



Ministry of Transport



Vietnam Expressway Corporation



Project Management Unit No. 85



THE WORLD BANK

IDA Credit No. : 4779-VN

Project ID No. : P106235

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

Detailed Engineering Design Report (Final)

Volume 4: Structural Calculation Report (PKG A2)

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

Section 4.2.1

0. Typical calculation sheet

July 25, 2013

The Joint Venture of



NIPPON KOEI CO.,LTD.



NIPPON ENGINEERING CONSULTANTS CO.,LTD.



CHODAI CO.,LTD.



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IDA Credit No. : 4779-VN
(IDA tín dụng số : 4779-VN)
Project ID No. : P106235
(Mã dự án : P106235)

**Consulting Services
for**


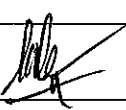
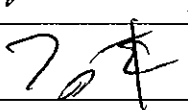
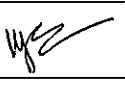
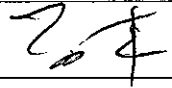
**Detailed Design for Danang - Quang Ngai Expressway Development Project
(Dịch vụ tư vấn Thiết kế kỹ thuật dự án Đường cao tốc Đà Nẵng - Quảng Ngãi)**

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

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

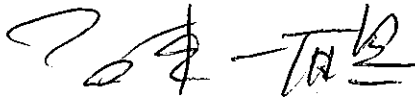
**Volume 4.2: Structural Calculation Report (PKG A2, Bridges)
(Tập 4.2: Báo cáo tính toán kết cấu (Gói thầu A2, Phần cầu))**

Section 4.2.1

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

Project Manager/Giám đốc Dự án


Ichizuru Ishimoto

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

MINISTRY OF TRANSPORT

VIETNAM EXPRESSWAY CORPORATION - PROJECT MANAGEMENT UNIT NO.85

DA NANG - QUANG NGAI EXPRESSWAY DEVELOPMENT PROJECT

PACKAGE: A2

TYPICAL CALCULATION SHEETS

Table of content-Typical calculation sheet

Package A2

A. General Input Data

B. Superstructure design

1. I girder, L=21m
2. I girder, L=27m
3. I girder, L=33m
4. I girder, L=40m
5. Link slab of I girder
6. Deck slab of I girder

C. Miscellaneous

1. Bearing I21 Girder
2. Bearing I27 Girder
3. Bearing I33 Girder
4. Bearing I40 Girder
5. Approach slab

MINISTRY OF TRANSPORT

VIETNAM EXPRESSWAY CORPORATION - PROJECT MANAGEMENT UNIT NO.85

DA NANG - QUANG NGAI EXPRESSWAY DEVELOPMENT PROJECT

PACKAGE: A2

CALCULATION SHEETS

GENERAL INPUT DATA

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	DETAIL DESIGN		

1. GENARAL

1.1 Location

Package A2 from Km81+364 to Km92+359, at Quang Nam province.

No	Bridge	Station	District	Commune
1	CB23	081+364	Núi Thành	Tam Anh
2	ORB22	082+349	Núi Thành	Tam Anh
3	OP18a	082+986	Núi Thành	Tam Hiệp
4	OP19	085+737	Núi Thành	Tam Hiệp
5	ORB23	087+709	Núi Thành	Tam Mỹ
6	LRB12a	087+970	Núi Thành	Tam Mỹ
7	FO09	089+158	Núi Thành	Tam Mỹ
8	ORB25a	091+140	Núi Thành	Tam Mỹ
9	CB25	092+359	Núi Thành	Tam Nghĩa

1.2 Structure

Package A2

No	Bridge	Type of girder	Arrange
1	CB23	I Girder	1x40
2	ORB22	I Girder	3x21
3	OP18a	I Girder	1x27
4	OP19	I Girder	1x27
5	ORB23	I Girder	1x33
6	LRB12a	I Girder	4x33
7	FO09	Void Slab Girder	2x24
8	ORB25a	I Girder	1x21
9	CB25	I Girder	1x33

2. SPECIFICATIONS AND MATERIALS

2.1. Specifications

[1]. Specification for bridge design 22TCN-272-05

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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 1st edition, 1998 specification for bridge construction
- [6]. Transportation works in seismic zone - *22TCN221-95*

2.2. Materials characteristics

a. Concrete:

Items	Unit	I girder	Void Slab girder	Pier	Abutment	Bored pile
Density	kg/m ³	2400	2400	2400	2400	2400
Compressive strength at 28 age days, f_c , cylinder specimen	MPa	45	40	30	30	30
Elastic modulus $E_c = 0.043 \gamma_c^{1.5} \sqrt{f_c}$	MPa	33915	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	20.25	18.00			
Limited tensile stress - $0.5\sqrt{f_c}$ At service stage	MPa	-3.35	-3.16			

b. Reinforcement:

Reinforcement type	Grade	Elastic Modulus E(MPa)	f_y (MPa)	f_u (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

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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	mm ²	140	98.7
Ultimate strength f_{pu}	MPa	1860	1860
Yield strength f_{py}	MPa	1670	1670
Elastic modulus	MPa	197000	197000
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°C and 0.7P _M	
Prestressing force before 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,	Up to 100	Up to 100	Up to 100	Up to 100	Over 65 to

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mm	incl.	incl.	incl.	incl.	100 incl.
Shapes	All Groups	All Groups	All Groups	Not applicable	Not applicable
Minimum Strength, F_u , MPa	400	450	485	485	690
Minimum yield strength, F_y , MPa	250	345	345	345	620

* F_y : 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, F_u , MPa		414
Minimum yield strength, F_y , MPa	290	

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3. LOADS

3.1. Selfweight

Unit weight of reinforcement concrete	24.50 kN/m³
Unit weight of structural steel	77.01 kN/m³
Unit weight of Asphalt concrete	22.1 kN/m³

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:

$$(0.56+0.408)\text{m}^2 \times 24.5\text{kN/m}^3 = \mathbf{23.7\text{ kN/m}}$$

+ *For Flyover bridges:*

Parapet + steel rail for 2 side:

$$(0.416)\text{m}^2 \times 2 \times 24.5\text{kN/m}^3 + 1.25 \times 2 \text{ kN/m} = \mathbf{22.9\text{ kN/m}}$$

c) Miscellaneous dead loads:

$$(\text{traffic lights, water drainage ...}) = \mathbf{2.00\text{ kN/m}} \text{ assumed}$$

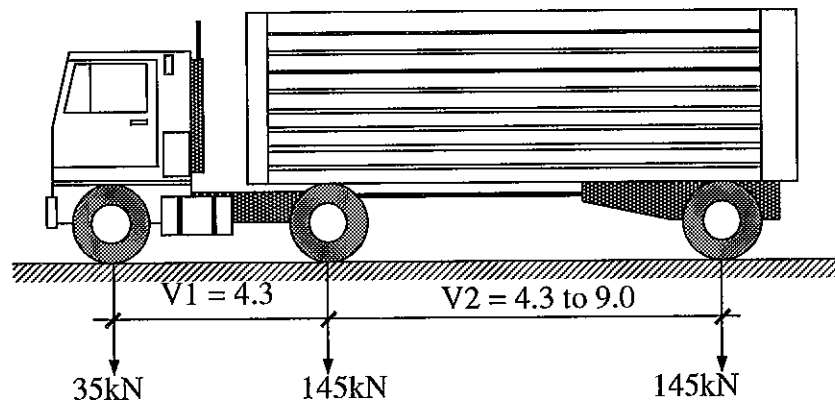
	<i>Project: Da Nang-- Quang Ngai Expressway</i> <i>General Input Data – Package A2</i>	Date: / /12	Page: 6
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3.3. Live Loads

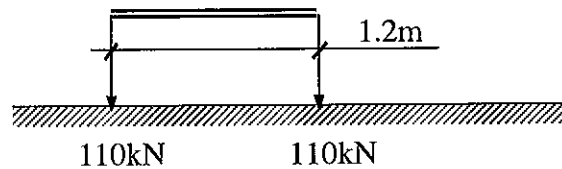
Vehicular live loading on the roadways of bridges or incidental structures, designated HL-93, shall consist of a combination of the:

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

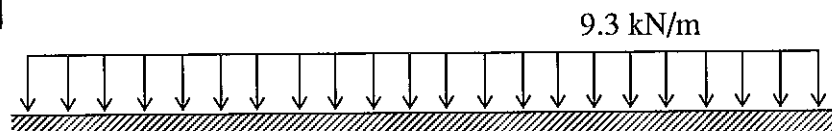
Design Truck



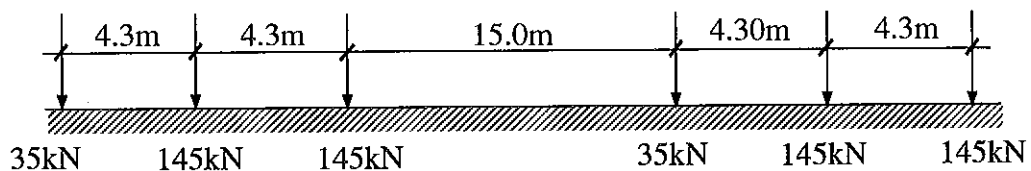
Design Tandem



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%

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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:

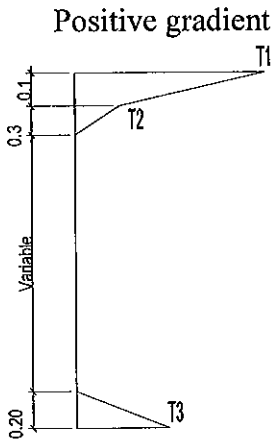
Annual average Temp.	Maximum Temp.	Minimum Temp.
+25 ⁰ C	+47 ⁰ C	10 ⁰ C

- Reference temperature: +25⁰C.
- Uniform temperature:

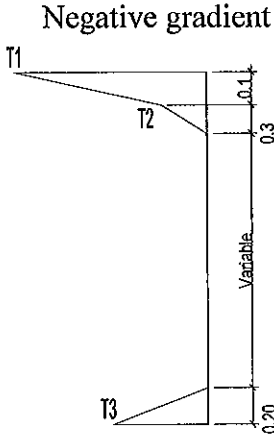
Structure (°C)	Increase	Decrease
Reinforcement concrete	+22.0	-15.0

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3.5.2. Gradient temperature:



$T1 = 23^{\circ} \text{C}$
 $T2 = 6^{\circ} \text{C}$
 $T3 = 3^{\circ} \text{C}$



$T1 = - 7^{\circ} \text{C}$
 $T2 = - 1^{\circ} \text{C}$
 $T3 = 0^{\circ} \text{C}$

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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 $V_B = 53.0 \text{ m/s}$

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

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 transversal direction, is 1.5 kN/m at 1.8m height from asphalt surface. Wind load on vehicular in longitudinal direction is 0.75 kN/m at 1.8m height from asphalt surface.

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	$6.5 < \text{grade} \leq 7.5$
$0.19 < A < 0.29$	3	$7.5 < \text{grade} \leq 8$

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

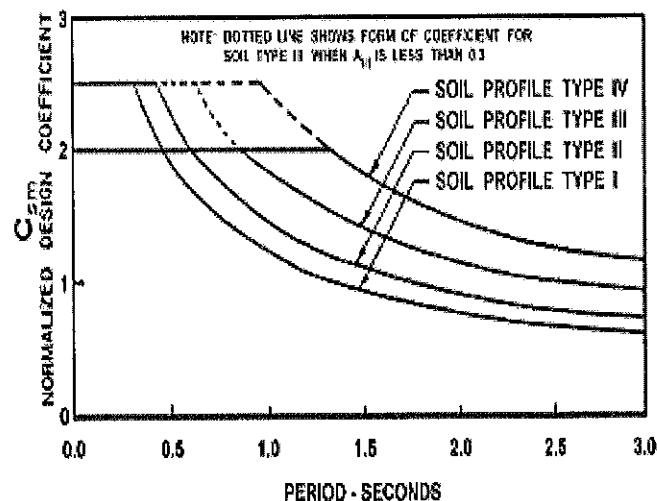
No	Bridge	A
1	CB23	0.058
2	ORB22	0.058
3	OP18a	0.058
4	OP19	0.058
5	ORB23	0.058
6	LRB12a	0.058
7	FO09	0.058
8	ORB25a	0.058
9	CB25	0.058

Coefficient site S: according to geological data survey, at Package A2, *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

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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.

$$\sum \eta_i \cdot \gamma_i \cdot Q_i \leq \phi \cdot R_n = R_r$$

Where:

- η_i : load modifier factors
- γ_i : load factors
- ϕ : resistance factors
- Q_i : forces effect
- R_n : nominal resistance
- R_r : factored resistance

For loads for which a maximum value of γ_i is appropriate: $\eta_i = \eta_D \times \eta_R \times \eta_I \geq 0.95$

For loads for which a minimum value of γ_i 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

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Load modifier factors used in design:

Load modifier factors for loads for which	η_i
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	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

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	DETAIL DESIGN		

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.

MINISTRY OF TRANSPORT

VIETNAM EXPRESSWAY CORPORATION - PROJECT MANAGEMENT UNIT NO.85

DA NANG - QUANG NGAI EXPRESSWAY DEVELOPMENT PROJECT

PACKAGE: A2

CALCULATION SHEETS

SUPERSTRUCTURE

CALCULATION SHEET

I Girder

121 Glrder, S=2.55m

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1. INPUT DATA:

1.1. General Data

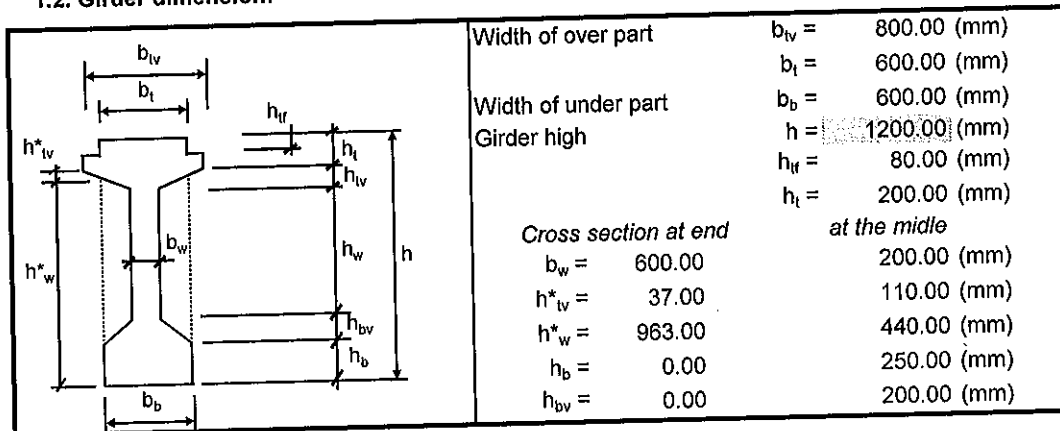
Specification for bridge design:
 Live load
 Impact (or dynamic) of the live load
 Pedestrian
 Length of Girder
 Span between support
 Carriageway width in bridge
 Parapet width
 Bridge width
 Number of girder
 Space between 2 girders
 Distance from inside of parapet to exterior girder center
 Width of bridge deck
 Length of the overhang (cantilever arm length)
 Thickness of bridge deck
 Precast plank width
 Precast plank thick
 Pavement thick

TCN 272-05

HL93

IM = 0.25
 PL = 0.00 (kN/m²)
 L_d = 21.00 (m)
 L_{it} = 20.30 (m)
 w = 11.75 (m)
 c = 0.50 (m)
 B = 12.75 (m)
 N_d = 5.00 girder
 S = 2.55 (m)
 d_e = 0.78 (m)
 b_{ds} = 12.48 (m)
 L_h = 1.28 (m)
 t_s = 0.22 (m)
 b_p = 1.95 (m)
 h_p = 0.08 (m)
 h_{pa} = 0.084 (m)

1.2. Girder dimension:



1.3. MATERIAL PROPERTIES:

1.3.1 Concrete:

Girder concrete

Girder concrete strength at the 28 age days

Unit weight of Concrete

Modulus of elasticity

Deck concrete

Deck concrete strength at the 28 age days

Unit weight of concrete

Modulus of elasticity

f_c = 45.00 MPa
 γ_c = 2450.00 kG/m³
 E_c = 0.043 γ_c^{1.5} sqrt(f_c) = 34980.32 MPa (5.4.2.4-1)

f_c = 35.00 MPa
 γ_c = 2450.00 kG/m³
 E_c = 0.043 γ_c^{1.5} sqrt(f_c) = 30849.75 MPa (5.4.2.4-1)

1.3.2 Prestressing steel

Diameter of one strand

Area of one strand

Ultimate Tendon strength

Yield strength of prestressing steel

Modulus of strand

Wobble friction coefficient (mm-1)

Coefficient of friction (1/RAD)

Number of Strands in one Tendon

Area of one Tendon

Stress in the prestressing steel at jacking

Jacking force for one tendon

Anchorage set

Area of one duck

Number of Tendons

D = 12.70 mm
 A_s^{12.7} = 98.70 mm²
 f_{pu} = 1860.00 MPa
 f_{py} = 0.9 f_{pu} = 1674.00 MPa
 E_p = 195000.00 MPa
 K = 6.60E-07 mm⁻¹
 μ = 0.25
 n = 9.00 Strands
 A_s = 888.30 mm²
 f_{pl} = 0.75 f_{pu} = 1395.00 MPa
 P_j = 1239.18 kN
 ΔL = 6.00 mm
 A_g = 3318.31 mm²
 N = 4.00 Tendons

1.3.3 Reinforcing Steel:

Yield strength (deformed bar)

Modulus of steel

f_{py} = 400.00 (MPa)
 E_s = 200000.00 (MPa)

2. INTERNAL FORCE:

2.1. Dead Load:

2.1.1 Load:

Interior Beam:

Bridge deck	DC _d =	13.58 (kN/m)
Precast plank & cross beam	DC _{pl} =	4.23 (kN/m)
Parapet	DC _{pa} =	4.74 (kN/m)
Pavement	DW _p =	4.44 (kN/m)

Exterior Beam:

Bridge deck	DC _d =	13.58 (kN/m)
Precast plank & cross beam	DC _{pl} =	2.12 (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 :

$$M = 0.5 q \cdot X_1 (L - X_1)$$

$$Q = q \cdot (0.5 \cdot L - X_1)$$

$$L_{eff} = 20.30 \text{ (m)}$$

INTERIOR GIRDER											
Section	X ₁ (m)	Girder (DC)		Concrete Deck (DC)		Plank & cr.beam (DC)		Parapet (DC)		Pavement (DW)	
		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	149.65	0.00	137.79	0.00	42.95	0.00	48.11	0.00	45.08
L/8	2.54	332.26	112.23	305.95	103.35	95.36	32.21	106.82	36.08	100.09	33.81
L/4	5.08	569.59	74.82	524.48	68.90	163.48	21.47	183.12	24.06	171.59	22.54
3L/8	7.61	711.99	37.41	655.60	34.45	204.35	10.74	228.90	12.03	214.49	11.27
L/2	10.15	759.45	0.00	699.30	0.00	217.97	0.00	244.16	0.00	228.79	0.00
EXTERIOR GIRDER											
Support	0.00	0.00	149.65	0.00	137.79	0.00	42.95	0.00	48.11	0.00	45.08
L/8	2.54	332.26	112.23	305.95	103.35	47.68	32.21	106.82	36.08	100.09	33.81
L/4	5.08	569.59	74.82	524.48	68.90	81.74	21.47	183.12	24.06	171.59	22.54
3L/8	7.61	711.99	37.41	655.60	34.45	102.17	10.74	228.90	12.03	214.49	11.27
L/2	10.15	759.45	0.00	699.30	0.00	108.98	0.00	244.16	0.00	228.79	0.00

2.2 Live Load:

2.2.1. Distribution factors for Live load:

Modular Ratio: Girder Concrete/Deck Concrete

Distance from girder centroid to bridge deck centroid

Longitudinal stiffness parameter

Ration

$$n = E_g / E_d = 1.13$$

$$e^I_g = 702.42 \text{ (mm)}$$

$$e^E_g = 702.42 \text{ (mm)}$$

$$K^I_g = n(lg + A e^2_g) = 7.8E+11$$

$$K^E_g = n(lg + A e^2_g) = 7.8E+11$$

$$K^I_g / (L t^3_g) = 3.52$$

$$K^E_g / (L t^3_g) = 3.52$$

$$S / L = 0.13$$

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

Interior Beam:

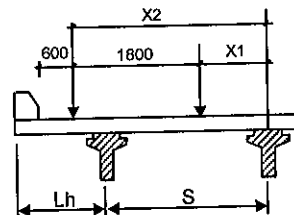
$$\text{For one lane} \quad 0.06 + \left(\frac{S}{4300} \right)^{0.4} \left(\frac{S}{L} \right)^{0.3} \left(\frac{K_g}{L t^3_g} \right)^{0.1} = 0.554$$

$$\text{Two or more lanes} \quad 0.075 + \left(\frac{S}{2900} \right)^{0.6} \left(\frac{S}{L} \right)^{0.2} \left(\frac{K_g}{L t^3_g} \right)^{0.1} = 0.768$$

Exterior Beam:

For one lane, follow the lever rule

$$\Rightarrow g(M) = 0.5 \sum y_i = 0.716$$



Two or more lanes

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

$$\text{Choice } e = 1.047 \quad \text{IF } (e > 1, 1.e)$$

$$\Rightarrow g(M) = e \cdot g_{\text{strong}} = 0.804$$

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

Interior Beam:

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

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

$$g(Q) = 0.5 \cdot \sum y_i = \boxed{0.716}$$

Two or more lanes

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

$$\Rightarrow g(Q) = e \cdot g_{\text{strong}} = \boxed{0.731}$$

(c) Correction factor for skew bridge:

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

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

Area of applications
 $300 \leq \theta \leq 600$
 $1100 \leq S \leq 4900$
 $6000 \leq L \leq 73000$
 $Nb \geq 4$

$$CF(M) = 1.0 - c1 \cdot (\tan \theta)^{1.5}$$

$$c1 = 0.25 \left(\frac{Kg}{L \cdot ts^3} \right)^{0.25} \left(\frac{S}{L} \right)^{0.5}$$

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

Correction Factor $CF(Q) = 1.050$

Area of applications
 $00 \leq \theta \leq 600$
 $1100 \leq S \leq 4900$
 $6000 \leq L \leq 73000$
 $Nb \geq 4$

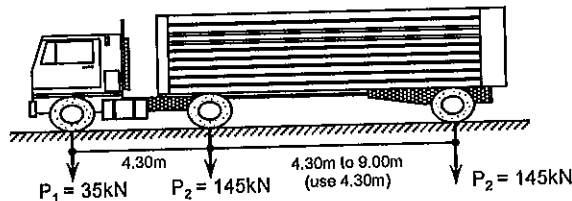
$$CF(Q) = 1.0 + 0.20 \left(\frac{L \cdot ts^3}{Kg} \right)^{0.3} \cdot \tan \theta$$

(d) Table of Distribution factors for Live load:

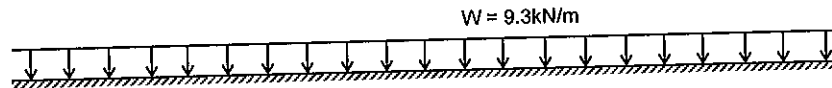
Interior Beam	$g(M)$	$g(Q)$	m	$m \cdot g(M)$	$m \cdot g(Q)$	$m \cdot g(M) \cdot CF(M)$	$m \cdot g(Q) \cdot CF(Q)$
1 lane	0.554	0.696	1.20	0.665	0.835	0.665	0.876
2 or more lanes	0.768	0.852	1.00	0.768	0.852	0.768	0.894
Exterior Beam							
1 lane	0.716	0.716	1.20	0.859	0.859	0.859	0.902
2 or more lanes	0.804	0.731	1.00	0.804	0.731	0.804	0.767

2.2.2 Live Load:

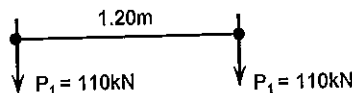
Design Truck



Design Lane Load



Design Tandem



- 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
- Dynamic load	IM = 0.25

2.2.3 Internal Force due to Live load:

Design truck or Tandem

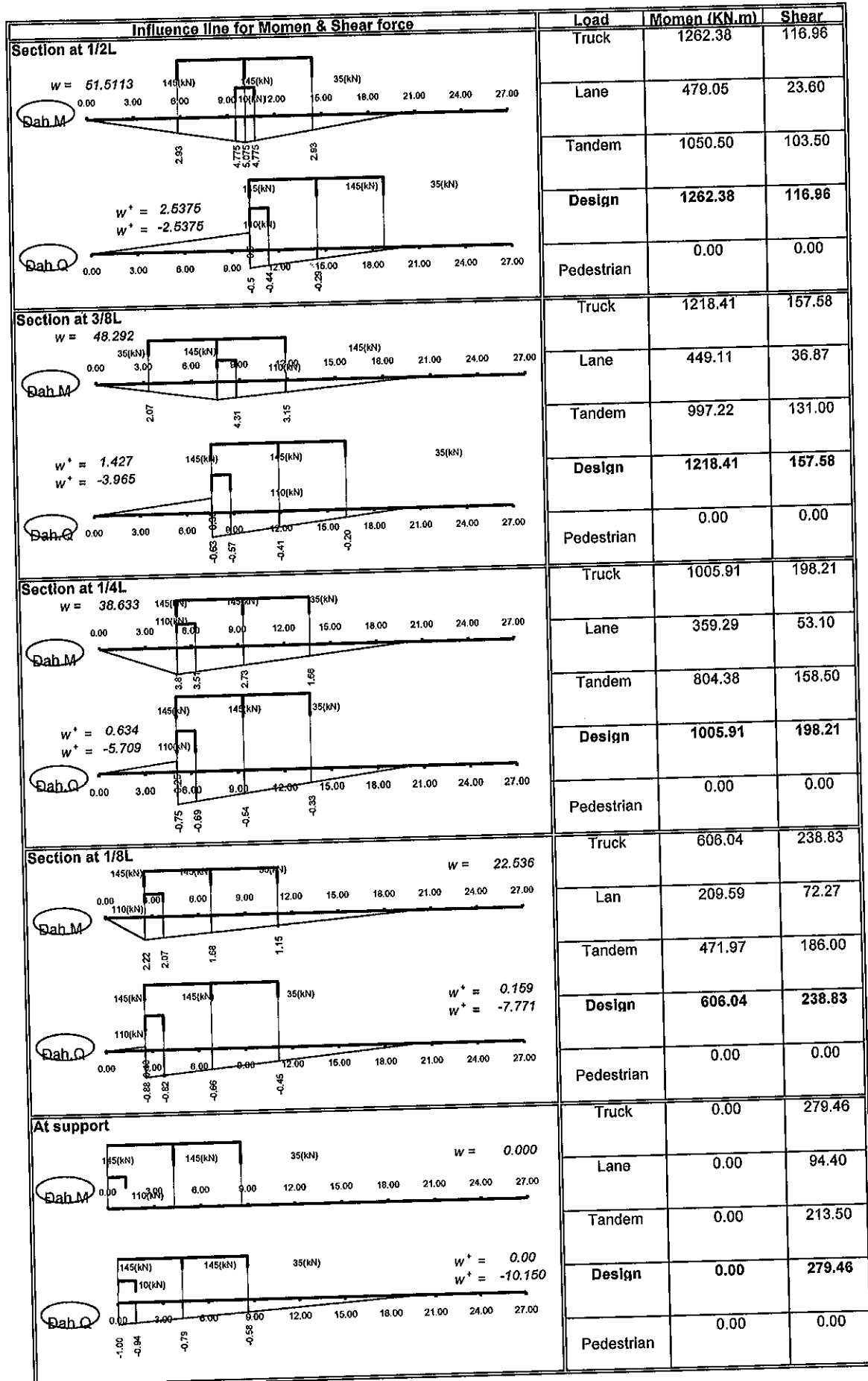
Momen $M_{TR(Ta)} = \sum P_i y_i$ (kNm)
 Shear force $Q_{TR(Ta)} = \sum P_i y_i$ (kN)

Lane load

Momen $M_{Ln} = W \cdot F_M$ (kNm)
 Shear force $Q_{Ln} = W \cdot F_Q$ (kN)

Pedestrian

Momen $M_{PL} = PL \cdot F_M$ (kNm)
 Shear force $Q_{PL} = PL \cdot F_Q$ (kN)



Internal Force due to Live load :

$$M_{(LL+IM)} = m \cdot g(M) \cdot [\max\{M_{TR}, M_{Ta}\} \cdot (1+IM) + M_{Ln}]$$

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

Internal Force due to pedestrian :

$$M = g(M) \cdot M_{PL}$$

$$Q = g(Q) \cdot Q_{PL}$$

In which:

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

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

y_l Value of influence line

m Lane factor

F

Area of influence line

g

Distribution factor

	$m \cdot g(M)$	$m \cdot g(Q)$
Interior	0.768	0.894
Exterior	0.859	0.902

TABLE OF INTERNAL FORCE DUE TO LIVE LOAD

Section	Xl (m)	Interior Girder		Exterior Girder	
		M (kNm)	Q (kN)	M (kNm)	Q (kN)
Support	0.00	0.00	396.70	0.00	400.09
L/8	2.54	743.07	331.52	830.60	334.36
L/4	5.08	1242.12	268.98	1388.44	271.28
3L/8	7.61	1515.22	209.07	1693.71	210.86
L/2	10.15	1580.45	151.80	1766.62	153.10

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 factord

$$\eta = \eta_D \eta_R \eta_I$$

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	
		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	473.13	1050.49	354.84	1800.83	236.56	2251.04	118.28	2401.11	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	694.23	1300.37	580.16	2173.71	470.71	2651.64	365.88	2765.78	265.66
Total		0.00	1234.97	2501.00	986.72	4231.93	741.08	5224.42	501.06	5510.08	265.66

STATE Service											
Load	Load Factor	Section									
		Support		L/8		L/4		3L/8		L/2	
		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	378.50	840.39	283.88	1440.67	189.25	1800.83	94.63	1920.89	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	396.70	743.07	331.52	1242.12	268.98	1515.22	209.07	1580.45	151.80
Total		0.00	820.28	1683.55	649.21	2854.38	480.77	3530.55	314.97	3730.13	151.80

2.3.2 Load combination - Exterior Girder:

STATE Strength											
Load	Load factor	Section									
		Supprt		L/8		L/4		3L/8		L/2	
		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	473.13	990.89	354.84	1698.66	236.56	2123.33	118.28	2264.88	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	700.17	1453.55	585.12	2429.77	474.74	2964.00	369.01	3091.59	267.93
Total		0.00	1240.91	2594.57	990.68	4385.81	745.11	5409.06	504.19	5699.65	267.93

STATE Service											
Load	load factor	Section									
		Support		L/8		L/4		3L/8		L/2	
		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	378.50	792.71	283.88	1358.93	189.25	1698.66	94.63	1811.91	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	400.09	830.60	334.36	1388.44	271.28	1693.71	210.86	1766.62	153.10
Total		0.00	823.68	1723.40	652.04	2918.96	483.07	3606.86	316.76	3807.31	153.10

3. TENDON PROFILE AND PROPERTY OF GIRDER CROSS SECTION

3.1. Tendon profile:

Tendon profile follow Parabol equation:

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

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 = X₂ - X₁ Distance between 2 point under consider
- angle of rotation of tendon for X_i-axis $\tan(\alpha) = (4 \cdot f \cdot (1 - 2 \cdot X_i / L)) / L$
- $\alpha = 2 \cdot f / 0.5 \cdot L - \tan(\alpha)$

L _{span} =	21000	(mm)
L _{su} =	20300	(mm)
L _{cap} =	20700	(mm)

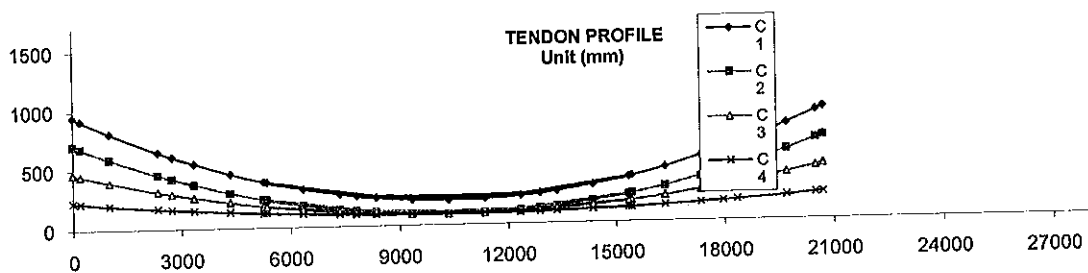
TENDON No	Section	f =	Xi	Yi	Lp	ΣLcáp	Tan(α)	(α)	Σα	Section
		(mm)	(mm)	(mm)	(mm)	(mm)		(rad)	(rad)	
1	Anchorage	945	0.00	945.00	0.00	0.00	0.1826	0.0000	0.0000	Anchorage
	Support		200.00	916.87	200.00	200.00	0.1791	0.0035	0.0035	Support
	L/8		2737.50	607.61	2537.50	2737.50	0.1343	0.0483	0.0518	L/8
	L/4		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

TENDON No	Section	f =	Xi	Yi	Lp	ΣLcáp	Tan(α)	(α)	Σα	Section
		(mm)	(mm)	(mm)	(mm)	(mm)		(rad)	(rad)	
2	Anchorage	705	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

TENDON No	Section	f =	Xi	Yi	Lp	ΣLcáp	Tan(α)	(α)	Σα	Section
		(mm)	(mm)	(mm)	(mm)	(mm)		(rad)	(rad)	
3	Anchorage	465	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

TENDON No	Section	f =	Xi	Yi	Lp	ΣLcáp	Tan(α)	(α)	Σα	Section
		(mm)	(mm)	(mm)	(mm)	(mm)		(rad)	(rad)	
4	Anchorage	225	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

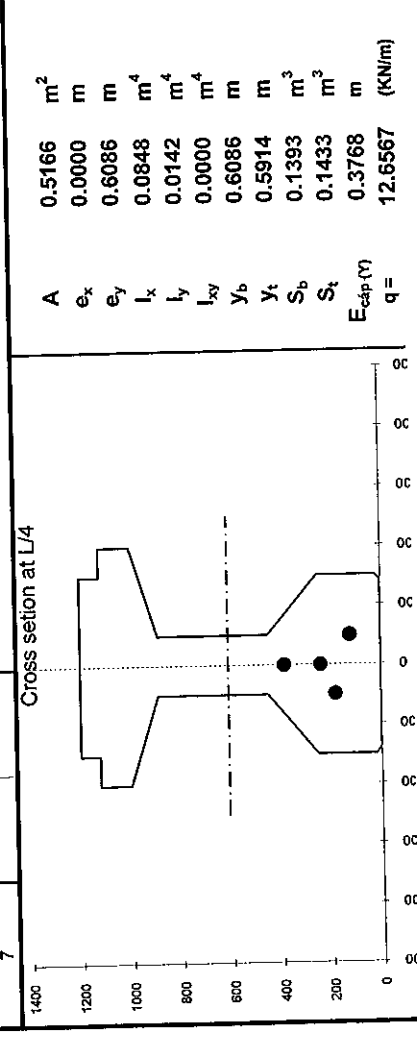
Section	TENDON No 1		TENDON No 2		TENDON No 3		TENDON No 4	
	X _i (mm)	Y _i (mm)	X _i (mm)	Y _i (mm)	X _i (mm)	Y _i (mm)	X _i (mm)	Y _i (mm)
Anchorage	0.00	945.00	0.0	705.00	0.0	465.00	0.00	225.00
Support	200.00	916.87	200.0	681.46	200.0	450.65	200.00	219.83
1	1000.00	809.83	1000.0	591.90	1000.0	396.04	1000.00	200.17
2	2350.00	649.12	2350.0	457.43	2350.0	314.04	2350.00	170.66
3	3350.00	546.20	3350.0	371.31	3350.0	261.53	3350.00	151.75
L/8	2737.50	607.61	2737.5	422.70	2737.5	292.86	2737.50	163.03
4	4350.00	457.01	4350.0	296.68	4350.0	216.02	4350.00	135.37
5	5350.00	381.53	5350.0	233.53	5350.0	177.52	5350.00	121.51
6	6350.00	319.78	6350.0	181.86	6350.0	146.01	6350.00	110.16
7	7350.00	271.75	7350.0	141.67	7350.0	121.51	7350.00	101.34
L/4	5275.00	386.72	5275.0	237.87	5275.0	180.16	5275.00	122.46
8	8350.00	237.45	8350.0	112.96	8350.0	104.00	8350.00	95.04
9	9350.00	216.86	9350.0	95.74	9350.0	93.50	9350.00	91.26
10	10350.00	210.00	10350.0	90.00	10350.0	90.00	10350.00	90.00
11	11350.00	216.86	11350.0	95.74	11350.0	93.50	11350.00	91.26
3L/8	7812.50	254.18	7812.5	126.97	7812.5	112.54	7812.50	98.11
12	12350.00	237.45	12350.0	112.96	12350.0	104.00	12350.00	95.04
13	13350.00	271.75	13350.0	141.67	13350.0	121.51	13350.00	101.34
14	14350.00	319.78	14350.0	181.86	14350.0	146.01	14350.00	110.16
15	15350.00	381.53	15350.0	233.53	15350.0	177.52	15350.00	121.51
L/2	10350.00	210.00	10350.0	90.00	10350.0	90.00	10350.00	90.00
2	5350.00	381.53	5350.0	233.53	5350.0	177.52	5350.00	121.51
3	6350.00	319.78	6350.0	181.86	6350.0	146.01	6350.00	110.16
4	7350.00	271.75	7350.0	141.67	7350.0	121.51	7350.00	101.34
5	8350.00	237.45	8350.0	112.96	8350.0	104.00	8350.00	95.04
-	12887.50	254.18	12887.5	126.97	12887.5	112.54	12887.50	98.11
6	9350.00	216.86	9350.0	95.74	9350.0	93.50	9350.00	91.26
7	10350.00	210.00	10350.0	90.00	10350.0	90.00	10350.00	90.00
8	11350.00	216.86	11350.0	95.74	11350.0	93.50	11350.00	91.26
9	12350.00	237.45	12350.0	112.96	12350.0	104.00	12350.00	95.04
-	15425.00	386.72	15425.0	237.87	15425.0	180.16	15425.00	122.46
10	13350.00	271.75	13350.0	141.67	13350.0	121.51	13350.00	101.34
11	14350.00	319.78	14350.0	181.86	14350.0	146.01	14350.00	110.16
12	15350.00	381.53	15350.0	233.53	15350.0	177.52	15350.00	121.51
13	16350.00	457.01	16350.0	296.68	16350.0	216.02	16350.00	135.37
-	17962.50	607.61	17962.5	422.70	17962.5	292.86	17962.50	163.03
14	17350.00	546.20	17350.0	371.31	17350.0	261.53	17350.00	151.75
14	18350.00	649.12	18350.0	457.43	18350.0	314.04	18350.00	170.66
16	19700.00	809.83	19700.0	591.90	19700.0	396.04	19700.00	200.17
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



ĐẶC TRƯNG HÌNH HỌC - MẶT CẮT L/4

Ái-lam giới	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	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

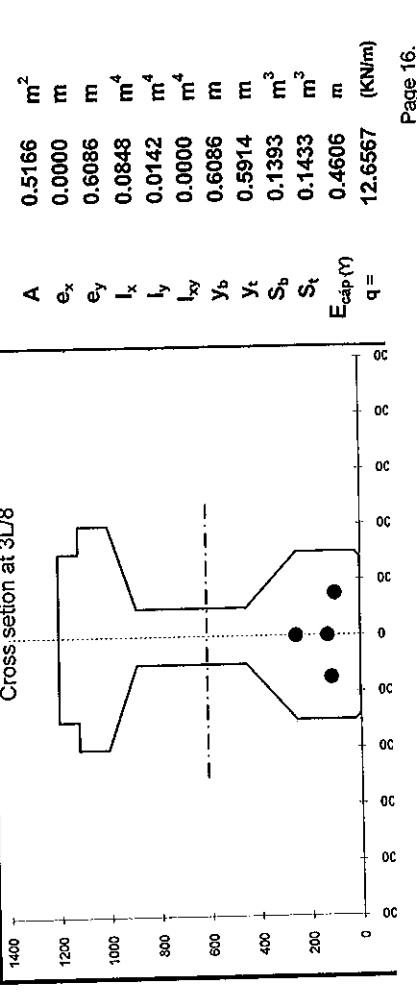
Cấp	1	0.00	386.72	0.00	0.00	0.00	0.00	0.00
2	0.00	237.87	0.00	0.00	0.00	0.00	0.00	0.00
3	-98.00	180.16	0.00	0.00	0.00	0.00	0.00	0.00
4	98.00	122.46	0.00	0.00	0.00	0.00	0.00	0.00
5								
6								
7								



ĐẶC TRƯNG HÌNH HỌC - MẶT CẮT 3L/8

Ái-lam giới	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	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

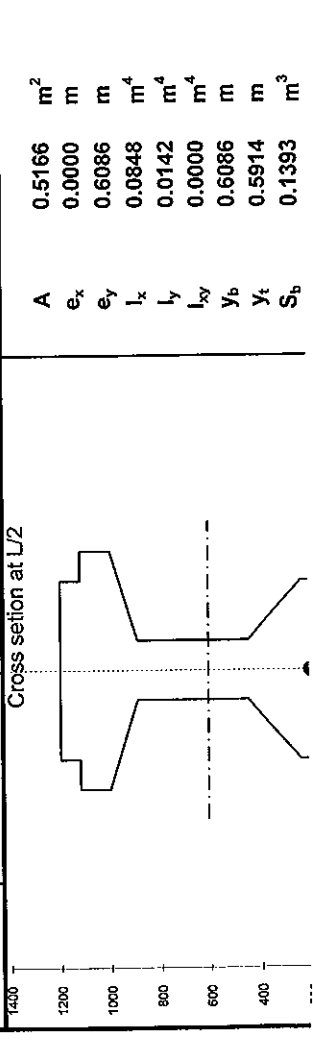
Cấp	1	0.00	254.18	0.00	0.00	0.00	0.00	0.00
2	0.00	126.97	0.00	0.00	0.00	0.00	0.00	0.00
3	-150.00	112.54	0.00	0.00	0.00	0.00	0.00	0.00
4	150.00	98.11	0.00	0.00	0.00	0.00	0.00	0.00
5								
6								
7								

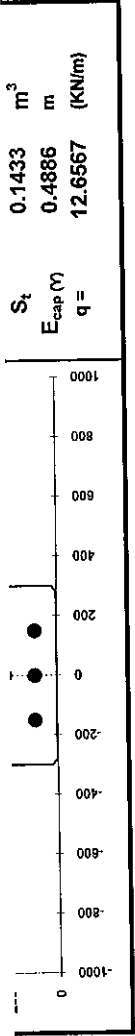


ĐẶC TRƯNG HÌNH HỌC - MẶT CẮT L/2

Âi lõm góc	x (mm)	y (mm)	A (mm ²)	Q _x [*] (mm ³)	Q _y [*] (mm ³)	I _x [*] (mm ⁴)	I _y [*] (mm ⁴)	I _{xy} [*] (mm 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	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				As =	888.30 (mm ²)	
2	0.00	90.00				n =	4.00 bó	
3	-150.00	90.00				Σ As Yl =	426384.00 (mm ³)	
4	150.00	90.00				Σ As =	3553.20 (mm ²)	
5						K. cách từ trọng tâm bó cáp đến TTH =	488.58 (mm)	
6								
7								

Cross section at L/2





Uniform load due to self weigh of Girder in Stage 1: $Q = 14.74 \text{ (KN/m)}$

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

3.3.1. Effective flange width

Modular Ratio: Deck Concrete/Girder Concrete

$$n = E_b / E_d = 0.88$$

For Interior Girder:

$$b_l = \min \left\{ \begin{array}{l} 1/4 L_u \\ 12h_f + \max(0.5b_w, b_w) \end{array} \right. \Rightarrow n^* b_l = 2248.88861 \text{ (mm)}$$

For Exterior Girder:

$$b_E = 0.5b_l + \min \left\{ \begin{array}{l} 1/8 L_u \\ 6h_f + \max(0.5b_w, 0.25b) \end{array} \right. \Rightarrow n^* b_E = 2248.88861 \text{ (mm)}$$

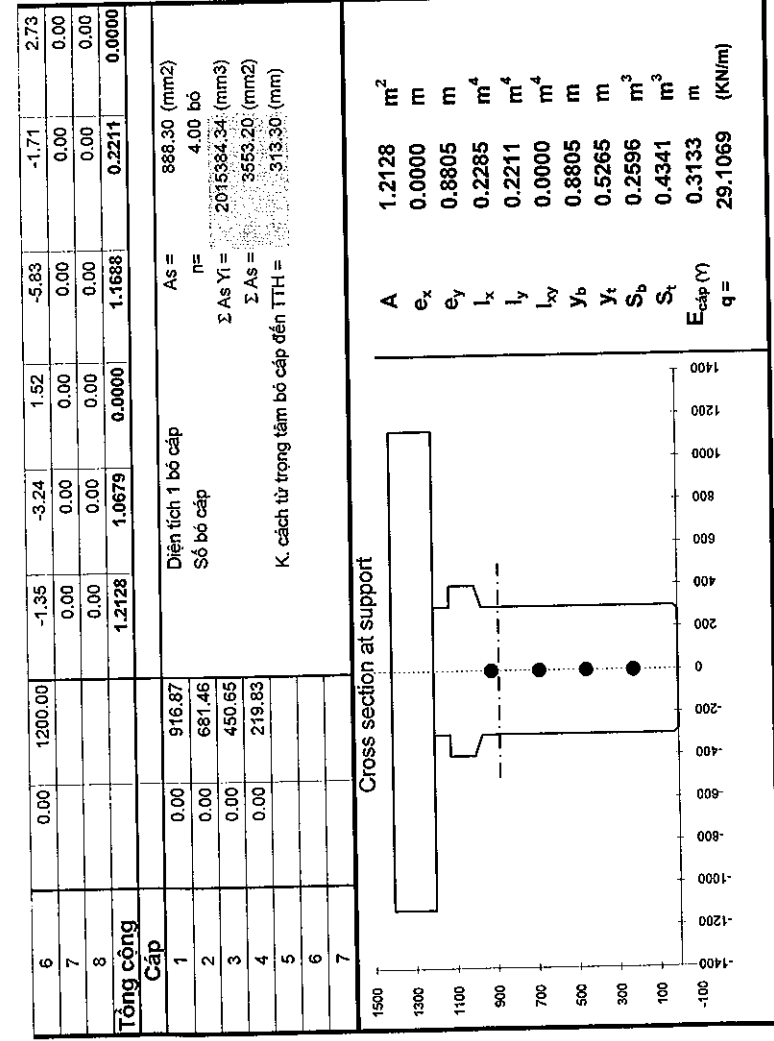
3.3.2. Property of Girder cross section in stage II (service stage):

Interior Girder:

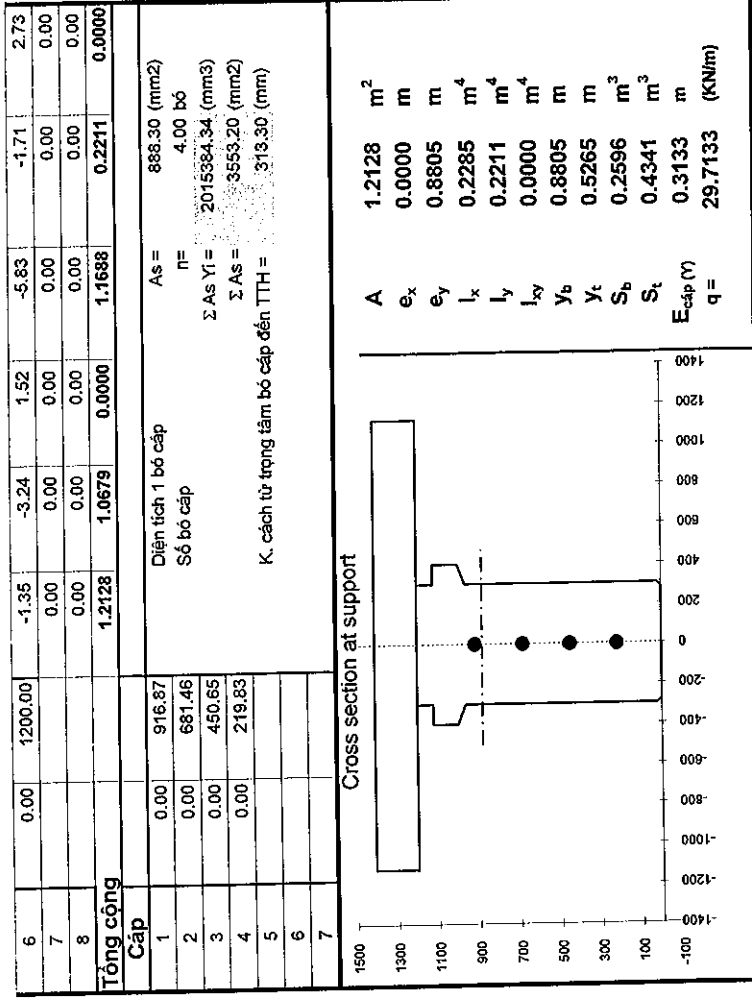
Áiãøm gõiç	x (mm)	y (mm)	A (m²)	Q _x ⁱ (m³)	Q _y ⁱ (m³)	I _x ⁱ (m⁴)	I _y ⁱ (m⁴)	I _{x'y} ⁱ (m⁴)
Đường bao	0.00	0.00	-	-	-	-	-	-
1	280.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	300.00	20.00	0.01	0.00	0.00	0.00	0.00	0.00
3	300.00	963.33	0.28	0.28	0.17	0.27	0.08	0.13
4	400.00	1000.00	-0.09	-0.17	-0.06	-0.25	-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	963.33	-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	1407.00	0.23	0.61	0.52	1.19	0.88	1.02
4	-1124.44	1407.00	3.16	8.90	0.00	18.79	4.00	0.00
5	-1124.44	1200.00	0.23	0.61	-0.52	1.19	0.88	-1.02

Exterior Girder:

Áiãøm gõiç	x (mm)	y (mm)	A (m²)	Q _x ⁱ (m³)	Q _y ⁱ (m³)	I _x ⁱ (m⁴)	I _y ⁱ (m⁴)	I _{x'y} ⁱ (m⁴)
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	963.33	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	963.33	-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	1407.00	0.23	0.61	0.52	1.19	0.88	1.02
4	-1124.44	1407.00	3.16	8.90	0.00	18.79	4.00	0.00
5	-1124.44	1200.00	0.23	0.61	-0.52	1.19	0.88	-1.02



Ái ãøm 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 _{xy'} (m ⁴)
Đường bao								
1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	280.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00
3	300.00	20.00	0.07	0.02	0.04	0.00	0.02	0.01
4	300.00	250.00	0.09	0.06	0.04	0.03	0.02	0.02
5	193.00	450.00	0.08	0.11	0.03	0.12	0.01	0.03
6	193.00	890.00	-0.16	-0.31	-0.10	-0.44	-0.04	-0.14
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	-193.00	890.00	-0.16	-0.31	0.10	-0.44	-0.04	0.14
15	-193.00	450.00	0.08	0.11	-0.03	0.12	0.01	-0.03
16	-300.00	250.00	0.09	0.06	-0.04	0.03	0.02	-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



Ái ãøm 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 _{xy'} (m ⁴)
Đường bao								
1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	280.00	0.00	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	300.00	450.00	0.09	0.06	0.04	0.03	0.02	0.02
5	193.00	890.00	0.08	0.11	0.03	0.12	0.01	0.03
6	400.00	1000.00	-0.16	-0.31	-0.10	-0.44	-0.04	-0.14
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	-193.00	890.00	-0.16	-0.31	0.10	-0.44	-0.04	0.14
15	-193.00	450.00	0.08	0.11	-0.03	0.12	0.01	-0.03
16	-300.00	250.00	0.09	0.06	-0.04	0.03	0.02	-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

[illegible]

Interior Girder:

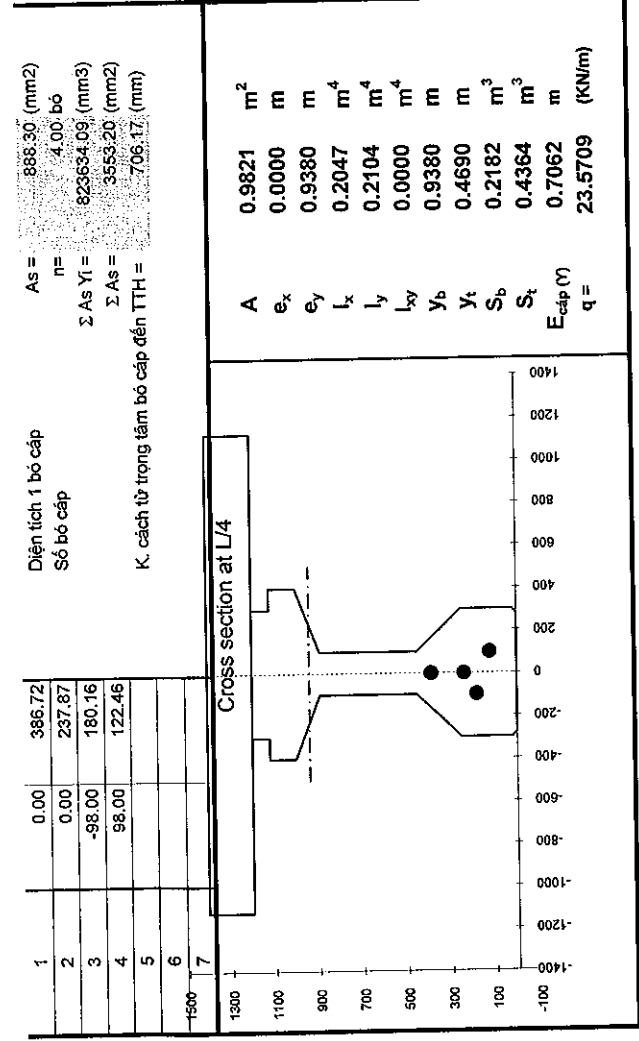
ĐẶC TRƯNG HÌNH HỌC - MẶT CẮT L/4

Âi/làm gối	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 ' (m ⁴)	I _y ' (m ⁴)	I _x ' (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	0,00	0,00
2	300,00	20,00	0,01	0,00	0,00	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	0,02	0,01	0,01
4	100,00	450,00	0,11	0,08	0,04	0,04	0,01	0,01	0,02	0,02
5	100,00	890,00	0,04	0,06	0,01	0,06	0,00	0,00	0,01	0,01
6	400,00	1000,00	-0,26	-0,48	-0,13	-0,69	-0,05	-0,18	-0,06	-0,18
7	400,00	1120,00	0,05	0,10	0,04	0,16	0,02	0,06	0,01	0,06
8	300,00	1120,00	0,11	0,25	0,08	0,42	0,04	0,13	0,03	0,13
9	300,00	1200,00	0,02	0,06	0,01	0,10	0,01	0,03	0,00	0,03
10	-300,00	1200,00	0,72	1,73	0,00	3,11	0,06	0,00	0,00	0,00
11	-300,00	1120,00	0,02	0,06	-0,01	0,10	0,01	-0,03	0,01	-0,03
12	-400,00	1120,00	0,11	0,25	-0,08	0,42	0,04	-0,13	0,04	-0,13
13	-400,00	1000,00	0,05	0,10	-0,04	0,16	0,02	-0,06	0,02	-0,06
14	-100,00	890,00	-0,26	-0,48	-0,13	-0,69	-0,05	-0,18	-0,05	-0,18
15	-100,00	450,00	0,04	0,06	-0,01	0,06	0,00	-0,01	0,00	-0,01
16	-300,00	250,00	0,11	0,08	-0,04	0,04	0,01	-0,02	0,01	-0,02
17	-300,00	20,00	0,07	0,02	-0,04	0,00	0,02	-0,01	0,02	-0,01
18	-280,00	0,00	0,01	0,00	0,00	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	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	0,00	0,00
2	1124,44	1200,00	-1,35	-3,24	-1,52	-5,83	-1,71	-2,73	-1,71	-2,73
3	1124,44	1407,00	0,23	0,61	0,52	1,19	0,88	1,02	0,88	1,02
4	-1124,44	1407,00	3,16	8,90	0,00	18,79	4,00	0,00	4,00	0,00
5	-1124,44	1200,00	0,23	0,61	-0,52	1,19	0,88	-1,02	0,88	-1,02
6	0,00	1200,00	-1,35	-3,24	1,52	-5,83	-1,71	2,73	-1,71	2,73
7										
8										
Tổng cộng			0,9821	0,9212	0,0000	1,0687	0,2104	0,0000	0,2104	0,0000
Cáp										

Exterior Girder:

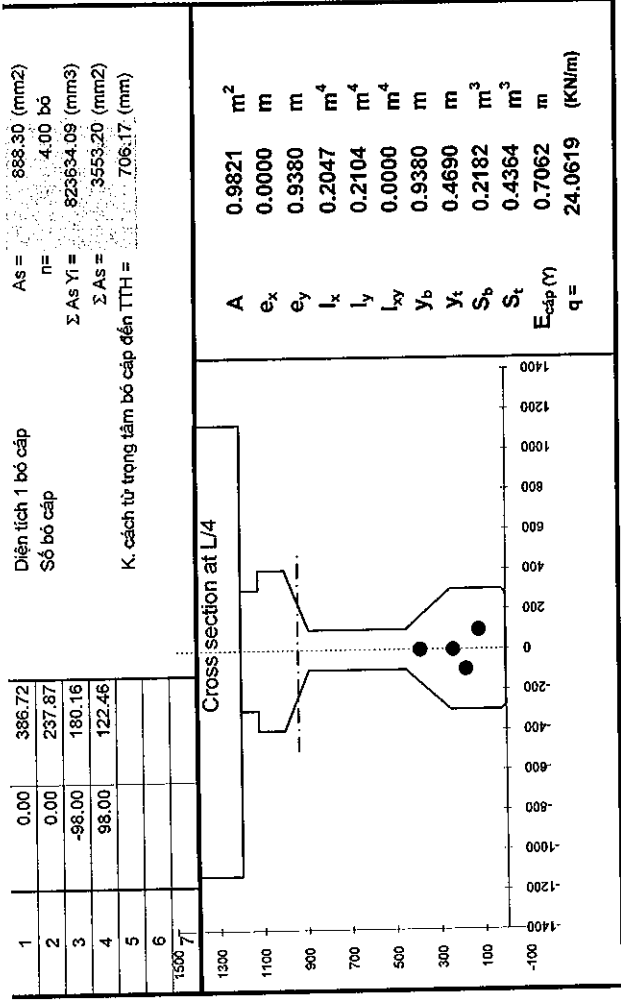
ĐẶC TRƯNG HÌNH HỌC - MẶT CẮT L/4

Âi/làm gối	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 ' (m ⁴)	I _y ' (m ⁴)	I _x ' (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	0,00	0,00
2	300,00	20,00	0,01	0,00	0,00	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	0,02	0,01	0,01
4	100,00	450,00	0,11	0,08	0,04	0,04	0,01	0,01	0,02	0,02
5	100,00	890,00	0,04	0,06	0,01	0,06	0,00	0,00	0,01	0,01
6	400,00	1000,00	-0,26	-0,48	-0,13	-0,69	-0,05	-0,18	-0,06	-0,18
7	400,00	1120,00	0,05	0,10	0,04	0,16	0,02	0,06	0,01	0,06
8	300,00	1120,00	0,11	0,25	0,08	0,42	0,04	0,13	0,03	0,13
9	300,00	1200,00	0,02	0,06	0,01	0,10	0,01	0,03	0,00	0,03
10	-300,00	1200,00	0,72	1,73	0,00	3,11	0,06	0,00	0,00	0,00
11	-300,00	1120,00	0,02	0,06	-0,01	0,10	0,01	-0,03	0,01	-0,03
12	-400,00	1120,00	0,11	0,25	-0,08	0,42	0,04	-0,13	0,04	-0,13
13	-400,00	1000,00	0,05	0,10	-0,04	0,16	0,02	-0,06	0,02	-0,06
14	-100,00	890,00	-0,26	-0,48	-0,13	-0,69	-0,05	-0,18	-0,05	-0,18
15	-100,00	450,00	0,04	0,06	-0,01	0,06	0,00	-0,01	0,00	-0,01
16	-300,00	250,00	0,11	0,08	-0,04	0,04	0,01	-0,02	0,01	-0,02
17	-300,00	20,00	0,07	0,02	-0,04	0,00	0,02	-0,01	0,02	-0,01
18	-280,00	0,00	0,01	0,00	0,00	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	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	0,00	0,00
2	1124,44	1200,00	-1,35	-3,24	-1,52	-5,83	-1,71	-2,73	-1,71	-2,73
3	1124,44	1407,00	0,23	0,61	0,52	1,19	0,88	1,02	0,88	1,02
4	-1124,44	1407,00	3,16	8,90	0,00	18,79	4,00	0,00	4,00	0,00
5	-1124,44	1200,00	0,23	0,61	-0,52	1,19	0,88	-1,02	0,88	-1,02
6	0,00	1200,00	-1,35	-3,24	1,52	-5,83	-1,71	2,73	-1,71	2,73
7										
8										
Tổng cộng			0,9821	0,9212	0,0000	1,0687	0,2104	0,0000	0,2104	0,0000
Cáp										



ĐẶC TRƯNG HÌNH HỌC - MẶT CẮT 3L/8

Ái-lô-m-gi-ơc	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	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	-300.00	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	-400.00	890.00	-0.26	-0.48	0.13	-0.69	-0.05	0.18
15	-100.00	890.00	0.04	0.06	-0.01	0.06	0.00	-0.01
16	-300.00	450.00	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								

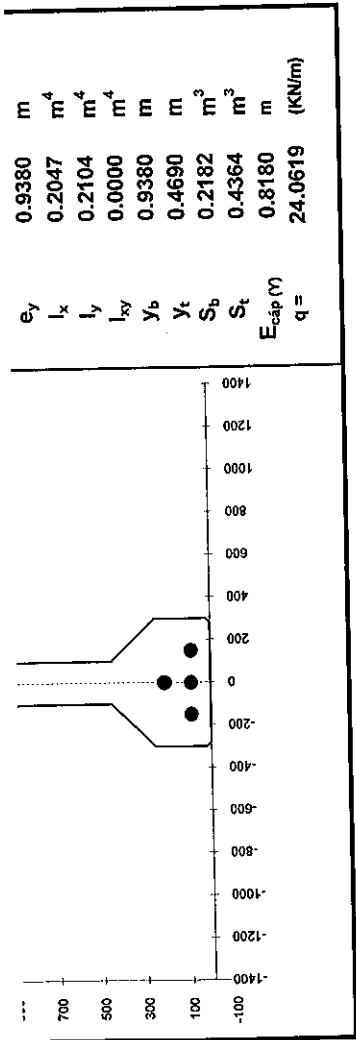
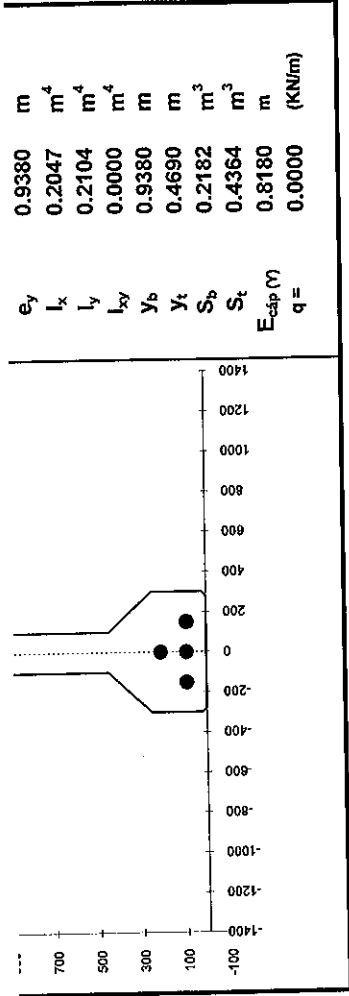


ĐẶC TRƯNG HÌNH HỌC - MẶT CẮT 3L/8

Ái-lô-m-gi-ơc	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	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	-300.00	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	-400.00	890.00	-0.26	-0.48	0.13	-0.69	-0.05	0.18
15	-100.00	890.00	0.04	0.06	-0.01	0.06	0.00	-0.01
16	-300.00	450.00	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								

3	300.00	250.00	0.07	0.02	0.04	0.00	0.02	0.01	0.01
4	100.00	450.00	0.11	0.08	0.04	0.04	0.01	0.02	0.02
5	100.00	890.00	0.04	0.06	0.01	0.06	0.00	0.01	0.01
6	400.00	1000.00	-0.26	-0.48	-0.13	-0.69	-0.05	-0.18	-0.18
7	400.00	1120.00	0.05	0.10	0.04	0.16	0.02	0.06	0.06
8	300.00	1120.00	0.11	0.25	0.08	0.42	0.04	0.13	0.13
9	300.00	1200.00	0.02	0.06	0.01	0.10	0.01	0.03	0.03
10	-300.00	1200.00	0.72	1.73	0.00	3.11	0.06	0.00	0.00
11	-300.00	1120.00	0.02	0.06	-0.01	0.10	0.01	-0.03	-0.03
12	-400.00	1120.00	0.11	0.25	-0.08	0.42	0.04	-0.13	-0.13
13	-400.00	1000.00	0.05	0.10	-0.04	0.16	0.02	-0.06	-0.06
14	-100.00	890.00	-0.26	-0.48	0.13	-0.69	-0.05	0.18	0.18
15	-100.00	450.00	0.04	0.06	-0.01	0.06	0.00	-0.01	-0.01
16	-300.00	250.00	0.11	0.08	-0.04	0.04	0.01	-0.02	-0.02
17	-300.00	20.00	0.07	0.02	-0.04	0.00	0.02	-0.01	-0.01
18	-280.00	0.00	0.01	0.00	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	0.00
20									
21									
22									
23									
24									
25									
26									
27									
28									
29									
30									
31									
32									
33									
1	0.00	1200.00	0.00	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	-2.73
3	1124.44	1407.00	0.23	0.61	0.52	1.19	0.88	1.02	1.02
4	-1124.44	1407.00	3.16	8.90	0.00	18.79	4.00	0.00	0.00
5	-1124.44	1200.00	0.23	0.61	-0.52	1.19	0.88	-1.02	-1.02
6	0.00	1200.00	-1.35	-3.24	1.52	-5.83	-1.71	2.73	2.73
7									
8									
Tổng cộng									
Cáp									
1	0.00	210.00	Diện tích 1 bó cáp		As =		888.30 (mm2)		
2	0.00	90.00	Số bó cáp		n=		4.00 bó		
3	-150.00	90.00			Σ As Yi =		426384.00 (mm3)		
4	150.00	90.00			Σ As =		3553.20 (mm2)		
5	0.00	0.00			K. cách từ trọng tâm bó cáp đến TTH =		817.97 (mm)		
1500 - 6									
1300									
1100									
Cross section at L/2									
A 0.9821 m ² 0.0000 m									

3	300.00	250.00	0.07	0.02	0.04	0.00	0.02	0.01	0.01
4	100.00	450.00	0.11	0.08	0.04	0.04	0.01	0.02	0.02
5	100.00	890.00	0.04	0.06	0.01	0.06	0.00	0.01	0.01
6	400.00	1000.00	-0.26	-0.48	-0.13	-0.69	-0.05	-0.18	-0.18
7	400.00	1120.00	0.05	0.10	0.04	0.16	0.02	0.06	0.06
8	300.00	1120.00	0.11	0.25	0.08	0.42	0.04	0.13	0.13
9	300.00	1200.00	0.02	0.06	0.01	0.10	0.01	0.03	0.03
10	-300.00	1200.00	0.72	1.73	0.00	3.11	0.06	0.00	0.00
11	-300.00	1120.00	0.02	0.06	-0.01	0.10	0.01	-0.03	-0.03
12	-400.00	1120.00	0.11	0.25	-0.08	0.42	0.04	-0.13	-0.13
13	-400.00	1000.00	0.05	0.10	-0.04	0.16	0.02	-0.06	-0.06
14	-100.00	890.00	-0.26	-0.48	0.13	-0.69	-0.05	0.18	0.18
15	-100.00	450.00	0.04	0.06	-0.01	0.06	0.00	-0.01	-0.01
16	-300.00	250.00	0.11	0.08	-0.04	0.04	0.01	-0.02	-0.02
17	-300.00	20.00	0.07	0.02	-0.04	0.00	0.02	-0.01	-0.01
18	-280.00	0.00	0.01	0.00	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	0.00
20									
21									
22									
23									
24									
25									
26									
27									
28									
29									
30									
31									
32									
33									
1	0.00	1200.00	0.00	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	-2.73
3	1124.44	1407.00	0.23	0.61	0.52	1.19	0.88	1.02	1.02
4	-1124.44	1407.00	3.16	8.90	0.00	18.79	4.00	0.00	0.00
5	-1124.44	1200.00	0.23	0.61	-0.52	1.19	0.88	-1.02	-1.02
6	0.00	1200.00	-1.35	-3.24	1.52	-5.83	-1.71	2.73	2.73
7									
8									
Tổng cộng									
Cáp			0.9821	0.9212	0.0000	1.0687	0.2104	0.0000	0.0000
1	0.00	210.00	Diện tích 1 bó cáp		As =		888.30 (mm2)		
2	0.00	90.00	Số bó cáp		n=		4.00 bó		
3	-150.00	90.00			Σ As Yi =		426384.00 (mm3)		
4	150.00	90.00			Σ As =		3553.20 (mm2)		
5	0.00	0.00			K. cách từ trọng tâm bó cáp đến TTH =		817.97 (mm)		
1500									
7									
Cross-section at L/2									
1300									
1100									
A 0.9821 m ² 0.0000 m									



4. LOSS OF PRESTRESS

4.1 Material:

Concrete:

Girder concrete strength at the 28 age days	$f_c =$	45.00 MPa	
Unit weight of concrete	$\gamma_c =$	2450.00 kg/m ³	
Modulus of elasticity	$E_c = 0.043 \gamma_c^{1.5} \sqrt{f_c} =$	34980.32 MPa	(5.4.2.4-1)
Concrete strength at transfer	$f_{ci}' = 0.9 f_c =$	40.50 MPa	
Compression stress Limit at transfer	$0.6 f_{ci}' =$	24.30 MPa	(5.9.4.1.1)
Tension stress Limit at transfer	$0.25 \sqrt{f_{ci}'} =$	1.59 MPa < (=) 1.38	(5.9.4.1.2)
	\Rightarrow	1.38 MPa	

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 elasticity	$E_p =$	195000.00 MPa	
Wobble friction coefficient (mm-1)	$K =$	6.60E-07 mm ⁻¹	
Coefficient of friction (1/RAD)	$\mu =$	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	$f_{pj} = 0.75 f_{pu} =$	1395.00 MPa	
Jacking force for one tendon	$P_j =$	1239.18 kN	
Anchorage set	$\Delta L =$	6.00 mm	
Area of one duck	$A_g =$	3318.31 mm ²	

4.1 Loss of prestressing force immediately (Instantaneous losses):

4.1.1 Friction between Prestressing Tendon and Duck:

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

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

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	200.00	0.0035	1.41	0.0026	1.10	0.0017	0.79	0.0008	0.48	0.95
L/8	2737.50	0.0518	20.44	0.0387	15.91	0.0255	11.37	0.0123	6.81	13.63
L/4	5275.00	0.1449	54.30	0.1081	41.91	0.0713	29.41	0.0345	16.79	35.60
3L/8	7812.50	0.2827	101.88	0.2109	78.46	0.1391	54.61	0.0673	30.34	66.32
L/2	10350.00	0.4653	161.66	0.3472	124.67	0.2290	86.58	0.1108	47.35	105.07

4.1.2 Anchorage seating or Set:

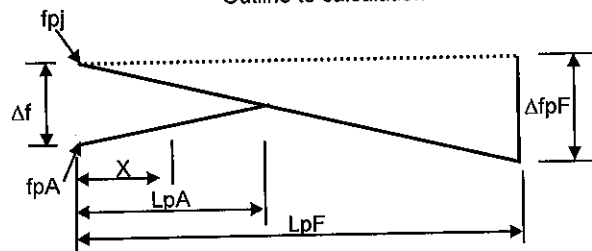
Formula

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

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

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

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 length from anchorage to 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 (L_{pF}) and calculation follow:

Tendon no.1	X_i (mm)	Δf_{pA} (MPa)
$L_{pF} =$	10350	0
$\Delta f_{pF} =$	161.66	200
$L_{pA} =$	8654.9	2738
$\Delta f =$	270.37	5275
	7813	26.32
	10350	0.00

Tendon no.2	X_i (mm)	Δf_{pA} (MPa)
$L_{pF} =$	10350	0
$\Delta f_{pF} =$	124.67	200
$L_{pA} =$	9855.4	2738
$\Delta f =$	237.43	5275
	7813	49.22
	10350	0.00

Tendon no.3	X_i (mm)	Δf_{pA} (MPa)
$L_{pF} =$	10350	0
$\Delta f_{pF} =$	86.58	200
$L_{pA} =$	10350.0	2738

Tendon no.4	X_i (mm)	Δf_{pA} (MPa)
$L_{pF} =$	10350	0
$\Delta f_{pF} =$	47.35	200
$L_{pA} =$	10350.0	2738

$\Delta f =$	173.18	5275	84.91
		7813	42.45
		10350	0.00

$\Delta f =$	94.69	5275	46.43
		7813	23.22
		10350	0.00

4.1.3 Elastic deformation of concrete:

Formula

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

In which:

Number of tendon

N = 4.00 (Tendon)

Cable modulus of elasticity

E_p = 195000.0 MPa

Concrete strength at transfer

f_{cl} = 40.50 MPa

Unit weight of concrete

γ_c = 2450.00 kg/m³

Concrete modulus of elasticity at transfer

E_{cl} = 33185.3 MPa

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

$$f_{cgp} = \frac{F_I}{A} + \frac{F_J e^2}{I_x} - \frac{M_{DC} e}{I_x}$$

Compression force due to prestressing consider loss stress:

$$F_J = N \cdot f_{pj} \cdot A_s - A_s \cdot \Sigma (\Delta f_{pFi} + \Delta f_{pAi})$$

A Area of girder cross section

I_x 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 (mm)	Tendon1 $\Delta f_{pF} + \Delta f_{pA}$ (MPa)	Tendon2 $\Delta f_{pF} + \Delta f_{pA}$ (MPa)	Tendon3 $\Delta f_{pF} + \Delta f_{pA}$ (MPa)	Tendon4 $\Delta f_{pF} + \Delta f_{pA}$ (MPa)	SUM $\Sigma(\Delta f_{pF} + \Delta f_{pA})$ (MPa)	ΣF_j (kN)
		(MPa)	(MPa)	(MPa)	(MPa)	(MPa)	(kN)
Anchorage	0	270.37	237.43	173.16	94.69	775.66	4267.70
Support	200	265.53	233.72	170.61	93.34	763.20	4278.77
L/8	2738	205.29	187.40	138.73	76.46	607.88	4416.74
L/4	5275	159.89	152.26	114.32	63.22	489.69	4521.73
3L/8	7813	128.20	127.68	97.07	53.55	406.50	4595.62
L/2	10350	161.66	124.67	86.58	47.35	420.26	4583.40

Loss stress due to Elastic deformation of concrete

Section	Xi (mm)	Fj (kN)	A (mm ²)	Ix (mm ⁴)	e (mm)	M _{DC} (kNm)	f _{cgp} (MPa)	Δf_{ES} (MPa)
Anchorage	0	4267.70	7.5E+05	9.2E+10	49.79	0.00	5.83	12.84
Support	200	4278.77	7.5E+05	9.2E+10	49.79	0.00	5.84	12.87
L/8	2738	4416.74	6.3E+05	8.8E+10	243.55	332.26	9.09	20.03
L/4	5275	4521.73	5.2E+05	8.5E+10	376.78	569.59	13.79	30.40
3L/8	7813	4595.62	5.2E+05	8.5E+10	460.63	711.99	16.53	36.43
L/2	10350	4583.40	5.2E+05	8.5E+10	488.58	759.45	17.40	38.35

Total loss of prestressing force immediately - Remaining prestressing force:

Tendon1	Xi (mm)	Δf_{pF} (MPa)	Δf_{pA} (MPa)	Δf_{ES} (MPa)	$\Sigma \Delta$ (MPa)	F _j ¹ (kN)	(α) (rad)	F _j ¹ *Cos(α) (kN)	F _j ¹ *Sin(α) (kN)
Section	(mm)	(MPa)	(MPa)	(MPa)	(MPa)	(kN)	(rad)	(kN)	(kN)
anchorage	0	0.00	270.37	12.84	283.20	987.61	0.1806	971.54	177.41
Support	200	1.41	264.12	12.87	278.40	991.87	0.1772	976.34	174.84
L/8	2738	20.44	184.85	20.03	225.32	1039.02	0.1335	1029.78	138.31
L/4	5275	54.30	105.58	30.40	190.28	1070.15	0.0893	1065.88	95.44
3L/8	7813	101.88	26.32	36.43	164.63	1092.94	0.0447	1091.85	48.88
L/2	10350	161.66	0.00	38.35	200.01	1061.51	0.0000	1061.51	0.00

Tendon2	Xi (mm)	Δf_{pF} (MPa)	Δf_{pA} (MPa)	Δf_{ES} (MPa)	$\Sigma \Delta$ (MPa)	F _j ² (kN)	(α) (rad)	F _j ² *Cos(α) (kN)	F _j ² *Sin(α) (kN)
Section	(mm)	(MPa)	(MPa)	(MPa)	(MPa)	(kN)	(rad)	(kN)	(kN)
anchorage	0	0.00	237.43	12.84	250.27	1016.86	0.1354	1007.56	137.26
Support	200	1.10	232.61	12.87	246.59	1020.13	0.1328	1011.15	135.09
L/8	2738	15.91	171.48	20.03	207.43	1054.92	0.0999	1049.67	105.18
L/4	5275	41.91	110.35	30.40	182.66	1076.92	0.0667	1074.53	71.78
3L/8	7813	78.46	49.22	36.43	164.11	1093.40	0.0334	1092.79	36.50
L/2	10350	124.67	0.00	38.35	163.02	1094.36	0.0000	1094.36	0.00

Tendon3	Xi (mm)	Δf_{pF} (MPa)	Δf_{pA} (MPa)	Δf_{ES} (MPa)	$\Sigma \Delta$ (MPa)	F _j ³ (kN)	(α) (rad)	F _j ³ *Cos(α) (kN)	F _j ³ *Sin(α) (kN)
Section	(mm)	(MPa)	(MPa)	(MPa)	(MPa)	(kN)	(rad)	(kN)	(kN)
anchorage	0	0.00	173.16	11.41	184.58	1075.22	0.0896	1070.90	96.23
Support	200	0.79	169.82	11.44	182.05	1077.46	0.0879	1073.31	94.58
L/8	2738	11.37	127.36	17.80	156.54	1100.13	0.0660	1097.73	72.55
L/4	5275	29.41	84.91	27.02	141.34	1113.63	0.0440	1112.55	49.02
3L/8	7813	54.61	42.45	32.38	129.45	1124.19	0.0220	1123.92	24.76
L/2	10350	86.58	0.00	34.09	120.67	1131.99	0.0000	1131.99	0.00

Tendon4	Xi (mm)	Δf_{pF} (MPa)	Δf_{pA} (MPa)	Δf_{ES} (MPa)	$\Sigma \Delta$ (MPa)	F _j ⁴ (kN)	(α) (rad)	F _j ⁴ *Cos(α) (kN)	F _j ⁴ *Sin(α) (kN)
Section	(mm)	(MPa)	(MPa)	(MPa)	(MPa)	(kN)	(rad)	(kN)	(kN)
anchorage	0	0.00	94.69	8.56	103.25	1147.46	0.0435	1146.38	49.84
Support	200	0.48	92.86	8.58	101.92	1148.64	0.0426	1147.60	48.93
L/8	2738	6.81	69.65	13.35	89.81	1159.40	0.0320	1158.81	37.06
L/4	5275	16.79	46.43	20.26	83.48	1165.02	0.0213	1164.76	24.83
3L/8	7813	30.34	23.22	24.28	77.84	1170.04	0.0107	1169.97	12.47
L/2	10350	47.35	0.00	25.57	72.91	1174.41	0.0000	1174.41	0.00

SUM 1to4	Xi	ΣF_j	$F_j \cdot \cos(\alpha)$	$F_j \cdot \sin(\alpha)$	$\theta_{cáp}$	$M_j = \Sigma F_j \cos(\alpha) \cdot \theta_{cáp}$
Section	(mm)	(kN)	(kN)	(kN)	(mm)	(kNm)
anchorage	0	4227.15	4196.38	460.74	49.79	208.94
Support	0	4238.11	4208.39	453.44	49.79	209.54
L/8	0	4353.47	4335.98	353.09	243.55	1056.04
L/4	0	4425.72	4417.72	241.07	376.78	1664.52
3L/8	0	4480.57	4478.53	122.61	460.63	2062.96
L/2	0	4462.27	4462.27	0.00	488.58	2180.19

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

4.2.1 Loss of prestress due to Shrinkage:

Formula: $\Delta f_{SH} = (93 - 0.85 \cdot H)$
Relative humidity of environment $H = 80.00 \%$
 $\Delta f_{SH} = 25.00 \text{ (MPa)}$

4.2.2 Loss of prestress due to Creep:

Formula: $\Delta f_{CR} = 12.0 f_{cgp} - 7.0 \cdot \Delta f_{cdp}$
In which:
 f_{cgp} Stress in concrete at tendons centroid (f_{cgp}) due to prestressing tendon and self weigh of girder
 Δf_{cdp} Stress at tendons centroid changes due to permanent load, except dead load action at transfer

Section	Xi	Interior Girder			Exterior Girder	
		f_{cgp}	Δf_{cdp}	Δf_{PCR}	Δf_{cdp}	Δf_{PCR}
	(m)	(MPa)	(MPa)	(MPa)	(MPa)	(MPa)
Support	0.00	5.84	0.00	70.10	0.00	70.10
L/8	2.54	9.09	0.84	103.18	1.49	98.67
L/4	5.08	13.79	4.28	135.56	3.92	138.10
3L/8	7.61	16.53	4.67	165.66	5.83	157.57
L/2	10.15	17.40	7.18	158.59	6.55	162.99

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
 f_{py} : Specified yeild strength of prestressing steel $f_{py} = 1674.00 \text{ (MPa)}$
 f_{pj} : Initial stress in the tendon at the end of stressing

Section	Xi	f_{pj}	Δf_{PR1}
	(m)	(MPa)	(MPa)
Support	0.00	1382.13	0.00
L/8	2.54	1374.97	0.00
L/4	5.08	1364.60	0.00
3L/8	7.61	1358.57	0.00
L/2	10.15	1356.65	0.00

(b) After Transfer:

Formula: $\Delta f_{PR2} = 30\% \cdot (138 - 0.3 \Delta f_{pF} - 0.4 \Delta f_{ES} - 0.2 (\Delta f_{pSH} + \Delta f_{PCR}))$

Interior Girder						
Section	Xi	Δf_{pF}	Δf_{pES}	Δf_{pSH}	Δf_{PCR}	Δf_{PR2}
	(m)	(MPa)	(MPa)	(MPa)	(MPa)	(MPa)
Support	0.00	0.95	12.87	25.00	17.52	37.22
L/8	2.54	13.63	20.03	25.00	25.79	34.72
L/4	5.08	35.60	30.40	25.00	33.89	31.01
3L/8	7.61	66.32	36.43	25.00	41.41	27.07
L/2	10.15	105.07	38.35	25.00	39.65	23.46

Exterior Girder						
Section	Xi	Δf_{pF}	Δf_{pES}	Δf_{pSH}	Δf_{PCR}	Δf_{PR2}
	(m)	(MPa)	(MPa)	(MPa)	(MPa)	(MPa)
Support	0.00	0.95	12.87	25.00	17.52	37.22
L/8	2.54	13.63	20.03	25.00	24.67	34.79
L/4	5.08	35.60	30.40	25.00	34.53	30.98
3L/8	7.61	66.32	36.43	25.00	39.39	27.20
L/2	10.15	105.07	38.35	25.00	40.75	23.40

TOTAL LOSS STRESS AT SERVICE STAGE

Interior Girder						
Section	Xi	Δf_{pSH}	Δf_{pCR}	Δf_{pR1}	Δf_{pR2}	Sum
	(m)	(MPa)	(MPa)	(MPa)	(MPa)	(MPa)
Support	0.00	25.00	70.10	0.00	37.22	132.32
L/8	2.54	25.00	103.18	0.00	34.72	162.90
L/4	5.08	25.00	135.56	0.00	31.01	191.57
3L/8	7.61	25.00	165.66	0.00	27.07	217.73
L/2	10.15	25.00	158.59	0.00	23.46	207.06

Exterior Girder						
Section	Xi	Δf_{pSH}	Δf_{pCR}	Δf_{pR1}	Δf_{pR2}	Sum
	(m)	(MPa)	(MPa)	(MPa)	(MPa)	(MPa)
Support	0.00	25.00	70.10	0.00	37.22	132.32
L/8	2.54	25.00	98.67	0.00	34.79	158.46
L/4	5.08	25.00	138.10	0.00	30.98	194.08
3L/8	7.61	25.00	157.57	0.00	27.20	209.76
L/2	10.15	25.00	162.99	0.00	23.40	211.39

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

Tendon1	Xi	$\Sigma \Delta_{pT}$	F_j^1	(α)	$F_j^1 \cdot \cos(\alpha)$	$F_j^1 \cdot \sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	410.72	874.33	0.1772	860.64	154.12
L/8	2.54	388.22	894.32	0.1335	886.36	119.05
L/4	5.08	381.86	899.97	0.0893	896.39	80.26
3L/8	7.61	382.36	899.53	0.0447	898.63	40.23
L/2	10.15	407.06	877.58	0.0000	877.58	0.00

Tendon2	Xi	$\Sigma \Delta_{pT}$	F_j^2	(α)	$F_j^2 \cdot \cos(\alpha)$	$F_j^2 \cdot \sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	378.91	902.60	0.1354	894.34	121.84
L/8	2.54	370.32	910.22	0.1328	902.20	120.53
L/4	5.08	374.23	906.75	0.0999	902.23	90.40
3L/8	7.61	381.84	899.99	0.0667	897.99	59.99
L/2	10.15	370.08	910.44	0.0334	909.93	30.39

Tendon3	Xi	$\Sigma \Delta_{pT}$	F_j^3	(α)	$F_j^3 \cdot \cos(\alpha)$	$F_j^3 \cdot \sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	316.89	957.68	0.0879	953.99	84.06
L/8	2.54	344.95	932.76	0.0660	930.73	61.51
L/4	5.08	348.11	929.95	0.0440	929.05	40.93
3L/8	7.61	359.07	920.22	0.0220	919.99	20.27
L/2	10.15	336.51	940.26	0.0000	940.26	0.00

Tendon4	Xi	$\Sigma \Delta_{pT}$	F_j^4	(α)	$F_j^4 \cdot \cos(\alpha)$	$F_j^4 \cdot \sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	235.57	1029.92	0.0426	1028.99	43.87
L/8	2.54	264.82	1003.94	0.0320	1003.42	32.09
L/4	5.08	281.38	989.23	0.0213	989.00	21.08
3L/8	7.61	301.22	971.61	0.0107	971.55	10.36
L/2	10.15	284.89	986.11	0.0000	986.11	0.00

SUM 1to4	Xi	ΣF_j	$F_j \cdot \cos(\alpha)$	$V_p = F_j \cdot \sin(\alpha)$	e_{cable}	$M_j = \Sigma F_j \cos(\alpha) \cdot e_{cap}$
Section	(mm)	(kN)	(kN)	(kN)	(m)	(kNm)
Support	0.00	3764.53	3737.95	403.90	0.05	186.1
L/8	0.00	3741.24	3722.72	333.18	0.24	906.7
L/4	0.00	3725.90	3716.67	232.68	0.38	1400.4
3L/8	0.00	3691.34	3688.16	130.84	0.46	1698.9
L/2	0.00	3714.39	3713.88	30.39	0.49	1814.5

Exterior Girder

Tendon1	Xi	$\Sigma\Delta_{PT}$	F_j^1	(α)	$F_j^1 \cdot \cos(\alpha)$	$F_j^1 \cdot \sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	410.72	874.33	0.1772	860.64	154.12
L/8	0.00	383.78	898.26	0.1335	890.27	119.57
L/4	0.00	384.36	897.75	0.0893	894.17	80.06
3L/8	0.00	374.39	906.61	0.0447	905.70	40.55
L/2	0.00	411.40	873.74	0.0000	873.74	0.00

Tendon2	Xi	$\Sigma\Delta_{PT}$	F_j^2	(α)	$F_j^2 \cdot \cos(\alpha)$	$F_j^2 \cdot \sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	378.91	902.60	0.1354	894.34	121.84
L/8	0.00	365.88	914.16	0.1328	906.11	121.06
L/4	0.00	376.74	904.52	0.0999	900.01	90.18
3L/8	0.00	373.87	907.07	0.0667	905.05	60.46
L/2	0.00	374.41	906.59	0.0334	906.08	30.26

Tendon3	Xi	$\Sigma\Delta_{PT}$	F_j^3	(α)	$F_j^3 \cdot \cos(\alpha)$	$F_j^3 \cdot \sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	316.89	957.68	0.0879	953.99	84.06
L/8	0.00	340.51	936.71	0.0660	934.67	61.77
L/4	0.00	350.61	927.73	0.0440	926.83	40.84
3L/8	0.00	351.10	927.30	0.0220	927.07	20.42
L/2	0.00	340.84	936.41	0.0000	936.41	0.00

Tendon4	Xi	$\Sigma\Delta_{PT}$	F_j^4	(α)	$F_j^4 \cdot \cos(\alpha)$	$F_j^4 \cdot \sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	235.57	1029.92	0.0426	1028.99	43.87
L/8	0.00	260.38	1007.88	0.0320	1007.37	32.21
L/4	0.00	283.89	987.00	0.0213	986.78	21.04
3L/8	0.00	293.25	978.69	0.0107	978.63	10.43
L/2	0.00	289.23	982.26	0.0000	982.26	0.00

SUM 1to4	Xi	ΣF_j	$F_j \cdot \cos(\alpha)$	$V_p = F_j \cdot \sin(\alpha)$	e_{caplo}	$M_i = \Sigma F_j \cdot \cos(\alpha) \cdot e_{cap}$
Section	(mm)	(kN)	(kN)	(kN)	(m)	(kNm)
Support	0.00	3764.53	3737.95	403.90	0.05	186.1
L/8	0.00	3757.02	3738.42	334.61	0.24	910.5
L/4	0.00	3717.00	3707.79	232.12	0.38	1397.0
3L/8	0.00	3719.67	3716.46	131.86	0.46	1711.9
L/2	0.00	3699.00	3698.49	30.26	0.49	1807.0

5. FIBRE STRESS CHECK:

Formula:

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

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

Concrete strength at transfer $f_{ci}' = 0.9 f_c = 40.50 \text{ MPa}$

Compression stress Limit at transfer $0.6 f_{ci}' = 24.30 \text{ MPa}$

Tension stress Limit at transfer $0.25 \text{ SQRT}(f_{ci}') < 1.38 = -1.38 \text{ MPa}$

(5.9.4.1.2-1)

Setion	Xi	A	St	Sb	F _i *Cos(α)	e	M _{DC}	f _{ti}	f _{bi}	Kiểm tra	
	(mm)	(mm ²)	(mm ³)	(mm ³)	(kN)	(mm)	(kNm)	(MPa)	(MPa)	f _{ti}	f _{bi}
irder en	0	7.47E+05	1.57E+08	1.49E+08	4196.38	49.79	0.00	4.29	7.02	OK	OK
Support	200	7.47E+05	1.57E+08	1.49E+08	4208.39	49.79	0.00	4.30	7.04	OK	OK
L/8	2738	6.27E+05	1.51E+08	1.44E+08	4335.98	243.55	332.26	2.12	11.95	OK	OK
L/4	5275	5.17E+05	1.43E+08	1.39E+08	4417.72	376.78	569.59	0.91	16.41	OK	OK
3L/8	7813	5.17E+05	1.43E+08	1.39E+08	4478.53	460.63	711.99	-0.76	18.37	OK	OK
L/2	10350	5.17E+05	1.43E+08	1.39E+08	4462.27	488.58	759.45	-1.28	18.84	OK	OK

5.2 Stress check during construction the deck:

5.2.1 Increase load:

Exterior Diaphragms beam $DC_{dn1} = 36.31 \text{ (kN)}$

Interior Diaphragms beam $DC_{dn1} = 8.31 \text{ (kN)}$

Precast plank $DC_{VK} = 3.82 \text{ (kN/m)}$

Wet concrete of deck $DC_{mc} = 13.58 \text{ (kN/m)}$

5.2.2 Stress check:

Compression strength of concrete $f_c = 45.00 \text{ MPa}$

Compression stress limit $0.45 f_c = 20.25 \text{ MPa} \quad (5.9.4.2.1-1)$

Tension stress limit $0.5 \text{ SQRT}(f_c) = -3.35 \text{ MPa} \quad (5.9.4.2.2-1)$

Setion	Xi	A	St	Sb	Fi	e	M _{DC}	f _{ti}	f _{bi}	Kiểm tra	
	(mm)	(mm ²)	(mm ³)	(mm ³)	(kN)	(mm)	(kNm)	(MPa)	(MPa)	f _{ti}	f _{bi}
irder en	0	7.47E+05	1.57E+08	1.49E+08	4196.38	49.79	0.00	4.29	7.02	OK	OK
Support	200	7.47E+05	1.57E+08	1.49E+08	4208.39	49.79	0.00	4.30	7.04	OK	OK
L/8	2738	6.27E+05	1.51E+08	1.44E+08	4335.98	243.55	1039.51	6.80	7.03	OK	OK
L/4	5275	5.17E+05	1.43E+08	1.39E+08	4417.72	376.78	1257.54	5.71	11.47	OK	OK
3L/8	7813	5.17E+05	1.43E+08	1.39E+08	4478.53	460.63	1571.93	5.24	12.19	OK	OK
L/2	10350	5.17E+05	1.43E+08	1.39E+08	4462.27	488.58	1676.73	5.12	12.25	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: $0.45 f_c = 20.25 \text{ MPa} \quad (5.9.4.2.1-1)$

Tension Stress Limit: $-0.5 \text{ SQRT}(f_c) = -3.35 \text{ MPa}$

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

Interior Girder

Setion	Xi	A	S _i	S _{ig}	P _{pe}	P _{pe} *e _c	M _g + M _s	M _{SDL}	f _i	Check
	(mm)	(mm ²)	(mm ³)	(mm ³)	(kN)	(kNm)	(kNm)	(kNm)	(MPa)	f _i
Support	200	7.47E+05	1.57E+08	7.51E+08	3737.95	186.12	0.00	0.00	3.819	OK
L/8	2738	6.27E+05	1.51E+08	7.83E+08	3722.72	906.68	733.57	206.92	5.053	OK
L/4	5275	5.17E+05	1.43E+08	8.29E+08	3716.67	1400.38	1257.54	354.71	6.626	OK
3L/8	7813	5.17E+05	1.43E+08	8.29E+08	3688.16	1698.89	1571.93	443.39	6.789	OK
L/2	10350	5.17E+05	1.43E+08	8.29E+08	3713.88	1814.54	1676.73	472.95	6.798	OK

Exterior Girder

Setion	Xi	A	S _i	S _{ig}	P _{pe}	P _{pe} *e _c	M _g + M _s	M _{SDL}	f _i	Check
	(mm)	(mm ²)	(mm ³)	(mm ³)	(kN)	(kNm)	(kNm)	(kNm)	(MPa)	f _i
Support	200	7.47E+05	1.57E+08	7.51E+08	3737.95	186.12	0.00	0.00	3.819	OK
L/8	2738	6.27E+05	1.51E+08	7.83E+08	3738.42	910.50	685.89	206.92	4.738	OK
L/4	5275	5.17E+05	1.43E+08	8.29E+08	3707.79	1397.03	1175.81	354.71	6.062	OK
3L/8	7813	5.17E+05	1.43E+08	8.29E+08	3716.46	1711.92	1469.76	443.39	6.039	OK
L/2	10350	5.17E+05	1.43E+08	8.29E+08	3698.49	1807.02	1567.74	472.95	6.061	OK

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

Compression Stress Limit: $0.40 f_c = 18.00 \text{ MPa}$ (5.9.4.2.1-1)
 Tension Stress Limit: $-0.5 \cdot \text{SQRT}(f_c) = -3.35 \text{ MPa}$

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

Interior Girder

Setion	Xi (mm)	A (mm ²)	S _t (mm ³)	S _{lg} (mm ³)	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	7.51E+08	3737.95	186.12	0.00	0.00	0.00	1.909	OK
L/8	2738	6.27E+05	1.51E+08	7.83E+08	3722.72	906.68	733.57	206.92	743.07	3.475	OK
L/4	5275	5.17E+05	1.43E+08	8.29E+08	3716.67	1400.38	1257.54	354.71	1242.12	4.812	OK
3L/8	7813	5.17E+05	1.43E+08	8.29E+08	3688.16	1698.89	1571.93	443.39	1515.22	5.223	OK
L/2	10350	5.17E+05	1.43E+08	8.29E+08	3713.88	1814.54	1676.73	472.95	1580.45	5.307	OK

Exterior Girder

Setion	Xi (mm)	A (mm ²)	S _t (mm ³)	S _{lg} (mm ³)	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	747266.7	1.57E+08	7.51E+08	3737.95	186.12	0.00	0.00	0.00	1.909	OK
L/8	2738	627270.0	1.51E+08	7.83E+08	3738.42	910.50	685.89	206.92	830.60	3.429	OK
L/4	5275	516600.0	1.43E+08	8.29E+08	3707.79	1397.03	1175.81	354.71	1388.44	4.707	OK
3L/8	7813	516600.0	1.43E+08	8.29E+08	3716.46	1711.92	1469.76	443.39	1693.71	5.064	OK
L/2	10350	516600.0	1.43E+08	8.29E+08	3698.49	1807.02	1567.74	472.95	1766.62	5.163	OK

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

Compression Stress Limit: $0.60 f_c = 27.00 \text{ MPa}$ (5.9.4.2.1-1)
 Tension Stress Limit: $-0.5 \cdot \text{SQRT}(f_c) = -3.35 \text{ MPa}$

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

Interior Girder

Setion	Xi (mm)	A (mm ²)	S _t (mm ³)	S _{lg} (mm ³)	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	7.51E+08	3737.95	186.12	0.00	0.00	0.00	3.819	OK
L/8	2738	6.27E+05	1.51E+08	7.83E+08	3722.72	906.68	733.57	206.92	743.07	6.002	OK
L/4	5275	5.17E+05	1.43E+08	8.29E+08	3716.67	1400.38	1257.54	354.71	1242.12	8.125	OK
3L/8	7813	5.17E+05	1.43E+08	8.29E+08	3688.16	1698.89	1571.93	443.39	1515.22	8.617	OK
L/2	10350	5.17E+05	1.43E+08	8.29E+08	3713.88	1814.54	1676.73	472.95	1580.45	8.706	OK

Exterior Girder

Setion	Xi (mm)	A (mm ²)	S _t (mm ³)	S _{lg} (mm ³)	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	7.51E+08	3737.95	186.12	0.00	0.00	0.00	3.819	OK
L/8	2738	6.27E+05	1.51E+08	7.83E+08	3738.42	910.50	685.89	206.92	830.60	5.798	OK
L/4	5275	5.17E+05	1.43E+08	8.29E+08	3707.79	1397.03	1175.81	354.71	1388.44	7.738	OK
3L/8	7813	5.17E+05	1.43E+08	8.29E+08	3716.46	1711.92	1469.76	443.39	1693.71	8.084	OK
L/2	10350	5.17E+05	1.43E+08	8.29E+08	3698.49	1807.02	1567.74	472.95	1766.62	8.193	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: $0.45 f_c = 15.75 \text{ MPa}$ (5.9.4.2.1-1)

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

Setion	Xi (mm)	MSDL (kNm)		S _{lc} (mm ³)		f _t (MPa)		Check	
		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	3.83E+08	0.000	0.000	OK	OK
L/8	2737.50	206.92	206.92	3.8E+08	3.83E+08	0.540	0.540	OK	OK
L/4	5275.00	354.71	354.71	3.8E+08	3.85E+08	0.922	0.922	OK	OK
3L/8	7812.50	443.39	443.39	3.8E+08	3.85E+08	1.152	1.152	OK	OK
L/2	10350.00	472.95	472.95	3.8E+08	3.85E+08	1.229	1.229	OK	OK

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

Compression Stress Limit: $0.6 f_c = 21.00 \text{ MPa}$ (5.9.4.2.1-1)

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

Setion	Xi	MSDL + MLL (kNm)		S _{ic} (mm ³)		f _i (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	3.83E+08	0.000	0.000	OK	OK
L/8	2737.50	949.98	1037.51	3.8E+08	3.83E+08	2.479	2.707	OK	OK
L/4	5275.00	1596.83	1743.15	3.8E+08	3.85E+08	4.149	4.530	OK	OK
3L/8	7812.50	1958.62	2137.10	3.8E+08	3.85E+08	5.089	5.553	OK	OK
L/2	10350.00	2053.40	2239.57	3.8E+08	3.85E+08	5.336	5.820	OK	OK

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

Compression Stress Limit: $0.60 f_c = 27.00 \text{ MPa}$ (5.9.4.2.1-1)

Tension Stress Limit: $-0.5 \cdot \text{SQRT}(f_c) = -3.35 \text{ MPa}$ (5.9.4.2.1-1)

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

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)	(mm ²)	(mm ³)	(mm ³)	(kN)	(kNm)	(kNm)	(kNm)	(kNm)	(MPa)	f _b
Support	200	7.47E+05	1.49E+08	2.60E+08	3737.95	186.12	0.00	0.00	0.00	6.254	OK
L/8	2738	6.27E+05	1.44E+08	2.39E+08	3722.72	906.68	733.57	206.92	743.07	3.780	OK
L/4	5275	5.17E+05	1.39E+08	2.18E+08	3716.67	1400.38	1257.54	354.71	1242.12	2.040	OK
3L/8	7813	5.17E+05	1.39E+08	2.18E+08	3688.16	1698.89	1571.93	443.39	1515.22	0.464	OK
L/2	10350	5.17E+05	1.39E+08	2.18E+08	3713.88	1814.54	1676.73	472.95	1580.45	0.217	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)	(mm ²)	(mm ³)	(mm ³)	(kN)	(kNm)	(kNm)	(kNm)	(kNm)	(MPa)	f _b
Support	200	7.47E+05	1.49E+08	2.60E+08	3737.95	186.12	0.00	0.00	0.00	6.254	OK
L/8	2738	6.27E+05	1.44E+08	2.39E+08	3738.42	910.50	685.89	206.92	830.60	3.870	OK
L/4	5275	5.17E+05	1.39E+08	2.18E+08	3707.79	1397.03	1175.81	354.71	1388.44	2.050	OK
3L/8	7813	5.17E+05	1.39E+08	2.18E+08	3716.46	1711.92	1469.76	443.39	1693.71	0.691	OK
L/2	10350	5.17E+05	1.39E+08	2.18E+08	3698.49	1807.02	1567.74	472.95	1766.62	0.233	OK

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

Compression Stress Limit: $0.60 f_c = 27.00 \text{ MPa}$ (5.9.4.2.1-1)

Tension Stress Limit: $-0.5 \cdot \text{SQRT}(f_c) = -3.35 \text{ MPa}$ (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)	(mm ²)	(mm ³)	(mm ³)	(kN)	(kNm)	(kNm)	(kNm)	(kNm)	(MPa)	f _b
Support	200	7.47E+05	1.49E+08	2.60E+08	3737.95	186.12	0.00	0.00	0.00	6.254	OK
L/8	2738	6.27E+05	1.44E+08	2.39E+08	3722.72	906.68	733.57	206.92	743.07	3.157	OK
L/4	5275	5.17E+05	1.39E+08	2.18E+08	3716.67	1400.38	1257.54	354.71	1242.12	0.902	OK
3L/8	7813	5.17E+05	1.39E+08	2.18E+08	3688.16	1698.89	1571.93	443.39	1515.22	-0.925	OK
L/2	10350	5.17E+05	1.39E+08	2.18E+08	3713.88	1814.54	1676.73	472.95	1580.45	-1.232	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)	(mm ²)	(mm ³)	(mm ³)	(kN)	(kNm)	(kNm)	(kNm)	(kNm)	(MPa)	f _b
Support	200	7.47E+05	1.49E+08	2.60E+08	3737.95	186.12	0.00	0.00	0.00	6.254	OK
L/8	2738	6.27E+05	1.44E+08	2.39E+08	3738.42	910.50	685.89	206.92	830.60	3.174	OK
L/4	5275	5.17E+05	1.39E+08	2.18E+08	3707.79	1397.03	1175.81	354.71	1388.44	0.777	OK
3L/8	7813	5.17E+05	1.39E+08	2.18E+08	3716.46	1711.92	1469.76	443.39	1693.71	-0.861	OK
L/2	10350	5.17E+05	1.39E+08	2.18E+08	3698.49	1807.02	1567.74	472.95	1766.62	-1.386	OK

a	Depth of equivalent stress block	m	0.077	0.077	0.078	0.078	0.078
de	Corresp. effective depth from extreme comp. fiber to centroid of tensile force in the tensile reinf.	m	0.888	1.071	1.201	1.280	1.306
Mn	Nominal resistance	kNm	4997	6199	7044	7598	7783
Mr	Factored resistance	kNm	4497	5579	6340	6838	7005
Mu	Flexural moment	kNm	0	2595	4386	5409	5700
(5.7.3.2)	Flexural moment Checking		OK	OK	OK	OK	OK
	Limits for reinforcement						
c/de	Maximum reinforcement		0.11	0.09	0.08	0.08	0.08
	Maximum reinforcement Checking	≤ 0.42	OK	OK	OK	OK	OK
r min	Minimum reinforcement		0.09%	0.10%	0.12%	0.12%	0.12%
	Minimum reinforcement Checking for RC	0.30%	N.a	N.a	N.a	N.a	N.a
1.2*Mcr	Cracking moment	kNm	827	784	741	741	741
(5.7.3.3.2)	Checking $M_r \geq \min(1.2M_{cr}, 1.33M_u)$		OK	OK	OK	OK	OK
(5.8.3.5)	Tensile force in steel should be satisfied - F_{yc}	kN	1353	3873	5068	5311	5245
	Checking $A_s f_y + A_{ps} f_{ps} \geq F_{yc}$		OK	OK	OK	OK	OK
(5.7.3.4)	Control of cracking by distr. of reinf for RC member- Check?		No	No	No	No	No
	Existing condition for structure	1,2 or 3	1	1	1	1	1
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.058	0.058	0.058	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	m ²	0.049	0.049	0.049	0.049	0.049
f _{sa}	Value	Mpa	211	211	211	211	211
0.6*f _y		Mpa	240	240	240	240	240
	Tensile stress in reinf $\min(f_{sa}, 0.6f_y)$	Mpa	211	211	211	211	211
x	Dist. From compression fiber to centroid	m	-	-	-	-	-
J.d	Arm	m	-	-	-	-	-
I _{cr}	Moment of Inertia of the cracked section	m ⁴	-	-	-	-	-
f _s	Tensile stress in reinforcement $f_s = M_{sls} / (A_s * J.d)$	Mpa	-	-	-	-	-
	Checking for control cracking $f_s < f_{sa}$		N.a	N.a	N.a	N.a	N.a
(5.10.8.2)	Shrinkage and temperature Reinforcement (side distribution)						
A _{req}	Area of required reinf	m ²	0.00029	0.00026	0.00023	0.00023	0.00023
	Distribution on sides 14 D12	m ²	0.00158	0.00158	0.00158	0.00158	0.00158
	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	5.7	3.3	2.4	2.4
θ	Angle of inclination of diagonal compressive	degree	27.00	27.00	25.26	30.90	31.07
α	Angle of inclination of transv. reinf. to long. Axis	degree	90	90	90	90	90
b _v	Effective web width as minimum web width - In dv	m	0.600	0.386	0.200	0.200	0.200
dv	Effective shear depth	m	1.024	1.032	1.163	1.241	1.267
	($d_e - a/2$)	m	0.850	1.032	1.163	1.241	1.267
s	Spacing of stirrups	m	0.150	0.150	0.150	0.150	0.150
n _{cat}	Amount of bars in spacing S	bars	2	2	2	2	2
A _v	Shear reinf area in spacing S	m ²	0.0003	0.0003	0.0003	0.0003	0.0003
β	Assume		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/m ²	2244	2762	3560	2257	1175
f _{po}	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1136	1138	1141	1141	1141
e _x	Strain in tensile reinforcement		-3.71E-03	-1.12E-03	9.97E-05	6.06E-04	6.29E-04
	If $e_x < 0$, multiple with reduce factor		-2.35E-04	-1.07E-04	-	-	-
	Strain checking	$\leq 2.00E-3$	OK	OK	OK	OK	OK
v/f _c	Ratio of shear stress and f _c		0.056	0.069	0.089	0.056	0.029
β	Final value		6.8	5.7	3.3	2.4	2.4
θ	Final value	degree	27.00	27.00	25.26	30.90	31.07
V _c	Nominal shear resistance provided by tensile stresses in the concrete	kN	2186	1193	397	318	324
V _s	Shear resistance provided by shear reinforcement	kN	1618	1632	1984	1670	1693
V _p	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0	0
V _{n1}	$V_{n1} = V_c + V_s + V_p$	kN	3805	2825	2381	1988	2018
V _{n2}	V _{n2}	kN	6143	3985	2325	2482	2534
V _n	Nominal shear resistance $V_n = \min(V_{n1}, V_{n2})$	kN	3805	2825	2325	1988	2018
V _r	Factored shear resistance	kN	3424	2542	2093	1789	1816
V _u	Shear	kN	1241	991	745	504	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	m ²	0.0001	0.0001	0.0000	0.0000	0.0000
	Minimum shear reinforcement Checking		OK	OK	OK	OK	OK
	$0.1 * f_c * b_v * dv$	kN	2457	1594	930	993	1014
	S _{max}	m	0.60	0.60	0.60	0.60	0.60
	Maximum spacing S _{max}		OK	OK	OK	OK	OK

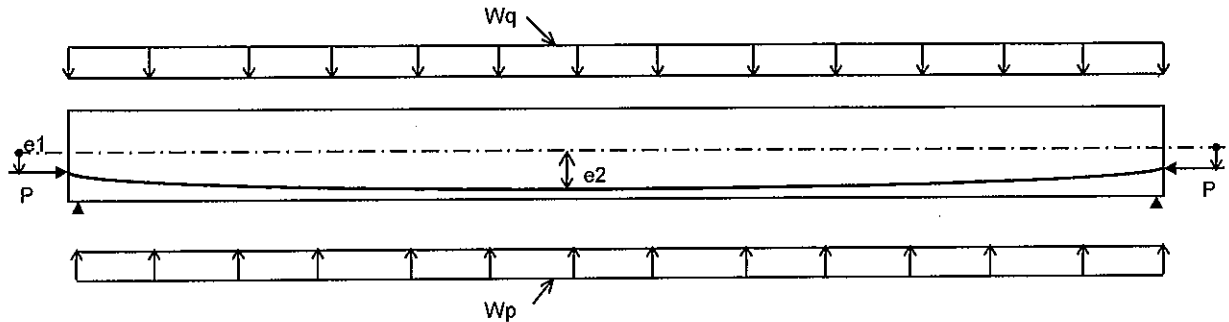
REINFORCEMENT OF GIRDER CHECKING - SERVICE LOAD COMBINATION

MATERIALS				
NORMAL CONCRETE				
fc	Compressive Strength of concrete at 28 days	Mpa	40	
Ec	Modulus of Elasticity	Mpa	31975	
fr	Modulus of Rupture	Mpa	4.0	
gc	Unit weight of concrete	kN/m3	24.5	
PRESTRESSING STEEL				
fpu	Tensile strength of prestressing steel	Mpa	1860	
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		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	824	652	483	317	153
Mu	Flexural Moment	kNm	0	1723	2919	3607	3807
Nu	Axial load	kN	0	0	0	0	0
Tu	Torsional Moment	kNm	0	0	0	0	0
6.1 FLEXURAL MOMENT CHECKING							
H	Section height	m	1.422	1.422	1.422	1.422	1.422
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	1.361	1.361	1.361	1.361	1.361
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	0.567	0.372	0.232	0.148	0.120
dps	Dis. From comp. fiber to centroid of tension prestressing steel	m	0.855	1.050	1.190	1.274	1.302
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	0.386	0.200	0.200	0.200
hf	Compression flange depth	m	0.222	0.222	0.222	0.222	0.222
Iz	Moment of inertia of section	m4	0.229	0.217	0.205	0.205	0.205
Amc	Section area	m2	1.213	1.093	0.982	0.982	0.982
	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	4	4	4	4	4
		Area	m2	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	0	0	0	0	0
		Area	m2	0.00000	0.00000	0.00000	0.00000
As	Tension Reinforcement	Number	6	6	6	6	6
		Diameter	16	16	16	16	16
		Area	m2	0.00114	0.00114	0.00114	0.00114
A's	Compression Reinforcement	Number	4	4	4	4	4
		Diameter	12	12	12	12	12
		Area	m2	0.00045	0.00045	0.00045	0.00045
A'c	Shear reinforcement	Number	2	2	2	2	2
		Diameter	14	14	14	14	14
		Area	m2	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		0.764	0.764	0.764	0.764	0.764
c	Dis. Between centroid and top fiber	m	0.101	0.101	0.101	0.102	0.102
	For T section behavior	m	-0.245	-0.474	-0.985	-1.000	-1.005
	For rectangular section behavior	m	0.101	0.101	0.101	0.102	0.102
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	1798	1809	1815	1818	1819
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	0.077	0.077	0.078	0.078	0.078
de	Corresp. effective depth from extreme comp. fiber to centroid of tensile force in the tensile reinf.	m	0.888	1.071	1.201	1.280	1.306
Mn	Nominal resistance	kNm	4997	6199	7044	7598	7783
Mr	Factored resistance	kNm	4997	6199	7044	7598	7783
Mu	Flexural moment	kNm	0	1723	2919	3607	3807
(5.7.3.2)	Flexural moment Checking		OK	OK	OK	OK	OK
	Limits for reinforcement						
c/de	Maximum reinforcement		0.11	0.09	0.08	0.08	0.08
	Maximum reinforcement Checking	<= 0.42	OK	OK	OK	OK	OK
r min	Minimum reinforcement		0.09%	0.10%	0.12%	0.12%	0.12%
	Minimum reinforcement Checking for RC	0.30%	N.a	N.a	N.a	N.a	N.a
1.2*Mcr	Cracking moment	kNm	827	784	741	741	741
(5.7.3.3.2)	Checking $M_r \geq \min(1.2M_{cr}, 1.33M_u)$		OK	OK	OK	OK	OK
(5.8.3.5)	Tensile force in steel should be satisfied - F_{yc}	kN	808	2309	2985	3217	3155
	Checking $A_s \cdot f_y + A_{ps} \cdot f_{ps} \geq F_{yc}$		OK	OK	OK	OK	OK
(5.7.3.4)	Control of cracking by distr. of reinf for RC member- Check?		Yes	Yes	Yes	Yes	Yes
	Existing condition for structure	1,2 or 3	1	1	1	1	1
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.058	0.058	0.058	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
f _{sa}	Value	Mpa	211	211	211	211	211
0.6*f _y	Tensil stress in reinf Min(f _{sa} ,0.6f _y)	Mpa	240	240	240	240	240
x	Dist. From compression fiber to centroid	m	0.066	0.073	0.078	0.08	0.081
J.d	Arm	m	0.866	1.047	1.176	1.253	1.279
icr	Moment of inertia of the cracked section	m4	0.012	0.012	0.012	0.012	0.012
f _s	Tensile stress in reinforcement $f_s = M_{sls} / (A_s \cdot J.d)$	Mpa	-	1446	2180	2527	2614
	Checking for control cracking $f_s < f_{sa}$		N.a	N.a	N.a	N.a	N.a
(5.10.8.2)	Shrinkage and temperature Reinforcement (side distribution)						
A _{req}	Area of required reinf	m2	0.00029	0.00026	0.00023	0.00023	0.00023
	Distribution on sides 14 D12	m2	0.00158	0.00158	0.00158	0.00158	0.00158
	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	6.5	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	0.386	0.200	0.200	0.200
dv	Effective shear depth	m	1.024	1.032	1.163	1.241	1.267
	(de - a/2)	m	0.850	1.032	1.163	1.241	1.267
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	1636	2077	1276	604
f _{po}	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1136	1138	1141	1141	1141
e _x	Strain in tensile reinforcement		-3.94E-03	-2.23E-03	-1.41E-03	-1.07E-03	-1.06E-03
	If e _x <0, multiple with reduce factor		-2.49E-04	-2.11E-04	-2.38E-04	-1.81E-04	-1.78E-04
	Strain checking	<=2.00E-3	OK	OK	OK	OK	OK
v/f _c	Ratio of shear stress and f _c		0.034	0.041	0.052	0.032	0.015
β	Final value		6.8	6.8	6.8	6.5	6.5
θ	Final value	degree	27.00	27.00	27.00	27.00	27.00
V _c	Nominal shear resistance provided by tensile stresses in the concrete	kN	2186	1418	828	853	866
V _s	Shear resistance provided by shear reinforcement	kN	1618	1632	1838	1961	2003
V _p	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0	0
V _{n1}	V _{n1} =V _c +V _s +V _p	kN	3805	3050	2665	2814	2868
V _{n2}	V _{n2}	kN	6143	3985	2325	2482	2534
V _n	Nominal shear resistance V _n =min(V _{n1} ,V _{n2})	kN	3805	3050	2325	2482	2534
V _r	Factored shear resistance	kN	3805	3050	2325	2482	2534
V _u	Shear	kN	824	652	483	317	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
	Minimum shear reinforcement Checking		-	-	OK	-	-
	0.1*f _c *b*v*dv	kN	2457	1594	930	993	1014
	S _{max}	m	0.60	0.60	0.60	0.60	0.60
	Maximum spacing S _{max}		-	-	OK	-	-

DEFLECTION OF GIRDER



7.1 Deflection of Girder due to Dead Load in Stage I

Calculation Formula $Y1 = \frac{5 \cdot q \cdot L^4}{384 E_c \cdot J}$

Where :

Uniform Dead Load	$q =$	14.74 KN/m
Span Length	$L =$	20.30 m
Elastic Modulus of Concrete	$E_c =$	33185.25 MPa
Exchanged Inertial Moment	$J =$	0.085 m ⁴
Rigidity Reduced Factor		1
Deflection	$Y1 =$	1.16 cm

7.2 Deflection of Girder due to Concentrate Prestressing Moment

Deflection of Girder due to Concentrate Moment Force at the end of Girder

Formula : $Y2 = \frac{M \cdot L^2}{8 \cdot E_c \cdot J}$

Where

Moment due to Prestressing at Mid-Span

	$M = P \cdot e$	KN.m
Prestressing Stress at Jacking Point	$P^* =$	1395.00 Mpa
Number of Cables	$n =$	4.00
Stress in Cable Including Losses	$T_x =$	1115.57 MPa
Area of One Cable	$A_i =$	0.00089 m ²
Total Compression Force in Cables	$P =$	4956.714 KN
Eccentricity of Cable Group and Section	$e =$	0.024 m
	$M =$	116.9 KNm
Span Length	$L =$	20.30 m
Elastic Modulus of Concrete	$E_c =$	33185.25 MPa
Exchanged Inertial Moment	$J =$	0.087 m ⁴
Rigidity Reduced Factor		1
Deflection	$Y2 =$	0.21 cm

7.3 Deflection of Girder due to Uniform Prestressing Forces

Formula to Calculating $Y3 = \frac{5 \cdot q \cdot L^4}{384 E_c \cdot J}$

Where:

Uniform Equivalence Load	$q = 8 \times P \times (e2 - e1) / L^2$	
	$P =$	4956.714 KN
	$e1 =$	0.032 m
	$e2 =$	0.489 m
	$q =$	43.936 KN/m
Span Length	$L =$	20.30 m
Elastic Modulus of Concrete	$E_c =$	33185.25 MPa
Exchanged Inertial Moment	$J =$	0.087 m ⁴
Deflection	$Y3 =$	3.370 cm

7.4 Deflection of Girder in Stage I

Formula $\Sigma Y = Y2 + Y3 - Y1 = 2.420 \text{ cm}$

CALCULATION SHEET
127 GIRDER

CONTENT

1. INPUT DATA

- 1.1 General data
- 1.2. Girder dimension
- 1.3. Material properties
 - 1.3.1 Concrete:
 - 1.3.2 Prestressing steel
 - 1.3.3 Reinforcing Steel:

2. INTERNAL FORCE

- 2.1. Dead load
 - 2.1.1 Load:
 - 2.1.2 Internal Force due to dead load:
- 2.2. Live load
 - 2.2.1. Distribution factors for Live load:
 - 2.2.2 Live Load:
 - 2.2.3 Internal Force due to Live load:
- 2.3 Load combination:
 - 2.3.1 Load combination - - Interior Girder:
 - 2.3.2 Load combination - Exterior Girder:

3. TENDON PROFILE AND PROPERTY OF GIRDER CROSS SECTION

- 3.1. Tendon profile
- 3.2. Property of girder cross section at transfer (net cross section)
- 3.2. Property of girder cross section at service stage (composite cross section)
 - 3.3.1. Effective flange width
 - 3.3.2. Property of Girder cross section in stage II (service stage):

4. LOSS OF PRESTRESS

- 4.1. Loss of prestressing force immediately (Instantaneous losses):
 - 4.1.1 Friction between Prestressing Tendon and Duck:
 - 4.1.2 Anchorage seating or Set:
 - 4.1.3 Elastic deformation of concrete:
- 4.2. Loss of prestressing force at service stage (time - dependent losses):
 - 4.2.1 Loss of prestress due to Shrinkage:
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5. FIBRE STRESS CHECK:

- 5.1 Stress check during construction the Girder:
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 - 5.2.1 Increase load:
 - 5.2.2 Stress check:
- 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:
 - 5.3.2 Due to 1/2 (Prestressing tendon + self weigh of girder) and Live load - Service limit stage I:
 - 5.3.3 Due to prestressing tendon + self weigh of girder + live load - Service limit stage I:
- 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:
 - 5.4.2 Due to additional load (dead load part 2) and live load - Service limit stage I:
- 5.5 Stress check at the bottom fibre of girder - Service III (stage III):
- 5.6 Stress check at the bottom fibre of girder - Service I (stage III):

6. ULTIMATE LOAD CHECK AND SHEAR CAPACITY CHECK

- 6.1 Flexural moment checking
- 6.2. Ultimate load check

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/m²)

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 between 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)

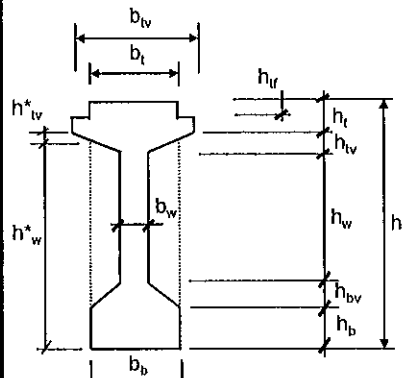
Precast plank thick

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 = 1500.00 (mm)
		h _{tv} = 80.00 (mm)
		h _t = 200.00 (mm)
Cross section at end		
	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)
at the middle		

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/m³

Modulus of elasticity

E_c = 0.043 γ_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/m³

Modulus of elasticity

E_c = 0.043 γ_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 mm²

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

Coefficient of friction (1/RAD)

μ = 0.25

Number of Strands in one Tendon

n = 12.00 Strands

Area of one Tendon

A_s = 1184.40 mm²

Stress in the prestressing steel at jacking

f_{pi} = 0.75 f_{pu} = 1395.00 MPa

Jacking force for one tendon

P_j = 1652.24 kN

Anchorage set

ΔL = 6.00 mm

Area of one duck

A_g = 3318.31 mm²

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 :

$$M = 0.5 q_i (L - X_i)$$

$$Q = q_i (0.5L - X_i)$$

$$L_{it} = 26.10 \text{ (m)}$$

INTERIOR GIRDER											
Section	X _i (m)	Girder (DC)		Concrete Deck (DC)		Plank & cr.beam (DC)		Parapet (DC)		Pavement (DW)	
		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	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											
Gđi	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

$$e'g = 853.99 \text{ (mm)}$$

$$e^Eg = 853.99 \text{ (mm)}$$

Longitudinal stiffness parameter

$$K^I_g = n(lg + A e'^2_g) = 1.2E+12$$

$$K^Eg = n(lg + A e^2_g) = 1.2E+12$$

Ration

$$K^I_g / (L t^3_s) = 4.37$$

$$K^Eg / (L t^3_s) = 4.37$$

$$S / L = 0.10$$

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

Interior Beam:

$$\text{For one lane} \quad 0.06 + \left(\frac{S}{4300} \right)^{0.4} \left(\frac{S}{L} \right)^{0.3} \left(\frac{K^I_g}{L t^3_s} \right)^{0.1} = 0.528$$

$$\text{Two or more lanes} \quad 0.075 + \left(\frac{S}{2900} \right)^{0.6} \left(\frac{S}{L} \right)^{0.2} \left(\frac{K^I_g}{L t^3_s} \right)^{0.1} = 0.749$$

Exterior Beam:

For one lane, follow the lever rule

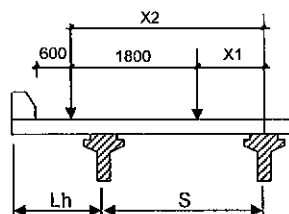
$$X_1 = 925.000$$

$$X_2 = 2725.00$$

$$Y_1 = 0.363$$

$$Y_2 = 1.069$$

$$\Rightarrow g(M) = 0.5 \sum y_i = 0.716$$



Two or more lanes

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

$$\text{Choice} \quad e = 1.047 \quad \text{IF}(e > 1, 1, e)$$

$$\Rightarrow g(M) = e \cdot g_{\text{strong}} = 0.784$$

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

Interior Beam:

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

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

$$g(Q) = 0.5 \cdot \sum y_i = \boxed{0.716}$$

Two or more lanes

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

$$\Rightarrow g(Q) = e \cdot g_{\text{wrong}} = \boxed{0.731}$$

(c) Correction factor for skew bridge:

* Correction factor of distribution factor for moment (Table 4.6.2.2d-1)

Skew angle	$\theta = 10$	Degree.	Area of applications
Factor	$c1 = 0.000$		$300 \leq \theta \leq 600$
Correction factor	$CF(M) = 1.000$		$1100 \leq S \leq 4900$
			$6000 \leq L \leq 73000$
			$Nb \geq 4$

$$CF(M) = 1.0 - c1 \cdot (\tan \theta)^{1.5}$$

$$c1 = 0.25 \cdot \left(\frac{Kg}{L \cdot S^3} \right)^{0.25} \cdot \left(\frac{S}{L} \right)^{0.5}$$

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

Correction Factor $CF(Q) = 1.023$

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

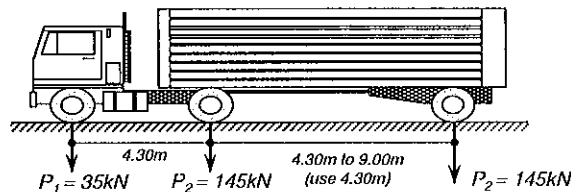
Area of applications
$00 \leq \theta \leq 600$
$1100 \leq S \leq 4900$
$6000 \leq L \leq 73000$
$Nb \geq 4$

(d) Table of Distribution factors for Live load:

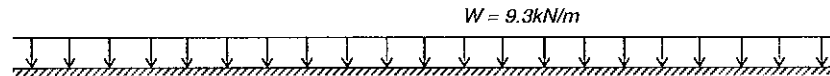
Interior Beam	$g(M)$	$g(Q)$	m	$m \cdot g(M)$	$m \cdot g(Q)$	$m \cdot g(M) \cdot CF(M)$	$m \cdot g(Q) \cdot CF(Q)$
1 lane	0.528	0.696	1.20	0.634	0.835	0.634	0.854
2 or more lanes	0.749	0.852	1.00	0.749	0.852	0.749	0.871
Exterior Beam							
1 lane	0.716	0.716	1.20	0.859	0.859	0.859	0.878
2 or more lanes	0.784	0.731	1.00	0.784	0.731	0.784	0.747

2.2.2 Live Load:

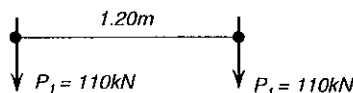
Design Truck



Design Lane Load



Design Tandem



- 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
- Dynamic load	IM = 0.25

2.2.3 Internal Force due to Live load:

Design truck or Tandem

Momen	$M_{TR(Ta)} = \sum P_i y_i$	(kNm)
Shear force	$Q_{TR(Ta)} = \sum P_i y_i$	(kN)

Lane load

Momen	$M_{Ln} = W \cdot F_M$	(kNm)
Shear force	$Q_{Ln} = W \cdot F_Q$	(kN)

Pedestrian

Momen	$M_{PL} = PL \cdot F_M$	(kNm)
Shear force	$Q_{PL} = PL \cdot F_Q$	(kN)

Influence line for Momen & Shear force		Load	Momen (kN.m)	Shear
Section at 1/2L		Truck	1733.63	127.08
$w = 85.1513$ $w^* = 3.2625$ $w^* = -3.2625$		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
$w = 79.829$ $w^* = 1.835$ $w^* = -5.098$		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
$w = 63.863$ $w^* = 0.816$ $w^* = -7.341$		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
$w = 37.254$ $w^* = 0.204$ $w^* = -9.991$		Lane	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
$w = 0.000$ $w^* = 0.00$ $w^* = -13.050$		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 :

$$M_{(LL+IM)} = m \cdot g(M) \cdot [\max\{M_{TR}, M_{Ta}\} \cdot (1+IM) + M_{Ln}]$$

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

Internal Force due to pedestrian :

$$M = g(M) \cdot M_{PL}$$

$$Q = g(Q) \cdot Q_{PL}$$

In which:

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

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

y_i Value of influence line

F

Area of influence line

m Lane factor

g

Distribution factor

	Interior	$m \cdot g(M)$	$m \cdot g(Q)$
		0.749	0.871
	Exterior		
		0.859	0.878

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	420.90	0.00	424.51
L/8	3.26	1019.67	351.91	1169.48	354.92
L/4	6.53	1717.10	286.22	1969.38	288.67
3L/8	9.79	2109.90	223.84	2419.89	225.75
L/2	13.05	2215.68	164.75	2541.21	166.16

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		Section									
Load	Load Factor	Support		L/8		L/4		3L/8		L/2	
		M	Q	M	Q	M	Q	M	Q	M	Q
	γ	(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	736.58	1784.43	615.85	3004.93	500.89	3692.33	391.71	3877.44	288.32
Total		0.00	1463.42	3859.32	1160.98	6561.90	864.31	8138.54	573.42	8620.07	288.32

STATE Service		Section									
Load	Load Factor	Support		L/8		L/4		3L/8		L/2	
		M	Q	M	Q	M	Q	M	Q	M	Q
	γ	(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	420.90	1019.67	351.91	1717.10	286.22	2109.90	223.84	2215.68	164.75
Total		0.00	990.78	2646.50	779.32	4505.95	571.16	5595.96	366.31	5934.14	164.75

2.3.2 Load combination - Exterior Girder:

STATE Strength		Section									
Load	Load factor	Support		L/8		L/4		3L/8		L/2	
		M	Q	M	Q	M	Q	M	Q	M	Q
	γ	(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	742.88	2046.59	621.12	3446.41	505.17	4234.80	395.06	4447.11	290.78
Total		0.00	1469.72	4014.77	1166.24	6820.44	868.59	8452.34	576.77	8945.82	290.78

STATE Service		Section									
Load	load factor	Support		L/8		L/4		3L/8		L/2	
		M	Q	M	Q	M	Q	M	Q	M	Q
	γ	(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	424.51	1169.48	354.92	1969.38	288.67	2419.89	225.75	2541.21	166.16
Total		0.00	994.38	2710.94	782.33	4611.87	573.61	5723.01	368.22	6064.53	166.16

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 = X₂-X₁ Distance between 2 point under consider

angle of rotation of tendon for X_i-axis

Tan(α) = (4.f (1-2.X_i / L)) / L

α = 2 f / 0.5 L - tan(α)

L _{span} =	27000	(mm)
L _{su.} =	26100	(mm)
L _{cap} =	26700	(mm)

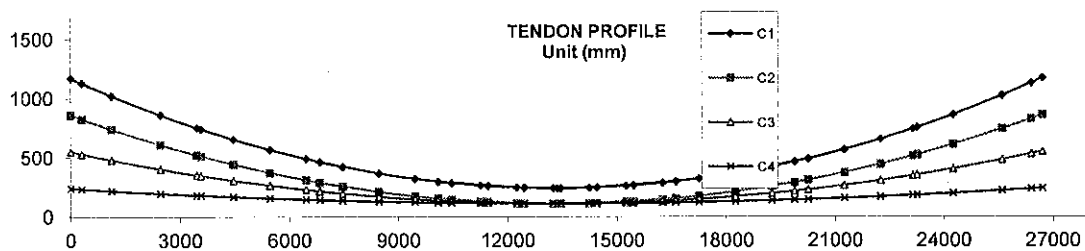
TENDON No 1	f =	1170	(mm)	L _{cap} =	26700	(mm)	C =	240	(mm)
	Section	Xi	Yi	Lp	ΣL _{cap}	Tan(α)	(α)	Σα	Section
		(mm)	(mm)	(mm)	(mm)		(rad)	(rad)	
	Anchorage	0.00	1170.00	0.00	0.00	0.1753	0.0000	0.0000	Anchorage
	Support	300.00	1128.67	300.00	300.00	0.1713	0.0039	0.0039	Support
	L/8	3562.50	739.88	3262.50	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

TENDON No 2	f =	860	(mm)	L _{cap} =	26700	(mm)	C =	110	(mm)
	Section	Xi	Yi	Lp	ΣL _{cap}	Tan(α)	(α)	Σα	Section
		(mm)	(mm)	(mm)	(mm)		(rad)	(rad)	
	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

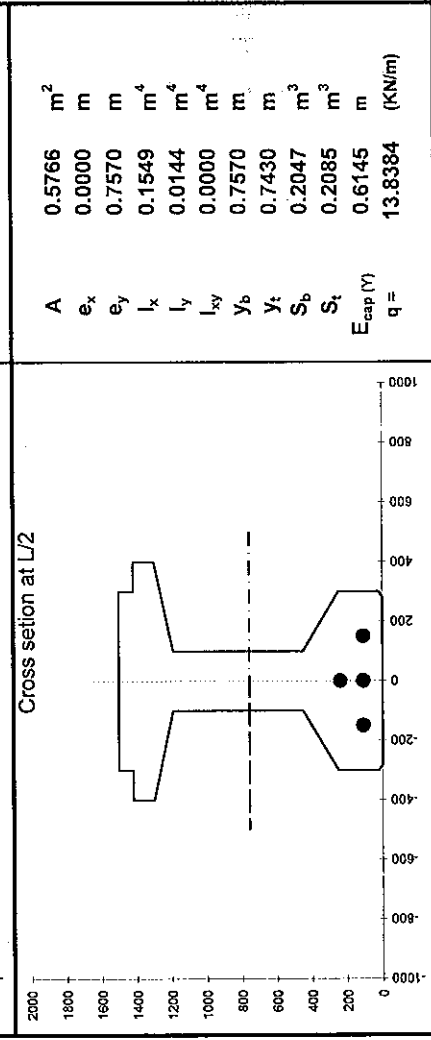
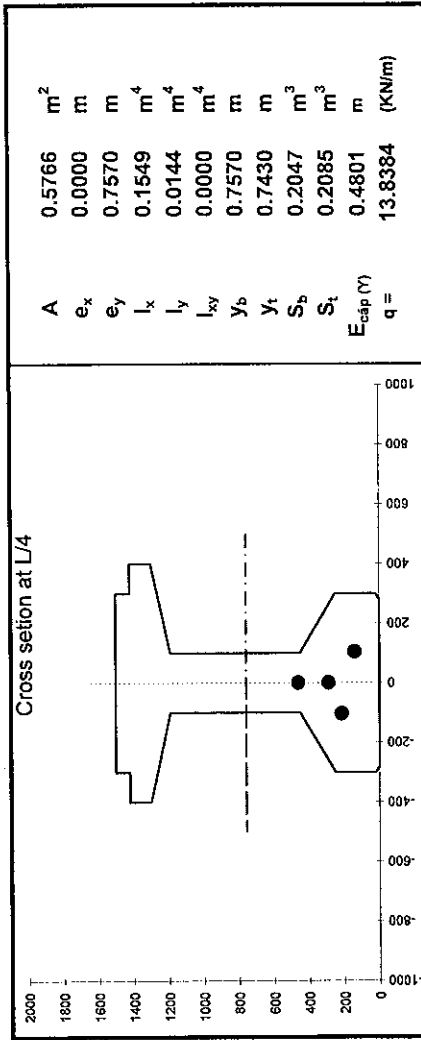
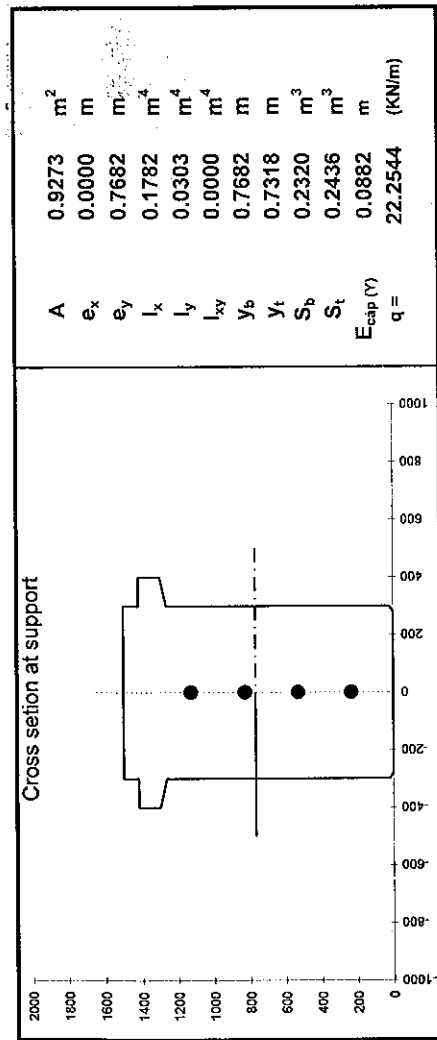
TENDON No 3	f =	550	(mm)	L _{cap} =	26700	(mm)	C =	110	(mm)
	Section	Xi	Yi	Lp	ΣL _{cap}	Tan(α)	(α)	Σα	Section
		(mm)	(mm)	(mm)	(mm)		(rad)	(rad)	
	Anchorage	0.00	550.00	0.00	0.00	0.0824	0.0000	0.0000	Anchorage
	Support	300.00	530.45	300.00	300.00	0.0805	0.0019	0.0019	Support
	L/8	3562.50	346.50	3262.50	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

TENDON No 4	f =	240	(mm)	L _{cap} =	26700	(mm)	C =	110	(mm)
	Section	Xi	Yi	Lp	ΣL _{cap}	Tan(α)	(α)	Σα	Section
		(mm)	(mm)	(mm)	(mm)		(rad)	(rad)	
	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

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	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):



Uniform load due to self 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

Modular Ratio: Deck Concrete/Girder Concrete

$$n = E_b / E_d = 0.88$$

For Interior Girder:

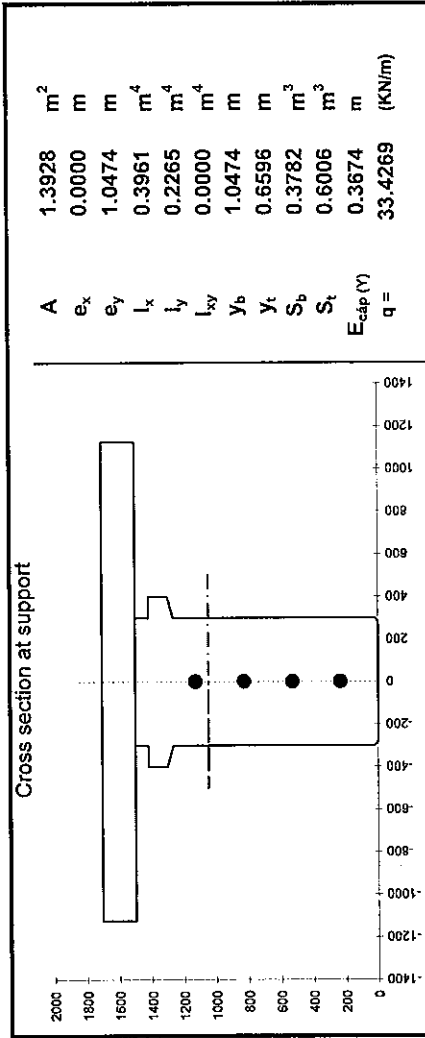
$$b_f = \min \left\{ \begin{array}{l} 1/4 L_{II} \\ 12h_f + \max(0.5b_w, b_w) \end{array} \right\} S \Rightarrow n^* b_f = 2248.88861 \text{ (mm)}$$

For Exterior Girder:

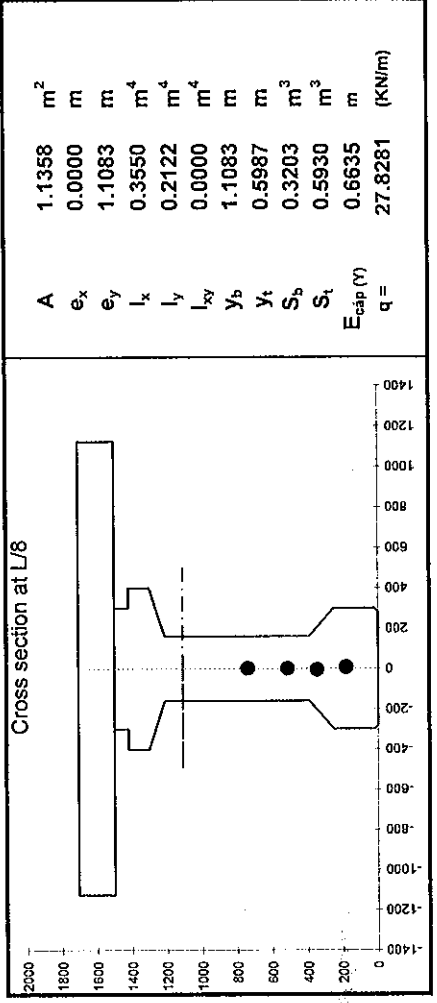
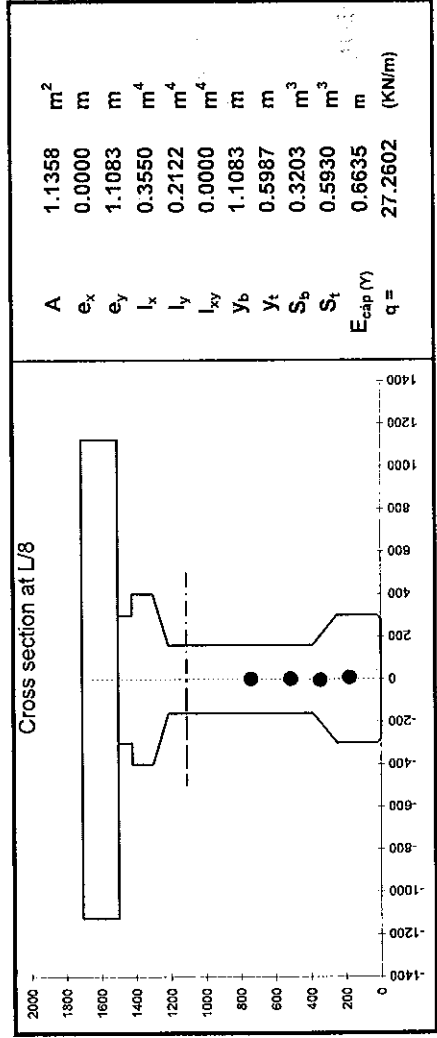
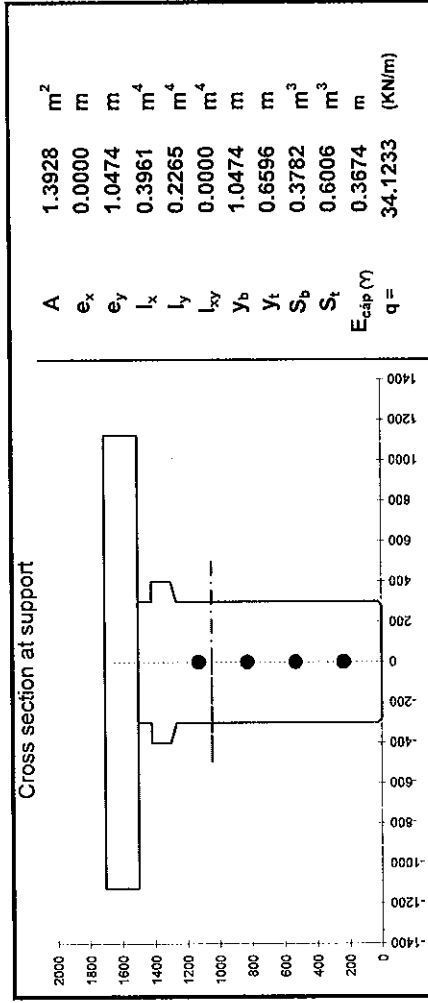
$$b_E = 0.5b_f + \min \left\{ \begin{array}{l} 1/8 L_{II} \\ 6h_f + \max(0.5b_w, 0.25b) \end{array} \right\} L_h \Rightarrow n^* b_E = 2248.88861 \text{ (mm)}$$

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

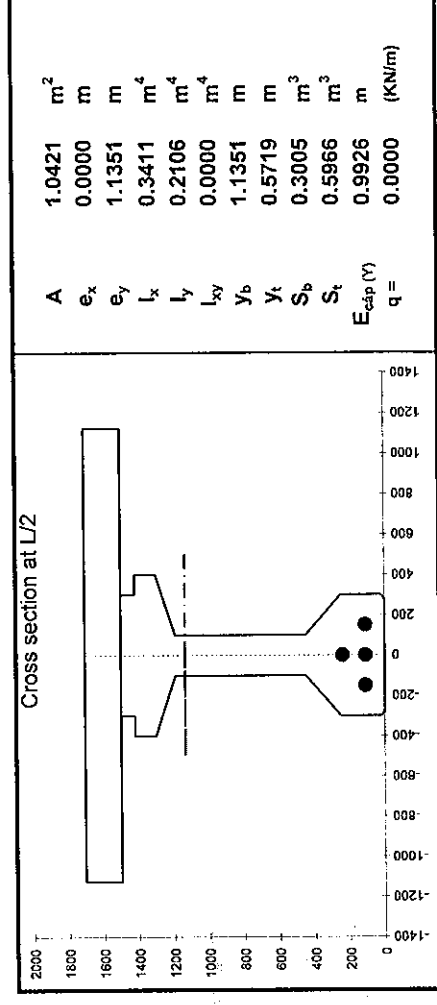
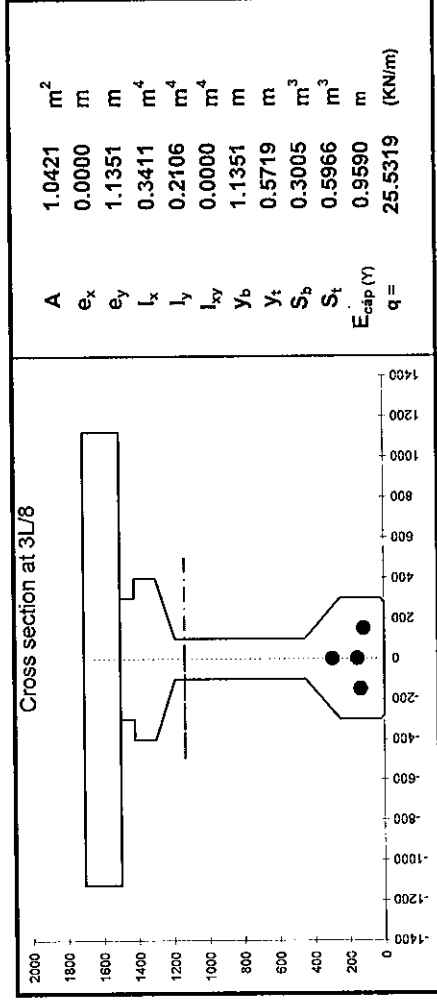
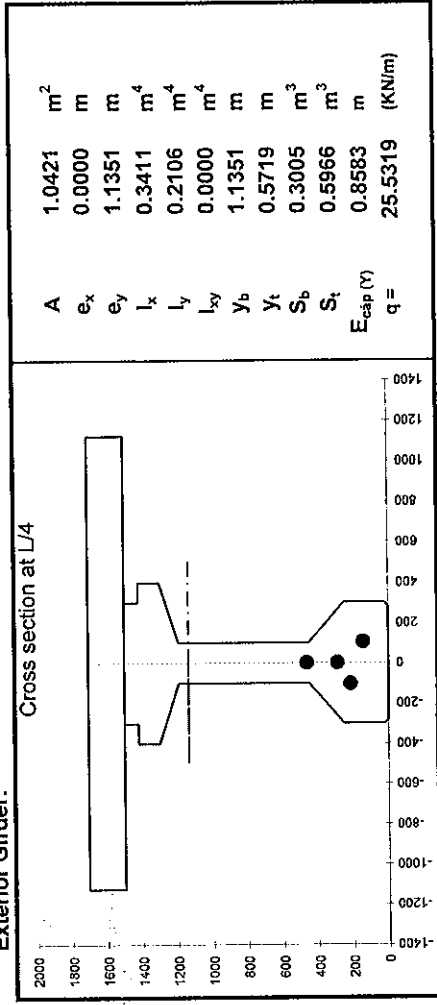
Interior Girder:



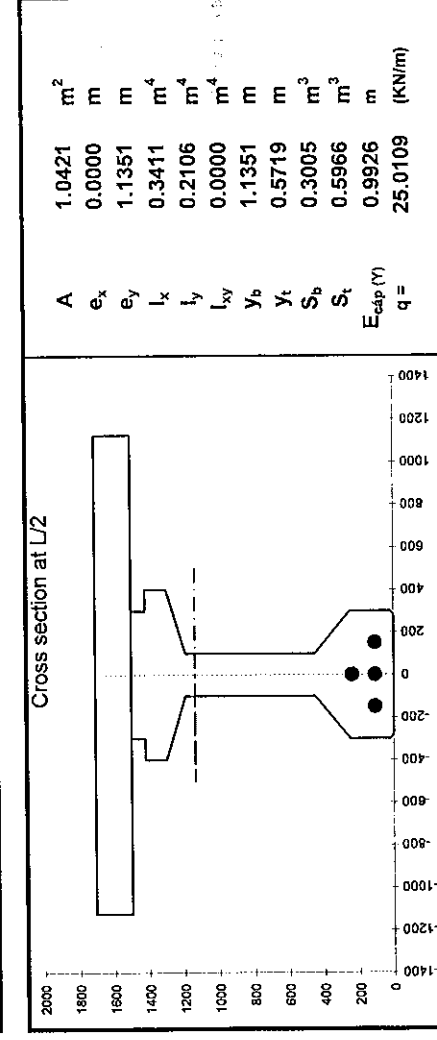
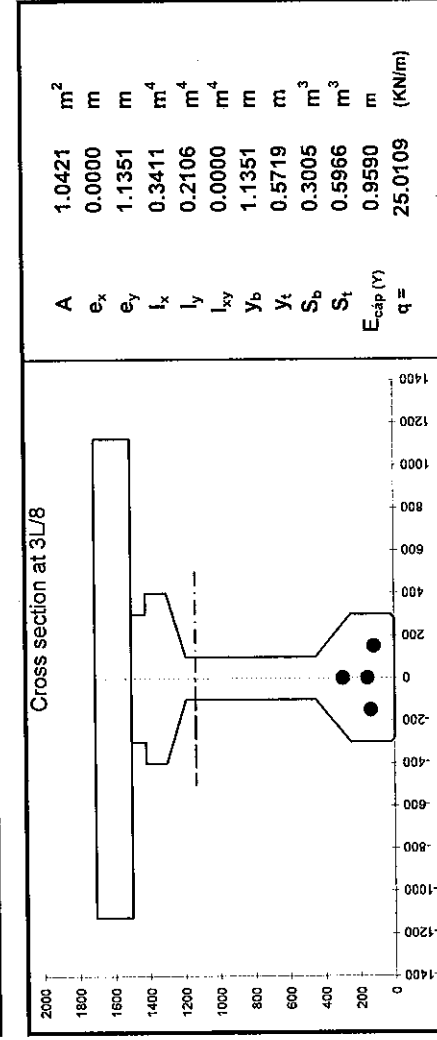
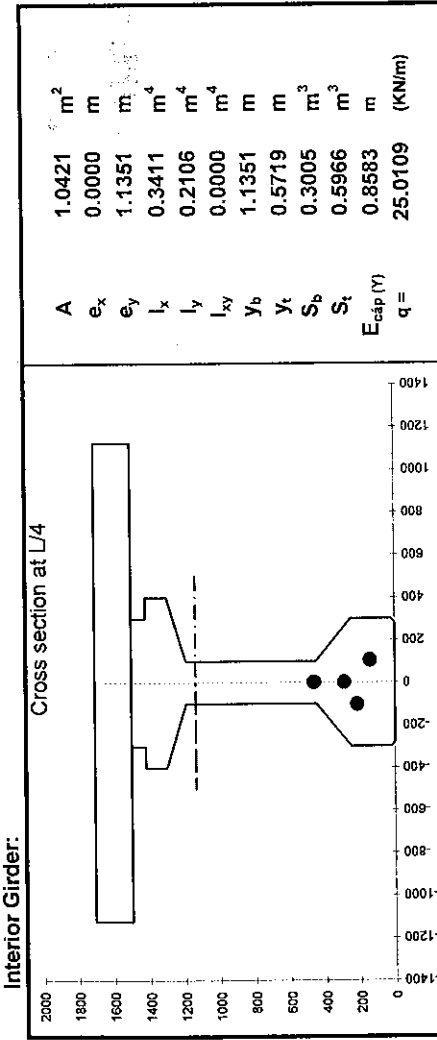
Exterior Girder:



Exterior Girder:



Interior Girder:



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

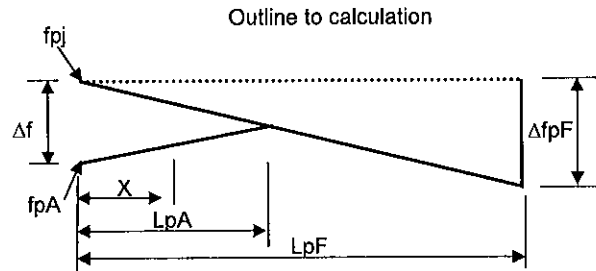
4.1.2 Anchorage seating or Set:

Formula

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

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

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



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 length from anchorage to it is L_{pF}
- Δf The loss stress value at Anchorage

Choose the length from anchorage to point that loss stress due to friction was known (L_{pF}) 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)
$L_{pF} =$	13350	0	166.33
$\Delta f_{pF} =$	83.16	300	162.59
$L_{pA} =$	13350.0	3563	121.94
$\Delta f =$	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

In which:

Number of tendon

Cable modulus of elasticity

Concrete strength at transfer

Unit weight of concrete

Concrete modulus of elasticity at transfer

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

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

$N =$ 4.00 (Tendon)

$E_p =$ 197000.0 MPa

$f_{ci} =$ 40.50 MPa

$\gamma_c =$ 2450.00 kg/m³

$E_{ci} =$ 33185.3 MPa

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

Compression force due to prestressing consider loss stress:

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

A Area of girder cross section

I_x 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 $\Delta f_{pF} + \Delta f_{pA}$	Tendon2 $\Delta f_{pF} + \Delta f_{pA}$	Tendon3 $\Delta f_{pF} + \Delta f_{pA}$	Tendon4 $\Delta f_{pF} + \Delta f_{pA}$	SUM $\Sigma(\Delta f_{pF} + \Delta f_{pA})$	ΣF_j
	(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)	(mm ²)	(mm ⁴)	(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}	$\Sigma \Delta$	F _j ¹	(α)	F _j ¹ *Cos(α)	F _j ¹ *Sin(α)
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}	$\Sigma \Delta$	F _j ²	(α)	F _j ² *Cos(α)	F _j ² *Sin(α)
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}	$\Sigma \Delta$	F _j ³	(α)	F _j ³ *Cos(α)	F _j ³ *Sin(α)
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}	$\Sigma \Delta$	F _j ⁴	(α)	F _j ⁴ *Cos(α)	F _j ⁴ *Sin(α)
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	$F_j \cos(\alpha)$	$F_j \sin(\alpha)$	e_{cdp}	$M_j = \Sigma F_j \cos(\alpha) * e_{cdp}$
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: $\Delta f_{pSH} = (93 - 0.85 * H)$
Relative humidity of environment $H = 80.00 \%$
 $\Delta f_{pSH} = 25.00 \text{ (MPa)}$

4.2.2 Loss of prestress due to Creep:

Formula $\Delta f_{pCR} = 12.0 f_{cgp} - 7.0 * \Delta f_{cdp}$
In which:
 f_{cgp} Stress in concrete at tendons centroid (f_{cgp}) due to prestressing tendon and self weigh of girder
 Δf_{cdp} Stress at tendons centroid changes due to permanent load, except dead load action at transfer

Section	Xi	Interior Girder			Exterior Girder	
		f_{cgp}	Δf_{cdp}	Δf_{pCR}	Δf_{cdp}	Δf_{pCR}
	(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
 f_{py} : Specified yield strength of prestressing steel $f_{py} = 1674.00 \text{ (MPa)}$
 f_{pj} : Initial stress in the tendon at the end of stressing

Section	Xi	f_{pj}	Δf_{pR1}
	(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: $\Delta f_{pR2} = 30\% * (138 - 0.3 \Delta f_{pF} - 0.4 \Delta f_{pES} - 0.2 (\Delta f_{pSH} + \Delta f_{pCR}))$

Interior Girder						
Section	Xi	Δf_{pF}	Δf_{pES}	Δf_{pSH}	Δf_{pCR}	Δf_{pR2}
	(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}	Δf_{pSH}	Δf_{pCR}	Δf_{pR2}
	(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 (m)	Δf_{pSH} (MPa)	Δf_{pCR} (MPa)	Δf_{pR1} (MPa)	Δf_{pR2} (MPa)	Sum (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 (m)	Δf_{pSH} (MPa)	Δf_{pCR} (MPa)	Δf_{pR1} (MPa)	Δf_{pR2} (MPa)	Sum (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	$\Sigma \Delta p_T$	F_j^1	(α)	$F_j^1 \cdot \cos(\alpha)$	$F_j^1 \cdot \sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	386.09	1194.95	0.1697	1177.79	201.80
L/8	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	$\Sigma \Delta p_T$	F_j^2	(α)	$F_j^2 \cdot \cos(\alpha)$	$F_j^2 \cdot \sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	357.74	1228.54	0.1281	1218.46	156.99
L/8	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	$\Sigma \Delta p_T$	F_j^3	(α)	$F_j^3 \cdot \cos(\alpha)$	$F_j^3 \cdot \sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	319.14	1274.25	0.0804	1270.14	102.30
L/8	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	$\Sigma \Delta p_T$	F_j^4	(α)	$F_j^4 \cdot \cos(\alpha)$	$F_j^4 \cdot \sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	236.89	1371.66	0.0351	1370.82	48.18
L/8	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	$F_j \cdot \cos(\alpha)$	$V_p = F_j \cdot \sin(\alpha)$	e_{cable}	$M_j = \Sigma F_j \cos(\alpha) \cdot e_{cap}$
Section	(mm)	(kN)	(kN)	(kN)	(m)	(kNm)
Support	0.00	5069.40	5037.21	509.27	0.09	444.4
L/8	0.00	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)	$F_j^1 \cdot \cos(\alpha)$	$F_j^1 \cdot \sin(\alpha)$
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)	$F_j^2 \cdot \cos(\alpha)$	$F_j^2 \cdot \sin(\alpha)$
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)	$F_j^3 \cdot \cos(\alpha)$	$F_j^3 \cdot \sin(\alpha)$
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)	$F_j^4 \cdot \cos(\alpha)$	$F_j^4 \cdot \sin(\alpha)$
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	$F_j \cdot \cos(\alpha)$	$V_p = F_j \cdot \sin(\alpha)$	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:

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

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

Concrete strength at transfer $f_{ci}' = 0.9 f_c = 40.50 \text{ MPa}$

Compression stress Limit at transfer $0.6 f_{ci}' = 24.30 \text{ MPa}$

Tension stress Limit at transfer $0.25 \text{ SQRT}(f_{ci}') < 1.38 = -1.38 \text{ MPa}$

(5.9.4.1.2-1)

Section	Xi	A	St	Sb	Fj*Cos(α)	e	M _{DC}	f _{ti}	f _{bi}	Kiểm tra	
	(mm)	(mm ²)	(mm ³)	(mm ³)	(kN)	(mm)	(kNm)	(MPa)	(MPa)	f _{ti}	f _{bi}
Girder end	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 construction 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	f _c =	45.00 MPa	
Compression stress limit	0.45 f _c =	20.25 MPa	(5.9.4.2.1-1)
Tension stress limit	0.5 SQRT(f _c) =	-3.35 MPa	(5.9.4.2.2-1)

Section	Xi	A	St	Sb	Fi	e	M _{DC}	f _{ti}	f _{bi}	Kiểm tra	
	(mm)	(mm ²)	(mm ³)	(mm ³)	(kN)	(mm)	(kNm)	(MPa)	(MPa)	f _{ti}	f _{bi}
Girder end	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 f _c =	20.25 MPa	(5.9.4.2.1-1)
Tension Stress Limit:	- 0.5*SQRT(f _c) =	-3.35 MPa	

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

Interior Girder

Section	Xi	A	S _i	S _{ig}	P _{pe}	P _{pe} *e _c	M _g + M _s	M _{SDL}	f _i	Check
	(mm)	(mm ²)	(mm ³)	(mm ³)	(kN)	(kNm)	(kNm)	(kNm)	(MPa)	f _i
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

Section	Xi	A	S _i	S _{ig}	P _{pe}	P _{pe} *e _c	M _g + M _s	M _{SDL}	f _i	Check
	(mm)	(mm ²)	(mm ³)	(mm ³)	(kN)	(kNm)	(kNm)	(kNm)	(MPa)	f _i
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	4939.38	1578.74	1199.41	342.04	5.987	OK
L/4	6825	5.77E+05	2.09E+08	9.75E+08	4881.06	2343.58	2056.13	586.36	7.688	OK
3L/8	10088	5.77E+05	2.09E+08	9.75E+08	4881.45	2835.73	2570.17	732.95	7.944	OK
L/2	13350	5.77E+05	2.09E+08	9.75E+08	4812.43	2957.30	2741.51	781.82	8.113	OK

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

Compression Stress Limit: $0.40 f_c = 18.00 \text{ MPa}$ (5.9.4.2.1-1)
 Tension Stress Limit: $-0.5 \cdot \text{SQRT}(f_c) = -3.35 \text{ MPa}$

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

Interior Girder

Setion	Xi (mm)	A (mm ²)	S _t (mm ³)	S _{ig} (mm ³)	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	1019.67	4.271	OK
L/4	6825	5.77E+05	2.09E+08	9.75E+08	4895.84	2350.68	2202.48	586.36	1717.10	5.952	OK
3L/8	10088	5.77E+05	2.09E+08	9.75E+08	4836.58	2809.66	2753.10	732.95	2109.90	6.598	OK
L/2	13350	5.77E+05	2.09E+08	9.75E+08	4837.71	2972.84	2936.64	781.82	2215.68	6.781	OK

Exterior Girder

Setion	Xi (mm)	A (mm ²)	S _t (mm ³)	S _{ig} (mm ³)	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	9.05E+08	5037.21	444.44	0.00	0.00	0.00	1.804	OK
L/8	3563	670320.0	2.17E+08	9.43E+08	4939.38	1578.74	1199.41	342.04	1169.48	4.234	OK
L/4	6825	576600.0	2.09E+08	9.75E+08	4881.06	2343.58	2056.13	586.36	1969.38	5.864	OK
3L/8	10088	576600.0	2.09E+08	9.75E+08	4881.45	2835.73	2570.17	732.95	2419.89	6.454	OK
L/2	13350	576600.0	2.09E+08	9.75E+08	4812.43	2957.30	2741.51	781.82	2541.21	6.663	OK

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

Compression Stress Limit: $0.60 f_c = 27.00 \text{ MPa}$ (5.9.4.2.1-1)
 Tension Stress Limit: $-0.5 \cdot \text{SQRT}(f_c) = -3.35 \text{ MPa}$

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

Interior Girder

Setion	Xi (mm)	A (mm ²)	S _t (mm ³)	S _{ig} (mm ³)	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	1019.67	7.461	OK
L/4	6825	5.77E+05	2.09E+08	9.75E+08	4895.84	2350.68	2202.48	586.36	1717.10	10.142	OK
3L/8	10088	5.77E+05	2.09E+08	9.75E+08	4836.58	2809.66	2753.10	732.95	2109.90	11.032	OK
L/2	13350	5.77E+05	2.09E+08	9.75E+08	4837.71	2972.84	2936.64	781.82	2215.68	11.290	OK

Exterior Girder

Setion	Xi (mm)	A (mm ²)	S _t (mm ³)	S _{ig} (mm ³)	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	4939.38	1578.74	1199.41	342.04	1169.48	7.228	OK
L/4	6825	5.77E+05	2.09E+08	9.75E+08	4881.06	2343.58	2056.13	586.36	1969.38	9.708	OK
3L/8	10088	5.77E+05	2.09E+08	9.75E+08	4881.45	2835.73	2570.17	732.95	2419.89	10.426	OK
L/2	13350	5.77E+05	2.09E+08	9.75E+08	4812.43	2957.30	2741.51	781.82	2541.21	10.719	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: $0.45 f_c = 15.75 \text{ MPa}$ (5.9.4.2.1-1)

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

Setion	Xi (mm)	MSDL (kNm)		S _{ic} (mm ³)		f _t (MPa)		Check	
		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	5.3E+08	0.000	0.000	OK	OK
L/8	3562.50	342.04	342.04	5.2E+08	5.23E+08	0.654	0.654	OK	OK
L/4	6825.00	586.36	586.36	5.3E+08	5.26E+08	1.115	1.115	OK	OK
3L/8	10087.50	732.95	732.95	5.3E+08	5.26E+08	1.393	1.393	OK	OK
L/2	13350.00	781.82	781.82	5.3E+08	5.26E+08	1.486	1.486	OK	OK

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

Compression Stress Limit:

$$0.6 f_c = 21.00 \text{ MPa}$$

(5.9.4.2.1-1)

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

Setion	Xi	MSDL + MLL (kNm)		S _{ic} (mm ³)		f _i (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	5.3E+08	0.000	0.000	OK	OK
L/8	3562.50	1361.72	1511.52	5.2E+08	5.23E+08	2.604	2.890	OK	OK
L/4	6825.00	2303.47	2555.74	5.3E+08	5.26E+08	4.378	4.858	OK	OK
3L/8	10087.50	2842.86	3152.84	5.3E+08	5.26E+08	5.403	5.993	OK	OK
L/2	13350.00	2997.50	3323.02	5.3E+08	5.26E+08	5.697	6.316	OK	OK

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

Compression Stress Limit:

$$0.45 f_c = 20.25 \text{ MPa}$$

(5.9.4.2.1-1)

Tension Stress Limit:

$$-0.5 * \text{SQRT}(f_c) = -3.35 \text{ MPa}$$

(5.9.4.2.1-1)

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

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)	(mm ²)	(mm ³)	(mm ³)	(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	1019.67	5.080	OK
L/4	6825	5.77E+05	2.05E+08	3.01E+08	4895.84	2350.68	2202.48	586.36	1717.10	2.693	OK
3L/8	10088	5.77E+05	2.05E+08	3.01E+08	4836.58	2809.66	2753.10	732.95	2109.90	0.609	OK
L/2	13350	5.77E+05	2.05E+08	3.01E+08	4837.71	2972.84	2936.64	781.82	2215.68	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)	(mm ²)	(mm ³)	(mm ³)	(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	1169.48	5.193	OK
L/4	6825	5.77E+05	2.05E+08	3.01E+08	4881.06	2343.58	2056.13	586.36	1969.38	2.676	OK
3L/8	10088	5.77E+05	2.05E+08	3.01E+08	4881.45	2835.73	2570.17	732.95	2419.89	0.883	OK
L/2	13350	5.77E+05	2.05E+08	3.01E+08	4812.43	2957.30	2741.51	781.82	2541.21	0.035	OK

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

Compression Stress Limit:

$$0.45 f_c = 20.25 \text{ MPa}$$

(5.9.4.2.1-1)

Tension Stress Limit:

$$-0.5 * \text{SQRT}(f_c) = -3.35 \text{ MPa}$$

(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)	(mm ²)	(mm ³)	(mm ³)	(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	1019.67	4.443	OK
L/4	6825	5.77E+05	2.05E+08	3.01E+08	4895.84	2350.68	2202.48	586.36	1717.10	1.550	OK
3L/8	10088	5.77E+05	2.05E+08	3.01E+08	4836.58	2809.66	2753.10	732.95	2109.90	-0.795	OK
L/2	13350	5.77E+05	2.05E+08	3.01E+08	4837.71	2972.84	2936.64	781.82	2215.68	-1.407	OK

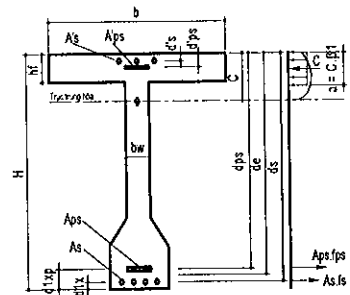
Exterior Girder

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)	(mm ²)	(mm ³)	(mm ³)	(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	1169.48	4.463	OK
L/4	6825	5.77E+05	2.05E+08	3.01E+08	4881.06	2343.58	2056.13	586.36	1969.38	1.366	OK
3L/8	10088	5.77E+05	2.05E+08	3.01E+08	4881.45	2835.73	2570.17	732.95	2419.89	-0.727	OK
L/2	13350	5.77E+05	2.05E+08	3.01E+08	4812.43	2957.30	2741.51	781.82	2541.21	-1.656	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
fy	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	1470	1166	869	577	291
Mu	Flexural Moment	kNm	0	4015	6820	8452	8946
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
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
As	Tension Reinforcement	Number	6	6	6	6	6
		Diameter	mm 16	16	16	16	16
		Area	m2	0.00114	0.00114	0.00114	0.00114
A's	Compression Reinforcement	Number	4	4	4	4	4
		Diameter	mm 12	12	12	12	12
		Area	m2	0.00045	0.00045	0.00045	0.00045
A'c	Shear reinforcement	Number	2	2	2	2	2
		Diameter	mm 14	14	14	14	14
		Area	m2	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 resistance	kNm	7634	9426	10727	11525	11791
Mu	Flexural moment	kNm	0	4015	6820	8452	8946
(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	Minimum reinforcement		0.08%	0.10%	0.11%	0.11%	0.11%
	Minimum reinforcement Checking for RC	0.34%	N.a	N.a	N.a	N.a	N.a
1.2*Mcr	Cracking moment	kNm	1287	1154	1110	1110	1110
(5.7.3.3.2)	Checking $M_r \geq \min(1.2M_{cr}, 1.33M_u)$		OK	OK	OK	OK	OK
(5.8.3.5)	Tensile force in steel should be satisfied - F_{yc}	kN	1602	4916	6443	6937	6885
	Checking $A_s f_y + A_{ps} f_{ps} \geq F_{yc}$		Ok	Ok	Ok	Ok	Ok
(5.7.3.4)	Control of cracking by distr. of reinf for RC member- Check?		No	No	No	No	No
	Existing condition for structure	1, 2 or 3	1	1	1	1	1
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.058	0.058	0.058	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	m ²	0.043	0.043	0.043	0.043	0.043
f _{sa}	Value	Mpa	220	220	220	220	220
0.6*f _y		Mpa	240	240	240	240	240
	Tensile stress in reinf $\min(f_{sa}, 0.6f_y)$	Mpa	220	220	220	220	220
x	Dist. From compression fiber to centroid	m	-	-	-	-	-
J.d	Arm	m	-	-	-	-	-
I _{cr}	Moment of inertia of the cracked section	m ⁴	-	-	-	-	-
f _s	Tensile stress in reinforcement $f_s = M_{sls} / (A_s * J.d)$	Mpa	-	-	-	-	-
	Checking for control cracking $f_s < f_{sa}$		N.a	N.a	N.a	N.a	N.a
(5.10.8.2)	Shrinkage and temperature Reinforcement (side distribution)						
Area _q	Area of required reinf	m ²	0.00033	0.00027	0.00025	0.00025	0.00025
	Distribution on sides 7 D12	m ²	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	26.64	28.77	29.71
α	Angle of Inclination of transv. reinf. to long. Axis	degree	90	90	90	90	90
b _v	Effective web width as minimum web width - in dv	m	0.600	0.320	0.200	0.200	0.200
d _v	Effective shear depth	m	1.224	1.224	1.383	1.478	1.510
	($d_e - a/2$)	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
n _{cat}	Amount of bars in spacing S	bars	2	2	2	2	2
A _v	Shear reinf area in spacing S	m ²	0.0003	0.0003	0.0003	0.0003	0.0003
β	Assume		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/m ²	2224	3308	3490	2168	1070
f _{po}	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1219	1221	1231	1223	1171
e _x	Strain in tensile reinforcement		-3.73E-03	-1.17E-03	-2.57E-05	3.85E-04	5.44E-04
	if $e_x < 0$, multiple with reduce factor		-2.35E-04	-1.31E-04	-4.31E-06	-	-
	Strain checking	$\leq 2.00E-3$	Ok	Ok	Ok	Ok	Ok
v/f _c	Ratio of shear stress and f _c		0.049	0.074	0.078	0.048	0.024
β	Final value		6.8	6.0	4.8	3.0	2.5
θ	Final value	degree	27.00	27.00	26.64	28.77	29.71
V _c	Nominal shear resistance provided by tensile stresses in the concrete	kN	2772	1301	732	487	418
V _s	Shear resistance provided by shear reinforcement	kN	1935	1935	1110	1084	1066
V _p	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0	0
V _{n1}	$V_{n1} = V_c + V_s + V_p$	kN	4707	3235	1841	1571	1483
V _{n2}	V _{n2}	kN	8262	4406	3111	3326	3398
V _n	Nominal shear resistance $V_n = \min(V_{n1}, V_{n2})$	kN	4707	3235	1841	1571	1483
V _r	Factored shear resistance	kN	4238	2912	1657	1414	1335
V _u	Shear	kN	1470	1166	869	577	291
(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	m ²	0.0001	0.0001	0.0001	0.0001	0.0001
	Minimum shear reinforcement Checking		OK	OK	OK	OK	OK
	$0.1 * f_c * b_v * d_v$	kN	3305	1763	1244	1330	1359
	S _{max}	m	0.60	0.60	0.60	0.60	0.60
	Maximum spacing S _{max}		OK	OK	OK	OK	OK

CALCULATION SHEET

***IGIRDER, $l=33.0m$, $S=2.55m$,
Skew=90 degree***

CONTENT

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- 1.2. Girder dimension**
- 1.3. Material properties**
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 - 1.3.2 Prestressing steel
 - 1.3.3 Reinforcing Steel:

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 - 2.1.1 Load:
 - 2.1.2 Internal Force due to dead load:
- 2.2. Live load**
 - 2.2.1. Distribution factors for Live load:
 - 2.2.2 Live Load:
 - 2.2.3 Internal Force due to Live load:
- 2.3 Load combination:**
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3. TENDON PROFILE AND PROPERTY OF GIRDER CROSS SECTION

- 3.1. Tendon profile**
- 3.2. Property of girder cross section at transfer (net cross section)**
- 3.2. Property of girder cross section at service stage (composite cross section)**
 - 3.3.1. Effective flange width
 - 3.3.2. Property of Girder cross section in stage II (service stage):

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 - 4.1.2 Anchorage seating or Set:
 - 4.1.3 Elastic deformation of concrete:
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 - 4.2.1 Loss of prestress due to Shrinkage:
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- 5.1 Stress check during contruction the Girder:**
- 5.2 Stress check during contruction the deck:**
 - 5.2.1 Increase load:
 - 5.2.2 Stress check:
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 - 5.3.2 Due to 1/2 (Prestressing tendon + self weigh of girder) and Live load - Service limit stage I:
 - 5.3.3 Due to prestressing tendon + self weigh of girder + live load - Service limit stage I:
- 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:
 - 5.4.2 Due to additional load (dead load part 2) and live load - Service limit stage I:
- 5.5 Stress check at the bottom fibre of girder - Service III (stage III):**
- 5.6 Stress check at the bottom fibre of girder - Service I (stage III):**

6. ULTIMATE LOAD CHECK AND SHEAR CAPACITY CHECK

- 6.1 Flexural moment checking**
- 6.2. Ultimate load check**

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/m²)

Length of Girder

L_d = 33.00 (m)

Span between support

L_{ti} = 32.20 (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 between 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)

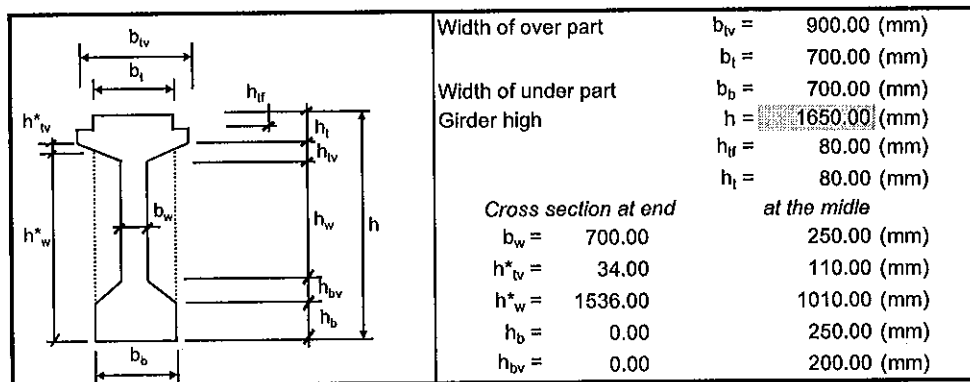
Precast plank thick

h_p = 0.08 (m)

Pavement thick

h_{pa} = 0.084 (m)

1.2. Girder dimension:



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/m³

Modulus of elasticity

E_c = 0.043 γ_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/m³

Modulus of elasticity

E_c = 0.043 γ_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 mm²

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

Coefficient 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 mm²

Stress in the prestressing steel at jacking

f_{pi} = 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 mm²

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	$DC_d =$	13.32 (kN/m)
Precat plank & cross beam	$DC_{pi} =$	4.64 (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	$DC_{pi} =$	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 :

$$M = 0.5 q_d X_i (L - X_i)$$

$$Q = q_d (0.5L - X_i)$$

$$L_n = 32.20 \text{ (m)}$$

INTERIOR GIRDER											
Section	X_i (m)	Girder (DC)		Concrete Deck (DC)		Plank & cr beam (DC)		Parapet (DC)		Pavement (DW)	
		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	326.91	0.00	214.42	0.00	74.77	0.00	76.31	0.00	71.51
L/8	4.03	1151.35	245.19	755.16	160.81	263.33	56.08	268.77	57.24	251.84	53.63
L/4	8.05	1973.74	163.46	1294.56	107.21	451.42	37.38	460.75	38.16	431.73	35.75
3L/8	12.08	2467.17	81.73	1618.19	53.60	564.28	18.69	575.93	19.08	539.66	17.88
L/2	16.10	2631.65	0.00	1726.07	0.00	601.90	0.00	614.33	0.00	575.64	0.00
EXTERIOR GIRDER											
Gđl	0.00	0.00	326.91	0.00	214.42	0.00	74.77	0.00	77.20	0.00	71.51
L/8	4.03	1151.35	245.19	755.16	160.81	131.66	56.08	271.89	57.90	251.84	53.63
L/4	8.05	1973.74	163.46	1294.56	107.21	225.71	37.38	466.09	38.60	431.73	35.75
3L/8	12.08	2467.17	81.73	1618.19	53.60	282.14	18.69	582.61	19.30	539.66	17.88
L/2	16.10	2631.65	0.00	1726.07	0.00	300.95	0.00	621.46	0.00	575.64	0.00

2.2 Live Load:

2.2.1. Distribution factors for Live load:

Modular Ratio: Girder Concrete/Deck Concrete

Distance from girder centroid to bridge deck centroid

Longitudinal stiffness parameter

Ration

$$n = E_g / E_d = 1.13$$

$$e^I_g = 934.27 \text{ (mm)}$$

$$e^E_g = 934.27 \text{ (mm)}$$

$$K^I_g = n(I_g + A e^I_g) = 1.7E+12$$

$$K^E_g = n(I_g + A e^E_g) = 1.7E+12$$

$$K^I_g / (L^3) = 4.83$$

$$K^E_g / (L^3) = 4.83$$

$$S / L = 0.08$$

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

Interior Beam:

$$\text{For one lane} \quad 0.06 + \left(\frac{S}{4300} \right)^{0.4} \left(\frac{S}{L} \right)^{0.3} \left(\frac{K_g}{L^3} \right)^{0.1} = 0.504$$

$$\text{Two or more lanes} \quad 0.075 + \left(\frac{S}{2900} \right)^{0.6} \left(\frac{S}{L} \right)^{0.2} \left(\frac{K_g}{L^3} \right)^{0.1} = 0.727$$

Exterior Beam:

For one lane, follow the lever rule

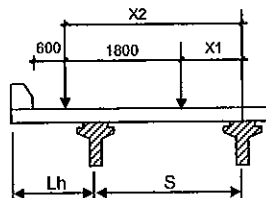
$$X_1 = 920.000$$

$$X_2 = 2720.00$$

$$Y_1 = 0.361$$

$$Y_2 = 1.067$$

$$\Rightarrow g(M) = 0.5 \sum y_i = 0.714$$



Two or more lanes

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

$$\text{Choice } e = 1.045 \quad \text{IF}(e > 1, 1, e)$$

$$\Rightarrow g(M) = e \cdot g_{\text{long}} = 0.760$$

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

Interior Beam:

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

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

$$g(Q) = 0.5 \sum y_i = 0.714$$

Two or more lanes

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

$$\Rightarrow g(Q) = e \cdot g_{\text{long}} = 0.729$$

(c) Correction factor for skew bridge:

* Correction factor of distribution factor for moment (Table 4.6.2.2d-1)

Skew angle	$\theta = 0$	Degree.	Area of applications
Factor	$c1 = 0.000$		$300 \leq \theta \leq 600$
Correction factor	$CF(M) = 1.000$		$1100 \leq S \leq 4900$
			$6000 \leq L \leq 73000$
			$Nb \geq 4$

$$CF(M) = 1.0 - c1 (\tan \theta)^{1.5}$$

$$c1 = 0.25 \left(\frac{Kg}{Ls^3} \right)^{0.25} \left(\frac{S}{L} \right)^{0.5}$$

* Regulation factor of distribution factor for shear force (Table 4.6.2.2c-1)

Correction Factor $CF(Q) = 1.000$

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

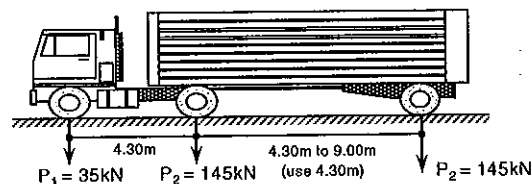
Area of applications
$00 \leq \theta \leq 600$
$1100 \leq S \leq 4900$
$6000 \leq L \leq 73000$
$Nb \geq 4$

(d) Table of Distribution factors for Live load:

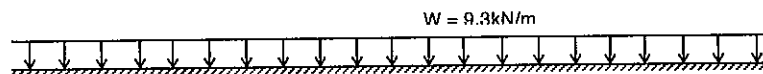
Interior Beam	$g(M)$	$g(Q)$	m	$m^*g(M)$	$m^*g(Q)$	$m^*g(M) \cdot CF(M)$	$m^*g(Q) \cdot CF(Q)$
1 lane	0.504	0.696	1.20	0.605	0.835	0.605	0.835
2 or more lanes	0.727	0.852	1.00	0.727	0.852	0.727	0.852
Exterior Beam							
1 lane	0.714	0.714	1.20	0.856	0.856	0.856	0.856
2 or more lanes	0.760	0.729	1.00	0.760	0.729	0.760	0.729

2.2.2 Live Load:

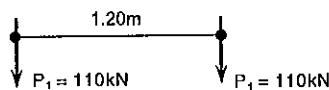
Design Truck



Design Lane Load



Design Tandem



- Truck	$P1 = 35.00 \text{ (kN)}$
	$P2 = 145.00 \text{ (kN)}$
- Lane load	$W = 9.30 \text{ (kN)}$
- Tandem	$P1 = 110.00 \text{ (kN)}$
- Pedestrian	$PL = 0.00 \text{ kN/m}^2$
- Dynamic load	$IM = 0.25$

2.2.3 Internal Force due to Live load:

Design truck or Tandem

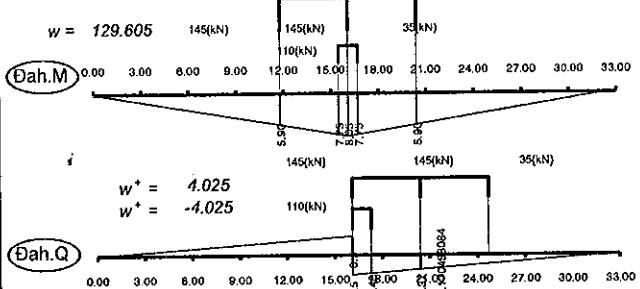
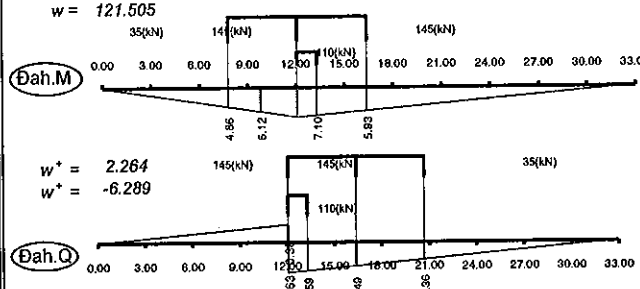
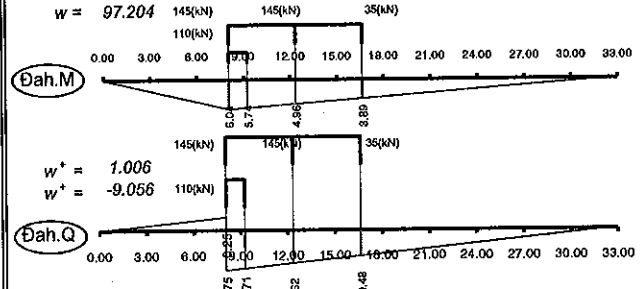
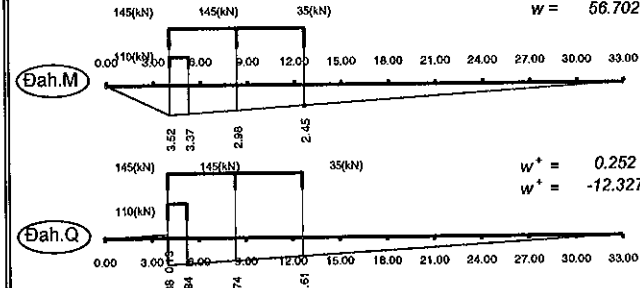
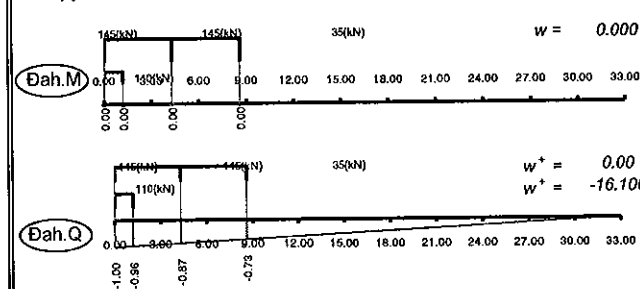
Momen	$M_{TR(Te)} = \sum P_i y_i$	(kNm)
Shear force	$Q_{TR(Te)} = \sum P_i y_i$	(kN)

Lane load

Momen	$M_{Ln} = W \cdot F_M$	(kNm)
Shear force	$Q_{Ln} = W \cdot F_Q$	(kN)

Pedestrian

Momen	$M_{PL} = PL \cdot F_M$	(kNm)
Shear force	$Q_{PL} = PL \cdot F_Q$	(kN)

Influence line for Momen & Shear force		Load	Momen (KN.m)	Shear
Section at 1/2L		Truck	2229.25	133.79
 <p> $w = 129.605$ $w^+ = 4.025$ $w^- = -4.025$ </p>		Lane	1205.33	37.43
		Tandem	1705.00	105.90
		Design	2229.25	133.79
		Pedestrian	0.00	0.00
Section at 3/8L		Truck	2124.86	174.41
 <p> $w = 121.505$ $w^+ = 2.264$ $w^- = -6.289$ </p>		Lane	1129.99	58.49
		Tandem	1610.81	133.40
		Design	2124.86	174.41
		Pedestrian	0.00	0.00
Section at 1/4L		Truck	1731.06	215.04
 <p> $w = 97.204$ $w^+ = 1.006$ $w^- = -9.056$ </p>		Lane	903.99	84.22
		Tandem	1295.25	160.90
		Design	1731.06	215.04
		Pedestrian	0.00	0.00
Section at 1/8L		Truck	1029.05	255.66
 <p> $w = 56.702$ $w^+ = 0.252$ $w^- = -12.327$ </p>		Lane	527.33	114.64
		Tandem	758.31	188.40
		Design	1029.05	255.66
		Pedestrian	0.00	0.00
At support		Truck	0.00	296.29
 <p> $w = 0.000$ $w^+ = 0.00$ $w^- = -16.100$ </p>		Lane	0.00	149.73
		Tandem	0.00	215.90
		Design	0.00	296.29
		Pedestrian	0.00	0.00

Internal Force due to Live load :

$$M_{(LL+IM)} = m \cdot g(M) \cdot [\max(M_{TR}, M_{Ta}) \cdot (1+IM) + M_{Ln}]$$

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

Internal Force due to pedestrian :

$$M = g(M) \cdot M_{PL}$$

$$Q = g(Q) \cdot Q_{PL}$$

In which:

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

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

yl Value of influence line

F Area of influence line

m Lane factor

g

Distribution factor

	$m \cdot g(M)$	$m \cdot g(Q)$
Interior	0.727	0.852
Exterior	0.856	0.856

TABLE OF INTERNAL FORCE DUE TO LIVE LOAD

Setion	Xi (m)	Interior Girder		Exterior Girder	
		M (kNm)	Q (kN)	M (kNm)	Q (kN)
Support	0.00	0.00	442.88	0.00	445.44
L/8	4.03	1319.41	369.75	1553.33	371.89
L/4	8.05	2231.82	300.61	2627.50	302.35
3L/8	12.08	2754.34	235.45	3242.66	236.82
L/2	16.10	2904.08	174.28	3418.94	175.29

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	
		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	865.52	3048.25	649.14	5225.58	432.76	6531.97	216.38	6967.44	0.00
DW	1.50	0.00	107.26	377.76	80.45	647.60	53.63	809.49	26.82	863.46	0.00
LL+IM	1.75	0.00	775.04	2308.97	647.07	3905.69	526.07	4820.10	412.05	5082.14	305.00
Total		0.00	1747.82	5734.99	1376.65	9778.87	1012.46	12161.57	655.24	12913.03	305.00

STATE		Service									
Load	Load Factor	Section									
		Support		L/8		L/4		3L/8		L/2	
		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	692.42	2438.60	519.31	4180.46	346.21	5225.58	173.10	5573.95	0.00
DW	1.00	0.00	71.51	251.84	53.63	431.73	35.75	539.66	17.88	575.64	0.00
LL+IM	1.00	0.00	442.88	1319.41	369.75	2231.82	300.61	2754.34	235.45	2904.08	174.28
Total		0.00	1206.80	4009.86	942.70	6844.02	682.57	8519.59	426.44	9053.67	174.28

2.3.2 Load combination - Exterior Girder:

STATE		Strength									
Load	Load factor	Section									
		Supprt		L/8		L/4		3L/8		L/2	
		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	866.63	2887.57	649.97	4950.12	433.31	6187.65	216.66	6600.16	0.00
DW	1.50	0.00	107.26	377.76	80.45	647.60	53.63	809.49	26.82	863.46	0.00
LL+IM	1.75	0.00	779.52	2718.32	650.81	4598.13	529.12	5674.65	414.43	5983.14	306.76
Total		0.00	1753.41	5983.66	1381.23	10195.84	1016.06	12671.80	657.91	13446.76	306.76

STATE		Service									
Load	load factor	Section									
		Support		L/8		L/4		3L/8		L/2	
		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	693.30	2310.06	519.98	3960.10	346.65	4950.12	173.33	5280.13	0.00
DW	1.00	0.00	71.51	251.84	53.63	431.73	35.75	539.66	17.88	575.64	0.00
LL+IM	1.00	0.00	445.44	1553.33	371.89	2627.50	302.35	3242.66	236.82	3418.94	175.29
Total		0.00	1210.25	4115.23	945.50	7019.33	684.76	8732.44	428.02	9274.71	175.29

3. TENDON PROFILE AND PROPERTY OF GIRDER CROSS SECTION

3.1. Tendon profile:

Tendon profile follow Parabol equation:

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

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 = X₂ - X₁ Distance between 2 point under consider

angle of rotation of tendon for X_i-axis

$$\tan(\alpha) = (4 \cdot f \cdot (1 - 2 \cdot X_i / L)) / L$$

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

L _{span}	=	33000	(mm)
L _{su.}	=	32200	(mm)
L _{cap}	=	32700	(mm)

TENDON No 1	f =	1440	(mm)	Lcáp =	32700	(mm)	C =	390	(mm)
	Section	XI	Yi	Lp	ΣLcáp	Tan(α)	(α)	Σα	Section
		(mm)	(mm)	(mm)	(mm)		(rad)	(rad)	
	Anchorage	0.00	1440.00	0.00	0.00	0.1761	0.0000	0.0000	Anchorage
	Support	250.00	1408.14	250.00	250.00	0.1735	0.0027	0.0027	Support
	L/8	4275.00	962.70	4025.00	4275.00	0.1301	0.0461	0.0488	L/8
	L/4	8300.00	644.53	4025.00	8300.00	0.0867	0.0894	0.1382	L/4
	3L/8	12325.00	453.63	4025.00	12325.00	0.0434	0.1328	0.2710	3L/8
	L/2	16350.00	390.00	4025.00	16350.00	0.0000	0.1761	0.4471	L/2

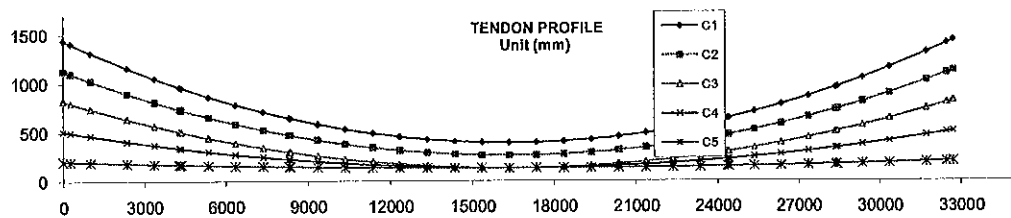
TENDON No 2	f =	1130	(mm)	Lcáp =	32700	(mm)	C =	260	(mm)
	Section	Xi	Yi	Lp	ΣLcáp	Tan(α)	(α)	Σα	Section
		(mm)	(mm)	(mm)	(mm)		(rad)	(rad)	
	Anchorage	0.00	1130.00	0.00	0.00	0.1382	0.0000	0.0000	Anchorage
	Support	250.00	1103.60	250.00	250.00	0.1361	0.0021	0.0021	Support
	L/8	4275.00	734.52	4025.00	4275.00	0.1021	0.0361	0.0383	L/8
	L/4	8300.00	470.90	4025.00	8300.00	0.0681	0.0702	0.1084	L/4
	3L/8	12325.00	312.72	4025.00	12325.00	0.0340	0.1042	0.2126	3L/8
	L/2	16350.00	260.00	4025.00	16350.00	0.0000	0.1382	0.3508	L/2

TENDON No 3	f =	820	(mm)	L _{cap} =	32700	(mm)	C =	130	(mm)
	Section	X _i	Y _i	L _p	ΣL _{cap}	Tan(α)	(α)	Σα	Section
		(mm)	(mm)	(mm)	(mm)		(rad)	(rad)	
	Anchorage	0.00	820.00	0.00	0.00	0.1003	0.0000	0.0000	Anchorage
	Support	250.00	799.06	250.00	250.00	0.0988	0.0015	0.0015	Support
	L/8	4275.00	506.35	4025.00	4275.00	0.0741	0.0262	0.0278	L/8
	L/4	8300.00	297.27	4025.00	8300.00	0.0494	0.0509	0.0787	L/4
	3L/8	12325.00	171.82	4025.00	12325.00	0.0247	0.0756	0.1543	3L/8
	L/2	16350.00	130.00	4025.00	16350.00	0.0000	0.1003	0.2546	L/2

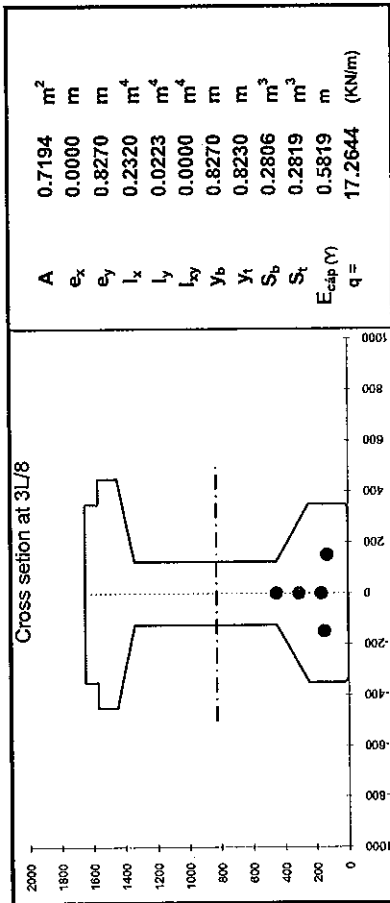
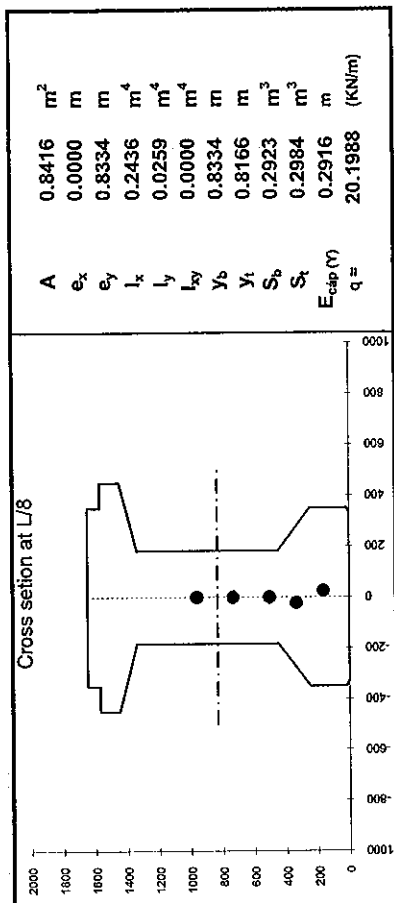
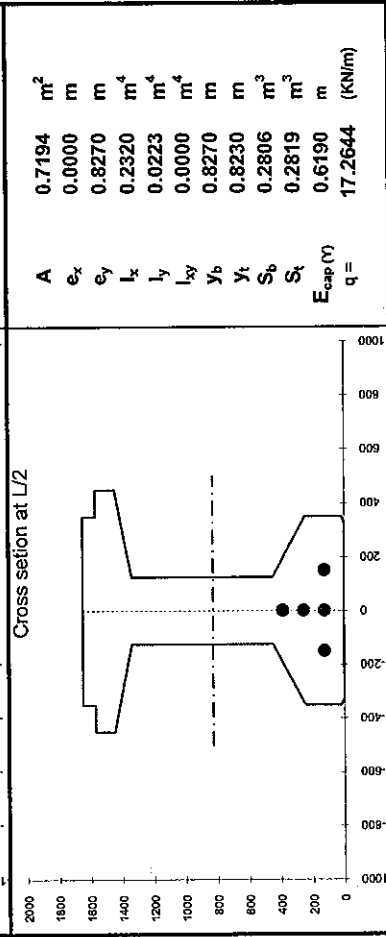
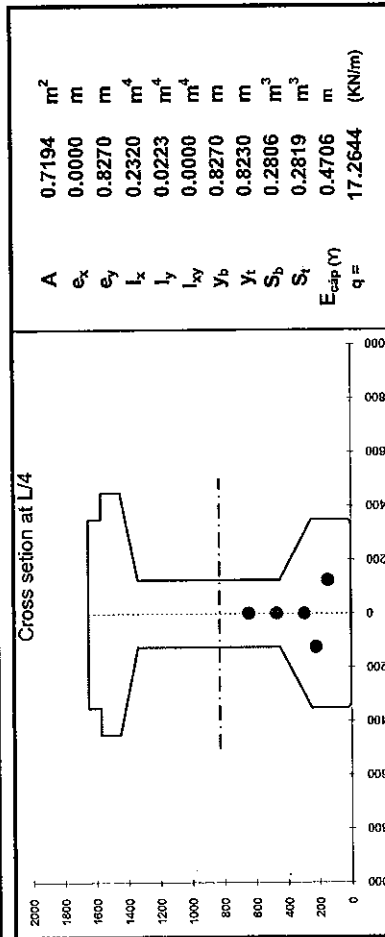
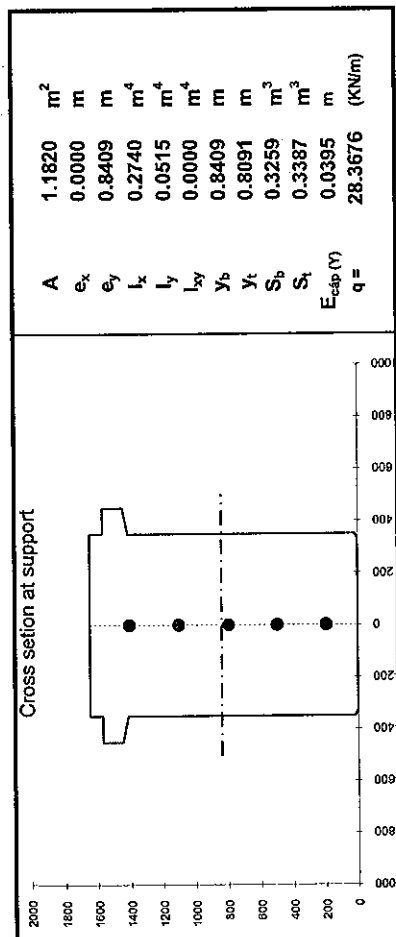
TENDON No 4	f =	510	(mm)	Lcáp =	32700	(mm)	C =	130	(mm)
	Section	Xi	Yi	Lp	ΣLcáp	Tan(α)	(α)	Σα	Section
		(mm)	(mm)	(mm)	(mm)		(rad)	(rad)	
	Anchorage	0.00	510.00	0.00	0.0	0.0624	0.0000	0.0000	Anchorage
	Support	250.00	498.47	250.00	250.0	0.0614	0.0010	0.0010	Support
	L/8	4275.00	337.26	4025.00	4275.0	0.0461	0.0163	0.0173	L/8
	L/4	8300.00	222.12	4025.00	8300.0	0.0307	0.0317	0.0489	L/4
	3L/8	12325.00	153.03	4025.00	12325.0	0.0154	0.0470	0.0960	3L/8
	L/2	16350.00	130.00	4025.00	16350.0	0.0000	0.0624	0.1583	L/2

TENDON No 5	f =	200	(mm)	Lcáp =	32700	(mm)	C =	130	(mm)
	Section	XI	YI	Lp	ΣLcáp	Tan(α)	(α)	Σα	Section
		(mm)	(mm)	(mm)	(mm)		(rad)	(rad)	
	Anchorage	0.00	200.00	0.00	0.0	0.0245	0.0000	0.0000	Anchorage
	Support	250.00	197.88	250.00	250.0	0.0241	0.0004	0.0004	Support
	L/8	4275.00	168.18	4025.00	4275.0	0.0181	0.0064	0.0068	L/8
	L/4	8300.00	146.97	4025.00	8300.0	0.0120	0.0124	0.0192	L/4
	3L/8	12325.00	134.24	4025.00	12325.0	0.0060	0.0184	0.0376	3L/8
	L/2	16350.00	130.00	4025.00	16350.0	0.0000	0.0245	0.0621	L/2

Section	TENDON No 1		TENDON No 2		TENDON No 3		TENDON No 4		TENDON No 5	
	X _i (mm)	Y _i (mm)	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	1440.00	0.0	1130.00	0.0	820.00	0.00	510.00	0.00	200.00
Support	250.00	1408.14	250.0	1103.60	250.0	799.06	250.00	498.47	250.00	197.88
1	1000.00	1315.49	1000.0	1026.83	1000.0	738.18	1000.00	464.94	1000.00	191.70
2	2350.00	1159.86	2350.0	897.88	2350.0	635.91	2350.00	408.61	2350.00	181.32
3	3350.00	1053.80	3350.0	810.01	3350.0	566.21	3350.00	370.23	3350.00	174.25
L/8	4275.00	962.70	4275.0	734.52	4275.0	506.35	4275.00	337.26	4275.00	168.18
4	4350.00	955.61	4350.0	728.65	4350.0	501.69	4350.00	334.70	4350.00	167.71
5	5350.00	865.27	5350.0	653.79	5350.0	442.32	5350.00	302.00	5350.00	161.68
6	6350.00	782.78	6350.0	585.45	6350.0	388.12	6350.00	272.15	6350.00	156.19
7	7350.00	708.16	7350.0	523.61	7350.0	339.07	7350.00	245.14	7350.00	151.21
L/4	8300.00	644.53	8300.0	470.90	8300.0	297.27	8300.00	222.12	8300.00	146.97
8	8350.00	641.38	8350.0	468.29	8350.0	295.19	8350.00	220.98	8350.00	146.76
9	9350.00	582.46	9350.0	419.47	9350.0	256.48	9350.00	199.65	9350.00	142.83
10	10350.00	531.40	10350.0	377.16	10350.0	222.92	10350.00	181.17	10350.00	139.43
11	11350.00	488.20	11350.0	341.36	11350.0	194.53	11350.00	165.54	11350.00	136.55
3L/8	12325.00	453.63	12325.0	312.72	12325.0	171.82	12325.00	153.03	12325.00	134.24
12	12350.00	452.85	12350.0	312.07	12350.0	171.30	12350.00	152.74	12350.00	134.19
13	13350.00	425.35	13350.0	289.29	13350.0	153.23	13350.00	142.79	13350.00	132.36
14	14350.00	405.71	14350.0	273.02	14350.0	140.32	14350.00	135.69	14350.00	131.05
15	15350.00	393.93	15350.0	263.25	15350.0	132.58	15350.00	131.42	15350.00	130.26
L/2	16350.00	390.00	16350.0	260.00	16350.0	130.00	16350.00	130.00	16350.00	130.00
2	17350.00	393.93	17350.0	263.25	17350.0	132.58	17350.00	131.42	17350.00	130.26
3	18350.00	405.71	18350.0	273.02	18350.0	140.32	18350.00	135.69	18350.00	131.05
4	19350.00	425.35	19350.0	289.29	19350.0	153.23	19350.00	142.79	19350.00	132.36
5	20350.00	452.85	20350.0	312.07	20350.0	171.30	20350.00	152.74	20350.00	134.19
-	20375.00	453.63	20375.0	312.72	20375.0	171.82	20375.00	153.03	20375.00	134.24
6	21350.00	488.20	21350.0	341.36	21350.0	194.53	21350.00	165.54	21350.00	136.55
7	22350.00	531.40	22350.0	377.16	22350.0	222.92	22350.00	181.17	22350.00	139.43
8	23350.00	582.46	23350.0	419.47	23350.0	256.48	23350.00	199.65	23350.00	142.83
9	24350.00	641.38	24350.0	468.29	24350.0	295.19	24350.00	220.98	24350.00	146.76
-	24400.00	644.53	24400.0	470.90	24400.0	297.27	24400.00	222.12	24400.00	146.97
10	25350.00	708.16	25350.0	523.61	25350.0	339.07	25350.00	245.14	25350.00	151.21
11	26350.00	782.78	26350.0	585.45	26350.0	388.12	26350.00	272.15	26350.00	156.19
12	27350.00	865.27	27350.0	653.79	27350.0	442.32	27350.00	302.00	27350.00	161.68
13	28350.00	955.61	28350.0	728.65	28350.0	501.69	28350.00	334.70	28350.00	167.71
-	28425.00	962.70	28425.0	734.52	28425.0	506.35	28425.00	337.26	28425.00	168.18
14	29350.00	1053.80	29350.0	810.01	29350.0	566.21	29350.00	370.23	29350.00	174.25
14	30350.00	1159.86	30350.0	897.88	30350.0	635.91	30350.00	408.61	30350.00	181.32
16	31700.00	1315.49	31700.0	1026.83	31700.0	738.18	31700.00	464.94	31700.00	191.70
Support	32450.00	1408.14	32450.0	1103.60	32450.0	799.06	32450.00	498.47	32450.00	197.88
Anchorage	32700.00	1440.00	32700.0	1130.00	32700.0	820.00	32700.00	510.00	32700.00	200.00



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



Uniform load due to self weight of Girder in Stage 1: $Q = 20.31 \text{ (KN/m)}$

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

3.3.1. Effective flange width

Modular Ratio: Deck Concrete/Girder Concrete $n = E_b / E_d = 0.88$

For Interior Girder:

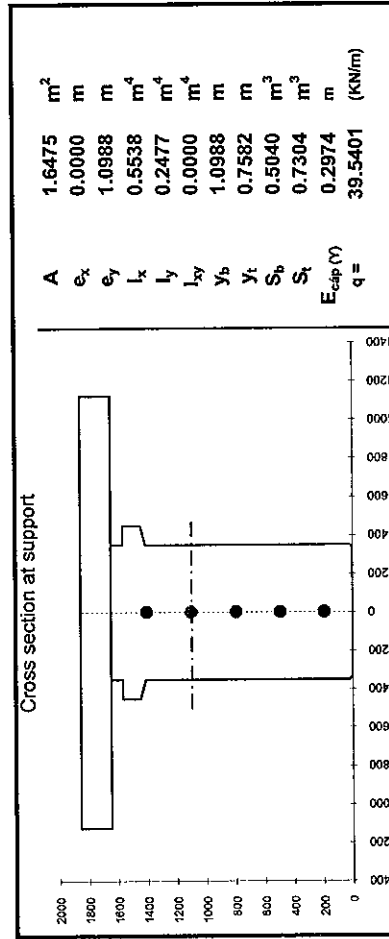
$$b_f = \min \left\{ \begin{array}{l} 1/4 L_u \\ 12 h_f + \max(0.5 b_w, b_w) \end{array} \right\} S \Rightarrow n^* b_f = 2248.88861 \text{ (mm)}$$

For Exterior Girder:

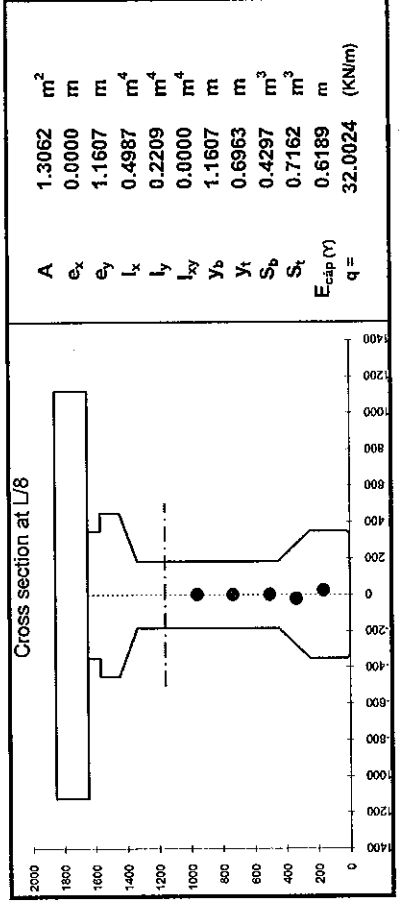
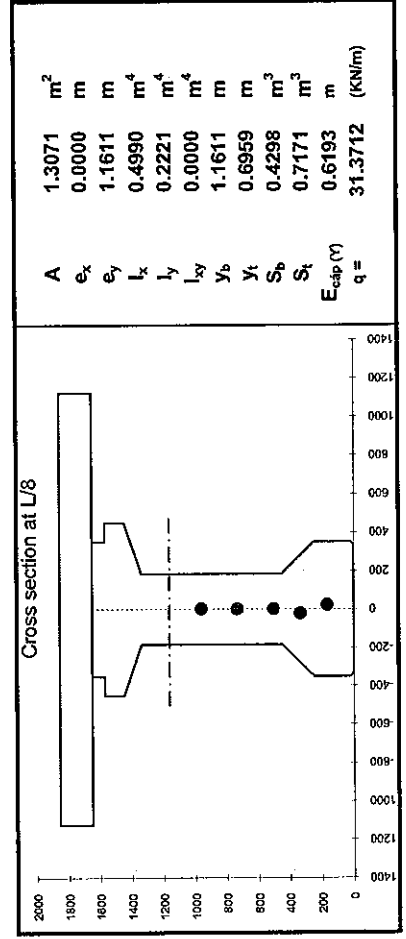
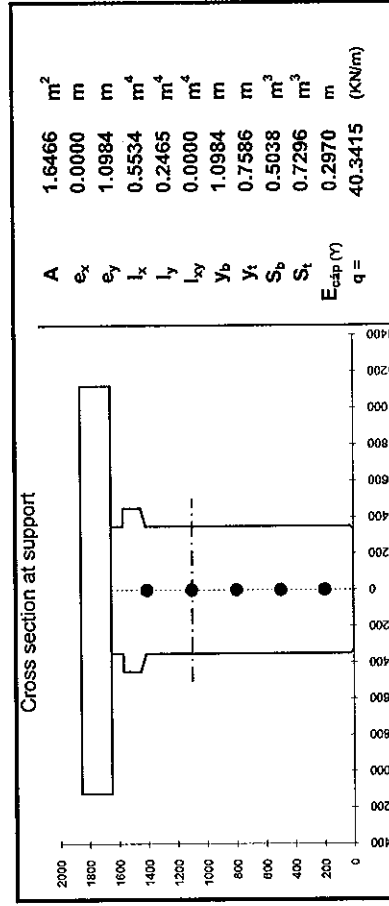
$$b_E = 0.5 b_f + \min \left\{ \begin{array}{l} 1/8 L_u \\ 6 h_f + \max(0.5 b_w, 0.25 b) \end{array} \right\} L_n \Rightarrow n^* b_E = 2244.47903 \text{ (mm)}$$

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

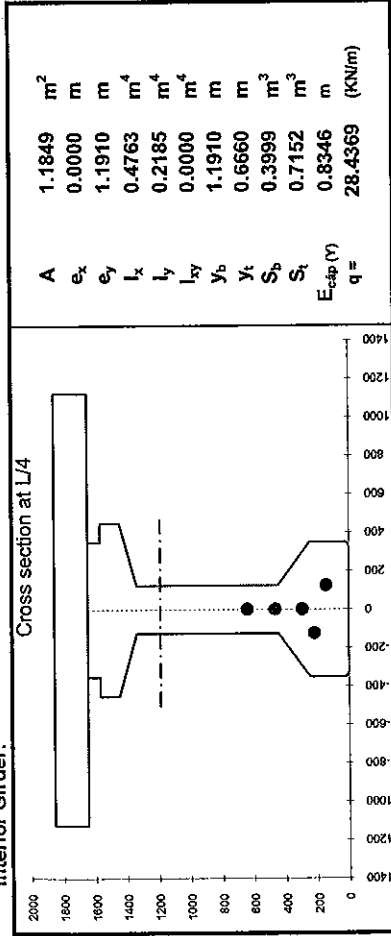
Interior Girder:



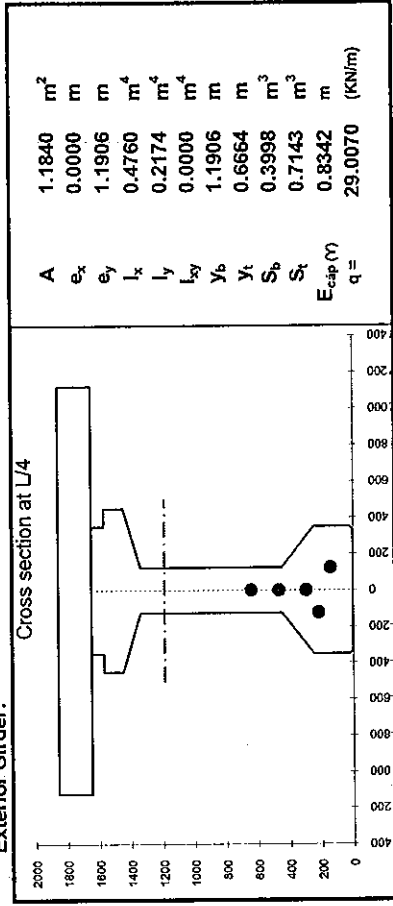
Exterior Girder:



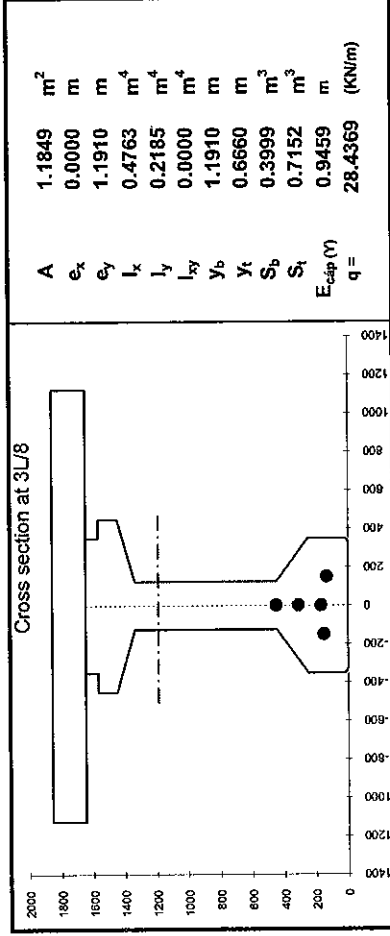
Interior Girder:



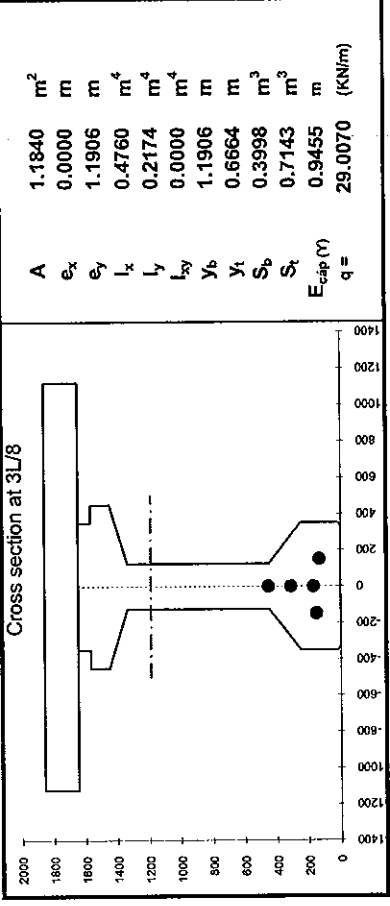
Exterior Girder:



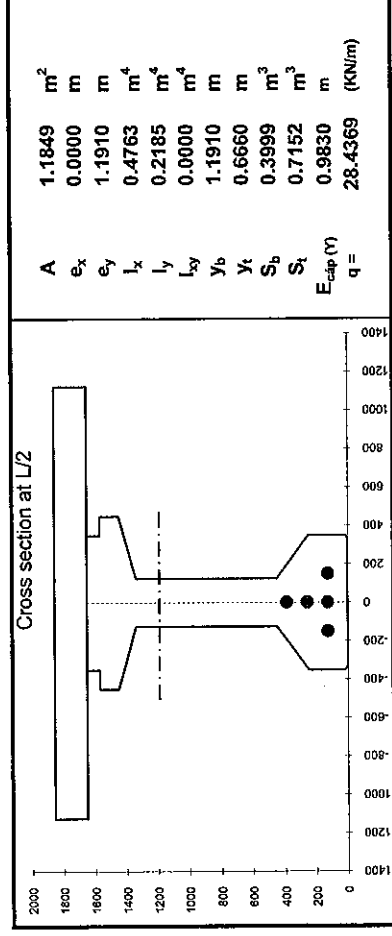
Cross section at 3L/8



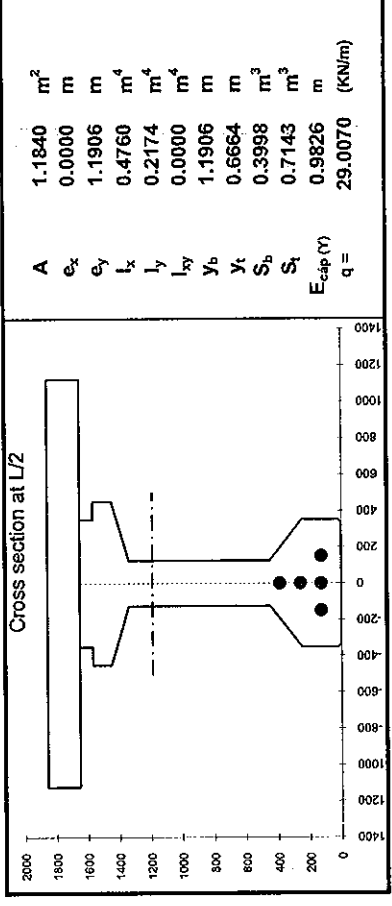
Cross section at 3L/8



Cross section at L/2



Cross section at L/2



4. LOSS OF PRESTRESS

4.1 Loss of prestressing force immediately (Instantaneous losses):

4.1.1 Friction between Prestressing Tendon and Duck:

Formula: $\Delta f_{pF} = f_{p1} (1 - e^{-(kx + \mu\alpha)})$ (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	Tendon no. 5	SUM/5
	$\Sigma\alpha$ (rad)	Δf_{pF} (MPa)	$\Sigma\alpha$ (rad)	Δf_{pF} (MPa)	$\Sigma\alpha$ (rad)	Δf_{pF} (MPa)
Ancho.	0.00	0.00	0.00	0.00	0.00	0.00
Support	0.0027	1.09	0.0021	0.90	0.0015	0.71
L/8	0.0488	19.40	0.0383	16.03	0.0278	12.65
L/4	0.1382	51.08	0.1084	41.74	0.0787	32.33
3L/8	0.2710	95.13	0.2126	77.40	0.1543	59.42
L/2	0.4471	150.19	0.3508	122.14	0.2546	93.40

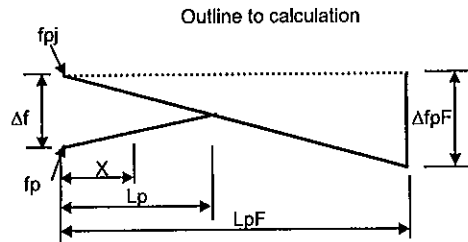
4.1.2 Anchorage seating or Set:

Formula

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

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

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



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 (L_{pF}) and calculation follow:

Tendon no.1	Xi (mm)	Δf_{pA} (MPa)
$L_{pF} =$	16350	0
$\Delta f_{pF} =$	150.19	250
$L_{pA} =$	11343.5	4275
$\Delta f =$	208.40	8300
	12325	0.00
	16350	0.00

Tendon no.2	Xi (mm)	Δf_{pA} (MPa)
$L_{pF} =$	16350	0
$\Delta f_{pF} =$	122.14	250
$L_{pA} =$	12578.9	4275
$\Delta f =$	187.93	8300
	12325	3.79
	16350	0.00

Tendon no.3	Xi (mm)	Δf_{pA} (MPa)
$L_{pF} =$	16350	0
$\Delta f_{pF} =$	93.40	250
$L_{pA} =$	14384.3	4275
$\Delta f =$	164.35	8300
	12325	23.53
	16350	0.00

Tendon no.4	Xi (mm)	Δf_{pA} (MPa)
$L_{pF} =$	16350	0
$\Delta f_{pF} =$	63.97	250
$L_{pA} =$	16350.0	4275
$\Delta f =$	127.93	8300
	12325	31.49
	16350	0.00

Tendon no.5	Xi (mm)	Δf_{pA} (MPa)
$L_{pF} =$	16350	0
$\Delta f_{pF} =$	33.82	250
$L_{pA} =$	16350.0	4275
$\Delta f =$	67.63	8300
	12325	16.65
	16350	0.00

4.1.3 Elastic deformation of concrete:

Formula

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

In which:

Number of tendon	N = 5.00 (Tendon)
Cable modulus of elasticity	$E_p = 197000.0$ MPa
Concrete strength at transfer	$f_{ci} = 40.50$ MPa
Unit weight of concrete	$\gamma_c = 2450.00$ kg/m ³
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

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

Compression force due to prestressing consider loss stress:

$$F_j = N \cdot f_{pj} \cdot A_g - A_g \cdot \Sigma (\Delta f_{pF} + \Delta f_{pA})$$

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

Total loss stress due to friction and Anchorage:

Section	X_i (mm)	Tendon1 $\Delta f_{pF} + \Delta f_{pA}$ (MPa)	Tendon2 $\Delta f_{pF} + \Delta f_{pA}$ (MPa)	Tendon3 $\Delta f_{pF} + \Delta f_{pA}$ (MPa)	Tendon4 $\Delta f_{pF} + \Delta f_{pA}$ (MPa)	Tendon5 $\Delta f_{pF} + \Delta f_{pA}$ (MPa)	SUM ($\Delta f_{pF} + \Delta f_{pA}$) (MPa)	ΣF_j (kN)
Anchorage	0	208.40	187.93	164.35	127.93	67.83	756.25	8518.43
Support	250	204.90	185.10	162.20	126.50	66.93	745.64	8534.13
L/8	4275	149.26	140.09	128.15	103.74	55.81	577.05	8783.73
L/4	8300	108.99	105.67	101.85	85.85	46.61	446.97	8976.32
3L/8	12325	95.13	81.20	82.94	72.66	39.29	371.22	9088.46
L/2	16350	150.19	122.14	93.40	63.97	33.82	463.51	8951.83

Loss stress due to Elastic deformation of concrete

Section	X_i (mm)	F_j (kN)	A (mm ²)	I_x (mm ⁴)	e (mm)	M_{oc} (kNm)	f_{cgp} (MPa)	Δf_{ES} (MPa)
Anchorage	0	8518.43	1.2E+06	2.7E+11	39.52	0.00	7.26	17.23
Support	250	8534.13	1.2E+06	2.7E+11	39.52	0.00	7.27	17.26
L/8	4275	8783.73	8.4E+05	2.4E+11	291.63	1151.35	12.12	28.79
L/4	8300	8976.32	7.2E+05	2.3E+11	470.62	1973.74	17.04	40.47
3L/8	12325	9088.46	7.2E+05	2.3E+11	581.89	2467.17	19.71	46.80
L/2	16350	8951.83	7.2E+05	2.3E+11	618.98	2631.65	20.20	47.98

Total loss of prestressing force immediately - Remaining prestressing force:

Tendon1	X_i (mm)	Δf_{pF} (MPa)	Δf_{pA} (MPa)	Δf_{ES} (MPa)	$\Sigma \Delta$ (MPa)	F_j^1 (kN)	(α) (rad)	$F_j^1 \cos(\alpha)$ (kN)	$F_j^1 \sin(\alpha)$ (kN)
Section	(mm)	(MPa)	(MPa)	(MPa)	(MPa)	(kN)	(rad)	(kN)	(kN)
anchorage	0	0.00	208.40	17.23	225.63	1593.57	0.1744	1569.41	276.45
Support	250	1.09	203.81	17.26	222.16	1598.70	0.1717	1575.18	273.22
L/8	4275	19.40	129.86	28.79	178.05	1664.01	0.1294	1650.11	214.66
L/4	8300	51.08	55.92	40.47	147.46	1709.29	0.0865	1702.90	147.69
3L/8	12325	95.13	0.00	46.80	141.93	1717.48	0.0433	1715.87	74.41
L/2	16350	150.19	0.00	47.98	198.17	1634.23	0.0000	1634.23	0.00

Tendon2	X_i (mm)	Δf_{pF} (MPa)	Δf_{pA} (MPa)	Δf_{ES} (MPa)	$\Sigma \Delta$ (MPa)	F_j^2 (kN)	(α) (rad)	$F_j^2 \cos(\alpha)$ (kN)	$F_j^2 \sin(\alpha)$ (kN)
Section	(mm)	(MPa)	(MPa)	(MPa)	(MPa)	(kN)	(rad)	(kN)	(kN)
anchorage	0	0.00	187.93	16.15	204.08	1625.46	0.1374	1610.15	222.57
Support	250	0.90	184.20	16.18	201.28	1629.61	0.1353	1614.72	219.78
L/8	4275	16.03	124.06	26.99	167.08	1680.25	0.1017	1671.56	170.64
L/4	8300	41.74	63.93	37.94	143.61	1715.00	0.0680	1711.04	116.45
3L/8	12325	77.40	3.79	43.88	125.07	1742.44	0.0340	1741.43	59.26
L/2	16350	122.14	0.00	44.98	167.12	1680.20	0.0000	1680.20	0.00

Tendon3	X_i (mm)	Δf_{pF} (MPa)	Δf_{pA} (MPa)	Δf_{ES} (MPa)	$\Sigma \Delta$ (MPa)	F_j^3 (kN)	(α) (rad)	$F_j^3 \cos(\alpha)$ (kN)	$F_j^3 \sin(\alpha)$ (kN)
Section	(mm)	(MPa)	(MPa)	(MPa)	(MPa)	(kN)	(rad)	(kN)	(kN)
anchorage	0	0.00	164.35	14.36	178.70	1663.04	0.1000	1654.74	165.98
Support	250	0.71	161.49	14.38	176.59	1666.17	0.0985	1658.11	163.77
L/8	4275	12.65	115.50	23.99	152.14	1702.36	0.0739	1697.71	125.76
L/4	8300	32.33	69.52	33.72	135.57	1726.89	0.0493	1724.79	85.18
3L/8	12325	59.42	23.53	39.00	121.94	1747.07	0.0247	1746.54	43.13
L/2	16350	93.40	0.00	39.98	133.38	1730.14	0.0000	1730.14	0.00

Tendon4	X_i (mm)	Δf_{pF} (MPa)	Δf_{pA} (MPa)	Δf_{ES} (MPa)	$\Sigma \Delta$ (MPa)	F_j^4 (kN)	(α) (rad)	$F_j^4 \cos(\alpha)$ (kN)	$F_j^4 \sin(\alpha)$ (kN)
Section	(mm)	(MPa)	(MPa)	(MPa)	(MPa)	(kN)	(rad)	(kN)	(kN)
anchorage	0	0.00	127.93	10.77	138.70	1722.26	0.0623	1718.92	107.24
Support	250	0.53	125.98	10.79	137.29	1724.35	0.0614	1721.11	105.73
L/8	4275	9.26	94.48	17.99	121.74	1747.38	0.0460	1745.53	80.42
L/4	8300	22.86	62.99	25.29	111.14	1763.07	0.0307	1762.24	54.13
3L/8	12325	41.16	31.49	29.25	101.91	1776.74	0.0154	1776.53	27.28
L/2	16350	63.97	0.00	29.99	93.95	1788.51	0.0000	1788.51	0.00

Tendon5	Xi	Δf_{pF}	Δf_{pA}	Δf_{ES}	$\Sigma \Delta$	F_j^0	(α)	$F_j^0 \cos(\alpha)$	$F_j^0 \sin(\alpha)$
Section	(mm)	(MPa)	(MPa)	(MPa)	(MPa)	(kN)	(rad)	(kN)	(kN)
anchorage	0	0.00	67.63	0.00	67.63	1827.48	0.0245	1826.94	44.70
Support	250	0.34	66.60	0.00	66.93	1828.52	0.0241	1827.99	44.04
L/8	4275	5.86	49.95	0.00	55.81	1844.98	0.0181	1844.68	33.33
L/4	8300	13.31	33.30	0.00	46.61	1858.61	0.0120	1858.47	22.39
3L/8	12325	22.64	16.65	0.00	39.29	1869.44	0.0060	1869.41	11.26
L/2	16350	33.82	0.00	0.00	33.82	1877.55	0.0000	1877.55	0.00

SUM 1to5	Xi	ΣF_j	$F_j \cos(\alpha)$	$F_j \sin(\alpha)$	θ_{cAp}	$M_j = \Sigma F_j \cos(\alpha) \cdot \theta_{cAp}$
Section	(mm)	(kN)	(kN)	(kN)	(mm)	(kNm)
anchorage	0	8431.82	8380.15	816.92	39.52	331.15
Support	250	8447.36	8397.11	806.55	39.52	331.82
L/8	4275	8638.98	8609.58	624.82	291.63	2510.81
L/4	8300	8772.86	8759.44	425.83	470.62	4122.41
3L/8	12325	8853.17	8849.78	215.33	581.89	5149.62
L/2	16350	8710.62	8710.62	0.00	618.98	5391.71

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

4.2.1 Loss of prestress due to Shrinkage:

Formula:

$$\Delta f_{pSH} = (93 - 0.85 \cdot H)$$

Relative humidity of environment

$$H = 80.00 \%$$

$$\Delta f_{pSH} = 25.00 \text{ (MPa)}$$

4.2.2 Loss of prestress due to Creep:

Formula

$$\Delta f_{pCR} = 12.0 f_{cgp} - 7.0 \cdot \Delta f_{odp}$$

In which:

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

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

Section	Xi	Interior Girder			Exterior Girder	
		f_{cgp}	Δf_{odp}	Δf_{pCR}	Δf_{odp}	Δf_{pCR}
	(m)	(MPa)	(MPa)	(MPa)	(MPa)	(MPa)
Support	0.00	7.27	0.00	87.23	0.00	87.23
L/8	4.03	12.12	0.90	139.17	1.71	133.52
L/4	8.05	17.04	5.10	168.78	4.66	171.92
3L/8	12.08	19.71	5.47	198.20	6.99	187.55
L/2	16.10	20.20	8.67	181.80	7.88	187.32

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

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

f_{pj} : Initial stress in the tendon at the end of stressing

Section	Xi	f_{pj}	Δf_{pR1}
	(m)	(MPa)	(MPa)
Support	0.00	1284.74	0.00
L/8	4.03	1273.21	0.00
L/4	8.05	1261.53	0.00
3L/8	12.08	1255.20	0.00
L/2	16.10	1254.02	0.00

(b) After Transfer:

Formula:

$$\Delta f_{pR2} = 30\% \cdot (138 - 0.3 \Delta f_{pF} - 0.4 \Delta f_{ES} - 0.2 (\Delta f_{pSH} + \Delta f_{pCR}))$$

Interior Girder						
Section	Xi	Δf_{pF}	Δf_{pES}	Δf_{pSH}	Δf_{pCR}	Δf_{pR2}
	(m)	(MPa)	(MPa)	(MPa)	(MPa)	(MPa)
Support	0.00	0.71	17.26	25.00	17.45	36.72
L/8	4.03	12.64	28.79	25.00	27.83	33.64
L/4	8.05	32.26	40.47	25.00	33.76	30.11
3L/8	12.08	59.15	46.80	25.00	39.64	26.58
L/2	16.10	92.70	47.98	25.00	36.36	23.62

Exterior Girder						
Section	X_i	Δf_{pF}	Δf_{pES}	Δf_{pSH}	Δf_{pCR}	Δf_{pR2}
	(m)	(MPa)	(MPa)	(MPa)	(MPa)	(MPa)
Support	0.00	0.71	17.26	25.00	17.45	36.72
L/8	4.03	12.64	28.79	25.00	26.70	33.71
L/4	8.05	32.26	40.47	25.00	34.38	30.08
3L/8	12.08	59.15	46.80	25.00	37.51	26.71
L/2	16.10	92.70	47.98	25.00	37.46	23.55

TOTAL LOSS STRESS AT SERVICE STAGE

Interior Girder						
Section	X_i	Δf_{pSH}	Δf_{pCR}	Δf_{pR1}	Δf_{pR2}	Sum
	(m)	(MPa)	(MPa)	(MPa)	(MPa)	(MPa)
Support	0.00	25.00	87.23	0.00	36.72	148.94
L/8	4.03	25.00	139.17	0.00	33.64	197.81
L/4	8.05	25.00	168.78	0.00	30.11	223.90
3L/8	12.08	25.00	198.20	0.00	26.58	249.78
L/2	16.10	25.00	181.80	0.00	23.62	230.42

Exterior Girder						
Section	X_i	Δf_{pSH}	Δf_{pCR}	Δf_{pR1}	Δf_{pR2}	Sum
	(m)	(MPa)	(MPa)	(MPa)	(MPa)	(MPa)
Support	0.00	25.00	87.23	0.00	36.72	148.94
L/8	4.03	25.00	133.52	0.00	33.71	192.22
L/4	8.05	25.00	171.92	0.00	30.08	227.00
3L/8	12.08	25.00	187.55	0.00	26.71	239.26
L/2	16.10	25.00	187.32	0.00	23.55	235.87

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

Tendon1	X_i	$\Sigma \Delta_{pT}$	F_j^1	(α)	$F_j^1 \cos(\alpha)$	$F_j^1 \sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	371.10	1378.19	0.1717	1357.92	235.54
L/8	4.03	375.86	1371.16	0.1294	1359.70	176.88
L/4	8.05	371.36	1377.81	0.0865	1372.66	119.05
3L/8	12.08	391.71	1347.68	0.0433	1346.41	58.39
L/2	16.10	428.58	1293.09	0.0000	1293.09	0.00

Tendon2	X_i	$\Sigma \Delta_{pT}$	F_j^2	(α)	$F_j^2 \cos(\alpha)$	$F_j^2 \sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	350.23	1409.10	0.1374	1395.83	192.94
L/8	4.03	364.89	1387.39	0.1353	1374.72	187.12
L/4	8.05	367.51	1383.52	0.1017	1376.36	140.51
3L/8	12.08	374.85	1372.64	0.0680	1369.47	93.20
L/2	16.10	397.53	1339.06	0.0340	1338.29	45.54

Tendon3	X_i	$\Sigma \Delta_{pT}$	F_j^3	(α)	$F_j^3 \cos(\alpha)$	$F_j^3 \sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	327.65	1442.53	0.0985	1435.55	141.79
L/8	4.03	374.39	1373.32	0.0739	1369.57	101.46
L/4	8.05	376.04	1370.88	0.0493	1369.21	67.62
3L/8	12.08	385.36	1357.09	0.0247	1356.68	33.50
L/2	16.10	352.36	1405.94	0.0000	1405.94	0.00

Tendon4	X_i	$\Sigma \Delta_{pT}$	F_j^4	(α)	$F_j^4 \cos(\alpha)$	$F_j^4 \sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	287.65	1501.75	0.0614	1498.93	92.08
L/8	4.03	335.10	1431.50	0.0460	1429.98	65.88
L/4	8.05	345.64	1415.90	0.0307	1415.23	43.47
3L/8	12.08	360.92	1393.27	0.0154	1393.10	21.40
L/2	16.10	332.33	1435.60	0.0000	1435.60	0.00

Tendon5	X_i	$\Sigma \Delta_{PT}$	F_j^5	(α)	$F_j^5 \cos(\alpha)$	$F_j^5 \sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	216.57	1606.97	0.0241	1606.51	38.70
L/8	4.03	264.74	1535.66	0.0181	1535.41	27.74
L/4	8.05	279.71	1513.50	0.0120	1513.39	18.23
3L/8	12.08	296.39	1488.80	0.0060	1488.78	8.97
L/2	16.10	269.71	1528.31	0.0000	1528.31	0.00

SUM 1to5	X_i	ΣF_j	$F_j \cos(\alpha)$	$V_p = F_j \sin(\alpha)$	e_{cable}	$M_j = \Sigma F_j \cos(\alpha) * e_{cap}$
Section	(mm)	(kN)	(kN)	(kN)	(m)	(kNm)
Support	0.00	7338.55	7294.73	701.05	0.04	288.3
L/8	4.03	7099.03	7069.38	559.08	0.29	2061.6
L/4	8.05	7061.61	7046.86	388.87	0.47	3316.4
3L/8	12.08	6959.48	6954.44	215.45	0.58	4046.7
L/2	16.10	7002.00	7001.23	45.54	0.62	4333.6

Exterior Girder

Tendon1	X_i	$\Sigma \Delta_{PT}$	F_j^1	(α)	$F_j^1 \cos(\alpha)$	$F_j^1 \sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	371.10	1378.19	0.1717	1357.92	235.54
L/8	4.03	370.27	1379.43	0.1294	1367.90	177.95
L/4	8.05	374.46	1373.22	0.0865	1368.09	118.65
3L/8	12.08	381.19	1363.25	0.0433	1361.97	59.06
L/2	16.10	434.04	1285.02	0.0000	1285.02	0.00

Tendon2	X_i	$\Sigma \Delta_{PT}$	F_j^2	(α)	$F_j^2 \cos(\alpha)$	$F_j^2 \sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	350.23	1409.10	0.1374	1395.83	192.94
L/8	4.03	359.30	1395.66	0.1353	1382.91	188.23
L/4	8.05	370.61	1378.92	0.1017	1371.79	140.04
3L/8	12.08	364.33	1388.21	0.0680	1385.01	94.26
L/2	16.10	402.98	1330.99	0.0340	1330.22	45.27

Tendon3	X_i	$\Sigma \Delta_{PT}$	F_j^3	(α)	$F_j^3 \cos(\alpha)$	$F_j^3 \sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	327.65	1442.53	0.0985	1435.55	141.79
L/8	4.03	368.81	1381.59	0.0739	1377.81	102.07
L/4	8.05	379.14	1366.29	0.0493	1364.63	67.39
3L/8	12.08	374.84	1372.67	0.0247	1372.25	33.88
L/2	16.10	357.81	1397.87	0.0000	1397.87	0.00

Tendon4	X_i	$\Sigma \Delta_{PT}$	F_j^4	(α)	$F_j^4 \cos(\alpha)$	$F_j^4 \sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	287.65	1501.75	0.0614	1498.93	92.08
L/8	4.03	329.51	1439.77	0.0460	1438.24	66.26
L/4	8.05	348.74	1411.30	0.0307	1410.64	43.33
3L/8	12.08	350.40	1408.84	0.0154	1408.68	21.63
L/2	16.10	337.78	1427.53	0.0000	1427.53	0.00

Tendon5	X_i	$\Sigma \Delta_{PT}$	F_j^5	(α)	$F_j^5 \cos(\alpha)$	$F_j^5 \sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	216.57	1606.97	0.0241	1606.51	38.70
L/8	4.03	259.16	1543.93	0.0181	1543.68	27.89
L/4	8.05	282.81	1508.91	0.0120	1508.80	18.17
3L/8	12.08	285.87	1504.38	0.0060	1504.35	9.06
L/2	16.10	275.16	1520.24	0.0000	1520.24	0.00

SUM 1to5	X_i	ΣF_j	$F_j \cos(\alpha)$	$V_p = F_j \sin(\alpha)$	e_{cable}	$M_j = \Sigma F_j \cos(\alpha) * e_{cap}$
Section	(mm)	(kN)	(kN)	(kN)	(m)	(kNm)
Support	0.00	7338.55	7294.73	701.05	0.04	288.3
L/8	4.03	7140.37	7110.54	562.41	0.29	2073.6
L/4	8.05	7038.65	7023.95	387.59	0.47	3305.6
3L/8	12.08	7037.36	7032.26	217.90	0.58	4092.0
L/2	16.10	6961.66	6960.89	45.27	0.62	4308.7

5. FIBRE STRESS CHECK:

Formula:

$$\text{Top fibre: } f_t = \frac{F_t}{A} - \frac{F_t e}{S_t} + \frac{M_{DC}}{S_t} \quad \text{Bottom fibre } f_b = \frac{F_t}{A} + \frac{F_t e}{S_b} - \frac{M_{DC}}{S_b}$$

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

Concrete strength at transfer $f_{ci}' = 0.9 f_c = 40.50 \text{ MPa}$

Compression stress Limit at transfer $0.6 f_{ci}' = 24.30 \text{ MPa}$

Tension stress Limit at transfer $0.25 \text{ SQRT}(f_{ci}') < 1.38 = -1.38 \text{ MPa}$

(5.9.4.1.2-1)

Setion	Xi	A	St	Sb	F _t *Cos(α)	e	M _{DC}	f _{ti}	f _{bi}	Kiểm tra	
	(mm)	(mm ²)	(mm ³)	(mm ³)	(kN)	(mm)	(kNm)	(MPa)	(MPa)	f _{ti}	f _{bi}
irder en	0	1.18E+06	3.39E+08	3.26E+08	8380.15	39.52	0.00	6.11	8.11	OK	OK
Support	250	1.18E+06	3.39E+08	3.26E+08	8397.11	39.52	0.00	6.12	8.12	OK	OK
L/8	4275	8.42E+05	2.98E+08	2.92E+08	8609.58	291.63	1151.35	5.67	14.88	OK	OK
L/4	8300	7.19E+05	2.82E+08	2.81E+08	8759.44	470.62	1973.74	4.56	19.83	OK	OK
3L/8	12325	7.19E+05	2.82E+08	2.81E+08	8849.78	581.89	2467.17	2.79	21.86	OK	OK
L/2	16350	7.19E+05	2.82E+08	2.81E+08	8710.62	618.98	2631.65	2.32	21.95	OK	OK

5.2 Stress check during contruction the deck:

5.2.1 Increase load:

Exterior Diaphragms beam	DC _{dn1} =	49.73 (kN)
Interior Diaphragms beam	DC _{dn1} =	35.16 (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 $f_c = 45.00 \text{ MPa}$

Compression stress limit $0.45 f_c = 20.25 \text{ MPa}$ (5.9.4.2.1-1)

Tension stress limit $0.5 \text{ SQRT}(f_c) = -3.35 \text{ MPa}$ (5.9.4.2.2-1)

Setion	Xi	A	St	Sb	Fi	e	M _{DC}	f _{ti}	f _{bi}	Kiểm tra	
	(mm)	(mm ²)	(mm ³)	(mm ³)	(kN)	(mm)	(kNm)	(MPa)	(MPa)	f _{ti}	f _{bi}
irder en	0	1.18E+06	3.39E+08	3.26E+08	8380.15	39.52	0.00	6.11	8.11	OK	OK
Support	250	1.18E+06	3.39E+08	3.26E+08	8397.11	39.52	0.00	6.12	8.12	OK	OK
L/8	4275	8.42E+05	2.98E+08	2.92E+08	8609.58	291.63	2924.99	11.62	8.81	OK	OK
L/4	8300	7.19E+05	2.82E+08	2.81E+08	8759.44	470.62	3719.72	10.75	13.61	OK	OK
3L/8	12325	7.19E+05	2.82E+08	2.81E+08	8849.78	581.89	4649.65	10.53	14.08	OK	OK
L/2	16350	7.19E+05	2.82E+08	2.81E+08	8710.62	618.98	4959.62	10.58	13.65	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: $0.45 f_c = 20.25 \text{ MPa}$ (5.9.4.2.1-1)

Tension Stress Limit: $-0.5 \text{ SQRT}(f_c) = -3.35 \text{ MPa}$

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

Interior Girder

Setion	Xi	A	S _t	S _{Ig}	P _{pe}	P _{pe} *e _c	M _g + M _s	M _{SDL}	f _t	Check
	(mm)	(mm ²)	(mm ³)	(mm ³)	(kN)	(kNm)	(kNm)	(kNm)	(MPa)	f _t
Support	250	1.18E+06	3.39E+08	1.03E+09	7294.73	288.26	0.00	0.00	5.321	OK
L/8	4275	8.42E+05	2.98E+08	1.05E+09	7069.38	2061.64	2169.83	520.61	9.256	OK
L/4	8300	7.19E+05	2.82E+08	1.07E+09	7046.86	3316.42	3719.72	892.48	12.067	OK
3L/8	12325	7.19E+05	2.82E+08	1.07E+09	6954.44	4046.73	4649.65	1115.60	12.845	OK
L/2	16350	7.19E+05	2.82E+08	1.07E+09	7001.23	4333.63	4959.62	1189.97	13.061	OK

Exterior Girder

Setion	Xi	A	S _t	S _{Ig}	P _{pe}	P _{pe} *e _c	M _g + M _s	M _{SDL}	f _t	Check
	(mm)	(mm ²)	(mm ³)	(mm ³)	(kN)	(kNm)	(kNm)	(kNm)	(MPa)	f _t
Support	250	1.18E+06	3.39E+08	1.03E+09	7294.73	288.26	0.00	0.00	5.321	OK
L/8	4275	8.42E+05	2.98E+08	1.05E+09	7110.54	2073.64	2038.17	523.73	8.827	OK
L/4	8300	7.19E+05	2.82E+08	1.07E+09	7023.95	3305.64	3494.01	897.82	11.270	OK
3L/8	12325	7.19E+05	2.82E+08	1.07E+09	7032.26	4092.02	4367.51	1122.28	11.800	OK
L/2	16350	7.19E+05	2.82E+08	1.07E+09	6960.89	4308.66	4658.67	1197.10	12.034	OK

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

Compression Stress Limit: $0.40 f_c = 18.00 \text{ MPa}$ (5.9.4.2.1-1)

Tension Stress Limit: $-0.5 \cdot \text{SQRT}(f_c) = -3.35 \text{ MPa}$

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

Interior Girder

Setion	Xi (mm)	A (mm ²)	S _t (mm ³)	S _{lg} (mm ³)	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	1.18E+06	3.39E+08	1.03E+09	7294.73	288.26	0.00	0.00	0.00	2.660	OK
L/8	4275	8.42E+05	2.98E+08	1.05E+09	7069.38	2061.64	2169.83	520.61	1319.41	5.880	OK
L/4	8300	7.19E+05	2.82E+08	1.07E+09	7046.86	3316.42	3719.72	892.48	2231.82	8.107	OK
3L/8	12325	7.19E+05	2.82E+08	1.07E+09	6954.44	4046.73	4649.65	1115.60	2754.34	8.987	OK
L/2	16350	7.19E+05	2.82E+08	1.07E+09	7001.23	4333.63	4959.62	1189.97	2904.08	9.234	OK

Exterior Girder

Setion	Xi (mm)	A (mm ²)	S _t (mm ³)	S _{lg} (mm ³)	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	1181984.6	3.39E+08	1.03E+09	7294.73	288.26	0.00	0.00	0.00	2.660	OK
L/8	4275	841615.0	2.98E+08	1.05E+09	7110.54	2073.64	2038.17	523.73	1553.33	5.889	OK
L/4	8300	719350.0	2.82E+08	1.07E+09	7023.95	3305.64	3494.01	897.82	2627.50	8.085	OK
3L/8	12325	719350.0	2.82E+08	1.07E+09	7032.26	4092.02	4367.51	1122.28	3242.66	8.924	OK
L/2	16350	719350.0	2.82E+08	1.07E+09	6960.89	4308.66	4658.67	1197.10	3418.94	9.206	OK

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

Compression Stress Limit: $0.60 f_c = 27.00 \text{ MPa}$ (5.9.4.2.1-1)

Tension Stress Limit: $-0.5 \cdot \text{SQRT}(f_c) = -3.35 \text{ MPa}$

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

Interior Girder

Setion	Xi (mm)	A (mm ²)	S _t (mm ³)	S _{lg} (mm ³)	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	1.18E+06	3.39E+08	1.03E+09	7294.73	288.26	0.00	0.00	0.00	5.321	OK
L/8	4275	8.42E+05	2.98E+08	1.05E+09	7069.38	2061.64	2169.83	520.61	1319.41	10.508	OK
L/4	8300	7.19E+05	2.82E+08	1.07E+09	7046.86	3316.42	3719.72	892.48	2231.82	14.136	OK
3L/8	12325	7.19E+05	2.82E+08	1.07E+09	6954.44	4046.73	4649.65	1115.60	2754.34	15.409	OK
L/2	16350	7.19E+05	2.82E+08	1.07E+09	7001.23	4333.63	4959.62	1189.97	2904.08	15.765	OK

Exterior Girder

Setion	Xi (mm)	A (mm ²)	S _t (mm ³)	S _{lg} (mm ³)	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	1.18E+06	3.39E+08	1.03E+09	7294.73	288.26	0.00	0.00	0.00	5.321	OK
L/8	4275	8.42E+05	2.98E+08	1.05E+09	7110.54	2073.64	2038.17	523.73	1553.33	10.303	OK
L/4	8300	7.19E+05	2.82E+08	1.07E+09	7023.95	3305.64	3494.01	897.82	2627.50	13.720	OK
3L/8	12325	7.19E+05	2.82E+08	1.07E+09	7032.26	4092.02	4367.51	1122.28	3242.66	14.824	OK
L/2	16350	7.19E+05	2.82E+08	1.07E+09	6960.89	4308.66	4658.67	1197.10	3418.94	15.223	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: $0.45 f_c = 15.75 \text{ MPa}$ (5.9.4.2.1-1)

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

Setion	Xi (mm)	MSDL (kNm)		S _{ic} (mm ³)		f _t (MPa)		Check	
		in. Girder	Ex. Girder	in. Girder	Ex. Girder	in. Girder	Ex. Girder	in. Girder	Ex. Girder
Support	250.00	0.00	0.00	6.4E+08	6.43E+08	0.000	0.000	OK	OK
L/8	4275.00	520.61	523.73	6.3E+08	6.32E+08	0.823	0.829	OK	OK
L/4	8300.00	892.48	897.82	6.3E+08	6.3E+08	1.415	1.425	OK	OK
3L/8	12325.00	1115.60	1122.28	6.3E+08	6.3E+08	1.769	1.782	OK	OK
L/2	16350.00	1189.97	1197.10	6.3E+08	6.3E+08	1.887	1.900	OK	OK

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

Compression Stress Limit: $0.6 f_c = 21.00 \text{ MPa}$ (5.9.4.2.1-1)

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

Setion	Xi	MSDL + MLL (kNm)		S _{ic} (mm ³)		f _i (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	6.4E+08	6.43E+08	0.000	0.000	OK	OK
L/8	4275.00	1840.02	2077.06	6.3E+08	6.32E+08	2.909	3.288	OK	OK
L/4	8300.00	3124.30	3525.32	6.3E+08	6.3E+08	4.953	5.598	OK	OK
3L/8	12325.00	3869.94	4364.93	6.3E+08	6.3E+08	6.135	6.929	OK	OK
L/2	16350.00	4094.05	4616.03	6.3E+08	6.3E+08	6.491	7.328	OK	OK

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

Compression Stress Limit: $0.45 f_c = 20.25 \text{ MPa}$ (5.9.4.2.1-1)

Tension Stress Limit: $-0.5 \cdot \text{SQRT}(f_c) = -3.35 \text{ MPa}$ (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)	(mm ²)	(mm ³)	(mm ³)	(kN)	(kNm)	(kNm)	(kNm)	(kNm)	(MPa)	f _b
Support	250	1.18E+06	3.26E+08	5.04E+08	7294.73	288.26	0.00	0.00	0.00	7.056	OK
L/8	4275	8.42E+05	2.92E+08	4.30E+08	7069.38	2061.64	2169.83	520.61	1319.41	4.362	OK
L/4	8300	7.19E+05	2.81E+08	4.00E+08	7046.86	3316.42	3719.72	892.48	2231.82	1.663	OK
3L/8	12325	7.19E+05	2.81E+08	4.00E+08	6954.44	4046.73	4649.65	1115.60	2754.34	-0.780	OK
L/2	16350	7.19E+05	2.81E+08	4.00E+08	7001.23	4333.63	4959.62	1189.97	2904.08	-1.283	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)	(mm ²)	(mm ³)	(mm ³)	(kN)	(kNm)	(kNm)	(kNm)	(kNm)	(MPa)	f _b
Support	250	1.18E+06	3.26E+08	5.04E+08	7294.73	288.26	0.00	0.00	0.00	7.056	OK
L/8	4275	8.42E+05	2.92E+08	4.30E+08	7110.54	2073.64	2038.17	523.73	1553.33	4.459	OK
L/4	8300	7.19E+05	2.81E+08	4.00E+08	7023.95	3305.64	3494.01	897.82	2627.50	1.590	OK
3L/8	12325	7.19E+05	2.81E+08	4.00E+08	7032.26	4092.02	4367.51	1122.28	3242.66	-0.501	OK
L/2	16350	7.19E+05	2.81E+08	4.00E+08	6960.89	4308.66	4658.67	1197.10	3418.94	-1.405	OK

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

Compression Stress Limit: $0.45 f_c = 20.25 \text{ MPa}$ (5.9.4.2.1-1)

Tension Stress Limit: $-0.5 \cdot \text{SQRT}(f_c) = -3.35 \text{ MPa}$ (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

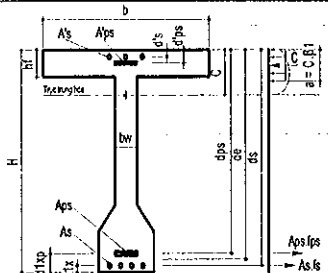
Setion	Xi	A	S _b	S _{bc}	P _{pe}	P _{pe} *e _c	M _g + M _s	M _{SDL}	M _{LL}	f _b	Check
	(mm)	(mm ²)	(mm ³)	(mm ³)	(kN)	(kNm)	(kNm)	(kNm)	(kNm)	(MPa)	f _b
Support	250	1.18E+06	3.26E+08	5.04E+08	7294.73	288.26	0.00	0.00	0.00	7.06	OK
L/8	4275	8.42E+05	2.92E+08	4.30E+08	7069.38	2061.64	2169.83	520.61	1319.41	3.75	OK
L/4	8300	7.19E+05	2.81E+08	4.00E+08	7046.86	3316.42	3719.72	892.48	2231.82	0.55	OK
3L/8	12325	7.19E+05	2.81E+08	4.00E+08	6954.44	4046.73	4649.65	1115.60	2754.34	-2.16	OK
L/2	16350	7.19E+05	2.81E+08	4.00E+08	7001.23	4333.63	4959.62	1189.97	2904.08	-2.73	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)	(mm ²)	(mm ³)	(mm ³)	(kN)	(kNm)	(kNm)	(kNm)	(kNm)	(MPa)	f _b
Support	250	1.18E+06	3.26E+08	5.04E+08	7294.73	288.26	0.00	0.00	0.00	7.056	OK
L/8	4275	8.42E+05	2.92E+08	4.30E+08	7110.54	2073.64	2038.17	523.73	1553.33	3.736	OK
L/4	8300	7.19E+05	2.81E+08	4.00E+08	7023.95	3305.64	3494.01	897.82	2627.50	0.276	OK
3L/8	12325	7.19E+05	2.81E+08	4.00E+08	7032.26	4092.02	4367.51	1122.28	3242.66	-2.123	OK
L/2	16350	7.19E+05	2.81E+08	4.00E+08	6960.89	4308.66	4658.67	1197.10	3418.94	-3.116	OK

REINFORCEMENT OF GIRDER CHECKING - STRENGTH LOAD COMBINATION

MATERIALS			
NORMAL CONCRETE			
f _c	Compressive Strength of concrete at 28 days	Mpa	45
E _c	Modulus of Elasticity	Mpa	33915
f _r	Modulus of Rupture	Mpa	4.2
γ _c	Unit weight of concrete	kN/m3	24.0
PRESTRESSING STEEL			
f _{pu}	Tensile strength of prestressing steel	Mpa	1860
f _{py}	Yield strength of prestressing steel	Mpa	1674
E _p	Modulus of Elasticity	Mpa	197000
REINFORCEMENT			
f _y	Yield strength	Mpa	400
E _s	Modulus of Elasticity	Mpa	200000
n _c	Ratio E _s /E _c		6

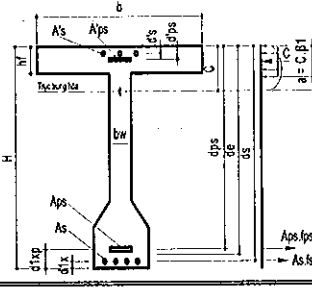


Sign	Parameters	Unit	Support	L/8	L/4	3L/8	L/2	
INTERNAL FORCES AT SECTION								
Q _u	Combination Shear	kN	1753	1381	1016	658	307	
M _u	Flexural Moment	kNm	0	5984	10196	12672	13447	
N _u	Axial load	kN						
T _u	Torsional Moment	kNm						
6.1 FLEXURAL MOMENT CHECKING								
H	Section height	m	1.850	1.850	1.850	1.850	1.850	
d _s	Dis. From comp. fiber to centroid of comp. Reinf	m	0.062	0.062	0.062	0.062	0.062	
d _{1x}	Dis. From tens. fiber to centroid of tension Reinf	m	0.062	0.062	0.062	0.062	0.062	
cover	Cover to reinf	m	0.040	0.040	0.040	0.040	0.040	
d _s	Dis. From comp. fiber to centroid of tension Reinf	m	1.788	1.788	1.788	1.788	1.788	
d _{ps}	Dis. From comp. fiber to centroid of comp. prestressing steel	m	0.000	0.000	0.000	0.000	0.000	
d _{1xp}	Dis. From tens. fiber to centroid of tension prestressing steel	m	0.801	0.542	0.356	0.245	0.208	
d _{ps}	Dis. From comp. fiber to centroid of tension prestressing steel	m	1.049	1.308	1.494	1.605	1.642	
b	Width of the compression face of member	m	2.249	2.249	2.249	2.249	2.249	
b _w	Web width or diameter of a circular section	m	0.700	0.700	0.250	0.250	0.250	
h _f	Compression flange depth	m	0.200	0.200	0.200	0.200	0.200	
I _z	Moment of inertia of section	m ⁴	0.554	0.499	0.476	0.476	0.476	
A _{mc}	Section area	m ²	1.648	1.307	1.185	1.185	1.185	
Steel choice								
A _{ps}	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	m ²	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	m ²	0.00000	0.00000	0.00000	0.00000	0.00000
A _s	Tension Reinforcement	Number	bars	6	6	6	6	6
		Diameter	mm	16	16	16	16	16
		Area	m ²	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	m ²	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	m ²	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	
f _v	Resistance factors for shear		0.90	0.90	0.90	0.90	0.90	
f _n	Resistance factors for axial force		1.00	1.00	1.00	1.00	1.00	
b ₁	Stress block factor		0.729	0.729	0.729	0.729	0.729	
c	Dis. Between centroid and top fiber		m	0.235	0.242	0.307	0.313	0.315
		For T section behavior	m	0.235	0.242	0.307	0.313	0.315
		For rectangular section behavior	m	0.212	0.214	0.216	0.216	0.216
f _{pe}	Effective stress in the prestressing steel after losses	Mpa	1378	1371	1371	1348	1293	
f _{ps}	Aver. stress in pres. steel at the time for which the nominal resistance	Mpa	1743	1764	1753	1758	1760	
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	0.176	0.224	0.228	0.229	
d _e	Corresp. effective depth from extreme comp. fiber to centroid of tensile force in the tensile reinf.		m	1.075	1.325	1.504	1.611	1.647
M _n	Nominal resistance	kNm	13134	16653	18884	20384	20885	
M _r	Factored resistance	kNm	11821	14988	16996	18346	18796	
M _u	Flexural moment	kNm	0	5984	10196	12672	13447	
(5.7.3.2)	Flexural moment Checking		OK	OK	OK	OK	OK	
Limits for reinforcement								
c/d _e	Maximum reinforcement		0.22	0.18	0.20	0.19	0.19	
		<= 0.42	OK	OK	OK	OK	OK	
r _{min}	Minimum reinforcement		0.07%	0.09%	0.10%	0.10%	0.10%	
		0.34%	N.a	N.a	N.a	N.a	N.a	
1.2*M _{cr}	Cracking moment	kNm	1739	1574	1566	1572	1573	
(5.7.3.3.2)	Checking M _r >= min(1.2M _{cr} , 1.33M _u)		OK	OK	OK	OK	OK	
(5.8.3.5)	Tensile force in steel should be satisfied - F _{yc}		kN	1912	6497	9244	10120	10083
				OK	OK	OK	OK	OK

(5.7.3.4)	Control of cracking by distr. of reinf for RC member- Check?		No	No	No	No	No
dc	Existing condition for structure	1,2 or 3	1	1	1	1	1
Z	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.058	0.058	0.058	0.058	0.058
A	Crack width parameter	N/mm	30000	30000	30000	30000	30000
fsa	Area of concr. with same centroid as tens. Reinf	m2	0.043	0.043	0.043	0.043	0.043
0.6*fy	Value	Mpa	220	220	220	220	220
	Tensile stress in reinf Min(fs,0.6fy)	Mpa	240	240	240	240	240
x	Dist. From compression fiber to centroid	m	220	220	220	220	220
J.d	Arm	m	-	-	-	-	-
Icr	Moment of inertia of the cracked section	m4	-	-	-	-	-
fs	Tensile stress in reinforcement fs = MsIs / (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 Reinforcement (side distribution)						
Areq	Area of required reinf	m2	0.00038	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	6.7	6.8	5.9	5.5
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	0.700	0.700	0.250	0.250	0.250
dv	Effective shear depth	m	1.332	1.332	1.392	1.498	1.533
	(de - a/2)	m	0.990	1.237	1.392	1.498	1.533
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	2089	1846	3243	1953	890
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1414	1416	1421	1397	1340
εx	Strain in tensile reinforcement		-5.14E-03	-2.73E-03	-1.29E-03	-7.24E-04	-4.97E-04
	if εx<0, multiple with reduce factor		-3.70E-04	-1.96E-04	-2.30E-04	-1.29E-04	-8.85E-05
	Strain checking	<=2.00E-3	OK	OK	OK	OK	OK
v/fc	Ratio of shear stress and fc		0.046	0.037	0.072	0.043	0.020
β	Final value		6.8	6.7	6.8	5.9	5.5
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	3520	3494	1314	1239	1183
Vs	Shear resistance provided by shear reinforcement	kN	2816	2816	1472	1583	1620
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	6336	6310	2786	2822	2803
Vn2	Vn2	kN	10490	10490	3916	4212	4310
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	6336	6310	2786	2822	2803
Vr	Factored shear resistance	kN	5703	5679	2507	2540	2523
Vu	Shear	kN	1753	1381	1016	658	307
(5.8.2.7)	Shear checking		OK	OK	OK	OK	OK
	Region requiring transverse reinf Checking		Need	No need	Need	Need	No need
	Minimum shear reinf area	m2	0.0001	0.0001	0.0001	0.0001	0.0001
	Minimum shear reinforcement Checking		OK	-	OK	OK	-
	0.1*fc*bv*dv	kN	4196	4196	1566	1685	1724
	Smax	m	0.60	0.60	0.60	0.60	0.60
	Maximum spacing Smax		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		
fpv	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							
Qu	Shear	kN	1210	946	685	428	175
Mu	Flexural Moment	kNm	0	4115	7019	8732	9275
Nu	Axial load	kN					
Tu	Torsional Moment	kNm					
6.1 FLEXURAL MOMENT CHECKING							
H	Section height	m	1.873	1.873	1.873	1.873	1.873
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
ds	Dis. From comp. fiber to centroid of tension Reinf	m	0.040	0.040	0.040	0.040	0.040
dps	Dis. From comp. fiber to centroid of tension prestressing steel	m	1.811	1.811	1.811	1.811	1.811
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	0.700	0.700	0.700
hf	Compression flange depth	m	0.223	0.223	0.223	0.223	0.223
Iz	Moment of inertia of section	m4	0.554	0.499	0.476	0.476	0.476
Amc	Section area	m2	1.648	1.307	1.185	1.185	1.185
Aps	Tension prestressing steel	tendons	15 T12.7	15 T12.7	15 T12.7	15 T12.7	15 T12.7
A's	Compression prestressing steel	tendons	0	0	0	0	0
As	Tension Reinforcement	bars	6	6	6	6	6
A's	Compression Reinforcement	bars	4	4	4	4	4
A'c	Shear reinforcement	bars	2	2	2	2	2
f	Resistance factors for flexure		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	0.188	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	1768	1786	1794	1799	1800
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	1.097	1.348	1.527	1.634	1.670
Mn	Nominal resistance	kNm	13479	17037	19465	20993	21503
Mr	Factored resistance	kNm	13479	17037	19465	20993	21503
Mu	Flexural moment	kNm	0	4115	7019	8732	9275
(5.7.3.2)	Flexural moment Checking		OK	OK	OK	OK	OK
c/de	Limits for reinforcement		0.17	0.14	0.13	0.12	0.11
r min	Maximum reinforcement Checking	<= 0.42	OK	OK	OK	OK	OK
1.2*Mcrc	Minimum reinforcement	0.07%	0.07%	0.09%	0.10%	0.10%	0.10%
(5.7.3.3.2)	Minimum reinforcement Checking for RC	0.34%	N.a	N.a	N.a	N.a	N.a
(5.8.3.5)	Cracking moment	kNm	1668	1504	1437	1437	1437
	Checking Mr >= min(1.2Mcrc, 1.33Mu)		OK	OK	OK	OK	OK
	Tensile force in steel should be satisfied - Fyc	kN	1188	3980	5490	6003	5970
	Checking As.fy + Aps.fps >= Fyc		OK	OK	OK	OK	OK

(5.7.3.4)	Control of cracking by distr. of reinf for RC member- Check?		Yes	Yes	Yes	Yes	Yes
	Existing condition for structure	1,2 or 3	1	1	1	1	1
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.058	0.058	0.058	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
f _{sa}	Value	Mpa	211	211	211	211	211
0.6*f _y	Tensile stress in reinf Min(f _{sa} ,0.6f _y)	Mpa	240	240	240	240	240
x	Dist. From compression fiber to centroid	m	0.076	0.085	0.091	0.094	0.095
J.d	Arm	m	1.072	1.319	1.486	1.603	1.638
I _{cr}	Moment of inertia of the cracked section	m4	0.022	0.022	0.022	0.022	0.022
f _s	Tensile stress in reinforcement f _s = M _s / (A _s *J.d)	Mpa	-	2573	3870	4496	4672
	Checking for control cracking f _s <f _{sa}		N.a	N.a	N.a	N.a	N.a
(5.10.8.2)	Shrinkage and temperature Reinforcement (side distribution)						
A _{req}	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
b _v	Effective web width as minimum web width - in dv	m	0.700	0.700	0.250	0.250	0.250
d _v	Effective shear depth	m	1.348	1.348	1.457	1.564	1.600
	(d _e - a/2)	m	1.029	1.278	1.457	1.564	1.600
s	Spacing of stirrups	m	0.150	0.150	0.150	0.150	0.150
n _{cat}	Amount of bars in spacing S	bars	2	2	2	2	2
A _v	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	1282	1002	1880	1095	438
f _{po}	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1145	1153	1156	1156	1156
e _s	Strain in tensile reinforcement		-4.63E-03	-2.94E-03	-2.00E-03	-1.63E-03	-1.57E-03
	if e _s <0, multiple with reduce factor		-3.29E-04	-2.09E-04	-3.53E-04	-2.87E-04	-2.78E-04
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok	Ok
v/f _c	Ratio of shear stress and f _c		0.028	0.022	0.042	0.024	0.010
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
V _c	Nominal shear resistance provided by tensile stresses in the concrete	kN	3563	3563	1375	1476	1510
V _s	Shear resistance provided by shear reinforcement	kN	2851	2851	3080	3307	3383
V _p	Component in the direction of the applied shear of the effective P.S	kN	0	0	0	0	0
V _{n1}	V _{n1} =V _c +V _s +V _p	kN	6413	6413	4455	4783	4892
V _{n2}	V _{n2}	kN	10617	10617	4097	4399	4499
V _n	Nominal shear resistance V _n =min(V _{n1} ,V _{n2})	kN	6413	6413	4097	4399	4499
V _r	Factored shear resistance	kN	6413	6413	4097	4399	4499
V _u	Shear	kN	1210	946	685	428	175
(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
	Minimum shear reinforcement Checking		-	-	-	-	-
	0.1*f _c *b _v *d _v	kN	4247	4247	1639	1760	1800
	S _{max}	m	0.60	0.60	0.60	0.60	0.60
	Maximum spacing S _{max}		-	-	-	-	-

CALCULATION SHEET
I40 GIRDER

CONTENT

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- 3.2. Property of girder cross section at service stage (composite cross section)
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- 5.1 Stress check during contruction the Girder:
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- 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:
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- 5.5 Stress check at the bottom fibre of girder - Service III (stage III):
- 5.6 Stress check at the bottom fibre of girder - Service I (stage III):

6. ULTIMATE LOAD CHECK AND SHEAR CAPACITY CHECK

- 6.1 Flexural moment checking
- 6.2. Ultimate load check

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/m²)

Length of Girder

L_d = 40.00 (m)

Span between support

L_{tt} = 39.10 (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 between 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 = 1.80 (m)

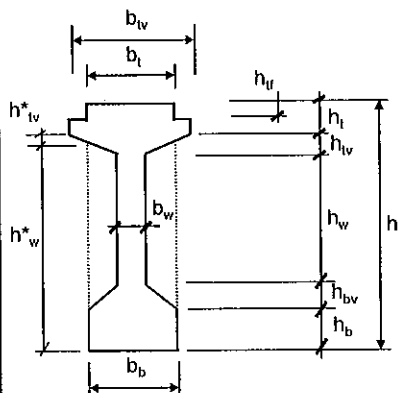
Precast plank thick

h_p = 0.08 (m)

Pavement thick

h_{pa} = 0.084 (m)

1.2. Girder dimension:

	Width of over part	b _{iv} = 950.00 (mm)
		b _t = 750.00 (mm)
	Width of under part	b _b = 750.00 (mm)
	Girder high	h = 2100.00 (mm)
		h _{iv} = 80.00 (mm)
		h _t = 200.00 (mm)
Cross section at end		
	b _w = 750.00	300.00 (mm)
	h* _{iv} = 34.00	110.00 (mm)
	h* _w = 1866.00	1340.00 (mm)
	h _b = 0.00	250.00 (mm)
	h _{bv} = 0.00	200.00 (mm)
at the middle		

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/m³

Modulus of elasticity

E_c = 0.043 γ_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/m³

Modulus of elasticity

E_c = 0.043 γ_c^{1.5} sqrt(f_c) = 29910.20 MPa (5.4.2.4-1)

1.3.2 Prestressing steel

Diameter of one strand

D = 15.20 mm

Area of one strand

A_s"15.2" = 140.00 mm²

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

Coefficient of friction (1/RAD)

μ = 0.25

Number of Strands in one Tendon

n = 12.00 Strands

Area of one Tendon

A_s = 1680.00 mm²

Stress in the prestressing steel at jacking

f_{pj} = 0.7 f_{pu} = 1302.00 MPa

Jacking force for one tendon

P_j = 2187.36 kN

Anchorage set

ΔL = 6.00 mm

Area of one duck

A_g = 5026.55 mm²

Number of Tendons

N = 6.00 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 =	13.33 (kN/m)
Precast plank & cross beam	DC _{pl} =	5.22 (kN/m)
Parapet	DC _{pa} =	4.74 (kN/m)
Pavement	DW _p =	4.44 (kN/m)

Exterior Beam:

Bridge deck	DC _d =	13.33 (kN/m)
Precast plank & cross beam	DC _{pl} =	2.61 (kN/m)
Parapet	DC _{pa} =	5.48 (kN/m)
Pavement	DW _p =	4.44 (kN/m)

2.1.2 Internal Force due to dead load:

Formula :

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

$$Q = q \cdot (0.5 \cdot L - X_i)$$

$$L_{it} = 39.10 \text{ (m)}$$

INTERIOR GIRDER											
Section	X _i (m)	Girder (DC)		Concrete Deck (DC)		Plank & cr.beam (DC)		Parapet (DC)		Pavement (DW)	
		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	513.14	0.00	260.57	0.00	101.97	0.00	92.67	0.00	86.83
L/8	4.89	2194.47	384.85	1114.36	195.43	436.10	76.48	396.30	69.50	371.34	65.12
L/4	9.78	3761.95	256.57	1910.34	130.29	747.59	50.99	679.36	46.33	636.58	43.42
3L/8	14.66	4702.43	128.28	2387.92	65.14	934.49	25.49	849.21	23.17	795.73	21.71
L/2	19.55	5015.93	0.00	2547.12	0.00	996.79	0.00	905.82	0.00	848.78	0.00
EXTERIOR GIRDER											
Gđi	0.00	0.00	513.14	0.00	260.57	0.00	101.97	0.00	107.13	0.00	86.83
L/8	4.89	2194.47	384.85	1114.36	195.43	218.05	76.48	458.17	80.35	371.34	65.12
L/4	9.78	3761.95	256.57	1910.34	130.29	373.80	50.99	785.43	53.57	636.58	43.42
3L/8	14.66	4702.43	128.28	2387.92	65.14	467.25	25.49	981.78	26.78	795.73	21.71
L/2	19.55	5015.93	0.00	2547.12	0.00	498.40	0.00	1047.23	0.00	848.78	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

$$e_g^l = 1161.49 \text{ (mm)}$$

$$e_g^E = 1161.49 \text{ (mm)}$$

Longitudinal stiffness parameter

$$K_g^l = n(lg + A e_g^2) = 3.2E+12$$

$$K_g^E = n(lg + A e_g^2) = 3.2E+12$$

Ration

$$K_g^l / (L t_s^3) = 7.33$$

$$K_g^E / (L t_s^3) = 7.33$$

$$S / L = 0.07$$

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

Interior Beam:

$$\text{For one lane} \quad 0.06 + \left(\frac{S}{4300} \right)^{0.4} \left(\frac{S}{L} \right)^{0.3} \left(\frac{K_g}{L t_s^3} \right)^{0.1} = 0.497$$

$$\text{Two or more lanes} \quad 0.075 + \left(\frac{S}{2900} \right)^{0.6} \left(\frac{S}{L} \right)^{0.2} \left(\frac{K_g}{L t_s^3} \right)^{0.1} = 0.729$$

Exterior Beam:

For one lane, follow the lever rule

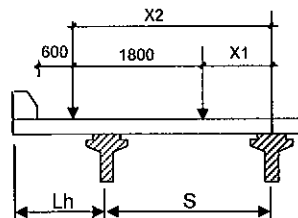
$$X_1 = 925.000$$

$$X_2 = 2725.00$$

$$Y_1 = 0.363$$

$$Y_2 = 1.069$$

$$\Rightarrow g(M) = 0.5 \cdot \Sigma y_i = 0.716$$



Two or more lanes

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

$$\text{Choice } e = 1.047 \quad \text{IF}(e > 1, 1, e)$$

$$\Rightarrow g(M) = e \cdot g_{\text{long}} = 0.764$$

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

Interior Beam:

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

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

$$g(Q) = 0.5 \cdot \Sigma y_i = \boxed{0.716}$$

Two or more lanes

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

$$\Rightarrow g(Q) = e \cdot g_{\text{strong}} = \boxed{0.731}$$

(c) Correction factor for skew bridge:

* Correction factor of distribution factor for moment (Table 4.6.2.2d-1)

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

Area of applications
 $300 \leq \theta \leq 600$
 $1100 \leq S \leq 4900$
 $6000 \leq L \leq 73000$
 $Nb \geq 4$

$$CF(M) = 1.0 - c1 \cdot (\tan \theta)^{1.5}$$

$$c1 = 0.25 \left(\frac{Kg}{L \cdot ts^3} \right)^{0.25} \left(\frac{S}{L} \right)^{0.5}$$

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

Correction Factor $CF(Q) = 1.040$

Area of applications
 $00 \leq \theta \leq 600$
 $1100 \leq S \leq 4900$
 $6000 \leq L \leq 73000$
 $Nb \geq 4$

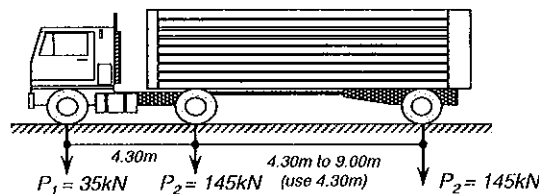
$$CF(Q) = 1.0 + 0.20 \left(\frac{L \cdot ts^3}{Kg} \right)^{0.3} \cdot \tan \theta$$

(d) Table of Distribution factors for Live load:

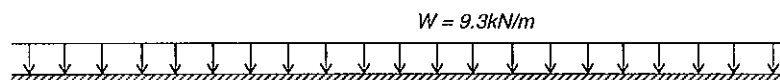
Interior Beam	g(M)	g(Q)	m	m*g(M)	m*g(Q)	m*g(M)*CF(M)	m*g(Q)*CF(Q)
1 lane	0.497	0.696	1.20	0.596	0.835	0.596	0.868
2 or more lanes	0.729	0.852	1.00	0.729	0.852	0.729	0.886
Exterior Beam							
1 lane	0.716	0.716	1.20	0.859	0.859	0.859	0.893
2 or more lanes	0.764	0.731	1.00	0.764	0.731	0.764	0.760

2.2.2 Live Load:

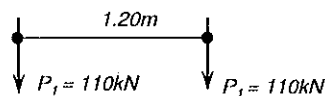
Design Truck



Design Lane Load



Design Tandem



- 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
- Dynamic load	IM = 0.25

2.2.3 Internal Force due to Live load:

Design truck or Tandem

Momen $M_{TR(Ta)} = \Sigma P_i y_i$ (kNm)Shear forc $Q_{TR(Ta)} = \Sigma P_i y_i$ (kN)

Lane load

Momen $M_{Ln} = W \cdot F_M$ (kNm)Shear forc $Q_{Ln} = W \cdot F_Q$ (kN)

Pedestrian

Momen $M_{PL} = PL \cdot F_M$ (kNm)Shear forc $Q_{PL} = PL \cdot F_Q$ (kN)

Influence line for Momen & Shear force		Load	Momen (KN.m)	Shear
Section at 1/2L		Truck	2789.88	138.86
		Lane	1777.24	45.45
		Tandem	2084.50	106.62
		Design	2789.88	138.86
		Pedestrian	0.00	0.00
Section at 3/8L		Truck	2650.45	179.48
		Lane	1666.16	71.02
		Tandem	1966.59	134.12
		Design	2650.45	179.48
		Pedestrian	0.00	0.00
Section at 1/4L		Truck	2151.53	220.11
		Lane	1332.93	102.27
		Tandem	1579.88	161.62
		Design	2151.53	220.11
		Pedestrian	0.00	0.00
Section at 1/8L		Truck	1274.32	260.73
		Lane	777.54	139.20
		Tandem	924.34	189.12
		Design	1274.32	260.73
		Pedestrian	0.00	0.00
At support		Truck	0.00	301.36
		Lane	0.00	181.82
		Tandem	0.00	216.62
		Design	0.00	301.36
		Pedestrian	0.00	0.00

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

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

Internal Force due to pedestrian :

$M = g(M) \cdot M_{PL}$

$Q = g(Q) \cdot Q_{PL}$

In which:

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

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

y_i Value of influence line

F Area of influence line

m Lane factor

g Distribution factor

Interior	$m \cdot g(M)$	$m \cdot g(Q)$
	0.729	0.886
Exterior		
	0.859	0.893

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	494.64	0.00	498.87
L/8	4.89	1729.13	411.92	2035.79	415.45
L/4	9.78	2934.11	334.24	3454.48	337.10
3L/8	14.66	3632.11	261.59	4276.27	263.83
L/2	19.55	3840.27	193.97	4521.35	195.63

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 factord

$\eta = \eta_D \eta_R \eta_I$

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	
		M	Q	M	Q	M	Q	M	Q	M	Q
	γ	(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)
DC	1.25	0.00	1210.44	5176.53	907.83	8874.06	605.22	11092.57	302.61	11832.08	0.00
DW	1.50	0.00	130.25	557.01	97.69	954.87	65.12	1193.59	32.56	1273.16	0.00
LL+IM	1.75	0.00	865.61	3025.97	720.87	5134.70	584.92	6356.19	457.79	6720.47	339.46
Total		0.00	2206.30	8759.52	1726.38	14963.63	1255.27	18642.36	792.96	19825.71	339.46

STATE		Service									
Load	Load Factor	Section									
		Support		L/8		L/4		3L/8		L/2	
		M	Q	M	Q	M	Q	M	Q	M	Q
	γ	(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)
DC	1.00	0.00	968.35	4141.23	726.27	7099.25	484.18	8874.06	242.09	9465.66	0.00
DW	1.00	0.00	86.83	371.34	65.12	636.58	43.42	795.73	21.71	848.78	0.00
LL+IM	1.00	0.00	494.64	1729.13	411.92	2934.11	334.24	3632.11	261.59	3840.27	193.97
Total		0.00	1549.82	6241.70	1203.31	10869.94	861.83	13301.90	525.39	14154.71	193.97

2.3.2 Load combination - Exterior Girder:

STATE		Strength									
Load	Load factor	Section									
		Supprt		L/8		L/4		3L/8		L/2	
		M	Q	M	Q	M	Q	M	Q	M	Q
	γ	(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)
DC	1.25	0.00	1228.53	4981.31	921.39	8539.39	614.26	10674.24	307.13	11385.85	0.00
DW	1.50	0.00	130.25	557.01	97.69	954.87	65.12	1193.59	32.56	1273.16	0.00
LL+IM	1.75	0.00	873.02	3562.64	727.03	6045.35	589.93	7483.48	461.70	7912.36	342.36
Total		0.00	2231.79	9100.96	1746.11	15539.61	1269.31	19351.30	801.40	20571.38	342.36

STATE		Service									
Load	load factor	Section									
		Support		L/8		L/4		3L/8		L/2	
		M	Q	M	Q	M	Q	M	Q	M	Q
	γ	(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)	(kNm)	(kN)
DC	1.00	0.00	982.82	3985.05	737.12	6831.51	491.41	8539.39	245.71	9108.68	0.00
DW	1.00	0.00	86.83	371.34	65.12	636.58	43.42	795.73	21.71	848.78	0.00
LL+IM	1.00	0.00	498.87	2035.79	415.45	3454.48	337.10	4276.27	263.83	4521.35	195.63
Total		0.00	1568.52	6392.18	1217.69	10922.58	871.93	13611.39	531.24	14478.81	195.63

3. TENDON PROFILE AND PROPERTY OF GIRDER CROSS SECTION

3.1. Tendon profile:

Tendon profile follow Parabol equation:

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

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 = X₂ - X₁ Distance between 2 point under consider
 angle of rotation of tendon for X_i-axis $\tan(\alpha) = (4 \cdot f \cdot (1 - 2 \cdot X_i / L)) / L$
 $\alpha = 2 \cdot f / 0.5 \cdot L - \tan(\alpha)$

L _{span} =	40000 (mm)
L _{su} =	39100 (mm)
L _{cap} =	39700 (mm)

TENDON No 1	f =	1860	(mm)	Lcáp =	39700	(mm)	C =	530	(mm)
	Section	XI	Yi	Lp	ΣLcáp	Tan(αi)	(α)	Σα	Section
		(mm)	(mm)	(mm)	(mm)		(rad)	(rad)	
	Anchorag	0.00	1860.00	0.00	0.00	0.1874	0.0000	0.0000	Anchorage
	Support	300.00	1820.10	300.00	300.00	0.1846	0.0028	0.0028	Support
	L/8	5187.50	1255.68	4887.50	5187.50	0.1384	0.0490	0.0518	L/8
	L/4	10075.00	852.53	4887.50	10075.00	0.0923	0.0951	0.1469	L/4
	3L/8	14962.50	610.63	4887.50	14962.50	0.0461	0.1413	0.2882	3L/8
	L/2	19850.00	530.00	4887.50	19850.00	0.0000	0.1874	0.4756	L/2

TENDON No 2	f =	1530	(mm)	Lcáp =	39700	(mm)	C =	400	(mm)
	Section	XI	Yi	Lp	ΣLcáp	Tan(α)	(α)	Σα	Section
		(mm)	(mm)	(mm)	(mm)		(rad)	(rad)	
	Anchorag	0.00	1530.00	0.00	0.00	0.1542	0.0000	0.0000	Anchorage
	Support	300.00	1496.10	300.00	300.00	0.1518	0.0023	0.0023	Support
	L/8	5187.50	1016.56	4887.50	5187.50	0.1139	0.0403	0.0426	L/8
	L/4	10075.00	674.03	4887.50	10075.00	0.0759	0.0782	0.1209	L/4
	3L/8	14962.50	468.51	4887.50	14962.50	0.0380	0.1162	0.2371	3L/8
	L/2	19850.00	400.00	4887.50	19850.00	0.0000	0.1542	0.3912	L/2

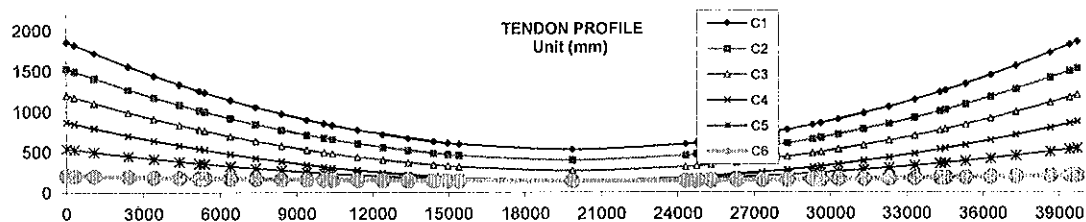
TENDON No 3	f =	1200	(mm)	Lcáp =	39700	(mm)	C =	270	(mm)
	Section	Xi	Yi	Lp	ΣLcáp	Tan(α)	(α)	Σα	Section
		(mm)	(mm)	(mm)	(mm)		(rad)	(rad)	
	Anchorag	0.00	1200.00	0.00	0.00	0.1209	0.0000	0.0000	Anchorage
	Support	300.00	1172.10	300.00	300.00	0.1191	0.0018	0.0018	Support
	L/8	5187.50	777.43	4887.50	5187.50	0.0893	0.0316	0.0334	L/8
	L/4	10075.00	495.53	4887.50	10075.00	0.0595	0.0614	0.0948	L/4
	3L/8	14962.50	326.38	4887.50	14962.50	0.0298	0.0911	0.1859	3L/8
	L/2	19850.00	270.00	4887.50	19850.00	0.0000	0.1209	0.3068	L/2

TENDON No 4	f =	870	(mm)	L _{cap} =	39700	(mm)	C =	140	(mm)
	Section	Xi	Yi	L _p	ΣL _{cap}	Tan(α)	(α)	Σα	Section
		(mm)	(mm)	(mm)	(mm)		(rad)	(rad)	
	Anchorag	0.00	870.00	0.00	0.0	0.0877	0.0000	0.0000	Anchorage
	Support	300.00	848.10	300.00	300.0	0.0863	0.0013	0.0013	Support
	L/8	5187.50	538.31	4887.50	5187.5	0.0647	0.0229	0.0242	L/8
	L/4	10075.00	317.03	4887.50	10075.0	0.0432	0.0445	0.0687	L/4
	3L/8	14962.50	184.26	4887.50	14962.5	0.0216	0.0661	0.1348	3L/8
	L/2	19850.00	140.00	4887.50	19850.0	0.0000	0.0877	0.2225	L/2

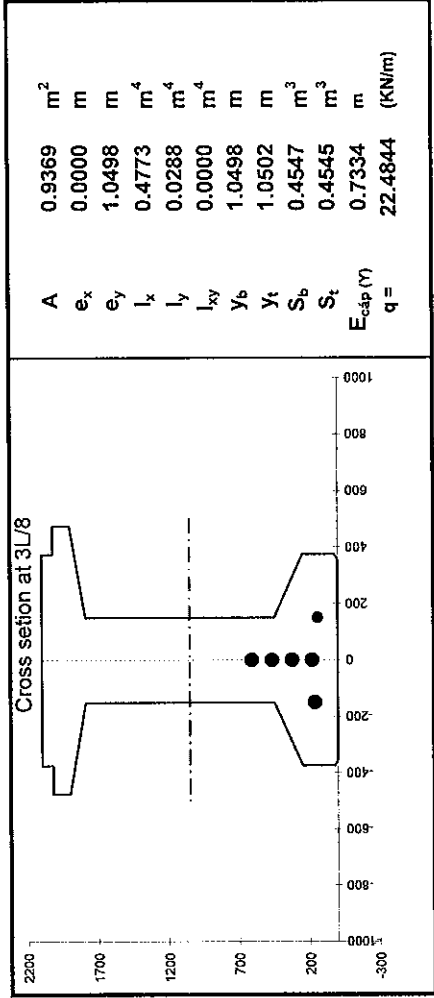
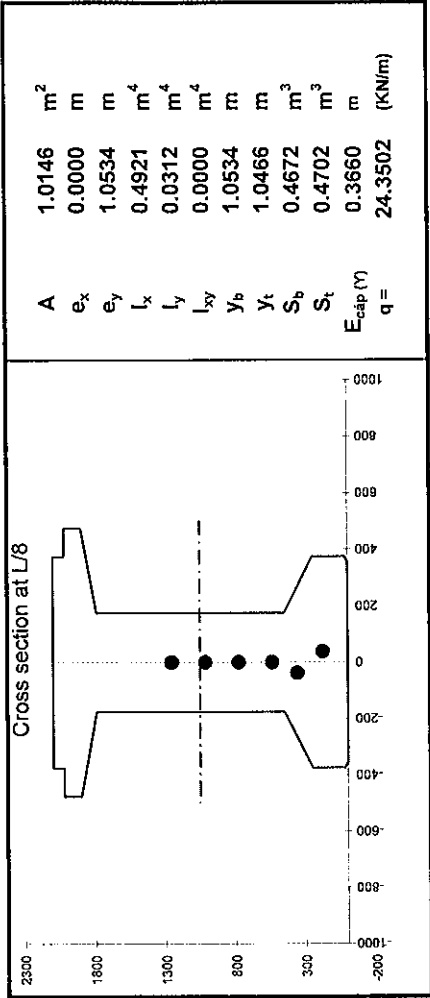
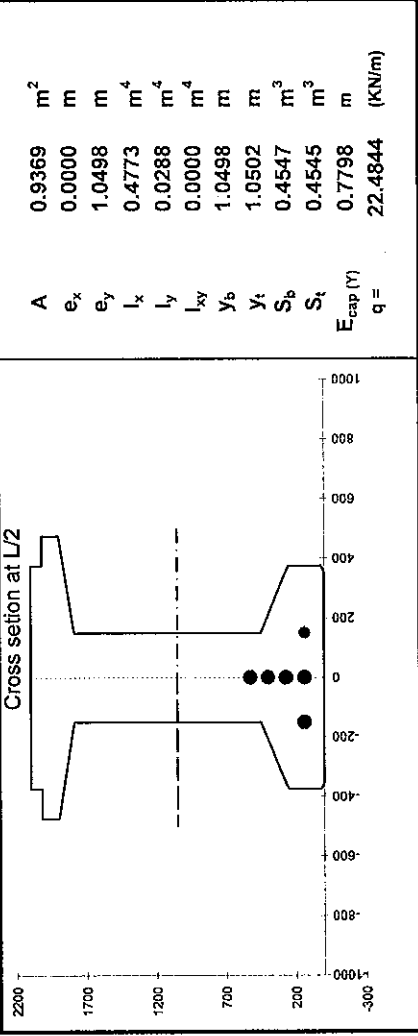
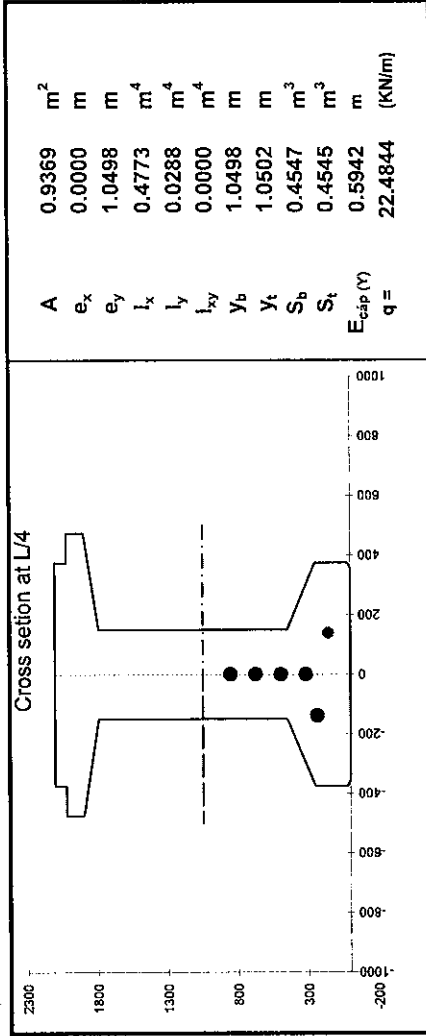
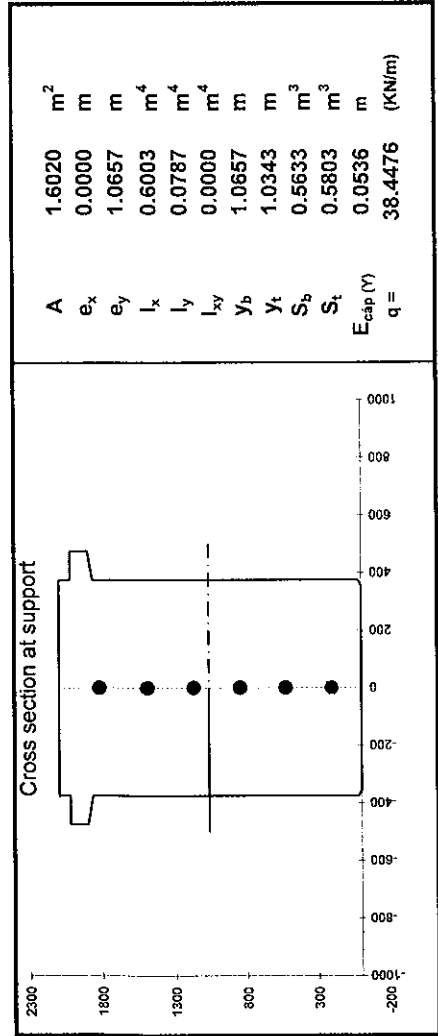
TENDON No 5	f =	540	(mm)	Lcáp =	39700	(mm)	C =	140	(mm)
	Section	Xi	Yi	Lp	ΣLcáp	Tan(α)	(α)	Σα	Section
		(mm)	(mm)	(mm)	(mm)		(rad)	(rad)	
	Anchorag	0.00	870.00	0.00	0.0	0.0877	0.0000	0.0000	Anchorage
	Support	300.00	848.10	300.00	300.0	0.0863	0.0013	0.0013	Support
	L/8	5187.50	538.31	4887.50	5187.5	0.0647	0.0229	0.0242	L/8
	L/4	10075.00	317.03	4887.50	10075.0	0.0432	0.0445	0.0687	L/4
	3L/8	14962.50	184.26	4887.50	14962.5	0.0216	0.0661	0.1348	3L/8
	L/2	19850.00	140.00	4887.50	19850.0	0.0000	0.0877	0.2225	L/2

TENDON No 6	f =	210	(mm)	Lcáp =	39700	(mm)	C =	140	(mm)
	Section	Xi	Yi	Lp	ΣLcáp	Tan(α)	(α)	Σα	Section
		(mm)	(mm)	(mm)	(mm)		(rad)	(rad)	
	Anchorag	0.00	210.00	0.00	0.0	0.0212	0.0000	0.0000	Anchorage
	Support	300.00	207.90	300.00	300.0	0.0208	0.0003	0.0003	Support
	L/8	5187.50	178.19	4887.50	5187.5	0.0156	0.0055	0.0058	L/8
	L/4	10075.00	156.98	4887.50	10075.0	0.0104	0.0107	0.0166	L/4
	3L/8	14962.50	144.24	4887.50	14962.5	0.0052	0.0159	0.0325	3L/8
	L/2	19850.00	140.00	4887.50	19850.0	0.0000	0.0212	0.0537	L/2

Section	TENDON No 1		TENDON No 2		TENDON No 3		TENDON No 4		TENDON No 5		TENDON No 6	
	X _i	Y _i	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)	(mm)	(mm)
Anchorage	0.00	1860.00	0.0	1530.00	0.0	1200.00	0.00	870.00	0.00	540.00	0.00	210.00
Support	300.00	1820.10	300.0	1496.10	300.0	1172.10	300.00	848.10	300.00	528.00	300.00	207.90
1	1050.00	1723.02	1050.0	1413.62	1050.0	1104.21	1050.00	794.81	1050.00	498.80	1050.00	202.79
2	2400.00	1557.83	2400.0	1273.27	2400.0	988.71	2400.00	704.15	2400.00	449.12	2400.00	194.10
3	3400.00	1443.40	3400.0	1176.05	3400.0	908.70	3400.00	641.34	3400.00	414.71	3400.00	188.07
L/8	5187.50	1255.68	5187.5	1016.56	5187.5	777.43	5187.50	538.31	5187.50	358.25	5187.50	178.19
4	4400.00	1335.73	4400.0	1084.56	4400.0	833.40	4400.00	582.24	4400.00	382.32	4400.00	182.41
5	5400.00	1234.80	5400.0	998.82	5400.0	762.83	5400.00	526.85	5400.00	351.97	5400.00	177.09
6	6400.00	1140.63	6400.0	918.80	6400.0	696.98	6400.00	475.16	6400.00	323.65	6400.00	172.14
7	7400.00	1053.20	7400.0	844.52	7400.0	635.85	7400.00	427.17	7400.00	297.35	7400.00	167.54
L/4	10075.00	852.53	10075.0	674.03	10075.0	495.53	10075.00	317.03	10075.00	237.00	10075.00	156.98
8	8400.00	972.53	8400.0	775.98	8400.0	579.44	8400.00	382.89	8400.00	273.09	8400.00	163.29
9	9400.00	898.61	9400.0	713.18	9400.0	527.75	9400.00	342.32	9400.00	250.86	9400.00	159.40
10	10400.00	831.44	10400.0	656.11	10400.0	480.78	10400.00	305.45	10400.00	230.66	10400.00	155.87
11	11400.00	771.01	11400.0	604.77	11400.0	438.53	11400.00	272.29	11400.00	212.49	11400.00	152.68
3L/8	14962.50	610.63	14962.5	468.51	14962.5	326.38	14962.50	184.26	14962.50	164.25	14962.50	144.24
12	12400.00	717.35	12400.0	559.17	12400.0	401.00	12400.00	242.83	12400.00	196.34	12400.00	149.86
13	13400.00	670.43	13400.0	519.31	13400.0	368.19	13400.00	217.08	13400.00	182.23	13400.00	147.39
14	14400.00	630.26	14400.0	485.18	14400.0	340.11	14400.00	195.03	14400.00	170.15	14400.00	145.28
15	15400.00	596.84	15400.0	456.79	15400.0	316.74	15400.00	176.69	15400.00	160.10	15400.00	143.52
L/2	19850.00	530.00	19850.0	400.00	19850.0	270.00	19850.00	140.00	19850.00	140.00	19850.00	140.00
2	24300.00	596.84	24300.0	456.79	24300.0	316.74	24300.00	176.69	24300.00	160.10	24300.00	143.52
3	25300.00	630.26	25300.0	485.18	25300.0	340.11	25300.00	195.03	25300.00	170.15	25300.00	145.28
4	26300.00	670.43	26300.0	519.31	26300.0	368.19	26300.00	217.08	26300.00	182.23	26300.00	147.39
5	27300.00	717.35	27300.0	559.17	27300.0	401.00	27300.00	242.83	27300.00	196.34	27300.00	149.86
-	24737.50	610.63	24737.5	468.51	24737.5	326.38	24737.50	184.26	24737.50	164.25	24737.50	144.24
6	28300.00	771.01	28300.0	604.77	28300.0	438.53	28300.00	272.29	28300.00	212.49	28300.00	152.68
7	29300.00	831.44	29300.0	656.11	29300.0	480.78	29300.00	305.45	29300.00	230.66	29300.00	155.87
8	30300.00	898.61	30300.0	713.18	30300.0	527.75	30300.00	342.32	30300.00	250.86	30300.00	159.40
9	31300.00	972.53	31300.0	775.98	31300.0	579.44	31300.00	382.89	31300.00	273.09	31300.00	163.29
-	29625.00	852.53	29625.0	674.03	29625.0	495.53	29625.00	317.03	29625.00	237.00	29625.00	156.98
10	32300.00	1053.20	32300.0	844.52	32300.0	635.85	32300.00	427.17	32300.00	297.35	32300.00	167.54
11	33300.00	1140.63	33300.0	918.80	33300.0	696.98	33300.00	475.16	33300.00	323.65	33300.00	172.14
12	34300.00	1234.80	34300.0	998.82	34300.0	762.83	34300.00	526.85	34300.00	351.97	34300.00	177.09
13	35300.00	1335.73	35300.0	1084.56	35300.0	833.40	35300.00	582.24	35300.00	382.32	35300.00	182.41
-	34512.50	1255.68	34512.5	1016.56	34512.5	777.43	34512.50	538.31	34512.50	358.25	34512.50	178.19
14	36300.00	1443.40	36300.0	1176.05	36300.0	908.70	36300.00	641.34	36300.00	414.71	36300.00	188.07
14	37300.00	1557.83	37300.0	1273.27	37300.0	988.71	37300.00	704.15	37300.00	449.12	37300.00	194.10
16	38650.00	1723.02	38650.0	1413.62	38650.0	1104.21	38650.00	794.81	38650.00	498.80	38650.00	202.79
Support	39400.00	1820.10	39400.0	1496.10	39400.0	1172.10	39400.00	848.10	39400.00	528.00	39400.00	207.90
Anchorage	39700.00	1860.00	39700.0	1530.00	39700.0	1200.00	39700.00	870.00	39700.00	540.00	39700.00	210.00



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



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

3.3.1. Effective flange width

Modular Ratio: Deck Concrete/Girder Concrete

(4.6.2.6)

$$n = E_b / E_d =$$

0.88

For Interior Girder:

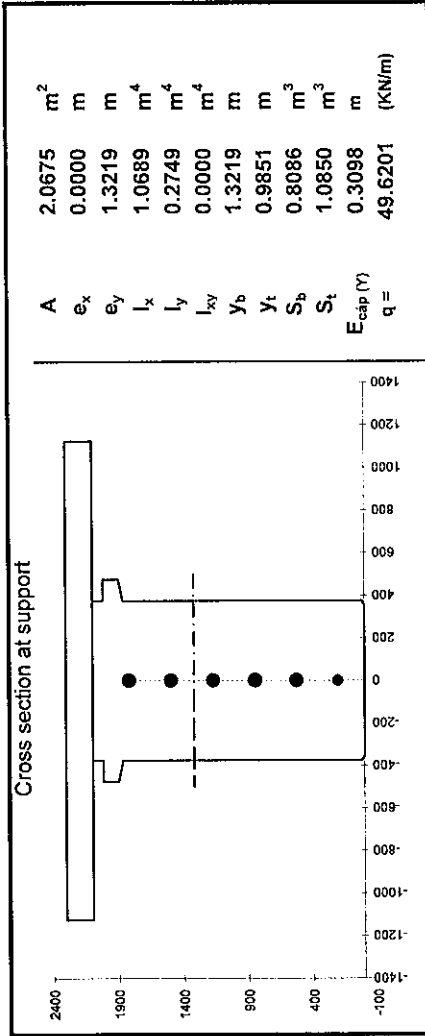
$$b_f = \min \left\{ \begin{array}{l} 1/4 L_{II} \\ 12h_f + \max(0.5b_w, b_w) \end{array} \right\} S \Rightarrow n^* b_i = 2248.88861 \text{ (mm)}$$

For Exterior Girder:

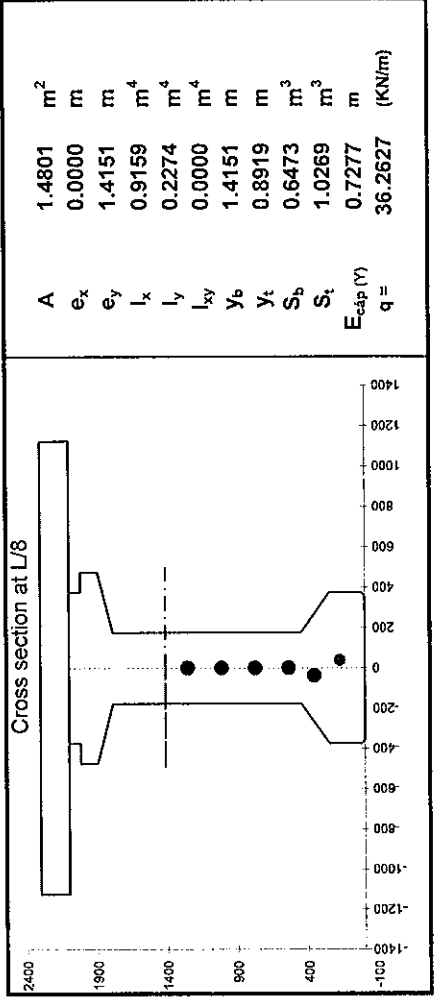
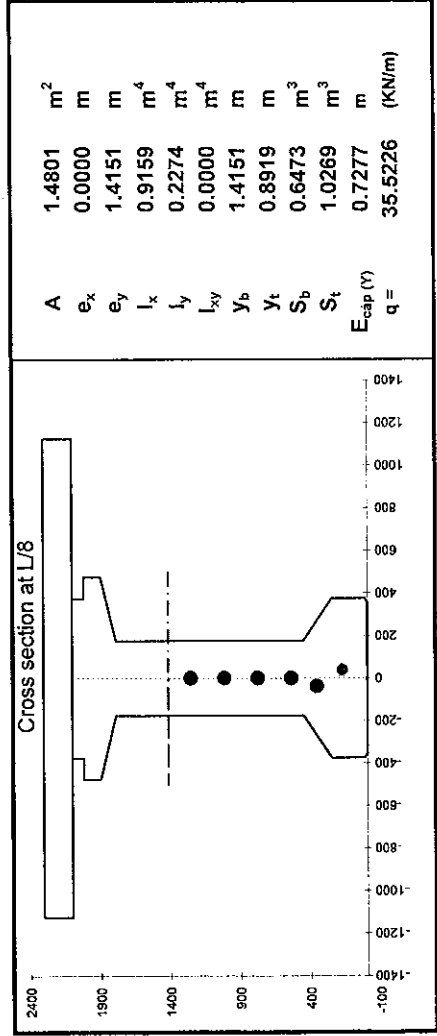
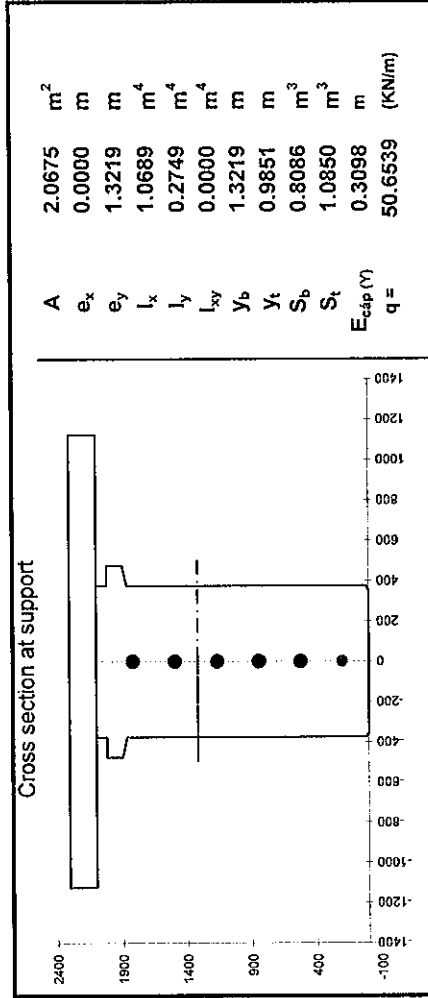
$$b_E = 0.5b_f + \min \left\{ \begin{array}{l} 1/8 L_{II} \\ 6h_f + \max(0.5b_w, 0.25b) \end{array} \right\} L_h \Rightarrow n^* b_E = 2248.88861 \text{ (mm)}$$

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

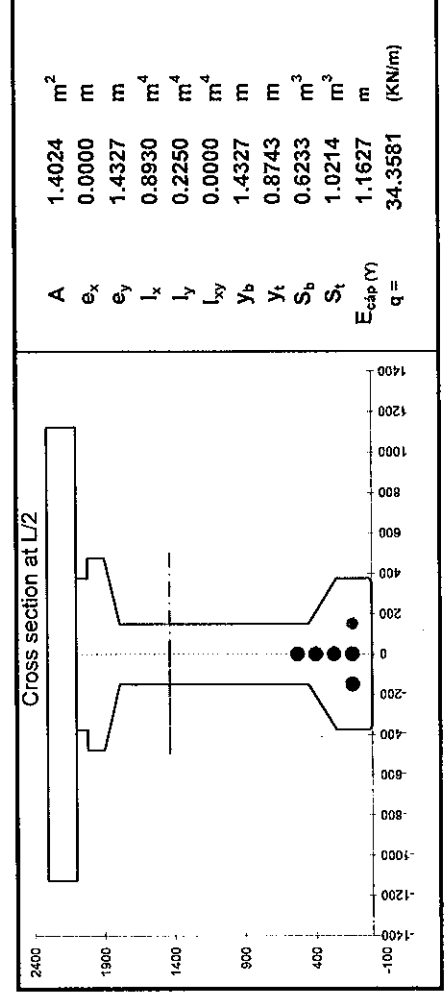
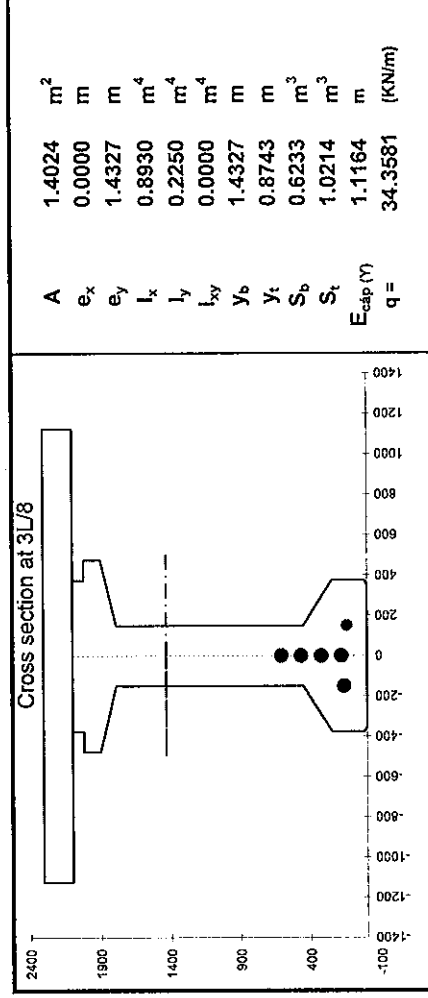
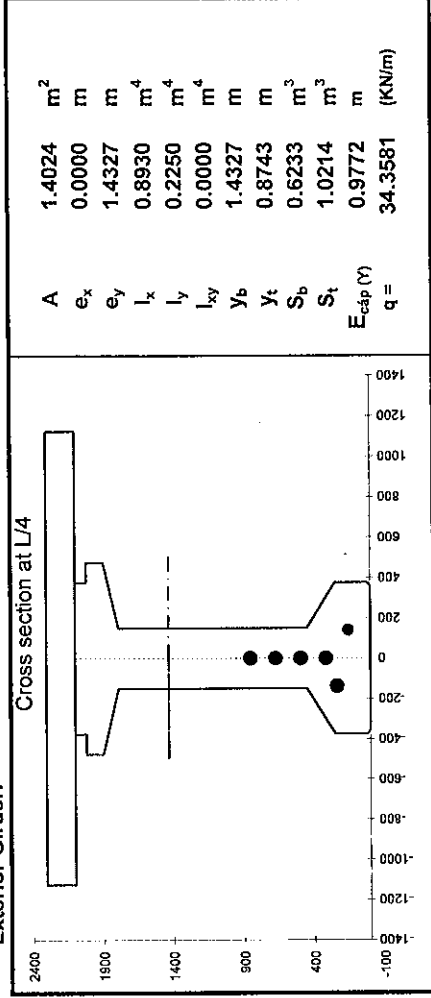
Interior Girder:



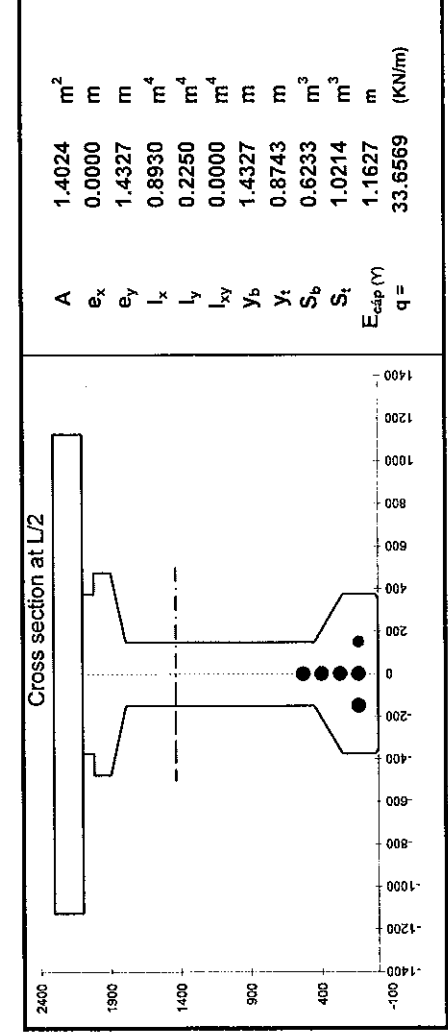
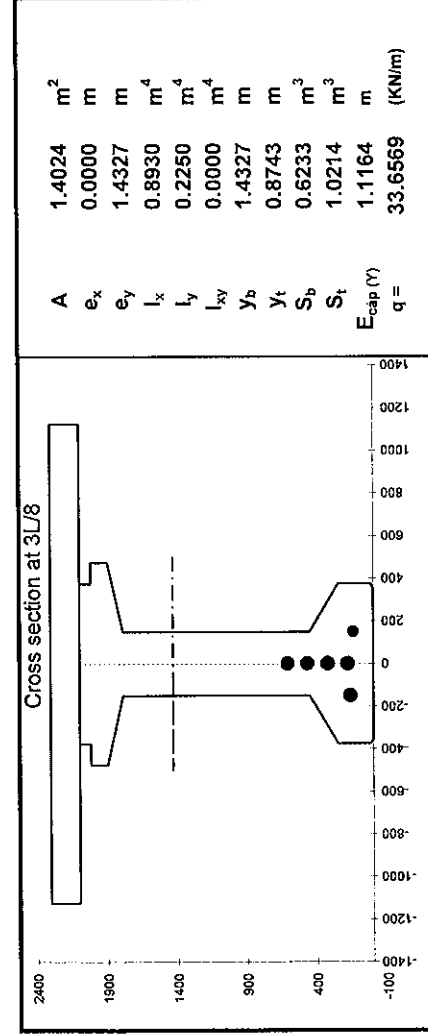
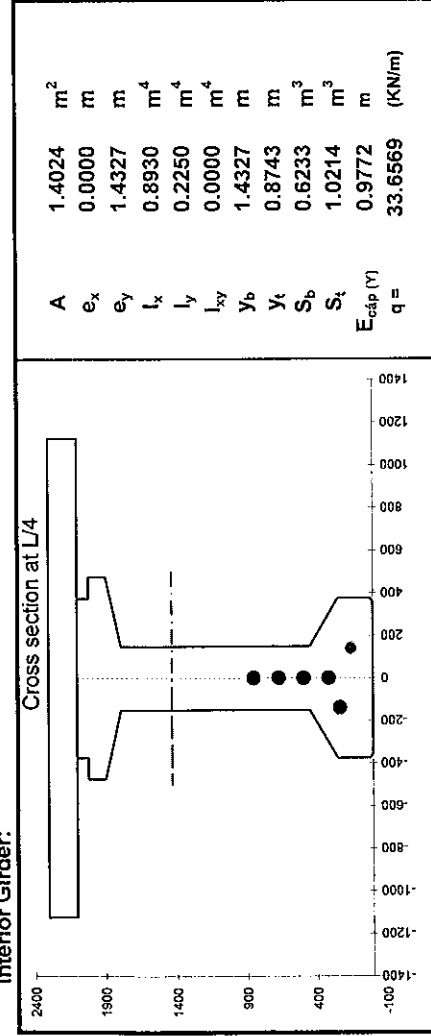
Exterior Girder:



Exterior Girder:



Interior Girder:



4. LOSS OF PRESTRESS

4.1 Loss of prestressing force Immediately (Instantaneous losses):

4.1.1 Friction between Prestressing Tendon and Duck:

Formula: $\Delta f_{pF} = f_{pj} (1 - e^{-(kx + \mu \theta)})$ (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		Tendon no. 6		SUM/6
	X_i	$\Sigma\alpha$	Δf_{pF}	$\Sigma\alpha$	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)	(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	0.0000	0.00	0.00
Support	300.00	0.0028	1.18	0.0023	1.02	0.0018	0.85	0.0013	0.69	0.0013	0.69	0.0003	0.36	0.80
L/8	5187.50	0.0518	21.15	0.0426	18.20	0.0334	15.25	0.0242	12.29	0.0242	12.29	0.0058	6.35	14.25
L/4	10075.00	0.1469	55.27	0.1209	47.12	0.0948	38.92	0.0687	30.66	0.0687	30.66	0.0166	13.98	36.10
3L/8	14982.50	0.2882	102.41	0.2371	86.98	0.1859	71.35	0.1348	55.52	0.1348	55.52	0.0325	23.24	65.83
L/2	19850.00	0.4756	161.00	0.3912	136.68	0.3068	111.84	0.2225	86.46	0.2225	86.46	0.0537	34.08	102.75

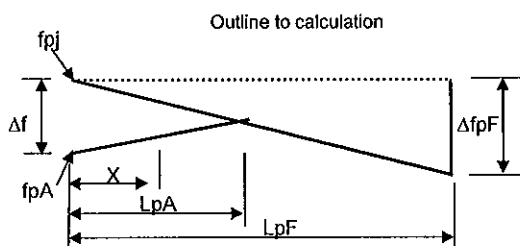
4.1.2 Anchorage seating or Set:

Formula

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

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

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



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 length from anchorage to 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 (L_{pF}) and calculation follow:

Tendon no.1	XI (mm)	Δf_{pA} (MPa)
LpF = 19850	0	195.83
Δf_{pF} = 161.00	300	190.96
LpA = 12071.8	5188	111.68
Δf = 195.83	10075	32.39
	14963	0.00
	19850	0.00

Tendon no.2	XI (mm)	Δf_{pA} (MPa)
L _{pF} =	19850	0
Δf_{pF} =	136.68	300
L _{pA} =	13102.0	5188
Δf =	180.43	10075
	14963	0.00
	19850	0.00

Tendon no.3	XI (mm)	Δf_{pA} (MPa)
$L_{pF} =$ 19850	0	163.21
$\Delta f_{pF} =$ 111.84	300	159.83
$L_{pA} =$ 14484.4	5188	104.76
$\Delta f =$ 163.21	10075	49.68
	14963	0.00
	19850	0.00

Tendon no.4	XI (mm)	Δf_{pA} (MPa)
LpF = 19850	0	143.51
Δf_{pF} = 86.46	300	140.89
LpA = 16473.1	5188	98.32
Δf = 143.51	10075	55.74
	14963	13.16
	19850	0.00

Tendon no.5	Xi (mm)	Δf_{pA} (MPa)
LpF = 19850	0	143.51
Δf_{pF} = 86.46	300	140.89
LpA = 16473.1	5188	98.32
Δf = 143.51	10075	55.74
	14983	13.16
	19850	0.00

Tendon no.6		Xi (mm)	Δf_{pA} (MPa)
$L_{pF} =$	19850	0	143.51
$\Delta f_{pF} =$	34.08	300	140.89
$L_{pA} =$	19850.0	5188	98.32
$\Delta f =$	68.16	10075	55.74
		14963	13.16
		19850	0.00

4.1.3 Elastic deformation of concrete:

Formula

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

In which:

Number of tendon	N =	6.00 (Tendon)
Cable modulus of elasticity	E _p =	197000.0 MPa
Concrete strength at transfer	f _{ci} =	40.50 MPa
Unit weight of concrete	γ _c =	2450.00 kg/m ³
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:

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

A	Area of girder cross section
I _x	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 (mm)	Tendon1 ΔfpF+ΔfpA (MPa)	Tendon2 ΔfpF+ΔfpA (MPa)	Tendon3 ΔfpF+ΔfpA (MPa)	Tendon4 ΔfpF+ΔfpA (MPa)	Tendon5 ΔfpF+ΔfpA (MPa)	Tendon8 ΔfpF+ΔfpA (MPa)	SUM (MPa)	ΣFj (kN)
Anchorage	0	195.83	180.43	163.21	143.51	143.51	143.51	969.99	11494.58
Support	300	192.14	177.31	160.68	141.58	141.58	141.25	954.56	11520.50
L/8	5188	132.82	127.19	120.00	110.60	110.60	104.66	705.89	11938.27
L/4	10075	87.67	88.81	88.60	86.40	86.40	69.72	507.59	12271.40
3L/8	14963	102.41	86.98	71.35	68.68	68.68	36.40	434.49	12394.22
L/2	19850	161.00	136.68	111.84	86.46	86.46	34.08	616.52	12088.40

Loss stress due to Elastic deformation of concrete

Section	Xi (mm)	Fj (kN)	A (mm ²)	I _x (mm ⁴)	e (mm)	M _{DC} (kNm)	f _{cgp} (MPa)	Δf _{ES} (MPa)
Anchorage	0	11494.58	1.6E+06	6.0E+11	53.61	0.00	7.23	17.88
Support	300	11520.50	1.6E+06	6.0E+11	53.61	0.00	7.25	17.92
L/8	5188	11938.27	1.0E+06	4.9E+11	365.95	2194.47	13.38	33.10
L/4	10075	12271.40	9.4E+05	4.8E+11	594.24	3761.95	17.49	43.27
3L/8	14963	12394.22	9.4E+05	4.8E+11	733.38	4702.43	19.97	49.40
L/2	19850	12088.40	9.4E+05	4.8E+11	779.76	5015.93	20.11	49.74

Total loss of prestressing force immediately - Remaining prestressing force:

Tendon1	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)
Section	(mm)	(MPa)	(MPa)	(MPa)	(MPa)	(kN)	(rad)	(kN)	(kN)
anchorage	0	0.00	195.83	17.88	213.71	1828.32	0.1853	1797.04	336.77
Support	300	1.18	190.96	17.92	210.07	1834.45	0.1825	1803.98	332.97
L/8	5188	21.15	111.68	33.10	165.93	1908.60	0.1376	1890.57	261.71
L/4	10075	55.27	32.39	43.27	130.94	1967.39	0.0920	1959.06	180.80
3L/8	14963	102.41	0.00	49.40	151.81	1932.32	0.0461	1930.27	89.07
L/2	19850	161.00	0.00	49.74	210.74	1833.32	0.0000	1833.32	0.00

Tendon2	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)
Section	(mm)	(MPa)	(MPa)	(MPa)	(MPa)	(kN)	(rad)	(kN)	(kN)
anchorage	0	0.00	180.43	17.17	197.60	1855.39	0.1530	1833.73	282.68
Support	300	1.02	176.30	17.21	194.52	1860.56	0.1507	1839.48	279.28
L/8	5188	18.20	108.99	31.78	158.97	1920.29	0.1134	1907.96	217.26
L/4	10075	47.12	41.69	41.54	130.35	1968.38	0.0758	1962.73	149.00
3L/8	14963	86.98	0.00	47.42	134.40	1961.57	0.0379	1960.16	74.40
L/2	19850	136.68	0.00	47.75	184.42	1877.53	0.0000	1877.53	0.00

Tendon3	Xi	Δf_{pF}	Δf_{pA}	Δf_{ES}	$\Sigma \Delta$	F_j^3	(α)	$F_j^3 \cdot \cos(\alpha)$	$F_j^3 \cdot \sin(\alpha)$
Section	(mm)	(MPa)	(MPa)	(MPa)	(MPa)	(kN)	(rad)	(kN)	(kN)
anchorage	0	0.00	163.21	42.47	205.68	1841.82	0.1203	1828.51	221.08
Support	300	0.85	159.83	42.56	203.24	1845.91	0.1185	1832.96	218.27
L/8	5188	15.25	104.76	78.61	198.61	1853.69	0.0891	1846.34	164.90
L/4	10075	38.92	49.68	102.75	191.35	1865.89	0.0595	1862.59	110.90
3L/8	14963	71.35	0.00	117.29	188.64	1870.44	0.0298	1869.61	55.66
L/2	19850	111.84	0.00	118.10	229.93	1801.07	0.0000	1801.07	0.00

Tendon4	Xi	Δf_{pF}	Δf_{pA}	Δf_{ES}	$\Sigma \Delta$	F_j^4	(α)	$F_j^4 \cdot \cos(\alpha)$	$F_j^4 \cdot \sin(\alpha)$
Section	(mm)	(MPa)	(MPa)	(MPa)	(MPa)	(kN)	(rad)	(kN)	(kN)
anchorage	0	0.00	143.51	14.31	157.81	1922.23	0.0874	1914.89	167.85
Support	300	0.69	140.89	14.34	155.92	1925.41	0.0861	1918.28	165.61
L/8	5188	12.29	98.32	26.48	137.09	1957.06	0.0647	1952.97	126.45
L/4	10075	30.66	55.74	34.62	121.01	1984.06	0.0431	1982.21	85.56
3L/8	14963	55.52	13.16	39.52	108.19	2005.60	0.0216	2005.13	43.28
L/2	19850	86.46	0.00	39.79	126.25	1975.26	0.0000	1975.26	0.00

Tendon5	Xi	Δf_{pF}	Δf_{pA}	Δf_{ES}	$\Sigma \Delta$	F_j^5	(α)	$F_j^5 \cdot \cos(\alpha)$	$F_j^5 \cdot \sin(\alpha)$
Section	(mm)	(MPa)	(MPa)	(MPa)	(MPa)	(kN)	(rad)	(kN)	(kN)
anchorage	0	0.00	143.51	10.73	154.24	1928.24	0.0874	1920.88	168.38
Support	300	0.69	140.89	10.75	152.34	1931.43	0.0861	1924.28	166.13
L/8	5188	12.29	98.32	19.86	130.46	1968.18	0.0647	1964.07	127.17
L/4	10075	30.66	55.74	25.96	112.36	1998.60	0.0431	1996.74	86.19
3L/8	14963	55.52	13.16	29.64	98.31	2022.19	0.0216	2021.72	43.64
L/2	19850	86.46	0.00	29.84	116.30	1991.97	0.0000	1991.97	0.00

Tendon6	Xi	Δf_{pF}	Δf_{pA}	Δf_{ES}	$\Sigma \Delta$	F_j^6	(α)	$F_j^6 \cdot \cos(\alpha)$	$F_j^6 \cdot \sin(\alpha)$
Section	(mm)	(MPa)	(MPa)	(MPa)	(MPa)	(kN)	(rad)	(kN)	(kN)
anchorage	0	0.00	143.51	0.00	143.51	1946.27	0.0212	1945.83	41.17
Support	300	0.36	140.89	0.00	141.25	1950.05	0.0208	1949.63	40.63
L/8	5188	6.35	98.32	0.00	104.66	2011.53	0.0156	2011.28	31.43
L/4	10075	13.98	55.74	0.00	69.72	2070.23	0.0104	2070.12	21.57
3L/8	14963	23.24	13.16	0.00	36.40	2126.21	0.0052	2126.18	11.08
L/2	19850	34.08	0.00	0.00	34.08	2130.10	0.0000	2130.10	0.00

SUM 1to6	Xi	ΣF_j	$F_j \cdot \cos(\alpha)$	$F_j \cdot \sin(\alpha)$	$e_{c\Delta p}$	$M_i = \Sigma F_j \cos(\alpha) \cdot e_{c\Delta p}$
Section	(mm)	(kN)	(kN)	(kN)	(mm)	(kNm)
anchorage	0	11322.28	11240.88	1217.94	53.61	602.67
Support	300	11347.82	11268.60	1202.88	53.61	604.16
L/8	5188	11619.34	11573.19	928.93	365.95	4235.22
L/4	10075	11854.54	11833.45	634.02	594.24	7031.95
3L/8	14963	11918.34	11913.08	317.12	733.38	8736.79
L/2	19850	11609.25	11609.25	0.00	779.76	9052.38

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

4.2.1 Loss of prestress due to Shrinkage:

Formula:

$$\Delta f_{pSH} = (93 - 0.85 \cdot H)$$

Relative humidity of environment

$$H = 80.00 \%$$

$$\Delta f_{pSH} = 25.00 \text{ (MPa)}$$

4.2.2 Loss of prestress due to Creep:

Formula

$$\Delta f_{pCR} = 12.0 f_{cgp} - 7.0 \cdot \Delta f_{c\Delta p}$$

In which:

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

$\Delta f_{c\Delta p}$ Stress at tendons centroid changes due to permanent load, except dead load action at transfer

Section	Xi (m)	Interior Girder			Exterior Girder	
		f_{cgp} (MPa)	$\Delta f_{c\Delta p}$ (MPa)	Δf_{pCR} (MPa)	$\Delta f_{c\Delta p}$ (MPa)	Δf_{pCR} (MPa)
Support	0.00	7.25	0.00	86.96	0.00	86.96
L/8	4.89	13.38	0.83	154.80	1.65	149.05
L/4	9.78	17.49	4.75	176.68	4.40	179.12
3L/8	14.66	19.97	5.10	203.91	6.61	193.38
L/2	19.55	20.11	8.07	184.77	7.44	189.18

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
 f_{py} : Specified yield strength of prestressing steel f_{py} = 1674.00 (MPa)
 f_{pj} : Initial stress in the tendon at the end of stressing

Section	Xi (m)	f _{pj} (MPa)	Δf _{pR1} (MPa)
Support	0.00	1284.08	0.00
L/8	4.89	1268.90	0.00
L/4	9.78	1258.73	0.00
3L/8	14.66	1252.60	0.00
L/2	19.55	1252.26	0.00

(b) After Transfer:

Formula:
$$\Delta f_{pR2} = 30\% \cdot (138 - 0.3 \Delta f_{pF} - 0.4 \Delta f_{pES} - 0.2 (\Delta f_{pSH} + \Delta f_{pCR}))$$

Interior Girder						
Section	Xi (m)	Δf _{pF} (MPa)	Δf _{pES} (MPa)	Δf _{pSH} (MPa)	Δf _{pCR} (MPa)	Δf _{pR2} (MPa)
Support	0.00	0.80	17.92	25.00	14.49	36.81
L/8	4.89	14.25	33.10	25.00	25.80	33.10
L/4	9.78	36.10	43.27	25.00	29.45	29.69
3L/8	14.66	65.83	49.40	25.00	33.99	26.01
L/2	19.55	102.75	49.74	25.00	30.80	22.84

Exterior Girder						
Section	Xi (m)	Δf _{pF} (MPa)	Δf _{pES} (MPa)	Δf _{pSH} (MPa)	Δf _{pCR} (MPa)	Δf _{pR2} (MPa)
Support	0.00	0.80	17.92	25.00	14.49	36.81
L/8	4.89	14.25	33.10	25.00	24.84	33.15
L/4	9.78	36.10	43.27	25.00	29.85	29.67
3L/8	14.66	65.83	49.40	25.00	32.23	26.11
L/2	19.55	102.75	49.74	25.00	31.53	22.79

TOTAL LOSS STRESS AT SERVICE STAGE

Interior Girder						
Section	Xi (m)	Δf _{pSH} (MPa)	Δf _{pCR} (MPa)	Δf _{pR1} (MPa)	Δf _{pR2} (MPa)	Sum (MPa)
Support	0.00	25.00	86.96	0.00	36.81	148.77
L/8	4.89	25.00	154.80	0.00	33.10	212.90
L/4	9.78	25.00	176.68	0.00	29.69	231.37
3L/8	14.66	25.00	203.91	0.00	26.01	254.92
L/2	19.55	25.00	184.77	0.00	22.84	232.61

Exterior Girder						
Section	Xi (m)	Δf _{pSH} (MPa)	Δf _{pCR} (MPa)	Δf _{pR1} (MPa)	Δf _{pR2} (MPa)	Sum (MPa)
Support	0.00	25.00	86.96	0.00	36.81	148.77
L/8	4.89	25.00	149.05	0.00	33.15	207.21
L/4	9.78	25.00	179.12	0.00	29.67	233.79
3L/8	14.66	25.00	193.38	0.00	26.11	244.49
L/2	19.55	25.00	189.18	0.00	22.79	236.97

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

Tendon1	Xi	$\Sigma \Delta_{PT}$	F_1^1	(α)	$F_j^1 \cdot \cos(\alpha)$	$F_j^1 \cdot \sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	358.83	1584.52	0.1825	1558.20	287.60
L/8	4.89	378.83	1550.93	0.1376	1536.28	212.67
L/4	9.78	362.31	1578.69	0.0920	1572.01	145.08
3L/8	14.66	406.73	1504.06	0.0461	1502.46	69.33
L/2	19.55	443.35	1442.54	0.0000	1442.54	0.00

Tendon2	Xi	$\Sigma \Delta_{PT}$	F_1^2	(α)	$F_j^2 \cdot \cos(\alpha)$	$F_j^2 \cdot \sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	343.29	1610.64	0.1530	1591.83	245.39
L/8	4.89	371.87	1562.62	0.1507	1544.91	234.56
L/4	9.78	361.72	1579.67	0.1134	1569.53	178.72
3L/8	14.66	389.32	1533.31	0.0758	1528.91	116.06
L/2	19.55	417.03	1486.75	0.0379	1485.68	56.39

Tendon3	Xi	$\Sigma \Delta_{PT}$	F_1^3	(α)	$F_j^3 \cdot \cos(\alpha)$	$F_j^3 \cdot \sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	354.44	1591.90	0.1185	1580.73	188.23
L/8	4.89	416.14	1488.24	0.0891	1482.34	132.39
L/4	9.78	429.98	1464.99	0.0595	1462.40	87.07
3L/8	14.66	446.27	1437.63	0.0298	1436.99	42.78
L/2	19.55	421.25	1479.66	0.0000	1479.66	0.00

Tendon4	Xi	$\Sigma \Delta_{PT}$	F_1^4	(α)	$F_j^4 \cdot \cos(\alpha)$	$F_j^4 \cdot \sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	306.58	1672.31	0.0861	1666.11	143.84
L/8	4.89	368.82	1567.74	0.0647	1564.47	101.30
L/4	9.78	368.46	1568.36	0.0431	1566.90	67.64
3L/8	14.66	375.93	1555.79	0.0216	1555.43	33.57
L/2	19.55	340.80	1614.82	0.0000	1614.82	0.00

Tendon5	Xi	$\Sigma \Delta_{PT}$	F_1^5	(α)	$F_j^5 \cdot \cos(\alpha)$	$F_j^5 \cdot \sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	303.00	1678.31	0.0874	1671.90	146.55
L/8	4.89	365.23	1573.77	0.0861	1567.93	135.36
L/4	9.78	361.83	1579.48	0.0647	1576.18	102.06
3L/8	14.66	367.28	1570.33	0.0431	1568.87	67.72
L/2	19.55	330.92	1631.41	0.0216	1631.03	35.20

Tendon6	Xi	$\Sigma \Delta_{PT}$	F_1^6	(α)	$F_j^6 \cdot \cos(\alpha)$	$F_j^6 \cdot \sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	292.27	1696.34	0.0208	1695.97	35.34
L/8	4.89	354.15	1592.38	0.0156	1592.19	24.88
L/4	9.78	336.03	1622.83	0.0104	1622.74	16.91
3L/8	14.66	324.64	1641.97	0.0052	1641.95	8.55
L/2	19.55	269.01	1735.43	0.0000	1735.43	0.00

SUM 1to6	Xi	ΣF_j	$F_j \cdot \cos(\alpha)$	$\sum p = F_j \cdot \sin(\alpha)$	e_{cable}	$M_j = \Sigma F_j \cdot \cos(\alpha) \cdot e_{cable}$
Section	(mm)	(kN)	(kN)	(kN)	(m)	(kNm)
Support	0.00	9834.01	9764.75	1046.96	0.05	523.5
L/8	4.89	9335.69	9288.13	841.16	0.37	3399.0
L/4	9.78	9394.01	9369.75	597.47	0.59	5567.9
3L/8	14.66	9243.09	9234.61	338.02	0.73	6772.5
L/2	19.55	9390.60	9389.16	91.59	0.78	7321.3

Exterior Girder

Tendon1	Xi	$\Sigma \Delta_{PT}$	F_1^1	(α)	$F_1^1 \cdot \cos(\alpha)$	$F_1^1 \cdot \sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	358.83	1584.52	0.1825	1558.20	287.60
L/8	4.89	373.13	1560.49	0.1376	1545.75	213.98
L/4	9.78	384.73	1574.62	0.0920	1567.96	144.70
3L/8	14.66	396.30	1521.57	0.0461	1519.96	70.14
L/2	19.55	447.71	1435.20	0.0000	1435.20	0.00

Tendon2	Xi	$\Sigma \Delta_{PT}$	F_2^2	(α)	$F_2^2 \cdot \cos(\alpha)$	$F_2^2 \cdot \sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	343.29	1610.64	0.1530	1591.83	245.39
L/8	4.89	366.18	1572.18	0.1507	1554.37	235.99
L/4	9.78	364.14	1575.61	0.1134	1565.49	178.26
3L/8	14.66	378.89	1550.82	0.0758	1546.37	117.39
L/2	19.55	421.40	1479.41	0.0379	1478.35	56.11

Tendon3	Xi	$\Sigma \Delta_{PT}$	F_3^3	(α)	$F_3^3 \cdot \cos(\alpha)$	$F_3^3 \cdot \sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	354.44	1591.90	0.1185	1580.73	188.23
L/8	4.89	410.45	1497.80	0.0891	1491.86	133.24
L/4	9.78	432.40	1460.93	0.0595	1458.34	86.83
3L/8	14.66	435.84	1455.14	0.0298	1454.50	43.30
L/2	19.55	425.61	1472.33	0.0000	1472.33	0.00

Tendon4	Xi	$\Sigma \Delta_{PT}$	F_4^4	(α)	$F_4^4 \cdot \cos(\alpha)$	$F_4^4 \cdot \sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	306.58	1672.31	0.0861	1668.11	143.84
L/8	4.89	363.13	1577.31	0.0647	1574.01	101.92
L/4	9.78	370.88	1564.29	0.0431	1562.83	67.46
3L/8	14.66	365.51	1573.31	0.0216	1572.94	33.95
L/2	19.55	345.17	1607.48	0.0000	1607.48	0.00

Tendon5	Xi	$\Sigma \Delta_{PT}$	F_5^5	(α)	$F_5^5 \cdot \cos(\alpha)$	$F_5^5 \cdot \sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	303.00	1678.31	0.0874	1671.90	146.55
L/8	4.89	359.54	1583.33	0.0861	1577.46	136.19
L/4	9.78	364.25	1575.41	0.0647	1572.12	101.79
3L/8	14.66	356.85	1587.85	0.0431	1586.37	68.48
L/2	19.55	335.29	1624.08	0.0216	1623.70	35.04

Tendon6	Xi	$\Sigma \Delta_{PT}$	F_6^6	(α)	$F_6^6 \cdot \cos(\alpha)$	$F_6^6 \cdot \sin(\alpha)$
Section	(mm)	(MPa)	(kN)	(rad)	(kN)	(kN)
Support	0.00	292.27	1696.34	0.0208	1695.97	35.34
L/8	4.89	348.46	1601.94	0.0156	1601.75	25.03
L/4	9.78	338.45	1618.76	0.0104	1618.67	16.87
3L/8	14.66	314.21	1659.48	0.0052	1659.46	8.65
L/2	19.55	273.37	1728.10	0.0000	1728.10	0.00

SUM 1to6	Xi	ΣF_i	$F_i^i \cdot \cos(\alpha)$	$\sum F_i^i \cdot \sin(\alpha)$	e_{caple}	$M_i = \Sigma F_i \cdot \cos(\alpha) \cdot e_{cap}$
Section	(mm)	(kN)	(kN)	(kN)	(m)	(kNm)
Support	0.00	9834.01	9764.75	1046.96	0.05	523.5
L/8	4.89	9393.05	9345.20	846.35	0.37	3419.9
L/4	9.78	9369.61	9345.42	595.91	0.59	5553.5
3L/8	14.66	9348.17	9339.60	341.90	0.73	6849.5
L/2	19.55	9346.60	9345.15	91.16	0.78	7286.9

5. FIBRE STRESS CHECK:

Formula:

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

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

Concrete strength at transfer $f_{ci}' = 0.9 f_c = 40.50 \text{ MPa}$

Compression stress Limit at transfer $0.6 f_{ci}' = 24.30 \text{ MPa}$

Tension stress Limit at transfer $0.25 \text{ SQRT}(f_{ci}') < 1.38 = -1.38 \text{ MPa} \quad (5.9.4.1.2-1)$

Setion	Xi	A	St	Sb	F _i *Cos(α)	e	M _{DC}	f _{ti}	f _{bi}	Kiểm tra	
	(mm)	(mm ²)	(mm ³)	(mm ³)	(kN)	(mm)	(kNm)	(MPa)	(MPa)	f _{ti}	f _{bi}
Girder end	0	1.60E+06	5.80E+08	5.63E+08	11240.88	53.61	0.00	5.98	8.09	OK	OK
Support	300	1.60E+06	5.80E+08	5.63E+08	11268.60	53.61	0.00	5.99	8.11	OK	OK
L/8	5188	1.01E+06	4.70E+08	4.67E+08	11573.19	365.95	2194.47	7.07	15.77	OK	OK
L/4	10075	9.37E+05	4.54E+08	4.55E+08	11833.45	594.24	3761.95	5.44	19.82	OK	OK
3L/8	14963	9.37E+05	4.54E+08	4.55E+08	11913.08	733.38	4702.43	3.84	21.59	OK	OK
L/2	19850	9.37E+05	4.54E+08	4.55E+08	11609.25	779.76	5015.93	3.51	21.27	OK	OK

5.2 Stress check during contruction the deck:

5.2.1 Increase load:

Exterior Diaphragms beam	DC _{dn1} =	95.90 (kN)
Interior Diaphragms beam	DC _{dn1} =	68.82 (kN)
Precast plank	DC _{VK} =	3.46 (kN/m)
Wet concrete of deck	DC _{mc} =	13.33 (kN/m)

5.2.2 Stress check:

Compression strength of concrete	f _c =	45.00 MPa
Compression stress limit	0.45 f _c =	20.25 MPa (5.9.4.2.1-1)
Tension stress limit	0.5 SQRT(f _c) =	-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)	(mm ²)	(mm ³)	(mm ³)	(kN)	(mm)	(kNm)	(MPa)	(MPa)	f _{ti}	f _{bi}
Girder end	0	1.60E+06	5.80E+08	5.63E+08	11240.88	53.61	0.00	5.98	8.09	OK	OK
Support	300	1.60E+06	5.80E+08	5.63E+08	11268.60	53.61	0.00	5.99	8.11	OK	OK
L/8	5188	1.01E+06	4.70E+08	4.67E+08	11573.19	365.95	4859.30	12.73	10.07	OK	OK
L/4	10075	9.37E+05	4.54E+08	4.55E+08	11833.45	594.24	6419.88	11.28	13.98	OK	OK
3L/8	14963	9.37E+05	4.54E+08	4.55E+08	11913.08	733.38	8024.85	11.15	14.28	OK	OK
L/2	19850	9.37E+05	4.54E+08	4.55E+08	11609.25	779.76	8559.84	11.31	13.47	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 f _c =	20.25 MPa (5.9.4.2.1-1)
Tension Stress Limit:	- 0.5*SQRT(f _c) =	-3.35 MPa

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

Interior Girder

Setion	Xi	A	S _i	S _{ig}	P _{pe}	P _{pe} *e _c	M _g + M _s	M _{SDL}	f _{ti}	Check
	(mm)	(mm ²)	(mm ³)	(mm ³)	(kN)	(kNm)	(kNm)	(kNm)	(MPa)	f _{ti}
Support	300	1.60E+06	5.80E+08	1.40E+09	9764.75	523.53	0.00	0.00	5.193	OK
L/8	5188	1.01E+06	4.70E+08	1.37E+09	9288.13	3399.00	3744.93	767.64	10.451	OK
L/4	10075	9.37E+05	4.54E+08	1.37E+09	9369.75	5567.91	6419.88	1315.95	12.836	OK
3L/8	14963	9.37E+05	4.54E+08	1.37E+09	9234.61	6772.46	8024.85	1644.93	13.813	OK
L/2	19850	9.37E+05	4.54E+08	1.37E+09	9389.16	7321.25	8559.84	1754.60	14.028	OK

Exterior Girder

Setion	Xi	A	S _i	S _{ig}	P _{pe}	P _{pe} *e _c	M _g + M _s	M _{SDL}	f _{ti}	Check
	(mm)	(mm ²)	(mm ³)	(mm ³)	(kN)	(kNm)	(kNm)	(kNm)	(MPa)	f _{ti}
Support	300	1.60E+06	5.8E+08	1.4E+09	9764.75	523.53	0.00	0.00	5.193	OK
L/8	5188	1.01E+06	4.7E+08	1.37E+09	9345.20	3419.88	3526.88	829.50	10.045	OK
L/4	10075	9.37E+05	4.54E+08	1.37E+09	9345.42	5553.45	6046.08	1422.01	12.097	OK
3L/8	14963	9.37E+05	4.54E+08	1.37E+09	9339.60	6849.45	7557.61	1777.51	12.825	OK
L/2	19850	9.37E+05	4.54E+08	1.37E+09	9345.15	7286.94	8061.45	1896.01	13.063	OK

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

Compression Stress Limit: $0.40 f_c = 18.00 \text{ MPa}$ (5.9.4.2.1-1)

Tension Stress Limit: $-0.5 \cdot \text{SQRT}(f_c) = -3.35 \text{ MPa}$

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

Interior Girder

Setion	Xi (mm)	A (mm ²)	S _t (mm ³)	S _{ig} (mm ³)	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	1.60E+06	5.80E+08	1.40E+09	9764.75	523.53	0.00	0.00	0.00	2.597	OK
L/8	5188	1.01E+06	4.70E+08	1.37E+09	9288.13	3399.00	3744.93	767.64	1729.13	6.489	OK
L/4	10075	9.37E+05	4.54E+08	1.37E+09	9369.75	5567.91	6419.88	1315.95	2934.11	8.560	OK
3L/8	14963	9.37E+05	4.54E+08	1.37E+09	9234.61	6772.46	8024.85	1644.93	3632.11	9.558	OK
L/2	19850	9.37E+05	4.54E+08	1.37E+09	9389.16	7321.25	8559.84	1754.60	3840.27	9.817	OK

Exterior Girder

Setion	Xi (mm)	A (mm ²)	S _t (mm ³)	S _{ig} (mm ³)	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	1601984.6	5.8E+08	1.4E+09	9764.75	523.53	0.00	0.00	0.00	2.597	OK
L/8	5188	1014590.0	4.7E+08	1.37E+09	9345.20	3419.88	3526.88	829.50	2035.79	6.510	OK
L/4	10075	936850.0	4.54E+08	1.37E+09	9345.42	5553.45	6046.08	1422.01	3454.48	8.570	OK
3L/8	14963	936850.0	4.54E+08	1.37E+09	9339.60	6849.45	7557.61	1777.51	4276.27	9.533	OK
L/2	19850	936850.0	4.54E+08	1.37E+09	9345.15	7286.94	8061.45	1896.01	4521.35	9.832	OK

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

Compression Stress Limit: $0.60 f_c = 27.00 \text{ MPa}$ (5.9.4.2.1-1)

Tension Stress Limit: $-0.5 \cdot \text{SQRT}(f_c) = -3.35 \text{ MPa}$

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

Interior Girder

Setion	Xi (mm)	A (mm ²)	S _t (mm ³)	S _{ig} (mm ³)	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	1.60E+06	5.80E+08	1.40E+09	9764.75	523.53	0.00	0.00	0.00	5.193	OK
L/8	5188	1.01E+06	4.70E+08	1.37E+09	9288.13	3399.00	3744.93	767.64	1729.13	11.715	OK
L/4	10075	9.37E+05	4.54E+08	1.37E+09	9369.75	5567.91	6419.88	1315.95	2934.11	14.978	OK
3L/8	14963	9.37E+05	4.54E+08	1.37E+09	9234.61	6772.46	8024.85	1644.93	3632.11	16.464	OK
L/2	19850	9.37E+05	4.54E+08	1.37E+09	9389.16	7321.25	8559.84	1754.60	3840.27	16.831	OK

Exterior Girder

Setion	Xi (mm)	A (mm ²)	S _t (mm ³)	S _{ig} (mm ³)	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	1.60E+06	5.80E+08	1.40E+09	9764.75	523.53	0.00	0.00	0.00	5.193	OK
L/8	5188	1.01E+06	4.70E+08	1.37E+09	9345.20	3419.88	3526.88	829.50	2035.79	11.532	OK
L/4	10075	9.37E+05	4.54E+08	1.37E+09	9345.42	5553.45	6046.08	1422.01	3454.48	14.619	OK
3L/8	14963	9.37E+05	4.54E+08	1.37E+09	9339.60	6849.45	7557.61	1777.51	4276.27	15.946	OK
L/2	19850	9.37E+05	4.54E+08	1.37E+09	9345.15	7286.94	8061.45	1896.01	4521.35	16.363	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: $0.45 f_c = 15.75 \text{ MPa}$ (5.9.4.2.1-1)

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

Setion	Xi (mm)	MSDL (kNm)		S _{tc} (mm ³)		f _t (MPa)		Check	
		in. Girder	Ex. Girder	in. Girder	Ex. Girder	in. Girder	Ex. Girder	in. Girder	Ex. Girder
Support	300.00	0.00	0.00	9.6E+08	9.57E+08	0.000	0.000	OK	OK
L/8	5187.50	767.64	829.50	9.1E+08	9.06E+08	0.848	0.916	OK	OK
L/4	10075.00	1315.95	1422.01	9E+08	9.01E+08	1.461	1.579	OK	OK
3L/8	14962.50	1644.93	1777.51	9E+08	9.01E+08	1.826	1.973	OK	OK
L/2	19850.00	1754.60	1896.01	9E+08	9.01E+08	1.948	2.105	OK	OK

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

Compression Stress Limit: $0.6 f_c = 21.00 \text{ MPa}$ (5.9.4.2.1-1)

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

Setion	Xi	MSDL + MLL (kNm)		S _{ic} (mm ³)		f _i (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	9.6E+08	9.57E+08	0.000	0.000	OK	OK
L/8	5187.50	2496.76	2865.30	9.1E+08	9.06E+08	2.757	3.164	OK	OK
L/4	10075.00	4250.06	4876.49	9E+08	9.01E+08	4.718	5.414	OK	OK
3L/8	14962.50	5277.04	6053.78	9E+08	9.01E+08	5.858	6.721	OK	OK
L/2	19850.00	5594.87	6417.36	9E+08	9.01E+08	6.211	7.124	OK	OK

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

Compression Stress Limit: $0.45 f_c = 20.25 \text{ MPa}$ (5.9.4.2.1-1)

Tension Stress Limit: $-0.5 \cdot \text{SQRT}(f_c) = -3.35 \text{ MPa}$ (5.9.4.2.1-1)

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

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)	(mm ²)	(mm ³)	(mm ³)	(kN)	(kNm)	(kNm)	(kNm)	(kNm)	(MPa)	f _b
Support	300	1.60E+06	5.63E+08	8.09E+08	9764.75	523.53	0.00	0.00	0.00	7.025	OK
L/8	5188	1.01E+06	4.67E+08	6.47E+08	9288.13	3399.00	3744.93	767.64	1729.13	5.091	OK
L/4	10075	9.37E+05	4.55E+08	6.23E+08	9369.75	5567.91	6419.88	1315.95	2934.11	2.250	OK
3L/8	14963	9.37E+05	4.55E+08	6.23E+08	9234.61	6772.46	8024.85	1644.93	3632.11	-0.199	OK
L/2	19850	9.37E+05	4.55E+08	6.23E+08	9389.16	7321.25	8559.84	1754.60	3840.27	-0.446	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)	(mm ²)	(mm ³)	(mm ³)	(kN)	(kNm)	(kNm)	(kNm)	(kNm)	(MPa)	f _b
Support	300	1.60E+06	5.63E+08	8.09E+08	9764.75	523.53	0.00	0.00	0.00	7.025	OK
L/8	5188	1.01E+06	4.67E+08	6.47E+08	9345.20	3419.88	3526.88	829.50	2035.79	5.184	OK
L/4	10075	9.37E+05	4.55E+08	6.23E+08	9345.42	5553.45	6046.08	1422.01	3454.48	2.176	OK
3L/8	14963	9.37E+05	4.55E+08	6.23E+08	9339.60	6849.45	7557.61	1777.51	4276.27	0.071	OK
L/2	19850	9.37E+05	4.55E+08	6.23E+08	9345.15	7286.94	8061.45	1896.01	4521.35	-0.574	OK

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

Compression Stress Limit: $0.45 f_c = 20.25 \text{ MPa}$ (5.9.4.2.1-1)

Tension Stress Limit: $-0.5 \cdot \text{SQRT}(f_c) = -3.35 \text{ MPa}$ (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)	(mm ²)	(mm ³)	(mm ³)	(kN)	(kNm)	(kNm)	(kNm)	(kNm)	(MPa)	f _b
Support	300	1.60E+06	5.63E+08	8.09E+08	9764.75	523.53	0.00	0.00	0.00	7.025	OK
L/8	5188	1.01E+06	4.67E+08	6.47E+08	9288.13	3399.00	3744.93	767.64	1729.13	4.557	OK
L/4	10075	9.37E+05	4.55E+08	6.23E+08	9369.75	5567.91	6419.88	1315.95	2934.11	1.309	OK
3L/8	14963	9.37E+05	4.55E+08	6.23E+08	9234.61	6772.46	8024.85	1644.93	3632.11	-1.364	OK
L/2	19850	9.37E+05	4.55E+08	6.23E+08	9389.16	7321.25	8559.84	1754.60	3840.27	-1.679	OK

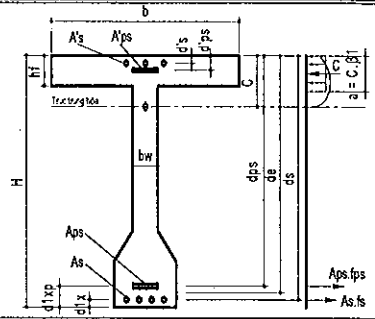
Exterior Girder

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)	(mm ²)	(mm ³)	(mm ³)	(kN)	(kNm)	(kNm)	(kNm)	(kNm)	(MPa)	f _b
Support	300	1.60E+06	5.63E+08	8.09E+08	9764.75	523.53	0.00	0.00	0.00	7.025	OK
L/8	5188	1.01E+06	4.67E+08	6.47E+08	9345.20	3419.88	3526.88	829.50	2035.79	4.555	OK
L/4	10075	9.37E+05	4.55E+08	6.23E+08	9345.42	5553.45	6046.08	1422.01	3454.48	1.068	OK
3L/8	14963	9.37E+05	4.55E+08	6.23E+08	9339.60	6849.45	7557.61	1777.51	4276.27	-1.301	OK
L/2	19850	9.37E+05	4.55E+08	6.23E+08	9345.15	7286.94	8061.45	1896.01	4521.35	-2.025	OK

REINFORCEMENT OF GIRDER CHECKING - STRENGTH LOAD COMBINATION

MATERIALS

NORMAL CONCRETE			
fc	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							
Qu	Combination Shear	kN	Strength 2232	Strength 1746	Strength 1269	Strength 801	Strength 342
Mu	Flexural Moment	kNm	0	9101	15540	19351	20571
Nu	Axial load	kN					
Tu	Torsional Moment	kNm					
6.1 FLEXURAL MOMENT CHECKING							
H	Section height	m	2.300	2.300	2.300	2.300	2.300
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	2.238	2.238	2.238	2.238	2.238
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	1.012	0.687	0.456	0.316	0.270
dps	Dis. From comp. fiber to centroid of tension prestressing steel	m	1.288	1.613	1.844	1.984	2.030
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	0.750	0.352	0.300	0.300	0.300
hf	Compression flange depth	m	0.200	0.200	0.200	0.200	0.200
Iz	Moment of inertia of section	m4	1.069	0.916	0.893	0.893	0.893
Amc	Section area	m2	2.068	1.480	1.402	1.402	1.402
	Steel choice						
Aps	Tension prestressing steel	P.S type	12 T15.2	12 T15.2	12 T15.2	12 T15.2	12 T15.2
		Number	tendons 6	6	6	6	6
		Area	m2 0.01008	0.01008	0.01008	0.01008	0.01008
	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
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
	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
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	0.428	0.649	0.731	0.744	0.748
	For T section behavior	m	0.428	0.649	0.731	0.744	0.748
	For rectangular section behavior	m	0.285	0.289	0.291	0.292	0.292
fpe	Effective stress in the prestressing steel after losses	Mpa	1585	1488	1465	1438	1443
fps	Aver. stress in pres. steel at the time for which the nominal resistance	Mpa	1687	1650	1654	1665	1668
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	0.312	0.473	0.532	0.542	0.545
de	Corresp. effective depth from extreme comp. fiber to centroid of tensile force in the tensile reinf.	m	1.314	1.630	1.856	1.991	2.036
Mn	Nominal resistance	kNm	20740	25338	29106	31585	32415
Mr	Factored resistance	kNm	18666	22804	26195	28427	29173
Mu	Flexual moment	kNm	0	9101	15540	19351	20571
(5.7.3.2)	Flexural moment Checking		OK	OK	OK	OK	OK
	Limits for reinforcement						
c/de	Maximum reinforcement		0.33	0.40	0.39	0.37	0.37
	Maximum reinforcement Checking	<= 0.42	OK	OK	OK	OK	OK
r min	Minimum reinforcement		0.06%	0.08%	0.09%	0.09%	0.09%
	Minimum reinforcement Checking for RC	0.34%	N.a	N.a	N.a	N.a	N.a
1.2*Mcr	Craking moment	kNm	2896	2814	2886	2911	2918
(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	2433	8010	11723	12874	12848
	Checking As.fy+Aps.fps >= F _{yc}		Ok	Ok	Ok	Ok	Ok

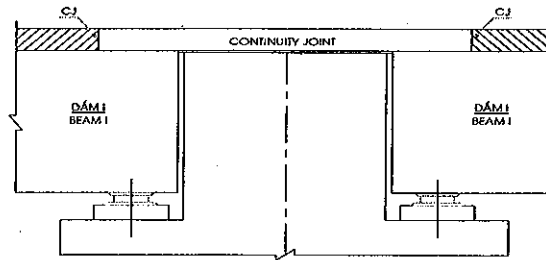
(5.7.3.4)	Control of cracking by distr. of reinf. for RC member- Check?		No	No	No	No	No
dc	Existing condition for structure	1,2 or 3	1	1	1	1	1
Z	Concr. thickness fro. Tens. fiber to tens. reinf. nearest	m	0.058	0.058	0.058	0.058	0.058
A	Crack width parameter	N/mm	30000	30000	30000	30000	30000
fsa	Area of concr. with same centroid as tens. Reinf	m ²	0.043	0.043	0.043	0.043	0.043
0.6*fy	Value	Mpa	220	220	220	220	220
x	Tensile stress in reinf. Min(fs,0.6fy)	Mpa	240	240	240	240	240
J.d	Dist. From compression fiber to centroid	m	220	220	220	220	220
lcr	Arm	m	-	-	-	-	-
fs	Moment of inertia of the cracked section	m ⁴	-	-	-	-	-
	Tensile stress in reinforcement fs = Ms/s / (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 Reinforcement (side distribution)						
Areq	Area of required reinf	m ²	0.00043	0.00031	0.00029	0.00029	0.00029
	Distribution on sides 10 D12	m ²	0.00113	0.00113	0.00113	0.00113	0.00113
	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	0.750	0.352	0.300	0.300	0.300
dv	Effective shear depth	m	1.656	1.656	1.670	1.792	1.832
	(de - a/2)	m	1.158	1.394	1.589	1.720	1.763
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	m ²	0.0004	0.0004	0.0004	0.0004	0.0004
β	Assume		6.8	6.8	6.8	6.8	6.8
θ	Assume	degree	27.00	27.00	27.00	27.00	27.00
v	Shear stress in concrete	kN/m ²	1997	3328	2815	1657	692
fpo	Parameter taken as modulus of elasticity of prestressing tendons	Mpa	1629	1547	1526	1498	1503
ε _s	Strain in tensile reinforcement		-6.39E-03	-3.76E-03	-2.17E-03	-1.57E-03	-1.61E-03
	if ε _s <0, multiple with reduce factor		-4.52E-04	-5.26E-04	-3.47E-04	-2.52E-04	-2.57E-04
	Strain checking	≤2.00E-3	Ok	Ok	Ok	Ok	Ok
v/fc	Ratio of shear stress and fc		0.044	0.074	0.063	0.037	0.015
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	4689	2200	1891	2029	2075
Vs	Shear resistance provided by shear reinforcement	kN	3501	3501	1766	1894	1937
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	8190	5702	3657	3923	4012
Vn2	Vn2	kN	13973	6558	5636	6047	6184
Vn	Nominal shear resistance Vn=min(Vn1,Vn2)	kN	8190	5702	3657	3923	4012
Vr	Factored shear resistance	kN	7371	5132	3291	3531	3611
Vu	Shear	kN	2232	1746	1269	801	342
(5.8.2.7)	Shear checking		OK	OK	OK	OK	OK
	Region requiring transverse reinf Checking		Need	Need	Need	No need	No need
	Minimum shear reinf area	m ²	0.0002	0.0001	0.0001	0.0001	0.0001
	Minimum shear reinforcement Checking		OK	OK	OK	-	-
	0.1*fc*bv*dv	kN	5589	2623	2255	2419	2474
	Smax	m	0.60	0.60	0.60	0.60	0.60
	Maximum spacing Smax		OK	OK	OK	-	-

CALCULATION SHEET
LINK SLAB

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	TYPICAL CACULATION	Design			
	DETAIL DESIGN	Check			
	CONTINUITY JOINT	Revise			

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I GENERAL DATA



1.1.Dimension data

LEFT SPAN				RIGHT SPAN			
ITEM	SYMBOL	VALUES	UNITS	ITEM	SYMBOL	VALUES	UNITS
Length of left span	Ll	33	m	Length of right span	Lr	33	m
Caculation length	Ltl	32.1	m	Caculation length	Ltr	32.1	m
Depth of pavement	t	0.084	m	Depth of pavement	t	0.084	m
Depth of deck slab	d	0.22	m	Depth of deck slab	d	0.22	m
Bridge width	B	12.74	m	Bridge with	B	12.74	m
Lane width	B1	11.75	m	Lane with	B1	11.75	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

$f_c' = 35$ Mpa

Modulus of elasticity

$E_c = 31799$ Mpa

Modulus of Rupture

$f_r = 3.73$ Mpa

Unit weight of concrete

$g_c = 2500$ kg/m³

Compressive Strength at 28 days of beam concrete

$f_c = 40$ Mpa

1.3.Self weight of superstructure

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	603.44	KN	Weight of girder	Gdc	603.44	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	103.1747	KN/m	Dead load	DC	103.1747	KN/m
Live load	LL	57.60	KN/m	Live load	LL	57.60	KN/m
Uniform load	p	160.78	KN/m	Uniform load	p	160.78	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	m ⁴	Equivalent moment of inertia	J	2.19	m ⁴
Angular displacement	ϕ	0.002980408	rad	Angular displacement	ϕ	0.00298041	rad

Calculated length of continuity joint

$L_b = 1.803$ m

Reduction factor of rigidity

$k = 0.8$

Equivalent moment of inertia

$J_b = 0.01130463$ m⁴

Modulus of elasticity

$E_b = 31799$ Mpa

Bending moment of restrained section

$M_n = -950.76$ KN.m

Shear force at restrained section

$Q_n = 0$ KN

Bending moment of middle section

$M_g = 0$ KN.m

2.2.Internal forces due to slope of bridge

Weight of superstructure

$W = 7685.0298$ KN

Longitudinal grade

$i = 2.000\%$

Number of superstructure

$n = 1$

$N_d = 153.700596$ KN

2.3.Internal forces due to live loads

Dynamic Load Allowance

$I+IM = 1.75$

Weight of wheel load

$P = 369.75$ KN

Uniform load of live load

$q_{LL} = 23.715$ KN/m

Bending moment of restrained section

$M_n = -124.37466$ KN.m

Shear force at restrained section

$Q = 340.634508$ KN

Bending moment of middle section

$M_g = 124.374662$ KN.m

Shear force at middle section

$Q_g = 323.53125$ KN

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	DETAIL DESIGN	Check			
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2.4. Internal forces due to self weight

Uniform load	qS	68.6686 KN/m
Bending moment of restrained section	Mn	-14.88 KN.m
Shear force at restrained section	Qn	49.52 KN
Bending moment of middle section	Mg	11.16 KN.m

2.5. Internal forces due to pavement and parapet

Uniform load	qPP	35.9075 KN/m
Bending moment of restrained section	Mn	-7.78 KN.m
Shear force at restrained section	Qn	25.90 KN
Bending moment of middle section	Mg	5.84 KN.m

2.6. Internal forces due to Load of Temperature

Area of bearing	Fb	0.175 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 _{apdat}	25.0 °C
Maximal temperature	t _{max}	47.0 °C
Minimal temperature	t _{min}	10.0 °C
Compressive force	N =	76.64 kN
Tensile force	N =	112.41 kN
Number of bearing that accomodate 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	950.8	0.0	0.0	0.0		
Slope of bridge	0.0	0.0	0.0	0.0	153.7	
Live load	124.4	340.6	124.4	323.5		
Self weight of continuity joint	14.9	49.5	11.2	0.0		
Load of pavement and parapet	7.8	25.9	5.8	0.0		
Temperature					76.6	112.4
Total	1097.8	416.1	124.4	323.5	230.3	112.4

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

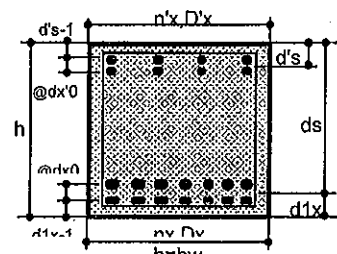
LOAD COMBINATIONS	RESTRAINED SECTION		MIDDLE SECTION		COMP. FORCE	TENSILE FORCE
	M (KN.m)	Q (KN)	M (KN.m)	Q (KN)	KN	KN
Strength Str-IA	1436.4	696.9	240.4	566.2	230.4	56.2
Service I	1097.8	416.1	141.4	323.5	230.3	112.4

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	TYPICAL CALCULATION		Design			
	DETAIL DESIGN		Check			
	CHECK SECTION OF CONTINUITY JOINT		Revise			

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REINFORCEMENTS CHECKING

Materials				
Normal concrete				
f_c	Compressive Strength of concrete at 28 days	Mpa	35	
E_c	Modulus of Elasticity	Mpa	29910	
f_r	Modulus of Rupture	Mpa	3.7	
γ_c	Unit weight of concrete	kN/m ³	24.5	
Reinforcement				
f_y	Yield strength	Mpa	400	
E_s	Modulus of Elasticity	Mpa	200000	
n_c	Ratio E_s/E_c		7	



			Restrained section		Middle section			
Internal forces at section								
	Combination		Strength	Service	Strength	Service		
Qu	Shear	kN	697	416	566	324		
Mu	Flexual Moment	kNm	1436	1098	240	141		
Nu	Axial load	kN	230	230	230	230		
Tu	Torsional Moment	kNm						
flexural Moment checking								
H	Section height	m	0.220	0.220	0.220	0.220		
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	0.162	0.162	0.162	0.162		
Rn	Mu / (j*ds ²)	kN/m2	60812.8	41830.3	10176.4	5386.9		
b	Width of section	m	12.740	12.740	12.740	12.740		
A _{mc}	Section area	m2	2.803	2.803	2.803	2.803		
	Chose							
A _s	Tension Reinforcement	Number	bars	83	83	83	83	
		Diameter	mm	25	25	25	25	
		Area	m2	0.04075	0.04075	0.04075	0.04075	
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	21	21	21	21	
		Diameter	mm	12	12	12	12	
		Area	m2	0.00238	0.00238	0.00238	0.00238	
j	Resistance factors for flexure	5.5.4.2	0.90	1.00	0.90	1.00		
jv	Resistance factors for shear		0.90	0.90	0.90	0.90		
bl	Stress block factor		0.800	0.800	0.800	0.800		
c	Dis. Between centroid and top fiber	m	0.054	0.054	0.054	0.054		
a	Depth of equivalent stress block	m	0.043	0.043	0.043	0.043		
de	Corresp. effective depth from extreme comp. fiber to centroid of tensile force in the tensile reinf.	m	0.162	0.162	0.162	0.162		
M _n	Nominal resistance	kNm	2290	2290	2290	2290		
M _r	Factored reistance	kNm	2061	2290	2061	2290		
M _u	Flexual moment	kNm	1436	1098	240	141		
(5.7.3.2)	Flexural moment Checking		OK	OK	OK	OK		
	Limits for reinforcement	m						
c/de	Maximum reinforcement		0.33	0.33	0.33	0.33		
	Maximum reinforcement Checking	<= 0.42	OK	OK	OK	OK		
r min	Minimum reinforcement		1.45%	1.45%	1.45%	1.45%		
	Minimum reinforcement Checking	0.26%	OK	OK	OK	OK		
1.2*M _{cr}	Craking moment	kNm	460	460	460	460		
(5.7.3.3.2)	Checking M _r >=min(1.2M _{cr} ,1.33M _u)		OK	OK	OK	OK		

DA NANG - QUANG NGAI EXPRESSWAY PROJECT			Item.	Eng.	Date.	Sign.
TYPICAL CALCULATION			Design			
DETAIL DESIGN			Check			
CHECK SECTION OF CONTINUITY JOINT			Revise			

(5.7.3.4)	Control of cracking by distr. of reinf - check?		No	Yes	No	Yes
	Existing condition for structure	1,2 or 3	1	1	1	1
dc	Concr. thickness fro. Tens. fiber to tens. reinf 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	m ²	0.015	0.015	0.015	0.015
f _{sa}	Value	Mpa	328	328	328	328
0.6*f _y		Mpa	240	240	240	240
	Tensile stress in reinf Min(f _{sa} , 0.6f _y)	Mpa	240	240	240	240
x	Depth of compr. area - Assu. A's in compr. Area so assumption is:	m	-	0.066	-	0.066
			-	x<d's	-	x<d's
I _{cr}	Moment of inertia of the cracked section	m ⁴	-	0.004	-	0.004
f _s	So tensile stress in reinf	Mpa	-	153	-	-15
	Checking for control cracking f _s <f _{sa}		N.a	OK	N.a	OK
(5.10.8.2)	Shrinkage and temperature Reinforcement					
A _{req}	Area of required reinf	m ²	0.00023	0.00023	0.00023	0.00023
	Distribute reinf 7 D16	m ²	0.00141	0.00141	0.00141	0.00141
	Required Spacing not larger than	m	0.45	0.45	0.45	0.45
	Checking		OK	OK	OK	OK

Shear and torsion checking						
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
b _v	Effective web width as minimum web width - in d _v	m	12.740	12.740	12.740	12.740
d _v	Effective shear depth	m	0.158	0.158	0.158	0.158
s	Spacing of stirrups	m	0.600	0.600	0.600	0.600
n _c /4	Amount of bars in spacing S	bars	21	21	21	21
A _v	Shear reinf area in spacing S	m ²	0.0024	0.0024	0.0024	0.0024
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/m ²	384	229	312	178
e _s	Strain in tensile reinforcement		1.14E-03	8.62E-04	2.07E-04	1.15E-04
	if e _s <0, multiple with reduce factor		-	-	-	-
	Strain checking	<=2.00E-3	Ok	Ok	Ok	Ok
v/f _c	Ratio of shear stress and f _c		0.011	0.007	0.009	0.005
b	Final value		2.0	2.0	2.0	2.0
q	Final value	degree	45.00	45.00	45.00	45.00
V _c	Nominal shear resistance in the concrete	kN	1982	1982	1982	1982
V _s	Shear resistance prov. by shear reinforcement	kN	251	251	251	251
V _p	Component in direct. of applied shear of prestres.	kN	0	0	0	0
V _{n1}	V _{n1} =V _c +V _s +V _p	kN	2233	2233	2233	2233
V _{n2}	V _{n2}	kN	17658	17658	17658	17658
V _n	Nominal shear resistance V _n =min(V _{n1} , V _{n2})	kN	2233	2233	2233	2233
V _r	Factored shear resistance	kN	2009	2009	2009	2009
V _u	Shear	kN	697	416	566	324
(5.8.2.7)	Shear checking		OK	OK	OK	OK

CALCULATION SHEET

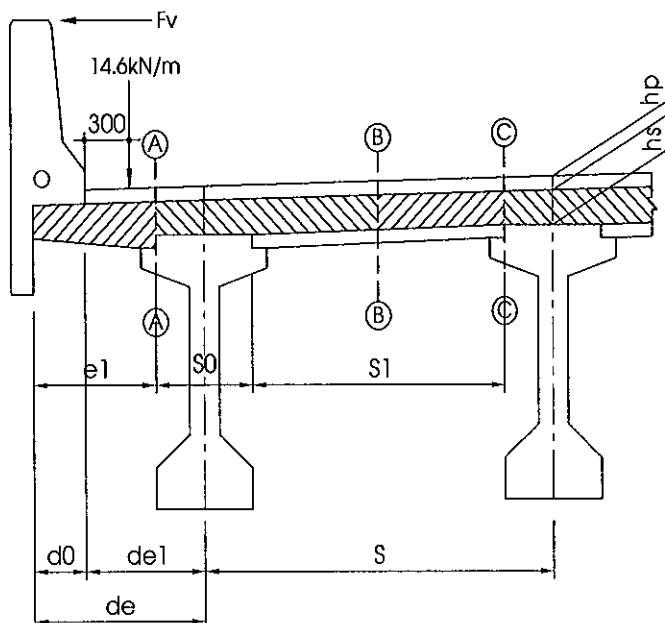
DECK SLAB

	DA NANG - QUANG NGAI EXPRESSWAY PROJECT TYPICAL CALCULATION DETAIL DESIGN DECK SLAB - I GIRDER	Item.	Eng.	Date.	Sign.
		Design			
		Check			
		Revise			

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/m ³	24.50
gas	kN/m ³	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.

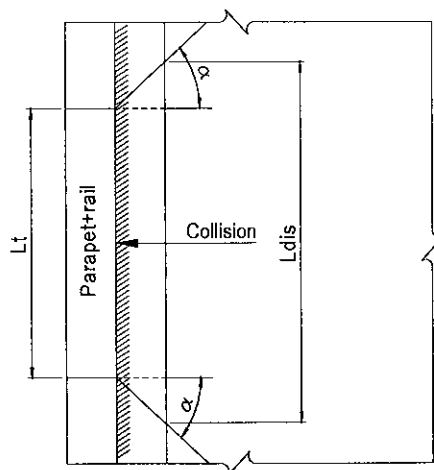
Lane load	Plan =	3.1 kN/m ²
Pedestrian load	Pdb =	0.0 kN/m ²

Collision force to barriers

Test Levels	L5
Horizontal 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 (m2)	Li (m)	ei (m)	N (kN/m)	Q (kN/m)	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)

Modification 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

$\eta = 1.00$

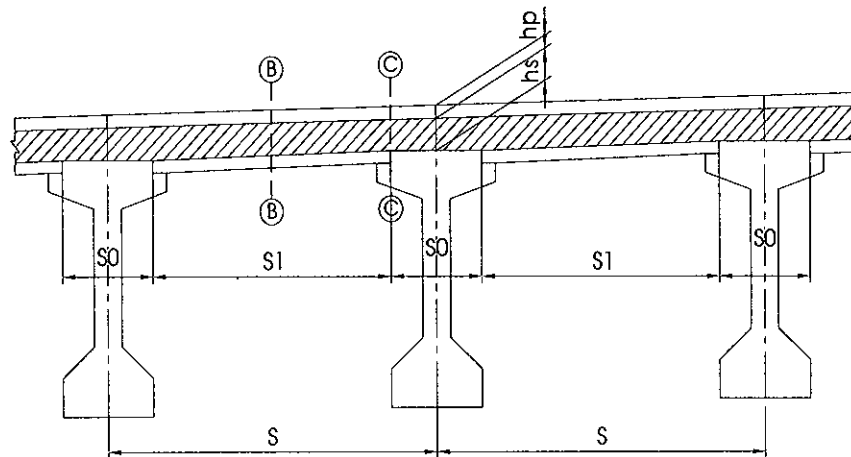
Loads	Combinations		
	Service	Strength	Extreme
Deck slab	sec1	1.00	1.25
	sec2	1.00	1.25
Pavement		1.00	1.50
Curb&handrail		1.00	1.25
Pedestrian		1.00	1.75
Lane load		1.00	1.75
Vehicle wheel		1.00	2.19
Horizontal collision			1.00
Verical collision			1.00

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

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		Design			
		Check			
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INNER DECK SLAB

1. Dead load



<S.4.6.2.1.6>

Distance between long. Girder

$$S = 2.55 \text{ m}$$

Distance between two supports

$$S_0 = 0.60 \text{ m}$$

$$S_1 = 1.95 \text{ m}$$

Deck slab

Unit weight of RC

$$g_c = 24.5 \text{ kN/m}^3$$

Thickkness of Deckslab

$$t = 0.207 \text{ m}$$

Deck strip for calculation

$$L = 1.0 \text{ m}$$

Dead load of deckslab

$$P_{ds} = 5.1 \text{ kN/m}$$

Pavement

Density

$$g_p = 22.5 \text{ kN/m}^3$$

Thickkness of Pavement

$$t = 0.084 \text{ m}$$

Dead load of Pavement

$$P_{ws} = 1.9 \text{ kN/m}$$

Parapet

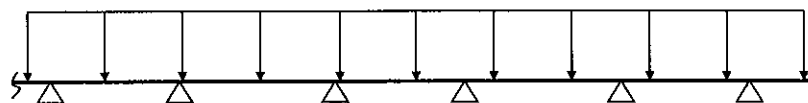
Dead load of Curb&handrail at outside

$$P_{pa} = 13.7 \text{ kN/m}$$

Dead load of median strip

$$P_{me} = 10.0 \text{ kN/m}$$

$q \text{ (kN/m)}$



Momens due to dead load are calculated base on:

$$M = q \cdot l^2 / c$$

With c is constant, is taken as

for positive

$$c_1 = 10.0$$

for negative

$$c_2 = 8.0$$

2. Live load

Liveload

Lane load for 1 lane

$$P_l = 9.3 \text{ kN/m}$$

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

$$W_{po.strip} = 660 + 0.55 \cdot S$$

$$S = 1950 \text{ mm}$$

$$W_{po.strip} = 1733 \text{ mm}$$

$$M = 22510 \text{ Nmm/mm}$$

Look up the unfactored in table A4.1

Negative Moment

Width of primary strip for

$$W_{ne.strip} = 1220 + 0.25 \cdot S$$

$$W_{ne.strip} = 1708 \text{ mm}$$

$$e = 0.0 \text{ mm}$$

$$M = 23655 \text{ Nmm/mm}$$

Dist. from CL of girder to sec. of Neg.

Look up the unfactored in table A4.1

3. Combinations

Modification factor

$$\eta = \eta_D \cdot \eta_R \cdot \eta_I$$

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) Unfactored load	Sign	Moment	
		Positive	Negative
Deckslab	DC	1.93	-2.41
Pavement	DW	0.72	-0.90
Parapet	DC	2.08	-2.60
Liveload	LL	22.51	-23.66

Combination (kNm/m)	Moment	
	Positive	Negative
Service	27.2	-29.6
Strength	45.5	-49.0

	DA NANG - QUANG NGAI EXPRESSWAY PROJECT TYPICAL CALCULATION DETAIL DESIGN DECK SLAB - I GIRDER	Item.	Eng.	Date.	Sign.
		Design			
		Check			
		Revise			

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REINFORCEMENT CHECKING - CANTILEVER SLAB CHECK

MATERIALS				
NORMAL CONCRETE				
fc	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 SECTION						
	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	0	0	0	
FLEXURAL MOMENT CHECKING						
H	Section height	m	0.200	0.200	0.200	
d's	Dis. From comp. fiber to centroid of comp. Reinf	m	0.077	0.077	0.077	
d _{lx}	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	
d _s	Dis. From comp. fiber to centroid of tension Reinf	m	0.148	0.148	0.148	
d'ps	Dis. From comp. fiber to centroid of comp. prestressing steel	m	0.000	0.000	0.000	
d _{lxp}	Dis. From tens. fiber to centroid of tension prestressing steel	m	0.000	0.000	0.000	
dps	Dis. From comp. fiber to centroid of tension prestressing steel	m	0.200	0.200	0.200	
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	
I _z	Moment of inertia of section	m4	0.001	0.001	0.001	
A _{mc}	Section area	m2	0.200	0.200	0.200	
	Steel choice					
A _{ps}	Tension prestressing steel	P.S type	0 T00.0	0 T00.0	0 T00.0	
		Number	tendons	0	0	0
		Area	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
A _s	Tension Reinforcement	Number	bars	7	7	7
		Diameter	mm	20	20	20
		Area	m2	0.00220	0.00220	0.00220
A's	Compression Reinforcement	Number	bars	0	0	0
		Diameter	mm	20	20	20
		Area	m2	0.00000	0.00000	0.00000
A's	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	
φ _v	Resistance factors for shear		0.90	1.00	1.00	
φ _n	Resistance factors for axial force		1.00	1.00	1.00	
β ₁	Stress block factor		0.800	0.800	0.800	
c	Dis. Between centroid and top fiber	m	0.037	0.037	0.037	
	For T section behavior	m	0.037	0.037	0.037	
	For rectangular section behavior	m	0.037	0.037	0.037	
f _{pe}	Effective stress in the prestressing steel after losses	Mpa	1116	1116	1116	
f _{ps}	Aver. stress in pres. steel at the time for which the nominal resistance	Mpa	1762	1762	1762	
k	Factor depends on type of P.S, Low relaxation strand k = 0.28		0.28	0.28	0.28	

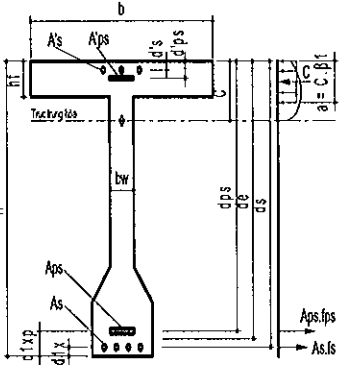
a	Depth of equivalent stress block	m	0.030	0.030	0.030		
de	Corresp. effective depth from extreme comp. fiber to centroid of tensile force in the tensile reinf.	m	0.148	0.148	0.148		
Mn	Nominal resistance	kNm	117	117	117		
Mr	Factored resistance	kNm	105	117	117		
Mu	Flexural moment	kNm	24	17	95		
(5.7.3.2)	Flexural moment Checking		OK	OK	OK		
	Limits for reinforcement						
c/de	Maximum reinforcement		0.25	0.25	0.25		
	Maximum reinforcement Checking	≤ 0.42	OK	OK	OK		
r min	Minimum reinforcement		1.10%	1.10%	1.10%		
	Minimum reinforcement Checking for RC	0.26%	OK	OK	OK		
1.2*Mcr	Cracking moment	kNm	18	18	18		
(5.7.3.3.2)	Checking $M_r \geq \min(1.2M_{cr}, 1.33M_u)$		OK	OK	OK		
(5.8.3.5)	Tensile force in steel should be satisfied - F_{yc}	kN	240	145	720		
	Checking $A_s f_y + A_{ps} f_{ps} \geq F_{yc}$		Ok	Ok	Ok		
(5.7.3.4)	Control of cracking by distr. of reinf for RC member- Check?		No	Yes	No		
	Existing condition for structure	1,2 or 3	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	m ²	0.015	0.015	0.015		
f _{sa}	Value	Mpa	327	327	327		
0.6*f _y		Mpa	240	240	240		
	Tensile stress in reinf $\min(f_{sa}, 0.6f_y)$	Mpa	240	240	240		
x	Dist. From compression fiber to centroid	m	-	0.054	-		
J.d	Arm	m	-	0.13	-		
I _{cr}	Moment of inertia of the cracked section	m ⁴	-	0.00	-		
f _s	Tensile stress in reinforcement $f_s = M_{s1} / (A_s * J.d)$	Mpa	-	59	-		
	Checking for control cracking $f_s < f_{sa}$		N.a	OK	N.a		
(5.10.8.2)	Shrinkage and temperature Reinforcement (side distribution)						
A _{req}	Area of required reinf	m ²	0.00023	0.00023	0.00023		
	Distribution on sides 2 D14	m ²	0.00030	0.00030	0.00030		
	Required Spacing not larger than	m	0.45	0.45	0.45		
	Checking		OK	OK	OK		

DA NANG - QUANG NGAI EXPRESSWAY PROJECT		Item.	Eng.	Date.	Sign.
TYPICAL CALCULATION		Design			
DETAIL DESIGN		Check			
DECK SLAB - I GIRDER		Revise			

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REINFORCEMENT CHECKING - INNER SLAB CHECK

MATERIALS			
NORMAL CONCRETE			
fc	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
fpv	Yield strength of prestressing steel	Mpa	1670
Ep	Modulus of Elasticity	Mpa	197000
REINFORCEMENT			
fy	Yield strength	Mpa	400
Es	Modulus of Elasticity	Mpa	200000
nc	Ratio Es/Ec		7



Sign	Parameters	Unit	Section - INNER SLAB			
			B-B	B-B	C-C	C-C
INTERNAL FORCES AT SECTION						
	Combination		Strength	Service	Strength	Service
Qu	Shear	kN	0	0	0	0
Mu	Flexural Moment	kNm	45	27	49	30
Nu	Axial load	kN	0	0	0	0
Tu	Torsional Moment	kNm	0	0	0	0
FLEXURAL MOMENT CHECKING						
H	Section height	m	0.200	0.200	0.200	0.200
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	0.148	0.148	0.148	0.148
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	0.200	0.200	0.200	0.200
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	0.200	0.200	0.200	0.200
	Steel choice					
Aps	Tension prestressing steel	P.S type	0 T00.0	0 T00.0	0 T00.0	0 T00.0
		Number	0	0	0	0
		Area	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	0	0	0	0
		Area	0.00000	0.00000	0.00000	0.00000
As	Tension Reinforcement	Number	7	7	7	7
		Diameter	20	20	20	20
		Area	0.00220	0.00220	0.00220	0.00220
A's	Compression Reinforcement	Number	0	0	0	0
		Diameter	20	20	20	20
		Area	0.00000	0.00000	0.00000	0.00000
A'c	Shear reinforcement	Number	1	1	1	1
		Diameter	14	14	14	14
		Area	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
β1	Stress block factor		0.800	0.800	0.800	0.800
c	Dis. Between centroid and top fiber	m	0.037	0.037	0.037	0.037
	For T section behavior	m	0.037	0.037	0.037	0.037
	For rectangular section behavior	m	0.037	0.037	0.037	0.037
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	1762	1762	1762	1762
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	0.030	0.030	0.030	0.030
de	Corresp. effective depth from extreme comp. fiber to centroid of tensile force in the tensile reinf.	m	0.148	0.148	0.148	0.148
Mn	Nominal resistance	kNm	117	117	117	117
Mr	Factored resistance	kNm	105	117	105	117
Mu	Flexural moment	kNm	45	27	49	30
(5.7.3.2)	Flexural moment Checking		OK	OK	OK	OK
	Limits for reinforcement					
c/de	Maximum reinforcement		0.25	0.25	0.25	0.25
	Maximum reinforcement Checking	≤ 0.42	OK	OK	OK	OK
r min	Minimum reinforcement		1.10%	1.10%	1.10%	1.10%
	Minimum reinforcement Checking for RC	0.26%	OK	OK	OK	OK
1.2*Mcr	Cracking moment	kNm	18	18	18	18
(5.7.3.2)	Checking $M_r \geq \min(1.2M_{cr}, 1.33M_u)$		OK	OK	OK	OK
(5.7.3.4)	Control of cracking by distr. of reinf for RC member- Check?		No	Yes	No	Yes
	Existing condition for structure	1,2 or 3	1	1	1	1
dc	Concr. thickness fro. Tens. fiber to tens. reinf nearest	m	0.052	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	m ²	0.015	0.015	0.015	0.015
fsa	Value	Mpa	327	327	327	327
0.6*fy		Mpa	240	240	240	240
	Tensile stress in reinf Min(fsa,0.6fy)	Mpa	240	240	240	240
x	Dist. From compression fiber to centroid	m	-	0.054	-	0.054
J.d	Arm	m	-	0.13	-	0.13
Icr	Moment of inertia of the cracked section	m ⁴	-	0.00	-	0.00
fs	Tensile stress in reinforcement $f_s = M_{sls} / (A_s * J.d)$	Mpa	-	95	-	103
	Checking for control cracking $f_s < f_{sa}$		N.a	OK	N.a	OK
(5.10.8.2)	Shrinkage and temperature Reinforcement (side distribution)					
Areq	Area of required reinf	m ²	0.00023	0.00023	0.00023	0.00023
	Distribution on sides 2 D14	m ²	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

MINISTRY OF TRANSPORT

VIETNAM EXPRESSWAY CORPORATION - PROJECT MANAGEMENT UNIT NO.85

DA NANG - QUANG NGAI EXPRESSWAY DEVELOPMENT PROJECT

Package A2

CALCULATION SHEETS

MISCELLANEOUS

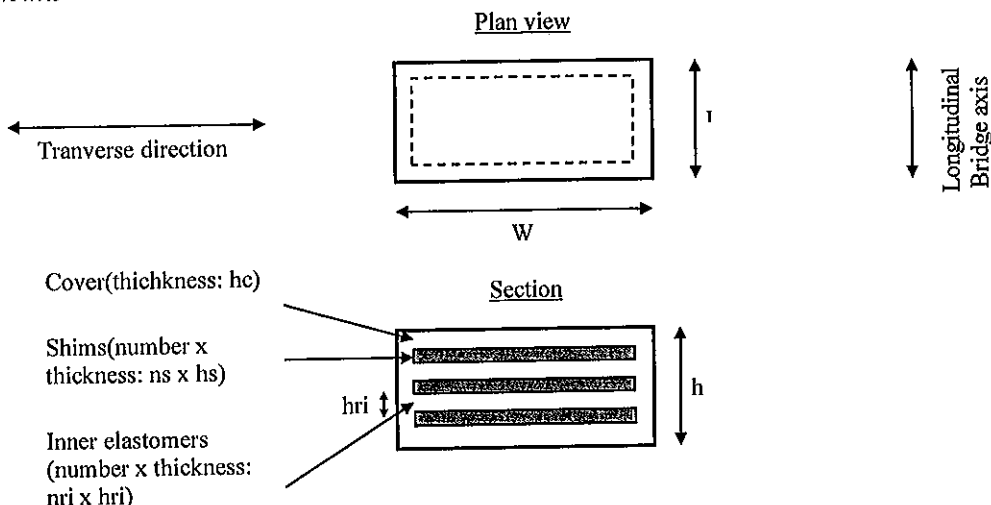
CALCULATION SHEET
BEARINGS

	DA NANG - QUANG NGAI EXPRESSWAY PROJECT PACKAGE 5 - BRIDGE DETAIL DESIGN STEEL-REINFORCED ELASTOMERIC BEARING I-21m	Item.	Eng.	Date.	Sign.
		Design			
		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

Yield strength of shims plate and soles steel (ASTM A709M grade 250)

G = 1.00 Mpa
Fy = 250.0 Mpa

2. Load

Bearing type:

"1: bearing subject to shear deformation (moveable bear.); 2: bearing fixed against shear deformation (fixed bear.)"

Design force on bearing

Combination	Max. Factored Reaction		Rotation		Horizontal movement Δ_0 (mm)
	All (kN)	Live load (kN)	θ_s Long. (rad)	θ_s Trans. (rad)	
Strength 1	1240.9	700.2			
Service 1	823.7	400.1	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	
	Transverse 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	
	Transverse axis	Ws	mm	390	
Cover	Top thickness	hct	mm	2.5	34.28 Ok
	Bottom thickness	hcb	mm	2.5	34.28 Ok
Inner elastomer 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 $S_i = L.W / [2.hri.(L+W)]$ $L.W / [2(L+W)] = 85.7$
 * 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 $\sigma_s = 6.86$ Mpa
 Service average compressive stress due to live load $\sigma_L = 3.33$ Mpa
 Shape factor of the thickest layer of the bearing $S = 4.59$

For bearings subject to shear deformation $\sigma_s \leq 1.66.G.S \leq 11.0$ Mpa $1.66.G.S = 7.62$ Mpa
 Check = Ok

$\sigma_L \leq 0.66.G.S$ $0.66.G.S = 3.03$ Mpa
 Check = N.G

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

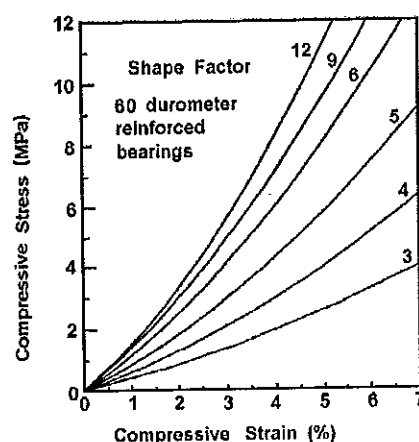
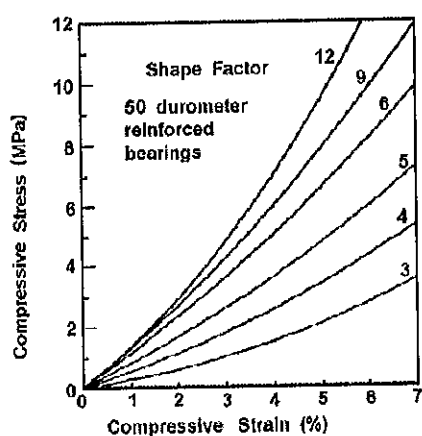
Only when deck joints or seals are present on the bridge: "1: check , 2: not check"

Instantaneous deflection shall be taken $\delta = \sum \epsilon_i . h_{ri} \leq 3$ 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

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 h_{ri}

No	Elastomer Layer Number	σ_L (Mpa)	S	h_{ri} (mm)	ϵ_i (%)	$\epsilon_i . h_{ri} . n_i$ (mm)
1	Top cover 1	3.33	4.59	2.5	3.5	0.09
2	Bottom cover 1	3.33	4.59	2.5	3.5	0.09
3	Inner layers 3	3.33	4.59	18.7	3.5	1.96
4						
5						
						$\delta_{cr} = 0.75$
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, Δ_S , shall be taken as Δ_0 , modified to account for the substructure stiffness and construction procedures.

If a low friction sliding surface is installed, Δ_S need not be taken to be larger than the deformation corresponding to first slip.

The horizontal movement of the bridge superstructure $\Delta_0 = \Delta_{cr} + \Delta_{sh} + \Delta_{ps} + \Delta_{temp}$ $\Delta_0 = 8.5$ mm
 (extreme displacement caused by creep, shrinkage, post-tensioning, combined with thermal effects)

Total elastomer thickness $h_{rt} = 61$ mm
 $2.\Delta_S = 17.01$ mm
 The bearing shall satisfy $h_{rt} \geq 2.\Delta_S$ 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 = 3 layers

Stress in elastomer

σ_s = 6.86 Mpa

Length of pad if rotation is about its transverse axis or width of pad if rotation is about its longitudinal axis

B

Maximum service rotation due to the total load about long. Or trans. axis

θ_s

No	Direction	B (mm)	hri (mm)	θ_s (rad)
1	Longitudinal rotation	300.0	18.7	0.008
2	Transverse rotation	400.0	18.7	0.000

No	Direction	F1 (Mpa)	Check F1	F2 (Mpa)	Check F2	F3 (Mpa)	Check F3
1	Longitudinal rotation	3.2	Ok	7.4	Ok	-	-
2	Transverse rotation	0.0	Ok	8.6	Ok	-	-

f.Stability of Elastomeric Bearings S.14.7.5.3.6

Bearings satisfying equation here shall be considered stable $2A \leq B$

Where

$$A = (1.92.hri/L)/\sqrt{1+2.0.L/W}$$

A = 0.247

$$B = 2.67 / [(S+2.0) \cdot (1+L/(4.0.W))]$$

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 $2A \leq B$, the stress due to total load shall be satisfy below equations:

If the bridge deck is free to translate horizontally $\sigma_s \leq G.S / (2A-B)$

G.S / (2A-B) = 30.1 Mpa

Check = Stable

If the bridge deck is fixed against horizontal translation $\sigma_s \leq G.S / (A-B)$

(A-B) = -0.094

G.S / (A-B) = -

Check = Stable

If the value $A-B \leq 0$, the bearing is stable and is not dependent on σ_s .

g.Reinforcement S.14.7.5.3.7

The thickness of the steel reinforcement, h_s , shall satisfy the provisions of Article 14.7.5.3.7 of the AASHTO LRFD Bridge Construction Specifications and:

At service limit state $h_s \geq 3.h_{max}.\sigma_s / F_y$

Thickness of thickest elastomeric layer in elastomeric bearing

h_{max} = 18.7 mm

Service average compressive stress due to total load

σ_s = 6.86 Mpa

Yield strength of steel reinforcement

F_y = 250.0 Mpa

h_s = 2.0 mm

$3.h_{max}.\sigma_s / F_y$ = 1.54 mm

Check = Ok

At fatigue limit state $h_s \geq 2.0.h_{max}.\sigma_L / \Delta F_{TH}$

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

σ_L = 3.33 Mpa

$2.0.h_{max}.\sigma_L / \Delta F_{TH}$ = 0.75 mm

Check = Ok

h.Seismic Provisions

S.14.7.5.3.8

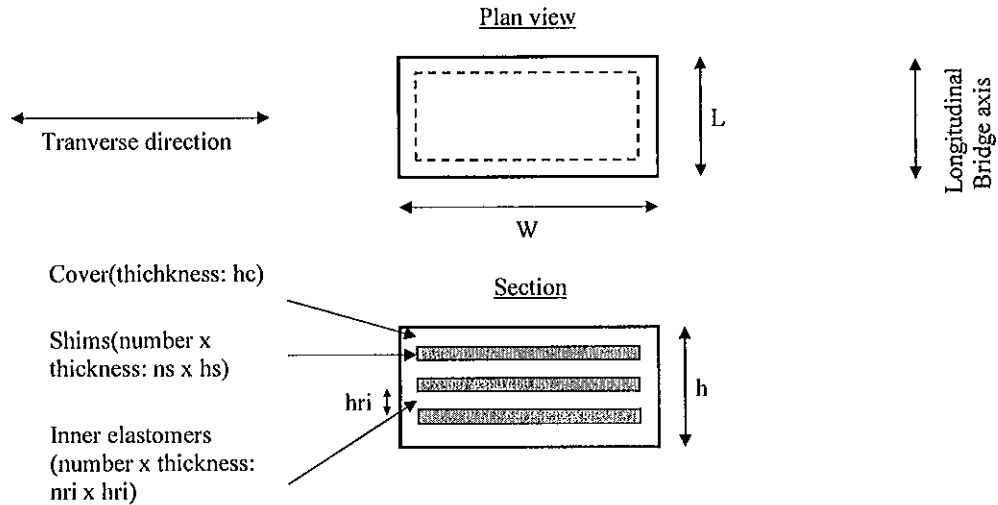
Elastomeric expansion bearings shall be provided with adequate seismic resistant anchorage to resist the horizontal forces in excess of those accommodated by shear in the pad. The sole plate and the base plate shall be made wider to accommodate the anchor bolts. The anchor bolts shall be designed for 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 TYPICAL CALCULATION DETAIL DESIGN STEEL-REINFORCED ELASTOMERIC BEARING 1-27m	Item.	Eng.	Date.	Sign.
		Design			
		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

Yield strength of shims plate and soles steel (ASTM A709M grade 250)

G = 1.00 Mpa
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)	θ_s Long. (rad)	θ_s Trans. (rad)	
Strength I	1493.5	760.04			
Service I	1009.4	434.5	0.010	0.000	21.9

*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	
	Transverse axis	W	mm	450	
	Height	h	mm	84	
Shims	Number of shim	ns	shims	4	
	Thickness	ts	mm	4	
	Longitudinal axis of bridge	Ls	mm	340	
	Transverse axis	Ws	mm	440	
	Top thickness	hct	mm	2.5	39.36 Ok
Cover	Bottom thickness	hcb	mm	2.5	39.36 Ok
Inner elastomer layers					
	Number layers	nr	layers	3	
	Layer thickness	hri	mm	21	4.69
Check total height of bearing		Ok	mm	84	

$$\text{Shape factor of an elastomer layer} \quad S_i = L.W / [2.hri.(L+W)] \quad L.W / [2(L+W)] = 98.4$$

* 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:

$$\begin{aligned} \text{Service average compressive stress due to the total load} & \quad \sigma_s = 6.41 \text{ Mpa} \\ \text{Service average compressive stress due to live load} & \quad \sigma_L = 2.76 \text{ Mpa} \\ \text{Shape factor of the thickest layer of the bearing} & \quad S = 4.69 \end{aligned}$$

$$\begin{aligned} \text{For bearings subject to shear deformation} \quad \sigma_s & \leq 1.66.G.S \leq 11.0 \text{ Mpa} & 1.66.G.S & = 7.79 \text{ Mpa} \\ & & \text{Check} & = \text{Ok} \end{aligned}$$

$$\begin{aligned} \sigma_L & \leq 0.66.G.S & 0.66.G.S & = 3.10 \text{ Mpa} \\ & & \text{Check} & = \text{Ok} \end{aligned}$$

$$\begin{aligned} \text{For bearings fixed against shear deformation} \quad \sigma_s & \leq 2.00.G.S \leq 12.0 \text{ Mpa} & 2.00.G.S & = - \text{ Mpa} \\ & & \text{Check} & = - \end{aligned}$$

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"

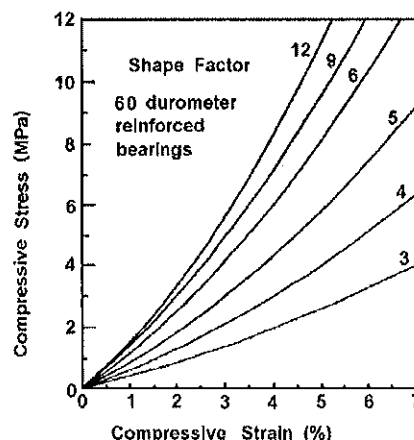
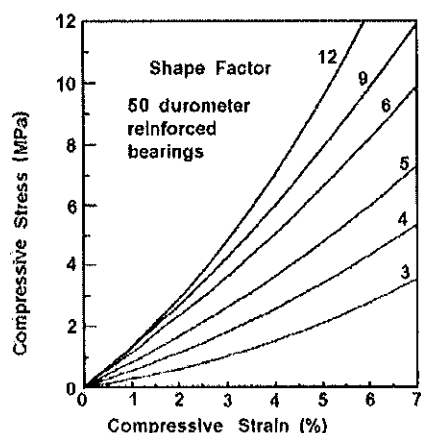
1

Instantaneous deflection shall be taken $\delta = \sum \epsilon_i \cdot h_{ri} \leq 3 \text{ mm}$

A maximum relative deflection across a joint of 3 mm is suggested

Instantaneous compressive strain in i^{th} elastomer layer of a laminated bearing ϵ_i

In the absence of information specific to the particular elastomer to be used, Figure below may be used as a guide.



$$\text{Creep deflection factor} \quad \phi_{cr} = 0.35$$

$$\text{Creep effect to instantaneous load} \quad \delta_{cr} = \delta \cdot \phi_{cr}$$

$$\text{Thickness of } i^{\text{th}} \text{ elastomeric layer in a laminated bearing} \quad h_{ri}$$

No	Elastomer Layer Number	σ_L (Mpa)	S	h_{ri} (mm)	ϵ_i (%)	$\epsilon_i \cdot h_{ri} \cdot n_i$ (mm)
1	Top cover 1	2.76	4.69	2.5	2.2	0.06
2	Bottom cover 1	2.76	4.69	2.5	2.2	0.06
3	Inner layers 3	2.76	4.69	21.0	2.2	1.39
4						
5						
						$\delta_{cr} = 0.52$
Check Ok						$\delta + \delta_{cr} = 2.02$

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 Δ_0 , modified to account for the substructure stiffness and construction procedures.

If a low friction sliding surface is installed, ΔS need not be taken to be larger than the deformation corresponding to first slip.

$$\begin{aligned} \text{The horizontal movement of the bridge superstructure} \quad \Delta_0 & = \Delta_{cr} + \Delta_{sh} + \Delta_{ps} + \Delta_{temp} & \Delta_0 & = 21.9 \text{ mm} \\ \text{(extreme displacement caused by creep, shrinkage, post-tensioning, combined with thermal effects)} & & & \end{aligned}$$

$$\begin{aligned} \text{Total elastomer thickness} & \quad h_{rt} = 68 \text{ mm} \\ & \quad 2.\Delta_s = 43.74 \text{ mm} \\ \text{The bearing shall satisfy} & \quad h_{rt} \geq 2.\Delta_s & \text{Check} & = \text{Ok} \end{aligned}$$

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:

$$\sigma_s > 1.0.G.S \ (\theta_s / n). \ (B / h_{ri})^2 = F1$$

Rectangular bearings subjected to shear deformation shall also satisfy:

$$\sigma_s < 1.875.G.S \ [1-0.2(\theta_s / n). \ (B / h_{ri})^2] = F2$$

Rectangular bearings fixed against shear deformation shall also satisfy:

$$\sigma_s < 2.25.G.S \ [1-0.167(\theta_s / n). \ (B / h_{ri})^2] = F3$$

Number of interior layers of elastomer.

n = 3 layers

Stress in elastomer

$\sigma_s = 6.41 \text{ Mpa}$

Length of pad if rotation is about its transverse axis or width of pad if rotation is about its longitudinal

B

Maximum service rotation due to the total load about long. Or trans. axis

θ_s

No	Direction	B (mm)	h_{ri} (mm)	θ_s (rad)
1	Longitudinal rotation	350.0	21.0	0.010
2	Transverse rotation	450.0	21.0	0.000

No	Direction	F1 (Mpa)	Check F1	F2 (Mpa)	Check F2	F3 (Mpa)	Check F3
1	Longitudinal rotation	4.5	Ok	7.1	Ok	-	-
2	Transverse rotation	0.0	Ok	8.8	Ok	-	-

f.Stability of Elastomeric Bearings S.14.7.5.3.6

Bearings satisfying equation here shall be considered stable $2A \leq B$

Where

$$A = (1.92.h_{ri}/L)/\sqrt{(1+2.0.L/W)}$$

A = 0.233

$$B = 2.67/ [(S+2.0) \cdot (1+L/(4.0.W))]$$

B = 0.334

2A = 0.467

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 satisfy below equations:

If the bridge deck is free to translate horizontally $\sigma_s \leq G.S/ (2A-B)$

$G.S/ (2A-B) = 35.4 \text{ Mpa}$

Check = Stable

If the bridge deck is fixed against horizontal translation $\sigma_s \leq G.S/ (A-B)$

$(A-B) = -0.101$

$G.S/ (A-B) = -$

If the value $A-B \leq 0$, the bearing is stable and is not dependent on σ_s .

Check = Stable

g.Reinforcement S.14.7.5.3.7

The thickness of the steel reinforcement, h_s , shall satisfy the provisions of Article 14.7.5.3.7 of the AASHTO LRFD Bridge Construction Specifications and:

At service limit state

$$h_s \geq 3.h_{max}.\sigma_s/ F_y$$

Thickness of thickest elastomeric layer in elastomeric bearing

$h_{max} = 21.0 \text{ mm}$

Service average compressive stress due to total load

$\sigma_s = 6.41 \text{ Mpa}$

Yield strength of steel reinforcement

$F_y = 250.0 \text{ Mpa}$

$h_s = 4.0 \text{ mm}$

$3.h_{max}.\sigma_s/ F_y = 1.61 \text{ mm}$

Check = Ok

At fatigue limit state

$$h_s \geq 2.0.h_{max}.\sigma_L/ \Delta F_{TH}$$

Constant amplitude fatigue threshold for Category A as specified in Article 6.6

$\Delta F_{TH} = 165.0 \text{ Mpa}$

Service average compressive stress due to live load

$\sigma_L = 2.76 \text{ Mpa}$

$2.0.h_{max}.\sigma_L/ \Delta F_{TH} = 0.70 \text{ mm}$

Check = Ok

h.Seismic Provisions S.14.7.5.3.8

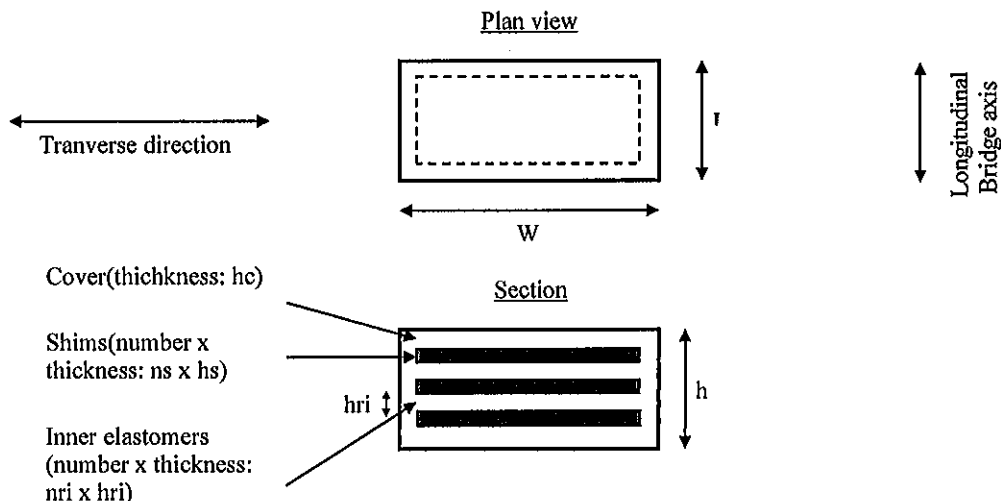
Elastomeric expansion bearings shall be provided with adequate seismic resistant anchorage to resist the horizontal forces in excess of those accommodated by shear in the pad. The sole plate and the base plate shall be made wider to accommodate the anchor bolts. The anchor bolts shall be designed for 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 PKGA2 DETAIL DESIGN STEEL-REINFORCED ELASTOMERIC BEARING I-33m	Item.	Eng.	Date.	Sign.
		Design			
		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

Yield strength of shims plate and soles steel (ASTM A709M grade 250)

G = 1.00 Mpa

Fy = 250.0 Mpa

2. Load

Bearing type:

"1: bearing subject to shear deformation (moveable bear.); 2: bearing fixed against shear deformation (fixed bear.)"

Design force on bearing

Combination	Max. Factored Reaction		Rotation		Horizontal movement Δ_0 (mm)
	All (kN)	Live load (kN)	θ_s Long. (rad)	θ_s Trans. (rad)	
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		
	Transverse 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		
	Transverse axis	Ws	mm	490		
Cover	Top thickness	hct	mm	2.5	41.16	Ok
	Bottom thickness	hcb	mm	2.5	41.16	Ok
Inner elastomer 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 $S_i = L.W / [2.hri.(L+W)]$ $L.W / [2(L+W)] = 102.9$
 * 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 $\sigma_s = 7.07 \text{ Mpa}$
 Service average compressive stress due to live load $\sigma_L = 2.66 \text{ Mpa}$
 Shape factor of the thickest layer of the bearing $S = 6.43$

For bearings subject to shear deformation $\sigma_s \leq 1.66.G.S \leq 11.0 \text{ Mpa}$ $1.66.G.S = 10.67 \text{ Mpa}$
 Check = Ok

$\sigma_L \leq 0.66.G.S$ $0.66.G.S = 4.24 \text{ Mpa}$
 Check = Ok

For bearings fixed against shear deformation $\sigma_s \leq 2.00.G.S \leq 12.0 \text{ Mpa}$ $2.00.G.S = - \text{ Mpa}$
 Check = -

c. Compressive Deflection S.14.7.5.3.3

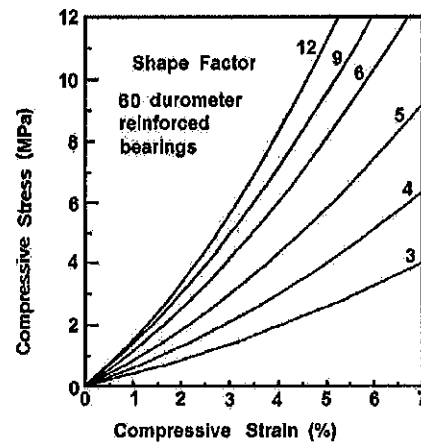
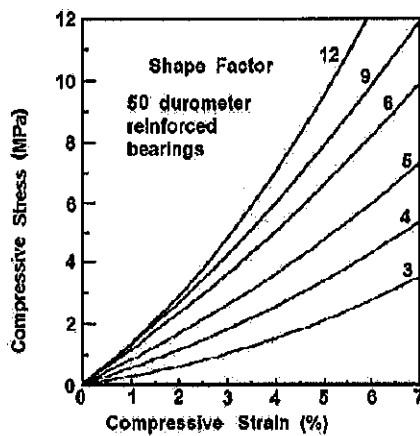
Only when deck joints or seals are present on the bridge: "1: check , 2: not check"

Instantaneous deflection shall be taken $\delta = \sum \epsilon_i \cdot h_{ri} \leq 3 \text{ mm}$

A maximum relative deflection across a joint of 3 mm is suggested

Instantaneous compressive strain in i^{th} elastomer layer of a laminated bearing ϵ_i

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 $\delta_{cr} = \delta \cdot \phi_{cr}$

Thickness of i^{th} elastomeric layer in a laminated bearing h_{ri}

No	Elastomer Layer Number	σ_L (Mpa)	S	h_{ri} (mm)	ϵ_i (%)	$\epsilon_i \cdot h_{ri} \cdot n_i$ (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						
						$\delta_{cr} = 0.60$
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 Δ_0 , modified to account for the substructure stiffness and construction procedures.

If a low friction sliding surface is installed, ΔS need not be taken to be larger than the deformation corresponding to first slip.

The horizontal movement of the bridge superstructure $\Delta_0 = \Delta_{cr} + \Delta_{sh} + \Delta_{ps} + \Delta_{temp}$ $\Delta_0 = 26.7 \text{ mm}$

(extreme displacement caused by creep, shrinkage, post-tensioning, combined with thermal effects)

Total elastomer thickness $h_{rt} = 69 \text{ mm}$

$2.\Delta_s = 53.46 \text{ mm}$

The bearing shall satisfy $h_{rt} \geq 2.\Delta_s$ 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:

$$\sigma_s > 1.0.G.S (\theta_s / n). (B / h_{ri})^2 = F1$$

Rectangular bearings subjected to shear deformation shall also satisfy:

$$\sigma_s < 1.875.G.S [1-0.2(\theta_s / n). (B / h_{ri})^2] = F2$$

Rectangular bearings fixed against shear deformation shall also satisfy:

$$\sigma_s < 2.25.G.S [1-0.167(\theta_s / n). (B / h_{ri})^2] = F3$$

Number of interior layers of elastomer.

n = 4 layers

Stress in elastomer

σ_s = 7.07 Mpa

Length of pad if rotation is about its transverse axis or width of pad if rotation is about its longitudinal axis

B

Maximum service rotation due to the total load about long. Or trans. axis

θ_s

No	Direction	B (mm)	h_{ri} (mm)	θ_s (rad)
1	Longitudinal rotation	350.0	16.0	0.007
2	Transverse rotation	500.0	16.0	0.000

No	Direction	F1 (Mpa)	Check F1	F2 (Mpa)	Check F2	F3 (Mpa)	Check F3
1	Longitudinal rotation	5.4	Ok	10.0	Ok	-	-
2	Transverse rotation	0.0	Ok	12.1	Ok	-	-

f. Stability of Elastomeric Bearings S.14.7.5.3.6

Bearings satisfying equation here shall be considered stable $2A \leq B$

Where

$$A = (1.92.h_{ri}/L)/\sqrt{1+2.0.L/W}$$

A = 0.244

$$B = 2.67 / [(S+2.0) \cdot (1+L/(4.0.W))]$$

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 satisfy below equations:

If the bridge deck is free to translate horizontally $\sigma_s \leq G.S / (2A-B)$

$G.S / (2A-B) = 29.3$ Mpa

Check = Stable

If the bridge deck is fixed against horizontal translation $\sigma_s \leq G.S / (A-B)$

(A-B) = -0.025

$G.S / (A-B) = -$

If the value $A-B \leq 0$, the bearing is stable and is not dependent on σ_s .

Check = Stable

g. Reinforcement S.14.7.5.3.7

The thickness of the steel reinforcement, h_s , shall satisfy the provisions of Article 14.7.5.3.7 of the AASHTO LRFD Bridge Construction Specifications and:

At service limit state $h_s \geq 3.h_{max}.\sigma_s / F_y$

Thickness of thickest elastomeric layer in elastomeric bearing

$h_{max} = 16.0$ mm

Service average compressive stress due to total load

$\sigma_s = 7.07$ Mpa

Yield strength of steel reinforcement

$F_y = 250.0$ Mpa

$h_s = 3.0$ mm

$3.h_{max}.\sigma_s / F_y = 1.36$ mm

Check = Ok

At fatigue limit state $h_s \geq 2.0.h_{max}.\sigma_L / \Delta F_{TH}$

Constant amplitude fatigue threshold for Category A as specified in Article 6.6

$\Delta F_{TH} = 165.0$ Mpa

Service average compressive stress due to live load

$\sigma_L = 2.66$ Mpa

$2.0.h_{max}.\sigma_L / \Delta F_{TH} = 0.52$ mm

Check = Ok

h. Seismic Provisions S.14.7.5.3.8

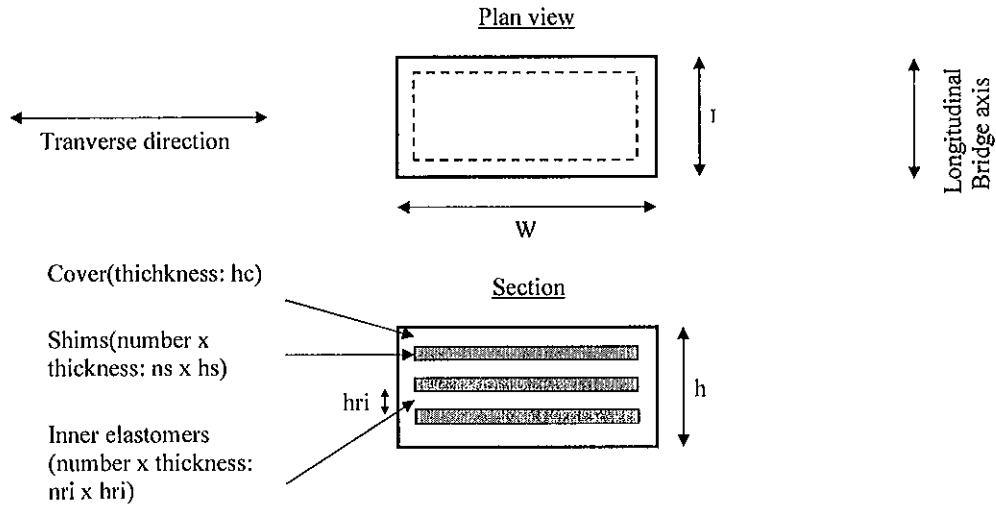
Elastomeric expansion bearings shall be provided with adequate seismic resistant anchorage to resist the horizontal forces in excess of those accommodated by shear in the pad. The sole plate and the base plate shall be made wider to accommodate the anchor bolts. The anchor bolts shall be designed for 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.

	DA NANG - QUANG NGAI EXPRESSWAY PROJECT	Item.	Eng.	Date.	Sign.
	TYPICAL CALCULATION	Design			
	DETAIL DESIGN	Check			
	STEEL-REINFORCED ELASTOMERIC BEARING I-40m	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

Yield strength of shims plate and soles steel (ASTM A709M grade 250)

G = 1.00 Mpa
Fy = 250.0 Mpa

2.Load

Bearing type:

"1: bearing subject to shear deformation (movable 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)	θ_s Long. (rad)	θ_s Trans. (rad)	
Strength I	2207.9	839.4			
Service I	1557.1	479.7	0.012	0.000	32.4

*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		
	Transverse axis	W	mm	550		
	Height	h	mm	99		
Shims	Number of shim	ns	shims	5		
	Thickness	ts	mm	3		
	Longitudinal axis of bridge	Ls	mm	340		
	Transverse axis	Ws	mm	540		
Cover	Top thickness	hct	mm	2.5	42.76	Ok
	Bottom thickness	hcb	mm	2.5	42.76	Ok
Inner elastomer layers						
	Number layers	nr	layers	4		
	Layer thickness	hri	mm	19.8	5.41	
Check total height of bearing		Ok	mm	99		

Shape factor of an elastomer layer $S_i = L.W / [2.hri.(L+W)]$ $L.W / [2(L+W)] = 106.9$
 * 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 $\sigma_s = 8.09$ Mpa
 Service average compressive stress due to live load $\sigma_L = 2.49$ Mpa
 Shape factor of the thickest layer of the bearing $S = 5.41$

For bearings subject to shear deformation $\sigma_s \leq 1.66.G.S \leq 11.0$ Mpa $1.66.G.S = 8.98$ Mpa
 Check = Ok

$\sigma_L \leq 0.66.G.S$ $0.66.G.S = 3.57$ 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"

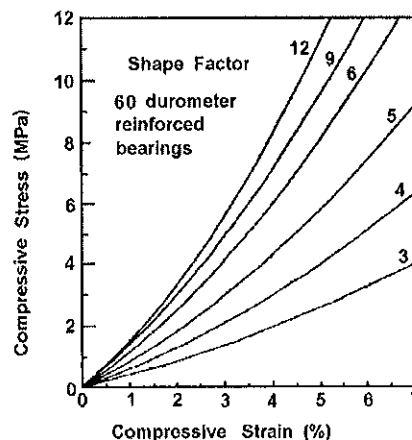
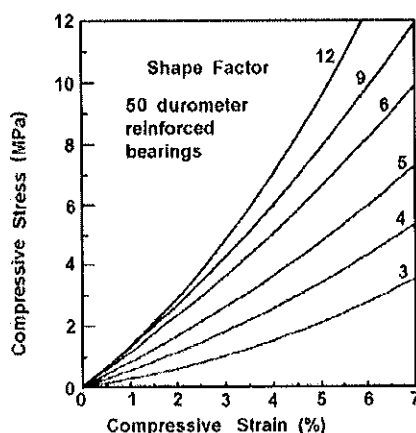
1

Instantaneous deflection shall be taken $\delta = \sum \epsilon_i . h_{ri} \leq 3$ 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 ϵ_i

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 h_{ri}

No	Elastomer Layer Number	σ_L (Mpa)	S	h_{ri} (mm)	ϵ_i (%)	$\epsilon_i . h_{ri} . n_i$ (mm)
1	Top cover 1	2.49	5.41	2.5	2.5	0.06
2	Bottom cover 1	2.49	5.41	2.5	2.5	0.06
3	Inner layers 4	2.49	5.41	19.8	2.5	1.98
4						
5						
						$\delta_{cr} = 0.74$
Check Ok						$\delta + \delta_{cr} = 2.84$

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, Δ_s , shall be taken as Δ_0 , modified to account for the substructure stiffness and construction procedures.

If a low friction sliding surface is installed, Δ_s need not be taken to be larger than the deformation corresponding to first slip.

The horizontal movement of the bridge superstructure $\Delta_0 = \Delta_{cr} + \Delta_{sh} + \Delta_{ps} + \Delta_{temp}$ $\Delta_0 = 32.4$ mm
 (extreme displacement caused by creep, shrinkage, post-tensioning, combined with thermal effects)

Total elastomer thickness $h_{rt} = 84$ mm
 $2.\Delta_s = 64.80$ mm

The bearing shall satisfy $h_{rt} \geq 2.\Delta_s$ 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:

$$\sigma_s > 1.0 \cdot G.S \cdot (\theta_s / n) \cdot (B / h_{ri})^2 = F1$$

Rectangular bearings subjected to shear deformation shall also satisfy:

$$\sigma_s < 1.875 \cdot G.S \cdot [1 - 0.2(\theta_s / n) \cdot (B / h_{ri})^2] = F2$$

Rectangular bearings fixed against shear deformation shall also satisfy:

$$\sigma_s < 2.25 \cdot G.S \cdot [1 - 0.167(\theta_s / n) \cdot (B / h_{ri})^2] = F3$$

Number of interior layers of elastomer.

n = 4 layers

Stress in elastomer

σ_s = 8.09 Mpa

Length of pad if rotation is about its transverse axis or width of pad if rotation is about its longitudinal axis

B

Maximum service rotation due to the total load about long. Or trans. axis

θ_s

No	Direction	B (mm)	h_{ri} (mm)	θ_s (rad)
1	Longitudinal rotation	350.0	19.8	0.012
2	Transverse rotation	550.0	19.8	0.000

No	Direction	F1 (Mpa)	Check F1	F2 (Mpa)	Check F2	F3 (Mpa)	Check F3
1	Longitudinal rotation	5.2	Ok	8.2	Ok	-	-
2	Transverse rotation	0.0	Ok	10.1	Ok	-	-

f. Stability of Elastomeric Bearings S.14.7.5.3.6

Bearings satisfying equation here shall be considered stable $2A \leq B$

Where

$$A = (1.92 \cdot h_{ri} / L) / \sqrt{1 + 2.0 \cdot L / W}$$

$$A = 0.306$$

$$B = 2.67 / [(S + 2.0) \cdot (1 + L / (4.0 \cdot W))]$$

$$B = 0.311$$

$$2A = 0.611$$

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 satisfied below equations:

If the bridge deck is free to translate horizontally $\sigma_s \leq G.S / (2A - B)$

$$G.S / (2A - B) = 18.0 \text{ Mpa}$$

Check = Stable

If the bridge deck is fixed against horizontal translation $\sigma_s \leq G.S / (A - B)$

$$(A - B) = -0.005$$

$$G.S / (A - B) = -$$

If the value $A - B \leq 0$, the bearing is stable and is not dependent on σ_s .

Check = Stable

g. Reinforcement S.14.7.5.3.7

The thickness of the steel reinforcement, h_s , shall satisfy the provisions of Article 14.7.5.3.7 of the AASHTO LRFD Bridge Construction Specifications and:

At service limit state $h_s \geq 3 \cdot h_{max} \cdot \sigma_s / F_y$

Thickness of thickest elastomeric layer in elastomeric bearing

$$h_{max} = 19.8 \text{ mm}$$

Service average compressive stress due to total load

$$\sigma_s = 8.09 \text{ Mpa}$$

Yield strength of steel reinforcement

$$F_y = 250.0 \text{ Mpa}$$

$$h_s = 3.0 \text{ mm}$$

$$3 \cdot h_{max} \cdot \sigma_s / F_y = 1.92 \text{ mm}$$

Check = Ok

At fatigue limit state $h_s \geq 2.0 \cdot h_{max} \cdot \sigma_L / \Delta F_{TH}$

Constant amplitude fatigue threshold for Category A as specified in Article 6.6

$$\Delta F_{TH} = 165.0 \text{ Mpa}$$

Service average compressive stress due to live load

$$\sigma_L = 2.49 \text{ Mpa}$$

$$2.0 \cdot h_{max} \cdot \sigma_L / \Delta F_{TH} = 0.60 \text{ mm}$$

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

APPROACH SLAB

	DA NANG - QUANG NGAI EXPRESSWAY PROJECT TYPICAL CALCULATION DETAIL DESIGN APPROACH SLAB (5M)	Item:	Eng:	Date:	Sign:
		Design			
		Check			
		Revise			

I. Loads

Calculating for a 1m wide strip of approach slab:
Supposed calculating approach slab as simple beam

I.1. Dimension of approach slab

L=	5 m	Hmd=	0.78 m	gbt=	24.5 KN/m ³
B=	11.59 m			gmd=	22.1 KN/m ³
h=	0.3 m				
L _{tt} =	3.5 m	(Use L _{tt} = 0.7 L)			

I.2. Selfweight

P _{bt} =	426.4 KN		
q _{bt} =	7.4 KN/m	(Uniform following length of approach slab)	

I.3. Surface pressure

P _{md} =	997.7 KN
q _{md} =	17.2 KN/m

I.4. Live load

Putting a truck in approach slab at one lane

P_b= 72.5 KN

Tire contact area

(Units m)	Long	Tran
On deck bridge	0.362	0.510
On slab surface	1.922	2.070

Putting a tandem in approach slab at one lane

P_b= 55 KN

(Units m)	Long	Tran
On deck bridge	0.274	0.51
On slab surface	1.83	2.07

q_{ht}= 18.2 KN/m
(Uniform following length of approach slab)

q_{ht}= 14.49 KN/m

I.5. Factored internal forces

	q _{md} =	17.2 KN/m	L _{tt} =	3.5 m
	q _{bt} =	7.4 KN/m		
Truck	q _{ht} =	18.2 KN/m	L _{ott} =	1.922 m
Tandem	q _{ht} =	14.49 KN/m	L _{ott} =	1.83 m

I.6 Load combination

Vehicles	Load combination	Unit	Strength I	Service
Truck	Shear	kN	91.9	60.5
	Moment	kNm	93	60
Tandem	Shear	kN	84.5	56.3
	Moment	kNm	114	72

.Factored internal forces

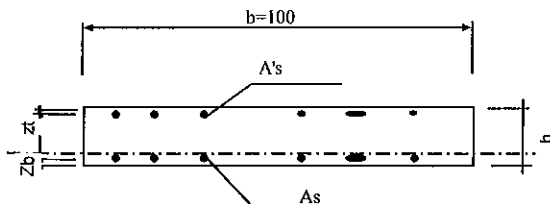
M_{ttmax}= 113.67 KNm

Q_{ttmax}= 91.93 KN

II. Checking

II.1. Checking limits for reinforcement

h= 30 cm b= 100 cm



Compressive Strength of concrete	f _c =	25 Mpa
Yield strength of tensile steel	f _y =	400 Mpa
Dimension of tension reinforcement	D=	20 mm
Number of tension reinforcement	n=	10
Area of tension reinforcement	A _s =	3141.59 mm ²
Dimension of compressive reinforcement	D' =	16 mm
Number of compressive reinforcement	n' =	5
Area of compressive reinforcement	A' s=	1005.31 mm ²