	0.062
	Cover to reinf		m	0.040	0.040	0.040	0.040	0.040
ds	Dis. From comp. fiber to centroid of tension Reinf		m	1.738	1.738	1.738	1.738	1.738
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	0.739	0.477	0.290	0.177	0.140
dps	Dis. From comp. fiber to centroid of tension prestressing steel		m	1.061	1.323	1.510	1.623	1.660
b	Width of the compression face of member		m	2.249	2.249	2.249	2.249	2.249
bw	Web width or diameter of a circular section		m	0.700	0.700	0.250	0.250	0.250
hf	Compression flange depth		m	0.200	0.200	0.200	0.200	0.200
Iz	Moment of inertia of section		m4	0.513	0.464	0.443	0.443	0.443
Amc	Section area		m2	1.613	1.289	1.172	1.172	1.172
	Steel choice							
Aps	Tension prestressing steel	P.S type		15 T12.7	15 T12.7	15 T12.7	15 T12.7	15 T12.7
		Number	tendons	5	5	5	5	5
		Area	m2	0.00740	0.00740	0.00740	0.00740	0.00740
A'ps	Compression prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	0 T00.0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
As	Tension Reinforcement	Number	bars	6	6	6	6	6
		Diameter	mm	16	16	16	16	16
		Area	m2	0.00121	0.00121	0.00121	0.00121	0.00121
A's	Compression Reinforcement	Number	bars	4	4	4	4	4
		Diameter	mm	12	12	12	12	12
		Area	m2	0.00045	0.00045	0.00045	0.00045	0.00045
A'c	Shear reinforcement	Number	bars	2	2	2	2	2
		Diameter	mm	16	16	16	16	16
		Area	m2	0.00040	0.00040	0.00040	0.00040	0.00040
f	Resistance factors for flexure		5.5.4.2	0.90	0.90	0.90	0.90	0.90
fv	Resistance factors for shear			0.90	0.90	0.90	0.90	0.90
fn	Resistance factors for axial force			1.00	1.00	1.00	1.00	1.00
b1	Stress block factor			0.729	0.729	0.729	0.729	0.729
c	Dis. Between centroid and top fiber		m	0.235	0.243	0.308	0.314	0.316
	For T section behavior		m	0.235	0.243	0.308	0.314	0.316
	For rectangular section behavior		m	0.212	0.215	0.216	0.216	0.217
fpe	Effective stress in the prestressing steel after losses		Mpa	1398	1395	1403	1387	1333
fps	Aver. stress in pres. steel at the time for which the nominal resistance		Mpa	1745	1764	1754	1759	1761
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	0.171	0.177	0.224	0.229	0.230
de	Corresp. effective depth from extreme comp. fiber to centroid of tensile force in the tensile reinf.		m	1.085	1.338	1.518	1.627	1.663
Mn	Nominal resistance		kNm	13274	16829	19082	20598	21104
Mr	Factored reistance		kNm	11946	15146	17174	18538	18993
Mu	Flexural moment		kNm	0	5046	8588	10662	11303
(5.7.3.2)	Flexural moment Checking			OK	OK	OK	OK	OK

	Limits for reinforcement						
c/de	Maximum reinforcement		0.22	0.18	0.20	0.19	0.19
	Maximum reinforcement Checking	<= 0.42	OK	OK	OK	OK	OK
r min	Mimimum reinforcement		0.08%	0.09%	0.10%	0.10%	0.10%
	Mimimum reinforcement Checking for RC	0.34%	N.a	N.a	N.a	N.a	N.a
1.2*Mcr	Craking moment	kNm	1661	1510	1507	1513	1515
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**		OK	OK	OK	OK	OK
(5.8.3.5)	Tensile force in steel should be satisfied - F_yc	kN	1779	5730	7823	8509	8436
	Checking As.fy+Aps.fps >= F_yc		Ok	Ok	Ok	Ok	Ok
(5.7.3.4)	**Conctrol of craking by distr. of reinf for RC member- Check?**		No	No	No	No	No
	Existing condition for structrure	1,2 or 3	1	1	1	1	1
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.058	0.058	0.058	0.058	0.058
Z	Crack width parameter	N/mm	30000	30000	30000	30000	30000
A	Area of concr. with same centroid as tens. Reinf	m2	0.043	0.043	0.043	0.043	0.043
fsa	Value	Mpa	220	220	220	220	220
0.6*fy		Mpa	240	240	240	240	240
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	220	220	220	220	220
x	Dist. From compression fiber to centroid	m	-	-	-	-	-
J.d	Arm	m	-	-	-	-	-
Icr	Moment of inertia of the cracked section	m4	-	-	-	-	-
fs	Tensile stress in reinforcement fs = Msls / (As*J.d)	Mpa	-	-	-	-	-
	Checking for control cracking fs<fsa		N.a	N.a	N.a	N.a	N.a
(5.10.8.2)	**Shrinkage and temperature Reinfocement (side distribution)**						
Areq	Area of required reinf	m2	0.00037	0.00030	0.00027	0.00027	0.00027
	Distribution on sides 8 D12	m2	0.00090	0.00090	0.00090	0.00090	0.00090
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45	0.45
	Checking		OK	OK	OK	OK	OK
	6.2 SHEAR FORCE CHECKING						
β	Factor indicating diag. cracked concr. to tension		6.8	6.8	6.8	6.8	6.8
θ	Angle of inclination of diagonal compressive	degree	27.00	27.00	27.00	27.00	27.00
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	90
bv	Effective web width as minimum web width - in dv	m	0.700	0.700	0.250	0.260	0.250
dv	Effective shear depth	m	1.296	1.296	1.406	1.512	1.548
	(de - a/2)	m	1.000	1.249	1.406	1.512	1.548
s	Spacing of stirrups	m	0.150	0.150	0.300	0.300	0.300
ncat	Amount of bars in spacing S	bars	2	2	2	2	2
Av	Shear reinf area in spacing S	m2	0.0004	0.0004	0.0004	0.0004	0.0004
β	Assume		6.8	6.7	6.8	5.9	5.5
θ	Assume	degree	27.00	27.00	27.00	27.00	27.00
v	Shear stress in concrete	kN/m2	1998	1577	3006	1823	850
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1435	1441	1454	1438	1381
e_x	Strain in tensile reinforcement		-5.31E-03	-3.24E-03	-2.19E-03	-1.76E-03	-1.55E-03
	if ex<0, multiple with reduce factor		-3.91E-04	-2.39E-04	-3.99E-04	-3.20E-04	-2.82E-04
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok	Ok
v/f'c	Ratio of shear stress and f'c		0.044	0.035	0.067	0.041	0.019
β	Final value		6.8	6.8	6.8	6.8	6.8
θ	Final value	degree	27.00	27.00	27.00	27.00	27.00
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	3425	3425	1327	1427	1461
Vs	Shear resistance provided by shear reinforcement	kN	2740	2740	1487	1599	1636
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	6165	6165	2814	3026	3097
Vn2	Vn2	kN	10206	10206	3955	4254	4353
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	6165	6165	2814	3026	3097
Vr	Factored shear resistance	kN	**5548**	**5548**	**2532**	**2724**	**2787**
Vu	Shear	kN	1631	1288	951	620	296
(5.8.2.7)	**Shear checking**		OK	OK	OK	OK	OK
	Region requiring transverse reinf Checking		Need	No need	Need	No need	No need
	Minimum shear reinf area	m2	0.0001	0.0001	0.0001	0.0001	0.0001
	Mimimum shear reinforcement Checking		OK	-	OK	-	-
	0.1*f'c*bv*dv	kN	4082	4082	1582	1701	1741
	Smax	m	0.60	0.60	0.60	0.60	0.60
	Maximum spacing Smax		OK	-	OK	-	-

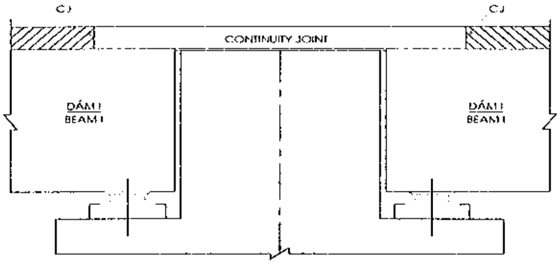
CALCULATION SHEET

LINK SLAB

	DA NANG - QUANG NGAI EXPRESSWAY PROJECT	Item.	Eng.	Date.	Sign.
	TYPICAL CALCULATION	Design			
	DETAIL DESIGN	Check			
	CONTINUITY JOINT,I40 GIRDER	Revise			

22TCN272-05; AASHTO LRFD 2nd - 1998

I GENERAL DATA



1.1.Dimension data

LEFT SPAN				RIGHT SPAN			
ITEM	SYMBOL	VALUES	UNITS	ITEM	SYMBOL	VALUES	UNITS
Length of left span	Ll	40	m	Length of right span	Lr	40	m
Caculation length	Ltl	39.1	m	Caculation length	Ltr	39.1	m
Depth of pavement	t	0.084	m	Depth of pavement	t	0.084	m
Depth of deck slab	d	0.24	m	Depth of deck slab	d	0.24	m
Bridge width	B	12	m	Bridge with	B	12	m
Lane width	B1	11	m	Lane with	B1	11	m
Parapet width	B2	0.5	m	Parapet with	B2	0.5	m
Number of lane	Nl	3		Number of lane	Nl	3	
Muntiple lane factor	m	0.85		Muntiple lane factor	m	0.85	

1.2.Material data

Compressive Strength at 28 days of deck slab concrete	fc' =	30	Mpa
Modulus of elasticity	Ec =	29440	Mpa
Modulus of Rupture	fr =	3.45	Mpa
Unit weight of concrete	gc=	2500	kg/m3
Compressive Strength at 28 days of beam concrete	f'c=	40	Mpa

1.3.Self weight of superstructere

LEFT SPAN				RIGHT SPAN			
ITEM	SYMBOL	VALUES	UNITS	ITEM	SYMBOL	VALUES	UNITS
Number of girder	nl	5		Number of girder	nr	5	
Weight of girder	Gdc	1056.24	KN	Weight of girder	Gdc	1056.24	KN
Unit weight of parapet	Gdw	13.7	KN/m	Unit weight of parapet	Gdw	13.7	KN/m

II CACULATE INTERNAL FORCES

2.1.Internal forces due to angular displacement

LEFT SPAN				RIGHT SPAN			
ITEM	SYMBOL	VALUES	UNITS	ITEM	SYMBOL	VALUES	UNITS
Dead load	DC	103.5212	KN/m	Dead load	DC	103.5212	KN/m
Live load	LL	54.79	KN/m	Live load	LL	54.79	KN/m
Uniform load	p	158.31	KN/m	Uniform load	p	158.31	KN/m
Coefficient of space	η	1.00		Coefficient of space	η	1.00	
Modulus of elasticity	E	33994.48	Mpa	Modulus of elasticity	E	33994.48	Mpa
Equivalent moment of inertia	J	2.19	m4	Equivalent moment of inerti	J	2.19	m4
Angular displacement	φ	0.005303506	rad	Angular displacement	φ	0.00530351	rad

Calculated length of continuity joint	Lb=	4	m
Reduction facteur of rigidity	k	0.8	
Equivalent moment of inertia	Jb	0.013824	m4
Modulus of elasticity	Eb	29440	Mpa
Bending moment of restrained section	Mn	-863.36785	KN.m
Shear force at restrained section	Qn	0	KN
Bending moment of middle section	Mg	0	KN.m

2.2.Internal forces due to slope of bridge

Weight of superstructure	W	12456.04	KN
Longitudinal grade	i	0.560%	
Number of superstructure	n	1	
	Nd	69.753824	KN

2.3.Internal forces due to live loads

Dynamic Load Allowance	1+IM	1.75	
Weight of wheel load	P	369.75	KN
Uniform load of live load	qLL	23.715	KN/m
Bending moment of restrained section	Mn	-296.769	KN.m
Shear force at restrained section	Q	361.47525	KN
Bending moment of middle section	Mg	296.769	KN.m
Shear force at middle section	Qg	323.53125	KN

2.4.Internal forces due to self weight

Uniform load	qS	70.56	KN/m

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Bending moment of restrained section	Mn	-75.26 KN.m
Shear force at restrained section	Qn	112.90 KN
Bending moment of middle section	Mg	56.45 KN.m

2.5.Internal forces due to pavement and parapet

Uniform load	qPP	34.49 KN/m
Bending moment of restrained section	Mn	-36.79 KN.m
Shear force at restrained section	Qn	55.18 KN
Bending moment of middle section	Mg	27.59 KN.m

2.6.Internal forces due to Load of Temperature

Area of bearing	Fb	0.1925 m2
Number of bearing	n	5
Height of bearing	h	0.084 m
Friction coefficient of bearing		1
Shear modulus of bearing	Gp	981 KN/m2
Install temperature	t_{lapdat}	25.0 °C
Maximal temparature	t_{max}	47.0 °C
Minimal temparature	t_{min}	10.0 °C
Compressive force	N =	84.30 kN
Tensile force	N =	123.65 kN
Number of bearing that acommondates displacement		5

III TABLE SUMMARY FORCES

Loads

ITEM	RESTRAINED SECTION		MIDDLE SECTION		COMP. FORCE	TENSILE FORCE
	M (KN.m)	Q (KN)	M (KN.m)	Q (KN)	KN	KN
Angular displacement	863.4	0.0	0.0	0.0		
Slope of bridge	0.0	0.0	0.0	0.0	69.8	
Live load	296.8	361.5	296.8	323.5		
Self weight of continuity joint	75.3	112.9	56.4	0.0		
Load of pavement and parapet	36.8	55.2	27.6	0.0		
Temperature					84.3	123.6
Total	**1272.2**	**529.6**	**296.8**	**323.5**	**154.1**	**123.6**

Load factors

LOADS	L.COMBINATION	
	Str-IA	Ser-I
Angular displacement	1.25	1.00
Slope of bridge	1.25	1.00
Live load	1.75	1.00
Self weight of continuity joint	1.25	1.00
Load of pavement and parapet	1.50	1.00
Temperature	0.50	1.00

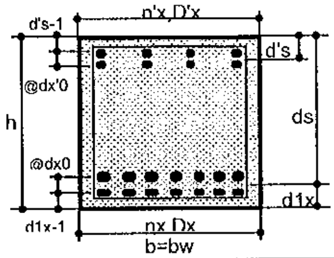
Load combinations

LOAD COMBINATIONS	RESTRAINED SECTION		MIDDLE SECTION		COMP. FORCE	TENSILE FORCE
	M (KN.m)	Q (KN)	M (KN.m)	Q (KN)	KN	KN
Strength Str-IA	*1747.8*	*856.5*	*631.3*	*566.2*	*129.3*	*61.8*
Service I	*1272.2*	*529.6*	*380.8*	*323.5*	*154.1*	*123.6*

REINFORCEMENTS checking

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
	Reinforcement		
fy	Yield strength	Mpa	420
Es	Modulus of Elasticity	Mpa	200000
nc	Ratio Es/Ec		7



			Restrained section		Middle section	
	Internal forces at section					
	Combination		Strength	Service	Strength	Service
Qu	Shear	kN	856	530	566	324
Mu	Flexual Moment	kNm	1748	1272	631	381
Nu	Axial load	kN	129	154	129	154
Tu	Torsional Moment	kNm	0	0	0	0
	flexural Moment checking					
H	Section height	m	0.240	0.240	0.240	0.240
d's	Dis. From comp. fiber to centroid of comp. Reinf	m	0.083	0.083	0.083	0.083
d1x	Dis. From tens. fiber to centroid of tension Reinf	m	0.058	0.058	0.058	0.058
	Cover to reinf	m	0.050	0.050	0.050	0.050
ds	Dis. From comp. fiber to centroid of tension Reinf	m	0.182	0.182	0.182	0.182
Rn	Mu / (j*ds^2)	kN/m2	58628.8	42674.3	21176.1	12773.9
b	Width of section	m	12.000	12.000	12.000	12.000
Amc	Section area	m2	2.880	2.880	2.880	2.880
	Chose					
As	Tension Reinforcement — Number	bars	78	78	78	78
	Diameter	mm	25	25	25	25
	Area	m2	0.03830	0.03830	0.03830	0.03830
A's	Compression Reinforcement — Number	bars	0	0	0	0
	Diameter	mm	25	25	25	25
	Area	m2	0.00000	0.00000	0.00000	0.00000
A'c	Shear reinforcement — Number	bars	20	20	20	20
	Diameter	mm	12	12	12	12
	Area	m2	0.00226	0.00226	0.00226	0.00226
j	Resistance factors for flexure	5.5.4.2	0.90	0.90	0.90	0.90
jv	Resistance factors for shear		0.90	0.90	0.90	0.90
b1	Stress block factor		0.836	0.836	0.836	0.836
c	Dis. Between centroid and top fiber	m	0.063	0.063	0.063	0.063
a	Depth of equivalent stress block	m	0.053	0.053	0.053	0.053
de	Corresp. effective depth from extreme comp. fiber to centroid of tensile force in the tensile reinf.	m	0.182	0.182	0.182	0.182
Mn	Nominal resistance	kNm	2505	2505	2505	2505
Mr	Factored reistance	kNm	2254	2254	2254	2254
Mu	Flexual moment	kNm	1748	1272	631	381
(5.7.3.2)	**Flexural moment Checking**		**OK**	**OK**	**OK**	**OK**
	Limits for reinforcement	m				
c/de	Maximum reinforcement		0.35	0.35	0.35	0.35
	Maximum reinforcement Checking	<= 0.42	**OK**	**OK**	**OK**	**OK**
r min	Mimimum reinforcement		1.33%	1.33%	1.33%	1.33%
	Mimimum reinforcement Checking	0.21%	**OK**	**OK**	**OK**	**OK**
1.2*Mcr	Craking moment	kNm	477	477	477	477
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**		**OK**	**OK**	**OK**	**OK**
(5.7.3.4)	**Conctrol of craking by distr. of reinf - check?**		No	Yes	No	Yes
	Existing condition for structrure	1,2 or 3	1	1	1	1

	DA NANG - QUANG NGAI EXPRESSWAY PROJECT	Item.	Eng.	Date.	Sign.
	TYPICAL CALCULATION	Design			
	DETAIL DESIGN	Check			
	CHECK SECTION OF CONTINUITY JOINT	Revise			

dc	Concr. thickness fro. Tens. fiber to tens. reinf nea.	m	0.050	0.050	0.050	0.050	
Z	Crack width parameter	N/mm	30000	30000	30000	30000	
A	Area of concr. with same centroid as tens. reinf	m2	0.015	0.015	0.015	0.015	
fsa	Value	Mpa	327	327	327	327	
0.6*fy		Mpa	252	252	252	252	
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	252	252	252	252	
x	Depth of compr. area - Assu. A's in compr. Area	m	-	0.071	-	0.071	
	so assumtion is:		-	x<d's	-	x<d's	
Icr	Moment of inertia of the cracked section	m4	-	0.005	-	0.005	
fs	So tensil stress in reinf	Mpa	-	181	-	35	
	Checking for control cracking fs<fsa		**N.a**	**OK**	**N.a**	**OK**	
(5.10.8.2)	**Shrinkage and temperature Reinfocement**						
Areq	Area of required reinf	m2	0.00023	0.00023	0.00023	0.00023	
	Distribute reinf 7 D16	m2	0.00141	0.00141	0.00141	0.00141	
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45	
	Checking		**OK**	**OK**	**OK**	**OK**	

Shear and torsion checking

b	Factor indicating diag. cracked concr. to tension		2.0	2.0	2.0	2.0	
q	Angle of inclination of diagonal compressive	degree	45.00	45.00	45.00	45.00	
a	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	12.000	12.000	12.000	12.000	
dv	Effective shear depth	m	0.173	0.173	0.173	0.173	
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	
nc¾t	Amount of bars in spacing S	bars	20	20	20	20	
Av	Shear reinf area in spacing S	m2	0.0023	0.0023	0.0023	0.0023	
b	Assume		2.0	2.0	2.0	2.0	
q	Assume	degree	45.00	45.00	45.00	45.00	
v	Shear stress in concrete	kN/m2	459	284	303	173	
e_x	Strain in tensile reinforcement		1.37E-03	9.86E-04	5.05E-04	2.99E-04	
	if ex<0, multiple with reduce factor		-	-	-	-	
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok	
v/f'c	Ratio of shear stress and f'c		0.015	0.009	0.010	0.006	
b	Final value		2.0	2.0	2.0	2.0	
q	Final value	degree	45.00	45.00	45.00	45.00	
Vc	Nominal shear resistance in the concrete	kN	1885	1885	1885	1885	
Vs	Shear resistance prov. by shear reinfocement	kN	274	274	274	274	
Vp	Component in direct. of applied shear of prestres.	kN	0	0	0	0	
Vn1	Vn1=Vc+Vs+Vp	kN	2159	2159	2159	2159	
Vn2	Vn2	kN	15552	15552	15552	15552	
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	2159	2159	2159	2159	
Vr	Factored shear resistance	kN	1943	1943	1943	1943	
Vu	Shear	kN	856	530	566	324	
(5.8.2.7)	**Shear checking**		**OK**	**OK**	**OK**	**OK**	

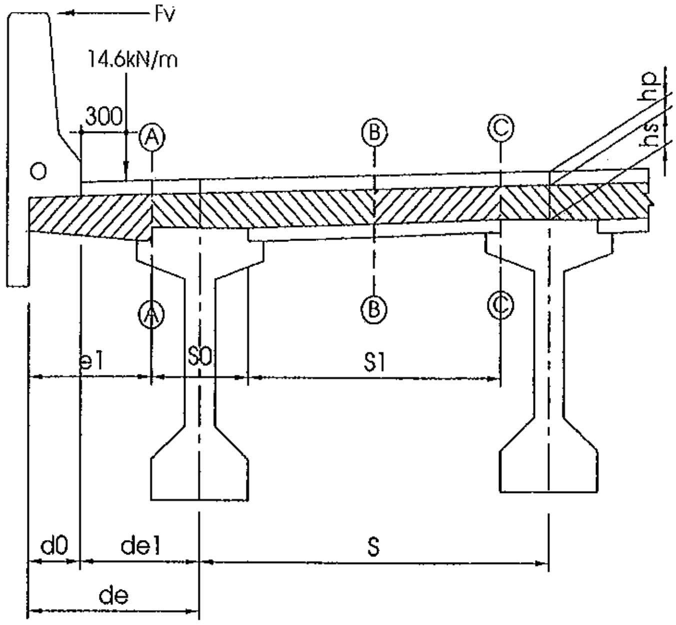
CALCULATION SHEET

DECK SLAB

CANTILEVER DECK SLAB

1. Cantilever design

Sign	Unit	Value
de	m	1.170
de1	m	0.770
d0	m	0.400
e1	m	0.870
S	m	2.550
S0	m	0.600
S1	m	1.950
hp	m	0.084
hs	m	0.207
gc	kN/m3	24.50
gas	kN/m3	22.50



Live load on cantilver S3.6.1.3.4

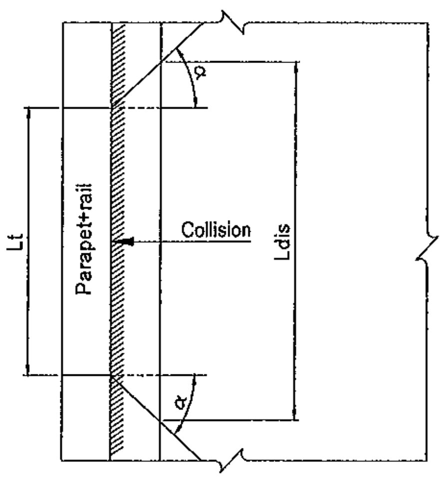
For the design of deck overhangs with a cantilever, not exceeding 1800 mm from the centerline of the exterior girder to the face of a structurally continuous concrete railing, the outside row of wheel loads may be replaced with a uniformly distributed line load of 14.6 N/mm intensity, located 300 mm from the face of the railing.

Lane load	Plan =	3.1 kN/m2
Pedestrian load	Pdb =	0.0 kN/m2

Collision force to barries

Test Levels		L5
Horziontal force	Ft =	550 kN
Distributed length	Lt =	2.44 m
Vertical force	Fv =	355 kN
Horizontal length	Lv =	12.2 m
Height of collision force	He =	1.07 m

Distributed length of collision force in plan / 1m length of deck	Ldis =	2.983 m
	α =	30 (degree)



Cantilever Loads

Dead load of curb&handrail	Plc =	13.7 kN/m	
Dead load of pavement	Plp =	0.9 kN/m	
Design section for overhang slab from centerline of exterior girder	Lse =	0.300 m	<=0.38m

Loads		Amc	Li	ei	N	Q	M
		(m2)	(m)	(m)	(kN/m)	(kN/m)	(kNm/m)
Deck slab	sec1	0.18	1.00	0.435		4.4	1.9
	sec2	0.00	1.00	0.000		0.0	0.0
Pavement		0.04	1.00	0.235		0.9	0.2
Curb&handrail			1.00	0.870		13.7	11.9
Pedestrian			1.00	0.000		0.0	0.0
Lane load			1.00	0.235		1.5	0.3
Vehicle wheel	14.6kN/m		1.00	0.170		14.6	2.5
Horizontal collision			1.00	1.070	68.0		72.8
Vertical collision			1.00	0.470		9.8	4.6

(Li: longitudinal calculation length, ei: distance from load to design section)

Modifcation factor $\eta = \eta_D*\eta_R*\eta_I$

Factor relating to ductility	$\eta_D =$	1.00
Factor relating to redundancy	$\eta_R =$	1.00
Factor relating to operational	$\eta_I =$	1.00
Modification factor	$\eta =$	1.00

Loads		Combinations		
		Service	Strength	Extreme
Deck slab	sec1	1.00	1.25	1.25
	sec2	1.00	1.25	1.25
Pavement		1.00	1.50	1.50
Curb&handrail		1.00	1.25	1.25
Pedestrian		1.00	1.75	
Lane load		1.00	1.75	
Vehicle wheel		1.00	2.19	
Horizontal collision				1.00
Verical collision				1.00

Load combinations	N	Q	M
	(kN/m)	(kN/m)	(kNm/m)
Service	0.0	35.1	16.9
Strength	0.0	58.5	23.6
Extreme	68.0	33.7	95.0

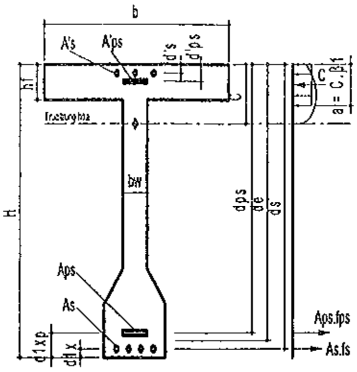
	DA NANG - QUANG NGAI EXPRESSWAY PROJECT	Item.	Eng.	Date.	Sign.
	PKG6 - TYPICAL CALCULATION	Design			
	DETAIL DESIGN	Check			
	DECK SLAB - SPACING OF GIRDERS, S=2.55M	Revise			

22TCN272-05; AASHTO LRFD 2nd - 1998

REINFORCEMENT CHECKING - CANTILEVER SLAB CHECK

MATERIALS

	NORMAL CONCRETE		
f'c	Compressive Strength of concrete at 28 days	Mpa	35
Ec	Modulus of Elasticity	Mpa	29910
fr	Modulus of Rupture	Mpa	3.7
gc	Unit weight of concrete	kN/m3	24.5
	PRESTRESSING STEEL		
fpu	Tensile strength of prestressing steel	Mpa	1860
fpy	Yield strength of prestressing steel	Mpa	1670
Ep	Modulus of Elasticity	Mpa	197000
	REINFORCEMENT		
fy	Yield strength	Mpa	400
Es	Modulus of Elasticity	Mpa	200000
nc	Ratio Es/Ec		7

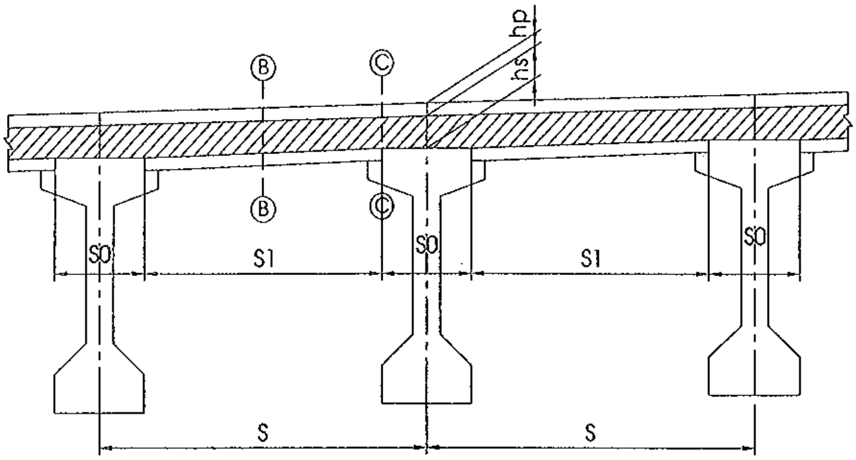


Sign	Parameters		Unit	Section - CANTILEVER SLAB				
				A-A	A-A	A-A		
	INTERNAL FORCES AT SECTIOIN							
	Combination			Strength	Service	Extreme		
Qu	Shear		kN	58	35	34		
Mu	Flexural Moment		kNm	24	17	95		
Nu	Axial load		kN	0	0	68		
Tu	Torsional Moment		kNm					
	FLEXURAL MOMENT CHECKING							
H	Section height		m	0.207	0.207	0.207		
d's	Dis. From comp. fiber to centroid of comp. Reinf		m	0.077	0.077	0.077		
d1x	Dis. From tens. fiber to centroid of tension Reinf		m	0.052	0.052	0.052		
	Cover to reinf		m	0.050	0.050	0.050		
ds	Dis. From comp. fiber to centroid of tension Reinf		m	0.155	0.155	0.155		
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	0.207	0.207	0.207		
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	0.001	0.001	0.001		
Amc	Section area		m2	0.207	0.207	0.207		
	Steel choice							
Aps	Tension prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0		
		Number	tendons	0	0	0		
		Area	m2	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	m2	0.00000	0.00000	0.00000		
As	Tension Reinforcement	Number	bars	6	6	6		
		Diameter	mm	20	20	20		
		Area	m2	0.00177	0.00177	0.00177		
A's	Compression Reinforcement	Number	bars	0	0	0		
		Diameter	mm	20	20	20		
		Area	m2	0.00000	0.00000	0.00000		
A'c	Shear reinforcement	Number	bars	1	1	1		
		Diameter	mm	14	14	14		
		Area	m2	0.00015	0.00015	0.00015		
ϕ	Resistance factors for flexure		5.5.4.2	0.90	1.00	1.00		
ϕv	Resistance factors for shear			0.90	1.00	1.00		
ϕn	Resistance factors for axial force			1.00	1.00	1.00		
$\beta 1$	Stress block factor			0.800	0.800	0.800		
c	Dis. Between centroid and top fiber		m	0.030	0.030	0.030		
	For T section behavior		m	0.030	0.030	0.030		
	For rectangular section behavior		m	0.030	0.030	0.030		
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 resistanc		Mpa	1784	1784	1784		
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	0.024	0.024	0.024		
de	Corresp. effective depth from extreme comp. fiber to centroid of tensile force in the tensile reinf.	m	0.155	0.155	0.155		
Mn	Nominal resistance	kNm	101	101	101		
Mr	Factored reistance	kNm	91	101	101		
Mu	Flexual moment	kNm	24	17	95		
(5.7.3.2)	**Flexural moment Checking**		OK	OK	OK		
	Limits for reinforcement						
c/de	Maximum reinforcement		0.19	0.19	0.19		
	Maximum reinforcement Checking	<= 0.42	OK	OK	OK		
r min	Mimimum reinforcement		0.86%	0.86%	0.86%		
	Mimimum reinforcement Checking for RC	0.26%	OK	OK	OK		
1.2*Mcr	Craking moment	kNm	19	19	19		
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**		OK	OK	OK		
(5.8.3.5)	Tensile force in steel should be satisfied - F_yc	kN	234	141	697		
	Checking As.fy+Aps.fps >= F_yc		Ok	Ok	Ok		
(5.7.3.4)	**Conctrol of craking by distr. of reinf for RC member- Check?**		No	Yes	No		
	Existing condition for structrure	1,2 or 3	1	1	1		
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.052	0.052	0.052		
Z	Crack width parameter	N/mm	30000	30000	30000		
A	Area of concr. with same centroid as tens. Reinf	m2	0.017	0.017	0.017		
fsa	Value	Mpa	311	311	311		
0.6*fy		Mpa	240	240	240		
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	240	240	240		
x	Dist. From compression fiber to centroid	m	-	0.051	-		
J.d	Arm	m	-	0.138	-		
Icr	Moment of inertia of the cracked section	m4	-	0.00	-		
fs	Tensile stress in reinforcement fs = Msls / (As*J.d)	Mpa	-	69	-		
	Checking for control cracking fs<fsa		N.a	OK	N.a		
(5.10.8.2)	**Shrinkage and temperature Reinfocement (side distribution)**						
Areq	Area of required reinf	m2	0.00023	0.00023	0.00023		
	Distribution on sides 2 D14	m2	0.00030	0.00030	0.00030		
	Required Spacing not larger than	m	0.45	0.45	0.45		
	Checking		OK	OK	OK		

INNER DECK SLAB

1. Dead load



<S.4.6.2.1.6>

Distance between long. Girder	S =	2.55	m
Distance between two supports	So =	0.60	m
	S1 =	1.95	m

Deck slab

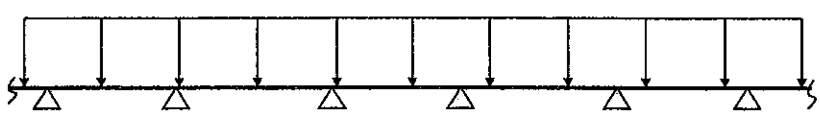
Unit weight of RC	gc =	24.5	kN/m3
Thichkness of Deckslab	t =	0.207	m
Deck strip for calculation	L =	1.0	m
Dead load of deckslab	Pds =	5.1	kN/m

Pavement

Density	gp =	22.5	kN/m3
Thichkness of Pavement	t =	0.084	m
Dead load of Pavement	Pws =	1.9	kN/m

Parapet

Dead load of Curb&handrail at outside	Ppa =	13.7	kN/m
Dead load of median strip	Pme =	10.0	kN/m



Momens due to dead load are calculated base on:

$M = q \cdot l^2 / c$

With c is constant, is taken as	for positive	c1 =	10.0
	for negative	c2 =	8.0

2. Live load

Liveload

Lane load for 1 lane	Pl =	9.3	kN/m
Combine with (1Truck or 1Tandem)			
Dynamic load allowance	1+IM =	1.25	

Using equivalent Strip S.4.6.2.1.3

Positive Moment

Width of primary strip for Wpo.strip = 660 + 0.55 . S

	S =	1950 mm
	Wpo.strip =	1733 mm
Look up the unfactored in table A4.1	M =	22510 Nmm/mm
Corrected factor positive moment (included IM)	Mf =	25336 Nmm/mm

Negative Moment

Width of primary strip for Wne.strip = 1220 + 0.25 . S

	Wne.strip =	1708 mm
Dist. from CL of girder to sec. of Neg.	e =	0.0 mm
Look up the unfactored in table A4.1	M =	23655 Nmm/mm
Corrected factor negative moment (included IM)	Mf =	27014 Nmm/mm

3. Combinations

Modifcation factor $\eta = \eta_D * \eta_R * \eta_I$

Factor relating to ductility	$\eta_D =$	1.00
Factor relating to redundancy	$\eta_R =$	1.00
Factor relating to operational	$\eta_I =$	1.00
Modification factor	$\eta =$	1.00

Load combinations and load factors

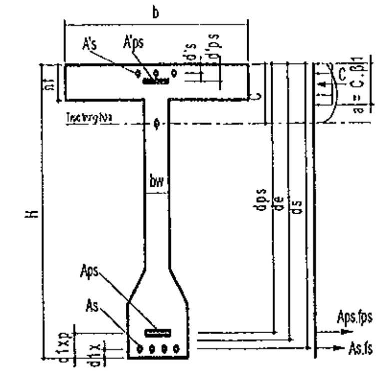
Loads	Sign	Combination	
		Strength	Service
Deckslab	DC	1.25	1.00
Pavement	DW	1.50	1.00
Parapet	DC	1.25	1.00
Liveload	LL	1.75	1.00

Loads (kNm/m)	Sign	Moment	
Unfactored load		Positive	Negative
Deckslab	DC	1.93	-2.41
Pavement	DW	0.72	-0.90
Parapet	DC	2.08	-2.60
Liveload	LL	25.34	-27.01

Combination	Moment	
(kNm/m)	Positive	Negative
Service	30.1	-32.9
Strength	50.4	-54.9

REINFORCEMENT CHECKING - INNER SLAB CHECK

MATERIALS



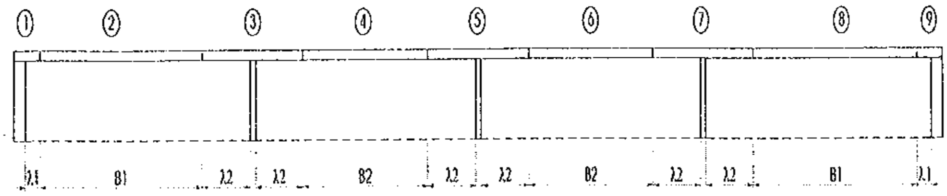
	NORMAL CONCRETE		
f'c	Compressive Strength of concrete at 28 days	Mpa	35
Ec	Modulus of Elasticity	Mpa	29910
fr	Modulus of Rupture	Mpa	3.7
gc	Unit weight of concrete	kN/m3	24.5
	PRESTRESSING STEEL		
fpu	Tensile strength of prestressing steel	Mpa	1860
fpy	Yield strength of prestressing steel	Mpa	1670
Ep	Modulus of Elasticity	Mpa	197000
	REINFORCEMENT		
fy	Yield strength	Mpa	400
Es	Modulus of Elasticity	Mpa	200000
nc	Ratio Es/Ec		7

Sign	Parameters		Unit	Section - INNER SLAB B-B	B-B	C-C	C-C	
	INTERNAL FORCES AT SECTIOIN							
	Combination			Strength	Service	Strength	Service	
Qu	Shear		kN	0	0	0	0	
Mu	Flexural Moment		kNm	50	30	55	33	
Nu	Axial load		kN					
Tu	Torsional Moment		kNm					
	FLEXURAL MOMENT CHECKING							
H	Section height		m	0.207	0.207	0.207	0.207	
d's	Dis. From comp. fiber to centroid of comp. Reinf		m	0.077	0.077	0.077	0.077	
d1x	Dis. From tens. fiber to centroid of tension Reinf		m	0.052	0.052	0.052	0.052	
	Cover to reinf		m	0.050	0.050	0.050	0.050	
ds	Dis. From comp. fiber to centroid of tension Reinf		m	0.155	0.155	0.155	0.155	
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	0.207	0.207	0.207	0.207	
b	Width of the compression face of member		m	1.000	1.000	1.000	1.000	
bw	Web width or diameter of a circular section		m	1.000	1.000	1.000	1.000	
hf	Compression flange depth		m	0.000	0.000	0.000	0.000	
Iz	Moment of inertia of section		m4	0.0007	0.0007	0.0007	0.0007	
Amc	Section area		m2	0.207	0.207	0.207	0.207	
	Steel choice							
Aps	Tension prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	
		Number	tendons	0	0	0	0	
		Area	m2	0.00000	0.00000	0.00000	0.00000	
A'ps	Compression prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	
		Number	tendons	0	0	0	0	
		Area	m2	0.00000	0.00000	0.00000	0.00000	
As	Tension Reinforcement	Number	bars	6	6	6	6	
		Diameter	mm	20	20	20	20	
		Area	m2	0.00177	0.00177	0.00177	0.00177	
A's	Compression Reinforcement	Number	bars	0	0	0	0	
		Diameter	mm	20	20	20	20	
		Area	m2	0.00000	0.00000	0.00000	0.00000	
A'c	Shear reinforcement	Number	bars	1	1	1	1	
		Diameter	mm	14	14	14	14	
		Area	m2	0.00015	0.00015	0.00015	0.00015	
ϕ	Resistance factors for flexure		5.5.4.2	0.90	1.00	0.90	1.00	
ϕv	Resistance factors for shear			0.90	1.00	0.90	1.00	
ϕn	Resistance factors for axial force			1.00	1.00	1.00	1.00	
$\beta 1$	Stress block factor			0.800	0.800	0.800	0.800	
c	Dis. Between centroid and top fiber		m	0.030	0.030	0.030	0.030	
	For T section behavior		m	0.030	0.030	0.030	0.030	
	For rectangular section behavior		m	0.030	0.030	0.030	0.030	
fpe	Effective stress in the prestressing steel after losses		Mpa	1116	1116	1116	1116	
fps	Aver. stress in pres. steel at the time for which the nominal resistanc		Mpa	1784	1784	1784	1784	
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	0.024	0.024	0.024	0.024	
de	Corresp. effective depth from extreme comp. fiber						
	to centroid of tensile force in the tensile reinf.	m	0.155	0.155	0.155	0.155	
Mn	Nominal resistance	kNm	101	101	101	101	
Mr	Factored reistance	kNm	91	101	91	101	
Mu	Flexual moment	kNm	50	30	55	33	
(5.7.3.2)	**Flexural moment Checking**		**OK**	**OK**	**OK**	**OK**	
	Limits for reinforcement						
c/de	Maximum reinforcement		0.19	0.19	0.19	0.19	
	Maximum reinforcement Checking	<= 0.42	**OK**	**OK**	**OK**	**OK**	
r min	Mimimum reinforcement		0.86%	0.86%	0.86%	0.86%	
	Mimimum reinforcement Checking for RC	0.26%	**OK**	**OK**	**OK**	**OK**	
1.2*Mcr	Craking moment	kNm	19	19	19	19	
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**		**OK**	**OK**	**OK**	**OK**	
(5.7.3.4)	**Conctrol of craking by distr. of reinf for RC member- Check?**		No	Yes	No	Yes	
	Existing condition for structrure	1,2 or 3	1	1	1	1	
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.052	0.052	0.052	0.052	
Z	Crack width parameter	N/mm	30000	30000	30000	30000	
A	Area of concr. with same centroid as tens. Reinf	m2	0.017	0.017	0.017	0.017	
fsa	Value	Mpa	311	311	311	311	
0.6*fy		Mpa	240	240	240	240	
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	240	240	240	240	
x	Dist. From compression fiber to centroid	m	-	0.051	-	0.051	
J.d	Arm	m	-	0.138	-	0.138	
Icr	Moment of inertia of the cracked section	m4	-	0.00	-	0.00	
fs	Tensile stress in reinforcement fs = Msls / (As*J.d)	Mpa	-	123	-	135	
	Checking for control cracking fs<fsa		**N.a**	**OK**	**N.a**	**OK**	
(5.10.8.2)	**Shrinkage and temperature Reinfocement (side distribution)**						
Areq	Area of required reinf	m2	0.00023	0.00023	0.00023	0.00023	
	Distribution on sides 2 D14	m2	0.00030	0.00030	0.00030	0.00030	
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45	
	Checking		**OK**	**OK**	**OK**	**OK**	

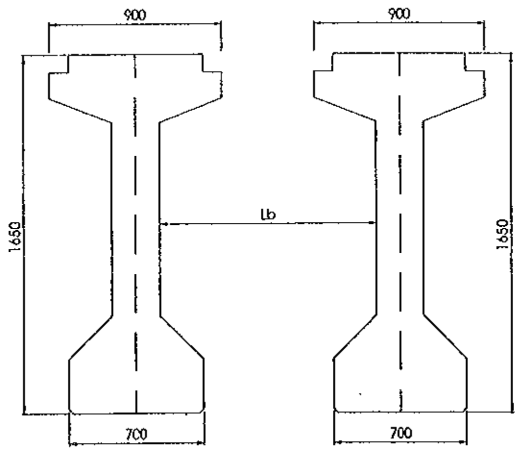
CALCULATION SHEET FOR DIAPHRAGM OF I33M GIRDER

MODELLING BY MIDAS SOFTWARE



I.Parameters :

1.Main girder:



Length of main girder	L=	33	m
Spacing of girder	S=	2.55	m
Width of web	bw=	0.25	m
Width of bottom girder	b=	0.7	m
Number of girder	n=	6	
Distance beetween edge of main girder	Lb=	2.3	m
Distance beetween End cross beam and Middle cross beam	L1=	7.8	m
Distance beetween MC and MC	L2=	7.8	m

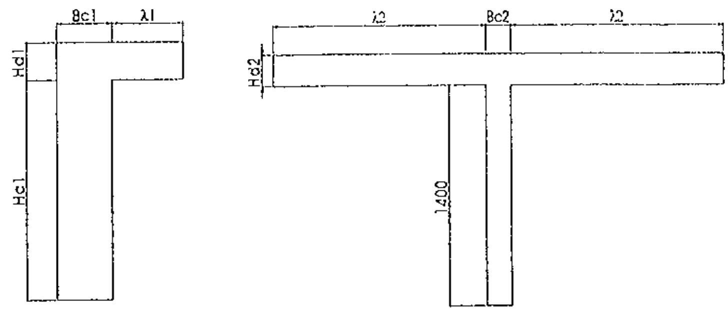
Distance along longitudinal direction:

λ_1=	S/8=	0.31875	m
λ_2=	(n-1)/6x(Lb+bw)=	1.7	m
	B1=	5.78125	m
	B2=	4.4	m

2. Cross Beam:

Head cross beam

Middle cross beam



Head cross beam	Bc1=	400 mm	Hd1=	240 mm
	Hc1=	1400 mm		
Middle Cross beam	Bc2=	200 mm	Hd2=	200 mm
	Hc2=	1400 mm		

	Head cross beam	Middle Cross beam
Area(m2)	1.00E+00	7.44E-01
Asy(m2)	6.14E-01	2.82E-01
Asz(m2)	2.15E-01	5.37E-01
Ixx(m4)	1.39E-02	3.08E-02
Iyy(m4)	1.77E-01	1.81E-01
Izz(m4)	7.79E-01	2.97E-02
Cent:y(m)	1.8	2.64E-01
Cent:z(m)	1.276	8.98E-01

2.Loads in model: (According to ASSHTO LRFD standard)

+ Seft weight (SW): Calculated by midas software

+ Pavement (DW):

Carriageway width in bridge	w=	11.75 m
Pavement height	hp=	0.084 m
Unit weight of pavement	γp=	22.5 kN/m3
Load of pavement	DW=	4.44 kN/m

+ Precast plate (DC):

Precast plate width	b=	1.85 m
Precast plate height	hp=	0.08 m
Unit weight of concrete	γc=	24 kN/m3
Load of pavement	DC=	3.552 kN/m

+ Parapet (DC):

Load of parapet	DC=	4.80 kN/m

+ Liveload (LL):

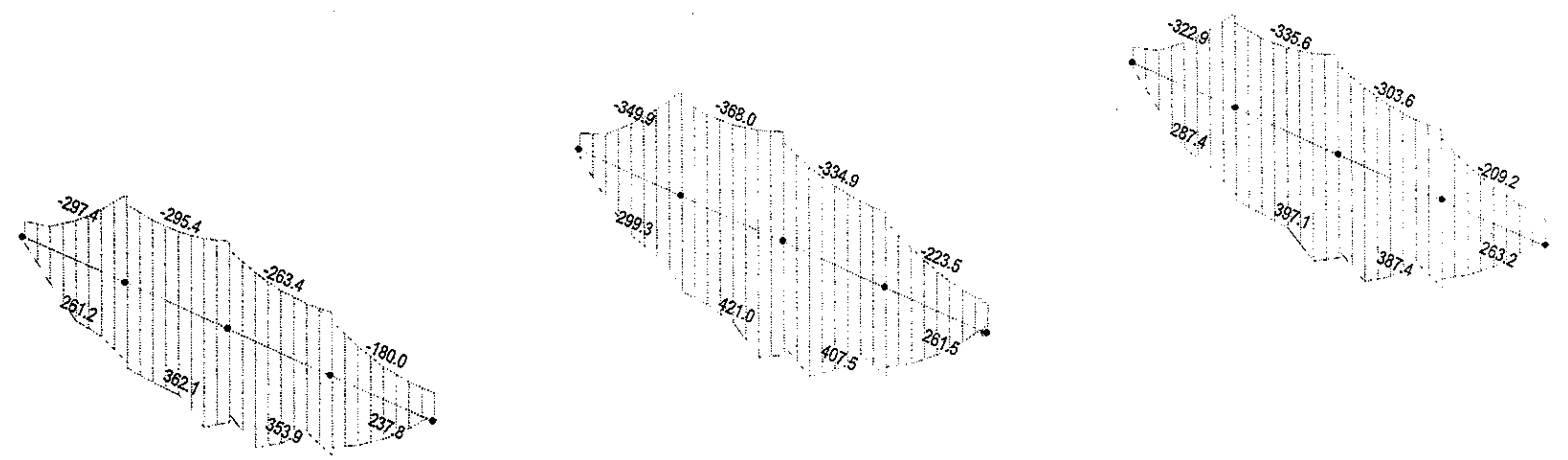
Lane load	PL =	9.3 kN/m
Combine with 1Truck	P1=	145 kN
	P2=	145 kN
	P3=	35 kN
Dynamic load allowance	1+IM =	1.25

Load combinations and load factors

Loads	Sign	Combination	
		Strength	Service
Precast plate	DC	1.25	1.00
Parapet	DW	1.25	1.00
Pavement	DC	1.50	1.00
Liveload	LL	1.75	1.00

RESULTS OF INTERNAL FORCE

MIDDLE DIAPHRAGM



MIDAS/Civil
POST-PROCESSOR

BEAM DIAGRAM

MOMENT-y

	4.21042e+002
J	3.49312e+002
I	2.77583e+002
H	2.05854e+002
G	1.34125e+002
F	6.23961e+001
E	0.00000e+000
D	-8.10620e+001
C	-1.52791e+002
B	-2.24520e+002
A	-2.96249e+002
	--3.67978e+002

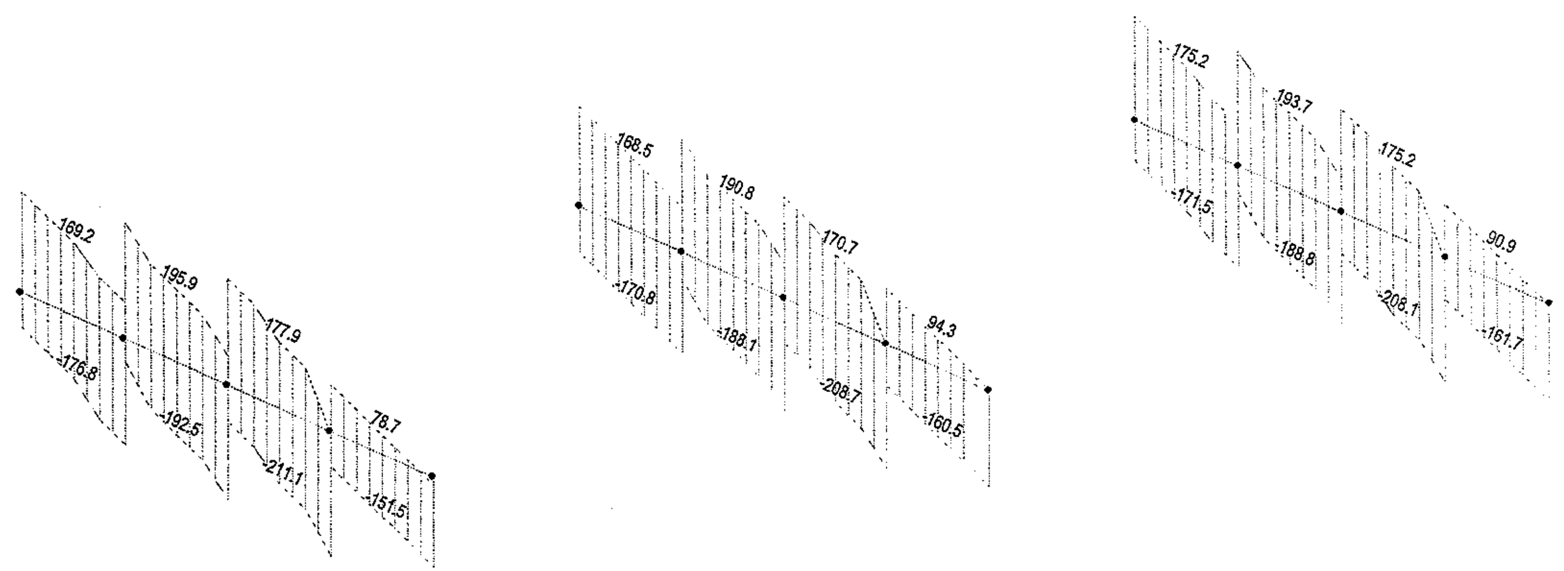
CBall: Streng3A

MAX : 1012
MIN : 1012

FILE: DIAPHRAGM(~
UNIT: kN·m
DATE: 12/15/2012

VIEW-DIRECTION

Z: 0.259



MIDAS/Civil
POST-PROCESSOR

BEAM DIAGRAM

SHEAR-z

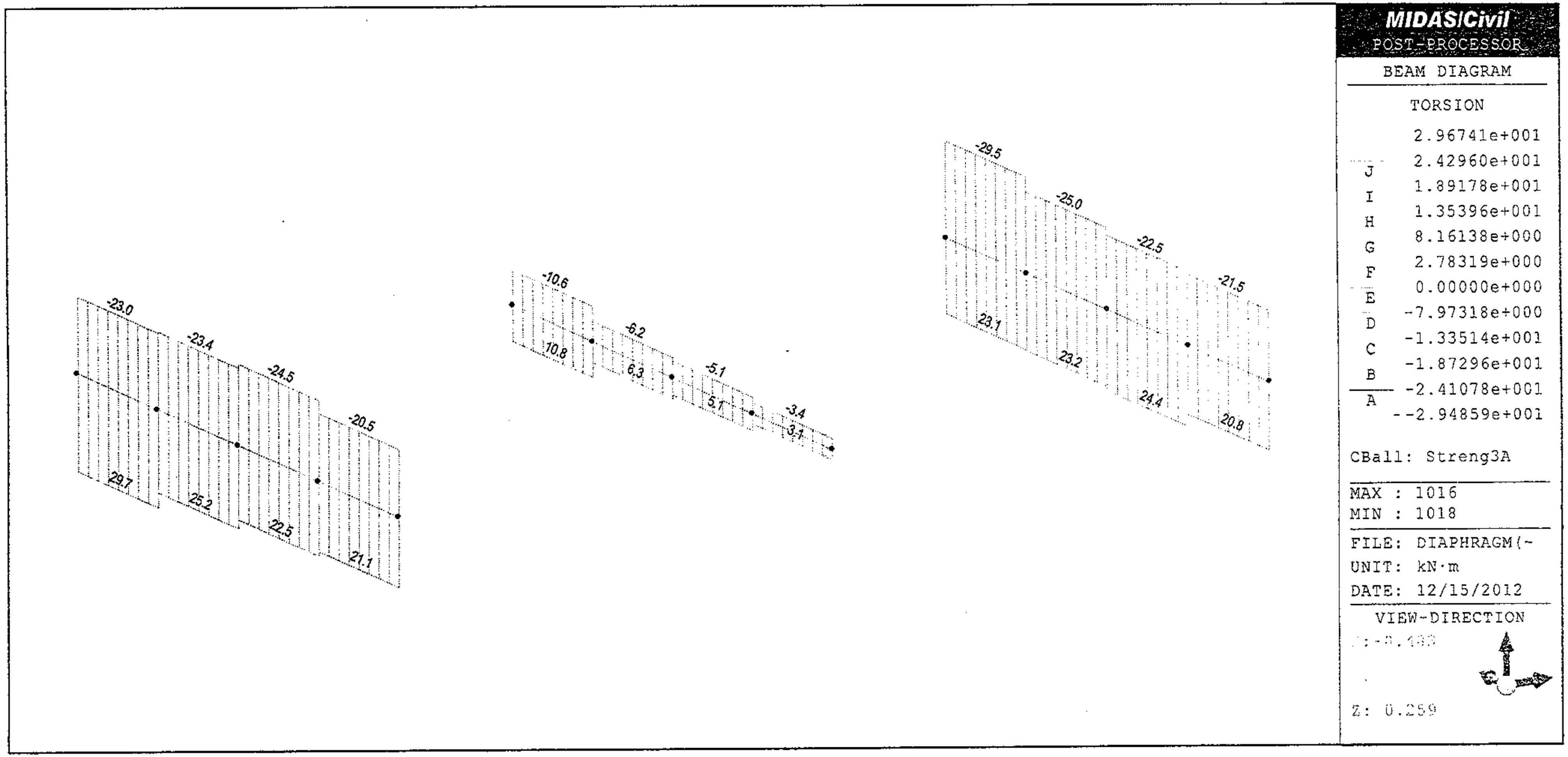
	1.95893e+002
J	1.58895e+002
I	1.21897e+002
H	8.48984e+001
G	4.79002e+001
F	0.00000e+000
E	-2.60963e+001
D	-6.30945e+001
C	-1.00093e+002
B	-1.37091e+002
A	-1.74089e+002
	-2.11087e+002

CBall: Streng3A

MAX : 1011
MIN : 1006

FILE: DIAPHRAGM(~
UNIT: kN
DATE: 12/15/2012

VIEW-DIRECTION
X: -0.483
Z: 0.259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25.2
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21.1
-10.6
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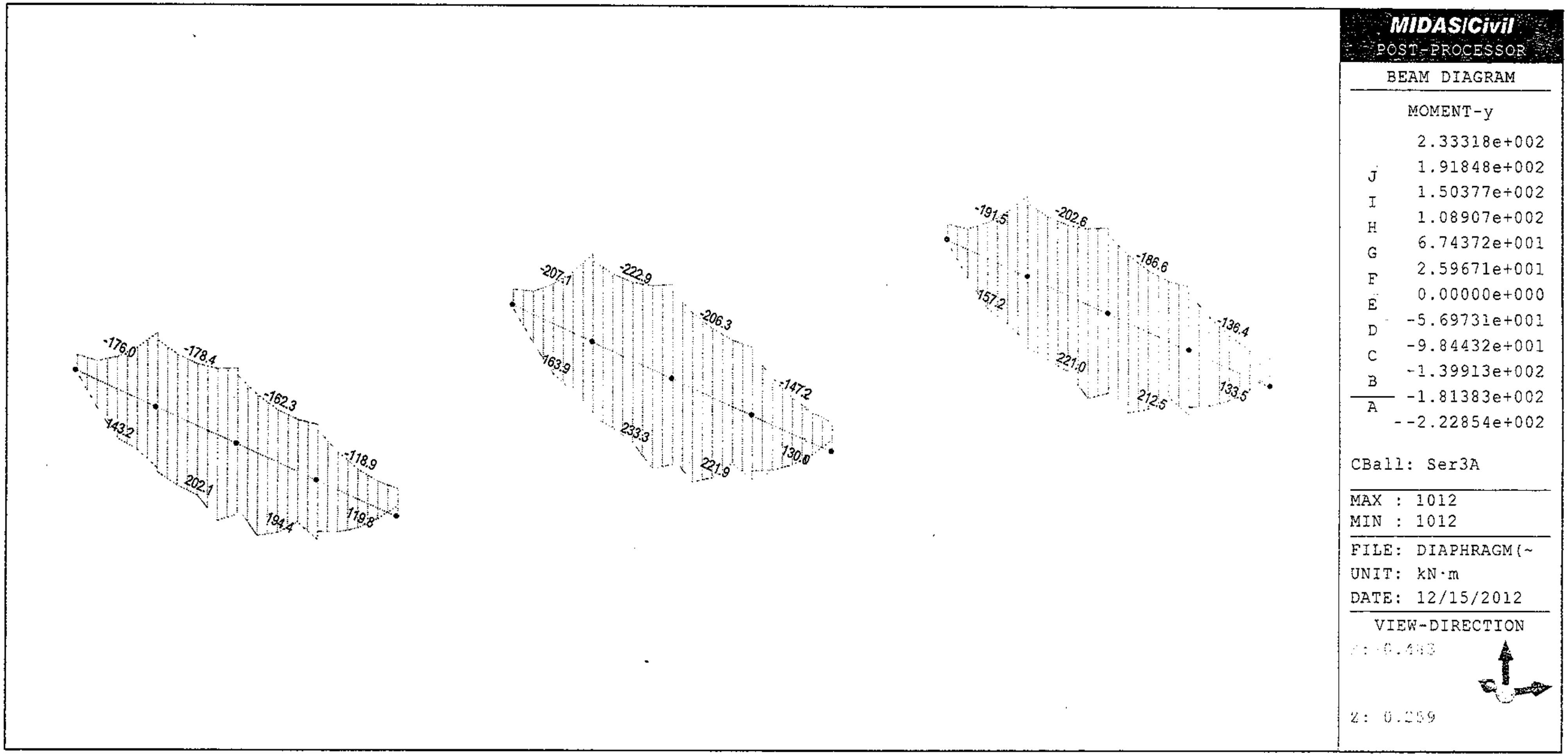
-3.4
10.8
6.3
5.1
3.1
-29.5
-25.0
-22.5
-21.5
23.1
23.2
24.4
20.8
MIDAS/Civil
POST-PROCESSOR
BEAM DIAGRAM
TORSION
2.96741e+001
J 2.42960e+001
I 1.89178e+001
H 1.35396e+001
G 8.16138e+000
F 2.78319e+000
E 0.00000e+000
D -7.97318e+000
C -1.33514e+001
B -1.87296e+001
A -2.41078e+001
-2.94859e+001
CBall: Streng3A
MAX : 1016
MIN : 1018
FILE: DIAPHRAGM(~
UNIT: kN·m
DATE: 12/15/2012
VIEW-DIRECTION
Z: 0.259


MIDAS/Civil
POST-PROCESSOR
BEAM DIAGRAM
MOMENT-y
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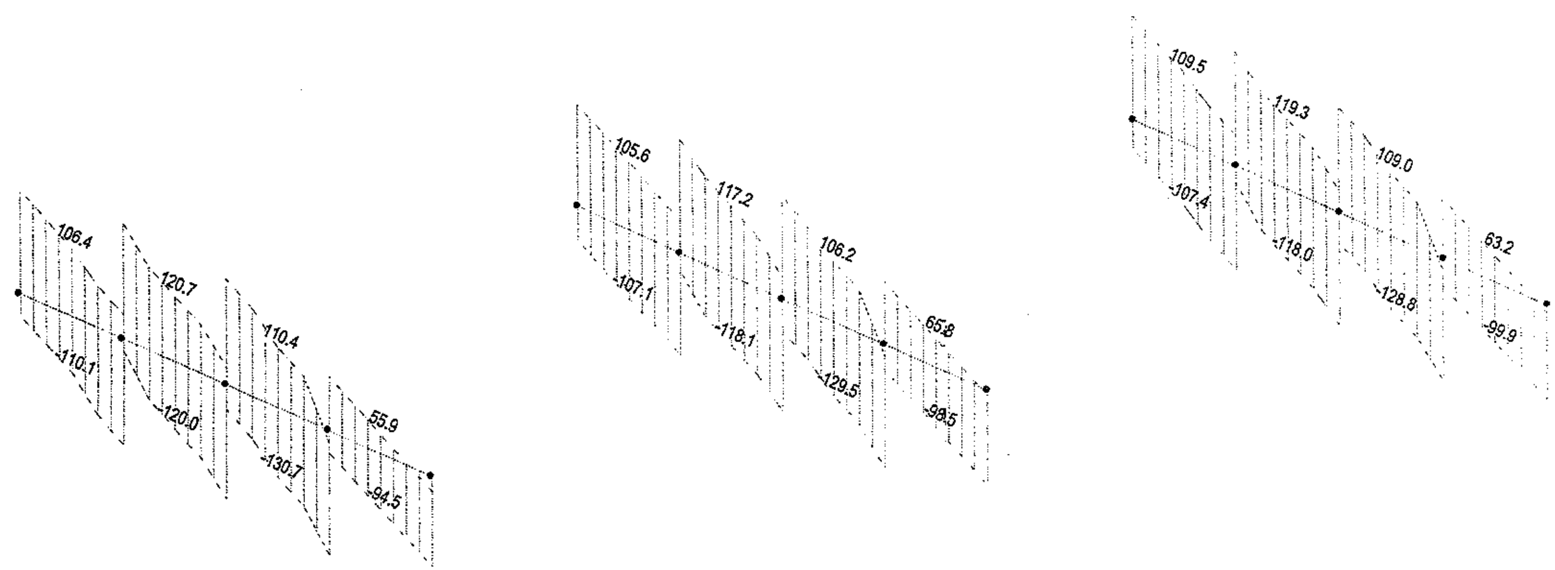
J 1.91848e+002
I 1.50377e+002
H 1.08907e+002
G 6.74372e+001
F 2.59671e+001
E 0.00000e+000
D -5.69731e+001
C -9.84432e+001
B -1.39913e+002
A -1.81383e+002
-2.22854e+002
CBall: Ser3A
MAX : 1012
MIN : 1012
FILE: DIAPHRAGM(~
UNIT: kN·m
DATE: 12/15/2012
VIEW-DIRECTION
-176.0
-178.4
-162.3
-118.9
143.2
202.1
194.4
119.8
-207.1
-222.9
-206.3
-147.2
163.9
233.3
221.9
130.0
-191.5
-202.6
-186.6
-136.4
157.2
221.0
212.5
133.5



MIDAS/Civil
POST-PROCESSOR

BEAM DIAGRAM

SHEAR-z

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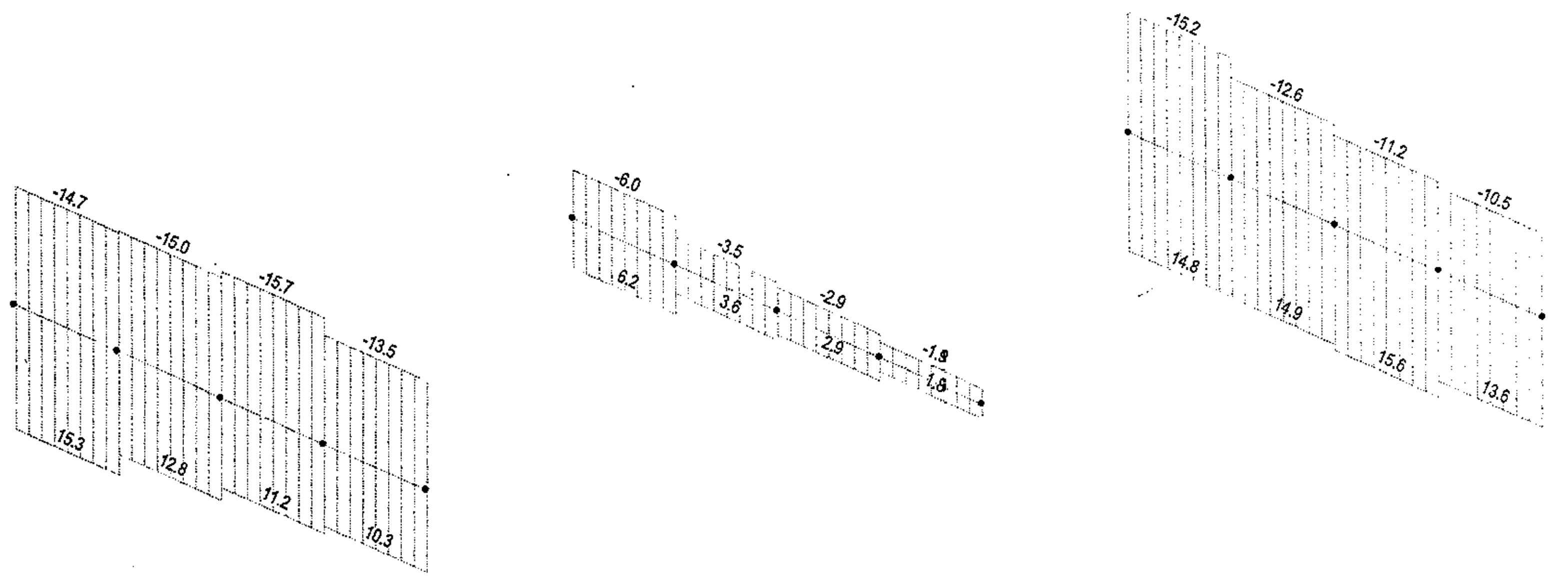
I	7.50249e+001
H	5.21690e+001
G	2.93130e+001
F	0.00000e+000
E	-1.63990e+001
D	-3.92550e+001
C	-6.21109e+001
B	-8.49669e+001
A	-1.07823e+002
	-1.30679e+002

CBall: Ser3A

MAX : 1011
MIN : 1006

FILE: DIAPHRAGM(~
UNIT: kN
DATE: 12/15/2012

VIEW-DIRECTION
Y: -0.483
Z: 0.259



MIDAS/Civil
POST-PROCESSOR

BEAM DIAGRAM

TORSION

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J	1.27948e+001
I	9.94515e+000
H	7.09549e+000
G	4.24583e+000
F	0.00000e+000
E	-1.45350e+000
D	-4.30316e+000
C	-7.15282e+000
B	-1.00025e+001
A	-1.28521e+001
	-1.57018e+001

CBall: Ser3A

MAX : 1008
MIN : 1006

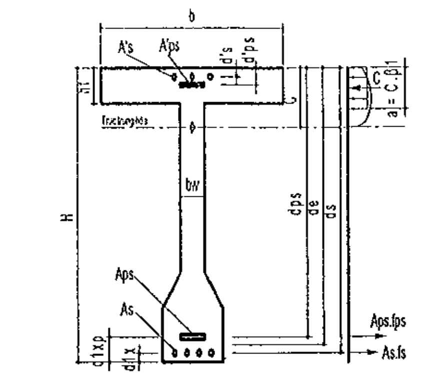
FILE: DIAPHRAGM(~
UNIT: kN·m
DATE: 12/15/2012

VIEW-DIRECTION
X:-0.4e3
Z: 0.259

REINFORCEMENT CHECKING - MIDLE DIAPHRAGM

MATERIALS

	NORMAL CONCRETE		
f'c	Compressive Strength of concrete at 28 days	Mpa	35
Ec	Modulus of Elasticity	Mpa	29910
fr	Modulus of Rupture	Mpa	3.7
gc	Unit weight of concrete	kN/m3	24.5
	PRESTRESSING STEEL		
fpu	Tensile strength of prestressing steel	Mpa	1860
fpy	Yield strength of prestressing steel	Mpa	1670
Ep	Modulus of Elasticity	Mpa	195000
	REINFORCEMENT		
fy	Yield strength	Mpa	400
Es	Modulus of Elasticity	Mpa	200000
nc	Ratio Es/Ec		7



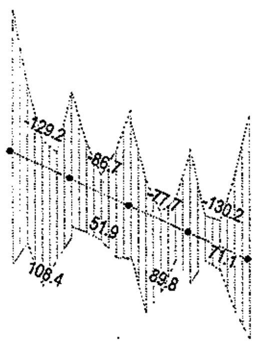
Sign	Parameters		Unit	Section - Middle Diaphragm			
				Pos.	Pos.	Neg.	Neg.
	INTERNAL FORCES AT SECTIOIN						
	Combination			Strength	Service	Strength	Service
Qu	Shear		kN	195.89	120.73	211.08	130.67
Mu	Flexural Moment		kNm	421.10	233.31	367.90	222.85
Nu	Axial load		kN				
Tu	Torsional Moment		kNm	29.67	15.60	29.48	15.70
	FLEXURAL MOMENT CHECKING						
H	Section height		m	1.600	1.600	1.600	1.600
d's	Dis. From comp. fiber to centroid of comp. Reinf		m	0.074	0.074	0.078	0.078
d1x	Dis. From tens. fiber to centroid of tension Reinf		m	0.078	0.078	0.074	0.074
	Cover to reinf		m	0.050	0.050	0.050	0.050
ds	Dis. From comp. fiber to centroid of tension Reinf		m	1.522	1.522	1.526	1.526
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	1.600	1.600	1.600	1.600
b	Width of the compression face of member		m	0.200	0.200	0.200	0.200
bw	Web width or diameter of a circular section		m	0.200	0.200	0.200	0.200
hf	Compression flange depth		m	0.200	0.200	0.200	0.200
Iz	Moment of inertia of section		m4	0.0683	0.0683	0.0683	0.0683
Amc	Section area		m2	0.320	0.3200	0.3200	0.3200
	Steel choice						
Aps	Tension prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0
		Number	tendons	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000
A'ps	Compression prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0
		Number	tendons	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000
As	Tension Reinforcement	Number	bars	2	2	4	4
		Diameter	mm	**28**	**28**	**20**	**20**
		Area	m2	0.00116	0.00116	0.00118	0.00118
A's	Compression Reinforcement	Number	bars	0	0	0	0
		Diameter	mm	20	20	28	28
		Area	m2	0.00000	0.00000	0.00000	0.00000
A'c	Shear reinforcement	Number	bars	2	2	2	2
		Diameter	mm	14	14	14	14
		Area	m2	0.00030	0.00030	0.00030	0.00030
ϕ	Resistance factors for flexure		5.5.4.2	0.90	1.00	0.90	1.00
ϕv	Resistance factors for shear			0.90	1.00	0.90	1.00
ϕn	Resistance factors for axial force			1.00	1.00	1.00	1.00
β1	Stress block factor			0.800	0.800	0.800	0.800
c	Dis. Between centroid and top fiber		m	0.097	0.097	0.099	0.099
	For T section behavior		m	0.097	0.097	0.099	0.099
	For rectangular section behavior		m	0.097	0.097	0.099	0.099
fpe	Effective stress in the prestressing steel after losses		Mpa	0	0	0	1116
fps	Aver. stress in pres. steel at the time for which the nominal resistance		Mpa	0	0	0	1827
k	Factor depends on type of P.S, Low relaxation strand k = 0.28			0.00	0.00	0.00	0.28

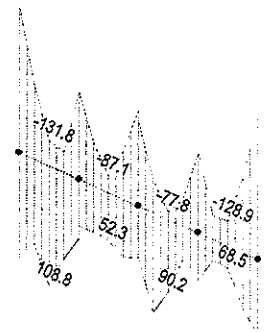
a	Depth of equivalent stress block	m	0.078	0.078	0.079	0.079	
de	Corresp. effective depth from extreme comp. fiber to centroid of tensile force in the tensile reinf.	m	1.522	1.522	1.526	1.526	
Mn	Nominal resistance	kNm	687	687	702	702	
Mr	Factored reistance	kNm	618	687	632	702	
Mu	Flexual moment	kNm	421	233	368	223	
(5.7.3.2)	**Flexural moment Checking**		**OK**	**OK**	**OK**	**OK**	
	Limits for reinforcement						
c/de	Maximum reinforcement		0.06	0.06	0.06	0.06	
	Maximum reinforcement Checking	<= 0.42	**OK**	**OK**	**OK**	**OK**	
r min	Mimimum reinforcement		0.36%	0.36%	0.37%	0.37%	
	Mimimum reinforcement Checking for RC	0.26%	**OK**	**OK**	**OK**	**OK**	
1.2*Mcr	Craking moment	kNm	203	203	203	203	
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**		**OK**	**OK**	**OK**	**OK**	
(5.7.3.4)	**Conctrol of craking by distr. of reinf for RC member- Check?**		No	Yes	No	Yes	
	Existing condition for structrure	1,2 or 3	1	1	1	1	
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.064	0.064	0.060	0.060	
Z	Crack width parameter	N/mm	30000	30000	30000	30000	
A	Area of concr. with same centroid as tens. Reinf	m2	0.013	0.013	0.006	0.006	
fsa	Value	Mpa	321	321	422	422	
0.6*fy		Mpa	240	240	240	240	
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	240	240	240	240	
x	Dist. From compression fiber to centroid	m	-	0.313	-	0.316	
J.d	Arm	m	-	1.418	-	1.421	
Icr	Moment of inertia of the cracked section	m4	-	0.014	-	0.014	
fs	Tensile stress in reinforcement fs = Msls / (As*J.d)	Mpa	-	142	-	133	
	Checking for control cracking fs<fsa		**N.a**	**OK**	**N.a**	**OK**	
(5.10.8.2)	**Shrinkage and temperature Reinfocement (side distribution)**						
Areq	Area of required reinf	m2	0.00023	0.00023	0.00023	0.00023	
	Distribution on sides 4 D16	m2	0.00081	0.00081	0.00081	0.00081	
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45	
	Checking		**OK**	**OK**	**OK**	**OK**	
	SHEAR AND TORSION CHECKING						
β	Factor indicating diag. cracked concr. to tension		1.9	2.2	1.9	2.2	
θ	**Angle of inclination of diagonal compressive**	**degree**	**41.80**	**36.35**	**41.24**	**36.16**	
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	0.200	0.200	0.200	0.200	
dv	Effective shear depth	m	1.483	1.483	1.486	1.486	
	(de - a/2)	m	1.483	1.483	1.486	1.486	
s	Spacing of stirrups	m	0.150	0.150	0.150	0.150	
ncat	Amount of bars in spacing S	bars	2	2	2	2	
Av	Shear reinf area in spacing S	m2	0.0003	0.0003	0.0003	0.0003	
β	Assume		1.9	2.2	1.9	2.2	
θ	Assume	degree	41.71	36.23	41.16	36.0	
v	Shear stress in concrete	kN/m2	734	407	789	440	
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	0	0	0	1116	
e_x	Strain in tensile reinforcement		1.70E-03	1.04E-03	1.56E-03	1.02E-03	
	if ex<0, multiple with reduce factor		-	-	-	-	
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok	
v/f'c	Ratio of shear stress and f'c		0.021	0.012	0.023	0.013	
β	Final value		1.9	2.2	1.9	2.2	
θ	Final value	degree	41.80	36.35	41.24	36.2	
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	271	322	281	324	
Vs	Shear resistance provided by shear reinforcement	kN	1336	1623	1365	1638	
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	1606	1945	1646	1962	
Vn2	Vn2	kN	2595	2595	2601	2601	
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	1606	1945	1646	1962	
Vr	Factored shear resistance	kN	1446	1945	1481	1962	
Vu	Shear	kN	196	121	211	131	
(5.8.2.7)	**Shear checking**		**OK**	**OK**	**OK**	**OK**	
	Region requiring transverse reinf Checking		Need	No need	Need	No need	
	Minimum shear reinf area	m2	0.0000	0.0000	0.0000	0.0000	
	Mimimum shear reinforcement Checking		**OK**	-	**OK**	-	
	0.1*f'c*bv*dv	kN	1038	1038	1040	1040	
	Smax	m	0.60	0.60	0.60	0.60	
	Maximum spacing Smax		**OK**	-	**OK**	-	

φt	Resistance factor for torsion	(5.5.4.2)	0.90	1.00	0.90	1.00	
pc	Outer perimeter of concrete section	m	3.200	3.200	3.200	3.200	
Acp	Area in outer perimeter of concrete section	m2	0.320	0.320	0.320	0.320	
fpc	Comp. stress in concrete after all prestress losses at the centroid of se	Mpa	0.00	0.00	0.00	0.00	
Tcr	Crack moment due to torsion	kNm	62.1	62.1	62.1	62.1	
	0.25*φ*Tcr	kNm	14.0	15.5	14.0	15.5	
Tu	Torsional moment by external forces	kNm	**29.7**	**15.6**	**29.5**	**15.7**	
	Shear and Torsion combine if Tu>0.25φTcr		**Yes**	**Yes**	**Yes**	**Yes**	
Ao	Area enclosed by shear flow path	m2	0.158	0.158	0.158	0.158	
At	Area of one leg of closed transverse torsion reinforcement	m2	0.0002	0.0002	0.0002	0.0002	
ph	Perimeter of the centerline of the closed transverse torsion reinf.	m	3.144	3.144	3.144	3.144	
Aoh	Area enclosed by centerline of ext. closed transverse torsion reinf.	m2	0.186	0.186	0.186	0.186	
Vu1	Modified Vu in case shear and torsion combine	kN	330	185	338	192	
v1	Determine θt in case shear and torsion combine	kN/m2	3251	1553	3243	1571	
θ	Assume	degree	39.92	38.24	39.54	37.95	
e_x	Strain in tensile reinforcement		2.03E-03	1.22E-03	1.87E-03	1.20E-03	
	if ex<0, multiple with reduce factor		-	-	-	-	
v1/f'c	Ratio of shear stress and f'c		0.093	0.044	0.093	0.045	
θt	Crack angle (S.5.8.3.4) upadated modified Vu	degree	39.92	38.24	39.54	37.95	
Tn	Nominal torsion resistance	kN	152	161	154	163	
Tr	Factored torsional resistance	kN	**137**	**161**	**139**	**163**	
(5.8.3.6.2)	**Torsional checking**		**OK**	**OK**	**OK**	**OK**	

RESULTS OF INTERNAL FORCE

END DIAPHRAGM





MIDAS/Civil
POST-PROCESSOR

BEAM DIAGRAM

MOMENT-y

	1.08825e+002
J	8.69498e+001
I	6.50747e+001
H	4.31995e+001
G	2.13244e+001
F	0.00000e+000
E	-2.24259e+001
D	-4.43010e+001
C	-6.61761e+001
B	-8.80513e+001
A	-1.09926e+002
	--1.31802e+002

CBall: Streng3A

MAX : 1019
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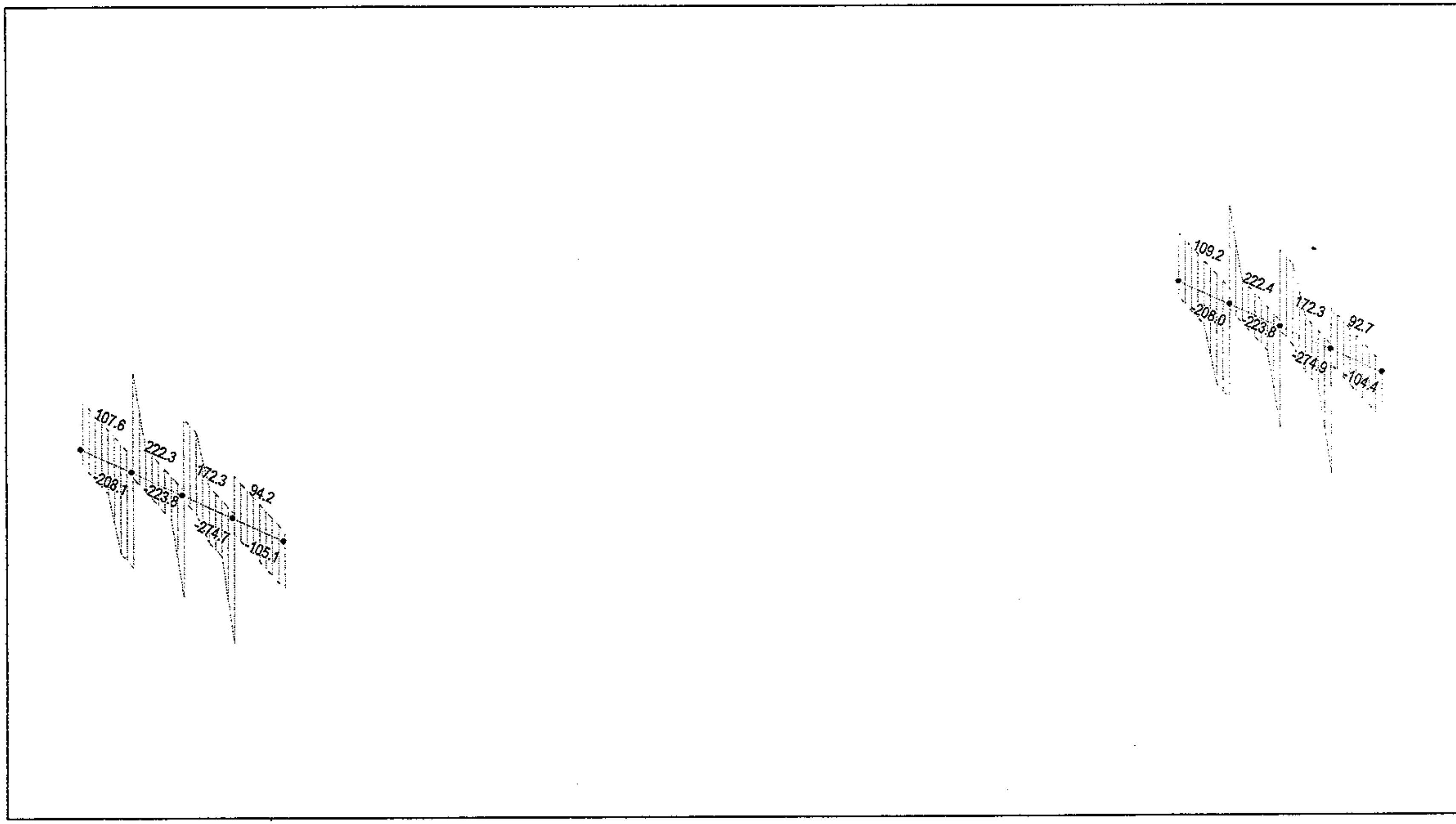
FILE: DIAPHRAGM(~
UNIT: kN·m
DATE: 12/15/2012

VIEW-DIRECTION
Y:-0.483
Z: 0.259



MIDAS/Civil
POST-PROCESSOR

BEAM DIAGRAM

SHEAR-z

	2.22439e+002
J	1.77229e+002
I	1.32019e+002
H	8.68097e+001
G	4.16000e+001
F	0.00000e+000
E	-4.88193e+001
D	-9.40289e+001
C	-1.39239e+002
B	-1.84448e+002
A	-2.29658e+002
	-2.74868e+002

CBall: Streng3A

MAX : 1014
MIN : 1009

FILE: DIAPHRAGM(~
UNIT: kN
DATE: 12/15/2012

VIEW-DIRECTION
Y:-0.663
Z: 0.259

MIDAS/Civil
POST-PROCESSOR

BEAM DIAGRAM

TORSION

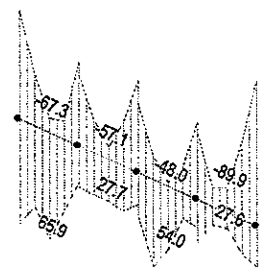
	7.24528e+001
J	5.87999e+001
I	4.51470e+001
H	3.14941e+001
G	1.78412e+001
F	0.00000e+000
E	-9.46456e+000
D	-2.31175e+001
C	-3.67703e+001
B	-5.04232e+001
A	-6.40761e+001
	--7.77290e+001

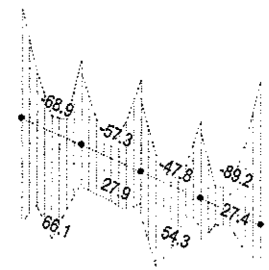
CBall: Streng3A

MAX : 1015
MIN : 1019

FILE: DIAPHRAGM(~
UNIT: kN·m
DATE: 12/15/2012

VIEW-DIRECTION
X:-0.483
Z: 0.259





MIDAS/Civil
POST-PROCESSOR

BEAM DIAGRAM

MOMENT-y

	6.61288e+001
J	5.19466e+001
I	3.77643e+001
H	2.35821e+001
G	9.39994e+000
F	0.00000e+000
E	-1.89645e+001
D	-3.31467e+001
C	-4.73289e+001
B	-6.15111e+001
A	-7.56933e+001
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CBall: Ser3A

MAX : 1019
MIN : 1000

FILE: DIAPHRAGM(~
UNIT: kN·m
DATE: 12/15/2012

VIEW-DIRECTION
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Z: 0.259

MIDAS/Civil
POST-PROCESSOR

BEAM DIAGRAM

SHEAR-z

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J	1.03729e+002
I	7.65841e+001
H	4.94394e+001
G	2.22947e+001
F	0.00000e+000
E	-3.19948e+001
D	-5.91395e+001
C	-8.62843e+001
B	-1.13429e+002
A	-1.40574e+002
	--1.67718e+002

CBall: Ser3A

MAX : 1014
MIN : 1009

FILE: DIAPHRAGM(~
UNIT: kN
DATE: 12/15/2012

VIEW-DIRECTION

Z: 0.259

MIDAS/Civil
POST-PROCESSOR

BEAM DIAGRAM

TORSION

	4.60905e+001
J	3.78867e+001
I	2.96829e+001
H	2.14791e+001
G	1.32753e+001
F	5.07145e+000
E	0.00000e+000
D	-1.13362e+001
C	-1.95400e+001
B	-2.77438e+001
A	-3.59476e+001
	-4.41514e+001

CBall: Ser3A

MAX : 1009
MIN : 1010

FILE: DIAPHRAGM(~
UNIT: kN·m
DATE: 12/15/2012

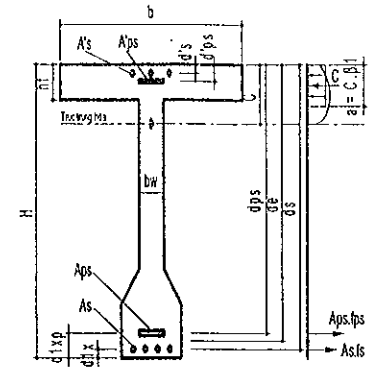
VIEW-DIRECTION

Z: 0.259

REINFORCEMENT CHECKING -END DIAPHRAGM

MATERIALS

NORMAL CONCRETE			
f'c	Compressive Strength of concrete at 28 days	Mpa	35
Ec	Modulus of Elasticity	Mpa	29910
fr	Modulus of Rupture	Mpa	3.7
gc	Unit weight of concrete	kN/m3	24.5
PRESTRESSING STEEL			
fpu	Tensile strength of prestressing steel	Mpa	1860
fpy	Yield strength of prestressing steel	Mpa	1670
Ep	Modulus of Elasticity	Mpa	195000
REINFORCEMENT			
fy	Yield strength	Mpa	400
Es	Modulus of Elasticity	Mpa	200000
nc	Ratio Es/Ec		7



Sign	Parameters		Unit	Section - End Diaphragm			
.				Pos.	Pos.	Neg.	Neg.
INTERNAL FORCES AT SECTIOIN							
	Combination			Strength	Service	Strength	Service
Qu	Shear		kN	222.40	130.80	274.80	167.7
Mu	Flexural Moment		kNm	108.80	66.10	131.80	89.8
Nu	Axial load		kN				
Tu	Torsional Moment		kNm	72.40	46.10	77.7	44.15
FLEXURAL MOMENT CHECKING							
H	Section height		m	1.600	1.600	1.600	1.600
d's	Dis. From comp. fiber to centroid of comp. Reinf		m	0.074	0.074	0.078	0.078
d1x	Dis. From tens. fiber to centroid of tension Reinf		m	0.078	0.078	0.074	0.074
	Cover to reinf		m	0.050	0.050	0.050	0.050
ds	Dis. From comp. fiber to centroid of tension Reinf		m	1.522	1.522	1.526	1.526
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	1.600	1.600	1.600	1.600
b	Width of the compression face of member		m	0.400	0.400	0.400	0.400
bw	Web width or diameter of a circular section		m	0.400	0.400	0.400	0.400
hf	Compression flange depth		m	0.000	0.000	0.000	0.000
Iz	Moment of inertia of section		m4	0.1365	0.1365	0.1365	0.1365
Amc	Section area		m2	0.640	0.640	0.640	0.640
	Steel choice						
Aps	Tension prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0
		Number	tendons	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000
A'ps	Compression prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0
		Number	tendons	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000
As	Tension Reinforcement	Number	bars	2	2	3	3
		Diameter	mm	28	28	20	20
		Area	m2	0.00116	0.00116	0.00089	0.00089
A's	Compression Reinforcement	Number	bars	0	0	0	0
		Diameter	mm	20	20	28	28
		Area	m2	0.00000	0.00000	0.00000	0.00000
A'c	Shear reinforcement	Number	bars	2	2	2	2
		Diameter	mm	14	14	14	14
		Area	m2	0.00030	0.00030	0.00030	0.00030
ϕ	Resistance factors for flexure		5.5.4.2	0.90	1.00	0.90	1.00
ϕv	Resistance factors for shear			0.90	1.00	0.90	1.00
ϕn	Resistance factors for axial force			1.00	1.00	1.00	1.00
β1	Stress block factor			0.800	0.800	0.800	0.800
c	Dis. Between centroid and top fiber		m	0.049	0.049	0.037	0.037
	For T section behavior		m	0.049	0.049	0.037	0.037
	For rectangular section behavior		m	0.049	0.049	0.037	0.037
fpe	Effective stress in the prestressing steel after losses		Mpa	0	0	0	0
fps	Aver. stress in pres. steel at the time for which the nominal resistanc		Mpa	0	0	0	0
k	Factor depends on type of P.S, Low relaxation strand k = 0.28			0.00	0.00	0.00	0.00

a	Depth of equivalent stress block	m	0.039	0.039	0.030	0.030	
de	Corresp. effective depth from extreme comp. fiber to centroid of tensile force in the tensile reinf.	m	1.522	1.522	1.526	1.526	
Mn	Nominal resistance	kNm	696	696	535	535	
Mr	Factored resistance	kNm	626	696	482	535	
Mu	Flexual moment	kNm	109	66	132	90	
(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*Mcr	Craking moment	kNm	394	394	391	391	
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**		**OK**	**OK**	**OK**	**OK**	
(5.7.3.4)	**Conctrol of craking by distr. of reinf for RC member- Check?**		No	Yes	No	Yes	
	Existing condition for structrure	1,2 or 3	1	1	1	1	
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.064	0.064	0.060	0.060	
Z	Crack width parameter	N/mm	30000	30000	30000	30000	
A	Area of concr. with same centroid as tens. Reinf	m2	0.026	0.026	0.016	0.016	
fsa	Value	Mpa	254	254	304	304	
0.6*fy		Mpa	240	240	240	240	
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	240	240	240	240	
x	Dist. From compression fiber to centroid	m	-	0.229	-	0.202	
J.d	Arm	m	-	1.446	-	1.459	
Icr	Moment of inertia of the cracked section	m4	-	0.015	-	0.012	
fs	Tensile stress in reinforcement fs = Msls / (As*J.d)	Mpa	-	39	-	70	
	Checking for control cracking fs<fsa		**N.a**	**OK**	**N.a**	**OK**	
(5.10.8.2)	**Shrinkage and temperature Reinfocement (side distribution)**						
Areq	Area of required reinf	m2	0.00030	0.00030	0.00030	0.00030	
	Distribution on sides 4 D16	m2	0.00081	0.00081	0.00081	0.00081	
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45	
	Checking		**OK**	**OK**	**OK**	**OK**	

SHEAR AND TORSION CHECKING

β	Factor indicating diag. cracked concr. to tension		2.2	2.4	2.0	2.2	
θ	Angle of inclination of diagonal compressive	degree	35.76	31.44	40.13	35.88	
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	0.400	0.400	0.400	0.400	
dv	Effective shear depth	m	1.503	1.503	1.511	1.511	
	(de - a/2)	m	1.503	1.503	1.511	1.511	
s	Spacing of stirrups	m	0.150	0.150	0.150	0.150	
ncat	Amount of bars in spacing S	bars	2	2	2	2	
Av	Shear reinf area in spacing S	m2	0.0003	0.0003	0.0003	0.0003	
β	Assume		2.2	2.4	2.0	2.20	
θ	Assume	degree	35.76	31.43	40.13	35.89	
v	Shear stress in concrete	kN/m2	411	218	505	277	
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	0	0	0	0	
e_x	Strain in tensile reinforcement		9.80E-04	6.52E-04	1.41E-03	9.90E-04	
	if ex<0, multiple with reduce factor		-	-	-	-	
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok	
v/f'c	Ratio of shear stress and f'c		0.012	0.006	0.014	0.008	
β	Final value		2.2	2.4	2.0	2.2	
θ	Final value	degree	35.76	31.44	40.13	35.9	
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	661	716	593	664	
Vs	Shear resistance provided by shear reinforcement	kN	1681	1980	1443	1682	
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	2342	2695	2037	2346	
Vn2	Vn2	kN	5259	5259	5289	5289	
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	2342	2695	2037	2346	
Vr	Factored shear resistance	kN	**2108**	**2695**	**1833**	**2346**	
Vu	Shear	kN	222	131	275	168	
(5.8.2.7)	**Shear checking**		**OK**	**OK**	**OK**	**OK**	
	Region requiring transverse reinf Checking		No need	No need	Need	No need	
	Minimum shear reinf area	m2	0.0001	0.0001	0.0001	0.0001	
	Mimimum shear reinforcement Checking		-	-	**OK**	-	
	0.1*f'c*bv*dv	kN	2104	2104	2116	2116	
	Smax	m	0.60	0.60	0.60	0.60	
	Maximum spacing Smax		-	-	**OK**	-	

φt	Resistance factor for torsion	(5.5.4.2)	0.90	1.00	0.90	1.00	
pc	Outer perimeter of concrete section	m	3.600	3.600	3.600	3.600	
Acp	Area in outer perimeter of concrete section	m2	0.640	0.640	0.640	0.640	
fpc	Comp. stress in concrete after all prestress losses at the centroid of se	Mpa	0.00	0.00	0.00	0.00	
Tcr	Crack moment due to torsion	kNm	221	221	221	221	
	0.25*φ*Tcr	kNm	50	55	50	55	
Tu	Torsional moment by external forces	kNm	72	46	78	44	
	Shear and Torsion combine if Tu>0.25φTcr		Yes	No	Yes	No	
Ao	Area enclosed by shear flow path	m2	0.172	0.172	0.383	0.383	
At	Area of one leg of closed transverse torsion reinforcement	m2	0.0002	0.0002	0.0002	0.0002	
ph	Perimeter of the centerline of the closed transverse torsion reinf.	m	3.544	3.544	3.544	3.544	
Aoh	Area enclosed by centerline of ext. closed transverse torsion reinf.	m2	0.203	0.203	0.450	0.450	
Vul	Modified Vu in case shear and torsion combine	kN	707	447	425	249	
vl	Determine θt in case shear and torsion combine	kN/m2	7074	4053	1701	876	
θ	Assume	degree	37.23	37.91	42.66	39.08	
e_x	Strain in tensile reinforcement		2.43E-03	1.77E-03	1.92E-03	1.31E-03	
	if ex<0, multiple with reduce factor		-	-	-	-	
vl/f'c	Ratio of shear stress and f'c		0.202	0.116	0.049	0.025	
θt	Crack angle (S.5.8.3.4) upadated modified Vu	degree	37.23	37.91	42.66	39.08	
Tn	Nominal torsion resistance	kN	182	-	334	-	
Tr	Factored torsional resistance	kN	164	-	301	-	
(5.8.3.6.2)	**Torsional checking**		**OK**	**N.a**	**OK**	**N.a**	

MINISTRY OF TRANSPORT

VIETNAM EXPRESSWAY CORPORATION - PROJECT MANAGEMENT UNIT NO.85

DA NANG - QUANG NGAI EXPRESSWAY DEVELOPMENT PROJECT

PACKAGE: 6

CALCULATION SHEETS

MISCELLANEOUS

HA NOI, 2012

CALCULATION SHEET

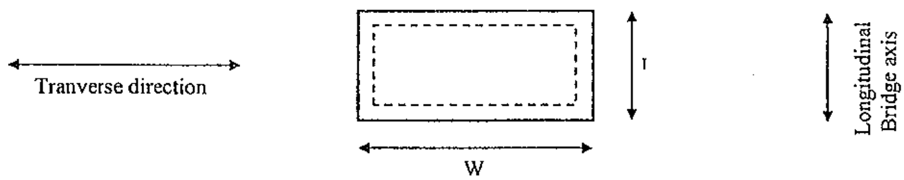
BEARING

22TCN272-05; AASHTO LRFD 3rd - 2004

STEEL-REINFORCED ELASTOMERIC BEARING - design checking

Method B - S.14.7.5

Plan view



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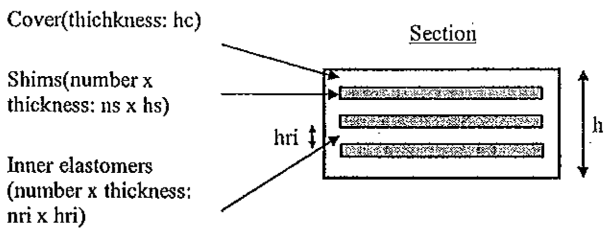


1. Materials

The elastomer shall have a shear modulus between 0.60 and 1.3 MPa and a nominal hardness between 50 and 60 on the Shore A scale.
It shall conform to the requirements of Section 18.2 of the AASHTO LRFD Bridge Construction

The shear modulus of the elastomer at 23°C shall be used as the basis for design.

Item	Hardness (Shore A)		
	50	60	70
Shear modulus @ 23^{0}C (MPa)	0.66-0.90	0.90-1.38	1.38-2.07
Creep deflection @25 years divided by instantaneous deflection	0.25	0.35	0.45

Choose Shear modulus for elastomer material at 23^{0}C G = 1.00 Mpa
Yield strength of shims plate and soles steel (ASTM A709M grade 250) Fy = 250.0 Mpa

2.Load

Bearing type:
"1: bearing subject to shear deformation (moveable bear.); 2: bearing fixed against shear deformation (fixed bear.)" 1
Design force on bearing

Combination	Max. Factored Reaction		Rotation		Horizontal movement
	All (kN)	Live load (kN)	θs Long. (rad)	θs Trans. (rad)	Δ_0(mm)
Strength 1	1631.3	753.1			
Service 1	1119.4	430.3	0.006	0.000	24.3

*Dynamic Impact load included in design for conservative approach

3.Design checking

a. Bearing configuration S.14.7.5.1

In any elastomeric bearing layer, the average compressive stress at the service limit state shall satisfy:

Item		Sign	Unit	Value	Shape factor	Check.*
Dimensions	Longitudinal axis of bridge	L	mm	350		
	Tranverse axis	W	mm	500		
	Height	h	mm	84		
Shims	Number of shim	ns	shims	5		
	Thickness	ts	mm	3		
	Longitudinal axis of bridge	Ls	mm	340		
	Tranverse axis	Ws	mm	490		
Cover	Top thickness	hct	mm	2.5	41.16	**Ok**
	Bottom thickness	hcb	mm	2.5	41.16	**Ok**
Inner elastormer layers						
	Number layers	nr	layers	4		
	Layer thickness	hri	mm	16.0	6.43	
Check total height of bearing		**Ok**	mm	84		

Shape factor of an elastomer layer $S_i = L.W / [2.h_{ri}.(L+W)]$ $L.W / [2(L+W)] =$ 102.9

* To ensure that top or bottom cover elastomer layer is not thicker than 70% of inner elastomer layer

b. Compressive Stress   S.14.7.5.3.2

In any elastomeric bearing layer, the average compressive stress at the service limit state shall satisfy:

Service average compressive stress due to the total load		$\sigma s =$	6.40 Mpa
Service average compressive stress due to live load		$\sigma L =$	2.46 Mpa
Shape factor of the thickest layer of the bearing		S =	6.43
For bearings subject to shear deformation	$\sigma s \leq 1.66.G.S \leq 11.0$ Mpa	1.66.G.S =	10.67 Mpa
		Check =	Ok
	$\sigma L \leq 0.66.G.S$	0.66.G.S =	4.24 Mpa
		Check =	Ok
For bearings fixed against shear deformation	$\sigma s \leq 2.00.G.S \leq 12.0$ Mpa	2.00.G.S =	- Mpa
		Check =	-

c. Compressive Deflection   S.14.7.5.3.3

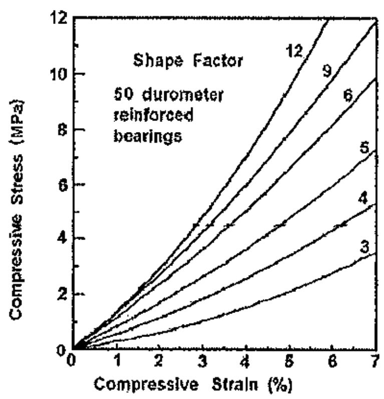
Only when deck joints or seals are present on the bridge: "1: check, 2: not check"   1

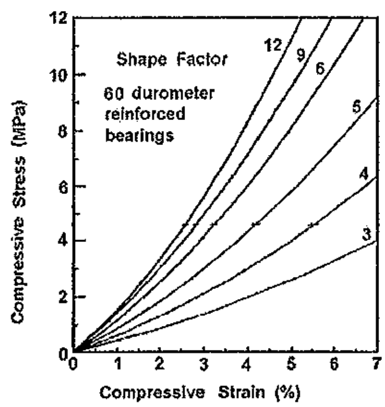
Instantaneous deflection shall be taken $\delta = \Sigma\varepsilon i.\ hri \leq 3mm$

A maximum relative deflection across a joint of 3 mm is suggested

Instantaneous compressive strain in i[th] elastomer layer of a laminated bearing εi

In the absence of information specific to the particular elastomer to be used, Figure below may be used as a guide.





Creep deflection factor		$\phi cr =$	0.35
Creep effect to instantaneous load	$\delta cr = \delta.\phi cr$		
Thickness of i[th] elastomeric layer in a laminated bearing		hri	

No	Elastomer Layer Number		σL (Mpa)	S	hri (mm)	εi (%)	εi. hri.ni (mm)
1	Top cover	1	2.46	6.43	2.5	2.5	0.06
2	Bottom cover	1	2.46	6.43	2.5	2.5	0.06
3	Inner layers	4	2.46	6.43	16.0	2.5	1.60
4							
5							
						δcr =	0.60
	Check	Ok				δ + δcr =	2.33

d. Shear Deformation   S.14.7.5.3.4

Only for moveable bearing (For bearings subject to shear deformation)

The maximum shear deformation of the bearing, at the service limit state, ΔS, shall be taken as Δo, modified to account for the substructure stiffness and construction procedures.

If a low friction sliding surface is installed, DSneed not be taken to be larger than the deformation corresponding to first slip.

The horizontal movement of the bridge superstructure $\Delta_0 = \Delta cr\&sh + \Delta ps + \Delta temp$ $\Delta_0 =$ 24.3 mm

(extreme displacement caused by creep, shrinkage, post-tensioning, combined with thermal effects)

Total elastomer thickness		hrt =	69 mm
		2.Δs =	48.60 mm
The bearing shall satisfy	$hrt \geq 2.\Delta s$	Check =	Ok

e.Combined Compression & Rotation S.14.7.5.3.5

The provisions of this section shall apply at the service limit state.
The goal is to prevent uplift of any corner of the bearing under any combination of loading and corresponding rotation.

Rectangular bearings may be taken to satisfy uplift requirements if they satisfy:

$$\sigma s > 1.0.G.S\ (\theta s/n).\ (B/hri)^2 = F1$$

Rectangular bearings subjected to shear deformation shall also satisfy:

$$\sigma s < 1.875.G.S\ [1-0.2(\theta s/n).\ (B/hri)^2] = F2$$

Rectangular bearings fixed against shear deformation shall also satisfy:

$$\sigma s < 2.25.G.S\ [1-0.167(\theta s/n).\ (B/hri)^2] = F3$$

Number of interior layers of elastomer.	n =	4	layers
Stress in elastomer	σs =	6.40	Mpa
Length of pad if rotation is about its transverse axis or width of pad if rotation is about its longitudir	B		
Maximum service rotation due to the total load about long. Or trans. axis	θs		

No	Direction	B (mm)	hri (mm)	θs (rad)
1	Longitudinal rotation	350.0	16.0	0.006
2	Transverse rotaiton	500.0	16.0	0.000

No	Direction	F1 (Mpa)	Check F1	F2 (Mpa)	Check F2	F3 (Mpa)	Check F3
1	Longitudinal rotation	4.3	Ok	10.5	Ok	-	-
2	Transverse rotaiton	0.0	Ok	12.1	Ok	-	-

f.Stability of Elastomeric Bearings S.14.7.5.3.6

Bearings satisfying equation here shall be considered sta $2A \le B$

Where	$A = (1.92.hrt/L)/sqrt(1+2.0.L/W)$	A =	0.244
	$B = 2.67/[(S+2.0).(1+L/(4.0.W))]$	B =	0.270
		2A =	0.489
		Check =	Need check

For a rectangular bearing where L is greater than W, stability shall be investigated by interchanging L and W
For a rectangular bearing not satisfying $2A \le B$, the stress due to total load shall be satify below equations:

If the bridge deck is free to translate horizontally	$\sigma s \le G.S/(2A-B)$	G.S/ (2A-B) =	29.3 Mpa
		Check =	**Stable**
If the bridge deck is fixed against horizontal translation	$\sigma s \le G.S/(A-B)$	(A-B) =	-0.025
		G.S/ (A-B) =	-
If the value $A-B \le 0$, the bearing is stable and is not dependent on σs.		Check =	**Stable**

g.Reinforcement S.14.7.5.3.7

The thickness of the steel reinforcement, hs, shall satisfy the provisions of Article 14.7.5.3.7 of the AASHTO LRFD Bridge Construction Specifications and:

At service limit state	$hs \ge 3.hmax.\sigma s/Fy$		
Thickness of thickest elastomeric layer in elastomeric bearing		hmax =	16.0 mm
Service average compressive stress due to total load		σs =	6.40 Mpa
Yield strength of steel reinforcement		Fy =	250.0 Mpa
		hs =	3.0 mm
		$3.hmax.\sigma s/Fy$ =	1.23 mm
		Check =	**Ok**

At fatigue limit state	$hs \ge 2.0.hmax.\sigma L/\Delta F_{TH}$		
Constant amplitude fatigue threshold for Category A as specified in Article 6.6		ΔF_{TH} =	165.0 Mpa
Service average compressive stress due to live load		σL =	2.46 Mpa
		$2.0.hmax.\sigma L/\Delta F_{TH}$ =	0.48 mm
		Check =	**Ok**

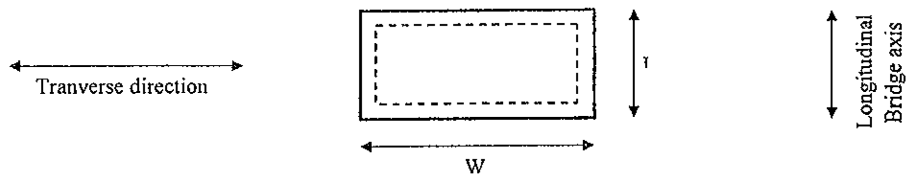
h.Seismic Provisions S.14.7.5.3.8

Elastomeric expansion bearings shall be provided with adequate seismic resistant anchorage to resist the horizontal forces in excess of those accommodated by shear in the pad. The sole plate and the base plate shall be made wider to accommodate the anchor bolts. The anchor bolts shall be designed for the combined effect of bending and shear for seismic loads as specified in Article 14.6.5.3. Elastomeric fixed bearings shall be provided with horizontal restraint adequate for the full horizontal load.

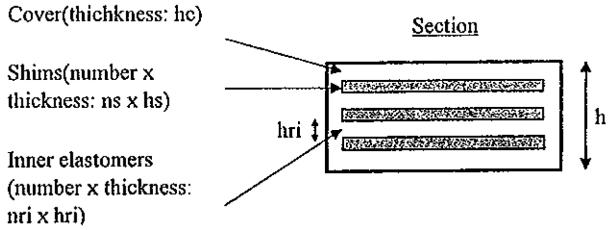
STEEL-REINFORCED ELASTOMERIC BEARING - design checking

Method B - S.14.7.5

Plan view



Section



1. Materials

The elastomer shall have a shear modulus between 0.60 and 1.3 MPa and a nominal hardness between 50 and 60 on the Shore A scale.
It shall conform to the requirements of Section 18.2 of the AASHTO LRFD Bridge Construction

The shear modulus of the elastomer at 23°C shall be used as the basis for design.

Item	Hardness (Shore A)		
	50	60	70
Shear modulus @ 23°C (MPa)	0.66-0.90	0.90-1.38	1.38-2.07
Creep deflection @25 years divided by instantaneous deflection	0.25	0.35	0.45

Choose Shear modulus for elastomer material at 23°C G = 1.00 Mpa
Yield strength of shims plate and soles steel (ASTM A709M grade 250) Fy = 250.0 Mpa

2.Load

Bearing type:
"1: bearing subject to shear deformation (moveable bear.); 2: bearing fixed against shear deformation (fixed bear.)" 1

Design force on bearing

Combination	Max. Factored Reaction		Rotation		Horizontal movement
	All (kN)	Live load (kN)	θs Long. (rad)	θs Trans. (rad)	Δ_0(mm)
Strength I	1232.8	545.7			
Service I	851.5	311.8	0.008	0.000	8.5

*Dynamic Impact load included in design for conservative approach

3.Design checking

a. Bearing configuration S.14.7.5.1

In any elastomeric bearing layer, the average compressive stress at the service limit state shall satisfy:

Item		Sign	Unit	Value	Shape factor	Check.*
Dimensions	Longitudinal axis of bridge	L	mm	300		
	Tranverse axis	W	mm	400		
	Height	h	mm	69		
Shims	Number of shim	ns	shims	4		
	Thickness	ts	mm	2		
	Longitudinal axis of bridge	Ls	mm	290		
	Tranverse axis	Ws	mm	390		
Cover	Top thickness	hct	mm	2.5	34.28	Ok
	Bottom thickness	hcb	mm	2.5	34.28	Ok
Inner elastormer layers						
	Number layers	nr	layers	3		
	Layer thickness	hri	mm	18.7	4.59	
Check total height of bearing		Ok	mm	69		

Shape factor of an elastomer layer Si = L.W/ [2.hri.(L+W)] L.W/ [2(L+W)] = 85.7

* To ensure that top or bottom cover elastomer layer is not thicker than 70% of inner elastomer layer

b. Compressive Stress S.14.7.5.3.2

In any elastomeric bearing layer, the average compressive stress at the service limit state shall satisfy:

Service average compressive stress due to the total load		σs =	7.10 Mpa
Service average compressive stress due to live load		σL =	2.60 Mpa
Shape factor of the thickest layer of the bearing		S =	4.59
For bearings subject to shear deformation	$\sigma s \le 1.66.G.S \le 11.0$ Mpa	1.66.G.S =	7.62 Mpa
		Check =	Ok
	$\sigma L \le 0.66.G.S$	0.66.G.S =	3.03 Mpa
		Check =	Ok
For bearings fixed against shear deformation	$\sigma s \le 2.00.G.S \le 12.0$ Mpa	2.00.G.S =	- Mpa
		Check =	-

c. Compressive Deflection S.14.7.5.3.3

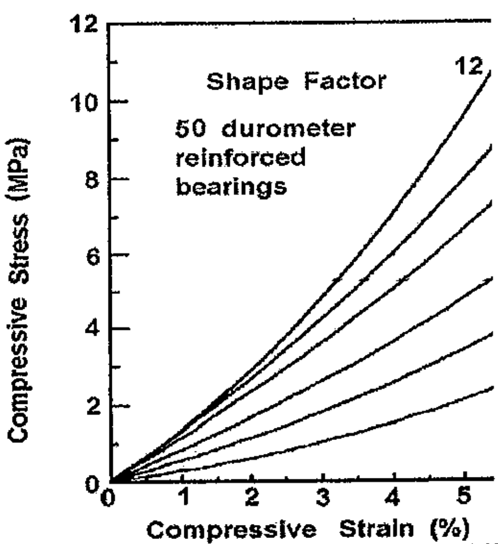
Only when deck joints or seals are present on the bridge: "1: check , 2: not check" 1

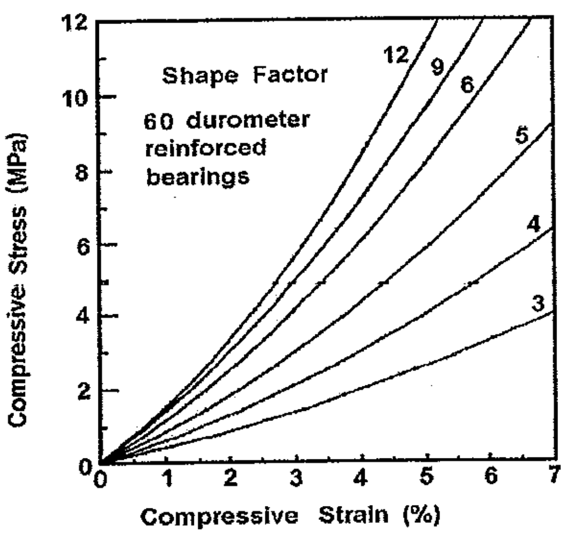
Instantaneous deflection shall be taken $\delta = \Sigma \varepsilon i.\ hri \le 3$mm

A maximum relative deflection across a joint of 3 mm is suggested

Instantaneous compressive strain in i[th] elastomer layer of a laminated bearing εi

In the absence of information specific to the particular elastomer to be used, Figure below may be used as a guide.





Creep defle

Creep effec

Thickness c

No	Elastomer Layer	Number	σL (Mpa)	S	hri (mm)	εi (%)	εi. hri.ni (mm)
1	Top cover	1	2.60	4.59	2.5	3.5	0.09
2	Bottom cover	1	2.60	4.59	2.5	3.5	0.09
3	Inner layers	3	2.60	4.59	18.7	3.5	1.96
4							
5							
						δcr =	0.75
	Check	Ok				$\delta + \delta cr$ =	2.88

d. Shear Deformation S.14.7.5.3.4

Only for moveable bearing (For bearings subject to shear deformation)

The maximum shear deformation of the bearing, at the service limit state, ΔS, shall be taken as Δo, modified to account for the substructure stiffness and construction procedures.

If a low friction sliding surface is installed, DSneed not be taken to be larger than the deformation corresponding to first slip.

The horizontal movement of the bridge superstructure $\Delta_0 = \Delta cr\&sh + \Delta ps + \Delta temp$ Δ_0 = 8.5 mm

(extreme displacement caused by creep, shrinkage, post-tensioning, combined with thermal effects)

Total elastomer thickness		hrt =	61 mm
		$2.\Delta s$ =	17.01 mm
The bearing shall satisfy	hrt $\ge 2.\Delta s$	Check =	Ok

e.Combined Compression & Rotation S.14.7.5.3.5

The provisions of this section shall apply at the service limit state.

The goal is to prevent uplift of any corner of the bearing under any combination of loading and corresponding rotation.

Rectangular bearings may be taken to satisfy uplift requirements if they satisfy:

$$\sigma s > 1.0.G.S\ (\theta s\ /n).\ (B\ /hri)^2 = F1$$

Rectangular bearings subjected to shear deformation shall also satisfy:

$$\sigma s < 1.875.G.S\ [1\text{-}0.2(\theta s\ /n).\ (B\ /hri)^2] = F2$$

Rectangular bearings fixed against shear deformation shall also satisfy:

$$\sigma s < 2.25.G.S\ [1\text{-}0.167(\theta s\ /n).\ (B\ /hri)^2] = F3$$

Number of interior layers of elastomer.	n =	3 layers
Stress in elastomer	σs =	7.10 Mpa
Length of pad if rotation is about its transverse axis or width of pad if rotation is about its longitudir	B	
Maximum service rotation due to the total load about long. Or trans. axis	θs	

No	Direction	B (mm)	hri (mm)	θs (rad)
1	Longitudinal rotation	300.0	18.7	0.008
2	Transverse rotaiton	400.0	18.7	0.000

No	Direction	F1 (Mpa)	Check F1	F2 (Mpa)	Check F2	F3 (Mpa)	Check F3
1	Longitudinal rotation	3.1	Ok	7.4	Ok	-	-
2	Transverse rotaiton	0.0	Ok	8.6	Ok	-	-

f.Stability of Elastomeric Bearings S.14.7.5.3.6

Bearings satisfying equation here shall be considered stal $2A \leq B$

Where A = (1.92.hrt/L)/sqrt(1+2.0.L/ W) A = 0.247

B = 2.67/ [(S+2.0) . (1+L/(4.0.W))] B = 0.341

2A = 0.494

Check = Need check

For a rectangular bearing where L is greater than W, stability shall be investigated by interchanging L and W

For a rectangular bearing not satisfying 2A≤B, the stress due to total load shall be satify below equations:

If the bridge deck is free to translate horizontally	σs ≤ G.S/ (2A-B)	G.S/ (2A-B) =	30.1 Mpa
		Check =	Stable
If the bridge deck is fixed against horizontal translation	σs ≤ G.S/ (A-B)	(A-B) =	-0.094
		G.S/ (A-B) =	-
If the value A−B≤0, the bearing is stable and is not dependent on σs.		Check =	Stable

g.Reinforcement S.14.7.5.3.7

The thickness of the steel reinforcement, hs, shall satisfy the provisions of Article 14.7.5.3.7 of the AASHTO LRFD Bridge Construction Specifications and:

At service limit state hs ≥ 3.hmax.σs/ Fy

Thickness of thickest elastomeric layer in elastomeric bearing	hmax =	18.7 mm
Service average compressive stress due to total load	σs =	7.10 Mpa
Yield strength of steel reinforcement	Fy =	250.0 Mpa
	hs =	2.0 mm
	3.hmax.σs/ Fy =	1.59 mm
	Check =	Ok

At fatigue limit state hs ≥ 2.0.hmax.σL/ ΔF_{TH}

Constant amplitude fatigue threshold for Category A as specified in Article 6.6	ΔF_{TH} =	165.0 Mpa
Service average compressive stress due to live load	σL =	2.60 Mpa
	2.0.hmax.σL/ ΔF_{TH} =	0.59 mm
	Check =	Ok

h.Seismic Provisions S.14.7.5.3.8

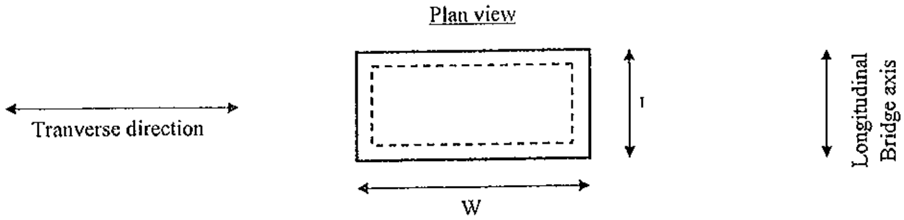
Elastomeric expansion bearings shall be provided with adequate seismic resistant anchorage to resist the horizontal forces in excess of those accommodated by shear in the pad. The sole plate and the base plate shall be made wider to accommodate the anchor bolts. The anchor bolts shall be designed for the combined effect of bending and shear for seismic loads as specified in Article 14.6.5.3. Elastomeric fixed bearings shall be provided with horizontal restraint adequate for the full horizontal load.

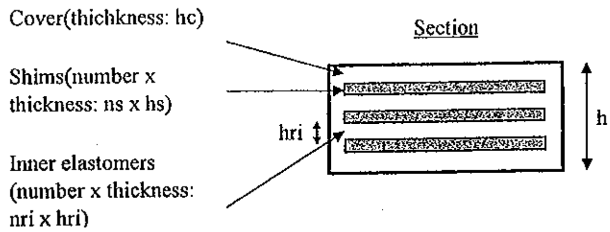
	DA NANG - QUANG NGAI EXPRESSWAY PROJECT PACKAGE 6 - BRIDGE	Item.	Eng.	Date.	Sign.
		Design			
	DETAIL DESIGN STEEL-REINFORCED ELASTOMERIC BEARING I-24m	Check			
		Revise			

22TCN272-05; AASHTO LRFD 3rd - 2004

STEEL-REINFORCED ELASTOMERIC BEARING - design checking

Method B - S.14.7.5





1. Materials

The elastomer shall have a shear modulus between 0.60 and 1.3 MPa and a nominal hardness between 50 and 60 on the Shore A scale.
It shall conform to the requirements of Section 18.2 of the AASHTO LRFD Bridge Construction

The shear modulus of the elastomer at 23°C shall be used as the basis for design.

Item	Hardness (Shore A)		
	50	60	70
Shear modulus @ 23°C (MPa)	0.66-0.90	0.90-1.38	1.38-2.07
Creep deflection @25 years divided by instantaneous deflection	0.25	0.35	0.45

Choose Shear modulus for elastomer material at 23°C — G = 1.00 Mpa
Yield strength of shims plate and soles steel (ASTM A709M grade 250) — Fy = 250.0 Mpa

2.Load

Bearing type:
"1: bearing subject to shear deformation (moveable bear.); 2: bearing fixed against shear deformation (fixed bear.)" 1
Design force on bearing

Combination	Max. Factored Reaction		Rotation		Horizontal movement Δ_0(mm)
	All (kN)	Live load (kN)	θs Long. (rad)	θs Trans. (rad)	
Strength 1	1408.9	623.6			
Service 1	973.1	356.4	0.010	0.000	9.7

*Dynamic Impact load included in design for conservative approach

3.Design checking

a. Bearing configuration S.14.7.5.1

In any elastomeric bearing layer, the average compressive stress at the service limit state shall satisfy:

Item		Sign	Unit	Value	Shape factor	Check.*
Dimensions	Longitudinal axis of bridge	L	mm	350		
	Tranverse axis	W	mm	400		
	Height	h	mm	69		
Shims	Number of shim	ns	shims	4		
	Thickness	ts	mm	2		
	Longitudinal axis of bridge	Ls	mm	340		
	Tranverse axis	Ws	mm	390		
Cover	Top thickness	hct	mm	2.5	37.32	Ok
	Bottom thickness	hcb	mm	2.5	37.32	Ok
Inner elastormer layers						
	Number layers	nr	layers	3		
	Layer thickness	hri	mm	18.7	5	
Check total height of bearing		Ok	mm	69		

Shape factor of an elastomer layer $Si = L.W / [2.hri.(L+W)]$ $L.W / [2(L+W)] =$ 93.3

* To ensure that top or bottom cover elastomer layer is not thicker than 70% of inner elastomer layer

b. Compressive Stress S.14.7.5.3.2

In any elastomeric bearing layer, the average compressive stress at the service limit state shall satisfy:

Service average compressive stress due to the total load	$\sigma s =$	6.95 Mpa
Service average compressive stress due to live load	$\sigma L =$	2.55 Mpa
Shape factor of the thickest layer of the bearing	$S =$	5.00

For bearings subject to shear deformation	$\sigma s \le 1.66.G.S \le 11.0$ Mpa	1.66.G.S =	8.30 Mpa
		Check =	Ok
	$\sigma L \le 0.66.G.S$	0.66.G.S =	3.30 Mpa
		Check =	Ok
For bearings fixed against shear deformation	$\sigma s \le 2.00.G.S \le 12.0$ Mpa	2.00.G.S =	- Mpa
		Check =	-

c. Compressive Deflection S.14.7.5.3.3

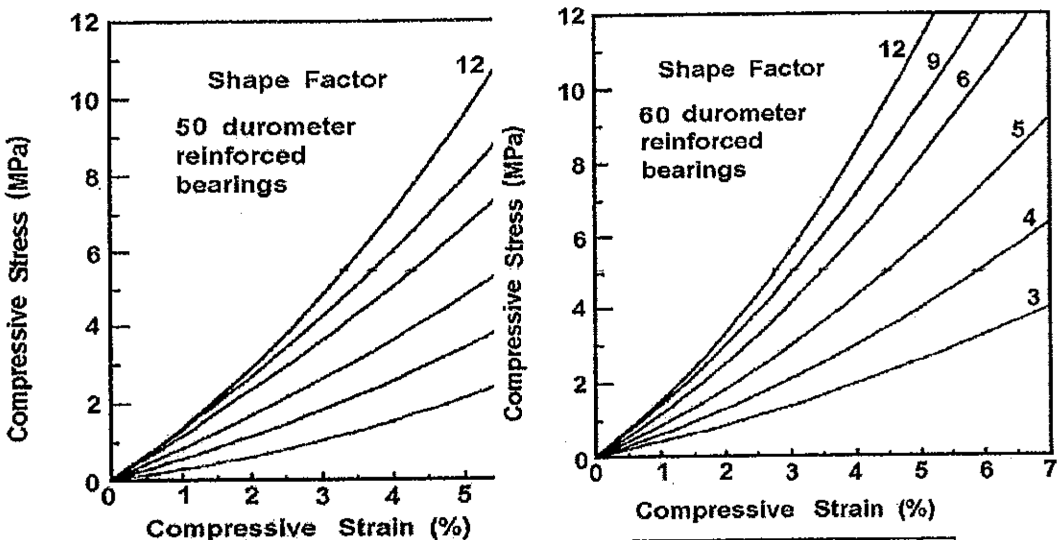
Only when deck joints or seals are present on the bridge: "1: check , 2: not check" 1

Instantaneous deflection shall be taken $\delta = \Sigma \varepsilon i.\ hri \le 3$mm

A maximum relative deflection across a joint of 3 mm is suggested

Instantaneous compressive strain in i[th] elastomer layer of a laminated bearing εi

In the absence of information specific to the particular elastomer to be used, Figure below may be used as a guide.



Creep defle

Creep effec

Thickness (

No	Elastomer Layer	Number	σL (Mpa)	S	hri (mm)	εi (%)	εi. hri.ni (mm)
1	Top cover	1	2.55	5.00	2.5	3.0	0.08
2	Bottom cover	1	2.55	5.00	2.5	3.0	0.08
3	Inner layers	3	2.55	5.00	18.7	3.0	1.68
4							
5							
						δcr =	0.64
	Check	Ok				δ + δcr =	2.47

d. Shear Deformation S.14.7.5.3.4

Only for moveable bearing (For bearings subject to shear deformation)

The maximum shear deformation of the bearing, at the service limit state, ΔS, shall be taken as Δo, modified to account for the substructure stiffness and construction procedures.

If a low friction sliding surface is installed, DSneed not be taken to be larger than the deformation corresponding to first slip.

The horizontal movement of the bridge superstructure $\Delta_0 = \Delta cr\&sh + \Delta ps + \Delta temp$ $\Delta_0 =$ 9.7 mm

(extreme displacement caused by creep, shrinkage, post-tensioning, combined with thermal effects)

Total elastomer thickness		hrt =	61 mm
		2.Δs =	19.44 mm
The bearing shall satisfy	hrt ≥ 2.Δs	Check =	Ok

e.Combined Compression & Rotation S.14.7.5.3.5

The provisions of this section shall apply at the service limit state.

The goal is to prevent uplift of any corner of the bearing under any combination of loading and corresponding rotation.

Rectangular bearings may be taken to satisfy uplift requirements if they satisfy:

$$\sigma s > 1.0.G.S\ (\theta s\ /n).\ (B\ /hri)^2 = F1$$

Rectangular bearings subjected to shear deformation shall also satisfy:

$$\sigma s < 1.875.G.S\ [1\text{-}0.2(\theta s\ /n).\ (B\ /hri)^2] = F2$$

Rectangular bearings fixed against shear deformation shall also satisfy:

$$\sigma s < 2.25.G.S\ [1\text{-}0.167(\theta s\ /n).\ (B\ /hri)^2] = F3$$

Number of interior layers of elastomer.	n =	3	layers
Stress in elastomer	σs =	6.95	Mpa
Length of pad if rotation is about its transverse axis or width of pad if rotation is about its longitudir	B		
Maximum service rotation due to the total load about long. Or trans. axis	θs		

No	Direction	B (mm)	hri (mm)	θs (rad)
1	Longitudinal rotation	350.0	18.7	0.010
2	Transverse rotaiton	400.0	18.7	0.000

No	Direction	F1 (Mpa)	Check F1	F2 (Mpa)	Check F2	F3 (Mpa)	Check F3
1	Longitudinal rotation	5.7	Ok	7.2	Ok	-	-
2	Transverse rotaiton	0.0	Ok	9.4	Ok	-	-

f.Stability of Elastomeric Bearings S.14.7.5.3.6

Bearings satisfying equation here shall be considered stal $2A \le B$

Where $A = (1.92.hrt/L)/sqrt(1+2.0.L/W)$ A = 0.202

$B = 2.67/[(S+2.0).(1+L/(4.0.W))]$ B = 0.313

2A = 0.404

Check = Need check

For a rectangular bearing where L is greater than W, stability shall be investigated by interchanging L and W

For a rectangular bearing not satisfying 2A≤B, the stress due to total load shall be satify below equations:

If the bridge deck is free to translate horizontally $\sigma s \le G.S/(2A-B)$ G.S/ (2A-B) = 55.2 Mpa

Check = Stable

If the bridge deck is fixed against horizontal translation $\sigma s \le G.S/(A-B)$ (A-B) = -0.111

G.S/ (A-B) = -

If the value A−B≤0, the bearing is stable and is not dependent on σs. Check = Stable

g.Reinforcement S.14.7.5.3.7

The thickness of the steel reinforcement, hs, shall satisfy the provisions of Article 14.7.5.3.7 of the AASHTO LRFD Bridge Construction Specifications and:

At service limit state $hs \ge 3.hmax.\sigma s/Fy$

Thickness of thickest elastomeric layer in elastomeric bearing	hmax =	18.7	mm
Service average compressive stress due to total load	σs =	6.95	Mpa
Yield strength of steel reinforcement	Fy =	250.0	Mpa
	hs =	2.0	mm
	3.hmax.σs/ Fy =	1.56	mm
	Check =	Ok	

At fatigue limit state $hs \ge 2.0.hmax.\sigma L/\Delta F_{TH}$

Constant amplitude fatigue threshold for Category A as specified in Article 6.6	ΔF_{TH} =	165.0	Mpa
Service average compressive stress due to live load	σL =	2.55	Mpa
	$2.0.hmax.\sigma L/\Delta F_{TH}$ =	0.58	mm
	Check =	Ok	

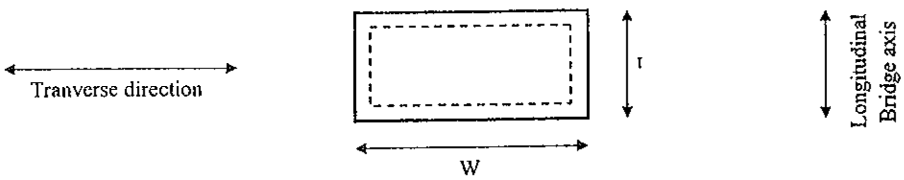
h.Seismic Provisions S.14.7.5.3.8

Elastomeric expansion bearings shall be provided with adequate seismic resistant anchorage to resist the horizontal forces in excess of those accommodated by shear in the pad. The sole plate and the base plate shall be made wider to accommodate the anchor bolts. The anchor bolts shall be designed for the combined effect of bending and shear for seismic loads as specified in Article 14.6.5.3. Elastomeric fixed bearings shall be provided with horizontal restraint adequate for the full horizontal load.

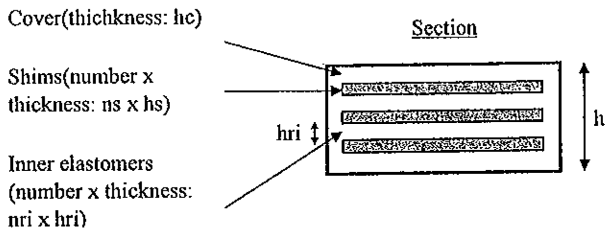
STEEL-REINFORCED ELASTOMERIC BEARING - design checking

Method B - S.14.7.5

Plan view



Section



1. Materials

The elastomer shall have a shear modulus between 0.60 and 1.3 MPa and a nominal hardness between 50 and 60 on the Shore A scale. It shall conform to the requirements of Section 18.2 of the AASHTO LRFD Bridge Construction

The shear modulus of the elastomer at 23°C shall be used as the basis for design.

Item	Hardness (Shore A)		
	50	60	70
Shear modulus @ 23°C (MPa)	0.66-0.90	0.90-1.38	1.38-2.07
Creep deflection @25 years divided by instantaneous deflection	0.25	0.35	0.45

Choose Shear modulus for elastomer material at 23°C G = 1.00 Mpa

Yield strength of shims plate and soles steel (ASTM A709M grade 250) Fy = 250.0 Mpa

2.Load

Bearing type:

"1: bearing subject to shear deformation (moveable bear.); 2: bearing fixed against shear deformation (fixed bear.) 1

Design force on bearing

Combination	Max. Factored Reaction		Rotation		Horizontal movement Δ_0(mm)
	All (kN)	Live load (kN)	θs Long. (rad)	θs Trans. (rad)	
Strength 1	1585.0	701.6			
Service 1	1094.8	400.9	0.011	0.000	10.9

*Dynamic Impact load included in design for conservative approach

3.Design checking

a. Bearing configuration S.14.7.5.1

In any elastomeric bearing layer, the average compressive stress at the service limit state shall satisfy:

Item		Sign	Unit	Value	Shape factor	Check.*
Dimensions	Longitudinal axis of bridge	L	mm	350		
	Tranverse axis	W	mm	450		
	Height	h	mm	84		
Shims	Number of shim	ns	shims	5		
	Thickness	ts	mm	3		
	Longitudinal axis of bridge	Ls	mm	340		
	Tranverse axis	Ws	mm	440		
Cover	Top thickness	hct	mm	2.5	39.36	Ok
	Bottom thickness	hcb	mm	2.5	39.36	Ok
Inner elastomer layers						
	Number layers	nr	layers	4		
	Layer thickness	hri	mm	16.0	6.15	
Check total height of bearing		Ok	mm	84		

Shape factor of an elastomer layer $Si = L.W/[2.hri.(L+W)]$ $L.W/[2(L+W)] =$ 98.4

* To ensure that top or bottom cover elastomer layer is not thicker than 70% of inner elastomer layer

b. Compressive Stress   S.14.7.5.3.2

In any elastomeric bearing layer, the average compressive stress at the service limit state shall satisfy:

Service average compressive stress due to the total load		$\sigma s =$	6.95 Mpa
Service average compressive stress due to live load		$\sigma L =$	2.55 Mpa
Shape factor of the thickest layer of the bearing		$S =$	6.15
For bearings subject to shear deformation	$\sigma s \le 1.66.G.S \le 11.0$ Mpa	$1.66.G.S =$	10.21 Mpa
		Check =	Ok
	$\sigma L \le 0.66.G.S$	$0.66.G.S =$	4.06 Mpa
		Check =	Ok
For bearings fixed against shear deformation	$\sigma s \le 2.00.G.S \le 12.0$ Mpa	$2.00.G.S =$	- Mpa
		Check =	-

c. Compressive Deflection   S.14.7.5.3.3

Only when deck joints or seals are present on the bridge: "1: check, 2: not check"   1

Instantaneous deflection shall be taken $\delta = \Sigma \varepsilon i.\ hri \le 3mm$

A maximum relative deflection across a joint of 3 mm is suggested

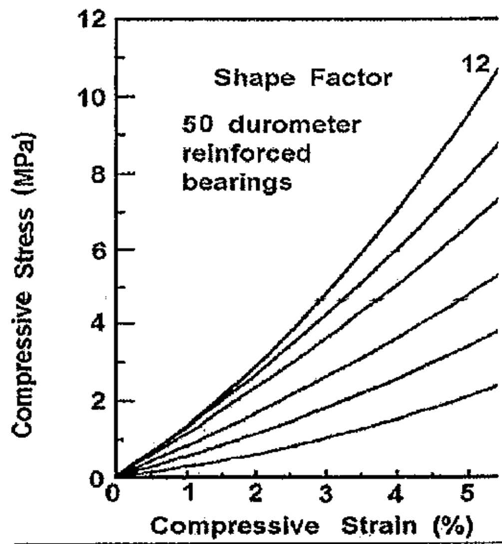
Instantaneous compressive strain in i[th] elastomer layer of a laminated bearing εi

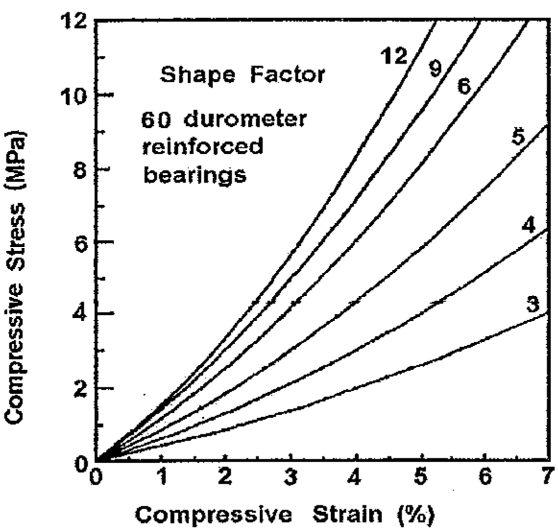
In the absence of information specific to the particular elastomer to be used, Figure below may be used as a guide.

Creep defle

Creep effec

Thickness (





No	Elastomer Layer	Number	σL (Mpa)	S	hri (mm)	εi (%)	$\varepsilon i.hri.ni$ (mm)
1	Top cover	1	2.55	6.15	2.5	2.5	0.06
2	Bottom cover	1	2.55	6.15	2.5	2.5	0.06
3	Inner layers	4	2.55	6.15	16.0	2.5	1.60
4							
5							
						$\delta cr =$	0.60
	Check	Ok				$\delta + \delta cr =$	2.33

d. Shear Deformation   S.14.7.5.3.4

Only for moveable bearing (For bearings subject to shear deformation)

The maximum shear deformation of the bearing, at the service limit state, ΔS, shall be taken as Δo, modified to account for the substructure stiffness and construction procedures.

If a low friction sliding surface is installed, DSneed not be taken to be larger than the deformation corresponding to first slip.

The horizontal movement of the bridge superstructure $\Delta_0 = \Delta cr\&sh + \Delta ps + \Delta temp$ $\Delta_0 =$ 10.9 mm

(extreme displacement caused by creep, shrinkage, post-tensioning, combined with thermal effects)

Total elastomer thickness		hrt =	69 mm
		$2.\Delta s =$	21.87 mm
The bearing shall satisfy	$hrt \ge 2.\Delta s$	Check =	Ok

e.Combined Compression & Rotation S.14.7.5.3.5

The provisions of this section shall apply at the service limit state.

The goal is to prevent uplift of any corner of the bearing under any combination of loading and corresponding rotation.

Rectangular bearings may be taken to satisfy uplift requirements if they satisfy:

$\sigma s > 1.0.G.S\ (\theta s\ /n).\ (B\ /hri)^2 = F1$

Rectangular bearings subjected to shear deformation shall also satisfy:

$\sigma s < 1.875.G.S\ [1\text{-}0.2(\theta s\ /n).\ (B\ /hri)^2] = F2$

Rectangular bearings fixed against shear deformation shall also satisfy:

$\sigma s < 2.25.G.S\ [1\text{-}0.167(\theta s\ /n).\ (B\ /hri)^2] = F3$

Number of interior layers of elastomer.	n =	4 layers
Stress in elastomer	σs =	6.95 Mpa
Length of pad if rotation is about its transverse axis or width of pad if rotation is about its longitudir	B	
Maximum service rotation due to the total load about long. Or trans. axis	θs	

No	Direction	B (mm)	hri (mm)	θs (rad)
1	Longitudinal rotation	350.0	16.0	0.011
2	Transverse rotaiton	450.0	16.0	0.000

No	Direction	F1 (Mpa)	Check F1	F2 (Mpa)	Check F2	F3 (Mpa)	Check F3
1	Longitudinal rotation	8.3	N.G	8.4	Ok	-	-
2	Transverse rotaiton	0.0	Ok	11.5	Ok	-	-

f.Stability of Elastomeric Bearings S.14.7.5.3.6

Bearings satisfying equation here shall be considered stal $2A \leq B$

Where	A = (1.92.hrt/L)/sqrt(1+2.0.L/ W)	A =	0.237
	B = 2.67/ [(S+2.0) . (1+L/(4.0.W))]	B =	0.274
		2A =	0.474
		Check =	Need check

For a rectangular bearing where L is greater than W, stability shall be investigated by interchanging L and W

For a rectangular bearing not satisfying 2A≤B, the stress due to total load shall be satify below equations:

If the bridge deck is free to translate horizontally	σs ≤ G.S/ (2A-B)	G.S/ (2A-B) =	30.9 Mpa
		Check =	Stable
If the bridge deck is fixed against horizontal translation	σs ≤ G.S/ (A-B)	(A-B) =	-0.037
		G.S/ (A-B) =	-
If the value A−B≤0, the bearing is stable and is not dependent on σs.		Check =	Stable

g.Reinforcement S.14.7.5.3.7

The thickness of the steel reinforcement, hs, shall satisfy the provisions of Article 14.7.5.3.7 of the AASHTO LRFD Bridge Construction Specifications and:

At service limit state	hs ≥ 3.hmax.σs/ Fy		
Thickness of thickest elastomeric layer in elastomeric bearing		hmax =	16.0 mm
Service average compressive stress due to total load		σs =	6.95 Mpa
Yield strength of steel reinforcement		Fy =	250.0 Mpa
		hs =	3.0 mm
		3.hmax.σs/ Fy =	1.33 mm
		Check =	Ok

At fatigue limit state	$hs \geq 2.0.hmax.\sigma L/ \Delta F_{TH}$		
Constant amplitude fatigue threshold for Category A as specified in Article 6.6		ΔF_{TH} =	165.0 Mpa
Service average compressive stress due to live load		σL =	2.55 Mpa
		$2.0.hmax.\sigma L/ \Delta F_{TH}$ =	0.49 mm
		Check =	Ok

h.Seismic Provisions S.14.7.5.3.8

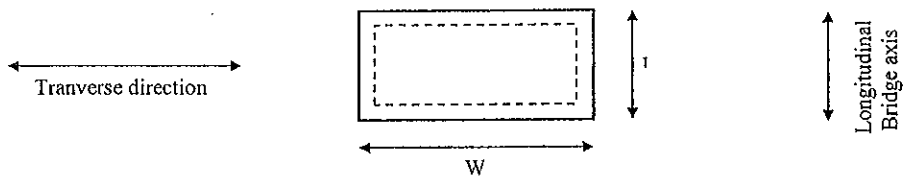
Elastomeric expansion bearings shall be provided with adequate seismic resistant anchorage to resist the horizontal forces in excess of those accommodated by shear in the pad. The sole plate and the base plate shall be made wider to accommodate the anchor bolts. The anchor bolts shall be designed for the combined effect of bending and shear for seismic loads as specified in Article 14.6.5.3. Elastomeric fixed bearings shall be provided with horizontal restraint adequate for the full horizontal load.

22TCN272-05; AASHTO LRFD 3rd - 2004

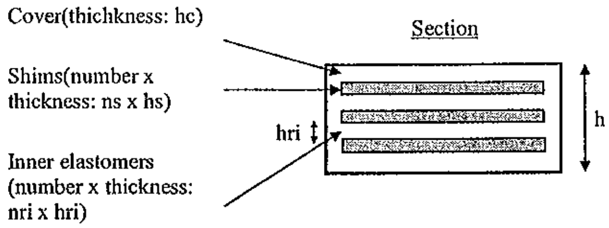
STEEL-REINFORCED ELASTOMERIC BEARING - design checking

Method B - S.14.7.5

Plan view



Section



1. Materials

The elastomer shall have a shear modulus between 0.60 and 1.3 MPa and a nominal hardness between 50 and 60 on the Shore A scale. It shall conform to the requirements of Section 18.2 of the AASHTO LRFD Bridge Construction

The shear modulus of the elastomer at 23°C shall be used as the basis for design.

Item	Hardness (Shore A)		
	50	60	70
Shear modulus @ 23°C (MPa)	0.66-0.90	0.90-1.38	1.38-2.07
Creep deflection @25 years divided by instantaneous deflection	0.25	0.35	0.45

Choose Shear modulus for elastomer material at 23°C G = 1.00 Mpa

Yield strength of shims plate and soles steel (ASTM A709M grade 250) Fy = 250.0 Mpa

2.Load

Bearing type:

"1: bearing subject to shear deformation (moveable bear.); 2: bearing fixed against shear deformation (fixed bear.)" 1

Design force on bearing

Combination	Max. Factored Reaction		Rotation		Horizontal movement
	All (kN)	Live load (kN)	θs Long. (rad)	θs Trans. (rad)	Δ_0(mm)
Strength 1	1796.5	814.9			
Service 1	1236.7	465.7	0.007	0.000	26.7

*Dynamic Impact load included in design for conservative approach

3.Design checking

a. Bearing configuration S.14.7.5.1

In any elastomeric bearing layer, the average compressive stress at the service limit state shall satisfy:

Item		Sign	Unit	Value	Shape factor	Check.*
Dimensions	Longitudinal axis of bridge	L	mm	350		
	Tranverse axis	W	mm	500		
	Height	h	mm	84		
Shims	Number of shim	ns	shims	5		
	Thickness	ts	mm	3		
	Longitudinal axis of bridge	Ls	mm	340		
	Tranverse axis	Ws	mm	490		
Cover	Top thickness	hct	mm	2.5	41.16	Ok
	Bottom thickness	hcb	mm	2.5	41.16	Ok
Inner elastormer layers						
	Number layers	nr	layers	4		
	Layer thickness	hri	mm	16.0	6.43	
Check total height of bearing		Ok	mm	84		

Shape factor of an elastomer layer	Si = L.W/ [2.hri.(L+W)]	L.W/ [2(L+W)] =	102.9

* To ensure that top or bottom cover elastomer layer is not thicker than 70% of inner elastomer layer

b. Compressive Stress S.14.7.5.3.2

In any elastomeric bearing layer, the average compressive stress at the service limit state shall satisfy:

Service average compressive stress due to the total load		σs =	7.07 Mpa
Service average compressive stress due to live load		σL =	2.66 Mpa
Shape factor of the thickest layer of the bearing		S =	6.43
For bearings subject to shear deformation	σs ≤ 1.66.G.S ≤ 11.0 Mpa	1.66.G.S =	10.67 Mpa
		Check =	**Ok**
	σL ≤ 0.66.G.S	0.66.G.S =	4.24 Mpa
		Check =	**Ok**
For bearings fixed against shear deformation	σs ≤ 2.00.G.S ≤ 12.0 Mpa	2.00.G.S =	- Mpa
		Check =	-

c. Compressive Deflection S.14.7.5.3.3

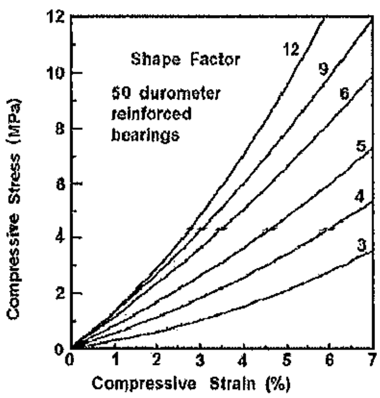
Only when deck joints or seals are present on the bridge: "1: check, 2: not check" 1

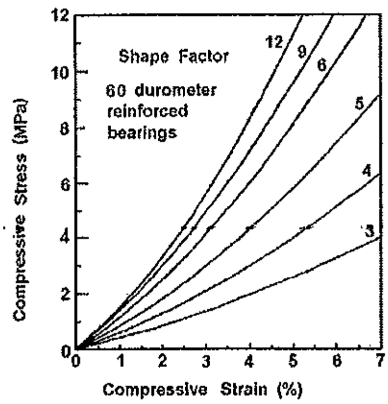
Instantaneous deflection shall be taken δ=Σεi. hri ≤ 3mm

A maximum relative deflection across a joint of 3 mm is suggested

Instantaneous compressive strain in i[th] elastomer layer of a laminated bearing εi

In the absence of information specific to the particular elastomer to be used, Figure below may be used as a guide.





Creep deflection factor		ϕcr =	0.35
Creep effect to instantaneous load	δcr =δ.ϕcr		
Thickness of i[th] elastomeric layer in a laminated bearing		hri	

No	Elastomer Layer	Number	σL (Mpa)	S	hri (mm)	εi (%)	εi. hri.ni (mm)
1	Top cover	1	2.66	6.43	2.5	2.5	0.06
2	Bottom cover	1	2.66	6.43	2.5	2.5	0.06
3	Inner layers	4	2.66	6.43	16.0	2.5	1.60
4							
5							
						δcr =	0.60
	Check	Ok				δ + δcr =	2.33

d. Shear Deformation S.14.7.5.3.4

Only for moveable bearing (For bearings subject to shear deformation)

The maximum shear deformation of the bearing, at the service limit state, ΔS, shall be taken as Δo, modified to account for the substructure stiffness and construction procedures.

If a low friction sliding surface is installed, DSneed not be taken to be larger than the deformation corresponding to first slip.

The horizontal movement of the bridge superstructure Δ_0=Δcr&sh +Δps + Δtemp Δ_0= 26.7 mm

(extreme displacement caused by creep, shrinkage, post-tensioning, combined with thermal effects)

Total elastomer thickness		hrt =	69 mm
		2.Δs =	53.46 mm
The bearing shall satisfy	hrt ≥ 2.Δs	Check =	**Ok**

c.Combined Compression & Rotation S.14.7.5.3.5

The provisions of this section shall apply at the service limit state.

The goal is to prevent uplift of any corner of the bearing under any combination of loading and corresponding rotation.

Rectangular bearings may be taken to satisfy uplift requirements if they satisfy:

$$\sigma s > 1.0.G.S\ (\theta s\ /n).\ (B\ /hri)^2 = F1$$

Rectangular bearings subjected to shear deformation shall also satisfy:

$$\sigma s < 1.875.G.S\ [1\text{-}0.2(\theta s\ /n).\ (B\ /hri)^2] = F2$$

Rectangular bearings fixed against shear defomation shall also satisfy:

$$\sigma s < 2.25.G.S\ [1\text{-}0.167(\theta s\ /n).\ (B\ /hri)^2] = F3$$

Number of interior layers of elastomer.	n =	4 layers
Stress in elastomer	σs =	7.07 Mpa
Length of pad if rotation is about its transverse axis or width of pad if rotation is about its longitudir	B	
Maximum service rotation due to the total load about long. Or trans. axis	θs	

No	Direction	B (mm)	hri (mm)	θs (rad)
1	Longitudinal rotation	350.0	16.0	0.007
2	Transverse rotaiton	500.0	16.0	0.000

No	Direction	F1 (Mpa)	Check F1	F2 (Mpa)	Check F2	F3 (Mpa)	Check F3
1	Longitudinal rotation	5.7	**Ok**	9.9	**Ok**	-	-
2	Transverse rotaiton	0.0	**Ok**	12.1	**Ok**	-	-

f.Stability of Elastomeric Bearings S.14.7.5.3.6

Bearings satisfying equation here shall be considered sta $2A \leq B$

Where

	$A = (1.92.hrt/L)/sqrt(1+2.0.L/W)$	A =	0.244
	$B = 2.67/[(S+2.0).(1+L/(4.0.W))]$	B =	0.270
		2A =	0.489
		Check =	Need check

For a rectangular bearing where L is greater than W, stability shall be investigated by interchanging L and W

For a rectangular bearing not satisfying $2A \leq B$, the stress due to total load shall be satify below equations:

If the bridge deck is free to translate horizontally	$\sigma s \leq G.S/(2A\text{-}B)$	G.S/ (2A-B) =	29.3 Mpa
		Check =	**Stable**
If the bridge deck is fixed against horizontal translation	$\sigma s \leq G.S/(A\text{-}B)$	(A-B) =	-0.025
		G.S/ (A-B) =	-
If the value $A-B \leq 0$, the bearing is stable and is not dependent on σs.		Check =	**Stable**

g.Reinforcement S.14.7.5.3.7

The thickness of the steel reinforcement, hs, shall satisfy the provisions of Article 14.7.5.3.7 of the AASHTO LRFD Bridge Construction Specifications and:

At service limit state	$hs \geq 3.hmax.\sigma s/Fy$		
Thickness of thickest elastomeric layer in elastomeric bearing		hmax =	16.0 mm
Service average compressive stress due to total load		σs =	7.07 Mpa
Yield strength of steel reinforcement		Fy =	250.0 Mpa
		hs =	3.0 mm
		$3.hmax.\sigma s/Fy$ =	1.36 mm
		Check =	**Ok**

At fatigue limit state	$hs \geq 2.0.hmax.\sigma L/\Delta F_{TH}$		
Constant amplitude fatigue threshold for Category A as specified in Article 6.6		ΔF_{TH} =	165.0 Mpa
Service average compressive stress due to live load		σL =	2.66 Mpa
		$2.0.hmax.\sigma L/\Delta F_{TH}$ =	0.52 mm
		Check =	**Ok**

h.Seismic Provisions S.14.7.5.3.8

Elastomeric expansion bearings shall be provided with adequate seismic resistant anchorage to resist the horizontal forces in excess of those accommodated by shear in the pad. The sole plate and the base plate shall be made wider to accommodate the anchor bolts. The anchor bolts shall be designed for the combined effect of bending and shear for seismic loads as specified in Article 14.6.5.3. Elastomeric fixed bearings shall be provided with horizontal restraint adequate for the full horizontal load.

CALCULATION SHEET

PARAPET

CALCULATION SHEET FOR OUTSIDE BARRIER OF THRUWAY 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		*3.00*	*(lanes)*
Design earthquake class	*Class*	*VII*	

2. Bridge width

Width of carriageway	B_{CAR} =	*11.74*	*(m)*
Width of barrier wall	B_{lc} =	*0.50*	*(m)*
Bridge width	B =	*12.74*	*(m)*

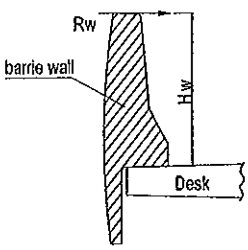
3. Material properties:

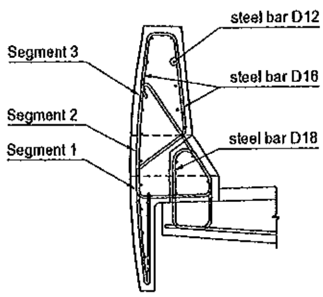
Concrete			
Compressive strength of cylindrical at 28 days age	$f'c$ =	*25.00*	*MPa*
Concrete density	g =	*24.50*	*KN/m3*
Elastic modulus	E_c =	25278.73	*MPa*
Tensile strength of concrete	f_r =	*3.15*	*MPa*
Steel CB-400-V			
Modulus of elasticity	E_s =	*200000*	*MPa*
Yield strength of steel bar	f_y =	*400*	*MPa*

4. Dimensions of RC barrier wall:

b1	*500*	*(mm)*
b2	*325*	*(mm)*
b3	*250*	*(mm)*
h1	*159*	*(mm)*
h2	*255*	*(mm)*
h3	*740*	*(mm)*

5. Diagram of Calculation





6. Railing shall be proportioned such that:

$$\boxed{R \geq Ft} \quad (13.7.3.3\text{-}1)$$

In which:

R - Total resistance of the barrier wall

Ft - Transverse vehicle impact force

8. General value:

- Diameter of longitual 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	*550 (KN)*	(AASHTO2007 Table 13.2-1)
He(min)	*1070*	

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 As	$a=\frac{A_s.f_y}{0,85.f'_c.b}$	$\Phi.Mn(+)$ $\Phi.A_s.f_y\left(d-\frac{a}{2}\right)$
	(mm)	(Bar)	(mm)	(mm^2)	(mm)	(KNmm)
1	159	1	183	154	18	10707.18
2	255	1	258	154	11	15536.56
3	740	4	433	616	16	104719.37

+ Mw for Int-face

Segment	Width of Segment $b' = h$	Number of bars n	Effective Depth $d(+)$	Area of bars As	$a=\frac{A_s.f_y}{0,85.f'_c.b}$	$\Phi.Mn(-)$ $\Phi.A_s.f_y\left(d-\frac{a}{2}\right)$
	(mm)	(Bar)	(mm)	(mm2)	(mm)	(KNmm)
1	159	1	183	154	18	10707.18
2	255	1	248	154	11	14920.80
3	740	4	423	616	16	102256.36

+ Resistance of concrete wall for vertical axial (Mw.H)

Segment	Width of Segment $b' = h$	$\Phi.Mn(+)$ Out-face	$\Phi.Mn(-)$ Int-face	$\Phi.Mni$ Average of two face	Mw.H $\sum \Phi.M\,ni$
	(mm)	(KNmm)	(KNmm)	(KNmm)	(KNmm)
1	159	10707.18	10707.18	10707.18	129423.73
2	255	15536.56	14920.80	15228.68	
3	740	104719.37	102256.36	103487.86	

Where:

d - Average distance from compression face to centroid of tension reinforcement (mm)

a - Thickness of the equivalent stress block (mm)

As - 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) $\boxed{As = \frac{\pi.\Phi^2}{4*D}}$ *(with D is Reinf. Spacing of shear) and b =1 m*

Segment	Hight of Segment h	Shear contact area As	Effective Depth d	$a=\frac{A_s.f_y}{0,85.f'_c.b}$	Mci $\Phi.A_s.f_y\left(d-\frac{a}{2}\right)$	Mc $\frac{\sum \Phi.M.hi}{\sum hi}$
	(mm)	(mm2/mm)	(mm)	(mm)	(KNmm)	(KNmm)
1	159	2.094	310	39.42	243.191	211.628
2	255	2.094	139.5	39.42	100.353	
3	740	2.094	310	39.42	243.191	

+ Total ultimate resistance of RC barrier wall:

- For impacts within a wall segment :

$$R_w = \left(\frac{2}{2.L_c - L_t}\right)\left(8.M_b + 8.M_w.H + \frac{M_c.L_c^2}{H_w}\right)$$ (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 :

$$L_c = \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + \frac{8.H_w.(M_b + M_w.H)}{M_c}}$$ (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	1154	0	211.63	129423.73	3891	1427.12

- For impacts at end of wall or at joint :

$$R_w = \left(\frac{2}{2.L_c - L}\right)\left(M_b + M_w.H + \frac{M_c.L_c^2}{H_w}\right)$$ (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	1154	0	211.63	129423.73	2701	990.75

9. RESISTANCE CHECK FOR RC BARRIER WALL

- Condition 1 *R = R w≥ Ft* *With : Ft = 550* *(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	990.75	550	**OK**
2. Impact at a wall segment	1427.12	550	**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 (m2)*

+ Load due to weight of wall $DC_c = \gamma_c.A_c$

$DC_{lc} =$ *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 Rw spreads out at a 1:1 slope from Lc

- The tensile force per unit of length in the overhang, is given by:

$$T = \frac{R_{max}}{L_c + 2.H_w}$$

- Height of barrier	$Hw =$	*1154 (mm)*
- Maximum of load impact on barrier wall	$Rmax =$	*1427.12 (KN)*
	$Lc =$	*3891 (mm)*
	$T_1 =$	*230.22 (N/mm)*
- For Impact at end of barrier wall	$Rmax =$	*990.75 (KN)*
	$Lc =$	*2701 (mm)*
	$T_2 =$	*197.78 (N/mm)*
- Shear load for calculate	$T = Max(T_1,T_2)$	
	$T =$	*230.22 (N/mm)*

- The nominal shear resistance Vn of the interface plane following:

$$V_n = c.A_{cv} + Õ.(A_{vf}.f_y + P_c)$$

Which shall not exceed $0.2f'_c$ or $5.5A_{cv}$

Where:

- Shear contact area:	$Acv = b_1.1\ mm$		
	$Acv =$	*500 (mm2/mm)*	
- Dowel area across shear plane:	$A_{vf} = \frac{\pi.\Phi^2}{4 * D}$		*(Determined in 9.2.2)*
	$Avf =$	*2.094 (mm2/mm)*	
- Yield strenght of reinforcement	$fy =$	*400 (MPa)*	
- Permanent compressive force:	$Pc = DC_{lc}.1\ mm$		
	$Pc =$	*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]
	$Vn =$	*770.43 (N/mm)*	
	$0.2f'_c.A_{cv} =$	*2500 (N/mm)*	
	$5.5A_{cv} =$	*2750 (N/mm)*	

- Nominal shear resistance: *Vn = Min(Vn , 0.2f'c.Acv , 5.5Acv)*

Vn = *770.43 (N/mm)* ***> T = 230.22 (N/mm) : OK***

+ The minimum cross-sectional area of dowels across the shear plane:

$$A'_{vf} = 0.35\frac{b_1.s}{f_y}$$ *[5.8.4.1 - 22TCN 272-05]*

$A'_{vf} =$ *65.63 (mm2)*

- Number of stirrup input deck $n =$ *2*

- Cross-sectional area of stirrup input deck

$As = n.Avf.s$

$As =$ 628.32 (mm2) $> A'vf$: OK

- The development length l_n shall not less than 3 values then:

$\frac{100.\Phi}{\sqrt{30}} =$ 365 (mm) With Φ =20 (mm)

$8.\Phi =$ 160 (mm)

And 150 (mm)

- The development length: $l_n =$ 365 (mm) *(The required modify)*
- Modification factor for adequate cover: $k_1 =$ 0.7

$l'_n = k_1.l_n$

$l'_n =$ 256 (mm)

- The development length after modify: $l_n =$ 256 (mm)
- The Available development length: $l_c = hf$ Ô $as(+)$

$l_c =$ 160 *mm (which is not adequate)*

- Unless the required area is reduced to $A_{vf}(hc) = A_{vf}.l_c / l_n$

$A_{vf}(hc) =$ 1.311 (mm2)

By using this area to recalculate Mc, Lc, Rw (The determined following 5.2.2)

Segment	Height of Segment h	Shear contact area As	Effective Depth d	$a=\frac{A_s.f_y}{0,85.f'_c.b}$	Mci $\Phi.A_s.f_y\left(d-\frac{a}{2}\right)$	Mc $\frac{\sum \Phi.M.hi}{\sum hi}$
	(mm)	(mm2/mm)	(mm)	(mm)	(KNmm)	(KNmm)
1	159	1.311	310	24.68	156.096	136.339
2	255	1.311	139.5	24.68	66.684	
3	740	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	1154	0	136.34	129423.73	4422	1044.85

- 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	1154	0	136.34	129423.73	2827	668.09

+ Check railing following Condition 1

Combination	Resistance of Wall		Check
	Rw (KN)	R (KN)	Condition (1)
1. Impact at end of wall or joint	668.09	550.00	**OK**
2. Impact at a wall segment	1044.85	550.00	**OK**

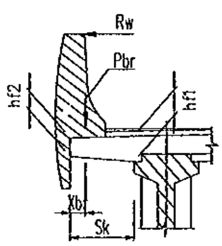
12.Ovehang of deck

Over hang length Sk= 750 mm

Thickness of overhang (inside) hf1= 265 mm

Thickness of overhang (outside) hf2= 280 mm

Thickness of wearing surface ha= 84 mm



The top reinforcement must resist the negative bending moment over the exterior beam due to the collision and the deadload of overhang:

$$M_{CT} = \frac{R_w.H_w + R_r.H_r}{L_c + 2.H_w}$$

In which $Hw =$ 1154 (mm)

Combination	Resistance of barrier Rw (KN)	Effect length Lc (mm)	Momen (M_{CT}) (KNmm/mm)
1. Impact at end of wall or joint	668.09	2701	153.91
2. Impact at a wall segment	1044.85	3891	194.51

- The Collision moment: $M_{CT} =$ 153.91 (KNmm/mm)

Combination momen due to dead load and collision

$Mu = 1.Ù\ 1.25M_{DC} + 1.5M_{DW} + 1.0M_{CT}\ Ù$

In which:

Momen due to barrier and overhang $M_{DC} = DC_{lc}.(S_k - X_{lc}) + \gamma_c.h_f.S_k^2$

Distance from centre of barrier to edge of overhang $X_{lc} =$ 245 (mm)

$M_{DC} =$ 8.42 (KNmm/mm)

Momen due to wearing surface

$$M_{DW} = \begin{cases} 0 : (S_k - b_1) \le 0 \\ (\gamma_a.h_a + \gamma_p.h_p).(S_k - b_1)^2 \end{cases}$$

$M_{DW} =$ 0.49 (KNmm/mm)

Momen due to Deadload and collision load for the Extreme event limit state:

$Mu =$ 165.18 (KNmm/mm)

$Mu =$ 165179 (Nmm/mm)

Thickness of cover: as(-) = 50 (mm)

Reinforcement: D20@150

$As =$ 4.189 (mm2/mm)

Distance from compression face to centroid of tension reinforcement (mm)

ds(Ô) = hf - as(-)

ds(Ô) = 215 (mm)

Thickness of the equivalent stress block (mm)

$$a = \frac{A_s . f_y}{0.85\ f_c'.b}$$

$a =$ 78.85 (mm)

Blending moment factor Þ = 1.0

Moment resistance: $\Phi.Mn = \Phi.As.fy.[\ ds(-) - a(-)/2\ ]$

$\Phi.Mn =$ 294181 (Nmm/mm)

This moment strength will be reduced because of the axial tension force:

$$T = \frac{R_{max}}{L_c + 2.H_w}$$

Barrier height: $Hw =$ 1154 (mm)

For impact at a wall segment

$Rmax =$ 1044.85 (KN)

$Lc =$ 3891 (mm)

$T_1 =$ 168.55 (N/mm)

For impact at end of wall or joint

$Rmax =$ 668.09 (KN)

$Lc =$ 2701 (mm)

$T_2 =$ 133.37 (N/mm)

The shear were used is: $T = Max(T_1, T_2)$

$T =$ 168.55 (N/mm)

By assuming the interaction curve between moment and axial tension is a straight line:

$$\frac{P_u}{\Phi.P_n} + \frac{M_u}{\Phi.M_n} \leq 1 \qquad (*)$$

In which: $Pu = T$

$Pu =$ 168.55 (N/mm)

$\Phi.Pn = \Phi.As.fy$

$\Phi.Pn =$ 3351.03 (N/mm)

To find out:

$$M_u \leq \Phi.M_n.\left(1 - \frac{P_u}{\Phi.P_n}\right)$$

$Mu \leq$ 279384 (Nmm/mm)

With $Mu =$ 165179 (Nmm/mm) *(OK)*

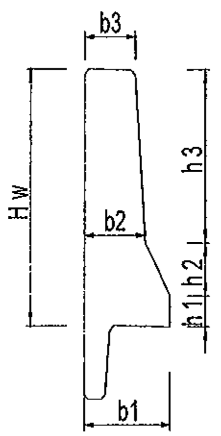
CALCULATION SHEET FOR MEDIAN BARRIER OF THRUWAY BRIDGES

- Bridge Design Standard 22 TCN - 272 - 05 (considered with AASHTO LRFD 2007)

A. GENERAL DATA:

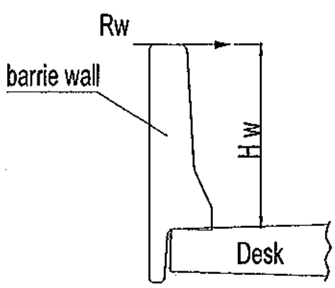
1. Design live load			
Design vehicle load		*HL93*	
Number of lanes		*3.00*	*(lanes)*
Design earthquake class	*Class*	*VII*	
2. Bridge width			
Width of carriageway	B_{CAR} =	*11.74*	*(m)*
Width of barrier wall	B_{lc} =	*0.50*	*(m)*
Bridge width	B =	*12.74*	*(m)*
3. Material properties:			
Concrete			
Compressive strength of cylindrical at 28 days age	$f'c$ =	*25.00*	*MPa*
Concrete density	g =	*24.50*	*KN/m3*
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*

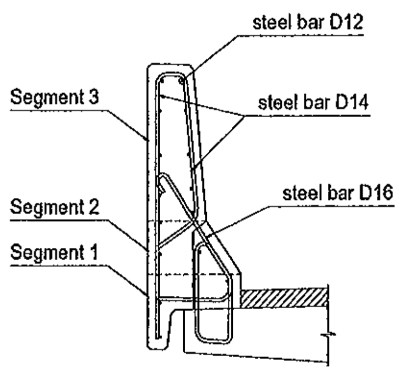
4. Dimensions of RC median barrier wall:



b1	*490*	*(mm)*
b2	*315*	*(mm)*
b3	*240*	*(mm)*
h1	*159*	*(mm)*
h2	*255*	*(mm)*
h3	*740*	*(mm)*

5. Diagram of Calculation





6. Railing shall be proportioned such that:

$\boxed{R \geq Ft}$ (13.7.3.3-1)

In which:

R - Total resistance of the barrier wall

Ft - Transverse vehicle impact force

7. General value:

- Diameter of longitual steel bar 14 (mm)
- Diameter of stirrup 20 (mm)
- Reinf. Spacing of stirrup 150 (mm)
- Φ Bending resistance factor 1

7.1 Choose Design force for barrier wall :

- **Barrier wall containment level:**

Ft	550 (KN)	(AASHTO2007 Table 13.2-1)
He(min)	1070	

7.2 Total capacity of Barrier wall:

7.2.1. 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 As	$a = \frac{A_s.f_y}{0{,}85.f'_c.b}$	Φ.Mn(+) $\Phi.A_s.f_y\left(d-\frac{a}{2}\right)$
	(mm)	(Bar)	(mm)	(mm^2)	(mm)	(KNmm)
1	159	1	423	154	18	25485.24
2	255	1	248	154	11	14920.80
3	740	5	173	770	20	50248.65

+ Mw for Int-face

Segment	Width of Segment $b' = h$	Number of bars n	Effective Depth $d(+)$	Area of bars As	$a = \frac{A_s.f_y}{0{,}85.f'_c.b}$	Φ.Mn(-) $\Phi.A_s.f_y\left(d-\frac{a}{2}\right)$
	(mm)	(Bar)	(mm)	(mm2)	(mm)	(KNmm)
1	159	1	413	154	18	24869.48
2	255	1	248	154	11	14920.80
3	740	5	173	770	20	50248.65

+ Resistance of concrete wall for vertical axial (Mw.H)

Segment	Width of Segment $b' = h$	Φ.Mn(+) Out-face	Φ.Mn(-) Int-face	Φ.Mni Average of two face	Mw.H $\sum \Phi.M ni$
	(mm)	(KNmm)	(KNmm)	(KNmm)	(KNmm)
1	159	25485.24	24869.48	25177.36	90346.81
2	255	14920.80	14920.80	14920.80	
3	740	50248.65	50248.65	50248.65	

Where:

d - Average distance from compression face to centroid of tension reinforcement (mm)

a - Thickness of the equivalent stress block (mm)

As - Area of tension reinforcement (mm2)

7.2.2. *Transverse Ultimate resistance of wall (Mc)*

+ Transverse resistance of RC barrier wall (Mc)

Shear contact area: (mm2/mm) $\boxed{As = \frac{\pi.\Phi^2}{4 * D}}$ *(with D is Reinf. Spacing of shear)*

and b =1 m

Segment	Height of Segment h	Shear contact area As	Effective Depth d	$a=\frac{A_s.f_y}{0,85.f'_c.b}$	Mci $\Phi.A_s.f_y\left(d-\frac{a}{2}\right)$	Mc $\frac{\sum \Phi.M.hi}{\sum hi}$
	(mm)	(mm2/mm)	(mm)	(mm)	(KNmm)	(KNmm)
1	159	2.094	300	39.42	234.814	177.862
2	255	2.094	210	39.42	159.415	
3	740	2.094	225	39.42	171.982	

+ Total ultimate resistance of RC barrier wall:

<u>- For impacts within a wall segment :</u>

$$\boxed{R_w=\left(\frac{2}{2.L_c-L_t}\right)\left(8.M_b+8.M_w.H+\frac{M_c.L_c^2}{H_w}\right)}$$ (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 :

$$\boxed{L_c=\frac{L_t}{2}+\sqrt{\left(\frac{L_t}{2}\right)^2+\frac{8.H_w.(M_b+M_w.H)}{M_c}}}$$ (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	1154	0	177.86	90346.81	3706	1142.24

<u>- For impacts at end of wall or at joint :</u>

$$\boxed{R_w=\left(\frac{2}{2.L_c-L}\right)\left(M_b+M_w.H+\frac{M_c.L_c^2}{H_w}\right)}$$ (TCN 13.7.3.4-1)

ỏ Critical length of yield line failure pattern Lc :

$$\boxed{L_c=\frac{L_t}{2}+\sqrt{\left(\frac{L_t}{2}\right)^2+\frac{H_w.(M_b+M_w.H)}{M_c}}}$$

Lt (mm)	Hw (mm)	Mb (KN.mm)	Mc (KN.mm/mm)	Mw.H (KN.mm)	Lc (mm)	Rw (KN)
2440	1154	0	177.86	90346.81	2660	820.06

8. RESISTANCE CHECK FOR RC BARRIER WALL

- Condition 1 $R = R_w \geq F_t$ *With : Ft = 550* *(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	820.06	550	OK
2. Impact at a wall segment	1142.24	550	OK

9. 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 Ac = 0.4387 (m2)

+ Load due to weight of wall $DC_c = \gamma_c.A_c$

$DC_{lc} =$ 10.75 (KN/m)

10. CHECK SHEAR-RESISTANCE OF RC AT BASE OF THE WALL JOINT WITH DECK

- Arrangement of stirrup *D20* *attach in overhang*

Assuming that Rw spreads out at a 1:1 slope from Lc

- The tensile force per unit of length in the overhang, is given by:

$$T = \frac{R_{max}}{L_c + 2.H_w}$$

- Height of barrier Hw = 1154 (mm)
- Maximum of load impact on barrier wall Rmax = 1142.24 (KN)

Lc = 3706 (mm)

$T_1 =$ 189.95 (N/mm)

- For Impact at end of barrier wall Rmax = 820.06 (KN)

Lc = 2660 (mm)

$T_2 =$ 165.06 (N/mm)

- Shear load for calculate $T = Max(T_1, T_2)$

T = 189.95 (N/mm)

- The nominal shear resistance Vn of the interface plane following:

$$V_n = c.A_{cv} + \mu.(A_{vf}.f_y + P_c)$$

Which shall not exceed $0.2f'_c$ or $5.5A_{cv}$

Where:

- Shear contact area: $Acv = b_1.1$ mm

Acv = 490 (mm2/mm)

- Dowel area across shear plane:

$$A_{vf} = \frac{\pi.\Phi^2}{4 * D}$$ *(Determined in 9.2.2)*

Avf = 2.094 (mm2/mm)

- Yield strenght of reinforcement fy = 400 (MPa)
- Permanent compressive force: $Pc = DC_{lc}.1$ mm

Pc = 10.75 (N/mm)

- Strength of weaker concrete f'c = 25 (MPa)
- Cohesion factor c = 0.52 [5.8.4.2 - 22TCN 272-05]
- Friction factor μ = 0.60 [A5.8.4.2 - 22TCN 272-05]

Vn = 763.90 (N/mm)

$0.2f'_c.A_{cv}$ = 2450 (N/mm)

$5.5A_{cv}$ = 2695 (N/mm)

- Nominal shear resistance: Vn = Min(Vn , 0.2f'c.Acv , 5.5Acv)

Vn = 763.90 (N/mm) > T = 189.95 (N/mm) : OK

+ The minimum cross-sectional area of dowels across the shear plane:

$$A'_{vf} = 0.35\frac{b_1.s}{f_y}$$ [5.8.4.1 - 22TCN 272-05]

A'_{vf} = 64.31 (mm2)

- Number of stirrup input deck n = 2
- Cross-sectional area of stirrup input deck

As = n.Avf.s

As = 628.32 (mm2) > A'vf : OK

- The development length l_n shall not less than 3 values then:

$$\frac{100\,.\Phi}{\sqrt{30}} =$$ 365 (mm) With Φ =20 (mm)

8.Φ = 160 (mm)

And 150 (mm)

- The development length: l_n = 365 (mm) (The required modify)
- Modification factor for adequate cover: k_1 = 0.7

$l'_n = k_1.l_n$

l'_n = 256 (mm)

- The development length after modify: l_n = 256 (mm)
- The Available development length: l_c = hf - as(+)

l_c = 170 mm (which is not adequate)

- Unless the required area is reduced to $A_{vf}(hc) = A_{vf}.l_c / l_n$

$A_{vf}(hc)$ = 1.393 (mm2)

By using this area to recalculate Mc, Lc, Rw (The determined following 5.2.2)

Segment	Height of Segment h	Shear contact area As	Effective Depth d	$a=\frac{A_s.f_y}{0,85.f'_c.b}$	Mci $\Phi.A_s.f_y\left(d-\frac{a}{2}\right)$	Mc $\frac{\sum \Phi.M.hi}{\sum hi}$
	(mm)	(mm2/mm)	(mm)	(mm)	(KNmm)	(KNmm)
1	159	1.393	300	26.22	159.851	121.973
2	255	1.393	210	26.22	109.704	
3	740	1.393	225	26.22	118.062	

- 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	1154	0	121.97	90346.81	4106	867.89

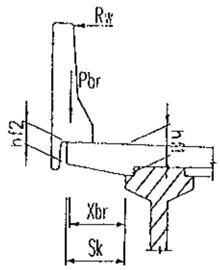
- For impacts at end of wall or joint :

Lt (mm)	Hw (mm)	Mb (KN.mm)	Mc (KN.mm/mm)	Mw.H (KN.mm)	Lc (mm)	Rw (KN)
2440	1154	0	121.97	90346.81	2751	581.48

+ Check railing following Condition 1

Combination	Resistance of Wall Rw (KN)	R (KN)	Check Condition (1)
1. Impact at end of wall or joint	581.48	550.00	OK
2. Impact at a wall segment	867.89	550.00	OK

11. *Ovehang of deck*



Over hang length	*Sk=*	*740*	*mm*
Thickness of overhang (inside)	*hf1=*	*270*	*mm*
Thickness of overhang (outside)	*hf2=*	*200*	*mm*
Thickness of wearing surface	*ha=*	*84*	*mm*

The top reinforcement must resist the negative bending moment over the exterior beam due to the collision and the deadload of overhang:

$$M_{CT} = \frac{R_w.H_w + R_r.H_r}{L_c + 2.H_w}$$

In which *Hw =* *1154 (mm)*

Combination	Resistance of barrier Rw (KN)	Effect length Lc (mm)	Momen (M_{CT}) (KNmm/mm)
1. Impact at end of wall or joint	*581.48*	*2660*	*135.06*
2. Impact at a wall segment	*867.89*	*3706*	*166.55*

- The Collision moment: M_{CT} = *166.55 (KNmm/mm)*

Combination momen due to dead load and collision

Mu = 1.Ú 1.25M_{DC} + 1.5M_{DW} + 1.0M_{CT} Ù

In which:

Momen due to barrier and overhang $M_{DC} = DC_{lc}.(S_k - X_{lc}) + \gamma_c.h_f.S_k^2$

Distance from centre of barrier to edge of overhang X_{lc} = 245 *(mm)*

M_{DC} = *6.90 (KNmm/mm)*

Momen due to wearing surface

$$M_{DW} = \begin{cases} 0 : (S_k - b_1) \le 0 \\ (\gamma_a.h_a + \gamma_p.h_p).(S_k - b_1)^2 \end{cases}$$

M_{DW} = *0.47 (KNmm/mm)*

Momen due to Deadload and collision load for the Extreme event limit state:

Mu = *175.88 (KNmm/mm)*

Mu = *175879 (Nmm/mm)*

Thickness of cover: as(-) = 50 *(mm)*

Reinforcement: D20@150

As = 4.189 *(mm2/mm)*

Distance from compression face to centroid of tension reinforcement (mm)

ds(-) = hf - as(-)

ds(-) = 220 *(mm)*

Thickness of the equivalent stress block (mm)

$$a = \frac{A_s . f_y}{0.85\ f_c^{'}.b}$$

a = *78.85 (mm)*

Blending moment factor Φ = *1.0*

Moment resistance:

Φ.Mn = Φ.As.fy.[ds(-) - a(-)/2]

Φ.Mn = *302558 (Nmm/mm)*

This moment strength will be reduced because of the axial tension force:

$$T = \frac{R_{max}}{L_c + 2.H_w}$$

Barrier height:	$Hw =$	1154 (mm)
For impact at a wall segment	$Rmax =$	867.89 (KN)
	$Lc =$	3706 (mm)
	$T_1 =$	144.32 (N/mm)
For impact at end of wall or joint	$Rmax =$	581.48 (KN)
	$Lc =$	2660 (mm)
	$T_2 =$	117.04 (N/mm)
The shear were used is:	$T = Max(T_1, T_2)$	
	$T =$	144.32 (N/mm)

By assuming the interaction curve between moment and axial tension is a straight line:

$$\frac{P_u}{\Phi.P_n} + \frac{M_u}{\Phi.M_n} \leq 1 \qquad (*)$$

In which:	$Pu = T$	
	$Pu =$	144.32 (N/mm)
	$\Phi.Pn = \Phi.As.fy$	
	$\Phi.Pn =$	3351.03 (N/mm)

To find out:

$$M_u \leq \Phi.M_n.\left(1 - \frac{P_u}{\Phi.P_n}\right)$$

	$Mu \leq$	289528 (Nmm/mm)	
With	$Mu =$	175879 (Nmm/mm)	*(OK)*

MINISTRY OF TRANSPORT

VIETNAM EXPRESSWAY CORPORATION - PROJECT MANAGEMENT UNIT NO.85

DA NANG - QUANG NGAI EXPRESSWAY DEVELOPMENT PROJECT

PACKAGE: 6

BRIDGE

OP11

CALCULATION SHEETS

HA NOI, 2012

Table of content - OP11 Bridge

A. Substructure design

Da Nang Quang Ngai Expressway project

BRIDGE

OP11

CALCULATION SHEETS

ABUTMENT A1

Da Nang, 2013

Table of content

Assumptions :

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

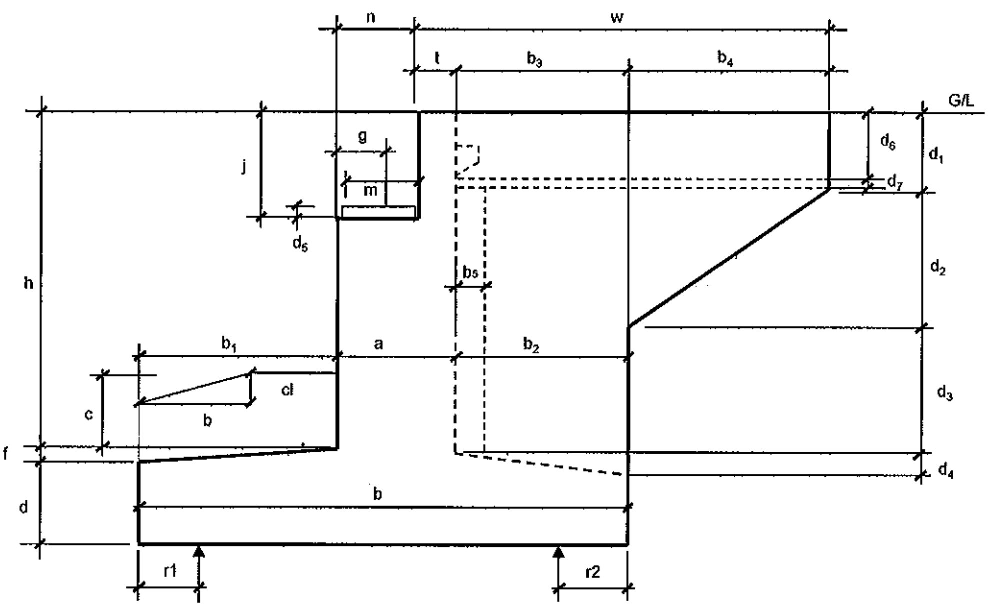
Input :

Level Table(at center of abutment)

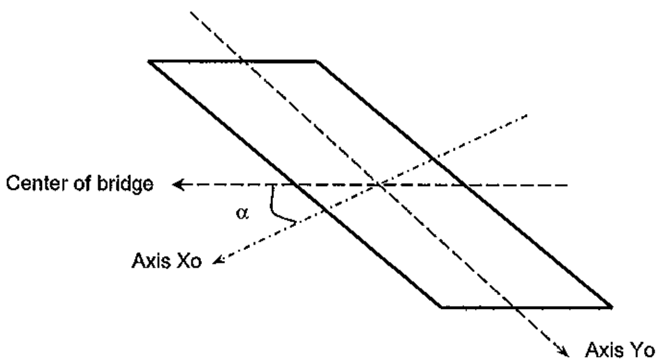
Level of top of headwall	HTwL	**21.416**	m
Level of top of bearing	BTL	**19.906**	m
Level of top of stem abutment	HTL	**19.627**	m
Level of top of footing	FTL	**13.500**	m
Level of bottom of footing	FBL	**11.500**	m
Ground level	GL	**14.560**	m
Lowest water level	HWL	**14.560**	m
Skew angle	α	**20.00**	deg

I.Loads from substructure

Abutment dimensions VERTICAL VIEW



PLAN VIEW

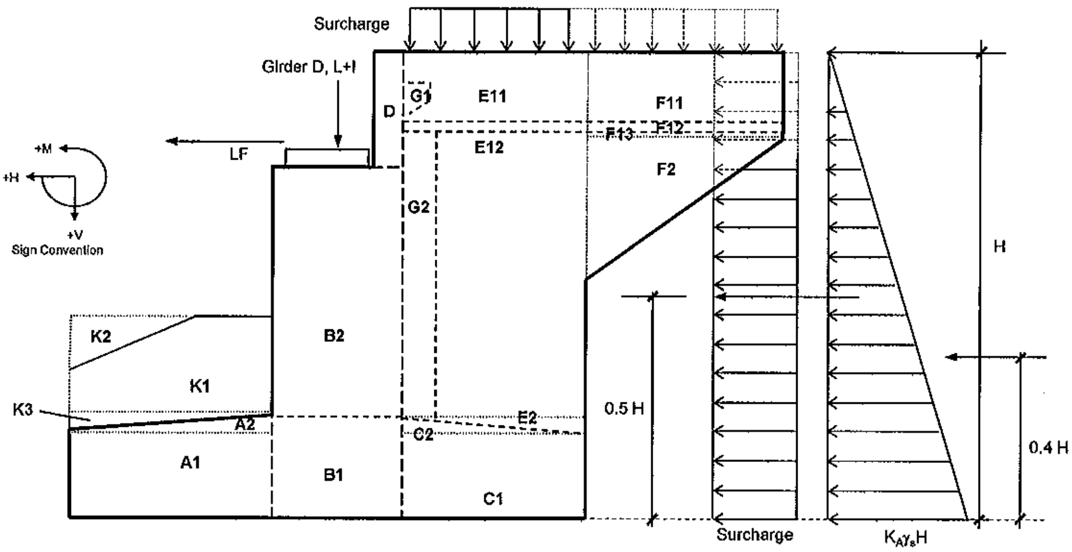


Material Unit Weights

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

ABUTMENT DIMENSIONS (IN METRES)					
Item	Symbol	Value	Item	Symbol	Value
Height of stem	h	7.916	Horizontal Dimension	b_3	3.000
Footing Width	b	6.500	Horizontal Dimension	b_4	3.548
Stem Width	a	1.500	Horizontal Dimension	b_5	0.500
Footing Depth	d	2.000	Vertical Dimension	d_1	0.930
Footing Slope	f	0.000	Vertical Dimension	d_2	3.548
Width of stem at bearing	n	1.000	Vertical Dimension	d_3	3.438
Ballast Wall Height	j	1.789	Vertical Dimension	d_4	0.000
Ballast Wall Thickness	t	0.500	Vertical Dimension	d_5	0.279
Wingwall Length	w	7.500	Vertical Dimension	d_6	1.200
Soil Cover at Toe	c	1.060	Vertical Dimension	d_7	0.300
Girder Reaction	g	0.550	With of bearing pad	m	0.550
Distance to cl of pile	r1	1.100	Wingwall Thickness	u1	0.500
Horizontal Dimension	b_1	2.000	Wingwall Thickness	u2	0.500
Horizontal Dimension	b_2	3.000	Distance to cl of pile	r2	1.100

	$\cos(\alpha)$	=	0.94
Slope front of abutment	cl	=	0.00 m
	bl	=	0.00 m
Width of Abutment	L		12.600 m
Width of abutment (inclined direction)	Ltr		13.409 m
Height of Abutment	Ht	=	9.92 m
Distance from CG of footing to edge of Abutment	b/2	=	3.25 m



1. Self weight of Abutment (DC)

Description	Area (m^2)	Length (m)	Force (kN)	$X^{(1)}$ (m)	$Arm^{(2)}$ (m)	Moment (kN•m)
SW of Abutment (DC)						
Section A1	4.000	13.409	1314	1.000	2.250	2957
Section A2	-	13.409	-	1.333	1.917	-
Section B1	3.000	13.409	986	2.750	0.500	493
Section B2	9.191	13.409	3019	2.750	0.500	1510
Section C1	6.000	13.409	1971	5.000	-1.750	-3449
Section C2	-	13.409	-	4.500	-1.250	-
Section D	0.895	13.409	294	3.250	-	-
Section E11	1.890	0.500	23	5.000	-1.750	-41
Section E12	20.958	0.500	257	5.000	-1.750	-449
Part extra stem	4.958	0.740	90	5.750	-2.500	-225
Section F11	4.258	0.500	52	8.274	-5.024	-262
Section F12	0.982	0.500	12	6.774	-3.524	-42
Section F13	-2.022	0.500	-25	8.274	-5.024	124
Section F2	6.294	0.500	77	7.683	-4.433	-342
Section G1	0.135	12.909	255	3.650	-0.400	-102
Section G2	0.125	12.832	39	3.750	-0.500	-20
Bearing seats (w1seat= 0.60m)	0.153	3.000	14	2.550	0.700	10
Curbs +Handrail on Abutment	0.50	7.500	99	6.750	-3.500	-348
Total SW of Abutment (DC)			*8478*			*-186*
Tranverser moment			*474*		*6.175*	*2929*

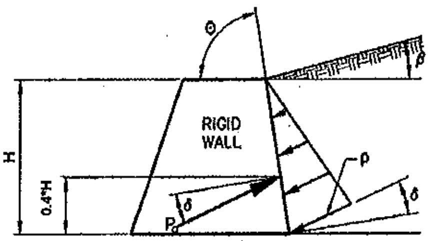
Notes: 1. Distance 'X' is measured horizontally from Toe of Retaining to CG of Section
2. Moment 'Arm' is measured from CG horizontally and from Underside of Footing Vertically.

2. Earth on Abutment (EV)

Description	Area (m^2)	Length (m)	Force (kN)	X[1] (m)	Arm[2] (m)	Moment (kN•m)
Earth on Abutment (EV)						
Section E1	23.75	12.909	5518	5.000	-1.750	-9656
Section E2	-	12.909	-	5.500	-2.250	-
Section E3	-	0.500	-	6.500	-3.250	-
Section K1	2.120	13.409	512	1.000	2.250	-
Section K2	-	13.409	-	-	3.250	-
Section K3	-	13.409	-	0.667	2.583	-
Total Earth on Footing			*6030*			*-9656*

3. Horizontal Earth Pressure on Abutment (EH)

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



- Height for horizontal earth pressure — H = 9.92 m
- Width for horizontal earth pressure — W 13.41 m
- Density of Soil — γ_s = 1835 kg/m^3
- Internal Friction Angle of Soil — ϕ'_f = 30.0 deg
- Incline angle of back face wall — θ = 90.0 deg
- Friction angle between fill and wall — δ = 0.0 deg
- Incline angle of fill soil — β = 0.0 deg
- Gravitational acceleration — g = 9.81 m/s^2
- Basic earth pressure

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

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

Γ = 2.250
Ka = 0.333
p = 0.059 Mpa

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

Horizontal earth pressure:

- Ea = 0,5. p. Z . B . 10^3 (kN) — Ea = 3955 kN
- M= Ea . 0,4H — M = 15688 kNm
- Horizontal Earth Pressure act at a height of 0.4 H — <S 3.11.5.1>

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

Equivalent height of soil for highway loading taken from Table 3.11.6.2.1

H=	1.50m heq=	1.7 m	
H=	3.00m heq=	1.2 m	
H=	6.00m heq=	0.76 m	
H=	9.00m heq=	0.61 m	
H=	9.92m heq=	0.61 m	(Linear interpolation)

- **Vertical force**

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

- **Horizontal force**

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

ESh = 487 kN
eh = 4.96 m
M = 2413 kNm

5. Earthquake effects

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

• Peak ground acceleration coefficient A = 0.0310 g

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

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

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

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

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

• Seismic active lateral Earth pressure coefficient E_{AE} = 4214 kN

$M_{AE} = E_{AS}*0.3H+(E_{AE}-E_{AS})*0.6H$ M_{AE} = 13305 KNm <A.11.1.1.1>

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

5.2. Earthquake effects to abutment (EQ)

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

• Soil profile type	Soil type		I
• Site Coefficients.	S	=	1.0
• Elastic Seismic Response Coefficient	2.5A	=	0.078
Csm = 1.2*A*S/ Tm^2/3 ≤ 2.5*A	Csm	=	0.036
• Period of vibration of the fundamental mode			
Tm = 2*pi()*sqrt(m/k)	Tm	=	1.053 s

Decription	Area (m^2)	Length (m)	Force (kN)	X[1] (m)	Arm[2] (m)	Moment (kN•m)
Section A1	4.000	13.409	41	-	1.000	41
Section A2	-	13.409	-	-	2.000	-
Section B1	3.000	13.409	31	-	1.000	31
Section B2	9.191	13.409	109	-	5.064	550
Section C1	6.000	13.409	61	-	1.000	61
Section C2	-	13.409	-	-	2.000	-
Section D	0.895	13.409	11	-	9.022	95
Section E11	1.890	0.500	1	-	7.316	5
Section E12	20.958	0.500	8	-	3.208	
Section E2	4.958	0.740	3	-	2.000	6
Section F11	4.258	0.500	2	-	7.316	12
Section F12	0.982	0.500	0	-	6.566	
Section F13	-2.022	0.500	-1	-	8.201	
Section F2	6.294	0.500	2	-	7.803	19
Section G1	0.135	12.909	1	-	7.203	10
Section G2	0.125	12.832	1	-	3.208	4
Total EQ of Abutment Selfweight			***269***			***832***

6. Braking Force(BR)

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

• Number of lanes	n	=	3 lanes	
• Multiple presence factor	m	=	0.85	
• Take 25 % of Truck load				
BR = 25% * n * m * (2*145+35)	BR	=	104 kN	Long. Axis
• Acting at 1.8m higher of road face	e	=	11.8 m	
	Mlong	=	1221 KNm	Long. Axis

7. Centrifucal Force , CE (3.6.3)

• Plan of bridge (1:"straight",2: "Curve")			1	
• Design Speed	V	=	120 km/h	
	V	=	33.3 m/s	
	R		- m	
$C = 4/3*(V^2/gR)$	C		-	
Acting at 1.8m higher of road face				
CE = n * m * (2*145+35) * C	CE	=	0.00 KN	
	e	=	11.80 m	
	Mtrans	=	0.00 KNm	Trans. Axis

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

II.Loads from superstructure

Item	Sign	Value	Unit
Span length	Lsp	21.00	m
Span between bearings	Lb	20.30	m
Skew angle	α	20.00	deg
Deck slab length	Ldeck	21.00	m
Bridge Width	Bc	12.48	m
Girder height	hgi	1.20	m
Deck slab depth	hdkslab	0.22	m
Asphalt depth	has	0.084	m
Unit weight of concrete	γc	24.50	kN/m3
Unit weight of asphalt concrete	γa	22.10	kN/m3

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

Item	Sign	Value	Unit
1.1.Girders			
Weight of 1 girder	DC	295.47	kN
Number of girders	n	5	Girders
Sum of girders weight	DC	1477.35	kN
Precast Planks	DC	305.76	kN
Diaphragm	DC	193.18	kN
Total	DC	1976.29	kN
1.2.Deck slab			
Deck slab	DC	1387.15	kN
1.3.Pavement			
Asphalt concrete	DW	447.62	kN
1.4.Handrail			
Handrail + median	DC	568.80	kN

2.Live load (LL):

Truck

145 145 35 kN

4.3 4.3 m

Tandem

110 110 kN

1.2 m

Lane load

w_l 9.3 kN/m

Pedestrial Wpd= 0.0 kN/m2

Considerate structure as a simple span

Reaction Influence

20.3 m

1

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

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

Truck	P1(kN)	P2(kN)	P3(kN)	Sum(kN)	Total(kN)
Axle load	145	145	35		
Influence value	1.000	0.788	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 superstructrue (EQ)

Longitudinal moveable bearings at Abutment

Horizontal force from superstructure due to EQ - transverse direction

At bearing — Heq = 79 kN

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

Bearing displacement due to uniform temperature and shrinkage creep	Δu	=	0.026 m	
$H = G.A.\Delta u/h_{rt}$				<14.6.3.1-2>
Shear modulus G	G	=	1 MPa	
Bearing area	A	=	0.165 m2	
Height of elastomeric layers	hrt		0.065 m	
Number of bearing	nb	=	5 bears	
Horizontal force due to TU+SH&CR				
Acting at top of bearing	H(tu+sh+cr)	=	330 kN	

5.Wind loads (Ws)

5.1.Tranverse wind on superstructure (WS)

Wind zone	Zone		III	
Basic 3 second gust wind	Vb	=	53.00 m/s	
Correction factor	S	=	1.09	
Design wind velocity	V	=	57.77 m/s	
Drag coefficient	Cd	=	1.36	
Overall width of bridge	b	=	12.48 m	
Depth of superstructure(including solid parapet)	d	=	2.48 m	
	b/d	=	5.03	
Windy obstructed area of superstructure	At	=	52.10 m2	
Force due to tranverse wind				
Fhy=max(0.0006*V2*At*Cd,1.8*At) (kN)	Fhy	=	142.2 kN	<3.8.1>

5.2.Wind load on vehicles (WL)

Transverse wind on vehicles	Wltran	=	1.50 kN/m
Transverse horizontal force due to wind on liveload			
At 1.8m from surface	Fhy	=	31.50 kN

6.Combinations

Loads from superstructure to Abutment

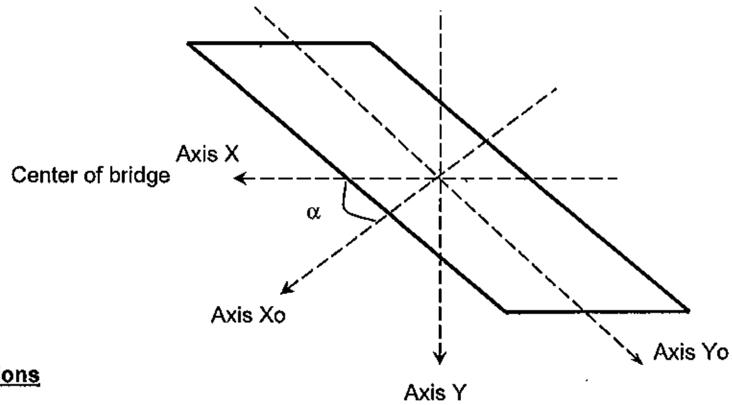
Loads at bottom of stem		Vertical		Longitudinal			Tranversal		
Loads	Sign	N (kN)	x (m)	Hx (kN)	z_1 (m)	My (kN•m)	Hy (kN)	y (m)	Mx (kN•m)
Girder + Deckslab	DC	1682	0.20			336			
Handrail	DC	284	0.20			57			
Pavement	DW	224	0.20			45			
LiveLoad	LL	1131	0.20			226		1.38	1556
Pedestrial	PL	0	0.20			0		-	-
Trans. wind on Struc.	WS						71	6.13	436
Trans. wind on vehi.	WL						16	7.93	125
Eearth quake	EQ						79	6.13	482
TU+SH&CR	TU+SH&CR			330	6.13	2022			

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	1682	0.70			1177			
Handrail	DC	284	0.70			199			
Pavement	DW	224	0.70			157			
LiveLoad	LL	1131	0.70			792		1.38	1556
Pedestrial	PL	0	0.70			0		-	-
Trans. wind on Struc.	WS						71	8.13	578
Trans. wind on vehi.	WL						16	9.93	156
Eearth quake	EQ						79	8.13	640
TU+SH&CR	TU+SH&CR			330	8.13	2682			

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

Load combinations at bottom of stem					
		Longitudinal		Tranversal	
Load combinations	N (kN)	Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	4773	165	1966	0	2723
Strength Str-IB	3895	165	1790	0	2723
Strength Str-IIIA	4321	165	1875	44	2399
Strength Str-IIIB	3442	165	1699	44	2399
Service Ser-I	3321	330	2686	37	1811
Extreme Ext-IA	3359	0	672	79	1260
Extreme Ext-IB	2481	0	496	79	1260

Load combinations at bottom of pilecap					
		Longitudinal		Tranversal	
Load combinations	N (kN)	Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	4773	165	4682	0	2723
Strength Str-IB	3895	165	4068	0	2723
Strength Str-IIIA	4321	165	4366	44	2488
Strength Str-IIIB	3442	165	3751	44	2488
Service Ser-I	3321	330	5007	37	1885
Extreme Ext-IA	3359	0	2351	79	1418
Extreme Ext-IB	2481	0	1737	79	1418



III.Load Combinations

1.Loads from substructure

Loads at bottom of pilecap	Sign	Vertical		Longitudinal			Tranversal		
		N (kN)	x (m)	H_x (kN)	z_1 (m)	M_y (kN•m)	H_y (kN)	y (m)	M_x (kN•m)
Self weight of Abutment	DC	8478				-186			599.557
Soils on pilecap	EV	6030				-9656			
Horizontal Earth Pressure	EH			3717		14742			
Vertical Surcharge	L_{Sv}	442				-773			
Horizontal Surcharge	L_{Sh}			457		2267			
Braking Force	BR			104		1221			
Centrifucal Force	CE			-		-	-		-
Buoyancy of Abutment	WA	-1950				-50			
Buoyancy of Earth on Abutment	WA	-666				50			
Earthquake effects to Abutment	EQ			269		832	81		250
Earthquake effects to soil	E_{AE}			3960		12503			

Table of load factors

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

Load Combination at bottom of pilecap					
Load combinations	N (kN)	Longitudinal		Tranversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	16895	6557	13596	0	749
Strength Str-IB	11214	4327	9161	0	540
Strength Str-IIIA	16718	6332	12510	0	749
Strength Str-IIIB	11037	4102	8075	0	540
Service Ser-I	12333	4278	7614	0	600
Extreme Ext-IA	16342	4509	1423	81	999
Extreme Ext-IB	10662	4509	5833	81	789

2. Loads from superstructure

Load Combination at bottom of pilecap					
		Longitudinal		Tranversal	
Load combinations	N (kN)	Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	4773	165	4682	0	2723
Strength Str-IB	3895	165	4068	0	2723
Strength Str-IIIA	4321	165	4366	44	2488
Strength Str-IIIB	3442	165	3751	44	2488
Service Ser-I	3321	330	5007	37	1885
Extreme Ext-IA	3359	0	2351	79	1418
Extreme Ext-IB	2481	0	1737	79	1418

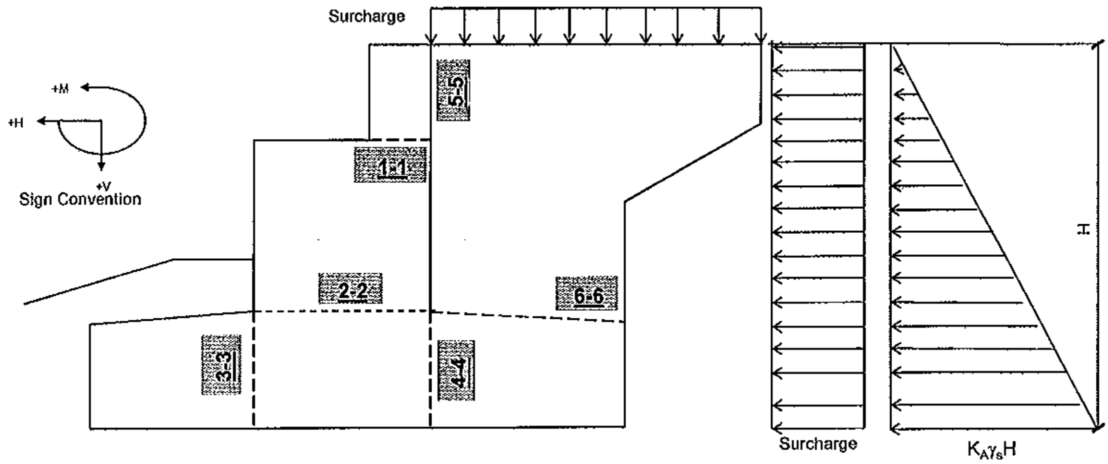
3. Total loads at bottom of pilecap

Load Combination at bottom of pilecap					
		Longitudinal		Tranversal	
Load combinations	N (kN)	Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	21668	6722	18278	0	3472
Strength Str-IB	15109	4492	13228	0	3262
Strength Str-IIIA	21039	6497	16875	44	3237
Strength Str-IIIB	14480	4267	11826	44	3027
Service Ser-I	15655	4608	12621	37	2485
Extreme Ext-IA	19702	4509	3774	159	2417
Extreme Ext-IB	13142	4509	7570	159	2207

IV.Elements checking

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

1. Calculate Internal force of sections



1.1 Section 1-1
Table of Load Factors

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

Unfactored Loads	**N** (kN)	Longitudinal **Hx** (kN)	**My** (kN.m)	Tranversal **Hy** (kN)	**Mx** (kN.m)
Selfweight	549		-102		
Horizontal Earth Pressure		137	98		
Surcharge (horizontal)		246	220		
Horizontal Seismic Earth Pressure		146	83		
Abutment earthquake force		12	11	4	3

Load Combination at bottom of headwall

Load combinations	**N** (kN)	Longitudinal **Hx** (kN)	**My** (kN.m)	Tranversal **Hy** (kN)	**Mx** (kN.m)
Service Ser-I	549	383	216	0	0
Strength Str-IA	687	635	404	0	0
Strength Str-IB	494	553	381	0	0
Extreme Ext-I	687	354	117	4	3

1.2 Section 2-2
Table of Load Factors

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

Unfactored Loads	N (kN)	Longitudinal Hx (kN)	Longitudinal My (kN.m)	Tranversal Hy (kN)	Tranversal Mx (kN.m)
Selfweight	3569		-172		
Superstructure Dead Load	1682		336		
Pavement	224		45		
Handrail+curb	284		57		
Live Load	1131		226		1556
Horizontal Earth Pressure		2682	8494		
Surchage (Horizontal)		450	1782		
TU+SH&CR		330	2022		
Horizontal Seismic Earth Pressure		2858	7203		
Abutment earthquake force		120	416	60	276

Load Combination at bottom of stem wall					
Load combinations	N (kN)	Longitudinal Hx (kN)	Longitudinal My (kN.m)	Tranversal Hy (kN)	Tranversal Mx (kN.m)
Service Ser-I	6890	3463	12789	0	1556
Strength Str-IA	9234	4976	17609	0	2723
Strength Str-IB	7107	3367	12397	0	2723
Extreme Ext-I	7820	4632	12568	60	1054

1.3 Section 3-3

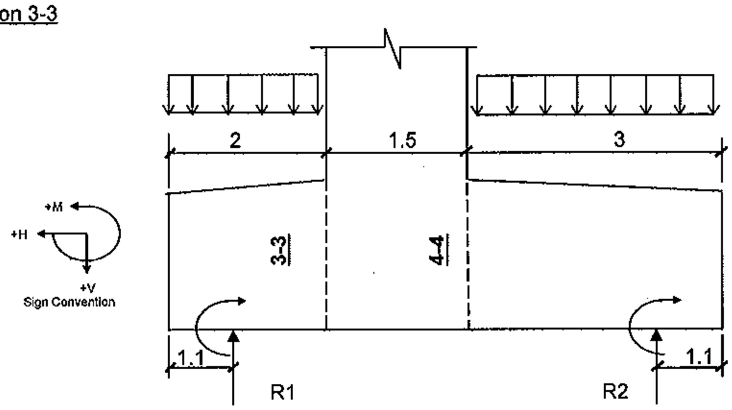


Table of Load Factors

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

Unfactored Loads		N (kN)	Longitudinal Hx (kN)	Longitudinal My (kN.m)	Tranversal Hy (kN)	Tranversal Mx (kN.m)
Selfweight at front side		1314		1314		
Vertical soil on foot at front side		512		512		
Reaction of piles	Ser-I	-11562	-2635	-7778	42	471
	Str-IA	-16232	-3840	-10789	93	719
	Str-IB	-11415	-2567	-7781	62	526
	Ext-I	-11944	-2579	-7630	-24	214

Load Combination at section 3-3					
Load combinations	N (kN)	Longitudinal Hx (kN)	Longitudinal My (kN.m)	Tranversal Hy (kN)	Tranversal Mx (kN.m)
Service Ser-I	-9737	-2635	-5953	42	471
Strength Str-IA	-13898	-3840	-8456	93	719
Strength Str-IB	-9772	-2567	-6138	62	526
Extreme Ext-I	-9611	-2579	-5297	-24	214

1.4 Section 4-4

Table of Load Factors

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

Unfactored Loads		N	Longitudinal		Tranversal	
			Hx	My	Hy	Mx
		(kN)	(kN)	(kN.m)	(kN)	(kN.m)
Selfweight of behind side		2457		-4067		
Vertical soil on foot at behind side		5518		-8277		
Surcharge(Vertical)		442		-663		
Reaction of piles	Ser-I	-4094	-1976	9750	-81	208
	Str-IA	-5439	-2880	13198	-93	330
	Str-IB	-3692	-1926	8885	-62	255
	Ext-I	-7754	-1934	17073	-133	12

Load Combination at section 4-4					
		Longitudinal		Tranversal	
Load combinations	N	Hx	My	Hy	Mx
	(kN)	(kN)	(kN.m)	(kN)	(kN.m)
Service Ser-I	4323	-1976	-3256	-81	208
Strength Str-IA	5855	-2880	-4219	-93	330
Strength Str-IB	4258	-1926	-3384	-62	255
Extreme Ext-I	2988	-1934	484	-133	12

1.4 Section 5-5 & 6-6

Slope of triang pressure $tg\beta$ = 6.00

Uniform horizontal pressure U.p = 3.66 kN/m2

Load Combination at section 5-5					
		Horizontal		Vertical	
Load combinations	N	Hx	Mx	Hy	My
	(kN)	(kN)	(kN.m)	(kN)	(kN.m)
Service Ser-I		89	206		
Strength Str-IA		139	328		

Load Combination at section 6-6					
		Horizontal		Vertical	
Load combinations	N	Hx	Mx	Hy	My
	(kN)	(kN)	(kN.m)	(kN)	(kN.m)
Service Ser-I				244	179
Strength Str-IA				378	334

2. Elements Checking

2.1. Bearing Resistance <S.5.7.5>

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

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

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

Where supporting surface is wider on all sides than loades area

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

where loaded area is subjected to nouniformly distributed bearing

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

Modification factor (case 1)	m	=	1.784	
Resistance factor	ϕ	=	0.700	<S.5.5.4.2>
Factored bearing resistance	Pr	=	10507 kN	> Pu
Bearing reaction of approach bridge	Pu	=	3641 kN	Ok

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

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

Factored bearing resistance shall be taken <S.5.10.9.7.2>

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

fn take the lesser of

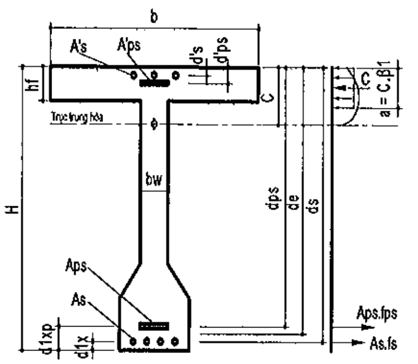
$fn = 0.7.\ f'ci.\ sqrt(A/Ag)$ and

$fn = 2.25.\ f'ci$

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

MATERIALS

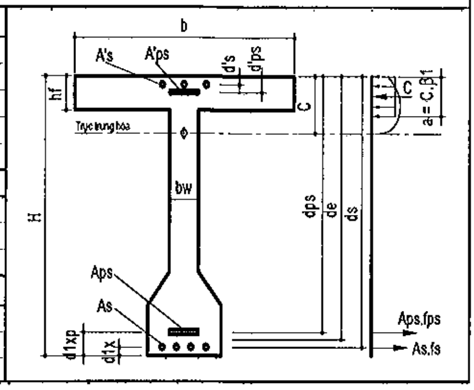
Sign	Parameter	Unit	Value
NORMAL CONCRETE			
f'c	Compressive Strength of concrete at 28 days	Mpa	30
Ec	Modulus of Elasticity	Mpa	27691
fr	Modulus of Rupture	Mpa	3.5
gc	Unit weight of concrete	kN/m3	24.5
PRESTRESSING STEEL			
fpu	Tensile strength of prestressing steel	Mpa	1860
fpy	Yield strength of prestressing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



Sign	Parameters		Unit	Sections 1-1	1-1	2-2	2-2	2-2
INTERNAL FORCES AT SECTIOIN								
	Combination			Strength	Service	Service	Strength	Extreme
Qu	Shear		kN	635	383	3463	4976	4632
Mu	Flexural Moment		kNm	404	216	12789	17609	12568
Nu	Axial load		kN	687	549	6890	9234	7820
Tu	Torsional Moment		kNm	0	0	0	0	0
FLEXURAL MOMENT CHECKING								
H	Section height		m	0.500	0.500	1.500	1.500	1.500
d's	Dis. From comp. fiber to centroid of comp. Reinf		m	0.058	0.058	0.058	0.058	0.058
d1x	Dis. From tens. fiber to centroid of tension Reinf		m	0.058	0.058	0.059	0.059	0.059
	Cover to reinf		m	0.050	0.050	0.050	0.050	0.050
ds	Dis. From comp. fiber to centroid of tension Reinf		m	0.442	0.442	1.441	1.441	1.441
d'ps	Dis. From comp. fiber to centroid of comp. prestressing steel		m	0.000	0.000	0.000	0.000	0.000
d1xp	Dis. From tens. fiber to centroid of tension prestressing steel		m	0.000	0.000	0.000	0.000	0.000
dps	Dis. From comp. fiber to centroid of tension prestressing steel		m	0.500	0.500	1.500	1.500	1.500
b	Width of the compression face of member		m	13.409	13.409	13.409	13.409	13.409
bw	Web width or diameter of a circular section		m	13.409	13.409	13.409	13.409	13.409
hf	Compression flange depth		m	0.000	0.000	0.000	0.000	0.000
Iz	Moment of inertia of section		m4	0.140	0.140	3.771	3.771	3.771
Amc	Section area		m2	6.704	6.704	20.113	20.113	20.113
	Steel choice							
Aps	Tension prestressing steel	P.S type		0	0	0	0	0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
A'ps	Compression prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	0 T00.0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
As	Tension Reinforcement	Number	bars	83	83	156	156	156
		Diameter	mm	16	16	22	22	22
		Area	m2	0.01677	0.01677	0.05928	0.05928	0.05928
A's	Compression Reinforcement	Number	bars	83	83	83	83	83
		Diameter	mm	16	16	16	16	16
		Area	m2	0.01677	0.01677	0.01677	0.01677	0.01677
A'c	Shear reinforcement	Number	bars	21	21	20	20	20
		Diameter	mm	14	14	14	14	14
		Area	m2	0.00317	0.00317	0.00302	0.00302	0.00302
ϕ	Resistance factors for flexure		5.5.4.2	0.90	1.00	1.00	0.90	1.00
ϕv	Resistance factors for shear			0.90	1.00	1.00	0.90	1.00
ϕn	Resistance factors for axial force			1.00	1.00	1.00	1.00	1.00
$\beta 1$	Stress block factor			0.836	0.836	0.836	0.836	0.836
c	Dis. Between centroid and top fiber		m	0.000	0.000	0.060	0.060	0.060
	For T section behavior		m	0.000	0.000	0.060	0.060	0.060
	For rectangular section behavior		m	0.000	0.000	0.060	0.060	0.060
fpe	Effective stress in the prestressing steel after losses		Mpa	1116	1116	1116	1116	1116
fps	Aver. stress in pres. steel at the time for which the nominal resistanc		Mpa	1860	1860	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	0.000	0.000	0.050	0.050	0.050
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	33357	33357	33357
Mr	Factored resistance		kNm	**2318**	**2575**	**33357**	**30021**	**33357**
Mu	Flexual moment		kNm	404	216	12789	17609	12568

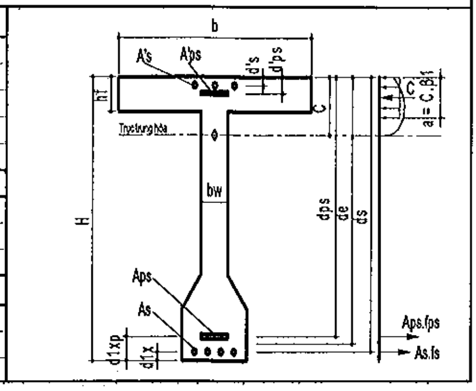
(5.7.3.2)	**Flexural moment Checking**		OK	OK	OK	OK	OK
	Limits for reinforcement						
c/de	Maximum reinforcement		0.00	0.00	0.04	0.04	0.04
	Maximum reinforcement Checking	<= 0.42	OK	OK	OK	OK	OK
1.2*Mcr	Craking moment	kNm	1157	1157	10841	10841	10841
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**		OK	OK	OK	OK	OK
(5.7.3.4)	**Conctrol of craking by distr. of reinf for RC member- Check?**		No	Yes	Yes	No	No
	Existing condition for structrure	1,2 or 3	1	1	1	1	1
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.058	0.058	0.059	0.059	0.059
Z	Crack width parameter	N/mm	30000	30000	30000	30000	30000
A	Area of concr. with same centroid as tens. Reinf	m2	0.019	0.019	0.010	0.010	0.010
fsa	Value	Mpa	292	292	356	356	356
0.6*fy		Mpa	240	240	240	240	240
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	240	240	240	240	240
x	Dist. From compression fiber to centroid	m	-	0.08	0.269	-	-
J.d	Arm	m	-	0.415	1.351	-	-
Icr	Moment of inertia of the cracked section	m4	-	0.018	0.661	-	-
fs	Tensile stress in reinforcement fs = Msls / (As*J.d)	Mpa	-	31	160	-	-
	Checking for control cracking fs<fsa		N.a	OK	OK	N.a	N.a
(5.10.8.2)	**Shrinkage and temperature Reinfocement (side distribution)**						
Areq	Area of required reinf	m2	0.00045	0.00045	0.00126	0.00126	0.00126
	Distribution on sides 7 D16	m2	0.00141	0.00141	0.00141	0.00141	
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45	0.45
	Checking		OK	OK	OK	OK	OK
	SHEAR AND TORSION CHECKING						
β	Factor indicating diag. cracked concr. to tension		3.1	3.8	2.4	2.2	2.4
θ	Angle of inclination of diagonal compressive	degree	28.68	27.54	32.77	36.21	33.04
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	90
bv	Effective web width as minimum web width - in dv	m	13.409	13.409	13.409	13.409	13.409
dv	Effective shear depth	m	0.442	0.442	1.416	1.416	1.416
	(de - a/2)	m	0.442	0.442	1.416	1.416	1.416
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	0.600
ncat	Amount of bars in spacing S	bars	21	21	20	20	20
Av	Shear reinf area in spacing S	m2	0.0032	0.0032	0.0030	0.0030	0.0030
θ	Assume	degree	28.87	28.19	28.88	30.16	30.27
v	Shear stress in concrete	kN/m2	119	65	182	291	244
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e_x	Strain in tensile reinforcement		3.42E-04	1.70E-04	7.36E-04	1.02E-03	7.54E-04
	if ex<0, multiple with reduce factor		-	-	-	-	-
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok	Ok
v/f'c	Ratio of shear stress and f'c		0.004	0.002	0.006	0.010	0.008
β	Final value		3.1	3.8	2.4	2.2	2.4
θ	Final value	degree	28.68	27.54	32.77	36.21	33.04
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	8432	10265	20527	19151	20442
Vs	Shear resistance provided by shear reinforcement	kN	1708	1792	4428	3895	4383
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	10140	12057	24955	23046	24825
Vn2	Vn2	kN	44450	44450	142413	142413	142413
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	10140	12057	24955	23046	24825
Vr	Factored shear resistance	kN	**9126**	**12057**	**24955**	**20741**	**24825**
Vu	Shear	kN	635	383	3463	4976	4632
(5.8.2.7)	**Shear checking**		OK	OK	OK	OK	OK
	Region requiring transverse reinf Checking		No need	No need	No need	No need	No need
	Minimum shear reinf area	m2	0.0091	0.0091	0.0091	0.0091	0.0091
	Minimum shear reinforcement Checking		-	-	-	-	-
	0.1*f'c*bv*dv	kN	17780	17780	56965	56965	56965
	Smax	m	0.35	0.35	0.60	0.60	0.60
	Maximum spacing Smax		-	-	-	-	-

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



Sign	Parameters		Unit	Sections 3-3	3-3	3-3	4-4	4-4
INTERNAL FORCES AT SECTIOIN								
	Combination			Service	Strength	Extreme	Extreme	Strength
Qu	Shear		kN	9737	13898	9611	2988	5855
Mu	Flexural Moment		kNm	5953	8456	5297	484	4219
Nu	Axial load		kN	2635	3840	2579	1934	2880
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	13.409	13.409	13.409	13.409	13.409
bw	Web width or diameter of a circular section		m	13.409	13.409	13.409	13.409	13.409
hf	Compression flange depth		m	0.000	0.000	0.000	0.000	0.000
Iz	Moment of inertia of section		m4	8.939	8.939	8.939	8.939	8.939
Amc	Section area		m2	26.817	26.817	26.817	26.817	26.817
	Steel choice							
Aps	Tension prestressing steel	P.S type		0	0	0	0	0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
A'ps	Compression prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	0 T00.0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
As	Tension Reinforcement	Number	bars	84	84	84	84	84
		Diameter	mm	25	25	25	20	20
		Area	m2	0.04124	0.04124	0.04124	0.02638	0.02638
A's	Compression Reinforcement	Number	bars	0	0	0	0	0
		Diameter	mm	20	20	20	25	25
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
A'c	Shear reinforcement	Number	bars	21	21	21	21	21
		Diameter	mm	16	16	16	16	16
		Area	m2	0.00424	0.00424	0.00424	0.00424	0.00424
ϕ	Resistance factors for flexure		5.5.4.2	1.00	0.90	1.00	1.00	0.90
ϕv	Resistance factors for shear			1.00	0.90	1.00	1.00	0.90
ϕn	Resistance factors for axial force			1.00	1.00	1.00	1.00	1.00
β1	Stress block factor			0.836	0.836	0.836	0.836	0.836
c	Dis. Between centroid and top fiber		m	0.058	0.058	0.058	0.037	0.037
	For T section behavior		m	0.058	0.058	0.058	0.037	0.037
	For rectangular section behavior		m	0.058	0.058	0.058	0.037	0.037
fpe	Effective stress in the prestressing steel after losses		Mpa	1116	1116	1116	1116	1116
fps	Aver. stress in pres. steel at the time for which the nominal resistance		Mpa	1845	1845	1845	1850	1850
k	Factor depends on type of P.S, Low relaxation strand k = 0.28			0.28	0.28	0.28	0.28	0.28
a	Depth of equivalent stress block		m	0.048	0.048	0.048	0.031	0.031
de	Corresp. effective depth from extreme comp. fiber							
	to centroid of tensile force in the tensile reinf.		m	1.838	1.838	1.838	1.916	1.916
Mn	Nominal resistance		kNm	29916	29916	29916	20052	20052
Mr	Factored reistance		kNm	**29916**	**26925**	**29916**	**20052**	**18047**
Mu	Flexual moment		kNm	5953	8456	5297	484	4219

(5.7.3.2)	**Flexural moment Checking**		**OK**	**OK**	**OK**	**OK**	**OK**
	Limits for reinforcement						
c/de	Maximum reinforcement		0.03	0.03	0.03	0.02	0.02
	Maximum reinforcement Checking	<= 0.42	**OK**	**OK**	**OK**	**OK**	**OK**
1.2*Mcr	Craking moment	kNm	19058	19058	19058	18856	18856
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**		**OK**	**OK**	**OK**	**OK**	**OK**
(5.7.3.4)	**Conctrol of craking by distr. of reinf for RC member- Check?**		Yes	No	No	No	No
	Existing condition for structrure	1,2 or 3	3	3	3	3	3
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.063	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	m2	0.020	0.020	0.020	0.019	0.019
fsa	Value	Mpa	163	163	163	167	167
0.6*fy		Mpa	240	240	240	240	240
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	163	163	163	167	167
x	Dist. From compression fiber to centroid	m	0.261	-	-	-	-
J.d	Arm	m	1.751	-	-	-	-
Icr	Moment of inertia of the cracked section	m4	0.797	-	-	-	-
fs	Tensile stress in reinforcement fs = Msls / (As*J.d)	Mpa	82	-	-	-	-
	Checking for control cracking fs<fsa		**OK**	**N.a**	**N.a**	**N.a**	**N.a**
(5.10.8.2)	**Shrinkage and temperature Reinfocement (side distribution)**						
Areq	Area of required reinf	m2	0.00127	0.00127	0.00127	0.00127	0.00127
	Distribution on sides 7 D16	m2	0.00141	0.00141	0.00141	0.00141	0.00141
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45	0.45
	Checking		**OK**	**OK**	**OK**	**OK**	**OK**
	SHEAR AND TORSION CHECKING						
β	Factor indicating diag. cracked concr. to tension		2.2	2.1	2.3	3.1	2.3
θ	Angle of inclination of diagonal compressive	degree	35.85	38.87	34.80	28.68	35.44
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	90
bv	Effective web width as minimum web width - in dv	m	13.409	13.409	13.409	13.409	13.409
dv	Effective shear depth	m	1.813	1.813	1.813	1.901	1.901
	(de - a/2)	m	1.813	1.813	1.813	1.901	1.901
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	0.600
ncat	Amount of bars in spacing S	bars	21	21	21	21	21
Av	Shear reinf area in spacing S	m2	0.0042	0.0042	0.0042	0.0042	0.0042
θ	Assume	degree	38.22	41.43	39.67	30.88	34.58
v	Shear stress in concrete	kN/m2	400	635	395	33	255
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e_x	Strain in tensile reinforcement		9.88E-04	1.29E-03	9.00E-04	3.39E-04	9.53E-04
	if ex<0, multiple with reduce factor		-	-	-	-	-
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok	Ok
v/f'c	Ratio of shear stress and f'c		0.013	0.021	0.013	0.001	0.009
β	Final value		2.2	2.1	2.3	3.1	2.3
θ	Final value	degree	35.85	38.87	34.80	28.68	35.44
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	24726	22873	25267	36413	26141
Vs	Shear resistance provided by shear reinforcement	kN	7097	6362	7378	9827	7553
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	31823	29235	32645	46239	33694
Vn2	Vn2	kN	182362	182362	182362	191131	191131
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	31823	29235	32645	46239	33694
Vr	Factored shear resistance	kN	**31823**	**26311**	**32645**	**46239**	**30325**
Vu	Shear	kN	9737	13898	9611	2988	5855
(5.8.2.7)	**Shear checking**		**OK**	**OK**	**OK**	**OK**	**OK**



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

Sign	Parameters		Unit	Sections				
				5-5	5-5	6-6	6-6	6-6
	INTERNAL FORCES AT SECTIOIN							
	Combination			Service	Strength	Service	Strength	Strength
Qu	Shear		kN	89	139	244	378	378
Mu	Flexural Moment		kNm	206	328	179	334	334
Nu	Axial load		kN	0	0	0	0	0
Tu	Torsional Moment		kNm	0	0	0	0	0
	FLEXURAL MOMENT CHECKING							
H	Section height		m	0.500	0.500	0.500	0.500	0.500
d's	Dis. From comp. fiber to centroid of comp. Reinf		m	0.058	0.058	0.058	0.058	0.058
d1x	Dis. From tens. fiber to centroid of tension Reinf		m	0.059	0.059	0.059	0.059	0.059
	Cover to reinf		m	0.050	0.050	0.050	0.050	0.050
ds	Dis. From comp. fiber to centroid of tension Reinf		m	0.441	0.441	0.441	0.441	0.441
d'ps	Dis. From comp. fiber to centroid of comp. prestressing steel		m	0.000	0.000	0.000	0.000	0.000
d1xp	Dis. From tens. fiber to centroid of tension prestressing steel		m	0.000	0.000	0.000	0.000	0.000
dps	Dis. From comp. fiber to centroid of tension prestressing steel		m	0.500	0.500	0.500	0.500	0.500
b	Width of the compression face of member		m	1.000	1.000	1.000	1.000	1.000
bw	Web width or diameter of a circular section		m	1.000	1.000	1.000	1.000	1.000
hf	Compression flange depth		m	0.000	0.000	0.000	0.000	0.000
Iz	Moment of inertia of section		m4	0.010	0.010	0.010	0.010	0.010
Amc	Section area		m2	0.500	0.500	0.500	0.500	0.500
	Steel choice							
Aps	Tension prestressing steel	P.S type		0	0	0	0	0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
A'ps	Compression prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	0 T00.0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
As	Tension Reinforcement	Number	bars	7	7	7	7	7
		Diameter	mm	22	22	22	22	22
		Area	m2	0.00266	0.00266	0.00266	0.00266	0.00266
A's	Compression Reinforcement	Number	bars	7	7	7	7	7
		Diameter	mm	16	16	16	16	16
		Area	m2	0.00141	0.00141	0.00141	0.00141	0.00141
A'c	Shear reinforcement	Number	bars	3	3	3	3	3
		Diameter	mm	12	12	12	12	12
		Area	m2	0.00034	0.00034	0.00034	0.00034	0.00034
ϕ	Resistance factors for flexure		5.5.4.2	1.00	0.90	1.00	0.90	0.90
ϕv	Resistance factors for shear			1.00	0.90	1.00	0.90	0.90
ϕn	Resistance factors for axial force			1.00	1.00	1.00	1.00	1.00
β1	Stress block factor			0.836	0.836	0.836	0.836	0.836
c	Dis. Between centroid and top fiber		m	0.023	0.023	0.023	0.023	0.023
	For T section behavior		m	0.023	0.023	0.023	0.023	0.023
	For rectangular section behavior		m	0.023	0.023	0.023	0.023	0.023
fpe	Effective stress in the prestressing steel after losses		Mpa	1116	1116	1116	1116	1116
fps	Aver. stress in pres. steel at the time for which the nominal resistance		Mpa	1835	1835	1835	1835	1835
k	Factor depends on type of P.S, Low relaxation strand k = 0.28			0.28	0.28	0.28	0.28	0.28

a	Depth of equivalent stress block	m	0.020	0.020	0.020	0.020	0.020
de	Corresp. effective depth from extreme comp. fiber						
	to centroid of tensile force in the tensile reinf.	m	0.441	0.441	0.441	0.441	0.441
Mn	Nominal resistance	kNm	432	432	432	432	432
Mr	Factored reistance	kNm	**432**	**388**	**432**	**388**	**388**
Mu	Flexual moment	kNm	206	328	179	334	334
(5.7.3.2)	**Flexural moment Checking**		**OK**	**OK**	**OK**	**OK**	**OK**
	Limits for reinforcement						
c/de	Maximum reinforcement		0.05	0.05	0.05	0.05	0.05
	Maximum reinforcement Checking	<= 0.42	**OK**	**OK**	**OK**	**OK**	**OK**
r min	Mimimum reinforcement		0.53%	0.53%	0.53%	0.53%	0.53%
	Minimum reinforcement Checking for RC	0.23%	**OK**	**OK**	**OK**	**OK**	**OK**
1.2*Mcr	Craking moment	kNm	90	90	90	90	90
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**		**OK**	**OK**	**OK**	**OK**	**OK**
(5.7.3.4)	**Conctrol of craking by distr. of reinf for RC member- Check?**		Yes	No	Yes	No	No
	Existing condition for structrure	1,2 or 3	1	1	1	1	1
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.059	0.059	0.059	0.059	0.059
Z	Crack width parameter	N/mm	30000	30000	30000	30000	30000
A	Area of concr. with same centroid as tens. Reinf	m2	0.017	0.017	0.017	0.017	0.017
fsa	Value	Mpa	301	301	301	301	301
0.6*fy		Mpa	240	240	240	240	240
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	240	240	240	240	240
x	Dist. From compression fiber to centroid	m	0.111	-	0.111	-	-
J.d	Arm	m	0.404	-	0.404	-	-
Icr	Moment of inertia of the cracked section	m4	0.003	-	0.003	-	-
fs	Tensile stress in reinforcement fs = Msls / (As*J.d)	Mpa	192	-	167	-	-
	Checking for control cracking fs<fsa		**OK**	N.a	**OK**	N.a	N.a
(5.10.8.2)	**Shrinkage and temperature Reinfocement (side distribution)**						
Areq	Area of required reinf	m2	0.00031	0.00031	0.00031	0.00031	0.00031
	Distribution on sides 7 D16	m2	0.00141	0.00141	0.00141	0.00141	0.00141
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45	0.45
	Checking		**OK**	**OK**	**OK**	**OK**	**OK**
	SHEAR AND TORSION CHECKING						
β	Factor indicating diag. cracked concr. to tension		2.2	1.9	2.2	1.8	1.8
θ	Angle of inclination of diagonal compressive	degree	36.14	41.31	36.70	42.37	42.37
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	0.431	0.431	0.431	0.431	0.431
	(de - a/2)	m	0.431	0.431	0.431	0.431	0.431
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	0.600
ncat	Amount of bars in spacing S	bars	3	3	3	3	3
Av	Shear reinf area in spacing S	m2	0.0003	0.0003	0.0003	0.0003	0.0003
β	Assume		2.0	2.0	2.0	2.0	2.0
θ	Assume	degree	36.15	41.31	38.30	42.39	42.39
v	Shear stress in concrete	kN/m2	206	359	565	974	974
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e_x	Strain in tensile reinforcement		1.01E-03	1.58E-03	1.07E-03	1.84E-03	1.84E-03
	if ex<0, multiple with reduce factor		-	-	-	-	-
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok	Ok
v/f'c	Ratio of shear stress and f'c		0.007	0.012	0.019	0.032	0.032
β	Final value		2.2	1.9	2.2	1.8	1.8
θ	Final value	degree	36.14	41.31	36.70	42.37	42.37
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	436	375	429	351	351
Vs	Shear resistance provided by shear reinforcement	kN	134	111	131	107	107
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	569	486	560	458	458
Vn2	Vn2	kN	3234	3234	3234	3234	3234
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	569	486	560	458	458
Vr	Factored shear resistance	kN	**569**	**438**	**560**	**412**	**412**
Vu	Shear	kN	89	139	244	378	378
(5.8.2.7)	**Shear checking**		**OK**	**OK**	**OK**	**OK**	**OK**

SPACE PILE FOUNDATION ANALYSIS PROGRM
Turbo BASIC

PROJECT: E.X Da Nang Quang Ngai
Bridge: OP11 - KM42+723

INITIA DATA

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

LOAD COMBINATIONS

COMB.	Hx	Hy	P	Mx	My	Mz
1	685.00	0.00	2209.00	-354.00	1863.00	0.00
2	458.00	0.00	1540.00	-333.00	1348.00	0.00
3	662.00	5.00	2145.00	-330.00	1720.00	0.00
4	435.00	5.00	1476.00	-309.00	1205.00	0.00
5	470.00	4.00	1596.00	-253.00	1287.00	0.00
6	460.00	16.00	2008.00	-246.00	385.00	0.00
7	460.00	16.00	1340.00	-225.00	772.00	0.00

PROPERTIES OF PILES

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

PILE COORD.

PILE	X	Y	Phi	Xi
1	0.67	4.75	0.000	0.00
2	1.88	1.40	0.000	0.00
3	3.10	-1.95	0.000	0.00
4	4.32	-5.30	0.000	0.00
5	-0.47	-5.30	0.000	0.00
6	-2.29	-0.28	0.000	0.00
7	-4.12	4.75	0.000	0.00

FORCES ON PILES

PILE	COMB.	N	Q2	Q3	M1	M2	M3
1	1	453.93	-103.64	0.26	0.619	15.605	104.772
	2	322.80	-69.29	0.17	0.414	11.596	68.493
	3	436.14	-100.22	-0.46	0.605	13.738	102.514
	4	305.02	-65.88	-0.55	0.400	9.730	66.235
	5	324.01	-71.16	-0.39	0.431	10.111	72.144
	6	340.93	-69.81	-2.10	0.439	3.530	84.791
	7	263.70	-69.81	-2.10	0.439	5.337	76.460
2	1	427.08	-99.78	1.66	0.619	17.411	99.810
	2	301.54	-66.72	1.11	0.414	12.804	65.175

	3	410.19	-96.46	0.91	0.605	15.505	97.661
	4	284.65	-63.39	0.36	0.400	10.898	63.025
	5	304.44	-68.48	0.58	0.431	11.367	68.693
	6	316.57	-67.08	-1.11	0.439	4.811	81.273
	7	246.18	-67.08	-1.11	0.439	6.618	72.941
3	1	400.22	-95.93	3.06	0.619	19.217	94.848
	2	280.27	-64.14	2.05	0.414	14.011	61.857
	3	384.23	-92.69	2.28	0.605	17.272	92.807
	4	264.28	-60.90	1.27	0.400	12.066	59.816
	5	284.87	-65.80	1.56	0.431	12.623	65.242
	6	292.22	-64.35	-0.12	0.439	6.092	77.754
	7	228.65	-64.35	-0.12	0.439	7.899	69.422
4	1	373.37	-92.08	4.46	0.619	21.023	89.886
	2	259.00	-61.57	2.98	0.414	15.219	58.539
	3	358.28	-88.92	3.65	0.605	19.038	87.953
	4	243.91	-58.41	2.17	0.400	13.234	56.607
	5	265.31	-63.12	2.53	0.431	13.880	61.790
	6	267.86	-61.62	0.88	0.439	7.372	74.236
	7	211.12	-61.62	0.88	0.439	9.179	65.904
5	1	144.52	-92.08	-1.04	0.619	13.930	89.886
	2	93.56	-61.57	-0.70	0.414	10.476	58.539
	3	146.46	-88.92	-1.74	0.605	12.100	87.953
	4	95.50	-58.41	-1.39	0.400	8.646	56.607
	5	109.77	-63.12	-1.30	0.431	8.946	61.790
	6	226.94	-61.62	-3.03	0.439	2.343	74.236
	7	103.82	-61.62	-3.03	0.439	4.150	65.904
6	1	184.80	-97.86	-3.15	0.619	11.220	97.329
	2	125.47	-65.43	-2.10	0.414	8.665	63.516
	3	185.39	-94.57	-3.79	0.605	9.450	95.234
	4	126.05	-62.14	-2.75	0.400	6.894	61.421
	5	139.13	-67.14	-2.76	0.431	7.062	66.967
	6	263.47	-65.71	-4.52	0.439	0.422	79.514
	7	130.12	-65.71	-4.52	0.439	2.229	71.182
7	1	225.08	-103.64	-5.25	0.619	8.511	104.772
	2	157.37	-69.29	-3.51	0.414	6.853	68.493
	3	224.32	-100.22	-5.85	0.605	6.800	102.514
	4	156.61	-65.88	-4.11	0.400	5.142	66.235
	5	168.48	-71.16	-4.22	0.431	5.177	72.144
	6	300.01	-69.81	-6.01	0.439	-1.500	84.791
	7	156.40	-69.81	-6.01	0.439	0.308	76.460

SUMMARY OF FORCES

	PILE	COMB.	N	Q2	Q3	M1	M2	M3
Nmin	5	2	93.56	-61.57	-0.70	0.414	10.476	58.539
Nmax	1	1	453.93	-103.64	0.26	0.619	15.605	104.772
Q2max	1	1	453.93	-103.64	0.26	0.619	15.605	104.772
Q3max	7	7	156.40	-69.81	-6.01	0.439	0.308	76.460
M1max	1	1	453.93	-103.64	0.26	0.619	15.605	104.772
M2max	4	1	373.37	-92.08	4.46	0.619	21.023	89.886
M3max	1	1	453.93	-103.64	0.26	0.619	15.605	104.772

CHECKING CALCULATI
IN COMPARISON WITH INITIA LOAD MATRIX

1	685.00	0.00	2209.00	-354.00	1863.00	0.00
2	458.00	0.00	1540.00	-333.00	1348.00	0.00
3	662.00	5.00	2145.00	-330.00	1720.00	0.00
4	435.00	5.00	1476.00	-309.00	1205.00	0.00
5	470.00	4.00	1596.00	-253.00	1287.00	0.00
6	460.00	16.00	2008.00	-246.00	385.00	0.00
7	460.00	16.00	1340.00	-225.00	772.00	0.00

f.BORED PILE DESIGN

I. BORED PILE DATA

1.Load Combinations at top of bored pile

No	Combinations	Sign	F_V (kN)	Longitudinal F_{HX} (kN)	Longitudinal My (kN•m)	Transvesal F_{HY} (kN)	Transvesal Mx (kN•m)
1	Strength Str-IB		942	599	-579	7	-95
2	Strength Str-IA		4320	1011	-1032	-3	-145
3	Strength Str-IA		2169	1011	-1032	51	-76
4	Strength Str-IA		3592	898	-887	-44	-198
5	Strength Str-IA		2169	1011	-1032	51	-76
6							

2. Bored pile Material

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

3. Bored pile Section

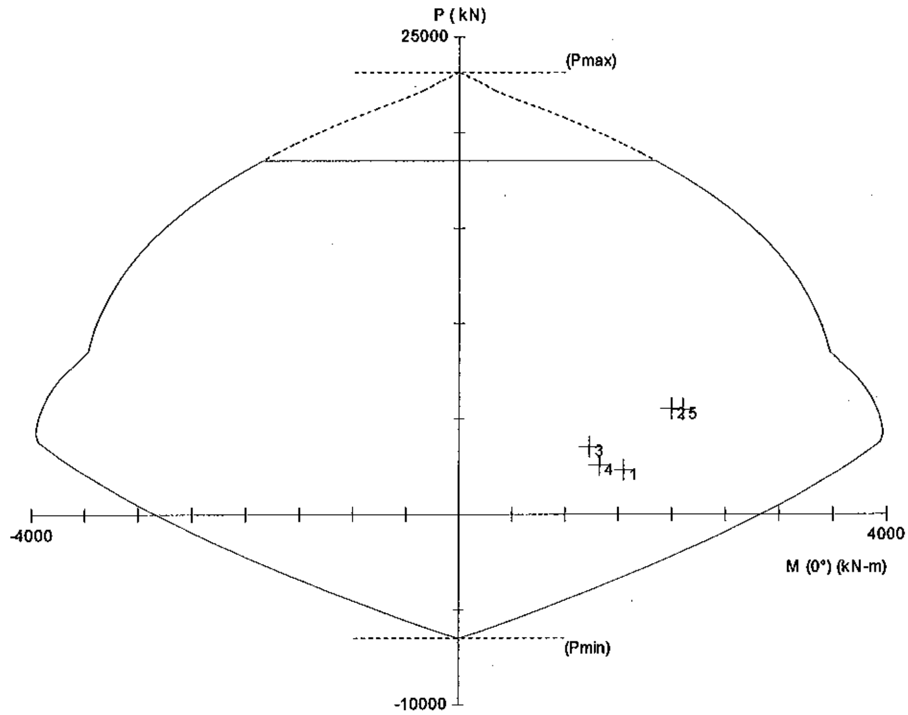
Pile diameter	D	1.20	m
Section area	A	1.131	m2
Moment inertia	Ix	0.102	m4
	Iy	0.102	m4
Radius of gyration of gross concrete section; r = sqrt(I/A)	rx	0.300	m
	ry	0.300	m

II. PILE DESIGN

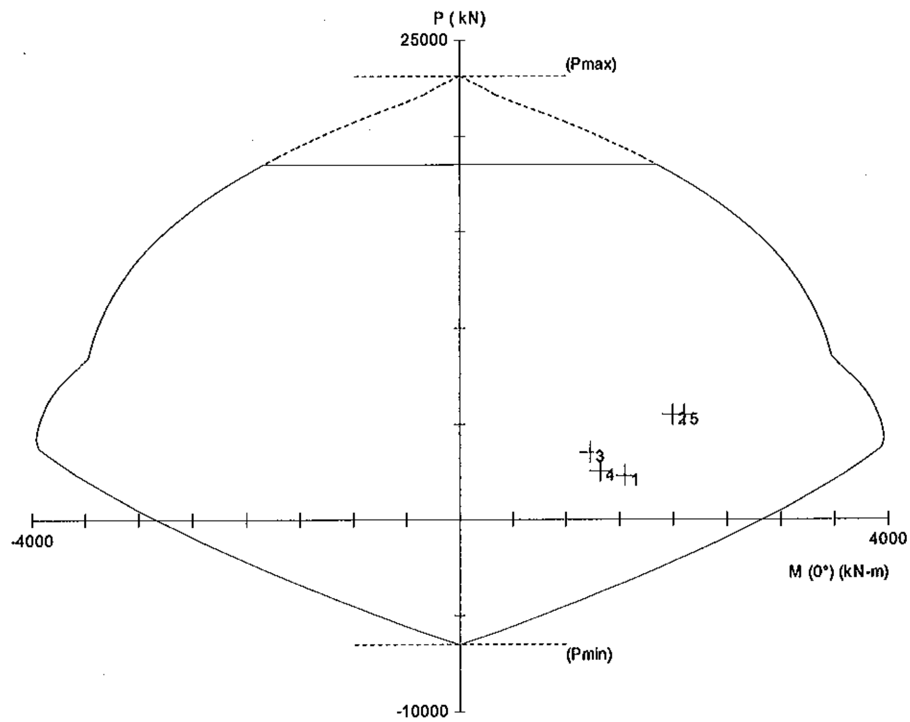
1. Limit of Reinforcement S.5.7.4.2

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

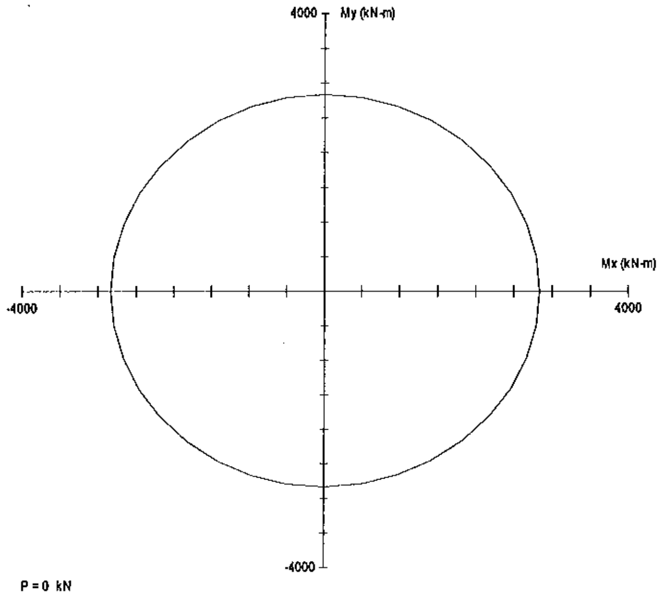
In Transverse Direction



In Longitudinal Direction



**In Both Direction



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

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

4. Shear Design

Shear resistance factors	ϕv	1.0	
Factored shear force	Vu	1011	kN
Required shear capacity Vn = Vu/ ϕv	Vn	1011	kN
Determine concrete shear capacity			
Minimum shear reinforcement will provided in cross section			
Therefore	β	2.0	
	θ	45.0	deg
Diameter of bored pile	D	1.20	m
Width of cross section	b	1.20	m
dv = 0.9*de de = D/2 + Dr/pi()			
Diameter of the circle passing through the centers of the long. reinf.	Dr	0.99	m
	de	0.92	m
	dv	0.82	m
Vc = 0.083 . β . sqrt(f'c) . bv. dv	Vc	900	kN
	A_v	1963	mm2
Angle of inclination of shear reinf. to long. axis	α	90	
Vs=Av*fy*dv*(cotgθ+cotgα)*sinα/s	Vs	8635	KN
Vn1 = Vc + Vs	V_{n1}	9535	
Vn2 = 0.25f'cbvdv	V_{n2}	7423	
	V_n	7423	
	Conclude		**OK**

ASSUMPTION:

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

DATA & CALCULATION:

Bored hole name		OP11-A1		Pile Concrete comp. strength	f'_c =	30.0	MPa
Bottom of pilecap elavation	EL1 =	11.50		Concrete Unit Weight	g_c =	24.5	kN/m^3
Top of socket elevation	EL2 =	8.90		Modulus of elasticity of concr	E_c =	27691	MPa
Pile tip elevation	EL3 =	5.50					
Pile Length	L =	6.00	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 =	4436.0	kN				
Working normal force at top of socket	P_i =	4396.7	kN				
Intack rock modulus	E_i =	25000	MPa	*Figure C10.8.3.5-2 Lrfd*			
Modulus modification ratio	K_e =	0.50		*Figure C10.8.3.5-3 Lrfd*			
Elastic modulus of the insitu rock	$E_r = K_e{*}E_i$ =	########	MPa				
Influence coefficient	$I_p = f(H_s/D_s, E_c/E_r)$ =	0.30		*Figure C10.8.3.5-1 Lrfd*			
	H_s/D_s =	3.40					
	E_c/E_r =	2.22					
Rock mass modulus/ intack rock modulus	E_m / E_i			*C.10.4.6.5-1-Lrfd 4th*			
Atmospheric pressure	p_a =	0.101	MPa				
Reduction factor to account for jointing	α_E			*10.8.3.5.4b-Lrfd 4th*			

The elastic shorterning of the drilled shaft

$r_e = (\Sigma P_i){*}H_s/(A_{soc}{*}E_c)$ = 0.687 mm

The settlement of base of drilled shaft

$r_{base} = (\Sigma P_i){*}I_p/(D_s{*}E_r)$ = 0.106 mm

$r_e + r_{base}$ = 0.793 mm < 10 mm

Compute the bearing capacity based on Sharft resistance alone

Unit side resistance

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

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

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

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

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

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

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

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

ϕ_s is the resistance factor - table 10.5.5-3 LRFD

q_u is the uniaxial compressive strength of the rock

Case1

No.	EL_T	EL_B	Depth (m)	RQD (%)	q_u (MPa)	q_s (MPa)	Q_{SR} (kN)	ϕ_s	Q_R (kN)
1	8.90	7.90	1.00	80	85.61	1.94	6104	0.65	3968
2	7.90	6.40	1.50	80	85.61	1.94	9156	0.65	5952
3	6.40	5.50	0.90	80	76.93	1.84	5208	0.65	3385
4									
5									
6									
7									
8									
Sum			**3.40**				**20469**		**13305**

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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	80.00	85.61	0.80	0.92	1	13.58	1.76	*1.76*	5524	0.55	3038
2	1.50	80.00	85.61	0.80	0.92	1	13.58	1.76	*1.76*	8286	0.55	4558
3	0.90	80.00	76.93	0.80	0.92	1	13.58	1.67	*1.67*	4713	0.55	2592
4	-	-	-	-	-	-	-	-	-	-	-	-
5	-	-	-	-	-	-	-	-	-	-	-	-
6	-	-	-	-	-	-	-	-	-	-	-	-
7	-	-	-	-	-	-	-	-	-	-	-	-
8	-	-	-	-	-	-	-	-	-	-	-	-
Sum	**3.40**									**18524**		**10188**

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

Limit pressure determined from presuremeter tests	$p_1 =$	-	MPa	*C10.8.3.5-7*
At rest total horizontal stress measured at ther base elevation	$p_0 =$	-	MPa	
Coefficient that depen on diameter socket	$K_b =$	4.34		*Table C10.8.3.5-1*
Total vertical stress at the base elevation	$\sigma v =$	-	MPa	
	$q_p =$	-	MPa	
	$\phi =$	0.50		*Table 10.5.5-3*
$Q_{pR} =$ Ap . qp	$Q_{pR} =$	-	kN	
$Q_R = \phi.Q_{pR}$	$Q_R =$	-	kN	

ESTIMATED PILE CAPACITY:

Pile Structural Capacity Q_T = 0.75*0.85*0.85*fc*Ag	*12768*	kN	*1301*	T
Pile resistance Q_R	*10188*	kN	*1039*	T
Deducting pile weight	*-96*	kN	*-10*	T
Estimated Pile Capacity	*10093*	kN	*1029*	T
Maximum Reaction - ULS **Ok**	*4453*	kN	*453.93*	T

ASSUMPTION:

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

DATA & CALCULATION:

Bored hole name		OP11-A2		Pile Concrete comp. strength	f'_c =	30.0	MPa
Bottom of pilecap elavation	EL1 =	11.50		Concrete Unit Weight	g_c =	24.5	kN/m^3
Top of socket elevation	EL2 =	5.50		Modulus of elasticity of concre	E_c =	27691	MPa
Pile tip elevation	EL3 =	2.50					
Pile Length	L =	9.00	m	Depth of socket	H_s =	3.00	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 =	4493.8	kN				
Working normal force at top of socket	P_i =	4459.1	kN				
Intack rock modulus	E_i =	25000	MPa	*Figure C10.8.3.5-2 Lrfd*			
Modulus modification ratio	K_e =	0.03		*Figure C10.8.3.5-3 Lrfd*			
Elastic modulus of the insitu rock	$E_r = K_e{}^*E_i$ =	750.0	MPa				
Influence coefficient	$I_p = f(H_s/D_s, E_c/E_r)$ =	0.30		*Figure C10.8.3.5-1 Lrfd*			
	H_s/D_s =	3.00					
	E_c/E_r =	36.92					
Rock mass modulus/ intack rock modulus	E_m / E_i			*C.10.4.6.5-1-Lrfd 4th*			
Atmospheric pressure	p_a =	0.101	MPa				
Reduction factor to account for jointing	α_E			*10.8.3.5.4b-Lrfd 4th*			

The elastic shorterning of the drilled shaft

$r_c = (\Sigma P_i)^*H_s/(A_{soc}{}^*E_c) =$ 0.615 mm

The settlement of base of drilled shaft

$r_{base} = (\Sigma P_i)^*I_p/(D_s{}^*E_r) =$ 1.784 mm

$r_c + r_{base} =$ **2.399** mm < 10 mm

Compute the bearing capacity based on Sharft resistance alone

Unit side resistance

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

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

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

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

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

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

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

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

ϕ_s is the resistance factor - table 10.5.5-3 LRFD

q_u is the uniaxial compressive strength of the rock

Case1

No.	EL_T	EL_B	Depth (m)	RQD (%)	q_u (MPa)	q_s (MPa)	Q_{SR} (kN)	ϕ_s	Q_R (kN)
1	5.50	3.50	2.00	50	58.08	1.60	10056	0.65	6536
2	3.50	2.50	1.00	50	81.84	1.90	5968	0.65	3879
3									
4									
5									
6									
7									
8									
Sum			**3.00**				**16024**		**10416**

	DANANG QUANG NGAI EXPRESSWAY	Item.	Eng.	Date.	Sign.
	OP11 BRIDGE	Design			
	DETAIL DESIGN	Check			
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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	2.00	50.00	58.08	0.15	0.59	1	13.58	0.92	*0.92*	5811	0.55	3196
2	1.00	50.00	81.84	0.15	0.59	1	13.58	1.10	*1.10*	3449	0.55	1897
3	-	-	-	-	-	-	-	-	-	-	-	-
4	-	-	-	-	-	-	-	-	-	-	-	-
5	-	-	-	-	-	-	-	-	-	-	-	-
6	-	-	-	-	-	-	-	-	-	-	-	-
7	-	-	-	-	-	-	-	-	-	-	-	-
8	-	-	-	-	-	-	-	-	-	-	-	-
Sum	**3.00**									**9260**		**5093**

Unit base resistance	$q_p = Kb.(p1\text{-}p0) + \sigma v$				
Limit pressure determined from presuremeter tests		p_l =	5.89	MPa	*C10.8.3.5-7*
At rest total horizontal stress measured at ther base elevation		p_0 =	-	MPa	
Coefficient that depen on diameter socket		K_b =	4.20		*Table C10.8.3.5-1*
Total vertical stress at the base elevation		σv =	-	MPa	
		q_p =	-	MPa	
		ϕ =	0.50		*Table 10.5.5-3*
	Q_{pR} = Ap . qp	Q_{pR} =	-	kN	
	$Q_R = \phi.Q_{pR}$	Q_R =	-	kN	

ESTIMATED PILE CAPACITY:

Pile Structural Capacity	$Q_T = 0.75*0.85*0.85*fc*Ag$	*12768*	kN	*1301*	T
Pile resistance	Q_R	*5093*	kN	*519*	T
Deducting pile weight		*-127*	kN	*-13*	T
Estimated Pile Capacity		*4966*	kN	*506*	T
Maximum Reaction - ULS	**Ok**	*4453*	kN	*453.93*	T

Da Nang Quang Ngai Expressway project

BRIDGE

CB11

Km43+650.00

CALCULATION SHEETS

ABUTMENT A2

Da Nang, 2013

Table of content

Assumptions :

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

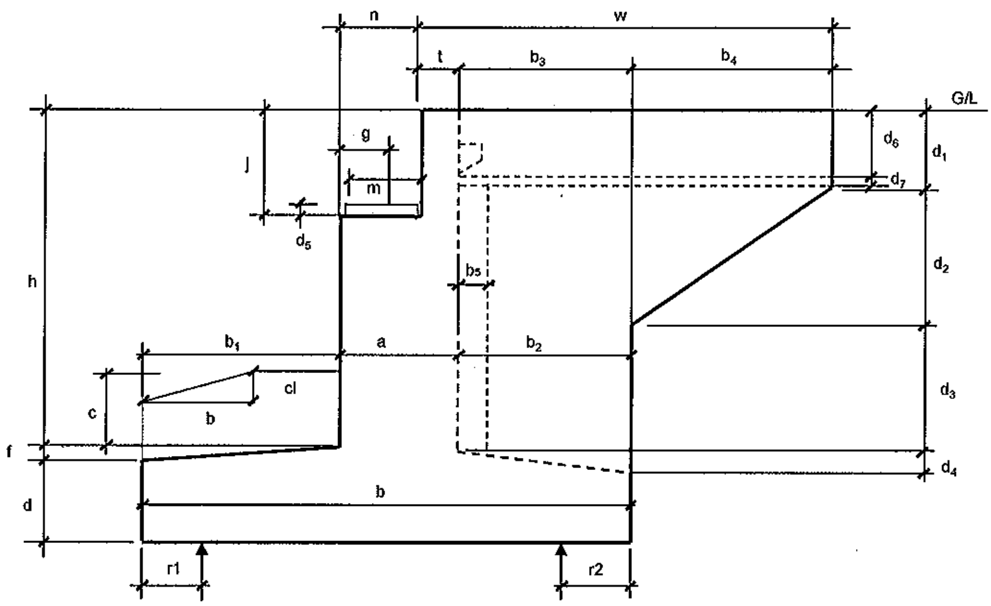
Input :

Level Table(at center of abutment)

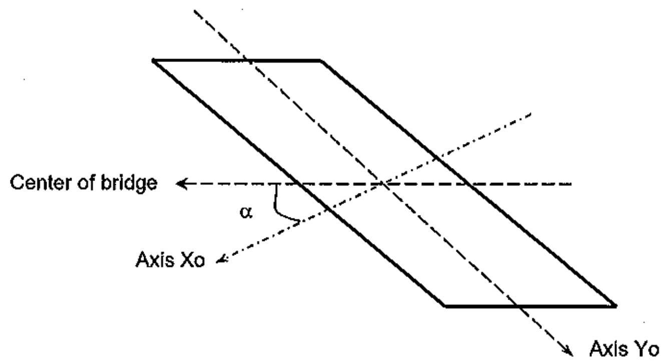
Level of top of headwall	HTwL	19.962	m
Level of top of bearing	BTL	17.998	m
Level of top of stem abutment	HTL	17.848	m
Level of top of footing	FTL	12.000	m
Level of bottom of footing	FBL	10.000	m
Ground level	GL	13.500	m
Lowest water level	HWL	13.500	m
Skew angle	α	10.00	deg

I.Loads from substructure

Abutment dimensions VERTICAL VIEW



PLAN VIEW

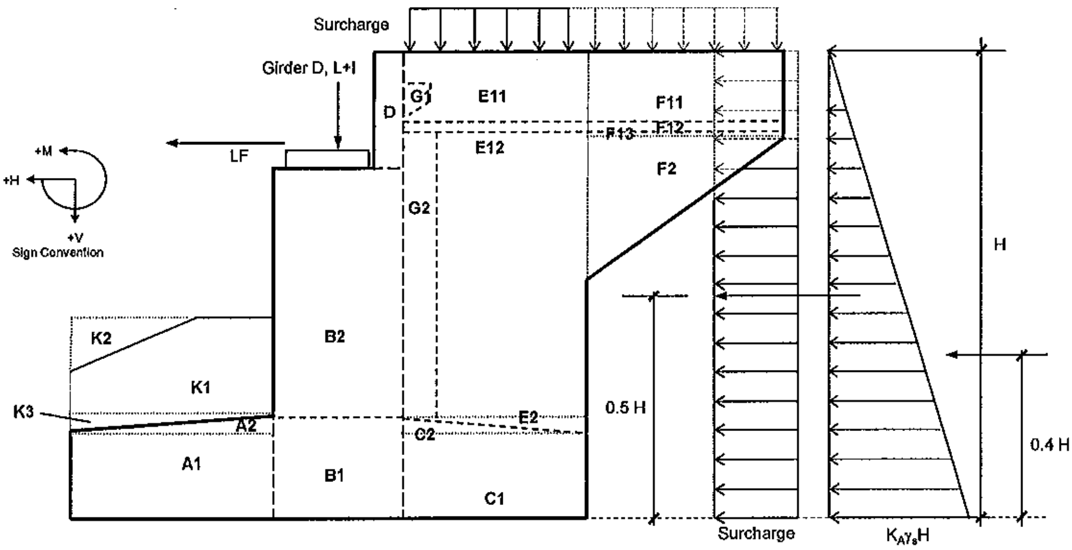


Material Unit Weights

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

ABUTMENT DIMENSIONS (IN METRES)					
Item	Symbol	Value	Item	Symbol	Value
Height of stem	h	7.962	Horizontal Dimension	b_3	3.000
Footing Width	b	6.500	Horizontal Dimension	b_4	2.446
Stem Width	a	1.500	Horizontal Dimension	b_5	0.500
Footing Depth	d	2.000	Vertical Dimension	d_1	2.000
Footing Slope	f	0.000	Vertical Dimension	d_2	2.446
Width of stem at bearing	n	1.000	Vertical Dimension	d_3	3.516
Ballast Wall Height	j	2.114	Vertical Dimension	d_4	0.000
Ballast Wall Thickness	t	0.500	Vertical Dimension	d_5	0.150
Wingwall Length	w	6.038	Vertical Dimension	d_6	1.200
Soil Cover at Toe	c	1.500	Vertical Dimension	d_7	0.300
Girder Reaction	g	0.550	With of bearing pad	m	0.550
Distance to cl of pile	r1	1.000	Wingwall Thickness	u1	0.500
Horizontal Dimension	b_1	2.000	Wingwall Thickness	u2	0.500
Horizontal Dimension	b_2	3.000	Distance to cl of pile	r2	1.000

	cos (α)	=	0.98
Slope front of abutment	cl	=	0.00 m
	bl	=	0.00 m
Width of Abutment	L		12.600 m
Width of abutment (inclined direction)	Ltr		12.794 m
Height of Abutment	Ht	=	9.96 m
Distance from CG of footing to edge of Abutment	b/2	=	3.25 m



1. Self weight of Abutment (DC)

Description	Area (m^2)	Length (m)	Force (kN)	X[(1)] (m)	Arm[(2)] (m)	Moment (kN•m)
SW of Abutment (DC)						
Section A1	4.000	12.794	1254	1.000	2.250	2821
Section A2	-	12.794	-	1.333	1.917	-
Section B1	3.000	12.794	940	2.750	0.500	470
Section B2	8.772	12.794	2750	2.750	0.500	1375
Section C1	6.000	12.794	1881	5.000	-1.750	-3291
Section C2	-	12.794	-	4.500	-1.250	-
Section D	1.057	12.794	331	3.250	-	-
Section E11	5.100	0.500	62	5.000	-1.750	-109
Section E12	17.886	0.500	219	5.000	-1.750	-383
Part extra stem	4.981	0.740	90	5.750	-2.500	-226
Section F11	2.935	0.500	36	7.723	-4.473	-161
Section F12	0.817	0.500	10	6.223	-2.973	-30
Section F13	1.223	0.500	15	7.723	-4.473	-67
Section F2	2.991	0.500	37	7.315	-4.065	-149
Section G1	0.135	12.294	283	3.650	-0.400	-113
Section G2	0.125	12.924	40	3.750	-0.500	-20
Bearing seats (w1seat= 0.65m)	0.083	3.250	9	2.550	0.700	6
Curbs +Handrail on Abutment	0.50	6.038	80	6.019	-2.769	-222
Total SW of Abutment (DC)			*8037*			*-99*
Tranverser moment			*459*		*6.175*	*2837*

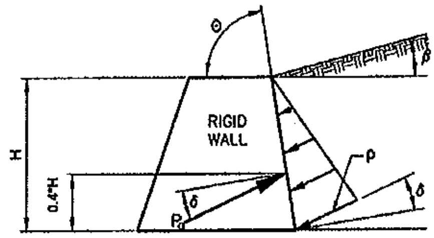
Notes: 1. Distance 'X' is measured horizontally from Toe of Retaining to CG of Section
2. Moment 'Arm' is measured from CG horizontally and from Underside of Footing Vertically.

2. Earth on Abutment (EV)

Description	Area (m^2)	Length (m)	Force (kN)	X[1] (m)	Arm[2] (m)	Moment (kN•m)
Earth on Abutment (EV)						
Section E1	23.89	12.294	5286	5.000	-1.750	-9250
Section E2	-	12.294	-	5.500	-2.250	-
Section E3	-	0.500	-	6.500	-3.250	-
Section K1	3.000	12.794	691	1.000	2.250	-
Section K2	-	12.794	-	-	3.250	-
Section K3	-	12.794	-	0.667	2.583	-
Total Earth on Footing			*5977*			*-9250*

3. Horizontal Earth Pressure on Abutment (EH)

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



- Height for horizontal earth pressure H = 9.96 m
- Width for horizontal earth pressure W 12.79 m
- Density of Soil γ_s = 1835 kg/m^3
- Internal Friction Angle of Soil ϕ'_f = 30.0 deg
- Incline angle of back face wall θ = 90.0 deg
- Friction angle between fill and wall δ = 30.0 deg
- Incline angle of fill soil β = 0.0 deg
- Gravitational acceleration g = 9.81 m/s^2
- Basic earth pressure

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

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

Γ = 2.914
Ka = 0.297
p = 0.053 Mpa

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

Horizontal earth pressure:
- Ea = 0,5. p. Z . B . 10^3 (kN) Ea = 3396 kN
- M= Ea . 0,4H M = 13532 kNm
- Horizontal Earth Pressure act at a height of 0.4 H <S 3.11.5.1>

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

Equivalent height of soil for highway loading taken from Table 3.11.6.2.1

H=	1.50m heq=	1.7 m	
H=	3.00m heq=	1.2 m	
H=	6.00m heq=	0.76 m	
H=	9.00m heq=	0.61 m	
H=	9.96m heq=	0.61 m	(Linear interpolation)

- **Vertical force**

ESv = 421 kN
ev = -1.75 m
M = -738 kNm

- **Horizontal force**

ESh = 416 kN
eh = 4.98 m
M = 2072 kNm

$$\Delta p = k\gamma_s g h_{eq} x10^{-9}$$

5. Earthquake effects

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

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

• Peak ground acceleration coefficient A = 0.0310 g

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

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

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

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

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

• Seismic active lateral Earth pressure coefficient E_{AE} = 3704 kN

$M_{AE} = E_{AS}*0.3H+(E_{AE} - E_{AS})*0.6H$ M_{AE} = 11988 KNm <A.11.1.1.1>

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

5.2. Earthquake effects to abutment (EQ)

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

• Soil profile type	Soil type		I
• Site Coefficients.	S	=	1.0
• Elastic Seismic Response Coefficient	2.5A	=	0.078
Csm = 1.2*A*S/ Tm^2/3 ≤ 2.5*A	Csm	=	0.036
• Period of vibration of the fundamental mode			
Tm = 2*pi()*sqrt(m/k)	Tm	=	1.032 s

Decription	Area (m^2)	Length (m)	Force (kN)	X[(1)] (m)	Arm[(2)] (m)	Moment (kN•m)
Section A1	4.000	12.794	39	-	1.000	39
Section A2	-	12.794	-	-	2.000	-
Section B1	3.000	12.794	29	-	1.000	29
Section B2	8.772	12.794	100	-	4.924	493
Section C1	6.000	12.794	58	-	1.000	58
Section C2	-	12.794	-	-	2.000	-
Section D	1.057	12.794	12	-	8.905	107
Section E11	5.100	0.500	2	-	7.362	14
Section E12	17.886	0.500	7	-	3.231	
Section E2	4.981	0.740	3	-	2.000	6
Section F11	2.935	0.500	1	-	7.362	8
Section F12	0.817	0.500	0	-	6.612	
Section F13	1.223	0.500	0	-	7.712	
Section F2	2.991	0.500	1	-	7.147	8
Section G1	0.135	12.294	1	-	7.249	9
Section G2	0.125	12.924	1	-	3.231	4
Total EQ of Abutment Selfweight			***256***			***776***

6. Braking Force(BR)

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

• Number of lanes	n	=	2 lanes	
• Multiple presence factor	m	=	1.00	
• Take 25 % of Truck load				
BR = 25% * n * m * (2*145+35)	BR	=	81 kN	Long. Axis
• Acting at 1.8m higher of road face	e	=	11.8 m	
	Mlong	=	961 KNm	Long. Axis

7. Centrifucal Force , CE (3.6.3)

• Plan of bridge (1:"straight",2: "Curve")			1	
• Design Speed	V	=	120 km/h	
	V	=	33.3 m/s	
	R		- m	
$C = 4/3*(V^2/gR)$	C		-	
Acting at 1.8m higher of road face				
CE = n * m * (2*145+35) * C	CE	=	0.00 KN	
	e	=	11.85 m	
	Mtrans	=	0.00 KNm	Trans. Axis

8. Water Load (WA) *:NA*

II.Loads from superstructure

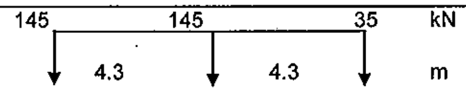
Item	Sign	Value	Unit
Span length	Lsp	27.00	m
Span between bearings	Lb	26.20	m
Skew angle	α	10.00	deg
Deck slab length	Ldeck	27.00	m
Bridge Width	Bc	12.48	m
Girder height	hgi	1.50	m
Deck slab depth	hdkslab	0.26	m
Asphalt depth	has	0.084	m
Unit weight of concrete	γc	24.50	kN/m3
Unit weight of asphalt concrete	γa	22.10	kN/m3

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

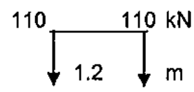
Item	Sign	Value	Unit
1.1.Girders			
Weight of 1 girder	DC	464.77	kN
Number of girders	n	5	Girders
Sum of girders weight	DC	2323.83	kN
Precast Planks	DC	397.49	kN
Diaphragm	DC	145.48	kN
Total	DC	2866.79	kN
1.2.Deck slab			
Deck slab	DC	2196.32	kN
1.3.Pavement			
Asphalt concrete	DW	575.51	kN
1.4.Handrail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

2.Live load (LL):

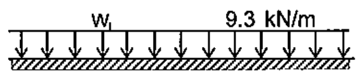
Truck



Tandem



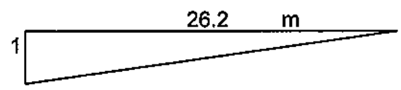
Lane load



Pedestrial Wpd= 0.0 kN/m2

Considerate structure as a simple span

Reaction Influence



Number of lanes	n	2	
Multiple presence factor	m	1.00	
Dynamic load allowance	1+IM	1.25	

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

Truck	P1(kN)	P2(kN)	P3(kN)	Sum(kN)	Total(kN)
Axle load	145	145	35		
Influence value	1.000	0.836	0.672		
Reaction	145.0	121.2	23.5	289.7	724.3

Tandem	P1(kN)	P2(kN)	Sum(kN)	Total(kN)
Axle load	110	110		
Influence value	1.000	0.954		
Reaction	110	105.0	215.0	537.4

Lane load	Wl(kN/m)	Total(kN)
Value	9.3	
Influence value	13.1	
Reaction	121.8	243.7

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

3.Earthquake effects on superstructrue (EQ)

Longitudinal moveable bearings at Abutment

Horizontal force from superstructure due to EQ - transverse direction

At bearing Heq = 114 kN

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

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

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

Shear modulus G G = 1 MPa

Bearing area A = 0.158 m2

Height of elastomeric layers hrt 0.064 m

Number of bearing nb = 5 bears

Horizontal force due to TU+SH&CR

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

5.Wind loads (Ws)

5.1.Tranverse wind on superstructure (WS)

Wind zone Zone III

Basic 3 second gust wind Vb = 53.00 m/s

Correction factor S = 1.09

Design wind velocity V = 57.77 m/s

Drag coefficient Cd = 1.40

Overall width of bridge b = 12.48 m

Depth of superstructure(including solid parapet) d = 2.82 m

b/d = 4.42

Windy obstructed area of superstructure At = 76.17 m2

Force due to tranverse wind

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

5.2.Wind load on vehicles (WL)

Transverse wind on vehicles Wltran = 1.50 kN/m

Transverse horizontal force due to wind on liveload

At 1.8m from surface Fhy = 40.50 kN

6.Combinations

Loads from superstructure to Abutment

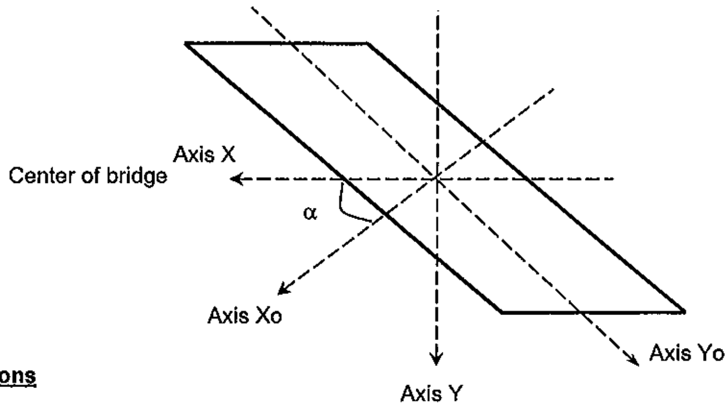
Loads at bottom of stem		Vertical		Longitudinal			Tranversal		
Loads	Sign	N (kN)	x (m)	Hx (kN)	z_1 (m)	My (kN•m)	Hy (kN)	y (m)	Mx (kN•m)
Girder + Deckslab	DC	2532	0.20			506			
Handrail	DC	320	0.20			64			
Pavement	DW	288	0.20			58			
LiveLoad	LL	968	0.20			194		1.38	1331
Pedestrial	PL	0	0.20			0		-	-
Trans. wind on Struc.	WS						107	5.85	625
Trans. wind on vehi.	WL						20	7.65	155
Eearth quake	EQ						114	5.85	669
TU+SH&CR	TU+SH&CR			320	5.85	1871			

Loads at bottom of pilecap		Vertical		Longitudinal			Tranversal		
Loads	Sign	N (kN)	x (m)	Hx (kN)	z_1 (m)	My (kN•m)	Hy (kN)	y (m)	Mx (kN•m)
Girder + Deckslab	DC	2532	0.70			1772			
Handrail	DC	320	0.70			224			
Pavement	DW	288	0.70			201			
LiveLoad	LL	968	0.70			678		1.38	1331
Pedestrial	PL	0	0.70			0		-	-
Trans. wind on Struc.	WS						107	7.85	839
Trans. wind on vehi.	WL						20	9.65	195
Eearth quake	EQ						114	7.85	897
TU+SH&CR	TU+SH&CR			320	7.85	2511			

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

Load combinations at bottom of stem		Longitudinal		Tranversal	
Load combinations	N (kN)	Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	5690	160	2073	0	2329
Strength Str-IB	4447	160	1825	0	2329
Strength Str-IIIA	5303	160	1996	63	2202
Strength Str-IIIB	4060	160	1747	63	2202
Service Ser-I	4107	320	2692	52	1673
Extreme Ext-IA	4480	0	896	114	1334
Extreme Ext-IB	3237	0	647	114	1334

Load combinations at bottom of pilecap		Longitudinal		Tranversal	
Load combinations	N (kN)	Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	5690	160	5238	0	2329
Strength Str-IB	4447	160	4368	0	2329
Strength Str-IIIA	5303	160	4967	63	2328
Strength Str-IIIB	4060	160	4097	63	2328
Service Ser-I	4107	320	5386	52	1778
Extreme Ext-IA	4480	0	3136	114	1563
Extreme Ext-IB	3237	0	2266	114	1563



III.Load Combinations

1.Loads from substructure

Loads at bottom of pilecap	Sign	Vertical		Longitudinal			Tranversal		
		N (kN)	x (m)	H_x (kN)	z_1 (m)	M_y (kN•m)	H_y (kN)	y (m)	M_x (kN•m)
Self weight of Abutment	DC	8037				-99			602.338
Soils on pilecap	EV	5977				-9250			
Horizontal Earth Pressure	EH			3344		13327			
Vertical Surcharge	L_{Sv}	421				-738			
Horizontal Surcharge	L_{Sh}			410		2040			
Braking Force	BR			81		961			
Centrifucal Force	CE			-		-	-		-
Buoyancy of Abutment	WA	-1958				-64			
Buoyancy of Earth on Abutment	WA	-897				64			
Earthquake effects to Abutment	EQ			256		776	77		233
Earthquake effects to soil	E_{AE}			3647		11806			

Table of load factors

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

Load Combination at bottom of pilecap					
Load combinations	N (kN)	Longitudinal		Tranversal	
		Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	15997	5876	11340	0	753
Strength Str-IB	10494	3869	7542	0	542
Strength Str-IIIA	15828	5679	10435	0	753
Strength Str-IIIB	10326	3673	6636	0	542
Service Ser-I	11580	3835	6241	0	602
Extreme Ext-IA	15470	4148	1102	77	986
Extreme Ext-IB	9968	4148	5300	77	775

2. Loads from superstructure

Load Combination at bottom of pilecap						
		Longitudinal		Tranversal	
Load combinations	**N**	**Hx**	**My**	**Hy**	**Mx**
	(kN)	(kN)	(kN.m)	(kN)	(kN.m)
Strength Str-IA	5690	160	5238	0	2329
Strength Str-IB	4447	160	4368	0	2329
Strength Str-IIIA	5303	160	4967	63	2328
Strength Str-IIIB	4060	160	4097	63	2328
Service Ser-I	4107	320	5386	52	1778
Extreme Ext-IA	4480	0	3136	114	1563
Extreme Ext-IB	3237	0	2266	114	1563

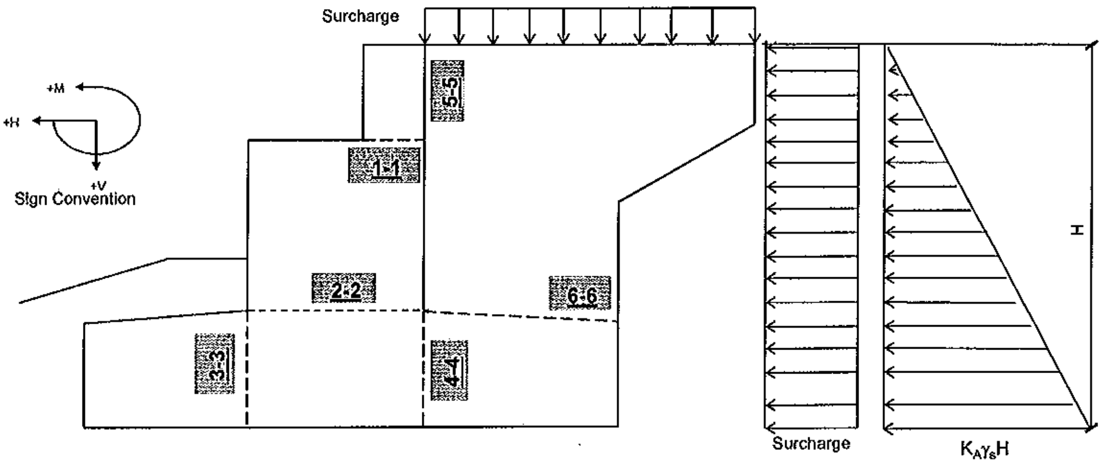
3. Total loads at bottom of pilecap

Load Combination at bottom of pilecap					
		Longitudinal		Tranversal	
Load combinations	**N**	**Hx**	**My**	**Hy**	**Mx**
	(kN)	(kN)	(kN.m)	(kN)	(kN.m)
Strength Str-IA	21687	6035	16579	0	3082
Strength Str-IB	14942	4029	11910	0	2871
Strength Str-IIIA	21131	5839	15402	63	3081
Strength Str-IIIB	14386	3833	10733	63	2870
Service Ser-I	15687	4155	11627	52	2380
Extreme Ext-IA	19950	4148	4238	191	2549
Extreme Ext-IB	13205	4148	7566	191	2338

IV.Elements checking

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

1. Calculate internal force of sections



1.1 Section 1-1

Table of Load Factors

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

Unfactored Loads	N	Longitudinal		Tranversal	
		Hx	My	Hy	Mx
	(kN)	(kN)	(kN.m)	(kN)	(kN.m)
Selfweight	615		-113		
Horizontal Earth Pressure		155	131		
Surcharge (horizontal)		220	232		
Horizontal Seismic Earth Pressure		169	116		
Abutment earthquake force		13	14	4	4

Load Combination at bottom of headwall					
Load combinations	N	Longitudinal		Tranversal	
		Hx	My	Hy	Mx
	(kN)	(kN)	(kN.m)	(kN)	(kN.m)
Service Ser-I	615	375	250	0	0
Strength Str-IA	768	617	462	0	0
Strength Str-IB	553	524	423	0	0
Extreme Ext-I	768	377	163	4	4

1.2 Section 2-2

Table of Load Factors

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

Unfactored Loads	N (kN)	Longitudinal Hx (kN)	Longitudinal My (kN.m)	Tranversal Hy (kN)	Tranversal Mx (kN.m)
Selfweight	3364		-194		
Superstructure Dead Load	2532		506		
Pavement	288		58		
Handrail+curb	320		64		
Live Load	968		194		1331
Horizontal Earth Pressure		2203	7015		
Surchage (Horizontal)		366	1458		
TU+SH&CR		320	1871		
Horizontal Seismic Earth Pressure		2402	6215		
Abutment earthquake force		113	385	68	321

Load Combination at bottom of stem wall

Load combinations	N (kN)	Longitudinal Hx (kN)	Longitudinal My (kN.m)	Tranversal Hy (kN)	Tranversal Mx (kN.m)
Service Ser-I	7471	2889	10972	0	1331
Strength Str-IA	9895	4105	14905	0	2329
Strength Str-IB	7475	2783	10516	0	2329
Extreme Ext-I	8685	3900	11090	68	987

1.3 Section 3-3

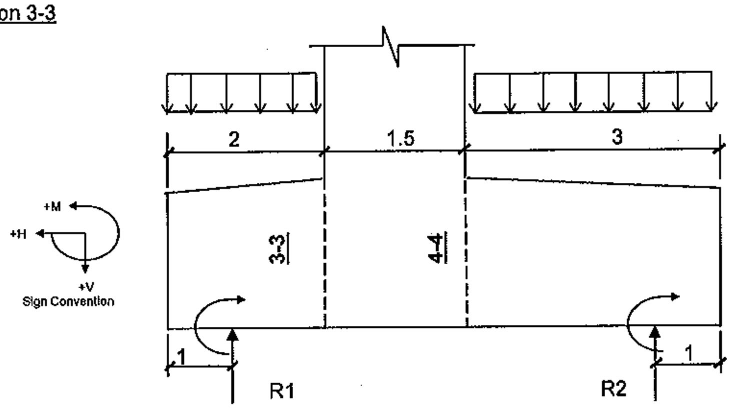


Table of Load Factors

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

Unfactored Loads		N (kN)	Longitudinal Hx (kN)	Longitudinal My (kN.m)	Tranversal Hy (kN)	Tranversal Mx (kN.m)
Selfweight at front side		1254		1254		
Vertical soil on foot at front side		691		691		
Reaction of piles	Ser-I	-11891	-2377	-7715	41	93
	Str-IA	-16671	-3448	-10621	99	200
	Str-IB	-11506	-2304	-7487	66	145
	Ext-I	-12546	-2371	-7983	-36	34

Load Combination at section 3-3

Load combinations	N (kN)	Longitudinal Hx (kN)	Longitudinal My (kN.m)	Tranversal Hy (kN)	Tranversal Mx (kN.m)
Service Ser-I	-9947	-2377	-5770	41	93
Strength Str-IA	-14171	-3448	-8121	99	200
Strength Str-IB	-9755	-2304	-5736	66	145
Extreme Ext-I	-10046	-2371	-5482	-36	34

1.4 Section 4-4

Table of Load Factors

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

Unfactored Loads		N	Longitudinal		Tranversal	
			Hx	My	Hy	Mx
		(kN)	(kN)	(kN.m)	(kN)	(kN.m)
Selfweight of behind side		2350		-3821		
Vertical soil on foot at behind side		5286		-7929		
Surcharge(Vertical)		421		-632		
Reaction of piles	Ser-I	-3795	-1783	10722	-90	-175
	Str-IA	-5018	-2586	14575	-99	-201
	Str-IB	-3435	-1728	9884	-66	-126
	Ext-I	-7407	-1778	18238	-150	-225

Load Combination at section 4-4					
		Longitudinal		Tranversal	
Load combinations	N	Hx	My	Hy	Mx
	(kN)	(kN)	(kN.m)	(kN)	(kN.m)
Service Ser-I	4263	-1783	-1660	-90	-175
Strength Str-IA	5793	-2586	-2012	-99	-201
Strength Str-IB	4175	-1728	-1797	-66	-126
Extreme Ext-I	2877	-1778	2441	-150	-225

1.4 Section 5-5 & 6-6

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

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

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

2. Elements Checking

2.1. Bearing Resistance

<S.5.7.5>

The case of absence of confinement reinforcement in the conrete supporting the bearing device					
Factored bearing resistance shall be taken					
$Pr = \phi.\ Pn = \phi.\ 0.85.\ f'c.\ A1.\ m$					
Dimension of bearing plate	w0	=	0.550	m	
	b0	=	0.650	m	
Area under bearing device	A1	=	0.358	m2	
Distributed width and length	w	=	1.000	m	
	b	=	1.100	m	
Notational area	A2	=	1.100	m2	
Where supporting surface is wider on all sides than loades area					
$m = sqrt(A2/A1) \le 2.0$ case 1					
where loaded area is subjected to nouniformly distributed bearing					
$m = 0.75*sqrt(A2/A1) \le 1.5$ case 2					
Modification factor case 1	m	=	1.754		
Resistance factor	ϕ	=	0.700		<S.5.5.4.2>
Factored bearing resistance	Pr	=	11194	kN	> Pu
Bearing reaction of approach bridge	Pu	=	4591	kN	Ok
$Pu = 1.25*DC + 1.5*DW + 1.75*LL$					

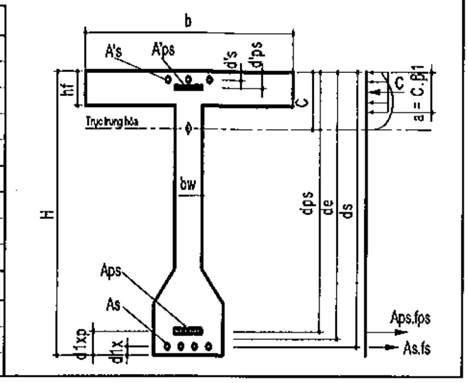
In case factored applied load exceeds the factored resistance,
provision shall be made to resist the bursting and spalling force in article 5.10.9

<S.5.10.9.7.2>

Factored bearing resistance shall be taken					
$Pr = \phi.\ fn.\ Ab$					
fn take the lesser of					
$fn = 0.7.\ f'ci.\ sqrt(A/Ag)$ and	fn	=	36.84	MPa	
$fn = 2.25.\ f'ci$					
Maximum area of the portion of supporting surface	A	=	1.100	m2	
Gross area of bearing plate	Ag	=	0.358	m2	
Effective net area of bearing plate, Ag minus stud of bearing	Ab	=	0.358	m2	
Nominal concrete strength at time of application	f'ci	=	30	MPa	
Factored bearing resistance	Pr	=	9218	kN	Ok

MATERIALS

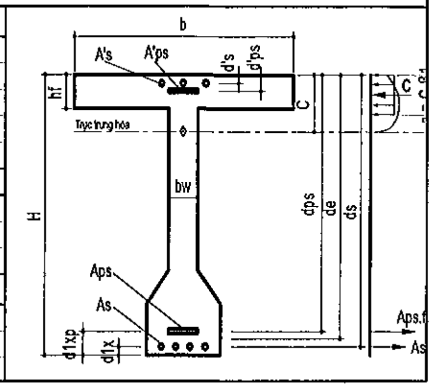
	NORMAL CONCRETE		
f'c	Compressive Strength of concrete at 28 days	Mpa	30
Ec	Modulus of Elasticity	Mpa	27691
fr	Modulus of Rupture	Mpa	3.5
gc	Unit weight of concrete	kN/m3	24.5
	PRESTRESSING STEEL		
fpu	Tensile strength of prestressing steel	Mpa	1860
fpy	Yield strength of prestressing steel	Mpa	1670
Ep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



Sign	Parameters		Unit	Sections 1-1	1-1	2-2	2-2	2-2
	INTERNAL FORCES AT SECTIOIN							
	Combination			Strength	Service	Service	Strength	Extreme
Qu	Shear		kN	617	375	2889	4105	3900
Mu	Flexural Moment		kNm	462	250	10972	14905	11090
Nu	Axial load		kN	768	615	7471	9895	8685
Tu	Torsional Moment		kNm	0	0	0	0	0
	FLEXURAL MOMENT CHECKING							
H	Section height		m	0.500	0.500	1.500	1.500	1.500
d's	Dis. From comp. fiber to centroid of comp. Reinf		m	0.058	0.058	0.058	0.058	0.058
d1x	Dis. From tens. fiber to centroid of tension Reinf		m	0.058	0.058	0.059	0.059	0.059
	Cover to reinf		m	0.050	0.050	0.050	0.050	0.050
ds	Dis. From comp. fiber to centroid of tension Reinf		m	0.442	0.442	1.441	1.441	1.441
d'ps	Dis. From comp. fiber to centroid of comp. prestressing steel		m	0.000	0.000	0.000	0.000	0.000
d1xp	Dis. From tens. fiber to centroid of tension prestressing steel		m	0.000	0.000	0.000	0.000	0.000
dps	Dis. From comp. fiber to centroid of tension prestressing steel		m	0.500	0.500	1.500	1.500	1.500
b	Width of the compression face of member		m	12.794	12.794	12.794	12.794	12.794
bw	Web width or diameter of a circular section		m	12.794	12.794	12.794	12.794	12.794
hf	Compression flange depth		m	0.000	0.000	0.000	0.000	0.000
Iz	Moment of inertia of section		m4	0.133	0.133	3.598	3.598	3.598
Amc	Section area		m2	6.397	6.397	19.192	19.192	19.192
	Steel choice							
Aps	Tension prestressing steel	P.S type		0	0	0	0	0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
A'ps	Compression prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	0 T00.0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
As	Tension Reinforcement	Number	bars	83	83	77	77	77
		Diameter	mm	16	16	25	25	25
		Area	m2	0.01677	0.01677	0.03781	0.03781	0.03781
A's	Compression Reinforcement	Number	bars	83	83	77	77	77
		Diameter	mm	16	16	16	16	16
		Area	m2	0.01677	0.01677	0.01555	0.01555	0.01555
A'c	Shear reinforcement	Number	bars	20	20	19	19	19
		Diameter	mm	14	14	14	14	14
		Area	m2	0.00302	0.00302	0.00287	0.00287	0.00287
ϕ	Resistance factors for flexure		5.5.4.2	0.90	1.00	1.00	0.90	1.00
ϕv	Resistance factors for shear			0.90	1.00	1.00	0.90	1.00
ϕn	Resistance factors for axial force			1.00	1.00	1.00	1.00	1.00
β1	Stress block factor			0.836	0.836	0.836	0.836	0.836
c	Dis. Between centroid and top fiber		m	0.000	0.000	0.033	0.033	0.033
	For T section behavior		m	0.000	0.000	0.033	0.033	0.033
	For rectangular section behavior		m	0.000	0.000	0.033	0.033	0.033
fpe	Effective stress in the prestressing steel after losses		Mpa	1116	1116	1116	1116	1116
fps	Aver. stress in pres. steel at the time for which the nominal resistanc		Mpa	1860	1860	1848	1848	1848
k	Factor depends on type of P.S, Low relaxation strand k = 0.28			0.28	0.28	0.28	0.28	0.28
a	Depth of equivalent stress block		m	0.000	0.000	0.027	0.027	0.027
de	Corresp. effective depth from extreme comp. fiber							
	to centroid of tensile force in the tensile reinf.		m	0.442	0.442	1.441	1.441	1.441
Mn	Nominal resistance		kNm	2575	2575	21310	21310	21310
Mr	Factored reistance		kNm	**2318**	**2575**	**21310**	**19179**	**21310**
Mu	Flexual moment		kNm	462	250	10972	14905	11090

(5.7.3.2)	**Flexural moment Checking**		**OK**	**OK**	**OK**	**OK**	**OK**
	Limits for reinforcement						
c/de	Maximum reinforcement		0.00	0.00	0.02	0.02	0.02
	Maximum reinforcement Checking	<= 0.42	**OK**	**OK**	**OK**	**OK**	**OK**
1.2*Mcr	Craking moment	kNm	1104	1104	10155	10155	10155
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**		**OK**	**OK**	**OK**	**OK**	**OK**
(5.7.3.4)	**Conctrol of craking by distr. of reinf for RC member- Check?**		No	Yes	Yes	No	No
	Existing condition for structrure	1,2 or 3	1	1	1	1	1
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.058	0.058	0.059	0.059	0.059
Z	Crack width parameter	N/mm	30000	30000	30000	30000	30000
A	Area of concr. with same centroid as tens. Reinf	m2	0.018	0.018	0.020	0.020	0.020
fsa	Value	Mpa	296	296	286	286	286
0.6*fy		Mpa	240	240	240	240	240
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	240	240	240	240	240
x	Dist. From compression fiber to centroid	m	-	0.081	0.224	-	-
J.d	Arm	m	-	0.415	1.366	-	-
Icr	Moment of inertia of the cracked section	m4	-	0.018	0.442	-	-
fs	Tensile stress in reinforcement fs = Msls / (As*J.d)	Mpa	-	36	212	-	-
	Checking for control cracking fs<fsa		**N.a**	**OK**	**OK**	**N.a**	**N.a**
(5.10.8.2)	**Shrinkage and temperature Reinfocement (side distribution)**						
Areq	Area of required reinf	m2	0.00045	0.00045	0.00126	0.00126	0.00126
	Distribution on sides 7 D16	m2	0.00141	0.00141	0.00141	0.00141	
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45	0.45
	Checking		**OK**	**OK**	**OK**	**OK**	**OK**
	SHEAR AND TORSION CHECKING						
β	Factor indicating diag. cracked concr. to tension		3.0	3.8	2.3	2.1	2.3
θ	Angle of inclination of diagonal compressive	degree	28.73	27.68	34.43	37.94	34.74
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	90
bv	Effective web width as minimum web width - in dv	m	12.794	12.794	12.794	12.794	12.794
dv	Effective shear depth	m	0.442	0.442	1.427	1.427	1.427
	(de - a/2)	m	0.442	0.442	1.427	1.427	1.427
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	0.600
ncat	Amount of bars in spacing S	bars	20	20	19	19	19
Av	Shear reinf area in spacing S	m2	0.0030	0.0030	0.0029	0.0029	0.0029
θ	Assume	degree	28.87	28.19	28.88	30.16	30.27
v	Shear stress in concrete	kN/m2	121	66	158	250	214
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e_x	Strain in tensile reinforcement		3.64E-04	1.82E-04	8.69E-04	1.19E-03	8.95E-04
	if ex<0, multiple with reduce factor		-	-	-	-	-
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok	Ok
v/f'c	Ratio of shear stress and f'c		0.004	0.002	0.005	0.008	0.007
β	Final value		3.0	3.8	2.3	2.1	2.3
θ	Final value	degree	28.73	27.68	34.43	37.94	34.74
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	7824	9677	19124	17613	19002
Vs	Shear resistance provided by shear reinforcement	kN	1624	1697	3983	3502	3937
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	9448	11373	23107	21115	22938
Vn2	Vn2	kN	42413	42413	136966	136966	136966
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	9448	11373	23107	21115	22938
Vr	Factored shear resistance	kN	**8503**	**11373**	**23107**	**19003**	**22938**
Vu	Shear	kN	617	375	2889	4105	3900
(5.8.2.7)	**Shear checking**		**OK**	**OK**	**OK**	**OK**	**OK**
	Region requiring transverse reinf Checking		No need	No need	No need	No need	No need
	Minimum shear reinf area	m2	0.0087	0.0087	0.0087	0.0087	0.0087
	Minimum shear reinforcement Checking		-	-	-	-	-
	0.1*f'c*bv*dv	kN	16965	16965	54786	54786	54786
	Smax	m	0.35	0.35	0.60	0.60	0.60
	Maximum spacing Smax		-	-	-	-	-

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

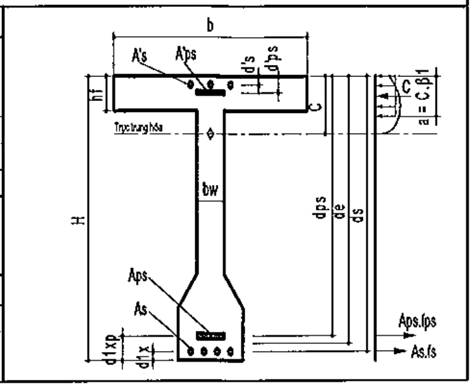


Sign	Parameters		Unit	Sections				
				3-3	3-3	3-3	4-4	4-4
INTERNAL FORCES AT SECTIOIN								
	Combination			Service	Strength	Extreme	Extreme	Strength
Qu	Shear		kN	9947	14171	10046	2877	5793
Mu	Flexural Moment		kNm	5770	8121	5482	2441	2012
Nu	Axial load		kN	2377	3448	2371	1778	2586
Tu	Torsional Moment		kNm	0	0	0	0	0
FLEXURAL MOMENT CHECKING								
H	Section height		m	2.000	2.000	2.000	2.000	2.000
d's	Dis. From comp. fiber to centroid of comp. Reinf		m	0.084	0.084	0.084	0.161	0.161
d1x	Dis. From tens. fiber to centroid of tension Reinf		m	0.161	0.161	0.161	0.084	0.084
	Cover to reinf		m	0.050	0.050	0.050	0.050	0.050
ds	Dis. From comp. fiber to centroid of tension Reinf		m	1.839	1.839	1.839	1.916	1.916
d'ps	Dis. From comp. fiber to centroid of comp. prestressing steel		m	0.000	0.000	0.000	0.000	0.000
d1xp	Dis. From tens. fiber to centroid of tension prestressing steel		m	0.000	0.000	0.000	0.000	0.000
dps	Dis. From comp. fiber to centroid of tension prestressing steel		m	2.000	2.000	2.000	2.000	2.000
b	Width of the compression face of member		m	12.794	12.794	12.794	12.794	12.794
bw	Web width or diameter of a circular section		m	12.794	12.794	12.794	12.794	12.794
hf	Compression flange depth		m	0.000	0.000	0.000	0.000	0.000
Iz	Moment of inertia of section		m4	8.530	8.530	8.530	8.530	8.530
Amc	Section area		m2	25.589	25.589	25.589	25.589	25.589
	Steel choice							
Aps	Tension prestressing steel	P.S type		0	0	0	0	0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
A'ps	Compression prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	0 T00.0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
As	Tension Reinforcement	Number	bars	84	84	84	84	84
		Diameter	mm	22	22	22	20	20
		Area	m2	0.03192	0.03192	0.03192	0.02638	0.02638
A's	Compression Reinforcement	Number	bars	0	0	0	0	0
		Diameter	mm	20	20	20	22	22
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
A'c	Shear reinforcement	Number	bars	20	20	20	20	20
		Diameter	mm	16	16	16	16	16
		Area	m2	0.00404	0.00404	0.00404	0.00404	0.00404
ϕ	Resistance factors for flexure		5.5.4.2	1.00	0.90	1.00	1.00	0.90
ϕv	Resistance factors for shear			1.00	0.90	1.00	1.00	0.90
ϕn	Resistance factors for axial force			1.00	1.00	1.00	1.00	1.00
β1	Stress block factor			0.836	0.836	0.836	0.836	0.836
c	Dis. Between centroid and top fiber		m	0.047	0.047	0.047	0.039	0.039
	For T section behavior		m	0.047	0.047	0.047	0.039	0.039
	For rectangular section behavior		m	0.047	0.047	0.047	0.039	0.039
fpe	Effective stress in the prestressing steel after losses		Mpa	1116	1116	1116	1116	1116
fps	Aver. stress in pres. steel at the time for which the nominal resistance		Mpa	1848	1848	1848	1850	1850
k	Factor depends on type of P.S, Low relaxation strand k = 0.28			0.28	0.28	0.28	0.28	0.28
a	Depth of equivalent stress block		m	0.039	0.039	0.039	0.032	0.032
de	Corresp. effective depth from extreme comp. fiber							
	to centroid of tensile force in the tensile reinf.		m	1.839	1.839	1.839	1.916	1.916
Mn	Nominal resistance		kNm	23231	23231	23231	20044	20044
Mr	Factored reistance		kNm	**23231**	**20907**	**23231**	**20044**	**18040**
Mu	Flexual moment		kNm	5770	8121	5482	2441	2012

(5.7.3.2)	**Flexural moment Checking**		**OK**	**OK**	**OK**	**OK**	**OK**
	Limits for reinforcement						
c/de	Maximum reinforcement		0.03	0.03	0.03	0.02	0.02
	Maximum reinforcement Checking	<= 0.42	**OK**	**OK**	**OK**	**OK**	**OK**
1.2*Mcr	Craking moment	kNm	18083	18083	18083	18008	18008
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**		**OK**	**OK**	**OK**	**OK**	**OK**
(5.7.3.4)	**Conctrol of craking by distr. of reinf for RC member- Check?**		Yes	No	No	No	No
	Existing condition for structrure	1,2 or 3	3	3	3	3	3
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.061	0.061	0.061	0.060	0.060
Z	Crack width parameter	N/mm	17500	17500	17500	17500	17500
A	Area of concr. with same centroid as tens. Reinf	m2	0.019	0.019	0.019	0.018	0.018
fsa	Value	Mpa	168	168	168	170	170
0.6*fy		Mpa	240	240	240	240	240
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	168	168	168	170	170
x	Dist. From compression fiber to centroid	m	0.237	-	-	-	-
J.d	Arm	m	1.76	-	-	-	-
Icr	Moment of inertia of the cracked section	m4	0.63	-	-	-	-
fs	Tensile stress in reinforcement fs = Msls / (As*J.d)	Mpa	103	-	-	-	-
	Checking for control cracking fs<fsa		**OK**	**N.a**	**N.a**	**N.a**	**N.a**
(5.10.8.2)	**Shrinkage and temperature Reinfocement (side distribution)**						
Areq	Area of required reinf	m2	0.00127	0.00127	0.00127	0.00127	0.00127
	Distribution on sides 7 D16	m2	0.00141	0.00141	0.00141	0.00141	0.00141
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45	0.45
	Checking		**OK**	**OK**	**OK**	**OK**	**OK**
	SHEAR AND TORSION CHECKING						
β	Factor indicating diag. cracked concr. to tension		2.1	1.9	2.1	2.5	2.4
θ	Angle of inclination of diagonal compressive	degree	39.00	41.75	38.35	29.50	33.03
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	90
bv	Effective web width as minimum web width - in dv	m	12.794	12.794	12.794	12.794	12.794
dv	Effective shear depth	m	1.819	1.819	1.819	1.900	1.900
	(de - a/2)	m	1.819	1.819	1.819	1.900	1.900
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	0.600
ncat	Amount of bars in spacing S	bars	20	20	20	20	20
Av	Shear reinf area in spacing S	m2	0.0040	0.0040	0.0040	0.0040	0.0040
θ	Assume	degree	38.22	41.43	39.67	30.88	34.58
v	Shear stress in concrete	kN/m2	427	676	432	33	265
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e_x	Strain in tensile reinforcement		1.30E-03	1.69E-03	1.24E-03	5.31E-04	7.52E-04
	if ex<0, multiple with reduce factor		-	-	-	-	-
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok	Ok
v/f'c	Ratio of shear stress and f'c		0.014	0.023	0.014	0.001	0.009
β	Final value		2.1	1.9	2.1	2.5	2.4
θ	Final value	degree	39.00	41.75	38.35	29.50	33.03
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	21822	19727	22206	27544	26175
Vs	Shear resistance provided by shear reinforcement	kN	6052	5491	6194	9046	7871
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	27874	25218	28400	36590	34047
Vn2	Vn2	kN	174589	174589	174589	182304	182304
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	27874	25218	28400	36590	34047
Vr	Factored shear resistance	kN	**27874**	**22696**	**28400**	**36590**	**30642**
Vu	Shear	kN	9947	14171	10046	2877	5793
(5.8.2.7)	**Shear checking**		**OK**	**OK**	**OK**	**OK**	**OK**

MATERIALS

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



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

a	Depth of equivalent stress block	m	0.012	0.012	0.012	0.012	0.012
de	Corresp. effective depth from extreme comp. fiber						
	to centroid of tensile force in the tensile reinf.	m	0.441	0.441	0.441	0.441	0.441
Mn	Nominal resistance	kNm	353	353	353	353	353
Mr	Factored resistance	kNm	**353**	**318**	**353**	**318**	**318**
Mu	Flexual moment	kNm	172	273	179	278	278
(5.7.3.2)	**Flexural moment Checking**		**OK**	**OK**	**OK**	**OK**	**OK**
	Limits for reinforcement						
c/de	Maximum reinforcement		0.03	0.03	0.03	0.03	0.03
	Maximum reinforcement Checking	<= 0.42	**OK**	**OK**	**OK**	**OK**	**OK**
r min	Mimimum reinforcement		0.44%	0.44%	0.44%	0.44%	0.44%
	Mimimum reinforcement Checking for RC	0.23%	**OK**	**OK**	**OK**	**OK**	**OK**
1.2*Mcr	Craking moment	kNm	89	89	89	89	89
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**		**OK**	**OK**	**OK**	**OK**	**OK**
(5.7.3.4)	**Conctrol of craking by distr. of reinf for RC member- Check?**		Yes	No	Yes	No	No
	Existing condition for structrure	1,2 or 3	1	1	1	1	1
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.059	0.059	0.059	0.059	0.059
Z	Crack width parameter	N/mm	30000	30000	30000	30000	30000
A	Area of concr. with same centroid as tens. Reinf	m2	0.017	0.017	0.017	0.017	0.017
fsa	Value	Mpa	301	301	301	301	301
0.6*fy		Mpa	240	240	240	240	240
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	240	240	240	240	240
x	Dist. From compression fiber to centroid	m	0.102	-	0.102	-	-
J.d	Arm	m	0.407	-	0.407	-	-
Icr	Moment of inertia of the cracked section	m4	0.002	-	0.002	-	-
fs	Tensile stress in reinforcement fs = Msls / (As*J.d)	Mpa	192	-	200	-	-
	Checking for control cracking fs<fsa		**OK**	**N.a**	**OK**	**N.a**	**N.a**
(5.10.8.2)	**Shrinkage and temperature Reinfocement (side distribution)**						
Areq	Area of required reinf	m2	0.00031	0.00031	0.00031	0.00031	0.00031
	Distribution on sides 7 D16	m2	0.00141	0.00141	0.00141	0.00141	0.00141
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45	0.45
	Checking		**OK**	**OK**	**OK**	**OK**	**OK**
	SHEAR AND TORSION CHECKING						
β	Factor indicating diag. cracked concr. to tension		2.2	1.9	2.1	1.8	1.8
θ	Angle of inclination of diagonal compressive	degree	36.15	41.31	38.29	42.39	42.39
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	90
bv	Effective web width as minimum web width - in dv	m	1.000	1.000	1.000	1.000	1.000
dv	Effective shear depth	m	0.435	0.435	0.435	0.435	0.435
	(de - a/2)	m	0.435	0.435	0.435	0.435	0.435
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	0.600
ncat	Amount of bars in spacing S	bars	3	3	3	3	3
Av	Shear reinf area in spacing S	m2	0.0003	0.0003	0.0003	0.0003	0.0003
β	Assume		2.0	2.0	2.0	2.0	2.0
θ	Assume	degree	36.15	41.31	38.30	42.39	42.39
v	Shear stress in concrete	kN/m2	170	296	467	805	805
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e_x	Strain in tensile reinforcement		1.01E-03	1.58E-03	1.23E-03	1.85E-03	1.85E-03
	if ex<0, multiple with reduce factor		-	-	-	-	-
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok	Ok
v/f'c	Ratio of shear stress and f'c		0.006	0.010	0.016	0.027	0.027
β	Final value		2.2	1.9	2.1	1.8	1.8
θ	Final value	degree	36.15	41.31	38.29	42.39	42.39
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	439	378	416	354	354
Vs	Shear resistance provided by shear reinforcement	kN	135	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	Vn1=Vc+Vs+Vp	kN	574	490	540	462	462
Vn2	Vn2	kN	3261	3261	3261	3261	3261
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	574	490	540	462	462
Vr	Factored shear resistance	kN	**574**	**441**	**540**	**416**	**416**
Vu	Shear	kN	74	116	203	315	315
(5.8.2.7)	**Shear checking**		**OK**	**OK**	**OK**	**OK**	**OK**

SPACE PILE FOUNDATION ANALYSIS PROGRM
Turbo BASIC

PROJECT: E.X Da Nang - Quang Ngai
Bridge: CB11 - KM43+655

INITIA DATA

Kn = 0.00 Ax = 6.50 By = 12.60 Cz = 2.00
E v.uon = 2944008 E r.uon = 2944008 E v.nen = 2944008
E r.nen = 2944008
Mq = 0 (t/m4) Md = 0 (t/m4) m = 2038.735961914062 (t/m4)

LOAD COMBINATIONS

COMB.	Hx	Hy	P	Mx	My	Mz
1	615.00	0.00	2211.00	-314.00	1690.00	0.00
2	411.00	0.00	1523.00	-293.00	1214.00	0.00
3	595.00	6.00	2154.00	-314.00	1570.00	0.00
4	391.00	6.00	1466.00	-293.00	1094.00	0.00
5	424.00	5.00	1599.00	-243.00	1185.00	0.00
6	423.00	19.00	2034.00	-260.00	432.00	0.00
7	423.00	19.00	1346.00	-238.00	771.00	0.00

PROPERTIES OF PILES

PILE	Lo	H	Bpx	Bpy	A	B	Cday	Fo	Io	Po	Co	Ct
1	0.00	11.50	1.384	1.384	1.00	1.000	0.000	1.000	0.098	0	150000	75000
2					n	t						
3					n	t						
4					n	t						
5					n	t						
6					n	t						
7					n	t						

PILE COORD.

PILE	X	Y	Phi	Xi
1	3.11	4.66	0.000	0.00
2	2.52	1.34	0.000	0.00
3	1.94	-1.98	0.000	0.00
4	1.35	-5.30	0.000	0.00
5	-3.22	-5.30	0.000	0.00
6	-2.34	-0.32	0.000	0.00
7	-1.46	4.66	0.000	0.00

FORCES ON PILES

PILE	COMB.	N	Q2	Q3	M1	M2	M3
1	1	473.45	-94.26	3.65	1.720	7.384	167.203
	2	330.23	-62.99	2.44	1.149	5.224	111.115
	3	457.68	-91.24	2.70	1.678	5.715	162.701
	4	314.47	-59.98	1.49	1.108	3.556	106.614
	5	339.33	-65.03	1.82	1.198	3.956	115.484
	6	363.97	-65.00	-0.11	1.228	2.513	125.586
	7	283.33	-65.00	-0.11	1.228	0.439	119.220
2	1	441.05	-89.99	2.89	1.720	5.855	158.531
	2	305.55	-60.14	1.93	1.149	4.203	105.320

	3	425.54	-87.08	1.97	1.678	4.223	154.240
	4	290.03	-57.23	1.01	1.108	2.571	101.029
	5	315.14	-62.06	1.30	1.198	2.892	109.446
	6	334.48	-61.95	-0.65	1.228	1.421	119.395
	7	261.05	-61.95	-0.65	1.228	-0.653	113.030
3	1	408.66	-85.72	2.14	1.720	4.326	149.858
	2	280.87	-57.29	1.43	1.149	3.181	99.524
	3	393.39	-82.92	1.23	1.678	2.731	145.778
	4	265.60	-54.48	0.52	1.108	1.586	95.444
	5	290.94	-59.09	0.78	1.198	1.827	103.407
	6	304.98	-58.91	-1.19	1.228	0.330	113.204
	7	238.77	-58.91	-1.19	1.228	-1.744	106.839
4	1	376.27	-81.46	1.39	1.720	2.797	141.186
	2	256.19	-54.44	0.93	1.149	2.159	93.729
	3	361.25	-78.76	0.50	1.678	1.239	137.317
	4	241.17	-51.74	0.04	1.108	0.601	89.859
	5	266.75	-56.12	0.25	1.198	0.762	97.369
	6	275.48	-55.86	-1.72	1.228	-0.762	107.013
	7	216.49	-55.86	-1.72	1.228	-2.836	100.648
5	1	121.93	-81.46	-4.48	1.720	-9.139	141.186
	2	79.70	-54.44	-3.00	1.149	-5.818	93.729
	3	123.83	-78.76	-5.23	1.678	-10.407	137.317
	4	81.59	-51.74	-3.75	1.108	-7.085	89.859
	5	92.65	-56.12	-3.84	1.198	-7.548	97.369
	6	207.45	-55.86	-5.92	1.228	-9.282	107.013
	7	82.03	-55.86	-5.92	1.228	-11.357	100.648
6	1	170.52	-87.86	-3.36	1.720	-6.845	154.195
	2	116.72	-58.71	-2.24	1.149	-4.285	102.422
	3	172.05	-85.00	-4.13	1.678	-8.169	150.009
	4	118.24	-55.86	-3.02	1.108	-5.608	98.236
	5	128.95	-60.57	-3.05	1.198	-5.951	106.426
	6	251.70	-60.43	-5.11	1.228	-7.645	116.300
	7	115.45	-60.43	-5.11	1.228	-9.719	109.934
7	1	219.11	-94.26	-2.23	1.720	-4.552	167.203
	2	153.74	-62.99	-1.49	1.149	-2.752	111.115
	3	220.27	-91.24	-3.03	1.678	-5.931	162.701
	4	154.89	-59.98	-2.29	1.108	-4.131	106.614
	5	165.24	-65.03	-2.27	1.198	-4.354	115.484
	6	295.94	-65.00	-4.30	1.228	-6.008	125.586
	7	148.87	-65.00	-4.30	1.228	-8.082	119.220

SUMMARY OF FORCES

	PILE	COMB.	N	Q2	Q3	M1	M2	M3
Nmin	5	2	79.70	-54.44	-3.00	1.149	-5.818	93.729
Nmax	1	1	473.45	-94.26	3.65	1.720	7.384	167.203
Q2max	1	1	473.45	-94.26	3.65	1.720	7.384	167.203
Q3max	5	6	207.45	-55.86	-5.92	1.228	-9.282	107.013
M1max	1	1	473.45	-94.26	3.65	1.720	7.384	167.203
M2max	5	7	82.03	-55.86	-5.92	1.228	-11.357	100.648
M3max	1	1	473.45	-94.26	3.65	1.720	7.384	167.203

CHECKING CALCULATI
IN COMPARISON WITH INITIA LOAD MATRIX

1	615.00	0.00	2211.00	-314.00	1690.00	0.00
2	411.00	0.00	1523.00	-293.00	1214.00	0.00
3	595.00	6.00	2154.00	-314.00	1570.00	0.00
4	391.00	6.00	1466.00	-293.00	1094.00	0.00
5	424.00	5.00	1599.00	-243.00	1185.00	0.00
6	423.00	19.00	2034.00	-260.00	432.00	0.00
7	423.00	19.00	1346.00	-238.00	771.00	0.00

f.BORED PILE DESIGN

I. BORED PILE DATA

1.Load Combinations at top of bored pile

No	Combinations	Sign	F_V (kN)	Longitudinal F_{HX} (kN)	Longitudinal My (kN•m)	Transvesal F_{HY} (kN)	Transvesal Mx (kN•m)
1	Strength Str-IB		782	534	-919	29	57
2	Strength Str-IA		4645	925	-1640	-36	-72
3	Strength Str-IA		4645	925	-1640	-36	-72
4	Strength Str-IA		4645	925	-1640	-36	-72
5	Strength Str-IA		4645	925	-1640	-36	-72
6							

2. Bored pile Material

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

3. Bored pile Section

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

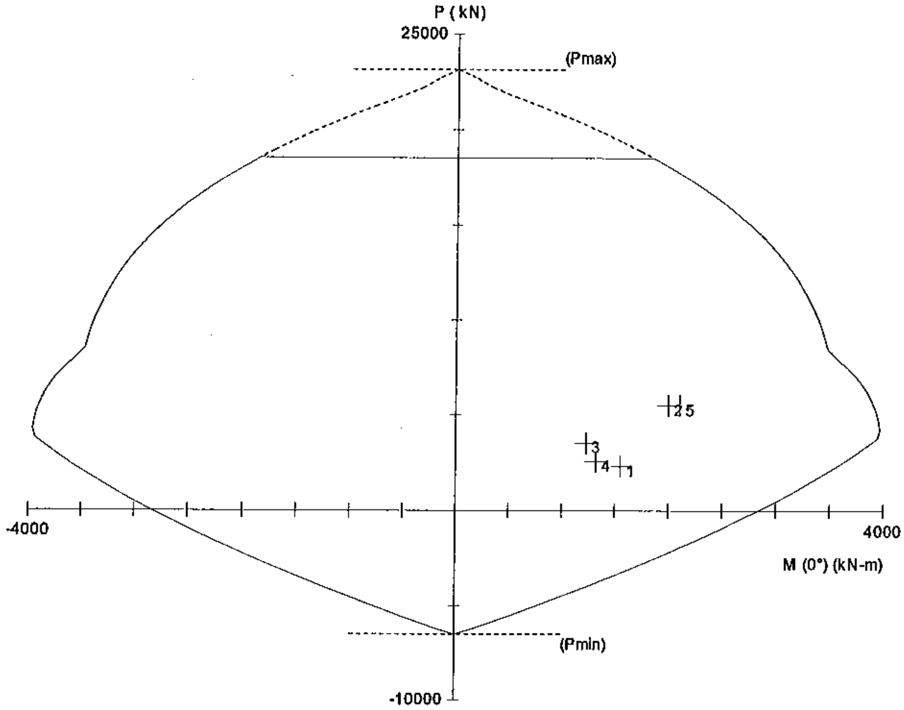
II. PILE DESIGN

1. Limit of Reinforcement

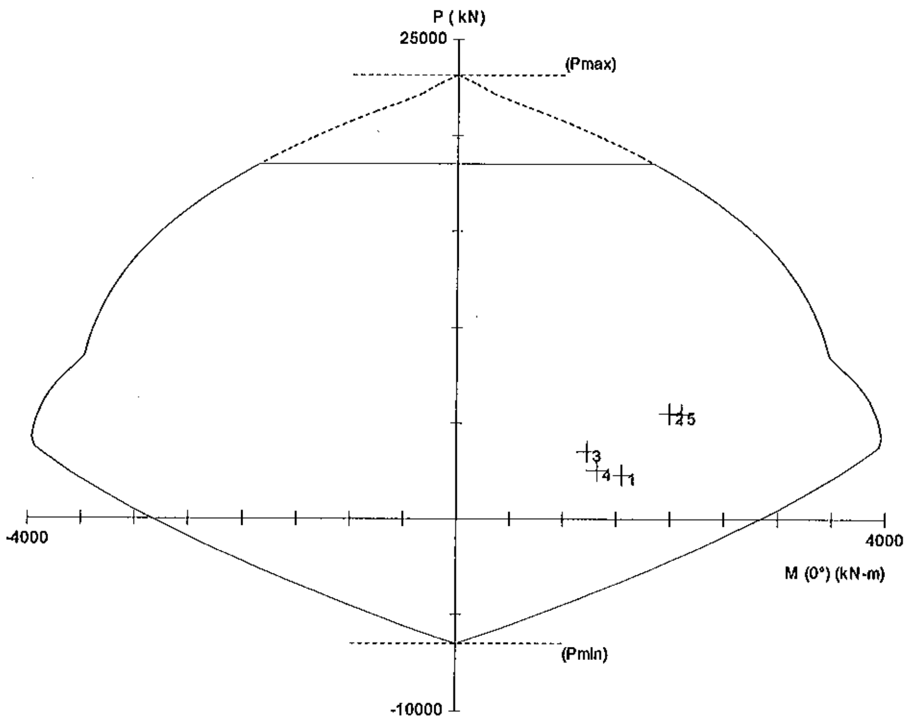
S.5.7.4.2

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

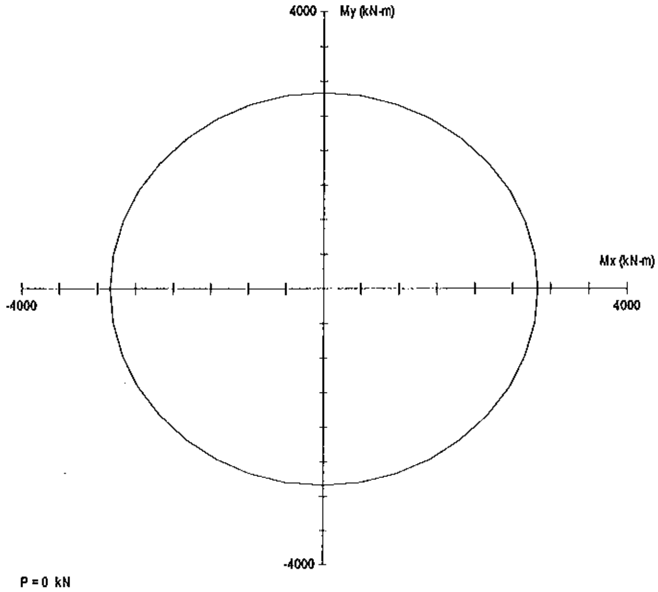
**In Transverse Direction



**In Longitudinal Direction



In Both Direction



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

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

4. Shear Design

Shear resistance factors	ϕv	1.0	
Factored shear force	Vu	925	kN
Required shear capacity Vn = Vu/ ϕv	Vn	925	kN
Determine concrete shear capacity			
Minimum shear reinforcement will provided in cross section			
Therefore	β	2.0	
	θ	45.0	deg
Diameter of bored pile	D	1.20	m
Width of cross section	b	1.20	m
dv = 0.9*de de = D/2 + Dr/pi()			
Diameter of the circle passing through the centers of the long. reinf.	Dr	0.99	m
	de	0.92	m
	dv	0.82	m
Vc = 0.083 . β . sqrt(f'c) . bv. dv	Vc	900	kN
	A_v	1963	mm2
Angle of inclination of shear reinf. to long. axis	α	90	
Vs=Av*fy*dv*(cotgθ+cotgα)*sinα/s	Vs	8635	KN
Vn1 = Vc + Vs	V_{n1}	9535	
Vn2 = 0.25f'cbvdv	V_{n2}	7423	
	V_n	7423	
	Conclude		**OK**

PILE CAPACITY AND SETTLEMENT OF GROUP PILE

1. DATA

- Code of boreholes		CB11_A1	
- Pile diameter :		1000	mm
- Elevation of the underground water	EL_3 =	13.26	m
- Ground elevation after scour	EL_4 =	10.500	m
- Base bottom elevation	EL_1 =	10.500	m
- Expected pipe tip elevation	EL_2 =	-14.500	m
- Pipe length	L =	25.0	m
- Perimeter of cross section	P =	3.14	m
- Area of the pile cross section	A_b =	0.79	m2
- Concrete weight	γ_c =	24.50	kN/m3
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'Nell and Reese (1999)

$$q_s = \beta\sigma'_v \le 0.19MPa; \qquad \text{with } 0.25 < \beta < 1.2$$

+ For sandy soil:

$$\beta = 1.5 - 7.7x10^{-3}(z)^{0.5} \qquad \text{for } N_{60} \ge 15$$

$$\beta = N_{60}/15[1.5 - 7.7x10^{-3}(z)^{0.5}] \qquad \text{for } N_{60} < 15$$

+ For Gravelly sand and gravels and SPT values are more than 50 blows/30cm:

$$\beta = 2\text{-}0.00082\ (z)^{0.5} \qquad \text{with } 0.25 < \beta < 1.8$$

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 conhesive soil, side wall friction is calculated according to the method α

$$q_s = \alpha S_u \qquad (10.8.3.3.1\text{-}1)$$

In which:

+ The cohesive factor a 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:

$$S_u = 0.06{*}N_{60}\,(bar) = 0.006{*}N_{60}\,(MPa) \qquad \text{(Terzaghi \& Peck)}$$

* Calculation method α: (10.8.3.5.1b AASHTO LRFD 2007)

$$\alpha = 0.55 \qquad \text{with } S_u/0.101 \le 1.5$$

$$\alpha = 0.55 - 0.1(S_u/0.101 - 1.5) \qquad \text{with } 1.5 \le S_u/0.101 \le 2.5$$

*** 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 : $\sigma_v = \gamma.z_i$; $u_z = 0$

At the soil layer under the ground water level : $\sigma_v = \gamma_{sat}.z_i$; $u_z = z_i$

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/m^2)
+ σ'_v : effective stress at the depth z (kN/m^2)
+ u : Pore water pressure (kN/m^2)
+ z_i : Depth to the middle point of the i layer counted from the surfac (m)
+ l_i : thickness of the i soil layer (m)
+ γ : Natural density of the soil layer above the ground water level (kN/m^3)
+ γ_{sat}: the saturated desnsity of the soil layer under the ground wate (kN/m^3)

$$\gamma_{sat} = \gamma_d(1 - \gamma_w/\gamma_s) + \gamma_w$$

+ γ_d : Dry density (kN/m^3)
+ γ_s : Unit weight (kN/m^3)

+ γ_w : water density γ_w = 10 (kN/m³)

+ With: Energy efficiency E_h = 50

Name of layer	Layer thickness	Elevation of the layer	depth	Unit weight	Dry density	saturated density	Total stress	Pore water pressure	Effective stress
	l_i		z_i	γ_s	γ_d	γ_{sat}	σ_{vz}	u	σ'_z
	(m)	(m)	(m)	(kN/m³)	(kN/m³)	(kN/m³)	(kN/m²)	(kN/m²)	(kN/m²)
		10.50							
2C	2.61	7.89	1.31	0.00	0.00	0.00	0.00	0.00	0.00
4A	5.60	2.29	5.41	26.68	25.21	25.76	139.38	54.10	85.28
4B	6.00	-3.71	11.21	26.68	25.21	25.76	288.81	112.10	176.71
5A	3.00	-6.71	15.71	26.68	25.21	25.76	404.74	157.10	247.64
5A	3.00	-9.71	18.71	26.68	25.21	25.76	482.03	187.10	294.93
5A	2.00	-11.71	21.21	26.68	25.21	25.76	546.44	212.10	334.34
5A	2.79	-14.50	23.61	26.68	25.21	25.76	608.14	236.05	372.09

Name of layer	Soil type	σ'_z (N/mm²)	SPT N (Blow/30cm)	N_{60} = N*E_h/60	S_u (N/mm²)	α	β	q_s (N/mm²)	Resistance factor φs	value $\varphi_s l_i q_s$ (N/mm)
2C	1	0.000	4	3	-	-			0.55	0.00
4A	2	0.085	11	9	0.054	0.55	-	0.030	0.45	75.08
4B	2	0.177	22	18	0.110	0.55	-	0.061	0.45	163.35
5A	3	0.248	100	83	-	-	1.800	0.190	0.60	342.00
5A	3	0.295	100	83	-	-	1.800	0.190	0.60	342.00
5A	3	0.334	100	83	-	-	1.800	0.190	0.60	228.00
5A	3	0.372	100	83	-	-	1.800	0.190	0.60	318.06
	Total									1468.49

Total resistance of the side wall: $Q_s = \varphi_s \phi_i A_s = \Sigma \varphi_s \phi_i l_i P$ = **4613.4** (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: 3 IGM

2.2.1. Pile tip resistance: Method Reese and O'Neill (1999)

- Nominal Tip resistance follow:

$q_p = 0.057N$ with $N \leq 50$

$q_p = 0.59\sigma'_v((N60(Pa/\sigma'_v))$^0.8 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

σ'_v = 0.37 (Mpa) effective vertical prestress

=> Nominal tip resistance q_p = **3.079** (Mpa)

- Tip resistance of pile:

$Q_b = \phi * q_p * A_b$ = 1330.12 (kN)

With tip resistance factor : ϕ = **0.55** (Table 10.5.5.2.4-1 AASHTO2007)

2.3. Pile capacity:

- Design resistance of the single pile :

$Q_T = (Q_s + Q_P)$ = 5943.5 kN

- Resistance of the resisting pile according to the pile group:

$Q_R = \eta . Q_T$ = **5071.19** kN

In which

+ η = 0.85 Effective factor due to the pile working by group (AASHTO 2007 - 10.8.3.6.3)

+ d = 3.37 (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)	$N_{60} = N*E_h/60$	S_u (N/mm^2)	α	β	q_s (N/mm^2)	Resistance factor φs	value $\varphi_s l_i q_s$ (N/mm)
2C	1	0.00	4	3	0.00	0.00	0.00	0.00	1.00	0.00
4A	2	0.09	11	9	0.05	0.55	0.00	0.03	1.00	-166.83
4B	2	0.18	22	18	0.11	0.55	0.00	0.06	1.00	-363.00
5A	3	0.25	100	83	0.00	0.00	1.80	0.19	1.00	-570.00
5A	3	0.29	100	83	0.00	0.00	1.80	0.19	1.00	-570.00
5A	3	0.33	100	83	0.00	0.00	1.80	0.19	1.00	-380.00
5A	3	0.37	100	83	0.00	0.00	1.80	0.19	1.00	-530.10
	Total									-2579.93

Total uplift resistance of the side wall: $Q_s = \varphi_s \phi_i A_s = \Sigma\varphi_s \phi_i l_i P =$ **-8105.1** (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 :

$Q_U = Q_s =$ -8105.1 kN

- Uplift Resistance of the resisting pile according to the pile group:

$Q_R = \eta . Q_T =$ **-6915.54** kN

In which

+ η = 0.85 Effective factor due to the pile working by group (AASHTO 2007 - 10.8.3.6.3)

+ d = 3.37 (m) The distance from the center to center of pile.

2.4. Conclusion

Internal force of pile		Seftweight	Factored force of pile		Pile capacity		Check	
PMax	PMin	of Pile W	QMax	QMin	Nominal resistance	Uplift resistance		
(KN)	(KN)	(KN)	(KN)	(KN)	Qr (KN)	Qu (KN)	Qmax<Qr	Qmin>Qu
4644.5	781.857	285	4929	1067	5071.19	-6915.54	OK	OK

3. THE SETTLEMENT OF PILE GROUP:

- The Settlement of pile groups in cohesionless soils using SPT value:

$$\rho = (30 q I \sqrt{B})/N\_60 \quad \text{(mm)} \quad (10.7.2.3.3\text{-}1)$$

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)	ρ <25.4
47577.86	2.04E+08	5800	7790	5193.33	0.89	83	0.23	5.68	Ok

PILE CAPACITY AND SETTLEMENT OF GROUP PILE

1. DATA

- Code of boreholes $\boxed{CB11\_A2}$
- Pile diameter : 1000 mm
- Elevation of the underground water EL_3 = 13.26 m
- Ground elevation after scour EL_4 = 10.500 m
- Base bottom elevation EL_1 = 10.500 m
- Expected pipe tip elevation EL_2 = -9.500 m
- Pipe length L = 20.0 m
- Perimeter of cross section P = 3.14 m
- Area of the pile cross section A_b = 0.79 m2
- Concrete weight γ_c = 24.50 kN/m3

Soil layer at Tip of pile: $\boxed{3}$

('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)

$$q_s = \beta\sigma'_v \le 0.19\text{MPa}; \qquad \text{with } 0.25 < \beta < 1.2$$

+ For sandy soil:

$$\beta = 1.5 - 7.7\text{x}10^{-3}(z)^{0.5} \qquad \text{for } N_{60} \ge 15$$

$$\beta = N_{60}/15[1.5 - 7.7\text{x}10^{-3}(z)^{0.5}] \qquad \text{for } N_{60} < 15$$

+ For Gravelly sand and gravels and SPT values are more than 50 blows/30cm:

$$\beta = 2\text{-}0.00082\ (z)^{0.5} \qquad \text{with } 0.25 < \beta < 1.8$$

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 conhesive soil, side wall friction is calculated according to the method α

$$q_s = \alpha S_u \qquad (10.8.3.3.1\text{-}1)$$

In which:

+ The cohesive factor a 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:

$$S_u = 0.06{*}N_{60}\ (\text{bar}) = 0.006{*}N_{60}\ (\text{MPa}) \qquad \text{(Terzaghi \& Peck)}$$

* Calculation method α: (10.8.3.5.1b AASHTO LRFD 2007)

$$\alpha = 0.55 \qquad \text{with } S_u/0.101 \le 1.5$$

$$\alpha = 0.55 - 0.1(S_u/0.101 - 1.5) \qquad \text{with } 1.5 \le S_u/0.101 \le 2.5$$

*** 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 : $\sigma_v = \gamma.z_i$; $u_z = 0$

At the soil layer under the ground water level : $\sigma_v = \gamma_{sat}.z_i$; $u_z = z_i$

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/m^2)
+ σ'_v : effective stress at the depth z (kN/m^2)
+ u : Pore water pressure (kN/m^2)
+ z_i : Depth to the middle point of the i layer counted from the surfac (m)
+ l_i : thickness of the i soil layer (m)
+ γ : Natural density of the soil layer above the ground water level (kN/m^3)
+ γ_{sat}: the saturated desnsity of the soil layer under the ground wate (kN/m^3)

$$\gamma_{sat} = \gamma_d(1 - \gamma_w/\gamma_s) + \gamma_w$$

+ γ_d : Dry density (kN/m^3)
+ γ_s : Unit weight (kN/m^3)
+ γ_w : water density γ_w = 10 (kN/m^3)

+ With: Energy efficiency E_h = 50

Name of layer	Layer thickness	Elevation of the layer	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/m³)	(kN/m³)	(kN/m³)	(kN/m²)	(kN/m²)	(kN/m²)
		10.50							
2C	2.10	8.40	1.05	0.00	0.00	0.00	0.00	0.00	0.00
4A	4.70	3.70	4.45	26.68	25.21	25.76	114.65	44.50	70.15
4B	1.20	2.50	7.40	26.68	25.21	25.76	190.65	74.00	116.65
5A	2.00	0.50	9.00	26.68	25.21	25.76	231.87	90.00	141.87
5A	2.00	-1.50	11.00	26.68	25.21	25.76	283.39	110.00	173.39
5A	3.00	-4.50	13.50	26.68	25.21	25.76	347.80	135.00	212.80
5A	5.00	-9.50	17.50	26.68	25.21	25.76	450.86	175.00	275.86

Name of layer	Soil type	σ'_z (N/mm²)	SPT N (Blow/30cm)	N_{60} = N*E_h/60	S_u (N/mm²)	α	β	q_s (N/mm²)	Resistance factor φs	value $\varphi_s l_i q_s$ (N/mm)
2C	1	0.000	4	3	-	-			0.55	0.00
4A	2	0.070	11	9	0.054	0.55	-	0.030	0.45	63.01
4B	2	0.117	27	23	0.135	0.55	-	0.074	0.45	40.10
5A	3	0.142	100	83	-	-	1.800	0.190	0.60	228.00
5A	3	0.173	110	92	-	-	1.800	0.190	0.60	228.00
5A	3	0.213	120	100	-	-	1.800	0.190	0.60	342.00
5A	3	0.276	130	108	-	-	1.800	0.190	0.60	570.00
	Total									1471.10

Total resistance of the side wall: $Q_s = \varphi_s \phi_i A_s = \Sigma \varphi_s \phi_i l_i P$ = **4621.6** (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: 3 IGM

2.2.1. Pile tip resistance: Method Reese and O'Neill (1999)

- Nominal Tip resistance follow:

$q_p = 0.057N$ with $N \leq 50$

$q_p = 0.59\sigma'_v((N60(Pa/\sigma'_v))^{0.8}$ with $N > 50$

In which

N = 130 (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

σ'_v = 0.28 (Mpa) effective vertical prestress

=> Nominal tip resistance q_p = **3.578** (Mpa)

- Tip resistance of pile:

$Q_b = \phi * q_p * A_b$ = 1545.44 (kN)

With tip resistance factor : ϕ = **0.55** (Table 10.5.5.2.4-1 AASHTO2007)

2.3. Pile capacity:

- Design resistance of the single pile :

$Q_T = (Q_s + Q_P)$ = 6167.1 kN

- Resistance of the resisting pile according to the pile group:

$Q_R = \eta . Q_T$ = **5261.93** kN

In which

+ η = 0.85 Effective factor due to the pile working by group (AASHTO 2007 - 10.8.3.6.3)

+ d = 3.37 (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)	$N_{60} = N*E_h/60$	S_u (N/mm^2)	α	β	q_s (N/mm^2)	Resistance factor φs	value $\varphi_s l_i q_s$ (N/mm)
2C	1	0.00	4	3	0.00	0.00	0.00	0.00	1.00	0.00
4A	2	0.07	11	9	0.05	0.55	0.00	0.03	1.00	-140.02
4B	2	0.12	27	23	0.14	0.55	0.00	0.07	1.00	-89.10
5A	3	0.14	100	83	0.00	0.00	1.80	0.19	1.00	-380.00
5A	3	0.17	110	92	0.00	0.00	1.80	0.19	1.00	-380.00
5A	3	0.21	120	100	0.00	0.00	1.80	0.19	1.00	-570.00
5A	3	0.28	130	108	0.00	0.00	1.80	0.19	1.00	-950.00
	Total									-2509.12

Total uplift resistance of the side wall: $Q_s = \varphi_s \phi_i A_s = \Sigma \varphi_s \phi_i l_i P =$ **-7882.6** (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 :

$Q_U = Q_s =$ -7882.6 kN

- Uplift Resistance of the resisting pile according to the pile group:

$Q_R = \eta . Q_T =$ **-6725.73** kN

In which

+ η = 0.85 Effective factor due to the pile working by group (AASHTO 2007 - 10.8.3.6.3)

+ d = 3.37 (m) The distance from the center to center of pile.

2.4. Conclusion

Internal force of pile		Seftweight	Factored force of pile		Pile capacity		Check	
PMax	PMin	of Pile W	QMax	QMin	Nominal resistance	Uplift resistance		
(KN)	(KN)	(KN)	(KN)	(KN)	Qr (KN)	Qu (KN)	Qmax<Qr	Qmin>Qu
4644.5	781.857	228	4872	1010	5261.93	-6725.73	OK	OK

3. THE SETTLEMENT OF PILE GROUP:

- The Settlement of pile groups in cohesionless soils using SPT value:

$$\rho = (30 q I \sqrt{B}) / N\_60 \quad \text{(mm)} \quad (10.7.2.3.3\text{-}1)$$

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)	ρ <25.4
47577.86	2.32E+08	5800	10000	6666.67	0.86	108	0.21	3.70	Ok

MINISTRY OF TRANSPORT

VIETNAM EXPRESSWAY CORPORATION - PROJECT MANAGEMENT UNIT NO.85

DA NANG - QUANG NGAI EXPRESSWAY DEVELOPMENT PROJECT

PACKAGE: 6

BRIDGE

ORB11

CALCULATION SHEETS

HA NOI, 2012

Table of content - ORB11 Bridge

A. Substructure design

B. Miscellaneous

Da Nang Quang Ngai Expressway project

BRIDGE

ORB11

CALCULATION SHEETS

ABUTMENT A1 (RIGHT)

DA NANG, 2013

Table of content

Assumptions :

1. Bridge is considered to be in seismic with acceleration coeff. A = 0.0912 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

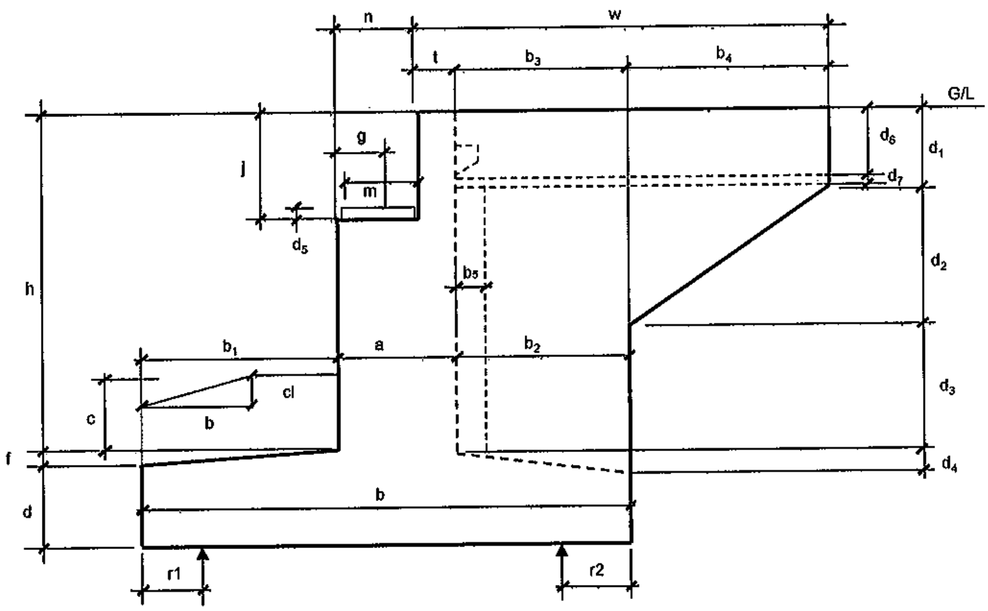
Input :

Level Table(at center of abutment)

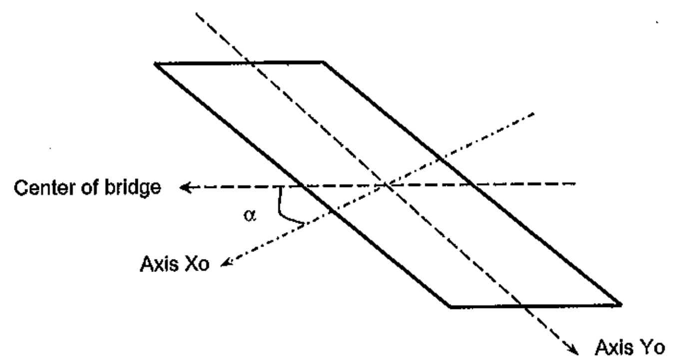
Level of top of headwall	HTwL	17.196	m
Level of top of bearing	BTL	15.297	m
Level of top of stem abutment	HTL	15.147	m
Level of top of footing	FTL	12.000	m
Level of bottom of footing	FBL	10.000	m
Ground level	GL	12.900	m
Lowest water level	HWL	14.220	m
Skew angle	α	0.00	deg

I.Loads from substructure

Abutment dimensions VERTICAL VIEW



PLAN VIEW

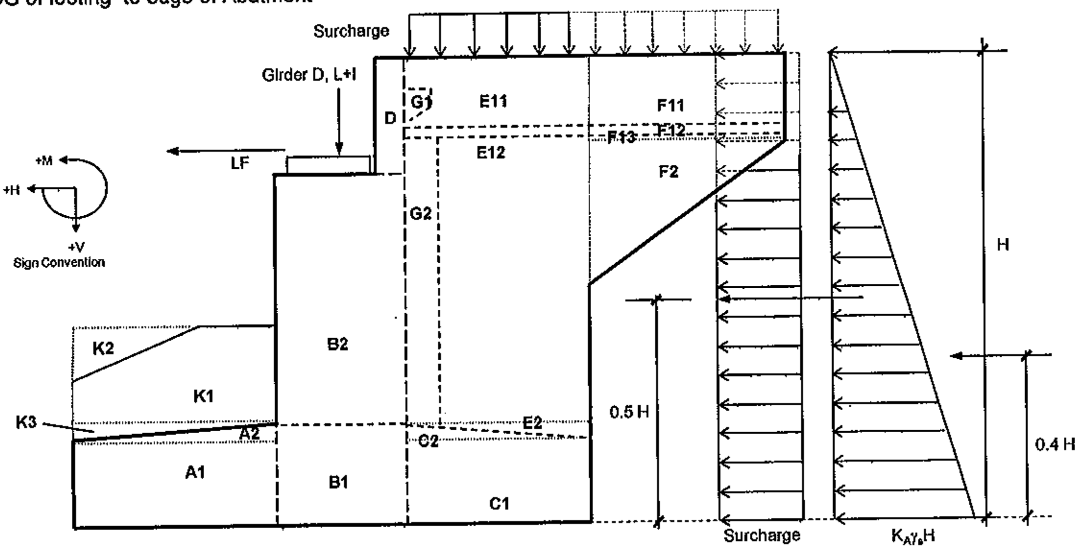


Material Unit Weights

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

ABUTMENT DIMENSIONS (IN METRES)					
Item	**Symbol**	**Value**	**Item**	**Symbol**	**Value**
Height of stem	h	5.196	Horizontal Dimension	b_3	2.000
Footing Width	b	5.500	Horizontal Dimension	b_4	1.500
Stem Width	a	1.500	Horizontal Dimension	b_5	0.300
Footing Depth	d	2.000	Vertical Dimension	d_1	0.930
Footing Slope	f	0.000	Vertical Dimension	d_2	1.500
Width of stem at bearing	n	1.000	Vertical Dimension	d_3	2.766
Ballast Wall Height	j	2.049	Vertical Dimension	d_4	0.000
Ballast Wall Thickness	t	0.500	Vertical Dimension	d_5	0.150
Wingwall Length	w	4.000	Vertical Dimension	d_6	1.070
Soil Cover at Toe	c	0.900	Vertical Dimension	d_7	0.300
Girder Reaction	g	0.500	With of bearing pad	m	0.650
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

	cos (α)	=	1.00
Slope front of abutment	cl	=	0.00 m
	bl	=	0.00 m
Width of Abutment	L		12.600 m
Width of abutment (inclined direction)	Ltr		12.600 m
Height of Abutment	Ht	=	7.20 m
Distance from CG of footing to edge of Abutment	b/2	=	2.75 m



1. Self weight of Abutment (DC)

Description	Area (m^2)	Length (m)	Force (kN)	X[1] (m)	Arm[2] (m)	Moment (kN•m)
SW of Abutment (DC)						
Section A1	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	4.721	12.600	1457	2.750	-	-
Section C1	4.000	12.600	1235	4.500	-1.750	-2161
Section C2	-	12.600	-	4.167	-1.417	-
Section D	1.025	12.600	316	3.250	-0.500	-158
Section E11	1.260	0.500	15	4.500	-1.750	-27
Section E12	8.532	0.500	105	4.500	-1.750	-183
Part extra stem	3.598	0.740	65	5.083	-2.333	-152
Section F11	1.605	0.500	20	6.250	-3.500	-69
Section F12	0.525	0.500	6	5.250	-2.500	-16
Section F13	-0.660	0.500	-8	6.250	-3.500	28
Section F2	1.125	0.500	14	6.000	-3.250	-45
Section G1	0.135	12.100	253	3.650	-0.900	-228
Section G2	0.045	7.652	8	3.650	-0.900	-8
Bearing seats (w1seat= 0.70m)	0.098	3.500	11	2.500	0.250	3
Curbs +Handrail on Abutment	0.50	4.000	53	5.000	-2.250	-119
Total SW of Abutment (DC)			*5712*			*-973*
Tranverser moment			*211*		*6.175*	*1300*

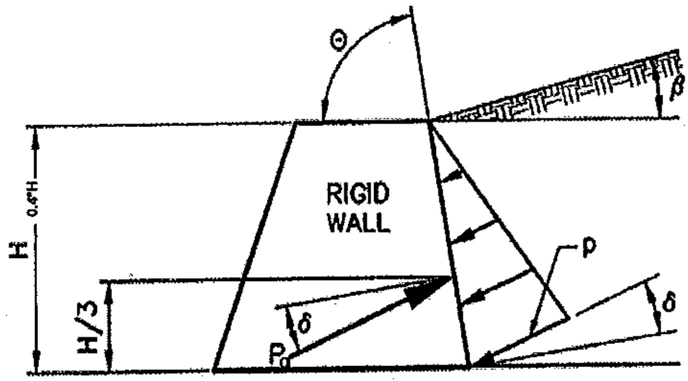
Notes: 1. Distance 'X' is measured horizontally from Toe of Retaining to CG of Section
2. Moment 'Arm' is measured from CG horizontally and from Underside of Footing Vertically.

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	10.39	12.100	2263	4.500	-1.750	-3961
Section E2	-	12.100	-	4.833	-2.083	-
Section E3	-	0.500	-	5.500	-2.750	-
Section K1	1.800	12.600	408	1.000	1.750	-
Section K2	-	12.600	-	-	2.750	-
Section K3	-	12.600	-	0.667	2.083	-
Total Earth on Footing			***2672***			***-3961***

3. Horizontal Earth Pressure on Abutment (EH)

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



- Height for horizontal ear
- Width for horizontal eart
- Density of Soil
- Internal Friction Angle of
- Incline angle of back face wall — θ = 90.0 deg
- Friction angle between fill and wall — δ = 20.0 deg
- Incline angle of fill soil — β = 0.0 deg
- Gravitational acceleration — g = 9.81 m/s^2
- Basic earth pressure

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

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

Γ	=	2.684
Ka	=	0.297
p	=	0.039 Mpa

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

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

Ea	=	1746 kN
M	=	5025 kNm

<S 3.11.5.1>

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

Equivalent height of soil for highway loading taken from Table 3.11.6.2.1

H=	1.50m heq=	1.7 m	
H=	3.00m heq=	1.2 m	
H=	6.00m heq=	0.76 m	
H=	9.00m heq=	0.61 m	
H=	7.20m heq=	0.70 m	(Linear Interpolation)

- **Vertical force**

ESv	=	318 kN
ev	=	-1.75 m
M	=	-556 kNm

- **Horizontal force**

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

ESh	=	340 kN
eh	=	3.60 m
M	=	1222 kNm

5. Earthquake effects

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

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

- Peak ground acceleration coefficient A = 0.0912 g

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

- Backfill slop angle i = 0.0 deg
- Slope of wall to vertical β' = 0.0 deg
- Angle of friction of soil ϕ = 30.0 deg
- Angle of friction between soil and abutment δ = 20.0 deg
- Horizontal acceleration coefficient kh = 0.137
- Vertical acceleration coefficient kv = 0.055
- Angle $\theta = \arctan(k_h/(1-k_v))$ θ = 8.2 deg

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

- Seismic active lateral Earth pressure coefficient K_{AE} = 0.40

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

- Seismic active lateral Earth pressure coefficient E_{AE} = 2234 kN

$M_{AE} = E_{AS}*0.3H+(E_{AE}-E_{AS})*0.6H$ M_{AE} = 5876 KNm <A.11.1.1.1>

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

5.2. Earthquake effects to abutment (EQ)

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

- Soil profile type Soil type II
- Site Coefficients. S = 1.2
- Elastic Seismic Response Coefficient 2.5A = 0.228

Csm = 1.2*A*S/ Tm^2/3 ≤ 2.5*A Csm = 0.199

- Period of vibration of the fundamental mode

Tm = 2*pi()*sqrt(m/k) Tm = 0.534 s

Decription	Area (m^2)	Length (m)	Force (kN)	X[(1)] (m)	Arm[(2)] (m)	Moment (kN•m)
Section A1	4.000	12.600	135	-	1.000	135
Section A2	-	12.600	-	-	2.000	-
Section B1	3.000	12.600	101	-	1.000	101
Section B2	4.721	12.600	291	-	3.574	1039
Section C1	4.000	12.600	135	-	1.000	135
Section C2	-	12.600	-	-	2.000	-
Section D	1.025	12.600	63	-	6.172	389
Section E11	1.260	0.500	2	-	4.661	8
Section E12	8.532	0.500	11	-	1.913	
Section E2	3.598	0.740	7	-	2.000	14
Section F11	1.605	0.500	2	-	4.661	10
Section F12	0.525	0.500	1	-	3.976	
Section F13	-0.660	0.500	-1	-	5.416	
Section F2	1.125	0.500	2	-	5.766	9
Section G1	0.135	12.100	4	-	4.483	20
Section G2	0.045	7.652	1	-	1.913	2
Total EQ of Abutment Selfweight			***754***			***1862***

6. Braking Force(BR)

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

• Number of lanes	n	=	3 lanes	
• Multiple presence factor	m	=	0.85	
• Take 25 % of Truck load				
BR = 25% * n * m * (2*145+35)	BR	=	104 kN	Long. Axis
• Acting at 1.8m higher of road face	e	=	9.1 m	
	Mlong	=	939 KNm	Long. Axis

7. Centrifucal Force , CE (3.6.3)

• Plan of bridge (1:"straight",2: "Curve")			1	
• Design Speed	V	=	120 km/h	
	V	=	33.3 m/s	
	R		- m	
$C = 4/3*(V^2/gR)$	C		-	
Acting at 1.8m higher of road face				
CE = n * m * (2*145+35) * C	CE	=	0.00 KN	
	e	=	9.08 m	
	Mtrans	=	0.00 KNm	Trans. Axis

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

II.Loads from superstructure

Item	Sign	Value	Unit
Span length	Lsp	24.00	m
Span between bearings	Lb	23.20	m
Skew angle	α	0.00	deg
Deck slab length	Ldeck	24.00	m
Bridge Width	Bc	12.48	m
Girder height	hgi	1.45	m
Deck slab depth	hdkslab	0.22	m
Asphalt depth	has	0.084	m
Unit weight of concrete	γc	24.50	kN/m3
Unit weight of asphalt concrete	γa	22.10	kN/m3

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

Item	Sign	Value	Unit
1.1.Girders			
Weight of 1 girder	DC	384.41	kN
Number of girders	n	5	Girders
Sum of girders weight	DC	1922.03	kN
Precast Planks	DC	337.43	kN
Diaphragm	DC	275.01	kN
Total	DC	2534.46	kN
1.2.Deck slab			
Deck slab	DC	1575.30	kN
1.3.Pavement			
Asphalt concrete	DW	511.56	kN
1.4.Handrail			
Handrail + median	DC	568.80	kN

2.Live load (LL):

Truck	145 145 35 kN; 4.3 4.3 m
Tandem	110 110 kN; 1.2 m
Lane load	w_l 9.3 kN/m
Pedestrial	Wpd= 0.0 kN/m2
Considerate structure as a simple span	
Reaction Influence	1; 23.2 m

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

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

Truck	P1(kN)	P2(kN)	P3(kN)	Sum(kN)	Total(kN)
Axle load	145	145	35		
Influence value	1.000	0.815	0.629		
Reaction	145.0	118.1	22.0	285.2	908.9

Tandem	P1(kN)	P2(kN)	Sum(kN)	Total(kN)
Axle load	110	110		
Influence value	1.000	0.948		
Reaction	110	104.3	214.3	683.1

Lane load	Wl(kN/m)	Total(kN)
Value	9.3	
Influence value	11.6	
Reaction	107.9	275.1

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

3.Earthquake effects on superstructrue (EQ)

Longitudinal moveable bearings at Abutment

Horizontal force from superstructure due to EQ - transverse direction

At bearing — Heq = 518 kN

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

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

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

Shear modulus G — G = 1 MPa

Bearing area — A = 0.158 m2

Height of elastomeric layers — hrt 0.064 m

Number of bearing — nb = 5 bears

Horizontal force due to TU+SH&CR

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

5.Wind loads (Ws)

5.1.Tranverse wind on superstructure (WS)

Wind zone	Zone		III
Basic 3 second gust wind	Vb	=	53.00 m/s
Correction factor	S	=	1.09
Design wind velocity	V	=	57.77 m/s
Drag coefficient	Cd	=	1.39
Overall width of bridge	b	=	12.48 m
Depth of superstructure(including solid parapet)	d	=	2.73 m
	b/d	=	4.57
Windy obstructed area of superstructure	At	=	65.54 m2

Force due to tranverse wind

Fhy=max(0.0006*V2*At*Cd,1.8*At) (kN) — Fhy = 182.8 kN <3.8.1>

5.2.Wind load on vehicles (WL)

Transverse wind on vehicles — Wltran = 1.50 kN/m

Transverse horizontal force due to wind on liveload

At 1.8m from surface — Fhy = 36.00 kN

6.Combinations

Loads from superstructure to Abutment

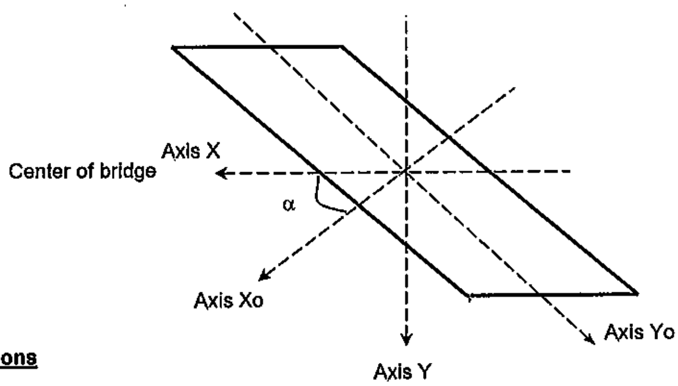
Loads at bottom of stem		Vertical		Longitudinal			Tranversal		
Loads	Sign	N (kN)	x (m)	Hx (kN)	z_1 (m)	My (kN•m)	Hy (kN)	y (m)	Mx (kN•m)
Girder + Deckslab	DC	2055	0.25			514			
Handrail	DC	284	0.25			71			
Pavement	DW	256	0.25			64			
LiveLoad	LL	1184	0.25			296		1.38	1628
Pedestrial	PL	0	0.25			0		-	-
Trans. wind on Struc.	WS						91	3.15	288
Trans. wind on vehi.	WL						18	4.95	89
Eearth quake	EQ						518	3.15	1629
TU+SH&CR	TU+SH&CR			320	3.15	1007			

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	2055	0.25			514			
Handrail	DC	284	0.25			71			
Pavement	DW	256	0.25			64			
LiveLoad	LL	1184	0.25			296		1.38	1628
Pedestrial	PL	0	0.25			0		-	-
Trans. wind on Struc.	WS						91	5.15	470
Trans. wind on vehi.	WL						18	6.95	125
Eearth quake	EQ						518	5.15	2664
TU+SH&CR	TU+SH&CR			320	5.15	1647			

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

Load combinations at bottom of stem		Longitudinal		Tranversal	
Load combinations	N (kN)	Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	5380	160	1848	0	2849
Strength Str-IB	4344	160	1589	0	2849
Strength Str-IIIA	4906	160	1730	55	2402
Strength Str-IIIB	3870	160	1471	55	2402
Service Ser-I	3779	320	1952	45	1803
Extreme Ext-IA	3900	0	975	518	2443
Extreme Ext-IB	2864	0	716	518	2443

Load combinations at bottom of pilecap		Longitudinal		Tranversal	
Load combinations	N (kN)	Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	5380	160	2168	0	2849
Strength Str-IB	4344	160	1909	0	2849
Strength Str-IIIA	4906	160	2050	55	2511
Strength Str-IIIB	3870	160	1791	55	2511
Service Ser-I	3779	320	2591	45	1894
Extreme Ext-IA	3900	0	975	518	3478
Extreme Ext-IB	2864	0	716	518	3478



III.Load Combinations

1.Loads from substructure

Loads at bottom of pilecap	Sign	Vertical N (kN)	Vertical x (m)	Longitudinal H_x (kN)	Longitudinal z_1 (m)	Longitudinal M_y (kN•m)	Tranversal H_y (kN)	Tranversal y (m)	Tranversal M_x (kN•m)
Self weight of Abutment	DC	5712				-973			435.096
Soils on pilecap	EV	2672				-3961			
Horizontal Earth Pressure	EH			1746		5025			
Vertical Surcharge	L_{Sv}	318				-556			
Horizontal Surcharge	L_{Sh}			340		1222			
Braking Force	BR			104		939			
Centrifucal Force	CE			-		-	-		-
Buoyancy of Abutment	WA	-1815				76			
Buoyancy of Earth on Abutment	WA	-728				495			
Earthquake effects to Abutment	EQ			754		1862	226		559
Earthquake effects to soil	E_{AE}			2234		5876			

Table of load factors

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

Load Combination at bottom of pilecap					
Load combinations	N (kN)	Longitudinal Hx (kN)	Longitudinal My (kN.m)	Tranversal Hy (kN)	Tranversal Mx (kN.m)
Strength Str-IA	8759	3395	4355	0	544
Strength Str-IB	5558	2347	3463	0	392
Strength Str-IIIA	8632	3217	3713	0	544
Strength Str-IIIB	5431	2170	2821	0	392
Service Ser-I	6158	2189	2268	0	435
Extreme Ext-IA	8362	3210	2548	226	1102
Extreme Ext-IB	5161	3210	4671	226	950

2. Loads from superstructure

Load Combination at bottom of pilecap					
		Longitudinal		Tranversal	
Load combinations	N (kN)	Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	5380	160	2168	0	2849
Strength Str-IB	4344	160	1909	0	2849
Strength Str-IIIA	4906	160	2050	55	2511
Strength Str-IIIB	3870	160	1791	55	2511
Service Ser-I	3779	320	2591	45	1894
Extreme Ext-IA	3900	0	975	518	3478
Extreme Ext-IB	2864	0	716	518	3478

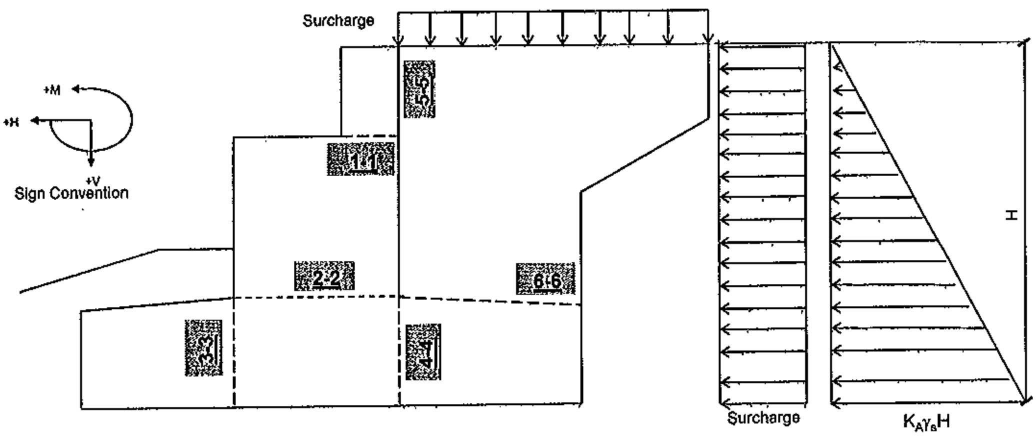
3. Total loads at bottom of pilecap

Load Combination at bottom of pilecap					
		Longitudinal		Tranversal	
Load combinations	N (kN)	Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Strength Str-IA	14139	3555	6524	0	3393
Strength Str-IB	9902	2507	5373	0	3241
Strength Str-IIIA	13539	3377	5763	55	3055
Strength Str-IIIB	9301	2330	4612	55	2903
Service Ser-I	9937	2509	4859	45	2329
Extreme Ext-IA	12262	3210	3523	744	4581
Extreme Ext-IB	8025	3210	5387	744	4428

IV.Elements checking

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

1. Calculate internal force of sections



1.1 Section 1-1
Table of Load Factors

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

Unfactored Loads	N (kN)	Longitudinal Hx (kN)	Longitudinal My (kN.m)	Tranversal Hy (kN)	Tranversal Mx (kN.m)
Selfweight	569		-101		
Horizontal Earth Pressure		142	116		
Surcharge (horizontal)		210	215		
Horizontal Seismic Earth Pressure		181	136		
Abutment earthquake force		67	69	20	21

Load Combination at bottom of headwall

Load combinations	N (kN)	Longitudinal Hx (kN)	Longitudinal My (kN.m)	Tranversal Hy (kN)	Tranversal Mx (kN.m)
Service Ser-I	569	351	230	0	0
Strength Str-IA	711	579	423	0	0
Strength Str-IB	512	494	389	0	0
Extreme Ext-I	711	444	254	20	21

1.2 Section 2-2
Table of Load Factors

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

Unfactored Loads	N (kN)	Longitudinal Hx (kN)	Longitudinal My (kN.m)	Tranversal Hy (kN)	Tranversal Mx (kN.m)
Selfweight	2026		-183		
Superstructure Dead Load	2055		514		
Pavement	256		64		
Handrail+curb	284		71		
Live Load	1184		296		1628
Horizontal Earth Pressure		910	1892		
Surchage (Horizontal)		280	728		
TU+SH&CR		320	1007		
Horizontal Seismic Earth Pressure		1165	2212		
Abutment earthquake force		358	740	263	734

Load Combination at bottom of stem wall

Load combinations	N (kN)	Longitudinal Hx (kN)	Longitudinal My (kN.m)	Tranversal Hy (kN)	Tranversal Mx (kN.m)
Service Ser-I	5805	1511	4388	0	1628
Strength Str-IA	7913	2016	5732	0	2849
Strength Str-IB	6167	1470	4402	0	2849
Extreme Ext-I	6433	2245	5168	263	1548

1.3 Section 3-3

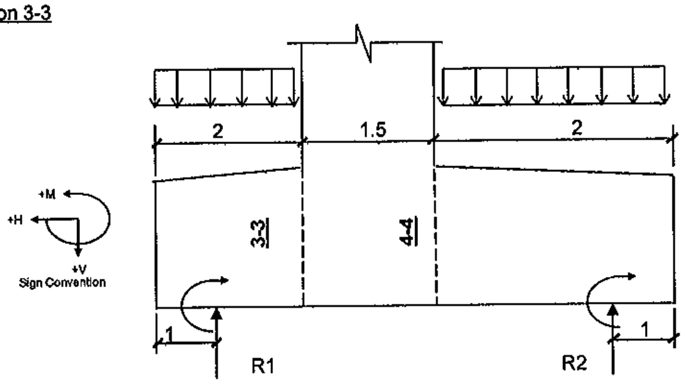


Table of Load Factors

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

Unfactored Loads		N (kN)	Longitudinal Hx (kN)	Longitudinal My (kN.m)	Tranversal Hy (kN)	Tranversal Mx (kN.m)
Selfweight at front side		1235		1235		
Vertical soil on foot at front side		408		408		
Reaction of piles	Ser-I	-13477	-2487	-7914	31	110
	Str-IA	-18925	-3655	-10761	75	230
	Str-IB	-13103	-2418	-7736	49	144
	Ext-I	-14223	-2578	-8025	-44	-67

Load Combination at section 3-3

Load combinations	N (kN)	Longitudinal Hx (kN)	Longitudinal My (kN.m)	Tranversal Hy (kN)	Tranversal Mx (kN.m)
Service Ser-I	-11834	-2487	-6270	31	110
Strength Str-IA	-16830	-3655	-8666	75	230
Strength Str-IB	-11624	-2418	-6257	49	144
Extreme Ext-I	-12128	-2578	-5930	-44	-67

1.4 Section 4-4

Table of Load Factors

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

Unfactored Loads		N	Longitudinal		Tranversal	
		(kN)	Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Selfweight of behind side		1452		-1526		
Vertical soil on foot at behind side		2263		-2263		
Surcharge(Vertical)		318		-318		
Reaction of piles	Ser-I	-4284	-1990	8735	-62	-135
	Str-IA	-5490	-2924	12021	-68	-141
	Str-IB	-3819	-1934	8113	-45	-100
	Ext-I	-8231	-2063	13190	-128	-290

Load Combination at section 4-4

Load combinations	N	Longitudinal		Tranversal	
	(kN)	Hx (kN)	My (kN.m)	Hy (kN)	Mx (kN.m)
Service Ser-I	-251	-1990	4628	-62	-135
Strength Str-IA	-64	-2924	6502	-68	-141
Strength Str-IB	80	-1934	4147	-45	-100
Extreme Ext-I	-3202	-2063	8068	-128	-290

1.4 Section 5-5 & 6-6

Slope of triang pressure   tgβ = 5.35

Uniform horizontal pressure   U.p = 3.75 kN/m2

Load Combination at section 5-5

Load combinations	N	Horizontal		Vertical	
	(kN)	Hx (kN)	Mx (kN.m)	Hy (kN)	My (kN.m)
Service Ser-I		78	181		
Strength Str-IA		122	287		

Load Combination at section 6-6

Load combinations	N	Horizontal		Vertical	
	(kN)	Hx (kN)	Mx (kN.m)	Hy (kN)	My (kN.m)
Service Ser-I				213	188
Strength Str-IA				331	292

2. Elements Checking

2.1. Bearing Resistance

<S.5.7.5>

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

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

Dimension of bearing plate	w0	=	0.650	m	
	b0	=	0.700	m	
Area under bearing device	A1	=	0.455	m2	
Distributed width and length	w	=	1.000	m	
	b	=	1.050	m	
Notational area	A2	=	1.050	m2	

Where supporting surface is wider on all sides than loades area

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

where loaded area is subjected to nouniformly distributed bearing

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

Modification factor case 1	m	=	1.519		
Resistance factor	ϕ	=	0.700		<S.5.5.4.2>
Factored bearing resistance	Pr	=	12338	kN	> Pu
Bearing reaction of approach bridge	Pu	=	4046	kN	Ok

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

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

<S.5.10.9.7.2>

Factored bearing resistance shall be taken

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

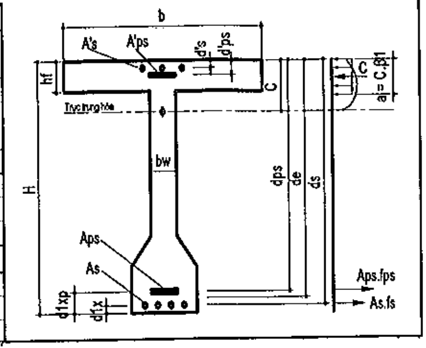
fn take the lesser of

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

$fn = 2.25.\ f'ci$

	fn	=	31.90	MPa	
Maximum area of the portion of supporting surface	A	=	1.050	m2	
Gross area of bearing plate	Ag	=	0.455	m2	
Effective net area of bearing plate, Ag minus stud of bearing	Ab	=	0.455	m2	
Nominal concrete strength at time of application	f'ci	=	30	MPa	
Factored bearing resistance	Pr	=	10161	kN	Ok

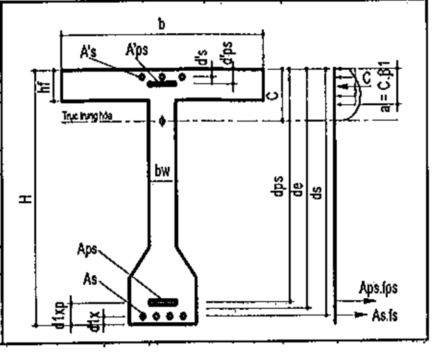
MATERIALS			
NORMAL CONCRETE			
f'c	Compressive Strength of concrete at 28 days	Mpa	30
Ec	Modulus of Elasticity	Mpa	27691
fr	Modulus of Rupture	Mpa	3.5
gc	Unit weight of concrete	kN/m3	24.5
PRESTRESSING STEEL			
fpu	Tensile strength of prestressing steel	Mpa	1860
fpy	Yield strength of prestressing steel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



Sign	Parameters		Unit	Sections				
				1-1	1-1	2-2	2-2	2-2
INTERNAL FORCES AT SECTIOIN								
	Combination			Strength	Service	Service	Strength	Extreme
Qu	Shear		kN	579	351	1511	2016	2245
Mu	Flexural Moment		kNm	423	230	4388	5732	5168
Nu	Axial load		kN	711	569	5805	7913	6433
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	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
A'ps	Compression prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	0 T00.0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
As	Tension Reinforcement	Number	bars	83	83	77	77	77
		Diameter	mm	16	16	22	22	22
		Area	m2	0.01677	0.01677	0.02926	0.02926	0.02926
A's	Compression Reinforcement	Number	bars	83	83	77	77	77
		Diameter	mm	16	16	16	16	16
		Area	m2	0.01677	0.01677	0.01555	0.01555	0.01555
A'c	Shear reinforcement	Number	bars	20	20	19	19	19
		Diameter	mm	14	14	14	14	14
		Area	m2	0.00302	0.00302	0.00287	0.00287	0.00287
ϕ	Resistance factors for flexure		5.5.4.2	0.90	1.00	1.00	0.90	1.00
ϕv	Resistance factors for shear			0.90	1.00	1.00	0.90	1.00
ϕn	Resistance factors for axial force			1.00	1.00	1.00	1.00	1.00
β1	Stress block factor			0.836	0.836	0.836	0.836	0.836
c	Dis. Between centroid and top fiber		m	0.000	0.000	0.020	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 reistance		kNm	2318	2575	16458	14812	16458
Mu	Flexual moment		kNm	423	230	4388	5732	5168

(5.7.3.2)	**Flexural moment Checking**		**OK**	**OK**	**OK**	**OK**	**OK**
	Limits for reinforcement						
c/de	Maximum reinforcement		0.00	0.00	0.01	0.01	0.01
	Maximum reinforcement Checking	<= 0.42	**OK**	**OK**	**OK**	**OK**	**OK**
1.2*Mcr	Craking moment	kNm	1087	1087	9918	9918	9918
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**		**OK**	**OK**	**OK**	**OK**	**OK**
(5.7.3.4)	**Conctrol of craking by distr. of reinf for RC member- Check?**		No	Yes	Yes	No	No
	Existing condition for structrure	1,2 or 3	1	1	1	1	1
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.058	0.058	0.059	0.059	0.059
Z	Crack width parameter	N/mm	30000	30000	30000	30000	30000
A	Area of concr. with same centroid as tens. Reinf	m2	0.018	0.018	0.019	0.019	0.019
fsa	Value	Mpa	298	298	287	287	287
0.6*fy		Mpa	240	240	240	240	240
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	240	240	240	240	240
x	Dist. From compression fiber to centroid	m	-	0.082	0.201	-	-
J.d	Arm	m	-	0.415	1.374	-	-
Icr	Moment of inertia of the cracked section	m4	-	0.018	0.351	-	-
fs	Tensile stress in reinforcement fs = Msls / (As*J.d)	Mpa	-	33	109	-	-
	Checking for control cracking fs<fsa		**N.a**	**OK**	**OK**	**N.a**	**N.a**
(5.10.8.2)	**Shrinkage and temperature Reinfocement (side distribution)**						
Areq	Area of required reinf	m2	0.00045	0.00045	0.00126	0.00126	0.00126
	Distribution on sides 7 D16	m2	0.00141	0.00141	0.00141	0.00141	
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45	0.45
	Checking		**OK**	**OK**	**OK**	**OK**	**OK**
	SHEAR AND TORSION CHECKING						
β	Factor indicating diag. cracked concr. to tension		3.2	3.8	3.4	3.3	2.9
θ	Angle of inclination of diagonal compressive	degree	28.67	27.51	28.52	28.61	28.79
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	90
bv	Effective web width as minimum web width - in dv	m	12.600	12.600	12.600	12.600	12.600
dv	Effective shear depth	m	0.442	0.442	1.432	1.432	1.432
	(de - 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
ncat	Amount of bars in spacing S	bars	20	20	19	19	19
Av	Shear reinf area in spacing S	m2	0.0030	0.0030	0.0029	0.0029	0.0029
0	Assume	degree	28.87	28.19	28.88	30.16	30.27
v	Shear stress in concrete	kN/m2	116	63	84	124	124
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e_x	Strain in tensile reinforcement		3.36E-04	1.68E-04	2.61E-04	3.04E-04	3.96E-04
	if ex<0, multiple with reduce factor		-	-	-	-	-
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok	Ok
v/f'c	Ratio of shear stress and f'c		0.004	0.002	0.003	0.004	0.004
β	Final value		3.2	3.8	3.4	3.3	2.9
θ	Final value	degree	28.67	27.51	28.52	28.61	28.79
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	7980	9669	28267	26896	23954
Vs	Shear resistance provided by shear reinforcement	kN	1627	1709	5041	5024	4986
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	9607	11377	33309	31920	28940
Vn2	Vn2	kN	41769	41769	135368	135368	135368
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	9607	11377	33309	31920	28940
Vr	Factored shear resistance	kN	**8647**	**11377**	**33309**	**28728**	**28940**
Vu	Shear	kN	579	351	1511	2016	2245
(5.8.2.7)	**Shear checking**		**OK**	**OK**	**OK**	**OK**	**OK**
	Region requiring transverse reinf Checking		No need	No need	No need	No need	No need
	Minimum shear reinf area	m2	0.0086	0.0086	0.0086	0.0086	0.0086
	Mimimum shear reinforcement Checking		-	-	-	-	-
	0.1*f'c*bv*dv	kN	16708	16708	54147	54147	54147
	Smax	m	0.35	0.35	0.60	0.60	0.60
	Maximum spacing Smax		-	-	-	-	-

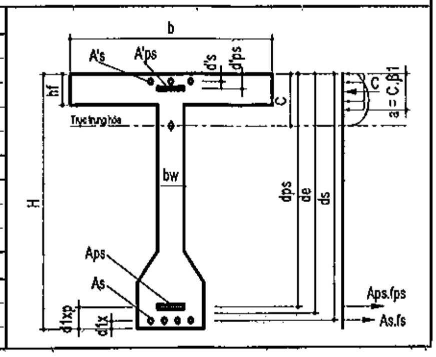
MATERIALS			
NORMAL CONCRETE			
f'c	Compressive Strength of concrete at 28 days	Mpa	30
Ec	Modulus of Elasticity	Mpa	27691
fr	Modulus of Rupture	Mpa	3.5
gc	Unit weight of concrete	kN/m3	24.5
PRESTRESSING STEEL			
fpu	Tensile strength of prestressing steel	Mpa	1860
fpy	Yield strength of prestressing steel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



Sign	Parameters		Unit	Sections 3-3	3-3	3-3	4-4	4-4
INTERNAL FORCES AT SECTIOIN								
	Combination			Service	Strength	Extreme	Extreme	Strength
Qu	Shear		kN	11834	16830	12128	3202	64
Mu	Flexural Moment		kNm	6270	8666	5930	8068	6502
Nu	Axial load		kN	2487	3655	2578	2063	2924
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	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
A'ps	Compression prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	0 T00.0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
As	Tension Reinforcement	Number	bars	84	84	84	84	84
		Diameter	mm	25	25	25	20	20
		Area	m2	0.04124	0.04124	0.04124	0.02638	0.02638
A's	Compression Reinforcement	Number	bars	84	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
$\beta 1$	Stress block factor			0.836	0.836	0.836	0.836	0.836
c	Dis. Between centroid and top fiber		m	0.022	0.022	0.022	-0.022	-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	Corresp. 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 reistance		kNm	29373	26436	29373	17479	15731
Mu	Flexual moment		kNm	6270	8666	5930	8068	6502

(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	17586	17586	17586	17201	17201
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**		**OK**	**OK**	**OK**	**OK**	**OK**
(5.7.3.4)	**Conctrol of craking by distr. of reinf for RC member- Check?**		Yes	No	No	No	No
	Existing condition for structrure	1,2 or 3	3	3	3	3	3
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.063	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	m2	0.019	0.019	0.019	0.018	0.018
fsa	Value	Mpa	166	166	166	171	171
0.6*fy		Mpa	240	240	240	240	240
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	166	166	166	171	171
x	Dist. From compression fiber to centroid	m	0.268	-	-	-	-
J.d	Arm	m	1.748	-	-	-	-
Icr	Moment of inertia of the cracked section	m4	0.797	-	-	-	-
fs	Tensile stress in reinforcement fs = Msls / (As*J.d)	Mpa	87	-	-	-	-
	Checking for control cracking fs<fsa		**OK**	N.a	N.a	N.a	N.a
(5.10.8.2)	**Shrinkage and temperature Reinfocement (side distribution)**						
Areq	Area of required reinf	m2	0.00127	0.00127	0.00127	0.00127	0.00127
	Distribution on sides 7 D16	m2	0.00141	0.00141	0.00141	0.00141	0.00141
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45	0.45
	Checking		**OK**	**OK**	**OK**	**OK**	**OK**
	SHEAR AND TORSION CHECKING						
β	Factor indicating diag. cracked concr. to tension		2.1	1.9	2.2	2.2	3.0
θ	Angle of inclination of diagonal compressive	degree	37.76	41.04	37.23	37.06	28.74
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
	(de - 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
Av	Shear reinf area in spacing S	m2	0.0040	0.0040	0.0040	0.0040	0.0040
θ	Assume	degree	38.22	41.43	39.67	30.88	34.58
v	Shear stress in concrete	kN/m2	514	812	527	33	3
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e_x	Strain in tensile reinforcement		1.18E-03	1.51E-03	1.12E-03	1.11E-03	3.72E-04
	if ex<0, multiple with reduce factor		-	-	-	-	-
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok	Ok
v/fc	Ratio of shear stress and f'c		0.017	0.027	0.018	0.001	0.000
β	Final value		2.1	1.9	2.2	2.2	3.0
θ	Final value	degree	37.76	41.04	37.23	37.06	28.74
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	22322	20378	22630	23936	33220
Vs	Shear resistance provided by shear reinforcement	kN	6357	5657	6479	6865	9454
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	28679	26035	29109	30801	42674
Vn2	Vn2	kN	172769	172769	172769	181937	181937
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	28679	26035	29109	30801	42674
Vr	Factored shear resistance	kN	**28679**	**23432**	**29109**	**30801**	**38407**
Vu	Shear	kN	11834	16830	12128	3202	64
(5.8.2.7)	**Shear checking**		**OK**	**OK**	**OK**	**OK**	**OK**

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



Sign	Parameters		Unit	Sections				
				5-5	5-5	6-6	6-6	6-6
INTERNAL FORCES AT SECTIOIN								
	Combination			Service	Strength	Service	Strength	Strength
Qu	Shear		kN	78	122	213	331	331
Mu	Flexural Moment		kNm	181	287	188	292	292
Nu	Axial load		kN	0	0	0	0	0
Tu	Torsional Moment		kNm	0	0	0	0	0
FLEXURAL MOMENT CHECKING								
H	Section height		m	0.500	0.500	0.500	0.500	0.500
d's	Dis. From comp. fiber to centroid of comp. Reinf		m	0.058	0.058	0.058	0.058	0.058
d1x	Dis. From tens. fiber to centroid of tension Reinf		m	0.059	0.059	0.059	0.059	0.059
	Cover to reinf		m	0.050	0.050	0.050	0.050	0.050
ds	Dis. From comp. fiber to centroid of tension Reinf		m	0.441	0.441	0.441	0.441	0.441
d'ps	Dis. From comp. fiber to centroid of comp. prestressing steel		m	0.000	0.000	0.000	0.000	0.000
d1xp	Dis. From tens. fiber to centroid of tension prestressing steel		m	0.000	0.000	0.000	0.000	0.000
dps	Dis. From comp. fiber to centroid of tension prestressing steel		m	0.500	0.500	0.500	0.500	0.500
b	Width of the compression face of member		m	1.000	1.000	1.000	1.000	1.000
bw	Web width or diameter of a circular section		m	1.000	1.000	1.000	1.000	1.000
hf	Compression flange depth		m	0.000	0.000	0.000	0.000	0.000
Iz	Moment of inertia of section		m4	0.010	0.010	0.010	0.010	0.010
Amc	Section area		m2	0.500	0.500	0.500	0.500	0.500
	Steel choice							
Aps	Tension prestressing steel	P.S type		0	0	0	0	0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
A'ps	Compression prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	0 T00.0
		Number	tendons	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000	0.00000
As	Tension Reinforcement	Number	bars	7	7	7	7	7
		Diameter	mm	20	20	20	20	20
		Area	m2	0.00220	0.00220	0.00220	0.00220	0.00220
A's	Compression Reinforcement	Number	bars	7	7	7	7	7
		Diameter	mm	16	16	16	16	16
		Area	m2	0.00141	0.00141	0.00141	0.00141	0.00141
A'c	Shear reinforcement	Number	bars	3	3	3	3	3
		Diameter	mm	12	12	12	12	12
		Area	m2	0.00034	0.00034	0.00034	0.00034	0.00034
ϕ	Resistance factors for flexure		5.5.4.2	1.00	0.90	1.00	0.90	0.90
ϕv	Resistance factors for shear			1.00	0.90	1.00	0.90	0.90
ϕn	Resistance factors for axial force			1.00	1.00	1.00	1.00	1.00
$\beta 1$	Stress block factor			0.836	0.836	0.836	0.836	0.836
c	Dis. Between centroid and top fiber		m	0.015	0.015	0.015	0.015	0.015
	For T section behavior		m	0.015	0.015	0.015	0.015	0.015
	For rectangular section behavior		m	0.015	0.015	0.015	0.015	0.015
fpe	Effective stress in the prestressing steel after losses		Mpa	1116	1116	1116	1116	1116
fps	Aver. stress in pres. steel at the time for which the nominal resistance		Mpa	1844	1844	1844	1844	1844
k	Factor depends on type of P.S, Low relaxation strand k = 0.28			0.28	0.28	0.28	0.28	0.28

a	Depth of equivalent stress block	m	0.012	0.012	0.012	0.012	0.012
de	Corresp. effective depth from extreme comp. fiber						
	to centroid of tensile force in the tensile reinf.	m	0.441	0.441	0.441	0.441	0.441
Mn	Nominal resistance	kNm	353	353	353	353	353
Mr	Factored reistance	kNm	**353**	**318**	**353**	**318**	**318**
Mu	Flexual moment	kNm	181	287	188	292	292
(5.7.3.2)	**Flexural moment Checking**		**OK**	**OK**	**OK**	**OK**	**OK**
	Limits for reinforcement						
c/de	Maximum reinforcement		0.03	0.03	0.03	0.03	0.03
	Maximum reinforcement Checking	<= 0.42	**OK**	**OK**	**OK**	**OK**	**OK**
r min	Mimimum reinforcement		0.44%	0.44%	0.44%	0.44%	0.44%
	Mimimum reinforcement Checking for RC	0.23%	**OK**	**OK**	**OK**	**OK**	**OK**
1.2*Mcr	Craking moment	kNm	89	89	89	89	89
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**		**OK**	**OK**	**OK**	**OK**	**OK**
(5.7.3.4)	**Conctrol of craking by distr. of reinf for RC member- Check?**		Yes	No	Yes	No	No
	Existing condition for structrure	1,2 or 3	1	1	1	1	1
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.059	0.059	0.059	0.059	0.059
Z	Crack width parameter	N/mm	30000	30000	30000	30000	30000
A	Area of concr. with same centroid as tens. Reinf	m2	0.017	0.017	0.017	0.017	0.017
fsa	Value	Mpa	301	301	301	301	301
0.6*fy		Mpa	240	240	240	240	240
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	240	240	240	240	240
x	Dist. From compression fiber to centroid	m	0.102	-	0.102	-	-
J.d	Arm	m	0.407	-	0.407	-	-
Icr	Moment of inertia of the cracked section	m4	0.002	-	0.002	-	-
fs	Tensile stress in reinforcement fs = Msls / (As*J.d)	Mpa	202	-	210	-	-
	Checking for control cracking fs<fsa		**OK**	**N.a**	**OK**	**N.a**	**N.a**
(5.10.8.2)	**Shrinkage and temperature Reinfocement (side distribution)**						
Areq	Area of required reinf	m2	0.00031	0.00031	0.00031	0.00031	0.00031
	Distribution on sides 7 D16	m2	0.00141	0.00141	0.00141	0.00141	0.00141
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45	0.45
	Checking		**OK**	**OK**	**OK**	**OK**	**OK**
	SHEAR AND TORSION CHECKING						
β	Factor indicating diag. cracked concr. to tension		2.2	1.9	2.1	1.7	1.7
θ	Angle of inclination of diagonal compressive	degree	36.66	41.63	38.90	42.76	42.76
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	90
bv	Effective web width as minimum web width - in dv	m	1.000	1.000	1.000	1.000	1.000
dv	Effective shear depth	m	0.435	0.435	0.435	0.435	0.435
	(de - a/2)	m	0.435	0.435	0.435	0.435	0.435
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600	0.600
ncat	Amount of bars in spacing S	bars	3	3	3	3	3
Av	Shear reinf area in spacing S	m2	0.0003	0.0003	0.0003	0.0003	0.0003
β	Assume		2.0	2.0	2.0	2.0	2.0
θ	Assume	degree	36.15	41.31	38.30	42.39	42.39
v	Shear stress in concrete	kN/m2	179	311	490	845	845
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	1116
e_x	Strain in tensile reinforcement		1.07E-03	1.66E-03	1.29E-03	1.94E-03	1.94E-03
	if ex<0, multiple with reduce factor		-	-	-	-	-
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok	Ok
v/f'c	Ratio of shear stress and f'c		0.006	0.010	0.016	0.028	0.028
β	Final value		2.2	1.9	2.1	1.7	1.7
θ	Final value	degree	36.66	41.63	38.90	42.76	42.76
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	434	371	409	346	346
Vs	Shear resistance provided by shear reinforcement	kN	132	111	122	106	106
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	566	482	531	452	452
Vn2	Vn2	kN	3261	3261	3261	3261	3261
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	566	482	531	452	452
Vr	Factored shear resistance	kN	**566**	**434**	**531**	407	407
Vu	Shear	kN	78	122	213	331	331
(5.8.2.7)	**Shear checking**		**OK**	**OK**	**OK**	**OK**	**OK**

SPACE PILE FOUNDATION ANALYSIS PROGRM
Turbo BASIC

PROJECT: : Cau ORB11 - KM44+440.00

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 = 1500 (t/m4)

LOAD COMBINATIONS

COMB.	Hx	Hy	P	Mx	My	Mz
1	362.00	0.00	1441.00	-346.00	665.00	0.00
2	256.00	0.00	1009.00	-330.00	548.00	0.00
3	344.00	6.00	1380.00	-311.00	587.00	0.00
4	237.00	6.00	948.00	-296.00	470.00	0.00
5	256.00	5.00	1013.00	-237.00	495.00	0.00
6	327.00	76.00	1250.00	-467.00	359.00	0.00
7	327.00	76.00	818.00	-451.00	549.00	0.00

PROPERTIES OF PILES

PILE	Lo	H	Bpx	Bpy	A	B	Cday	Fo	Io	Po	Co	Ct
1	0.00	10.00	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	5.30	0.000	0.00
2	-1.75	0.31	0.000	0.00
3	-1.75	-4.67	0.000	0.00
4	1.75	-4.67	0.000	0.00
5	1.75	0.31	0.000	0.00
6	1.75	5.30	0.000	0.00

DISPLACEMENTS

COMB.	X	Y	Z	Fix	Fiy	Fiz
1	0.00628	-0.00002	0.002643	0.000012	0.000758	-0.000069
2	0.00452	0.00000	0.001848	-0.000001	0.000577	-0.000049
3	0.00591	0.00005	0.002532	0.000012	0.000697	-0.000066
4	0.00415	0.00008	0.001736	-0.000001	0.000515	-0.000045
5	0.00447	0.00005	0.001858	0.000008	0.000549	-0.000049
6	0.00540	0.00103	0.002282	-0.000025	0.000557	-0.000063
7	0.00561	0.00106	0.001486	-0.000038	0.000658	-0.000063

FORCES ON PILES

PILE	COMB.	N	Q2	Q3	M1	M2	M3
1	1	114.16	-55.84	1.58	-1.310	2.921	90.908
	2	76.84	-39.49	1.12	-0.927	2.410	62.803
	3	113.64	-53.06	0.50	-1.245	0.625	87.247
	4	76.54	-36.56	0.03	-0.858	0.109	58.855
	5	77.88	-39.49	0.28	-0.927	0.319	63.816
	6	131.14	-50.44	-11.24	-1.183	-22.898	86.739
	7	49.00	-50.44	-11.24	-1.183	-22.430	83.107
2	1	119.45	-60.33	1.58	-1.310	2.921	100.427
	2	76.24	-42.67	1.12	-0.927	2.410	69.534
	3	119.07	-57.33	0.50	-1.245	0.625	96.292
	4	76.05	-39.50	0.03	-0.858	0.109	65.087
	5	81.38	-42.67	0.28	-0.927	0.319	70.547
	6	119.66	-54.50	-11.24	-1.183	-22.898	95.337
	7	31.64	-54.50	-11.24	-1.183	-22.430	91.705
3	1	124.73	-64.83	1.58	-1.310	2.921	109.945
	2	75.65	-45.84	1.12	-0.927	2.410	76.265
	3	124.51	-61.60	0.50	-1.245	0.625	105.337
	4	75.55	-42.44	0.03	-0.858	0.109	71.319
	5	84.88	-45.84	0.28	-0.927	0.319	77.278
	6	108.19	-58.56	-11.24	-1.183	-22.898	103.935
	7	14.29	-58.56	-11.24	-1.183	-22.430	100.303
4	1	366.17	-64.83	-1.58	-1.310	-3.762	109.945
	2	259.50	-45.84	-1.12	-0.927	-2.316	76.265
	3	346.36	-61.60	-2.50	-1.245	-5.726	105.337
	4	239.46	-42.44	-2.03	-0.858	-4.266	71.319
	5	259.79	-45.84	-1.95	-0.927	-4.407	77.278
	6	285.53	-58.56	-14.09	-1.183	-28.935	103.935
	7	223.67	-58.56	-14.09	-1.183	-28.467	100.303
5	1	360.89	-60.33	-1.58	-1.310	-3.762	100.427
	2	260.09	-42.67	-1.12	-0.927	-2.316	69.534
	3	340.93	-57.33	-2.50	-1.245	-5.726	96.292
	4	239.95	-39.50	-2.03	-0.858	-4.266	65.087
	5	256.29	-42.67	-1.95	-0.927	-4.407	70.547
	6	297.00	-54.50	-14.09	-1.183	-28.935	95.337
	7	241.02	-54.50	-14.09	-1.183	-28.467	91.705
6	1	355.60	-55.84	-1.58	-1.310	-3.762	90.908

2	260.69	-39.49	-1.12	-0.927	-2.316	62.803
3	335.49	-53.06	-2.50	-1.245	-5.726	87.247
4	240.45	-36.56	-2.03	-0.858	-4.266	58.855
5	252.79	-39.49	-1.95	-0.927	-4.407	63.816
6	308.47	-50.44	-14.09	-1.183	-28.935	86.739
7	258.38	-50.44	-14.09	-1.183	-28.467	83.107

SUMMARY OF FORCES

	PILE	COMB.	N	Q2	Q3	M1	M2	M3
Nmin	3	7	14.29	-58.56	-11.24	-1.183	-22.430	100.303
Nmax	4	1	366.17	-64.83	-1.58	-1.310	-3.762	109.945
Q2max	3	1	124.73	-64.83	1.58	-1.310	2.921	109.945
Q3max	4	6	285.53	-58.56	-14.09	-1.183	-28.935	103.935
M1max	1	1	114.16	-55.84	1.58	-1.310	2.921	90.908
M2max	4	6	285.53	-58.56	-14.09	-1.183	-28.935	103.935
M3max	3	1	124.73	-64.83	1.58	-1.310	2.921	109.945

CHECKING CALCULATI
IN COMPARISON WITH INITIA LOAD MATRIX

1	362.00	0.00	1441.00	-346.00	665.00	0.00
2	256.00	0.00	1009.00	-330.00	548.00	0.00
3	344.00	6.00	1380.00	-311.00	587.00	0.00
4	237.00	6.00	948.00	-296.00	470.00	0.00
5	256.00	5.00	1013.00	-237.00	495.00	0.00
6	327.00	76.00	1250.00	-467.00	359.00	0.00
7	327.00	76.00	818.00	-451.00	549.00	0.00

	DANANG QUANG NGAI EXPRESSWAY	Item.	Eng.	Date.	Sign.
	ORB11 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		**ORB11-A1**	
Bottom of pilecap elavation	EL1 =	10.00	
Top of socket elevation	EL2 =	3.00	
Pile tip elevation	EL3 =	0.00	
Pile Length	L =	10.00	m
Diameter of drilled-shaft	D_p =	1.00	m
Pile Cross-Sectional Perimeter	P =	3.14	m
Pile Cross-Sectional Area	A_b =	0.79	m^2
Working normal force at pile head	N =	3784.7	kN
Working normal force at top of socket	P_i =	3750.1	kN
Intack rock modulus	E_i =	25000	MPa
Modulus modification ratio	K_e =	0.05	
Elastic modulus of the insitu rock	$E_r = K_e*E_i$ =	1250.0	MPa
Influence coefficient	$I_p = f(H_s/D_s, E_c/E_r)$ =	0.30	
	H_s/D_s =	3.00	
	E_c/E_r =	22.15	

Pile Concrete comp. strength	f_c =	30.0	MPa
Concrete Unit Weight	g_c =	24.5	kN/m^3
Modulus of elasticity of concre	E_c =	27691	MPa
Depth of socket	H_s =	3.00	m
Diameter of socket	D_s =	1.00	m
Socket Cross-Sect. Perimeter	P_{soc} =	3.14	m
Socket Cross-Sectional Area	A_{soc} =	0.79	m^2

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

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

Atmospheric pressure p_a = 0.101 MPa

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

The elastic shorterning of the drilled shaft

$r_e = (\Sigma P_i)*H_s/(A_{soc}*E_c) =$ 0.517 mm

The settlement of base of drilled shaft

$r_{base} = (\Sigma P_i)*I_p/(D_s*E_r) =$ 0.900 mm

$r_e + r_{base} =$ **1.417** mm < 10 mm

Compute the bearing capacity based on Sharft resistance alone

Unit side resistance

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

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

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

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

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

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

Drilling method used in construction: *Case 2*

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

ϕ_s is the resistance factor - table 10.5.5-3 LRFD

q_u is the uniaxial compressive strength of the rock

Case1

No.	EL_T	EL_B	Depth (m)	RQD (%)	q_u (MPa)	q_s (MPa)	Q_{SR} (kN)	ϕ_s	Q_R (kN)
1	3.00	2.00	1.00	50	42.72	1.37	4312	0.65	2803
2	2.00	1.00	1.00	50	42.72	1.37	4312	0.65	2803
3	1.00	-	1.00	75	42.72	1.37	4312	0.65	2803
4						-	-	-	-
5									
6									
7									
8									
Sum			**3.00**				**12936**		**8409**

	DANANG QUANG NGAI EXPRESSWAY	Item.	Eng.	Date.	Sign.
	ORB11 BRIDGE	Design			
	DETAIL DESIGN	Check			
	EMPIRICAL ESTIMATION OF PILE CAPACITY-ABUTMENT A1	Revise			

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

No.	Depth (m)	RQD (%)	q_u (MPa)	E_m / E_i	α_E	Type	qs0 (MPa)	q_s (MPa)	q_s - *used* (MPa)	Q_{SR} (kN)	ϕ_s	Q_R (kN)
1	1.00	50.00	42.72	0.15	0.59	1	13.58	0.79	*0.79*	2492	0.55	1371
2	1.00	50.00	42.72	0.15	0.59	1	13.58	0.79	*0.79*	2492	0.55	1371
3	1.00	75.00	42.72	0.75	0.90	1	13.58	1.22	*1.22*	3818	0.55	2100
4	-	-	-	-	-	-	-	-	-	-	-	-
5	-	-	-	-	-	-	-	-	-	-	-	-
6	-	-	-	-	-	-	-	-	-	-	-	-
7	-	-	-	-	-	-	-	-	-	-	-	-
8	-	-	-	-	-	-	-	-	-	-	-	-
Sum	**3.00**									**8802**		**4841**

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

Limit pressure determined from presuremeter tests	$p_l =$	5.89	MPa	*C10.8.3.5-7*
At rest total horizontal stress measured at ther base elevation	$p_0 =$	-	MPa	
Coefficient that depen on diameter socket	$K_b =$	4.20		*Table C10.8.3.5-1*
Total vertical stress at the base elevation	$\sigma v =$	-	MPa	
	$q_p =$	-	MPa	
	$\phi =$	0.50		*Table 10.5.5-3*
$Q_{pR} = Ap . qp$	$Q_{pR} =$	-	kN	
$Q_R = \phi.Q_{pR}$	$Q_R =$	-	kN	

ESTIMATED PILE CAPACITY:

Pile Structural Capacity $Q_T = 0.75*0.85*0.85*f'c*Ag$		*12768*	kN	*1301*	T
Pile resistance Q_R		*4841*	kN	*493*	T
Deducting pile weight		*-139*	kN	*-14*	T
Estimated Pile Capacity		*4702*	kN	*479*	T
Maximum Reaction - ULS	**Ok**	*3592*	kN	*366*	T

	DANANG QUANG NGAI EXPRESSWAY	Item.	Eng.	Date.	Sign.
	ORB11 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		**ORB11-A2**	
Bottom of pilecap elavation	$EL1 =$	10.00	
Top of socket elevation	$EL2 =$	6.95	
Pile tip elevation	$EL3 =$	4.00	
Pile Length	$L =$	6.00	m
Diameter of drilled-shaft	$D_p =$	1.00	m
Pile Cross-Sectional Perimeter	$P =$	3.14	m
Pile Cross-Sectional Area	$A_b =$	0.79	m^2
Working normal force at pile head	$N =$	3707.7	kN
Working normal force at top of socket	$P_i =$	3673.6	kN
Intack rock modulus	$E_i =$	25000	MPa
Modulus modification ratio	$K_o =$	0.05	
Elastic modulus of the insitu rock	$E_r = K_o*E_i =$	1250.0	MPa
Influence coefficient	$I_p = f(H_s/D_s, E_c/E_r) =$	0.30	
	$H_s/D_s =$	2.95	
	$E_c/E_r =$	22.15	

Pile Concrete comp. strength	$f'_c =$	30.0	MPa
Concrete Unit Weight	$g_c =$	24.5	kN/m^3
Modulus of elasticity of concre	$E_c =$	27691	MPa
Depth of socket	$H_s =$	2.95	m
Diameter of socket	$D_s =$	1.00	m
Socket Cross-Sect. Perimeter	$P_{soc} =$	3.14	m
Socket Cross-Sectional Area	$A_{soc} =$	0.79	m^2

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

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

Atmospheric pressure $p_a =$ 0.101 MPa

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

The elastic shorterning of the drilled shaft

$r_o = (\Sigma P_i)*H_s/(A_{soc}*E_c) =$ 0.498 mm

The settlement of base of drilled shaft

$r_{baso} = (\Sigma P_i)*I_p/(D_s*E_r) =$ 0.882 mm

$r_e + r_{base} =$ **1.380** mm < 10 mm

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

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

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

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

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

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

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

Drilling method used in construction: *Case 2*

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

ϕ_s is the resistance factor - table 10.5.5-3 LRFD

q_u is the uniaxial compressive strength of the rock

Case1

No.	EL_T	EL_B	Depth (m)	RQD (%)	q_u (MPa)	q_s (MPa)	Q_{SR} (kN)	ϕ_s	Q_R (kN)
1	6.95	5.95	1.00	50	42.72	1.37	4312	0.65	2803
2	5.95	4.95	1.00	50	42.72	1.37	4312	0.65	2803
3	4.95	4.00	0.95	75	42.72	1.37	4096	0.65	2663
4						-	-	-	-
5									
6									
7									
8									
Sum			**2.95**				**12721**		**8268**

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

No.	Depth (m)	RQD (%)	q_u (MPa)	E_m/E_i	α_E	Type	qs0 (MPa)	q_s (MPa)	*q_s - used* (MPa)	Q_{SR} (kN)	ϕ_s	Q_R (kN)
1	1.00	50.00	42.72	0.15	0.59	1	13.58	0.79	*0.79*	2492	0.55	1371
2	1.00	50.00	42.72	0.15	0.59	1	13.58	0.79	*0.79*	2492	0.55	1371
3	0.95	75.00	42.72	0.75	0.90	1	13.58	1.22	*1.22*	3627	0.55	1995
4	-	-	-	-	-	-	-	-	-	-	-	-
5	-	-	-	-	-	-	-	-	-	-	-	-
6	-	-	-	-	-	-	-	-	-	-	-	-
7	-	-	-	-	-	-	-	-	-	-	-	-
8	-	-	-	-	-	-	-	-	-	-	-	-
Sum	**2.95**									**8611**		**4736**

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

Limit pressure determined from presuremeter tests $p_l =$ 5.89 MPa *C10.8.3.5-7*

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

Coefficient that depen on diameter socket $K_b =$ 4.17 *Table C10.8.3.5-1*

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

$q_p =$ - MPa

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

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

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

ESTIMATED PILE CAPACITY:

Pile Structural Capacity	$Q_T = 0.75*0.85*0.85*fc*Ag$	*12768*	kN	*1301*	T
Pile resistance	Q_R	*4736*	kN	*483*	T
Deducting pile weight		*-92*	kN	*-9*	T
Estimated Pile Capacity		*4644*	kN	*473*	T
Maximum Reaction - ULS	**Ok**	*3592*	kN	*366*	T

DANANG QUANG NGAI EXPRESSWAY ORB11 BRIDGE	Item.	Eng.	Date.	Sign.
	Design			
DETAIL DESIGN CHECK REINFORCEMENT OF BORED PILE	Check			
	Revise			

BORED PILE DESIGN

I. BORED PILE DATA

1.Load Combinations at top of bored pile

No	Combinations	Sign	F_V (kN)	Longitudinal F_{HX} (kN)	Longitudinal My (kN•m)	Transvesal F_{HY} (kN)	Transvesal Mx (kN•m)
1	Strength Str-IA		0	0	0	0	0
2	Strength Str-IA		3592	636	-1079	15	37
3	Strength Str-IA		0	0	0	0	0
4	Strength Str-IA		1120	548	-892	-15	-29
5	Strength Str-IA		1224	636	-1079	-15	-29
6							

2. Bored pile Material

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

3. Bored pile Section

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

II. PILE DESIGN

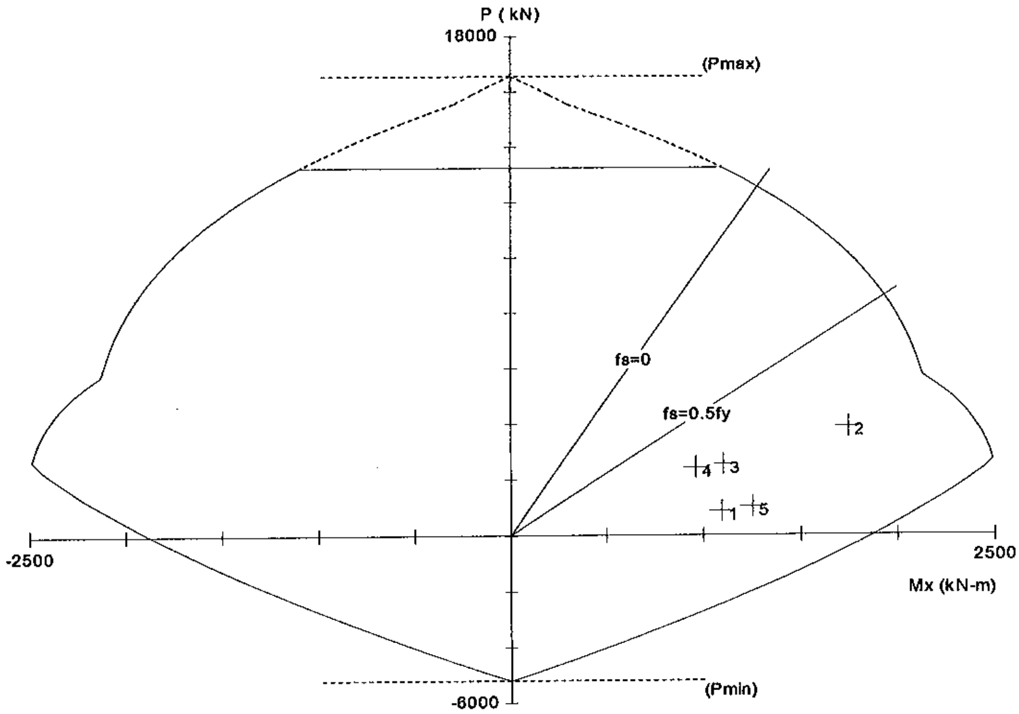
1. Limit of Reinforcement

S.5.7.4.2

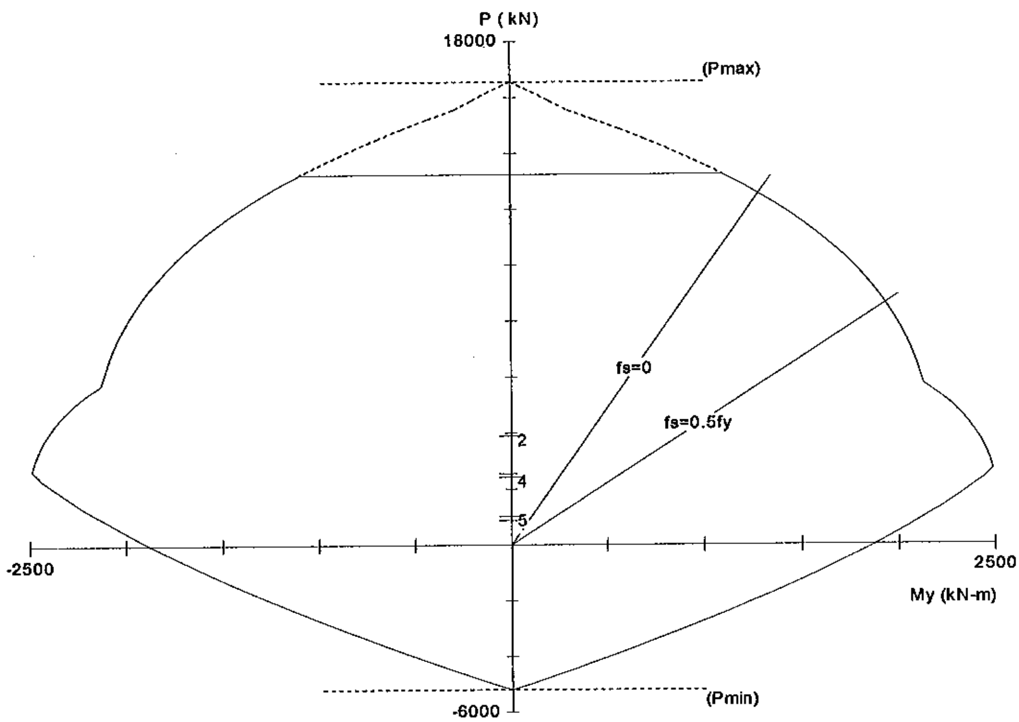
Minimum area of longitudinal reinforcement in column			
As.fy / (Ag . f'c) >= 0.135	As ≥	0.008	m2
As / Ag >= 0.01	As ≥	0.008	m2
Maximum area of longitudinal reinforcement in column			
As / Ag <= 0.08	As ≤	0.063	m2
Trial Rebars: Ok	As	**0.015**	m2
1layers x 24 = 24 bars D28 @150	As1	0.015	m2

2. Interation diagram M-P

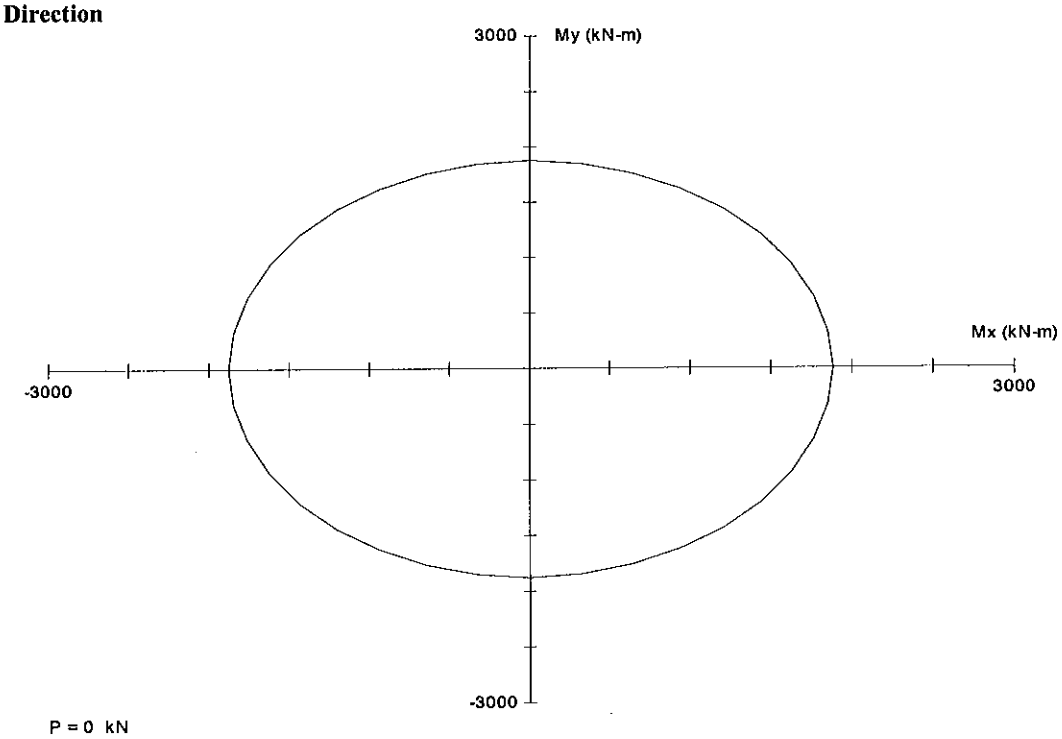
****In Transverse Direction**



****In Longitudinal Direction**



**In Both Direction



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

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

4. Shear Design

Shear resistance factors	ϕv	1.0	
Factored shear force	Vu	636	kN
Required shear capacity Vn = Vu/ ϕv	Vn	636	kN
Determine concrete shear capacity			
Minimum shear reinforcement will provided in cross section			
Therefore	β	2.0	
	θ	45.0	deg
Diameter of bored pile	D	1.00	m
Width of cross section	b	1.00	m
dv = 0.9*de de = D/2 + Dr/pi()			
Diameter of the circle passing through the centers of the long. reinf.	Dr	0.79	m
	de	0.75	m
	dv	0.68	m

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

a.STRUCTURE DIMENSIONS & LOAD COMPONENTS

I. GENERAL DATA

Assumptions :

1.The Design of the Pier conforms to "Specification for bridge design 22-TCN-272-05" and AASHTO LRFD 2004, JIS,... for reference.

2.Design live load: HL-93 and lane loading 9.3 KN/m

3.Bridge is considered to be in seismic with acceleration coefficient A = 0.0310 g

Input data:

Bridge type	*Simple PC I girder L=30m with link slab*					
Span length	Left	=	24.05	Right	=	24.05 m
Girder length between bearings	Left	=	23.20	Right	=	23.20 m
Bridge width	B	=	12.74 m			

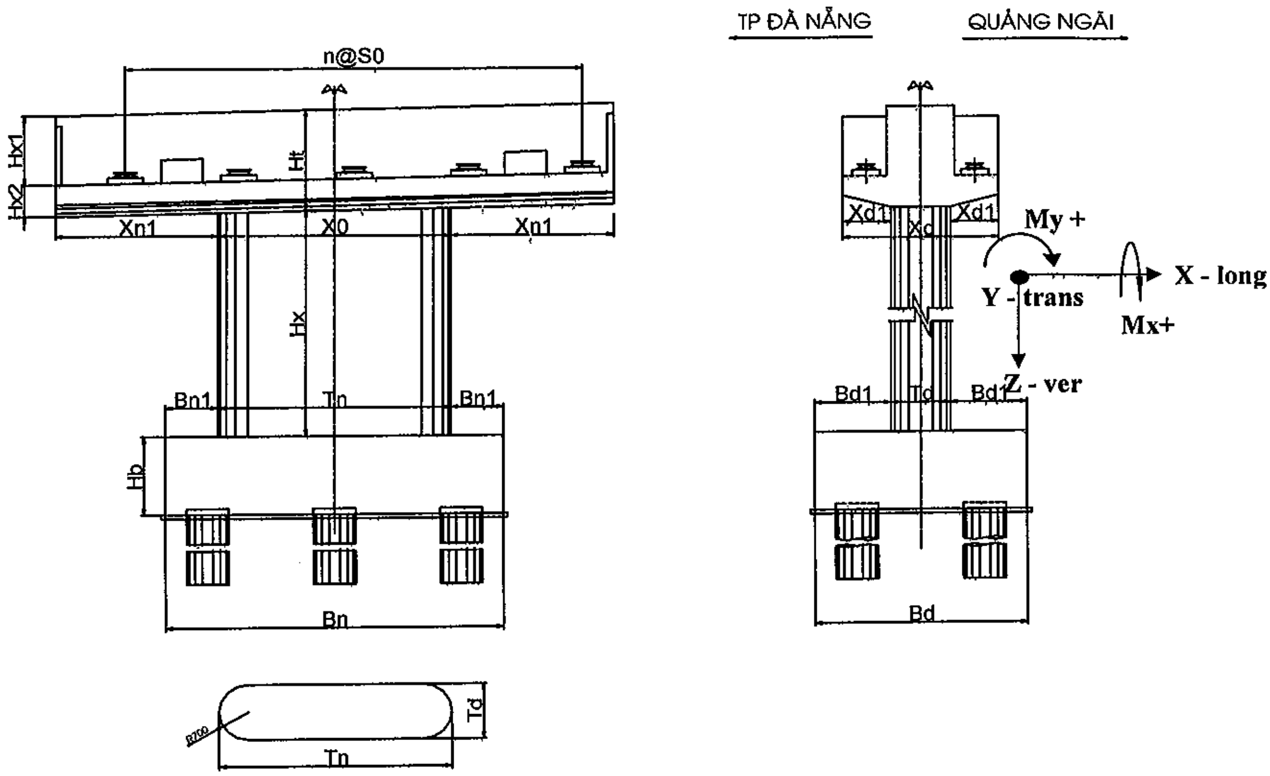
Level Table(at center of pier)

Top of pier cap	ThL	16.438	m
Top of pier column	TcL	14.119	m
Bottom of upper pier column	BucL	14.119	m
Bottom of pier column	BcL	4.500	m
Bottom of upper pilecap	BupL	4.500	m
Bottom of pilecap	BpL	2.500	m
Tip of pile	TpL		m
Skew angle	Ska	90.000	deg
Ground level	GL	8.720	m
Maximum water level (H1%)	HWL	14.220	m
Navigation water level (H5%)	NWL	10.040	m
Minimum water level	MWL	10.040	m
Average Annual water level	AWL	10.040	m
Local scour level (at water level H1%)	LsL	10.040	m

Material unit weight

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

II. PIER DIMENSIONS



Pier Dimensions Table

Notation	Dimensions Transverse direction	Value (m)	Notation	Dimensions Longitudinal direction	Value (m)
*	***Bearing distribution***			Dist. from bearing to pier c.line	
nbear	Number of bearing	5.00	e1	Girder 1	5.10
S0	Bearings spacing	2.55	e2	Girder 2	2.55
			e3	Girder 3	0.00
			e4	Girder 4	-2.55
			e5	Girder 5	-5.10
*	***Bearing pedestal*** Width	0.85	*	***Anchorage block*** Width	0.40
	Length	0.65		Length	1.00
	Height	0.15		Height	0.52
	Number	10.00		Number	4.00
*	***Pier Cap***				
Hx1	Haunch 1 height	1.52	xd	Pier cap width	1.40
Hx2	Haunch 2 height	0.80	GL	Left bearing to pier c.line	1.250
Hx	Pier cap height	2.32	GR	Right bearing to pier c.line	1.250
Xn1	Haunch width	3.49			
Xn0	Bottom of pier cap width	5.50	Hc	Curtain wall height	1.50
Xnt	Top of pier cap width	12.48	tc	Curtain wall thickness	0.15
*	***Pier Column***				
tn	Pier column width	5.50	td	Pier column thichness	1.40
Htt	Pier column height	9.62	Rv	Round nose radius	0.70
tnb	Upper pier column width	0.00	tdb	Upper pier column thichness	0.00
Htb	Upper pier column height	0.00	Rvb	Upper round nose radius	0.00
Ht	Column height	9.62			

Notation	Dimensions Transverse direction	Value (m)	Notation	Dimensions Longitudinal direction	Value (m)
*	***Pile Cap***				
Bn	Pile cap width	8.00	Bd	Pile cap length	5.00
Hb	Pile cap depth	2.00			
Bn1	Transverse cantilever	1.25	bd1	Long. Cantilever	1.80
Bnb	Upper pile cap width	0.00	Bdb	Upper pile cap length	0.00
Hbb	Upper pile cap depth	0.00			

III. SUBSTRUCTURE LOADS

1.Pier Selfweight

Item	Volume	Vertical	Longitudinal			Transversal		
		F_V	F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m3)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Bearing pedestal	0.83	20.3						
Bearing devices		50.0						
Anchorage block	0.84	20.5						
Pier Cap	70.25	1721.1						
Curtain wall	0.50	12.1						
Upper pier column	0.00	0.0						
Pier Column	70.02	1715.5						
Upper pilecap	0.00	0.0						
PileCap	80.00	1960.0						
Shear key	0.00	0.0						
Total at bottom of Column		***3539.6***						
Total at bottom of pilecap		***5499.6***						

2.Soil on pilecap

Item	Volume	Vertical	Longitudinal			Transversal		
		F_V	F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m3)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Soil on pile cap	138.08	2444.0						
Total at bottom of Column								
Total at bottom of pilecap		***2444.0***						

3.Buoyancy on pier

Case1: Maximum water level (H1%)

Item	Volume	Vertical	Longitudinal			Transversal		
		F_V	F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m3)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Upper pier column	0.00	0.0						
Pier Column	70.02	-686.9						
Upper pilecap	0.00	0.0						
PileCap	80.00	-784.8						
Shear key	0.00	0.0						
Total at bottom of Column		***-686.9***						
Total at bottom of pilecap		***-1471.7***						

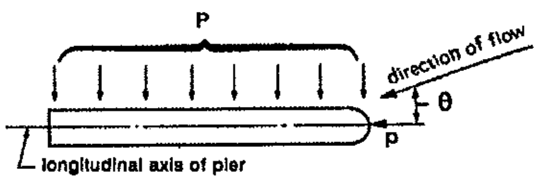
Case2: Minimum water level (Hmin)

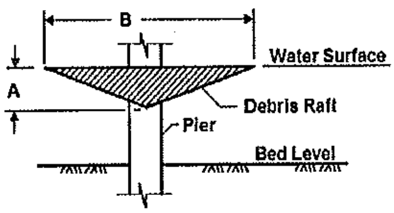
Item	Volume	Vertical	Longitudinal			Transversal		
		F_V	F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m3)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Upper pier column	0.00	0.0						
Pier Column	40.33	-395.6						
Upper pilecap	0.00	0.0						
PileCap	80.00	-784.8						
Shear key	0.00	0.0						
Total at bottom of Column		***-395.6***						
Total at bottom of pilecap		***-1180.4***						

Case3: average Annual water level

Item	Volume	Vertical	Longitudinal			Transversal		
		F_V	F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m3)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Upper pier column	0.00	0.0						
Pier Column	40.33	-395.6						
Upper pilecap	0.00	0.0						
PileCap	80.00	-784.8						
Shear key	0.00	0.0						
Total at bottom of Column		***-395.6***						
Total at bottom of pilecap		***-1180.4***						

4.Stream Pressure





Stream pressure data

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

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

Type of pier colum: "1:Semicircular - nosed pier"; "2:Square - ended pier";		1	
"3:Debris lodged against the pier "; "4:Wedged - nosed pier with nose angle 90deg or less"			
Drag coefficient	C_D	0.70	
Stream pressure at H1%	pL1%	0.44	kN/m2
Stream pressure at minimum water level	pLmin	0.05	kN/m2
Stream pressure at average annual water level	pLannual	0.44	kN/m2
Additional stream pressure due to driftwood raft lodged against pier			
Drag coefficient	C_D	0.50	
Height of debris raft	A	2.8	m
Width of debris raft	B	14.0	m
Area of debris raft	Area	19.3	m2
Stream pressure due to driftwood raft at H1%	pLdebris	0.32	kN/m2
Equivalent force	Fhdebris	6.1	kN

Lateral axis of pier	$pT = 5.14 \times 10^{-4} C_L V^2$		
Drag coefficient	C_L	0.00	
Stream pressure at H1%	pT1%	0.00	kN/m2
Stream pressure at minimum water level	pTmin	0.00	kN/m2
Stream pressure at average annual water level	pTannual	0.00	kN/m2

Case1: Maximum water level (H1%)

Item	Exposed height	Vertical F_V	Longitudinal			Transversal		
			F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Upper pier Column	0.10		0.0	9.7	0.0	0.0	9.7	0.0
Pier Column	9.62		0.0	4.8	0.0	6.0	4.8	28.7
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***	***6.0***		***28.7***
Upper pier Column	0.10		0.0	11.7	0.0	0.0	11.7	0.0
Pier Column	9.62		0.0	6.8	0.0	6.0	6.8	40.7
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***	***6.0***		***40.7***

Additional stream pressure due to driftwood raft

Item	Exposed height	Vertical F_V	Longitudinal			Transversal		
			F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Pier Column					0.0	6.1	9.7	59.2
Total at bottom of Column					***0.0***	***6.1***		***59.2***
Pier Column					0.0	6.1	11.7	71.4
Total at bottom of pilecap					***0.0***	***6.1***		***71.4***

Case2: Minimum water level (Hmin)

Item	Exposed height	Vertical F_V	Longitudinal			Transversal		
			F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Upper pier Column	0.00		0.0	9.6	0.0	0.0	9.6	0.0
Pier Column	5.54		0.0	2.8	0.0	0.4	2.8	1.2
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***	***0.4***		***1.2***
Upper pier Column	0.00		0.0	11.6	0.0	0.0	11.6	0.0
Pier Column	5.54		0.0	4.8	0.0	0.4	4.8	2.0
Upper pilecap	0.00		0.0	2.0	0.0	0.0	2.0	0.0
PileCap	0.00		0.0	0.0	0.0	0.0	0.0	0.0
Total at bottom of pilecap			***0.0***		***0.0***	***0.4***		***2.0***

Case3: average Annual water level

Item	Exposed height	Vertical F_V	Longitudinal			Transversal		
			F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Upper pier Column	0.00		0.0	9.6	0.0	0.0	9.6	0.0
Pier Column	5.54		0.0	2.8	0.0	3.4	2.8	9.5
Upper pilecap	0.00		0.0	0.0	0.0	0.0	0.0	0.0
Total at bottom of Column			***0.0***		***0.0***	***3.4***		***9.5***
Upper pier Column	0.00		0.0	11.6	0.0	0.0	11.6	0.0
Pier Column	5.54		0.0	4.8	0.0	3.4	4.8	16.4
Upper pilecap	0.00		0.0	2.0	0.0	0.0	2.0	0.0
PileCap	0.00		0.0	0.0	0.0	0.0	0.0	0.0
Total at bottom of pilecap			***0.0***		***0.0***	***3.4***		***16.4***

5.Wind Loads

Wind loads data	$P_D = 0.0006\ V^2\ C_d \geq 1.8$ (kN/m2)		
Reference standard "Wind region TCVN 2737-1995"			
Wind region (I, II, III, IV) at bridge location		III	
3 second gust wind velocity with 100 years return period	Vb	53.0	m/s
Type of terrain at bridge location		1	
"1:exposed area"; "2: forest, houses,... with height 10m"; "3:houses area..with height>10m"			
Average elevation of pier upper ground or water plane level	Hele_p	6.2	m
Correct coefficient for wind zone and elevation of pier	S	1.09	
Design wind speed V = S*Vb	V	57.8	m/s
Obstacle coefficient for pier	Cd	1.3	
Wind pressure on pier	P_D	2.60	kN/m2

At Maximum water level (H1%)

Item	Exposed height	Vertical F_V	Longitudinal			Transversal		
			F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Curtain wall	1.50		1.2	12.7	14.9	5.5	12.7	69.4
Pier Cap	2.22		72.1	10.7	773.0	8.1	10.7	86.7
Upper pier Column	0.00		0.0	9.7	0.0	0.0	9.7	0.0
Pier Column	0.00		0.0	9.7	0.0	0.0	9.7	0.0
Upper pilecap	0.00		0.0	9.7	0.0	0.0	9.7	0.0
Total at bottom of Column			***73.2***		***787.9***	***13.5***		***156.1***
Curtain wall	1.50		1.2	14.7	17.2	5.5	14.7	80.3
Pier Cap	2.22		72.1	12.7	917.1	8.1	12.7	102.9
Upper pier Column	0.00		0.0	11.7	0.0	0.0	11.7	0.0
Pier Column	0.00		0.0	11.7	0.0	0.0	11.7	0.0
Upper pilecap	0.00		0.0	11.7	0.0	0.0	11.7	0.0
PileCap	0.00		0.0	11.7	0.0	0.0	11.7	0.0
Total at bottom of pilecap			***73.2***		***934.3***	***13.5***		***183.2***

At Minimum water level (Hmin)

Item	Exposed height	Vertical F_V	Longitudinal			Transversal		
			F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Curtain wall	1.50		1.2	12.7	14.9	5.5	12.7	69.4
Pier Cap	2.32		75.3	10.8	812.0	8.5	10.8	91.1
Upper pier Column	0.00		0.0	5.5	0.0	0.0	5.5	0.0
Pier Column	4.08		58.4	7.6	442.6	14.9	7.6	112.7
Upper pilecap	0.00		0.0	5.5	0.0	0.0	5.5	0.0
Total at bottom of Column			***134.9***		***1269.5***	***28.8***		***273.1***
Curtain wall	1.50		1.2	14.7	17.2	5.5	14.7	80.3
Pier Cap	2.32		75.3	12.8	962.7	8.5	12.8	108.0
Upper pier Column	0.00		0.0	7.5	0.0	0.0	7.5	0.0
Pier Column	4.08		58.4	9.6	559.4	14.9	9.6	142.4
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			***134.9***		***1539.4***	***28.8***		***330.7***

At average Annual water level

Item	Exposed height	Vertical F_V	Longitudinal			Transversal		
			F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Curtain wall	1.50		1.2	12.7	14.9	5.5	12.7	69.4
Pier Cap	2.32		75.3	10.8	812.0	8.5	10.8	91.1

Upper pier Column	0.00		0.0	5.5	0.0	0.0	5.5	0.0
Pier Column	4.08		58.4	7.6	442.6	14.9	7.6	112.7
Upper pilecap	0.00		0.0	5.5	0.0	0.0	5.5	0.0
Total at bottom of Column			***134.9***		***1269.5***	***28.8***		***273.1***
Curtain wall	1.50		1.2	14.7	17.2	5.5	14.7	80.3
Pier Cap	2.32		75.3	12.8	962.7	8.5	12.8	108.0
Upper pier Column	0.00		0.0	7.5	0.0	0.0	7.5	0.0
Pier Column	4.08		58.4	9.6	559.4	14.9	9.6	142.4
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			***134.9***		***1539.4***	***28.8***		***330.7***

6.Vessel Collision

Vessel collision load data

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

Ship Collision force on pier P_S = 1.2e5*V*sqrt(DWT)

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

Towed barge Collision force on pier Pb = 6e4.ab

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

Vessel collision is applied for pier P1 or P2, this force is transmitfed to pier P3.
The values here are taken from whole bridge model analysis (refer to annex)

Item	**Height**	**Vertical**	**Longitudinal**			**Transversal**		
		F_V	F_{HX}	Arm.$_{HX}$	M_y	F_{HY}	Arm.$_{HY}$	M_x
	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Pier column	5.54		173.0	5.5	1447.0	404.0	5.54	3310.0
Total at bottom of Column								
Pier column	7.54		173.0	7.5	1794.0	404.0	7.54	4111.0
Total at bottom of pilecap			***173.0***		***1794.0***	***404.0***		***4111.0***

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

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

7.Vehicular Collision Force "1:yes"; "0:no" | 0 |

Item	Height	Vertical	Longitudinal			Transversal		
		F_V	F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Pier column	10.82		0.0	10.8	0.0	0.0	10.82	0.0
Total at bottom of Column			***0.0***		***0.0***	***0.0***		***0.0***
Pier column	12.82		0.0	12.8	0.0	0.0	12.82	0.0
Total at bottom of pilecap			***0.0***		***0.0***	***0.0***		***0.0***

IV. SUPERSTRUCTURE LOADS

1.Dead Loads

Left side Span

Item	Volume	Vertical	Longitudinal			Transversal		
		F_V	F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m3)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Stage1 (DC)								
Girders	78.45	961.0		1.250	-1201.3			
Diaphragm	11.55	141.5		1.250	-176.9			
Precast plank	12.18	149.3		1.250	-186.6			
Deck slab	69.55	852.0		1.250	-1065.0			
Total at bottom of Column		***2103.8***			***-2629.7***			
Total at bottom of pilecap		***2103.8***			***-2629.7***			
Stage2 (DW)								
Pavement	25.74	284.4		1.250	-355.5			
Parapet + railing		391.1		1.250	-488.8			
Lighting post + mis.		33.0		1.250	-41.3			
Total at bottom of Column		***708.4***			***-885.6***			
Total at bottom of pilecap		***708.4***			***-885.6***			

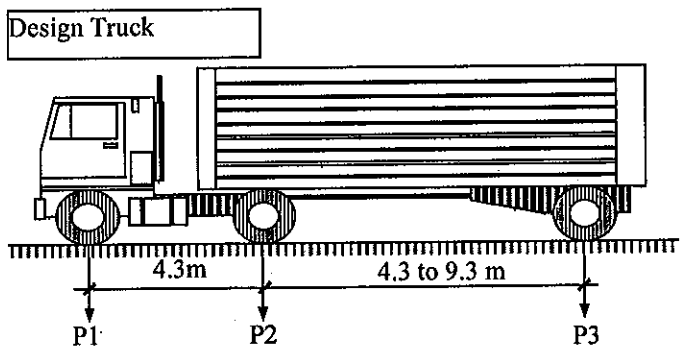
Right side Span

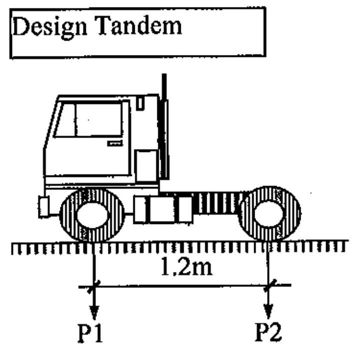
Item	Volume	Vertical	Longitudinal			Transversal		
		F_V	F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m3)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Stage1 (DC)								
Girders	78.45	961.0		1.250	1201.3			
Diaphragm	11.55	141.5		1.250	176.9			
Precast plank	12.18	149.3		1.250	186.6			
Deck slab	69.55	852.0		1.250	1065.0			
Total at bottom of Column		***2103.8***			***2629.7***			
Total at bottom of pilecap		***2103.8***			***2629.7***			
Stage2 (DW)								
Pavement	25.74	284.4		1.250	355.5			
Parapet + railing		391.1		1.250	488.8			
Lighting post + mis.		33.0		1.250	41.3			
Total at bottom of Column		***708.4***			***885.6***			
Total at bottom of pilecap		***708.4***			***885.6***			

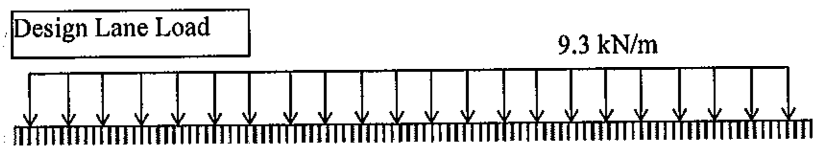
2.Live Load

Live load data

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

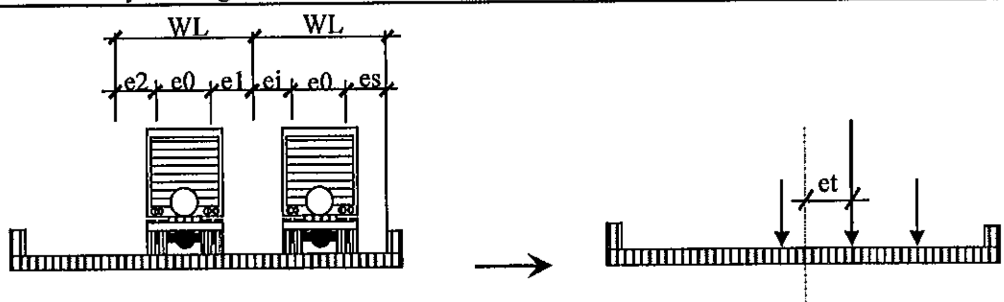






Transverse Distribution

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



Longitudinal Distribution

Case1a: 1Truck + Lane load on 1 span

Case1b: 1Truck + Lane load on 2 spans

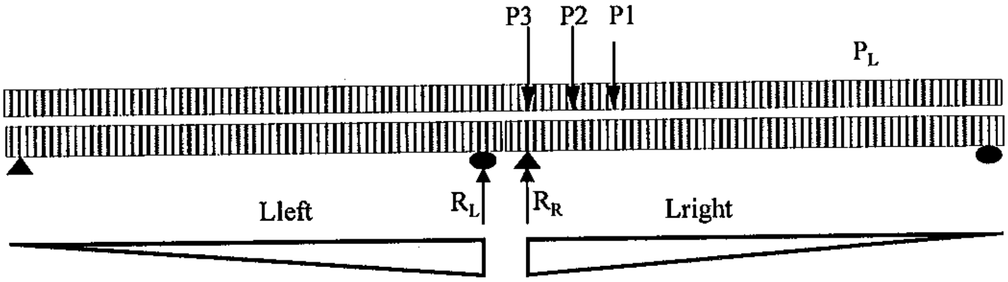
Case2a: 2Trucks + Lane load on 1 span
Case2b: 2Trucks + Lane load on 2 spans
Case3a: 1Tandem + Lane load on 1 span
Case3b: 1Tandem + Lane load on 2 spans

Influence line value

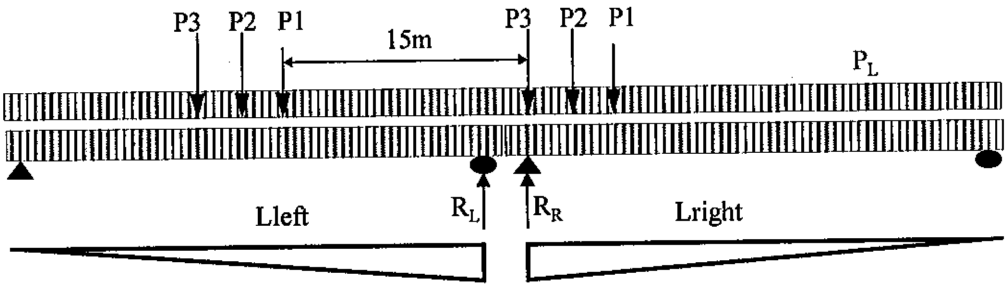
Case	Left span				Right span			
	P3	P2	P1	Aleft	P3	P2	P1	Aright
Case1a:				11.60	1.00	0.81	0.63	11.60
Case1b:	0.92			11.60		1.00	0.81	11.60
Case2a:*	1.00	0.81	0.63	11.60	-0.02	-0.20	0.00	11.60
Case2b:	0.09	0.28	0.46	11.60	1.00	0.81	0.63	11.60
Case3a:				11.60		1.00	0.95	11.60
Case3b:		1.06		11.60			1.00	11.60

* 2 Trucks in right span

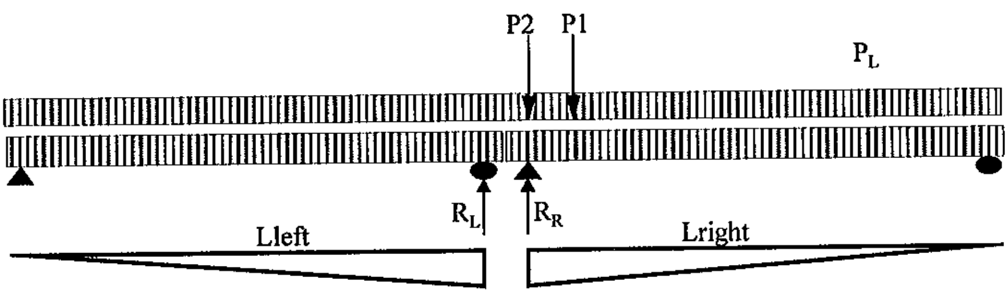
Truck + Lane load



2Trucks + Lane load



Tandem + Lane load



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

1 Loaded Lane m = 1.20

Item	Reaction left	Reaction right	Vertical F_V	Longitudinal			Transversal		
				F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(kN)	(kN)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Case1a:	129.5	557.2	686.6			534.7			3426.3
Case1b:	330.1	389.7	719.8			74.6			3591.8

Case2a:	116.5	458.4	574.9			427.4			2869.0
Case2b:	210.0	501.5	711.5			364.3			3550.3
Case3a:	129.5	450.9	580.4			401.8			2896.1
Case3b:	303.7	294.5	598.2			-11.6			2984.8

2 Loaded Lane m = 1.00

Item	Reaction left (kN)	Reaction right (kN)	Vertical F_V (kN)	Longitudinal F_{HX} (kN)	Longitudinal $Arm._{HX}$ (m)	Longitudinal M_y (kN•m)	Transversal F_{HY} (kN)	Transversal $Arm._{HY}$ (m)	Transversal M_x (kN•m)
Case1a:	215.8	928.6	1144.4			891.1			3993.9
Case1b:	550.1	649.5	1199.7			124.3			4186.9
Case2a:	194.2	764.1	958.2			712.3			3344.3
Case2b:	350.0	835.8	1185.8			607.2			4138.5
Case3a:	215.8	751.5	967.3			669.7			3375.9
Case3b:	506.2	490.8	996.9			-19.3			3479.3

3 Loaded Lane m = 0.85

Item	Reaction left (kN)	Reaction right (kN)	Vertical F_V (kN)	Longitudinal F_{HX} (kN)	Longitudinal $Arm._{HX}$ (m)	Longitudinal M_y (kN•m)	Transversal F_{HY} (kN)	Transversal $Arm._{HY}$ (m)	Transversal M_x (kN•m)
Case1a:	275.1	1184.0	1459.1			1136.1			2903.6
Case1b:	701.4	828.2	1529.6			158.4			3043.9
Case2a:	247.6	974.2	1221.8			908.2			2431.3
Case2b:	446.3	1065.6	1511.9			774.1			3008.7
Case3a:	275.1	958.2	1233.3			853.9			2454.3
Case3b:	645.4	625.7	1271.1			-24.6			2529.5

4 Loaded Lane m = 0.65

Item	Reaction left (kN)	Reaction right (kN)	Vertical F_V (kN)	Longitudinal F_{HX} (kN)	Longitudinal $Arm._{HX}$ (m)	Longitudinal M_y (kN•m)	Transversal F_{HY} (kN)	Transversal $Arm._{HY}$ (m)	Transversal M_x (kN•m)
Case1a:	280.5	1207.2	1487.7			1158.4			729.0
Case1b:	715.2	844.4	1559.6			161.5			764.2
Case2a:	252.4	993.3	1245.7			926.0			610.4
Case2b:	455.0	1086.5	1541.6			789.3			755.4
Case3a:	280.5	977.0	1257.5			870.6			616.2
Case3b:	658.0	638.0	1296.0			-25.0			635.0

Item Live Load	Vertical F_V (kN)	Longitudinal F_{HX} (kN)	Longitudinal $Arm._{HX}$ (m)	Longitudinal M_y (kN•m)	Transversal F_{HY} (kN)	Transversal $Arm._{HY}$ (m)	Transversal M_x (kN•m)
Total at bottom of Column	*1185.8*	*0.0*	*0.0*	*607.2*	*0.0*	*0.0*	*4138.5*
Total at bottom of pilecap	*1185.8*	*0.0*	*0.0*	*607.2*	*0.0*	*0.0*	*4138.5*

Pedestrian Load

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

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

Centrifugal force data			
Axle weights of design Truck	P	325.0	kN
Number of loaded lanes	n	3.0	lanes
	m	0.85	

Factor, C = (4/3)* V^2/ (g*R)	C	0.0	kN
Highway design speed	V	11.1	m/s
Radius of curvature of traffic lane	R	-	m
Centrifugal force	CE	0.0	kN

Item	From surface	Vertical F_V	Longitudinal			Transversal		
			F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Centrifugal force	1.80							
Total at bottom of Column						***0.0***	14.049	***0.0***
Total at bottom of pilecap						***0.0***	16.049	***0.0***

4.Braking Force

Braking force data

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

Item	From surface	Vertical F_V	Longitudinal			Transversal		
			F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
Take 50 %	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Braking force	1.80							
Total at bottom of Column			***103.6***	14.049	***1455.4***			
Total at bottom of pilecap			***103.6***	16.049	***1662.6***			

5.Uniform Temperature

Uniform temperature data

Installing temperature	t0	27.0	deg
Maximum temperature	tmax	47.0	deg
Minimum temperature	tmin	10.00	deg
Plus temperature amplitude	Δtmax	20.0	deg
Minus temperature amplitude	Δtmin	17.0	deg
Coefficient of Thermal Expansion	α	1.08E-05	
Strain due to minus temperature	ε_T	1.84E-04	
Span length from longitudinal center of inter-span	Lsp	0.1	m
Displacement along girder	Δs	9.18E-06	m
Horizontal force applies to top of pier: Fhx =(3.E.I) . Δs / H^3	Fhx	1	kN
	E	27691	MPa
	I	1.126	m4

Item	From surface	Vertical F_V	Longitudinal			Transversal		
			F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Total at bottom of Column			***0.5***	12.09	***6.1***			
Total at bottom of pilecap			***0.5***	14.09	***7.1***			

6.Creep & Shrinkage

Creep & shrinkage data

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

Item	From surface	Vertical F_V	Longitudinal F_{HX}	Longitudinal $Arm._{HX}$	Longitudinal M_y	Transversal F_{HY}	Transversal $Arm._{HY}$	Transversal M_x
	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Total at bottom of Column			***0.5***	12.09	***5.7***			
Total at bottom of pilecap			***0.5***	14.09	***6.7***			

7.Wind on Structure

Wind loads data $P_D = 0.0006\ V^2\ C_d \geq 1.8$ (kN/m2)

Average elevation of deck girder upper ground or water plane level	Hele_g	9.6	m
Correct coefficient for wind zone and elevation of pier	S	1.09	
Design wind speed V = S*Vb	V	57.8	m/s
Overall width between handrails	b	12.7	m
Superstructure height including solid parapet	d	3.03	m
	b/d	4.21	
Obstacle coefficient for pier	Cd	1.36	
Wind pressure on pier	P_D	2.72	kN/m2

Item	Exposed height	Vertical F_V	Longitudinal F_{HX}	Longitudinal $Arm._{HX}$	Longitudinal M_y	Transversal F_{HY}	Transversal $Arm._{HY}$	Transversal M_x
	(m)	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Superstructure	3.03		49.6	13.5	666.7	198.3	13.5	2666.8
Total at bottom of Column			***49.6***		***666.7***	***198.3***		***2666.8***
Superstructure	3.03		49.6	15.5	765.8	198.3	15.5	3063.3
Total at bottom of pilecap			***49.6***		***765.8***	***198.3***		***3063.3***

8.Wind on Vehicle

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

Item	Vertical F_V	Longitudinal F_{HX}	Longitudinal $Arm._{HX}$	Longitudinal M_y	Transversal F_{HY}	Transversal $Arm._{HY}$	Transversal M_x
	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Superstructure		18.0	15.8	285.9	36.1	15.8	571.8
Total at bottom of Column		***18.0***		***285.9***	***36.1***		***571.8***
Superstructure		18.0	17.8	322.0	36.1	17.8	643.9
Total at bottom of pilecap		***18.0***		***322.0***	***36.1***		***643.9***

9.Earth Quake N/A

Earth Quake data

Acceleration coefficient	A	0.0310	g
Seismic zone	Sz	1	
Soil profile type: according to geological data survey		I	
Coeffient site	S	1.00	
Bridge importance category: "1:critical"; "2:essential"; "3:other"	IC	3	other
Response Modification Factor			
Column		1.0	
Connection		1.0	
Foundation		1.0	

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

Item	Vertical	Longitudinal			Transversal		
	F_V	F_{HX}	$Arm._{HX}$	M_y	F_{HY}	$Arm._{HY}$	M_x
	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Total at bottom of Column							
Total at bottom of pilecap							

b.LOAD COMBINATIONS

I. LOAD COMBINATIONS

Loads at Bottom of Column						
Loads	Sign	F_V (kN)	Longitudinal F_{HX} (kN)	Longitudinal My (kN•m)	Transvesal F_{HY} (kN)	Transvesal Mx (kN•m)
Superstructure Loads						
1.Stage1 - Dead Load	DC	4208		0		
2.Stage2 - Pave.+Parapet+Railing+Mis.	DW	1417		0		
3.Live Load	LL	1186		607		4138
4.Pedestrian	LL	0		0		0
5.Centrifugal force	CE				0	0
6.Braking force	BR		104	1455		
7.Uniform temperature	TU		1	6		
8.Creep and Shrinkage	CR&SH		0	6		
9.Wind pressure on superstructure	WS		50	667	198	2667
10.Wind pressure on vehicles	WL		18	286	36	572
11.Earthequake						
a - Longitudinal direction	EQ		0	0		
b - Transverse direction	EQ				0	0
Substructure Loads						
1.Pier selfweight	DC	3540				
2.Soil on pile cap	EV	0				
3.Bouyancy on pier						
a - Maximum water level	WA	-687				
b - Minimum water level	WA	-396				
c - Average annual water level	WA	-396				
4.Stream pressure						
a - Maximum water level	WA		0	0	12	88
b - Minimum water level	WA		0	0	0	1
c - Average annual water level	WA		0	0	3	10
5.Wind pressure						
a - Maximum water level	WS		73	788	14	156
b - Minimum water level	WS		135	1270	29	273
c - Average annual water level	WS		135	1270	29	273
6.Vessel collision force						
a - Longitudinal direction	CV		0	0		
b - Transverse direction	CV				0	0
7.Vehicular collision force						
a - Longitudinal direction	CT		0	0		
b - Transverse direction	CT				0	0

Loads at Bottom of Pilecap

Loads	Sign	F_V (kN)	Longitudinal F_{HX} (kN)	Longitudinal My (kN•m)	Transvesal F_{HY} (kN)	Transvesal Mx (kN•m)
Superstructure Loads						
1.Stage1 - Dead Load	DC	4208		0		
2.Stage2 - Pave.+Parapet+Railing+Mis.	DW	1417		0		
3.Live Load	LL	1186		607		4138
4.Pedestrian	LL	0		0		0
5.Centrifugal force	CE				0	0
6.Braking force	BR		104	1663		
7.Uniform temperature	TU		1	7		
8.Creep and Shrinkage	CR&SH		0	7		
9.Wind pressure on superstructure	WS		50	766	198	3063
10.Wind pressure on vehicles	WL		18	322	36	644
11.Earthequake						
a - Longitudinal direction	EQ		0	0		
b - Transverse direction	EQ				0	0
Substructure Loads						
1.Pier selfweight	DC	5500				
2.Soil on pile cap	EV	2444				
3.Bouyancy on pier						
a - Maximum water level	WA	-1472				
b - Minimum water level	WA	-1180				
c - Average annual water level	WA	-1180				
4.Stream pressure						
a - Maximum water level	WA		0	0	12	100
b - Minimum water level	WA		0	0	0	2
c - Average annual water level	WA		0	0	3	16
5.Wind pressure						
a - Maximum water level	WS		73	934	14	183
b - Minimum water level	WS		135	1539	29	331
c - Average annual water level	WS		135	1539	29	331
6.Vessel collision force						
a - Longitudinal direction	CV		173	1794		
b - Transverse direction	CV				404	4111
7.Vehicular collision force						
a - Longitudinal direction	CT		0	0		
b - Transverse direction	CT				0	0

Load Factors and Load Combinations

Loads	Sign	Str1a 2	Str1b 3	Str2a 4	Str2b 5	Str3a 6	Str3b 7
Superstructure Loads							
1.Stage1 - Dead Load	DC	1.25	0.90	1.25	0.90	1.25	0.90
2.Stage2 - Pave.+Mis.	DW	1.50	0.65	1.50	0.65	1.50	0.65
3.Live Load	LL	1.75	1.75	-	-	1.35	1.35
4.Pedestrian	LL	1.75	1.75	-	-	1.35	1.35
5.Centrifugal force	CE	1.75	1.75	-	-	1.35	1.35
6.Braking force	BR	1.75	1.75	-	-	1.35	1.35
7.Uniform temperature	TU	0.50	0.50	0.50	0.50	0.50	0.50
8.Creep and Shrinkage	CR&SH	8.00	0.50	0.50	0.50	0.50	5.00
9.Wind pressure on superst.	WS	-	-	1.40	1.40	0.40	0.40
10.Wind pressure on vehicles	WL	-	-	-	-	1.00	1.00
11.Earthequake							
a - Longitudinal direction	EQL	-	-	-	-	-	-
b - Transverse direction	EQT	-	-	-	-	-	-
Substructure Loads							
1.Pier selfweight	DC	1.25	0.90	1.25	0.90	1.25	0.90
2.Soil on pile cap	EV	1.35	0.90	1.35	0.90	1.35	0.90
3.Bouyancy on pier							
a - Maximum water level	WA		1.00		1.00		1.00
b - Minimum water level	WA	1.00		1.00		1.00	
c - Average annual WL	WA						
4.Stream pressure							
a - Maximum water level	WA		1.00		1.00		1.00
b - Minimum water level	WA	1.00		1.00		1.00	
c - Average annual WL	WA						
5.Wind pressure							
a - Maximum water level	WS	-	-		1.40		0.40
b - Minimum water level	WS	-	-	1.40		0.40	
c - Average annual WL	WS	-	-				
6.Vessel collision force							
a - Longitudinal direction	CV	-	-	-	-	-	-
b - Transverse direction	CV	-	-	-	-	-	-
7.Vehicular collision force							
a - Longitudinal direction	CT	-	-	-	-	-	-
b - Transverse direction	CT	-	-	-	-	-	-

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

Load Factors and Load Combinations

Loads	Sign	Ext2a	Ext2b	Ext2c	Ext2d		

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

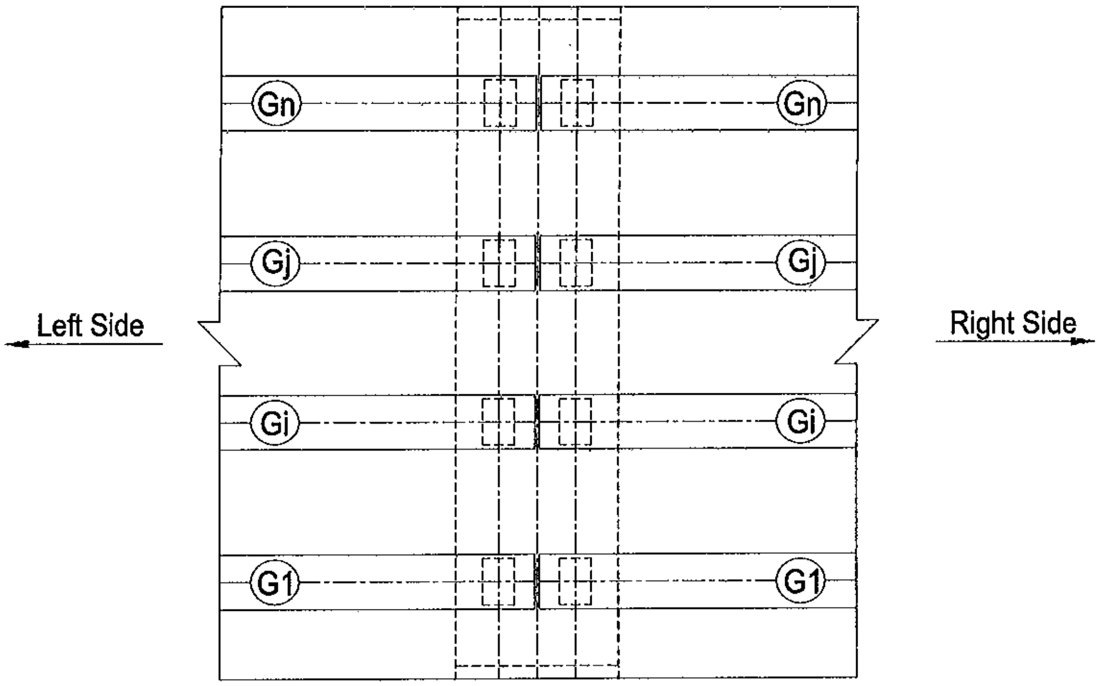
II. LOAD COMBINATIONS RESULT

No	Combinations		Sign	F_V (kN)	Longitudinal F_{HX} (kN)	Longitudinal My (kN•m)	Transvesal F_{HY} (kN)	Transvesal Mx (kN•m)
1	Strength 1a		Str1a	13489	185	3658	0	7243
2	Strength 1b		Str1b	9282	182	3615	12	7330
3	Strength 2a		Str2a	11414	259	2717	318	4117
4	Strength 2b		Str2b	7207	172	2042	309	4040
5	Strength 3a		Str3a	13015	232	3851	127	7336
6	Strength 3b		Str3b	8807	210	3684	133	7376
7	Service 1		Ser1	9954	178	2941	105	5593
8	Extreme 1a	EQL	Ext1a	12007	52	1031	3	2079
9	Extreme 1b	EQL	Ext1b	8091	52	1031	3	2079
10	Extreme 1c	EQT	Ext1c	12007	52	1031	3	2079
11	Extreme 1d	EQT	Ext1d	8091	52	1031	3	2079
12	Extreme 2a	CTL	Ext2a	12007	52	1031	3	2079
13	Extreme 2b	CTL	Ext2b	8091	52	1031	3	2079
14	Extreme 2c	CTT	Ext2c	12007	52	1031	3	2079
15	Extreme 2d	CTT	Ext2d	8091	52	1031	3	2079

Load Combinations at Bottom of PileCap

No	Combinations		Sign	F_V (kN)	Longitudinal F_{HX} (kN)	Longitudinal My (kN•m)	Transvesal F_{HY} (kN)	Transvesal Mx (kN•m)
1	Strength 1a		Str1a	18454	185	4029	0	7244
2	Strength 1b		Str1b	12461	182	3979	12	7342
3	Strength 2a		Str2a	16378	259	3234	318	4754
4	Strength 2b		Str2b	10385	172	2387	309	4645
5	Strength 3a		Str3a	17979	232	4315	127	7590
6	Strength 3b		Str3b	11986	210	4103	133	7629
7	Service 1		Ser1	13574	178	3297	105	5803
8	Extreme 1a	EQL	Ext1a	16971	52	1135	3	2086
9	Extreme 1b	EQL	Ext1b	11270	52	1135	3	2086
10	Extreme 1c	EQT	Ext1c	16971	52	1135	3	2086
11	Extreme 1d	EQT	Ext1d	11270	52	1135	3	2086
12	Extreme 2a	CTL	Ext2a	16971	52	1135	3	2086
13	Extreme 2b	CTL	Ext2b	13857	52	1135	3	2086
14	Extreme 2c	CTT	Ext2c	16971	52	1135	3	2086
15	Extreme 2d	CTT	Ext2d	11270	52	1135	3	2086

c.PIER CAP ANALYSIS



1.DEAD LOAD

Stage1 - Dead load

Load: ***Girders***	***G1***	***G2***	***G3***	***G4***	***G5***	***G6***	***G7***
Left side Reactions (kN)	192.2	192.2	192.2	192.2	192.2	192.2	
Right side Reactions (kN)	192.2	192.2	192.2	192.2	192.2	192.2	
Total reactions both side (kN)	***384.4***	***384.4***	***384.4***	***384.4***	***384.4***	***384.4***	***0.0***

Load: ***Diaphragm***	***G1***	***G2***	***G3***	***G4***	***G5***	***G6***	***G7***
Left side Reactions (kN)	28.3	28.3	28.3	28.3	28.3	28.3	
Right side Reactions (kN)	28.3	28.3	28.3	28.3	28.3	28.3	
Total reactions both side (kN)	***56.6***	***56.6***	***56.6***	***56.6***	***56.6***	***56.6***	***0.0***

Load: ***Precast plank***	***G1***	***G2***	***G3***	***G4***	***G5***	***G6***	***G7***
Left side Reactions (kN)	29.9	29.9	29.9	29.9	29.9	29.9	
Right side Reactions (kN)	29.9	29.9	29.9	29.9	29.9	29.9	
Total reactions both side (kN)	***59.7***	***59.7***	***59.7***	***59.7***	***59.7***	***59.7***	***0.0***

Load: ***DeckSlab***	***G1***	***G2***	***G3***	***G4***	***G5***	***G6***	***G7***
Left side Reactions (kN)	170.4	170.4	170.4	170.4	170.4	170.4	
Right side Reactions (kN)	170.4	170.4	170.4	170.4	170.4	170.4	
Total reactions both side (kN)	***340.8***	***340.8***	***340.8***	***340.8***	***340.8***	***340.8***	

Load: ***Stage1 (DC)***	***G1***	***G2***	***G3***	***G4***	***G5***	***G6***	***G7***
Total reactions both side (kN)	***841.5***	***841.5***	***841.5***	***841.5***	***841.5***	***841.5***	

Stage2 - Dead load

Load: ***Pavement***	***G1***	***G2***	***G3***	***G4***	***G5***	***G6***	***G7***
Left side Reactions (kN)	56.9	56.9	56.9	56.9	56.9	56.9	

Right side Reactions (kN)	56.9	56.9	56.9	56.9	56.9	56.9	
Total reactions both side (kN)	***113.8***	***113.8***	***113.8***	***113.8***	***113.8***	***113.8***	

Load: ***Parapet+railing***	***G1***	***G2***	***G3***	***G4***	***G5***	***G6***	***G7***
Left side Reactions (kN)	195.5	0.0	0.0	0.0	0.0	195.5	
Right side Reactions (kN)	195.5	0.0	0.0	0.0	0.0	195.5	
Total reactions both side (kN)	***391.1***	***0.0***	***0.0***	***0.0***	***0.0***	***391.1***	

Load: ***Lighting post.+mis***	***G1***	***G2***	***G3***	***G4***	***G5***	***G6***	***G7***
Left side Reactions (kN)	6.6	6.6	6.6	6.6	6.6	6.6	
Right side Reactions (kN)	6.6	6.6	6.6	6.6	6.6	6.6	
Total reactions both side (kN)	***13.2***	***13.2***	***13.2***	***13.2***	***13.2***	***13.2***	

Load: ***Stage2 (DC)***	***G1***	***G2***	***G3***	***G4***	***G5***	***G6***	***G7***
Total reactions both side (kN)	***518.0***	***127.0***	***127.0***	***127.0***	***127.0***	***518.0***	

2. LIVE LOAD

Live load reactions is calculated by level rule

Summary of Live load Reactions:

8 5

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

Live load (kN)	Factor	***G1***	***G2***	***G3***	***G4***	***G5***	***G6***
Truck + Lane load		468.5	251.3	0.0	0.0	0.0	0.0
Tandem + Lane load		389.3	208.8	0.0	0.0	0.0	0.0
0.9*(2Truck+Lane)	0.9	463.1	248.4	0.0	0.0	0.0	0.0

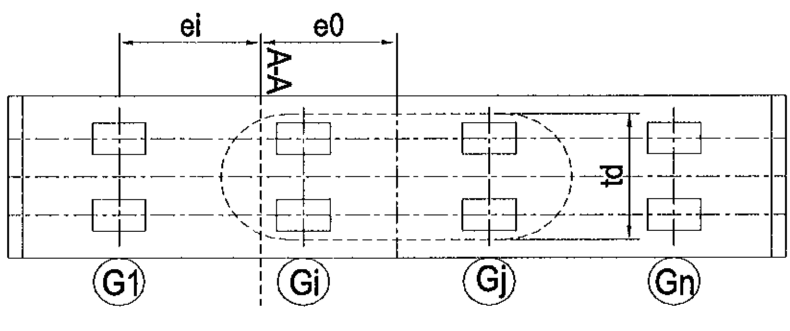
Reaction = [(1+IM)*Vehicle + Laneload]*n*m

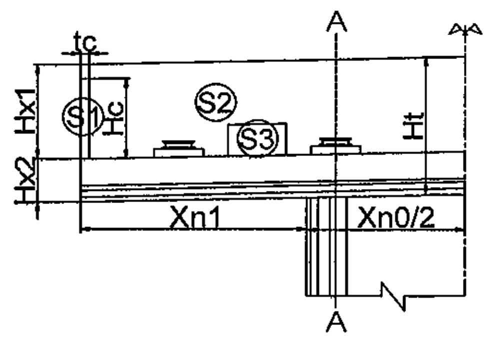
2 Loaded Lanes n = 2.0 m = 1.00 1+IM = 1.25

Live load (kN)	Factor	***G1***	***G2***	***G3***	***G4***	***G5***	***G6***
Truck + Lane load		390.4	433.1	367.1	9.0	9.0	0.0
Tandem + Lane load		324.4	359.9	305.1	7.5	7.5	0.0
0.9*(2Truck+Lane)	0.9	385.9	428.1	362.9	8.9	8.9	0.0

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

3. PIER CAP DESIGN





Cantilever section (A-A)

Distance from centerline of pier to section A-A e0 = 2.28 m

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

Dead load of substructure

Notation	Dimensions	Value(m)	Notation	Dimensions	Value(m)
Hx1	Haunch 1 height	1.52	Hc	Curtain wall height	1.50
Hx2	Haunch 2 height	0.80	tc	Curtain wall thickness	0.15
Xn1	Haunch width	3.49	xd	Pier cap width	1.40
Xn0	Bottom of pier cap width	5.50		Xn0/2 - e0	0.47
γc	Concrete unit weight (kN/m3)	24.50	e0	Dist. from CL of pier to sec.A-A	2.28

Item	**Volume**	**Section G1**			**Volume**	**Section A-A**		
		Fv	**Arm.$_{Fv}$**	**M$_x$**		**Fv**	**Arm.$_{Fv}$**	**M$_x$**
Component	(m3)	(kN)	(m)	(kN•m)	(m3)	(kN)	(m)	(kN•m)
S1	0.50	12.1	-0.01	-0.1	0.50	12.1	3.88	47.1
S2	6.35	155.6	0.61	94.3	28.26	692.3	1.98	1369.6
S3					0.68	16.6	1.56	26.0
Total at section G1		***167.7***		***94.3***				
Total at section A-A						***721.0***		***1442.7***

Load components at section bearing G1

Item	**Vertical**	**Torsion Moment**			**Bending Moment**		
	F$_V$	**F$_{HX}$**	**Arm.$_{HX}$**	**M$_y$**	**F$_{HY}$**	**Arm.$_{HY}$**	**M$_x$**
	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Pier Selfweight	167.7						94.3
DC stage1	842					0.0	0.0
DW stage2	518.0					0.0	0.0
Live Load	463.1			247.9		0.0	0.0
Pedestrian	0.0					0.0	0.0

Load components at section A-A

Item	**Vertical**	**Torsion Moment**			**Bending Moment**		
	F$_V$	**F$_{HX}$**	**Arm.$_{HX}$**	**M$_y$**	**F$_{HY}$**	**Arm.$_{HY}$**	**M$_x$**
	(kN)	(kN)	(m)	(kN•m)	(kN)	(m)	(kN•m)
Pier Selfweight	721.0						1442.7
DC stage1	1683.0					3.89	4467.1
DW stage2	645.0					3.89	2194.6
Live Load	711.5			247.9		3.89	2153.4
Pedestrian	0.0					3.89	0.0

Load factors and Load combinations

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

Load Combinations

Section	**Comb.**	**Vertical**	**Bending**	**Torsion**
		Fv	**Mx**	**My**
G1	Ser1	*1990*	*94*	*248*
	Str1a	*2849*	*118*	*434*
A-A	Ser1	*3761*	*10258*	*248*
	Str1a	*5218*	*14448*	*434*

d.COLUMN DESIGN

I. COLUMN DATA

1.Load Combinations at Bottom of Pier Column

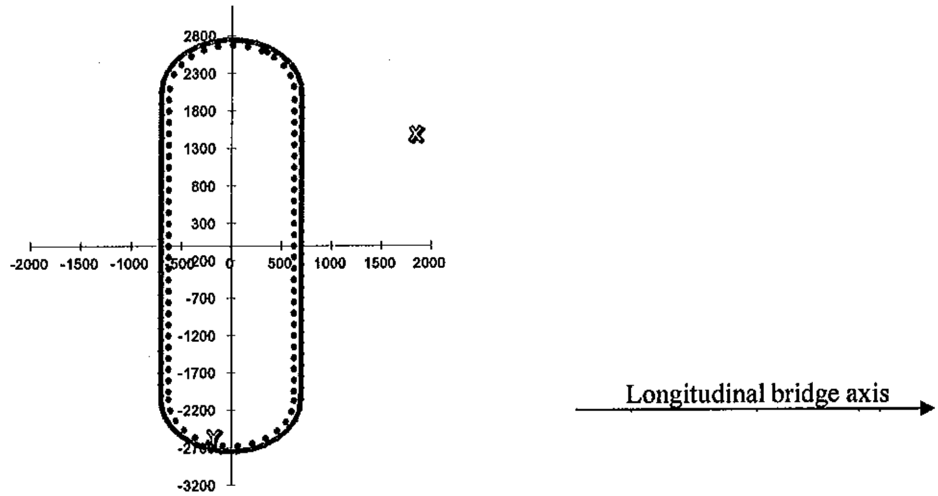
No	Combinations		Sign	F_V (kN)	Longitudinal F_{HX} (kN)	Longitudinal My (kN•m)	Transvesal F_{HY} (kN)	Transvesal Mx (kN•m)
1	Strength 1a		Str1a	13489	185	3658	0	7243
2	Strength 1b		Str1b	9282	182	3615	12	7330
3	Strength 2a		Str2a	11414	259	2717	318	4117
4	Strength 2b		Str2b	7207	172	2042	309	4040
5	Strength 3a		Str3a	13015	232	3851	127	7336
6	Strength 3b		Str3b	8807	210	3684	133	7376
7	Service 1		Ser1	9954	178	2941	105	5593
8	Extreme 1a	EQL	Ext1a	12007	52	1031	3	2079
9	Extreme 1b	EQL	Ext1b	8091	52	1031	3	2079
10	Extreme 1c	EQT	Ext1c	12007	52	1031	3	2079
11	Extreme 1d	EQT	Ext1d	8091	52	1031	3	2079
12	Extreme 2a	CTL	Ext2a	12007	52	1031	3	2079
13	Extreme 2b	CTL	Ext2b	8091	52	1031	3	2079
14	Extreme 2c	CTT	Ext2c	12007	52	1031	3	2079
15	Extreme 2d	CTT	Ext2d	8091	52	1031	3	2079

2. Pier Column Material

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

3. Pier Column Section

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



4. Slenderness Effect

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

Transverse direction: Fixed at bottom; translation free, rotation free at top	Kt	2.10	
Longitudinal direction: Fixed at bottom; translation free, rotation free at top	Kl	2.10	
Unsupported length from top to bottom of column	Lu	11.94	m
Slenderness ratio: if K.Lu / r > 22 than considered	Kt.Lu/rx	16.7	no
	Kl.Lu/ry	63.7	yes
Moment inertia of longitudinal reinforcements	Is	0	m4
Ratio of max factored Per. load moment to max factored total load moment	βd	0	

P - Δ analysis

****Longitudinal Direction**

P - Δ moment dertermination procedure:

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

Combination	Fv	My	Initial					
	(kN)	(kNm)	Fx (kN)	Δxg (m)	Fcr (kN)	Δxcr (m)	M P-Δ (kNm)	ΔFx (kN)
Strength 1a	13489	3658	306	0.006	2.5	0.014	188	15.7
Strength 1b	9282	3615	303	0.006	2.5	0.014	128	10.7
Strength 2a	11414	2717	228	0.004	2.5	0.010	118	9.9
Strength 2b	7207	2042	171	0.003	2.5	0.008	56	4.7
Strength 3a	13015	3851	323	0.006	2.5	0.015	191	16.0
Strength 3b	8807	3684	309	0.006	2.5	0.014	124	10.4
Service 1	9954	2941	246	0.004	2.5	0.011	112	9.3
Extreme 1a	12007	1031	86	0.002	2.5	0.004	47	4.0
Extreme 1b	8091	1031	8	0.000	2.5	0.000	3	5.0
Extreme 1c	12007	1031	86	0.002	2.5	0.004	47	4.0
Extreme 1d	8091	1031	86	0.002	2.5	0.004	32	2.7
Extreme 2a	12007	1031	86	0.002	2.5	0.004	47	4.0
Extreme 2b	8091	1031	86	0.002	2.5	0.004	32	2.7
Extreme 2c	12007	1031	86	0.002	2.5	0.004	47	4.0
Extreme 2d	8091	1031	86	0.002	2.5	0.004	32	2.7

Combination	Fv	My	1st Trial					
	(kN)	(kNm)	Fx (kN)	Δxg (m)	Fcr (kN)	Δxcr (m)	M P-Δ (kNm)	ΔFx (kN)
Strength 1a	13489	3658	322	0.006	2.5	0.015	198	16.6
Strength 1b	9282	3615	314	0.006	2.5	0.014	132	11.1
Strength 2a	11414	2717	237	0.004	2.5	0.011	123	10.3
Strength 2b	7207	2042	176	0.003	2.5	0.008	58	4.8
Strength 3a	13015	3851	339	0.006	2.5	0.015	200	16.8
Strength 3b	8807	3684	319	0.006	2.5	0.015	128	10.7
Service 1	9954	2941	256	0.005	2.5	0.012	116	9.7
Extreme 1a	12007	1031	90	0.002	2.5	0.004	49	4.1
Extreme 1b	8091	1031	13	0.000	2.5	0.001	5	0.4
Extreme 1c	12007	1031	90	0.002	2.5	0.004	49	4.1
Extreme 1d	8091	1031	89	0.002	2.5	0.004	33	2.7
Extreme 2a	12007	1031	90	0.002	2.5	0.004	49	4.1
Extreme 2b	8091	1031	89	0.002	2.5	0.004	33	2.7
Extreme 2c	12007	1031	90	0.002	2.5	0.004	49	4.1
Extreme 2d	8091	1031	89	0.002	2.5	0.004	33	2.7

Combination	Fv	My	2nd Trial					
	(kN)	(kNm)	Fx (kN)	Δxg (m)	Fcr (kN)	Δxcr (m)	M P-Δ (kNm)	ΔFx (kN)
Strength 1a	13489	3658	323	0.006	2.5	0.015	198	16.6
Strength 1b	9282	3615	314	0.006	2.5	0.014	132	11.1
Strength 2a	11414	2717	238	0.004	2.5	0.011	123	10.3
Strength 2b	7207	2042	176	0.003	2.5	0.008	58	4.8
Strength 3a	13015	3851	339	0.006	2.5	0.015	201	16.8
Strength 3b	8807	3684	319	0.006	2.5	0.015	128	10.7
Service 1	9954	2941	256	0.005	2.5	0.012	116	9.7
Extreme 1a	12007	1031	91	0.002	2.5	0.004	49	4.1
Extreme 1b	8091	1031	8	0.000	2.5	0.000	3	0.3
Extreme 1c	12007	1031	91	0.002	2.5	0.004	49	4.1
Extreme 1d	8091	1031	89	0.002	2.5	0.004	33	2.7
Extreme 2a	12007	1031	91	0.002	2.5	0.004	49	4.1
Extreme 2b	8091	1031	89	0.002	2.5	0.004	33	2.7
Extreme 2c	12007	1031	91	0.002	2.5	0.004	49	4.1
Extreme 2d	8091	1031	89	0.002	2.5	0.004	33	2.7

****Transverse Direction**

Combination	Fv	Mx	Initial					
			Fx	Δxg	Fcr	Δxcr	M P-Δ	ΔFx
	(kN)	(kNm)	(kN)	(m)	(kN)	(m)	(kNm)	(kN)
Strength 1a	13489	7243	607	0.001	2.5	0.002	25	2.1
Strength 1b	9282	7330	614	0.001	2.5	0.002	18	1.5
Strength 2a	11414	4117	345	0.000	2.5	0.001	12	1.0
Strength 2b	7207	4040	338	0.000	2.5	0.001	8	0.6
Strength 3a	13015	7336	614	0.001	2.5	0.002	25	2.1
Strength 3b	8807	7376	618	0.001	2.5	0.002	17	1.4
Service 1	9954	5593	469	0.001	2.5	0.001	14	1.2
Extreme 1a	12007	2079	174	0.000	2.5	0.001	6	0.5
Extreme 1b	8091	2079	174	0.000	2.5	0.001	4	0.4
Extreme 1c	12007	2079	174	0.000	2.5	0.001	6	0.5
Extreme 1d	8091	2079	174	0.000	2.5	0.001	4	0.4
Extreme 2a	12007	2079	174	0.000	2.5	0.001	6	0.5
Extreme 2b	8091	2079	174	0.000	2.5	0.001	4	0.4
Extreme 2c	12007	2079	174	0.000	2.5	0.001	6	0.5
Extreme 2d	8091	2079	174	0.000	2.5	0.001	4	0.4

Combination	Fv	Mx	1st Trial					
			Fx	Δxg	Fcr	Δxcr	M P-Δ	ΔFx
	(kN)	(kNm)	(kN)	(m)	(kN)	(m)	(kNm)	(kN)
Strength 1a	13489	7243	609	0.001	2.5	0.002	25	2.1
Strength 1b	9282	7330	616	0.001	2.5	0.002	18	1.5
Strength 2a	11414	4117	346	0.000	2.5	0.001	12	1.0
Strength 2b	7207	4040	339	0.000	2.5	0.001	8	0.6
Strength 3a	13015	7336	617	0.001	2.5	0.002	25	2.1
Strength 3b	8807	7376	619	0.001	2.5	0.002	17	1.4
Service 1	9954	5593	470	0.001	2.5	0.001	15	1.2
Extreme 1a	12007	2079	175	0.000	2.5	0.001	7	0.5
Extreme 1b	8091	2079	174	0.000	2.5	0.001	4	0.4
Extreme 1c	12007	2079	175	0.000	2.5	0.001	7	0.5
Extreme 1d	8091	2079	174	0.000	2.5	0.001	4	0.4
Extreme 2a	12007	2079	175	0.000	2.5	0.001	7	0.5
Extreme 2b	8091	2079	174	0.000	2.5	0.001	4	0.4
Extreme 2c	12007	2079	175	0.000	2.5	0.001	7	0.5
Extreme 2d	8091	2079	174	0.000	2.5	0.001	4	0.4

Combination	Fv	Mx	2nd Trial					
			Fx	Δxg	Fcr	Δxcr	M P-Δ	ΔFx
	(kN)	(kNm)	(kN)	(m)	(kN)	(m)	(kNm)	(kN)
Strength 1a	13489	7243	609	0.001	2.5	0.002	25	2.1
Strength 1b	9282	7330	616	0.001	2.5	0.002	18	1.5
Strength 2a	11414	4117	346	0.000	2.5	0.001	12	1.0
Strength 2b	7207	4040	339	0.000	2.5	0.001	8	0.6
Strength 3a	13015	7336	617	0.001	2.5	0.002	25	2.1
Strength 3b	8807	7376	619	0.001	2.5	0.002	17	1.4
Service 1	9954	5593	470	0.001	2.5	0.001	15	1.2
Extreme 1a	12007	2079	175	0.000	2.5	0.001	7	0.5
Extreme 1b	8091	2079	174	0.000	2.5	0.001	4	0.4
Extreme 1c	12007	2079	175	0.000	2.5	0.001	7	0.5
Extreme 1d	8091	2079	174	0.000	2.5	0.001	4	0.4
Extreme 2a	12007	2079	175	0.000	2.5	0.001	7	0.5
Extreme 2b	8091	2079	174	0.000	2.5	0.001	4	0.4
Extreme 2c	12007	2079	175	0.000	2.5	0.001	7	0.5
Extreme 2d	8091	2079	174	0.000	2.5	0.001	4	0.4

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

Combination	Fv Vert. (kN)	Mx Trans. (kNm)	Mx P-Δ (kNm)	Mx Total (kNm)	My Long. (kNm)	My P-Δ (kNm)	My Total (kNm)
Strength 1a	13489	7243	25	7269	3658	198	3857
Strength 1b	9282	7330	18	7348	3615	132	3748
Strength 2a	11414	4117	12	4129	2717	123	2840
Strength 2b	7207	4040	8	4048	2042	58	2100
Strength 3a	13015	7336	25	7361	3851	201	4052
Strength 3b	8807	7376	17	7393	3684	128	3812
Service 1	9954	5593	15	5608	2941	116	3057
Extreme 1a	12007	2079	7	2085	1031	49	1081
Extreme 1b	8091	2079	4	2083	1031	3	1034
Extreme 1c	12007	2079	7	2085	1031	49	1081
Extreme 1d	8091	2079	4	2083	1031	33	1064
Extreme 2a	12007	2079	7	2085	1031	49	1081
Extreme 2b	8091	2079	4	2083	1031	33	1064
Extreme 2c	12007	2079	7	2085	1031	49	1081
Extreme 2d	8091	2079	4	2083	1031	33	1064

II. PIER COLUMN DESIGN

1. Limit of Reinforcement

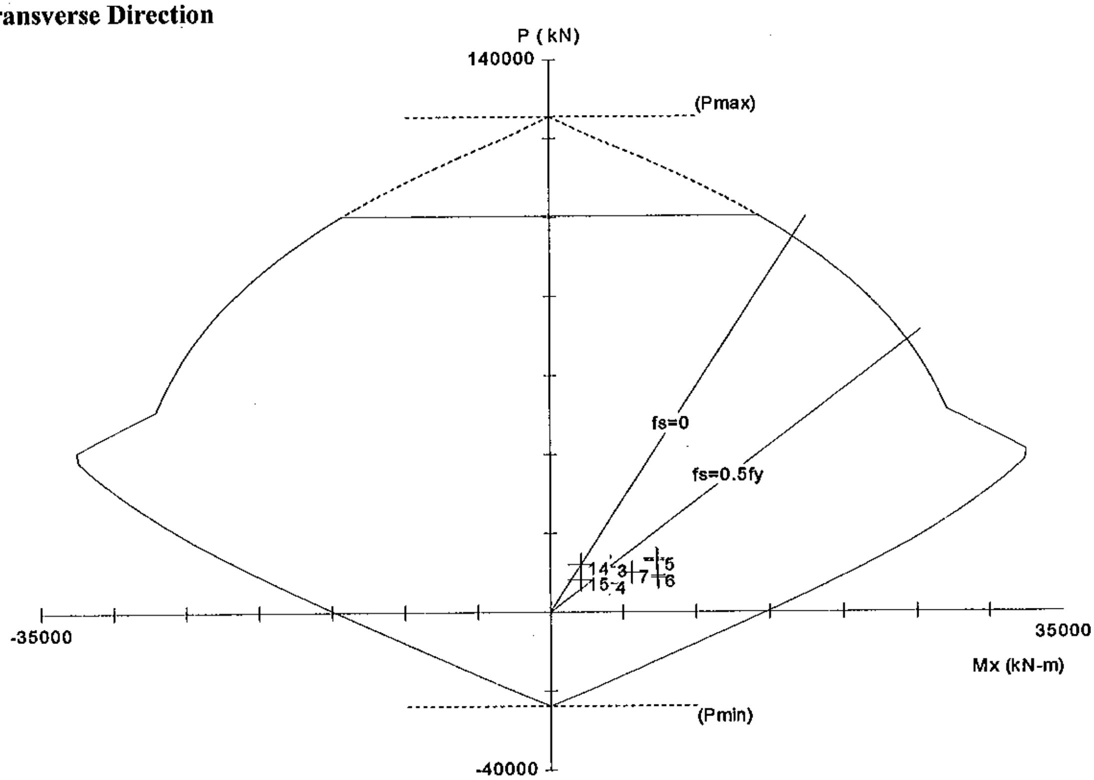
S.5.7.4.2

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

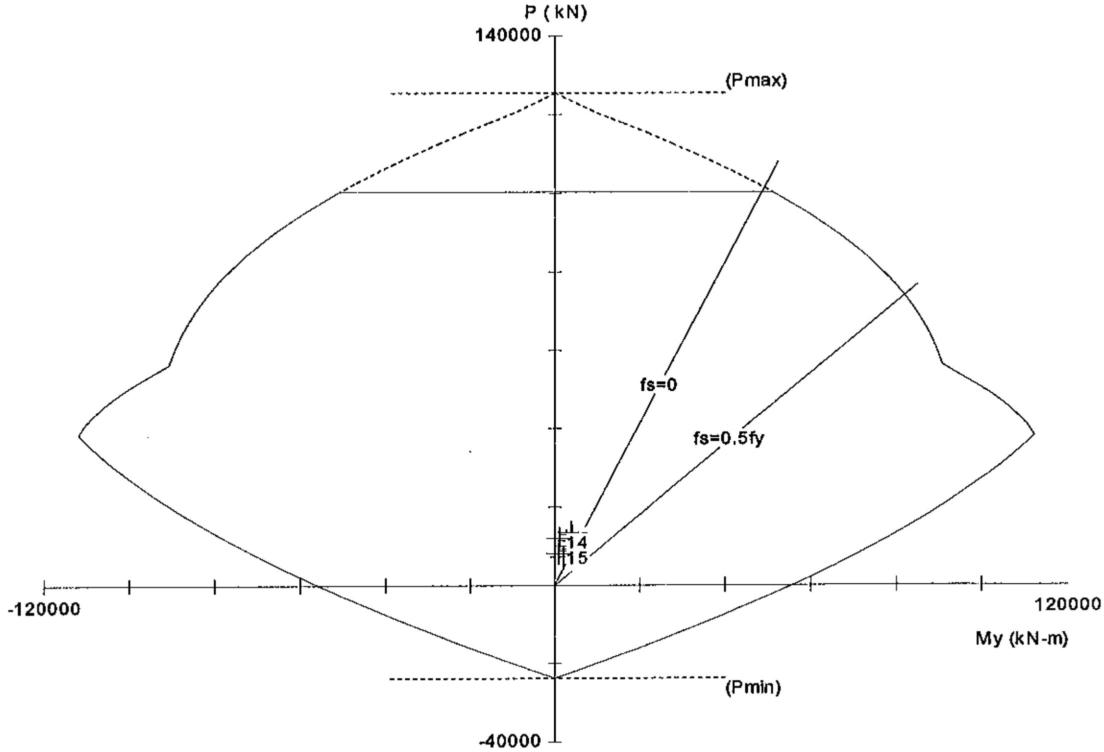
2. Interation diagram M-P

Using Pca-Column software

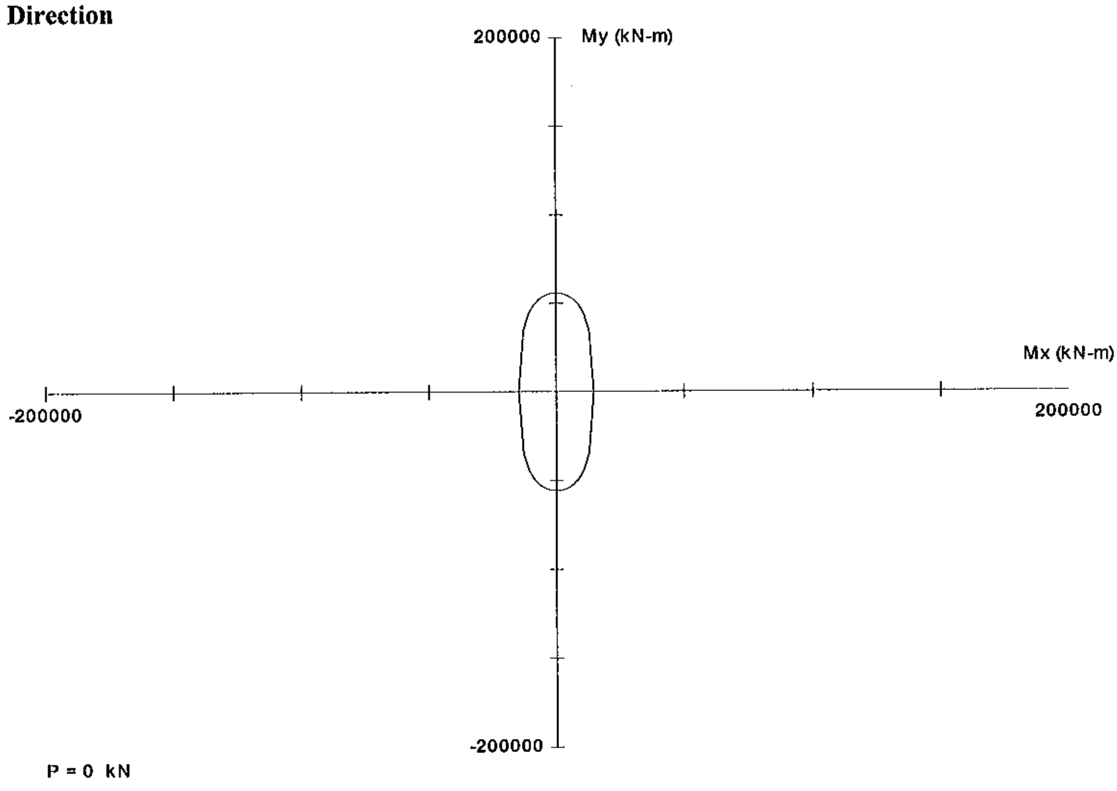
****In Transverse Direction**



****In Longitudinal Direction**



**In Both Direction



3. Column Ties

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

Bridge is in seismic zone	Sz	1	
Area of concrete core measured out-to-out of ties	Ac	6.719	m2
Tie diameter	Dtie	16	mm2
Cross section area of 1 tie	As-tr	0.0002	m2
Spacing of hoops	s	150	mm
Length of reinfocement tie in 1 hoop	Ltie	19.90	m
Ratio of ties reinf. in one hoop/ volume of conc. core one pitch spacing			
ρs = As-tr . Ltie / (Ac * spacing)	ρs	0.0040	
Ratio of spiral reinf. To total volume of concrete core shall satisfy			S.5.7.4.6
ρs >= 0.45 . (Ag/Ac - 1). f'c/ fy = Req1	Req1	0.0028	N/A
			S.5.10.11.3

Transverse Reinforcement for Confinement at Plastic Hinges		S.5.10.11.4.1.d	
For a circular column "1:applied","2:Not applied"		2	
ρs >= 0.12 . f'c/ fy = Req2	Req2	0.0090	**N/A**
For a rectangular column			
Rectangular hoop reinforcement shall satisfy			
Either Ash >= 0.30 . s.hc.f'c/ fy. [Ag/Ac -1] = Req1			
or Ash >= 0.12 . s.hc.f'c/ fy = Req2			
In longitudinal direction "1:applied","2:Not applied"		2	
Number of cross tie	nt_x	8	ties
Total cross-sectional area of tie reinf.	Ash_x	0.0016	m2
Core dimension of tied column	hc_x	1.30	m
Rectangular hoop reinforcement shall satisfy	Req1_x	0.0004	m2
	Req2_x	0.0018	m2
	Conclude		**N/A**
In transverse direction			
Number of cross tie	nt_y	4	ties
Total cross-sectional area of tie reinf.	Ash_y	0.0008	m2
Core dimension of tied column	hc_y	5.00	m
Rectangular hoop reinforcement shall satisfy	Req1_y	0.0014	m2
	Req2_y	0.0068	m2
	Conclude		**N/A**
Spacing of Transverse Reinforcement for Confinement		S.5.10.11.4.1.e	
Transverse reinforcement for confinement shall be:			
* Provided at the top and bottom of the column over a length not less than the greatest of the			
maximum cross-sectional column dimensions,one-sixth of the clear height of the column, or 450 mm;			
Maximum cross-sectional column dimensions	L1	5.50	m
1/6 of clear height of column	L2	1.60	m
or 450mm	L3	0.45	m
Chosen value: L = max(L1,L2,L3)	L	5.50	m
* Spaced not to exceed one-quarter of the minimum member dimension or 100 mm center-to-center.			
	Spacing	0.10	m
Column connections		S.5.10.11.4.3	
* Development length for all longitudinal steel shall be 1.25 times that required in S.5.11			
* Column transverse reinforcement, as specified in Article 5.10.11.4.1d, shall be continued for a distance			
not less than one-half the maximum column dimension or 380 mm from the face of the column connection into			
the adjoining member.			
1/2 maximum column dimension	L4	2.75	m
or 380mm	L5	0.38	m
Chosen value: Le = max(L4,L5)	Le	2.75	m

4. Shear Design

Direction		Long.- X	Trans.-Y	Unit
Shear resistance factors	ϕv	1.0	1.0	
Factored shear force in longitudinal	Vu	52	3	kN
Required shear capacity Vn = Vu/ ϕv	Vn	52	3	kN
Determine concrete shear capacity				
Minimum shear reinforcement will provided in cross section				
Therefore	β	2.0	2.0	
	θ	45.0	45.0	
Cross section equivalent height	h	1.40	5.20	m
width	b	5.20	1.40	m
d = h - cover - d1x	d	1.31	5.11	m
dv = max(0.72*h; 0.9*d)	dv	1.18	4.60	m
Vc = min(0.083 . β . sqrt(f'c) . bv. dv, 0.25.f'c.bv.dv)	Vc	5591	5858	kN
Difference between required shear capacity and the capacity provided by concrete				
is the minimum required capacity for shear reinforcements				
Vs = Vn - Vc	Vs	0	0	kN
In this case Vc > Vn so shear reinforcement is no need				

Stirrup diameter	Ds	16	16	
Number of stirrup legs / cross section	ns	6	2	
Shear legs area	Av	0.0012	0.0004	m2
Angle of inclination of shear reinf. to long. axis	α	90	90	deg
Vs = Av. fy. dv. (cot θ + cot α). sin α / s	s <=	-	-	m
Stirrup spacing used	s	0.10	0.10	m
Check minimum shear reinforcement requirement		**OK**	**OK**	
Av >= 0.083. sqrt(f'c). bv. s/ fy = Req	Req	0.0006	0.0002	m2
Check maximum shear reinforcement spacing requirement		**OK**	**OK**	
F = 0.1*f'c*bv*dv	F	18447	19329	kN
If Vu < 0.1*f'c*bv*dv then Smax=0.8*dv <= 600mm				
If Vu > 0.1*f'c*bv*dv then Smax=0.4*dv <= 300mm	Smax	0.60	0.60	m

Interface shear transfer S.5.8.4

Area of concrete engaged in shear transfer	Acv	7.279	m2
Area of shear reinforcement crossing the shear plane	Avf	0.066	m2
For concrete placed against clean, hardened concrete with surface roughened			
Cohesion factor specified in Article 5.8.4.2	c	0.7	MPa
Friction factor	μ	1	
For normal density concrete	λ	1	
Nominal shear resistance of the interface plane shall be taken as			
Vn = c. Acv + μ. Avf. fy	Vn	31368	kN
Vn <= 0.2 . f'c . Acv	Vn <=	43676	kN
Vn <= 5.5 . Acv . (1MPa)	Vn <=	40037	kN
Norminal shear resistance	Vn	31368	kN
Factor for shear friction		1.0	
Factored shear resistance	Vr	31368	kN
Horizontal force at bottom of pier column	Vu	52	kN
	Conclude		**OK**

5. Control of cracking by distribution of reinforcement

Tensile stress in rebars should be satisfied equation: fs <= fsa = Z/[(dc.A)^1/3] and fs <= 0.6.fy

Direction		Long.- X	Trans.-Y	Unit
Existing condition for structrure	1,2 or 3	1	1	
Crack width parameter	Z	30000	30000	N/mm
Flexural moment	Ms	3057	5608	kNm
Axial thrust at service limit state	Ns	9954	9954	kN
Cross section equivalent height	h	1.40	5.20	m
width	b	5.20	1.40	m
Concrete thickness from tension fiber to tension reinf.	dc	0.05	0.05	m
Concrete thickness from compression fiber to tension reinf.	d	1.31	5.11	kN
Number of rebars	N	76	22	bars
Area of rebars	As	0.0609	0.0176	m2
Area of concrete assumed to participate with reinf.				
A = 2 . dc . b / N	A	0.0068	0.0064	m2
	fsa	429	439	MPa
	0.6fy	240	240	MPa
Min (fsa, 0.6fy) = fs1	fs1	240	240	MPa
e = Ms/Ns+d-h/2	e	0.92	3.08	m
e/d > 1.15	e/d	1.15	1.15	
j = 0.74 + 0.1(e/d) ≤ 0.9	j	0.86	0.86	
i = 1/(1-j.d/e)	i	3.90	3.90	
Stress in rebars: fs = (Ms+Ns(d-h/2))/(As.j.i.d)	fs	34	102	MPa
	Conclude	**OK**	**OK**	
Maximum width of crack: an = 0.076.β.fs.(dc.A)^(1/3)	an	0.030	0.088	mm
Where	β	0.167	0.167	

e.PILECAP DESIGN

I. PILECAP DATA

1.Load Combinations at Bottom of Pilecap

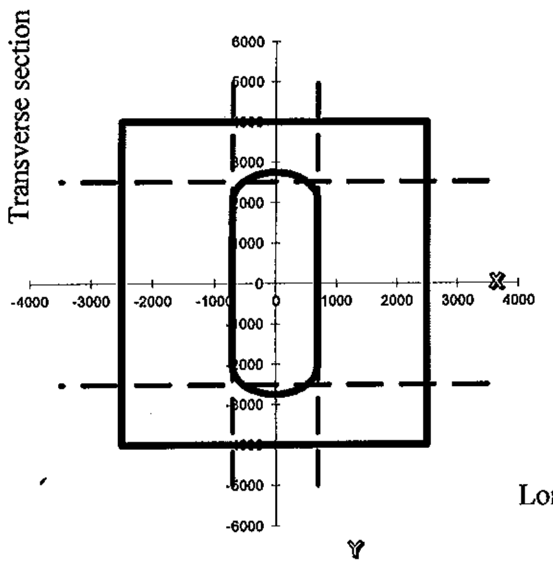
No	Combinations		Sign	F_V (kN)	Longitudinal F_{HX} (kN)	Longitudinal My (kN•m)	Transvesal F_{HY} (kN)	Transvesal Mx (kN•m)
1	Strength 1a		Str1a	18454	185	4029	0	7244
2	Strength 1b		Str1b	12461	182	3979	12	7342
3	Strength 2a		Str2a	16378	259	3234	318	4754
4	Strength 2b		Str2b	10385	172	2387	309	4645
5	Strength 3a		Str3a	17979	232	4315	127	7590
6	Strength 3b		Str3b	11986	210	4103	133	7629
7	Service 1		Ser1	13574	178	3297	105	5803
8	Extreme 1a	EQL	Ext1a	16971	52	1135	3	2086
9	Extreme 1b	EQL	Ext1b	11270	52	1135	3	2086
10	Extreme 1c	EQT	Ext1c	16971	52	1135	3	2086
11	Extreme 1d	EQT	Ext1d	11270	52	1135	3	2086
12	Extreme 2a	CTL	Ext2a	16971	52	1135	3	2086
13	Extreme 2b	CTL	Ext2b	13857	52	1135	3	2086
14	Extreme 2c	CTT	Ext2c	16971	52	1135	3	2086
15	Extreme 2d	CTT	Ext2d	11270	52	1135	3	2086

2. PileCap Material

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

3. Pilecap dimensions - piles arrangement

Pilecap dimensions	Longitudinal		Bd	5.00	m
	Transverse		Bn	8.00	m
	Height		Hb	2.00	m
Distance from edge to	transverse section		ex	1.80	m
	longitudinal section		ey	1.48	m
Soil on top of pilecap	"1:consider","0:not consider"	0	hsoil	0.00	m



Longitudinal bridge axis

Longitudinal section

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 dv from face of equivalent square column.			
Estimated distance between internal flexural force components dv, we may take dv = 0.9*de			
de = H - cover - dx1	de	1.81	m
	dv	1.63	m

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	4	0	1	1	0	kN
	Strength 1a	5	0	Strength 1a	2	0	kN
		6	0		3	0	
Selfweight of pilecap							
Load factor	1.25			1.25			
bouyancy	0		-84			-84	kN
Total shear force at section			**-84**			**-84**	

Longitudinal section	Upper side			Lower side			
Reactions of piles	Comb	Pile No	Reaction	Comb	Pile No	Reaction	
	1	3	0	1	1	0	kN
	Strength 1a	6	0	Strength 1a	4	0	kN
Selfweight of pilecap							
Load factor	1.25			1.25			
bouyancy	0		44			44	kN
Total shear force at section			**44**			**44**	

One-way Shear Design 8 5

Direction			Long.- X	Trans.-Y	Unit
Shear resistance factors		ϕv	0.9	0.9	
Factored shear force in longitudinal		Vu	44	-84	kN
Required shear capacity Vn = Vu/ ϕv		Vn	49	-94	kN
Determine concrete shear capacity					
Minimum shear reinforcement will provided in cross section					
Therefore		β	2.0	2.0	
		θ	45.0	45.0	
Cross section	height	h	2.00	2.00	m
	width	b	5.00	8.00	m
d = h - cover - d1x		d	1.81	1.81	m
dv = max(0.72*h; 0.9*d)		dv	1.63	1.63	m
Vc = min(0.083 . β . sqrt(f'c) . bv. dv, 0.25.f'c.bv.dv)		Vc	7422	11875	kN
Difference between required shear capacity and the capacity provided by concrete					
is the minimum required capacity for shear reinforcements					
Vs = Vn - Vc		Vs	0	0	kN
In this case Vc > Vn so shear reinforcement is no need					
Stirrup diameter		Ds	16	16	
Number of stirrup legs / cross section		ns	20	36	
Shear legs area		Av	0.0040	0.0073	m2
Angle of inclination of shear reinf. to long. axis		α	90	90	deg
Vs = Av. fy. dv. (cot θ + cot α). sin α / s		s <=	-	-	m
Stirrup spacing used		s	0.30	0.30	m
Check minimum shear reinforcement requirement			**OK**	**OK**	
Av >= 0.083. sqrt(f'c). bv. s/ fy = Req		Req	0.0017	0.0027	m2
Check maximum shear reinforcement spacing requirement			**OK**	**OK**	
F = 0.1*f'c*bv*dv		F	24489	39182	kN
If Vu < 0.1*f'c*bv*dv then Smax=0.8*dv <= 600mm					
If Vu > 0.1*f'c*bv*dv then Smax=0.4*dv <= 300mm		Smax	0.60	0.60	m

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	1.40	m
	Transverse axis	tn	5.50	m
Perimeter of two-way shear				
b0 = (td+tn)*2 + 4*dv		b0	18.28	m
Compressive strength of pilecap concrete		f'c	30	Mpa
Yield strength of rebar		fy	400	Mpa
Section with transverse reinforcement				
Norminal shear resistance shall be taken as				
Vn = Vc + Vs <= 0.504.sqrt(f'c). b0 . dv = Va				
Vc = 0.166 . sqrt(f'c). b0 . dv				
Vs = Av. fy . dv / s				
Shear resistance of concrete		Vc	27136	kN
Assumed stirrup diameter		Ds	16	mm
Number of stirrup legs / cross section		ns	0	
Shear legs area		Av	0.0000	m2
Stirrup spacing used		s	600	mm
Shear resistance of reinforcement		Vs	0	kN
		Va	82390	kN
		Vn	27136	kN
Maximum reaction at bottom of column		Vu	13489	kN
Resistance factor for shear		ϕv	0.9	
Factored shear resistance		ϕv*Vn	24423	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	f'c	30	Mpa
Yield strength of rebar	fy	400	Mpa
Section with transverse reinforcement			
Norminal shear resistance shall be taken as			
Vn = Vc + Vs <= 0.504.sqrt(f'c). b0 . dv = Va			
Vc = 0.166 . sqrt(f'c). b0 . dv			
Vs = Av. fy . dv / s			
Shear resistance of concrete	Vc	6038	kN
Assumed stirrup diameter	Ds	16	mm
Number of stirrup legs / cross section	ns	0	
Shear legs area	Av	0.0000	m2
Stirrup spacing used	s	300	mm
Shear resistance of reinforcement	Vs	0	kN
	Va	18332	kN
	Vn	6038	kN
Maximum reaction of conner pile	Vu	0	kN
Resistance factor for shear	ϕv	0.9	
Factored shear resistance	ϕv*Vn	5434	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 1a	(kN)	(kN)	(kNm)	(m)	(m)	(kN.m)
Pile number			1	0	0	0	1.0	0.48	0
			4	0	0	0	1.0	0.48	0
Distance - ey			1.48						
Self. of pilecap		1.25	1	-218		-162			-162
Soil on pilecap		1.35	1	0		0			0
Reaction from column									
				Q	H				M
Sum				**-218**	**0**				**-162**

Section	Long.	Comb.	7	N	Q3	M2	e0	e=ey-e0	M*
	Lower		Service 1	(kN)	(kN)	(kNm)	(m)	(m)	(kN.m)
Pile number			1	0	0	0	1.0	0.48	0
			4	0	0	0	1.0	0.48	0
			0						
Distance - ey			1.48						
Self. of pilecap		1.00	1	-218		-162			-162
Soil on pilecap		1.00	1	0		0			0
Reaction from column									
				Q	H				M
Sum				**-218**	**0**				**-162**

Section	Long.	Comb.	14	N	Q3	M2	e0	e=ey-e0	M*
	Lower		Extreme 2c	(kN)	(kN)	(kNm)	(m)	(m)	(kN.m)
Pile number			1	0	0	0	1.0	0.48	0
			4	0	0	0	1.0	0.48	0
			0						
Distance - ey			1.48						
Self. of pilecap		1.25	1	-218		-162			-162
Soil on pilecap		1.35	1	0		0			0
Reaction from column									
				Q	H				M
Sum				**-218**	**0**				**-162**

M* = N.e + M2 + Q3.Hb/2

Section	Long.	Comb.	1	N	Q3	M2	e0	e=ey-e0	M*
	Upper		Strength 1a	(kN)	(kN)	(kNm)	(m)	(m)	(kN.m)
Pile number			3	0	0	0	1.0	0.48	0
			6	0	0	0	1.0	0.48	0
Distance - ey			1.48						
Self. of pilecap		1.25	1	-218		-162			-162
Soil on pilecap		1.35	1	0		0			0
Reaction from column									
				Q	**H**				**M**
Sum				**-218**	**0**				**-162**

Section	Long.	Comb.	7	N	Q3	M2	e0	e=ey-e0	M*
	Upper		Service 1	(kN)	(kN)	(kNm)	(m)	(m)	(kN.m)
Pile number			3	0	0	0	1.0	0.48	0
			6	0	0	0	1.0	0.48	0
			0						
Distance - ey			1.48						
Self. of pilecap		1.00	1	-218		-162			-162
Soil on pilecap		1.00	1	0		0			0
Reaction from column									
				Q	**H**				**M**
Sum				**-218**	**0**				**-162**

Section	Long.	Comb.	8	N	Q3	M2	e0	e=ey-e0	M*
	Upper		Extreme 1a	(kN)	(kN)	(kNm)	(m)	(m)	(kN.m)
Pile number			3	0	0	0	1.0	0.48	0
			6	0	0	0	1.0	0.48	0
			0						
Distance - ey			1.48						
Self. of pilecap		1.25	1	-218		-162			-162
Soil on pilecap		1.35	1	0		0			0
Reaction from column									
				Q	**H**				**M**
Sum				**-218**	**0**				**-162**

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*
	Left		Strength 1a	(kN)	(kN)	(kNm)	(m)	(m)	(kN.m)
Pile number			4	0	0	0	1.0	0.80	0
			5	0	0	0	1.0	0.80	0
			6	0	0	0	1.0	0.80	0
Distance - ey			1.80						
Self. of pilecap		1.25	1	-349		-314			-314
Soil on pilecap		1.35	1	0		0			0
Reaction from column									
				Q	H				M
Sum				**-349**	**0**				**-314**

Section	Trans.	Comb.	7	N	Q2	M3	e0	e=ey-e0	M*
	Left		Service 1	(kN)	(kN)	(kNm)	(m)	(m)	(kN.m)
Pile number			4	0	0	0	1.0	0.80	0
			5	0	0	0	1.0	0.80	0
			6	0	0	0	1.0	0.80	
Distance - ey			1.80						
Self. of pilecap		1.00	1	-349		-314			-314
Soil on pilecap		1.00	1	0		0			0
Reaction from column									
				Q	H				M
Sum				**-349**	**0**				**-314**

Section	Trans.	Comb.	9	N	Q2	M3	e0	e=ey-e0	M*
	Left		Extreme 1b	(kN)	(kN)	(kNm)	(m)	(m)	(kN.m)
Pile number			4	0	0	0	1.0	0.80	0
			5	0	0	0	1.0	0.80	0
			6	0	0	0	1.0	0.80	0
Distance - ey			1.80						
Self. of pilecap		0.90	1	-349		-314			-314
Soil on pilecap		0.90	1	0		0			0
Reaction from column									
				Q	H				M
Sum				**-349**	**0**				**-314**

M* = N.e + M3 - Q2.Hb/2

Section	Trans.	Comb.	1	N	Q2	M3	e0	e=ey-e0	M*
	Right		Strength 1a	(kN)	(kN)	(kNm)	(m)	(m)	(kN.m)
Pile number			1	0	0	0	1.0	0.80	0
			2	0	0	0	1.0	0.80	0
			3	0	0	0	1.0	0.80	0
Distance - ey			1.80						
Self. of pilecap		1.25	1	-349		-314			-314
Soil on pilecap		1.35	1	0		0			0
Reaction from column									
				Q	H				M
Sum				**-349**	**0**				**-314**

Section	Trans.	Comb.	7	N	Q2	M3	e0	e=ey-e0	M*
	Right		Service 1	(kN)	(kN)	(kNm)	(m)	(m)	(kN.m)
Pile number			1	0	0	0	1.0	0.80	0
			2	0	0	0	1.0	0.80	0
			3	0	0	0	1.0	0.80	0
Distance - ey			1.80						
Self. of pilecap		1.00	1	-349		-314			-314
Soil on pilecap		1.00	1	0		0			0
Reaction from column									
				Q	**H**				**M**
Sum				**-349**	**0**				**-314**

Section	Trans.	Comb.	8	N	Q2	M3	e0	e=ey-e0	M*
	Right		Extreme 1a	(kN)	(kN)	(kNm)	(m)	(m)	(kN.m)
Pile number			1	0	0	0	1.0	0.80	0
			2	0	0	0	1.0	0.80	0
			3	0	0	0	1.0	0.80	0
Distance - ey			1.80						
Self. of pilecap		1.25	1	-349		-314			-314
Soil on pilecap		1.35	1	0		0			0
Reaction from column									
				Q	**H**				**M**
Sum				**-349**	**0**				**-314**

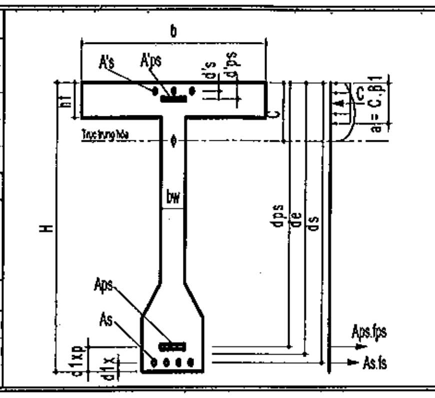
	DA NANG - QUANG NGAI EXPRESS WAY PROJECT VD1a BRIDGE	Item.	Eng.	Date.	Sign.
		Design			
	DETAIL DESIGN PIER CAP-P1R - CHECK STRENGTH	Check			
		Revise			

22TCN272-05; AASHTO LRFD 2nd - 1998

REINFORCEMENT CHECKING - PIER CAP

MATERIALS

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



Sign	Parameters		Unit	Section - CANTILEVER				
				A-A	A-A	G1	G1	
	INTERNAL FORCES AT SECTIOIN							
	Combination			Strength	Service	Strength	Service	
Qu	Shear		kN	5218	3761	2849	1990	
Mu	Flexural Moment		kNm	14448	10258	118	94	
Nu	Axial load		kN	0	0	0	0	
Tu	Torsional Moment		kNm	0	17	0	284	
	FLEXURAL MOMENT CHECKING							
H	Section height		m	2.724	2.724	2.724	2.724	
d's	Dis. From comp. fiber to centroid of comp. Reinf		m	0.058	0.058	0.058	0.058	
d1x	Dis. From tens. fiber to centroid of tension Reinf		m	0.133	0.133	0.133	0.133	
	Cover to reinf		m	0.050	0.050	0.050	0.050	
ds	Dis. From comp. fiber to centroid of tension Reinf		m	2.591	2.591	2.591	2.591	
d'ps	Dis. From comp. fiber to centroid of comp. prestressing steel		m	0.000	0.000	0.000	0.000	
d1xp	Dis. From tens. fiber to centroid of tension prestressing steel		m	0.000	0.000	0.000	0.000	
dps	Dis. From comp. fiber to centroid of tension prestressing steel		m	2.724	2.724	2.724	2.724	
b	Width of the compression face of member		m	1.600	1.600	1.600	1.600	
bw	Web width or diameter of a circular section		m	1.600	1.600	1.600	1.600	
hf	Compression flange depth		m	1.000	1.000	1.000	1.000	
Iz	Moment of inertia of section		m4	2.695	2.695	2.695	2.695	
Amc	Section area		m2	4.358	4.358	4.358	4.358	
	Steel choice							
Aps	Tension prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	
		Number	tendons	0	0	0	0	
		Area	m2	0.00000	0.00000	0.00000	0.00000	
A'ps	Compression prestressing steel	P.S type		0 T00.0	0 T00.0	0 T00.0	0 T00.0	
		Number	tendons	0	0	0	0	
		Area	m2	0.00000	0.00000	0.00000	0.00000	
As	Tension Reinforcement	Number	bars	39	39	39	39	
		Diameter	mm	32	32	32	32	
		Area	m2	0.03124	0.03124	0.03124	0.03124	
A's	Compression Reinforcement	Number	bars	0	0	0	0	
		Diameter	mm	28	28	28	32	
		Area	m2	0.00000	0.00000	0.00000	0.00000	
A'c	Shear reinforcement	Number	bars	4	4	4	4	
		Diameter	mm	20	20	20	20	
		Area	m2	0.00126	0.00126	0.00126	0.00126	
ϕ	Resistance factors for flexure		5.5.4.2	0.90	1.00	0.90	1.00	
ϕv	Resistance factors for shear			0.90	1.00	0.90	1.00	
ϕn	Resistance factors for axial force			1.00	1.00	1.00	1.00	
β1	Stress block factor			0.836	0.836	0.836	0.836	
c	Dis. Between centroid and top fiber		m	0.366	0.366	0.366	0.366	
	For T section behavior		m	0.366	0.366	0.366	0.366	
	For rectangular section behavior		m	0.366	0.366	0.366	0.366	
fpe	Effective stress in the prestressing steel after losses		Mpa	1116	1116	1116	1116	
fps	Aver. stress in pres. steel at the time for which the nominal resistance		Mpa	1789	1789	1789	1789	
k	Factor depends on type of P.S, Low relaxation strand k = 0.28			0.28	0.28	0.28	0.28	

a	Depth of equivalent stress block	m	0.306	0.306	0.306	0.306	
de	Corresp. effective depth from extreme comp. fiber						
	to centroid of tensile force in the tensile reinf.	m	2.591	2.591	2.591	2.591	
Mn	Nominal resistance	kNm	30463	30463	30463	30463	
Mr	Factored reistance	kNm	**27416**	**30463**	**27416**	**30463**	
Mu	Flexual moment	kNm	14448	10258	118	94	
(5.7.3.2)	**Flexural moment Checking**		**OK**	**OK**	**OK**	**OK**	
	Limits for reinforcement						
c/de	Maximum reinforcement		0.14	0.14	0.14	0.14	
	Maximum reinforcement Checking	<= 0.42	**OK**	**OK**	**OK**	**OK**	
r min	Mimimum reinforcement		0.72%	0.72%	0.72%	0.72%	
	Mimimum reinforcement Checking for RC	0.23%	**OK**	**OK**	**OK**	**OK**	
1.2*Mcr	Craking moment	kNm	4734	4734	4734	4734	
(5.7.3.3.2)	**Checking Mr>=min(1.2Mcr,1.33Mu)**		**OK**	**OK**	**OK**	**OK**	
(5.7.3.4)	**Conctrol of craking by distr. of reinf for RC member- Check?**		No	Yes	No	Yes	
	Existing condition for structrure	1,2 or 3	1	1	1	1	
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.066	0.066	0.066	0.066	
Z	Crack width parameter	N/mm	30000	30000	30000	30000	
A	Area of concr. with same centroid as tens. Reinf	m2	0.005	0.005	0.005	0.005	
fsa	Value	Mpa	423	423	423	423	
0.6*fy		Mpa	240	240	240	240	
	Tensil stress in reinf Min(fsa,0.6fy)	Mpa	240	240	240	240	
x	Dist. From compression fiber to centroid	m	-	0.716	-	0.716	
J.d	Arm	m	-	2.352	-	2.352	
Icr	Moment of inertia of the cracked section	m4	-	0.965	-	0.965	
fs	Tensile stress in reinforcement fs = Msls / (As*J.d)	Mpa	-	140	-	1	
	Checking for control cracking fs<fsa		**N.a**	**OK**	**N.a**	**OK**	
(5.10.8.2)	**Shrinkage and temperature Reinfocement (side distribution)**						
Areq	Area of required reinf	m2	0.00094	0.00094	0.00094	0.00094	
	Distribution on sides 6 D16	m2	0.00121	0.00121	0.00121	0.00121	
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45	
	Checking		**OK**	**OK**	**OK**	**OK**	
	SHEAR AND TORSION CHECKING						
β	Factor indicating diag. cracked concr. to tension		2.0	2.2	3.5	3.5	
θ	Angle of inclination of diagonal compressive	degree	40.12	36.37	28.51	28.43	
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	
bv	Effective web width as minimum web width - in dv	m	1.600	1.600	1.600	1.600	
dv	Effective shear depth	m	2.438	2.438	2.438	2.438	
	(de - a/2)	m	2.438	2.438	2.438	2.438	
s	Spacing of stirrups	m	0.150	0.150	0.150	0.150	
ncat	Amount of bars in spacing S	bars	4	4	4	4	
Av	Shear reinf area in spacing S	m2	0.0013	0.0013	0.0013	0.0013	
β	Assume		2.0	2.0	2.0	2.0	
θ	Assume	degree	42.03	39.62	42.52	33.75	
v	Shear stress in concrete	kN/m2	1486	964	812	510	
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1116	1116	1116	1116	
e_x	Strain in tensile reinforcement		1.41E-03	1.04E-03	2.56E-04	2.45E-04	
	if ex<0, multiple with reduce factor		-	-	-	-	
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok	
v/f'c	Ratio of shear stress and f'c		0.050	0.032	0.027	0.017	
β	Final value		2.0	2.2	3.5	3.5	
θ	Final value	degree	40.12	36.37	28.51	28.43	
Vc	Nominal shear resistance provided by tensile stresses in the concrete	kN	3545	3918	6144	6227	
Vs	Shear resistance provided by shear reinforcement	kN	9690	11087	15031	15079	
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	13236	15005	21175	21307	
Vn2	Vn2	kN	29254	29254	29254	29254	
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	13236	15005	21175	21307	
Vr	Factored shear resistance	kN	**11912**	**15005**	**19057**	**21307**	
Vu	Shear	kN	5218	3761	2849	1990	
(5.8.2.7)	**Shear checking**		**OK**	**OK**	**OK**	**OK**	
	Region requiring transverse reinf Checking		Need	Need	Need	No need	
	Minimum shear reinf area	m2	0.0003	0.0003	0.0003	0.0003	
	Mimimum shear reinforcement Checking		**OK**	**OK**	**OK**	-	
	0.1*f'c*bv*dv	kN	11702	11702	11702	11702	
	Smax	m	0.60	0.60	0.60	0.60	
	Maximum spacing Smax		**OK**	**OK**	**OK**	-	

φt	Resistance factor for torsion	(5.5.4.2)	0.90	1.00	0.90	1.00	
pc	Outer perimeter of concrete section	m	6.000	6.000	6.000	5.400	
Acp	Area in outer perimeter of concrete section	m2	4.358	4.358	4.358	4.358	
fpc	Comp. stress in concrete after all prestress losses at the centroid of se	Mpa	0.00	0.00	0.00	0.00	
Tcr	Crack moment due to torsion	kNm	5688	5688	5688	6320	
	0.25*φ*Tcr	kNm	1280	1422	1280	1580	
Tu	Torsional moment by external forces	kNm	0	17	0	284	
	Shear and Torsion combine if Tu>0.25φTcr		**No**	**No**	**No**	**No**	
Ao	Area enclosed by shear flow path	m2	2.839	2.839	3.346	3.346	
At	Area of one leg of closed transverse torsion reinforcement	m2	0.0003	0.0003	0.0003	0.0003	
ph	Perimeter of the centerline of the closed transverse torsion reinf.	m	8.168	8.168	8.168	8.168	
Aoh	Area enclosed by centerline of ext. closed transverse torsion reinf.	m2	3.340	3.340	3.936	3.936	
Vu1	Modified Vu in case shear and torsion combine	kN	5218	3761	2849	2015	
v1	Determine θt in case shear and torsion combine	kN/m2	1486	964	812	538	
θ	Assume	degree	40.61	37.29	27.00	32.05	
e_x	Strain in tensile reinforcement		1.41E-03	1.04E-03	2.56E-04	2.47E-04	
	if ex<0, multiple with reduce factor		-	-	-	-	
v1/f'c	Ratio of shear stress and f'c		0.050	0.032	0.027	0.018	
θt	Crack angle (S.5.8.3.4) upadated modified Vu	degree	40.12	36.37	28.51	28.47	
Tn	Nominal torsion resistance	kN	-	-	-	-	
Tr	Factored torsional resistance	kN	-	-	-	-	
(5.8.3.6.2)	**Torsional checking**		**N.a**	**N.a**	**N.a**	**N.a**	

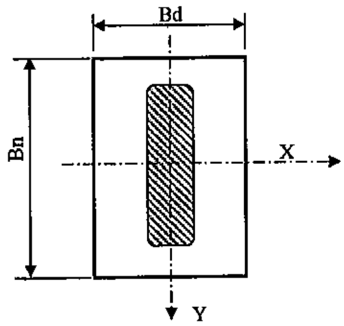
D. SHALLOW FOUNDATION CHECKING

1.LOAD COMBINATIONS AT BOTTOM OF PILE CAP

Load Combinations at Bottom of PileCap

No	Combinations	Sign	Vertical	Longitudinal		Transvesal	
			F_V	F_{HX}	My	F_{HY}	Mx
			(kN)	(kN)	(kN•m)	(kN)	(kN•m)
1	Strength Str-IA	Str-IA	18454	185	4029	0	7244
2	Strength Str-IB	Str-IB	12461	182	3979	12	7342
3	Strength Str-IIIA	Str-IIIA	17979	232	4315	127	7590
4	Strength Str-IIIB	Str-IIIB	11986	210	4103	133	7629
5	Service Ser-I	Ser-I	13574	178	3297	105	5803
6	Extreme Ext-IA	Ext-IA	16971	52	1135	3	2086
7	Extreme Ext-IB	Ext-IB	11270	52	1135	3	2086

2.CHECK BEARING RESISTANCE OF SHALLOW FOUNDATION



Pile cap properties

Longitudinal dimension		Bd	5.5	m
Transverse dimension		Bn	9.0	m
Longitudinal reduced effective dimension		B'd=Bd-2*ed		
Transverse reduced effective dimension		B'n=Bn-2*en		
Pile cap reduced effective area		A'=B'd*B'n		m2
Bending inertia moment		$W'x=B'd*B'n^2/6$		m3
		$W'y=B'n*B'd^2/6$		m3
Stress at corner points				
σ max= Fv/A+Mx/Wx+My/Wy				
σ min= Fv/A-Mx/Wx-My/Wy				
Resistance factor for bearing capacity - SLS, shallow foundation		ϕb	0.60	
Resistance factor for bearing capacity - other limit state		ϕb	1.00	
Unfactored compression strength	709.3 kg/cm2	Qu	10451	kN/m2
Factored bearing resistance		Qr	6271	kN/m2

FHWA manual 1988		
qu = σc.Ksq		
Ksq = [9+3.C/B]/[10.(1+300.g/C)^0.5]		
qu: ultimate end bearing pressure		
C: spacing of discontinuities	C=	0.25 m
B: pile width or (d pile diameter)	B=	5.5 m
g: aperture of discontinuite	g=	0.03 m
D = 1+0.4. (L/d) <=3.4 - depth factor	D=	1.828
L: length of the socket	L=	12 m
d: diameter of pile	d=	5.8 m
	Ksq=	0.1502
σc: unconfined compressive strength	σc =	69.582 Mpa
	Qu=	10.4513 Mpa
	Qu=	10451 kN/m2

Bearing Resistance AT BOTTOM OF FOOTING

Load Combination	Vertical	Longitudinal		Transvesal		Value		Area
	F_V	F_{HX}	My	F_{HY}	Mx	Bd	Bn	A
	(kN)	(kN)	(kN•m)	(kN)	(kN•m)	(m)	(m)	(m^2)
Strength Str-IA	18454	185	4029	0	7244	5.500	9.000	49.500
Strength Str-IB	12461	182	3979	12	7342	5.500	9.000	49.500
Strength Str-IIIA	17979	232	4315	127	7590	5.500	9.000	49.500
Strength Str-IIIB	11986	210	4103	133	7629	5.500	9.000	49.500
Service Ser-I	13574	178	3297	105	5803	5.500	9.000	49.500
Extreme Ext-IA	16971	52	1135	3	2086	5.500	9.000	49.500
Extreme Ext-IB	11270	52	1135	3	2086	5.500	9.000	49.500

Load Combination	Fv /A	Mx/Wx	My/Wy	σmax	σmin	Qr	Check
	(kPa)	(kPa)	(kPa)	(kPa)	(kPa)	(kPa)	
Strength Str-IA	373	98	89	559	186	6271	OK
Strength Str-IB	252	99	88	438	65	6271	OK
Strength Str-IIIA	363	102	95	561	166	6271	OK
Strength Str-IIIB	242	103	90	435	49	6271	OK
Service Ser-I	274	78	73	425	123	6271	OK
Extreme Ext-IA	343	28	25	396	290	10451	OK
Extreme Ext-IB	228	28	25	281	175	10451	OK

Bearing Resistance use to reduced effective area AT BOTTOM OF FOOTING

Load Combination	Vertical	Longitudinal		Transvesal		Value reduce		Area
	F_V	F_{HX}	My	F_{HY}	Mx	B'd	B'n	A'
	(kN)	(kN)	(kN•m)	(kN)	(kN•m)	(m)	(m)	(m^2)
Strength Str-IA	18454	185	4029	0	7244	5.063	8.215	41.594
Strength Str-IB	12461	182	3979	12	7342	4.861	7.822	38.023
Strength Str-IIIA	17979	232	4315	127	7590	5.020	8.156	40.941
Strength Str-IIIB	11986	210	4103	133	7629	4.815	7.727	37.208
Service Ser-I	13574	178	3297	105	5803	5.014	8.145	40.841
Extreme Ext-IA	16971	52	1135	3	2086	5.366	8.754	46.977
Extreme Ext-IB	11270	52	1135	3	2086	5.299	8.630	45.726

Load Combination	Fv /A'	Qr	Check
	(kPa)	(kPa)	
Strength Str-IA	444	6271	OK
Strength Str-IB	328	6271	OK
Strength Str-IIIA	439	6271	OK
Strength Str-IIIB	322	6271	OK
Service Ser-I	332	6271	OK
Extreme Ext-IA	361	10451	OK
Extreme Ext-IB	246	10451	OK

Bearing Resistance use to AASHTO 2004&2007 AT BOTTOM OF FOOTING

1. If the resultant is within the middle one-third of the base

$$\sigma_{vmax} = \frac{F_v}{B_d}\left(1+6\frac{e_d}{B_d}\right) \quad 11.6.3.2\text{-}2 \qquad \sigma_{vmin} = \frac{F_v}{B_d}\left(1-6\frac{e_d}{B_d}\right) \quad 11.6.3.2\text{-}3$$

2. If the resultant is outside the middle one-third of the base

$$\sigma_{vmax} = \frac{2F_v}{3\left[\left(\frac{B_d}{2}\right)-e_d\right]} \quad 11.6.3.2\text{-}2 \qquad \sigma_{v\,min} = 0 \quad 11.6.3.2\text{-}3$$

Load Combination	e_d (m)	$\lvert e_d \rvert$ (m)	σ_{max} (kPa)	σ_{min} (kPa)	Qr (kPa)	Check
Strength Str-IA	0.218	0.917	462	284	6271	OK
Strength Str-IB	0.319	0.917	339	164	6271	OK
Strength Str-IIIA	0.240	0.917	458	268	6271	OK
Strength Str-IIIB	0.342	0.917	333	152	6271	OK
Service Ser-I	0.243	0.917	347	202	6271	OK
Extreme Ext-IA	0.067	0.917	368	318	10451	OK
Extreme Ext-IB	0.101	0.917	253	203	10451	OK

3.CHECK SLIDING AT THE BASE OF FOOTING S.10.6.3.3

Horizontal force $H = (F_{HX}\text{^}2 + F_{HY}\text{^}2)\text{^}0.5$			
Factored resistance against failure by sliding			
$Q_r = \phi\, Q_n = \phi_t Q_t + \phi_{ep}\, Q_{ep}$			
Norminal shear resistance between soil and foundation Qt = Fv.tan(ϕ)	Qt		
For concrete cast against soil: tan (ϕ) = tan (ϕf)	tan(ϕf)	0.70	
Internal friction angle of soil	ϕf	35	deg
Resistance factor for shear reistance between soil and foundation	ϕt	0.80	
Norminal passive resistance	Qep	0.00	kN
Resistance factor for passive reistance	ϕep	0.50	

Load Combination	Resist. Factor ϕ	Fv (kN)	F_{HX} (kN)	F_{HY} (kN)	H (kN)	Qr (kN)	Check H< Hr
Strength Str-IA	0.80	18454	185	0	185	10337	OK
Strength Str-IB	0.80	12461	182	12	182	6980	OK
Strength Str-IIIA	0.80	17979	232	127	265	10071	OK
Strength Str-IIIB	0.80	11986	210	133	248	6714	OK
Service Ser-I	0.80	13574	178	105	206	7603	OK
Extreme Ext-IA	1.00	16971	52	3	52	11883	OK
Extreme Ext-IB	1.00	11270	52	3	52	7891	OK

4.CHECK OVERTURNNING AT THE BASE OF FOOTING

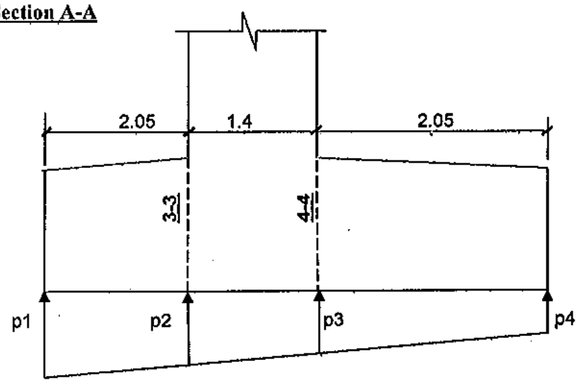
S.11.6.3.3, S.11.6.3.7

The location of the resultant of the reaction forces shall be within the middle three-fourths of the base.				
Longitudinal direction	ex =My / Fv <= 3.Bd / 8 = [ex]	[ex]	2.06	m
Transverse direction	ey =Mx / Fv <= 3.Bn / 8 = [ey]	[ey]	3.38	m
For seismic provision				
Longitudinal direction	ex =My / Fv <= 0.6 Bd / 2 = [ex]	[ex]	1.65	m
Transverse direction	ey =Mx / Fv <= 0.6 Bn / 2 = [ey]	[ey]	2.70	m
According to LRFD 2004: **2**				
Where γEQ = 0				
Longitudinal direction	ex =My / Fv <= 2/3. Bd / 2 = [ex]	[ex]	1.83	m
Transverse direction	ey =Mx / Fv <= 2/3. Bn / 2 = [ey]	[ey]	3.00	m
Where γEQ = 1				
Longitudinal direction	ex =My / Fv <= 8/10. Bd / 2 = [ex]	[ex]	2.20	m
Transverse direction	ey =Mx / Fv <= 8/10. Bn / 2 = [ey]	[ey]	3.60	m
Where γEQ between 0 and 1, restrictions of the location can get by linear interpolation				
Choosing value for seismic: following LRFD 2004, with γEQ = 0.5				
Longitudinal direction		[ex]	2.02	m
Transverse direction		[ey]	3.30	m

Load Combination	Fv	Mx	My	Longitudinal		Transverse	
				ex	Check	ey	Check
	(kN)	(kN•m)	(kN•m)	(m)	ex <[ex]	(m)	ey <[ey]
Strength Str-IA	18454	7244	4029	0.22	OK	0.39	OK
Strength Str-IB	12461	7342	3979	0.32	OK	0.59	OK
Strength Str-IIIA	17979	7590	4315	0.24	OK	0.42	OK
Strength Str-IIIB	11986	7629	4103	0.34	OK	0.64	OK
Service Ser-I	13574	5803	3297	0.24	OK	0.43	OK
Extreme Ext-IA	16971	2086	1135	0.07	OK	0.12	OK
Extreme Ext-IB	11270	2086	1135	0.10	OK	0.19	OK

5. BENDING MOMENT AND SHEAR OF FOOTING

Transverse Section A-A



Transverse section (3-3)-(4-4)
p1 = Fv /A + My/ Wy
p4 = Fv /A - My/ Wy

Sec tion 3-3

Load Combination	p1 (kPa)	p2 (kPa)	p3 (kPa)	p4 (kPa)	M_{3-3} (kN•m)	Q_{3-3} (kN)	N_{3-3} (kN)
Strength Str-IA	462	395	350	284	8312	7906	69
Strength Str-IB	339	274	229	164	6007	5659	68
Strength Str-IIIA	458	387	339	268	8220	7802	87
Strength Str-IIIB	333	265	219	152	5864	5514	78
Service Ser-I	347	293	256	202	6218	5900	66
Extreme Ext-IA	368	349	336	318	6839	6615	19
Extreme Ext-IB	253	234	221	203	4661	4490	19

Sec tion 4-4

Load Combination	p1 (kPa)	p2 (kPa)	p3 (kPa)	p4 (kPa)	M_{4-4} (kN•m)	Q_{4-4} (kN)	N_{4-4} (kN)
Strength Str-IA	462	395	350	284	5788	5850	69
Strength Str-IB	339	274	229	164	3514	3630	68
Strength Str-IIIA	458	387	339	268	5517	5601	87
Strength Str-IIIB	333	265	219	152	3294	3421	78
Service Ser-I	347	293	256	202	4153	4218	66
Extreme Ext-IA	368	349	336	318	6128	6036	19
Extreme Ext-IB	253	234	221	203	3950	3911	19

CALCULATION SHEET

EXPANSION JOINT

EXPANSION JOINT

I. Displacement

Maximum allowable displacements in longitudinal direction at pier A1 =	50 mm	
Maximum displacement	23.1	OK
Maximum allowable displacements in longitudinal direction at abutment A2=	50 mm	
Maximum displacement	28.9	OK

A1

Unit (mm)

Tải trọng	Symbol	Sign	Displacement		Service	
			Case1	Case2	a	b
TU+	TU	+	10.95	10.95	1.20	1.20
TU-	TU	-	-9.95	-9.95	1.20	1.20
Cr&Sh	CR&SH	-	-7.46	-7.46	1.20	0.50
Other loads		±	2.19	-2.19	1.00	1.00

Max Stretch	=	11.6
Max Shrink	=	-23.1
Maximum displacement		23.1

A2

Unit (mm)

Tải trọng	Ký hiệu	Dấu	Displacement		Service	
			Case1	Case2	a	b
TU+	TU	+	13.69	13.69	1.20	1.20
TU-	TU	-	-12.44	-12.44	1.20	1.20
Cr&Sh	CR&SH	-	-9.33	-9.33	1.20	0.50
Other loads		±	2.74	-2.74	1.00	1.00

Max Stretch	=	14.5
Max Shrink	=	-28.9
Maximum displacement		28.9