Ministry of Transport (B.GTVT)
Vietnam Expressway Corporation (VEC)
Project Management Unit No. 85 (PMU. 85)

THE WORLD BANK

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

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

DETAILED DESIGN / THIẾT KẾ KỸ THUẬT
PACKAGE/GÓI THẦU: PKG3A (KM 16+880.00 -:- KM18+100.00)

VOLUME 2: STRUCTURAL CALCULATION REPORT / TẬP 2: TÍNH TOÁN KẾT CẤU

(Updated in according to Decision No. 439/QĐ-VEC, on November-23-2012 / Hoàn thiện theo QĐ phê duyệt số 439/QĐ-VEC, ngày 23/11/2012)

November, 2012 / Tháng 11, 2012

The Joint Venture of / Liên danh Tư vấn:
NIPPON KOEI CO.,LTD.
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VOLUME 2: STRUCTURAL CALCULATION REPORT
/ TẬP 2: TÍNH TOÁN KẾT CẤU

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THE JOINT VENTURE OF NK-NE-CHODAI-TEC/LIÊN DANH TƯ VẤN
Project Manager/Giám đốc Dự án

Ichizuru Ishimoto

1.1 PC BOX GIRDER
1.1 KẾT CẤU DẦM HỘP

MỤC LỤC

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Đặc điểm chung về cầu Kỳ Lam được thể hiện trong bảng sau:

Bảng 1.1 Đặc điểm chính của cầu

STT	Đặc điểm chính	
1	Tuyến	Đường cao tốc Đà Nẵng – Quảng Ngãi
2	Tên cầu	Cầu Kỳ Lam
3	Cắt ngang cầu	500+12000+1000+12000+500 = 26000
4	Hoạt tải	HL-93
5	Cấp động đất	Cấp 7
6	Loại đất	II
7	Hệ số gia tốc	A = 0.0341
8	Kết cấu cầu	10-Dầm Super BTDƯL về phía Bắc và 7 dầm hộp, nhịp liên tục BTDƯL về phía Nam
9	Chiều dài cầu	L=1044.80m
10	Sơ đồ nhịp	39.15m+8@40+39.15m+65m+5@100m+65m

1.2 Bản vẽ chung

(xem bản vẽ đính kèm)

1.3 Bản vẽ kết cấu

(xem bản vẽ đính kèm)

1.4 Bố trí cáp dự ứng lực

(xem bản vẽ đính kèm)

1.5 Mặt bằng thi công

1.5.1 Trình tự thi công:

Trình tự tính toán được thực hiện theo trình tự thi công như sau: Biện pháp thi công của cầu được thực hiện với công nghệ đúc hẫng cân bằng từ trụ và đúc trên đà giáo cho đốt dầm biên.

Bảng 1.5.1 Thời gian thi công

STT	**Giai đoạn**	**Hạng mục thi công**
1	Bước_00	- Thi công trụ (P10 ~ P16)
2	Bước_01	- Lắp dựng đà giáo ván khuôn của đốt K_0 trên các trụ(P11 ~ P16) - Đúc đốt K_0trên các trụ - Căng kéo cáp dự ứng lực
3	Bước_02-13	- Lắp đặt xe đúc và ván khuôn trên các trụ chuẩn bị thi công đúc các đốt từ 1~12 (K1 ~K12) - Đúc đốt K1~K12 - Căng kéo cáp dự ứng lựcK1 ~ K13 (từng bước một).
4	Bước_14	- Tháo giỡ xe đúc và ván khuôn
5	Bước_15	- Lắp đặt hệ thống chống đỡ và ván khuôn thi công đốtbiên (L= 13.5m) - Đúc đốt dầm biên - Căng kéo cáp dự ứng lựcNhóm C
6	Bước_16	- Lắp dựng ván khuôn thi công đốt hợp long (đoạn: P12 ~ P13; P14 ~ P15)

1 GENERAL

1.1 General condition

General condition of Ky Lam bridge is as shown bellow:

Table 1.1 Bridge main feature

No.	Main feature	
1	Route	Da Nang – QuangNgai Expressway
2	Bridge name	Ky Lam Bridge
3	Width compsition	500+12000+1000+12000+500 = 26000
4	Live Load	HL-93
5	Seismic class	Class 7
6	Soil Profile Type	II
7	Acceleration Coefficient	A =0.0341
8	Structural Type	10-Span PC Supert Girder at Northbound and 7-Span Continuous PC Box Girder at Southbound
9	Bridge Length	L=1044.80m
10	Span Arrangement	39.15m+8@40+39.15m+65m+5@100m+65m

1.2 General Drawing

(See attached Drawing)

1.3 Structural Drawing

(See attached Drawing)

1.4 PC Cable Arrangement

(See attached Drawing)

1.5 Construction Planning

1.5.1 Contruction order :

The calculation order is based on the decisive construction order as follows:Method of structure construction for bridge by balanced cantilever technology from the pier and casting on the falsework for segments near the bank.

Table 1.5.1 Schedule of Contruction

No	Stage	Construcion item
1	Stage_00	- Construction Piers(P10 ~ P16)
2	Stage_01	- Installation formwork of Piers head segment K_o(P11 ~ P16) - Casting Piers headsegment Ko - Prestressed cable tensiling
3	Stage_02-13	- Installation traveller and formwork for Segments 1~12 (K1 ~K12) - Casting segment K1~K12 - Prestressed cable tensiling K1 ~ K13 (step by step).
4	Stage_14	- Remove traveller and formwork
5	Stage_15	- Installation support system and formwork for side segments (L= 13.5m) - Casting side segment - Tensiling prestressed cable Group C

		- Đúc các đốt hợp long (đoạn: P12 ~ P13; P14 ~ P15) - Căng kéo cáp dự ứng lực nhóm B (đoạn: P12 ~ P13; P14 ~ P15)
7	Bước_17	- Lắp dựng ván khuôn thi công đốt hợp long (đoạn: P11 ~ P12; P15 ~ P16) - Đúc đốt hợp long (đoạn: P11 ~ P12; P15 ~ P16) - Căng kéo cáp dự ứng lực nhóm B (đoạn: P11 ~ P12; P15 ~ P16)
8	Bước _18	- Lắp dựng ván khuôn thi công đốt hợp long (đoạn: P13 ~ P14) - Đúc đốt hợp long (đoạn: P13 ~ P14) - Căng kéo cáp dự ứng lựcnhóm B (đoạn: P13 ~ P14)
9	Bước_19	- Tháo dỡ ván khuôn tại đốt hợp long. - Thi công lan can cầu, lề đường

1.5.2 *Thời gian thi công:*

(xem bản vẽ đính kèm)

2 ĐIỀU KIỆN THIẾT KẾ

2.1 Cơ sở thiết kế

2.1.1 Tổng quát

Kết cấu phần trên phải thỏa mãncông thức (1) đối với từng trạng thái giới hạn, trừ khi có yêu cầu đặc biệt khác. Đối với trạng thái giới hạn sử dụng và trạng thái giới hạn đặc biệt, hệ số sức kháng lấy bằng 1.0.

2.1.2 Hệ số điều chỉnh tải trọng η

Kết cấu phần trên phải thỏa mãn công thức (1) đối với từng trạng thái giới hạn, trừ khi có yêu cầu đặc biệt khác. Đối với trạng thái giới hạn sử dụng và trạng thái giới hạn đặc biệt, hệ số sức kháng lấy bằng 1.0.

$\sum \eta_i \gamma_i Q_i \leq \phi R_n = R_r$ công thức (1)

η_D = 1.00 (tính dẻo).

η_R = 1.00 (tính dư).

η_I = 1.00 (tầm quan trọng trong khai thác).

2.1.3 Hệ số sức kháng φ

Hệ số sức kháng được thể hiện trong bảng sau. Đối với trạng thái giới hạn sử dụngvà trạng thái giới hạn đặc biệt, hệ số sức kháng φ bằng 1.00.

Bảng 2.1.3Hệ số sức khángφ

Cấu kiện	Uốn và kéo	Cắt và xoắn	Nén với đai xoắn và đai đơn	Gối và nén trong mô hình chống giằng	Vùng neo	
					Chịu nén	Thép chịu kéo
CK Bê tông cốt thép	0.90	0.90	0.75	0.80	1.00	1.00
CK Bê tông DƯL (Thông thường)	1.00	0.90	0.75	0.70	0.80	1.00
CK Bê tông DƯL (Thi công phân đoạn)	0.95	0.90	0.75	0.70	0.80	1.00

* Đối với trạng thái giới hạn sử dụng và trạng thái giới hạn đặc biệt, φ=1.00

* Tham khảo tiêu chuẩn Việt Nam “Tiêu chuẩn thiết kế cầu 22TCN-272-05”

Đối với các phần tử chịu nén uốn, giá trị φ có thể tăng tuyến tính cho kết cấu chịu uốn như sức kháng tải trọng dọc trục tính toán, φP_n giảm từ $0,10f'_c A_g$ tới 0.

6	Stage_16	- Installation formwork for close segment (setion: P12 ~ P13; P14 ~ P15) - Casting close segment (setion: P12 ~ P13; P14 ~ P15) - Tensiling prestressed cable group B (setion: P12 ~ P13; P14 ~ P15)
7	Stage_17	- Installation formwork for close segment (setion: P11 ~ P12; P15 ~ P16) - Casting close segment (setion: P11 ~ P12; P15 ~ P16) - Tensiling prestressed cable group B (setion: P11 ~ P12; P15 ~ P16)
8	Stage_18	- Installation formwork for close segment (setion: P13 ~ P14) - Casting close segment (setion: P13 ~ P14) - Tensiling prestressed cable group B (setion: P13 ~ P14)
9	Stage_19	- Removing formwork at close segment. - Contruction Parapet, Pavement

1.5.2 Contruction Schedule :

(See attached Drawing)

2 DESIGN CONDITION

2.1 Design Basic

2.1.1 General

Bridge Superstructure shall satisfy Eq.1 for each limit state, unless otherwise specified. For SERVICEand EXTREME EVENT limit stats, resistance factors shall be taken as 1.0.

2.1.2 Load modifier factor η

Bridge Superstructure shall satisfy Eq.1 for each limit state, unless otherwise specified. For SERVICEand EXTREME EVENT limit stats, resistance factors shall be taken as 1.0.

$$\sum \eta_i \gamma_i Q_i \leq \phi R_n = R_r \quad \text{(Eq.1)}$$

η_D = 1.00 (for ductility).

η_R = 1.00 (for redundancy).

η_I = 1.00 (for operational Importance).

2.1.3 Resistance Factorφ

Resistance factor is shown bellow. For service and extreme event limit state, resistance facter φ is 1.00.

Table 2.1.3Resistance Factor φ

Component	Flexure and Tension	Shear and Torsion	Compression with Spirals and Ties	Bearing and Compression in Strut and Tie models	For Anchorage Zones	
					Compression	Tension in Steel
Reinforced Concrete	0.90	0.90	0.75	0.70	0.80	1.00
Prestressed Concrete (conventional)	1.00	0.90	0.75	0.70	0.80	1.00
Prestressed Concrete (segmental construction)	0.95	0.90	0.75	0.70	0.80	1.00

* For service and extreme event limit state, φ=1.00
* Refer to Vietnamese “Specifications for Bridge Design 22TCN-272-05”

For compression members with flexure, the value of φ may be increased linearly to the value for flexure as the factored axial load resistance, φP_n, decreases from $0.10 f'_c A_g$ to 0.

2.2 Tải trọng và tổ hợp tải trọng:

2.2.1 Tải trọng:

(1) *Tĩnh tải*

Tĩnh tải của kết cấu và tĩnh tải khác(DC) được tính toán bằng cách sử dụng tỷ trọng như sau:

Bảng 2.2.1-1 Tỷ trọng của vật liệu

STT	Vật liệu	Trọng lượng (kg/m^3)	Trọng lượng riêng (kN/m^3)
1	Bê tông thường	2400	23.5
2	Bê tông cốt thép dự ứng lực	2500	24.5
3	Lớp phủ mặt đường	2250	22.0

(2) *Hoạt tải và lực xung kích*

Hoạt tải và lực xung kích căn cứ theo Điều 3.6.1 và 3.6.2 của tiêu chuẩn 22TCN-272-05.

(3) *Nhiệt độ đều*

Phạm vi nhiệt độ áp dụng theo Điều 3.12.2 của tiêu chuẩn 22TCN-272-05. Nhiệt độ bình quân là 25^o theo nhiệt độ trung bình hàng năm ở tỉnh Quảng Nam.

Biên độ nhiệt độ: $+10^o$ đến$+47^o$.

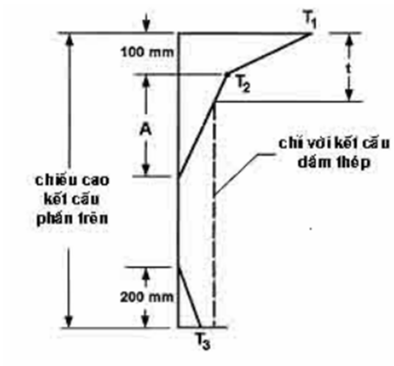
Nhiệt độ trung bình: $+25^o$

(4) *Gradient nhiệt độ*

Gradient nhiệt độ áp dụng Điều 3.12.3 của tiêu chuẩn 22TCN-272-05.

Hình 1 được cho trong bảng 1 cho cả hai trường hợp chênh nhiệt dương và âm. Kích thước "A" cho trong hình 1 được lấy như sau:

- 300 mm cho kết cấu nhịp BTCT có chiều cao 400mm hay lớn hơn.
- Đối với mặt cắt BTCT có chiều cao thấp hơn 400mm thì lấy nhỏ hơn chiều cao thực tế100mm.
- Đối với kết cấu nhịp thép bê tông liên hợp cự ly "t" phải lấy bằng chiều dày bản mặt cầu bằng bê tông.



Hình 3.12.3-1. Gradiend nhiệt trong phương thẳng đứng trong kết cấu nhịp thép và bê tông

Bảng 3.12.3-1- Gradient nhiệt

Thông số	Gradien nhiệt dương	Gradien nhiệt âm
T_1	+23	-7
T_2	+6	-1
T_3	+3	0

2.2 Load and Load Combinations:

2.2.1 Load:

(1) *Dead Load*

Dead load of structural components and nonstructural attachments (DC) are calculated by use of the densities noted below:

Table 2.2.1-1 Material Density

No	Material	Density (kg/m^3)	Unit Weight (kN/m^3)
1	Plain Concrete	2400	23.5
2	Reinforced and Prestressed Concrete	2500	24.5
3	Asphalt Pavement	2250	22.0

(2) *Live Load and Dynamic Load Allowance*

Live load and dynamic load allowance is based on Article 3.6.1 and 3.6.2 of 22TCN-272-05.

(3) *Uniform Temperature*

Temperature ranges are applied based on Article 3.12.2 of 22TCN-272-05. Center temperature of temperature change is 25deg from annual mean temperature in Quang Nam.

Temperature Range = from +10deg. to+47deg.

Mean Temperature = +25deg

(4) *Temperature Gradient*

Temperature gradient is applied based on Article 3.12.3 of 22TCN-272-05.

Figure 1 are given in Table 1 for both positive and negative temperature differentials.Dimension "A" in Figure 1 shall be taken as:

- For concrete superstructures that are 400 mm or more in depth – 300 mm
- For concrete sections shallower than 400 mm – 100 mm less than the actual depth
- For steel/concrete composite superstructures, the distance "t" shall be taken as the depth of the concrete deck

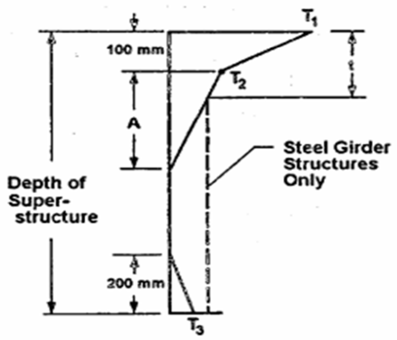


Figure 3.12.3-1 – Vertical Temperature Gradient in Concrete and Steel Superstructures

Table 3.12.3-1 - Temperature Gradients

Parameter	Positive temperature gradient (°c)	Negative temperature gradient (°c)
T_1	+23	-7
T_2	+6	-1
T_3	+3	0

(5) *Hệ số giãn nở nhiệt*

Đối với nhịp dẫn, hệ số giãn nở nhiệt được trình bày như sau.

Bê tông (Điều 5.4.2.2. of 22TCN-272-05)

10.8×10^{-6} /deg (đối với bê tông thông thường)

(6) *Sự co ngót và từ biến của bê tông*

Hệ số từ biến và biến dạng do co ngót được tính theo "Tiêu chuẩn thiết kế kết cấu bê tông theo mô hình EB-FIP, 1990 (MC90)". Điều kiện cơ bản dùng để tính toán hệ số từ biến và co ngót được trình bày trong bảng dưới đây.

Bảng 2.2.1-2Điều kiện cơ bản của từ biến và co ngót

STT	Hạng mục	Giá trị
1	Cường độ chịu nén của bê tông	45 MPa
2	Tuổi thọ của bê tông lúc kết thúc từ biến và co ngót	10,950 ngày (=30năm)
3	Nhiệt độ trung bình	25độ
4	Độ ẩm tương đối	80%
5	Chu vi tiếp xúc với môi trường	Mặt ngoài (100%) + Mặt trong (50%)
6	Loại xi măng	Xi măng Portland đông cứng nhanh hoặc xi măng thường

(7) *Độ lún*

Đối với nhịp chính, căn cứ Tiêu chí thiết kế cầu của dự án đường cao tốc Đà Nẵng – Quảng Ngãi xem xét lún gối 10mm giữa các trụ.

2.2.2 *Tổ hợp tải trọng:*

Bảng 2.2.2-1Tổ hợp tải trọng và hệ số tải trọng

Tổ hợp tải trọng	DC DW	LL&IM BR PL	WA	WS	WL	TU CR SH	TG	SE	CV or EQ
Cường độ– I	γ_p	1.75	1.00	-	-	0.5 / 1.2	γ_{TG}	γ_{SE}	-
Cường độ– II	γ_p	1.35	1.00	-	-	0.5 / 1.2	γ_{TG}	γ_{SE}	-
Cường độ– III	γ_p	-	1.00	1.40	-	0.5 / 1.2	γ_{TG}	γ_{SE}	-
Giới hạn đặc biệt	γ_p	0.50	1.00	-	-	-	-	-	1.00
Giới hạn sử dụng	1.00	1.00	1.00	0.3	1.00	1.0 / 1.2	γ_{TG}	γ_{SE}	-

Bảng 2.2.2-2Tổ hợp tải trọng và hệ số tải trọng

STT	Tổ hợp	DC	DW	LL BR, PL	WA	WS	WL	TU	CR	TG	SE	CV	EQ1	EQ2
1	Cường độ-Ia	1.25	1.50	1.75	1.00	0.00	0.00	0.50	0.50	0.00	0.50	0.00	0.00	0.00
2	Cường độ - Ib	0.90	0.65	1.75	1.00	0.00	0.00	0.50	0.50	0.00	0.50	0.00	0.00	0.00
3	Cường độ - IIa	1.25	1.50	0.00	1.00	1.40	0.00	0.50	0.50	0.00	0.50	0.00	0.00	0.00
4	Cường độ - IIb	0.9	0.65	0.00	1.00	1.40	0.00	0.50	0.50	0.00	0.50	0.00	0.00	0.00
5	Cường độ - IIIa	1.25	1.5	1.35	1.00	0.40	1.00	0.50	0.50	0.00	0.50	0.00	0.00	0.00
6	Cường độ - IIIb	0.9	0.65	1.35	1.00	0.40	1.00	0.50	0.50	0.00	0.50	0.00	0.00	0.00
7	Đặc biệt – Ia	1.25	1.50	0.50	1.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.00	0.00
8	Đặc biệt – Ib	0.90	0.65	0.50	1.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.00	0.00

***(5)* Thermal Expansion Coefficient**

For approach bridges, thermal expansion coefficient is shown below.

Concrete (Article 5.4.2.2. of 22TCN-272-05)

10.8×10^{-6} /deg (for normal density concrete)

***(6)* Concrete Shrinkage and Creep**

Creep Coefficient and Shrinkage Strain are calculated based on "CEB-FIP Model Code for structures, 1990 (MC90)". Basic condition for calculation of creep coefficient and shrinkage strain is shown below.

Table 2.2.1-2 Basic Conditions for Creep and Shrinkage

No	Item	Value
1	Compressive strength of concrete 4	45 MPa
2	Concrete age at the end of creep and shrinkage	10,950 days (=30years)
3	Average temperature 2	25 deg
4	Relative humidity 8	80%
5	Perimeter exposed to the atmosphere O	Outside (100%) + Inside (50%)
6	Type of cement	Normal or Rapid-Hardening Portland Cement

***(7)* Settlement**

For main bridges, 10mm support settlement between 8 supports is considered according to Bridge Design Criteria for Da Nang – QuangNgai Expressway Project.

2.2.2 Load combination:

Table 2.2.2-1Load Combinations and Load Factors

Load Combinaticon	DC DW	LL&IM BR PL	WA	WS	WL	TU CR SH	TG	SE	CV or EQ
Strength – I	γ_p	1.75	1.00	-	-	0.5 / 1.2	γ_{TG}	γ_{SE}	-
Strength – II	γ_p	1.35	1.00	-	-	0.5 / 1.2	γ_{TG}	γ_{SE}	-
Strength – III	γ_p	-	1.00	1.40	-	0.5 / 1.2	γ_{TG}	γ_{SE}	-
Extreme event	γ_p	0.50	1.00	-	-	-	-	-	1.00
Service	1.00	1.00	1.00	0.3	1.00	1.0 / 1.2	γ_{TG}	γ_{SE}	-

Table 2.2.2-2Load Combinations and Load Factors

No	Combination	DC	DW	LL BR, PL	WA	WS	WL	TU	CR	TG	SE	CV	EQ1	EQ2
1	Strength-Ia	1.25	1.50	1.75	1.00	0.00	0.00	0.50	0.50	0.00	0.50	0.00	0.00	0.00
2	Strength - Ib	0.90	0.65	1.75	1.00	0.00	0.00	0.50	0.50	0.00	0.50	0.00	0.00	0.00
3	Strength - IIa	1.25	1.50	0.00	1.00	1.40	0.00	0.50	0.50	0.00	0.50	0.00	0.00	0.00
4	Strength - IIb	0.9	0.65	0.00	1.00	1.40	0.00	0.50	0.50	0.00	0.50	0.00	0.00	0.00
5	Strength - IIIa	1.25	1.5	1.35	1.00	0.40	1.00	0.50	0.50	0.00	0.50	0.00	0.00	0.00
6	Strength - IIIb	0.9	0.65	1.35	1.00	0.40	1.00	0.50	0.50	0.00	0.50	0.00	0.00	0.00
7	Extreme – Ia	1.25	1.50	0.50	1.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.00	0.00

9	Đặc biệt – IIa	1.25	1.50	0.50	1.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.00
10	Đặc biệt - IIb	1.25	1.50	0.50	1.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00
11	Đặc biệt - IIc	0.90	0.65	0.50	1.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.00
12	Đặc biệt - IId	0.90	0.65	0.50	1.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00
13	Sử dụng– I	1.00	1.00	1.00	1.00	0.30	1.00	1.00	1.00	0.50	0.50	0.00	0.00	0.00
14	Sử dụng - III	1.00	1.00	0.80	1.00	0.30	1.00	1.00	1.00	0.50	0.50	0.00	0.00	0.00

Trong đó :

DC- tải trọng bản thân của các bộ phận kết cấu và thiết bị phụ phi kết cấu;

DW – Tải trọng tĩnh bổ sung

TG –Gradient nhiệt; TU – Nhiệt độ đều

LL – Hoạt tải xe; BR – Lực hãm xe

IM –Lực xung kích (lực động) của xe; PL – Tải trọng người đi

WA– Tải trọng nước và áp lực dòng chảy; WS– Tải trọng gió trên kết cấu

WL – Gió trên hoạt tải; SE –Lún

CV – Lực va tàu; EQ – Động đất

CR – Từ biến; SH – Co ngót

2.3 *Các tính chất của vật liệu:*

2.3.1 *Bê tông*

Sử dụngbê tông mác f’c=45MPa (Cấp bê tông :C45) để thiết kế kết cấu thượng bộ dầm hộp. Cường độ bê tông, f’c, là cường độ chịu nén tạithời điểm 28 ngày với mẫu thử hình trụ.

Bảng 2.3.1-1Tính chất của bê tông

STT	Hạng mục	Ký hiệu	Dầm hộp	Kết cấu hạ bộ
1	Cường độ chịu nén trong 28 ngày	f’c	45 Mpa	30 Mpa
2	Modun đàn hồi	Ec	36,057 MPa	29,440 MPa
3	Modun phá hoại	fr	4.23MPa	3.45MPa
4	Hệ số giãn nở do nhiệt	α	10.8 x 10^{-6}/deg	10.8 x 10^{-6}/deg
5	Hệ số Poisson	-	0.2	0.2

**Đường cong ứng suất – biến dạng được thiết lập theo tiêu chuẩn 22TCN-272-05.*

2.3.2 *Cốt thép*

- Cốt thép thường:
  + Cường độ thép áp dụng tiêu chuẩn TCVN1651 - 2008
  + Thép trơn(CB300-T) : f_{sy} = 300 MPa.
  + Thép có gân(CB400-V) : f_{sy} = 400 Mpa
  + Modun đàn hồi: Es = 200000 Mpa.

2.3.3 *Cốt thép dự ứng lực*

Cáp dự ứng lực:Hệ thống cáp dự ứng lực phải đáp ứng tiêu chuẩn ASTM A416-99 Loại 270. Hệ thống cáp và neo sau đây được sử dụng trong tính toán và bố trí kết cấu:

8	Extreme – Ib	0.90	0.65	0.50	1.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.00	0.00
9	Extreme – IIa	1.25	1.50	0.50	1.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.00
10	Extreme - IIb	1.25	1.50	0.50	1.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00
11	Extreme - IIc	0.90	0.65	0.50	1.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.00
12	Extreme - IId	0.90	0.65	0.50	1.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00
13	Service – I	1.00	1.00	1.00	1.00	0.30	1.00	1.00	1.00	0.50	0.50	0.00	0.00	0.00
14	Service - III	1.00	1.00	0.80	1.00	0.30	1.00	1.00	1.00	0.50	0.50	0.00	0.00	0.00

Where :

DC- dead load of structuralcomponents and nonstructural attachments; DW –Additional static load

TG - temperature gradient; TU - uniform temperature

LL - live load; BR - breaking force

IM - vehicular dynamic load allowance; PL - pedestrian live load

WA - water load and stream pressure; WS - wind load on structure

WL - wind on live load; SE - settlement

CV - vessel collision force; EQ - earthquake

CR- creep; SH – shrinkage

2.3 Material Properties:

2.3.1 Concrete

Use of concrete grade of f'c=45MPa (Concrete Class :C45) is assumed for superstructure of main bridge in the design. Concrete strength, f'c, shall be based on the 28-day compressive strength of cylinder specimens.

Table 2.3.1-1 Concrete Properties

No	Item	Symbol	Box girder	Substructure
1	Compressive Strength at 28 days	f'c	45 Mpa	30 Mpa
2	Modulus of Elasticity	Ec	36,057 MPa	29,440 MPa
3	Modulus of Rupture	fr	4.23MPa	3.45MPa
4	Coefficient of Thermal Expansion	α	10.8 x 10^{-6}/deg	10.8 x 10^{-6}/deg
5	Poisson's Ratio	-	0.2	0.2

**The stress-strain curve was set up based on 22TCN-272-05.*

2.3.2 Reinforcing Steel

- Reinforcement:
  + The strength of reinforcement under the standard TCVN1651 - 2008
  + Plain bar (CB300-T) : f_{sy} = 300 MPa.
  + Ribbed bar (CB400-V) : f_{sy} = 400 Mpa
  + Elastic modulus : Es = 200000 Mpa.

2.3.3 Prestressing Tendon

Prestressed cable:Prestressed cable system shall meet the standard ASTM A416-99 Grade270. In order to calculate and locate the structure, the following cable system and anchorage are used:

Bảng 2.3.1-3 Tính chất của cáp dự ứng lực

STT	Hạng mục	Tính chất và giới hạn ứng lực
1	Cáp dự ứng lực	19 tao 15,2mm
2	Diện tích 1 tao cáp	140 mm^2
3	Diện tích 1 bố cáp	2660 mm^2
4	Cường độ kéo căng f_{pu}	1860 Mpa
5	Giới hạn chảy f_{py}	1670 Mpa
6	Modun đàn hồi	197000 Mpa
7	Hệ số ma sát/ chiều dài đơn vị	0.002 m^{-1}
8	Hệ số ma sát góc	0.25 Deg^{-1}
9	Độ tụt neo tính toán	6 mm
10	Mất mát ứng suất	2.5% (ít mất)
		Sau 1000^h, at 20^oC, 0.7 P_M
11	Lực căng kéo 1 bó cáp dự ứng lực	3711 KN
12	Ống bọc cáp	D100/D107 mm

2.4 *Tiêu chuẩn kỹ thuật áp dụng và phần mềm tham khảo*

2.4.1 *Tiêu chuẩn kỹ thuật*

[1]. Tiêu chuẩn thiết kế cầu 22TCN-272-05

Tài liệu tham khảo:

[2]. CEB – FIP model code 1990 (comite euro – international du beton)

[3]. Tiêu chuẩn thiết kế tải trọng và tác động TCVN 2737-1995

[4]. Tiêu chuẩn thiết kế công trình chịu động đất TCXDVN375-2006

[5]. AASHTO LRFD tiêu chuẩn thiết kế cầu, xuất bản lần thứ 4, 2007

[6]. Công trình giao thông trong vùng động đất - 22TCN221-95

2.4.2 *Phần mềm và hệ đơn vị sử dụng trong phân tích kết cấu*

- Phần mềm : RM Bridge V8i - Space frame @Copyright Bentley Systems Austria 2011.
- Đơn vị đo lường gồm có :
  + Chiều dài : mm.Góc : rad.
  + Khối lượng : kg.Lực : kN.
  + Áp lực : kN/m^2.Nhiệt độ : ^oC.

3 *THIẾT KẾ DẦM HỘP BÊ TÔNG DỰ ỨNG LỰC THEO CHIỀU DỌC*

3.1 *Mô hình tính*

Kết cấu cầu được mô hình với một đơn nguyên cầu riêng biệt nhằm giảm số lượng phép tính và thời gian tính toán. Tuy nhiên tư vấn đã xem xét tải trọng lệch tâm do tĩnh tải khi thi công (2 nguyên đơn không thi công đồng thời), hoặc hoạt tải khi khai thác (hoạt tải trên một cầu) là không gây tổ hợp bất lợi cho hệ móng. Phương pháp cộng tác dụng sẽ được áp dụng đối với các trường hợp tải trọng khác để xem xét cho hiệu ứng bất lợi nhất có thể xảy ra cho kết cấu.

Table 2.3.1-3 PrestressingCable Properties

No.	Iterm	Properties and stress limit
1	Prestressed cable	19 strands 15,2mm
2	Area of 1 strand	140 mm^2
3	Area of 1 tendon	2660 mm^2
4	Tensile strength f_{pu}	1860 Mpa
5	Yield point stress f_{py}	1670 Mpa
6	Elastic modulus	197000 Mpa
7	Friction coefficient/ length unit	0.002 m^{-1}
8	Angle friction coefficient	0.25 Deg^{-1}
9	Calculated anchor sliding	6 mm
10	Loose of stress	2.5% (low loose)
		after 1000^h, at 20^oC, 0.7 P_M
11	Tensile force of 1 prestressed tendon	3711 KN
12	Cable duct	D100/D107 mm

2.4 Applied Specification and References Software

2.4.1 Specification

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

References:

[2]. CEB – FIP model code 1990 (comite euro – international du beton)

[3]. Loads and effects design standard TCVN 2737-1995

[4]. Seismic standard TCXDVN375-2006

[5]. AASHTO LRFD 4th edition, 2007 specification for bridge design

[6]. Transportation works in seismic zone - 22TCN221-95

2.4.2 Software and unit system are used for structure analysis

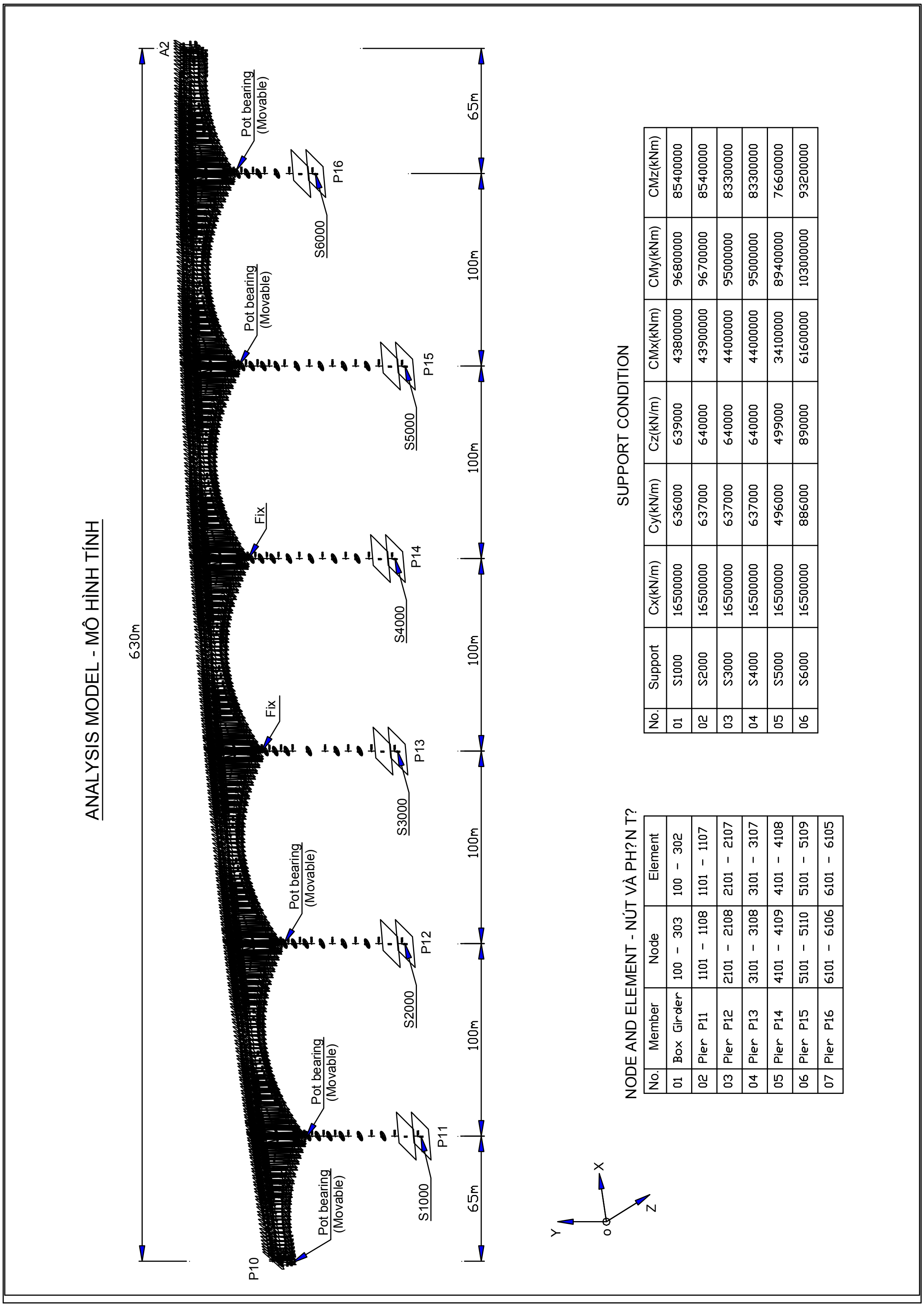
- Software : RM Bridge V8i - Space frame @Copyright Bentley Systems Austria 2011.
- Measurement units consist of :
  + Length : mm.Angle : rad.
  + Weight : kg.Force : kN.
  + Stress : kN/m^2.Temperature : ^oC.

3 LONGGITUDINAL PC BOX GIRDER DESIGN

3.1 Analysis Model

Bridge structural is modeled as one bridge to reduce the amount of calculation and computation time. But consultants have considered eccentric load due static load to construction (2 superstructure of bridge was not constructed at the same time), or live loads (1 section has live load) are not result on unfavorable combination for foundation system. The method of superposition will be applied to the other loadcase to consider for the most unfavorable effects may occur for structures.

3.1.1 ***Mô hình tính****(Xem hình vẽ và Phụ lục về mô hình phân tích dưới đây)*



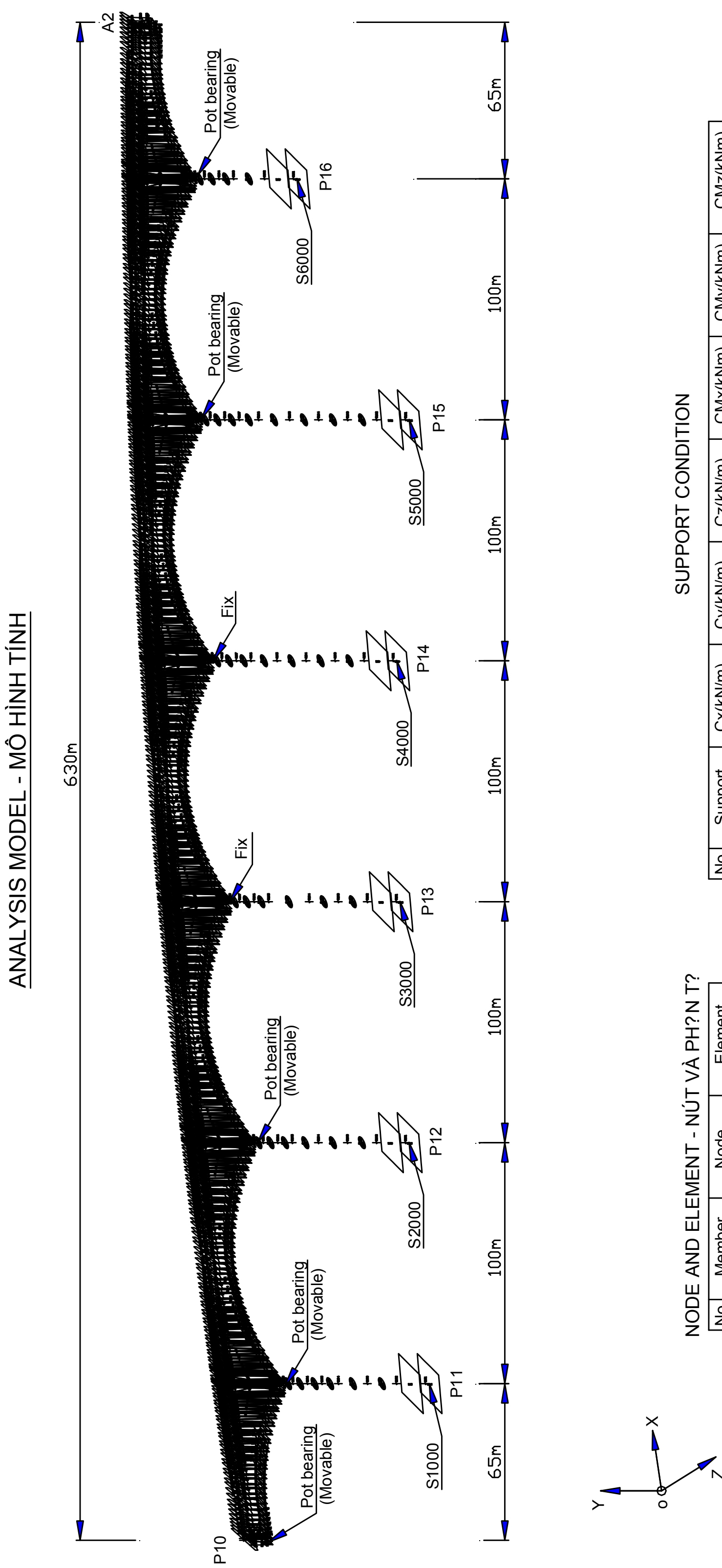
SUPPORT CONDITION

No.	Support	Cx(kN/m)	Cy(kN/m)	Cz(kN/m)	CMx(kNm)	CMy(kNm)	CMz(kNm)
01	S1000	16500000	636000	639000	43800000	96800000	85400000
02	S2000	16500000	637000	640000	43900000	96700000	85400000
03	S3000	16500000	637000	640000	44000000	95000000	83300000
04	S4000	16500000	637000	640000	44000000	95000000	83300000
05	S5000	16500000	496000	499000	34100000	89400000	76600000
06	S6000	16500000	886000	890000	61600000	103000000	93200000

NODE AND ELEMENT - NÚT VÀ PH?N T?

No.	Member	Node	Element
01	Box Girder	100 - 303	100 - 302
02	Pier P11	1101 - 1108	1101 - 1107
03	Pier P12	2101 - 2108	2101 - 2107
04	Pier P13	3101 - 3108	3101 - 3107
05	Pier P14	4101 - 4109	4101 - 4108
06	Pier P15	5101 - 5110	5101 - 5109
07	Pier P16	6101 - 6106	6101 - 6105

3.1.1 Analysis Model*(See below image and AppendixAnalysis Model)*



SUPPORT CONDITION

No.	Support	Cx(kN/m)	Cy(kN/m)	Cz(kN/m)	CMx(kNm)	CMy(kNm)	CMz(kNm)
01	S1000	16500000	636000	639000	43800000	96800000	85400000
02	S2000	16500000	637000	640000	43900000	96700000	85400000
03	S3000	16500000	637000	640000	44000000	95000000	83300000
04	S4000	16500000	637000	640000	44000000	95000000	83300000
05	S5000	16500000	496000	499000	34100000	89400000	76600000
06	S6000	16500000	886000	890000	61600000	103000000	93200000

NODE AND ELEMENT - NÚT VÀ PH?N T?

No.	Member	Node	Element
01	Box Girder	100 – 303	100 – 302
02	Pier P11	1101 – 1108	1101 – 1107
03	Pier P12	2101 – 2108	2101 – 2107
04	Pier P13	3101 – 3108	3101 – 3107
05	Pier P14	4101 – 4109	4101 – 4108
06	Pier P15	5101 – 5110	5101 – 5109
07	Pier P16	6101 – 6106	6101 – 6105

3.1.2 ***Thông số mặt cắt ngang** (Xem Phụ lục, thông số mặt cắt ngang)*

3.2 *Tải trọng thiết kế*

- Các tải trọng dưới đây được xem xét khi tính toán trong thi công:
  + Trọng lượng bản thân đốt dầm.
  + Trọng lượng bê tông ướt.
  + Trọng lượng xe đúc, ván khuôn.
  + Lực dự ứng lực.
  + Tác động của co ngót và từ biến trong thi công.
  + Gió tác động lên đốt dầm và trụ.

- Các tải trọng dưới đây cần xem xét thêm trong giai đoạn khai thác:
  + Tác động của co ngót và từ biến trong giai đoạn khai thác.
  + Tĩnh tải bổ sung (lớp phủ mặt cầu, lan can, và các công trình phụ trợ khác).
  + Tác động của việc thay đổi nhiệt độ (tăng, giảm nhiệt độ và gradient nhiệt tại mặt cắt ngang dầm).
  + Tác động của lún trụ.
  + Hoạt tải.
  + Gió tác động lên kết cấu và xe cộ.

3.2.1 *Tải trọng bản thân của kết cấu: 100*

- Chương trình tự động tính toán về trọng lượng bản thân dầm thông qua kích thước hình học và trọng lượng đơn vị được khai báo.
  + Tải trọng tổng cộng do tĩnh tải : 100
  + Tương ứng trong giai đoạn thi công thứ i là: 10i (i= 1,2,3….)
  + Tĩnh tải bản thân của lan can : 12.25kN/m (cho mỗi bên).

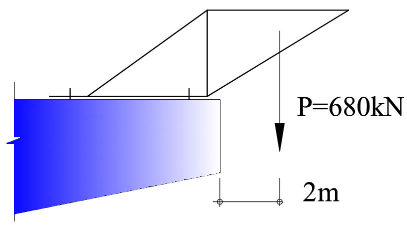
3.2.2 *Tĩnh tải của ván khuôn và xe đúc: LC200*

- Trọng lượng ván khuôn và xe đúc được mô tả bằng lực thẳng đứng P vàmomen M:

P = 680 kN, M =1360 kN.m.

(Tính toán với tổng trọng lượng ván khuôn, xe đúc là 680kN)

- Tải trọng tổng cộng do ván khuôn, xe đúclà : 200
- Tương ứng trong giai đoạn thi công thứ i là: 20i (i= 1,2,3….)



Các thiết bị thi công khác (như: cần cẩu, v.v.) hoặc vật liệu để trên mặt cầu như thiết bị căng kéo dự ứng lực ...không bao gồm trong tải trọng xe đúc, ván khuôn và phải được đặt tại các điểm gần đỉnh trụ khi đo đạc độ vồng và đường khống chế cao độ thi công. Các tải trọng này không được sử dụng trong quá trình tính toán kết cấu cầu.

3.2.3 *Trọng lượng bê tông ướt: LC300*

Chương trình tự động tính toán trọng lượng bê tông ướt của các đốt dầm thông qua kích thước hình học và trọng lượng đơn vị được khai báo. Trọng lượng đơn vị của bê tông ướt: γ_c = 24.5kN/m^3.

Trọng lượng bê tông ướt tác dụng qua ván khuôn và xe đúc lên phần tử dầm đã đúc trước đó và điểm tác dụng đặt tại điểm ngoài cùng của mép ngoài khối đã đúc trước đó:

- Tải trọng tổng cộng do bê tông ướt : LC300
- Tương ứng trong giai đoạn thi công thứ i là: LC30i (i =1,2,3…).

3.2.4 *Lực kéo căng cáp dự ứng lực: LC500*

- Kết cấu dầm hộp sử dụng cáp dự ứng lực 6-19 (19tao 15.2mm).
- Lực kích cho mỗi bó 3711 kN.

3.1.2 Cross section Properties(See Appendix: Cross section Properties)

3.2 Design Loads

- The following loads are considered for calculation in the construction:
  + Self weight of girder segment.
  + Self weight of wet concrete segment.
  + Traveller weight, formwork weight.
  + Tensile force of prestressed cable.
  + Effect of shrinkage and creep in the construction.
  + The wind affecting to girder segment and pier.
- The following loads are further considered in the service stage:
  + Effect of shrinkage and creep in the service stage.
  + Additional static weight (wearing coast, parapet, and other auxiliary structures).
  + Effect of temperature change (increasing tempertature, decreasing temperature andgradient of temperature in the girder cross section).
  + Effect of settlement of pier.
  + Live load.
  + Wind affecting to the structure and Live load.

3.2.1 Dead load of structural components: 100

- Automatic calculation program of self weight of girder through geometric dimension and volume weight which are set up.
  + Total load due to modeled dead load : 100
  + Accordingly, in the ith construction : 10i (i= 1,2,3….)
  + Self weight of Parapet :12.25kN/m (for each side).

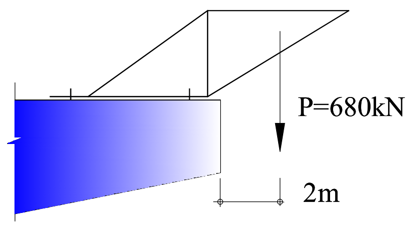
3.2.2 Dead load of formwork andtraveler: LC200

- Formwork andtraveller is modeled in a vertical force P and a moment M:

P = 680 kN, M =1360 kN.m.

(Supposing that the weight of formwork, form traveller is 680kN)

- Total load due to formwork, form traveller is named : 200
- Accordingly, in the ith construction : 20i (i= 1,2,3….)



Other construction equipments (such as : crane, etc.) or materials located on bridge deck such as prestressed tensioning equipment ...excluding the load of form traveller, formwork must be arranged near the pier peak when measuring and elevation control line. These loads are not used for the calculation of bridge structure.

3.2.3 Self weight of wet concrete segments: LC300

The program automatic calculation the self weightwet concrete segments which have just been casted through geometric dimension and volume weight are set up. The volume weight of wet concrete: $\gamma_c = 24.5kN/m^3$.

The self weight of wet concrete effecting to the element which was previously casted through the formwork and traveller and point of application is located at outermost point of outer edge of the previous casted mass:

- Total load due to wet concrete is named : LC300
- Accordingly, in the ith construction: LC30i (i =1,2,3…).

3.2.4 Tensile force ofprestressed cable: LC500

- Box girder structure is used prestressed cable 6-19 (19 strands 15.2mm).
- Tensile force for per tendon 3711 kN.

- Các giá trị về mất mát ứng suất trong quá trình thi công và khai thác được chương trình tự động tính toán, bao gồm:
  + Do ma sát giữa cáp dự ứng lực và đường ống cáp.
  + Do tụt đầu neo.
  + Do biến dạng đàn hồi bê tông.
  + Do co ngót bê tông.
  + Do từ biến.
  + Do tự chùng thép dự ứng lực.
- Tải trọng tổng cộng do dự ứng lực : LC500
- Tương ứng trong giai đoạn thi công thứ i là : LC50i (i =1,2,3…).

3.2.5 *Co ngót và từ biến CR &SH: LC600*

- Co ngót và từ biến được chương trình tự động tính toán theo quy trình CEB-FIP 90
- Tải trọng tổng cộng do co ngót và từ biến : LC600
- Tương ứng trong giai đoạn thi công thứ i là : LC60i (i =1,2,3…).
- Trong quá trình khai thác, co ngót và từ biến được tính toán sau 30 năm khi thác sử dụng.

3.2.6 *Tĩnh tải lớp phủ mặt cầu: DW*

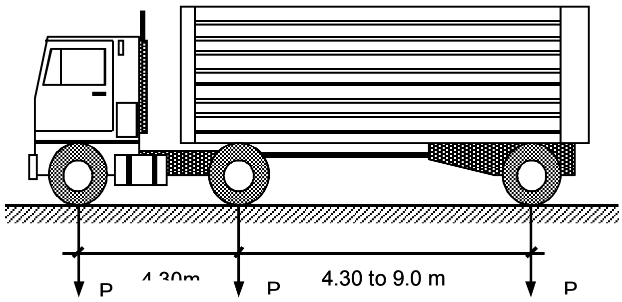
- Tải trọng bản thân của lớp phủ mặt cầu(bê tông atphan 8.4cm) :$\gamma = 22.0kN/m^3$.
- Tính toán tải với tải trọng các cấu kiện khác là 2 Kn/m (cột đèn chiếu sáng, ống thoát nước…)

3.2.7 *Hoạt tải và tải trọng xung kích: LL & IM*

- Chiều rộng phần xe chạy: W=12.0m
- Số làn xe thiết kế: n=3 lanes.
- Hệ số làn xe: m=0.85.
- Hoạt tải thiết kế : HL-93
  + Xe tải thiết kế và tải trọng làn
  + hoặc xe hai trục thiết kế và tải trọng làn

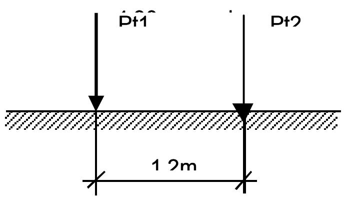
(1) Xe tải thiết kế:

- Hệ số xung kích lấy theo Điều 7.3.5.3.2 - 22TCN272-05 bằng 0.25

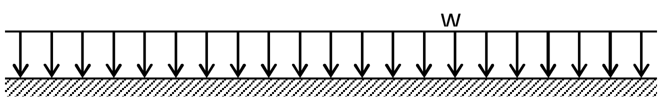


(2) Xe hai trục thiết kế

- Xe hai trục gồm một cặp trục 110kN cách nhau 1,2m. Cự ly theo chiều ngang của các bánh xe lấy bằng 1.8m, tổng trọng lượng xe là: 220kN.
- Hệ số xung kích lấy theo điều 7.3.5.3.2-22TCN272-05 bằng 0.25



(3) Tải trọng làn thiết kế



- Ảnh hưởng của tải trọng làn thiết kế không xét lực xung kích do hoạt tải.

3.2.8 *Tải trọng hãm xe: BR*

- Tải trọng hãm xe tính bằng 25% của trọng lượng các trục xe tải hay xe hai trục thiết kế

- Values of stress loss during the construction and service stage are calculated by automatic program, including:
  + Friction between prestressing tendon and duct.
  + Anchorage seating or set.
  + Elastic deformation of concrete.
  + Shrinkage of concrete.
  + Creep of concrete.
  + Relaxation of prestressing steel.
- Total load due Prestressing force to modeled : LC500
- Accordingly, in the construction : LC50i (i =1,2,3…).

3.2.5 Shrinkage and creep CR &SH: LC600

- Shrinkage and creep are calculated by automatic program under the procedure CEB-FIP 90
- Total load due to modeled shrinkage and creep : LC600
- Accordingly, in the construction : LC60i (i =1,2,3…).
- In the service stage, shrinkage and creep are calculated after 30 years of service.

3.2.6 Dead load of Pavement: DW

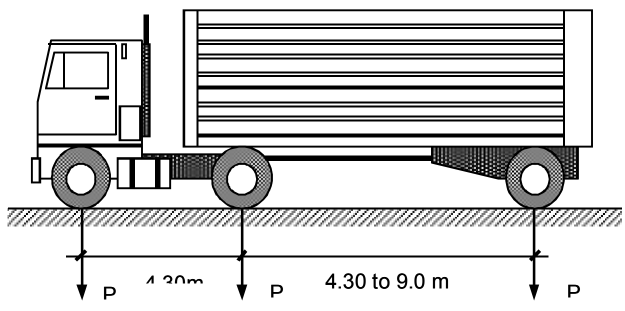
- Self weight of pavement (8.4cm asphaltwaterproof) : $\gamma = 22.0kN/m^3$.
- Miscellaneous dead load: assumed 2 Kn/m for seftweight of traffic light and drainage pipe…

3.2.7 Vehicular live load& vehicular dynamic load: LL & IM

- Carriage-way width: W=12.0m
- Number of design lanes : n=3 lanes.
- Multiple presence factors : m=0.85.
- Vehicular live load : HL-93
  + Design truck and lane load
  + or design tandem and lane load

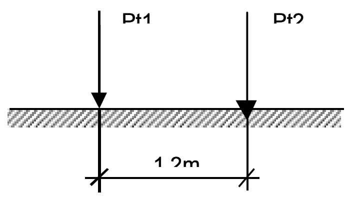
(1) Design truck:

- Impact coefficient is extracted from the article 7.3.5.3.2 - 22TCN272-05 to be as 0.25

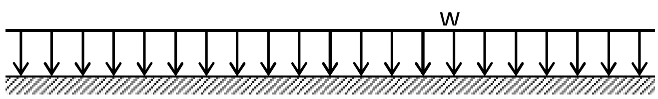


(2) Design tandem

- The design tandem shall consist of a pair of 110kN axles spaced 1.2m apart. The transverse spacing of wheels shall be taken as 1.8m, total wieght of truck is: 220kN.
- Impact coefficient shall be considered as in Article 7.3.5.3.2-22TCN272-05 to be as 0.25



(3) Design lane load



- The force effects from the design lane load shall not be subject to a dynamic load allowance.

3.2.8 Vehicular braking force: BR

- Breaking force shall be taken as 25% of the axle weights of the design truck or tandem cho mỗi làn được đặt trong tất cả các làn thiết kế, lực hãm được coi là tác dụng nằm ngang cách mặt cầu 1.8m.

+ Số làn xe thiết kế: n = 3 lanes.
+ Trọng lượng 1 xe tải : P_{truck}= 325 kN.
+ Lực hãm đối với 3 làn xe tải: BR_{Truck}= 243.75 kN.
+ Trọng lượng xe 2 trục: P_{tandem}= 220 kN.
+ Lực hãm đối với 3 làn xe hai trục: BR_{Tandem}= 165 kN.

➔ Lực hãm lựa chọn tính toánBR = 243.75 kN.

3.2.9 *Tải trọng nước và áp lực dòng chảy: WA*

- Theo chiều dọc: Áp lực nước chảy theo chiều dọc của kế cấu phần dưới tính bằng:

$P = 5.14 \times 10^{-4} C_d V^2 = 5.14\text{x}10^{-4}\text{x}0.7\text{x}3.44^2 = 0.0043\text{MPa} = 4.3\text{kN/m}^2$

+ P –áp lực của nước chảy (MPa)
+ C_d – hệ số cản đối với các trụ được trình bày trong Bảng 3.7.3.1.1– 22TCN272-05

Đối với trụ mũi hình bán nguyệt C_d = 0.7.

+ V –tốc độ nước chảy đối với lũ thiết kế,V = 3.44m/s

- Theo chiều ngang: Theo chiều ngang áp lực phân bố đồng đều trên kết cấu phần dưới do dòng nước tại góc θ, với trụ thẳng đứng trụ được tính theo:

$P = 5.14 \times 10^{-4} C_L V^2 = 5.14\text{x}10^{-4}\text{x}1\text{x}3.44^2 = 0.0061\ \text{MPa} = 6.1\text{kN/m}^2$

+ P –áp lực theo chiều ngang của nước chảy (MPa).
+ C_L – hệ số cản theo phương ngangđược trình bày trong bảng 3.7.3.2.1– 22TCN272-05,

for $\theta \geq 30^oC_L = 1.0$.

3.2.10 *Tải trọng gió lên kết cấu: WS*

- Tốc độ gió thiết kế : $V = V_B \text{ x } S$
  + Cầu Kỳ Lam nằm trong vùng gió cấp III có vận tốc gió giật cơ bản là V_B = 53m/s (TCVN 2737-1995)
  + S –hệ số điều chỉnh đối với vùng gió và độ cao mặt cầu, tra bảng S = 1.14

  V = 53x1.14 = 60.42 m/s
- Tải trọng gió ngang : $P_D = 0.0006\ V^2\ A_t\ C_d$ (kN)
  + A_t–diện tích kết cấu đối với việc tính tải trọng gió ngang, (tính với trụ không xét At)
  + C_d–Hệ số cản phụ thuộc vào tỉ số b/d.
  + b –chiều rộng toàn bộ cầu giữa các bề mặt lan can, b= 13.0m
  + d –chiều cao của kết cấu gồm cả lan can, d = (2.5+6.0)/2+1.2 = 5.45m
  + b/d = 13.0/5.45 = 2.39, dựa vào sơ đồ 3.8.1.2.1.1 22TCN 272 - 05, C_d = 1.50

  $P_D = 0.0006 \text{ x } 60.42^2 \text{x } 1.5 = 3.29\ (\text{kN/m}^2)$
- Tải trọng gió dọc: lấy bằng 25% tải trọng gió ngang: 0.25x3.29=0.82(kN/m^2).

3.2.11 *Tải trọng gió lên hoạt tải: WL*

Tải trọng gió lên hoạt tải: Khi xét tổ hợp cường độ III, phải xét tải trọng gió trên cả kết cấu và xe cộ, áp lực gió tác động lên xe cộ sẽ là tải trọng phân bố 1.50 kN/m tác dụng thẳng góc, phía trên mặt đường 1.80m và truyền vào kết cấu.

3.2.12 *Tải trọng nhiệt độ: TU & TG*

(1) Nhiệt độ thay đổi đều: TU

- Nhiệt độ tham chiếu : T_{tc} = 25^{o}C.

per lane placed in all design lanes, the breaking forces shall be assumed to act horizontally at a distance of 1.8m above the roadway surface.

+ Number of design lanes : n = 3 lanes.
+ Weight of 1 truck : P_{truck}= 325 kN.
+ Breaking force for 3 lanes of truck: BR_{Truck}= 243.75 kN.
+ Weight of tandem : P_{tandem}= 220 kN.
+ Breaking force for 3 lanes of tandem : BR_{Tandem}= 165 kN.
  ➔ Braking force is calculated BR = 243.75 kN.

3.2.9 Water load and Stream pressure: WA

- Longitudinal: The pressure of flowing water acting in the longitudinal direction of substructures shall be takens as:
  $P = 5.14 \times 10^{-4} C_d V^2 = 5.14\text{x}10^{-4}\text{x}0.7\text{x}3.44^2 = 0.0043\text{MPa} = 4.3\text{kN/m}^2$
  + P - pressure of flowing water (MPa)
  + C_d – drag coefficient for piers as specified in Table 3.7.3.1.1– 22TCN272-05,

for semicircular-nosed pier C_d = 0.7.

  + V - velocity of water for design flood,V = 3.44m/s

- Lateral: The lateral, uniformly distributed pressure on a substructure due to water flowing at an angle θ, to the longitudinal axis of the pier shall be takens as:
  $P = 5.14 \times 10^{-4} C_L V^2 = 5.14\text{x}10^{-4}\text{x}1\text{x}3.44^2 = 0.0061\text{ MPa} = 6.1\text{kN/m}^2$
  + P –lateral pressure of flowing water (MPa).
  + C_L – lateral drag coefficient for piers as specified in Table 3.7.3.2.1– 22TCN272-05,

for $\theta \geq 30^oC_L = 1.0$.

3.2.10 Wind load on structure: WS

- Design wind speed : $V = V_B \text{ x } S$
  + Ky Lam bridge is in the area of grade III wind with a basic gust as V_B = 53m/s (TCVN 2737-1995)
  + S - adjustment coefficient for wind area and bridge deck height, see the Table S = 1.14

  V = 53x1.14 = 60.42 m/s
- Horizontal wind load : $P_D = 0.0006\ V^2\ A_t\ C_d$ (kN)
  + A_t - area of the structure for calculation of transverve wind load, (for pier without considering At)
  + C_d - drag coefficient depending on ratio b/d.
  + b - overall width of bridge between outer faces of parapets, b= 13.0m
  + d - depth of superstructure, including solid parapets, d = (2.5+6.0)/2+1.2 = 5.45m
  + b/d = 13.0/5.45 = 2.39, base on chart 3.8.1.2.1.1 22TCN 272 - 05, C_d = 1.50

  $P_D = 0.0006 \text{ x } 60.42^2 \text{x } 1.5 = 3.29\ (\text{kN/m}^2)$
- Longitudinal wind load:Shall be taken as 25% horizontal wind load: 0.25x3.29=0.82(kN/m^2).

3.2.11 Wind on live load: WL

- Wind load on live load: When considering the strength III load combination, the design shall be applied to both structure and vehicles,wind force on vehicles shall be a line load of 1.50 kN/m acting horizontally, and 1.80m above the roadway and shall be transmitted to the structure.

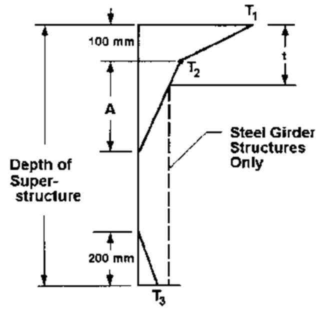
3.2.12 Temperature load: TU & TG

(1) Uniform temperature: TU

- Reference temperature : T_{tc} = 25^{o}C.

- Nhiệt độ cao nhất khu vực: $T_{max} = 47^oC$. (Article 3.12.2.1).
- Nhiệt độ thấp nhất khu vực: $T_{min} = 10^oC$. (Article 3.12.2.1)
- Nhiệt độ thay đổi đều : tăng : +22 ^oC ; giảm: -15^oC

(2) Gradient nhiệt:TG



Nhiệt độ dương   Nhiệt độ âm

T1 = +23^{o}C   T1 = -7.0^{o}C

T2 = +6.0^{o}C   T2 = -1.0^{o}C

T3 = +3.0^{o}C   T3 = -0.0^{o}C

3.2.13 Hiệu ứng lún: SE

- Hiệu ứng lún không đều được tự động tính toán bằng chương trình RM.

3.2.14 Tải trọng động đất: EQ

Hệ số gia tốc khu vực cầu Kỳ Lam là khá nhỏ. Vì vậy sau khi xem xét tính toán động đất theo 2 phương pháp: phương pháp phân tích tĩnh tương đương theo phổ dạng đơn và phân tích phổ dạng phức. Đề xuất sử dụng hiệu ứng theo đơn phổ để có thể xem xét hiệu ứng bất lợi nhất có thể xảy ra cho công trình trong quá trình chịu động đất. Ảnh hưởng động đất của kết cấu phần trên đối với một nguyên đơn cầu được tính chuyển về hệ móng bằng cách nhân đôi giá trị các thành phần lực.

- Vùng động đất: vùng II.
- Hệ số gia tốc A = 0.0341 (TCXDVN 375:2006).
- Hệ số thực địa: S = 1.20 (điều 3.10.5.1.1).
- Trọng lượng bản thân của kết cấu phần trên tác động (đối với 1m): Wt = 257.77 kN/m.
- Chiều dài kết cấu phần trên ảnh hưởng đến trụ, L = 630.0m.
- Tải trọng đều phân bổ trên nhịp : P_o = 1.00 kN/m.
- Xác định độ cứng của thân trụ: Kx(y) = 3.E.Ix(y)/H^3.
- Biến dạng kết cấu do P_o: Vsx(y) = P_o.L/Kx(y).

H	E_c	I_x	I_y	K_x	K_y	V_{sx}	V_{sy}
(m)	(MPa)	(m^4)	(m^4)	(KN/m)	(KN/m)	(mm)	(mm)
15.0	29440.09	12.566	12.566	1.32E+06	1.32E+06	0.2395	0.2395

Trong đó :

E_c : Mô đun đàn hồi của bê tông.

I_x, I_y : Momen quán tính mặt cắt ngang trụ theo trục X và trục Y.

H : Chiều cao ảnh hưởng của trụ.

- Tính các hệ số α , β và γ bằng công thức:

$$\alpha = \int v_s(x)dx \qquad \beta = \int w(x)v_s(x)dx \qquad \gamma = \int w(x)v_s^2(x).dx$$

Trong đó: $w(x)$: Tĩnh tải của kết cấu phần trên và tải trọng bổ sung của kết cấu phần dưới.

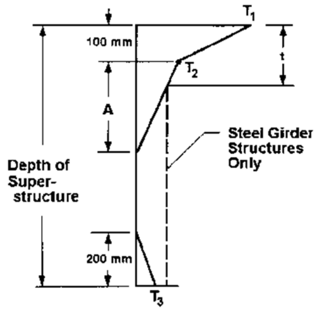
- Tính chu kỳ dao động T_m bằng công thức: $T_m = 2\pi\sqrt{\frac{\gamma}{p_0 g.\alpha}} \leq 4$(s)

Kết quả tính toán theo hướng dọc và hướng ngang

α_x	α_y	β_x	β_y	γ_x	γ_y	T_x	T_y	**$T_x \leq 4$(s)**	**$T_y \leq 4$(s)**
(m^2)	(m^2)	(KNm)	(KNm)	(KN/m^2)	(KN/m^2)	(s)	(s)	(s)	(s)
0.1509	0.1509	38.889	38.889	9.31E-03	9.31E-03	0.498	0.498	**0.498**	**0.498**

- Maximum temperature of the area: T_{max} = 47°C. (Article 3.12.2.1).
- Minimum temperature : T_{min} = 10°C. (Article 3.12.2.1)
- Uniform temperature : increasing : +22 °C ; decreasing: -15°C

(2) Temperature gradient:TG



Positive temperature Negative temperature

T1 = +23°C T1 = -7.0°C

T2 = +6.0°C T2 = -1.0°C

T3 = +3.0°C T3 = -0.0°C

3.2.13 Settlement: SE

- The effect of unequal settlement was automatic calculated by RM Program.

3.2.14 Seismic load: EQ

Hệ số gia tốc khu vực cầu Kỳ Lam là khá nhỏ. Vì vậy sau khi xem xét tính toán động đất theo 2 phương pháp: namely uniform load method followed single-mode method and multi-mode spectral method. The single-mode method proposed to be able to consider the most adverse effects can occur during earthquake. Earthquake effect of superstructure for one bridge section is estimated to be moved into foundation system by doubling the value of force components.

The acceleration coefficient at Ky Lam bridge area is quite small. So after considering the earthquake force which calculated by two methods:

- Seismic Zone : Zone II.
- Acceleration coefficient A = 0.0341 (TCXDVN 375:2006).
- Site coefficient: S = 1.20 (article 3.10.5.1.1).
- Self weigh of Super structure have affect (for 1m): Wt = 257.77 kN/m.
- The length of the superstructure have affect to the pier that is considered, L = 630.0m.
- Uniform load distributed over span : P_o = 1.00 kN/m.
- Determination of stiffness of pier shaft: $Kx(y) = 3.E.Ix(y)/H^3$.
- Deformation of structural due to P_o: $Vsx(y) = P_o.L/Kx(y)$.

H	**E_c**	**I_x**	**I_y**	**K_x**	**K_y**	**V_{sx}**	**V_{sy}**
(m)	(MPa)	(m^4)	(m^4)	(KN/m)	(KN/m)	(mm)	(mm)
15.0	29440.09	12.566	12.566	1.32E+06	1.32E+06	0.2395	0.2395

Where :

E_c : Elastic modulus of concrete.

I_x, I_y : Inertia moment of pier cross section follow axis X and axis Y.

H : Effective high of pier.

- Calculation coefficients α , β và γ by formula:

$$\alpha = \int v_s(x)dx \qquad \beta = \int w(x)v_s(x)dx \qquad \gamma = \int w(x)v_s^2(x).dx$$

Where: $w(x)$: Dead load of superstructure and addition load of sub structure.

- Calculation period of vibration T_m by formula: $T_m = 2\pi\sqrt{\frac{\gamma}{p_0 g.\alpha}} \leq 4(s)$

Result of calculation follow longitudinal direction and transver direction

α_x	**α_y**	**β_x**	**β_y**	**γ_x**	**γ_y**	**T_x**	**T_y**	**$T_x \leq 4(s)$**	**$T_y \leq 4(s)$**
(m^2)	(m^2)	(KNm)	(KNm)	(KN/m^2)	(KN/m^2)	(s)	(s)	(s)	(s)
0.1509	0.1509	38.889	38.889	9.31E-03	9.31E-03	0.498	0.498	**0.498**	**0.498**

Trong đó:

T_x, T_y : Chu kỳ dao động theo hướng X và hướng Y.

g: gia tốc trọng lực(m/s^2).

Sxd(x) = L : Chiều dài kết cấu phần trên(m).

- Hệ số đáp ứng động đất đàn hồi bằng công thức: $C_{sm} = \frac{1.2AS}{T_m^{2/3}} \leq 2.5A$

- Tải trọng động đất tương đương $p_e(x)$ tính bằng công thức : $p_e(x) = \frac{\beta.C_{sm}}{\gamma} w(x) v_s(x)$

Kết quả

C_{xsm}	C_{ysm}	C_{xsm}	C_{ysm}	$P_{e(x)}$	$P_{e(y)}$
Lý thuyết		So sánh với 2.5A và lựa chọn		(KN/m)	(KN/m)
0.097	0.097	0.085	0.085	**21.91**	**21.91**

- Tổ hợp ứng lực động đất: EQ_1: $EQ_{longitudinal}$ + 30%$EQ_{transver}$

EQ_2: $EQ_{transver}$+ 30%$EQ_{longitudinal}$

3.2.15 *Tải trọng va tàu : CV*

Ảnh hưởng của lực va tàu được tính cho một tàu va vào một trụ để xét cho một đơn nguyên cầu. Ảnh hưởng lực va tàu từ một nguyên đơn cầu này được tính chuyển về hệ móng bằng cách dời thành phần lực nằm ngang và mô men uốn về trọng tâm hệ móng.

- Tính theo tiêu chuẩn thiết kế cầu 22-TCN272-05, trọng lượng và kích thước tàu như sau:

Tấn tải trọng	Tàu tự hành	Xà lan kéo
DWT	200	400

- Lực va tàu trên trụ:

Kích thước tàu

	Tàu tự hành	Xà lan kéo
Chiều dài lớn nhất (m)	34.00	41.00
Bề rộng lớn nhất (m)	6.60	11.20
Mớn nước đầy tải (m)	1.70	1.30

- Vận tốc va thiết kế :

Tàu tự hành	V = 2.5 + Vs = 3.65 m/s
Xà lan kéo	V = 1.6 + Vs = 2.75 m/s

Vs : vận tốc trung bình hàng năm = 1.15 m/s.

- Lực va tàu tự hành trên trụ :
  $Ps = 1.2 \times 10^5 V \sqrt{DWT} = 1.2 \times 10^5 \times 3.65 \times 200^{0.5} = 6194255$ N = 6194.26kN.

- Lực va xà lan kéo trên trụ:
  + Năng lượng va tàu: $KE = 500C_HMV^2 = 500 \times 1.05 \times 400 \times 2.75 = 577500$J
    M - trọng lượng chuyển vị tàu = 400 Mg.
    C_H–hệ số khối lượng thủy động = 1.05
  + Chiều dài hư hỏng mũi xà lan:
    $a_B = 3100(\sqrt{(1+1.3 \times 10^{-7}KE)} -1) = 305$ mm

Where:

T_x, T_y : Period of vibration follow direction X and direction Y.

g: acceleration of gravity (m/s^2).

Sxd(x) = L : Length of superstructure (m).

- Elastic seismic response coefficient, by formula: $C_{sm} = \dfrac{1.2AS}{T_m^{2/3}} \leq 2.5A$

- Equivalence seismic load $p_e(x)$ by formula : $p_e(x) = \dfrac{\beta.C_{sm}}{\gamma} w(x)v_s(x)$

Result

C_{xsm}	C_{ysm}	C_{xsm}	C_{ysm}	$P_{e(x)}$	$P_{e(y)}$
Theory		Compare with 2.5A and choise		(KN/m)	(KN/m)
0.097	0.097	0.085	0.085	**21.91**	**21.91**

- Combination of seismic force effects: EQ_1: $EQ_{longitudinal}$ + 30%$EQ_{transver}$
  EQ_2: $EQ_{transver}$+ 30%$EQ_{longitudinal}$

3.2.15 Vessel collision load : CV

The effect of Vessel collision force is estimated to a vessel collides against a pier shaft and consided for a bridge section. This effect will be consided to foundation system by shifting force and flexural moment to central of foundation.

- Shall be calculated under the specification for bridge design 22-TCN272-05, weight and dimension of design vessel as follows:

Design vessel tonnage	Self-propelled vessel	Towed barge
DWT	200	400

- Vessel collision force on pier:

Dimension of design vessels

	Self-propelled vessel	Towed barge
Maximum length (m)	34.00	41.00
Maximum breadth (m)	6.60	11.20
Laden draught (m)	1.70	1.30

- Design collision velocity :

Self-propelled vessel	V = 2.5 + Vs = 3.65 m/s
Towed barge	V = 1.6 + Vs = 2.75 m/s

Vs : mean annual velocity = 1.15 m/s.

- Self-propelled vessel collision force on pier :
  $Ps = 1.2 \times 10^5 V \sqrt{DWT} = 1.2x10^5x3.65x200^{0.5}$ = 6194255 N = 6194.26kN.

- Towed barge collision force on pier:
  + Vessel collision energy: $KE = 500C_HMV^2$ = 500x1.05x400x2.75 = 577500J
    M - vessel displacement tonnage = 400 Mg.
    C_H - hydrodynamic mass coefficient = 1.05
  + Barge bow damage length:
    $a_B = 3100(\sqrt{(1+1.3x10^{-7}KE)} -1)$ = 305 mm

+ Lực va, N, vào trụ do xà lan chạy trên sông được tính như sau:

$P_B = 6.0x104a_B$ nếu a_B<100 mm

$P_B = 6.0x10^6 + 1600\ a_B$ nếu a_B>100 mm

➔ Chọn $P_B = 6.0x10^6 + 1600\ a_B = 6488000N = 6488.00kN$

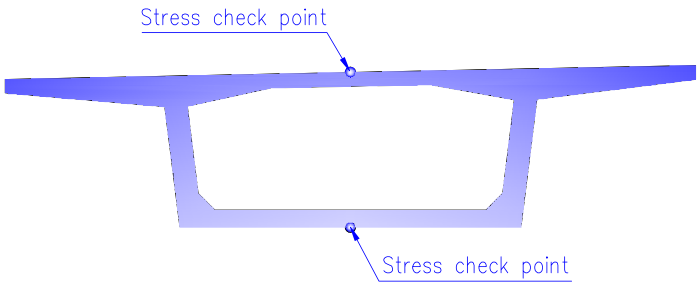
- Kết luận :Trên cơ sở so sánh lực va giữa tàu tự hành và xà lan kéo, lựa chọn lực va tàu trên trụ như sau:
  + Theo hướng vuông góc với tim cầu: P = 100% P_B = 6488.0 kN
  + Theo hướng song song với tim cầu: P = 50 % P_B = 3244.0kN

3.3 *Nội lực mặt cắt (xem Phụ lục: Nội lực mặt cắt)*

3.4 *Thiết kế cho dầm hộp*

3.4.1 *Kiểm tra ứng suất thớ dầm*

Dùng mô đun FIBRE STRESS CHECKcủa chương trình RM để kiểm tra ứng suất tại thớ dầm. Kết quả kiểm tra được trình bày trong sơ đồ của phần Phụ lục. Kết quả tính toán thể hiện ở đây là tại 2 điểm sau: (xem hình dưới)



Tổng hợp kiểm tra ứng suất

Tổ hợp	Ứng suất lớn nhất–nhỏ nhất (kN/m^2)		Giới hạn ứng suất (kN/m^2)		Kết luận
	σ_t	σ_c	$[\sigma_t]$	$[\sigma_c]$	
Giai đoạn thi công	1308	-13810	3180	-20250	OK
TTGH sử dụng I	1146	-13515	3354	-20250	OK
TTGH sử dụng III	1619	-13291	3354	-20250	OK

3.4.2 *Kiểm tra tải trọng cực hạn:*

Sử dụng mô đun ULTIMATE CHECK của chương trình để kiểm tra tải trọng cực hạn. Kết quả kiểm tra được trình bày trong sơ đồ của phần Phụ lục.

+ M_{max}, M_{min}momen lớn nhất, momen nhỏ nhất tại trụ theo kết quả cường độ.

+ ϕM_{max} , ϕM_{min}momen lớn nhất, momen nhỏ nhất tại mặt cắt ngoài trụ.

Bảng 3.4.2Tổng hợp kiểm tra tải trọng cực hạn

Tổ hợp	M_{u_max}	M_{u_min}	ϕM_{n_max}	ϕM_{n_min}	Kết luận
	(kNm)	(kNm)	(kNm)	(kNm)	
TTGH cường độ	138927	-450391	193593	-861307	Ok

3.4.3 *Kiểm tra khả năng chịu cắt.*

Sử dụng chương trình mô đunSHEAR CAPACITY CHECK để kiểm tra khả năng chịu cắt. Kết quả kiểm tra được trình bày trong sơ đồ của phần Phụ lục.

Bảng 3.4.3 Tổng hợp kiểm tra khả năng chịu cắt

Tình trạng	V_{max}	φV_n	Kết luận
	(kN)	(kN)	
Trạng thái giới hạn	27804	103039	Ok

+ The collision impact force, N, on a pier for a river-going barge shall be taken as:

$P_B = 6.0x104a_B$ if a_B<100 mm

$P_B = 6.0x10^6 + 1600\ a_B$ if a_B>100 mm

➔ Selecting $P_B = 6.0x10^6 + 1600\ a_B$ = 6488000N = 6488.00kN

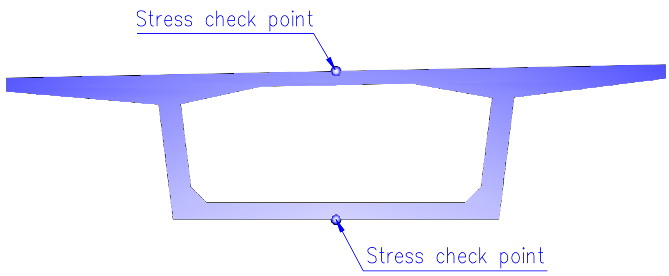
- Conclusion :On the basis of comparison of collision force between self-propelled vessel and towed barge, selection of vessel collision force on pier as follow:
  + As perpendicular direction with bridge centreline: P = 100% P_B = 6488.0 kN
  + As perpendicular direction with bridge centreline: P = 50 % P_B = 3244.0kN

3.3 Sectional force (see Appendix: Sectional Force)

3.4 Design for Box Girder

3.4.1 FibreStress Check

The modulus program FIBRE STRESS CHECK is used to check on fibre stress. The results of checking is shown in diagrams in the Appendix.The calculation result shown in this document is based on 2 checking points: (see the image)



Summaryof Fibre stress check

Combination	Stress max - min (kN/m^2)		Limitof stress (kN/m^2)		Conclusion
	σ_t	σ_c	[σ_t]	[σ_c]	
Contruction stage	1308	-13810	3180	-20250	OK
Service I limit state	1146	-13515	3354	-20250	OK
Service III limit state	1619	-13291	3354	-20250	OK

3.4.2 Ultimate Load Check:

The modulus program ULTIMATE CHECK is used to check on Ultimate load. The results of checking is shown in diagrams in the Appendix.

+ M_{max}, M_{min} maximum moment, minimum moment at per section under the result of strength.

+ ϕM_{max} , ϕM_{min} minimum and maximum extreme section moment at per section.

Table 3.4.2Summary ofUltimate load Check

Status	M_{u_max}	M_{u_min}	ϕM_{n_max}	ϕM_{n_min}	Conclusion
	(kNm)	(kNm)	(kNm)	(kNm)	
Strength limit state	138927	-450391	193593	-861307	Ok

3.4.3 Shear Capacity Check.

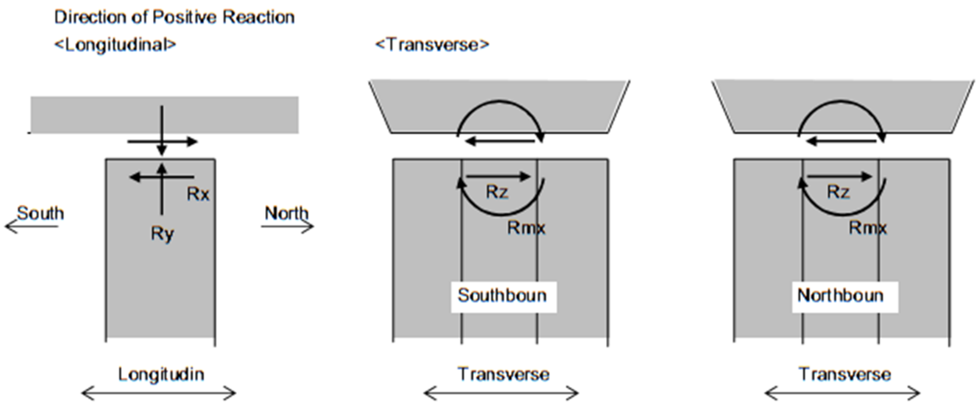
The modulus program SHEAR CAPACITY CHECK is used to check on shear capacity. The results of checking is shown in diagrams in the Appendix.

Table 3.4.3 Summary of Shear capacity Check

Status	V_{max}	φV_n	Conclusion
	(kN)	(kN)	
Strength limit state	27804	103039	Ok

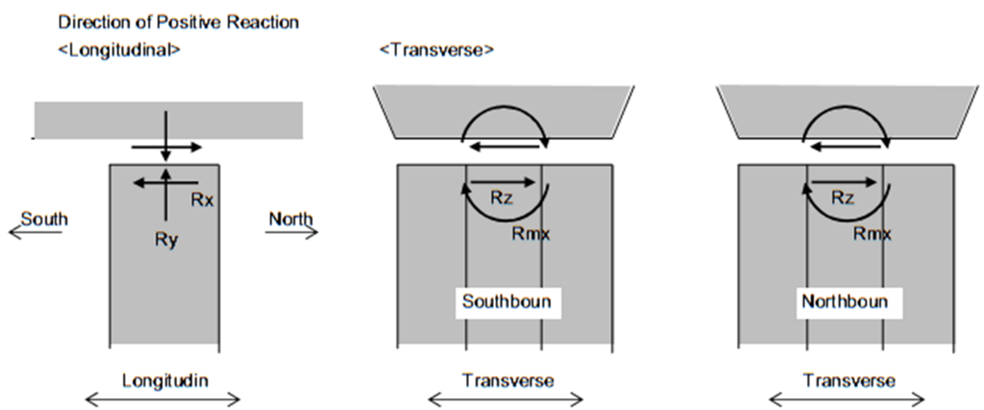
3.5 Phản lực gối tựa

Load	Symbol		P10				P11				P12			
			Rx	Ry	Rz	Rmx	Rx	Ry	Rz	Rmx	Rx	Ry	Rz	Rmx
			(kN)	(kN)	(kN)	(kNm)	(kN)	(kN)	(kN)	(kNm)	(kN)	(kN)	(kN)	(kNm)
Dead Load	DC			4499.07				29016.99				29665.1		
Super-imposed Load	DW			401.564				2177.903				2321.198		
Creep & Shrinkage	CR & SH		-4.797	-87.824			4.564	111.385			0.613	-159.879		
Permanent Load Summary	DC+DW+CR+SH		**-4.797**	**4812.81**			**4.564**	**31306.28**			**0.613**	**31826.42**		
Live Load	LL	MAX												
		MIN												
Live Load + Impact	LL+IM	MAX		1967.689				4629.642				4988.703		
		MIN		-703.306				-954.939				-1097.71		
Wind Load on Structure	WS	Longi.												
		Trans.			284.442				1056.852				1369.756	
Wind Load on Live Load	WL	Longi.												
		Trans.				-549.048								
Uniform Temperature	TU(+) +22deg													
	TU(-) -19deg													
Temperature Gradient	TG	+		282.358				106.86				179.744		
		-		-79.801				-380.731				-58.095		
Settlement	SE	MAX		99.951				154.489				260.533		
		MIN		-61.32				-229.489				-205.955		



3.5 Support Reaction

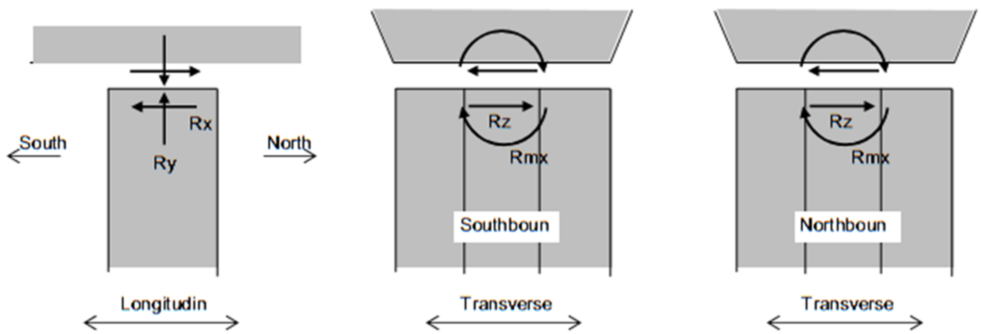
Load	Symbol		P10				P11				P12			
			Rx (kN)	Ry (kN)	Rz (kN)	Rmx (kNm)	Rx (kN)	Ry (kN)	Rz (kN)	Rmx (kNm)	Rx (kN)	Ry (kN)	Rz (kN)	Rmx (kNm)
Dead Load	DC			4499.07				29016.99				29665.1		
Super-imposed Load	DW			401.564				2177.903				2321.198		
Creep & Shrinkage	CR & SH		-4.797	-87.824			4.564	111.385			0.613	-159.879		
Permanent Load Summary	DC+DW+CR+SH		**-4.797**	**4812.81**			**4.564**	**31306.28**			**0.613**	**31826.42**		
Live Load	LL	MAX												
		MIN												
Live Load + Impact	LL+IM	MAX		1967.689				4629.642				4988.703		
		MIN		-703.306				-954.939				-1097.71		
Wind Load on Structure	WS	Longi.												
		Trans.			284.442				1056.852				1369.756	
Wind Load on Live Load	WL	Longi.												
		Trans.				-549.048								
Uniform Temperature	TU(+) +22deg													
	TU(-) -19deg													
Temperature Gradient	TG	+		282.358				106.86				179.744		
		-		-79.801				-380.731				-58.095		
Settlement	SE	MAX		99.951				154.489				260.533		
		MIN		-61.32				-229.489				-205.955		



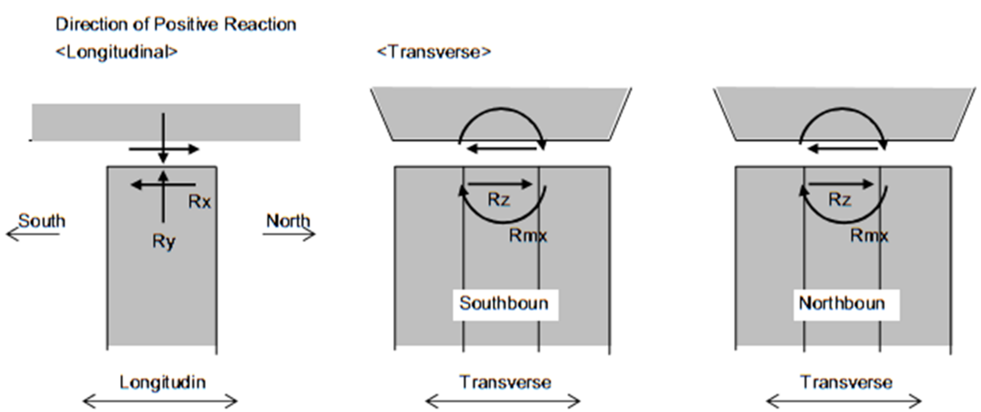
Load	Symbol	P15				P16				A2			
		Rx (kN)	Ry (kN)	Rz (kN)	Rmx (kNm)	Rx (kN)	Ry (kN)	Rz (kN)	Rmx (kNm)	Rx (kN)	Ry (kN)	Rz (kN)	Rmx (kNm)
Dead Load	DC		29733.8				28949.02				4574.01		
Super-imposed Load	DW		2320.804				2178.774				401.075		
Creep & Shrinkage	CR & SH	-0.272	-195.006			6.38	146.2			-5.899	-104.02		
Permanent Load Summary	DC+DW+CR+SH	**-0.272**	**31859.6**			**6.38**	**31273.99**			**-5.899**	**4871.065**		
Live Load	LL												
Live Load + Impact	LL+IM		4829.462				4826.26				2027.448		
			-963.712				-1093.79				-710.932		
Wind Load on Structure	WS												
				1086.751				1413.243				120.116	
Wind Load on Live Load	WL												
Uniform Temperature	TU(+) +22deg												
	TU(-) -19deg												
Temperature Gradient	TG		168.29				280.145				282.635		
			-45.488				-554.605				-79.869		
Settlement	SE		259.817				154.114				99.832		
			-205.189				-229.138				-61.192		

Direction of Positive Reaction

<Longitudinal> <Transverse>

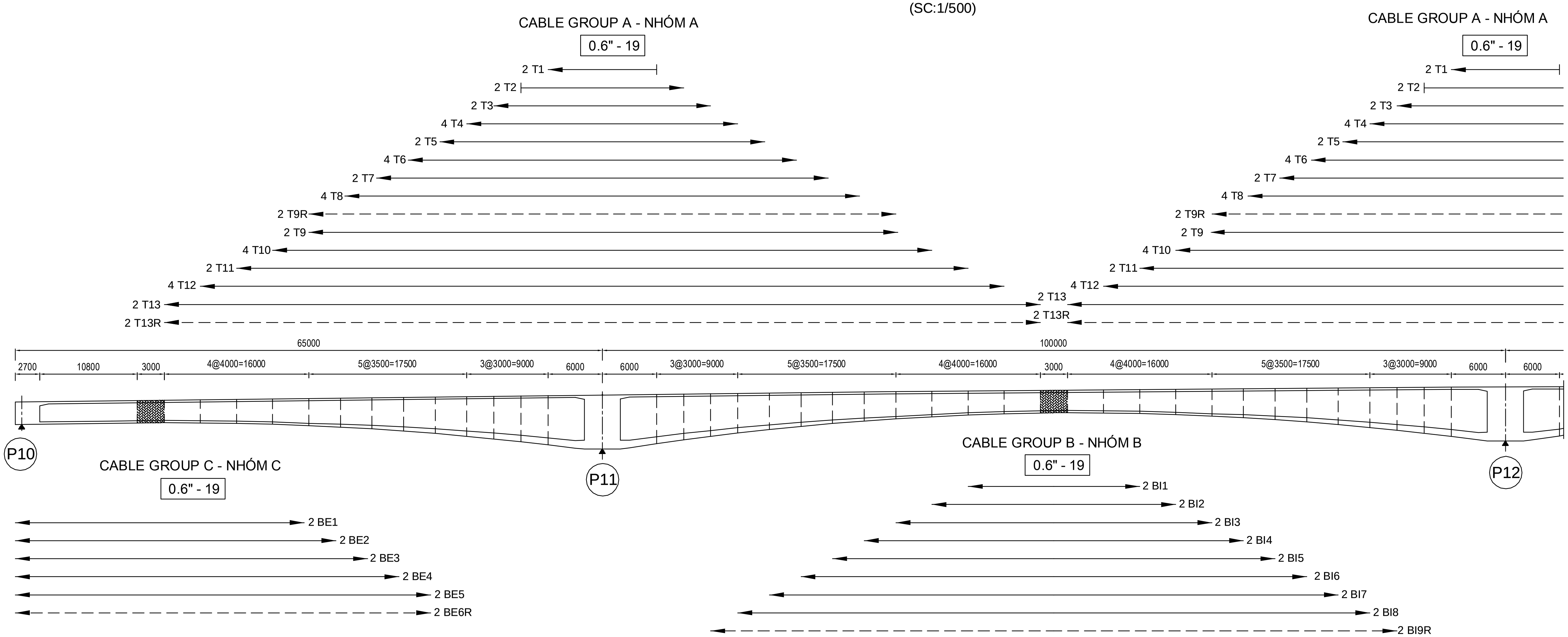


Load	Symbol	P15				P16				A2			
		Rx	Ry	Rz	Rmx	Rx	Ry	Rz	Rmx	Rx	Ry	Rz	Rmx
		(kN)	(kN)	(kN)	(kNm)	(kN)	(kN)	(kN)	(kNm)	(kN)	(kN)	(kN)	(kNm)
Dead Load	DC		29733.8				28949.02				4574.01		
Super-imposed Load	DW		2320.804				2178.774				401.075		
Creep & Shrinkage	CR & SH	-0.272	-195.006			6.38	146.2			-5.899	-104.02		
Permanent Load Summary	DC+DW+CR+SH	**-0.272**	**31859.6**			**6.38**	**31273.99**			**-5.899**	**4871.065**		
Live Load	LL												
Live Load + Impact	LL+IM		4829.462				4826.26				2027.448		
			-963.712				-1093.79				-710.932		
Wind Load on Structure	WS												
				1086.751				1413.243				120.116	
Wind Load on Live Load	WL												
Uniform Temperature	TU(+) +22deg												
	TU(-) -19deg												
Temperature Gradient	TG		168.29				280.145				282.635		
			-45.488				-554.605				-79.869		
Settlement	SE		259.817				154.114				99.832		
			-205.189				-229.138				-61.192		



MAIN CABLE ARRANGEMENT - BỐ TRÍ CÁP CHỦ (1/4)

GENERAL VIEW OF MAIN CABLE - BỐ TRÍ CHUNG CÁP CHỦ
(SC:1/500)



NOTES:

- All dimension are in mm.
- Prestressing tendons shall be formed 7 wire low relaxation strand, be in accordance with ASTM 416-99 Grade 270 :
  - + Norminal diameter of 1 strand : 15.2mm.
  - + Area of 1 strand:140mm2.
  - + Yield strength: fpy = 1670 Mpa
  - + Ultimate strength: fpu = 1860 Mpa
  - + Modulus of elasticity : 197 GPa.
- Cables used type of 6-19 (19 strand of 15.2mm), Ducts diameter : Ø100/107, Jacking force: 3711 KN.
- Length of tendons is added 1,0m for end of tensile, added 0.5m for end of no-tensile.
- Each tendon must be placed at least 2 pipe for checking grouting at the highest position and lowest position.
- Before grouting into duct for reserve cables, it must be approved by consultant engineer.
- The elongation of tendon is not included in length of tendon section in the jack.

GHI CHÚ

- Kých thưíc ghi trªn b¶n vÏ dïng ®¬n vÞ mm..
- VËt liÖu c¸p dïng lo¹i tao 7 sîi cã ®é tù chïng thÊp theo tiªu chuÈn ASTM 416-99 Grade 270 :
  - + §ường kÝnh danh ®Þnh 15.2mm.
  - + DiÖn tÝch tao c¸p 140mm2.
  - + Cường ®é ch¶y : 1670 Mpa.
  - + Cường ®é kÐo ®øt : 1860 Mpa.
  - + M« ®uyn ®µn håi : 197 GPa.
- C¸c bã c¸p dïng lo¹i bã 6-19 (19 tao 15.2mm), èng ghen : Ø100/107, lùc c¨ng mçi bã : 3711 KN.
- ChiÒu dµi c¸p ®ưîc céng thªm 1,0m cho ®Çu c¨ng kÐo, céng 0.5m cho ®Çu kh«ng c¨ng kÐo.
- Mçi bã c¸p ph¶i ®ưîc bè trÝ tèi thiÓu 2 èng th¨m v÷a, ë vÞ trÝ cao nhÊt vµ thÊp nhÊt.
- Trưíc khi b¬m v÷a vµo c¸c bã c¸p dù phßng ph¶i ®ưîc sù chÊp thuËn cña TVGS vµ TVTK.
- §é gi·n dµi cña c¸c bã c¸p chưa tÝnh ®Õn ®é dµi c¸c ®o¹n c¸p trong kÝch.

REMARKS - KÝ HIỆU:

Tensile both side - C¨ng 2 ®Çu

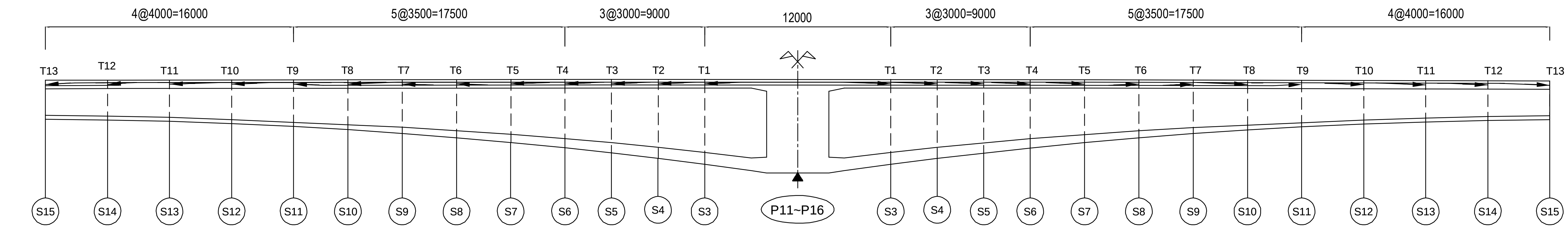
Tensile one side - C¨ng 1 ®Çu

Reserve Cable - C¸p dù phßng

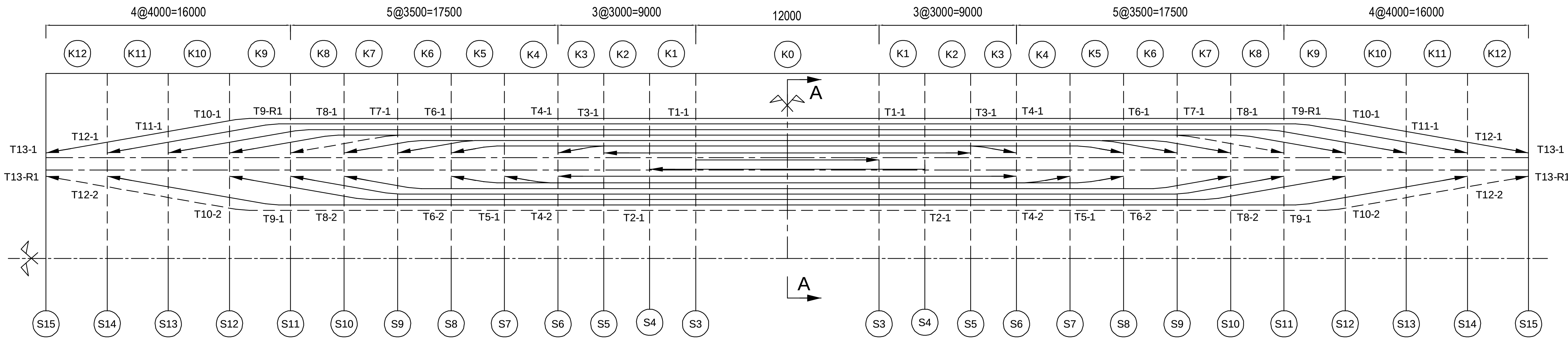
MINISTRY OF TRANSPORT VIETNAM		ENGINEERING DESIGN CONSULTANT	REMARKS:	DA NANG-QUANG NGAI EXPRESSWAY DEVELOPMENT PROJECT Package: 3A Station: KM16+880.00 - KM18+100.00				
CLIENT	PROJECT MANAGEMENT CONSULTANT	The Joint Venture of Nippon Koei Co., Ltd. Nippon Engineering Consultants Co., Ltd. Chodai Co., Ltd. Thai Engineering Consultants Co., Ltd.			PREPARED BY	CHECKED BY	APPROVED BY	DRAWING TITLE: MAIN CABLE ARRANGEMENT (1/4) BỐ TRÍ CÁP CHỦ (1/4)
				NAME	Nguyen Van Le	Hiroyuki Yokoyama	Yoshito Oba	
VIETNAM EXPRESSWAY CORPORATION	PROJECT MANAGEMENT UNIT NO.85			SIGNATURE				SCALE: AS SHOWN / DRAWING NO.: PKG3A-BR-SP1-0030 / REV. NO.: 1
				DATE	2012.09.July	2012.09.July	2012.09.July	

MAIN CABLE ARRANGEMENT - BỐ TRÍ CÁP CHỦ (2/4)

PROFILE OF CABLE GROUP A - CHÍNH DIỆN CÁP NHÓM A
(SC:1/300)

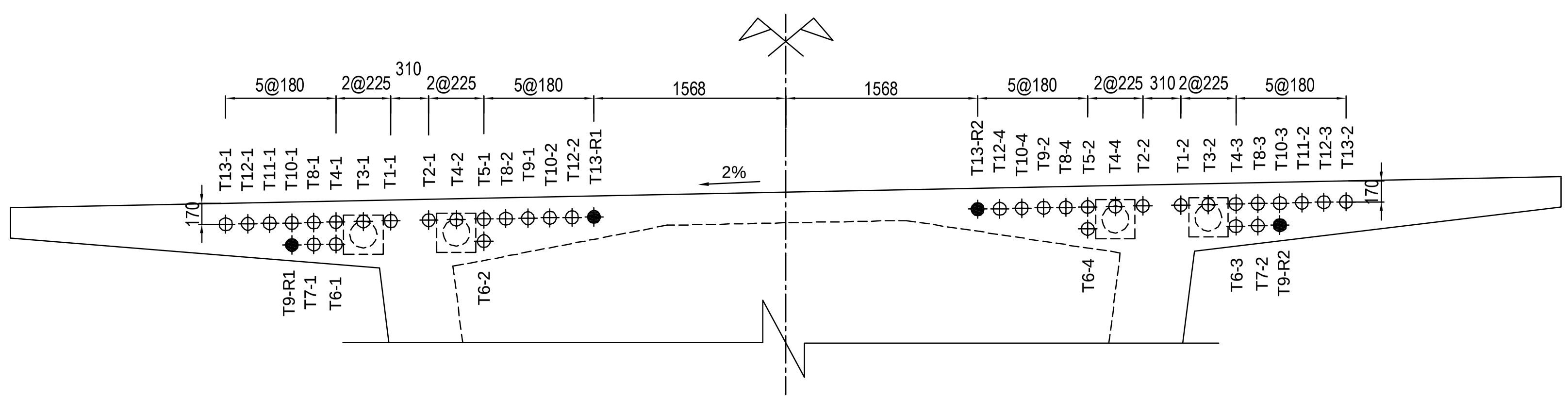


1/2 PLAN OF CABLE GROUP A - 1/2 MẶT BẰNG CÁP NHÓM A
(SC:1/300 - 1/150)

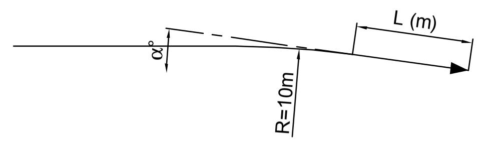


CROSS SECTION A - A

(SC:1/60)



PLAN OF TENDONS AT TOP SLAB
MẶT BẰNG CÁP DƯL BẢN ĐỈNH



NOTES:

- Tensile both side
- Tensile one side
- Reserve Cable

PROPERTIES OF TENDONS TOP SLAB
THÔNG SỐ CÁP DƯL BẢN ĐỈNH

Tendon's name Tên bó cáp	T1:T3	T4 -1	T4 -2	T5	T6	T7	T8	T9	T10	T11	T12	T13
α°	0°	5°	0°	5°	5°	5°	5°	5°	5°	5°	5°	5°
L(m)	–	2.145	–	2.145	2.145	4.21	4.21	6.276	6.276	6.276	8.341	12.471

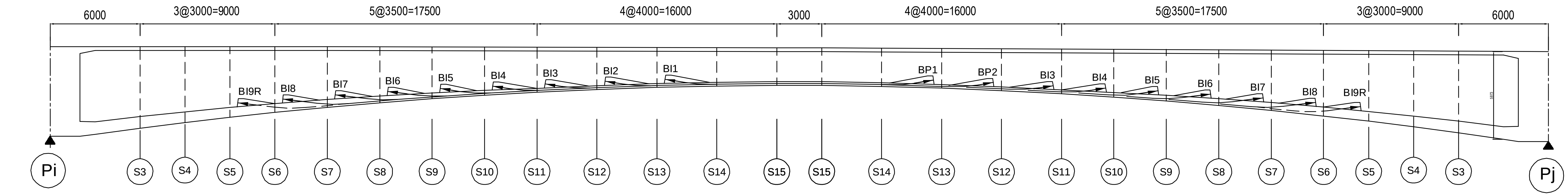
MINISTRY OF TRANSPORT VIETNAM		ENGINEERING DESIGN CONSULTANT	REMARKS:
CLIENT	PROJECT MANAGEMENT CONSULTANT	The Joint Venture of Nippon Koei Co., Ltd. Nippon Engineering Consultants Co., Ltd. Chodai Co., Ltd. Thai Engineering Consultants Co., Ltd.	
VIETNAM EXPRESSWAY CORPORATION	PROJECT MANAGEMENT UNIT NO.85		

DA NANG-QUANG NGAI EXPRESSWAY DEVELOPMENT PROJECT
Package: 3A Station: KM16+880.00 - KM18+100.00

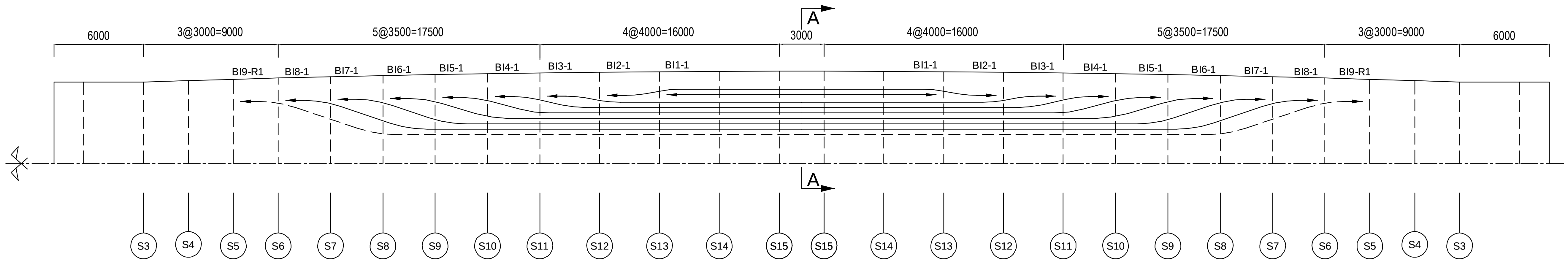
	PREPARED BY	CHECKED BY	APPROVED BY	DRAWING TITLE	MAIN CABLE ARRANGEMENT (2/4) BỐ TRÍ CÁP CHỦ (2/4)	
NAME	Nguyen Van Le	Hiroyuki Yokoyama	Yoshito Oba			
SIGNATURE				SCALE	DRAWING NO.	REV. NO.
DATE	2012.09.July	2012.09.July	2012.09.July	AS SHOWN	PKG3A-BR-SP1-0031	1

MAIN CABLE ARRANGEMENT - BỐ TRÍ CÁP CHỦ (3/4)

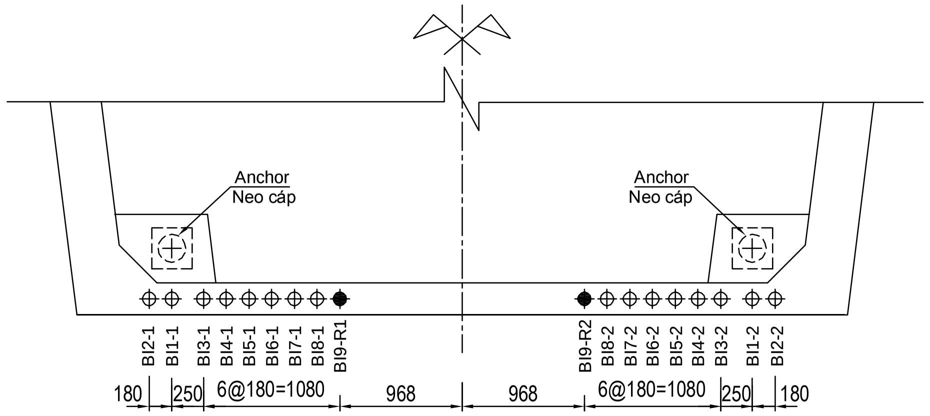
PROFILE OF CABLE GROUP B - CHÍNH DIỆN CÁP NHÓM B
(SC:1/300)



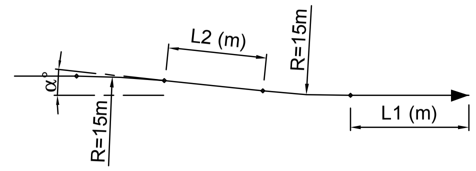
1/2 PLAN OF CABLE GROUP B - 1/2 MẶT BẰNG CÁP NHÓM B
(SC:1/300 - 1/150)



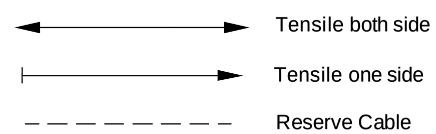
A - A
(SC:1/20)



PLAN OF TENDONS (SPANS 100m)
MẶT BẰNG CÁP DƯL (NHỊP 100m)



NOTES:



PROPERTIES OF TENDONS BOTTOM SLAB
THÔNG SỐ CÁP DƯL BẢN ĐÁY

Tendon's name Tên bó cáp	BI1	BI2	BI3	BI4	BI5	BI6	BI7	BI8	BI9R
α°	0°	5°	5°	6.5°	7.5°	8.5°	9.5°	9.5°	9.5°
L1(m)	-	1.200	1.200	1.200	1.200	1.200	1.200	1.200	1.200
L2(m)	-	1.019	1.077	1.538	2.025	2.310	2.457	3.348	4.220

MINISTRY OF TRANSPORT VIETNAM		ENGINEERING DESIGN CONSULTANT	REMARKS:
CLIENT	PROJECT MANAGEMENT CONSULTANT	The Joint Venture of Nippon Koei Co., Ltd. Nippon Engineering Consultants Co., Ltd. Chodai Co., Ltd. Thai Engineering Consultants Co., Ltd.	
VIETNAM EXPRESSWAY CORPORATION	PROJECT MANAGEMENT UNIT NO.85		

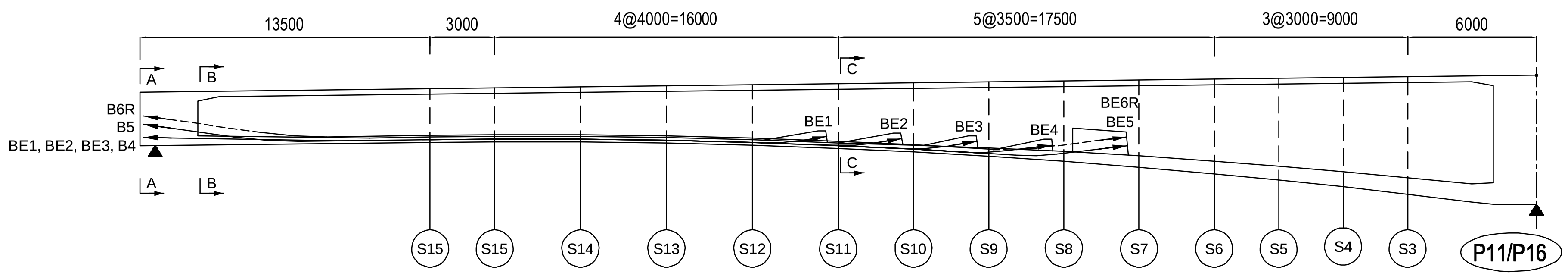
DA NANG-QUANG NGAI EXPRESSWAY DEVELOPMENT PROJECT
Package: 3A Station: KM16+880.00 - KM18+100.00

	PREPARED BY	CHECKED BY	APPROVED BY	DRAWING TITLE	MAIN CABLE ARRANGEMENT (3/4) BỐ TRÍ CÁP CHỦ (3/4)	
NAME	Nguyen Van Le	Hiroyuki Yokoyama	Yoshito Oba			
SIGNATURE				SCALE	DRAWING NO.	REV. NO.
DATE	2012.09.July	2012.09.July	2012.09.July	AS SHOWN	PKG3A-BR-SP1-0032	1

MAIN CABLE ARRANGEMENT - BỐ TRÍ CÁP CHỦ (4/4)

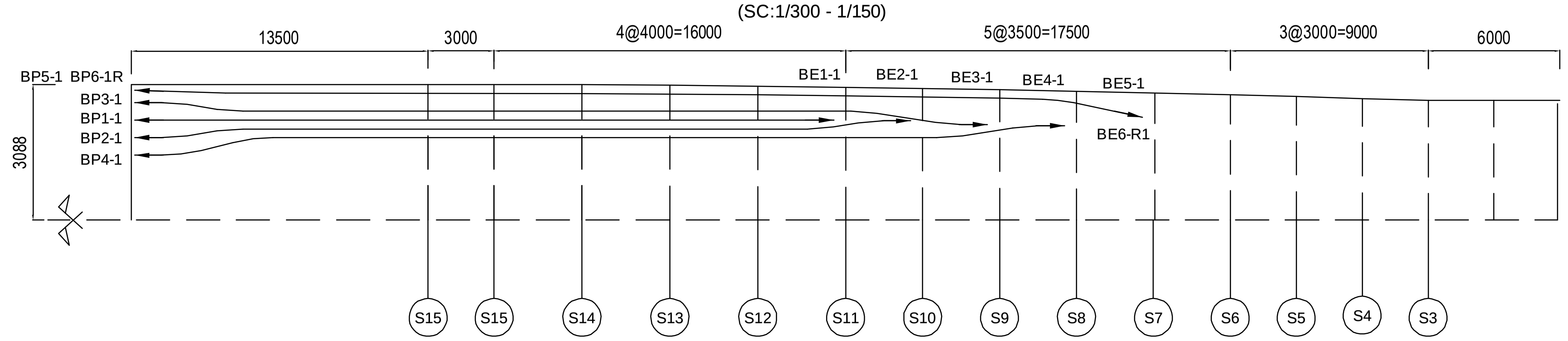
PROFILE OF CABLE GROUP C - CHÍNH DIỆN CÁP NHÓM C

(SC:1/300)



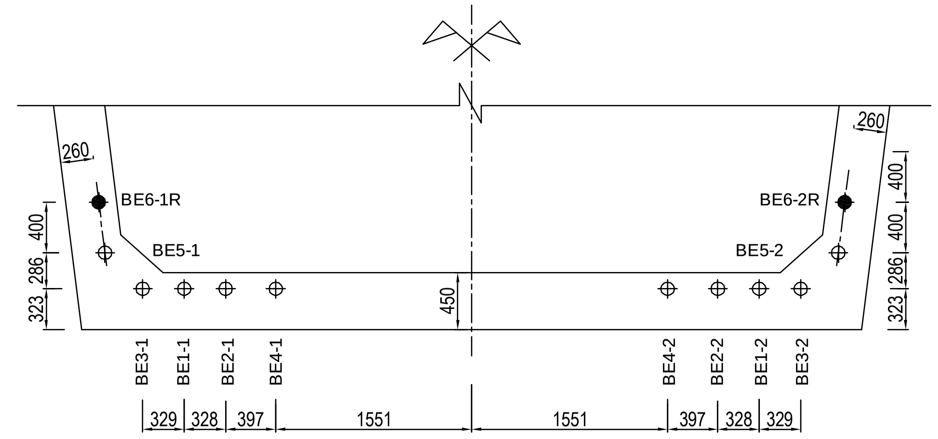
1/2 PLAN OF CABLE GROUP C - MẶT BẰNG CÁP NHÓM C

(SC:1/300 - 1/150)



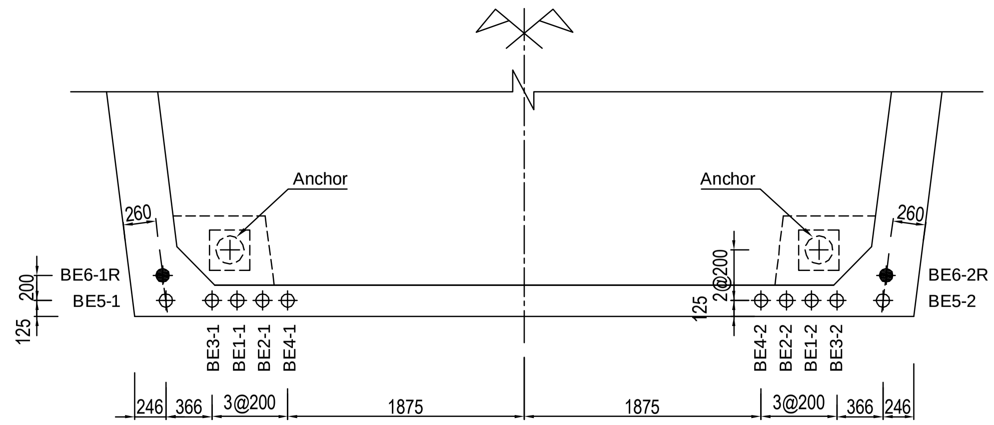
B - B

(SC:1/60)



C - C

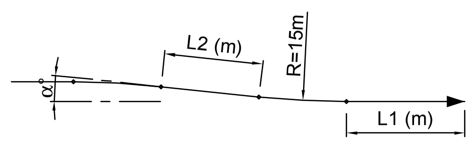
(SC:1/60)



A - A

(SC:1/60)

PLAN OF TENDONS (SPANS 65m)
MẶT BẰNG CÁP DƯL (NHỊP 65m)



PROPERTIES OF TENDONS BOTTOM SLAB
THÔNG SỐ CÁP DƯL BẢN ĐÁY

Tendon's name Tên bó cáp	BE1	BE2	BE3	BE4	BE5	BE6R
α°	0°	4.5°	4.5°	4.5°	6.0°	6.0°
L1(m)	-	1.200	1.200	1.200	-	-
L2(m)	-	1.103	1.969	2.933	2.755	2.755

NOTES:

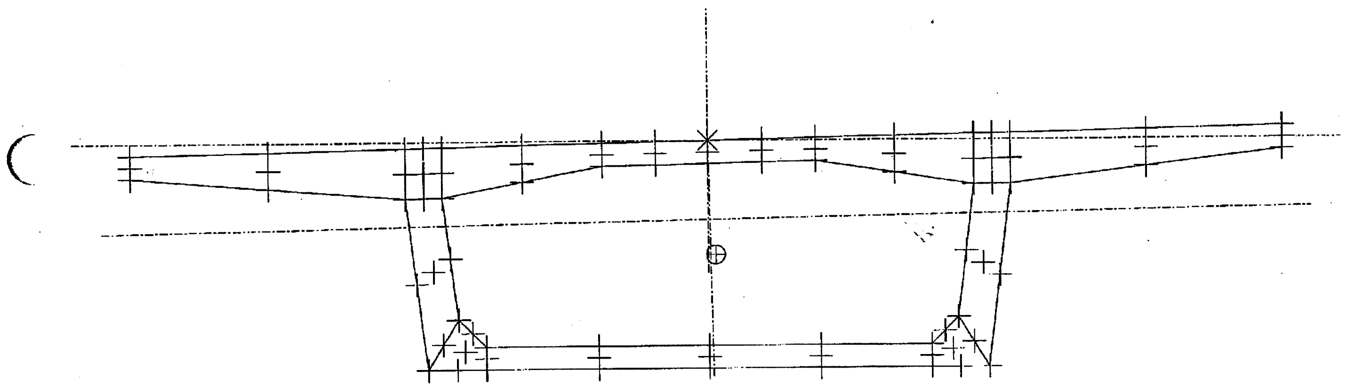
- Tensile both side
- Tensile one side
- Reserve Cable

MINISTRY OF TRANSPORT VIETNAM		ENGINEERING DESIGN CONSULTANT	REMARKS:	DA NANG-QUANG NGAI EXPRESSWAY DEVELOPMENT PROJECT Package: 3A Station: KM16+880.00 - KM18+100.00				
CLIENT	PROJECT MANAGEMENT CONSULTANT	The Joint Venture of Nippon Koei Co., Ltd. Nippon Engineering Consultants Co., Ltd. Chodai Co., Ltd. Thai Engineering Consultants Co., Ltd.			PREPARED BY	CHECKED BY	APPROVED BY	DRAWING TITLE: MAIN CABLE ARRANGEMENT (4/4) BỐ TRÍ CÁP CHỦ (4/4)
VIETNAM EXPRESSWAY CORPORATION	PROJECT MANAGEMENT UNIT NO.85			NAME	Nguyen Van Le	Hiroyuki Yokoyama	Yoshito Oba	
				SIGNATURE				SCALE: AS SHOWN / DRAWING NO.: PKG3A-BR-SP1-0033 / REV. NO.: 1
				DATE	2012.09.July	2012.09.July	2012.09.July	

Cross-section : Box001

Part : 1 Variant : 1

Description : Box



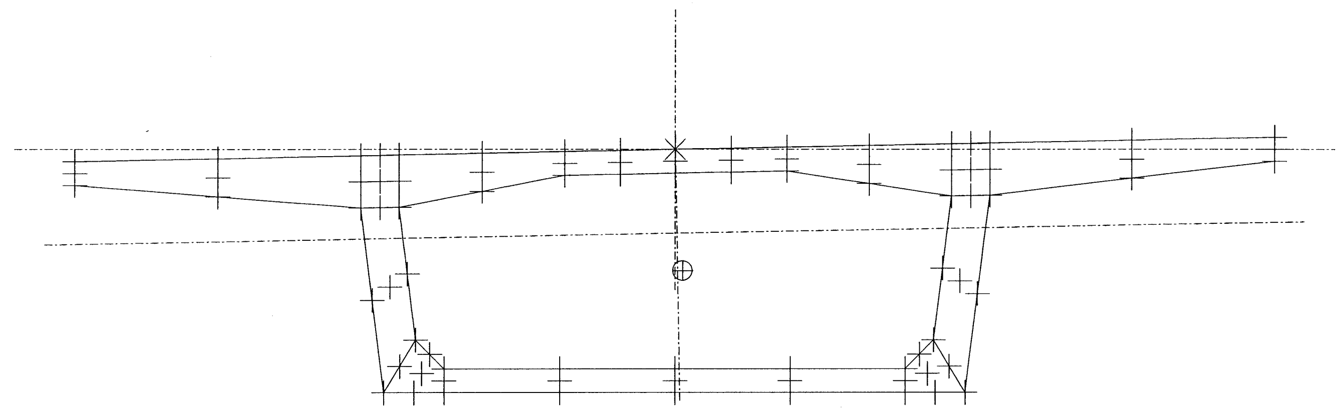
C R O S S - S E C T I O N D A T A

Cross-section area	0.78370E+01 m2
Shear area - Bending about Z-axis	0.13507E+01 m2
Shear area - Bending about Y-axis	0.50306E+01 m2
Torsional moment of inertia I	0.14612E+02 m4
Shear lag factor X (Ax-shear/Ax)	0.10000E+01
Moment of inertia about Y-axis	0.83182E+02 m4
Section moment about Y-axis - min	0.13089E+02 m3
Section moment about Y-axis - max	0.13172E+02 m3
Eccentricity Z (Shear lag)	0.00000E+00 m
Shear lag factor Y	0.10000E+01
Moment of inertia about Y-axis (Shear lag)	0.83182E+02 m4
Moment of inertia about Z-axis	0.66159E+01 m4
Section moment about Z-axis - min	0.40415E+01 m3
Section moment about Z-axis - max	0.66826E+01 m3
Eccentricity Y (Shear lag)	0.00000E+00 m
Shear lag factor Z	0.10000E+01
Moment of inertia about Z-axis (Shear lag)	0.66159E+01 m4
Warping moment of inertia	0.13564E+02 m6
Bending axis origin - Eccentricity ey	-0.86302 m
Bending axis origin - Eccentricity ez	0.02008 m
Main axis angle	1.01438 Deg
Shear axis origin - Eccentricity ey	-1.25277 m
Shear axis origin - Eccentricity ez	0.08260 m
Main axis angle - Shear	0.00000 Deg
Y-below : Gravity centre - minY	1.63698 m
Y-above : Gravity centre - maxY	0.99002 m
Z-left : Gravity centre - minZ	6.35508 m
Z-right : Gravity centre - maxZ	6.31492 m
Perimeter exposed to drying (outside)	29.31785 m
Perimeter (inside)	14.48838 m

Cross-section : Box002

Part : 1 Variant : 1

Description : Box



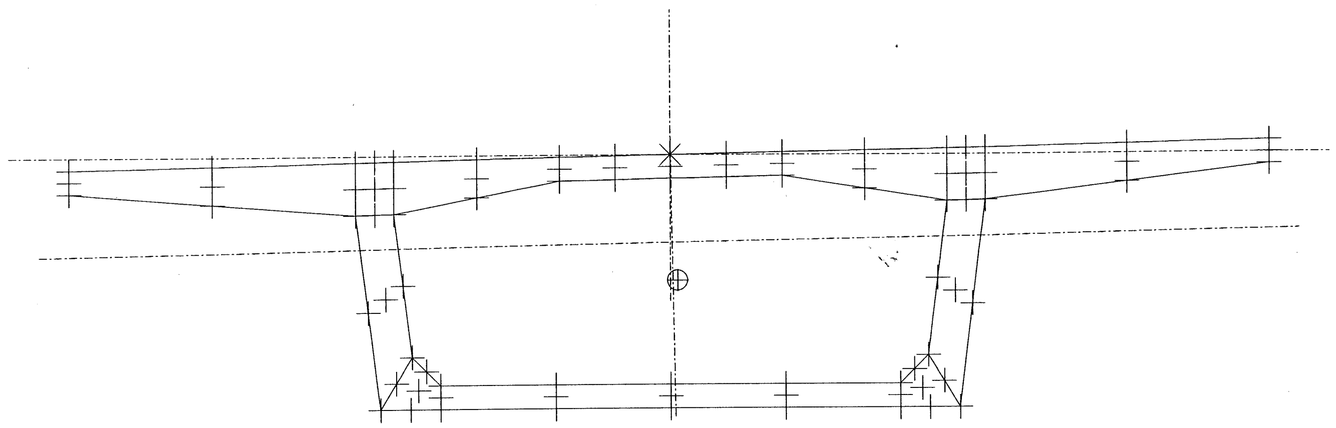
C R O S S - S E C T I O N D A T A

Cross-section area	0.78810E+01 m2
Shear area - Bending about Z-axis	0.14002E+01 m2
Shear area - Bending about Y-axis	0.50208E+01 m2
Torsional moment of inertia I	0.15287E+02 m4
Moment of inertia about Y-axis	0.83546E+02 m4
Eccentricity Z (Shear lag)	0.00000E+00 m
Shear lag factor Y	0.10000E+01
Moment of inertia about Y-axis (Shear lag)	0.83546E+02 m4
Moment of inertia about Z-axis	0.69931E+01 m4
Eccentricity Y (Shear lag)	0.00000E+00 m
Shear lag factor Z	0.10000E+01
Moment of inertia about Z-axis (Shear lag)	0.69931E+01 m4
Warping moment of inertia	0.13962E+02 m6
Bending axis origin - Eccentricity ey	-0.88301 m
Bending axis origin - Eccentricity ez	0.01993 m
Main axis angle	1.01720 Deg
Shear axis origin - Eccentricity ey	-1.27718 m
Shear axis origin - Eccentricity ez	0.07910 m
Main axis angle - Shear	0.00000 Deg
Y-below : Gravity centre - minY	1.67599 m
Y-above : Gravity centre - maxY	1.01001 m
Z-left : Gravity centre - minZ	6.35493 m
Z-right : Gravity centre - maxZ	6.31507 m
Perimeter exposed to drying (outside)	29.42093 m
Perimeter (inside)	14.59146 m

Cross-section : Box003

Part : 1 Variant : 1

Description : Box



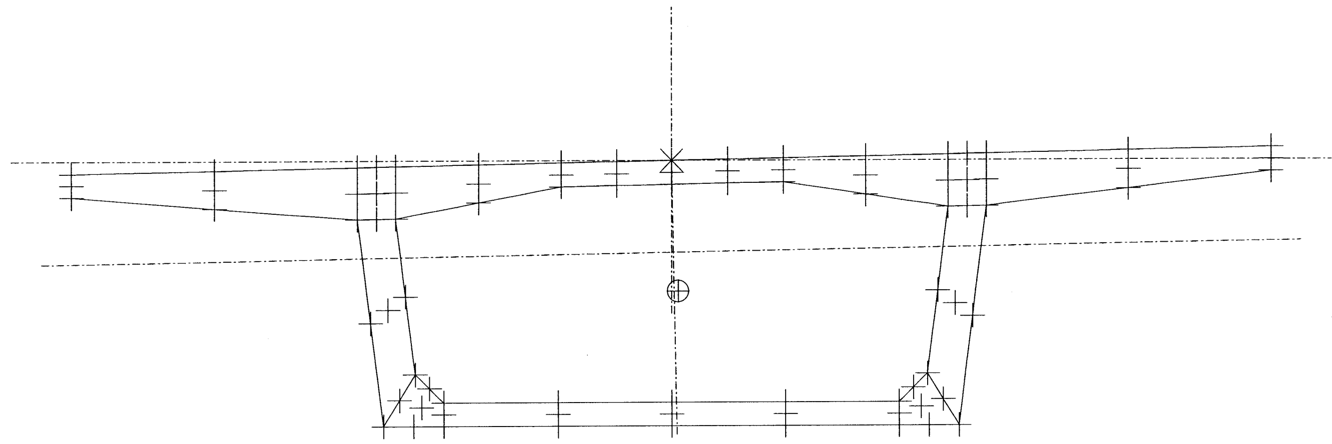
C R O S S - S E C T I O N D A T A

Cross-section area	0.79532E+01	m2
Shear area - Bending about Z-axis	0.14823E+01	m2
Shear area - Bending about Y-axis	0.50043E+01	m2
Torsional moment of inertia I	0.16442E+02	m4
Moment of inertia about Y-axis	0.84143E+02	m4
Eccentricity Z (Shear lag)	0.00000E+00	m
Shear lag factor Y	0.10000E+01	
Moment of inertia about Y-axis (Shear lag)	0.84143E+02	m4
Moment of inertia about Z-axis	0.76522E+01	m4
Eccentricity Y (Shear lag)	0.00000E+00	m
Shear lag factor Z	0.10000E+01	
Moment of inertia about Z-axis (Shear lag)	0.76522E+01	m4
Warping moment of inertia	0.14598E+02	m6
Bending axis origin - Eccentricity ey	-0.91667	m
Bending axis origin - Eccentricity ez	0.02030	m
Main axis angle	1.01966	Deg
Shear axis origin - Eccentricity ey	-1.31752	m
Shear axis origin - Eccentricity ez	0.07520	m
Main axis angle - Shear	0.00000	Deg
Y-below : Gravity centre - minY	1.74133	m
Y-above : Gravity centre - maxY	1.04367	m
Z-left : Gravity centre - minZ	6.35530	m
Z-right : Gravity centre - maxZ	6.31470	m
Perimeter exposed to drying (outside)	29.59638	m
Perimeter (inside)	14.76690	m

Cross-section : Box004

Part : 1 Variant : 1

Description : Box



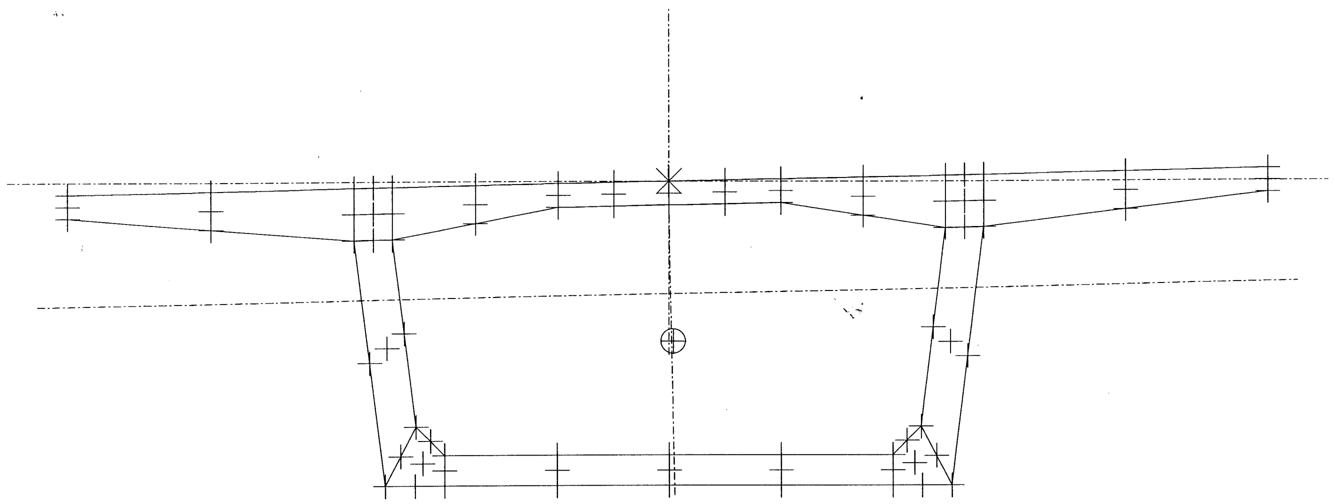
C R O S S - S E C T I O N D A T A

Cross-section area	0.80617E+01	m2
Shear area - Bending about Z-axis	0.16049E+01	m2
Shear area - Bending about Y-axis	0.49803E+01	m2
Torsional moment of inertia I	0.18200E+02	m4
Moment of inertia about Y-axis	0.85034E+02	m4
Eccentricity Z (Shear lag)	0.00000E+00	m
Shear lag factor Y	0.10000E+01	
Moment of inertia about Y-axis (Shear lag)	0.85034E+02	m4
Moment of inertia about Z-axis	0.86858E+01	m4
Eccentricity Y (Shear lag)	0.00000E+00	m
Shear lag factor Z	0.10000E+01	
Moment of inertia about Z-axis (Shear lag)	0.86858E+01	m4
Warping moment of inertia	0.15505E+02	m6
Bending axis origin - Eccentricity ey	-0.96686	m
Bending axis origin - Eccentricity ez	0.02003	m
Main axis angle	1.02752	Deg
Shear axis origin - Eccentricity ey	-1.37646	m
Shear axis origin - Eccentricity ez	0.06823	m
Main axis angle - Shear	0.00000	Deg
Y-below : Gravity centre - minY	1.83714	m
Y-above : Gravity centre - maxY	1.09386	m
Z-left : Gravity centre - minZ	6.35503	m
Z-right : Gravity centre - maxZ	6.31497	m
Perimeter exposed to drying (outside)	29.85459	m
Perimeter (inside)	15.02512	m

Cross-section : Box:006

Part : 1 Variant : 1

Description : Box



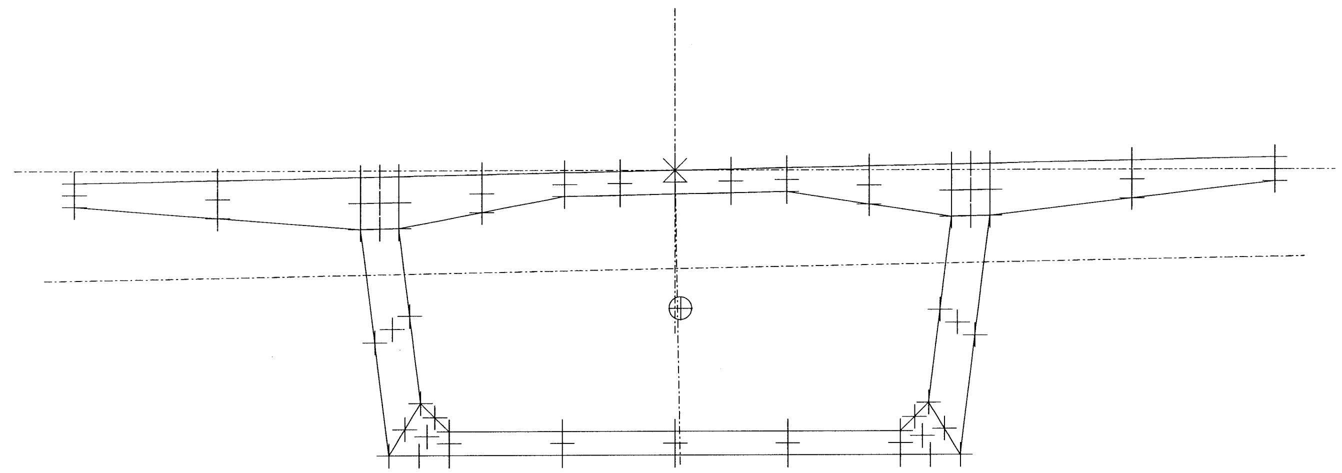
C R O S S - S E C T I O N D A T A

Cross-section area	0.87245E+01	m2
Shear area - Bending about Z-axis	0.19629E+01	m2
Shear area - Bending about Y-axis	0.53036E+01	m2
Torsional moment of inertia I	0.25022E+02	m4
Moment of inertia about Y-axis	0.88273E+02	m4
Eccentricity Z (Shear lag)	0.00000E+00	m
Shear lag factor Y	0.10000E+01	
Moment of inertia about Y-axis (Shear lag)	0.88273E+02	m4
Moment of inertia about Z-axis	0.13040E+02	m4
Eccentricity Y (Shear lag)	0.00000E+00	m
Shear lag factor Z	0.10000E+01	
Moment of inertia about Z-axis (Shear lag)	0.13040E+02	m4
Warping moment of inertia	0.17011E+02	m6
Bending axis origin - Eccentricity ey	-1.18215	m
Bending axis origin - Eccentricity ez	0.01859	m
Main axis angle	1.06872	Deg
Shear axis origin - Eccentricity ey	-1.68689	m
Shear axis origin - Eccentricity ez	0.04587	m
Main axis angle - Shear	0.00000	Deg
Y-below : Gravity centre - minY	2.02385	m
Y-above : Gravity centre - maxY	1.30915	m
Z-left : Gravity centre - minZ	6.35359	m
Z-right : Gravity centre - maxZ	6.31641	m
Perimeter exposed to drying (outside)	30.56303	m
Perimeter (inside)	15.61239	m

Cross-section : Box:005

Part : 1 Variant : 1

Description : Box



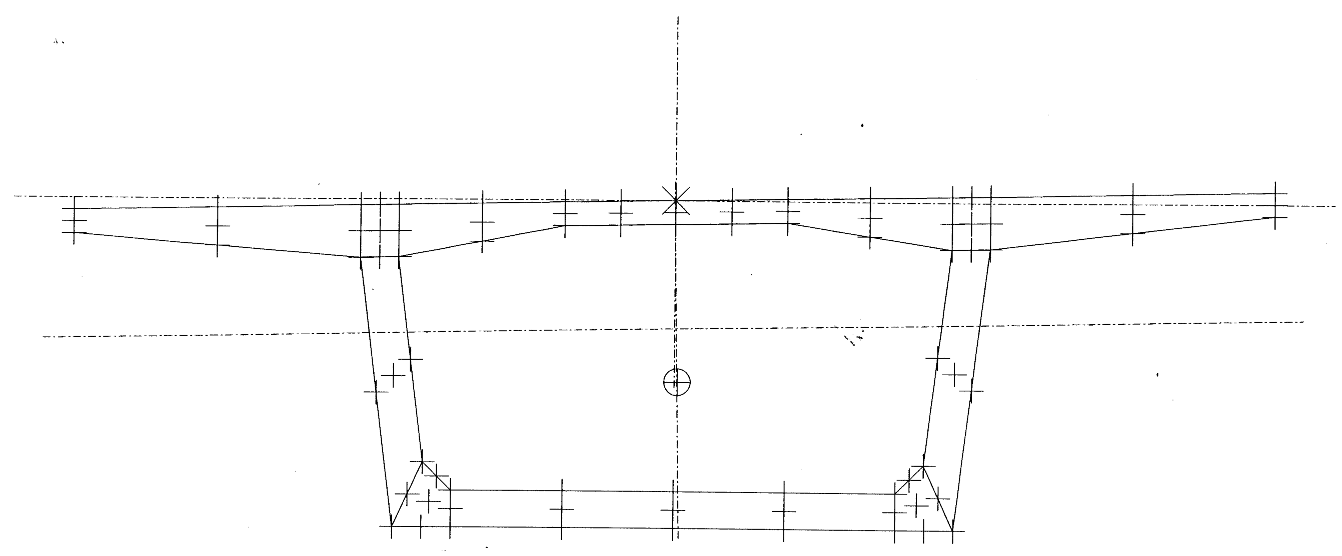
C R O S S - S E C T I O N D A T A

Cross-section area	0.82052E+01	m2
Shear area - Bending about Z-axis	0.17677E+01	m2
Shear area - Bending about Y-axis	0.49492E+01	m2
Torsional moment of inertia I	0.20612E+02	m4
Moment of inertia about Y-axis	0.86193E+02	m4
Eccentricity Z (Shear lag)	0.00000E+00	m
Shear lag factor Y	0.10000E+01	
Moment of inertia about Y-axis (Shear lag)	0.86193E+02	m4
Moment of inertia about Z-axis	0.10164E+02	m4
Eccentricity Y (Shear lag)	0.00000E+00	m
Shear lag factor Z	0.10000E+01	
Moment of inertia about Z-axis (Shear lag)	0.10164E+02	m4
Warping moment of inertia	0.16605E+02	m6
Bending axis origin - Eccentricity ey	-1.03387	m
Bending axis origin - Eccentricity ez	0.01966	m
Main axis angle	1.04002	Deg
Shear axis origin - Eccentricity ey	-1.45309	m
Shear axis origin - Eccentricity ez	0.06008	m
Main axis angle - Shear	0.00000	Deg
Y-below : Gravity centre - minY	1.96313	m
Y-above : Gravity centre - maxY	1.16087	m
Z-left : Gravity centre - minZ	6.35466	m
Z-right : Gravity centre - maxZ	6.31534	m
Perimeter exposed to drying (outside)	30.19556	m
Perimeter (inside)	15.36609	m

Cross-section : Box007

Part : 1 Variant : 1

Description : Box



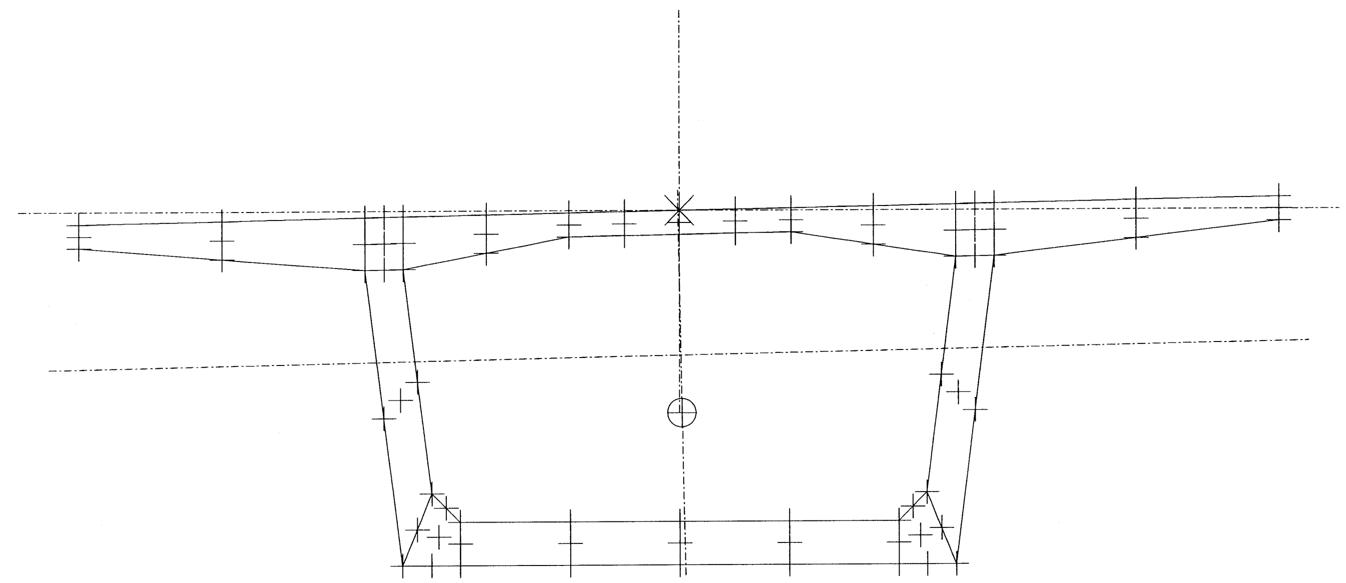
CROSS-SECTION DATA

Cross-section area	0.92618E+01	m2
Shear area - Bending about Z-axis	0.21857E+01	m2
Shear area - Bending about Y-axis	0.56057E+01	m2
Torsional moment of inertia I	0.29831E+02	m4
Moment of inertia about Y-axis	0.90474E+02	m4
Eccentricity Z (Shear lag)	0.00000E+00	m
Shear lag factor Y	0.10000E+01	
Moment of inertia about Y-axis (Shear lag)	0.90474E+02	m4
Moment of inertia about Z-axis	0.16605E+02	m4
Eccentricity Y (Shear lag)	0.00000E+00	m
Shear lag factor Z	0.10000E+01	
Moment of inertia about Z-axis (Shear lag)	0.16605E+02	m4
Warping moment of inertia	0.17991E+02	m6
Bending axis origin - Eccentricity ey	-1.34326	m
Bending axis origin - Eccentricity ez	0.01682	m
Main axis angle	1.11137	Deg
Shear axis origin - Eccentricity ey	-1.90856	m
Shear axis origin - Eccentricity ez	0.03261	m
Main axis angle - Shear	0.00000	Deg
Y-below : Gravity centre - minY	2.10674	m
Y-above : Gravity centre - maxY	1.47026	m
Z-left : Gravity centre - minZ	6.35182	m
Z-right : Gravity centre - maxZ	6.31818	m
Perimeter exposed to drying (outside)	30.99470	m
Perimeter (inside)	15.92115	m

Cross-section : Box008

Part : 1 Variant : 1

Description : Box



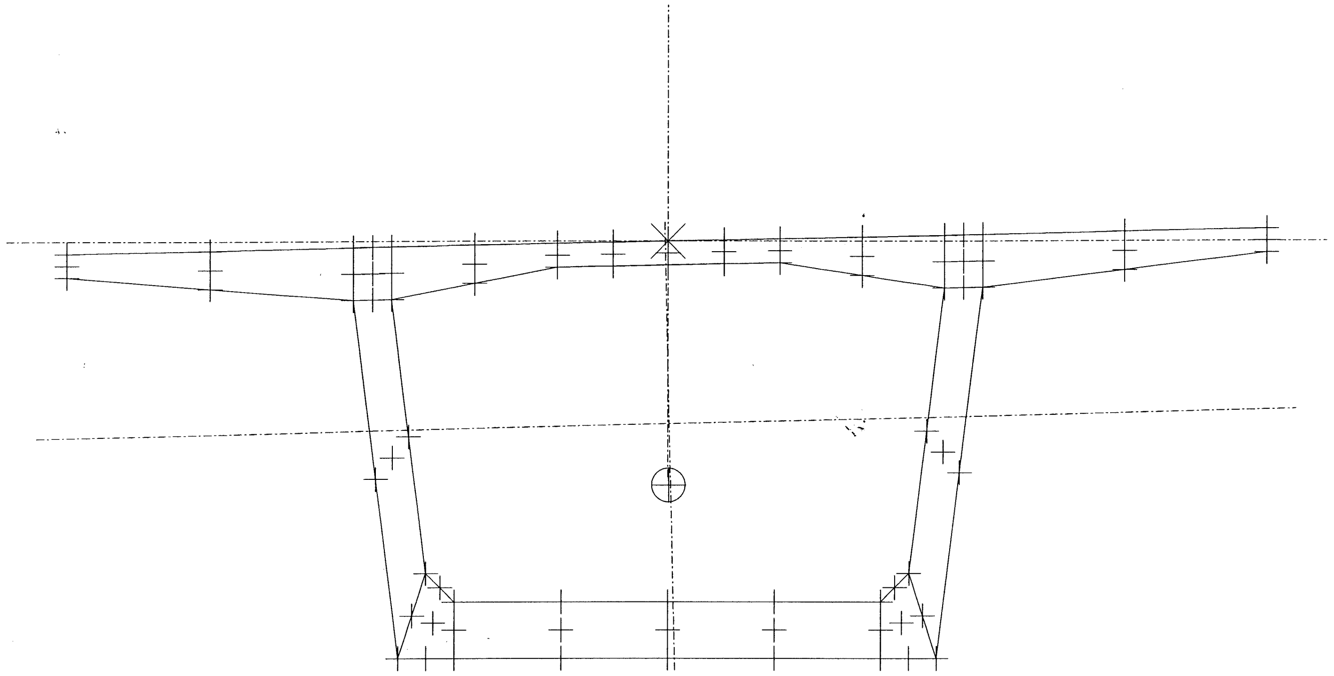
CROSS-SECTION DATA

Cross-section area	0.98123E+01	m2
Shear area - Bending about Z-axis	0.24387E+01	m2
Shear area - Bending about Y-axis	0.58545E+01	m2
Torsional moment of inertia I	0.35209E+02	m4
Moment of inertia about Y-axis	0.92780E+02	m4
Eccentricity Z (Shear lag)	0.00000E+00	m
Shear lag factor Y	0.10000E+01	
Moment of inertia about Y-axis (Shear lag)	0.92780E+02	m4
Moment of inertia about Z-axis	0.21052E+02	m4
Eccentricity Y (Shear lag)	0.00000E+00	m
Shear lag factor Z	0.10000E+01	
Moment of inertia about Z-axis (Shear lag)	0.21052E+02	m4
Warping moment of inertia	0.19621E+02	m6
Bending axis origin - Eccentricity ey	-1.51879	m
Bending axis origin - Eccentricity ez	0.01648	m
Main axis angle	1.16391	Deg
Shear axis origin - Eccentricity ey	-2.12695	m
Shear axis origin - Eccentricity ez	0.02485	m
Main axis angle - Shear	0.00000	Deg
Y-below : Gravity centre - minY	2.21221	m
Y-above : Gravity centre - maxY	1.64579	m
Z-left : Gravity centre - minZ	6.35148	m
Z-right : Gravity centre - maxZ	6.31852	m
Perimeter exposed to drying (outside)	31.49105	m
Perimeter (inside)	16.29655	m

Cross-section : Box010

Part : 1 Variant : 1

Description : Box



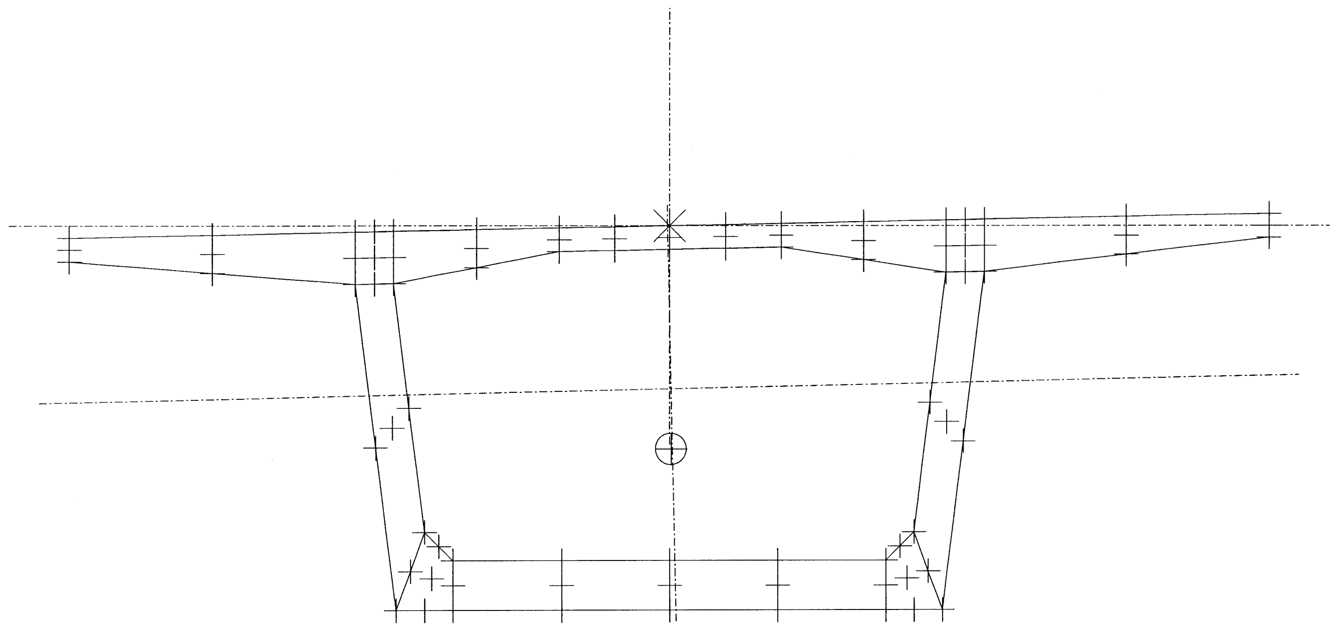
C R O S S - S E C T I O N D A T A

Cross-section area	0.10955E+02	m2
Shear area - Bending about Z-axis	0.30282E+01	m2
Shear area - Bending about Y-axis	0.62175E+01	m2
Torsional moment of inertia I	0.47855E+02	m4
Moment of inertia about Y-axis	0.97589E+02	m4
Eccentricity Z (Shear lag)	0.00000E+00	m
Shear lag factor Y	0.10000E+01	
Moment of inertia about Y-axis (Shear lag)	0.97589E+02	m4
Moment of inertia about Z-axis	0.33371E+02	m4
Eccentricity Y (Shear lag)	0.00000E+00	m
Shear lag factor Z	0.10000E+01	
Moment of inertia about Z-axis (Shear lag)	0.33371E+02	m4
Warping moment of inertia	0.26045E+02	m6
Bending axis origin - Eccentricity ey	-1.91639	m
Bending axis origin - Eccentricity ez	0.01409	m
Main axis angle	1.35953	Deg
Shear axis origin - Eccentricity ey	-2.56999	m
Shear axis origin - Eccentricity ez	0.00882	m
Main axis angle - Shear	0.00000	Deg
Y-below : Gravity centre - minY	2.47761	m
Y-above : Gravity centre - maxY	2.04339	m
Z-left : Gravity centre - minZ	6.34909	m
Z-right : Gravity centre - maxZ	6.32091	m
Perimeter exposed to drying (outside)	32.66140	m
Perimeter (inside)	17.22679	m

Cross-section : Box009

Part : 1 Variant : 1

Description : Box



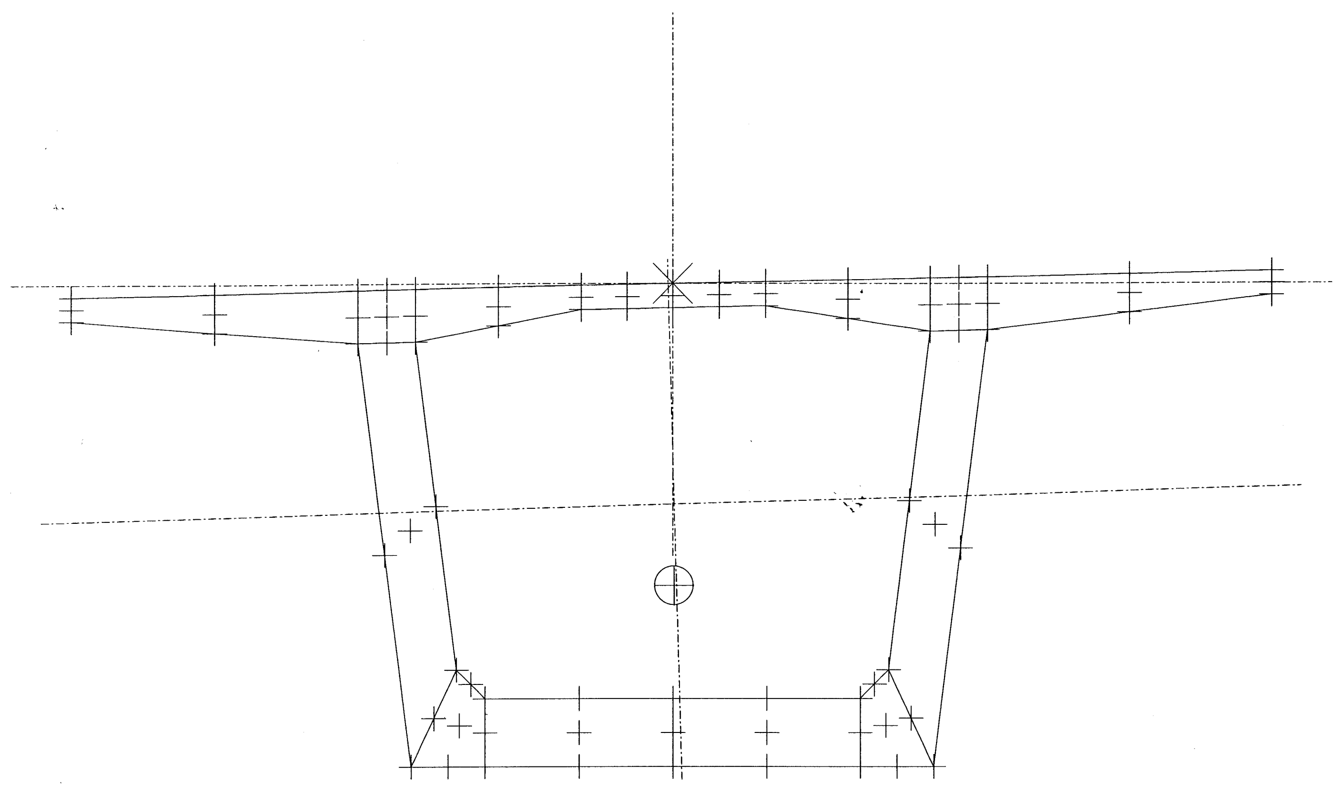
C R O S S - S E C T I O N D A T A

Cross-section area	0.10383E+02	m2
Shear area - Bending about Z-axis	0.27216E+01	m2
Shear area - Bending about Y-axis	0.60603E+01	m2
Torsional moment of inertia I	0.41258E+02	m4
Moment of inertia about Y-axis	0.95183E+02	m4
Eccentricity Z (Shear lag)	0.00000E+00	m
Shear lag factor Y	0.10000E+01	
Moment of inertia about Y-axis (Shear lag)	0.95183E+02	m4
Moment of inertia about Z-axis	0.26613E+02	m4
Eccentricity Y (Shear lag)	0.00000E+00	m
Shear lag factor Z	0.10000E+01	
Moment of inertia about Z-axis (Shear lag)	0.26613E+02	m4
Warping moment of inertia	0.22175E+02	m6
Bending axis origin - Eccentricity ey	-1.71117	m
Bending axis origin - Eccentricity ez	0.01545	m
Main axis angle	1.24553	Deg
Shear axis origin - Eccentricity ey	-2.34872	m
Shear axis origin - Eccentricity ez	0.01662	m
Main axis angle - Shear	0.00000	Deg
Y-below : Gravity centre - minY	2.33683	m
Y-above : Gravity centre - maxY	1.83817	m
Z-left : Gravity centre - minZ	6.35045	m
Z-right : Gravity centre - maxZ	6.31955	m
Perimeter exposed to drying (outside)	32.05007	m
Perimeter (inside)	16.73617	m

Cross-section : Box012

Part : 1 Variant : 1

Description : Box



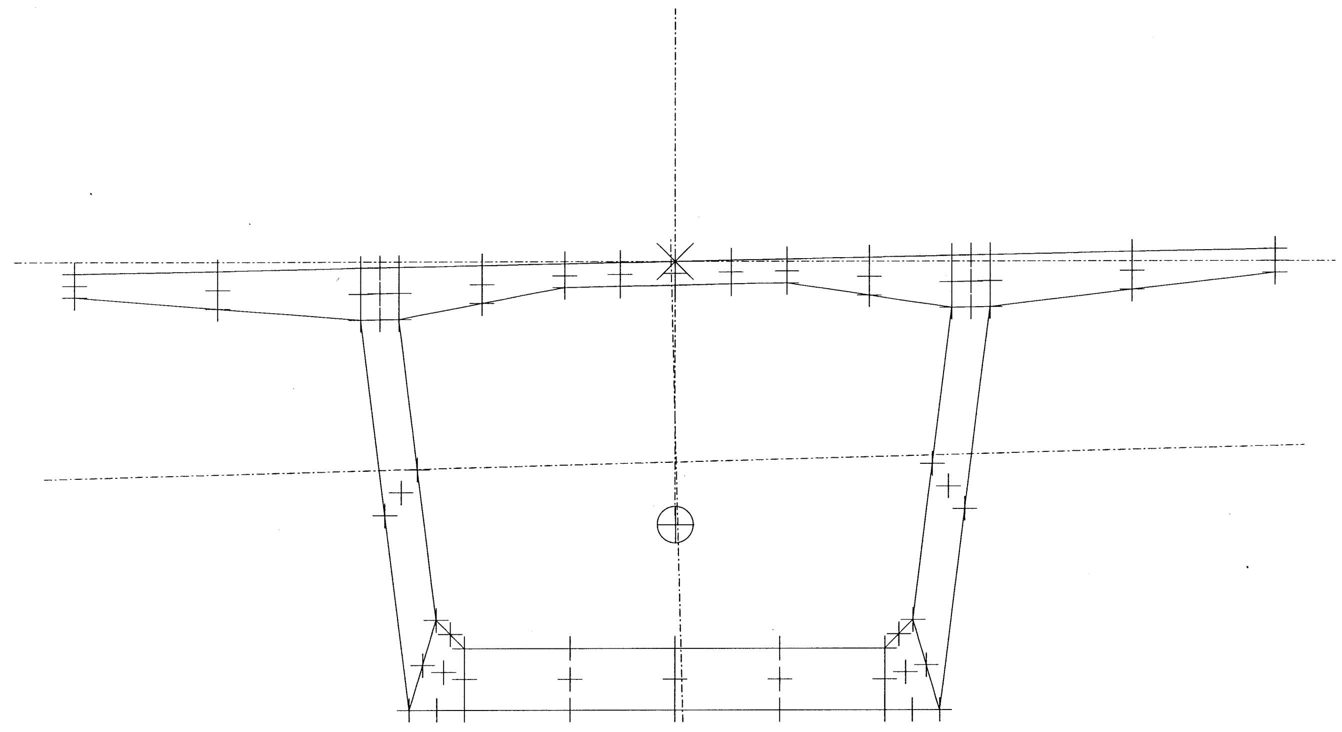
C R O S S - S E C T I O N D A T A

Cross-section area	0.13647E+02 m2
Shear area - Bending about Z-axis	0.51605E+01 m2
Shear area - Bending about Y-axis	0.67834E+01 m2
Torsional moment of inertia I	0.69018E+02 m4
Moment of inertia about Y-axis	0.11263E+03 m4
Eccentricity Z (Shear lag)	0.00000E+00 m
Shear lag factor Y	0.10000E+01
Moment of inertia about Y-axis (Shear lag)	0.11263E+03 m4
Moment of inertia about Z-axis	0.51284E+02 m4
Eccentricity Y (Shear lag)	0.00000E+00 m
Shear lag factor Z	0.10000E+01
Moment of inertia about Z-axis (Shear lag)	0.51284E+02 m4
Warping moment of inertia	0.47911E+02 m6
Bending axis origin - Eccentricity ey	-2.31278 m
Bending axis origin - Eccentricity ez	0.01629 m
Main axis angle	1.60039 Deg
Shear axis origin - Eccentricity ey	-3.17593 m
Shear axis origin - Eccentricity ez	0.01175 m
Main axis angle - Shear	0.00000 Deg
Y-below : Gravity centre - minY	2.76022 m
Y-above : Gravity centre - maxY	2.43978 m
Z-left : Gravity centre - minZ	6.35129 m
Z-right : Gravity centre - maxZ	6.31871 m
Perimeter exposed to drying (outside)	33.86000 m
Perimeter (inside)	17.40832 m

Cross-section : Box011

Part : 1 Variant : 1

Description : Box



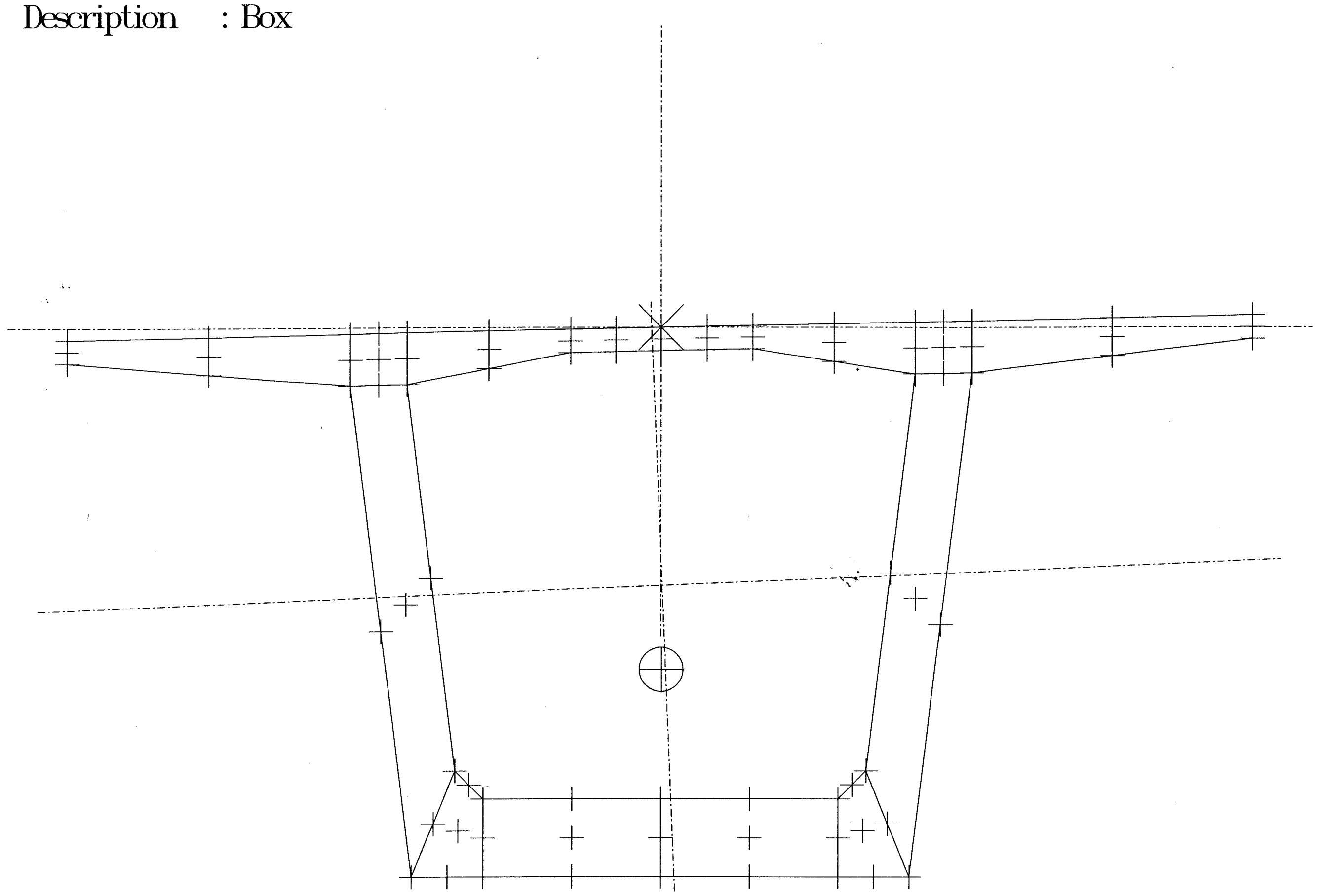
C R O S S - S E C T I O N D A T A

Cross-section area	0.11472E+02 m2
Shear area - Bending about Z-axis	0.33167E+01 m2
Shear area - Bending about Y-axis	0.63293E+01 m2
Torsional moment of inertia I	0.54131E+02 m4
Moment of inertia about Y-axis	0.99730E+02 m4
Eccentricity Z (Shear lag)	0.00000E+00 m
Shear lag factor Y	0.10000E+01
Moment of inertia about Y-axis (Shear lag)	0.99730E+02 m4
Moment of inertia about Z-axis	0.40487E+02 m4
Eccentricity Y (Shear lag)	0.00000E+00 m
Shear lag factor Z	0.10000E+01
Moment of inertia about Z-axis (Shear lag)	0.40487E+02 m4
Warping moment of inertia	0.31061E+02 m6
Bending axis origin - Eccentricity ey	-2.10912 m
Bending axis origin - Eccentricity ez	0.01399 m
Main axis angle	1.50322 Deg
Shear axis origin - Eccentricity ey	-2.76725 m
Shear axis origin - Eccentricity ez	0.00587 m
Main axis angle - Shear	0.00000 Deg
Y-below : Gravity centre - minY	2.61088 m
Y-above : Gravity centre - maxY	2.23612 m
Z-left : Gravity centre - minZ	6.34899 m
Z-right : Gravity centre - maxZ	6.32101 m
Perimeter exposed to drying (outside)	33.23653 m
Perimeter (inside)	17.69663 m

Cross-section : Box014

Part : 1 Variant : 1

Description : Box



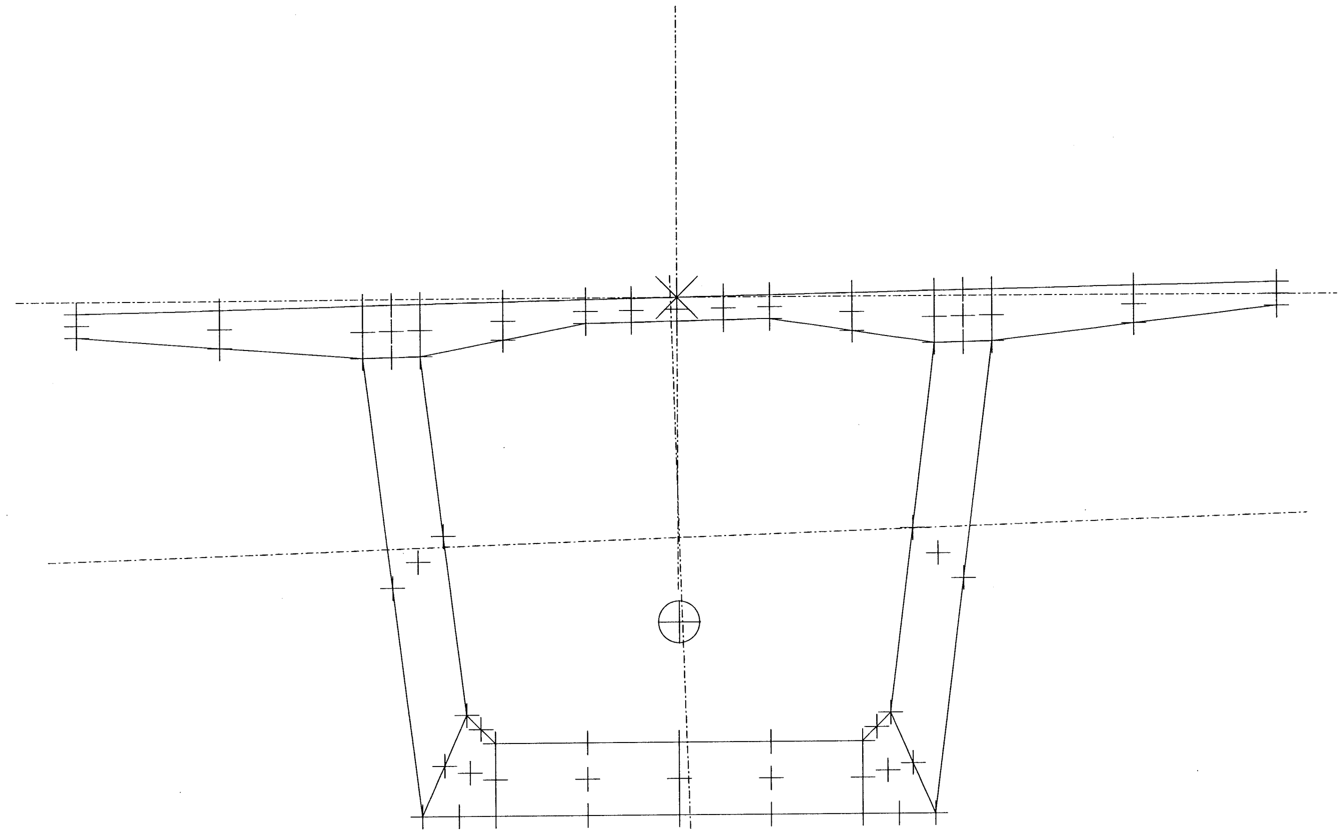
C R O S S - S E C T I O N D A T A

Cross-section area	0.14970E+02 m2
Shear area - Bending about Z-axis	0.61773E+01 m2
Shear area - Bending about Y-axis	0.68733E+01 m2
Torsional moment of inertia I	0.87405E+02 m4
Moment of inertia about Y-axis	0.11833E+03 m4
Eccentricity Z (Shear lag)	0.00000E+00 m
Shear lag factor Y	0.10000E+01
Moment of inertia about Y-axis (Shear lag)	0.11833E+03 m4
Moment of inertia about Z-axis	0.74630E+02 m4
Eccentricity Y (Shear lag)	0.00000E+00 m
Shear lag factor Z	0.10000E+01
Moment of inertia about Z-axis (Shear lag)	0.74630E+02 m4
Warping moment of inertia	0.67555E+02 m6
Bending axis origin - Eccentricity ey	-2.74799 m
Bending axis origin - Eccentricity ez	0.01552 m
Main axis angle	2.36690 Deg
Shear axis origin - Eccentricity ey	-3.65451 m
Shear axis origin - Eccentricity ez	0.00616 m
Main axis angle - Shear	0.00000 Deg
Y-below : Gravity centre - minY	3.11501 m
Y-above : Gravity centre - maxY	2.87499 m
Z-left : Gravity centre - minZ	6.35052 m
Z-right : Gravity centre - maxZ	6.31948 m
Perimeter exposed to drying (outside)	35.25610 m
Perimeter (inside)	18.59653 m

Cross-section : Box013

Part : 1 Variant : 1

Description : Box



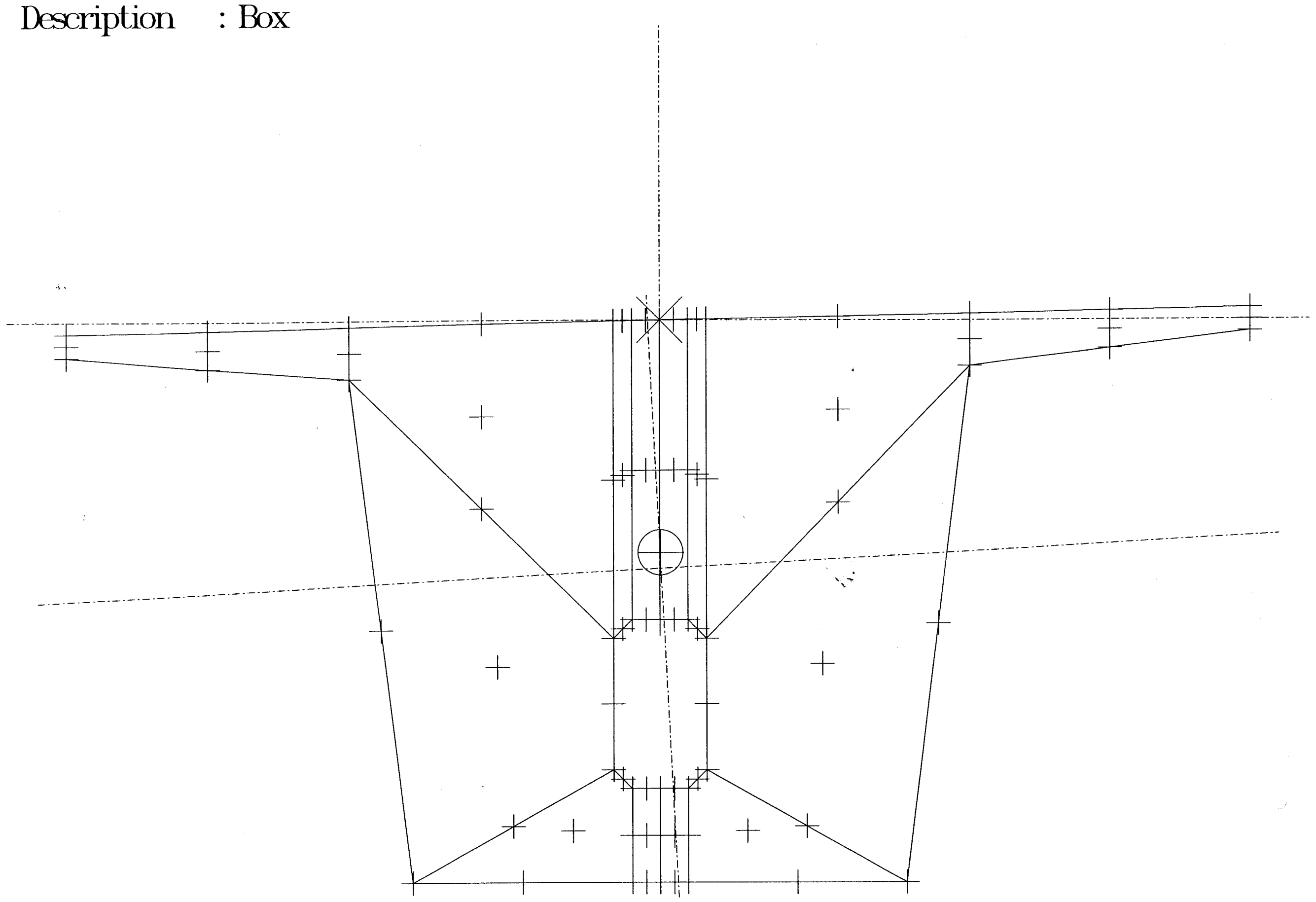
C R O S S - S E C T I O N D A T A

Cross-section area	0.14297E+02 m2
Shear area - Bending about Z-axis	0.56484E+01 m2
Shear area - Bending about Y-axis	0.68472E+01 m2
Torsional moment of inertia I	0.77820E+02 m4
Moment of inertia about Y-axis	0.11545E+03 m4
Eccentricity Z (Shear lag)	0.00000E+00 m
Shear lag factor Y	0.10000E+01
Moment of inertia about Y-axis (Shear lag)	0.11545E+03 m4
Moment of inertia about Z-axis	0.61895E+02 m4
Eccentricity Y (Shear lag)	0.00000E+00 m
Shear lag factor Z	0.10000E+01
Moment of inertia about Z-axis (Shear lag)	0.61895E+02 m4
Warping moment of inertia	0.56273E+02 m6
Bending axis origin - Eccentricity ey	-2.52296 m
Bending axis origin - Eccentricity ez	0.01561 m
Main axis angle	1.88261 Deg
Shear axis origin - Eccentricity ey	-3.41170 m
Shear axis origin - Eccentricity ez	0.00811 m
Main axis angle - Shear	0.00000 Deg
Y-below : Gravity centre - minY	2.92904 m
Y-above : Gravity centre - maxY	2.64996 m
Z-left : Gravity centre - minZ	6.35061 m
Z-right : Gravity centre - maxZ	6.31939 m
Perimeter exposed to drying (outside)	34.52980 m
Perimeter (inside)	17.97330 m

Cross-section : Box016

Part : 1 Variant : 1

Description : Box



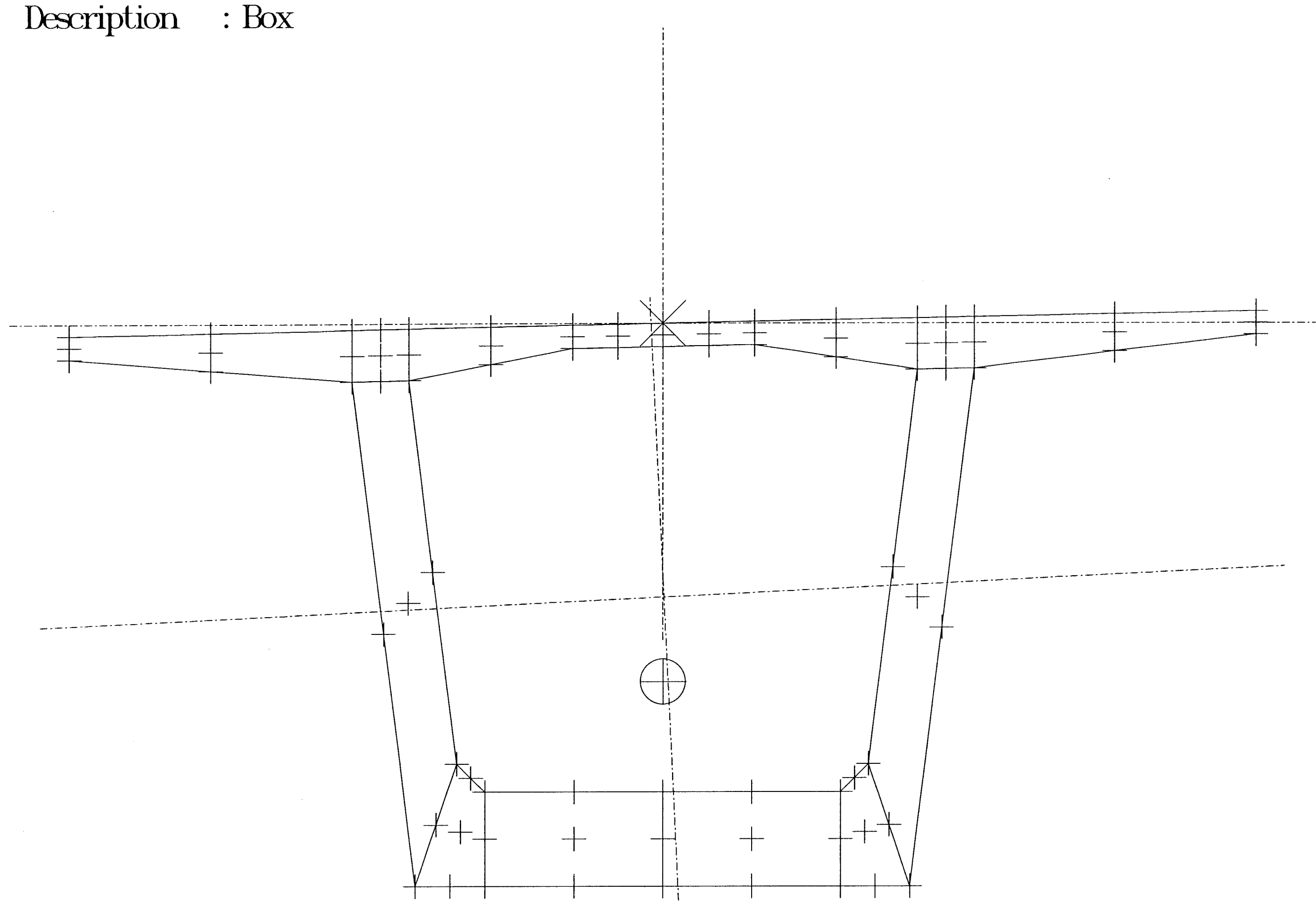
C R O S S - S E C T I O N D A T A

Cross-section area	0.36949E+02 m2
Shear area - Bending about Z-axis	0.26714E+02 m2
Shear area - Bending about Y-axis	0.27344E+02 m2
Torsional moment of inertia I	0.18247E+03 m4
Moment of inertia about Y-axis	0.16548E+03 m4
Eccentricity Z (Shear lag)	0.00000E+00 m
Shear lag factor Y	0.10000E+01
Moment of inertia about Y-axis (Shear lag)	0.16548E+03 m4
Moment of inertia about Z-axis	0.12033E+03 m4
Eccentricity Y (Shear lag)	0.00000E+00 m
Shear lag factor Z	0.10000E+01
Moment of inertia about Z-axis (Shear lag)	0.12033E+03 m4
Warping moment of inertia	0.34427E+02 m6
Bending axis origin - Eccentricity ey	-2.64821 m
Bending axis origin - Eccentricity ez	0.01354 m
Main axis angle	3.03769 Deg
Shear axis origin - Eccentricity ey	-2.48281 m
Shear axis origin - Eccentricity ez	0.00872 m
Main axis angle - Shear	0.00000 Deg
Y-below : Gravity centre - minY	3.35179 m
Y-above : Gravity centre - maxY	2.77521 m
Z-left : Gravity centre - minZ	6.34854 m
Z-right : Gravity centre - maxZ	6.32146 m
Perimeter exposed to drying (outside)	35.49645 m
Perimeter (inside)	5.13137 m

Cross-section : Box015

Part : 1 Variant : 1

Description : Box

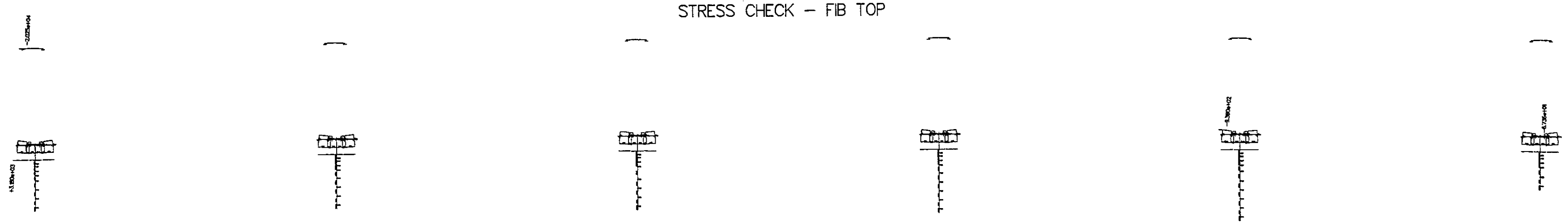


C R O S S - S E C T I O N D A T A

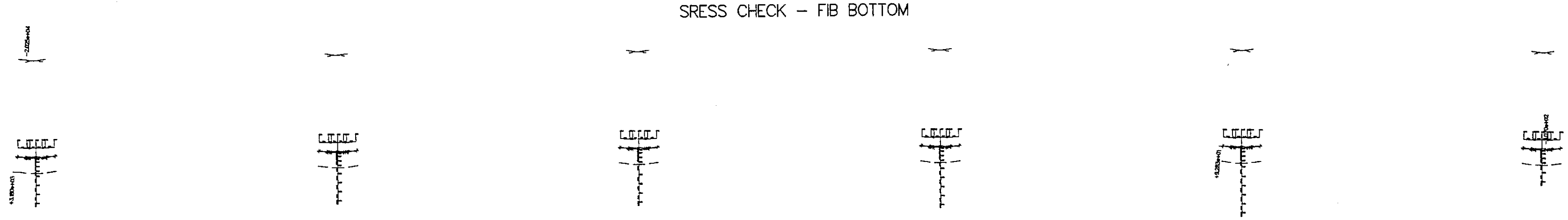
Cross-section area	0.15843E+02 m2
Shear area - Bending about Z-axis	0.64041E+01 m2
Shear area - Bending about Y-axis	0.73456E+01 m2
Torsional moment of inertia I	0.91431E+02 m4
Moment of inertia about Y-axis	0.12028E+03 m4
Eccentricity Z (Shear lag)	0.00000E+00 m
Shear lag factor Y	0.10000E+01
Moment of inertia about Y-axis (Shear lag)	0.12028E+03 m4
Moment of inertia about Z-axis	0.82295E+02 m4
Eccentricity Y (Shear lag)	0.00000E+00 m
Shear lag factor Z	0.10000E+01
Moment of inertia about Z-axis (Shear lag)	0.82295E+02 m4
Warping moment of inertia	0.77193E+02 m6
Bending axis origin - Eccentricity ey	-2.91471 m
Bending axis origin - Eccentricity ez	0.01402 m
Main axis angle	2.78242 Deg
Shear axis origin - Eccentricity ey	-3.81961 m
Shear axis origin - Eccentricity ez	0.00335 m
Main axis angle - Shear	0.00000 Deg
Y-below : Gravity centre - minY	3.08529 m
Y-above : Gravity centre - maxY	3.04171 m
Z-left : Gravity centre - minZ	6.34902 m
Z-right : Gravity centre - maxZ	6.32098 m
Perimeter exposed to drying (outside)	35.49645 m
Perimeter (inside)	18.53629 m

Mz – BENDING MOMENT
(SC: 1/10000 Secondary)

STRESS CHECK – FIB TOP



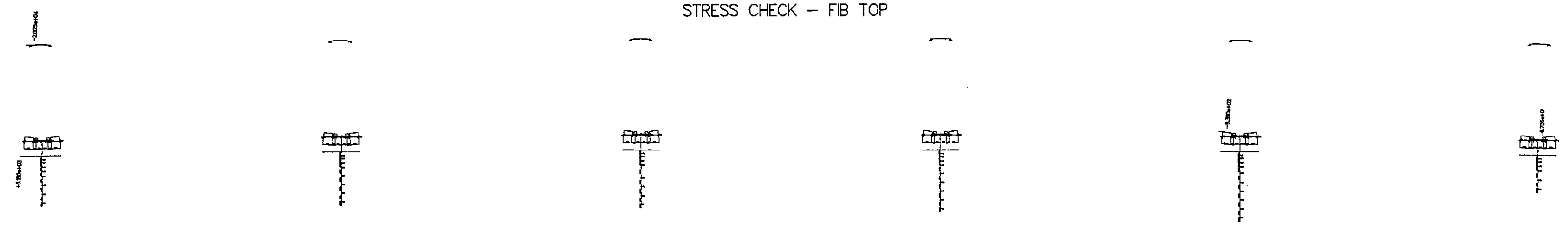
SRESS CHECK – FIB BOTTOM



Ky Lam Bridge
Construction stage CS 1

Mz – BENDING MOMENT
(SC: 1/10000 Secondary)

STRESS CHECK – FIB TOP

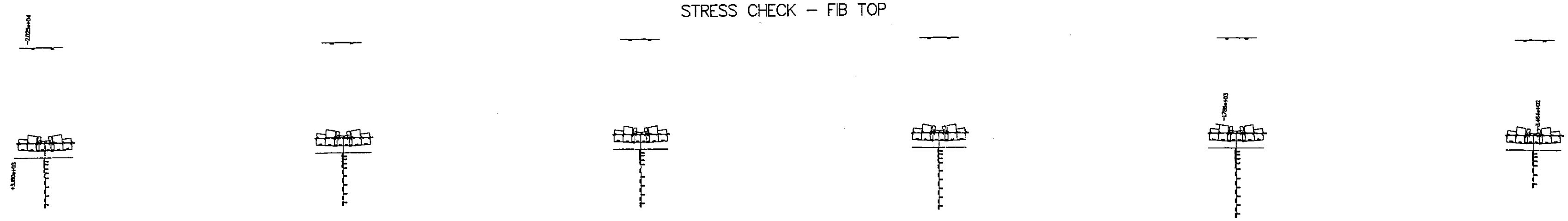


SRESS CHECK – FIB BOTTOM

Ky Lam Bridge
Construction stage CS 2

Mz – BENDING MOMENT
(SC: 1/10000 Secondary)

STRESS CHECK – FIB TOP



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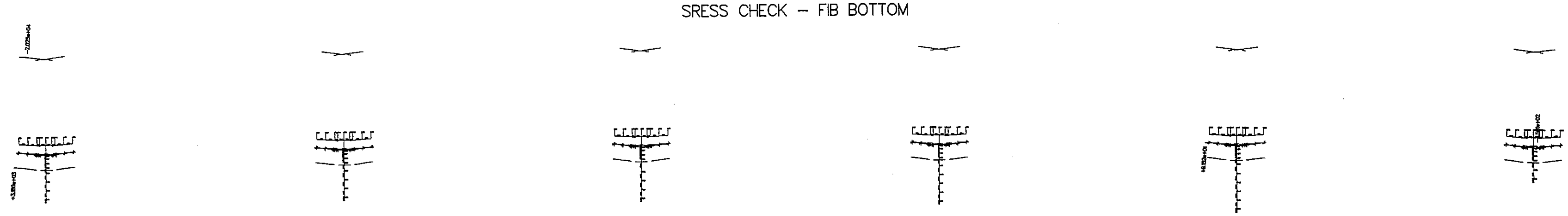
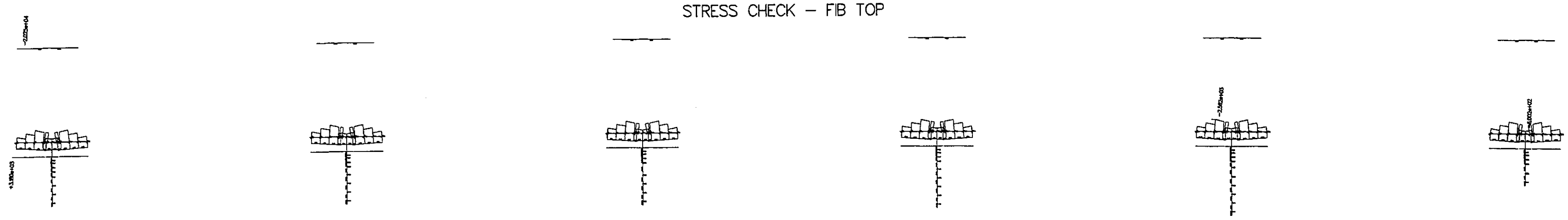


Ky Lam Bridge
Construction stage CS 3

Mz – BENDING MOMENT
(SC: 1/10000 Secondary)

STRESS CHECK – FIB TOP

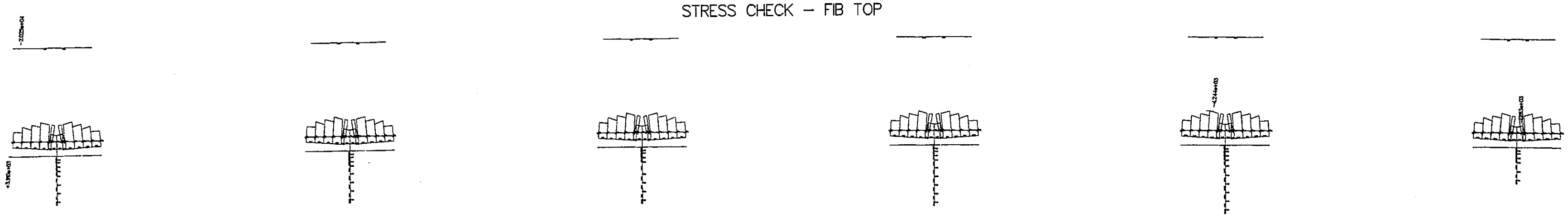


SRESS CHECK – FIB BOTTOM

Ky Lam Bridge
Construction stage CS 4

Mz – BENDING MOMENT
(SC: 1/10000 Secondary)

STRESS CHECK – FIB TOP



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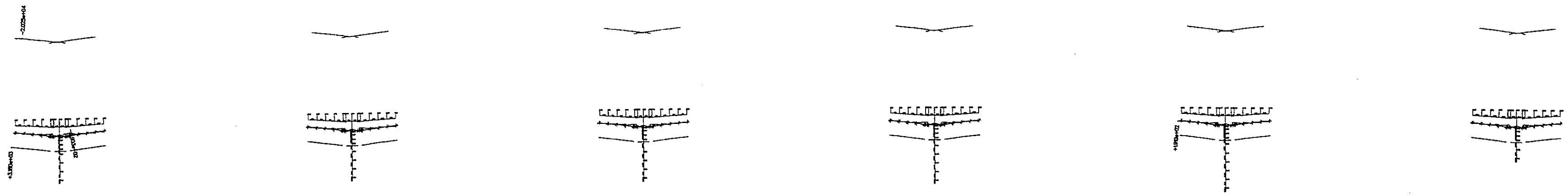
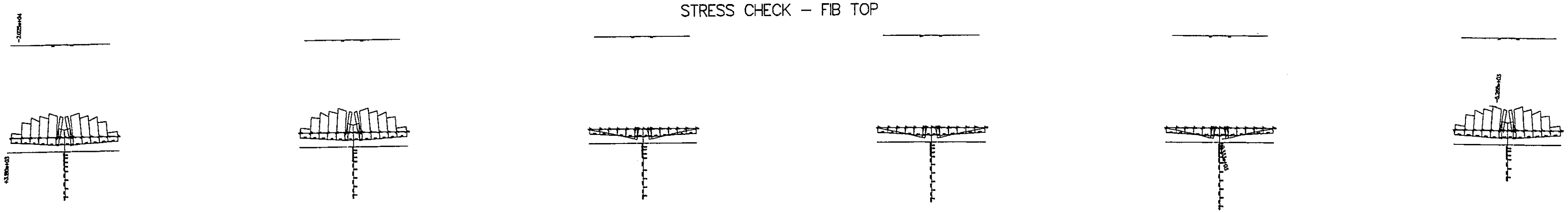


Ky Lam Bridge
Construction stage CS5

Mz – BENDING MOMENT
(SC: 1/10000 Secondary)

STRESS CHECK – FIB TOP



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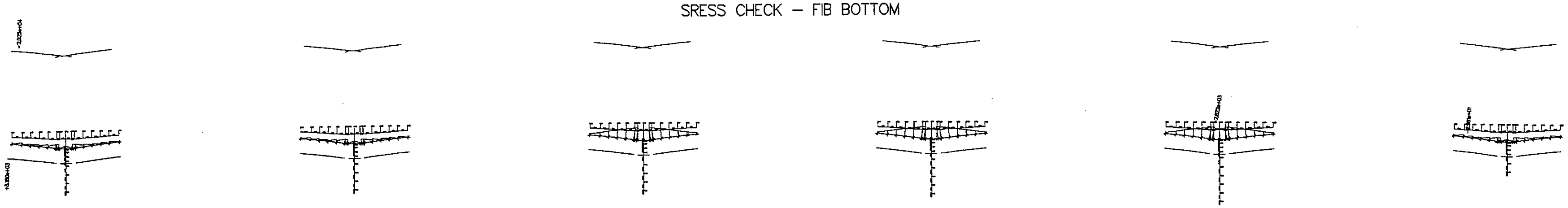
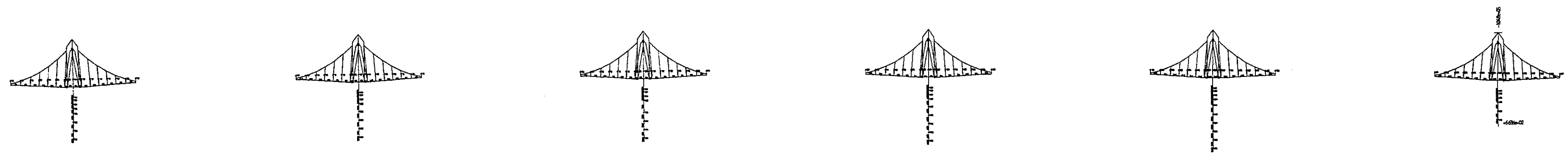


Ky Lam Bridge
Construction stage CS 6

Mz – BENDING MOMENT
(SC: 1/10000 Secondary)



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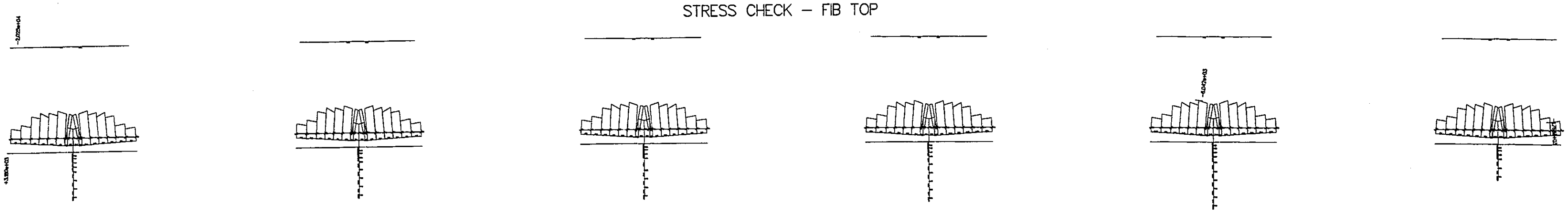

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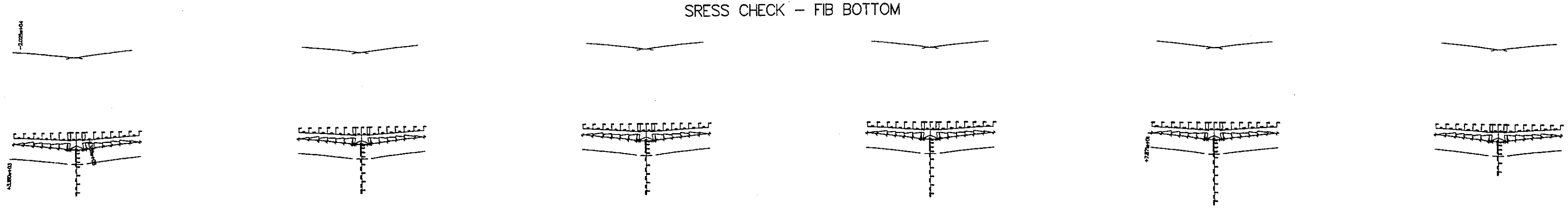
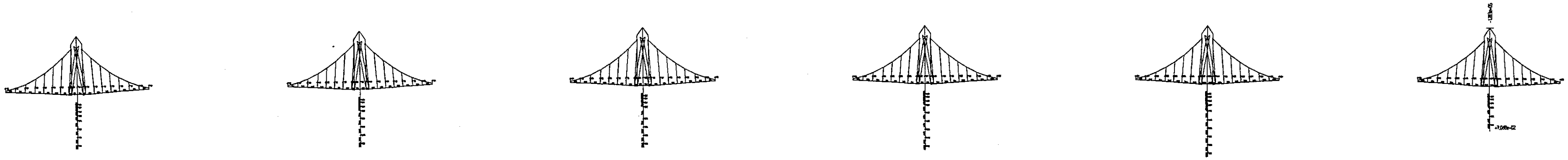
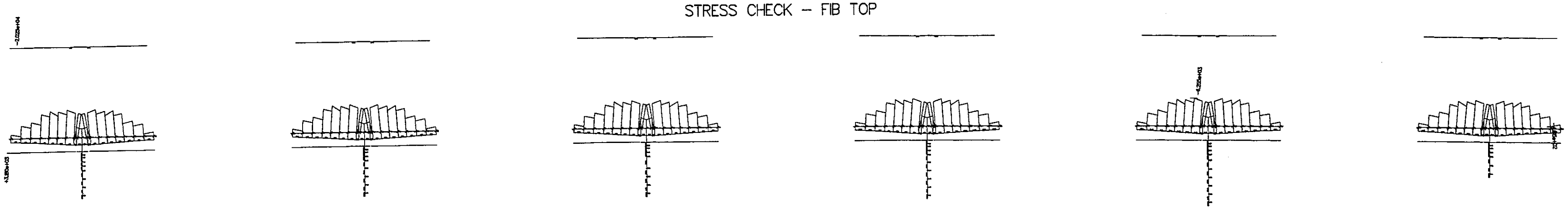


Ky Lam Bridge
Construction stage CS7

Mz – BENDING MOMENT
(SC: 1/10000 Secondary)



STRESS CHECK – FIB TOP



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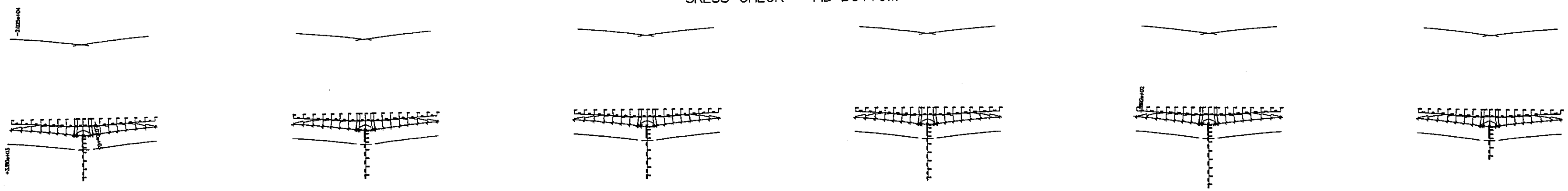
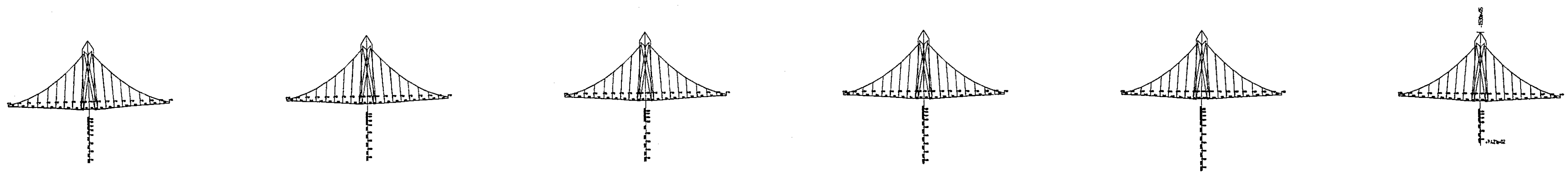
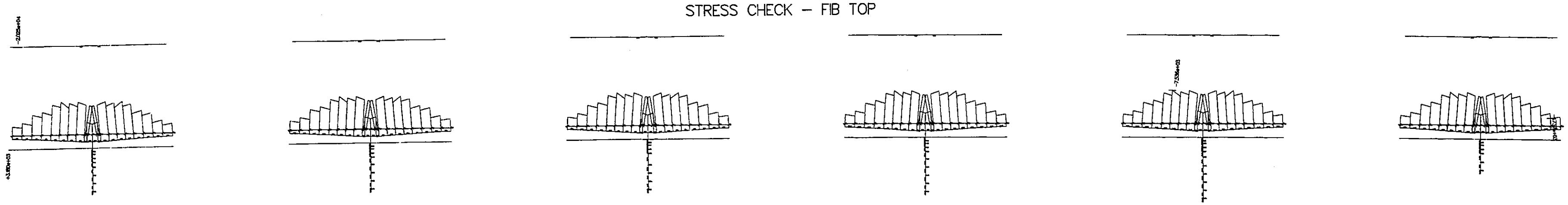


Ky Lam Bridge
Construction stage CS 8

Mz – BENDING MOMENT
(SC: 1/10000 Secondary)



STRESS CHECK – FIB TOP



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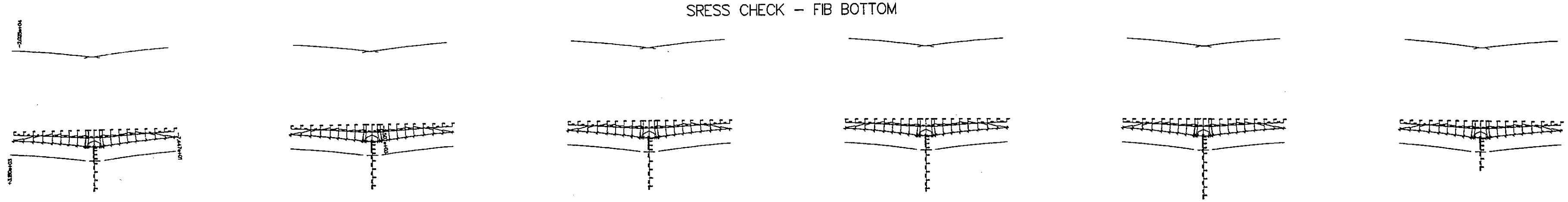
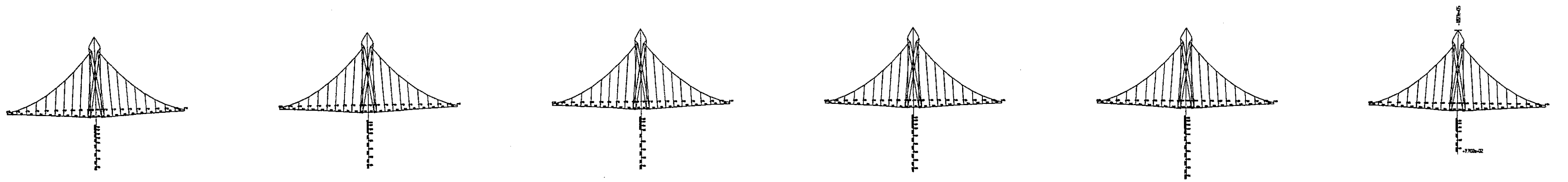
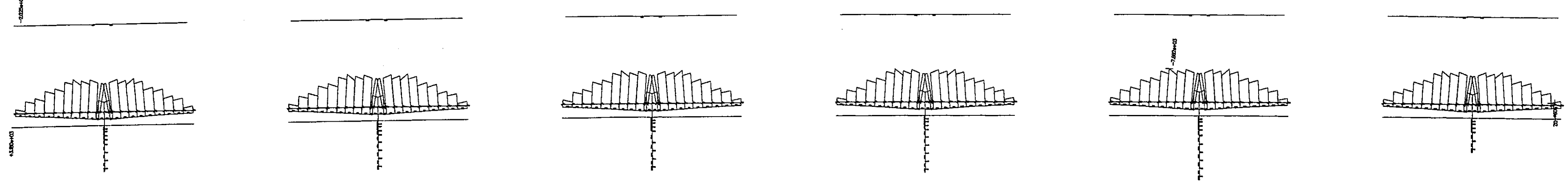


Ky Lam Bridge
Construction stage CS 9

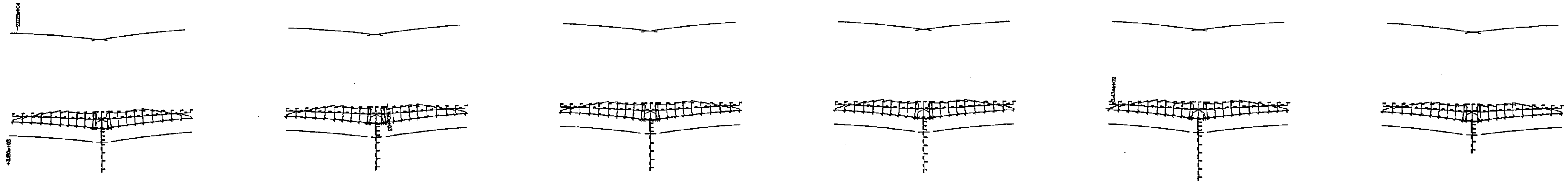
Mz – BENDING MOMENT
(SC: 1/10000 Secondary)



STRESS CHECK – FIB TOP

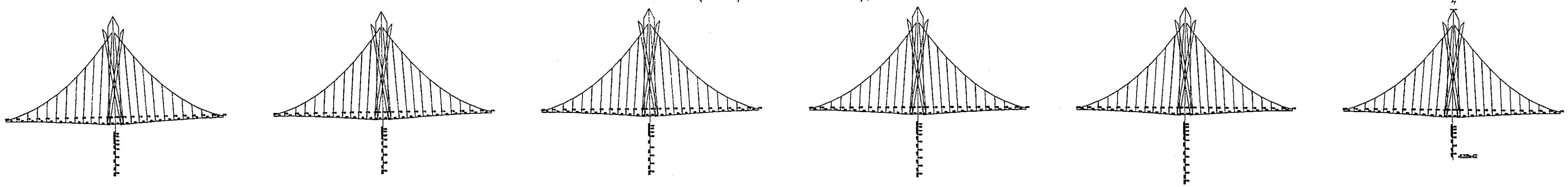


SRESS CHECK – FIB BOTTOM

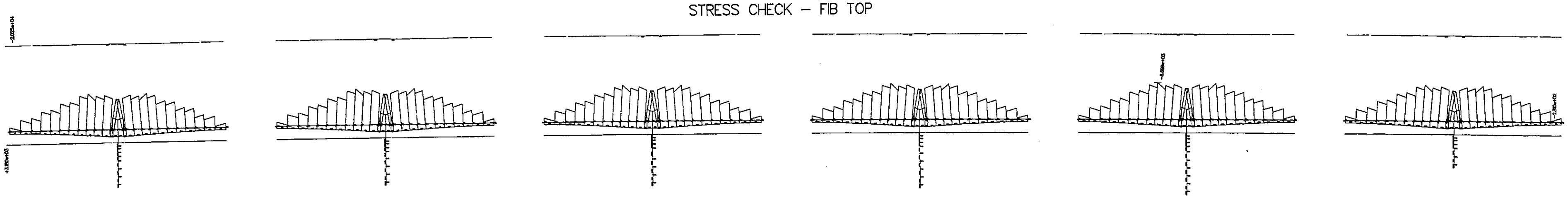


Ky Lam Bridge
Construction stage CS 10

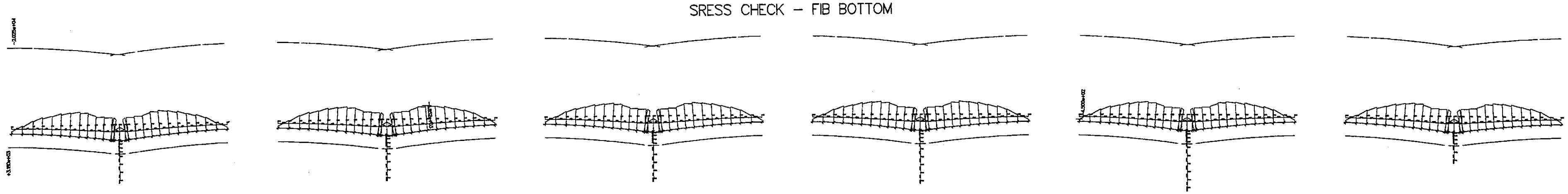
Mz – BENDING MOMENT
(SC: 1/10000 Secondary)



STRESS CHECK – FIB TOP

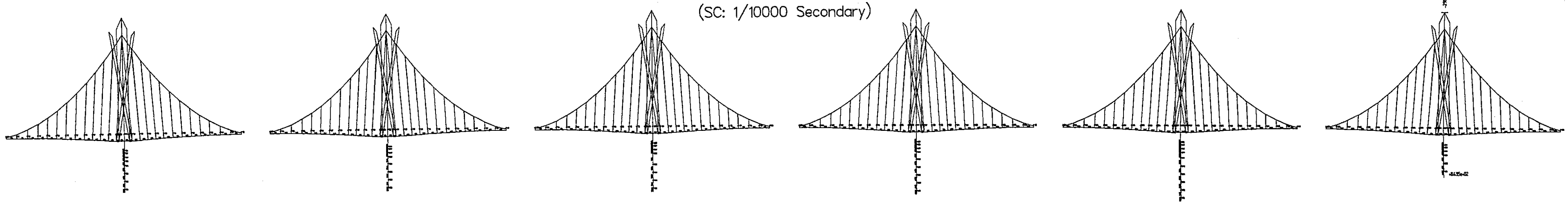


SRESS CHECK – FIB BOTTOM

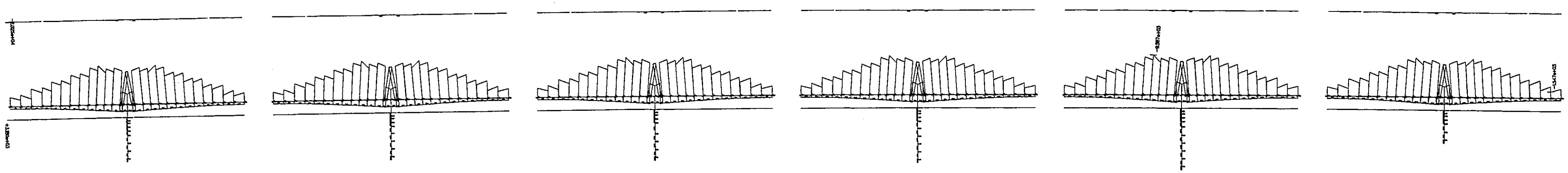


Ky Lam Bridge
Construction stage CS 11

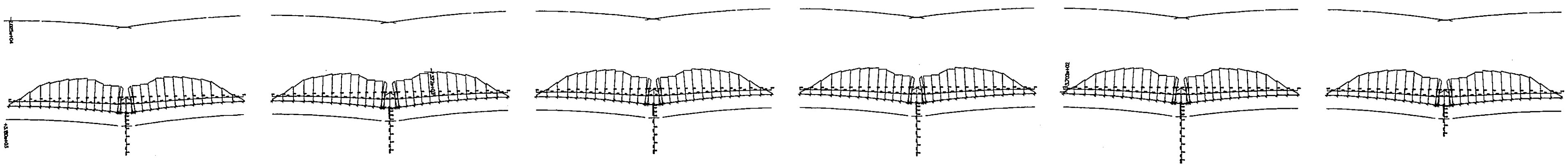
Mz – BENDING MOMENT
(SC: 1/10000 Secondary)



STRESS CHECK – FIB TOP

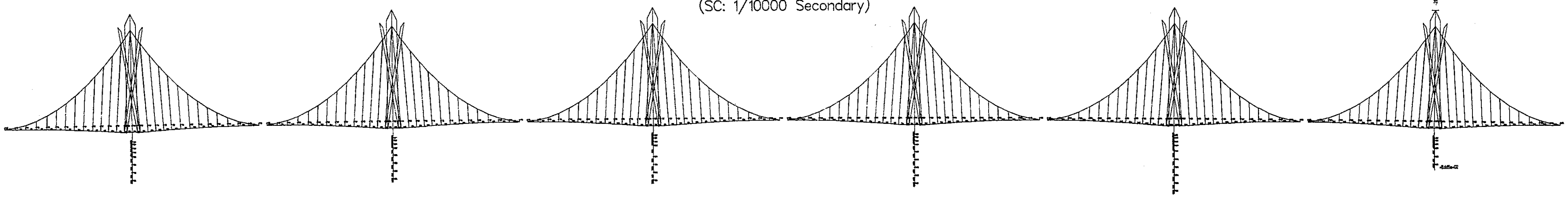


SRESS CHECK – FIB BOTTOM

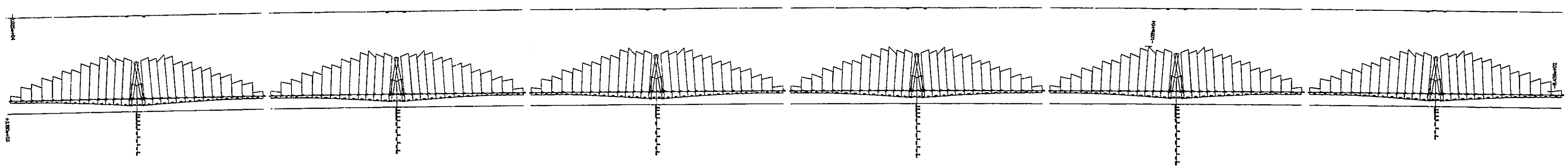


Ky Lam Bridge
Construction stage CS 12

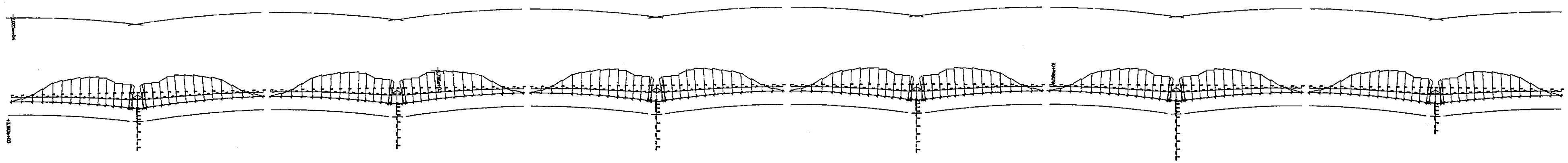
Mz – BENDING MOMENT
(SC: 1/10000 Secondary)



STRESS CHECK – FIB TOP

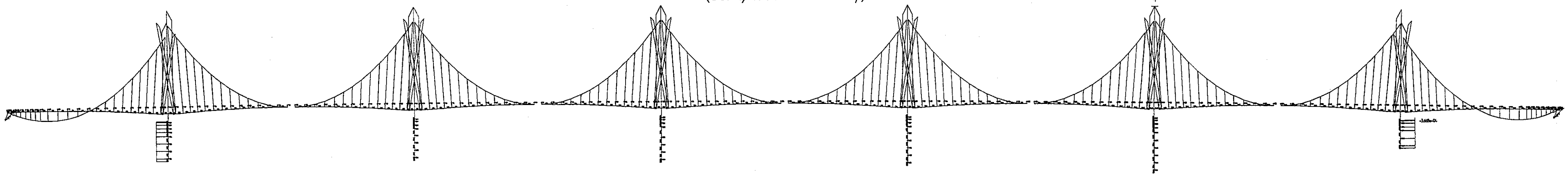


SRESS CHECK – FIB BOTTOM

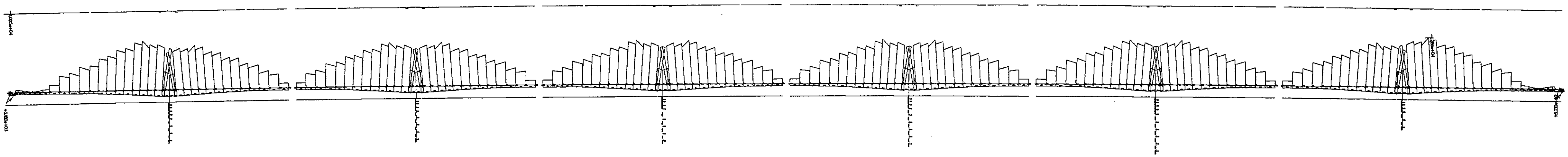


Ky Lam Bridge
Construction stage CS 13

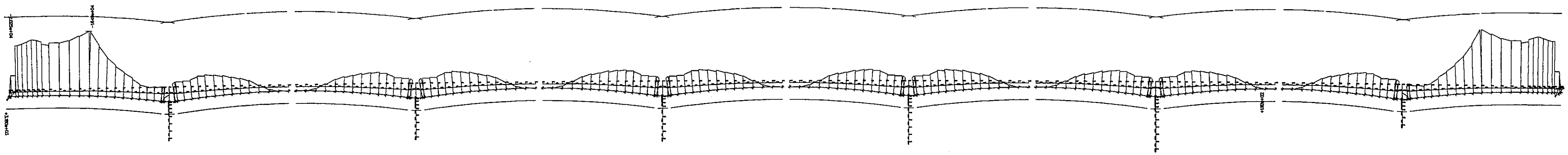
Mz – BENDING MOMENT
(SC: 1/10000 Secondary)



STRESS CHECK – FIB TOP

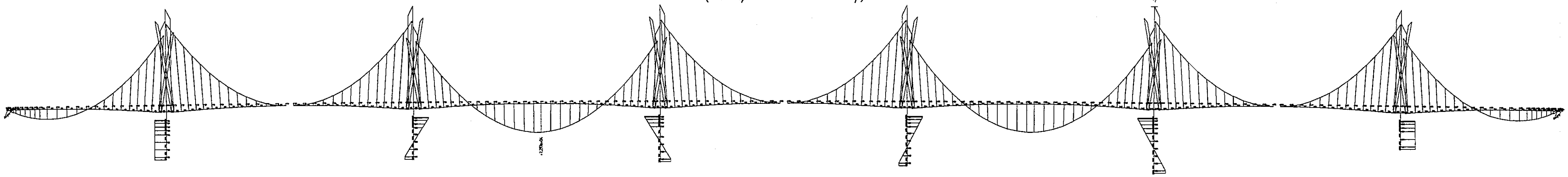


SRESS CHECK – FIB BOTTOM

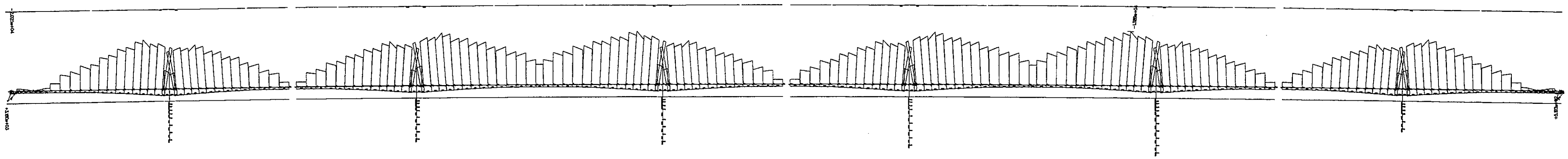


Ky Lam Bridge
Construction stage CS 15

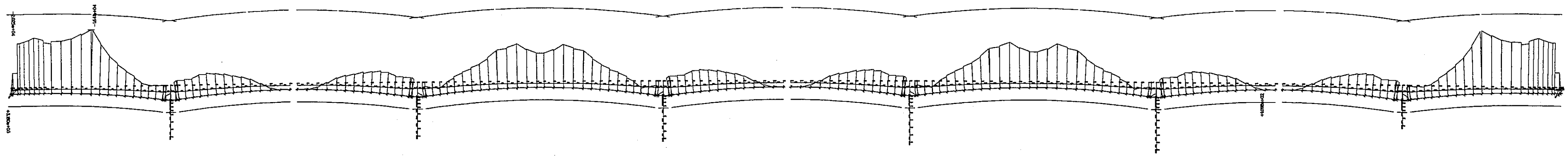
Mz – BENDING MOMENT
(SC: 1/10000 Secondary)



STRESS CHECK – FIB TOP

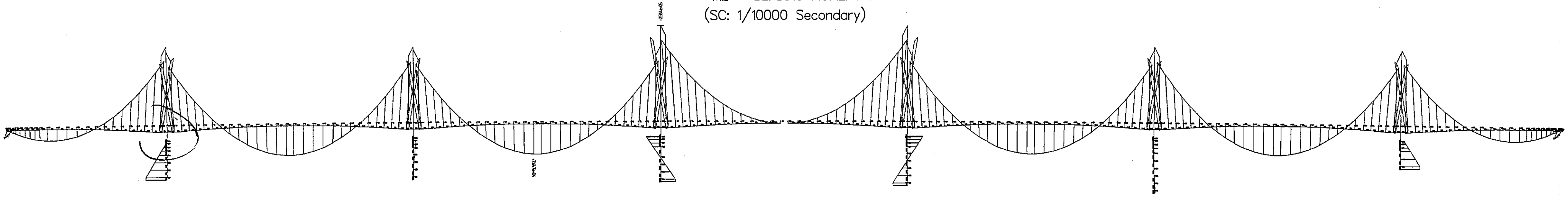


SRESS CHECK – FIB BOTTOM

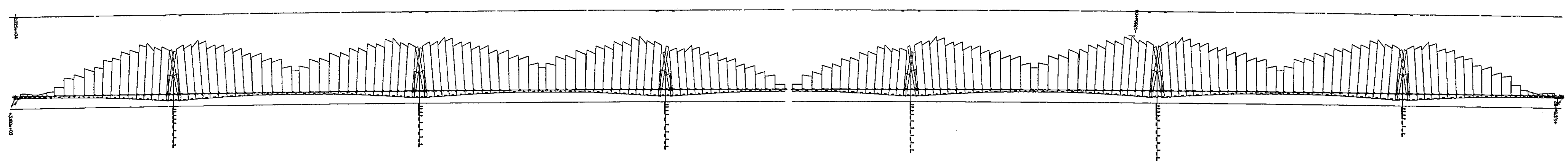


Ky Lam Bridge
Construction stage CS 16

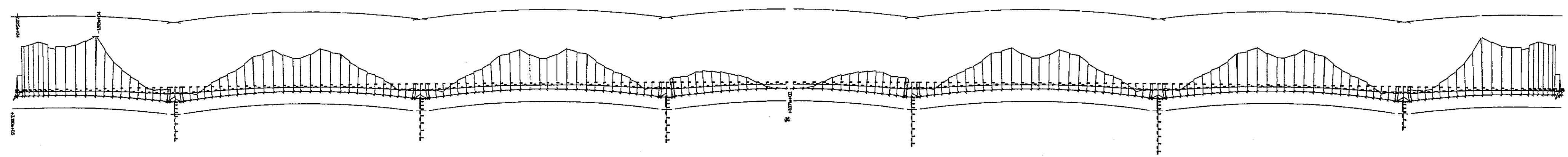
Mz – BENDING MOMENT
(SC: 1/10000 Secondary)



STRESS CHECK – FIB TOP

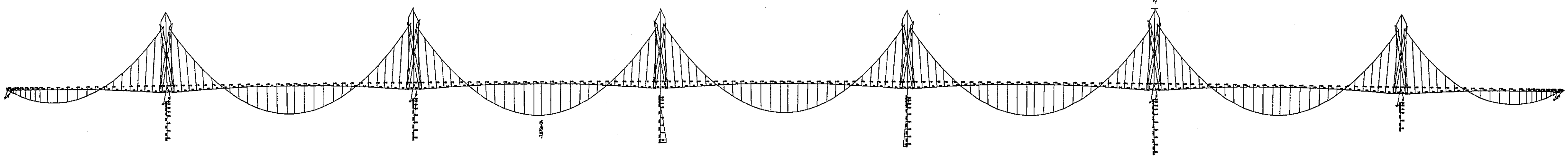


SRESS CHECK – FIB BOTTOM

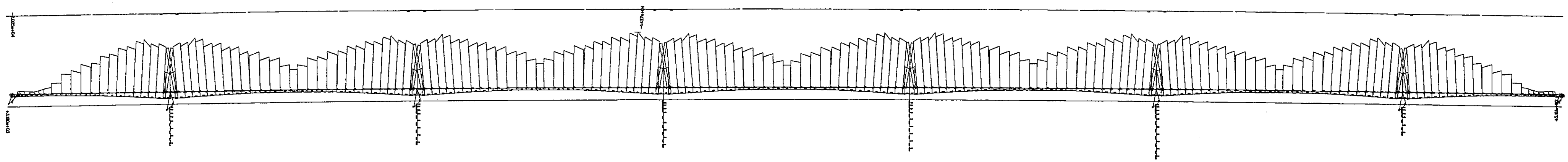


Ky Lam Bridge
Construction stage CS17

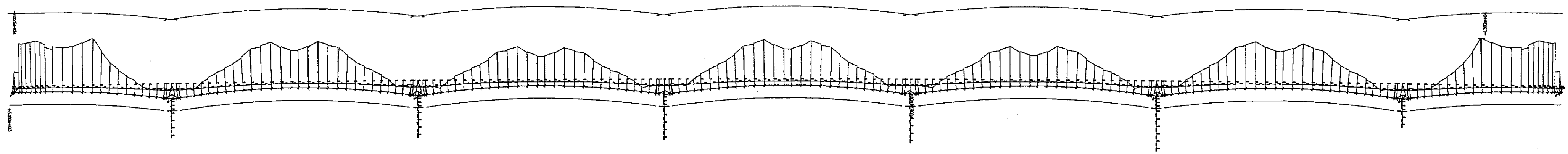
Mz – BENDING MOMENT
(SC: 1/10000 Secondary)



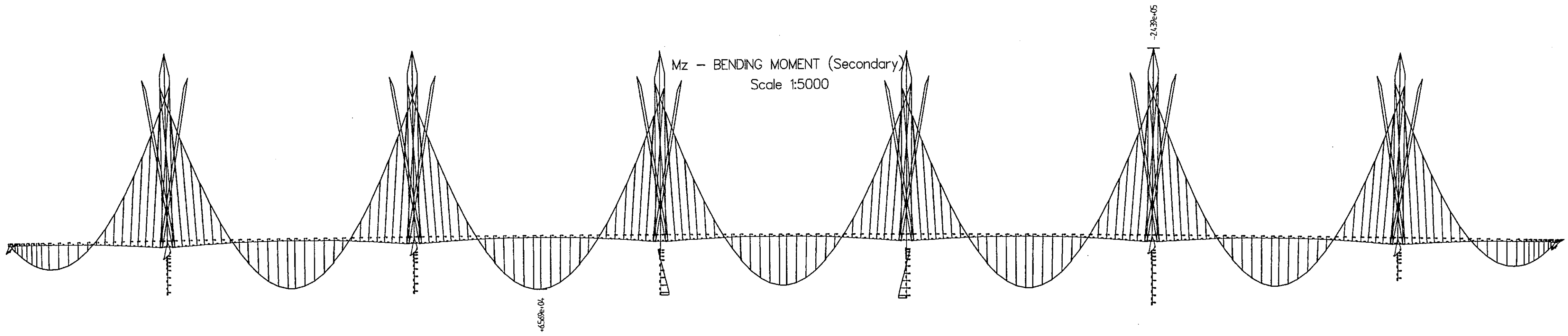
STRESS CHECK – FIB TOP

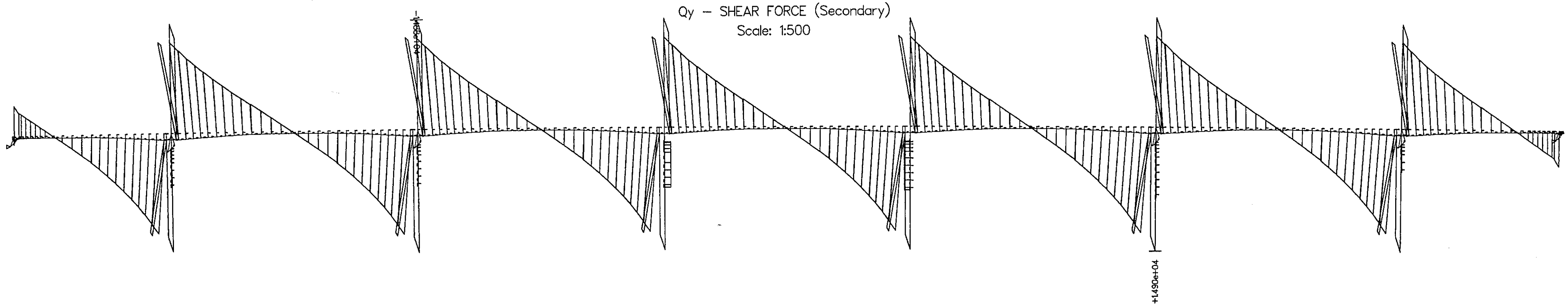


SRESS CHECK – FIB BOTTOM



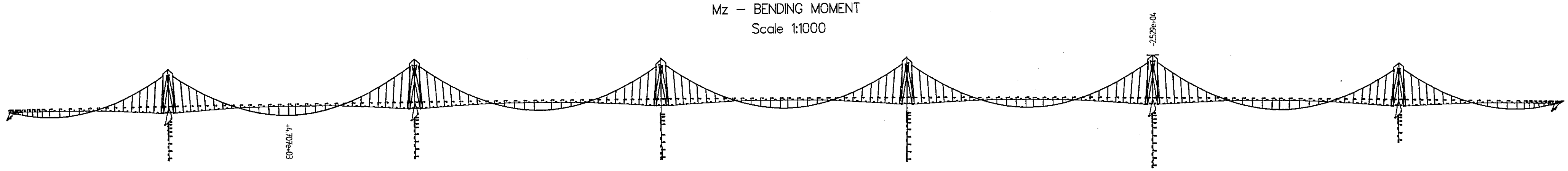
Ky Lam Bridge
Construction stage CS 18



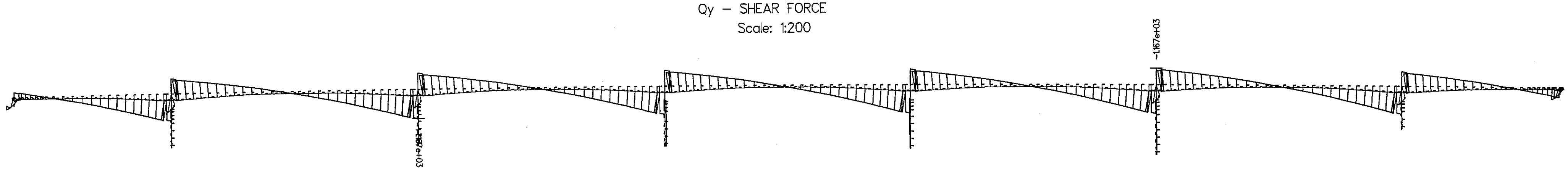


Ky Lam Bridge
DC.sup

Mz – BENDING MOMENT
Scale 1:1000

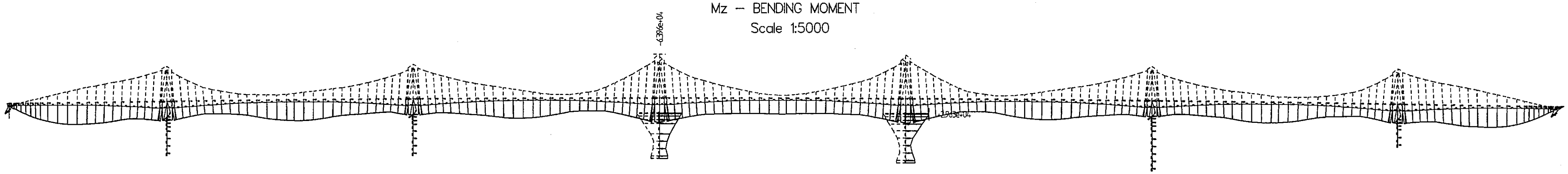


Qy – SHEAR FORCE
Scale: 1:200

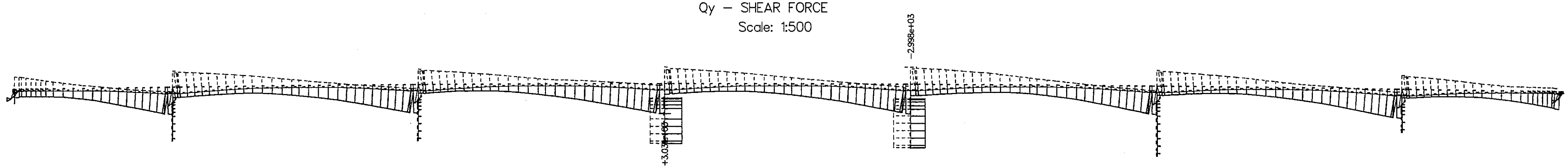


Ky Lam Bridge
DW.sup

Mz — BENDING MOMENT
Scale 1:5000

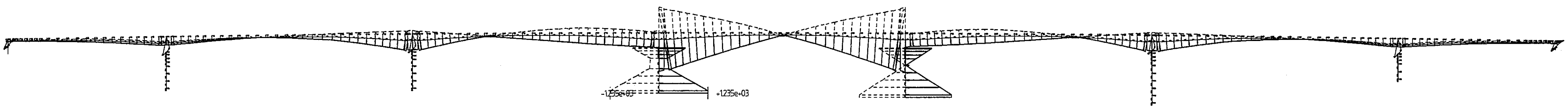


Qy — SHEAR FORCE
Scale: 1:500

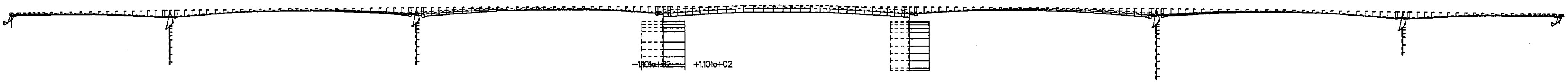


Ky Lam Bridge
LL.sup

Mz – BENDING MOMENT
Scale 1:100

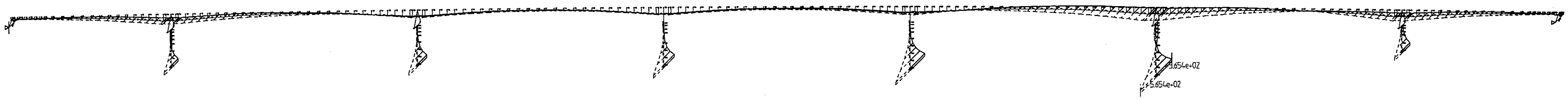


Qy – SHEAR FORCE
Scale: 1:20

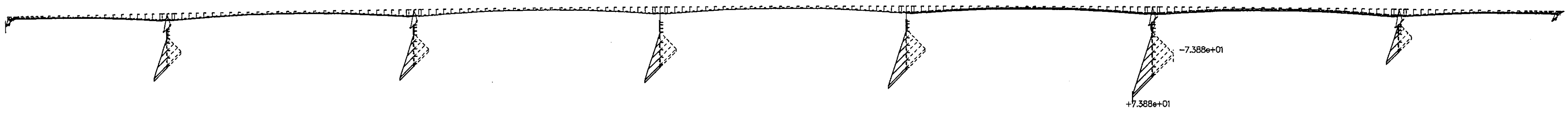


Ky Lam Bridge
BR.sup

My – BENDING MOMENT
Scale 1:100

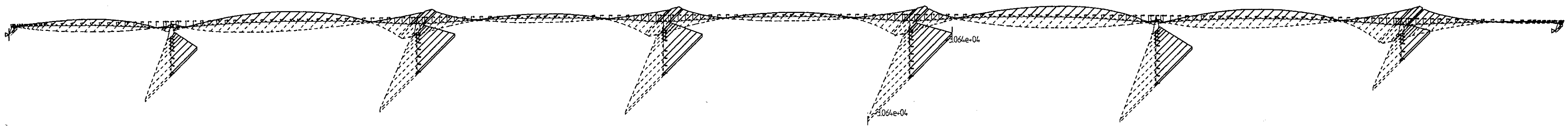


Qz – SHEAR FORCE
Scale: 1:10

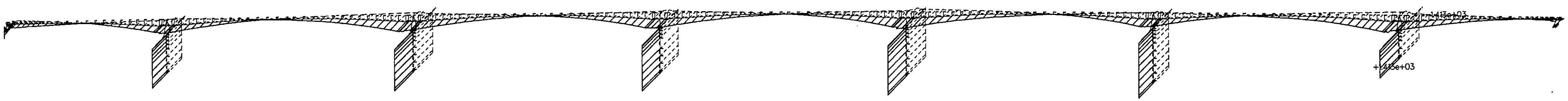


Ky Lam Bridge
WA.sup

My – BENDING MOMENT
Scale 1:2000

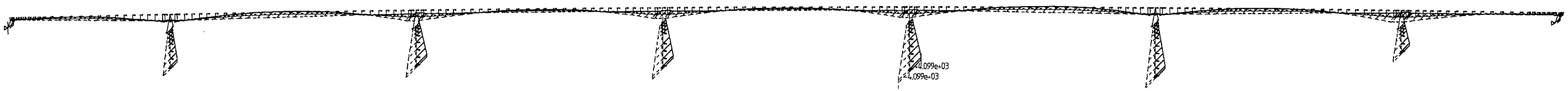


Qz – SHEAR FORCE
Scale: 1:200

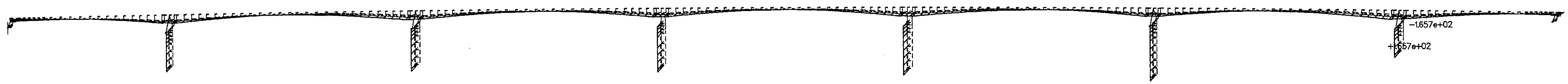


Ky Lam Bridge
WS.sup

My – BENDING MOMENT
Scale 1:1000

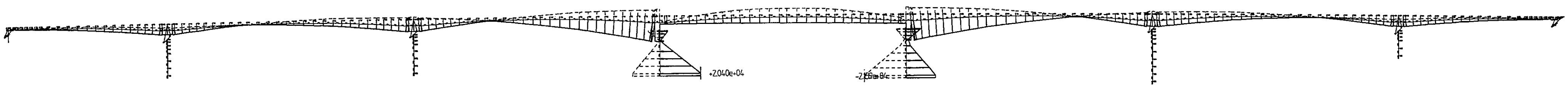


Qz – SHEAR FORCE
Scale: 1:100

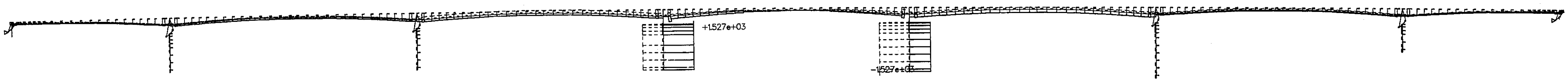


Ky Lam Bridge
WL.sup

Mz – BENDING MOMENT
Scale 1:10000

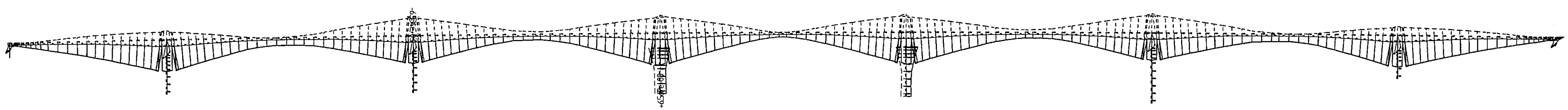


Qy – SHEAR FORCE
Scale: 1:200

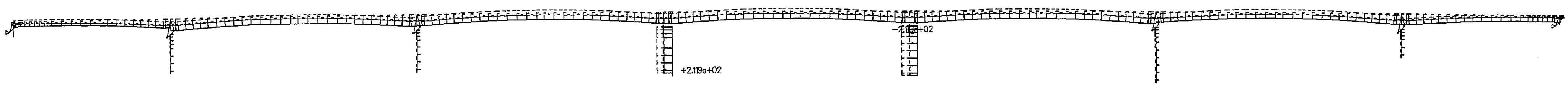


Ky Lam Bridge
TU.sup

Mz – BENDING MOMENT
Scale 1:1000

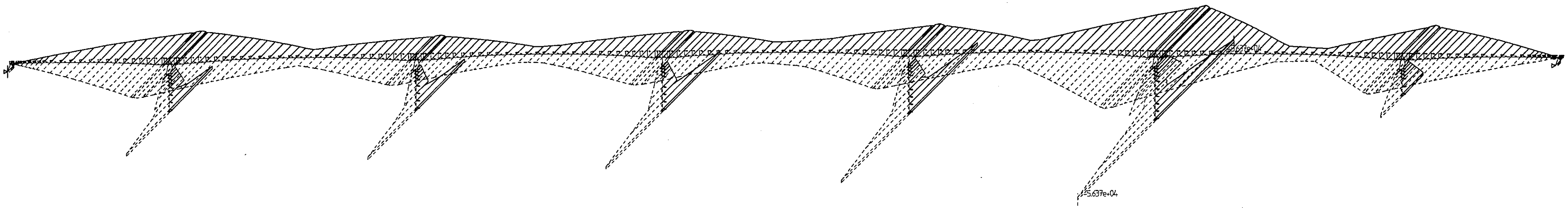


Qy – SHEAR FORCE
Scale: 1:100

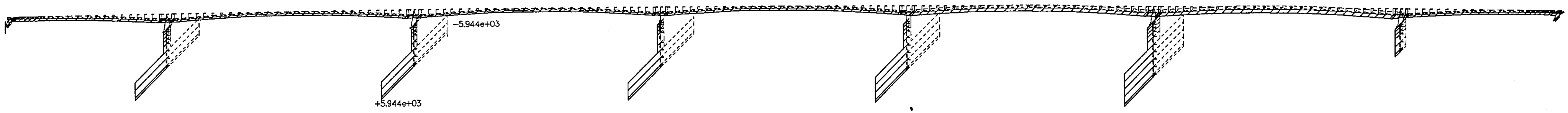


Ky Lam Bridge
SE.sup

My – BENDING MOMENT
Scale 1:2000

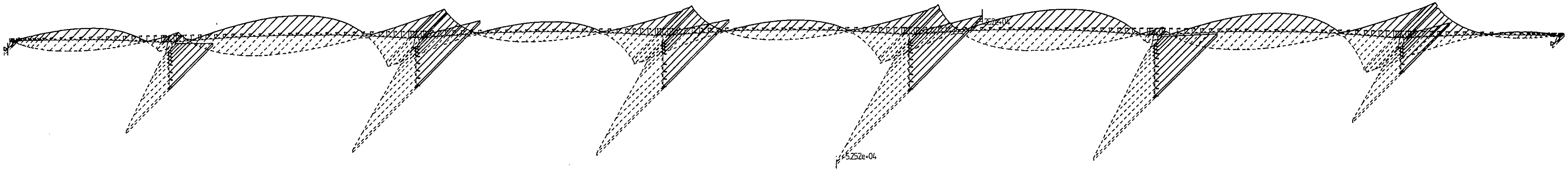


Qz – SHEAR FORCE
Scale: 1:500

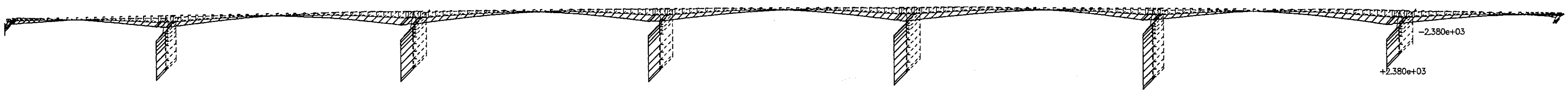


Ky Lam Bridge
CV.sup

My – BENDING MOMENT
Scale 1:2000

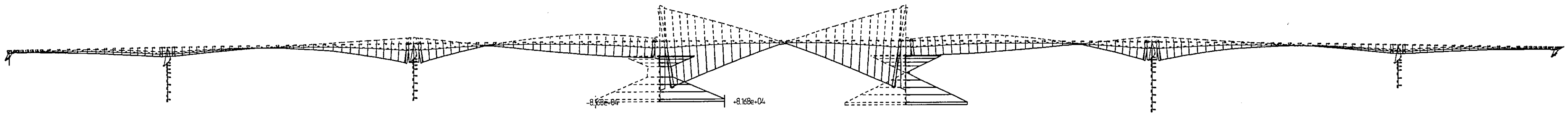


Qz – SHEAR FORCE
Scale: 1:500

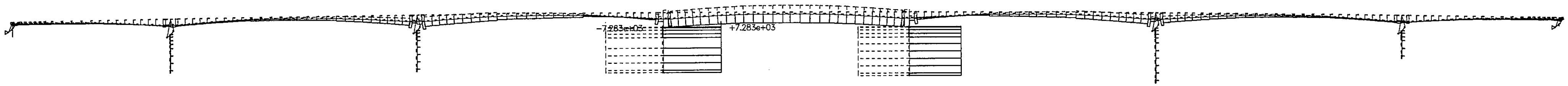


Ky Lam Bridge
EQ1.sup

Mz – BENDING MOMENT
Scale 1:2000

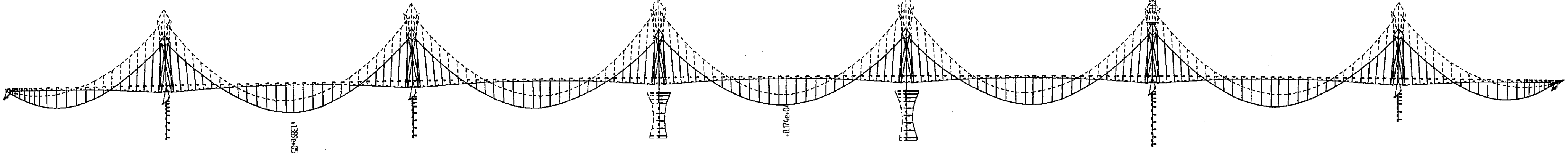


Qy – SHEAR FORCE
Scale: 1:500

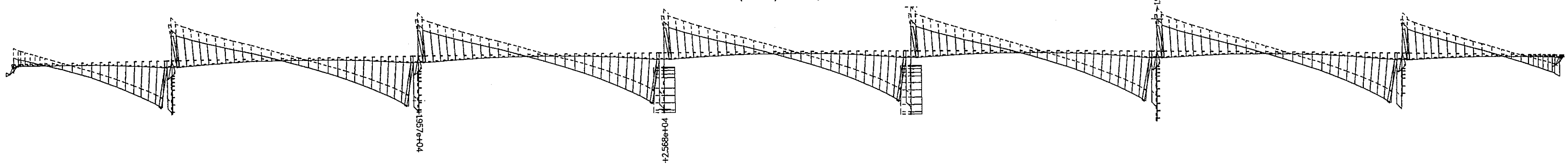


Ky Lam Bridge
EQ2.sup

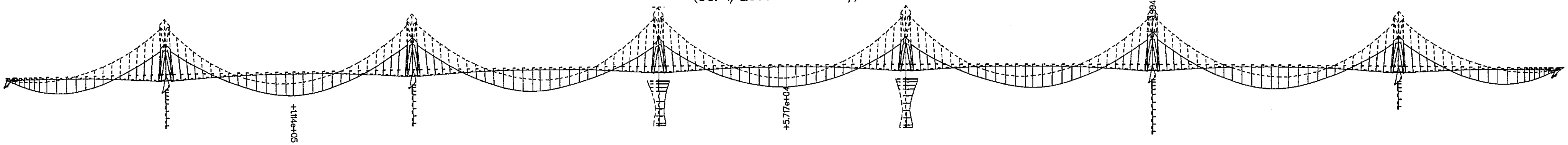
Mz – BENDING MOMENT (Strength 1a)
(SC: 1/20000 Secondary)



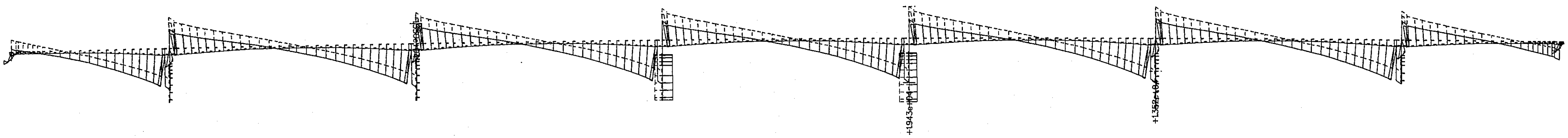
SHEAR FORCE (Strength 1a)
(SC: 1/2000)



Mz–BENDING MOMENT (Strength 1b)
(SC: 1/20000 Secondary)

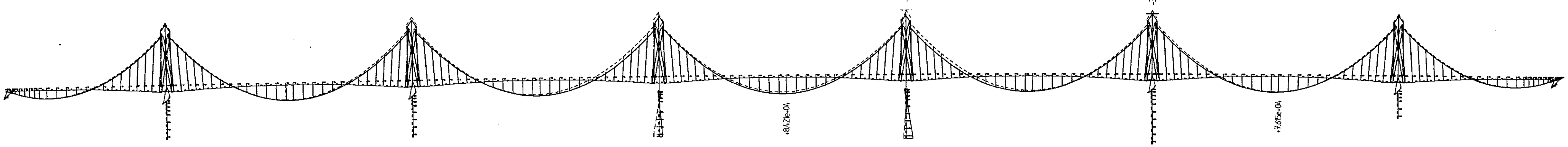


SHEAR FORCE (Strength 1b)
(SC: 1/2000)

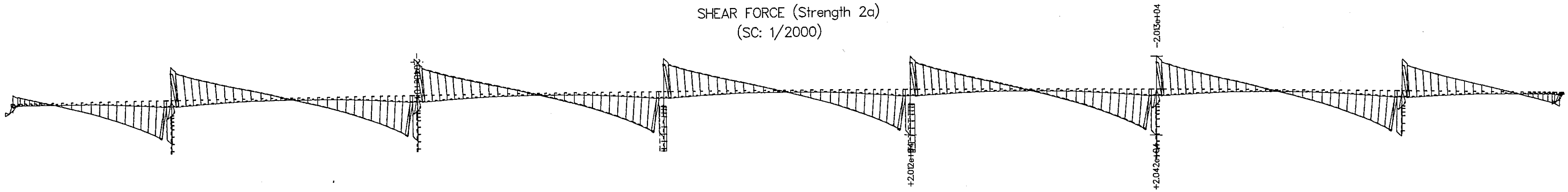


Ky Lam Bridge
Strength limit state 1

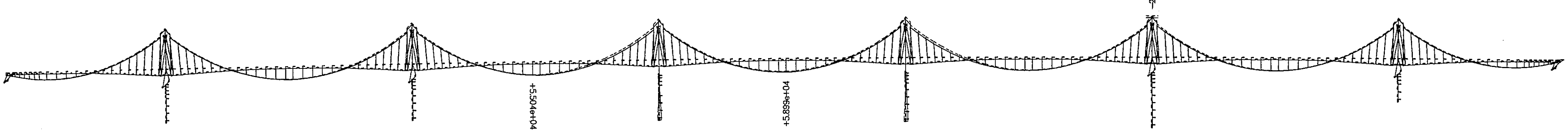
Mz – BENDING MOMENT (Strength 2a)
(SC: 1/20000 Secondary)



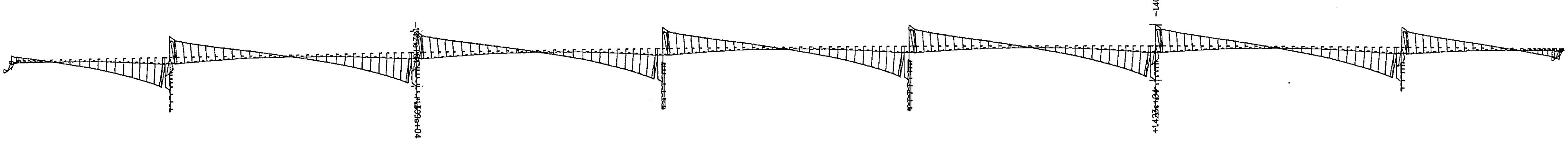
SHEAR FORCE (Strength 2a)
(SC: 1/2000)



Mz–BENDING MOMENT (Strength 2b)
(SC: 1/20000 Secondary)

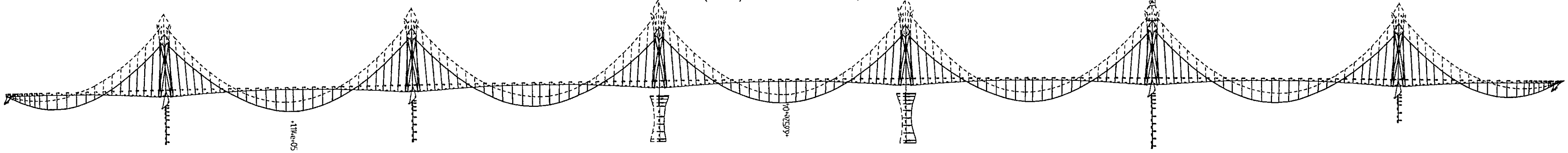


SHEAR FORCE (Strength 2b)
(SC: 1/2000)

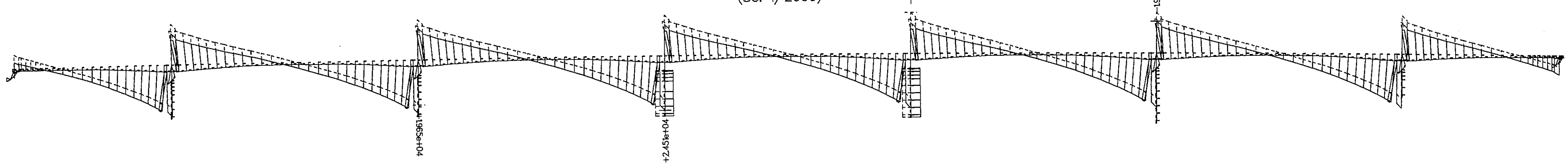


Ky Lam Bridge
Strength limit state 2

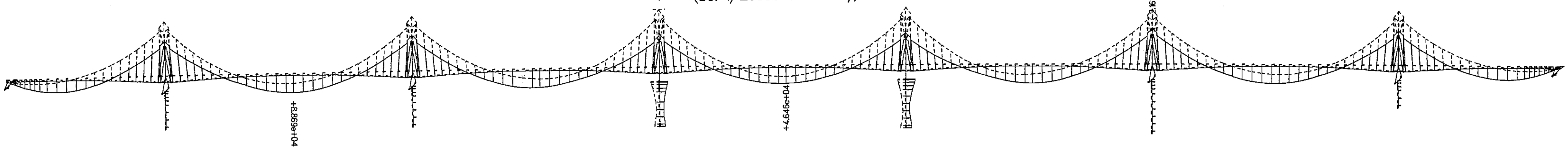
Mz – BENDING MOMENT (Strength 3a)
(SC: 1/20000 Secondary)



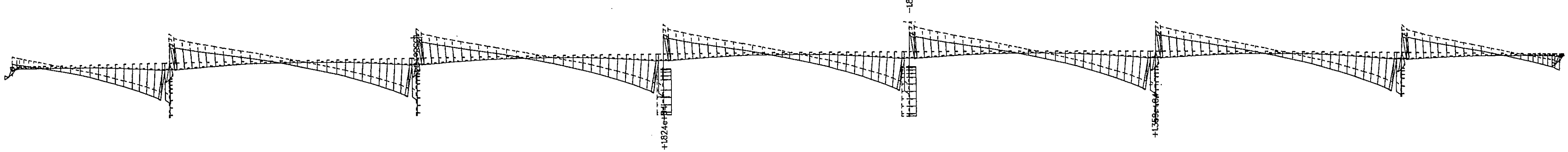
SHEAR FORCE (Strength 3a)
(SC: 1/2000)



Mz–BENDING MOMENT (Strength 3b)
(SC: 1/20000 Secondary)

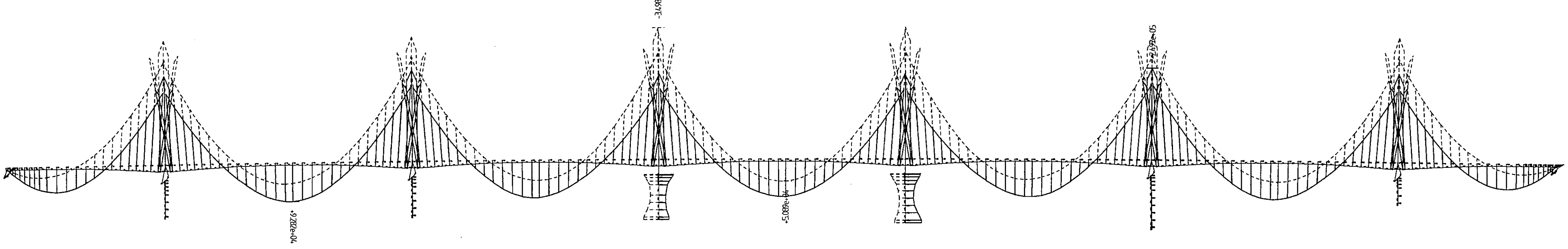


SHEAR FORCE (Strength 3b)
(SC: 1/2000)

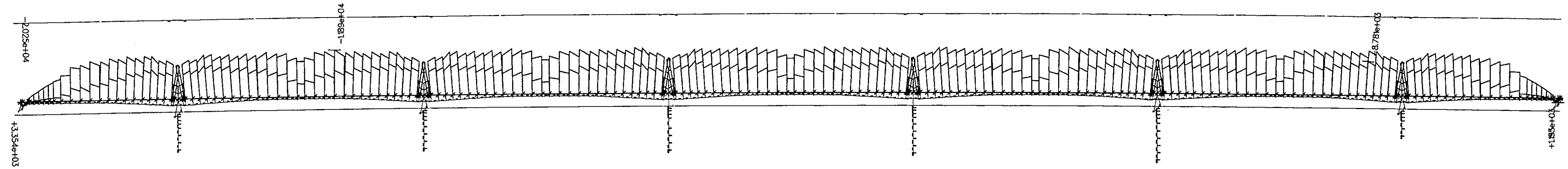


Ky Lam Bridge
Strength limit state 3

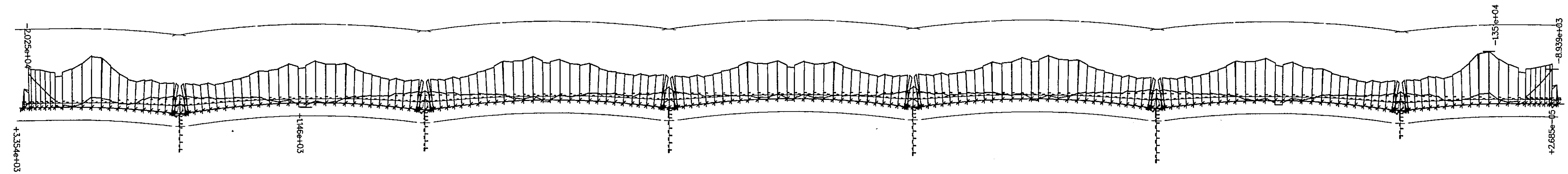
Mz – BENDING MOMENT
(SC: 1/10000 Secondary)



FIBRE STRESS CHECK – FIB TOP

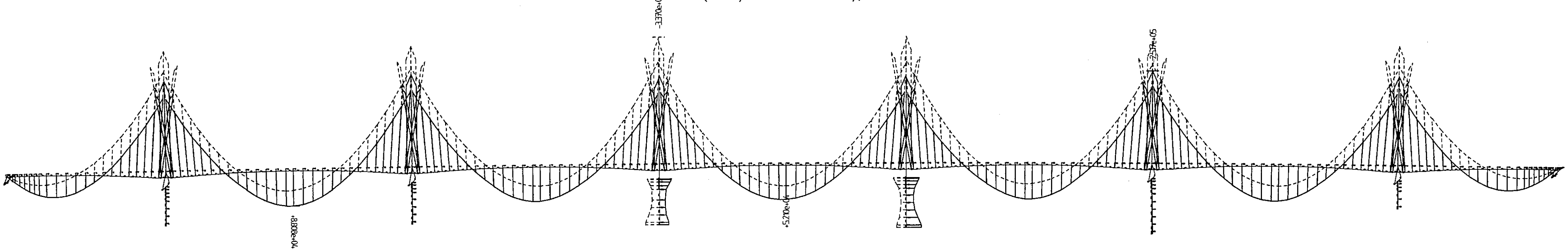


FIBRE SRESS CHECK – FIB BOTTOM

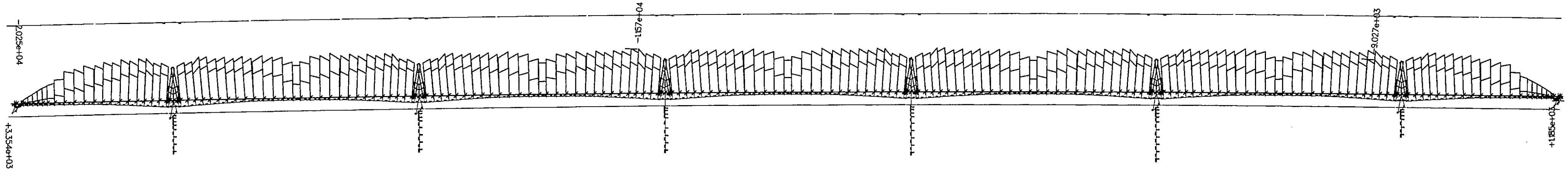


Ky Lam Bridge
Service limit state 1

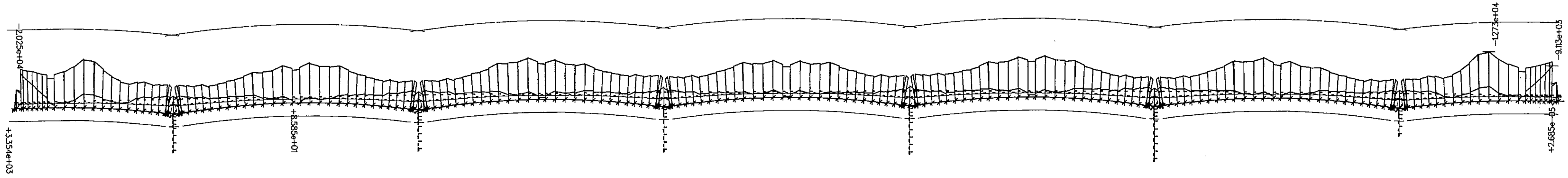
Mz – BENDING MOMENT
(SC: 1/10000 Secondary)



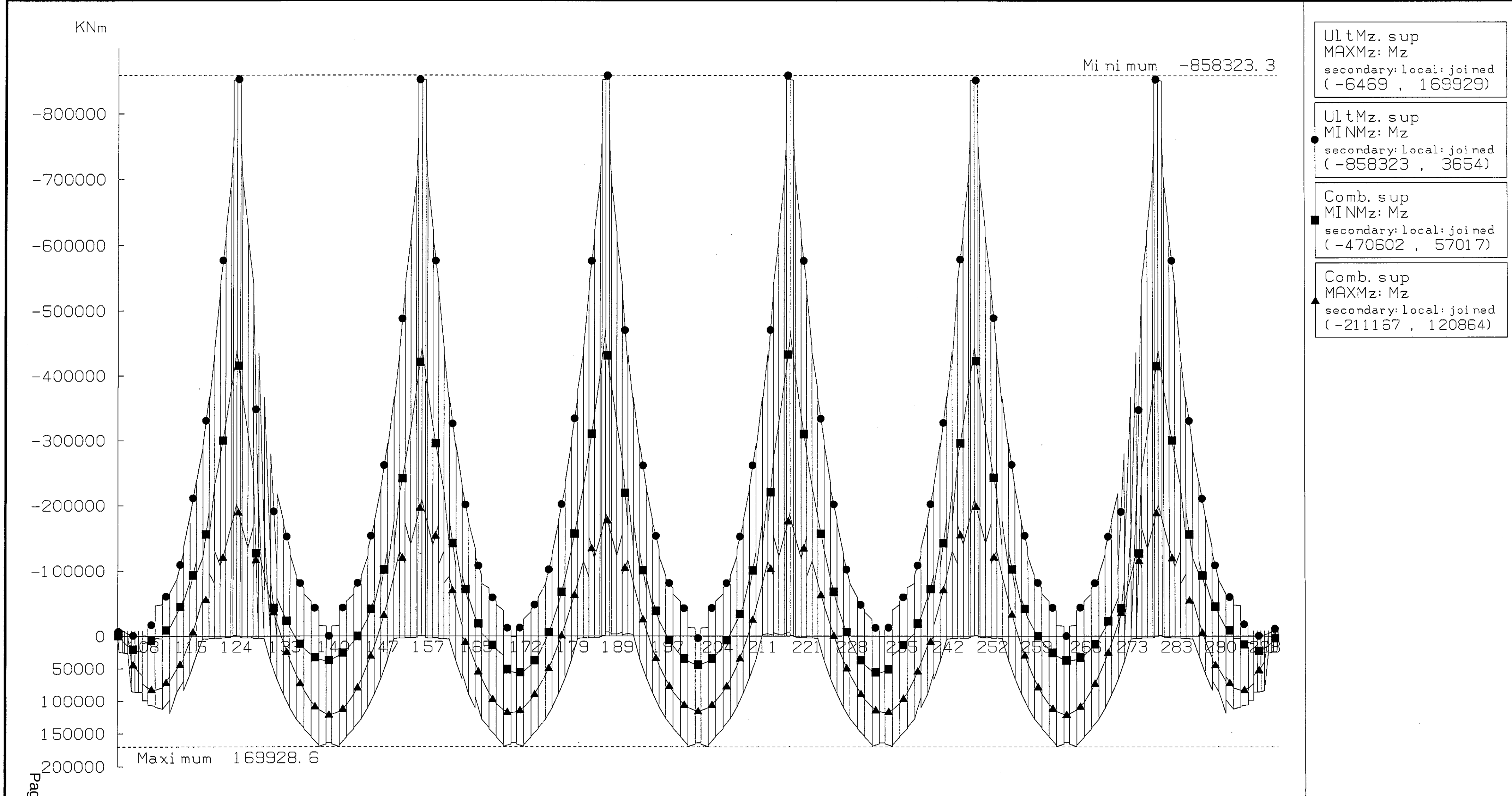
STRESS CHECK – FIB TOP



SRESS CHECK – FIB BOTTOM



Ky Lam Bridge
Service limit state 3



RM Bridge V8i 08.09.30.01

Author: Nippon Koei Co. LtdChiyoda-ku

Project: Ky Lam Bridge
DaNang-QuangNgai Expressway

: ULTMz, Stage: 23
Ultimate load check
(DefaultSchedule)/Dgm-ULTMz

1 cm Plot = 81445.7 KNm

0 81445.7 162891.4 244337.1 325782.8 407228.5

12/06/2012
15:20

3. Shear Capacity Check (5.8.3.3)

3.3. Nominal shear resistance

Nimonal shear resistace Vn shall be determined as the lesser value of :

$V_n = V_c + V_s + V_p$

$V_n = 0{,}25f'_c b_v d_v + V_p$

Where:

$$V_c = 0{,}083\beta\sqrt{f'_c}\, b_v d_v$$

$$V_s = \frac{A_v f_y d_v (\cot g\theta + \cot g\alpha)\sin\alpha}{s}$$

+ b_v : Effective web width taken as the minimum web width within the depth d_v as determined in Article 5.8.2.7 (mm)
+ d_v : Effective shear depth as determined in Article 5.8.2.7 (mm), d_v = max(0,9de ; 0,72h)

$$d_e = \frac{A_{ps}.f_{ps}.d_p + A_s.f_y.d_s}{A_{ps}.f_{ps} + A_s.f_y}$$

+ f_{ps} : Average stress in prestressing force reinforcement

$$f_{ps} = f_{pu}.(1 - k\frac{c}{d_p})$$

+ d_p (d_s): The distance from the extreme compression fiber to the centroid of the tensile reinforcement (tension nonprestressed reinforcement)
+ c : Distance between the neutral axis and the compressive face (m) reinforcement (tension nonprestressed reinforcement)
+ k : Coefficent depend on reinforcement nature

$$k = 2.(1{,}04 - \frac{f_{py}}{f_{pu}}) \quad =0.28$$

+ f_{pu} : Tension strength of prestressing force reinforcement =1860 Mpa
+ f_{py} : Minimum yield strength of tensile reinforcement =1674 Mpa
+ f_y : Specified yield strength of reinforcing bars =400 Mpa
+ f'_y : Specified yield strength of compression reinforcement =400 Mpa
+ A_{ps} : Area of prestressing force reinforcement
+ A_s : Area of nonprestressed tensile reinforcement
+ s : Spacing of stirrups reinforcement (mm)
+ β : Ceoficent indicating ability of diagonal cracked concrete
+ θ : angle of inclination of diagonal compressive stresses (DEG)
+ α : angle of inclination of transverse reinforcement to longitudinal axis(DEG)
+ A_v : Area of shear reinforcement within a distance s (mm2)

$$A_{vmin} = 0{,}083\sqrt{f'_c}\frac{b_v s}{f_y}$$

+ V_p : Component in the direction of the appiled shear of the effective prestressing force positive if resisting the appiled shear (N)

The Plan for disposes stirrups reinforcement the following as:

Notations		K0_1	K0_2	K0_3	K1	K2	K3	K4	K5	K6	K7	K8	K9	K10	K11	K12	Midspan
Diameter	d (mm)	22	22	22	20	18	18	18	18	18	18	18	16	16	16	16	16
Liquid limits	f_y (Mpa)	400	400	400	400	400	400	400	400	400	400	400	400	400	400	400	400
Spacing	s (mm)	125	125	125	125	125	125	125	125	125	125	125	125	125	125	125	125
Shear bar in s	n	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4
Area	**A_v (mm^2)**	**1520.53**	**1520.53**	**1520.53**	**1256.64**	**1017.88**	**1017.88**	**1017.88**	**1017.88**	**1017.88**	**1017.88**	**1017.88**	**804.25**	**804.25**	**804.25**	**804.25**	**804.25**
	$A_{v\,min}$	**922.17**	**208.79**	**208.79**	**208.79**	**208.79**	**139.20**	**139.20**	**139.20**	**139.20**	**139.20**	**139.20**	**139.20**	**139.20**	**139.20**	**139.20**	**139.20**
Conclusion	**$A_v > A_{v\,min}$**	**Ok**	**Ok**	**Ok**	**Ok**	**Ok**	**Ok**	**Ok**	**Ok**	**Ok**	**Ok**	**Ok**	**Ok**	**Ok**	**Ok**	**Ok**	**Ok**

3.3.1. Determination V_p

Have:

$$V_p = \Sigma f_p.A_s.\sin\alpha_i$$

Component of axis longitudinal impact to beam due to prestressing force N_p

$$N_p = \Sigma f_p.A_s.\cos\alpha_i$$

Where:

+ f_p : Stresses in a cable bunch after excepted the losses
+ A_s : Area of primary cable bunch
+ α_i : Angle of inclination of cable with transverse

Notations	Unit	K0_1	K0_2	K0_3	K1	K2	K3	K4	K5	K6	K7	K8	K9	K10	K11	K12	Midspan
V_p	N	1248329	1186669	1158108	948145	891271	972981	816754	548048	2212013	2062617	1991481	2001976	1970710	1839934	1439950	659662
N_p	N	119742324	119756774	119770700	113045797	106150040	99095600	90929810	89671920	82252680	82232240	75548500	74693080	68739120	68622610	55729840	48883580

3.3.2. Determination b and q

β and θ listed in article 5.8.3.4.2-1 depend on ratio v/f'c and the strain in the reinforcement on the flexural tension side

The shear stress on the concrete v :

$$v = \frac{V_u - j V_p}{j b_v d_v} \qquad \text{(N)}$$

Improvisation in tensile reinforcement ε_x:

$$\varepsilon_x = \frac{\frac{M_u}{d_v} + 0,5N_u + 0,5V_u \cot g\theta - A_{ps}f_{po}}{E_s A_s + E_p A_{ps}} \le 0,002 \qquad (5.8.3.4.2\text{-}2)$$

If the value of ε_x is negative, it shall be multiplied by the factor F_ε

$$F_\varepsilon = \frac{E_s A_s + E_{ps} A_{ps}}{E_c A_c + E_s A_s + E_{ps} A_{ps}}$$

Where:

+ f'_c : compression strength of concrete f'_c = 45 MPa
+ E_c : elastic modulus of concrete E_c = 34980 MPa
+ E_p : Elastic modulus of prestressing force reinforcement E_p = 197000 MPa
+ E_s : Modulus of elasticity of reinforcement bars E_s = 200000 MPa

+ A_c : Area of concrete on the flexural tension side of the member as shown in Figure 3 (mm2)

+ φ : Resistance factor for shear specified in Article 5.5.4.2   φ =   0.9

+ f_{po} : The stress in prestressing force reinforcement when the stress around it with 0

$$f_{po} = f_{pe} + f_{pc}.E_p/E_c$$

+ f_{pe} : Effective stress in preforcement force after excepted the losses

+ f_{pc} : Compression stress in the centroid of sectional area $f_{pc} = F/A$

+ V_u : Factored shear force (N)

+ M_u : Factored Moment (N.mm)

Parameter determination of b **and** q

Section	Units	K0_1	K0_2	K0_3	K1	K2	K3	K4	K5	K6	K7	K8	K9	K10	K11	K12	Midspan
V_u	N	26828767	17735661	22985047	21024841	19306661	17838380	16373820	14723670	13786930	12599720	11773390	10547930	9027080	7391060	5492440	3188530
M_u	N.mm	9.887E+09	1.859E+09	2.97E+09	1.692E+09	3.8524E+08	2.0709E+09	2.9777E+09	2.2915E+09	2.4959E+09	2.1727E+09	2.4115E+09	1.9886E+09	2.7477E+09	2.7567E+09	2.6613E+09	2.6929E+09
N_u	N	119742324	119756774	119770700	113045797	106150040	99095600	90929810	89671920	82252680	82232240	75548500	74693080	68739120	68622610	55729840	48883580
h	mm	6000	6000	5873	5452	5073	4720	4394	4048	3731	3450	3206	2997	2804	2658	2559	2500
b_v	mm	5300	1200	1200	1200	1200	800	800	800	800	800	800	800	800	800	800	800
A_{ps}	mm^2	95760.00	95760.00	95760.00	90440.00	85120.00	79800.00	69160.00	69160.00	63840.00	63840.00	58520.00	58520.00	53200.00	53200.00	42560.00	37240
A_g	mm^2	36990000	16950000	14990000	14320000	13647000	11472000	10955000	10383000	9812300	9261800	8724500	8205200	8061700	7953200	7881000	7837000
c	mm	3351.00	3085.00	3115.00	2929.00	2760.00	2610.00	2477.00	2336.00	2212.00	1470.00	1309.00	1160.00	1093.00	1043.00	1010.00	990
d_s	mm	5804.30	5804.30	5677.30	5256.30	4877.30	4524.30	4198.30	3852.30	3535.30	3254.30	3010.30	2801.30	2608.30	2462.30	2363.30	2304.30
d_p	mm	5830.00	5830.00	5703.00	5282.00	4903.00	4550.00	4224.00	3878.00	3561.00	3280.00	3036.00	2827.00	2634.00	2488.00	2389.00	2330.00
A_s	mm^2	35185.8	35185.8	35185.8	31365.7	31767.8	32169.9	32169.9	32169.9	18899.8	18899.8	18899.8	18899.8	18899.8	18899.8	18899.8	18899.82
f_{ps}	MPa	1560.65	1584.41	1575.54	1571.20	1566.83	1561.26	1554.60	1546.28	1536.49	1626.59	1635.45	1646.30	1643.89	1641.67	1639.82	1638.72
d_e	mm	5827.79	5827.82	5700.81	5279.92	4900.76	4547.59	4221.25	3875.24	3559.16	3278.26	3034.12	2825.13	2631.96	2485.95	2386.49	2327.17
d_v	mm	5245.0	5245.0	5130.7	4751.9	4410.7	4092.8	3799.1	3487.7	3203.2	2950.4	2730.7	2542.6	2368.8	2237.4	2147.8	2094.5
A_c	mm^2	19525295	7839193	7649334	7365150	7111198	6115893	5960297	5795027	5639431	5599928	5466649	5347677	5294107	5254098	5227677	5211659
θ	Degree	27.00	27.00	25.36	25.45	25.58	25.65	25.01	25.25	24.50	24.53	24.46	24.21	24.52	26.49	27.00	27.00
f_{po}	Mpa	1132.27	1151.51	1156.15	1155.69	1155.20	1159.72	1155.68	1157.86	1156.89	1159.32	1158.16	1160.83	1157.48	1158.04	1149.94	1145.87
f_{pe}	Mpa	1116.00	1116.00	1116.00	1116.00	1116.00	1116.00	1116.00	1116.00	1116.00	1116.00	1116.00	1116.00	1116.00	1116.00	1116.00	1116.00
f_{pc}	Mpa	2.89	6.30	7.13	7.05	6.96	7.76	7.05	7.43	7.26	7.69	7.49	7.96	7.36	7.47	6.03	5.30
v	N	1.027	2.942	3.960	3.930	3.885	5.756	5.717	5.667	5.115	5.057	5.077	4.778	4.253	3.560	2.714	1.721
v/f'_c	-	0.023	0.065	0.088	0.087	0.086	0.128	0.127	0.126	0.114	0.112	0.113	0.106	0.095	0.079	0.060	0.038
F_ε	-	0.037	0.086	0.088	0.086	0.085	0.094	0.088	0.090	0.077	0.077	0.074	0.076	0.071	0.072	0.062	0.057
ε_x	-	-0.001	-0.001	-0.001	-0.001	-0.001	-0.001	-0.001	-0.001	-0.001	-0.001	-0.001	-0.001	-0.001	-0.001	-0.001	-0.001
ε_xx1000	-	0.029	0.109	0.089	0.091	0.092	0.101	0.071	0.085	0.079	0.086	0.078	0.089	0.081	0.094	0.074	0.071
β	Degree	4.68	3.92	3.38	3.40	3.43	2.57	2.58	2.58	2.70	2.70	2.71	2.74	3.11	3.76	4.27	4.37
θ	Degree	27.00	27.00	25.36	25.45	25.58	25.65	25.01	25.25	24.50	24.53	24.46	24.21	24.52	26.49	27.00	27.00

Shear Capacity Check																
Iterm	Unit	K0_1	K0_2	K0_3	K1	K2	K3	K4	K5	K6	K7	K8	K9	K10	K11	K12	Midspan
α	Degree	90	90	90	90	90	90	90	90	90	90	90	90	90	90	90	90
A_v	mm^2	1520.53	1520.53	1520.53	1256.64	1017.88	1017.88	1017.88	1017.88	1017.88	1017.88	1017.88	804.25	804.25	804.25	804.25	804.25
s	mm	125.00	125.00	125.00	125.00	125.00	125.00	125.00	125.00	125.00	125.00	125.00	125.00	125.00	125.00	125.00	125.00
V_s	kN	50087.07	50087.33	52659.46	40150.39	30010.51	27757.02	26523.56	24090.84	22890.65	21056.95	19557.45	14553.70	13363.70	11556.02	10848.65	10578.99
V_c	kN	72368.82	13750.69	11585.02	10780.14	10121.31	4681.52	4362.62	4003.70	3857.04	3542.85	3300.62	3105.51	3283.55	3745.86	4084.65	4078.20
V_p	kN	1248.33	1186.67	1158.11	948.15	891.27	972.98	816.75	548.05	2212.01	2062.62	1991.48	2001.98	1970.71	1839.93	1439.95	659.66
$V_c + V_s + V_p$	kN	123704.22	65024.69	65402.59	51878.68	41023.09	33411.52	31702.93	28642.59	28959.69	26662.41	24849.55	19661.18	18617.96	17141.81	16373.25	15316.85
$0,25f'_c b_v d_v + V_p$	kN	313982.00	71994.66	70422.91	65099.11	60435.56	37808.49	35008.90	31937.49	31041.22	28616.49	26567.84	24885.53	23289.55	21976.15	20770.50	19509.72
V_n	kN	123704.22	65024.69	65402.59	51878.68	41023.09	33411.52	31702.93	28642.59	28959.69	26662.41	24849.55	19661.18	18617.96	17141.81	16373.25	15316.85
j V_n	kN	**111333.8**	**58522.22**	**58862.33**	**46690.81**	**36920.78**	**30070.37**	**28532.64**	**25778.33**	**26063.72**	**23996.17**	**22364.60**	**17695.07**	**16756.16**	**15427.63**	**14735.92**	**13785.17**
V_u	kN	**26828.77**	**17735.66**	**22985.05**	**21024.84**	**19306.66**	**17838.38**	**16373.82**	**14723.67**	**13786.93**	**12599.72**	**11773.39**	**10547.93**	**9027.08**	**7391.06**	**5492.44**	**3188.53**
$V_u \le j V_n$	Conclusion	**Ok**	**Ok**	**Ok**	**Ok**	**Ok**	**Ok**	**Ok**	**Ok**	**Ok**	**Ok**	**Ok**	**Ok**	**Ok**	**Ok**	**Ok**	**Ok**

4. Combined Shear and torsion Capacity Check (5.8.3.6)

For combined shear and torsion, ε_x shall be determined using Equation 5.8.3.4.2-2, with V_n replaced by:

$$V_u = \sqrt{V_u^2 + \left(\frac{0.9 p_h T_u}{2A_o}\right)^2} \qquad (5.8.3.6.2\text{-}2)$$

Where:

+ A_o: Area enclosed by the shear flow path, including any area of holes therein (mm2)

+ The angle θ shall be as specified in Table 5.8.3.4.2-1 or Table 5.8.3.4.2-2, as appropriate, with the shear stress, v, taken as:

For box sections: $$v = \frac{V_u - j V_p}{j b_v d_v} + \frac{T_u p_h}{j A_{0h}^2}$$

+ p_h: Perimeter of the centerline of the closed transverse torsion reinforcement (mm)

+ A_{oh}: Area enclosed by centerline of exterior closed transverse torsion reinforcement, including area of any holes (mm2)

+ T_u: Factored torsional moment (N.mm)

Parameter determination of b and q

Section	Units	K0_1	K0_2	K0_3	K1	K2	K3	K4	K5	K6	K7	K8	K9	K10	K11	K12	Midspan
V_u	N	26830798	17738383	22987348	21027369	19309392	17841419	16377126	14727351	13791701	12604968	11795724	10553191	9033295	7398890	5503125	3207372
T_u	N.mm	-659572000	-620831000	-642213000	-617382000	-589572000	-572417000	-547028000	-519305000	-542125000	-515559000	-977636000	-428213000	-410599000	-401276000	-393153000	-391444000
M_u	N.mm	9.887E+09	1.859E+09	2.97E+09	1.692E+09	3.8524E+08	2.0709E+09	2.9777E+09	2.2915E+09	2.4959E+09	2.1727E+09	2.4115E+09	1.9886E+09	2.7477E+09	2.7567E+09	2.6613E+09	2.6929E+09
N_u	N	119742324	119756774	119770700	113045797	106150040	99095600	90929810	89671920	82252680	82232240	75548500	74693080	68739120	68622610	55729840	48883580
h	mm	6000	6000	5873	5452	5073	4720	4394	4048	3731	3450	3206	2997	2804	2658	2559	2500
b_v	mm	5300	1200	1200	1200	1200	800	800	800	800	800	800	800	800	800	800	800
A_o	mm^2	31398542	31398542	30833889	28937634	27198419	25551130	24006377	22342210	20795250	19406160	18186398	17131567	16149266	15400932	14890929	14586000
A_{oh}	mm^2	36939461	36939461	36275163	34044275	31998140	30060153	28242796	26284953	24465000	22830776	21395762	20154785	18999136	18118743	17518740	17160000
p_h	mm	34927	34927	34702	33959	33289	32666	32090	31479	30919	30423	29992	29623	29282	29024	28849	28745
A_{ps}	mm^2	95760.00	95760.00	95760.00	90440.00	85120.00	79800.00	69160.00	69160.00	63840.00	63840.00	58520.00	58520.00	53200.00	53200.00	42560.00	37240
A_g	mm^2	36990000	16950000	14990000	14320000	13647000	11472000	10955000	10383000	9812300	9261800	8724500	8205200	8061700	7953200	7881000	7837000
c	mm	3351.00	3085.00	3115.00	2929.00	2760.00	2610.00	2477.00	2336.00	2212.00	1470.00	1309.00	1160.00	1093.00	1043.00	1010.00	990
d_s	mm	5804.30	5804.30	5677.30	5256.30	4877.30	4524.30	4198.30	3852.30	3535.30	3254.30	3010.30	2801.30	2608.30	2462.30	2363.30	2304.30
d_p	mm	5830.00	5830.00	5703.00	5282.00	4903.00	4550.00	4224.00	3878.00	3561.00	3280.00	3036.00	2827.00	2634.00	2488.00	2389.00	2330.00
A_s	mm^2	35185.8	35185.8	35185.8	31365.7	31767.8	32169.9	32169.9	32169.9	18899.8	18899.8	18899.8	18899.8	18899.8	18899.8	18899.8	18899.82
f_{ps}	MPa	1560.65	1584.41	1575.54	1571.20	1566.83	1561.26	1554.60	1546.28	1536.49	1626.59	1635.45	1646.30	1643.89	1641.67	1639.82	1638.72
d_e	mm	5827.79	5827.82	5700.81	5279.92	4900.76	4547.59	4221.25	3875.24	3559.16	3278.26	3034.12	2825.13	2631.96	2485.95	2386.49	2327.17
d_v	mm	5245.0	5245.0	5130.7	4751.9	4410.7	4092.8	3799.1	3487.7	3203.2	2950.4	2730.7	2542.6	2368.8	2237.4	2147.8	2094.5
A_c	mm^2	19525295	7839193	7649334	7365150	7111198	6115893	5960297	5795027	5639431	5599928	5466649	5347677	5294107	5254098	5227677	5211659
θ	Degree	27.00	27.00	25.42	25.51	25.64	25.63	24.98	25.22	24.47	24.49	24.39	27.00	24.62	26.58	27.00	27.00
f_{po}	Mpa	1132.27	1151.51	1156.15	1155.69	1155.20	1159.72	1155.68	1157.86	1156.89	1159.32	1158.16	1160.83	1157.48	1158.04	1149.94	1145.87
f_{pe}	Mpa	1116.00	1116.00	1116.00	1116.00	1116.00	1116.00	1116.00	1116.00	1116.00	1116.00	1116.00	1116.00	1116.00	1116.00	1116.00	1116.00
f_{pc}	Mpa	2.89	6.30	7.13	7.05	6.96	7.76	7.05	7.43	7.26	7.69	7.49	7.96	7.36	7.47	6.03	5.30
v	N	1.009	2.925	3.942	3.911	3.864	5.734	5.694	5.642	5.086	5.026	5.017	4.746	4.220	3.526	2.679	1.69
v/f'_c	-	0.022	0.065	0.088	0.087	0.086	0.127	0.127	0.125	0.113	0.112	0.111	0.105	0.094	0.078	0.060	0.038
F_ε	-	0.037	0.086	0.088	0.086	0.085	0.094	0.088	0.090	0.077	0.077	0.074	0.076	0.071	0.072	0.062	0.057
ε_x	-	-0.001	-0.001	-0.001	-0.001	-0.001	-0.001	-0.001	-0.001	-0.001	-0.001	-0.001	-0.001	-0.001	-0.001	-0.001	-0.001
ε_xx1000	-	0.029	0.109	0.089	0.091	0.092	0.101	0.070	0.085	0.079	0.086	0.078	0.096	0.081	0.094	0.074	0.071
β	Degree	4.68	3.93	3.40	3.41	3.46	2.57	2.58	2.58	2.71	2.70	2.73	2.72	3.15	3.79	4.28	4.37
θ	Degree	27.00	27.00	25.42	25.51	25.64	25.63	24.98	25.22	24.47	24.49	24.39	24.22	24.62	26.58	27.00	27.00

Combined Shear and torsion Capacity Check																	
Iterm	Unit	K0_1	K0_2	K0_3	K1	K2	K3	K4	K5	K6	K7	K8	K9	K10	K11	K12	Midspan
α	Degree	90	90	90	90	90	90	90	90	90	90	90	90	90	90	90	90
A_v	mm^2	1520.53	1520.53	1520.53	1256.64	1017.88	1017.88	1017.88	1017.88	1017.88	1017.88	1017.88	804.25	804.25	804.25	804.25	804.25
s	mm	125.00	125.00	125.00	125.00	125.00	125.00	125.00	125.00	125.00	125.00	125.00	125.00	125.00	125.00	125.00	125.00
V_s	kN	50087.07	50087.33	52536.64	40051.22	29932.65	27785.88	26556.64	24121.03	22926.92	21095.70	19621.29	14546.60	13305.39	11507.67	10848.65	10578.99
V_c	kN	72369.13	13766.79	11648.06	10842.03	10181.90	4683.92	4364.81	4005.92	3867.47	3551.76	3319.33	3080.81	3321.26	3781.00	4090.92	4078.84
V_p	kN	1248.33	1186.67	1158.11	948.15	891.27	972.98	816.75	548.05	2212.01	2062.62	1991.48	2001.98	1970.71	1839.93	1439.95	659.66
$V_c + V_s + V_p$	kN	123704.53	65040.79	65342.81	51841.40	41005.81	33442.78	31738.20	28675.01	29006.40	26710.08	24932.10	19629.38	18597.36	17128.60	16379.52	15317.49
$0{,}25f'_c b_v d_v + V_p$	kN	313982.00	71994.66	70422.91	65099.11	60435.56	37808.49	35008.90	31937.49	31041.22	28616.49	26567.84	24885.53	23289.55	21976.15	20770.50	19509.72
V_n	kN	123704.53	65040.79	65342.81	51841.40	41005.81	33442.78	31738.20	28675.01	29006.40	26710.08	24932.10	19629.38	18597.36	17128.60	16379.52	15317.49
j V_n	kN	**111334.1**	**58536.71**	**58808.53**	**46657.26**	**36905.23**	**30098.50**	**28564.38**	**25807.51**	**26105.76**	**24039.07**	**22438.89**	**17666.45**	**16737.62**	**15415.74**	**14741.57**	**13785.74**
V_u	kN	**26830.80**	**17738.38**	**22987.35**	**21027.37**	**19309.39**	**17841.42**	**16377.13**	**14727.35**	**13791.70**	**12604.97**	**11795.72**	**10553.19**	**9033.29**	**7398.89**	**5503.12**	**3207.37**
$V_u \le$ j V_n	Conclusion	**Ok**	**Ok**	**Ok**	**Ok**	**Ok**	**Ok**	**Ok**	**Ok**	**Ok**	**Ok**	**Ok**	**Ok**	**Ok**	**Ok**	**Ok**	**Ok**

1.1.4 Transversal Box Girder section Design
1.1.4 Tính toán dầm hộp theo phương ngang

4. Transversal Design for Box Section of Girder

4.1. Design Principle

4.2. Basics of Design

4.3. Design Condition

4.3.1. Matreial and Stress limits

4.3.2. Load Combination and Each Load

4.4. Rebar Arrangement Due to Calculation of the Box Section

4.4. Result of Calculation

4.5.1. Section 1 (Hg=2.5m,Hfu=0.25m,Hfl=0.25m,Bw=0.40m)

4.5.2. Section 2 (Hg=5.788m,Hfu=0.25m,Hfl=0.525m,Bw=0.40m)

4.5.3. Section 3 (Hg=6.0m,Hfu=0.25m,Hfl=1.00m,Bw=1.00m)

4.5.4. Check the Overhang at Vehicle Collision

Appendix 1-1 The Pucher Influence Surface and Bending Moment

Appendix 1-2 Bending Moment due to Design Truck and Design Tandem Load

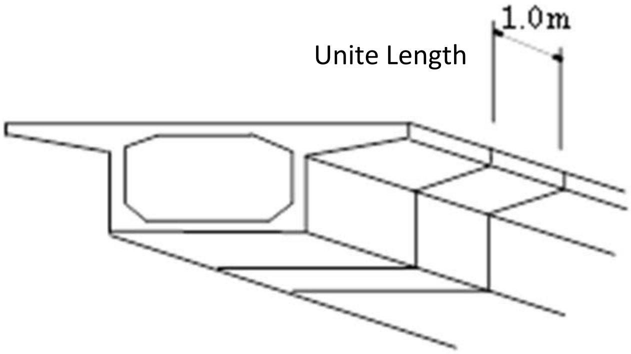
Appendix 2 Creep and Shrinkage follow the MC90

4. Transversal Box Girder Section Design

4.1 Design Principal

Article 4.6.2.1 4.6.2.9.4

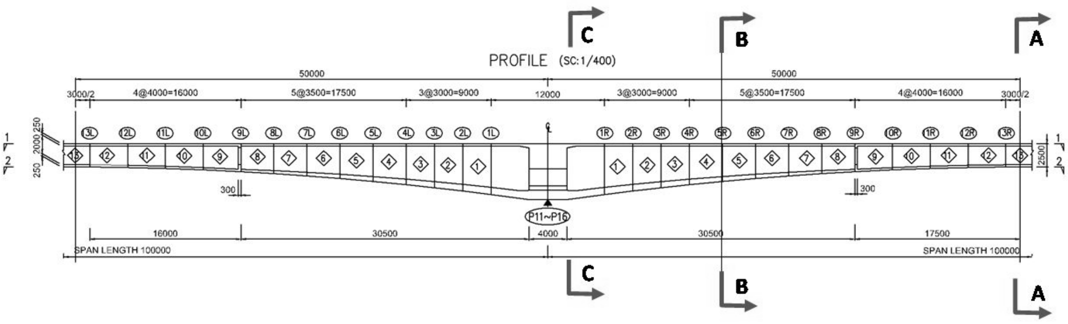
I divide a principal beam section by unit length to show below Figure and I rearrange it for the rigid frame structure that established the fulcrum right under Web and do the top, a lower slab and the Web and perform the design as the version out of the aspect. In addition, the upper slab assumes it Prestressted Concrete structure, and Web and the lower slab do it with Reinforcement Concrete structure.



Analysis Model

-Considered Design Section

1. At Spans Center Section (Section A)
2. At Interior Support Section(section B)
3. At Interior Span Section(Section C)
4. Check the Overhang at Vehicle Corrosion



Property of Box Section

	Girder Height	Thickness of Upper Flange	Thickness of Lower Flange	Thickness of Web
	hg(m)	Hfu(m)	Hfl(m)	Bw(m)
Section A-A	2.500	0.250	0.250	0.400
Section B-B	5.788	0.250	0.525	0.400
Section C-C	6.000	0.250	1.000	0.600

4.2 Basics of Design

The members of box section made by deck slab,bottom slab and web shall be satisfied Eq.1 for each limit state. For Service limit states, resistance factors ϕ shall be taken as 1.0

- Load modifier factor ηi

$\Sigma\eta_i \cdot \gamma_i \cdot Q_i \leqq \phi R_n = R_n$ **Eq.1**

η_D = 1.00 (for ductility)

η_R = 1.00 (for redundancy)

η_I = 1.00 (for operational importance)

- Resistance Factor φ for the Strength Limit Strength

Table Resistance Factor φ for Reinforcing Concrete

Condition	Resistance Factor φ
Flexure and Tension	0.9
Shear and Torsion	0.9
Bearing and S&T Model	0.7
For Anchorage Zone(Compression)	0.8

- Verification for service limit state

* For Prstressing Concrete For Deck Slab

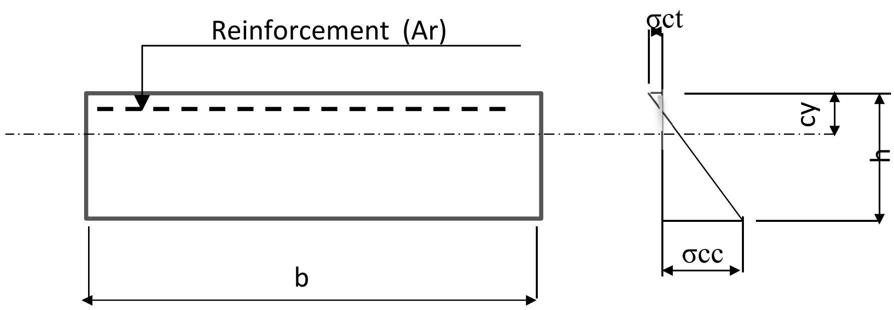
In service limit stage , tensile and compressive stress calcurated by the gross section is not exeeded the tensile and compressive limite stress.

- Crack control

* For Prestressing Concrete

Tesile stress calculated by gross section is not exceede the tensile limit stress.

The areas where tensile stress is occurred are reinforced by binded reinforcing bars.



$$c_y = \frac{\sigma ct}{\sigma cc + \sigma ct} h$$

$$T = \frac{1}{2} \sigma ct \cdot b \cdot c_y$$

$$reqA_r = \frac{T}{fsa}$$

$$where\_\ fsa = 0.5 f_{ry} \leq 206 Mps$$

* For Reinforcing Concrete(RC) Bottom splab and Web

The tensile stress fr in the mild steel reinforcement at service limit state fra doas not exceed:

$$f_r = f_{ra} = \min\left(\frac{Z}{(d_c A)^{(1/3)}}, 0.6 \cdot f_{ry}\right)$$

fr: tensile stress in reinforcement

fra limit stress of reinforcement

Z: crack width papameter(N Z= 30,000

dc: depth of concrete measured from extreme tension fiber to center of bar(mm),dc≦50mm

A: Area of concrete having same centroid as principle tensile reinforcement and bounded by the surface of the cross-section and straight line parallel to the neutral axis; divided by number of bars, for calculation purpose, the thickness of cover used to compute A shall not be taken to be greater than 50mm.

4.3 Design Condition

4.3.1 Matreial and Stress limts

(1) Property of Matreial

a. Concrete

Table Property of Concrte

Item	Symbol	Unit	Desidn Value	Remark	
Compressive Strength at 28 days	f'c	Mpa	45		
Modulas of Elastity	Ec	Mpa	32,200	4800√ f'c	
Poisson's Ratio	ν	-	0.2		
Tensile Bending Strength	fctr	Mpa	4.23	0.63*√ f'c	5.4.2.6
UltimateLimit Strein	εcfu	-	0.003		5.7.2
Liner Thermal Coefficient	εt	-	0.0000108		5.4.2.2

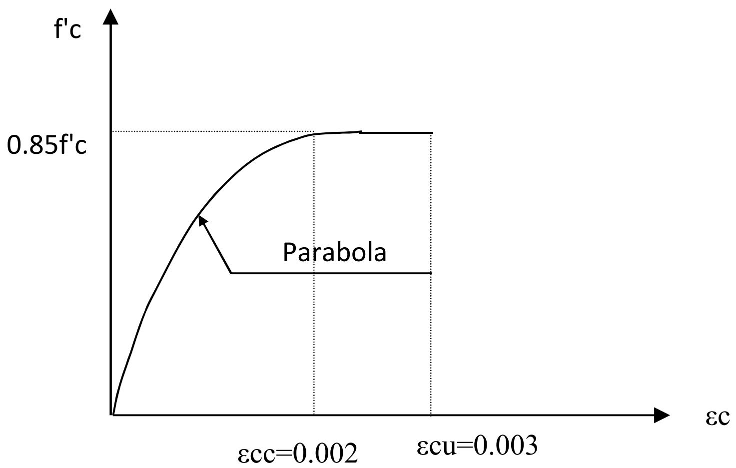
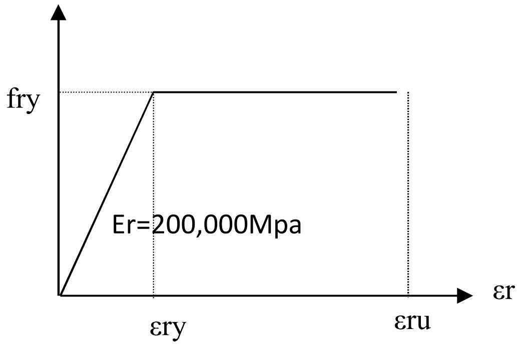


Figure Assumed Stress- Train Curve of Concrete

b. Reinforcing Steel

Table Property of Reinforcing Bar

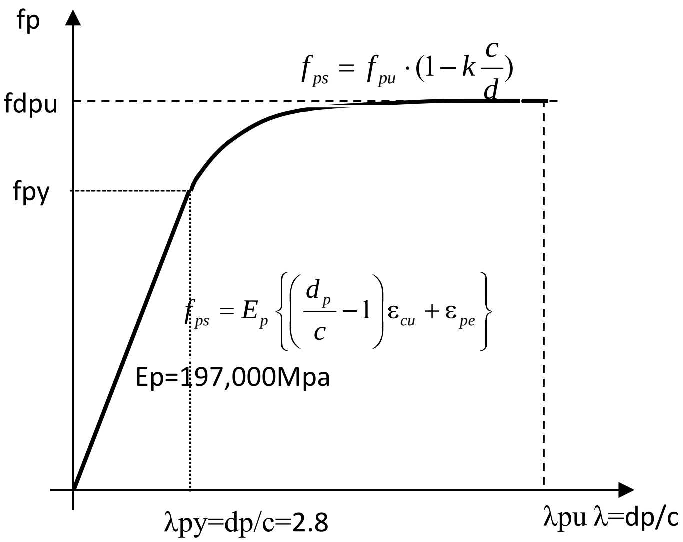
Item	Symbol	Unit	Desidn Value	Remark	Artticl
Yield Strength	fry	Mpa	400	CB400-V	
Tensile Strength	fru	Mpa	570		
Modulus of Elasticit	Er	Mpa	200,000		
Limit of elongation	εru	%	15.0	-	



c. Prestresing Steel

Table Propety of Prstressing Steel

Item		Symbol	Unit	Design Value	Remark	Artticl
Property of the Material	Type	-	-	Low Relaxation		
	Standaerd	-	-	ASSOT M203		5.4.4
	Symbol	-	-	grade270ksi		5.4.4.1
	Diamiter	D	mm	15.24		
	Nominal Strength	fpu	Mpa	1860		5.4.4.1
	Yield Strength	fpy	Mpa	1674	0.9*fpu	5.4.4.1
	Elastic Modulus	Es	Mpa	197000		5.4.4.2
	Ulimate Strain	εpu	-	Over 3.5%		
	Nominal Relaxation Cofficient	r	%	Bellow 2.5%		
	Sectional Aerea	Ap1	mm2	140.0		
	For Value k	fpy/fpu	-	0.9	k=2(1.04-fpy/fpu)	5.7.3.1
		k	-	0.280		
Property of the Prestressing Tendon	Number of Strand	Nst	nos	3		
	Anchorage Set	dl	mm	6.0		
	Firction Due to Duct	K	1/m	0.0066		5.9.5.2.2
		μ	1/rad	0.25		5.9.5.2.2



$$f_{ps} = f_{pu} \cdot (1 - k\frac{c}{d})$$

$$k = 2 \cdot (1.04 - \frac{f_{py}}{f_{pu}})$$

$$f_{pu} = 0.9 f_{pu}$$

$$k = 2 \cdot (1.04 - \frac{0.9 f_{pu}}{f_{pu}}) = 0.28$$

where

$$f_{ps} = f_{py}$$

$$f_{py} = (f_{py} / 0.9)(1 - 0.28\frac{c}{d})$$

$$\lambda = d / c = k / 0.1 = 0.28 / 0.10 = 2.80$$

$$\therefore \lambda = \frac{d}{c} = 0.28$$

$$\lambda u = \frac{d_p}{c} = \frac{\varepsilon_{pu} - \varepsilon_{pe}}{\varepsilon_{cu}} + 1 \qquad f_{ps} = E_p(\frac{d_p - c}{c}\varepsilon_{cu} + \varepsilon_{pe}) = E_p \left\{ \left(\frac{d_p}{c} - 1 \right) \varepsilon_{cu} + \varepsilon_{pe} \right\}$$

Figure Assumed Stress-Strain Curve for Prestressing Steel

(2) Limit Stress At Sevice limit State
For Prstressing Concrete

Table Limit stress of concrete

Item			Symbol	Unit	Limit Stress	Remark	Load Combinat	Artticl
Specified Compressive Stress Strength of Concrete at 28days			f'c	Mpa	45	-	-	-
Specified compressive strength of at time prestressing			f'c	Mpa	36.0	0.8f'c	-	-
Compressive Stress	After Loass	The Sum of Effective Prestress and Permanent	f'ca1	Mpa	20.3	0.45f'c	D+Ps	5.9.4.2.1
		Live Load and One-half the sum of effective prestress and permanent loads	f'ca2	Mpa	18.0	0.40f'c	D+L/2+Ps	5.9.4.2.1
	Before Loss		fca3	Mpa	21.6	0.6f'ci	-	5.9.4.1
Tensale Stress	After Prestress Losses	Tranversal Stress Through Joints; -Tesion in the tranverse direction in precompressed zone	fcta1	Mpa	1.68	$0.25\sqrt{f'c}$	Service III	5.6.4.2.2
		-In areas with bonded reinforcment suffcient to resist the tensile force in the concrete comuted assuming an uncracked section ,where reinforment is propertioned using a stresss of 0.5fy(200Mpa),not to exeed 205Mpa	fcta2	Mpa	3.35	$0.50\sqrt{f'c}$	Service III	5.6.4.2.2
	Before Prestress Losses	Tranversal Stress Through Joints; -Tesion in the tranverse direction in precompressed zone	fcta1	Mpa	1.06	$0.25\sqrt{f'ci}$	-	5.6.4.2.2
		-In areas with bonded reinforcment suffcient to resist the tensile force in the concrete comuted assuming an uncracked section ,where reinforment is propertioned using a stresss of 0.5fy(200Mpa),not to exeed 210Mpa	fcta2	Mpa	2.12	$0.50\sqrt{f'ci}$	-	5.6.4.2.2

b. Reinforcing Steel

At Service Limit State for Crack Control
maxmum tensile stress fsa= 240 Mpa

c. Prestressing Steel

Table Limit stress of Prestressing Steel

Item	Symbol	Unit	Design Value	Remark	Article
Design ultimate strength	fpu	Mpa	1,860	-	
Design yield strength	fpy	Mpa	1,674	0.9*fpu	5.9.3
Prior to seating -fpbt me be allowed	fpaj	Mpa	1,507	0.90fpy	5.9.3
At anchorages and couplers immediately after anchor set	fpta1	Mpa	1,302	0.70fpu	5.9.3 5.9.3
Elsewhere along length of member away from anchorages and couplers immediately after anchor set	fpta2	Mpa	1,376	0.74fpu	5.9.3
At service limit state after losses (fpe)	fpea	Mpa	1,339	0.80fpy	5.9.3

4.3.2. Load Combination and Each Load

(1) Load Combination for Box Section

		1	2	3	4	5	6	7	8	9	10	11
		DC	**DW**	**EL**	**CR**	**SH**	**LL +IM**	**TU**	**TG**	**WS**	**WL**	**CT**
		Self weight of Girder	Surfaces Load	Secondary Force of Prestressing	Creep Effect	Shrinkage Effect	Vehicle Load Impact	Temperature Effect	Gradient of Temperature	Wind Load to Structure	Wind Load to Vehicle	Vehicle Collision
Service Limit State	Service- Ⅰ	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.50	0.30	1.00	-
		1.00	1.00	1.00	1.00	1.00	-	1.00	1.00	0.30	1.00	-
	Service-Ⅲ	1.00	1.00	1.00	1.00	1.00	0.80	1.00	0.50	1.00	0.30	-
		1.00	1.00	1.00	1.00	1.00	-	1.00	1.00	1.00	0.30	-
Strength Limit State	Strength- Ⅰ	1.25	1.50	1.00	0.50	0.50	1.75	0.50	0.50	-	-	-
		0.90	0.65	1.00	0.50	0.50	1.75	0.50	0.50	-	-	-
	Strength- Ⅱ	1.25	1.50	1.00	0.50	0.50	1.35	0.50	0.50	-	-	-
		0.90	0.65	1.00	0.50	0.50	1.35	0.50	0.50	-	-	-
	Strength-Ⅲ	1.25	1.50	1.00	0.50	0.50	-	0.50	1.00	1.40	-	-
		0.90	0.65	1.00	0.50	0.50	-	0.50	1.00	1.40	-	-
	Strength-Ⅳ	1.50	1.50	1.00	0.50	0.50	-	0.50	1.00	-	-	-
	Strength- Ⅴ	1.25	1.50	1.00	0.50	0.50	1.35	0.50	0.50	0.40	1.00	-
		0.90	0.65	1.00	0.50	0.50	1.35	0.50	0.50	0.40	1.00	-
Extreme Event Limit State	Extreme II	1.25	1.50	1.00	-	-	0.50	-	-	-	-	1.00
		0.90	0.65	1.00	-	-	0.50	-	-	-	-	1.00

Verification of Service limit state

Service- Ⅰ Check of concrete compression stress of full prostressing component and cluck control.

Service-Ⅲ Check of concrete tension stress of full prostressing component .

Verification of Strength and Extreme event state

For Strength and Extreme Event stage, each component all satisfy blow equation.

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

For loads for which a maximum value of γi is appropriate.

$$\eta_i = \eta_D \cdot \eta_R \cdot \eta_i \geq 0.95$$

For loads for which a minimum value of γi is appropriate.

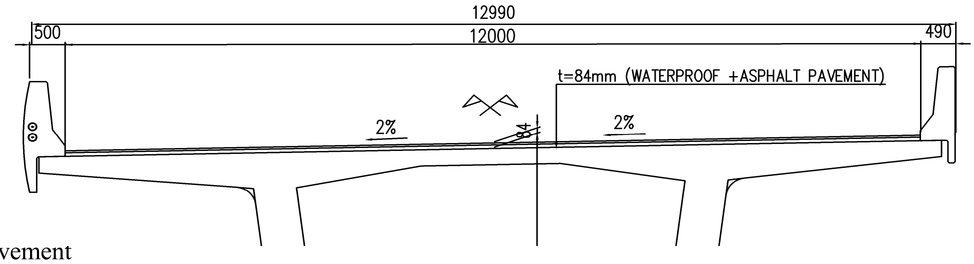
$$\eta_i = \frac{1}{\eta_D \cdot \eta_R \cdot \eta_i} \leq 1.0$$

(2) Load

1) Dead Load Dc and Dw

Unite Weight

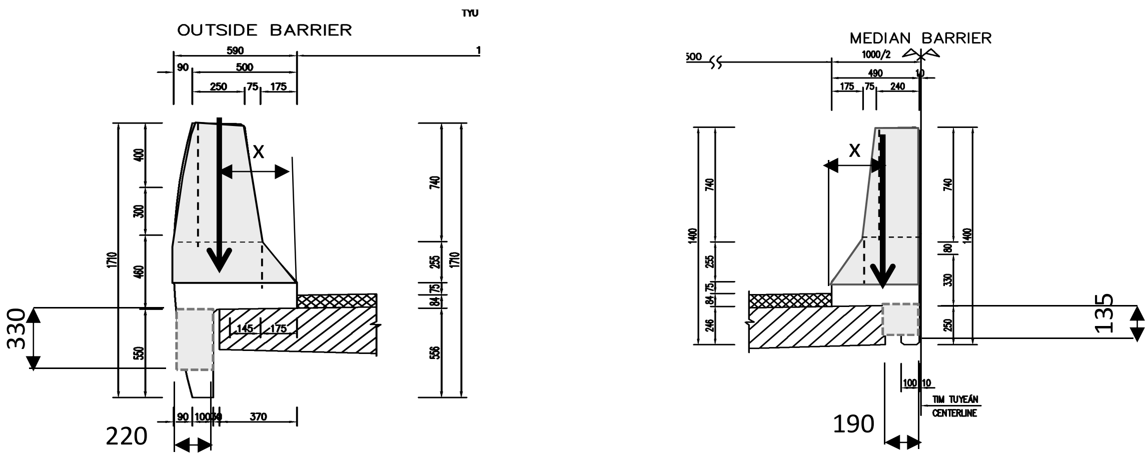
	Symble	Unit	Design Value	Rmark
Reeinfocing Bar and Prestressing Concrete	γpv	kN/m3	24.5	
Pavement(Asphalt)	γc	kN/m3	22.5	



Pavement

wpv= 0.084 * 22.5 = 1.890 kN/m

Concrete Barria



Outside Barria

	k	b	h	A	γ	N	w	x	w・x
	-	m	m	m2	kN/m3	n	kN/m	m	kN/m ・m
1	1	0.250	0.740	0.185	24.5	1	4.533	0.375	1.700
2	2	0.090	0.740	0.033	24.5	1	0.816	0.560	0.457
3	2	0.075	0.740	0.028	24.5	1	0.680	0.225	0.153
4	1	0.415	0.225	0.093	24.5	1	2.288	0.383	0.875
5	2	0.075	0.255	0.010	24.5	1	0.234	0.117	0.027
Sum						5	8.550		3.2120

Weight w 8.5502 kN/m

Position x＝ w・x / w 3.2120 / 8.550 = 0.3757 m

Insade Barria

	k	b	h	A	γ	N	w	x	w・x
	-	m	m	m2	kN/m3	n	kN/m	m	kN/m ・m
1	1	0.240	0.740	0.178	24.5	1	4.351	0.370	1.610
2	2	0.075	0.740	0.028	24.5	1	0.680	0.225	0.153
3	2	0.075	0.740	0.028	24.5	1	0.680	0.117	0.080
4	1	0.315	0.225	0.071	24.5	1	1.736	0.333	0.578
0	0	0.000	0.000	0.000	24.5	0	0.000		0.000
Sum						4	7.447		2.4207

Weight w 7.447 kN/m

Position x＝ w・x / w 2.4207 / 7.447 = 0.3250 m

2) EL Secondary Moment of Prestressing

This Interal Force is calurated automatically in the softwear .

Used Prestressing tendon is 3S15.2(B) (Low relaxsaision strands) for tarnsversal tendns.

Jacking Force Pij Grade 270

Tensil Strenth	fpu=	1860	Mpa		------>	Pfu=	773.9	kN
Yeald Strenth	fpy=	1674	Mpa	(0.9*fpu)	----->	Pfu=	696.6	kN
Closs Section Area	Ap=	138.7	*	3 =	416.1 mm2			

Friction of Anckore and Juack is considered η= 5% .

So Jackin force as blow,

fpj=(0.9 -η)*fpy= (0.9 - 0.050) * 1674 = 1422.9 -------> 1400.0 Mpa

Force Pfj= 1400.0 * 416.1 = 582,540 N --------> **582.5** kN

3) Creep and Shrinkage Cofficient

Creep and Shrinkage Cofficient is calucureted according to MC90.

Age of concrete is assumed berow.

At Prestressing t1= 5 days

φ(5,30000)= 2.2 εsh(5,30000)= 250 x10-6

At loading the saface load t2= 90 days

φ(90,30000)= 1.0 εsh(90,30000)= 190 x10-6

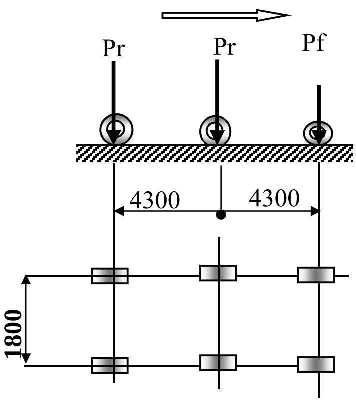
4) Vehiculer Live Load

Vehiculer Live Load effect is calucurated according to elastic analysia in the AASHTO LRFD 2007 4the Edition,4.6.2.9.4.

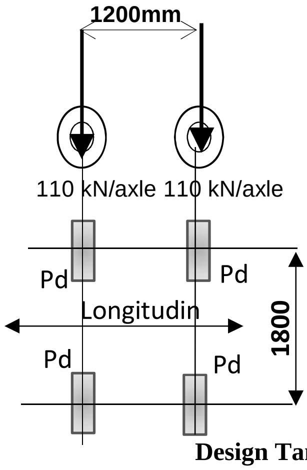
The Bending moment is determed byt the infuluence surface such as by Pucher(1964).
(See Appendix 1-and1-2)

The Design moment for vehicular live load is used large moment of Design Truck and Design Tandem.

4.1) Design Vehicular Load



Design Truck



Design Tandem

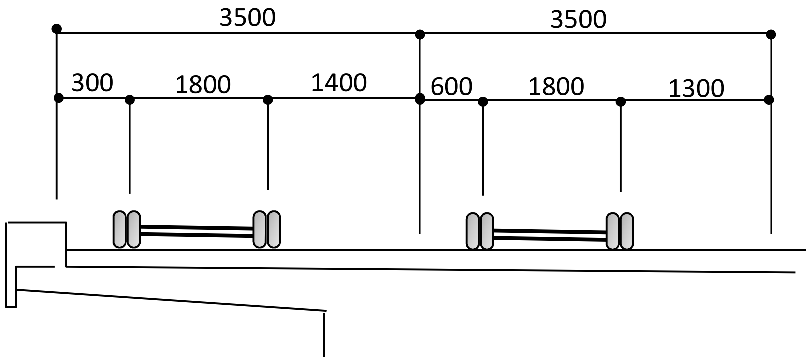
One Unite Load

			Pr	Pf	Pd	Sum.
Truck	Wheel Number	nos	2	4	0	6
	Load	kN	17.5	72.5	0	
	Axle Load	kN	35	290	0	325
Tandem	Wheel Number	nos	0	0	4	4
	Load	kN	0	0	55	
	Axle Load	kN	0	0	220	220

Modification Factor fm

Lane Number	fm
1	1.2
2	1
3	0.85
≥4	0.65

Position of Wheel Load at Overhang



4.2) Design Impact IM IM= 0.25

4.3) Tire Contact Area for Design of Deck Slab 3.6.1.2.5

Tire contact area of a wheel consisting of one or two tires shall be assumed to be a single rectangle ,whose width is 510mm and whose length in mm shall be taken as below.

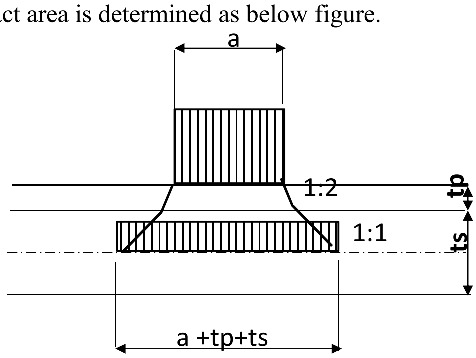
$$l = 2.28x10^{-3}\gamma(1+IM)P$$

The Contact area is used

I use the value that I found by load at the time of the use for a design calculation.

For Design ,the contact aria that is decided at service limit state is used considered conservatively.

Deck contact area is determined as below figure.



Contact Aria for Truck Load

				At Service	At Strength
Tire Contact Area	B		mm	510	510
	Pr	P	kN	72.5	72.5
		IM	-	0.25	0.25
		γ	-	1.00	1.75
		k	-	2.28	2.28
		L	mm	207	362
		Pd	kN	90.6	158.6
	Pf	P	kN	35.0	35.0
		IM	-	0.25	0.25
		γ	-	1.00	1.75
		k	-	2.28	2.28
		L	mm	100	175
		Pd	kN	43.8	76.6
Distributed Uniformed Load	Pr	tp	mm	80	80
		ts	mm	250	250
		Be	mm	840	840
		Le	mm	537	692
		qPr	kN/m2	201.0	273.0
	Pf	tp	mm	80	80
		ts	mm	250	250
		Be	mm	840	692
		Le	mm	430	505
		qPf	kN/m2	121.2	219.4

Contact Aria for Tandem Load

				At Service	At Strength
Tire Contact Area	B		mm	510	510
	Pi	P	kN	55	55
		IM	-	0.25	0.25
		γ	-	1.00	1.75
		k	-	2.28	2.28
		L	mm	157	274
		Pd	kN	68.8	120.3
Distributed Uniformed Load	Pi	tp	mm	80	80
		ts	mm	250	250
		Be	mm	840	840
		Le	mm	487	604
		qPr	kN/m2	168.1	237.0

4.4) Design Moment due to Truck or Tandem

The blow table is the design moment due to truck or tandem load using the Puncher Influence Surface.

Table Design Moment for Deck Slab

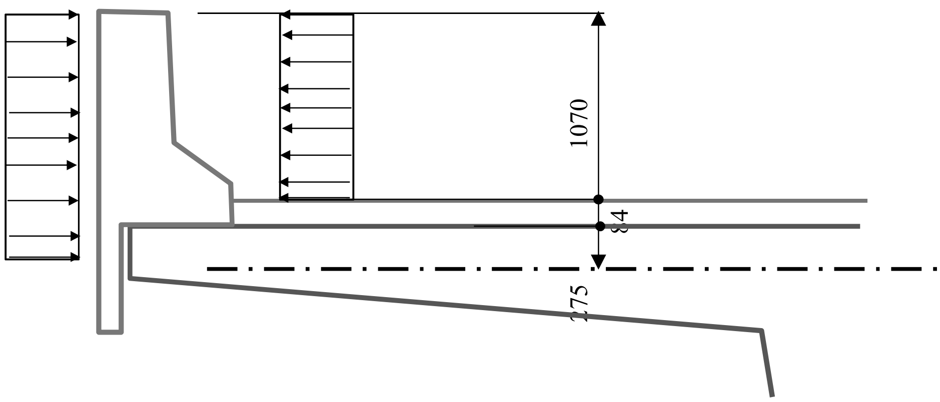
		Unit	Truck	Tandem	Result
Box Center	Mx	kNm	60.29	63.32	**63.32**
On The Web	Mx	kNm	-77.27	-73.75	**-77.27**
Overhang	Mx	kNm	-82.62	-103.86	**-103.86**
Box Center	My	kNm	29.95	31.52	**31.52**
End of Overhang	My	kNm	33.06	29.58	**33.06**

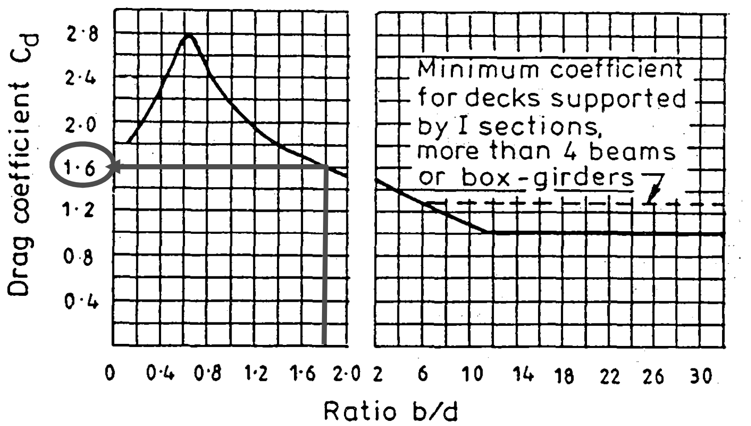
5) Temperature Effect **TG**

Only Deck Slab ∆T= **+ 7** degree Celsius.

6) Wind Load to Structure **WS**

Ratio b/d= 12.99 / 7.15 = 1.82
Cd= = 1.6
Vb= = 53 m/sec
V=Vb*S= 53 * 1.14 = 60.42 m/sec
Pd=0.0006*V^2*At*Cd≥1.8At (kN)
Pd1= 0.0006 * 60.42 ^2*Al * 1.6 = 3.505 *Al kN/m2
Pd= 3.505 * 1.0700 = 3.750 kN/m
Mwd= 3.750 * 1.429 = 5.359 kNm/m

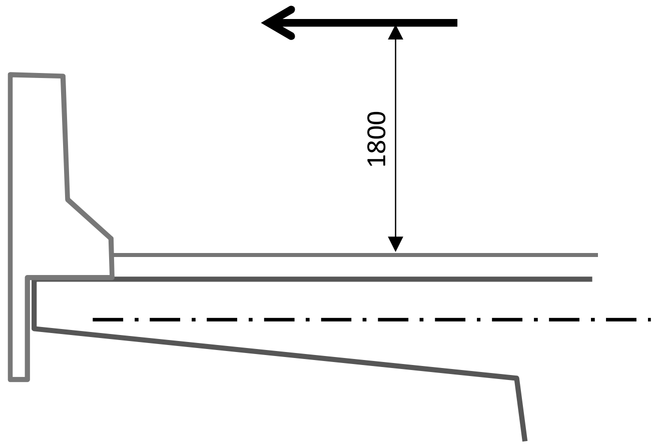




Drag Coefficient C_d for Superstructures with Solid Elevation

7) Wind Lord to Vehicle **WL**

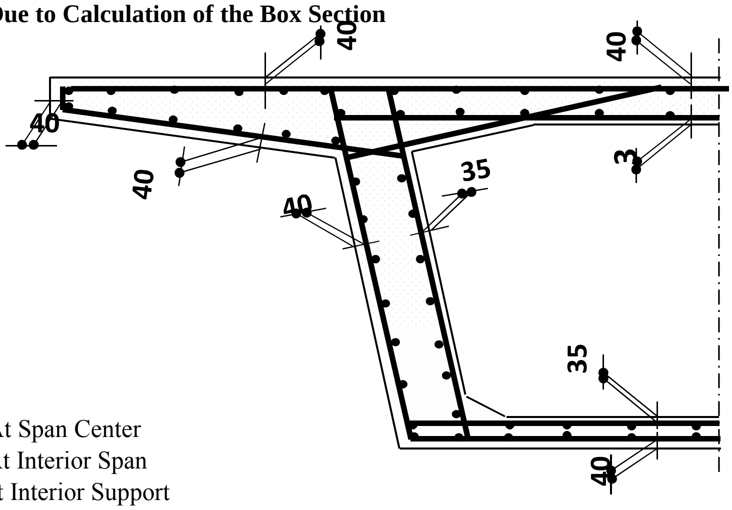
WL=				=	1.500	kN/m
Pwl=				=	1.500	kN/m
Mel=	1.500	*	1.8	=	2.700	kNm



8) Vehicle Collision for Railing A13.1.2

Item	Symbol	Unit	Value	Rmark
Reiling Testlevel	-	-	TL-5	
Ft transverse	Ft	kN	550	
Fl Longitudinal	Fl	kN	80	
Fv Vertical Down	Fv	kN	80	
Lt and Ll	L	m	2440	
Hirht of working	He	m	1070	
Minmum Heigt Rail	H	m	1070	

4. 4. Rebar Arrangement Due to Calculation of the Box Section



1 Section A :At Span Center
2 Section B :At Interior Span
3 Section C:At Interior Support

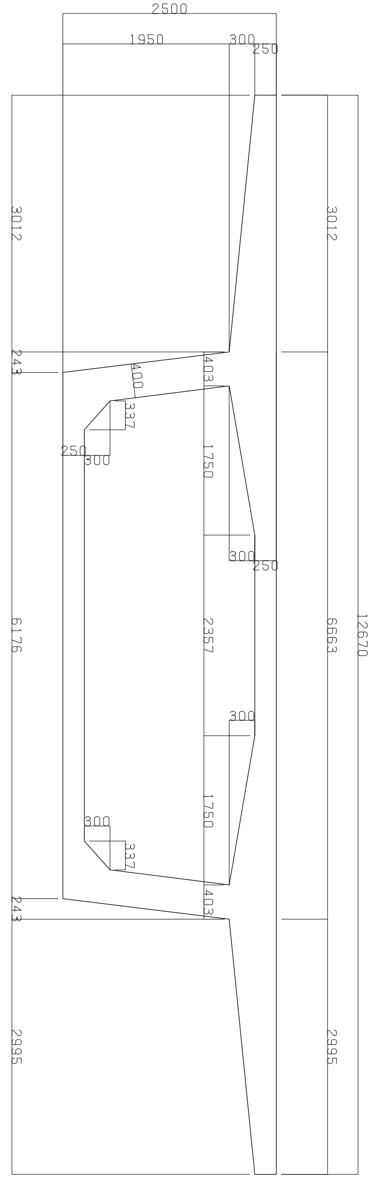
f'c= 45 Mpa fry= 400 Mpa

							Thickness	Requirement Rebar Area				Rebar Arrangement			Judge	Ratio Ar/Req Ar
								Crack Control	At Strength	Minimum	Req.Ar	Diameter	Pitch	Ar		
							mm	mm2/m	mm2/m	mm2/m	mm2/m	mm	mm	mm2/m	-	-
A		Transversal	Upper Flange	Top	Box		550	0.0	0	332.7	332.7	16	125	1608.5	OK	4.84
Section					Cantilever		550	38.4	0.0	332.7	332.7	16	125	1608.5	OK	4.84
				Bottom	Box		250	326.7	0.0	187.5	326.7	14	250	615.8	OK	1.88
	Hg 2500				Cantilever		550	46.0	0.0	332.7	332.7	14	125	1231.5	OK	3.70
	Hfu 250		Lower Flange	Top		23	250	583.0	518.6	187.5	583.0	16	250	804.2	OK	1.38
	HfL 250			Bottom		27	250	304.9	242.1	187.5	304.9	16	250	804.2	OK	2.64
	Bw 400		Web(Stirrup)				400	1333.2	1420.5	267.9	1420.5	18	125	2035.8	OK	1.43
		Longitudinal	Upper Flange	Top	Box		550	0.0	0.0	332.7	332.7	14	250	615.8	OK	1.85
					Cantilever		550	0.0	0.0	332.7	332.7	14	250	615.8	OK	1.85
				Bottom	Box		250	808.0	1225.6	187.5	1225.6	16	125	1608.5	OK	1.31
					Cantilever		550	628.0	964.4	332.7	964.4	14	125	1231.5	OK	1.28
			Lower Flange	Top			250	0.0	0.0	187.5	187.5	14	250	615.8	OK	3.28
				Bottom			250	0.0	0.0	187.5	187.5	14	250	615.8	OK	3.28
			Web				400	0.0	0.0	267.9	267.9	14	250	615.8	OK	2.30
B		Transversal	Upper Fringe	Top	Box		550	0.0	0	332.7	332.7	16	125	1608.5	OK	4.84
Section					Cantilever		550	0.0	0.0	332.7	332.7	16	125	1608.5	OK	4.84
				Bottom	Box		250	283.9	0.0	187.5	283.9	14	250	615.8	OK	2.17
	Hg 4048				Cantilever		550	0.0	0.0	332.7	332.7	14	250	615.8	OK	1.85
	Hfu 250		Lower Flange	Top		23	525	257.6	300.0	322.7	322.7	14	125	1231.5	OK	3.82
	HfL 525			Bottom		27	525	641.5	578.2	322.7	641.5	14	125	1231.5	OK	1.92
	Bw 400		Web(Stirrup)				400	1320.9	1423.4	267.9	1423.4	16	125	1608.5	OK	1.13
		Longitudinal	Upper Flange	Top	Box		550	0.0	0.0	332.7	332.7	14	250	615.8	OK	1.85
					Cantilever		550	658.4	0.0	332.7	658.4	14	125	1231.5	OK	1.87
				Bottom	Box		250	808.0	1225.6	187.5	1225.6	16	125	1608.5	OK	1.31
					Cantilever		250	808.0	964.4	187.5	964.4	16	125	1608.5	OK	1.67
			Lower Flange	Top			525	0.0	0.0	322.7	322.7	14	250	615.8	OK	1.91
				Bottom			525	0.0	0.0	322.7	322.7	14	250	615.8	OK	1.91
			Web				400	0.0	0.0	267.9	267.9	14	250	615.8	OK	2.30
C		Transversal	Upper Flange	Top	Box		550	0.0	0	332.7	332.7	16	125	1608.5	OK	4.84
Section					Cantilever		550	38.5	0.0	332.7	332.7	16	125	1608.5	OK	4.84
				Bottom	Box		250	290.1	0.0	187.5	290.1	14	250	615.8	OK	2.12
	Hg 6000				Cantilever		550	116.1	0.0	332.7	332.7	14	250	615.8	OK	1.85
	Hfu 250		Lower Flange	Top		23	1000	97.9	328.1	468.8	468.8	16	250	804.2	OK	1.72
	HfL 1000			Bottom		27	1000	601.1	541.8	468.8	601.1	16	250	804.2	OK	1.34
	Bw 600		Web(Stirrup)				600	942.8	1013.4	351.6	1013.4	16	125	1608.5	OK	1.59
		Longitudinal	Upper Flange	Top	Box		550	0.0	0.0	332.7	332.7	14	250	615.8	OK	1.85
					Cantilever		550	0.0	0.0	332.7	332.7	14	125	1231.5	OK	3.70
				Bottom	Box		250	808.0	1225.6	187.5	1225.6	16	125	1608.5	OK	1.31
					Cantilever		550	628.0	964.4	332.7	964.4	16	125	1608.5	OK	1.67
			Lower Flange	Top			1000	0.0	543.9	468.8	543.9	14	250	615.8	OK	1.13
				Bottom			1000	0.0	329.8	468.8	468.8	14	250	615.8	OK	1.31
			Web				600	0.0	0.0	351.6	351.6	14	250	615.8	OK	1.75

4. 5. 1. Section 1(Hg=2.5m,Hfu=0.25m,Hfl=0.25m,Bw=0.40m)

No

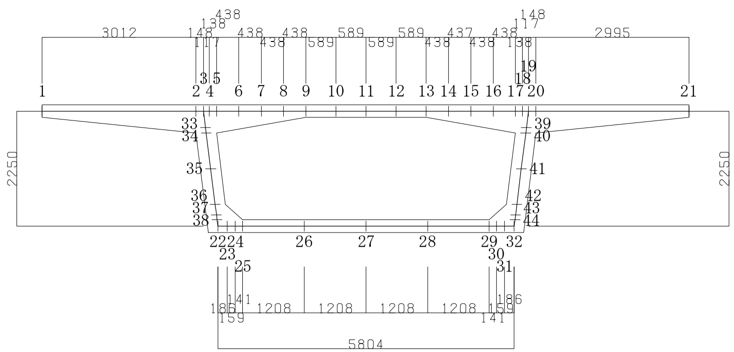
1. CROSS SECTION



No

2. ANALYSIS FOR SECTIONAL FORCE

2. 1. CENTROID AND PROPERTY



MEMBER	A (m2)	I (m4)	X (m)	Y (m)	MAT
1- 2	0.400	0.00758	3.012	0.000	1
2- 3	50.000	50.00000	0.148	0.000	1
3- 4	50.000	50.00000	0.117	0.000	1
4- 5	0.550	0.01386	0.138	0.000	1
5- 6	0.512	0.01140	0.438	0.000	1
6- 7	0.438	0.00713	0.438	0.000	1
7- 8	0.363	0.00410	0.438	0.000	1
8- 9	0.287	0.00208	0.438	0.000	1
9- 10	0.250	0.00130	0.589	0.000	1
10- 11	0.250	0.00130	0.589	0.000	1
11- 12	0.250	0.00130	0.589	0.000	1
12- 13	0.250	0.00130	0.589	0.000	1
13- 14	0.287	0.00208	0.438	0.000	1
14- 15	0.363	0.00410	0.438	0.000	1
15- 16	0.438	0.00713	0.438	0.000	1
16- 17	0.512	0.01140	0.438	0.000	1
17- 18	0.550	0.01386	0.138	0.000	1
18- 19	50.000	50.00000	0.117	0.000	1
19- 20	50.000	50.00000	0.148	0.000	1
20- 21	0.400	0.00758	2.995	0.000	1
22- 23	50.000	50.00000	0.186	0.000	1
23- 24	50.000	50.00000	0.159	0.000	1

MEMBER	A (m2)	I (m4)	X (m)	Y (m)	MAT
24- 25	0.250	0.00130	0.141	0.000	1
25- 26	0.250	0.00130	1.208	0.000	1
26- 27	0.250	0.00130	1.208	0.000	1
27- 28	0.250	0.00130	1.208	0.000	1
28- 29	0.250	0.00130	1.208	0.000	1
29- 30	0.250	0.00130	0.141	0.000	1
30- 31	50.000	50.00000	0.159	0.000	1
31- 32	50.000	50.00000	0.186	0.000	1
3- 33	50.000	50.00000	0.040	0.322	1
33- 34	0.400	0.00533	0.013	0.103	1
34- 35	0.400	0.00533	0.087	0.700	1
35- 36	0.400	0.00533	0.087	0.700	1
36- 37	0.400	0.00533	0.025	0.202	1
37- 38	50.000	50.00000	0.012	0.098	1
38- 22	50.000	50.00000	0.016	0.125	1
19- 39	50.000	50.00000	0.040	0.322	1
39- 40	0.400	0.00533	0.013	0.103	1
40- 41	0.400	0.00533	0.087	0.700	1
41- 42	0.400	0.00533	0.087	0.700	1
42- 43	0.400	0.00533	0.025	0.202	1
43- 44	50.000	50.00000	0.012	0.098	1
44- 32	50.000	50.00000	0.016	0.125	1

SUPPORT CONDITION

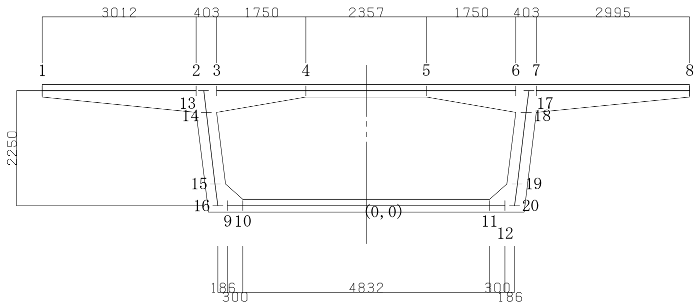
No.	CONDITION
22	H
32	Rv

MATERIAL

No.	MODULUS ELASTICITY	THERMAL EXPANSION
1	0.3220E+05	0.000010

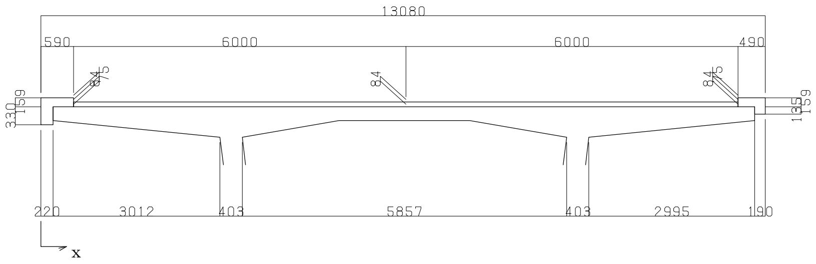
2. 2. LOADING

2. 2. 1. GIRDER-SELF LOAD



SEC.	X (m)	Y (m)	h (m)	W(kN/m)	P(kN)
1	-6.344	2.250	0.250	6.125	
2	-3.332	2.250	0.550	13.475	
3	-2.928	2.250	0.550	13.475	
4	-1.178	2.250	0.250	6.125	
5	1.178	2.250	0.250	6.125	
6	2.928	2.250	0.550	13.475	
7	3.332	2.250	0.550	13.475	
8	6.327	2.250	0.250	6.125	
9	-2.716	0.000	0.517	12.659	
10	-2.416	0.000	0.250	6.125	
11	2.416	0.000	0.250	6.125	
12	2.716	0.000	0.517	12.659	
13	-3.183	2.250	0.400	9.800	1.235
14	-3.130	1.825	0.400	9.800	
14	-3.130	1.825	0.400	9.800	
15	-2.955	0.425	0.400	9.800	
15	-2.955	0.425	0.409	10.030	
16	-2.902	0.000	0.409	10.030	1.163
17	3.183	2.250	0.400	9.800	1.235
18	3.130	1.825	0.400	9.800	
18	3.130	1.825	0.400	9.800	
19	2.955	0.425	0.400	9.800	
19	2.955	0.425	0.409	10.030	
20	2.902	0.000	0.409	10.030	1.163

2. 2. 2. SUPERIMPOSED



ITEMS	LOCATION x (m)	UNIT(kN/m3)	P(kN)	Q(kN/m)
EDGE BLOCK(L)	0.110	24.500	1.779	
PARAPET(L)	0.295	24.500	2.298	
HANDRAIL(L)	0.214		8.550	
PAVEMENT(L)	0.590	22.500		1.890
PAVEMENT(MID)	6.590	22.500		1.890
PAVEMENT(R)	12.590	22.500		1.890
HANDRAIL(R)	12.915		7.447	
PARAPET(R)	12.835	24.500	1.909	
EDGE BLOCK(R)	12.985	24.500	0.628	

WIND LOAD→
W1L = 5.360
W1R = 5.360

WUND LOAD←
W2L = 5.360
W2R = 5.360

WIND LOAD→
W3L = 1.500× (1.800+ 0.125+ 0.084) = 3.013 kN·m
W3R = 1.500× (1.800+ 0.125+ 0.084) = 3.013 kN·m

WUND LOAD←
W4L = 1.500× (1.800+ 0.125+ 0.084) = 3.013 kN·m
W4R = 1.500× (1.800+ 0.125+ 0.084) = 3.013 kN·m

2. 2. 3. SECONDARY FORCE DUE TO PRE-STRESS

1) AXIAL FORCE AND MOMENT DUE TO PRE-STRESS IMMEDIATELY AFTER ANCHOR SET

PRE-STRESS AFTER SET $Pt = \sigma pt \cdot Ap / P$

ECCENTRICITY MOMENT $Mpt = Pt \cdot ep$

$\sigma pt'$: TENSIL STRESS AT EACH SECTION

$\Delta \sigma p$: ELASTIC SHORTNING

σpt : TENDON STRESS IMMEDIATELY AFTER ANCHOR SET

Ap : AREA 416.100 mm^2

P : SPACING 0.750 m

ep : ECCENTRICITY

SEC.	σpt' Mpa	Δσp Mpa	σpt Mpa	Pt kN	ep m	Mpt kN·m	ΔMpt kN·m	ΔL m	ΔMpt/ΔL kN	P kN
							143.989			
3	1261.88	8.10	1253.78	695.599	0.207	143.989	-6.944	0.117	-59.290	59.290
4	1261.99	8.10	1253.89	695.659	0.197	137.045	-8.335	0.138	-60.618	1.328
5	1262.12	8.10	1254.02	695.729	0.185	128.710	-25.709	0.438	-58.763	-1.854
6	1262.52	8.10	1254.42	695.952	0.148	103.001	-26.838	0.438	-61.344	2.581
7	1267.55	8.10	1259.45	698.742	0.109	76.163	-32.035	0.438	-73.222	11.878
8	1270.63	8.10	1262.53	700.449	0.063	44.128	-32.918	0.438	-75.240	2.018
9	1271.03	8.10	1262.93	700.671	0.016	11.211	-8.402	0.589	-14.260	-60.980
10	1273.75	8.10	1265.65	702.182	0.004	2.809	-2.809	0.589	-4.767	-9.493
11	1279.36	8.10	1271.26	705.294	0.000	0.000	2.834	0.589	4.809	-9.576
12	1284.93	8.10	1276.83	708.387	0.004	2.834	8.525	0.589	14.468	-9.659
13	1287.64	8.10	1279.54	709.889	0.016	11.358	33.379	0.438	76.294	-61.826
14	1288.03	8.10	1279.93	710.107	0.063	44.737	32.848	0.438	75.081	1.213
15	1291.06	8.10	1282.96	711.785	0.109	77.585	28.165	0.438	64.377	10.704
16	1295.99	8.10	1287.89	714.523	0.148	105.749	26.478	0.438	60.520	3.857
17	1296.38	8.10	1288.28	714.740	0.185	132.227	8.590	0.138	62.475	-1.955
18	1296.51	8.10	1288.41	714.808	0.197	140.817	7.160	0.117	61.133	1.342
19	1296.61	8.10	1288.51	714.866	0.207	147.977	-147.977			61.133

2) TEMPERATURE FORCE EQUIVALENT TO SECONDARY FORCE DUE TO ELASTIC SHORTNING

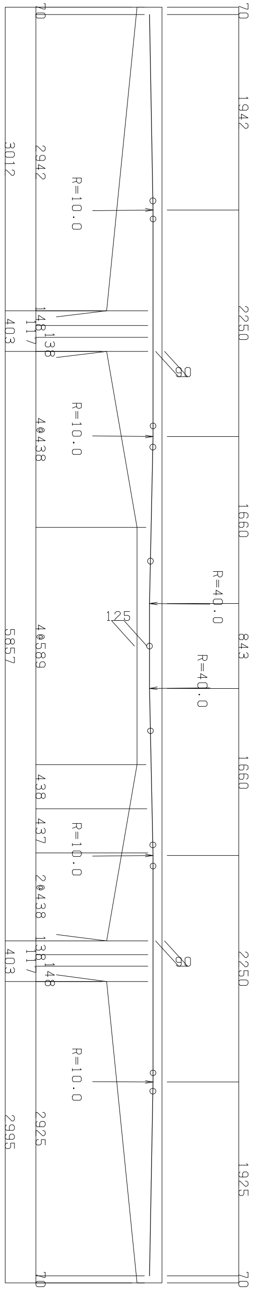
AVERAGE AREA OF TOP SLAB $A = 0.340\ m^2$

$\sigma c = Pt / A = 705.3 / 0.340 = 2076.5\ kN/m^2$

$\sigma c / Ec = 2076.5 / 28800000.0 = 7.2\times1^{0-}$

THEREFORE, 7.2℃ DIFFERENCE IS LOADED.

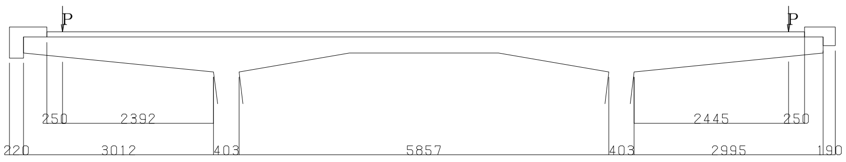
TENDON ARRANGEMENT



ep (m)	0.004	0.185 0.207 0.185 0.185	0.148	0.109	0.063	0.016	0.004	0.000	0.004	0.016	0.063	0.109	0.148	0.185 0.185 0.207 0.185	0.003
yp (m)	0.125	0.090 0.090 0.090 0.090	0.090	0.091	0.100	0.109	0.121	0.125	0.121	0.109	0.100	0.091	0.090	0.090 0.090 0.090 0.090	0.125
tu (m)	0.257	0.550 0.594 0.550 0.550	0.475	0.400	0.325	0.250	0.250	0.250	0.250	0.250	0.325	0.400	0.475	0.550 0.550 0.594 0.550	0.257

No

2. 2. 4. LIVE LOAD



TRUCK LOAD ON CANTILEVER(L)

CANTILEVER LEGTH l = 2.392 m IMPACT K1 = 1.000

M = -103.860 kN·m

TRUCK LOAD ON SPAN (AT WEB)

SPAN LEGTH l = 5.857 m IMPACT K1 = 1.000

M = -77.270 kN·m

TRUCK LOAD (AT MID SPAN)

SPAN LEGTH l = 5.857 m IMPACT K1 = 1.000

M = 63.320 kN·m

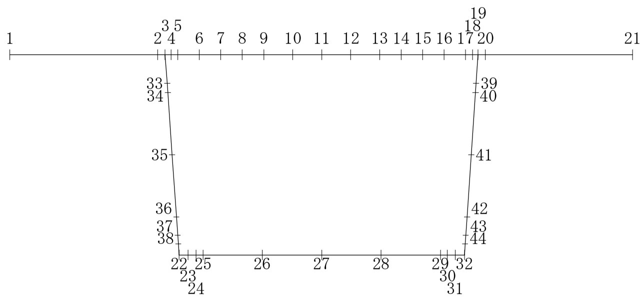
TRUCK LOAD ON CANTILEVER(R)

CANTILEVER LEGTH l = 2.445 m IMPACT K1 = 1.000

M = -103.860 kN·m

No

2. 2. 5. LOADING CASE



CASE 1 (GIRDER-SELF LOAD)

ALL MEMBERS ARE LOADED

CASE 2 (SUPERIMPOSED)

TOP SLAB 1 ～ 21 ARE LOADED

CASE 3 (SECONDARY FORCE DUE TO PRE-STRESS)

TOP SLAB 3 IS LOADED144.0 kN·m AND 59.3 kN

TOP SLAB 19 IS LOADED 148.0 kN·m AND 61.1 kN

TOP SLAB 4 ～ 18 ARE LOADED

CASE 4 (TRUCK LOAD ON CANTILEVER(L))

SEC. 3 IS LOADED MOMENT (-103.860 kN·m) DUE TO TRUCK LOAD

CASE 5 (TRUCK LOAD ON CANTILEVER(R))

SEC.19 IS LOADED MOMENT (-103.860 kN·m) DUE TO TRUCK LOAD

CASE 6 (TRUCK LOAD ON SLAB)

SEC. 3 AND 19 ARE LOADED MOMENT (-77.270 kN·m) DUE TO TRUCK LOAD

CASE 7 (TEMPERATURE GRADIENT)

TOP SLAB 3 ～19 ARE LOADED ＋ 7.0 ℃

CASE 8(WIND LOAD ON STRUCTURE(L))

No

2．2．6． LOAD COMBINATIONS

			1	2	3	4	5	6	7	8	9	10	11				
			DC	DW	EL	CR	SH	LL+IM	PL	TG	WS	WL	CT				
SLS	Service-1	SE1-1	1.00	1.00	1.00	1.00		1.00	1.00	0.50	0.30	1.00		Lmax	R	±	1
		SE1-2	1.00	1.00	1.00	1.00		1.00	1.00	0.50	0.30	1.00		Lmin	R	±	2
		SE1-3	1.00	1.00	1.00	1.00		1.00	1.00	0.50	-0.30	-1.00		Lmax	L	±	3
		SE1-4	1.00	1.00	1.00	1.00		1.00	1.00	0.50	-0.30	-1.00		Lmin	L	±	4
		SE1-5	1.00	1.00	1.00	1.00				1.00	0.30			Wind	R	±	5
		SE1-6	1.00	1.00	1.00	1.00				1.00	-0.30			Wind	L	±	6
	Service-3	SE3-1	1.00	1.00	1.00	1.00		0.80	0.80	0.50				Lmax	-	±	7
		SE3-2	1.00	1.00	1.00	1.00		0.80	0.80	0.50				Lmin	-	±	8
		SE3-3	1.00	1.00	1.00	1.00				1.00					-	±	9

No

CASE 9(WIND LOAD ON STRUCTURE(R))

CASE 10(WIND LOAD ON VEHICLE(L))

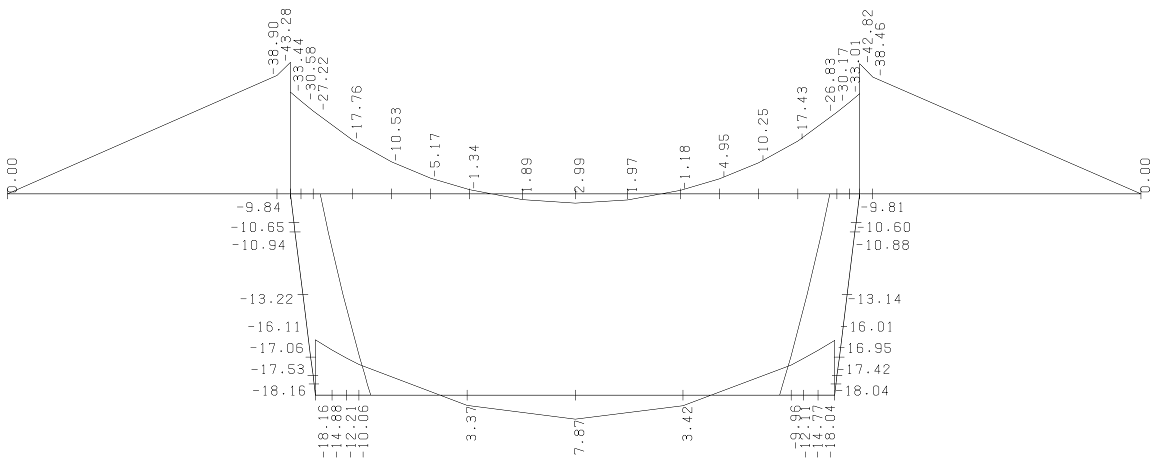
CASE 11(WIND LOAD ON VEHICLE(R))

CASE 12 (TEMPERATURE GRADIENT)

TOP SLAB 3 ～19 ARE LOADED － 2.0 ℃

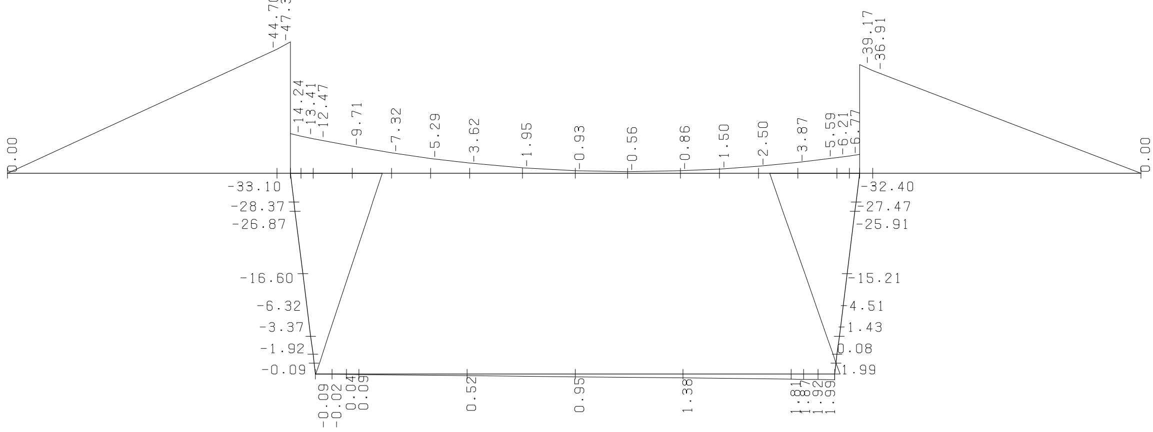
2. 3. BENDING MOMENT

CASE 1 (GIRDER-SELF LOAD)

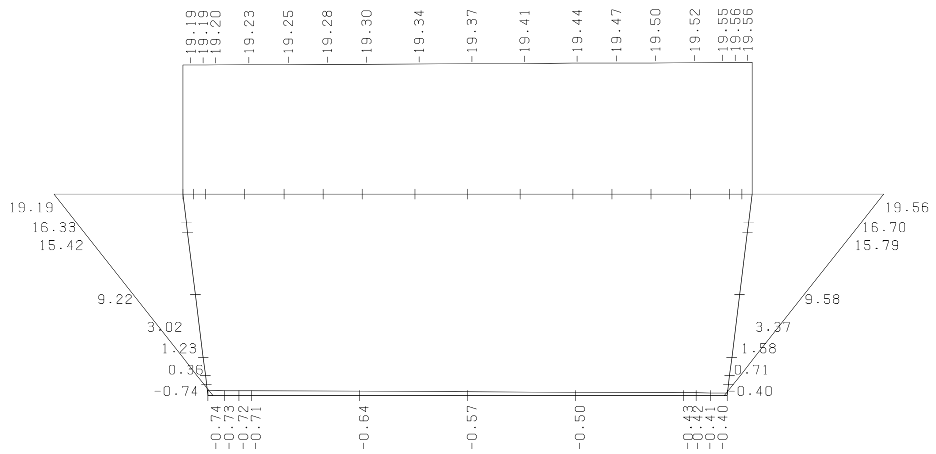


No

CASE 2 (SUPERIMPOSED)

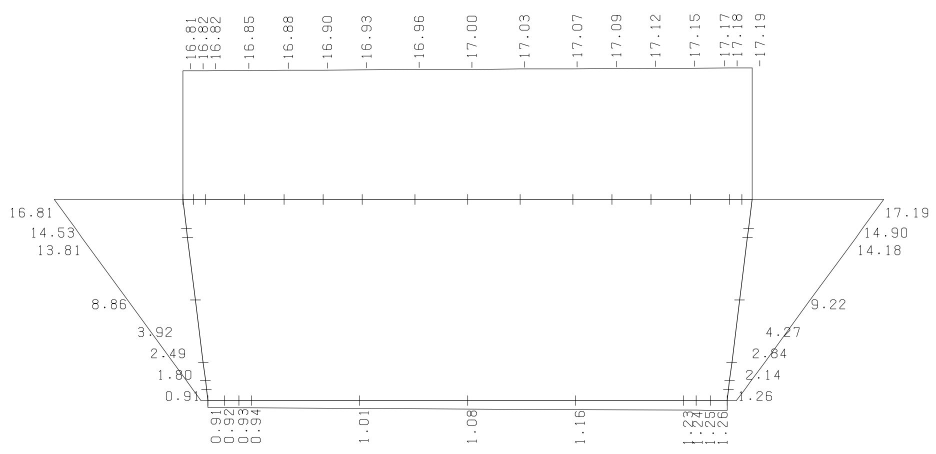


No



CASE 3-1 (MOMENT DUE TO PRESTRES)

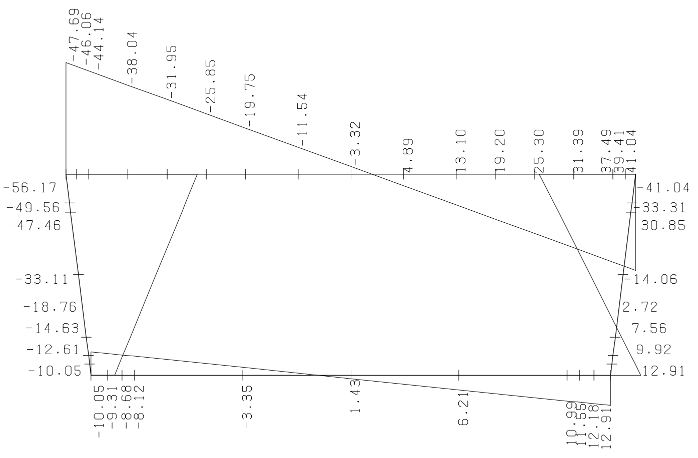
No



CASE 3-2 (SECONDARY MOMENT DUE TO PRESTRESS - ECCENTRICITY+ELASTIC SHORTNING)

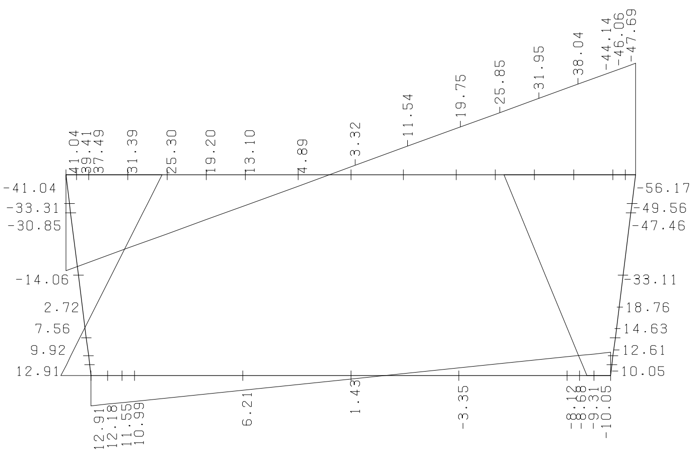
No

CASE 4 (TRUCK LOAD ON CANTILEVER(L))

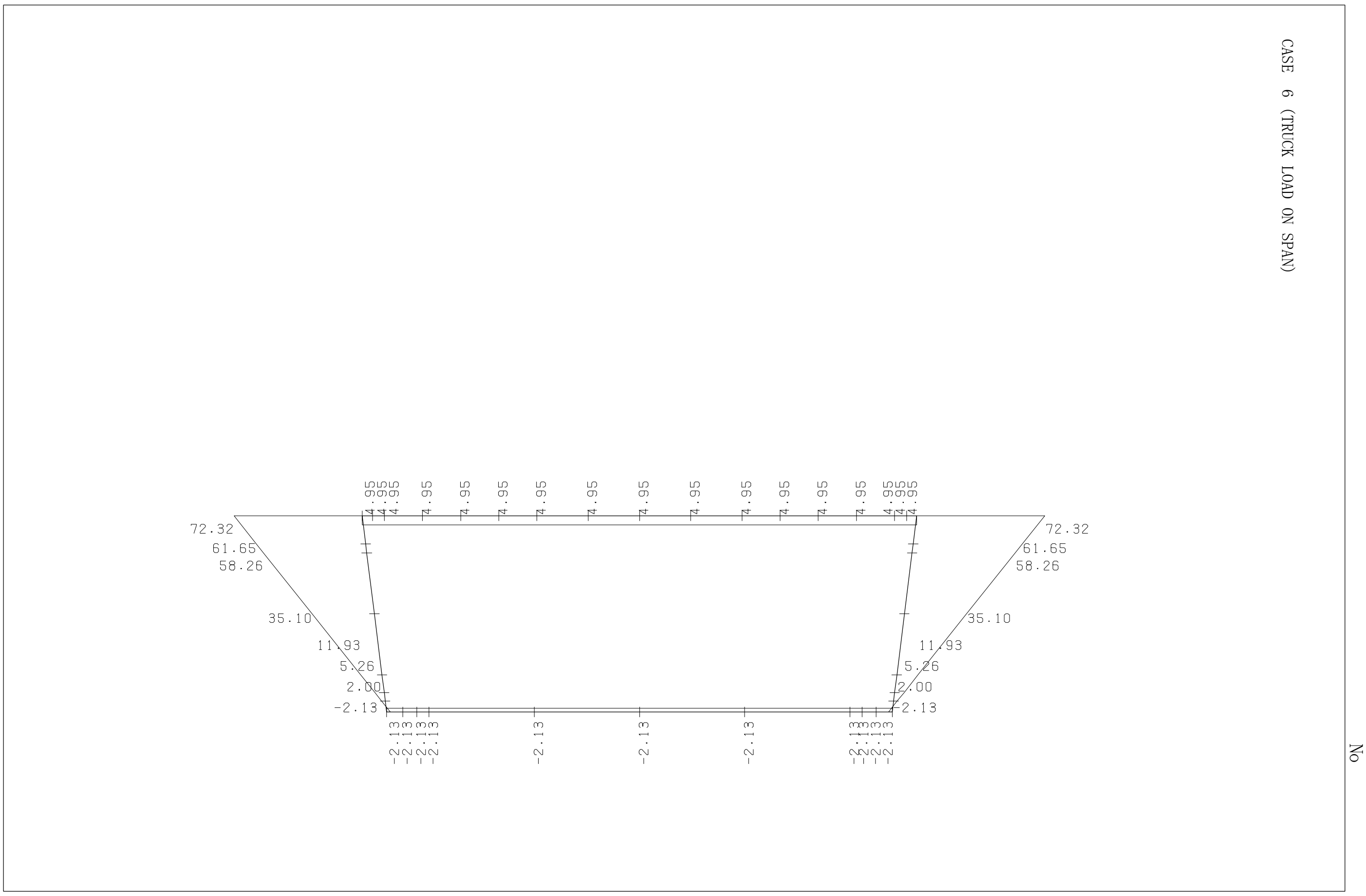


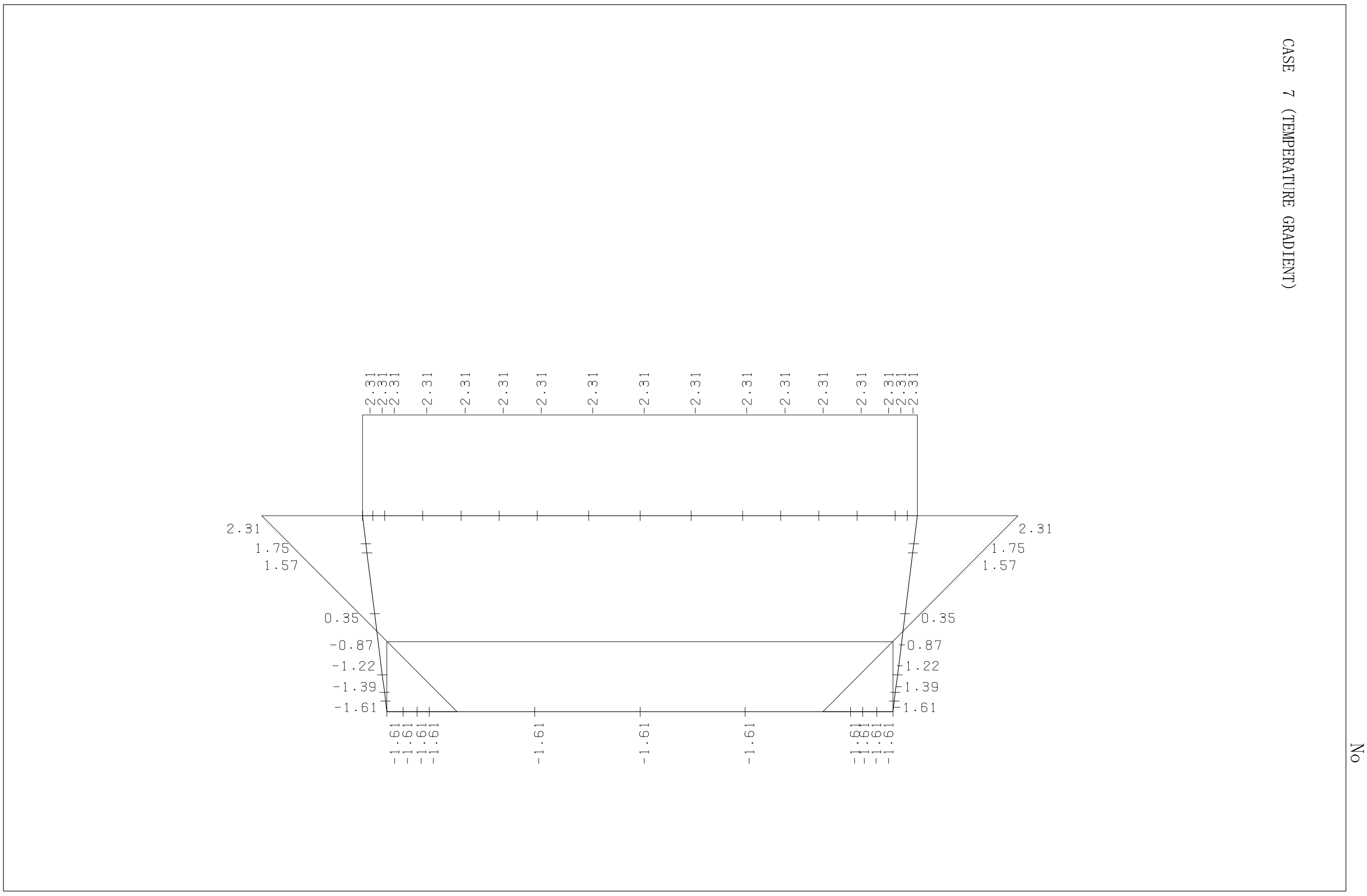
No

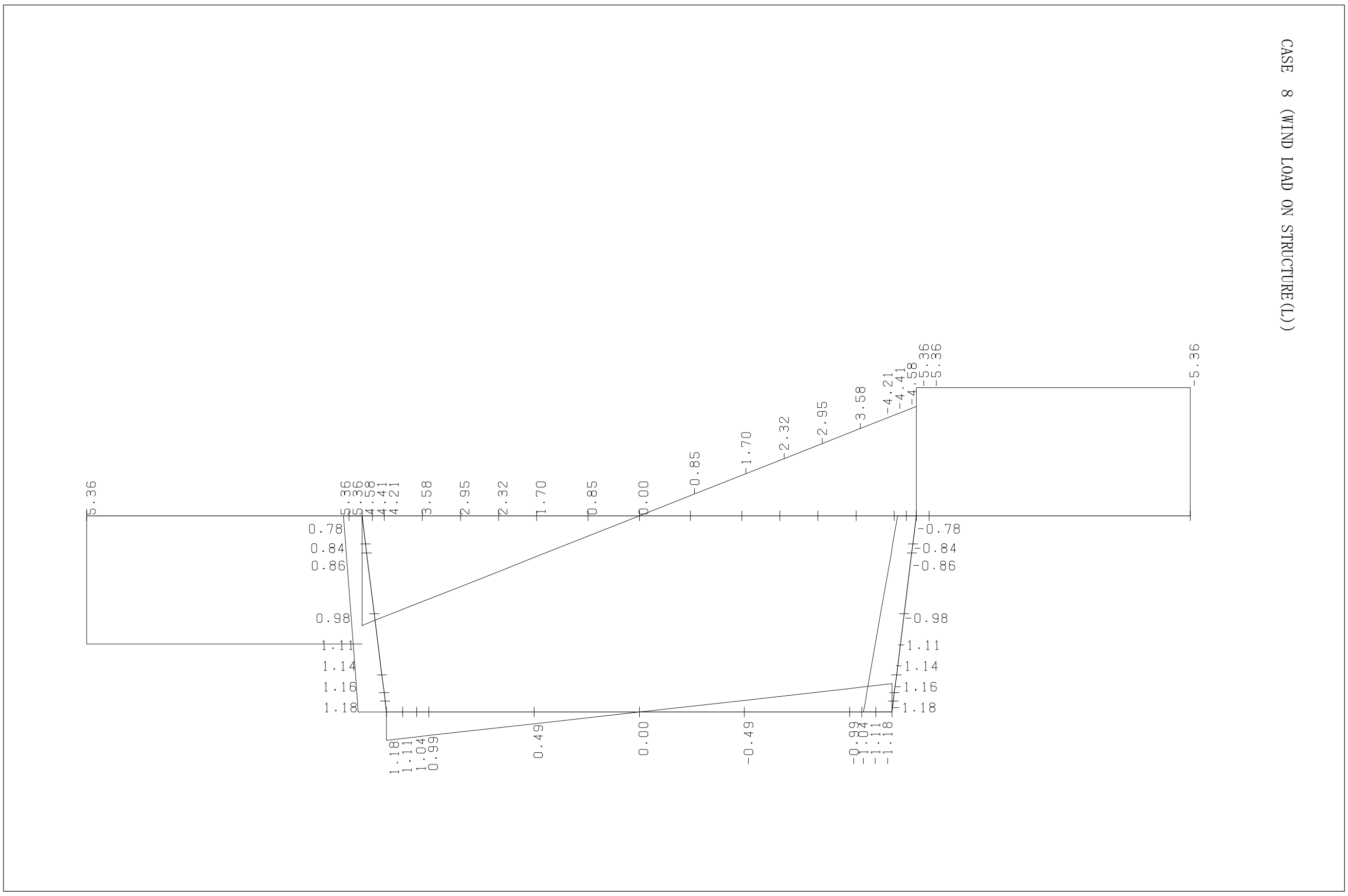
CASE 5 (TRUCK LOAD ON CANTILEVER(R))

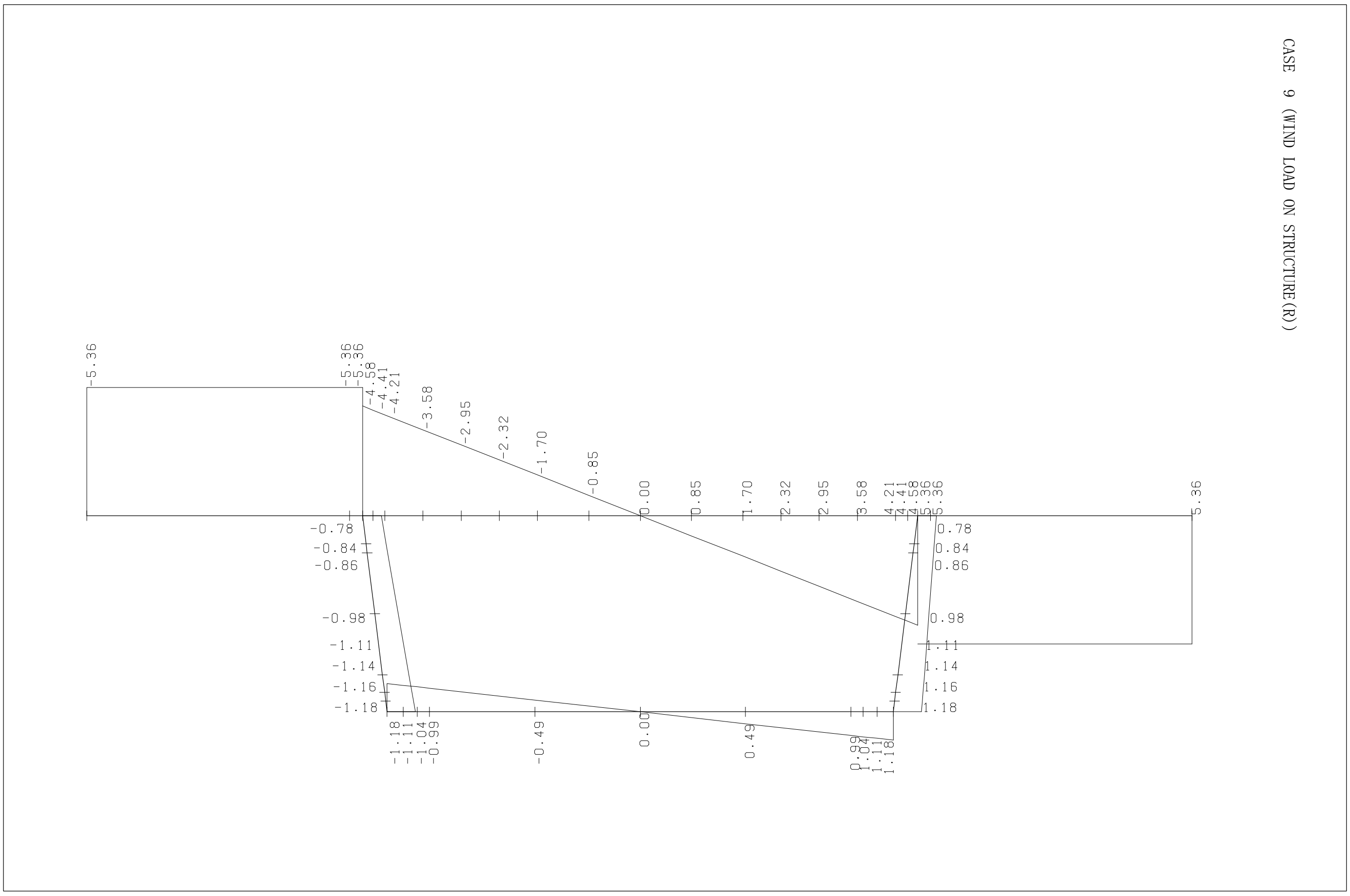


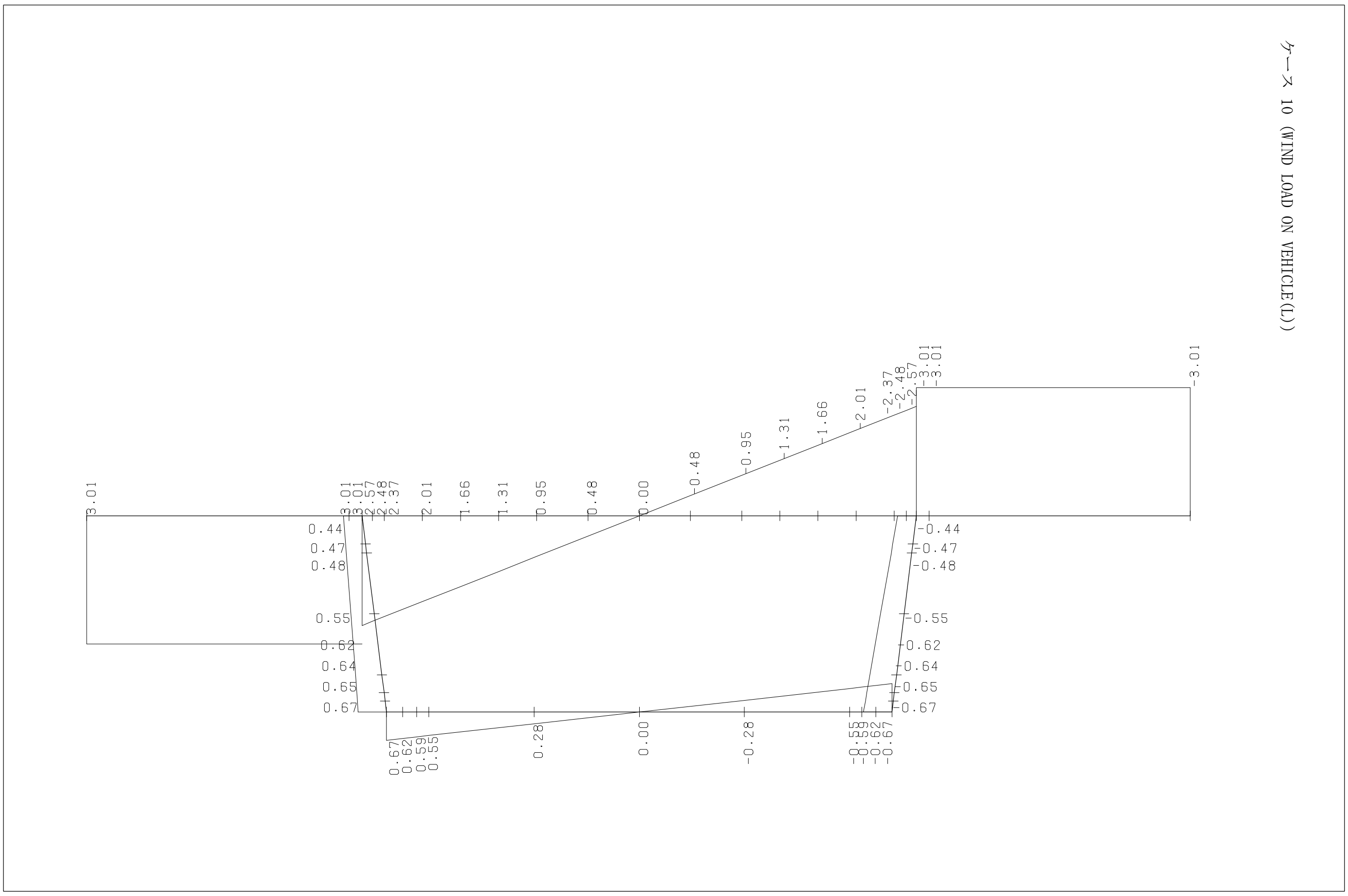
No

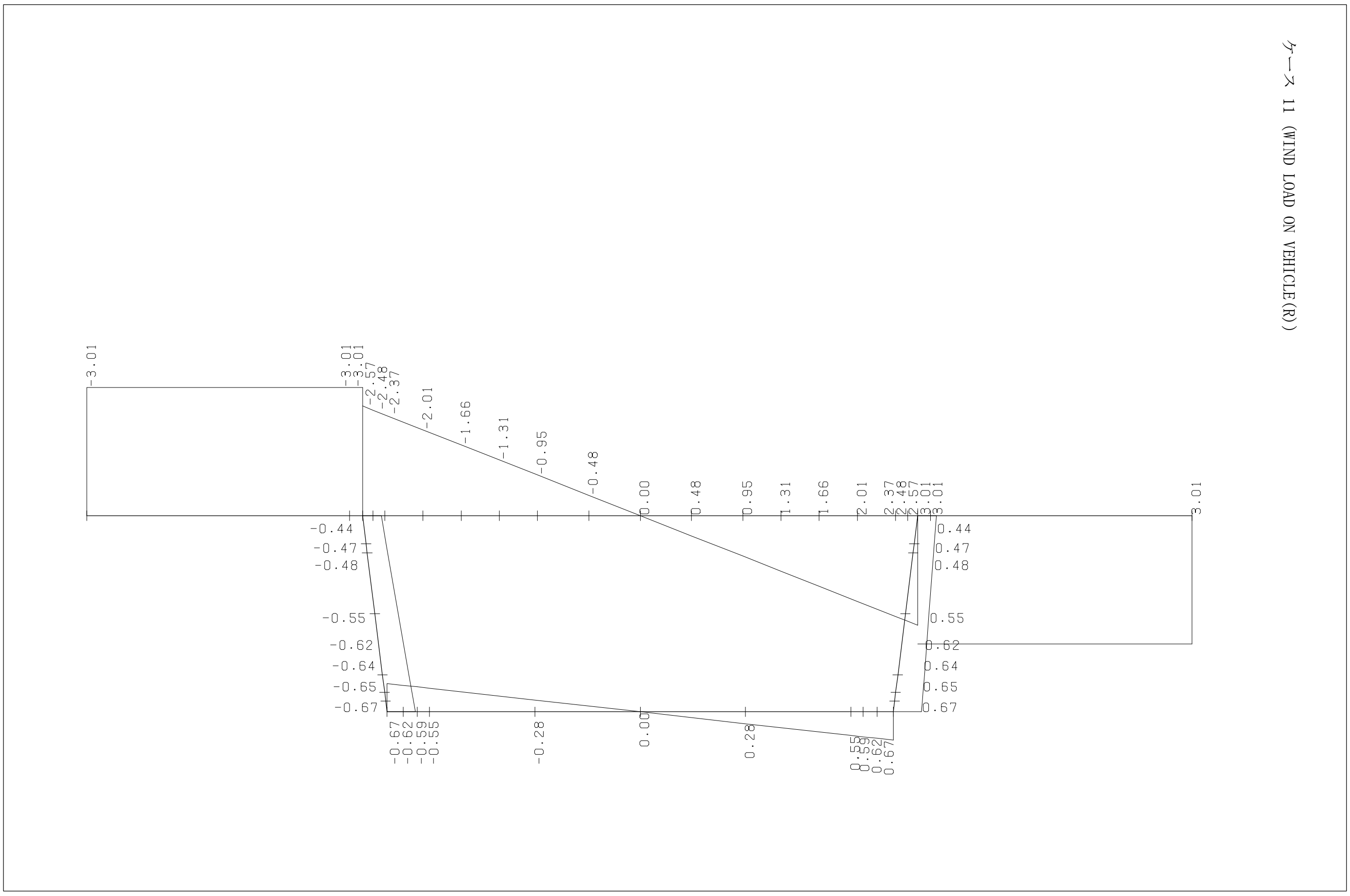

CASE 6 (TRUCK LOAD ON SPAN)
4.95
72.32
61.65
58.26
35.10
11.93
5.26
2.00
-2.13
No


CASE 7 (TEMPERATURE GRADIENT)
-2.31
2.31
1.75
1.57
0.35
-0.87
-1.22
-1.39
-1.61
No

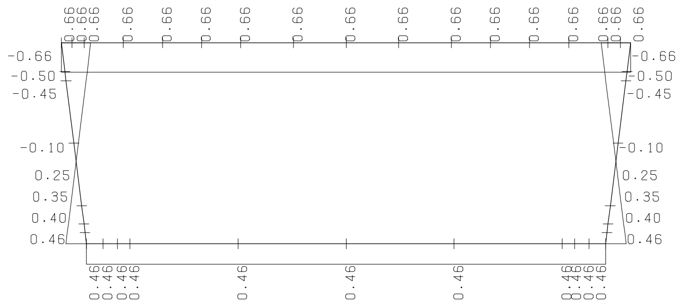

CASE 8 (WIND LOAD ON STRUCTURE(L))
No


CASE 9 (WIND LOAD ON STRUCTURE(R))
No


ケース 10 (WIND LOAD ON VEHICLE(L))


ケース 11 (WIND LOAD ON VEHICLE(R))

ケース 12 (TEMPERATURE GRADIENT)

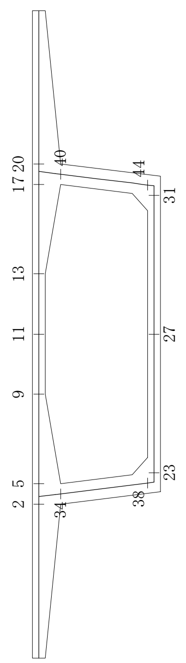


No

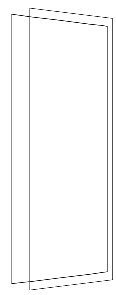
No

2．4．COMBINATION

BOX-SECTION MODEL FRAME



MOMENT SHALL BE POSITIVE AS THE DOT LINE SIDE IS TENSILE



No

SERVICE LIMIT STATE

COLLECTION TABLE (TOP SLAB kN·m)

			2	5	9	11	13	17	20
GIRDER-SELF LOAD			-38.90	-27.22	-1.34	2.99	-1.18	-26.83	-38.46
SUPERIMPOSED			-44.70	-12.47	-3.62	-0.93	-0.86	-5.59	-36.91
SECONDARY MOMENT	CONST.			-16.82	-16.93	-17.00	-17.07	-17.17	
	SERVICE			-15.50	-15.60	-15.66	-15.73	-15.82	
CREEP				1.32	1.33	1.34	1.34	1.35	
LIVE LOAD(LL+IM)	MAX.					63.32			
	MIN.		-103.86	-77.27				-77.27	-103.86
+TEMP.	7.00	1.00·TG		-2.31	-2.31	-2.31	-2.31	-2.31	
	3.50	0.50·TG		-1.15	-1.15	-1.15	-1.15	-1.15	
-TEMP.	-2.00	1.00·TG		0.66	0.66	0.66	0.66	0.66	
	-1.00	0.50·TG		0.33	0.33	0.33	0.33	0.33	
WS	→	0.30·WS	1.61	1.26	0.51	0.00	-0.51	-1.26	-1.61
	←	0.30·WS	-1.61	-1.26	-0.51	0.00	0.51	1.26	1.61
WL	→	1.00·WL	3.01	2.37	0.95	0.00	-0.95	-2.37	-3.01
	←	1.00·WL	-3.01	-2.37	-0.95	0.00	0.95	2.37	3.01
AFTER ANCHOR SET			-38.90	-44.04	-18.27	-14.01	-18.25	-44.00	-38.46
PERMANENT LOAD			-83.60	-55.19	-20.56	-13.60	-17.77	-48.24	-75.37
SLS	Service-1	SE1- 1	-78.98	-52.71	-20.25	50.05	-20.38	-53.03	-79.99
		SE1- 2	-182.84	-129.98	-20.25	-14.75	-20.38	-130.30	-183.85
		SE1- 3	-88.22	-59.98	-23.18	50.05	-17.46	-45.76	-70.74
		SE1- 4	-192.08	-137.25	-23.18	-14.75	-17.46	-123.03	-174.60
		SE1- 5	-81.99	-56.23	-22.36	-15.91	-20.58	-51.81	-76.97
		SE1- 6	-85.21	-58.76	-23.38	-15.91	-19.56	-49.29	-73.76
		MAX.	-78.98	-52.71	-20.25	50.05	-17.46	-45.76	-70.74
		MIN.	-192.08	-137.25	-23.38	-15.91	-20.58	-130.30	-183.85
	Service-3	SE3- 1	-83.60	-56.34	-21.71	37.39	-18.92	-49.40	-75.37
		SE3- 2	-166.69	-118.16	-21.71	-14.75	-18.92	-111.21	-158.45
		SE3- 3	-83.60	-57.50	-22.87	-15.91	-20.07	-50.55	-75.37
		MAX.	-83.60	-56.34	-21.71	37.39	-18.92	-49.40	-75.37
		MIN.	-166.69	-118.16	-22.87	-15.91	-20.07	-111.21	-158.45

No

COLLECTION TABLE (BOTTOM SLAB kN·m)

			23	27	31
CASE 4			-9.31	1.43	12.18
CASE 5			-1.72	12.91	1.67
CASE 6			12.18	1.43	-9.31
CASE 7			-27.25	7.92	-24.90
CASE 8			-2.13	-2.13	-2.13
CASE 9			-5.11	12.34	-2.76
GIRDER-SELF LOAD			-14.88	7.87	-14.77
SUPERIMPOSED			-0.02	0.95	1.92
SECONDARY MOMENT	CONST.		0.92	1.08	1.25
	SERVICE		0.85	1.00	1.15
CREEP			-0.07	-0.09	-0.10
LIVE LOAD(LL+IM)	MAX.		12.18	2.87	12.18
	MIN.		-11.44	-2.13	-11.44
+TEMP.	7.00	1.00·TG	-1.61	-1.61	-1.61
	3.50	0.50·TG	-0.80	-0.80	-0.80
-TEMP.	-2.00	1.00·TG	0.46	0.46	0.46
	-1.00	0.50·TG	0.23	0.23	0.23
WS	→	0.30·WS	0.33	0.00	-0.33
	←	0.30·WS	-0.33	0.00	0.33
WL	→	1.00·WL	0.62	0.00	-0.62
	←	1.00·WL	-0.62	0.00	0.62
AFTER ANCHOR SET			-13.96	8.95	-13.52
PERMANENT LOAD			-14.05	9.82	-11.70
SLS	Service-1	SE1- 1	-1.72	12.91	-1.28
		SE1- 2	-25.34	7.92	-24.90
		SE1- 3	-3.63	12.91	1.67
		SE1- 4	-27.25	7.92	-22.99
		SE1- 5	-15.32	10.28	-13.64
		SE1- 6	-15.99	10.28	-12.97
		MAX.	-1.72	12.91	1.67
		MIN.	-27.25	7.92	-24.90
	Service-3	SE3- 1	-5.11	12.34	-2.76
		SE3- 2	-24.01	8.34	-21.66
		SE3- 3	-15.66	10.28	-13.30
		MAX.	-5.11	12.34	-2.76
		MIN.	-24.01	8.34	-21.66

No

3.STRESS CHECK FOR TOP SLAB

3．1．TRANSVERSAL

3．1．1.STRESS DUE TO LOAD

(kN·m)

			2	5	9	11	13	17	20
MOMENT	AFTER ANCHOR SET		-38.90	-44.04	-18.27	-14.01	-18.25	-44.00	-38.46
	PERMANENT LOAD		-83.60	-55.19	-20.56	-13.60	-17.77	-48.24	-75.37
	SERVICE	MAX.	-78.98	-52.71	-20.25	50.05	-17.46	-45.76	-70.74
		MIN.	-192.08	-137.25	-23.38	-15.91	-20.58	-130.30	-183.85
	Service-3	MAX.	-83.60	-56.34	-21.71	37.39	-18.92	-49.40	-75.37
		MIN.	-166.69	-118.16	-22.87	-15.91	-20.07	-111.21	-158.45
I / yc (m3)			0.0504	0.0504	0.0104	0.0104	0.0104	0.0504	0.0504
STRESS	AFTER ANCHOR SET		-0.77	-0.87	-1.75	-1.34	-1.75	-0.87	-0.76
			0.77	0.87	1.75	1.34	1.75	0.87	0.76
	PERMANENT LOAD		-1.66	-1.09	-1.97	-1.31	-1.71	-0.96	-1.49
			1.66	1.09	1.97	1.31	1.71	0.96	1.49
	Service-1	MAX.	-1.57	-1.05	-1.94	4.80	-1.68	-0.91	-1.40
			1.57	1.05	1.94	-4.80	1.68	0.91	1.40
		MIN.	-3.81	-2.72	-2.24	-1.53	-1.98	-2.58	-3.65
			3.81	2.72	2.24	1.53	1.98	2.58	3.65
	Service-3	MAX.	-1.66	-1.12	-2.08	3.59	-1.82	-0.98	-1.49
			1.66	1.12	2.08	-3.59	1.82	0.98	1.49
		MIN.	-3.31	-2.34	-2.20	-1.53	-1.93	-2.21	-3.14
			3.31	2.34	2.20	1.53	1.93	2.21	3.14

(Mpa)

IN STRESS TABLE, UPPER COLUMN SHOWS EXTREME TOP AND LOWER COLUMN SHOWS EXTREME BOTTOM

STRESS CALCULATION

$$\left.\begin{matrix}\sigma_{cu}\\ \sigma_{cl}\end{matrix}\right\} = \pm \frac{M}{Z}$$

M : BENDING MOMENT (kN·m)

Z : I/yc (m3)

σ_{cu} : EXTREMELY FIBER STRESS(TOP) (Mpa)

σ_{cu} : EXTREMELY FIBER STRESS(BOTTOM) (Mpa)

No

COLLECTION TABLE (WEB kN·m)

			34	38	40	44
CASE 4			-47.46	-12.61	-30.85	9.92
CASE 5			34.70	-5.57	36.06	-3.14
CASE 6			-30.85	9.92	-47.46	-12.61
CASE 7			-104.35	-32.11	-103.00	-29.67
CASE 8			58.26	2.00	58.26	2.00
CASE 9			22.31	-8.96	23.67	-6.52
GIRDER-SELF LOAD			-10.94	-17.53	-10.88	-17.42
SUPERIMPOSED			-26.87	-1.92	-25.91	0.08
SECONDARY MOMENT	CONST.		13.81	1.80	14.18	2.14
	SERVICE		12.72	1.65	13.07	1.97
CREEP			-1.08	-0.14	-1.11	-0.17
LIVE LOAD(LL+IM)	MAX.		58.26	11.92	58.26	11.92
	MIN.		-78.31	-12.61	-78.31	-12.61
+TEMP.	7.00	1.00·TG	1.57	-1.39	1.57	-1.39
	3.50	0.50·TG	0.78	-0.70	0.78	-0.70
-TEMP.	-2.00	1.00·TG	-0.45	0.40	-0.45	0.40
	-1.00	0.50·TG	-0.22	0.20	-0.22	0.20
WS	→	0.30·WS	0.26	0.35	-0.26	-0.35
	←	0.30·WS	-0.26	-0.35	0.26	0.35
WL	→	1.00·WL	0.48	0.65	-0.48	-0.65
	←	1.00·WL	-0.48	-0.65	0.48	0.65
AFTER ANCHOR SET			2.87	-15.74	3.29	-15.28
PERMANENT LOAD			-25.08	-17.80	-23.73	-15.37
SLS	Service-1	SE1- 1	34.70	-5.57	34.58	-5.14
		SE1- 2	-102.87	-30.10	-103.00	-29.67
		SE1- 3	33.22	-7.58	36.06	-3.14
		SE1- 4	-104.35	-32.11	-101.52	-27.67
		SE1- 5	-25.27	-18.84	-24.43	-17.10
		SE1- 6	-25.79	-19.54	-23.92	-16.41
		MAX.	34.70	-5.57	36.06	-3.14
		MIN.	-104.35	-32.11	-103.00	-29.67
	Service-3	SE3- 1	22.31	-8.96	23.67	-6.52
		SE3- 2	-87.95	-28.58	-86.60	-26.15
		SE3- 3	-25.53	-19.19	-24.17	-16.76
		MAX.	22.31	-8.96	23.67	-6.52
		MIN.	-87.95	-28.58	-86.60	-26.15

No

3. 1. 2. PRE-STRESS

1) PRE-STRESS IMMEDIATELY AFTER ANCHOR SET

TYPE : STRAND CABLE

STRESSING : ONE SIDE

$$\sigma_{pti} = \sigma_{pt0} \times e^{-(\lambda \cdot l + \mu \cdot \alpha)}$$

σ_{pt0} = 1400.0 Mpa

λ = 0.001

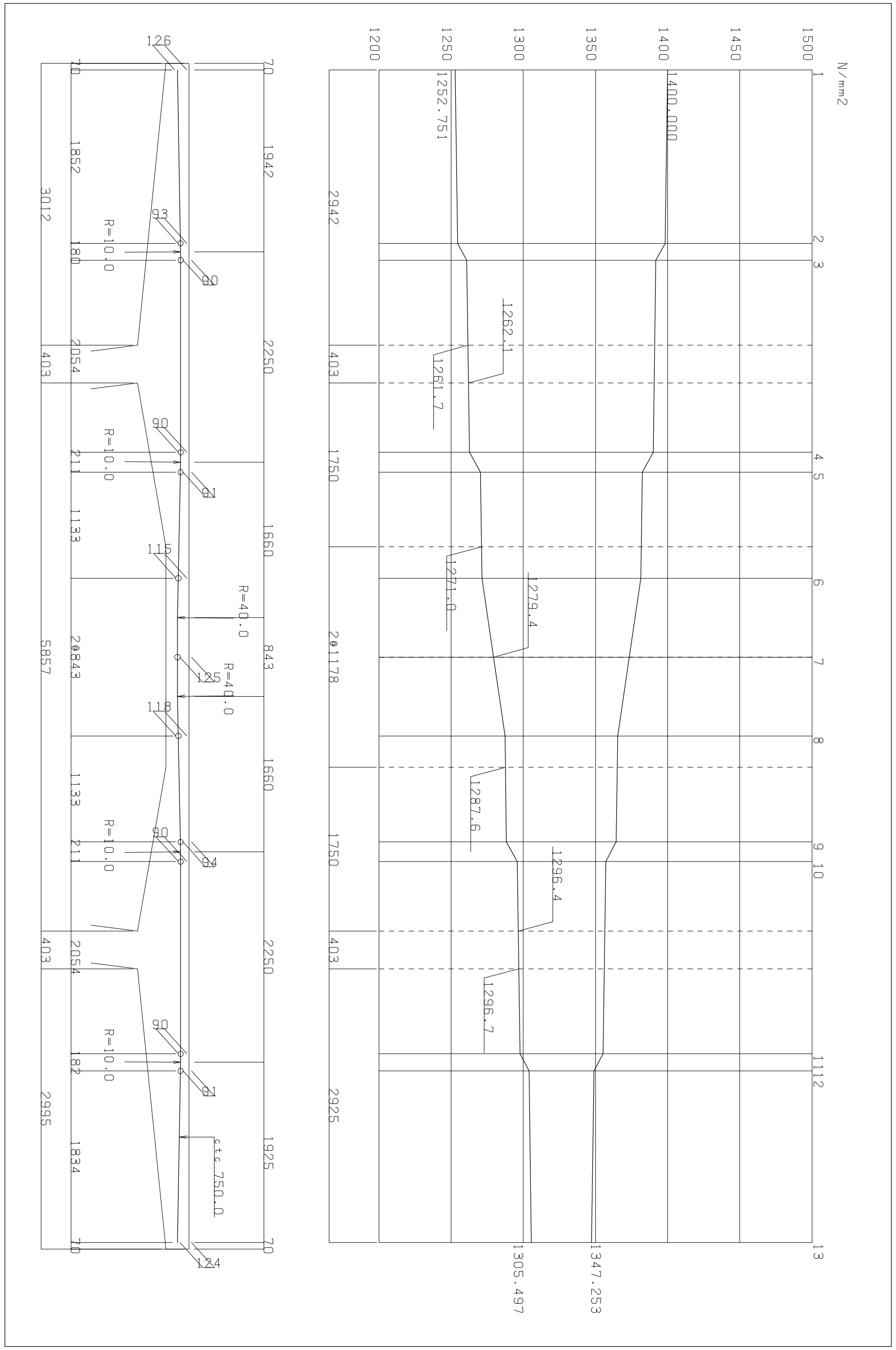
μ = 0.250

STRESS IMMEDIATELY AFTER ANCHOR SET AT INFLICTION POINTS

POINTS	l (m)	α (Rad)	λ・l	μ・α	λ・l+μ・α	e−(λ・l+μ・α)	σpti (Mpa)	ANCHOR SET(Mpa)
1	0.000	0.0000	0.0000	0.0000	0.0000	1.0000	1400.00	1252.75
2	1.852	0.0000	0.0012	0.0000	0.0012	0.9988	1398.29	1254.46
3	2.032	0.0180	0.0013	0.0045	0.0058	0.9942	1391.84	1260.91
4	4.087	0.0180	0.0027	0.0045	0.0072	0.9928	1389.95	1262.80
5	4.298	0.0391	0.0028	0.0098	0.0126	0.9875	1382.45	1270.30
6	5.431	0.0391	0.0036	0.0098	0.0134	0.9867	1381.42	1271.33
7	6.274	0.0602	0.0041	0.0150	0.0192	0.9810	1373.39	1279.36
8	7.117	0.0813	0.0047	0.0203	0.0250	0.9753	1365.41	1287.34
9	8.251	0.0813	0.0054	0.0203	0.0258	0.9746	1364.39	1288.36
10	8.461	0.1024	0.0056	0.0256	0.0312	0.9693	1357.03	1295.72
11	10.515	0.1024	0.0069	0.0256	0.0325	0.9680	1355.19	1297.56
12	10.697	0.1205	0.0071	0.0301	0.0372	0.9635	1348.89	1303.87
13	12.531	0.1205	0.0083	0.0301	0.0384	0.9623	1347.25	1305.50

STRESS IMMEDIATELY AFTER ANCHOR SET AT DESIGN SECTION

SEC.	X (m)	ANCHOR SET(Mpa)
2	3.012	1261.75
5	3.415	1262.12
9	5.165	1271.03
11	6.344	1279.36
13	7.522	1287.64
17	9.272	1296.38
20	9.675	1296.74



No

STRESS IMMEDIATELY AFTER ANCHOR SET AT DESIGN SECTION

$$\sigma pt = \sigma pt' - \Delta \sigma p$$

$\sigma pt'$: TENSILE STRESS AT DESIGN
$\Delta \sigma p$: ELASTIC SHORTNING

$$Pt = \frac{1.0\ m}{0.75\ m} \cdot Ap \cdot \sigma pt$$

$Ap = 416.1\ mm^2$

SEC.	σpt′ Mpa	Δσp Mpa	σpt Mpa	Pt kN
2	1261.75	8.10	1253.65	695.523
5	1262.12	8.10	1254.02	695.729
9	1271.03	8.10	1262.93	700.671
11	1279.36	8.10	1271.26	705.294
13	1287.64	8.10	1279.54	709.889
17	1296.38	8.10	1288.28	714.740
20	1296.74	8.10	1288.64	714.940

2) PRE-STRESS AT SERVICE

STRESS AFTER ANCHOR SET AT CENTROID OF TENDONS

$$\sigma cpt = \frac{Pt}{A} + \frac{Pt \cdot ep}{Zp}$$

SEC.	Pt kN	A m2	ep m	Zp m3	σcpt kN/m2
2	695.523	0.550	0.185	0.0749	2981.5
5	695.729	0.550	0.185	0.0749	2982.4
9	700.671	0.250	0.016	0.0814	2940.4
11	705.294	0.250	0.000	0.0000	2821.2
13	709.889	0.250	0.016	0.0814	2979.1
17	714.740	0.550	0.185	0.0749	3063.9
20	714.940	0.550	0.185	0.0749	3064.7

STRESS DUE TO LOAD AT CENTROID OF TENDONS

SEC.	Zp m3	Mdo kN·m	Md1 kN·m	M2p kN·m	σcpdo kN/m2	σcd1 kN/m2	σcp2p kN/m2
2	0.0749	-38.897	-44.703		-519.0	-596.5	
5	0.0749	-27.219	-12.469	-16.824	-363.2	-166.4	-224.5
9	0.0814	-1.340	-3.623	-16.928	-16.5	-44.5	-208.0
11	0.0000	2.992	-0.928	-16.998	0.0	0.0	0.0
13	0.0814	-1.182	-0.857	-17.068	-14.5	-10.5	-209.7
17	0.0749	-26.826	-5.594	-17.172	-357.9	-74.6	-229.1
20	0.0749	-38.459	-36.907		-513.2	-492.5	

No

COMPOSITE STRESS AT CENTROID OF TENDONS

$$\sigma cp1 = \sigma cpt + \sigma cpd0 + \sigma cp2p + 1/2(\eta - 1) \cdot \sigma cp2p \quad (\text{ASSUMED } \eta = 0.92)$$

SEC.	σcpt kN/m2	σcpdo kN/m2	σcp2p kN/m2	σcp1 kN/m2
2	2981.5	-519.0		2462.5
5	2982.4	-363.2	-224.5	2403.5
9	2940.4	-16.5	-208.0	2724.1
11	2821.2	0.0	0.0	2821.2
13	2979.1	-14.5	-209.7	2763.1
17	3063.9	-357.9	-229.1	2485.8
20	3064.7	-513.2		2551.6

PRESTRESSING LOSS DUE TO CREEP AND SHRINKAGE

$$\Delta \sigma p\phi = \frac{n \cdot \phi 1 \cdot \sigma cp1 + Ep \cdot \varepsilon s}{1 + n \cdot \sigma cpt \cdot (1+\rho 1 \cdot \phi 1) / \sigma pt} + \frac{n \cdot \phi 2 \cdot \sigma cd1}{1 + n \cdot \sigma cpt \cdot (1+\rho 2 \cdot \phi 2) / \sigma pt}$$
$$= \Delta \sigma p\phi 1 + \Delta \sigma p\phi 2$$

$\phi 1 = 2.20$ $\phi 2 = 1.00$ $n = 6.10$ $\rho 1 = 0.70$ $\rho 2 = 0.70$

$Ep = 0.2 \times 10^6$ Mpa、 $\varepsilon s = 25.0 \times 10\text{-}5$

SEC.	σpt Mpa	σcpt Mpa	σcp1 Mpa	σcd1 Mpa	Δσpφ1 Mpa	Δσpφ2 Mpa	Δσpφ Mpa
2	1253.65	2.98	2.46	-0.60	79.37	-3.55	75.82
5	1254.02	2.98	2.40	-0.17	78.61	-0.99	77.62
9	1262.93	2.94	2.72	-0.04	82.82	-0.27	82.56
11	1271.26	2.82	2.82	0.00	84.21	0.00	84.21
13	1279.54	2.98	2.76	-0.01	83.33	-0.06	83.26
17	1288.28	3.06	2.49	-0.07	79.67	-0.44	79.23
20	1288.64	3.06	2.55	-0.49	80.52	-2.93	77.59

PRESTRESSING LOSS DUE TO RELAXATION

$$\Delta \sigma p\gamma = \gamma \cdot \sigma pt \quad (\gamma = 1.50\ \%)$$

SEC.	σpt Mpa	Δσpγ Mpa
2	1253.65	18.80
5	1254.02	18.81
9	1262.93	18.94
11	1271.26	19.07
13	1279.54	19.19
17	1288.28	19.32
20	1288.64	19.33

3）PRE-STRESS AT EXTREMELY FIBER

$$\left.\begin{matrix}\sigma_{ptu}\\ \sigma_{ptl}\end{matrix}\right\} = \frac{P_t}{A} \pm \frac{P_t \cdot e_p}{Z}$$

$$\left.\begin{matrix}\sigma_{peu}\\ \sigma_{pel}\end{matrix}\right\} = \eta \times \left\{\begin{matrix}\sigma_{ptu}\\ \sigma_{ptl}\end{matrix}\right.$$

P_t = STRESSING FORCE IMMEDATELY AFTER ANCHOR SET(kN)

A = AREA (m^2)

Z = I / yc (m^3)

e_p = EXCENTRCITY (m)

η = EFFECTIVE RATIO

σ_{ptu}、σ_{ptl} = PRE-STRESS AT EXTREMELY FIBER AFTER ANCHOR SET (Mpa)

σ_{ptu}、σ_{ptl} = PRE-STRESS AT EXTREMELY FIBER IN SERVICE (Mpa)

SEC.		A m2	Z m3	Pt kN	e p m	η	ANCHOR SET Mpa	AT SERVICE Mpa
2	TOP.	0.550	0.0504	695.523	0.185	0.925	3.82	3.53
	BOT.						-1.29	-1.19
5	TOP.	0.550	0.0504	695.729	0.185	0.923	3.82	3.52
	BOT.						-1.29	-1.19
9	TOP.	0.250	0.0104	700.671	0.016	0.920	3.88	3.57
	BOT.						1.73	1.59
11	TOP.	0.250	0.0104	705.294	0.000	0.919	2.82	2.59
	BOT.						2.82	2.59
13	TOP.	0.250	0.0104	709.889	0.016	0.920	3.93	3.62
	BOT.						1.75	1.61
17	TOP.	0.550	0.0504	714.740	0.185	0.924	3.92	3.62
	BOT.						-1.32	-1.22
20	TOP.	0.550	0.0504	714.940	0.185	0.925	3.92	3.63
	BOT.						-1.32	-1.22

EFFECTIVE STRESS AT SERVICE

$\sigma_{pe} = \sigma_{pt} - \Delta\sigma_{p\phi} - \Delta\sigma_{p\gamma}$

$\eta = \sigma_{pe} / \sigma_{pt}$

$\bar{\eta}$ = 0.922

SEC.	σ pt Mpa	Δ σ p φ Mpa	Δ σ p γ Mpa	σ pe Mpa	η i
2	1253.65	75.82	18.80	1159.02	0.925
5	1254.02	77.62	18.81	1157.59	0.923
9	1262.93	82.56	18.94	1161.43	0.920
11	1271.26	84.21	19.07	1167.97	0.919
13	1279.54	83.26	19.19	1177.08	0.920
17	1288.28	79.23	19.32	1189.73	0.924
20	1288.64	77.59	19.33	1191.72	0.925

3.1.3.COMPOSITE STRESS

SEC.		STRESS DUE TO LOAD						PRE-STRESS		COMPOSITE STRESS					
		ANCHOR SET	PERMA NENT	SERVICE-1		SERVICE-3		ANCHOR SET	SERVICE	ANCHOR SET	PERMA NENT	SERVICE-1		SERVICE-3	
				MAX	MIN	MAX	MIN					MAX	MIN	MAX	MIN
2	TOP.	-0.77	-1.66	-1.57	-3.81	-1.66	-3.31	3.82	3.53	3.05	1.87	1.96	-0.28	1.87	0.22
	BOT.	0.77	1.66	1.57	3.81	1.66	3.31	-1.29	-1.19	-0.52	0.47	0.38	2.62	0.47	2.12
5	TOP.	-0.87	-1.09	-1.05	-2.72	-1.12	-2.34	3.82	3.52	2.94	2.43	2.48	0.80	2.41	1.18
	BOT.	0.87	1.09	1.05	2.72	1.12	2.34	-1.29	-1.19	-0.41	-0.09	-0.14	1.53	-0.07	1.15
9	TOP.	-1.75	-1.97	-1.94	-2.24	-2.08	-2.20	3.88	3.57	2.13	1.59	1.62	1.32	1.48	1.37
	BOT.	1.75	1.97	1.94	2.24	2.08	2.20	1.73	1.59	3.48	3.56	3.53	3.83	3.67	3.78
11	TOP.	-1.34	-1.31	4.80	-1.53	3.59	-1.53	2.82	2.59	1.48	1.29	7.40	1.07	6.18	1.07
	BOT.	1.34	1.31	-4.80	1.53	-3.59	1.53	2.82	2.59	4.17	3.90	-2.21	4.12	-1.00	4.12
13	TOP.	-1.75	-1.71	-1.68	-1.98	-1.82	-1.93	3.93	3.62	2.18	1.91	1.94	1.64	1.80	1.69
	BOT.	1.75	1.71	1.68	1.98	1.82	1.93	1.75	1.61	3.50	3.31	3.28	3.58	3.43	3.54
17	TOP.	-0.87	-0.96	-0.91	-2.58	-0.98	-2.21	3.92	3.62	3.05	2.67	2.71	1.04	2.64	1.42
	BOT.	0.87	0.96	0.91	2.58	0.98	2.21	-1.32	-1.22	-0.45	-0.27	-0.31	1.36	-0.24	0.98
20	TOP.	-0.76	-1.49	-1.40	-3.65	-1.49	-3.14	3.92	3.63	3.16	2.13	2.23	-0.02	2.13	0.49
	BOT.	0.76	1.49	1.40	3.65	1.49	3.14	-1.32	-1.22	-0.56	0.27	0.18	2.42	0.27	1.92

ALLOWABLE STRESS

TENSILE STRESS LIMIT $0.50 \cdot \sqrt{fc'} = 0.5x\sqrt{45.0} = -3.35$ Mpa

COMPRESSIVE STRESS LIMIT $0.40 \cdot fc' = 0.4x\ 45.0 = 18.0$ Mpa

No

No

CRACK CONTROL

$T = 1/2 \cdot \sigma ct \cdot b \cdot x$
$reqAs = T / \sigma sa$

- T : SECTIONAL TENSILE FORCE (kN)
- σct : TENSILE STRESS AT EXTREME FIBER
- b : UNIT WIDTH (m)
- reqAs : REQUIRED REINFORCEMENT
- σsa : TENSILE STRESS OF REINFORCEMENT FOR CRACK ALLOWABLE
- x : $|\sigma cu| / (|\sigma cu| + |\sigma cl|) \cdot h$
- h : MEMBER THICKNESS
- σcu、σcl : CONCRETE STRESS AT EXTREME FIBER

ANCHOR SET

SEC.	σcu Mpa	σcl Mpa	h m	x m	T kN	σsa Mpa	reqAs mm2	ARRENGED REBAR mm2
2	3.05	-0.52	0.550	0.0797	20.57	200.00	102.8	
5	2.94	-0.41	0.550	0.0679	14.06	200.00	70.3	
17	3.05	-0.45	0.550	0.0708	15.94	200.00	79.7	
20	3.16	-0.56	0.550	0.0829	23.23	200.00	116.2	

PERMANENT

SEC.	σcu Mpa	σcl Mpa	h m	x m	T kN	σsa Mpa	reqAs mm2	ARRENGED REBAR mm2
5	2.43	-0.09	0.550	0.0205	0.97	195.00	5.0	
17	2.67	-0.27	0.550	0.0497	6.59	195.00	33.8	

SERVICE-Ⅰ (LL-MAX)

SEC.	σcu Mpa	σcl Mpa	h m	x m	T kN	σsa Mpa	reqAs mm2	ARRENGED REBAR mm2
5	2.48	-0.14	0.550	0.0301	2.16	195.00	11.1	
11	7.40	-2.21	0.250	0.0576	63.70	195.00	326.7	
17	2.71	-0.31	0.550	0.0571	8.97	195.00	46.0	

SERVICE-Ⅰ (LL-MIN)

SEC.	σcu Mpa	σcl Mpa	h m	x m	T kN	σsa Mpa	reqAs mm2	ARRENGED REBAR mm2
2	-0.28	2.62	0.550	0.0533	7.50	195.00	38.4	
20	-0.02	2.42	0.550	0.0041	0.04	195.00	0.2	

No

PC-STEEL STRES

$\sigma pmax = \sigma pe + \Delta \sigma p$

$\Delta \sigma p = -n \cdot (\sigma cpd1 + \sigma cp2p + \sigma cpl)$

$\sigma pmax$	: PC-STEEL STRESS AT LL-LOADING
$\Delta \sigma$	: ADDITIONAL PC-STEEL STRESS DUE TO LOAD
n	: ELASTIC RATIO OF CONCRETE TO PC-STEEL
$\sigma cpd1$	: CONCRETE STRESS AT CENTROID OF PS-STEEL DUE TO SURFACE LOADING
$\sigma cpd1$	: CONCRETE STRESS FOR DIFFERENCE OF SECONDARY FORCE DUE TO PRE-STRESSING LOSS AT CENTROID OF PC-STEEL
σcpl	: CONCRETE STRESS AT CENTROID OF PC-STEEL DUE TO LL-LOADING
Zp	: I/y AT CENTROID OF PC-STEEL

$\sigma cpi = Mi / Zp$

SEC.	Md1 kN·m	ΔM2p kN·m	Ml kN·m	Zp m3	σ cpd1 Mpa	σ cp2p Mpa	σ cpl Mpa	Σ σ cp Mpa	n	Δ σ p Mpa	σ pe Mpa	σ pmax Mpa
5	-12.469	-1.322	-77.270	0.0749	-0.17	-0.02	-1.03	-1.22	6.10	7.41	1157.59	1165.00
11	-0.928	-1.335	63.320	0.0000	—	—	—	—	6.10	—	1167.97	—
17	-5.594	-1.349	-77.270	0.0749	-0.07	-0.02	-1.03	-1.12	6.10	6.85	1189.73	1196.58

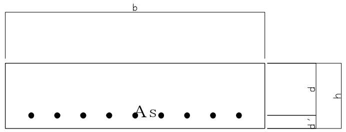
No

SERVICE-Ⅲ (MAZ)

SEC.	σ cu Mpa	σ cl Mpa	h m	x m	T kN	σ sa Mpa	reqAs mm2	ARRENGED REBAR mm2
5	2.41	-0.07	0.550	0.0158	0.57	195.00	2.9	
11	6.18	-1.00	0.250	0.0347	17.32	195.00	88.8	
17	2.64	-0.24	0.550	0.0462	5.59	195.00	28.7	

3. 2. 2. STRESS CHECK

CRACK CONTROL COMPUTED AS RC SECTION



REBAR STRESS

$$\sigma s = \frac{M}{As \cdot j \cdot d}$$

CONCRETE STRESS

$$\sigma c = \frac{\sigma s}{m}$$

ELASTIC RATIO n = 12.00

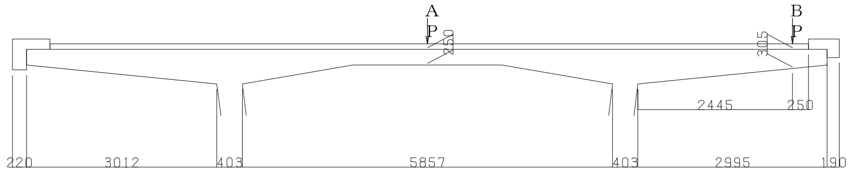
SECTION			A － A	B － B
MOMENT	M	kN·m	31.52	33.06
DIMENSION	B	mm	1000.0	1000.0
	d'	mm	65.0	62.0
	d	mm	185.0	243.1
REBARS	Asreq	mm2	808.0	628.0
	As	mm2	D16ctc125	D14ctc125
			1608.0	1232.0
CIE	100·p		0.009	0.005
	k		0.364	0.293
	j		0.879	0.902
	m		20.950	28.927
	σs	Mpa	120.60	122.35
ALLOWABLE CRACK	dc	mm	50.000	50.000
	ctc	mm	125.000	125.000
	A	mm2	12500.0	12500.0
	Z	N/mm	23000.0	23000.0
	Z/(dc·A)**1/3	Mpa	269.0	269.0
	0.6·fsy	Mpa	240.0	240.0
	fsa	Mpa	240.0	240.0
	JUDGE	-	OK	OK

fsa = Z / (dc·A)**1/3 ≦ 0.6·fsy

fsa : LIMIT STRESS FOR REBAR
dc : DEPTH OF CONCRETE MEASURED FROM TENTION FIBER (LESS THAN 50mm)
A : AREA OF CONCRETE FOR A TENSILE REBAR(mm2)
fsy : YIELD STRESS OF REBAR (400.0 Mpa)
Z : CRACK WIDTH PARAMETER (N/mm)

Asreq = M / (fsa·j·d)

3. 2. DESIGN FOR LONGITUDINAL DIRECTION



CALCULATION AS RC COMPONENT IS PERFORMED FOR THE SECTION SHOW ABOVE.

3. 2. 1. SECTIONAL FORCE

LIVE LOAD IS CONSIDERED AND PERMANENT LOAD IS DISREGARDED.

SECTION A (CENTER OF UPPER SLAB)

SPAN LENGTH l =5.857 m COEFFICIENT K =1.000

MA = = 31.520 kN·m

SECTION B (LOADING POINT OF AXLE FORCE)

SPAN LENGTH l =2.445 m COEFFICIENT K =1.000

MB = 33.060 kN·m

No

3.2.3. CHECK FOR MINIMUM REINFORCEMENT

REINFORCEMENT FOR SHRINKAGE AND TEMPERATURE

(1) UPPER SLAB OF BOX (LONGITUDINAL)

$Asreq① = 0.75 \cdot Ag / fy / L$

$= 0.75 \times 1989196.7 / 400.0 / 5.8568$

$= 636.8$ mm2/m

$Asreq② = 0.0015 \cdot Ag / L$

$= 0.0015 \times 1989196.7 / 5.8568$

$= 509.5$ mm2/m

(2) OVERHANG(L) OF BOX (LONGITUDINAL)

$Asreq① = 0.75 \cdot Ag / fy / L$

$= 0.75 \times 1204800.0 / 400.0 / 3.0120$

$= 750.0$ mm2/m

$Asreq② = 0.0015 \cdot Ag / L$

$= 0.0015 \times 1204800.0 / 3.0120$

$= 600.0$ mm2/m

(3) OVERHANG(R) OF BOX (LONGITUDINAL)

$Asreq① = 0.75 \cdot Ag / fy / L$

$= 0.75 \times 1198000.0 / 400.0 / 2.9950$

$= 750.0$ mm2/m

$Asreq② = 0.0015 \cdot Ag / L$

$= 0.0015 \times 1198000.0 / 2.9950$

$= 600.0$ mm2/m

(4) BOTTOM SLAB (TRANSVERSE)

$Asreq = 0.005 \cdot Ag / L$

$= 0.005 \times 773779.8 / 6.1760$

$= 626.4$ mm2/m

No

(5) WEB (LONGITUDINAL)

$Asreq① = 0.75 \cdot Ag / fy / L$

$= 0.75 \times 685281.1 / 400.0 / 1.7000$

$= 755.8$ mm2/m

$Asreq② = 0.0015 \cdot Ag / L$

$= 0.0015 \times 685281.1 / 1.7000$

$= 604.7$ mm2/m

(6) BOTTOM SLAB (TRANSVERSE)

$Asreq = 0.005 \cdot Ag / L$

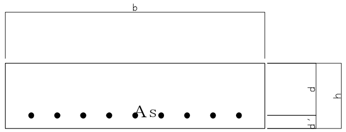
$= 0.005 \times 773779.8 / 6.1760$

$= 626.4$ mm2/m

No

BOTTOM SLAB (SERVICE)

CRACK CONTROL COMPUTED AS RC SECTION



REBAR STRESS

$$\sigma s = \frac{M}{As \cdot j \cdot d}$$

CONCRETE STRESS

$$\sigma c = \frac{\sigma s}{m}$$

ELASTIC RATIO n = 12.00

SECTION			27 MAX.	23 MIN.
MOMENT	M	kN·m	12.91	-25.34
DIMENSION	B	mm	1000.0	1000.0
	d'	mm	50.0	45.0
	d	mm	200.0	205.0
REBARS	Asreq	mm2	304.9	583.0
	As	mm2	D16ctc125	D16ctc125
			1608.0	1608.0
CIE	100·p		0.008	0.008
	k		0.353	0.350
	j		0.882	0.883
	m		21.969	22.301
	σ s	Mpa	45.51	87.02
ALLOWABLE CRACK	d c	mm	50.000	45.000
	ctc	mm	125.000	125.000
	A	mm2	12500.0	11250.0
	Z	N/mm	23000.0	23000.0
	Z/(dc·A)**1/3	Mpa	269.0	288.6
	0.6·fsy	Mpa	240.0	240.0
	fsa	Mpa	240.0	240.0
	JUDGE	-	OK	OK

fsa = Z / (dc·A)**1/3 ≦ 0.6·fsy

fsa : LIMIT STRESS FOR REBAR
dc : DEPTH OF CONCRETE MEASURED FROM TENTION FIBER (LESS THAN 50mm)
A : AREA OF CONCRETE FOR A TENSILE REBAR(mm2)
fsy : YIELD STRESS OF REBAR (400.0 Mpa)
Z : CRACK WIDTH PARAMETER (N/mm)

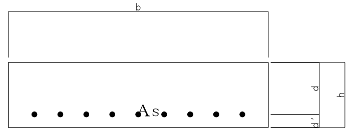
Asreq = M / (fsa·j·d)

No

4. STRESS CHECK FOR BOTTOM SLAB AND WEB

4.1 BOTTOM SLAB

CRACK CONTROL COMPUTED AS RC SECTION



REBAR STRESS

$$\sigma s = \frac{M}{As \cdot j \cdot d}$$

CONCRETE STRESS

$$\sigma c = \frac{\sigma s}{m}$$

ELASTIC RATIO n = 12.00

SECTION			27 MAX.	23 MIN.
MOMENT	M	kN·m	9.82	-14.05
DIMENSION	B	mm	1000.0	1000.0
	d'	mm	50.0	45.0
	d	mm	200.0	205.0
REBARS	Asreq	mm2	231.9	323.2
	As	mm2	D16ctc125	D16ctc125
			1608.0	1608.0
CIE	100·p		0.008	0.008
	k		0.353	0.350
	j		0.882	0.883
	m		21.969	22.301
	σ s	Mpa	34.60	48.24
ALLOWABLE CRACK	d c	mm	50.000	45.000
	ctc	mm	125.000	125.000
	A	mm2	12500.0	11250.0
	Z	N/mm	23000.0	23000.0
	Z/(dc·A)**1/3	Mpa	269.0	288.6
	0.6·fsy	Mpa	240.0	240.0
	fsa	Mpa	240.0	240.0
	JUDGE	-	OK	OK

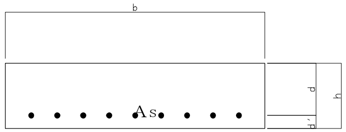
fsa = Z / (dc·A)**1/3 ≦ 0.6·fsy

fsa : LIMIT STRESS FOR REBAR
dc : DEPTH OF CONCRETE MEASURED FROM TENTION FIBER (LESS THAN 50mm)
A : AREA OF CONCRETE FOR A TENSILE REBAR(mm2)
fsy : YIELD STRESS OF REBAR (400.0 Mpa)
Z : CRACK WIDTH PARAMETER (N/mm)

Asreq = M / (fsa·j·d)

WEB (SERVICE)

CRACK CONTROL COMPUTED AS RC SECTION



REBAR STRESS

$$\sigma s = \frac{M}{As \cdot j \cdot d}$$

CONCRETE STRESS

$$\sigma c = \frac{\sigma s}{m}$$

ELASTIC RATIO n = 12.00

SECTION			40 MAX.	40 MIN.
MOMENT	M	kN·m	36.06	-103.00
DIMENSION	B	mm	1000.0	1000.0
	d’	mm	50.0	45.0
	d	mm	350.0	355.0
REBARS	Asreq	mm2	473.7	1333.2
	As	mm2	D16ctc125	D16ctc125
			1608.0	1608.0
CIE	100·p		0.005	0.005
	k		0.281	0.280
	j		0.906	0.907
	m		30.633	30.887
	σs	Mpa	70.70	198.99
ALLOWABLE CRACK	dc	mm	50.000	45.000
	ctc	mm	125.000	125.000
	A	mm2	12500.0	11250.0
	Z	N/mm	23000.0	23000.0
	Z/(dc·A)**1/3	Mpa	269.0	288.6
	0.6·fsy	Mpa	240.0	240.0
	fsa	Mpa	240.0	240.0
	JUDGE	-	OK	OK

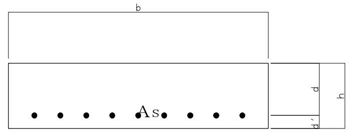
fsa = Z / (dc·A)**1/3 ≦ 0.6·fsy

fsa : LIMIT STRESS FOR REBAR
dc : DEPTH OF CONCRETE MEASURED FROM TENTION FIBER (LESS THAN 50mm)
A : AREA OF CONCRETE FOR A TENSILE REBAR(mm2)
fsy : YIELD STRESS OF REBAR (400.0 Mpa)
Z : CRACK WIDTH PARAMETER (N/mm)

Asreq = M / (fsa·j·d)

4.2 WEB

CRACK CONTROL COMPUTED AS RC SECTION



REBAR STRESS

$$\sigma s = \frac{M}{As \cdot j \cdot d}$$

CONCRETE STRESS

$$\sigma c = \frac{\sigma s}{m}$$

ELASTIC RATIO n = 12.00

SECTION			44 MAX.	34 MIN.
MOMENT	M	kN·m	-15.37	-25.08
DIMENSION	B	mm	1000.0	1000.0
	d’	mm	45.0	45.0
	d	mm	355.0	355.0
REBARS	Asreq	mm2	198.9	324.7
	As	mm2	D16ctc125	D16ctc125
			1608.0	1608.0
CIE	100·p		0.005	0.005
	k		0.280	0.280
	j		0.907	0.907
	m		30.887	30.887
	σs	Mpa	29.69	48.46
ALLOWABLE CRACK	dc	mm	45.000	45.000
	ctc	mm	125.000	125.000
	A	mm2	11250.0	11250.0
	Z	N/mm	23000.0	23000.0
	Z/(dc·A)**1/3	Mpa	288.6	288.6
	0.6·fsy	Mpa	240.0	240.0
	fsa	Mpa	240.0	240.0
	JUDGE	-	OK	OK

fsa = Z / (dc·A)**1/3 ≦ 0.6·fsy

fsa : LIMIT STRESS FOR REBAR
dc : DEPTH OF CONCRETE MEASURED FROM TENTION FIBER (LESS THAN 50mm)
A : AREA OF CONCRETE FOR A TENSILE REBAR(mm2)
fsy : YIELD STRESS OF REBAR (400.0 Mpa)
Z : CRACK WIDTH PARAMETER (N/mm)

Asreq = M / (fsa·j·d)

5. STRENGTH CHECK

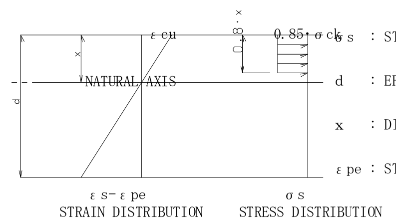
Ⅰ) BENDING MOMENT AT STRENGTH LIMIT STATE

LOAD COMBINATIONS

			1	2	3	4	5	6	7	8	9	10	11				
			DC	DW	EL	CR	SH	LL+IM	PL	TG	WS	WL	CT				
S		ST1-1	1.25	1.50	1.00	0.50		1.75	1.75					Lmax	-		1
T	Strength-1	ST1-2	1.25	1.50	1.00	0.50		1.75	1.75					Lmin	-		2
R		ST1-3	0.90	0.65	1.00	0.50		1.75	1.75					Lmax	-		3
E		ST1-4	0.90	0.65	1.00	0.50		1.75	1.75					Lmin	-		4
N		ST2-1	1.25	1.50	1.00	0.50		1.35	1.35					Lmax	-		5
G	Strength-2	ST2-2	1.25	1.50	1.00	0.50		1.35	1.35					Lmin	-		6
T		ST2-3	0.90	0.65	1.00	0.50		1.35	1.35					Lmax	-		7
H		ST2-4	0.90	0.65	1.00	0.50		1.35	1.35					Lmin	-		8
		ST3-1	1.25	1.50	1.00	0.50					1.40			Wind	R		9
L	Strength-3	ST3-2	1.25	1.50	1.00	0.50					-1.40			Wind	L		10
I		ST3-3	0.90	0.65	1.00	0.50					1.40			Wind	R		11
M		ST3-4	0.90	0.65	1.00	0.50					-1.40			Wind	L		12
I	Strength-4	ST4-1	1.50	1.50	1.00	0.50								D	-		13
T		ST5-1	1.25	1.50	1.00	0.50		1.35	1.35		0.40	1.00		Lx+W	R		14
		ST5-2	1.25	1.50	1.00	0.50		1.35	1.35		0.40	1.00		Ln+W	R		15
S		ST5-3	1.25	1.50	1.00	0.50		1.35	1.35		-0.40	-1.00		Lx+W	L		16
T	Strength-5	ST5-4	1.25	1.50	1.00	0.50		1.35	1.35		-0.40	-1.00		Ln+W	L		17
A		ST5-5	0.90	0.65	1.00	0.50		1.35	1.35		0.40	1.00		Lx+W	R		18
T		ST5-6	0.90	0.65	1.00	0.50		1.35	1.35		0.40	1.00		Ln+W	R		19
E		ST5-7	0.90	0.65	1.00	0.50		1.35	1.35		-0.40	-1.00		Lx+W	L		20
		ST5-8	0.90	0.65	1.00	0.50		1.35	1.35		-0.40	-1.00		Ln+W	L		21

Ⅱ) STRENGTH SECTIO

DISTRIBUTION OF STRAIN AND STRESS ON CALCULATION OF STRENGTH SECTION



ε cu : ULTIMATE STRAIN OF CONCRETE (=0.0030)

ε s : STRAIN OF REINFORCEMENT

σ ck : CONCRETE STRENGTH (Mpa)

σ s : STRESS OF REINFORCEMENT (Mpa)

d : EFFECTIVE HEIGHT OF SECTION (mm)

x : DISTANCE FROM COMPRESSIVE EXTREME FIBER TO NATURAL AXIS (mm)N

ε pe : STRAIN OF TENDON DUE TO FEECTIVE PRESTRESS

FORCE BALANCING

$$Cu = Tu$$

$$Cu = 0.8 \cdot x \cdot b \cdot 0.85 \cdot \sigma_{ck} = 0.680 \cdot x \cdot \sigma_{ck}$$

b : WIDTH OF COMPRESSION FLANGE

COMPATIBILITY CONDITION OF STAIN

$$\frac{x}{\varepsilon_{cu}} = \frac{d - x}{\varepsilon_s - \varepsilon_{pe}}$$

RESISTANCE OF SECTION

$$Mr = Tu \cdot (d - 0.4 \cdot x)$$

$$Tu = As \cdot \sigma_s$$

a) REINFORCEMENT BAR

$Es \cdot \varepsilon_s < \sigma_{sy}$ ·············· $\sigma_s = Es \cdot \varepsilon_s$

$Es \cdot \varepsilon_s \geqq \sigma_{sy}$ ·············· $\sigma_s = \sigma_{sy}$

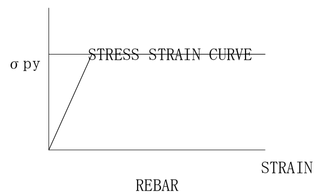
b) PC CABLE

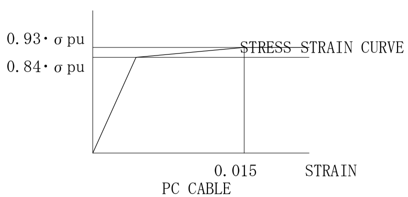
$Eps \cdot \varepsilon_s < 0.84 \cdot \sigma_{pu}$ ······· $\sigma_s = Eps \cdot \varepsilon_s$

$0.84 \cdot \sigma_{pu} \leqq Eps \cdot \varepsilon_s < 0.93 \cdot \sigma_{pu}$ ·····

$$\sigma_s = 0.84 \cdot \sigma_{pu} + \frac{\varepsilon_s - \frac{0.84 \cdot \sigma_{pu}}{Ep}}{0.015 - \frac{0.84 \cdot \sigma_p}{Eps}} \cdot 0.09 \cdot \sigma_{pu}$$

$\varepsilon_s \geqq 0.015$ ·················· $\sigma_s = 0.93 \cdot \sigma_{pu}$





Mr : NOMINAL RESISTANCE OF SECTION

σ sy : YIELD STRENGTH OF REBAR (σ sy = 400 M

σ pu : YIELD STRENGTH OF PC CABLE (σ pu = 1860 Mpa)

σ s : STRESS OF REBAR (Mpa)

Es : MODULUS OF ELASTICITY OF REINFORCEMENT (Mpa)

ε s : STRAIN OF REINFORCEMENT

As : ALL AREA OF TENSILE(mm^2)FORCEMENT (mm2)

d : EFFECTIVE HEIGHT OF SECTION (mm)

Ⅲ) SEFETY DEGREE FOR BENDING MOMENT STRENGTH

$$Fs = \frac{Mr}{Mu}$$

5．1 STRENGTH CHECK FOR TOP SLAB

5．1．1 MEMBER REINFORCED STRESSING STEEL

			2	5	9	11	13	17	20
GIRDER-SELF LOAD			-38.90	-27.22	-1.34	2.99	-1.18	-26.83	-38.46
SUPERIMPOSED			-44.70	-12.47	-3.62	-0.93	-0.86	-5.59	-36.91
SECONDARY MOMENT	CONST.			-16.82	-16.93	-17.00	-17.07	-17.17	
	SERVICE			-15.50	-15.60	-15.66	-15.73	-15.82	
CREEP				1.32	1.33	1.34	1.34	1.35	
LIVE LOAD(LL+IM)	MAX.					63.32			
	MIN.		-103.86	-77.27				-77.27	-103.86
WS	→	1.00·WS	5.36	4.21	1.70	0.00	-1.70	-4.21	-5.36
	←	1.00·WS	-5.36	-4.21	-1.70	0.00	1.70	4.21	5.36
WL	→	1.00·WL	3.01	2.37	0.95	0.00	-0.95	-2.37	-3.01
	←	1.00·WL	-3.01	-2.37	-0.95	0.00	0.95	2.37	3.01
STRENGTH LIMIT STATE	Strength-1	ST1- 1	-115.68	-68.89	-23.37	96.83	-19.16	-58.42	-103.43
		ST1- 2	-297.43	-204.11	-23.37	-13.98	-19.16	-193.64	-285.19
		ST1- 3	-64.06	-48.76	-19.82	96.57	-18.02	-44.28	-58.60
		ST1- 4	-245.82	-183.99	-19.82	-14.24	-18.02	-179.50	-240.36
		MAX.	-64.06	-48.76	-19.82	96.83	-18.02	-44.28	-58.60
		MIN.	-297.43	-204.11	-23.37	-14.24	-19.16	-193.64	-285.19
	Strength-2	ST2- 1	-115.68	-68.89	-23.37	71.50	-19.16	-58.42	-103.43
		ST2- 2	-255.89	-173.20	-23.37	-13.98	-19.16	-162.73	-243.65
		ST2- 3	-64.06	-48.76	-19.82	71.24	-18.02	-44.28	-58.60
		ST2- 4	-204.28	-153.08	-19.82	-14.24	-18.02	-148.59	-198.81
		MAX.	-64.06	-48.76	-19.82	71.50	-18.02	-44.28	-58.60
		MIN.	-255.89	-173.20	-23.37	-14.24	-19.16	-162.73	-243.65
	Strength-3	ST3- 1	-108.17	-62.99	-21.00	-13.98	-21.53	-64.32	-110.94
		ST3- 2	-123.18	-74.79	-25.75	-13.98	-16.79	-52.52	-95.93
		ST3- 3	-56.56	-42.87	-17.45	-14.24	-20.39	-50.17	-66.11
		ST3- 4	-71.57	-54.66	-22.20	-14.24	-15.64	-38.38	-51.10
		MAX.	-56.56	-42.87	-17.45	-13.98	-15.64	-38.38	-51.10
		MIN.	-123.18	-74.79	-25.75	-14.24	-21.53	-64.32	-110.94
	Strength-4	ST4- 1	-125.40	-75.69	-23.71	-13.23	-19.45	-65.13	-113.05
	Strength-5	ST5- 1	-110.52	-64.84	-21.74	71.50	-20.79	-62.47	-108.59
		ST5- 2	-250.73	-169.15	-21.74	-13.98	-20.79	-166.79	-248.80
		ST5- 3	-120.83	-72.94	-25.00	71.50	-17.53	-54.37	-98.28
		ST5- 4	-261.04	-177.26	-25.00	-13.98	-17.53	-158.68	-238.49
		ST5- 5	-58.91	-44.71	-18.19	71.24	-19.65	-48.33	-63.76
		ST5- 6	-199.12	-149.03	-18.19	-14.24	-19.65	-152.64	-203.97
		ST5- 7	-69.22	-52.82	-21.45	71.24	-16.39	-40.22	-53.45
		ST5- 8	-209.43	-157.13	-21.45	-14.24	-16.39	-144.54	-193.66
		MAX.	-58.91	-44.71	-18.19	71.50	-16.39	-40.22	-53.45
		MIN.	-261.04	-177.26	-25.00	-14.24	-20.79	-166.79	-248.80

STRENGTH RESISTANCE AND RATIO

$$Mn = Aps \cdot fps \cdot (dp-a/2) + As \cdot fy \cdot (ds-a/2) - As' \cdot fy' \cdot (ds'-a/2)$$

Aps = ctc 750.0 = 554.8 mm2
As = D14 ctc 125.0 = 1232.0 mm2
As' = D14 ctc 125.0 = 1232.0 mm2

$$fps = fpu \cdot (1-k \cdot c/dp)$$

$$k = 2 \cdot (1.04-fpy/fpu)$$

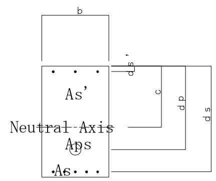
$$c = \frac{Aps \cdot fpu + As \cdot fy - As' \cdot fy}{0.85 \cdot fc' \cdot \beta 1 \cdot b + k \cdot Aps \cdot fpu/dp}$$

$$a = \beta 1 \cdot c$$

$$\beta 1 = 0.85 - (0.05 \cdot (fc'-28)/7)$$

$$\phi = 0.9 + 0.10 \cdot \frac{Aps \cdot fpy}{Aps \cdot fpy + As \cdot fy}$$

fpu = 1860.0 Mpa
fpy = 1674.0 Mpa
fy = 400.0 Mpa
fc' = 45.0 Mpa



			2	5	11	17	20
EFFECTIVE HEIGHT	dp	mm	460.0	460.0	125.0	460.0	460.0
	ds	mm	485.0	485.0	185.0	485.0	485.0
	ds'	mm	65.0	65.0	65.0	65.0	65.0
PC-STEEL STRESS	β1		0.7286	0.7286	0.7286	0.7286	0.7286
	k		0.2800	0.2800	0.2800	0.2800	0.2800
	a	mm	26.384	26.384	24.912	26.384	26.384
	fps	Mpa	1819.0	1819.0	1717.5	1819.0	1819.0
STRENGTH RESISTANCE	c	mm	36.2	36.2	34.2	36.2	36.2
MOMENT AT STRENGTH LIMIT STATE	Mu	kN·m	-297.4	-204.1	96.8	-193.6	-285.2
NOMINAL FLEXURAL STRENGTH	Mn	kN·m	657.9	657.9	166.4	657.9	657.9
RESISTANCE FACTER	φ		0.965	0.965	0.965	0.965	0.965
	φ·Mn	kN·m	635.1	635.1	160.6	635.1	635.1
RESISTANCE	Fs		2.135	3.111	1.659	3.280	2.227

No

5．2 STRENGTH CHECK FOR BOTTOM SLAB AND WEB

COLLECTION TABLE (WEB kN·m)

			23	27	31	34	38	40	44
CASE 4			-9.31	1.43	12.18	-47.46	-12.61	-30.85	9.92
CASE 5			-1.72	12.91	1.67	34.70	-5.57	36.06	-3.14
CASE 6			12.18	1.43	-9.31	-30.85	9.92	-47.46	-12.61
CASE 7			-27.25	7.92	-24.90	-104.35	-32.11	-103.00	-29.67
CASE 8			-2.13	-2.13	-2.13	58.26	2.00	58.26	2.00
CASE 9			-5.11	12.34	-2.76	22.31	-8.96	23.67	-6.52
GIRDER-SELF LOAD			-14.88	7.87	-14.77	-10.94	-17.53	-10.88	-17.42
SUPERIMPOSED			-0.02	0.95	1.92	-26.87	-1.92	-25.91	0.08
SECONDARY MOMENT	CONST.		0.92	1.08	1.25	13.81	1.80	14.18	2.14
	SERVICE		0.85	1.00	1.15	12.72	1.65	13.07	1.97
CREEP			-0.07	-0.09	-0.10	-1.08	-0.14	-1.11	-0.17
LIVE LOAD(LL+IM)	MAX.		12.18	2.87	12.18	58.26	11.92	58.26	11.92
	MIN.		-11.44	-2.13	-11.44	-78.31	-12.61	-78.31	-12.61
WS	→ 0.30·WS		0.33	0.00	-0.33	0.26	0.35	-0.26	-0.35
	← 0.30·WS		-0.33	0.00	0.33	-0.26	-0.35	0.26	0.35
WL	→ 1.00·WL		0.62	0.00	-0.62	0.48	0.65	-0.48	-0.65
	← 1.00·WL		-0.62	0.00	0.62	-0.48	-0.65	0.48	0.65
STRENGTH LIMIT STATE	Strength-1	ST1- 1	3.57	17.32	6.93	61.25	-2.21	63.11	1.26
		ST1- 2	-37.77	8.57	-34.40	-177.74	-45.14	-175.88	-41.66
		ST1- 3	8.79	13.76	10.47	87.91	5.56	88.94	7.29
		ST1- 4	-32.54	5.01	-30.87	-151.08	-37.37	-150.05	-35.63
		MAX.	8.79	17.32	10.47	87.91	5.56	88.94	7.29
		MIN.	-37.77	5.01	-34.40	-177.74	-45.14	-175.88	-41.66
	Strength-2	ST2- 1	-1.30	16.17	2.06	37.94	-6.98	39.81	-3.51
		ST2- 2	-33.19	9.43	-29.83	-146.42	-40.10	-144.56	-36.62
		ST2- 3	3.92	12.61	5.60	64.61	0.79	65.64	2.52
		ST2- 4	-27.97	5.86	-26.29	-119.76	-32.33	-118.73	-30.59
		MAX.	3.92	16.17	5.60	64.61	0.79	65.64	2.52
		MIN.	-33.19	5.86	-29.83	-146.42	-40.10	-144.56	-36.62
	Strength-3	ST3- 1	-16.19	12.30	-15.93	-39.51	-21.44	-40.04	-21.22
		ST3- 2	-19.29	12.30	-12.83	-41.91	-24.70	-37.64	-17.97
		ST3- 3	-10.96	8.74	-12.40	-12.84	-13.68	-14.21	-15.19
		ST3- 4	-14.07	8.74	-9.29	-15.24	-16.93	-11.81	-11.94
		MAX.	-10.96	12.30	-9.29	-12.84	-13.68	-11.81	-11.94
		MIN.	-19.29	8.74	-15.93	-41.91	-24.70	-40.04	-21.22
	Strength-4	ST4- 1	-21.46	14.27	-18.07	-43.44	-27.46	-41.56	-23.95
	Strength-5	ST5- 1	-0.24	16.17	0.99	38.77	-5.86	38.98	-4.62
		ST5- 2	-32.12	9.43	-30.89	-145.60	-38.98	-145.38	-37.74
		ST5- 3	-2.37	16.17	3.13	37.12	-8.10	40.63	-2.39
		ST5- 4	-34.26	9.43	-28.76	-147.25	-41.21	-143.73	-35.50
		ST5- 5	4.99	12.61	4.53	65.43	1.91	64.81	1.41
		ST5- 6	-26.90	5.86	-27.36	-118.93	-31.21	-119.55	-31.71
		ST5- 7	2.85	12.61	6.66	63.78	-0.33	66.46	3.64
		ST5- 8	-29.03	5.86	-25.23	-120.58	-33.44	-117.90	-29.47
		MAX.	4.99	16.17	6.66	65.43	1.91	66.46	3.64
		MIN.	-34.26	5.86	-30.89	-147.25	-41.21	-145.38	-37.74

No

5．1．2 DESIGN FOR LONGITUDINAL (RC COMPONENT)

BENDING MOMENT AT STRENGTH LIMIT STATE

A － A SECTION (BOX CENTER OF SPAN)

M1 = 31.520 (kN·m)

Mu = 1.75×31.520 = 78.800 (kN·m)

B － B SECTION (NEAR TIP OF OVERHANG)

M1 = 33.060 (kN·m)

Mu = 1.75×33.060 = 82.650 (kN·m)

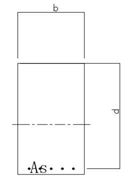
STRENGTH RESISTANCE AND RATIO

Mn = As·fy·(d-a/2)

a = As·fy/(0.85·fc'·b)

fy = 400.0 Mpa

fc' = 45.0 Mpa



Asreq = 1/fy·(X-√(X^2-2·(X/d)·(Mu/φ)))

X = 0.85·fc'·b·d

		A-A	B-B
Asreq	mm2	1225.6	964.4
As	mm2	D16ctc125	D14ctc125
		1608.0	1232.0
d	mm	185.0	243.1
Mn	kN·m	113.58	116.62
φ		0.900	0.900
φ·Mn	kN·m	102.23	104.96
Mu	〃	78.80	82.65
Fs		1.297	1.270

No

MINIMUM REINFORCEMENT OF FLAXURAL COMPONENT (UPPER SLAB)

Mr AT LEAST EQUAL TO THE LESSER OF ;
1.2 TIMES THE CRACKING STRENGTH (1.2Mcr)
1.33 TIMES THE FACTORED MOMENT (1.33Mu)

$Mn \geqq 1.20 \cdot Mcr$
OR
$Mn \geqq 1.33 \cdot Mu$

$Mcr = Sc \cdot (fr + fcpe) - Mdnc \cdot (Sc/Snc - 1)$

Mn : NOMINAL BENDING STRENGTH (Mn/φ)
Mcr : CRACKING MOMENT
Mu : MOMENT AT STRENGTH LIMIT STATE
fr : MODULUS OF RUPTURE
$fr = 0.63 \cdot \sqrt{(fc')}$
fc' : CONCRETE STRENGTH
Sc : SECTION COEFFICIENT
Sc/Snc : NON COMPOSIT >>> 1

$$fcpe = \frac{Np \cdot Pe}{Ac} + \frac{Np \cdot ep \cdot Pe}{Sc}$$

		2	5	11	17	20
b	m	1.000	1.000	1.000	1.000	1.000
d	m	0.550	0.550	0.250	0.550	0.550
Sc	m3	0.050	0.050	0.010	0.050	0.050
fr	Mpa	4.23	4.23	4.23	4.23	4.23
fcpe	Mpa	-1.19	-1.19	2.59	-1.22	-1.22
Mcr	kN·m	153.1	153.1	71.0	151.5	151.4
1.20·Mcr	kN·m	183.7	183.8	85.2	181.8	181.6
Mu	kN·m	-297.4	-204.1	96.8	-193.6	-285.2
1.33·Mu	kN·m	-395.6	-271.5	128.8	-257.5	-379.3
Mn	kN·m	657.9	657.9	166.4	657.9	657.9
Fs		3.582	3.580	1.952	3.620	3.622

No

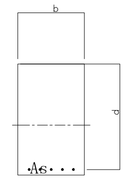
STRENGTH RESISTANCE AND RATIO

$Mn = As \cdot fy \cdot (d - a/2)$

$a = As \cdot fy / (0.85 \cdot fc' \cdot b)$

fy = 400.0 Mpa

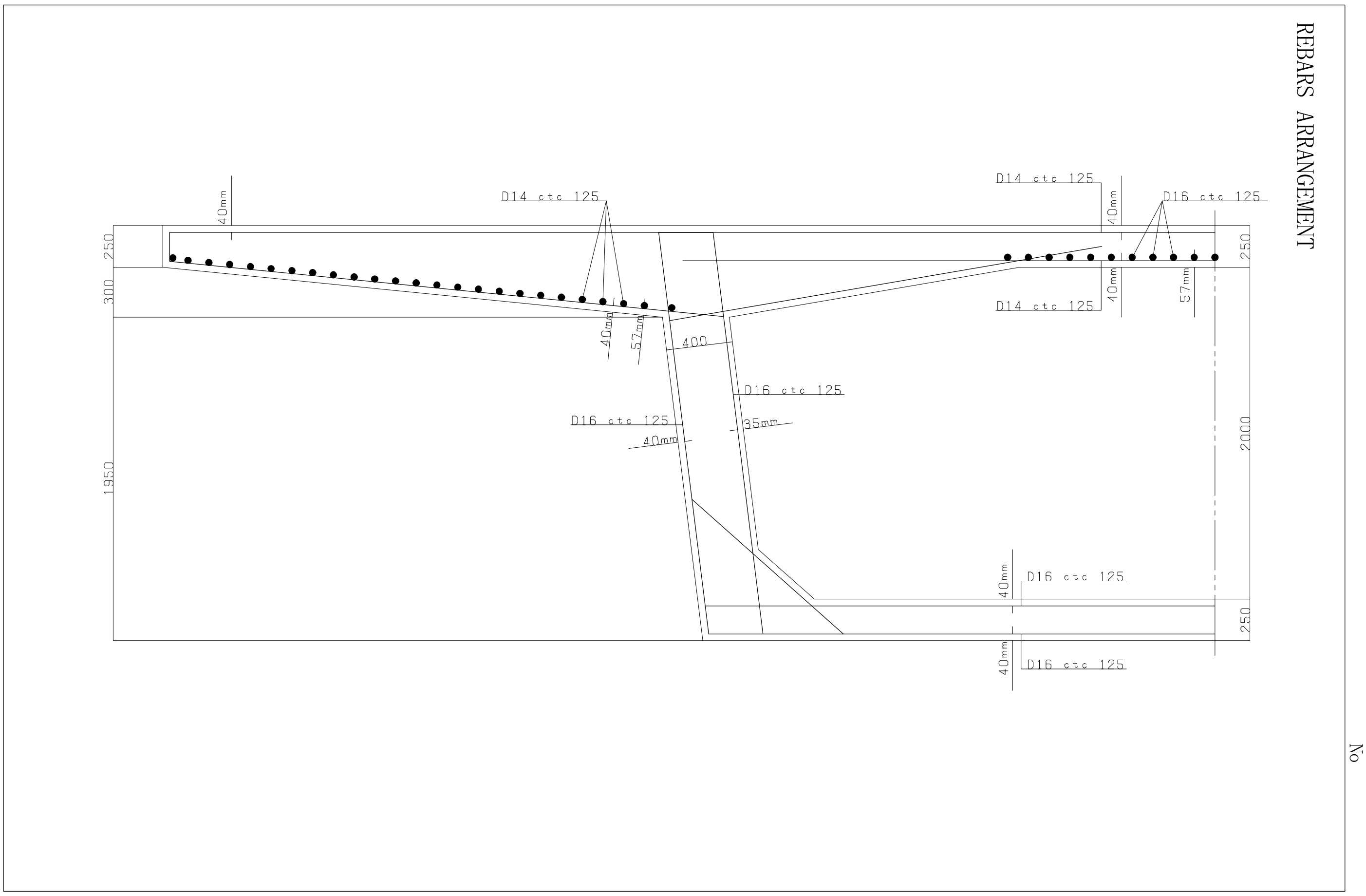
fc' = 45.0 Mpa



$Asreq = 1/fy \cdot (X - \sqrt{(X^2 - 2 \cdot (X/d) \cdot (Mu/\phi))})$

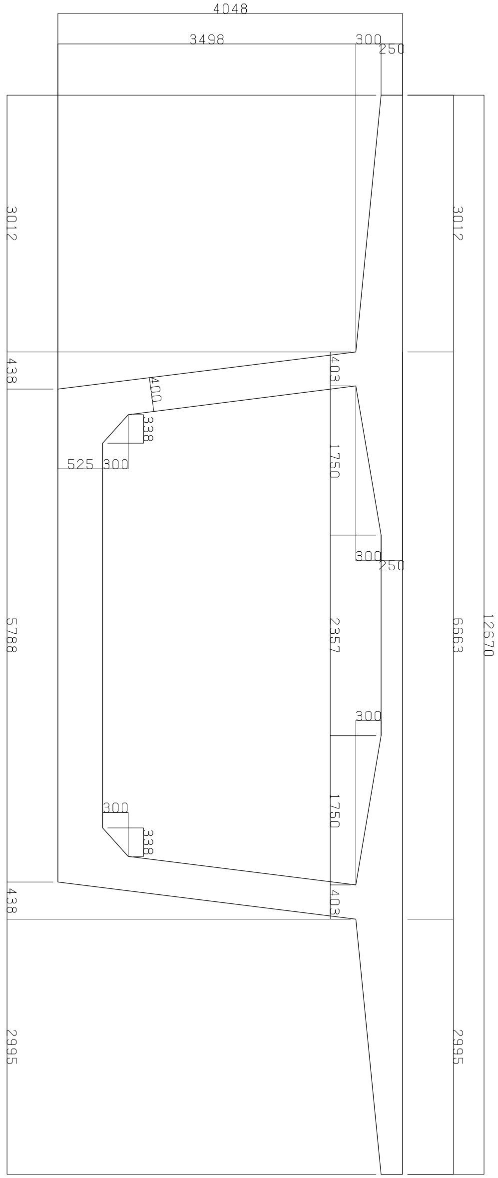
$X = 0.85 \cdot fc' \cdot b \cdot d$

		BOTTOM SLAB		WEB	
		23 AT WEB	27 MID SPAN	44 OUTSIDE	34 INSIDE
Asreq	mm2	518.6	242.1	332.3	1420.5
As		D16ctc125	D16ctc125	D16ctc125	D16ctc125
	mm2	1608.0	1608.0	1608.0	1608.0
d	mm	205.0	200.0	350.0	355.0
Mn	kN·m	126.45	123.23	219.71	222.93
φ		0.900	0.900	0.900	0.900
φ·Mn	kN·m	113.80	110.91	197.74	200.64
Mu	〃	-37.77	17.32	-41.66	-177.74
Fs		3.013	6.404	4.746	1.129


REBARS ARRANGEMENT
D14 ctc 125
D14 ctc 125
D16 ctc 125
D14 ctc 125
40mm
40mm
57mm
40mm
57mm
400
D16 ctc 125
D16 ctc 125
35mm
40mm
D16 ctc 125
D16 ctc 125
40mm
40mm
250
300
1950
250
2000
250
No

4. 5. 2. Section 2(Hg=5.788m,Hfu=0.25m,Hfl=0.525m,Bw=0.40m)

1. CROSS SECTION

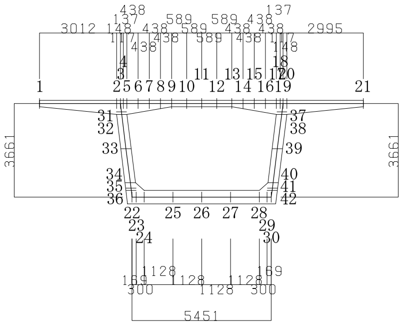


No

No

2. ANALYSIS FOR SECTIONAL FORCE

2. 1. CENTROID AND PROPERTY



MEMBER	A (m2)	I (m4)	X (m)	Y (m)	MAT
1- 2	0.400	0.00758	3.012	0.000	1
2- 3	50.000	50.00000	0.148	0.000	1
3- 4	50.000	50.00000	0.117	0.000	1
4- 5	0.550	0.01386	0.138	0.000	1
5- 6	0.512	0.01140	0.438	0.000	1
6- 7	0.438	0.00713	0.438	0.000	1
7- 8	0.363	0.00410	0.438	0.000	1
8- 9	0.287	0.00208	0.438	0.000	1
9- 10	0.250	0.00130	0.589	0.000	1
10- 11	0.250	0.00130	0.589	0.000	1
11- 12	0.250	0.00130	0.589	0.000	1
12- 13	0.250	0.00130	0.589	0.000	1
13- 14	0.287	0.00208	0.438	0.000	1
14- 15	0.363	0.00410	0.438	0.000	1
15- 16	0.438	0.00713	0.438	0.000	1
16- 17	0.512	0.01140	0.438	0.000	1
17- 18	0.550	0.01386	0.138	0.000	1
18- 19	50.000	50.00000	0.117	0.000	1
19- 20	50.000	50.00000	0.148	0.000	1
20- 21	0.400	0.00758	2.995	0.000	1
22- 23	0.625	0.02035	0.169	0.000	1

MEMBER	A (m2)	I (m4)	X (m)	Y (m)	MAT
23- 24	0.575	0.01620	0.300	0.000	1
24- 25	0.525	0.01206	1.128	0.000	1
25- 26	0.525	0.01206	1.128	0.000	1
26- 27	0.525	0.01206	1.128	0.000	1
27- 28	0.525	0.01206	1.128	0.000	1
28- 29	0.575	0.01620	0.300	0.000	1
29- 30	0.625	0.02035	0.169	0.000	1
3- 31	50.000	50.00000	0.040	0.322	1
31- 32	0.400	0.00533	0.013	0.103	1
32- 33	0.400	0.00533	0.167	1.337	1
33- 34	0.400	0.00533	0.167	1.337	1
34- 35	0.400	0.00533	0.025	0.203	1
35- 36	50.000	50.00000	0.012	0.097	1
36- 22	50.000	50.00000	0.033	0.263	1
19- 37	50.000	50.00000	0.040	0.322	1
37- 38	0.400	0.00533	0.013	0.103	1
38- 39	0.400	0.00533	0.167	1.337	1
39- 40	0.400	0.00533	0.167	1.337	1
40- 41	0.400	0.00533	0.025	0.203	1
41- 42	50.000	50.00000	0.012	0.097	1
42- 30	50.000	50.00000	0.033	0.263	1

SUPPORT CONDITION

No.	CONDITION
22	H
30	Rv

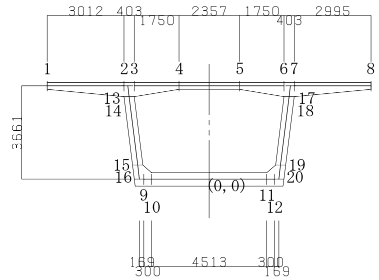
MATERIAL

No.	MODULUS ELASTICITY	THERMAL EXPANSION
1	0.3220E+05	0.000010

No

2．2．LOADING

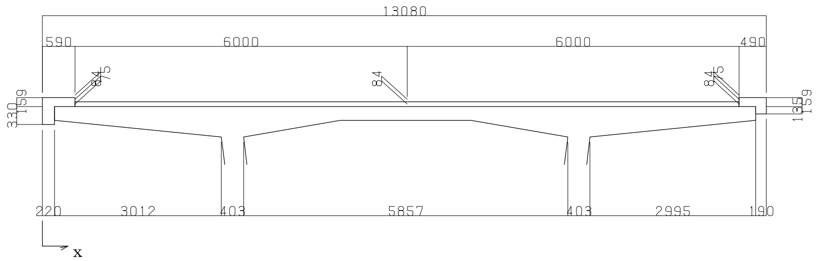
2．2．1．GIRDER-SELF LOAD



SEC.	X (m)	Y (m)	h (m)	W(kN/m)	P(kN)
1	-6.344	3.660	0.250	6.125	
2	-3.332	3.660	0.550	13.475	
3	-2.928	3.660	0.550	13.475	
4	-1.178	3.660	0.250	6.125	
5	1.178	3.660	0.250	6.125	
6	2.928	3.660	0.550	13.475	
7	3.332	3.660	0.550	13.475	
8	6.327	3.660	0.250	6.125	
9	-2.557	0.000	0.792	19.395	
10	-2.257	0.000	0.525	12.863	
11	2.257	0.000	0.525	12.863	
12	2.557	0.000	0.792	19.395	
13	-3.183	3.660	0.400	9.800	1.235
14	-3.130	3.236	0.400	9.800	
14	-3.130	3.236	0.400	9.800	
15	-2.796	0.562	0.400	9.800	
15	-2.796	0.562	0.401	9.830	
16	-2.725	0.000	0.401	9.830	2.276
17	3.183	3.660	0.400	9.800	1.235
18	3.130	3.236	0.400	9.800	
18	3.130	3.236	0.400	9.800	
19	2.796	0.562	0.400	9.800	
19	2.796	0.562	0.401	9.830	
20	2.725	0.000	0.401	9.830	2.276

No

2．2．2．SUPERIMPOSED



ITEMS	LOCATION x (m)	UNIT(kN/m3)	P(kN)	Q(kN/m)
EDGE BLOCK(L)	0.110	24.500	1.779	
PARAPET(L)	0.295	24.500	2.298	
HANDRAIL(L)	0.214		8.550	
PAVEMENT(L)	0.590	22.500		1.890
PAVEMENT(MID)	6.590	22.500		1.890
PAVEMENT(R)	12.590	22.500		1.890
HANDRAIL(R)	12.915		7.447	
PARAPET(R)	12.835	24.500	1.909	
EDGE BLOCK(R)	12.985	24.500	0.628	

WIND LOAD→
W1L = 5.360
W1R = 5.360

WUND LOAD←
W2L = 5.360
W2R = 5.360

WIND LOAD→
W3L = 1.500× (1.800+ 0.125+ 0.084) = 3.013 kN·m
W3R = 1.500× (1.800+ 0.125+ 0.084) = 3.013 kN·m

WUND LOAD←
W4L = 1.500× (1.800+ 0.125+ 0.084) = 3.013 kN·m
W4R = 1.500× (1.800+ 0.125+ 0.084) = 3.013 kN·m

2.2.3. SECONDARY FORCE DUE TO PRE-STRESS

1) AXIAL FORCE AND MOMENT DUE TO PRE-STRESS IMMEDIATELY AFTER ANCHOR SET

PRE-STRESS AFTER SET $Pt = \sigma pt \cdot Ap / P$

ECCENTRICITY MOMENT $Mpt = Pt \cdot ep$

$\sigma pt'$: TENSIL STRESS AT EACH SECTION

$\Delta \sigma p$: ELASTIC SHORTNING

σpt : TENDON STRESS IMMEDIATELY AFTER ANCHOR SET

Ap : AREA 416.100 mm^2

P : SPACING 0.750 m

ep : ECCENTRICITY

SEC.	σpt' Mpa	Δσp Mpa	σpt Mpa	Pt kN	ep m	Mpt kN·m	ΔMpt kN·m	ΔL m	ΔMpt/ΔL kN	P kN
							143.989			
3	1261.88	8.10	1253.78	695.599	0.207	143.989	-6.944	0.117	-59.244	59.244
4	1261.99	8.10	1253.89	695.659	0.197	137.045	-8.335	0.138	-60.618	1.374
5	1262.12	8.10	1254.02	695.729	0.185	128.710	-25.709	0.438	-58.763	-1.854
6	1262.52	8.10	1254.42	695.952	0.148	103.001	-26.838	0.438	-61.344	2.581
7	1267.55	8.10	1259.45	698.742	0.109	76.163	-32.035	0.438	-73.222	11.878
8	1270.63	8.10	1262.53	700.449	0.063	44.128	-32.918	0.438	-75.240	2.018
9	1271.03	8.10	1262.93	700.671	0.016	11.211	-8.402	0.589	-14.260	-60.980
10	1273.75	8.10	1265.65	702.182	0.004	2.809	-2.809	0.589	-4.767	-9.493
11	1279.36	8.10	1271.26	705.294	0.000	0.000	2.834	0.589	4.809	-9.576
12	1284.93	8.10	1276.83	708.387	0.004	2.834	8.525	0.589	14.468	-9.659
13	1287.64	8.10	1279.54	709.889	0.016	11.358	33.379	0.438	76.294	-61.825
14	1288.03	8.10	1279.93	710.107	0.063	44.737	32.848	0.438	75.081	1.213
15	1291.06	8.10	1282.96	711.785	0.109	77.585	28.165	0.438	64.377	10.704
16	1295.99	8.10	1287.89	714.523	0.148	105.749	26.478	0.438	60.520	3.857
17	1296.38	8.10	1288.28	714.740	0.185	132.227	8.590	0.138	62.475	-1.955
18	1296.51	8.10	1288.41	714.808	0.197	140.817	7.160	0.117	61.086	1.389
19	1296.61	8.10	1288.51	714.866	0.207	147.977	-147.977			61.086

2) TEMPERATURE FORCE EQUIVALENT TO SECONDARY FORCE DUE TO ELASTIC SHORTNING

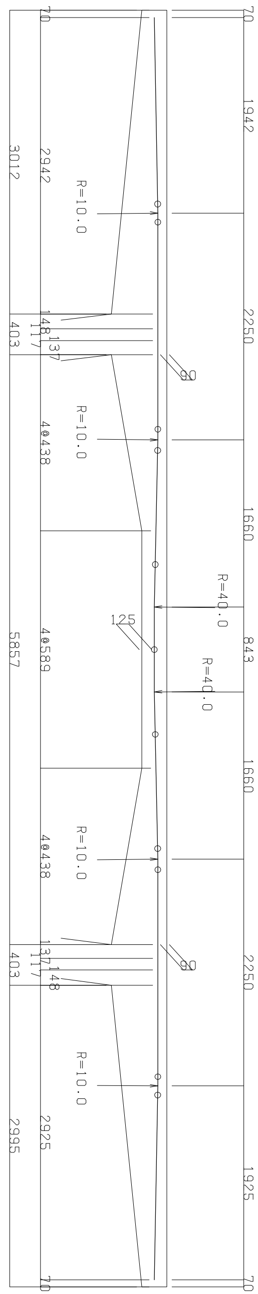
AVERAGE AREA OF TOP SLAB $A = 0.340\ m^2$

$\sigma c = Pt / A = 705.3 / 0.340 = 2076.5\ kN/m^2$

$\sigma c / Ec = 2076.5 / 28800000.0 = 7.2 \times 1^{0-}$

THEREFORE, 7.2℃ DIFFERENCE IS LOADED.

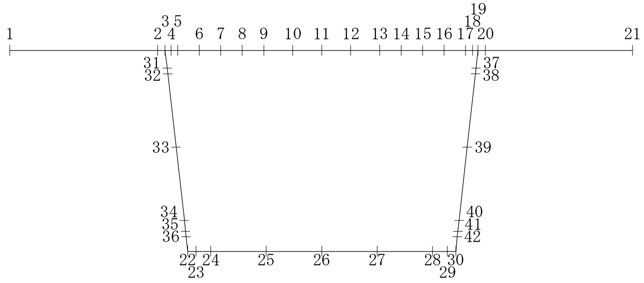
TENDON ARRANGEMENT



ep (m)	0.004	0.185 0.207 0.185 0.185	0.148	0.109	0.063	0.016	0.004	0.000	0.004	0.016	0.063	0.109	0.148	0.185 0.185 0.207 0.185	0.003
yp (m)	0.125	0.090 0.090 0.090 0.090	0.090	0.091	0.100	0.109	0.121	0.125	0.121	0.109	0.100	0.091	0.090	0.090 0.090 0.090 0.090	0.125
tu (m)	0.257	0.550 0.594 0.550 0.550	0.475	0.400	0.325	0.250	0.250	0.250	0.250	0.250	0.325	0.400	0.475	0.550 0.550 0.594 0.550	0.257

No

2．2．5．LOADING CASE



CASE　1 (GIRDER-SELF LOAD)

ALL MEMBERS ARE LOADED

CASE　2 (SUPERIMPOSED)

TOP SLAB　1 ～ 21 ARE LOADED

CASE　3 (SECONDARY FORCE DUE TO PRE-STRESS)

TOP SLAB　3 IS LOADED144.0 kN·m AND 59.2 kN

TOP SLAB 19 IS LOADED 148.0 kN·m　AND 61.1 kN

TOP SLAB　4 ～ 18 ARE LOADED

CASE　4 (TRUCK LOAD ON CANTILEVER(L))

SEC.　3 IS LOADED MOMENT (-103.860 kN·m) DUE TO TRUCK LOAD

CASE　5 (TRUCK LOAD ON CANTILEVER(R))

SEC.19 IS LOADED MOMENT (-103.860 kN·m) DUE TO TRUCK LOAD

CASE　6 (TRUCK LOAD ON SLAB)

SEC.　3 AND 19 ARE LOADED MOMENT (-77.270 kN·m) DUE TO TRUCK LOAD

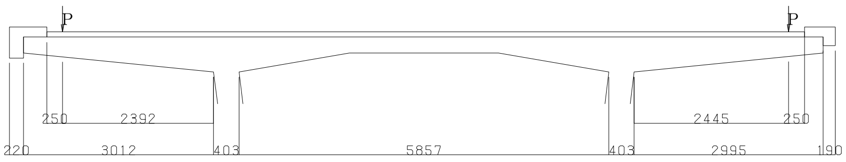
CASE　7 (TEMPERATURE GRADIENT)

TOP SLAB 3 ～19 ARE LOADED ＋ 7.0 ℃

CASE　8(WIND LOAD ON STRUCTURE(L))

No

2．2．4．LIVE LOAD



TRUCK LOAD ON CANTILEVER(L)

CANTILEVER LEGTH　l　= 2.392 m　　IMPACT　　K1 = 1.000

M =　-103.860 kN·m

TRUCK LOAD ON SPAN (AT WEB)

SPAN LEGTH　l　= 5.857 m　　IMPACT　　K1 = 1.000

M =　 -77.270 kN·m

TRUCK LOAD (AT MID SPAN)

SPAN LEGTH　l　= 5.857 m　　IMPACT　　K1 = 1.000

M =　　63.320 kN·m

TRUCK LOAD ON CANTILEVER(R)

CANTILEVER LEGTH　l　= 2.445 m　　IMPACT　　K1 = 1.000

M =　-103.860 kN·m

2.2.6. LOAD COMBINATIONS

			1	2	3	4	5	6	7	8	9	10	11				
			DC	DW	EL	CR	SH	LL+IM	PL	TG	WS	WL	CT				
	Service-1	SE1-1	1.00	1.00	1.00	1.00		1.00	1.00	0.50	0.30	1.00		Lmax	R	±	1
		SE1-2	1.00	1.00	1.00	1.00		1.00	1.00	0.50	0.30	1.00		Lmin	R	±	2
		SE1-3	1.00	1.00	1.00	1.00		1.00	1.00	0.50	-0.30	-1.00		Lmax	L	±	3
S		SE1-4	1.00	1.00	1.00	1.00		1.00	1.00	0.50	-0.30	-1.00		Lmin	L	±	4
L		SE1-5	1.00	1.00	1.00	1.00				1.00	0.30			Wind	R	±	5
S		SE1-6	1.00	1.00	1.00	1.00				1.00	-0.30			Wind	L	±	6
	Service-3	SE3-1	1.00	1.00	1.00	1.00		0.80	0.80	0.50				Lmax	-	±	7
		SE3-2	1.00	1.00	1.00	1.00		0.80	0.80	0.50				Lmin	-	±	8
		SE3-3	1.00	1.00	1.00	1.00				1.00					-	±	9

CASE 9(WIND LOAD ON STRUCTURE(R))

CASE 10(WIND LOAD ON VEHICLE(L))

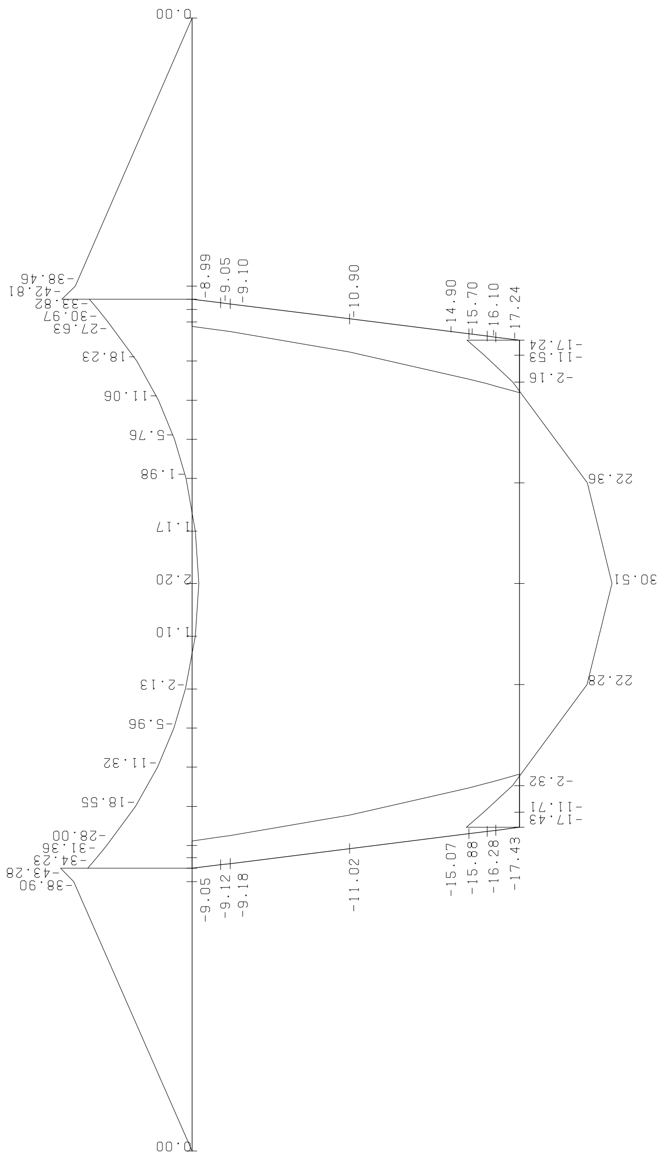
CASE 11(WIND LOAD ON VEHICLE(R))

CASE 12 (TEMPERATURE GRADIENT)

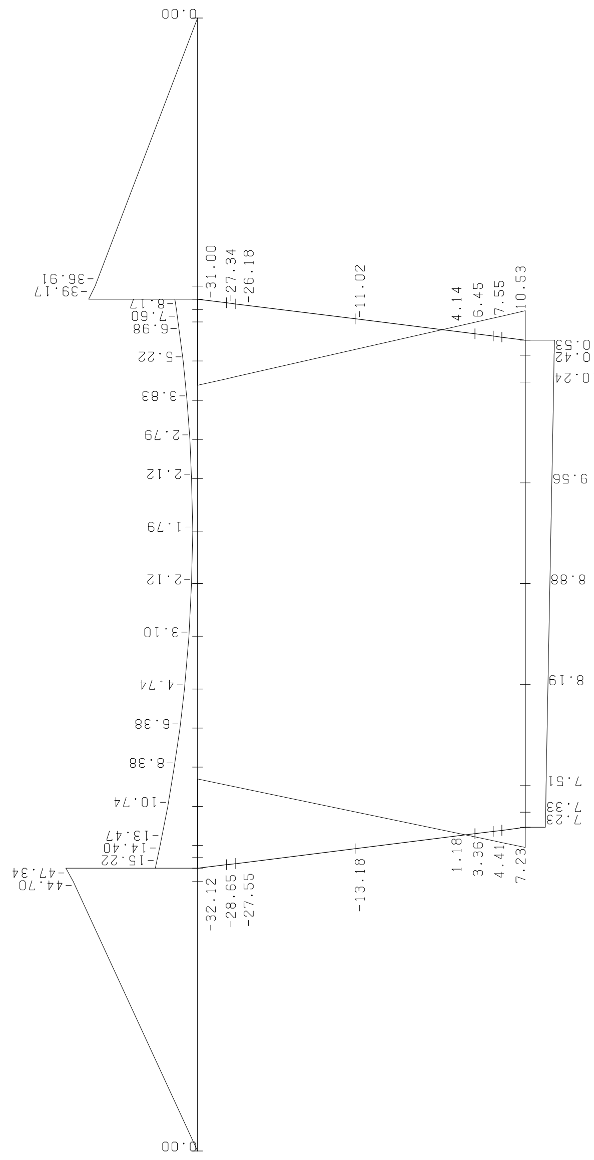
TOP SLAB 3 ～19 ARE LOADED － 2.0 ℃

2. 3.BENDING MOMENT

CASE 1 (GIRDER-SELF LOAD)

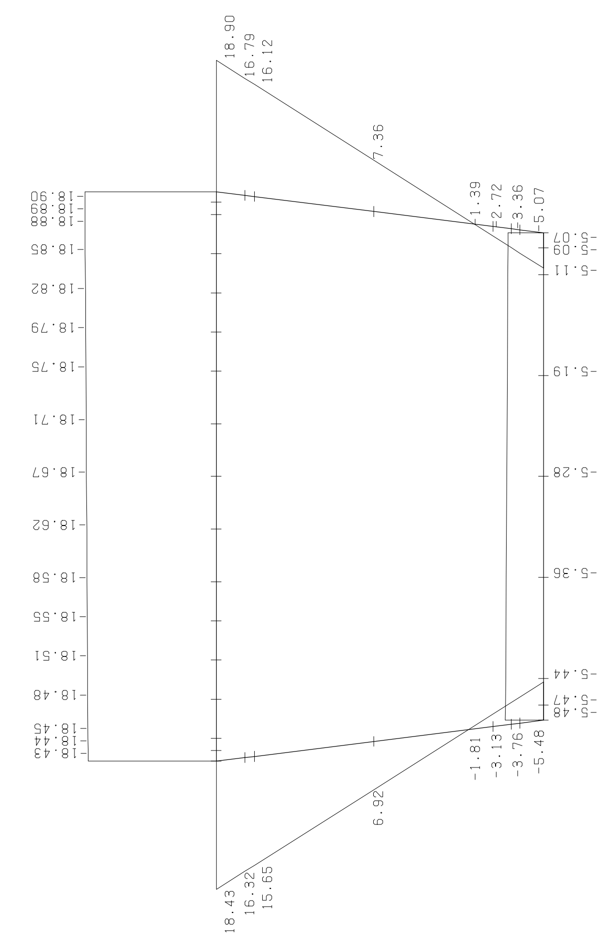


CASE 2 (SUPERIMPOSED)



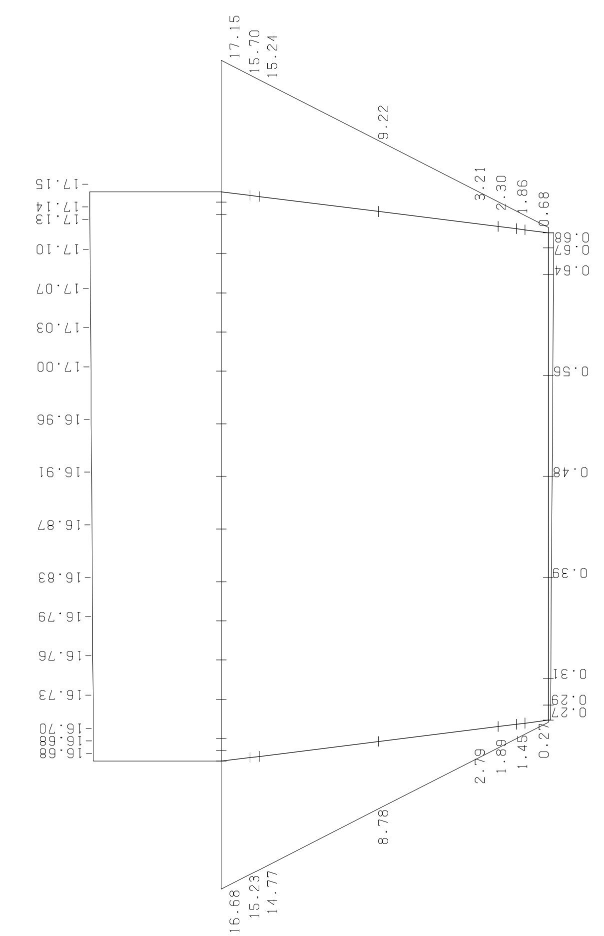
No

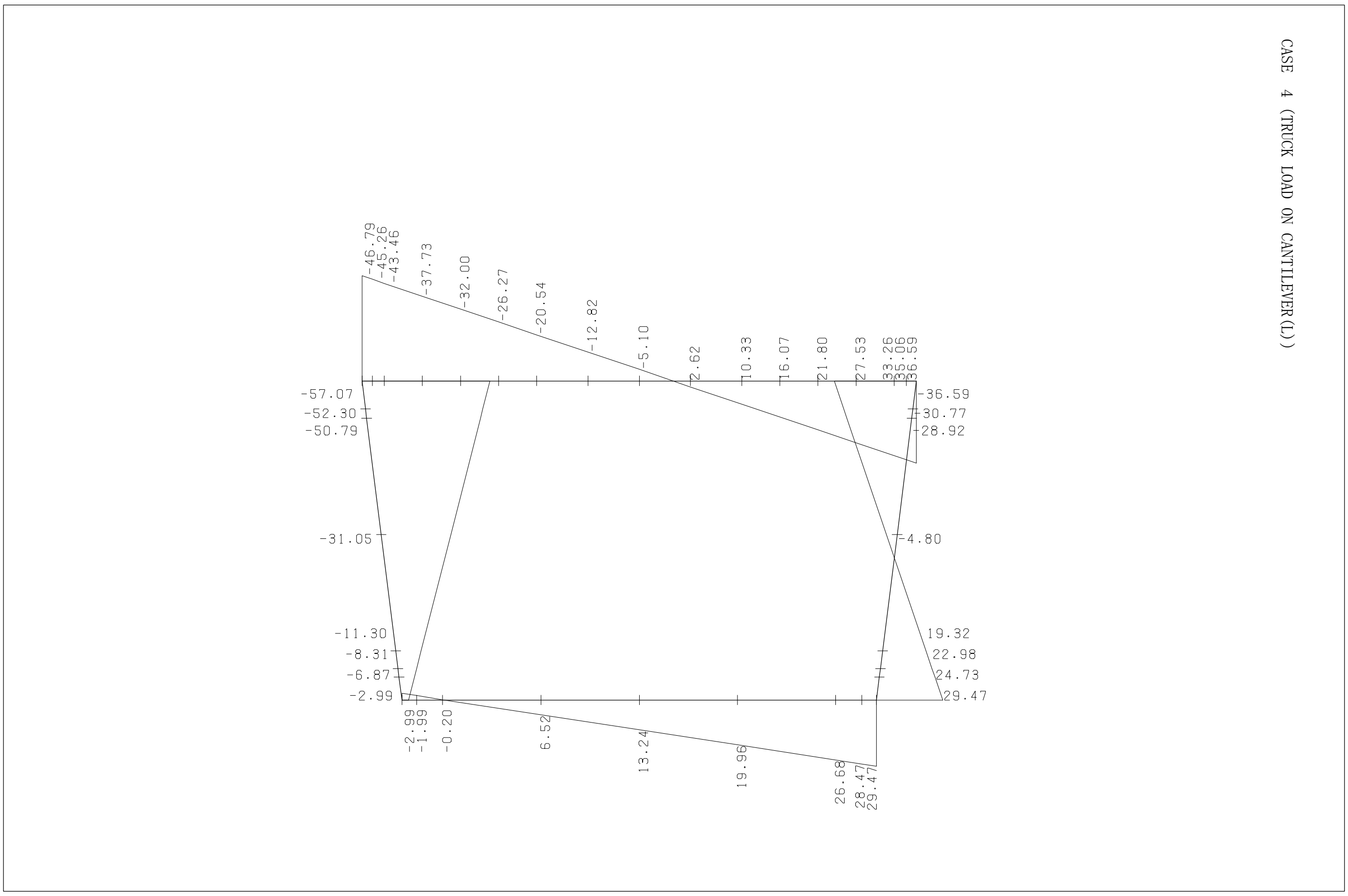
CASE 3-1 (MOMENT DUE TO PRESTRES)

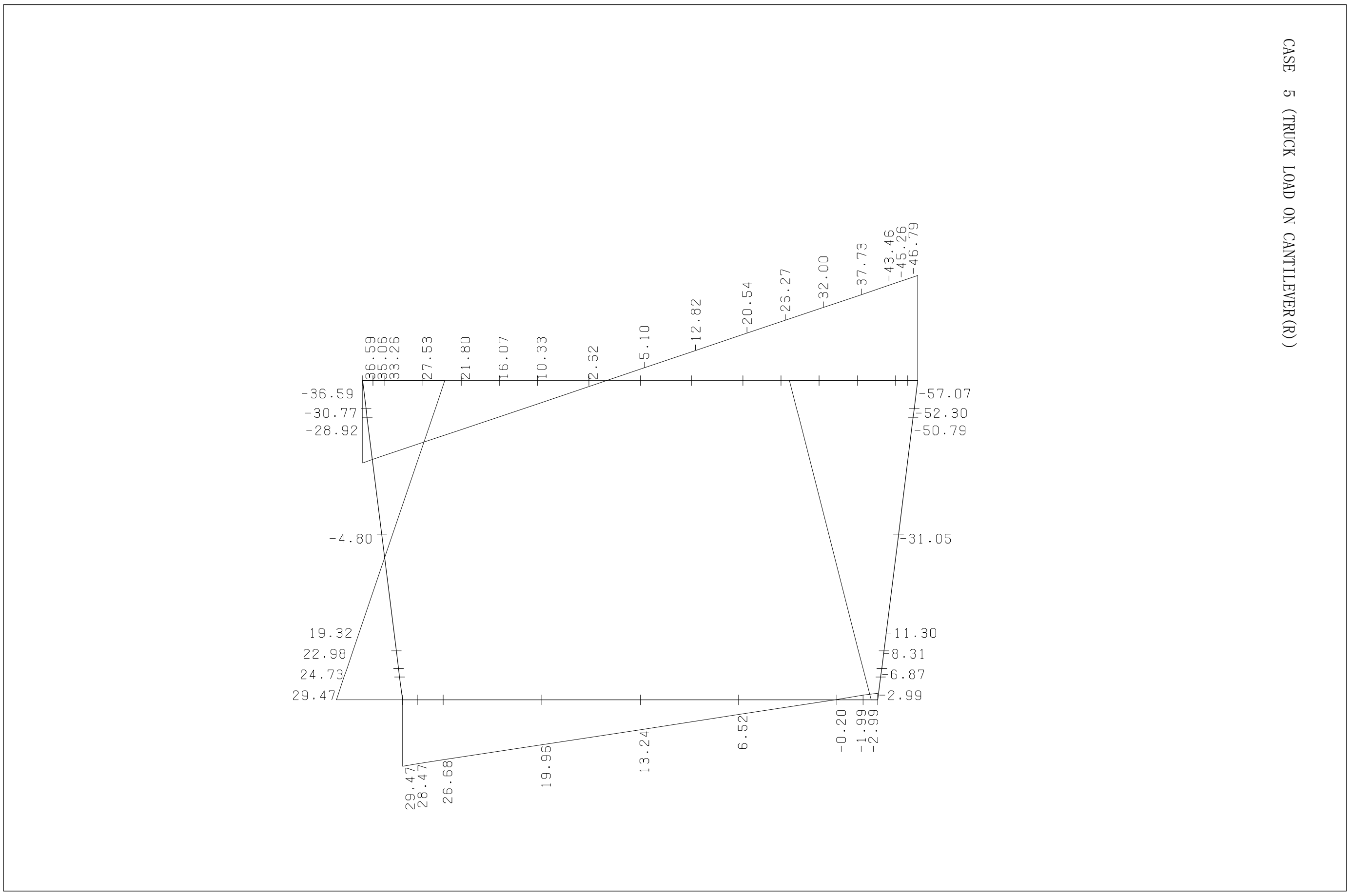


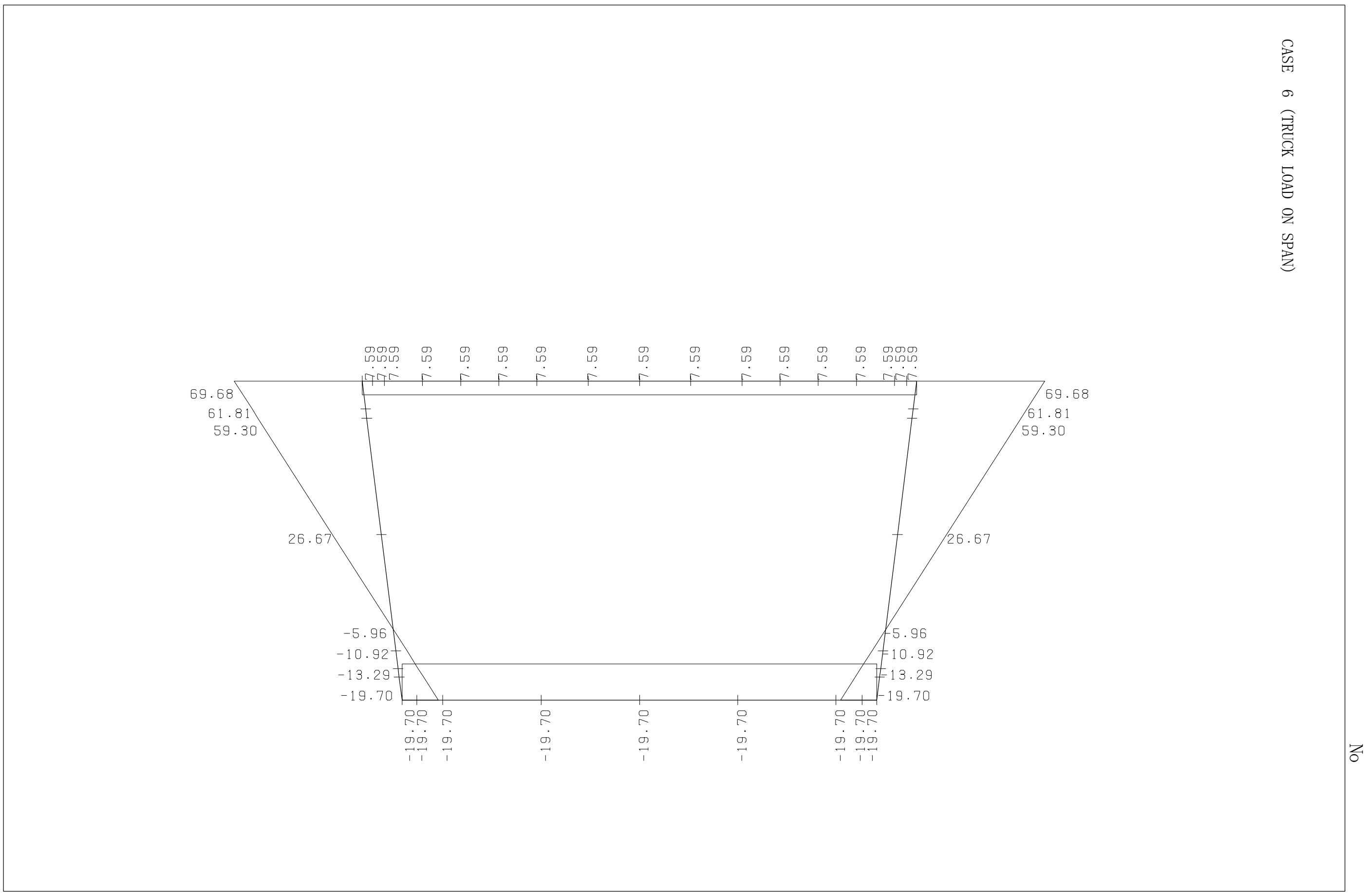
No

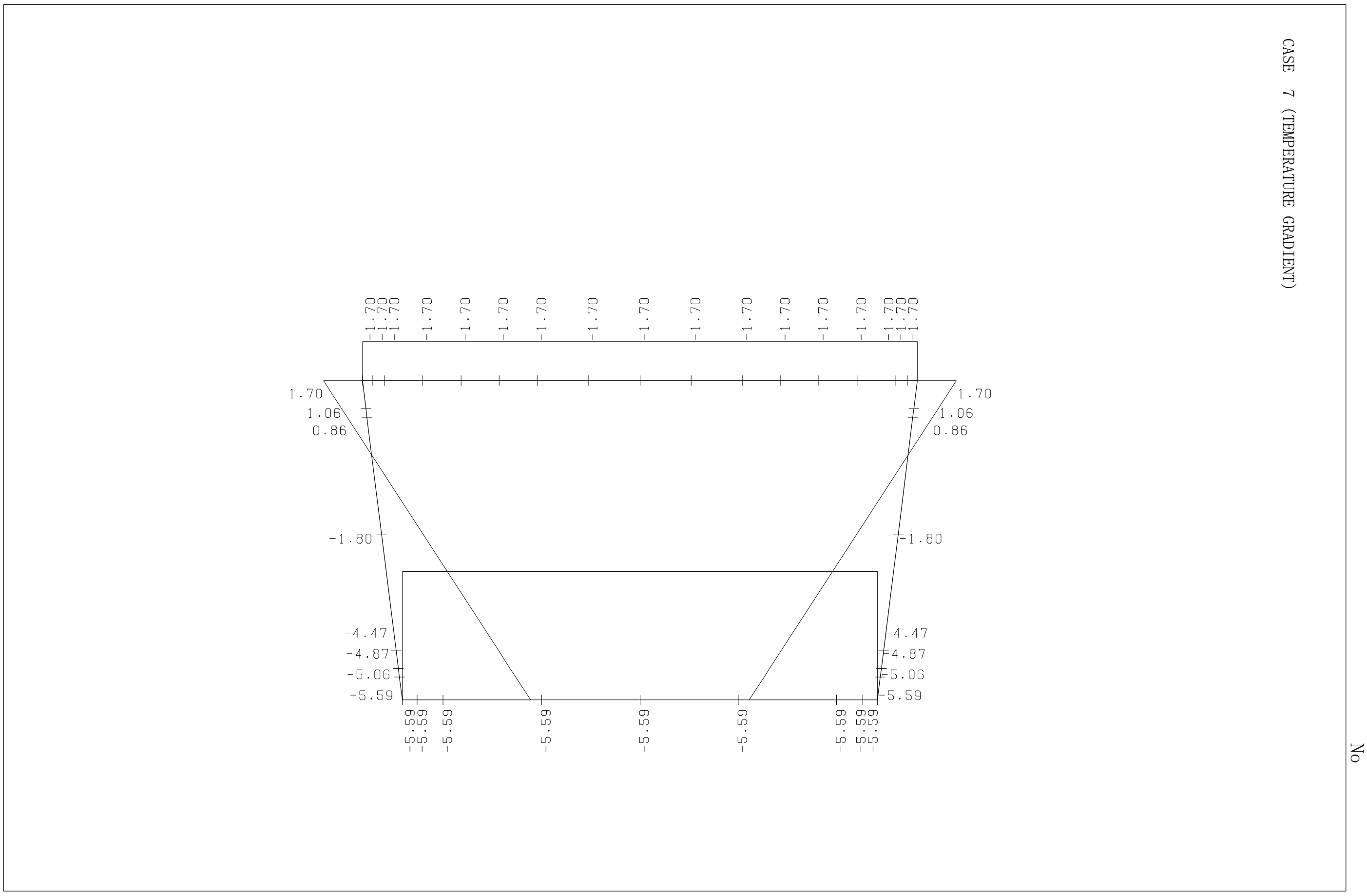
CASE 3-2 (SECONDARY MOMENT DUE TO PRESTRESS - ECCENTRICITY+ELASTIC SHORTNING)

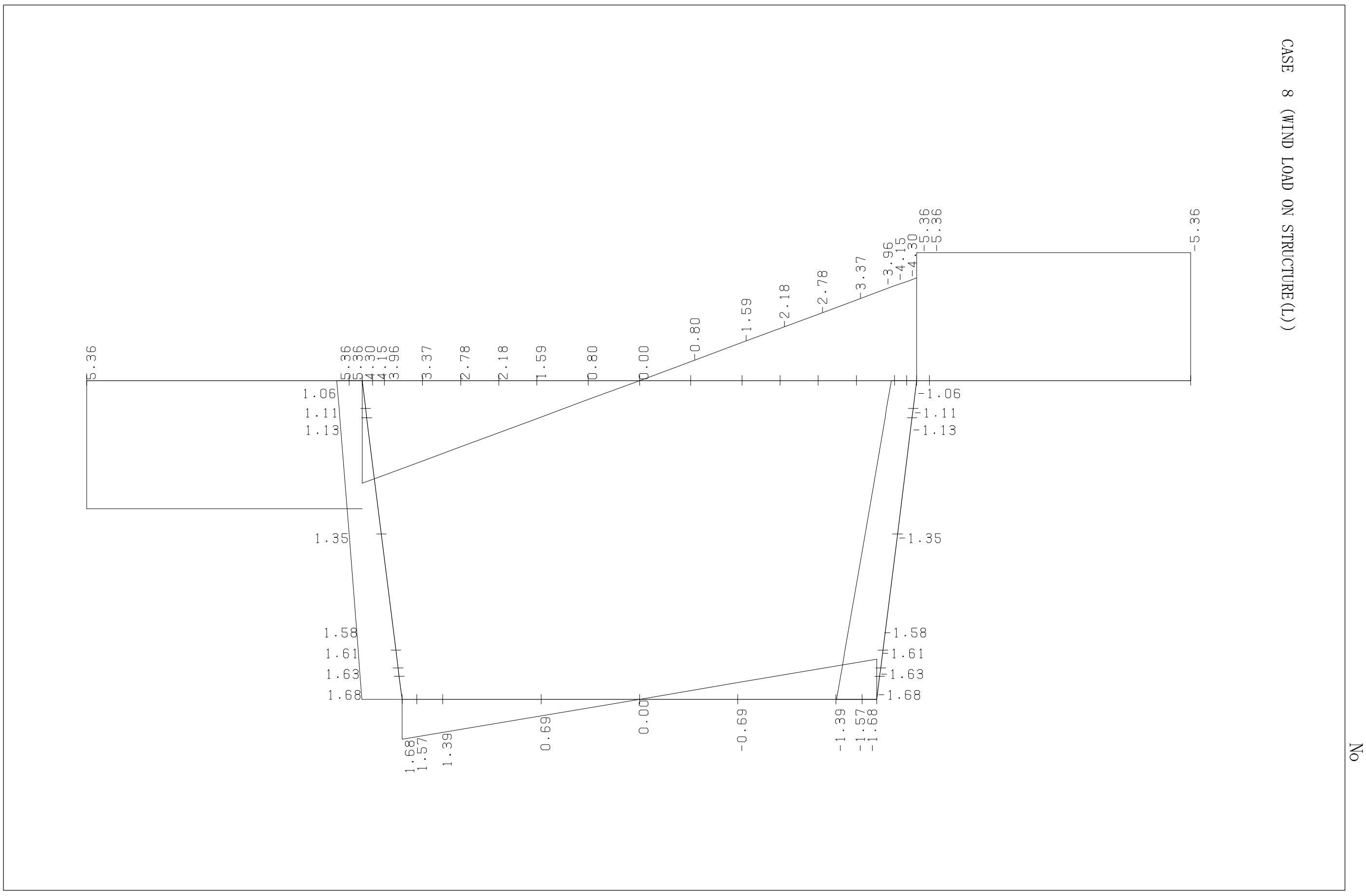


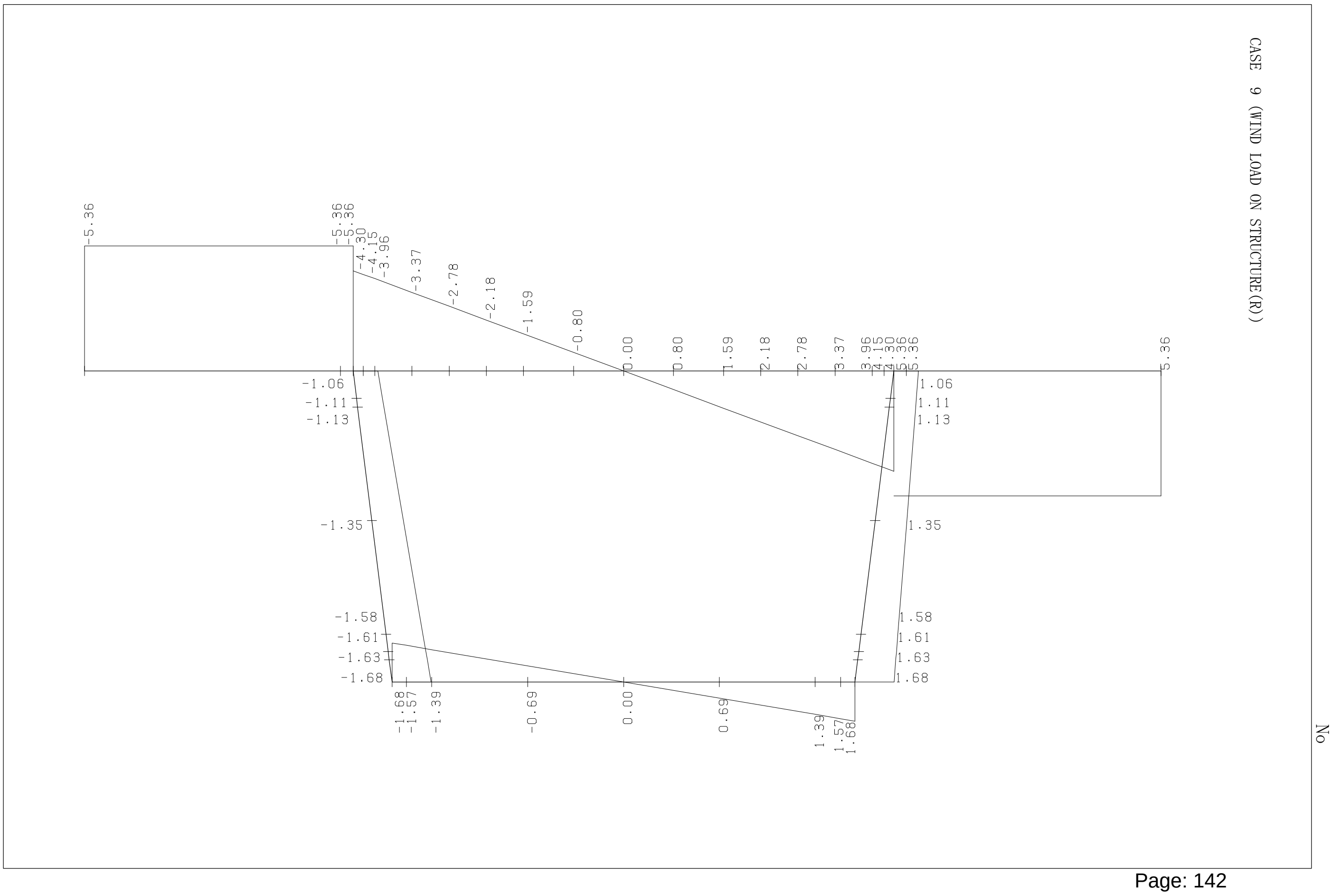

CASE 4 (TRUCK LOAD ON CANTILEVER(L))
-46.79
-45.26
-43.46
-37.73
-32.00
-26.27
-20.54
-12.82
-5.10
2.62
10.33
16.07
21.80
27.53
33.26
35.06
36.59
-57.07
-52.30
-50.79
-31.05
-11.30
-8.31
-6.87
-2.99
-36.59
30.77
28.92
4.80
19.32
22.98
24.73
29.47
-2.99
-1.99
-0.20
6.52
13.24
19.96
26.68
28.47
29.47
No


CASE 5 (TRUCK LOAD ON CANTILEVER(R))
36.59
35.06
33.26
27.53
21.80
16.07
10.33
2.62
-5.10
-12.82
-20.54
-26.27
-32.00
-37.73
-43.46
-45.26
-46.79
-36.59
-30.77
-28.92
-4.80
19.32
22.98
24.73
29.47
-57.07
-52.30
-50.79
-31.05
-11.30
-8.31
-6.87
-2.99
29.47
28.47
26.68
19.96
13.24
6.52
-0.20
-1.99
-2.99
No


CASE 6 (TRUCK LOAD ON SPAN)
69.68
61.81
59.30
26.67
-5.96
-10.92
-13.29
-19.70
7.59
No

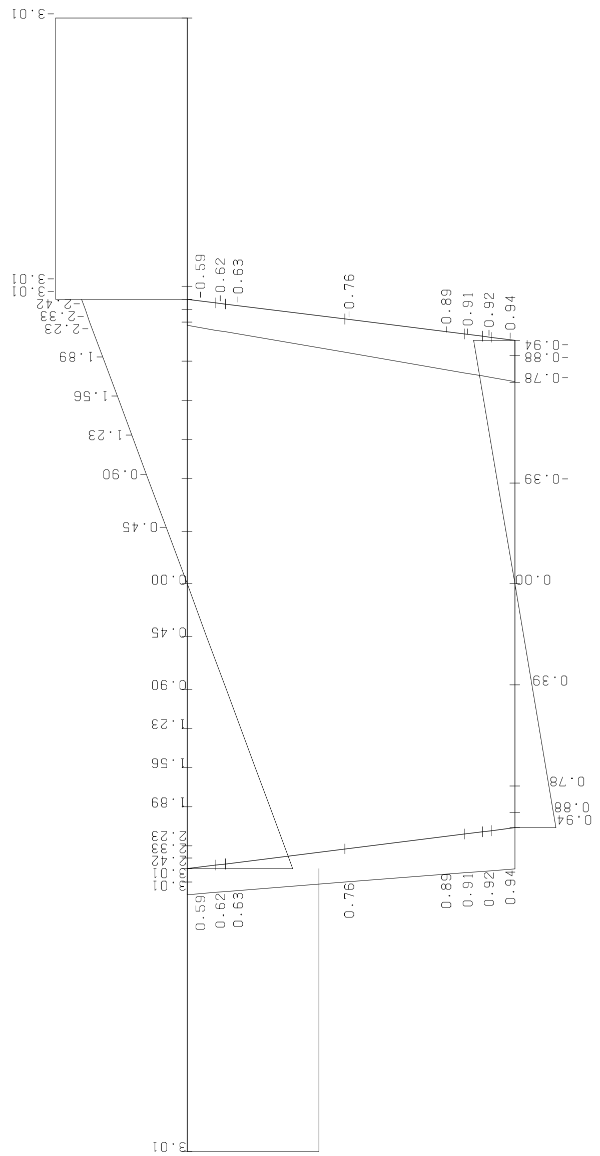

CASE 7 (TEMPERATURE GRADIENT)
-1.70
1.70
1.06
0.86
-1.80
-4.47
-4.87
-5.06
-5.59
No


CASE 8 (WIND LOAD ON STRUCTURE(L))
No


CASE 9 (WIND LOAD ON STRUCTURE(R))
No

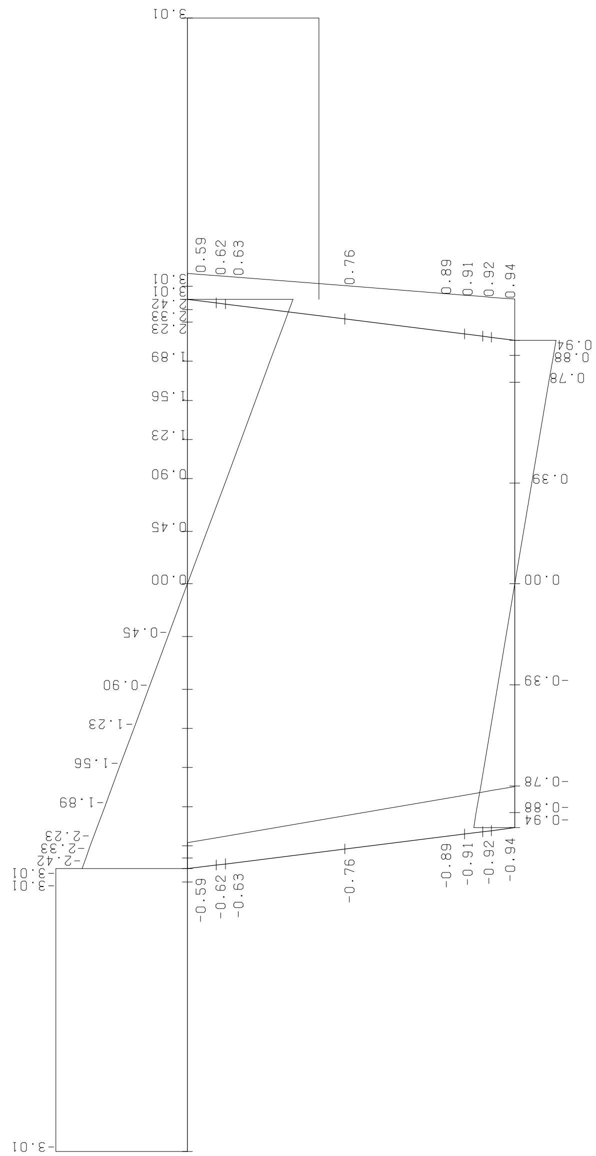
No

ケース 10 (WIND LOAD ON VEHICLE(L))

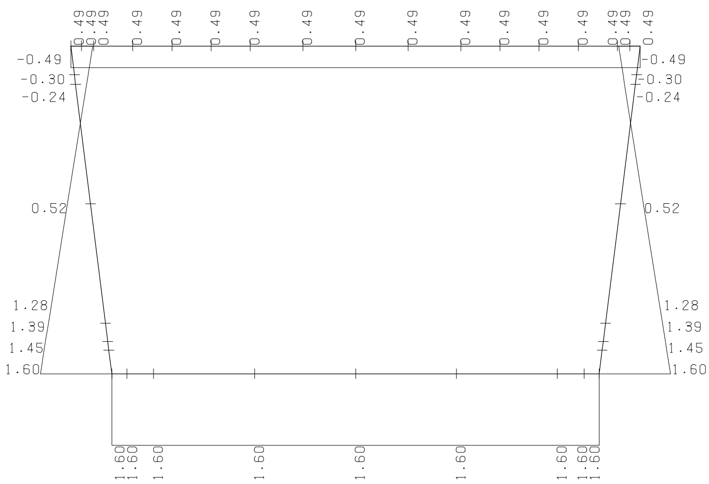


No

ケース 11 (WIND LOAD ON VEHICLE(R))



ケース 12 (TEMPERATURE GRADIENT)

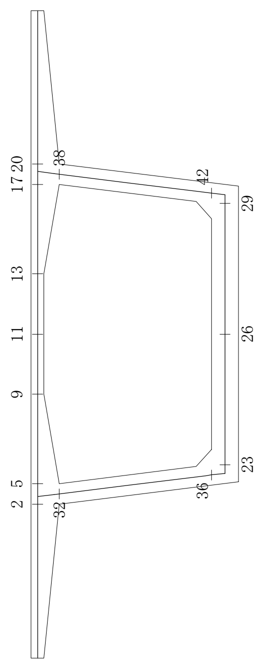


No

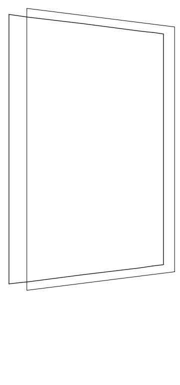
No

2．4．COMBINATION

BOX-SECTION MODEL FRAME



MOMENT SHALL BE POSITIVE AS THE DOT LINE SIDE IS TENSILE



No

SERVICE LIMIT STATE

COLLECTION TABLE (TOP SLAB kN·m)

			2	5	9	11	13	17	20
GIRDER-SELF LOAD			-38.90	-28.00	-2.13	2.20	-1.98	-27.63	-38.46
SUPERIMPOSED			-44.70	-13.47	-4.74	-2.12	-2.12	-6.98	-36.91
SECONDARY MOMENT	CONST.			-16.69	-16.83	-16.91	-17.00	-17.13	
	SERVICE			-15.39	-15.51	-15.59	-15.67	-15.79	
CREEP				1.31	1.32	1.33	1.33	1.34	
LIVE LOAD(LL+IM)	MAX.					63.32			
	MIN.		-103.86	-77.27				-77.27	-103.86
+TEMP.	7.00	1.00·TG		-1.70	-1.70	-1.70	-1.70	-1.70	
	3.50	0.50·TG		-0.85	-0.85	-0.85	-0.85	-0.85	
-TEMP.	-2.00	1.00·TG		0.49	0.49	0.49	0.49	0.49	
	-1.00	0.50·TG		0.24	0.24	0.24	0.24	0.24	
WS	→	0.30·WS	1.61	1.19	0.48	0.00	-0.48	-1.19	-1.61
	←	0.30·WS	-1.61	-1.19	-0.48	0.00	0.48	1.19	1.61
WL	→	1.00·WL	3.01	2.23	0.90	0.00	-0.90	-2.23	-3.01
	←	1.00·WL	-3.01	-2.23	-0.90	0.00	0.90	2.23	3.01
AFTER ANCHOR SET			-38.90	-44.70	-18.96	-14.72	-18.98	-44.76	-38.46
PERMANENT LOAD			-83.60	-56.86	-22.37	-15.51	-19.77	-50.40	-75.37
SLS	Service-1	SE1- 1	-78.98	-54.29	-21.85	48.06	-22.00	-54.66	-79.99
		SE1- 2	-182.84	-131.56	-21.85	-16.36	-22.00	-131.93	-183.85
		SE1- 3	-88.22	-61.12	-24.60	48.06	-19.25	-47.83	-70.74
		SE1- 4	-192.08	-138.39	-24.60	-16.36	-19.25	-125.10	-174.60
		SE1- 5	-81.99	-57.37	-23.60	-17.21	-21.95	-53.29	-76.97
		SE1- 6	-85.21	-59.75	-24.55	-17.21	-21.00	-50.91	-73.76
		MAX.	-78.98	-54.29	-21.85	48.06	-19.25	-47.83	-70.74
		MIN.	-192.08	-138.39	-24.60	-17.21	-22.00	-131.93	-183.85
	Service-3	SE3- 1	-83.60	-57.71	-23.22	35.39	-20.62	-51.25	-75.37
		SE3- 2	-166.69	-119.52	-23.22	-16.36	-20.62	-113.06	-158.45
		SE3- 3	-83.60	-58.56	-24.07	-17.21	-21.48	-52.10	-75.37
		MAX.	-83.60	-57.71	-23.22	35.39	-20.62	-51.25	-75.37
		MIN.	-166.69	-119.52	-24.07	-17.21	-21.48	-113.06	-158.45

No

COLLECTION TABLE (BOTTOM SLAB kN·m)

			23	26	29
CASE 4			-1.99	13.24	28.47
CASE 5			26.50	67.10	30.13
CASE 6			28.47	13.24	-1.99
CASE 7			-29.95	20.93	-26.33
CASE 8			-19.70	-19.70	-19.70
CASE 9			19.45	61.81	23.08
GIRDER-SELF LOAD			-11.71	30.51	-11.53
SUPERIMPOSED			7.33	8.88	10.42
SECONDARY MOMENT	CONST.		0.29	0.48	0.67
	SERVICE		0.26	0.44	0.61
CREEP			-0.02	-0.04	-0.05
LIVE LOAD(LL+IM)	MAX.		28.47	26.48	28.47
	MIN.		-21.69	-19.70	-21.69
+TEMP.	7.00	1.00·TG	-5.59	-5.59	-5.59
	3.50	0.50·TG	-2.79	-2.79	-2.79
-TEMP.	-2.00	1.00·TG	1.60	1.60	1.60
	-1.00	0.50·TG	0.80	0.80	0.80
WS	→	0.30·WS	0.47	0.00	-0.47
	←	0.30·WS	-0.47	0.00	0.47
WL	→	1.00·WL	0.88	0.00	-0.88
	←	1.00·WL	-0.88	0.00	0.88
AFTER ANCHOR SET			-11.43	30.99	-10.86
PERMANENT LOAD			-4.12	39.83	-0.49
SLS	Service-1	SE1- 1	26.50	67.10	27.42
		SE1- 2	-27.24	20.93	-26.33
		SE1- 3	23.79	67.10	30.13
		SE1- 4	-29.95	20.93	-23.62
		SE1- 5	-9.23	41.42	-6.55
		SE1- 6	-10.17	41.42	-5.61
		MAX.	26.50	67.10	30.13
		MIN.	-29.95	20.93	-26.33
	Service-3	SE3- 1	19.45	61.81	23.08
		SE3- 2	-24.26	24.87	-20.63
		SE3- 3	-9.70	41.42	-6.08
		MAX.	19.45	61.81	23.08
		MIN.	-24.26	24.87	-20.63

No

3.STRESS CHECK FOR TOP SLAB

3．1．TRANSVERSAL

3．1．1.STRESS DUE TO LOAD

(kN·m)

			2	5	9	11	13	17	20
MOMENT	AFTER ANCHOR SET		-38.90	-44.70	-18.96	-14.72	-18.98	-44.76	-38.46
	PERMANENT LOAD		-83.60	-56.86	-22.37	-15.51	-19.77	-50.40	-75.37
	SERVICE	MAX.	-78.98	-54.29	-21.85	48.06	-19.25	-47.83	-70.74
		MIN.	-192.08	-138.39	-24.60	-17.21	-22.00	-131.93	-183.85
	Service-3	MAX.	-83.60	-57.71	-23.22	35.39	-20.62	-51.25	-75.37
		MIN.	-166.69	-119.52	-24.07	-17.21	-21.48	-113.06	-158.45
I / yc (m3)			0.0504	0.0504	0.0104	0.0104	0.0104	0.0504	0.0504
STRESS	AFTER ANCHOR SET		-0.77	-0.89	-1.82	-1.41	-1.82	-0.89	-0.76
			0.77	0.89	1.82	1.41	1.82	0.89	0.76
	PERMANENT LOAD		-1.66	-1.13	-2.15	-1.49	-1.90	-1.00	-1.49
			1.66	1.13	2.15	1.49	1.90	1.00	1.49
	Service-1	MAX.	-1.57	-1.08	-2.10	4.61	-1.85	-0.95	-1.40
			1.57	1.08	2.10	-4.61	1.85	0.95	1.40
		MIN.	-3.81	-2.74	-2.36	-1.65	-2.11	-2.62	-3.65
			3.81	2.74	2.36	1.65	2.11	2.62	3.65
	Service-3	MAX.	-1.66	-1.14	-2.23	3.40	-1.98	-1.02	-1.49
			1.66	1.14	2.23	-3.40	1.98	1.02	1.49
		MIN.	-3.31	-2.37	-2.31	-1.65	-2.06	-2.24	-3.14
			3.31	2.37	2.31	1.65	2.06	2.24	3.14

(Mpa)

IN STRESS TABLE, UPPER COLUMN SHOWS EXTREME TOP AND LOWER COLUMN SHOWS EXTREME BOTTOM

STRESS CALCULATION

$$\left.\begin{matrix}\sigma cu \\ \sigma cl\end{matrix}\right\} = \pm \frac{M}{Z}$$

M : BENDING MOMENT (kN·m)

Z : I/yc (m3)

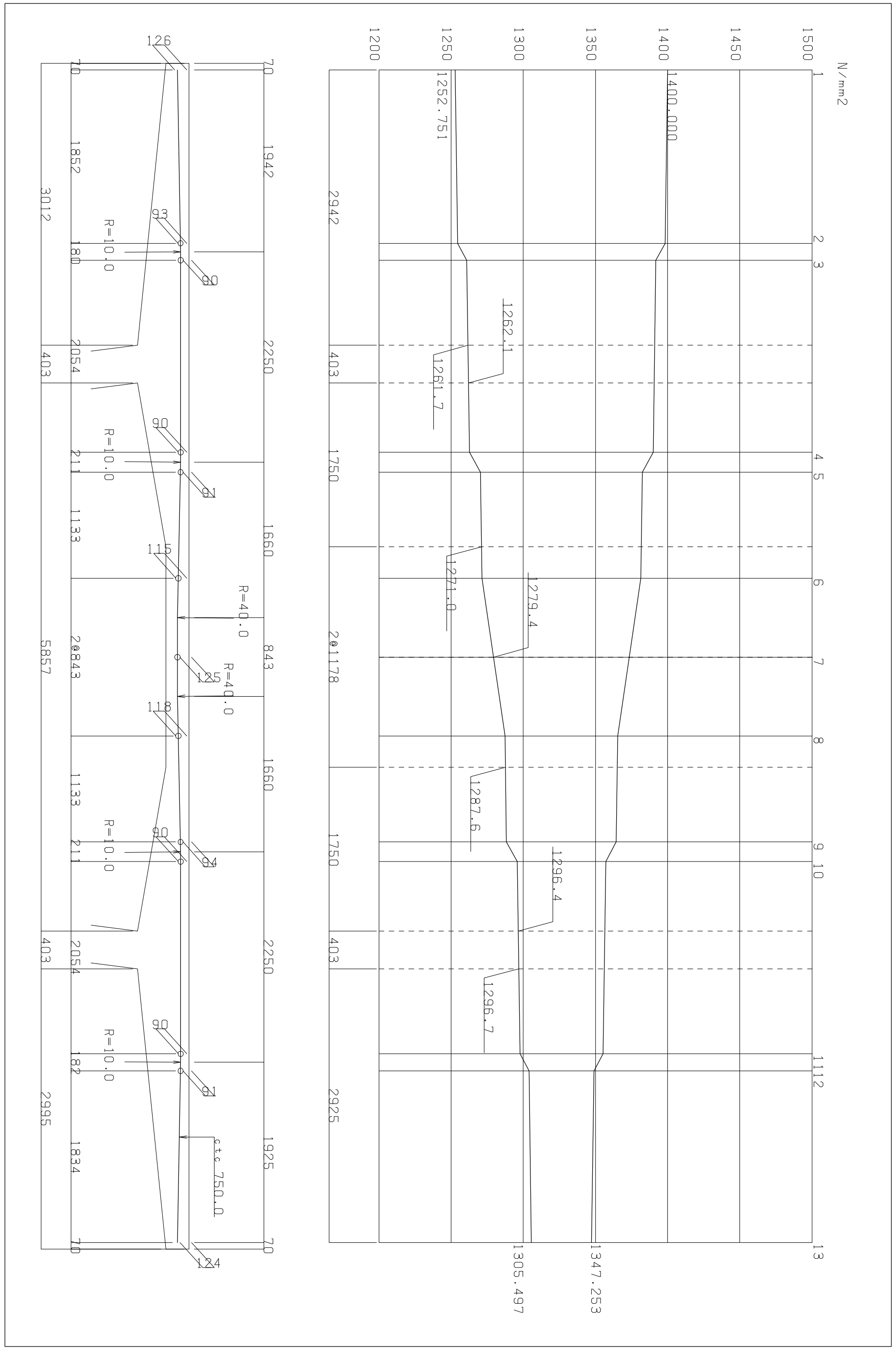
σcu : EXTREMELY FIBER STRESS(TOP) (Mpa)

σcu : EXTREMELY FIBER STRESS(BOTTOM) (Mpa)

No

COLLECTION TABLE (WEB kN·m)

			32	36	38	42
CASE 4			-50.79	-6.87	-28.92	24.73
CASE 5			37.59	16.32	39.46	20.02
CASE 6			-28.92	24.73	-50.79	-6.87
CASE 7			-103.92	-34.64	-102.05	-30.94
CASE 8			59.30	-13.29	59.30	-13.29
CASE 9			24.75	9.97	26.63	13.67
GIRDER-SELF LOAD			-9.18	-16.28	-9.10	-16.10
SUPERIMPOSED			-27.55	4.41	-26.18	7.55
SECONDARY MOMENT	CONST.		14.77	1.45	15.24	1.86
	SERVICE		13.61	1.34	14.04	1.71
CREEP			-1.16	-0.11	-1.20	-0.15
LIVE LOAD(LL+IM)	MAX.		59.30	24.73	59.30	24.73
	MIN.		-79.71	-20.16	-79.71	-20.16
+TEMP.	7.00	1.00·TG	0.86	-5.06	0.86	-5.06
	3.50	0.50·TG	0.43	-2.53	0.43	-2.53
-TEMP.	-2.00	1.00·TG	-0.24	1.45	-0.24	1.45
	-1.00	0.50·TG	-0.12	0.72	-0.12	0.72
WS	→	0.30·WS	0.34	0.49	-0.34	-0.49
	←	0.30·WS	-0.34	-0.49	0.34	0.49
WL	→	1.00·WL	0.63	0.92	-0.63	-0.92
	←	1.00·WL	-0.63	-0.92	0.63	0.92
AFTER ANCHOR SET			5.59	-14.83	6.13	-14.24
PERMANENT LOAD			-23.12	-10.54	-21.24	-6.84
SLS	Service-1	SE1- 1	37.59	16.32	37.52	17.21
		SE1- 2	-101.98	-31.83	-102.05	-30.94
		SE1- 3	35.64	13.51	39.46	20.02
		SE1- 4	-103.92	-34.64	-100.10	-28.12
		SE1- 5	-23.02	-15.12	-21.82	-12.39
		SE1- 6	-23.70	-16.09	-21.15	-11.41
		MAX.	37.59	16.32	39.46	20.02
		MIN.	-103.92	-34.64	-102.05	-30.94
	Service-3	SE3- 1	24.75	9.97	26.63	13.67
		SE3- 2	-87.01	-29.20	-85.13	-25.50
		SE3- 3	-23.36	-15.60	-21.49	-11.90
		MAX.	24.75	9.97	26.63	13.67
		MIN.	-87.01	-29.20	-85.13	-25.50



3. 1. 2. PRE-STRESS

1) PRE-STRESS IMMEDIATELY AFTER ANCHOR SET

TYPE : STRAND CABLE

STRESSING : ONE SIDE

$$\sigma pti = \sigma pt0 \times e^{-(\lambda \cdot l + \mu \cdot \alpha)}$$

$\sigma pt0$ = 1400.0 Mpa

λ = 0.001

μ = 0.250

STRESS IMMEDIATELY AFTER ANCHOR SET AT INFLICTION POINTS

POINTS	l (m)	α (Rad)	λ·l	μ·α	λ·l+μ·α	e−(λ·l+μ·α)	σpti (Mpa)	ANCHOR SET(Mpa)
1	0.000	0.0000	0.0000	0.0000	0.0000	1.0000	1400.00	1252.75
2	1.852	0.0000	0.0012	0.0000	0.0012	0.9988	1398.29	1254.46
3	2.032	0.0180	0.0013	0.0045	0.0058	0.9942	1391.84	1260.91
4	4.087	0.0180	0.0027	0.0045	0.0072	0.9928	1389.95	1262.80
5	4.298	0.0391	0.0028	0.0098	0.0126	0.9875	1382.45	1270.30
6	5.431	0.0391	0.0036	0.0098	0.0134	0.9867	1381.42	1271.33
7	6.274	0.0602	0.0041	0.0150	0.0192	0.9810	1373.39	1279.36
8	7.117	0.0813	0.0047	0.0203	0.0250	0.9753	1365.41	1287.34
9	8.251	0.0813	0.0054	0.0203	0.0258	0.9746	1364.39	1288.36
10	8.461	0.1024	0.0056	0.0256	0.0312	0.9693	1357.03	1295.72
11	10.515	0.1024	0.0069	0.0256	0.0325	0.9680	1355.19	1297.56
12	10.697	0.1205	0.0071	0.0301	0.0372	0.9635	1348.89	1303.87
13	12.531	0.1205	0.0083	0.0301	0.0384	0.9623	1347.25	1305.50

STRESS IMMEDIATELY AFTER ANCHOR SET AT DESIGN SECTION

SEC.	X (m)	ANCHOR SET(Mpa)
2	3.012	1261.75
5	3.415	1262.12
9	5.165	1271.03
11	6.344	1279.36
13	7.522	1287.64
17	9.272	1296.38
20	9.675	1296.74

No

COMPOSITE STRESS AT CENTROID OF TENDONS

$\sigma cp1 = \sigma cpt + \sigma cpd0 + \sigma cp2p + 1/2(\eta - 1) \cdot \sigma cp2p$ (ASSUMED η = 0.92)

SEC.	σ cpt kN/m2	σ cpdo kN/m2	σ cp2p kN/m2	σ cp1 kN/m2
2	2981.5	-519.0		2462.5
5	2982.4	-373.7	-222.8	2394.7
9	2940.4	-26.2	-206.7	2715.6
11	2821.2	0.0	0.0	2821.2
13	2979.1	-24.4	-208.9	2754.1
17	3063.9	-368.7	-228.6	2475.6
20	3064.7	-513.2		2551.6

PRESTRESSING LOSS DUE TO CREEP AND SHRINKAGE

$$\Delta \sigma p\phi = \frac{n \cdot \phi 1 \cdot \sigma cp1 + Ep \cdot \varepsilon s}{1 + n \cdot \sigma cpt \cdot (1+\rho 1 \cdot \phi 1) / \sigma pt} + \frac{n \cdot \phi 2 \cdot \sigma cd1}{1 + n \cdot \sigma cpt \cdot (1+\rho 2 \cdot \phi 2) / \sigma pt}$$

$$= \Delta \sigma p\phi 1 + \Delta \sigma p\phi 2$$

$\phi 1 = 2.20$ $\phi 2 = 1.00$ $n = 6.10$ $\rho 1 = 0.70$ $\rho 2 = 0.70$

$Ep = 0.2 \times 10^6$ Mpa、 $\varepsilon s = 25.0 \times 10^{-5}$

SEC.	σ pt Mpa	σ cpt Mpa	σ cp1 Mpa	σ cd1 Mpa	Δ σ p φ 1 Mpa	Δ σ p φ 2 Mpa	Δ σ p φ Mpa
2	1253.65	2.98	2.46	-0.60	79.37	-3.55	75.82
5	1254.02	2.98	2.39	-0.18	78.49	-1.07	77.42
9	1262.93	2.94	2.72	-0.06	82.71	-0.35	82.36
11	1271.26	2.82	2.82	0.00	84.21	0.00	84.21
13	1279.54	2.98	2.75	-0.03	83.21	-0.16	83.05
17	1288.28	3.06	2.48	-0.09	79.54	-0.55	78.99
20	1288.64	3.06	2.55	-0.49	80.52	-2.93	77.59

PRESTRESSING LOSS DUE TO RELAXATION

$\Delta \sigma p\gamma = \gamma \cdot \sigma pt$ (γ = 1.50 %)

SEC.	σ pt Mpa	Δ σ p γ Mpa
2	1253.65	18.80
5	1254.02	18.81
9	1262.93	18.94
11	1271.26	19.07
13	1279.54	19.19
17	1288.28	19.32
20	1288.64	19.33

No

STRESS IMMEDIATELY AFTER ANCHOR SET AT DESIGN SECTION

$\sigma pt = \sigma pt' - \Delta \sigma p$

σ pt′ : TENSILE STRESS AT DESIGN
Δ σ p : ELASTIC SHORTNING

$$Pt = \frac{1.0\ m}{0.75\ m} \cdot Ap \cdot \sigma pt$$

$Ap = 416.1\ mm^2$

SEC.	σ pt′ Mpa	Δ σ p Mpa	σ pt Mpa	Pt kN
2	1261.75	8.10	1253.65	695.523
5	1262.12	8.10	1254.02	695.729
9	1271.03	8.10	1262.93	700.671
11	1279.36	8.10	1271.26	705.294
13	1287.64	8.10	1279.54	709.889
17	1296.38	8.10	1288.28	714.740
20	1296.74	8.10	1288.64	714.940

2) PRE-STRESS AT SERVICE

STRESS AFTER ANCHOR SET AT CENTROID OF TENDONS

$$\sigma cpt = \frac{Pt}{A} + \frac{Pt \cdot ep}{Zp}$$

SEC.	Pt kN	A m2	e p m	Zp m3	σ cpt kN/m2
2	695.523	0.550	0.185	0.0749	2981.5
5	695.729	0.550	0.185	0.0749	2982.4
9	700.671	0.250	0.016	0.0814	2940.4
11	705.294	0.250	0.000	0.0000	2821.2
13	709.889	0.250	0.016	0.0814	2979.1
17	714.740	0.550	0.185	0.0749	3063.9
20	714.940	0.550	0.185	0.0749	3064.7

STRESS DUE TO LOAD AT CENTROID OF TENDONS

SEC.	Zp m3	Mdo kN·m	Md1 kN·m	M2p kN·m	σ cpdo kN/m2	σ cd1 kN/m2	σ cp2p kN/m2
2	0.0749	-38.897	-44.703		-519.0	-596.5	
5	0.0749	-28.004	-13.467	-16.695	-373.7	-179.7	-222.8
9	0.0814	-2.131	-4.736	-16.825	-26.2	-58.2	-206.7
11	0.0000	2.196	-2.118	-16.913	0.0	0.0	0.0
13	0.0814	-1.982	-2.124	-17.000	-24.4	-26.1	-208.9
17	0.0749	-27.632	-6.976	-17.130	-368.7	-93.1	-228.6
20	0.0749	-38.459	-36.907		-513.2	-492.5	

3) PRE-STRESS AT EXTREMELY FIBER

$$\left.\begin{matrix}\sigma_{ptu}\\ \sigma_{ptl}\end{matrix}\right\} = \frac{Pt}{A} \pm \frac{Pt \cdot ep}{Z}$$

$$\left.\begin{matrix}\sigma_{peu}\\ \sigma_{pel}\end{matrix}\right\} = \eta \times \left\{\begin{matrix}\sigma_{ptu}\\ \sigma_{ptl}\end{matrix}\right.$$

Pt = STRESSING FORCE IMMEDATELY AFTER ANCHOR SET(kN)

A = AREA (m^2)

Z = I / yc (m^3)

ep = EXCENTRCITY (m)

η = EFFECTIVE RATIO

σ_{ptu}、σ_{ptl} = PRE-STRESS AT EXTREMELY FIBER AFTER ANCHOR SET (Mpa)

σ_{ptu}、σ_{ptl} = PRE-STRESS AT EXTREMELY FIBER IN SERVICE (Mpa)

SEC.		A m2	Z m3	Pt kN	ep m	η	ANCHOR SET Mpa	AT SERVICE Mpa
2	TOP.	0.550	0.0504	695.523	0.185	0.925	3.82	3.53
	BOT.						-1.29	-1.19
5	TOP.	0.550	0.0504	695.729	0.185	0.923	3.82	3.52
	BOT.						-1.29	-1.19
9	TOP.	0.250	0.0104	700.671	0.016	0.920	3.88	3.57
	BOT.						1.73	1.59
11	TOP.	0.250	0.0104	705.294	0.000	0.919	2.82	2.59
	BOT.						2.82	2.59
13	TOP.	0.250	0.0104	709.889	0.016	0.920	3.93	3.62
	BOT.						1.75	1.61
17	TOP.	0.550	0.0504	714.740	0.185	0.924	3.92	3.62
	BOT.						-1.32	-1.22
20	TOP.	0.550	0.0504	714.940	0.185	0.925	3.92	3.63
	BOT.						-1.32	-1.22

EFFECTIVE STRESS AT SERVICE

$\sigma_{pe} = \sigma_{pt} - \Delta\sigma_{p\phi} - \Delta\sigma_{p\gamma}$

$\eta = \sigma_{pe} / \sigma_{pt}$

$\bar{\eta} = 0.922$

SEC.	σ_{pt} Mpa	$\Delta\sigma_{p\phi}$ Mpa	$\Delta\sigma_{p\gamma}$ Mpa	σ_{pe} Mpa	ηi
2	1253.65	75.82	18.80	1159.02	0.925
5	1254.02	77.42	18.81	1157.78	0.923
9	1262.93	82.36	18.94	1161.62	0.920
11	1271.26	84.21	19.07	1167.97	0.919
13	1279.54	83.05	19.19	1177.29	0.920
17	1288.28	78.99	19.32	1189.97	0.924
20	1288.64	77.59	19.33	1191.72	0.925

3.1.3.COMPOSITE STRESS

SEC.		STRESS DUE TO LOAD						PRE-STRESS		COMPOSITE STRESS					
		ANCHOR SET	PERMA NENT	SERVICE-1		SERVICE-3		ANCHOR SET	SERVICE	ANCHOR SET	PERMA NENT	SERVICE-1		SERVICE-3	
				MAX	MIN	MAX	MIN					MAX	MIN	MAX	MIN
2	TOP.	-0.77	-1.66	-1.57	-3.81	-1.66	-3.31	3.82	3.53	3.05	1.87	1.96	-0.28	1.87	0.22
	BOT.	0.77	1.66	1.57	3.81	1.66	3.31	-1.29	-1.19	-0.52	0.47	0.38	2.62	0.47	2.12
5	TOP.	-0.89	-1.13	-1.08	-2.74	-1.14	-2.37	3.82	3.52	2.93	2.40	2.45	0.78	2.38	1.15
	BOT.	0.89	1.13	1.08	2.74	1.14	2.37	-1.29	-1.19	-0.40	-0.06	-0.11	1.56	-0.04	1.18
9	TOP.	-1.82	-2.15	-2.10	-2.36	-2.23	-2.31	3.88	3.57	2.06	1.42	1.47	1.21	1.34	1.26
	BOT.	1.82	2.15	2.10	2.36	2.23	2.31	1.73	1.59	3.55	3.74	3.69	3.95	3.82	3.90
11	TOP.	-1.41	-1.49	4.61	-1.65	3.40	-1.65	2.82	2.59	1.41	1.10	7.21	0.94	5.99	0.94
	BOT.	1.41	1.49	-4.61	1.65	-3.40	1.65	2.82	2.59	4.23	4.08	-2.02	4.24	-0.81	4.24
13	TOP.	-1.82	-1.90	-1.85	-2.11	-1.98	-2.06	3.93	3.62	2.11	1.72	1.77	1.50	1.64	1.55
	BOT.	1.82	1.90	1.85	2.11	1.98	2.06	1.75	1.61	3.57	3.51	3.46	3.72	3.59	3.67
17	TOP.	-0.89	-1.00	-0.95	-2.62	-1.02	-2.24	3.92	3.62	3.03	2.62	2.67	1.01	2.61	1.38
	BOT.	0.89	1.00	0.95	2.62	1.02	2.24	-1.32	-1.22	-0.44	-0.22	-0.27	1.39	-0.21	1.02
20	TOP.	-0.76	-1.49	-1.40	-3.65	-1.49	-3.14	3.92	3.63	3.16	2.13	2.23	-0.02	2.13	0.49
	BOT.	0.76	1.49	1.40	3.65	1.49	3.14	-1.32	-1.22	-0.56	0.27	0.18	2.42	0.27	1.92

ALLOWABLE STRESS

TENSILE STRESS LIMIT $0.50 \cdot \sqrt{fc'} = 0.5x\sqrt{45.0} = -3.35$ Mpa

COMPRESSIVE STRESS LIMIT $0.40 \cdot fc' = 0.4x\ 45.0 = 18.0$ Mpa

No

No

CRACK CONTROL

$T = 1/2 \cdot \sigma ct \cdot b \cdot x$
$reqAs = T / \sigma sa$

T	: SECTIONAL TENSILE FORCE (kN)
σct	: TENSILE STRESS AT EXTREME FIBER
b	: UNIT WIDTH (m)
reqAs	: REQUIRED REINFORCEMENT
σsa	: TENSILE STRESS OF REINFORCEMENT FOR CRACK ALLOWABLE
x	: $\lvert\sigma cu\rvert / (\lvert\sigma cu\rvert + \lvert\sigma cl\rvert) \cdot h$
h	: MEMBER THICKNESS
σcu、σcl	: CONCRETE STRESS AT EXTREME FIBER

ANCHOR SET

SEC.	σcu Mpa	σcl Mpa	h m	x m	T kN	σsa Mpa	reqAs mm2	ARRENGED REBAR mm2
2	3.05	-0.52	0.550	0.0797	20.57	200.00	102.8	
5	2.93	-0.40	0.550	0.0662	13.29	200.00	66.5	
17	3.03	-0.44	0.550	0.0690	15.02	200.00	75.1	
20	3.16	-0.56	0.550	0.0829	23.23	200.00	116.2	

PERMANENT

SEC.	σcu Mpa	σcl Mpa	h m	x m	T kN	σsa Mpa	reqAs mm2	ARRENGED REBAR mm2
5	2.40	-0.06	0.550	0.0137	0.42	195.00	2.2	
17	2.62	-0.22	0.550	0.0430	4.79	195.00	24.6	

SERVICE-Ⅰ (LL-MAX)

SEC.	σcu Mpa	σcl Mpa	h m	x m	T kN	σsa Mpa	reqAs mm2	ARRENGED REBAR mm2
5	2.45	-0.11	0.550	0.0241	1.35	195.00	6.9	
11	7.21	-2.02	0.250	0.0548	55.35	195.00	283.9	
17	2.67	-0.27	0.550	0.0510	6.98	195.00	35.8	

SERVICE-Ⅰ (LL-MIN)

SEC.	σcu Mpa	σcl Mpa	h m	x m	T kN	σsa Mpa	reqAs mm2	ARRENGED REBAR mm2
2	-0.28	2.62	0.550	0.0533	7.50	195.00	38.4	
20	-0.02	2.42	0.550	0.0041	0.04	195.00	0.2	

PC-STEEL STRES

$\sigma pmax = \sigma pe + \Delta \sigma p$

$\Delta \sigma p = -n \cdot (\sigma cpd1 + \sigma cp2p + \sigma cpl)$

$\sigma pmax$	: PC-STEEL STRESS AT LL-LOADING
$\Delta \sigma$	: ADDITIONAL PC-STEEL STRESS DUE TO LOAD
n	: ELASTIC RATIO OF CONCRETE TO PC-STEEL
$\sigma cpd1$	: CONCRETE STRESS AT CENTROID OF PS-STEEL DUE TO SURFACE LOADING
$\sigma cpd1$	: CONCRETE STRESS FOR DIFFERENCE OF SECONDARY FORCE DUE TO PRE-STRESSING LOSS AT CENTROID OF PC-STEEL
σcpl	: CONCRETE STRESS AT CENTROID OF PC-STEEL DUE TO LL-LOADING
Zp	: I/y AT CENTROID OF PC-STEEL

$\sigma cpi = Mi / Zp$

SEC.	Md1 kN·m	ΔM2p kN·m	Ml kN·m	Zp m3	σ cpd1 Mpa	σ cp2p Mpa	σ cpl Mpa	Σ σ cp Mpa	n	Δ σ p Mpa	σ pe Mpa	σ pmax Mpa
5	-13.467	-1.310	-77.270	0.0749	-0.18	-0.02	-1.03	-1.23	6.10	7.49	1157.78	1165.27
11	-2.118	-1.327	63.320	0.0000	—	—	—	—	6.10	—	1167.97	—
17	-6.976	-1.344	-77.270	0.0749	-0.09	-0.02	-1.03	-1.14	6.10	6.97	1189.97	1196.94

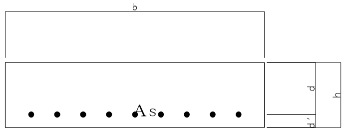
SERVICE-Ⅲ (MAZ)

SEC.	σ cu Mpa	σ cl Mpa	h m	x m	T kN	σ sa Mpa	reqAs mm2	ARRENGED REBAR mm2
5	2.38	-0.04	0.550	0.0101	0.22	195.00	1.2	
11	5.99	-0.81	0.250	0.0296	11.94	195.00	61.2	
17	2.61	-0.21	0.550	0.0402	4.14	195.00	21.2	

No

3．2．2． STRESS CHECK

CRACK CONTROL COMPUTED AS RC SECTION



REBAR STRESS

$$\sigma s = \frac{M}{As \cdot j \cdot d}$$

CONCRETE STRESS

$$\sigma c = \frac{\sigma s}{m}$$

ELASTIC RATIO n = 12.00

SECTION			A － A	B － B
MOMENT	M	kN·m	31.52	33.06
DIMENSION	B	mm	1000.0	1000.0
	d’	mm	65.0	62.0
	d	mm	185.0	243.1
REBARS	Asreq	mm2	808.0	628.0
	As	mm2	D16ctc125	D14ctc125
			1608.0	1232.0
CIE	100·p		0.009	0.005
	k		0.364	0.293
	j		0.879	0.902
	m		20.950	28.927
	σ s	Mpa	120.60	122.35
ALLOWABLE CRACK	d c	mm	50.000	50.000
	ctc	mm	125.000	125.000
	A	mm2	12500.0	12500.0
	Z	N/mm	23000.0	23000.0
	Z/(dc·A)**1/3	Mpa	269.0	269.0
	0.6·fsy	Mpa	240.0	240.0
	fsa	Mpa	240.0	240.0
	JUDGE	-	OK	OK

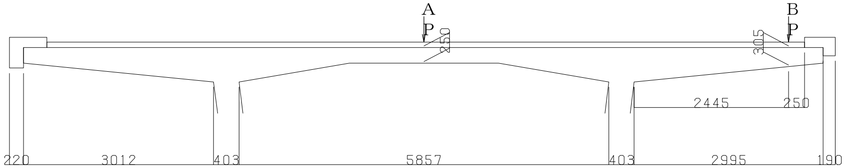
fsa = Z / (dc·A)**1/3 ≦ 0.6·fsy

fsa : LIMIT STRESS FOR REBAR
dc : DEPTH OF CONCRETE MEASURED FROM TENTION FIBER (LESS THAN 50mm)
A : AREA OF CONCRETE FOR A TENSILE REBAR(mm2)
fsy : YIELD STRESS OF REBAR (400.0 Mpa)
Z : CRACK WIDTH PARAMETER (N/mm)

Asreq = M / (fsa·j·d)

No

3．2.DESIGN FOR LONGITUDINAL DIRECTION



CALCULATION AS RC COMPONENT IS PERFORMED FOR THE SECTION SHOW ABOVE.

3． 2． 1． SECTIONAL FORCE

LIVE LOAD IS CONSIDERED AND PERMANENT LOAD IS DISREGARDED.

SECTION A (CENTER OF UPPER SLAB)

SPAN LENGTH l =5.857 m COEFFICIENT K =1.000

MA = = 31.520 kN·m

SECTION B (LOADING POINT OF AXLE FORCE)

SPAN LENGTH l =2.445 m COEFFICIENT K =1.000

MB = 33.060 kN·m

3. 2. 3. CHECK FOR MINIMUM REINFORCEMENT

REINFORCEMENT FOR SHRINKAGE AND TEMPERATURE

(1) UPPER SLAB OF BOX (LONGITUDINAL)

Asreq① = 0.75·Ag / fy / L

= 0.75 × 1989191.8 / 400.0 / 5.8568

= 636.8 mm2/m

Asreq② = 0.0015·Ag / L

= 0.0015 × 1989191.8 / 5.8568

= 509.5 mm2/m

(2) OVERHANG(L) OF BOX (LONGITUDINAL)

Asreq① = 0.75·Ag / fy / L

= 0.75 × 1204800.0 / 400.0 / 3.0120

= 750.0 mm2/m

Asreq② = 0.0015·Ag / L

= 0.0015 × 1204800.0 / 3.0120

= 600.0 mm2/m

(3) OVERHANG(R) OF BOX (LONGITUDINAL)

Asreq① = 0.75·Ag / fy / L

= 0.75 × 1198000.0 / 400.0 / 2.9950

= 750.0 mm2/m

Asreq② = 0.0015·Ag / L

= 0.0015 × 1198000.0 / 2.9950

= 600.0 mm2/m

(4) BOTTOM SLAB (TRANSVERSE)

Asreq = 0.005·Ag / L

= 0.005 × 1534084.4 / 5.7880

= 1325.2 mm2/m

(5) WEB (LONGITUDINAL)

Asreq① = 0.75·Ag / fy / L

= 0.75 × 1198465.2 / 400.0 / 2.9730

= 755.8 mm2/m

Asreq② = 0.0015·Ag / L

= 0.0015 × 1198465.2 / 2.9730

= 604.7 mm2/m

(6) BOTTOM SLAB (TRANSVERSE)

Asreq = 0.005·Ag / L

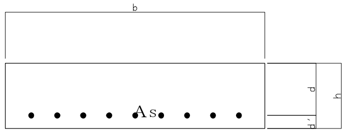
= 0.005 × 1534084.4 / 5.7880

= 1325.2 mm2/m

No

BOTTOM SLAB (SERVICE)

CRACK CONTROL COMPUTED AS RC SECTION



REBAR STRESS

$$\sigma s = \frac{M}{As \cdot j \cdot d}$$

CONCRETE STRESS

$$\sigma c = \frac{\sigma s}{m}$$

ELASTIC RATIO $n = 12.00$

SECTION			26 MAX.	23 MIN.
MOMENT	M	kN·m	67.10	-27.24
DIMENSION	B	mm	1000.0	1000.0
	d’	mm	50.0	45.0
	d	mm	475.0	480.0
REBARS	Asreq	mm2	641.5	257.6
	As	mm2	D16ctc125	D16ctc125
			1608.0	1608.0
CIE	100·p		0.003	0.003
	k		0.247	0.246
	j		0.918	0.918
	m		36.525	36.744
	σs	Mpa	95.74	38.45
ALLOWABLE CRACK	dc	mm	50.000	45.000
	ctc	mm	125.000	125.000
	A	mm2	12500.0	11250.0
	Z	N/mm	23000.0	23000.0
	Z/(dc·A)**1/3	Mpa	269.0	288.6
	0.6·fsy	Mpa	240.0	240.0
	fsa	Mpa	240.0	240.0
	JUDGE	-	OK	OK

fsa = Z / (dc·A)**1/3 ≦ 0.6·fsy

fsa : LIMIT STRESS FOR REBAR
dc : DEPTH OF CONCRETE MEASURED FROM TENTION FIBER (LESS THAN 50mm)
A : AREA OF CONCRETE FOR A TENSILE REBAR(mm2)
fsy : YIELD STRESS OF REBAR (400.0 Mpa)
Z : CRACK WIDTH PARAMETER (N/mm)

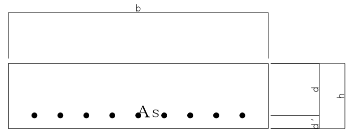
Asreq = M / (fsa·j·d)

No

4. STRESS CHECK FOR BOTTOM SLAB AND WEB

4.1 BOTTOM SLAB

CRACK CONTROL COMPUTED AS RC SECTION



REBAR STRESS

$$\sigma s = \frac{M}{As \cdot j \cdot d}$$

CONCRETE STRESS

$$\sigma c = \frac{\sigma s}{m}$$

ELASTIC RATIO $n = 12.00$

SECTION			26 MAX.	23 MIN.
MOMENT	M	kN·m	39.83	-4.12
DIMENSION	B	mm	1000.0	1000.0
	d’	mm	50.0	45.0
	d	mm	475.0	480.0
REBARS	Asreq	mm2	380.7	38.9
	As	mm2	D16ctc125	D16ctc125
			1608.0	1608.0
CIE	100·p		0.003	0.003
	k		0.247	0.246
	j		0.918	0.918
	m		36.525	36.744
	σs	Mpa	56.83	5.81
ALLOWABLE CRACK	dc	mm	50.000	45.000
	ctc	mm	125.000	125.000
	A	mm2	12500.0	11250.0
	Z	N/mm	23000.0	23000.0
	Z/(dc·A)**1/3	Mpa	269.0	288.6
	0.6·fsy	Mpa	240.0	240.0
	fsa	Mpa	240.0	240.0
	JUDGE	-	OK	OK

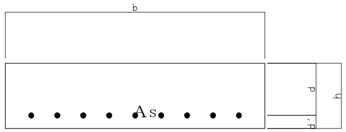
fsa = Z / (dc·A)**1/3 ≦ 0.6·fsy

fsa : LIMIT STRESS FOR REBAR
dc : DEPTH OF CONCRETE MEASURED FROM TENTION FIBER (LESS THAN 50mm)
A : AREA OF CONCRETE FOR A TENSILE REBAR(mm2)
fsy : YIELD STRESS OF REBAR (400.0 Mpa)
Z : CRACK WIDTH PARAMETER (N/mm)

Asreq = M / (fsa·j·d)

WEB (SERVICE)

CRACK CONTROL COMPUTED AS RC SECTION



REBAR STRESS

$$\sigma s = \frac{M}{As \cdot j \cdot d}$$

CONCRETE STRESS

$$\sigma c = \frac{\sigma s}{m}$$

ELASTIC RATIO n = 12.00

SECTION			38 MAX.	38 MIN.
MOMENT	M	kN·m	39.46	-102.05
DIMENSION	B	mm	1000.0	1000.0
	d'	mm	50.0	45.0
	d	mm	350.0	355.0
REBARS	Asreq	mm2	518.4	1320.9
	As	mm2	D16ctc125	D16ctc125
			1608.0	1608.0
CIE	100·p		0.005	0.005
	k		0.281	0.280
	j		0.906	0.907
	m		30.633	30.887
	σs	Mpa	77.38	197.15
ALLOWABLE CRACK	dc	mm	50.000	45.000
	ctc	mm	125.000	125.000
	A	mm2	12500.0	11250.0
	Z	N/mm	23000.0	23000.0
	Z/(dc·A)**1/3	Mpa	269.0	288.6
	0.6·fsy	Mpa	240.0	240.0
	fsa	Mpa	240.0	240.0
	JUDGE	-	OK	OK

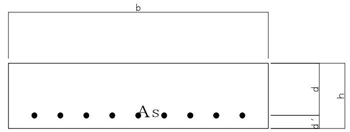
fsa = Z / (dc·A)**1/3 ≦ 0.6·fsy

fsa : LIMIT STRESS FOR REBAR
dc : DEPTH OF CONCRETE MEASURED FROM TENTION FIBER (LESS THAN 50mm)
A : AREA OF CONCRETE FOR A TENSILE REBAR(mm2)
fsy : YIELD STRESS OF REBAR (400.0 Mpa)
Z : CRACK WIDTH PARAMETER (N/mm)

Asreq = M / (fsa·j·d)

4.2 WEB

CRACK CONTROL COMPUTED AS RC SECTION



REBAR STRESS

$$\sigma s = \frac{M}{As \cdot j \cdot d}$$

CONCRETE STRESS

$$\sigma c = \frac{\sigma s}{m}$$

ELASTIC RATIO n = 12.00

SECTION			42 MAX.	32 MIN.
MOMENT	M	kN·m	-6.84	-23.12
DIMENSION	B	mm	1000.0	1000.0
	d'	mm	45.0	45.0
	d	mm	355.0	355.0
REBARS	Asreq	mm2	88.5	299.2
	As	mm2	D16ctc125	D16ctc125
			1608.0	1608.0
CIE	100·p		0.005	0.005
	k		0.280	0.280
	j		0.907	0.907
	m		30.887	30.887
	σs	Mpa	13.21	44.66
ALLOWABLE CRACK	dc	mm	45.000	45.000
	ctc	mm	125.000	125.000
	A	mm2	11250.0	11250.0
	Z	N/mm	23000.0	23000.0
	Z/(dc·A)**1/3	Mpa	288.6	288.6
	0.6·fsy	Mpa	240.0	240.0
	fsa	Mpa	240.0	240.0
	JUDGE	-	OK	OK

fsa = Z / (dc·A)**1/3 ≦ 0.6·fsy

fsa : LIMIT STRESS FOR REBAR
dc : DEPTH OF CONCRETE MEASURED FROM TENTION FIBER (LESS THAN 50mm)
A : AREA OF CONCRETE FOR A TENSILE REBAR(mm2)
fsy : YIELD STRESS OF REBAR (400.0 Mpa)
Z : CRACK WIDTH PARAMETER (N/mm)

Asreq = M / (fsa·j·d)

5. STRENGTH CHECK

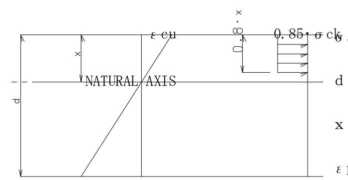
Ⅰ) BENDING MOMENT AT STRENGTH LIMIT STATE

LOAD COMBINATIONS

			1	2	3	4	5	6	7	8	9	10	11				
			DC	DW	EL	CR	SH	LL+IM	PL	TG	WS	WL	CT				
S		ST1-1	1.25	1.50	1.00	0.50		1.75	1.75					Lmax	-		1
T	Strength-1	ST1-2	1.25	1.50	1.00	0.50		1.75	1.75					Lmin	-		2
R		ST1-3	0.90	0.65	1.00	0.50		1.75	1.75					Lmax	-		3
E		ST1-4	0.90	0.65	1.00	0.50		1.75	1.75					Lmin	-		4
N		ST2-1	1.25	1.50	1.00	0.50		1.35	1.35					Lmax	-		5
G	Strength-2	ST2-2	1.25	1.50	1.00	0.50		1.35	1.35					Lmin	-		6
T		ST2-3	0.90	0.65	1.00	0.50		1.35	1.35					Lmax	-		7
H		ST2-4	0.90	0.65	1.00	0.50		1.35	1.35					Lmin	-		8
		ST3-1	1.25	1.50	1.00	0.50					1.40			Wind	R		9
L	Strength-3	ST3-2	1.25	1.50	1.00	0.50					-1.40			Wind	L		10
I		ST3-3	0.90	0.65	1.00	0.50					1.40			Wind	R		11
M		ST3-4	0.90	0.65	1.00	0.50					-1.40			Wind	L		12
I	Strength-4	ST4-1	1.50	1.50	1.00	0.50								D	-		13
T		ST5-1	1.25	1.50	1.00	0.50		1.35	1.35		0.40	1.00		Lx+W	R		14
		ST5-2	1.25	1.50	1.00	0.50		1.35	1.35		0.40	1.00		Ln+W	R		15
S		ST5-3	1.25	1.50	1.00	0.50		1.35	1.35		-0.40	-1.00		Lx+W	L		16
T	Strength-5	ST5-4	1.25	1.50	1.00	0.50		1.35	1.35		-0.40	-1.00		Ln+W	L		17
A		ST5-5	0.90	0.65	1.00	0.50		1.35	1.35		0.40	1.00		Lx+W	R		18
T		ST5-6	0.90	0.65	1.00	0.50		1.35	1.35		0.40	1.00		Ln+W	R		19
E		ST5-7	0.90	0.65	1.00	0.50		1.35	1.35		-0.40	-1.00		Lx+W	L		20
		ST5-8	0.90	0.65	1.00	0.50		1.35	1.35		-0.40	-1.00		Ln+W	L		21

Ⅱ) STRENGTH SECTIO

DISTRIBUTION OF STRAIN AND STRESS ON CALCULATION OF STRENGTH SECTION



STRAIN DISTRIBUTION   STRESS DISTRIBUTION

ε_{cu} : ULTIMATE STRAIN OF CONCRETE (=0.0030)

ε_s : STRAIN OF REINFORCEMENT

σ_{ck} : CONCRETE STRENGTH (Mpa)

σ_s : STRESS OF REINFORCEMENT (Mpa)

d : EFFECTIVE HEIGHT OF SECTION (mm)

x : DISTANCE FROM COMPRESSIVE EXTREME FIBER TO NATURAL AXIS (mm)N

ε_{pe} : STRAIN OF TENDON DUE TO FEECTIVE PRESTRESS

FORCE BALANCING

$$C_u = T_u$$

$$C_u = 0.8\cdot x\cdot b\cdot 0.85\cdot \sigma_{ck} = 0.680\cdot x\cdot \sigma_{ck}$$

b : WIDTH OF COMPRESSION FLANGE

COMPATIBILITY CONDITION OF STAIN

$$\frac{x}{\varepsilon_{cu}} = \frac{d - x}{\varepsilon_s - \varepsilon_{pe}}$$

RESISTANCE OF SECTION

$$M_r = T_u\cdot(d - 0.4\cdot x)$$

$$T_u = A_s\cdot \sigma_s$$

a) REINFORCEMENT BAR

$E_s\cdot\varepsilon_s < \sigma_{sy}$ ·············· $\sigma_s = E_s\cdot\varepsilon_s$

$E_s\cdot\varepsilon_s \geqq \sigma_{sy}$ ·············· $\sigma_s = \sigma_{sy}$

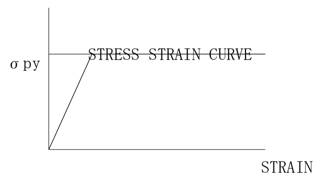
b) PC CABLE

$E_{ps}\cdot\varepsilon_s < 0.84\cdot\sigma_{pu}$ ······· $\sigma_s = E_{ps}\cdot\varepsilon_s$

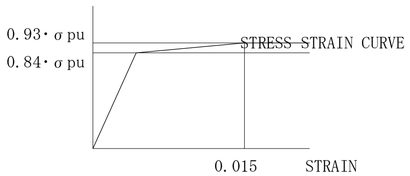
$0.84\cdot\sigma_{pu} \leqq E_{ps}\cdot\varepsilon_s < 0.93\cdot\sigma_{pu}$ ·····

$$\sigma_s = 0.84\cdot\sigma_{pu} + \frac{\varepsilon_s - \frac{0.84\cdot\sigma_{pu}}{E_p}}{0.015 - \frac{0.84\cdot\sigma_p}{E_{ps}}}\cdot 0.09\cdot\sigma_{pu}$$

$\varepsilon_s \geqq 0.015$ ·················· $\sigma_s = 0.93\cdot\sigma_{pu}$



REBAR



PC CABLE

Mr : NOMINAL RESISTANCE OF SECTION

σ_{sy} : YIELD STRENGTH OF REBAR (σ_{sy} = 400 M

σ_{pu} : YIELD STRENGTH OF PC CABLE (σ_{pu} = 1860 Mpa)

σ_s : STRESS OF REBAR (Mpa)

Es : MODULUS OF ELASTICITY OF REINFORCEMENT (Mpa)

ε_s : STRAIN OF REINFORCEMENT

As : ALL AREA OF TENSILE(mm^2)FORCEMENT (mm2)

d : EFFECTIVE HEIGHT OF SECTION (mm)

Ⅲ) SEFETY DEGREE FOR BENDING MOMENT STRENGTH

$$F_s = \frac{M_r}{M_u}$$

No

STRENGTH RESISTANCE AND RATIO

Mn = Aps·fps·(dp-a/2) + As·fy·(ds-a/2) - As'·fy'·(ds'-a/2)

Aps = ctc 750.0 = 554.8 mm2
As = D14 ctc 125.0 = 1232.0 mm2
As' = D14 ctc 125.0 = 1232.0 mm2

fps = fpu·(1-k·c/dp)

k = 2·(1.04-fpy/fpu)

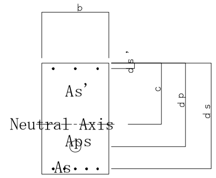
$$c = \frac{Aps \cdot fpu + As \cdot fy - As' \cdot fy}{0.85 \cdot fc' \cdot \beta 1 \cdot b + k \cdot Aps \cdot fpu/dp}$$

a = β1·c

β1 = 0.85 - (0.05·(fc'-28)/7)

$$\phi = 0.9 + 0.10 \cdot \frac{Aps \cdot fpy}{Aps \cdot fpy + As \cdot fy}$$

fpu = 1860.0 Mpa
fpy = 1674.0 Mpa
fy = 400.0 Mpa
fc' = 45.0 Mpa



			2	5	11	17	20
EFFECTIVE HEIGHT	dp	mm	460.0	460.0	125.0	460.0	460.0
	ds	mm	485.0	485.0	185.0	485.0	485.0
	ds'	mm	65.0	65.0	65.0	65.0	65.0
PC-STEEL STRESS	β1		0.7286	0.7286	0.7286	0.7286	0.7286
	k		0.2800	0.2800	0.2800	0.2800	0.2800
	a	mm	26.384	26.384	24.912	26.384	26.384
	fps	Mpa	1819.0	1819.0	1717.5	1819.0	1819.0
STRENGTH RESISTANCE	c	mm	36.2	36.2	34.2	36.2	36.2
MOMENT AT STRENGTH LIMIT STATE	Mu	kN·m	-297.4	-206.5	95.2	-196.7	-285.2
NOMINAL FLEXURAL STRENGTH	Mn	kN·m	657.9	657.9	166.4	657.9	657.9
RESISTANCE FACTER	φ		0.965	0.965	0.965	0.965	0.965
	φ·Mn	kN·m	635.1	635.1	160.6	635.1	635.1
RESISTANCE	Fs		2.135	3.076	1.688	3.229	2.227

No

5.1 STRENGTH CHECK FOR TOP SLAB

5.1.1 MEMBER REINFORCED STRESSING STEEL

			2	5	9	11	13	17	20
GIRDER-SELF LOAD			-38.90	-28.00	-2.13	2.20	-1.98	-27.63	-38.46
SUPERIMPOSED			-44.70	-13.47	-4.74	-2.12	-2.12	-6.98	-36.91
SECONDARY MOMENT	CONST.			-16.69	-16.83	-16.91	-17.00	-17.13	
	SERVICE			-15.39	-15.51	-15.59	-15.67	-15.79	
CREEP				1.31	1.32	1.33	1.33	1.34	
LIVE LOAD(LL+IM)	MAX.					63.32			
	MIN.		-103.86	-77.27				-77.27	-103.86
WS	→	1.00·WS	5.36	3.96	1.59	0.00	-1.59	-3.96	-5.36
	←	1.00·WS	-5.36	-3.96	-1.59	0.00	1.59	3.96	5.36
WL	→	1.00·WL	3.01	2.23	0.90	0.00	-0.90	-2.23	-3.01
	←	1.00·WL	-3.01	-2.23	-0.90	0.00	0.90	2.23	3.01
STRENGTH LIMIT STATE	Strength-1	ST1- 1	-115.68	-71.25	-25.93	94.13	-22.00	-61.46	-103.43
		ST1- 2	-297.43	-206.47	-25.93	-16.68	-22.00	-196.69	-285.19
		ST1- 3	-64.06	-50.00	-21.16	95.16	-19.50	-45.86	-58.60
		ST1- 4	-245.82	-185.22	-21.16	-15.65	-19.50	-181.08	-240.36
		MAX.	-64.06	-50.00	-21.16	95.16	-19.50	-45.86	-58.60
		MIN.	-297.43	-206.47	-25.93	-16.68	-22.00	-196.69	-285.19
	Strength-2	ST2- 1	-115.68	-71.25	-25.93	68.80	-22.00	-61.46	-103.43
		ST2- 2	-255.89	-175.56	-25.93	-16.68	-22.00	-165.78	-243.65
		ST2- 3	-64.06	-50.00	-21.16	69.83	-19.50	-45.86	-58.60
		ST2- 4	-204.28	-154.31	-21.16	-15.65	-19.50	-150.18	-198.81
		MAX.	-64.06	-50.00	-21.16	69.83	-19.50	-45.86	-58.60
		MIN.	-255.89	-175.56	-25.93	-16.68	-22.00	-165.78	-243.65
	Strength-3	ST3- 1	-108.17	-65.70	-23.70	-16.68	-24.23	-67.01	-110.94
		ST3- 2	-123.18	-76.79	-28.16	-16.68	-19.77	-55.92	-95.93
		ST3- 3	-56.56	-44.45	-18.93	-15.65	-21.73	-51.40	-66.11
		ST3- 4	-71.57	-55.54	-23.39	-15.65	-17.27	-40.32	-51.10
		MAX.	-56.56	-44.45	-18.93	-15.65	-17.27	-40.32	-51.10
		MIN.	-123.18	-76.79	-28.16	-16.68	-24.23	-67.01	-110.94
	Strength-4	ST4- 1	-125.40	-78.25	-26.47	-16.13	-22.49	-68.37	-113.05
	Strength-5	ST5- 1	-110.52	-67.44	-24.40	68.80	-23.53	-65.27	-108.59
		ST5- 2	-250.73	-171.75	-24.40	-16.68	-23.53	-169.59	-248.80
		ST5- 3	-120.83	-75.05	-27.47	68.80	-20.46	-57.65	-98.28
		ST5- 4	-261.04	-179.37	-27.47	-16.68	-20.46	-161.97	-238.49
		ST5- 5	-58.91	-46.19	-19.63	69.83	-21.03	-49.67	-63.76
		ST5- 6	-199.12	-150.50	-19.63	-15.65	-21.03	-153.99	-203.97
		ST5- 7	-69.22	-53.81	-22.69	69.83	-17.96	-42.05	-53.45
		ST5- 8	-209.43	-158.12	-22.69	-15.65	-17.96	-146.37	-193.66
		MAX.	-58.91	-46.19	-19.63	69.83	-17.96	-42.05	-53.45
		MIN.	-261.04	-179.37	-27.47	-16.68	-23.53	-169.59	-248.80

5．2 STRENGTH CHECK FOR BOTTOM SLAB AND WEB

COLLECTION TABLE (WEB kN·m)

			23	26	29	32	36	38	42
CASE 4			-1.99	13.24	28.47	-50.79	-6.87	-28.92	24.73
CASE 5			26.50	67.10	30.13	37.59	16.32	39.46	20.02
CASE 6			28.47	13.24	-1.99	-28.92	24.73	-50.79	-6.87
CASE 7			-29.95	20.93	-26.33	-103.92	-34.64	-102.05	-30.94
CASE 8			-19.70	-19.70	-19.70	59.30	-13.29	59.30	-13.29
CASE 9			19.45	61.81	23.08	24.75	9.97	26.63	13.67
GIRDER-SELF LOAD			-11.71	30.51	-11.53	-9.18	-16.28	-9.10	-16.10
SUPERIMPOSED			7.33	8.88	10.42	-27.55	4.41	-26.18	7.55
SECONDARY MOMENT	CONST.		0.29	0.48	0.67	14.77	1.45	15.24	1.86
	SERVICE		0.26	0.44	0.61	13.61	1.34	14.04	1.71
CREEP			-0.02	-0.04	-0.05	-1.16	-0.11	-1.20	-0.15
LIVE LOAD(LL+IM)	MAX.		28.47	26.48	28.47	59.30	24.73	59.30	24.73
	MIN.		-21.69	-19.70	-21.69	-79.71	-20.16	-79.71	-20.16
WS	→ 0.30·WS		0.47	0.00	-0.47	0.34	0.49	-0.34	-0.49
	← 0.30·WS		-0.47	0.00	0.47	-0.34	-0.49	0.34	0.49
WL	→ 1.00·WL		0.88	0.00	-0.88	0.63	0.92	-0.63	-0.92
	← 1.00·WL		-0.88	0.00	0.88	-0.63	-0.92	0.63	0.92
STRENGTH LIMIT STATE	Strength-1	ST1- 1	46.45	98.25	51.68	65.17	30.93	67.77	36.27
		ST1- 2	-41.32	17.44	-36.09	-178.10	-47.63	-175.50	-42.30
		ST1- 3	44.31	80.02	46.85	91.81	32.89	93.21	35.49
		ST1- 4	-43.45	-0.79	-40.91	-151.47	-45.68	-150.06	-43.08
		MAX.	46.45	98.25	51.68	91.81	32.89	93.21	36.27
		MIN.	-43.45	-0.79	-40.91	-178.10	-47.63	-175.50	-43.08
	Strength-2	ST2- 1	35.06	87.65	40.29	41.45	21.04	44.05	26.37
		ST2- 2	-32.65	25.32	-27.41	-146.22	-39.57	-143.62	-34.23
		ST2- 3	32.93	69.43	35.47	68.08	22.99	69.49	25.59
		ST2- 4	-34.78	7.09	-32.24	-119.58	-37.61	-118.18	-35.01
		MAX.	35.06	87.65	40.29	68.08	22.99	69.49	26.37
		MIN.	-34.78	7.09	-32.24	-146.22	-39.57	-143.62	-35.01
	Strength-3	ST3- 1	-1.17	51.91	-0.34	-37.03	-10.07	-37.59	-9.30
		ST3- 2	-5.57	51.91	4.06	-40.19	-14.64	-34.43	-4.73
		ST3- 3	-3.30	33.69	-5.16	-10.40	-8.11	-12.15	-10.08
		ST3- 4	-7.70	33.69	-0.76	-13.56	-12.68	-8.99	-5.51
		MAX.	-1.17	51.91	4.06	-10.40	-8.11	-8.99	-4.73
		MIN.	-7.70	33.69	-5.16	-40.19	-14.64	-37.59	-10.08
	Strength-4	ST4- 1	-6.30	59.54	-1.02	-40.90	-16.42	-38.28	-11.04
	Strength-5	ST5- 1	36.57	87.65	38.78	42.54	22.61	42.96	24.80
		ST5- 2	-31.13	25.32	-28.93	-145.13	-38.00	-144.70	-35.80
		ST5- 3	33.55	87.65	41.80	40.37	19.47	45.14	27.94
		ST5- 4	-34.16	25.32	-25.90	-147.30	-41.14	-142.53	-32.66
		ST5- 5	34.44	69.43	33.95	69.17	24.56	68.40	24.02
		ST5- 6	-33.27	7.09	-33.75	-118.50	-36.05	-119.26	-36.58
		ST5- 7	31.42	69.43	36.98	67.00	21.42	70.58	27.16
		ST5- 8	-36.29	7.09	-30.73	-120.67	-39.18	-117.09	-33.44
		MAX.	36.57	87.65	41.80	69.17	24.56	70.58	27.94
		MIN.	-36.29	7.09	-33.75	-147.30	-41.14	-144.70	-36.58

5．1．2 DESIGN FOR LONGITUDINAL (RC COMPONENT)

BENDING MOMENT AT STRENGTH LIMIT STATE

A - A SECTION (BOX CENTER OF SPAN)

M1 = 31.520 (kN·m)

Mu = 1.75×31.520 = 78.800 (kN·m)

B - B SECTION (NEAR TIP OF OVERHANG)

M1 = 33.060 (kN·m)

Mu = 1.75×33.060 = 82.650 (kN·m)

STRENGTH RESISTANCE AND RATIO

Mn = As·fy·(d-a/2)

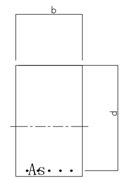
a = As·fy/(0.85·fc'·b)

fy = 400.0 Mpa

fc' = 45.0 Mpa

Asreq = 1/fy·(X-√(X^2-2·(X/d)·(Mu/ϕ)))

X = 0.85·fc'·b·d



		A-A	B-B
Asreq	mm2	1225.6	964.4
As	mm2	D16ctc125	D14ctc125
		1608.0	1232.0
d	mm	185.0	243.1
Mn	kN·m	113.58	116.62
ϕ		0.900	0.900
ϕ·Mn	kN·m	102.23	104.96
Mu	〃	78.80	82.65
Fs		1.297	1.270

No

MINIMUM REINFORCEMENT OF FLAXURAL COMPONENT (UPPER SLAB)

Mr AT LEAST EQUAL TO THE LESSER OF ;
1.2 TIMES THE CRACKING STRENGTH (1.2Mcr)
1.33 TIMES THE FACTORED MOMENT (1.33Mu)

$Mn \geqq 1.20 \cdot Mcr$
OR
$Mn \geqq 1.33 \cdot Mu$

$Mcr = Sc \cdot (fr + fcpe) - Mdnc \cdot (Sc/Snc - 1)$

Mn : NOMINAL BENDING STRENGTH (Mn/ϕ)
Mcr : CRACKING MOMENT
Mu : MOMENT AT STRENGTH LIMIT STATE
fr : MODULUS OF RUPTURE
$fr = 0.63 \cdot \sqrt{(fc')}$
fc' : CONCRETE STRENGTH
Sc : SECTION COEFFICIENT
Sc/Snc : NON COMPOSIT >>> 1

$$fcpe = \frac{Np \cdot Pe}{Ac} + \frac{Np \cdot ep \cdot Pe}{Sc}$$

		2	5	11	17	20
b	m	1.000	1.000	1.000	1.000	1.000
d	m	0.550	0.550	0.250	0.550	0.550
Sc	m3	0.050	0.050	0.010	0.050	0.050
fr	Mpa	4.23	4.23	4.23	4.23	4.23
fcpe	Mpa	-1.19	-1.19	2.59	-1.22	-1.22
Mcr	kN·m	153.1	153.1	71.0	151.5	151.4
1.20·Mcr	kN·m	183.7	183.7	85.2	181.7	181.6
Mu	kN·m	-297.4	-206.5	95.2	-196.7	-285.2
1.33·Mu	kN·m	-395.6	-274.6	126.6	-261.6	-379.3
Mn	kN·m	657.9	657.9	166.4	657.9	657.9
Fs		3.582	3.581	1.952	3.620	3.622

No

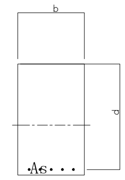
STRENGTH RESISTANCE AND RATIO

$Mn = As \cdot fy \cdot (d - a/2)$

$a = As \cdot fy / (0.85 \cdot fc' \cdot b)$

fy = 400.0 Mpa

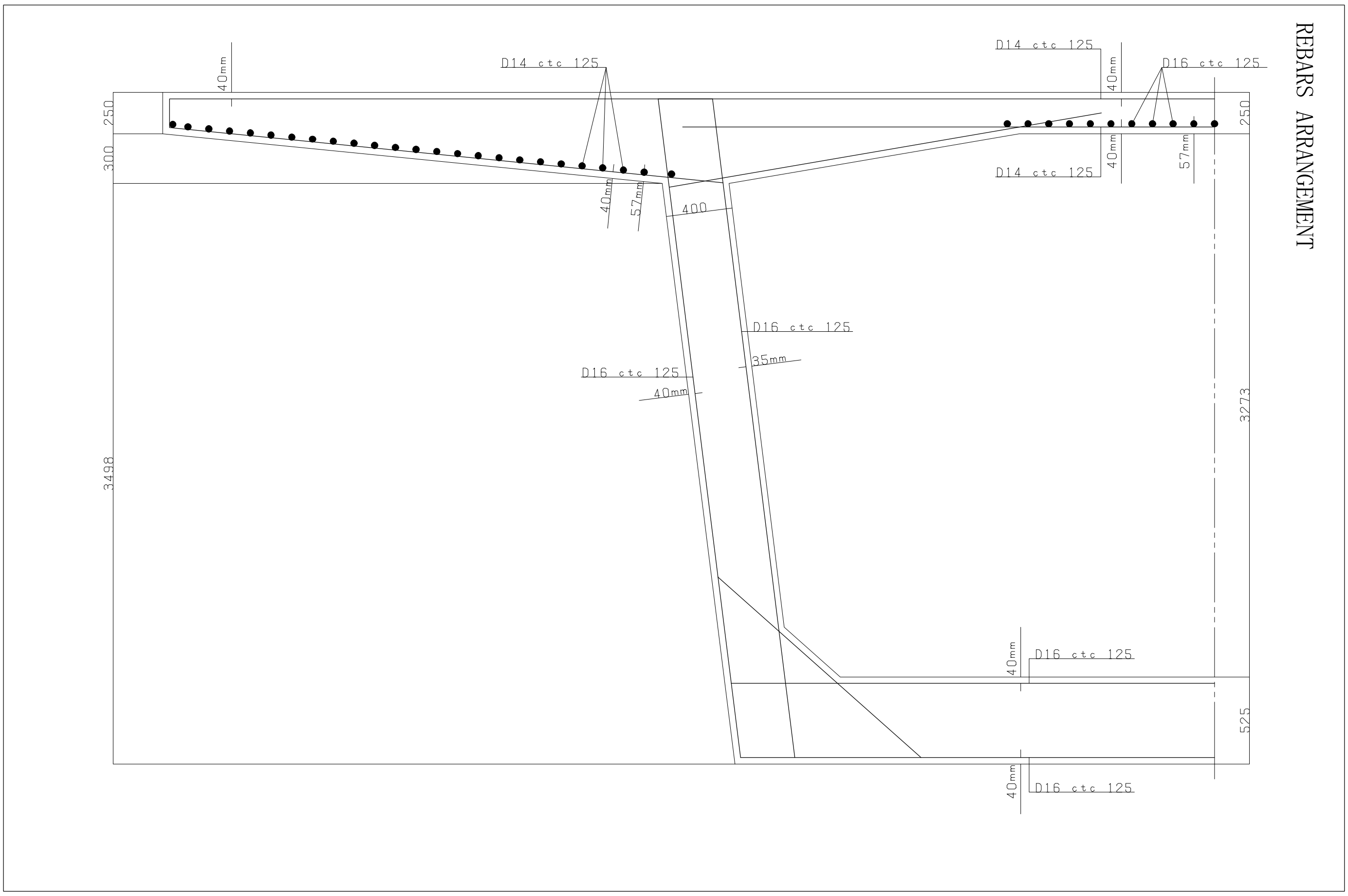
fc' = 45.0 Mpa



$Asreq = 1/fy \cdot (X - \sqrt{(X^2 - 2 \cdot (X/d) \cdot (Mu/\phi))})$

$X = 0.85 \cdot fc' \cdot b \cdot d$

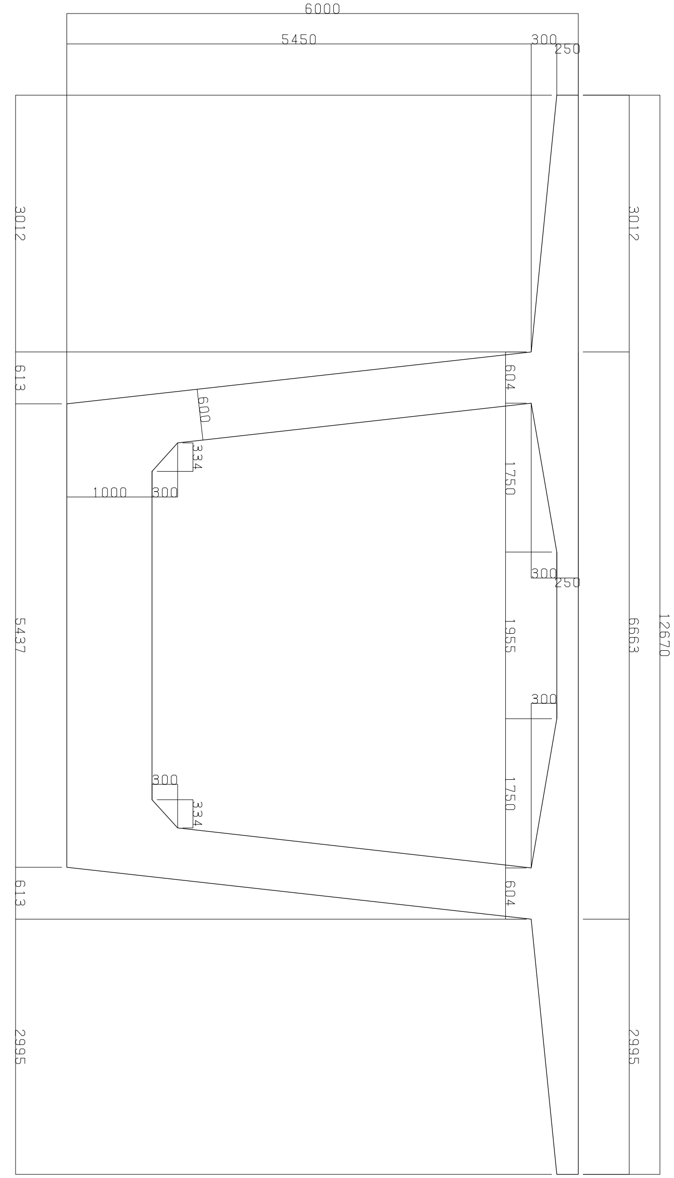
		BOTTOM SLAB		WEB	
		29 AT WEB	26 MID SPAN	42 OUTSIDE	32 INSIDE
Asreq	mm2	300.0	578.2	343.7	1423.4
As	mm2	D16ctc125	D16ctc125	D16ctc125	D16ctc125
		1608.0	1608.0	1608.0	1608.0
d	mm	480.0	475.0	350.0	355.0
Mn	kN·m	303.33	300.11	219.71	222.93
ϕ		0.900	0.900	0.900	0.900
ϕ·Mn	kN·m	273.00	270.10	197.74	200.64
Mu	〃	51.68	98.25	-43.08	-178.10
Fs		5.283	2.749	4.590	1.127


REBARS ARRANGEMENT
D14 ctc 125
D14 ctc 125
D16 ctc 125
D14 ctc 125
40mm
40mm
57mm
400
40mm
57mm
D16 ctc 125
35mm
D16 ctc 125
40mm
250
300
3498
250
3273
D16 ctc 125
40mm
525
D16 ctc 125
40mm
No

4. 5. 3. Section 3(Hg=6.0m,Hfu=0.25m,Hfl=1.00m,Bw=1.00m)

No

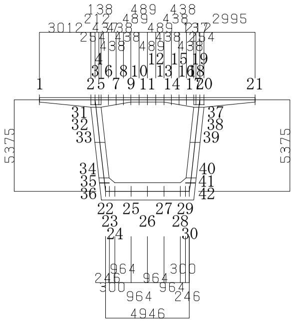
1. CROSS SECTION



No

2. ANALYSIS FOR SECTIONAL FORCE

2. 1. CENTROID AND PROPERTY



MEMBER	A (m2)	I (m4)	X (m)	Y (m)	MAT
1- 2	0.400	0.00758	3.012	0.000	1
2- 3	50.000	50.00000	0.254	0.000	1
3- 4	50.000	50.00000	0.212	0.000	1
4- 5	0.550	0.01386	0.138	0.000	1
5- 6	0.512	0.01140	0.438	0.000	1
6- 7	0.438	0.00713	0.438	0.000	1
7- 8	0.363	0.00410	0.438	0.000	1
8- 9	0.287	0.00208	0.438	0.000	1
9- 10	0.250	0.00130	0.489	0.000	1
10- 11	0.250	0.00130	0.489	0.000	1
11- 12	0.250	0.00130	0.489	0.000	1
12- 13	0.250	0.00130	0.489	0.000	1
13- 14	0.287	0.00208	0.438	0.000	1
14- 15	0.363	0.00410	0.438	0.000	1
15- 16	0.438	0.00713	0.438	0.000	1
16- 17	0.512	0.01140	0.438	0.000	1
17- 18	0.550	0.01386	0.138	0.000	1
18- 19	50.000	50.00000	0.212	0.000	1
19- 20	50.000	50.00000	0.254	0.000	1
20- 21	0.400	0.00758	2.995	0.000	1
22- 23	1.100	0.11092	0.246	0.000	1

MEMBER	A (m2)	I (m4)	X (m)	Y (m)	MAT
23- 24	1.050	0.09713	0.300	0.000	1
24- 25	1.000	0.08333	0.964	0.000	1
25- 26	1.000	0.08333	0.964	0.000	1
26- 27	1.000	0.08333	0.964	0.000	1
27- 28	1.000	0.08333	0.964	0.000	1
28- 29	1.050	0.09713	0.300	0.000	1
29- 30	1.100	0.11092	0.246	0.000	1
3- 31	50.000	50.00000	0.031	0.273	1
31- 32	0.600	0.01800	0.017	0.152	1
32- 33	0.600	0.01800	0.233	2.075	1
33- 34	0.600	0.01800	0.233	2.075	1
34- 35	0.600	0.01800	0.034	0.300	1
35- 36	0.600	0.01800	0.000	0.003	1
36- 22	50.000	50.00000	0.056	0.497	1
19- 37	50.000	50.00000	0.031	0.273	1
37- 38	0.600	0.01800	0.017	0.152	1
38- 39	0.600	0.01800	0.233	2.075	1
39- 40	0.600	0.01800	0.233	2.075	1
40- 41	0.600	0.01800	0.034	0.300	1
41- 42	0.600	0.01800	0.000	0.003	1
42- 30	50.000	50.00000	0.056	0.497	1

SUPPORT CONDITION

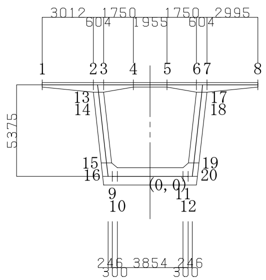
No.	CONDITION
22	H
30	Rv

MATERIAL

No.	MODULUS ELASTICITY	THERMAL EXPANSION
1	0.3220E+05	0.000010

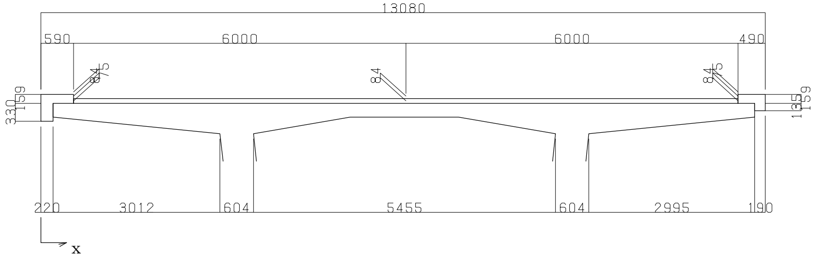
2. 2. LOADING

2. 2. 1. GIRDER-SELF LOAD



SEC.	X (m)	Y (m)	h (m)	W(kN/m)	P(kN)
1	-6.344	5.375	0.250	6.125	
2	-3.332	5.375	0.550	13.475	
3	-2.728	5.375	0.550	13.475	
4	-0.978	5.375	0.250	6.125	
5	0.978	5.375	0.250	6.125	
6	2.728	5.375	0.550	13.475	
7	3.332	5.375	0.550	13.475	
8	6.327	5.375	0.250	6.125	
9	-2.227	0.000	1.270	31.107	
10	-1.927	0.000	1.000	24.500	
11	1.927	0.000	1.000	24.500	
12	2.227	0.000	1.270	31.107	
13	-3.077	5.375	0.600	14.700	1.849
14	-3.030	4.950	0.600	14.700	
14	-3.030	4.950	0.600	14.700	
15	-2.563	0.800	0.600	14.700	
15	-2.563	0.800	0.588	14.411	
16	-2.473	0.000	0.588	14.411	6.363
17	3.077	5.375	0.600	14.700	1.849
18	3.030	4.950	0.600	14.700	
18	3.030	4.950	0.600	14.700	
19	2.563	0.800	0.600	14.700	
19	2.563	0.800	0.588	14.411	
20	2.473	0.000	0.588	14.411	6.363

2. 2. 2. SUPERIMPOSED



ITEMS	LOCATION x (m)	UNIT(kN/m3)	P(kN)	Q(kN/m)
EDGE BLOCK(L)	0.110	24.500	1.779	
PARAPET(L)	0.295	24.500	2.298	
HANDRAIL(L)	0.214		8.550	
PAVEMENT(L)	0.590	22.500		1.890
PAVEMENT(MID)	6.590	22.500		1.890
PAVEMENT(R)	12.590	22.500		1.890
HANDRAIL(R)	12.915		7.447	
PARAPET(R)	12.835	24.500	1.909	
EDGE BLOCK(R)	12.985	24.500	0.628	

WIND LOAD→

W1L = 5.360
W1R = 5.360

WUND LOAD←

W2L = 5.360
W2R = 5.360

WIND LOAD→

W3L = 1.500× (1.800+ 0.125+ 0.084) = 3.013 kN·m
W3R = 1.500× (1.800+ 0.125+ 0.084) = 3.013 kN·m

WUND LOAD←

W4L = 1.500× (1.800+ 0.125+ 0.084) = 3.013 kN·m
W4R = 1.500× (1.800+ 0.125+ 0.084) = 3.013 kN·m

2．2．3．SECONDARY FORCE DUE TO PRE-STRESS

1) AXIAL FORCE AND MOMENT DUE TO PRE-STRESS IMMEDIATELY AFTER ANCHOR SET

PRE-STRESS AFTER SET $Pt = \sigma pt \cdot Ap / P$

ECCENTRICITY MOMENT $Mpt = Pt \cdot ep$

$\sigma pt'$: TENSIL STRESS AT EACH SECTION

$\Delta\sigma p$: ELASTIC SHORTNING

σpt : TENDON STRESS IMMEDIATELY AFTER ANCHOR SET

Ap : AREA 416.100 mm^2

P : SPACING 0.750 m

ep : ECCENTRICITY

SEC.	σpt' Mpa	Δσp Mpa	σpt Mpa	Pt kN	ep m	Mpt kN·m	ΔMpt kN·m	ΔL m	ΔMpt/ΔL kN	P kN
							149.553			
3	1261.98	8.20	1253.78	695.597	0.215	149.553	-12.499	0.212	-58.906	58.906
4	1262.18	8.20	1253.98	695.705	0.197	137.054	-8.336	0.138	-60.622	1.716
5	1262.30	8.20	1254.10	695.776	0.185	128.718	-25.711	0.438	-58.767	-1.855
6	1262.70	8.20	1254.50	695.998	0.148	103.008	-29.479	0.438	-67.381	8.613
7	1270.41	8.20	1262.21	700.274	0.105	73.529	-32.900	0.438	-75.200	7.820
8	1270.81	8.20	1262.61	700.495	0.058	40.629	-32.220	0.438	-73.646	-1.554
9	1271.21	8.20	1263.01	700.717	0.012	8.409	-6.301	0.489	-12.888	-60.758
10	1274.70	8.20	1266.50	702.657	0.003	2.108	-2.108	0.489	-4.312	-8.576
11	1279.36	8.20	1271.16	705.238	0.000	0.000	2.123	0.489	4.344	-8.656
12	1283.98	8.20	1275.78	707.805	0.003	2.123	6.393	0.489	13.078	-8.735
13	1287.46	8.20	1279.26	709.733	0.012	8.517	32.660	0.438	74.652	-61.574
14	1287.85	8.20	1279.65	709.952	0.058	41.177	33.391	0.438	76.322	-1.669
15	1288.25	8.20	1280.05	710.170	0.105	74.568	31.158	0.438	71.219	5.102
16	1295.81	8.20	1287.61	714.367	0.148	105.726	26.472	0.438	60.507	10.712
17	1296.20	8.20	1288.00	714.585	0.185	132.198	8.588	0.138	62.462	-1.955
18	1296.33	8.20	1288.13	714.653	0.197	140.787	12.886	0.212	60.729	1.732
19	1296.52	8.20	1288.32	714.758	0.215	153.673	-153.673			60.729

2) TEMPERATURE FORCE EQUIVALENT TO SECONDARY FORCE DUE TO ELASTIC SHORTNING

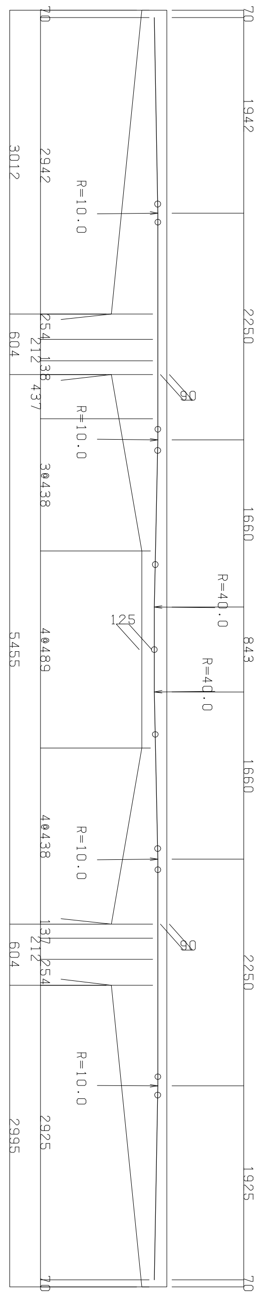
AVERAGE AREA OF TOP SLAB $A = 0.346\ m^2$

$\sigma c = Pt / A = 705.2 / 0.346 = 2036.8\ kN/m^2$

$\sigma c / Ec = 2036.8 / 28800000.0 = 7.1\times1^{0-}$

THEREFORE, 7.1℃ DIFFERENCE IS LOADED.

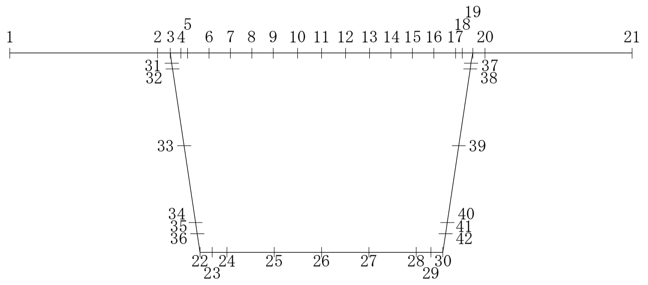
TENDON ARRANGEMENT



ep (m)	0.004	0.185	0.215	0.185	0.185	0.148	0.105	0.058	0.012	0.003	0.000	0.003	0.012	0.058	0.105	0.148	0.185	0.185	0.215	0.185	0.003
yp (m)	0.125	0.090	0.090	0.090	0.090	0.090	0.095	0.104	0.113	0.122	0.125	0.122	0.113	0.104	0.095	0.090	0.090	0.090	0.090	0.090	0.125
tu (m)	0.257	0.550	0.610	0.550	0.550	0.475	0.400	0.325	0.250	0.250	0.250	0.250	0.250	0.325	0.400	0.475	0.550	0.550	0.610	0.550	0.257

No

2. 2. 5. LOADING CASE



CASE 1 (GIRDER-SELF LOAD)

ALL MEMBERS ARE LOADED

CASE 2 (SUPERIMPOSED)

TOP SLAB 1 ～ 21 ARE LOADED

CASE 3 (SECONDARY FORCE DUE TO PRE-STRESS)

TOP SLAB 3 IS LOADED149.6 kN·m AND 58.9 kN

TOP SLAB 19 IS LOADED 153.7 kN·m AND 60.7 kN

TOP SLAB 4 ～ 18 ARE LOADED

CASE 4 (TRUCK LOAD ON CANTILEVER(L))

SEC. 3 IS LOADED MOMENT (-103.860 kN·m) DUE TO TRUCK LOAD

CASE 5 (TRUCK LOAD ON CANTILEVER(R))

SEC.19 IS LOADED MOMENT (-103.860 kN·m) DUE TO TRUCK LOAD

CASE 6 (TRUCK LOAD ON SLAB)

SEC. 3 AND 19 ARE LOADED MOMENT (-77.270 kN·m) DUE TO TRUCK LOAD

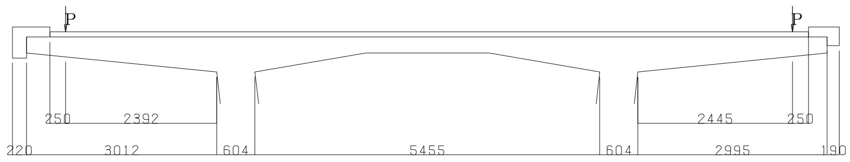
CASE 7 (TEMPERATURE GRADIENT)

TOP SLAB 3 ～19 ARE LOADED ＋ 7.0 ℃

CASE 8(WIND LOAD ON STRUCTURE(L))

No

2. 2. 4.LIVE LOAD



TRUCK LOAD ON CANTILEVER(L)

CANTILEVER LEGTH l = 2.392 m IMPACT K1 = 1.000

M = -103.860 kN·m

TRUCK LOAD ON SPAN (AT WEB)

SPAN LEGTH l = 5.455 m IMPACT K1 = 1.000

M = -77.270 kN·m

TRUCK LOAD (AT MID SPAN)

SPAN LEGTH l = 5.455 m IMPACT K1 = 1.000

M = 63.320 kN·m

TRUCK LOAD ON CANTILEVER(R)

CANTILEVER LEGTH l = 2.445 m IMPACT K1 = 1.000

M = -103.860 kN·m

2．2．6． LOAD COMBINATIONS

			1	2	3	4	5	6	7	8	9	10	11				
			DC	DW	EL	CR	SH	LL+IM	PL	TG	WS	WL	CT				
		SE1-1	1.00	1.00	1.00	1.00		1.00	1.00	0.50	0.30	1.00		Lmax	R	±	1
		SE1-2	1.00	1.00	1.00	1.00		1.00	1.00	0.50	0.30	1.00		Lmin	R	±	2
	Service-1	SE1-3	1.00	1.00	1.00	1.00		1.00	1.00	0.50	-0.30	-1.00		Lmax	L	±	3
S		SE1-4	1.00	1.00	1.00	1.00		1.00	1.00	0.50	-0.30	-1.00		Lmin	L	±	4
L		SE1-5	1.00	1.00	1.00	1.00				1.00	0.30			Wind	R	±	5
S		SE1-6	1.00	1.00	1.00	1.00				1.00	-0.30			Wind	L	±	6
		SE3-1	1.00	1.00	1.00	1.00		0.80	0.80	0.50				Lmax	-	±	7
	Service-3	SE3-2	1.00	1.00	1.00	1.00		0.80	0.80	0.50				Lmin	-	±	8
		SE3-3	1.00	1.00	1.00	1.00				1.00					-	±	9

CASE 9(WIND LOAD ON STRUCTURE(R))

CASE 10(WIND LOAD ON VEHICLE(L))

CASE 11(WIND LOAD ON VEHICLE(R))

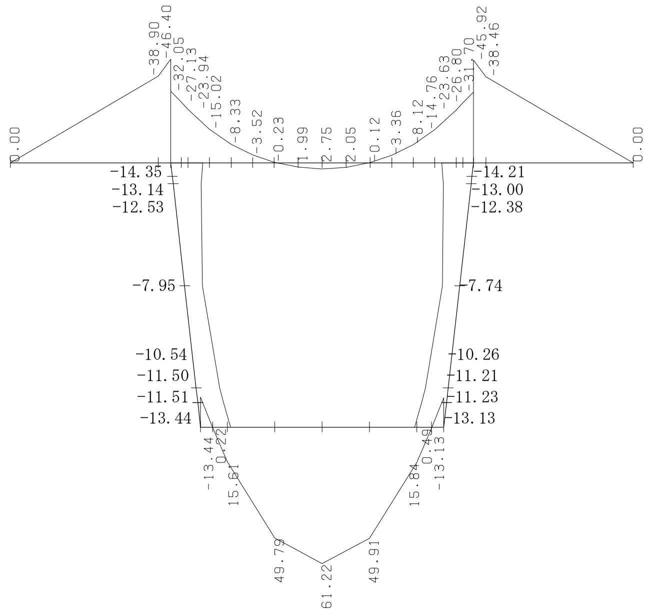
CASE 12 (TEMPERATURE GRADIENT)

TOP SLAB 3 ～19 ARE LOADED － 2.0 ℃

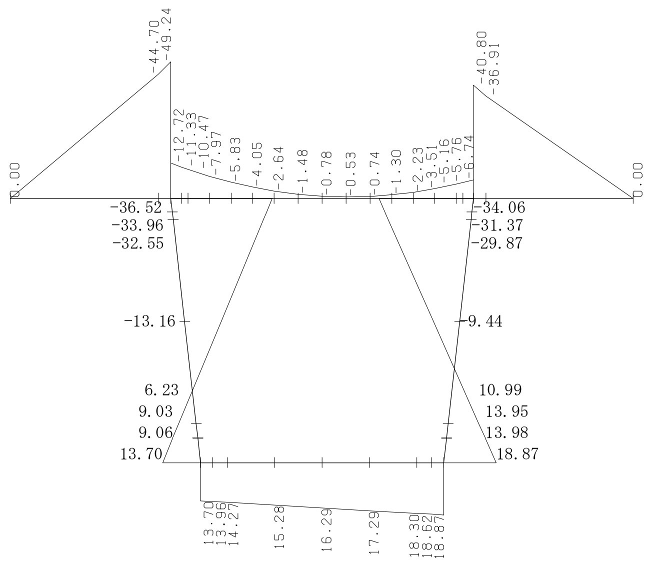
No

2. 3. BENDING MOMENT

CASE 1 (GIRDER-SELF LOAD)

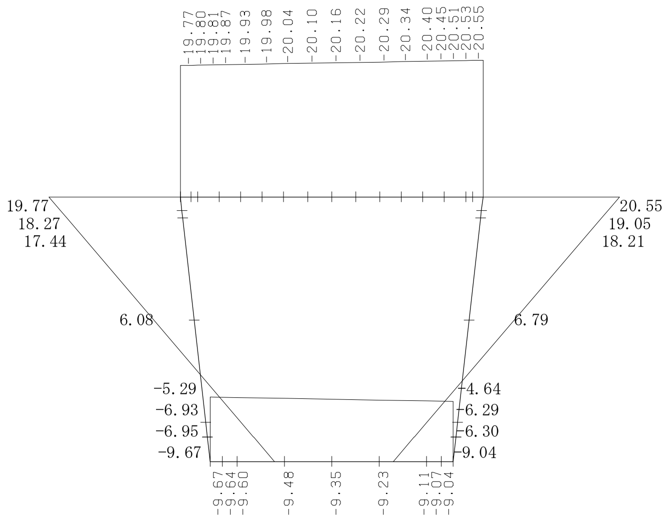


CASE 2 (SUPERIMPOSED)

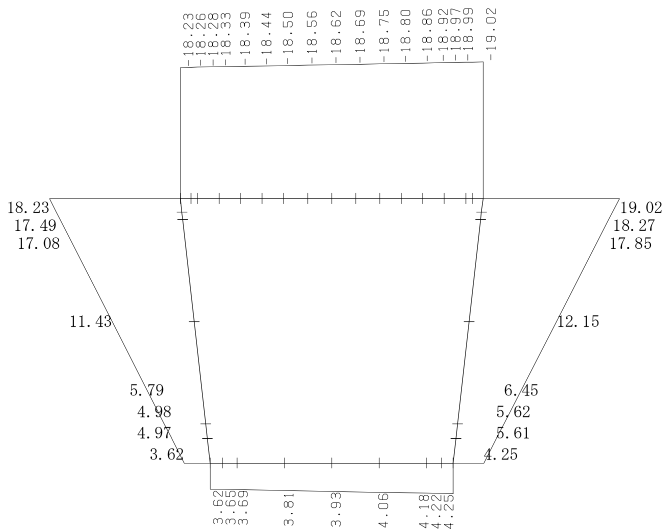


No

CASE 3-1 (MOMENT DUE TO PRESTRES)

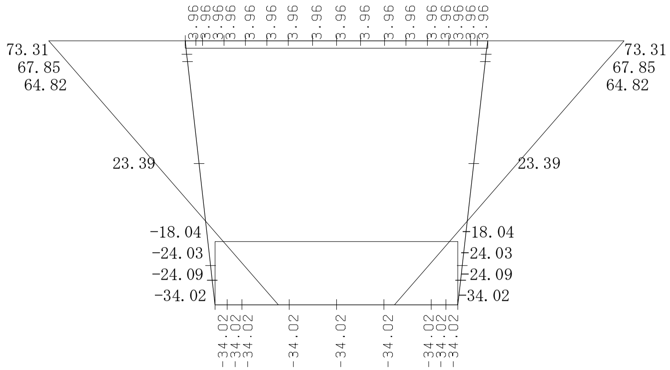


CASE 3-2 (SECONDARY MOMENT DUE TO PRESTRESS - ECCENTRICITY+ELASTIC SHORTNING)

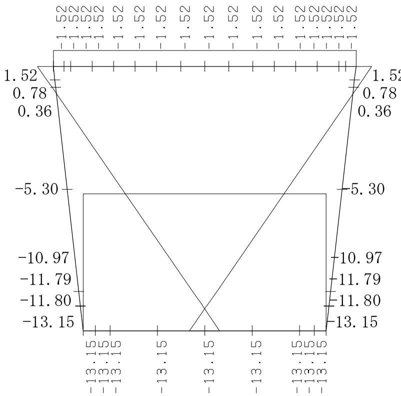


No

CASE 6 (TRUCK LOAD ON SPAN)

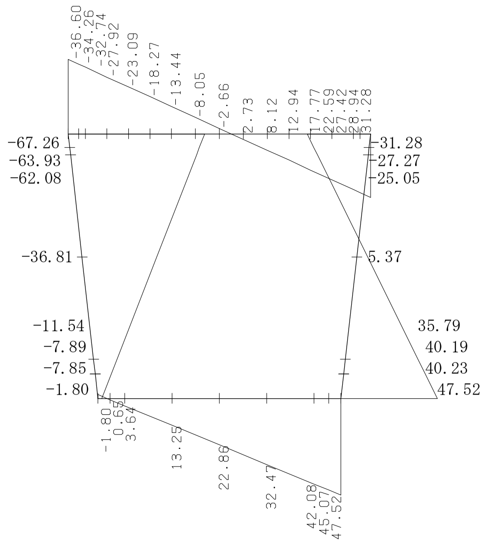


CASE 7 (TEMPERATURE GRADIENT)

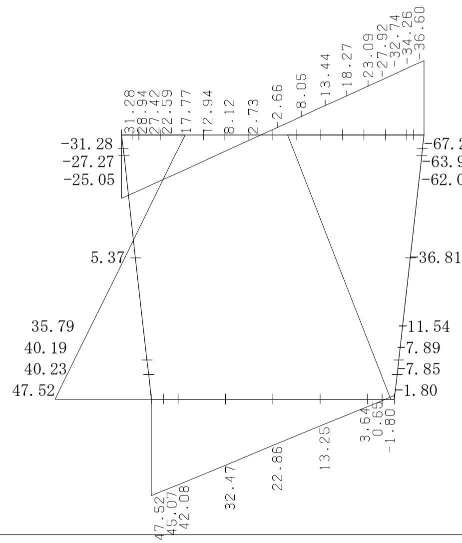


No

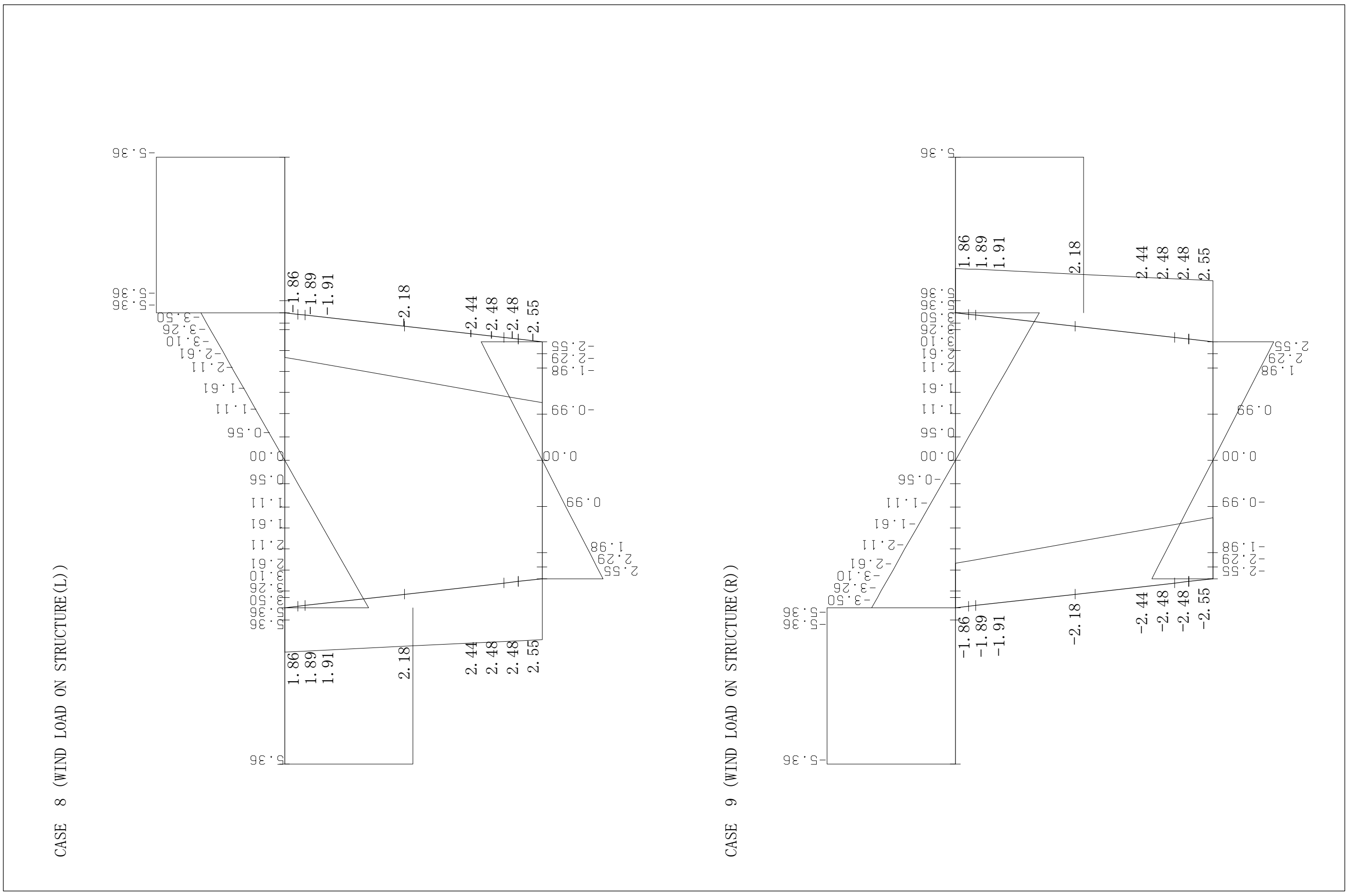
CASE 4 (TRUCK LOAD ON CANTILEVER(L))



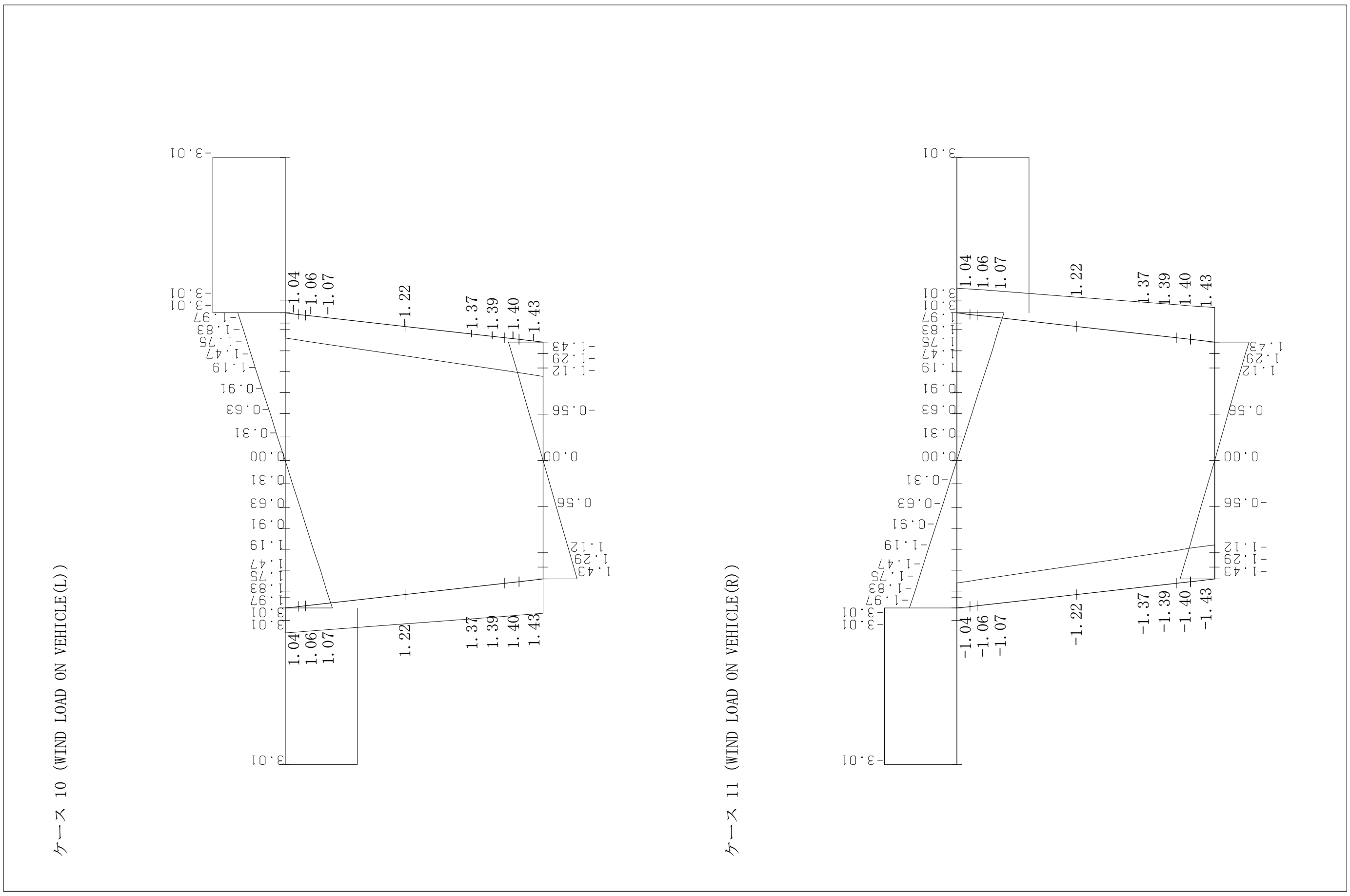
CASE 5 (TRUCK LOAD ON CANTILEVER(R))



No


CASE 8 (WIND LOAD ON STRUCTURE(L))
CASE 9 (WIND LOAD ON STRUCTURE(R))

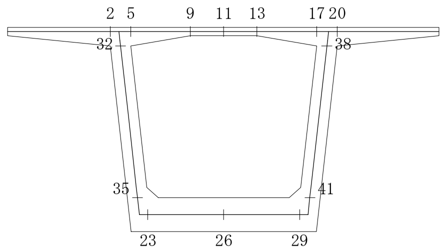
No


ケース 10 (WIND LOAD ON VEHICLE(L))
ケース 11 (WIND LOAD ON VEHICLE(R))

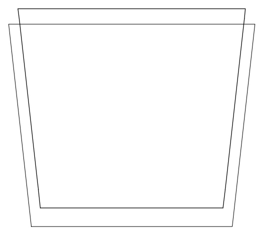
No

2．4．COMBINATION

BOX-SECTION MODEL FRAME

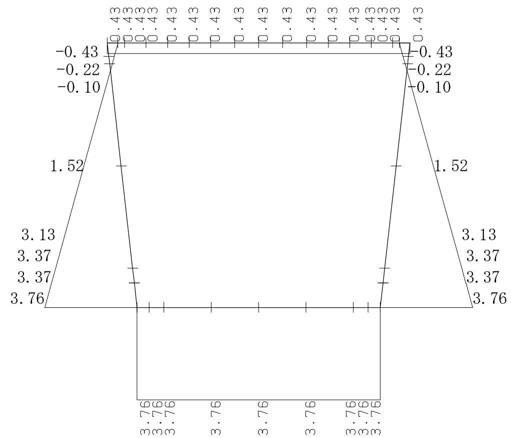


MOMENT SHALL BE POSITIVE AS THE DOT LINE SIDE IS TENSILE



No

ケース 12 (TEMPERATURE GRADIENT)



No

SERVICE LIMIT STATE

COLLECTION TABLE (TOP SLAB kN·m)

			2	5	9	11	13	17	20
GIRDER-SELF LOAD			-38.90	-23.94	-0.23	2.75	-0.12	-23.63	-38.46
SUPERIMPOSED			-44.70	-10.47	-2.64	-0.78	-0.74	-5.16	-36.91
SECONDARY MOMENT	CONST.			-18.28	-18.50	-18.62	-18.75	-18.97	
	SERVICE			-16.84	-17.04	-17.16	-17.27	-17.48	
CREEP				1.44	1.45	1.46	1.47	1.49	
LIVE LOAD(LL+IM)	MAX.					63.32			
	MIN.		-103.86	-77.27				-77.27	-103.86
+TEMP.	7.00	1.00·TG		-1.52	-1.52	-1.52	-1.52	-1.52	
	3.50	0.50·TG		-0.76	-0.76	-0.76	-0.76	-0.76	
-TEMP.	-2.00	1.00·TG		0.43	0.43	0.43	0.43	0.43	
	-1.00	0.50·TG		0.22	0.22	0.22	0.22	0.22	
WS	→	0.30·WS	1.61	0.93	0.33	0.00	-0.33	-0.93	-1.61
	←	0.30·WS	-1.61	-0.93	-0.33	0.00	0.33	0.93	1.61
WL	→	1.00·WL	3.01	1.75	0.63	0.00	-0.63	-1.75	-3.01
	←	1.00·WL	-3.01	-1.75	-0.63	0.00	0.63	1.75	3.01
AFTER ANCHOR SET			-38.90	-42.21	-18.73	-15.87	-18.87	-42.60	-38.46
PERMANENT LOAD			-83.60	-51.24	-19.91	-15.19	-18.13	-46.27	-75.37
SLS	Service-1	SE1- 1	-78.98	-49.33	-19.71	48.35	-19.85	-49.71	-79.99
		SE1- 2	-182.84	-126.60	-19.71	-15.95	-19.85	-126.98	-183.85
		SE1- 3	-88.22	-54.68	-21.63	48.35	-17.93	-44.36	-70.74
		SE1- 4	-192.08	-131.95	-21.63	-15.95	-17.93	-121.63	-174.60
		SE1- 5	-81.99	-51.83	-21.10	-16.71	-19.98	-48.73	-76.97
		SE1- 6	-85.21	-53.70	-21.77	-16.71	-19.32	-46.86	-73.76
		MAX.	-78.98	-49.33	-19.71	48.35	-17.93	-44.36	-70.74
		MIN.	-192.08	-131.95	-21.77	-16.71	-19.98	-126.98	-183.85
	Service-3	SE3- 1	-83.60	-52.00	-20.67	35.69	-18.89	-47.03	-75.37
		SE3- 2	-166.69	-113.82	-20.67	-15.95	-18.89	-108.85	-158.45
		SE3- 3	-83.60	-52.76	-21.43	-16.71	-19.65	-47.80	-75.37
		MAX.	-83.60	-52.00	-20.67	35.69	-18.89	-47.03	-75.37
		MIN.	-166.69	-113.82	-21.43	-16.71	-19.65	-108.85	-158.45

No

COLLECTION TABLE (BOTTOM SLAB kN·m)

			23	26	29
CASE 4			0.65	22.86	45.07
CASE 5			67.12	128.73	72.57
CASE 6			45.07	22.86	0.65
CASE 7			-25.03	49.00	-19.57
CASE 8			-34.02	-34.02	-34.02
CASE 9			56.00	119.59	61.45
GIRDER-SELF LOAD			0.22	61.22	0.49
SUPERIMPOSED			13.96	16.29	18.62
SECONDARY MOMENT	CONST.		3.65	3.93	4.22
	SERVICE		3.36	3.62	3.89
CREEP			-0.29	-0.31	-0.33
LIVE LOAD(LL+IM)	MAX.		45.72	45.72	45.72
	MIN.		-34.02	-34.02	-34.02
+TEMP.	7.00	1.00·TG	-13.15	-13.15	-13.15
	3.50	0.50·TG	-6.58	-6.58	-6.58
-TEMP.	-2.00	1.00·TG	3.76	3.76	3.76
	-1.00	0.50·TG	1.88	1.88	1.88
WS	→	0.30·WS	0.69	0.00	-0.69
	←	0.30·WS	-0.69	0.00	0.69
WL	→	1.00·WL	1.29	0.00	-1.29
	←	1.00·WL	-1.29	0.00	1.29
AFTER ANCHOR SET			3.87	65.15	4.71
PERMANENT LOAD			17.54	81.13	23.00
SLS	Service-1	SE1- 1	67.12	128.73	68.62
		SE1- 2	-21.07	49.00	-19.57
		SE1- 3	63.16	128.73	72.57
		SE1- 4	-25.03	49.00	-15.62
		SE1- 5	21.99	84.89	26.07
		SE1- 6	20.61	84.89	27.44
		MAX.	67.12	128.73	72.57
		MIN.	-25.03	49.00	-19.57
	Service-3	SE3- 1	56.00	119.59	61.45
		SE3- 2	-16.25	55.80	-10.79
		SE3- 3	21.30	84.89	26.75
		MAX.	56.00	119.59	61.45
		MIN.	-16.25	55.80	-10.79

No

3. STRESS CHECK FOR TOP SLAB

3. 1. TRANSVERSAL

3. 1. 1. STRESS DUE TO LOAD

(kN·m)

			2	5	9	11	13	17	20
MOMENT	AFTER ANCHOR SET		-38.90	-42.21	-18.73	-15.87	-18.87	-42.60	-38.46
	PERMANENT LOAD		-83.60	-51.24	-19.91	-15.19	-18.13	-46.27	-75.37
	SERVICE	MAX.	-78.98	-49.33	-19.71	48.35	-17.93	-44.36	-70.74
		MIN.	-192.08	-131.95	-21.77	-16.71	-19.98	-126.98	-183.85
	Service-3	MAX.	-83.60	-52.00	-20.67	35.69	-18.89	-47.03	-75.37
		MIN.	-166.69	-113.82	-21.43	-16.71	-19.65	-108.85	-158.45
I / yc (m3)			0.0504	0.0504	0.0104	0.0104	0.0104	0.0504	0.0504
STRESS	AFTER ANCHOR SET		-0.77 0.77	-0.84 0.84	-1.80 1.80	-1.52 1.52	-1.81 1.81	-0.85 0.85	-0.76 0.76
	PERMANENT LOAD		-1.66 1.66	-1.02 1.02	-1.91 1.91	-1.46 1.46	-1.74 1.74	-0.92 0.92	-1.49 1.49
	Service-1	MAX.	-1.57 1.57	-0.98 0.98	-1.89 1.89	4.64 -4.64	-1.72 1.72	-0.88 0.88	-1.40 1.40
		MIN.	-3.81 3.81	-2.62 2.62	-2.09 2.09	-1.60 1.60	-1.92 1.92	-2.52 2.52	-3.65 3.65
	Service-3	MAX.	-1.66 1.66	-1.03 1.03	-1.98 1.98	3.43 -3.43	-1.81 1.81	-0.93 0.93	-1.49 1.49
		MIN.	-3.31 3.31	-2.26 2.26	-2.06 2.06	-1.60 1.60	-1.89 1.89	-2.16 2.16	-3.14 3.14

(Mpa)

IN STRESS TABLE, UPPER COLUMN SHOWS EXTREME TOP AND LOWER COLUMN SHOWS EXTREME BOTTOM

STRESS CALCULATION

$$\left.\begin{matrix}\sigma cu \\ \sigma cl\end{matrix}\right\} = \pm \frac{M}{Z}$$

M : BENDING MOMENT (kN·m)

Z : I/yc (m3)

σcu : EXTREMELY FIBER STRESS(TOP) (Mpa)

σcu : EXTREMELY FIBER STRESS(BOTTOM) (Mpa)

No

COLLECTION TABLE (WEB kN·m)

			32	35	38	41
CASE 4			-62.08	-7.89	-25.05	40.19
CASE 5			37.31	46.13	40.84	51.93
CASE 6			-25.05	40.19	-62.08	-7.89
CASE 7			-118.17	-37.83	-114.64	-32.03
CASE 8			64.82	-24.03	64.82	-24.03
CASE 9			22.70	35.96	26.23	41.76
GIRDER-SELF LOAD			-12.53	-11.50	-12.38	-11.21
SUPERIMPOSED			-32.55	9.03	-29.87	13.95
SECONDARY MOMENT	CONST.		17.08	4.98	17.85	5.62
	SERVICE		15.73	4.59	16.45	5.18
CREEP			-1.34	-0.39	-1.40	-0.44
LIVE LOAD(LL+IM)	MAX.		64.82	40.19	64.82	40.19
	MIN.		-87.13	-31.92	-87.13	-31.92
+TEMP.	7.00	1.00·TG	0.36	-11.79	0.36	-11.79
	3.50	0.50·TG	0.18	-5.89	0.18	-5.89
-TEMP.	-2.00	1.00·TG	-0.10	3.37	-0.10	3.37
	-1.00	0.50·TG	-0.05	1.68	-0.05	1.68
WS	→	0.30·WS	0.57	0.74	-0.57	-0.74
	←	0.30·WS	-0.57	-0.74	0.57	0.74
WL	→	1.00·WL	1.07	1.39	-1.07	-1.39
	←	1.00·WL	-1.07	-1.39	1.07	1.39
AFTER ANCHOR SET			4.55	-6.52	5.47	-5.59
PERMANENT LOAD			-29.34	2.12	-25.81	7.92
SLS	Service-1	SE1- 1	37.31	46.13	37.55	47.65
		SE1- 2	-114.87	-33.55	-114.64	-32.03
		SE1- 3	34.01	41.85	40.84	51.93
		SE1- 4	-118.17	-37.83	-111.34	-27.75
		SE1- 5	-28.87	-8.92	-26.48	10.54
		SE1- 6	-30.02	-10.41	-25.34	12.03
		MAX.	37.31	46.13	40.84	51.93
		MIN.	-118.17	-37.83	-114.64	-32.03
	Service-3	SE3- 1	22.70	35.96	26.23	41.76
		SE3- 2	-99.10	-29.31	-95.56	-23.51
		SE3- 3	-29.44	-9.67	-25.91	11.29
		MAX.	22.70	35.96	26.23	41.76
		MIN.	-99.10	-29.31	-95.56	-23.51

No

3. 1. 2. PRE-STRESS

1) PRE-STRESS IMMEDIATELY AFTER ANCHOR SET

TYPE : STRAND CABLE

STRESSING : ONE SIDE

$\sigma_{pti} = \sigma_{pt0} \times e^{-(\lambda \cdot l + \mu \cdot \alpha)}$

$\sigma_{pt0} = 1400.0$ Mpa

$\lambda = 0.001$

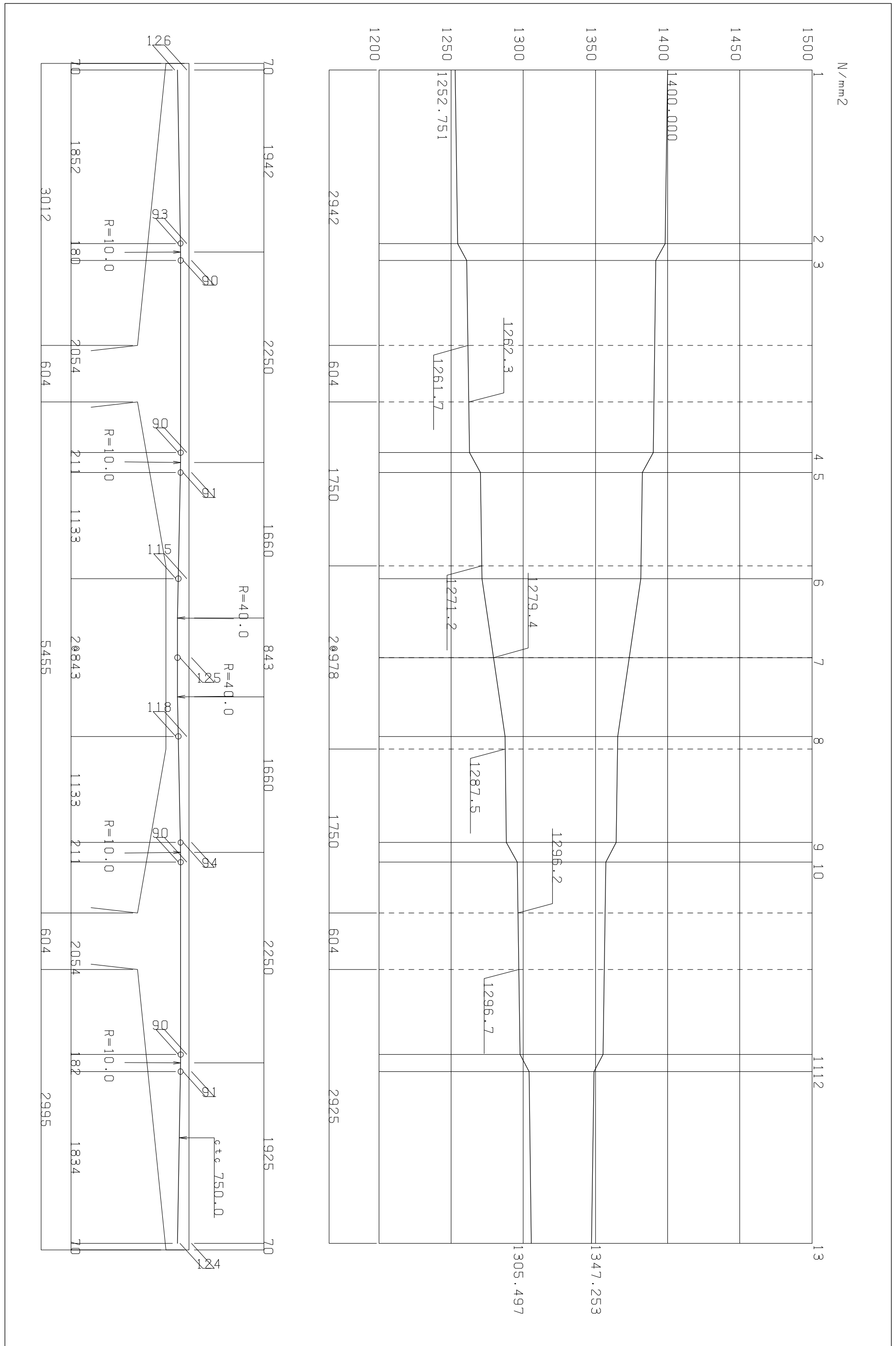
$\mu = 0.250$

STRESS IMMEDIATELY AFTER ANCHOR SET AT INFLICTION POINTS

POINTS	l (m)	α (Rad)	λ・l	μ・α	λ・l+μ・α	e-(λ・l+μ・α)	σpti (Mpa)	ANCHOR SET(Mpa)
1	0.000	0.0000	0.0000	0.0000	0.0000	1.0000	1400.00	1252.75
2	1.852	0.0000	0.0012	0.0000	0.0012	0.9988	1398.29	1254.46
3	2.032	0.0180	0.0013	0.0045	0.0058	0.9942	1391.84	1260.91
4	4.087	0.0180	0.0027	0.0045	0.0072	0.9928	1389.95	1262.80
5	4.298	0.0391	0.0028	0.0098	0.0126	0.9875	1382.45	1270.30
6	5.431	0.0391	0.0036	0.0098	0.0134	0.9867	1381.42	1271.33
7	6.274	0.0602	0.0041	0.0150	0.0192	0.9810	1373.39	1279.36
8	7.117	0.0813	0.0047	0.0203	0.0250	0.9753	1365.41	1287.34
9	8.251	0.0813	0.0054	0.0203	0.0258	0.9746	1364.39	1288.36
10	8.461	0.1024	0.0056	0.0256	0.0312	0.9693	1357.03	1295.72
11	10.515	0.1024	0.0069	0.0256	0.0325	0.9680	1355.19	1297.56
12	10.697	0.1205	0.0071	0.0301	0.0372	0.9635	1348.89	1303.87
13	12.531	0.1205	0.0083	0.0301	0.0384	0.9623	1347.25	1305.50

STRESS IMMEDIATELY AFTER ANCHOR SET AT DESIGN SECTION

SEC.	X (m)	ANCHOR SET(Mpa)
2	3.012	1261.75
5	3.616	1262.30
9	5.366	1271.21
11	6.344	1279.36
13	7.321	1287.46
17	9.071	1296.20
20	9.675	1296.74



No

STRESS IMMEDIATELY AFTER ANCHOR SET AT DESIGN SECTION

$$\sigma pt = \sigma pt' - \Delta \sigma p$$

$\sigma pt'$: TENSILE STRESS AT DESIGN
$\Delta \sigma p$: ELASTIC SHORTNING

$$Pt = \frac{1.0\ m}{0.75\ m} \cdot Ap \cdot \sigma pt$$

$$Ap = 416.1\ mm^2$$

SEC.	σpt′ Mpa	Δσp Mpa	σpt Mpa	Pt kN
2	1261.75	8.20	1253.55	695.468
5	1262.30	8.20	1254.10	695.776
9	1271.21	8.20	1263.01	700.717
11	1279.36	8.20	1271.16	705.238
13	1287.46	8.20	1279.26	709.733
17	1296.20	8.20	1288.00	714.585
20	1296.74	8.20	1288.54	714.884

2) PRE-STRESS AT SERVICE

STRESS AFTER ANCHOR SET AT CENTROID OF TENDONS

$$\sigma cpt = \frac{Pt}{A} + \frac{Pt \cdot ep}{Zp}$$

SEC.	Pt kN	A m2	ep m	Zp m3	σcpt kN/m2
2	695.468	0.550	0.185	0.0749	2981.3
5	695.776	0.550	0.185	0.0749	2982.6
9	700.717	0.250	0.012	0.1085	2880.4
11	705.238	0.250	0.000	0.0000	2821.0
13	709.733	0.250	0.012	0.1085	2917.4
17	714.585	0.550	0.185	0.0749	3063.2
20	714.884	0.550	0.185	0.0749	3064.5

STRESS DUE TO LOAD AT CENTROID OF TENDONS

SEC.	Zp m3	Mdo kN·m	Md1 kN·m	M2p kN·m	σcpdo kN/m2	σcd1 kN/m2	σcp2p kN/m2
2	0.0749	-38.897	-44.703		-519.0	-596.5	
5	0.0749	-23.936	-10.466	-18.276	-319.4	-139.7	-243.9
9	0.1085	-0.228	-2.637	-18.499	-2.1	-24.3	-170.5
11	0.0000	2.754	-0.783	-18.623	0.0	0.0	0.0
13	0.1085	-0.118	-0.736	-18.749	-1.1	-6.8	-172.8
17	0.0749	-23.631	-5.162	-18.972	-315.3	-68.9	-253.1
20	0.0749	-38.459	-36.907		-513.2	-492.5	

COMPOSITE STRESS AT CENTROID OF TENDONS

$$\sigma cp1 = \sigma cpt + \sigma cpd0 + \sigma cp2p + 1/2(\eta - 1) \cdot \sigma cp2p \quad (\text{ASSUMED } \eta = 0.92)$$

SEC.	σcpt kN/m2	σcpdo kN/m2	σcp2p kN/m2	σcp1 kN/m2
2	2981.3	-519.0		2462.3
5	2982.6	-319.4	-243.9	2428.9
9	2880.4	-2.1	-170.5	2714.5
11	2821.0	0.0	0.0	2821.0
13	2917.4	-1.1	-172.8	2750.3
17	3063.2	-315.3	-253.1	2504.7
20	3064.5	-513.2		2551.3

PRESTRESSING LOSS DUE TO CREEP AND SHRINKAGE

$$\Delta \sigma p\phi = \frac{n \cdot \phi 1 \cdot \sigma cp1 + Ep \cdot \varepsilon s}{1 + n \cdot \sigma cpt \cdot (1 + \rho 1 \cdot \phi 1) / \sigma pt} + \frac{n \cdot \phi 2 \cdot \sigma cd1}{1 + n \cdot \sigma cpt \cdot (1 + \rho 2 \cdot \phi 2) / \sigma pt}$$
$$= \Delta \sigma p\phi 1 + \Delta \sigma p\phi 2$$

$\phi 1 = 2.20$ $\phi 2 = 1.00$ $n = 6.10$ $\rho 1 = 0.70$ $\rho 2 = 0.70$

$Ep = 0.2 \times 10^6$ Mpa、 $\varepsilon s = 25.0 \times 10\text{-}5$

SEC.	σpt Mpa	σcpt Mpa	σcp1 Mpa	σcd1 Mpa	Δσpφ1 Mpa	Δσpφ2 Mpa	Δσpφ Mpa
2	1253.55	2.98	2.46	-0.60	79.37	-3.55	75.82
5	1254.10	2.98	2.43	-0.14	78.94	-0.83	78.11
9	1263.01	2.88	2.71	-0.02	82.75	-0.14	82.61
11	1271.16	2.82	2.82	0.00	84.21	0.00	84.21
13	1279.26	2.92	2.75	-0.01	83.22	-0.04	83.18
17	1288.00	3.06	2.50	-0.07	79.92	-0.41	79.51
20	1288.54	3.06	2.55	-0.49	80.52	-2.93	77.59

PRESTRESSING LOSS DUE TO RELAXATION

$$\Delta \sigma p\gamma = \gamma \cdot \sigma pt \quad (\gamma = 1.50\ \%)$$

SEC.	σpt Mpa	Δσpγ Mpa
2	1253.55	18.80
5	1254.10	18.81
9	1263.01	18.95
11	1271.16	19.07
13	1279.26	19.19
17	1288.00	19.32
20	1288.54	19.33

3) PRE-STRESS AT EXTREMELY FIBER

$$\left.\begin{matrix}\sigma_{ptu}\\ \sigma_{ptl}\end{matrix}\right\} = \frac{Pt}{A} \pm \frac{Pt \cdot ep}{Z}$$

$$\left.\begin{matrix}\sigma_{peu}\\ \sigma_{pel}\end{matrix}\right\} = \eta \times \left\{\begin{matrix}\sigma_{ptu}\\ \sigma_{ptl}\end{matrix}\right.$$

Pt = STRESSING FORCE IMMEDATELY AFTER ANCHOR SET(kN)

A = AREA (m^2)

Z = I / yc (m^3)

ep = EXCENTRCITY (m)

η = EFFECTIVE RATIO

σ_{ptu}、σ_{ptl} = PRE-STRESS AT EXTREMELY FIBER AFTER ANCHOR SET (Mpa)

σ_{ptu}、σ_{ptl} = PRE-STRESS AT EXTREMELY FIBER IN SERVICE (Mpa)

SEC.		A m2	Z m3	Pt kN	ep m	η	ANCHOR SET Mpa	AT SERVICE Mpa
2	TOP.	0.550	0.0504	695.468	0.185	0.925	3.82	3.53
	BOT.						-1.29	-1.19
5	TOP.	0.550	0.0504	695.776	0.185	0.923	3.82	3.52
	BOT.						-1.29	-1.19
9	TOP.	0.250	0.0104	700.717	0.012	0.920	3.61	3.32
	BOT.						2.00	1.84
11	TOP.	0.250	0.0104	705.238	0.000	0.919	2.82	2.59
	BOT.						2.82	2.59
13	TOP.	0.250	0.0104	709.733	0.012	0.920	3.66	3.36
	BOT.						2.02	1.86
17	TOP.	0.550	0.0504	714.585	0.185	0.923	3.92	3.62
	BOT.						-1.32	-1.22
20	TOP.	0.550	0.0504	714.884	0.185	0.925	3.92	3.63
	BOT.						-1.32	-1.22

EFFECTIVE STRESS AT SERVICE

$\sigma_{pe} = \sigma_{pt} - \Delta\sigma_{p\phi} - \Delta\sigma_{p\gamma}$

$\eta = \sigma_{pe} / \sigma_{pt}$

$\bar{\eta}$ = 0.922

SEC.	σ_{pt} Mpa	$\Delta\sigma_{p\phi}$ Mpa	$\Delta\sigma_{p\gamma}$ Mpa	σ_{pe} Mpa	η_i
2	1253.55	75.82	18.80	1158.93	0.925
5	1254.10	78.11	18.81	1157.18	0.923
9	1263.01	82.61	18.95	1161.45	0.920
11	1271.16	84.21	19.07	1167.88	0.919
13	1279.26	83.18	19.19	1176.89	0.920
17	1288.00	79.51	19.32	1189.18	0.923
20	1288.54	77.59	19.33	1191.63	0.925

3.1.3.COMPOSITE STRESS

SEC.		STRESS DUE TO LOAD						PRE-STRESS		COMPOSITE STRESS					
		ANCHOR SET	PERMA NENT	SERVICE-1		SERVICE-3		ANCHOR SET	SERVICE	ANCHOR SET	PERMA NENT	SERVICE-1		SERVICE-3	
				MAX	MIN	MAX	MIN					MAX	MIN	MAX	MIN
2	TOP.	-0.77	-1.66	-1.57	-3.81	-1.66	-3.31	3.82	3.53	3.04	1.87	1.96	-0.28	1.87	0.22
	BOT.	0.77	1.66	1.57	3.81	1.66	3.31	-1.29	-1.19	-0.52	0.47	0.38	2.62	0.47	2.12
5	TOP.	-0.84	-1.02	-0.98	-2.62	-1.03	-2.26	3.82	3.52	2.98	2.51	2.54	0.91	2.49	1.27
	BOT.	0.84	1.02	0.98	2.62	1.03	2.26	-1.29	-1.19	-0.45	-0.17	-0.21	1.43	-0.16	1.07
9	TOP.	-1.80	-1.91	-1.89	-2.09	-1.98	-2.06	3.61	3.32	1.81	1.41	1.43	1.23	1.34	1.26
	BOT.	1.80	1.91	1.89	2.09	1.98	2.06	2.00	1.84	3.79	3.75	3.73	3.92	3.82	3.89
11	TOP.	-1.52	-1.46	4.64	-1.60	3.43	-1.60	2.82	2.59	1.30	1.13	7.23	0.99	6.02	0.99
	BOT.	1.52	1.46	-4.64	1.60	-3.43	1.60	2.82	2.59	4.34	4.05	-2.05	4.20	-0.83	4.20
13	TOP.	-1.81	-1.74	-1.72	-1.92	-1.81	-1.89	3.66	3.36	1.85	1.62	1.64	1.45	1.55	1.48
	BOT.	1.81	1.74	1.72	1.92	1.81	1.89	2.02	1.86	3.83	3.60	3.58	3.78	3.67	3.75
17	TOP.	-0.85	-0.92	-0.88	-2.52	-0.93	-2.16	3.92	3.62	3.08	2.70	2.74	1.10	2.69	1.46
	BOT.	0.85	0.92	0.88	2.52	0.93	2.16	-1.32	-1.22	-0.48	-0.30	-0.34	1.30	-0.29	0.94
20	TOP.	-0.76	-1.49	-1.40	-3.65	-1.49	-3.14	3.92	3.63	3.16	2.13	2.22	-0.02	2.13	0.49
	BOT.	0.76	1.49	1.40	3.65	1.49	3.14	-1.32	-1.22	-0.56	0.27	0.18	2.42	0.27	1.92

ALLOWABLE STRESS

TENSILE STRESS LIMIT 0.50･√fc'=0.5x√ 45.0= -3.35 Mpa

COMPRESSIVE STRESS LIMIT 0.40･fc'=0.4x 45.0= 18.0 Mpa

No

No

CRACK CONTROL

$T = 1/2 \cdot \sigma ct \cdot b \cdot x$
$reqAs = T / \sigma sa$

T : SECTIONAL TENSILE FORCE (kN)
σct : TENSILE STRESS AT EXTREME FIBER
b : UNIT WIDTH (m)
reqAs : REQUIRED REINFORCEMENT
σsa : TENSILE STRESS OF REINFORCEMENT FOR CRACK ALLOWABLE
x : |σcu| / (|σcu| + |σcl|)･h
h : MEMBER THICKNESS
σcu、σcl : CONCRETE STRESS AT EXTREME FIBER

ANCHOR SET

SEC.	σcu Mpa	σcl Mpa	h m	x m	T kN	σsa Mpa	reqAs mm2	ARRENGED REBAR mm2
2	3.04	-0.52	0.550	0.0797	20.56	200.00	102.8	
5	2.98	-0.45	0.550	0.0722	16.28	200.00	81.4	
17	3.08	-0.48	0.550	0.0739	17.67	200.00	88.3	
20	3.16	-0.56	0.550	0.0829	23.23	200.00	116.1	

PERMANENT

SEC.	σcu Mpa	σcl Mpa	h m	x m	T kN	σsa Mpa	reqAs mm2	ARRENGED REBAR mm2
5	2.51	-0.17	0.550	0.0353	3.04	195.00	15.6	
17	2.70	-0.30	0.550	0.0555	8.43	195.00	43.2	

SERVICE-Ⅰ (LL-MAX)

SEC.	σcu Mpa	σcl Mpa	h m	x m	T kN	σsa Mpa	reqAs mm2	ARRENGED REBAR mm2
5	2.54	-0.21	0.550	0.0420	4.41	195.00	22.6	
11	7.23	-2.05	0.250	0.0552	56.58	195.00	290.1	
17	2.74	-0.34	0.550	0.0609	10.41	195.00	53.4	

SERVICE-Ⅰ (LL-MIN)

SEC.	σcu Mpa	σcl Mpa	h m	x m	T kN	σsa Mpa	reqAs mm2	ARRENGED REBAR mm2
2	-0.28	2.62	0.550	0.0534	7.51	195.00	38.5	
20	-0.02	2.42	0.550	0.0042	0.04	195.00	0.2	

PC-STEEL STRES

$\sigma pmax = \sigma pe + \Delta \sigma p$

$\Delta \sigma p = -n \cdot (\sigma cpd1 + \sigma cp2p + \sigma cpl)$

σ pmax	:PC-STEEL STRESS AT LL-LOADING
Δ σ	:ADDITIONAL PC-STEEL STRESS DUE TO LOAD
n	:ELASTIC RATIO OF CONCRETE TO PC-STEEL
σ cpd1	:CONCRETE STRESS AT CENTROID OF PS-STEEL DUE TO SURFACE LOADING
σ cpd1	:CONCRETE STRESS FOR DIFFERENCE OF SECONDARY FORCE DUE TO PRE-STRESSING LOSS AT CENTROID OF PC-STEEL
σ cpl	:CONCRETE STRESS AT CENTROID OF PC-STEEL DUE TO LL-LOADING
Zp	:I/y AT CENTROID OF PC-STEEL

$\sigma cpi = Mi / Zp$

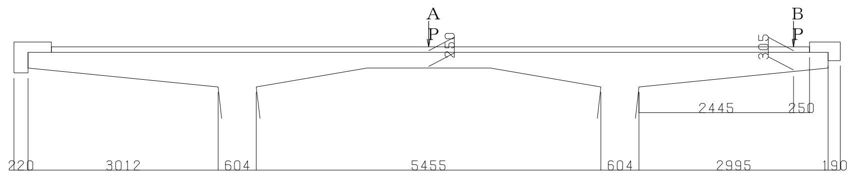
SEC.	Md1 kN·m	ΔM2p kN·m	Ml kN·m	Zp m3	σ cpd1 Mpa	σ cp2p Mpa	σ cpl Mpa	Σ σ cp Mpa	n	Δ σ p Mpa	σ pe Mpa	σ pmax Mpa
5	-10.466	-1.437	-77.270	0.0749	-0.14	-0.02	-1.03	-1.19	6.10	7.26	1157.18	1164.44
11	-0.783	-1.464	63.320	0.0000	—	—	—	—	6.10	—	1167.88	—
17	-5.162	-1.492	-77.270	0.0749	-0.07	-0.02	-1.03	-1.12	6.10	6.83	1189.18	1196.01

SERVICE-Ⅲ (MAZ)

SEC.	σ cu Mpa	σ cl Mpa	h m	x m	T kN	σ sa Mpa	reqAs mm2	ARRENGED REBAR mm2
5	2.49	-0.16	0.550	0.0326	2.56	195.00	13.1	
11	6.02	-0.83	0.250	0.0304	12.69	195.00	65.1	
17	2.69	-0.29	0.550	0.0533	7.69	195.00	39.4	

No

3．2．DESIGN FOR LONGITUDINAL DIRECTION



CALCULATION AS RC COMPONENT IS PERFORMED FOR THE SECTION SHOW ABOVE.

3．2．1．SECTIONAL FORCE

LIVE LOAD IS CONSIDERED AND PERMANENT LOAD IS DISREGARDED.

SECTION A (CENTER OF UPPER SLAB)

SPAN LENGTH l =5.455 m COEFFICIENT K =1.000

MA = = 31.520 kN·m

SECTION B (LOADING POINT OF AXLE FORCE)

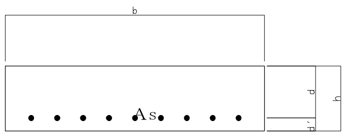
SPAN LENGTH l =2.445 m COEFFICIENT K =1.000

MB = 33.060 kN·m

No

3．2．2．STRESS CHECK

CRACK CONTROL COMPUTED AS RC SECTION



REBAR STRESS

$$\sigma s = \frac{M}{As \cdot j \cdot d}$$

CONCRETE STRESS

$$\sigma c = \frac{\sigma s}{m}$$

ELASTIC RATIO n = 12.00

SECTION			A － A	B － B
MOMENT	M	kN·m	31.52	33.06
DIMENSION	B	mm	1000.0	1000.0
	d'	mm	65.0	62.0
	d	mm	185.0	243.1
REBARS	Asreq	mm2	808.0	628.0
	As	mm2	D16ctc125	D14ctc125
			1608.0	1232.0
CIE	100·p		0.009	0.005
	k		0.364	0.293
	j		0.879	0.902
	m		20.950	28.927
	σs	Mpa	120.60	122.35
ALLOWABLE CRACK	dc	mm	50.000	50.000
	ctc	mm	125.000	125.000
	A	mm2	12500.0	12500.0
	Z	N/mm	23000.0	23000.0
	Z/(dc·A)**1/3	Mpa	269.0	269.0
	0.6·fsy	Mpa	240.0	240.0
	fsa	Mpa	240.0	240.0
	JUDGE	-	OK	OK

fsa = Z / (dc·A)**1/3 ≦ 0.6·fsy

fsa : LIMIT STRESS FOR REBAR
dc : DEPTH OF CONCRETE MEASURED FROM TENTION FIBER (LESS THAN 50mm)
A : AREA OF CONCRETE FOR A TENSILE REBAR(mm2)
fsy : YIELD STRESS OF REBAR (400.0 Mpa)
Z : CRACK WIDTH PARAMETER (N/mm)

Asreq = M / (fsa·j·d)

3. 2. 3. CHECK FOR MINIMUM REINFORCEMENT

REINFORCEMENT FOR SHRINKAGE AND TEMPERATURE

(1) UPPER SLAB OF BOX (LONGITUDINAL)

Asreq① = 0.75·Ag / fy / L

= 0.75 × 1888858.3 / 400.0 / 5.4554

= 649.2 mm2/m

Asreq② = 0.0015·Ag / L

= 0.0015 × 1888858.3 / 5.4554

= 519.4 mm2/m

(2) OVERHANG(L) OF BOX (LONGITUDINAL)

Asreq① = 0.75·Ag / fy / L

= 0.75 × 1204800.0 / 400.0 / 3.0120

= 750.0 mm2/m

Asreq② = 0.0015·Ag / L

= 0.0015 × 1204800.0 / 3.0120

= 600.0 mm2/m

(3) OVERHANG(R) OF BOX (LONGITUDINAL)

Asreq① = 0.75·Ag / fy / L

= 0.75 × 1198000.0 / 400.0 / 2.9950

= 750.0 mm2/m

Asreq② = 0.0015·Ag / L

= 0.0015 × 1198000.0 / 2.9950

= 600.0 mm2/m

(4) BOTTOM SLAB (TRANSVERSE)

Asreq = 0.005·Ag / L

= 0.005 × 2799684.9 / 5.4370

= 2574.7 mm2/m

(5) WEB (LONGITUDINAL)

Asreq① = 0.75·Ag / fy / L

= 0.75 × 2686836.1 / 400.0 / 4.4500

= 1132.1 mm2/m

Asreq② = 0.0015·Ag / L

= 0.0015 × 2686836.1 / 4.4500

= 905.7 mm2/m

(6) BOTTOM SLAB (TRANSVERSE)

Asreq = 0.005·Ag / L

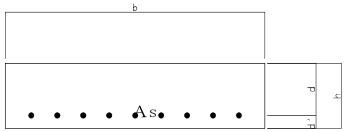
= 0.005 × 2799684.9 / 5.4370

= 2574.7 mm2/m

No

BOTTOM SLAB (SERVICE)

CRACK CONTROL COMPUTED AS RC SECTION



REBAR STRESS

$$\sigma s = \frac{M}{As \cdot j \cdot d}$$

CONCRETE STRESS

$$\sigma c = \frac{\sigma s}{m}$$

ELASTIC RATIO $n = 12.00$

SECTION			26 MAX.	23 MIN.
MOMENT	M	kN·m	128.73	-21.07
DIMENSION	B	mm	1000.0	1000.0
	d’	mm	50.0	45.0
	d	mm	950.0	955.0
REBARS	Asreq	mm2	601.1	97.9
	As	mm2	D16ctc125	D16ctc125
			1608.0	1608.0
CIE	100·p		0.002	0.002
	k		0.182	0.182
	j		0.939	0.939
	m		53.840	53.995
	σ s	Mpa	89.72	14.61
ALLOWABLE CRACK	d c	mm	50.000	45.000
	ctc	mm	125.000	125.000
	A	mm2	12500.0	11250.0
	Z	N/mm	23000.0	23000.0
	Z/(dc·A)**1/3	Mpa	269.0	288.6
	0.6·fsy	Mpa	240.0	240.0
	fsa	Mpa	240.0	240.0
	JUDGE	-	OK	OK

fsa = Z / (dc·A)**1/3 ≦ 0.6·fsy

fsa : LIMIT STRESS FOR REBAR
dc : DEPTH OF CONCRETE MEASURED FROM TENTION FIBER (LESS THAN 50mm)
A : AREA OF CONCRETE FOR A TENSILE REBAR(mm2)
fsy : YIELD STRESS OF REBAR (400.0 Mpa)
Z : CRACK WIDTH PARAMETER (N/mm)

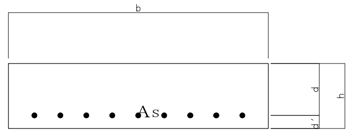
Asreq = M / (fsa·j·d)

No

4. STRESS CHECK FOR BOTTOM SLAB AND WEB

4.1 BOTTOM SLAB

CRACK CONTROL COMPUTED AS RC SECTION



REBAR STRESS

$$\sigma s = \frac{M}{As \cdot j \cdot d}$$

CONCRETE STRESS

$$\sigma c = \frac{\sigma s}{m}$$

ELASTIC RATIO $n = 12.00$

SECTION			26 MAX.	23 MIN.
MOMENT	M	kN·m	81.13	17.54
DIMENSION	B	mm	1000.0	1000.0
	d’	mm	50.0	50.0
	d	mm	950.0	950.0
REBARS	Asreq	mm2	378.9	81.9
	As	mm2	D16ctc125	D16ctc125
			1608.0	1608.0
CIE	100·p		0.002	0.002
	k		0.182	0.182
	j		0.939	0.939
	m		53.840	53.840
	σ s	Mpa	56.55	12.23
ALLOWABLE CRACK	d c	mm	50.000	50.000
	ctc	mm	125.000	125.000
	A	mm2	12500.0	12500.0
	Z	N/mm	23000.0	23000.0
	Z/(dc·A)**1/3	Mpa	269.0	269.0
	0.6·fsy	Mpa	240.0	240.0
	fsa	Mpa	240.0	240.0
	JUDGE	-	OK	OK

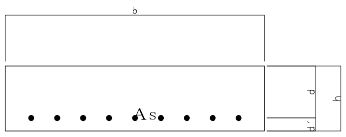
fsa = Z / (dc·A)**1/3 ≦ 0.6·fsy

fsa : LIMIT STRESS FOR REBAR
dc : DEPTH OF CONCRETE MEASURED FROM TENTION FIBER (LESS THAN 50mm)
A : AREA OF CONCRETE FOR A TENSILE REBAR(mm2)
fsy : YIELD STRESS OF REBAR (400.0 Mpa)
Z : CRACK WIDTH PARAMETER (N/mm)

Asreq = M / (fsa·j·d)

WEB (SERVICE)

CRACK CONTROL COMPUTED AS RC SECTION



REBAR STRESS

$$\sigma s = \frac{M}{As \cdot j \cdot d}$$

CONCRETE STRESS

$$\sigma c = \frac{\sigma s}{m}$$

ELASTIC RATIO n = 12.00

SECTION			41 MAX.	32 MIN.
MOMENT	M	kN·m	51.93	-114.87
DIMENSION	B	mm	1000.0	1000.0
	d'	mm	50.0	45.0
	d	mm	550.0	555.0
REBARS	Asreq	mm2	430.3	942.8
	As	mm2	D18ctc125	D18ctc125
			2032.0	2032.0
CIE	100·p		0.004	0.004
	k		0.257	0.256
	j		0.914	0.915
	m		34.743	34.924
	σ s	Mpa	50.82	111.35
ALLOWABLE CRACK	d c	mm	50.000	45.000
	ctc	mm	125.000	125.000
	A	mm2	12500.0	11250.0
	Z	N/mm	23000.0	23000.0
	Z/(dc·A)**1/3	Mpa	269.0	288.6
	0.6·fsy	Mpa	240.0	240.0
	fsa	Mpa	240.0	240.0
	JUDGE	-	OK	OK

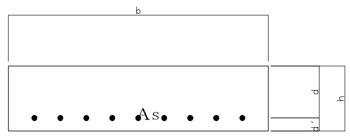
fsa = Z / (dc·A)**1/3 ≦ 0.6·fsy

fsa : LIMIT STRESS FOR REBAR
dc : DEPTH OF CONCRETE MEASURED FROM TENTION FIBER (LESS THAN 50mm)
A : AREA OF CONCRETE FOR A TENSILE REBAR(mm2)
fsy : YIELD STRESS OF REBAR (400.0 Mpa)
Z : CRACK WIDTH PARAMETER (N/mm)

Asreq = M / (fsa·j·d)

4.2 WEB

CRACK CONTROL COMPUTED AS RC SECTION



REBAR STRESS

$$\sigma s = \frac{M}{As \cdot j \cdot d}$$

CONCRETE STRESS

$$\sigma c = \frac{\sigma s}{m}$$

ELASTIC RATIO n = 12.00

SECTION			41 MAX.	32 MIN.
MOMENT	M	kN·m	7.92	-29.34
DIMENSION	B	mm	1000.0	1000.0
	d'	mm	50.0	45.0
	d	mm	550.0	555.0
REBARS	Asreq	mm2	65.6	240.8
	As	mm2	D18ctc125	D18ctc125
			2032.0	2032.0
CIE	100·p		0.004	0.004
	k		0.257	0.256
	j		0.914	0.915
	m		34.743	34.924
	σ s	Mpa	7.75	28.44
ALLOWABLE CRACK	d c	mm	50.000	45.000
	ctc	mm	125.000	125.000
	A	mm2	12500.0	11250.0
	Z	N/mm	23000.0	23000.0
	Z/(dc·A)**1/3	Mpa	269.0	288.6
	0.6·fsy	Mpa	240.0	240.0
	fsa	Mpa	240.0	240.0
	JUDGE	-	OK	OK

fsa = Z / (dc·A)**1/3 ≦ 0.6·fsy

fsa : LIMIT STRESS FOR REBAR
dc : DEPTH OF CONCRETE MEASURED FROM TENTION FIBER (LESS THAN 50mm)
A : AREA OF CONCRETE FOR A TENSILE REBAR(mm2)
fsy : YIELD STRESS OF REBAR (400.0 Mpa)
Z : CRACK WIDTH PARAMETER (N/mm)

Asreq = M / (fsa·j·d)

5. STRENGTH CHECK

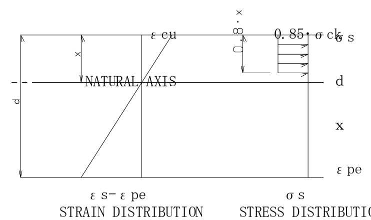
Ⅰ) BENDING MOMENT AT STRENGTH LIMIT STATE

LOAD COMBINATIONS

			1	2	3	4	5	6	7	8	9	10	11				
			DC	DW	EL	CR	SH	LL+IM	PL	TG	WS	WL	CT				
S	Strength-1	ST1-1	1.25	1.50	1.00	0.50		1.75	1.75					Lmax	-		1
T		ST1-2	1.25	1.50	1.00	0.50		1.75	1.75					Lmin	-		2
R		ST1-3	0.90	0.65	1.00	0.50		1.75	1.75					Lmax	-		3
E		ST1-4	0.90	0.65	1.00	0.50		1.75	1.75					Lmin	-		4
N	Strength-2	ST2-1	1.25	1.50	1.00	0.50		1.35	1.35					Lmax	-		5
G		ST2-2	1.25	1.50	1.00	0.50		1.35	1.35					Lmin	-		6
T		ST2-3	0.90	0.65	1.00	0.50		1.35	1.35					Lmax	-		7
H		ST2-4	0.90	0.65	1.00	0.50		1.35	1.35					Lmin	-		8
	Strength-3	ST3-1	1.25	1.50	1.00	0.50					1.40			Wind	R		9
L		ST3-2	1.25	1.50	1.00	0.50					-1.40			Wind	L		10
I		ST3-3	0.90	0.65	1.00	0.50					1.40			Wind	R		11
M		ST3-4	0.90	0.65	1.00	0.50					-1.40			Wind	L		12
I	Strength-4	ST4-1	1.50	1.50	1.00	0.50								D	-		13
T	Strength-5	ST5-1	1.25	1.50	1.00	0.50		1.35	1.35		0.40	1.00		Lx+W	R		14
		ST5-2	1.25	1.50	1.00	0.50		1.35	1.35		0.40	1.00		Ln+W	R		15
S		ST5-3	1.25	1.50	1.00	0.50		1.35	1.35		-0.40	-1.00		Lx+W	L		16
T		ST5-4	1.25	1.50	1.00	0.50		1.35	1.35		-0.40	-1.00		Ln+W	L		17
A		ST5-5	0.90	0.65	1.00	0.50		1.35	1.35		0.40	1.00		Lx+W	R		18
T		ST5-6	0.90	0.65	1.00	0.50		1.35	1.35		0.40	1.00		Ln+W	R		19
E		ST5-7	0.90	0.65	1.00	0.50		1.35	1.35		-0.40	-1.00		Lx+W	L		20
		ST5-8	0.90	0.65	1.00	0.50		1.35	1.35		-0.40	-1.00		Ln+W	L		21

Ⅱ) STRENGTH SECTIO

DISTRIBUTION OF STRAIN AND STRESS ON CALCULATION OF STRENGTH SECTION



ε_{cu} : ULTIMATE STRAIN OF CONCRETE (=0.0030)

ε_s : STRAIN OF REINFORCEMENT

σ_{ck} : CONCRETE STRENGTH (Mpa)

σ_s : STRESS OF REINFORCEMENT (Mpa)

d : EFFECTIVE HEIGHT OF SECTION (mm)

x : DISTANCE FROM COMPRESSIVE EXTREME FIBER TO NATURAL AXIS (mm)N

ε_{pe} : STRAIN OF TENDON DUE TO FEECTIVE PRESTRESS

FORCE BALANCING

$$Cu = Tu$$

$$Cu = 0.8 \cdot x \cdot b \cdot 0.85 \cdot \sigma_{ck} = 0.680 \cdot x \cdot \sigma_{ck}$$

b : WIDTH OF COMPRESSION FLANGE

COMPATIBILITY CONDITION OF STAIN

$$\frac{x}{\varepsilon_{cu}} = \frac{d - x}{\varepsilon_s - \varepsilon_{pe}}$$

RESISTANCE OF SECTION

$$Mr = Tu \cdot (d - 0.4 \cdot x)$$

$$Tu = As \cdot \sigma_s$$

a) REINFORCEMENT BAR

$Es \cdot \varepsilon_s < \sigma_{sy}$ ·············· $\sigma_s = Es \cdot \varepsilon_s$

$Es \cdot \varepsilon_s \geqq \sigma_{sy}$ ·············· $\sigma_s = \sigma_{sy}$

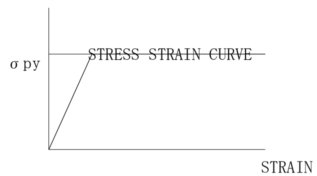
b) PC CABLE

$Eps \cdot \varepsilon_s < 0.84 \cdot \sigma_{pu}$ ······· $\sigma_s = Eps \cdot \varepsilon_s$

$0.84 \cdot \sigma_{pu} \leqq Eps \cdot \varepsilon_s < 0.93 \cdot \sigma_{pu}$ ·····

$$\sigma_s = 0.84 \cdot \sigma_{pu} + \frac{\varepsilon_s - \frac{0.84 \cdot \sigma_{pu}}{Ep}}{0.015 - \frac{0.84 \cdot \sigma_p}{Eps}} \cdot 0.09 \cdot \sigma_{pu}$$

$\varepsilon_s \geqq 0.015$ ·················· $\sigma_s = 0.93 \cdot \sigma_{pu}$



REBAR

Mr : NOMINAL RESISTANCE OF SECTION

σ_{sy} : YIELD STRENGTH OF REBAR (σ_{sy} = 400 M

σ_{pu} : YIELD STRENGTH OF PC CABLE (σ_{pu} = 1860 Mpa)

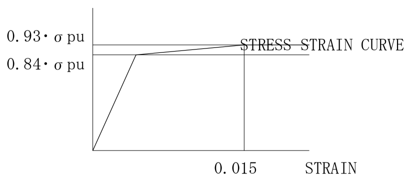
σ_s : STRESS OF REBAR (Mpa)

Es : MODULUS OF ELASTICITY OF REINFORCEMENT (Mpa)

ε_s : STRAIN OF REINFORCEMENT

As : ALL AREA OF TENSILE(mm^2)FORCEMENT (mm2)

d : EFFECTIVE HEIGHT OF SECTION (mm)



PC CABLE

Ⅲ) SEFETY DEGREE FOR BENDING MOMENT STRENGTH

$$Fs = \frac{Mr}{Mu}$$

No

5．1 STRENGTH CHECK FOR TOP SLAB

5．1．1 MEMBER REINFORCED STRESSING STEEL

			2	5	9	11	13	17	20
GIRDER-SELF LOAD			-38.90	-23.94	-0.23	2.75	-0.12	-23.63	-38.46
SUPERIMPOSED			-44.70	-10.47	-2.64	-0.78	-0.74	-5.16	-36.91
SECONDARY MOMENT	CONST.			-18.28	-18.50	-18.62	-18.75	-18.97	
	SERVICE			-16.84	-17.04	-17.16	-17.27	-17.48	
CREEP				1.44	1.45	1.46	1.47	1.49	
LIVE LOAD(LL+IM)		MAX.				63.32			
		MIN.	-103.86	-77.27				-77.27	-103.86
WS	→	1.00·WS	5.36	3.10	1.11	0.00	-1.11	-3.10	-5.36
	←	1.00·WS	-5.36	-3.10	-1.11	0.00	1.11	3.10	5.36
WL	→	1.00·WL	3.01	1.75	0.63	0.00	-0.63	-1.75	-3.01
	←	1.00·WL	-3.01	-1.75	-0.63	0.00	0.63	1.75	3.01
STRENGTH LIMIT STATE	Strength-1	ST1- 1	-115.68	-63.18	-22.01	95.19	-19.26	-55.51	-103.43
		ST1- 2	-297.43	-198.40	-22.01	-15.62	-19.26	-190.73	-285.19
		ST1- 3	-64.06	-45.90	-19.69	94.89	-18.60	-42.85	-58.60
		ST1- 4	-245.82	-181.13	-19.69	-15.92	-18.60	-178.07	-240.36
		MAX.	-64.06	-45.90	-19.69	95.19	-18.60	-42.85	-58.60
		MIN.	-297.43	-198.40	-22.01	-15.92	-19.26	-190.73	-285.19
	Strength-2	ST2- 1	-115.68	-63.18	-22.01	69.86	-19.26	-55.51	-103.43
		ST2- 2	-255.89	-167.49	-22.01	-15.62	-19.26	-159.82	-243.65
		ST2- 3	-64.06	-45.90	-19.69	69.56	-18.60	-42.85	-58.60
		ST2- 4	-204.28	-150.22	-19.69	-15.92	-18.60	-147.16	-198.81
		MAX.	-64.06	-45.90	-19.69	69.86	-18.60	-42.85	-58.60
		MIN.	-255.89	-167.49	-22.01	-15.92	-19.26	-159.82	-243.65
	Strength-3	ST3- 1	-108.17	-58.83	-20.45	-15.62	-20.82	-59.85	-110.94
		ST3- 2	-123.18	-67.52	-23.57	-15.62	-17.71	-51.16	-95.93
		ST3- 3	-56.56	-41.56	-18.13	-15.92	-20.15	-47.20	-66.11
		ST3- 4	-71.57	-50.25	-21.25	-15.92	-17.04	-38.50	-51.10
		MAX.	-56.56	-41.56	-18.13	-15.62	-17.04	-38.50	-51.10
		MIN.	-123.18	-67.52	-23.57	-15.92	-20.82	-59.85	-110.94
	Strength-4	ST4- 1	-125.40	-69.16	-22.07	-14.93	-19.29	-61.42	-113.05
	Strength-5	ST5- 1	-110.52	-60.19	-20.94	69.86	-20.33	-58.50	-108.59
		ST5- 2	-250.73	-164.50	-20.94	-15.62	-20.33	-162.81	-248.80
		ST5- 3	-120.83	-66.17	-23.08	69.86	-18.19	-52.52	-98.28
		ST5- 4	-261.04	-170.48	-23.08	-15.62	-18.19	-156.83	-238.49
		ST5- 5	-58.91	-42.92	-18.62	69.56	-19.67	-45.84	-63.76
		ST5- 6	-199.12	-147.23	-18.62	-15.92	-19.67	-150.15	-203.97
		ST5- 7	-69.22	-48.89	-20.76	69.56	-17.53	-39.86	-53.45
		ST5- 8	-209.43	-153.21	-20.76	-15.92	-17.53	-144.18	-193.66
		MAX.	-58.91	-42.92	-18.62	69.86	-17.53	-39.86	-53.45
		MIN.	-261.04	-170.48	-23.08	-15.92	-20.33	-162.81	-248.80

No

STRENGTH RESISTANCE AND RATIO

Mn = Aps·fps·(dp-a/2) + As·fy·(ds-a/2) - As'·fy'·(ds'-a/2)

Aps = ctc 750.0 = 554.8 mm2
As = D14 ctc 125.0 = 1232.0 mm2
As' = D14 ctc 125.0 = 1232.0 mm2

fps = fpu·(1-k·c/dp)

k = 2·(1.04-fpy/fpu)

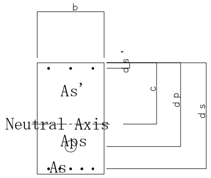
$$c = \frac{Aps\cdot fpu + As\cdot fy - As'\cdot fy}{0.85\cdot fc'\cdot \beta 1\cdot b + k\cdot Aps\cdot fpu/dp}$$

a = β1·c

β1 = 0.85 - (0.05·(fc'-28)/7)

$$\phi = 0.9 + 0.10\cdot \frac{Aps\cdot fpy}{Aps\cdot fpy + As\cdot fy}$$

fpu = 1860.0 Mpa
fpy = 1674.0 Mpa
fy = 400.0 Mpa
fc' = 45.0 Mpa



			2	5	11	17	20
EFFECTIVE HEIGHT	dp	mm	460.0	460.0	125.0	460.0	460.0
	ds	mm	485.0	485.0	185.0	485.0	485.0
	ds'	mm	65.0	65.0	65.0	65.0	65.0
PC-STEEL STRESS	β1		0.7286	0.7286	0.7286	0.7286	0.7286
	k		0.2800	0.2800	0.2800	0.2800	0.2800
	a	mm	26.384	26.384	24.912	26.384	26.384
	fps	Mpa	1819.0	1819.0	1717.5	1819.0	1819.0
STRENGTH RESISTANCE	c	mm	36.2	36.2	34.2	36.2	36.2
MOMENT AT STRENGTH LIMIT STATE	Mu	kN·m	-297.4	-198.4	95.2	-190.7	-285.2
NOMINAL FLEXURAL STRENGTH	Mn	kN·m	657.9	657.9	166.4	657.9	657.9
RESISTANCE FACTER	φ		0.965	0.965	0.965	0.965	0.965
	φ·Mn	kN·m	635.1	635.1	160.6	635.1	635.1
RESISTANCE	Fs		2.135	3.201	1.687	3.330	2.227

No

5．2 STRENGTH CHECK FOR BOTTOM SLAB AND WEB

COLLECTION TABLE (WEB kN·m)

			23	26	29	32	35	38	41
CASE 4			0.65	22.86	45.07	-62.08	-7.89	-25.05	40.19
CASE 5			67.12	128.73	72.57	37.31	46.13	40.84	51.93
CASE 6			45.07	22.86	0.65	-25.05	40.19	-62.08	-7.89
CASE 7			-25.03	49.00	-19.57	-118.17	-37.83	-114.64	-32.03
CASE 8			-34.02	-34.02	-34.02	64.82	-24.03	64.82	-24.03
CASE 9			56.00	119.59	61.45	22.70	35.96	26.23	41.76
GIRDER-SELF LOAD			0.22	61.22	0.49	-12.53	-11.50	-12.38	-11.21
SUPERIMPOSED			13.96	16.29	18.62	-32.55	9.03	-29.87	13.95
SECONDARY MOMENT	CONST.		3.65	3.93	4.22	17.08	4.98	17.85	5.62
	SERVICE		3.36	3.62	3.89	15.73	4.59	16.45	5.18
CREEP			-0.29	-0.31	-0.33	-1.34	-0.39	-1.40	-0.44
LIVE LOAD(LL+IM)		MAX.	45.72	45.72	45.72	64.82	40.19	64.82	40.19
		MIN.	-34.02	-34.02	-34.02	-87.13	-31.92	-87.13	-31.92
WS	→	0.30·WS	0.69	0.00	-0.69	0.57	0.74	-0.57	-0.74
	←	0.30·WS	-0.69	0.00	0.69	-0.57	-0.74	0.57	0.74
WL	→	1.00·WL	1.29	0.00	-1.29	1.07	1.39	-1.07	-1.39
	←	1.00·WL	-1.29	0.00	1.29	-1.07	-1.39	1.07	1.39
STRENGTH LIMIT STATE	Strength-1	ST1- 1	104.73	184.75	112.60	65.36	74.29	70.30	82.65
		ST1- 2	-34.81	45.21	-26.93	-200.55	-51.90	-195.61	-43.54
		ST1- 3	92.79	149.47	96.61	97.41	70.64	100.03	74.71
		ST1- 4	-46.75	9.94	-42.93	-168.50	-55.55	-165.89	-51.48
		MAX.	104.73	184.75	112.60	97.41	74.29	100.03	82.65
		MIN.	-46.75	9.94	-42.93	-200.55	-55.55	-195.61	-51.48
	Strength-2	ST2- 1	86.44	166.46	94.32	39.43	58.21	44.37	66.57
		ST2- 2	-21.20	58.82	-13.33	-165.70	-39.13	-160.76	-30.78
		ST2- 3	74.50	131.19	78.32	71.48	54.56	74.10	58.64
		ST2- 4	-33.14	23.54	-29.32	-133.65	-42.79	-131.04	-38.71
		MAX.	86.44	166.46	94.32	71.48	58.21	74.10	66.57
		MIN.	-33.14	23.54	-29.32	-165.70	-42.79	-160.76	-38.71
	Strength-3	ST3- 1	27.93	104.74	29.38	-45.40	7.43	-45.81	8.84
		ST3- 2	21.51	104.74	35.80	-50.75	0.48	-40.46	15.79
		ST3- 3	15.99	69.46	13.39	-13.35	3.78	-16.09	0.91
		ST3- 4	9.57	69.46	19.81	-18.70	-3.17	-10.74	7.85
		MAX.	27.93	104.74	35.80	-13.35	7.43	-10.74	15.79
		MIN.	9.57	69.46	13.39	-50.75	-3.17	-45.81	0.91
	Strength-4	ST4- 1	24.77	120.04	32.72	-51.21	1.08	-46.23	9.51
	Strength-5	ST5- 1	88.65	166.46	92.11	41.27	60.60	42.53	64.18
		ST5- 2	-19.00	58.82	-15.53	-163.86	-36.75	-162.60	-33.16
		ST5- 3	84.24	166.46	96.52	37.60	55.83	46.21	68.96
		ST5- 4	-23.41	58.82	-11.12	-167.54	-41.52	-158.92	-28.39
		ST5- 5	76.71	131.19	76.11	73.32	56.95	72.26	56.25
		ST5- 6	-30.94	23.54	-31.53	-131.81	-40.40	-132.88	-41.10
		ST5- 7	72.29	131.19	80.52	69.64	52.17	75.94	61.02
		ST5- 8	-35.35	23.54	-27.12	-135.49	-45.17	-129.20	-36.32
		MAX.	88.65	166.46	96.52	73.32	60.60	75.94	68.96
		MIN.	-35.35	23.54	-31.53	-167.54	-45.17	-162.60	-41.10

No

5．1．2 DESIGN FOR LONGITUDINAL (RC COMPONENT)

BENDING MOMENT AT STRENGTH LIMIT STATE

A - A SECTION (BOX CENTER OF SPAN)

Ml = 31.520 (kN·m)

Mu = 1.75×31.520 = 78.800 (kN·m)

B - B SECTION (NEAR TIP OF OVERHANG)

Ml = 33.060 (kN·m)

Mu = 1.75×33.060 = 82.650 (kN·m)

STRENGTH RESISTANCE AND RATIO

Mn = As·fy·(d-a/2)

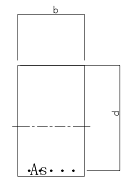
a = As·fy/(0.85·fc'·b)

fy = 400.0 Mpa

fc' = 45.0 Mpa

Asreq = 1/fy·(X-√(X^2-2·(X/d)·(Mu/φ)))

X = 0.85·fc'·b·d



		A-A	B-B
Asreq	mm2	1225.6	964.4
As	mm2	D16ctc125	D14ctc125
		1608.0	1232.0
d	mm	185.0	243.1
Mn	kN·m	113.58	116.62
φ		0.900	0.900
φ·Mn	kN·m	102.23	104.96
Mu	〃	78.80	82.65
Fs		1.297	1.270

MINIMUM REINFORCEMENT OF FLAXURAL COMPONENT (UPPER SLAB)

Mr AT LEAST EQUAL TO THE LESSER OF ;
1.2 TIMES THE CRACKING STRENGTH (1.2Mcr)
1.33 TIMES THE FACTORED MOMENT (1.33Mu)

$Mn \geqq 1.20 \cdot Mcr$
OR
$Mn \geqq 1.33 \cdot Mu$

$Mcr = Sc \cdot (fr+fcpe) - Mdnc \cdot (Sc/Snc-1)$

Mn : NOMINAL BENDING STRENGTH (Mn/ϕ)
Mcr : CRACKING MOMENT
Mu : MOMENT AT STRENGTH LIMIT STATE
fr : MODULUS OF RUPTURE
$fr = 0.63 \cdot \sqrt{(fc')}$
fc' : CONCRETE STRENGTH
Sc : SECTION COEFFICIENT
Sc/Snc : NON COMPOSIT >>> 1

$$fcpe = \frac{Np \cdot Pe}{Ac} + \frac{Np \cdot ep \cdot Pe}{Sc}$$

		2	5	11	17	20
b	m	1.000	1.000	1.000	1.000	1.000
d	m	0.550	0.550	0.250	0.550	0.550
Sc	m3	0.050	0.050	0.010	0.050	0.050
fr	Mpa	4.23	4.23	4.23	4.23	4.23
fcpe	Mpa	-1.19	-1.19	2.59	-1.22	-1.22
Mcr	kN·m	153.1	153.1	71.0	151.5	151.4
1.20·Mcr	kN·m	183.7	183.8	85.2	181.8	181.6
Mu	kN·m	-297.4	-198.4	95.2	-190.7	-285.2
1.33·Mu	kN·m	-395.6	-263.9	126.6	-253.7	-379.3
Mn	kN·m	657.9	657.9	166.4	657.9	657.9
Fs		3.582	3.580	1.952	3.619	3.622

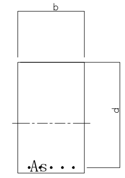
STRENGTH RESISTANCE AND RATIO

$Mn = As \cdot fy \cdot (d-a/2)$

$a = As \cdot fy/(0.85 \cdot fc' \cdot b)$

fy = 400.0 Mpa

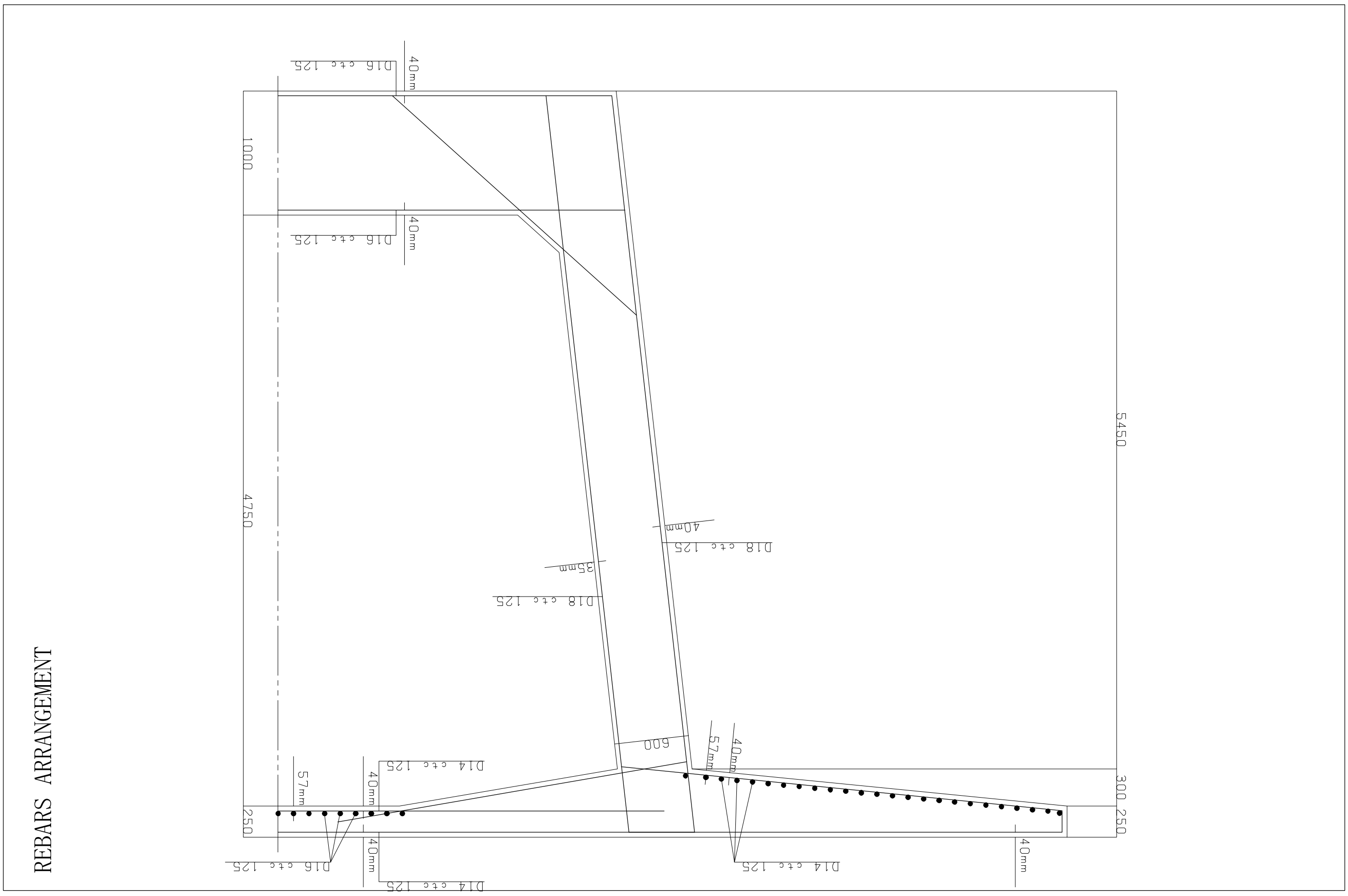
fc' = 45.0 Mpa



$Asreq = 1/fy \cdot (X-\sqrt{(X^2-2 \cdot (X/d) \cdot (Mu/\phi))})$

$X = 0.85 \cdot fc' \cdot b \cdot d$

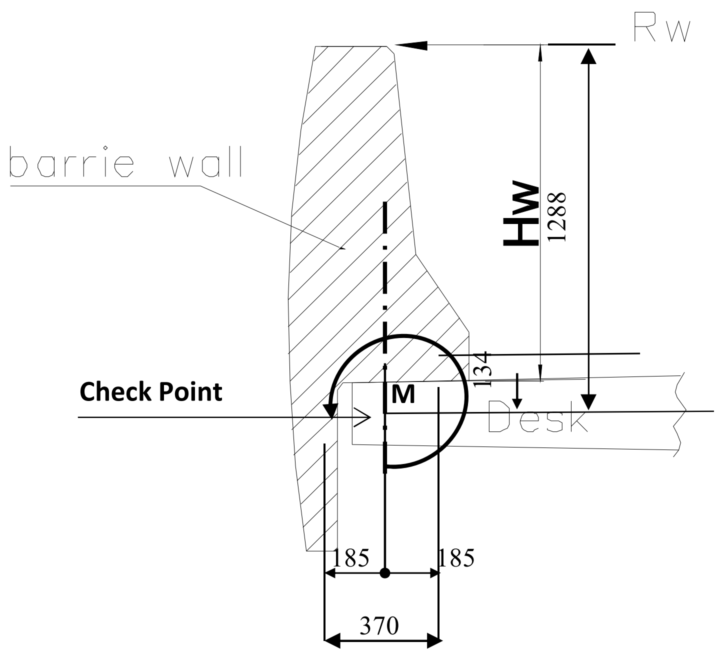
		BOTTOM SLAB		WEB	
		29 AT WEB	26 MID SPAN	41 OUTSIDE	32 INSIDE
Asreq	mm2	328.1	541.8	419.1	1013.4
As	mm2	D16ctc125	D16ctc125	D18ctc125	D18ctc125
		1608.0	1608.0	2032.0	2032.0
d	mm	955.0	950.0	550.0	555.0
Mn	kN·m	608.85	605.63	438.40	442.47
ϕ		0.900	0.900	0.900	0.900
ϕ·Mn	kN·m	547.96	545.07	394.56	398.22
Mu	〃	112.60	184.75	82.65	-200.55
Fs		4.866	2.950	4.774	1.986


No
REBARS ARRANGEMENT
D16 ctc 125
40mm
1000
D16 ctc 125
40mm
4750
5450
40mm
D18 ctc 125
35mm
D18 ctc 125
600
57mm
40mm
D14 ctc 125
57mm
40mm
300
250
250
D16 ctc 125
40mm
D14 ctc 125
D14 ctc 125
40mm

5.4 Check the Overhang at Vehicle Corrosion

(1) Check near the concrete barrier

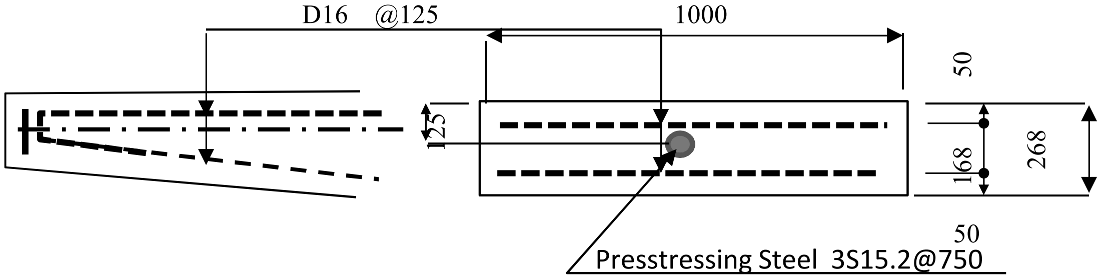
(1.1) Corrosion Load and Working Moment



	1070	
	84	
Hw=	1154	mm
	134	mm
h=	1288	mm
t=	268	mm
Rw=	881	kN
Lc=	4566	mm
Be=	5103	mm

M=	881	*	1.2882			=	1134.9	kNm
N=						=	881	kN
Mo=	M / Be	=	1135	/	5.103	=	222.4	kNm/m
No=	N /Be	=	881	/	5.103	=	172.6	kNm/m

(1.2) Check for Extreme Limit Event



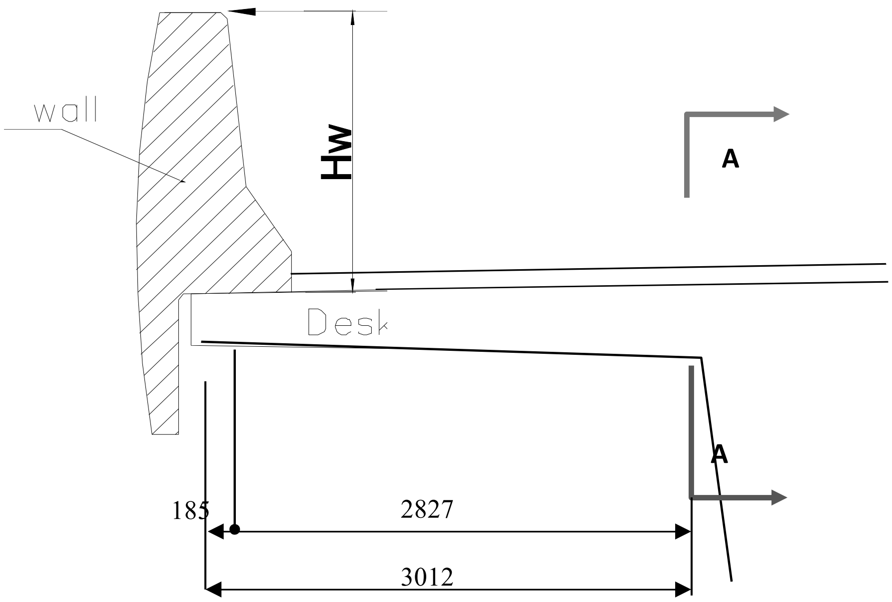
Prestressing Force after All Loss Pe= 620 kN/m

Area of Presstressing Steel Ap= 416.2 *1000 /750 = 554.9 mm2

Resistance Factor φm= 1.00

Item	Symbol	Unit	Result	Item	Symbol	Unit	Result
Rebar 1	As	mm2	1,608	Axis Force	Nu	kN	-172.6
Distance 1	ds	mm	218	Neutral Axis	cy	m	0.04819
Rebar 2	As'	mm2	1,608	Resistance Moment	Mn	kNm	222.92
Distance 2	ds'	mm	50	Reduction Factor	φm	-	1.00
Effective Prestress	Pe	kN	620	Factored Resistance Moment	Mr	kink	222.92
Prestressing Steel	Ap	mm2	554.9	Design Bending Moment	Mu	kNm	222.4
Distance	dip	mm	125	Safety	F	-	1.002
-	-	-	-	Judge	-	-	OK

(2) Check at Web Position **A - A Section**



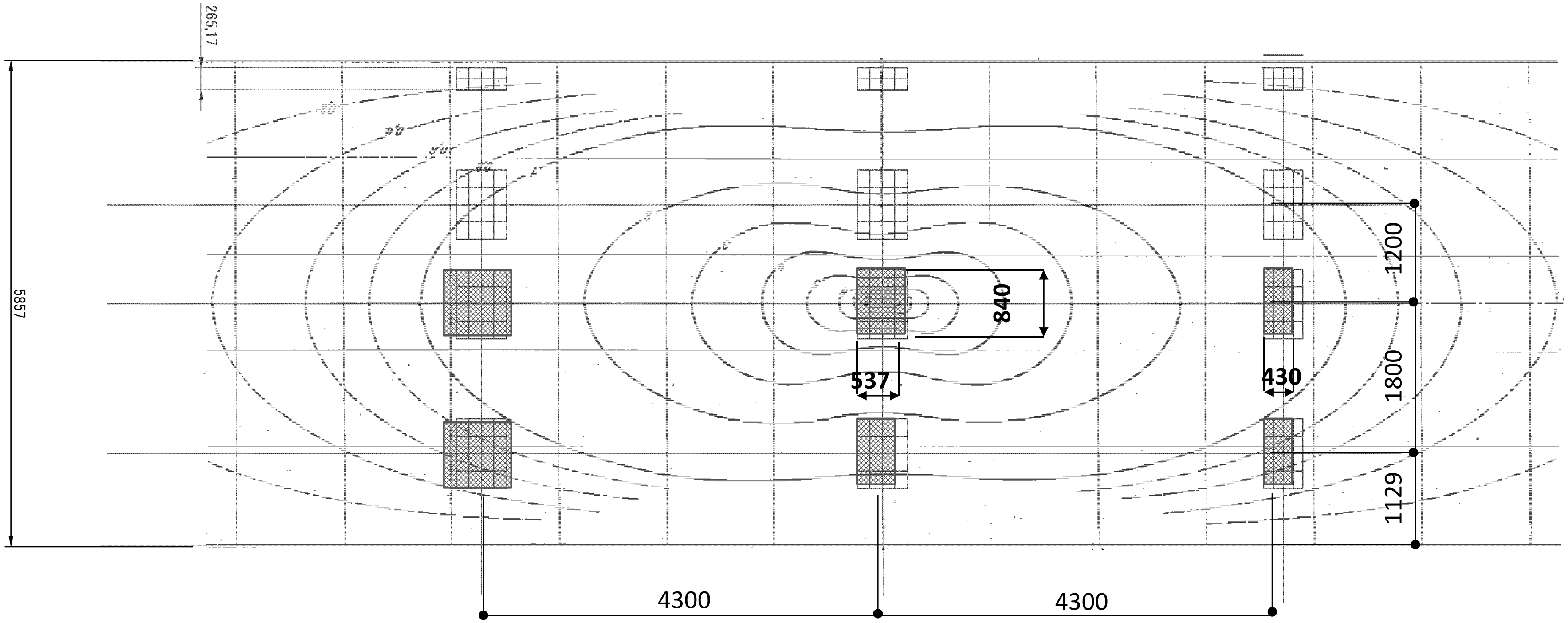
Bending Moment	Dead Load	DC=	33.9	kink/m	1.30
		Mew=	44.7	ken/m	1.50
	Live Load	Ml=	77.3	kink/m	1.00
	Collision Load	Mo	222.4	kink/m	
		No=	172.6	ken/m	
	Effective Width	Be=	10.757	m	(Be=5.103+2*2.827)
		Ma=	20.68	kink/m	
		Na=	-16.05	ken/m	1.00
	At Extleme Event	Mex=	188.4	kNm/m	1.00
		Nex=	-16.05	kn/m	

Item	Symbol	Unit	Result	Item	Symbol	Unit	Result
Rebar 1	As	mm2	16	Axis Force	Nu	ken	-16.1
Distance 1	ds	mm	8	Neutral Axis	cy	m	0.04819
Rebar 2	As'	mm2	16	Resistance Moment	Mn	kNm	222.92
Distance 2	ds'	mm	8	Reduction Factor	φm	-	1.00
Effective Prestress	Pe	kN	620	Factored Resistance Moment	Mr	kink	736.9
Prestressing Steel	Ap	mm2	554.9	Design Bending Moment	Mu	kNm	188.4
Distance	dp	mm	90	Safety	F	-	3.911
-	-	-	-	Judge	-	-	OK

Appedex 1-1 **The Pucher Influence Surface and Bending Moment**

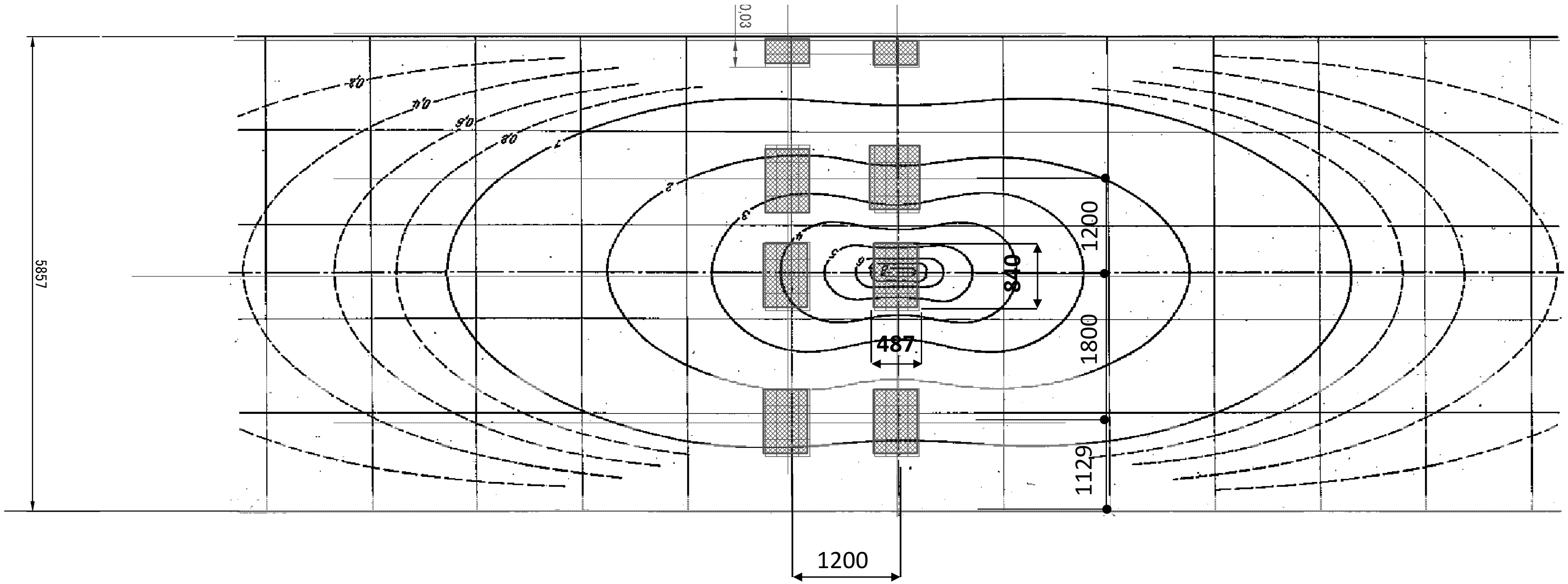
1 At Box Center Moment Mx At Box Center

1. 1 For Design Truck Load



Number of Vehicle	Nv=	1	
Pr =	90.6	kN	(At service and Included Impact)
Pf =	43.8	kN	(At service and Included Impact)
Cf=	12.614	-	
Cr=	3.02	-	
km=	1.2	-	
ki=	1.1	-	
kf=	0.9	-	
Mx=	60.29	kNm	(At service and Included Impact)

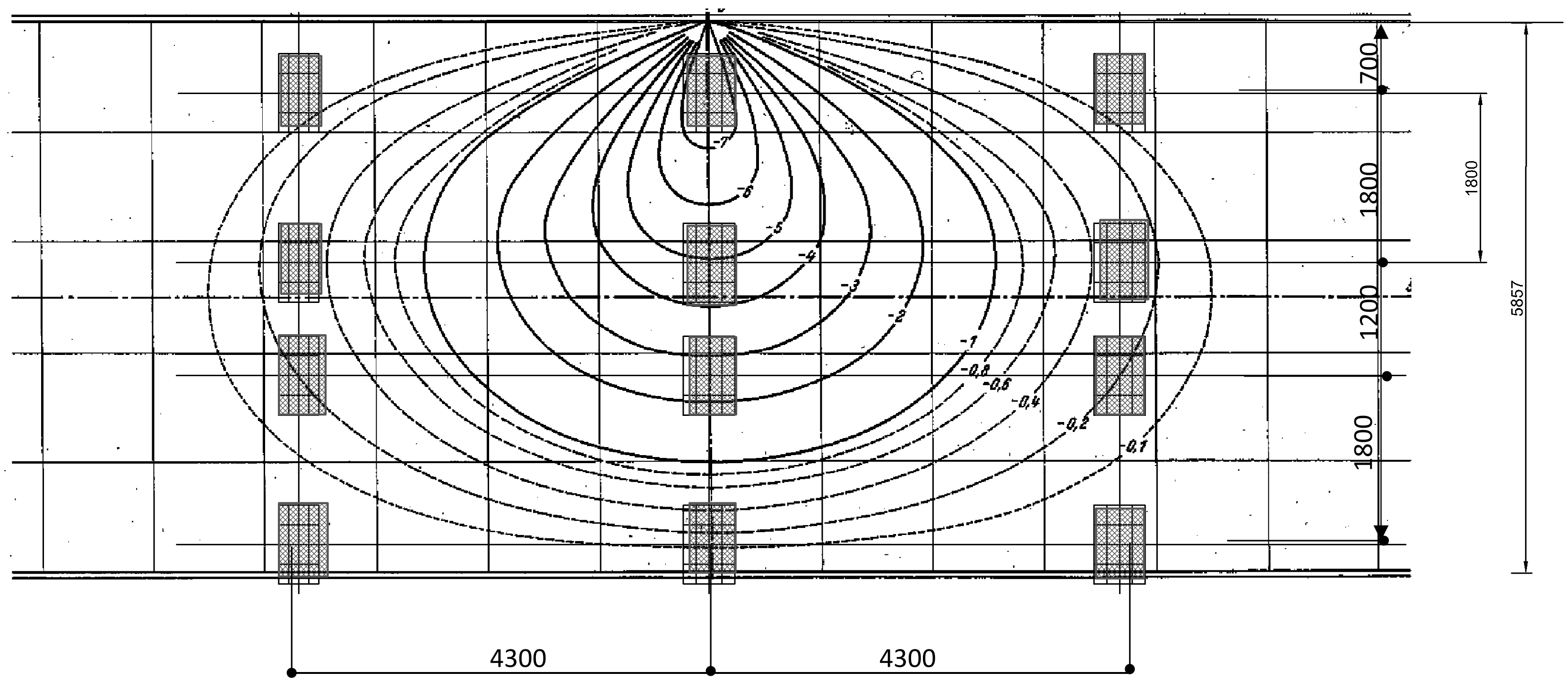
1\. 2 For Tandem Load Mx



Number of Vehicle	Nv=	2	
Pr =	68.75	kN	(At service and Included Impact)
Pf =	68.75	kN	(At service and Included Impact)
Cf=	19.13	-	
Cr=	4.256	-	
km=	1	-	
ki=	1.1	-	
kf=	9	-	
Mx=	63.32	kNm	(At service and Included Impact)

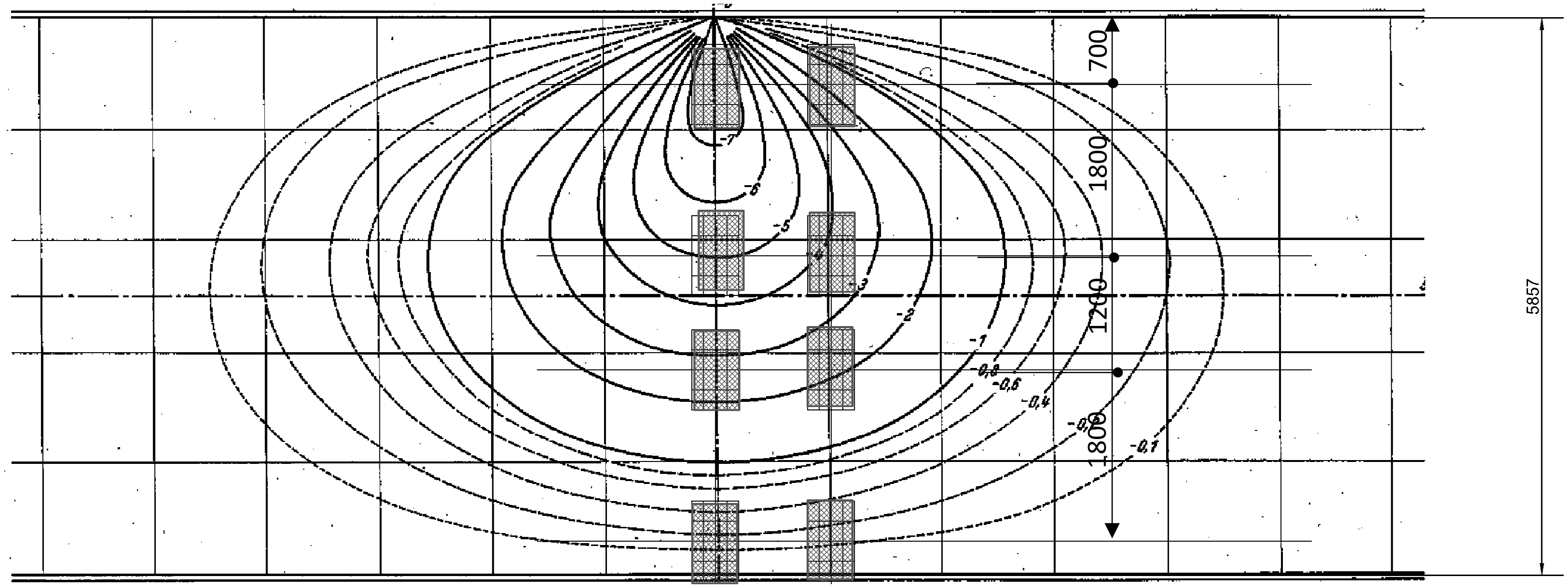
2 Moment Mx on The Web

2. 1 For Design Truck Load



Number of Vehicle	Nv=	2	
Pr =	90.6	kN	(At service and Included Impact)
Pf =	43.8	kN	(At service and Included Impact)
Cf=	16.89	-	
Cr=	1.6	-	
km=	1	-	
ki=	1.1	-	
kf=	1	-	
Mx=	77.27	kNm	(At service and Included Impact)

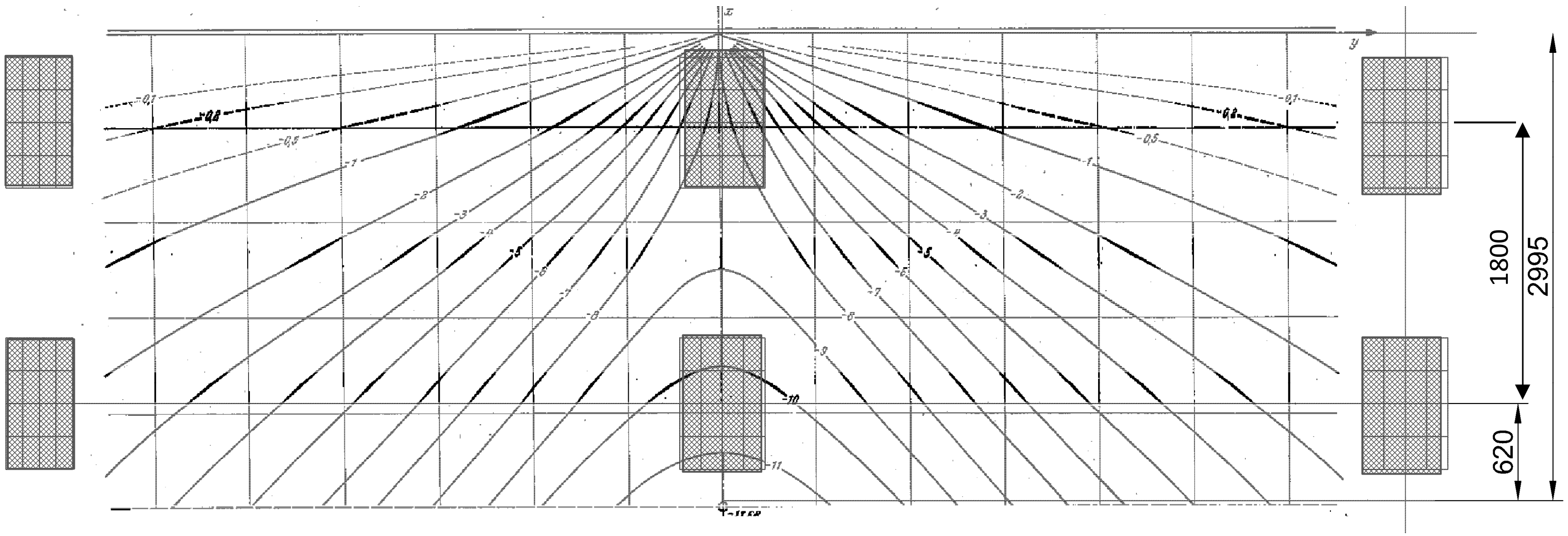
2 Moment Mx on The Web



Number of Vehicle	Nv=	2	
Pr =	68.75	kN	(At service and Included Impact)
Pr =	68.75	kN	(At service and Included Impact)
Cf=	20.43	-	
Cr=		-	
km=	1.0	-	
ki=	1.1	-	
kf=	1.0	-	
Mx=	73.75	kNm	(At service and Included Impact)

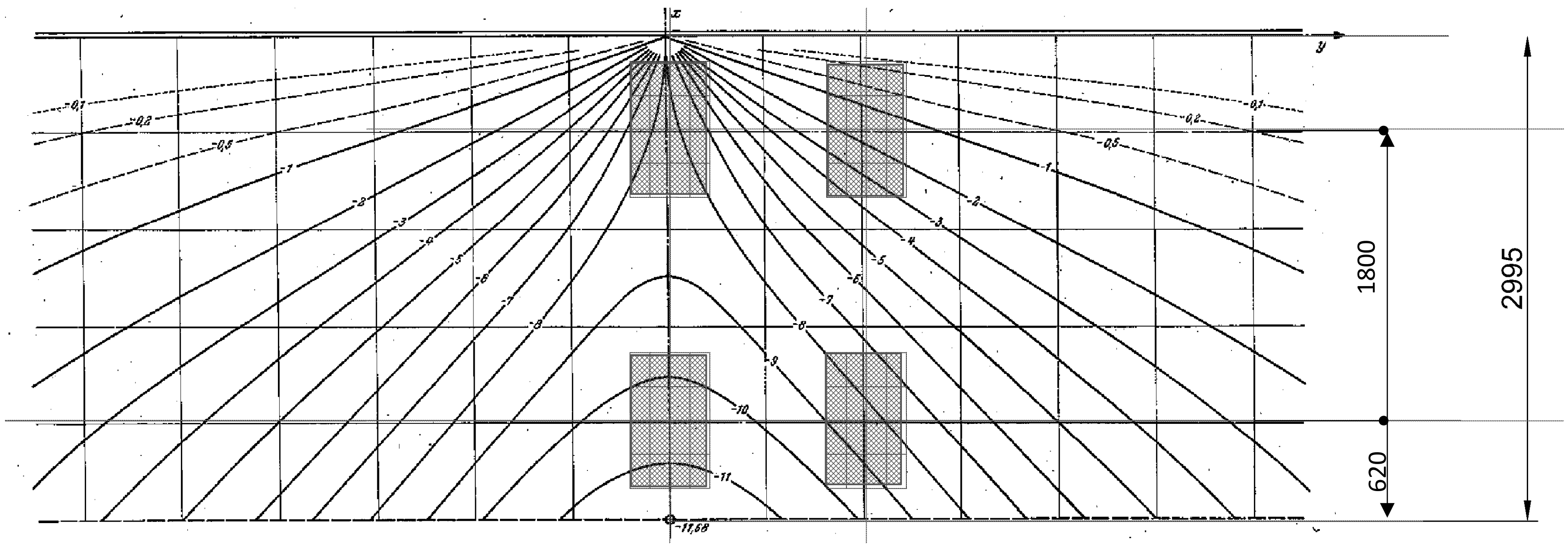
3 Moment Mx at Overhang

3. 1 For Design Truck Load



Number of Vehicle	Nv=	2	
Pr =	90.6	kN	(At service and Included Impact)
Pf =	43.8	kN	(At service and Included Impact)
Cf=	20.78	-	
Cr=	2.1	-	
km=	1	-	
ki=	1.1	-	
kf=	1	-	
Mx=	82.62	kNm	(At service and Included Impact)

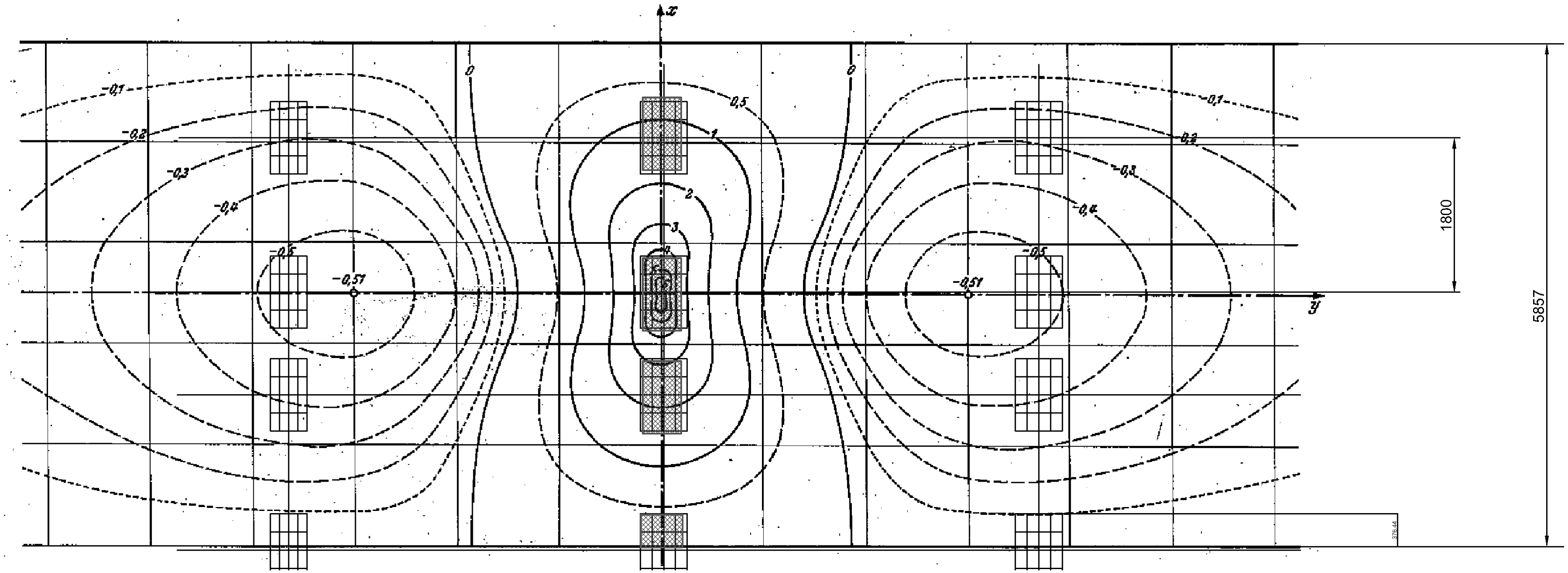
3. 2 For Design Tandem Load



Number of Vehicle	Nv=	2		
Pr =	68.75	kN		(At service and Included Impact)
Pr =	68.75	kN		(At service and Included Impact)
Cf=	28.76	-		
Cr=		-		
km=	1.0	-		
ki=	1.1	-		
kf=	1.0	-		
Mx=	103.86	kNm		(At service and Included Impact)

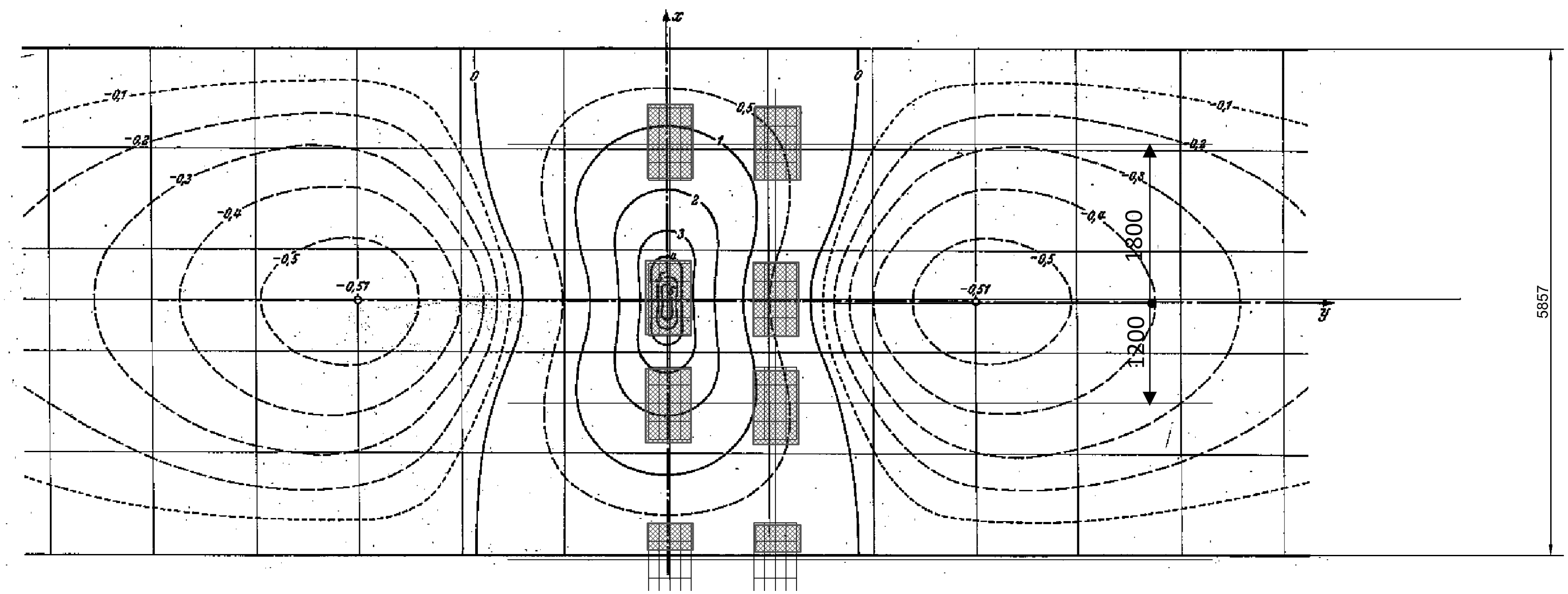
4 Moment My at Center My

4. 1 For Design Truck Load My



Number of Vehicle	Nv=	2	
Pr =	90.6	kN	(At service and Included Impact)
Pf =	43.8	kN	(At service and Included Impact)
Cf=	7.5	-	
Cr=	0.00	-	
km=	1	-	
ki=	1.1	-	
kf=	1	-	
Mx=	29.95	kNm	(At service and Included Impact)

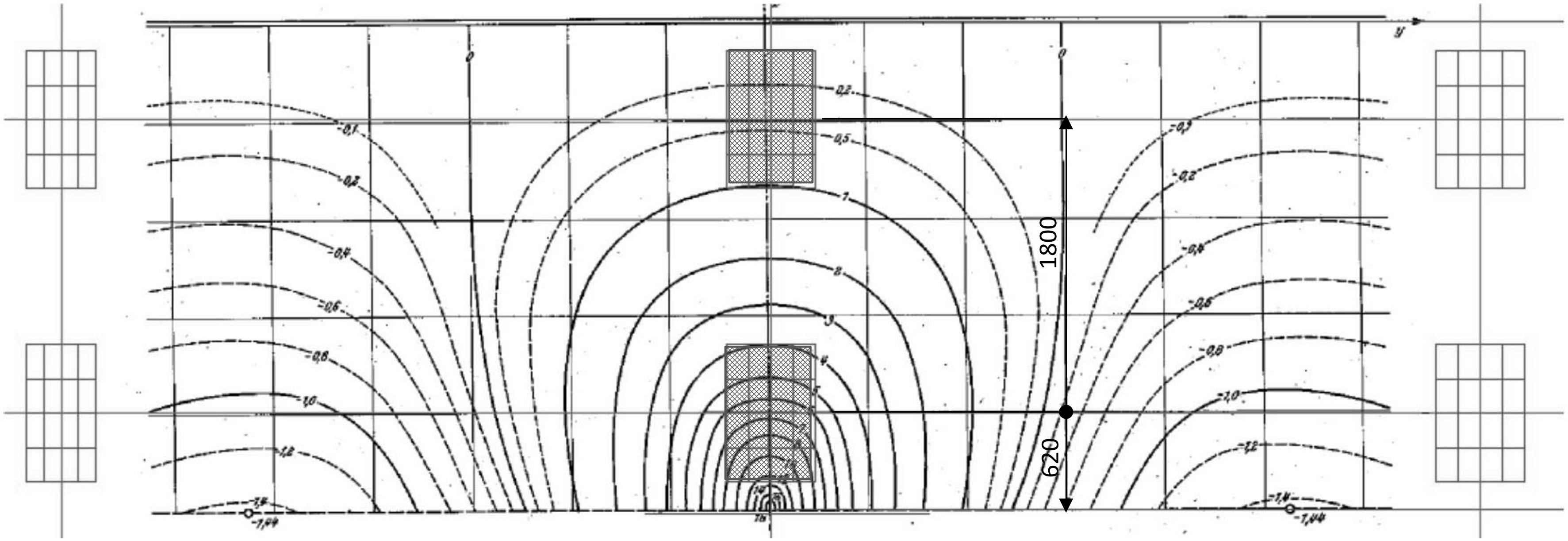
4. 2 For Design Tandem Load My



Number of Vehicle	Nv=	2	
Pr =	68.75	kN	(At service and Included Impact)
Pr =	68.75	kN	(At service and Included Impact)
Cf=	10.48	-	
Cr=	0	-	
km=	1.0	-	
ki=	1.1	-	
kf=	1.0	-	
Mx=	31.52	kNm	(At service and Included Impact)

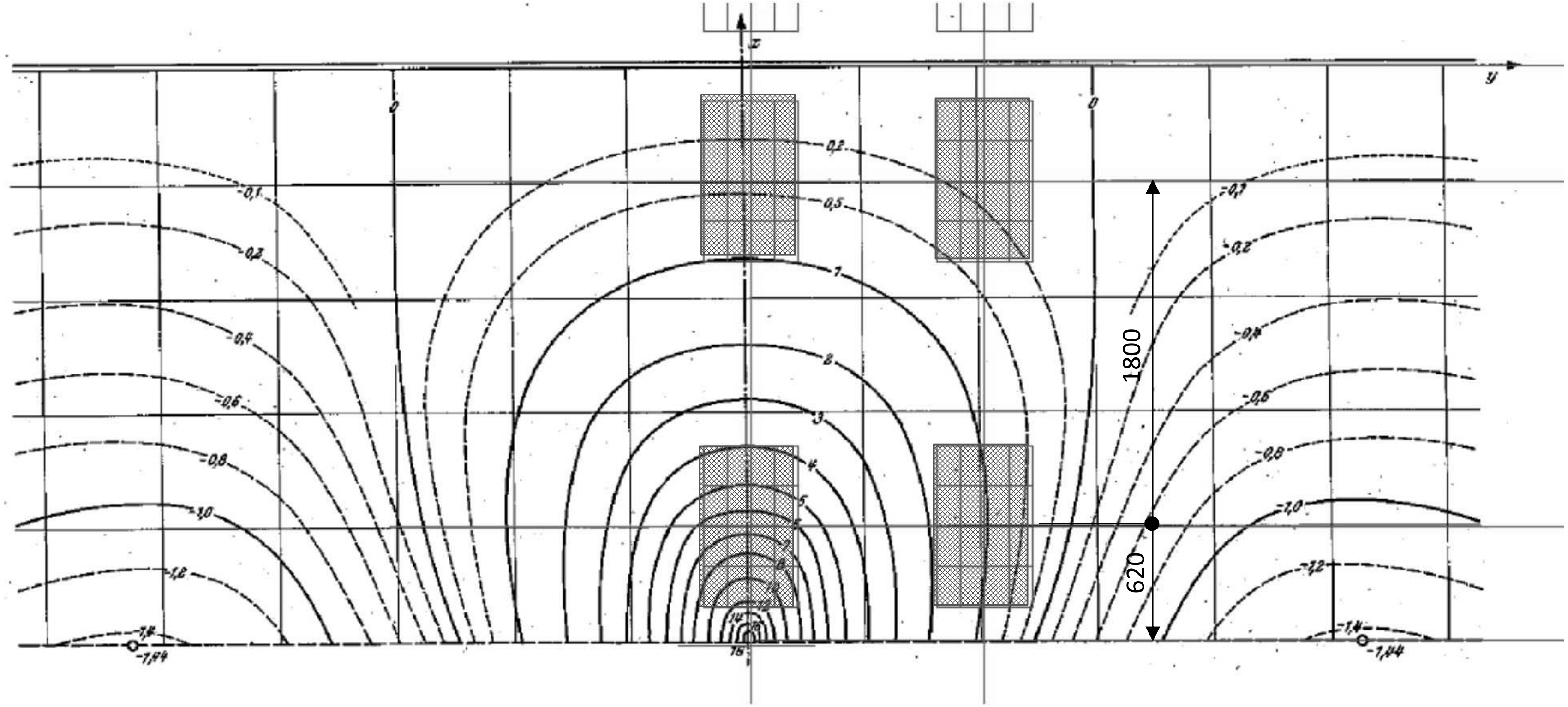
5 Moment My at Overhang

5. 2 For Design Truck Load My



Number ofVehiclee	Nv=	2	
Pr =	90.6	kN	(At service and Included Impact)
Pf =	43.8	kN	(At service and Included Impact)
Cf=	6.95	-	
Cr=	0.00	-	
km=	1	-	
ki=	1.1	-	
kf=	1	-	
Mx=	33.06	kNm	(At service and Included Impact)

5. 2 For Design Tandem Load My



Number of Viecle	Nv=	1		
Pr =	68.75	kN	(At service and Included Impact)
Pr =	68.75	kN	(At service and Included Impact)
Cf=	10.48	-	
Cr=	0	-	
km=	1.0	-	
ki=	1.1	-	
kf=	1.0	-	
Mx=	31.52	kNm	(At service and Included Impact)

Appendex 1-2 **Bending Moment due to Design Truck and Design Tandem Load**

Caluculation Method using the "Pucher Chart"

$$Ci = \iint f(x,y)dxdy$$

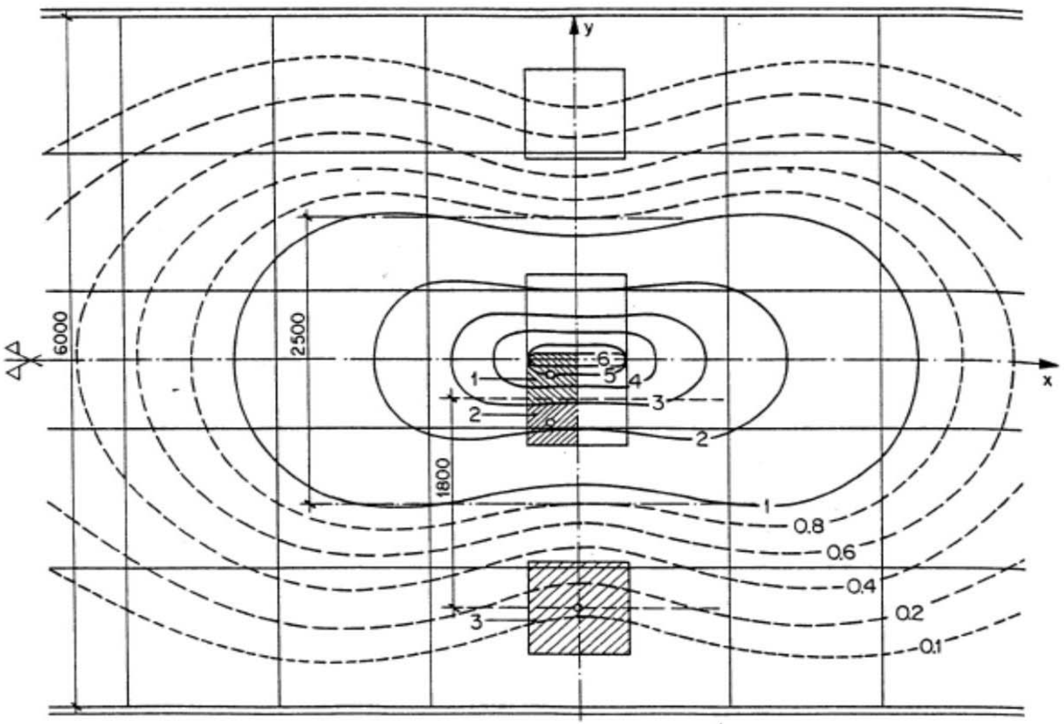
$$M = \frac{1}{8\pi}\left(P_r \cdot \sum C_{ri} + P_f \cdot \sum C_{fi}\right)$$

Ci : Influence Values are caluculated infuluence suface of Pucher Chart

Pr : Rear Wheel Load (kN)

Pf : Forwaord Wheel Load (kN)

M : Bending moment for the deck slab (kNm/m)



Exaple of Ifuence surface

Truck Load

		Unit	Load Case 1	2	3	4	Design Bending Moment
Center	Mx	kNm	60.29	45.67			60.29
Suport	Mx	kNm	−60.64	−60.15	−75.57	−77.27	−77.27
OverHang	Mx	kNm	−82.62				−82.62
Center	My	kNm	26.26	29.95			29.95
Overhang	My	kNm	33.06				33.06

Tandem Load

			Case 2: 1	2	3	4	Design Bending Moment
Center	Mx	kNm	62.64	63.32	0.00	0.00	63.32
Suport	Mx	kNm	−59.72	−68.60	−73.75	−68.53	−73.75
OverHang	Mx	kNm	−103.86	0.00	0.00	0.00	−103.86
Center	My	kNm	25.67	31.52	0.00	0.00	31.52
Overhang	My	kNm	29.58				29.58

(4) Tire Contact Area for Design of Deck Slab

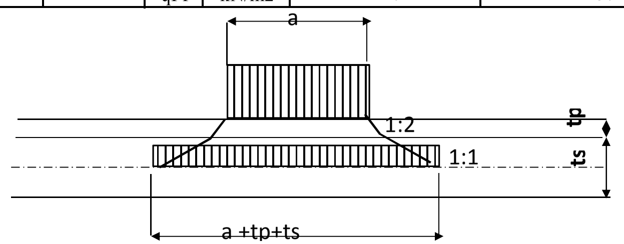
VN Standard

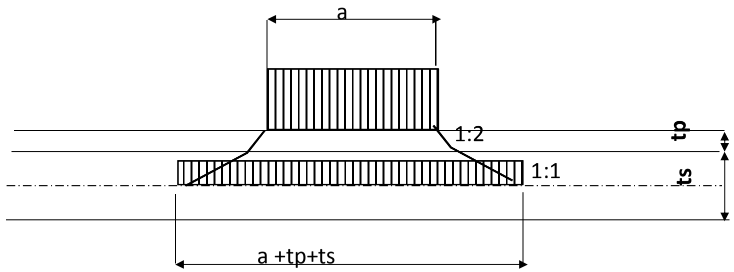
Truck Load

				At Service	At Strength
Tire Contact Area	B		mm	510	510
	Pr	P	kN	72.5	72.5
		IM	-	0.25	0.25
		γ	-	1.00	1.75
		k	-	2.28	2.28
		L	mm	207	362
		Pd	kN	90.6	158.6
	Pf	P	kN	35.0	35.0
		IM	-	0.25	0.25
		γ	-	1.00	1.75
		k	-	2.28	2.28
		L	mm	100	175
		Pd	kN	43.8	76.6
Distributed Unifomed Load	Pr	tp	mm	80	80
		ts	mm	250	250
		Be	mm	840	840
		Le	mm	537	692
		qPr	kN/m2	201.0	273.0
	Pf	tp	mm	80	80
		ts	mm	250	250
		Be	mm	840	692
		Le	mm	430	505
		qPf	kN/m2	121.2	219.4

Tandem Load

				At Service	At Strength
Tire Contact Area	B		mm	510	510
	Pi	P	kN	55	55
		IM	-	0.25	0.25
		γ	-	1.00	1.75
		k	-	2.28	2.28
		L	mm	157	274
		Pd	kN	68.8	120.3
Distributed Unifomed Load	Pi	tp	mm	80	80
		ts	mm	250	250
		Be	mm	840	840
		Le	mm	487	604
		qPr	kN/m2	168.1	237.0





1 **1.Centar Mx** Span Ls= 5.806 m

1 1 **CASE.1** Weel Number Nw= 1 Units

Multiple Presence Factor km= 1.20

Rear Wheel(Include Iimpact) Pr= 90.63 kN Nr= 4

Front Wheel(Include Impact) Pf= 43.75 kN Nf= 2

Pr Effective Area Ae= 0.840 0.537 0.450765 m2

Pf Effective Area Ae= 0.840 0.430 0.36099 m2

No	Iss	a	b	Ae	k	Cio	Ci
1	1	0.840	0.537	0.450765	1.000	6.09	6.094
2	1	0.840	0.537	0.450765	1.000	2.10	2.100
3	1	0.840	0.537	0.450765	1.000	1.40	1.400
4	1	0.840	0.537	0.450765	1.000	1.30	1.300
5	1	0.840	0.537	0.450765	1.000	1.00	1.000
6	1	0.840	0.537	0.450765	1.000	0.72	0.720
7	1	0.840	0	0	0.000	0.00	0.000
8	1	0.840	0	0	0.000	0.00	0.000
9	2	0.840	0.430	0.36099	1.000	1.30	1.300
10	2	0.840	0.430	0.36099	1.000	1.00	1.000
11	2	0.840	0.430	0.36099	1.000	0.72	0.720
12	2	0.840	0.265	0.2226	0.617	0.00	0.000
SumCr =	12.614	Pr=		90.63	kN	Pr*SumCr=	1143.25
SumCf =	3.020	Pf=		43.75	kN	Pf*SumCf=	132.13
Sum Ci*P	SumCr+SumCf			1275.37			
Mo	(1/8/π)*SumCi*P			50.75			
km	-			1.20			
ki	-			1.10			
M	km*ki*Mo			66.98	kNm		
kf				0.90			
Md	kf*M			60.29	kNm		

Influence Coefficient Pr1

1	Nx	4	I	Ⅱ	Ⅲ	IV	V
	Ny	4	1	2	3	4	5
	NO	nx	η1	η2	η3	η4	η5
	1	1	4.8	4.7	4.6	4.7	4.8
	2	4	6.0	6.0	6.0	6.0	6.0
	3	2	7.2	8.0	8.0	8.0	7.2
	4	4	6.0	6.0	6.0	6.0	6.0
	5	1	4.8	4.7	4.6	4.7	4.8
	Sum A		72.0	73.4	73.2	73.4	72.0
	Area Ai		6.00	6.12	6.10	6.12	6.00
	ny		1	4	2	4	1
	SumV		73.13				
	Cr1		6.09				

1 **1.Centar Mx** Tandem Span Ls= 5.806 m

1 1 **CASE.1** Weel Number Nw= 1 Units

Multiple Presence Factor km= 1.20

Rear Wheel(Include Iimpact) Pr= 68.75 kN Nr= 4

Front Wheel(Include Impact) Pf= 68.75 kN Nf= 4

Effective Area Ae= 0.840 0.487 0.40908 m2

No	Iss	a	b	Ae	k	Cio	Ci
1	1	0.84	0.487	0.40908	1.000	6.50	6.500
2	1	0.84	0.487	0.4091	1.000	4.20	4.200
3	1	0.84	0.487	0.4091	1.000	2.20	2.200
4	1	0.84	0.487	0.4091	1.000	2.50	2.500
5	1	0.84	0.487	0.4091	1.000	1.50	1.500
6	1	0.84	0.487	0.4091	1.000	1.50	1.500
7	1	0.84	0.330	0.2772	0.678	0.69	0.468
8	1	0.84	0.330	0.2772	0.678	0.60	0.407
9	0						
10	0						
11	0	0	0	0.0000	0.000	0.00	0.000
12	0	0	0	0.0000	0.000	0.00	0.000
SumCr =	19.27	Pr=		68.75	kN	Pr*SumCr=	1325.10
SumCf =	0.000	Pf=		68.75	kN	Pf*SumCf=	0.00
Sum Ci*P	SumCr+SumCf			1325.10			
Mo	(1/8/π)*SumCi*P			52.72			
km	-			1.20			
ki	-			1.10			
M	km*ki*Mo			69.60	kNm		
kf				0.90			
Md	kf*M			62.64	kNm		

Influence Coefficient

1	Nx	4	I	Ⅱ	Ⅲ	IV	V
	Ny	4	1	2	3	4	5
	NO	nx	η1	η2	η3	η4	η5
	1	1	4.6	4.5	4.4	4.5	4.6
	2	4	6.0	7.0	6.0	7.0	6.0
	3	2	7.0	8.0	8.0	8.0	7.0
	4	4	6.0	7.0	6.0	7.0	6.0
	5	1	4.6	4.5	4.4	4.5	4.6
	Sum A		71.2	81.0	72.8	81.0	71.2
	Area Ai		5.93	6.75	6.07	6.75	5.93
	ny		1	4	2	4	1
	SumV		78.00				
	Cr1		6.50				

1 **1.Centar Mx** Truck Span Ls= 5.806 m
1 2 **CASE.2** Weel Number Nw= 2 Units
Multiple Presence Factor km= 1.00
Rear Wheel(Include Iimpact) Pr= 90.63 kN Nr= 8
Front Wheel(Include Impact) Pf= 43.75 kN Nf= 4
Effective Area Ae= 0.840 0.537 0.450765 m2
Effective Area Ae= 0.840 0.430 0.3612 m2

No	Iss	a	b	Ae	k	Cio	Ci
1	1	0.840	0.537	0.4508	1.000	6.09	6.094
2	1	0.840	0.537	0.4508	1.000	2.10	2.100
3	1	0.840	0.537	0.4508	1.000	1.40	1.400
4	1	0.840	0.537	0.4508	1.000	1.30	1.300
5	1	0.840	0.537	0.4508	1.000	1.00	1.000
6	1	0.840	0.537	0.4508	1.000	0.72	0.720
7	0	0.840	0.430	0.3612	0.000	0.00	0.000
8	0	0.840	0.430	0.3612	0.000	0.00	0.000
9	2	0.840	0.430	0.3612	1.000	1.30	1.300
10	2	0.840	0.430	0.3612	1.000	1.00	1.000
11	2	0.840	0.430	0.3612	1.000	0.72	0.720
12	0	0.840	0.430	0.3612	0.000	0.00	0.000
SumCr =	12.614	Pr=	90.63	kN		Pr*SumCr=	1143.25
SumCf =	3.020	Pf=	43.75	kN		Pf*SumCf=	132.13
Sum Ci*P	SumCr+SumCf		1275.37				
Mo	(1/8/π)*SumCi*P		50.75				
km	-		1.00				
ki	-		1.00				
M	km*ki*Mo		50.75	kNm			
kf			0.90				
Md	kf*M		45.67	kNm			

Influence Coefficient

	Nx	4	I	Ⅱ	Ⅲ	IV	V
	Ny	4	1	2	3	4	5
	NO	nx	η1	η2	η3	η4	η5
	1	1	4.8	4.7	4.6	4.7	4.8
	2	4	6.0	6.0	6.0	6.0	6.0
	3	2	7.2	8.0	8.0	8.0	7.2
1	4	4	6.0	6.0	6.0	6.0	6.0
	5	1	4.8	4.7	4.6	4.7	4.8
	Sum A		72.0	73.4	73.2	73.4	72.0
	Area Ai		6.00	6.12	6.10	6.12	6.00
	ny		1	4	2	4	1
	SumV		73.13				
	Cr1		6.09				

1 **1.Centar Mx** Tandem Span Ls= 5.806 m
1 2 **CASE.2** Weel Number Nw= 2 Units
Multiple Presence Factor km= 1.00
Rear Wheel(Include Iimpact) Pr= 68.75 kN Nr= 4
Front Wheel(Include Impact) Pf= 68.75 kN Nf= 2
Effective Area Ae= 0.840 0.430 0.3612 m2

No	Iss	a	b	Ae	k	Cio	Ci
1	1	0.84	0.43	0.3612	1.000	6.50	6.500
2	1	0.84	0.61	0.5124	1.419	4.20	5.958
3	1	0.84	0.61	0.5124	1.419	2.20	3.121
4	1	0.84	0.61	0.5124	1.419	2.50	3.547
5	2	0.84	0.61	0.5124	1.419	1.50	2.128
6	2	0.84	0.61	0.5124	1.419	1.50	2.128
7	0	0.84	0.61	0.5124	1.419	0.69	0.979
8	0	0.84	0.61	0.5124	1.419	0.60	0.851
9	0	0.84	0.61	0.5124	1.419	0.00	0.000
10	0	0.84	0.61	0.5124	1.419	0.00	0.000
SumCr =	19.13	Pr=	68.75	kN		Pr*SumCr=	1314.88
SumCf =	4.256	Pf=	68.75	kN		Pf*SumCf=	292.60
Sum Ci*P	SumCr+SumCf		1607.48				
Mo	(1/8/π)*SumCi*P		63.96				
km	-		1.00				
ki	-		1.10				
M	km*ki*Mo		70.36	kNm			
kf			0.90				
Md	kf*M		63.32	kNm			

Influence Coefficient

	Nx	4	I	Ⅱ	Ⅲ	IV	V
	Ny	4	1	2	3	4	5
	NO	nx	η1	η2	η3	η4	η5
	1	1	4.6	4.5	4.4	4.5	4.6
	2	4	6.0	7.0	6.0	7.0	6.0
	3	2	7.0	8.0	8.0	8.0	7.0
1	4	4	6.0	7.0	6.0	7.0	6.0
	5	1	4.6	4.5	4.4	4.5	4.6
	Sum A		71.2	81.0	72.8	81.0	71.2
	Area Ai		5.93	6.75	6.07	6.75	5.93
	ny		1	4	2	4	1
	SumV		78.00				
	Cr1		6.50				

3 **Suport** **Mx** Truck Span Ls= 5.857 m

3 1 **Case1** Weel Number Nw= 1 Units

Multiple Presence Factor km= 1.20

Rear Wheel(Include Iimpact) Pr= 90.63 kN Nr= 4

Front Wheel(Include Impact) Pf= 43.75 kN Nf= 2

Effective Area Ae= 0.840 0.537 0.45108 m2

Effective Area Ae= 0.840 0.430 0.3612 m2

No	Iss	a	b	Ae	k	Cio	Ci
1	1	0.840	0.537	0.45108	1.000	6.65	6.647
2	1	0.840	0.537	0.4511	1.000	5.50	5.500
3	0	0.840	0.537	0.4511	0.000	3.20	0.000
4	0	0.840	0.537	0.4511	0.000	0.40	0.000
5	1	0.840	0.537	0.4511	1.000	0.10	0.100
6	1	0.840	0.537	0.4511	1.000	0.30	0.300
7	0	0.840	0.537	0.4511	0.000	0.25	0.000
8	0	0.840	0.537	0.4511	0.000	0.10	0.000
9	2	0.840	0.430	0.3612	1.000	0.10	0.100
10	2	0.840	0.430	0.3612	1.000	0.30	0.300
11	0	0.840	0.430	0.3612	0.000	1.10	0.000
12	0	0.840	0.430	0.3612	0.000	0.10	0.000
SumCr =	12.55		Pr=	90.63	kN	Pr*SumCr=	1137.15
SumCf =	0.400		Pf=	43.75	kN	Pf*SumCf=	17.50
Sum Ci*P	SumCr+SumCf			1154.65			
Mo	(1/8/π)*SumCi*P			45.94			
km	-			1.20			
ki	-			1.10			
M	km*ki*Mo			60.64	kNm		
kf				1.00			
Md	kf*M			60.64	kNm		

Influence Coefficient

1	Nx	4	I	Ⅱ	Ⅲ	IV	V
	Ny	4	1	2	3	4	5
	NO	nx	η1	η2	η3	η4	η5
	1	1	4.0	7.0	8.0	7.0	4.0
	2	4	4.0	6.0	7.8	6.0	4.0
	3	2	6.0	7.0	7.5	7.0	6.0
	4	4	6.8	7.0	7.4	7.0	6.8
	5	1	7.0	7.3	7.4	7.3	7.0
	Sum A		66.2	80.3	91.2	80.3	66.2
	Area Ai		5.52	6.69	7.60	6.69	5.52
	ny		1	4	2	4	1
	SumV		79.77				
	Cr1		6.65				

3 **Suport** **Mx** Dandem Span Ls= 5.857 m

3 1 **Case1** Weel Number Nw= 1 Units

Multiple Presence Factor km= 1.20

Rear Wheel(Include Iimpact) Pr= 68.75 kN Nr= 4

Front Wheel(Include Impact) Pf= 68.75 kN Nf= 0

Effective Area Ae= 0.840 0.487 0.40908 m2

No	Iss	a	b	Ae	k	Cio	Ci
1	1	0.84	0.487	0.4091	1.000	6.44	6.439
2	1	0.84	0.487	0.4091	1.000	5.50	5.500
3	1	0.84	0.487	0.4091	1.000	0.60	0.600
4	1	0.84	0.487	0.4091	1.000	4.00	4.000
5	0	0.84	0.487	0.4091	0.000	3.10	0.000
6	0	0.84	0.487	0.4091	0.000	0.38	0.000
7	0	0.84	0.487	0.4091	0.000	2.50	0.000
8	0	0.84	0.487	0.4091	0.000	0.28	0.000
9	0	0.84	0.487	0.4091	0.000	0.00	0.000
10	0	0.84	0.487	0.4091	0.000	0.00	0.000
SumCr =	16.54		Pr=	68.75	kN	Pr*SumCr=	1137.05
SumCf =	0.000		Pf=	68.75	kN	Pf*SumCf=	0.00
Sum Ci*P	SumCr+SumCf			1137.05			
Mo	(1/8/π)*SumCi*P			45.24			
km	-			1.20			
ki	-			1.10			
M	km*ki*Mo			59.72	kNm		
kf				1.00			
Md	kf*M			59.72	kNm		

Influence Coefficient

1	Nx	4	I	Ⅱ	Ⅲ	IV	V
	Ny	4	1	2	3	4	5
	NO	nx	η1	η2	η3	η4	η5
	1	1	0.1	0.5	8.0	0.5	0.1
	2	4	3.0	6.8	7.8	6.8	3.0
	3	2	6.0	7.1	7.7	7.1	6.0
	4	4	7.0	7.2	7.5	7.2	7.0
	5	1	7.1	7.4	7.4	7.4	7.1
	Sum A		59.2	78.1	92.0	78.1	59.2
	Area Ai		4.93	6.51	7.67	6.51	4.93
	ny		1	4	2	4	1
	SumV		77.27				
	Cr1		6.44				

3 Suport Mx Truck Span Ls= 5.857 m
3 2 Case2 Weel Number Nw= 1 Units
Multiple Presence Factor km= 1.20
Rear Wheel(Include Iimpact) Pr= 90.63 kN Nr= 4
Front Wheel(Include Impact) Pf= 43.75 kN Nf= 2
Effective Area Ae= 0.840 0.537 0.45108 m2
Effective Area Ae= 0.840 0.430 0.3612 m2

No	Iss	a	b	Ae	k	Cio	Ci
1	1	0.840	0.537	0.4511	1.000	7.04	7.043
2	1	0.840	0.537	0.4511	1.000	5.00	5.000
3	1	0.840	0.537	0.4511	1.000	0.10	0.100
4	1	0.840	0.537	0.4511	1.000	0.30	0.300
5	2	0.840	0.430	0.3612	1.000	0.10	0.100
6	2	0.840	0.430	0.3612	1.000	0.30	0.300
7	0	0.000	0	0.0000	0.000	0.00	0.000
8	0	0.000	0	0.0000	0.000	0.00	0.000
9	0	0.000	0	0.0000	0.000	0.00	0.000
10	0	0.000	0	0.0000	0.000	0.00	0.000
SumCr =	12.44		Pr=	90.63	kN	Pr*SumCr=	1127.71
SumCf =	0.400		Pf=	43.75	kN	Pf*SumCf=	17.50
Sum Ci*P	SumCr+SumCf			1145.21			
Mo	(1/8/π)*SumCi*P			45.57			
km	-			1.20			
ki	-			1.10			
M	km*ki*Mo			60.15	kNm		
kf				1.00			
Md	kf*M			60.15	kNm		

Influence Coefficient

	Nx	4	I	Ⅱ	Ⅲ	IV	V
	Ny	4	1	2	3	4	5
	NO	nx	η1	η2	η3	η4	η5
	1	1	5.0	7.0	7.7	7.0	5.0
	2	4	6.0	6.7	7.6	6.7	6.0
	3	2	7.0	7.3	7.7	7.3	7.0
1	4	4	6.0	7.5	7.6	7.5	6.0
	5	1	7.0	7.1	7.2	7.1	7.0
	Sum A		74.0	85.5	91.1	85.5	74.0
	Area Ai		6.17	7.13	7.59	7.13	6.17
	ny		1	4	2	4	1
	SumV		84.52				
	Cr1		7.04				

3 Suport Mx Dandem Span Ls= 5.806 m
3 2 Case2 Weel Number Nw= 2 Units
Multiple Presence Factor km= 1.00
Rear Wheel(Include Iimpact) Pr= 68.75 kN Nr= 4
Front Wheel(Include Impact) Pf= 68.75 kN Nf= 4
Effective Area Ae= 0.840 0.487 0.40908 m2

No	Iss	a	b	Ae	k	Cio	Ci
1	1	0.840	0.487	0.4091	1.000	6.44	6.439
2	1	0.840	0.487	0.4091	1.000	5.50	5.500
3	1	0.840	0.487	0.4091	1.000	0.60	0.600
4	1	0.840	0.487	0.4091	1.000	4.00	4.000
5	1	0.840	0.487	0.4091	1.000	3.10	3.100
6	1	0.840	0.487	0.4091	1.000	0.38	0.380
7	1	0.840	0.487	0.4091	1.000	2.50	2.500
8	1	0.840	0.487	0.4091	1.000	0.28	0.280
9	0	0.840	0.487	0.4091	0.000	0.00	0.000
10	0	0.840	0.487	0.4091	0.000	0.00	0.000
SumCr =	22.80		Pr=	68.75	kN	Pr*SumCr=	1567.42
SumCf =	0.000		Pf=	68.75	kN	Pf*SumCf=	0.00
Sum Ci*P	SumCr+SumCf			1567.42			
Mo	(1/8/π)*SumCi*P			62.37			
km	-			1.00			
ki	-			1.10			
M	km*ki*Mo			68.60	kNm		
kf				1.00			
Md	kf*M			68.60	kNm		

Influence Coefficient

	Nx	4	I	Ⅱ	Ⅲ	IV	V
	Ny	4	1	2	3	4	5
	NO	nx	η1	η2	η3	η4	η5
	1	1	0.1	0.5	8.0	0.5	0.1
	2	4	3.0	6.8	7.8	6.8	3.0
	3	2	6.0	7.1	7.7	7.1	6.0
1	4	4	7.0	7.2	7.5	7.2	7.0
	5	1	7.1	7.4	7.4	7.4	7.1
	Sum A		59.2	78.1	92.0	78.1	59.2
	Area Ai		4.93	6.51	7.67	6.51	4.93
	ny		1	4	2	4	1
	SumV		77.27				
	Cr1		6.44				

3 **Suport** **Mx** Truck Span Ls= 5.806 m
3 3 **Case3** Weel Number Nw= 2 Units
Multiple Presence Factor km= 1.00
Rear Wheel(Include Iimpact) Pr= 98.00 kN Nr= 8
Front Wheel(Include Impact) Pf= 68.75 kN Nf= 4
Effective Area Ae= 0.840 0.537 0.45108 m2
Effective Area Ae= 0.840 0.430 0.3612 m2

No	Iss	a	b	Ae	k	Cio	Ci
1	1	0.840	0.537	0.45108	1.000	6.65	6.647
2	1	0.840	0.537	0.4511	1.000	5.50	5.500
3	1	0.840	0.537	0.4511	1.000	3.20	3.200
4	1	0.840	0.537	0.4511	1.000	0.40	0.400
5	1	0.840	0.537	0.4511	1.000	0.10	0.100
6	1	0.840	0.537	0.4511	1.000	0.30	0.300
7	1	0.840	0.537	0.4511	1.000	0.25	0.250
8	1	0.840	0.537	0.4511	1.000	0.10	0.100
9	2	0.840	0.430	0.3612	1.000	0.10	0.100
10	2	0.840	0.430	0.3612	1.000	0.30	0.300
11	2	0.840	0.430	0.3612	1.000	1.10	1.100
12	2	0.840	0.430	0.3612	1.000	0.10	0.100
SumCr =	16.50	Pr=	98.00	kN		Pr*SumCr=	1616.73
SumCf =	1.600	Pf=	68.75	kN		Pf*SumCf=	110.00
Sum Ci*P	SumCr+SumCf		1726.73				
Mo	(1/8/π)*SumCi*P		68.70				
km	-		1.00				
ki	-		1.10				
M	km*ki*Mo		75.57	kNm			
kf			1.00				
Md	kf*M		75.57	kNm			

Influence Coefficient

	Nx	4	I	Ⅱ	Ⅲ	IV	V
	Ny	4	1	2	3	4	5
	NO	nx	η1	η2	η3	η4	η5
	1	1	4.0	7.0	8.0	7.0	4.0
	2	4	4.0	6.0	7.8	6.0	4.0
	3	2	6.0	7.0	7.5	7.0	6.0
1	4	4	6.8	7.0	7.4	7.0	6.8
	5	1	7.0	7.3	7.4	7.3	7.0
	Sum A		66.2	80.3	91.2	80.3	66.2
	Area Ai		5.52	6.69	7.60	6.69	5.52
	ny		1	4	2	4	1
	SumV		79.77				
	Cr1		6.65				

3 **Suport** **Mx** Tandem Span Ls= 5.857 m
3 3 **Case3** Weel Number Nw= 1 Units
Multiple Presence Factor km= 1.20
Rear Wheel(Include Iimpact) Pr= 68.75 kN Nr= 4
Front Wheel(Include Impact) Pf= 68.75 kN Nf= 0
Effective Area Ae= 0.840 0.487 0.40908 m2

No	Iss	a	b	Ae	k	Cio	Ci
1	1	0.84	0.487	0.40908	1.000	7.13	7.125
2	1	0.84	0.487	0.4091	1.000	5.00	5.000
3	1	0.84	0.487	0.4091	1.000	1.90	1.900
4	1	0.84	0.487	0.4091	1.000	3.80	3.800
5	1	0.84	0.487	0.4091	1.000	2.60	2.600
6	0	0.84	0.487	0.4091	0.000	0.15	0.000
7	0	0.84	0.487	0.4091	0.000	2.10	0.000
8	0	0.84	0.487	0.4091	0.000	0.10	0.000
9	0	0.84	0.487	0.4091	0.000	0.00	0.000
10	0	0.84	0.487	0.4091	0.000	0.00	0.000
SumCr =	20.43	Pr=	68.75	kN		Pr*SumCr=	1404.22
SumCf =	0.000	Pf=	68.75	kN		Pf*SumCf=	0.00
Sum Ci*P	SumCr+SumCf		1404.22				
Mo	(1/8/π)*SumCi*P		55.87				
km	-		1.20				
ki	-		1.10		Coeficient of Analys		
M	km*ki*Mo		73.75	kNm			
kf			1.00				
Md	kf*M		73.75	kNm			

Influence Coefficient

	Nx	4	I	Ⅱ	Ⅲ	IV	V
	Ny	4	1	2	3	4	5
	NO	nx	η1	η2	η3	η4	η5
	1	1	5.0	7.0	7.7	7.0	5.0
	2	4	6.2	7.1	7.6	7.1	6.2
	3	2	7.0	7.3	7.5	7.3	7.0
1	4	4	7.1	7.2	7.4	7.2	7.1
	5	1	6.8	7.2	7.3	7.2	6.8
	Sum A		79.0	86.0	90.0	86.0	79.0
	Area Ai		6.58	7.17	7.50	7.17	6.58
	ny		1	4	2	4	1
	SumV		85.50				
	Cr1		7.13				

3 **Suport** **Mx** Truck Span Ls= 5.806 m
3 4 **Case4** Weel Number Nw= 2 Units
Multiple Presence Factor km= 1.00
Rear Wheel(Include Iimpact) Pr= 98.00 kN Nr= 8
Front Wheel(Include Impact) Pf= 68.75 kN Nf= 4
Effective Area Ae= 0.840 0.537 0.45108 m2
Effective Area Ae= 0.840 0.430 0.3612 m2

No	Iss	a	b	Ae	k	Cio	Ci
1	1	0.840	0.537	0.45108	1.000	7.04	7.043
2	1	0.840	0.537	0.4511	1.000	5.50	5.500
3	1	0.840	0.537	0.4511	1.000	3.20	3.200
4	1	0.840	0.537	0.4511	1.000	0.40	0.400
5	1	0.840	0.537	0.4511	1.000	0.10	0.100
6	1	0.840	0.537	0.4511	1.000	0.30	0.300
7	1	0.840	0.537	0.4511	1.000	0.25	0.250
8	1	0.840	0.537	0.4511	1.000	0.10	0.100
9	2	0.840	0.430	0.3612	1.000	0.10	0.100
10	2	0.840	0.430	0.3612	1.000	0.30	0.300
11	2	0.840	0.430	0.3612	1.000	1.10	1.100
12	2	0.840	0.430	0.3612	1.000	0.10	0.100

SumCr =	16.89 Pr=	98.00	kN	Pr*SumCr=	1655.52
SumCf =	1.600 Pf=	68.75	kN	Pf*SumCf=	110.00
Sum Ci*P	SumCr+SumCf	1765.52			
Mo	(1/8/π)*SumCi*P	70.25			
km	-	1.00			
ki	-	1.10			
M	km*ki*Mo	77.27	kNm		
kf		1.00			
Md	kf*M	77.27	kNm		

Influence Coefficient

	Nx	4	I	Ⅱ	Ⅲ	IV	V
	Ny	4	1	2	3	4	5
	NO	nx	η1	η2	η3	η4	η5
	1	1	5.0	7.0	7.7	7.0	5.0
	2	4	6.0	6.7	7.6	6.7	6.0
	3	2	7.0	7.3	7.7	7.3	7.0
1	4	4	6.0	7.5	7.6	7.5	6.0
	5	1	7.0	7.1	7.2	7.1	7.0
	Sum A		74.0	85.5	91.1	85.5	74.0
	Area Ai		6.17	7.13	7.59	7.13	6.17
	ny		1	4	2	4	1
	SumV		84.52				
	Cr1		7.04				

3 **Suport** **Mx** Tandem Span Ls= 5.806 m
3 4 **Case4** Weel Number Nw= 2 Units
Multiple Presence Factor km= 1.00
Rear Wheel(Include Iimpact) Pr= 68.75 kN Nr= 4
Front Wheel(Include Impact) Pf= 68.75 kN Nf= 0
Effective Area Ae= 0.840 0.487 0.40908 m2

No	Iss	a	b	Ae	k	Cio	Ci
1	1	0.840	0.487	0.4091	1.000	7.13	7.125
2	1	0.840	0.487	0.4091	1.000	5.00	5.000
3	1	0.840	0.487	0.4091	1.000	1.90	1.900
4	1	0.840	0.487	0.4091	1.000	3.80	3.800
5	1	0.840	0.487	0.4091	1.000	2.60	2.600
6	1	0.840	0.487	0.4091	1.000	0.15	0.150
7	1	0.840	0.487	0.4091	1.000	2.10	2.100
8	1	0.840	0.487	0.4091	1.000	0.10	0.100
9	0	0.840	0.487	0.4091	0.000	0.00	0.000
10	0	0.840	0.487	0.4091	0.000	0.00	0.000

SumCr =	22.78 Pr=	68.75	kN	Pr*SumCr=	1565.78
SumCf =	0.000 Pf=	68.75	kN	Pf*SumCf=	0.00
Sum Ci*P	SumCr+SumCf	1565.78			
Mo	(1/8/π)*SumCi*P	62.30			
km	-	1.00			
ki	-	1.10		Coeficient of Analys	
M	km*ki*Mo	68.53	kNm		
kf		1.00			
Md	kf*M	68.53	kNm		

Influence Coefficient

	Nx	4	I	Ⅱ	Ⅲ	IV	V
	Ny	4	1	2	3	4	5
	NO	nx	η1	η2	η3	η4	η5
	1	1	5.0	7.0	7.7	7.0	5.0
	2	4	6.2	7.1	7.6	7.1	6.2
	3	2	7.0	7.3	7.5	7.3	7.0
1	4	4	7.1	7.2	7.4	7.2	7.1
	5	1	6.8	7.2	7.3	7.2	6.8
	Sum A		79.0	86.0	90.0	86.0	79.0
	Area Ai		6.58	7.17	7.50	7.17	6.58
	ny		1	4	2	4	1
	SumV		85.50				
	Cr1		7.13				

4 **OverHang** **Mx** Truck Span Ls= 2.9950 m
4 1 **Case.1** Weel Number Nw= 1 Units
Multiple Presence Factor km= 1.20
Rear Wheel(Include Iimpact) Pr= 68.75 kN Nr= 3
Front Wheel(Include Impact) Pf= 68.75 kN Nf= 3
Effective Area Ae= 0.840 0.537 0.45108 m2
Effective Area Ae= 0.840 0.430 0.3612 m2

No	Iss	a	b	Ae	k	Cio	Ci
1	1	0.84	0.537	0.4511	1.000	10.38	10.382
2	1	0.84	0.537	0.4511	1.000	8.30	8.300
3	1	0.84	0.537	0.4511	1.000	2.00	2.000
4	1	0.84	0.537	0.4511	1.000	0.10	0.100
5	2	0.84	0.430	0.3612	1.000	2.00	2.000
6	2	0.84	0.430	0.3612	1.000	0.10	0.100
7	0			0.0000	0.000	0.00	0.000
8	0			0.0000	0.000	0.00	0.000
9	0			0.0000	0.000	0.00	0.000
10	0			0.0000	0.000	0.00	0.000
SumCr =	20.78		Pr=	68.75	kN	Pr*SumCr=	1428.76
SumCf =	2.100		Pf=	68.75	kN	Pf*SumCf=	144.38
Sum Ci*P	SumCr+SumCf			1573.14			
Mo	(1/8/π)*SumCi*P			62.59			
km	-			1.20			
ki	-			1.10			
M	km*ki*Mo			82.62	kNm		
kf				1.00			
Md	kf*M			82.62	kNm		

Influence Coefficient

	Nx	4	I	Ⅱ	Ⅲ	IV	V
	Ny	4	1	2	3	4	5
	NO	nx	η1	η2	η3	η4	η5
	1	1	11.2	11.2	11.3	11.2	11.2
	2	4	10.8	10.8	10.9	10.8	10.8
	3	2	10.4	10.1	10.5	10.1	10.4
1	4	4	10.0	10.0	10.1	10.0	10.0
	5	1	9.5	9.6	9.7	9.6	9.5
	Sum A		124.7	124.2	126.0	124.2	124.7
	Area Ai		10.39	10.35	10.50	10.35	10.39
	ny		1	4	2	4	1
	SumV		124.58				
	Cr1		10.38				

4 **OverHang** **Mx** Tandem Span Ls= 2.1270 m
4 1 **Case.1** Weel Number Nw= 1 Units
Multiple Presence Factor km= 1.20
Rear Wheel(Include Iimpact) Pr= 68.75 kN Nr= 4
Front Wheel(Include Impact) Pf= 68.75 kN Nf= 0
Effective Area Ae= 0.840 0.487 0.40908 m2

No	Iss	a	b	Ae	k	Cio	Ci
1	1	0.840	0.487	0.4091	1.000	10.46	10.46
2	1	0.840	0.487	0.4091	1.000	8.30	8.300
3	1	0.840	0.487	0.4091	1.000	8.20	8.200
4	1	0.840	0.487	0.4091	1.000	1.80	1.800
5	0			0.0000	0.000	0.00	0.000
6	0			0.0000	0.000	0.00	0.000
7	0			0.0000	0.000	0.00	0.000
8	0			0.0000	0.000	0.00	0.000
9	0			0.0000	0.000	0.00	0.000
10	0			0.0000	0.000	0.00	0.000
SumCr =	28.76		Pr=	68.75	kN	Pr*SumCr=	1977.42
SumCf =	0.000		Pf=	68.75	kN	Pf*SumCf=	0.00
Sum Ci*P	SumCr+SumCf			1977.42			
Mo	(1/8/π)*SumCi*P			78.68			
km	-			1.20			
ki	-			1.10			
M	km*ki*Mo			103.86	kNm		
kf				1.00			
Md	kf*M			103.86	kNm		

Influence Coefficient

	Nx	4	I	Ⅱ	Ⅲ	IV	V
	Ny	4	1	2	3	4	5
	NO	nx	η1	η2	η3	η4	η5
	1	1	11.2	11.3	11.4	11.3	11.2
	2	4	10.8	10.9	10.9	10.9	10.8
	3	2	10.4	10.5	10.5	10.5	10.4
1	4	4	10.0	10.0	10.1	10.0	10.0
	5	1	9.6	9.7	9.7	9.7	9.6
	Sum A		124.8	125.6	126.1	125.6	124.8
	Area Ai		10.40	10.47	10.51	10.47	10.40
	ny		1	4	2	4	1
	SumV		125.55				
	Cr1		10.46				

5 Center My 1 Span Ls= 5.806 m

5 1 CASE.1 Weel Number Nw= 1 Units

Multiple Presence Factor km= 1.20

Rear Wheel(Include Iimpact) Pr= 90.63 kN Nr= 4

Front Wheel(Include Impact) Pf= 43.75 kN Nf= 2

Effective Area Ae= 0.840 0.537 0.4511 m2

Effective Area Ae= 0.840 0.430 0.3612 m2

No	Iss	a	b	Ae	k	Cio	Ci
1	1	0.840	0.537	0.45108	1.000	4.22	4.217
2	1	0.840	0.537	0.45108	1.000	1.30	1.300
3	1	0.840	0.537	0.45108	1.000	0.00	0.000
4	1	0.840	0.537	0.45108	1.000	0.00	0.000
5	2	0.840	0.430	0.3612	1.000	0.00	0.000
6	2	0.840	0.430	0.3612	1.000	0.00	0.000
7	0	0.840	0.537	0.45108	0.000	2.20	0.000
8	0	0.325	0.537	0.174525	0.000	0.30	0.000
9	0	0.840	0.537	0.45108	0.000	0.00	0.000
10	0	0.325	0.537	0.174525	0.000	0.00	0.000
11	0	0.840	0.537	0.45108	0.000	0.00	0.000
12	0	0.325	0.537	0.174525	0.000	0.00	0.000
SumCr =	5.517	Pr=	90.63	kN		Pr*SumCr=	499.98
SumCf =	0.000	Pf=	43.75	kN		Pf*SumCf=	0.00
Sum Ci*P	SumCr+SumCf		499.98				
Mo	(1/8/π)*SumCi*P		19.89				
km	-		1.20				
ki	-		1.10				
M	km*ki*Mo		26.26	kNm			
kf			1.00				
Md	kf*M		26.26	kNm			

Influence Coefficient

1	Nx	4	I	Ⅱ	Ⅲ	IV	V
	Ny	4	1	2	3	4	5
	NO	nx	η1	η2	η3	η4	η5
	1	1	3.2	4.0	4.3	4.0	3.2
	2	4	3.1	4.2	5.7	4.2	3.1
	3	2	3.1	4.2	6.0	4.2	3.1
	4	4	3.1	4.2	5.7	4.2	3.1
	5	1	3.2	4.0	4.3	4.0	3.2
	Sum A		37.4	50.0	66.2	50.0	37.4
	Area Ai		3.12	4.17	5.52	4.17	3.12
	ny		1	4	2	4	1
	SumV		50.60				
	Cr1		4.22				

5 Center My Tandem Span Ls= 5.806 m

5 1 CASE.1 Weel Number Nw= 1 Units

Multiple Presence Factor km= 1.20

Rear Wheel(Include Iimpact) Pr= 68.75 kN Nr= 2

Front Wheel(Include Impact) Pf= 68.75 kN Nf= 0

Effective Area Ae= 0.840 0.487 0.40908 m2

No	Iss	a	b	Ae	k	Cio	Ci
1	1	0.840	0.487	0.4091	1.000	4.71	4.708
2	1	0.840	0.487	0.4091	1.000	1.30	1.300
3	1	0.840	0.487	0.4091	1.000	0.50	0.500
4	1	0.840	0.487	0.4091	1.000	0.60	0.600
5	0	0.840	0.487	0.4091	0.000	2.30	0.000
6	0	0.840	0.325	0.2730	0.000	0.30	0.000
7	0	0.840	0.487	0.4091	0.000	0.80	0.000
8	0	0.840	0.325	0.2730	0.000	0.10	0.000
9	0			0.0000	0.000	0.00	0.000
10	0			0.0000	0.000	0.00	0.000
SumCr =	7.11	Pr=	68.75	kN		Pr*SumCr=	488.70
SumCf =	0.000	Pf=	68.75	kN		Pf*SumCf=	0.00
Sum Ci*P	SumCr+SumCf		488.70				
Mo	(1/8/π)*SumCi*P		19.44				
km	-		1.20				
ki	-		1.10				
M	km*ki*Mo		25.67	kNm			
kf			1.00				
Md	kf*M		25.67	kNm			

Influence Coefficient

1	Nx	4	I	Ⅱ	Ⅲ	IV	V
	Ny	4	1	2	3	4	5
	NO	nx	η1	η2	η3	η4	η5
	1	1	3.5	4.2	4.3	4.2	3.5
	2	4	3.2	5.0	5.8	5.0	3.2
	3	2	3.3	4.9	6.0	4.9	3.3
	4	4	3.2	5.0	5.8	5.0	3.2
	5	1	3.5	4.2	4.3	4.2	3.5
	Sum A		39.2	58.2	67.0	58.2	39.2
	Area Ai		3.27	4.85	5.58	4.85	3.27
	ny		1	4	2	4	1
	SumV		56.50				
	Cr1		4.71				

5 2 CASE.2 Weel Number Nw= 2 Units
Multiple Presence Factor km= 1.00
Rear Wheel(Include Iimpact) Pr= 90.63 kN Nr= 4
Front Wheel(Include Impact) Pf= 43.75 kN Nf= 2
Effective Area Ae= 0.840 0.430 0.3612 m2
Effective Area Ae= 0.840 0.430 0.3612 m2

No	Iss	a	b	Ae	k	Cio	Ci
1	1	0.840	0.430	0.3612	1.000	4.22	4.217
2	1	0.840	0.430	0.3612	0.801	1.30	1.041
3	1	0.840	0.430	0.3612	0.801	0.00	0.000
4	1	0.840	0.430	0.3612	0.801	0.00	0.000
5	2	0.840	0.430	0.3612	1.000	0.00	0.000
6	2	0.840	0.430	0.3612	1.000	0.00	0.000
7	1	0.840	0.537	0.45108	1.000	2.20	2.200
8	1	0.325	0.430	0.13975	0.310	0.30	0.093
9	1	0.840	0.430	0.3612	0.801	0.00	0.000
10	1	0.325	0.430	0.13975	0.310	0.00	0.000
11	2	0.840	0.430	0.3612	1.000	0.00	0.000
12	2	0.325	0.430	0.13975	0.387	0.00	0.000

SumCr =	7.551	Pr=	90.63	kN	Pr*SumCr=	684.31
SumCf =	0.000	Pf=	43.75	kN	Pf*SumCf=	0.00

Sum Ci*P	SumCr+SumCf	684.31			
Mo	(1/8/π)*SumCi*P	27.23			
km	-	1.00			
ki	-	1.10			
M	km*ki*Mo	29.95	kNm		
kf		1.00			
Md	kf*M	29.95	kNm		

Influence Coefficient

	Nx	4	I	II	III	IV	V
	Ny	4	1	2	3	4	5
	NO	nx	η1	η2	η3	η4	η5
	1	1	3.2	4.0	4.3	4.0	3.2
	2	4	3.1	4.2	5.7	4.2	3.1
	3	2	3.1	4.2	6.0	4.2	3.1
1	4	4	3.1	4.2	5.7	4.2	3.1
	5	1	3.2	4.0	4.3	4.0	3.2
	Sum A		37.4	50.0	66.2	50.0	37.4
	Area Ai		3.12	4.17	5.52	4.17	3.12
	ny		1	4	2	4	1
	SumV		50.60				
	Cr1		4.22				

5 2 CASE.2 Weel Number Nw= 2 Units
Multiple Presence Factor km= 1.00
Rear Wheel(Include Iimpact) Pr= 68.75 kN Nr= 2
Front Wheel(Include Impact) Pf= 68.75 kN Nf= 0
Effective Area Ae= 0.840 0.487 0.40908 m2

No	Iss	a	b	Ae	k	Cio	Ci
1	1	0.840	0.487	0.4091	1.000	4.71	4.708
2	1	0.840	0.487	0.4091	1.000	1.30	1.300
3	1	0.840	0.487	0.4091	1.000	0.50	0.500
4	1	0.840	0.487	0.4091	1.000	0.60	0.600
5	1	0.840	0.487	0.4091	1.000	2.30	2.300
6	1	0.840	0.325	0.2730	0.667	0.30	0.200
7	1	0.840	0.487	0.4091	1.000	0.80	0.800
8	1	0.840	0.325	0.2730	0.667	0.10	0.067
9	0			0.0000	0.000	0.00	0.000
10	0			0.0000	0.000	0.00	0.000

SumCr =	10.48	Pr=	68.75	kN	Pr*SumCr=	720.18
SumCf =	0.000	Pf=	68.75	kN	Pf*SumCf=	0.00

Sum Ci*P	SumCr+SumCf	720.18			
Mo	(1/8/π)*SumCi*P	28.65			
km	-	1.00			
ki	-	1.10			
M	km*ki*Mo	31.52	kNm		
kf		1.00			
Md	kf*M	31.52	kNm		

Influence Coefficient

	Nx	4	I	II	III	IV	V
	Ny	4	1	2	3	4	5
	NO	nx	η1	η2	η3	η4	η5
	1	1	3.5	4.2	4.3	4.2	3.5
	2	4	3.2	5.0	5.8	5.0	3.2
	3	2	3.3	4.9	6.0	4.9	3.3
1	4	4	3.2	5.0	5.8	5.0	3.2
	5	1	3.5	4.2	4.3	4.2	3.5
	Sum A		39.2	58.2	67.0	58.2	39.2
	Area Ai		3.27	4.85	5.58	4.85	3.27
	ny		1	4	2	4	1
	SumV		56.50				
	Cr1		4.71				

6 **Overhang My 1** Span Ls= 2.995 m

6 1 **CASE.1** Weel Number Nw= 1 Units

Multiple Presence Factor km= 1.20

Rear Wheel(Include Iimpact) Pr= 90.63 kN Nr= 4

Front Wheel(Include Impact) Pf= 43.75 kN Nf= 2

Effective Area Ae= 0.840 0.537 0.4511 m2

Effective Area Ae= 0.840 0.430 0.3612 m2

No	Iss	a	b	Ae	k	Cio	Ci
1	1	0.840	0.537	0.45108	1.000	6.53	6.525
2	1	0.840	0.537	0.45108	1.000	0.42	0.420
3	0			0	0.000	0.00	0.000
4	0			0	0.000	0.00	0.000
5	0			0	0.000	0.00	0.000
6	0			0	0.000	0.00	0.000
7	0			0	0.000	0.00	0.000
8	0			0	0.000	0.00	0.000
9	0			0	0.000	0.00	0.000
10	0			0	0.000	0.00	0.000
11	0			0	0.000	0.00	0.000
12	0			0	0.000	0.00	0.000
SumCr =	6.945		Pr=	90.63	kN	Pr*SumCr=	629.43
SumCf =	0.000		Pf=	43.75	kN	Pf*SumCf=	0.00
Sum Ci*P	SumCr+SumCf			629.43			
Mo	(1/8/π)*SumCi*P			25.04			
km	-			1.20			
ki	-			1.10			
M	km*ki*Mo			33.06	kNm		
kf				1.00			
Md	kf*M			33.06	kNm		

Influence Coefficient

	Nx	4	I	Ⅱ	Ⅲ	IV	V
	Ny	4	1	2	3	4	5
	NO	nx	η1	η2	η3	η4	η5
	1	1	8.0	10.8	12.0	10.8	8.0
	2	4	7.0	8.1	8.6	8.1	7.0
	3	2	5.9	6.4	6.5	6.4	5.9
1	4	4	4.6	4.8	5.0	4.8	4.6
	5	1	3.8	3.9	4.0	3.9	3.8
	Sum A		70.0	79.1	83.4	79.1	70.0
	Area Ai		5.83	6.59	6.95	6.59	5.83
	ny		1	4	2	4	1
	SumV		78.30				
	Cr1		6.53				

6 **Overhang My** Tandem Span Ls= 2.995 m

6 1 **CASE.1** Weel Number Nw= 1 Units

Multiple Presence Factor km= 1.20

Rear Wheel(Include Iimpact) Pr= 68.75 kN Nr= 2

Front Wheel(Include Impact) Pf= 68.75 kN Nf= 0

Effective Area Ae= 0.840 0.487 0.40908 m2

No	Iss	a	b	Ae	k	Cio	Ci
1	1	0.840	0.487	0.4091	1.000	6.59	6.592
2	1	0.840	0.487	0.4091	1.000	0.40	0.400
3	1	0.840	0.487	0.4091	1.000	1.10	1.100
4	1	0.840	0.487	0.4091	1.000	0.10	0.100
5	0			0.0000	0.000	0.00	0.000
6	0			0.0000	0.000	0.00	0.000
7	0			0.0000	0.000	0.00	0.000
8	0			0.0000	0.000	0.00	0.000
9	0			0.0000	0.000	0.00	0.000
10	0	0.000	0.610	0.0000	0.000	0.00	0.000
SumCr =	8.19		Pr=	68.75	kN	Pr*SumCr=	563.18
SumCf =	0.000		Pf=	68.75	kN	Pf*SumCf=	0.00
Sum Ci*P	SumCr+SumCf			563.18			
Mo	(1/8/π)*SumCi*P			22.41			
km	-			1.20			
ki	-			1.10			
M	km*ki*Mo			29.58	kNm		
kf				1.00			
Md	kf*M			29.58	kNm		

Influence Coefficient

	Nx	4	I	Ⅱ	Ⅲ	IV	V
	Ny	4	1	2	3	4	5
	NO	nx	η1	η2	η3	η4	η5
	1	1	8.1	11.0	12.1	11.0	8.1
	2	4	7.2	8.2	8.4	8.2	7.2
	3	2	6.0	6.4	6.5	6.4	6.0
1	4	4	4.7	4.9	5.0	4.9	4.7
	5	1	3.8	3.9	4.0	3.9	3.8
	Sum A		71.5	80.1	82.7	80.1	71.5
	Area Ai		5.96	6.68	6.89	6.68	5.96
	ny		1	4	2	4	1
	SumV		79.10				
	Cr1		6.59				

Appendex 2

Creep and Shrinkage followed the MC90

CREEP MC90 ψ(t0,t)

Item	Symbol	Unit	Super Girder			
			t0'=		3	
			φg1	φg2	φg3	φg4
			φ(5 ,to)	φ(90 ,to)	φ(210 ,to)	φ(30000 ,to)
Type Cement		-	1	1	1	1
Concrete Strength at Age 28 days	fck	Mpa	45	45	45	45
Concrete Age at Imidiat Loaded	to'	day	3	3	3	3
Concrete Age	t	day	5	90	210	30,000
Average Temperature from Loaded	T0（⊿ti)	℃	25	25	25	25
Average Temperature	T（⊿ti)	℃	25	25	25	25
Relative Humidity	RH	%	85	85	85	85
Effective Thickness	ho	mm	125	125	125	125
Coefficient by Cement Type	α	—	1.0	1.0	1.0	1.0
Basic strength for elastic coefficient	fcmo	Mpa	10	10	10	10
Revision strength for the design standard strength	⊿f	Mpa	8	8	8	8
The coefficient for the kind of the cement	α	-	0	0	0	0
The coefficient for the kind of the cement	s	-	0.25	0.25	0.25	0.25
Compressive strength mean for materials age 28 days	fcm	Mpa	53	53	53	53
A coefficient elastic for materials age 28 days	Ec（28)	Mpa	37,486	37,486	37,486	37,486
A sustained load loading time materials age	$t_{0,T}$	day	3.8	3.8	3.8	3.8
A sustained load loading time existence effect materials age	to	day	3.8	3.8	3.8	3.8
The effective materials age for the creep coefficient	t	day	6.3	113.3	264.3	37,758.0
A correction factor for elastic coefficient calculation of the arbitrary materials age	βcc（to)	-	0.6498	0.6498	0.6498	0.6498
A correction factor to face each other on materials age 28th for arbitrary materials age elasticity coefficient calculation	β_E(to)	-	0.8061	0.8061	0.8061	0.8061
Mean compressive strength of materials age to	fcm(to)	Mpa	34.4	34.4	34.4	34.4
Mean compressive strength of materials age to	Ec(to)	Mpa	30,217	30,217	30,217	30,217
A basic virtual member thickness	her	mm	100	100	100	100
A correction factor for the humidity	ψ_{RH}	—	1.3027	1.3027	1.3027	1.3027
A correction factor for the mean compressive strength	β（fcm)	—	2.302	2.302	2.302	2.302
The correction factor for the materials age	$\beta(t_o)$	-	0.712	0.712	0.712	0.712
The correction factor for the member thickness	β_H	mm	705.3	705.3	705.3	705.3
A basic creep coefficient	ψ0	-	2.136	2.136	2.136	2.136
A creep progress degree to materials age to - t	βc（t-to)	-	0.184	0.548	0.675	0.994
A creep coefficient	ψ(t,t0)	-	0.394	1.170	1.442	2.124
The correction factor for the elastic coefficient	β_E(to)	-	0.806	0.806	0.806	0.806
A creep coefficient CEB	$\psi_{CEB}(t,t_0)$	-	0.488	0.943	1.162	1.712
From creep coefficient (t1 to t2)	$\psi_{CEB}(t,t1)$	B(t2,t0)-φCEB(t				

Shrinkage MC90 εsc(t,ts)

Item	Symbol	単位	Super Girder			
			Tiger			
			εsh(5 ,to)	εsh(90 ,to)	εsh(210 ,to)	εsh(30000 ,to)
Type of cement		-	1	1	1	1
Characstiric Strength	fck	Mpa	45	45	45	45
The Age of Concrete at the beginning of shrinkage	ts'	day	3	3	3	3
The Age of Concrete	t'	day	5	90	210	30,000
The average temperature until loading time	T0（⊿ti）	℃	25	25	25	25
Average temperature	T（⊿ti）	℃	25	25	25	25
the relative humidity	RH	%	85	85	85	85
Effective Thickness	h	mm	125	125	125	125
The coefficient for the cement kind	βsc	—	5	5	5	5
Relative Humidity	Rho	%	100	100	100	100
Basic Thickness	ho	mm	100	100	100	100
Basic Concrete Age	t1	day	3	3	3	3
A good self-care end time loading time materials age	ts	day	4	4	4	4
The effective materials age for the dry shrinkage degree	t	day	6	113	264	37,758
The coefficient which depend on humidity1	β_{sRH}	-	0.3859	0.3859	0.3859	0.3859
The coefficient which depend on humidity2	β_{RH}	-	-0.5981	-0.5981	-0.5981	-0.5981
The correction factor about the materials age	βs(t-ts)		0.0392	0.2502	0.3702	0.9790
Basic Concrete Strength	fcmo	Mpa	10	10	10	10
Concrete Strength at 28 Ages	fcm	Mpa	53	53	53	53
Basic Shrinkage for Basic Strength	εs(fcm)	$x10^{-6}$	345	345	345	345
Basic Coefficient of Shrinkage	ε_{cso}	$x10^{-6}$	-206	-206	-206	-206
Shrinkage Coefficient of Concrete Age from ts to t	εcs(t-ts)	$x10^{-6}$	-8	-52	-76	-202
From dry shrinkage degree (t1 to t2)	εcu(t2,t1)	$x10^{-6}$	44			

1.1.5 Anchor Design & Spalling check
1.1.5 Tính toán ụ neo cáp & kiểm toán lực ép vỡ tại đáy dầm hộp

STRESS CHECKING FOR ANCHORAGE ZONE

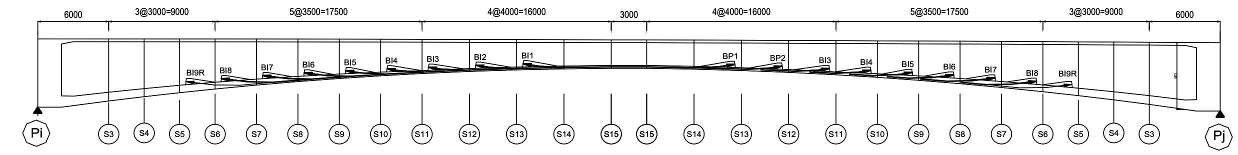
I. GENERAL DATA

A. STRUCTURES

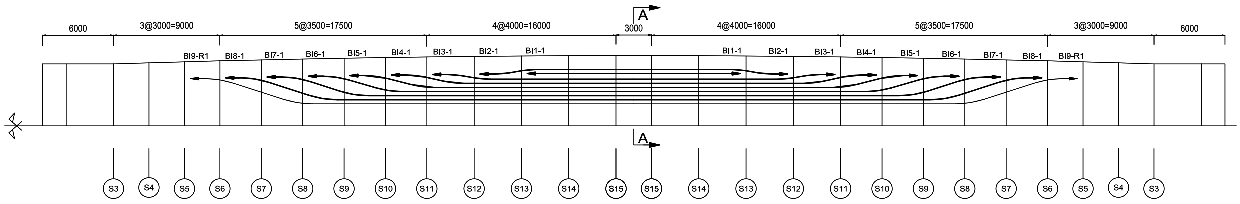
1. General

Ky lam Bridge have continuous Box Girder with layout spans 65 + 5@100 + 65 = 630 m

PROFILE OF CABLE GROUP B - CHÍNH DIỆN CÁP NHÓM B
(SC:1/300)

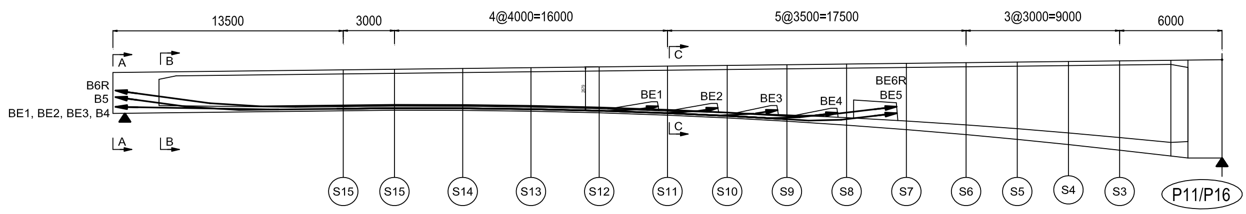


1/2 PLAN OF CABLE GROUP B - 1/2 MẶT BẰNG CÁP NHÓM B
(SC:1/300 - 1/150)

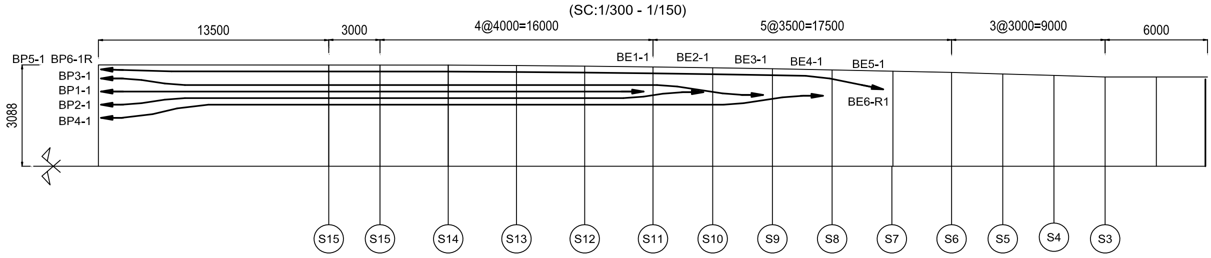


Arrangement of main cable at bottom of box girder (mainsan L=100m)

PROFILE OF CABLE GROUP C - CHÍNH DIỆN CÁP NHÓM C
(SC:1/300)



1/2 PLAN OF CABLE GROUP C - MẶT BẰNG CÁP NHÓM C
(SC:1/300 - 1/150)



Arrangement of main cable at bottom of box girder (Sidespan L=65m)

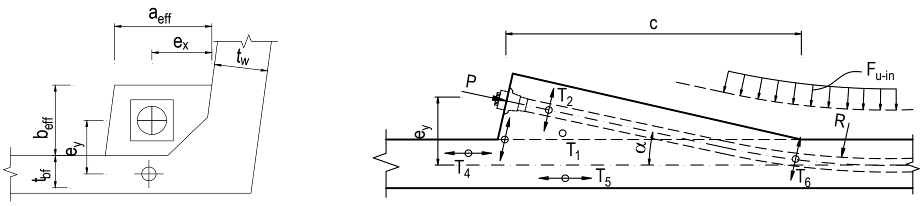


Diagram of anchor zone with one anchorage

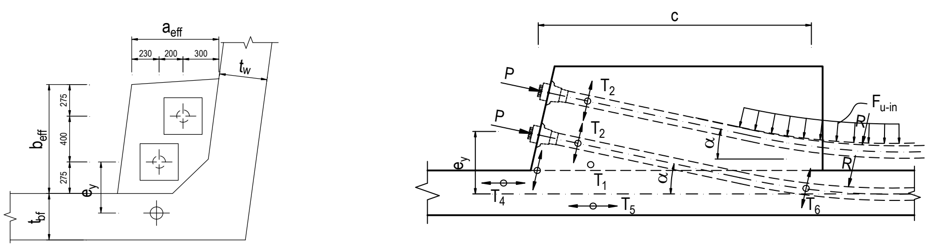


Diagram of anchor zone with two anchorage

B. MATERIAL

1. Concrete

Concrete data of Main Girder

Item	Symbol	Unit	
		Mpa	KN/m^2
Concrete class	f'_c	45.00	45000
Minimum compressive at transfer Tendons	f'_{ci}	40.00	40000
Modulus of elasticity	E_c	33994	3.40E+07
Coefficient of thermal expansion	α	1.08E-05	/^{o}C
Concrete unit weigh	γ_c	24.500	KN /m^3

2. Prestressing steel data

2.1. Prestressing Cable data

Item	Symbol	Value	Unit
Prestressing Cable Grade 270 - ASTM A416, Nominal diameter 15.2mm			
Yield strength	f_{py}	1670	Mpa
Tensile strength	f_{pu}	1860	MPa
Young's modulus	E_p	195000	MPa
Allowable stress at service	f_{pe}	1336	MPa
Nominal diameter	D	15.2	mm
Min. breaking load	P_{pu}	183.7	KN
Nominal area		140.0	mm
Max. Relaxation after 1000 hrs at 20^{o}C (at 70% UTS)		2.5	%

2.2. Bridge Tendons data

Item	Symbol	Value	Unit
Nominal area of strands	D_{st}	140	mm^2
Number srands for one Tendon	n	19	strands
Tendon nominal area	A_t	2660	mm^2
Outer duct diameter	D_d	107	mm
Duct area	A_d	8992	mm^2
Tensioning force at transfer (before release wedge) $0.75f_{pu}$	$\mathbf{P_o}$	**3710.7**	**KN**
Coefficient of friction	μ	0.2	DIM
Wobble friction	$\beta = k/\mu$	0.286	^{o}/m
Max. Wedge slip		6	mm
Tendons force limit at service	P_{pe}	3554	KN

3. Steel Reinforcing data

Item	Symbol	Value	Unit
Yield strength	f'_y	400	MPa
Young's modulus	E_c	200000	MPa

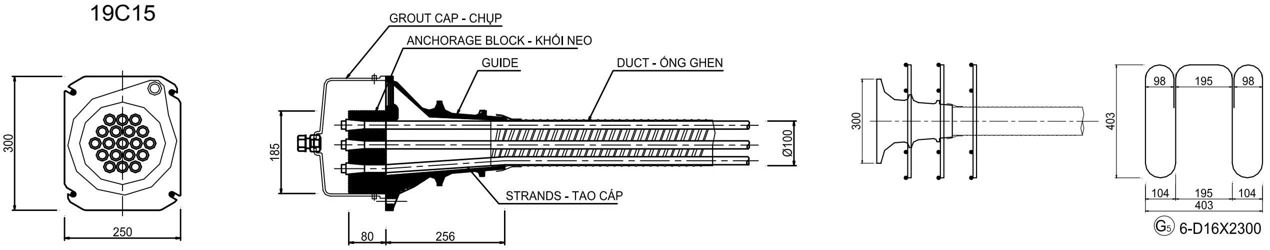
C. APPLICATION CODE

1. TCN 22TCN 272 -05, AASHTO LRFD-1998

2. Detailing for Post-Tensioned - VSL user guider

DETAILED OF ANCHORAGE ELEMENTS

19C15



II. ANALYSIS

A. MODELING AND METHOD

Approximate stress analyses and design

B. GENERAL ZONE COMPRESSIVE STRESSES

Art.5.10.9.6.2

1. Checking at the time of transfering Blister (One anchorage)

Determine the allowable concrete compressive stress from :

$f_{ca} = (0.6P_u.K) / (A_b.(1 + l_c(1/b_{eff} - 1/t)))$

for which:

s	▪ Center-to-center spacing of anchorages	s	=	0	mm
a_{eff}	▪ Lateral dimension of the effective bearing area	a_{eff}	=	730	mm
b_{eff}	▪ Vertical dimension of the effective bearing area	b_{eff}	=	550	mm
n	▪ Number of anchorages in a row	n	=	1	
K	▪ Correction factor	K	=	1.00	
l_c	▪ Longitudinal extent of confining reinforcement	l_c	=	633	mm
A_b	▪ Effective bearing area	A_b	=	392508	mm^2
t	▪ Width of web	t	=	400	mm
P_u	▪ Factor Tendon force	P_u	=	4452840	N
f_{ca}	▪ Concrete compressive stress	f_{ca}	=	11.97	Mpa
Check compressive stress		$\mathbf{0.7\varphi.f_{ci}}$	=	22.40	**OK**

2. Checking at the time of transfering Blister (Two anchorages)

Determine the allowable concrete compressive stress from :

$f_{ca} = (0.6P_u.K) / (A_b.(1 + l_c(1/b_{eff} - 1/t)))$

for which:

s	▪ Center-to -center spacing of anchorages	s	=	400	mm
a_{eff}	▪ Lateral dimension of the effective bearing area	a_{eff}	=	730	mm
b_{eff}	▪ Lateral dimension of the effective bearing area	b_{eff}	=	950	mm
n	▪ Number of anchorages in a row	n	=	2	
K	▪ Correction factor	K	=	1.00	
l_c	▪ Longitudinal extent of confining reinforcement	l_c	=	1093	mm
Ab	▪ Effective bearing area	A_b	=	693500	mm^2
t	▪ Width of web	t	=	400	mm
P_u	▪ Factor Tendon force	P_u	=	8905680	N
f_{ca}	▪ Concrete compressive stress	f_{ca}	=	13.26	Mpa
Check compressive stress		$\mathbf{0.7\varphi.f_c}$	=	25.20	**OK**

C. GENERAL ZONE BURSTING FORCE

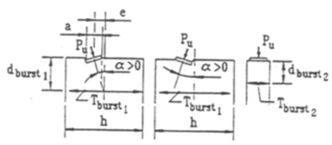
Art.5.10.9.6.3

1. The bursting force in the anchorage is caculated as (One anchorage)

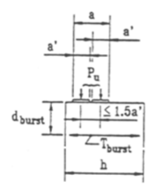
$T_{burst} = 0.25.P_u(1\text{-}a/h)+0.5.P_u\sin\alpha$

The location of the bursting force, d_{burst} may be taken as:

$d_{burst}=0.5(h\text{-}2e)+5e.\sin\alpha$



a) Các bó nghiêng



b) Các thiết bị neo đặt gần nhau

Where :

a	▪ Lateral dimension of the anchorage (mm)										
e	▪ Eccentricity of the anchorage device (mm)										
h	▪ Lateral dimension of the section (mm)										
α	▪ Angle of inclination of a Tendon force (Deg)										
T_{burst}	▪ Bursting force (KN)										
d_{burst}	▪ distance from anchorage devide to centroid of the T_{brust}										
A_s	▪ Requirement bursting reinforcement area (mm^2)										
D	▪ Diameter of bursting reinforcement	D	=	16	mm	and	D	=	16	mm	
n	▪ Number of bar spacing @ = 80	n	=	10	bar		n	=	12	bar	
$A_{s\ req}$	▪ required rebar area										

Check bursting reinforcement to transerve direction:

Anchor name	a	h	α	Pu	e	T_{burst}	d_{burst}	$A_{s\,req}$	$A_{s\,provide}$	Check
	(mm)	(mm)	(deg)	(KN)	(mm)	(Kn)	(mm)	(mm^2)	(mm^2)	$A_{s\,req} < A_{s\,pro}$
BI 1	250	730	0	3710.7	85	731.97	280.00	2614.2	4423	OK
BI 2	250	730	5	3710.7	85	926.02	317.04	3307.2	4423	OK
BI 3	250	730	5	3710.7	85	926.02	317.04	3307.2	4423	OK
BI 4	250	730	6.5	3710.7	85	984.01	328.11	3514.3	4423	OK
BI 5	250	730	7.5	3710.7	85	1022.58	335.47	3652.1	4423	OK
BI 6	250	730	8.5	3710.7	85	1061.06	342.82	3789.5	4423	OK
BI 7	250	730	9.5	3710.7	85	1099.44	350.15	3926.6	4423	OK
BI 8	250	730	9.5	3710.7	85	1099.44	350.15	3926.6	4423	OK
BI 9R	250	730	9.5	3710.7	85	1099.44	350.15	3926.6	4423	OK
BE 1	250	730	0	3710.7	85	731.97	280.00	2614.2	4423	OK
BE 2	250	730	4.5	3710.7	85	906.66	313.35	3238.1	4423	OK
BE 3	250	730	4.5	3710.7	85	906.66	313.35	3238.1	4423	OK
BE 4	250	730	4.5	3710.7	85	906.66	313.35	3238.1	4423	OK

Check bursting reinforcement to vertical direction:

Anchor name	a	h	α	Pu	e	T_{burst}	d_{burst}	$A_{s\,req}$	$A_{s\,provide}$	Check
	(mm)	(mm)	(deg)	(KN)	(mm)	(Kn)	(mm)	(mm^2)	(mm^2)	$A_{s\,req} < A_{s\,pro}$
BI 1	300	550	9	3710.7	0	854.29	275.00	3051.0	4423	OK
BI 2	300	550	9	3710.7	0	854.29	275.00	3051.0	4423	OK
BI 3	300	550	9	3710.7	0	854.29	275.00	3051.0	4423	OK
BI 4	300	550	9	3710.7	0	854.29	275.00	3051.0	4423	OK
BI 5	300	550	9	3710.7	0	854.29	275.00	3051.0	4423	OK
BI 6	300	550	7	3710.7	0	777.34	275.00	2776.2	4423	OK
BI 7	300	550	7	3710.7	0	777.34	275.00	2776.2	4423	OK
BI 8	300	550	7	3710.7	0	777.34	275.00	2776.2	4423	OK
BI 9R	300	550	7	3710.7	0	777.34	275.00	2776.2	4423	OK
BE 1	300	550	9	3710.7	0	854.29	275.00	3051.0	4423	OK
BE 2	300	550	9	3710.7	0	854.29	275.00	3051.0	4423	OK
BE 3	300	550	7	3710.7	0	777.34	275.00	2776.2	4423	OK
BE 4	300	550	7	3710.7	0	777.34	275.00	2776.2	4423	OK

2. The bursting force in the anchorage is caculated as (Two anchorages)

$T_{burst} = 0.25.P_u(1\text{-}a/h)+0.5.P_u\sin\alpha$

The location of the bursting force, dburst may be taken as:

$d_{burst}=0.5(h\text{-}2e)+5e.\sin\alpha$

Where :

a	▪ Lateral dimension of the anchorage (mm)											
e	▪ Eccentricity of the anchorage device (mm)											
h	▪ Lateral dimension of the section (mm)											
α	▪ Angle of inclination of a Tendon force (Deg)											
T_{burst}	▪ Bursting force (KN)											
d_{burst}	▪ distance from anchorage devide to centroid of the T_{brust} (mm)											
A_s	▪ Requirement bursting reinforcement area (mm^2)											
D	▪ Diameter of bursting reinforcement			D	=	16	mm	and	D	=	16	mm
n	▪ Number of bar	spacing @ =	80	n	=	10	bar		n	=	12	bar

Check bursting reinforcement to transerve direction:

Anchor name	a	h	α	Pu	e	T_{burst}	d_{burst}	$A_{s\,req}$	$A_{s\,provide}$	Check
	(mm)	(mm)	(deg)	(KN)	(mm)	(Kn)	(mm)	(mm^2)	(mm^2)	$A_{s\,req} < A_{s\,pro}$
BE5 & BE6R	450	730	6	7421.4	35	1319.42	348.29	4712.2	6836	OK

Check bursting reinforcement to vertical direction:

Anchor name	a (mm)	h (mm)	α (deg)	Pu (KN)	e (mm)	T_{burst} (Kn)	d_{burst} (mm)	$A_{s\ req}$ (mm^2)	$A_{s\ provide}$ (mm^2)	Check $A_{s\ req} < A_{s\ pro}$
BE5 & BE6R	650	950	7	7421.4	0	1245.74	475.00	4449.1	6836	OK

D. GENERAL ZONE EDGE TENSION FORCE

Art.5.10.9.6.4

1. With the Blister (One anchorage)

Spalling force may be taken as 4% of total post-Tension force

$T_{spall} = 0.04Pu$

h	▪ Lateral dimension of the section	h	=	730	mm
P_u	▪ Total post-Tensioning force	P_u	=	4453	kN
T_{spall}	▪ Eccentricity of the anchorage device	T_{spall}	=	178	kN
A_s	▪ Requirement spalling reinforcement area	A_s	=	700	mm^2
D	▪ Diameter of spalling reinforcement	D	=	18	mm
n	▪ Number of bar spacing @ = 140	n	=	5	bar

Check bursting reinforcement	n.D2.pi()/4 ≥ A_s	⇒	272>700 mm^2	**OK**

2. With the Blister Type (Two anchorages)

Spalling force may be taken as 4% of total post-Tension force

$T_{spall} = 0.04Pu$

h	▪ Lateral dimension of the section	h	=	730	mm
P_u	▪ Total post-Tensioning force	P_u	=	8906	Mpa
T_{spall}	▪ Eccentricity of the anchorage device	T_{spall}	=	356	Mpa
A_s	▪ Requirement spalling reinforcement area	A_s	=	891	mm^2
D	▪ Diameter of spalling reinforcement	D	=	18	mm
n	▪ Number of bar spacing @ = 140	n	=	5	bar

Check bursting reinforcement	n.D2.pi()/4 ≥ A_s	⇒	272>891 mm^2	**OK**

E. CHECKING AT DEVIATION SADDLES (T6 FORCE)

The force at deviation saddles T6 = Pu.sinα

α (deg)	Pu (KN)	T6 (Kn)	f_{sa} (Mpa)	As req (mm^2)	$A_{s\ provide}$ (mm^2)	Check $A_{s\ req} < A_{s\ provide}$
9	3710.7	580.481	240	2418.67	3619.11	OK

P_u ▪ The Tendon force factored

α ▪ The angle of tendon follow vertical direction

$A_{s\ provide}$ ▪ The reinforcement area using 18 bar D16

1. General

Tendons for continuity prestress are arranged a lot in the bottom flange. The girder is used the variable-depth member so that the bottom flange has a curvature in the vertical plane, which must be followed by prestress tendon. Figure 1 shows the free body diagrams of due to the curvature .Curvature of a tendon induces a downward radial load ,which must be resisted by transverse bending of the bottom flange between the webs. Longitudinal compressive stresses in the bottom flange similarly induce an upward radial reaction in the flange, counteracting at least in part the effect of the tendons.

The internal forces in the bottom flange are calculated using rigid frame model which is same analysis model for the box-section calculated(See figure 1)

The check of the bottom flange for crack shall be estimated at Service Limit State III and at the prestressing tendon tranferd .

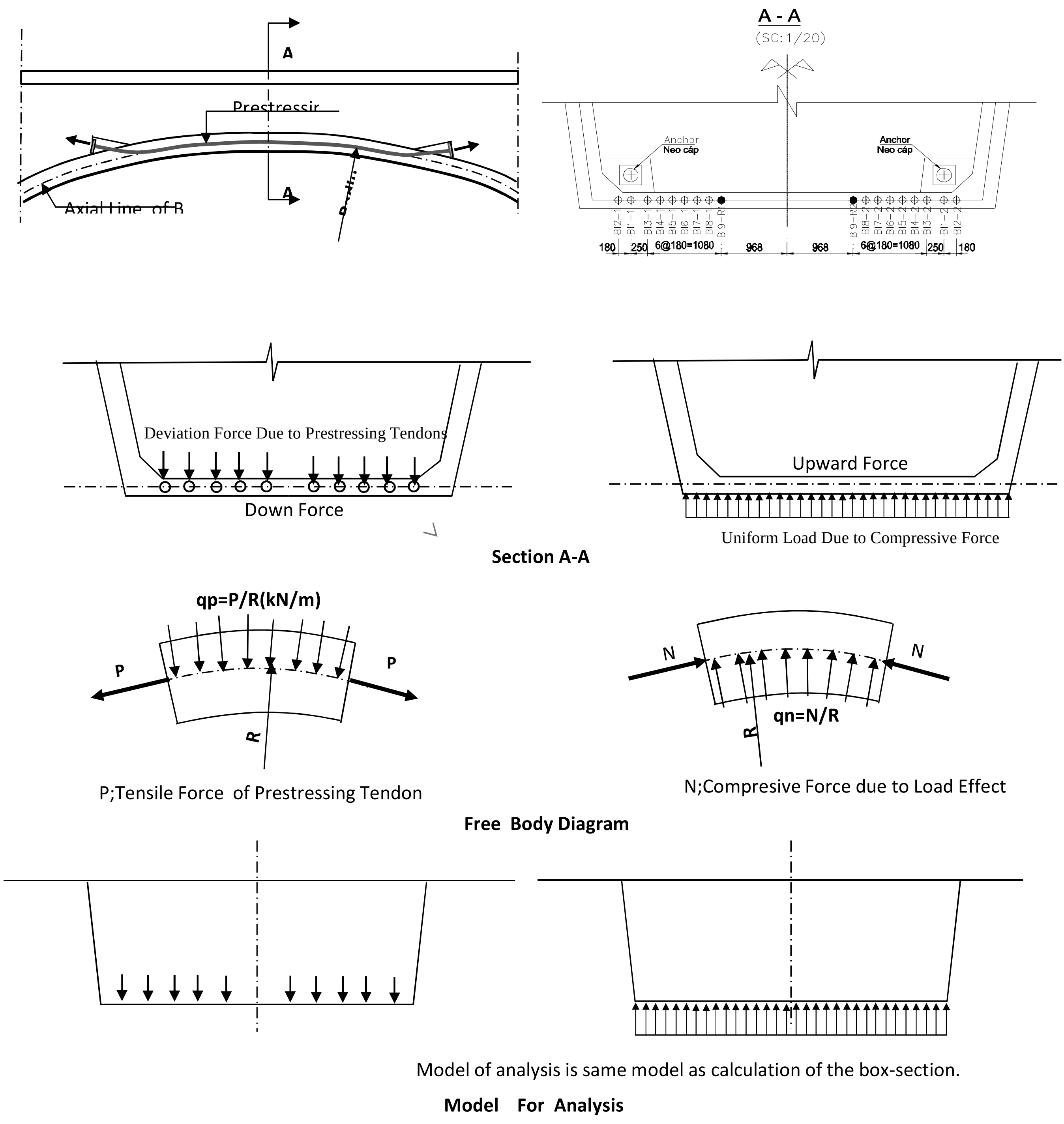
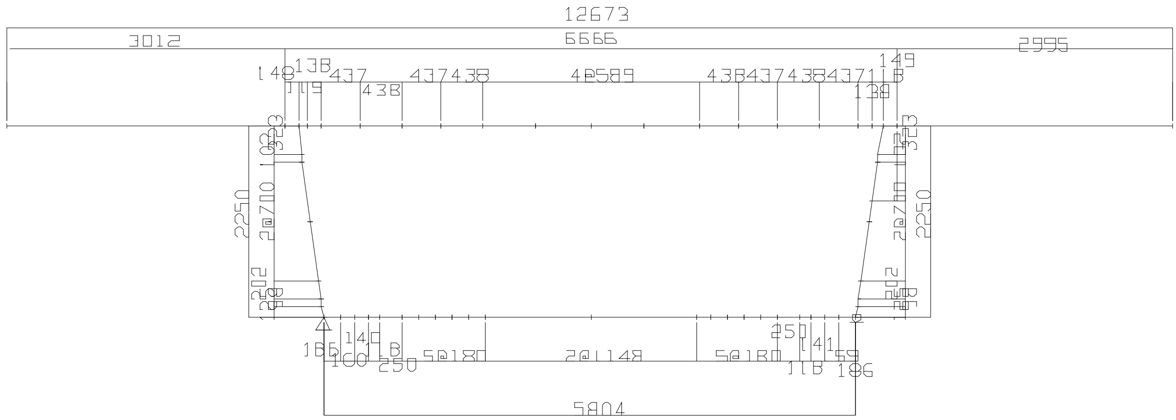
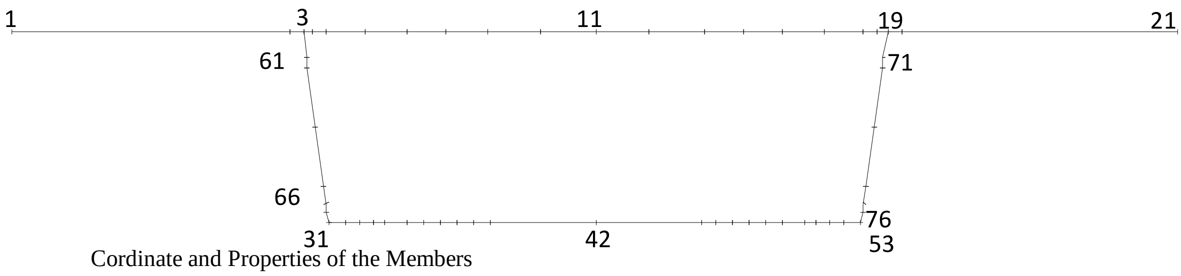


Figure 1 Secondary Stresses due to Curved Tendon in the Bottom Flange and Model for Analysis

2. Calculation of Bending Moment

Anlaysis Model





Cordinate and Properties of the Members

Codinate

	x	y		x	y		x	y
	m	m		m	m		m	m
1	-6.345	0.000	31	-2.904	-2.250	61	-3.144	-0.323
2	-3.333	0.000	32	-2.718	-2.250	62	-3.131	-0.425
3	-3.184	0.000	33	-2.558	-2.250	63	-3.044	-1.125
4	-3.066	0.000	34	-2.418	-2.250	64	-2.957	-1.825
5	-2.928	0.000	35	-2.3	-2.250	65	-2.931	-2.027
6	-2.491	0.000	36	-2.05	-2.250	66	-2.919	-2.125
7	-2.053	0.000	37	-1.87	-2.250	71	3.144	-0.323
8	-1.616	0.000	38	-1.69	-2.250	72	3.131	-0.425
9	-1.178	0.000	39	-1.51	-2.250	73	3.044	-1.125
10	-0.589	0.000	40	-1.33	-2.250	74	2.957	-1.825
11	0.000	0.000	41	-1.15	-2.250	75	2.931	-2.027
12	0.589	0.000	42	-0.002	-2.250	76	2.919	-2.125
13	1.178	0.000	43	1.146	-2.250			
14	1.616	0.000	44	1.326	-2.250			
15	2.053	0.000	45	1.506	-2.250			
16	2.491	0.000	46	1.686	-2.250			
17	2.928	0.000	47	1.866	-2.250			
18	3.066	0.000	48	2.046	-2.250			
19	3.184	0.000	49	2.296	-2.250			
20	3.333	0.000	50	2.414	-2.250			
21	6.328	0.000	51	2.555	-2.250			
			52	2.714	-2.250			
			53	2.9	-2.250			

Support

31	Moval
53	Hinge

Properties of the Members

I	J	A	I	I	J	A	I	I	J	A	I
		m2	m4			m2	m4			m2	m4
1	2	0.4000	0.0076	13	14	0.2875	0.0021	31	32	50.0000	50.0000
2	3	50.0000	50.0000	14	15	0.3625	0.0041	32	33	50.0000	50.0000
3	4	50.0000	50.0000	15	16	0.4375	0.0071	33	-51	0.2500	0.0013
4	5	0.5500	0.0139	16	17	0.5125	0.0114	51	52	50.0000	50.0000
5	6	0.5125	0.0114	17	18	0.5500	0.0139	52	53	50.0000	50.0000
6	7	0.4375	0.0071	18	19	50.0000	50.0000	61	62	0.4000	0.0053
7	8	0.3625	0.0041	19	20	50.0000	50.0000	62	63	0.4000	0.0053
8	9	0.2875	0.0021	20	21	0.4000	0.0076	63	64	0.4000	0.0053
9	-13	0.2500	0.0013					64	65	0.4000	0.0053
								65	66	50.0000	50.0000

3. Bending Moment

Service Limit State III

Load

Reinforcement of Lower Flange for Deviation of Presstressing

Position of Prestressing Steel

NO	z from Center	DZ	y	Pt	R	Pv
	m	m	m	kN	m	kN/m
1	0.968		-0.125	3700	310	11.94
2	1.328		-0.125	3700	310	11.94
3	1.508		-0.125	3700	310	11.94
4	1.688		-0.125	3700	310	11.94
5	1.868		-0.125	3700	310	11.94
6	2.048		-0.125	3700	310	11.94
7	2.228		-0.125	3700	310	11.94
8	2.408		-0.125	3700	310	11.94
9	-0.968		-0.125	3700	310	11.94
10	-1.328		-0.125	3700	310	11.94
11	-1.508		-0.125	3700	310	11.94
12	-1.688		-0.125	3700	310	11.94
13	-1.868		-0.125	3700	310	11.94
14	-2.048		-0.125	3700	310	11.94
15	-2.228		-0.125	3700	310	11.94
16	-2.408		-0.125	3700	310	11.94

Section		Symbol	Unit	Cable No.								Total
				0.610	0.790	1.040	1.220	1.400	1.580	1.760	1.940	
				1	2	3	4	5	6	7	8	
186	P	PtorPe	kN	0	0	0	0	0	0	0	0	0
		R	m	0	0	0	0	0	0	0	0	
		Pvp	kN/m	0	0	0	0	0	0	0	0	0
187	PL1	PtorPe	kN	0	0	0	0	0	0	0	0	0
		R	m	0	0	0	0	0	0	0	0	
		Pvp	kN/m	0	0	0	0	0	0	0	0	0
188	PL2	PtorPe	kN	0	0	0	0	0	0	0	0	0
		R	m	0	0	0	0	0	0	0	0	
		Pvp	kN/m	0	0	0	0	0	0	0	0	0
189	1	PtorPe	kN	0	0	0	0	0	0	0	0	0
		R	m	318	318	318	318	318	318	318	318	
		Pvp	kN/m	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0
190	2	PtorPe	kN	0	0	0	0	0	0	0	0	0
		R	m	317	317	317	317	317	317	317	317	
		Pvp	kN/m	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0
191	3	PtorPe	kN	0	0	0	0	0	0	0	0	0
		R	m	315	315	315	315	315	315	315	315	
		Pvp	kN/m	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0
192	4	PtorPe	kN	0	0	0	0	0	0	0	0	0
		R	m	314	314	314	314	314	314	314	314	
		Pvp	kN/m	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0
193	5	PtorPe	kN	3200	0	0	0	0	0	0	0	6400
		R	m	313	313	313	313	313	313	313	313	
		Pvp	kN/m	10.21	0.00	0.00	0.00	0.00	0.00	0.00	0.00	20
194	6	PtorPe	kN	3200	3200	0	0	0	0	0	0	12800
		R	m	312	312	312	312	312	312	312	312	
		Pvp	kN/m	10.25	10.25	0.00	0.00	0.00	0.00	0.00	0.00	41
195	7	PtorPe	kN	3200	3200	3200	0	0	0	0	0	19200
		R	m	311	311	311	311	311	311	311	311	
		Pvp	kN/m	10.27	10.27	10.27	0.00	0.00	0.00	0.00	0.00	62
196	8	PtorPe	kN	3200	3200	3200	3200	0	0	0	0	25600
		R	m	311	311	311	311	311	311	311	311	
		Pvp	kN/m	10.30	10.30	10.30	10.30	0.00	0.00	0.00	0.00	82
197	9	PtorPe	kN	3200	3200	3200	3200	3200	0	0	0	32000
		R	m	310	310	310	310	310	310	310	310	
		Pvp	kN/m	10.32	10.32	10.32	10.32	10.32	0.00	0.00	0.00	103
198	10	PtorPe	kN	3200	3200	3200	3200	3200	3200	0	0	38400
		R	m	310	310	310	310	310	310	310	310	
		Pvp	kN/m	10.34	10.34	10.34	10.34	10.34	10.34	0.00	0.00	124
199	11	PtorPe	kN	3200	3200	3200	3200	3200	3200	3200	0	44800
		R	m	309	309	309	309	309	309	309	309	
		Pvp	kN/m	10.35	10.35	10.35	10.35	10.35	10.35	10.35	0.00	145
200	12	PtorPe	kN	3200	3200	3200	3200	3200	3200	3200	3200	51200
		R	m	309	309	309	309	309	309	309	309	
		Pvp	kN/m	10.36	10.36	10.36	10.36	10.36	10.36	10.36	10.36	166
201	13	PtorPe	kN	3200	3200	3200	3200	3200	3200	3200	3200	51200
		R	m	0	0	0	0	0	0	0	0	
		Pvp	kN/m	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0
202	CL	PtorPe	kN	3200	3200	3200	3200	3200	3200	3200	3200	51200
		R	m	0	0	0	0	0	0	0	0	
		Pvp	kN/m	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0

Fiber Stress of Concrete at Service Limit State III

Section		x	σtop	σbottm	Hg	Bfl	Hfl	cy	Bflu	σftop	N
		m	Mpa	Mpa	m	m	m	m	m	Mpa	kN
186	P	0.00	3.1	2.3	6.000	5.300	1.000	23.250	5.550	2.43	12,839
187	PL1	2.00	3.7	1.5	6.000	5.300	1.000	10.091	5.550	1.87	9,132
188	PL2	3.00	8.7	4.2	5.851	5.337	0.943	11.312	5.573	4.93	23,464
189	1	6.000	9.7	3.5	5.424	5.437	0.771	8.486	5.630	4.38	16,812
190	2	9.000	9.9	2.9	5.026	5.532	0.712	7.108	5.710	3.89	13,591
191	3	12.000	11.1	2.9	4.656	5.620	0.653	6.303	5.783	4.05	12,938
192	4	15.000	11.1	2.0	4.317	5.701	0.594	5.265	5.850	3.25	9,009
193	5	18.500	10.7	2.1	3.957	5.788	0.525	4.923	5.919	3.24	8,207
194	6	22.000	11.1	2.0	3.637	5.867	0.456	4.436	5.981	3.14	6,944
195	7	25.500	10.7	1.8	3.356	5.937	0.388	4.035	6.034	2.83	5,375
196	8	29.000	11.2	1.6	3.116	5.998	0.319	3.635	6.078	2.58	4,028
197	9	32.500	10.8	1.7	2.914	6.051	0.250	3.459	6.114	2.48	3,178
198	10	36.500	11.2	1.0	2.733	6.099	0.250	3.001	6.162	1.93	2,248
199	11	40.500	10.6	1.1	2.604	6.135	0.250	2.91	6.198	2.01	2,399
200	12	44.500	10.5	1.9	2.526	6.160	0.250	3.084	6.223	2.75	3,600
201	13	48.500	9.2	0.9	2.500	6.176	0.250	2.771	6.239	1.73	2,041
202	CL	50.000	8.3	2.7	2.500	6.176	0.250	3.705	6.239	3.26	4,624

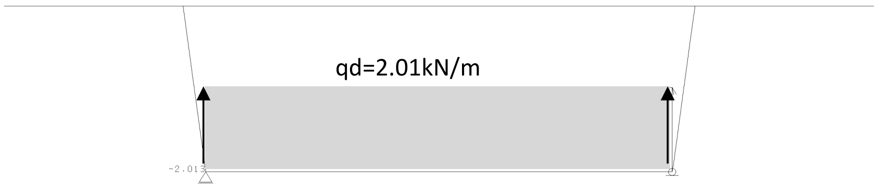
Load toBottom Flange

Section		x	N	R	Pvc	Bf	Bw	θ	Bc	pvc
		m	kN	m	kN/m	m	m	rad	m	kN/m/m
186	P	0.00	12,839	0.00	0.000	5.425	0.600	0.124	4.820	0.000
187	PL1	2.00	9,132	0.00	0.000	5.425	0.600	0.124	4.820	0.000
188	PL2	3.00	23,464	319.00	73.555	5.455	0.600	0.124	4.850	15.165
189	1	6.000	16,812	317.71	52.916	5.533	0.600	0.124	4.929	10.736
190	2	9.000	13,591	316.50	42.941	5.621	0.600	0.124	5.016	8.560
191	3	12.000	12,938	315.38	41.022	5.702	0.400	0.124	5.299	7.742
192	4	15.000	9,009	314.36	28.658	5.775	0.400	0.124	5.372	5.335
193	5	18.500	8,207	313.27	26.197	5.854	0.400	0.124	5.451	4.806
194	6	22.000	6,944	312.31	22.234	5.924	0.400	0.124	5.521	4.027
195	7	25.500	5,375	311.47	17.257	5.986	0.400	0.124	5.582	3.091
196	8	29.000	4,028	310.74	12.964	6.038	0.400	0.124	5.635	2.301
197	9	32.500	3,178	310.14	10.249	6.082	0.400	0.124	5.679	1.805
198	10	36.500	2,248	309.59	7.260	6.130	0.400	0.124	5.727	1.268
199	11	40.500	2,399	309.20	7.758	6.166	0.400	0.124	5.763	1.346
200	12	44.500	3,600	308.97	11.650	6.191	0.400	0.124	5.788	2.013
201	13	48.500	2,041	0.00	0.000	6.207	0.400	0.124	5.804	0.000
202	CL	50.000	4,624	0.00	0.000	6.207	0.400	0.124	5.804	0.000

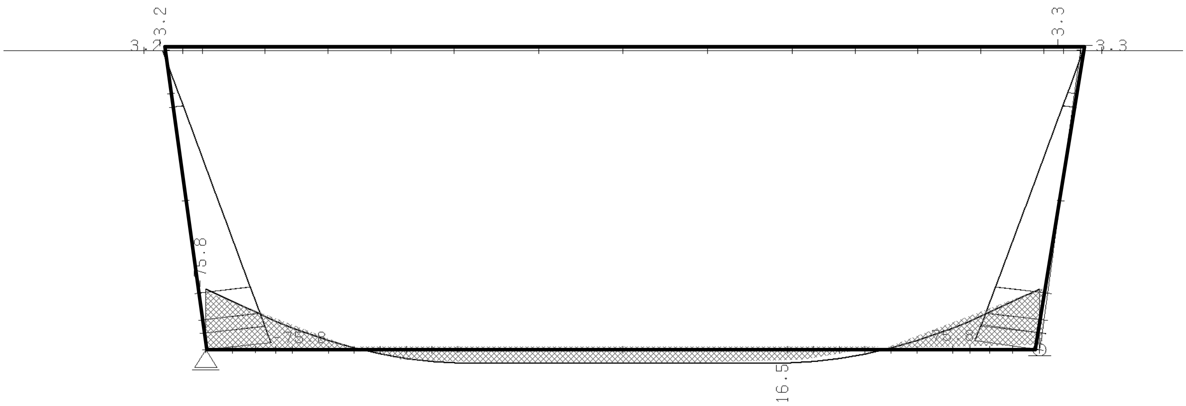
Moment Diagram At Service III

Load due to Prestressing Tendon

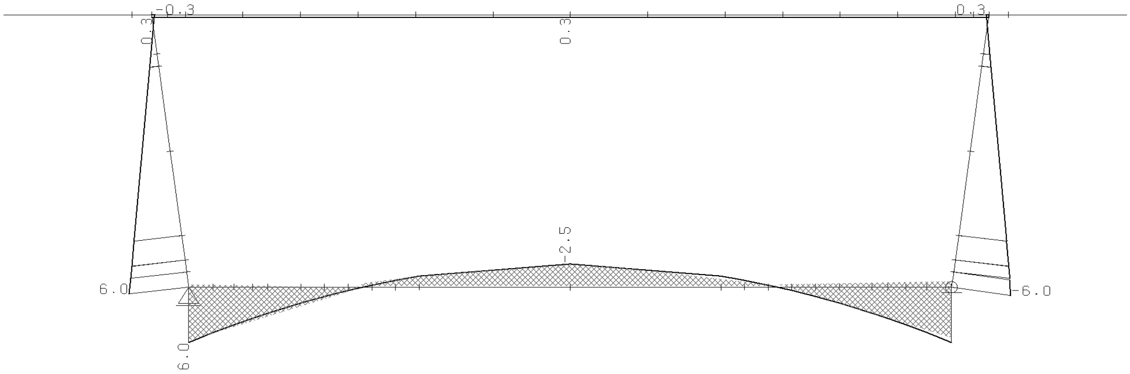
Load due to Stress in the Bottom Flange by Load Effect



Bending Moment due to Prestressing



Bending Moment due to Stress in the Bottom Flange by the Load



Moment Diagram At Anchorages immediately after Anchor Set

Reinforcement of Lower Frange for Deviation of Presstressing

Posion of Prestresing Steel

NO	z from Center	DZ	y	Pt	R	Pv
	m	m	m	kN	m	kN/m
1	0.968		-0.125	3700	310	11.94
2	1.328		-0.125	3700	310	11.94
3	1.508		-0.125	3700	310	11.94
4	1.688		-0.125	3700	310	11.94
5	1.868		-0.125	3700	310	11.94
6	2.048		-0.125	3700	310	11.94
7	2.228		-0.125	3700	310	11.94
8	2.408		-0.125	3700	310	11.94
9	-0.968		-0.125	3700	310	11.94
10	-1.328		-0.125	3700	310	11.94
11	-1.508		-0.125	3700	310	11.94
12	-1.688		-0.125	3700	310	11.94
13	-1.868		-0.125	3700	310	11.94
14	-2.048		-0.125	3700	310	11.94
15	-2.228		-0.125	3700	310	11.94
16	-2.408		-0.125	3700	310	11.94

Force of Tendon

Section		Symbol	Unit	Cable No.								Tptal
				0.610	0.790	1.040	1.220	1.400	1.580	1.760	1.940	
				1	2	3	4	5	6	7	8	
		PtorPe	kN	0	0	0	0	0	0	0	0	0
186	P	R	m	0	0	0	0	0	0	0	0	
		Pvp	kN/m	0	0	0	0	0	0	0	0	0
		PtorPe	kN	3700	3700	3700	3700	3700	3700	3700	3700	59200
187	PL1	R	m	0	0	0	0	0	0	0	0	
		Pvp	kN/m	0	0	0	0	0	0	0	0	0
		PtorPe	kN	3700	3700	3700	3700	3700	3700	3700	3700	59200
188	PL2	R	m	0	0	0	0	0	0	0	0	
		Pvp	kN/m	0	0	0	0	0	0	0	0	0
		PtorPe	kN	3700	3700	3700	3700	3700	3700	3700	3700	59200
189	1	R	m	318	318	318	318	318	318	318	318	
		Pvp	kN/m	11.65	11.65	11.65	11.65	11.65	11.65	11.65	11.65	186
		PtorPe	kN	3700	3700	3700	3700	3700	3700	3700	3700	59200
190	2	R	m	317	317	317	317	317	317	317	317	
		Pvp	kN/m	11.69	11.69	11.69	11.69	11.69	11.69	11.69	11.69	187
		PtorPe	kN	0	0	0	0	0	0	0	0	0
191	3	R	m	315	315	315	315	315	315	315	315	
		Pvp	kN/m	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0
		PtorPe	kN	0	0	0	0	0	0	0	0	0
192	4	R	m	314	314	314	314	314	314	314	314	
		Pvp	kN/m	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0
		PtorPe	kN	3700	0	0	0	0	0	0	0	7400
193	5	R	m	313	313	313	313	313	313	313	313	
		Pvp	kN/m	11.81	0.00	0.00	0.00	0.00	0.00	0.00	0.00	24
		PtorPe	kN	3700	3700	0	0	0	0	0	0	14800
194	6	R	m	312	312	312	312	312	312	312	312	
		Pvp	kN/m	11.85	11.85	0.00	0.00	0.00	0.00	0.00	0.00	47
		PtorPe	kN	3700	3700	3700	0	0	0	0	0	22200
195	7	R	m	311	311	311	311	311	311	311	311	
		Pvp	kN/m	11.88	11.88	11.88	0.00	0.00	0.00	0.00	0.00	71
		PtorPe	kN	3700	3700	3700	3700	0	0	0	0	29600
196	8	R	m	311	311	311	311	311	311	311	311	
		Pvp	kN/m	11.91	11.91	11.91	11.91	0.00	0.00	0.00	0.00	95
		PtorPe	kN	3700	3700	3700	3700	3700	0	0	0	37000
197	9	R	m	310	310	310	310	310	310	310	310	
		Pvp	kN/m	11.93	11.93	11.93	11.93	11.93	0.00	0.00	0.00	119
		PtorPe	kN	3700	3700	3700	3700	3700	3700	0	0	44400
198	10	R	m	310	310	310	310	310	310	310	310	
		Pvp	kN/m	11.95	11.95	11.95	11.95	11.95	11.95	0.00	0.00	143
		PtorPe	kN	3700	3700	3700	3700	3700	3700	3700	0	51800
199	11	R	m	309	309	309	309	309	309	309	309	
		Pvp	kN/m	11.97	11.97	11.97	11.97	11.97	11.97	11.97	0.00	168
		PtorPe	kN	3700	3700	3700	3700	3700	3700	3700	3700	59200
200	12	R	m	309	309	309	309	309	309	309	309	
		Pvp	kN/m	11.98	11.98	11.98	11.98	11.98	11.98	11.98	11.98	192
		PtorPe	kN	3700	3700	3700	3700	3700	3700	3700	3700	59200
201	13	R	m	0	0	0	0	0	0	0	0	
		Pvp	kN/m	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0
		PtorPe	kN	3700	3700	3700	3700	3700	3700	3700	3700	59200
202	CL	R	m	0	0	0	0	0	0	0	0	
		Pvp	kN/m	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0

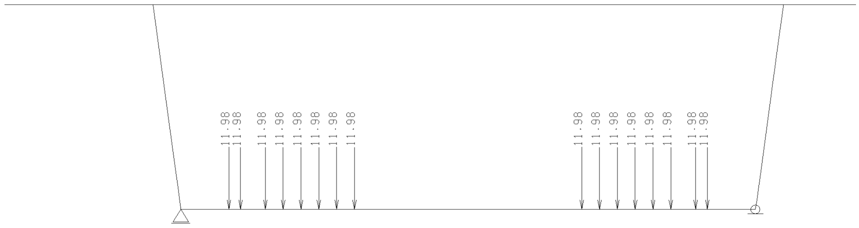
Fiber Stress of Concrete at Transfer

Section		x	σtop	σbottom	Hg	Bfl	Hfl	cy	Bflu	σftop	N
		m	Mpa	Mpa	m	m	m	m	m	Mpa	kN
186	P	0.00	4.4	1.4	6.000	5.300	1.000	8.800	5.550	1.90	8,951
187	PL1	2.00	10.9	2.9	6.000	5.300	1.000	8.175	5.550	4.23	19,349
188	PL2	3.00	11.2	3.1	5.851	5.337	0.943	8.090	5.573	4.41	19,299
189	1	6.000	12.2	2.6	5.424	5.437	0.771	6.893	5.630	3.96	14,003
190	2	9.000	12.2	2.2	5.026	5.532	0.712	6.131	5.710	3.62	11,640
191	3	12.000	13.4	2.4	4.656	5.620	0.653	5.672	5.783	3.94	11,807
192	4	15.000	13.3	1.8	4.317	5.701	0.594	4.992	5.850	3.38	8,889
193	5	18.500	12.2	2.6	3.957	5.788	0.525	5.028	5.919	3.87	9,947
194	6	22.000	12.1	3.4	3.637	5.867	0.456	5.058	5.981	4.49	10,658
195	7	25.500	11.0	4.3	3.356	5.937	0.388	5.510	6.034	5.07	10,886
196	8	29.000	10.9	5.5	3.116	5.998	0.319	6.289	6.078	6.05	11,126
197	9	32.500	9.6	7.4	2.914	6.051	0.250	12.717	6.114	7.59	11,396
198	10	36.500	9.4	8.0	2.733	6.099	0.250	18.351	6.162	8.13	12,359
199	11	40.500	8.0	9.3	2.604	6.135	0.250	-16.02	6.198	9.18	14,240
200	12	44.500	7.2	9.4	2.526	6.160	0.250	-8.267	6.223	9.18	14,381
201	13	48.500	5.3	10.4	2.500	6.176	0.250	-2.598	6.239	9.89	15,743
202	CL	50.000	4.2	10.2	2.500	6.176	0.250	-1.750	6.239	9.60	15,363

Section		x	N	R	Pvc	Bf	Bw	θ	Bc	pvc
		m	kN	m	kN/m	m	m	rad	m	kN/m/m
186	P	0.00	8,951	0.00	0.000	5.425	0.600	0.124	4.820	0.000
187	PL1	2.00	19,349	0.00	0.000	5.425	0.600	0.124	4.820	0.000
188	PL2	3.00	19,299	319.00	60.497	5.455	0.600	0.124	4.850	12.473
189	1	6.000	14,003	317.71	44.076	5.533	0.600	0.124	4.929	8.943
190	2	9.000	11,640	316.50	36.777	5.621	0.600	0.124	5.016	7.331
191	3	12.000	11,807	315.38	37.437	5.702	0.400	0.124	5.299	7.066
192	4	15.000	8,889	314.36	28.278	5.775	0.400	0.124	5.372	5.264
193	5	18.500	9,947	313.27	31.753	5.854	0.400	0.124	5.451	5.826
194	6	22.000	10,658	312.31	34.126	5.924	0.400	0.124	5.521	6.181
195	7	25.500	10,886	311.47	34.950	5.986	0.400	0.124	5.582	6.261
196	8	29.000	11,126	310.74	35.804	6.038	0.400	0.124	5.635	6.354
197	9	32.500	11,396	310.14	36.744	6.082	0.400	0.124	5.679	6.470
198	10	36.500	12,359	309.59	39.919	6.130	0.400	0.124	5.727	6.970
199	11	40.500	14,240	309.20	46.055	6.166	0.400	0.124	5.763	7.991
200	12	44.500	14,381	308.97	46.545	6.191	0.400	0.124	5.788	8.041
201	13	48.500	15,743	0.00	0.000	6.207	0.400	0.124	5.804	0.000
202	CL	50.000	15,363	0.00	0.000	6.207	0.400	0.124	5.804	0.000

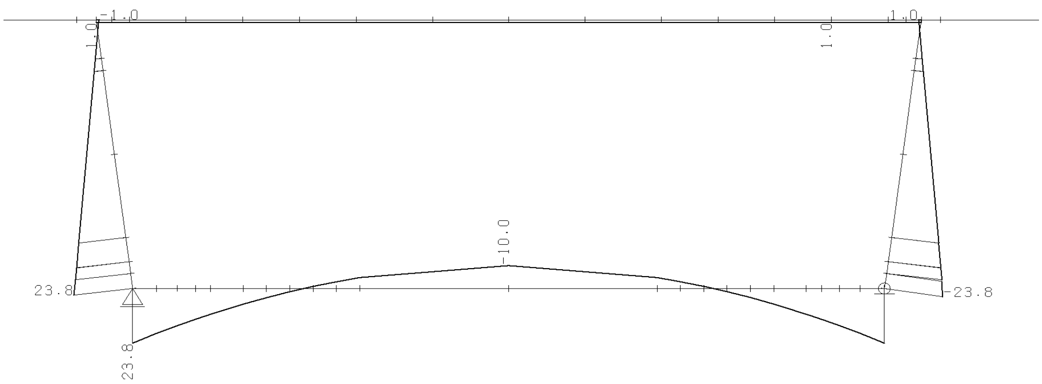
Moment Diagram

Load due to Prestressing Tendon

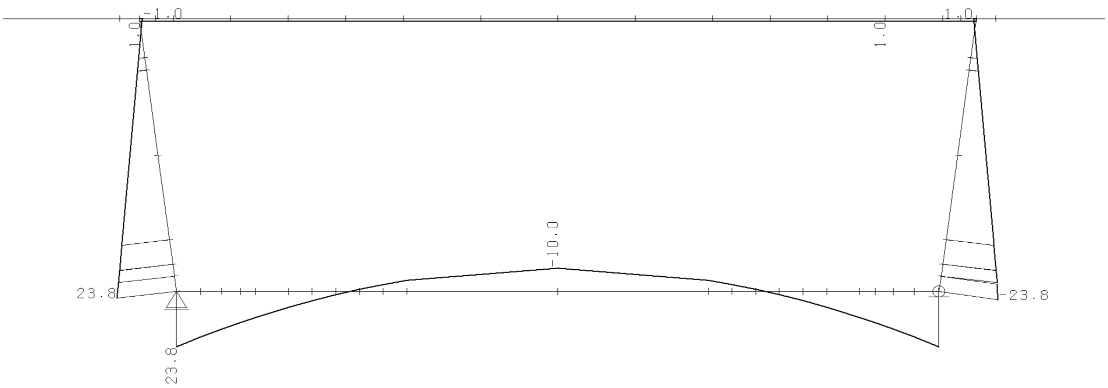


Load due to Stress in the Bottom Flange by Load Effect

Bending Moment due to Prestressing



Bending Moment due to Stress in the Bottom Flange by the Load



Summation of Bending Moment
At Sevice Limit State III

** SUM. TITLE ** SUM 1 2 SUM. CASE- 1

** SUPERPOSITION **
ALLOWABLE STRESS FACTOR 1.000
SELECTED LOAD CASE NO 1 2

		*** AXIAL FORCE	***	*** BENDING MOMENT	***	*** SHEAR FORCE	***
NO	I - J	N-I	N-J	M-I	M-J	Q-I	Q-J
		(kN)	(kN)	(kN*m)	(kN*m)	(kN)	(kN)
1	1- 2	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
2	2- 3	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
3	3- 4	-29.5923	-29.5923	-2.7488	-2.7489	-0.0012	-0.0012
4	4- 5	-29.5923	-29.5923	-2.7489	-2.7491	-0.0012	-0.0012
5	5- 6	-29.5923	-29.5923	-2.7491	-2.7496	-0.0012	-0.0012
6	6- 7	-29.5923	-29.5923	-2.7496	-2.7501	-0.0012	-0.0012
7	7- 8	-29.5923	-29.5923	-2.7501	-2.7506	-0.0012	-0.0012
8	8- 9	-29.5923	-29.5923	-2.7506	-2.7511	-0.0012	-0.0012
9	9- 10	-29.5923	-29.5923	-2.7511	-2.7518	-0.0012	-0.0012
10	10- 11	-29.5923	-29.5923	-2.7518	-2.7525	-0.0012	-0.0012
11	11- 12	-29.5923	-29.5923	-2.7525	-2.7532	-0.0012	-0.0012
12	12- 13	-29.5923	-29.5923	-2.7532	-2.7539	-0.0012	-0.0012
13	13- 14	-29.5923	-29.5923	-2.7539	-2.7544	-0.0012	-0.0012
14	14- 15	-29.5923	-29.5923	-2.7544	-2.7549	-0.0012	-0.0012
15	15- 16	-29.5923	-29.5923	-2.7549	-2.7554	-0.0012	-0.0012
16	16- 17	-29.5923	-29.5923	-2.7554	-2.7559	-0.0012	-0.0012
17	17- 18	-29.5923	-29.5923	-2.7559	-2.7561	-0.0012	-0.0012
18	18- 19	-29.5923	-29.5923	-2.7561	-2.7562	-0.0012	-0.0012
19	19- 20	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
20	20- 21	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
21	31- 32	29.5923	29.5923	-63.8336	-50.2084	72.5062	74.0018
22	32- 33	29.5923	29.5923	-50.2084	-38.3104	74.0018	75.2835
23	33- 34	29.5923	29.5923	-38.3104	-27.6460	75.2835	76.4141
24	34- 35	29.5923	29.5923	-27.6460	-19.9868	64.4341	65.3830
25	35- 36	29.5923	29.5923	-19.9868	-6.3848	53.4030	55.4132
26	36- 37	29.5923	29.5923	-6.3848	1.5635	43.4332	44.8806
27	37- 38	29.5923	29.5923	1.5635	7.6158	32.9006	34.3480
28	38- 39	29.5923	29.5923	7.6158	11.7723	22.3680	23.8153
29	39- 40	29.5923	29.5923	11.7723	14.0329	11.8353	13.2827
30	40- 41	29.5923	29.5923	14.0329	14.3977	1.3027	2.7501
31	41- 42	29.5923	29.5923	14.3977	9.1004	-9.2299	0.0012
32	42- 43	29.5923	29.5923	9.1004	14.4004	0.0012	9.2322
33	43- 44	29.5923	29.5923	14.4004	14.0361	-2.7478	-1.3004
34	44- 45	29.5923	29.5923	14.0361	11.7759	-13.2804	-11.8330
35	45- 46	29.5923	29.5923	11.7759	7.6198	-23.8130	-22.3656
36	46- 47	29.5923	29.5923	7.6198	1.5678	-34.3456	-32.8982
37	47- 48	29.5923	29.5923	1.5678	-6.3800	-44.8782	-43.4309
38	48- 49	29.5923	29.5923	-6.3800	-19.9814	-55.4109	-53.4006
39	49- 50	29.5923	29.5923	-19.9814	-27.6404	-65.3806	-64.4318
40	50- 51	29.5923	29.5923	-27.6404	-38.3044	-76.4118	-75.2812
41	51- 52	29.5923	29.5923	-38.3044	-50.2020	-75.2812	-73.9995
42	52- 53	29.5923	29.5923	-50.2020	-63.8268	-73.9995	-72.5038
43	3- 61	3.6705	3.6705	2.7488	-6.7947	-29.3638	-29.3638
44	61- 62	3.6681	3.6681	-6.7947	-9.8279	-29.3641	-29.3641
45	62- 63	3.6675	3.6675	-9.8279	-30.5424	-29.3642	-29.3642
46	63- 64	3.6675	3.6675	-30.5424	-51.2570	-29.3642	-29.3642
47	64- 65	3.6698	3.6698	-51.2570	-57.2257	-29.3639	-29.3639
48	65- 66	3.6459	3.6459	-57.2257	-60.1346	-29.3669	-29.3669
49	66- 31	3.6659	3.6659	-60.1346	-63.8336	-29.3644	-29.3644
50	19- 71	3.6682	3.6682	-2.7562	6.7873	29.3641	29.3641
51	71- 72	3.6658	3.6658	6.7873	9.8206	29.3644	29.3644
52	72- 73	3.6652	3.6652	9.8206	30.5353	29.3645	29.3645
53	73- 74	3.6652	3.6652	30.5353	51.2500	29.3645	29.3645
54	74- 75	3.6675	3.6675	51.2500	57.2188	29.3642	29.3642
55	75- 76	3.6436	3.6436	57.2188	60.1278	29.3672	29.3672
56	76- 53	4.4000	4.4000	60.1278	63.8268	29.2634	29.2634

At Anchorages immediately after Anchor Set

** SUM. TITLE ** SUM 1+2 SUM. CASI

** SUPERPOSITION **
ALLOWABLE STRESS FACTOR 1.000
SELECTED LOAD CASE NO 1 2
(NO COMPARISON)

		*** AXIAL FORCE ***		*** BENDING MOMENT ***		*** SHEAR FORCE ***	
NO	I - J	N-I	N-J	M-I	M-J	Q-I	Q-J
		(kN)	(kN)	(kN*m)	(kN*m)	(kN)	(kN)
1	1- 2	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
2	2- 3	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
3	3- 4	-32.3772	-32.3772	-2.9876	-2.9878	-0.0013	-0.0013
4	4- 5	-32.3772	-32.3772	-2.9878	-2.9880	-0.0013	-0.0013
5	5- 6	-32.3772	-32.3772	-2.9880	-2.9885	-0.0013	-0.0013
6	6- 7	-32.3772	-32.3772	-2.9885	-2.9891	-0.0013	-0.0013
7	7- 8	-32.3772	-32.3772	-2.9891	-2.9896	-0.0013	-0.0013
8	8- 9	-32.3772	-32.3772	-2.9896	-2.9902	-0.0013	-0.0013
9	9 10	32.3772	32.3772	2.9902	2.9909	0.0013	0.0013
10	10- 11	-32.3772	-32.3772	-2.9909	-2.9917	-0.0013	-0.0013
11	11- 12	-32.3772	-32.3772	-2.9917	-2.9924	-0.0013	-0.0013
12	12- 13	-32.3772	-32.3772	-2.9924	-2.9932	-0.0013	-0.0013
13	13- 14	-32.3772	-32.3772	-2.9932	-2.9938	-0.0013	-0.0013
14	14- 15	-32.3772	-32.3772	-2.9938	-2.9943	-0.0013	-0.0013
15	15- 16	-32.3772	-32.3772	-2.9943	-2.9949	-0.0013	-0.0013
16	16- 17	-32.3772	-32.3772	-2.9949	-2.9954	-0.0013	-0.0013
17	17- 18	-32.3772	-32.3772	-2.9954	-2.9956	-0.0013	-0.0013
18	18- 19	-32.3772	-32.3772	-2.9956	-2.9958	-0.0013	-0.0013
19	19- 20	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
20	20- 21	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
21	31- 32	32.3772	32.3772	-69.8607	-55.4965	77.0396	77.4140
22	32- 33	32.3772	32.3772	-55.4965	-43.1312	77.4140	77.7348
23	33- 34	32.3772	32.3772	-43.1312	-32.1818	77.7348	78.0179
24	34- 35	32.3772	32.3772	-32.1818	-24.1841	67.6579	67.8954
25	35- 36	32.3772	32.3772	-24.1841	-9.7374	57.5354	58.0387
26	36- 37	32.3772	32.3772	-9.7374	-1.1226	47.6787	48.0410
27	37- 38	32.3772	32.3772	-1.1226	5.6926	37.6810	38.0433
28	38- 39	32.3772	32.3772	5.6926	10.7082	27.6833	28.0457
29	39- 40	32.3772	32.3772	10.7082	13.9242	17.6857	18.0480
30	40- 41	32.3772	32.3772	13.9242	15.3407	7.6880	8.0504
31	41- 42	32.3772	32.3772	15.3407	14.0157	-2.3096	0.0013
32	42- 43	32.3772	32.3772	14.0157	15.3436	0.0013	2.3122
33	43- 44	32.3772	32.3772	15.3436	13.9276	-8.0478	-7.6855
34	44- 45	32.3772	32.3772	13.9276	10.7121	-18.0455	-17.6831
35	45- 46	32.3772	32.3772	10.7121	5.6969	-28.0431	-27.6808
36	46- 47	32.3772	32.3772	5.6969	-1.1178	-38.0408	-37.6784
37	47- 48	32.3772	32.3772	-1.1178	-9.7321	-48.0384	-47.6761
38	48- 49	32.3772	32.3772	-9.7321	-24.1782	-58.0361	-57.5329
39	49- 50	32.3772	32.3772	-24.1782	-32.1756	-67.8929	-67.6553
40	50- 51	32.3772	32.3772	-32.1756	-43.1246	-78.0153	-77.7323
41	51- 52	32.3772	32.3772	-43.1246	-55.4896	-77.7323	-77.4114
42	52- 53	32.3772	32.3772	-55.4896	-69.8533	-77.4114	-77.0370
43	3- 61	4.0159	4.0159	2.9876	-7.4540	-32.1272	-32.1272
44	61- 62	4.0133	4.0133	-7.4540	-10.7726	-32.1275	-32.1275
45	62- 63	4.0126	4.0126	-10.7726	-33.4365	-32.1276	-32.1276
46	63- 64	4.0126	4.0126	-33.4365	-56.1005	-32.1276	-32.1276
47	64- 65	4.0152	4.0152	-56.1005	-62.6309	-32.1273	-32.1273
48	65- 66	3.9890	3.9890	-62.6309	-65.8136	-32.1305	-32.1305
49	66- 31	4.0108	4.0108	-65.8136	-69.8607	-32.1278	-32.1278
50	19- 71	4.0134	4.0134	-2.9958	7.4459	32.1275	32.1275
51	71- 72	4.0108	4.0108	7.4459	10.7646	32.1278	32.1278
52	72- 73	4.0101	4.0101	10.7646	33.4288	32.1279	32.1279
53	73- 74	4.0101	4.0101	33.4288	56.0929	32.1279	32.1279
54	74- 75	4.0127	4.0127	56.0929	62.6234	32.1276	32.1276
55	75- 76	3.9865	3.9865	62.6234	65.8061	32.1308	32.1308
56	76- 53	4.8141	4.8141	65.8061	69.8533	32.0173	32.0173

4. Check for Crack Control
At Service Limit State III

Item			Symbol	Unit	12 Upper	12 Lower
Input Data	Bending Moment		Md	kNm	**-50.20**	**14.4**
	Shape	Width	B	m	**1.000**	**1.000**
		Height	H	m	**0.250**	**0.250**
		Inner Cover	ci	mm	**35**	**35**
		Outer Cover	co	mm	**40**	**40**
	Reinforcement	Yield Stress	fry	Mpa	**400**	**400**
		Reinforcement Area 1	D	mm	**D16**	**D16**
			@	mm	**125**	**125**
			Ari	mm2	**201.1**	**201.1**
			Ar	mm2	**1608.5**	**1608.5**
		Effective Hight1	ds	m	**0.207**	**0.207**
		Reinforcement Area 2	D	mm	**D16**	**D16**
			ctc	mm	**125**	**125**
			Ari	mm2	**201.062**	**201.062**
			Ar	mm2	**1608.5**	**1608.5**
		Effective Hight2	ds	m	**0.043**	**0.043**
		Modular Ratio	ns	-	**15**	**15**
Crack Control Check	Reinforcement Ratio		p	-	0.007771	0.00777
	Neutral Axis Ratio		k	-	0.38013	0.38013
	Arm Ratio		j	-	0.87329	0.87329
	Neutral Axis		cy	m	0.07869	0.07869
	Active Stress	Concrete	σc	Mpa	7.1	2.0
		Reinforcement	σs	〃	172.6	49.5
	Crack Control Check	Crack With Para.	Z	N/mm	**27500**	**27500**
		Clear Cover	dc	mm	**35**	**40**
		Distance to Rebar	d	mm	**43**	**48**
		Concrte Area	A	mm2	**10750**	**12000**
		Limit Stress for Rebar	fsa1	Mpa	**381**	**351**
			fsa2	Mpa	**240**	**240**
			fsa	Mpa	**240**	**240**
		Active Stress	σs	〃	172.6	49.5
		Judge	-	-	OK	OK

Check Strength

Item		Symbol	Unit	12 Upper	12 Lower
Moment at Strength Limit (1.2*M)		Mu	kNm	60.24	17.28
Shape	With	b	m	1.000	1.000
	Height	h	m	0.250	0.250
Concrete Strength		fcu	Mpa	45.0	45.0
Design Straingt of Concrte		fcd	〃	38.250	38.250
Ultimate Strain for Concrete		εcu	-	0.0030	0.0030
Yield Stress		fsy	Mpa	400	400
Yield Strain		εsy	-	0.002	0.002
Reinforcement 1		ds1	m	0.207	0.207
		As1	mm2	1608.5	1608.5
Reinforcement 2		ds2	m	0.043	0.043
		As2	mm2	1608.5	1608.5
Stress Block Factor		β1	-	0.729	0.729
Force of Rebar		Tr	kN	643.40	643.40
Force of Concrete		Cc'	kN/m	27867.9	27867.9
Neutral Axis		cy	m	0.023	0.023
Force of Concrete		Cc	kN	643.40	643.40
Flexural Resistance		Mn	kNm	127.77	127.77
Resistance Factor		φ	-	0.900	0.900
Factored Resistance Moment		Mr	kNm	114.995	114.995
Moment at Strength Limit (1.2*M)		Mu	kNm	60.24	17.28
Safety Degree		F	-	1.909	6.655
Judge		-	-	OK	OK

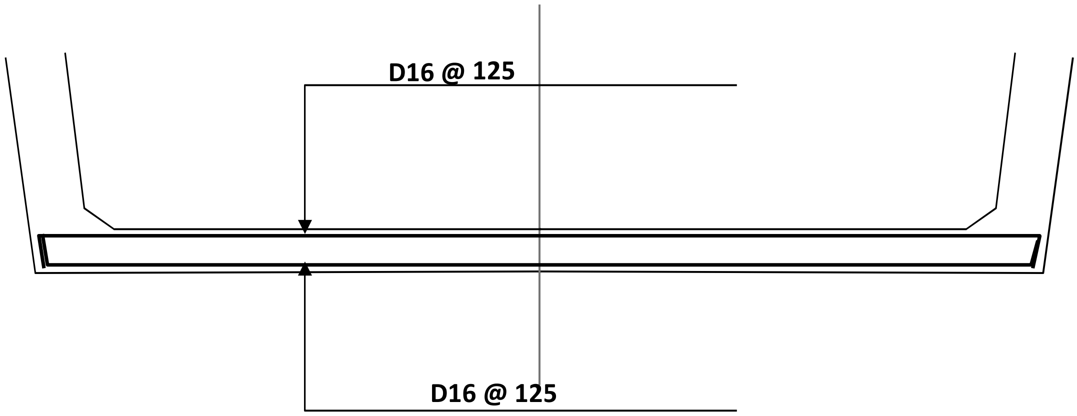
At Anchorages immediately after Anchor Set

Item			Symbpl	Unit	12 Upper	12 Lower
Input Data	Bending Moment		Md	kNm	**-55.5**	**15.3**
	Shap	Width	B	m	**1.000**	**1.000**
		Hight	H	m	**0.250**	**0.250**
		Inner Cover	ci	mm	**35**	**35**
		Outer Cover	co	mm	**40**	**40**
	Reinforcemet	Yeild Stress	fry	Mpa	**400**	**400**
		Reeinforcement Area 1	D	mm	**D16**	**D16**
			@	mm	**125**	**125**
			Ari	mm2	**201.1**	**201.1**
			Ar	mm2	**1608.5**	**1608.5**
		Efective Hight1	ds	m	**0.207**	**0.207**
		Reeinforcement Area 2	D	mm	**D16**	**D16**
			ctc	mm	**125**	**125**
			Ari	mm2	**201.062**	**201.062**
			Ar	mm2	**1608.5**	**1608.5**
		Efective Hight2	ds	m	**0.043**	**0.043**
		Modular Ratio	ns	-	**12**	**12**
Crack Control Check	Reinforcement Ratio		p	-	0.007771	0.00777
	Neutral Axis Ratio		k	-	0.34855	0.34855
	Aerm Ratio		j	-	0.88382	0.88382
	Neutral Axis		cy	m	0.07215	0.07215
	Active Stress	Conrete	σc	Mpa	8.4	2.3
		Reinforcement	σs	〃	188.6	52.0
	Crack Control Check	CrackWith Para.	Z	N/mm	**27500**	**27500**
		Clear Cover	dc	mm	**35**	**40**
		Distance to Rebar	d	mm	**43**	**48**
		Concrte Area	A	mm2	**10750**	**12000**
		Limit Stress for Rebar	fsa1	Mpa	**381**	**351**
			fsa2	Mpa	**240**	**240**
			fsa	Mpa	**240**	**240**
		Active Stress	σs	〃	188.6	52.0
		Judge	-	-	OK	OK

Check Strength

Item		Symbpl	Unit	12 Upper	12 Lower
Moment at Strength Limit (1.2*M)		Mu	kNm	66.60	18.36
Shape	Witht	b	m	1.000	1.000
	Hight	h	m	0.250	0.250
Conctret Strength		fcu	Mpa	45.0	45.0
Design Straingt of Concrte		fcd	〃	38.250	38.250
Utimate Strain for Concrete		εcu	-	0.0030	0.0030
Yeild Stress		fsy	Mpa	400	400
Yeild Strain		εsy	-	0.002	0.002
Reinfocemnt 1		ds1	m	0.207	0.207
		As1	mm2	1608.5	1608.5
Reinfocemnt 2		ds2	m	0.043	0.043
		As2	mm2	1608.5	1608.5
Stress Block Factor		β1	-	0.729	0.729
Force of ReBar		Tr	kN	643.40	643.40
Force of Concrete		Cc'	kN/m	27867.9	27867.9
Neutral Axis		cy	m	0.023	0.023
Force of Concrete		Cc	kN	643.40	643.40
Flexural Resistance		Mn	kNm	127.77	127.77
Resistance Factor		φ	-	0.900	0.900
Factored Resistance Moment		Mr	kNm	114.99	114.99
Moment at Strength Limit (1.2*M)		Mu	kNm	66.60	18.36
Safty Degree		F	-	1.727	6.263
Judge		-	-	OK	OK

5. Arrangmnet of Reinfocing Bar



1.1.6 Calculation of the Temporary Bearing at Contruction
1.1.6 Tính toán liên kết gối tạm.

1.1.6 Calculation of Temporary Support at Cantilever Construction

1.1.6 .1 Principle of Design

— Number of th restressing bar is determined so as not to be up from concrete base at the service limite state.
— For the following load combination, F = 1.5 to ensure the safety factor for falls. . Art.5.14.2.4
Design Velocity of Wind is "V=90km/hr".
Load Combination at the Service

			1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
			Dead Load			Live Load			Wind Load			Other Load					Earth Load
			DC	DIFF	U	CE CLL	IE	CLE	WS	WUP	WE	CR	SH	TU	TG	WA	EH,EV ,ES
Service	a	1	1.00	1.00	0.00	1.00	1.00	0.00	0.00	0.00	0.00	1.00	1.00	1.00	1.00	1.00	1.00
	b	2	1.00	0.00	1.00	1.00	1.00	0.00	0.00	0.00	0.00	1.00	1.00	1.00	1.00	1.00	1.00
	c	3	1.00	1.00	0.00	0.00	0.00	0.00	0.70	0.70	0.00	1.00	1.00	1.00	1.00	1.00	1.00
	d	4	1.00	1.00	0.00	0.00	0.00	0.00	0.70	1.00	0.70	1.00	1.00	1.00	1.00	1.00	1.00
	e	5	1.00	0.00	1.00	1.00	1.00	0.00	0.30	0.00	0.30	1.00	1.00	1.00	1.00	1.00	1.00
	f	6	1.00	0.00	0.00	1.00	1.00	1.00	0.30	0.00	0.30	1.00	1.00	1.00	1.00	1.00	1.00

1.1.6 .2 Load and Load Combination

DC ; Self weight of the girder
DIFF ; Error of the self Wight of girder to consider at the time of construction only one side
For one side, 2 % of self Wight is increased.
U ; Self weight of one segment
U=Acg*Lb*γc= 8.00 * 4.00 * 24.5 = 784 kN
CLL ; Live Load at Construction
One Side of Canchiler Part qCL1L= 4.8 x10 $^{-4}$ Mpa = 0.48 kN/m2
Other side wCLL2= 2.4 x10 $^{-4}$ Mpa = 0.24 kN\m2

IE ; Impact Load of the Election Equipment 10 % of Self Weight of the one Segment
PIE= 784 * 0.1 = 78.4 kN
CE ; Self Weight of the Traver WCE= 680 kN
CLE ; Longitudinal Load from Election Equipment qCLL1= 0.25 kN/m2
qCLL2= 0.50 kN/m2

	Symbol	Unit	Left side	Right side
Uniform Load	qCLL	kN/m2	0.25	0.50
Bridge Wight	B	m	12.5	12.5
Line Load	pCll	kN/m	3.125	6.25

WS ; Horizontal Wind Load
Basic Design Wind Velocity V_B= 53 m/s V_B= 191 m/hr
Design Velocity V= V_B*S
Design Unite Load qD= 0.0006* C_d* V^2
Transversal Direction

	Symbol	Unit	Superstructure	Substructure
Wind Zone	—		II	II
Basic Design Wind Velocity	V_B	m・s	53	53
Elevation	aveh	m	20	10
Coefficient for Elevation	S	-	1.14	1.09
Design Velocity	V	m/s	60.42	57.77
b/d	-		3	Circle
Drag Coefficient	Cd	-	1.6	0.8
Design Unite Load	qWS	kN/m2	3.50	1.60
Width of Resistance	bw	m	4.30	4.00
Unite Line Load	pWS	kN/m	15.05	6.40

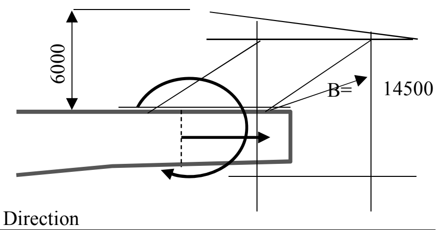
Longitudinal direction

	Symbol	Unit	Superstructure	Substructure
Wind Zone	—		II	II
Basic Design Wind Velocity	V_B	m・s	53	53
Elevation	aveh	m	20	10
Coefficient for Elevation	S	-	1.14	1.09
Design Velocity	V	m/s	60.42	57.77
b/d	-		3	Circle
Drag Coefficient	Cd	-	1.6	0.8
Design Unite Load	pWS	kN/m2	3.50	1.60
Width of Resistance	bw	m	-	5.00
Unite Line Load	pWS	kN/m	-	**8.00**
Area of Resistance	Ag	m2	37.00	-
Concentrate Load	PWS	kN	**129.50**	-

WUP ; Wind Uplift on Cantilever qWUP= 2.4×10^{-4} Mpa
Apply to one side only. qWUP= 0.24 kN/m2

	Symbol	Unit	Design Value
Basic Wind Load	qWUP	Mpa	0.00024
Basic Wind Load(kN/m2)	qWUP	kN/m2	0.24
Width	B	m	12.500
Uniform Load	pWUP	kN/m2	3.00

WE ; Horizontal Wind Load on Equipment qWE= 4.8×10^{-4} Mpa
Apply to Formtraver qWE= 0.48 kN/m2

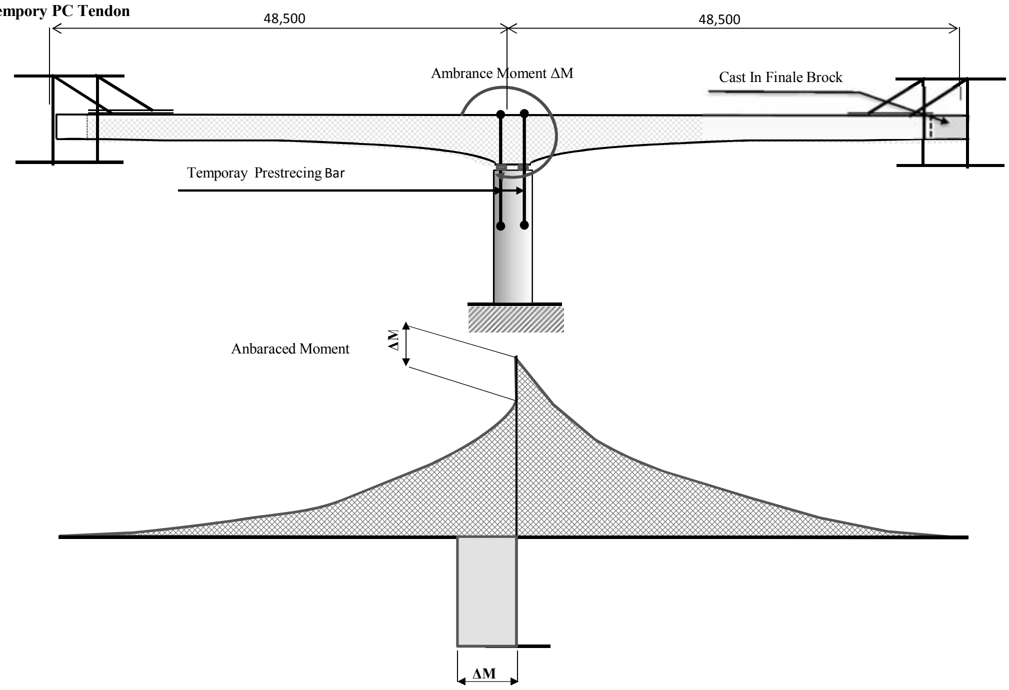


Longitudinal Direction

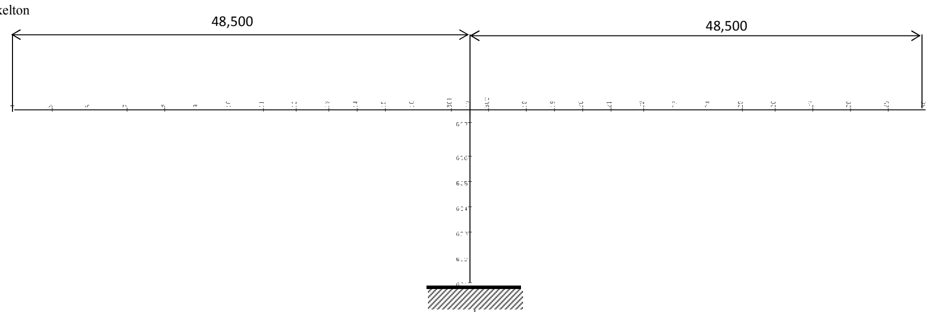
		Symbol	Unit	Design Value
Basic Wind Load		qWE	Mpa	0.00024
Basic Wind Load(kN/m2)		qWE	kN/m2	0.24
Shape	Height	H	m	6.000
	Width	B	m	14.500
Neutral of the Girder		yg	m	1.000
Concentrate Load	Force	PWE	kN	20.88
	Moment	MWE	kNm	83.5

CR ; Effect of creep
SH ; Effect of shrinkage
TU ; Effect of temperature
TG ; Effect of temperature gradient — are not considered.
WA ; Water pressure
EH ; Earth pressure
EV ; Self Wight of Soil
ES ; Earth Surcharge Load

Tempory PC Tendon

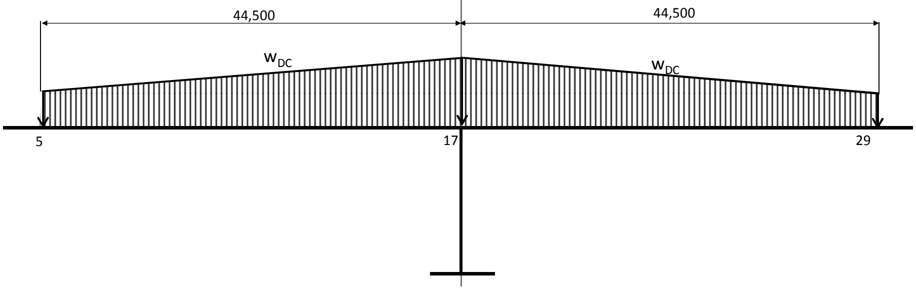


Skelton

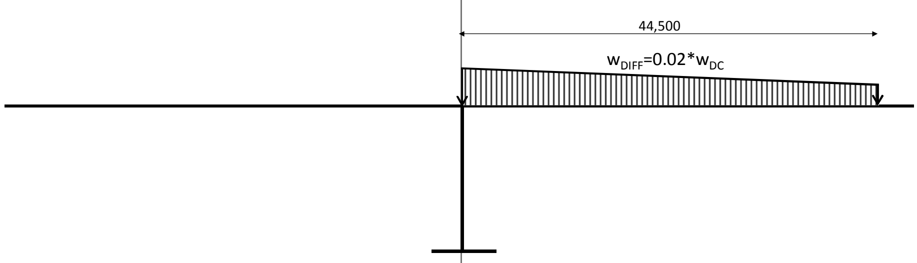


Load

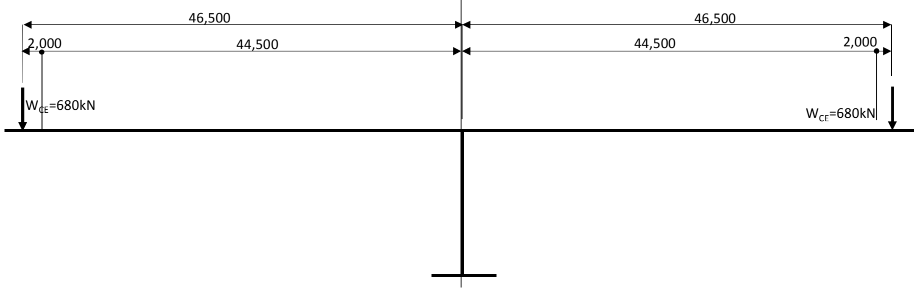
DC Self wieght of Box Girder



DIFF DIFF = 0.02*DC

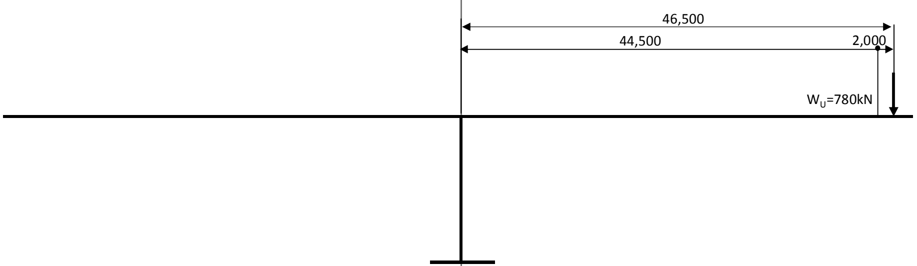


CE Traverer Self Wight W_{CE}= 680 kN



U

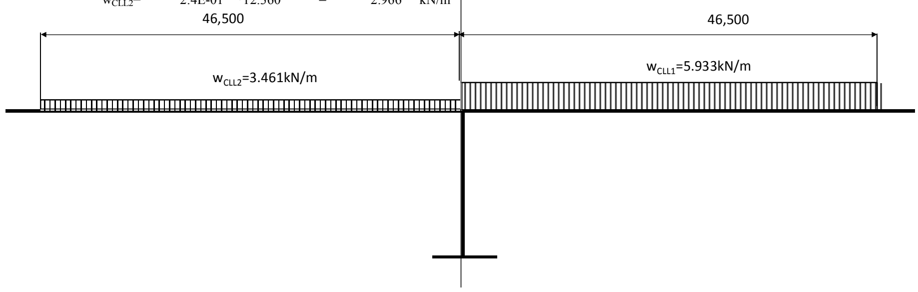
		m2	m	kN/m3					
Wu=		7.938	*4.000	*24.5	=	777.9	----->	**780**	kN



CLL

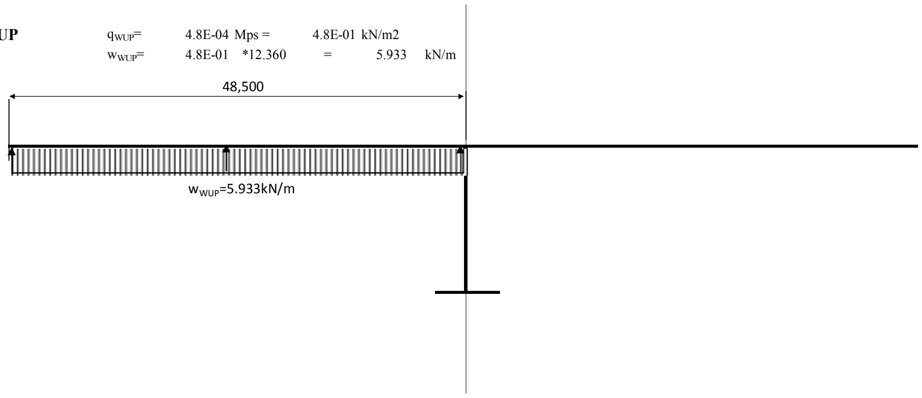
q_{CLL1}= 4.8E-04 Mps = 4.8E-01 kN/m2
q_{CLL2}= 2.4E-04 Mps = 2.4E-01 kN/m2

$w_{CLL} = q_{CLL} * Bd$
w_{CLL1}= 4.8E-01 *12.360 = 5.933 kN/m
w_{CLL2}= 2.4E-01 *12.360 = 2.966 kN/m



WUP q_{WUP}= 4.8E-04 Mps = 4.8E-01 kN/m2

w_{WUP}= 4.8E-01 *12.360 = 5.933 kN/m



Load Combination

8001

Load		N	Mz	H
		kN	kNm	kN
DC		26,139	0	0
DIFF		236	4,567	0
U		779	36,175	0
CE		1,360	0	0
CLL		420	4,041	0
IE		78	3,489	0
CLE		442	4,257	0
WS		0	0	130
WUP		-288	6,978	0
WE		0	167	40
D-1	1	26,918	36,175	0
Service	1	28,233	12,097	0
	2	28,776	43,705	0
	3	26,173	9,452	91
	4	26,866	47,837	119
	5	27,997	7,580	51
	6	28,439	11,837	51
ST1-1	1	31,327	10,277	0
ST-2	2	27,919	4,041	0

LOAD COMBINATION FATORS

			1	2	3	4	5	6	7	8	9	10	11
			DC	DIFF	U	CE	CLL	IE	CLE	WS	WUP	WE	
DEAD	D-1	1	1.00	0.00	1.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Service	a	1	1.00	1.00	0.00	1.00	1.00	1.00	0.00	0.00	0.00	0.00	0.00
	b	2	1.00	0.00	1.00	1.00	1.00	1.00	0.00	0.00	0.00	0.00	0.00
	c	3	1.00	1.00	0.00	0.00	0.00	0.00	0.00	0.70	0.70	0.00	0.00
	d	4	1.00	1.00	1.00	0.00	0.00	0.00	0.00	0.70	1.00	0.70	0.00
	e	5	1.00	0.00	0.00	1.00	1.00	1.00	0.00	0.30	0.00	0.30	0.00
	f	6	1.00	0.00	0.00	1.00	1.00	1.00	1.00	0.30	0.00	0.30	0.00
Strength	ST1-1	1	1.10	1.10	0.00	1.30	1.30	0.00	0.00	0.00	0.00	0.00	0.00
	ST-2	2	1.00	0.00	0.00	1.00	1.00	0.00	0.00	0.00	0.00	0.00	0.00

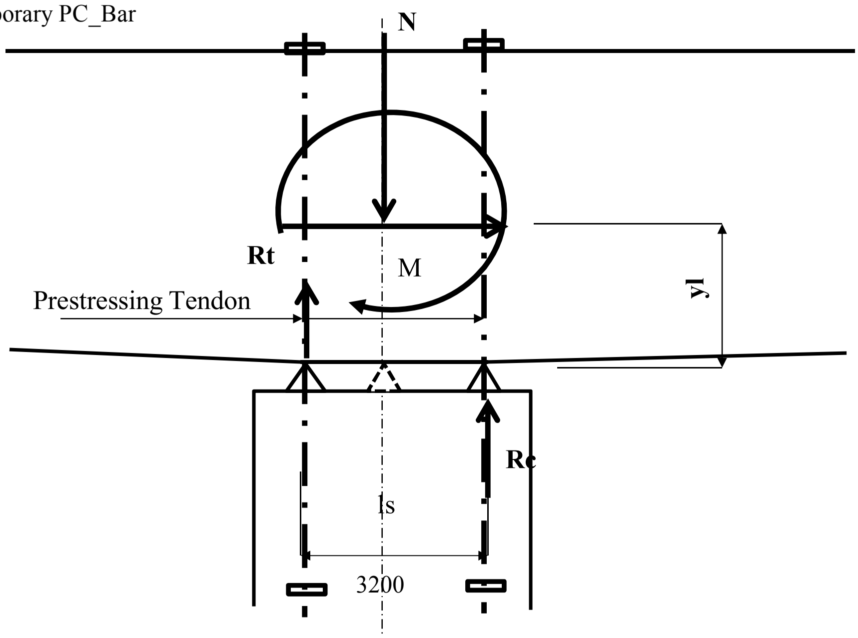
1.1.6 .3 Verifivation

(1) Check for Service Stage

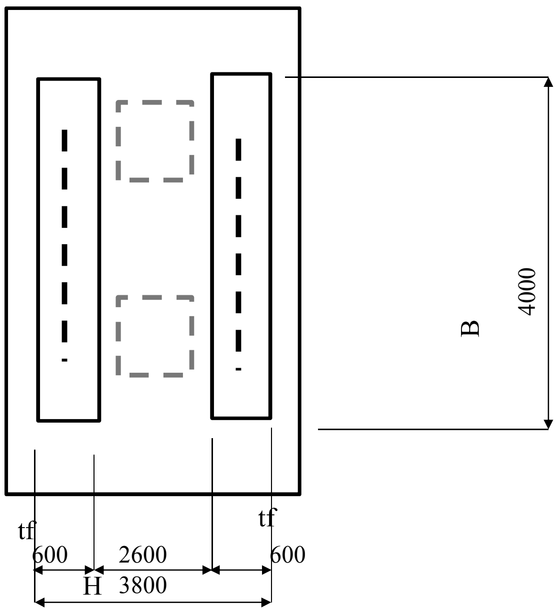
Sectinal Force

	N	M	H
	kN	kNm	kN
D+U+CE	26,918	36,175	0
M-max	26,866	47,837	119
N-min	26,173	9,452	91
yl(m)	3.093		
Mdmax=M-max*H*yl	26,866	48,205	119
Ndmin=N-min+H*yl	26,173	9,733	91

Requirment of temporary PC_Bar



Symbl	Unit	Value
B	m	4.000
H	m	3.800
tf	m	0.600



Concrte strength

f'c= 45 Mpa

PC-Bar

fpu= 1035 Mpa

Colculation of PC Bar Number and Steree of Concrete

The number of the PC steel stick is decided so that Temopray Bearing does not rise.

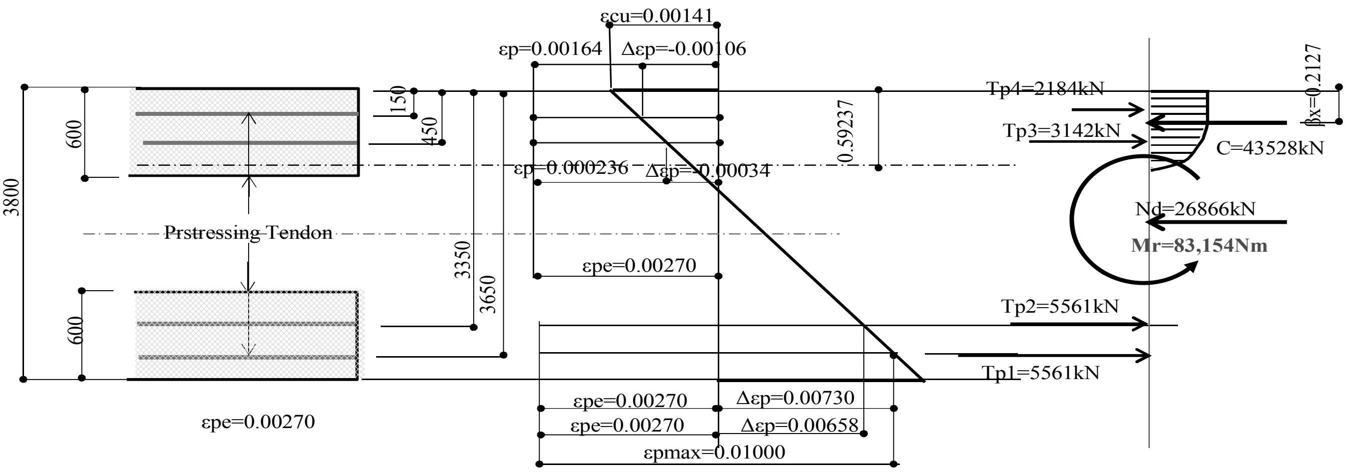
Calculation follows the next expression.

$$\mathrm{Rt} = \frac{\mathrm{N}}{\mathrm{ns}} - \frac{\mathrm{M}}{\mathrm{ls}} \qquad \mathrm{reqNp} = \mathrm{Rt} / \mathrm{Pe}$$

Item			Symbol	Unit	at M_max	Remark
Caluculation of PC Bar Number	Internal Force	Normal Force	N	kN	26,866	
		Moment	M	kNm	48,205	
	Nunber of Suport		ns	each	3	
	Arm Length		ls	m	3.200	
	Uplift Force		Rt	kN	-6109	
	Diamiter of PC Bar		ϕ p	mm	32	
	Effective prestressing		Pe	kN	500	
	Required Number of PC Bars		req Npt	nos/one side	12.2	
	Arrengement of PC Bar for one side		Npc	nos/one side	14	
	Total Number of PC Bar		Np	nos	28	
Check for Copmpression Stress of Temporary Bearing	Propertiy of Concrete	Width	B	m	4.000	
		Hight	H	m	3.800	
		Thikness	tf	m	0.600	
		Area	A	m2	4.800	
	Sectional Force	Normal Force	N	kN	40,866	
		Moment	M	kNm	48,205	
	CluculationCondition	Number of Bearing	Ns	nos	2.000	
		Length of Arm	ls	m	3.200	
	Reaction of Temprary Bering	Compression	Rc	kN	35,497	
		Tension	Rt	kN	5,369	
	Compression Stress		σc	Mpa	14.8	
	Limit Stress 05*f'c		fca	Mpa	22.5	
	Jajge	σc≤fca--->OK	-	-	OK	

Verification for ovr-turn

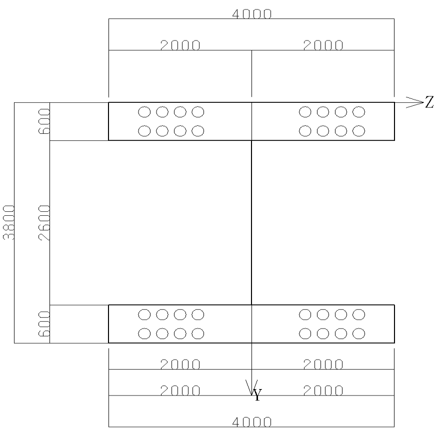
		Symbol	Unit	Design Value	Remark
Caluculation Condition	Sectional Force	N	kN	26,866	
		M	kNm	48,205	
	Design Conrete Srength	f'c	Mpa	45	
	Width	b	m	4.000	
	Hight	H	m	3.800	
	Thikness	ti	m	0.600	
Propaty of PC Bar	Ulutimate Stress of PC_Bar	fpu	Mpa	1,035	
	Yeild Stress of PC_Bar	fpy	Mpa	879.8	
	Diamiter of PC_Bar	Dp	mm	32	
	Area of PC_Bar	Ap	mm2	804.2	
	Effective Presstress of PC_Bar1	Pe1	kN	500	
	Number of PC_Bar1	Np1	each	8	
	Depth of PC_Bar1	dp1	m	0.150	
	Effective Presstress of PC_Bar2	Pe2	kN	500	
	Number of PC_Bar2	Np2	nos	8	
	Depth of PC_Bar2	dp2	m	0.450	
	Effective Presstress of PC_Bar3	Pe3	kN	500	
	Number of PC_Bar3	Np3	nos	8	
	Depth of PC_Bar3	dp3	m	3.350	
	Effective Presstress of PC_Bar4	Pe4	kN	500	
	Number of PC_Bar4	Np4	nos	8	
	Depth of PC_Bar4	dp4	m	3.650	
Result	Total of PC_Bar Number	SumNp	nos	32	
	Total of Effective Prestress	SumPe	kN	16,000	
	Maxmum Stress of PC Bar	σpmax	kN	880	
	Total Force of PC_Bar	SumPct	kN	16,662	
	Position of the Tatal Tension	yp	m	2.483	
	Tatal Force of Concrete	C	kN	43,528	
	Stress Block factor	β1	-	0.213	
	Distance from Nutral Axicis to Top	cx	m	0.592	
	Caoacity of Bending Moment	Mr	kNm	83,154	
	Working Moment	M	kNm	48,205	
Jajgemen	**Safety Factor**	**Fa**	**-**	**1.72**	
	Jajge **Fa>=1.5** **-->OK**	**-**	**-**	**OK**	



(2) Check for Strength Stage

Frow AASHTO 5.7.4.8

		Symbol	Unit	ST_Nmax	ST-Nmi
Factored Axiall Force		Nu	kN	**31327**	**27919**
Factored Bending Moment	X - Axial	Mux	kN·m	**10277**	**4041**
	Y -Axial	Muy	kN·m	**0**	**0**
Resistance Factor	For Bnding Moment	φmo	—	**0.90**	**0.90**
	For Axical Force	φno	—	**0.75**	**0.75**
Nominal Axial Resistamce		Pno	kN	**14688**	**14688**
Factored Axiial Resitance		Pro	kN	**110160**	**110160**
10% of Nominal Axial Resitance		P10	kN	**16200**	**14400**
Check Method		—	—	AXIAL FORCE	AXIAL FORCE
Eccentricity	For X Axial	ex	m	**0.3281**	**0.1447**
Eccentricity for Y Direction	For Y Axial	ey	m	—	—
Factored Axiial Resitance	On ex	Prx	m	**102295**	**110160**
	On ey	Pry	m	**110160**	**110160**
Nominal Axial Resistamce		Po	m	**183600**	**183600**
Resistance Foctor for Axial Force		φ	m	**0.75**	**0.75**
Factored Axial Resisitance		φ·Po	m	**137700**	**137700**
Factored Biaxial Resistance		Prxy	m	86,272	91,800
Safty Factor		**Nu/Prxy**	**m**	**0.363**	**0.304**
Judge Nu/Prxy≦1 --->OK		**JUDGE**	**-**	**OK**	**OK**
Resistance Factor for Bending Moment		φ	-	—	—
Factored Resistance Moment for X-Axial		Mrx	kN·m	—	—
Resio of Mux/Mrx		Mux/Mrx	—	—	—
Factored Resistance Moment for Y-Axial		Mry	kN·m	—	—
Resio of Mux/Mrx		Muy/Mry	—	—	—
Summation	Mux/Mur + Muy/Mir	ΣMu/Mr	—	—	—
Judge	ΣMu /Mr≦1--->Ok	JUDGE	—		



IN-PC STEEL (Yu= 1.9000 m、Zr= 0.0000 m、Ep= 0.21E+06 N/mm²、np= 12.0000)

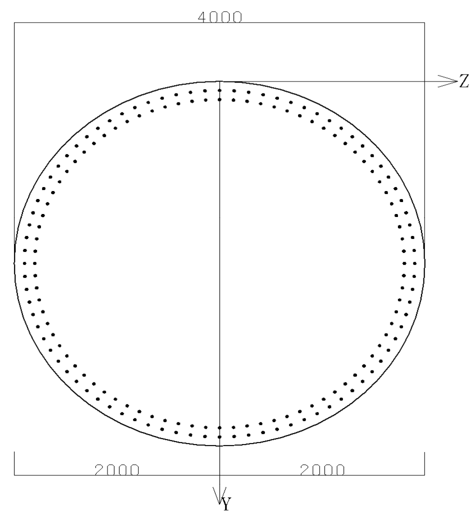
No. —	Np NOS.	Pt kN	η —	Pe kN	Np·Pe kN	偏心 方向	始点 m	終点 m	ﾋﾟｯﾁ m	ep m	Mp kN·m	Ap mm2	σ pe Mpa	ε pe —	D mm
1	4	450.000	1.0000	450.000	1800.0	Y	0.150	0.150	0.000	1.7500	3150.0	804.2	559.56	0.00270	45
						Z	−0.750	−1.500	−0.250	1.1250	2025.0				
2	4	450.000	1.0000	450.000	1800.0	Y	0.150	0.150	0.000	1.7500	3150.0	804.2	559.56	0.00270	45
						Z	0.750	1.500	0.250	−1.1250	−2025.0				
3	4	450.000	1.0000	450.000	1800.0	Y	0.450	0.450	0.000	1.4500	2610.0	804.2	559.56	0.00270	45
						Z	−0.750	−1.500	−0.250	1.1250	2025.0				
4	4	450.000	1.0000	450.000	1800.0	Y	0.450	0.450	0.000	1.4500	2610.0	804.2	559.56	0.00270	45
						Z	0.750	1.500	0.250	−1.1250	−2025.0				
5	4	450.000	1.0000	450.000	1800.0	Y	3.650	3.650	0.000	−1.7500	−3150.0	804.2	559.56	0.00270	45
						Z	−0.750	−1.500	−0.250	1.1250	2025.0				
6	4	450.000	1.0000	450.000	1800.0	Y	3.650	3.650	0.000	−1.7500	−3150.0	804.2	559.56	0.00270	45
						Z	0.750	1.500	0.250	−1.1250	−2025.0				
7	4	450.000	1.0000	450.000	1800.0	Y	3.350	3.350	0.000	−1.4500	−2610.0	804.2	559.56	0.00270	45
						Z	−0.750	−1.500	−0.250	1.1250	2025.0				
8	4	450.000	1.0000	450.000	1800.0	Y	3.350	3.350	0.000	−1.4500	−2610.0	804.2	559.56	0.00270	45
						Z	0.750	1.500	0.250	−1.1250	−2025.0				
SUM. AVE.	32	450.000	1.0000	450.000	14400.0	Y	1.9000			0.0000	0.0	25734.4	559.56	0.00270	
						Z	0.0000			0.0000	0.0				

Check of Pier

Stress of Reinforcement Due to Anbarance Moment at Construction

			Symbol	Unit	Design Value	Remark
Sectional Force		Normal Force	N	kN	26,866	
		Bending Moment	M	kNm	48,205	
Pier Shape		Diamiter	D	m	4.000	
		Area of Section	A	m2	12.566	
		Inatia	I	m4	12.566	
Concrete Strenght		-	f'c	Mpa	30	
Yield Strength of Rebar		-	fry	Mpa	400	
Gloss Area Considered	Stress of Concrete	Compression	σcc	Mpa	8.07	
		Tension	σct	Mpa	-4.31	
Calculation for RC Component	Condition	Cover	d'	m	0.100	
		Rebar Diameter	D	mm	32	
		Number of Rebar	Nr	nos	84	
		Cover	d'	m	0.200	
		Rebar Diameter	D	mm	32	
		Number of Rebar	Nr	nos	84	
		Total Area of Rebar	Ar	mm2	135,113.6	
	Result	Neutral Axis's	cx	m	**1.995**	
		Stress of Concrete	σc	Mpa	**10.1**	
		Sreress of Rebar	σr	Mpa	**135**	
	Limited Stress	**Concrete**	**fca**	**Mpa**	**15.0**	
		Rebar	**fra**	**Mpa**	**240**	
	Judgment	**σc or r ≤fa---->OK**	**-**	**-**	**OK**	

DESIGN SECTION

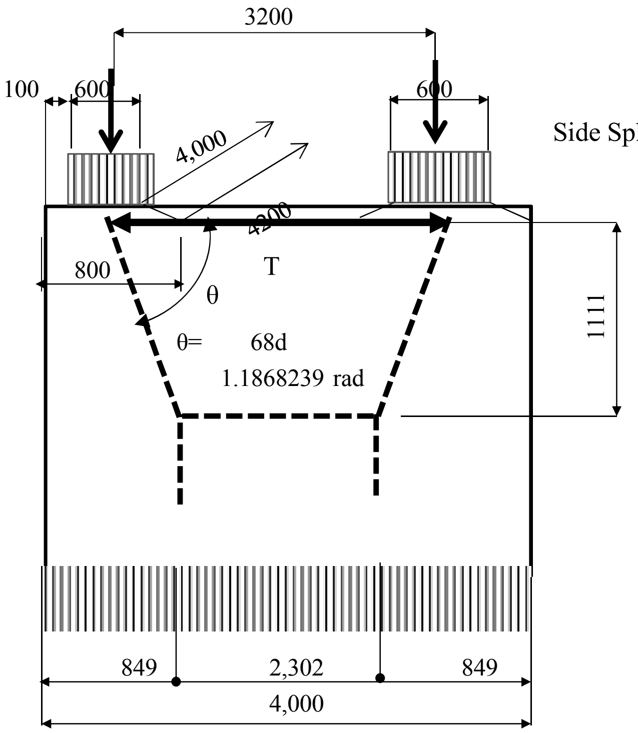


SECTION PROPERTY(REBAR - EQUIVALENT COEFFICIENT ns14.0000)

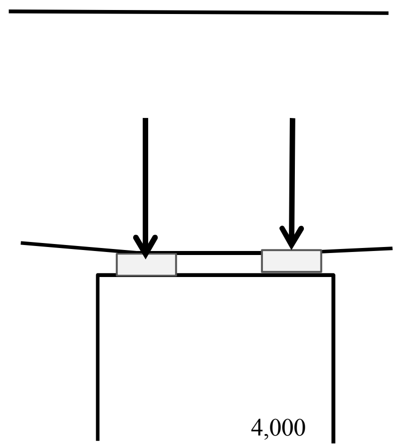
		AREA m2	CENTROID m		INERTIA m4	SECTION MODULUS m3	
		A	Yu	Yl	I	Wu	Wl
Z AX IS	SECTION PROPERTY	12.5664	2.0000	2.0000	12.56635	6.2832	6.2832
	EQUIVALENT REBAR	12.4313	2.0000	2.0000	12.33498	6.1675	6.1675
	EQUIVALENT REBAR	14.3227	2.0000	2.0000	15.57414	7.7871	7.7871
	EQUIVALENT TENDON	14.3227	2.0000	2.0000	15.57414	7.7871	7.7871

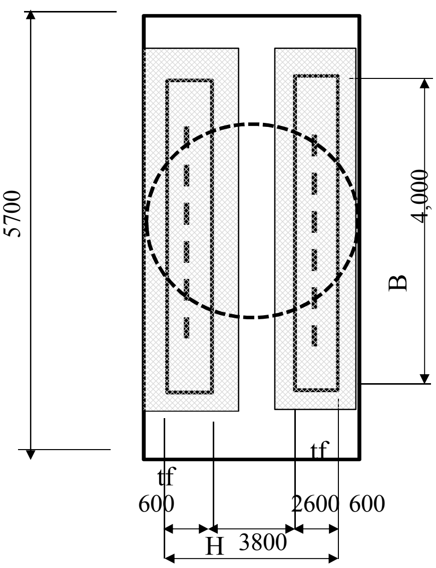
Check of Bearing Stress and Bursting at the Top of Pier

Bearing Stress of Concrete of Pier



Side Sple 1:2

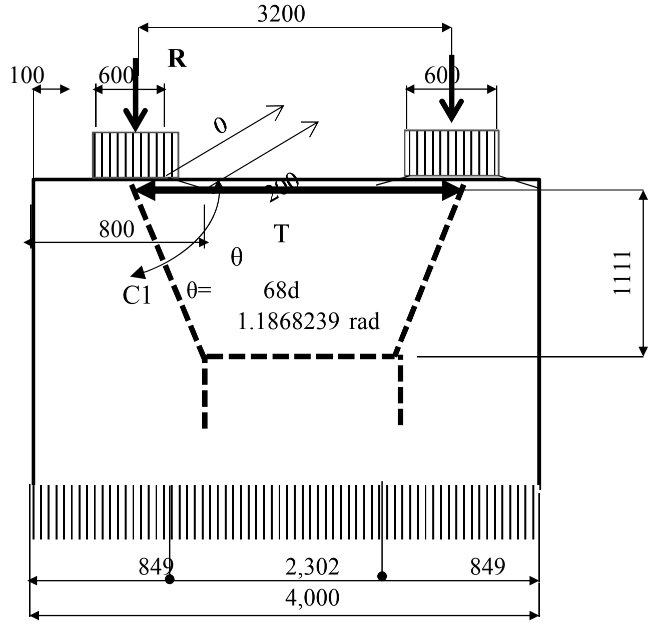




The Factored bearing Resitance

Item				Smbol	Unit	Design Value
Strength of Concrete				f'c	Mpa	**30**
Reaction				R	kN	43527.5
Shape of Bearing	Longitudinal			L	mm	**600**
	Transversal			W	mm	**4,000**
ective Width of Conc	Distance from End of Concrete dw			dlmin	mm	400
	Distance from End of Concrete dl			dwmin	mm	2850
	Transversal Cover			dw0	mm	100
	Longitudinal Cover			dl0	mm	850
	Longitudinal			Lc	mm	**800**
	Tranversal			Wc	mm	**4400**
Check of Bearing Stress of Concrete	Modification factor	Condition	Ic=1,Nonuniformy load	Ic	-	0
		Factor of Support Condition	Ic=0->k=1.0,Ic=1 ->k=0.75	k	-	1
		Effective Erea of Concrete	Lc*Wc	A2	mm2	3,520,000
		Area of Bearing Devie	L*W	A1	mm2	2,400,000
		Modification Factore	k*Sqrt(A2/A1)	m1	-	1.21
			Ic=0,m≦2.0,Ic=1,m≦1.5	m	-	1.2
Bearing Resitance	Nominal resitance		0.85*f'c*A1*m	Pn	kN	74116.9
	Restance Foctor		-	φ	-	0.7
	Design Bearing Rsistance		φ*Pn	Pr	kN	51,882
	Design Bearing Stress		Pr/A1	fb	Mpa	21.6
Judgement	Safty Factor		Pr/Rumax	Fa	-	1.19
	Judgement		Fa≧1---"OK",Fa<1.0--"NG"	-	-	OK

Check for Birsting Force due to Temprary Bearings



Reinforcement for Birsting Force

Item		Symbl	Unit	Design Value
Raction		R	kN	15,663.5
Angle		θ	rad	1.186824
Tensile Force	R/tan(θ)	T	kN	6,328.5
Reinforcemint	T/fr/φ	reqAr	mm2	22,602
	Diameter	D	mm	**20**
	Requred Rebar	reqNr	nos	72
	Acural Rebar	Nr	nos	**84**
	Area of Rebar	Ar	mm2	26,389
	Yield of Rebar	fry	Mpa	**400**
Nominal Resitance	fry*Ar	Tn	kN	10,556
Resitance Factor	-	φ	-	**0.7**
Factored Resitance	φTn	Tr	kN	7,389
Fafty	Tr/R	Fa	-	1.17
Judgement	Fa≧1-->"OK"	-	-	OK

Rebar; eainforcing Bar

1.1.7 Calculation of Pot bearing
1.1.7 Tính toán gối chậu.

POT BEARING CALCULATION SHEET

PROJECT :

TYPE : GUIDED

1. SPECIFICATIONS

AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS, 2007

2. DESIGN CONDITIONS

2.1 Load and combination Loads

	Unit	Service limit state		Ultimate limit state					kN
Maximum Vertical Load	kN	20000	kN	20000	x	1.35	=		27000
Maximum Horizontal Load	kN	3000	kN	3000	x	1.35	=		4050

2.2 Rotation

- Minimum rotation θr = 0.01 rad
- Design rotation θu = 0.005 + 0.005 + 0.01 = 0.02 rad

-Vertical deflection δ u = 3 mm

2.3 Displacement

・ Transverse = ± 0 mm

・ Longitudinal = ± 200 mm

2.4 Compressive Strength of concrete

・ Superstructure : 40 Mpa

・ Supstructure : 40 Mpa

3. MATERIALS

3.1 Steel ASTM A 709M, Grade 345S, 345W

Minimum tensile Strength Fu = 450 (Mpa)

Minimun Yield Strength Fy = 345 (Mpa)

3.2 Rubber Natural rubber

3.3 Bolt

・ Guide bolt ASTM A 307 Grade A Fubmin = 414 Mpa

・Anchor bolt ASTM A 325 / ASTM F 1554

・Anchor bar ASTM F1554

3.4 PTFE ASTM D4894

3.5 Stainless steel

3.6 Internal ring Brass ring

3.7 External ring Chloroprene rubber

4. Calculation of Pot Bearing

4.1 The diameter of the pot and the elastomeric pad 14.7.4.4

4.1.1 Average stress is satisfied

$$Ap \geq \frac{\text{Service vertical load}}{25} = \frac{20000 \times 1000}{25} = 800000 \quad (mm^2)$$

4.1.2 Diameter of the elastomeric

$$Dp = \sqrt{\frac{4 \times A_p}{\pi}} \geq \frac{\sqrt{(4 \times 800000)}}{\sqrt{(3.14)}} = 1010 \quad (mm^2)$$

$\boxed{\text{Use: Dp} = 1150 \text{ (mm)}}$ **OK**

4.1.3 The thickness of the elastomeric pad 14.4.4.3-1

$$h_r \geq 3.33 D_p \theta_u \Leftrightarrow hr \geq 3.33 \times 1150 \times 0.02 = 77 \quad (mm)$$

$\boxed{\text{Use: hr} = 90 \text{ (mm)}}$ **OK**

4.2 Pot 14.7.4.6

4.2.1 Minimum thickness of base (Set on Concrete)

＊Set on concrete

$$t_b \geq \max \begin{cases} 0.06 \times D_p = 0.06 \times 1150 = 69 \text{ mm} \\ 20mm = 20 \text{ mm} \\ \sqrt{\dfrac{25 H_u \theta_u}{F_y}} = \sqrt{\dfrac{25 \times 4050 \times 1000 \times 0.02}{345}} = 76.6 \text{ (mm)} \end{cases}$$

$\boxed{\text{Use: tb} = 85 \text{ (mm)}}$ **OK**

＊Set on steel girder or distribution plate

$$t_b \geq \max \begin{cases} 0.04 \times D_p = 0.04 \times 1150 = 46 \text{ mm} \\ 12.5mm = 12.5 \text{ mm} \\ \sqrt{\dfrac{25 H_u \theta_u}{F_y}} = \sqrt{\dfrac{25 \times 4050 \times 1000 \times 0.02}{345}} = 76.6 \text{ (mm)} \end{cases}$$

$\boxed{\text{Use: tb} = 80 \text{ (mm)}}$ **OK**

＊Use: tb = 85 mm

4.2.2 Thickness of pot wall

$$t_w \geq \max \begin{cases} \dfrac{D_p \sigma_s}{1.25 F_y} = \dfrac{1150 \times 25.9943}{1.25 \times 345} = 69.32 \text{ (mm)} & 14.7.4.6\text{-}5 \\ 20mm = 20 \text{ (mm)} & 14.7.6\text{-}6 \\ \sqrt{\dfrac{25 H_u \theta_u}{F_y}} = \sqrt{\dfrac{25 \times 4050 \times 1000 \times 0.02}{345}} = 77 \text{ (mm)} \end{cases}$$

14.7.4.7-1

$\boxed{\text{Use: tw} = 85 \text{ (mm)}}$ **OK**

$$\sigma s = \frac{27000 \times 1000}{3.14 \times \dfrac{1150^2}{4}} = 26.0 \quad (N/mm^2)$$

4.2.3 The pot cavity height , hp1 C14.7.4.3-1

$$h_{p1} \geq (0.5 D_p \theta_u) + h_r + h_w$$

hp1 ≥ (0.5 x 1150 x 0.02) + 90 + 35 = 137 mm

Use: hp1 = 140 (mm)	**OK**

4.3 Piston

4.3.1 Piston thickness

The piston shall have the same shape as the inside of the pot. Its thickness shall be adequate to resist the loads imposed on it, but shall not be less than 6 percent of the inside diameter of the pot, Dp, except at the rim.

4.3.2 Height of piston rim 14.7.4.7

$$h_w \geq \max \begin{cases} \dfrac{1.5 H_u}{D_p F_y} = \dfrac{1.5 \times 4050 \times 1000}{1150 \times 345} = 15.3 \text{ (mm)} \\ 3mm = 3 \text{ (mm)} \\ 0.03 D_p = 0.03 \times 1150 = 34.5 \text{ (mm)} \end{cases}$$

Use: hw = 35 (mm)	**OK**

4.3.3 The vertical clearance, hp2 C14.7.4.3-2

$$h_{p2} \geq R_0 \theta_u + 2\delta_u + 3$$

hp2 ≥ 660 x 0.02 + 2 x 3 + 3 = 22.2 (mm)

Radial distance from center of pot to edge of pot wall

R0 = Dp/2 + tw = 1150 / 2 + 85 = 660 (mm)

Use: hp2 = 30 (mm)	**OK**

4.3.4 Clearance, c C14.7.4.7

The diameter of the piston rim shall be the inside diameter of the pot less a clearance

c ≥ 0.5 (mm)

$$c \geq \Theta_u \text{ x } \left(h_w - \frac{D_p \times \Theta_u}{2} \right) = 0.020 \text{ x } \left(35 - \frac{1150 \text{ x } 0.02}{2} \right) = 0.470 \text{ mm}$$

Use: c = 1 (mm)

4.4 Brass ring 3 pieces 14.7.4.5

4.4.1 Width of each ring

$19 \geq t_b \geq 0.02\, D_p$ and, 6.0

19 ≥ 19 < 23.0 and, 6.0

Use: tb = 19 (mm)

4.4.2 Thickness of each ring

$h_b \geq 0.2\, t_b$

4 ≥ 3.8

Use: hb = 4 (mm)

4.5 PTFE Confined Sheet PTFE

Check contact pressure at service limit state

Total load = 20000 (kN)

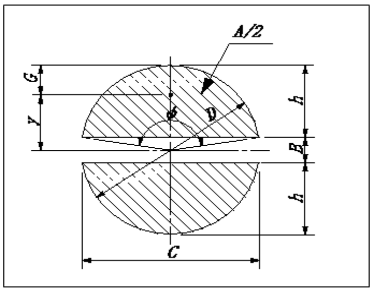
Allowable average contact pressure = 31 (N/mm^2) (AASHTO 14.7.2.4-1)

Allowable area

$$A = \frac{Rv(SLS)}{Stress} = \frac{20000 \times 1000}{31.0} = 645161 \quad mm^2$$

D	B	φ	A	A/2
1100	100	2.960	840483	420242

OK



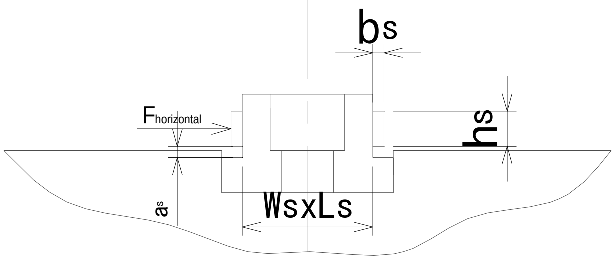
Dimension of PTFE plate

D	r	h	C	G	y	a1	a2
1100	550	500	1095.45	289.3	260.7	3	7
						OK	**OK**

a1 = thickness of the part of PTFE out the piston (a1 ≥ 2 mm) if D > 600 mm, a1>3)

a2 = thichness of the part of PTFE in the piston (a2 ≥ 5 mm) if D > 600 mm, a2>7)

4.6 Guide



4.6.1 Dimension of bolt of guide key

Number of bolt in guide key = 11

Diameter of bolt = 24 mm

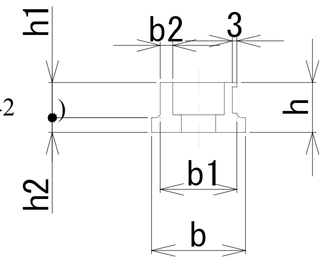
Root diameter = 20.752 mm

Stress area = 338.2 mm2

4.6.2 Dimension of guide key

hs	bs	as	Ws	Ls
21	3	3	88	1235

mm



Height of guide key = max (hs + 6 + 15 ; Dbolt + 2 + 16 ; 42)

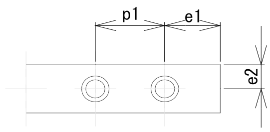
= max (42 ; 42 ; 42) = 42 mm

Height of the part in the backing plate = 15 mm < 16 mm **OK**

b1	b2	b	h1	h2	h	Ls
88	24.5	100	27	15	42	1235

(mm)

4.6.3 Check edge distance of guide bolt



Standard edge distance = 42 mm

p1 = 100 mm e1 = 118 mm e2 = 44 mm

\> 72 mm **OK** **OK**

4.6.4 Shear resistance at the guide

Vn ≥ Ultimate Horizontal Load

here, Vn = Φv 0.58 Fy (Ws Ls - NbAhole bolt)

= 1 × 0.58 × 345 × (88 × 1235 - 11 × π × 39 ² ÷ 4)

= 19117455 N

Check 19117 > 4050 (kN) **OK**

4.6.5 Bending resistance at the guide

$P_r \geq P_{uh}$

here, $P_r = \Phi_f F_y$ = 1.0 × 345 = 345 Mpa

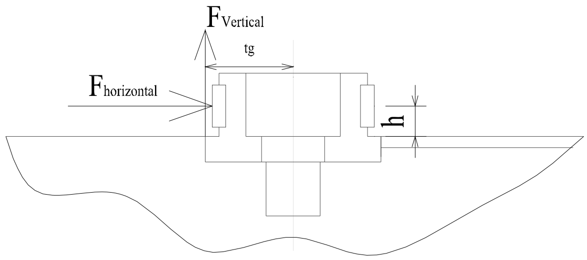
$$P_{uh} = \frac{M_{uh}}{Z_{y\text{-}y}} = \frac{54675000}{1578767} = 34.6 \text{ Mpa}$$

M_{uh} = Ul.Hor. Load × (hs/2 + as) = 4050 × 13.5 = 54675 kN.㎜

$$Z_{y\text{-}y} = \frac{Ls \times Ws^2}{6} - 0.1 \times D\text{\^{}}3xn = \frac{1235 \times 88^2}{6} - 0.1 \times 24^3 \times 11 = 1578767 \text{ mm3}$$

Check 345 ≥ 35 Mpa **OK**

4.6.6 Tensile resistance of guide bolt M 24 Root diameter = 20.752 mm



Fhorizontal = 4050 kN (ASTM A307)

tg = 50 mm

h = 13.5 mm

$$\text{Fvertical} = \frac{4050 \times 13.5}{50 \times 2}$$

= 1093.5 kN

$T_n \geq$ Fvertical

here, $T_n = 0.76\ NbA_b F_{ub}$ = 11 x 0.76 × 338 × 414 = 1170621 N

F_{ub} = specified minimum tensile strength of the bolt = 414 Mpa

A_b : area of the bolt corresponding to the nominal diameter (㎟)

$$A_b = \frac{\pi \times d_b^2}{4} = \frac{\pi \times 20.752^2}{4} = 338 \text{ mm2}$$

Check

1171 ≥ 1094 kN **OK**

4.7 Middle plate

4.7.1 Dimension of middle plate

Longitudinal dimension = 1235 mm

Transverse dimension = 1235 mm

Thickness of middle plate = 20 mm

Height of whole piston (include middle) = 100 mm

4.7.2 Check bending of middle plate

(1)Bending Moment

M=1/2×R×{ y-2/(3×π)×D1) = 0.5 x 4050 x | 260.7 - 2 ÷ (3 x π) x 1150 |

= 33681 kN

(2)Check bending yield resistance of middle plate

$\sigma = M/(1/6 \times D2 \times t^2) \leqq fy$

σ = 33681119 ÷ (0.17 x 1130 x 100 ^2) = 18 N/mm2 < 345 N/mm2

OK

4.8 Anchor bolt
High Strength Bolt

Dbolt	=	80	mm	Fubmin	=	725	Mpa	Fy =	560	Mpa	ASTM A 325
Lb	=	1100	mm	Root diameter	=	73.505	mm				AASHTO 6.4.3
Nbolt	=	4									

The minimum edge distance of anchor bolt = 120 mm

The pitch of the anchor bolt = 1100 mm

4.8.1 Tensile resistance

$T_n \geq T_{uv}$

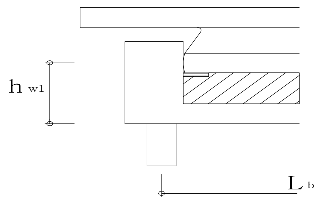
here, $T_n = 0.76\ A_b\ F_{ub}$ = 0.76 × 4244 × 725 = 2338444 N

F_{ub} = specified minimum tensile strength of the bolt = 725 Mpa

A_b : area of the bolt corresponding to the nominal diameter (㎟)

$$A_b = \frac{\pi \times d_{bolt}^{\ 2}}{4} = \frac{\pi \times 73.505^{\ 2}}{4} = 4244 \text{ mm2}$$

T_{uv} = UHL × h_{w1} / L_b
= 4050 × 193 / 1100
= 708.8 kN



2338 ≥ 709	kN	**OK**

4.8.2 Shear resistance

$R_n \geq$ USL

here, R_n = 4677 kN

- Where threads are excluded from the shear plane

$R_n = 0.48\ A_b\ F_{ub}\ N_{bolt}$ = 0.48×4244×725×4 = 5907648 N

- Where threads are included in the shear plane

$R_n = 0.38\ A_b\ F_{ub}\ N_{bolt}$ = 0.38×4244×725×4 = 4676888 N

Fub = specified minimum tensile strength of the bolt = 725 Mpa

4677 ≥ 4050	kN	**OK**

4.9 Anchor Socket Check

a. Dimension of Socket Check

Diameter of Socket	ds	160
Length of Socket	Ls	750
Number of Socket	Ns	4

b. Shear Resistance of the concrete around the rod

Qn= 0.5 Asc √(f'c Ec)x Ns = 0.5 x 15080 x √(40 x 30358) x 4 = 33234314 N

$$Asc = \frac{\Pi \times (160^2 - 80^2)}{4} = 15080 \text{ mm}^2$$

f'c = 40 (Mpa)

Ec = 4800√(f'c) = 4800 x √(40) = 30358 Mpa

Check Qn > Horizontal Force

33234 > 4050	kN	**OK**

4.10 Anchor Rod Check G345S fy = 345 Mpa fu = 450 Mpa

a. Shear resistance

Fs,r = Φs x Asc/2 x fy = 1 x 7540 x 345 = 2601239 (N)

Check Fs,r > Fh/4

2601 > 1013 kN **OK**

b. Tensile resistance

Ft,r = Φt x Asc x fu = 1 x 15080 x 450 = 6785840.13 (N)

Check Ft,r > Fh/2

6786 > 2025 kN **OK**

4.11 Check Compression strength of concrete of pier 5.7.5

Bottom f'c = 40 (Mpa)

The dimension of bottom plate

1320 x 1320 / 4 x pi + 4 x (340 + 300) x 258 / 2 = 1698373 (mm²)

The dimension of mortal （assume) 1935.46 x 1935.46 = 3746008.13 (mm²)

The factor bearing resistance shall be taken as $P_r = \phi P_n$

$$\left.\begin{array}{l} P_n = 0.85 f_c' A_1 m \\ \phi = 0.7 \\ m = \sqrt{\dfrac{A_2}{A_1}} \end{array}\right\}$$

Pr = 0.7 x 0.85 x 40 x 1698373 x 1.49 = 60031308 (N)

Check: Ultimate vertical load < Concrete resistance

27000 (kN) < 60031 (kN) **O.K**

Stress at concrete surface:

σ = N / Area of bearing place = 27000 * 1000 / 1698373 = 15.9 Mpa

Check 15.9 < 20 Mpa **OK**

4.12 Adhesive stress between anchor rod and non-shringkage mortar of substructer

Tad.allow = Ad x fad.allow

= Π x drod x Lrod x fad.allow

= Π x 160 x 750 x 1

= 376991.12 N

Tad.allow ≥ Tu/2 (kN)

377 ＞ 354 kN **O.K**

drod	160	mm
Lrod	750	mm
f'c	40	Mpa
fad.allow	1	

Design Strength of mortar (Mpa)	21	24	27	30	40	50	60
Round rebar	0.7	0.8	0.85	0.9	1	1	1
Deform rebar	1.4	1.6	1.7	1.8	2	2	2

4.13. Anchor Plate check

4.13.1 Welding check : Strength limit state

Weld Metal	E70xx	with	Fexx =	485	MPa	Table 6.6.2-2	a	=	340	mm
Minimum size of fillet weld			W =	10	mm		b	=	300	mm
Effective Length	0.707	x	W =	7.07	mm		t	=	60	mm

The resistance of fillet weld in shear length shall be taken as:

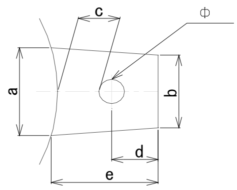
$$\begin{cases} R_r = 0.6\phi_{e2}F_{exx}0.707w \times 8a = 0.6 \times 0.8 \times 485 \times 0.707 \times W \times 8a = 4476837 \text{ N} \\ R_r = \phi_v R_n = \phi_v 0.58 A_g F_y \times 4 = 1 \times 0.58 \times 17000 \times 345 \times 4 = 13606800 \text{ N} \end{cases}$$

ϕ_{e2} = 0.8 6.5.4.2

ϕ_v = 1

Welding resistance Rr = 4476837 N

Check 4477 > 4050 kN **OK**

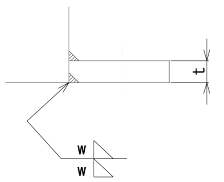


4.13.2 Minimum edges distance Table 6.13.2.6.6-1

Diameter of hole = 86 mm

Sheared Edges = 136 mm

d = 140 mm **OK**



4.14 Upper Plate

4.14.1 Dimension of upper plate

a. Determined by the requested displacement of bearing Longitudinal Displacement = 200 mm

Longitudinal dimension = 1530 mm

Transverse dimension = 1130 mm

Thickness of Upper plate = 50 mm

b. Height of sliding system of upper plate = 22 mm

Width of sliding system of upper plate = 100 mm

Length of sliding system of upper plate = 1530 mm

4.15 Anchor Bar

4.15.1 Diameter and arrangment of anchor bar

a. Anchor Bar: ASTM F 1554-07a (Pg. 397) Grade 55 (AASHTO 6.4.3.1)

Diameter Db = 48 mm

Ab = 1424.44 mm2 (root diameter = 42.587 mm)

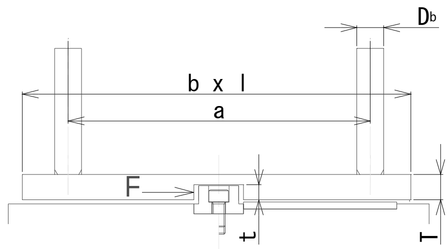
Fub = 517 Mpa

b. Arrangement of anchor bar

Longitudinal direction = 4 rows

Transverse direction = 4 rows

Number of anchor bar = 16 pieces



4.15.2 Edge Distance and Spacing Distance of anchor bars

Spacing between bolts not less than 3 times the diameter of the anchor bars 6.13.2.6.1

Arrangement of anchor bar on one side of sliding system

Choice spacing between bars:	Transverse direction	=	335	mm	>	144	= 3 * D bolt	**OK**
	Longitudinal direction	=	450	mm	>	144	= 3 * D bolt	**OK**

Edge Distance of bolts

Transverse	=	90	mm	>	80	mm	= 1.67 * D bolt	**OK**
Longitudinal	=	90.0	mm	>	80	mm	= 1.67 * D bolt	**OK**

4.15.3 Tension resistance of anchor bars

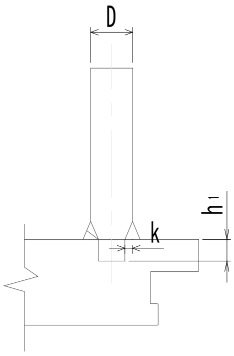
Ft,r = Φy x Ab x fy

= 0.95 x 1424.4 x 560 = 757802 N

Check 758 > 157 kN **OK**

4.15.4 Shear resistance with welding

4.15.4.1 Design of connection



Diameter of anchor bar D = 48 mm (root diameter = 42.587 mm)

Diamension of welding line k = 9 mm

Length of anchor bar leg in backing plate h1 = 0.8×(D-2×k) = 24 mm

Diameter of anchor bar inside upper plate M 30

Minimum thickness of backing plate = 50 mm

Welding metal E70XX with Fexx = 485 Mpa

Effective length of welding connection k' = $\sqrt{3}\times k$ = 15.6 mm

Perimeter of sheared section = $\pi\times d$ = 94.2 mm

Area of sheared section = $\pi\times D^2 \div 4$ = 1424.4 mm2

4.15.4.2 Check resistance of welding connection

4.15.4.2.1 Shear resistance of one anchor bar

$$\begin{cases} R_r = 0.6\phi_{e2}F_{exx}\sqrt{3}k \times \pi d = 0.6 \times 0.8 \times 485 \times 1.73 \times 9 \times 94.2 = 342025 \text{ N} \\ R_r = \phi_v R_n = \phi_v 0.58 A_g F_y = 1 \times 0.58 \times 1424.4 \times 345 = 285030 \text{ N} \end{cases}$$

ϕ_{e2} = 0.8 6.5.4.2

ϕ_v = 1

Checking shear resistance of anchor bar

Shear Resistance of anchor bar = min (shear resistance of anchor bar; shear resistance of welded connection)

= min(342.1;285.1) = 285.03 kN

Check 285.03 × 16 = 4560 > 4050 kN **OK**

4.15.5 Adhesive stress between anchor bar and superstructer

dbar	48	mm
Lbar	450	mm
f'c	40	Mpa
fad.allow	1	

Tad.allow = Ad x fad.allow

= Π x drod x Lrod x fad.allow

= Π x 48 x 450 x 1

= 67858.40

Fv = F x (T - t / 2) / a

= 4050 x (50 - 22 / 2) / 1005

= 157.164 kN

Tad.allow ≥ Tu/4 (kN)

67.86 > 39.29 **O.K**

Design Strength of mortar (Mpa)	21	24	27	30	40	50	60
Round rebar	0.7	0.8	0.85	0.9	1	1	1
Deform rebar	1.4	1.6	1.7	1.8	2	2	2

4.16 Check Compression strength of concrete of superstructure

f'c = 40 (Mpa)

The dimension of top plate 1530 x 1130 = 1728900 (mm2)

The dimension of bearing area 1630 x 1230 = 2004900 (mm²)

The factor bearing resistance shall be taken as $P_r = \phi P_n$

$$\left.\begin{aligned} P_n &= 0.85 f_c' A_1 m \\ \phi &= 0.7 \\ m &= \sqrt{\frac{A_2}{A_1}} \end{aligned}\right\}$$

Pr = 0.7 x 0.85 x 40 x 1728900 x 1.07687

= 44310663 (N)

Check: Ultimate vertical load < Concrete resistance

27000 (kN) < 44311 (kN) **O.K**

Stress at concrete surface:

σ = N / Area of bearing place = 27000 * 1000 / 1728900 = 15.6 Mpa

Check 15.62 < 20 Mpa **OK**

POT BEARING CALCULATION SHEET

PROJECT :
TYPE : FIX

1. SPECIFICATIONS
AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS, 2007

2. DESIGN CONDITIONS
2.1 Load and combination Loads

	Unit	Service limit state		Ultimate limit state				kN
Maximum Vertical Load	kN	20000	kN	20000	x	1.35	=	27000
Maximum Horizontal Load	kN	3000	kN	3000	x	1.35	=	4050

2.2 Rotation
- Minimum rotation θr = 0.01 rad
- Design rotation θu = 0.005 + 0.005 + 0.01 = 0.02 rad
-Vertical deflection δ u = 3 mm

2.3 Displacement
・ Transverse = ± 0 mm
・ Longitudinal = ± 0 mm

2.4 Compressive Strength of concrete
・ Superstructure : 40 Mpa
・ Supstructure : 40 Mpa

3. MATERIALS
3.1 Steel ASTM A 709M, Grade 345S, 345W
Minimum tensile Strength Fu = 450 (Mpa)
Minimun Yield Strength Fy = 345 (Mpa)
3.2 Rubber Natural rubber
3.3 Bolt
・Anchor bolt ASTM A 325
・Anchor bar ASTM F1554
3.4 Internal ring Brass ring
3.5 External ring Chloroprene rubber

4. Calculation of Pot Bearing

4.1 The diameter of the pot and the elastomeric pad 14.7.4.4

4.1.1 Average stress is satisfied

$$Ap \geq \frac{\text{Service vertical load}}{25} = \frac{20000 \times 1000}{25} = 800000 \quad (mm^2)$$

4.1.2 Diameter of the elastomeric

$$Dp = \sqrt{\frac{4 \times A_p}{\pi}} \geq \frac{\sqrt{(4 \times 800000)}}{\sqrt{(3.14)}} = 1010 \quad (mm^2)$$

Use: Dp = 1150 (mm)	**OK**

4.1.3 The thickness of the elastomeric pad 14.4.4.3-1

$$h_r \geq 3.33 D_p \theta_u \Leftrightarrow hr \geq 3.33 \times 1150 \times 0.02 = 77 \quad (mm)$$

Use: hr = 90 (mm)	**OK**

4.2 Pot 14.7.4.6

4.2.1 Minimum thickness of base (Set on Concrete)

＊Set on concrete

$$t_b \geq \max \begin{cases} 0.06 \times D_p = 0.06 \times 1150 = 69 \text{ mm} \\ 20mm = 20 \text{ mm} \\ \sqrt{\dfrac{25 H_u \theta_u}{F_y}} = \sqrt{\dfrac{25 \times 4050 \times 1000 \times 0.02}{345}} = 76.6 \quad (mm) \end{cases}$$

Use: tb = 85 (mm)	**OK**

＊Set on steel girder or distribution plate

$$t_b \geq \max \begin{cases} 0.04 \times D_p = 0.04 \times 1150 = 46 \text{ mm} \\ 12.5mm = 12.5 \text{ mm} \\ \sqrt{\dfrac{25 H_u \theta_u}{F_y}} = \sqrt{\dfrac{25 \times 4050 \times 1000 \times 0.02}{345}} = 76.6 \quad (mm) \end{cases}$$

Use: tb = 85 (mm)	**OK**

＊Use: tb = 85 mm

4.2.2 Thickness of pot wall

$$t_w \geq \max \begin{cases} \dfrac{D_p \sigma_s}{1.25 F_y} = \dfrac{1150 \times 25.9943}{1.25 \times 345} = 69.32 \quad (mm) & 14.7.4.6\text{-}5 \\ 20mm = 20 \quad (mm) & 14.7.6\text{-}6 \\ \sqrt{\dfrac{25 H_u \theta_u}{F_y}} = \sqrt{\dfrac{25 \times 4050 \times 1000 \times 0.02}{345}} = 77 \quad (mm) & \end{cases}$$

14.7.4.7-1

Use: tw = 85 (mm)	**OK**

$$\sigma s = \frac{27000 \times 1000}{\dfrac{3.14 \times 1150^2}{4}} = 26.0 \quad (N/mm^2)$$

4.2.3 The pot cavity height , hp1 C14.7.4.3-1

$$h_{p1} \geq (0.5 D_p \theta_u) + h_r + h_w$$

hp1 ≥ (0.5 x 1150 x 0.02) + 90 + 35 = 137

Use: hp1 = 140 (mm)	**OK**

4.3 Piston

4.3.1 Piston thickness

The piston shall have the same shape as the inside of the pot. Its thickness shall be adequate to resist the loads imposed on it, but shall not be less than 6 percent of the inside diameter of the pot, Dp, except at the rim.

4.3.2 Height of piston rim 14.7.4.7

$$h_w \geq \max \begin{cases} \dfrac{1.5 H_u}{D_p F_y} = \dfrac{1.5 \times 4050 \times 1000}{1150 \times 345} = 15.3 \text{ (mm)} \\ 3mm = 3 \text{ (mm)} \\ 0.03 D_p = 0.03 \times 1150 = 34.5 \text{ (mm)} \end{cases}$$

Use: hw = 35 (mm)	**OK**

4.3.3 The vertical clearance, hp2 C14.7.4.3-2

$$h_{p2} \geq R_0 \theta_u + 2\delta_u + 3$$

hp2 ≥ 660 x 0.02 + 2 x 3 + 3 = 22.2 (mm)

Radial distance from center of pot to edge of pot wall

R0 = Dp/2 + tw = 1150 / 2 + 85 = 660 (mm)

Use: hp2 = 30 (mm)	**OK**

4.3.4 Clearance, c C14.7.4.7

The diameter of the piston rim shall be the inside diameter of the pot less a clearance

c ≥ 0.5 (mm)

$$c \geq \Theta_u \times \left(h_w - \frac{D_p \times \Theta_u}{2} \right) = 0.020 \times \left(35 - \frac{1150 \times 0.02}{2} \right) = 0.470 \text{ mm}$$

Use: c = 1 (mm)

4.4 Brass ring 14.7.4.5

4.4.1 Width of each ring

$19 \geq t_b \geq 0.02\ D_p$ and, 6.0

19 ≥ 19 < 23.0 and, 6.0

Use: tb = 19 (mm)

4.4.2 Depth of each ring

$h_b \geq 0.2\ t_b$

4 ≥ 3.8

Use: hb = 4 (mm)

5. Anchor bolt

High Strength Bolt

Dbolt =	80	mm	Fubmin =	725	Mpa	Fy =	560	Mpa
Lb =	1100	mm	Root diameter =	73.505	mm			
Nbolt =	4							

ASTM A 325
AASHTO 6.4.3

The minimum edge distance of anchor bolt = 120 mm

The pitch of the anchor bolt = 1100 mm

5.1 Tensile resistance

$T_n \geq T_{uv}$

here, $T_n = 0.76\ A_b\ F_{ub}$ = 0.76 × 4244 × 725 = 2338444 N

F_{ub} = specified minimum tensile strength of the bolt = 725 Mpa

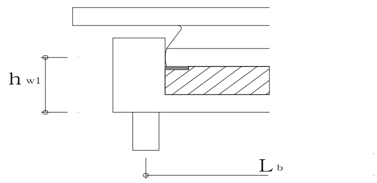
A_b : area of the bolt corresponding to the nominal diameter (㎟)

$$A_b = \frac{\pi \times d_{bolt}^{\ 2}}{4} = \frac{\pi \times 73.505^{\ 2}}{4} = 4244 \text{ mm2}$$

T_{uv} = UHL × h_{w1} / L_b

= 4050 × 193 / 1100

= 708.8 kN



$\boxed{2338 \geq 709}$ kN **OK**

5.2 Shear resistance

$R_n \geq$ USL

here, R_n = 4677 kN

- Where threads are excluded from the shear plane

$R_n = 0.48\ A_b\ F_{ub}\ N_{bolt}$ = 0.48×4244×725×4 = 5907648 N

- Where threads are included in the shear plane

$R_n = 0.38\ A_b\ F_{ub}\ N_{bolt}$ = 0.38×4244×725×4 = 4676888 N

Fub = specified minimum tensile strength of the bolt = 725 Mpa

$\boxed{4677 \geq 4050}$ kN **OK**

5.3 Anchor Socket Check

a. Dimension of Socket Check

Diameter of Socket	ds	160
Length of Socket	Ls	750
Number of Socket	Ns	4

b. Shear Resistance of the concrete around the rod

Qn= 0.5 Asc √(f'c Ec)x Ns = 0.5 x 15080 x √(40 x 30358) x 4 = 33234314 N

$$Asc = \frac{\Pi \times (160^2 - 80^2)}{4} = 15080 \text{ mm}^2$$

f'c = 40 (Mpa)

Ec = 4800√(f'c) = 4800 x √(40) = 30358 Mpa

Check Qn > Horizontal Force

$\boxed{33234 > 4050}$ kN **OK**

5.4 Anchor Rod Check G345S fy = 345 Mpa fu = 450 Mpa

a. Shear resistance

Fs,r = Φs x Asc/2 x fy = 1 x 7540 x 345 = 2601239 (N)

Check Fs,r > Fh/4

2601 > 1013 kN **OK**

b. Tensile resistance

Ft,r = Φt x Asc x fu = 1 x 15080 x 450 = 6785840.13 (N)

Check Ft,r > Fh/2

6786 > 2025 kN **OK**

5.5 Check Compression strength of concrete of pier 5.7.5

Bottom $f'c$ = 40 (Mpa)

The dimension of bottom plate

1320 x 1320 / 4 x pi + 4 x (340 + 300) x 273 / 2 = 1717918 (mm²)

The dimension of mortal 1966 x 1966 = 3865156 (mm²)

The factor bearing resistance shall be taken as $P_r = \phi P_n$

$$\left.\begin{array}{l} P_n = 0.85 f'_c A_1 m \\ \phi = 0.7 \\ m = \sqrt{\dfrac{A_2}{A_1}} \end{array}\right\}$$

Pr = 0.7 x 0.85 x 40 x 1717918 x 1.50 = 61328402.9 (N)

Check: Ultimate vertical load < Concrete resistance

27000 (kN) < 61328 (kN) **O.K**

Stress at concrete surface:

σ = N / Area of bearing place = 27000 * 1000 / 1717918 = 15.7 Mpa

Check 15.7 < 20 Mpa **OK**

5.6 Adhesive stress between anchor rod and non-shringkage mortar of substructer

drod	160	mm
Lrod	750	mm
f'c	40	Mpa
fad.allow	1	

Tad.allow = Ad x fad.allow

= Π x drod x Lrod x fad.allow

= Π x 160 x 750 x 1

= 376991.12 N

Tad.allow ≥ Tu/2 (kN)

377 > 354 kN **O.K**

Design Strength of mortar (Mpa)	21	24	27	30	40	50	60
Round rebar	0.7	0.8	0.85	0.9	1	1	1
Deform rebar	1.4	1.6	1.7	1.8	2	2	2

6. Anchor Plate check

6.1 Welding check : Strength limit state

Weld Metal E70xx with Fexx = 485 MPa Table 6.6.2-2 a = 340 mm

Minimum size of fillet weld W = 6 mm b = 300 mm

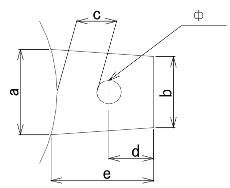
Effective Length 0.707 x W = 4.242 mm t = 60 mm

The resistance of fillet weld in shear per 1mm length shall be taken as:

$$\begin{cases} R_r = 0.6\phi_{e2} F_{exx} 0.707 w \times 8a = 0.6 \times 0.8 \times 485 \times 0.707 \times W \times 8a = 2686102 \text{ N} \\ R_r = \phi_v R_n = \phi_v 0.58 A_g F_y \times 4 = 1 \times 0.58 \times 18360 \times 345 \times 4 = 14695344 \text{ N} \end{cases}$$

ϕ_{e2} = 0.8 6.5.4.2

ϕ_v = 1



Welding resistance Rr = 2686102 N

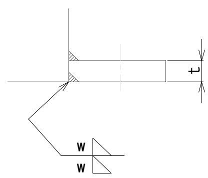
Check 2686 < 4050 kN **NG**

6.2 Minimum edges distance Table 6.13.2.6.6-1

Diameter of hole = 86 mm

Sheared Edges = 136 mm

d = 140 mm **OK**



7 Upper Plate

7.1 Dimension of upper plate

a. Determined by the requested displacement of bearing — Longitudinal Displacement = 0 mm

Longitudinal dimension = 1320 mm

Transverse dimension = 1320 mm

Thickness of Upper plate = 50 mm

8 Anchor Bar

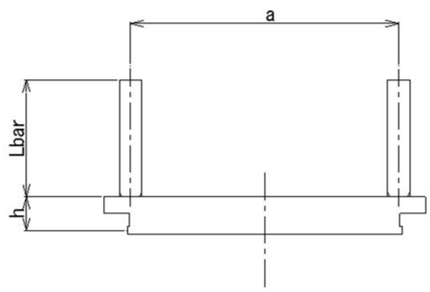
8.1 Diameter and arrangment of anchor bar

a. Anchor Bar: ASTM F 1554-07a (Pg. 397) Grade 55 (AASHTO 6.4.3.1)

Diameter Db = 48 mm

Ab = 1424.44 mm2 (root diameter = 42.587 mm)

Fub = 517 Mpa



b. Arrangement of anchor bar

Longitudinal direction = 4 rows

Transverse direction = 4 rows

Number of anchor bar = 16 pieces

8.2 Edge Distance and Spacing Distance of anchor bars

Spacing between bolts not less than 3 times the diameter of the anchor bars 6.13.2.6.1

Arrangement of anchor bar on one side of sliding system

Choice spacing between bars: Transverse direction = 380 mm > 144 = 3 * D bolt **OK**

Longitudinal direction = 380 mm > 144 = 3 * D bolt **OK**

Edge Distance of bolts

Transverse = 90 mm > 80 mm = 1.67 * D bolt **OK**

Longitudinal = 90.0 mm > 80 mm = 1.67 * D bolt **OK**

8.3 Tension resistance of anchor bars

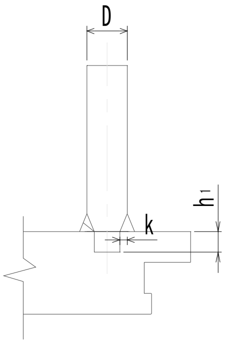
$F_{t,r} = \Phi y \times Ab \times fy$

$= 0.95 \times 1424.4 \times 560 = 757802$ N

Check 758 > 400 kN **OK**

8.4 Shear resistance with welding

8.4.1 Design of connection



Diameter of anchor bar D = 48 mm (root diameter = 42.587 mm)

Diamension of welding line k = 9 mm

Length of anchor bar leg in backing plate h1 = 0.8×(D-2×k) = 24 mm

Diameter of anchor bar inside upper plate M 30

Minimum thickness of backing plate = 50 mm

Welding metal E70XX with Fexx = 485 Mpa

Effective length of welding connection k' = $\sqrt{3}\times k$ = 15.6 mm

Perimeter of sheared section = π× d = 94.2 mm

Area of sheared section = π ×D²÷4 = 1424.4 mm2

8.4.2 Check resistance of welding connection

8.4.2.1 Shear resistance of one anchor bar

$$\begin{cases} R_r = 0.6\phi_{e2} F_{exx}\sqrt{3}k & \times \pi d = 0.6 \times 0.8 \times 485 \times 1.73 \times 9 \times 94.2 = 342025 \text{ N} \\ R_r = \phi_v R_n = \phi_v 0.58 A_g F_y & = 1 \times 0.58 \times 1424.4 \times 345 = 285030 \text{ N} \end{cases}$$

ϕ_{e2} = 0.8 6.5.4.2

ϕ_v = 1

Checking shear resistance of anchor bar

Shear Resistance of anchor bar = min (shear resistance of anchor bar; shear resistance of welded connection)

= min(342.1;285.1) = 285.03 kN

Check 285.03 × 16 = 4560 ＞ 4050 kN **OK**

8.5 Adhesive stress between anchor bar and superstructer

dbar	48	mm
Lbar	670	mm
f'c	40	Mpa
fad.allow	1	

Tad.allow = Ad x fad.allow

= Π x drod x Lrod x fad.allow

= Π x 48 x 670 x 1

= 101033.62

Fv = F x (H / 2) / a

= 4050 x (113 / 2) / 1140

= 399.671 kN

Tad.allow ≥ Tu/4 (kN)

101.03 ＞ 99.92 **O.K**

Design Strength of mortar (Mpa)	21	24	27	30	40	50	60
Round rebar	0.7	0.8	0.85	0.9	1	1	1
Deform rebar	1.4	1.6	1.7	1.8	2	2	2

8.6 Check Compression strength of concrete of superstructure

f'c = 40 (Mpa)

The dimension of top plate	1320	x	1320	=	1742400	(mm2)
The dimension of bearing area	1420	x	1420	=	2016400	(mm²)

The factor bearing resistance shall be taken as $P_r = \phi P_n$

$$\left.\begin{array}{l} P_n = 0.85 f_c' A_1 m \\ \phi = 0.7 \\ m = \sqrt{\dfrac{A_2}{A_1}} \end{array}\right\}$$

Pr = 0.7 x 0.85 x 40 x 1742400 x 1.07576

= 44610720 (N)

Check: Ultimate vertical load < Concrete resistance

27000 (kN) < 44611 (kN) **O.K**

Stress at concrete surface:

σ = N / Area of bearing place = 27000 * 1000 / 1742400 = 15.5 Mpa

Check 15.50 < 20 Mpa **OK**

POT BEARING CALCULATION SHEET

PROJECT :

TYPE : FREE

1. SPECIFICATIONS

AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS, 2007

2. DESIGN CONDITIONS

2.1 Load and combination Loads

	Unit	Service limit state		Ultimate limit state				kN
Maximum Vertical Load	kN	20000	kN	20000	x	1.35	=	27000
Maximum Horizontal Load	kN	2000	kN	2000	x	1.35	=	2700

2.2 Rotation

- Minimum rotation θr = 0.01 rad
- Design rotation θu = 0.005 + 0.005 + 0.01 = 0.02 rad

-Vertical deflection δu = 3 mm

2.3 Displacement

・ Transverse = ± 40 mm

・ Longitudinal = ± 190 mm

2.4 Compressive Strength of concrete

・ Superstructure : 40 Mpa

・ Supstructure : 40 Mpa

3. MATERIALS

3.1 Steel ASTM A 709M, Grade 345S, 345W

Minimum tensile Strength Fu = 450 (Mpa)

Minimun Yield Strength Fy = 345 (Mpa)

3.2 Rubber Natural rubber

3.3 Bolt

・Anchor bolt ASTM A 325 / ASTM F 1554

・Anchor bar ASTM F1554

3.4 PTFE ASTM D4894

3.5 Stainless steel

3.6 Internal ring Brass ring

3.7 External ring Chloroprene rubber

4. Calculation of Pot Bearing

4.1 The diameter of the pot and the elastomeric pad — 14.7.4.4

4.1.1 Average stress is satisfied

$$Ap \geq \frac{\text{Service vertical loac}}{25} = \frac{20000 \times 1000}{25} = 800000 \quad (mm^2)$$

4.1.2 Diameter of the elastomeric

$$Dp = \sqrt{\frac{4 \times A_p}{\pi}} \geq \frac{\sqrt{(4 \times 800000)}}{\sqrt{(3.14)}} = 1010 \quad (mm^2)$$

Use: Dp = 1150 (mm)	**OK**

4.1.3 The thickness of the elastomeric pad — 14.4.4.3-1

$$h_r \geq 3.33 D_p \theta_u \iff hr \geq 3.33 \times 1150 \times 0.02 = 77 \quad (mm)$$

Use: hr = 90 (mm)	**OK**

4.2 Pot — 14.7.4.6

4.2.1 Minimum thickness of base (Set on Concrete)

＊Set on concrete

$$t_b \geq \max \begin{cases} 0.06 \times D_p = 0.06 \times 1150 = 69 \text{ mm} \\ 20mm = 20 \text{ mm} \\ \sqrt{\dfrac{25 H_u \theta_u}{F_y}} = \sqrt{\dfrac{25 \times 2700 \times 1000 \times 0.02}{345}} = 62.6 \quad (mm) \end{cases}$$

Use: tb = 85 (mm)	**OK**

＊Set on steel girder or distribution plate

$$t_b \geq \max \begin{cases} 0.04 \times D_p = 0.04 \times 1150 = 46 \text{ mm} \\ 12.5mm = 12.5 \text{ mm} \\ \sqrt{\dfrac{25 H_u \theta_u}{F_y}} = \sqrt{\dfrac{25 \times 2700 \times 1000 \times 0.02}{345}} = 62.6 \quad (mm) \end{cases}$$

Use: tb = 85 (mm)	**OK**

＊Use: tb = 85 mm

4.2.2 Thickness of pot wall

$$t_w \geq \max \begin{cases} \dfrac{D_p \sigma_s}{1.25 F_y} = \dfrac{1150 \times 25.9943}{1.25 \times 345} = 69.32 \quad (mm) & 14.7.4.6\text{-}5 \\ 20mm = 20 \quad (mm) & 14.7.6\text{-}6 \\ \sqrt{\dfrac{25 H_u \theta_u}{F_y}} = \sqrt{\dfrac{25 \times 2700 \times 1000 \times 0.02}{345}} = 63 \quad (mm) & 14.7.4.7\text{-}1 \end{cases}$$

Use: tw = 85 (mm)	**OK**

$$\sigma s = \frac{27000 \times 1000}{3.14 \times \dfrac{1150^2}{4}} = 26.0 \quad (N/mm^2)$$

4.2.3 The pot cavity height , hp1 — C14.7.4.3-1

$$h_{p1} \geq (0.5 D_p \theta_u) + h_r + h_w$$

$$hp1 \geq (0.5 \times 1150 \times 0.02) + 90 + 35 = 137$$

Use: hp1 = 140 (mm)	**OK**

4.3 Piston

4.3.1 Piston thickness

The piston shall have the same shape as the inside of the pot. Its thickness shall be adequate to resist the loads imposed on it, but shall not be less than 6 percent of the inside diameter of the pot, Dp, except at the rim.

4.3.2 Height of piston rim 14.7.4.7

$$h_w \geq \max \begin{cases} \dfrac{1.5H_u}{D_p F_y} = \dfrac{1.5 \times 2700 \times 1000}{1150 \times 345} = 10.2 \text{ (mm)} \\ 3mm = 3 \text{ (mm)} \\ 0.03D_p = 0.03 \times 1150 = 34.5 \text{ (mm)} \end{cases}$$

Use: hw =	35	(mm)	OK

4.3.3 The vertical clearance, hp2 C14.7.4.3-2

$$h_{p2} \geq R_0\theta_u + 2\delta_u + 3$$

hp2 ≥ 660 x 0.02 + 2 x 3 + 3 = 22.2 (mm)

Radial distance from center of pot to edge of pot wall

R0 = Dp/2 + tw = 1150 / 2 + 85 = 660 (mm)

Use: hp2 =	30	(mm)	OK

4.3.4 Clearance, c C14.7.4.7

The diameter of the piston rim shall be the inside diameter of the pot less a clearance

c ≥ 0.5 (mm)

$$c \geq \Theta_u \times \left(h_w - \frac{D_p \times \Theta_u}{2} \right) = 0.020 \times \left(35 - \frac{1150 \times 0.02}{2} \right) = 0.470 \text{ mm}$$

Use: c =	1	(mm)

4.4 Brass ring 14.7.4.5

4.4.1 Width of each ring

$19 \geq t_b \geq 0.02\ D_p$ and, 6.0

19 ≥ 19 < 23.0 and, 6.0

Use: tb =	19	(mm)

4.4.2 Depth of each ring

$h_b \geq 0.2\ t_b$

4 ≥ 3.8 mm

Use: hb =	4	(mm)

4.5 PTFE

Check contact pressure at service limit state

Total load = 20000 (kN)

Allowable average contact pressure = 31 (N/mm²)

(AASHTO 14.7.2.4-1)

Allowable area

$$A = \frac{Rv(SLS)}{Stress} = \frac{20000 \times 1000}{31.0} = 645161 \text{ mm}^2$$

D	A	
1100	950332	OK

Dimension of PTFE plate

D	a1	a2
1100	3	7
	OK	OK

a1 = thickness of the part of PTFE out the piston (a1 ≥ 2 mm) if D > 600 mm, a1>3)

a2 = thichness of the part of PTFE in the piston (a2 ≥ 5 mm) if D > 600 mm, a2>7)

4.6 Middle plate:

Thickness of middle plate: h = 0 mm

Dimension of middle plate: Circle with D = 1128 mm

5. Anchor bolt

High Strength Bolt

Dbolt	=	72	mm	Fubmin	=	725	Mpa	Fy =	560 Mpa
Lb	=	1089	mm	Root diameter	=	65.505	mm		
Nbolt	=	4							

ASTM A 325
AASHTO 6.4.3

The minimum edge distance of anchor bolt = 108 mm

The pitch of the anchor bolt = 1089 mm

5.1 Tensile resistance

$T_n \geq T_{uv}$

here, $T_n = 0.76\, A_b\, F_{ub}$ = 0.76 × 3371 × 725 = 1857421 N

F_{ub} = specified minimum tensile strength of the bolt = 725 Mpa

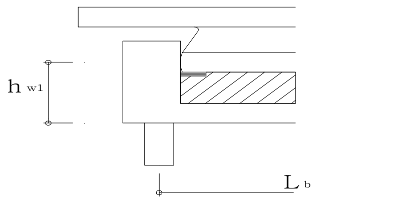
A_b : area of the bolt corresponding to the nominal diameter (㎟)

$$A_b = \frac{\pi \times d_{bolt}^{\ 2}}{4} = \frac{\pi \times 65.505^{\ 2}}{4} = 3371 \text{ mm2}$$

T_{uv} = UHL × h_{w1} / L_b

= 2700 × 193 / 1089

= 477.1 kN



1857 ≥ 477	kN	**OK**

5.2 Shear resistance

$R_n \geq$ USL

here, R_n = 3715 kN

- Where threads are excluded from the shear plane

$R_n = 0.48\, A_b\, F_{ub}\, N_{bolt}$ = 0.48×3371×725×4 = 4692432 N

- Where threads are included in the shear plane

$R_n = 0.38\, A_b\, F_{ub}\, N_{bolt}$ = 0.38×3371×725×4 = 3714842 N

Fub = specified minimum tensile strength of the bolt = 725 Mpa

3715 ≥ 2700	kN	**OK**

5.3 Anchor Socket Check

a. Dimension of Socket Check

Diameter of Socket	ds	160
Length of Socket	Ls	650
Number of Socket	Ns	4

b. Shear Resistance of the concrete around the rod

Qn= 0.5 Asc √(f'c Ec)x Ns = 0.5 x 16035 x √ (40 x 30358) x 4 = 35339154 N

$$Asc = \frac{\Pi \times (160^2 - 72^2)}{4} = 16035 \text{ mm}^2$$

f'c = 40 (Mpa)

Ec = 4800√(f'c) = 4800 x √ (40) = 30358 Mpa

Check Qn > Horizontal Force

35339 > 2700	kN	**OK**

5.4 Anchor Rod Check G345S fy = 345 Mpa fu = 450 Mpa

a. Shear resistance

Fs,r = Φs x Asc/2 x fy = 1 x 8017 x 345 = 2765984 (N)

Check Fs,r > Fh/4

2766 > 675 kN **OK**

b. Tensile resistance

Ft,r = Φt x Asc x fu = 1 x 16035 x 450 = 7215610.01 (N)

Check Ft,r > Fh/2

7216 > 1350 kN **OK**

6 Check Compression strength of concrete of pier 5.7.5

Bottom $f'c$ = 40 (Mpa)

The dimension of bottom plate

1320 x 1320 / 4 x pi + 4 x (320 + 280) x 253 / 2 = 1672078 (mm²)

The dimension of mortal 1926 x 1926 = 3709476 (mm²)

The factor bearing resistance shall be taken as $P_r = \phi P_n$

$$\left.\begin{aligned} P_n &= 0.85 f_c' A_1 m \\ \phi &= 0.7 \\ m &= \sqrt{\frac{A_2}{A_1}} \end{aligned}\right\}$$

Pr = 0.7 x 0.85 x 40 x 1672078 x 1.49 = 59273623.2 (N)

Check: Ultimate vertical load < Concrete resistance

27000 (kN) < 59274 (kN) **O.K**

Stress at concrete surface:

σ = N / Area of bearing place = 27000 * 1000 / 1672078 = 16.1 Mpa

Check 16.1 < 20 **OK**

7. Adhesive stress between anchor rod and non-shringkage mortar of substructer

Tad.allow = Ad x fad.allow

= Π x drod x Lrod x fad.allow

= Π x 160 x 650 x 1

= 326725.64 N

Tad.allow ≥ Tu/2 (kN)

327 > 239 kN **O.K**

drod	160	mm
Lrod	650	mm
f'c	40	Mpa
fad.allow	1	

Design Strength of mortar (Mpa)	21	24	27	30	40	50	60
Round rebar	0.7	0.8	0.85	0.9	1	1	1
Deform rebar	1.4	1.6	1.7	1.8	2	2	2

8. Anchor Plate check

8.1 Welding check : Strength limit state

Weld Metal E70xx with Fexx = 485 MPa Table 6.6.2-2

Minimum size of fillet weld W = 10 mm

Effective Length 0.707 x W = 7.07 mm

a = 320 mm

b = 280 mm

t = 60 mm

The resistance of fillet weld in shear per 1mm length shall be taken as:

$$\begin{cases} R_r = 0.6\phi_{e2} F_{exx} 0.707w \times 8a \\ R_r = \phi_v R_n = \phi_v 0.58 A_g F_y \times 4 \end{cases}$$

= 0.6 x 0.8 x 485 x 0.707 x W x 8a = 4213494 N

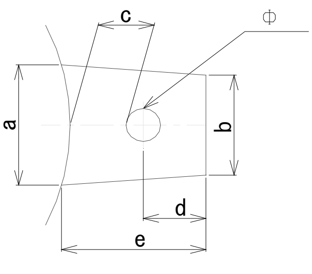
= 1 x 0.58 x 19200 x 345 × 4 = 15367680 N

ϕ_{e2} = 0.8 6.5.4.2

ϕ_v = 1

Welding resistance Rr = 4213494 N

Check 4213 > 2700 kN **OK**

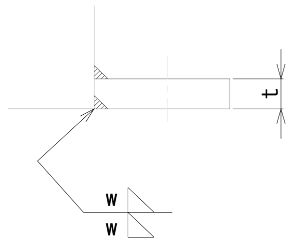


8.2 Minimum edges distance Table 6.13.2.6.6-1

Diameter of hole = 78 mm

Sheared Edges = 124 mm

d = 130 mm **OK**



9. Upper Plate

9.1 Dimension of upper plate

a. Determined by the requested displacement of bearing

Longitudinal dimension = 1660 mm

Transverse dimension = 1360 mm

Thickness of Upper plate = 50 mm

Longitudinal Displacement = 190 mm

Tranverse Displacement = 40 mm

10. Anchor Bar

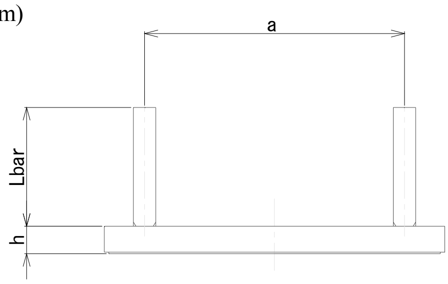
10.1 Diameter and arrangment of anchor bar

a. Anchor Bar: ASTM F 1554-07a (Pg. 397) Grade 55 (AASHTO 6.4.3.1)

Diameter Db = 48 mm

Ab = 1424.44 mm2 (root diameter = 42.587 mm)

Fub = 517 Mpa



b. Arrangement of anchor bar

Longitudinal direction = 4 rows

Transverse direction = 4 rows

Number of anchor bar = 16 pieces

10.2 Edge Distance and Spacing Distance of anchor bars

Spacing between bolts not less than 3 times the diameter of the anchor bars 6.13.2.6.1

Arrangement of anchor bar on one side of sliding system

Choice spacing between bars:	Transverse direction	=	390	mm	>	144	= 3 * D bolt	OK
	Longitudinal direction	=	490	mm	>	144	= 3 * D bolt	OK

Edge Distance of bolts

Transverse	=	95	mm	>	80	mm	= 1.67 * D bolt	OK
Longitudinal	=	95.0	mm	>	80	mm	= 1.67 * D bolt	OK

10.3 Tension resistance of anchor bars

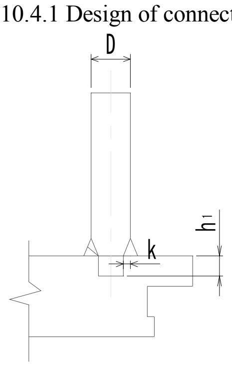
Ft,r = Φy x Ab x fy

= 0.95 x 1424.4 x 560 = 757802 N

Check 758 > 58 kN **OK**

10.4 Shear resistance with welding

10.4.1 Design of connection



Diameter of anchor bar D = 48 mm (root diameter = 42.587 mm)

Diamension of welding lin k = 9 mm

Length of anchor bar leg in backing plate h1 = 0.8×(D-2×k) = 24 mm

Diameter of anchor bar inside upper plate M 30

Minimum thickness of backing plate = 50 mm

Welding metal E70XX with Fexx = 485 Mpa

Effective length of welding connection k' = √3×k = 15.6 mm

Perimeter of sheared section = π× d = 94.2 mm

Area of sheared section = π ×D²÷4 = 1424.4 mm2

10.4.2 Check resistance of welding connection

10.4.2.1 Shear resistance of one anchor bar

$$\begin{cases} R_r = 0.6\phi_{e2} F_{exx}\sqrt{3}k \times \pi d = 0.6 \times 0.8 \times 485 \times 1.73 \times 9 \times 94.2 = 342025 \text{ N} \\ R_r = \phi_v R_n = \phi_v 0.58 A_g F_y = 1 \times 0.58 \times 1424.4 \times 345 = 285030 \text{ N} \end{cases}$$

ϕ_{e2} = 0.8 6.5.4.2

ϕ_v = 1

Checking shear resistance of anchor bar

Shear Resistance of anchor bolt = min (shear resistance of anchor bar; shear resistance of welded connection

= min(342.1;285.1) = 285.03 kN

Check 285.03 × 16 = 4560 > 2700 kN **OK**

10.4.3 Adhesive stress between anchor bar and superstructer

dbar	48	mm
Lbar	450	mm
f'c	40	Mpa
fad.allow	1	

Tad.allow = Ad x fad.allow

= Π x drod x Lrod x fad.allow

= Π x 48 x 450 x 1

= 67858.40

Fv = F x (T / 2) / a
= 2700 x (50 / 2) / 1170
= 57.6923 kN

Tad.allow ≥ Tu/4 (kN)

67.86 > 14.42		**O.K**

Design Strength of mortar (Mpa)	21	24	27	30	40	50	60
Round rebar	0.7	0.8	0.85	0.9	1	1	1
Deform rebar	1.4	1.6	1.7	1.8	2	2	2

10.5 Check Compression strength of concrete of superstructure

f'c = 40 (Mpa)

The dimension of top plate 1660 x 1360 = 2257600 (mm2)

The dimension of bearing area 1760 x 1460 = 2569600 (mm²)

The factor bearing resistance shall be taken as $P_r = \phi P_n$

$$\left.\begin{aligned} P_n &= 0.85 f'_c A_1 m \\ \phi &= 0.7 \\ m &= \sqrt{\frac{A_2}{A_1}} \end{aligned}\right\}$$

Pr = 0.7 x 0.85 x 40 x 2257600 x 1.06686
= 57323568 (N)

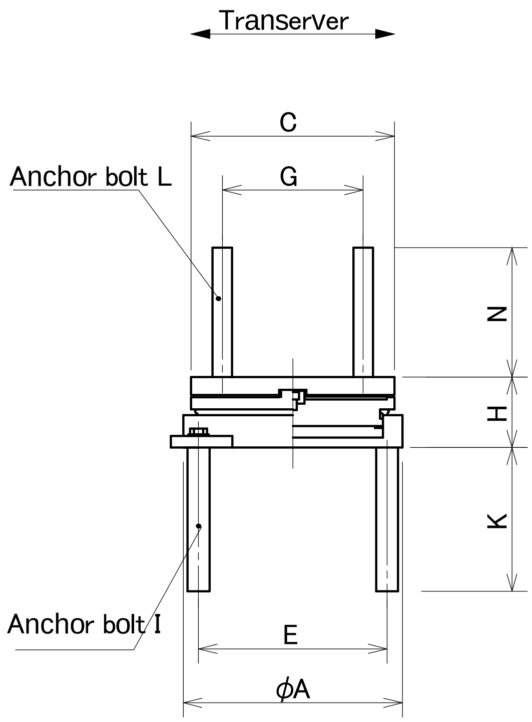
Check: Ultimate vertical load < Concrete resistance

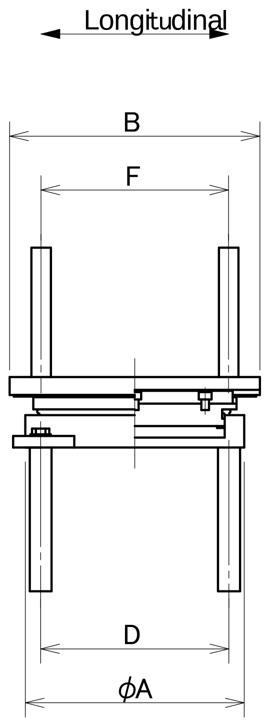
27000 (kN) < 57324 (kN) **O.K**

Stress at concrete surface:

σ = N / Area of bearing place = 27000 * 1000 / 2257600 = 12.0 Mpa

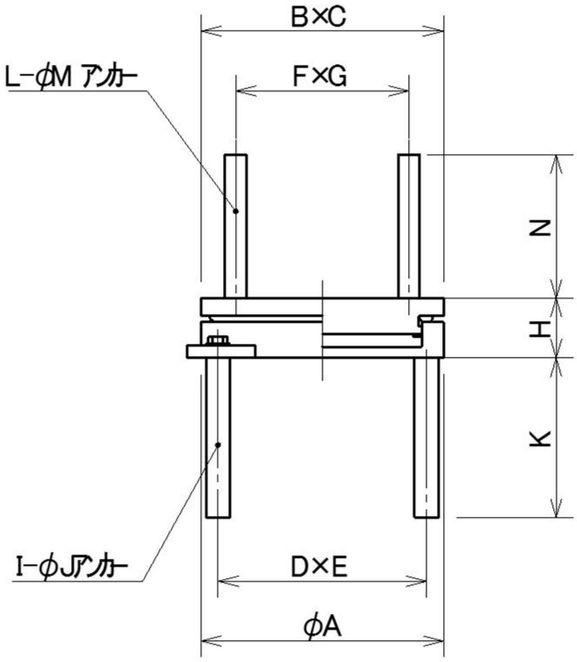
Check 11.96 < 20 Mpa **OK**



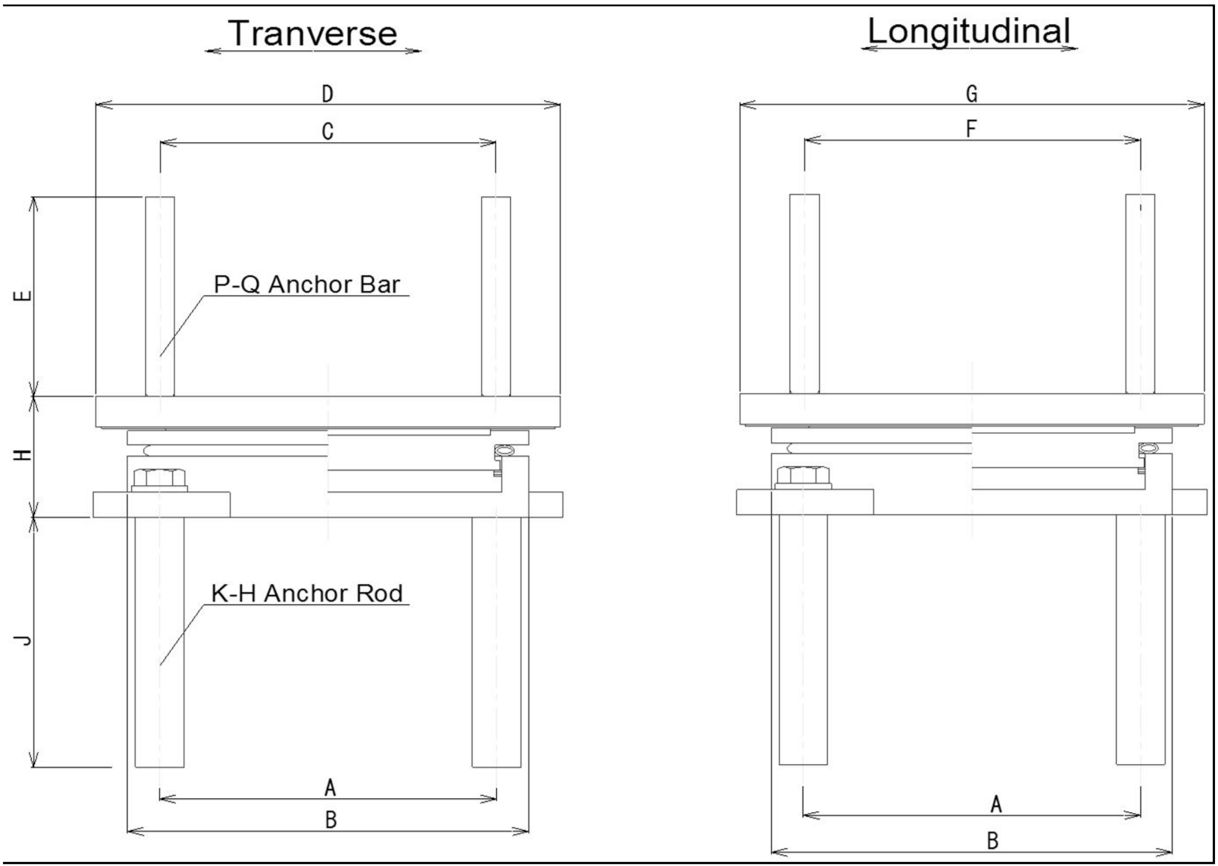


A	1320	mm
E	1100	mm
K	750	mm
H	331	mm
N	466	mm
I	4	
J	160	mm
G	950	mm
C	1130	mm
L	16	
M	48	mm

A	1320	mm
D	1100	mm
F	1350	mm
B	1530	mm



A	1320	mm
B	1320	mm
C	1320	mm
D	1100	mm
E	1100	mm
F	1140	mm
G	1140	mm
K	750	mm
H	305	mm
N	686	mm
L	16	
M	48	mm
I	4	
J	160	mm



A	1088	mm
B	1320	mm
C	1170	mm
D	1360	mm
E	466	mm
H	311	mm
J	650	mm
P	16	
Q	48	mm
K	4	
H	160	mm

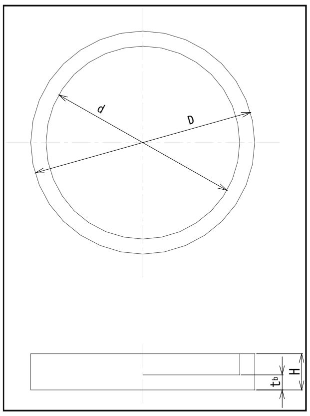
Rod

A	1088	mm
B	1320	mm
F	1470	mm
G	1660	mm

Information Only **WEIGHT CALCULATION**

1. DIMENSION TABLE

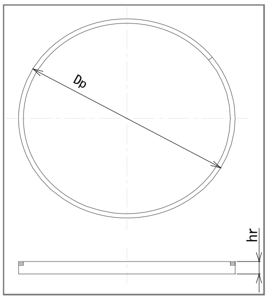
1.1 Pot



Diameter of inner pot	=	1150	mm
Diameter of extorior pot	=	1320	mm
Thickness of pot base	=	85	mm
Height of pot	=	225	mm

V1 = 162491026 mm3

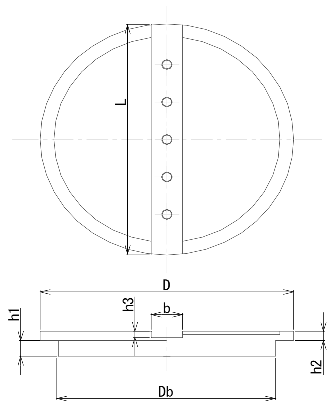
1.2 Elastomeric pad



Diameter of elastomeric Pad	=	1150	mm
Height of elastomeric Pad	=	90	mm

V2 = 93482016.4 mm3

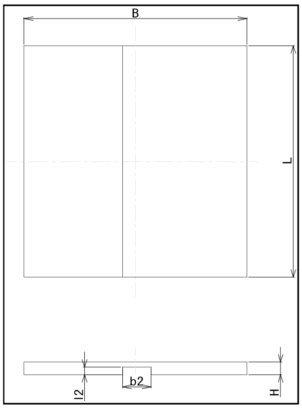
1.3 Piston



Diameter of exterior of piston	=	1150	mm
Thickness of piston	=	80	mm
Diameter of below backing plate	=	1320	mm
Thickness of below backing plate	=	20	mm
Width of guide key	=	100	mm
Depth of guide key	=	15	mm
Length of the guide key	=	1320	mm

V3 = 102601296.5 mm3

1.4 the above backing plate



the minor dimension of plate	=	1130	mm
The major dimension of plate	=	1530	mm
thickness of the plate	=	50	mm
Width of the guide key	=	100	mm
Thickness of the guide key	=	22	mm

V4 = 83079000 mm3

1.5 PTFE

Area of PTFE	=	840483.5	mm2
Thickness of PTFE	=	10	mm
V5	=	8404835	mm3

1.6 Anchor bar

Diameter of anchor bar =	48	mm	
Length of anchor bar =	490	mm	
V6 =	886683.1		mm3

1.7 Anchor Rod

Diameter of anchor rod =	160	mm	
Length of bolt in anchor rod =	80	mm	
Length of anchor rod =	750	mm	
Diameter of anchor bolt =	80	mm	
V7 =	14677521		mm3

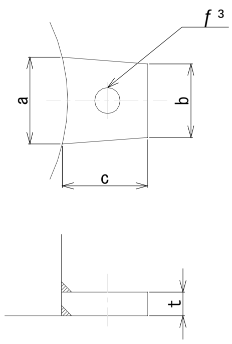
1.8 Hexagon Bolt Wash = 10.37 (kg) per piece

Washer Thickness 12

D = 80 L = 140

Col. Num 23 Row Num 24

1.9 Anchor Plate



a =	340	mm
b =	300	mm
t =	60	mm
c =	280	mm

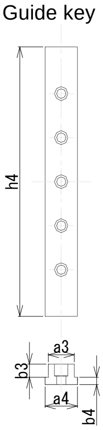
V9 = 5376000 mm3

1.10 Stainless Steel

Dimesion of stainless Steel

Transverse =	537	mm
Longitudinal =	1520	mm
Thickness =	3	mm
V10 =	2448720	mm3

1.11 Guide key



a4	=	100	mm
b3	=	27	mm
b4	=	15	mm
a3	=	88	mm
h4	=	1235	mm

V11 = 4786860 mm3

1.12 Guide PTFE

	h	L	t
Dimension	21	1100	6
V12 =		138600	mm3

1.13 Guide Bolt = 0.266 (kg) per piece

D =	24	L =	40
col	8	row	6
Nos =	11		

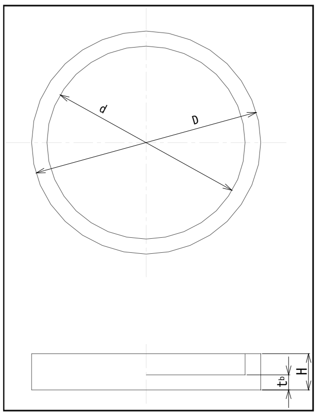
1.14 Sealing ring = 2.308599 (kg)

No	Part	Volume (mm3)	Quantity	Mass(kg)
1	POT	162491026	1	1275.6
2	Elastomeric	93482016	1	107.5
3	Piston	102601296.5	1	805.4
4	Middle Plate	83079000	1	652.2
5	PTFE	8404835	1	19.3
6	Anchor Bar	886683	16	111.4
7	Anchor Rod	14677521	4	460.9
8	Hexagon.Wash		4	41.5
9	Anchor plate	5376000	4	168.8
10	Stainless Plate	2448720	2	38.4
11	Guide Plate	4786860	1	34.7
12	PTFE of guide	138600	2	0.6
13	Hexagon bolt(Guide)		11	2.9
14	Seal ring		1	2.3
			Sum	3721.5

WEIGHT CALCULATION

1. DIMENSION TABLE

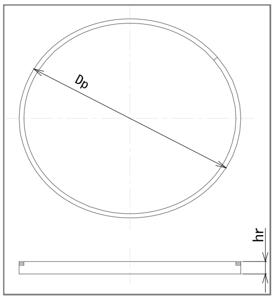
1.1 POT



Diameter of inner pot	=	1150	mm
Diameter of extorior pot	=	1320	mm
Thickness of pot base	=	85	mm
Height of pot	=	225	mm

V1 = 162491026 mm3

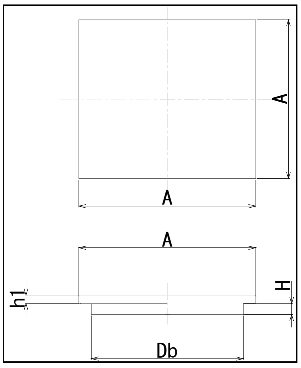
1.2 Elastomeric pad



Diameter of elastomeric Pad	=	1150	mm
Height of elastomeric Pad	=	90	mm

V2 = 93482016.4 mm3

1.3 Piston



A	=	1320	mm
Db	=	1150	mm
H	=	80	mm
h1	=	50	mm
V3	=	170215126	mm

1.4 Anchor Bar

Diameter of anchor bar	=	48	mm
Length of anchor bar	=	710	mm
V4	=	1284785.7	mm3

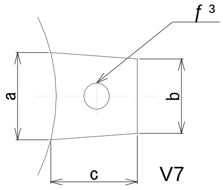
1.5 Anchor Rod

Diameter of anchor rod	=	160	mm
Length of bolt in anchor rod	=	80	mm
Length of anchor rod	=	750	mm
Diameter of anchor bolt	=	80	mm
V5	=	14677520.9	mm3

1.6 Hexagon Wash = 10.37 (kg) per piece

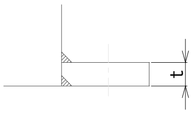
Washer	Thickness	12			
D =		80		L =	140
Col. Num		23		Row Num	24

1.7 Anchor Plate



a = 340 mm
b = 300 mm
t = 60 mm
c = 280 mm

V7 = 5376000 mm3



1.8 Sealing Ring = 2.308599165 kg

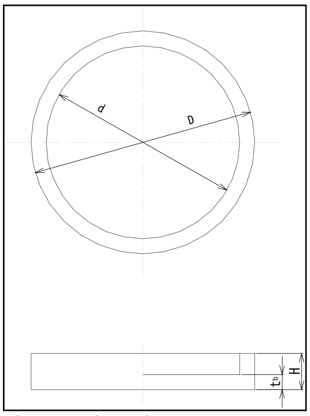
Fix

No	Parts	Volume(mm3)	Quantity	Mass(kg)
1	POT	162491026	1	1275.6
2	Elastomeric	93482016.4	1	107.5
3	Piston	170215125.7	1	1336.2
4	Anchor Bar	1284785.7	16	161.4
5	Anchor Rod	14677520.9	4	460.9
6	Hex. Wash		4	41.5
7	Anchor Pla	5376000.0	4	168.8
8	Seal Ring		1	2.3
			Sum =	3554.1

WEIGHT CALCULATION

1. DIMENSION TABLE

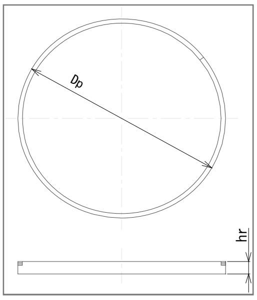
1.1 POT



Diameter of inner pot	=	1150	mm
Diameter of extorior pot	=	1320	mm
Thickness of pot base	=	85	mm
Height of pot	=	225	mm

V1 = 162491026 mm3

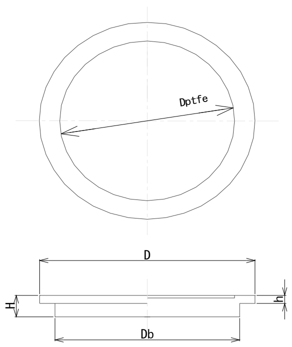
1.2 Elastomeric pad



Diameter of elastomeric Pad	=	1150	mm
Height of elastomeric Pad	=	90	mm

V2 = 93482016.4 mm3

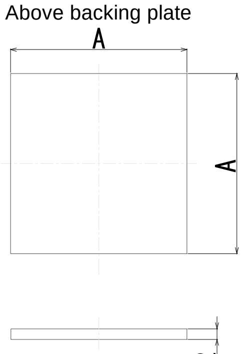
1.3 Piston



Db	=	1150
D	=	1150
Dptfe	=	1100
Thickness of ptfe in pot	=	7
h	=	0
H	=	80

V3 = 76442803.24 mm3

1.4 Above backing plate



A = 1660 x 1360
H2 = 50

V4 = 1.13E+08 mm3

1.5 PTFE

Area of ptfe = 950331.8 mm
Thickness of ptfe = 10 mm

V5 = 9503318 mm3

1.6 Anchor bar

Diameter of anchor bar = 48 mm
Length of anchor bar = 490 mm
V4 = 886683.1 mm3

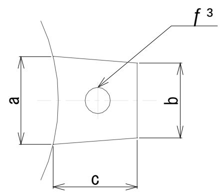
1.7 Anchor rod

Diameter of anchor rod =		160	mm	
Length of bolt in anchor rod =		60	mm	
Length of anchor rod =		650	mm	
Diameter of anchor bolt =		72	mm	
V7	=	12824735.2		mm3

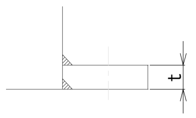
1.8 Hexagon Bolt Wash = 7.66 (kg) per piece

Washer	Thickness	10		
D =	72		L =	130
Col. Num	21		Row Num	23

1.9 Anchor plate



a =	320	mm	
b =	280	mm	
t =	60	mm	
c =	260	mm	
V9	=	4680000	mm3



1.10 Stainless plate

Dimension of Stainless plate

Longitudinal = 1650 mm

Transverse = 1350 mm

Thickness of stainless = 3 mm

V10 = 6682500 mm3

1.11 Sealing ring = 2.292567 (kg)

No	Part	Volume (mm3)	Quantity	Mass(kg)
1	POT	162491026	1	1275.6
2	Elastomeric	93482016	1	107.5
3	Piston	76442803	1	600.1
4	Middle Plate	112880000	1	886.1
5	PTFE	9503318	1	21.9
6	Anchor Bar	886683	16	111.4
7	Anchor Rod	12824735	4	402.7
8	Hexagon.Wash		4	30.6
9	Anchor plate	4680000	4	147.0
10	Stainless Plate	6682500	1	52.5
11	Seal ring		1	2.3
			Sum =	3637.5

Calculation of Anchor Bar For Box Girder

A1 & Abutment and P5 & P10 Anchorbolt

Device for Limiting Longitudinal Movement of Girder
Design Force is 1.5*Rd*Cm

				Symbol	Unit	P10 B	P11	P12	P15	P16	A2 E
Design Condition	Support Condition	Longitudinal		-	-	Fix	Move	Move	Move	Move	Move
		Transversal		-	-	Fix	Fix	Fix	Fix	Fix	Fix
	Rreaction of Dead Load	Self Weight of Superstructure		Dc	kN	4850					4850
		Saface Load		Dw	kN	410					410
		Total		SumD	kN	5260					5260
	Movement	Longitudinal	Creep +Shrinkage	dL(Cr+sh)	mm	0					0
			Temperature(+)	dL(T+)	mm	0					0
			Temperature(-)	dL(T-)	mm	0					0
			Braking Force	dL(BE)	mm	0					0
			Earthquake Effect	dl(EqL)	mm	0					0
	Sizmic Coefficient and Horizontal Force	Seizmic Coefficient(1.5*A)		Cm	-	0.05					0.05
		Horizontal Force		Hd	-	273.52					273.52
	Load Factor — Horizontal Force	Extreme Event		γeq'	-	1.5					1.5
	Load Factor — Movement	Strength	Creep +Shrinkage	γCr,Sh	-	1.2					1.2
			Temperature	γTE	-	1.2					1.2
			Braking Force	γBR	-	1.75					1.75
		Extreme Event	Earthquake Effect	γEq	-	1.0					1.0
	Factored Movement	Longitudinal	Strength	dLst	mm	0					0
			Extreme Event	dLex	mm	0					0
	Design Horizontal Force	Extreme Event		Heq	kN	650.0	1860.0	2430.0	1970.0	2460.0	410.3
	Concrete	Compressive Strength at 28 days		f'c	Mpa	30.0	30.0	30.0	30.0	30.0	30.0
Material	Steel	Anchor Bar	Type	-	-	C300-T	C300-T	C300-T	C300-T	C300-T	C300-T
			Yield Tensil Strength	fry	Mpa	300	300	300	300	300	300
			Yield Shear Strength	τry	Mpa	170	170	170	170	170	170
	Resistance Factor	Concrete	Strength	φstc	-	0.7	0.7	0.7	0.7	0.7	0.7
			Extreme Event	φexr	-	1.0	1.0	1.0	1.0	1.0	1.0
		Rebar	Extreme Event	φexr	-	1.0	1.0	1.0	1.0	1.0	1.0
Calculation Result	Required Anchor Bar Area	at Extreme		reqAr1	mm2	3,824	10,941	14,294	11,588	14,471	2,413
	Verification of Anchore Bar	Arrengement Anchor Bar	Nuber of Anchor Bar	Nb	nos	4	3	3	3	3	4
			Required Dimaiter	reqDs	mm	34.9	68.2	77.9	70.2	78.4	27.8
		Extreme Event	Diamiter	Ds	nos	**40**	**80**	**80**	**80**	**80**	**40**
			Area of Rebar	Ar	mm2	5,027	15,080	15,080	15,080	15,080	5,027
			Nomonal Rejistance	Hrn	kN	854.5	2563.5	2563.5	2563.5	2563.5	854.5
			Resistance Factor	φstc	-	1.00	1.00	1.00	1.00	1.00	1.00
			Factored Resistance	Hur	kN	855	2564	2564	2564	2564	855
			Active Force	Hu	kN	650.0	1860.0	2430.0	1970.0	2460.0	410.3
			Safety Factor	Fa	-	0.761	0.726	0.948	0.768	0.960	0.480
			Juadge	-	-	OK	OK	OK	OK	OK	OK

1.2 PC SUPER T GIRDER
1.2 KẾT CẤU DẦM SUPER T

CONTENT

I. DATA

1. General Information

• Total of length	$L =$	38.30	m
• Effective span length	$L_s =$	37.60	m
• Total width of deck	$B =$	13.00	m
• Width of wearing surface on the deck	$W =$	11.50	m
• Height of girder	$H =$	1750.00	mm
• Distance between center of girders	$S =$	2120.00	mm
• Average thickness of C.I.P slab	$h_s =$	196	mm
• Thickness of wearing surface	$h_w =$	84.00	mm

2. Material

2.1.Concrete

• Unit weight of asphant concrete, or wearing concrete	$\gamma_c =$	22.50	kN/m^3
• Unit weight of reinforcement	$\gamma_c =$	24.50	kN/m^3

a. Slab Concrete

• Concrete strength at 28 days	$f'_{cs} =$	35.00	Mpa
• Stress block factor	$\beta =$	0.80	
• Elastic modulus of concrete at 28 days	$E_{cs} =$	31750.32	Mpa

b. Girder concrete

• Compressive strength of concrete at 28 days	$f'_c =$	50.00	Mpa
• Tensile strength of concrete at 28 days	$f_r =$	4.45	Mpa
• Elastic modulus of concrete at 28 days	$E_c =$	37948.89	Mpa
• Ratio of elastic modulus between slab and girder	$n =$	0.84	Mpa
• Compressive strength of concrete at jack release	$f'_{ci} \geq 0.80f'_c =$	40.00	Mpa
• Modulus of elasticity of concrete at jack release	$E_c =$	33942.52	Mpa

2.1.1.Stress Limit

• Compressive stress before all losses	$f'_c = 0.60f'_{ci} =$	24.00	Mpa
• Tensile stress before all losses	$f'_{ct} = 0.5f'_{ci}{}^{0.5} =$	3.16	Mpa
• Compressive stress at service limit state	$f'_c = 0.45f'_c =$	22.50	Mpa
• Tensile stress at service limit state	$f'_{ct} = 0.5f'_c{}^{0.5} =$	3.54	Mpa
• Tensile stress at service limit state (debonding)	$f'_c =$	0.00	Mpa

2.1.2.Poission ratio and modulus of rupture

• Poission ratio	$n =$	0.25	Mpa
• Modulus of rupture	$G_c = E/(2.(n+1)) =$	15179.56	Mpa

2.2.Strand

• Strand 15.2 mm, low relaxation strand which complies with : ASTM A416, Grade 270

• Cross section area of strand		$A_{tps} =$	140.00	mm^2
• Unit weight		$W_{ps} =$	1.10	kg/m
• Tensile Strength		$f_{pu} =$	1860.00	Mpa
• Yield Strength		$f_{py} = 0.9f_{pu} =$	1674.00	Mpa
• Stress Limit before all of losses	$f_{pi} \leq 0.75\ f_{pu}$,	$f_{pi} \leq$	1395.00	Mpa
• Stress Limit at service limit state	$f_{pe} \leq 0.80\ f_{py}$,	$f_{pe} \leq$	1339.20	Mpa
• Modulus of elasticity of strand		$E_{ps} =$	197000.00	Mpa

2.3.Reinforcment

• Complying with TCVN

• Yield strength	$f_y =$	400.00	Mpa
• Modulus of elasticity	$E_s =$	200000.00	Mpa

Exterior Girder

Tendon	x	y	L_{total} (m)	L_{debond} (m)	L_{bond} (m)	A_{ps} (mm^2)	W_{ten}(kN/m)	W (kN)
1	-250	75	37.5	8	29.5	140.00	1.10	0.42
2	-200	75	37.5	0	37.5	140.00	1.10	0.42
3	-150	75	37.5	0	37.5	140.00	1.10	0.42
4	-100	75	37.5	4	33.5	140.00	1.10	0.42
5	-50	75	37.5	0	37.5	140.00	1.10	0.42
6	0	75	37.5	5	32.5	140.00	1.10	0.42
7	50	75	37.5	0	37.5	140.00	1.10	0.42
8	100	75	37.5	4	33.5	140.00	1.10	0.42
9	150	75	37.5	0	37.5	140.00	1.10	0.42
10	200	75	37.5	0	37.5	140.00	1.10	0.42
11	250	75	37.5	8	29.5	140.00	1.10	0.42
12	-250	125	37.5	6	31.5	140.00	1.10	0.42
13	-200	125	37.5	0	37.5	140.00	1.10	0.42
14	-150	125	37.5	4	33.5	140.00	1.10	0.42
15	-100	125	37.5	0	37.5	140.00	1.10	0.42
16	-50	125	37.5	0	37.5	140.00	1.10	0.42
17	0	125	37.5	0	37.5	140.00	1.10	0.42
18	50	125	37.5	0	37.5	140.00	1.10	0.42
19	100	125	37.5	0	37.5	140.00	1.10	0.42
20	150	125	37.5	4	33.5	140.00	1.10	0.42
21	200	125	37.5	0	37.5	140.00	1.10	0.42
22	250	125	37.5	6	31.5	140.00	1.10	0.42
23	-300	175	37.5	6	31.5	140.00	1.10	0.42
24	-250	175	37.5	0	37.5	140.00	1.10	0.42
25	-200	175	37.5	2	35.5	140.00	1.10	0.42
26	-150	175	37.5	0	37.5	140.00	1.10	0.42
27	-100	175	37.5	2	35.5	140.00	1.10	0.42
28	-50	175	37.5	0	37.5	140.00	1.10	0.42
29	50	175	37.5	0	37.5	140.00	1.10	0.42
30	100	175	37.5	2	35.5	140.00	1.10	0.42
31	150	175	37.5	0	37.5	140.00	1.10	0.42
32	200	175	37.5	2	35.5	140.00	1.10	0.42
33	250	175	37.5	0	37.5	140.00	1.10	0.42
34	300	175	37.5	6	31.5	140.00	1.10	0.42
35	-300	225	37.5	8	29.5	140.00	1.10	0.42
36	-250	225	37.5	0	37.5	140.00	1.10	0.42
37	-200	225	37.5	0	37.5	140.00	1.10	0.42
38	-150	225	37.5	0	37.5	140.00	1.10	0.42
39	150	225	37.5	0	37.5	140.00	1.10	0.42
40	200	225	37.5	0	37.5	140.00	1.10	0.42
41	250	225	37.5	0	37.5	140.00	1.10	0.42
42	300	225	37.5	8	29.5	140.00	1.10	0.42
43	-520	1690	38.3	1	37.3	140.00	1.10	0.43
44	520	1690	38.3	1	37.3	140.00	1.10	0.43
Total			**1651.6**	**87**	**1564.6**	**6160.00**		**18.553142**

Eccentricity of prestress tendon from bottom girder y_b = 145.24 mm

Eccentricity of prestress tendon from top girder y_t = 60 mm

Interior Girder

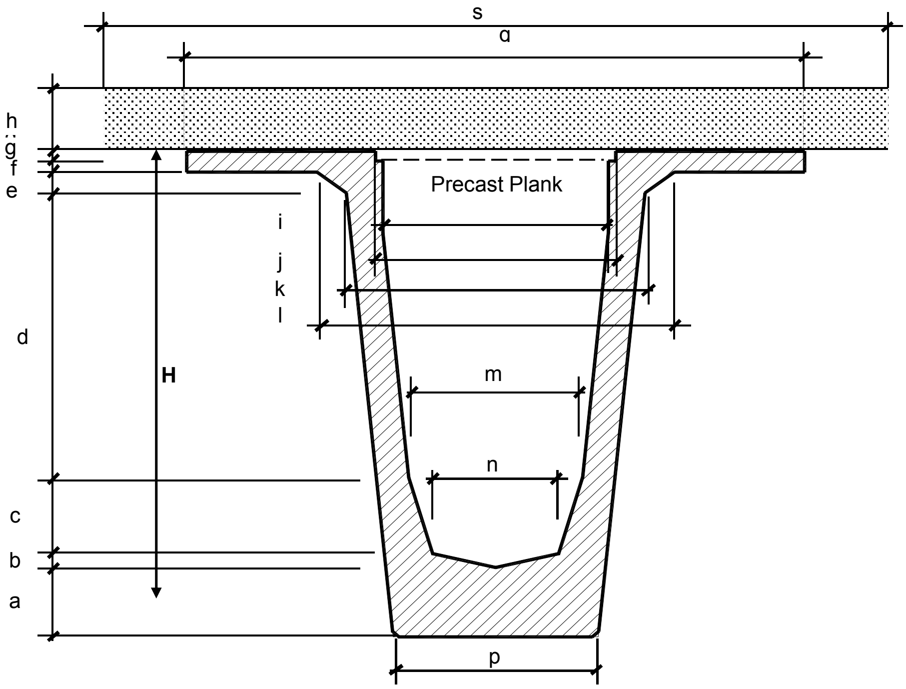
Tendon	x	y	L_{total} (m)	L_{debond} (m)	L_{bond} (m)	A_{ps} (mm^2)	W_{ten}(kN/m)	W (kN)
1	-250	75	37.5	8	29.5	140.00	1.10	0.42
2	-200	75	37.5	0	37.5	140.00	1.10	0.42
3	-150	75	37.5	0	37.5	140.00	1.10	0.42
4	-100	75	37.5	0	37.5	140.00	1.10	0.42
5	-50	75	37.5	4	33.5	140.00	1.10	0.42
6	0	75	37.5	0	37.5	140.00	1.10	0.42
7	50	75	37.5	4	33.5	140.00	1.10	0.42
8	100	75	37.5	0	37.5	140.00	1.10	0.42
9	150	75	37.5	0	37.5	140.00	1.10	0.42
10	200	75	37.5	0	37.5	140.00	1.10	0.42
11	250	75	37.5	8	29.5	140.00	1.10	0.42
12	-250	125	37.5	6	31.5	140.00	1.10	0.42
13	-200	125	37.5	0	37.5	140.00	1.10	0.42
14	-150	125	37.5	4	33.5	140.00	1.10	0.42
15	-100	125	37.5	0	37.5	140.00	1.10	0.42
16	-50	125	37.5	0	37.5	140.00	1.10	0.42
17	0	125	37.5	0	37.5	140.00	1.10	0.42
18	50	125	37.5	0	37.5	140.00	1.10	0.42
19	100	125	37.5	0	37.5	140.00	1.10	0.42
20	150	125	37.5	4	33.5	140.00	1.10	0.42
21	200	125	37.5	0	37.5	140.00	1.10	0.42
22	250	125	37.5	6	31.5	140.00	1.10	0.42
23	-300	175	37.5	6	31.5	140.00	1.10	0.42
24	-250	175	37.5	0	37.5	140.00	1.10	0.42
25	-200	175	37.5	2	35.5	140.00	1.10	0.42
26	-150	175	37.5	0	37.5	140.00	1.10	0.42
27	-100	175	37.5	2	35.5	140.00	1.10	0.42
28	-50	175	37.5	0	37.5	140.00	1.10	0.42
29	50	175	37.5	0	37.5	140.00	1.10	0.42
30	100	175	37.5	2	35.5	140.00	1.10	0.42
31	150	175	37.5	0	37.5	140.00	1.10	0.42
32	200	175	37.5	2	35.5	140.00	1.10	0.42
33	250	175	37.5	0	37.5	140.00	1.10	0.42
34	300	175	37.5	6	31.5	140.00	1.10	0.42
35	-300	225	37.5	8	29.5	140.00	1.10	0.42
36	-250	225	37.5	0	37.5	140.00	1.10	0.42
37	-200	225	37.5	0	37.5	140.00	1.10	0.42
38	200	225	37.5	0	37.5	140.00	1.10	0.42
39	250	225	37.5	0	37.5	140.00	1.10	0.42
40	300	225	37.5	8	29.5	140.00	1.10	0.42
41	-520	1690	38.3	1	37.3	140.00	1.10	0.43
42	520	1690	38.3	1	37.3	140.00	1.10	0.43
Total			**1576.6**	**82**	**1494.6**	**5880.00**		**17.710634**

Eccentricity of prestress tendon from bottom girder y_b = 141.25 mm

Eccentricity of prestress tendon from top girder y_t = 60.00 mm

II. GEOMETRIC PROPERTIES

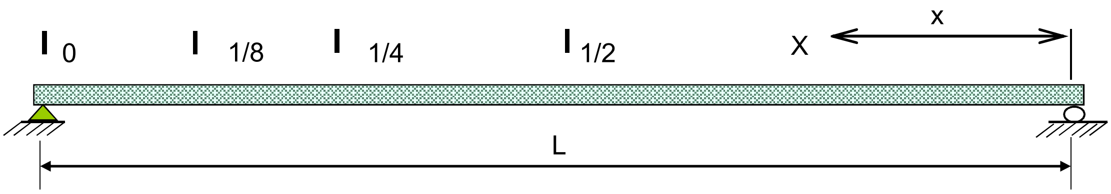
1. Section Properties



Dimention (mm)	
a	250
b	50
c	275
d	1025
e	75
f	40
g	35
h	196
i	770
j	820
k	1020
l	1220
m	594
n	430
p	700
q	2110
H	**1750**

2. Property and position of sections

• Position of check section



2.1.Properties of girder

Section	0	1/8L	1/4L	1/2L	X=2.0 m	Unit
A	1.618	0.635	0.635	0.635	1.618	m^2
I_x	0.435	0.241	0.241	0.241	0.435	m^4
I_y	0.155	0.117	0.117	0.117	0.155	m^4
I_{xy}	0.008	0.000	0.000	0.000	0.008	m^4
y_b	0.975	0.823	0.823	0.823	0.975	m
y_t	0.775	0.927	0.927	0.927	0.775	m
S_b	0.446	0.293	0.293	0.293	0.446	m^3
S_t	0.561	0.260	0.260	0.260	0.561	m^3

2.2.Properties of composite section

2.2.1. Effective flange width (A.4.6.2.6.)

• Effective span L_s = 37.60 m
• Thickness of web h_{sb} = 250.00 mm
• Width of flange side b_{st} = 1055.00 mm
• Average thickness of deck h_s = 196.20 mm
• Average spacing of girders S = 2120.00 mm
• *Effective flange width of interior girder* b_{ef} = 2120.00 mm
• *Effective flange width of exterior girder* b_{ef} = 2120.00 mm

Consided different of elastic modulus between desk slab and girder

• *Effective flange width of interior girder* $n.b_{ef}$ = 1773.72 mm
• *Effective flange width of exterior girder* $n.b_{ef}$ = 1773.72 mm

2.2.2.Geometric properties of interior girder

Section	0	1/8L	1/4L	1/2L	X=2.0 m	Unit
A	1.966	0.983	0.983	0.983	1.966	m^2
I_{xc}	0.618	0.413	0.413	0.413	0.618	m^4
I_{yc}	0.247	0.208	0.208	0.208	0.247	m^4
I_{xy}	0.028	0.009	0.009	0.009	0.028	m^4
y_b	1.130	1.186	1.186	1.186	1.130	m
y_t	0.816	0.760	0.760	0.760	0.816	m
S_{bc}	0.547	0.348	0.348	0.348	0.547	m^3
S_{tc}	0.757	0.543	0.543	0.543	0.757	m^3

2.2.3.Geometric properties of exterior girder

Section	0	1/8L	1/4L	1/2L	X=2.0 m	Unit
A	1.966	0.983	0.983	0.983	1.966	m^2
I_{xc}	0.618	0.413	0.413	0.413	0.618	m^4
I_{yc}	0.247	0.208	0.208	0.208	0.247	m^4
I_{xyc}	0.008	0.009	0.009	0.009	0.008	m^4
y_{bc}	1.130	1.186	1.186	1.186	1.130	m
y_{tc}	0.816	0.760	0.760	0.760	0.816	m
S_{bc}	0.547	0.348	0.348	0.348	0.547	m^3
S_{tc}	0.757	0.543	0.543	0.543	0.757	m^3

III. DISTRIBUTION FACTOR OF LIVELOAD (A. 4.6.2.2.)

1.Distribution factor of moment

• Spacing of girders	S =	2120.00	mm
• Depth of girder	d =	1750.00	mm
• Effective length of span	L =	37600.00	mm
• Number of girder	N_b =	6	girders

1.1. Interior girder

Strength limit state, service limit state

1.1.1. For one design lane loaded

$\mathbf{Gi = (S / 910)^{0.35} (S.d / L^2)^{0.25}}$	Gi =	0.304

1.1.2. Two or more design lanes loaded

$\mathbf{Gi = (S / 1900)^{0.6} (S.d / L^2)^{0.125}}$	Gi =	0.508
• Conclusion : (use larger value of 1.1.1 & 1.1.2)	**Gi =**	**0.508**

For fatigue limit state

$\mathbf{G_F = Gi / 1.2}$	G_F =	0.254

1.2. Exterior girder

1.2.1. For one design lane loaded (lever rule)

Ge= 1.2(0.545+(S -1800)/2S)	Gi =	0.745	

1.2.2. Two or more design lanes loaded

• Distance from the ex-web of ex-girder to in-edge of curb	de =	1002.00	mm
Ge = e.Gi = (0.97 + de/8700).Gi	Ge =	0.551	
• Conclusion : (use larger value of 1.2.1 & 1.2.2)	**Gi =**	**0.745**	

2.Distribution factor of shear force

2.1. Interior girder

Strength limit state, service limit state

2.1.1. For one design lane loaded

$\mathbf{Gi = (S / 3050)^{0.6} (d / L)^{0.1}}$	Gi =	0.592

2.1.2. Two or more design lanes loaded

$\mathbf{Gi = (S / 2250)^{0.8} (d / L)^{0.1}}$	Gi =	0.702
• Conclusion : (use larger value of 2.1.1 & 2.1.2)	**Gi =**	**0.702**

For fatigue limit state

$\mathbf{G_F = Gi / 1.2}$	G_F =	0.493

2.2. Exterior girder

2.2.1. For one design lane loaded (lever rule)

Ge= 1.2(0.545+(S -1800)/2S)	Gi =	0.745

2.2.2. Two or more design lanes loaded

Ge = e.Gi = (0.8 + de/3050).Gi	Ge =	0.792
• Conclusion : (use larger value of 2.2.1 & 2.2.2)	**Gi =**	**0.792**

IV. LOADS AND ACTIONS

1. General concept

- All limit states shall be considered of equal

$$\eta\sum(\gamma_i.Q_i) \le \varphi.R_n = R_r$$

- In which

$$\eta = \eta_D.\eta_R.\eta_I \ge 0.95$$

State	Strength	Extreme	Service	Fatigue
η_D	1.00	1.00	1.00	1.00
η_R	1.00	1.00	1.00	1.00
η_I	1.00	1.05	1.00	1.00
η	1.00	1.05	1.00	1.00

- There are three loaded stages of Super-T girder
- Stage 1: Prestress and self weight
- Stage 2: Self weight , Prestress, Live load and other action (wearing,etc …)
- Stage 3: All action of stage 2 and temperature action

2. Moment and shear force (due to DC, DW)

- Moment and shear force due to Self weight of girder are calculated in appendix 1 *(on page 9)*
- Self weight of stay-in-placed concrete formwork *(plank)* W_{sg}= 0.70 kN/m
- Permanent Weight of deck and weight on the deck are taked as distribution of each girder

if it satifies follow requirements

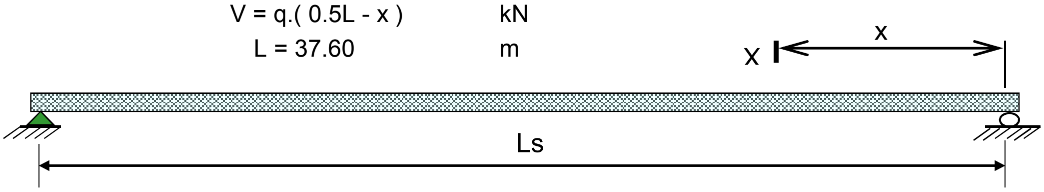
The width of deck is constant			OK
Number of girder	N_b=	6	OK
Girder are paralle and approximate rigid			OK
The width of lane in overhang part (mm)	de =	518	OK
Curvature in horizontal plan	α =	0	OK
Type of transverse section	Loại =	c	OK

- Weight of deck W_s= 10.20 kN/m
- Weight of wearing surface W_w= 3.62 kN/m
- Weight of parapet W_{rp1}= 13.48 kN/m
- Weight of median parapet W_{rp2}= 11.03 kN/m
- Weight of parapet and other Item act on this girder W_{rp}= 4.08 kN/m
- Caculated Equation

$M = 0.5q.x.(L - x)$ kNm

$V = q.(0.5L - x)$ kN

$L = 37.60$ m



Section	DC				DW		(DC + DW)	
x (m)	Girder		Slab		Wearing+Parapet		Total	
	V (kN)	M (kNm)	V (kN)	M (kNm)	V (kN)	M (kNm)	V (kN)	M (kNm)
0.00	356.43	0.00	191.73	0.00	144.87	0.00	693.04	0.00
2.00	212.40	632.94	171.34	363.07	129.46	274.33	513.20	1270.34
3.00	196.13	906.57	161.14	529.31	121.75	399.93	479.02	1835.81
4.00	179.86	1163.94	150.94	685.35	114.05	517.83	444.85	2367.12
5.00	163.59	1405.03	140.74	831.19	106.34	628.03	410.67	2864.24
6.00	147.32	1629.85	130.54	966.83	98.63	730.51	376.50	3327.20
8.00	114.78	2030.69	110.15	1207.52	83.22	912.37	308.15	4150.58
9.40	83.58	2294.03	95.87	1351.73	72.43	1021.33	251.88	4667.09
18.80	2.11	3012.86	0.00	1802.30	0.00	1361.77	2.11	6176.94

3. Moment and shear force (due to LL, IM, PL)

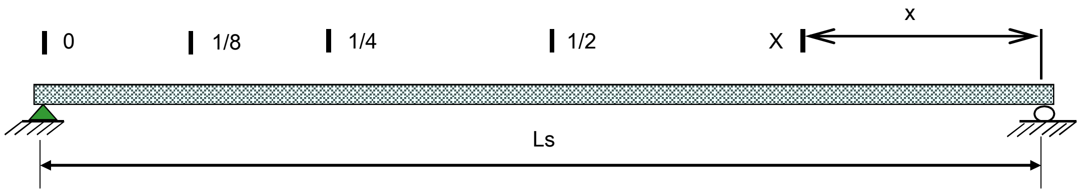
3.1.Dynamic load allowance IM

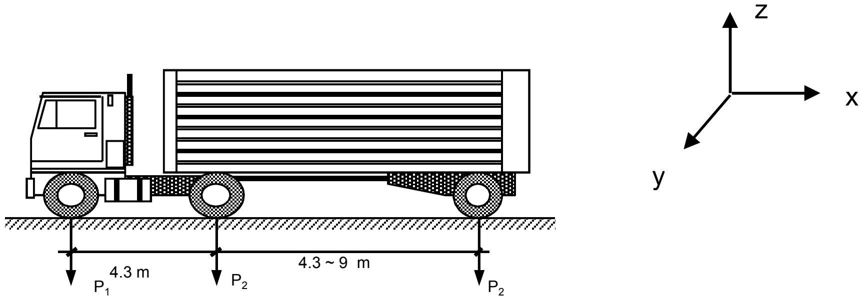
- Joint and slab — IM = 75.00 %
- Other part
  - Fatigue and Fracture limit state — IM = 15.00 %
  - Other states — IM = 25.00 %

M = 0.5q.x.(L - x) kNm

V = q.(0.5L - x) kN

L = 37.60 m





- In the below table, caculation for one design lane loaded, is not composed of distribution factor
- With HL-93 : Vz =(145.(L-x)+ 145.(L-x-4.3) + 35(L-x-8.6))/L
- With lane load : Vz =9.3(L-x)^2/(2L)
- If x ≥ 4.3 m : My =(145.(L-x)x + 145.(L-x-4.3).x + 35(x-4.3).(L-x))/L
- if x < 4.3 m : My =(145.(L-x)x + 145.(L-x-4.3).x + 35(L-x-8.6).x)/L
- In fatigue My =(145.(L-x)x + 145.(L-x-9).x + 35(x-4.3).(L-x))/L

Section	HL93 + IM				Lane Load		Fatigue	
x (m)	HL93		LT=HL93+IM		LL		LT	LT + IM
	Vz (kN)	My (kNm)	Vz (kN)	My (kNm)	Vz (kN)	My(kNm)	My (kNm)	My (kNm)
0.00	300.41	0.00	375.52	0.00	174.84	0.00		0.00
2.00	283.13	566.25	353.91	707.81	156.73	331.08	403.52	464.04
3.00	274.48	823.44	343.10	1029.31	148.05	482.67	654.59	752.78
4.00	265.84	1063.35	332.30	1329.19	139.62	624.96	888.38	1021.64
5.00	257.19	1195.51	321.49	1494.39	131.43	757.95	1104.89	1270.62
6.00	248.55	1412.85	310.69	1766.06	123.49	881.64	1304.10	1499.72
8.00	231.26	1795.67	289.08	2244.59	108.35	1101.12	1650.67	1898.27
9.40	219.16	2022.50	273.95	2528.13	98.35	1232.62	1852.13	2129.94
18.80	137.91	2668.00	172.39	3335.00	43.71	1643.50	2327.25	2676.34

V. LOAD COMBINATIONS (A.3.4)

1. General concept

• Load combinations are taked as:

$Q = \eta\sum (\gamma_i.Q_i)$

• In which

η - Load modifier; a factor relating to ductivity, redundancy, and operational importance

γ_i - Load factor

2. Load combinations

• **Service I** : For Checking of compressive stress in prestressed concrete components under service limit state

Q =1.00(DC + DW) + 1.00(LL + IM)

• **Service III** : For Checking of tensile stress and crack control in prestressed concrete components under service limit state

Q =1.00(DC + DW) + 0.80(LL + IM)

• **Strength I** : For checking of resistance and stability of components under strength limit state

Q =0.90(DC) + 0.65(DW) + 1.75(LL + IM)

• **Fratigue** : For checking of stress due to liveload and impact under fatigue limit state

Q =0.75(LL+IM)

VI. CHECK EXTERIOR GIRDER

1. Loss of prestress (A.5.9.5.)

• Total final loss

$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR2}$ Mpa

• Where

Δf_{pES} - Loss due to elastic shortening

Δf_{pSR} - Loss due to shrinkage

Δf_{pCR} - Loss due to creep of concrete

Δf_{pR2} - loss due to relaxation of steel after transfer

1.1.Loss due to elastic shortening (A.5.9.5.2.3.)

$\Delta f_{pES} = E_p / E_{ci} \cdot f_{cpg}$ Mpa

• Where

f_{cpg} - Sum of concrete stresses at the center of gravity of prestressing tendons due to the prestressing force at transfer and the self-weight of the member at the sections of maximum moment

f_{cgp} - be assumed to be $0.75 f_{pu}$ for low-relaxation strands	$0.75 f_{pu}$ =	1395	Mpa
pretension force after allowing for the initial losses	P =	195.3	kN
Assuming loss of stress after release	Loss =	0.929	%
• concrete stresses at the center of gravity of the prestressing steel	f_{pi} =	1382.045	Mpa

$f_{cpg} = P_i / A + P_i.e_c^2 / I - M_g.e_c / I$

• Number of tendon	n =	42	Stranc
• pretension force after allowing for the initial losses	P_{pi} =	8126.425	kN
• Moment due to self weight of girder	M_g =	3012.8594	kNm
• Average eccentricity of tendon	e_c =	0.678	m
• Area of girder section	A =	0.635	m^2
• Moment of inertia of girder	I_x =	0.241	m^4
• f_{cpg} - Sum of concrete stresses at the center of gravity of tendons	f_{cpg} =	19.80	Mpa
• Modulus of elasticity of concrete at jack release	E_{ci} =	33942.52	Mpa
• Modulus of elasticity of strand	E_p =	197000.00	Mpa
	Δf_{pES} =	**114.94**	**Mpa**

1.2.Loss due to shrinkage (A 5.9.5.4.2.)

$\Delta f_{pSR} = 117 - 1.03H$ Mpa

• the average annual ambient relative humidity	H =	80.000	%
	Δf_{pSR} =	**34.6**	**Mpa**

1.3.Loss due to creep of concrete (A 5.9.5.4.3.)

$\Delta f_{pCR} = 12 f_{cgp} - 7 \Delta f_{cdp} \geq 0$ Mpa

• Where

Δf_{cdp} - change of stresses at the center of gravity of the prestressing steel due to permanent loads except the dead load present at the time the prestress force is applied

• Moment due to weight of slab	M_s=	1802.30	kNm
• Moment due to weight of wearing and parapet	M_{w+u} =	1361.77	kNm
• Average eccentricity of prestress tendon	e_c =	1.041	m
• Moment of inertia of composite section	I_x =	0.4127	m^4
	Δf_{cdp} =	8.50	Mpa
	Δf_{pCR} =	**178.17**	**Mpa**

1.4.Loss due to relaxation of steel after transfer (A 5.9.5.4.4c.)

$\Delta f_{pR2} = 30\%(138 - 0.4 \Delta f_{pES} + 0.2(\Delta f_{pSR} + \Delta f_{pCR}))$

• loss due to relaxation after transfer	**Δf_{pR2} =**	**14.84**	**Mpa**
• Initial loss	**$\Delta f_{initial}$ =**	**12.95**	**Mpa**

Sum of loss stress

$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR2}$	**Δf_{pT} =**	**342.56**	**Mpa**
Initial Prestress loss	Loss =	0.929	%

<u>Conclusion</u>

Different from assume and cacualtion of initial loss		Δ =	0.000 **OK**
• Effective initial prestress		**f_{pi} =**	**1382.05 Mpa**
Check:	f_{pi} = $1382.05 \le 1395.00$		**OK**
• Effective pretension force after allowing for the initial losses		**P_{pi} =**	**8126.43 kN**
• Effective prestress at jack release			**1267.10 Mpa**
• Effective final prestress after all of losses		**f_{pe} =**	**1039.49 Mpa**
Check:	f_{pi} = $1039.49 \le 1339.20$		**OK**
• Total prestressing force after all losses		**P_{pe} =**	**6112.19 kN**

2. Check stress at jack release

- For top fiber

$f_t = P_i/A - P_i.e/S_t + M_g/S_t + f_{top}$

$f_{top} = p/A + p.e_t / S_t$; p - prestress after initial loss of 2 tendons on top fiber

x	n	A	P_i	e	St	Mg	f_{top}	f_t	Check
(m)	strands	m^2	kN	m	m^3	kNm	Mpa	Mpa	
2.00	25.00	1.62	4434.86	0.826	0.561	632.94	0.239	-2.423	**OK**
3.00	29.00	0.64	5144.44	0.671	0.260	906.57	0.609	-1.060	**OK**
4.00	29.00	0.64	5144.44	0.671	0.260	1163.94	0.609	-0.072	**OK**
5.00	33.00	0.64	5854.01	0.677	0.260	1405.03	0.609	0.000	**OK**
6.00	34.00	0.64	6031.41	0.679	0.260	1629.85	0.609	0.633	**OK**
8.00	38.00	0.64	6740.98	0.679	0.260	2030.69	0.609	1.454	**OK**
9.40	42.00	0.64	7450.56	0.678	0.260	2294.03	0.609	1.748	**OK**
18.80	42.00	0.64	7450.56	0.678	0.260	3012.86	0.609	4.507	**OK**

e_t : Eccentricity of tendons on top with neutral axis, e : Eccentricity of tendons on bottom with neutral axis

- For bottom fiber

$f_b = P_i/A + P_i.e/S_b - M_g/S_b - f_{btop}$

$f_{btop} = p/A + p.e_t / S_b$; p - prestress after initial loss of 2 tendons on top fiber

x	n	A	P_i	e	S_b	M_g	f_{btop}	f_b	Check
(m)	strands	m^2	kN	m	m^3	kNm	Mpa	Mpa	
2.00	25.00	1.62	4434.86	0.826	0.446	632.94	-0.239	9.304	**OK**
3.00	29.00	0.64	5144.44	0.671	0.293	906.57	-0.609	16.163	**OK**
4.00	29.00	0.64	5144.44	0.671	0.293	1163.94	-0.617	15.278	**OK**
5.00	33.00	0.64	5854.01	0.677	0.293	1405.03	-0.617	17.322	**OK**
6.00	34.00	0.64	6031.41	0.679	0.293	1629.85	-0.617	17.288	**OK**
8.00	38.00	0.64	6740.98	0.679	0.293	2030.69	-0.617	18.667	**OK**
9.40	42.00	0.64	7450.56	0.678	0.293	2294.03	-0.617	20.515	**OK**
18.80	42.00	0.64	7450.56	0.678	0.293	3012.86	-0.617	18.063	**OK**

e_t : Eccentricity of tendons on top with neutral axis, e : Eccentricity of tendons on bottom with neutral axis

3. Check compressive stress at service limit state I

- Due to effecive prestress and permanent loads

$f_t = P_{pe}/A - P_{pe}.e/S_t + (M_g+M_s)/S_t + M_{SDL}/S_{tc} \le 0.45\ f'_c$

- Area of girder section A = 0.64 m^2
- Check stress of top fiber

x	n	P_{pe}	e	S_t	S_{tc}	M_g+M_s	M_{SDL}	$M_{LT}+M_{LL}$	f_t	Check
(m)	strands	kN	m	m^3	$1/m^3$	kNm	kNm	kNm	Mpa	
2.00	25.00	3638.21	0.826	0.561	0.757	996.01	274.33	773.52	-1.13	**OK**
3.00	29.00	4220.32	0.671	0.260	0.543	1435.88	399.93	1125.77	1.50	**OK**
4.00	29.00	4220.32	0.671	0.260	0.543	1849.28	517.83	1454.99	3.16	**OK**
5.00	33.00	4802.43	0.677	0.260	0.543	2236.22	628.03	1677.01	4.00	**OK**
6.00	34.00	4947.96	0.679	0.260	0.543	2596.68	730.51	1971.39	5.25	**OK**
8.00	38.00	5530.07	0.679	0.260	0.543	3238.20	912.37	2491.10	7.22	**OK**
9.40	42.00	6112.19	0.678	0.260	0.543	3645.76	1021.33	2800.12	8.26	**OK**
18.80	42.00	6112.19	0.678	0.260	0.543	4815.16	1361.77	3706.82	12.93	**OK**

- Due to 1/2.(effecive prestress + permanent loads) and transient loads

$f_t = 0.5(P_{pe}/A - P_{pe}.e/S_t + (M_g+M_s)/S_t + M_{SDL}.S_{tc}) + (M_{LL}+M_{LT})S_{tc} \le 0.40\ f'_c$

- Check stress of the top fiber

x	n	P_{pe}	e	S_t	S_{tc}	M_g+M_s	M_{SDL}	$M_{LT}+M_{LL}$	f_t	Check
(m)	strands	kN	m	m^3	$1/m^3$	kNm	kNm	kNm	Mpa	
2.00	25.00	3638.21	0.826	0.561	0.757	996.01	274.33	773.52	0.02	**OK**

3.00	29.00	4220.32	0.671	0.260	0.543	1435.88	399.93	1125.77	3.70	**OK**
4.00	29.00	4220.32	0.671	0.260	0.543	1849.28	517.83	1454.99	5.20	**OK**
5.00	33.00	4802.43	0.677	0.260	0.543	2236.22	628.03	1677.01	6.11	**OK**
6.00	34.00	4947.96	0.679	0.260	0.543	2596.68	730.51	1971.39	7.34	**OK**
8.00	38.00	5530.07	0.679	0.260	0.543	3238.20	912.37	2491.10	9.40	**OK**
9.40	42.00	6112.19	0.678	0.260	0.543	3645.76	1021.33	2800.12	10.56	**OK**
18.80	42.00	6112.19	0.678	0.260	0.543	4815.16	1361.77	3706.82	14.79	**OK**

• Due to effecive prestress and permanent loads and transient loads

$f_t = P_{pe}/A - P_{pe}.e/S_t + (M_g+M_s)/S_t + M_{SDL}.S_{tc} + (M_{LL}+M_{LT})S_{tc} \leq 0.40\ f'_c$

• Check stress of the top fiber

x	n	P_{pe}	e	S_t	S_{tc}	M_g+M_s	M_{SDL}	$M_{LT}+M_{LL}$	f_t	Check
(m)	strands	kN	m	m^3	$1/m^3$	kNm	kNm	kNm	Mpa	
2.00	25.00	3638.21	0.826	0.561	0.757	996.01	274.33	773.52	2.94	**OK**
3.00	29.00	4220.32	0.671	0.260	0.543	1435.88	399.93	1125.77	2.12	**OK**
4.00	29.00	4220.32	0.671	0.260	0.543	1849.28	517.83	1454.99	3.95	**OK**
5.00	33.00	4802.43	0.677	0.260	0.543	2236.22	628.03	1677.01	4.91	**OK**
6.00	34.00	4947.96	0.679	0.260	0.543	2596.68	730.51	1971.39	6.32	**OK**
8.00	38.00	5530.07	0.679	0.260	0.543	3238.20	912.37	2491.10	8.58	**OK**
9.40	42.00	6112.19	0.678	0.260	0.543	3645.76	1021.33	2800.12	9.78	**OK**
18.80	42.00	6112.19	0.678	0.260	0.543	4815.16	1361.77	3706.82	14.95	**OK**

4. Check tensile stress at service limit state III

• For bottom fiber

$f_b = P_{pe}/A + P_{pe}.e/S_b - (M_g+M_s)/S_b - M_{SDL}/S_{bc} - 0.8(M_{LL}+M_{LT})/S_{bc}$

• Check stress of the bottom fiber

x	n	P_{pe}	e	S_b	S_{bc}	M_g+M_s	M_{SDL}	$M_{LT}+M_{LL}$	f_b	Check
(m)	strands	kN	m	m^3	m^4	kNm	kNm	kNm	Mpa	
2.00	25.00	3638.21	0.826	0.446	0.547	996.01	274.33	618.82	5.35	**OK**
3.00	29.00	4220.32	0.671	0.293	0.348	1435.88	399.93	900.61	8.18	**OK**
4.00	29.00	4220.32	0.671	0.293	0.348	1849.28	517.83	1163.99	5.82	**OK**
5.00	33.00	4802.43	0.677	0.293	0.348	2236.22	628.03	1341.61	6.13	**OK**
6.00	34.00	4947.96	0.679	0.293	0.348	2596.68	730.51	1577.11	4.67	**OK**
8.00	38.00	5530.07	0.679	0.293	0.348	3238.20	912.37	1992.88	3.25	**OK**
9.40	42.00	6112.19	0.678	0.293	0.348	3645.76	1021.33	2240.10	3.24	**OK**
18.80	42.00	6112.19	0.678	0.293	0.348	4815.16	1361.77	2965.46	-3.40	**OK**

5. Check stress range in strands at fatigue limit state

• Compressive stress due to prestress and permanent load

$f_b = P_{pe}/A + P_{pe}.e/S_b - (M_g+M_s)/S_b - M_{SDL}/S_{bc}$

• Tensile stress due to frague

$f_t = -M_f / S_{bc}$

x	n	P_{pe}	e	S_{bc}	M_g+M_s	M_{SDL}	M_f	f_b	f_t	Check
(m)	strands	kN	m	m^4	kNm	kNm	kNm	Mpa	Mpa	
2.00	25.00	3638.21	0.826	0.547	996.01	274.33	117.67	6.354	-0.161	**OK**
3.00	29.00	4220.32	0.671	0.348	1435.88	399.93	190.89	10.250	-0.411	**OK**
4.00	29.00	4220.32	0.671	0.348	1849.28	517.83	259.07	8.501	-0.558	**OK**
5.00	33.00	4802.43	0.677	0.348	2236.22	628.03	322.21	9.217	-0.694	**OK**
6.00	34.00	4947.96	0.679	0.348	2596.68	730.51	380.30	8.293	-0.820	**OK**
8.00	38.00	5530.07	0.679	0.348	3238.20	912.37	481.37	7.835	-1.037	**OK**
9.40	42.00	6112.19	0.678	0.348	3645.76	1021.33	540.12	8.385	-1.164	**OK**

6. Strength limit state

• Ultimate moment

M_u = 1.25(DC) + 1.5(DW) + 1.75(LL+IM)

• Determine neutral axis, consider section is like as Retangular-shape (R) or T-shape (T)

• *Effective width of flange with exterior girder* b_{ef} = 2120.00 mm

x	n	A_s	A'_s	A_{ps}	b_w	k	c	a	hs	Check
(m)	strands	mm^2	mm^2	mm^2	mm		mm	mm	mm	
2.00	25	1570.80	5629.73	3500.00	924.51	0.280	94.82	75.85	196.20	**R**
3.00	29	1570.80	5629.73	4060.00	363.09	0.280	114.65	91.72	196.20	**R**
4.00	29	1570.80	5629.73	4060.00	363.09	0.280	114.65	91.72	196.20	**R**
5.00	33	1570.80	5629.73	4620.00	363.09	0.280	134.34	107.47	196.20	**R**
6.00	34	1570.80	5629.73	4760.00	363.09	0.280	139.25	111.40	196.20	**R**
8.00	38	1570.80	5629.73	5320.00	363.09	0.280	158.78	127.02	196.20	**R**
9.40	42	1570.80	5629.73	5880.00	363.09	0.280	178.18	142.55	196.20	**R**
18.80	42	1570.80	5629.73	5880.00	363.09	0.280	178.18	142.55	196.20	**R**

x (m)	f_{ps}	A_s	A'_s	A_{ps}	b_w	d_p	a	Mr	Mu	Check
	Mpa	mm^2	mm^2	mm^2	mm	mm	mm	kNm	kNm	
2.00	1833	1570.80	5629.73	3500.00	924.51	1800.96	75.85	12325	3010.17	**OK**
3.00	1827	1570.80	5629.73	4060.00	363.09	1800.96	91.72	14048	4364.84	**OK**
4.00	1827	1570.80	5629.73	4060.00	363.09	1800.96	91.72	14048	5634.59	**OK**
5.00	1821	1570.80	5629.73	4620.00	363.09	1800.96	107.47	15743	6672.09	**OK**
6.00	1820	1570.80	5629.73	4760.00	363.09	1800.96	111.40	16163	7791.56	**OK**
8.00	1814	1570.80	5629.73	5320.00	363.09	1800.96	127.02	17827	9775.74	**OK**
9.40	1808	1570.80	5629.73	5880.00	363.09	1800.96	142.55	19464	10989.4	**OK**
18.80	1808	1570.80	5629.73	5880.00	363.09	1800.96	142.55	19464	14548.5	**OK**

7. Check maximum ratio of reinforcement

• Distance from extreme compressive fiber to neutral axis c = 760.25 mm

• Depth from to Steel Centroid to extreme compressive fiber d_e = 1811.84 mm

• Ratio of maximum reinforcement **c/d_e = 0.41**

Check : $c/d_e = 0.41 \leq 0.42$ **OK**

8. Check minimum ratio of reinforcement

• Check : $M_r \geq \min(1.2M_{cr}, 1.33M_u)$

• Caculation M_{cr}

$M_{cr} = (f_r + f_{pe})S_{bc} - M_{d\text{-}nc}(S_{bc}/S_b - 1)$

• Where

• Tensile strength of concrete at 28 days f_r = 4.45 Mpa

• Moment due to weight of girder and slab $M_{d\text{-}nc}$

• Effective prestress in concrete

$f_{pe} = P_{pe}/A + P_{pe}.e/S_b$

x	$M_{d\text{-}nc}$	P_{pe}	e	S_{bc}	S_b	f_{pe}	$1.2M_{cr}$	$1.33M_u$	M_r	Check
(m)	kNm	kN	m	m^3	m^3	Mpa	kNm	kNm	kNm	
2.00	996.01	3638.21	0.826	0.547	0.446	8.60	8296.3	4003.53	12325.3	**OK**
3.00	1435.88	4220.32	0.671	0.348	0.293	16.30	8343.2	5805.24	14047.6	**OK**
4.00	1849.28	4220.32	0.671	0.348	0.293	16.30	8250.5	7494.01	14047.6	**OK**
5.00	2236.22	4802.43	0.677	0.348	0.293	18.65	9146.0	8873.88	15743.5	**OK**
6.00	2596.68	4947.96	0.679	0.348	0.293	19.25	9315.9	10362.77	16163.3	**OK**
8.00	3238.20	5530.07	0.679	0.348	0.293	21.50	10112.8	13001.7	17826.7	**OK**

9.40	3645.76	6112.19	0.678	0.348	0.293	23.75	10962.2	14615.9	19464.5	**OK**
18.80	4815.16	6112.19	0.678	0.348	0.293	23.75	10699.9	19349.6	19464.5	**OK**

9. Check shear resistance

- Shear Reinforcement Spacing — s — mm
- Area of one stirrup leg — A_v — mm^2
- Max. Spacing of Reinf. According to A_v — s_1 — mm
- Max. Spacing of Reinf. According to V_u — s_2 — mm
- minimum transverse reinforcement — A_{vmin} — mm

  $V_u = 1.25(DC) + 1.5(DW) + 1.75(LL+IM)$ — V_u — kN
- Check: Av, s

x	b_v	V_u	d_v	A_v	s	s_1	s_2	A_{vmin}	s≤ min(s_1,s_2)	$A_v \geq A_{vmin}$
(m)	mm	kN	mm	mm^2	mm	mm	mm	mm^2	kNm	mm^2
0.00	924.51	1477.22	1260.0	402.12	150.0	296.4	600.0	203.47	**OK**	**OK**
2.00	924.51	1241.14	1260.0	402.12	150.0	296.4	600.0	203.47	**OK**	**OK**
3.00	363.09	1178.79	1260.0	402.12	150.0	755	600.0	79.91	**OK**	**OK**
4.00	363.09	1116.78	1260.0	402.12	150.0	755	600.0	79.91	**OK**	**OK**
5.00	363.09	1055.12	1260.0	402.12	150.0	755	600.0	79.91	**OK**	**OK**
6.00	363.09	993.79	1260.0	402.12	150.0	755	600.0	79.91	**OK**	**OK**
8.00	363.09	872.17	1260.0	402.12	150.0	755	600.0	79.91	**OK**	**OK**
9.40	363.09	779.52	1260.0	402.12	150.0	755	600.0	79.91	**OK**	**OK**
18.80	363.09	301.53	1260.0	402.12	300.0	755	600.0	159.82	**OK**	**OK**

- Assumption angle of inclination — θ_1 — deg
- Choose angle of inclination — θ — deg
- Factor — β
- Shear stress in concrete — v — kN/m^2

  $v = (V_u - \varphi.V_P)/(\varphi.b_v.d_v)$
- Shear resistance factor — $\varphi =$ 0.90
- Shear Resistance due to prestress — $V_p =$ 0.00 kN
- Strain in the tensile reinforcement

  $\varepsilon_x = (M_u/d_v+0.5N_u+0.5V_u.\cot\theta-A_{ps}.f_{po})/(E_s.A_s + E_p.A_{ps}) \leq 0.002$

  if ε_x <0 then multiplier it with reduction factor Fe
- Reduction factor

  $F\varepsilon = (E_s.A_s+E_p.A_{ps})/(E_c.A_c+E_s.A_s+E_p.A_{ps})$
- Stress in tendon in where stress in surrounding concrete is equal to zero — $f_{po} =$ 1395.00 Mpa

- Check : $V_u \leq \varphi.V_n$

x	θ_1	v/f'c	$1000.\varepsilon_x$	θ	β	Vc	Vs	Vn	Vu	Check
(m)	Deg		mm	Deg		kN	kN	kN	kN	
0.00	27.00	0.028	0.000	27.00	3.25	2224.2	2651.8	4875.96	1477.2	**OK**
2.00	27.00	0.024	0.000	27.00	3.25	2224.2	2651.8	4875.96	1241.1	**OK**
3.00	27.00	0.057	0.000	27.00	3.73	1000.3	2651.8	3652.09	1178.8	**OK**
4.00	38.55	0.054	1.453	40.38	1.39	373.7	1588.7	1962.38	1116.8	**OK**
5.00	41.28	0.051	1.513	41.00	1.29	347.4	1554.2	1901.57	1055.1	**OK**
6.00	42.82	0.048	2.040	43.00	1.10	295.3	1448.9	1744.27	993.8	**OK**
8.00	43.00	0.042	2.621	43.00	1.10	295.3	1448.9	1744.27	872.2	**OK**
9.40	43.00	0.038	2.712	43.00	1.10	295.3	1448.9	1744.27	779.5	**OK**
18.80	43.00	0.015	4.456	43.00	1.10	295.3	724.5	1019.81	301.5	**OK**

10. Check tension in longitudinal reinforcement due to shear

- Expression

$A = A_s.f_y + A_{ps}.f_{ps} \geq M^{pe}_u/(d_v.\varphi) + 0.5N_u/\varphi + (V_u/\varphi - 0.5V_s - V_p)\cot\theta = B$

x	A_s	A_{ps}	M^{pe}_u	N_u	θ	Vu	Vs	A	B	Check
(m)	mm^2		kNm	kN	Deg	kN	kN	kN	kN	
2.00	1570.80	3500.0	4.3	3638.2	27.00	1241.1	2651.8	7042.35	1926.9	**OK**
3.00	1570.80	4060.0	1534.3	4220.3	27.00	1178.8	2651.8	8045.32	3296.2	**OK**
4.00	1570.80	4060.0	2804.0	4220.3	40.38	1116.8	1588.7	8045.32	4860.6	**OK**
5.00	1570.80	4620.0	3420.5	4802.4	41.00	1055.1	1554.2	9042.04	5570.5	**OK**
6.00	1570.80	4760.0	4431.0	4948.0	43.00	993.8	1448.9	9290.2	6397.9	**OK**
8.00	1570.80	5320.0	6023.3	5530.1	43.00	872.2	1448.9	10279.2	7807.7	**OK**
9.40	1570.80	5880.0	6845.0	6112.2	43.00	779.5	1448.9	11262.1	8640.6	**OK**

11. Check interface shear reinforcement (A 5.8.4.)

- Equation

$V_n = \min(c.A_{cv} + \mu.(A_{vf}.f_y + P_c), 0.2f'_c.A_{cv}, 5.5A_{cv})$ (5.8.4.1-1)

- Where
  - Cohesion factor — c = 1.000 Mpa
  - Friction factor — $\mu = 1.4\lambda =$ 1.400
  - area of concrete engaged in shear transfer — $A_{cv} = 2x0.643x1.0 =$ 1.286 m^2
  - area of shear reinforcement crossing the shear plane for 1m — $A_{vf} =$ 3217.0 mm^2
  - Width of interface section — $b_v = 2x0.643 =$ 1.286 m
- Check :
  - Minimum Ratio of reinforcement — $A_{vf} \geq 0.35b_v/f_y =$ 1125.25 **OK**
  - permanent net compressive force normal to the shear plane — $P_c =$ 0.000 kN
  - Nomal Interface Shear resistance for 1m in length — $V_n =$ 3087.51 kN
  - Distance from reinforcment to slab — $d_v =$ 1.730 m
  - Shear force act on 1m in length — $V_h =$ 717.55 kN / m
- **Check :** $V_h \leq \varphi.V_n$ ⇔ $717.55 \leq 2778.76$ kN **OK**

12. Check composite slab

- Stress in top fiber of slab

$f_{bs} = M_s/S_t + M_{SDL}/I_c + 0.8M_{LT+LL}/I_c \leq f'_{ct} = 0.45f'_c =$ 15.750 Mpa

x	y_t	y_{tc}	h_s	S_t	M_s	M_{SDL}	M_{LT+LL}	I_c	f_{bs}	Check
(m)	m	m	m	m^3	kNm	kNm	kNm	m^4	Mpa	
2.00	0.775	0.816	196.20	0.561	270.33	204.26	773.52	0.618	1.81	**OK**
3.00	0.927	0.760	196.20	0.260	394.11	297.78	1125.77	0.413	4.42	**OK**
4.00	0.927	0.760	196.20	0.260	510.29	385.56	1454.99	0.413	5.71	**OK**
5.00	0.927	0.760	196.20	0.260	618.88	467.61	1677.01	0.413	6.76	**OK**
6.00	0.927	0.760	196.20	0.260	719.87	543.92	1971.39	0.413	7.90	**OK**
8.00	0.927	0.760	196.20	0.260	899.08	679.32	2491.10	0.413	9.93	**OK**
9.40	0.927	0.760	196.20	0.260	1006.45	760.45	2800.12	0.413	11.13	**OK**
18.80	0.927	0.760	196.20	0.260	1341.93	1013.93	3706.82	0.413	14.79	**OK**

13. Camber and deflection

13.1.At Jack Release

13.1.1. Camber

- Prestress load for one tendon after jack release — p_{pe}= 177.39 kN
- Moment of inertia of girder section — Ix = 0.241 m^4
- Effective span length — Ls= 37.600 m
- Elastic modulus of concrete at jack release — E_c = 33942.519 Mpa

Number of	Ldebond	Debonding bi (m)	Distance e	ΣP_{pe} (kN)	Camber (m)	θ_1 (rad)	θ_2 (rad)
2.00	0.0 m	0.027	0.867	355	-0.007	-0.00073	0.00073
4.00	2.0 m	0.125	0.648	710	0.009	0.00089	-0.00089
4.00	4.0 m	0.231	0.723	710	0.009	0.00038	-0.00038
1.00	5.0 m	0.285	0.748	177	0.002	0.00004	-0.00004
4.00	6.0 m	0.338	0.673	710	0.006	-0.00001	0.00001
4.00	8.0 m	0.444	0.673	710	0.002	-0.00030	0.00030
25.00	0.0 m	0.009	0.674	4435	0.096	0.01018	-0.01018
Sum				7805	C=0.117	0.010	-0.010

13.1.2. Defelection

- Self weight of girder — W_g= 18.93 kN/m
- Deflection due to self weight of girder — $De=5W_g.L_s^4/(384.E.I_x)$ — De = -0.060 m ↓

13.1.3. Camber at release

- Camber at release — Δ = C - De — $\boxed{\Delta = 0.057 \text{ m} \uparrow}$

13.1.4. Camber at erection due to creep and shrinkage effect

- Long term deflection — Δ = 1.7C - 1.75De — $\boxed{\Delta = 0.093 \text{ m} \uparrow}$

13.2.At final stage

13.2.1. Camber and defleclection at release time

- Moment of inertia of composite section — Ix = 0.413 m^4
- Effective length — Ls= 37.600 m
- Elastic modulus of girder concrete at 28 days — E_c = 37948.890 Mpa
- Deflection due to self weight of girder — $De=5W_g.L_s^4/(384.E.I_x)$ — De = -0.060 m ↓
- Camber due to prestressing — C = 0.117 m

13.2.2. Defelection

- Elastic modulus of concrete at 28 days of composite member — E_c = 37948.89 Mpa
- Self weight of slab — W_s= 10.20 kN/m
- Deflection due to self weight of slab — $D_{slab}=5W_g.L_s^4/(384.E.I_x)$ — D_{slab} = -0.017 m ↓
- Self weight of parapet — W_s= 4.08 kN/m
- Deflection due to self weight of parapet — $D_p=5W_g.L_s^4/(384.E.I_x)$ — D_p = -0.007 m ↓
- Self weight of wearing — W_w= 3.62 kN/m
- Deflection due to self weight of girder — $D_w=5W_g.L_s^4/(384.E.I_x)$ — D_w= -0.006 m ↓

13.2.3. Camber at final time

- Camber at final — $\Delta = C - D_e - D_{slab} - D_p - D_w$ — $\boxed{\Delta = 0.03 \text{ m} \uparrow}$

13.2.4. Camber at final time due to creep effect

- Long term deflection — $\Delta = 2.1C - 2.2D_e - 2.15D_{slab} - 2.75(D_p+D_w)$ — $\boxed{\Delta = 0.041 \text{ m} \uparrow}$

13.3.Live Load Deflection

- Uniform load due to lane load is : w= 9.300 kN/m
- Deflection due to lane load is:

Δ_{lane} =5.w.L^4/(384.E.I) Δ_{lane} = 0.015 m

- Deflection due to truck is:

P (kN)	a (m)	b (m)	x (m)	D (m)
145	18.80	18.80	18.80	0.020
145	14.50	23.10	18.80	0.017
35	23.10	14.50	18.80	0.004
Sum			Δ_{truck} =	0.041

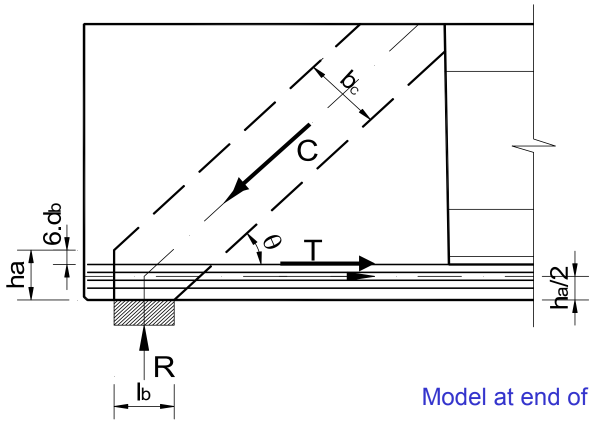
- Deflection due to live load is:

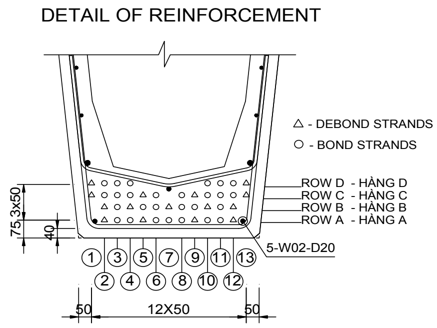
Δ_{LL}=(Δ_{lane} + Δ_{truck}).DFM = 0.029 m ↓

- Limited Deflection due to live load is: Δ_{Limit} = 0.047 m

Check :	$0.029 \leq 0.047$	m **OK**

14, Check end of Girder:





Model at end of Girder

Checking for end of Super-T girder follow strut and tie mode:

• l_b: The width of bearing	l_b =	350	(mm)
• h_a: The effective height of tension bar	h_a =	316	(mm)
• b_c: The effective thickness of compression bar	b_c =	471	(mm)
• θ: The effective angle	θ =	45	(deg)
• d: The effective width of compression bar	d =	600	(mm)
• n: Number of bond tendon	n =	25	(tendon)
• Effective final prestress after all of losses	f_{pe} =	1056.57	(Mpa)

- Bearing reaction at Strength state

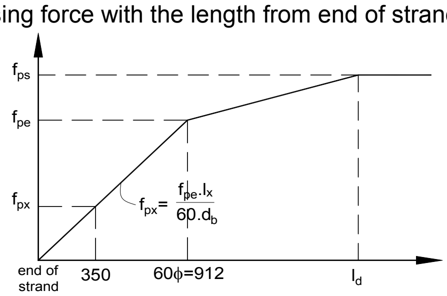
Combination Loading	DC (KN)	DW (KN)	LL+IM (KN)	TOTAL (KN)
Strength state	668.84	75.03	409.78	1665.69
Factor	1.25	1.5	1.75	

- Checking for Compressive bar (Strut bar C):

- Effective area of compression bar:	Ac =	282644.72	(mm)
- The limiting compressive stress:	f_{cu} =	42.50	(KN)
- Resistance factor:	φ =	0.70	
- The factor compresive stress	$C = R/\sin(\theta)$ =	2355.64	(Kn)
- The factored resistance of strut	$P_r = \varphi.f_{cu}.Ac$	8408.68	(Kn)
- Check condition	$P_r \geq C$	OK	

- Checking for tension bar (Tie bar T):

• Effective of pretressing force with the length from end of strand:



Development of prestressing of strand

- Distance from bearing section to end of girder	lx =	350	(mm)
- Effective final prestress after all of losses	f_{pe} =	1056.57	(Mpa)
- Effective prestressing stress at bearing section	f_{px} =	405.48	(Mpa)
- Total prestressing force at bearing section:	P_{px} =	1419.19	(KN)
- Resistance factor:	φ =	1.00	
- Tension resistance of prestressing force:	$P_{pr} = \varphi.P_{px}$ =	1419.19	(KN)
- The Area of reinforcement - bar D20:	As =	1570.80	(mm^2)
- The stress in reinforcement - bar D20:	f_y =	240.00	(Mpa)
- Tension resistance of reinforcement:	$P_{sr} = \varphi.A_s.f_y$ =	376.99	(KN)
- Final Tension resistance:	$T = P_{pr}+P_{sr}$ =	1796.18	(KN)
- Check tie bar follow condion:	$T \geq R/tg(\theta)$ =>	1796.18 > 1665.69	(KN)
		OK	

VII. CHECK INTERIOR GIRDER

1. Loss of prestress (A.5.9.5.)

• Total final loss

$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR2}$ Mpa

• Where

Δf_{pES} - Loss due to elastic shortening

Δf_{pSR} - Loss due to shrinkage

Δf_{pCR} - Loss due to creep of concrete

Δf_{pR2} - loss due to relaxation of steel after transfer

1.1.Loss due to elastic shortening (A.5.9.5.2.3.)

$\Delta f_{pES} = E_p / E_{ci} \cdot f_{cpg}$ Mpa

• Where

f_{cpg} - Sum of concrete stresses at the center of gravity of prestressing tendons due to the prestressing force at transfer and the self-weight of the member at the sections of maximum moment

f_{cgp} - be assumed to be $0.75f_{pu}$ for low-relaxation strands	$0.75f_{pu}$ =	1395	Mpa
pretension force after allowing for the initial losses	P =	195.3	kN
Assuming loss of stress after release	Loss =	0.929	%
• concrete stresses at the center of gravity of the prestressing steel	f_{pi} =	1382.045	Mpa

$f_{cpg} = P_i / A + P_i.e_c^2 / I - M_g.e_c / I$

• Number of tendon	n =	40	Stranc
• pretension force after allowing for the initial losses	P_{pi} =	7739.453	kN
• Moment due to self weight of girder	M_g =	3012.8594	kNm
• Average eccentricity of tendon	e_c =	0.689	m
• Area of girder section	A =	0.635	m^2
• Moment of inertia of girder	I_x =	0.241	m^4
• f_{cpg} - Sum of concrete stresses at the center of gravity of tendons	f_{cpg} =	18.79	Mpa
• Modulus of elasticity of concrete at jack release	E_{ci} =	33942.52	Mpa
• Modulus of elasticity of strand	E_p =	197000.00	Mpa
	Δf_{pES} =	**109.08**	**Mpa**

1.2.Loss due to shrinkage (A 5.9.5.4.2.)

$\Delta f_{pSR} = 117 - 1.03H$ Mpa

• the average annual ambient relative humidity	H =	80.000	%
	Δf_{pSR} =	**34.6**	**Mpa**

1.3.Loss due to creep of concrete (A 5.9.5.4.3.)

$\Delta f_{pCR} = 12f_{cpg} - 7\Delta f_{cdp} \geq 0$ Mpa

• Where

Δf_{cdp} - change of stresses at the center of gravity of the prestressing steel due to permanent loads except the dead load present at the time the prestress force is applied

• Moment due to weight of slab	M_s=	1802.30	kNm
• Moment due to weight of wearing and parapete	M_{w+u} =	1361.77	kNm
• Average eccentricity of prestress tendon	e_c =	1.041	m
• Moment of inertia of composite section	I_x =	0.4127	m^4
	Δf_{cdp} =	8.58	Mpa
	Δf_{pCR} =	**165.49**	**Mpa**

1.4.Loss due to relaxation of steel after transfer (A 5.9.5.4.4c.)

$\Delta f_{pR2} = 30\%(138 - 0.4\Delta f_{pES} + 0.2(\Delta f_{pSR} + \Delta f_{pCR}))$

• loss due to relaxation after transfer	**Δf_{pR2} =**	**16.31**	**Mpa**
• Initial loss	**$\Delta f_{initial}$ =**	**12.95**	**Mpa**

Sum of loss stress

$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR2}$	**Δf_{pT} =**	**325.48**	**Mpa**
Initial Prestress loss	Loss =	0.929	%

Conclusion

Different from assume and cacualtion of initial loss		Δ =	0.000 **OK**
• Effective initial prestress		**f_{pi} =**	**1382.05 Mpa**
Check:	f_{pi} = $1382.05 \le 1395.00$		**OK**
• Effective pretension force after allowing for the initial losses		**P_{pi} =**	**7739.45 kN**
• Effective prestress at jack release			**1272.97 Mpa**
• Effective final prestress after all of losses		**f_{pe} =**	**1056.57 Mpa**
Check:	f_{pi} = $1056.57 \le 1339.20$		**OK**
• Total prestressing force after all losses		**P_{pe} =**	**5916.79 kN**

2. Check stress at jack release

• For top fiber

$f_t = P_i/A - P_i.e/S_t + M_g/S_t + f_{top}$

$f_{top} = p/A + p.e_t / S_t$; p - prestress after initial loss of 2 tendons on top fiber

x	n	A	P_i	e	St	Mg	f_{top}	f_t	Check
(m)	strands	m^2	kN	m	m^3	kNm	Mpa	Mpa	
2.00	24.00	1.62	4643.67	0.836	0.561	632.94	0.220	-2.699	**OK**
3.00	28.00	0.64	5417.62	0.671	0.260	906.57	0.561	-1.382	**OK**
4.00	28.00	0.64	5417.62	0.671	0.260	1163.94	0.561	-0.394	**OK**
5.00	32.00	0.64	6191.56	0.677	0.260	1405.03	0.561	-0.394	**OK**
6.00	32.00	0.64	6191.56	0.679	0.260	1629.85	0.561	0.419	**OK**
8.00	36.00	0.64	6965.51	0.679	0.260	2030.69	0.561	1.174	**OK**
9.40	40.00	0.64	7739.45	0.678	0.260	2294.03	0.561	1.402	**OK**
18.80	40.00	0.64	7739.45	0.682	0.260	3012.86	0.561	4.043	**OK**

e_t : Eccentricity of tendons on top with neutral axis, e : Eccentricity of tendons on bottom with neutral axis

• For bottom fiber

$f_b = P_i/A + P_i.e/S_b - M_g/S_b - f_{btop}$

$f_{btop} = p/A + p.e_t / S_b$; p - prestress after initial loss of 2 tendons on top fiber

x	n	A	P_i	e	S_b	M_g	f_{btop}	f_b	Check
(m)	strands	m^2	kN	m	m^3	kNm	Mpa	Mpa	
2.00	24.00	1.62	4643.67	0.836	0.446	632.94	-0.220	9.937	**OK**
3.00	28.00	0.64	5417.62	0.671	0.293	906.57	-0.561	17.267	**OK**
4.00	28.00	0.64	5417.62	0.671	0.293	1163.94	-0.561	16.389	**OK**
5.00	32.00	0.64	6191.56	0.677	0.293	1405.03	-0.561	18.690	**OK**
6.00	32.00	0.64	6191.56	0.679	0.293	1629.85	-0.561	17.967	**OK**
8.00	36.00	0.64	6965.51	0.679	0.293	2030.69	-0.561	19.596	**OK**
9.40	40.00	0.64	7739.45	0.678	0.293	2294.03	-0.561	21.694	**OK**
18.80	40.00	0.64	7739.45	0.682	0.293	3012.86	-0.561	19.348	**OK**

e_t : Eccentricity of tendons on top with neutral axis, e : Eccentricity of tendons on bottom with neutral axis

3. Check compressive stress at service limit state I

• Due to effecive prestress and permanent loads

$f_t = P_{pe}/A - P_{pe}.e/S_t + (M_g+M_s)/S_t + M_{SDL}/S_{tc} \leq 0.45\ f'_c$

• Area of girder section A = 0.64 m^2

• Check stress of top fiber

x	n	P_{pe}	e	S_t	S_{tc}	M_g+M_s	M_{SDL}	$M_{LT}+M_{LL}$	f_t	Check
(m)	strands	kN	m	m^3	$1/m^3$	kNm	kNm	kNm	Mpa	
2.00	24.00	3550.07	0.836	0.561	0.757	996.01	274.33	527.83	-1.11	**OK**
3.00	28.00	4141.75	0.671	0.260	0.543	1435.88	399.93	768.19	1.58	**OK**
4.00	28.00	4141.75	0.671	0.260	0.543	1849.28	517.83	992.85	3.23	**OK**
5.00	32.00	4733.43	0.677	0.260	0.543	2236.22	628.03	1144.35	4.07	**OK**
6.00	32.00	4733.43	0.679	0.260	0.543	2596.68	730.51	1345.22	5.47	**OK**
8.00	36.00	5325.11	0.679	0.260	0.543	3238.20	912.37	1699.86	7.44	**OK**
9.40	40.00	5916.79	0.678	0.260	0.543	3645.76	1021.33	1910.73	8.46	**OK**
18.80	40.00	5916.79	0.682	0.260	0.543	4815.16	1361.77	2529.43	13.04	**OK**

• Due to 1/2.(effecive prestress + permanent loads) and transient loads

$f_t = 0.5(P_{pe}/A - P_{pe}.e/S_t + (M_g+M_s)/S_t + M_{SDL}.S_{tc}) + (M_{LL}+M_{LT})S_{tc} \leq 0.40\ f'_c$

• Check stress of the top fiber

x	n	P_{pe}	e	S_t	S_{tc}	M_g+M_s	M_{SDL}	$M_{LT}+M_{LL}$	f_t	Check
(m)	strands	kN	m	m^3	$1/m^3$	kNm	kNm	kNm	Mpa	
2.00	24.00	3550.07	0.836	0.561	0.757	996.01	274.33	527.83	-0.16	**OK**
3.00	28.00	4141.75	0.671	0.260	0.543	1435.88	399.93	768.19	3.03	**OK**
4.00	28.00	4141.75	0.671	0.260	0.543	1849.28	517.83	992.85	4.34	**OK**

5.00	32.00	4733.43	0.677	0.260	0.543	2236.22	628.03	1144.35	5.11	**OK**
6.00	32.00	4733.43	0.679	0.260	0.543	2596.68	730.51	1345.22	6.25	**OK**
8.00	36.00	5325.11	0.679	0.260	0.543	3238.20	912.37	1699.86	8.00	**OK**
9.40	40.00	5916.79	0.678	0.260	0.543	3645.76	1021.33	1910.73	8.98	**OK**
18.80	40.00	5916.79	0.682	0.260	0.543	4815.16	1361.77	2529.43	12.63	**OK**

- Due to effecive prestress and permanent loads and transient loads

$f_t = P_{pe}/A - P_{pe}.e/S_t + (M_g+M_s)/S_t + M_{SDL}.S_{tc} + (M_{LL}+M_{LT})S_{tc} \leq 0.40\ f'_c$

- Check stress of the top fiber

x	n	P_{pe}	e	S_t	S_{tc}	M_g+M_s	M_{SDL}	$M_{LT}+M_{LL}$	f_t	Check
(m)	strands	kN	m	m^3	$1/m^3$	kNm	kNm	kNm	Mpa	
2.00	24.00	3550.07	0.836	0.561	0.757	996.01	274.33	527.83	2.68	**OK**
3.00	28.00	4141.75	0.671	0.260	0.543	1435.88	399.93	768.19	2.00	**OK**
4.00	28.00	4141.75	0.671	0.260	0.543	1849.28	517.83	992.85	3.77	**OK**
5.00	32.00	4733.43	0.677	0.260	0.543	2236.22	628.03	1144.35	4.69	**OK**
6.00	32.00	4733.43	0.679	0.260	0.543	2596.68	730.51	1345.22	6.20	**OK**
8.00	36.00	5325.11	0.679	0.260	0.543	3238.20	912.37	1699.86	8.36	**OK**
9.40	40.00	5916.79	0.678	0.260	0.543	3645.76	1021.33	1910.73	9.50	**OK**
18.80	40.00	5916.79	0.682	0.260	0.543	4815.16	1361.77	2529.43	14.42	**OK**

4. Check tensile stress at service limit state III

- For bottom fiber

$f_b = P_{pe}/A + P_{pe}.e/S_b - (M_g+M_s)/S_b - M_{SDL}/S_{bc} - 0.8(M_{LL}+M_{LT})/S_{bc}$

- Check stress of the bottom fiber

x	n	P_{pe}	e	S_b	S_{bc}	M_g+M_s	M_{SDL}	$M_{LT}+M_{LL}$	f_b	Check
(m)	strands	kN	m	m^3	m^4	kNm	kNm	kNm	Mpa	
2.00	24.00	3550.07	0.836	0.446	0.547	996.01	274.33	422.26	5.50	**OK**
3.00	28.00	4141.75	0.671	0.293	0.348	1435.88	399.93	614.55	8.53	**OK**
4.00	28.00	4141.75	0.671	0.293	0.348	1849.28	517.83	794.28	6.37	**OK**
5.00	32.00	4733.43	0.677	0.293	0.348	2236.22	628.03	915.48	6.84	**OK**
6.00	32.00	4733.43	0.679	0.293	0.348	2596.68	730.51	1076.18	4.98	**OK**
8.00	36.00	5325.11	0.679	0.293	0.348	3238.20	912.37	1359.88	3.91	**OK**
9.40	40.00	5916.79	0.678	0.293	0.348	3645.76	1021.33	1528.58	4.11	**OK**
18.80	40.00	5916.79	0.682	0.293	0.348	4815.16	1361.77	2023.54	-1.91	**OK**

5. Check stress range in strands at fatigue limit state

- Compressive stress due to prestress and permanent load

$f_b = P_{pe}/A + P_{pe}.e/S_b - (M_g+M_s)/S_b - M_{SDL}/S_{bc}$

- Tensile stress due to frague

$f_t = - M_f / S_{bc}$

x	n	P_{pe}	e	S_{bc}	M_g+M_s	M_{SDL}	M_f	f_b	f_t	Check
(m)	strands	kN	m	m^4	kNm	kNm	kNm	Mpa	Mpa	
2.00	24.00	3550.07	0.836	0.547	996.01	274.33	117.67	6.208	-0.161	**OK**
3.00	28.00	4141.75	0.671	0.348	1435.88	399.93	190.89	9.946	-0.411	**OK**
4.00	28.00	4141.75	0.671	0.348	1849.28	517.83	259.07	8.197	-0.558	**OK**
5.00	32.00	4733.43	0.677	0.348	2236.22	628.03	322.21	8.949	-0.694	**OK**
6.00	32.00	4733.43	0.679	0.348	2596.68	730.51	380.30	7.459	-0.820	**OK**
8.00	36.00	5325.11	0.679	0.348	3238.20	912.37	481.37	7.038	-1.037	**OK**
9.40	40.00	5916.79	0.678	0.348	3645.76	1021.33	540.12	7.626	-1.164	**OK**

6. Strength limit state

• Ultimate moment

M_u = 1.25(DC) + 1.5(DW) + 1.75(LL+IM)

• Determine neutral axis, consider section is like as Retangular-shape (R) or T-shape (T)

• *Effective width of flange with exterior girder* b_{ef} = 2120.00 mm

x	n	A_s	A'_s	A_{ps}	b_w	k	c	a	hs	Check
(m)	strands	mm^2	mm^2	mm^2	mm		mm	mm	mm	
2.00	24	1570.80	5629.73	3360.00	924.51	0.280	89.84	71.87	196.20	R
3.00	28	1570.80	5629.73	3920.00	363.09	0.280	109.70	87.76	196.20	R
4.00	28	1570.80	5629.73	3920.00	363.09	0.280	109.70	87.76	196.20	R
5.00	32	1570.80	5629.73	4480.00	363.09	0.280	129.43	103.54	196.20	R
6.00	32	1570.80	5629.73	4480.00	363.09	0.280	129.43	103.54	196.20	R
8.00	36	1570.80	5629.73	5040.00	363.09	0.280	149.03	119.22	196.20	R
9.40	40	1570.80	5629.73	5600.00	363.09	0.280	168.50	134.80	196.20	R
18.80	40	1570.80	5629.73	5600.00	363.09	0.280	168.50	134.80	196.20	R

x (m)	f_{ps}	A_s	A'_s	A_{ps}	b_w	d_p	a	Mr	Mu	Check
	Mpa	mm^2	mm^2	mm^2	mm	mm	mm	kNm	kNm	
2.00	1834	1570.80	5629.73	3360.00	924.51	1800.96	71.87	11891	2580.21	OK
3.00	1828	1570.80	5629.73	3920.00	363.09	1800.96	87.76	13620	3739.08	OK
4.00	1828	1570.80	5629.73	3920.00	363.09	1800.96	87.76	13620	4825.83	OK
5.00	1823	1570.80	5629.73	4480.00	363.09	1800.96	103.54	15322	5739.92	OK
6.00	1823	1570.80	5629.73	4480.00	363.09	1800.96	103.54	15322	6695.76	OK
8.00	1817	1570.80	5629.73	5040.00	363.09	1800.96	119.22	16998	8391.06	OK
9.40	1811	1570.80	5629.73	5600.00	363.09	1800.96	134.80	18649	9432.96	OK
18.80	1811	1570.80	5629.73	5600.00	363.09	1800.96	134.80	18649	12488.11	OK

7. Check maximum ratio of reinforcement

• Distance from extreme compressive fiber to neutral axis c = 760.25 mm

• Depth from to Steel Centroid to extreme compressive fiber d_e = 1812.34 mm

• Ratio of maximum reinforcement **c/d_e = 0.41**

Check : $c/d_e = 0.41 \leq 0.42$ **OK**

8. Check minimum ratio of reinforcement

• Check : $M_r \geq \min(1.2M_{cr}, 1.33M_u)$

• Caculation M_{cr}

$M_{cr} = (f_r + f_{pe})S_{bc} - M_{d\text{-}nc}(S_{bc}/S_b - 1)$

• Where

• Tensile strength of concrete at 28 days f_r = 4.45 Mpa

• Moment due to weight of girder and slab $M_{d\text{-}nc}$

• Effective prestress in concrete

$f_{pe} = P_{pe}/A + P_{pe}.e/S_b$

x	$M_{d\text{-}nc}$	P_{pe}	e	S_{bc}	S_b	f_{pe}	$1.2M_{cr}$	$1.33M_u$	M_r	Check
(m)	kNm	kN	m	m^3	m^3	Mpa	kNm	kNm	kNm	
2.00	996.01	3550.07	0.836	0.547	0.446	8.46	8208.9	3431.68	11890.5	OK
3.00	1435.88	4141.75	0.671	0.348	0.293	15.99	8216.5	4972.98	13619.5	OK
4.00	1849.28	4141.75	0.671	0.348	0.293	15.99	8123.8	6418.36	13619.5	OK
5.00	2236.22	4733.43	0.677	0.348	0.293	18.38	9034.1	7634.09	15321.9	OK
6.00	2596.68	4733.43	0.679	0.348	0.293	18.41	8967.4	8905.36	15321.9	OK
8.00	3238.20	5325.11	0.679	0.348	0.293	20.70	9780.0	11160.1	16998.2	OK
9.40	3645.76	5916.79	0.678	0.348	0.293	23.00	10645.0	12545.8	18648.7	OK
18.80	4815.16	5821.13	0.682	0.348	0.293	22.70	10260.7	16609.2	18648.7	OK

9. Check shear resistance

- Shear Reinforcement Spacing — s — mm
- Area of one stirrup leg — A_v — mm^2
- Max. Spacing of Reinf. According to A_v — s_1 — mm
- Max. Spacing of Reinf. According to V_u — s_2 — mm
- minimum transverse reinforcement — A_{vmin} — mm

  $V_u = 1.25(DC) + 1.5(DW) + 1.75(LL+IM)$ — V_u — kN
- Check: Av, s

x	b_v	V_u	d_v	A_v	s	s_1	s_2	A_{vmin}	s≤ min(s_1,s_2)	$A_v \geq A_{vmin}$
(m)	mm	kN	mm	mm^2	mm	mm	mm	mm^2	kNm	mm^2
2.00	924.51	1099.79	1260.0	402.12	150.0	296.4	600.0	203.47	**OK**	**OK**
3.00	363.09	1044.54	1260.0	402.12	150.0	754.8	600.0	79.91	**OK**	**OK**
4.00	363.09	989.59	1260.0	402.12	150.0	754.8	600.0	79.91	**OK**	**OK**
5.00	363.09	934.95	1260.0	402.12	150.0	754.8	600.0	79.91	**OK**	**OK**
6.00	363.09	880.61	1260.0	402.12	150.0	754.8	600.0	79.91	**OK**	**OK**
8.00	363.09	772.84	1260.0	402.12	150.0	754.8	600.0	79.91	**OK**	**OK**
9.40	363.09	690.74	1260.0	402.12	300.0	754.8	600.0	159.82	**OK**	**OK**
18.80	363.09	267.19	1260.0	402.12	300.0	754.8	600.0	159.82	**OK**	**OK**

- Assumption angle of inclination — θ_1 — deg
- Choose angle of inclination — θ — deg
- Factor — β
- Shear stress in concrete — v — kN/m^2

  $v = (V_u - \varphi.V_P)/(\varphi.b_v.d_v)$
- Shear resistance factor — $\varphi =$ 0.90
- Shear Resistance due to prestress — $V_p =$ 0.00 kN
- Strain in the tensile reinforcement

  $\varepsilon_x = (M_u/d_v+0.5N_u+0.5V_u.\cot\theta-A_{ps}.f_{po})/(E_s.A_s + E_p.A_{ps}) \leq 0.002$

  if ε_x <0 then multiplier it with reduction factor Fe
- Reduction factor

  $F\varepsilon = (E_s.A_s+E_p.A_{ps})/(E_c.A_c+E_s.A_s+E_p.A_{ps})$
- Stress in tendon in where stress in surrounding concrete is equal to zezo — $f_{po} =$ 1395.00 Mpa

- Check : $V_u \leq \varphi.V_n$

x	θ_1	v/f'c	$1000.\varepsilon_x$	θ	β	Vc	Vs	Vn	Vu	Check
(m)	Deg		mm	Deg		kN	kN	kN	kN	
2.00	27.00	0.021	0.000	27.00	4.88	3336.3	2651.8	5988.07	1099.8	**OK**
3.00	27.00	0.051	0.000	27.00	4.88	1310.3	2651.8	3962.03	1044.5	**OK**
4.00	38.55	0.048	0.970	35.64	2.25	603.3	1884.8	2488.07	989.6	**OK**
5.00	41.28	0.045	1.007	36.07	2.23	597.7	1854.9	2452.61	935.0	**OK**
6.00	42.82	0.043	1.593	41.37	1.91	512.1	1534.1	2046.16	880.6	**OK**
8.00	43.00	0.038	2.070	43.00	1.72	461.8	1448.9	1910.74	772.8	**OK**
9.40	43.00	0.034	2.119	43.00	1.72	461.8	724.5	1186.28	690.7	**OK**
18.80	43.00	0.013	3.636	43.00	1.72	461.8	724.5	1186.28	267.2	**OK**

10. Check tension in longitudinal reinforcement due to shear

• Expression

$A = A_s.f_y + A_{ps}.f_{ps} \geq M^{pe}_u/(d_v.\varphi) + 0.5N_u/\varphi + (V_u/\varphi - 0.5V_s - V_p)\cot\theta = B$

x	A_s	A_{ps}	M^{pe}_u	N_u	θ	Vu	Vs	A	B	Check
(m)	mm^2		kNm	kN	Deg	kN	kN	kN	kN	
2.00	1570.80	3360.0	-386.3	3550.1	27.00	1099.8	2651.8	6790.63	1264.6	**OK**
3.00	1570.80	3920.0	961.2	4141.8	27.00	1044.5	2651.8	7795.16	2509.4	**OK**
4.00	1570.80	3920.0	2047.9	4141.8	35.64	989.6	1884.8	7795.16	3915.5	**OK**
5.00	1570.80	4480.0	2535.0	4733.4	36.07	935.0	1854.9	8793.44	4531.5	**OK**
6.00	1570.80	4480.0	3480.9	4733.4	41.37	880.6	1534.1	8793.4	5369.4	**OK**
8.00	1570.80	5040.0	4777.7	5325.1	43.00	772.8	1448.9	9785.5	6598.3	**OK**
9.40	1570.80	5600.0	5421.1	5916.8	43.00	690.7	724.5	10771.5	7695.4	**OK**

11. Check interface shear reinforcement (A 5.8.4.)

• Equation

V_n =min $(c.A_{cv} + \mu.(A_{vf}.f_y + P_c), 0.2f'_c.A_{cv}, 5.5A_{cv})$ (5.8.4.1-1)

• Where

• Cohesion factor $c =$ 1.000 Mpa

• Friction factor $\mu = 1.4\lambda =$ 1.400

• area of concrete engaged in shear transfer $A_{cv} = 2x0.643x1.0 =$ 1.286 m^2

• area of shear reinforcement crossing the shear plane for 1m $A_{vf} =$ 3217.0 mm^2

• Width of interface section $b_v = 2x0.643 =$ 1.286 m

• Check :

• Minimum Ratio of reinforcement $A_{vf} \geq 0.35b_v/f_y =$ 1125.25 **OK**

• permanent net compressive force normal to the shear plane $P_c =$ 0.000 kN

• Nomal Interface Shear resistance for 1m in length $V_n =$ 3087.51 kN

• Distance from reinforcment to slab $d_v =$ 1.734 m

• Shear force act on 1m in length $V_h =$ 634.41 kN / m

• Check : $V_h \leq \varphi.V_n$ ⇔ $634.41 \leq 2778.76$ kN **OK**

12. Check composite slab

• Stress in top fiber of slab

$f_{bs} = M_s/S_t + M_{SDL}/I_c + 0.8M_{LT+LL}/I_c \leq f'_{ct} = 0.45f'_c =$ 15.750 Mpa

x	y_t	y_{tc}	h_s	S_t	M_s	M_{SDL}	M_{LT+LL}	I_c	f_{bs}	Check
(m)	m	m	m	m^3	kNm	kNm	kNm	m^4	Mpa	
2.00	0.775	0.816	196.20	0.561	184.47	204.26	773.52	0.618	1.66	**OK**
3.00	0.927	0.760	196.20	0.260	268.93	297.78	1125.77	0.413	3.94	**OK**
4.00	0.927	0.760	196.20	0.260	348.21	385.56	1454.99	0.413	5.09	**OK**
5.00	0.927	0.760	196.20	0.260	422.30	467.61	1677.01	0.413	6.01	**OK**
6.00	0.927	0.760	196.20	0.260	491.22	543.92	1971.39	0.413	7.03	**OK**
8.00	0.927	0.760	196.20	0.260	613.50	679.32	2491.10	0.413	8.83	**OK**
9.40	0.927	0.760	196.20	0.260	686.77	760.45	2800.12	0.413	9.91	**OK**
18.80	0.927	0.760	196.20	0.260	915.70	1013.93	3706.82	0.413	13.16	**OK**

13. Camber and deflection

13.1.At Jack Release

13.1.1. Camber

- Prestress load for one tendon after jack release p_{pe}= 178.22 kN
- Moment of inertia of girder section Ix = 0.241 m^4
- Effective span length Ls= 37.600 m
- Elastic modulus of concrete at jack release E_c = 33942.519 Mpa

Number of	Ldebond	Debonding bi (m)	Distance e	ΣP_{pe} (kN)	Camber (m)	θ_1 (rad)	θ_2 (rad)
2	0.0 m	0.027	0.867	356	-0.007	-0.00073	0.00073
4.00	2.0 m	0.125	0.648	713	0.009	0.00089	-0.00089
4.00	4.0 m	0.231	0.723	713	0.009	0.00038	-0.00038
0.00	5.0 m	0.285	0.748	0	0.000	0.00000	0.00000
4.00	6.0 m	0.338	0.673	713	0.006	-0.00001	0.00001
4.00	8.0 m	0.444	0.673	713	0.002	-0.00030	0.00030
24.00	0.0 m	0.009	0.668	4277	0.092	0.00981	-0.00981
Sum				7485	C=0.112	0.010	-0.010

13.1.2. Defelection

- Self weight of girder W_g= 18.93 kN/m
- Deflection due to self weight of girder $De=5W_g.L_s^4/(384.E.I_x)$ De = -0.060 m ↓

13.1.3. Camber at release

- Camber at release $\Delta = C - De$ $\Delta =$ 0.051 m ↑

13.1.4. Camber at erection due to creep and shrinkage effect

- Long term deflection $\Delta = 1.7C - 1.75De$ $\Delta =$ 0.084 m ↑

13.2.At final stage

13.2.1. Camber and defleclection at release time

- Moment of inertia of composite section Ix = 0.413 m^4
- Effective length Ls= 37.600 m
- Elastic modulus of concrete of girder at 28 days E_c = 37948.89 Mpa
- Deflection due to self weight of girder $De=5W_g.L_s^4/(384.E.I_x)$ De = -0.060 m ↓
- Camber due to prestressing C = 0.112 m

13.2.2. Defelection

- Elastic modulus of concrete of composite member at 28 days E_c = 37948.89 Mpa
- Self weight of slab W_s= 10.20 kN/m
- Deflection due to self weight of slab $D_{slab}=5W_g.L_s^4/(384.E.I_x)$ D_{slab} = -0.017 m ↓
- Self weight of parapet W_s= 4.08 kN/m
- Deflection due to self weight of parapet $D_p=5W_g.L_s^4/(384.E.I_x)$ D_p = -0.007 m ↓
- Self weight of wearing W_w= 3.62 kN/m
- Deflection due to self weight of girder $D_w=5W_g.L_s^4/(384.E.I_x)$ D_w= -0.006 m ↓

13.2.3. Camber at final time

- Camber at final $\Delta = C - D_e - D_{slab} - D_p - D_w$ $\Delta =$ 0.022 m ↑

13.2.4. Camber at final time due to creep effect

- Long term deflection $\Delta = 2.1C - 2.2D_e - 2.15D_{slab} - 2.75(D_p+D_w)$ $\Delta =$ 0.03 m ↑

13.3.Live Load Deflection

- Uniform load due to lane load is : w= 9.300 kN/m
- Deflection due to lane load is:

Δ_{lane} =5.w.L^4/(384.E.I) Δ_{lane} = 0.015 m

- Deflection due to truck is:

P (kN)	a (m)	b (m)	x (m)	D (m)
145	18.80	18.80	18.80	0.020
145	14.50	23.10	18.80	0.017
35	23.10	14.50	18.80	0.004
Sum			Δ_{truck} =	0.041

- Deflection due to live load is:

Δ_{LL}=(Δ_{lane} + Δ_{truck}).DFM = 0.029 m ↓

- Limited Deflection due to live load is: Δ_{Limit} = 0.047 m

Check :	$0.029 \leq 0.047$	m **OK**

1.2.4 Check Link Slab
1.2.4 Kiểm toán bản liên tục nhiệt

LINK SLAB - CALCULATION SHEET

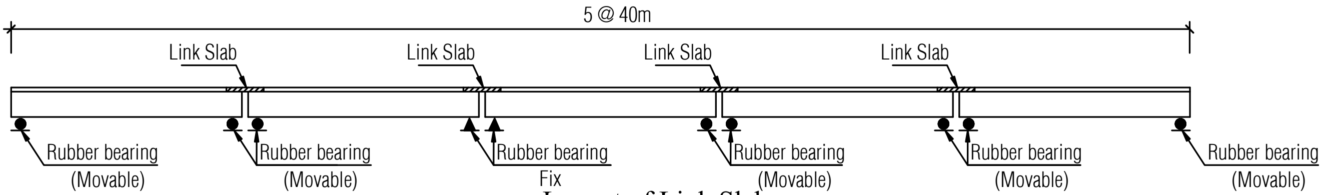
1. DATA FOR CALCUALATION

1.1. Design standards and references:

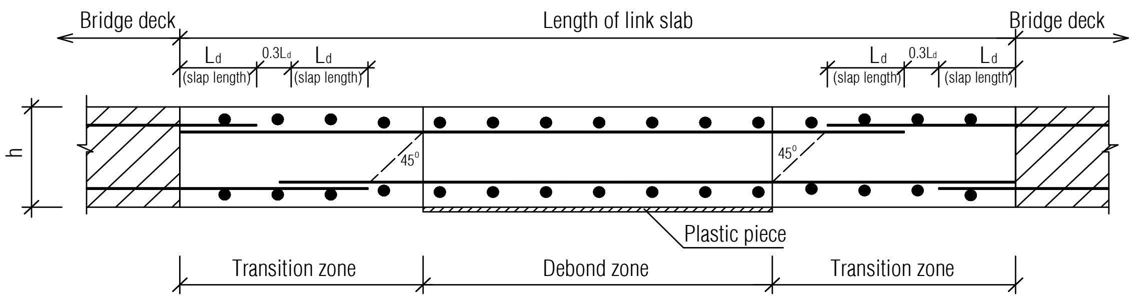
- Specifications for bridge design: **22TCN 272 - 05.**
- Specifications for bridge design: **AASHTO 1998 - LRFD.**
- Bridge standard **22 TCN 18- 79**

1.2. Structure:

- Length of Super T - girder	$L =$	38.30	m
- Distance between 2 bearings	$L_s =$	37.60	m
- Bridge width	$W =$	13.00	m
- Carriageway width	$W_c =$	12.00	m
- Thickness of deck	$h_{tb} =$	0.21	m
- Height of Super T - girder	$h =$	1.75	m
- Distance between 2 girder center	$b_e =$	2.12	m
- Parameter for link-slab			
+ Length of link-slab (debond zone)	$l_b =$	1.70	m
+ Thicknes of link-slab	$h_b =$	0.21	m
+ Width of link slab	$b =$	2.12	m
+ Moment of inertia of link-slab.	$J_b =$	0.00164	m^4



Layout of Link Slab



Detailed of Link Slab

- Dead load: (self weigh of concrete deck).	$DC =$	10.91	kN/m
- Adition load (wearing coat and parapet)	$DW =$	7.88	kN/m

1.3. Materials:

1.3.1 Reinforcement (TCVN 1651:2008)

Elastic modulus	$E_s =$	200 000	(MPa)
Tensile strength of ribbed reinforcement (CB400-V)	$f_{sy} =$	400	(MPa)
Tensile strength of ribbed reinforcement at service stage	$f_{sa} = 0.6f_{sy} =$	240	(MPa)
Tensile strength of plain reinforcement (CB300-V)	$f_{syr} =$	300	(MPa)
Tensile strength of plain reinforcement at service stage	$f_{sar} = 0.6f_{syr} =$	180	(MPa)

1.3.2. Concrete (22 TCN 272:05)

Concrete density	$y_c =$	2 500	(kg/m^3)
Thermal expansion coefficient of concrete	$\alpha_T =$	1.08E-05	$/ ^0C$

Proportinality factor between concrete and reinforcement	p =	0.20
Average Humidity	H =	80 %

1.3.2.1 Super T - girder

Compressive strength of concrete at 28 days age	f_c =	50.00 (MPa)
Moment of inertia of Girder	J_d =	0.43786 m^4
Elastic Modulus	$E_d = 0.043y_c^{1.5}f_c^{0.5}$ =	38 007 (MPa)
Tensile strength of concrete	$f_r = 0.63f_c^{0.5}$ =	4.45 (MPa)

1.3.2.2 Link slab

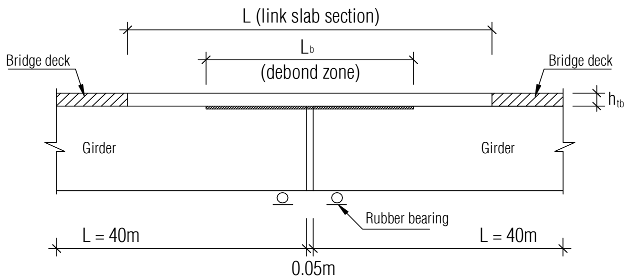
Compressive strength of concrete at 28 days age	f_{cs} =	35.00 (MPa)
Elastic Modulus	$E_b = 0.043y_c^{1.5}f_{cs}^{0.5}$ =	31 799 (MPa)
Tensile strength of concrete	$f_{rs} = 0.63f_{cs}^{0.5}$ =	3.73 (MPa)

1.3.2.3 Converion factor of materials:

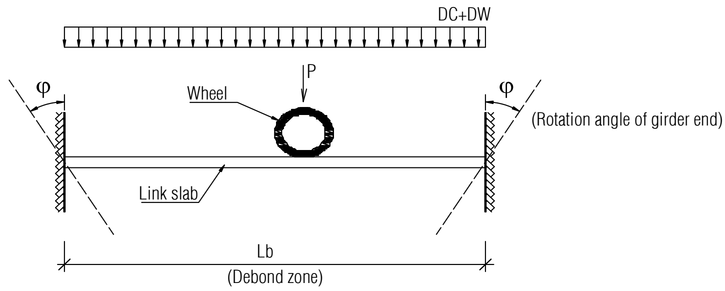
Reinforcement/concrete (of Super T - girder)	$R_{sc} = E_s/E_d$ =	6.00
Reinforcement/concrete (of link slab)	$R_{scs} = E_s/E_b$ =	7.00
Concrete of link slab/ Concrete of super T- girder	$R_{dc} = E_b/E_d$ =	0.84

2. DEFINE THE INTERNAL FORCE OF LINK SLAB

Model for calculation of link slab is fixed-end slab. Internal forces in link slab result from the followings:



General view of link slab



Model for calculation

2.1. Due to the deformation of Super T - girder :

2.1.1. Deformation of Super T - Girder due to dead load

Define the rotation Angle of end girder due to dead load

$$\varphi = \frac{0.7ML_s}{3E_dJ_b} = \quad 0.0018 \text{ (rad)}$$

Where M = 3320.63 kNm (moment at the midle of Super T girder due to dead load)

L_s = 37.60 m

E_dJ_d = 16641559 (kNm^2) - Girder stiffness

0.70 Experimental factor in consideration of theoretical rotation angle not fit to actual

- At the end of link slab (position link slab and girder was bonded)

$$M_1 = -\frac{2E_bJ_bK}{l_b}\varphi = \quad -91.08 \text{ kNm}$$

Q_1 = 0.00 (kN)

E_b = 31798929 (kN/m^2) - Elastic modular of the link slab material

J_b = 0.00164 (m^4) - Moment of inertia of link slab cross section

l_b = 1.70 (m) - Length of link-slab (debond zone)

K = 0.85 - Reduction stiffness factor when calculate for bending deflection and angular deflection

2.1.2. Deformation of Super T - Girder due to live load

Define the rotation Angle of end girder due to live load: $\varphi = (\varphi_1 + \varphi_2)$

* Rotation angle φ_1 due to Truck is defined : $\varphi_1 = \dfrac{(g_e.P).L^2 s}{16.E_d.J_d} =$ 0.0012 (rad)

g_e = 0.70 Distribution factor of Live load

P = 325 (kN) Total weigh of design Truck

Ls = 37.6 (m) Length of calculated span

* Rotation angle φ_2 due to Lane load is defined : $\varphi_2 = \dfrac{0.7ML_s}{3E_dJ_b} =$ 0.0006 (rad)

$$M = g_e \frac{qL^2{}_s}{8} =$$ 1153.73 kNm (moment at the middle of Super T girder due to design lane load)

Where

q = 9.3 kN/m design lane load

- At the debond zone end of link slab :

$$M_1 = -\frac{2E_bJ_bK}{l_b}(\varphi_1+\varphi_2) =$$ -94.67 kNm

Q_1 = 0.00 (kN)

2.2. Due to dead load and live load on the link slab section:

2.2.1. Dead load on link slab section:

- At the end of debond zone of link slab:

$$M_1 = \frac{(\gamma_{DC}DC+\gamma_{DW}DW)l_b^2}{12} =$$ -4.53 kNm (when dead load factor =1)

-6.13 kNm (when dead load factor =1.25, 1.5)

$$Q_1 = \frac{(\gamma_{DC}DC+\gamma_{DW}DW).l_b}{2} =$$ 15.97 kN (when dead load factor =1)

21.64 kN (when dead load factor =1.25, 1.5)

DC = 10.91 (kN/md)- link slab self weigh

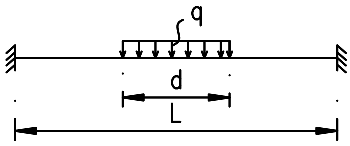
DW = 7.88 (kN/md)- wearing coat and parapet.

γ_{DC} = 1/1.25 Dead load factor (for link slab self weight)

γ_{DW} = 1/1.5 Dead load factor (for wearing coat and parapet)

2.2.2. Live load on link slab section:

Partial internal force in link slab due to the effection of live load directly on link-slab in consideration with distribution through bridge flooring's coating



- At the end of debond zone of link slab:

$$M_1 = -\frac{q.d.l_b}{24}\left(3-\frac{d^2}{l_b^2}\right)\gamma(1+IM) =$$ -37.68 kNm (when live load factor γ_{LL}=1.0)

-65.94 kNm (when live load factor γ_{LL}=1.75)

$$Q_1 \frac{q.d}{2}\gamma(1+IM) =$$ 93.57 kN (when live load factor γ_{LL}=1.0)

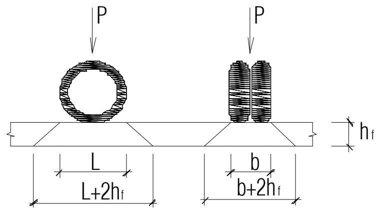
163.74 kN (when live load factor γ_{LL}=1.75)

- At the center of debond zone of link slab:

$$M_2 = \frac{q.d.l_b}{24}\left(3+\frac{d^2}{l_b^2}-\frac{3d}{l_b}\right)\gamma(1+IM) =$$ 26.08 kNm (when live load factor γ_{LL}=1.0)

45.64 kNm (when live load factor γ_{LL}=1.75)

$Q_2 =$ 0.00 kN



q = 158.60 (kN/m) - distribute load due to wheel pressure.

P = 72500 (N) - Load due to design vehicle (1/2 of axle load) .

$h_1 =$ 8.40 (cm) - Thickness of wearing coat

$l_b =$ 1.70 (m) - Length of link slab (debond zone)

The tire contact area of a wheel consisting of one or two tires shall be assumed to be a single rectangle, whose width b=510mm and whose length in mm (L) shall be taken as:

$L = 2.28 \times 10^{-3}\gamma(1+IM/100)xP =$ 506.2 mm

$\gamma_{LL} =$ 1/1.75 Live load factor

IM = 75 Dynamic load (%.)

$L+2h_f =$ 0.67 (m) - Length of load distribution (longitudinal of bridge).

$b+2h_f =$ 0.68 (m) - Width of load distribution (transever of bridge).

2.3. Due to Temperature, Braking force and longitudinal gradient of girder:

2.3.1. Temperature

Axial force of axis N created by thermal at link slab will be total shearing forces at all movable bearings of the initial part of nearest strings:

$$Nt = \pm\gamma_{TU}\sum_{i=1}^{j}\Delta_i.\left(\frac{FpGp}{hp}\right)_i = \begin{matrix} 29.79 & \text{kN (when load factor } \gamma_{TU}=1) \\ 35.75 & \text{kN (when load factor } \gamma_{TU}=1.2) \end{matrix}$$

$\gamma_{TU} =$ 1/1.2 Load factor

$\Delta_i =$ 1.39 (cm) - Longitudinal displacement in each bearing of span structure-strings .
get from elastometric bearing calculation sheet.

$F_p =$ 2100 (cm2) - Area of bearings on lay-out.

$h_p =$ 14.1 (cm) - Total thickness of rubber layers of bearing

$G_p =$ 0.08 (kN/cm2) - Anti-shearing modular of rubber (as temperature).

t = 30 (degree)- Calculated temperature of rubber bearing.

No.	Δ_i	F_p	G_p	h_p	Nt (kN)
1	0.00	2100	0.08	8.4	0.00
2	0.66	2100	0.08	8.4	13.18
3	1.39	2100	0.08	14.1	16.61
Total					29.79

2.3.2 Braking force:

Axial force due to braking force is calculated as follows.

For design Truck

BR =0.25*n*m*(35+2*145) = 207.19 kN

For Design Tandem

BR =0.25*n*m*(2*110) = 140.25 kN

=> N = 34.53 kN - Braking force for 1 girder (load factor γ_{LL}=1.0).
60.43 kN - Braking force for 1 girder (load factor γ_{LL}=1.75).

+ n: number of traffic lane
+ m: coefficient of traffice lane

2.3.3 Longitudinal gradient of girder:

Axial force generated in link slab will be :

$$N = \sum^{n} P_j.i = 26.18 \text{ kN (with load factor } \gamma_{DC} = 1.0)$$

$j=i$ 32.72 kN (with load factor γ_{DC} = 1.25)

γ_{DC} = 1/1.25 Load factor

P_j = 727.16 (kN)- Seflt weight of one girder and plank slab.

i = longitudinal gradient of girder. 1.2%

n = Number of span, n = 3.0

2.4. Load combination:

Load combination table

Strength stage		Internal force		
Combine	Load	M(kNm)	N(kN)	Q(kN)
Strength 1	Due to the deformation of Super T - Girder consider dead load	-91.08		
	Due to dead load on the link slab section	-6.13		21.64
	Due to Live load on the link slab section	-65.94		163.74
	Axial force generated by variation of temperature		35.75	
	Axial force due to longitudinal gradient of girder		32.72	
	Axial force due to Braking force		60.43	
	Total	**-163.15**	**128.90**	**185.38**
Strength 2	Due to the deformation of Super T - Girder consider dead load	-91.08		
	Due to dead load on the link slab section	-6.13		21.64
	Due to Live load on span	-94.67		0.00
	Axial force generated by variation of temperature		35.75	
	Axial force due to longitudinal gradient of girder		32.72	
	Axial force due to Braking force		60.43	
	Total	**-191.88**	**128.90**	**21.64**
Service 1	Due to the deformation of Super T - Girder consider dead load	-91.08		
	Due to dead load on the link slab section	-4.53		15.97
	Due to Live load on the link slab section	-37.68		93.57
	Axial force generated by variation of temperature		29.79	
	Axial force generated by bridge's gradient		26.18	
	Axial force generated by braking force		34.53	
	Total	**-133.28**	**90.50**	**109.54**
Service 2	Due to the deformation of Super T - Girder consider dead load	-91.08		
	Due to dead load on the link slab section	-4.53		0.00
	Due to Live load on span	-94.67		0.00
	Axial force generated by variation of temperature		29.79	
	Axial force generated by bridge's gradient		26.18	
	Axial force generated by braking force		34.53	
	Total	**-190.27**	**90.50**	**0.00**
Max(Combine1, Combine2)		**190.27**	**90.50**	**109.54**

3. STRENGTH LIMIT STATES CHECK

3.1. Limits for reinforcement (5.7.3.3)

3.1.1. Maximum reinforcement

The maximum amount of reinforcement shall be such that:

$$\frac{c}{d_e} \leq 0.42$$

for which :

+ c : the distance from the extreme compression fiber to the neutral axis (mm)

$$c = \frac{A_s f_y - 0.85\beta_1 f'_c (b - b_w) h_f}{0.85\beta_1 f'_c b_w} \qquad (5.7.3.1.2\text{-}3)$$

In case c < h_f , the neutral axis in the flang of cross secction, same the rectangular cross section.

$$c = \frac{A_s f_y}{0.85\ \beta_1 f'_c\ b} \qquad (5.7.3.1.2\text{-}4)$$

+ d_e : the corresponding effective depth from the extreme compression fiber to the centroid of the tensile force in the tensile reinforcement (mm)

For the cross section have only nonprestressed reinforcement $d_e = d_s$

+ β_1 : stress block factor specified

$$\beta_1 = 0,85 - \frac{f'_c - 28}{7}.0,05 \geq 0,65 \qquad (5.7.2.2)$$

+ b : width of compression flange (mm)
+ h_f : depth of compression flange (mm)
+ f'_c : compression strength of concrete at 28 days age
+ A_s : area of mild steel tension reinforcement (mm^2)
+ f_y : yield strength of tension reinforcement (MPa)
+ d_s: distance from extreme compression fiber to the centroid of reinforcement (mm)

Design tensile reinforcement for link slab in the calculation width:

n =	15	bar steel (number of steel bar)
a =	150	mm (center - to - center spacing of steel bars)
d =	22	mm (diameter of steel bar)
Cover	50	mm (concrete cover)

Parameter	A_s	f_y	β_1	f'_c	b	d_e
Units	mm^2	MPa		MPa	mm	mm
Value	5701.99	400.00	0.80	35.00	2120.00	149
c/d_e	0.303					
Conclusion	**OK**					

3.1.2. Minimum reinforcement

For components containing no prestressing steel, the minimum reinforcement provision herein may be considered satisfied if : $P_{min} = \frac{A_s}{A_g} \geq 0,03\frac{f'_c}{f_y}$

Parameter	A_s	A_g	A_s/A_g	f'_c	f_y	$0,03.f'_c/f_y$
Units	mm^2	mm^2		MPa	MPa	
Value	5701.99	439498.01	0.0130	35.00	400.00	0.00263
Conclusion	**OK**					

3.2. Flexural resistance check (5.7.3.2)

The resistance moment Mr, shall be taken as:

$$M_r = \varphi.M_n \geq M_u \quad (kN.m)$$

for which :

$$M_n = A_s.f_y.(d_s - \frac{a}{2}) + 0,85.f_c'.(b - b_w).\beta_1.h_f.(\frac{a}{2} - \frac{h_f}{2}) \quad (kN.m)$$

In case the neutral axis in the flange of cross section $b_w = b$ (same rectangular cross section)

for which:

+ M_u : moment due to load
+ M_r : resistance moment of link slab
+ M_n : nominal resistance moment of link slab
+ φ =0.9 : resistance factor as specified in aticle 5.5.4.2
+ d_s : distance from extreme compression fiber to the centroid of nonprestressed tensile inforcement (mm)
+ $a = c.\beta_1$: depth of the equivalent stress block (mm)

Flexural resistance check							
Parameter	A_s	f_y	d_s	c	$a = c.\beta_1$	$\varphi.M_n$	M_u
Units	mm^2	MPa	mm	mm	mm	kN.m	kN.m
Value	5701.99	400.00	149.00	45.20	36.16	268.74	191.88
Conclusion:$\varphi.M_n \geq M_u$	**OK**						

3.3. Shear resistance check (5.8.3.3)

The nominal shear resistance, V_n shall be determined as the lesser of :

$$\text{Min} \begin{cases} V_n = V_c + V_s \\ V_n = 0,25f'_c b_v d_v \end{cases} \quad (5.8.3.3\text{-}2)$$

for which :

$$V_c = 0,083\beta\sqrt{f'_c}\,b_v d_v$$

$$V_s = \frac{A_v f_y d_v (\cot g\theta + \cot g\alpha)\sin\alpha}{s} \quad (5.8.3.3\text{-}4)$$

+ φ = 0.9 : resistance factor as specified in aticle 5.5.4.2
+ b_v = 2120 : width of link slab in the calculation (mm)
+ d_v = 151.2 : effective shear depth (mm), as determined in Article 5.8.2.7
  $d_v = \max(0.9d_e; 0.72h; d_e\text{-}a/2)$
+ s : spacing of stirrups (mm)
+ β = 2 : factor indicating ability of diagonally cracked concrete to transmit tension as specified in Acticle 5.8.3.4
+ θ = 45 : angle of inclination of diagonal compressive stresses (degree) as determined in Acticle 5.8.3.4
+ α = 90 : angle of inclination of transverse reinforcement to longitudinal axis (degree)
+ A_v : area of shear reinforcement within a distance s (mm^2)

* For link slab, the effect of stirrups is location for the reinforcement. So calculation is not necessary for the shear resistance of stirrups where consider only the shear resistance of the concrete section

=> not consider the V_s in calculation

Shear resistance check					
Parameter	d_v	f'_c	$0,25f'_cb_vd_v$	$V_n=\varphi.Vn$	V_u
Units	mm	MPa	kN	kN	kN
Value	151.2	35.00	2804.76	2524.28	185.38
Conclusion : $V_u \le \varphi V_n$	**OK**				

4. SLAB AUDIT ACCORDANCE WITH SERVICE LIMIT STATES

4.1. Checking crack on cross-section

Firstly, it is needed to carry out checking concrete section , if concrete section is cracked, all tension force will be transmited to reinforcement. That is why the tensile stress in reinforcement must be controlled.

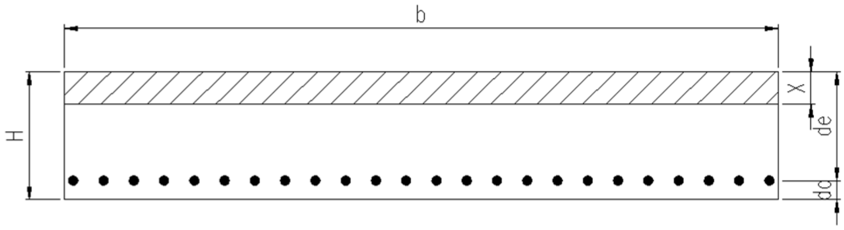
In order to confirm that section is cracked or not, it is needed to calculate the tensile stress in the fiber concrete to define whether $f_{cs} > 80\% f_r$ or not, if $f_{cs} > 80\% f_r$ is mean the section was crecked

$$f_{cs} = M_u . y_d / J_b$$

$$f_r = 0{,}63\sqrt{f'_c}$$

Description	Parameter	Value	Unit
Max bending moment in link slab	M_u	190.27	kN.m
Axial force	N	90.50	kN
Moment of inertia of link slab	J_b	0.00164	m^4
Distance from tensile fiber to the centroid	y_d	0.065	m
Concrete compressive strength	f'_c	35.00	MPa
Concrete tensile strength	f_r	3.73	MPa
Tensile stess at top-fiber of link slab	f_c	7.56	MPa
Limit of cracking of concrete	$0{,}8.f_r$	2.98	MPa
conclusion	**Required check crack**		

4.2. Determine the Characteristics of cracked section:



Distance from extreme compression fiber to center of cracked section, x :

$$0{,}5.x^2.b = n.A_s.(d_e\text{-}x)$$

From the above equation, : x = 61.04 mm

Moment of inertia of the cracked section : $I_{cr} = b.x^3/3 + n.A_s.(d_e\text{-}x)^2$ = 5.5E+08 mm^4

4.3. Reinforcement arragement for cracking controlling (5.7.3.4)

The components should be so proportioned that the tensile stress in the mild steel reinforcement at the service limit state, f_{sa} does not exceed:

$$f_s \leq f_{sa} = \min\left(\frac{Z}{(d_c A)^{1/3}} ; 0{,}6.f_y\right)$$

for which:

+ f_s : tensile tress in reinforcement $f_s = n.\frac{M(d_e - x)}{I_{cr}} + \frac{N}{A_s}$

+ Z : Crack width diameter (N/mm), In moderate exposure condition, Z= 30000

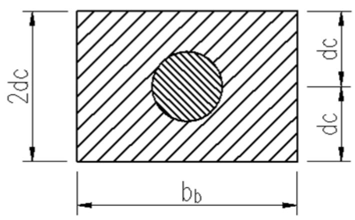
+ d_c : depth of concrete measured from extreme tension fiber to center of bar (mm), dc≤ 50mm

+ d_e : effective depth from extreme compression fiber to centroid of the tensile force in the tensile reinforcement (mm)

+ A : Area of concrete having same centroid as the principle tensile reinforcement and bounded by the surfaces of the cross-section and a straight line parrallel to the neutral axis; divided by the number of bars, for calculation purposes, the thickness of clear concrete cover used to compute A shall nots be taken to be greater than 50mm

+ n : modular ratio Es/Ec between reinforcement and concrete
+ n_s : number of bar of tensile reinforcement in the wide of b_b
+ I_{cr} : Moment of interia of the cracked section (if section is cracked), I_g in case of uncracked section
+ x : distance from extreme compression fiber to center of cracked section

f_s			$A = 2d_cb_b/n_s$		
Parameter	Value	Unit	Parameter	Value	Unit
d_e	160	mm	d_c	50	mm
x	61.04	mm	b_b	2120	mm
I_{cr}	6E+08	mm^4	n_s	15	bar
M	2E+08	N.m	A_s	5701.99	mm^2
N	9E+04	N	A	14133.3	mm^2
n	7.00		f_{sa}	336.81	MPa
f_s	238.95	MPa	$0,6.f_y$	240.0	MPa
Conclusion					
f_s	238.95	MPa	$min(f_{sa,}0,6f_{y)}$	240.0	MPa
OK					



1.2.5 Check Elastomeric bearing
1.2.5 Kiểm toán gối cao su

Contents

1. General

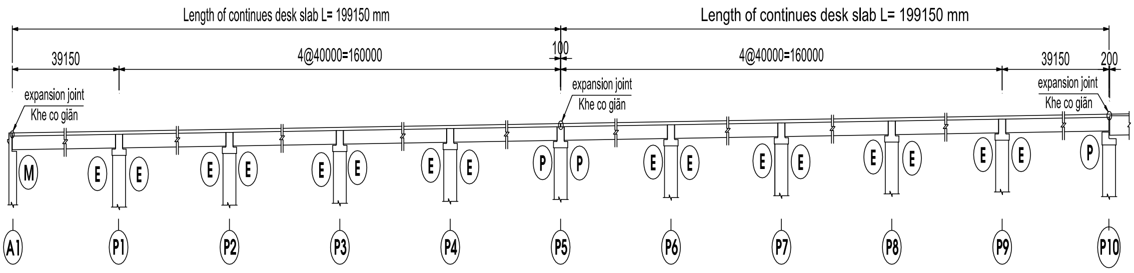
1. 1. Out Line

The super structure that are supported by these bearings is five-span super T girder. The girders are connected by Link-Slab. Support condition of the bearings are showed as bellow table. the design seismic intensity is very small, so that I assume that one bearing is hinged for longitudinal. All transversal support are hinged by the anchored bar.

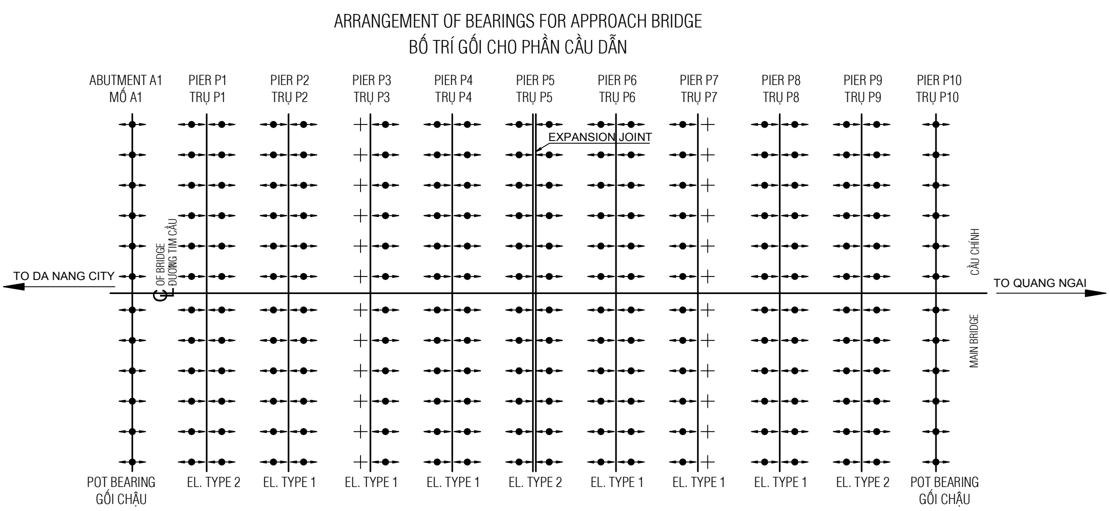
This calculation sheets are regarding the elastomeric bearings.

Table Support condition and bearing type

Name		Condition		Bearing Type	
		Long.	Trans.	Long.	Trans.
A1	-	M	H	Pot Bearing	Anchored Bar
P1	BS	E	H	Elastomeric Bearing	Anchored Bar
	ES	E	H	Elastomeric Bearing	Anchored Bar
P2	BS	E	H	Elastomeric Bearing	Anchored Bar
	ES	E	H	Elastomeric Bearing	Anchored Bar
P3	BS	H	H	Elastomeric Bearing	Anchored Bar
	ES	E	H	Elastomeric Bearing	Anchored Bar
P4	BS	E	H	Elastomeric Bearing	Anchored Bar
	ES	E	H	Elastomeric Bearing	Anchored Bar
P5	-	E	H	Elastomeric Bearing	Anchored Bar



E :Elastomeric Bearing P :Pot Bearing



+ :Fix Position

:Movable Position

1.2. Summary of Calculation of Elastomeric Bearing

Super T girder

1 Material of Elastomeric Bearing

					S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
					A1	P1		P2		P3		P4		P5
Bearing Type		Is=1;Pot Bearing Is=0,Elastmeric Bearing	Is	-	1	0	0	0	0	2	0	0	0	0
			-	-	Pot Bearing	Elastomeric Bearing	Elastomeric Bearing	Elastomeric Bearing	Elastomeric Bearing	Elastomeric Bearing	Elastomeric Bearing	Elastomeric Bearing	Elastomeric Bearing	Elastomeric Bearing
Elastomer	Hardness Grade				60	60	60	60	60	60	60	60	60	60
	Shear modulus at 23°C	0.90 -1.38	Ge	Mpa	-	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Insert Reinforcement	ASTM A709 Grad 250	Yield Strength	fsy	Mpa	250	250	250	250	250	250	250	250	250	250
	ASTM A709 Grad 250	Fatigue stress	fsfy	Mpa	165	165	165	165	165	165	165	165	165	165
Number of the bearing on each line			Nb	nos	6	6	6	6	6	6	6	6	6	6

2 Property of elastomeric bearings Input Data

					S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
					A1	P1		P2		P3		P4		P5
Condition of support		Remark	-	-	1	0	0	0	0	2	0	0	0	0
		Condition	-	-	Mv	E	E	E	E	F	E	E	E	E
Dimension	Longitudinal	-	L0	mm	-	350	350	350	350	350	350	350	350	350
	Transversal	-	W0	mm	-	600	600	600	600	600	600	600	600	600
Necessary dimensions	Longitudinal	-	reqL	mm	-	230	230	230	230	220	230	230	230	230
	Transversal	-	reqW	mm	-	230	230	230	230	220	230	230	230	230
Using dimension for design	Longitudinal	Input Data	L	mm	-	340	340	340	340	340	340	340	340	340
	Transversal	Input Data	W	mm	-	590	590	590	590	590	590	590	590	590
Modulus of Rigidity		−	G	Mpa	-	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Thickness of layer of elastomeric	Thickness of necessity	−	Σte	mm	-	87	99	33	53	9	7	54	37	102
	Thickness of one layer	Input Data	hri	mm	-	16	16	16	16	16	16	16	16	16
	Req. Number layer for Displacement	-	reqn	layer	-	6	7	3	4	1	1	4	3	7
	Number layer		n	layer	-	7	7	4	4	4	4	4	4	7
	Total thickness of elastomeric	Σ te=te*n	hrt	mm	-	112	112	64	64	64	64	64	64	112
	Thickness of top layer	Input Data	hrit	mm	-	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
	Thickness of bottom layer	Input Data	hrib	mm	-	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
Steel Plate	Medium Plate	Input Data	tsm	mm	-	3	3	3	3	3	3	3	3	3
	Top Plate	Input Data	tst	mm	-	3	3	3	3	3	3	3	3	3
	Bottom Plate	Input Data	tsb	mm	-	3	3	3	3	3	3	3	3	3
Total area for calculation		F=a•b	F	m2	-	0.2006	0.2006	0.2006	0.2006	0.2006	0.2006	0.2006	0.2006	0.2006
Shape factor		S=a•b/(2(a+b)te)	S	-	-	6.741	6.741	6.741	6.741	6.741	6.741	6.741	6.741	6.741
Spring of elastomeric bearing		AG/hrt	Kh	kN/m	0	1,791	1,791	3,134	3,134	3,134	3,134	3,134	3,134	1,791

2 Reaction and movements Pre-shearing IS= 0 Not pre-shearing

						SP1	SP2	SP3	SP4	SP5	SP6	SP7	SP8	SP9	SP10
						A1	P1		P2		P3		P4		P5
Reaction	Reaction of permanent load		-	Rd	kN	750	750	750	750	750	750	750	750	750	750
	Reaction of live load	Max	-	Rmax	kN	550	550	550	550	550	550	550	550	550	550
		Min	-	Rmin	kN	0	0	0	0	0	0	0	0	0	0
	Service	Max	-	Rmax	kN	1300	1300	1300	1300	1300	1300	1300	1300	1300	1300
		Min	-	Rmin	kN	750	750	750	750	750	750	750	750	750	750
Movement	At time of working dead-load		-	ld	mm	44	22	28	7	16	-4	4	-15	-8	-28
	SH+CR		-	sh+cr	mm	44	22	28	7	16	-4	4	-15	-8	-28
	Service		-	ls	mm	79	43	49	16	26	4	3	27	18	51
	Rotation st service limit state		-	θ	rad	0.0070	0.0070	0.0070	0.0070	0.0070	0.0070	0.0070	0.0070	0.0070	0.0070

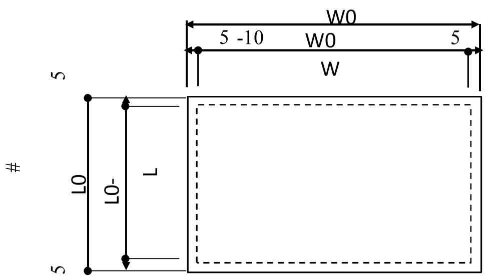
3 Check of the compressive Deflection

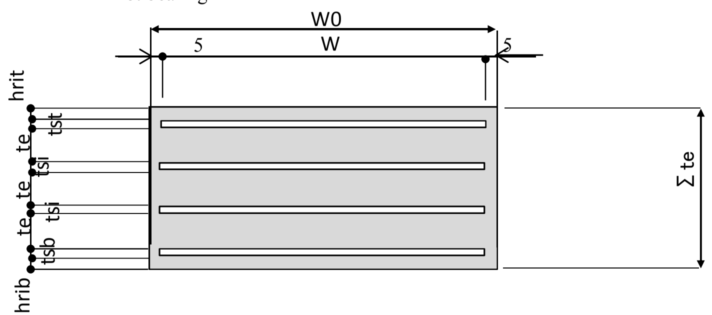
						SP1	SP2	SP3	SP4	SP5	SP6	SP7	SP8	SP9	SP10
						A1	P1		P2		P3		P4		P5
Check of the compression of elastomeric bearings	Compressive stress	Dead Load	Rd/F	σd	Mpa	-	3.74	3.74	3.74	3.74	3.74	3.74	3.74	3.74	3.74
		Service Stage	R/F	σs	Mpa	-	6.48	6.48	6.48	6.48	6.48	6.48	6.48	6.48	6.48
		Live load	RL/F	σl	Mpa	-	2.74	2.74	2.74	2.74	2.74	2.74	2.74	2.74	2.74
	Judgment	Maximum compressive stress	σs≦σsa		Mpa	-	OK	OK	OK	OK	OK	OK	OK	OK	OK
		due to live load	σl≦σla		Mpa	-	OK	OK	OK	OK	OK	OK	OK	OK	OK
Check of the compressive Deflection	Compressive Deflection due to live load	Strain of each layer	-	εi	%	-	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00
		Deflection of inner layer	ΣεLii*hrii	δLi	mm	-	2.2400	2.2400	1.2800	1.2800	1.2800	1.2800	1.2800	1.2800	2.2400
		Deflection of top layer	ΣεLt*hrt	δLt	mm	-	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05
		Deflection of bottom layer	ΣεLt*hrt	δLb	mm	-	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05
		Total deflection	-	δL	mm	-	2.34	2.34	1.38	1.38	1.38	1.38	1.38	1.38	2.34
	Judgment	Limit deflection	-	δLa	mm	-	3.00	3.00	3.00	3.00	3.00	3.00	3.00	3.00	3.00
		Judgment	δL≦δLa	-	mm	-	OK	OK	OK	OK	OK	OK	OK	OK	OK
Check of Shear deformation	Check of Shear deformation	Total thickness of elastomeric layers	—	hrt	mm	-	112	112	64	64	64	64	64	64	112
		Design Movement		dl	mm	79	43	49	16	26	4	3	27	18	51
		At service limit state	ls/hrt	γs	%	-	39	44	25	41	7	5	42	29	45
		Judgment	Allowable		%	-	50	50	50	50	50	50	50	50	50
			Safty	F	-	-	1.30	1.13	1.97	1.23	7.49	9.72	1.19	1.74	1.10
				—	—	-	OK	OK	OK	OK	OK	OK	OK	OK	OK
Check of compressive stress due to rotation and angle of rotation of the girder	Angle of rotation		-	θ	rad	0.007	0.007	0.007	0.007	0.007	0.007	0.007	0.007	0.007	0.007
	Judgment	Limit angle due to rotation	1.0GS(θ/n)(B/hri)^2	σamin	Mpa	-	3.23	3.23	5.64	5.64	5.64	5.64	5.64	5.64	3.23
		Local Compressive Stress due to Rotation	2.25GS(1-0.167(θs/n)(B/hri)^2	σamax	Mpa	-	11.43	11.43	10.52	10.52	13.05	10.52	10.52	10.52	11.43
		Accrual Stress	R/(L*B)	σs	Mpa	-	6.48	6.48	6.48	6.48	6.48	6.48	6.48	6.48	6.48
		Check for Edge Uplift	σs＞σsmin	-	-	-	OK	OK	OK	OK	OK	OK	OK	OK	OK
		Check for Compression	σs＜σsmax	-	-	-	OK	OK	OK	OK	OK	OK	OK	OK	OK
		Judgment	-	-	-	-	OK	OK	OK	OK	OK	OK	OK	OK	OK
Check for buckling of elastomeric bearing	Result of calculation	Factor A	(1.92/hrt/L)/(S･Sqrt(1+2L/W))	A	-	-	0.4311	0.4311	0.2463	0.2463	0.2463	0.2463	0.2463	0.2463	0.4311
		Factor B	2.67/(S+2)/(1+L/4W)	B	-	-	0.2670	0.2670	0.2670	0.2670	0.2670	0.2670	0.2670	0.2670	0.2670
		2*A	2*A	2A	-	-	0.862	0.862	0.493	0.493	0.493	0.493	0.493	0.493	0.862
	Judgment		2A≦B	-	-	-	NG	NG	NG	NG	NG	NG	NG	NG	NG
	Check for the case of 2A>B														
	Result of calculation	Maximum compressive stress		σs	Mpa	-	6.48	6.48	6.48	6.48	6.48	6.48	6.48	6.48	6.48
		Limit of compressive stress	GS/(2A-B) or GS/(A-B	σsa	Mpa	-	11.3	11.3	29.9	29.9	-325.7	29.9	29.9	29.9	11.3
		Judgment	σs≦GS/(A-B)	-	-	-	OK	OK	OK	OK	OK	OK	OK	OK	OK
	Comprehensive Judgment			—	—	-	OK	OK	OK	OK	OK	OK	OK	OK	OK
Check of the reinforcement steel	Working stress	Maximum compressive stress		σs	m2	-	6.48	6.48	6.48	6.48	6.48	6.48	6.48	6.48	6.48
		Compressive stress due to live load		σl		-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Necessary thickness	For maximum compressive stress	3hmaxσs/Fy	reqhs1		-	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
		For compressive stress due to live load	2.0hmaxσL/ΔF_TH	reqHs2		-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
		Necessary thickness		reqhs		-	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
	Judgment		hs≧reqhs	-		-	OK	OK	OK	OK	OK	OK	OK	OK	OK

Shape of Elastomeric Bearimg　　　　Information Only

					SP1	SP2	SP3	SP4	SP5	SP6	SP7	SP8	SP9	SP10
					A1	P1		P2		P3		P4		P5
Dimension	Longitudinal	-	L0	mm	-	350	350	350	350	350	350	350	350	350
	Transversal	-	W0	mm	-	600	600	600	600	600	600	600	600	600
Outside Elastomeric			tso	mm	-	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0
Using dimension for design	Longitudinal	-	L	mm	-	340	340	340	340	340	340	340	340	340
	Transversal	-	W	mm	-	590	590	590	590	590	590	590	590	590
Thickness of elastomer	Thickness of one layer		te	mm	-	16	16	16	16	16	16	16	16	16
	Number of layers		ns	nos	-	7	7	4	4	4	4	4	4	7
	Total thickness of elastomer	te*ne	Σte	mm	-	112	112	64	64	64	64	64	64	112
	Thickness of top layer		hrit	mm	-	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
	Thickness of bottom layer		hrib	mm	-	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
Thickness of steel plate	Thickness of medium steel plate		tsi	mm	-	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0
	Thickness of top steel plate		tsT	mm	-	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0
	Thickness of bottom steel plate		tsB	mm	-	3	3	3	3	3	3	3	3	3
	Total thickness of steel plate	tsi*(ne-1)+tsT+tsB	Σts	mm	-	24	24	15	15	15	15	15	15	24
Thickness of Manufacture			Σts	mm	-	141	141	84	84	84	84	84	84	141

Pot bearing





2. General Design Condition

2. 1. Design Requirments

The method of steel-reinfoced elastmeric bearings are uesed Method B.Design requirements are showed below table.The design stage is consided the service limit stgaeI.

Design Requrement of steel reinforced-elasymeric bearing Table 2. 1

Item	Contents	Article number of LRFD
Design Rotation	- The maximum factored service rotation due to total load θs. Plus allowance for uncertainties $\theta unk=\pm0.0005$rad	14.4.2.1
Maximum compressive stress and Compressive stress due to live load	- In any elastomeric bearing layer, the average compressive stress at service stress limit stage and due to live load shall be less that limit stress.	14.7.5.3.2
Compressive defection	- Instantaneous deflection due to live load shall be less than 3mm and using this value ,rotation capacity can be estimated. - Long-term dead load deflections should be considered where joints and seals between sections of the bridge rest on bearings of different design and when estimating redistribution of force in continuous bridges caused by settlement.	14.7.5.3.3
Shear Deflection	- The maximum horizontal displacement of the bridge superstructure,Δo shall be taken as 65 percent of the design thermal movement range ,Δt,compputented in accordance with the temperature range combined with movements caused by creep, shrinkage and post-tensioning. - The maximum shear deformation of the bearing, at the service state,Δs shall be taken as Δmodified to account for the substructure stiffness and construction procedures. - The total elastomer thickness ,hrt,shall be more than twice design-deformation($hrt \geqq 2*\Delta s$).	14.7.5.3.4
Combined Compression and Rotation	- Rotation shall be taken as the maximum sum of the effects of initial lack of parallelism and subsequent girder end rotation due to imposed loads and movements. - Bearings shall be designed that uplift dose not occur under any combination of loads and corresponding rotation. Plus allowance for uncertainties $\theta unk=\pm0.0005$rad	14.7.5.3.5
Stability of Elastomeric Bearings	- The bearing bad should be designed to prevent instability at service limit state load combinations by limiting the average compressive stress to one-half estimated buckling stress.	14.7.5.3.6
Tensile stress of inner Reinforcing plate	- Tensile stress of inner steel plates due to max vertical reaction must be less than limited stress and fatigue stress.	14.7.5.3.7
Horizontal Force	- Horizontal forces and movements induced in the the bridge by restraint of movement at the bearings shall be determined using the movements and bearing characteristics - This horizontal forces shall be considered at structural design.	14.6.3.1

2. General Design Condition

2. 2. Design Specification

The applicable design standerd for the bearing is showen bellow.

- 22 TCN 272-05 Specifications for Bridge Design
- AASHTO LRFD Bridge design Specification ,Fourth Edition 2007
- AASHTO LRFD Bridge Construction Specifications
- AASHTO Materials Specifications

2. 3. Load Combination

Load combination in Service Limit State is called Service and shown on below table

Load Combintion for the Elastomeric Bearing Table 2. 2

	DC	DW	EL SP	LL IM BR	WS	WL	TU CR SH	TG	SE	EQ
Service 1	1.0	1.0	1.0	1.0	0.3	1.0	1.2/1.0	1.2/1.0	0.5	-

Load combination of AASHTO LRFD

The reaction and movement caused by load combination of Strength and Extreme State become dominated and shown as next table.

Table 2. 3

	DC	DW	EL SP	LL IM BR	WS	WL	TU CR SH	TG	SE	EQ
Strength1	1.25/0.9	1.5/0.65	1.0	1.75	-	-	1.2/0.5	-	1.00	-
Strength2	1.25/0.9	1.5/0.65	1.0	1.35	-	-	1.2/0.5	-	1.00	-
Strength3	1.25/0.9	1.5/0.65	1.0	-	1.40	-	1.2/0.5	-	1.00	-
Strength4	1.25/0.9	1.5/0.65	1.0	-	-	-	1.2/0.5	-	-	-
Strength5	1.25/0.9	1.5/0.65	1.0	1.35	0.40	1.00	1.2/0.5	-	1.00	-
Extreme1	1.25/0.9	1.5/0.65	1.0	-	-	-	-	-	-	1.00
Extreme4	1.25/0.9	1.5/0.65	-	-	1.40	-	-	-	-	1.00

2. General Design Condition

2. 4. Material Properties And Allowable Values

2. 4. 1. Material Properties

(1) Rubber

Type of materials ; Natural rubber or Neoprene rubber
Hardness(ShoreA Durmeter) ; Hardness "60" is slected
Design code ; AASHTO LRFD Bridge Construction Specifications
and AASHTO Materials Specification M251
Minimum low-temperture elastomer grade 0
Minimum low-temperture elastomer grade 0

Material Properties Table 2. 4

				Hardness(Shore A)			Remark
				50	60	70	D2240
Natural Rubber	Shera Modulas@23pc(Mpa)	range		066~0.90	0.90~1.38	1.38~2.07	A_table14.7.6.2-1
		design value		-	1.0	-	Engineer selected
	Creep defection@25years divided by initial deflection			0.25	0.35	0.45	A_table14.7.6.2-1
	Tensile Strength,Minimum		Mpa	15.5	15.5	15.5	D412
	Ultimate		%	450	400	300	
Neoprene Rubber	Shera Modulas@23pc(Mpa)	range		066~0.90	0.90~1.38	1.38~2.07	A_table14.7.6.2-1
		design value		-	1.0	-	Engineer selected
	Creep defection@25years divided by initial deflection			0.25	0.35	0.45	A_table14.7.6.2-1
	Tensile Strength,Minimum		Mpa	15.5	15.5	15.5	D412
	Ultimate Elongation,Minimum		%	400	350	300	

(2) Steel material

Physical constant of steel materials Table 2. 5

Items	Sign	Unit	Design value	Reference
Elastic coefficient	Es	Mpa	$2.0x10^{5}$	
Poisson ratio	ν	-	0.3	
Linear expansion coefficient	εct	-	$12.0x10^{-6}$	

TCXDVN 338:2005 Rated strength and caluculates yeild strength of compound steel Table 2. 6

Type of Materials	Steel Grade	Sign	Steel bar's thickness	Yielding strength	Caluculates Yield Strength	Tensile strength	Fatigu Stress
			t1≦t<t2(mm)	Mpa	Mpa	Mpa	Mpa
Steel Plate	10Mn2Si	SS345	t ≦ 20	360	345	510	165
			20 <t≦ 30	350	335	500	
			30 <t≦ 60	340	325	480	
				-	-	-	-
Anchor Bar	CB300-T	-	-	300	300	440	-

(3) Conrete of Bearing Seat

Concrete ; fck = 30 Mpa Table 2. 7

Items	Sign	Unit	Design value	Reference
Strength of basic design	fck	Mpa	30	
Elastic coefficient	Ec	Mpa	27,700	
Poisson ratio	ν	-	0.2	
Linear expansion coefficient	εct	1/°C	1.08.E-05	
Ultimate strain	εcu	-	0.003	
Adhesion stress	τpu	-		

2. General Design Condition

2. 4. 2. Limitation

(1) Rubber bearing

Allowable value of rubber bearing Table 2. 8

Items			Symbol	Unit	Allowable value	Reference
Compressive stress	For Subject to Shear Deformation	Maximum	σ_s	Mpa	1.66GS≦11.0Mpa	S1 is primary shape coefficient
		Live Load	σ_L	Mpa	0.66GS	
	For Subject to Fix Against Shear Deformation	Maximum	σ_s	Mpa	2GS≦12.0Mpa	
		Live Load	σ_L	Mpa	1.00GS	
Shear strain	Service		Δs	mm	0.5hrt	hrt is total elastmeric thickness
	Strength/ Extreme			-	-	
Combined Compression and Rotation	Minimum Compression		σ_s	Mpa	$1.0GS(\theta s/n)(B/hri)^2$	Check for Uplift
	Minimum Compression	For Subject to Shear Deformation	σ_s	Mpa	$1.875GS(1-0.20(\theta s/n)(B/hri)^2)$	For Requtangular Bearings
		For Subject to Fix Against Shear Deformation	σ_s	Mpa	2GS≦12.0Mpa	
Tensile stress	Service			Mpa	0	

(2) Steel material

Limit stress of the Steel p.Pate in the Elastmeric Bearing Table 2. 9

Type of stress		Basic Yielding point	Limt tensile stress at service limit stage Fy	Limt tensile stress at fatigue limit stage ΔF_{TH}
Type of steel materials	Scope of plate	Mpa	Mpa	Mpa
A709M g250	t ≦ -	250	250	165

Limit Shear stress of Anckor Bar Table 2. 10

			CB300-T	Remark
			Mpa	
Yield tensile stress		fsy	300	-
Shear stress at All limit States	Basic value	τsyo	173	fsy/√ fsy
	Design Value	τsy	170	

2. General Design Condition

(3) Concrete

Limit bearing stress and Push-off shear stress of Concrete Table 2. 11

Items	Sign	Unit	Design value	Reference
Strength of basic design	fck	Mpa	30	
Limit bearing stress	σcb	Mpa	25.5	0.85f'c
Nominal punching stress	σcp	MPa	1.80	$0.328\sqrt{f'c}$

(4) Adhesion stress between mortar and steel bar

Allowable adhesion stress between mortar and steel bar Table 2. 12

Strength of basic design	fck	Mpa	30
Round bar	30	Mpa	1.0
Deformed bar	30	MPa	2.0

The strength of mortar shall be stronger than concrete strength supporting bearing

2.5. Rteaction and Displacement

2.5.1. Reaction

Reaction of T Girder

			Exteria1	Interia	Extria2	Total	Load Factor: Service	Strength I max	Strength I min	Extreme I max	Extreme I min
Number of Girder		nos	1	4	1	6	–	max	min	max	min
Girder	Rdcg	kN	670	670	670	4020	-	-	-	-	-
Deck Slab	Rdcs	kN				0	-	-	-	-	-
Co_Parapet	Rdcb	kN				0	-	-	-	-	-
Total	Rdc	kN	670	670	670	4020	1.0	1.25	0.9	1.25	0.9
Pavement ,etc	Rdw	kN	75.6	75.6	75.6	453.6	1.0	1.5	0.65	1.5	0.65
Live Load	Rlmax	kN	547.3	322	404.5		1.0	1.75	1.0	1.75	0.0
	Rlmin	kN	0	0	0		1.0	1.75	1.0	1.75	0.0
Addisional dead load	Rd'	kN				107.28	Not Considered the desgn of bearings				
All Dead Load	Rd	kN	745.6	745.6	745.6	4580.9					
Servie I	Rsv_max	kN	1292.9	1514.4	1056.6						
	Rsv_min	kN	745.6	950.9	652.1						
Strength I	Rst_max	kN	1908.7	974.1	1658.8						
	Rst_min	kN	652.1	950.9	652.1						
Extreme I	Rex_max	kN	1908.7	1658.8	652.1						
	Rex_min	kN	652.1	652.1	652.1						

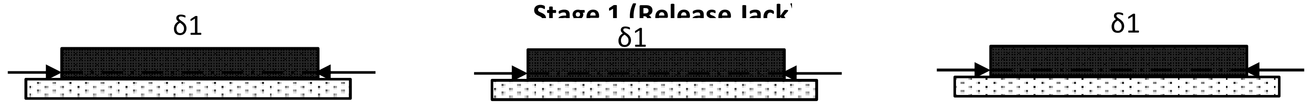
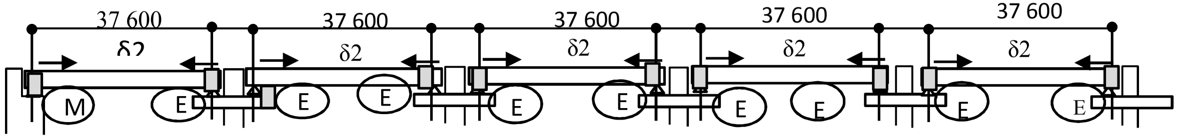
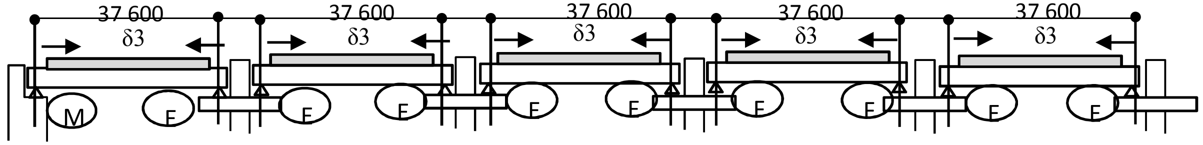
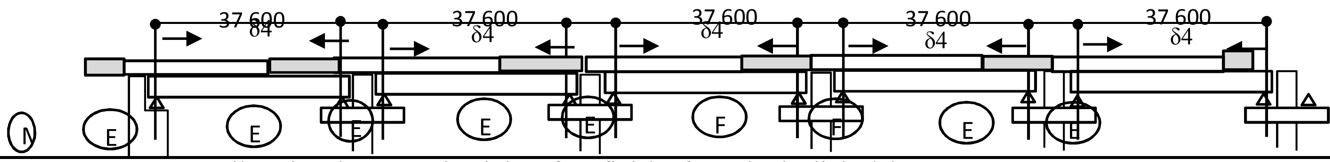
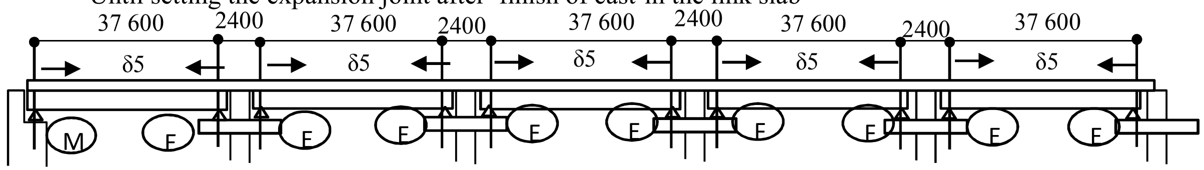
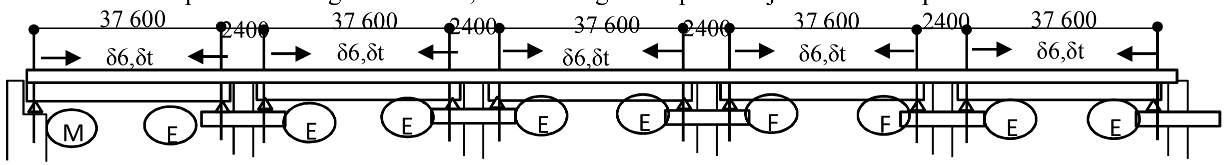
670	2680	670		4020
Addisonal Dead Load on Pier Head				
Deck Slab	24.5	kN/m3		
0.25	12.47	1.70/2		64.92
Co_Parapet				
26.27		1.70/2		22.3295
			Toatal Rcd'=	87.25
Pavement	22.5	kN/m3		
0.084	12.47	1.70/2		20.03

Design Reaction for Caluculation of Bearings

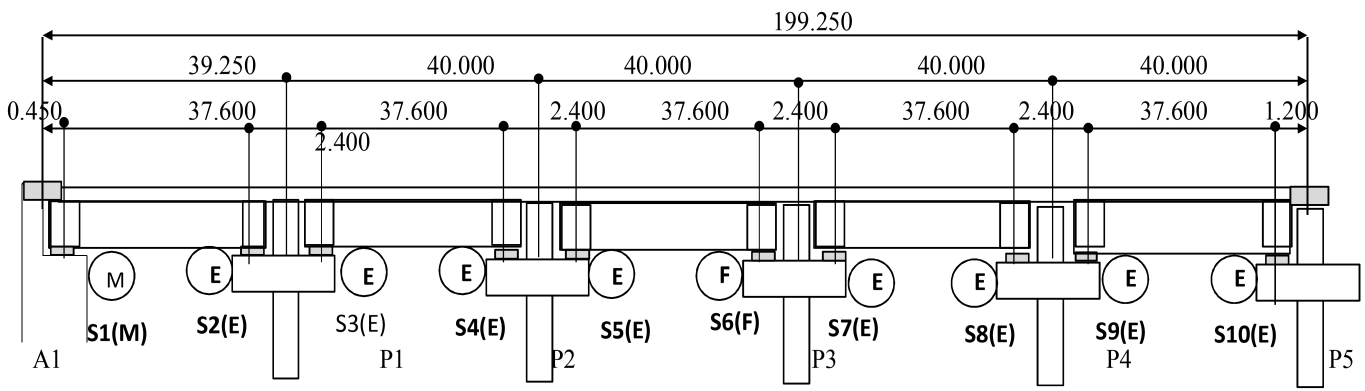
					S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
					A1	S1	S1	S2	S2	S3	S3	S4	S4	S5
Number of Bearing			Nb	nos	6	6	6	6	6	6	6	6	6	6
Total Dead load	Dead load		Rd	kN	4581.0	4581.0	4581.0	4581.0	4581.0	4581.0	4581.0	4581.0	4581.0	4581.0
	Live Load	Maxmam	Rlmax	kN	373.3	373.3	373.3	373.3	373.3	373.3	373.3	373.3	373.3	373.3
		Minimum	Rlmin	kN	0.0	0.0	0.0	0.0	1.0	1.8	1.0	1.8	0.0	0.3
	Wind load			kN	0	0	0	0	0	0	0	0	0	0
Reaction for Bearing	Dead load	For Concrete	Rdc	kN	670	670	670	670	670	670	670	670	670	670
		Surface,etc	Rdw	kN	80	80	80	80	80	80	80	80	80	80
		Total	Rd	kN	750	750	750	750	750	750	750	750	750	750
	Live Load	Maxmam	Rlmax	kN	550	550	550	550	550	550	550	550	550	550
		Average	Rlmin	kN	373	373	373	373	373	373	373	373	373	373
		Minimum	Rlmin	kN	0	0	0	0	0	0	0	0	0	0
	Service I	Max_Rmax	Rsmax	kN	1,300	1,300	1,300	1,300	1,300	1,300	1,300	1,300	1,300	1,300
		Max Ave_Rmax	Ramax	kN	1,123	1,123	1,123	1,123	1,123	1,123	1,123	1,123	923	923
		Min_Rmin	Rsmin	kN	750	750	750	750	750	750	750	750	750	750
	Strength	Max_Rmax	Rmax	kN	1,910	1,910	1,910	1,910	1,910	1,910	1,910	1,910	1,910	1,910
		Min_Rmin	Rmin	kN	652	652	652	652	652	652	652	652	652	652
Reaction of Taotal Dead Load				kN	4,581	4,581	4,581	4,581	4,581	4,581	4,581	4,581	4,581	4,581
Weight of superstructure				kN	45,810									
Ratio					0.1000	0.1000	0.1000	0.1000	0.1000	0.1000	0.1000	0.1000	0.1000	0.1000
Method of desitribution				—	Mov	E	E	E	E	Fix	E	E	E	E
Number bearing				nos	6	6	6	6	6	6	6	6	6	6

2.5.2. Displacement Calculation Method

The shear deformation is considered the construction procedure and structural syster Considering that mater is describe bellow table.

Construction Stage	Outline Drawing	Structural System
Step 1	Until the Super T girders are elected after presitressings are transferred.(in the cast-in yard) 	Simple Beam
Step 2	Until finish of cast-in the cross beam after finish of election of the girder 	Simple Beam
Step 3	Until finish of cast-in the deck slab after finish of cast-in the cross beam 	Simple Beam
Step 4	Until finish of cast-in link slab after finish of cast-in the deck slab at medium. 	Simple Beam
Step 5	Until setting the expansion joint after finish of cast-in the link slab 	Continuance Beam
Stap6 and temperature eff	Until creep and shrinkage is finished, after setting the expansion joint. And temperature effect. 	Continuance Beam

Displace ment of each span



Elastic shortening by 1.1 priestesses $\varDelta le = -(Pt/Ect/Ac) = -(\sigma ct/Ect)*l$

Quantity of movement by the creep of 1.2 concrete $\varDelta l_{\varphi} = -(Pt/Ec/Ac)' = -(\sigma ct/Ec)*\varphi*li$

Quantity of movement by the dry shrinkage of 1.3 concrete $\varDelta lsh = = -\varepsilon sh*li$

Quantity of movement by 1.4 temperature changes $\varDelta lt = \pm \varDelta T*\alpha*l$

	Ect	Ec
	Mpa	Mpa
Super T Girder	30 345	35 700
Deck Spab	-	29 900

Where

- $\varDelta le$: Elastic shortening by the prestressing force (mm)
- $\varDelta l_{\varphi}$: Quantity of movement by the creep of the concrete (mm)
- $\varDelta lsh$: Quantity of movement by the dry shrinkage of the concrete (mm)
- $\varDelta lt$: Quantity of movement by the temperature change
- Pt : Mean tension to act on PC steel materials just after presto Lessing
- Ac : A cross section of the concrete
- σct : A mean prestress just after the prestress introduction
- Ect : A modulus of elasticity of the prestress introduction concrete
- Ec : A modulus of elasticity of the concrete
- li Each Span Length
- φ : A creep coefficient of the concrete
- εsh : A basic dry shrinkage degree of the concrete
- α : A linear expansion coefficient α = 1.08E-05 =11 $*10^{-o}$
- $\varDelta T$: A temperature change $\varDelta T(+)$= +22 * 1.15 Degree $\varDelta T$= 10 ~ 47 Degree

$\varDelta T(-)$= - 15 * 1.15 Degree Tmean= 25 Degree

Coefficient of Creep and Shrinkage

	Creep		Shrinkage	
	Composite φc(t2,t1)		Composite εc(t2,t1)(x10-6	
Stage1	φg(3 ,055)	1.136	shg(3 ,055)	-79.1
Stage2	φg(100 ,120)	0.142	shg(55 ,085)	-17.3
Stage3	φg(120 ,180)	0.426	shg(85 ,110)	-22.0
Stage4	φg(180 ,240)	0.155	shg(110 ,160)	-14.3
Stage5	φg(240 ,285)	0.233	shg(160 ,285)	-48.6
Stage6	φs(28 ,51000)	0.624	shg(2851 ,100)	-89.4
Total		2.716		-270.8

Movements of Each Span

The expansion assumes the course that a beam shorts plus.

					1 Span	2 Span	3 Span	4 Span	5 Span	6 Span	7 Span	8 Span	9 Span	0 Span	0 Span
					1	2	3	4	5	6	7	8	9	10	11
Step 1		Span Length	Ls	m	37.60	0.00	37.60	0.00	37.60	0.00	37.60	0.00	37.60	0.00	0.00
	Elastic Shortening due to Prestressing	Modification Factor	ke		1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
		Expansion Length	l	m	37.6	0	37.6	0	37.6	0	37.6	0	37.6	0	0
		Concrete Stress of Prestressing	σct	N/mm2	10.8	0.0	10.8	0.0	10.8	0.0	10.8	0.0	10.8	0.0	10.8
		Elastic Modulus at Prestressing	Ect	N/mm2	3.0E+04	3.0E+04	3.0E+04	3.0E+04	3.0E+04	3.0E+04	3.0E+04	3.0E+04	3.0E+04	3.0E+04	3.0E+04
		Unit Shortening	k=Δle/Ls	1/m	0.357	0.000	0.357	0.000	0.357	0.000	0.357	0.000	0.357	0.000	0.357
		Shortening	Δle	mm	13.4	0.0	13.4	0.0	13.4	0.0	13.4	0.0	13.4	0.0	0.0
	Shortening due to Creep	Average Compressive Stress of Concrete	σct	N/mm2	10.8	0.0	10.8	0.0	10.8	0.0	10.8	0.0	10.8	0.0	10.8
		Coefficient of Creep	φ	—	1.14	0.00	1.14	0.00	1.14	0.00	1.14	0.00	1.14	0.00	1.14
		Elastic Modulus	Ec	N/mm2	3.6E+04	3.6E+04	3.6E+04	3.6E+04	3.6E+04	3.6E+04	3.6E+04	3.6E+04	3.6E+04	3.6E+04	3.6E+04
		Unit Shorting	k=Δlφ/Ls	1/m	0.345	0.000	0.345	0.000	0.345	0.000	0.345	0.000	0.345	0.000	0.345
		Shortening	$⊿l_{φ0}$	mm	13.0	0.0	13.0	0.0	13.0	0.0	13.0	0.0	13.0	0.0	0.0
	Shortening due to Shrinkage	Coefficient of Shortening	$εsh \times 10^{-6}$	—	79	0.0	79	0.0	79	0.0	79	0.0	79	0.0	79
		Unit Shorting	k・l	mm	0.079	0.000	0.079	0.000	0.079	0.000	0.079	0.000	0.079	0.000	0.079
		Unit Shorting	⊿lsh0	mm	3.0	0.0	3.0	0.0	3.0	0.0	3.0	0.0	3.0	0.0	0.0
		Sum of Shortening	⊿l0	mm	29.4	0.0	29.4	0.0	29.4	0.0	29.4	0.0	29.4	0.0	0.0
Step 2		Span Length	Ls	m	37.60	0.00	37.60	0.00	37.60	0.00	37.60	0.00	37.60	0.00	0.00
	Shortening due to Creep	Average Compressive Stress of Concrete	σct	N/mm2	10.3	0.0	10.3	0.0	10.3	0.0	10.3	0.0	10.3	0.0	10.3
		Coefficient of Creep	φ	—	0.142	0.000	0.142	0.000	0.142	0.000	0.142	0.000	0.142	0.000	0.142
		Elastic Modulus	Ec	N/mm2	3.6E+04	3.6E+04	3.6E+04	3.6E+04	3.6E+04	3.6E+04	3.6E+04	3.6E+04	3.6E+04	3.6E+04	3.6E+04
		Unit Shorting	k=Δlφ/Ls	1/m	0.041	0.000	0.041	0.000	0.041	0.000	0.041	0.000	0.041	0.000	0.041
		Shortening	$⊿l_{φ0}$	mm	1.5	0.0	1.5	0.0	1.5	0.0	1.5	0.0	1.5	0.0	0.0
	Shortening due to Shrinkage	Coefficient of Shortening	$εsh \times 10^{-6}$	—	17	0.0	17	0.0	17	0.0	17	0.0	17	0.0	17
		Unit Shorting	k・l	mm	0.017	0.000	0.017	0.000	0.017	0.000	0.017	0.000	0.017	0.000	0.017
		Unit shorting	⊿lsh0	mm	0.7	0.0	0.7	0.0	0.7	0.0	0.7	0.0	0.7	0.0	0.0
		Sum of Shortening	⊿l0	mm	2.2	0.0	2.2	0.0	2.2	0.0	2.2	0.0	2.2	0.0	0.0

					1 Span	2 Span	3 Span	4 Span	5 Span	6 Span	7 Span	8 Span	9 Span	0 Span	0 Span
					1	2	3	4	5	6	7	8	9	10	11
Step 3		Span Length	Ls	m	37.60	0.00	37.60	0.00	37.60	0.00	37.60	0.00	37.60	0.00	0.00
	Shortening due to Creep	Average Compressive Stress of Concrete	σct	N/mm2	10.3	0.0	10.3	0.0	10.3	0.0	10.3	0.0	10.3	0.0	10.3
		Coefficient of Creep	φ	—	0.43	0.00	0.43	0.00	0.43	0.00	0.43	0.00	0.43	0.00	0.43
		Elastic Modulus	Ec	N/mm2	3.6E+04	3.6E+04	3.6E+04	3.6E+04	3.6E+04	3.6E+04	3.6E+04	3.6E+04	3.6E+04	3.6E+04	3.6E+04
		Unit shorting	k=Δlφ/Ls	1/m	0.123	0.000	0.123	0.000	0.123	0.000	0.123	0.000	0.123	0.000	0.123
		Shortening	$⊿l_{φ0}$	mm	4.6	0.0	4.6	0.0	4.6	0.0	4.6	0.0	4.6	0.0	0.0

	Shortening due to Shrinkage	Coefficient of Shortening	εsh x10^{-6}	—	22	0.0	22	0.0	22	0.0	22	0.0	22	0.0	22
		Unit Shortening	k・l	mm	0.022	0.000	0.022	0.000	0.022	0.000	0.022	0.000	0.022	0.000	0.022
		Unit Shortening	⊿lsh0	mm	0.8	0.0	0.8	0.0	0.8	0.0	0.8	0.0	0.8	0.0	0.0
	Sum of Shortening		⊿l0	mm	5.4	0.0	5.4	0.0	5.4	0.0	5.4	0.0	5.4	0.0	0.0
Step 4	Span Length		Ls	m	37.60	0.00	37.60	0.00	37.60	0.00	37.60	0.00	37.60	0.00	0.00
	Shortening due to Creep	Average Comprecive Stress of Concrete	σct	N/mm2	10.3	0.0	10.3	0.0	10.3	0.0	10.3	0.0	10.3	0.0	10.3
		Coefficient of Creep	φ	—	0.15	0.00	0.15	0.00	0.15	0.00	0.15	0.00	0.15	0.00	0.15
		Elastic Modulus	Ec	N/mm2	3.6E+04	3.6E+04	3.6E+04	3.6E+04	3.6E+04	3.6E+04	3.6E+04	3.6E+04	3.6E+04	3.6E+04	3.6E+04
		Unit Shorting	k=Δlφ/Ls	1/m	0.045	0.000	0.045	0.000	0.045	0.000	0.045	0.000	0.045	0.000	0.045
		Shortening	$⊿l_{φ0}$	mm	1.7	0.0	1.7	0.0	1.7	0.0	1.7	0.0	1.7	0.0	0.0
	Shortening due to Shrinkage	Coefficient of Shortening	εsh x10^{-6}	—	14	0.0	14	0.0	14	0.0	14	0.0	14	0.0	14
		Unit Shorting	k・l	mm	0.014	0.000	0.014	0.000	0.014	0.000	0.014	0.000	0.014	0.000	0.014
		Unit Shorting	⊿lsh0	mm	0.5	0.0	0.5	0.0	0.5	0.0	0.5	0.0	0.5	0.0	0.0
	Sum of Shortning		⊿l0	mm	2.2	0.0	2.2	0.0	2.2	0.0	2.2	0.0	2.2	0.0	0.0
Step 5	Span Length		Ls	m	37.60	2.40	37.60	2.40	37.60	2.40	37.60	2.40	37.60	0.00	0.00
	Shortening due to Creep	Average Compressive Stress of Concrete	σct	N/mm2	6.9	0.0	6.9	0.0	6.9	0.0	6.9	0.0	6.9	0.0	6.9
		Coefficient of Creep	φ	—	0.23	0.00	0.23	0.00	0.23	0.00	0.23	0.00	0.23	0.00	0.23
		Elastic Modulus	Ec	N/mm2	3.6E+04	3.6E+04	3.6E+04	3.6E+04	3.6E+04	3.6E+04	3.6E+04	3.6E+04	3.6E+04	3.6E+04	3.6E+04
		Unit Shorting	k=Δlφ/Ls	1/m	0.045	0.000	0.045	0.000	0.045	0.000	0.045	0.000	0.045	0.000	0.045
		Shortening	$⊿l_{φ0}$	mm	1.7	0.0	1.7	0.0	1.7	0.0	1.7	0.0	1.7	0.0	0.0
	Shortening due to Shrinkage	Coefficient of Shortening	εsh x10^{-6}	—	48.6	48.6	48.6	48.6	48.6	48.6	48.6	48.6	48.6	48.6	48.6
		Unit Shortning	k・l	mm	0.049	0.049	0.049	0.049	0.049	0.049	0.049	0.049	0.049	0.049	0.049
		Unit Shortening	⊿lsh0	mm	1.8	0.1	1.8	0.1	1.8	0.1	1.8	0.1	1.8	0.0	0.0
	Sum of Shortening		⊿l0	mm	3.5	0.1	3.5	0.1	3.5	0.1	3.5	0.1	3.5	0.0	0.0

					1 Span	2 Span	3 Span	4 Span	5 Span	6 Span	7 Span	8 Span	9 Span	0 Span	0 Span
					1	2	3	4	5	6	7	8	9	10	11
Step 6	Span Length		Ls	m	37.60	2.40	37.60	2.40	37.60	2.40	37.60	2.40	37.60	2.40	0.00
	Shortening due to Creep	Average Compressive Stress of Concrete	σct	N/mm2	6.9	0.0	6.9	0.0	6.9	0.0	6.9	0.0	6.9	0.0	6.9
		Coefficient of Creep	φ	—	0.62	0.62	0.62	0.62	0.62	0.62	0.62	0.62	0.62	0.62	0.62
		Elastic Modulus	Ec	N/mm2	3.6E+04	3.6E+04	3.6E+04	3.6E+04	3.6E+04	3.6E+04	3.6E+04	3.6E+04	3.6E+04	3.6E+04	3.6E+04
		Unit Shortening	k=Δlφ/Ls	1/m	0.121	0.000	0.121	0.000	0.121	0.000	0.121	0.000	0.121	0.000	0.121
		Shortening	$\varDelta l_{\varphi 0}$	mm	4.6	0.0	4.6	0.0	4.6	0.0	4.6	0.0	4.6	0.0	0.0
	Shortening due to Shrinkage	Coefficient of Shortening	$\varepsilon sh \times 10^{-6}$	—	89	89.4	89	89.4	89	89.4	89	89.4	89	89.4	89
		Unit Shortening	k･l	mm	0.089	0.089	0.089	0.089	0.089	0.089	0.089	0.089	0.089	0.089	0.089
		Shortening	⊿lsh0	mm	3.4	0.2	3.4	0.2	3.4	0.2	3.4	0.2	3.4	0.2	0.0
	Sum of Shortening		⊿l0	mm	7.9	0.2	7.9	0.2	7.9	0.2	7.9	0.2	7.9	0.2	0.0
Temperature Effect	Span Length		Ls	m	37.60	2.40	37.60	2.40	37.60	2.40	37.60	2.40	37.60	2.40	0.00
	Temperature Effect(Increase 15%)	Rise	T1	deg	+25.3	+25.3	+25.3	+25.3	+25.3	+25.3	+25.3	+25.3	+25.3	+25.3	+25.3
		Fall	T2	deg	-17.3	-17.3	-17.3	-17.3	-17.3	-17.3	-17.3	-17.3	-17.3	-17.3	-17.3
		Coefficient of thermal expansion	$\varepsilon t x 10^{-6}$	mm	10.8	10.8	10.8	10.8	10.8	10.8	10.8	10.8	10.8	10.8	10.8
		Unit Expansion	kt+	1/m	-0.273	-0.273	-0.273	-0.273	-0.273	-0.273	-0.273	-0.273	-0.273	-0.273	-0.273
		Unit Shortening	kt-	1/m	0.186	0.186	0.186	0.186	0.186	0.186	0.186	0.186	0.186	0.186	0.186
		Expansion	Δlt+	mm	-10.3	-0.7	-10.3	-0.7	-10.3	-0.7	-10.3	-0.7	-10.3	-0.7	0.0
		Shortening	Δlt-	mm	7.0	0.4	7.0	0.4	7.0	0.4	7.0	0.4	7.0	0.4	0.0
		Total movement	l	m	17.3	1.1	17.3	1.1	17.3	1.1	17.3	1.1	17.3	1.1	0.0

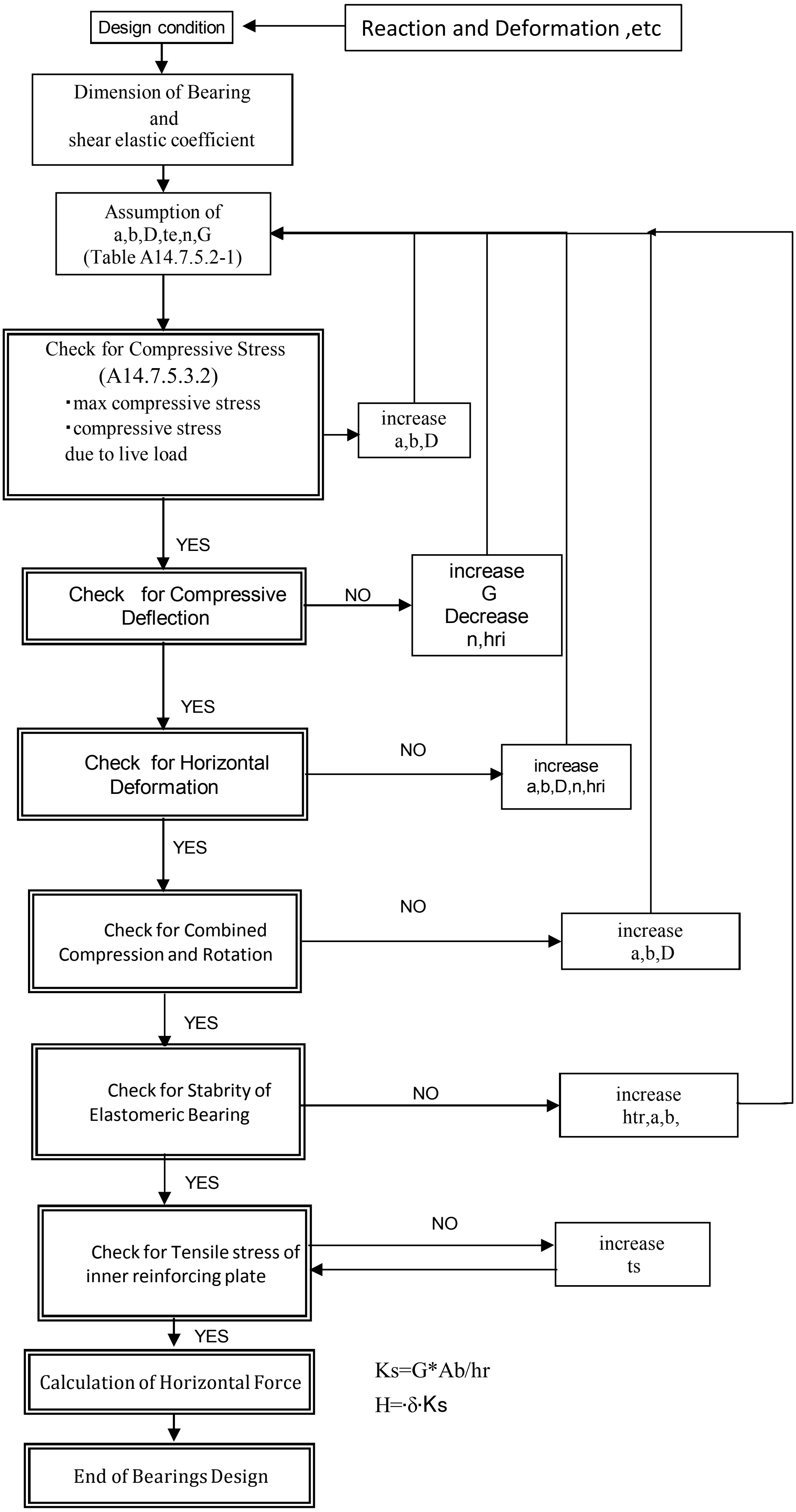
Average Prestress of Super T Girder

x	Super T girder						Composite girder							
	Pt	Ac	σct	aveσc	dx	dx*aveσc	Pt	Pe	(Pt+Pe)/2	Ac	σct	aveσc	dx	dx*aveσc
m	kN	m2	Mpa	Mpa	m		kN	kN	kN	m2	Mpa	Mpa	m	
0.000	0	1.628	0.00				0	0	0	1.973	0.00			
2.000	4837	0.635	7.62	3.81	2.00	7.62	4837	3628	4232.5	0.990	4.89	2.44	2.00	4.89
3.000	5611	0.635	8.84	8.23	1.00	8.23	5611	4208	4909.5	0.990	5.67	5.28	1.00	5.28
4.000	5611	0.635	8.84	8.84	1.00	8.84	5611	4208	4909.5	0.990	5.67	5.67	1.00	5.67
5.000	6385	0.635	10.06	9.45	1.00	9.45	6385	4789	5587	0.990	6.45	6.06	1.00	6.06
6.000	6579	0.635	10.36	10.21	1.00	10.21	6579	4934	5756.5	0.990	6.65	6.55	1.00	6.55
8.000	7353	0.635	11.58	10.97	2.00	21.94	7353	5514	6433.5	0.990	7.43	7.04	2.00	14.07
9.400	8127	0.635	12.80	12.19	1.40	17.06	8127	6095	7111	0.990	8.21	7.82	1.40	10.95
18.800	8127	0.635	12.80	12.80	9.40	120.31	8127	6095	7111	0.990	8.21	8.21	9.40	77.17
Sum					18.8	203.64							18.8	130.62
Average	σct=	10.83	Mpa	at release the jack					σct=	6.95	Mpa	Stage 5,6		
	η1=	0.95												
	σct=	10.29	Mpa	Stage 2 t0 4										

3. Elastomeric Bearing

3. 1 Design Method for Elastomeric Bearing

3. 1. 1. Flow Chart for Design of Elastomeric Bearing



Flow chart for Design of Elastomeric Bearing

3. 1. 2. Equation for Design of Elastomeric Bearing

(1) **Compressive Stress** A 14.7.5.3.2

- For bearings subject to shear deformation

  $\sigma s \leqq 1.66GS \leqq 11.0$ Mpa

  $\sigma_L \leqq 0.66GS$

- For bearings fixed against shear deformation

  $\sigma s \leqq 2.00GS \leqq 12.0$ Mpa

  $\sigma_L \leqq 1.00GS$

$$\sigma_s = \frac{R}{L \cdot W} \qquad \sigma_L = \frac{R_L}{L \cdot W} \qquad S_i = \frac{LW}{2h_{ri}(L+W)}$$

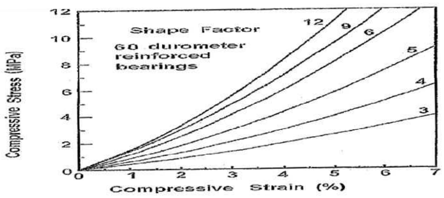
(2) **Compressive Deflection** A14.7.5.3.3

$\delta e = \Sigma \varepsilon_{Li} \cdot h_{ri}$

$\delta d = \Sigma \varepsilon_{di} \cdot h_{ri}$

$\delta_{lt} = \delta_d + a_{cr}\delta_d$

$$\varepsilon = \frac{\sigma}{6GS^2}$$



(3) **Shear Deformation**

$\Delta h_r \leqq 2\Delta_s$

(4) **Combined Compression and Rotation**

a. For uplift

$$\sigma_s > \sigma_{up\min} = 1.0GS\left(\frac{\theta_s}{n}\right)\left(\frac{B}{h_{ri}}\right)^2$$

b. For maximum compression

b1. Rectangular bearings subjected to shear deformation

$$\sigma_s < \sigma_{c\max} = 1.875GS\left[1 - 0.20\left(\frac{\theta_s}{n}\right)\left(\frac{B}{h_{ri}}\right)^2\right]$$

b2. Rectangular bearings fixed against shear deformation

$$\sigma_s < \sigma_{c\max} = 2.00GS\left[1 - 0.20\left(\frac{\theta_s}{n}\right)\left(\frac{B}{h_{ri}}\right)^2\right]$$

(5) **Stability of Elastomeric Bearing** A14.4.5.3.6

Bearing satisfying $2A \leqq B$(Eq-1) shall be considered stable, and no further investigation of stability. When the bearing is not satisfied,the buckling stress should be checked.

The bearing pad should be designed to prevent instability at the service limit state load combinations by limiting the average compressive to one-half the estimated buckling stress.

Equation -1 $2A \leqq B$ (Eq 1)

$$A = \frac{1.92\frac{h_{rt}}{L}}{\sqrt{1+\frac{2.0L}{W}}} \qquad B = \frac{2.67}{(S+2)\left(1+\frac{L}{4.0W}\right)}$$

For rectangular bearing not satisfying Eq.1, the stress due to the total load shall next equation.

- If the bridge deck is free to translate horizontal:

$$\sigma_s \leq \sigma cr = \frac{GS}{2A-B}$$

- If the bridge deck is fix to translate horizontal:

$$\sigma_s \leq \sigma cr = \frac{GS}{A-B}$$

(6) **Reinforcement**

Thickness of the steel reinforcement, hs, shall satisfy bellow equations. A14.7.5.3.7

- At the service limit state

$$h_s \geq \frac{3h_{\max}\sigma_s}{F_y}$$

- At the fatigue limit state

$$h_s \geq \frac{2h_{\max}\sigma_L}{\Delta F_{TH}}$$

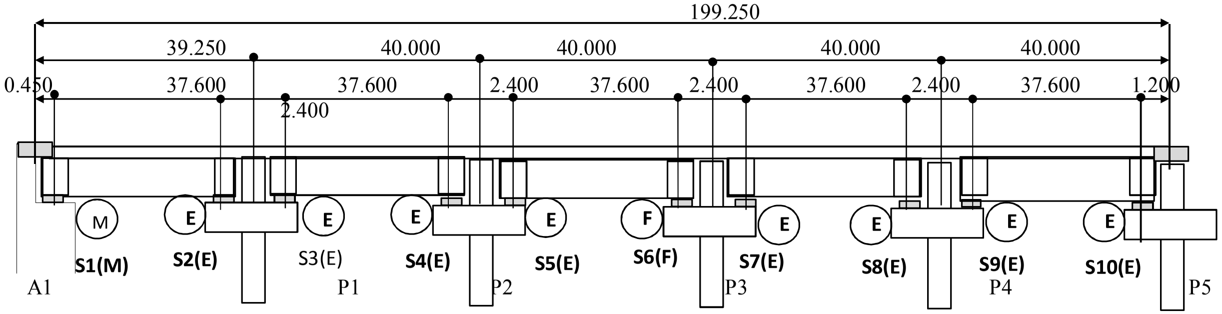
where

σs	:	Service average compressive due to total load	(Mpa)
σ_L	:	Service average compressive due to live load	(Mpa)
G	:	Shear modules of elastomer	(Mpa)
S	:	shape factor of thickness layer of the bearing	(-)
L	:	Length of longitudinal elastomeric bearing	(mm)
W	:	Width of transversal elastomeric bearing	(mm)
hri	:	Thickness of the i-th elastomer layer	(mm)
R	:	Total reaction of service limit state	(N)
RL	:	Reaction of live load at service limit sate	(N)
ε_{Li}	:	Instantaneous live load compressive strain in/th elastomer layer of a laminated bearin	(-)
δe	:	Instantaneous live load deflection	(mm)
hri	:	Thickness of i-th elastomeric layer in a laminated bearing	(mm)
δd	:	Initial dead load deflection	(mm)
a_{cr}	:	Creep deflection ith elastomeric layer in a laminated bearin a_{cr} = 0.35	(dim.)
δ_{lt}	:	Long-term dead load deflection ,including the effects of creep	(mm)
σ	:	Instantaneous live load compressive stress or dead load compressive stress in an individual elastomer layer of a laminated bearing	(Mpa)
σ_{upmin}	:	Minimum limit compressive stress for uplift requirement	(Mpa)
n	:	Number of interior layers, where interior layers are defined as those layers which are bonded on each face. Exterior layers are defined aas those layers which are bonded only on one face. When the thickness of the exterior layer of elastomer is more than one-half the thickness of an interior layer ,the parameter,n,may be increased by one-half for each such exterior layer.(for checking the rotation.)	(layer)
σ_{cmax}	:	Maximum limit compressive stress due to rotation	(Mpa)
B	:	Length of pad if rotation is about its transverse axis or width of pad if rotation is about its longitudinal axis.	(mm)
θ_s	:	Maximum service rotation due to the total load	(rad)
hrt	:	Total elastomer thickness	(mm)
Δs	:	Maximum total shear deformation of the elastomer at the service limit state	(mm)
σ_{cr}	:	Buckling stress of the bearing	(Mpa)
h_s	:	The thickness of the steel reinforcement	(mm)
F_y	:	Yield strength of steel reinforcement	(Mpa)
ΔF_{TH}	:	Constant amplitude fatigue threshold Category A	(Mpa)
hrr	:	Thickness of thickness elastomeric layer in elastomeric bearing	(mm)

3.2. Design Condition

Tra Bong Bridge
Supper Structure — Super T Girder

1.1.1.Span Arrangement



Total Number of the Span 11 Spans

	1	2	3	4	5	6	7	8	9	
Span Arrangement1	37.600 m	0.000 m	37.600 m	0.000 m	37.600 m	0.000 m	37.600 m	0.000 m	37.600 m	
Span Arrangement 2	37.600 m	2.400 m	37.600 m	2.400 m	37.600 m	2.400 m	37.600 m	2.400 m	37.600 m	

Name of Support and Distance from First Support to Each Support

Name of Support		S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
Name of Substructure		A1	P1	P1	P2	P2	P3	P3	P4	P4	P5
Distance 1		0.000 m	37.600 m	0.000 m	37.600 m	0.000 m	37.600 m	0.000 m	37.600 m	0.000 m	37.600 m
Distance 2		0.000 m	37.600 m	40.000 m	77.600 m	80.000 m	117.600 m	120.000 m	157.600 m	160.000 m	197.600 m
Support Condition	IS	1	0	0	0	0	2	0	0	0	0
	-	Mov	E	E	E	E	Fix	E	E	E	E

IS=0,Support by Elastic Spring IS -1; Movable Bearing IS=2;Fix

1.1.2.Seizmic Effect

A= 0.034 II S= 1.200

1.1.3. Reaction Table

				S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
				A1	P1	P1	P2	P2	P3	P3	P4	P4	P5
Total Reaction	Dead Load4			4,581	4,581	4,581	4,581	4,581	4,581	4,581	4,581	4,581	4,581
	Live Load max			0	0	0	0	0	0	0	0	0	0
	Live load min			0	0	0	0	0	0	0	0	0	0
	Maximum Reaction			4,581	4,581	4,581	4,581	4,581	4,581	4,581	4,581	4,581	4,581
Reaction for Bearing	Dead Load4		kN	750.0	750.0	750.0	750.0	750.0	750.0	750.0	750.0	750.0	750.0
	Live Load max		kN	550.0	550.0	550.0	550.0	550.0	550.0	550.0	550.0	550.0	550.0
	Live load min		kN	373.3	373.3	373.3	373.3	373.3	373.3	373.3	373.3	373.3	373.3
	Maximum Reaction		kN	1300.0	1300.0	1300.0	1300.0	1300.0	1300.0	1300.0	1300.0	1300.0	1300.0
Reaction of Dead Load			kN	4581.0	4581.0	4581.0	4581.0	4581.0	4581.0	4581.0	4581.0	4581.0	4581.0
Total Wight of Superstructure			kN	45810.0									
Ratio of Reaction				0.1000	0.1000	0.1000	0.1000	0.1000	0.1000	0.1000	0.1000	0.1000	0.1000
Method of Distribution of Horizontal Reaction			—	Mov	E	E	E	E	Fix	E	E	E	E
Number of Bearing			nos	6	6	6	6	6	6	6	6	6	6

E: Distribution Bearing M: Moving Bearing

1.3 Design Movement pre-shear ing for Elastomeric bearing

0 Pre-shearing is not considered.

1.3.1 Movements for each span

Number of Span				S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
Step 1	Length of Span	ls	m	37.600	0.000	37.600	0.000	37.600	0.000	37.600	0.000	37.600	0.000
	Until Election of Girder	dl1	mm	29.4	0.0	29.4	0.0	29.4	0.0	29.4	0.0	29.4	0.0
Step 2	Length of Span	ls	m	37.600	0.000	37.600	0.000	37.600	0.000	37.600	0.000	37.600	0.000
	Form Election of girder to Cast in of Cross Beam	dl2	mm	2.2	0.0	2.2	0.0	2.2	0.0	2.2	0.0	2.2	0.0
Step 3	Length of Span	ls	m	37.600	0.000	37.600	0.000	37.600	0.000	37.600	0.000	37.600	0.000
	Form Cast in of Cross Beam to Deck Slab Cast in	dl3	mm	5.4	0.0	5.4	0.0	5.4	0.0	5.4	0.0	5.4	0.0
Step 4	Length of Span	ls	m	37.600	0.000	37.600	0.000	37.600	0.000	37.600	0.000	37.600	0.000
	Form Deck Slab Cast in to Link Slab Cast in	dl4	mm	2.2	0.0	2.2	0.0	2.2	0.0	2.2	0.0	2.2	0.0
Step 5	Length of Span	ls	m	37.600	2.400	37.600	2.400	37.600	2.400	37.600	2.400	37.600	0.000
	Form Link Slab Cast in to Setting of Expansion Joint	dl5	mm	3.5	0.1	3.5	0.1	3.5	0.1	3.5	0.1	3.5	0.0
Step 6	Length of Span	ls	m	37.600	2.400	37.600	2.400	37.600	2.400	37.600	2.400	37.600	2.400
	Form Setting of Expansion Joint to After 30 years	dl6	mm	7.9	0.2	7.9	0.2	7.9	0.2	7.9	0.2	7.9	0.2
Temperature Effect	Length of Span	ls	m	37.600	2.400	37.600	2.400	37.600	2.400	37.600	2.400	37.600	2.400
	Expansion	Δlt+	mm	-10.3	-0.7	-10.3	-0.7	-10.3	-0.7	-10.3	-0.7	-10.3	-0.7
	Shortening	Δlt-	mm	7.0	0.4	7.0	0.4	7.0	0.4	7.0	0.4	7.0	0.4

1.3.2. Displacement of each bearind

				S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
				1	5	7	11	13	17	19	23	25	29
Step 2		dl2	mm	**2.19**	**0.00**	**1.33**	**-0.76**	**1.02**	**-1.02**	**0.99**	**-0.99**	**0.73**	**-1.27**
Step 3		dl3	mm	**5.45**	**0.00**	**3.29**	**-1.88**	**2.54**	**-2.54**	**2.47**	**-2.47**	**1.80**	**-3.15**
Step 4		dl4	mm	**2.22**	**0.00**	**1.34**	**-0.77**	**1.94**	**0.00**	**1.00**	**-1.00**	**0.73**	**-1.28**
Step 5		dl5	mm	**10.64**	**6.82**	**6.71**	**3.21**	**3.10**	**0.00**	**-0.30**	**-3.40**	**-3.50**	**-6.97**
Step 6		dl6	mm	**23.74**	**15.18**	**14.98**	**7.15**	**6.95**	**0.00**	**-0.64**	**-7.57**	**-7.76**	**-15.54**
Temperature Effect(+15% Marge)	+25.3degree	dl7	mm	**-31.56**	**-20.44**	**-19.81**	**-9.66**	**-9.05**	**0.00**	**1.14**	**10.12**	**10.69**	**20.81**
	-17.25 degree	dl8	mm	**21.52**	**13.94**	**13.51**	**6.59**	**6.17**	**0.00**	**-0.78**	**-6.90**	**-7.29**	**-14.19**
Creep and Shrinkage		dld	mm	44.23	22.00	27.64	6.96	15.55	-3.56	3.52	-15.43	-8.00	-28.21
Servise I	Shortening	dL1	mm	78.90	43.12	49.38	16.25	26.07	-4.27	3.29	-26.80	-18.35	-50.88
	Expansion	dL2	mm	6.35	-2.53	3.86	-4.63	4.70	-3.56	4.89	-3.29	4.83	-3.25
	Total(Absolute)	dL	mm	85.25	45.66	53.24	20.89	30.77	7.84	8.18	30.09	23.18	54.13

4 Rotation of the girder

		S1	S2	S3	S4	S5	S5	S5	S5	S5	S5
		P5	P6	P7	P8	P9	P9	P9	P9	P9	P9
Value of Rotation	rad	0.007	0.007	0.007	0.007	0.007	0.007	0.007	0.007	0.007	0.007

1.6 Spring of Substructure and Foundation

				A1	S1	S1	S2	S2	S3	S3	S4	S4	S5
Longitudinal	Spring of Pier	Kp	kN/m	1,082,516	259,462	259,462	259,462	259,462	259,462	259,462	137,646	137,646	137,646
Transversal	Spring of Pier	Kp	kN/m	1,082,516	259,462	259,462	259,462	259,462	259,462	259,462	137,646	137,646	137,646

"-" is shown infinite.

1.7 Property of Elastomer

				A1	S1	S1	S2	S2	S3	S3	S4	S4	S5
				5	1	1	1	1	1	1	1	1	5
Type of Elastomer			-	-	NR	NR	NR	NR	NR	NR	NR	NR	-
Elastic Shear Modulus		G0		-	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Ultimate Elongation		γu	%	-	400	400	400	400	400	400	400	400	400
Factor of Shape		S	-	-	6<S	6<S	6<S	6<S	6<S	6<S	6<S	6<S	6<S
Allowable shear strain	Service state	γsa	%	-	50	50	50	50	50	50	50	50	50
	Ultimate	γeqa	%	-	-	-	-	-	-	-	-	-	-
Steel plate in the elastomer	Material	-	-	-	A709M g250	A709M g250	A709M g250	A709M g250	A709M g250	A709M g250	A709M g250	A709M g250	A709M g250
	yield strength	fsy	Mpa	-	250	250	250	250	250	250	250	250	250
	fatigue strenght	fft	Moa	-	165	165	165	165	165	165	165	165	165

Friction factor of Sliding-Bearing

3.3. Calculation Sheet of Elastomeric Bearing

Super T girder

1 Material of Elastmeric Bearing

					S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
					A1	P1		P2		P3		P4		P5
Bearing Type		Is=1;Pot Bearing Is=0,Elastmeric Bearing	Is	-	1	0	0	0	0	2	0	0	0	0
			-	-	Pot Bearing	Elastomeric Bearing	Elastomeric Bearing	Elastomeric Bearing	Elastomeric Bearing	Elastomeric Bearing	Elastomeric Bearing	Elastomeric Bearing	Elastomeric Bearing	Elastomeric Bearing
Elastomer	Hardness Grade				60	60	60	60	60	60	60	60	60	60
	Shear modulus at 23°C	0.90 -1.38	Ge	Mpa	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Insert Reinfocement	ASTM A709 Grad 250	Yield Strenght	fsy	Mpa	250	250	250	250	250	250	250	250	250	250
	ASTM A709 Grad 250	Fatigue stress	fsfy	Mpa	165	165	165	165	165	165	165	165	165	165

2 Property of elastomeric bearings

					S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
					A1	P1		P2		P3		P4		P5
Condition of support		Remark	-	-	1	0	0	0	0	2	0	0	0	0
		Condition	-	-	Mv	E	E	E	E	F	E	E	E	E
Dimension	Longitudinal	-	L0	mm	-	350	350	350	350	350	350	350	350	350
	Transversal	-	W0	mm	-	600	600	600	600	600	600	600	600	600
Necessary dimensions	Longitudinal	-	reqL	mm	-	230	230	230	230	220	230	230	230	230
	Transversal	-	reqW	mm	-	230	230	230	230	220	230	230	230	230
Using dimension for design	Longitudinal	-	L	mm	-	340	340	340	340	340	340	340	340	340
	Transversal	-	W	mm	-	590	590	590	590	590	590	590	590	590
Modulus of Rigidity		-	G	Mpa	-	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Thickness of layer of elastomeric	Thickness of necessity	-	Σte	mm	-	87	99	33	53	9	7	54	37	102
	Thickness of one layer	-	hri	mm	-	16	16	16	16	16	16	16	16	16
	Req Number layer	-	reqn	layer	-	6	7	3	4	1	1	4	3	7
	Number layer		n	layer	-	7	7	4	4	4	4	4	4	7
	Total thickness of elastomeric	Σ te=te*n	hrt	mm	-	112	112	64	64	64	64	64	64	112
	Thickness of top layer		hrit	mm	-	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
	Thickness of bottom layer		hrib	mm	-	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
Steel Plate	Mediam Plate		tsm	mm	-	3	3	3	3	3	3	3	3	3
	Top Plate		tst	mm	-	3	3	3	3	3	3	3	3	3
	Bottom Plate		tsb	mm	-	3	3	3	3	3	3	3	3	3
Total area for calculation		F=a・b	F	m2	-	0.2006	0.2006	0.2006	0.2006	0.2006	0.2006	0.2006	0.2006	0.2006
Shape factor		S=a・b/(2(a+b)te)	S	-	-	6.741	6.741	6.741	6.741	6.741	6.741	6.741	6.741	6.741
Spring of elasotmeric bearing		AG/hrt	Kh	kN/m	0	1,791	1,791	3,134	3,134	3,134	3,134	3,134	3,134	1,791

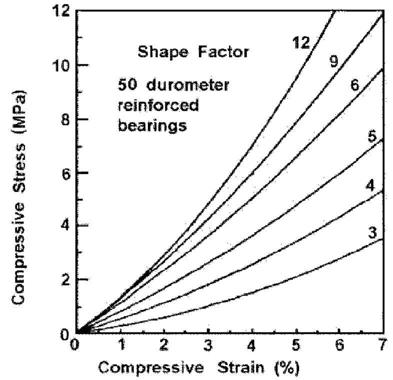
2 Reaction and movements — Pre-shearing IS= 0 Not pre-shearing

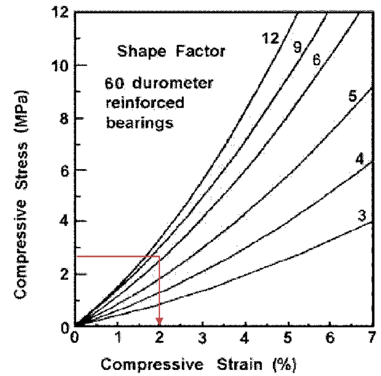
						SP1	SP2	SP3	SP4	SP5	SP6	SP7	SP8	SP9	SP10
						A1	P1		P2		P3		P4		P5
Reaction	Reaction of permanent load		-	Rd	kN	750	750	750	750	750	750	750	750	750	750
	Reaction of live load	Max	-	Rmax	kN	550	550	550	550	550	550	550	550	550	550
		Min	-	Rmin	kN	0	0	0	0	0	0	0	0	0	0
	Service	Max	-	Rmax	kN	1300	1300	1300	1300	1300	1300	1300	1300	1300	1300
		Min	-	Rmin	kN	750	750	750	750	750	750	750	750	750	750
Movement	At time of working dead-load		-	ld	mm	44.2	22.0	27.6	7.0	15.6	-3.6	3.5	-15.4	-8.0	-28.2
	SH+CR		-	sh+cr	mm	44.2	22.0	27.6	7.0	15.6	-3.6	3.5	-15.4	-8.0	-28.2
	T(-15.25)			lt	mm	21.5	13.9	13.5	6.6	6.2	0.0	-0.8	-6.9	-7.3	-14.2
	Service		-	ls	mm	78.9	43.1	49.4	16.3	26.1	4.3	3.3	26.8	18.4	50.9
	Rotation st service limit state		-	θ	rad	0.0070	0.0070	0.0070	0.0070	0.0070	0.0070	0.0070	0.0070	0.0070	0.0070

3 Check of the compression of elastomeric bearings

					SP1	SP2	SP3	SP4	SP5	SP6	SP7	SP8	SP9	SP10
					A1	P1		P2		P3		P4		P5
Limit of compressive stress	Modulus of Rigidity	-	G	Mpa	-	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
	Shape factor	-	S	-	-	6.741	6.741	6.741	6.741	6.741	6.741	6.741	6.741	6.741
	Due to the total load	1.66GC≦11.0 or 2.0GS≦12.0Mpa	σsa	Mpa	-	11.0	11.0	11.0	11.0	12.0	11.0	11.0	11.0	11.0
	Due to the live load	0.66Gs or 1.00GS	σla	Mpa	-	4.4	4.4	4.4	4.4	6.7	4.4	4.4	4.4	4.4
Reaction	Dead Load	-	Rd	kN	750.0	750.0	750.0	750.0	750.0	750.0	750.0	750.0	750.0	750.0
	At service limit state	Rmax	R	kN	1300	1300	1300	1300	1300	1300	1300	1300	1300	1300
	Live load	-	Rl	kN	550	550	550	550	550	550	550	550	550	550
Total area of elastomeric bearing	Longitudinal	-	L	mm	-	340	340	340	340	340	340	340	340	340
	Transversal	-	W	mm	-	590	590	590	590	590	590	590	590	590
	Area	L*W	F	m2	-	0.2006	0.2006	0.2006	0.2006	0.2006	0.2006	0.2006	0.2006	0.2006
Compressive stress	Dead Load	Rd/F	σd	Mpa	-	3.74	3.74	3.74	3.74	3.74	3.74	3.74	3.74	3.74
	At service limit state	R/F	σs	Mpa	-	6.48	6.48	6.48	6.48	6.48	6.48	6.48	6.48	6.48
	Live load	RL/F	σl	Mpa	-	2.74	2.74	2.74	2.74	2.74	2.74	2.74	2.74	2.74
Judgment	Maximum compressive stress	σs≦σsa		Mpa	-	OK	OK	OK	OK	OK	OK	OK	OK	OK
	due to live load	σl≦σla		Mpa	-	OK	OK	OK	OK	OK	OK	OK	OK	OK

4 Check of the compressive Deflection





Check of the Compreesive Deflection

					SP1	SP2	SP3	SP4	SP5	SP6	SP7	SP8	SP9	SP10
					A1	P1		P2		P3		P4		P5
Property of elastomeric bearing	Modulus of Rigidity	-	G	kN	-	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
	Shape factor	-	S	-	-	6.741	6.741	6.741	6.741	6.741	6.741	6.741	6.741	6.741
	Thickness of inner layer	-	hri	mm	-	16.0	16.0	16.0	16.0	16.0	16.0	16.0	16.0	16.0
	Number of layers	-	Nl	nos	-	7.0	7.0	4.0	4.0	4.0	4.0	4.0	4.0	7.0
	Thickness of top layer	-	hrio	mm	-	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
	Thickness of bottom layer	-	hrib	mm	-	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
Compressive stress	Due to dead load	-	σd	Mpa	-	3.74	3.74	3.74	3.74	3.74	3.74	3.74	3.74	3.74
	Due to live load	-	σs	Mpa	-	2.74	2.74	2.74	2.74	2.74	2.74	2.74	2.74	2.74
Compressive Deflection due to live load	Strain of each layer	-	εi	%	-	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00
	Deflection of inner layer	ΣεLii*hrii	δLi	mm	-	2.240	2.240	1.280	1.280	1.280	1.280	1.280	1.280	2.240
	Deflection of top layer	ΣεLt*hrt	δLt	mm	-	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05
	Deflection of bottom layer	ΣεLt*hrt	δLb	mm	-	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05
	Total deflection	-	δL	mm	-	2.34	2.34	1.38	1.38	1.38	1.38	1.38	1.38	2.34
Judgment	Limit deflection	-	δLa	mm	-	3.00	3.00	3.00	3.00	3.00	3.00	3.00	3.00	3.00
	Judgment	δL≦δLa	-	mm	-	OK	OK	OK	OK	OK	OK	OK	OK	OK

4 Check of Shear deformation

					SP1	SP2	SP3	SP4	SP5	SP6	SP7	SP8	SP9	SP10
					A1	P1		P2		P3		P4		P5
Shear deformation at Service limit state		—	ls	mm	79	43	49	16	26	4	3	27	18	51
Capacity of shear deformation		—	Ss	-	-	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
Required Thickness	Required Thickness	δs/Ss	hrts	mm	-	86.2	98.8	32.5	52.1	8.5	6.6	53.6	36.7	101.8
	Required Thickness	Max(hrts,htrst,hrtex)	reqhrt	mm	-	87	99	33	53	9	7	54	37	102
	Thickness of one layer	—	hri	mm	-	16.0	16.0	16.0	16.0	16.0	16.0	16.0	16.0	16.0
	Number of layers	—	ns	layer	-	7	7	4	4	4	4	4	4	7
	Total thickness of elastomeric layers	hri*ns	hrt	mm	-	112	112	64	64	64	64	64	64	112
	Jug dement	hrt≧reqhrt		-	-	OK	OK	OK	OK	OK	OK	OK	OK	OK
Check of Shear deformation	Total thickness of elastomeric layers	—	hrt	mm	-	112	112	64	64	64	64	64	64	112
	At service limit state	ls/hrt	γs	%	-	39	44	25	41	7	5	42	29	45
	Jug dement	Allowable		%	-	50	50	50	50	50	50	50	50	50
		Safty	F	-	-	1.30	1.13	1.97	1.23	7.49	9.72	1.19	1.74	1.10
			—	—	-	OK	OK	OK	OK	OK	OK	OK	OK	OK

5 Check of compressive stress due to rotation and angle of rotation of the girder

					SP1	SP2	SP3	SP4	SP5	SP6	SP7	SP8	SP9	SP10
					A1	P1		P2		P3		P4		P5
Modulus of Rigidity		-	G	Mpa	-	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Reaction	Service	-	R	kN	1300	1300	1300	1300	1300	1300	1300	1300	1300	1300
Dimension of elastomeric bearing	Longitudinal	B=L	B	mm	-	350	350	350	350	350	350	350	350	350
	Transversal	-	W	mm	-	600	600	600	600	600	600	600	600	600
	Thickness of one layer	-	hri	mm	-	16	16	16	16	16	16	16	16	16
	Req Number layer	-	reqn	layer	-	7	7	4	4	4	4	4	4	7
	Total thickness of elastomeric layers	hri*ns	hrt	mm	-	112	112	64	64	64	64	64	64	112
	Shape factor	S=a・b/(2(a+b)te)	S	-	-	6.741	6.741	6.741	6.741	6.741	6.741	6.741	6.741	6.741
Angle of rotation		-	θ	rad	0.007	0.007	0.007	0.007	0.007	0.007	0.007	0.007	0.007	0.007
Judgment	Limit angle due to rotation	1.0GS(θ/n)(B/hri)^2	σamin	Mpa	-	3.23	3.23	5.64	5.64	5.64	5.64	5.64	5.64	3.23
	Local Commpresive Stress due to Rotation	2.25GS(1-0.167(θs/n)(B/hri)^2	σamax	Mpa	-	11.43	11.43	10.52	10.52	13.05	10.52	10.52	10.52	11.43
	Acual Stress	R/(L*B)	σs	Mpa	-	6.48	6.48	6.48	6.48	6.48	6.48	6.48	6.48	6.48
	Check for Edge Uplift	σs＞σsmin	-	-	-	OK	OK	OK	OK	OK	OK	OK	OK	OK
	Check for Compresssion	σs＜σsmax	-	-	-	OK	OK	OK	OK	OK	OK	OK	OK	OK
	Judgement	-	-	-	-	OK	OK	OK	OK	OK	OK	OK	OK	OK

6 Check for buckling of elastomeric bearing

					SP1	SP2	SP3	SP4	SP5	SP6	SP7	SP8	SP9	SP10
					A1	P1		P2		P3		P4		P5
Condition of support		Remark	-	-	1	0	0	0	0	2	0	0	0	0
		Condition	-	-	Mv	E	E	E	E	F	E	E	E	E
Modulus of Rigidity			G	Mpa	-	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Dimension of bearing	Longitudinal		L	mm	-	340	340	340	340	340	340	340	340	340
	Transversal		W	mm	-	590	590	590	590	590	590	590	590	590
	Thickness of one elastmeric layer		hri	mm	-	16	16	16	16	16	16	16	16	16
	Total thickness of elastomeric layers		hrt	mm	-	112	112	64	64	64	64	64	64	112
	Shape factor	S=a・b/(2(a+b)te)	S	-	-	6.741	6.741	6.741	6.741	6.741	6.741	6.741	6.741	6.741
Result of caluculation	Factor A	(1.92/hrt/L)/(S・Sqrt(1+2L/W))	A	-	-	0.4311	0.4311	0.2463	0.2463	0.2463	0.2463	0.2463	0.2463	0.4311
	Factor B	2.67/(S+2)/(1+L/4W)	B	-	-	0.2670	0.2670	0.2670	0.2670	0.2670	0.2670	0.2670	0.2670	0.2670
	2*A	2*A	2A	-	-	0.862	0.862	0.493	0.493	0.493	0.493	0.493	0.493	0.862
Jadgement		2A≦B	-	-	-	NG	NG	NG	NG	NG	NG	NG	NG	NG
Check for the case of 2A>B														
Result of caluculation	Maximum compresssive stress		σs	Mpa	-	6.48	6.48	6.48	6.48	6.48	6.48	6.48	6.48	6.48
	Limit of compressive stress	GS/(2A-B) or GS/(A-B)	σsa	Mpa	-	11.3	11.3	29.9	29.9	-325.7	29.9	29.9	29.9	11.3
	Jadgement	σs≦GS/(A-B)	-	-	-	OK	OK	OK	OK	OK	OK	OK	OK	OK
Comprehensive Jadgement			―	―	-	OK	OK	OK	OK	OK	OK	OK	OK	OK

7 Check of the reinforcement steel

					SP1	SP2	SP3	SP4	SP5	SP6	SP7	SP8	SP9	SP10
					A1	P1		P2		P3		P4		P5
Prooperty of steel palte	Yeild strenth of steel plate	-	Fy	Mpa	-	250	250	250	250	250	250	250	250	250
	Fatigue strenght of teel plate	-	ΔFTH	Mpa	-	165	165	165	165	165	165	165	165	165
	Thickess of steel plate	-	hs	mm	-	3	3	3	3	3	3	3	3	3
Maximum thickness of one layer of elastmeric			hrimax	mm	-	16	16	16	16	16	16	16	16	16
Working stress	Maxmam commpressive stress		σs	m2	-	6.48	6.48	6.48	6.48	6.48	6.48	6.48	6.48	6.48
	Compressive stress due to live load		σl		-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Necessary thickness	For maxmam compressive stress	3hmaxσs/Fy	reqhs1		-	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
	For compressive stress due to live load	2.0hmaxσL/ΔF$_{TH}$	reqHs2		-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Necessary thickness		reqhs		-	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
Jadgement		hs≧reqhs	-		-	OK	OK	OK	OK	OK	OK	OK	OK	OK

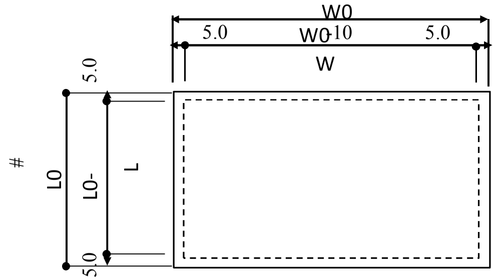
8 Longitudinal Horizontal Force

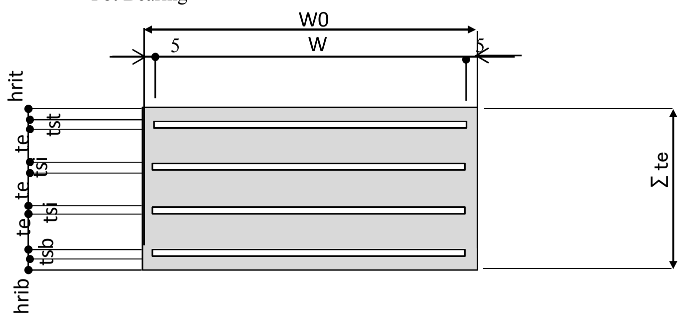
I.tem			Symbol	Unit	SP1	SP2	SP3	SP4	SP5	SP6	SP7	SP8	SP9	SP10
					A1	P1		P2		P3		P4		P5
			SC	-	Mv	E	E	E	E	F	E	E	E	E
Spring	Due to Bearing	Each Bearing	Ks1	kN/m	0	1,791	1,791	3,134	3,134	3,134	3,134	3,134	3,134	1,791
		Total of Shoe's Line	Ks	kN/m	0	10,746	10,746	18,806	18,806	18,806	18,806	18,806	18,806	10,746
	Due to Substrcture and Foundation		Kp	kN/m	1,082,516	259,462	259,462	259,462	259,462	259,462	259,462	137,646	137,646	137,646
	Composite		K	kN/m	0	10,319	10,319	17,535	17,535	17,535	17,535	16,546	16,546	9,968
Displasement	Creep and Shrinkage		sh+cr	mm	44.2	22.0	27.6	7.0	15.6	-3.6	3.5	-15.4	-8.0	-28.2
	Tempratur e Effect	T(+25)	t+	mm	-31.6	-20.4	-19.8	-9.7	-9.0	0.0	1.1	10.1	10.7	20.8
		T(-17)	t-	mm	21.5	13.9	13.5	6.6	6.2	0.0	-0.8	-6.9	-7.3	-14.2
Horizontal Force	Creep and Shrinkage		Hd	mm	0.0	39.4	49.5	21.8	48.8	-11.2	11.0	-48.4	-25.1	-50.5
	Tempratur e Effect	T(+25)	Ht(+)	mm	0.0	-210.9	-204.4	-169.4	-158.7	0.0	20.0	167.4	176.9	207.4
		T(-17)	Ht(-)	mm	0.0	143.8	139.4	115.5	108.2	0.0	-13.7	-114.1	-120.6	-141.4

Information Only

					SP1	SP2	SP3	SP4	SP5	SP6	SP7	SP8	SP9	SP10
					A1	P1		P2		P3		P4		P5
Dimension	Longitudinal	-	L0	mm	-	350	350	350	350	350	350	350	350	350
	Transversal	-	W0	mm	-	600	600	600	600	600	600	600	600	600
Outeside Elastmeric			tso	mm	-	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0
Using dimension for design	Longitudinal	-	L	mm	-	340	340	340	340	340	340	340	340	340
	Transversal	-	W	mm	-	590	590	590	590	590	590	590	590	590
Thikness of elastomer	Thickness of one layer		te	mm	-	16	16	16	16	16	16	16	16	16
	Number of layers		ns	nos	-	7	7	4	4	4	4	4	4	7
	Total thickness of elastomer	te*ne	Σte	mm	-	112	112	64	64	64	64	64	64	112
	Thickness of top layer		hrit	mm	-	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
	Thickness of bottom layer		hrib	mm	-	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
Thickness of steel plate	Thickness of medium steel plate		tsi	mm	-	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0
	Thickness of top steel plate		tsT	mm	-	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0
	Thickness of bottom steel plate		tsB	mm	-	3	3	3	3	3	3	3	3	3
	Total thickness of steel plate	tsi*(ne-1)+tsT+tsB	Σts	mm	-	24	24	15	15	15	15	15	15	24
Thikness of Manufacture			Σts	mm	-	141	141	84	84	84	84	84	84	141

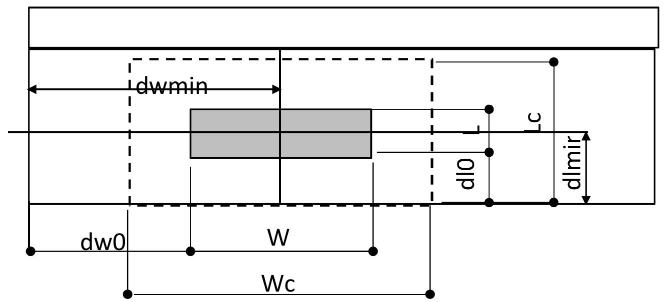
Pot Bearing





3.4 Chek for bearibg stress of bed concrete

						S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
						A1	P1		P2		P3		P4		P5
Bearing Type		Is=1;Pot Bearing Is=0,Elastmeric Bearing		Is	-	1	0	0	0	0	2	0	0	0	0
				-	-	Pot Bearing	Elastomeric Bearing	Elastomeric Bearing	Elastomeric Bearing	Elastomeric Bearing	Elastomeric Bearing	Elastomeric Bearing	Elastomeric Bearing	Elastomeric Bearing	Elastomeric Bearing
Strength of Concrete				f'c	Mpa	30	30	30	30	30	30	30	30	30	30
Reaction for Bearing		Dead Load1		Dc	kN	670.0	670.0	670.0	670.0	670.0	670.0	670.0	670.0	670.0	670.0
		Dead Load2		Dw	kN	80.0	80.0	80.0	80.0	80.0	80.0	80.0	80.0	80.0	80.0
		Live Load max		Lmax	kN	550.0	550.0	550.0	550.0	550.0	550.0	550.0	550.0	550.0	550.0
		Live load min		Lmin	kN	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Sevive limit state		Maximum Reaction		Rmax	kN	1300.0	1300.0	1300.0	1300.0	1300.0	1300.0	1300.0	1300.0	1300.0	1300.0
Strength Limit State		Maximum Reaction		Rumnax	kN	1920.0	1920.0	1920.0	1920.0	1920.0	1920.0	1920.0	1920.0	1920.0	1920.0
Shape of Bearing		Longitudinal		L	mm	330	350	350	350	350	350	350	350	350	350
		Transversal		W	mm	330	600	600	600	600	600	600	600	600	600
fective Width of Concr		tance from End of Concrete		dlmin	mm	1000	1000	1000	1000	1000	1000	1000	1000	1000	1000
		stance from End of Concrete		dwmin	mm	350	350	350	350	350	350	350	350	350	350
		Longitudinal Cover		dlo	mm	185	175	175	175	175	175	175	175	175	175
		Transversal Cover		dwo	mm	835	825	825	825	825	825	825	825	825	825
		Longitudinal		Lc	mm	700	700	700	700	700	700	700	700	700	700
		Tranversal		Wc	mm	700	950	950	950	950	950	950	950	950	950
Check of Bearing Stress of Concrete	Modification factor	Condition	Ic=1,Nonuniformy load	Ic	-	0	0	0	0	0	0	0	0	0	0
		Factor of Support Condition	Ic=0->k=1.0,Ic=1 ->k=0.75	k	-	1	1	1	1	1	1	1	1	1	1
		Effective Erea of Concrete	Lc*Wc	A2	mm2	490,000	665,000	665,000	665,000	665,000	665,000	665,000	665,000	665,000	665,000
		Area of Bearing Devie	L*W	A1	mm2	108,900	210,000	210,000	210,000	210,000	210,000	210,000	210,000	210,000	210,000
		Modification	k*Sqrt(A2/A1)	m1	-	2.12	1.78	1.78	1.78	1.78	1.78	1.78	1.78	1.78	1.78
		Factore	Ic=0,m≦2.0,Ic=1,m≦1.5	m	-	2.0	1.8	1.8	1.8	1.8	1.8	1.8	1.8	1.8	1.8
Bearing Resitance		Nominal resitance	0.85*f'c*A1*m	Pn	kN	5553.9	9529.3	9529.3	9529.3	9529.3	9529.3	9529.3	9529.3	9529.3	9529.3
		Restance Foctor	-	φ	-	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7
		Design Bearing Rsistance	φ*Pn	Pr	kN	3887.73	6,670.5	6,670.5	6,670.5	6,670.5	6,670.5	6,670.5	6,670.5	6,670.5	6,670.5
		Design Bearing Stress	Pr/A1	fb	Mpa	35.7	31.8	31.8	31.8	31.8	31.8	31.8	31.8	31.8	31.8
Judgement		Safty Factor	Pr/Rumax	Fa	-	2.02	3.47	3.47	3.47	3.47	3.47	3.47	3.47	3.47	3.47
		Judgement	Fa≧1---"OK",Fa<1.0--"NG	-	-	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK



4. Cluculationof Anchor Bar   Calculation of Anchor bar for Pier and Abutment under Super T

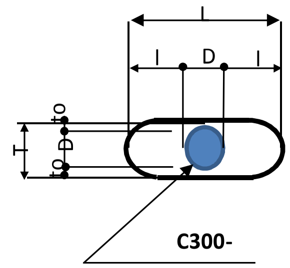
Design Condition

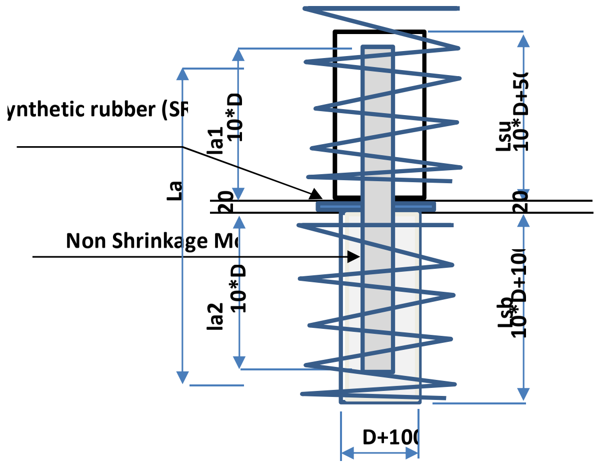
Design Condition			Symbol	Unit	A2	P1 B	P1 E	P2 B	P2 E	P3 B	P3 E	P4 B	P4 E	P5 B	P5 E	P6 B	P6 E	P7 B	P7 E	P8 B	P8 E	P9 B	P9 E	P10 B
Support Condition	Longitudinal		-	-	Mov	E	E	E	E	E	Fix	E	E	E	E	E	E	Fix	E	E	E	E	E	Mov
	Transversal		-	-	Fix	Fix	Fix	Fix	Fix	Fix	Fix	Fix	Fix	Fix	Fix	Fix	Fix	Fix	Fix	Fix	Fix	Fix	Fix	Fix
Horizontal Reaction	Longitudinal	Braking Force	BR	kN	0	0	0	0	0	0	210	0	0	0	0	210	0	0	0	0	0	0	0	0
		Earthquake Effect	EqL	kN	0	0	0	0	0	0	3000	0	0	0	0	3000	0	0	0	0	0	0	0	0
	Transversal	Earthquake Effect	EqR	kN	350	390	390	390	390	390	390	390	390	390	390	390	390	390	390	390	390	390	390	350
Movement	Longitudinal	Creep +Shrinkage	dL(Cr+sh)	mm	30	30	30	30	30	30	30	30	30	30	30	30	30	30	30	30	30	30	30	30
		Temperature(+)	dL(T+)	mm	20	20	20	20	20	20	20	20	20	20	20	20	20	20	20	20	20	20	20	20
		Temperature(-)	dL(T-)	mm	-15	-15	-15	-15	-15	-15	-15	-15	-15	-15	-15	-15	-15	-15	-15	-15	-15	-15	-15	-15
		Braking Force	dL(BE)	mm	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10
		Earthquake Effect	dl(EqL)	mm	20	20	20	20	20	20	20	20	20	20	20	20	20	20	20	20	20	20	20	20
	Transversal	Creep +Shrinkage	dLt(Cr+sh)	mm	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
		Temperature(+)	dLt(T+)	mm	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
		Temperature-+)	dLt(T+)	mm	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
		Earthquake Effect	dL(EqT)	mm	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Load Factor	Strength	Creep +Shrinkage	γCr,Sh	-	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
		Temperature	γTE	-	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
		Braking Force	γBR	-	1.75	1.75	1.75	1.75	1.75	1.75	1.75	1.75	1.75	1.75	1.75	1.75	1.75	1.75	1.75	1.75	1.75	1.75	1.75	1.75
	Extreme Event	Earthquake Effect	γEq	-	1.5	1.0	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1.5
Factored Reaction	Longitudinal	Strength	Hst	kN	0	0	0	0	0	0	367.5	0	0	0	0	367.5	0	0	0	0	0	0	0	0
		Extreme Invent	Heq	kN	0	0	0	0	0	0	3,000	0	0	0	0	3,000	0	0	0	0	0	0	0	0
	Transversal	Extreme Invent	Heq	kN	525	390	390	390	390	390	390	390	390	390	390	390	390	390	390	390	390	390	390	525
Factored Movement	Longitudinal	Strength	dLst	mm	77.5	77.5	77.5	77.5	77.5	77.5	77.5	77.5	77.5	77.5	77.5	77.5	77.5	77.5	77.5	77.5	77.5	77.5	77.5	77.5
		Extreme Event	dLex	mm	30	20	20	20	20	20	20	20	20	20	20	20	20	20	20	20	20	20	20	30
	Transversal	Strength	dLst	mm	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
		Extreme Event	dLex	mm	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Design Horizontal Force	Strength		Hst	-	0	0	0	0	0	0	367.5	0	0	0	0	367.5	0	0	0	0	0	0	0	0
	Extreme Event		Hex	-	525.0	390.0	390	390	390	390	3000	390	390	390	390	3000	390	390	390	390	390	390	390	525.0

Material and Result

				Symbol	Unit	A2	P1		P2		P3		P4		P5		P6		P7		P8		P9		A2
							B	E	B	E	B	E	B	E	B	E	B	E	B	E	B	E	B	E	
Material	Concrete	Compressive Strength at 28 days		f'c	Mpa	30	30.0	30	30	30	30	30	30	30	30	30	30	30	30	30	30	30	30	30	30
	Steel	Anchor Bar	Type	-	-	C300-T	C300-T	C300-T	C300-T	C300-T	C300-T	C300-T	C300-T	C300-T	C300-T	C300-T	C300-T	C300-T	C300-T	C300-T	C300-T	C300-T	C300-T	C300-T	C300-T
			Yield Tensil Strength	fry	Mpa	300	300	300	300	300	300	300	300	300	300	300	300	300	300	300	300	300	300	300	300
			Yield Shear Strength	τry	Mpa	170	170	170	170	170	170	170	170	170	170	170	170	170	170	170	170	170	170	170	170
	Resistance Factor	Concrete	Strength	φstc	-	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7
			Extreme Event	φexr	-	1	1.0	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
		Rebar	Strength	φexc	-	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7
			Extreme Event	φexr	-	1	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1
Calculation Result	Required Anchor Bar Area	at Strength		reqAr1	mm2	0	0	0	0	0	0	3088.2	0	0	0	0	3088.2	0	0	0	0	0	0	0	0
		at Extreme		reqAr2	mm2	3,088.2	2,294.1	2,294.1	2,294.1	2,294.1	2,294.1	17,647.1	2,294.1	2,294.1	2,294.1	2,294.1	17,647.1	2,294.1	2,294.1	2,294.1	2,294.1	2,294.1	2,294.1	2,294.1	3,088.2
		Result		reqAr	mm2	3,088.2	2,294.1	2,294.1	2,294.1	2,294.1	2,294.1	17,647.1	2,294.1	2,294.1	2,294.1	2,294.1	17,647.1	2,294.1	2,294.1	2,294.1	2,294.1	2,294.1	2,294.1	2,294.1	3,088.2
	Verification of Anchore Bar	Arrengement Anchor Bar	Nuber of Anchor Bar	Nb	nos	5	5	5	5	5	5	10	5	5	5	5	10	5	5	5	5	5	5	5	5
			Required Dimaiter	reqDs	mm	28.1	24.2	24.2	24.2	24.2	24.2	47.5	24.2	24.2	24.2	24.2	47.5	24.2	24.2	24.2	24.2	24.2	24.2	24.2	28.1
		Strength	Diamiter	Ds	nos	30	30	30	30	30	30	50	30	30	30	30	50	30	30	30	30	30	30	30	30
			Area of Rebar	Ar	mm2	3534.3	3534.3	3534.3	3534.3	3534.3	3534.3	19635.0	3534.3	3534.3	3534.3	3534.3	19635.0	3534.3	3534.3	3534.3	3534.3	3534.3	3534.3	3534.3	3534.3
			Nomonal Rejistance	Hrn	kN	600.8	600.8	600.8	600.8	600.8	600.8	3337.9	600.8	600.8	600.8	600.8	3337.9	600.8	600.8	600.8	600.8	600.8	600.8	600.8	600.8
			Resistance Factor	φstc	-	0.7	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.7
			Factored Resistance	Hur	kN	421	421	421	421	421	421	2337	421	421	421	421	2337	421	421	421	421	421	421	421	421
			Active Force	Hu	kN	0	0	0	0	0	0	367.5	0	0	0	0	367.5	0	0	0	0	0	0	0	0
			Safety Factor	Fa	-	0	0.000	0.000	0.000	0.000	0.000	0.157	0.000	0.000	0.000	0.000	0.157	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0
			Juadge	-	-	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK
		Extreme Event	Diamiter	Ds	mm	30	30	30	30	30	30	50	30	30	30	30	50	30	30	30	30	30	30	30	30
			Area of Rebar	Ar	mm2	3534.3	3534.3	3534.3	3534.3	3534.3	3534.3	19635.0	3534.3	3534.3	3534.3	3534.3	19635.0	3534.3	3534.3	3534.3	3534.3	3534.3	3534.3	3534.3	3534.3
			Nomonal Rejistance	Hrn	kN	600.8	600.8	600.8	600.8	600.8	600.8	3337.9	600.8	600.8	600.8	600.8	3337.9	600.8	600.8	600.8	600.8	600.8	600.8	600.8	600.8
			Resistance Factor	φstc	-	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
			Factored Resistance	Hur	kN	601	601	601	601	601	601	3338	601	601	601	601	3338	601	601	601	601	601	601	601	601
			Active Force	Hu	kN	525.0	390.0	390.0	390.0	390.0	390.0	3000.0	390.0	390.0	390.0	390.0	3000.0	390.0	390.0	390.0	390.0	390.0	390.0	390.0	525.0
			Safety Factor	Fa	-	0.874	0.649	0.649	0.649	0.649	0.649	0.899	0.649	0.649	0.649	0.649	0.899	0.649	0.649	0.649	0.649	0.649	0.649	0.649	0.874
			Juadge	-	-	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK

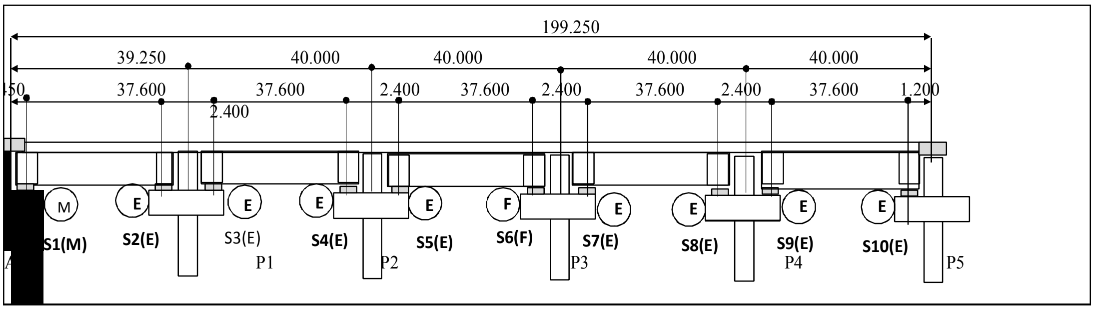
Shape of Anchor Bar





		Symbol	Unit	A1	P1		P2		P3		P4		P5		P6		P7		P8		P9		P10
					B	E	B	E	B	E	B	E	B	E	B	E	B	E	B	E	B	E	B
Support Condition	Longitudinal	-	-	Move	E	E	E	E	E	Fix	E	E	E	E	E	E	Fix	E	E	E	E	E	Move
	Transversal	-	-	Fix	Fix	Fix	Fix	Fix	Fix	Fix	Fix	Fix	Fix	Fix	Fix	Fix	Fix	Fix	Fix	Fix	Fix	Fix	Fix
Anhor Bar		Nb	nos	5	5	5	5	5	5	10	5	5	5	5	10	5	5	5	5	5	5	5	5
		D	mm	30	30	30	30	30	30	50	30	30	30	30	50	30	30	30	30	30	30	30	30
		la1	mm	300	300	300	300	300	300	500	300	300	300	300	500	300	300	300	300	300	300	300	300
		la2	mm	300	300	300	300	300	300	500	300	300	300	300	500	300	300	300	300	300	300	300	300
		La	mm	620	620	620	620	620	620	1020	620	620	620	620	1020	620	620	620	620	620	620	620	620
Sleeve Pipe		l	mm	100	100	100	100	100	100	5	100	100	100	100	100	100	5	100	100	100	100	100	100
		L	mm	230	230	230	230	230	230	60	230	230	230	230	250	230	40	230	230	230	230	230	230
		Lsu	mm	350	350	350	350	350	350	550	350	350	350	350	550	350	350	350	350	350	350	350	350
		to	mm	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5
		T	mm	40	40	40	40	40	40	60	40	40	40	40	60	40	40	40	40	40	40	40	40
		Lsb	mm	400	400	400	400	400	400	600	400	400	400	400	600	400	400	400	400	400	400	400	400

Appendix 1. **Desplacement at the bearings** First Assump Not Final



Movements for each span

The expansion assumes the course that a beam shorts plus.

				Symbol	Unit	NO.1Span 1	NO.2Span 2	NO.3Span 3	NO.4Span 4	NO.5Span 5	NO.6Span 6	NO.7Span 7	NO.8Span 8	NO.9Span 9	SUM
Simple Beam	Step 1		Span Length	ls	m	37.60	0.00	37.60	0.00	37.60	0.00	37.60	0.00	37.60	188.00
			Shortning	d1	mm	29.4	0.0	29.4	0.0	29.4	0.0	29.4	0.0	29.4	146.77
	Step 2		Span Length	ls	m	37.60	0.00	37.60	0.00	37.60	0.00	37.60	0.00	37.60	188.00
			Shortning	d2	mm	2.2	0.0	2.2	0.0	2.2	0.0	2.2	0.0	2.2	10.97
	Step 3		Span Length	ls	m	37.60	0.00	37.60	0.00	37.60	0.00	37.60	0.00	37.60	188.00
			Shortning	d3	mm	5.4	0.0	5.4	0.0	5.4	0.0	5.4	0.0	5.4	27.23
	Step 4		Span Length	ls	m	37.60	0.00	37.60	0.00	37.60	0.00	37.60	0.00	37.60	188.00
			Shortning	d4	mm	2.2	0.0	2.2	0.0	2.2	0.0	2.2	0.0	2.2	11.09
	Step 5		Span Length	ls	m	37.60	2.40	37.60	2.40	37.60	2.40	37.60	2.40	37.60	197.60
			Shortning	d5	mm	3.5	0.1	3.5	0.1	3.5	0.1	3.5	0.1	3.5	18.15
Contunued Span	Step 6		Span Length	ls	m	37.60	2.40	37.60	2.40	37.60	2.40	37.60	2.40	37.60	197.60
			Shortning	d6	mm	7.9	0.2	7.9	0.2	7.9	0.2	7.9	0.2	7.9	40.49
	Temperature Effect		Span Length	ls	m	37.60	2.40	37.60	2.40	37.60	2.40	37.60	2.40	37.60	197.60
			Expansion	Δlt+	mm	-10.3	-0.7	-10.3	-0.7	-10.3	-0.7	-10.3	-0.7	-10.3	-53.99
			Shortening	Δlt-	mm	7.00	0.45	7.00	0.45	7.00	0.45	7.00	0.45	7.00	36.81
	Dead Load		Shortning	⊿l(+)	mm	21.3	0.3	21.3	0.3	21.3	0.3	21.3	0.3	21.3	107.93
	Simple Beam	Step 2+3	Shortning	DL1	mm	7.6	0.0	7.6	0.0	7.6	0.0	7.6	0.0	7.6	38.20
	Cross Beam	Step 4+5	Shortning	DL2	mm	5.8	0.1	5.8	0.1	5.8	0.1	5.8	0.1	5.8	29.24
	Continus Beam	Step 6	Shortning	DL3	mm	7.9	0.2	7.9	0.2	7.9	0.2	7.9	0.2	7.9	40.49
		Tempertuer EfFect	Expansion	DLt(+)	mm	-10.3	-0.7	-10.3	-0.7	-10.3	-0.7	-10.3	-0.7	-10.3	-53.99
			Shortning	DLt(-)	mm	7.0	0.4	7.0	0.4	7.0	0.4	7.0	0.4	7.0	36.81
Service I for Bearing			Shortning	⊿l(+)	mm	34.0	0.9	34.0	0.9	34.0	0.9	34.0	0.9	34.0	173.69
			Expansion	⊿l(-)	mm	-12.3	-0.8	-12.3	-0.8	-12.3	-0.8	-12.3	-0.8	-12.3	-64.79
			Total	⊿l	mm	46.3	1.7	46.3	1.7	46.3	1.7	46.3	1.7	46.3	238.48
Service I for Expansion Joints			Shortning	⊿l(+)	mm	17.9	0.8	17.9	0.8	17.9	0.8	17.9	0.8	17.9	92.76
			Expansion	⊿l(-)	mm	-12.3	-0.8	-12.3	-0.8	-12.3	-0.8	-12.3	-0.8	-12.3	-64.79
			Total	⊿l	mm	30.2	1.6	30.2	1.6	30.2	1.6	30.2	1.6	30.2	157.55

Desplacement for each bearings Iskp= 0 The movement direction of the fulcrum assumes that symbol(->) is plus.

			Symbol	Unit	1	2	3	4	5	6	7	8	9	10
Number Support					A1	P1	P1	P2	P2	P3	P3	P4	P4	P5
Condition of Support			Ibc		1	0	0	0	0	2	0	0	0	0
			-		Mv	E	E	E	E	F	E	E	E	E
			Ns	nos	6	6	6	6	6	6	6	6	6	6
Data of Support condition		Distance from S1	L	m	0.0	37.6	40.0	77.6	80.0	117.6	120.0	157.6	160.0	197.6
		Spring for One Bearing	Ks1	kN/m	0	1,791	1,791	3,134	3,134	3,134	3,134	3,134	3,134	1,791
		Total Spring of Bearing	Ks	kN/m	0	10,746	10,746	18,806	18,806	18,806	18,806	18,806	18,806	10,746
		Supring of Substructure	Kp	kN/m	1,082,516	259,462	259,462	259,462	259,462	259,462	259,462	137,646	137,646	137,646
		Compositting Spring	K	kN/m	0	10,319	10,319	17,535	17,535	17,535	17,535	16,546	16,546	9,968
		All Toal Spring	ΣK	kN/m	133,838.6									
		Spring x Distance	K*L	kN	0.0	387,996	412,761	1,360,737	1,402,821	2,062,147	2,104,232	2,607,595	2,647,305	1,969,713
		Total Spring x Distance	Σk*L	kN	14,955,306									
		Distannce from S1 to Fixed Point	x	m	111.741									
Simple Beam	Step 2	Condition of Support	-	-	Mv	E	E	E	E	F	E	E	E	E
		Total Spring of Bearing	Ks1		0	10,746	10,746	18,806	18,806	18,806	18,806	18,806	18,806	10,746
		Supring of Substructure	Kp	kN/m	1,082,516	259,462	259,462	259,462	259,462	259,462	259,462	137,646	137,646	137,646
		Compositting Spring	Ksp	kN/m	0.0	10319.0	10319.0	17535.3	17535.3	17535.3	17535.3	16545.7	16545.7	9968.2
		Span Length	lsi	m	37.600	0.000	37.600	0.000	37.600	0.000	37.600	0.000	37.600	0.000
		Distance from S1 to Si	L	m	0.0	37.6	0.0	37.6	0.0	37.6	0.0	37.6	0.0	37.6
		Total Spring(Each Span)	Σksi	kN/m	10319.03		27854.3		35070.5		34080.9		26513.8	
		Movement of Each Span	δdi	mm	2.2	0.0	2.2	0.0	2.2	0.0	2.2	0.0	2.2	0.0
		Si is Asummed fix,Movement of S		〃	0.0	2.2	0.0	2.2	0.0	2.2	0.0	2.2	0.0	2.2
		Reaction of Each Supports	F and Fi	kN	22639.5	22639.5	38471.6	38471.6	38471.6	38471.6	36300.5	36300.5	21869.8	21869.8
		Total Movement	Δsl(1)	mm	2.2	0.0	1.4	-0.8	1.1	-1.1	1.1	-1.1	0.8	-1.4
		Movemnt of pier	Δpl(1)	mm	0.0	0.0	0.1	-0.1	0.1	-0.1	0.1	-0.1	0.1	-0.1
		Movemen of Bearing	Δsl(1)	mm	2.2	0.0	1.3	-0.8	1.0	-1.0	1.0	-1.0	0.7	-1.3
	Step 3	Condition of Support	-	-	Mv	E	E	E	E	F	E	E	E	E
		Total Spring of Bearing	Ks1		0	10,746	10,746	18,806	18,806	18,806	18,806	18,806	18,806	10,746
		Supring of Substructure	Kp	kN/m	1,082,516	259,462	259,462	259,462	259,462	259,462	259,462	137,646	137,646	137,646
		Compositting Spring	Ksp	kN/m	0.0	10319.0	10319.0	17535.3	17535.3	17535.3	17535.3	16545.7	16545.7	9968.2
		Span Length	lsi	m	37.600	2.400	37.600	2.400	37.600	2.400	37.600	2.400	37.600	0.000
		Distance from S1 to Si	L	m	0.0	40.0	0.0	40.0	0.0	40.0	0.0	40.0	0.0	37.6
		Total Spring(Each Span)	Σksi	kN/m	10319.03		27854.3		35070.5		34080.9		26513.8	
		Movement of Each Span	δdi	mm	5.4	0.0	5.4	0.0	5.4	0.0	5.4	0.0	5.4	0.0
		Si is Asummed fix,Movement of S		〃	0.0	5.4	0.0	5.4	0.0	5.4	0.0	5.4	0.0	5.4
		Reaction of Each Supports	F and Fi	kN	56206.0	56206.0	95511.6	95511.6	95511.6	95511.6	90121.4	90121.4	54295.0	54295.0
		Total Movement	Δsl(1)	mm	5.4	0.0	3.4	-2.0	2.7	-2.7	2.6	-2.8	2.0	-3.4
		Movemnt of pier	Δpl(1)	mm	0.0	0.0	0.1	-0.1	0.2	-0.2	0.2	-0.3	0.2	-0.2
		Movemen of Bearing	Δsl(1)	mm	5.4	0.0	3.3	-1.9	2.5	-2.5	2.5	-2.5	1.8	-3.2
	Step 4	Condition of Support	-	-	Mv	E	E	E	E	F	E	E	E	E
		Total Spring of Bearing	Ks1		0	10,746	10,746	18,806	18,806	25,946,225,010	18,806	18,806	18,806	10,746
		Supring of Substructure	Kp	kN/m	1,082,516	259,462	259,462	259,462	259,462	259,462	259,462	137,646	137,646	137,646
		Compositting Spring	Ks	kN/m	0.0	10319.0	10319.0	17535.3	17535.3	259459.7	17,535	16,546	16545.7	9968.2
		Span Length	lsi	m	37.600	0.000	37.600	0.000	37.600	0.000	37.600	0.000	37.600	0.000
		Distance from S1 to Si	L	m	0.0	37.6	0.0	37.6	0.0	37.6	0.0	37.6	0.0	37.6
		Total Spring(Each Span)	Σksi	kN/m	10319.03		27854.3		276994.9		34,081		26513.8	
		Movement of Each Span	δdi	mm	2.2	0.0	2.2	0.0	2.2	0.0	2.2	0.0	2.2	0.0
		Si is Asummed fix,Movement of S		〃	0.0	2.2	0.0	2.2	0.0	2.2	0.0	2.2	0.0	2.2
		Reaction of Each Supports	F and Fi	kN	22879.4	22879.4	38879.3	38879.3	575275.9	575275.9	36,685	36,685	22101.5	22101.5
		Total Movement	Δsl(1)	mm	2.2	0.0	1.4	-0.8	2.1	-0.1	1.1	-1.1	0.8	-1.4
		Movemnt of pier	Δpl(1)	mm	0.0	0.0	0.1	-0.1	0.1	-0.1	0.1	-0.1	0.1	-0.1
		Movemen of Bearing	Δsl(1)	mm	2.2	0.0	1.3	-0.8	1.9	0.0	1.0	-1.0	0.7	-1.3

Number Support			Symbol	Unit	1	2	3	4	5	6	7	8	9	10
					A1	P1	P1	P2	P2	P3	P3	P4	P4	P5
Continuas Beam	Step 5	Condition of Support	-	-	Mv	E	E	E	E	F	E	E	E	E
		Total Spring of Bearing	Ks1		0	10,746	10,746	18,806	18,806	25,946,225,010	18,806	18,806	18,806	10,746
		Supring of Substructure	Ks	kN/m	1,082,516	259,462	259,462	259,462	259,462	259,462	259,462	137,646	137,646	137,646
		Compositting Spring	Kp	kN/m	0	10,319	10,319	17,535	17,535	259,460	17,535	16,546	16,546	9,968
		Span Length	lsi	m	37.600	2.400	37.600	2.400	37.600	2.400	37.600	2.400	37.600	0.000
		Distance from S1 to Si	L	m	0.0	37.6	40.0	77.6	80.0	117.6	120.0	157.6	160.0	197.6
		Total Spring(Each Span)	Σksi	kN/m	375,763									
		Movement of Each Span	δdi	mm	3.5	0.1	3.5	0.1	3.5	0.1	3.5	0.1	3.5	0.0
		Si is Asummed fix,Movement of S	δb	〃	0.0	3.5	3.7	7.2	7.3	10.8	11.0	14.5	14.6	18.2
		Reaction of Each Supports	F and Fi	kN	3,996,735	36,496	37,701	126,084	128,131	2,813,530	192,196	239,868	241,799	180,931
		Total Movement	Δsl(1)	mm	10.6	7.1	7.0	3.4	3.3	-0.2	-0.3	-3.9	-4.0	-7.5
		Movemnt of pier	Δpl(1)	mm	0.0	0.3	0.3	0.2	0.2	-0.2	0.0	-0.5	-0.5	-0.5
		Movemen of Bearing	Δsl(1)	mm	10.6	6.8	6.7	3.2	3.1	0.0	-0.3	-3.4	-3.5	-7.0
	Step 6	Condition of Support	-	-	Mv	E	E	E	E	F	E	E	E	E
		Total Spring of Bearing	Ks1		0	10,746	10,746	18,806	18,806	25,946,225,010	18,806	18,806	18,806	10,746
		Supring of Substructure	Ks	kN/m	1,082,516	259,462	259,462	259,462	259,462	259,462	259,462	137,646	137,646	137,646
		Compositting Spring	Kp	kN/m	0	10,319	10,319	17,535	17,535	259,460	17,535	16,546	16,546	9,968
		Span Length	lsi	m	37.600	2.400	37.600	2.400	37.600	2.400	37.600	2.400	37.600	0.000
		Distance from S1 to Si	L	m	0.0	37.6	40.0	77.6	80.0	117.6	120.0	157.6	160.0	197.6
		Total Spring(Each Span)	Σksi	kN/m	375,763									
		Movement of Each Span	δdi	mm	7.9	0.2	7.9	0.2	7.9	0.2	7.9	0.2	7.9	0.0
		Si is Asummed fix,Movement of S	δb	〃	0.0	7.9	8.1	16.1	16.3	24.2	24.4	32.3	32.6	40.5
		Reaction of Each Supports	F and Fi	kN	8,918,984	81,782	83,997	281,711	285,475	6,280,335	428,212	535,177	538,728	403,567
		Total Movement	Δsl(1)	mm	23.7	15.8	15.6	7.7	7.5	-0.5	-0.7	-8.6	-8.8	-16.7
		Movemnt of pier	Δpl(1)	mm	0.0	0.6	0.6	0.5	0.5	-0.5	0.0	-1.0	-1.1	-1.2
		Movemen of Bearing	Δsl(1)	mm	23.7	15.2	15.0	7.2	7.0	0.0	-0.6	-7.6	-7.8	-15.5
	Temperture EfFect	Condition of Support	-	-	Mv	E	E	E	E	F	E	E	E	E
		Total Spring of Bearing	Ks1		0	10,746	10,746	18,806	18,806	25,946,225,010	18,806	18,806	18,806	10,746
		Supring of Substructure	Ks	kN/m	1,082,516	259,462	259,462	259,462	259,462	259,462	259,462	137,646	137,646	137,646
		Compositting Spring	Kp	kN/m	0	10,319	10,319	17,535	17,535	259,460	17,535	16,546	16,546	9,968
		Span Length	lsi	m	37.600	2.400	37.600	2.400	37.600	2.400	37.600	2.400	37.600	0.000
		Distance from S1 to Si	L	m	0.0	37.6	40.0	77.6	80.0	117.6	120.0	157.6	160.0	197.6
		Total Spring(Each Span)	Σksi	kN/m	375,763									
		Rise in Temperture: ovement of each sp	δdi	mm	-10.3	-0.7	-10.3	-0.7	-10.3	-0.7	-10.3	-0.7	-10.3	0.0
		Rise in Temperture: Total Movement	δb	〃	0	-10.3	-10.9	-21.2	-21.9	-32.1	-32.8	-43.1	-43.7	-54.0
		Rise in Temperture: Reaction	F and Fi	kN	-11860150	-106016	-112783	-371808	-383307	-8337223	-574960	-712499	-723349	-538204
		Rise in Temperture: Total Movement	Δsl(1)	mm	-31.6	-21.3	-20.6	-10.4	-9.7	0.6	1.2	11.5	12.2	22.4
		Rise in Temperture: Movemnt of pier	Δpl(1)	mm	0.0	-0.8	-0.8	-0.7	-0.7	0.6	0.1	1.4	1.5	1.6
		Rise in Temperture: Movemen of Bearin	Δsl(1)	mm	-31.6	-20.4	-19.8	-9.7	-9.0	0.0	1.1	10.1	10.7	20.8
		Fall in Temperture: ovement of each sp	δdi	mm	7.0	0.4	7.0	0.4	7.0	0.4	7.0	0.4	7.0	0.0
		Fall in Temperture: Total Movement	δb	〃	0	7.0	7.5	14.5	14.9	21.9	22.4	29.4	29.8	36.8
		Fall in Temperture: Reaction	F and Fi	kN	8,086,466	72,283.6	76,897.4	253,505.2	261,345.6	5,684,470.5	392,018.4	485,794.9	493,192.8	366,957.5
		Fall in Temperture: Total Movement	Δsl(1)	mm	21.5	14.5	14.1	7.1	6.6	-0.4	-0.8	-7.8	-8.3	-15.3
		Fall in Temperture: Movemnt of pier	Δpl(1)	mm	0.0	0.6	0.6	0.5	0.4	-0.4	-0.1	-0.9	-1.0	-1.1
		Fall in Temperture: Iovemen of Beari	Δsl(1)	mm	21.5	13.9	13.5	6.6	6.2	0.0	-0.8	-6.9	-7.3	-14.2

Combination of Movement at Suppoert for Barings

Number of Bearing		Symbol	Unit	1	2	3	4	5	6	7	8	9	10
Name of Substructure				S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
Step 2	Shorthing	δi	mm	2.2	0.0	1.3	-0.8	1.0	-1.0	1.0	-1.0	0.7	-1.3
Step 3	Shorthing	δi	mm	5.4	0.0	3.3	-1.9	2.5	-2.5	2.5	-2.5	1.8	-3.2
Step 4	Shorthing	δi	mm	2.2	0.0	1.3	-0.8	1.9	0.0	1.0	-1.0	0.7	-1.3
Step 5	Shorthing	δi	mm	10.6	6.8	6.7	3.2	3.1	0.0	-0.3	-3.4	-3.5	-7.0
Step 6	Shorthing	δi	mm	23.7	15.2	15.0	7.2	7.0	0.0	-0.6	-7.6	-7.8	-15.5
Dead Load	Shorthing	δdi	mm	44.2	22.0	27.6	7.0	15.6	-3.6	3.5	-15.4	-8.0	-28.2
Temperture Effect	Rise in Temperture	δsI(T+)	mm	-31.6	-20.4	-19.8	-9.7	-9.0	0.0	1.1	10.1	10.7	20.8
	Fall in Temperture	δsI(T-)	mm	21.5	13.9	13.5	6.6	6.2	0.0	-0.8	-6.9	-7.3	-14.2
Service I	Expansion	δsI(T+)	mm	15.2	1.9	9.4	-3.2	7.8	-4.3	5.6	-6.4	3.2	-8.9
	Shorthing	δsI(T-)	mm	78.9	43.1	49.4	16.3	26.1	-4.3	3.3	-26.8	-18.4	-50.9
	TotaL Movement	δsI	mm	94.1	45.0	58.8	19.5	33.9	8.5	8.9	33.2	21.6	59.8

Number of Bearing		Symbol	Unit	1	2	3	4	5	6	7	8	9	10
Name of Substructure				S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
Step 2	Shorthing	δi	mm	2.2	0.0	1.4	-0.8	1.1	-1.1	1.1	-1.1	0.8	-1.4
Step 3	Shorthing	δi	mm	5.4	0.0	3.3	-1.9	2.5	-2.5	2.5	-2.5	1.8	-3.2
Step 4	Shorthing	δi	mm	2.2	0.0	1.4	-0.8	2.1	-0.1	1.1	-1.1	0.8	-1.4
Step 5	Shorthing	δi	mm	10.6	6.8	6.7	3.2	3.1	0.0	-0.3	-3.4	-3.5	-7.0
Step 6	Shorthing	δi	mm	23.7	15.8	15.6	7.7	7.5	-0.5	-0.7	-8.6	-8.8	-16.7
Dead Load	Shorthing	δdi	mm	44.2	22.6	28.4	7.4	16.3	-4.2	3.6	-16.7	-8.9	-29.6
Temperture Effect	Rise in Temperture	δsI(T+)	mm	-31.6	-21.3	-20.6	-10.4	-9.7	0.6	1.2	11.5	12.2	22.4
	Fall in Temperture	δsI(T-)	mm	21.5	14.5	14.1	7.1	6.6	-0.4	-0.8	-7.8	-8.3	-15.3
Service I	Expansion	δsI(T+)	mm	12.7	1.3	7.7	-3.0	6.6	-3.7	4.8	-5.2	3.3	-7.2
	Shorthing	δsI(T-)	mm	65.8	37.1	42.4	14.4	22.9	-4.6	2.8	-24.6	-17.2	-44.9
	TotaL Movement	δsI	mm	78.4	38.5	50.2	17.4	29.5	8.3	7.6	29.8	20.4	52.1

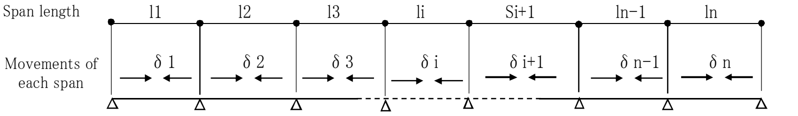
Combination of Movement at Suppoert for Expansion Joints

Number of Bearing		Symbol	Unit	1	2	3	4	5	6	7	8	9	10
Name of Substructure				S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
Dead Load	Shorthing	δdi	mm	23.7	15.2	15.0	7.2	7.0	0.0	-0.6	-7.6	-7.8	-15.5
Temperture Effect	Rise in Temperture	δsI(T+)	mm	-31.6	-20.4	-19.8	-9.7	-9.0	0.0	1.1	10.1	10.7	20.8
	Movemen of Bearing	δsI(T-)	mm	21.5	13.9	13.5	6.6	6.2	0.0	-0.8	-6.9	-7.3	-14.2
Service I	Expansion	δsI(T+)	mm	24.9	17.6	18.6	12.0	13.0	7.2	7.8	2.0	3.0	-3.5
	Shorthing	δsI(T-)	mm	28.5	18.2	18.0	8.6	8.3	0.0	-0.8	-9.1	-9.3	-18.6
	TotaL Movement	δsI	mm	53.4	35.8	36.5	20.5	21.3	7.2	8.5	11.1	12.4	22.2

Caluculation Metho for Movement of Each Bearing

When quantity of movement of each span is know Elastic shortening of each span is ignore.

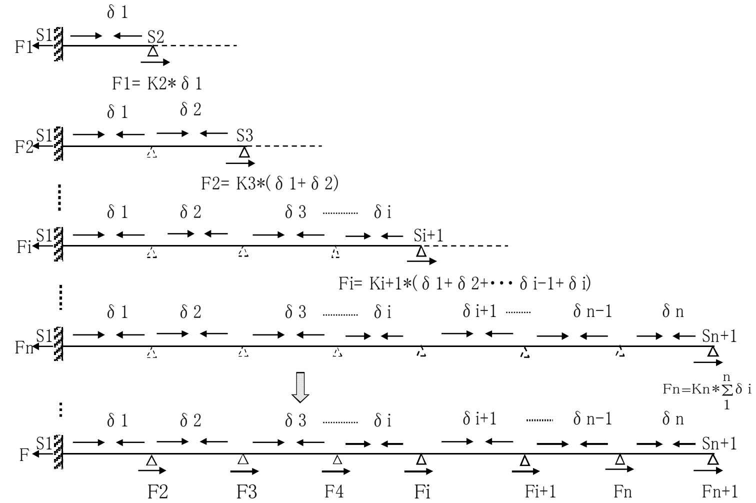
n ; Number of span



Name of Support	S1	S2	S3	Si	Si+1	Sn−1	Sn	Sn+1
Spring of Support	K1	K2	K3	Ki	Ki+1	Kn−1	Kn	Kn+1

(1) Reaction of S1 when S1 is fixed F= Σ K∗ Δ l(1)

S1 reaction force is determined by fixing the S1, the total reaction force of each bearing.



$$F = F_2 + F_3 + \cdots F_i + \cdots + F_{n+1} = K2 * \delta_1 + K3 * (\delta_1 + \delta_2) + \cdots + K_{i+1} * (\delta_1 + \delta_2 + \cdots \delta_i) + \cdots + K_{n+1} * \sum_{1}^{n} \delta_i = \sum_{i=1}^{n} (K_{i+1} \sum_{j=1}^{i} \delta_i)$$

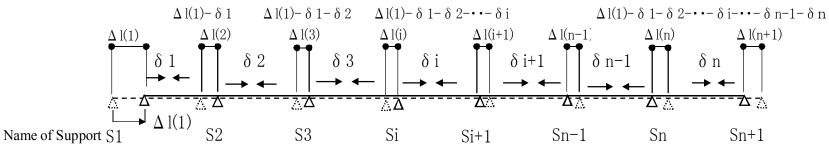
(2) Calculation of (1) Δl amount of movement of the bearing-S1

from F=K∗ δ so δ =F/K

$$\Delta l(1) = \frac{F}{K} = \frac{\sum_{i=1}^{n} Fi}{\sum_{i=1}^{n+1} Ki} = \frac{\sum_{i=1}^{n} K_{i+1}(\sum_{i=1}^{n} \delta_i)}{\sum_{i=1}^{n+1} K_i}$$

(3) Calculation of the movement Δl(i) at each bearing.

The movement Δl(i) at each bearing. is calculated by subtracting the sum of the movement amount δ from each span up (1) Δl amount of movement of the fulcrum S1 Δl amount of movement of each bearing.



S1 Δ l(1) = Δ l(1)

S2 Δ l(2) = Δ l(1)− δ 1 = Δ l(1)− Σ δ i (i=1,1)

S3 Δ l(3) = Δ l(1)− δ 1− δ 2 = Δ l(1)− Σ δ i (i=1.2)

・

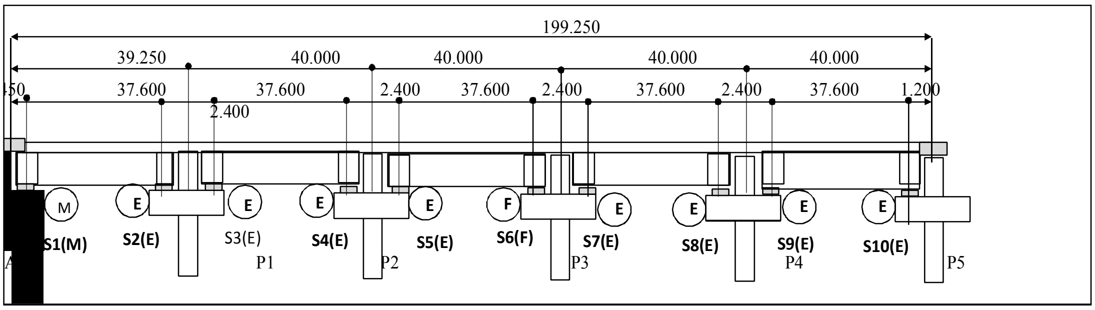
Si Δ l(i) = Δ l(1)− δ 1− δ 2− δ 3−・・・・− δ $_{i-1}$ = Δ l(1)− Σ δ $_{i\,(i=1,i-1)}$

・

Sn Δ l(n) = Δ l(1)− δ 1− δ 2− δ 3−・・・・− δ i−・・・・− δ n−1 = Δ l(1)− Σ δ $_{i\,(i=1,n-1)}$

Sn+1 Δ l(n+1) = Δ l(1)− δ 1− δ 2− δ 3−・・・− δ i−・・・・− δ n−1− δ n = Δ l(1)− Σ δ $_{i\,(i=1,n)}$

Appendix 1. **Desplacement at the bearings** First Assump Not Final



Movements for each span The expansion assumes the course that a beam shorts plus.

						NO.1Span	NO.2Span	NO.3Span	NO.4Span	NO.5Span	NO.6Span	NO.7Span	NO.8Span	NO.9Span			SUM
						1	2	3	4	5	6	7	8	9			
Simple Beam	Step 1		Span Length	ls	m	37.60	0.00	37.60	0.00	37.60	0.00	37.60	0.00	37.60			188.00
			Shortning	d1	mm	29.4	0.0	29.4	0.0	29.4	0.0	29.4	0.0	29.4			146.77
	Step 2		Span Length	ls	m	37.60	0.00	37.60	0.00	37.60	0.00	37.60	0.00	37.60			188.00
			Shortning	d2	mm	2.2	0.0	2.2	0.0	2.2	0.0	2.2	0.0	2.2			10.97
	Step 3		Span Length	ls	m	37.60	0.00	37.60	0.00	37.60	0.00	37.60	0.00	37.60			188.00
			Shortning	d3	mm	5.4	0.0	5.4	0.0	5.4	0.0	5.4	0.0	5.4			27.23
	Step 4		Span Length	ls	m	37.60	0.00	37.60	0.00	37.60	0.00	37.60	0.00	37.60			188.00
			Shortning	d4	mm	2.2	0.0	2.2	0.0	2.2	0.0	2.2	0.0	2.2			11.09
	Step 5		Span Length	ls	m	37.60	2.40	37.60	2.40	37.60	2.40	37.60	2.40	37.60			197.60
			Shortning	d5	mm	3.5	0.1	3.5	0.1	3.5	0.1	3.5	0.1	3.5			18.15
Contunued Span	Step 6		Span Length	ls	m	37.60	2.40	37.60	2.40	37.60	2.40	37.60	2.40	37.60			197.60
			Shortning	d6	mm	7.9	0.2	7.9	0.2	7.9	0.2	7.9	0.2	7.9			40.49
	Temperature Effect		Span Length	ls	m	37.60	2.40	37.60	2.40	37.60	2.40	37.60	2.40	37.60			197.60
			Expansion	Δlt+	mm	-10.3	-0.7	-10.3	-0.7	-10.3	-0.7	-10.3	-0.7	-10.3			-53.99
			Shortening	Δlt-	mm	7.00	0.45	7.00	0.45	7.00	0.45	7.00	0.45	7.00			36.81
	Dead Load		Shortning	⊿l(+)	mm	21.3	0.3	21.3	0.3	21.3	0.3	21.3	0.3	21.3			107.93
	Simple Beam	Step 2+3	Shortning	DL1	mm	7.6	0.0	7.6	0.0	7.6	0.0	7.6	0.0	7.6			38.20
	Cross Beam	Step 4+5	Shortning	DL2	mm	5.8	0.1	5.8	0.1	5.8	0.1	5.8	0.1	5.8			29.24
	Continus Beam	Step 6	Shortning	DL3	mm	7.9	0.2	7.9	0.2	7.9	0.2	7.9	0.2	7.9			40.49
		Temperture EfFect	Expansion	DLt(+)	mm	-10.3	-0.7	-10.3	-0.7	-10.3	-0.7	-10.3	-0.7	-10.3			-53.99
			Shortning	DLt(-)	mm	7.0	0.4	7.0	0.4	7.0	0.4	7.0	0.4	7.0			36.81
Service I for Bearing			Shortning	⊿l(+)	mm	34.0	0.9	34.0	0.9	34.0	0.9	34.0	0.9	34.0			173.69
			Expansion	⊿l(-)	mm	-12.3	-0.8	-12.3	-0.8	-12.3	-0.8	-12.3	-0.8	-12.3			-64.79
			Total	⊿l	mm	46.3	1.7	46.3	1.7	46.3	1.7	46.3	1.7	46.3			238.48
Service I for Expansion Joints			Shortning	⊿l(+)	mm	17.9	0.8	17.9	0.8	17.9	0.8	17.9	0.8	17.9			92.76
			Expansion	⊿l(-)	mm	-12.3	-0.8	-12.3	-0.8	-12.3	-0.8	-12.3	-0.8	-12.3			-64.79
			Total	⊿l	mm	30.2	1.6	30.2	1.6	30.2	1.6	30.2	1.6	30.2			157.55

Desplacement for each bearings Iskp= 0 The movement direction of the fulcrum assumes that symbol(->) is plus.

			Symbol	Unit	1	2	3	4	5	6	7	8	9	10
Number Support					A1	P1	P1	P2	P2	P3	P3	P4	P4	P5
Condition of Support			Ibc		1	0	0	0	0	2	0	0	0	0
			-		Mv	E	E	E	E	F	E	E	E	E
			Ns	nos	6	6	6	6	6	6	6	6	6	6
Data of Support condition		Distance from S1	L	m	0.0	37.6	40.0	77.6	80.0	117.6	120.0	157.6	160.0	197.6
		Spring for One Bearing	Ks1	kN/m	0	1,791	1,791	3,134	3,134	3,134	3,134	3,134	3,134	1,791
		Total Spring of Bearing	Ks	kN/m	0	10,746	10,746	18,806	18,806	18,806	18,806	18,806	18,806	10,746
		Suppring of Substructure	Kp	kN/m	1,082,516	259,462	259,462	259,462	259,462	259,462	259,462	137,646	137,646	137,646
		Compositting Spring	K	kN/m	0	10,319	10,319	17,535	17,535	17,535	17,535	16,546	16,546	9,968
		All Toal Spring	ΣK	kN/m	133,838.6									
		Spring x Distance	K*L	kN	0.0	387,996	412,761	1,360,737	1,402,821	2,062,147	2,104,232	2,607,595	2,647,305	1,969,713
		Total Spring x Distance	Σk*L	kN	14,955,306									
		Distannce from S1 to Fixed Point	x	m	111.741									
Simple Beam	Step 2	Condition of Support	-	-	Mv	E	E	E	E	F	E	E	E	E
		Total Spring of Bearing	Ks1		0	10,746	10,746	18,806	18,806	18,806	18,806	18,806	18,806	10,746
		Suppring of Substructure	Kp	kN/m	1,082,516	259,462	259,462	259,462	259,462	259,462	259,462	137,646	137,646	137,646
		Compositting Spring	Ksp	kN/m	0.0	10319.0	10319.0	17535.3	17535.3	17535.3	17535.3	16545.7	16545.7	9968.2
		Span Length	lsi	m	37.600	0.000	37.600	0.000	37.600	0.000	37.600	0.000	37.600	0.000
		Distance from S1 to Si	L	m	0.0	37.6	0.0	37.6	0.0	37.6	0.0	37.6	0.0	37.6
		Total Spring(Each Span)	Σksi	kN/m	10319.03		27854.3		35070.5		34080.9		26513.8	
		Movement of Each Span	δdi	mm	2.2	0.0	2.2	0.0	2.2	0.0	2.2	0.0	2.2	0.0
		Si is Asummed fix,Movement of S		〃	0.0	2.2	0.0	2.2	0.0	2.2	0.0	2.2	0.0	2.2
		Reaction of Each Supports	F and Fi	kN	22639.5	22639.5	38471.6	38471.6	38471.6	38471.6	36300.5	36300.5	21869.8	21869.8
		Total Movement	Δsl(1)	mm	2.2	0.0	1.4	-0.8	1.1	-1.1	1.1	-1.1	0.8	-1.4
		Movemnt of pier	Δpl(1)	mm	0.0	0.0	0.1	-0.1	0.1	-0.1	0.1	-0.1	0.1	-0.1
		Movemen of Bearing	Δsl(1)	mm	2.2	0.0	1.3	-0.8	1.0	-1.0	1.0	-1.0	0.7	-1.3
	Step 3	Condition of Support	-	-	Mv	E	E	E	E	F	E	E	E	E
		Total Spring of Bearing	Ks1		0	10,746	10,746	18,806	18,806	18,806	18,806	18,806	18,806	10,746
		Suppring of Substructure	Kp	kN/m	1,082,516	259,462	259,462	259,462	259,462	259,462	259,462	137,646	137,646	137,646
		Compositting Spring	Ksp	kN/m	0.0	10319.0	10319.0	17535.3	17535.3	17535.3	17535.3	16545.7	16545.7	9968.2
		Span Length	lsi	m	37.600	2.400	37.600	2.400	37.600	2.400	37.600	2.400	37.600	0.000
		Distance from S1 to Si	L	m	0.0	40.0	0.0	40.0	0.0	40.0	0.0	40.0	0.0	37.6
		Total Spring(Each Span)	Σksi	kN/m	10319.03		27854.3		35070.5		34080.9		26513.8	
		Movement of Each Span	δdi	mm	5.4	0.0	5.4	0.0	5.4	0.0	5.4	0.0	5.4	0.0
		Si is Asummed fix,Movement of S		〃	0.0	5.4	0.0	5.4	0.0	5.4	0.0	5.4	0.0	5.4
		Reaction of Each Supports	F and Fi	kN	56206.0	56206.0	95511.6	95511.6	95511.6	95511.6	90121.4	90121.4	54295.0	54295.0
		Total Movement	Δsl(1)	mm	5.4	0.0	3.4	-2.0	2.7	-2.7	2.6	-2.8	2.0	-3.4
		Movemnt of pier	Δpl(1)	mm	0.0	0.0	0.1	-0.1	0.2	-0.2	0.2	-0.3	0.2	-0.2
		Movemen of Bearing	Δsl(1)	mm	5.4	0.0	3.3	-1.9	2.5	-2.5	2.5	-2.5	1.8	-3.2
	Step 4	Condition of Support	-	-	Mv	E	E	E	E	F	E	E	E	E
		Total Spring of Bearing	Ks1		0	10,746	10,746	18,806	18,806	25,946,225,010	18,806	18,806	18,806	10,746
		Suppring of Substructure	Kp	kN/m	1,082,516	259,462	259,462	259,462	259,462	259,462	259,462	137,646	137,646	137,646
		Compositting Spring	Ks	kN/m	0.0	10319.0	10319.0	17535.3	17535.3	259459.7	17,535	16,546	16545.7	9968.2
		Span Length	lsi	m	37.600	0.000	37.600	0.000	37.600	0.000	37.600	0.000	37.600	0.000
		Distance from S1 to Si	L	m	0.0	37.6	0.0	37.6	0.0	37.6	0.0	37.6	0.0	37.6
		Total Spring(Each Span)	Σksi	kN/m	10319.03		27854.3		276994.9		34,081		26513.8	
		Movement of Each Span	δdi	mm	2.2	0.0	2.2	0.0	2.2	0.0	2.2	0.0	2.2	0.0
		Si is Asummed fix,Movement of S		〃	0.0	2.2	0.0	2.2	0.0	2.2	0.0	2.2	0.0	2.2
		Reaction of Each Supports	F and Fi	kN	22879.4	22879.4	38879.3	38879.3	575275.9	575275.9	36,685	36,685	22101.5	22101.5
		Total Movement	Δsl(1)	mm	2.2	0.0	1.4	-0.8	2.1	-0.1	1.1	-1.1	0.8	-1.4
		Movemnt of pier	Δpl(1)	mm	0.0	0.0	0.1	-0.1	0.1	-0.1	0.1	-0.1	0.1	-0.1
		Movemen of Bearing	Δsl(1)	mm	2.2	0.0	1.3	-0.8	1.9	0.0	1.0	-1.0	0.7	-1.3

Number Support				Symbol	Unit	1	2	3	4	5	6	7	8	9	10
						A1	P1	P1	P2	P2	P3	P3	P4	P4	P5
Continuas Beam	Step 5	Condition of Support		-	-	Mv	E	E	E	E	F	E	E	E	E
		Total Spring of Bearing		Ks1		0	10,746	10,746	18,806	18,806	25,946,225,010	18,806	18,806	18,806	10,746
		Supring of Substructure		Ks	kN/m	1,082,516	259,462	259,462	259,462	259,462	259,462	259,462	137,646	137,646	137,646
		Compositting Spring		Kp	kN/m	0	10,319	10,319	17,535	17,535	259,460	17,535	16,546	16,546	9,968
		Span Length		lsi	m	37.600	2.400	37.600	2.400	37.600	2.400	37.600	2.400	37.600	0.000
		Distance from S1 to Si		L	m	0.0	37.6	40.0	77.6	80.0	117.6	120.0	157.6	160.0	197.6
		Total Spring(Each Span)		Σksi	kN/m	375,763									
		Movement of Each Span		δdi	mm	3.5	0.1	3.5	0.1	3.5	0.1	3.5	0.1	3.5	0.0
		Si is Asummed fix,Movement of S		δb	〃	0.0	3.5	3.7	7.2	7.3	10.8	11.0	14.5	14.6	18.2
		Reaction of Each Supports		F and Fi	kN	3,996,735	36,496	37,701	126,084	128,131	2,813,530	192,196	239,868	241,799	180,931
		Total Movement		Δsl(1)	mm	10.6	7.1	7.0	3.4	3.3	-0.2	-0.3	-3.9	-4.0	-7.5
		Movemnt of pier		Δpl(1)	mm	0.0	0.3	0.3	0.2	0.2	-0.2	0.0	-0.5	-0.5	-0.5
		Movemen of Bearing		Δsl(1)	mm	10.6	6.8	6.7	3.2	3.1	0.0	-0.3	-3.4	-3.5	-7.0
	Step 6	Condition of Support		-	-	Mv	E	E	E	E	F	E	E	E	E
		Total Spring of Bearing		Ks1		0	10,746	10,746	18,806	18,806	25,946,225,010	18,806	18,806	18,806	10,746
		Supring of Substructure		Ks	kN/m	1,082,516	259,462	259,462	259,462	259,462	259,462	259,462	137,646	137,646	137,646
		Compositting Spring		Kp	kN/m	0	10,319	10,319	17,535	17,535	259,460	17,535	16,546	16,546	9,968
		Span Length		lsi	m	37.600	2.400	37.600	2.400	37.600	2.400	37.600	2.400	37.600	0.000
		Distance from S1 to Si		L	m	0.0	37.6	40.0	77.6	80.0	117.6	120.0	157.6	160.0	197.6
		Total Spring(Each Span)		Σksi	kN/m	375,763									
		Movement of Each Span		δdi	mm	7.9	0.2	7.9	0.2	7.9	0.2	7.9	0.2	7.9	0.0
		Si is Asummed fix,Movement of S		δb	〃	0.0	7.9	8.1	16.1	16.3	24.2	24.4	32.3	32.6	40.5
		Reaction of Each Supports		F and Fi	kN	8,918,984	81,782	83,997	281,711	285,475	6,280,335	428,212	535,177	538,728	403,567
		Total Movement		Δsl(1)	mm	23.7	15.8	15.6	7.7	7.5	-0.5	-0.7	-8.6	-8.8	-16.7
		Movemnt of pier		Δpl(1)	mm	0.0	0.6	0.6	0.5	0.5	-0.5	0.0	-1.0	-1.1	-1.2
		Movemen of Bearing		Δsl(1)	mm	23.7	15.2	15.0	7.2	7.0	0.0	-0.6	-7.6	-7.8	-15.5
	Temperture EfFect	Condition of Support		-	-	Mv	E	E	E	E	F	E	E	E	E
		Total Spring of Bearing		Ks1		0	10,746	10,746	18,806	18,806	25,946,225,010	18,806	18,806	18,806	10,746
		Supring of Substructure		Ks	kN/m	1,082,516	259,462	259,462	259,462	259,462	259,462	259,462	137,646	137,646	137,646
		Compositting Spring		Kp	kN/m	0	10,319	10,319	17,535	17,535	259,460	17,535	16,546	16,546	9,968
		Span Length		lsi	m	37.600	2.400	37.600	2.400	37.600	2.400	37.600	2.400	37.600	0.000
		Distance from S1 to Si		L	m	0.0	37.6	40.0	77.6	80.0	117.6	120.0	157.6	160.0	197.6
		Total Spring(Each Span)		Σksi	kN/m	375,763									
		Rise in Temperture	ovement of each sp	δdi	mm	-10.3	-0.7	-10.3	-0.7	-10.3	-0.7	-10.3	-0.7	-10.3	0.0
			Total Movement	δb	〃	0	-10.3	-10.9	-21.2	-21.9	-32.1	-32.8	-43.1	-43.7	-54.0
			Reaction	F and Fi	kN	-11860150	-106016	-112783	-371808	-383307	-8337223	-574960	-712499	-723349	-538204
			Total Movement	Δsl(1)	mm	-31.6	-21.3	-20.6	-10.4	-9.7	0.6	1.2	11.5	12.2	22.4
			Movemnt of pier	Δpl(1)	mm	0.0	-0.8	-0.8	-0.7	-0.7	0.6	0.1	1.4	1.5	1.6
			Movemen of Bearin	Δsl(1)	mm	-31.6	-20.4	-19.8	-9.7	-9.0	0.0	1.1	10.1	10.7	20.8
		Fall in Temperture	ovement of each sp	δdi	mm	7.0	0.4	7.0	0.4	7.0	0.4	7.0	0.4	7.0	0.0
			Total Movement	δb	〃	0	7.0	7.5	14.5	14.9	21.9	22.4	29.4	29.8	36.8
			Reaction	F and Fi	kN	8,086,466	72,283.6	76,897.4	253,505.2	261,345.6	5,684,470.5	392,018.4	485,794.9	493,192.8	366,957.5
			Total Movement	Δsl(1)	mm	21.5	14.5	14.1	7.1	6.6	-0.4	-0.8	-7.8	-8.3	-15.3
			Movemnt of pier	Δpl(1)	mm	0.0	0.6	0.6	0.5	0.4	-0.4	-0.1	-0.9	-1.0	-1.1
			Iovemen of Bearii	Δsl(1)	mm	21.5	13.9	13.5	6.6	6.2	0.0	-0.8	-6.9	-7.3	-14.2

Combination of Movement at Suppoert for Barings

Number of Bearing		Symbol	Unit	1	2	3	4	5	6	7	8	9	10
Name of Substructure				S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
Step 2	Shorthing	δi	mm	2.2	0.0	1.3	-0.8	1.0	-1.0	1.0	-1.0	0.7	-1.3
Step 3	Shorthing	δi	mm	5.4	0.0	3.3	-1.9	2.5	-2.5	2.5	-2.5	1.8	-3.2
Step 4	Shorthing	δi	mm	2.2	0.0	1.3	-0.8	1.9	0.0	1.0	-1.0	0.7	-1.3
Step 5	Shorthing	δi	mm	10.6	6.8	6.7	3.2	3.1	0.0	-0.3	-3.4	-3.5	-7.0
Step 6	Shorthing	δi	mm	23.7	15.2	15.0	7.2	7.0	0.0	-0.6	-7.6	-7.8	-15.5
Dead Load	Shorthing	δdi	mm	44.2	22.0	27.6	7.0	15.6	-3.6	3.5	-15.4	-8.0	-28.2
Temperture Effect	Rise in Temperture	δsI(T+)	mm	-31.6	-20.4	-19.8	-9.7	-9.0	0.0	1.1	10.1	10.7	20.8
	Fall in Temperture	δsI(T-)	mm	21.5	13.9	13.5	6.6	6.2	0.0	-0.8	-6.9	-7.3	-14.2
Service I	Expansion	δsI(T+)	mm	15.2	1.9	9.4	-3.2	7.8	-4.3	5.6	-6.4	3.2	-8.9
	Shorthing	δsI(T-)	mm	78.9	43.1	49.4	16.3	26.1	-4.3	3.3	-26.8	-18.4	-50.9
	TotaL Movement	δsI	mm	94.1	45.0	58.8	19.5	33.9	8.5	8.9	33.2	21.6	59.8

Number of Bearing		Symbol	Unit	1	2	3	4	5	6	7	8	9	10
Name of Substructure				S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
Step 2	Shorthing	δi	mm	2.2	0.0	1.4	-0.8	1.1	-1.1	1.1	-1.1	0.8	-1.4
Step 3	Shorthing	δi	mm	5.4	0.0	3.3	-1.9	2.5	-2.5	2.5	-2.5	1.8	-3.2
Step 4	Shorthing	δi	mm	2.2	0.0	1.4	-0.8	2.1	-0.1	1.1	-1.1	0.8	-1.4
Step 5	Shorthing	δi	mm	10.6	6.8	6.7	3.2	3.1	0.0	-0.3	-3.4	-3.5	-7.0
Step 6	Shorthing	δi	mm	23.7	15.8	15.6	7.7	7.5	-0.5	-0.7	-8.6	-8.8	-16.7
Dead Load	Shorthing	δdi	mm	44.2	22.6	28.4	7.4	16.3	-4.2	3.6	-16.7	-8.9	-29.6
Temperture Effect	Rise in Temperture	δsI(T+)	mm	-31.6	-21.3	-20.6	-10.4	-9.7	0.6	1.2	11.5	12.2	22.4
	Fall in Temperture	δsI(T-)	mm	21.5	14.5	14.1	7.1	6.6	-0.4	-0.8	-7.8	-8.3	-15.3
Service I	Expansion	δsI(T+)	mm	12.7	1.3	7.7	-3.0	6.6	-3.7	4.8	-5.2	3.3	-7.2
	Shorthing	δsI(T-)	mm	65.8	37.1	42.4	14.4	22.9	-4.6	2.8	-24.6	-17.2	-44.9
	TotaL Movement	δsI	mm	78.4	38.5	50.2	17.4	29.5	8.3	7.6	29.8	20.4	52.1

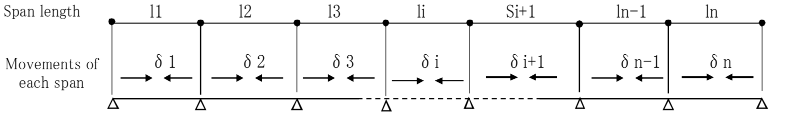
Combination of Movement at Suppoert for Expansion Joints

Number of Bearing		Symbol	Unit	1	2	3	4	5	6	7	8	9	10
Name of Substructure				S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
Dead Load	Shorthing	δdi	mm	23.7	15.2	15.0	7.2	7.0	0.0	-0.6	-7.6	-7.8	-15.5
Temperture Effect	Rise in Temperture	δsI(T+)	mm	-31.6	-20.4	-19.8	-9.7	-9.0	0.0	1.1	10.1	10.7	20.8
	Movemen of Bearing	δsI(T-)	mm	21.5	13.9	13.5	6.6	6.2	0.0	-0.8	-6.9	-7.3	-14.2
Service I	Expansion	δsI(T+)	mm	24.9	17.6	18.6	12.0	13.0	7.2	7.8	2.0	3.0	-3.5
	Shorthing	δsI(T-)	mm	28.5	18.2	18.0	8.6	8.3	0.0	-0.8	-9.1	-9.3	-18.6
	TotaL Movement	δsI	mm	53.4	35.8	36.5	20.5	21.3	7.2	8.5	11.1	12.4	22.2

Caluculation Metho for Movement of Each Bearing

When quantity of movement of each span is know Elastic shortening of each span is ignore.

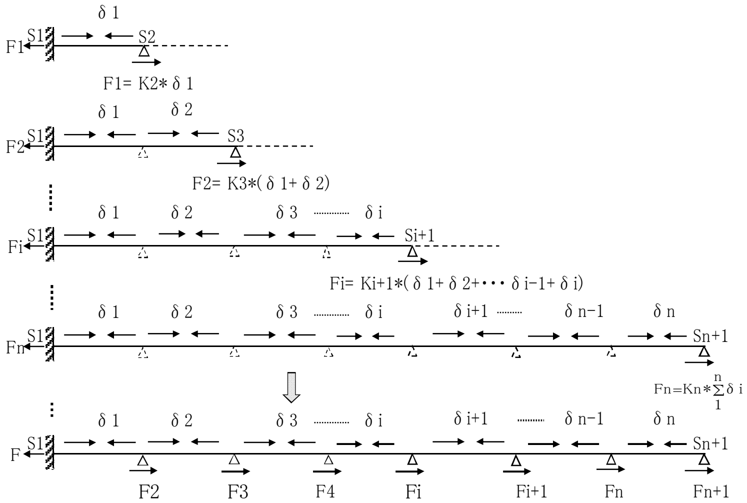
n ; Number of span



Name of Support	S1	S2	S3	Si	Si+1	Sn−1	Sn	Sn+1
Spring of Support	K1	K2	K3	Ki	Ki+1	Kn−1	Kn	Kn+1

(1) Reaction of S1 when S1 is fixed $F=\Sigma K*\Delta l(1)$

S1 reaction force is determined by fixing the S1, the total reaction force of each bearing.



$$F = F_2 + F_3 + \cdots F_i + \cdots + F_{n+1} = K2*\delta_1 + K3*(\delta_1+\delta_2) + \cdots + K_{i+1}*(\delta_1+\delta_2+\cdots\delta_i) + \cdots + K_{n+1}*\sum_{1}^{n}\delta_i = \sum_{i=1}^{n}(K_{i+1}\sum_{j=1}^{i}\delta_i)$$

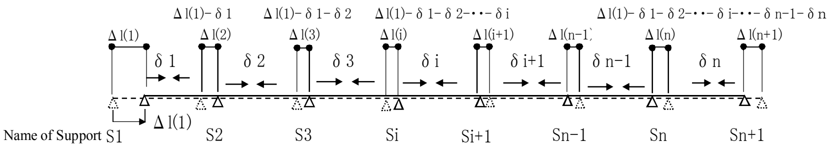
(2) Calculation of (1) Δl amount of movement of the bearing-S1

from $F=K*\delta$ so $\delta = F/K$

$$\Delta l(1) = \frac{F}{K} = \frac{\sum_{i=1}^{n} Fi}{\sum_{i=1}^{n+1} Ki} = \frac{\sum_{i=1}^{n} K_{i+1}(\sum_{i=1}^{n}\delta_i)}{\sum_{i=1}^{n+1} K_i}$$

(3) Calculation of the movement Δl(i) at each bearing.

The movement Δl(i) at each bearing. is calculated by subtracting the sum of the movement amount δ from each span up (1) Δl amount of movement of the fulcrum S1 Δl amount of movement of each bearing.



S1 $\Delta l(1) = \Delta l(1)$

S2 $\Delta l(2) = \Delta l(1) - \delta 1$ $= \Delta l(1) - \Sigma\ \delta i\ (i=1,1)$

S3 $\Delta l(3) = \Delta l(1) - \delta 1 - \delta 2$ $= \Delta l(1) - \Sigma\ \delta i\ (i=1.2)$

・

Si $\Delta l(i) = \Delta l(1) - \delta 1 - \delta 2 - \delta 3 - \cdots - \delta_{i-1}$ $= \Delta l(1) - \Sigma\ \delta_{i\,(i=1,i-1)}$

・

Sn $\Delta l(n) = \Delta l(1) - \delta 1 - \delta 2 - \delta 3 - \cdots - \delta i - \cdots - \delta n-1$ $= \Delta l(1) - \Sigma\ \delta_{i\,(i=1,n-1)}$

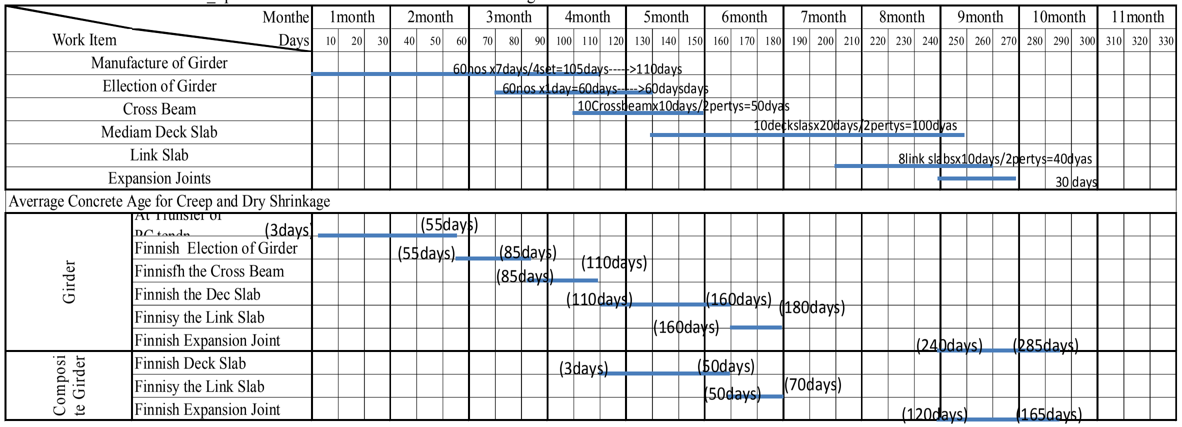
Sn+1 $\Delta l(n+1) = \Delta l(1) - \delta 1 - \delta 2 - \delta 3 - \cdots - \delta i - \cdots - \delta n-1 - \delta n = \Delta l(1) - \Sigma\ \delta_{i\,(i=1,n)}$

Appendix2

Coefficient of Creep and Shrinkage

Assumed progress and concrte age

A1 to P5 6_Span   Number of Girder   Ng= 60 nos



At presstressing tranfer
At finsh election of the girder
At casting concrete of the cross beams
At casting concrete of the deck slab except the link slab
At castiing concrete of link slab
At setting the expansion joints
Concrete age at end of creep and shrinkage

Coefficient of Creep

Assumtion of the concrte age at each construction stage

	Super T girder		**Deck Slab**			
At presstressing tranfer	t0=	3 days	ts0=	3 days		Non Composite girder
At finsh election of the girder	t1=	55 days				Non Composite girder
At casting concrete of the cross beams	t2=	85 days			bearing	Non Composite girder
At casting concrete of the deck slab except the link slab	t3=	110 days	ts1=	50 days	Composite girder	Composite girder
At castiing concrete of link slab	t4=	160 days	ts2=	100 days	Secondary intermidiate force	Composite girder
At setting the expansion joints	t5=	285 days	ts3=	225 days	Expansion joint	Composite girder
Concrete age at end of creep and shrinkage	t6=	11000 days	ts4=	10915 days		Composite girder

Suprt T Girder				Deck Slab			
Item		Unit	Value	Item		Unit	Value
Acg	Area of Girder	m2	0.6377	Acs	Area of Deck Slab	m2	0.3384
φg1	φ(55 ,to)	-	1.136				
φg2	φ(85 ,to)	-	1.278				
φg3	φ(110 ,to)	-	1.364	φd1	φ(50 ,to)	-	1.067
φg4	φ(160 ,to)	-	1.489	φd2	φ(100 ,to)	-	1.278
φg5	φ(285 ,to)	-	1.674	φd3	φ(225 ,to)	-	1.603
φg6	φ(11000 ,to)	-	2.175	φd4	φ(10915 ,to)	-	2.457

For Space of from girder to girder
Untile composite beam

Suprt T Girder φg(t2,t1)				Deck Slab φs(t2,t1)				Compsite φc(t2,t1)	
φg(3 ,055)	1.136	0.000	1.136					φg(3 ,055)	1.136
φg(100 ,120)	1.278	1.136	0.142					φg(100 ,120)	0.142
φg(120 ,180)	1.364	1.278	0.086	φg(3 ,110)	1.067	0.000	1.067	φc(120 ,180)	0.426
φg(180 ,240)	1.489	1.364	0.125	φs(110 ,180)	1.278	1.067	0.210	φc(180 ,240)	0.155
φg(240 ,285)	1.674	1.489	0.185	φs(180 ,200)	1.603	1.278	0.326	φc(240 ,285)	0.233
φg(285 ,11000)	2.175	1.674	0.501	φs(200 ,10820)	2.457	1.603	0.854	φs(285 ,11000)	0.624
φg(3 ,11000)			2.175	φs(3 ,10820)			2.457		2.716

Creep and Shrinkage from MC90

CREEP MC90 $\psi(t0,t)$ A creep coefficient of concrete increcing by materials age t from materials age to.

	項目	記号	計算式	単位	Super Girder						Deck Slab			
					t1	t2	t3	t4	t5	t6	ts1	ts2	ts3	ts4
					φg1	φg2	φg3	φg4	φg5	φg6	φd1	φd2	φd3	φd4
1					φ(55 ,to)	φ(85 ,to)	φ(110 ,to)	φ(160 ,to)	φ(285 ,to)	φ(11000 ,to)	φ(50 ,to)	φ(100 ,to)	φ(225 ,to)	φ(10915 ,to)
2	Type of the cement	Tcp		-	1	1	1	1	1	1	1	1	1	1
3	Copmressive stress at age of 28 days	fck	—	Mpa	50	50	50	50	50	50	30	30	30	30
4	Concrete age at uniaaiallly loaded	to'	—	day	3	3	3	3	3	3	3	3	3	3
5	At time of comcidering the creep strain	t	—	day	55	85	110	160	285	11,000	50	100	225	10,915
6	Average temperture before loading	Ts（⊿ti）		℃	25	25	25	25	25	25	25	20	25	25
7	Average temperture	T（⊿ti）		℃	25	25	25	25	25	25	25	20	25	25
8	Relative humidity	RH		%	80	80	80	80	80	80	80	80	80	80
9	The notitional size of member	h	h=2Ac/u	mm	125	125	125	125	125	125	320	320	320	320
10	Coefficient for cement type	α		—	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
11	Basic concrete strength for elasitic modulus	fcmo		Mpa	10	10	10	10	10	10	10	10	10	10
12	Modifinasional strength for concrte strength	⊿f		Mpa	8	8	8	8	8	8	8	8	8	8
13	Cofficient for type of cement	α		-	0	0	0	0	0	0	0	0	0	0
14	Cofficient for type of cement	s		-	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25
15	Mean value of compresive strength at the age of 28 days	fcm	fcm=fck+Δf	Mpa	58	58	58	58	58	58	38	38	38	38
16	Elastic modulus at age of 28days	Ec（28）		Mpa	38,629	38,629	38,629	38,629	38,629	38,629	33,551	33,551	33,551	33,551
17	The age of concrte at loading	$t_{0,T}$		day	3.772	3.772	3.772	3.8	3.8	3.8	3.8	3.0	3.772	3.772
18	The effective age at loading	to		day	3.772	3.772	3.772	3.8	3.8	3.8	3.8	3.0000	3.772	3.772
19	The effective age at caluculating creep cofficient	t		day	69.2	107	138.32	201	359	13,845	63	100	282.918	13,724.689
20	Collectiong cofficient for elastic modulus	βcc（to）		-	0.6498	0.6498	0.6498	0.6498	0.6498	0.6498	0.6498	0.5982	0.6498	0.6498
21	A correction factor to face each other on materials age 28th for arbitrary materials age elasticity coefficient	β_E(to)		-	0.8061	0.8061	0.8061	0.8061	0.8061	0.8061	0.8061	0.7735	0.8061	0.8061
22	Concrte strenth at time t0	fcm(to)		Mpa	37.7	37.7	37.7	37.7	37.7	37.7	24.7	22.7	24.69	24.69
23	Elastic moduluws at time t0	Ec(to)		Mpa	31,139	31,139	31,139	31,139	31,139	31,139	27,045	25,950	27,045	27,045
24	Basic size of the menber	her		mm	100	100	100	100	100	100	100	100	100	100
25	Cofficient for humdity and size of member	ψ_{RH}		—	1.4036	1.4036	1.4036	1.4036	1.4036	1.4036	1.2950	1.2950	1.2950	1.2950
26	Collectiong cofficient for mean strength	β（fcm）		—	2.201	2.201	2.201	2.201	2.201	2.201	2.719	2.719	2.719	2.719
27	Collecting coffient for the concrte age	$\beta(t_o)$		-	0.712	0.712	0.712	0.712	0.712	0.712	0.712	0.743	0.712	0.712
28	Collecting cofficient for the size of menber	β_H		mm	527.4	527.4	527.4	527.4	527.4	527.4	960.2	960.2	960.2	960.2
29	Cofficient for humdity and size of member	φRH		-	1.404	1.404	1.404	1.404	1.404	1.404	1.295	1.295	1.295	1.295
30	Basic creep coefficient	ψ0		-	2.200	2.200	2.200	2.200	2.200	2.200	2.508	2.616	2.508	2.508
31	A creep progress degree to conrete age to - t	βc（t-to）		-	0.516	0.581	0.620	0.677	0.761	0.989	0.426	0.488	0.639	0.980
32	Ceep Coefficent	$\psi_{CEB}(t,t_0)$	MC90	-	1.136	1.278	1.364	1.489	1.674	2.175	1.067	1.278	1.603	2.457
33	Collecting coefficient for elastic modulus	β_E(to)		-	0.806	0.806	0.806	0.806	0.806	0.806	0.806	0.773	0.806	0.806
34	Creep coefficient for EC92	$\psi(t,t_0)$	EC2	-	0.916	1.030	1.100	1.201	1.349	1.754	1.324	0.988	1.293	1.981
35	Ceep Coefficent	$\psi_{CEB}(t,t1)$	φCEB(t2,t0)-φCEB(t1,to)		-0.11						0.34			

Coefficient of Shrinkage

Assumtion of the concrte age at each construction stage

	Super T girder			Deck Slab				
At presstressing tranfer	t0=	3	days	ts0=	3	days		Non Composite girder
At finsh election of the girder	t1=	55	days					Non Composite girder
At casting concrete of the cross beams	t2=	85	days				bearing	Non Composite girder
At casting concrete of the deck slab except the link slab	t3=	110	days	ts1=	55	days	Composite girder	Composite girder
At castiing concrete of link slab	t4=	160	days	ts2=	100	days	Secondary intermidiate force	Composite girder
At setting the expansion joints	t5=	285	days	ts3=	225	days	Expansion joint	Composite girder
Concrete age at end of creep and shrinkage	t6=	11000	days	ts4=	10915	days		Composite girder

For Space of from girder to girder
Untile composite beam

Suprt T Girder($x10^{-6}$)				Deck Slab($x10^{-6}$)					
Item		Unit	Value	Item		Unit	Value		
Acg	Area of Girder	m2	0.6377	Acs	Area of Deck Slab	m2	0.3384		
εsg1	φ(55 ,to)	-	-79.1						
εsg2	φ(85 ,to)	-	-96.4						
εsg3	φ(110 ,to)	-	-107.6	εsd1	φ(55 ,to)	-	-42.5		
εsg4	φ(160 ,to)	-	-124.7	εsd2	φ(100 ,to)	-	-51.6		
εsg5	φ(285 ,to)	-	-151.8	εsd3	φ(225 ,to)	-	-140.7		
εsg6	φ(11000 ,to)	-	-237.4	εsd4	φ(10915 ,to)	-	-237.4		
Suprt T Girder εg(t2,t1) ($x10^{-6}$)				Deck Slab εd(t2,t1) ($x10^{-6}$)				Compsite εc(t2,t1)($x10^{-6}$)	
shg(3 ,055)	-79	0	-79					shg(3 ,055)	-79
shg(55 ,085)	-96	-79	-17					shg(55 ,085)	-17
shg(85 ,110)	-108	-96	-11	shd(3 ,110)	-43	0	-43	shc(85 ,110)	-22
shg(110 ,160)	-125	-108	-17	shd(110 ,180)	-52	-43	-9	shc(110 ,160)	-14
shg(160 ,285)	-152	-125	-27	shd(180 ,200)	-141	-52	-89	shc(160 ,285)	-49
shg(28 ,51100)	-237	-152	-86	shd(200 ,10820)	-237	-141	-97	shc(28 ,51100)	-89
			-237				-237		-271

Shrinkage MC90 εsc(t,ts) A dry shrinkage degree to go by arbitrary materials age t from materials age ts at good self-care end time

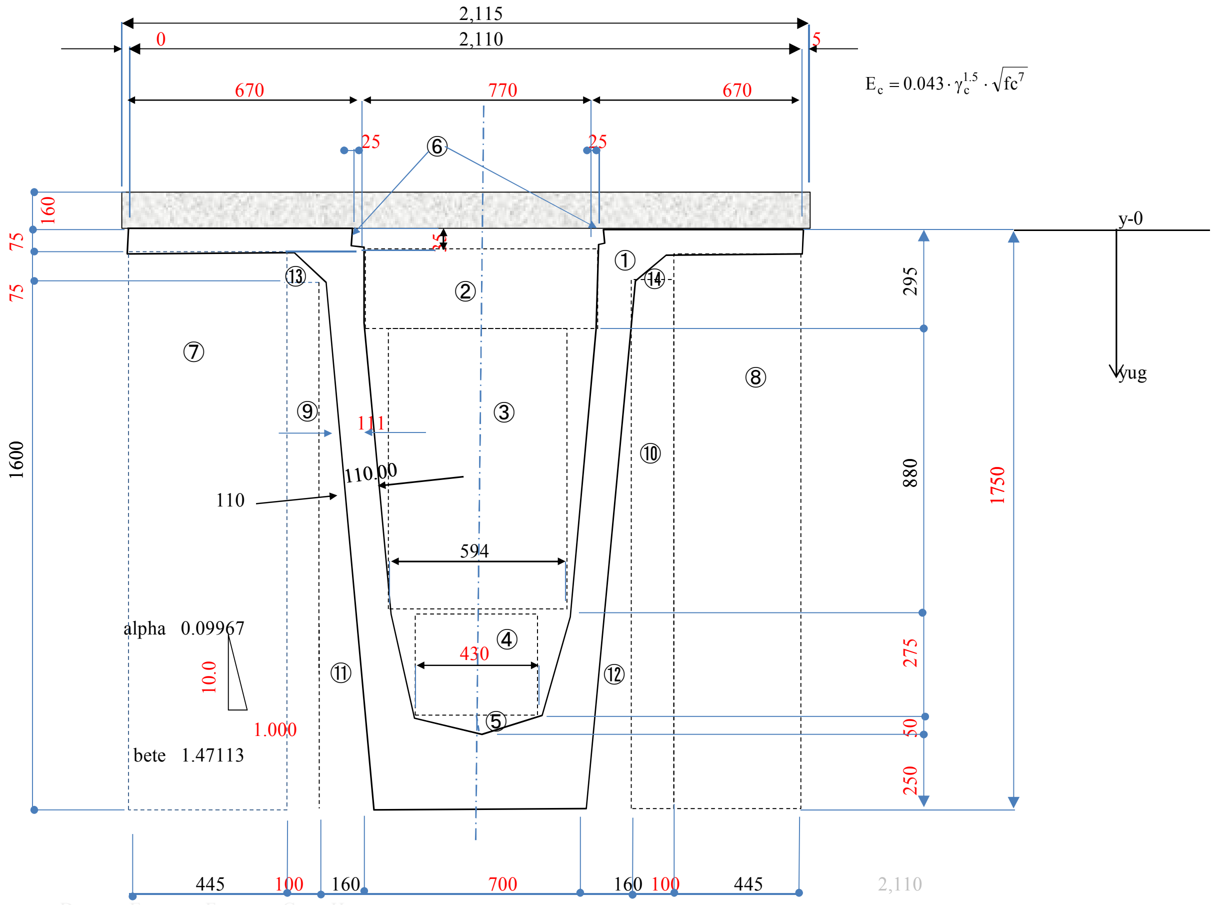
Item	Symbol	Equation	単位	Super Girder						Deck			
				t1	t2	t3	t4	t5	t6	ts1	ts2	ts3	ts4
				εsg1	εsg2	εsg3	εsg4	εsg5	εsg6	εsd1	εsd2	εsd3	εsd4
				φ(55 ,to)	φ(85 ,to)	φ(110 ,to)	φ(160 ,to)	φ(285 ,to)	φ(11000 ,to)	φ(55 ,to)	φ(100 ,to)	φ(225 ,to)	φ(10915 ,to)
Type of cement		Normal or Rapid Harding Cemen	-	1	1	1	1	1	1	1	1	1	1
Copmressional strength at age of 28 days	fck	—	Mpa	50	50	50	50	50	50	30	30	50	50
At concrete age at finishing cure	ts'	—	day	3	3	3	3	3	3	3	3	3	3
An arbitrary concretes age	t'	—	day	55	85	110	160	285	11,000	55	100	225	10915
The average temperature until loading time	T0（⊿ti)		℃	25	25	25	25	25	25	25	20	25	25
Averge temperture	T（⊿ti)		℃	25	25	25	25	25	25	25	20	25	25
Relative humidity	RH		%	80	80	80	80	80	80	80	80	80	80
Effective size of the menber	h0	h=2Ac/u	mm	125	125	125	125	125	125	320	320	125	125
Coeffient foy type of cement	βsc		—	5	5	5	5	5	5	5	5	5	5
Basic strength for elastic modulus	fcmo		Mpa	10	10	10	10	10	10	10	10	10	10
Collecting strength for conretes strength of the age of 28 day	⊿f		mm	8	8	8	8	8	8	8	8	8	8
The concrte age of funushing cure	ts		day	4	4	4	4	4	4	4	3	4	4
The effective age for caluculation of shrinkage	t		day	69	107	138	201	359	13,845	69	100	283	13,738
Averege strength ofconcrete st sge of 28 days	fcm			58	58	58	58	58	58	38	38	58	58
Collectin gcoefficient of humidity	β_{RH}	40%≤RH<99% βRH=-1.55βsRH RH≥99% βRH=+0.25	-	-0.7564	-0.7564	-0.7564	-0.7564	-0.7564	-0.7564	-0.7564	-0.7564	-0.7564	-0.7564
Collection cofficient of age of concrte	βs(t-ts)			0.327	0.398	0.444	0.515	0.627	0.981	0.134	0.162	0.582	0.981
Coefficient of compresive strength of concrte	εs(fcm)		$x10^{-6}$	320	320	320	320	320	320	420	420	320	320
Basic coeffient of shrinkage	ε_{cso}		$x10^{-6}$	-242	-242	-242	-242	-242	-242	-318	-318	-242	-242
A shrinkage degree from ts to t	εcs(t、ts)		$x10^{-6}$	-79	-96	-108	-125	-152	-237	-43	-52	-141	-237
A shrinkage degee (from t1 to t2)	εcu(t2,t1)	εCEB(t2,t0)-εCEB(t1,to)	$x10^{-6}$										

Appendix 3 **Effective Thickness of the Menber**

Exteria Gerder

Shape of the Gerder fcg= 50 fcs= 35 γc= 2400 kg/m3

Eg= 35,700Mpa Es= 29,900Mpa ns= 0.838



External Gerder

Number NO.0

Precast Gerder 0 Xs= = 0.0000 m X = 0.0000 m

Gerdr Hight Hg= 1.750 m Bft= 2.11 m Web Thikness Bw= 0.000 m

Conversion Coefficent n= 0

														from top	from Bot	higt
No.	Shape	bu or R	bl	h	yu or yl	nos.	yug	A0	n	A	A・y	A・y^2	Io	y1	y2	h
1	1	2.1100	2.1100	1.7500	0.0000	1	0.87500	3.6925	1.000	3.6925	3.23094	2.82707	0.9423568	0.87500	0.87500	1.75000
2	1	0.7700	0.7700	0.2945	0.0000	-1	0.14726	-0.2268	1.000	-0.2268	-0.03339	-0.00492	-0.0016392	0.14726	0.14726	0.29451
3	1	0.7700	0.5939	0.8805	0.2945	-1	0.71581	-0.6004	1.000	-0.6004	-0.42981	-0.30766	-0.0385763	0.42130	0.45919	0.88049
4	1	0.5939	0.4300	0.2750	-0.2750	-1	1.33016	-0.1408	1.000	-0.1408	-0.18727	-0.24910	-0.0008797	0.13016	0.14484	0.27500
5	1	0.4300	0.0000	0.0500	-0.2250	-1	1.49167	-0.0108	1.000	-0.0108	-0.01604	-0.02392	-0.0000015	0.01667	0.03333	0.05000
6	1	0.0250	0.0250	0.0350	0.0000	-2	0.01750	-0.0018	1.000	-0.0018	-0.00003	0.00000	-0.0000002	0.01750	0.01750	0.03500
7	1	0.4450	0.4450	1.6750	0.0750	-1	0.91250	-0.7454	1.000	-0.7454	-0.68015	-0.62064	-0.1742702	0.83750	0.83750	1.67500
8	1	0.4450	0.4450	1.6750	0.0750	-1	0.91250	-0.7454	1.000	-0.7454	-0.68015	-0.62064	-0.1742702	0.83750	0.83750	1.67500
9	1	0.1000	0.1000	1.6000	0.1500	-1	0.95000	-0.1600	1.000	-0.1600	-0.15200	-0.14440	-0.0341333	0.80000	0.80000	1.60000
10	1	0.1000	0.1000	1.6000	0.1500	-1	0.95000	-0.1600	1.000	-0.1600	-0.15200	-0.14440	-0.0341333	0.80000	0.80000	1.60000
11	1	0.0000	0.1600	1.6000	0.1500	-1	1.21667	-0.1280	1.000	-0.1280	-0.15573	-0.18948	-0.0182044	0.53333	1.06667	1.60000
12	1	0.0000	0.1600	1.6000	0.1500	-1	1.21667	-0.1280	1.000	-0.1280	-0.15573	-0.18948	-0.0182044	0.53333	1.06667	1.60000
13	1	0.0000	0.1000	0.0750	0.0750	-1	0.12500	-0.0038	1.000	-0.0038	-0.00047	-0.00006	-0.0000012	0.02500	0.05000	0.07500
14	1	0.0000	0.1000	0.0750	0.0750	-1	0.12500	-0.0038	1.000	-0.0038	-0.00047	-0.00006	-0.0000012	0.02500	0.05000	0.07500
Sum1								0.6377		0.6377	0.5877	0.3323	0.4480			

A0 = 0.6377 m^2

A = 0.6377 m^2

yu= A・y/A = 0.5877 / 0.6377 = 0.92152 m

yl= h-yu = 1.7500 - 0.9215 = 0.82848 m

I= A・y2+Io-A・yu^2 = 0.3323 + 0.44804 - 0.6377・ 0.92152^2 = 0.23880 m^4

Wt= I / yu = 0.23880 / 0.9215 = 0.25914 m^3

Wb= I/yl = 0.23880 / 0.8285 = 0.28824 m^3

Only Desc Slab

Gider Hight Hg= 0.160 m Width B= 2.115 m

Reducet coeffient n 1.000

No.	Shape	bu or R	bl	h	yu or yl	nos.	yug	A0	n	A	A・y	A・y^2	Io	from top y1	from Bot y2	higt h
1	1	2.1150	2.1150	0.1600	0.0000	1	0.08000	0.3384	1.000	0.3384	0.02707	0.00217	0.0007219	0.08000	0.08000	0.16000
0	0	0.0000	0.0000	0.0000	0.0000	0	0.00000	0.0000	1.000	0.0000	0.00000	0.00000	0.0000000	0.00000	0.00000	0.00000
Sum1								0.3384		0.3384	0.0271	0.0022	0.0007			

A0 = 0.3384 m^2

A = 0.3384 m^2

yu= A・y/A = 0.0271 / 0.3384 = 0.08000 m

yl= h-yu = 0.1600 - 0.0800 = 0.08000 m

I= A・y2+Io-A・yu^2 = 0.0022 + 0.00072 - 0.3384・ 0.08000^2 = 0.00072 m^4

Wt= I / yu = 0.00072 / 0.0800 = 0.00902 m^3

Wb= I/yl = 0.00072 / 0.0800 = 0.00902 m^3

Reduce Deckslab to Main Girder

Reduced Coeffiectn=Eg/Es= 0.838 Thikness of Deck Slab I 1.910 m

	Shape	bu or R	bl	h	yu	nos.	yug	A0	n	A	A・y	A・y2	nIo	Io
1	1	2.115	2.115	0.160	-0.1600	1	-0.0800	0.33840	0.8375	0.28342	-0.02267	0.00181	0.00060	0.0007219
2	0	0	0.000	0.000	0.0000	1	0.0000	0.00000	0.8375	0.00000	0.00000	0.00000	0.00000	0
G								0.63774		0.63774	0.58769	0.33232	0.44804	
Sum2								0.97614		0.92116	0.56501	0.33414	0.44865	

A0 = 0.97614 m^2

A = 0.92116 m^2

ygu= A・y/A = 0.5650 / 0.9212 = 0.61337 m

ysu= Hgc-yl= 1.9100 - 1.1366 = 0.77337 m 0.77337

yl= hg-yu = 1.7500 - 0.6134 = 1.13663 m

I= A・y^2+Io-A・yu^2 = 0.3341 + 0.44865 - 0.9212・ 0.61337^2 = 0.43622 m^4

Wt= I / yu = 0.43622 / 0.6134 = 0.71119 m^3

Wb= I/yl = 0.43622 / 1.1366 = 0.38378 m^3

Torsinal Inatia

Deduced Coefficient n=Eg/Es= 0.838

Girder Hight hg = 1.750

Width bu = 1.082 m bl= 0.700 m

Thikness tu = 0.160 m tl= 0.250 m

tw = 0.1100 m two= 0.111 m

Total Hifht hgs= 1.910 m tuo= 0.134006 m

θ=atan(1600.0 /160)= 1.47113 rad 84.28941 deg

Ht=hgs-(tu+tl)/2= 1.718

Btl=bl+tl/tan(θ)-2two= 0.5039 m

Btu=Btl+2*Ht/tanθ= 0.8475 m

Hto=Ht/sinθ= 1.7266 m

Am=Ht*(Btu+Btl)/2= 1.1609 m

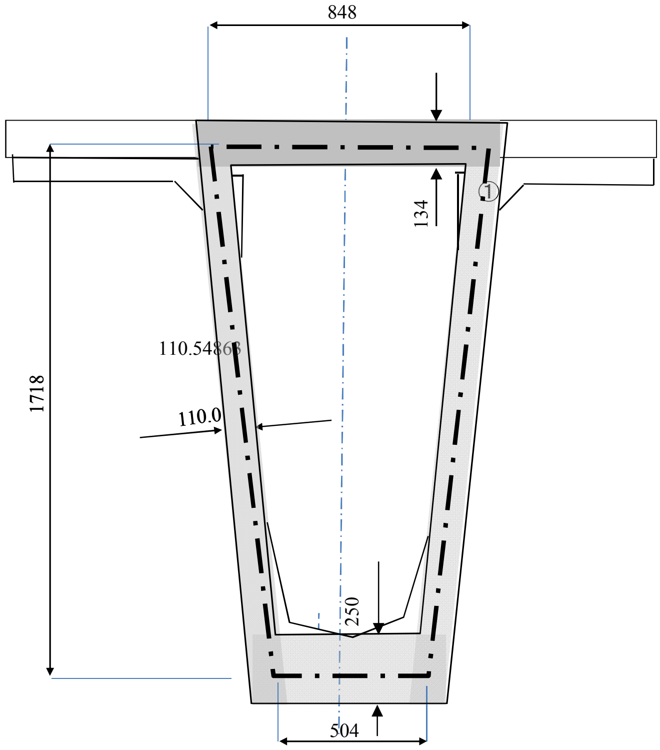
S1=Btu/tuo= 6.3244 m

S2=Btl/rl= 2.0156 m

S3=2*Hto/tw= 31.3921 m

S= 39.7321 m

Ix=4*Am^2/S= 0.13567 m4



Effective Thickness for Creep and Shrinkage

Only Gerder ho=2Ac/u Ac = 0.6377 m2

perimeter for outside of the Gerder

NO.	dx	dy	n	li	l
1	0.000	0.075	2	0.0750	0.1500
2	0.445	0.000	2	0.4450	0.8900
3	0.100	0.075	2	0.1250	0.2500
4	0.160	1.600	2	1.6080	3.2160
5	0.670	0.000	2	0.6700	1.3400
6	0.000	0.295	2	0.2945	0.5890
7	0.088	0.880	2	0.8849	1.7698
8	0.082	0.275	2	0.2870	0.5739
9	0.215	0.050	2	0.2207	0.4415
10	0.700	0.000	1	0.7000	0.7000
				Sum Lo	9.9201

ho = 2 * 0.6377 / 9.9201 0.129 m
= 129 mm

AASHTO V/S= 0.6377 / 9.9201 0.0643
= 64 mm

Deck Slab ho=2Ac/u Ac = 0.3384 m2

Perimeter for outside of the Gerder

NO.	dx	dy	n	li	l
1	2.115	0.000	1	2.1150	2.1150
2	0.000	0.000	0	0.0000	0.0000
				Sum Lo	2.1150

Perimeter for intside of the Gerder

NO.	dx	dy	n	li	L
1	0.000	0.000	0	0.0000	0.0000
2	0.000	0.000	0	0.0000	0.0000
				Sum Li	0.0000

L= 2.115 + 0.0000 / 2 = 2.1150 m
ho = 2 * 0.3384 / 2.1150 0.320 m
= 320 mm

AASHTO V/S= 0.3384 / 2.1150 0.1600
= 160 mm

Composite Member

ho=2Ac/u Ac = 0.9761 m2

Perimeter for outside of the Gerder

NO.	dx	dy	n	li	l
1	0.000	0.075	2	0.0750	0.1500
2	0.445	0.000	2	0.4450	0.8900
3	0.100	0.075	2	0.1250	0.2500
4	0.160	1.600	2	1.6080	3.2160
5	0.700	0.000	1	0.7000	0.7000
6	2.115	0.000	1	2.1150	2.1150
7	0.000	0.000	0	0.0001	0.0000
8	0.000	0.000	0	0.0000	0.0000
				Sum Lo	7.3210

Perimeter for intside of the Gerder

NO.	dx	dy	n	li	L
1	0.770	0.000	2	0.7700	1.5400
2	0.000	0.260	2	0.2595	0.5190
3	0.088	0.880	2	0.8849	1.7698
4	0.082	0.275	2	0.2870	0.5739
5	0.215	0.050	2	0.2207	0.4415
	0.000	0.000	0	0.0000	0.0000
				Sum Li	3.8288

L= 7.321 + 3.8288 / 2 = 9.2354 m
ho = 2 * 0.9761 / 9.2354 0.211 m
= 211 mm

AASHTO V/S= 0.9761 / 9.2354 0.1057
= 106 mm

Appendix 4 **Condition of Substructure and Fundation**

Stifness of Substructure and Foundation Direction of Logitudinal

Item			Symbol	Unit	S1	S2	S3	S4	S5	S6	S7	S8	S9	S10
					A1	P1		P2		P3		P4		P5
Distance from Pier's Top to Position of Horizontal Force of Superstructure			h0	m	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
Hight of Pier			hp	m	8.500	8.500	8.500	8.500	8.500	8.500	8.500	10.500	10.500	10.500
Thikness of Pile Cap			tf	m	2.000	2.000	2.000	2.000	2.000	2.000	2.000	2.000	2.000	2.000
Distance from Bottom of Pile cap to Position of Horizontal Force of Superstructure	Hight of Pier Shaft		hp	m	8.500	8.500	8.500	8.500	8.500	8.500	8.500	8.500	8.500	8.500
	Thickness of Pier Cap		hh	m	0.000	0.800	0.800	0.800	0.800	0.800	0.800	0.800	0.800	0.800
	Toatal		h	m	8.500	9.300	9.300	9.300	9.300	9.300	9.300	9.300	9.300	9.300
Elastic Modulus of Concrete			Ec	Mpa	2.77E+04	2.77E+04	2.77E+04	2.77E+04	2.77E+04	2.77E+04	2.77E+04	2.77E+04	2.77E+04	2.77E+04
Self wight of Pier			Wp	kN	0	1375	1495	1495	1495	1495	1495	1495	1495	1495
Cofficient of center gravity of pier			cp	-	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
Spring of Fpoundation	Considering or Not Considering of Spring of Foundation		Iss	-	0	0	0	0	0	0	0	0	0	0
			-	-	Not Consider	Not Consider	Not Consider	Not Consider	Not Consider	Not Consider	Not Consider	Not Consider	Not Consider	Not Consider
	Horizontal Spring	K11	Kh	kN/m	-	-	-	-	-	-	-	-	-	-
	Vertical Spring	K22	Kv	kN/m	-	-	-	-	-	-	-	-	-	-
	Rotation Spring	K33	Kθ	kN・m/rad	-	-	-	-	-	-	-	-	-	-
	Coupled Spring	K13	Khθ	kN・m/rad	-	-	-	-	-	-	-	-	-	-
	Center of Rotation	K13/K11	h	m	-	-	-	-	-	-	-	-	-	-
Stifness of Pire	Inertia	—	Ic	m4	8.000	1.917	1.917	1.917	1.917	1.917	1.917	1.917	1.917	1.917
	Stifness	E・Ic	Ei_e	kN・m^2	2.216E+08	5.311E+07	5.311E+07	5.311E+07	5.311E+07	5.311E+07	5.311E+07	5.311E+07	5.311E+07	5.311E+07
	Spring Pire	Kep=Eic/hp^3	Kep	kN/m	1.083E+06	2.595E+05	2.595E+05	2.595E+05	2.595E+05	2.595E+05	2.595E+05	1.376E+05	1.376E+05	1.376E+05
Displace ment at Superstructure	Displacement of Foundation — flexibility matrix	K33/(K11・K33-K13^2)	K_{11}'	m/kN	-	-	-	-	-	-	-	-	-	-
		K11/(K11・K33-K13^2)	K33'	rad/kN・m	-	-	-	-	-	-	-	-	-	-
		-K13/(K11・K33-K13^2)	K13'	rad/kN	-	-	-	-	-	-	-	-	-	-
	Displacement of Foundation — Horizontal	K11'+K13'*(h+ho)	δ0	m	-	-	-	-	-	-	-	-	-	-
	Displacement of Foundation — Rotation	K13'+K33'・(h+ho)	θ0	rad	-	-	-	-	-	-	-	-	-	-
	Displacement of Foundation — Total Horizontal	δ0+θ0・(h+ho)	δf	m	-	-	-	-	-	-	-	-	-	-
	Displacement of Foundation — Spring	1/δf	Kk	kN/m	-	-	-	-	-	-	-	-	-	-
	Displacement of Pier Shaft	h^3/3EI+h^2/2EI・h0	δpe	m	9.238E-07	3.854E-06	3.854E-06	3.854E-06	3.854E-06	3.854E-06	3.854E-06	7.265E-06	7.265E-06	7.265E-06
	Total displacement	δf+δpe	δke	m	9.238E-07	3.854E-06	3.854E-06	3.854E-06	3.854E-06	3.854E-06	3.854E-06	7.265E-06	7.265E-06	7.265E-06
Spring of Substructure and Foundation		1/δke	Kpe	kN/m	1.083E+06	2.595E+05	2.595E+05	2.595E+05	2.595E+05	2.595E+05	2.595E+05	1.376E+05	1.376E+05	1.376E+05

1.3 Parapet design
1.3 Kiểm toán lan can bêtông

CALCULATION PROCEDURE & STANDARD:

- Bridge Design Standard 22 TCN - 272 - 05 (considered with AASHTO LRFD 2007)

A. GENERAL DATA:

1. Design live load

Design vehicle load		HL93	
Number of lanes		3.00	(lanes)
Design earthquake class	Class	VII	

2. Bridge width

Width of carriageway	B_{CAR} =	12.00	(m)
Width of barrier wall	B_{lc} =	0.50	(m)
Bridge width	B =	13.00	(m)

3. Material properties:

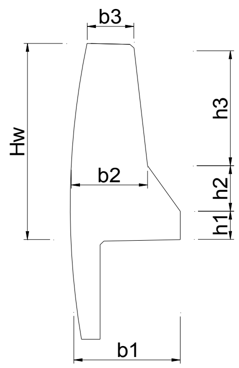
Concrete

Compressive strength of cylindrical at 28 days age	f'c =	25.00	MPa
Concrete density	g =	24.50	KN/m3
Elastic modulus	E_c =	26875.00	MPa
Tensile strength of concrete	f_r =	3.15	MPa

Steel CB-400-V

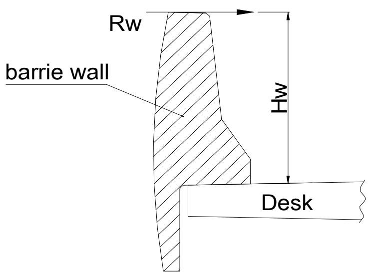
Modulus of elasticity	E_s =	200000	MPa
Yield strength of steel bar	f_y =	400	MPa

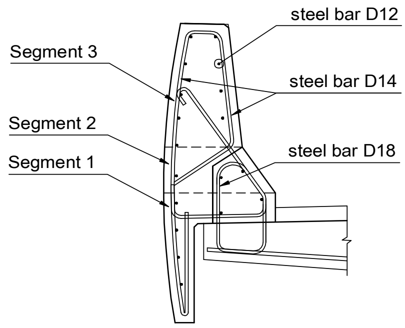
4. Dimensions of RC barrier wall:



b1	590	(mm)
b2	409	(mm)
b3	250	(mm)
h1	159	(mm)
h2	255	(mm)
h3	740	(mm)

5. Diagram of Calculation





6. Railing shall be proportioned such that:

$$\boxed{R \geq Ft} \qquad (13.7.3.3\text{-}1)$$

In which:

R - Total resistance of the barrier wall

Ft - Transverse vehicle impact force

7. General value:

- Diameter of longitual steel bar 12 (mm)
- Diameter of stirrup 18 (mm)
- Reinf. Spacing of stirrup 150 (mm)
- Φ Bending resistance factor 1

7.1 Choose Design force for barrier wall : (AASHTO2007 Table 13.2-1)

- Barrier wall containment level: L5

Ft 550 (KN)

He(min) 1070

7.2 Total capacity of Barrier wall:

7.2.1. Resistance of concrete wall for vertical axial (Mw.H)

+ Mw for out-face

Segment	Width of Segment b' = h	Number of bars n	Effective Depth d(+)	Area of bars As	$a = \frac{A_s.f_y}{0,85.f'_c.b}$	Φ.Mn(+) $\Phi.A_s.f_y.\left(d - \frac{a}{2}\right)$
	(mm)	(Bar)	(mm)	(mm^2)	(mm)	(KNmm)
1	159	1	186	113	13	8111.58
2	255	1	345	113	8	15418.59
3	740	4	526	452	12	94141.54

+ Mw for Int-face

Segment	Width of Segment b' = h	Number of bars n	Effective Depth d(+)	Area of bars As	$a = \frac{A_s.f_y}{0,85.f'_c.b}$	Φ.Mn(-) $\Phi.A_s.f_y.\left(d - \frac{a}{2}\right)$
	(mm)	(Bar)	(mm)	(mm2)	(mm)	(KNmm)
1	159	1	186	113	13	8111.58
2	255	1	335	113	8	14966.20
3	740	4	516	452	12	92331.98

+ Resistance of concrete wall for vertical axial (Mw.H)

Segment	Width of Segment b' = h	Φ.Mn(+) Out-face	Φ.Mn(-) Int-face	Φ.Mni Average of two face	Mw.H $\sum \Phi.M\,ni$
	(mm)	(KNmm)	(KNmm)	(KNmm)	(KNmm)
1	159	8111.58	8111.58	8111.58	116540.74
2	255	15418.59	14966.20	15192.40	
3	740	94141.54	92331.98	93236.76	

Where:

d - Average distance from compression face to centroid of tension reinforcement (mm)

a - Thickness of the equivalent stress block (mm)

As - Area of tension reinforcement (mm2)

7.2.2. Transverse Ultimate resistance of wall (Mc)

+ Transverse resistance of RC barrier wall (Mc)

Shear contact area: (mm2/mm) $As = \frac{\pi.\Phi^2}{4 * D}$ (with D is Reinf. Spacing of shear)

and b =1 m

Segment	Hight of Segment h	Shear contact area As	Effective Depth d	$a = \frac{A_s.f_y}{0,85.f'_c.b}$	Mci $\Phi.A_s.f_y.\left(d - \frac{a}{2}\right)$	Mc $\frac{\sum \Phi.M.hi}{\sum hi}$
	(mm)	(mm2/mm)	(mm)	(mm)	(KNmm)	(KNmm)
1	159	1.696	311	31.93	200.205	174.639
2	255	1.696	140.5	31.93	84.506	
3	740	1.696	311	31.93	200.205	

+ Total ultimate resistance of RC barrier wall:

- For impacts within a wall segment :

$$R_w = \left(\frac{2}{2.L_c - L_t}\right)\left(8.M_b + 8.M_w.H + \frac{M_c.L_c^2}{H_w}\right)$$

(TCN 13.7.3.4-1)

In which :

- Rw - Total transverse resistance of the RC barrier wall (N)
- Lc - Critical length of yield line failure pattern (mm)
- Lt - Longitudinal length of distribution of impact force Ft (mm)
- Mw - Flexural resistance of a wall (KNmm/mm)
- Mc - Transverse flexural resistance of wall (KNmm/mm)
- Mb - Additional flexural resistance of beam in addition to Mw, if any, at top of wall (KNmm/mm)
- Hw - Height of barrier wall Hw (mm)

- Critical length of yield line failure pattern Lc :

$$L_c = \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + \frac{8.H_w.(M_b + M_w.H)}{M_c}}$$

(TCN 13.7.3.4-2)

Lt (mm)	Hw (mm)	Mb (KN.mm)	Mc (KN.mm/mm)	Mw.H (KN.mm)	Lc (mm)	Rw (KN)
2440	1154	0	174.64	116540.74	3986	1206.34

- For impacts at end of wall or at joint :

$$R_w = \left(\frac{2}{2.L_c - L}\right)\left(M_b + M_w.H + \frac{M_c.L_c^2}{H_w}\right)$$

(TCN 13.7.3.4-1)

- Critical length of yield line failure pattern Lc :

$$L_c = \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + \frac{H_w.(M_b + M_w.H)}{M_c}}$$

Lt (mm)	Hw (mm)	Mb (KN.mm)	Mc (KN.mm/mm)	Mw.H (KN.mm)	Lc (mm)	Rw (KN)
2440	1154	0	174.64	116540.74	2723	824.11

8. RESISTANCE CHECK FOR RC BARRIER WALL

- Condition 1 R = R w≥ Ft With : Ft = 550 (KN)

+ Resistance Check for RC barrier wall accordant Condition 1

Combination	Resistance barrier Wall Rw (KN)	Ft (KN)	Check Condition (1)
1. Impact at end of wall or joint	824.11	550	**OK**
2. Impact at a wall segment	1206.34	550	**OK**

9. SEFT WEIGHT OF RC BARRIER WALL (DC_{lc})

+ Seft weight of concrete	γ_c	24.5	(KN/m^3)
+ Seft weight of steel	γ_s	7.85	(KN/m^3)
+ seft weight of Asphalt concrete	γ_a	23	(KN/m^3)

- Seft weight of concrete wall

+ Area of concrete wall Ac = 0.529 (m2)

+ Load due to weight of wall $DC_c = \gamma_c.A_c$

DC_{lc} = 12.96 (KN/m)

10. CHECK SHEAR-RESISTANCE OF RC AT BASE OF THE WALL JOINT WITH DECK

- Arrangement of stirrup D18 attach in overhang

Assuming that Rw spreads out at a 1:1 slope from Lc

- The tensile force per unit of length in the overhang, is given by:

$$T = \frac{R_{max}}{L_c + 2.H_w}$$

- Height of barrier	Hw =	1154	(mm)
- Maximum of load impact on barrier wall	Rmax =	1206.34	(KN)
	Lc =	3986	(mm)
	T_1 =	191.67	(N/mm)
- For Impact at end of barrier wall	Rmax =	824.11	(KN)
	Lc =	2723	(mm)
	T_2 =	163.81	(N/mm)
- Shear load for calculate	T =	$Max(T_1,T_2)$	
	T =	191.67	(N/mm)

- The nominal shear resistance Vn of the interface plane following:

$$V_n = c.A_{cv} + \mu.(A_{vf}.f_y + P_c)$$

Which shall not exceed $0.2f'_c$ or $5.5A_{cv}$

Where:

- Shear contact area:	Acv =	b_1.1 mm		
	Acv =	590	(mm2/mm)	
- Dowel area across shear plane:	$A_{vf} = \frac{\pi.\Phi^2}{4*D}$			(Determined in 9.2.2)
	Avf =	1.696	(mm2/mm)	
- Yield strenght of reinforcement	fy =	400	(MPa)	
- Permanent compressive force:	Pc =	DC_{lc}.1 mm		
	Pc =	12.96	(N/mm)	
- Strength of weaker concrete	f'c =	25	(MPa)	
- Cohesion factor	c =	0.52		[5.8.4.2 - 22TCN 272-05]
- Friction factor	μ =	0.60		[A5.8.4.2 - 22TCN 272-05]
	Vn =	721.73	(N/mm)	
	$0.2f'_c.A_{cv}$ =	2950	(N/mm)	
	$5.5A_{cv}$ =	3245	(N/mm)	

- Nominal shear resistance: Vn = Min(Vn , 0.2f'c.Acv , 5.5Acv)

Vn = 721.73 (N/mm) **> T = 191.67 (N/mm) : OK**

+ The minimum cross-sectional area of dowels across the shear plane:

$$A'_{vf} = 0.35\frac{b_1.s}{f_y}$$ *[5.8.4.1 - 22TCN 272-05]*

A'_{vf} = 77.44 (mm2)

- Number of stirrup input deck n = 1
- Cross-sectional area of stirrup input deck

As = n.Avf.s

As = 254.47 (mm2) **> A'vf : OK**

- The development length l_n shall not less than 3 values then:

$$\frac{100.\Phi}{\sqrt{30}} =$$ 329 (mm) With Φ =18 (mm)

8.Φ = 144 (mm)

And 150 (mm)

- The development length: l_n = 329 (mm) ***(The required modify)***
- Modification factor for adequate cover: k_1 = 0.7

$l'_n = k_1.l_n$

	l'_n =	230	(mm)
- The development length after modify:	l_n =	230	(mm)
- The Available development length:	l_c = hf - as(+)		
	l_c =	191	***mm (which is not adequate)***
- Unless the required area is reduced to	A_{vf} (hc) = $A_{vf}.l_c / l_n$		
	A_{vf} (hc) =	1.409	(mm2)

By using this area to recalculate Mc, Lc, Rw (The determined following 5.2.2)

Segment	Height of Segment h	Shear contact area As	Effective Depth d	$a = \frac{A_s.f_y}{0{,}85.f'_c.b}$	Mci $\Phi.A_s.f_y.\left(d - \frac{a}{2}\right)$	Mc $\frac{\sum \Phi.M.hi}{\sum hi}$
	(mm)	(mm2/mm)	(mm)	(mm)	(KNmm)	(KNmm)
1	159	0.000	311	0.00	0.000	123.412
2	255	1.409	140.5	26.51	71.691	
3	740	1.409	311	26.51	167.752	

- For impacts within a wall segment :

Lt (mm)	Hw (mm)	Mb (KN.mm)	Mc (KN.mm/mm)	Mw.H (KN.mm)	Lc (mm)	Rw (KN)
2440	1154	0	123.41	116540.74	4415	944.25

- For impacts at end of wall or joint :

Lt (mm)	Hw (mm)	Mb (KN.mm)	Mc (KN.mm/mm)	Mw.H (KN.mm)	Lc (mm)	Rw (KN)
2440	1154	0	123.41	116540.74	2826	604.37

+ Check railing following Condition 1

Combination	Resistance of Wall Rw (KN)	R (KN)	Check Condition (1)
1. Impact at end of wall or joint	604.37	550.00	**OK**
2. Impact at a wall segment	944.25	550.00	**OK**

CALCULATION PROCEDURE & STANDARD:

- Bridge Design Standard 22 TCN - 272 - 05 (considered with AASHTO LRFD 2007)

A. GENERAL DATA:

1. Design live load

Design vehicle load		HL93	
Number of lanes		3.00	(lanes)
Design earthquake class	Class	VII	

2. Bridge width

Width of carriageway	B_{CAR} =	12.00	(m)
Width of barrier wall	B_{lc} =	0.50	(m)
Bridge width	B =	13.00	(m)

3. Material properties:

Concrete

Compressive strength of cylindrical at 28 days age	f'c =	25.00	MPa
Concrete density	g =	24.50	KN/m3
Elastic modulus	E_c =	26875.00	MPa
Tensile strength of concrete	f_r =	3.15	MPa

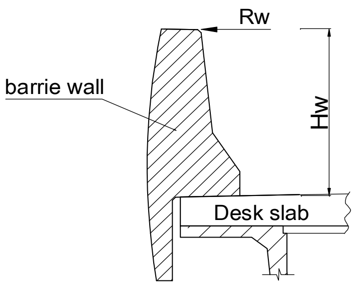
Steel CB-400-V

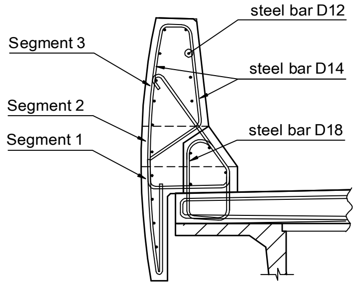
Modulus of elasticity	E_s =	200000	MPa
Yield strength of steel bar	f_y =	400	MPa

4. Dimensions of RC barrier wall:

b1	590	(mm)
b2	409	(mm)
b3	250	(mm)
h1	159	(mm)
h2	255	(mm)
h3	740	(mm)

5. Diagram of Calculation





6. Parapet shall be proportioned such that:

$\boxed{R \geq Ft}$ (13.7.3.3-1)

In which:

R - Total resistance of the barrier wall

Ft - Transverse vehicle impact force

8. General value:

- Diameter of longitual steel bar	12	(mm)
- Diameter of stirrup	18	(mm)
- Reinf. Spacing of stirrup	150	(mm)
- Φ Bending resistance factor	1	

8.1 Choose Design force for barrier wall :

- Barrier wall containment level:

Ft	550	(KN)	(AASHTO2007 Table 13.2-1)
He(min)	1070		

8.2 Total capacity of Barrier wall:

8.2.1. Resistance of concrete wall for vertical axial (Mw.H)

+ Mw for out-face

Segment	Width of Segment b' = h	Number of bars n	Effective Depth d(+)	Area of bars As	$a = \frac{A_s.f_y}{0{,}85.f'_c.b}$	Φ.Mn(+) $\Phi.A_s.f_y.\left(d - \frac{a}{2}\right)$
	(mm)	(Bar)	(mm)	(mm^2)	(mm)	(KNmm)
1	159	1	186	113	13	8111.58
2	255	1	345	113	8	15418.59
3	740	4	526	452	12	94141.54

+ Mw for Int-face

Segment	Width of Segment b' = h	Number of bars n	Effective Depth d(+)	Area of bars As	$a = \frac{A_s.f_y}{0{,}85.f'_c.b}$	Φ.Mn(-) $\Phi.A_s.f_y.\left(d - \frac{a}{2}\right)$
	(mm)	(Bar)	(mm)	(mm2)	(mm)	(KNmm)
1	159	1	186	113	13	8111.58
2	255	1	335	113	8	14966.20
3	740	4	516	452	12	92331.98

+ Resistance of concrete wall for vertical axial (Mw.H)

Segment	Width of Segment b' = h	Φ.Mn(+) Out-face	Φ.Mn(-) Int-face	Φ.Mni Average of two face	Mw.H $\sum \Phi.M\,ni$
	(mm)	(KNmm)	(KNmm)	(KNmm)	(KNmm)
1	159	8111.58	8111.58	8111.58	116540.74
2	255	15418.59	14966.20	15192.40	
3	740	94141.54	92331.98	93236.76	

Where:

d - Average distance from compression face to centroid of tension reinforcement (mm)

a - Thickness of the equivalent stress block (mm)

As - Area of tension reinforcement (mm2)

8.2.2. Transverse Ultimate resistance of wall (Mc)

+ Transverse resistance of RC barrier wall (Mc)

Shear contact area: (mm2/mm)

$$As = \frac{\pi.\Phi^2}{4 * D}$$

(with D is Reinf. Spacing of shear)

and b =1 m

Segment	Hight of Segment h	Shear contact area As	Effective Depth d	$a = \frac{A_s.f_y}{0{,}85.f'_c.b}$	Mci $\Phi.A_s.f_y.\left(d - \frac{a}{2}\right)$	Mc $\frac{\sum \Phi.M.hi}{\sum hi}$
	(mm)	(mm2/mm)	(mm)	(mm)	(KNmm)	(KNmm)
1	159	1.696	311	31.93	200.205	174.639
2	255	1.696	140.5	31.93	84.506	
3	740	1.696	311	31.93	200.205	

+ Total ultimate resistance of RC barrier wall:

<u>- For impacts within a wall segment :</u>

$$R_w = \left(\frac{2}{2.L_c - L_t}\right)\left(8.M_b + 8.M_w.H + \frac{M_c.L_c^2}{H_w}\right)$$

(TCN 13.7.3.4-1)

In which :

Rw	- Total transverse resistance of the RC barrier wall (N)
Lc	- Critical length of yield line failure pattern (mm)
Lt	- Longitudinal length of distribution of impact force Ft (mm)
Mw	- Flexural resistance of a wall (KNmm/mm)
Mc	- Transverse flexural resistance of wall (KNmm/mm)
Mb	- Additional flexural resistance of beam in addition to Mw, if any, at top of wall (KNmm/mm)
Hw	- Height of barrier wall Hw (mm)

- Critical length of yield line failure pattern Lc :

$$L_c = \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + \frac{8.H_w.(M_b + M_w.H)}{M_c}}$$

(TCN 13.7.3.4-2)

Lt (mm)	Hw (mm)	Mb (KN.mm)	Mc (KN.mm/mm)	Mw.H (KN.mm)	Lc (mm)	Rw (KN)
2440	1154	0	174.64	116540.74	3986	1206.34

- For impacts at end of wall or at joint :

$$R_w = \left(\frac{2}{2.L_c - L}\right).\left(M_b + M_w.H + \frac{M_c.L_c^2}{H_w}\right)$$

(TCN 13.7.3.4-1)

- Critical length of yield line failure pattern Lc :

$$L_c = \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + \frac{H_w.(M_b + M_w.H)}{M_c}}$$

Lt (mm)	Hw (mm)	Mb (KN.mm)	Mc (KN.mm/mm)	Mw.H (KN.mm)	Lc (mm)	Rw (KN)
2440	1154	0	174.64	116540.74	2723	824.11

9. RESISTANCE CHECK FOR RC BARRIER WALL

- Condition 1 R = R w≥ Ft With : Ft = 550 (KN)

+ Resistance Check for RC barrier wall accordant Condition 1

Combination	Resistance barrier Wall Rw (KN)	Ft (KN)	Check Condition (1)
1. Impact at end of wall or joint	824.11	550	**OK**
2. Impact at a wall segment	1206.34	550	**OK**

10. SEFT WEIGHT OF RC BARRIER WALL (DC_{lc})

+ Seft weight of concrete	γ_c	24.5	(KN/m^3)
+ Seft weight of steel	γ_s	7.85	(KN/m^3)
+ seft weight of Asphalt concrete	γ_a	23	(KN/m^3)

- Seft weight of concrete wall

+ Area of concrete wall Ac = 0.529 (m2)

+ Load due to weight of wall $DC_c = \gamma_c.A_c$

DC_{lc} = 12.96 (KN/m)

11. CHECK SHEAR-RESISTANCE OF RC AT BASE OF THE WALL JOINT WITH DECK

- Arrangement of stirrup D18 attach in overhang

Assuming that Rw spreads out at a 1:1 slope from Lc

- The tensile force per unit of length in the overhang, is given by:

$$T = \frac{R_{max}}{L_c + 2.H_w}$$

- Height of barrier	Hw =	1154	(mm)
- Maximum of load impact on barrier wall	Rmax =	1206.34	(KN)
	Lc =	3986	(mm)
	T_1 =	191.67	(N/mm)
- For Impact at end of barrier wall	Rmax =	824.11	(KN)
	Lc =	2723	(mm)
	T_2 =	163.81	(N/mm)
- Shear load for calculate	T =	Max(T_1,T_2)	
	T =	191.67	(N/mm)

- The nominal shear resistance Vn of the interface plane following:

$$V_n = c.A_{cv} + \mu.(A_{vf}.f_y + P_c)$$

Which shall not exceed $0.2f'_c$ or $5.5A_{cv}$

Where:

- Shear contact area:	Acv =	b_1.1 mm		
	Acv =	590	(mm2/mm)	
- Dowel area across shear plane:	$A_{vf} = \frac{\pi.\Phi^2}{4 * D}$			(Determined in 9.2.2)
	Avf =	1.696	(mm2/mm)	
- Yield strenght of reinforcement	fy =	400	(MPa)	
- Permanent compressive force:	Pc =	DC_{lc}.1 mm		
	Pc =	12.96	(N/mm)	
- Strength of weaker concrete	f'c =	25	(MPa)	
- Cohesion factor	c =	0.52		[5.8.4.2 - 22TCN 272-05]
- Friction factor	μ =	0.60		[A5.8.4.2 - 22TCN 272-05]
	Vn =	721.73	(N/mm)	
	$0.2f'_c.A_{cv}$ =	2950	(N/mm)	
	$5.5A_{cv}$ =	3245	(N/mm)	

- Nominal shear resistance: Vn = Min(Vn , 0.2f'c.Acv , 5.5Acv)

Vn = 721.73 (N/mm) **> T = 191.67 (N/mm) : OK**

+ The minimum cross-sectional area of dowels across the shear plane:

$$A'_{vf} = 0.35\frac{b_1.s}{f_y}$$ *[5.8.4.1 - 22TCN 272-05]*

A'_{vf} = 77.44 (mm2)

- Number of stirrup input deck n = 1
- Cross-sectional area of stirrup input deck

As = n.Avf.s

As = 254.47 (mm2) **> A'vf : OK**

- The development length l_n shall not less than 3 values then:

$\frac{100.\Phi}{\sqrt{30}}$ =	329	(mm)	With Φ =18 (mm)
8.Φ =	144	(mm)	
And	150	(mm)	

- The development length:	l_n =	329	(mm)	***(The required modify)***
- Modification factor for adequate cover:	k_1 =	0.7		
	l'_n =	$k_1.l_n$		
	l'_n =	230	(mm)	
- The development length after modify:	l_n =	230	(mm)	
- The Available development length:	l_c =	hf - as(+)		
	l_c =	146	***mm (which is not adequate)***	
- Unless the required area is reduced to	A_{vf} (hc) =	$A_{vf}.l_c / l_n$		

A_{vf} (hc) = 1.077 (mm2)

By using this area to recalculate Mc, Lc, Rw (The determined following 5.2.2)

Segment	Height of Segment h	Shear contact area As	Effective Depth d	$a = \frac{A_s.f_y}{0,85.f'_c.b}$	Mci $\Phi.A_s.f_y.\left(d-\frac{a}{2}\right)$	Mc $\frac{\sum \Phi.M.hi}{\sum hi}$
	(mm)	(mm2/mm)	(mm)	(mm)	(KNmm)	(KNmm)
1	159	1.077	311	20.27	129.575	113.349
2	255	1.077	140.5	20.27	56.145	
3	740	1.077	311	20.27	129.575	

- For impacts within a wall segment :

Lt (mm)	Hw (mm)	Mb (KN.mm)	Mc (KN.mm/mm)	Mw.H (KN.mm)	Lc (mm)	Rw (KN)
2440	1154	0	113.35	116540.74	4534	890.62

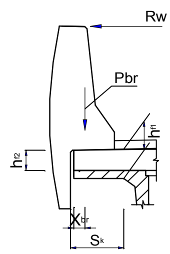
- For impacts at end of wall or joint :

Lt (mm)	Hw (mm)	Mb (KN.mm)	Mc (KN.mm/mm)	Mw.H (KN.mm)	Lc (mm)	Rw (KN)
2440	1154	0	113.35	116540.74	2856	560.95

+ Check railing following Condition 1

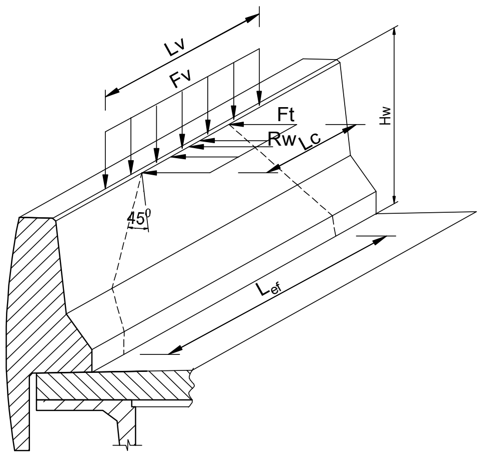
Combination	Resistance of Wall Rw (KN)	R (KN)	Check Condition (1)
1. Impact at end of wall or joint	560.95	550.00	**OK**
2. Impact at a wall segment	890.62	550.00	**OK**

7.2.Ovehang of deck



Over hang length	S_k=	455	(mm)
Thickness of overhang (inside)	h_{f1}=	274	(mm)
Thickness of overhang (outside)	h_{f2}=	265	(mm)
Thickness of wearing surface	h_a=	84	(mm)
Height of parapet	H_w =	1154	(mm)
Distance from centre of barrier to edge of overhang	X_{lc} =	100	(mm)

The top reinforcement must resist the negative bending moment over the exterior beam due to the collision and the deadload of overhang:



$$M_{CT} = \frac{R_w.H_w + R_r.H_r}{L_c + 2.H_w}$$

$$M_{CV} = \frac{F_V}{L_V}.S_k$$

Fv = 355 (KN)

Lv = 12200 (mm)

- Design case 1: The transverse and longitudinal forces on parapet (Rw) - (Extreme state)
- Design case 2: The vertical forces on parapet (Fv) - (Extreme state)
- Design case 3: The loads occupy the overhang - (strength state). This case not consided because overhang is very short, live load not act to overhang part.

- Combination momen due to dead load and collision

$$M_u = [\,1.25M_{DC} + 1.5M_{DW} + 1.0M_{CT}\,]$$

- Moment due to collision forces on parapet act to desk slab:

Combination	Resistance of barrier Rw (KN)	Effect length Lc (mm)	Momen (M_{CT}) (KN.m)
1. Impact at end of wall or joint	560.95	2723	128.67
2. Impact at a wall segment	890.62	3986	163.30

+ The Moment due to transverse collision: M_{CT} = 128.67 (KNmm/mm)
+ The Moment due to vertical forces: M_{CV} = 13.24 (KNmm/mm)

- Moment due to seftweight of desk slab and parapet:

$$M_{DC} = DC_{lc}.(S_k - X_{lc}) + \gamma_c.h_f.S_k^2$$

M_{DC} = 5.28 (KNmm/mm)

- Momen due to wearing surface

$$M_{DW} = \begin{cases} 0 : (S_k - b_1) \le 0 \\ (\gamma_a.h_a + \gamma_p.h_p).(S_k - b_1)^2 \end{cases}$$

M_{DW} = 0.24 (KNmm/mm)

- Combination loading:

Design Case	Unit	M_{DC}	M_{DW}	M_{CT}	M_{CV}	Mu
Design case 1	(Nmm/mm)	5.28	0.24	128.67	0	135645.29
Design case 2	(Nmm/mm)	5.28	0.24	0	13.24	20210.69

Checking for resistance moment

Item		Value	Unit
- The factored moment	Mu =	135645	(Nmm/mm)
- Tension Reinforcement		D 16/22@300	
- Area of tension Reinforcement	As =	1.937	(mm2/mm)
- Thickness of cover:	as(-) =	50	(mm)
- Distance from compression face to centroid of tension reinf.	ds(-) = hf - as(-) =	215	(mm)
- Thickness of the equivalent stress block (mm)	$a = \frac{A_s.f_y}{0.85f'_c.b} =$	36.47	(mm)
- Resistance factor	ϕ =	1.0	
- Moment resistance:	Φ.Mn =Φ.As.fy.[ds(-) - a(-)/2]=	152479	(Nmm/mm)
- This moment strength will be reduced because of the axial tension force:		$T = \frac{R_{max}}{L_c + 2.H_w}$	
- Barrier height:	Hw =	1154	(mm)
	Rmax =	890.62	(KN)
- For impact at a wall segment	Lc =	3986	(mm)
	T_1 =	141.51	(N/mm)
- For impact at end of wall or joint	Rmax =	560.95	(KN)
	Lc =	2723	(mm)
	T_2 =	111.50	(N/mm)
- The shear consided	T =Max(T1,T2)=	141.51	(N/mm)
By assuming the interaction curve between moment and axial tension is a straight line:		$\frac{P_u}{\Phi.P_n} + \frac{M_u}{\Phi.M_n} \le 1$	
- Nominal forces	Pu = T =	141.51	(N/mm)
	Φ.Pn =Φ.As.fy	1549.85	(N/mm)
- Final resistance moment	$M_r = \Phi.M_n.\left(1 - \frac{P_u}{\Phi.P_n}\right)$	138557	(Nmm/mm)
=> Check condition	Mu ≤ Mr	OK	

CALCULATION PROCEDURE & STANDARD:

- Bridge Design Standard 22 TCN - 272 - 05 (considered with AASHTO LRFD 2007)

A. GENERAL DATA:

1. Design live load

Design vehicle load		HL93	
Number of lanes		3.00	(lanes)
Design earthquake class	Class	VII	

2. Bridge width

Width of carriageway	B_{CAR} =	11.50	(m)
Width of barrier wall	B_{lc} =	0.50	(m)
Bridge width	B =	12.50	(m)

3. Material properties:

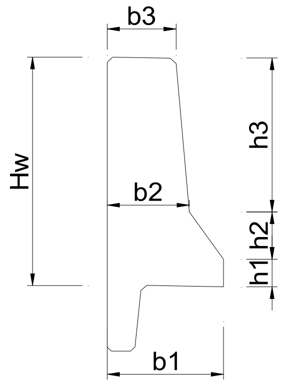
Concrete

Compressive strength of cylindrical at 28 days age	f'c =	25.00	MPa
Concrete density	g =	24.50	KN/m3
Elastic modulus	E_c =	26875.00	MPa
Tensile strength of concrete	f_r =	3.15	MPa

Steel CB-400-V

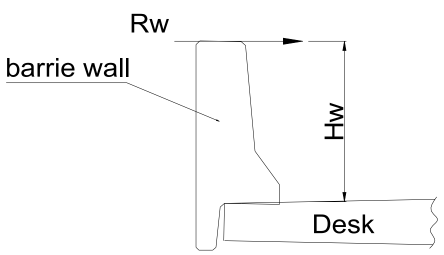
Modulus of elasticity	E_s =	200000	MPa
Yield strength of steel bar	f_y =	400	MPa

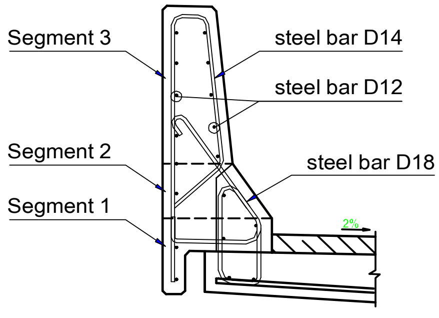
4. Dimensions of RC median barrier wall:



b1	490	(mm)
b2	310	(mm)
b3	240	(mm)
h1	159	(mm)
h2	255	(mm)
h3	740	(mm)

5. Diagram of Calculation





6. Railing shall be proportioned such that:

$\boxed{R \geq Ft}$ (13.7.3.3-1)

In which:

R - Total resistance of the barrier wall

Ft - Transverse vehicle impact force

7. General value:

- Diameter of longitual steel bar 12 (mm)
- Diameter of stirrup 18 (mm)
- Reinf. Spacing of stirrup 150 (mm)
- Φ Bending resistance factor 1

7.1 Choose Design force for barrier wall : (AASHTO2007 Table 13.2-1)

- Barrier wall containment level: L5

Ft 550 (KN)

He(min) 1070

7.2 Total capacity of Barrier wall:

7.2.1. Resistance of concrete wall for vertical axial (Mw.H)

+ Mw for out-face

Segment	Width of Segment b' = h	Number of bars n	Effective Depth d(+)	Area of bars As	$a=\frac{A_s.f_y}{0,85.f'_c.b}$	Φ.Mn(+) $\Phi.A_s.f_y.\left(d-\frac{a}{2}\right)$
	(mm)	(Bar)	(mm)	(mm^2)	(mm)	(KNmm)
1	159	1	426	113	13	18968.93
2	255	1	246	113	8	10939.94
3	740	5	176	565	14	38183.42

+ Mw for Int-face

Segment	Width of Segment b' = h	Number of bars n	Effective Depth d(+)	Area of bars As	$a=\frac{A_s.f_y}{0,85.f'_c.b}$	Φ.Mn(-) $\Phi.A_s.f_y.\left(d-\frac{a}{2}\right)$
	(mm)	(Bar)	(mm)	(mm2)	(mm)	(KNmm)
1	159	1	416	113	13	18516.54
2	255	1	246	113	8	10939.94
3	740	5	176	565	14	38183.42

+ Resistance of concrete wall for vertical axial (Mw.H)

Segment	Width of Segment b' = h	Φ.Mn(+) Out-face	Φ.Mn(-) Int-face	Φ.Mni Average of two face	Mw.H $\sum \Phi.M ni$
	(mm)	(KNmm)	(KNmm)	(KNmm)	(KNmm)
1	159	18968.93	18516.54	18742.73	67866.10
2	255	10939.94	10939.94	10939.94	
3	740	38183.42	38183.42	38183.42	

Where:

d - Average distance from compression face to centroid of tension reinforcement (mm)

a - Thickness of the equivalent stress block (mm)

As - Area of tension reinforcement (mm2)

7.2.2. Transverse Ultimate resistance of wall (Mc)

+ Transverse resistance of RC barrier wall (Mc)

Shear contact area: (mm2/mm) $As = \frac{\pi.\Phi^2}{4 * D}$ (with D is Reinf. Spacing of shear)

and b =1 m

Segment	Hight of Segment h	Shear contact area As	Effective Depth d	$a=\frac{A_s.f_y}{0,85.f'_c.b}$	Mci $\Phi.A_s.f_y.\left(d-\frac{a}{2}\right)$	Mc $\frac{\sum \Phi.M.hi}{\sum hi}$
	(mm)	(mm2/mm)	(mm)	(mm)	(KNmm)	(KNmm)
1	159	1.696	301	31.93	193.419	147.288
2	255	1.696	211	31.93	132.346	
3	740	1.696	226	31.93	142.525	

+ Total ultimate resistance of RC barrier wall:

- For impacts within a wall segment :

$$R_w = \left(\frac{2}{2.L_c - L_t}\right).\left(8.M_b + 8.M_w.H + \frac{M_c.L_c^2}{H_w}\right)$$

(TCN 13.7.3.4-1)

In which :

- Rw - Total transverse resistance of the RC barrier wall (N)
- Lc - Critical length of yield line failure pattern (mm)
- Lt - Longitudinal length of distribution of impact force Ft (mm)
- Mw - Flexural resistance of a wall (KNmm/mm)
- Mc - Transverse flexural resistance of wall (KNmm/mm)
- Mb - Additional flexural resistance of beam in addition to Mw, if any, at top of wall (KNmm/mm)
- Hw - Height of barrier wall Hw (mm)

- Critical length of yield line failure pattern Lc :

$$L_c = \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + \frac{8.H_w.(M_b + M_w.H)}{M_c}}$$

(TCN 13.7.3.4-2)

Lt (mm)	Hw (mm)	Mb (KN.mm)	Mc (KN.mm/mm)	Mw.H (KN.mm)	Lc (mm)	Rw (KN)
2440	1154	0	147.29	67866.10	3616	923.12

- For impacts at end of wall or at joint :

$$R_w = \left(\frac{2}{2.L_c - L}\right).\left(M_b + M_w.H + \frac{M_c.L_c^2}{H_w}\right)$$

(TCN 13.7.3.4-1)

- Critical length of yield line failure pattern Lc :

$$L_c = \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + \frac{H_w.(M_b + M_w.H)}{M_c}}$$

Lt (mm)	Hw (mm)	Mb (KN.mm)	Mc (KN.mm/mm)	Mw.H (KN.mm)	Lc (mm)	Rw (KN)
2440	1154	0	147.29	67866.10	2641	674.24

8. RESISTANCE CHECK FOR RC BARRIER WALL

- Condition 1 R = R w≥ Ft With : Ft = 550 (KN)

+ Resistance Check for RC barrier wall accordant Condition 1

Combination	Resistance barrier Wall Rw (KN)	Ft (KN)	Check Condition (1)
1. Impact at end of wall or joint	674.24	550	**OK**
2. Impact at a wall segment	923.12	550	**OK**

9. SEFT WEIGHT OF RC BARRIER WALL (DC_{lc})

+ Seft weight of concrete γ_c 24.5 (KN/m^3)
+ Seft weight of steel γ_s 7.85 (KN/m^3)
+ seft weight of Asphalt concrete γ_a 23 (KN/m^3)

- Seft weight of concrete wall

+ Area of concrete wall Ac = 0.4387 (m2)
+ Load due to weight of wall $DC_c = \gamma_c.A_c$

DC_{lc} = 10.75 (KN/m)

10. CHECK SHEAR-RESISTANCE OF RC AT BASE OF THE WALL JOINT WITH DECK

- Arrangement of stirrup D18 attach in overhang

Assuming that Rw spreads out at a 1:1 slope from Lc

- The tensile force per unit of length in the overhang, is given by:

$$T = \frac{R_{\max}}{L_c + 2.H_w}$$

- Height of barrier	Hw =	1154	(mm)
- Maximum of load impact on barrier wall	Rmax =	923.12	(KN)
	Lc =	3616	(mm)
	T_1 =	155.82	(N/mm)
- For Impact at end of barrier wall	Rmax =	674.24	(KN)
	Lc =	2641	(mm)
	T_2 =	136.23	(N/mm)
- Shear load for calculate	T =	$Max(T_1,T_2)$	
	T =	155.82	(N/mm)

- The nominal shear resistance Vn of the interface plane following:

$$V_n = c.A_{cv} + \mu.(A_{vf}.f_y + P_c)$$

Which shall not exceed $0.2f'_c$ or $5.5A_{cv}$

Where:

- Shear contact area:	Acv =	b_1.1 mm	
	Acv =	490	(mm2/mm)
- Dowel area across shear plane:	$A_{vf} = \frac{\pi.\Phi^2}{4*D}$		(Determined in 9.2.2)
	Avf =	1.696	(mm2/mm)
- Yield strenght of reinforcement	fy =	400	(MPa)
- Permanent compressive force:	Pc =	DC_{lc}.1 mm	
	Pc =	10.75	(N/mm)
- Strength of weaker concrete	f'c =	25	(MPa)
- Cohesion factor	c =	0.52	[5.8.4.2 - 22TCN 272-05]
- Friction factor	μ =	0.60	[A5.8.4.2 - 22TCN 272-05]
	Vn =	668.40	(N/mm)
	$0.2f'_c.A_{cv}$ =	2450	(N/mm)
	$5.5A_{cv}$ =	2695	(N/mm)

- Nominal shear resistance: Vn = Min(Vn , 0.2f'c.Acv , 5.5Acv)

Vn = 668.40 (N/mm) **> T = 155.82 (N/mm) : OK**

+ The minimum cross-sectional area of dowels across the shear plane:

$$A'_{vf} = 0.35\frac{b_1.s}{f_y}$$ *[5.8.4.1 - 22TCN 272-05]*

A'_{vf} = 64.31 (mm2)

- Number of stirrup input deck n = 1
- Cross-sectional area of stirrup input deck

As = n.Avf.s

As = 254.47 (mm2) **> A'vf : OK**

- The development length l_n shall not less than 3 values then:

$\frac{100.\Phi}{\sqrt{30}} =$ 329 (mm) With Φ =18 (mm)

8.Φ = 144 (mm)

And 150 (mm)

- The development length: l_n = 329 (mm) ***(The required modify)***
- Modification factor for adequate cover: k_1 = 0.7

$l'_n = k_1.l_n$

l'_n = 230 (mm)

- The development length after modify: l_n = 230 (mm)
- The Available development length: l_c = hf - as(+)

l_c = 196 ***mm (which is not adequate)***

- Unless the required area is reduced to $A_{vf}(hc) = A_{vf}.l_c / l_n$

$A_{vf}(hc)$ = 1.445 (mm2)

By using this area to recalculate Mc, Lc, Rw (The determined following 5.2.2)

Segment	Height of Segment h	Shear contact area As	Effective Depth d	$a = \frac{A_s.f_y}{0,85.f'_c.b}$	Mci $\Phi.A_s.f_y.\left(d - \frac{a}{2}\right)$	Mc $\frac{\sum \Phi.M.hi}{\sum hi}$
	(mm)	(mm2/mm)	(mm)	(mm)	(KNmm)	(KNmm)
1	159	1.445	301	27.21	166.162	126.858
2	255	1.445	211	27.21	114.127	
3	740	1.445	226	27.21	122.799	

- For impacts within a wall segment :

Lt (mm)	Hw (mm)	Mb (KN.mm)	Mc (KN.mm/mm)	Mw.H (KN.mm)	Lc (mm)	Rw (KN)
2440	1154	0	126.86	67866.10	3755	825.61

- For impacts at end of wall or joint :

Lt (mm)	Hw (mm)	Mb (KN.mm)	Mc (KN.mm/mm)	Mw.H (KN.mm)	Lc (mm)	Rw (KN)
2440	1154	0	126.86	67866.10	2671	587.27

+ Check railing following Condition 1

Combination	Resistance of Wall		Check
	Rw (KN)	R (KN)	Condition (1)
1. Impact at end of wall or joint	587.27	550.00	**OK**
2. Impact at a wall segment	825.61	550.00	**OK**

CALCULATION PROCEDURE & STANDARD:

- Bridge Design Standard 22 TCN - 272 - 05 (considered with AASHTO LRFD 2007)

A. GENERAL DATA:

1. Design live load

Design vehicle load		HL93	
Number of lanes		3.00	(lanes)
Design earthquake class	Class	VII	

2. Bridge width

Width of carriageway	B_{CAR} =	11.50	(m)
Width of barrier wall	B_{lc} =	0.50	(m)
Bridge width	B =	12.50	(m)

3. Material properties:

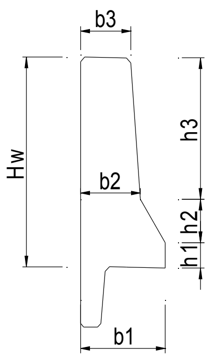
Concrete

Compressive strength of cylindrical at 28 days age	f'c =	25.00	MPa
Concrete density	g =	24.50	KN/m3
Elastic modulus	E_c =	26875.00	MPa
Tensile strength of concrete	f_r =	3.15	MPa

Steel CB-400-V

Modulus of elasticity	E_s =	200000	MPa
Yield strength of steel bar	f_y =	400	MPa

4. Dimensions of RC median barrier wall:



b1	490	(mm)
b2	310	(mm)
b3	240	(mm)
h1	159	(mm)
h2	255	(mm)
h3	740	(mm)

5. Diagram of Calculation

6. Railing shall be proportioned such that:

$$\boxed{R \geq Ft} \qquad (13.7.3.3\text{-}1)$$

In which:

R - Total resistance of the barrier wall

Ft - Transverse vehicle impact force

7. General value:

- Diameter of longitual steel bar	12	(mm)
- Diameter of stirrup	18	(mm)
- Reinf. Spacing of stirrup	150	(mm)
- Φ Bending resistance factor	1	

7.1 Choose Design force for barrier wall : (AASHTO2007 Table 13.2-1)

- Barrier wall containment level:

Ft	550	(KN)
He(min)	1070	

7.2 Total capacity of Barrier wall:

7.2.1. Resistance of concrete wall for vertical axial (Mw.H)

+ Mw for out-face

Segment	Width of Segment b' = h	Number of bars n	Effective Depth d(+)	Area of bars As	$a=\frac{A_s.f_y}{0,85.f'_c.b}$	Φ.Mn(+) $\Phi.A_s.f_y\left(d-\frac{a}{2}\right)$
	(mm)	(Bar)	(mm)	(mm^2)	(mm)	(KNmm)
1	159	1	426	113	13	18968.93
2	255	1	246	113	8	10939.94
3	740	5	176	565	14	38183.42

+ Mw for Int-face

Segment	Width of Segment b' = h	Number of bars n	Effective Depth d(+)	Area of bars As	$a=\frac{A_s.f_y}{0,85.f'_c.b}$	Φ.Mn(-) $\Phi.A_s.f_y\left(d-\frac{a}{2}\right)$
	(mm)	(Bar)	(mm)	(mm2)	(mm)	(KNmm)
1	159	1	416	113	13	18516.54
2	255	1	246	113	8	10939.94
3	740	5	176	565	14	38183.42

+ Resistance of concrete wall for vertical axial (Mw.H)

Segment	Width of Segment b' = h	Φ.Mn(+) Out-face	Φ.Mn(-) Int-face	Φ.Mni Average of two face	Mw.H $\sum \Phi.M\,ni$
	(mm)	(KNmm)	(KNmm)	(KNmm)	(KNmm)
1	159	18968.93	18516.54	18742.73	67866.10
2	255	10939.94	10939.94	10939.94	
3	740	38183.42	38183.42	38183.42	

Where:

d - Average distance from compression face to centroid of tension reinforcement (mm)
a - Thickness of the equivalent stress block (mm)
As - Area of tension reinforcement (mm2)

7.2.2. Transverse Ultimate resistance of wall (Mc)

+ Transverse resistance of RC barrier wall (Mc)

Shear contact area: (mm2/mm) $As = \frac{\pi.\Phi^2}{4 * D}$ (with D is Reinf. Spacing of shear) and b =1 m

Segment	Hight of Segment h	Shear contact area As	Effective Depth d	$a=\frac{A_s.f_y}{0,85.f'_c.b}$	Mci $\Phi.A_s.f_y\left(d-\frac{a}{2}\right)$	Mc $\frac{\sum \Phi.M.hi}{\sum hi}$
	(mm)	(mm2/mm)	(mm)	(mm)	(KNmm)	(KNmm)
1	159	1.696	301	31.93	193.419	147.288
2	255	1.696	211	31.93	132.346	
3	740	1.696	226	31.93	142.525	

+ Total ultimate resistance of RC barrier wall:

- For impacts within a wall segment :

$$R_w = \left(\frac{2}{2.L_c - L_t}\right).\left(8.M_b + 8.M_w.H + \frac{M_c.L_c^2}{H_w}\right)$$ (TCN 13.7.3.4-1)

In which :

Rw - Total transverse resistance of the RC barrier wall (N)
Lc - Critical length of yield line failure pattern (mm)
Lt - Longitudinal length of distribution of impact force Ft (mm)
Mw - Flexural resistance of a wall (KNmm/mm)
Mc - Transverse flexural resistance of wall (KNmm/mm)
Mb - Additional flexural resistance of beam in addition to Mw, if any, at top of wall (KNmm/mm)
Hw - Height of barrier wall Hw (mm)

- Critical length of yield line failure pattern Lc :

$$L_c = \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + \frac{8.H_w.(M_b + M_w.H)}{M_c}}$$ (TCN 13.7.3.4-2)

Lt (mm)	Hw (mm)	Mb (KN.mm)	Mc (KN.mm/mm)	Mw.H (KN.mm)	Lc (mm)	Rw (KN)
2440	1154	0	147.29	67866.10	3616	923.12

- For impacts at end of wall or at joint :

$$R_w = \left(\frac{2}{2.L_c - L}\right).\left(M_b + M_w.H + \frac{M_c.L_c^2}{H_w}\right)$$ (TCN 13.7.3.4-1)

- Critical length of yield line failure pattern Lc :

$$L_c = \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + \frac{H_w.(M_b + M_w.H)}{M_c}}$$

Lt (mm)	Hw (mm)	Mb (KN.mm)	Mc (KN.mm/mm)	Mw.H (KN.mm)	Lc (mm)	Rw (KN)
2440	1154	0	147.29	67866.10	2641	674.24

8. RESISTANCE CHECK FOR RC BARRIER WALL

- Condition 1 R = R w≥ Ft With : Ft = 550 (KN)

+ Resistance Check for RC barrier wall accordant Condition 1

Combination	Resistance barrier Wall Rw (KN)	Ft (KN)	Check Condition (1)
1. Impact at end of wall or joint	674.24	550	**OK**
2. Impact at a wall segment	923.12	550	**OK**

9. SEFT WEIGHT OF RC BARRIER WALL (DC_{lc})

+ Seft weight of concrete γ_c 24.5 (KN/m^3)
+ Seft weight of steel γ_s 7.85 (KN/m^3)
+ seft weight of Asphalt concrete γ_a 23 (KN/m^3)

- Seft weight of concrete wall

+ Area of concrete wall Ac = 0.45 (m2)
+ Load due to weight of wall $DC_c = \gamma_c.A_c$

DC_{lc} = 11.03 (KN/m)

10. CHECK SHEAR-RESISTANCE OF RC AT BASE OF THE WALL JOINT WITH DECK

- Arrangement of stirrup D18 attach in overhang

Assuming that Rw spreads out at a 1:1 slope from Lc

- The tensile force per unit of length in the overhang, is given by:

$$T = \frac{R_{max}}{L_c + 2.H_w}$$

- Height of barrier Hw = 1154 (mm)
- Maximum of load impact on barrier wall Rmax = 923.12 (KN)

Lc = 3616 (mm)

T_1 = 155.82 (N/mm)

- For Impact at end of barrier wall Rmax = 674.24 (KN)

Lc = 2641 (mm)

T_2 = 136.23 (N/mm)

- Shear load for calculate $T = Max(T_1,T_2)$

T = 155.82 (N/mm)

- The nominal shear resistance Vn of the interface plane following:

$$V_n = c.A_{cv} + \mu.(A_{vf}.f_y + P_c)$$

Which shall not exceed $0.2f'_c$ or $5.5A_{cv}$

Where:

- Shear contact area: $Acv = b_1.1$ mm

$Acv =$ 490 (mm2/mm)

- Dowel area across shear plane:

$$A_{vf} = \frac{\pi.\Phi^2}{4*D}$$ (Determined in 9.2.2)

Avf = 1.696 (mm2/mm)

- Yield strenght of reinforcement fy = 400 (MPa)
- Permanent compressive force: $Pc = DC_{lc}.1$ mm

Pc = 11.03 (N/mm)

- Strength of weaker concrete f'c = 25 (MPa)
- Cohesion factor c = 0.52 [5.8.4.2 - 22TCN 272-05]
- Friction factor μ = 0.60 [A5.8.4.2 - 22TCN 272-05]

Vn = 668.57 (N/mm)

$0.2f'_c.A_{cv} =$ 2450 (N/mm)

$5.5A_{cv} =$ 2695 (N/mm)

- Nominal shear resistance: Vn = Min(Vn , 0.2f'c.Acv , 5.5Acv)

Vn = 668.57 (N/mm) **> T = 155.82 (N/mm) : OK**

+ The minimum cross-sectional area of dowels across the shear plane:

$$A'_{vf} = 0.35\frac{b_1.s}{f_y}$$ *[5.8.4.1 - 22TCN 272-05]*

$A'_{vf} =$ 64.31 (mm2)

- Number of stirrup input deck n = 1
- Cross-sectional area of stirrup input deck

As = n.Avf.s

As = 254.47 (mm2) **> A'vf : OK**

- The development length l_n shall not less than 3 values then:

$$\frac{100.\Phi}{\sqrt{30}} =$$ 329 (mm) With Φ =18 (mm)

8.Φ = 144 (mm)

And 150 (mm)

- The development length: $l_n =$ 329 (mm) ***(The required modify)***
- Modification factor for adequate cover: $k_1 =$ 0.7

$l'_n = k_1.l_n$

$l'_n =$ 230 (mm)

- The development length after modify: $l_n =$ 230 (mm)
- The Available development length: $l_c =$ hf - as(+)

$l_c =$ 194 ***mm (which is not adequate)***

- Unless the required area is reduced to $A_{vf}(hc) = A_{vf}.l_c / l_n$

$A_{vf}(hc) =$ 1.431 (mm2)

By using this area to recalculate Mc, Lc, Rw (The determined following 5.2.2)

Segment	Height of Segment h	Shear contact area As	Effective Depth d	$a=\frac{A_s.f_y}{0,85.f'_c.b}$	Mci $\Phi.A_s.f_y.\left(d-\frac{a}{2}\right)$	Mc $\frac{\sum \Phi.M.hi}{\sum hi}$
	(mm)	(mm2/mm)	(mm)	(mm)	(KNmm)	(KNmm)
1	159	1.431	301	26.93	164.546	125.643
2	255	1.431	211	26.93	113.042	
3	740	1.431	226	26.93	121.626	

- For impacts within a wall segment :

Lt (mm)	Hw (mm)	Mb (KN.mm)	Mc (KN.mm/mm)	Mw.H (KN.mm)	Lc (mm)	Rw (KN)
2440	1154	0	125.64	67866.10	3765	819.75

- For impacts at end of wall or joint :

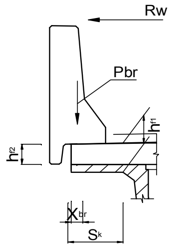
Lt (mm)	Hw (mm)	Mb (KN.mm)	Mc (KN.mm/mm)	Mw.H (KN.mm)	Lc (mm)	Rw (KN)
2440	1154	0	125.64	67866.10	2673	582.09

+ Check railing following Condition 1

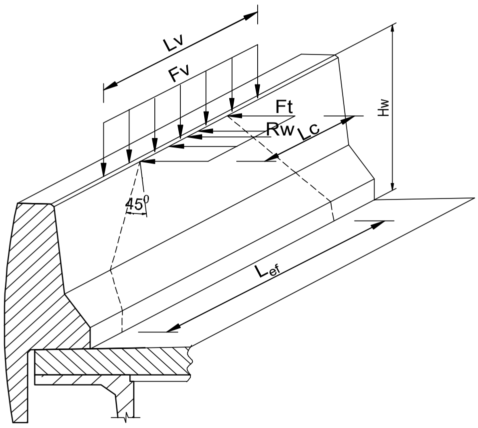
Combination	Resistance of Wall		Check
	Rw (KN)	R (KN)	Condition (1)
1. Impact at end of wall or joint	582.09	550.00	**OK**
2. Impact at a wall segment	819.75	550.00	**OK**

11. Ovehang of deck

Over hang length	Sk=	455	mm
Thickness of overhang (inside)	hf1=	278	mm
Thickness of overhang (outside)	hf2=	287	mm
Thickness of wearing surface	ha=	84	mm
Height of parapet	Hw =	1154	(mm)
Distance from centre of barrier to edge of overhang	X_{lc} =	100	(mm)



The top reinforcement must resist the negative bending moment over the exterior beam due to the collision and the deadload of overhang:



$$M_{CT} = \frac{R_w.H_w + R_r.H_r}{L_c + 2.H_w}$$

$$M_{CV} = \frac{F_V}{L_V}.S_k$$

Fv = 355 (KN)
Lv = 12200 (mm)

- Design case 1: The transverse and longitudinal forces on parapet (Rw) - (Extreme state)
- Design case 2: The vertical forces on parapet (Fv) - (Extreme state)
- Design case 3: The loads occupy the overhang - (strength state). This case not consided because overhang is very short, live load not act to overhang part.
- Combination momen due to dead load and collision

$$Mu = [\,1.25M_{DC} + 1.5M_{DW} + 1.0M_{CT}\,]$$

- Moment due to collision forces on parapet act to desk slab:

Combination	Resistance of barrier Rw (KN)	Effect length Lc (mm)	Momen (M_{CT}) (KNmm/mm)
1. Impact at end of wall or joint	582.09	2641	135.72
2. Impact at a wall segment	819.75	3616	159.68

+ The Moment due to transverse collision: M_{CT} = 135.72 (KNmm/mm)
+ The Moment due to vertical forces: M_{CV} = 13.24 (KNmm/mm)

- Moment due to seftweight of desk slab and parapet:

$$M_{DC} = DC_{lc}.(S_k - X_{lc}) + \gamma_c.h_f.S_k^2$$

M_{DC} = 4.63 (KNmm/mm)

- Momen due to wearing surface

$$M_{DW} = \begin{cases} 0 : (S_k - b_1) \le 0 \\ (\gamma_a.h_a + \gamma_p.h_p).(S_k - b_1)^2 \end{cases}$$

M_{DW} = 0.24 (KNmm/mm)

- Combination loading:

Design Case	Unit	M_{DC}	M_{DW}	M_{CT}	M_{CV}	Mu
Design case 1	(Nmm/mm)	4.63	0.24	135.72	0	141875.3
Design case 2	(Nmm/mm)	4.63	0.24	0	13.24	19393.0

1.4 Expansion Joint
1.4 Tính khe co giãn

§ Design of P10 expanstion joint

§ 1 . Design condition

Live load	P ＝ 72.5 kN
Dynamic load allowance	IM ＝ 0.75 (STRENGTH Ⅰ)
Temperature change	＋10 ℃ ～ ＋47 ℃ (Main and Aproch Bridge)
Standard temperature	＋25 ℃

§ 2 . Movement

Unit(mm)

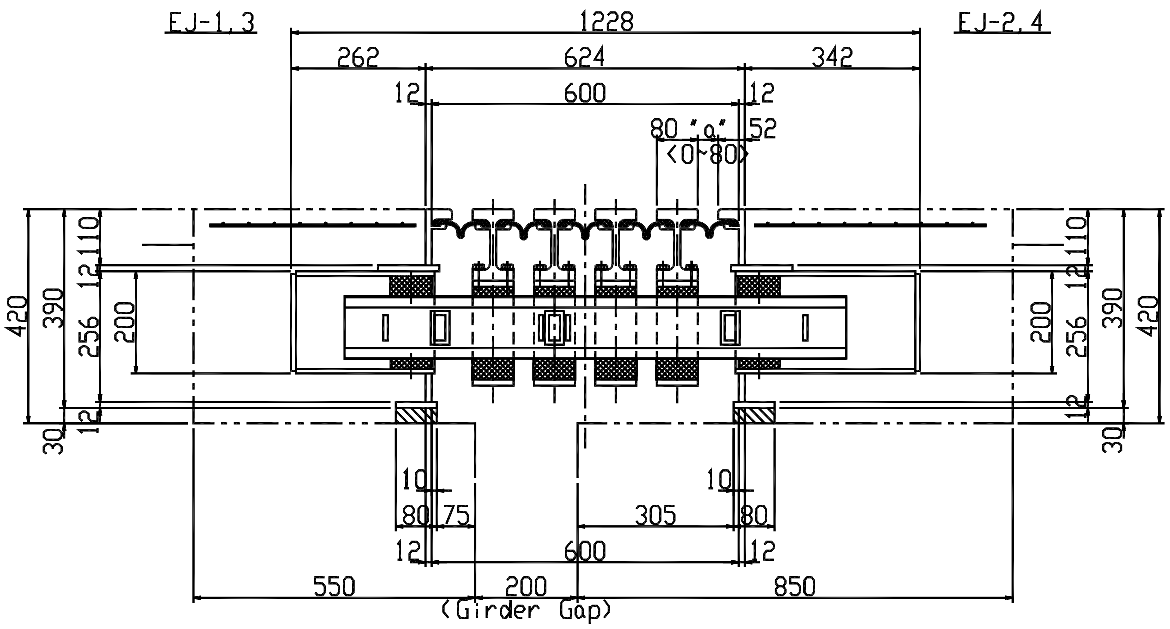
P10		ΔL(+)	ΔL(-)	Total
Ordinary movement	Temperature	103.4	70.4	173.8
	Creep,Shrinkage		106.9	106.9
	allowance (20%)	20.7	35.5	56.1
	Total	124.1	212.8	336.8
	Design of movement	340.0		

§ 3 . Selection of a type

A expansion joint type is selected as the maximal movement per one cell 80mm(±40mm)

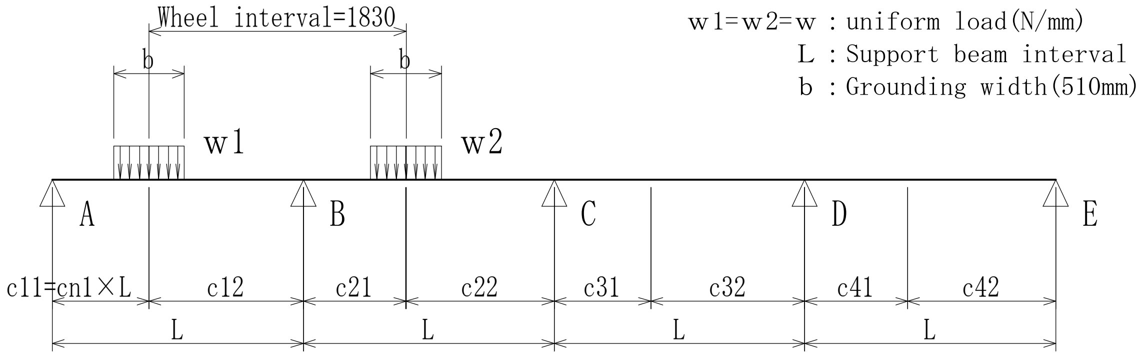
	P10
The number of a cells	5
Type	LR-5S
Maximal movement(mm)	400 (±200)

§ 4 . Section of a support system



§5. Calculation of a middle beam

A middle beam is calculation as 4continues beam.
EI(Stiffness)is fixed



State of loading on a middle beam

(1) Bending moment on each support position

by the theorem of three moments in case of the continuous beam

$$MA\cdot L + 4\cdot MB\cdot L + MC\cdot L - \frac{w1\cdot b\cdot c11\cdot\{4(L^2 - c11^2) - b^2\}}{4\cdot L} - \frac{w2\cdot b\cdot c22\cdot\{4(L^2 - c22^2) - b^2\}}{4\cdot L} = 0$$

$$MB\cdot L + 4\cdot MC\cdot L + MD\cdot L - \frac{w2\cdot b\cdot c21\cdot\{4(L^2 - c21^2) - b^2\}}{4\cdot L} = 0$$

$$MC\cdot L + 4\cdot MD\cdot L + ME\cdot L = 0$$

Now above equations are arranged as follows,

$$B = \frac{w1\cdot b\cdot c11\cdot\{4(L^2 - c11^2) - b^2\}}{4\cdot L^2} + \frac{w2\cdot b\cdot c22\cdot\{4(L^2 - c22^2) - b^2\}}{4\cdot L^2}$$

$$C = \frac{w2\cdot b\cdot c21\cdot\{4(L^2 - c21^2) - b^2\}}{4\cdot L^2}$$

$$D = 0$$

$$MA + 4 \cdot MB + MC = B$$

$$MB + 4 \cdot MC + MD = C$$

$$MC + 4 \cdot MD + ME = D$$

MA and ME as "0" on account of a end support

$$MA = 0$$

$$MB = -\frac{1}{56}(15B - 4C + D)$$

$$Mc = \frac{1}{14}(B - 4C + D)$$

$$MD = -\frac{1}{56}(B - 4C + 15D)$$

$$ME = 0$$

(2) Reaction foce on each support section

Reaction force on a support position of a continuous beam is found with a next equation.

$$RA = RA0 - \frac{MA - MB}{L}$$

$$RB = RB0^{l} + RB0^{r} + \frac{MA - MB}{L} - \frac{MB - MC}{L}$$

$$RC = RC0^{l} + RC0^{r} + \frac{MB - MC}{L} - \frac{MC - MD}{L}$$

$$RD = RD0^{l} + RD0^{r} + \frac{MC - MD}{L} - \frac{MD - ME}{L}$$

$$RE = RE0 + \frac{MD - ME}{L}$$

"0"is an answer that was calculated as a simple beam.
$RB0^{l}$ is reaction force on left side of a support position B.
$RB0^{r}$ is reaction force on right side of a support position B.
Other reaction forces are found with the same equation.

(3) Bending moment on each span

Bending moment on each span "X" is a maximum, if a shearing force is "0".

A position on a span AB

$$XAB = \frac{RA}{W} + \left(c11 - \frac{b}{2}\right)$$

A position on a span BC

$$XBC = \frac{RA + RB - w \cdot b}{W} + \left(c21 - \frac{b}{2}\right)$$

A maximal bending moment "Mx" on the span AB

$$MX = MX0 + MA\left\{1 - \left(\frac{XAB}{L}\right)\right\} + MB\left(\frac{XAB}{L}\right)$$

A maximal bending moment "Mx" on the span BC

$$MX = MX0 + MA\left\{1 - \left(\frac{XBC}{L}\right)\right\} + MB\left(\frac{XBC}{L}\right)$$

MX0 is a maximal bending moment in a loading position on a simpie beam.
RA0 and RB0 are a reaction foce of a simple beam.

$$MX0 = RA0 \cdot X - \frac{w}{2}\left\{X - \left(c11 - \frac{b}{2}\right)\right\}^2$$

$$RA0 = P' \times \left(\frac{L - c11}{L}\right)$$

P': see next page

$$RB0 = P' - RA0$$

（4）Loading condition and bending moment diagram

Live load　$P = 72.5$ (kN)

Dynamic load allowance　$IM = 0.75$

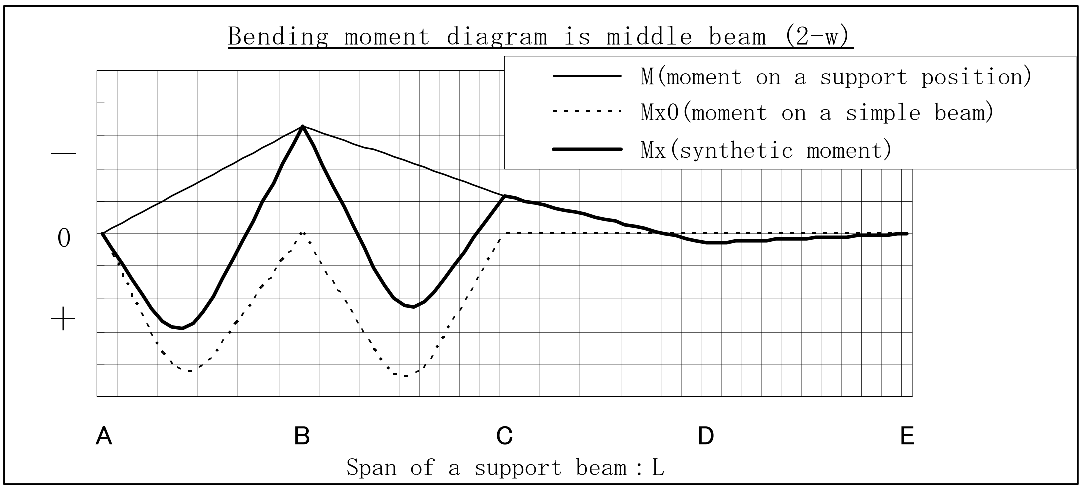
Load　$P' = P(1 + IM)$ (kN)

uniform load　$W = \dfrac{P(1 + IM)}{b} = \dfrac{P'}{b}$ (N/mm)

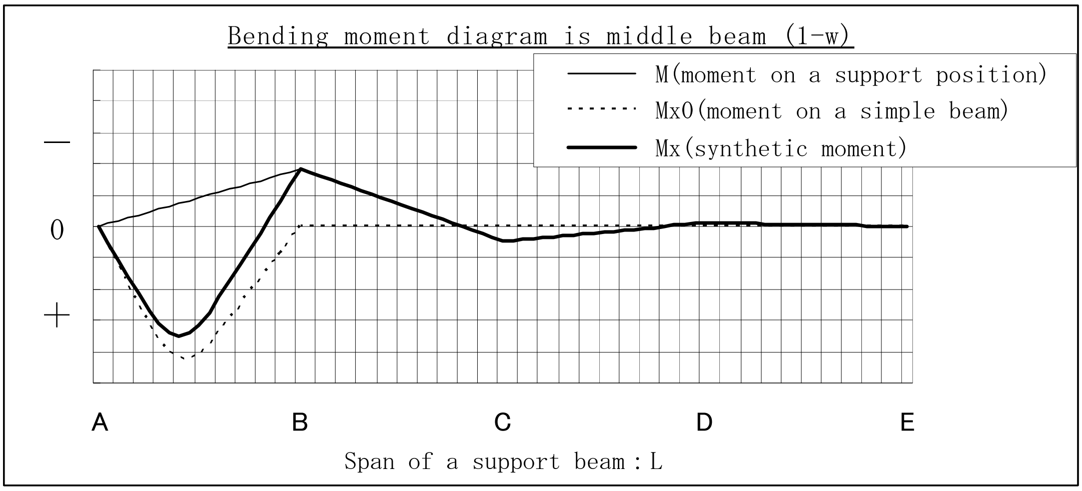
Loading position　$c11 = 0.4 \cdot L$

Hereafter, it is calculated in the state of the loading of span AB and apan BC.

4-1)　Loding to span AB and span BC(2-W)



4-2)　only loding to span AB(1-w)

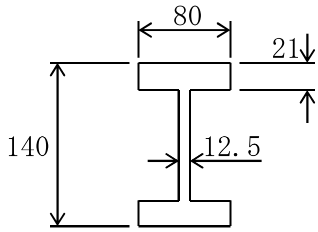


(5) Calculation of a middle beam

$$\text{Bending stress} \quad \sigma b = \frac{M}{Z} \quad (N/mm^2)$$

It is caluculated in the maximal moment position of a following table.

Section of a middle beam



Modulus of section Z = 192087.2 (mm^3)

Allowable fatigue stress range σa = 210 (N/mm^2)

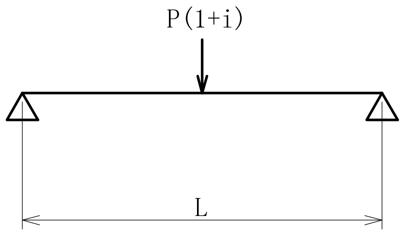
material … SM490YB

Result of a calculation

		Unit	P10	
			2-w	1-w
Loding conditions	L	(mm)	1650	1650
	IM		0.750	0.750
	P’	(kN)	127	127
	w	(N/mm)	248.8	248.8
	c 11	(mm)	660.0	660.0
Theorem of three moments	B	(N・mm)	143902003	68346491
	C	(N・mm)	76415939	0
Bending moment of a support position	M_B	(N・mm)	-33086898	-18307096
	M_C	(N・mm)	-11554411	4881892
	M_D	(N・mm)	2888603	-1220473
Bending moment of a span in case of a simpie	M_{X0} : span AB	(N・mm)	41674081	42234784
	M_{X0} : span BC	(N・mm)	43895095	0
Effective span synthetic moment of a continuation	M_X : span AB	(N・mm)	29032881	34840936
	M_X : span BC	(N・mm)	22393980	0
Position of a maximal moment			MB	MX : span AB
maximal moment		(N・mm)	33086898	34840936
Bending stress	$\sigma b_{(MAX)}$	(N/mm^2)	172	181

§ 6 . Calculation of a support beam

Support beam is a simple beam that has a span of a bearing"L" in case of a neutral joint gap.



L : Neutral joint gap

$$L = 80 + 5 \times 40 + 4 \times 80 + 80$$
$$= 680 \text{(mm)}$$

Bearing span $L' = \dfrac{L}{\sin\theta}$ (mm) θ : Angle of skew

Live load $P = 72.5$ (kN)

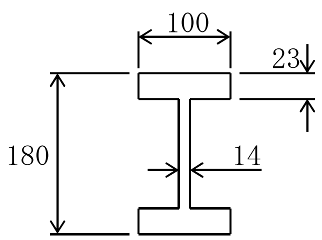
Dynamic load allowance $IM = 0.75$

Load $P' = P(1 + IM)$ (kN)

Bending moment $M = \dfrac{P' \times L}{4}$ (N・mm)

Bending stress $\sigma b = \dfrac{M}{Z}$ (N/mm^2)

Section of a spport beam



Modulus of section $Z = 361610.7$ (mm^3)

Allowable fatigue stress range $\sigma a = 210$ (N/mm^2) material … SM490YB

Result of a calculation

	nuit	P10
θ	(° ' ")	90° 0' 0"
L'	(mm)	680.0
P'	(kN)	126.88
M	(N・mm)	21569600
σ b	(N/mm^2)	60

§　Design of A2 expanstion joint

§１. Design condition

Live load	P ＝ 72.5　kN
Dynamic load allowance	IM ＝ 0.75　(STRENGTH Ⅰ)
Temperature change	＋10 ℃ ～ ＋47 ℃ (Main and Aproch Bridge)
Standard temperature	＋25 ℃

§２. Movement

Unit(mm)

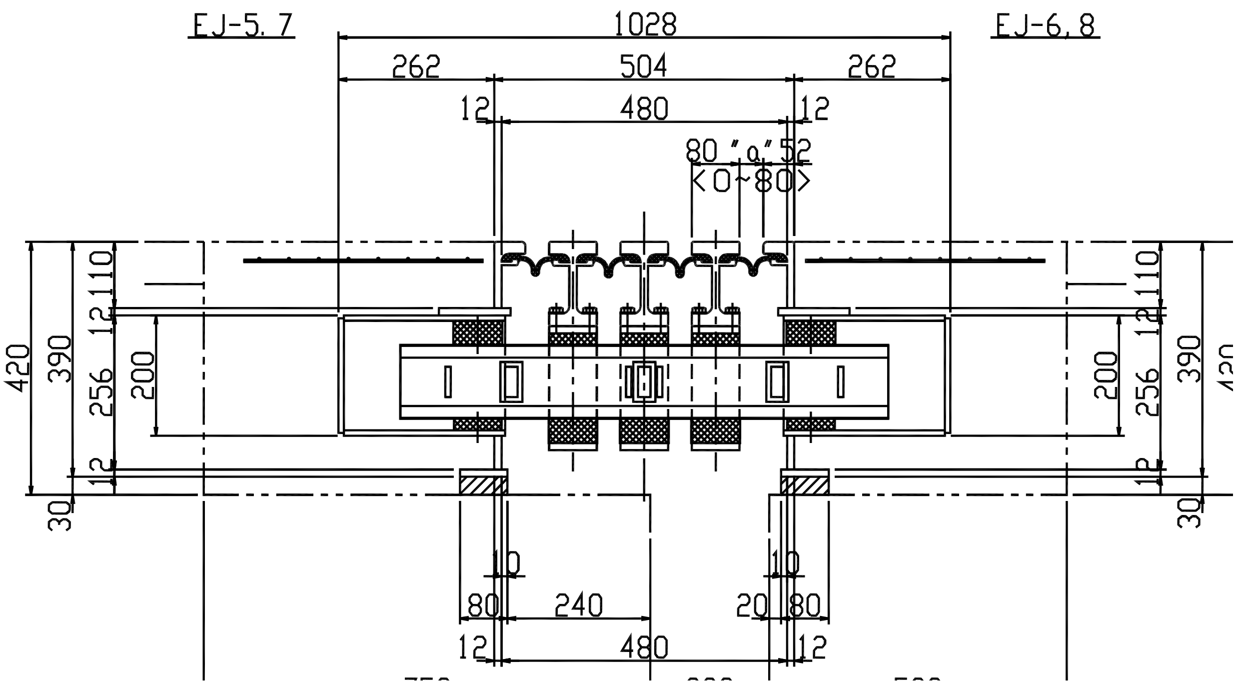
A2		ΔL(+)	ΔL(-)	Total
Ordinary movement	Temperature	74.9	51.0	125.9
	Creep, Shrinkage		84.0	84.0
	allowance (20%)	15.0	27.0	42.0
	Total	89.9	162.0	251.9
	Design of movement	340.0		

§３. Selection of a type

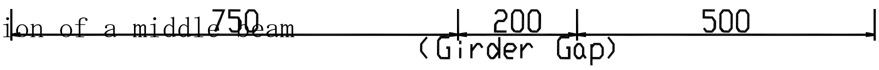
A expansion joint type is selected as the maximal movement per one cell 80mm(±40mm)

	A2
The number of a cells	4
Type	LR-4S
Maximal movement(mm)	320 (±160)

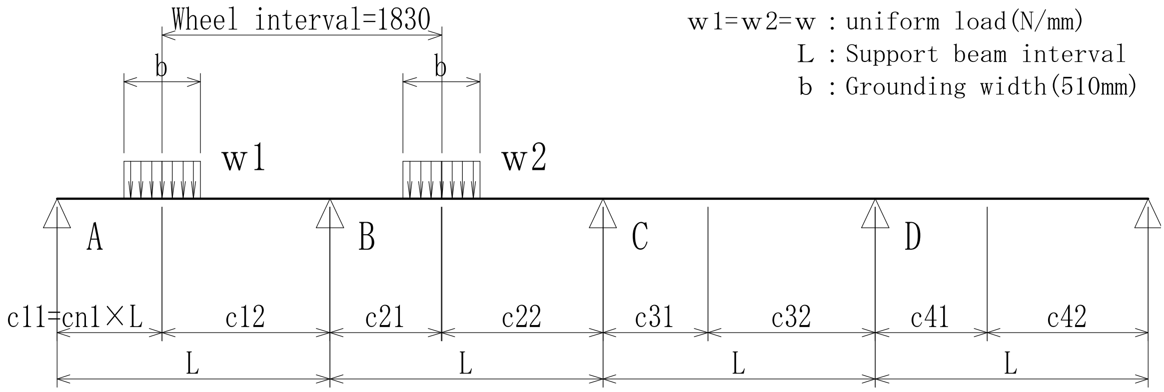
§４. Section of a support system



§５．Calculation of a middle beam



A middle beam is calculation as 4continues beam.
EI(Stiffness)is fixed



State of loading on a middle beam

（１）Bending moment on each support position

by the theorem of three moments in case of the continuous beam

$$MA\cdot L + 4\cdot MB\cdot L + MC\cdot L - \frac{w1\cdot b\cdot c11\cdot\{4(L^2 - c11^2) - b^2\}}{4\cdot L}$$

$$- \frac{w2\cdot b\cdot c22\cdot\{4(L^2 - c22^2) - b^2\}}{4\cdot L} = 0$$

$$MB\cdot L + 4\cdot MC\cdot L + MD\cdot L - \frac{w2\cdot b\cdot c21\cdot\{4(L^2 - c21^2) - b^2\}}{4\cdot L} = 0$$

$$MC\cdot L + 4\cdot MD\cdot L + ME\cdot L = 0$$

Now above equations are arranged as follows,

$$B = \frac{w1\cdot b\cdot c11\cdot\{4(L^2 - c11^2) - b^2\}}{4\cdot L^2} + \frac{w2\cdot b\cdot c22\cdot\{4(L^2 - c22^2) - b^2\}}{4\cdot L^2}$$

$$C = \frac{w2\cdot b\cdot c21\cdot\{4(L^2 - c21^2) - b^2\}}{4\cdot L^2}$$

$$D = 0$$

$$MA + 4 \cdot MB + MC = B$$
$$MB + 4 \cdot MC + MD = C$$
$$MC + 4 \cdot MD + ME = D$$

MA and ME as ″0″ on account of a end support

$$MA = 0$$

$$MB = -\frac{1}{56}(15B - 4C + D)$$

$$Mc = \frac{1}{14}(B - 4C + D)$$

$$MD = -\frac{1}{56}(B - 4C + 15D)$$

$$ME = 0$$

(2) Reaction foce on each support section

Reaction force on a support position of a continuous beam is found with a next equation.

$$RA = RA0 - \frac{MA - MB}{L}$$

$$RB = RB0^{l} + RB0^{r} + \frac{MA - MB}{L} - \frac{MB - MC}{L}$$

$$RC = RC0^{l} + RC0^{r} + \frac{MB - MC}{L} - \frac{MC - MD}{L}$$

$$RD = RD0^{l} + RD0^{r} + \frac{MC - MD}{L} - \frac{MD - ME}{L}$$

$$RE = RE0 + \frac{MD - ME}{L}$$

″0″is an answer that was calculated as a simple beam.
$RB0^{l}$ is reaction force on left side of a support position B.
$RB0^{r}$ is reaction force on right side of a support position B.
Other reaction forces are found with the same equation.

(3) Bending moment on each span

Bending moment on each span "X" is a maximum, if a shearing force is "0".

A position on a span AB

$$XAB = \frac{RA}{W} + \left(c11 - \frac{b}{2}\right)$$

A position on a span BC

$$XBC = \frac{RA + RB - w \cdot b}{W} + \left(c21 - \frac{b}{2}\right)$$

A maximal bending moment "Mx" on the span AB

$$MX = MX0 + MA\left\{1 - \left(\frac{XAB}{L}\right)\right\} + MB\left(\frac{XAB}{L}\right)$$

A maximal bending moment "Mx" on the span BC

$$MX = MX0 + MA\left\{1 - \left(\frac{XBC}{L}\right)\right\} + MB\left(\frac{XBC}{L}\right)$$

MX0 is a maximal bending moment in a loading position on a simpie beam.
RA0 and RB0 are a reaction foce of a simple beam.

$$MX0 = RA0 \cdot X - \frac{w}{2}\left\{X - \left(c11 - \frac{b}{2}\right)\right\}^2$$

$$RA0 = P' \times \left(\frac{L - c11}{L}\right)$$

P': see next page

$$RB0 = P' - RA0$$

（4）Loading condition and bending moment diagram

Live load　P ＝ 72.5 (kN)

Dynamic load allowance　IM ＝ 0.75

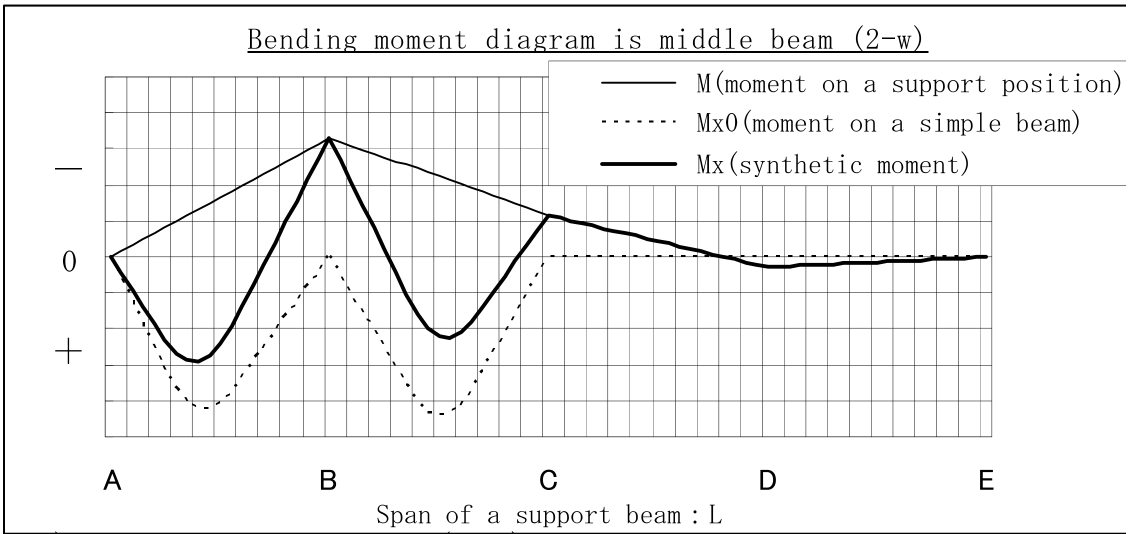
Load　$P' = P(1 + IM)$ (kN)

uniform load　$W = \dfrac{P(1 + IM)}{b} = \dfrac{P'}{b}$ (N/mm)

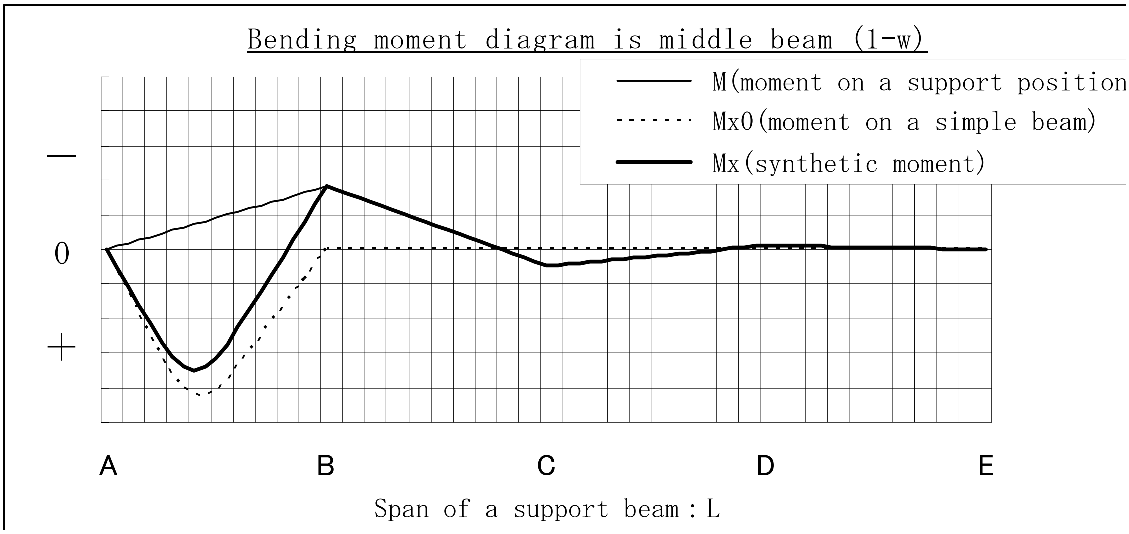
Loading position　$c11 = 0.4 \cdot L$

Hereafter, it is calculated in the state of the loading of span AB and apan BC.

4-1) Loding to span AB and span BC(2-W)



4-2) only loding to span AB(1-w)

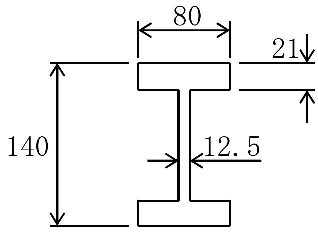


(5) Calculation of a middle beam

Bending stress $\sigma b = \dfrac{M}{Z}$ (N/mm^2)

It is caluculated in the maximal moment position of a following table.

Section of a middle beam



Modulus of section Z = 192087.2 (mm^3)

Allowable fatigue stress range σa = 210 (N/mm^2)

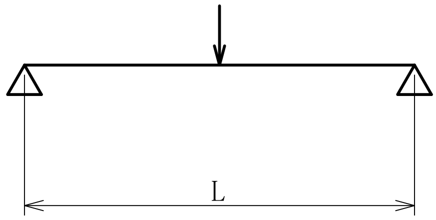
material … SM490YB

Result of a calculation

		Unit	A2	
			2-w	1-w
Loding conditions	L	(mm)	1650	1650
	IM		0.750	0.750
	P'	(kN)	127	127
	w	(N/mm)	248.8	248.8
	c 11	(mm)	660.0	660.0
Theorem of three moments	B	(N・mm)	143902003	68346491
	C	(N・mm)	76415939	0
Bending moment of a support position	M_B	(N・mm)	-33086898	-18307096
	M_C	(N・mm)	-11554411	4881892
	M_D	(N・mm)	2888603	-1220473
Bending moment of a span in case of a simpie	M_{X0} : span AB	(N・mm)	41674081	42234784
	M_{X0} : span BC	(N・mm)	43895095	0
Effective span synthetic moment of a continuation	M_X : span AB	(N・mm)	29032881	34840936
	M_X : span BC	(N・mm)	22393980	0
Position of a maximal moment			MB	MX : span AB
maximal moment		(N・mm)	33086898	34840936
Bending stress	σb (MAX)	(N/mm^2)	172	181

§ 6 . Calculation of a support beam

Support beam is a simple beam that has a span of a bearing"L" in case of a neutral joint gap.



L : Neutral joint gap

$L = 80 + 4 \times 40 + 3 \times 80 + 80$
$= 560$ (mm)

Bearing span $L' = \dfrac{L}{\sin\theta}$ (mm) θ : Angle of skew

Live load $P = 72.5$ (kN)

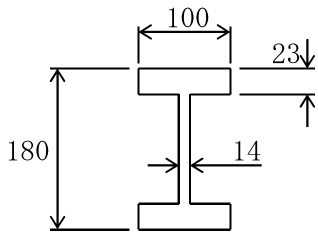
Dynamic load allowance $IM = 0.75$

Load $P' = P(1 + IM)$ (kN)

Bending moment $M = \dfrac{P' \times L}{4}$ (N･mm)

Bending stress $\sigma b = \dfrac{M}{Z}$ (N/mm^2)

Section of a spport beam



Modulus of section $Z = 361610.7$ (mm^3)

Allowable fatigue stress range $\sigma a = 210$ (N/mm^2) material … SM490YB

Result of a calculation

	nuit	A2
θ	(° ' ")	90° 0'0"
L'	(mm)	560.0
P'	(kN)	126.88
M	(N・mm)	17763200
σ_b	(N/mm^2)	49

2 CALCULATION SHEET FOR SUBSTRUCTURE – BẢNG TÍNH KẾT CẤU PHẦN DƯỚI

2.1 Abutment A1

2.1.1 General Data – Số liệu chung.
2.1.2 Load actions – Tải trọng
2.1.3 Combination Loading – Tổ hợp tải trọng
2.1.4 Ultimate check and Shear capacity check
2.1.5 Check for Piles cap – Kiểm toán bệ móng

2.2 Abutment A2

2.2.1 General Data – Số liệu chung.
2.2.2 Load actions – Tải trọng
2.2.3 Combination Loading – Tổ hợp tải trọng
2.2.4 Ultimate check and Shear capacity check
2.2.5 Check for Piles cap - Kiểm toán bệ móng

2.3 Pier P7

2.3.1 General Data – Số liệu chung.
2.3.2 Loads Actions – Tải trọng
2.3.3 Combination Loading – Tổ hợp tải trọng
2.3.4 Check for Pier cap – Kiểm toán xà mũ trụ
2.3.5 Check for Pier cap - Kiểm toán than trụ
2.3.6 Determinal force at top of Pile – Tính toán nội lực đầu cọc
2.3.7 Check for Piles shaft - Kiểm toán thân trụ
2.3.8 Check for Piles cap - Kiểm toán bệ móng

2.4 Pier P9

2.4.1 General Data – Số liệu chung.
2.4.2 Loads Actions – Tải trọng
2.4.3 Combination Loading– Tổ hợp tải trọng
2.4.4 Determinal force at top of Pile – Tính toán nội lực đầu cọc
2.4.5 Check for Piles shaft - Kiểm toán thân trụ
2.4.6 Check for Piles cap - Kiểm toán bệ móng

2.5 Pier P10

2.5.1 General Data – Số liệu chung.
2.5.2 Loads Actions – Tải trọng
2.5.3 Combination Loading – Tổ hợp tải trọng
2.5.4 Check for Pier cap – Kiểm toán xà mũ trụ
2.5.5 Determinal force at top of Pile – Tính toán nội lực đầu cọc
2.5.6 Check for Piles shaft - Kiểm toán thân trụ
2.5.7 Check for Piles cap - Kiểm toán bệ móng

2.6 Pier P11

2.6.1 General Data – Số liệu chung
2.6.2 Loads and Combination – Tải trọng và tổ hợp
2.6.3 Check for Piles shaft - Kiểm toán thân trụ
2.6.4 Check for Pile - Kiểm toán cọc khoan nhồi
2.6.5 Check for Piles cap - Kiểm toán bệ móng

2.7 Pier P12

2.7.1 General Data – Số liệu chung
2.7.2 Loads and Combination – Tải trọng và tổ hợp
2.7.3 Check for Piles shaft - Kiểm toán thân trụ
2.7.4 Check for Pile - Kiểm toán cọc khoan nhồi
2.7.5 Check for Piles cap - Kiểm toán bệ móng

2.8 Pier P13

2.8.1 General Data – Số liệu chung
2.8.2 Loads and Combination – Tải trọng và tổ hợp
2.8.3 Check for Piles shaft - Kiểm toán thân trụ
2.8.4 Check for Pile - Kiểm toán cọc khoan nhồi

2.1 ABUTMENT A1
2.1 MỐ CẦU A1

ABUTMENT A1 - CALCULATION SHEET

CONTENT

2.1.1. GENERAL DATA

2.1.1.1. General

- Type of Girder SUPER-T Girder
- Designed Live load: HL-93
- Design Specification: 22 TCN 272-05

2.1.1.2. Super-Structure

- Girder type		Precast	
- Girder Number	N	12.00	Girder
- Girder Length	L	38.30	m
- Efective length of girder	L_s	37.60	m
- Efective width of lanes	B	24.00	m
- The Overall Width of bridge cross section	W	26.00	m
- Lane number	n	2x3	Lane
- Pedestrian width	b_{ng}	0.00	m
- Pedestrian load	q	0.00	KN/m2
- Lane factor	m	0.85	
- Impact factor	IM	0.25	
- Number of Cross beam	n_g	20.00	Girder
- Average area of cross beam	F_{ng}	1.33	m^2
- Width of cross beam (Longitudinal)	b	0.75	m
- Area of parapet	F_{lc}	0.50	m^2
- Deck thickness	t	0.21	m
- Bearing height	h_g	0.120	m
- Total heigh of super-structure	h_{girder}	2.04	m

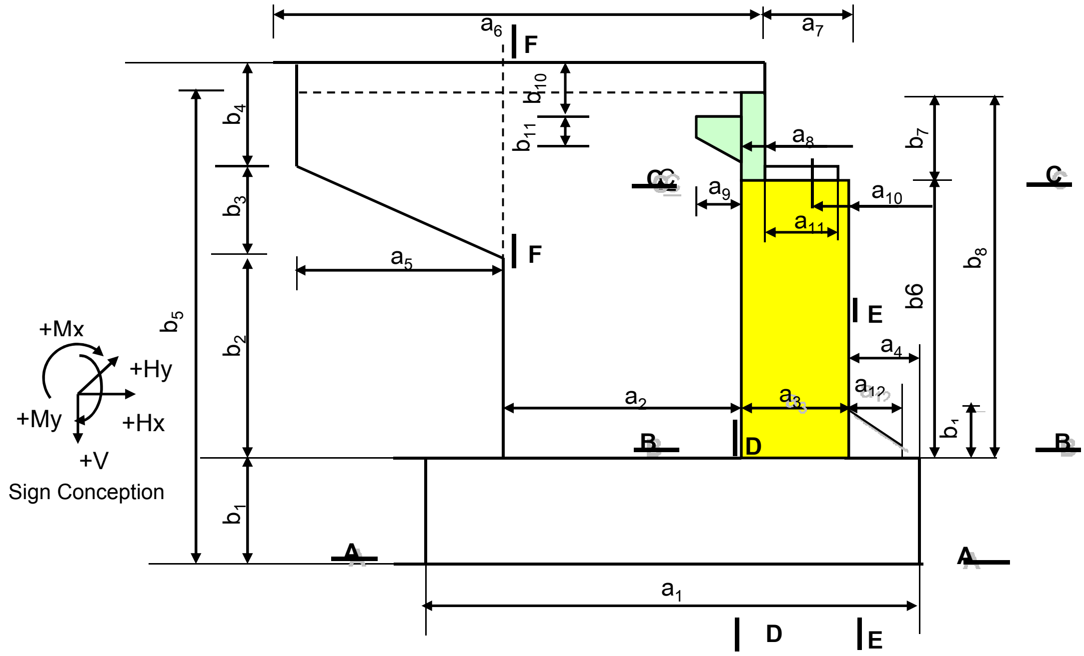
2.1.1.3. Sub-Structure

- Abutment Name A1
- Type of U-Shape
- Type of Foundation Drilled Shaft, Bored Pile, D1500
- Pile Number n_c = 12 Pile
- Anticipating Length of pile L_c = 50.00 m

2.1.1.4. Material properties:

- Concrete							
- Selfweight Concrete		= 2500	kg/m^3	γ_c	=	24.50	kN/m^3
- Compression Strength	f'_c	= 30	MPa				
- Concrete Elastic Modulus	Ec	=29440.09	kN/m^2				
- Reinforcement							
- Yield strength of rebar	f_y	= 400.00	MPa				
- Elastic Modulus	Es	= 200000	kN/m^2				
- Asphal concrete							
- Selftweight of asphalt concrete		= 2350	kg/m^3	γ_s	=	23.50	kN/m^3
- Soil properties							
- Selftweight of soil		= 1766	kg/m^3	γ_s	=	18.00	kN/m^3
- Angle of friction soil				φ_s	=	30.00	(degree)
- Angle of friction between soil and wall				δ_s	=	15.00	(degree)

GENERAL DIMENSION



Dimension accordance with Longitudinal

STT	Dimension	Notation	Value	Unit
1	Width of pile cap (longitudinal)	a_1	7.00	m
2	Width of wing wall	a_2	3.60	m
3	Thickness of body wall	a_3	1.40	m
4	Front ovehang	a_4	2.00	m
5	Width of wing wall (part 2)	a_5	2.00	m
6	Overall width of wing wall	a_6	6.00	m
7	Width of body wall for bearing	a_7	1.00	m
8	Width of head wall	a_8	0.40	m
9	Width of bracket	a_9	0.30	m
10	CL-Bearing and edge of head wall	a_{10}	0.55	m
11	Horizontal of bearing pad	a_{11}	0.80	m
12	Effective width of soil front body wall	a_{12}	0.00	m
13	Depth of pilecap	b_1	2.00	m
14	Dimension of wing wall (vertical)	b_2	4.975	m
15	Dimension of wing wall (vertical)	b_3	1.60	m
16	Dimension of wing wall (vertical)	b_4	1.00	m
17	The overall hight	b_5	9.85	m
18	Body wall height	b_6	5.50	m
19	Head wall height	b_7	2.35	m
20	Total height of head wall and body wall	b_8	7.85	m
21	Height of bearing pad	b_9	0.21	m
22	The dimension from backet to head wall	b_{10}	0.63	m
23	Hight of bracket	b_{11}	0.30	m

Dimension accordance with Transever direction

STT	Dimension	Notation	Value	Unit
1	Thichness of wing wall	c_1	0.50	m
2	Width of pile cap (transever direction)	c_2	26.00	m
3	Width of body wall	c_3	26.00	m
4	Width of bearing pad	c_4	0.80	m
5	Number of bearing pad	n_g	12.00	pad

2.1.2. LOAD ACTIONS

2.1.2.1. Dead load (DC)

$P = V.\gamma$

Inwhich:

V : Volume of structure

γ : selfweight of concrete

Dead load of Superstructure

Item	Value	Unit
Selfweight of Girder	4280.64	kN
Selfweight of Deck slab	2563.37	kN
Selfweight of Lighting	0.00	kN
Selfweight of Cross beam	243.69	kN
Selfweight of Parapet	938.35	kN
Total	**8026.06**	**kN**
Selfweight of Surface	907.25	kN

Dead load of Substructure

STT	Item	Formular	Volume (m^3)	Value (KN)
1	Pile cap	$V_{bm}= b_1*a_1*c_2$	364.0	8918.00
2	Body wall	$V_{tt}=a_3.b_6.c_3$	200.2	4904.90
3	Head wall	$V_{tđ}=a_8.b_7.c_3$	24.5	599.29
4	Bracket	$V_{mđ}=(b_{11}+a_9/2)*a_9*(c_3-2*c_1)$	3.4	82.69
5	Wing wall (back part)	$V_{tcd}=(2b_4+b_3)*a_5*c1$	3.6	88.20
6	Wing wall (front part)	$V_{tct}=2*(b_2+ b_3 + b_4)*a_2*c_1$	27.3	668.12
7	Bearing pad	$V_{đkg}=ng*(a_{11}*b_9*c_4)$	1.6	38.93
8	Wing wall (on body wall)		0.5	12.64
			Total	**15312.76**

The Moment at calculated section:

$M = P.e$

Inwhich :

P : The factor acting to calculated section

e : The eccentricity of force

Internal Force applied on section A-A

Item	Section A-A		
	P(KN)	**e**(m)	**M**(KN.m)
1. Pile cap **P1**	8918.00	0.00	0.00
2. Body wall **P2**	4904.90	0.80	3923.92
3. Head wall **P3**	599.29	0.30	179.79
4. Bracket **P4**	82.69	-0.05	-4.13
5. Wing wall (back part) **P5**	88.20	-4.35	-383.83
6. Wing wall (Front part) **P6**	668.12	-1.70	-1135.80
7. Bearing pad **P7**	38.93	1.05	40.88
8. Wing wall (on body wall) **P8**	12.64	1.00	12.64
Total	**15312.76**	**-**	**2633.46**

Internal Force applied on section B-B

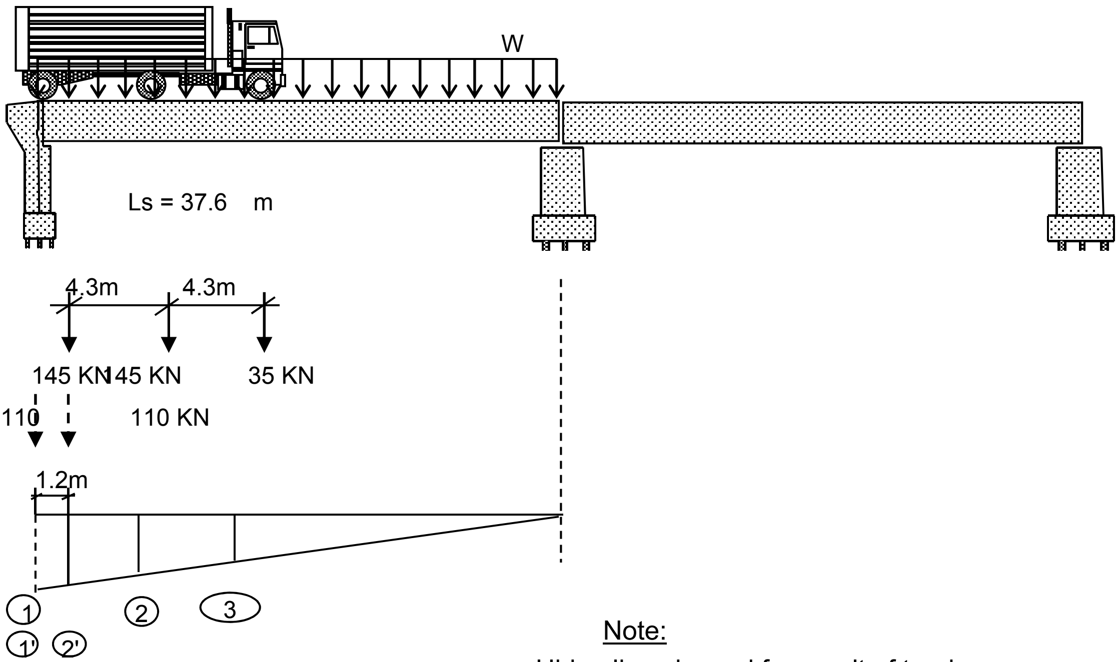
Item	Section B-B		
	P(KN)	**e**(m)	**M**(KN.m)
1. Body wall **P2**	4904.90	0.00	0.00
2. Head wall **P3**	599.29	-0.50	-299.64
3. Bracket **P4**	82.69	-0.85	-70.28
4. Bearing pad **P7**	38.93	0.25	9.73
5. Wing wall (on body wall) **P8**	12.64	0.20	2.53
Total	**5638.45**		**-357.67**

Internal Force applied on section C-C

Item	Section C-C		
	P(KN)	**e**(m)	**M**(KN.m)
1.Head wall **P3**	599.29	0.00	0.00
2.Braket **P4**	82.69	-0.35	-28.94
Total	**681.98**		**-28.94**

2.1.2.2. Live Load (LL+IM)

2.1 Live load due to vehicle (LL)



Note:

Hiden line showed for result of tandem.

Load type	Position	Influline	Load	Reaction Ri	Unit
Tandem	1'	1.00	110.0	110.00	kN
	2'	0.97	110.0	106.49	kN
Truck	3	0.77	35.0	26.99	kN
	2	0.89	145.0	128.42	kN
	1	1.00	145.0	145.00	kN
Lane load	W_L	18.80	9.3	174.84	kN
Total				**2806.81**	kN

2.2. Pedestrian load (PL)

PL - KN

2.1.2.3. Bracking load (BR)

- Braking force shall be taken as 25% of the axle weights of the design truck or tandem per lane placed in all design lanes

The force act horizontal at a 1.8m above the roadway surface = 1.8 m

Move bearing placed at abutment position, therefore:

BR = 0.00 kN

2.1.2.4. Friction force (FR)

Friction force is deteminer follow:

$$FR = f_{max} * N(KN)$$

Inwhich: f_{max} is friction factor of pot bearing (FPTE plate) = 0.06

N is reaction from deadload and live load (without Impact load) = 11357 (KN)

FR = 681.4 KN

2.1.2.5. Wind Load (WS, WL)

1. Wind load on structure (WS)

a. Transverse win load:

- Transverse win load P_D shall b taken as acting horizontally at the centroids of the appropriate areas, and shall be calculated as:

$$P_D = 0.0006*V^{2}*A_t*C_d >= 1.8*A_t \text{ (KN)}$$

Inwhich:

V Designed velocity = 53.00 (m/s)

$V = V_B*S$

V_B basic 3 second gust wind velocity with 100 year

Wind Zone	VB(m/s)
III	53

S Correction factor for upwind terrain and deck height

Height of bridge deck	S
10	1.00

A_t Area of structure for calculation of transverse wind load (m^2)

C_d drag cofficcient specified depended ratio b/d 1.19 (3.8.1.2.1.1 22 TCN 272-05)

b Overall width of bridge between outer faces of parapets = 26000 (mm)

d depth of superstructure, includeing solid parapets if applicable = 3518 (mm)

Transverse wind load WS on section A-A:

Item	ez (m)	A_t (m2)	P_D (KN)	Mx (KNm)
Abutment	6.69	37.37	74.95	501.04
Superstructure	8.67	59.06	118.45	1027.19
Total			**193.40**	**1528.24**

Transverse wind load WS on section B-B:

item	ez (m)	A_t (m2)	P_D (KN)	Mx (KNm)
Abutment	4.69	37.37	74.95	351.14
Superstructure	6.67	59.06	118.45	790.29
Total			**193.40**	**1141.44**

b. Longitudinal wind load : Not consider

2. Wind load on vehicles (WL)

a.Transverse wind load:

- Transverse wind load on vehicles shall be represented by a line load of 1.5 KN/m, acting horizontally transverse to the longitudinal centreline of the structure and above roadway: = 1.80 (m)

- Value of transverse wind load:

WL_N = 28.725 (KN)

3. Vertical wind load

- This load shall be aplied only for limit states that do not involve wind on live load.

and only when the direction of wind is taken to be perpendicular to the longitudinal axis of the bridge.

$P_V = 0.00045 * V^2 * A_V$ (KN)

Inwhich:	V	Designed velocity	=	53.00	(m/s)
	A_V	plan area of the bridge deck	=	497.90	(m^2)

Value of vertical wind load P_V :

P_V = 1244.75 (KN)

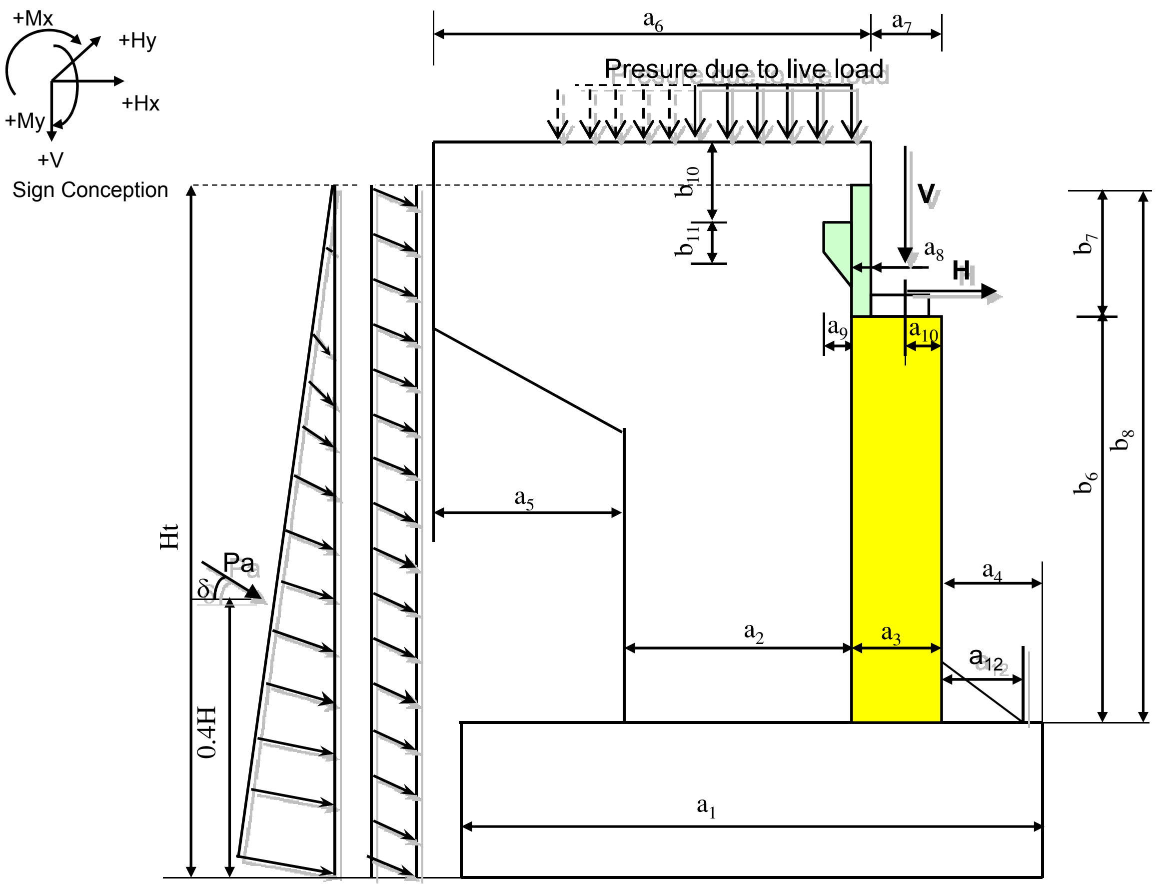
2.1.2.6. Internal force due to weight of backfill:

Height of backfill:	b_8 =	7.852	(m)
Width of backfill layer (c5=c3-2*c1):	c_5 =	25.000	(m)
Effective area of backfill (Std=c5*(a1-a3-a4)):	S_{td} =	90.00	(m^2)
Density of soil:	gd =	18.00	(KN/m^3)

Internal force due to seft weight of backfill aplied on section A-A

Item	Section A-A			
	Formular	P(KN)	e(m)	**M**(KN.m)
1. At back body wall	$P_s = b_8 * S_{td} * gd$	12721.681	-1.700	-21626.9
2.At front of body wal	$P_{tr} = b_{12} * a_{12} * c_3 * gd * 1/2$	0.000	1.500	0.000
Total		**12721.68**		**-21626.9**

2.1.2.7. Earth Pressure EH,LS



1. Earth Pressure EH (3.11.5 22 TCN 272-05)

- Formula:

$$EH=(\gamma*H^2*K*c_5)/2 \quad (KN)$$

Inwhich:

H	Total wall height		
H1	Height of pressure applied at section A-A	=	9.852 (m)
H2	Height of pressure applied at section B-B	=	7.852 (m)
H3	Height of pressure applied at section C-C	=	2.352 (m)
K	Coefficient of lateral earth pressure. For walls that deflect K =Ka		

Ka is coefficient of active pressure. (3.11.5.3)

$$Ka=\sin^2(\theta+\varphi')/(T*\sin^2(\theta)*\sin(\theta-\delta))$$

Inwhich: $T= [1+\mathrm{sqrt}(\sin(\varphi'+\delta)*\sin(\varphi'-\beta)/(\sin(\theta+\delta)+\sin(\theta+\beta)))]^2$

δ_1	Friction angle between fill and fill	=	30 (degree)
δ_2	Friction angle between fill and wall	=	15 (degree)
β	Angle of fill to horizontal	=	0.00 (degree)
θ	Angle of backfill of wall to the vertical	=	90 (degree)
φf	Effective angle of internal friction	=	30 (degree)

With: T_1 = 3.097 T_2 = 2.610

K_{a1} = 0.280 K_{a2} = 0.297

Section	Lateral earth pressure (E_H)		
	E_H(KN)	e(m)	M(KNm)
A-A	6107.55	3.941	24068.63
B-B	4127.18	3.141	12962.65
C-C	370.31	0.941	348.39

2. Live load surcharge *LS*

- A live load surchage shall be aplied where vehicular load is expected to act on the surface of the backfill within a distance equal to the wall height behind back face of he wall $h_{eq.}$

- Horizontal earth pressure maybe detemined follow below formula:

$$LS=K*h_{eq}*\gamma*H*c_5 \ (KN)$$

- Assumed to act at height of 0.5H.

In which

K = 0.280

$h_{eq:}$ Equivalent height of soil (m)

Section	Horizontal pressure due to live load surcharge (LS)			
	H(m)	h_{eq}(m)	LS(KN)	M(KNm)
A-A	9.852	0.61	756.31	1862.80
B-B	7.852	0.667	701.60	1377.24
C-C	2.352	1.416	445.89	262.18

- Vertical earth pressure (VS) due to live load surcharge to section A-A

Vaue of VS sau:

$$VS=h_{eq}*\gamma*(a_1-a_3-a_4)*c_5$$

With a1-a3-a4 is equivelent height of soil h_{eq} applied vertical load to section A-A

a_1-a_3-a_4 = 3.6 (m)

VS = 988.31 (KN)

MS = -1680.13 (KN.m)

2.1.2.8. Internal force due to earthquake - E_{AE}

Acceleration coefficient specified A = **0.034**

1. Earth pressure due to Earthquake (E_{AE})

δ_1	Friction angle between fill and fill		=	30.00 (degree)
δ_2	Friction angle between fill and wall		=	15.00 (degree)
β	Slope of wall to the vertical		=	0.00 (degree)
i	Backfill slope angle		=	0.00 (degree)
θ	Seismic intertial angle of soil	=arctan(Kh/(1-Kv)=	=	2.98 (degree)

Kv: earthquake effect factor to vertical

Kh: earthquake effect factor to horizontal

Foundation type : **1** ("1" : Drilled pile, "0" :shallow foundation)

Kh= 0.051 (dim) Kv= 0.0204 (dim)

Static earth pressure factor to section A-A K_{AS1} = 0.30

Static earth pressure factor to section B-B, C-C K_{AS2} = 0.30

Active earth pressure factor to section A-A K_{AE1} = 0.334

Active earth pressure factor to section B-B, C-C K_{AE2} = 0.334

Active earth pressure when eathquake occur

$$E_{AE} = 0.5*(1-Kv)(\gamma*H^2*Kae*c_5)\ \text{(KN)}$$

Moment due to earth pressure :

$$M_{AE} = E_{AS}*H/3+(E_{AE}-E_{AS})*0.6H$$

Section	Horizontal earth pressure (E_{AE})			
	E_{AS}(KN)	E_{AE}(KN)	H(m)	M_{AE}(KNm)
A-A	6490.67	7140.33	9.852	25155.64
B-B	4181.76	4541.92	7.852	12641.82
C-C	375.21	407.52	2.352	339.77

2. Earthquake effects to abutment (E_Q)

- At section A-A :

STT	Component	Volume (m^3)	E_Q (KN)	d (m)	M (KNm)
1	Pile cap	364.0	303.21	1.00	303.21
2	Body wall	200.2	166.77	4.75	792.14
3	Head wall	24.5	20.38	8.00	163.01
4	Bracket	3.4	2.81	9.20	25.87
5	Wing wall (back part)	3.6	3.00	8.54	25.61
6	Wing wall (front part)	27.3	22.72	6.93	157.33
7	Bearing pad	1.6	1.32	7.90	10.46
8	Wing wall (on body wall)	0.5	0.43	8.38	3.60
	Total		**520.63**		**1481.23**

- At section B-B :

STT	Component	Volume (m^3)	E_Q (KN)	d (m)	M (KNm)
1	Body wall	200.2	166.77	2.75	458.61
2	Head wall	24.5	20.38	6.68	136.03
3	Bracket	3.4	2.81	7.20	20.25
4	Wing wall (back part)	3.6	3.00	6.54	19.62
5	Wing wall (front part)	27.3	22.72	4.93	111.90
6	Bearing pad	1.6	1.32	5.65	7.48
7	Wing wall (on body wall)	0.5	0.43	6.38	2.74
	Total		**217.42**		**756.62**

- At section C-C :

STT	Component	Volume (m^3)	E_Q (KN)	d (m)	M (KNm)
1	Head wall	24.5	20.38	1.18	23.96
2	Bracket	3.4	2.81	1.70	4.78
3	Wing wall (back part)	2.0	1.67	1.18	1.96
4	Wing wall (front part)	8.5	7.05	1.18	8.29
5	Wing wall (on body wall)	0.5	0.43	0.88	0.38
	Total		**32.34**		**39.38**

2.1.3. COMBINATION LOADING

2.1.3.1. COMBINATON LOADING TO SECTION A-A

LOADING		β	ΣV	ΣHx	ΣHy	ΣMx	ΣMy
			(kN)	(kN)	(kN)	(kN•m)	(kN•m)
Superstructure & Substructure (DC)		β_{DC}	23338.8				10258.21
Wearing surface (DW)		β_{DW}	868.6				825.21
Horizontal earth pressure (EH)		β_{EH}		6107.55			24068.63
Ar-rest pressure (EV)		β_{EV}	12721.7				-21626.86
Live load (LL)		β_{LL}	2806.8				2666.47
Bracking load (BR)		β_{BR}		0.00			0.00
Pedestrian load (PL)		β_{PL}	0.0				0.00
Horizontal Live load surcharge (LS)		β_{LS}		756.31			1862.80
Horizontal Live load surcharge (VS)		β_{LS}	988.3				-1680.13
Earthquake effect to abutment		β_{EQ}		520.63	156.19	444.37	1481.23
Active earth pressure due to earthquake		β_{EQ}		6695.72			22527.46
Wind load on vehicles (WS)	Horizontal	β_{WS}			193.40	1528.2	
	Longitudinal	β_{WS}		0.00			0.00
Wind load on vehicle (WL)	Horizontal	β_{WL}			28.73	224.83	
	Longitudinal	β_{WL}		0.00			0.00
Vertical wind load (P_V)		β_{WS}	1244.8				1182.52
Friction force (FR)		β_{FR}		679.11			5306.56

COMBINATION LOADING TABLE

Max factor

Comb.	Factor β									ΣV	ΣHx	ΣHy	ΣMx	ΣMy
	β_{DC}	β_{DW}	β_{EH}	β_{EV}	β_{A}	β_{WS}	β_{WL}	β_{FR}	β_{EQ}	(kN)	(kN)	(kN)	(kN•m)	(kN•m)
Strength I	1.25	1.50	1.50	1.35	1.75	-	-	1.00	-	54292	11164.0	0.0	0.0	31260
Strength II	1.25	1.50	1.50	1.35	-	1.40	-	1.00	-	49393	9840.4	270.8	2139.5	27929
Strength III	1.25	1.50	1.50	1.35	1.35	0.40	1.00	1.00	-	53272	10861.5	28.7	224.8	30593
Service	1.00	1.00	1.00	1.00	1.00	0.30	1.00	1.00	-	41098	7543.0	28.7	224.8	22036
Extreme	1.25	1.50	1.50	1.35	0.50	-	-	1.00	1.00	49548	17434.9	156.2	444.4	26868

Min factor

Comb.	Factor β									ΣV	ΣHx	ΣHy	ΣMx	ΣMy
	β_{DC}	β_{DW}	β_{EH}	β_{EV}	β_{A}	β_{WS}	β_{WL}	β_{FR}	β_{EQ}	(kN)	(kN)	(kN)	(kN•m)	(kN•m)
Strength I	0.90	0.65	0.90	1.00	1.75	-	-	1.00	-	40933	7499.5	0.0	0.0	20096
Strength II	0.90	0.65	0.90	1.00	-	1.40	-	1.00	-	36034	6175.9	270.8	2139.5	16766
Strength III	0.90	0.65	0.90	1.00	1.35	0.40	1.00	1.00	-	39913	7196.9	28.7	224.8	19430
Service	1.00	1.00	1.00	1.00	1.00	0.30	1.00	1.00	-	41098	7543.0	28.7	224.8	22036
Extreme	0.90	0.65	0.90	1.00	0.50	-	-	1.00	1.00	36189	13770.4	156.2	444.4	16629

Note: $\beta_{LL,BR,PL,LS,VS} = \beta_{A}$

2.1.3.2. COMBINATON LOADING TO SECTION B - B

LOADING		β	ΣV (kN)	ΣHx (kN)	ΣHy (kN)	ΣMx (kN•m)	ΣMy (kN•m)
Superstructure & Substructure (DC)		β_{DC}	13664.5				846.24
Wearing surface (DW)		β_{DW}	868.64				130.30
Horizontal earth pressure (EH)		β_{EH}		4127.18			12962.65
Live load (LL)		β_{LL}	2806.8				421.02
Bracking load (BR)		β_{BR}		0.00			0.00
Pedestrian load (PL)		β_{PL}	0.00				0.00
Horizontal Live load surcharge (LS)		β_{LS}		701.60			1377.24
Earthquake effect to abutment		β_{EQ}		217.42	65.23	226.99	756.62
Active earth pressure due to earthquak		β_{EQ}		4296.27			11484.53
Wind load on vehicles (WS)	Horizontal	β_{WS}			193.40	1141.4	
	Longitudinal	β_{WS}		0.00			0.00
Wind load on vehicle (WL)	Horizontal	β_{WL}			28.73	167.38	
	Longitudinal	β_{WL}		0.00			0.00
Vertical wind load (PV)		β_{WS}	1244.75				186.71
Friction force (FR)		β_{FR}		679.11			3948.34

COMBINATION LOADING TABLE

Max factor

Comb.	Factor β								ΣV	ΣHx	ΣHy	ΣMx	ΣMy
	β_{DC}	β_{DW}	β_{EH}	β_{B}	β_{WS}	β_{WL}	β_{FR}	β_{EQ}	(kN)	(kN)	(kN)	(kN•m)	(kN•m)
Strength I	1.25	1.50	1.50	1.75	-	-	1.00	-	23295.5	8097.7	-	-	23844.2
Strength II	1.25	1.50	1.50	-	1.40	-	1.00	-	20126.3	6869.9	270.8	1598.0	20697.2
Strength III	1.25	1.50	1.50	1.35	0.40	1.00	1.00	-	22670.7	7817.0	28.7	167.4	23311.6
Service	1.00	1.00	1.00	1.00	0.30	1.00	1.00	-	17713.4	5507.9	28.7	167.4	15924.2
Extreme	1.25	1.50	1.50	0.50	-	-	1.00	1.00	19787.0	11734.4	65.2	227.0	20135.8

Min factor

Comb.	Factor β								ΣV	ΣHx	ΣHy	ΣMx	ΣMy
	β_{DC}	β_{DW}	β_{EH}	β_{B}	β_{WS}	β_{WL}	β_{FR}	β_{EQ}	(kN)	(kN)	(kN)	(kN•m)	(kN•m)
Strength I	0.90	0.65	0.90	1.75	-	-	1.00	-	17774.6	5621.4	-	-	15659.6
Strength II	0.90	0.65	0.90	-	1.40	-	1.00	-	14605.3	4393.6	270.8	1598.0	12512.7
Strength III	0.90	0.65	0.90	1.35	0.40	1.00	1.00	-	17149.8	5340.7	28.7	167.4	15127.1
Service	1.00	1.00	1.00	1.00	0.30	1.00	1.00	-	17713.4	5507.9	28.7	167.4	15924.2
Extreme	0.90	0.65	0.90	0.50	-	-	1.00	1.00	14266.1	9258.1	65.2	227.0	12838.1

Note: $\beta_{LL,BR,PL,LS} = \beta_B$

2.1.3.3. COMBINATON LOADING TO SECTION C - C

LOADING	β	ΣV (kN)	ΣHx (kN)	ΣHy (kN)	ΣMx (kN•m)	ΣMy (kN•m)
Substructure (DC)	β_{DC}	681.98				-28.94
Horizontal earth pressure (EH)	β_{EH}		370.31			348.39
Horizontal Live load surcharge (LS)	β_{LS}		445.89			262.18
Earthquake effect to abutment	β_{EQ}		32.34	9.70	11.81	39.38
Active earth pressure due to earthquake	β_{EQ}		385.48			308.66

COMBINATION LOADING TABLE

Max factor

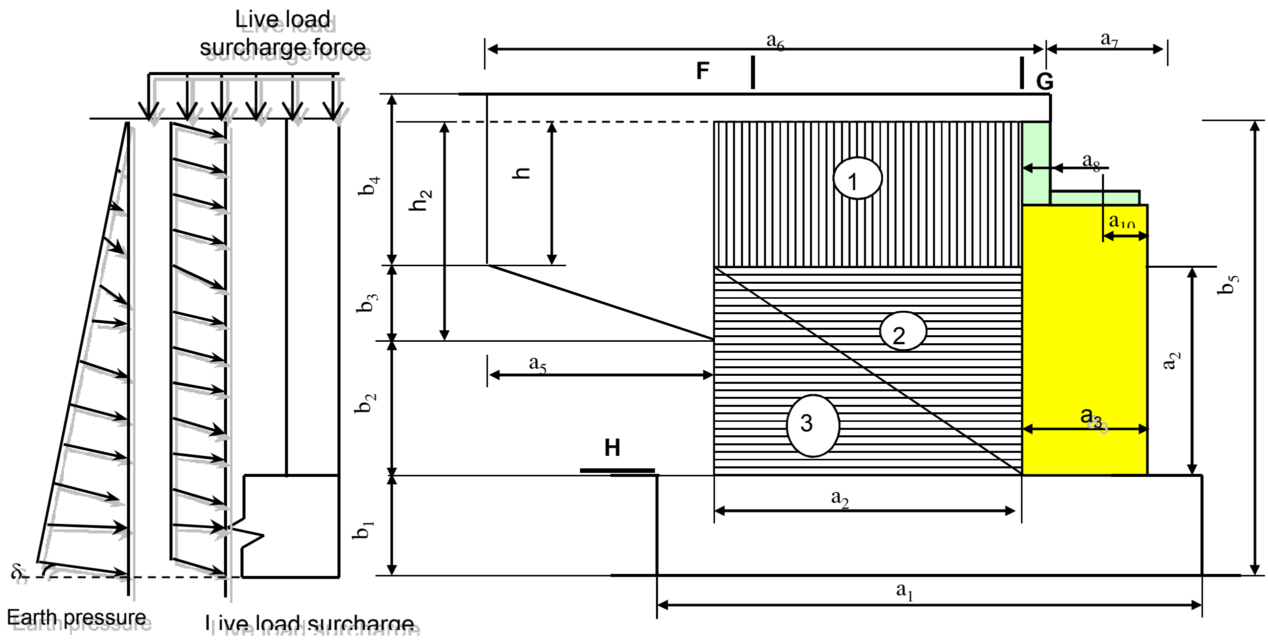
Comb.	Factor β				ΣV	ΣHx	ΣHy	ΣMx	ΣMy
	β_{DC}	β_{EH}	β_{LS}	β_{EQ}	(kN)	(kN)	(kN)	(kN•m)	(kN•m)
Strength I	1.25	1.50	1.75	-	852.47	1335.77	-	-	945.22
Strength II	1.25	1.50	-	-	852.47	555.47	-	-	486.41
Strength III	1.25	1.50	1.35	-	852.47	1157.41	-	-	840.35
Service	1.00	1.00	1.00	-	681.98	816.20	-	-	581.63
Extreme	1.25	1.50	0.50	1.00	852.47	1196.23	9.70	11.81	965.54

Min factor

Comb.	Factor β				ΣV	ΣHx	ΣHy	ΣMx	ΣMy
	β_{DC}	β_{EH}	β_{LS}	β_{EQ}	(kN)	(kN)	(kN)	(kN•m)	(kN•m)
Strength I	0.90	0.90	1.75	-	613.78	1113.58	-	-	746.32
Strength II	0.90	0.90	-	-	613.78	333.28	-	-	287.50
Strength III	0.90	0.90	1.35	-	613.78	935.23	-	-	641.45
Service	1.00	1.00	1.00	-	681.98	816.20	-	-	581.63
Extreme	0.90	0.90	0.50	1.00	613.78	974.04	9.70	11.81	766.63

2.1.3.4. Combination loading of wing wall

Diagram of wingwall analysis



Loading table to section G1

Loading	Formula	ΣQy	e	ΣMz
		kN	m	kNm
1. Horizontal earth pressure (EH)	$Qy=K_a*\gamma_đ*0.5*(b_5-b_1-a_2)*(b_5-b_1-a_2)*a_2$	163.82	1.800	294.88
2. Live load surcharge (LS)	$Qy=K_a*\gamma_đ*h_{eq}*a2*(b_5-b_1-a_2)$	78.32	1.800	140.97

Notice:

h_{eq} = 1.016 The height of converted earth pressure corelative with it's retaining wall = 4.252 (m)

Combination loading table to section G1

With max factor

Combination	Factor β		ΣQy	ΣMz
	β_{EH}	β_{LS}	kN	kNm
Strength I	1.50	1.75	382.79	689.01
Strength II	1.50	0.00	245.73	442.31
Strength III	1.50	1.35	351.46	632.62
Service	1.00	1.00	242.14	435.85

With min factor

Combination	Factor β		ΣQy	ΣMz
	β_{EH}	β_{LS}	kN	kNm
Strength I	0.9	1.75	284.49	512.09
Strength II	0.9	0.00	147.44	265.39
Strength III	0.9	1.35	253.17	455.70
Service	1.00	1.00	242.14	435.85

Loading table to section G2

Loading	Formula	ΣQy	e	ΣMz
		kN	m	kNm
1. Horizontal earth pressure (EH)	$Qy=K_a*a_2*a_2*\gamma_đ*(b_5-b_1-2*a_2/3)*0.5$	177.84	1.20	213.41
2. Live load surcharge (LS)	$Qy=K_a*gđ*heq*a_2*a_2/2$	24.71	1.20	29.65

Notice:

h_{eq} = 0.757 The height of converted earth pressure corelative with it's retaining wall = 6.052 (m)

Combination loading table to section G2

With max factor

Combination	Factor β		ΣQy	ΣMz
	β_{EH}	β_{LS}	kN	kNm
Strength I	1.50	1.75	310.00	372.00
Strength II	1.50	0.00	266.77	320.12
Strength III	1.50	1.35	300.12	360.14
Service	1.00	1.00	202.55	243.06

With min factor

Combination	Factor β		ΣQy	ΣMz
	β_{EH}	β_{LS}	kN	kNm
Strength I	0.9	1.75	203.30	243.95
Strength II	0.9	-	160.06	192.07
Strength III	0.9	1.35	193.41	232.10
Service	1.00	1.00	202.55	243.06

COMBINATON LOADING TO SECTION F - F

LOAD	Formula	ΣQy	e	ΣMz
		(kN)	(m)	(kN)
1. Earth pressure (EH)	$Qy=0.5*K_a*\gamma_đ*h_{tb}*a_5*(h_1+h_2)$	22.79	0.87	19.86
2. Live load surcharge (LS)	$Qy=0.5*K_a*\gamma đ*heq*a5*(h1+h2)$	35.55	0.87	30.98

Inwhich:

h1 = b5-b1-b2-b3 = 1.3 (m)

h2 = b5-b1-b2 = 2.9 (m)

h_{eq} = 1.7 Height of backfill aquivalent with height of wall h2/2 = 1.44 (m)

h_{tb} = 1.090 Height of centre of backpart of wingwall

Combination loading table to section F-F

With max factor

Comb.	Factor β		ΣQy	ΣMz
	β_{EH}	β_{LS}	(kN)	(kN•m)
Strength I	1.50	1.75	96.40	84.02
Strength II	1.50	-	34.18	29.80
Strength III	1.50	1.35	82.18	71.63
Service	1.00	1.00	58.34	50.85
Extreme	1.50	0.50	51.96	45.29

With min factor

Comb.	Factor β		ΣQy	ΣMz
	β_{EH}	β_{LS}	(kN)	(kN•m)
Strength I	0.90	1.75	82.72	72.10
Strength II	0.90	-	20.51	17.88
Strength III	0.90	1.35	68.50	59.71
Service	1.00	1.00	58.34	50.85
Extreme	0.90	0.50	38.29	33.37

Loading table to section H3

Loading	Formula	ΣQy	e	ΣMz
		kN	m	kN
1. Horizontal earth pressure (EH)	$Qy=K_a*a_2*a_2*g_đ*(b_5-b_1-a_2/3)*0.5$	216.99	1.20	260.39
2. Live load surcharge (LS)	$Qy=K_a*gđ*heq*a_2*a_2/2$	21.77	1.20	26.12

Notice:

h_{eq} = 0.667 The height of converted earth pressure corelative with it's retaining wall = 7.852 (m)

Combination loading table to section H3

With max factor

Combination	Factor β		ΣQy	ΣMz
	β_{EH}	β_{LS}	kN	kNm
Strength I	1.50	1.75	363.58	436.30
Strength II	1.50	0.00	325.48	390.58
Strength III	1.50	1.35	354.87	425.85
Service	1.00	1.00	238.76	286.51

With min factor

Combination	Factor β		ΣQy	ΣMz
	β_{EH}	β_{LS}	kN	kNm
Strength I	0.9	1.75	233.39	280.06
Strength II	0.9	0.00	195.29	234.35
Strength III	0.9	1.35	224.68	269.62
Service	1.00	1.00	238.76	286.51

2.1.4. ULTIMATE CHECK AND SHEAR CAPACITY CHECK

2.1.4.1. CHECK FOR BODY SHAFT (SECTION B-B)

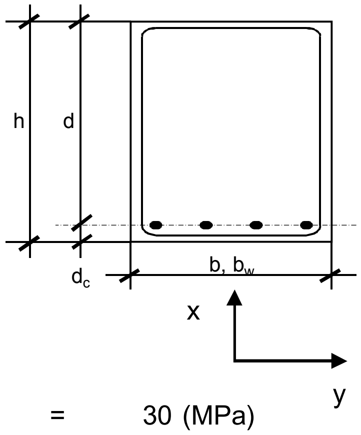
Combination loading to section B-B

Combination	ΣV	ΣHx	ΣHy	ΣMx	ΣMy
	kN	kN	kN	kN.m	kN.m
Strength I	23295.5	8097.7	0.0	0.0	23844.2
Strength II	20126.3	6869.9	270.8	1598.0	20697.2
Strength III	22670.7	7817.0	28.7	167.4	23311.6
Service	17713.4	5507.9	28.7	167.4	15924.2
Extreme	19787.0	11734.4	65.2	227.0	20135.8

The dimensions of calculated section.

- Effective Width of Section $b, b_w =$ 26000 mm
- Total Depth of Section $h =$ 1400 mm

1. Check Biaxial flexure



Used combination loading: Strength I

- Axial loading used in calculation: **N** = 23296 (kN)

If the fatored axial load is not less than 0,1.φ.f'c.Ag:

Mux/Mrx+Muy/Mry<=1 (1-a)

If the fatored axial load is not less than 0,1.φ.f'c.Ag:

1/Prxy = 1/Prx+1/Pry-1/φ.Po (1-b)

- Check for condition 0,1.φ.f'c.Ag:

φ = 0.75

f'c Compression Strength of concrete at 28 days = 30 (MPa)

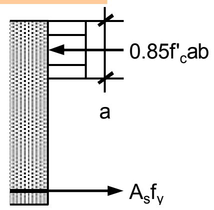
Ag Gros area of section\ = 36.40 (m^2)

Value: 0,1.φ.f'c.Ag = 81900 (kN)

- Compared N = 23296 < 0,1.φ.f'c.Ag = 81900 Check follow formula (1-a)
- Mrx x axial Flexure capacity (N.mm)

$Mrx = \varphi.As_x.fy.(ds-a/2)$

- Mry y axial Flexure capacity (N.mm)



$Mrx = \varphi.As_y.fy.(d_s-a/2)$

φ resistance factor for member in flexure = 0.90

A_s Reinforcement area

Longitudinal direction = 164 - D25 → A_s = 80503 mm^2

Horizontal direction = 6 - D14 → A_s = 924 mm^2

d_c Effective Cover to Steel Centroid

d_c = 75.0 mm (Longitudinal)

= 75.0 mm (Horizontal)

d_s Depth from to Steel Centroid

Longitudinal direction d_s = 1313 mm

Horizontal direction d_s = 25918 mm

$a = c\beta_1$ Depth of the equivalent stress block

β_1 stress block factor β_1 = 0.84

Longitudinal direction a = 49 mm

Horizontal direction a = 10 mm

Factored flexural resistance:

Longitudinal direction Mrx = 8618 kNm

Horizontal direction Mry = 38038 kNm

- Mux Factored flexural in direction of the x axial = 0.0 kNm
- Muy Factored flexural in direction of the y axial = 23844 kNm
- Detemined slendernes effects K.Lu/r

r The radius of gyration $rx = \sqrt{\frac{Ix}{A}}$ = 7.51 m $ry = \sqrt{\frac{Iy}{A}}$ = 0.4 m

A The gross concrete section = 36.40 m^2

I Moment of inertia I_x = 2050.53 m^4 I_Y = 5.95 m^4

k Effective length factor = 1.00

Lu Length (no support) of Abument shaft = 5.50

Horizontal slenderness ratio K.Lu/r = 1 < 22 Ignore slenderness effect

Longitudinal slenderness ratio K.Lu/r = 14 < 22 Ignore slenderness effect

- Consider for vertical slenderness effect:

$$\delta_b = \frac{C_m}{1 - \frac{Pu}{\varphi.Pe}} \geq 1$$

φ Compressive resistance factor = 0.75

Cm = 1

Pu Factored compressive = 23296 kN

Pe Vertical flexurel force Ơle, Pe = π^2.EI/(KLu)^2

E Concrete modulus 2.94E+07 kN/m2

I Moment of inertia = 5.95 m4

EI = 175031133.35 kNm2

Pe = 57049155.78 kN

=> δ_b = 1.001

- Moment increased effect by slendeness:

$M_{ux}^{tt} = \delta_b.M_{ux}$ = 0.00 kNm

$M_{uy}^{tt} = \delta_b.M_{uy}$ = 23857 kNm

- Check follow condition (1-a):

Mux	Muy	Mry	Mrx	Mux/Mrx	Muy/Mry	Mux/Mrx+Muy/Mry	Conclusion
kNm	kNm	kNm	kNm				
0.00	23857	38037.81	8617.89	0.000	0.627	0.6	OK

- Reinforcement ratio ρ = 0.24 %
- Check of minimum reinforcement ratio ρ ≥ 0.03f'c/fy = 0.23 % OK.
- Check of maximum reinforcement ratio 0.42 ≥ c/de = 0.044 OK.
- Check minimum longitudinal reinforcement:

As fy >(Mu/dv φ + 0.5*Nu/φ) + (Vu/φ - 0.5*Vs)*cotgθ OK.

As fy= **32201** kN

(Mu/dv φ + 0.5*Nu/φ) + (Vu/φ - 0.5*Vs)*cotgθ = **17905** kN

2. Checking shear capacity

Datas		Long.	Trans.	Unit
• Shear force	V_u	11734	271	kN
• Resistance factor	φ	0.90	0.90	
• The shear depth of structure	d_v	1313	25918	mm
• Effective web width taken as the minimum web width within the depth d	b_v	25918	1313	mm
• Angle of inclination of diagonal compressive stresses	θ	45	45	độ
• Angle of inclination of transverse reinforcement to long. Axis	α	90	90	độ
• Factor indicating ability of diagonally cracked concrete to transmit tensi	β	2	2	
•Value	$0.1*f'_c*b_v*d_v$	102052	102052	kN

• The spacing of stirrups	s	600	1200	mm
• Diameter of shear reinforcement	D	D12	D14	mm
• No. of shear reinforcement within a distance s	n	43	2	
• Total area of shear reinforcement	A_v	4863	308	mm^2
• Nominal shear resistance provided by tensile stresses in the concrete	V_c	30929	30929	kN
•Shear resistance provided by shear reinforcement	V_s	4255	2660	kN
• Value	$0.25 \cdot f'_c \cdot b_v \cdot d_v$	244287	255130	kN
• Nominal shear resistance	V_n	35185	33589	kN
• The factor shear resistance	V_r	31666	30230	kN
• Checking	$V_r > V_u$	**OK**	**OK**	

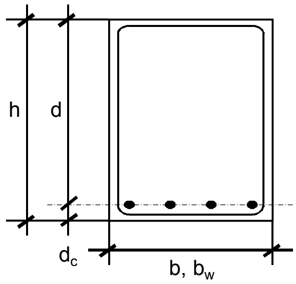
3. Check crack

Datas		**Long.**	**Trans.**	**Unit**
• Interior force combination	**Service**			
• Factored moment	M_u	15924	167	kNm
• Hight of Section	h	1400	26000	mm
• Width of section	b	26000	1400	mm
• Effective Cover to Steel Centroid	d_c	75	75	mm
• Distance from compressive reinf. to extreme Tension fiber	d_e	1325	25925	mm
• Tension reinforcement:	D	25	14	mm
- Number of Bar	n	164	6	Thanh
- Total area of reinf.	A_s	80503	924	mm^2
• Ratio of reinf. Modulus with concrete modulus	$n = E_s/E_c$	= 7.00	= 7.00	
• The distance from extreme fiber to the neutral axis	c	= 262.31	= 493.98	%
• Effective moment of inertia	J	7.93E+11	4.24E+12	
• Arm	de-c	1062.69	25431.02	
• Tension stress in reinforcement	fs =n*Ms*(de-c)/J	= 149.41	7.03	Mpa
• Crack width parameter	Z	23000	23000	N/mm
• Area of concrete having the same centroid as the principal tensile reinforcement divided by number of bars	A	15854	23333	mm^2
• Tension stress in reinforcement with service state	f_{sa}	230.7	209.1	Mpa
• Check condition	$f_s \le Min(0.6 \cdot f_y, f_{sa})$	**OK**	**OK**	

2.1.4.2. CHECKING HEADWALL (SECTION C-C)

Combination loading to section C-C

Combination	ΣV	ΣHx	ΣHy	ΣMx	ΣMy
	kN	kN	kN	kN.m	kN.m
Strength I	852.5	1335.8	0.0	0.0	945.2
Strength II	852.5	555.5	0.0	0.0	486.4
Strength III	852.5	1157.4	0.0	0.0	840.4
Service	682.0	816.2	0.0	0.0	581.6
Extreme	852.5	1196.2	9.7	11.8	965.5

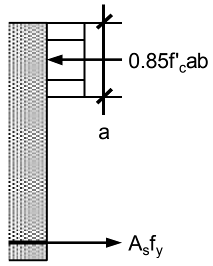


Data

- Effective Width of Section — b, b_w = 26000 mm
- Total Depth of Section — h = 400 mm
- Depth from to Steel Centroid — d = 342 mm
- Effective Cover to Steel Centroid — d_c = 50 mm

2.1. Check Flexural capacity

Factored combination loading: Extreme



-Resistance factor — ϕ = 0.90

-Reinforcement area = 170 - D16 — A_s = 34181 mm^2

-Reinforcement ratio — ρ = 0.33 %

-Stress block factor — β_1 = 0.84

-Depth of the equivalent stress block — a = 21 mm

Flexural resistance **M_r = 4208.3** kNm

Factored moment **M_u = 965.5** kNm

-Check condition — Mu ≤ Mr = 4208.3 **OK.**

-Check for minimum reinf. Ratio — ρ ≥ 0.03f'c/fy = 0.23 % **OK.**

-Check for maximum reinf. Ratio — 0.42 ≥ c/de = 0.072 **OK.**

2.2. Shear capacity check

-Resistance factor: — ϕ_v = 0.90

- Transverse reinforcement area: = 2 - D12 — A_v = 226.19 mm^2

-Spacing of Transverse reinforcement: — s = 200 mm

-Factor indicating ability of diagonally cracked concrete to transmit tension (5.8.3.4 22 TCN 272-05) — β = 2

- Angle of inclination of diagonal compressive stresses (5.8.3.4) — θ = 45 °

- The shear depth of structure: dv= max(de-0.5*a,0.9*dc,0.72*h) — dv = 332 mm

-The shear resistance of concrete: (5.8.2.4 22 TCN 272-05) — V_c = 7841 kN

-The shear resistance of stirrup, (5.8.3.3) — V_s = 150 kN

Vn = 0,25.fc.bv.dv = 64679 kN

Shear resistance **V_r = 7192** kN

Factored shear **V_u = 1336** kN

Check condition Vu ≤ Vr = **7192** kN **O.K.**

2.3. Check crack

- Interior force combination — **Service**
- Factored moment — Ms = 5.82E+05 KN.mm
- Modulus of rupture of concrete — fr = 0.63*sqrt(f'c) = 3.45 MPa
- Distance from extreme tension fiber to the neutral axis — y_t = h-c = 375 mm
- Stress of concrete at tension fiber — f'r = Ms*y_t/I_g = 1.57 MPa
- **Check condition**

Stress of concrete f'r<0.8*fr shouldn't be controled of cracking by distribution of reinforcement by condition

2.1.4.3. Check for Wing wall

Item		Section G1	Section G2	Section F-F	Section H3	Unit
• Factored Plexural moment	M_u	689.01	372.00	84.02	436.30	kN.m
• Factored Shear force	V_u	382.79	310.00	96.40	363.58	kN
• Hight of Section	h	500	500	500	500	mm
• Width of section	b	4252	3600	4154	3600	mm
• Section area	A_c	2126000	1800000	2077000	1800000	mm^2
• Moment of inertia of concrete section	I_g	4.4E+10	3.8E+10	4.3E+10	3.8E+10	mm^4
• Tension reinforcement: Distance from tension reinf. to extreme compression fiber	d_c	80	80	80	60	mm
Reinf. Diameter	∅	D 20	D 20	D 20	D 20	mm
Space	@	150	150	150	150	mm
Number of bar	n	28	23	17	25	bar
Total area of reinf.	A_s	8905	7121	5341	7854	mm^2
• comp. reinforcement: Distance from compressive reinf. to extreme		58	124	124	58	mm
Diameter		D 16	D 16	D 16	D 16	mm
Reinf. Space		150	150	150	150	mm
Number of bar		28	23	17	25	bar
Total area of reinf.	A'_s	5699	4557	3418	5027	mm^2
Check Flexural Moment						
• Resistance factor	Φ	0.90	0.90	0.90	0.90	
• The corresponding effective	d_e	420	420	420	440	mm
• Stress block factor	β_1	0.84	0.84	0.84	0.84	
• Depth of the equivalent stress block = c*β1	a	32.85	31.03	20.17	34.22	mm
• Distance from extreme compression fiber to the neutral axis	c	39.31	37.13	24.13	40.95	mm
• The nominal flexural resistance:	M_n	1438	1152	876	1329	kN.m
• Factored flexural resistance	$M_r = \Phi.M_n$	1294	1037	788	1196	kN.m
• **Check condition**	$M_r > M_u$	**O.K**	**O.K**	**O.K**	**O.K**	
Mimimum Reinforcement						
• Ratio of tension steel to gross area	$\rho = A_s/(b.d)$	0.50	0.47	0.31	0.50	%
• **Check**	$\rho > 0.03 \cdot f'_c/f'_y$	**O.K**	**O.K**	**O.K**	**O.K**	0.23
• Cracking moment	1.2Mcr	733.61	621.12	716.70	621.12	KN.m
• **Check**	Mr> min(1.2Mcr, 1.33Mu)	**O.K**	**O.K**	**O.K**	**O.K**	
Maximum Reinforcement						
• Obligation Condition	c/d_e	0.09	0.09	0.06	0.09	
• **Check**	$c/d_e < 0.42$	**O.K**	**O.K**	**O.K**	**O.K**	
Check shear resistance						
• Factored Shear force	V_u	382.79	310.00	96.40	363.58	kN
• Resistance factor	Φ	0.90	0.90	0.90	0.90	
• The effective shear deepth	d_v	404	404	410	423	mm
• Effective width	b_v	4252	3600	4154	3600	mm
• Angle of inclination of diagonal compressive stress	θ	45	45	45	45	degree
• Angle of inclination of transerverse reinf. To longitudinal axis	α	90	90	90	90	degree
• Factor indicating ability of diagonally cracked concrete to	β	2	2	2	2	
• Value	$0.1 \cdot f'_c \cdot b_v \cdot d_v$	5148	4368	5108	4567	kN
• Max spacing of transverse reinforcement	s	323	324	328	338	mm
• Spacing of stirrup	s	300	300	300	300	mm
• Diameter of transverse reinforcement	∅	D 22	D 22	D 22	D 22	
• Number of transverse reinf. within distance s	n	2	2	2	2	bar
• Total area of transverse reinf.	A_v	760	760	760	760	mm^2
• Diameter of stirrup	∅	D 12	D 12	D 12	D 12	mm
• Number of stirrup within distance s	n	9	7	9	7	bar
• Total area of stirrup	A_v	993.25	829.38	968.62	829.38	
• The shear resistance of concrete:	V_c	1560.22	1323.96	1548.21	1384.20	kN
• The shear resistance of stirrup	V_s	238.33	199.46	236.07	208.53	kN
• Value	$0.25 \cdot f'_c \cdot b_v \cdot d_v$	12869.95	10921.12	12770.94	11418.00	kN
• The nominal shear resistance:	V_n	1798.54	1523.42	1784.28	1592.73	kN
• The factored shear resistance	V_r	1618.69	1371.08	1605.86	1433.46	kN
• **Check**	$V_r > V_u$	**O.K**	**O.K**	**O.K**	**O.K**	
•**Requiring transverse reinforcement**	$V_u > 0.5 \cdot \Phi \cdot V_c$	Not required	Not required	Not required	Not required	
•**Check minimum transverse reinforcement**	$A_v > 0.083 \cdot (f'_c{}^{0.5}) \cdot b_v \cdot s/f_y$	Not required	Not required	Not required	Not required	
Check crack						
Interior force combination	**Service**					
• Factored moment	M_u	4.36E+02	2.43E+02	4.53E+01	2.87E+02	kN.m
• Modulus of rupture of concrete	fr = 0.63*sqrt(f'c)	3.45	3.45	3.45	3.45	MPa
• Distance from extreme tension fiber to the neutral axis	$y_t = h-c$	461	463	476	459	mm
• Stress of concrete at tension fiber	$f'r = Ms \cdot y_t/I_g$	5	3	0	4	MPa
• **Check**	f'r >	0.8*fr	0.8*fr	0.8*fr	0.8*fr	

			check	check	No check	check	
• Crack width parameter		Z	= 23000	= 23000	= 23000	= 23000	N/mm
• Ratio of reinf. Modulus with concrete modulus		$n = E_s/E_c$	= 7.00	= 7.00	= 7.00	= 7.00	
• The distance from extreme fiber to the neutral axis		c	= 126.60	= 122.58	= 96.41	= 132.20	mm
• Effective moment of inertia		J	8.24E+09	6.62E+09	5.16E+09	7.98E+09	mm4
• Arm		de-c	= 293.40	= 297.42	= 323.59	= 307.80	mm
• Tension stress in reinforcement		fs =n*Ms*(de-c)/J	= 108.61	= 76.45	= 19.90	= 77.35	MPa
• Area of concrete having the same centroid as the principal tensile r	A		= 15000	= 15882	= 24435	= 14400	mm2
• Tension stress in reinforcement with service state		$f_{sa} = Z/(d_c*A)^{1/3}$	= 238.22	= 233.73	= 202.46	= 241.48	Mpa
• Check condition		$f_s < f_{sa}$	**O.K**	**O.K**	**O.K**	**O.K**	
• Check condition		$f_s < 0.6*f_y$	**O.K**	**O.K**	**O.K**	**O.K**	

2.1.5.1. INTERNAL FORCE AT TOP OF PILE

1. SUMMARY OF EXTERNAL FORCE ACTING BOTTOM FOOTING

Combination Type	V	Hx	Hy	Mx	My
	(KN)	(Kn)	(Kn)	(Kn.m)	(Kn.m)
Strength I	54292	11164	0	0	31260
Strength II	49393	9840	271	2140	27929
Strength III	53272	10861	29	225	30593
Service I	41098	7543	29	225	22036
Extreme	49548	17435	156	444	26868

2. Piling material:

Concrete 30 Mpa

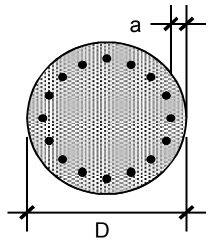
E_b (kg/cm^2)	294401
γ_b (T/m^3)	2.5

Steel bar

Type	CB-400-T
E_t (kg/cm^2)	200000

3. Piling dimension

+ Diameter **D** = 1.50 m
a = 0.100 m
+ Length **L** = 50.00 m



4. Maximum Internal force at top piling

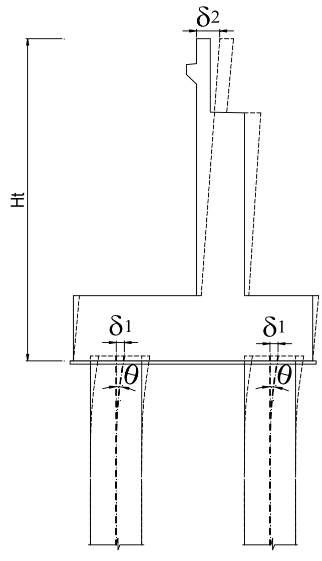
Internal force and diplacements *(Result follow Piling software)*

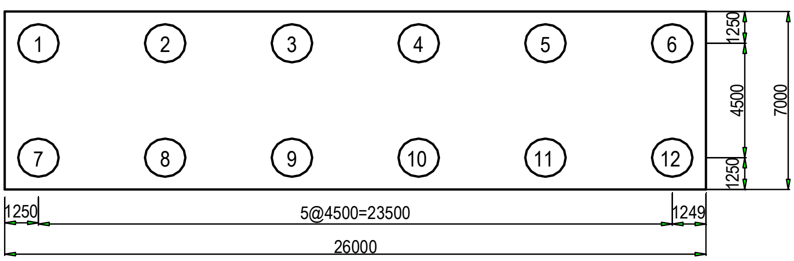
Combination Type	V	H	M	x	y	z
	(KN)	(KN)	(KN.m)	(m)	(m)	(rad)
Strength I	6966.91	930.50	2890.79	-	-	-
Strength II	6329.46	820.00	2542.90	-	-	-
Strength III	6826.27	905.08	2809.57	-	-	-
Service I	5109.13	628.58	1941.73	**0.011**	**-0.000**	**0.002**
Extreme	7264.23	1452.92	4778.66	-	-	-

- Check displacement of top pile not exceed 38mm (10.7.2.7) **OK**

- Displacement of top abutment

+ Displacement of foundation $\delta_1 =$ 10.84 (mm)
+ Rotation of foundation $\theta =$ -0.001 (deg)
+ Displacement at top of abutment $\delta_2 =$ 18.01 (mm)
+ Total height of abutment $H_t =$ 9.852 (m)





Arrangement of pile

- Result for internal force at top of each pile

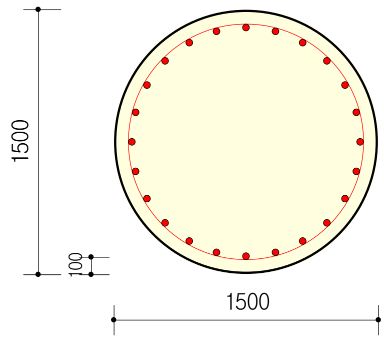
Combination	Pile Number	Hx	Hy	N	Mx	My
		(KN)	(KN)	(KN.m)	(m)	(m)
Strength I	1	930.50	0.00	2081.76	0.00	2890.79
	2	930.50	0.00	2081.76	0.00	2890.79
	3	930.50	0.00	2081.76	0.00	2890.79
	4	930.50	0.00	2081.76	0.00	2890.79
	5	930.50	0.00	2081.76	0.00	2890.79
	6	930.50	0.00	2081.76	0.00	2890.79
	7	930.50	0.00	6966.91	0.00	2890.79
	8	930.50	0.00	6966.91	0.00	2890.79
	9	930.50	0.00	6966.91	0.00	2890.79
	10	930.50	0.00	6966.91	0.00	2890.79
	11	930.50	0.00	6966.91	0.00	2890.79
	12	930.50	0.00	6966.91	0.00	2890.79
Strength II	1	820.00	-22.58	2000.29	89.19	2542.90
	2	820.00	-22.58	1980.77	89.19	2542.90
	3	820.00	-22.58	1961.26	89.19	2542.90
	4	820.00	-22.58	1941.74	89.19	2542.90
	5	820.00	-22.58	1922.23	89.19	2542.90
	6	820.00	-22.58	1902.71	89.19	2542.90
	7	820.00	-22.58	6329.46	89.19	2542.90
	8	820.00	-22.58	6309.94	89.19	2542.90
	9	820.00	-22.58	6290.42	89.19	2542.90
	10	820.00	-22.58	6270.91	89.19	2542.90
	11	820.00	-22.58	6251.39	89.19	2542.90
	12	820.00	-22.58	6231.88	89.19	2542.90
Strength III	1	905.08	-2.42	2062.72	9.55	2809.57
	2	905.08	-2.42	2060.66	9.55	2809.57
	3	905.08	-2.42	2058.59	9.55	2809.57
	4	905.08	-2.42	2056.53	9.55	2809.57
	5	905.08	-2.42	2054.47	9.55	2809.57
	6	905.08	-2.42	2052.40	9.55	2809.57
	7	905.08	-2.42	6826.27	9.55	2809.57
	8	905.08	-2.42	6824.20	9.55	2809.57
	9	905.08	-2.42	6822.14	9.55	2809.57
	10	905.08	-2.42	6820.07	9.55	2809.57
	11	905.08	-2.42	6818.01	9.55	2809.57
	12	905.08	-2.42	6815.94	9.55	2809.57
Service	1	628.58	-2.42	1750.86	9.55	1941.73
	2	628.58	-2.42	1748.79	9.55	1941.73
	3	628.58	-2.42	1746.73	9.55	1941.73
	4	628.58	-2.42	1744.66	9.55	1941.73
	5	628.58	-2.42	1742.60	9.55	1941.73
	6	628.58	-2.42	1740.53	9.55	1941.73
	7	628.58	-2.42	5109.13	9.55	1941.73
	8	628.58	-2.42	5107.07	9.55	1941.73
	9	628.58	-2.42	5105.01	9.55	1941.73
	10	628.58	-2.42	5102.94	9.55	1941.73
	11	628.58	-2.42	5100.88	9.55	1941.73
	12	628.58	-2.42	5098.81	9.55	1941.73
Extreme event	1	1452.92	-13.00	1026.31	52.21	4778.66
	2	1452.92	-13.00	1019.80	52.21	4778.66
	3	1452.92	-13.00	1013.30	52.21	4778.66
	4	1452.92	-13.00	1006.79	52.21	4778.66
	5	1452.92	-13.00	1000.28	52.21	4778.66
	6	1452.92	-13.00	993.77	52.21	4778.66
	7	1452.92	-13.00	7264.23	52.21	4778.66
	8	1452.92	-13.00	7257.72	52.21	4778.66
	9	1452.92	-13.00	7251.21	52.21	4778.66
	10	1452.92	-13.00	7244.71	52.21	4778.66
	11	1452.92	-13.00	7238.20	52.21	4778.66
	12	1452.92	-13.00	7231.69	52.21	4778.66

2.1.5.2. Check for pile:

Item		Mark	Unit	Value
• Factored Axial force		Nu	Kn	6966.91
• Factored Plexural moment		Mux	Kn.m	89.19
• Factored Plexural moment		Muy	Kn.m	2542.90
• Diameter of Pier shaft		D	m	1.50
• Section area		Ag	m2	1.77
• Moment of inertia of concrete section		Ic	m4	0.25
• Cover thickness		a	m	0.075
• Reinf. Diameter		Ds	mm	D32
• Number of rebar		n_s	nos	24
• Rebar area		As	mm2	19301.95
Check minimum reinforcement				
• Minimum rebar area required (0.135*f'c/fy)*Ag		As req	mm2	
• Check condition As > (0.135*f'c/fy)*Ag				**OK**
Check maximum reinforcement				
• Maximum rebar area 0.08*Ag		As max	mm2	141371.7
• Check condition As < 0.08*Ag				**OK**
Check ratio spiral or Tier (5.7.4.6)				
• Distance to outside of Spairal or Ties to concrete face			mm	68.00
• Effect diamete		Deff	m	1.36
• Area of core measured to the outside diameter of the spiral			m2	1.46
• Ratio spiral Rebar required		ρsa		0.00707
Required Area of Spiral Rebar	space		mm	75
	Effective length			1.36
	layer			1
	Area			180.7
	Requaired Dhs			15.2
Actuaral	Effective length	d	m	1.364
	Diameter	Dhr	mm	16
	Area of Rebar	Ah	mm2	201.1
	layer	Nl	nos	1
	Total area of spiral	Ac	m2	201.062
	space	s	mm	75
	Ratio spiral Rebar	ρs	-	0.00786166
• Check condition		$\rho_s > \rho_{sa}$		OK

	Company	Nipponkoei	**Project Name**	DN Qn Expressway
	Designer	vinh	**File Name**	D:\...\Pile D1.5m - A1.B01

1. Geometry and Materials



Design Code : ACI318M-02

Stress Profile : Equivalent Stress Block

Material Data : f_c' = 30 MPa (β_1 = 0.850)

f_y = 400, f_{ys} = 400 MPa

Section Dim. : Φ1500 mm

Effective Len. : KL_u = 15000 mm

Steel Distribut.: 24 - D32 (d_c = 100 mm)

Total Steel Area A_{st} = 19061 mm^2 (ρ_{st} = 0.0108)

2. Member Force and Moment

Unit : kN, kN-m

L.C.	P_u	M_{ux}	M_{uy}	R_{atioV}	V_{ux}	V_{uy}	R_{atioH}	Remark
1	6966.9	0.0	2890.8	0.539	930.5	0.0	0.195	
2	6329.5	89.2	2542.9	0.475	820.0	22.6	0.172	
3	6826.3	9.6	2809.6	0.524	905.1	2.4	0.190	
4	5109.1	9.6	1941.7	0.361	628.6	2.4	0.133	
5	7264.2	52.2	4778.7	0.817	1452.9	13.0	0.305	

3. Magnified Moment

KL_u/r_x = 15000/375 = 40.00 > 34-12(M_1/M_2) = 22.00

δ_x = MAX[1.00/(1-P_u/0.75/52256), 1.0] = 1.228

KL_u/r_y = 15000/375 = 40.00 > 34-12(M_1/M_2) = 22.00

δ_y = MAX[1.00/(1-P_u/0.75/52256), 1.0] = 1.228

4. Design Force and Moment

Design Load Combination No : 5

P_u = 7264.2 kN

M_{ux} = 52.2, M_{uy} = 4778.7 kN-m

$\delta_x M_{ux}$= δ_x*MAX[M_{ux}, $P_u e_{min}$] = 535.0 kN-m

$\delta_y M_{uy}$= δ_y*M_{uy}, = 5865.9 kN-m

5. Check Axial and Moment Capacity

Rotation Angle and Depth to the Neutral Axis θ = -5.21°, c = 566 mm

Strength Reduction Factor Φ = 0.8550

Maximum Axial Load $\Phi P_{n(max)}$ = 27144.3 kN

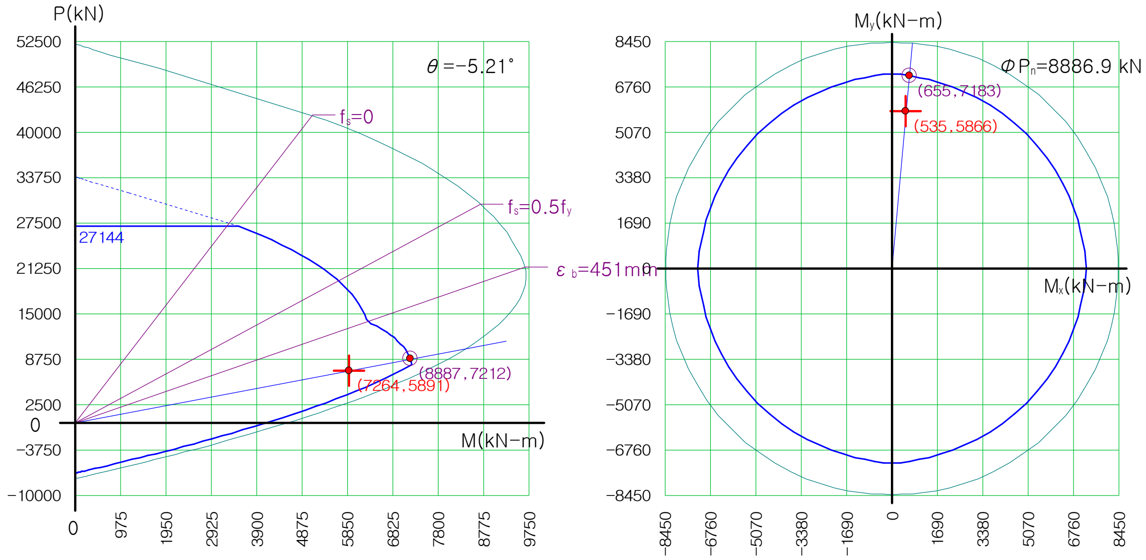
Design Axial Load Strength ΦP_n = 8886.9 kN

Design Moment Strength ΦM_{nx} = 655.1 kN-m

ΦM_{ny} = 7182.6 kN-m

Strength Ratio : Applied/Design = 0.817 < 1.000 .... O.K

	Company	Nipponkoei	Project Name	DN Qn Expressway
	Designer	vinh	File Name	D:\...\Pile D1.5m - A1.BOI



6. Check Shear Capacity

Design Load Combination No : 5

Strength Reduction Factor Φ= 0.750

Design Force Vu = 1453.0 kN (Pu = 7264.2 kN)

Required Hoop Spacing : D16 @ 508 mm

Provided Hoop Spacing : D16 @ 150 mm (Tie)

$\Phi V_c + \Phi V_s$ = 3847.0 + 924.5 = 4771.5 kN > V_u = 1453.0 kN .... O.K

2.1.5.3. COMBINATON LOADING TO SECTION D - D

Loading	Formular	Volume	ΣVz	e	ΣMy	β
		(m3)	(KN)	(m)	(KN.m)	
1. Pile cap	$V_{bm}= b_1*(a_1-a_3-a_4)*c_2$	0.0	0.00	-1.80	0.00	β_{DC}
2. Bracket	$V_{mđ} =(b_{11}+a_9/2)*a_9*(c_3-2*c_1)$	3.4	82.69	0.15	12.40	β_{DC}
3. Wing wall (back part)	$V_{tcd}=(2b_4+b_3)*a_5*c1$	3.6	88.20	-4.45	-392.65	β_{DC}
4. Wing wall (front part)	$V_{tct}=2*(b_2+ b_3 + b_4)*a_2*c_1$	27.3	668.12	-1.80	-1202.61	β_{DC}
5. Backfill	$V_s=b_8*S_{tđ}$	706.7	12721.68	-1.80	-22899.03	β_{EV}
6. Vertical earth pressure (LS)	$V_{ls}=heq*c_5*(a_1-a_3-a_4)$	54.9	988.31	-1.80	-1778.96	β_{LS}
Total			**14549.00**		**-26260.84**	

COMBINATION LOADING TABLE

Max factor

Comb.	Factor β			Internal force at top pile		Shear force	Moment
				ΣNi	ΣMi	Qz	My
	β_{DC}	β_{EV}	β_{LS}	(kN)	(kN•m)	(kN)	(kN•m)
Strength I	1.25	1.35	1.75	12490.57	29352.83	7462.00	-6652.61
Strength II	1.25	1.35	-	11709.00	27516.16	6514.02	-5376.10
Strength III	1.25	1.35	1.35	12345.37	29011.62	7211.87	-6282.24
Service	1.00	1.00	1.00	10474.16	24614.28	4074.83	-1646.56
Extreme	1.25	1.35	0.50	6060.25	14241.58	12656.93	-19540.16

Min factor

Comb.	Factor β			Internal force at top pile		Shear force	Moment
				ΣNi	ΣMi	Qz	My
	β_{DC}	β_{EV}	β_{LS}	(kN)	(kN•m)	(kN)	(kN•m)
Strength I	0.90	1.00	1.75	12490.57	29352.83	2715.76	1916.05
Strength II	0.90	1.00	-	11709.00	27516.16	1767.78	3192.56
Strength III	0.90	1.00	1.35	12345.37	29011.62	2465.64	2286.42
Service	1.00	1.00	1.00	10474.16	24614.28	4074.83	-1646.56
Extreme	0.90	1.00	0.50	6060.25	14241.58	7910.69	-10971.50

2.1.5.4. COMBINATON LOADING TO SECTION E - E

Component	Formular	Volume (m3)	ΣVz (KN)	e (m)	ΣMy (KN.m)
1. Pile cap	$V_{bm}= b_1*a_4*c_2$	0.00	0.00	1.00	0.00
2. Due to Backfill (front of abutment):	$V_s=b_{12}*a_{12}*c_3/2$	0.00	0.00	0.00	0.00
Total			**0.00**		**0.00**

COMBINATION LOADING TABLE

Max factor

Comb.	Factor β		Internal force at top pile		Shear force	Moment
			ΣNi	ΣMi	Qz	My
	β_{DC}	β_{EV}	(kN)	(kN•m)	(kN)	(kN•m)
Strength I	1.25	1.35	41801.44	31351.08	-41801.44	31351.08
Strength II	1.25	1.35	37684.00	28263.00	-37684.00	28263.00
Strength III	1.25	1.35	40926.63	30694.97	-40926.63	30694.97
Service	1.00	1.00	30623.84	22967.88	-30623.84	22967.88
Extreme	1.25	1.35	43487.75	32615.81	-43487.75	32615.81

Min factor

Comb.	Factor β		Internal force at top pile		Shear force	Moment
			ΣNi	ΣMi	Qz	My
	β_{DC}	β_{EV}	(kN)	(kN•m)	(kN)	(kN•m)
Strength I	0.90	1.00	41801.44	31351.08	-41801.44	31351.08
Strength II	0.90	1.00	37684.00	28263.00	-37684.00	28263.00
Strength III	0.90	1.00	40926.63	30694.97	-40926.63	30694.97
Service	1.00	1.00	30623.84	22967.88	-30623.84	22967.88
Extreme	0.90	1.00	43487.75	32615.81	-43487.75	32615.81

2.1.5.5. Ultimate check and shear capacity check for pile cap :

Item			Section D - D	Section E-E	Unit
• Factored Plexural moment		M_u	6652.61	31351.08	kN.m
• Factored Shear force		V_u	7462.00	41801.44	kN
• Hight of Section		h	2000	2000	mm
• Width of section		b	26000	26000	mm
• Section area		A_c	52000000	52000000	mm^2
• Moment of inertia of concrete section		I_g	1.7E+13	1.7E+13	mm^4
• Tension reinforcement:	Distance from tension reinf. to extreme compression fiber	d_c	100	164	mm
	Reinf. Diameter	∅	22	28	mm
	Space	@	150	150	mm
	Number of bar	n	173	173	bar
	Total area of reinf.	A_s	65636	106525	mm^2
• comp. reinforcement:	Distance from compressive reinf. to extreme Tension fiber		100	139	mm
	Diameter		28	22	mm
	Reinf. Space		150	150	mm
	Number of bar		172	172	bar
	Total area of reinf.	A'_s	105909	65383	mm^2
Check Flexural Moment at Strength state					
• Resistance factor		Φ	0.90	0.90	
• The corresponding effective		d_e	1900	1836	mm
• Stress block factor		β_1	0.8357	0.84	
• Depth of the equivalent stress block = $c*\beta 1$		a	39.60	64.27	mm
• Distance from extreme compression fiber to the neutral axis		c	47.38	76.90	mm
• The nominal flexural resistance:		M_n	49364	76863	kN.m
• Factored flexural resistance		$M_r = \Phi.M_n$	44427	69177	kN.m
• **Check condition**		$\mathbf{M_r > M_u}$	**O.K**	**O.K**	
Mimimum Reinforcement					
• Cracking moment		1.2Mcr	71773.56	60485.30	Kn.m
• **Check**		Mr> min(1.2Mcr, 1.33Mu)	**O.K**	**O.K**	
Maximum Reinforcement					
• Obligation Condition		c/d_e	0.02	0.04	
• **Check**		$c/d_e < 0.42$	**O.K**	**O.K**	
Check shear resistance					
• Factored Shear force		V_u	7462.00	41801.44	kN
• Resistance factor		Φ	0.90	0.90	
• The effective shear deepth		d_v	1880	1804	mm
• Effective width		b_v	26000	26000	mm
• Angle of inclination of diagonal compressive stress		θ	33	41	degree
• Angle of inclination of transerverse reinf. To longitudinal axis		α	90	90	degree
• Factor indicating ability of diagonally cracked concrete to transmit tension		β	2.40	1.95	
• Value		$0.1*f'_c*b_v*d_v$	146656	140702	kN
• Max spacing of transverse reinforcement		s	600	600	mm
• Spacing of stirrup		s	450	450	mm
• Diameter of transverse reinforcement		∅	D 32	D 32	
• Number of transverse reinf. within distance s		n	2	2	bar
• Assume		θ	33.00	41.00	degree
• Strain in tensile reinforcement		ε_x	7.07E-04	1.94E-03	
If ex<0, multiple with reduce factor		Φc	-	-	
• Ratio of shear stress and f'c		V/f'c	0.01	0.03	
• β final			2.40	1.95	
• θ final			33.00	41.00	degree
• Total area of transverse reinf.		A_v	1608	1608	mm^2
• Diameter of stirrup		∅	D 18	D 18	mm
• Number of stirrup within distance s		n	58	58	bar
• Total area of stirrup		A_v	14787.48	14787.48	
• The shear resistance of concrete:		V_c	53336.86	41576.78	kN

• The shear resistance of stirrup	V_s	22948.58	13760.59	kN
• Value	$0.25*f'_c \cdot b_v \cdot d_v$	366639.04	351753.82	kN
• The nominal shear resistance:	V_n	76285.44	55337.38	kN
• The factored shear resistance	V_r	68656.89	49803.64	kN
• Check	$V_r > V_u$	**O.K**	**O.K**	
•Requiring transverse reinforcement	$V_u > 0.5 \cdot \Phi \cdot V_c$	**Not required**	**Need**	
•Check minimum transverse reinforcement	$A_v > 0.083 \cdot (f'_c{}^{0.5}) \cdot b_v \cdot s/f_y$	**not required**	**O.K**	
Check Flexural and shear resistance at Extreme state				
• Factored Plexural moment	M_u	19540.16	32615.81	kN.m
• Factored Shear force	V_u	12656.93	43487.75	kN
• Resistance factor	Φ	1.00	1.00	
• The nominal flexural resistance:	M_n	49364	76863	kN.m
• Factored flexural resistance	$M_r = \Phi.M_n$	49364	76863	kN.m
• Check condition	$M_r > M_u$	**O.K**	**O.K**	
Check crack				
Interior force combination				
• Factored moment	M_u	1.65E+03	2.30E+04	kN.m
• Modulus of rupture of concrete	fr = 0.63*sqrt(f'c)	3.45	3.45	MPa
• Distance from extreme tension fiber to the neutral axis	$y_t = h-c$	1953	1923	mm
• Stress of concrete at tension fiber	$f'r = Ms*y_t/I_g$	0	3	MPa
• Check	f'r >	0.8*fr	0.8*fr	
		No check	No check	
• Crack width parameter	Z	= 23000	= 23000	N/mm
• Ratio of reinf. Modulus with concrete modulus	$n = E_s/E_c$	= 7.00	= 7.00	
• The distance from extreme fiber to the neutral axis	c	= 277.41	= 354.46	mm
• Effective moment of inertia	J	1.39E+12	2.02E+12	mm^4
• Arm	de-c	= 1622.59	= 1481.54	mm
• Tension stress in reinforcement	fs =n*Ms*(de-c)/J	= 13.41	= 117.76	MPa
• Area of concrete having the same centroid as the principal	A	= 15058	= 15029	mm2
• Tension stress in reinforcement with service state	$f_{sa} = Z/(d_c*A)^{1/3}$	= 236.61	= 233.00	Mpa
• Check condition	$f_s < f_{sa}$	**O.K**	**O.K**	
• Check condition	$f_s < 0.6*f_y$	**O.K**	**O.K**	

2.2 ABUTMENT A2
2.2 MỐ CẦU A2

CONTENT

2.2.1. GENERAL DATA

2.2.1.1. General

- Type of Girder	Box Girder
- Designed Live load:	HL-93
- Design Specification:	22 TCN 272-05

2.2.1.2. Super-Structure

- Girder type		Precast	
- Girder Number	N	2.00	Girder
- Girder Length	L	65.00	m
- Efective length of girder	L_s	64.35	m
- Efective width of lanes	B	24.00	m
- The Overall Width of bridge cross section	W	26.00	m
- Lane number	n	2x3	Lane
- Pedestrian width	b_{ng}	0.00	m
- Pedestrian load	q	0.00	KN/m2
- Lane factor	m	0.85	
- Impact factor	IM	0.25	
- Number of Cross beam	n_g	0.00	Girder
- Average area of cross beam	F_{ng}	0.00	m^2
- Width of cross beam (Longitudinal)	b	0.00	m
- Area of parapet	F_{lc}	0.50	m^2
- Deck thickness	t	0.00	m
- Bearing height	h_g	0.373	m
- Total heigh of super-structure	h_{girder}	2.61	m

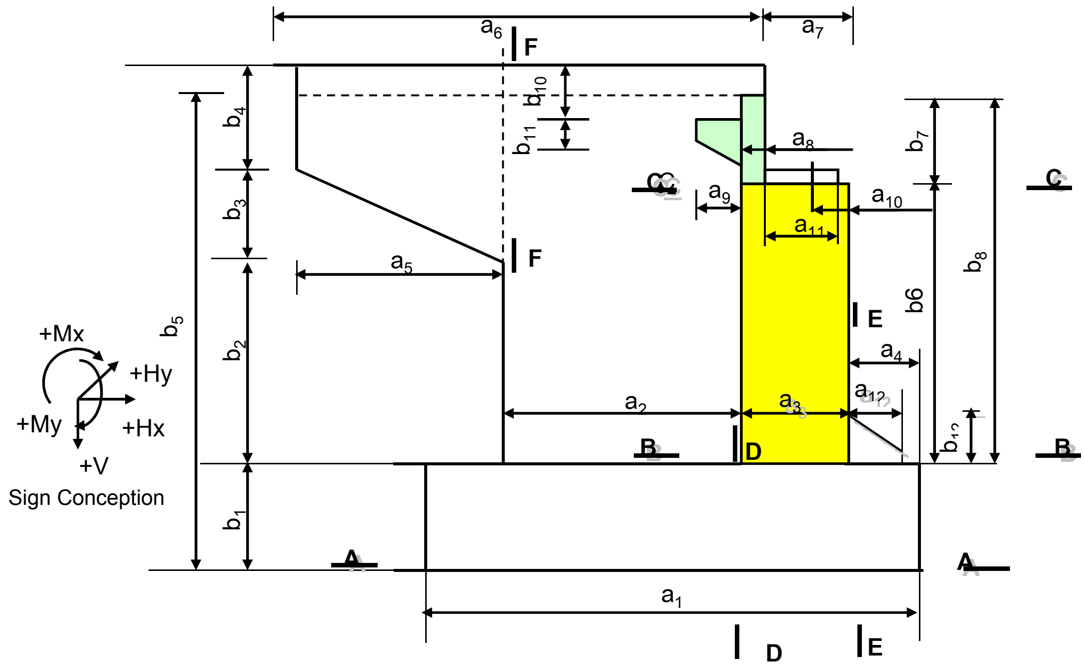
2.2.1.3. Sub-Structure

- Abutment Name	A2			
- Type of	U-Shape			
- Type of Foundation	Drilled Shaft, Bored Pile, D1500			
- Pile Number	n_c	=	18	Pile
- Anticipating Length of pile	L_c	=	65.00	m

2.2.1.4. Material properties:

- Concrete								
- Selfweight Concrete		=	2500	kg/m^3	γ_c	=	24.50	kN/m^3
- Compression Strength	f'_c	=	30	MPa				
- Concrete Elastic Modulus	Ec	=	29440.09	kN/m^2				
- Reinforcement								
- Yield strength of rebar	f_y	=	400.00	MPa				
- Elastic Modulus	Es	=	200000	kN/m^2				
- Asphal concrete								
- Selftweight of asphalt concrete		=	2350	kg/m^3	γ_s	=	23.50	kN/m^3
- Soil propeties								
- Selftweight of soil		=	1766	kg/m^3	γ_s	=	18.00	kN/m^3
- Angle of friction soil					φ_s	=	30.00	(degree)
- Angle of friction between soil and wall					δ_s	=	15.00	(degree)

GENERAL DIMENSION



Dimension accordance with Longitudinal

STT	Dimension	Notation	Value	Unit
1	Width of pile cap (longitudinal)	a_1	11.50	m
2	Width of wing wall	a_2	6.00	m
3	Thickness of body wall	a_3	2.00	m
4	Front ovehang	a_4	3.50	m
5	Width of wing wall (part 2)	a_5	3.50	m
6	Overall width of wing wall	a_6	10.00	m
7	Width of body wall for bearing	a_7	1.50	m
8	Width of head wall	a_8	0.50	m
9	Width of bracket	a_9	0.30	m
10	CL-Bearing and edge of head wall	a_{10}	0.65	m
11	Horizontal of bearing pad	a_{11}	1.10	m
12	Effective width of soil front body wall	a_{12}	2.00	m
13	Depth of pilecap	b_1	2.50	m
14	Dimension of wing wall (vertical)	b_2	7.76	m
15	Dimension of wing wall (vertical)	b_3	2.80	m
16	Dimension of wing wall (vertical)	b_4	1.00	m
17	The overall hight	b_5	14.45	m
18	Body wall height	b_6	9.00	m
19	Head wall height	b_7	2.95	m
20	Total height of head wall and body wall	b_8	11.95	m
21	Height of bearing pad	b_9	0.19	m
22	The dimension from backet to head wall	b_{10}	0.63	m
23	Hight of bracket	b_{11}	0.30	m

Dimension accordance with Transever direction

STT	Dimension	Notation	Value	Unit
1	Thichness of wing wall	c_1	0.80	m
2	Width of pile cap (transever direction)	c_2	26.00	m
3	Width of body wall	c_3	26.00	m
4	Width of bearing pad	c_4	1.10	m
5	Number of bearing pad	n_g	4.00	pad

2.2.2. LOAD ACTIONS

2.2.2.1. Dead load (DC)

$P = V.\gamma$

Inwhich:

V : Volume of structure

γ : selfweight of concrete

Dead load of Superstructure

Item	Value	Unit
Selfweight of Girder	9666.08	kN
Selfweight of Deck slab	0.00	kN
Selfweight of Lighting	0.00	kN
Selfweight of Cross beam	0.00	kN
Selfweight of Parapet	0.00	kN
Total	**9666.08**	**kN**
Selfweight of Surface	802.16	kN

Dead load of Substructure

STT	Item	Formular	Volume (m^3)	Value (KN)
1	Pile cap	$V_{bm} = b_1*a_1*c_2$	747.5	18313.75
2	Body wall	$V_{tt} = a_3.b_6.c_3$	468.0	11466.00
3	Head wall	$V_{tđ} = a_8.b_7.c_3$	38.4	939.58
4	Bracket	$V_{mđ} = (b_{11}+a_9/2)*a_9*(c_3-2*c_1)$	3.3	80.70
5	Wing wall (back part)	$V_{tcd} = (2b_4+b_3)*a_5*c1$	13.4	329.28
6	Wing wall (front part)	$V_{tct} = 2*(b_2+b_3+b_4)*a_2*c_1$	110.9	2718.21
7	Bearing pad	$V_{đkg} = ng*(a_{11}*b_9*c_4)$	0.9	22.53
8	Wing wall (on body wall)		1.0	24.14
			Total	**33894.19**

The Moment at calculated section:

$M = P.e$

Inwhich :

P : The factor acting to calculated section

e : The eccentricity of force

Internal Force applied on section A-A

Item	Section A-A		
	P(KN)	**e**(m)	**M**(KN.m)
1. Pile cap **P1**	18313.75	0.00	0.00
2. Body wall **P2**	11466.00	1.25	14332.50
3. Head wall **P3**	939.58	0.50	469.79
4. Bracket **P4**	80.70	0.10	8.07
5. Wing wall (back part) **P5**	329.28	-7.16	-2357.55
6. Wing wall (Front part) **P6**	2718.21	-2.75	-7475.07
7. Bearing pad **P7**	22.53	1.40	31.54
8. Wing wall (on body wall) **P8**	24.14	1.50	36.22
Total	**33894.19**	**-**	**5045.50**

Internal Force applied on section B-B

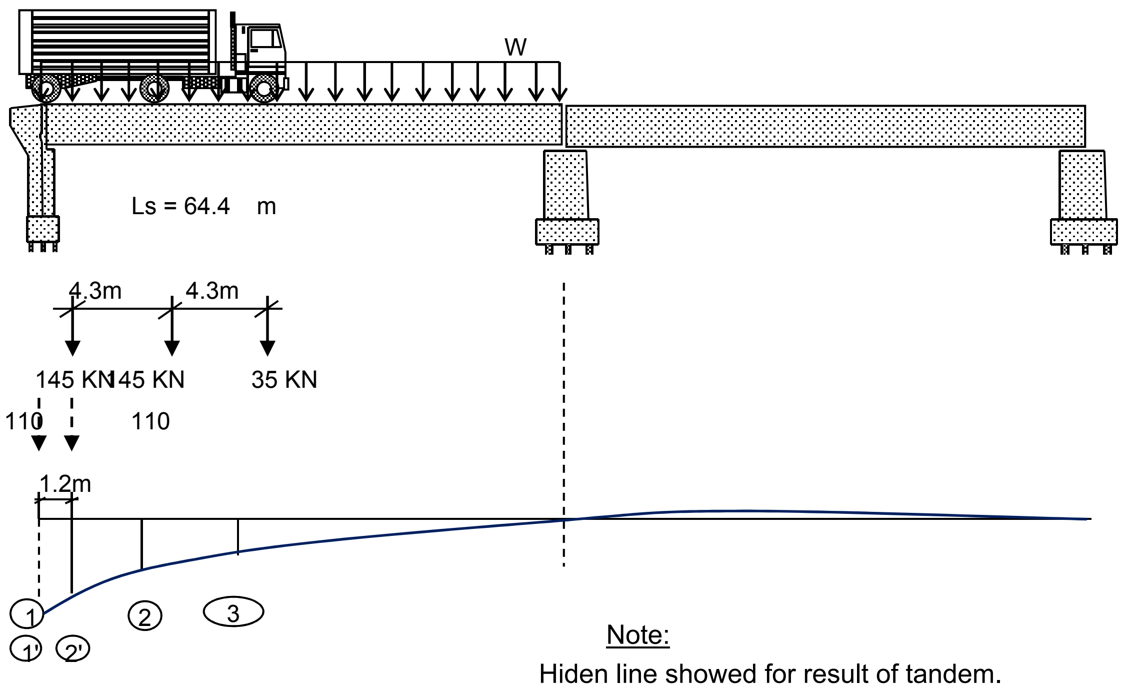
Item	Section B-B		
	P(KN)	**e**(m)	**M**(KN.m)
1. Body wall **P2**	11466.00	0.00	0.00
2. Head wall **P3**	939.58	-0.75	-704.68
3. Bracket **P4**	80.70	-1.15	-92.81
4. Bearing pad **P7**	22.53	0.15	3.38
5. Wing wall (on body wall) **P8**	24.14	0.25	6.04
Total	**12532.95**		**-788.07**

Internal Force applied on section C-C

Item	Section C-C		
	P(KN)	**e**(m)	**M**(KN.m)
1.Head wall **P3**	939.58	0.00	0.00
2.Braket **P4**	80.70	-0.40	-32.28
Total	**1020.28**		**-32.28**

2.2.2.2. Live Load (LL+IM)

1. Live load due to vehicle (LL)



Note:

Hiden line showed for result of tandem.

Load type	Position	Tung độ đường Influline	Load	Reaction Ri	Unit
Tandem	1'	1.00	110.0	110.00	kN
	2'	0.97	110.0	106.70	kN
Truck	3	0.77	35.0	26.83	kN
	2	0.88	145.0	127.92	kN
	1	1.00	145.0	145.00	kN
Lane load	W_L	22.60	9.3	210.15	kN
Total				**4784.38**	kN

2.2.2.3. Bracking load (BR)

- Braking force shall be taken as 25% of the axle weights of the design truck or tandem per lane placed in all design lanes

The force act horizontal at a 1.8m above the roadway surface = 1.8 m

Move bearing placed at abutment position, therefore:

BR = 0.00 kN

2.2.2.4. Friction force (FR)

Friction force is deteminer follow:

$FR = f_{max} * N(KN)$

Inwhich: f_{max} is friction factor of pot bearing (FPTE plate) = 0.06

N is reaction from deadload and live load (without Impact load) = 11768 (KN)

FR = 706.1 KN

2.2.2.5. Wind Load (WS, WL)

1. Wind load on structure (WS)

a. Transverse win load:

- Transverse win load P_D shall b taken as acting horizontally at the centroids of the appropriate areas, and shall be calculated as:

$$P_D = 0.0006*V^2*A_t*C_d >= 1.8*A_t \text{ (KN)}$$

Inwhich:

V Designed velocity = 53.00 (m/s)

$V = V_B*S$

V_B basic 3 second gust wind velocity with 100 year

Wind Zone	VB(m/s)
III	53

S Correction factor for upwind terrain and deck height

Height of bridge deck	S
10	1.00

A_t Area of structure for calculation of transverse wind load (m^2)

C_d drag cofficcient specified depended ratio b/d 1.19 (3.8.1.2.1.1 22 TCN 272-05)

b Overall width of bridge between outer faces of parapets = 26000 (mm)

d depth of superstructure, includeing solid parapets if applicable = 3654 (mm)

Transverse wind load WS on section A-A:

Item	ez (m)	A_t (m2)	P_D (KN)	Mx (KNm)
Abutment	9.05	91.34	183.20	1658.31
Superstructure	13.23	118.76	238.18	3150.86
Total			**421.38**	**4809.17**

Transverse wind load WS on section B-B:

item	ez (m)	A_t (m2)	P_D (KN)	Mx (KNm)
Abutment	6.55	91.34	183.20	1200.31
Superstructure	10.73	118.76	238.18	2555.41
Total			**421.38**	**3755.73**

2. Wind load on vehicles (WL)

a.Transverse wind load:

- Transverse wind load on vehicles shall be represented by a line load of 1.5 KN/m, acting horizontally transverse to the longitudinal centreline of the structure and above roadway: = 1.80 (m)

- Value of transverse wind load:

WL_N = 48.75 (KN)

3. Vertical wind load

- This load shall be aplied only for limit states that do not involve wind on live load.

and only when the direction of wind is taken to be perpendicular to the longitudinal axis of the bridge.

$P_V = 0.00045*V^2*A_V$ (KN)

Inwhich:	V	Designed velocity	=	53.00	(m/s)
	A_V	plan area of the bridge deck	=	845.00	(m^2)

Value of vertical wind load P_V :

P_V = 2112.51 (KN)

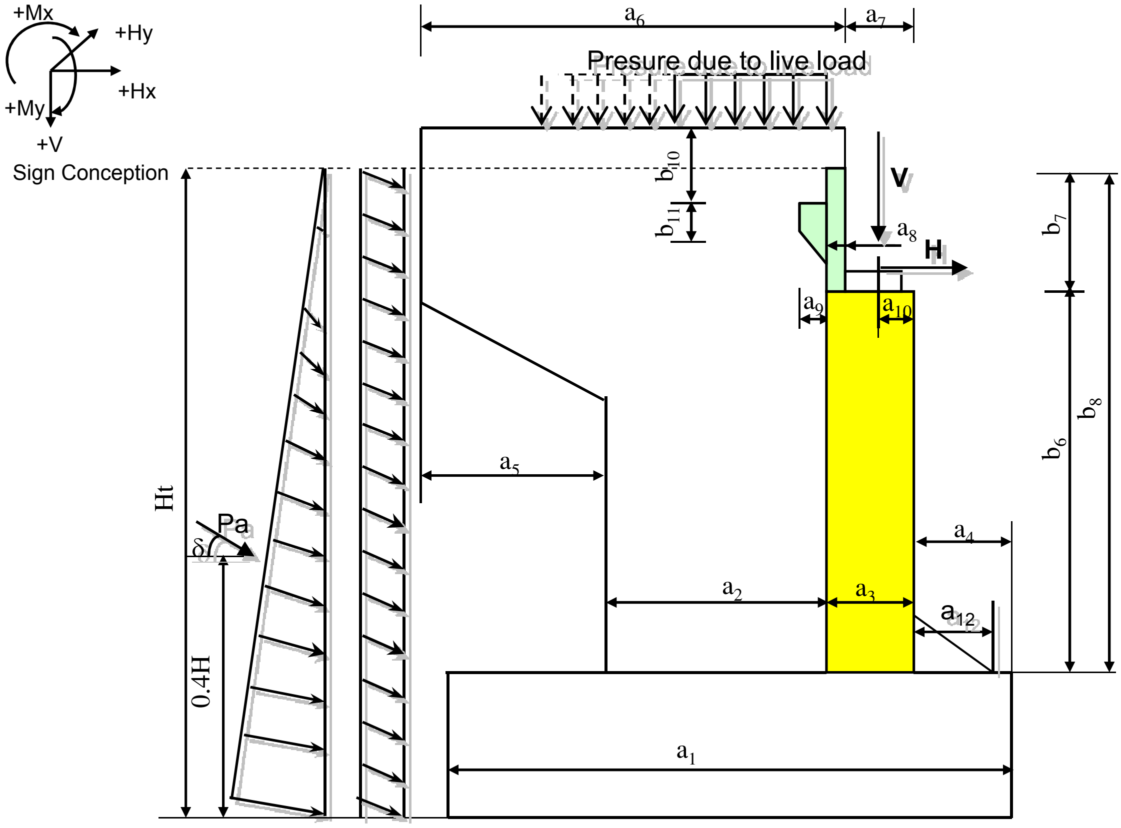
2.2.2.6. Internal force due to weight of backfill

Height of backfill:	b_8 =	11.950	(m)
Width of backfill layer (c5=c3-2*c1):	c_5 =	24.400	(m)
Effective area of backfill (Stđ=c5*(a1-a3-a4)):	$S_{tđ}$ =	146.40	(m^2)
Density of soil:	$g_đ$ =	18.00	(KN/m^3)
Height of backfill (front of abutment):	b_{12} =	0.500	(m)
Width of backfill (front of abutment):	a_{12} =	2.000	(m)

Internal force due to seft weight of backfill aplied on section A-A

Item	Section A-A			
	Formular	P(KN)	e(m)	**M**(KN.m)
1. At back body wall	$P_s=b_8*S_{tđ}*g_đ$	31494.207	-2.750	-86609.1
2.At front of body wal	$P_{tr}=b_{12}*a_{12}*c_3*g_đ*1/2$	234.027	2.917	682.577
Total		**31728.23**		**-85926.5**

2.2.2.7. Earth Pressure EH,LS



1. Earth Pressure EH (3.11.5 22 TCN 272-05)

- Formula:

$$EH=(\gamma*H^2*K*c_5)/2 \quad (KN)$$

Inwhich:

H	Total wall height	
H1	Height of pressure applied at section A-A	= 14.450 (m)
H2	Height of pressure applied at section B-B	= 11.950 (m)
H3	Height of pressure applied at section C-C	= 2.950 (m)
K	Coefficient of lateral earth pressure. For walls that deflect K =Ka	

Ka is coefficient of active pressure. (3.11.5.3)

$$Ka=\sin^2(\theta+\varphi')/(T*\sin^2(\theta)*\sin(\theta-\delta))$$

Inwhich: $T= [1+\mathrm{sqrt}(\sin(\varphi'+\delta)*\sin(\varphi'-\beta)/(\sin(\theta-\delta)^*\sin(\theta+\beta)))]^2$

δ_1	Friction angle between fill and fill	= 30 (degree)
δ_2	Friction angle between fill and wall	= 15 (degree)
β	Angle of fill to horizontal	= 0.00 (degree)
θ	Angle of backfill of wall to the vertical	= 90 (degree)
φf	Effective angle of internal friction	= 30 (degree)

With: T_1 = 3.097 T_1 = 2.610

K_{a1} = 0.280 K_{a2} = 0.251

Section	Lateral earth pressure (E_H)		
	E_H(KN)	e(m)	M(KNm)
A-A	12823.41	5.780	74119.32
B-B	7863.04	4.780	37585.34
C-C	479.18	1.180	565.43

2 Live load surcharge *LS*

- A live load surchage shall be aplied where vehicular load is expected to act on the surface of the backfill within a distance equal to the wall height behind back face of he wall $h_{eq.}$

- Horizontal earth pressure maybe detemined follow below formula:

$$LS=K*h_{eq}*\gamma*H*c_5 \ (KN)$$

- Assumed to act at height of 0.5H.

In which

K_{a1} = 0.280 K_{a2} = 0.251

h_{eq}: Equivalent height of soil (m)

Section	Horizontal pressure due to live load surcharge			
	H(m)	h_{eq}(m)	LS(KN)	M(KNm)
A-A	14.450	0.61	1082.67	3911.14
B-B	11.950	0.61	802.75	2398.23
C-C	2.950	1.217	395.26	291.50

- Vertical earth pressure (VS) due to live load surcharge to section A-A

Vaue of VS sau:

$$VS=h_{eq}*\gamma*(a_1-a_3-a_4)*c_5$$

With a1-a3-a4 is equivelent height of soil h_{eq} applied vertical load to section A-A

a_1-a_3-a_4 = 6

VS = 1607.65 (KN)

MS = -4421.05 (KN.m)

2.2.2.8. Internal force due to earthquake - E_{AE}

Acceleration coefficient specified A = **0.034**

1. Earth pressure due to Earthquake (E_{AE})

δ_1	Friction angle between fill and fill	= 30.00 (degree)
δ_2	Friction angle between fill and wall	= 15.00 (degree)

β	Angle of fill to horizontal		=	0.00 (degree)
i	Angle of fill to horizontal		=	0.00 (degree)
θ	Seismic intertial angle of soil	=arctan(Kh/(1-Kv)=	=	0.98 (degree)

Kv: earthquake effect factor to vertical

Kh: earthquake effect factor to horizontal

Foundation type : **1** ("1" : Drilled pile, "0" :shallow foundation)

Kh= 0.017 degree

Kv= 0.0068

Static earth pressure factor to section A-A K_{AS1} = 0.30

Static earth pressure factor to section B-B, C-C K_{AS2} = 0.30

Active earth pressure factor to section A-A K_{AE1} = 0.309

Active earth pressure factor to section B-B, C-C K_{AE2} = 0.312

Active earth pressure when eathquake occur

$$E_{AE} = 0.5*(1-Kv)(\gamma*H^2*Kae*c_5) \text{ (KN)}$$

Moment due to earth pressure :

$$M_{AE} = E_{AS}*H/3+(E_{AE}-E_{AS})*0.6H$$

Section	Horizontal earth pressure (E_{AE})			
	E_{AS}(KN)	E_{AE}(KN)	H(m)	M_{AE}(KNm)
A-A	13627.82	14058.35	14.450	69373.31
B-B	9453.32	9712.18	11.950	47389.50
C-C	576.09	591.87	2.950	2802.77

2. Earthquake effects to abutment (E_Q)

- At section A-A :

STT	Component	Volume (m^3)	E_Q (KN)	d (m)	M (KNm)
1	Pile cap	747.5	622.67	1.25	778.33
2	Body wall	468.0	389.84	7.00	2728.91
3	Head wall	38.4	31.95	12.25	391.33
4	Bracket	3.3	2.74	13.80	37.87
5	Wing wall (back part)	13.4	11.20	12.62	141.33
6	Wing wall (front part)	110.9	92.42	9.73	898.77
7	Bearing pad	0.9	0.77	12.05	9.23
8	Wing wall (on body wall)	1.0	0.82	12.38	10.16
	Total		**1152.40**		**4995.93**

- At section B-B :

STT	Component	Volume (m^3)	E_Q (KN)	d (m)	M (KNm)
1	Body wall	468.0	389.84	4.50	1754.30
2	Head wall	38.4	31.95	10.48	334.63
3	Bracket	3.3	2.74	11.30	31.01
4	Wing wall (back part)	13.4	11.20	10.12	113.34
5	Wing wall (front part)	110.9	92.42	7.23	667.73
6	Bearing pad	0.9	0.77	9.15	7.01
7	Wing wall (on body wall)	1.0	0.82	9.88	8.11
	Total		**529.73**		**2916.12**

- At section C-C :

STT	Component	Volume (m^3)	E_Q (KN)	d (m)	M (KNm)
1	Head wall	38.4	31.95	1.48	47.12
2	Bracket	3.3	2.74	2.30	6.31
3	Wing wall (back part)	5.6	4.66	1.48	6.88
4	Wing wall (front part)	28.3	23.59	1.48	34.80
5	Wing wall (on body wall)	1.0	0.82	0.88	0.72
	Total		**63.77**		**95.83**

2.2.3. - COMBINATION LOADING

2.2.3.1. COMBINATON LOADING TO SECTION A-A

LOADING		β	ΣV (kN)	ΣHx (kN)	ΣHy (kN)	ΣMx (kN•m)	ΣMy (kN•m)
Superstructure & Substructure (DC)		β_{DC}	43560.3				20511.22
Wearing surface (DW)		β_{DW}	802.2				1283.46
Horizontal earth pressure (EH)		β_{EH}		12823.41			74119.32
Ar-rest pressure (EV)		β_{EV}	31728.2				-85926.49
Live load (LL)		β_{LL}	4784.4				7655.01
Bracking load (BR)		β_{BR}		0.00			0.00
Pedestrian load (PL)		β_{PL}	0.0				0.00
Horizontal Live load surcharge (LS)		β_{LS}		1082.67			3911.14
Horizontal Live load surcharge (VS)		β_{LS}	1607.7				-4421.05
Earthquake effect to abutment		β_{EQ}		1152.40	345.72	1498.78	4995.93
Active earth pressure due to earthquake		β_{EQ}		14058.35			69373.31
Wind load on vehicles (WS)	Horizontal	β_{WS}			421.38	4809.2	
	Longitudinal	β_{WS}		0.00			0.00
Wind load on vehicle (WL)	Horizontal	β_{WL}			48.75	588.07	
	Longitudinal	β_{WL}		0.00			0.00
Vertical wind load (P_V)		β_{WS}	2112.5				3380.01
Friction load (FR)		β_{FR}		706.11			8341.96

COMBINATION LOADING TABLE

Max factor

Comb.	Factor β									ΣV	ΣHx	ΣHy	ΣMx	ΣMy
	β_{DC}	β_{DW}	β_{EH}	β_{EV}	β_{A}	β_{WS}	β_{WL}	β_{FR}	β_{EQ}	(kN)	(kN)	(kN)	(kN•m)	(kN•m)
Strength I	1.25	1.50	1.50	1.35	1.75	-	-	1.00	-	109673	21835.9	0.0	0.0	43588
Strength II	1.25	1.50	1.50	1.35	-	1.40	-	1.00	-	101444	19941.2	589.9	6732.8	35816
Strength III	1.25	1.50	1.50	1.35	1.35	0.40	1.00	1.00	-	107961	21402.8	48.8	588.1	42082
Service	1.00	1.00	1.00	1.00	1.00	0.30	1.00	1.00	-	83116	14612.2	48.8	588.1	26489
Extreme	1.25	1.50	1.50	1.35	0.50	-	-	1.00	1.00	101683	35693.3	345.7	1498.8	32534

Min factor

Comb.	Factor β									ΣV	ΣHx	ΣHy	ΣMx	ΣMy
	β_{DC}	β_{DW}	β_{EH}	β_{EV}	β_{A}	β_{WS}	β_{WL}	β_{FR}	β_{EQ}	(kN)	(kN)	(kN)	(kN•m)	(kN•m)
Strength I	0.90	0.65	0.90	1.00	1.75	-	-	1.00	-	82640	14141.8	0.0	0.0	20921
Strength II	0.90	0.65	0.90	1.00	-	1.40	-	1.00	-	74411	12247.2	589.9	6732.8	13149
Strength III	0.90	0.65	0.90	1.00	1.35	0.40	1.00	1.00	-	80928	13708.8	48.8	588.1	19415
Service	1.00	1.00	1.00	1.00	1.00	0.30	1.00	1.00	-	83116	14612.2	48.8	588.1	26489
Extreme	0.90	0.65	0.90	1.00	0.50	-	-	1.00	1.00	74650	27999.3	345.7	1498.8	12714

Note: $\beta_{LL,BR,PL,LS,VS} = \beta_{A}$

2.2.3.2. COMBINATON LOADING TO SECTION B - B

LOADING		β	ΣV	ΣHx	ΣHy	ΣMx	ΣMy
			(kN)	(kN)	(kN)	(kN•m)	(kN•m)
Superstructure & Substructure (DC)		β_{DC}	22199.0				2595.05
Wearing surface (DW)		β_{DW}	802.16				280.76
Horizontal earth pressure (EH)		β_{EH}		7863.04			37585.34
Live load (LL)		β_{LL}	4784.4				1674.53
Bracking load (BR)		β_{BR}		0.00			0.00
Pedestrian load (PL)		β_{PL}	0.00				0.00
Horizontal Live load surcharge (LS)		β_{LS}		802.75			2398.23
Earthquake effect to abutment		β_{EQ}		529.73	158.92	874.83	2916.12
Active earth pressure due to earthqual		β_{EQ}		9712.18			47389.50
Wind load on vehicles (WS)	Horizontal	β_{WS}			421.38	3755.7	
	Longitudinal	β_{WS}		0.00			0.00
Wind load on vehicle (WL)	Horizontal	β_{WL}			48.75	466.20	
	Longitudinal	β_{WL}		0.00			0.00
Vertical wind load (PV)		β_{WS}	2112.51				739.38
Friction load (FR)		β_{FR}		706.11			6576.69

COMBINATION LOADING TABLE

Max factor

Comb.	Factor β								ΣV	ΣHx	ΣHy	ΣMx	ΣMy
	β_{DC}	β_{DW}	β_{EH}	β_{B}	β_{WS}	β_{WL}	β_{FR}	β_{EQ}	(kN)	(kN)	(kN)	(kN•m)	(kN•m)
Strength I	1.25	1.50	1.50	1.75	-	-	1.00	-	37324.7	13905.5	-	-	67170.3
Strength II	1.25	1.50	1.50	-	1.40	-	1.00	-	31909.5	12500.7	589.9	5258.0	60043.0
Strength III	1.25	1.50	1.50	1.35	0.40	1.00	1.00	-	36255.9	13584.4	48.8	466.2	66280.6
Service	1.00	1.00	1.00	1.00	0.30	1.00	1.00	-	28419.3	9371.9	48.8	466.2	45273.3
Extreme	1.25	1.50	1.50	0.50	-	-	1.00	1.00	31344.2	23144.0	158.9	874.8	79701.7

Min factor

Comb.	Factor β								ΣV	ΣHx	ΣHy	ΣMx	ΣMy
	β_{DC}	β_{DW}	β_{EH}	β_{B}	β_{WS}	β_{WL}	β_{FR}	β_{EQ}	(kN)	(kN)	(kN)	(kN•m)	(kN•m)
Strength I	0.90	0.65	0.90	1.75	-	-	1.00	-	28873.2	9187.7	-	-	43472.2
Strength II	0.90	0.65	0.90	-	1.40	-	1.00	-	23458.0	7782.8	589.9	5258.0	36344.8
Strength III	0.90	0.65	0.90	1.35	0.40	1.00	1.00	-	27804.5	8866.6	48.8	466.2	42582.5
Service	1.00	1.00	1.00	1.00	0.30	1.00	1.00	-	28419.3	9371.9	48.8	466.2	45273.3
Extreme	0.90	0.65	0.90	0.50	-	-	1.00	1.00	22892.7	18426.1	158.9	874.8	50121.1

Note: $\beta_{LL,BR,PL,LS} = \beta_{B}$

2.2.3.3. COMBINATON LOADING TO SECTION C - C

LOADING	β	ΣV (kN)	ΣHx (kN)	ΣHy (kN)	ΣMx (kN•m)	ΣMy (kN•m)
Substructure (DC)	β_{DC}	1020.28				-32.28
Horizontal earth pressure (EH)	β_{EH}		479.18			565.43
Horizontal Live load surcharge (LS)	β_{LS}		395.26			291.50
Earthquake effect to abutment	β_{EQ}		63.77	19.13	28.75	95.83
Active earth pressure due to earthqua	β_{EQ}		591.87			2802.77

COMBINATION LOADING TABLE

Max factor

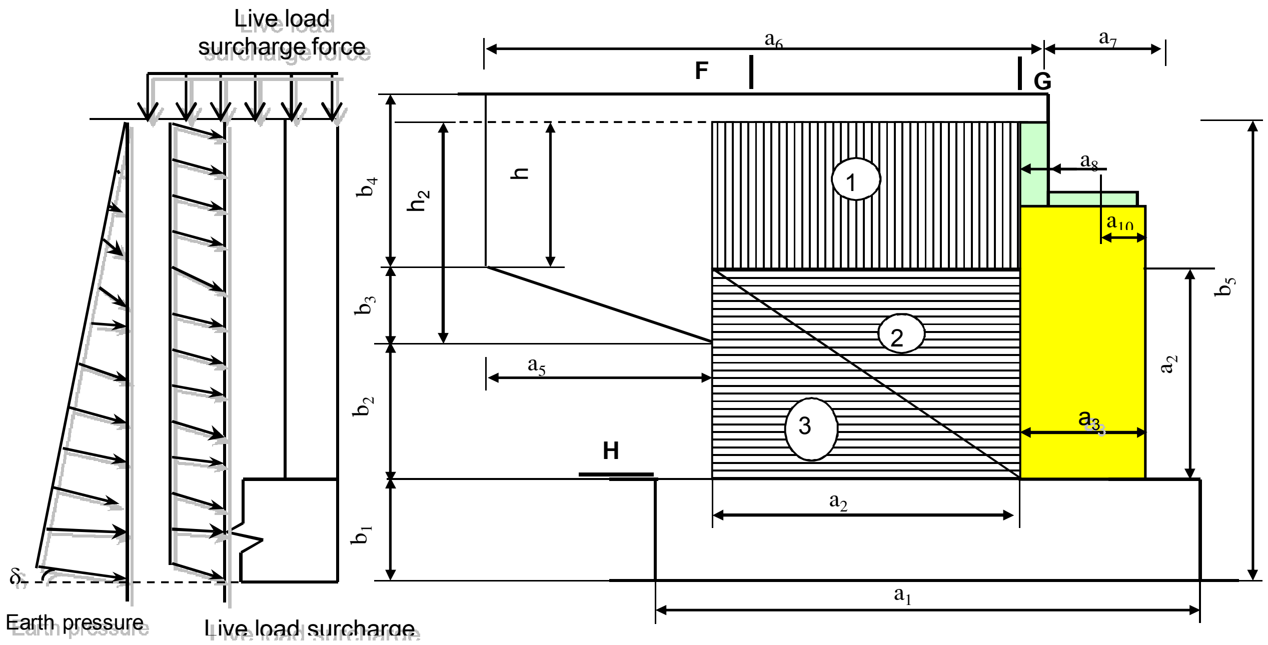
Comb.	Factor β				ΣV	ΣHx	ΣHy	ΣMx	ΣMy
	β_{DC}	β_{EH}	β_{LS}	β_{EQ}	(kN)	(kN)	(kN)	(kN•m)	(kN•m)
Strength I	1.25	1.50	1.75	-	1275.35	1410.47	-	-	1317.92
Strength II	1.25	1.50	-	-	1275.35	718.77	-	-	807.80
Strength III	1.25	1.50	1.35	-	1275.35	1252.37	-	-	1201.32
Service	1.00	1.00	1.00	-	1020.28	874.44	-	-	824.65
Extreme	1.25	1.50	0.50	1.00	1275.35	1572.03	19.13	28.75	3852.14

Min factor

Comb.	Factor β				ΣV	ΣHx	ΣHy	ΣMx	ΣMy
	β_{DC}	β_{EH}	β_{LS}	β_{EQ}	(kN)	(kN)	(kN)	(kN•m)	(kN•m)
Strength I	0.90	0.90	1.75	-	918.25	1122.96	-	-	989.96
Strength II	0.90	0.90	-	-	918.25	431.26	-	-	479.84
Strength III	0.90	0.90	1.35	-	918.25	964.86	-	-	873.36
Service	1.00	1.00	1.00	-	1020.28	874.44	-	-	824.65
Extreme	0.90	0.90	0.50	1.00	918.25	1284.52	19.13	28.75	3524.18

2.2.3.4. COMBINATION LOADING OF WING WALL

Diagram of wingwall analysis



Loading table to section G1

Loading	Formula	ΣQy	e	ΣMz
		kN	m	kNm
1. Horizontal earth pressure (EH)	$Qy=K_a*\gamma_đ*0.5*(b_5-b_1-a_2)*(b_5-b_1-a_2)*a_2$	534.64	3.000	1603.93
2. Live load surcharge (LS)	$Qy=K_a*\gamma_đ*h_{eq}*a2*(b_5-b_1-a_2)$	137.90	3.000	413.70

Notice:

h_{eq} = 0.767 The height of converted earth pressure corelative with = 5.950 (m)
it's retaining wall

Combination loading table to section G1

With max factor

Combination	Factor β		ΣQy	ΣMz
	β EH	β LS	kN	kNm
Strength I	1.50	1.75	1043.29	3129.86
Strength II	1.50	0.00	801.96	2405.89
Strength III	1.50	1.35	988.13	2964.38
Service	1.00	1.00	672.54	2017.62

With min factor

Combination	Factor β		ΣQy	ΣMz
	β EH	β LS	kN	kNm
Strength I	0.9	1.75	722.50	2167.50
Strength II	0.9	0.00	481.18	1443.53
Strength III	0.9	1.35	667.34	2002.02
Service	1.00	1.00	672.54	2017.62

Loading table to section G2

Loading	Formula	ΣQy	e	ΣMz
		kN	m	kNm
1. Horizontal earth pressure (EH)	$Qy=K_a*a_2*a_2*\gamma_đ*(b_5-b_1-2*a_2/3)*0.5$	720.36	2.00	1440.71
2. Live load surcharge (LS)	$Qy=K_a*gđ*heq*a_2*a_2/2$	55.50	2.00	111.00

Notice:

h_{eq} = 0.613 The height of converted earth pressure corelative with = 8.950 (m) it's retaining wall

Combination loading table to section G2

With max factor

Combination	Factor β		ΣQy	ΣMz
	β EH	β LS	kN	kNm
Strength I	1.50	1.75	1177.66	2355.32
Strength II	1.50	0.00	1080.54	2161.07
Strength III	1.50	1.35	1155.46	2310.92
Service	1.00	1.00	775.86	1551.71

With min factor

Combination	Factor β		ΣQy	ΣMz
	β EH	β LS	kN	kNm
Strength I	0.9	1.75	745.44	1490.89
Strength II	0.9	-	648.32	1296.64
Strength III	0.9	1.35	723.25	1446.49
Service	1.00	1.00	775.86	1551.71

COMBINATON LOADING TO SECTION F - F

LOAD	Formula	ΣQy	e	ΣMz
		(kN)	(m)	(kN)
1. Earth pressure (EH)	$Qy=0.5*K_a*\gamma_đ*h_{tb}*a_5*(h_1+h_2)$	53.63	1.46	78.17
2. Live load surcharge (LS)	$Qy=0.5*K_a*\gamma đ*heq*a5*(h1+h2)$	62.21	1.46	90.68

Inwhich:

h1 = b5-b1-b2-b3 = 1.4 (m)

h2 = b5-b1-b2 = 4.2 (m)

h_{eq} = 1.501 Height of backfill aquivalent with height of wall h2/2 = 2.10 (m)

h_{tb} = 1.513 Height of central backpart of wingwall

Combination loading table to section F-F

With max factor

Comb.	Factor β		ΣQy	ΣMz
	β EH	β LS	(kN)	(kN•m)
Strength I	1.50	1.75	96.40	84.02
Strength II	1.50	-	34.18	29.80
Strength III	1.50	1.35	82.18	71.63
Service	1.00	1.00	58.34	50.85
Extreme	1.50	0.50	51.96	45.29

With min factor

Comb.	Factor β		ΣQy	ΣMz
	β EH	β LS	(kN)	(kN•m)
Strength I	0.90	1.75	82.72	72.10
Strength II	0.90	-	20.51	17.88
Strength III	0.90	1.35	68.50	59.71
Service	1.00	1.00	58.34	50.85
Extreme	0.90	0.50	38.29	33.37

Loading table to section H3

Loading	Formula	ΣQy	e	ΣMz
		kN	m	kN
1. Horizontal earth pressure (EH)	$Qy=K_a*a_2*a_2*g_đ*(b_5-b_1-a_2/3)*0.5$	901.58	2.00	1803.16
2. Live load surcharge (LS)	$Qy=K_a*gđ*heq*a_2*a_2/2$	55.27	2.00	110.55

Notice:

h_{eq} = 0.61 The height of converted earth pressure corelative with it's retaining wall = 11.950 (m)

Combination loading table to section H3

With max factor

Combination	Factor β		ΣQy	ΣMz
	β_{EH}	β_{LS}	kN	kNm
Strength I	1.50	1.75	1449.10	2898.19
Strength II	1.50	0.00	1352.37	2704.74
Strength III	1.50	1.35	1426.99	2853.97
Service	1.00	1.00	956.85	1913.70

With min factor

Combination	Factor β		ΣQy	ΣMz
	β_{EH}	β_{LS}	kN	kNm
Strength I	0.9	1.75	908.15	1816.30
Strength II	0.9	0.00	811.42	1622.84
Strength III	0.9	1.35	886.04	1772.08
Service	1.00	1.00	956.85	1913.70

2.2.4. ULTIMATE CHECK AND SHEAR CAPACITY CHECK

2.2.4.1. CHECK FOR BODY SHAFT (SECTION B-B)

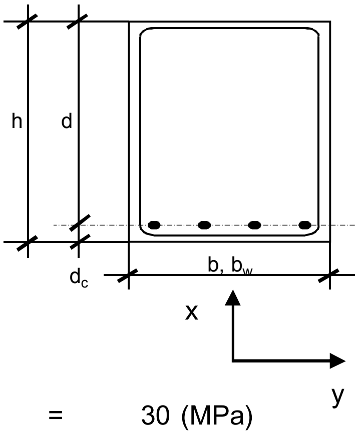
Combination loading to section B-B

Combination	ΣV	ΣHx	ΣHy	ΣMx	ΣMy
	kN	kN	kN	kN.m	kN.m
Strength I	37324.7	13905.5	0.0	0.0	67170.3
Strength II	31909.5	12500.7	589.9	5258.0	60043.0
Strength III	36255.9	13584.4	48.8	466.2	66280.6
Service	28419.3	9371.9	48.8	466.2	45273.3
Extreme	31344.2	23144.0	158.9	874.8	79701.7

The dimensions of calculated section.

- Effective Width of Section b, b_w = 26000 mm
- Total Depth of Section h = 2000 mm

1. Check Biaxial flexure



Used combination loading: Extreme

- Axial loading used in calculation: **N** = 31344 (kN)

If the fatored axial load is not less than 0,1.φ.f'c.Ag:

Mux/Mrx+Muy/Mry<=1 (1-a)

If the fatored axial load is not less than 0,1.φ.f'c.Ag:

1/Prxy = 1/Prx+1/Pry-1/φ.Po (1-b)

- Check for condition 0,1.φ.f'c.Ag:

φ = 0.75

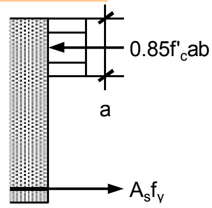
f'c Compression Strength of concrete at 28 days = 30 (MPa)

Ag Gros area of section\ = 52.00 (m^2)

Value: 0,1.φ.f'c.Ag = 117000 (kN)

- Compared N = 31344 $<$ 0,1.φ.f'c.Ag = 117000 Check follow formula (1-a)
- Mr x axial Flexure capacity (N.mm)

Mrx = $\varphi.As_x.fy.(ds-a/2)$



- Mr y axial Flexure capacity (N.mm)

Mrx = $\varphi.As_y.fy.(d_s-a/2)$

φ resistance factor for member in flexure = 0.90

A_s Reinforcement area

Longitudinal direction = 328 - D28 → A_s = 201967 mm^2

Horizontal direction = 12 - D16 → A_s = 2480 mm^2

d_c Effective Cover to Steel Centroid

d_c = 75.0 mm (Longitudinal)

= 75.0 mm (Horizontal)

d_s Depth from to Steel Centroid

Longitudinal direction d_s = 1811 mm

Horizontal direction d_s = 25917 mm

$a = c\beta_1$ Depth of the equivalent stress block

β_1 stress block factor β_1 = 0.84

Longitudinal direction a = 122 mm

Horizontal direction a = 19 mm

Factored flexural resistance:

Longitudinal direction Mrx = 23136 kNm

Horizontal direction Mry = 131674 kNm

- Mux Factored flexural in direction of the x axial = 874.8 kNm
- Muy Factored flexural in direction of the y axial = 79702 kNm
- Detemined slendernes effects K.Lu/r

r	The radius of gyration	$rx = \sqrt{\frac{Ix}{A}}$	= 7.51 m	$ry = \sqrt{\frac{Iy}{A}}$	= 0.6 m
A	The gross concrete section		= 52.00 m^2		
I	Moment of inertia		I_x = 2929.33 m^4		I_Y = 17.33 m^4
k	Effective length factor		= 1.00		
Lu	Length (no support) of Abument shaft		= 9.00		

Horizontal slenderness ratio K.Lu/r = 1 < 22 Ignore slenderness effect

Longitudinal slenderness ratio K.Lu/r = 16 < 22 Ignore slenderness effect

- Consider for vertical slenderness effect:

$$\delta_b = \frac{C_m}{1 - \frac{Pu}{\varphi . Pe}} \geq 1$$

φ	Compressive resistance factor	= 0.75
Cm	= 1	
Pu	Factored compressive	= 37325 kN
Pe	Vertical flexurel force Ơle, Pe = π^2.EI/(KLu)^2	
E	Concrete modulus	2.94E+07 kN/m2
I	Moment of inertia	= 17.33 m4

EI = 510294849.41 kNm2

Pe = 62114853.05 kN

=> δ_b = 1.001

- Mô men tính toán tăng lên phản ánh tác dụng của biến dạng như sau:

$M_{ux}^{tt} = \delta_b.M_{ux}$ = 875.54 kNm

$M_{uy}^{tt} = \delta_b.M_{uy}$ = 79766 kNm

- Check follow condition (1-a):

Mux kNm	Muy kNm	Mry kNm	Mrx kNm	Mux/Mrx	Muy/Mry	Mux/Mrx+Muy/Mry	Conclusion
875.54	79766	131674.22	23136.49	0.038	0.606	0.64	OK

- Reinforcement ratio ρ = 0.43 %
- Check of minimum reinforcement ratio ρ ≥ 0.03f'c/fy = 0.23 % **OK.**
- Check of maximum reinforcement ratio 0.42 ≥ c/de = 0.081 **OK.**
- Kiểm tra hàm lượng cốt thép dọc tham gia chịu cắt cùng cốt thép đai theo:

As fy >(Mu/dv φ + 0.5*Nu/φ) + (Vu/φ - 0.5*Vs)*cotgθ **OK.**

As fy= **80787** kN

(Mu/dv φ + 0.5*Nu/φ) + (Vu/φ - 0.5*Vs)*cotgθ = **47848** kN

2. Checking shear capacity

Datas		**Trans.**	**Long.**	**Unit**
• Shear force	V_u	23144	590	kN
• Resistance factor	φ	0.90	0.90	
• The shear depth of structure	d_v	1811	25917	mm
• Effective web width taken as the minimum web width within the depth d	b_v	25917	1811	mm
• Angle of inclination of diagonal compressive stresses	θ	45	45	độ
• Angle of inclination of transverse reinforcement to long. Axis	α	90	90	độ
• Factor indicating ability of diagonally cracked concrete to transmit tensi	β	2	2	
•Value	$0.1 \cdot f'_c \cdot b_v \cdot d_v$	140807	140807	kN

• The spacing of stirrups	s	600	600	mm
• Diameter of shear reinforcement	D	D12	D12	mm
• No. of shear reinforcement within a distance s	n	86	2	
• Total area of shear reinforcement	A_v	9726	226	mm^2
• Nominal shear resistance provided by tensile stresses in the concrete	V_c	42675	42675	kN
•Shear resistance provided by shear reinforcement	V_s	11743	3908	kN
• Value	$0.25 \cdot f'_c \cdot b_v \cdot d_v$	572029	352018	kN
• Nominal shear resistance	V_n	54418	46583	kN
• The factor shear resistance	V_r	48976	41925	kN
• Checking	$V_r > V_u$	**OK**	**OK**	

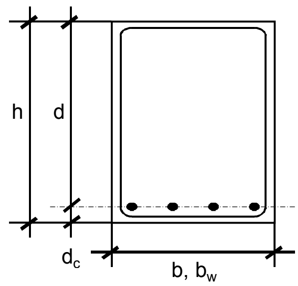
3. Check crack

Datas		**Trans.**	**Long.**	**Unit**
• Interior force combination	**Service**			
• Factored moment	M_u	466	45273	kNm
• Hight of Section	h	26000	2000	mm
• Width of section	b	2000	26000	mm
• Effective Cover to Steel Centroid	d_c	75	75	mm
• Distance from compressive reinf. to extreme Tension fiber	d_e	25925	1925	mm
• Tension reinforcement:	D	16	28	mm
- Number of Bar	n	12	328	Thanh
- Total area of reinf.	A_s	2480	201967	mm^2
• Ratio of reinf. Modulus with concrete modulus	$n = E_s/E_c$	= 7.00	= 7.00	
• The distance from extreme fiber to the neutral axis	c	= 679.57	= 515.14	%
• Effective moment of inertia	J	1.13E+13	3.99E+12	
• Arm	de-c	25245.43	1409.86	
• Tension stress in reinforcement	fs =n*Ms*(de-c)/J	= 7.31	111.84	Mpa
• Crack width parameter	Z	23000	23000	N/mm
• Area of concrete having the same centroid as the principal tensile reinforcement divided by number of bars	A	16216	15854	mm^2
• Tension stress in reinforcement with service state	f_{sa}	234.7	228.9	Mpa
• Check condition	$f_s \le Min(0.6 \cdot f_y, f_{sa})$	**OK**	**OK**	

2.2.4.2. CHECKING HEADWALL (SECTION C-C)

Combination loading to section B-B

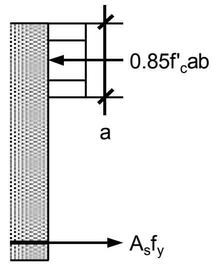
Combination	ΣV	ΣHx	ΣHy	ΣMx	ΣMy
	kN	kN	kN	kN.m	kN.m
Strength I	1275.3	1410.5	0.0	0.0	1317.9
Strength II	1275.3	718.8	0.0	0.0	807.8
Strength III	1275.3	1252.4	0.0	0.0	1201.3
Service	1020.3	874.4	0.0	0.0	824.7
Extreme	1275.3	1572.0	19.1	28.7	3852.1



Data

- Effective Width of Section — b, b_w = 26000 mm
- Total Depth of Section — h = 500 mm
- Depth from to Steel Centroid — d = 442 mm
- Effective Cover to Steel Centroid — d_c = 50 mm

2.1. Check Flexural capacity



Factored combination loading: Extreme

-Resistance factor ϕ = 0.90

-Reinforcement area = 170 - D16 — A_s = 34181 mm^2

-Reinforcement ratio ρ = 0.26 %

-Stress block factor β_1 = 0.84

-Depth of the equivalent stress block a = 21 mm

Flexural resistance **M_r = 5438.8** kNm

Factored moment **M_u = 3852.1** kNm

-Check condition — Mu ≤ Mr = 5438.8 — OK.

-Check for minimum reinf. Ratio — ρ ≥ 0.03f'c/fy = 0.23 % — OK.

-Check for maximum reinf. Ratio — 0.42 ≥ c/de = 0.056 — OK.

2.2. Shear capacity check

-Resistance factor: ϕ_v = 0.90

- Transverse reinforcement area: = 2 - D12 — A_v = 226.19 mm^2

-Spacing of Transverse reinforcement: s = 200 mm

-Factor indicating ability of diagonally cracked concrete to transmit tension (5.8.3.4 22 TCN 272-05) — β = 2

- Angle of inclination of diagonal compressive stresses (5.8.3.4) — θ = 45 °

- The shear depth of structure: dv= max(de-0.5*a,0.9*dc,0.72*h) — dv = 432 mm

-The shear resistance of concrete: (5.8.2.4 22 TCN 272-05) — V_c = 10205 kN

-The shear resistance of stirrup, (5.8.3.3) — V_s = 195 kN

Vn = 0,25.fc.bv.dv = 84179 kN

Shear resistance **V_r = 9360** kN

Factored shear **V_u = 1572** kN

Check condition Vu ≤ Vr = **9360** kN — O.K.

2.3. Check crack

- Interior force combination — **Service**
- Factored moment — Ms = 8.25E+05 KN.mm
- Modulus of rupture of concrete — fr = 0.63*sqrt(f'c) = 3.45 MPa
- Distance from extreme tension fiber to the neutral axis — y_t = h-c = 475 mm
- Stress of concrete at tension fiber — f'r = Ms*y_t/I_g = 1.45 MPa
- **Check condition**

Stress of concrete f'r<0.8*fr shouldn't be controled of cracking by distribution of reinforcement by condition

2.2.4.3. CHECK FOR WING WALL

Item			Section G1	Section G2	Section F-F	Section H3	Unit
• Factored Plexural moment		M_u	3129.86	2355.32	84.02	2898.19	kN.m
• Factored Shear force		V_u	1043.29	1177.66	96.40	1449.10	kN
• Hight of Section		h	800	800	800	800	mm
• Width of section		b	5950	6000	3800	6000	mm
• Section area		A_c	4760000	4800000	3040000	4800000	mm^2
• Moment of inertia of concrete section		I_g	2.5E+11	2.6E+11	1.6E+11	2.6E+11	mm^4
• Tension reinforcement:	Distance from tension reinf. to extreme tension fiber	d_c	88	88	88	63	mm
	Reinf. Diameter	∅	D 25	D 25	D 25	D 25	mm
	Space	@	150	150	150	150	mm
	Number of bar	n	34	40	16	40	bar
	Total area of reinf.	A_s	16690	19635	7854	19471	mm^2
• comp. reinforcement:	Distance from compressive reinf. to extreme Compression fiber		74	74	74	58	mm
	Diameter		D 16	D 16	D 16	D 16	mm
	Reinf. Space		150	150	150	150	mm
	Number of bar		34	40	16	40	bar
	Total area of reinf.	A'_s	6836	8042	3217	7975	mm^2
Check Flexural Moment							
• Resistance factor		Φ	0.90	0.90	0.90	0.90	
• The corresponding effective		d_e	713	713	713	738	mm
• Stress block factor		β_1	0.84	0.84	0.84	0.84	
• Depth of the equivalent stress block = c*β1		a	44.00	51.33	32.42	50.91	mm
• Distance from extreme compression fiber to the neutral axis		c	52.65	61.42	38.79	60.91	mm
• The nominal flexural resistance:		M_n	4610	5394	2187	5546	kN.m
• Factored flexural resistance		$M_r = \Phi.M_n$	4149	4855	1969	4991	kN.m
• Check condition		$M_r > M_u$	O.K	O.K	O.K	O.K	
Mimimum Reinforcement							
• Ratio of tension steel to gross area		$\rho = A_s/(b.d)$	0.39	0.46	0.29	0.44	%
• Check		$\rho > 0.03 \cdot f'_c/f'_y$	O.K	O.K	O.K	O.K	0.23
• Cracking moment		1.2Mcr	2628.02	2650.10	1678.40	2650.10	KN.m
• Check		Mr> min(1.2Mcr, 1.33Mu)	O.K	O.K	O.K	O.K	
Maximum Reinforcement							
• Obligation Condition		c/d_e	0.07	0.09	0.05	0.08	
• Check		$c/d_e < 0.42$	O.K	O.K	O.K	O.K	
Check shear resistance							
• Factored Shear force		V_u	1043.29	1177.66	96.40	1449.10	kN
• Resistance factor		Φ	0.90	0.90	0.90	0.90	
• The effective shear deepth		d_v	691	687	696	712	mm
• Effective width		b_v	5950	6000	3800	6000	mm
• Angle of inclination of diagonal compressive stress		θ	45	43	43	43	degree
• Angle of inclination of transerverse reinf. To longitudinal axis		α	90	90	90	90	degree
• Factor indicating ability of diagonally cracked concrete to transmit tension		β	2	2	2	2	
• Value		$0.1 \cdot f'_c \cdot b_v \cdot d_v$	12325	12363	7938	12817	kN
• Max spacing of transverse reinforcement		s	552	549	557	570	mm
• Spacing of stirrup		s	300	300	300	300	mm
• Diameter of transverse reinforcement		∅	D 25	D 25	D 25	D 25	
• Number of transverse reinf. within distance s		n	2	2	2	2	bar
• Total area of transverse reinf.		A_v	982	982	982	982	mm^2
• Diameter of stirrup		∅	D 12	D 12	D 12	D 12	mm
• Number of stirrup within distance s		n	19	19	12	19	bar
• Total area of stirrup		A_v	2130.00	2148.85	1319.47	2148.85	
• Assume		θ	43.00	43.00	43.00	43.00	degree
• Strain in tensile reinforcement		ex	8.96E-04	1.00E-03	1.34E-04	1.23E-03	
If ex<0, multiple with reduce factor		Fc	-	-	-	-	
• Ratio of shear stress and f'c		V/f'c	0.01	0.01	0.00	0.01	
• β final			2.00	1.75	1.75	1.75	
• θ final			45.00	43.00	43.00	43.00	degree
• The shear resistance of concrete:		V_c	3735.51	3278.53	2104.99	3398.89	kN

• The shear resistance of stirrup	V_s	874.46	1004.83	625.50	1041.72	kN
• Value	$0.25*f'_c*b_v*d_v$	30813.56	30907.50	19844.25	32042.13	kN
• The nominal shear resistance:	V_n	4609.97	4283.37	2730.49	4440.61	kN
• The factored shear resistance	V_r	4148.97	3855.03	2457.44	3996.55	kN
• Check	$V_r > V_u$	**O.K**	**O.K**	**O.K**	**O.K**	
•Requiring transverse reinforcement	$V_u > 0.5 \cdot \Phi \cdot V_c$	**No require**	**Not require**	**Not require**	**Not require**	
•Check minimum transverse reinforcement	$A_v > 0.083 \cdot (f'_c{}^{0.5}) \cdot b_v \cdot s/f_y$	**O.K**	**O.K**	**O.K**	**O.K**	
Check crack						
Interior force combination	**Service**					
• Factored moment	M_u	2.02E+03	1.55E+03	5.08E+01	1.91E+03	kN.m
• Modulus of rupture of concrete	fr = 0.63*sqrt(f'c)	3.45	3.45	3.45	3.45	MPa
• Distance from extreme tension fiber to the neutral axis	y_t = h-c	747	739	761	739	mm
• Stress of concrete at tension fiber	f'r = Ms*y_t/I_g	6	4	0	6	MPa
• Check	f'r >	0.8*fr	0.8*fr	0.8*fr	0.8*fr	
		check	check	No check	check	
• Crack width parameter	Z	= 23000	= 23000	= 23000	= 23000	N/mm
• Ratio of reinf. Modulus with concrete modulus	n = E_s/E_c	= 7.00	= 7.00	= 7.00	= 7.00	
• The distance from extreme fiber to the neutral axis	c	= 188.06	= 205.03	= 158.78	= 207.17	mm
• Effective moment of inertia	J	4.53E+10	5.26E+10	2.19E+10	5.61E+10	mm4
• Arm	de-c	= 524.44	= 507.47	= 553.72	= 530.33	mm
• Tension stress in reinforcement	fs =n*Ms*(de-c)/J	= 163.43	= 104.73	= 8.99	= 126.60	MPa
• Area of concrete having the same centroid as the principal tensile	A	= 17500	= 15000	= 23750	= 15126	mm2
• Tension stress in reinforcement with service state	$f_{sa} = Z/(d_c*A)^{1/3}$	= 218.12	= 226.29	= 206.71	= 225.84	Mpa
• Check condition	$f_s < f_{sa}$	**O.K**	**O.K**	**O.K**	**O.K**	
• Check condition	$f_s < 0.6*f_y$	**O.K**	**O.K**	**O.K**	**O.K**	

2.2.5.1. INTERNAL FORCE AT TOP OF PILE

1. Summary of external force acting bottom footing

Combination Type	N (KN)	Hx (Kn)	Hy (Kn)	Mx (Kn.m)	My (Kn.m)
Strength I	109673	21836	0	0	43588
Strength II	101444	19941	590	6733	35816
Strength III	107961	21403	49	588	42082
Service I	83116	14612	49	588	26489
Extreme	101683	35693	346	1499	32534

2. Piling material:

Concrete 30 Mpa

E_c (Mpa)	294401
γ_c (Kn/m^3)	24.5

Steel bar

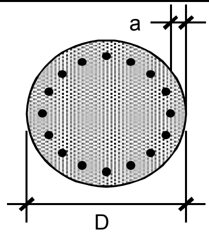
Type	CB-400-T
E_s (kg/cm^2)	200000

3. Piling dimension

+ Diameter **D** = 1.50 m

a = 0.075 m

+ Length **L** = 62.00 m



4. Maximum Internal force at top piling

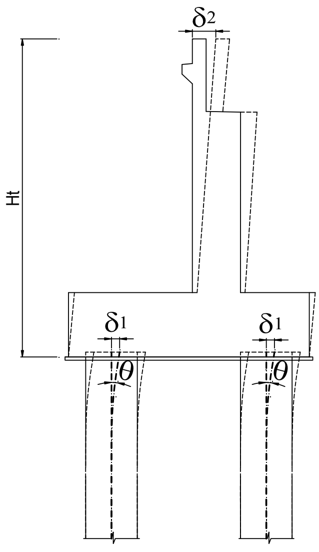
Internal force and diplacements *(Result follow Piling software)*

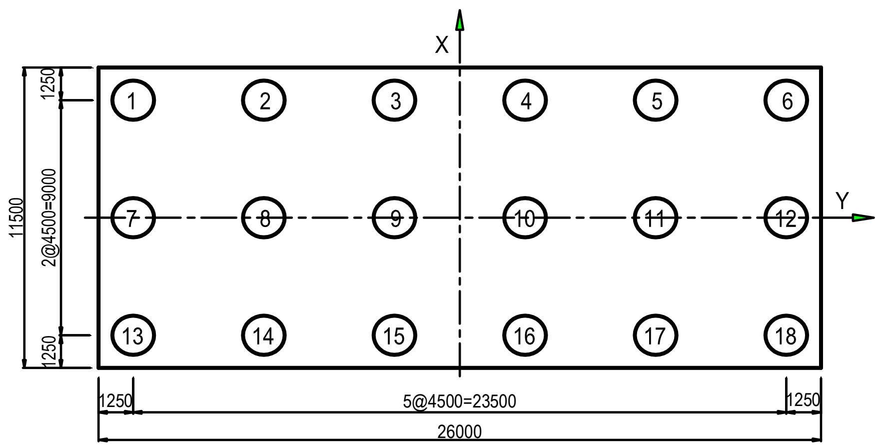
Combination Type	N (KN)	H (KN)	M (KN.m)	x (m)	y (m)	z (rad)
Strength I	8576.08	1213.11	5027.85	-	-	-
Strength II	7932.85	1107.83	4606.59	-	-	-
Strength III	8429.12	1189.06	4930.58	-	-	-
Service I	6241.42	811.78	3374.61	**0.016**	**-0.000**	**0.008**
Extreme	9071.92	1982.94	8364.73	-	-	-

- Check displacement of top pile not exceed 38mm (10.7.2.7) **OK**

- Displacement of top abutment

+ Displacement of foundation $\delta_1 =$ 16.38 (mm)
+ Rotation of foundation $\theta =$ -0.0005 (deg)
+ Displacement at top of abutment $\delta_2 =$ 22.96 (mm)
+ Total height of abutment $H_t =$ 14.45 (m)





Arrangement of pile

- Result for internal force at top of each pile

Combination	Pile Number	**Hx**	**Hy**	**N**	**Mx**	**My**
		(KN)	(KN)	(KN.m)	(m)	(m)
Strength I	1	1213.11	0.00	3609.81	0.00	5027.85
	2	1213.11	0.00	3609.81	0.00	5027.85
	3	1213.11	0.00	3609.81	0.00	5027.85
	4	1213.11	0.00	3609.81	0.00	5027.85
	5	1213.11	0.00	3609.81	0.00	5027.85
	6	1213.11	0.00	3609.81	0.00	5027.85
	7	1213.11	0.00	6092.94	0.00	5027.85
	8	1213.11	0.00	6092.94	0.00	5027.85
	9	1213.11	0.00	6092.94	0.00	5027.85
	10	1213.11	0.00	6092.94	0.00	5027.85
	11	1213.11	0.00	6092.94	0.00	5027.85
	12	1213.11	0.00	6092.94	0.00	5027.85
	13	1213.11	0.00	8576.08	0.00	5027.85
	14	1213.11	0.00	8576.08	0.00	5027.85
	15	1213.11	0.00	8576.08	0.00	5027.85
	16	1213.11	0.00	8576.08	0.00	5027.85
	17	1213.11	0.00	8576.08	0.00	5027.85
	18	1213.11	0.00	8576.08	0.00	5027.85
Strength II	1	1107.83	-32.78	3535.27	141.93	4606.59
	2	1107.83	-32.78	3495.96	141.93	4606.59
	3	1107.83	-32.78	3456.65	141.93	4606.59
	4	1107.83	-32.78	3417.33	141.93	4606.59
	5	1107.83	-32.78	3378.02	141.93	4606.59
	6	1107.83	-32.78	3338.71	141.93	4606.59
	7	1107.83	-32.78	5734.06	141.93	4606.59
	8	1107.83	-32.78	5694.75	141.93	4606.59
	9	1107.83	-32.78	5655.44	141.93	4606.59
	10	1107.83	-32.78	5616.12	141.93	4606.59
	11	1107.83	-32.78	5576.81	141.93	4606.59
	12	1107.83	-32.78	5537.50	141.93	4606.59
	13	1107.83	-32.78	7932.85	141.93	4606.59
	14	1107.83	-32.78	7893.54	141.93	4606.59
	15	1107.83	-32.78	7854.22	141.93	4606.59
	16	1107.83	-32.78	7814.91	141.93	4606.59
	17	1107.83	-32.78	7775.60	141.93	4606.59
	18	1107.83	-32.78	7736.28	141.93	4606.59
Strength III	1	1189.06	-2.72	3583.48	11.76	4930.58
	2	1189.06	-2.72	3580.09	11.76	4930.58
	3	1189.06	-2.72	3576.71	11.76	4930.58
	4	1189.06	-2.72	3573.32	11.76	4930.58
	5	1189.06	-2.72	3569.94	11.76	4930.58
	6	1189.06	-2.72	3566.55	11.76	4930.58
	7	1189.06	-2.72	6006.30	11.76	4930.58
	8	1189.06	-2.72	6002.91	11.76	4930.58
	9	1189.06	-2.72	5999.53	11.76	4930.58
	10	1189.06	-2.72	5996.14	11.76	4930.58
	11	1189.06	-2.72	5992.76	11.76	4930.58
	12	1189.06	-2.72	5989.37	11.76	4930.58
	13	1189.06	-2.72	8429.12	11.76	4930.58
	14	1189.06	-2.72	8425.73	11.76	4930.58
	15	1189.06	-2.72	8422.35	11.76	4930.58
	16	1189.06	-2.72	8418.96	11.76	4930.58
	17	1189.06	-2.72	8415.58	11.76	4930.58
	18	1189.06	-2.72	8412.19	11.76	4930.58

Combination	Pile Number	Hx	Hy	N	Mx	My
		(KN)	(KN)	(KN.m)	(m)	(m)
Service	1	811.78	-2.72	3010.61	11.76	3374.61
	2	811.78	-2.72	3007.23	11.76	3374.61
	3	811.78	-2.72	3003.84	11.76	3374.61
	4	811.78	-2.72	3000.46	11.76	3374.61
	5	811.78	-2.72	2997.07	11.76	3374.61
	6	811.78	-2.72	2993.69	11.76	3374.61
	7	811.78	-2.72	4626.02	11.76	3374.61
	8	811.78	-2.72	4622.63	11.76	3374.61
	9	811.78	-2.72	4619.25	11.76	3374.61
	10	811.78	-2.72	4615.86	11.76	3374.61
	11	811.78	-2.72	4612.48	11.76	3374.61
	12	811.78	-2.72	4609.09	11.76	3374.61
	13	811.78	-2.72	6241.42	11.76	3374.61
	14	811.78	-2.72	6238.04	11.76	3374.61
	15	811.78	-2.72	6234.65	11.76	3374.61
	16	811.78	-2.72	6231.27	11.76	3374.61
	17	811.78	-2.72	6227.88	11.76	3374.61
	18	811.78	-2.72	6224.50	11.76	3374.61
Extreme event	1	1982.94	-19.22	2290.47	85.47	8364.73
	2	1982.94	-19.22	2277.62	85.47	8364.73
	3	1982.94	-19.22	2264.76	85.47	8364.73
	4	1982.94	-19.22	2251.90	85.47	8364.73
	5	1982.94	-19.22	2239.05	85.47	8364.73
	6	1982.94	-19.22	2226.19	85.47	8364.73
	7	1982.94	-19.22	5681.20	85.47	8364.73
	8	1982.94	-19.22	5668.34	85.47	8364.73
	9	1982.94	-19.22	5655.48	85.47	8364.73
	10	1982.94	-19.22	5642.63	85.47	8364.73
	11	1982.94	-19.22	5629.77	85.47	8364.73
	12	1982.94	-19.22	5616.91	85.47	8364.73
	13	1982.94	-19.22	9071.92	85.47	8364.73
	14	1982.94	-19.22	9059.07	85.47	8364.73
	15	1982.94	-19.22	9046.21	85.47	8364.73
	16	1982.94	-19.22	9033.35	85.47	8364.73
	17	1982.94	-19.22	9020.50	85.47	8364.73
	18	1982.94	-19.22	9007.64	85.47	8364.73

2.2.5.2. Check for pile:

Item		Mark	Unit	Value
• Factored Axial force		Nu	Kn	8576.08
• Factored Plexural moment		Mux	Kn.m	141.93
• Factored Plexural moment		Muy	Kn.m	4606.59
• Diameter of Pier shaft		D	m	1.50
• Section area		Ag	m2	1.77
• Moment of inertia of concrete section		Ic	m4	0.25
• Cover thickness		a	m	0.075
• Reinf. Diameter		Ds	mm	D32
• Number of rebar		n_s	nos	48
• Rebar area		As	mm2	38603.89
Check minimum reinforcement				
• Minimum rebar area required (0.135*f'c/fy)*Ag		As req	mm2	17892.35
• Check condition As > (0.135*f'c/fy)*Ag				**OK**
Check maximum reinforcement				
• Maximum rebar area 0.08*Ag		As max	mm2	141371.7
• Check condition As < 0.08*Ag				**OK**
Check ratio spiral or Tier (5.7.4.6)				
• Distance to outside of Spairal or Ties to concrete face			mm	68.00
• Effect diamete		Deff	m	1.36
• Area of core measured to the outside diameter of the spiral			m2	1.46
• Ratio spiral Rebar required		ρsa		0.00707
Required Area of Spiral Rebar	space		mm	75
	Effective length			1.36
	layer			1
	Area			180.7
	Requaired Dhs			15.2
Actuaral	Effective length	d	m	1.364
	Diameter	Dhr	mm	16
	Area of Rebar	Ah	mm2	201.1
	layer	Nl	nos	1
	Total area of spiral	Ac	m2	201.062
	space	s	mm	75
	Ratio spiral Rebar	ρs	-	0.0078617
• Check condition		$\rho_s > \rho_{sa}$		OK

MIDAS/Set **Column Design [Pile D1.5m]**

	Company	Nipponkoei	**Project Name**	DN Qn Expressway
	Designer	vinh	**File Name**	D:\...\Pile D1.5m - A2.BOI

1. Geometry and Materials

Design Code : ACI318M-02

Stress Profile : Equivalent Stress Block

Material Data : f_c' = 30 MPa (β_1 = 0.850)

f_y = 400, f_{ys} = 400 MPa

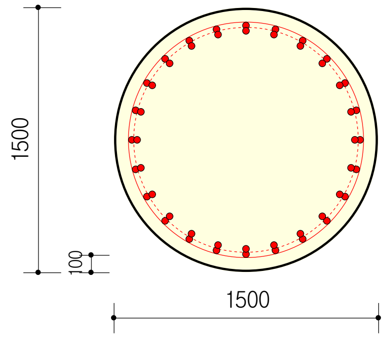
Section Dim. : Φ1500 mm

Effective Len. : KL_u = 9000 mm

Steel Distribut.: 24 - D32 (d_c = 100 mm)

: 24 - D32 (d_c = 132 mm)

Total Steel Area A_{st} = 38122 mm^2 (ρ_{st} = 0.0216)



2. Member Force and Moment

Unit : kN, kN-m

L.C.	P_u	M_{ux}	M_{uy}	R_{atioV}	V_{ux}	V_{uy}	R_{atioH}	Remark
1	6092.9	0.0	5027.9	0.589	1213.1	0.0	0.255	
2	7932.9	141.9	4606.6	0.598	1107.8	32.8	0.232	
3	8429.1	11.8	4930.6	0.639	1189.1	2.7	0.248	
4	6241.4	11.8	3374.6	0.447	811.8	2.7	0.171	
5	9071.9	85.5	8364.7	0.974	1982.9	19.2	0.413	

3. Magnified Moment

KL_u/r_x = 9000/375 = 24.00 > 34-12(M_1/M_2) = 22.00

δ_x = MAX[1.00/(1-P_u/0.75/195843), 1.0] = 1.066

KL_u/r_y = 9000/375 = 24.00 > 34-12(M_1/M_2) = 22.00

δ_y = MAX[1.00/(1-P_u/0.75/195843), 1.0] = 1.066

4. Design Force and Moment

Design Load Combination No : 5

P_u = 9071.9 kN

M_{ux} = 85.5, M_{uy} = 8364.7 kN-m

$\delta_x M_{ux}$= δ_x*MAX[M_{ux}, $P_u e_{min}$] = 580.1 kN-m

$\delta_y M_{uy}$= δ_y*M_{uy}, = 8915.4 kN-m

5. Check Axial and Moment Capacity

Rotation Angle and Depth to the Neutral Axis θ = -3.72°, c = 628 mm

Strength Reduction Factor Φ = 0.7919

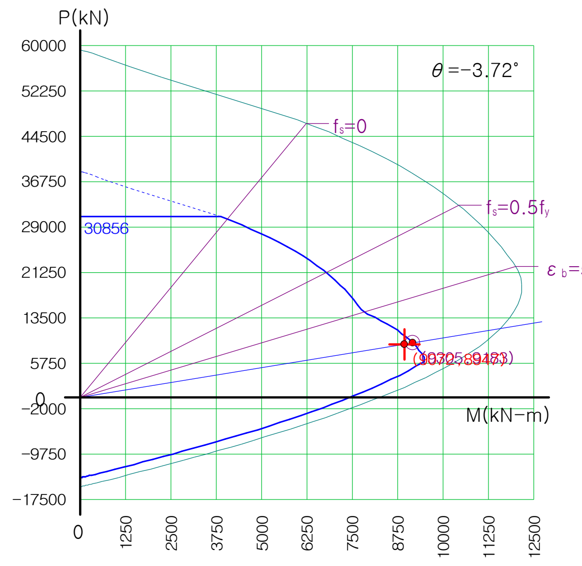
Maximum Axial Load $\Phi P_{n(max)}$ = 30856.2 kN

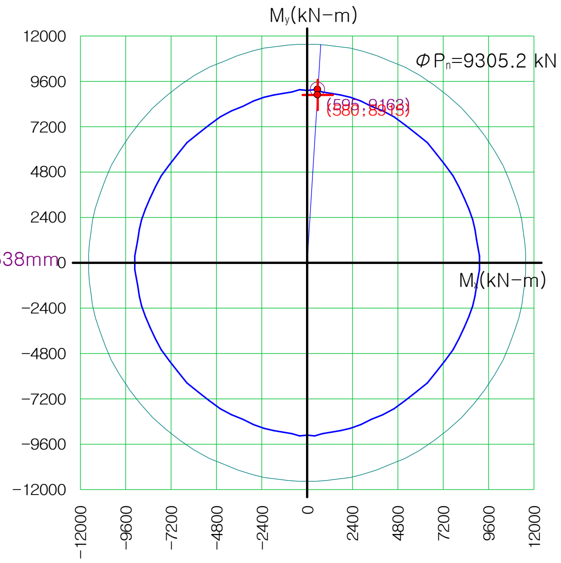
Design Axial Load Strength ΦP_n = 9305.2 kN

Design Moment Strength ΦM_{nx} = 595.5 kN-m

ΦM_{ny} = 9163.4 kN-m

Strength Ratio : Applied/Design = 0.974 < 1.000 .... O.K





6. Check Shear Capacity

Design Load Combination No : 5

Strength Reduction Factor Φ= 0.750

Design Force Vu = 1983.0 kN (Pu = 9071.9 kN)

Required Hoop Spacing : D16 @ 309 mm

Provided Hoop Spacing : D16 @ 150 mm (Tie)

$\Phi V_c + \Phi V_s$ = 3874.3 + 924.5 = 4798.8 kN > V_u = 1983.0 kN .... O.K

2.2.5.3. COMBINATON LOADING TO SECTION D - D

Component	Formular	Volume	ΣVz	e	ΣMy	β
		(m3)	(KN)	(m)	(KN.m)	
1. Pile cap	$V_{bm}= b_1*(a_1-a_3-a_4)*c_2$	0.0	0.00	-3.00	0.00	β DC
2. Bracket	$V_{mđ} =(b_{11}+a_9/2)*a_9*(c_3-2*c_1)$	3.3	80.70	0.15	12.11	β DC
3. Wing wall (back part)	$V_{tcd}=(2b_4+b_3)*a_5*c1$	13.4	329.28	-7.41	-2439.87	β DC
4. Wing wall (front part)	$V_{tct}=2*(b_2+ b_3 + b_4)*a_2*c_1$	110.9	2718.21	-3.00	-8154.62	β DC
5. Backfill	$V_s=b_8*S_{tđ}$	1749.5	31494.21	-3.00	-94482.62	β EV
6. Vertical earth pressure (LS)	$V_{ls}=heq*c_5*(a_1-a_3-a_4)$	89.3	1607.65	-3.00	-4822.96	β LS
Tổng cộng			**36230.05**		**-109887.97**	

COMBINATION LOADING TABLE

Max factor

Comb.	Factor β			Internal force at top pile		Shear force	Moment
				ΣNi	ΣMi	Qz	My
	β DC	β EV	β LS	(kN)	(kN•m)	(kN)	(kN•m)
Strength I	1.25	1.35	1.75	58216.52	141127.23	-8975.71	-8092.47
Strength II	1.25	1.35	-	54436.61	133626.18	-8009.19	-7153.35
Strength III	1.25	1.35	1.35	57437.08	139603.14	-8839.33	-7687.38
Service	1.00	1.00	1.00	45718.23	115346.71	-9488.18	5458.74
Extreme	1.25	1.35	0.50	47444.32	96558.16	-213.07	-46632.85

Min factor

Comb.	Factor β			Internal force at top pile		Shear force	Moment
				ΣNi	ΣMi	Qz	My
	β DC	β EV	β LS	(kN)	(kN•m)	(kN)	(kN•m)
Strength I	0.90	1.00	1.75	58216.52	141127.23	-21093.55	28680.28
Strength II	0.90	1.00	-	54436.61	133626.18	-20127.03	29619.41
Strength III	0.90	1.00	1.35	57437.08	139603.14	-20957.17	29085.38
Service	1.00	1.00	1.00	45718.23	115346.71	-9488.18	5458.74
Extreme	0.90	1.00	0.50	47444.32	96558.16	-12330.91	-9860.09

2.2.5.4. COMBINATON LOADING TO SECTION E - E

Component	Formular	Volume (m3)	ΣVz (KN)	e (m)	ΣMy (KN.m)
1. Pile cap	$V_{bm}= b_1*a_4*c_2$	0.00	0.00	1.75	0.00
2. Due to Backfill (front of abutment):	$V_s=b_{12}*a_{12}*c_3/2$	0.00	0.00	0.67	0.00
Total			**0.00**		**0.00**

COMBINATION LOADING TABLE

Max factor

Comb.	Factor β		Internal force at top pile		Shear force	Moment
			ΣNi	ΣMi	Qz	My
	β_{DC}	β_{EV}	(kN)	(kN•m)	(kN)	(kN•m)
Strength I	1.25	1.35	51456.48	90048.84	-51456.48	90048.84
Strength II	1.25	1.35	47007.40	82262.94	-47007.40	82262.94
Strength III	1.25	1.35	50523.93	88416.88	-50523.93	88416.88
Service	1.00	1.00	37397.77	65446.10	-37397.77	65446.10
Extreme	1.25	1.35	54238.69	94917.70	-54238.69	94917.70

Min factor

Comb.	Factor β		Internal force at top pile		Shear force	Moment
			ΣNi	ΣMi	Qz	My
	β_{DC}	β_{EV}	(kN)	(kN•m)	(kN)	(kN•m)
Strength I	0.90	1.00	51456.48	90048.84	-51456.48	90048.84
Strength II	0.90	1.00	47007.40	82262.94	-47007.40	82262.94
Strength III	0.90	1.00	50523.93	88416.88	-50523.93	88416.88
Service	1.00	1.00	37397.77	65446.10	-37397.77	65446.10
Extreme	0.90	1.00	54238.69	94917.70	-54238.69	94917.70

2.2.5.5. Ultimate check and shear capacity check for pile cap :

Item		Section D - D	Section E-E	Unit
• Factored Plexural moment	M_u	8092.47	90048.84	kN.m
• Factored Shear force	V_u	-8009.19	51456.48	kN
• Hight of Section	h	2500	2500	mm
• Width of section	b	26000	26000	mm
• Section area	A_c	65000000	65000000	mm^2
• Moment of inertia of concrete section	I_g	3.4E+13	3.4E+13	mm^4
• Tension reinforcement: Distance from tension reinf. to extreme compression fiber	d_c	100	164	mm
Reinf. Diameter	∅	28	28	mm
Space	@	150	150	mm
Number of bar	n	173	346	bar
Total area of reinf.	A_s	106320	213050	mm^2
• comp. reinforcement: Distance from compressive reinf. to extreme		100	142	mm
Diameter		28	28	mm
Reinf. Space		150	150	mm
Number of bar		172	172	bar
Total area of reinf.	A'_s	105909	105909	mm^2
Check Flexural Moment at Strength state				
• Resistance factor	Φ	0.90	0.90	
• The corresponding effective	d_e	2400	2336	mm
• Stress block factor	β_1	0.8357	0.84	
• Depth of the equivalent stress block = c*β1	a	64.14	128.54	mm
• Distance from extreme compression fiber to the neutral axis	c	76.75	153.81	mm
• The nominal flexural resistance:	M_n	100703	193597	kN.m
• Factored flexural resistance	$M_r = \Phi.M_n$	90633	174237	kN.m
• Check condition	$M_r > M_u$	**O.K**	**O.K**	
Mimimum Reinforcement				
• Cracking moment	1.2Mcr	112146.19	97915.22	Kn.m
• Check	Mr> min(1.2Mcr, 1.33Mu)	**O.K**	**O.K**	
Maximum Reinforcement				
• Obligation Condition	c/d_e	0.03	0.07	
• Check	$c/d_e < 0.42$	**O.K**	**O.K**	
Check shear resistance				
• Factored Shear force	V_u	-8009.19	51456.48	kN
• Resistance factor	Φ	0.90	0.90	
• The effective shear deepth	d_v	2368	2272	mm
• Effective width	b_v	26000	26000	mm
• Angle of inclination of diagonal compressive stress	θ	33	41	degree
• Angle of inclination of transerverse reinf. To longitudinal axis	α	90	90	degree
• Factor indicating ability of diagonally cracked concrete to	β	2.40	1.95	
• Value	$0.1{*}f'_c{*}b_v{*}d_v$	184698	177195	kN
• Max spacing of transverse reinforcement	s	600	600	mm
• Spacing of stirrup	s	450	450	mm
• Diameter of transverse reinforcement	∅	D 32	D 32	
• Number of transverse reinf. within distance s	n	2	2	bar
• Assume	θ	33.00	41.00	degree
• Strain in tensile reinforcement	ε_x	-1.29E-04	1.62E-03	
If ex<0, multiple with reduce factor	Φc	0.00	-	
• Ratio of shear stress and f'c	V/f'c	0.00	0.03	
• β final		2.40	1.95	
• θ final		33.00	41.00	degree
• Total area of transverse reinf.	A_v	1608	1608	mm^2
• Diameter of stirrup	∅	D 18	D 18	mm
• Number of stirrup within distance s	n	58	58	bar
• Total area of stirrup	A_v	14787.48	14787.48	
• The shear resistance of concrete:	V_c	67172.53	52360.49	kN
• The shear resistance of stirrup	V_s	28901.49	17329.66	kN

• Value	$0.25 \cdot f'_c \cdot b_v \cdot d_v$	461745.89	442987.63	kN
• The nominal shear resistance:	V_n	96074.02	69690.14	kN
• The factored shear resistance	V_r	86466.62	62721.13	kN
• Check	$V_r > V_u$	**O.K**	**O.K**	
•Requiring transverse reinforcement	$V_u > 0.5 \cdot \Phi \cdot V_c$	**Not required**	**Need**	
•Check minimum transverse reinforcement	$A_v > 0.083 \cdot (f'_c{}^{0.5}) \cdot b_v \cdot s / f_y$	**not required**	**O.K**	
Check Flexural and shear resistance at Extreme state				
• Factored Plexural moment	M_u	46632.85	94917.70	kN.m
• Factored Shear force	V_u	-213.07	54238.69	kN
• Resistance factor	Φ	1.00	1.00	
• The nominal flexural resistance:	M_n	100703	193597	kN.m
• Factored flexural resistance	$M_r = \Phi . M_n$	100703	193597	kN.m
• Check condition	$M_r > M_u$	**O.K**	**O.K**	
Check crack				
Interior force combination				
• Factored moment	M_u	5.46E+03	6.54E+04	kN.m
• Modulus of rupture of concrete	fr = 0.63*sqrt(f'c)	3.45	3.45	MPa
• Distance from extreme tension fiber to the neutral axis	y_t = h-c	2423	2346	mm
• Stress of concrete at tension fiber	f'r = Ms*y_t/I_g	0	5	MPa
• Check	f'r >	0.8*fr	0.8*fr	
		No check	check crack	
• Crack width parameter	Z	= 23000	= 23000	N/mm
• Ratio of reinf. Modulus with concrete modulus	n = E_s/E_c	= 7.00	= 7.00	
• The distance from extreme fiber to the neutral axis	c	= 400.40	= 578.20	mm
• Effective moment of inertia	J	3.53E+12	6.28E+12	mm^4
• Arm	de-c	= 1999.60	= 1757.80	mm
• Tension stress in reinforcement	fs =n*Ms*(de-c)/J	= 21.63	= 128.16	MPa
• Area of concrete having the same centroid as the principal	A	= 15058	= 7514	mm2
• Tension stress in reinforcement with service state	$f_{sa} = Z/(d_c{}^*A)^{1/3}$	= 232.85	= 293.56	Mpa
• Check condition	$f_s < f_{sa}$	**O.K**	**O.K**	
• Check condition	$f_s < 0.6^*f_y$	**O.K**	**O.K**	

2.3 PIER P7
2.3 TRỤ CẦU P7

PIER P7 - CALCULATION SHEET

(GROUP P3, P7 - FIX PIER)

CONTENT:

CALCULATION PROCEDURE & STANDARD:

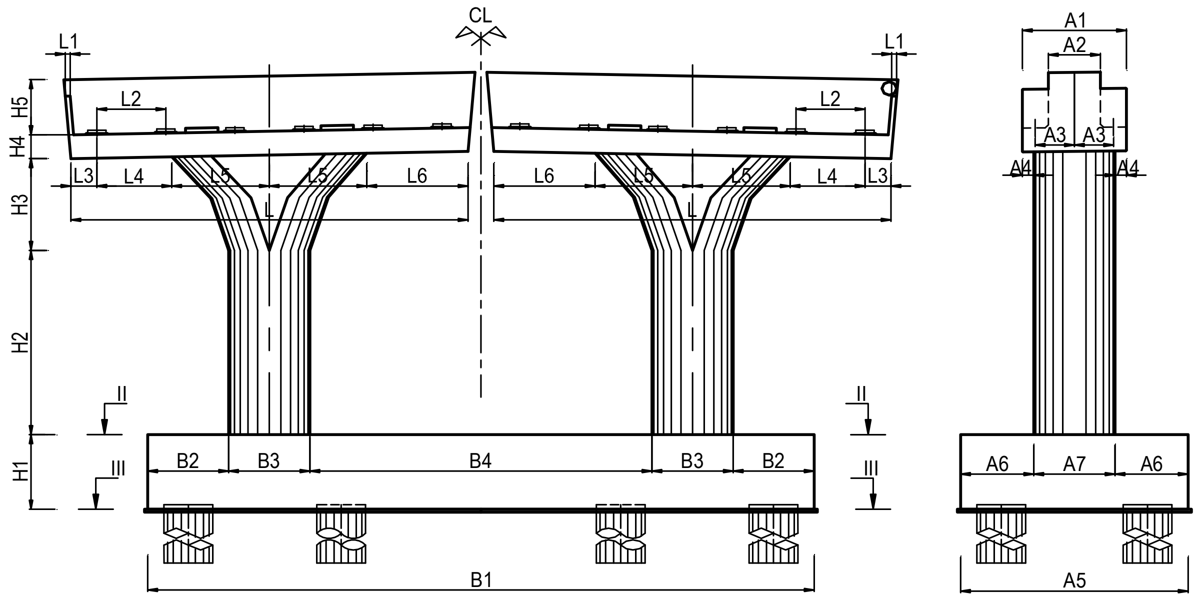
Design criteria: Bridge Design Standard 22 TCN - 272 - 05

2.3.1. GENERAL DATA:

1. Span length `	L_{left} =	38.30	(m)
	L_{right} =	38.30	(m)
2. Design live load			
Design vehicle load		HL93	22TCN 272 - 05
Number of lane		2x3	(lane)
Pedestrian		0.00	KG/m^2
3. Bridge width			
Width carriageway	B_{xe} =	12.00	(m)
Width of median guardrail	B_{pc} =	0.50	(m)
Width barrier	B_{lc} =	0.50	(m)
Bridge width	B =	13.00	(m)
4. cross sections:			
Pavement thick ness	d_{BTN} =	0.084	(m)
Deck Thickness	d_{BTCT} =	0.20	(m)
Number of Girder	n =	2x6	(m)
Girder distance	L_d =	2.12	(m)
Distance from center of outer girder to extreme of pier cap	L =	0.75	(m)
5. Material property:			
Concrete			
Compressive strength of cylindrical at 28 da	f 'c =	30.00	MPa
Concrete density	γ =	24.50	KN/m^3
Elastic modulus	Ec =	29440	MPa
Tension strength of concrete	fr =	3.45	MPa
Steel			
Concrete modulus	Es =	200000	MPa
Yeild strength of steel bar	fy =	400.00	MPa

6. THE PIER GEOMETRIC:

GENERAL DIMENTIONS OF PIER



vertical			Horizontal			Thickness		
Remark	Value	Unit	Remark	Value	Unit	Remark	Value	Unit
a_1 =	3.20	(m)	L =	12.22	(m)	h_1 =	2.50	(m)
a_2 =	1.60	(m)	L_1 =	0.15	(m)	h_2 =	8.500	(m)
a_3 =	1.20	(m)	L_2 =	2.12	(m)	h_3 =	3.000	(m)
a_4 =	0.35	(m)	L_3 =	0.80	(m)	h_4 =	0.800	(m)
a_5 =	7.00	(m)	L_4 =	3.10	(m)	h_5 =	1.850	(m)
a_6 =	2.25	(m)	L_5 =	3.00	(m)			
a_7 =	2.50	(m)	L_6 =	3.12	(m)			
			B_1 =	20.50	(m)			
			B_2 =	2.495	(m)			
			B_3 =	2.50	(m)			

1. The design elevation:

				Bearing Pad dimention		
Elevation of surface of deck	EL_{mc} =	16.288	(m)	Bearing Pad dimention		
Elevation of top of bearing	EL_{xm} =	14.75	(m)	a =	0.35	(m)
Hight water level (H1%)	EL_{MNTK} =	9.20	(m)	b =	0.7	(m)
Daily water level (H5%)	EL_{MNTB} =	4.07	(m)	d =	0.035	(m)
Existing height	EL_{TN} =	4.70	(m)			
Daily water level (H5%)	EL_{TT} =	4.07	(m)			
Section II - II (top footing)	$EL_{MC\ II\text{-}II}$ =	1.67	(m)			
Section III - III (bottom footing)	$EL_{MCIII\text{-}III}$ =	-0.826	(m)			

2.3.2. EXTERIAL FORCE INPUT TO PIER:

2.3.2.1. Dead load of super structure: DC

Load type	Vertical load **Nz**		Momen due to Left Span		Momen due to Right Span	
	Left span	Right span	e_{xt}	**My**	e_{xp}	**My**
	(KN)	(KN)	(m)	(KNm)	(m)	(KNm)
1 - Wearing surface - DW	907.25	907.25	1.200	1088.70	-1.200	-1088.70
2 - Median barrier	422.26	422.26	1.200	506.71	-1.200	-506.71
3 - Side Barrier	516.09	516.09	1.200	619.31	-1.200	-619.31
4 - Railing	0.00	0.00	1.200	0.00	-1.200	0.00
5 - Deck and permanent form (if any)	2574.36	2574.36	1.200	3089.24	-1.200	-3089.24
6 - Cross beam + jointions	268.90	268.90	1.200	322.68	-1.200	-322.68
7 - Super - T girder	4280.64	4280.64	1.200	5136.77	-1.200	-5136.77
Total	**8969.50**	**8969.50**		**10763.40**		**-10763.40**

2.3.2.2. Dead load of pier: DC

Load type	Mass	Unit Weight	Nz Load
	(m3)	(KN/m3)	(KN)
1 - Bearing stone	0.09	24.50	2.10
2 - Pier cap	136.61	24.50	3346.88
3 - Pier body	139.16	24.50	3409.420
4 - Footing	358.75	24.50	8789.38
Total			**15547.78**

2.3.2.3. Live Load: LL + IM

1. Design live load: HL-93

		Truck with 3 axle		Tandem		Lane load		
Weights of axles:	P1 =	35.00	(KN)	110.00	(KN)	W_L =	9.30	(KN/m)
	P2 =	145.00	(KN)	110.00	(KN)			
	P3 =	145.00	(KN)			Pedestrian load		
Spacings of axle	V1 =	4.30	(m)	1.20	(m)	W_N =	0.00	(KN/m^2)
	V2 =	4.30	(m)			Ble =	0.00	(m)

Eccentric (longitudinal) - Left span e^{Left}_x = 1.200 (m)

- Right span e^{Right}_x = -1.200 (m)

Number of lane K = 2x3 (lane)

Dynamic load factor IM = 1.25

2 Live load is mentioned in some case below:

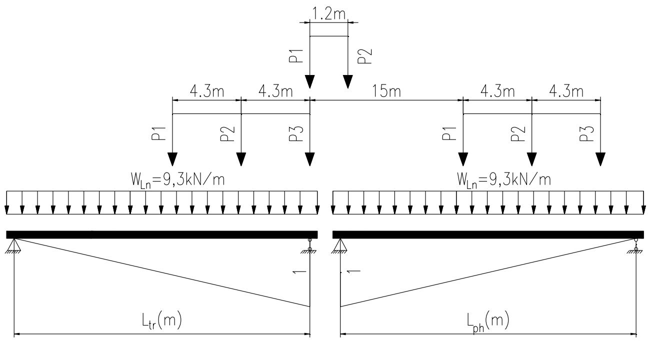
Live load applied on each spans follow below formular:

$$N_z^{Truck} = [P_3+P_2(L - V_2) / L+P_1 (L - V_1 - V_2) / L]*m* K * IM$$

$$N_z^{lane\ load} = 0.5*L*W_L*m*K \qquad N_z^{Tandem} = [P2+P1 (L-V1) / L] *m*K*IM$$

$$N_z^{Pedestrian} = 0.5*L*W_N*Ble*2$$

Caculation diagram



2.1 Live load effect on two spans - All vehicle lane. (Max Nz) - (Strength limit state IA)

Number of lane in this case k = 2x3 (lane)

Lane factor m = 0.85

Eccentric (Horizontal) - for vehicle $e^{vehicle}$ = 0.00 (m)

Live load type	**Left span**	*37.60*	*(m)*	**Right span**	*37.60*	*(m)*
	Nz	Mx =Nz*e_y	My =Nz*e_x	Nz	Mx =Nz*e_y	My =Nz*e_x
	(KN)	(KNm)	(KNm)	(KN)	(KNm)	(KNm)
1 - Truck	1915.13	0.00	2298.15	795.94	0.00	-955.13
2 - Tandem	701.25	0.00	841.50	723.63	0.00	-868.36
3 - Lane Load	891.68	0.00	1070.02	891.68	0.00	-1070.02
HL - 93	**2806.81**	**0.00**	**3368.17**	**1687.63**	**0.00**	**-2025.15**

Live load value:

Nz =	**4494.44**	**(KN)**
Mx =	**0.00**	**(KNm)**
My =	**1343.02**	**(KNm)**

2.2 Live load effect on two spans - Vehicle on 1/2 Bridge width (Max Mx) - (Strength IB)

Number of lane in this case K = 3.00 (lane)

Lane factor m = 0.85

Eccentric (Horizontal) - for vehicle $e^{vehicle}_y$ = 10.250 (m)

Live Load	**Left span**	*37.60*	*(m)*	**Right span**	*37.60*	*(m)*
	Nz	Mx =Nz*e_y	My =Nz*e_x	Nz	Mx =Nz*e_y	My =Nz*e_x
	(KN)	(KNm)	(KNm)	(KN)	(KNm)	(KNm)
1 - Truck	957.56	9815.03	1149.08	397.97	4079.21	-477.57
2 - Tandem	350.63	3593.91	420.75	361.82	3708.61	-434.18
3 - Lane Load	445.84	4569.88	535.01	445.84	4569.88	-535.01
HL - 93	**1403.41**	**14384.91**	**1684.09**	**843.81**	**8649.09**	**-1012.58**

Live load value:

Nz =	**2247.22**	**(KN)**
Mx =	**23034.00**	**(KNm)**
My =	**671.51**	**(KNm)**

2.3 Live load effect on one Span (Lmax) - All Lane (Max My) - (Strength IC)

Number of lane in this case K = 2x3 (lane)

Lane factor m = 0.85

Eccentric (Horizontal) - for vehicle $e^{vehicle}_y$ = 0.00 (m)

Eccentric (longitudinal)- Max Span e^{Lmax}_x = 1.200 (m)

Live Load	**Lmax**	*37.60*	*(m)*	
	Nz	Mx =Nz*e_y	My =Nz*e_x	
	(KN)	(KNm)	(KNm)	
1 - Truck	1915.13	0.00	2298.15	Live load value:
2 - Tandem	1380.12	0.00	1656.14	
3 - Lane Load	891.68	0.00	1070.02	**Nz = 2806.81 (KN)**
HL - 93	**2806.81**	**0.00**	**3368.17**	**My = 3368.17 (KNm)**

2.3.2.4. Breaking Force: BR

Breadking force is 25% of total weight of truck axles or of tandem on all lanes

These force shall be assumed to act horizontally at a distance of 1800mm above roadway surface

Section ID	Lane No.	Truck	Tandem	Lane	Truck	Tandem	Z	**My**
	(Lane)	(KN)	(KN)	Factor	(KN)	(KN)	(m)	**(KNm)**
II	2x3	325.00	220.00	0.85	414.38	280.50	16.41	**6801.55**
III	2x3	325.00	220.00	0.85	414.38	280.50	18.91	**7837.49**

2.3.2.5. Stream pressure : WA

$$P_{(D,L)} = 5.14 * 10^{-4} * C_{D(L)} * V^2 \quad \text{(Mpa)}$$

In Which: C_D : Drag coefficient in longitudinal direction.

C_D = 0.70 With circular pier cap

C_L : Drag coefficient in transverse direction

C_L = 0.50

V : Designed velocity of water

V = 2.54 (m/s)

=> Horizontal Stream pressure P_D = 0.00231 (Mpa)

P_D = 2.312 (KN/m^2)

=> Longitudinal Stream pressure P_L = 0.000145 (Mpa)

P_L = 0.145 (KN/m^2)

Water pressure acting on the section follow table below:

Section ID	Horizontal			Longitudinal			Momen		
	Pressure	Area	**Force**	Pressure	Area	**Water Force**	Z	**Mx**	**My**
	(KN/m^2)	(m2)	**(KN)**	(KN/m^2)	(m2)	**(KN)**	(m)	**(KNm)**	**(KNm)**
II	2.312	29.55	**68.34**	0.145	29.55	**4.27**	3.76	**257.18**	**16.07**
III	2.312	47.05	**108.81**	0.145	80.80	**11.68**	5.01	**545.47**	**58.54**

2.3.2.6. Win Load:

6.1 Win load on Structure: WS

(a) Designed win speed:

Designed win velocity follow formula: **$V = V_B . S$**

Trong âoï: V_B : basic 3 second gust wind velocity.

V_B = 53 (m/s) (Zone II)

S : correction factor for upwind terrain and deck height.
S = 1.09
Designed wind velocity. **V = 57.77 (m/s)**

(b) *Transverse wind load:* **$P_D = 0.0006 . V^2 . A_t . C_d$**

Inwhich: V : Designed wind velocity
C_d : drag coefficient specified, it's depened rario b/d.
b: overall width of bridge between outer faces of parapets.
b = 13.00 (m)
d: depth of superstructure, include solid parapets if applicable.
d = 3.11 (m)
Ratio (b / d) = 4.17 **C_d = 1.40**
A_t : Area of the structure for calculation of transverse wind load.
A_t = $A_t^{Rigth\ Span}$ = ($L^{Left\ Span}$ x d_T)
A_t = 119.27 (m^2)

Transverse wind load: **P_D^{trans} = 334.35 (KN)**

(c) *Longitudinal wind load:*

For superstructure with solid elevation, a longitudinal wind load equal to 0.25 times the transverse wind load calculated

Longitudinal wind load: **P_D^{Long} = 83.59 (KN)**

(d) *The wind load applied on the sections:*

Section	Transverse wind load	Longitudinal wind load	Height	Moment	
	P_D^{Trans}	P_D^{Long}	Z	Mx	My
	(KN)	(KN)	(m)	(KNm)	(KNm)
II	**334.35**	**83.59**	14.63	**4891.21**	**1222.80**
III	**334.35**	**83.59**	17.13	**5727.08**	**1431.77**

6.2 Wind load on vehicles: WL

(a) *Line wind load*

Transeverse direction: p_y = 1.50 (KN/m) A 3.8.1.3 22TCN 272-05
Longitudinal direction: p_x = 0.75 (KN/m)
Wind load assumed 1.8m above the r di = 1.80 (m)

(b) *Wind load*

Transverse direction _ Left spa P_D = 56.40 (KN)
_ Right span: P_D = 56.40 (KN)
Total: P^{Trans}_D = 112.80 (KN)

Longitudinal direction _ Left span: P_D = 28.20 (KN)
_ Right span: P_D = 28.20 (KN)
Total: P^{Long}_D = 56.40 (KN)

(c) *Wind load on vehicles:*

Section	Transverse wind load	Longitudinal wind load	Height	Moment	
	P_D^{Trans}	P_D^{Long}	Z	Mx	My
	(KN)	(KN)	(m)	(KNm)	(KNm)
II	112.80	56.40	16.41	3703.00	1851.50
III	112.80	56.40	18.91	4267.00	2133.50

2.3.2.7. Earthquake effects: EQ

Earthquake class: Class 7.00
Acceleration coefficient A = 0.034
Site coefficient S = 1.20
R : Response modification factors R = 1.00
Elastic modulus Ec = 29440 Mpa
Selftweight of super-structure applied on piers
Wt = 234.19 KN/m

(a) *Determined stiffness:* **K x(y) = 3.E.I x(y) / H^3**

(b) *Displacement accodance with unit force* **Vs x(y) = Po . L / K x(y)**

(c) *Determined factor a, b, g :*

$$\alpha = \int_{T(n+1)}^{T(n-1)} v_s(x)dx \qquad \beta = \int_{T(n+1)}^{T(n-1)} W_{(x)}V_s(x)dx \qquad \gamma = \int_{T(n+1)}^{T(n-1)} W_{(x)}V_s(x)^2 dx$$

(d) *Determination of period of vibration*

$$T = 2\pi\sqrt{\frac{\gamma}{P_0.g.\alpha}}$$

The summation of result :

Section	H	Ix	Iy	Kx	Ky	Vsx	Vsy
	(m)	(m^4)	(m^4)	(KN/m)	(KN/m)	(m)	(m)
II	13.07	3.83	3.83	151633.15	151633.15	0.0013	0.0003
III	15.57	3.83	3.83	151633.15	151633.15	0.0013	0.0003

Section	H	α_x	α_y	βx	βy	γ_x	γ_y	Tx	Ty
	(m)	(m^2)	(m^2)	KNm	KNm	KNm2	KNm2	(s)	(s)
II	13.07	0.242	0.010	56.639	2.266	0.072	0.0006	1.09	0.49
III	15.57	0.242	0.010	56.639	2.266	0.072	0.0006	1.09	0.49

Inwhich:

Tx, Ty: The period of vibration follow direct X and direct Y.

H: The distance from deck slab to consided sections (m)

$\int_{T(n+1)}^{T(n-1)} xd(x) = L$ The length of Superstructure applied to pier (m)

Ec: Reinforcement concrete elastic modulus. (Mpa)

Ix, Iy: Moment of inertia follow X axial, and follow Y axial. (m^4)

(e) Determination of elastic seismic response coefficient: $C_{sm} = \frac{1{,}2.A.S}{T_m^{2/3}} \le 2{,}5A$

(f) Equivalent uniform static seismic loading $P_e(x) = \frac{\beta C_{sm}}{\gamma} W(x).V(x)$

(g) Designed force applied on pier due to earthquake effect: **F = $Pe_{(x)}$ L / R**

Section	C_x sm	C_y sm	C_x sm	C_y sm	Pe(x)	Pe(y)	Hx	Hy	Z
	Follow theory		Compared with 2.5A		(KN/m)	(KN/m)	(KN)	(KN)	(m)
II	0.046	0.079	0.046	0.079	**10.82**	**18.49**	**2071.19**	**708.33**	**13.07**
III	0.046	0.079	0.046	0.079	**10.82**	**18.49**	**2071.19**	**708.33**	**15.57**

Section	Hx	Hy	Z	Mx	My
	(KN)	(KN)	(m)	(KNm)	(KNm)
II	**2071.19**	**708.33**	**13.07**	**9259.35**	**27074.64**
III	**2071.19**	**708.33**	**15.57**	**11030.18**	**32252.63**

(h) Earthquake effect due to substructure

Section II - II

The Component	Nz	C_x sm	C_y sm	Hx	Hy	Z	Mx	My
	(KN)			**(KN)**	**(KN)**	(m)	**(KNm)**	**(KNm)**
1 - Pier cap	3346.88	0.046	0.079	154.57	264.31	12.83	1982.35	3389.75
2 - Pier shaft	3409.42	0.046	0.079	157.46	269.25	5.75	905.38	1548.17
Total	**6756.30**			**312.03**	**533.56**		**2887.73**	**4937.92**

Section III - III

The Component	Nz	C_x sm	C_y sm	Hx	Hy	Z	Mx	My
	(KN)			**(KN)**	**(KN)**	(m)	**(KNm)**	**(KNm)**
1 - Pier cap	3346.88	0.05	0.08	154.57	264.31	15.33	2368.77	4050.52
2 - Pier shaft	3409.42	0.05	0.08	157.46	269.25	8.25	1299.02	2221.29
4 - Pile cap	8789.38	0.05	0.08	405.92	694.11	1.25	507.40	867.64
Total	**15545.68**			**717.95**	**1227.66**		**4175.20**	**7139.45**

(i) Designed load due to earthquake effect applied on the sections

Section	Hx	Hy	Mx	My
	(KN)	(KN)	(KN.m)	(KN.m)
II - II	**2383.22**	**1241.89**	**12147.08**	**32012.57**
III - III	**2789.14**	**1936.00**	**15205.38**	**39392.07**

2.3.2.8. Vessel Collision: CV (3.14 - 22TCN272-05)

Class of navigable waterway:	Class	IV	
Mean annual stream velocity:	V_{bq} =	1.15	(m/s)
Design vessel tonnage:	Self-propelled vessel	200.00	(DWT)
	Towed barge	400.00	(DWT)
Dimensions of design vessel:	Maximum length	34.00	(m)
	Maximum breadth	6.60	(m)
	Laden Draught	1.70	(m)
Design collision velocity	Self-propelled vessel	3.65	(m/s)
	Towed barge	2.75	(m/s)

(a) Collision energy of Towed barge:

KE = 500 C_H M V^2 = 1588125 (J)

M Towed barge displacement tonnage M = 400.00 (Mg)

C_H Hydrodynamic mass coefficient C_H = 1.05

Barge bow damage length

$$a_B = 3100.(\sqrt{1+1,3x10^{-7}KE}-1) =$$ 305.00 (mm)

Barge collision force on pier **P_B = 6488.00 (KN)**

(b) Vessel collision force pier:

It's detemined follow formula

V Design collision velocity (m/s)

DWT Deadweight tonnage of vessel (Mg)

$$P_s = 1,2\times10^5 V\sqrt{DWT}$$

Vessel collision force pier: **P_S = 6194.26 (KN)**

On basic of comparison of vessel conllision force beetwen Seft-ropelled vessel and Towed barge, Chose follow table below

The Force follow Perpendicular direction to centerline : CV = 100% max(P_B,P_S)

The force follow longitudinal direction of bridge: CV = 50% max(P_B,P_S)

Vessel collision force (CV) applied to pier at daily water level.

Section	H_x	H_y	Z	Mx	My
	(KN)	(KN)	(m)	(KNm)	(KNm)
II	3244.00	6488.00	2.40	15545.26	7772.63
III	3244.00	6488.00	4.90	31765.27	15882.64

2.3.3. COMBINATION OF EXTERNAL FORCE - SECTION II - II

	LOAD TYPE & NOTATIONS		Factor γ	Nz (KN)	Hx (KN)	Hy (KN)	Mx (KNm)	My (KNm)
Ia	**STRENGTH LIMIT STATE I_A**							
1	DC	Dead load of left span	1.25	10077.81	-	-	-	12093.38
2	DC	Dead load of right span	1.25	10077.81	-	-	-	-12093.38
3	DW	Dead load of wearing surface	1.50	2721.75	-	-	-	0.00
4	DC	Dead load of pier	1.25	8448.01	-	-	-	-
5	LL	Live load - (maxNz)	1.75	7865.27	-	-	0.00	2350.29
6	BR	Braking force (Start)	1.75	-	725.16	-	-	11902.71
7	PL	Pedestrian live load	1.75	0.00	-	-	-	0.00
8	WA	Water load and stream pressure	1.00	-	4.27	68.34	257.18	16.07
9	FR	Friction	1.00	-	0.00	-	-	0.00
10	TU+CR+SH	Temperature + Creep + Shinkage	0.50	-	-	-	-	-
	TOTAL			**39190.65**	**729.43**	**68.34**	**257.18**	**14269.08**

Ib	**STRENGTH LIMIT STATE I_B**							
1	DC	Dead load of left span	0.90	7256.03	-	-	-	8707.23
2	DC	Dead load of right span	0.90	7256.03	-	-	-	-8707.23
3	DW	Dead load of wearing surface	0.65	1179.43	-	-	-	0.00
4	DC	Dead load of pier	0.90	6082.56	-	-	-	-
5	LL	Vehicle Live load - (maxMx)	1.75	3932.63	-	-	40309.50	1175.14
7	BR	Braking force (Start)	1.75	-	725.16	-	-	11902.71
8	PL	Pedestrian	1.75	0.00	-	-	0.00	0.00
9	WA	Water load and stream pressure	1.00	-	4.27	68.34	257.18	16.07
10	FR	Friction	1.00	-	0.00	-	-	0.00
11	TU+CR+SH	Temperature + Creep + Shinkage	0.50	-	-	-	-	-
	TOTAL			**25706.68**	**729.43**	**68.34**	**40566.67**	**13093.93**

Ic	**STRENGTH LIMIT STATE I_C**							
1	DC	Dead load of left span	0.90	7256.03	-	-	-	8707.23
2	DC	Dead load of right span	0.90	7256.03	-	-	-	-8707.23
3	DW	Dead load of wearing surface	0.65	1179.43	-	-	-	0.00
4	DC	Dead load of pier	0.90	6082.56	-	-	-	-
5	LL	Vehicle Live load - (maxMy)	1.75	4911.92	-	-	0.00	5894.31
7	BR	Braking force (Start)	1.75	-	725.16	-	-	11902.71
8	PL	Pedestrian live load	1.75	0.00	-	-	-	0.00
9	WA	Water load and stream pressure	1.00	-	4.27	68.34	257.18	16.07
10	FR	Friction	1.00	-	0.00	-	-	0.00
11	TU+CR+SH	Temperature + Creep + Shinkage	0.50	-	-	-	-	-
	TOTAL			**26685.96**	**729.43**	**68.34**	**257.18**	**17813.09**

II	**STRENGTH LIMIT STATE II**							
1	DC	Dead load of left span	1.25	10077.81	-	-	-	12093.38
2	DC	Dead load of right span	1.25	7256.03	-	-	-	-12093.38
3	DW	Dead load of wearing surface	1.50	2721.75	-	-	-	0.00
4	DC	Dead load of pier	1.25	8448.01	-	-	-	0.00
5	WA	Water load and stream pressure	1.00	-	4.27	68.34	257.18	16.07
6	WS	Wind load on structure	1.40	-	117.02	468.09	6847.69	1711.92
7	FR	Friction	1.00	-	0.00	-	-	0.00
8	TU+CR+SH	Temperature + Creep + Shinkage	0.50	-	-	-	-	-
	TOTAL			**28503.60**	**121.29**	**536.43**	**7104.87**	**1727.99**

IIIa	**STRENGTH LIMIT STATE III_A**							
1	DC	Dead load of left span	1.25	10077.81	-	-	-	12093.38
2	DC	Dead load of right span	1.25	10077.81	-	-	-	-12093.38
3	DW	Dead load of wearing surface	1.50	2721.75	-	-	-	0.00
4	DC	Dead load of pier	1.25	8448.01	-	-	-	0.00
5	LL	Live load - (maxNz)	1.35	6067.49	-	-	0.00	1813.08
7	BR	Braking force (Start)	1.35	-	559.41	-	-	9182.09
8	PL	Pedestrian live load	1.35	0.00	-	-	-	0.00
9	WA	Water load and stream pressure	1.00	-	4.27	68.34	257.18	16.07
10	WS	Wind load on structure	0.40	-	33.44	133.74	1956.48	489.12
11	WL	Wind load on vehicle	1.00	-	56.40	112.80	3703.00	1851.50
13	FR	Bearing Friction	1.00	-	0.00	-	-	0.00
14	TU+CR+SH	Temperature + Creep + Shinkage	0.50	-	-	-	-	-
	TOTAL			**37392.88**	**653.51**	**314.88**	**5916.66**	**13351.87**

IIIb	STRENGTH III_B							
1	DC	Dead load of left span	0.90	7256.03	-	-	-	8707.23
2	DC	Dead load of right span	0.90	7256.03	-	-	-	-8707.23
3	DW	Dead load of wearing surface	0.65	1179.43	-	-	-	0.00
4	DC	Dead load of pier	0.90	6082.56	-	-	-	0.00
5	LL	Live load - (max Mx)	1.35	3033.75	-	-	31095.90	906.54
7	BR	Braking force (Start)	1.35	-	559.41	-	-	9182.09
8	PL	Pedestrian live load	1.35	0.00	-	-	0.00	0.00
9	WA	Water load and stream pressure	1.00	-	4.27	68.34	257.18	16.07
10	WS	Wind load on structure	0.40	-	33.44	133.74	1956.48	489.12
11	WL	Wind load on vehicle	1.00	-	56.40	112.80	3703.00	1851.50
13	FR	Bearing Friction	1.00	-	0.00	-	-	0.00
14	TU+CR+SH	Temperature + Creep + Shinkage	0.50	-	-	-	-	-
	TOTAL			**24807.79**	**653.51**	**314.88**	**37012.55**	**12445.33**

IIIc	STRENGTH III_C							
1	DC	Dead load of left span	0.90	7256.03	-	-	-	8707.23
2	DC	Dead load of right span	0.90	7256.03	-	-	-	-8707.23
3	DW	Dead load of wearing surface	0.65	1179.43	-	-	-	0.00
4	DC	Dead load of pier	0.90	6082.56	-	-	-	0.00
5	LL	Live load - (max My)	1.35	3789.20	-	-	0.00	4547.04
7	BR	Braking force (Start)	1.35	-	559.41	-	-	9182.09
8	PL	Pedestrian live load	1.35	0.00	-	-	-	0.00
9	WA	Water load and stream pressure	1.00	-	4.27	68.34	257.18	16.07
10	WS	Wind load on structure	0.40	-	33.44	133.74	1956.48	489.12
11	WL	Wind load on vehicle	1.00	-	56.40	112.80	3703.00	1851.50
13	FR	Bearing Friction	1.00	-	0.00	-	-	0.00
14	TU+CR+SH	Temperature + Creep + Shinkage	0.50	-	-	-	-	-
	TOTAL			**25563.24**	**653.51**	**314.88**	**5916.66**	**16085.82**

IV.A	EXTREME EVENT I_A (100% EQ long + 30% EQ trans)							
1	DC	Dead load of left span	1.25	10077.81	-	-	-	12093.38
2	DC	Dead load of right span	1.25	10077.81	-	-	-	-12093.38
3	DW	Dead load of wearing surface	1.50	2721.75	-	-	-	0.00
4	DC	Dead load of pier	1.25	8448.01	-	-	-	0.00
5	LL	Live load - (maxNz)	0.50	2247.22	-	-	0.00	671.51
7	BR	Braking force (Start)	0.50	-	207.19	-	-	3400.78
8	PL	Pedestrian live load	0.50	0.00	-	-	-	0.00
9	WA	Water load and stream pressure	1.00	-	4.27	68.34	257.18	16.07
10	FR	Bearing Friction	1.00	-	0.00	-	-	0.00
11	EQ	Earth Quake Load	1.00	-	2383.22	372.57	3644.12	32012.57
	TOTAL			**33572.60**	**2594.68**	**440.91**	**3901.30**	**36100.92**

IV. B	EXTREME EVENT I_B (100% EQ long + 30% EQ trans)							
1	DC	Dead load of left span	0.90	7256.03	-	-	-	8707.23
2	DC	Dead load of right span	0.90	7256.03	-	-	-	-8707.23
3	DW	Dead load of wearing surface	0.65	1179.43	-	-	-	0.00
4	DC	Dead load of pier	0.90	6082.56	-	-	-	0.00
5	LL	Live load - (maxMy)	0.50	1403.41	-	-	0.00	1684.09
6	BR	Braking force (Start)	0.50	-	207.19	-	31096	0.00
7	PL	Pedestrian live load	0.50	0.00	-	-	-	0.00
8	WA	Water load and stream pressure	1.00	-	4.27	68.34	257.18	16.07
9	FR	Bearing Friction	1.00	-	0.00	-	-	0.00
10	EQ	Earth Quake Load	1.00	-	2383.22	372.57	3644.12	32012.57
	TOTAL			**23177.45**	**2594.68**	**440.91**	**34997.20**	**33712.72**

IV. C	EXTREME EVENT I_C (30% EQ long + 100% EQ trans)							
1	DC	Dead load of left span	1.25	10077.81	-	-	-	12093.38
2	DC	Dead load of right span	1.25	10077.81	-	-	-	-12093.38
3	DW	Dead load of wearing surface	1.50	2721.75	-	-	-	0.00
4	DC	Dead load of pier	1.25	8448.01	-	-	-	0.00
5	LL	Live load - (maxNz)	0.50	2247.22	207	-	-	3401
6	BR	Braking force (Start)	0.50	-	-	-	-	-
7	PL	Pedestrian live load	0.50	-	-	-	-	-
8	WA	Water load and stream pressure	1.00	-	4	68	257	16
9	FR	Bearing Friction	1.00	-	-	-	-	-
10	EQ	Earth Quake Load	1.00	-	715	1242	12147	9604
	TOTAL			**33572.60**	**926.42**	**1310.23**	**12404.25**	**13020.62**

IV. D	**EXTREME EVENT I_D (30% EQ long + 100% EQ trans)**							
1	DC	Dead load of left span	0.90	7256.03	-	-	-	8707.23
2	DC	Dead load of right span	0.90	7256.03	-	-	-	-8707.23
3	DW	Dead load of wearing surface	0.65	1179.43	-	-	-	0.00
4	DC	Dead load of pier	0.90	6082.56	-	-	-	0.00
5	LL	Live load - (maxMx)	0.50	1123.61	-	-	11517	336
6	BR	Braking force (Start)	0.50	-	-	-	-	-
7	PL	Pedestrian live load	0.50	-	-	-	-	-
8	WA	Water load and stream pressure	1.00	-	4	68	257	16
9	FR	Bearing Friction	1.00	-	-	-	-	-
10	EQ	Earth Quake Load	1.00	-	715	1242	12147	9604
	TOTAL			**22897.65**	**719.24**	**1310.23**	**23921.25**	**9955.60**

V	**EXTREME EVENT II_A**							
1	DC	Dead load of left span	1.25	10077.81	-	-	-	12093.38
2	DC	Dead load of right span	1.25	10077.81	-	-	-	-12093.38
3	DW	Dead load of wearing surface	1.50	2721.75	-	-	-	0.00
4	DC	Dead load of pier	1.25	8448.01	-	-	-	0.00
5	LL	Live load - (maxNz)	0.50	2247.22	-	-	0.00	671.51
7	BR	Braking force (Start)	0.50	-	207.19	-	-	3400.78
8	PL	Pedestrian live load	0.50	0.00	-	-	-	0.00
9	WA	Water load and stream pressure	1.00	-	4.27	68.34	257.18	16.07
11	FR	Bearing Friction	1.00	-	0.00	-	-	0.00
12	CV	Vessels Impact	1.00	-	3244.00	6488.00	15545.26	7772.63
	TOTAL			**33572.60**	**3455.46**	**6556.35**	**15802.44**	**11860.99**

V	**EXTREME EVENT II_B**							
1	DC	Dead load of left span	0.90	7256.03	-	-	-	8707.23
2	DC	Dead load of right span	0.90	7256.03	-	-	-	-8707.23
3	DW	Dead load of wearing surface	0.65	1179.43	-	-	-	0.00
4	DC	Dead load of pier	0.90	6082.56	-	-	-	0.00
5	LL	Live load - (maxMx)	0.50	1123.61	-	-	11517.00	335.76
6	BR	Braking force (Start)	0.50	-	-	-	-	-
7	PL	Pedestrian live load	0.50	-	-	-	-	0.00
8	WA	Water load and stream pressure	1.00	-	4.27	68.34	257.18	16.07
9	FR	Bearing Friction	1.00	-	-	-	-	-
10	CV	Vessels Impact	1.00	-	3244	6488	15545	7773
	TOTAL			**22897.65**	**3248.27**	**6556.35**	**27319.43**	**8124.46**

VII	**SERVICE STATE I_A**							
1	DC	Dead load of left span	1.00	8062.25	-	-	-	9674.70
2	DC	Dead load of right span	1.00	8062.25	-	-	-	-9674.70
3	DW	Dead load of wearing surface	1.00	1814.50	-	-	-	0.00
4	DC	Dead load of pier	1.00	6758.41	-	-	-	0.00
5	LL	Live load - (max Nz)	1.00	4494.44	-	-	0.00	1343.02
7	BR	Braking force (Start)	1.00	-	414.38	-	-	6801.55
8	PL	Pedestrian live load	1.00	0.00	-	-	-	0.00
9	WA	Water load and stream pressure	1.00	-	4.27	68.34	257.18	16.07
10	WS	Wind load on structure	0.30	-	25.08	100.31	1467.36	366.84
11	WL	Wind load on vehicle	1.00	-	56.40	112.80	3703.00	1851.50
13	FR	Bearing Friction	1.00	-	0.00	-	-	0.00
14	TU+CR+SH	Temperature + Creep + Shinkage	1.00	-	-	-	-	-
	TOTAL			**29191.85**	**500.12**	**281.45**	**5427.54**	**10378.99**

VIII	**SERVICE STATE I_B**							
1	DC	Dead load of left span	1.00	8062.25	-	-	-	9674.70
2	DC	Dead load of right span	1.00	8062.25	-	-	-	-9674.70
3	DW	Dead load of wearing surface	1.00	1814.50	-	-	-	0.00
4	DC	Dead load of pier	1.00	6758.41	-	-	-	0.00
5	LL	Live load - (max Mx)	1.00	2247.22	-	-	23034.00	671.51
7	BR	Braking force (Start)	1.00	-	414.38	-	-	6801.55
8	PL	Pedestrian live load	1.00	0.00	-	-	-	0.00
9	WA	Water load and stream pressure	1.00	-	4.27	68.34	257.18	16.07
10	WS	Wind load on structure	0.30	-	25.08	100.31	1467.36	366.84
11	WL	Wind load on vehicle	1.00	-	56.40	112.80	3703.00	1851.50
13	FR	Bearing Friction	1.00	-	0.00	-	-	0.00
14	TU+CR+SH	Temperature + Creep + Shinkage	1.00	-	-	-	-	0.00
	TOTAL			**26944.63**	**500.12**	**281.45**	**28461.53**	**9707.47**

IX	SERVICE STATE I_A							
1	DC	Dead load of left span	1.00	8062.25	-	-	-	9674.70
2	DC	Dead load of right span	1.00	8062.25	-	-	-	-9674.70
3	DW	Dead load of wearing surface	1.00	1814.50	-	-	-	0.00
4	DC	Dead load of pier	1.00	6758.41	-	-	-	0.00
5	LL	Live load - (max My)	1.00	2806.81	-	-	0.00	3368.17
7	BR	Braking force (Start)	1.00	-	414.38	-	-	6801.55
8	PL	Pedestrian live load	1.00	0.00	-	-	-	0.00
9	WA	Water load and stream pressure	1.00	-	4.27	68.34	257.18	16.07
10	WS	Wind load on structure	0.30	-	25.08	100.31	1467.36	366.84
11	WL	Wind load on vehicle	1.00	-	56.40	112.80	3703.00	1851.50
13	FR	Bearing Friction	1.00	-	0.00	-	-	0.00
14	TU+CR+SH	Temperature + Creep + Shinkage	1.00	-	-	-	-	0.00
	TOTAL			**27504.22**	**500.12**	**281.45**	**5427.54**	**12404.14**

COMBINATION EXTERNAL FORCE ON SECTION II - II

State	Load factors specified			Nz	Hx	Hy	Mx	My
	η_D	η_R	η_I	(KN)	(KN)	(KN)	(KNm)	(KNm)
Strength I_A	1.00	1.00	1.00	39191	729	68	257	14269
Strength I_B	1.00	1.00	1.00	25707	729	68	40567	13094
Strength I_C	1.00	1.00	1.00	26686	729	68	257	17813
Strength II	1.00	1.00	1.00	28504	121	536	7105	1728
Strength III_A	1.00	1.00	1.00	37393	654	315	5917	13352
Strength III_B	1.00	1.00	1.00	24808	654	315	37013	12445
Strength III_C	1.00	1.00	1.00	25563	654	315	5917	16086
Extreme Event I_A	1.00	1.00	1.00	33573	2595	441	3901	36101
Extreme Event I_B	1.00	1.00	1.00	23177	2595	441	34997	33713
Extreme Event I_C	1.00	1.00	1.00	33573	926	1310	12404	13021
Extreme Event I_D	1.00	1.00	1.00	22898	719	1310	23921	9956
Extreme Event II_A	1.00	1.00	1.00	33573	3455	6556	15802	11861
Extreme Event II_B	1.00	1.00	1.00	22898	3248	6556	27319	8124
Service state I_A	1.00	1.00	1.00	29192	500	281	5428	10379
Service state I_B	1.00	1.00	1.00	26945	500	281	28462	9707
Service state I_C	1.00	1.00	1.00	27504	500	281	5428	12404

2.3.4. COMBINATION OF EXTERNAL FORCE - SECTION III - III

	LOAD TYPE & NOTATIONS		Factor γ	Nz (KN)	Hx (KN)	Hy (KN)	Mx (KNm)	My (KNm)
Ia	**STRENGTH STATE I_A**							
1	DC	Dead load of left span	1.25	10077.81	-	-	-	12093.38
2	DC	Dead load of right span	1.25	10077.81	-	-	-	-12093.38
3	DW	Dead load of wearing surface	1.50	2721.75	-	-	-	0.00
4	DC	Dead load of pier	1.25	19434.73	-	-	-	-
5	LL	Live load - (maxNz)	1.75	7865.27	-	-	0.00	2350.29
7	BR	Braking force (Start)	1.75	-	725.16	-	-	13715.61
8	PL	Pedestrian live load	1.75	0.00	-	-	0.00	0.00
9	WA	Water load and stream pressure	1.00	-	11.68	108.81	545.47	58.54
10	FR	Friction	1.00	-	0.00	-	-	0.00
11	TU+CR+SH	Temperature + Creep + Shinkage	0.50	-	-	-	-	-
	TOTAL			**50177.37**	**736.83**	**108.81**	**545.47**	**16124.43**

Ib	**STRENGTH STATE I_B**							
1	DC	Dead load of left span	1.25	10077.81	-	-	-	12093.38
2	DC	Dead load of right span	1.25	10077.81	-	-	-	-12093.38
3	DW	Dead load of wearing surface	1.50	2721.75	-	-	-	0.00
4	DC	Dead load of pier	1.25	19434.73	-	-	-	-
5	LL	Live load - (max Mx)	1.75	3932.63	-	-	40309.50	1175.14
7	BR	Braking force (Start)	1.75	-	725.16	-	-	13715.61
8	PL	Pedestrian live load	1.75	0.00	-	-	0.00	0.00
9	WA	Water load and stream pressure	1.00	-	11.68	108.81	545.47	58.54
10	FR	Friction	1.00	-	0.00	-	-	0.00
11	TU+CR+SH	Temperature + Creep + Shinkage	0.50	-	-	-	-	-
	TOTAL			**46244.74**	**736.83**	**108.81**	**40854.97**	**14949.29**

Ic	**STRENGTH STATE I_C**							
1	DC	Dead load of left span	1.25	10077.81	-	-	-	12093.38
2	DC	Dead load of right span	1.25	10077.81	-	-	-	-12093.38
3	DW	Dead load of wearing surface	1.50	2721.75	-	-	-	0.00
4	DC	Dead load of pier	1.25	19434.73	-	-	-	-
5	LL	Vehicle Live load - (maxMy)	1.75	4911.92	-	-	0.00	5894.31
7	BR	Braking force (Start)	1.75	-	725.16	-	-	13715.61
8	PL	Pedestrian live load	1.75	0.00	-	-	-	0.00
9	WA	Water load and stream pressure	1.00	-	11.68	108.81	545.47	58.54
10	FR	Friction	1.00	-	0.00	-	-	0.00
11	TU+CR+SH	Temperature + Creep + Shinkage	0.50	-	-	-	-	-
	TOTAL			**47224.03**	**736.83**	**108.81**	**545.47**	**19668.45**

II	**STRENGTH STATE II**							
1	DC	Dead load of left span	1.25	10077.81	-	-	-	12093.38
2	DC	Dead load of right span	1.25	10077.81	-	-	-	-12093.38
3	DW	Dead load of wearing surface	1.50	2721.75	-	-	-	0.00
4	DC	Dead load of pier	1.25	19434.73	-	-	-	0.00
5	WA	Water load and stream pressure	1.00	-	11.68	108.81	545.47	58.54
6	WS	Wind load on structure	1.40	-	117.02	468.09	8017.92	2004.48
7	FR	Friction	1.00	-	0.00	-	-	0.00
8	TU+CR+SH	Temperature + Creep + Shinkage	0.50	-	-	-	-	-
	TOTAL			**42312.10**	**128.70**	**576.90**	**8563.39**	**2063.02**

IIIa	**STRENGTH STATE III-A**							
1	DC	Dead load of left span	1.25	10077.81	-	-	-	12093.38
2	DC	Dead load of right span	1.25	10077.81	-	-	-	-12093.38
3	DW	Dead load of wearing surface	1.50	2721.75	-	-	-	0.00
4	DC	Dead load of pier	1.25	19434.73	-	-	-	0.00
5	LL	Live load - (maxNz)	1.35	6067.49	-	-	0.00	1813.08
7	BR	Braking force (Start)	1.35	-	559.41	-	-	10580.61
8	PL	Pedestrian live load	1.35	0.00	-	-	-	0.00
9	WA	Water load and stream pressure	1.00	-	11.68	108.81	545.47	58.54
10	WS	Wind load on structure	0.40	-	33.44	133.74	2290.83	572.71
11	WL	Wind load on vehicle	1.00	-	56.40	112.80	4267.00	2133.50
13	FR	Bearing Friction	1.00	-	0.00	-	-	0.00
14	TU+CR+SH	Temperature + Creep + Shinkage	0.50	-	-	-	-	-
	TOTAL			**48379.60**	**660.92**	**355.35**	**7103.30**	**15158.43**

IIIb	**STRENGTH STATE III-B**							
1	DC	Dead load of left span	1.25	10077.81	-	-	-	12093.38
2	DC	Dead load of right span	1.25	10077.81	-	-	-	-12093.38
3	DW	Dead load of wearing surface	1.50	2721.75	-	-	-	0.00
4	DC	Dead load of pier	1.25	19434.73	-	-	-	0.00
5	LL	Live load - (max Mx)	1.35	3033.75	-	-	31095.90	906.54
7	BR	Braking force (Start)	1.35	-	559.41	-	-	10580.61
8	PL	Pedestrian live load	1.35	0.00	-	-	0.00	0.00
9	WA	Water load and stream pressure	1.00	-	11.68	108.81	545.47	58.54
10	WS	Wind load on structure	0.40	-	33.44	133.74	2290.83	572.71
11	WL	Wind load on vehicle	1.00	-	56.40	112.80	4267.00	2133.50
13	FR	Bearing Friction	1.00	-	0.00	-	-	0.00
14	TU+CR+SH	Temperature + Creep + Shinkage	0.50	-	-	-	-	-
	TOTAL			**45345.85**	**660.92**	**355.35**	**38199.20**	**14251.89**

IIIc	**STRENGTH STATE III-C**							
1	DC	Dead load of left span	1.25	10077.81	-	-	-	12093.38
2	DC	Dead load of right span	1.25	10077.81	-	-	-	-12093.38
3	DW	Dead load of wearing surface	1.50	2721.75	-	-	-	0.00
4	DC	Dead load of pier	1.25	19434.73	-	-	-	0.00
5	LL	Live load - (max My)	1.35	3789.20	-	-	0.00	4547.04
7	BR	Braking force (Start)	1.35	-	559.41	-	-	10580.61
8	PL	Pedestrian live load	1.35	0.00	-	-	-	0.00
9	WA	Water load and stream pressure	1.00	-	11.68	108.81	545.47	58.54
10	WS	Wind load on structure	0.40	-	33.44	133.74	2290.83	572.71
11	WL	Wind load on vehicle	1.00	-	56.40	112.80	4267.00	2133.50
13	FR	Bearing Friction	1.00	-	0.00	-	-	0.00
14	TU+CR+SH	Temperature + Creep + Shinkage	0.50	-	-	-	-	-
	TOTAL			**46101.30**	**660.92**	**355.35**	**7103.30**	**17892.39**

IV	**EXTREME EVENT I_A**							
1	DC	Dead load of left span	1.25	10077.81	-	-	-	12093.38
2	DC	Dead load of right span	1.25	10077.81	-	-	-	-12093.38
3	DW	Dead load of wearing surface	1.50	2721.75	-	-	-	0.00
4	DC	Dead load of pier	1.25	15547.78	-	-	-	0.00
5	LL	Live load - (maxMx)	0.50	1123.61	-	-	11517.00	335.76
6	BR	Braking force (Start)	0.50	-	207.19	-	-	3918.74
7	PL	Pedestrian live load	0.50	0.00	-	-	-	0.00
8	WA	Water load and stream pressure	1.00	-	11.68	108.81	545.47	58.54
9	FR	Bearing Friction	1.00	-	0.00	-	-	0.00
10	EQ	Earth Quake Load	1.00	-	836.74	1936.00	15205.38	11817.62
	TOTAL			**39548.77**	**1055.61**	**2044.81**	**27267.85**	**16130.66**

IV	**EXTREME EVENT I_B**							
1	DC	Dead load of left span	1.25	10077.81	-	-	-	12093.38
2	DC	Dead load of right span	1.25	10077.81	-	-	-	-12093.38
3	DW	Dead load of wearing surface	1.50	2721.75	-	-	-	0.00
4	DC	Dead load of pier	1.25	15547.78	-	-	-	0.00
5	LL	Live load - (maxMy)	0.50	1403.41	-	-	0.00	1684.09
6	BR	Braking force (Start)	0.50	-	207	-	-	3919
7	WA	Water load and stream pressure	1.00	-	12	109	545	59
8	FR	Bearing Friction	1.00	-	-	-	-	-
9	EQ	Earth Quake Load	1.00	-	2789.14	580.80	4561.61	39392.07
	TOTAL			**39828.57**	**3008.00**	**689.61**	**5107.09**	**45053.44**

V	**EXTREME EVENT II**							
1	DC	Dead load of left span	1.25	10077.81	-	-	-	12093.38
2	DC	Dead load of right span	1.25	10077.81	-	-	-	-12093.38
3	DW	Dead load of wearing surface	1.50	2721.75	-	-	-	0.00
4	DC	Dead load of pier	1.25	15547.78	-	-	-	0.00
5	LL	Live load - (maxNz)	0.50	1123.61	-	-	11517.00	335.76
7	BR	Braking force (Start)	0.50	-	207.19	-	-	3918.74
8	PL	Pedestrian live load	0.50	0.00	-	-	-	0.00
9	WA	Water load and stream pressure	1.00	-	11.68	108.81	545.47	58.54
10	FR	Bearing Friction	1.00	-	0.00	-	-	0.00
11	CV	Vessels Impact	1.00	-	3244.00	6488.00	31765.27	15882.64
	TOTAL			**39548.77**	**3462.87**	**6596.82**	**43827.74**	**20195.67**

VII	SERVICE STATE I_A							
1	DC	Dead load of left span	1.00	8062.25	-	-	-	9674.70
2	DC	Dead load of right span	1.00	8062.25	-	-	-	-9674.70
3	DW	Dead load of wearing surface	1.00	1814.50	-	-	-	0.00
4	DC	Dead load of pier	1.00	15547.78	-	-	-	0.00
5	LL	Live load -(max Nz)	1.00	4494.44	-	-	0.00	1343.02
7	BR	Braking force (Start)	1.00	-	414.38	-	-	7837.49
8	PL	Pedestrian live load	1.00	0.00	-	-	-	0.00
9	WA	Water load and stream pressure	1.00	-	11.68	108.81	545.47	58.54
10	WS	Wind load on structure	0.30	-	25.08	100.31	1718.12	429.53
11	WL	Wind load on vehicle	1.00	-	56.40	112.80	4267.00	2133.50
13	FR	Bearing Friction	1.00	-	0.00	-	-	0.00
14	TU+CR+SH	Temperature + Creep + Shinkage	1.00	-	-	-	-	-
	TOTAL			**37981.22**	**507.53**	**321.92**	**6530.59**	**11802.08**

VIII	SERVICE STATE I_B							
1	DC	Dead load of left span	1.00	8062.25	-	-	-	9674.70
2	DC	Dead load of right span	1.00	8062.25	-	-	-	-9674.70
3	DW	Dead load of wearing surface	1.00	1814.50	-	-	-	0.00
4	DC	Dead load of pier	1.00	15547.78	-	-	-	0.00
5	LL	Live load - (max Mx)	1.00	2247.22	-	-	23034.00	671.51
7	BR	Braking force (Start)	1.00	-	414.38	-	-	7837.49
8	PL	Pedestrian live load	1.00	0.00	-	-	-	0.00
9	WA	Water load and stream pressure	1.00	-	11.68	108.81	545.47	58.54
10	WS	Wind load on structure	0.30	-	25.08	100.31	1718.12	429.53
11	WL	Wind load on vehicle	1.00	-	56.40	112.80	4267.00	2133.50
13	FR	Bearing Friction	1.00	-	0.00	-	-	0.00
14	TU+CR+SH	Temperature + Creep + Shinkage	1.00	-	-	-	-	0.00
	TOTAL			**35734.00**	**507.53**	**321.92**	**29564.59**	**11130.57**

IX	SERVICE STATE I_C							
1	DC	Dead load of left span	1.00	8062.25	-	-	-	9674.70
2	DC	Dead load of right span	1.00	8062.25	-	-	-	-9674.70
3	DW	Dead load of wearing surface	1.00	1814.50	-	-	-	0.00
4	DC	Dead load of pier	1.00	15547.78	-	-	-	0.00
5	LL	Live load - (max My)	1.00	2806.81	-	-	0.00	3368.17
7	BR	Braking force (Start)	1.00	-	414.38	-	-	7837.49
8	PL	Pedestrian live load	1.00	0.00	-	-	-	0.00
9	WA	Water load and stream pressure	1.00	-	11.68	108.81	545.47	58.54
10	WS	Wind load on structure	0.30	-	25.08	100.31	1718.12	429.53
11	WL	Wind load on vehicle	1.00	-	56.40	112.80	4267.00	2133.50
13	FR	Bearing Friction	1.00	-	0.00	-	-	0.00
14	TU+CR+SH	Temperature + Creep + Shinkage	1.00	-	-	-	-	0.00
	TOTAL			**36293.60**	**507.53**	**321.92**	**6530.59**	**13827.23**

COMBINATION EXTERNAL FORCE ON SECTION III - III

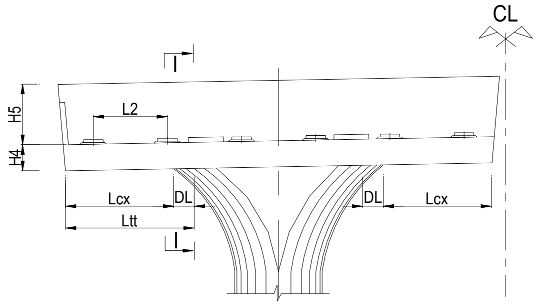
State	Load factors specified			Nz	Hx	Hy	Mx	My
	η_D	η_R	η_I	(KN)	(KN)	(KN)	(KNm)	(KNm)
Strength I A	1.00	1.00	1.00	50177.37	736.83	108.81	545.47	16124.43
Strength I B	1.00	1.00	1.00	46244.74	736.83	108.81	40854.97	14949.29
Strength I C	1.00	1.00	1.00	47224.03	736.83	108.81	545.47	19668.45
Strength II	1.00	1.00	1.00	42312.10	128.70	576.90	8563.39	2063.02
Strength III A	1.00	1.00	1.00	48379.60	660.92	355.35	7103.30	15158.43
Strength III B	1.00	1.00	1.00	45345.85	660.92	355.35	38199.20	14251.89
Strength III C	1.00	1.00	1.00	46101.30	660.92	355.35	7103.30	17892.39
Extreme event I A	1.00	1.00	1.00	39548.77	1055.61	2044.81	27267.85	16130.66
Extreme event I B	1.00	1.00	1.00	39828.57	3008.00	689.61	5107.09	45053.44
Extreme event II	1.00	1.00	1.00	39548.77	3462.87	6596.82	43827.74	20195.67
Service state I A	1.00	1.00	1.00	37981.22	507.53	321.92	6530.59	11802.08
Service state I B	1.00	1.00	1.00	35734.00	507.53	321.92	29564.59	11130.57
Service state I C	1.00	1.00	1.00	36293.60	507.53	321.92	6530.59	13827.23

2.3.5. CHECK FOR PIER CAP

2.3.5.1. Over hang Length:

$L_{tt} = L_{cx} + DL$

L_{tt}: Over hang length was calculated
L_{cx} : Read length of overhang
L_{cx} = 3.10 (m)
DL : addition length
R: Body pier radius
R = 1.25 (m)
DL = 0.42 (m)
=> Ltt = 3.52 (m)



2.3.5.2. Detemine internal force:

2.3.5.2.1 Distribution factor of Live load:

1 - Interior Girder:

*** Distribution factor for Momen**

- One Design lane loaded

$$g= (S / 910)^{0.35} (S.d / L2)^{0.25}$$

- Two or more Design lanes loaded

$$g= (S / 1900)^{0.6} (S.d / L^2)^{0.125}$$

*** Distribution factor for shear**

- One Design lane loaded

$$g= (S / 3050)^{0.6} (d / L)^{0.1}$$

- Two or more Design lanes loaded

$$g= (S / 2250)^{0.8} (d / L)^{0.1}$$

2 - Exterior Girder:

*** Distribution factor for Momen**

- One Design lane loaded: Use lever rule

Ge= 1.2(0.545+(S -1800)/2S)

- Two or more Design lanes loaded

$$g= e^*g_{interior} =(0.97 + de/8700).Gi$$

*** Distribution factor for shear**

- One Design lane loaded: Use lever rule

Ge= 1.2(0.545+(S -1800)/2S)

- Two or more Design lanes loaded

$$g= e^*g_{interior} = (0.8 + de/3050).Gi$$

Number Lanes	Girder possition	Live load (LL) Truck	tandem	Lane	Pedestrian (PL)
Distribution factor for Momen					
1 Lane	Interior Girder	0.304	0.304	0.304	
	Exterior Girder	0.745	0.745	0.745	
2 or more Lane	Interior Girder	0.508	0.508	0.508	
	Exterior Girder	0.523	0.523	0.523	
Distribution factor for Shear					
1 Lane	Interior Girder	0.592	0.592	0.592	
	Exterior Girder	0.745	0.745	0.745	
2 or more Lane	Interior Girder	0.702	0.702	0.702	
	Exterior Girder	0.792	0.792	0.792	

2.3.5.2.2 Exterior force impact on Over hang of pier cap:

Deadload: Seft weight of Over hang of Pier cap $DC_{(xm)}$ = 135.24 (KN/m)

Seft weight of super structure act to over hang

$DC_{(d)}$ = 671.85 (KN)

Wearing surface load DW = 75.60 (KN)

Live load:

Number Lanes	Girder possition	Live load (LL) Truck	tandem	Lane	Total Max (1;2) + (3)
		(KN)	(KN)	(KN)	
		(1)	(2)	(3)	(4)
Interior Girder		274.99	144.53	180.89	455.89
Exterior Girder		672.86	353.64	442.61	1115.47

Combination of Exterior load:

Type load / Symbol		Lever arm (m)	Service state Factor g	Nz (KN)	Strength I state Factor g	Nz (KN)
$DC_{(xm)}$	Seft weight of Pier cap	1.76	1.00	476.00	1.25	595.00
$DC^{exterior}_{(d)}$	Seft weight of Exterior Girde	2.72	1.00	671.85	1.25	839.82
$Dc^{interior}_{(d)}$	Seft weight of Interior Girder	0.60	1.00	671.85	1.25	839.82
$DW^{exterior}$	Seft weight of Deck	2.72	1.00	75.60	1.50	113.41
$DW^{interior}$	Seft weight of Deck	0.60	1.00	75.60	1.50	113.41
$LL^{exterior}_{(D)}$	Live load (Exterior Girder)	2.72	1.00	455.89	1.75	797.80
$LL^{interior}_{(D)}$	Live load (Interior Girder)	0.60	1.00	1115.47	1.75	1952.07
$PL^{exterior}$	Pedestrian (Exterior Girder)	2.72	1.00	0.00	1.75	0.00
$PL^{interior}$	Pedestrian (Interior Girder)	0.60	1.00	0.00	1.75	0.00
TOTAL				**3542.27**		**5251.32**

2.3.5.2.3 Combination loading aplied to section I - I

Type load / Symbol		Service state Nz (KN)	Y (m)	Mx (KNm)	Strength I state Nz (KN)	Y (m)	Mx (KNm)
$DC_{(xm)}$	Seft weight of Pier cap	476.00	1.76	837.68	595.00	1.76	1047.10
$DC^{exterior}_{(d)}$	Seft weight of Exterior Girde	671.85	2.72	1827.22	839.82	2.72	2284.02
$Dc^{interior}_{(d)}$	Seft weight of Interior Girder	671.85	0.60	402.89	839.82	0.60	503.61
$DW^{exterior}$	Seft weight of Deck	75.60	2.72	205.62	113.41	2.72	308.43
$DW^{interior}$	Seft weight of Deck	75.60	0.60	45.34	113.41	0.60	68.01
$LL^{exterior}_{(D)}$	Live load (Exterior Girder)	455.89	2.72	1239.86	797.80	2.72	2169.75
$LL^{interior}_{(D)}$	Live load (Interior Girder)	1115.47	0.60	668.91	1952.07	0.60	1170.59
TOTAL		**3542.27**		**5227.51**	**5251.32**		**7551.51**

Type Combonation	Load modifier specified η_D	η_R	η_I	N (KN)	Mx (KNm)
Service state (SLS)	1.00	1.00	1.00	**3542.27**	**5227.51**
Strength state (ULS)	1.00	1.00	1.00	**5251.32**	**7551.51**

Nz : *Vertical load*
Y: *Horizontal lever arm*
Mx: *Horizontal momen*

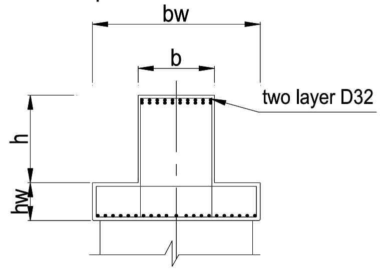
2.3.5.3. Ultimate check and shear capacity check - section I-I

DATA

- Width of section	bw	= 3.20	m
	b	= 1.60	m
- Hight of section	h	= 2.65	m
- Compression flange thickness	hf	= 0.80	m
- Clear cover thickness	d_c	= 0.18	m
- Flexural depth	d_e	= 2.48	m
- yeild strength of reinforcement	f_y	= 400.00	Mpa
- Elastic modulus of reinforcement	E_s	= 200000.00	Mpa
- Compressive strength of concrete at 28 days	f'_c	= 30.00	Mpa
- Unit weigth of concrete	γ_c	= 24.50	kN/m^3
- Elastic modulus of concrete	E_c	= 29440.09	Mpa

Combination interior force of I-I section

State	N (kN)	Mu (kNm)
Service state	3542.27	5227.51
Strength state	5251.32	7551.51



The Datas				Value	Unit
• Hight of Section			h	2650	mm
• Width of section			bw	3200	mm
			b	1600	mm
• Area of section			A_c	5520000	mm^2
• Moment of inertia of concrete section			I_g	9.8E+11	mm^4
• Tension reinforcement:	Distance from tension reinf. to extreme compression fiber		d_c	175	mm
	Diameter		∅	32.00	mm
	Number of bar		n	24.00	thanh
	Total of reinf.		A_s	19301.95	mm^2
• comp. reinforcement:	Distance from compressive reinf. to extreme Tension fiber		d_c	2600.00	mm
	Diameter		∅	16.00	mm
	Number of bar		n	20.00	thanh
	Total of reinf.		A'_s	4021.24	mm^2
Check plexural Moment					
Interior force Combination	**Strength state**				
• Factored Flexural moment			M_u	7.55E+06	kN.mm
• Resistance factor			Φ	0.90	
• The corresponding effective			d_e	2475.00	mm
• Stress block factor			β_1	0.84	
• Depth of the equivalent stress block			$a = c \cdot \beta_1$	479.34	mm
• Distance from extreme compression fiber to the neutral axis			$c=(A_s f_y - 0.85*\beta_1*f_c*(b-b_w)*h_f)/(0.85 f_c \beta_1 b_w)$	573.57	mm
• Due to neutral axis acrosses the flange, so the cross section should be check by rectangular section					
• The nominal flexural resistance:			Mn= As*fy*(de-a/2)	1.73E+07	kN.mm
• Factored flexural resistance			$M_r = \Phi \cdot M_n$	1.55E+07	kN.mm
• Check condition			$\mathbf{M_r > M_u}$	**O.K**	
Mimimum Reinforcement					
• Ratio of tension steel to gross area			$\rho_{min} = A_s/(b \cdot d)$	0.35	%
• Check			$\rho_{min} \geq 0.03 \cdot f'_c/f'_y$ = 0.23 %	**O.K**	
Maximum Reinforcement			A.5.7.3.3.1		
• Obligation Condition			c/d_e	0.23	
• Check			$c/d_e < 0.42$	**O.K**	

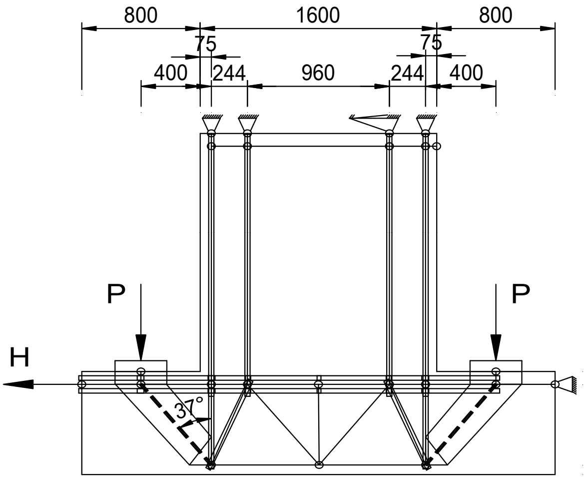
Check shear resistance			
Combination loading: **Strength state**			
• Factored Shear force	V_u	5251.32	kN
• Resistance factor	Φ	0.90	
• The effective shear deepth	d_v	2300.00	mm
• Effective width	b_v	1600.00	mm
• Angle of inclination of diagonal compressive stress	θ	43	độ
• Angle of inclination of transerverse reinf. To longitudinal axis	α	90	độ
• Factor indicating ability of diagonally cracked concrete to transmit tension	β	1.75	
• Value	$0.1 \cdot f'_c \cdot b_v \cdot d_v$	11040.00	kN
• Max spacing of transverse reinforcement	s	600.00	mm
• Spacing of stirrup	s	150.00	mm
• Diameter of transverse reinforcement	$\varnothing$	32	mm
• Number of transverse reinf. within distance s	n	2	
• Diameter of stirrup	$\varnothing$	25	mm
• Number of stirrup within distance s	n	4	
• Total area of stirrup	A_v	1963.50	mm^2
• Assume	θ	43.00	degree
• Strain in tensile reinforcement	ex	8.51E-01	
If ex<0, multiple with reduce factor	Fc	-	
• Ratio of shear stress and f'c	V/f'c	0.05	
• β final		1.75	
• θ final		43.00	degree
• The shear resistance of concrete:	V_c	2927.69	kN
• The shear resistance of stirrup	V_s	12914.29	kN
• Value	$0.25 \cdot f'_c \cdot b_v \cdot d_v$	27600.00	kN
• The nominal shear resistance:	$V_n = Min(0.25 \cdot f'_c \cdot b_v \cdot d_v; V_c + V_s)$	15841.98	kN
• The factored shear resistance	$V_r = \Phi * V_n$	14257.78	kN
• Check	$V_r > V_u$	**O.K**	
•Requiring transverse reinforcement	$V_u > 0.5 \cdot \Phi \cdot V_c$	**Need**	
•Check minimum transverse reinforcement	$A_v > 0.083 \cdot (f'_c{}^{0.5}) \cdot b_v \cdot s/f_y$	**O.K**	
Check crack			
Interior force combination		**Service state**	
• Factored moment	M_S	5.23E+06	kN.mm
• Modulus of rupture of concrete	fr = 0.63*sqrt(f'c)	3.45	MPa
• Distance from extreme tension fiber to the neutral axis	$y_t = h-c$	2076	mm
• Stress of concrete at tension fiber	$f'r = Ms * y_t / I_g$	11.07	MPa
• Check			
Stress of concrete f'r>0.8*fr should be controled of cracking by distribution of reinforcement by condition:			
• Crack width parameter	Z	30000.00	N/mm
• Ratio of reinf. Modulus with concrete modulus	$n = E_s/E_c$	7.00	
• The distance from extreme fiber to the neutral axis	c	= 736.47	mm
• Effective moment of inertia	J	6.21E+11	mm4
• Arm	de-c	= 1738.53	mm
• Tension stress in reinforcement	fs =n*Ms*(de-c)/J	= 102.37	Mpa
• Area of concrete having the same centroid as the principal tensile reinforcement divided by number of bars	A	26666.67	mm^2
• Tension stress in reinforcement with service state	$f_{sa} = Z/(d_c \cdot A)^{1/3}$	205.89	Mpa
• Check condition	$f_s < f_{sa}$	**O.K**	
• Check condition	$f_s < 0.6 \cdot f_y$	**O.K**	

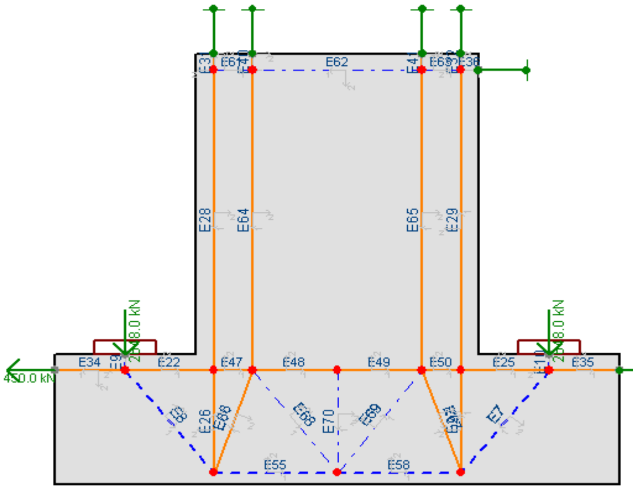
- **Checking of Overhang of Pier cap accordance with strut and tie model:**

The force apllied to ovehang part of Pier cap

Combination loading	Axial force (KN)	Longitudinal force (KN)
Strength I	5251.32	671.79
Service	2647.47	671.79

The model of pier cap:





- We will consider for compresive stress and tensile stress on strut and tier bar
  + Strut element need consider is E7, E8 (Line - - - - -)
  + Tier element need consider is E22, E25, E28, E28, E64, E65 (Line ____________) on the diagram above

Checking for strut element at Strength state I

Strut ID	F_u (kN)	β_s	ϕ	$\phi f_c = \phi(0.85)\beta_s f'_c$ (MPa)	Effective Width (mm)	Effective Thickness Scale Factor (mm)	ϕF_{ns} (kN)	Ratio $F_u / \phi F_{ns}$	Check
E7	-6543.7	1	0.75	19.13	270	1	16524	0.40	OK
E8	-6543.7	1	0.75	19.13	270	1	16524	0.40	OK

Checking for Tie element

Tie ID	F_u (kN)	Required A_s (mm²)	Provided A_s (mm²)	ϕ	ϕF_{ns} (kN)	Ratio $F_u / \phi F_{ns}$	Check $F_u/\phi F_{ns}<1$	Tie stress At service state (Mpa)	Limit stress $0.6f_y$ (Mpa)	Check $f_s<0.6f_y$
E22	4759.60	15865.3	21598.3	0.75	8467.5	0.56	OK	123.55	240	OK
E25	4759.60	15865.3	21598.3	0.75	8467.5	0.56	OK	123.55	240	OK
E28	3014.80	10049.2	10799.1	0.75	4233.7	0.71	OK	139.25	240	OK
E29	3014.80	10049.2	10799.1	0.75	4233.7	0.71	OK	139.25	240	OK
E64	2095.00	10799.1	10799.1	0.75	4233.8	0.49	OK	95.25	240	OK
E65	2095.00	10799.1	10799.1	0.75	4233.8	0.49	OK	95.25	240	OK

+ Tier element E28, E29, E64, E65 used 1 layer D25 @ 150
+ Tier element E22, E25 used 2 layer D25 @ 150

2.3.6. DETEMINAL INTERNAL FORCE AT TOP OF PILE

2.3.6.1. Summary of external force acting to bottom of footing:

Combination	V	Hx	Hy	Mx	My
	(KN)	(Kn)	(Kn)	(Kn.m)	(Kn.m)
Strength I	50177	737	109	40855	19668
Strength II	42312	129	577	8563	2063
Strength III	48380	661	355	38199	17892
Service I	37981	508	322	29565	13827
Extreme event IA (EQ)	39549	1056	2045	27268	16131
Extreme event IB (EQ)	39829	3008	690	5107	45053
Extreme event I (CV)	39549	3463	6597	43828	20196

2.3.6.2. Piling material:

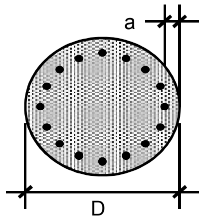
Concrete 30 Mpa

E_c (Mpa)	29440
γ_c (KN/m^3)	2450

Steel bar

Type	CB-400-T
E_s (Mpa)	200000

2.3.6.3. Piling dimension



+ Diameter **D** = 1.50 m

a = 0.075 m

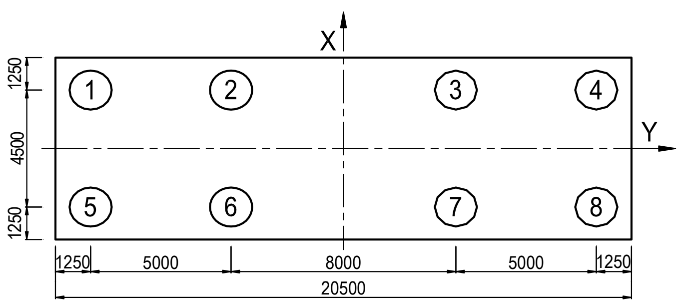
+ Length **L** = 62.00 m

2.3.6.4. Maximum Internal force and displacements at top piling

Internal force and diplacements *(Result follow Piling software)*

Combination	N	H	M	x	y	z
	(KN)	(KN)	(KN.m)	(m)	(m)	(rad)
Strength I	8305.81	92.13	-2.42	-	-	-
Strength II	5672.02	16.13	25.63	-	-	-
Strength III	7946.47	82.63	-6.93	-	-	-
Service I	6220.66	63.50	-6.77	**0.003**	**-0.001**	**0.007**
Extreme IA	6780.14	63.50	-6.77	-	-	-
Extreme IB	7963.05	376.00	655.34	-	-	-
Extreme II	8373.85	432.88	1352.66	-	-	-

- Check displacement of top pile not exceed 38mm (10.7.2.7) **OK**



Arrangement of pile

- Result for internal force at top pile

Combination	Pile Number	Hx	Hy	N	Mx	My
		(KN)	(KN)	(KN.m)	(m)	(m)
Strength I	1	92.13	-13.63	6122.63	-30.03	-2.42
	2	92.13	-13.63	5599.24	-30.03	-2.42
	3	92.13	-13.63	4761.82	-30.03	-2.42
	4	92.13	-13.63	4238.44	-30.03	-2.42
	5	92.13	-13.63	8305.81	-30.03	-2.42
	6	92.13	-13.63	7782.43	-30.03	-2.42
	7	92.13	-13.63	6945.01	-30.03	-2.42
	8	92.13	-13.63	6421.62	-30.03	-2.42
Strength II	1	16.13	-72.13	5420.01	314.63	25.63
	2	16.13	-72.13	5277.23	314.63	25.63
	3	16.13	-72.13	5048.77	314.63	25.63
	4	16.13	-72.13	4905.99	314.63	25.63
	5	16.13	-72.13	5672.02	314.63	25.63
	6	16.13	-72.13	5529.23	314.63	25.63
	7	16.13	-72.13	5300.78	314.63	25.63
	8	16.13	-72.13	5157.99	314.63	25.63
Strength III	1	82.63	-44.38	5964.63	118.50	-6.93
	2	82.63	-44.38	5460.16	118.50	-6.93
	3	82.63	-44.38	4653.00	118.50	-6.93
	4	82.63	-44.38	4148.53	118.50	-6.93
	5	82.63	-44.38	7946.47	118.50	-6.93
	6	82.63	-44.38	7442.00	118.50	-6.93
	7	82.63	-44.38	6634.84	118.50	-6.93
	8	82.63	-44.38	6130.37	118.50	-6.93
Service	1	63.50	-40.25	4690.35	119.07	-6.77
	2	63.50	-40.25	4297.08	119.07	-6.77
	3	63.50	-40.25	3667.85	119.07	-6.77
	4	63.50	-40.25	3274.59	119.07	-6.77
	5	63.50	-40.25	6220.66	119.07	-6.77
	6	63.50	-40.25	5827.40	119.07	-6.77
	7	63.50	-40.25	5198.17	119.07	-6.77
	8	63.50	-40.25	4804.90	119.07	-6.77
Extreme event IA	1	132.00	-255.63	4788.58	1122.14	224.13
	2	132.00	-255.63	4321.51	1122.14	224.13
	3	132.00	-255.63	3574.19	1122.14	224.13
	4	132.00	-255.63	3107.11	1122.14	224.13
	5	132.00	-255.63	6780.14	1122.14	224.13
	6	132.00	-255.63	6313.06	1122.14	224.13
	7	132.00	-255.63	5565.74	1122.14	224.13
	8	132.00	-255.63	5098.67	1122.14	224.13
Extreme event IB	1	376.00	-86.13	2374.64	387.37	655.34
	2	376.00	-86.13	2268.89	387.37	655.34
	3	376.00	-86.13	2099.70	387.37	655.34
	4	376.00	-86.13	1993.95	387.37	655.34
	5	376.00	-86.13	7963.05	387.37	655.34
	6	376.00	-86.13	7857.31	387.37	655.34
	7	376.00	-86.13	7688.11	387.37	655.34
	8	376.00	-86.13	7582.36	387.37	655.34
Extreme event II	1	432.88	-824.63	4927.48	3720.56	1352.66
	2	432.88	-824.63	3979.13	3720.56	1352.66
	3	432.88	-824.63	2461.76	3720.56	1352.66
	4	432.88	-824.63	1513.40	3720.56	1352.66
	5	432.88	-824.63	8373.85	3720.56	1352.66
	6	432.88	-824.63	7425.49	3720.56	1352.66
	7	432.88	-824.63	5908.12	3720.56	1352.66
	8	432.88	-824.63	4959.77	3720.56	1352.66

2.3.7. ULTIMATE LOAD CHECK, SHEAR CAPACITY AND CRACK CHECK

2.3.7.1. Check for pier shaft:

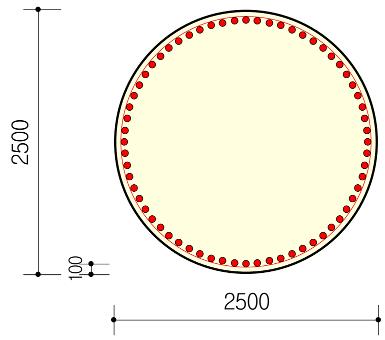
Item		Mark	Unit	Value
• Factored Axial force		Nu	Kn	13472.31
• Factored Plexural moment		Mux	Kn.m	14230.77
• Factored Plexural moment		Muy	Kn.m	4853.74
• Diameter of Pier shaft		D	m	2.50
• Section area		Ag	m2	4.91
• Moment of inertia of concrete section		Ic	m4	1.92
• Cover thickness		a	m	0.075
• Reinf. Diameter		Ds	mm	32.00
• Number of rebar		n_s	nos	64.00
• Rebar area		As	mm2	51471.85
Check minimum reinforcement				
• Minimum rebar area required (0.135*fy'c/fy)*Ag		As req	mm2	49700.98
• Check condition As > (0.135*f'c/fy)*Ag				**OK**
Check maximum reinforcement				
• Maximum rebar area 0.08*Ag		As max	mm2	392699.1
• Check condition As < 0.08*Ag				**OK**
Check ratio spiral or Tier (5.7.4.6)				
• Distance to outside of Spairal or Ties to concrete face			mm	62.00
• Effect diamete		Deff	m	2.38
• Area of core measured to the outside diameter of the spiral			m2	4.43
• Ratio spiral Rebar required		ρsa		0.00361
Required Area of Spiral Rebar	space		mm	150
	Effective length			2.38
	layer			1
	Area			322.1
	Requaired Dhs			20.3
Actuaral	Effective length	d	m	2.376
	Diameter	Dhr	mm	22
	Area of Rebar	Ah	mm2	380.1
	layer	Nl	nos	1
	Total area of spiral	Ac	m2	380.133
	space	s	mm	150
	Ratio spiral Rebar	ρs	-	0.0042664
• Check condition		$\rho_s > \rho_{sa}$		OK
Check Crack (At Service state)				
• Modulus of rupture of concrete		fr = 0.63*sqrt(f'c)	Mpa	3.45
• Stress of concrete at tension fiber		σ'_r	Mpa	6
• If f'r > 0.8fr require check crack		$\sigma'_r > 0.8*\sigma_r$	Mpa	Check
• Center of newtral axial		x	mm	1.25
• Maximum stress of Compression fiber of concrete		σ_c	Mpa	12.9
• Maximum stress of Compression Rebar		σ_{rc}	Mpa	-172.7
• Maximum stress of Tension Rebar		σ_{rt}	Mpa	172.8
• Check			σ_{rt} <0.6.fy	OK

MIDAS/Set Column Design [Check Pier Shaft - D2.5m- Pier P9]

Company	Nipponkoei	Project Name	DN Qn Expressway
Designer	vinh	File Name	D:\...Pier Shaft-D2.5m-Pier P7.BOI

1. Geometry and Materials

Design Code : ACI318M-02
Stress Profile : Equivalent Stress Block
Material Data : f_c' = 30 MPa (β_1 = 0.850)
f_y = 400, f_{ys} = 400 MPa
Section Dim. : Φ2500 mm
Effective Len. : KL_u = 11500 mm
Steel Distribut.: 64 - D32 (d_c = 100 mm)
Total Steel Area A_{st} = 50829 mm^2 (ρ_{st} = 0.0104)



2. Member Force and Moment

Unit : kN, kN-m

L.C.	P_u	M_{ux}	M_{uy}	R_{atioV}	V_{ux}	V_{uy}	R_{atioH}	Remark
1	19595.0	129.0	7135.0	0.307	365.0	34.0	0.026	
2	12853.0	20283.0	6547.0	0.660	365.0	34.0	0.027	
3	13343.0	129.0	8907.0	0.307	365.0	34.0	0.027	
4	14252.0	3552.0	864.0	0.198	61.0	268.0	0.020	
5	18696.0	2958.0	6676.0	0.305	327.0	157.0	0.026	
6	12404.0	18506.0	6223.0	0.591	327.0	157.0	0.026	
7	12782.0	2958.0	8043.0	0.296	327.0	157.0	0.026	
8	16786.0	1951.0	18050.0	0.544	1297.0	220.0	0.095	
9	11589.0	17499.0	16856.0	0.838	1297.0	220.0	0.095	
10	16786.0	6202.0	6510.0	0.329	463.0	655.0	0.058	
11	11449.0	11961.0	4978.0	0.386	360.0	655.0	0.054	
12	16786.0	15802.0	11861.0	0.589	3455.0	6556.0	0.534	
13	11449.0	21561.0	8124.0	0.779	3248.0	6556.0	0.531	
14	14596.0	2714.0	5189.0	0.241	250.0	141.0	0.021	
15	13472.0	14231.0	4854.0	0.449	250.0	141.0	0.021	
16	13752.0	2714.0	6202.0	0.255	250.0	141.0	0.021	

3. Magnified Moment

KL_u/r_x = 11500/625 = 18.40 < 34-12(M_1/M_2) = 22.00
δ_x = 1.000

KL_u/r_y = 11500/625 = 18.40 < 34-12(M_1/M_2) = 22.00
δ_y = 1.000

4. Design Force and Moment

Design Load Combination No : 9
P_u = 11589.0 kN
M_{ux} = 17499.0, M_{uy} = 16856.0 kN-m

	Company	Nipponkoei	Project Name	DN Qn Expressway
	Designer	vinh	File Name	D:\...Pier Shaft-D2.5m-Pier P7.BOI

5. Check Axial and Moment Capacity

Rotation Angle and Depth to the Neutral Axis θ = -46.07°, c = 741 mm

Strength Reduction Factor Φ = 0.9000

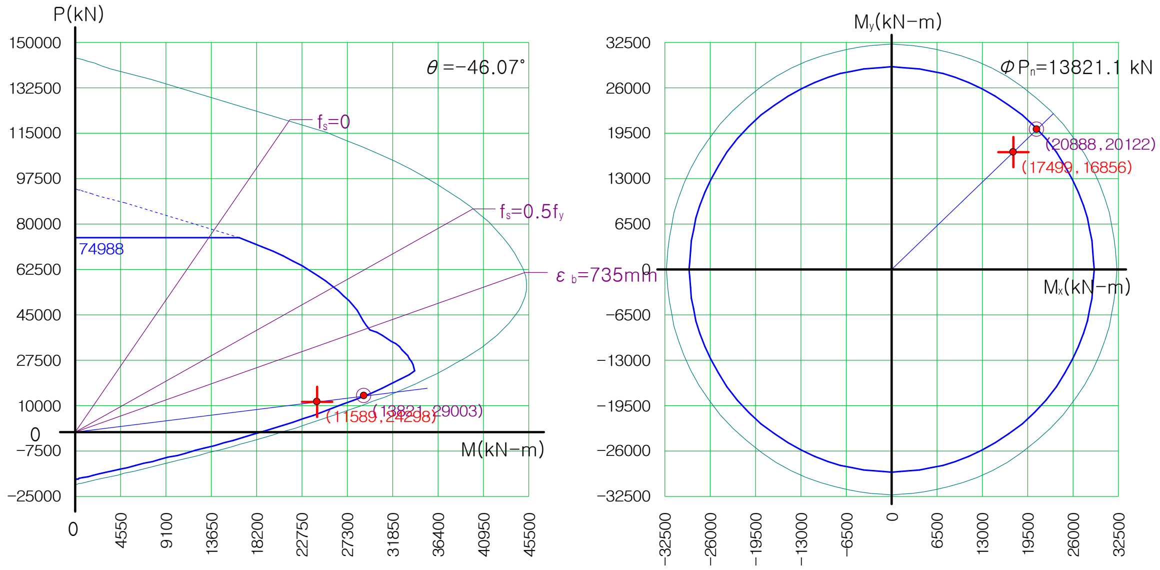
Maximum Axial Load $\Phi P_{n(max)}$ = 74988.3 kN

Design Axial Load Strength ΦP_n = 13821.1 kN

Design Moment Strength ΦM_{nx} = 20887.9 kN-m

ΦM_{ny} = 20121.9 kN-m

Strength Ratio : Applied/Design = 0.838 < 1.000 .... O.K



6. Check Shear Capacity

Design Load Combination No : 12

Strength Reduction Factor Φ= 0.750

Design Force Vu = 7410.7 kN (Pu = 16786.0 kN)

Required Hoop Spacing : D22 @ 361 mm

Provided Hoop Spacing : D22 @ 150 mm (Tie)

$\Phi V_c + \Phi V_s$ = 10801.4 + 3069.1 = 13870.5 kN > V_u = 7410.7 kN .... O.K

2.3.7.2. Check for pile:

Item		Mark	Unit	Value
• Factored Axial force		Nu	Kn	6220.66
• Factored Plexural moment		Mux	Kn.m	119.07
• Factored Plexural moment		Muy	Kn.m	-6.77
• Diameter of Pier shaft		D	m	1.50
• Section area		Ag	m2	1.77
• Moment of inertia of concrete section		Ic	m4	0.25
• Cover thickness		a	m	0.075
• Reinf. Diameter		Ds	mm	D32
• Number of rebar		n_s	nos	24
• Rebar area		As	mm2	19301.95
Check minimum reinforcement				
• Minimum rebar area required (0.135*fy'c/fy)*Ag		As req	mm2	17892.35
• Check condition As > (0.135*f'c/fy)*Ag				**OK**
Check maximum reinforcement				
• Maximum rebar area 0.08*Ag		As max	mm2	141371.7
• Check condition As < 0.08*Ag				**OK**
Check ratio spiral or Tier (5.7.4.6)				
• Distance to outside of Spairal or Ties to concrete face			mm	68.00
• Effect diamete		Deff	m	1.36
• Area of core measured to the outside diameter of the spiral			m2	1.46
• Ratio spiral Rebar required		ρsa		0.00707
Required Area of Spiral Rebar	space		mm	75
	Effective length			1.36
	layer			1
	Area			180.7
	Requaired Dhs			15.2
Actuaral	Effective length	d	m	1.364
	Diameter	Dhr	mm	16
	Area of Rebar	Ah	mm2	201.1
	layer	Nl	nos	1
	Total area of spiral	Ac	m2	201.062
	space	s	mm	75
	Ratio spiral Rebar	ρs	-	0.0078617
• Check condition		$\rho_s > \rho_{sa}$		OK

Company	Nipponkoei	**Project Name**	DN Qn Expressway
Designer	vinh	**File Name**	D:\...\Pile D1.5m - Pier P7.BOI

1. Geometry and Materials

Design Code : ACI318M-02

Stress Profile : Equivalent Stress Block

Material Data : f_c' = 30 MPa (β_1 = 0.850)

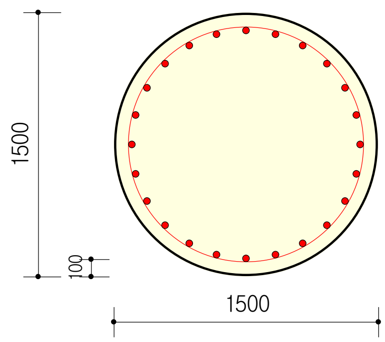
f_y = 400, f_{ys} = 400 MPa

Section Dim. : Φ1500 mm

Effective Len. : KL_u = 9000 mm

Steel Distribut.: 24 - D32 (d_c = 100 mm)

Total Steel Area A_{st} = 19061 mm^2 (ρ_{st} = 0.0108)



2. Member Force and Moment

Unit : kN, kN-m

L.C.	P_u	M_{ux}	M_{uy}	R_{atioV}	V_{ux}	V_{uy}	R_{atioH}	Remark
1	8305.8	-30.0	-2.4	0.306	92.1	13.6	0.019	
2	5672.0	314.6	25.6	0.209	16.1	72.1	0.016	
3	7946.5	118.5	-6.9	0.293	82.6	44.4	0.020	
4	6220.7	119.1	-6.8	0.229	63.5	40.3	0.016	
5	6780.1	1122.1	224.1	0.280	132.0	255.6	0.060	
6	7963.1	387.4	655.3	0.293	376.0	86.1	0.081	
7	8373.9	3720.6	1352.7	0.652	432.9	824.6	0.195	

3. Magnified Moment

KL_u/r_x = 9000/375 = 24.00 > 34-12(M_1/M_2) = 22.00

δ_x = MAX[1.00/(1-P_u/0.75/145156), 1.0] = 1.083

KL_u/r_y = 9000/375 = 24.00 > 34-12(M_1/M_2) = 22.00

δ_y = MAX[1.00/(1-P_u/0.75/145156), 1.0] = 1.083

4. Design Force and Moment

Design Load Combination No : 7

P_u = 8373.9 kN

M_{ux} = 3720.6, M_{uy} = 1352.7 kN-m

$\delta_x M_{ux}$ = δ_x*M_{ux} = 4030.6 kN-m

$\delta_y M_{uy}$ = δ_y*M_{uy}, = 1465.4 kN-m

5. Check Axial and Moment Capacity

Rotation Angle and Depth to the Neutral Axis θ = -70.02°, c = 791 mm

Strength Reduction Factor Φ = 0.6810

Maximum Axial Load $\Phi P_{n(max)}$ = 27144.3 kN

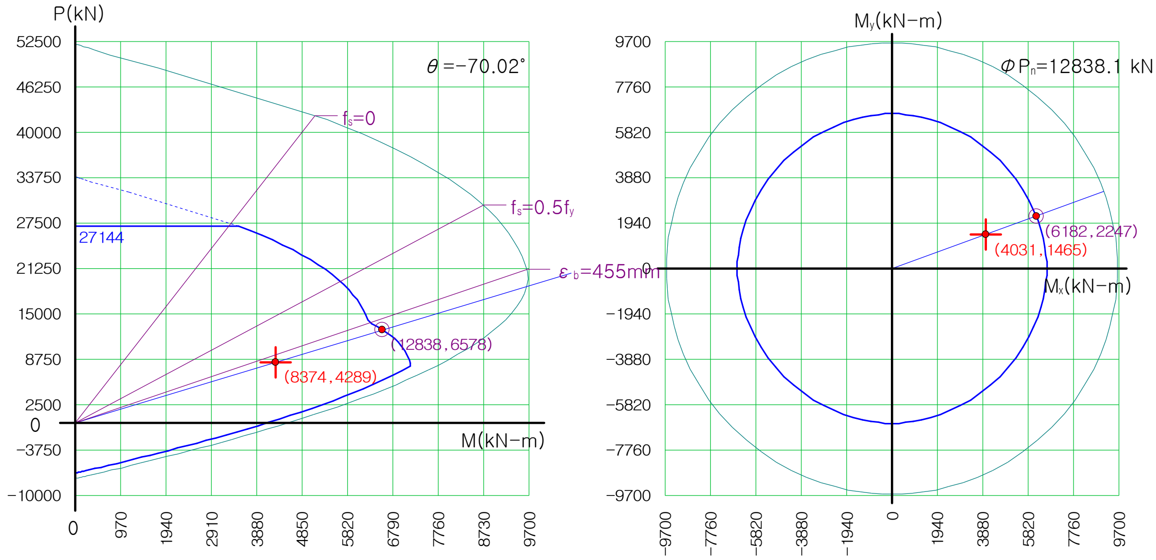
Design Axial Load Strength ΦP_n = 12838.1 kN

Design Moment Strength ΦM_{nx} = 6182.3 kN-m

ΦM_{ny} = 2247.1 kN-m

Strength Ratio : Applied/Design = 0.652 < 1.000 .... O.K

	Company	Nipponkoei	Project Name	DN Qn Expressway
	Designer	vinh	File Name	D:\...\Pile D1.5m - Pier P7.BOI



6. Check Shear Capacity

Design Load Combination No : 7

Strength Reduction Factor Φ= 0.750

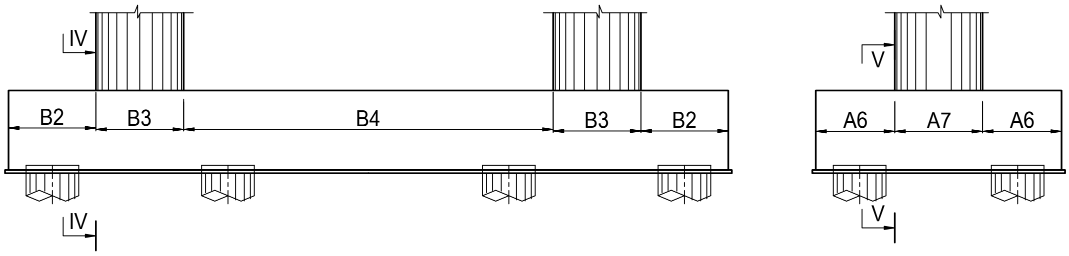
Design Force Vu = 931.3 kN (Pu = 8373.9 kN)

Required Hoop Spacing : D16 @ 508 mm

Provided Hoop Spacing : D16 @ 150 mm (Tie)

$\Phi V_c + \Phi V_s$ = 3863.8 + 924.5 = 4788.3 kN > V_u = 931.3 kN .... O.K

2.3.8. CHECK PILE CAP :
2.3.8.1. External force to section IV-IV, section V-V:



External force to section IV - IV

STATE	Longitudinal direction		
	Q (kN)	M(kN.m)	N (kN)
Strength I	29454.9	66273.5	368.5
Strength II	21660.0	48735.0	-288.5
Strength III	28153.7	63345.8	-177.5
Service	22051.1	49615.0	-161.0
Extreme IA	23757.6	53454.6	528.0
Extreme IB	31090.8	69954.4	1504.0
Extreme II	26667.2	60001.3	1731.5

External force to section V - V

STATE	Transverse direction		
	Q (kN)	M(kN.m)	N (kN)
Strength I	14428.4	23980.1	184.3
Strength II	11092.0	18434.9	32.3
Strength III	13911.1	23120.2	165.3
Service	10911.0	18134.1	127.0
Extreme IA	11568.7	19227.2	264.0
Extreme IB	10337.7	17181.2	752.0
Extreme II	13301.33	22106.8	865.8

2.3.8.2. Ultimate check and shear capacity check :

Item		Symbol	Section IV-IV (Bottom bar)	Section V-V (Bottom bar)	Unit
• Factored Plexural moment		M_u	66273.45	23980.07	kN.m
• Factored Shear force		V_u	29454.87	14428.44	kN
• Hight of Section		h	2500	2500	mm
• Width of section		b	20500	7000	mm
• Section area		A_c	51250000	17500000	mm^2
• Moment of inertia of concrete section		I_g	2.7E+13	9.1E+12	mm^4
• Tension reinforcement:	Distance from tension reinf. to extreme compression fiber	d_c	166	198	mm
	Reinf. Diameter	∅	32	32	mm
	Space	@	150	150	mm
	Number of bar	n	136	46	bar
	Total area of reinf.	A_s	109378	37263	mm^2
• comp. reinforcement:	Distance from compressive reinf. to extreme Tension fiber		100	138	mm
	Diameter		25	25	mm
	Reinf. Space		150	150	mm
	Number of bar		136	46	bar
	Total area of reinf.	A'_s	66922	22744	mm^2
Check Flexural Moment at Strength state					
• Resistance factor		Φ	0.90	0.90	
• The corresponding effective		d_e	2334	2302	mm
• Stress block factor		β_1	0.8357	0.84	
• Depth of the equivalent stress block = c*β1		a	83.69	83.50	mm
• Distance from extreme compression fiber to the neutral axis		c	100.15	99.92	mm
• The nominal flexural resistance:		M_n	100284	33690	kN.m
• Factored flexural resistance		$M_r = \Phi.M_n$	90256	30321	kN.m
• Check condition		$M_r > M_u$	**O.K**	**O.K**	
Mimimum Reinforcement					
• Ratio of tension steel to gross area		$\rho = A_s/(b.d)$	0.23	0.23	%
• Check		$\rho > 0.03{\bullet}f'_c/f'_y$	**O.K**	**O.K**	0.23
• Cracking moment		1.2Mcr	88422.96	25599.99	Kn.m
• Check		Mr> min(1.2Mcr, 1.33Mu)	**O.K**	**O.K**	
Maximum Reinforcement					
• Obligation Condition		c/d_e	0.04	0.04	
• Check		$c/d_e < 0.42$	**O.K**	**O.K**	
Check shear resistance					
• Factored Shear force		V_u	29454.87	14428.44	kN
• Resistance factor		Φ	0.90	0.90	
• The effective shear deepth		d_v	2292	2260	mm
• Effective width		b_v	20500	7000	mm
• Angle of inclination of diagonal compressive stress		θ	43	43	degree
• Angle of inclination of transerverse reinf. To longitudinal axis		α	90	90	degree
• Factor indicating ability of diagonally cracked concrete to transmit tension		β	1.75	1.75	
• Value		$0.1{*}f'_c{\bullet}b_v{\bullet}d_v$	140967	47465	kN
• Max spacing of transverse reinforcement		s	600	600	mm
• Spacing of stirrup		s	450	450	mm
• Diameter of transverse reinforcement		∅	D 32	D 32	
• Number of transverse reinf. within distance s		n	2	2	bar
• Assume		θ	43.00	39.00	degree
• Strain in tensile reinforcement		ε_x	2.04E-03	2.62E-03	
If ex<0, multiple with reduce factor		Φc	-	-	
• Ratio of shear stress and f'c		V/f'c	0.02	0.03	
• β final			1.75	1.75	
• θ final			43.00	43.00	degree
• Total area of transverse reinf.		A_v	1608	1608	mm^2
• Diameter of stirrup		∅	D 18	D 18	mm
• Number of stirrup within distance s		n	47	15	bar
• Total area of stirrup		A_v	11846.95	3788.76	
• The shear resistance of concrete:		V_c	37383.01	12587.25	kN
• The shear resistance of stirrup		V_s	12325.25	3886.86	kN

• Value	$0.25*f'_c \cdot b_v \cdot d_v$	352418.52	118663.03	kN
• The nominal shear resistance:	V_n	49708.25	16474.11	kN
• The factored shear resistance	V_r	44737.43	14826.70	kN
• Check	$V_r > V_u$	**O.K**	**O.K**	
•Requiring transverse reinforcement	$V_u > 0.5 \cdot \Phi \cdot V_c$	**Need**	**Need**	
•Check minimum transverse reinforcement	$A_v > 0.083 \cdot (f'_c{}^{0.5}) \cdot b_v \cdot s/f_y$	**O.K**	**O.K**	
Check Flexural and shear resistance at Extreme state				
• Factored Plexural moment	M_u	69954.36	22106.82	kN.m
• Factored Shear force	V_u	31090.83	13301.33	kN
• Resistance factor	Φ	0.90	0.90	
• The nominal flexural resistance:	M_n	100284	33690	kN.m
• Factored flexural resistance	$M_r = \Phi.M_n$	90256	30321	kN.m
• Check condition	$M_r > M_u$	**O.K**	**O.K**	
Check crack				
Interior force combination				
• Factored moment	M_u	4.96E+04	1.81E+04	kN.m
• Modulus of rupture of concrete	fr = 0.63*sqrt(f'c)	3.45	3.45	MPa
• Distance from extreme tension fiber to the neutral axis	$y_t = h-c$	2400	2400	mm
• Stress of concrete at tension fiber	$f'r = Ms*y_t/I_g$	4	5	MPa
• Check	f'r >	0.8*fr	0.8*fr	
		check crack	check crack	
• Crack width parameter	Z	= 23000	= 23000	N/mm
• Ratio of reinf. Modulus with concrete modulus	$n = E_s/E_c$	= 7.00	= 7.00	
• The distance from extreme fiber to the neutral axis	c	= 456.56	= 453.14	mm
• Effective moment of inertia	J	3.35E+12	1.11E+12	mm^4
• Arm	de-c	= 1877.44	= 1848.86	mm
• Tension stress in reinforcement	fs =n*Ms*(de-c)/J	= 194.70	= 211.67	MPa
• Area of concrete having the same centroid as the principal	A	= 15074	= 15108	mm2
• Tension stress in reinforcement with service state	$f_{sa} = Z/(d_c*A)^{1/3}$	= 230.40	= 230.22	Mpa
• Check condition	$f_s < f_{sa}$	**O.K**	**O.K**	
• Check condition	$f_s < 0.6*f_y$	**O.K**	**O.K**	

2.4 PIER P9
2.4 TRỤ CẦU P9

PIER P9 - CALCULATION SHEET

(GROUP P1, P2, P4, P5, P6, P8, P9 - PIER WITH RUBBER BEARING)

CONTENT:

CALCULATION PROCEDURE & STANDARD:

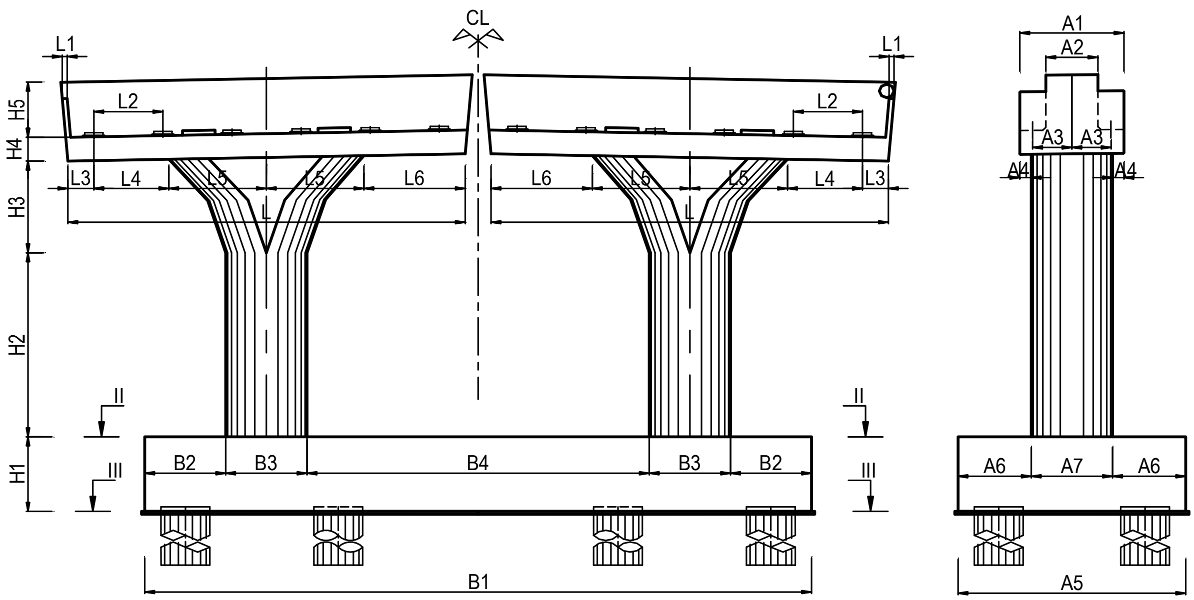
Design criteria: Bridge Design Standard 22 TCN - 272 - 05

2.4.1. GENERAL DATA:

1. Span length	`	L_{left} =	38.30	(m)
		L_{right} =	38.30	(m)
2. Design live load				
Design vehicle load			HL93	22TCN 272 - 05
Number of lane			2x3	(lane)
Pedestrian			0.00	KG/m^2
3. Bridge width				
Width carriageway		B_{xe} =	12.00	(m)
Width of median guardrail		B_{pc} =	0.50	(m)
Width barrier		B_{lc} =	0.50	(m)
Bridge width		B =	13.00	(m)
4. cross sections:				
Pavement thick ness		d_{BTN} =	0.084	(m)
Deck Thickness		d_{BTCT} =	0.20	(m)
Number of Girder		n =	2x6	(m)
Girder distance		L_d =	2.12	(m)
Distance from center of outer girder to extreme of pier cap		L =	0.75	(m)
5. Material property:				
Concrete				
Compressive strength of cylindrical at 28 da		f 'c =	30.00	MPa
Concrete density		γ =	24.50	KN/m^3
Elastic modulus		Ec =	29440	MPa
Tension strength of concrete		fr =	3.45	MPa
Steel				
Concrete modulus		Es =	200000	MPa
Yeild strength of steel bar		fy =	400.00	MPa

6. THE PIER GEOMETRIC:

GENERAL DIMENTIONS OF PIER



vertical			Horizontal			Thickness		
Remark	Value	Unit	Remark	Value	Unit	Remark	Value	Unit
a_1 =	3.20	(m)	L =	12.22	(m)	h_1 =	2.50	(m)
a_2 =	1.60	(m)	L_1 =	0.15	(m)	h_2 =	9.300	(m)
a_3 =	1.20	(m)	L_2 =	2.12	(m)	h_3 =	3.200	(m)
a_4 =	0.35	(m)	L_3 =	0.80	(m)	h_4 =	0.800	(m)
a_5 =	7.00	(m)	L_4 =	3.10	(m)	h_5 =	1.850	(m)
a_6 =	2.25	(m)	L_5 =	3.00	(m)			
a_7 =	2.50	(m)	L_6 =	3.12	(m)			
			B_1 =	20.50	(m)			
			B_2 =	2.495	(m)			
			B_3 =	2.50	(m)			

1. The design elevation:

				Bearing Pad dimention		
Elevation of surface of deck	EL_{mc} =	17.248	(m)			
Elevation of top of bearing	EL_{xm} =	15.71	(m)	a =	0.35	(m)
Hight water level (H1%)	EL_{MNTK} =	9.20	(m)	b =	0.7	(m)
Daily water level (H5%)	EL_{MNTB} =	4.07	(m)	d =	0.035	(m)
Existing height	EL_{TN} =	4.08	(m)			
Daily water level (H5%)	EL_{TT} =	4.07	(m)			
Section II - II (top footing)	$EL_{MC\ II\text{-}II}$ =	1.56	(m)			
Section III - III (bottom footing)	$EL_{MCIII\text{-}III}$ =	-0.936	(m)			

2.4.2. EXTERIAL FORCE INPUT TO PIER:
2.4.2.1. Dead load of super structure: DC

Load type	Vertical load Nz		Momen due to Left Span		Momen due to Right Span	
	Left span	Right span	e_{xt}	My	e_{xp}	My
	(KN)	(KN)	(m)	(KNm)	(m)	(KNm)
1 - Wearing surface - DW	907.25	907.25	1.200	1088.70	-1.200	-1088.70
2 - Median barrier	422.26	422.26	1.200	506.71	-1.200	-506.71
3 - Side Barrier	516.09	516.09	1.200	619.31	-1.200	-619.31
4 - Railing	0.00	0.00	1.200	0.00	-1.200	0.00
5 - Deck and permanent form (if any)	2574.36	2574.36	1.200	3089.24	-1.200	-3089.24
6 - Cross beam + jointions	268.90	268.90	1.200	322.68	-1.200	-322.68
7 - Super - T girder	4280.64	4280.64	1.200	5136.77	-1.200	-5136.77
Total	**8969.50**	**8969.50**		**10763.40**		**-10763.40**

2.4.2.2. Dead load of pier: DC

Load type	Mass	Unit Weight	Nz Load
	(m3)	(KN/m3)	(KN)
1 - Bearing stone	0.09	24.50	2.10
2 - Pier cap	136.61	24.50	3346.88
3 - Pier body	150.72	24.50	3692.640
4 - Footing	358.75	24.50	8789.38
Total			**15831.00**

2.4.2.3. Live Load: LL + IM

1. Design live load: HL-93

		Truck with 3 axle		Tandem		Lane load		
Weights of axles:	P1 =	35.00	(KN)	110.00	(KN)	W_L =	9.30	(KN/m)
	P2 =	145.00	(KN)	110.00	(KN)			
	P3 =	145.00	(KN)			Pedestrian load		
Spacings of axle	V1 =	4.30	(m)	1.20	(m)	W_N =	0.00	(KN/m^2)
	V2 =	4.30	(m)			Ble =	0.00	(m)

Eccentric (longitudinal) - Left span e^{Left}_x = 1.200 (m)

- Right span e^{Right}_x = -1.200 (m)

Number of lane K = 2x3 (lane)

Dynamic load factor IM = 1.25

2 Live load is mentioned in some case below:

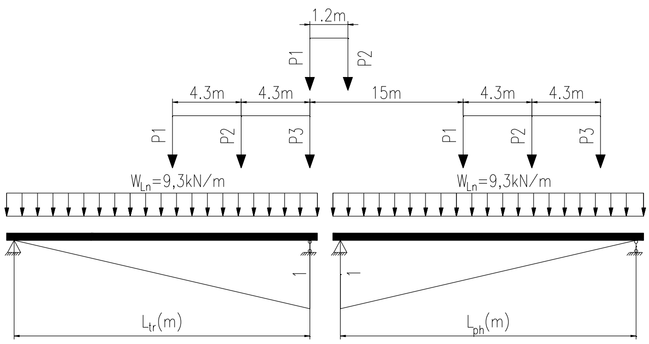
Live load applied on each spans follow below formular:

$$N_z^{Truck} = [P_3+P_2(L - V_2) / L+P_1 (L - V_1 - V_2) / L]*m*K*IM$$

$$N_z^{lane\ load} = 0.5*L*W_L*m*K \qquad N_z^{Tandem} = [P2+P1 (L-V1) / L] *m*K*IM$$

$$N_z^{Pedestrian} = 0.5*L*W_N*Ble*2$$

Caculation diagram



2.1 Live load effect on two spans - All vehicle lane. (Max Nz) - (Strength limit state IA)

Number of lane in this case k = 2x3 (lane)

Lane factor m = 0.85

Eccentric (Horizontal) - for vehicle $e^{vehicle}$ = 0.00 (m)

Live load type	Left span	37.60	(m)	Right span	37.60	(m)
	Nz	Mx =Nz*e_y	My =Nz*e_x	Nz	Mx =Nz*e_y	My =Nz*e_x
	(KN)	(KNm)	(KNm)	(KN)	(KNm)	(KNm)
1 - Truck	1915.13	0.00	2298.15	795.94	0.00	-955.13
2 - Tandem	701.25	0.00	841.50	723.63	0.00	-868.36
3 - Lane Load	891.68	0.00	1070.02	891.68	0.00	-1070.02
HL - 93	**2806.81**	**0.00**	**3368.17**	**1687.63**	**0.00**	**-2025.15**

Live load value:	**Nz = 4494.44 (KN)** **Mx = 0.00 (KNm)** **My = 1343.02 (KNm)**

2.2 Live load effect on two spans - Vehicle on 1/2 Bridge width (Max Mx) - (Strength IB)

Number of lane in this case K = 3.00 (lane)
Lane factor m = 0.85
Eccentric (Horizontal) - for vehicle $e^{vehicle}_{y}$ = 10.250 (m)

Live Load	**Left span**	*37.60*	*(m)*	**Right span**	*37.60*	*(m)*
	Nz	Mx =Nz*e_y	My =Nz*e_x	Nz	Mx =Nz*e_y	My =Nz*e_x
	(KN)	(KNm)	(KNm)	(KN)	(KNm)	(KNm)
1 - Truck	957.56	9815.03	1149.08	397.97	4079.21	-477.57
2 - Tandem	350.63	3593.91	420.75	361.82	3708.61	-434.18
3 - Lane Load	445.84	4569.88	535.01	445.84	4569.88	-535.01
HL - 93	**1403.41**	**14384.91**	**1684.09**	**843.81**	**8649.09**	**-1012.58**

Live load value:	**Nz = 2247.22 (KN)** **Mx = 23034.00 (KNm)** **My = 671.51 (KNm)**

2.3 Live load effect on one Span (Lmax) - All Lane (Max My) - (Strength IC)

Number of lane in this case K = 2x3 (lane)
Lane factor m = 0.85
Eccentric (Horizontal) - for vehicle $e^{vehicle}_{y}$ = 0.00 (m)
Eccentric (longitudinal)- Max Span e^{Lmax}_{x} = 1.200 (m)

Live Load	**Lmax**	*37.60*	*(m)*	
	Nz	Mx =Nz*e_y	My =Nz*e_x	
	(KN)	(KNm)	(KNm)	
1 - Truck	1915.13	0.00	2298.15	Live load value:
2 - Tandem	1380.12	0.00	1656.14	
3 - Lane Load	891.68	0.00	1070.02	**Nz = 2806.81 (KN)**
HL - 93	**2806.81**	**0.00**	**3368.17**	**My = 3368.17 (KNm)**

2.4.2.4. Breaking Force: BR

Breadking force is 25% of total weight of truck axles or of tandem on all lanes
These force shall be assumed to act horizontally at a distance of 1800mm above roadway surface
Assum that have 50% breaking force act to pier with rubber bearing

Section ID	Lane No.	Truck	Tandem	Lane	Truck	Tandem	Z	**My**
	(Lane)	(KN)	(KN)	Factor	(KN)	(KN)	(m)	**(KNm)**
II	2x3	325.00	220.00	0.85	207.19	140.25	17.48	**3622.47**
III	2x3	325.00	220.00	0.85	207.19	140.25	19.98	**4140.44**

2.4.2.5. Stream pressure : WA

$$P_{(D,L)} = 5.14 * 10^{-4} * C_{D(L)} * V^2 \quad \text{(Mpa)}$$

In Which: C_D : Drag coefficient in longitudinal direction.
C_D = 0.70 With circular pier cap
C_L : Drag coefficient in transverse direction
C_L = 0.50
V : Designed velocity of water
V = 2.54 (m/s)

=> Horizontal Stream pressure P_D = 0.00231 (Mpa)
P_D = 2.312 (KN/m^2)

=> Longitudinal Stream pressure P_L = 0.000145 (Mpa)
P_L = 0.145 (KN/m^2)

Water pressure acting on the section follow table below:

Section ID	Horizontal			Longitudinal			Momen		
	Pressure	Area	**Force**	Pressure	Area	**Water Force**	Z	**Mx**	**My**
	(KN/m^2)	(m2)	**(KN)**	(KN/m^2)	(m2)	**(KN)**	(m)	**(KNm)**	**(KNm)**
II	2.312	29.99	**69.34**	0.145	29.99	**4.33**	3.82	**264.75**	**16.54**
III	2.312	47.49	**109.81**	0.145	81.24	**11.74**	5.07	**556.52**	**59.50**

2.4.2.6. Win Load:

6.1 Win load on Structure: WS

(a) Designed win speed:

Designed win velocity follow formula: **$V = V_B . S$**

In which V_B : basic 3 second gust wind velocity.

V_B = 53 (m/s) (Zone II)
S : correction factor for upwind terrain and deck height.
S = 1.09
Designed wind velocity. **V = 57.77 (m/s)**

(b) *Transverse wind load:* $P_D = 0.0006 \cdot V^2 \cdot A_t \cdot C_d$

Inwhich:
V : Designed wind velocity
C_d : drag coefficient specified, it's depened rario b/d.
b: overall width of bridge between outer faces of parapets.
b = 13.00 (m)
d: depth of superstructure, include solid parapets if applicable.
d = 3.11 (m)
Ratio (b / d) = 4.17 **C_d = 1.40**
A_t : Area of the structure for calculation of transverse wind load.
A_t = $A_t^{Rigth\ Span}$ = ($L^{Left\ Span}$ x d_T)
A_t = 119.27 (m^2)
Transverse wind load: **P_D^{trans} = 334.35 (KN)**

(c) *Longitudinal wind load:*
For superstructure with solid elevation, a longitudinal wind load equal to 0.25 times the transverse wind load calculated
Longitudinal wind load: **P_D^{Long} = 83.59 (KN)**

(d) *The wind load applied on the sections:*

Section	Transverse wind load	Longitudinal wind load	Height	Moment	
	P_D^{Trans}	P_D^{Long}	Z	Mx	My
	(KN)	(KN)	(m)	(KNm)	(KNm)
II	334.35	83.59	15.70	5248.96	1312.24
III	334.35	83.59	18.20	6084.84	1521.21

6.2 Wind load on vehicles: WL

(a) *Line wind load*

Transeverse direction: p_y = 1.50 (KN/m) A 3.8.1.3 22TCN 272-05
Longitudinal direction: p_x = 0.75 (KN/m)
Wind load assumed 1.8m above the r di = 1.80 (m)

(b) *Wind load*

Transverse direction
_ Left spa P_D = 56.40 (KN)
_ Right span: P_D = 56.40 (KN)
Total: P^{Trans}_D = 112.80 (KN)

Longitudinal direction
_ Left span: P_D = 28.20 (KN)
_ Right span: P_D = 28.20 (KN)
Total: P^{Long}_D = 56.40 (KN)

(c) *Wind load on vehicles:*

Section	Transverse wind load	Longitudinal wind load	Height	Moment	
	P_D^{Trans}	P_D^{Long}	Z	Mx	My
	(KN)	(KN)	(m)	(KNm)	(KNm)
II	112.80	56.40	17.48	3944.39	1972.20
III	112.80	56.40	19.98	4508.39	2254.20

2.4.2.7. Earthquake effects: EQ

Earthquake class: Class 7.00
Acceleration coefficient A = 0.034
Site coefficient S = 1.20
R : Response modification factors R = 1.00
Elastic modulus Ec = 29440 Mpa
Selftweight of super-structure applied on piers
Wt = 234.19 KN/m

(a) *Determined stiffness:* **K** x(y) **= 3.E.I** x(y) **/ H^3**
(b) *Displacement accodance with unit force* **Vs** x(y) **= Po . L / K** x(y)
(c) *Determined factor a, b, g :*

$$\alpha = \int_{T(n+1)}^{T(n-1)} v_s(x)dx \qquad \beta = \int_{T(n+1)}^{T(n-1)} W_{(x)}V_s(x)dx \qquad \gamma = \int_{T(n+1)}^{T(n-1)} W_{(x)}V_s(x)^2 dx$$

(d) *Determination of period of vibration*

$$T = 2\pi\sqrt{\frac{\gamma}{P_0.g.\alpha}}$$

The summation of result :

Section	H	Ix	Iy	Kx	Ky	Vsx	Vsy
	(m)	(m^4)	(m^4)	(KN/m)	(KN/m)	(m)	(m)
II	14.14	3.83	3.83	119753.38	119753.38	0.0016	0.0003
III	16.64	3.83	3.83	119753.38	119753.38	0.0016	0.0003

Section	H	α_x	α_y	βx	βy	γ_x	γ_y	Tx	Ty
	(m)	(m^2)	(m^2)	KNm	KNm	KNm2	KNm2	(s)	(s)
II	14.14	0.306	0.012	71.717	2.869	0.115	0.0009	1.23	0.55
III	16.64	0.306	0.012	71.717	2.869	0.115	0.0009	1.23	0.55

Inwhich:

Tx, Ty: The period of vibration follow direct X and direct Y.

H: The distance from deck slab to consided sections (m)

$\int_{T(n+1)}^{T(n-1)} xd(x) = L$ The length of Superstructure applied to pier (m)

Ec: Reinforcement concrete elastic modulus. (Mpa)

Ix, Iy: Moment of inertia follow X axial, and follow Y axial. (m^4)

(e) Determination of elastic seismic response coefficient: $C_{sm} = \frac{1,2.A.S}{T_m^{2/3}} \leq 2,5A$

(f) Equivalent uniform static seismic loading $P_e(x) = \frac{\beta C_{sm}}{\gamma} W(x).V(x)$

(g) Designed force applied on pier due to earthquake effect: **F = $Pe_{(x)}$ L / R**

Section	C_x sm	C_y sm	C_x sm	C_y sm	Pe(x)	Pe(y)	Hx	Hy	Z
	Follow theory		Compared with 2.5A		(KN/m)	(KN/m)	(KN)	(KN)	(m)
II	0.043	0.073	0.043	0.073	**10.00**	**17.10**	**0.00**	**654.74**	**14.14**
III	0.043	0.073	0.043	0.073	**10.00**	**17.10**	**0.00**	**654.74**	**16.64**

Section	Hx	Hy	Z	Mx	My
	(KN)	(KN)	(m)	(KNm)	(KNm)
II	-	**654.74**	**14.14**	**9259.36**	-
III	-	**654.74**	**16.64**	**10896.21**	-

(h) Earthquake effect due to substructure

Section II - II								
The Component	**Nz**	**C_x sm**	**C_y sm**	**Hx**	**Hy**	**Z**	**Mx**	**My**
	(KN)			**(KN)**	**(KN)**	(m)	**(KNm)**	**(KNm)**
1 - Pier cap	3346.88	0.043	0.073	142.87	244.31	13.83	1975.24	3377.59
2 - Pier shaft	3692.64	0.043	0.073	157.63	269.55	6.25	985.21	1684.68
Total	**7039.52**			**300.51**	**513.86**		**2960.45**	**5062.28**

Section III - III								
The Component	**Nz**	**C_x sm**	**C_y sm**	**Hx**	**Hy**	**Z**	**Mx**	**My**
	(KN)			**(KN)**	**(KN)**	(m)	**(KNm)**	**(KNm)**
1 - Pier cap	3346.88	0.04	0.07	142.87	244.31	16.33	2332.43	3988.37
2 - Pier shaft	3692.64	0.04	0.07	157.63	269.55	8.75	1379.30	2358.56
4 - Pile cap	8789.38	0.04	0.07	375.21	641.59	1.25	469.01	801.99
Total	**15828.90**			**675.72**	**1155.45**		**4180.74**	**7148.92**

(i) Designed load due to earthquake effect applied on the sections

Section	Hx	Hy	Mx	My
	(KN)	(KN)	(KN.m)	(KN.m)
II - II	**300.51**	**1168.60**	**12219.81**	**5062.28**
III - III	**675.72**	**1810.19**	**15076.95**	**7148.92**

2.4.2.8. Vessel Collision: CV (3.14 - 22TCN272-05)

Class of navigable waterway:	Class	IV	
Mean annual stream velocity:	V_{bq} =	1.15	(m/s)
Design vessel tonnage:	Self-propelled vessel	200.00	(DWT)
	Towed barge	400.00	(DWT)
Dimensions of design vessel:	Maximum length	34.00	(m)
	Maximum breadth	6.60	(m)
	Laden Draught	1.70	(m)
Design collision velocity	Self-propelled vessel	3.65	(m/s)
	Towed barge	2.75	(m/s)

(a) Collision energy of Towed barge:

KE = 500 C_H M V^2 = 1588125 (J)

M Towed barge displacement tonnage M = 400.00 (Mg)

C_H Hydrodynamic mass coefficient C_H = 1.05

Barge bow damage length

$$a_B = 3100.(\sqrt{1+1,3x10^{-7}KE}-1) =$$ 305.00 (mm)

Barge collision force on pier **P_B = 6488.00 (KN)**

(b) Vessel collision force pier:

It's detemined follow formula

V Design collision velocity (m/s)

DWT Deadweight tonnage of vessel (Mg)

$$P_s = 1,2\times10^5 V\sqrt{DWT}$$

Vessel collision force pier: **P_S = 6194.26 (KN)**

On basic of comparison of vessel conllision force beetwen Seft-ropelled vessel and Towed barge, Chose follow table below

The Force follow Perpendicular direction to centerline : CV = 100% max(P_B,P_S)

The force follow longitudinal direction of bridge: CV = 50% max(P_B,P_S)

Vessel collision force (CV) applied to pier at daily water level.

Section	H_x	H_y	Z	Mx	My
	(KN)	(KN)	(m)	(KNm)	(KNm)
II	3244.00	6488.00	2.51	16258.94	8129.47
III	3244.00	6488.00	5.01	32478.95	16239.48

2.4.2.9. The force due to TU, CR, SH

Section	Force (Rx	Force (Ry)	Z	Mx	My
	(KN)	(KN)	(m)	(KNm)	(KNm)
II	273.60	0.00	14.14	0.00	3869.25
III	273.60	0.00	16.64	0.00	4553.25

2.4.3. COMBINATION OF EXTERNAL FORCE - SECTION II - II

	LOAD TYPE & NOTATIONS		Factor γ	Nz (KN)	Hx (KN)	Hy (KN)	Mx (KNm)	My (KNm)
Ia	**STRENGTH LIMIT STATE I_A**							
1	DC	Dead load of left span	1.25	10077.81	-	-	-	12093.38
2	DC	Dead load of right span	1.25	10077.81	-	-	-	-12093.38
3	DW	Dead load of wearing surface	1.50	2721.75	-	-	-	0.00
4	DC	Dead load of pier	1.25	8802.03	-	-	-	-
5	LL	Live load - (maxNz)	1.75	7865.27	-	-	0.00	2350.29
6	BR	Braking force (Start)	1.75	-	362.58	-	-	6339.32
7	WA	Water load and stream pressure	1.00	-	4.33	69.34	264.75	16.54
8	TU+CR+SH	Temperature + Creep + Shinkage	0.50	-	137	-	-	1935
	TOTAL			**39544.68**	**503.71**	**69.34**	**264.75**	**10640.78**

Ib	**STRENGTH LIMIT STATE I_B**							
1	DC	Dead load of left span	0.90	7256.03	-	-	-	8707.23
2	DC	Dead load of right span	0.90	7256.03	-	-	-	-8707.23
3	DW	Dead load of wearing surface	0.65	1179.43	-	-	-	0.00
4	DC	Dead load of pier	0.90	6337.46	-	-	-	-
5	LL	Vehicle Live load - (maxMx)	1.75	3932.63	-	-	40309.50	1175.14
6	BR	Braking force (Start)	1.75	-	362.58	-	-	6339.32
7	WA	Water load and stream pressure	1.00	-	4.33	69.34	264.75	16.54
8	TU+CR+SH	Temperature + Creep + Shinkage	0.50	-	137	-	-	1935
	TOTAL			**25961.57**	**503.71**	**69.34**	**40574.24**	**9465.63**

Ic	**STRENGTH LIMIT STATE I_C**							
1	DC	Dead load of left span	0.90	7256.03	-	-	-	8707.23
2	DC	Dead load of right span	0.90	7256.03	-	-	-	-8707.23
3	DW	Dead load of wearing surface	0.65	1179.43	-	-	-	0.00
4	DC	Dead load of pier	0.90	6337.46	-	-	-	-
5	LL	Vehicle Live load - (maxMy)	1.75	4911.92	-	-	0.00	5894.31
6	BR	Braking force (Start)	1.75	-	362.58	-	-	6339.32
7	WA	Water load and stream pressure	1.00	-	4.33	69.34	264.75	16.54
8	TU+CR+SH	Temperature + Creep + Shinkage	0.50	-	137	-	-	1935
	TOTAL			**26940.86**	**503.71**	**69.34**	**264.75**	**14184.79**

II	**STRENGTH LIMIT STATE II**							
1	DC	Dead load of left span	1.25	10077.81	-	-	-	12093.38
2	DC	Dead load of right span	1.25	7256.03	-	-	-	-12093.38
3	DW	Dead load of wearing surface	1.50	2721.75	-	-	-	0.00
4	DC	Dead load of pier	1.25	8802.03	-	-	-	0.00
5	WA	Water load and stream pressure	1.00	-	4.33	69.34	264.75	16.54
6	WS	Wind load on structure	1.40	-	117.02	468.09	7348.55	1837.14
7	TU+CR+SH	Temperature + Creep + Shinkage	0.50	-	137	-	-	1935
	TOTAL			**28857.62**	**258.16**	**537.43**	**7613.30**	**3788.31**

IIIa	**STRENGTH LIMIT STATE III_A**							
1	DC	Dead load of left span	1.25	10077.81	-	-	-	12093.38
2	DC	Dead load of right span	1.25	10077.81	-	-	-	-12093.38
3	DW	Dead load of wearing surface	1.50	2721.75	-	-	-	0.00
4	DC	Dead load of pier	1.25	8802.03	-	-	-	0.00
5	LL	Live load - (maxNz)	1.35	6067.49	-	-	0.00	1813.08
6	BR	Braking force (Start)	1.35	-	279.70	-	-	4890.33
7	WA	Water load and stream pressure	1.00	-	4.33	69.34	264.75	16.54
8	WS	Wind load on structure	0.40	-	33.44	133.74	2099.58	524.90
9	WL	Wind load on vehicle	1.00	-	56.40	112.80	3944.39	1972.20
10	TU+CR+SH	Temperature + Creep + Shinkage	0.50	-	137	-	-	1935
	TOTAL			**37746.90**	**510.67**	**315.88**	**6308.72**	**11151.67**

IIIb	**STRENGTH III_B**							
1	DC	Dead load of left span	0.90	7256.03	-	-	-	8707.23
2	DC	Dead load of right span	0.90	7256.03	-	-	-	-8707.23
3	DW	Dead load of wearing surface	0.65	1179.43	-	-	-	0.00
4	DC	Dead load of pier	0.90	6337.46	-	-	-	0.00
5	LL	Live load - (max Mx)	1.35	3033.75	-	-	31095.90	906.54
6	BR	Braking force (Start)	1.35	-	279.70	-	-	4890.33
7	WA	Water load and stream pressure	1.00	-	4.33	69.34	264.75	16.54
8	WS	Wind load on structure	0.40	-	33.44	133.74	2099.58	524.90
9	WL	Wind load on vehicle	1.00	-	56.40	112.80	3944.39	1972.20
10	TU+CR+SH	Temperature + Creep + Shinkage	0.50	-	137	-	-	1935
	TOTAL			**25062.69**	**510.67**	**315.88**	**37404.62**	**10245.13**

IIIc	STRENGTH III_C							
1	DC	Dead load of left span	0.90	7256.03	-	-	-	8707.23
2	DC	Dead load of right span	0.90	7256.03	-	-	-	-8707.23
3	DW	Dead load of wearing surface	0.65	1179.43	-	-	-	0.00
4	DC	Dead load of pier	0.90	6337.46	-	-	-	0.00
5	LL	Live load - (max My)	1.35	3789.20	-	-	0.00	4547.04
6	BR	Braking force (Start)	1.35	-	279.70	-	-	4890.33
7	WA	Water load and stream pressure	1.00	-	4.33	69.34	264.75	16.54
8	WS	Wind load on structure	0.40	-	33.44	133.74	2099.58	524.90
9	WL	Wind load on vehicle	1.00	-	56.40	112.80	3944.39	1972.20
10	TU+CR+SH	Temperature + Creep + Shinkage	0.50	-	137	-	-	1935
	TOTAL			**25818.14**	**510.67**	**315.88**	**6308.72**	**13885.63**

IV.A	EXTREME EVENT I_A (100% EQ long + 30% EQ trans)							
1	DC	Dead load of left span	1.25	10077.81	-	-	-	12093.38
2	DC	Dead load of right span	1.25	10077.81	-	-	-	-12093.38
3	DW	Dead load of wearing surface	1.50	2721.75	-	-	-	0.00
4	DC	Dead load of pier	1.25	8802.03	-	-	-	0.00
5	LL	Live load - (maxNz)	0.50	2247.22	-	-	0.00	671.51
6	BR	Braking force (Start)	0.50	-	103.59	-	-	1811.23
7	WA	Water load and stream pressure	1.00	-	4.33	69.34	264.75	16.54
8	FR	Bearing Friction	1.00	-	0.00	-	-	0.00
9	EQ	Earth Quake Load	1.00	-	300.51	350.58	3665.94	5062.28
	TOTAL			**33926.63**	**408.44**	**419.92**	**3930.69**	**7561.57**

IV. B	EXTREME EVENT I_B (100% EQ long + 30% EQ trans)							
1	DC	Dead load of left span	0.90	7256.03	-	-	-	8707.23
2	DC	Dead load of right span	0.90	7256.03	-	-	-	-8707.23
3	DW	Dead load of wearing surface	0.65	1179.43	-	-	-	0.00
4	DC	Dead load of pier	0.90	6337.46	-	-	-	0.00
5	LL	Live load - (maxMy)	0.50	1403.41	-	-	0.00	1684.09
6	BR	Braking force (Start)	0.50	-	103.59	-	31096	0.00
7	WA	Water load and stream pressure	1.00	-	4.33	69.34	264.75	16.54
8	EQ	Earth Quake Load	1.00	-	300.51	350.58	3665.94	5062.28
	TOTAL			**23432.35**	**408.44**	**419.92**	**35026.59**	**6762.91**

IV. C	EXTREME EVENT I_C (30% EQ long + 100% EQ trans)							
1	DC	Dead load of left span	1.25	10077.81	-	-	-	12093.38
2	DC	Dead load of right span	1.25	10077.81	-	-	-	-12093.38
3	DW	Dead load of wearing surface	1.50	2721.75	137	-	-	0.00
4	DC	Dead load of pier	1.25	8802.03	-	-	-	0.00
5	LL	Live load - (maxNz)	0.50	2247.22	-	-	-	672
6	BR	Braking force (Start)	0.50	-	104	-	-	1811
7	WA	Water load and stream pressure	1.00	-	4	69	265	17
8	EQ	Earth Quake Load	1.00	-	90	1169	12220	1519
	TOTAL			**33926.63**	**334.88**	**1237.94**	**12484.56**	**4017.97**

IV. D	EXTREME EVENT I_D (30% EQ long + 100% EQ trans)							
1	DC	Dead load of left span	0.90	7256.03	-	-	-	8707.23
2	DC	Dead load of right span	0.90	7256.03	-	-	-	-8707.23
3	DW	Dead load of wearing surface	0.65	1179.43	-	-	-	0.00
4	DC	Dead load of pier	0.90	6337.46	-	-	-	0.00
5	LL	Live load - (maxMx)	0.50	1123.61	-	-	11517	336
6	BR	Braking force (Start)	0.50	-	104	-	-	1811
7	WA	Water load and stream pressure	1.00	-	4	69	265	17
8	EQ	Earth Quake Load	1.00	-	90	1169	12220	1519
	TOTAL			**23152.55**	**198.08**	**1237.94**	**24001.56**	**3682.22**

V	EXTREME EVENT II_A							
1	DC	Dead load of left span	1.25	10077.81	-	-	-	12093.38
2	DC	Dead load of right span	1.25	10077.81	-	-	-	-12093.38
3	DW	Dead load of wearing surface	1.50	2721.75	-	-	-	0.00
4	DC	Dead load of pier	1.25	8802.03	-	-	-	0.00
5	LL	Live load - (maxNz)	0.50	2247.22	-	-	0.00	671.51
6	BR	Braking force (Start)	0.50	-	103.59	-	-	1811.23
7	WA	Water load and stream pressure	1.00	-	4.33	69.34	264.75	16.54
8	CV	Vessels Impact	1.00	-	3244.00	6488.00	16258.94	8129.47
	TOTAL			**33926.63**	**3351.93**	**6557.35**	**16523.69**	**10628.76**

V	**EXTREME EVENT II_B**							
1	DC	Dead load of left span	0.90	7256.03	-	-	-	8707.23
2	DC	Dead load of right span	0.90	7256.03	-	-	-	-8707.23
3	DW	Dead load of wearing surface	0.65	1179.43	-	-	-	0.00
4	DC	Dead load of pier	0.90	6337.46	-	-	-	0.00
5	LL	Live load - (maxMx)	0.50	1123.61	-	-	11517.00	335.76
6	BR	Braking force (Start)	0.50	-	104	-	-	1811
7	WA	Water load and stream pressure	1.00	-	4.33	69.34	264.75	16.54
8	CV	Vessels Impact	1.00	-	3244	6488	16259	8129
	TOTAL			**23152.55**	**3351.93**	**6557.35**	**28040.69**	**10293.00**

VII	**SERVICE STATE I_A**							
1	DC	Dead load of left span	1.00	8062.25	-	-	-	9674.70
2	DC	Dead load of right span	1.00	8062.25	-	-	-	-9674.70
3	DW	Dead load of wearing surface	1.00	1814.50	-	-	-	0.00
4	DC	Dead load of pier	1.00	7041.63	-	-	-	0.00
5	LL	Live load - (max Nz)	1.00	4494.44	-	-	0.00	1343.02
6	BR	Braking force (Start)	1.00	-	207.19	-	-	3622.47
7	WA	Water load and stream pressure	1.00	-	4.33	69.34	264.75	16.54
8	WS	Wind load on structure	0.30	-	25.08	100.31	1574.69	393.67
9	WL	Wind load on vehicle	1.00	-	56.40	112.80	3944.39	1972.20
10	TU+CR+SH	Temperature + Creep + Shinkage	1.00	-	274	-	-	4553
	TOTAL			**29475.07**	**566.60**	**282.45**	**5783.83**	**11901.15**

VIII	**SERVICE STATE I_B**							
1	DC	Dead load of left span	1.00	8062.25	-	-	-	9674.70
2	DC	Dead load of right span	1.00	8062.25	-	-	-	-9674.70
3	DW	Dead load of wearing surface	1.00	1814.50	-	-	-	0.00
4	DC	Dead load of pier	1.00	7041.63	-	-	-	0.00
5	LL	Live load - (max Mx)	1.00	2247.22	-	-	23034.00	671.51
6	BR	Braking force (Start)	1.00	-	207.19	-	-	3622.47
7	WA	Water load and stream pressure	1.00	-	4.33	69.34	264.75	16.54
8	WS	Wind load on structure	0.30	-	25.08	100.31	1574.69	393.67
9	WL	Wind load on vehicle	1.00	-	56.40	112.80	3944.39	1972.20
10	TU+CR+SH	Temperature + Creep + Shinkage	1.00	-	274	-	-	4553.25
	TOTAL			**27227.85**	**566.60**	**282.45**	**28817.83**	**11229.64**

IX	**SERVICE STATE I_A**							
1	DC	Dead load of left span	1.00	8062.25	-	-	-	9674.70
2	DC	Dead load of right span	1.00	8062.25	-	-	-	-9674.70
3	DW	Dead load of wearing surface	1.00	1814.50	-	-	-	0.00
4	DC	Dead load of pier	1.00	7041.63	-	-	-	0.00
5	LL	Live load - (max My)	1.00	2806.81	-	-	0.00	3368.17
6	BR	Braking force (Start)	1.00	-	207.19	-	-	3622.47
7	WA	Water load and stream pressure	1.00	-	4.33	69.34	264.75	16.54
8	WS	Wind load on structure	0.30	-	25.08	100.31	1574.69	393.67
9	WL	Wind load on vehicle	1.00	-	56.40	112.80	3944.39	1972.20
10	TU+CR+SH	Temperature + Creep + Shinkage	1.00	-	274	-	-	4553.25
	TOTAL			**27787.44**	**566.60**	**282.45**	**5783.83**	**13926.30**

COMBINATION EXTERNAL FORCE ON SECTION II - II

State	**Load factors specified**			**Nz**	**Hx**	**Hy**	**Mx**	**My**
	η_D	η_R	η_I	(KN)	(KN)	(KN)	(KNm)	(KNm)
Strength I_A	1.00	1.00	1.00	39545	504	69	265	10641
Strength I_B	1.00	1.00	1.00	25962	504	69	40574	9466
Strength I_C	1.00	1.00	1.00	26941	504	69	265	14185
Strength II	1.00	1.00	1.00	28858	258	537	7613	3788
Strength III_A	1.00	1.00	1.00	37747	511	316	6309	11152
Strength III_B	1.00	1.00	1.00	25063	511	316	37405	10245
Strength III_C	1.00	1.00	1.00	25818	511	316	6309	13886
Extreme Event I_A	1.00	1.00	1.00	33927	408	420	3931	7562
Extreme Event I_B	1.00	1.00	1.00	23432	408	420	35027	6763
Extreme Event I_C	1.00	1.00	1.00	33927	335	1238	12485	4018
Extreme Event I_D	1.00	1.00	1.00	23153	198	1238	24002	3682
Extreme Event II_A	1.00	1.00	1.00	33927	3352	6557	16524	10629
Extreme Event II_B	1.00	1.00	1.00	23153	3352	6557	28041	10293
Service state I_A	1.00	1.00	1.00	29475	567	282	5784	11901
Service state I_B	1.00	1.00	1.00	27228	567	282	28818	11230
Service state I_C	1.00	1.00	1.00	27787	567	282	5784	13926

2.4.4. COMBINATION OF EXTERNAL FORCE - SECTION III - III

	LOAD TYPE & NOTATIONS		Factor γ	Nz (KN)	Hx (KN)	Hy (KN)	Mx (KNm)	My (KNm)
Ia	**STRENGTH STATE I_A**							
1	DC	Dead load of left span	1.25	10077.81	-	-	-	12093.38
2	DC	Dead load of right span	1.25	10077.81	-	-	-	-12093.38
3	DW	Dead load of wearing surface	1.50	2721.75	-	-	-	0.00
4	DC	Dead load of pier	1.25	19788.75	-	-	-	-
5	LL	Live load - (maxNz)	1.75	7865.27	-	-	0.00	2350.29
6	BR	Braking force (Start)	1.75	-	362.58	-	-	7245.76
7	WA	Water load and stream pressure	1.00	-	11.74	109.81	556.52	59.50
8	TU+CR+SH	Temperature + Creep + Shinkage	0.50	-	137	-	-	2277
	TOTAL			**50531.40**	**511.12**	**109.81**	**556.52**	**11932.17**

Ib	**STRENGTH STATE I_B**							
1	DC	Dead load of left span	1.25	10077.81	-	-	-	12093.38
2	DC	Dead load of right span	1.25	10077.81	-	-	-	-12093.38
3	DW	Dead load of wearing surface	1.50	2721.75	-	-	-	0.00
4	DC	Dead load of pier	1.25	19788.75	-	-	-	-
5	LL	Live load - (max Mx)	1.75	3932.63	-	-	40309.50	1175.14
6	BR	Braking force (Start)	1.75	-	362.58	-	-	7245.76
7	WA	Water load and stream pressure	1.00	-	11.74	109.81	556.52	59.50
8	TU+CR+SH	Temperature + Creep + Shinkage	0.50	-	137	-	-	2277
	TOTAL			**46598.76**	**511.12**	**109.81**	**40866.01**	**10757.03**

Ic	**STRENGTH STATE I_C**							
1	DC	Dead load of left span	1.25	10077.81	-	-	-	12093.38
2	DC	Dead load of right span	1.25	10077.81	-	-	-	-12093.38
3	DW	Dead load of wearing surface	1.50	2721.75	-	-	-	0.00
4	DC	Dead load of pier	1.25	19788.75	-	-	-	-
5	LL	Vehicle Live load - (maxMy)	1.75	4911.92	-	-	0.00	5894.31
6	BR	Braking force (Start)	1.75	-	362.58	-	-	7245.76
7	WA	Water load and stream pressure	1.00	-	11.74	109.81	556.52	59.50
8	TU+CR+SH	Temperature + Creep + Shinkage	0.50	-	137	-	-	2277
	TOTAL			**47578.05**	**511.12**	**109.81**	**556.52**	**15476.19**

II	**STRENGTH STATE II**							
1	DC	Dead load of left span	1.25	10077.81	-	-	-	12093.38
2	DC	Dead load of right span	1.25	10077.81	-	-	-	-12093.38
3	DW	Dead load of wearing surface	1.50	2721.75	-	-	-	0.00
4	DC	Dead load of pier	1.25	19788.75	-	-	-	0.00
5	WA	Water load and stream pressure	1.00	-	11.74	109.81	556.52	59.50
6	WS	Wind load on structure	1.40	-	117.02	468.09	8518.77	2129.69
7	TU+CR+SH	Temperature + Creep + Shinkage	0.50	-	137	-	-	2277
	TOTAL			**42666.13**	**265.56**	**577.90**	**9075.29**	**4465.81**

IIIa	**STRENGTH STATE III-A**							
1	DC	Dead load of left span	1.25	10077.81	-	-	-	12093.38
2	DC	Dead load of right span	1.25	10077.81	-	-	-	-12093.38
3	DW	Dead load of wearing surface	1.50	2721.75	-	-	-	0.00
4	DC	Dead load of pier	1.25	19788.75	-	-	-	0.00
5	LL	Live load - (maxNz)	1.35	6067.49	-	-	0.00	1813.08
6	BR	Braking force (Start)	1.35	-	279.70	-	-	5589.59
7	WA	Water load and stream pressure	1.00	-	11.74	109.81	556.52	59.50
8	WS	Wind load on structure	0.40	-	33.44	133.74	2433.93	608.48
9	WL	Wind load on vehicle	1.00	-	56.40	112.80	4508.39	2254.20
10	TU+CR+SH	Temperature + Creep + Shinkage	0.50	-	137	-	-	2277
	TOTAL			**48733.62**	**518.08**	**356.35**	**7498.84**	**12601.47**

IIIb	**STRENGTH STATE III-B**							
1	DC	Dead load of left span	1.25	10077.81	-	-	-	12093.38
2	DC	Dead load of right span	1.25	10077.81	-	-	-	-12093.38
3	DW	Dead load of wearing surface	1.50	2721.75	-	-	-	0.00
4	DC	Dead load of pier	1.25	19788.75	-	-	-	0.00
5	LL	Live load - (max Mx)	1.35	3033.75	-	-	31095.90	906.54
7	BR	Braking force (Start)	1.35	-	279.70	-	-	5589.59
7	WA	Water load and stream pressure	1.00	-	11.74	109.81	556.52	59.50
8	WS	Wind load on structure	0.40	-	33.44	133.74	2433.93	608.48
9	WL	Wind load on vehicle	1.00	-	56.40	112.80	4508.39	2254.20
10	TU+CR+SH	Temperature + Creep + Shinkage	0.50	-	137	-	-	2277
	TOTAL			**45699.88**	**518.08**	**356.35**	**38594.74**	**11694.93**

IIIc	STRENGTH STATE III-C							
1	DC	Dead load of left span	1.25	10077.81	-	-	-	12093.38
2	DC	Dead load of right span	1.25	10077.81	-	-	-	-12093.38
3	DW	Dead load of wearing surface	1.50	2721.75	-	-	-	0.00
4	DC	Dead load of pier	1.25	19788.75	-	-	-	0.00
5	LL	Live load - (max My)	1.35	3789.20	-	-	0.00	4547.04
6	BR	Braking force (Start)	1.35	-	279.70	-	-	5589.59
7	WA	Water load and stream pressure	1.00	-	11.74	109.81	556.52	59.50
8	WS	Wind load on structure	0.40	-	33.44	133.74	2433.93	608.48
9	WL	Wind load on vehicle	1.00	-	56.40	112.80	4508.39	2254.20
10	TU+CR+SH	Temperature + Creep + Shinkage	0.50	-	137	-	-	2277
	TOTAL			**46455.33**	**518.08**	**356.35**	**7498.84**	**15335.42**

IV	EXTREME EVENT I_A							
1	DC	Dead load of left span	1.25	10077.81	-	-	-	12093.38
2	DC	Dead load of right span	1.25	10077.81	-	-	-	-12093.38
3	DW	Dead load of wearing surface	1.50	2721.75	-	-	-	0.00
4	DC	Dead load of pier	1.25	15831.00	-	-	-	0.00
5	LL	Live load - (maxMx)	0.50	1123.61	-	-	11517.00	335.76
6	BR	Braking force (Start)	0.50	-	103.59	-	-	2070.22
7	WA	Water load and stream pressure	1.00	-	11.74	109.81	556.52	59.50
8	EQ	Earth Quake Load	1.00	-	202.72	1810.19	15076.95	2144.68
	TOTAL			**39831.99**	**318.05**	**1920.00**	**27150.46**	**4610.14**

IV	EXTREME EVENT I_B							
1	DC	Dead load of left span	1.25	10077.81	-	-	-	12093.38
2	DC	Dead load of right span	1.25	10077.81	-	-	-	-12093.38
3	DW	Dead load of wearing surface	1.50	2721.75	-	-	-	0.00
4	DC	Dead load of pier	1.25	15831.00	-	-	-	0.00
5	LL	Live load - (maxMy)	0.50	1403.41	-	-	0.00	1684.09
6	BR	Braking force (Start)	0.50	-	104	-	-	2070
7	WA	Water load and stream pressure	1.00	-	12	110	557	59
8	EQ	Earth Quake Load	1.00	-	675.72	543.06	4523.08	7148.92
	TOTAL			**40111.79**	**791.05**	**652.87**	**5079.60**	**10962.72**

V	EXTREME EVENT II							
1	DC	Dead load of left span	1.25	10077.81	-	-	-	12093.38
2	DC	Dead load of right span	1.25	10077.81	-	-	-	-12093.38
3	DW	Dead load of wearing surface	1.50	2721.75	-	-	-	0.00
4	DC	Dead load of pier	1.25	15831.00	-	-	-	0.00
5	LL	Live load - (maxNz)	0.50	1123.61	-	-	11517.00	335.76
6	BR	Braking force (Start)	0.50	-	103.59	-	-	2070.22
7	WA	Water load and stream pressure	1.00	-	11.74	109.81	556.52	59.50
8	CV	Vessels Impact	1.00	-	3244.00	6488.00	32478.95	16239.48
	TOTAL			**39831.99**	**3359.34**	**6597.81**	**44552.47**	**18704.94**

VII	SERVICE STATE I_A							
1	DC	Dead load of left span	1.00	8062.25	-	-	-	9674.70
2	DC	Dead load of right span	1.00	8062.25	-	-	-	-9674.70
3	DW	Dead load of wearing surface	1.00	1814.50	-	-	-	0.00
4	DC	Dead load of pier	1.00	15831.00	-	-	-	0.00
5	LL	Live load -(max Nz)	1.00	4494.44	-	-	0.00	1343.02
6	BR	Braking force (Start)	1.00	-	207.19	-	-	4140.44
7	WA	Water load and stream pressure	1.00	-	11.74	109.81	556.52	59.50
8	WS	Wind load on structure	0.30	-	25.08	100.31	1825.45	456.36
9	WL	Wind load on vehicle	1.00	-	56.40	112.80	4508.39	2254.20
10	TU+CR+SH	Temperature + Creep + Shinkage	1.00	-	274	-	-	4553
	TOTAL			**38264.44**	**574.00**	**322.92**	**6890.36**	**12806.76**

VIII	SERVICE STATE I_B							
1	DC	Dead load of left span	1.00	8062.25	-	-	-	9674.70
2	DC	Dead load of right span	1.00	8062.25	-	-	-	-9674.70
3	DW	Dead load of wearing surface	1.00	1814.50	-	-	-	0.00
4	DC	Dead load of pier	1.00	15831.00	-	-	-	0.00
5	LL	Live load - (max Mx)	1.00	2247.22	-	-	23034.00	671.51
6	BR	Braking force (Start)	1.00	-	207.19	-	-	4140.44
7	WA	Water load and stream pressure	1.00	-	11.74	109.81	556.52	59.50
8	WS	Wind load on structure	0.30	-	25.08	100.31	1825.45	456.36
9	WL	Wind load on vehicle	1.00	-	56.40	112.80	4508.39	2254.20
10	TU+CR+SH	Temperature + Creep + Shinkage	1.00	-	274	-	-	4553.25
	TOTAL			**36017.22**	**574.00**	**322.92**	**29924.36**	**12135.25**

IX	SERVICE STATE I_C							
1	DC	Dead load of left span	1.00	8062.25	-	-	-	9674.70
2	DC	Dead load of right span	1.00	8062.25	-	-	-	-9674.70
3	DW	Dead load of wearing surface	1.00	1814.50	-	-	-	0.00
4	DC	Dead load of pier	1.00	15831.00	-	-	-	0.00
5	LL	Live load - (max My)	1.00	2806.81	-	-	0.00	3368.17
6	BR	Braking force (Start)	1.00	-	207.19	-	-	4140.44
7	WA	Water load and stream pressure	1.00	-	11.74	109.81	556.52	59.50
8	WS	Wind load on structure	0.30	-	25.08	100.31	1825.45	456.36
9	WL	Wind load on vehicle	1.00	-	56.40	112.80	4508.39	2254.20
10	TU+CR+SH	Temperature + Creep + Shinkage	1.00	-	274	-	-	4553.25
	TOTAL			**36576.82**	**574.00**	**322.92**	**6890.36**	**14831.91**

COMBINATION EXTERNAL FORCE ON SECTION III - III

State	Load factors specified			Nz	Hx	Hy	Mx	My
	η_D	η_R	η_I	(KN)	(KN)	(KN)	(KNm)	(KNm)
Strength I A	1.00	1.00	1.00	50531.40	511.12	109.81	556.52	11932.17
Strength I B	1.00	1.00	1.00	46598.76	511.12	109.81	40866.01	10757.03
Strength I C	1.00	1.00	1.00	47578.05	511.12	109.81	556.52	15476.19
Strength II	1.00	1.00	1.00	42666.13	265.56	577.90	9075.29	4465.81
Strength III A	1.00	1.00	1.00	48733.62	518.08	356.35	7498.84	12601.47
Strength III B	1.00	1.00	1.00	45699.88	518.08	356.35	38594.74	11694.93
Strength III C	1.00	1.00	1.00	46455.33	518.08	356.35	7498.84	15335.42
Extreme event I A	1.00	1.00	1.00	39831.99	318.05	1920.00	27150.46	4610.14
Extreme event I B	1.00	1.00	1.00	40111.79	791.05	652.87	5079.60	10962.72
Extreme event II	1.00	1.00	1.00	39831.99	3359.34	6597.81	44552.47	18704.94
Service state I A	1.00	1.00	1.00	38264.44	574.00	322.92	6890.36	12806.76
Service state I B	1.00	1.00	1.00	36017.22	574.00	322.92	29924.36	12135.25
Service state I C	1.00	1.00	1.00	36576.82	574.00	322.92	6890.36	14831.91

2.4.5. DETEMINAL INTERNAL FORCE AT TOP OF PILE

2.4.5.1. Summary of external force acting to bottom of footing:

Combination	V	Hx	Hy	Mx	My
	(KN)	(Kn)	(Kn)	(Kn.m)	(Kn.m)
Strength I	50531	511	110	40866	15476
Strength II	42666	266	578	9075	4466
Strength III	48734	518	356	38595	15335
Service I	38264	574	323	29924	14832
Extreme event IA (EQ)	39832	318	1920	27150	4610
Extreme event IB (EQ)	40112	791	653	5080	10963
Extreme event I (CV)	39832	3359	6598	44552	18705

2.4.5.2. Piling material:

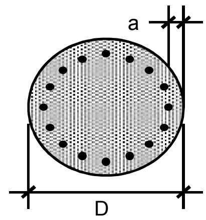
Concrete 30 Mpa

E_c (Mpa)	29440
γ_c (KN/m^3)	2450

Steel bar

Type	CB-400-T
E_s (Mpa)	200000

2.4.5.3. Piling dimension



+ Diameter **D** = 1.50 m

a = 0.075 m

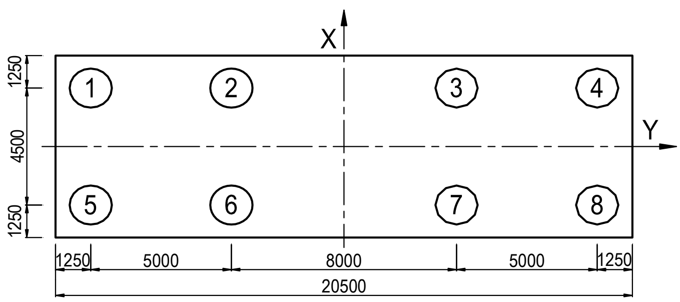
+ Length **L** = 62.00 m

2.4.5.4. Maximum Internal force and displacements at top piling

Internal force and diplacements *(Result follow Piling software)*

Combination	N	H	M	x	y	z
	(KN)	(KN)	(KN.m)	(m)	(m)	(rad)
Strength I	8262.38	63.88	-36.41	-	-	-
Strength II	5917.47	33.25	48.85	-	-	-
Strength III	8003.10	64.75	-30.25	-	-	-
Service I	6447.96	71.75	7.30	**0.003**	**-0.001**	**0.008**
Extreme IA	6231.76	71.75	7.30	-	-	-
Extreme IB	5924.48	98.88	189.04	-	-	-
Extreme II	8624.91	419.88	1328.75	-	-	-

- Check displacement of top pile not exceed 38mm (10.7.2.7) **OK**



Arrangement of pile

- Result for internal force at top pile

Combination	Pile Number	Hx	Hy	N	Mx	My
		(KN)	(KN)	(KN.m)	(m)	(m)
Strength I	1	63.88	-13.75	6575.20	-55.52	-36.41
	2	63.88	-13.75	5840.25	-55.52	-36.41
	3	63.88	-13.75	5105.31	-55.52	-36.41
	4	63.88	-13.75	4370.37	-55.52	-36.41
	5	63.88	-13.75	8262.38	-55.52	-36.41
	6	63.88	-13.75	7527.44	-55.52	-36.41
	7	63.88	-13.75	6792.50	-55.52	-36.41
	8	63.88	-13.75	6057.56	-55.52	-36.41
Strength II	1	33.25	-72.25	5377.83	306.61	48.85
	2	33.25	-72.25	5168.23	306.61	48.85
	3	33.25	-72.25	4958.63	306.61	48.85
	4	33.25	-72.25	4749.03	306.61	48.85
	5	33.25	-72.25	5917.47	306.61	48.85
	6	33.25	-72.25	5707.87	306.61	48.85
	7	33.25	-72.25	5498.27	306.61	48.85
	8	33.25	-72.25	5288.68	306.61	48.85
Strength III	1	64.75	-44.50	6326.09	92.82	-30.25
	2	64.75	-44.50	5610.86	92.82	-30.25
	3	64.75	-44.50	4895.63	92.82	-30.25
	4	64.75	-44.50	4180.41	92.82	-30.25
	5	64.75	-44.50	8003.10	92.82	-30.25
	6	64.75	-44.50	7287.87	92.82	-30.25
	7	64.75	-44.50	6572.64	92.82	-30.25
	8	64.75	-44.50	5857.41	92.82	-30.25
Service	1	71.75	-40.38	4793.47	99.04	7.30
	2	71.75	-40.38	4235.00	99.04	7.30
	3	71.75	-40.38	3676.52	99.04	7.30
	4	71.75	-40.38	3118.04	99.04	7.30
	5	71.75	-40.38	6447.96	99.04	7.30
	6	71.75	-40.38	5889.48	99.04	7.30
	7	71.75	-40.38	5331.01	99.04	7.30
	8	71.75	-40.38	4772.53	99.04	7.30
Extreme event IA	1	39.75	-240.00	5655.39	1027.23	72.17
	2	39.75	-240.00	5012.34	1027.23	72.17
	3	39.75	-240.00	4369.29	1027.23	72.17
	4	39.75	-240.00	3726.24	1027.23	72.17
	5	39.75	-240.00	6231.76	1027.23	72.17
	6	39.75	-240.00	5588.71	1027.23	72.17
	7	39.75	-240.00	4945.66	1027.23	72.17
	8	39.75	-240.00	4302.61	1027.23	72.17
Extreme event IB	1	98.88	-81.63	4538.34	361.45	189.04
	2	98.88	-81.63	4393.40	361.45	189.04
	3	98.88	-81.63	4248.46	361.45	189.04
	4	98.88	-81.63	4103.52	361.45	189.04
	5	98.88	-81.63	5924.48	361.45	189.04
	6	98.88	-81.63	5779.54	361.45	189.04
	7	98.88	-81.63	5634.60	361.45	189.04
	8	98.88	-81.63	5489.66	361.45	189.04
Extreme event II	1	419.88	-824.75	5365.46	3671.84	1328.75
	2	419.88	-824.75	4021.34	3671.84	1328.75
	3	419.88	-824.75	2677.22	3671.84	1328.75
	4	419.88	-824.75	1333.10	3671.84	1328.75
	5	419.88	-824.75	8624.91	3671.84	1328.75
	6	419.88	-824.75	7280.78	3671.84	1328.75
	7	419.88	-824.75	5936.66	3671.84	1328.75
	8	419.88	-824.75	4592.54	3671.84	1328.75

2.4.6. ULTIMATE LOAD CHECK, SHEAR CAPACITY AND CRACK CHECK

2.4.6.1. Check for pier shaft:

Item		Mark	Unit	Value
• Factored Axial force		Nu	Kn	13613.92
• Factored Plexural moment		Mux	Kn.m	14408.91
• Factored Plexural moment		Muy	Kn.m	5614.82
• Diameter of Pier shaft		D	m	2.50
• Section area		Ag	m2	4.91
• Moment of inertia of concrete section		Ic	m4	1.92
• Cover thickness		a	m	0.008
• Reinf. Diameter		Ds	mm	32.00
• Number of rebar		n_s	nos	64.00
• Rebar area		As	mm2	51471.85
Check minimum reinforcement				
• Minimum rebar area required (0.135*fy/f'c)*Ag		As req	mm2	49700.98
• Check condition As > (0.135*fy/f'c)*Ag				**OK**
Check maximum reinforcement				
• Maximum rebar area 0.08*Ag		As max	mm2	392699.1
• Check condition As < 0.08*Ag				**OK**
Check ratio spiral or Tier (5.7.4.6)				
• Distance to outside of Spairal or Ties to concrete face			mm	62.00
• Effect diamete		Deff	m	2.38
• Area of core measured to the outside diameter of the spiral			m2	4.43
• Ratio spiral Rebar required		ρsa		0.00361
Required Area of Spiral Rebar	space		mm	150
	Effective length			2.38
	layer			1
	Area			322.1
	Requaired Dhs			20.3
Actuaral	Effective length	d	m	2.376
	Diameter	Dhr	mm	22
	Area of Rebar	Ah	mm2	380.1
	layer	Nl	nos	1
	Total area of spiral	Ac	m2	380.133
	space	s	mm	150
	Ratio spiral Rebar	ρs	-	0.0042664
• Check condition		$\rho_s > \rho_{sa}$		OK
Check Crack (At Service state)				
• Modulus of rupture of concrete		fr = 0.63*sqrt(f'c)	Mpa	3.45
• Stress of concrete at tension fiber		σ'_r	Mpa	6
• If f'r > 0.8fr require check crack		$\sigma'_r > 0.8*\sigma_r$	Mpa	Check
• Center of newtral axial		x	mm	1.24
• Maximum stress of Compression fiber of concrete		σ_c	Mpa	13.3
• Maximum stress of Compression Rebar		σ_{rc}	Mpa	-177.6
• Maximum stress of Tension Rebar		σ_{rt}	Mpa	181.2
• Check			σ_{rt} <0.6.fy	OK

Company	Nipponkoei	**Project Name**	DN Qn Expressway
Designer	vinh	**File Name**	D:\...Pier Shaft-D2.5m-Pier P9.BOI

1. Geometry and Materials

Design Code : ACI318M-02

Stress Profile : Equivalent Stress Block

Material Data : f_c' = 30 MPa (β_1 = 0.850)

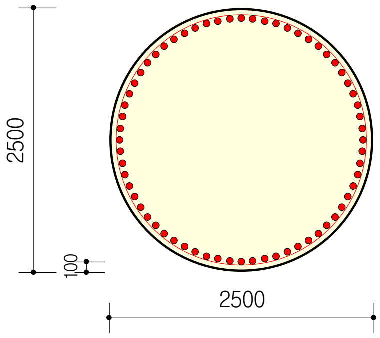
f_y = 400, f_{ys} = 400 MPa

Section Dim. : Φ2500 mm

Effective Len. : KL_u = 12500 mm

Steel Distribut.: 64 - P32 (d_c = 100 mm)

Total Steel Area A_{st} = 51472 mm^2 (ρ_{st} = 0.0105)



2. Member Force and Moment

Unit : kN, kN-m

L.C.	P_u	M_{ux}	M_{uy}	R_{atioV}	V_{ux}	V_{uy}	R_{atioH}	Remark
1	19772.0	132.0	5320.0	0.279	252.0	35.0	0.018	
2	12981.0	20287.0	4733.0	0.631	252.0	35.0	0.018	
3	13470.0	132.0	7092.0	0.260	252.0	35.0	0.018	
4	14429.0	3807.0	1894.0	0.209	129.0	269.0	0.022	
5	18873.0	3154.0	5576.0	0.289	255.0	158.0	0.022	
6	12531.0	18702.0	5123.0	0.577	255.0	158.0	0.022	
7	12909.0	3154.0	6943.0	0.270	255.0	158.0	0.022	
8	16963.0	1965.0	3781.0	0.234	204.0	210.0	0.021	
9	11716.0	17513.0	3381.0	0.527	204.0	210.0	0.021	
10	16963.0	6242.0	2009.0	0.275	167.0	619.0	0.046	
11	11576.0	12001.0	1841.0	0.365	99.0	619.0	0.045	
12	16963.0	16524.0	10629.0	0.584	3352.0	6557.0	0.531	
13	11576.0	22282.0	10293.0	0.842	3352.0	6557.0	0.534	
14	14738.0	2892.0	5951.0	0.258	283.0	141.0	0.023	
15	13614.0	14409.0	5615.0	0.459	283.0	141.0	0.023	
16	13894.0	2892.0	6963.0	0.274	283.0	141.0	0.023	

3. Magnified Moment

KL_u/r_x = 12500/625 = 20.00 < $34-12(M_1/M_2)$ = 22.00

δ_x = 1.000

KL_u/r_y = 12500/625 = 20.00 < $34-12(M_1/M_2)$ = 22.00

δ_y = 1.000

4. Design Force and Moment

Design Load Combination No : 13

P_u = 11576.0 kN

M_{ux} = 22282.0, M_{uy} = 10293.0 kN-m

5. Check Axial and Moment Capacity

Rotation Angle and Depth to the Neutral Axis θ = −65.21°, c = 735 mm

Strength Reduction Factor Φ = 0.9000

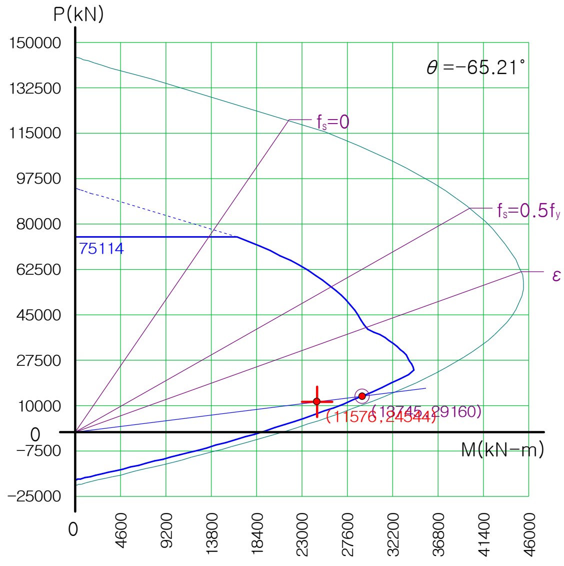
Maximum Axial Load $\Phi P_{n(max)}$ = 75113.5 kN

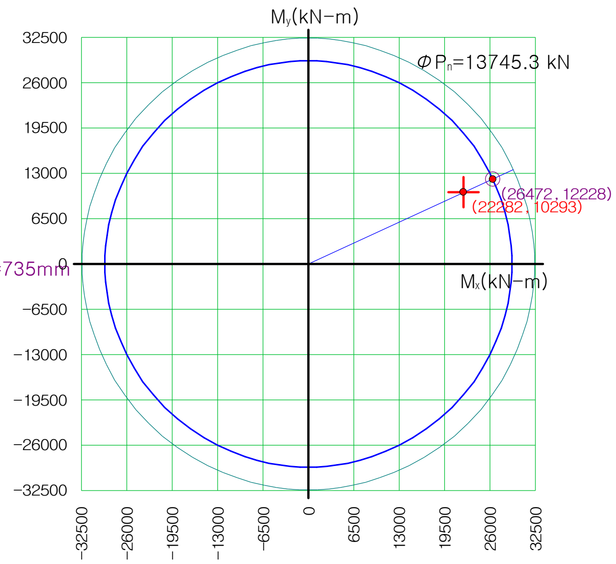
Design Axial Load Strength ΦP_n = 13745.3 kN

Design Moment Strength ΦM_{nx} = 26472.1 kN−m

ΦM_{ny} = 12227.9 kN−m

Strength Ratio : Applied/Design = 0.842 < 1.000 .... O.K





6. Check Shear Capacity

Design Load Combination No : 13

Strength Reduction Factor Φ= 0.750

Design Force Vu = 7364.1 kN (Pu = 11576.0 kN)

Required Hoop Spacing : D22 @ 361 mm

Provided Hoop Spacing : D22 @ 150 mm (Tie)

$\Phi V_c + \Phi V_s$ = 10721.5 + 3069.1 = 13790.6 kN > V_u = 7364.1 kN .... O.K

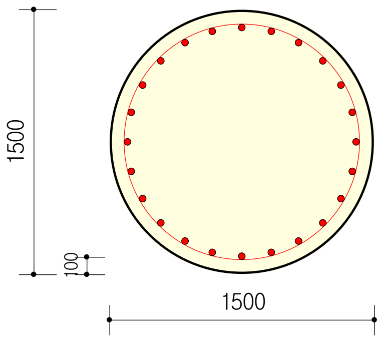
2.4.6.2. Check for pile:

Item		Mark	Unit	Value
• Factored Axial force		Nu	Kn	6447.96
• Factored Plexural moment		Mux	Kn.m	99.04
• Factored Plexural moment		Muy	Kn.m	7.30
• Diameter of Pier shaft		D	m	1.50
• Section area		Ag	m2	1.77
• Moment of inertia of concrete section		Ic	m4	0.25
• Cover thickness		a	m	0.075
• Reinf. Diameter		Ds	mm	D32
• Number of rebar		n_s	nos	24
• Rebar area		As	mm2	19301.95
Check minimum reinforcement				
• Minimum rebar area required (0.135*f'c/f'y)*Ag		As req	mm2	17892.35
• Check condition As > (0.135*f'c/fy)*Ag				**OK**
Check maximum reinforcement				
• Maximum rebar area 0.08*Ag		As max	mm2	141371.7
• Check condition As < 0.08*Ag				**OK**
Check ratio spiral or Tier (5.7.4.6)				
• Distance to outside of Spairal or Ties to concrete face			mm	68.00
• Effect diamete		Deff	m	1.36
• Area of core measured to the outside diameter of the spiral			m2	1.46
• Ratio spiral Rebar required		ρsa		0.00707
Required Area of Spiral Rebar	space		mm	75
	Effective length			1.36
	layer			1
	Area			180.7
	Requaired Dhs			15.2
Actuaral	Effective length	d	m	1.364
	Diameter	Dhr	mm	16
	Area of Rebar	Ah	mm2	201.1
	layer	Nl	nos	1
	Total area of spiral	Ac	m2	201.062
	space	s	mm	75
	Ratio spiral Rebar	ρs	-	0.0078617
• Check condition		$\rho_s > \rho_{sa}$		OK

	Company	Nipponkoei	Project Name	DN Qn Expressway
	Designer	vinh	File Name	D:\...\Pile D1.5m - Pier P9.BOI

1. Geometry and Materials

Design Code : ACI318M-02
Stress Profile : Equivalent Stress Block
Material Data : f_c' = 30 MPa (β_1 = 0.850)
f_y = 400, f_{ys} = 400 MPa
Section Dim. : Φ1500 mm
Effective Len. : KL_u = 9000 mm
Steel Distribut.: 24 - D32 (d_c = 100 mm)
Total Steel Area A_{st} = 19061 mm^2 (ρ_{st} = 0.0108)



2. Member Force and Moment

Unit : kN, kN-m

L.C.	P_u	M_{ux}	M_{uy}	R_{atioV}	V_{ux}	V_{uy}	R_{atioH}	Remark
1	8262.4	-55.5	-36.4	0.304	63.9	13.8	0.014	
2	5917.5	306.6	48.9	0.218	33.3	72.3	0.017	
3	8003.1	92.8	-30.3	0.295	64.8	44.5	0.016	
4	6448.0	99.0	7.3	0.238	71.8	40.4	0.017	
5	6231.8	1027.2	72.2	0.256	39.8	240.0	0.051	
6	5924.5	361.4	189.0	0.218	98.9	81.6	0.027	
7	8624.9	3671.8	1328.8	0.654	419.9	824.8	0.193	

3. Magnified Moment

KL_u/r_x = 9000/375 = 24.00 > 34-12(M_1/M_2) = 22.00
δ_x = MAX[1.00/(1-P_u/0.75/145156), 1.0] = 1.086

KL_u/r_y = 9000/375 = 24.00 > 34-12(M_1/M_2) = 22.00
δ_y = MAX[1.00/(1-P_u/0.75/145156), 1.0] = 1.086

4. Design Force and Moment

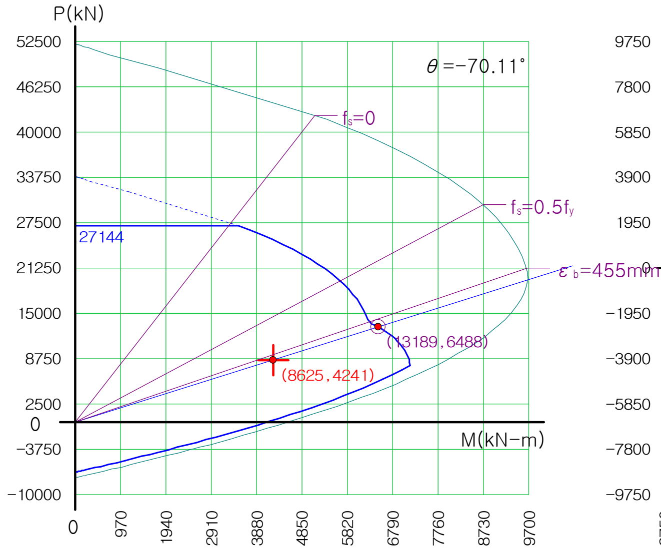
Design Load Combination No : 7
P_u = 8624.9 kN
M_{ux} = 3671.8, M_{uy} = 1328.8 kN-m
$\delta_x M_{ux}$= δ_x*M_{ux} = 3987.8 kN-m
$\delta_y M_{uy}$= δ_y*M_{uy}, = 1443.1 kN-m

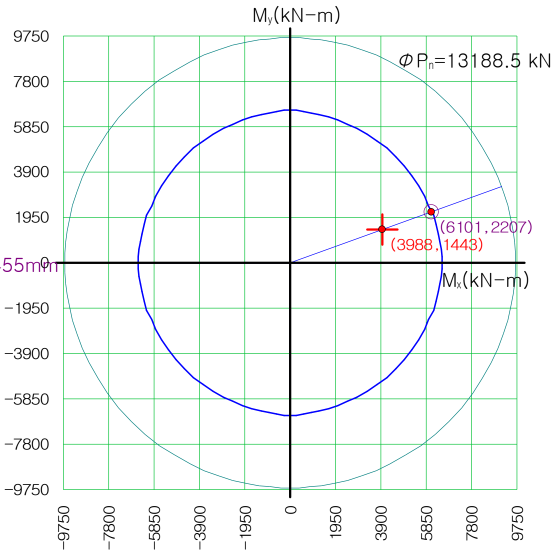
5. Check Axial and Moment Capacity

Rotation Angle and Depth to the Neutral Axis θ = -70.11°, c = 812 mm
Strength Reduction Factor Φ = 0.6698
Maximum Axial Load $\Phi P_{n(max)}$ = 27144.3 kN
Design Axial Load Strength ΦP_n = 13188.5 kN
Design Moment Strength ΦM_{nx} = 6100.7 kN-m
ΦM_{ny} = 2207.3 kN-m

Strength Ratio : Applied/Design = 0.654 < 1.000 .... O.K

	Company	Nipponkoei	Project Name	DN Qn Expressway
	Designer	vinh	File Name	D:\...\Pile D1.5m - Pier P9.BOI





6. Check Shear Capacity

Design Load Combination No : 7

Strength Reduction Factor ϕ= 0.750

Design Force Vu = 925.5 kN (Pu = 8624.9 kN)

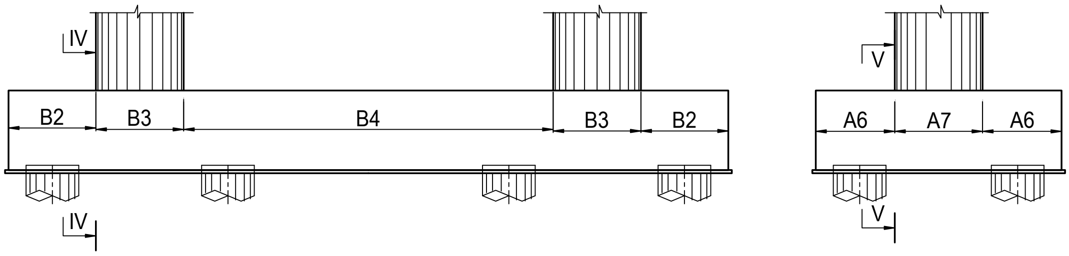
Required Hoop Spacing : D16 @ 508 mm

Provided Hoop Spacing : D16 @ 150 mm (Tie)

$\phi V_c + \phi V_s$ = 3867.6 + 924.5 = 4792.1 kN > V_u = 925.5 kN .... O.K

2.4.7. CHECK PILE CAP :

2.4.7.1. External force to section IV-IV, section V-V:



External force to section IV - IV

STATE	Longitudinal direction		
	Q (kN)	M(kN.m)	N (kN)
Strength I	28639.9	64439.7	255.5
Strength II	22412.3	50427.7	-289.0
Strength III	27721.0	62372.3	-178.0
Service	22441.0	50492.2	-161.5
Extreme IA	21068.7	47404.7	159.0
Extreme IB	22828.3	51363.6	395.5
Extreme II	26434.9	59478.5	1679.5

External force to section V - V

STATE	Transverse direction		
	Q (kN)	M(kN.m)	N (kN)
Strength I	14837.6	24660.1	127.8
Strength II	11295.3	18772.8	66.5
Strength III	14329.2	23815.1	129.5
Service	11241.4	18683.3	143.5
Extreme IA	11887.2	19756.4	79.5
Extreme IB	10462.8	17389.2	197.8
Extreme II	13990.37	23252.0	839.8

2.4.7.2. Ultimate check and shear capacity check :

Item			Section IV-IV (Bottom bar)	Section V-V (Bottom bar)	Unit
• Factored Plexural moment		M_u	64439.72	24660.05	kN.m
• Factored Shear force		V_u	28639.88	14837.58	kN
• Hight of Section		h	2500	2500	mm
• Width of section		b	20500	7000	mm
• Section area		A_c	51250000	17500000	mm^2
• Moment of inertia of concrete section		I_g	2.7E+13	9.1E+12	mm^4
• Tension reinforcement:	Distance from tension reinf. to extreme compression fiber	d_c	166	198	mm
	Reinf. Diameter	∅	32	32	mm
	Space	@	150	150	mm
	Number of bar	n	136	46	bar
	Total area of reinf.	A_s	109378	37263	mm^2
• comp. reinforcement:	Distance from compressive reinf. to extreme Tension fiber		100	138	mm
	Diameter		25	25	mm
	Reinf. Space		150	150	mm
	Number of bar		136	46	bar
	Total area of reinf.	A'_s	66922	22744	mm^2
Check Flexural Moment at Strength state					
• Resistance factor		Φ	0.90	0.90	
• The corresponding effective		d_e	2334	2302	mm
• Stress block factor		β_1	0.8357	0.84	
• Depth of the equivalent stress block = c*β1		a	83.69	83.50	mm
• Distance from extreme compression fiber to the neutral axis		c	100.15	99.92	mm
• The nominal flexural resistance:		M_n	100284	33690	kN.m
• Factored flexural resistance		$M_r = \Phi.M_n$	90256	30321	kN.m
• Check condition		$M_r > M_u$	**O.K**	**O.K**	
Mimimum Reinforcement					
• Ratio of tension steel to gross area		$\rho = A_s/(b.d)$	0.23	0.23	%
• Check		$\rho > 0.03 \cdot f'_c/f'_y$	**O.K**	**O.K**	0.23
• Cracking moment		1.2Mcr	88422.96	25599.99	Kn.m
• Check		Mr> min(1.2Mcr, 1.33Mu)	**O.K**	**O.K**	
Maximum Reinforcement					
• Obligation Condition		c/d_e	0.04	0.04	
• Check		$c/d_e < 0.42$	**O.K**	**O.K**	
Check shear resistance					
• Factored Shear force		V_u	28639.88	14837.58	kN
• Resistance factor		Φ	0.90	0.90	
• The effective shear deepth		d_v	2292	2260	mm
• Effective width		b_v	20500	7000	mm
• Angle of inclination of diagonal compressive stress		θ	43	43	degree
• Angle of inclination of transerverse reinf. To longitudinal axis		α	90	90	degree
• Factor indicating ability of diagonally cracked concrete to transmit tension		β	1.75	1.75	
• Value		$0.1 \cdot f'_c \cdot b_v \cdot d_v$	140967	47465	kN
• Max spacing of transverse reinforcement		s	600	600	mm
• Spacing of stirrup		s	450	450	mm
• Diameter of transverse reinforcement		∅	D 32	D 32	
• Number of transverse reinf. within distance s		n	2	2	bar
• Assume		θ	43.00	39.00	degree
• Strain in tensile reinforcement		ε_x	1.99E-03	2.69E-03	
If ex<0, multiple with reduce factor		Φc	-	-	
• Ratio of shear stress and f'c		V/f'c	0.02	0.03	
• β final			1.75	1.75	
• θ final			43.00	43.00	degree
• Total area of transverse reinf.		A_v	1608	1608	mm^2
• Diameter of stirrup		∅	D 18	D 18	mm
• Number of stirrup within distance s		n	47	16	bar
• Total area of stirrup		A_v	11846.95	4043.23	
• The shear resistance of concrete:		V_c	37383.01	12587.25	kN
• The shear resistance of stirrup		V_s	12325.25	4147.92	kN

• Value	$0.25*f'_c \cdot b_v \cdot d_v$	352418.52	118663.03	kN
• The nominal shear resistance:	V_n	49708.25	16735.17	kN
• The factored shear resistance	V_r	44737.43	15061.65	kN
• Check	$V_r > V_u$	**O.K**	**O.K**	
•Requiring transverse reinforcement	$V_u > 0.5 \cdot \Phi \cdot V_c$	**Need**	**Need**	
•Check minimum transverse reinforcement	$A_v > 0.083 \cdot (f'_c{}^{0.5}) \cdot b_v \cdot s/f_y$	**O.K**	**O.K**	
Check Flexural and shear resistance at Extreme state				
• Factored Plexural moment	M_u	59478.49	23251.99	kN.m
• Factored Shear force	V_u	26434.88	13990.37	kN
• Resistance factor	Φ	0.90	0.90	
• The nominal flexural resistance:	M_n	100284	33690	kN.m
• Factored flexural resistance	$M_r = \Phi.M_n$	90256	30321	kN.m
• Check condition	$M_r > M_u$	**O.K**	**O.K**	
Check crack				
Interior force combination				
• Factored moment	M_u	5.05E+04	1.87E+04	kN.m
• Modulus of rupture of concrete	fr = 0.63*sqrt(f'c)	3.45	3.45	MPa
• Distance from extreme tension fiber to the neutral axis	$y_t = h-c$	2400	2400	mm
• Stress of concrete at tension fiber	$f'r = Ms*y_t/I_g$	5	5	MPa
• Check	f'r >	0.8*fr	0.8*fr	
		check crack	check crack	
• Crack width parameter	Z	= 23000	= 23000	N/mm
• Ratio of reinf. Modulus with concrete modulus	$n = E_s/E_c$	= 7.00	= 7.00	
• The distance from extreme fiber to the neutral axis	c	= 456.56	= 453.14	mm
• Effective moment of inertia	J	3.35E+12	1.11E+12	mm^4
• Arm	de-c	= 1877.44	= 1848.86	mm
• Tension stress in reinforcement	fs =n*Ms*(de-c)/J	= 198.14	= 218.08	MPa
• Area of concrete having the same centroid as the principal	A	= 15074	= 15108	mm2
• Tension stress in reinforcement with service state	$f_{sa} = Z/(d_c*A)^{1/3}$	= 230.40	= 230.22	Mpa
• Check condition	$f_s < f_{sa}$	**O.K**	**O.K**	
• Check condition	$f_s < 0.6*f_y$	**O.K**	**O.K**	

2.5 PIER P10
2.5 TRỤ CẦU P10

PIER P10 - CALCULATION SHEET

CONTENT:

CALCULATION PROCEDURE & STANDARD:

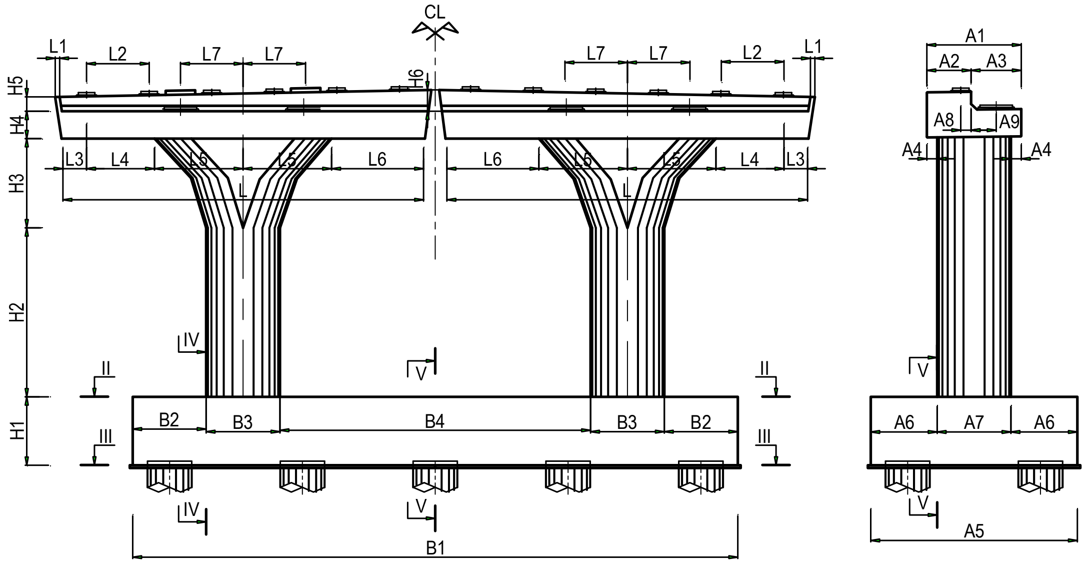
- Bridge Design Standard 22 TCN - 272 - 05

2.5.1. GENERAL DATA:

2.5.1.1. Span length	L_{left} =	38.30	(m)
	L_{right} =	65.00	(m)
2.5.1.2. Design live load			
Design vehicle load		HL93	22TCN 272 - 05
Number of lane		2x3	(lane)
Pedestrian		0.00	KG/m^2
2.5.1.3. Bridge width			
Width carriageway	B_{xe} =	12.00	(m)
Width of median guardrail	B_{pc} =	0.50	(m)
Width barrier	B_{lc} =	0.50	(m)
Bridge width	B =	13.00	(m)
2.5.1.4. Cross sections:			
Pavement thick ness	d_{BTN} =	0.08	(m)
Deck Thickness	d_{BTCT} =	0.20	(m)
Number of Girder	n =	2x6	(m)
Girder distance	L_d =	2.12	(m)
Distance from center of outer girder to extreme of pier cap	L =	0.75	(m)
2.5.1.5. Material property:			
Concrete			
Compressive strength of cylindrical at 28 da	f 'c =	30.00	MPa
Concrete density	g =	24.50	KN/m^3
Elastic modulus	Ec =	29440	MPa
Tension strength of concrete	fr =	3.45	MPa
Steel			
Concrete modulus	Es =	200000	MPa
Yeild strength of steel bar	fy =	400.00	MPa

2.4.1.6. THE PIER GEOMETRIC:

GENERAL DIMENSIONS OF PIER



vertical			Horizontal			Thickness		
Remark	Value	Unit	Remark	Value	Unit	Remark	Value	Unit
a_1 =	3.20	(m)	L =	12.30	(m)	H_1 =	2.50	(m)
a_2 =	1.50	(m)	L_1 =	0.15	(m)	H_2 =	8.800	(m)
a_3 =	1.70	(m)	L_2 =	2.11	(m)	H_3 =	3.200	(m)
a_4 =	0.35	(m)	L_3 =	0.85	(m)	H_4 =	1.000	(m)
a_5 =	7.00	(m)	L_4 =	3.15	(m)	H_5 =	0.673	(m)
a_6 =	2.25	(m)	L_5 =	3.00	(m)	H_6 =	0.927	(m)
a_7 =	2.50	(m)	L_6 =	3.12	(m)	H_7 =	1.80	(m)
a_8 =	0.45	(m)	L_7 =	2.09	(m)			
a_9 =	0.75	(m)	B_1 =	20.50	(m)			
			B_2 =	2.495	(m)			
			B_3 =	2.50	(m)			
			B_4 =	10.51	(m)			

The design elevation:

Elevation of surface of deck	EL_{mc} =	17.719	(m)	Bearing dimention of super-T girder		
Elevation of top of bearing	EL_{xm} =	16.18	(m)	a =	0.35	(m)
Hight water level (H1%)	EL_{MNTK} =	9.20	(m)	b =	0.7	(m)
Daily water level (H5%)	EL_{MNTB} =	4.07	(m)	δ =	0.035	(m)
Existing height	EL_{TN} =	3.69	(m)	Bearing dimention of box girder		
Daily water level (H5%)	EL_{TT} =	4.07	(m)	a =	1.2	(m)
Section II - II (top footing)	$EL_{MC\ II\text{-}II}$ =	1.61	(m)	b =	1.2	(m)
Section III - III (bottom footing)	$EL_{MCIII\text{-}III}$ =	-0.886	(m)	δ =	0.15	(m)

2.5.2. COMBINATION LOAD:

2.5.2.1. Dead load of super structure: DC

Load type	Vertical load **Nz**		Momen due to Left Span		Momen due to Right Span	
	Left span	Right span	e_{xt}	**My**	e_{xp}	**My**
	(KN)	(KN)	(m)	(KNm)	(m)	(KNm)
1 - Wearing surface - DW	907.25	802.09	0.450	408.26	-0.750	-601.57
2 - Median barrier	422.26	476.59	0.450	190.02	-0.750	-357.44
3 - Side Barrier	516.09	389.93	0.450	232.24	-0.750	-292.45
4 - Railing	0.00	0.00	0.450	0.00	-0.750	0.00
5 - Deck and permanent form (if any)	2574.36	0.00	0.450	1158.46	-0.750	0.00
6 - Cross beam + jointions	268.90	0.00	0.450	121.00	-0.750	0.00
7 - Super - T girder or Box girder	4280.64	8799.33	0.450	1926.29	-0.750	-6599.50
Total	**8969.50**	**10467.94**		**4036.28**		**-7850.96**

- Reaction of right span (box girder) received from Rm soft

2.5.2.2. Dead load of pier: DC

Load type	Mass	Unit Weight	Nz Load
	(m3)	(KN/m3)	(KN)
1 - Bearing pad	0.97	24.50	23.69
2 - Pier cap	108.73	24.50	2663.93
3 - Body pier	145.80	24.50	3572.100
4 - Pile cap	358.75	24.50	8789.38
Total			**15049.10**

2.5.2.3. Live Load: LL + IM

2.5.2.3.1 Design live load: HL-93

		Truck with 3 axle		Tandem		Lane load		
Weights of axles:	P1 =	35.00	(KN)	110.00	(KN)	W_L =	9.30	(KN/m)
	P2 =	145.00	(KN)	110.00	(KN)			
	P3 =	145.00	(KN)			Pedestrian load		
Spacings of axle	V1 =	4.30	(m)	1.20	(m)	W_N =	0.00	(KN/m^2)
	V2 =	4.30	(m)			Ble =	0.00	(m)

Eccentric (longitudinal)	- Left span	e^{Left}_x =	0.450	(m)
	- Right span	e^{Right}_x =	-0.750	(m)
Number of lane		K =	2x3	(lane)
Dynamic load factor		IM =	1.25	

2.5.2.3.2 Live load is mentioned in some case below:

Live load applied on each spans follow below formular:

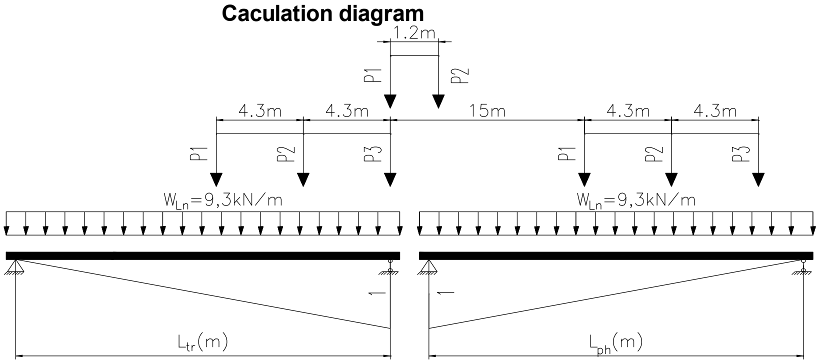
$$N_z^{Truck} = [P_3+P_2(L - V_2) / L+P_1 (L - V_1 - V_2) / L]*m* K * IM$$

$$N_z^{lane\ load} = 0.5*L*W_L*m*K$$

$$N_z^{Tandem} = [P2+P1 (L-V1) / L] *m*K*IM$$

$$N_z^{Pedestrian} = 0.5*L*W_N*Ble*2$$

Caculation diagram



1. Live load effect on two spans - All vehicle lane. (Max Nz) - (Strength limit state IA)

Number of lane in this case		k =	2x3	(lane)
Lane factor		m =	0.85	
Eccentric (Horizontal) - for vehicle		$e^{vehicle}$ =	0.00	(m)
	for pedestrian	$e^{pedestrian}_y$ =	0.00	(m)

Live load type	Left span	37.60	(m)	Right span	64.35	(m)
	Nz	Mx =Nz*e_y	My =Nz*e_x	Nz	Mx =Nz*e_y	My =Nz*e_x
	(KN)	(KNm)	(KNm)	(KN)	(KNm)	(KNm)
1 - Truck	1915.13	0.00	861.81	0.00	0.00	0.00
2 - Tandem	701.25	0.00	315.56	0.00	0.00	0.00
3 - Lane Load	891.68	0.00	401.26	0.00	0.00	0.00
HL - 93	**2806.81**	**0.00**	**1263.07**	**4784.38**	**0.00**	**-3588.29**

Live load value:

Nz =	**7591.20**	**(KN)**
Mx =	**0.00**	**(KNm)**
My =	**-2325.22**	**(KNm)**

2. Live load effect on two spans - Vehicle on 1/2 Bridge width (Max Mx) - (Strength IB)

Number of lane in this case K = 3.00 (lane)

Lane factor m = 0.85

Eccentric (Horizontal) - for vehicle $e^{Vehicle}_y$ = 6.505 (m)

Live Load	Left span	37.60	(m)	Right span	64.35	(m)
	Nz	Mx =Nz*e_y	My =Nz*e_x	Nz	Mx =Nz*e_y	My =Nz*e_x
	(KN)	(KNm)	(KNm)	(KN)	(KNm)	(KNm)
1 - Truck	957.56	6228.95	430.90	0.00	0.00	0.00
2 - Tandem	350.63	2280.82	157.78	0.00	0.00	0.00
3 - Lane Load	445.84	2900.20	200.63	0.00	0.00	0.00
HL - 93	**1403.41**	**9129.16**	**631.53**	**2392.19**	**15561.21**	**-1794.14**

Live load value:

Nz =	**3795.60**	**(KN)**
Mx =	**24690.36**	**(KNm)**
My =	**-1162.61**	**(KNm)**

3. Live load efect on one Span (Lmax) - All Lane (Max My) - (Strength IC)

Number of lane in this case k = 3.00 (lane)

Lane factor m = 0.85

Eccentric (Horizontal) - for vehicle $e^{vehicle}_y$ = 0.00 (m)

Eccentric (longitudinal)- Max Span e^{Lmax}_x = -0.750 (m)

Live Load	Lmax	64.35	(m)
	Nz	Mx =Nz*e_y	My =Nz*e_x
	(KN)	(KNm)	(KNm)
1 - Truck	0.00	0.00	0.00
2 - Tandem	0.00	0.00	0.00
3 - Lane Load	0.00	0.00	0.00
HL - 93	**4784.38**	**0.00**	**-3588.29**

Live load value:

Nz =	**4784.38**	**(KN)**
My =	**-3588.29**	**(KNm)**

2.5.2.4. Breaking Force: BR

Breadking force is 25% of total weight of truck axles or of tandem on all lanes

These force shall be assumed to act horizoltally at a distance of 1800mm above roadway surface.

Maximum friction force can act to pier FR = f*(ΣDC+ΣLL) = 1621.72 (Kn)

With friction factor of FPTE plate is: f = 0.06

If Breaking force is not much than Friction force, used breaking force to calculate:

So at pier used pot bearing assum 50% of breaking force act to this pier

Section ID	Lane Number	Truck	Tandem	Lane	**Truck**	**Tandem**	Z	**My**
	(Lane)	(KN)	(KN)	Factor	**(KN)**	**(KN)**	(m)	**(KNm)**
II	2x3	325.00	220.00	0.85	**-207.19**	**-140.25**	17.91	**-3709.69**
III	2x3	325.00	220.00	0.85	**-207.19**	**-140.25**	20.41	**-4227.66**

2.5.2.5. Stream pressure : WA

$$P_{(D,L)} = 5.14 * 10^{-4} * C_{D(L)} * V^2 \quad \text{(Mpa)}$$

In Which: C_D : Drag coefficient in longitudinal direction.

C_D = 0.70 With circular pier cap

C_L : Drag coefficient in transverse direction

C_L = 0.50

V : Designed velocity of water

V = 2.54 (m/s)

=> Horizontal Stream pressure P_D = 0.00231 (Mpa)

P_D = 2.312 (KN/m²)

=> Longitudinal Stream pressure P_L = 0.000145 (Mpa)

P_L = -0.145 (KN/m²)

Water pressure acting on the section follow table below:

Section ID	Horizontal			Longitudinal			Moment		
	Pressure	Area	**Force**	Pressure	Area	**Force**	Z	**Mx**	**My**
	(KN/m^2)	(m2)	**(KN)**	(KN/m^2)	(m2)	**(KN)**	(m)	**(KNm)**	**(KNm)**
II	2.312	29.79	**68.89**	-0.145	29.79	**-4.30**	3.79	**261.29**	**-16.33**
III	2.312	47.29	**109.36**	-0.145	81.04	**-11.71**	5.04	**551.48**	**-59.06**

2.5.2.6. Win Load:

2.5.2.6.1 Win load on Structure: WS

(a) *Designed win speed:*

Designed win velocity follow formula: **$V = V_B . S$**

Inwhich: V_B : basic 3 second gust wind velocity.

V_B = 53 (m/s) (Zone II)

S : correction factor for upwind terrain and deck height.

S = 1.20

Designed wind velocity. **V = 63.6 (m/s)**

(b) *Transverse wind load:* **$P_D = 0.0006 . V^2 . A_t . C_d$**

Inwhich: V : Designed wind velocity

C_d : drag coefficient specified, it's depened rario b/d.

b: overall width of bridge between outer faces of parapets.

b = 13.00 (m)

d: depth of superstructure, include solid parapets if applicable.

d = 3.02 (m)

Ratio (b / d) = 4.30 **C_d = 1.40**

A_t : Area of the structure for calculation of transverse wind load.

$A_t = A_t^{Left\ Span} = (L^{Left\ Span} \times d_T)$

A_t = 57.83 (m^2)

Transverse wind load: **P_D^{trans} = 436.74 (KN)**

(c) *Longitudinal wind load:*

For superstructure with solid elevation, a longitudinal wind load equal to 0.25 times the transverse wind load calculated

Longitudinal wind load: **P_D^{Long} = 0.00 (KN)**

(d) *The wind load applied on the sections:*

Section	**Transverse wind load**	**Longitudinal wind load**	Height	**Moment**	
	P_D^{Trans}	**P_D^{Long}**	Z	**Mx**	**My**
	(KN)	**(KN)**	(m)	**(KNm)**	**(KNm)**
II	**436.74**	**0.00**	16.07	**7019.64**	**0.00**
III	**436.74**	**0.00**	18.57	**8111.48**	**0.00**

2.5.2.6.2 Wind load on vehicles: WL

(a) *Line wind load*

Transeverse direction: p_y = 1.50 (KN/m) A 3.8.1.3 22TCN 272-05

Longitudinal direction: p_x = 0.75 (KN/m)

Wind load assumed 1.8m above the road di = 1.80 (m)

(b) *Wind load*

Transverse direction _ Left span: P_D = 56.40 (KN)

_ Right span: P_D = 43.82 (KN)

Total: P^{Trans}_D = 100.22 (KN)

Longitudinal direction _ Left span: P_D = 0.00 (KN)

_ Right span: P_D = 0.00 (KN)

Total: P^{Long}_D = 0.00 (KN)

(c) *Wind load on vehicles:*

Section	**Transverse wind load**	**Longitudinal wind load**	Height	**Moment**	
	P_D^{Trans}	**P_D^{Long}**	Z	**Mx**	**My**
	(KN)	**(KN)**	(m)	**(KNm)**	**(KNm)**
II	**100.22**	**0.00**	17.91	**3588.81**	**0.00**
III	**100.22**	**0.00**	20.41	**4089.90**	**0.00**

2.5.7. Earthquake effects: EQ

Earthquake class: Class 7.00

Acceleration coefficient A = 0.034

Site coefficient S = 1.20

R : Response modification factors R = 1.00

Steel Ec = 29440.09 Mpa

Selftweight of super-structure applied on piers

Wt = 234.19 KN/m

(a) *Determined stiffness:* **K** x(y) **= 3.E.I** x(y) **/ H^3**

(b) *Displacement accodance with unit force* **Vs** x(y) **= Po . L / K** x(y)

(c) *Determined factor a, b, g :*

$$\alpha = \int_{T(n+1)}^{T(n-1)} v_s(x)\,dx \qquad \beta = \int_{T(n+1)}^{T(n-1)} W_{(x)} V_s(x)\,dx \qquad \gamma = \int_{T(n+1)}^{T(n-1)} W_{(x)} V_s(x)^2\,dx$$

(d) *Determination of period of vibration*

$$T = 2\pi\sqrt{\frac{\gamma}{P_0.g.\alpha}}$$

The summation of result :

Section	H	Ix	Iy	Kx	Ky	Vsx	Vsy
	(m)	(m^4)	(m^4)	(KN/m)	(KN/m)	(m)	(m)
II	14.56	3.83	3.83	109664.92	109664.92	0.000	0.0005
III	17.06	3.83	3.83	109664.92	109664.92	0.000	0.0005

Section	H	αx	αy	βx	βy	γx	γy	Tx	Ty
	(m)	(m^2)	(m^2)	KNm	KNm	KNm2	KNm2	(s)	(s)
II	14.56	0.024	0.024	5.697	5.697	0.003	0.003	0.67	0.67
III	17.06	0.024	0.024	5.697	5.697	0.003	0.003	0.67	0.67

Inwhich:

Tx, Ty: The period of vibration follow direct X and direct Y.

H: The distance from desk slab to consided sections

$\int_{T(n+1)}^{T(n-1)} xd(x) = L$ The length of superstructure applied to pier (m)

Ec: Reinforcement concrete elastic modulus. (Mpa)

Ix, Iy: Moment of inertia follow X axial, and follow Y axial. (m^4)

(e) *Determination of elastic seismic response coefficient:* $C_{sm} = \frac{1{,}2.A.S}{T_m^{2/3}} \leq 2{,}5A$

(f) *Equivalent uniform static seismic coefficient:* $P_e(x) = \frac{\beta C_{sm}}{\gamma} W(x).V(x)$

(g) *Designed force applied on pier due to earthquake effect:* **H = Pe$_{(x)}$ L / R**

Section	$C_x$sm	$C_y$sm	$C_x$sm	$C_y$sm	Pe(x)	Pe(y)	Hx	Hy	Z
	Follow theory		Compared with 2.5A		(KN/m)	(KN/m)	(KN)	(KN)	(m)
II	0.064	0.064	0.064	0.064	**15.03**	**15.03**	**0.00**	**956.68**	**14.56**
III	0.064	0.064	0.064	0.064	**15.03**	**15.03**	**0.00**	**956.68**	**17.06**

Section	Hx	Hy	Z	Mx	My
	(KN)	(KN)	(m)	(KNm)	(KNm)
II	-	**956.68**	**14.56**	**13932.17**	-
III	-	**956.68**	**17.06**	**16323.87**	-

(h) *Earthquake effect due to substructure*

Section II - II								
The Component	**Nz**	**$C_x$sm**	**$C_y$sm**	**Hx**	**Hy**	**Z**	**Mx**	**My**
	(KN)			**(KN)**	**(KN)**	(m)	**(KNm)**	**(KNm)**
1 - Pier cap	2663.93	0.064	0.064	-170.92	170.92	12.84	2194.03	-2194.03
2 - Pier shaft	3572.10	0.064	0.064	-229.19	229.19	6.00	1375.14	-1375.14
Total	**6236.03**			**-400.11**	**400.11**		**3569.16**	**-3569.16**

Section III - III								
The Component	**Nz**	**$C_x$sm**	**$C_y$sm**	**Fx**	**Fy**	**Z**	**Mx**	**My**
	(KN)			**(KN)**	**(KN)**	(m)	**(KNm)**	**(KNm)**
1 - Pier cap	2663.93	0.06	0.06	-170.92	170.92	15.34	2621.33	-2621.33
2 - Pier shaft	3572.10	0.06	0.06	-229.19	229.19	8.50	1948.11	-1948.11
4 - Pile cap	8789.38	0.06	0.06	-563.94	563.94	1.25	704.92	-704.92
Total	**15025.41**			**-964.05**	**964.05**		**5274.36**	**-5274.36**

(i) *Designed load due to earthquake effect applied on the sections*

Section	Hx	Hy	Mx	My
	(KN)	(KN)	(KN.m)	(KN.m)
II - II	**-400.11**	**1356.79**	**17501.33**	**-3569.16**
III - III	**-964.05**	**1920.73**	**21598.23**	**-5274.36**

2.5.8. Vessel Collision: CV (3.14 - 22TCN272-05)

Class of navigable waterway:	Class	IV	
Mean annual stream velocity:	V_{bq} =	1.15	(m/s)
Design vessel tonnage:	Self-propelled vessel	200.00	(DWT)
	Towed barge	400.00	(DWT)
Dimensions of design vessel:	Maximum length	34.00	(m)
	Maximum breadth	6.60	(m)
	Laden Draught	1.70	(m)
Design collision velocity	Self-propelled vessel	3.65	(m/s)
	Towed barge	2.75	(m/s)

(a) Collision energy of Towed barge:

$KE = 500\ C_H\ M\ V^2$ = 1588125 (J)

M Towed barge displacement tonnage M = 400.00 (Mg)

C_H Hydrodynamic mass coefficient C_H = 1.05

Barge bow damage length

$$a_B = 3100.(\sqrt{1+1{,}3x10^{-7}KE}-1) = \quad 305.00 \text{ (mm)}$$

Barge collision force on pier **P_B = 6488.00 (KN)**

(b) Vessel collision force pier:

It's detemined follow formula

V Design collision velocity (m/s)

DWT Deadweight tonnage of vessel (Mg)

$$P_s = 1{,}2x10^5 V\sqrt{DWT}$$

Vessel collision force pier: **P_S = 6194.26 (KN)**

On basic of comparision of vessel collision force between seft-roelled and Towed barge, chose follow table below

The Force follow Perpendicular direction to centerline : CV = 100% max(P_B,P_S)

The force follow longitudinal direction of bridge: CV = 50% max(P_B,P_S)

Vessel collision force (CV) applied to pier at daily water level:

Section	H_x	H_y	Z	Mx	My
	(KN)	(KN)	(m)	(KNm)	(KNm)
II	3244.00	6488.00	2.46	15934.54	-7967.27
III	3244.00	6488.00	4.96	32154.55	-16077.28

2.5.3. COMBINATION LOAD - SECTION II - II

	LOAD TYPE & NOTATIONS		Factor γ	Nz (KN)	Hx (KN)	Hy (KN)	Mx (KNm)	My (KNm)
Ia	**STRENGTH LIMIT STATE I_A**							
1	DC	Dead load of left span	1.25	10077.81	-	-	-	4535.02
2	DC	Dead load of right span	1.25	12082.32	-	-	-	-9061.74
3	DW	Dead load of wearing surface	1.50	2564.01	-	-	-	-289.96
4	DC	Dead load of pier	1.25	7824.65	-	-	-	-
5	LL	Vehicle Live load - (maxNz)	1.75	13284.59	-	-	0.00	-4069.14
6	BR	Braking force (Start)	1.75	-	-245.44	-	-	-6491.96
7	WA	Water load and stream pressure	1.00	-	-4.30	68.89	261.29	-16.33
8	FR	Friction	1.00	-	0.00	-	-	0.00
	TOTAL			**45833.39**	**-249.74**	**68.89**	**261.29**	**-15394.11**
Ib	**STRENGTH LIMIT STATE I_B**							
1	DC	Dead load of left span	0.90	7256.03	-	-	-	3265.21
2	DC	Dead load of right span	0.90	8699.27	-	-	-	-6524.45
3	DW	Dead load of wearing surface	0.65	1111.07	-	-	-	-125.65
4	DC	Dead load of pier	0.90	5633.75	-	-	-	-
5	LL	Vehicle Live load - (maxMx)	1.75	6642.30	-	-	43208.14	-2034.57
6	BR	Braking force (Start)	1.75	-	-245.44	-	-	-6491.96
7	WA	Water load and stream pressure	1.00	-	-4.30	68.89	261.29	-16.33
8	FR	Friction	1.00	-	0.00	-	-	0.00
	TOTAL			**29342.41**	**-249.74**	**68.89**	**43469.43**	**-11927.75**
Ic	**STRENGTH LIMIT STATE I_C**							
1	DC	Dead load of left span	0.90	7256.03	-	-	-	3265.21
2	DC	Dead load of right span	0.90	8699.27	-	-	-	-6524.45
3	DW	Dead load of wearing surface	0.65	1111.07	-	-	-	-125.65
4	DC	Dead load of pier	0.90	5633.75	-	-	-	-
5	LL	Vehicle Live load - (maxMy)	1.75	8372.67	-	-	0.00	-6279.50
6	BR	Braking force (Start)	1.75	-	-245.44	-	-	-6491.96
7	WA	Water load and stream pressure	1.00	-	-4.30	68.89	261.29	-16.33
8	FR	Friction	1.00	-	0.00	-	-	0.00
	TOTAL			**31072.79**	**-249.74**	**68.89**	**261.29**	**-16172.68**
II	**STRENGTH LIMIT STATE II**							
1	DC	Dead load of left span	1.25	10077.81	-	-	-	4535.02
2	DC	Dead load of right span	1.25	12082.32	-	-	-	-9061.74
3	DW	Dead load of wearing surface	1.50	2564.01	-	-	-	-289.96
4	DC	Dead load of pier	1.25	7824.65	-	-	-	0.00
5	WA	Water load and stream pressure	1.00	-	-4.30	68.89	261.29	-16.33
6	WS	Wind load on structure	1.40	-	0.00	611.43	9827.50	0.00
	TOTAL			**32548.80**	**-4.30**	**680.32**	**10088.79**	**-4833.01**
IIIa	**STRENGTH LIMIT STATE III_A**							
1	DC	Dead load of left span	1.25	10077.81	-	-	-	4535.02
2	DC	Dead load of right span	1.25	12082.32	-	-	-	-9061.74
3	DW	Dead load of wearing surface	1.50	2564.01	-	-	-	-289.96
4	DC	Dead load of pier	1.25	7824.65	-	-	-	0.00
5	LL	Live load - (maxNz)	1.35	10248.11	-	-	0.00	-3139.05
6	BR	Braking force (Start)	1.35	-	-189.34	-	-	-5008.08
7	WA	Water load and stream pressure	1.00	-	-4.30	68.89	261.29	-16.33
8	WS	Wind load on structure	0.40	-	0.00	174.69	2807.86	0.00
9	WL	Wind load on vehicle	1.00	-	0.00	100.22	3588.81	0.00
10	TU+CR+SH	Temperature + Creep + Shinkage	0.50	-	-	-	-	-
	TOTAL			**42796.91**	**-193.64**	**343.80**	**6657.96**	**-12980.14**
IIIb	**STRENGTH III_B**							
1	DC	Dead load of left span	0.90	7256.03	-	-	-	3265.21
2	DC	Dead load of right span	0.90	8699.27	-	-	-	-6524.45
3	DW	Dead load of wearing surface	0.65	1111.07	-	-	-	-125.65
4	DC	Dead load of pier	0.90	5633.75	-	-	-	0.00
5	LL	Live load - (max Mx)	1.35	5124.06	-	-	33331.99	-1569.53
6	BR	Braking force (Start)	1.35	-	-189.34	-	-	-5008.08
7	WA	Water load and stream pressure	1.00	-	-4.30	68.89	261.29	-16.33
8	WS	Wind load on structure	0.40	-	0.00	174.69	2807.86	0.00
9	WL	Wind load on vehicle	1.00	-	0.00	100.22	3588.81	0.00
	TOTAL			**27824.17**	**-193.64**	**343.80**	**39989.95**	**-9978.83**

IIIc	STRENGTH III_C							
1	DC	Dead load of left span	0.90	7256.03	-	-	-	3265.21
2	DC	Dead load of right span	0.90	8699.27	-	-	-	-6524.45
3	DW	Dead load of wearing surface	0.65	1111.07	-	-	-	-125.65
4	DC	Dead load of pier	0.90	5633.75	-	-	-	0.00
5	LL	Live load - (max My)	1.35	6458.92	-	-	0.00	-4844.19
6	BR	Braking force (Start)	1.35	-	-189.34	-	-	-5008.08
7	WA	Water load and stream pressure	1.00	-	-4.30	68.89	261.29	-16.33
8	WS	Wind load on structure	0.40	-	0.00	174.69	2807.86	0.00
9	WL	Wind load on vehicle	1.00	-	0.00	100.22	3588.81	0.00
10	TU+CR+SH	Temperature + Creep + Shinkage	0.50	-	-	-	-	-
	TOTAL			**29159.03**	**-193.64**	**343.80**	**6657.96**	**-13253.49**

IV	EXTREME EVENT I (100% EQ trans)							
1	DC	Dead load of left span	1.25	10077.81	-	-	-	4535.02
2	DC	Dead load of right span	1.25	12082.32	-	-	-	-9061.74
3	DW	Dead load of wearing surface	1.50	2564.01	-	-	-	-289.96
4	DC	Dead load of pier	1.25	7824.65	-	-	-	0.00
5	LL	Live load - (maxMx)	0.50	1897.80	-	-	12345.18	-1162.61
6	BR	Braking force (Start)	0.50	-	-103.59	-	-	-581.31
7	WA	Water load and stream pressure	1.00	-	-4.30	68.89	261.29	-16.33
8	EQ	Earth Quake Load	1.00	-	-400.11	1356.79	17501.33	-3569.16
	TOTAL			**34446.60**	**-508.01**	**1425.68**	**30107.81**	**-10146.09**

V	EXTREME EVENT II							
1	DC	Dead load of left span	1.25	10077.81	-	-	-	4535.02
2	DC	Dead load of right span	1.25	12082.32	-	-	-	-9061.74
3	DW	Dead load of wearing surface	1.50	2564.01	-	-	-	-289.96
4	DC	Dead load of pier	1.25	7824.65	-	-	-	0.00
5	LL	Live load - (maxMx)	0.50	1897.80	-	-	12345.18	-1162.61
6	BR	Braking force (Start)	0.50	-	-103.59	-	-	-581.31
7	WA	Water load and stream pressure	1.00	-	-4.30	68.89	261.29	-16.33
8	CV	Vessels Impact	1.00	-	3244.00	6488.00	15934.54	-7967.27
	TOTAL			**34446.60**	**3136.10**	**6556.89**	**28541.02**	**-14544.19**

VII	SERVICE STATE I_A							
1	DC	Dead load of left span	1.00	8062.25	-	-	-	3628.01
2	DC	Dead load of right span	1.00	9665.85	-	-	-	-7249.39
3	DW	Dead load of wearing surface	1.00	1709.34	-	-	-	-193.31
4	DC	Dead load of pier	1.00	6259.72	-	-	-	0.00
5	LL	Live load - (max Nz)	1.00	7591.20	-	-	0.00	-2325.22
6	BR	Braking force (Start)	1.00	-	-140.25	-	-	-3709.69
7	WA	Water load and stream pressure	1.00	-	-4.30	68.89	261.29	-16.33
8	WS	Wind load on structure	0.30	-	0.00	131.02	2105.89	0.00
9	WL	Wind load on vehicle	1.00	-	0.00	100.22	3588.81	0.00
	TOTAL			**33288.36**	**-144.55**	**300.13**	**5955.99**	**-9865.93**

VIII	SERVICE STATE I_B							
1	DC	Dead load of left span	1.00	8062.25	-	-	-	3628.01
2	DC	Dead load of right span	1.00	9665.85	-	-	-	-7249.39
3	DW	Dead load of wearing surface	1.00	1709.34	-	-	-	-193.31
4	DC	Dead load of pier	1.00	6259.72	-	-	-	0.00
5	LL	Live load - (max Mx)	1.00	3795.60	-	-	24690.36	-1162.61
6	BR	Braking force (Start)	1.00	-	-140.25	-	-	-3709.69
7	WA	Water load and stream pressure	1.00	-	-4.30	68.89	261.29	-16.33
8	WS	Wind load on structure	0.30	-	0.00	131.02	2105.89	0.00
9	WL	Wind load on vehicle	1.00	-	0.00	100.22	3588.81	0.00
	TOTAL			**29492.77**	**-144.55**	**300.13**	**30646.36**	**-8703.31**

IX	SERVICE STATE I_C							
1	DC	Dead load of left span	1.00	8062.25	-	-	-	3628.01
2	DC	Dead load of right span	1.00	9665.85	-	-	-	-7249.39
3	DW	Dead load of wearing surface	1.00	1709.34	-	-	-	-193.31
4	DC	Dead load of pier	1.00	6259.72	-	-	-	0.00
5	LL	Live load - (max My)	1.00	4784.38	-	-	0.00	-3588.29
6	BR	Braking force (Start)	1.00	-	-140.25	-	-	-3709.69
7	WA	Water load and stream pressure	1.00	-	-4.30	68.89	261.29	-16.33
8	WS	Wind load on structure	0.30	-	0.00	131.02	2105.89	0.00
9	WL	Wind load on vehicle	1.00	-	0.00	100.22	3588.81	0.00
	TOTAL			**30481.55**	**-144.55**	**300.13**	**5955.99**	**-11128.99**

COMBINATION EXTERNAL FORCE ON SECTION II - II

State	Load factors specified			Nz	Hx	Hy	Mx	My
	η_D	η_R	η_I	(KN)	(KN)	(KN)	(KNm)	(KNm)
Strength I_A	1.00	1.00	1.00	45833.39	-249.74	68.89	261.29	-15394.11
Strength I_B	1.00	1.00	1.00	29342.41	-249.74	68.89	43469.43	-11927.75
Strength I_C	1.00	1.00	1.00	31072.79	-249.74	68.89	261.29	-16172.68
Strength II	1.00	1.00	1.00	32548.80	-4.30	680.32	10088.79	-4833.01
Strength III_A	1.00	1.00	1.00	42796.91	-193.64	343.80	6657.96	-12980.14
Strength III_B	1.00	1.00	1.00	27824.17	-193.64	343.80	39989.95	-9978.83
Strength III_C	1.00	1.00	1.00	29159.03	-193.64	343.80	6657.96	-13253.49
Extreme Event I	1.00	1.00	1.00	34446.60	-508.01	1425.68	30107.81	-10146.09
Extreme Event II	1.00	1.00	1.00	34446.60	3136.10	6556.89	28541.02	-14544.19
Service state I_A	1.00	1.00	1.00	33288.36	-144.55	300.13	5955.99	-9865.93
Service state I_B	1.00	1.00	1.00	29492.77	-144.55	300.13	30646.36	-8703.31
Service state I_C	1.00	1.00	1.00	30481.55	-144.55	300.13	5955.99	-11128.99

2.5.4. COMBINATION LOAD - SECTION III - III

STT	LOAD TYPE & NOTATIONS		Factor γ	Nz (KN)	Hx (KN)	Hy (KN)	Mx (KNm)	My (KNm)
Ia	**STRENGTH STATE I_A**							
1	DC	Dead load of left span	1.25	10077.81	-	-	-	4535.02
2	DC	Dead load of right span	1.25	12082.32	-	-	-	-9061.74
3	DW	Dead load of wearing surface	1.50	2564.01	-	-	-	-289.96
4	DC	Dead load of pier	1.25	18811.37	-	-	-	-
5	LL	Live load - (maxNz)	1.75	13284.59	-	-	0.00	-4069.14
6	BR	Braking force (Start)	1.75	-	-245.44	-	-	-7398.41
7	WA	Water load and stream pressure	1.00	-	-11.71	109.36	551.48	-59.06
8	TU+CR+SH	Temperature + Creep + Shinkage	0.50	-	-	-	-	-
	TOTAL			**56820.11**	**-257.15**	**109.36**	**551.48**	**-16343.28**

STT	LOAD TYPE & NOTATIONS		Factor γ	Nz (KN)	Hx (KN)	Hy (KN)	Mx (KNm)	My (KNm)
Ib	**STRENGTH STATE I_B**							
1	DC	Dead load of left span	1.25	10077.81	-	-	-	4535.02
2	DC	Dead load of right span	1.25	12082.32	-	-	-	-9061.74
3	DW	Dead load of wearing surface	1.50	2564.01	-	-	-	-289.96
4	DC	Dead load of pier	1.25	18811.37	-	-	-	-
5	LL	Live load - (max Mx)	1.75	6642.30	-	-	43208.14	-2034.57
6	BR	Braking force (Start)	1.75	-	-245.44	-	-	-7398.41
7	WA	Water load and stream pressure	1.00	-	-11.71	109.36	551.48	-59.06
8	TU+CR+SH	Temperature + Creep + Shinkage	0.50	-	-	-	-	-
	TOTAL			**50177.81**	**-257.15**	**109.36**	**43759.62**	**-14308.71**

STT	LOAD TYPE & NOTATIONS		Factor γ	Nz (KN)	Hx (KN)	Hy (KN)	Mx (KNm)	My (KNm)
Ic	**STRENGTH STATE I_C**							
1	DC	Dead load of left span	1.25	10077.81	-	-	-	4535.02
2	DC	Dead load of right span	1.25	12082.32	-	-	-	-9061.74
3	DW	Dead load of wearing surface	1.50	2564.01	-	-	-	-289.96
4	DC	Dead load of pier	1.25	18811.37	-	-	-	-
5	LL	Vehicle Live load - (maxMy)	1.75	8372.67	-	-	0.00	-6279.50
6	BR	Braking force (Start)	1.75	-	-245.44	-	-	-7398.41
7	WA	Water load and stream pressure	1.00	-	-11.71	109.36	551.48	-59.06
	TOTAL			**51908.19**	**-257.15**	**109.36**	**551.48**	**-18553.65**

STT	LOAD TYPE & NOTATIONS		Factor γ	Nz (KN)	Hx (KN)	Hy (KN)	Mx (KNm)	My (KNm)
II	**STRENGTH STATE II**							
1	DC	Dead load of left span	1.25	10077.81	-	-	-	4535.02
2	DC	Dead load of right span	1.25	12082.32	-	-	-	-9061.74
3	DW	Dead load of wearing surface	1.50	2564.01	-	-	-	-289.96
4	DC	Dead load of pier	1.25	18811.37	-	-	-	0.00
5	WA	Water load and stream pressure	1.00	-	-11.71	109.36	551.48	-59.06
6	WS	Wind load on structure	1.40	-	0.00	611.43	11356.07	0.00
	TOTAL			**43535.52**	**-11.71**	**720.79**	**11907.55**	**-4875.74**

STT	LOAD TYPE & NOTATIONS		Factor γ	Nz (KN)	Hx (KN)	Hy (KN)	Mx (KNm)	My (KNm)
IIIa	**STRENGTH STATE III-A**							
1	DC	Dead load of left span	1.25	10077.81	-	-	-	4535.02
2	DC	Dead load of right span	1.25	12082.32	-	-	-	-9061.74
3	DW	Dead load of wearing surface	1.50	2564.01	-	-	-	-289.96
4	DC	Dead load of pier	1.25	18811.37	-	-	-	0.00
5	LL	Live load - (maxNz)	1.35	10248.11	-	-	0.00	-3139.05
7	BR	Braking force (Start)	1.35	-	-189.34	-	-	-5707.34
8	WA	Water load and stream pressure	1.00	-	-11.71	109.36	551.48	-59.06
9	WS	Wind load on structure	0.40	-	0.00	174.69	3244.59	0.00
10	WL	Wind load on vehicle	1.00	-	0.00	100.22	4089.90	0.00
	TOTAL			**53783.63**	**-201.05**	**384.27**	**7885.97**	**-13722.13**

STT	LOAD TYPE & NOTATIONS		Factor γ	Nz (KN)	Hx (KN)	Hy (KN)	Mx (KNm)	My (KNm)
IIIb	**STRENGTH STATE III-B**							
1	DC	Dead load of left span	1.25	10077.81	-	-	-	4535.02
2	DC	Dead load of right span	1.25	12082.32	-	-	-	-9061.74
3	DW	Dead load of wearing surface	1.50	2564.01	-	-	-	-289.96
4	DC	Dead load of pier	1.25	18811.37	-	-	-	0.00
5	LL	Live load -(max Mx)	1.35	5124.06	-	-	33331.99	-1569.53
7	BR	Braking force (Start)	1.35	-	-189.34	-	-	-5707.34
9	WA	Water load and stream pressure	1.00	-	-11.71	109.36	551.48	-59.06
10	WS	Wind load on structure	0.40	-	0.00	174.69	3244.59	0.00
11	WL	Wind load on vehicle	1.00	-	0.00	100.22	4089.90	0.00
	TOTAL			**48659.57**	**-201.05**	**384.27**	**41217.96**	**-12152.61**

IIIc	STRENGTH STATE III-C							
1	DC	Dead load of left span	1.25	10077.81	-	-	-	4535.02
2	DC	Dead load of right span	1.25	12082.32	-	-	-	-9061.74
3	DW	Dead load of wearing surface	1.50	2564.01	-	-	-	-289.96
4	DC	Dead load of pier	1.25	18811.37	-	-	-	0.00
5	LL	Live load - (max My)	1.35	6458.92	-	-	0.00	-4844.19
6	BR	Braking force (Start)	1.35	-	-189.34	-	-	-5707.34
7	WA	Water load and stream pressure	1.00	-	-11.71	109.36	551.48	-59.06
8	WS	Wind load on structure	0.40	-	0.00	174.69	3244.59	0.00
9	WL	Wind load on vehicle	1.00	-	0.00	100.22	4089.90	0.00
	TOTAL			**49994.43**	**-201.05**	**384.27**	**7885.97**	**-15427.27**

IV	EXTREME EVENT I							
1	DC	Dead load of left span	1.25	10077.81	-	-	-	4535.02
2	DC	Dead load of right span	1.25	12082.32	-	-	-	-9061.74
3	DW	Dead load of wearing surface	1.50	2564.01	-	-	-	-289.96
4	DC	Dead load of pier	1.25	15049.10	-	-	-	0.00
5	LL	Live load - (maxNz)	0.50	3795.60	-	-	0.00	-1162.61
6	BR	Braking force (Start)	0.50	-	-70.13	-	-	-2113.83
7	WA	Water load and stream pressure	1.00	-	-11.71	109.36	551.48	-59.06
8	EQ	Earth Quake Load	1.00	-	-964.05	1920.73	21598.23	-5274.36
	TOTAL			**43568.84**	**-1045.88**	**2030.09**	**22149.72**	**-13426.54**

V	EXTREME EVENT II							
1	DC	Dead load of left span	1.25	10077.81	-	-	-	4535.02
2	DC	Dead load of right span	1.25	12082.32	-	-	-	-9061.74
3	DW	Dead load of wearing surface	1.50	2564.01	-	-	-	-289.96
4	DC	Dead load of pier	1.25	15049.10	-	-	-	0.00
5	LL	Live load - (maxNz)	0.50	3795.60	-	-	0.00	-1162.61
6	BR	Braking force (Start)	0.50	-	-70.13	-	-	-2113.83
7	WA	Water load and stream pressure	1.00	-	-11.71	109.36	551.48	-59.06
8	CV	Vessels Impact	1.00	-	3244.00	6488.00	32154.55	-16077.28
	TOTAL			**43568.84**	**3162.17**	**6597.36**	**32706.03**	**-24229.46**

VII	SERVICE STATE I_A							
1	DC	Dead load of left span	1.00	8062.25	-	-	-	3628.01
2	DC	Dead load of right span	1.00	9665.85	-	-	-	-7249.39
3	DW	Dead load of wearing surface	1.00	1709.34	-	-	-	-193.31
4	DC	Dead load of pier	1.00	15049.10	-	-	-	0.00
5	LL	Live load -(max Nz)	1.00	7591.20	-	-	0.00	-2325.22
6	BR	Braking force (Start)	1.00	-	-140.25	-	-	-4227.66
7	WA	Water load and stream pressure	1.00	-	-11.71	109.36	551.48	-59.06
8	WS	Wind load on structure	0.30	-	0.00	131.02	2433.44	0.00
9	WL	Wind load on vehicle	1.00	-	0.00	100.22	4089.90	0.00
	TOTAL			**42077.74**	**-151.96**	**340.59**	**7074.82**	**-10426.62**

VIII	SERVICE STATE I_B							
1	DC	Dead load of left span	1.00	8062.25	-	-	-	3628.01
2	DC	Dead load of right span	1.00	9665.85	-	-	-	-7249.39
3	DW	Dead load of wearing surface	1.00	1709.34	-	-	-	-193.31
4	DC	Dead load of pier	1.00	15049.10	-	-	-	0.00
5	LL	Live load - (max Mx)	1.00	3795.60	-	-	24690.36	-1162.61
6	BR	Braking force (Start)	1.00	-	-140.25	-	-	-4227.66
7	WA	Water load and stream pressure	1.00	-	-11.71	109.36	551.48	-59.06
8	WS	Wind load on structure	0.30	-	0.00	131.02	2433.44	0.00
9	WL	Wind load on vehicle	1.00	-	0.00	100.22	4089.90	0.00
	TOTAL			**38282.14**	**-151.96**	**340.59**	**31765.19**	**-9264.01**

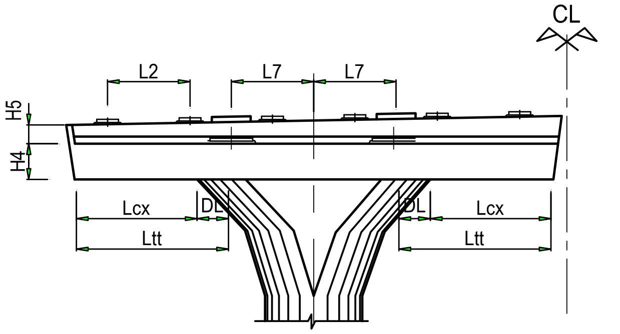
IX	SERVICE STATE I_C							
1	DC	Dead load of left span	1.00	8062.25	-	-	-	3628.01
2	DC	Dead load of right span	1.00	9665.85	-	-	-	-7249.39
3	DW	Dead load of wearing surface	1.00	1709.34	-	-	-	-193.31
4	DC	Dead load of pier	1.00	15049.10	-	-	-	0.00
5	LL	Live load - (max My)	1.00	4784.38	-	-	0.00	-3588.29
6	BR	Braking force (Start)	1.00	-	-140.25	-	-	-4227.66
7	WA	Water load and stream pressure	1.00	-	-11.71	109.36	551.48	-59.06
8	WS	Wind load on structure	0.30	-	0.00	131.02	2433.44	0.00
9	WL	Wind load on vehicle	1.00	-	0.00	100.22	4089.90	0.00
	TOTAL			**39270.93**	**-151.96**	**340.59**	**7074.82**	**-11689.69**

COMBINATION EXTERNAL FORCE ON SECTION III - III

State	Load factors specified			Nz	Hx	Hy	Mx	My
	η_D	η_R	η_I	(KN)	(KN)	(KN)	(KNm)	(KNm)
Strength I_A	1.00	1.00	1.00	56820.11	-257.15	109.36	551.48	-16343.28
Strength I_B	1.00	1.00	1.00	50177.81	-257.15	109.36	43759.62	-14308.71
Strength I_C	1.00	1.00	1.00	51908.19	-257.15	109.36	551.48	-18553.65
Strength II	1.00	1.00	1.00	43535.52	-11.71	720.79	11907.55	-4875.74
Strength III_A	1.00	1.00	1.00	53783.63	-201.05	384.27	7885.97	-13722.13
Strength III_B	1.00	1.00	1.00	48659.57	-201.05	384.27	41217.96	-12152.61
Strength III_C	1.00	1.00	1.00	49994.43	-201.05	384.27	7885.97	-15427.27
Extreme event I	1.00	1.00	1.00	43568.84	-1045.88	2030.09	22149.72	-13426.54
Extreme event II	1.00	1.00	1.00	43568.84	3162.17	6597.36	32706.03	-24229.46
Service state I_A	1.00	1.00	1.00	42077.74	-151.96	340.59	7074.82	-10426.62
Service state I_B	1.00	1.00	1.00	38282.14	-151.96	340.59	31765.19	-9264.01
Service state I_C	1.00	1.00	1.00	39270.93	-151.96	340.59	7074.82	-11689.69

2.5.5. CHECK FOR PIER CAP

2.5.5.1. Over hang Length:



$L_{tt} = L_{cx} + DL$		
L_{tt}:	Over hang length was calculated	
L_{cx} :	Read length of overhang	
L_{cx} =	3.15	(m)
DL :	addition length	
R:	Body pier radius	
R =	1.25	(m)
DL =	0.42	(m)
=> Ltt =	**3.57**	**(m)**

2.5.5.2. Detemine internal force:

2.5.5.2.1 Distribution factor of Live load:

Interior Girder:

*** Distribution factor for Momen**

- One Design lane loaded

$$g= (S / 910)^{0.35} (S.d / L2)^{0.25}$$

- Two or more Design lanes loaded

$$g= (S / 1900)^{0.6} (S.d / L^2)^{0.125}$$

*** Distribution factor for shear**

- One Design lane loaded

$$g= (S / 3050)^{0.6} (d / L)^{0.1}$$

- Two or more Design lanes loaded

$$g= (S / 2250)^{0.8} (d / L)^{0.1}$$

Exterior Girder:

*** Distribution factor for Momen**

- One Design lane loaded: Use lever rule

$$Ge= 1.2(0.5+(S -1800)/2S)$$

- Two or more Design lanes loaded

$$g= e^*g_{interior} =(0.97 + de/8700).Gi$$

*** Distribution factor for shear**

- One Design lane loaded: Use lever rule

$$Ge= 1.2(0.5+(S -1800)/2S)$$

- Two or more Design lanes loaded

$$g= e^*g_{interior} = (0.8 + de/3050).Gi$$

Number Lanes	Girder possition	Live load (LL) Truck	Live load (LL) tandem	Live load (LL) Lane	Pedestrian (PL)
Distribution factor for Momen					
1 Lane	Interior Girder	0.304	0.304	0.304	
	Exterior Girder	0.745	0.745	0.745	
2 or more Lane	Interior Girder	0.508	0.508	0.508	
	Exterior Girder	0.551	0.551	0.551	
Distribution factor for Shear					
1 Lane	Interior Girder	0.592	0.592	0.592	
	Exterior Girder	0.745	0.745	0.745	
2 or more Lane	Interior Girder	0.702	0.702	0.702	
	Exterior Girder	0.792	0.792	0.792	

2.5.5.2.2 Exterior force impact on Over hang of pier cap:

Deadload: Seft weight of Over hang of Pier cap $DC_{(xm)}$ = 107.80 (KN/m)

Seflt weight of super structure apllied to substructure

$DC_{(d)}$ = 671.85 (KN)

Wearing surface load DW = 75.60 (KN)

Live load:

Number Lanes	Girder possition	Live load (LL) Truck	Live load (LL) tandem	Live load (LL) Lane	Total Max (1;2) + (3)	Pedestrian (PL)
		(KN)	(KN)	(KN)		(KN)
		(1)	(2)	(3)	(4)	(5)
Interior Girder		194.26	71.13	90.45	284.70	0.00
Exterior Girder		475.31	174.04	221.31	696.62	0.00

Combination of Exterior load:

Type load / Symbol		Lever arm	Service state		Strength I state	
		(m)	Factor γ	Nz (KN)	Factor γ	Nz (KN)
$DC_{(xm)}$	Seft weight of Pier cap	1.78	1.00	384.49	1.25	480.61
$DC^{exterior}_{(d)}$	Seft weight of Exterior Girder	2.72	1.00	671.85	1.25	839.82
$Dc^{interior}_{(d)}$	Seft weight of Interior Girder	0.60	1.00	671.85	1.25	839.82
$DW^{exterior}$	Seft weight of Deck	2.72	1.00	75.60	1.50	113.41
$DW^{interior}$	Seft weight of Deck	0.60	1.00	75.60	1.50	113.41
$LL^{exterior}_{(D)}$	Live load (Exterior Girder)	2.72	1.00	284.70	1.75	498.23
$LL^{interior}_{(D)}$	Live load (Interior Girder)	0.60	1.00	696.62	1.75	1219.08
$PL^{exterior}$	Pedestrian (Exterior Girder)	2.72	1.00	0.00	1.75	0.00
$PL^{interior}$	Pedestrian (Interior Girder)	0.60	1.00	0.00	1.75	0.00
TOTAL				**2860.73**		**4104.37**

- Internal force at section I - I:

Type load / Symbol		Service state			Strength I state		
		Nz (KN)	**Y** (m)	**Mx** (KNm)	**Nz** (KN)	**Y** (m)	**Mx** (KNm)
$DC_{(xm)}$	Seft weight of Pier cap	384.49	1.78	685.67	480.61	1.78	857.08
$DC^{exterior}_{(d)}$	Seft weight of Exterior Girder	671.85	2.72	1825.20	839.82	2.72	2281.51
$Dc^{interior}_{(d)}$	Seft weight of Interior Girder	671.85	0.60	400.87	839.82	0.60	501.09
$DW^{exterior}$	Seft weight of Deck	75.60	2.72	205.39	113.41	2.72	308.09
$DW^{interior}$	Seft weight of Deck	75.60	0.60	45.11	113.41	0.60	67.67
$LL^{exterior}_{(D)}$	Live load (Exterior Girder)	284.70	2.72	773.45	498.23	2.72	1353.53
$LL^{interior}_{(D)}$	Live load (Interior Girder)	696.62	0.60	415.65	1219.08	0.60	727.39
$PL^{exterior}$	Pedestrian (Exterior Girder)	0.00	2.72	0.00	0.00	2.72	0.00
$PL^{interior}$	Pedestrian (Interior Girder)	0.00	0.60	0.00	0.00	0.60	0.00
TOTAL		**2860.73**		**4351.34**	**4104.37**		**6096.35**

Type Combonation	Load modifier specified			N	Mx
	η_D	η_R	η_I	(KN)	(KNm)
Service state (SLS)	1.00	1.00	1.00	**2860.73**	**4351.34**
Strength state (ULS)	1.00	1.00	1.00	**4104.37**	**6096.35**

Nz : *Vertical load*

Y: *Horizontal lever arm*

Mx: *Horizontal momen*

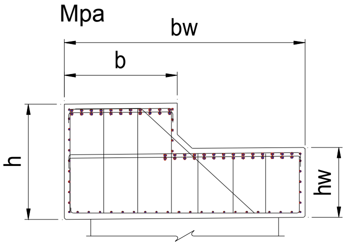
2.5.5.3. Ultimate check and shear capacity check - section I-I

DATA

- Width of flange	b	= 3.20	m
- Width of web	bw	= 1.50	m
- Hight of section	h	= 1.75	m
- Compression flange thickness	h_f	= 1.00	m
- Clear cover thickness	d_c	= 0.13	m
- Flexural depth	d_e	= 1.62	m
- yeild strength of reinforcement	f_y	= 400.00	Mpa
- Elastic modulus of reinforcement	E_s	= 200000.00	Mpa
- Compressive strength of concrete at 28 days	f_c	= 30.00	Mpa
- Unit weigth of concrete	γ_c	= 24.50	kN/m^3
- Elastic modulus of concrete	E_c	= 29440.09	Mpa

Combination interior force of I-I section

State	V (kN)	Mu (kNm)
Service state	2860.73	4351.34
Strength state	4104.37	6096.35



The Datas			Value	Unit
• Hight of Section		h	1750	mm
• Hight of flange		h_f	1000	mm
• Width of flange		b	3200	mm
• Width of web		bw	1500	mm
• Area of section		A_c	4209500	mm^2
• Moment of inertia of concrete section		I_g	7.1E+11	mm^4
• Tension reinforcement	Distance from tension reinf. to extreme compression fiber	d_c	132	mm
	Diameter	$\varnothing$	32.00	mm
	Number of bar	n	22.00	Bar
	Total of reinf.	A_s	17693.45	mm^2
• comp. reinforcement:	Distance from compressive reinf. to extreme Tension fiber	d_c	1700.00	mm
	Diameter	$\varnothing$	16.00	mm
	Number of bar	n	20.00	Bar
	Total of reinf.	A'_s	4021.24	mm^2
Check plexural Moment				
Interior force Combination	**Strength state**			
• Factored Plexural moment		M_u	6.10E+06	kN.mm
• Resistance factor		Φ	0.90	
• The corresponding effective		d_e	1618.00	mm
• Stress block factor		β_1	0.84	
• Depth of the equivalent stress block		$a = c \cdot \beta_1$	67.02	mm
• Distance from extreme compression fiber to the neutral axis		$c=(A_s f_y - 0.85*\beta_1*f_c*(b-b_w)*h_f)/(0.85 f_c \beta_1 b_w)$	-962.25	mm
• Due to neutral axis acrosses the flange, so the cross section should be check by rectangular section				
		$c=(A_s f_y - A'sf'y)/(0.85 f_c \beta_1 b_w)$	80.20	mm
• The nominal flexural resistance:		Mn= As*fy*(de-a/2)	1.12E+07	kN.mm
• Factored flexural resistance		$M_r = \Phi \cdot M_n$	1.01E+07	kN.mm
• Check condition		$\mathbf{M_r > M_u}$	**O.K**	
Mimimum Reinforcement				
• Ratio of tension steel to gross area		$\rho_{min} = A_s/(b \cdot d)$	0.42	%
• Check		$\rho_{min} \geq 0.03 \cdot f'_c/f'_y$ = 0.23 %	**O.K**	
• Cracking moment		1.2Mcr	3.17E+06	Kn.mm
• Check		Mr> min(1.2Mcr, 1.33Mu)	**O.K**	
Maximum Reinforcement		A.5.7.3.3.1		
• Obligation Condition		c/d_e	-0.59	

• **Check**	$c/d_e < 0.42$	O.K	
Check shear resistance			
Interior force Combination : Strength state			
• Factored Shear force	V_u	4104.37	kN
• Resistance factor	Φ	0.90	
• The effective shear deepth	d_v	1486.00	mm
• Effective width	b_v	1500.00	mm
• Angle of inclination of diagonal compressive stress	θ	43	degree
• Angle of inclination of transerverse reinf. To longitudinal axis	α	90	degree
• Factor indicating ability of diagonally cracked concrete to transmit tension	β	2	
• Value	$0.1 \cdot f'_c \cdot b_v \cdot d_v$	6687.00	kN
• Max spacing of transverse reinforcement	s	600.00	mm
• Spacing of stirrup	s	150.00	mm
• Diameter of transverse reinforcement	$\varnothing$	32	mm
• Number of transverse reinf. within distance s	n	2	
• Diameter of stirrup	$\varnothing$	25	mm
• Number of stirrup within distance s	n	4	
• Total area of stirrup	A_v	1963.50	mm^2
• Assume	θ	43.00	degree
• Strain in tensile reinforcement	ex	6.47E-03	
If ex<0, multiple with reduce factor	Fc	-	
• Ratio of shear stress and f'c	V/f'c	0.07	
• β final		1.75	
• θ final		43.00	degree
• The shear resistance of concrete:	V_c	1773.32	kN
• The shear resistance of stirrup	V_s	8343.76	kN
• Value	$0.25 \cdot f'_c \cdot b_v \cdot d_v$	16717.50	kN
• The nominal shear resistance:	$V_n = \mathrm{Min}(0.25 \cdot f'_c \cdot b_v \cdot d_v; V_c + V_s)$	10117.07	kN
• The factored shear resistance	$V_r = \Phi * V_n$	9105.37	kN
• **Check**	$V_r > V_u$	O.K	
•Requiring transverse reinforcement	$V_u > 0.5 \cdot \Phi \quad V_c$	Need	
•Check minimum transverse reinforcement	$A_v > 0.083 \quad (f'_c{}^{0.5}) \cdot b_v \cdot s / f_y$	O.K	
Check crack			
Interior force combination			
• Factored moment	M_S	4.35E+06	kN.mm
• Modulus of rupture of concrete	fr = 0.63*sqrt(f'c)	3.45	MPa
• Distance from extreme tension fiber to the neutral axis	$y_t = h - c$	2712	mm
• Stress of concrete at tension fiber	$f'r = Ms * y_t / I_g$	16.51	MPa
• **Check**			
Stress of concrete f'r>0.8*fr should be controled of cracking by distribution of reinforcement by condition:			
• Crack width parameter	Z	30000.00	N/mm
• Ratio of reinf. Modulus with concrete modulus	$n = E_s/E_c$	7.00	
• The distance from extreme fiber to the neutral axis	c	= 606.03	mm
• Effective moment of inertia	J	2.38E+11	mm4
• Arm	de-c	= 1011.97	mm
• Tension stress in reinforcement	fs =n*Ms*(de-c)/J	= 129.44	Mpa
• Area of concrete having the same centroid as the principal tensile reinforcement divided by number of bars	A	29090.91	mm^2
• Tension stress in reinforcement with service state	$f_{sa} = Z/(d_c \cdot A)^{1/3}$	200.01	Mpa
• **Check condition**	$f_s < f_{sa}$	O.K	
• **Check condition**	$f_s < 0.6 \cdot f_y$	O.K	

2.5.6. DETEMINAL INTERNAL FORCE AT TOP OF PILE

2.5.6.1. General value

Summary of external force acting to bottom pile cap

Combination	V	Hx	Hy	Mx	My
	(KN)	(Kn)	(Kn)	(Kn.m)	(Kn.m)
Strength I	56820	-257	109	43760	-18554
Strength II	43536	-12	721	11908	-4876
Strength III	53784	-201	384	41218	-15427
Service I	42078	-152	341	31765	-11690
Extreme event I (EQ)	43569	-1046	2030	22150	-13427
Extreme event I (CV)	43569	3162	6597	32706	-24229

2.5.6.2. Piling material:

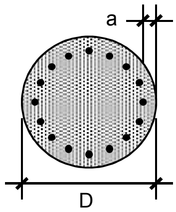
Concrete 30 Mpa

E_b (kg/cm^2)	294401
γ_b (T/m^3)	2.5

Steel bar

Type	CB-400-T
E_t (kg/cm^2)	200000

2.5.6.3. Piling dimension



+ Diameter **D** = 1.50 m

a = 0.08 m

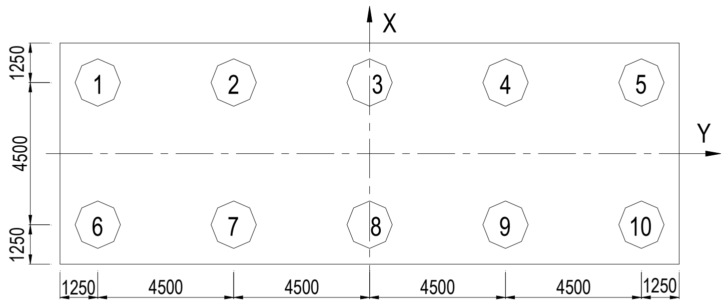
+ Length **L** = 62.00 m

2.5.6.4. Maximum Internal force and displacements at top piling

Internal force and diplacements *(Result follow Piling software)*

Combination	V	H	M	x	y	z
	(KN)	(KN)	(KN.m)	(m)	(m)	(rad)
Strength I	7386.33	25.70	-184.63	-	-	-
Strength II	4869.25	1.20	-69.91	-	-	-
Strength III	6927.33	20.10	-158.40	-	-	-
Service I	5397.96	15.20	-120.15	**0.001**	**-0.001**	**0.006**
Extreme I	5723.36	104.60	197.56	-	-	-
Extreme II	7172.76	316.20	847.28	-	-	-

- Check displacement of top pile not exceed 38mm (10.7.2.7) **OK**



Arrangement of pile

- Result for internal force at top pile for each pile

Combination	Pile Number	**Fx** (KN)	**Fy** (KN)	**N** (KN.m)	**Mx** (m)	**My** (m)
Strength I	1	25.70	-10.90	5901.20	-48.06	-184.63
	2	25.70	-10.90	5420.32	-48.06	-184.63
	3	25.70	-10.90	4939.43	-48.06	-184.63
	4	25.70	-10.90	4458.55	-48.06	-184.63
	5	25.70	-10.90	3977.67	-48.06	-184.63
	6	25.70	-10.90	7386.33	-48.06	-184.63
	7	25.70	-10.90	6905.45	-48.06	-184.63
	8	25.70	-10.90	6424.57	-48.06	-184.63
	9	25.70	-10.90	5943.68	-48.06	-184.63
	10	25.70	-10.90	5462.80	-48.06	-184.63
Strength II	1	1.20	-72.10	4497.97	294.24	-69.91
	2	1.20	-72.10	4332.96	294.24	-69.91
	3	1.20	-72.10	4167.96	294.24	-69.91
	4	1.20	-72.10	4002.95	294.24	-69.91
	5	1.20	-72.10	3837.95	294.24	-69.91
	6	1.20	-72.10	4869.25	294.24	-69.91
	7	1.20	-72.10	4704.25	294.24	-69.91
	8	1.20	-72.10	4539.24	294.24	-69.91
	9	1.20	-72.10	4374.24	294.24	-69.91
	10	1.20	-72.10	4209.23	294.24	-69.91
Strength III	1	20.10	-38.40	5696.84	79.79	-158.40
	2	20.10	-38.40	5230.00	79.79	-158.40
	3	20.10	-38.40	4763.15	79.79	-158.40
	4	20.10	-38.40	4296.31	79.79	-158.40
	5	20.10	-38.40	3829.47	79.79	-158.40
	6	20.10	-38.40	6927.33	79.79	-158.40
	7	20.10	-38.40	6460.49	79.79	-158.40
	8	20.10	-38.40	5993.65	79.79	-158.40
	9	20.10	-38.40	5526.80	79.79	-158.40
	10	20.10	-38.40	5059.96	79.79	-158.40
Service	1	15.20	-34.10	4465.65	81.53	-120.15
	2	15.20	-34.10	4103.65	81.53	-120.15
	3	15.20	-34.10	3741.64	81.53	-120.15
	4	15.20	-34.10	3379.64	81.53	-120.15
	5	15.20	-34.10	3017.64	81.53	-120.15
	6	15.20	-34.10	5397.96	81.53	-120.15
	7	15.20	-34.10	5035.96	81.53	-120.15
	8	15.20	-34.10	4673.96	81.53	-120.15
	9	15.20	-34.10	4311.96	81.53	-120.15
	10	15.20	-34.10	3949.95	81.53	-120.15
Extreme event I	1	104.60	-203.00	4354.24	853.54	197.56
	2	104.60	-203.00	4013.29	853.54	197.56
	3	104.60	-203.00	3672.34	853.54	197.56
	4	104.60	-203.00	3331.39	853.54	197.56
	5	104.60	-203.00	2990.44	853.54	197.56
	6	104.60	-203.00	5723.36	853.54	197.56
	7	104.60	-203.00	5382.41	853.54	197.56
	8	104.60	-203.00	5041.46	853.54	197.56
	9	104.60	-203.00	4700.51	853.54	197.56
	10	104.60	-203.00	4359.56	853.54	197.56
Extreme event II	1	316.20	-659.70	4265.93	2860.41	847.28
	2	316.20	-659.70	3584.71	2860.41	847.28
	3	316.20	-659.70	2903.49	2860.41	847.28
	4	316.20	-659.70	2222.26	2860.41	847.28
	5	316.20	-659.70	1541.04	2860.41	847.28
	6	316.20	-659.70	7172.76	2860.41	847.28
	7	316.20	-659.70	6491.54	2860.41	847.28
	8	316.20	-659.70	5810.31	2860.41	847.28
	9	316.20	-659.70	5129.09	2860.41	847.28
	10	316.20	-659.70	4447.87	2860.41	847.28

2.5.7. ULTIMATE LOAD CHECK, SHEAR CAPACITY AND CRACK CHECK

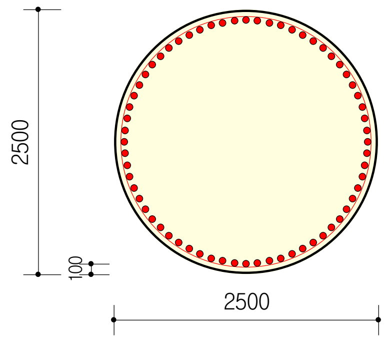
2.5.7.1. Check for pier shaft:

Item		Mark	Unit	Value
• Factored Axial force		Nu	Kn	16644.18
• Factored Plexural moment		Mux	Kn.m	15323.18
• Factored Plexural moment		Muy	Kn.m	-4351.66
• Diameter of Pier shaft		D	m	2.50
• Section area		Ag	m2	4.91
• Moment of inertia of concrete section		Ic	m4	1.92
• Cover thickness		a	m	0.008
• Reinf. Diameter		Ds	mm	32.00
• Number of rebar		n_s	nos	64.00
• Rebar area		As	mm2	51471.85
Check minimum reinforcement				
• Minimum rebar area required (0.135*f'c/fy)*Ag		As req	mm2	49700.98
• Check condition As > (0.135*f'c/fy)*Ag				**OK**
Check maximum reinforcement				
• Maximum rebar area 0.08*Ag		As max	mm2	392699.1
• Check condition As < 0.08*Ag				**OK**
Check ratio spiral or Tier (5.7.4.6)				
• Distance to outside of Spairal or Ties to concrete face			mm	62.00
• Effect diamete		Deff	m	2.38
• Area of core measured to the outside diameter of the spiral			m2	4.43
• Ratio spiral Rebar required		ρsa		0.00361
Required Area of Spiral Rebar	space		mm	150
	Effective length			2.38
	layer			1
	Area			322.1
	Requaired Dhs			20.3
Actuaral	Effective length	d	m	2.376
	Diameter	Dhr	mm	22
	Area of Rebar	Ah	mm2	380.1
	layer	Nl	nos	1
	Total area of spiral	Ac	m2	380.133
	space	s	mm	150
	Ratio spiral Rebar	ρs	-	0.0042664
• Check condition		$\rho_s > \rho_{sa}$		OK
Check Crack (At Service state)				
• Modulus of rupture of concrete		fr = 0.63*sqrt(f'c)	Mpa	3.45
• Stress of concrete at tension fiber		σ'_r	Mpa	5
• If f'r > 0.8fr require check crack		$\sigma'_r > 0.8*\sigma_r$	Mpa	Check
• Center of newtral axial		x	mm	1.3587
• Maximum stress of Compression fiber of concrete		σ_c	Mpa	13.6
• Maximum stress of Compression Rebar		σ_{rc}	Mpa	-182.5
• Maximum stress of Tension Rebar		σ_{rt}	Mpa	151
• Check			σ_{rt} <0.6.fy	OK

MIDAS/Column Design [Check Pier Shaft - D2.5m- Pier P10]

	Company	Nipponkoei	**Project Name**	DN Qn Expressway
	Designer	vinh	**File Name**	D:\...ier Shaft-D2.5m-Pier P10.BOI

1. Geometry and Materials



Design Code : ACI318M-02

Stress Profile : Equivalent Stress Block

Material Data : f_c' = 30 MPa (β_1 = 0.850)

f_y = 400, f_{ys} = 400 MPa

Section Dim. : Φ2500 mm

Effective Len. : KL_u = 12500 mm

Steel Distribut.: 64 - D32 (d_c = 100 mm)

Total Steel Area A_{st} = 50829 mm^2 (ρ_{st} = 0.0104)

2. Member Force and Moment

Unit : kN, kN-m

L.C.	P_u	M_{ux}	M_{uy}	R_{atioV}	V_{ux}	V_{uy}	R_{atioH}	Remark
1	22917.0	131.0	-7697.0	0.352	125.0	34.0	0.009	
2	14671.0	21735.0	-5964.0	0.674	125.0	34.0	0.009	
3	15536.0	131.0	-8086.0	0.297	125.0	34.0	0.009	
4	16274.0	5044.0	-2417.0	0.251	2.0	340.0	0.025	
5	21398.0	3329.0	-6490.0	0.328	97.0	172.0	0.014	
6	13912.0	19995.0	-4989.0	0.606	97.0	172.0	0.014	
7	14580.0	3329.0	-6627.0	0.276	97.0	172.0	0.014	
8	17223.0	15054.0	-5073.0	0.496	254.0	713.0	0.055	
9	17223.0	22368.0	-13963.0	0.788	3136.0	6557.0	0.524	
10	16644.0	2978.0	-4933.0	0.257	72.0	150.0	0.012	
11	14746.0	15323.0	-4352.0	0.479	72.0	150.0	0.012	
12	15241.0	2978.0	-5564.0	0.256	72.0	150.0	0.012	

3. Magnified Moment

KL_u/r_x = 12500/625 = 20.00 < 34-12(M_1/M_2) = 22.00

δ_x = 1.000

KL_u/r_y = 12500/625 = 20.00 < 34-12(M_1/M_2) = 22.00

δ_y = 1.000

4. Design Force and Moment

Design Load Combination No : 9

P_u = 17223.0 kN

M_{ux} = 22368.0, M_{uy} =-13963.0 kN-m

5. Check Axial and Moment Capacity

Rotation Angle and Depth to the Neutral Axis θ = -121.97°, c = 888 mm

Strength Reduction Factor Φ = 0.9000

Maximum Axial Load $\Phi P_{n(max)}$ = 74988.3 kN

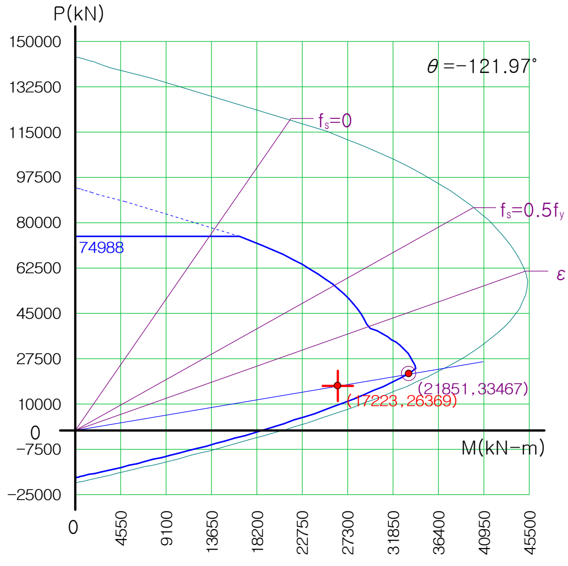
Design Axial Load Strength ΦP_n = 21850.8 kN

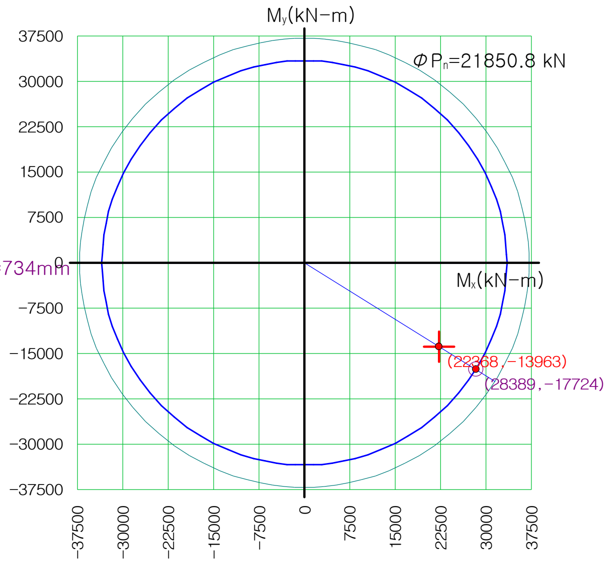
Design Moment Strength ΦM_{nx} = 28388.6 kN-m

ΦM_{ny} = -17723.6 kN-m

Strength Ratio : Applied/Design = 0.788 < 1.000 .... O.K

	Company	Nipponkoei	**Project Name**	DN Qn Expressway
	Designer	vinh	**File Name**	D:\...ier Shaft-D2.5m-Pier P10.BOI





6. Check Shear Capacity

Design Load Combination No : 9

Strength Reduction Factor Φ= 0.750

Design Force Vu = 7268.3 kN (Pu = 17223.0 kN)

Required Hoop Spacing : D22 @ 361 mm

Provided Hoop Spacing : D22 @ 150 mm (Tie)

$\Phi V_c + \Phi V_s$ = 10808.1 + 3069.1 = 13877.3 kN > V_u = 7268.3 kN .... O.K

2.5.7.2. Check for pile:

Item		Mark	Unit	Value
• Factored Axial force		Nu	Kn	5397.96
• Factored Plexural moment		Mux	Kn.m	81.53
• Factored Plexural moment		Muy	Kn.m	-120.15
• Diameter of Pier shaft		D	m	1.50
• Section area		Ag	m2	1.77
• Moment of inertia of concrete section		Ic	m4	0.25
• Cover thickness		a	m	0.075
• Reinf. Diameter		Ds	mm	D32
• Number of rebar		n_s	nos	24
• Rebar area		As	mm2	19301.95
Check minimum reinforcement				
• Minimum rebar area required (0.135*f'c/fy)*Ag		As req	mm2	17892.35
• Check condition As > (0.135*f'c/fy)*Ag				**OK**
Check maximum reinforcement				
• Maximum rebar area 0.08*Ag		As max	mm2	141371.7
• Check condition As < 0.08*Ag				**OK**
Check ratio spiral or Tier (5.7.4.6)				
• Distance to outside of Spairal or Ties to concrete face			mm	68.00
• Effect diamete		Deff	m	1.36
• Area of core measured to the outside diameter of the spiral			m2	1.46
• Ratio spiral Rebar required		ρsa		0.00707
Required Area of Spiral Rebar	space		mm	75
	Effective length			1.36
	layer			1
	Area			180.7
	Requaired Dhs			15.2
Actuaral	Effective length	d	m	1.364
	Diameter	Dhr	mm	16
	Area of Rebar	Ah	mm2	201.1
	layer	Nl	nos	1
	Total area of spiral	Ac	m2	201.062
	space	s	mm	75
	Ratio spiral Rebar	ps	-	0.0078617
• Check condition		$\rho_s > \rho_{sa}$		OK

	Company	Nipponkoei	**Project Name**	DN Qn Expressway
	Designer	vinh	**File Name**	D:\...\Pile D1.5m - Pier P10.B01

1. Geometry and Materials

Design Code : ACI318M-02

Stress Profile : Equivalent Stress Block

Material Data : f_c' = 30 MPa (β_1 = 0.850)

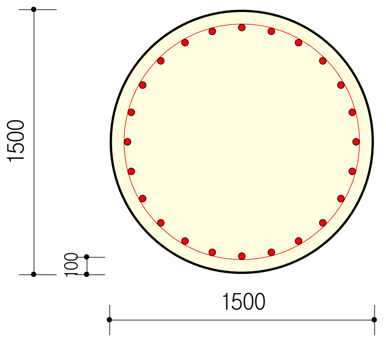
f_y = 400, f_{ys} = 400 MPa

Section Dim. : Φ1500 mm

Effective Len. : KL_u = 9000 mm

Steel Distribut.: 24 - D32 (d_c = 100 mm)

Total Steel Area A_{st} = 19061 mm^2 (ρ_{st} = 0.0108)



2. Member Force and Moment

Unit : kN, kN-m

L.C.	P_u	M_{ux}	M_{uy}	R_{atioV}	V_{ux}	V_{uy}	R_{atioH}	Remark
1	7386.3	-48.1	-184.6	0.272	25.7	10.9	0.006	
2	4869.3	294.2	-69.9	0.179	1.2	72.1	0.015	
3	6927.3	79.8	-158.4	0.255	20.1	38.4	0.009	
4	5398.0	81.5	-120.2	0.199	15.2	34.1	0.008	
5	5723.4	853.5	197.6	0.228	104.6	203.0	0.048	
6	7172.8	2860.4	847.3	0.509	316.2	659.7	0.153	

3. Magnified Moment

KL_u/r_x = 9000/375 = 24.00 > 34-12(M_1/M_2) = 22.00

δ_x = MAX[1.00/(1-P_u/0.75/145156), 1.0] = 1.071

KL_u/r_y = 9000/375 = 24.00 > 34-12(M_1/M_2) = 22.00

δ_y = MAX[1.00/(1-P_u/0.75/145156), 1.0] = 1.071

4. Design Force and Moment

Design Load Combination No : 6

P_u = 7172.8 kN

M_{ux} = 2860.4, M_{uy} = 847.3 kN-m

$\delta_x M_{ux}$= δ_x*M_{ux} = 3062.2 kN-m

$\delta_y M_{uy}$= δ_y*M_{uy}, = 907.0 kN-m

5. Check Axial and Moment Capacity

Rotation Angle and Depth to the Neutral Axis θ = -73.50°, c = 859 mm

Strength Reduction Factor Φ = 0.6500

Maximum Axial Load Φ$P_{n(max)}$ = 27144.3 kN

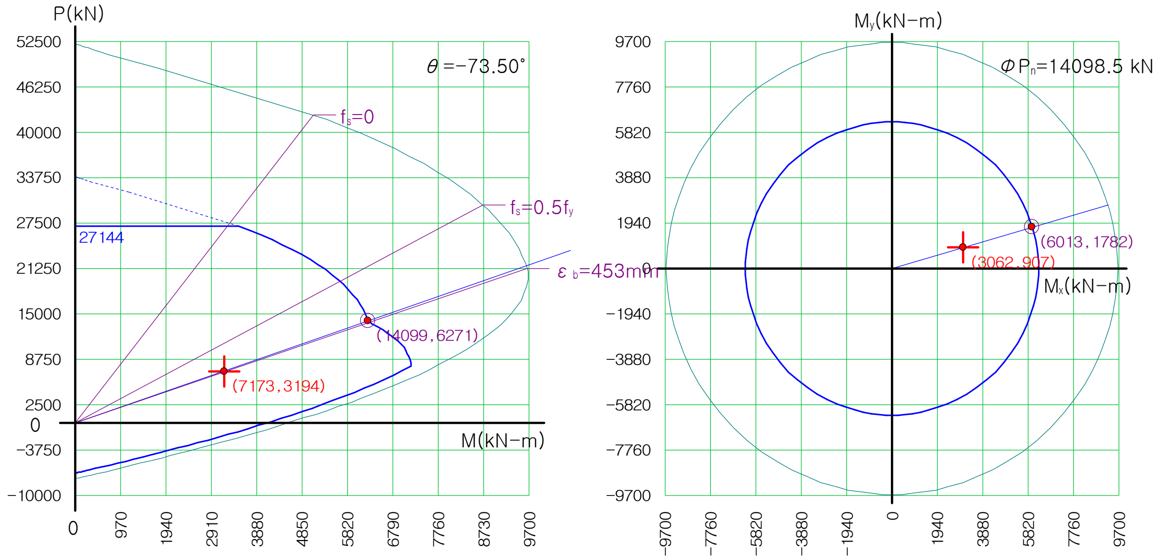
Design Axial Load Strength ΦP_n = 14098.5 kN

Design Moment Strength ΦM_{nx} = 6013.1 kN-m

ΦM_{ny} = 1781.5 kN-m

Strength Ratio : Applied/Design = 0.509 < 1.000 .... O.K

	Company	Nipponkoei	Project Name	DN Qn Expressway
	Designer	vinh	File Name	D:\...\Pile D1.5m - Pier P10.B01



6. Check Shear Capacity

Design Load Combination No : 6

Strength Reduction Factor Φ= 0.750

Design Force Vu = 731.6 kN (Pu = 7172.8 kN)

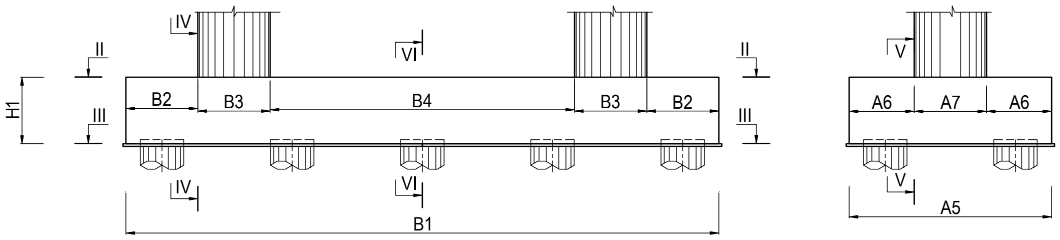
Required Hoop Spacing : D16 @ 508 mm

Provided Hoop Spacing : D16 @ 150 mm (Tie)

$\Phi V_c + \Phi V_s$ = 3845.6 + 924.5 = 4770.1 kN > V_u = 731.6 kN .... O.K

2.5.8. CHECK FOR PILE CAP :

2.5.8.1. External force to section IV-IV, section V-V, section VI-VI



External force to section IV - IV

STATE	Longitudinal direction		
	Q (kN)	M(kN.m)	N (kN)
Strength I	32122.8	72276.4	128.5
Strength II	22696.2	51066.5	6.0
Strength III	29968.2	67428.5	100.5
Service	23369.8	52582.0	76.0
Extreme I	25207.3	56716.4	523.0
Extreme II	29051.6	41166.1	1581.0

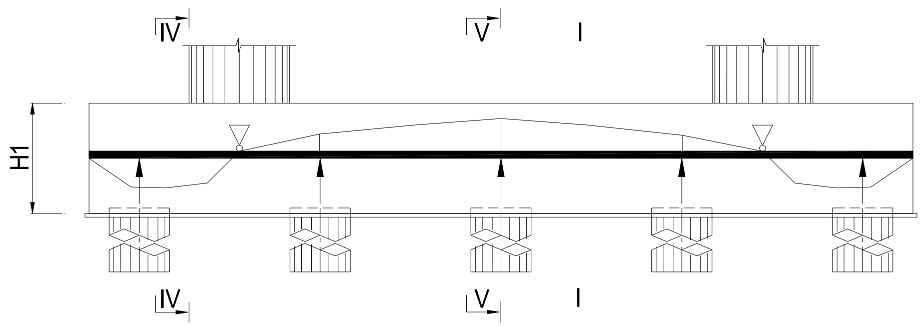
External force to section V - V

STATE	Transverse direction		
	Q (kN)	M(kN.m)	N (kN)
Strength I	13287.5	22083.9	51.4
Strength II	9367.2	15568.3	2.4
Strength III	12624.2	20981.4	40.2
Service	9863.6	16393.3	30.4
Extreme I	10077.6	16749.0	209.2
Extreme II	11438.7	19011.1	632.4

External force to section VI - VI

STATE	Transverse direction		
	Q (kN)	M(kN.m)	N (kN)
Strength I	6053.1	21621.3	51.4
Strength II	4870.8	16943.2	2.4
Strength III	5646.9	20458.2	40.2
Service	4471.8	15173.3	30.4
Extreme I	4731.4	16714.0	209.2
Extreme II	7013.3	18766.6	632.4

Note: External force to section V-V



2.5.8.2. Ultimate check and shear capacity check :

Item			Section IV-IV (Bottom bar)	Section V-V (Bottom bar)	Section VI-VI (Upper bar)	Unit
• Factored Plexural moment		M_u	72276.37	22083.87	21621.30	kN.m
• Factored Shear force		V_u	32122.83	13287.53	7013.30	kN
• Hight of Section		h	2500	2500	2500	mm
• Width of section		b	20500	7000	7000	mm
• Section area		A_c	51250000	17500000	17500000	mm^2
• Moment of inertia of concrete section		I_g	2.7E+13	9.1E+12	9.1E+12	mm^4
• Tension reinforcement:	Distance from tension reinf. to extreme compression fiber	d_c	166	198	141	mm
	Reinf. Diameter	∅	32	32	32	mm
	Space	@	150	150	150	mm
	Number of bar	n	136	46	46	bar
	Total area of reinf.	A_s	109646	37263	37263	mm^2
• comp. reinforcement:	Distance from compressive reinf. to extreme Tension fiber		100	141	148	mm
	Diameter		25	32	32	mm
	Reinf. Space		150	150	150	mm
	Number of bar		136	46	46	bar
	Total area of reinf.	A'_s	51825	37263	37263	mm^2
Check Flexural Moment						
• Resistance factor		Φ	0.90	0.90	0.90	
• The corresponding effective		d_e	2334	2302	2359	mm
• Stress block factor		β_1	0.84	0.84	0.84	
• Depth of the equivalent stress block = c*β1		a	83.90	83.50	83.50	mm
• Distance from extreme compression fiber to the neutral axis		c	100.39	99.92	99.92	mm
• The nominal flexural resistance:		M_n	100525	33690	34539	kN.m
• Factored flexural resistance		$M_r = \Phi.M_n$	90473	30321	31086	kN.m
• **Check condition**		$\mathbf{M_r > M_u}$	**O.K**	**O.K**	**O.K**	
Mimimum Reinforcement						
• Ratio of tension steel to gross area		$\rho = A_s/(b.d)$	0.23	0.23	0.23	%
• **Check**		$\rho > 0.03 \cdot f'_c/f'_y$	**O.K**	**O.K**	**O.K**	0.23
• Cracking moment		1.2Mcr	88422.96	30193.21	30193.21	Kn.m
• **Check**		Mr> min(1.2Mcr, 1.33Mu)	**O.K**	**O.K**	**O.K**	
Maximum Reinforcement						
• Obligation Condition		c/d_e	0.04	0.04	0.04	
• **Check**		$c/d_e < 0.42$	**O.K**	**O.K**	**O.K**	
Check shear resistance						
• Factored Shear force		V_u	32122.83	13287.53	7013.30	kN
• Resistance factor		Φ	0.90	0.90	0.90	
• The effective shear deepth		d_v	2292	2260	2317	mm
• Effective width		b_v	20500	7000	7000	mm
• Angle of inclination of diagonal compressive stress		θ	43	43	43	degree
• Angle of inclination of transerverse reinf. To longitudinal axis		α	90	90	90	degree
• Factor indicating ability of diagonally cracked concrete to transmit tension		β	1.75	1.75	1.75	
• Value		$0.1 \cdot f'_c \cdot b_v \cdot d_v$	140961	47465	48662	kN
• Max spacing of transverse reinforcement		s	600	600	600	mm
• Spacing of stirrup		s	450	450	450	mm
• Diameter of transverse reinforcement		∅	D 32	D 32	D 32	
• Number of transverse reinf. within distance s		n	2	2	2	bar
• Total area of transverse reinf.		A_v	1608	1608	1608	mm^2
• Diameter of stirrup		∅	D 18	D 18	D 18	mm
• Number of stirrup within distance s		n	45	15	15	bar
• Assume		θ	43.00	43.00	43.00	degree
• Strain in tensile reinforcement		ε_x	2.22E-03	2.27E-03	1.76E-03	
If ex<0, multiple with reduce factor		Φc	-	-	-	
• Ratio of shear stress and f'c		V/f'c	0.03	0.03	0.02	
• β final			1.75	1.75	1.75	
• θ final			43.00	43.00	43.00	degree
• Total area of stirrup		A_v	11422.83	3788.76	3788.76	

• The shear resistance of concrete:	V_c	37381.33	12587.25	12904.68	kN
• The shear resistance of stirrup	V_s	11883.48	3886.86	3984.88	kN
• Value	$0.25*f'_c \cdot b_v \cdot d_v$	352402.75	118663.03	121655.53	kN
• The nominal shear resistance:	V_n	49264.81	16474.11	16889.57	kN
• The factored shear resistance	V_r	44338.33	14826.70	15200.61	kN
• Check	$V_r > V_u$	**O.K**	**O.K**	**O.K**	
•Requiring transverse reinforcement	$V_u > 0.5 \cdot \Phi \quad V_c$	**Need**	**Need**	**Need**	
•Check minimum transverse reinforcement	$A_v > 0.083 \quad (f'^{0.5}_c) \cdot b_v \cdot s/f_y$	**O.K**	**O.K**	**O.K**	
Check crack					
Interior force combination	**Service**				
• Factored moment	M_u	5.26E+04	1.64E+04	1.52E+04	kN.m
• Modulus of rupture of concrete	fr = 0.63*sqrt(f'c)	3.45	3.45	3.45	MPa
• Distance from extreme tension fiber to the neutral axis	y_t = h-c	2400	2400	2400	mm
• Stress of concrete at tension fiber	$f'r = Ms^*y_t/I_g$	5	4	4	MPa
• Check	f'r >	0.8*fr	0.8*fr	0.8*fr	
		check crack	check crack	check crack	
• Crack width parameter	Z	= 23000	= 23000	= 23000	N/mm
• Ratio of reinf. Modulus with concrete modulus	$n = E_s/E_c$	= 7.00	= 7.00	= 7.00	
• The distance from extreme fiber to the neutral axis	c	= 457.17	= 453.14	= 458.21	mm
• Effective moment of inertia	J	3.36E+12	1.11E+12	1.17E+12	mm^4
• Arm	de-c	= 1876.83	= 1848.86	= 1900.79	mm
• Tension stress in reinforcement	fs =n*Ms*(de-c)/J	= 205.81	= 191.35	= 173.01	MPa
• Area of concrete having the same centroid as the principal tensile reinfor	A	= 15037	= 15108	= 15108	mm2
• Tension stress in reinforcement with service state	$f_{sa} = Z/(d_c{}^*A)^{1/3}$	= 230.58	= 230.22	= 230.22	Mpa
• Check condition	$f_s < f_{sa}$	**O.K**	**O.K**	**O.K**	
• Check condition	$f_s < 0.6{}^*f_y$	**O.K**	**O.K**	**O.K**	

2.6 PIER P11
2.6 TRỤ CẦU P11

PIER P11 - CALCULATION SHEET

CONTENT:

2.6.1. GENERAL DATA

CALCULATION PROCEDURE & STANDARD:

- Bridge Design Standard 22 TCN - 272 - 05

2.6.1.1. Design live load

Design vehicle load		HL93	22TCN 272 - 05
Number of lane		6	(lane)
Pedestrian		0.00	KG/m^2

2.6.1.2. Bridge width

Width carriageway	B =	12.00	(m)
Width of median guardrail	B_{pc} =	0.50	(m)
Width parapet	B_{lc} =	0.50	(m)
Bridge width	B =	13.00	(m)

2.6.1.3. Superstructure:

Span-arrangement		Continuous box girder 65+5@100+65m	
Height of box girder at pier section	H =	6.00	(m)
Height of box girder at mid-span section	h =	2.50	(m)
Pavement thickness	d_{BTN} =	0.084	(m)

2.6.1.4. Material property:

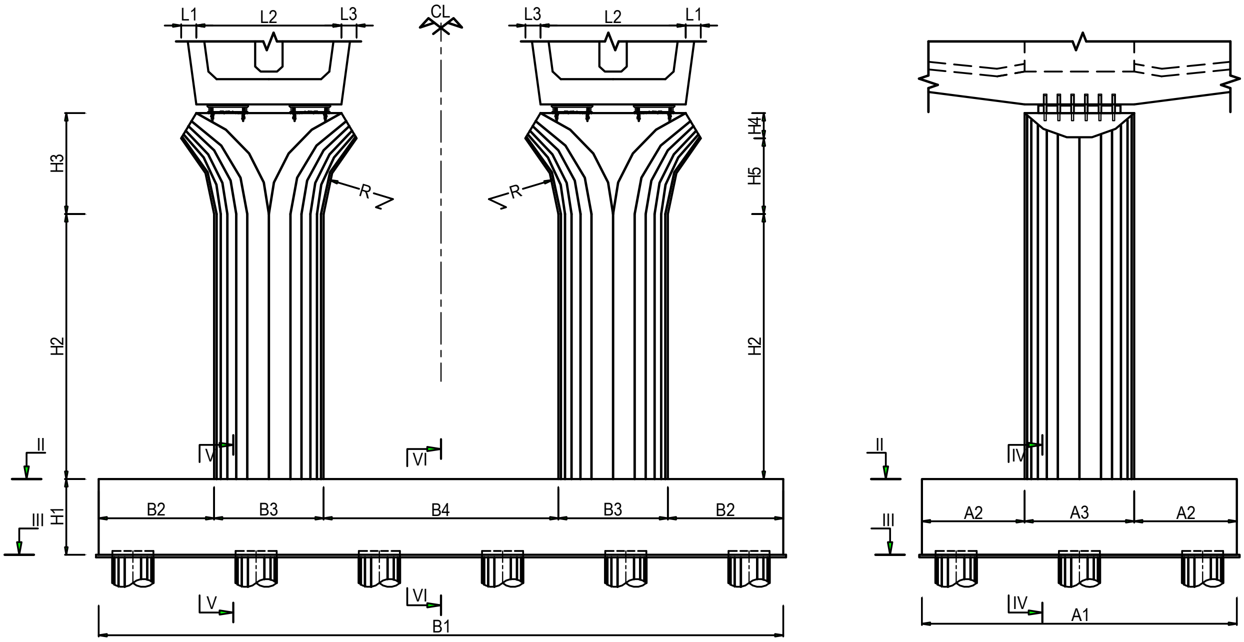
Concrete

Compressive strength of cylindrical at 28 day	f 'c =	30.00	MPa
Concrete density	g =	24.50	KN/m^3
Elastic modulus	Ec =	29440	MPa
Tension strength of concrete	fr =	3.45	MPa

Steel

Steel modulus	Es =	200000	MPa
Yeild strength of steel bar	fy =	400.00	MPa
Calculation unit:		KN, m, KN.m, Mpa, KN/m^2	

2.6.1.5. THE PIER GEOMETRIC



vertical			Horizontal			Thickness		
Remark	Value	Unit	Remark	Value	Unit	Remark	Value	Unit
a_1 =	11.50	(m)	L_1 =	0.55	(m)	h_1 =	3.00	(m)
a_2 =	3.75	(m)	L_2 =	5.30	(m)	h_2 =	7.000	(m)
a_3 =	4.00	(m)	L_3 =	0.55	(m)	h_3 =	4.000	(m)
			B_1 =	25.00	(m)	h_4 =	1.000	(m)
			B_2 =	3.965	(m)	h_5 =	3.000	(m)
			B_3 =	4.00	(m)			
			B_4 =	9.070	(m)			
			R =	4.34	(m)			

The design elevation:

Proposed height	EL_{mc} =	18.500	(m)			
Elevation of top of pier cap	EL_{xm} =	12.295	(m)			
Hight water level (H1%)	EL_{MNTK} =	9.200	(m)			
Daily water level (H5%)	EL_{MNTB} =	4.070	(m)			
Ground elevation	EL_{TN} =	3.090	(m)			
Daily water level (H5%)	EL_{TT} =	4.070	(m)			
Top of Pile cap (Section II - II)	$EL_{MC\ II\text{-}II}$ =	0.895	(m)			
Bottom of Pile cap (Section III - III)	$EL_{MCIII\text{-}III}$ =	-2.105	(m)			

2.6.2. CALCULATED PIER FORCE AND PILE FORCE

2.6.2.1. Loading combination at bottom of pier shaft

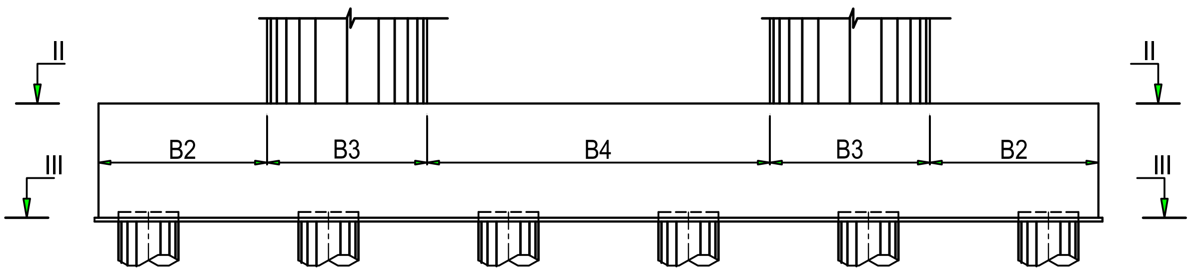
(Result from RM2010 SPACE-FRAME program)

Loading combination in accordance with 22 TCN 272-05, table 3.4.1.1 was mentioned in appendix of this report

LOADING COMBINATION AT TOP OF PILE CAP DUE TO SUPERSTRUCTURE + VESSEL COLLISION AND SEFTWEIGHT						
Limited state	Vertical axis	Longitudinal axis		Horizontal axis		Remarks
	N_x (kN)	H_y (kN)	M_z (kNm)	H_z (kN)	M_y (kNm)	
STRENGTH I	51377.00	56.00	258.00	200.00	6129.00	
STRENGTH II	43960.00	56.00	258.00	1529.00	22349.00	
STRENGTH III	49681.00	56.00	258.00	707.00	13206.00	
SERVICE I	39045.00	56.00	258.00	745.00	12839.00	
SERVICE III	39075.00	56.00	258.00	746.00	12848.00	
EXTREME EVENT (CV)	45866.00	3300.00	13461.00	5876.00	16073.00	
EXTREME EVENT (EQ trans)	45904.00	56.00	258.00	1859.00	28389.00	
EXTREME EVENT (EQ long)	46095.00	561.00	2582.00	946.00	11272.00	

2.6.2.2. Loading combination at bottom of pile cap

LOADING COMBINATION AT TOP OF PILE CAP DUE TO SUPERSTRUCTURE + VESSEL						
Limited state	Vertical axis	Longitudinal axis		Horizontal axis		Remarks
	N_x (kN)	H_y (kN)	M_z (kNm)	H_z (kN)	M_y (kNm)	
STRENGTH I	140503	149	908	426	12190	
STRENGTH II	125671	149	908	3084	53912	
STRENGTH III	137113	149	908	1439	29683	
SERVICE I	108491	149	908	1515	29437	
SERVICE III	108550	149	908	1517	29460	
EXTREME EVENT (CV)	129482	3318	23389	5889	33534	
EXTREME EVENT (EQ trans)	129558	149	908	3745	67599	
EXTREME EVENT (EQ long)	129940	1488	9079	2149	28232	



2.6.2.3. Loading combination at top of piling

2.6.2.3.1. Piling material:

Concrete 30 Mpa

E_c (Mpa)	294401
γ_c (KN/m^3)	24.5

Steel bar

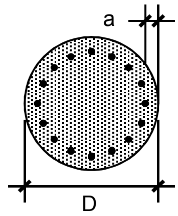
Type	CB-400-T
E_s (Mpa)	200000

2.6.2.3.2. Piling dimension

+ Diameter **D** = 1.50 m

a = 0.100 m

+ Length **L** = 62.00 m

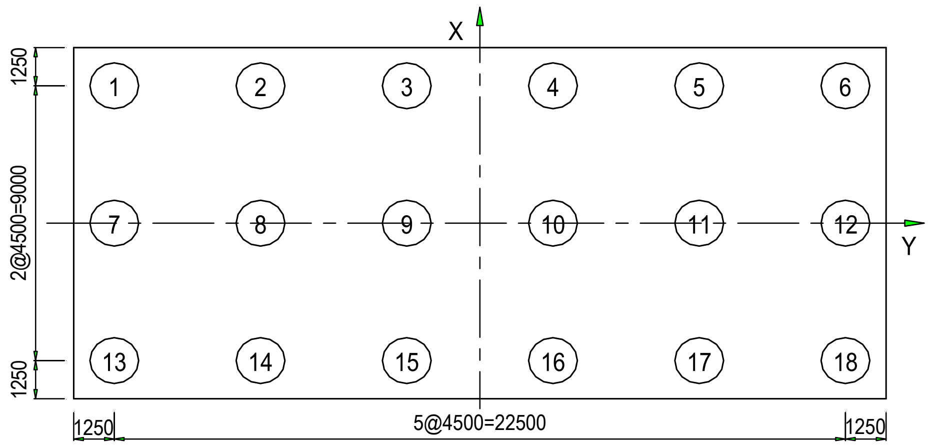


2.6.2.3.3. Maximum Internal force and displacement at top piling

Maximum Internal force and diplacements *(Result follow Piling software)*

Combination	N (KN)	Hx (KN)	My (KN.m)	x (m)	y (m)	z (rad)
Strength I	7980.50	8.28	32.11	-	-	-
Strength II	7717.65	8.28	32.11	-	-	-
Strength III	8022.62	8.28	32.11	-	-	-
Service I	6433.56	8.28	32.11	**0.000**	**-0.002**	**0.008**
Service III	6437.17	8.28	32.11	**0.000**	**-0.002**	**0.008**
Extreme I (CV)	8490.76	184.33	703.53	-	-	-
Extreme II (EQ trans)	8107.64	8.28	32.11	-	-	-
Extreme II (EQ long)	7890.28	82.67	320.59	-	-	-

- Check displacement of top pile not exceed 38mm (10.7.2.7) **OK**



Arrangement of pile

2.6.2.3.4. Internal force for each pile

Combination	Pile Number	Hx	Hy	N	Mx	My
		(KN)	(KN)	(KN.m)	(m)	(m)
Strength I	1	8.28	-23.67	7925.47	95.90	32.11
	2	8.28	-23.67	7866.56	95.90	32.11
	3	8.28	-23.67	7807.66	95.90	32.11
	4	8.28	-23.67	7748.75	95.90	32.11
	5	8.28	-23.67	7689.85	95.90	32.11
	6	8.28	-23.67	7630.94	95.90	32.11
	7	8.28	-23.67	7952.99	95.90	32.11
	8	8.28	-23.67	7894.08	95.90	32.11
	9	8.28	-23.67	7835.18	95.90	32.11
	10	8.28	-23.67	7776.27	95.90	32.11
	11	8.28	-23.67	7717.37	95.90	32.11
	12	8.28	-23.67	7658.46	95.90	32.11
	13	8.28	-23.67	7980.50	95.90	32.11
	14	8.28	-23.67	7921.60	95.90	32.11
	15	8.28	-23.67	7862.69	95.90	32.11
	16	8.28	-23.67	7803.79	95.90	32.11
	17	8.28	-23.67	7744.88	95.90	32.11
	18	8.28	-23.67	7685.98	95.90	32.11
Strength II	1	8.28	-171.33	7662.62	724.06	32.11
	2	8.28	-171.33	7379.25	724.06	32.11
	3	8.28	-171.33	7095.89	724.06	32.11
	4	8.28	-171.33	6812.52	724.06	32.11
	5	8.28	-171.33	6529.16	724.06	32.11
	6	8.28	-171.33	6245.79	724.06	32.11
	7	8.28	-171.33	7690.14	724.06	32.11
	8	8.28	-171.33	7406.77	724.06	32.11
	9	8.28	-171.33	7123.41	724.06	32.11
	10	8.28	-171.33	6840.04	724.06	32.11
	11	8.28	-171.33	6556.68	724.06	32.11
	12	8.28	-171.33	6273.31	724.06	32.11
	13	8.28	-171.33	7717.65	724.06	32.11
	14	8.28	-171.33	7434.29	724.06	32.11
	15	8.28	-171.33	7150.92	724.06	32.11
	16	8.28	-171.33	6867.56	724.06	32.11
	17	8.28	-171.33	6584.19	724.06	32.11
	18	8.28	-171.33	6300.83	724.06	32.11
Strength III	1	8.28	-79.94	7967.58	333.92	32.11
	2	8.28	-79.94	7816.50	333.92	32.11
	3	8.28	-79.94	7665.42	333.92	32.11
	4	8.28	-79.94	7514.33	333.92	32.11
	5	8.28	-79.94	7363.25	333.92	32.11
	6	8.28	-79.94	7212.16	333.92	32.11
	7	8.28	-79.94	7995.10	333.92	32.11
	8	8.28	-79.94	7844.02	333.92	32.11
	9	8.28	-79.94	7692.93	333.92	32.11
	10	8.28	-79.94	7541.85	333.92	32.11
	11	8.28	-79.94	7390.76	333.92	32.11
	12	8.28	-79.94	7239.68	333.92	32.11
	13	8.28	-79.94	8022.62	333.92	32.11
	14	8.28	-79.94	7871.53	333.92	32.11
	15	8.28	-79.94	7720.45	333.92	32.11
	16	8.28	-79.94	7569.36	333.92	32.11
	17	8.28	-79.94	7418.28	333.92	32.11
	18	8.28	-79.94	7267.20	333.92	32.11

- Result for internal force at top pile

Combination	Pile Number	**Hx** (KN)	**Hy** (KN)	**N** (KN.m)	**Mx** (m)	**My** (m)
Service I	1	8.28	-84.17	6378.53	353.13	32.11
	2	8.28	-84.17	6227.02	353.13	32.11
	3	8.28	-84.17	6075.51	353.13	32.11
	4	8.28	-84.17	5924.01	353.13	32.11
	5	8.28	-84.17	5772.50	353.13	32.11
	6	8.28	-84.17	5621.00	353.13	32.11
	7	8.28	-84.17	6406.04	353.13	32.11
	8	8.28	-84.17	6254.54	353.13	32.11
	9	8.28	-84.17	6103.03	353.13	32.11
	10	8.28	-84.17	5951.53	353.13	32.11
	11	8.28	-84.17	5800.02	353.13	32.11
	12	8.28	-84.17	5648.51	353.13	32.11
	13	8.28	-84.17	6433.56	353.13	32.11
	14	8.28	-84.17	6282.05	353.13	32.11
	15	8.28	-84.17	6130.55	353.13	32.11
	16	8.28	-84.17	5979.04	353.13	32.11
	17	8.28	-84.17	5827.54	353.13	32.11
	18	8.28	-84.17	5676.03	353.13	32.11
Service III	1	8.28	-84.28	6382.14	353.61	32.11
	2	8.28	-84.28	6230.50	353.61	32.11
	3	8.28	-84.28	6078.86	353.61	32.11
	4	8.28	-84.28	5927.22	353.61	32.11
	5	8.28	-84.28	5775.58	353.61	32.11
	6	8.28	-84.28	5623.94	353.61	32.11
	7	8.28	-84.28	6409.66	353.61	32.11
	8	8.28	-84.28	6258.02	353.61	32.11
	9	8.28	-84.28	6106.38	353.61	32.11
	10	8.28	-84.28	5954.74	353.61	32.11
	11	8.28	-84.28	5803.10	353.61	32.11
	12	8.28	-84.28	5651.46	353.61	32.11
	13	8.28	-84.28	6437.17	353.61	32.11
	14	8.28	-84.28	6285.53	353.61	32.11
	15	8.28	-84.28	6133.89	353.61	32.11
	16	8.28	-84.28	5982.25	353.61	32.11
	17	8.28	-84.28	5830.61	353.61	32.11
	18	8.28	-84.28	5678.97	353.61	32.11
Extreme Event I (with CV)	1	184.33	-327.17	7155.48	1442.78	703.53
	2	184.33	-327.17	6903.61	1442.78	703.53
	3	184.33	-327.17	6651.74	1442.78	703.53
	4	184.33	-327.17	6399.87	1442.78	703.53
	5	184.33	-327.17	6148.00	1442.78	703.53
	6	184.33	-327.17	5896.13	1442.78	703.53
	7	184.33	-327.17	7823.12	1442.78	703.53
	8	184.33	-327.17	7571.25	1442.78	703.53
	9	184.33	-327.17	7319.38	1442.78	703.53
	10	184.33	-327.17	7067.51	1442.78	703.53
	11	184.33	-327.17	6815.64	1442.78	703.53
	12	184.33	-327.17	6563.77	1442.78	703.53
	13	184.33	-327.17	8490.76	1442.78	703.53
	14	184.33	-327.17	8238.89	1442.78	703.53
	15	184.33	-327.17	7987.02	1442.78	703.53
	16	184.33	-327.17	7735.15	1442.78	703.53
	17	184.33	-327.17	7483.28	1442.78	703.53
	18	184.33	-327.17	7231.41	1442.78	703.53

- Result for internal force at top pile

Combination	Pile Number	**Hx** (KN)	**Hy** (KN)	**N** (KN.m)	**Mx** (m)	**My** (m)
Extreme Event I (with EQtran)	1	8.28	-208.06	8052.61	877.40	32.11
	2	8.28	-208.06	7699.63	877.40	32.11
	3	8.28	-208.06	7346.64	877.40	32.11
	4	8.28	-208.06	6993.66	877.40	32.11
	5	8.28	-208.06	6640.68	877.40	32.11
	6	8.28	-208.06	6287.69	877.40	32.11
	7	8.28	-208.06	8080.12	877.40	32.11
	8	8.28	-208.06	7727.14	877.40	32.11
	9	8.28	-208.06	7374.16	877.40	32.11
	10	8.28	-208.06	7021.18	877.40	32.11
	11	8.28	-208.06	6668.19	877.40	32.11
	12	8.28	-208.06	6315.21	877.40	32.11
	13	8.28	-208.06	8107.64	877.40	32.11
	14	8.28	-208.06	7754.66	877.40	32.11
	15	8.28	-208.06	7401.68	877.40	32.11
	16	8.28	-208.06	7048.69	877.40	32.11
	17	8.28	-208.06	6695.71	877.40	32.11
	18	8.28	-208.06	6342.73	877.40	32.11
Extreme Event I (with EQlong)	1	82.67	-119.39	7340.29	512.63	320.59
	2	82.67	-119.39	7181.74	512.63	320.59
	3	82.67	-119.39	7023.18	512.63	320.59
	4	82.67	-119.39	6864.62	512.63	320.59
	5	82.67	-119.39	6706.06	512.63	320.59
	6	82.67	-119.39	6547.50	512.63	320.59
	7	82.67	-119.39	7615.29	512.63	320.59
	8	82.67	-119.39	7456.73	512.63	320.59
	9	82.67	-119.39	7298.17	512.63	320.59
	10	82.67	-119.39	7139.61	512.63	320.59
	11	82.67	-119.39	6981.05	512.63	320.59
	12	82.67	-119.39	6822.49	512.63	320.59
	13	82.67	-119.39	7890.28	512.63	320.59
	14	82.67	-119.39	7731.72	512.63	320.59
	15	82.67	-119.39	7573.16	512.63	320.59
	16	82.67	-119.39	7414.60	512.63	320.59
	17	82.67	-119.39	7256.04	512.63	320.59
	18	82.67	-119.39	7097.49	512.63	320.59

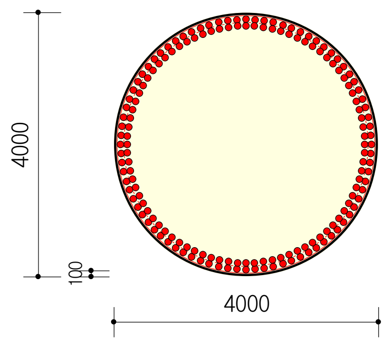
2.6.3. Check for pier shaft

Item		Mark	Unit	Value
• Factored Axial force		Nu	Kn	39075.00
• Factored Plexural moment		Mux	Kn.m	258.00
• Factored Plexural moment		Muy	Kn.m	12848.00
• Diameter of Pier shaft		D	m	4.00
• Section area		Ag	m2	12.57
• Moment of inertia of concrete section		Ic	m4	12.57
• Cover thickness		a	m	0.075
• Reinf. Diameter		Ds	mm	32.00
• Number of rebar		n_s	nos	168.00
• Rebar area		As	mm2	135113.62
Check minimum reinforcement				
• Minimum rebar area required (0.135*f'c/fy)*Ag		As req	mm2	127234.50
• Check condition As > (0.135*f'c/fy)*Ag				**OK**
Check maximum reinforcement				
• Maximum rebar area 0.08*Ag		As max	mm2	1005309.6
• Check condition As < 0.08*Ag				**OK**
Check ratio spiral or Tier (5.7.4.6)				
• Distance to outside of Spairal or Ties to concrete face			mm	66.00
• Effect diamete		Deff	m	3.87
• Area of core measured to the outside diameter of the spiral			m2	11.75
• Ratio spiral Rebar required		ρsa		0.00234
Required Area of Spiral Rebar	space		mm	200
	Effective length			3.87
	layer			2
	Area			226.6
	Requaired Dhs			17.0
Actuaral	Effective length	d	m	3.868
	Diameter	Dhr	mm	18
	Area of Rebar	Ah	mm2	254.5
	layer	Nl	nos	2
	Total area of spiral	Ac	m2	508.938
	space	s	mm	200
	Ratio spiral Rebar	ρs	-	0.0026315
• Check condition		$\rho_s > \rho_{sa}$		OK
Check Crack (At Service State)				
• Modulus of rupture of concrete		fr = 0.63*sqrt(f'c)	Mpa	3.45
• Stress of concrete at tension fiber		σ'_r	Mpa	2
• If f'r > 0.8fr require check crack		$\sigma'_r > 0.8*\sigma_r$	Mpa	No check
• Center of newtral axial		x	mm	3.02517
• Maximum stress of Compression fiber of concrete		σ_c	Mpa	7.7
• Maximum stress of Compression Rebar		σ_{rc}	Mpa	-116.5
• Maximum stress of Tension Rebar		σ_{rt}	Mpa	29.2
• Check			σ_{rt} <0.6.fy	OK

MIDAS/Set **Column Design [Pier Shaft D4.0m-P11]**

Company	Nipponkoei	**Project Name**	DN Qn Expressway
Designer	vinh	**File Name**	D:\...\bodypier-D4.0m-P11.BOI

1. Geometry and Materials



Design Code : ACI318M-02

Stress Profile : Equivalent Stress Block

Material Data : f_c' = 30 MPa (β_1 = 0.850)

f_y = 400, f_{ys} = 400 MPa

Section Dim. : Φ4000 mm

Effective Len. : KL_u = 11000 mm

Steel Distribut.: 84 - D32 (d_c = 100 mm)

: 84 - D32 (d_c = 200 mm)

Total Steel Area A_{st} = 131896 mm^2 (ρ_{st} = 0.0106)

2. Member Force and Moment

Unit : kN, kN-m

L.C.	P_u	M_{ux}	M_{uy}	R_{atioV}	V_{ux}	V_{uy}	R_{atioH}	Remark
1	51377.0	258.0	6129.0	0.271	56.0	200.0	0.007	
2	43960.0	258.0	22349.0	0.272	56.0	1529.0	0.050	
3	49681.0	258.0	13206.0	0.262	56.0	707.0	0.023	
4	39045.0	258.0	12839.0	0.210	56.0	745.0	0.024	
5	39075.0	258.0	12848.0	0.210	56.0	746.0	0.025	
6	45866.0	13461.0	16073.0	0.268	3300.0	5876.0	0.220	
7	45904.0	258.0	28389.0	0.311	56.0	1859.0	0.061	
8	46095.0	2582.0	11272.0	0.243	561.0	946.0	0.036	

3. Magnified Moment

KL_u/r_x = 11000/1000 = 11.00 < 34-12(M_1/M_2) = 22.00

δ_x = 1.000

KL_u/r_y = 11000/1000 = 11.00 > 34-12(M_1/M_2) = 22.00

δ_y = 1.000

4. Design Force and Moment

Design Load Combination No : 7

P_u = 45904.0 kN

M_{ux} = 258.0, M_{uy} = 28389.0 kN-m

$\delta_x M_{ux}$= δ_x*MAX[M_{ux}, $P_u e_{min}$] = 6740.1 kN-m

$\delta_y M_{uy}$= δ_y*M_{uy}, = 30876.7 kN-m

5. Check Axial and Moment Capacity

Rotation Angle and Depth to the Neutral Axis θ = -12.31°, c = 2986 mm

Strength Reduction Factor Φ = 0.6500

Maximum Axial Load $\Phi P_{n(max)}$ = 189731.1 kN

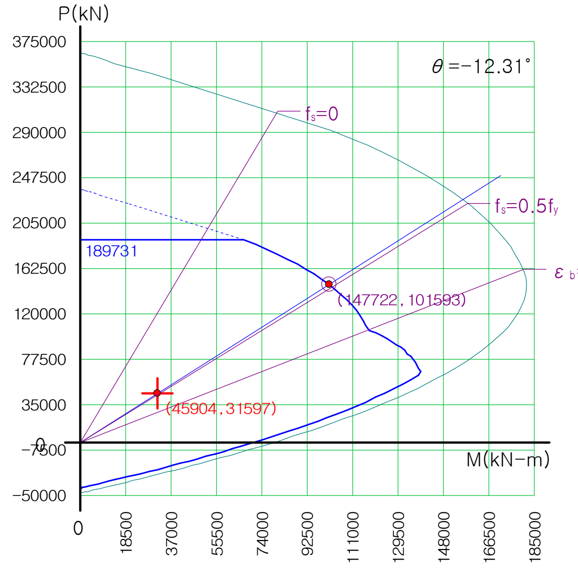
Design Axial Load Strength ΦP_n = 147721.9 kN

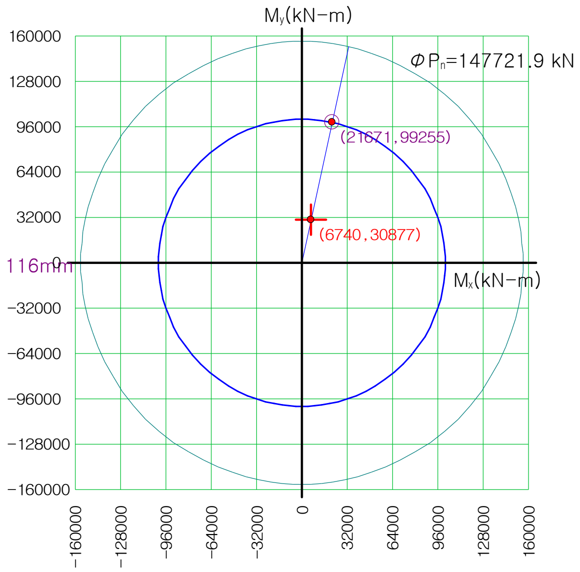
Design Moment Strength ΦM_{nx} = 21671.3 kN-m

ΦM_{ny} = 99254.6 kN-m

Strength Ratio : Applied/Design = 0.311 < 1.000 .... O.K

Company	Nipponkoei	**Project Name**	DN Qn Expressway
Designer	vinh	**File Name**	D:\...\bodypier-D4.0m-P11.BOI





6. Check Shear Capacity

Design Load Combination No : 6

Strength Reduction Factor Φ= 0.750

Design Force Vu = 6739.2 kN (Pu = 45866.0 kN)

Required Hoop Spacing : D18 @ 508 mm

Provided Hoop Spacing : D18 @ 200 mm (Tie)

$\Phi V_c + \Phi V_s$ = 27885.8 + 2734.6 = 30620.3 kN > V_u = 6739.2 kN .... O.K

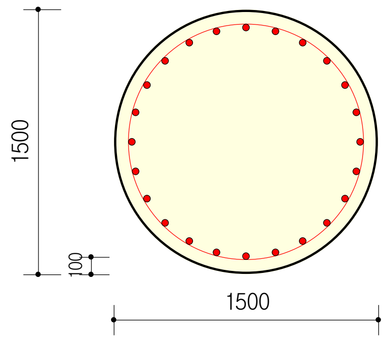
2.6.4. Check for pile

Item		Mark	Unit	Value
• Factored Axial force		Nu	Kn	6382.14
• Factored Plexural moment		Mux	Kn.m	353.61
• Factored Plexural moment		Muy	Kn.m	32.11
• Diameter of Pier shaft		D	m	1.50
• Section area		Ag	m2	1.77
• Moment of inertia of concrete section		Ic	m4	0.25
• Cover thickness		a	m	0.075
• Reinf. Diameter		Ds	mm	D32
• Number of rebar		n_s	nos	24
• Rebar area		As	mm2	19301.95
Check minimum reinforcement				
• Minimum rebar area required (0.135*f'c/fy)*Ag		As req	mm2	17892.35
• Check condition As > (0.135*f'c/fy)*Ag				**OK**
Check maximum reinforcement				
• Maximum rebar area 0.08*Ag		As max	mm2	141371.7
• Check condition As < 0.08*Ag				**OK**
Check ratio spiral or Tier (5.7.4.6)				
• Distance to outside of Spairal or Ties to concrete face			mm	68.00
• Effect diamete		Deff	m	1.36
• Area of core measured to the outside diameter of the spiral			m2	1.46
• Ratio spiral Rebar required		ρsa		0.00707
Required Area of Spiral Rebar	space		mm	75
	Effective length			1.36
	layer			1
	Area			180.7
	Requaired Dhs			15.2
Actuaral	Effective length	d	m	1.364
	Diameter	Dhr	mm	16
	Area of Rebar	Ah	mm2	201.1
	layer	Nl	nos	1
	Total area of spiral	Ac	m2	201.062
	space	s	mm	75
	Ratio spiral Rebar	ρs	-	0.0078617
• Check condition		$\rho_s > \rho_{sa}$		OK

	Company	Nipponkoei	Project Name	DN Qn Expressway
	Designer	vinh	File Name	D:\...\Pile D1.5m - Pier P11.BOI

1. Geometry and Materials

Design Code : ACI318M-02
Stress Profile : Equivalent Stress Block
Material Data : f_c' = 30 MPa (β_1 = 0.850)
f_y = 400, f_{ys} = 400 MPa
Section Dim. : Φ1500 mm
Effective Len. : KL_u = 15000 mm
Steel Distribut.: 24 - D32 (d_c = 100 mm)
Total Steel Area A_{st} = 19061 mm² (ρ_{st} = 0.0108)



2. Member Force and Moment

Unit : kN, kN-m

L.C.	P_u	M_{ux}	M_{uy}	R_{atioV}	V_{ux}	V_{uy}	R_{atioH}	Remark
1	7980.5	96.0	32.0	0.294	8.0	24.0	0.005	
2	7717.6	724.0	32.0	0.290	8.0	171.0	0.036	
3	8022.6	334.0	32.0	0.296	8.0	80.0	0.017	
4	6433.6	353.0	32.0	0.237	8.0	84.0	0.018	
5	6437.2	354.0	32.0	0.237	8.0	84.0	0.018	
6	8490.8	1443.0	704.0	0.391	184.0	327.0	0.078	
7	8107.6	877.0	32.0	0.314	8.0	208.0	0.044	
8	7890.3	513.0	321.0	0.291	83.0	119.0	0.030	

3. Magnified Moment

KL_u/r_x = 15000/375 = 40.00 > 34-12(M_1/M_2) = 22.00
δ_x = MAX[1.00/(1-P_u/0.75/52256), 1.0] = 1.277

KL_u/r_y = 15000/375 = 40.00 > 34-12(M_1/M_2) = 22.00
δ_y = MAX[1.00/(1-P_u/0.75/52256), 1.0] = 1.277

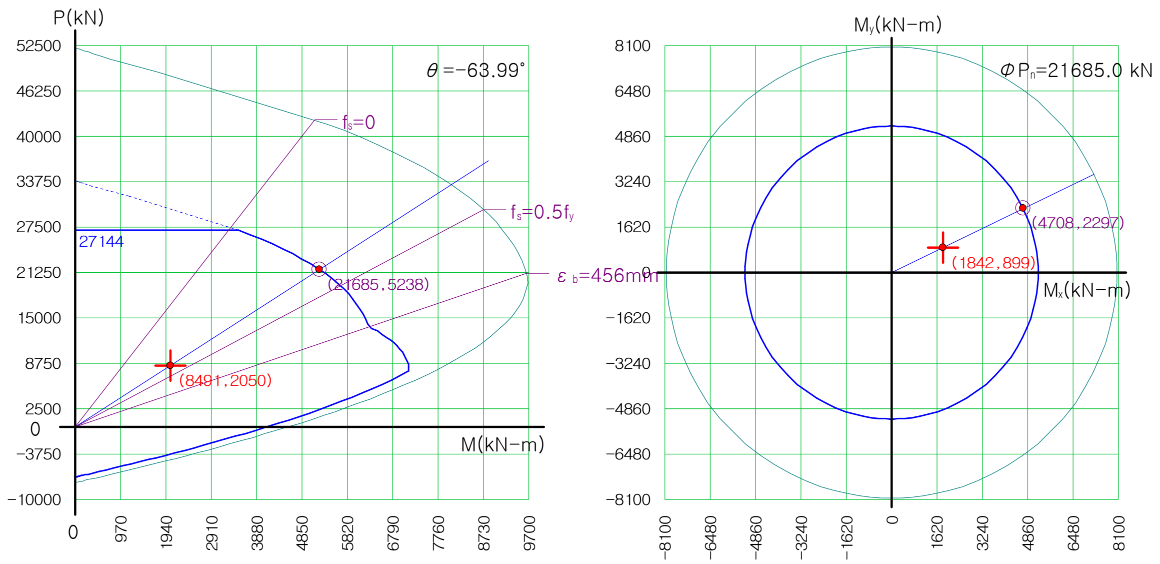
4. Design Force and Moment

Design Load Combination No : 6
P_u = 8490.8 kN
M_{ux} = 1443.0, M_{uy} = 704.0 kN-m
$\delta_x M_{ux}$= δ_x*M_{ux} = 1842.1 kN-m
$\delta_y M_{uy}$= δ_y*M_{uy}, = 898.7 kN-m

5. Check Axial and Moment Capacity

Rotation Angle and Depth to the Neutral Axis θ = -63.99°, c = 1154 mm
Strength Reduction Factor Φ = 0.6500
Maximum Axial Load $\Phi P_{n(max)}$ = 27144.3 kN
Design Axial Load Strength ΦP_n = 21685.0 kN
Design Moment Strength ΦM_{nx} = 4707.5 kN-m
ΦM_{ny} = 2296.7 kN-m
Strength Ratio : Applied/Design = 0.391 < 1.000 .... O.K

	Company	Nipponkoei	Project Name	DN Qn Expressway
	Designer	vinh	File Name	D:\...\Pile D1.5m - Pier P11.BOI



6. Check Shear Capacity

Design Load Combination No : 6

Strength Reduction Factor Φ= 0.750

Design Force Vu = 375.2 kN (Pu = 8490.8 kN)

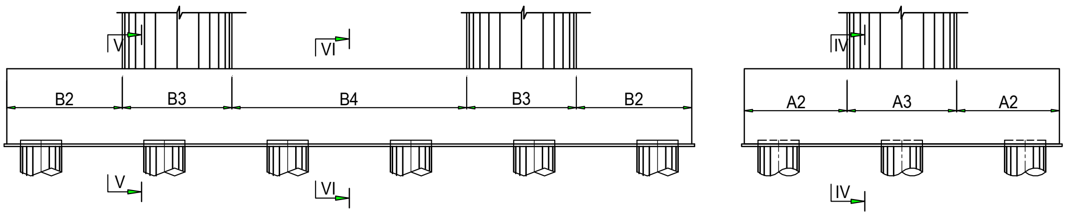
Required Hoop Spacing : D16 @ 508 mm

Provided Hoop Spacing : D16 @ 150 mm (Tie)

$\Phi V_c + \Phi V_s$ = 3865.5 + 924.5 = 4790.1 kN > V_u = 375.2 kN .... O.K

2.6.4. CHECK FOR PILE CAP

2.6.5.1. The Force to section IV-IV, section V-V, section VI-VI



At section IV - IV

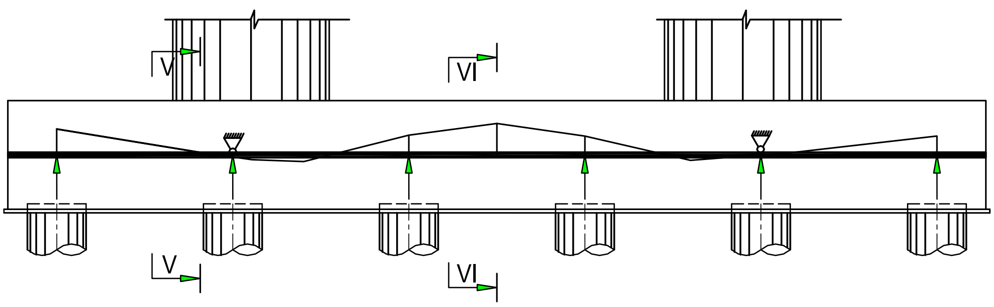
COMBINATION	Longitudinal direction		
	Q (kN)	M(kN.m)	V (kN.m)
Strength I	46999.4	148847.2	74.5
Strength II	42055.4	133189.6	74.5
Strength III	45869.4	145268.5	74.5
Service I	36328.8	115053.2	74.5
Service III	11477.2	36348.4	74.5
Extreme I (CV)	47166.5	149376.3	1659.0
Extreme II (EQ trans)	43351.1	137292.9	74.5
Extreme II (EQ long)	44963.3	142398.7	744.00

At section V - V

COMBINATION	Transverse direction		
	Q (kN)	M(kN.m)	V (kN.m)
Strength I	23859.0	86631.9	-47.3
Strength II	23070.4	83768.6	-342.7
Strength III	23985.3	87090.6	-159.9
Service I	19218.1	69781.0	-168.3
Service III	19229.0	69820.4	-168.6
Extreme I (CV)	15394.4	55896.9	-654.3
Extreme II (EQ trans)	24240.37	88016.8	-416.11
Extreme II (EQ long)	22845.85	82953.3	-238.78

At section VI - VI

COMBINATION	Transverse direction		
	Q (kN)	M(kN.m)	V (kN.m)
Strength I	27852.0	31562.0	-47.3
Strength II	25156.0	29521.0	-342.7
Strength III	26152.0	23125.0	-159.9
Service I	22352.0	24353.0	-168.3
Service III	21325.0	24215.0	-168.6
Extreme I (CV)	15263.0	19251.0	-654.3
Extreme II (EQ trans)	27562.00	30214.00	-416.1
Extreme II (EQ long)	34265.00	30125.00	-238.8



2.6.5.2. Ultimate load check, shear capacity check and crack control

*** Check flexure momen capacity of pile cap:**

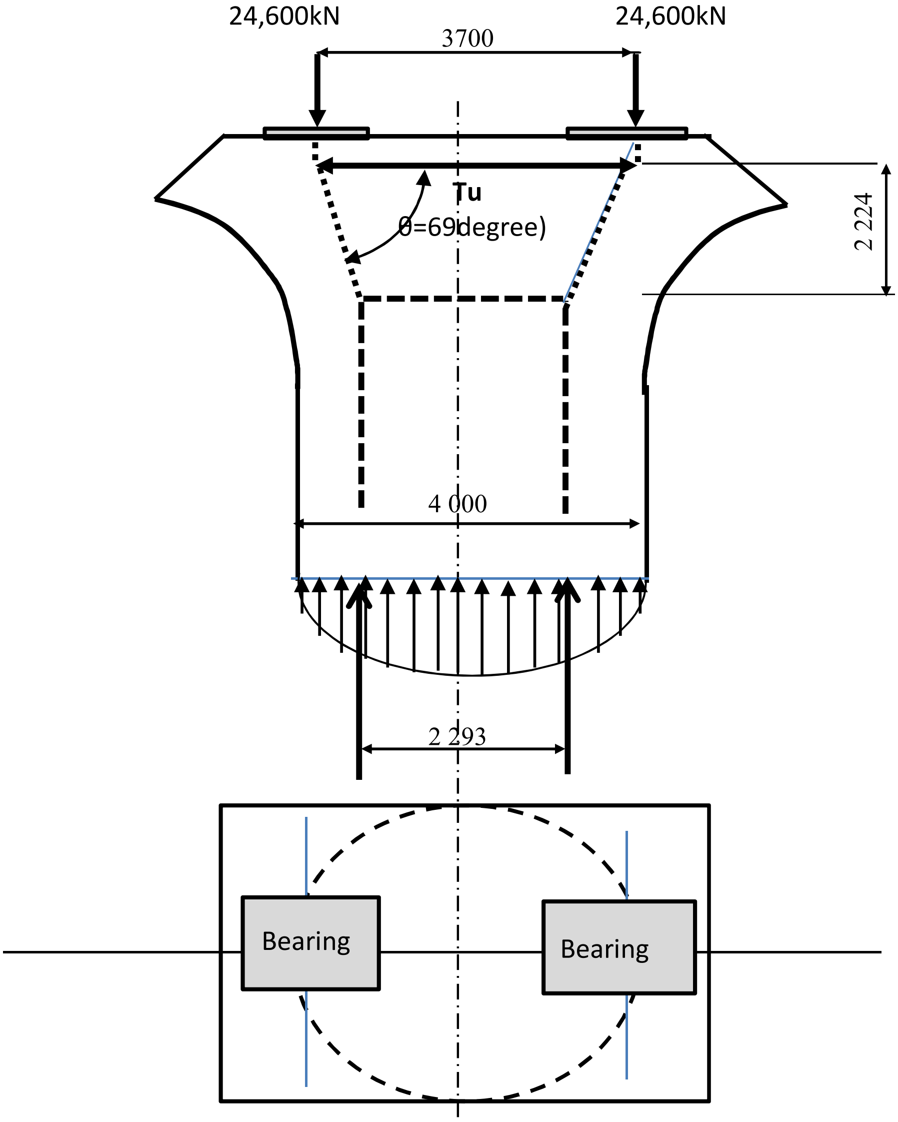
Item			Section IV-IV (Bottom bar)	Section V-V (Bottom bar)	Section VI-VI (Upper bar)	Unit
• Factored Plexural moment		M_u	148847.21	87090.62	31562.00	kN.m
• Factored Shear force		V_u	46999.44	23985.30	27852.00	kN
• Hight of Section		h	3000	3000	3000	mm
• Width of section		b	25000	11500	11500	mm
• Section area		A_c	75000000	34500000	34500000	mm^2
• Moment of inertia of concrete section		I_g	5.6E+13	2.6E+13	2.6E+13	mm^4
• Tension reinforcement:	Distance from tension reinf. to extreme compression fiber	d_c	228	260	146	mm
	Reinf. Diameter	∅	D28	D32	D36	mm
	Space	@	150	150	150	mm
	Number of bar	n	330	150	75	bar
	Total area of reinf.	A_s	203445	120959	76680	mm^2
• comp. reinforcement:	Distance from compressive reinf. to extreme Tension fiber		100	146	152	mm
	Diameter		D28	D36	D32	mm
	Reinf. Space		150	150	150	mm
	Number of bar		165	75	150	bar
	Total area of reinf.	A'_s	101804	76680	120959	mm^2
Check Flexural Moment at Strength state						
• Resistance factor		Φ	0.90	0.90	0.90	
• The corresponding effective		d_e	2772	2740	2854	mm
• Stress block factor		β_1	0.84	0.84	0.84	
• Depth of the equivalent stress block = $c*\beta 1$		a	127.65	164.99	104.59	mm
• Distance from extreme compression fiber to the neutral axis		c	152.75	197.42	125.15	mm
• The nominal flexural resistance:		M_n	220385	128579	85934	kN.m
• Factored flexural resistance		$M_r = \Phi.M_n$	198347	115722	77340	kN.m
• Check condition		$M_r > M_u$	**O.K**	**O.K**	**O.K**	
Mimimum Reinforcement						
• Ratio of tension steel to gross area		$\rho = A_s/(b.d)$	0.29	0.38	0.23	%
• Check		$\rho > 0.03•f'_c/f'_y$	**O.K**	**O.K**	**O.K**	0.23
• Cracking moment		1.2Mcr	155279.35	71428.50	71428.50	Kn.m
• Check		Mr> min(1.2Mcr, 1.33Mu)	**O.K**	**O.K**	**O.K**	
Maximum Reinforcement						
• Obligation Condition		c/d_e	0.06	0.07	0.04	
• Check		$c/d_e < 0.42$	**O.K**	**O.K**	**O.K**	
Check shear resistance						
• Factored Shear force		V_u	46999.44	23985.30	27852.00	kN
• Resistance factor		Φ	0.90	0.90	0.90	
• The effective shear deepth		d_v	2708	2658	2802	mm
• Effective width		b_v	25000	11500	11500	mm
• Angle of inclination of diagonal compressive stress		θ	43	43	43	degree
• Angle of inclination of transerverse reinf. To longitudinal axis		α	90	90	90	degree
• Factor indicating ability of diagonally cracked concrete to transmit tension		β	1.95	1.95	1.95	
• Value		$0.1*f'_c•b_v•d_v$	203113	91684	96659	kN
• Max spacing of transverse reinforcement		s_{max}	600	600	600	mm
• Spacing of stirrup		s	450	450	450	mm
• Diameter of transverse reinforcement		∅	D 28	D 32	D 36	
• Number of transverse reinf. within distance s		n	6	6	3	bar
• Total area of transverse reinf.		A_v	3695	4825	3054	mm^2
• Diameter of stirrup		∅	D 20	D 20	D 20	mm
• Number of stirrup within distance s		n	57	27	27	bar
• Total area of stirrup		A_v	17767.45	8342.67	8342.67	
• Assume		θ	43.00	43.00	43.00	degree
• Strain in tensile reinforcement		ex	1.97E-03	1.89E-03	1.71E-03	
If ex<0, multiple with reduce factor		Fc	-	-	-	
• Ratio of shear stress and f'c		V/f'c	0.03	0.03	0.03	
• β final			1.95	1.95	1.95	
• θ final			43.00	43.00	43.00	

• The shear resistance of concrete:	V_c	60019.16	27092.26	28562.31	kN
• The shear resistance of stirrup	V_s	21839.74	10062.94	10608.97	kN
• Value	$0.25*f'_c \cdot b_v \cdot d_v$	507782.68	229209.77	241646.91	kN
• The nominal shear resistance:	V_n	81858.90	37155.20	39171.28	kN
• The factored shear resistance	V_r	73673.01	33439.68	35254.15	kN
• Check	$V_r > V_u$	**O.K**	**O.K**	**O.K**	
•Requiring transverse reinforcement	$V_u > 0.5 \cdot \Phi \cdot V_c$	**Need**	**Need**	**Need**	
•Check minimum transverse reinforcement	$A_v > 0.083 \cdot (f'_c{}^{0.5}) \cdot b_v \cdot s/f_y$	**O.K**	**O.K**	**O.K**	
Check Flexural and shear resistance at Extreme state					
• Factored Plexural moment	M_u	149376.30	88016.79	30214.00	kN.m
• Factored Shear force	V_u	47166.50	24240.37	34265.00	kN
• Resistance factor	Φ	1.00	1.00	1.00	
• The nominal flexural resistance:	M_n	220385	128579	85934	kN.m
• Factored flexural resistance	$M_r = \Phi.M_n$	220385	128579	85934	kN.m
• The nominal flexural resistance:	V_n	81859	37155	39171	Kn
• Factored flexural resistance	$V_r = \Phi.V_n$	81859	37155	39171	Kn
• Check condition	$M_r > M_u$	**O.K**	**O.K**	**O.K**	
	$V_r > V_u$	**O.K**	**O.K**	**O.K**	
Check crack					
Interior force combination	**Service I**				
• Factored moment	M_u	1.15E+05	6.98E+04	2.44E+04	kN.m
• Modulus of rupture of concrete	fr = 0.63*sqrt(f'c)	3.45	3.45	3.45	MPa
• Distance from extreme tension fiber to the neutral axis	$y_t = h-c$	2847	2803	2875	mm
• Stress of concrete at tension fiber	$f'r = Ms*y_t/I_g$	5.82	7.56	2.71	MPa
• Check	f'r >	0.8*fr	0.8*fr	0.8*fr	
		check crack	check crack	No check	
• Crack width parameter	Z	= 23000	= 23000	= 23000	N/mm
• Ratio of reinf. Modulus with concrete modulus	$n = E_s/E_c$	= 7.00	= 7.00	= 7.00	
• The distance from extreme fiber to the neutral axis	c	= 621.81	= 713.08	= 564.94	mm
• Effective moment of inertia	J	8.59E+12	4.87E+12	3.50E+12	mm^4
• Arm	de-c	= 2150.19	= 2026.92	= 2289.06	mm
• Tension stress in reinforcement	fs =n*Ms*(de-c)/J	= 201.65	= 203.48	= 111.37	MPa
• Area of concrete having the same centroid as the principal tensile reinforcement divided by number of bars	A	= 15133	= 15293	= 15265	mm2
• Tension stress in reinforcement with service state	$f_{sa} = Z/(d_c*A)^{1/3}$	= 232.47	= 229.29	= 227.15	Mpa
• Check condition	$f_s < f_{sa}$	**O.K**	**O.K**	**O.K**	
• Check condition	$f_s < 0.6*f_y$	**O.K**	**O.K**	**O.K**	

Calculation of Pier Head

1. Reaction

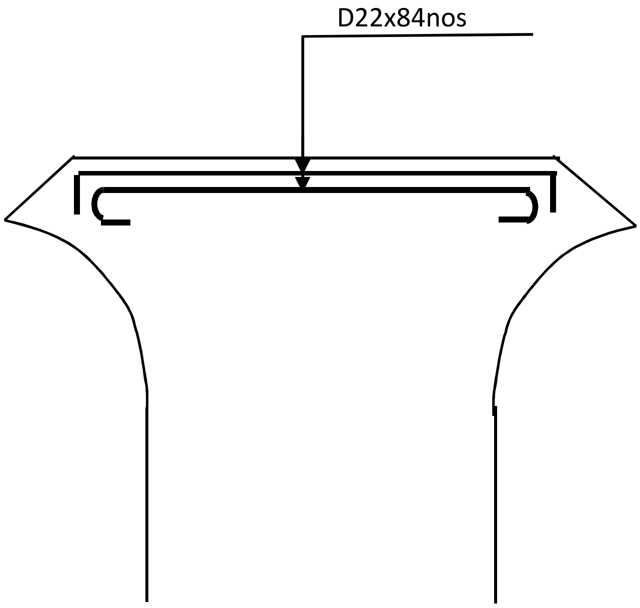
Item	Sumbol	Unit	R	γ
Dead Load	Dc	kN	14870	1.25
Super-Imposed Load	Dw	kN	1090	1.5
Live Load &Impact	L+Im	kN	2500	1.75
At Strength	Ru	kN	24600	

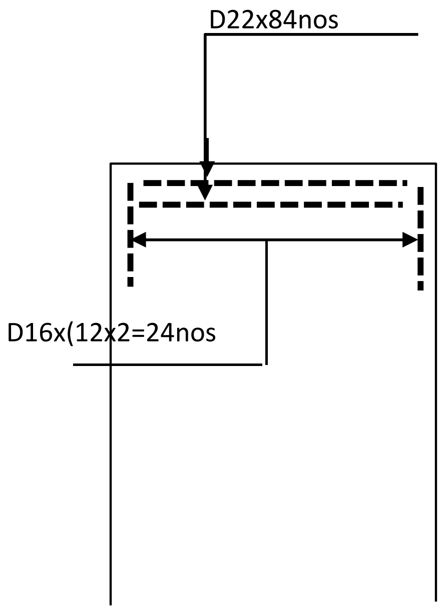


Check for Tennsion at Pier Head

Item		Symbol	Unit	Value
Reaction		Ru	kN	24600
Tension		Tu	kN	9 443
Yield Point of Rebar		fry	Mpa	400
resitance Factor		ϕ	-	0.7
Requred Rebar		reqAr		33 725
Actural Rebar	Diamiter	Dr1	mm	22
	Number Rebar	Nr1	nos	84
	Total Area	Ar1	mm2	31 931
	Diamiter	Dr2	mm	16
	Number Rebar	Nr2	nos	24
	Total Area	Ar2	mm2	4 825
	Total Area	Ar	mm2	36 757
Nominal Resitance		Trn	kN	14 703
Factored Resistance		Tr	kN	10 292
Jadgement(Tr≧Tu ----OK)		-	-	OK

Arrangement of Reinforcement





2.7 PIER P12
2.7 TRỤ CẦU P12

PIER P12 - CALCULATION SHEET

CONTENT:

2.7.1. GENERAL DATA

CALCULATION PROCEDURE & STANDARD:

- Bridge Design Standard 22 TCN - 272 - 05

2.7.1.1. Design live load

Design vehicle load		HL93	22TCN 272 - 05
Number of lane		6	(lane)
Pedestrian		0.00	KG/m^2

2.7.1.2. Bridge width

Width carriageway	B =	12.00	(m)
Width of median guardrail	B_{pc} =	0.50	(m)
Width parapet	B_{lc} =	0.50	(m)
Bridge width	B =	13.00	(m)

2.7.1.3. Superstructure:

Span-arrangement		Continuous box girder 65+5@100+65m	
Height of box girder at pier section	H =	6.00	(m)
Height of box girder at mid-span section	h =	2.50	(m)
Pavement thickness	d_{BTN} =	0.084	(m)

2.7.1.4. Material property:

Concrete

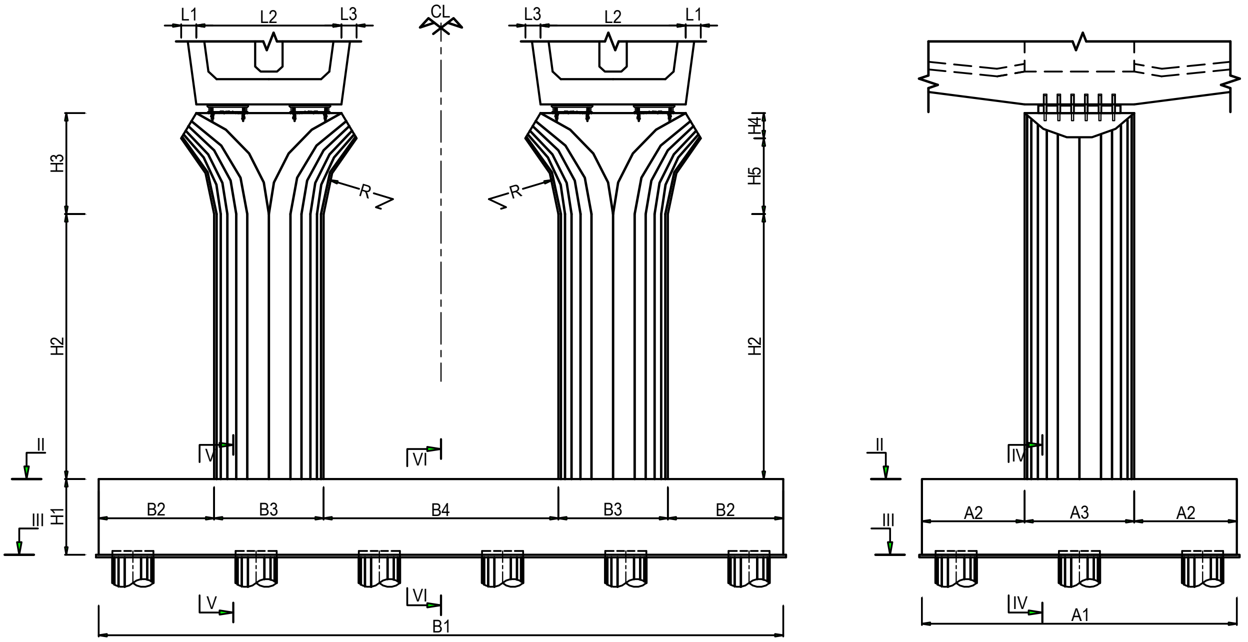
Compressive strength of cylindrical at 28 day	f 'c =	30.00	MPa
Concrete density	g =	24.50	KN/m^3
Elastic modulus	Ec =	29440	MPa
Tension strength of concrete	fr =	3.45	MPa

Steel

Steel modulus	Es =	200000	MPa
Yeild strength of steel bar	fy =	400.00	MPa
Calculation unit:		KN, m, KN.m, Mpa, KN/m^2	

2.7.1.5. THE PIER GEOMETRIC

GENERAL DIMENSIONS



vertical			Horizontal			Thickness		
Remark	Value	Unit	Remark	Value	Unit	Remark	Value	Unit
a_1 =	11.50	(m)	L_1 =	0.55	(m)	h_1 =	3.00	(m)
a_2 =	3.75	(m)	L_2 =	5.30	(m)	h_2 =	9.000	(m)
a_3 =	4.00	(m)	L_3 =	0.55	(m)	h_3 =	4.000	(m)
			B_1 =	25.00	(m)	h_4 =	1.000	(m)
			B_2 =	3.965	(m)	h_5 =	3.000	(m)
			B_3 =	4.00	(m)			
			B_4 =	9.070	(m)			
			R =	4.34	(m)			

The design elevation:

Proposed height	EL_{mc} =	19.390	(m)
Elevation of top of pier cap	EL_{xm} =	13.185	(m)
Hight water level (H1%)	EL_{MNTK} =	9.200	(m)
Daily water level (H5%)	EL_{MNTB} =	4.070	(m)
Ground elevation	EL_{TN} =	2.240	(m)
Daily water level (H5%)	EL_{TT} =	4.070	(m)
Top of Pile cap (Section II - II)	$EL_{MC\ II\text{-}II}$ =	-0.215	(m)
Bottom of Pile cap (Section III - III)	$EL_{MCIII\text{-}III}$ =	-3.215	(m)

2.7.2. CALCULATED PIER FORCE AND PILE FORCE

2.7.2.1. Loading combination at bottom of pier shaft

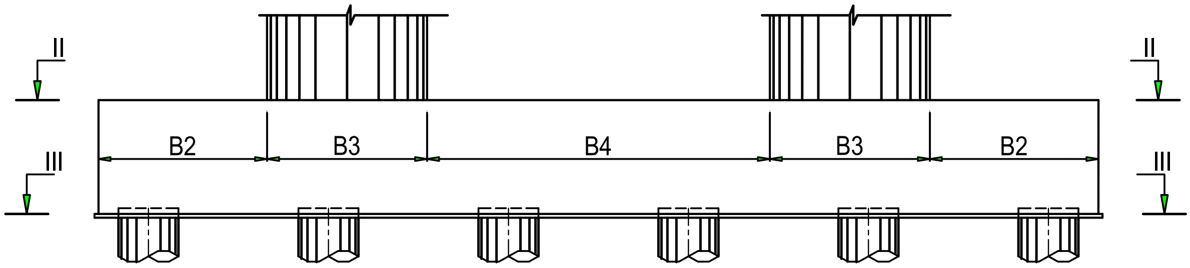
(Result from RM2010 SPACE-FRAME program)

Loading combination in accordance with 22 TCN 272-05, table 3.4.1.1 was mentioned in appendix of this report

LOADING COMBINATION AT TOP OF PILE CAP DUE TO SUPERSTRUCTURE + VESSEL COLLISION AND SEFTWEIGHT						
Limited state	Vertical axis	Longitudinal axis		Horizontal axis		Remarks
	N_x (kN)	H_y (kN)	M_z (kNm)	H_z (kN)	M_y (kNm)	
STRENGTH I	53296.00	53.00	231.00	186.00	7680.00	
STRENGTH II	45732.00	53.00	231.00	1879.00	32716.00	
STRENGTH III	51567.00	53.00	231.00	861.00	18415.00	
SERVICE I	40556.00	53.00	231.00	705.00	14934.00	
SERVICE III	40570.00	53.00	231.00	706.00	14936.00	
EXTREME EVENT (CV)	47766.00	3297.00	13807.00	6025.00	19750.00	
EXTREME EVENT (EQ trans)	47831.00	53.00	231.00	2413.00	41896.00	
EXTREME EVENT (EQ long)	48196.00	531.00	2309.00	1132.00	16215.00	

2.7.2.2. Loading combination at bottom of pile cap

LOADING COMBINATION AT TOP OF PILE CAP DUE TO SUPERSTRUCTURE + VESSEL						
Limited state	Vertical axis	Longitudinal axis		Horizontal axis		Remarks
	N_x (kN)	H_y (kN)	M_z (kNm)	H_z (kN)	M_y (kNm)	
STRENGTH I	144342	143	835	398	15509	
STRENGTH II	129214	143	835	3732	77214	
STRENGTH III	140884	143	835	1749	41262	
SERVICE I	111512	143	835	1437	33565	
SERVICE III	111540	143	835	1437	33570	
EXTREME EVENT (CV)	133283	3315	23726	6038	37698	
EXTREME EVENT (EQ trans)	133412	143	835	4852	98022	
EXTREME EVENT (EQ long)	134143	1427	8350	2522	39322	



2.7.2.3. Loading combination at top of piling

2.7.2.3.1. Piling material:

Concrete 30 Mpa

E_c (Mpa)	294401
γ_c (KN/m^3)	24.5

Steel bar

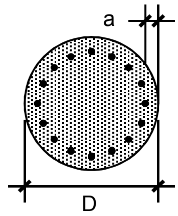
Type	CB-400-T
E_s (Mpa)	200000

2.7.2.3.2. Piling dimension

+ Diameter **D** = 1.50 m

a = 0.100 m

+ Length **L** = 64.00 m

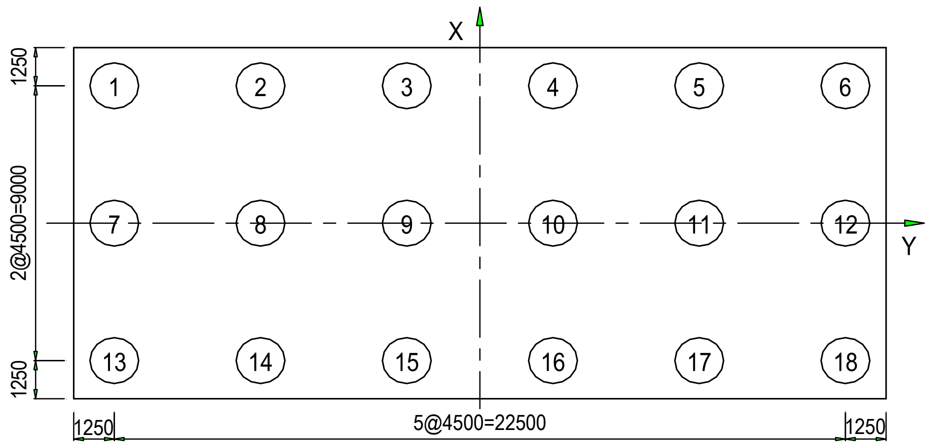


2.7.2.3.3. Maximum Internal force and displacement at top piling

Maximum Internal force and diplacements *(Result follow Piling software)*

Combination	N	Hx	My	x	y	z
	(KN)	(KN)	(KN.m)	(m)	(m)	(rad)
Strength I	8226.48	7.94	32.29	-	-	-
Strength II	8193.20	7.94	32.29	-	-	-
Strength III	8369.25	7.94	32.29	-	-	-
Service I	6641.90	7.94	32.29	**0.000**	**-0.002**	**0.008**
Service III	6643.50	7.94	32.29	**0.000**	**-0.002**	**0.008**
Extreme I (CV)	8778.26	184.17	733.51	-	-	-
Extreme II (EQ trans)	8698.40	7.94	32.29	-	-	-
Extreme II (EQ long)	8248.30	79.28	322.13	-	-	-

- Check displacement of top pile not exceed 38mm (10.7.2.7) **OK**



Arrangement of pile

2.7.2.3.4. Internal force for each pile

Combination	Pile Number	Hx (KN)	Hy (KN)	N (KN.m)	Mx (m)	My (m)
Strength I	1	7.94	-22.11	8174.03	89.96	32.29
	2	7.94	-22.11	8101.53	89.96	32.29
	3	7.94	-22.11	8029.03	89.96	32.29
	4	7.94	-22.11	7956.53	89.96	32.29
	5	7.94	-22.11	7884.02	89.96	32.29
	6	7.94	-22.11	7811.52	89.96	32.29
	7	7.94	-22.11	8200.25	89.96	32.29
	8	7.94	-22.11	8127.75	89.96	32.29
	9	7.94	-22.11	8055.25	89.96	32.29
	10	7.94	-22.11	7982.75	89.96	32.29
	11	7.94	-22.11	7910.25	89.96	32.29
	12	7.94	-22.11	7837.75	89.96	32.29
	13	7.94	-22.11	8226.48	89.96	32.29
	14	7.94	-22.11	8153.98	89.96	32.29
	15	7.94	-22.11	8081.48	89.96	32.29
	16	7.94	-22.11	8008.98	89.96	32.29
	17	7.94	-22.11	7936.48	89.96	32.29
	18	7.94	-22.11	7863.97	89.96	32.29
Strength II	1	7.94	-207.33	8140.75	899.53	32.29
	2	7.94	-207.33	7745.38	899.53	32.29
	3	7.94	-207.33	7350.01	899.53	32.29
	4	7.94	-207.33	6954.65	899.53	32.29
	5	7.94	-207.33	6559.28	899.53	32.29
	6	7.94	-207.33	6163.91	899.53	32.29
	7	7.94	-207.33	8166.97	899.53	32.29
	8	7.94	-207.33	7771.61	899.53	32.29
	9	7.94	-207.33	7376.24	899.53	32.29
	10	7.94	-207.33	6980.87	899.53	32.29
	11	7.94	-207.33	6585.50	899.53	32.29
	12	7.94	-207.33	6190.14	899.53	32.29
	13	7.94	-207.33	8193.20	899.53	32.29
	14	7.94	-207.33	7797.83	899.53	32.29
	15	7.94	-207.33	7402.46	899.53	32.29
	16	7.94	-207.33	7007.10	899.53	32.29
	17	7.94	-207.33	6611.73	899.53	32.29
	18	7.94	-207.33	6216.36	899.53	32.29
Strength III	1	7.94	-97.17	8316.80	417.40	32.29
	2	7.94	-97.17	8110.35	417.40	32.29
	3	7.94	-97.17	7903.89	417.40	32.29
	4	7.94	-97.17	7697.44	417.40	32.29
	5	7.94	-97.17	7490.98	417.40	32.29
	6	7.94	-97.17	7284.53	417.40	32.29
	7	7.94	-97.17	8343.03	417.40	32.29
	8	7.94	-97.17	8136.57	417.40	32.29
	9	7.94	-97.17	7930.12	417.40	32.29
	10	7.94	-97.17	7723.66	417.40	32.29
	11	7.94	-97.17	7517.21	417.40	32.29
	12	7.94	-97.17	7310.75	417.40	32.29
	13	7.94	-97.17	8369.25	417.40	32.29
	14	7.94	-97.17	8162.80	417.40	32.29
	15	7.94	-97.17	7956.34	417.40	32.29
	16	7.94	-97.17	7749.89	417.40	32.29
	17	7.94	-97.17	7543.43	417.40	32.29
	18	7.94	-97.17	7336.98	417.40	32.29

- Result for internal force at top pile

Combination	Pile Number	**Hx** (KN)	**Hy** (KN)	**N** (KN.m)	**Mx** (m)	**My** (m)
Service I	1	7.94	-79.83	6589.45	343.21	32.29
	2	7.94	-79.83	6421.22	343.21	32.29
	3	7.94	-79.83	6253.00	343.21	32.29
	4	7.94	-79.83	6084.77	343.21	32.29
	5	7.94	-79.83	5916.55	343.21	32.29
	6	7.94	-79.83	5748.33	343.21	32.29
	7	7.94	-79.83	6615.67	343.21	32.29
	8	7.94	-79.83	6447.45	343.21	32.29
	9	7.94	-79.83	6279.22	343.21	32.29
	10	7.94	-79.83	6111.00	343.21	32.29
	11	7.94	-79.83	5942.78	343.21	32.29
	12	7.94	-79.83	5774.55	343.21	32.29
	13	7.94	-79.83	6641.90	343.21	32.29
	14	7.94	-79.83	6473.67	343.21	32.29
	15	7.94	-79.83	6305.45	343.21	32.29
	16	7.94	-79.83	6137.23	343.21	32.29
	17	7.94	-79.83	5969.00	343.21	32.29
	18	7.94	-79.83	5800.78	343.21	32.29
Service III	1	7.94	-79.83	6591.05	343.21	32.29
	2	7.94	-79.83	6422.81	343.21	32.29
	3	7.94	-79.83	6254.56	343.21	32.29
	4	7.94	-79.83	6086.32	343.21	32.29
	5	7.94	-79.83	5918.08	343.21	32.29
	6	7.94	-79.83	5749.83	343.21	32.29
	7	7.94	-79.83	6617.28	343.21	32.29
	8	7.94	-79.83	6449.03	343.21	32.29
	9	7.94	-79.83	6280.79	343.21	32.29
	10	7.94	-79.83	6112.54	343.21	32.29
	11	7.94	-79.83	5944.30	343.21	32.29
	12	7.94	-79.83	5776.06	343.21	32.29
	13	7.94	-79.83	6643.50	343.21	32.29
	14	7.94	-79.83	6475.26	343.21	32.29
	15	7.94	-79.83	6307.01	343.21	32.29
	16	7.94	-79.83	6138.77	343.21	32.29
	17	7.94	-79.83	5970.53	343.21	32.29
	18	7.94	-79.83	5802.28	343.21	32.29
Extreme Event I (with CV)	1	184.17	-335.44	7410.51	1526.98	733.51
	2	184.17	-335.44	7134.60	1526.98	733.51
	3	184.17	-335.44	6858.69	1526.98	733.51
	4	184.17	-335.44	6582.78	1526.98	733.51
	5	184.17	-335.44	6306.87	1526.98	733.51
	6	184.17	-335.44	6030.96	1526.98	733.51
	7	184.17	-335.44	8094.39	1526.98	733.51
	8	184.17	-335.44	7818.48	1526.98	733.51
	9	184.17	-335.44	7542.57	1526.98	733.51
	10	184.17	-335.44	7266.66	1526.98	733.51
	11	184.17	-335.44	6990.75	1526.98	733.51
	12	184.17	-335.44	6714.84	1526.98	733.51
	13	184.17	-335.44	8778.26	1526.98	733.51
	14	184.17	-335.44	8502.35	1526.98	733.51
	15	184.17	-335.44	8226.44	1526.98	733.51
	16	184.17	-335.44	7950.53	1526.98	733.51
	17	184.17	-335.44	7674.62	1526.98	733.51
	18	184.17	-335.44	7398.71	1526.98	733.51

- Result for internal force at top pile

Combination	Pile Number	**Hx** (KN)	**Hy** (KN)	**N** (KN.m)	**Mx** (m)	**My** (m)
Extreme Event I (with EQtran)	1	7.94	-269.56	8645.95	1171.43	32.29
	2	7.94	-269.56	8141.79	1171.43	32.29
	3	7.94	-269.56	7637.63	1171.43	32.29
	4	7.94	-269.56	7133.47	1171.43	32.29
	5	7.94	-269.56	6629.31	1171.43	32.29
	6	7.94	-269.56	6125.15	1171.43	32.29
	7	7.94	-269.56	8672.18	1171.43	32.29
	8	7.94	-269.56	8168.02	1171.43	32.29
	9	7.94	-269.56	7663.86	1171.43	32.29
	10	7.94	-269.56	7159.70	1171.43	32.29
	11	7.94	-269.56	6655.54	1171.43	32.29
	12	7.94	-269.56	6151.38	1171.43	32.29
	13	7.94	-269.56	8698.40	1171.43	32.29
	14	7.94	-269.56	8194.24	1171.43	32.29
	15	7.94	-269.56	7690.08	1171.43	32.29
	16	7.94	-269.56	7185.92	1171.43	32.29
	17	7.94	-269.56	6681.76	1171.43	32.29
	18	7.94	-269.56	6177.60	1171.43	32.29
Extreme Event I (with EQlong)	1	79.28	-140.11	7724.29	618.44	322.13
	2	79.28	-140.11	7510.73	618.44	322.13
	3	79.28	-140.11	7297.16	618.44	322.13
	4	79.28	-140.11	7083.60	618.44	322.13
	5	79.28	-140.11	6870.04	618.44	322.13
	6	79.28	-140.11	6656.48	618.44	322.13
	7	79.28	-140.11	7986.29	618.44	322.13
	8	79.28	-140.11	7772.73	618.44	322.13
	9	79.28	-140.11	7559.17	618.44	322.13
	10	79.28	-140.11	7345.61	618.44	322.13
	11	79.28	-140.11	7132.05	618.44	322.13
	12	79.28	-140.11	6918.49	618.44	322.13
	13	79.28	-140.11	8248.30	618.44	322.13
	14	79.28	-140.11	8034.74	618.44	322.13
	15	79.28	-140.11	7821.18	618.44	322.13
	16	79.28	-140.11	7607.61	618.44	322.13
	17	79.28	-140.11	7394.05	618.44	322.13
	18	79.28	-140.11	7180.49	618.44	322.13

2.7.3. CHECK FOR PIER SHAFT

Item		Mark	Unit	Value
• Factored Axial force		Nu	Kn	40570.00
• Factored Plexural moment		Mux	Kn.m	231.00
• Factored Plexural moment		Muy	Kn.m	14936.00
• Diameter of Pier shaft		D	m	4.00
• Section area		Ag	m2	12.57
• Moment of inertia of concrete section		Ic	m4	12.57
• Cover thickness		a	m	0.075
• Reinf. Diameter		Ds	mm	32.00
• Number of rebar		n_s	nos	168.00
• Rebar area		As	mm2	135113.62
Check minimum reinforcement				
• Minimum rebar area required (0.135*f'c/fy)*Ag		As req	mm2	127234.50
• Check condition As > (0.135*f'c/fy)*Ag				**OK**
Check maximum reinforcement				
• Maximum rebar area 0.08*Ag		As max	mm2	1005309.6
• Check condition As < 0.08*Ag				**OK**
Check ratio spiral or Tier (5.7.4.6)				
• Distance to outside of Spairal or Ties to concrete face			mm	66.00
• Effect diamete		Deff	m	3.87
• Area of core measured to the outside diameter of the spiral			m2	11.75
• Ratio spiral Rebar required		ρsa		0.00234
Required Area of Spiral Rebar	space		mm	200
	Effective length			3.87
	layer			2
	Area			226.6
	Requaired Dhs			17.0
Actuaral	Effective length	d	m	3.868
	Diameter	Dhr	mm	18
	Area of Rebar	Ah	mm2	254.5
	layer	Nl	nos	2
	Total area of spiral	Ac	m2	508.938
	space	s	mm	200
	Ratio spiral Rebar	ρs	-	0.0026315
• Check condition		$\rho_s > \rho_{sa}$		OK
Check Crack (At Service state)				
• Modulus of rupture of concrete		fr = 0.63*sqrt(f'c)	Mpa	3.45
• Stress of concrete at tension fiber		σ'_r	Mpa	2
• If f'r > 0.8fr require check crack		$\sigma'_r > 0.8*\sigma_r$	Mpa	No check
• Center of newtral axial		x	mm	3.12517
• Maximum stress of Compression fiber of concrete		σ_c	Mpa	8.7
• Maximum stress of Compression Rebar		σ_{rc}	Mpa	-126.8
• Maximum stress of Tension Rebar		σ_{rt}	Mpa	32.4
• Check			σ_{rt} <0.6.fy	OK

MIDAS/Set **Column Design [Pier Shaft D4.0m-P12]**

Company	Nipponkoei	**Project Name**	DN Qn Expressway
Designer	vinh	**File Name**	D:\...\bodypier-D4.0m-P12.BOI

1. Geometry and Materials

Design Code : ACI318M-02

Stress Profile : Equivalent Stress Block

Material Data : f_c' = 30 MPa (β_1 = 0.850)

f_y = 400, f_{ys} = 400 MPa

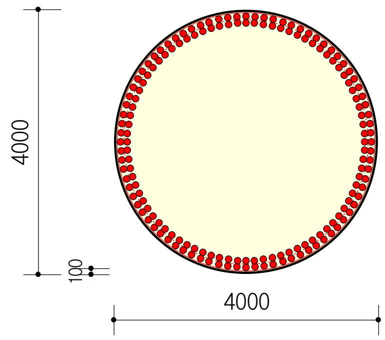
Section Dim. : Φ4000 mm

Effective Len. : KL_u = 14500 mm

Steel Distribut.: 84 - D32 (d_c = 100 mm)

: 84 - D32 (d_c = 200 mm)

Total Steel Area A_{st} = 131896 mm^2 (ρ_{st} = 0.0106)



2. Member Force and Moment

Unit : kN, kN-m

L.C.	P_u	M_{ux}	M_{uy}	R_{atioV}	V_{ux}	V_{uy}	R_{atioH}	Remark
1	53296.1	230.9	7680.3	0.277	53.1	185.9	0.006	
2	45732.1	230.8	32715.8	0.304	53.1	1878.8	0.061	
3	51567.2	230.8	18415.1	0.268	53.1	861.4	0.028	
4	47766.4	13807.0	19749.5	0.277	3297.1	6024.7	0.224	
5	47830.9	230.8	41895.6	0.424	53.1	2413.2	0.079	
6	48196.3	2308.5	16215.5	0.250	530.7	1131.9	0.041	
7	40556.2	230.8	14934.5	0.213	53.1	705.4	0.023	
8	40570.2	230.8	14936.5	0.211	53.1	705.5	0.023	

3. Magnified Moment

KL_u/r_x = 14500/1000 = 14.50 < 34-12(M_1/M_2) = 22.00

δ_x = 1.000

KL_u/r_y = 14500/1000 = 14.50 < 34-12(M_1/M_2) = 22.00

δ_y = 1.000

4. Design Force and Moment

Design Load Combination No : 5

P_u = 47830.9 kN

M_{ux} = 230.8, M_{uy} = 41895.6 kN-m

5. Check Axial and Moment Capacity

Rotation Angle and Depth to the Neutral Axis θ = -0.32°, c = 2435 mm

Strength Reduction Factor Φ = 0.6500

Maximum Axial Load $\Phi P_{n(max)}$ = 192613.4 kN

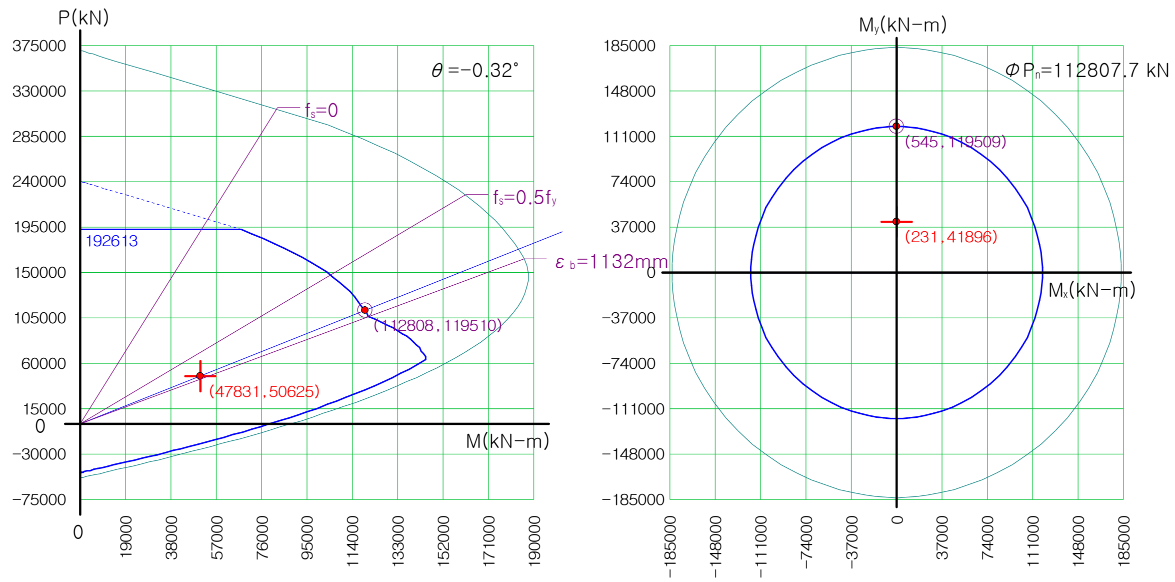
Design Axial Load Strength ΦP_n = 112807.7 kN

Design Moment Strength ΦM_{nx} = 545.0 kN-m

ΦM_{ny} = 119509.4 kN-m

Strength Ratio : Applied/Design = 0.424 < 1.000 .... O.K

	Company	Nipponkoei	Project Name	DN Qn Expressway
	Designer	vinh	File Name	D:\...\bodypier-D4.0m-P12.BOI



6. Check Shear Capacity

Design Load Combination No : 4

Strength Reduction Factor Φ= 0.750

Design Force Vu = 6867.8 kN (Pu = 47766.4 kN)

Required Hoop Spacing : D18 @ 508 mm

Provided Hoop Spacing : D18 @ 200 mm (Tie)

$\Phi V_c + \Phi V_s$ = 27915.1 + 2734.6 = 30649.7 kN > V_u = 6867.8 kN .... O.K

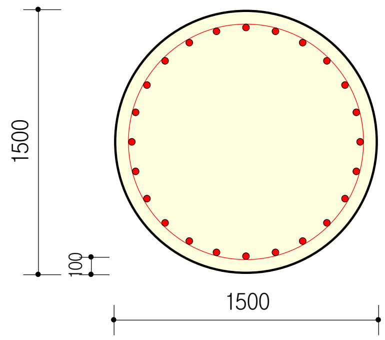
2.7.4. CHECK FOR PILE

Item		Mark	Unit	Value
• Factored Axial force		Nu	Kn	6591.05
• Factored Plexural moment		Mux	Kn.m	343.21
• Factored Plexural moment		Muy	Kn.m	32.29
• Diameter of Pier shaft		D	m	1.50
• Section area		Ag	m2	1.77
• Moment of inertia of concrete section		Ic	m4	0.25
• Cover thickness		a	m	0.075
• Reinf. Diameter		Ds	mm	D32
• Number of rebar		n_s	nos	24
• Rebar area		As	mm2	19301.95
Check minimum reinforcement				
• Minimum rebar area required (0.135*f'c/fy)*Ag		As req	mm2	17892.35
• Check condition As > (0.135*f'c/fy)*Ag				**OK**
Check maximum reinforcement				
• Maximum rebar area 0.08*Ag		As max	mm2	141371.7
• Check condition As < 0.08*Ag				**OK**
Check ratio spiral or Tier (5.7.4.6)				
• Distance to outside of Spairal or Ties to concrete face			mm	68.00
• Effect diamete		Deff	m	1.36
• Area of core measured to the outside diameter of the spiral			m2	1.46
• Ratio spiral Rebar required		ρsa		0.00707
Required Area of Spiral Rebar	space		mm	75
	Effective length			1.36
	layer			1
	Area			180.7
	Requaired Dhs			15.2
Actuaral	Effective length	d	m	1.364
	Diameter	Dhr	mm	16
	Area of Rebar	Ah	mm2	201.1
	layer	Nl	nos	1
	Total area of spiral	Ac	m2	201.062
	space	s	mm	75
	Ratio spiral Rebar	ρs	-	0.0078617
• Check condition		$\rho_s > \rho_{sa}$		OK

Company	Nipponkoei	Project Name	DN Qn Expressway
Designer	vinh	File Name	D:\...\Pile D1.5m - Pier P12.BOI

1. Geometry and Materials

Design Code : ACI318M-02
Stress Profile : Equivalent Stress Block
Material Data : f_c' = 30 MPa (β_1 = 0.850)
f_y = 400, f_{ys} = 400 MPa
Section Dim. : Φ1500 mm
Effective Len. : KL_u = 15000 mm
Steel Distribut.: 24 - D32 (d_c = 100 mm)
Total Steel Area A_{st} = 19061 mm^2 (ρ_{st} = 0.0108)



2. Member Force and Moment

Unit : kN, kN-m

L.C.	P_u	M_{ux}	M_{uy}	R_{atioV}	V_{ux}	V_{uy}	R_{atioH}	Remark
1	8226.5	90.0	32.0	0.303	8.0	22.0	0.005	
2	8193.2	900.0	32.0	0.319	8.0	207.0	0.043	
3	8369.3	417.0	32.0	0.308	8.0	97.0	0.020	
4	6641.9	343.0	32.0	0.245	8.0	80.0	0.017	
5	6643.5	343.0	32.0	0.245	8.0	80.0	0.017	
6	8778.3	1527.0	734.0	0.411	184.0	335.0	0.080	
7	8698.4	1171.0	32.0	0.360	8.0	270.0	0.056	
8	8248.3	618.0	322.0	0.304	79.0	140.0	0.034	

3. Magnified Moment

KL_u/r_x = 15000/375 = 40.00 > 34-12(M_1/M_2) = 22.00
δ_x = MAX[1.00/(1-P_u/0.75/52256), 1.0] = 1.289

KL_u/r_y = 15000/375 = 40.00 > 34-12(M_1/M_2) = 22.00
δ_y = MAX[1.00/(1-P_u/0.75/52256), 1.0] = 1.289

4. Design Force and Moment

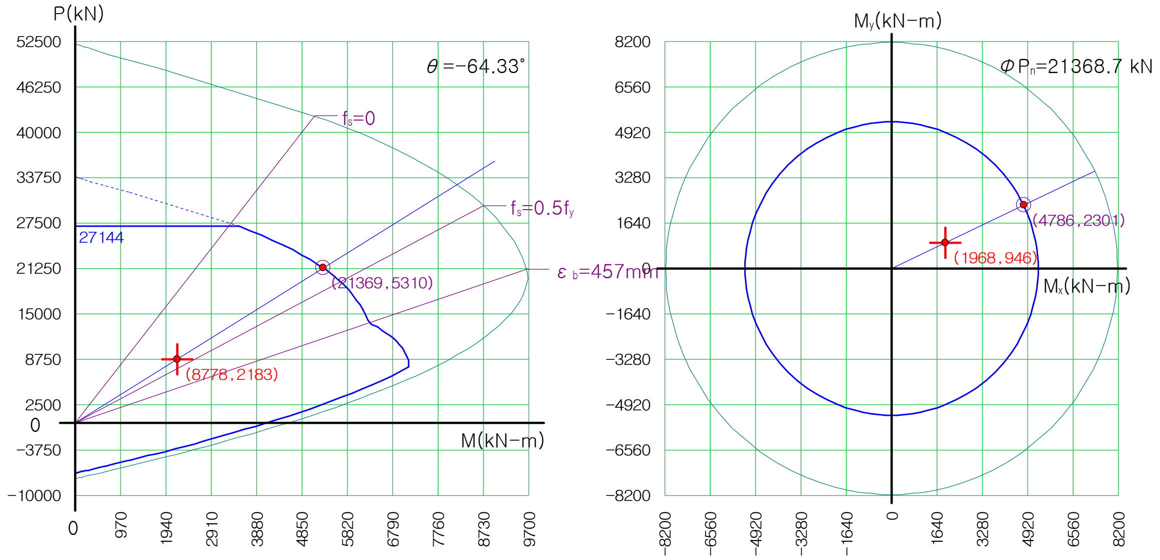
Design Load Combination No : 6
P_u = 8778.3 kN
M_{ux} = 1527.0, M_{uy} = 734.0 kN-m
$\delta_x M_{ux}$= δ_x*M_{ux} = 1967.7 kN-m
$\delta_y M_{uy}$= δ_y*M_{uy}, = 945.9 kN-m

5. Check Axial and Moment Capacity

Rotation Angle and Depth to the Neutral Axis θ = -64.33°, c = 1140 mm
Strength Reduction Factor Φ = 0.6500
Maximum Axial Load $\Phi P_{n(max)}$ = 27144.3 kN
Design Axial Load Strength ΦP_n = 21368.7 kN
Design Moment Strength ΦM_{nx} = 4786.3 kN-m
ΦM_{ny} = 2300.6 kN-m

Strength Ratio : Applied/Design = 0.411 < 1.000 .... O.K

	Company	Nipponkoei	Project Name	DN Qn Expressway
	Designer	vinh	File Name	D:\...\Pile D1.5m - Pier P12.BOI



6. Check Shear Capacity

Design Load Combination No : 6

Strength Reduction Factor Φ= 0.750

Design Force Vu = 382.2 kN (Pu = 8778.3 kN)

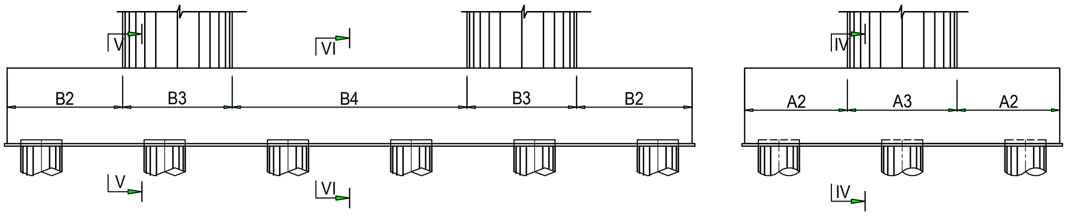
Required Hoop Spacing : D16 @ 508 mm

Provided Hoop Spacing : D16 @ 150 mm (Tie)

$\Phi V_c + \Phi V_s$ = 3869.9 + 924.5 = 4794.4 kN > V_u = 382.2 kN .... O.K

2.7.4. CHECK FOR PILE CAP

2.7.5.1. The Force to section IV-IV, section V-V, section VI-VI



At section IV - IV

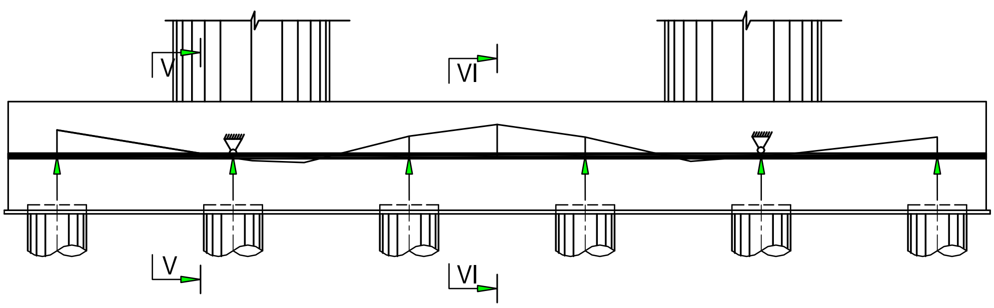
COMBINATION	Longitudinal direction		
	Q (kN)	M(kN.m)	V (kN.m)
Strength I	48271.4	152875.4	71.5
Strength II	43228.7	136905.2	71.5
Strength III	47118.7	149224.9	71.5
Service I	37328.0	118217.8	71.5
Service III	11789.5	37337.4	71.5
Extreme I (CV)	48530.9	153697.4	1657.5
Extreme II (EQ trans)	44628.0	141336.9	71.5
Extreme II (EQ long)	46286.4	146588.9	713.50

At section V - V

COMBINATION	Transverse direction		
	Q (kN)	M(kN.m)	V (kN.m)
Strength I	24600.8	89325.3	-44.2
Strength II	24500.9	88962.8	-414.7
Strength III	25029.1	90880.6	-194.3
Service I	19847.0	72064.5	-159.7
Service III	19851.8	72082.0	-159.7
Extreme I (CV)	15912.9	57779.6	-670.9
Extreme II (EQ trans)	26016.53	94466.0	-539.11
Extreme II (EQ long)	23958.88	86994.7	-280.22

At section VI - VI

COMBINATION	Transverse direction		
	Q (kN)	M(kN.m)	V (kN.m)
Strength I	29715.0	33559.5	-44.2
Strength II	26081.0	30040.2	-414.7
Strength III	27996.2	24874.1	-194.3
Service I	23256.1	25926.5	-159.7
Service III	23759.0	25933.0	-159.7
Extreme I (CV)	16783.6	20692.3	-670.9
Extreme II (EQ trans)	28338.20	31186.00	-539.1
Extreme II (EQ long)	35696.30	31018.50	-280.2



2.7.5.2. Ultimate load check, shear capacity check and crack control
*** Check flexure momen capacity of pile cap:**

Item			Section IV-IV (Bottom bar)	Section V-V (Bottom bar)	Section VI-VI (Upper bar)	Unit
• Factored Plexural moment		M_u	152875.37	90880.61	33559.50	kN.m
• Factored Shear force		V_u	48271.35	25029.09	29715.00	kN
• Hight of Section		h	3000	3000	3000	mm
• Width of section		b	25000	11500	11500	mm
• Section area		A_c	75000000	34500000	34500000	mm^2
• Moment of inertia of concrete section		I_g	5.6E+13	2.6E+13	2.6E+13	mm^4
• Tension reinforcement:	Distance from tension reinf. to extreme compression fiber	d_c	228	260	146	mm
	Reinf. Diameter	∅	28	32	36	mm
	Space	@	150	150	150	mm
	Number of bar	n	330	150	75	bar
	Total area of reinf.	A_s	203445	120959	76680	mm^2
• comp. reinforcement:	Distance from compressive reinf. to extreme Tension fiber		100	146	152	mm
	Diameter		28	36	32	mm
	Reinf. Space		150	150	150	mm
	Number of bar		165	75	150	bar
	Total area of reinf.	A'_s	101804	76680	120959	mm^2
Check Flexural Moment at Strength state						
• Resistance factor		Φ	0.90	0.90	0.90	
• The corresponding effective		d_e	2772	2740	2854	mm
• Stress block factor		β_1	0.84	0.84	0.84	
• Depth of the equivalent stress block = c*β1		a	127.65	164.99	104.59	mm
• Distance from extreme compression fiber to the neutral axis		c	152.75	197.42	125.15	mm
• The nominal flexural resistance:		M_n	220385	128579	85934	kN.m
• Factored flexural resistance		$M_r = \Phi.M_n$	198347	115722	77340	kN.m
• Check condition		$M_r > M_u$	**O.K**	**O.K**	**O.K**	
Mimimum Reinforcement						
• Ratio of tension steel to gross area		$\rho = A_s/(b.d)$	0.29	0.38	0.23	%
• Check		$\rho > 0.03 \cdot f'_c/f'_y$	**O.K**	**O.K**	**O.K**	0.23
• Cracking moment		1.2Mcr	155279.35	71428.50	71428.50	Kn.m
• Check		Mr> min(1.2Mcr, 1.33Mu)	**O.K**	**O.K**	**O.K**	
Maximum Reinforcement						
• Obligation Condition		c/d_e	0.06	0.07	0.04	
• Check		$c/d_e < 0.42$	**O.K**	**O.K**	**O.K**	
Check shear resistance						
• Factored Shear force		V_u	48271.35	25029.09	29715.00	kN
• Resistance factor		Φ	0.90	0.90	0.90	
• The effective shear deepth		d_v	2708	2658	2802	mm
• Effective width		b_v	25000	11500	11500	mm
• Angle of inclination of diagonal compressive stress		θ	43	43	43	degree
• Angle of inclination of transerverse reinf. To longitudinal axis		α	90	90	90	degree
• Factor indicating ability of diagonally cracked concrete to transmit tension		β	1.95	1.95	1.95	
• Value		$0.1 \cdot f'_c \cdot b_v \cdot d_v$	203113	91684	96659	kN
• Max spacing of transverse reinforcement		s_{max}	600	600	600	mm
• Spacing of stirrup		s	450	450	450	mm
• Diameter of transverse reinforcement		∅	D 28	D 32	D 36	
• Number of transverse reinf. within distance s		n	6	6	3	bar
• Total area of transverse reinf.		A_v	3695	4825	3054	mm^2
• Diameter of stirrup		∅	D 20	D 20	D 20	mm
• Number of stirrup within distance s		n	57	27	27	bar
• Total area of stirrup		A_v	17767.45	8342.67	8342.67	
• Assume		θ	43.00	43.00	43.00	degree
• Strain in tensile reinforcement		ex	2.02E-03	1.97E-03	1.82E-03	
If ex<0, multiple with reduce factor		Fc	-	-	-	
• Ratio of shear stress and f'c		V/f'c	0.03	0.03	0.03	
• β final			1.95	1.95	1.95	
• θ final			43.00	43.00	43.00	
• The shear resistance of concrete:		V_c	60019.16	27092.26	28562.31	kN

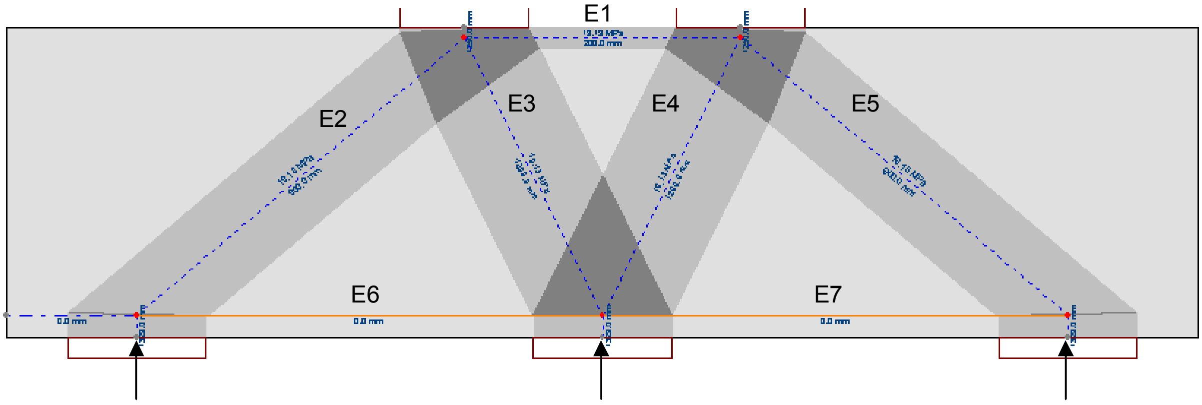
• The shear resistance of stirrup	V_s	21839.74	10062.94	10608.97	kN
• Value	$0.25*f'_c \cdot b_v \cdot d_v$	507782.68	229209.77	241646.91	kN
• The nominal shear resistance:	V_n	81858.90	37155.20	39171.28	kN
• The factored shear resistance	V_r	73673.01	33439.68	35254.15	kN
• Check	$V_r > V_u$	**O.K**	**O.K**	**O.K**	
•Requiring transverse reinforcement	$V_u > 0.5 \cdot \Phi \cdot V_c$	**Need**	**Need**	**Need**	
•Check minimum transverse reinforcement	$A_v > 0.083 \cdot (f'_c{}^{0.5}) \cdot b_v \cdot s/f_y$	**O.K**	**O.K**	**O.K**	
Check Flexural and shear resistance at Extreme state					
• Factored Plexural moment	M_u	153697.36	94466.02	31186.00	kN.m
• Factored Shear force	V_u	48530.90	26016.53	35696.30	kN
• Resistance factor	Φ	1.00	1.00	1.00	
• The nominal flexural resistance:	M_n	220385	128579	85934	kN.m
• Factored flexural resistance	$M_r = \Phi.M_n$	220385	128579	85934	kN.m
• The nominal flexural resistance:	V_n	81859	37155	39171	Kn
• Factored flexural resistance	$V_r = \Phi.V_n$	81859	37155	39171	Kn
• Check condition	$M_r > M_u$	**O.K**	**O.K**	**O.K**	
	$V_r > V_u$	**O.K**	**O.K**	**O.K**	
Check crack					
Interior force combination	**Service I**				
• Factored moment	M_u	1.18E+05	7.21E+04	2.59E+04	kN.m
• Modulus of rupture of concrete	fr = 0.63*sqrt(f'c)	3.45	3.45	3.45	MPa
• Distance from extreme tension fiber to the neutral axis	$y_t = h - c$	2847	2803	2875	mm
• Stress of concrete at tension fiber	$f'r = Ms*y_t/I_g$	5.98	7.81	2.88	MPa
• Check	f'r >	0.8*fr	0.8*fr	0.8*fr	
		check crack	check crack	check crack	
• Crack width parameter	Z	= 23000	= 23000	= 23000	N/mm
• Ratio of reinf. Modulus with concrete modulus	$n = E_s/E_c$	= 7.00	= 7.00	= 7.00	
• The distance from extreme fiber to the neutral axis	c	= 621.81	= 713.08	= 564.94	mm
• Effective moment of inertia	J	8.59E+12	4.87E+12	3.50E+12	mm^4
• Arm	de-c	= 2150.19	= 2026.92	= 2289.06	mm
• Tension stress in reinforcement	fs =n*Ms*(de-c)/J	= 207.20	= 210.07	= 118.60	MPa
• Area of concrete having the same centroid as the principal tensile reinforcement divided by number of bars	A	= 15133	= 15293	= 15265	mm2
• Tension stress in reinforcement with service state	$f_{sa} = Z/(d_c*A)^{1/3}$	= 232.47	= 229.29	= 227.15	Mpa
• Check condition	$f_s < f_{sa}$	**O.K**	**O.K**	**O.K**	
• Check condition	$f_s < 0.6*f_y$	**O.K**	**O.K**	**O.K**	

Check for pile cap of Pier P12
(Longitudinal direction)

General Properties

D-Region Thickness = 25000.0 mm
Concrete Cylinder Strength = 30.00 MPa
Non-Prestressed Reinforcement Yield Strength = 400.00 MPa

- **Truss Member Stress Limits and Effective Widths**



- **Summary of the Design**

  - Struts

Strut ID	F_u (kN)	β_s	ϕ	$\phi f_c = \phi(0.85)\beta_s f'_c$ (MPa)	Effective Width (mm)	ϕF_{ns} (kN)	Ratio $F_u / \phi F_{ns}$	Check $F_u / \phi F_{ns} < 1$
E2	-68584.9	1.0	0.75	19.13	900.0	430312.5	0.159	OK
E5	-68605.0	1.0	0.75	19.13	900.0	430312.5	0.159	OK
E3	-24738.2	1.0	0.75	19.13	1200.0	573750.0	0.043	OK
E4	-24772.7	1.0	0.75	19.13	1200.0	573750.0	0.043	OK

  - Ties

Tie ID	F_u (kN)	Required A_s (mm²)	Provided A_s (mm²)	ϕ	ϕF_{ns} (kN)	Ratio $F_u / \phi F_{ns}$	Check $F_u / \phi F_{ns} < 1$	Ties stress at Service state (Mpa)	Limit stress $0.6f_y$ =240 (Mpa)	Check $f_s < 0.6f_y$
E6	52330.4	174434.8	203181.0	0.750	60954.3	0.86	OK	216.86	240	OK
E7	52345.8	174486.0	203181.0	0.750	60954.3	0.86	OK	215.03	240	OK

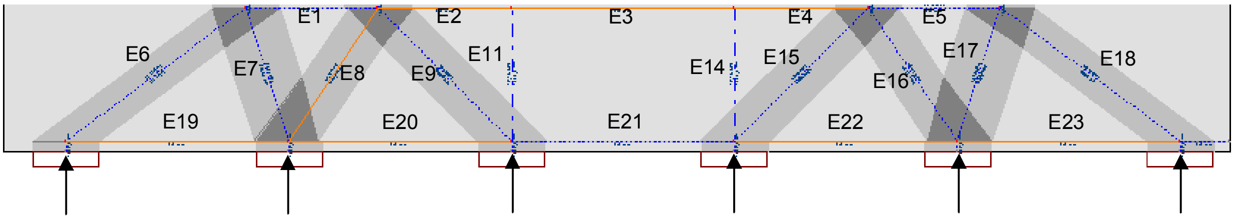
- Ties bar E6, E7 used 2x165 D28.

(Transverser direction)

General Properties

D-Region Thickness = 11500.0 mm
Concrete Cylinder Strength = 30.00 MPa
Non-Prestressed Reinforcement Yield Strength = 400.00 MPa

- **Truss Member Stress Limits and Effective Widths**



- **Summary of the Design**

  - Struts

Strut ID	F_u (kN)	β_s	ϕ	$\phi f_c = \phi(0.85)\beta_s f'_c$ (MPa)	**Effective Width** (mm)	ϕF_{ns} (kN)	Ratio $F_u/\phi F_{ns}$	Check $F_u/\phi F_{ns}<1$
E6	-44340.0	1.00	0.75	19.13	800.0	175950.0	0.252	OK
E7	-33014.2	1.00	0.75	19.13	1322.0	290757.4	0.114	OK
E9	-32798.1	1.00	0.75	19.13	1000.0	219937.5	0.149	OK
E15	-30489.0	1.00	0.75	19.13	1000.0	219937.5	0.139	OK
E16	-14855.2	1.00	0.75	19.13	1000.0	219937.5	0.07	OK
E17	-19612.1	1.00	0.75	19.13	1322.0	290757.4	0.07	OK
E18	-30657.8	1.00	0.75	19.13	800.0	175950.0	0.174	OK

  - Ties

Tie ID	F_u (kN)	Required A_s (mm²)	Provided A_s (mm²)	ϕ	ϕF_{ns} (kN)	Ratio $F_u/\phi F_{ns}$	Check $F_u/\phi F_{ns}$ <1	Ties stress at Service state (Mpa)	Limit stress $0.6f_y$ =240 (Mpa)	Check $f_s<0.6f_y$
E19	35603.3	118677.7	120600.0	0.75	36180.0	0.984	OK	221.70	240	OK
E20	21064.8	70216.0	120600.0	0.75	36180.0	0.582	OK	140.36	240	OK
E22	19430.2	64767.3	120600.0	0.75	36180.0	0.537	OK	140.23	240	OK
E23	24616.6	82055.3	120600.0	0.75	36180.0	0.680	OK	193.52	240	OK
E2	2152.7	7175.8	76340.7	0.75	22902.2	0.094	OK	25.58	240	OK
E3	2152.7	7175.8	76340.7	0.75	22902.2	0.094	OK	25.58	240	OK
E4	2152.7	7175.8	76340.7	0.75	22902.2	0.094	OK	25.58	240	OK

- Ties bar E19, E20, E22, E23 used 2x75 D32.
- Ties bar E2, E3, E4 used 75 D36.

2.8 PIER P13
2.8 TRỤ CẦU P13

PIER P13 - CALCULATION SHEET

CONTENT:

2.8.1. GENERAL DATA

CALCULATION PROCEDURE & STANDARD:

- Bridge Design Standard 22 TCN - 272 - 05

2.8.1.1. Design live load

Design vehicle load		HL93	22TCN 272 - 05
Number of lane		6	(lane)
Pedestrian		0.00	KG/m^2

2.8.1.2. Bridge width

Width carriageway	B_{xe} =	12.00	(m)
Width of median guardrail	B_{pc} =	0.50	(m)
Width parapet	B_{lc} =	0.50	(m)
Bridge width	B =	13.00	(m)

2.8.1.3. Superstructure:

Span-arrangement		Continuous box girder 65+5@100+65m	
Height of box girder at pier section	H =	6.00	(m)
Height of box girder at mid-span section	h =	2.50	(m)
Pavement thickness	d_{BTN} =	0.084	(m)

2.8.1.4. Material property:

Concrete

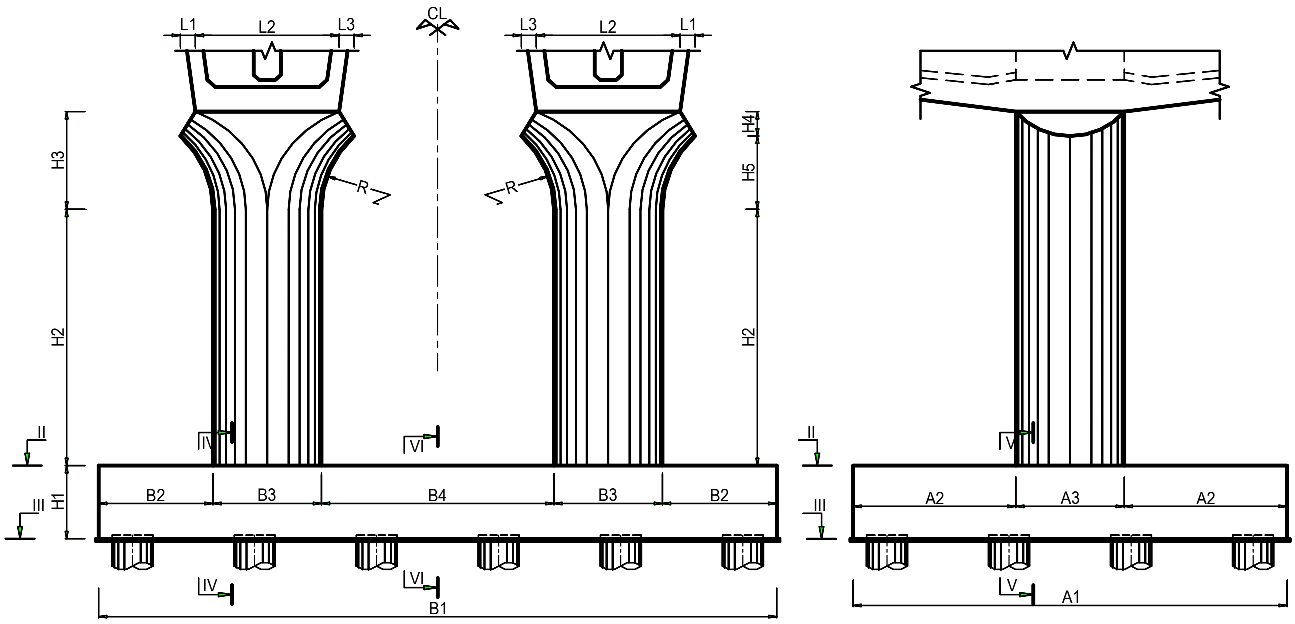
Compressive strength of cylindrical at 28 da	f 'c =	30.00	MPa
Concrete density	g =	24.50	KN/m^3
Elastic modulus	Ec =	29440	MPa
Tension strength of concrete	fr =	3.45	MPa

Steel

Steel modulus	Es =	200000	MPa
Yeild strength of steel bar	fy =	400.00	MPa
Calculation unit:		KN, m, KN.m, Mpa, KN/m^2	

2.8.1.5. THE PIER GEOMETRIC

GENERAL DIMENSIONS



vertical			Horizontal			Thickness		
Remark	Value	Unit	Remark	Value	Unit	Remark	Value	Unit
a_1 =	14.80	(m)	L_1 =	0.55	(m)	h_1 =	3.00	(m)
a_2 =	5.40	(m)	L_2 =	5.30	(m)	h_2 =	10.500	(m)
a_3 =	4.00	(m)	L_3 =	0.55	(m)	h_3 =	4.000	(m)
			B_1 =	25.00	(m)	h_4 =	1.000	(m)
			B_2 =	3.965	(m)	h_5 =	3.000	(m)
			B_3 =	4.00	(m)			
			B_4 =	9.070	(m)			
			R =	4.34	(m)			

The design elevation:

Proposed height	EL_{mc} =	19.660	(m)			
Elevation of top of pier cap	EL_{xm} =	13.455	(m)			
Hight water level (H1%)	EL_{MNTK} =	9.200	(m)			
Daily water level (H5%)	EL_{MNTB} =	4.070	(m)			
Ground elevation	EL_{TN} =	1.080	(m)			
Daily water level (H5%)	EL_{TT} =	4.070	(m)			
Top of Pile cap (Section II - II)	$EL_{MC\ II\text{-}II}$ =	-1.045	(m)			
Bottom of Pile cap (Section III - III)	$EL_{MCIII\text{-}III}$ =	-4.045	(m)			

2.8.2. CALCULATED PIER FORCE AND PILE FORCE

2.8.2.1. Loading combination at bottom of pier shaft

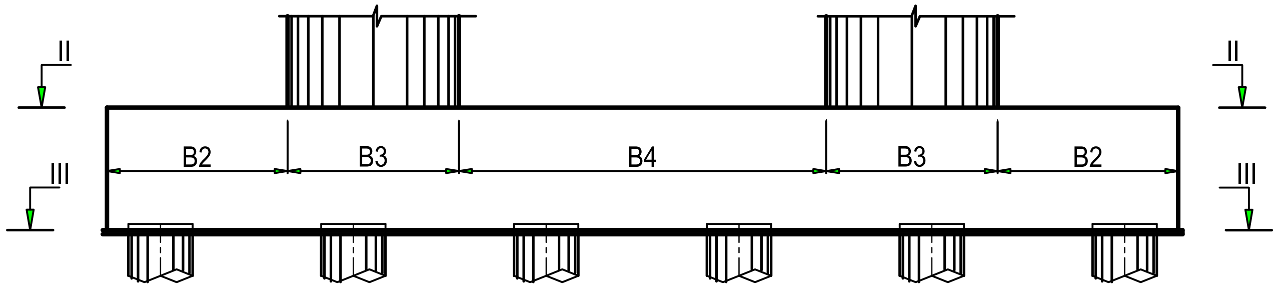
(Result from RM2010 SPACE-FRAME program)

Loading combination in accordance with 22 TCN 272-05, table 3.4.1.1 was mentioned in appendix of this report

LOADING COMBINATION AT TOP OF PILE CAP DUE TO SUPERSTRUCTURE + VESSEL COLLISION AND SEFTWEIGHT						
Limited state	Vertical axis	Longitudinal axis		Horizontal axis		Remarks
	N_x (kN)	Q_y (kN)	M_z (kNm)	Q_z (kN)	M_y (kNm)	
STRENGTH I	54045.62	5438.94	37643.59	191.91	7237.14	
STRENGTH II	46101.14	1985.98	21605.66	1766.36	34270.44	
STRENGTH III	52229.64	4605.98	33977.85	825.87	18615.49	
SERVICE I	41014.02	3721.34	28173.48	668.40	14853.70	
SERVICE III	41066.16	4258.39	33857.43	668.35	14582.69	
EXTREME EVENT (CV)	48013.70	2213.17	25707.18	5986.72	25905.06	
EXTREME EVENT (EQ trans)	48175.70	4790.32	35918.80	2286.13	43649.05	
EXTREME EVENT (EQ long)	48175.70	10479.76	79769.41	1138.67	17027.78	

2.8.2.2. Loading combination at bottom of pile cap

LOADING COMBINATION AT TOP OF PILE CAP DUE TO SUPERSTRUCTURE + VESSEL						
Limited state	Vertical axis	Longitudinal axis		Horizontal axis		Remarks
	N_x (kN)	Q_y (kN)	M_z (kNm)	Q_z (kN)	M_y (kNm)	
STRENGTH I	143636	14589	97442	10914	410	
STRENGTH II	127747	79702	54614	4008	3507	
STRENGTH III	140004	41395	87653	9249	1678	
SERVICE I	110223	33142	72661	7479	1363	
SERVICE III	110327	33139	87252	8553	1363	
EXTREME EVENT (CV)	131572	43731	36703	2231	6000	
EXTREME EVENT (EQ trans)	131896	100746	96757	9617	4598	
EXTREME EVENT (EQ long)	85354	40967	219089	21326	2535	



2.8.2.3. Loading combination at top of piling

2.8.2.3.1. Piling material:

Concrete 30 Mpa

E_c (kg/cm^2)	294401
γ_c (T/m^3)	2.5

Steel bar

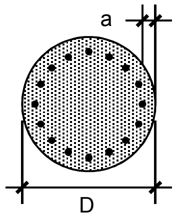
Type	CB-400-T
E_s (kg/cm^2)	200000

2.8.2.3.2. Piling dimension

+ Diameter **D** = 1.50 m

a = 0.100 m

+ Length **L** = 60.00 m

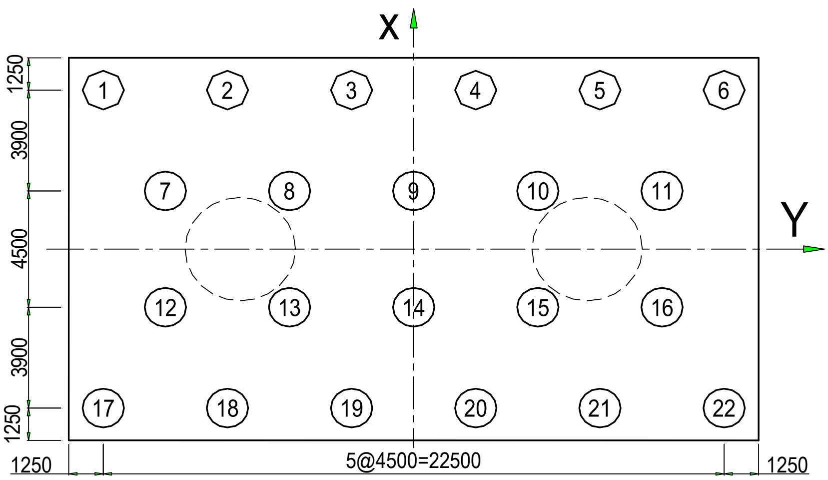


2.8.2.3.3. Maximim Internal force and displacement at top piling

Maximum Internal force and diplacements *(Result follow Piling software)*

Combination	N	Hx	My	x	y	z
	(KN)	(KN)	(KN.m)	(m)	(m)	(rad)
Strength I	8774.57	496.09	2931.80	-	-	-
Strength II	7871.32	182.18	1052.80	-	-	-
Strength III	8715.95	420.41	2478.11	-	-	-
Service I	6949.52	339.96	2001.61	**0.014**	**-0.002**	**0.008**
Service III	7207.76	388.77	2283.78	**0.016**	**-0.002**	**0.009**
Extreme I (CV)	7498.32	101.41	578.04	-	-	-
Extreme II (EQ trans)	9259.20	437.14	2569.59	-	-	-
Extreme II (EQ long)	10912.89	969.36	5692.41	-	-	-

- Check displacement of top pile not exceed 38mm (10.7.2.7) **OK**



Arrangement of pile

2.8.2.3.4. Internal force for each pile

Combination	Pile Number	Hx (KN)	Hy (KN)	N (KN.m)	Mx (m)	My (m)
Strength I	1	496.09	-18.59	4826.33	107.77	2931.80
	2	496.09	-18.59	4757.80	107.77	2931.80
	3	496.09	-18.59	4689.28	107.77	2931.80
	4	496.09	-18.59	4620.75	107.77	2931.80
	5	496.09	-18.59	4552.23	107.77	2931.80
	6	496.09	-18.59	4483.70	107.77	2931.80
	7	496.09	-18.59	6043.95	107.77	2931.80
	8	496.09	-18.59	5975.42	107.77	2931.80
	9	496.09	-18.59	5906.90	107.77	2931.80
	10	496.09	-18.59	5838.37	107.77	2931.80
	11	496.09	-18.59	5769.85	107.77	2931.80
	12	496.09	-18.59	7488.43	107.77	2931.80
	13	496.09	-18.59	7419.90	107.77	2931.80
	14	496.09	-18.59	7351.38	107.77	2931.80
	15	496.09	-18.59	7282.85	107.77	2931.80
	16	496.09	-18.59	7214.33	107.77	2931.80
	17	496.09	-18.59	8774.57	107.77	2931.80
	18	496.09	-18.59	8706.05	107.77	2931.80
	19	496.09	-18.59	8637.52	107.77	2931.80
	20	496.09	-18.59	8569.00	107.77	2931.80
	21	496.09	-18.59	8500.47	107.77	2931.80
	22	496.09	-18.59	8431.95	107.77	2931.80
Strength II	1	182.18	-159.41	5975.08	950.50	1052.80
	2	182.18	-159.41	5568.57	950.50	1052.80
	3	182.18	-159.41	5162.05	950.50	1052.80
	4	182.18	-159.41	4755.53	950.50	1052.80
	5	182.18	-159.41	4349.02	950.50	1052.80
	6	182.18	-159.41	3942.50	950.50	1052.80
	7	182.18	-159.41	6373.07	950.50	1052.80
	8	182.18	-159.41	5966.55	950.50	1052.80
	9	182.18	-159.41	5560.04	950.50	1052.80
	10	182.18	-159.41	5153.52	950.50	1052.80
	11	182.18	-159.41	4747.00	950.50	1052.80
	12	182.18	-159.41	7066.82	950.50	1052.80
	13	182.18	-159.41	6660.30	950.50	1052.80
	14	182.18	-159.41	6253.78	950.50	1052.80
	15	182.18	-159.41	5847.26	950.50	1052.80
	16	182.18	-159.41	5440.75	950.50	1052.80
	17	182.18	-159.41	7871.32	950.50	1052.80
	18	182.18	-159.41	7464.80	950.50	1052.80
	19	182.18	-159.41	7058.29	950.50	1052.80
	20	182.18	-159.41	6651.77	950.50	1052.80
	21	182.18	-159.41	6245.25	950.50	1052.80
	22	182.18	-159.41	5838.73	950.50	1052.80

- Result for internal force at top pile

Combination	Pile Number	**Hx** (KN)	**Hy** (KN)	**N** (KN.m)	**Mx** (m)	**My** (m)
Strength III	1	420.41	-76.27	5249.69	452.89	2478.11
	2	420.41	-76.27	5042.18	452.89	2478.11
	3	420.41	-76.27	4834.67	452.89	2478.11
	4	420.41	-76.27	4627.17	452.89	2478.11
	5	420.41	-76.27	4419.66	452.89	2478.11
	6	420.41	-76.27	4212.15	452.89	2478.11
	7	420.41	-76.27	6244.99	452.89	2478.11
	8	420.41	-76.27	6037.48	452.89	2478.11
	9	420.41	-76.27	5829.98	452.89	2478.11
	10	420.41	-76.27	5622.47	452.89	2478.11
	11	420.41	-76.27	5414.96	452.89	2478.11
	12	420.41	-76.27	7513.14	452.89	2478.11
	13	420.41	-76.27	7305.63	452.89	2478.11
	14	420.41	-76.27	7098.12	452.89	2478.11
	15	420.41	-76.27	6890.61	452.89	2478.11
	16	420.41	-76.27	6683.10	452.89	2478.11
	17	420.41	-76.27	8715.95	452.89	2478.11
	18	420.41	-76.27	8508.44	452.89	2478.11
	19	420.41	-76.27	8300.93	452.89	2478.11
	20	420.41	-76.27	8093.42	452.89	2478.11
	21	420.41	-76.27	7885.91	452.89	2478.11
	22	420.41	-76.27	7678.40	452.89	2478.11
Service I	1	339.96	-61.96	4104.37	368.15	2001.61
	2	339.96	-61.96	3937.74	368.15	2001.61
	3	339.96	-61.96	3771.10	368.15	2001.61
	4	339.96	-61.96	3604.47	368.15	2001.61
	5	339.96	-61.96	3437.84	368.15	2001.61
	6	339.96	-61.96	3271.21	368.15	2001.61
	7	339.96	-61.96	4923.17	368.15	2001.61
	8	339.96	-61.96	4756.54	368.15	2001.61
	9	339.96	-61.96	4589.91	368.15	2001.61
	10	339.96	-61.96	4423.28	368.15	2001.61
	11	339.96	-61.96	4256.65	368.15	2001.61
	12	339.96	-61.96	5964.08	368.15	2001.61
	13	339.96	-61.96	5797.45	368.15	2001.61
	14	339.96	-61.96	5630.82	368.15	2001.61
	15	339.96	-61.96	5464.19	368.15	2001.61
	16	339.96	-61.96	5297.56	368.15	2001.61
	17	339.96	-61.96	6949.52	368.15	2001.61
	18	339.96	-61.96	6782.89	368.15	2001.61
	19	339.96	-61.96	6616.26	368.15	2001.61
	20	339.96	-61.96	6449.62	368.15	2001.61
	21	339.96	-61.96	6282.99	368.15	2001.61
	22	339.96	-61.96	6116.36	368.15	2001.61

- Result for internal force at top pile

Combination	Pile Number	**Hx** (KN)	**Hy** (KN)	**N** (KN.m)	**Mx** (m)	**My** (m)
Service III	1	388.77	-61.96	3855.52	368.15	2283.78
	2	388.77	-61.96	3688.90	368.15	2283.78
	3	388.77	-61.96	3522.28	368.15	2283.78
	4	388.77	-61.96	3355.66	368.15	2283.78
	5	388.77	-61.96	3189.04	368.15	2283.78
	6	388.77	-61.96	3022.42	368.15	2283.78
	7	388.77	-61.96	4835.12	368.15	2283.78
	8	388.77	-61.96	4668.50	368.15	2283.78
	9	388.77	-61.96	4501.88	368.15	2283.78
	10	388.77	-61.96	4335.26	368.15	2283.78
	11	388.77	-61.96	4168.64	368.15	2283.78
	12	388.77	-61.96	6061.55	368.15	2283.78
	13	388.77	-61.96	5894.93	368.15	2283.78
	14	388.77	-61.96	5728.31	368.15	2283.78
	15	388.77	-61.96	5561.69	368.15	2283.78
	16	388.77	-61.96	5395.07	368.15	2283.78
	17	388.77	-61.96	7207.76	368.15	2283.78
	18	388.77	-61.96	7041.14	368.15	2283.78
	19	388.77	-61.96	6874.52	368.15	2283.78
	20	388.77	-61.96	6707.90	368.15	2283.78
	21	388.77	-61.96	6541.28	368.15	2283.78
	22	388.77	-61.96	6374.66	368.15	2283.78
Extreme Event I (with CV)	1	101.41	-272.73	6293.42	1680.15	578.04
	2	101.41	-272.73	5967.38	1680.15	578.04
	3	101.41	-272.73	5641.34	1680.15	578.04
	4	101.41	-272.73	5315.30	1680.15	578.04
	5	101.41	-272.73	4989.27	1680.15	578.04
	6	101.41	-272.73	4663.23	1680.15	578.04
	7	101.41	-272.73	6512.44	1680.15	578.04
	8	101.41	-272.73	6186.40	1680.15	578.04
	9	101.41	-272.73	5860.37	1680.15	578.04
	10	101.41	-272.73	5534.33	1680.15	578.04
	11	101.41	-272.73	5208.29	1680.15	578.04
	12	101.41	-272.73	6953.26	1680.15	578.04
	13	101.41	-272.73	6627.22	1680.15	578.04
	14	101.41	-272.73	6301.18	1680.15	578.04
	15	101.41	-272.73	5975.14	1680.15	578.04
	16	101.41	-272.73	5649.11	1680.15	578.04
	17	101.41	-272.73	7498.32	1680.15	578.04
	18	101.41	-272.73	7172.28	1680.15	578.04
	19	101.41	-272.73	6846.24	1680.15	578.04
	20	101.41	-272.73	6520.20	1680.15	578.04
	21	101.41	-272.73	6194.17	1680.15	578.04
	22	101.41	-272.73	5868.13	1680.15	578.04

- Result for internal force at top pile

Combination	Pile Number	**Hx** (KN)	**Hy** (KN)	**N** (KN.m)	**Mx** (m)	**My** (m)
Extreme Event I (with EQtran)	1	437.14	-209.00	5521.91	1248.38	2569.59
	2	437.14	-209.00	5003.89	1248.38	2569.59
	3	437.14	-209.00	4485.87	1248.38	2569.59
	4	437.14	-209.00	3967.85	1248.38	2569.59
	5	437.14	-209.00	3449.83	1248.38	2569.59
	6	437.14	-209.00	2931.80	1248.38	2569.59
	7	437.14	-209.00	6447.89	1248.38	2569.59
	8	437.14	-209.00	5929.87	1248.38	2569.59
	9	437.14	-209.00	5411.85	1248.38	2569.59
	10	437.14	-209.00	4893.83	1248.38	2569.59
	11	437.14	-209.00	4375.81	1248.38	2569.59
	12	437.14	-209.00	7815.19	1248.38	2569.59
	13	437.14	-209.00	7297.17	1248.38	2569.59
	14	437.14	-209.00	6779.15	1248.38	2569.59
	15	437.14	-209.00	6261.13	1248.38	2569.59
	16	437.14	-209.00	5743.11	1248.38	2569.59
	17	437.14	-209.00	9259.20	1248.38	2569.59
	18	437.14	-209.00	8741.17	1248.38	2569.59
	19	437.14	-209.00	8223.15	1248.38	2569.59
	20	437.14	-209.00	7705.13	1248.38	2569.59
	21	437.14	-209.00	7187.11	1248.38	2569.59
	22	437.14	-209.00	6669.09	1248.38	2569.59
Extreme Event I (with EQlong)	1	969.36	-115.23	2518.04	696.76	5692.41
	2	969.36	-115.23	2290.58	696.76	5692.41
	3	969.36	-115.23	2063.12	696.76	5692.41
	4	969.36	-115.23	1835.67	696.76	5692.41
	5	969.36	-115.23	1608.21	696.76	5692.41
	6	969.36	-115.23	1380.75	696.76	5692.41
	7	969.36	-115.23	5066.09	696.76	5692.41
	8	969.36	-115.23	4838.63	696.76	5692.41
	9	969.36	-115.23	4611.18	696.76	5692.41
	10	969.36	-115.23	4383.72	696.76	5692.41
	11	969.36	-115.23	4156.26	696.76	5692.41
	12	969.36	-115.23	8137.38	696.76	5692.41
	13	969.36	-115.23	7909.92	696.76	5692.41
	14	969.36	-115.23	7682.46	696.76	5692.41
	15	969.36	-115.23	7455.00	696.76	5692.41
	16	969.36	-115.23	7227.55	696.76	5692.41
	17	969.36	-115.23	10912.89	696.76	5692.41
	18	969.36	-115.23	10685.43	696.76	5692.41
	19	969.36	-115.23	10457.97	696.76	5692.41
	20	969.36	-115.23	10230.51	696.76	5692.41
	21	969.36	-115.23	10003.06	696.76	5692.41
	22	969.36	-115.23	9775.60	696.76	5692.41

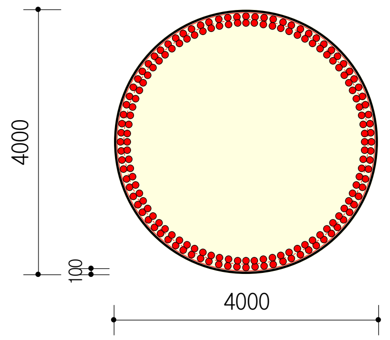
2.8.3. CHECK FOR PIER SHAFT

Item		Mark	Unit	Value
• Factored Axial force		Nu	Kn	41066.16
• Factored Plexural moment		Mux	Kn.m	33857.43
• Factored Plexural moment		Muy	Kn.m	14582.69
• Diameter of Pier shaft		D	m	4.00
• Section area		Ag	m2	12.57
• Moment of inertia of concrete section		Ic	m4	12.57
• Cover thickness		a	m	0.075
• Reinf. Diameter		Ds	mm	32.00
• Number of rebar		n_s	nos	168.00
• Rebar area		As	mm2	135113.62
Check minimum reinforcement				
• Minimum rebar area required (0.135*f'c/fy)*Ag		As req	mm2	127234.50
• Check condition As > (0.135*f'c/fy)*Ag				**OK**
Check maximum reinforcement				
• Maximum rebar area 0.08*Ag		As max	mm2	1005309.6
• Check condition As < 0.08*Ag				**OK**
Check ratio spiral or Tier (5.7.4.6)				
• Distance to outside of Spairal or Ties to concrete face			mm	66.00
• Effect diamete		Deff	m	3.87
• Area of core measured to the outside diameter of the spiral			m2	11.75
• Ratio spiral Rebar required		ρsa		0.00234
Required Area of Spiral Rebar	space		mm	200
	Effective length			3.87
	layer			2
	Area			226.6
	Requaired Dhs			17.0
Actuaral	Effective length	d	m	3.868
	Diameter	Dhr	mm	18
	Area of Rebar	Ah	mm2	254.5
	layer	Nl	nos	2
	Total area of spiral	Ac	m2	508.938
	space	s	mm	200
	Ratio spiral Rebar	ρs	-	0.0026315
• Check condition		$\rho_s > \rho_{sa}$		OK
Check Crack (at Service state)				
• Modulus of rupture of concrete		fr = 0.63*sqrt(f'c)	Mpa	3.45
• Stress of concrete at tension fiber		σ'_r	Mpa	1.80
• If f'r > 0.8fr require check crack		$\sigma'_r > 0.8*\sigma_r$	Mpa	No check
• Center of newtral axial		x	mm	3.08
• Maximum stress of Compression fiber of concrete		σ_c	Mpa	7.6
• Maximum stress of Compression Rebar		σ_{rc}	Mpa	-110.9
• Maximum stress of Tension Rebar		σ_{rt}	Mpa	30.6
• Check			σ_{rt} <0.6.fy	OK

MIDAS/Set **Column Design [Pier Shaft D4.0m-P13]**

Company	Nipponkoei	**Project Name**	DN Qn Expressway
Designer	vinh	**File Name**	D:\...\bodypier-D4.0m-P13.BOl

1. Geometry and Materials



Design Code : ACI318M-02
Stress Profile : Equivalent Stress Block
Material Data : f_c' = 30 MPa (β_1 = 0.850)
f_y = 400, f_{ys} = 400 MPa
Section Dim. : Φ4000 mm
Effective Len. : KL_u = 14500 mm
Steel Distribut.: 84 - D32 (d_c = 100 mm)
: 84 - D32 (d_c = 200 mm)
Total Steel Area A_{st} = 131896 mm^2 (ρ_{st} = 0.0106)

2. Member Force and Moment

Unit : kN, kN-m

L.C.	P_u	M_{ux}	M_{uy}	R_{atioV}	V_{ux}	V_{uy}	R_{atioH}	Remark
1	54045.6	37643.6	7237.1	0.365	5438.9	191.9	0.182	
2	46101.1	21605.7	34270.4	0.357	1986.0	1766.4	0.089	
3	52229.6	33977.9	18615.5	0.363	4606.0	825.9	0.157	
4	41014.0	28173.5	14853.7	0.293	3721.3	668.4	0.127	
5	41066.2	33857.4	14582.7	0.322	4258.4	668.4	0.145	
6	48013.7	25707.2	25905.1	0.338	2213.2	5986.7	0.214	
7	48175.7	35918.8	43649.1	0.464	4790.3	2286.1	0.178	
8	48175.7	79769.4	17027.8	0.589	10479.8	1138.7	0.353	

3. Magnified Moment

KL_u/r_x = 14500/1000= 14.50 < 34-12(M_1/M_2) = 22.00
δ_x = 1.000

KL_u/r_y = 14500/1000= 14.50 < 34-12(M_1/M_2) = 22.00
δ_y = 1.000

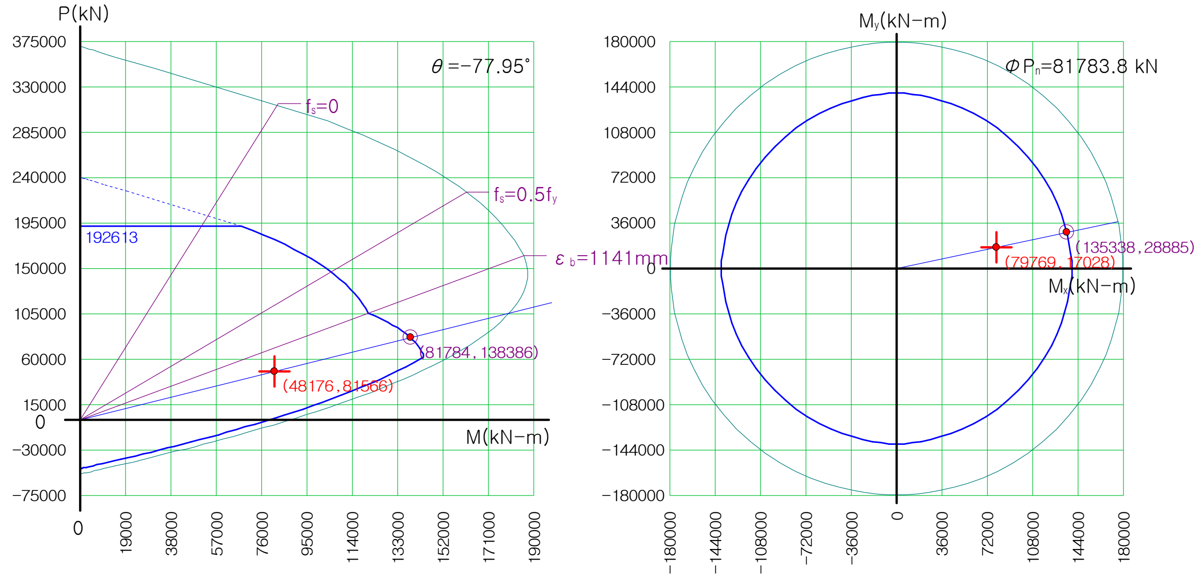
4. Design Force and Moment

Design Load Combination No : 8
P_u = 48175.7 kN
M_{ux} = 79769.4, M_{uy} = 17027.8 kN-m

5. Check Axial and Moment Capacity

Rotation Angle and Depth to the Neutral Axis θ = -77.95°, c = 1838 mm
Strength Reduction Factor Φ = 0.7693
Maximum Axial Load $\Phi P_{n(max)}$ = 192613.4 kN
Design Axial Load Strength ΦP_n = 81783.8 kN
Design Moment Strength ΦM_{nx} = 135338.0 kN-m
ΦM_{ny} = 28884.6 kN-m
Strength Ratio : Applied/Design = 0.589 < 1.000 .... O.K

	Company	Nipponkoei	Project Name	DN Qn Expressway
	Designer	vinh	File Name	D:\...\bodypier-D4.0m-P13.BOI



6. Check Shear Capacity

Design Load Combination No : 8

Strength Reduction Factor Φ= 0.750

Design Force Vu = 10541.5 kN (Pu = 48175.7 kN)

Required Hoop Spacing : D18 @ 508 mm

Provided Hoop Spacing : D18 @ 200 mm (Tie)

$\Phi V_c + \Phi V_s$ = 27921.5 + 1912.3 = 29833.7 kN > V_u = 10541.5 kN .... O.K

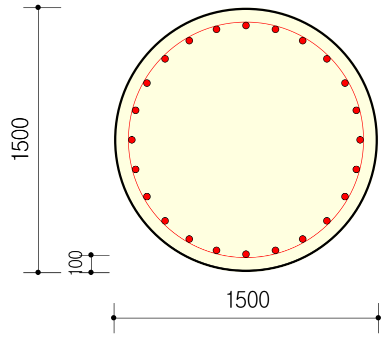
2.8.4. CHECK FOR PILE

Item		Mark	Unit	Value
• Factored Axial force		Nu	Kn	6616.26
• Factored Plexural moment		Mux	Kn.m	368.15
• Factored Plexural moment		Muy	Kn.m	2001.61
• Diameter of Pier shaft		D	m	1.50
• Section area		Ag	m2	1.77
• Moment of inertia of concrete section		Ic	m4	0.25
• Cover thickness		a	m	0.075
• Reinf. Diameter		Ds	mm	D32
• Number of rebar		n_s	nos	24
• Rebar area		As	mm2	19301.95
Check minimum reinforcement				
• Minimum rebar area required (0.135*f'c/fy)*Ag		As req	mm2	17892.35
• Check condition As > (0.135*f'c/fy)*Ag				**OK**
Check maximum reinforcement				
• Maximum rebar area 0.08*Ag		As max	mm2	141371.7
• Check condition As < 0.08*Ag				**OK**
Check ratio spiral or Tier (5.7.4.6)				
• Distance to outside of Spairal or Ties to concrete face			mm	68.00
• Effect diamete		Deff	m	1.36
• Area of core measured to the outside diameter of the spiral			m2	1.46
• Ratio spiral Rebar required		ρsa		0.00707
Required Area of Spiral Rebar	space		mm	75
	Effective length			1.36
	layer			1
	Area			180.7
	Requaired Dhs			15.2
Actuaral	Effective length	d	m	1.364
	Diameter	Dhr	mm	16
	Area of Rebar	Ah	mm2	201.1
	layer	Nl	nos	1
	Total area of spiral	Ac	m2	201.062
	space	s	mm	75
	Ratio spiral Rebar	ρs	-	0.0078617
• Check condition		$\rho_s > \rho_{sa}$		OK

	Company	Nipponkoei	Project Name	DN Qn Expressway
	Designer	vinh	File Name	

1. Geometry and Materials

Design Code : ACI318M-02
Stress Profile : Equivalent Stress Block
Material Data : f_c' = 30 MPa (β_1 = 0.850)
f_y = 400, f_{ys} = 400 MPa
Section Dim. : Φ1500 mm
Effective Len. : KL_u = 15000 mm
Steel Distribut.: 24 - D32 (d_c = 100 mm)
Total Steel Area A_{st} = 19061 mm^2 (ρ_{st} = 0.0108)



2. Member Force and Moment

Unit : kN, kN-m

L.C.	P_u	M_{ux}	M_{uy}	R_{atioV}	V_{ux}	V_{uy}	R_{atioH}	Remark
1	8774.6	107.8	2931.8	0.532	496.1	18.6	0.120	
2	7871.3	950.5	1052.8	0.329	182.2	159.4	0.059	
3	8716.0	452.9	2478.1	0.472	420.4	76.3	0.103	
4	6949.5	368.1	2001.6	0.375	340.0	62.0	0.084	
5	7207.8	368.1	2283.8	0.415	388.8	62.0	0.096	
6	7498.3	1680.2	578.0	0.355	101.4	272.7	0.071	
7	6261.1	1248.4	2569.6	0.471	437.1	209.0	0.118	
8	10912.9	696.8	5692.4	0.930	969.4	115.2	0.234	

3. Magnified Moment

KL_u/r_x = 15000/375 = 40.00 > 34-12(M_1/M_2) = 22.00
δ_x = MAX[1.00/(1-P_u/0.75/145156), 1.0] = 1.111

KL_u/r_y = 15000/375 = 40.00 > 34-12(M_1/M_2) = 22.00
δ_y = MAX[1.00/(1-P_u/0.75/145156), 1.0] = 1.111

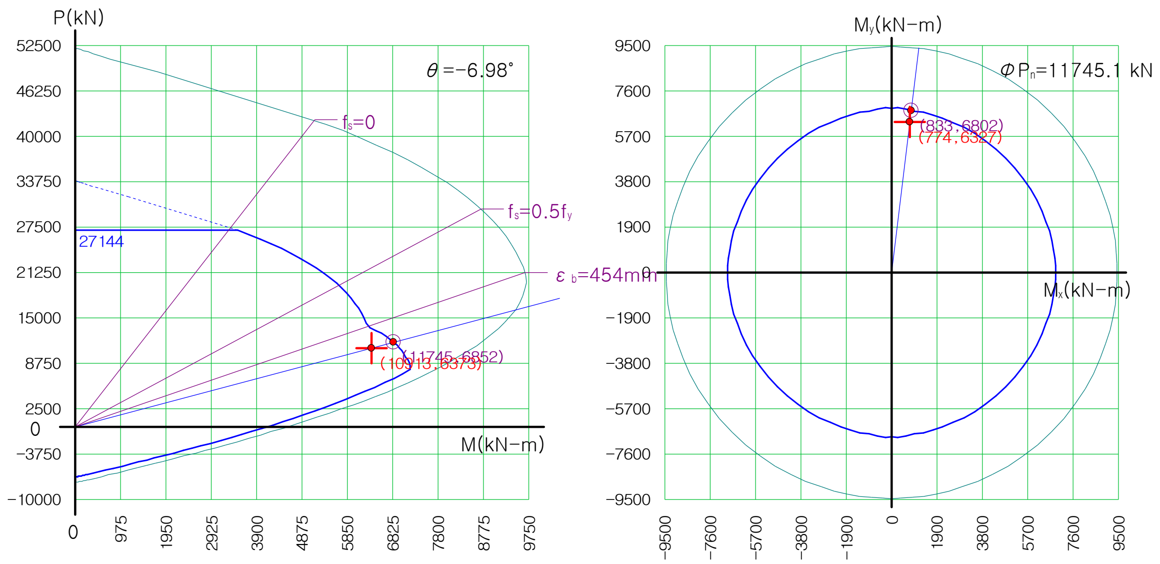
4. Design Force and Moment

Design Load Combination No : 8
P_u = 10912.9 kN
M_{ux} = 696.8, M_{uy} = 5692.4 kN-m
$\delta_x M_{ux}$= δ_x*M_{ux} = 774.4 kN-m
$\delta_y M_{uy}$= δ_y*M_{uy}, = 6326.6 kN-m

5. Check Axial and Moment Capacity

Rotation Angle and Depth to the Neutral Axis θ = -6.98°, c = 719 mm
Strength Reduction Factor Φ = 0.7230
Maximum Axial Load $\Phi P_{n(max)}$ = 27144.3 kN
Design Axial Load Strength ΦP_n = 11745.1 kN
Design Moment Strength ΦM_{nx} = 832.7 kN-m
ΦM_{ny} = 6801.7 kN-m
Strength Ratio : Applied/Design = 0.930 < 1.000 .... O.K

	Company	Nipponkoei	Project Name	DN Qn Expressway
	Designer	vinh	File Name	



6. Check Shear Capacity

Design Load Combination No : 8

Strength Reduction Factor Φ= 0.750

Design Force Vu = 976.2 kN (Pu = 10912.9 kN)

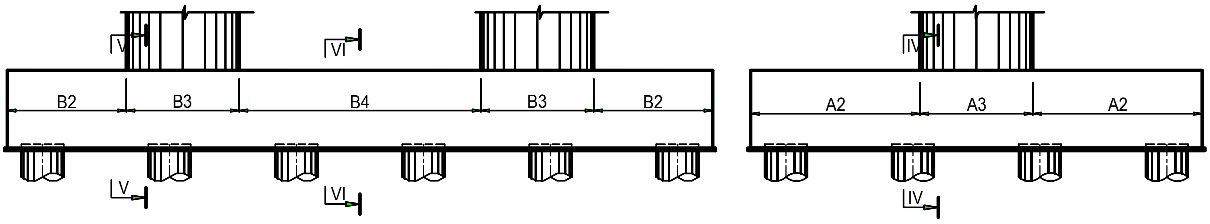
Required Hoop Spacing : D16 @ 508 mm

Provided Hoop Spacing : D16 @ 508 mm (Tie)

$\Phi V_c + \Phi V_s$ = 3902.1 + 273.0 = 4175.1 kN > V_u = 976.2 kN .... O.K

2.8.4. CHECK FOR PILE CAP

2.8.4.1. The Force to section IV-IV, section V-V, section VI-VI



At section IV - IV

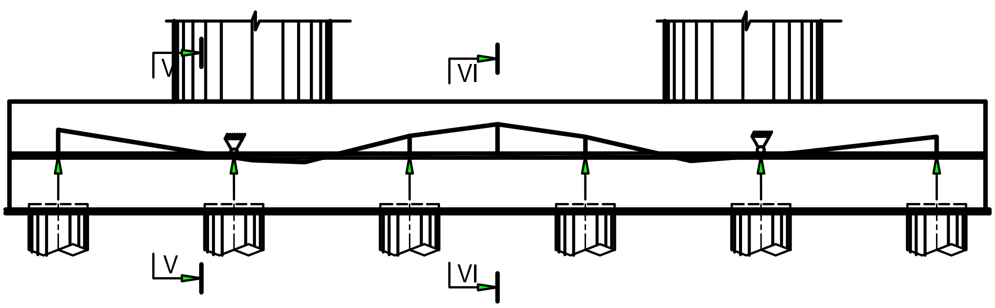
COMBINATION	Longitudinal direction		
	Q (kN)	M(kN.m)	V (kN.m)
Strength I	88376.4	282357.4	5457.0
Strength II	72399.1	226797.6	2004.0
Strength III	84673.6	269459.5	4624.5
Service I	67351.7	214632.3	3739.5
Service III	69388.8	222543.9	4276.5
Extreme I (CV)	71605.2	222049.4	1115.5
Extreme II (EQ trans)	81680.6	261262.0	4808.5
Extreme II (EQ long)	100477.77	334193.4	10663.00

At section V - V

COMBINATION	Transverse direction		
	Q (kN)	M(kN.m)	V (kN.m)
Strength I	27133.3	68100.2	-74.4
Strength II	27286.3	68864.1	-637.6
Strength III	27723.8	69736.9	-305.1
Service I	21941.1	55193.9	-247.8
Service III	21959.9	55241.0	-247.8
Extreme I (CV)	27257.4	68701.2	-1090.9
Extreme II (EQ trans)	29044.19	73396.6	-836.00
Extreme II (EQ long)	26634.39	67028.3	-460.91

At section VI - VI

COMBINATION	Transverse direction		
	Q (kN)	M(kN.m)	V (kN.m)
Strength I	28251.5	37356.2	-74.4
Strength II	23118.2	31726.2	-637.6
Strength III	28078.3	36045.4	-305.1
Service I	20738.5	28538.6	-247.8
Service III	21409.5	28846.1	-247.8
Extreme I (CV)	21730.5	32155.2	-1090.9
Extreme II (EQ trans)	33327.40	37446.70	-836.0
Extreme II (EQ long)	27742.20	34387.80	-460.9



2.8.4.2. Ultimate load check and shear capacity check and Crack control
*** Check flexure momen capacity of pile cap:**

Item			Section IV-IV (Bottom bar)	Section V-V (Bottom bar)	Section VI-VI (Upper bar)	Unit
• Factored Plexural moment		M_u	282357.44	69736.93	37356.20	kN.m
• Factored Shear force		V_u	88376.43	27723.77	28251.50	kN
• Hight of Section		h	3000	3000	3000	mm
• Width of section		b	25000	14800	14800	mm
• Section area		A_c	75000000	44400000	44400000	mm^2
• Moment of inertia of concrete section		I_g	5.6E+13	3.3E+13	3.3E+13	mm^4
• Tension reinforcement:	Distance from tension reinf. to extreme compression fiber	d_c	238	266	131	mm
	Reinf. Diameter	∅	38	28	36	mm
	Space	@	150	150	150	mm
	Number of bar	n	330	194	97	bar
	Total area of reinf.	A_s	374712	119702	99073	mm^2
• comp. reinforcement:	Distance from compressive reinf. to extreme Tension fiber		100	156	150	mm
	Diameter		38	36	28	mm
	Reinf. Space		150	150	150	mm
	Number of bar		165	97	194	bar
	Total area of reinf.	A'_s	168289	99073	119702	mm^2
Check Flexural Moment						
• Resistance factor		Φ	0.90	0.90	0.90	
• The corresponding effective		d_e	2762	2734	2869	mm
• Stress block factor		β_1	0.84	0.84	0.84	
• Depth of the equivalent stress block = c*β1		a	235.11	126.87	105.01	mm
• Distance from extreme compression fiber to the neutral axis		c	281.33	151.81	125.65	mm
• The nominal flexural resistance:		M_n	396361	127869	111616	kN.m
• Factored flexural resistance		$M_r = \Phi.M_n$	356725	115082	100454	kN.m
• Check condition		$M_r > M_u$	**O.K**	**O.K**	**O.K**	
Mimimum Reinforcement						
• Ratio of tension steel to gross area		$\rho = A_s/(b.d)$	0.54	0.30	0.23	%
• Check		$\rho > 0.03 \cdot f'_c/f'_y$	**O.K**	**O.K**	**O.K**	0.23
• Cracking moment		1.2Mcr	155279.35	91925.37	91925.37	Kn.m
• Check		Mr> min(1.2Mcr, 1.33Mu)	**O.K**	**O.K**	**O.K**	
Maximum Reinforcement						
• Obligation Condition		c/d_e	0.10	0.06	0.04	
• Check		$c/d_e < 0.42$	**O.K**	**O.K**	**O.K**	
Check shear resistance						
• Factored Shear force		V_u	88376.43	27723.77	28251.50	kN
• Resistance factor		Φ	0.90	0.90	0.90	
• The effective shear deepth		d_v	2644	2671	2816	mm
• Effective width		b_v	25000	14800	14800	mm
• Angle of inclination of diagonal compressive stress		θ	42	41	41	degree
• Angle of inclination of transerverse reinf. To longitudinal axis		α	90	90	90	degree
• Factor indicating ability of diagonally cracked concrete to transmit tension		β	1.75	1.95	1.95	
• Value		$0.1{*}f'_c{\cdot}b_v{\cdot}d_v$	198333	118573	125052	kN
• Max spacing of transverse reinforcement		s	600	600	600	mm
• Spacing of stirrup		s	300	450	450	mm
• Diameter of transverse reinforcement		∅	D 38	D 28	D 38	
• Number of transverse reinf. within distance s		n	4	4	2	bar
• Total area of transverse reinf.		A_v	4536	2463	2268	mm^2
• Diameter of stirrup		∅	D 20	D 20	D 20	mm
• Number of stirrup within distance s		n	84	32	32	bar
• Total area of stirrup		A_v	26494.10	10122.91	10122.91	
• Assume		θ	42.00	41.00	41.00	degree
• Strain in tensile reinforcement		ε_x	2.08E-03	1.76E-03	1.49E-03	
If ex<0, multiple with reduce factor		Fc	-	-	-	
• Ratio of shear stress and f'c		V/f'c	0.05	0.03	0.03	
• β final			1.75	1.95	1.95	
• θ final			43.00	43.00	43.00	degree
• The shear resistance of concrete:		V_c	52595.80	35037.91	36952.54	kN

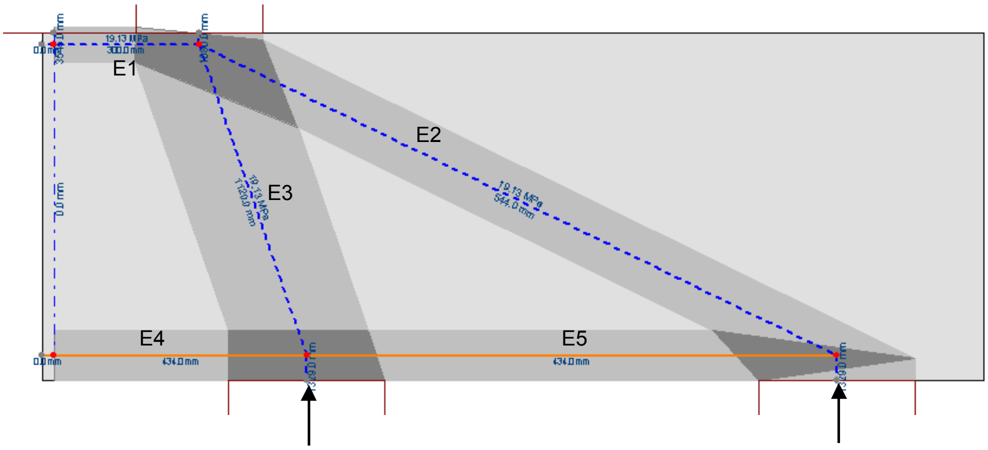
• The shear resistance of stirrup	V_s	50894.10	13945.93	14708.00	kN
• Value	$0.25*f'_c \cdot b_v \cdot d_v$	495833.14	296432.69	312631.16	kN
• The nominal shear resistance:	V_n	103489.90	48983.83	51660.54	kN
• The factored shear resistance	V_r	93140.91	44085.45	46494.49	kN
• Check	$V_r > V_u$	**O.K**	**O.K**	**O.K**	
•Requiring transverse reinforcement	$V_u > 0.5 \cdot \Phi\ V_c$	**Need**	**Need**	**Need**	
•Check minimum transverse reinforcement	$A_v > 0.083\ (f'_c{}^{0.5}) \cdot b_v \cdot s/f_y$	**O.K**	**O.K**	**O.K**	
Check Flexural and shear resistance at Extreme state					
• Factored Plexural moment	M_u	334193.41	73396.56	37446.70	kN.m
• Factored Shear force	V_u	100477.77	29044.19	33327.40	kN
• Resistance factor	Φ	1.00	1.00	1.00	
• The nominal flexural resistance:	M_n	396361	127869	111616	kN.m
• Factored flexural resistance	$M_r = \Phi.M_n$	396361	127869	111616	kN.m
• The nominal flexural resistance:	V_n	103490	48984	51661	Kn
• Factored flexural resistance	$V_r = \Phi.V_n$	103490	48984	51661	Kn
• Check condition	$M_r > M_u$	**O.K**	**O.K**	**O.K**	
	$V_r > V_u$	**O.K**	**O.K**	**O.K**	
Check crack					
Interior force combination	**Service III**				
• Factored moment	M_u	2.23E+05	5.52E+04	2.88E+04	kN.m
• Modulus of rupture of concrete	fr = 0.63*sqrt(f'c)	3.45	3.45	3.45	MPa
• Distance from extreme tension fiber to the neutral axis	$y_t = h-c$	2719	2848	2874	mm
• Stress of concrete at tension fiber	$f'r = Ms*y_t/I_g$	11	5	2	MPa
• Check	f'r >	0.8*fr	0.8*fr	0.8*fr	
		check crack	check crack	No check	
• Crack width parameter	Z	= 23000	= 23000	= 23000	N/mm
• Ratio of reinf. Modulus with concrete modulus	$n = E_s/E_c$	= 7.00	= 7.00	= 7.00	
• The distance from extreme fiber to the neutral axis	c	= 873.41	= 615.88	= 567.51	mm
• Effective moment of inertia	J	1.49E+13	4.91E+12	4.58E+12	mm^4
• Arm	de-c	= 1888.59	= 2118.12	= 2301.49	mm
• Tension stress in reinforcement	fs =n*Ms*(de-c)/J	= 197.35	= 166.75	= 101.58	MPa
• Area of concrete having the same centroid as the principal tensile reinfor	A	= 15133	= 15226	= 15205	mm2
• Tension stress in reinforcement with service state	$f_{sa} = Z/(d_c*A)^{1/3}$	= 226.71	= 231.99	= 227.45	Mpa
• Check condition	$f_s < f_{sa}$	**O.K**	**O.K**	**O.K**	
• Check condition	$f_s < 0.6*f_y$	**O.K**	**O.K**	**O.K**	

Check for pile cap of Pier P13
(Longitudinal direction)

General Properties

D-Region Thickness = 25000.0 mm
Concrete Cylinder Strength = 30.00 MPa
Non-Prestressed Reinforcement Yield Strength = 400.00 MPa

- **Truss Member Stress Limits and Effective Widths**



- **Summary of the Design**
  - Struts

Strut ID	F_u (kN)	β_s	ϕ	$\phi f_c = \phi(0.85)\beta_s f'_c$ (MPa)	**Effective Width** (mm)	ϕF_{ns} (kN)	Ratio $F_u/\phi F_{ns}$	Check $F_u/\phi F_{ns}<1$
E3	-38844.5	1.000	0.75	19.13	1120.0	535500.0	0.082	OK
E2	-106082.1	1.000	0.75	19.13	544.0	260100.0	0.508	OK

  - Ties

Tie ID	F_u (kN)	Required A_s (mm²)	Provided A_s (mm²)	ϕ	ϕF_{ns} (kN)	Ratio $F_u/\phi F_{ns}$	Check $F_u/\phi F_{ns}$ <1	Ties stress at Servic e state (Mpa)	Limit stress $0.6f_y$ =240 (Mpa)	Check $f_s<0.6f_y$
E5	105238	363877.7	376507.1	0.75	112952	0.96	OK	212.47	240	OK
E4	92676.1	318229.4	376507.1	0.75	112952	0.84	OK	186.91	240	OK

- Ties bar E4, E5 used 2x165 D38.

2.9 PIER P14
2.9 TRỤ CẦU P14

PIER P14 - CALCULATION SHEET

CONTENT:

2.9.1. GENERAL DATA

CALCULATION PROCEDURE & STANDARD:

- Bridge Design Standard 22 TCN - 272 - 05

2.9.1.1. Design live load

Design vehicle load		HL93	22TCN 272 - 05
Number of lane		6	(lane)
Pedestrian		0.00	KG/m^2

2.9.1.2. Bridge width

Width carriageway	B_{xe} =	12.00	(m)
Width of median guardrail	B_{pc} =	0.50	(m)
Width parapet	B_{lc} =	0.50	(m)
Bridge width	B =	13.00	(m)

2.9.1.3. Superstructure:

Span-arrangement		Continuous box girder 65+5@100+65m	
Height of box girder at pier section	H =	6.00	(m)
Height of box girder at mid-span section	h =	2.50	(m)
Pavement thickness	d_{BTN} =	0.084	(m)

2.9.1.4. Material property:

Concrete

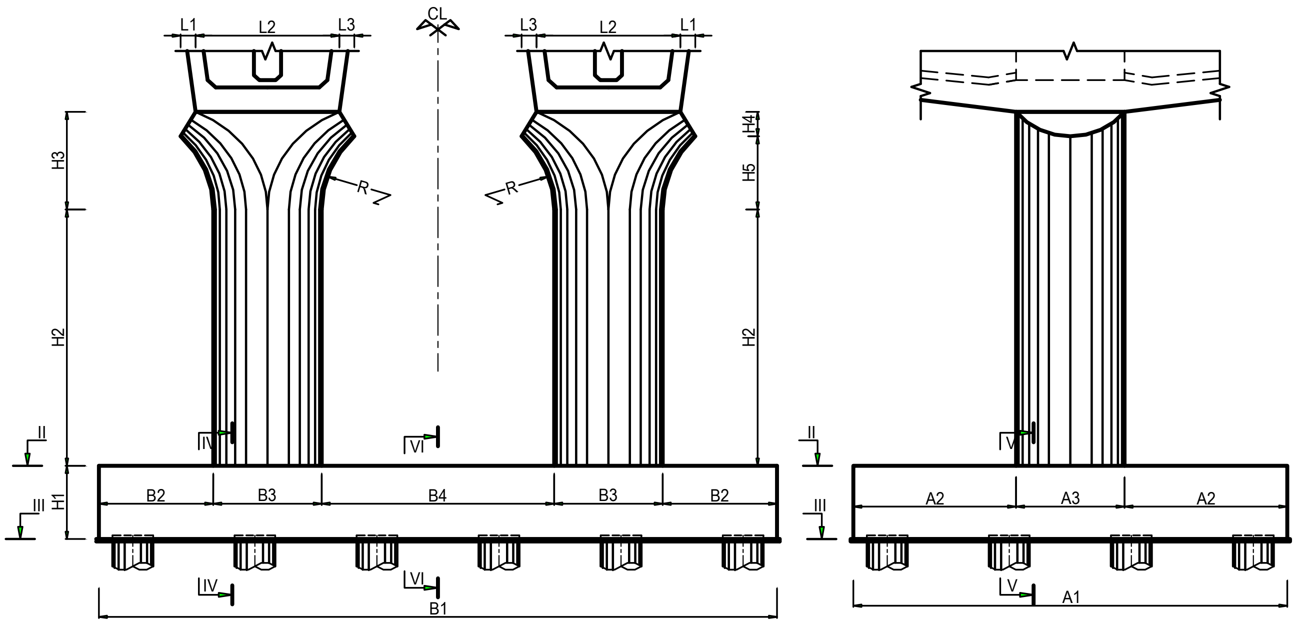
Compressive strength of cylindrical at 28 day	f 'c =	30.00	MPa
Concrete density	g =	24.50	KN/m^3
Elastic modulus	Ec =	29440	MPa
Tension strength of concrete	fr =	3.45	MPa

Steel

Steel modulus	Es =	200000	MPa
Yeild strength of steel bar	fy =	400.00	MPa
Calculation unit:		KN, m, KN.m, Mpa, KN/m^2	

2.9.1.5. THE PIER GEOMETRIC

GENERAL DIMENSIONS



vertical			Horizontal			Thickness		
Remark	Value	Unit	Remark	Value	Unit	Remark	Value	Unit
a_1 =	14.80	(m)	L_1 =	0.55	(m)	h_1 =	3.00	(m)
a_2 =	5.40	(m)	L_2 =	5.30	(m)	h_2 =	12.000	(m)
a_3 =	4.00	(m)	L_3 =	0.55	(m)	h_3 =	4.000	(m)
			B_1 =	25.00	(m)	h_4 =	1.000	(m)
			B_2 =	3.965	(m)	h_5 =	3.000	(m)
			B_3 =	4.00	(m)			
			B_4 =	9.070	(m)			
			R =	4.34	(m)			

The design elevation:

Proposed height	EL_{mc} =	19.310	(m)			
Elevation of top of pier cap	EL_{xm} =	13.105	(m)			
Hight water level (H1%)	EL_{MNTK} =	9.200	(m)			
Daily water level (H5%)	EL_{MNTB} =	4.070	(m)			
Ground elevation	EL_{TN} =	1.080	(m)			
Daily water level (H5%)	EL_{TT} =	4.070	(m)			
Top of Pile cap (Section II - II)	$EL_{MC\ II\text{-}II}$ =	-2.895	(m)			
Bottom of Pile cap (Section III - III)	$EL_{MCIII\text{-}III}$ =	-5.895	(m)			

2.9.2. CALCULATED PIER FORCE AND PILE FORCE

2.9.2.1. Loading combination at bottom of pier shaft

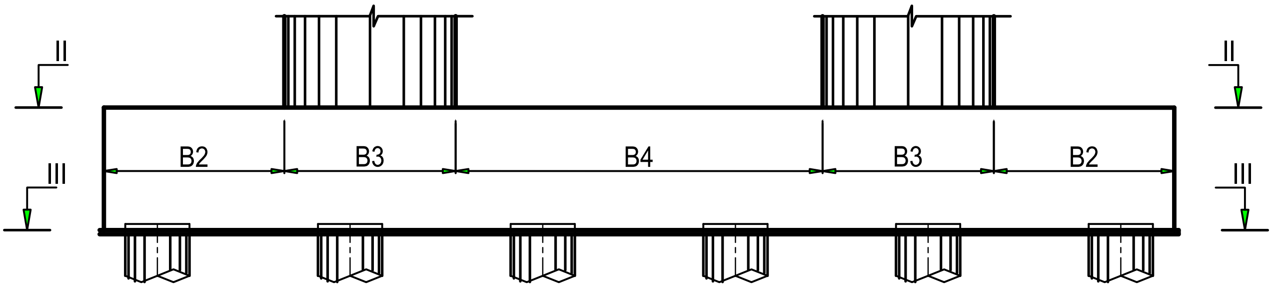
(Result from RM2010 SPACE-FRAME program)

Loading combination in accordance with 22 TCN 272-05, table 3.4.1.1 was mentioned in appendix of this report

LOADING COMBINATION AT TOP OF PILE CAP DUE TO SUPERSTRUCTURE + VESSEL COLLISION AND SEFTWEIGHT						
Limited state	**Vertical axis**	**Longitudinal axis**		**Horizontal axis**		**Remarks**
	N_x (kN)	**Q_y (kN)**	**M_z (kNm)**	**Q_z (kN)**	**M_y (kNm)**	
STRENGTH I	54600.00	5364.00	39912.00	207.00	7356.00	
STRENGTH II	46636.00	1988.00	23723.00	1978.00	38139.00	
STRENGTH III	52780.00	4508.00	36212.00	883.00	20128.00	
SERVICE I	41451.00	3712.00	30030.00	716.00	16093.00	
SERVICE III	41498.00	4249.00	36000.00	716.00	16092.00	
EXTREME EVENT (CV)	48590.00	5022.00	24277.00	5900.00	34393.00	
EXTREME EVENT (EQ trans)	48766.00	4563.00	34786.00	2437.00	48333.00	
EXTREME EVENT (EQ long)	49377.00	9751.00	77478.00	1279.00	19539.00	

2.8.2.2. Loading combination at bottom of pile cap

LOADING COMBINATION AT TOP OF PILE CAP DUE TO SUPERSTRUCTURE + VESSEL						
Limited state	**Vertical axis**	**Longitudinal axis**		**Horizontal axis**		**Remarks**
	N_x (kN)	**Q_y (kN)**	**M_z (kNm)**	**Q_z (kN)**	**M_y (kNm)**	
STRENGTH I	144745	10766	104341	440	14921	
STRENGTH II	128816	4013	58905	3982	88185	
STRENGTH III	141104	9053	93955	1792	44767	
SERVICE I	111098	7461	77748	1457	35905	
SERVICE III	111192	8535	92911	1457	35904	
EXTREME EVENT (CV)	132725	5040	38186	5913	51960	
EXTREME EVENT (EQ trans)	133076	9163	94638	4899	111018	
EXTREME EVENT (EQ long)	134299	19869	211644	2815	46832	



2.9.2.3. Loading combination at top of piling

2.9.2.3.1. Piling material:

Concrete 30 Mpa

E_c (kg/cm^2)	294401
γ_c (T/m^3)	2.5

Steel bar

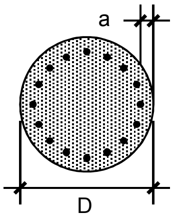
Type	CB-400-T
E_s (kg/cm^2)	200000

2.9.2.3.2. Piling dimension

+ Diameter **D** = 1.50 m

a = 0.100 m

+ Length **L** = 56.00 m

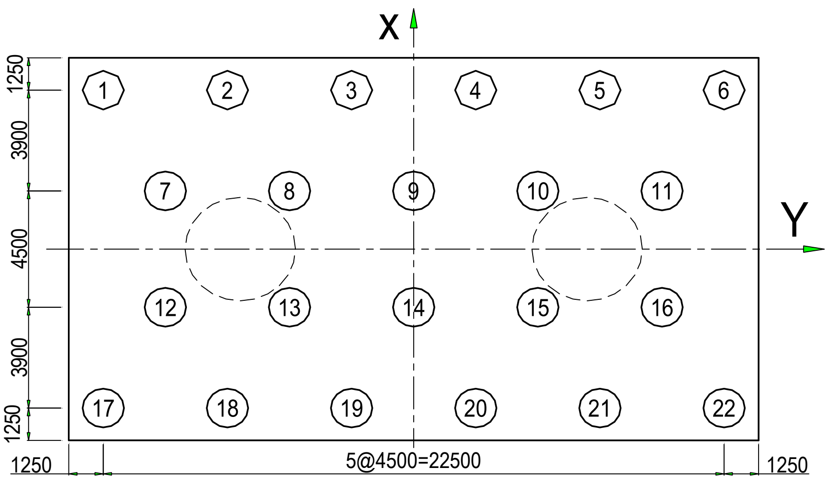


2.9.2.3.3. Maximim Internal force and displacement at top piling

Maximum Internal force and diplacements *(Result follow Piling software)*

Combination	N	Hx	My	x	y	z
	(KN)	(KN)	(KN.m)	(m)	(m)	(rad)
Strength I	8916.86	489.36	2939.11	-	-	-
Strength II	8096.30	182.41	1070.38	-	-	-
Strength III	8881.87	411.50	2463.65	-	-	-
Service I	7094.16	339.14	2030.01	**0.015**	**-0.003**	**0.009**
Service III	7360.30	387.96	2317.23	**0.018**	**-0.003**	**0.009**
Extreme I (CV)	7869.95	229.09	1389.33	-	-	-
Extreme II (EQ trans)	9394.09	416.50	2494.16	-	-	-
Extreme II (EQ long)	10881.21	903.14	5400.23	-	-	-

- Check displacement of top pile not exceed 38mm (10.7.2.7) **OK**



Arrangement of pile

2.9.2.3.4. Internal force for each pile

Combination	Pile Number	Hx	Hy	N	Mx	My
		(KN)	(KN)	(KN.m)	(m)	(m)
Strength I	1	489.36	-20.00	4796.46	118.78	2939.11
	2	489.36	-20.00	4725.62	118.78	2939.11
	3	489.36	-20.00	4654.77	118.78	2939.11
	4	489.36	-20.00	4583.93	118.78	2939.11
	5	489.36	-20.00	4513.08	118.78	2939.11
	6	489.36	-20.00	4442.24	118.78	2939.11
	7	489.36	-20.00	6067.51	118.78	2939.11
	8	489.36	-20.00	5996.66	118.78	2939.11
	9	489.36	-20.00	5925.81	118.78	2939.11
	10	489.36	-20.00	5854.97	118.78	2939.11
	11	489.36	-20.00	5784.12	118.78	2939.11
	12	489.36	-20.00	7574.97	118.78	2939.11
	13	489.36	-20.00	7504.12	118.78	2939.11
	14	489.36	-20.00	7433.28	118.78	2939.11
	15	489.36	-20.00	7362.43	118.78	2939.11
	16	489.36	-20.00	7291.59	118.78	2939.11
	17	489.36	-20.00	8916.86	118.78	2939.11
	18	489.36	-20.00	8846.01	118.78	2939.11
	19	489.36	-20.00	8775.17	118.78	2939.11
	20	489.36	-20.00	8704.32	118.78	2939.11
	21	489.36	-20.00	8633.48	118.78	2939.11
	22	489.36	-20.00	8562.63	118.78	2939.11
Strength II	1	182.41	-181.00	6086.02	1102.06	1070.38
	2	182.41	-181.00	5631.76	1102.06	1070.38
	3	182.41	-181.00	5177.49	1102.06	1070.38
	4	182.41	-181.00	4723.23	1102.06	1070.38
	5	182.41	-181.00	4268.96	1102.06	1070.38
	6	182.41	-181.00	3814.70	1102.06	1070.38
	7	182.41	-181.00	6496.29	1102.06	1070.38
	8	182.41	-181.00	6042.03	1102.06	1070.38
	9	182.41	-181.00	5587.77	1102.06	1070.38
	10	182.41	-181.00	5133.50	1102.06	1070.38
	11	182.41	-181.00	4679.24	1102.06	1070.38
	12	182.41	-181.00	7231.76	1102.06	1070.38
	13	182.41	-181.00	6777.50	1102.06	1070.38
	14	182.41	-181.00	6323.23	1102.06	1070.38
	15	182.41	-181.00	5868.97	1102.06	1070.38
	16	182.41	-181.00	5414.71	1102.06	1070.38
	17	182.41	-181.00	8096.30	1102.06	1070.38
	18	182.41	-181.00	7642.04	1102.06	1070.38
	19	182.41	-181.00	7187.77	1102.06	1070.38
	20	182.41	-181.00	6733.51	1102.06	1070.38
	21	182.41	-181.00	6279.24	1102.06	1070.38
	22	182.41	-181.00	5824.98	1102.06	1070.38

- Result for internal force at top pile

Combination	Pile Number	**Hx** (KN)	**Hy** (KN)	**N** (KN.m)	**Mx** (m)	**My** (m)
Strength III	1	411.50	-81.46	5269.72	493.02	2463.65
	2	411.50	-81.46	5045.02	493.02	2463.65
	3	411.50	-81.46	4820.32	493.02	2463.65
	4	411.50	-81.46	4595.62	493.02	2463.65
	5	411.50	-81.46	4370.92	493.02	2463.65
	6	411.50	-81.46	4146.22	493.02	2463.65
	7	411.50	-81.46	6302.69	493.02	2463.65
	8	411.50	-81.46	6077.99	493.02	2463.65
	9	411.50	-81.46	5853.29	493.02	2463.65
	10	411.50	-81.46	5628.59	493.02	2463.65
	11	411.50	-81.46	5403.89	493.02	2463.65
	12	411.50	-81.46	7624.21	493.02	2463.65
	13	411.50	-81.46	7399.50	493.02	2463.65
	14	411.50	-81.46	7174.80	493.02	2463.65
	15	411.50	-81.46	6950.10	493.02	2463.65
	16	411.50	-81.46	6725.40	493.02	2463.65
	17	411.50	-81.46	8881.87	493.02	2463.65
	18	411.50	-81.46	8657.17	493.02	2463.65
	19	411.50	-81.46	8432.47	493.02	2463.65
	20	411.50	-81.46	8207.77	493.02	2463.65
	21	411.50	-81.46	7983.07	493.02	2463.65
	22	411.50	-81.46	7758.37	493.02	2463.65
Service I	1	339.14	-66.23	4109.75	401.14	2030.01
	2	339.14	-66.23	3929.02	401.14	2030.01
	3	339.14	-66.23	3748.29	401.14	2030.01
	4	339.14	-66.23	3567.57	401.14	2030.01
	5	339.14	-66.23	3386.84	401.14	2030.01
	6	339.14	-66.23	3206.11	401.14	2030.01
	7	339.14	-66.23	4965.66	401.14	2030.01
	8	339.14	-66.23	4784.93	401.14	2030.01
	9	339.14	-66.23	4604.21	401.14	2030.01
	10	339.14	-66.23	4423.48	401.14	2030.01
	11	339.14	-66.23	4242.75	401.14	2030.01
	12	339.14	-66.23	6057.52	401.14	2030.01
	13	339.14	-66.23	5876.79	401.14	2030.01
	14	339.14	-66.23	5696.07	401.14	2030.01
	15	339.14	-66.23	5515.34	401.14	2030.01
	16	339.14	-66.23	5334.61	401.14	2030.01
	17	339.14	-66.23	7094.16	401.14	2030.01
	18	339.14	-66.23	6913.43	401.14	2030.01
	19	339.14	-66.23	6732.71	401.14	2030.01
	20	339.14	-66.23	6551.98	401.14	2030.01
	21	339.14	-66.23	6371.25	401.14	2030.01
	22	339.14	-66.23	6190.52	401.14	2030.01

- Result for internal force at top pile

Combination	Pile Number	**Hx** (KN)	**Hy** (KN)	**N** (KN.m)	**Mx** (m)	**My** (m)
Service III	1	387.96	-66.23	3852.14	401.14	2317.23
	2	387.96	-66.23	3671.42	401.14	2317.23
	3	387.96	-66.23	3490.69	401.14	2317.23
	4	387.96	-66.23	3309.97	401.14	2317.23
	5	387.96	-66.23	3129.25	401.14	2317.23
	6	387.96	-66.23	2948.52	401.14	2317.23
	7	387.96	-66.23	4874.12	401.14	2317.23
	8	387.96	-66.23	4693.40	401.14	2317.23
	9	387.96	-66.23	4512.67	401.14	2317.23
	10	387.96	-66.23	4331.95	401.14	2317.23
	11	387.96	-66.23	4151.23	401.14	2317.23
	12	387.96	-66.23	6157.59	401.14	2317.23
	13	387.96	-66.23	5976.87	401.14	2317.23
	14	387.96	-66.23	5796.15	401.14	2317.23
	15	387.96	-66.23	5615.42	401.14	2317.23
	16	387.96	-66.23	5434.70	401.14	2317.23
	17	387.96	-66.23	7360.30	401.14	2317.23
	18	387.96	-66.23	7179.57	401.14	2317.23
	19	387.96	-66.23	6998.85	401.14	2317.23
	20	387.96	-66.23	6818.13	401.14	2317.23
	21	387.96	-66.23	6637.40	401.14	2317.23
	22	387.96	-66.23	6456.68	401.14	2317.23
Extreme Event I (with CV)	1	229.09	-268.77	6193.74	1682.18	1389.33
	2	229.09	-268.77	5834.28	1682.18	1389.33
	3	229.09	-268.77	5474.81	1682.18	1389.33
	4	229.09	-268.77	5115.34	1682.18	1389.33
	5	229.09	-268.77	4755.88	1682.18	1389.33
	6	229.09	-268.77	4396.41	1682.18	1389.33
	7	229.09	-268.77	6545.49	1682.18	1389.33
	8	229.09	-268.77	6186.02	1682.18	1389.33
	9	229.09	-268.77	5826.56	1682.18	1389.33
	10	229.09	-268.77	5467.09	1682.18	1389.33
	11	229.09	-268.77	5107.63	1682.18	1389.33
	12	229.09	-268.77	7158.74	1682.18	1389.33
	13	229.09	-268.77	6799.27	1682.18	1389.33
	14	229.09	-268.77	6439.81	1682.18	1389.33
	15	229.09	-268.77	6080.34	1682.18	1389.33
	16	229.09	-268.77	5720.87	1682.18	1389.33
	17	229.09	-268.77	7869.95	1682.18	1389.33
	18	229.09	-268.77	7510.49	1682.18	1389.33
	19	229.09	-268.77	7151.02	1682.18	1389.33
	20	229.09	-268.77	6791.55	1682.18	1389.33
	21	229.09	-268.77	6432.09	1682.18	1389.33
	22	229.09	-268.77	6072.62	1682.18	1389.33

- Result for internal force at top pile

Combination	Pile Number	**Hx** (KN)	**Hy** (KN)	**N** (KN.m)	**Mx** (m)	**My** (m)
Extreme Event I (with EQtran)	1	416.50	-222.68	5748.93	1354.39	2494.16
	2	416.50	-222.68	5179.98	1354.39	2494.16
	3	416.50	-222.68	4611.03	1354.39	2494.16
	4	416.50	-222.68	4042.08	1354.39	2494.16
	5	416.50	-222.68	3473.13	1354.39	2494.16
	6	416.50	-222.68	2904.19	1354.39	2494.16
	7	416.50	-222.68	6620.23	1354.39	2494.16
	8	416.50	-222.68	6051.29	1354.39	2494.16
	9	416.50	-222.68	5482.34	1354.39	2494.16
	10	416.50	-222.68	4913.39	1354.39	2494.16
	11	416.50	-222.68	4344.44	1354.39	2494.16
	12	416.50	-222.68	7953.83	1354.39	2494.16
	13	416.50	-222.68	7384.88	1354.39	2494.16
	14	416.50	-222.68	6815.94	1354.39	2494.16
	15	416.50	-222.68	6246.99	1354.39	2494.16
	16	416.50	-222.68	5678.04	1354.39	2494.16
	17	416.50	-222.68	9394.09	1354.39	2494.16
	18	416.50	-222.68	8825.14	1354.39	2494.16
	19	416.50	-222.68	8256.19	1354.39	2494.16
	20	416.50	-222.68	7687.25	1354.39	2494.16
	21	416.50	-222.68	7118.30	1354.39	2494.16
	22	416.50	-222.68	6549.35	1354.39	2494.16
Extreme Event I (with EQlong)	1	903.14	-127.96	2824.59	788.05	5400.23
	2	903.14	-127.96	2565.32	788.05	5400.23
	3	903.14	-127.96	2306.05	788.05	5400.23
	4	903.14	-127.96	2046.78	788.05	5400.23
	5	903.14	-127.96	1787.51	788.05	5400.23
	6	903.14	-127.96	1528.24	788.05	5400.23
	7	903.14	-127.96	5249.49	788.05	5400.23
	8	903.14	-127.96	4990.23	788.05	5400.23
	9	903.14	-127.96	4730.96	788.05	5400.23
	10	903.14	-127.96	4471.69	788.05	5400.23
	11	903.14	-127.96	4212.42	788.05	5400.23
	12	903.14	-127.96	8197.04	788.05	5400.23
	13	903.14	-127.96	7937.77	788.05	5400.23
	14	903.14	-127.96	7678.50	788.05	5400.23
	15	903.14	-127.96	7419.23	788.05	5400.23
	16	903.14	-127.96	7159.96	788.05	5400.23
	17	903.14	-127.96	10881.21	788.05	5400.23
	18	903.14	-127.96	10621.94	788.05	5400.23
	19	903.14	-127.96	10362.67	788.05	5400.23
	20	903.14	-127.96	10103.40	788.05	5400.23
	21	903.14	-127.96	9844.13	788.05	5400.23
	22	903.14	-127.96	9584.86	788.05	5400.23

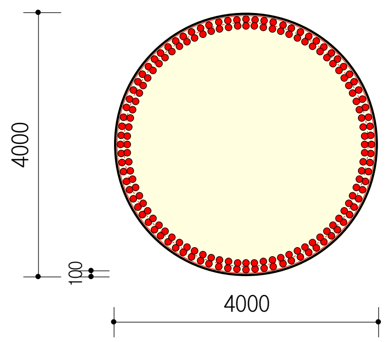
2.9.3. CHECK FOR PIER SHAFT

Item		Mark	Unit	Value
• Factored Axial force		Nu	Kn	41498.00
• Factored Plexural moment		Mux	Kn.m	36000.00
• Factored Plexural moment		Muy	Kn.m	16092.00
• Diameter of Pier shaft		D	m	4.00
• Section area		Ag	m2	12.57
• Moment of inertia of concrete section		Ic	m4	12.57
• Cover thickness		a	m	0.075
• Reinf. Diameter		Ds	mm	32.00
• Number of rebar		n_s	nos	168.00
• Rebar area		As	mm2	135113.62
Check minimum reinforcement				
• Minimum rebar area required (0.135*f'c/fy)*Ag		As req	mm2	127234.50
• Check condition As > (0.135*f'c/fy)*Ag				**OK**
Check maximum reinforcement				
• Maximum rebar area 0.08*Ag		As max	mm2	1005309.6
• Check condition As < 0.08*Ag				**OK**
Check ratio spiral or Tier (5.7.4.6)				
• Distance to outside of Spairal or Ties to concrete face			mm	66.00
• Effect diamete		Deff	m	3.87
• Area of core measured to the outside diameter of the spiral			m2	11.75
• Ratio spiral Rebar required		ρsa		0.00234
Required Area of Spiral Rebar	space		mm	200
	Effective length			3.87
	layer			2
	Area			226.6
	Requaired Dhs			17.0
Actuaral	Effective length	d	m	3.868
	Diameter	Dhr	mm	18
	Area of Rebar	Ah	mm2	254.5
	layer	Nl	nos	2
	Total area of spiral	Ac	m2	508.938
	space	s	mm	200
	Ratio spiral Rebar	ρs	-	0.0026315
• Check condition		$\rho_s > \rho_{sa}$		OK
Check Crack (at Service state)				
• Modulus of rupture of concrete		fr = 0.63*sqrt(f'c)	Mpa	3.45
• Stress of concrete at tension fiber		σ'_r	Mpa	1.80
• If f'r > 0.8fr require check crack		$\sigma'_r > 0.8*\sigma_r$	Mpa	No check
• Center of newtral axial		x	mm	3.08
• Maximum stress of Compression fiber of concrete		σ_c	Mpa	7.6
• Maximum stress of Compression Rebar		σ_{rc}	Mpa	-110.9
• Maximum stress of Tension Rebar		σ_{rt}	Mpa	30.6
• Check			σ_{rt} <0.6.fy	OK

	Company	Nipponkoei	**Project Name**	DN Qn Expressway
	Designer	vinh	**File Name**	D:\...\bodypier-D4.0m-P14.BOI

1. Geometry and Materials

Design Code : ACI318M-02
Stress Profile : Equivalent Stress Block
Material Data : f_c' = 30 MPa (β_1 = 0.850)
f_y = 400, f_{ys} = 400 MPa
Section Dim. : Φ4000 mm
Effective Len. : KL_u = 16000 mm
Steel Distribut.: 84 - D32 (d_c = 100 mm)
: 84 - D32 (d_c = 200 mm)
Total Steel Area A_{st} = 133426 mm^2 (ρ_{st} = 0.0106)



2. Member Force and Moment

Unit : kN, kN-m

L.C.	P_u	M_{ux}	M_{uy}	R_{atioV}	V_{ux}	V_{uy}	R_{atioH}	Remark
1	54600.0	39912.0	7356.0	0.379	5364.0	207.0	0.170	
2	46636.0	23723.0	38139.0	0.385	1988.0	1978.0	0.089	
3	52780.0	36212.0	20128.0	0.379	4508.0	883.0	0.145	
4	41451.0	30030.0	16093.0	0.306	3712.0	716.0	0.120	
5	41498.0	36000.0	16092.0	0.339	4249.0	716.0	0.137	
6	48590.0	24277.0	34393.0	0.372	5022.0	5900.0	0.245	
7	48766.0	34786.0	48333.0	0.479	4563.0	2437.0	0.164	
8	49377.0	77478.0	19539.0	0.583	9751.0	1279.0	0.311	

3. Magnified Moment

KL_u/r_x = 16000/1000 = 16.00 < 34-12(M_1/M_2) = 22.00
δ_x = 1.000

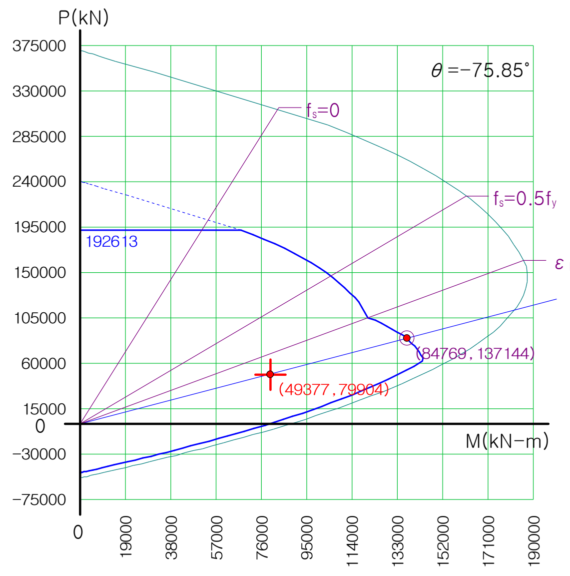
KL_u/r_y = 16000/1000 = 16.00 < 34-12(M_1/M_2) = 22.00
δ_y = 1.000

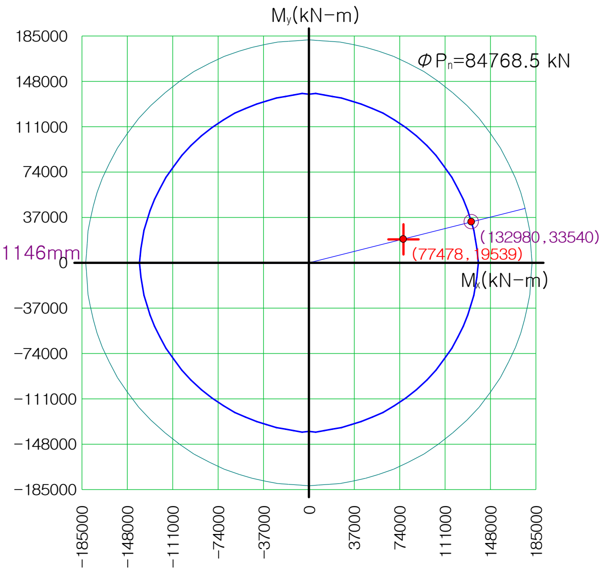
4. Design Force and Moment

Design Load Combination No : 8
P_u = 49377.0 kN
M_{ux} = 77478.0, M_{uy} = 19539.0 kN-m

5. Check Axial and Moment Capacity

Rotation Angle and Depth to the Neutral Axis θ = -75.85°, c = 1895 mm
Strength Reduction Factor Φ = 0.7533
Maximum Axial Load $\Phi P_{n(max)}$ = 192613.4 kN
Design Axial Load Strength ΦP_n = 84768.5 kN
Design Moment Strength ΦM_{nx} = 132979.8 kN-m
ΦM_{ny} = 33539.9 kN-m
Strength Ratio : Applied/Design = 0.583 < 1.000 .... O.K





6. Check Shear Capacity

Design Load Combination No : 8

Strength Reduction Factor Φ= 0.750

Design Force Vu = 9834.5 kN (Pu = 49377.0 kN)

Required Hoop Spacing : D18 @ 508 mm

Provided Hoop Spacing : D18 @ 150 mm (Tie)

$\Phi V_c + \Phi V_s = 27940.0 + 3646.1 = 31586.1$ kN > $V_u = 9834.5$ kN .... O.K

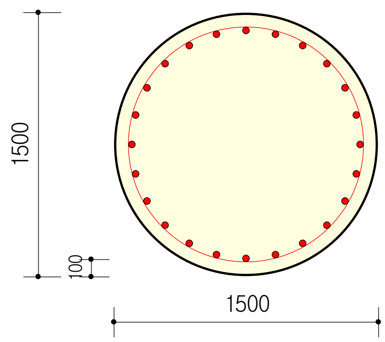
2.9.4. CHECK FOR PILE

Item		Mark	Unit	Value
• Factored Axial force		Nu	Kn	6732.71
• Factored Plexural moment		Mux	Kn.m	401.14
• Factored Plexural moment		Muy	Kn.m	2030.01
• Diameter of Pier shaft		D	m	1.50
• Section area		Ag	m2	1.77
• Moment of inertia of concrete section		Ic	m4	0.25
• Cover thickness		a	m	0.075
• Reinf. Diameter		Ds	mm	D32
• Number of rebar		n_s	nos	24
• Rebar area		As	mm2	19301.95
Check minimum reinforcement				
• Minimum rebar area required (0.135*f'c/fy)*Ag		As req	mm2	17892.35
• Check condition As > (0.135*f'c/fy)*Ag				**OK**
Check maximum reinforcement				
• Maximum rebar area 0.08*Ag		As max	mm2	141371.7
• Check condition As < 0.08*Ag				**OK**
Check ratio spiral or Tier (5.7.4.6)				
• Distance to outside of Spairal or Ties to concrete face			mm	68.00
• Effect diamete		Deff	m	1.36
• Area of core measured to the outside diameter of the spiral			m2	1.46
• Ratio spiral Rebar required		ρsa		0.00707
Required Area of Spiral Rebar	space		mm	75
	Effective length			1.36
	layer			1
	Area			180.7
	Requaired Dhs			15.2
Actuaral	Effective length	d	m	1.364
	Diameter	Dhr	mm	16
	Area of Rebar	Ah	mm2	201.1
	layer	Nl	nos	1
	Total area of spiral	Ac	m2	201.062
	space	s	mm	75
	Ratio spiral Rebar	ρs	-	0.0078617
• Check condition		$\rho_s > \rho_{sa}$		OK

Company	Nipponkoei	**Project Name**	DN Qn Expressway
Designer	vinh	**File Name**	

1. Geometry and Materials

Design Code : ACI318M-02
Stress Profile : Equivalent Stress Block
Material Data : f_c' = 30 MPa (β_1 = 0.850)
f_y = 400, f_{ys} = 400 MPa
Section Dim. : Φ1500 mm
Effective Len. : KL_u = 15000 mm
Steel Distribut.: 24 - D32 (d_c = 100 mm)
Total Steel Area A_{st} = 19061 mm^2 (ρ_{st} = 0.0108)



2. Member Force and Moment

Unit : kN, kN-m

L.C.	P_u	M_{ux}	M_{uy}	R_{atioV}	V_{ux}	V_{uy}	R_{atioH}	Remark
1	8916.9	118.8	2939.1	0.536	489.4	20.0	0.102	
2	8096.3	1102.1	1070.4	0.346	182.4	181.0	0.054	
3	8881.9	493.0	2463.7	0.473	411.5	81.5	0.087	
4	7094.2	401.1	2030.0	0.381	339.1	66.2	0.072	
5	7360.3	401.1	2317.2	0.423	388.0	66.2	0.082	
6	7869.9	1682.2	1389.3	0.411	229.1	268.8	0.074	
7	6247.0	1354.4	2494.2	0.468	416.5	222.7	0.099	
8	10881.2	788.0	5400.2	0.896	903.1	128.0	0.189	

3. Magnified Moment

KL_u/r_x = 15000/375 = 40.00 > 34-12(M_1/M_2) = 22.00
δ_x = MAX[1.00/(1-P_u/0.75/145156), 1.0] = 1.111

KL_u/r_y = 15000/375 = 40.00 > 34-12(M_1/M_2) = 22.00
δ_y = MAX[1.00/(1-P_u/0.75/145156), 1.0] = 1.111

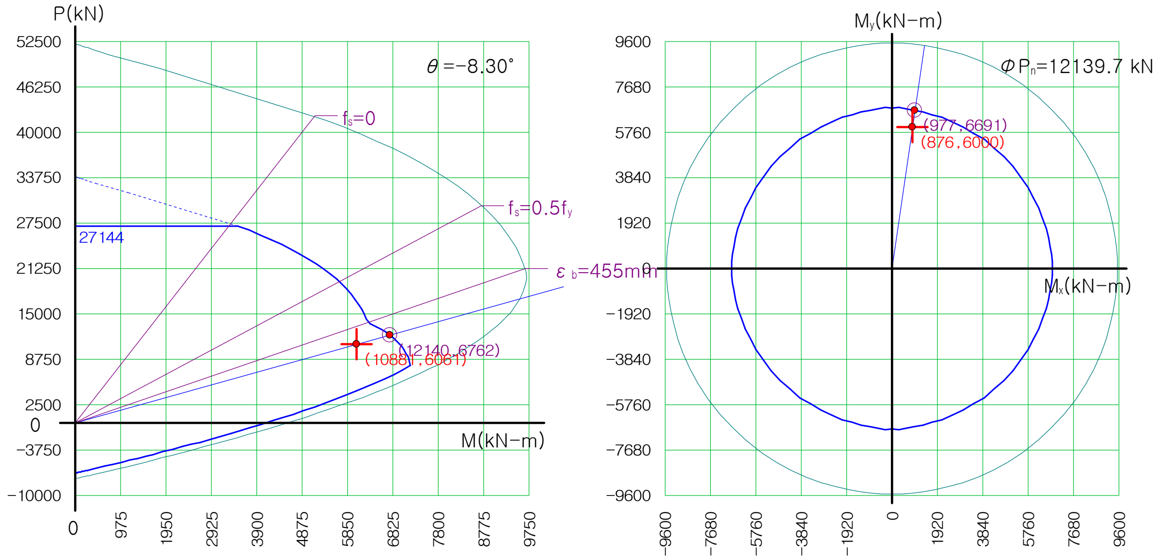
4. Design Force and Moment

Design Load Combination No : 8
P_u = 10881.2 kN
M_{ux} = 788.0, M_{uy} = 5400.2 kN-m
$\delta_x M_{ux}$= δ_x*M_{ux} = 875.6 kN-m
$\delta_y M_{uy}$= δ_y*M_{uy}, = 5999.9 kN-m

5. Check Axial and Moment Capacity

Rotation Angle and Depth to the Neutral Axis θ = -8.30°, c = 744 mm
Strength Reduction Factor Φ = 0.7072
Maximum Axial Load $\Phi P_{n(max)}$ = 27144.3 kN
Design Axial Load Strength ΦP_n = 12139.7 kN
Design Moment Strength ΦM_{nx} = 976.8 kN-m
ΦM_{ny} = 6690.9 kN-m
Strength Ratio : Applied/Design = 0.896 < 1.000 .... O.K

	Company	Nipponkoei	**Project Name**	DN Qn Expressway
	Designer	vinh	**File Name**	



6. Check Shear Capacity

Design Load Combination No : 8

Strength Reduction Factor Φ= 0.750

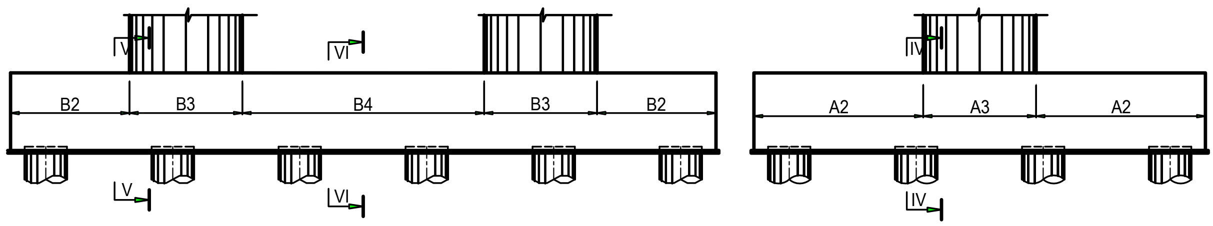
Design Force Vu = 912.2 kN (Pu = 10881.2 kN)

Required Hoop Spacing : D16 @ 508 mm

Provided Hoop Spacing : D16 @ 150 mm (Tie)

$\Phi V_c + \Phi V_s$ = 3901.6 + 924.5 = 4826.2 kN > V_u = 912.2 kN .... O.K

2.9.4. CHECK FOR PILE CAP
2.9.4.1. The Force to section IV-IV, section V-V, section VI-VI



At section IV - IV

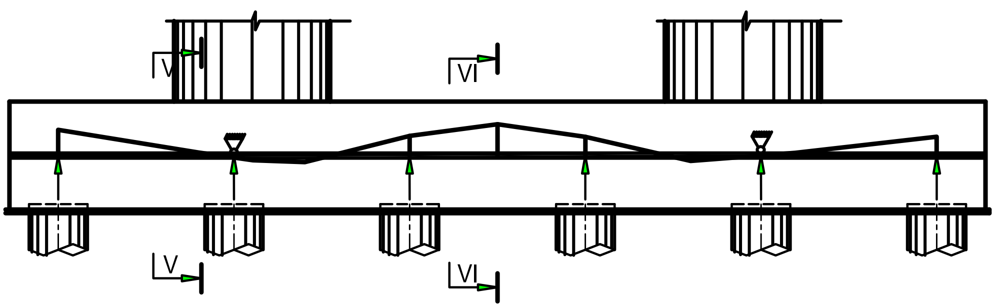
COMBINATION	Longitudinal direction		
	Q (kN)	M(kN.m)	V (kN.m)
Strength I	89604.8	286677.6	5383.0
Strength II	73380.0	230168.4	2006.5
Strength III	85794.7	273364.5	4526.5
Service I	68334.4	218093.5	3730.5
Service III	70431.7	226244.5	4267.5
Extreme I (CV)	74026.7	231010.6	2520.0
Extreme II (EQ trans)	81910.0	261649.7	4581.5
Extreme II (EQ long)	99790.71	330961.1	9934.50

At section V - V

COMBINATION	Transverse direction		
	Q (kN)	M(kN.m)	V (kN.m)
Strength I	27355.8	68660.7	-80.0
Strength II	27910.4	70482.4	-724.0
Strength III	28078.5	70645.6	-325.8
Service I	22227.1	55926.6	-264.9
Service III	22244.1	55969.4	-264.9
Extreme I (CV)	27767.9	70018.6	-1075.1
Extreme II (EQ trans)	29717.08	75140.8	-890.73
Extreme II (EQ long)	27152.33	68362.6	-511.82

At section VI - VI

COMBINATION	Transverse direction		
	Q (kN)	M(kN.m)	V (kN.m)
Strength I	28658.9	37562.3	-80.0
Strength II	23252.3	31825.6	-724.0
Strength III	28121.2	36123.0	-325.8
Service I	20795.3	28595.2	-264.9
Service III	21492.3	28892.2	-264.9
Extreme I (CV)	21798.3	32215.2	-1075.1
Extreme II (EQ trans)	33412.20	37485.20	-890.7
Extreme II (EQ long)	27812.30	34452.20	-511.8



2.9.4.2. Ultimate load check and shear capacity check and Crack control

*** Check flexure momen capacity of pile cap:**

Item			Section IV-IV (Bottom bar)	Section V-V (Bottom bar)	Section VI-VI (Upper bar)	Unit
• Factored Plexural moment		M_u	286677.62	70645.56	37562.25	kN.m
• Factored Shear force		V_u	89604.84	28078.49	28658.90	kN
• Hight of Section		h	3000	3000	3000	mm
• Width of section		b	25000	14800	14800	mm
• Section area		A_c	75000000	44400000	44400000	mm^2
• Moment of inertia of concrete section		I_g	5.6E+13	3.3E+13	3.3E+13	mm^4
• Tension reinforcement:	Distance from tension reinf. to extreme compression fiber	d_c	238	266	131	mm
	Reinf. Diameter	∅	38	28	36	mm
	Space	@	150	150	150	mm
	Number of bar	n	330	194	97	bar
	Total area of reinf.	A_s	374712	119702	99073	mm^2
• comp. reinforcement:	Distance from compressive reinf. to extreme Tension fiber		100	156	150	mm
	Diameter		38	36	28	mm
	Reinf. Space		150	150	150	mm
	Number of bar		165	97	194	bar
	Total area of reinf.	A'_s	168289	99073	119702	mm^2
Check Flexural Moment						
• Resistance factor		Φ	0.90	0.90	0.90	
• The corresponding effective		d_e	2762	2734	2869	mm
• Stress block factor		β_1	0.84	0.84	0.84	
• Depth of the equivalent stress block = c*β1		a	235.11	126.87	105.01	mm
• Distance from extreme compression fiber to the neutral axis		c	281.33	151.81	125.65	mm
• The nominal flexural resistance:		M_n	396361	127869	111616	kN.m
• Factored flexural resistance		$M_r = \Phi.M_n$	356725	115082	100454	kN.m
• Check condition		$M_r > M_u$	**O.K**	**O.K**	**O.K**	
Mimimum Reinforcement						
• Ratio of tension steel to gross area		$\rho = A_s/(b.d)$	0.54	0.30	0.23	%
• Check		$\rho > 0.03 \cdot f'_c/f'_y$	**O.K**	**O.K**	**O.K**	0.23
• Cracking moment		1.2Mcr	155279.35	91925.37	91925.37	Kn.m
• Check		Mr> min(1.2Mcr, 1.33Mu)	**O.K**	**O.K**	**O.K**	
Maximum Reinforcement						
• Obligation Condition		c/d_e	0.10	0.06	0.04	
• Check		$c/d_e < 0.42$	**O.K**	**O.K**	**O.K**	
Check shear resistance						
• Factored Shear force		V_u	89604.84	28078.49	28658.90	kN
• Resistance factor		Φ	0.90	0.90	0.90	
• The effective shear deepth		d_v	2644	2671	2816	mm
• Effective width		b_v	25000	14800	14800	mm
• Angle of inclination of diagonal compressive stress		θ	42	41	41	degree
• Angle of inclination of transerverse reinf. To longitudinal axis		α	90	90	90	degree
• Factor indicating ability of diagonally cracked concrete to transmit tension		β	1.75	1.95	1.95	
• Value		$0.1 \cdot f'_c \cdot b_v \cdot d_v$	198333	118573	125052	kN
• Max spacing of transverse reinforcement		s	600	600	600	mm
• Spacing of stirrup		s	300	450	450	mm
• Diameter of transverse reinforcement		∅	D 38	D 28	D 38	
• Number of transverse reinf. within distance s		n	4	4	2	bar
• Total area of transverse reinf.		A_v	4536	2463	2268	mm^2
• Diameter of stirrup		∅	D 20	D 20	D 20	mm
• Number of stirrup within distance s		n	84	32	32	bar
• Total area of stirrup		A_v	26494.10	10122.91	10122.91	
• Assume		θ	42.00	41.00	41.00	degree
• Strain in tensile reinforcement		ε_x	2.11E-03	1.78E-03	1.50E-03	
If ex<0, multiple with reduce factor		Fc	-	-	-	
• Ratio of shear stress and f'c		V/f'c	0.05	0.03	0.03	
• β final			1.75	1.95	1.95	
• θ final			43.00	43.00	43.00	degree
• The shear resistance of concrete:		V_c	52595.80	35037.91	36952.54	kN

• The shear resistance of stirrup	V_s	50894.10	13945.93	14708.00	kN
• Value	$0.25*f'_c \cdot b_v \cdot d_v$	495833.14	296432.69	312631.16	kN
• The nominal shear resistance:	V_n	103489.90	48983.83	51660.54	kN
• The factored shear resistance	V_r	93140.91	44085.45	46494.49	kN
• Check	$V_r > V_u$	**O.K**	**O.K**	**O.K**	
•Requiring transverse reinforcement	$V_u > 0.5 \cdot \Phi\ V_c$	**Need**	**Need**	**Need**	
•Check minimum transverse reinforcement	$A_v > 0.083\ (f'_c{}^{0.5}) \cdot b_v \cdot s/f_y$	**O.K**	**O.K**	**O.K**	
Check Flexural and shear resistance at Extreme state					
• Factored Plexural moment	M_u	330961.12	75140.78	37485.20	kN.m
• Factored Shear force	V_u	99790.71	29717.08	33412.20	kN
• Resistance factor	Φ	1.00	1.00	1.00	
• The nominal flexural resistance:	M_n	396361	127869	111616	kN.m
• Factored flexural resistance	$M_r = \Phi.M_n$	396361	127869	111616	kN.m
• The nominal flexural resistance:	V_n	103490	48984	51661	Kn
• Factored flexural resistance	$V_r = \Phi.V_n$	103490	48984	51661	Kn
• Check condition	$M_r > M_u$	**O.K**	**O.K**	**O.K**	
	$V_r > V_u$	**O.K**	**O.K**	**O.K**	
Check crack					
Interior force combination	**Service III**				
• Factored moment	M_u	2.26E+05	5.60E+04	2.89E+04	kN.m
• Modulus of rupture of concrete	fr = 0.63*sqrt(f'c)	3.45	3.45	3.45	MPa
• Distance from extreme tension fiber to the neutral axis	$y_t = h-c$	2719	2848	2874	mm
• Stress of concrete at tension fiber	$f'r = Ms*y_t/I_g$	11	5	2	MPa
• Check	f'r >	0.8*fr	0.8*fr	0.8*fr	
		check crack	check crack	No check	
• Crack width parameter	Z	= 23000	= 23000	= 23000	N/mm
• Ratio of reinf. Modulus with concrete modulus	$n = E_s/E_c$	= 7.00	= 7.00	= 7.00	
• The distance from extreme fiber to the neutral axis	c	= 873.41	= 615.88	= 567.51	mm
• Effective moment of inertia	J	1.49E+13	4.91E+12	4.58E+12	mm^4
• Arm	de-c	= 1888.59	= 2118.12	= 2301.49	mm
• Tension stress in reinforcement	fs =n*Ms*(de-c)/J	= 200.63	= 168.95	= 101.74	MPa
• Area of concrete having the same centroid as the principal tensile reinfor	A	= 15133	= 15226	= 15205	mm2
• Tension stress in reinforcement with service state	$f_{sa} = Z/(d_c*A)^{1/3}$	= 226.71	= 231.99	= 227.45	Mpa
• Check condition	$f_s < f_{sa}$	**O.K**	**O.K**	**O.K**	
• Check condition	$f_s < 0.6*f_y$	**O.K**	**O.K**	**O.K**	

2.10 PIER P15
2.10 TRỤ CẦU P15

PIER P15 - CALCULATION SHEET

CONTENT:

2.10.1. GENERAL DATA

CALCULATION PROCEDURE & STANDARD:

- Bridge Design Standard 22 TCN - 272 - 05

2.10.1.1. Design live load

Design vehicle load		HL93	22TCN 272 - 05
Number of lane		6	(lane)
Pedestrian		0.00	KG/m^2

2.10.1.2. Bridge width

Width carriageway	B_{xe} =	12.00	(m)
Width of median guardrail	B_{pc} =	0.50	(m)
Width parapet	B_{lc} =	0.50	(m)
Bridge width	B =	13.00	(m)

2.10.1.3. Superstructure:

Span-arrangement		Continuous box girder 65+5@100+65m	
Height of box girder at pier section	H =	6.00	(m)
Height of box girder at mid-span section	h =	2.50	(m)
Pavement thickness	d_{BTN} =	0.084	(m)

2.10.1.4. Material property:

Concrete

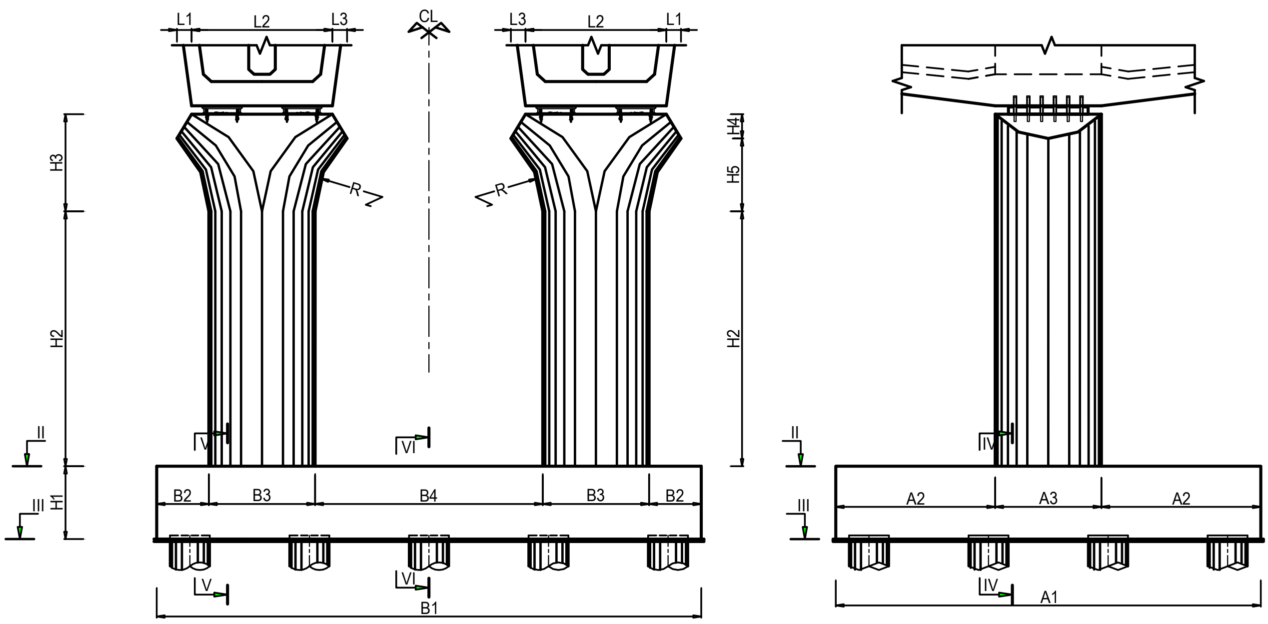
Compressive strength of cylindrical at 28 day	f 'c =	30.00	MPa
Concrete density	g =	24.50	KN/m^3
Elastic modulus	Ec =	29440	MPa
Tension strength of concrete	fr =	3.45	MPa

Steel

Steel modulus	Es =	200000	MPa
Yeild strength of steel bar	fy =	400.00	MPa
Calculation unit:		KN, m, KN.m, Mpa, KN/m^2	

2.10.1.5. THE PIER GEOMETRIC

GENERAL DIMENSIONS



vertical			Horizontal			Thickness		
Remark	Value	Unit	Remark	Value	Unit	Remark	Value	Unit
a_1 =	16.00	(m)	L_1 =	0.55	(m)	h_1 =	3.00	(m)
a_2 =	6.00	(m)	L_2 =	5.30	(m)	h_2 =	15.000	(m)
a_3 =	4.00	(m)	L_3 =	0.55	(m)	h_3 =	4.000	(m)
			B_1 =	20.50	(m)	h_4 =	1.000	(m)
			B_2 =	1.715	(m)	h_5 =	3.000	(m)
			B_3 =	4.00	(m)			
			B_4 =	9.070	(m)			
			R =	4.34	(m)			

The design elevation:

Proposed height	EL_{mc} =	18.340	(m)			
Elevation of top of pier cap	EL_{xm} =	12.135	(m)			
Hight water level (H1%)	EL_{MNTK} =	9.200	(m)			
Daily water level (H5%)	EL_{MNTB} =	4.070	(m)			
Ground elevation	EL_{TN} =	-3.680	(m)			
Daily water level (H5%)	EL_{TT} =	4.070	(m)			
Top of Pile cap (Section II - II)	$EL_{MC\ II\text{-}II}$ =	-7.265	(m)			
Bottom of Pile cap (Section III - III)	$EL_{MCIII\text{-}III}$ =	-10.265	(m)			

2.10.2. CALCULATED PIER FORCE AND PILE FORCE

2.10.2.1. Loading combination at bottom of pier Shaft

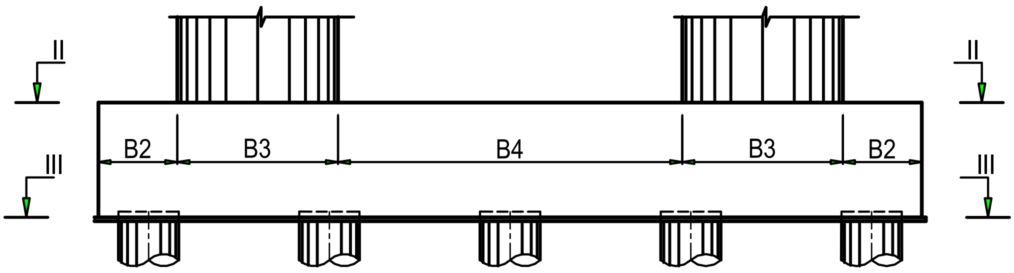
(Result from RM2010 SPACE-FRAME program)

Loading combination in accordance with 22 TCN 272-05, table 3.4.1.1 was mentioned in appendix of this report

LOADING COMBINATION AT TOP OF PILE CAP DUE TO SUPERSTRUCTURE + VESSEL COLLISION AND SEFTWEIGHT						
Limited state	Vertical axis	Longitudinal axis		Horizontal axis		Remarks
	N_x (kN)	Q_y (kN)	M_z (kNm)	Q_z (kN)	M_y (kNm)	
STRENGTH I	55478.80	95.77	751.80	214.04	6112.94	
STRENGTH II	47907.99	95.77	751.79	1579.83	32680.49	
STRENGTH III	53748.32	95.77	751.79	736.77	17059.98	
SERVICE I	49976.04	3339.77	35576.13	5442.24	42828.30	
SERVICE III	50033.28	95.77	751.79	1952.05	41340.60	
EXTREME EVENT (CV)	50352.49	957.70	7517.95	1206.11	17388.94	
EXTREME EVENT (EQ trans)	42300.06	95.77	751.79	598.96	13638.37	
EXTREME EVENT (EQ long)	42307.12	95.77	751.79	598.97	13634.57	

2.10.2.2. Loading combination at bottom of pile cap

LOADING COMBINATION AT TOP OF PILE CAP DUE TO SUPERSTRUCTURE + VESSEL						
Limited state	Vertical axis	Longitudinal axis		Horizontal axis		Remarks
	N_x (kN)	Q_y (kN)	M_z (kNm)	Q_z (kN)	M_y (kNm)	
STRENGTH I	148708	228	2133	454	12153	
STRENGTH II	133566	228	2133	3185	74881	
STRENGTH III	145247	228	2133	1499	37503	
SERVICE I	137702	3358	45623	5455	58962	
SERVICE III	137817	228	2133	3930	94034	
EXTREME EVENT (CV)	138455	2281	21331	2670	42003	
EXTREME EVENT (EQ trans)	115000	228	2133	1224	30112	
EXTREME EVENT (EQ long)	115014	228	2133	1224	30104	



2.10.2.3. Loading combination at top of piling

2.10.2.3.1. Piling material:

Concrete 30 Mpa

E_c (kg/cm^2)	294401
γ_c (T/m^3)	2.5

Steel bar

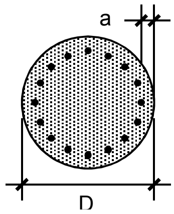
Type	CB-400-T
E_s (kg/cm^2)	200000

2.10.2.3.2. Piling dimension

+ Diameter **D** = 1.50 m

a = 0.100 m

+ Length **L** = 54.00 m

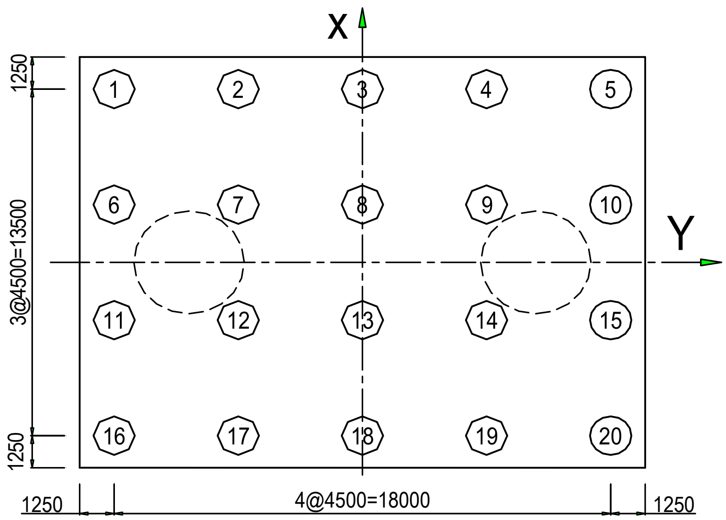


2.10.2.3.3. Maximum Internal force and displacement at top piling

Maximum Internal force and diplacements *(Result follow Piling software)*

Combination	N	Hx	My	x	y	z
	(KN)	(KN)	(KN.m)	(m)	(m)	(rad)
Strength I	7648.32	11.40	70.62	-	-	-
Strength II	7774.13	11.40	70.62	-	-	-
Strength III	7827.84	11.40	70.62	-	-	-
Service I	6214.84	11.40	70.62	**0.000**	**-0.003**	**0.008**
Service III	6215.45	11.40	70.62	**0.000**	**-0.003**	**0.008**
Extreme I (CV)	8804.16	167.90	1022.43	-	-	-
Extreme II (EQ trans)	8249.86	11.40	70.62	-	-	-
Extreme II (EQ long)	8046.93	114.05	706.53	-	-	-

- Check displacement of top pile not exceed 38mm (10.7.2.7) **OK**



Arrangement of pile

2.10.2.3.4. Internal force for each pile

Combination	Pile Number	Hx (KN)	Hy (KN)	N (KN.m)	Mx (m)	My (m)
Strength I	1	11.4	-22.7	7553.8	137.7	70.6
	2	11.4	-22.7	7470.9	137.7	70.6
	3	11.4	-22.7	7388.1	137.7	70.6
	4	11.4	-22.7	7305.3	137.7	70.6
	5	11.4	-22.7	7222.5	137.7	70.6
	6	11.4	-22.7	7585.3	137.7	70.6
	7	11.4	-22.7	7502.5	137.7	70.6
	8	11.4	-22.7	7419.6	137.7	70.6
	9	11.4	-22.7	7336.8	137.7	70.6
	10	11.4	-22.7	7254.0	137.7	70.6
	11	11.4	-22.7	7616.8	137.7	70.6
	12	11.4	-22.7	7534.0	137.7	70.6
	13	11.4	-22.7	7451.2	137.7	70.6
	14	11.4	-22.7	7368.3	137.7	70.6
	15	11.4	-22.7	7285.5	137.7	70.6
	16	11.4	-22.7	7648.3	137.7	70.6
	17	11.4	-22.7	7565.5	137.7	70.6
	18	11.4	-22.7	7482.7	137.7	70.6
	19	11.4	-22.7	7399.9	137.7	70.6
	20	11.4	-22.7	7317.0	137.7	70.6
Strength II	1	11.4	-159.3	7679.6	974.5	70.6
	2	11.4	-159.3	7155.3	974.5	70.6
	3	11.4	-159.3	6631.0	974.5	70.6
	4	11.4	-159.3	6106.7	974.5	70.6
	5	11.4	-159.3	5582.5	974.5	70.6
	6	11.4	-159.3	7711.1	974.5	70.6
	7	11.4	-159.3	7186.8	974.5	70.6
	8	11.4	-159.3	6662.5	974.5	70.6
	9	11.4	-159.3	6138.3	974.5	70.6
	10	11.4	-159.3	5614.0	974.5	70.6
	11	11.4	-159.3	7742.6	974.5	70.6
	12	11.4	-159.3	7218.3	974.5	70.6
	13	11.4	-159.3	6694.1	974.5	70.6
	14	11.4	-159.3	6169.8	974.5	70.6
	15	11.4	-159.3	5645.5	974.5	70.6
	16	11.4	-159.3	7774.1	974.5	70.6
	17	11.4	-159.3	7249.9	974.5	70.6
	18	11.4	-159.3	6725.6	974.5	70.6
	19	11.4	-159.3	6201.3	974.5	70.6
	20	11.4	-159.3	5677.0	974.5	70.6
Strength III	1	11.40	-74.95	7733.30	456.85	70.62
	2	11.40	-74.95	7474.19	456.85	70.62
	3	11.40	-74.95	7215.08	456.85	70.62
	4	11.40	-74.95	6955.97	456.85	70.62
	5	11.40	-74.95	6696.86	456.85	70.62
	6	11.40	-74.95	7764.81	456.85	70.62
	7	11.40	-74.95	7505.70	456.85	70.62
	8	11.40	-74.95	7246.59	456.85	70.62
	9	11.40	-74.95	6987.48	456.85	70.62
	10	11.40	-74.95	6728.37	456.85	70.62
	11	11.40	-74.95	7796.33	456.85	70.62
	12	11.40	-74.95	7537.22	456.85	70.62
	13	11.40	-74.95	7278.11	456.85	70.62
	14	11.40	-74.95	7019.00	456.85	70.62
	15	11.40	-74.95	6759.89	456.85	70.62
	16	11.40	-74.95	7827.84	456.85	70.62
	17	11.40	-74.95	7568.73	456.85	70.62
	18	11.40	-74.95	7309.62	456.85	70.62
	19	11.40	-74.95	7050.51	456.85	70.62
	20	11.40	-74.95	6791.40	456.85	70.62

- Result for internal force at top pile

Combination	Pile Number	**Hx** (KN)	**Hy** (KN)	**N** (KN.m)	**Mx** (m)	**My** (m)
Service I	1	11.40	-61.20	6120.29	373.44	70.62
	2	11.40	-61.20	5911.51	373.44	70.62
	3	11.40	-61.20	5702.73	373.44	70.62
	4	11.40	-61.20	5493.95	373.44	70.62
	5	11.40	-61.20	5285.16	373.44	70.62
	6	11.40	-61.20	6151.81	373.44	70.62
	7	11.40	-61.20	5943.02	373.44	70.62
	8	11.40	-61.20	5734.24	373.44	70.62
	9	11.40	-61.20	5525.46	373.44	70.62
	10	11.40	-61.20	5316.68	373.44	70.62
	11	11.40	-61.20	6183.32	373.44	70.62
	12	11.40	-61.20	5974.54	373.44	70.62
	13	11.40	-61.20	5765.76	373.44	70.62
	14	11.40	-61.20	5556.98	373.44	70.62
	15	11.40	-61.20	5348.19	373.44	70.62
	16	11.40	-61.20	6214.84	373.44	70.62
	17	11.40	-61.20	6006.05	373.44	70.62
	18	11.40	-61.20	5797.27	373.44	70.62
	19	11.40	-61.20	5588.49	373.44	70.62
	20	11.40	-61.20	5379.71	373.44	70.62
Service III	1	11.40	-61.20	6120.90	373.44	70.62
	2	11.40	-61.20	5912.17	373.44	70.62
	3	11.40	-61.20	5703.43	373.44	70.62
	4	11.40	-61.20	5494.69	373.44	70.62
	5	11.40	-61.20	5285.95	373.44	70.62
	6	11.40	-61.20	6152.42	373.44	70.62
	7	11.40	-61.20	5943.68	373.44	70.62
	8	11.40	-61.20	5734.94	373.44	70.62
	9	11.40	-61.20	5526.21	373.44	70.62
	10	11.40	-61.20	5317.47	373.44	70.62
	11	11.40	-61.20	6183.93	373.44	70.62
	12	11.40	-61.20	5975.20	373.44	70.62
	13	11.40	-61.20	5766.46	373.44	70.62
	14	11.40	-61.20	5557.72	373.44	70.62
	15	11.40	-61.20	5348.98	373.44	70.62
	16	11.40	-61.20	6215.45	373.44	70.62
	17	11.40	-61.20	6006.71	373.44	70.62
	18	11.40	-61.20	5797.97	373.44	70.62
	19	11.40	-61.20	5589.23	373.44	70.62
	20	11.40	-61.20	5380.50	373.44	70.62
Extreme Event I (with CV)	1	167.90	-272.75	7042.25	1723.38	1022.43
	2	167.90	-272.75	6523.20	1723.38	1022.43
	3	167.90	-272.75	6004.15	1723.38	1022.43
	4	167.90	-272.75	5485.09	1723.38	1022.43
	5	167.90	-272.75	4966.04	1723.38	1022.43
	6	167.90	-272.75	7629.56	1723.38	1022.43
	7	167.90	-272.75	7110.50	1723.38	1022.43
	8	167.90	-272.75	6591.45	1723.38	1022.43
	9	167.90	-272.75	6072.40	1723.38	1022.43
	10	167.90	-272.75	5553.34	1723.38	1022.43
	11	167.90	-272.75	8216.86	1723.38	1022.43
	12	167.90	-272.75	7697.81	1723.38	1022.43
	13	167.90	-272.75	7178.75	1723.38	1022.43
	14	167.90	-272.75	6659.70	1723.38	1022.43
	15	167.90	-272.75	6140.65	1723.38	1022.43
	16	167.90	-272.75	8804.16	1723.38	1022.43
	17	167.90	-272.75	8285.11	1723.38	1022.43
	18	167.90	-272.75	7766.05	1723.38	1022.43
	19	167.90	-272.75	7247.00	1723.38	1022.43
	20	167.90	-272.75	6727.95	1723.38	1022.43

- Result for internal force at top pile

Combination	Pile Number	**Hx** (KN)	**Hy** (KN)	**N** (KN.m)	**Mx** (m)	**My** (m)
Extreme Event I (with EQtran)	1	11.40	-196.50	8155.31	1201.11	70.62
	2	11.40	-196.50	7499.45	1201.11	70.62
	3	11.40	-196.50	6843.58	1201.11	70.62
	4	11.40	-196.50	6187.71	1201.11	70.62
	5	11.40	-196.50	5531.84	1201.11	70.62
	6	11.40	-196.50	8186.83	1201.11	70.62
	7	11.40	-196.50	7530.96	1201.11	70.62
	8	11.40	-196.50	6875.09	1201.11	70.62
	9	11.40	-196.50	6219.23	1201.11	70.62
	10	11.40	-196.50	5563.36	1201.11	70.62
	11	11.40	-196.50	8218.34	1201.11	70.62
	12	11.40	-196.50	7562.48	1201.11	70.62
	13	11.40	-196.50	6906.61	1201.11	70.62
	14	11.40	-196.50	6250.74	1201.11	70.62
	15	11.40	-196.50	5594.87	1201.11	70.62
	16	11.40	-196.50	8249.86	1201.11	70.62
	17	11.40	-196.50	7593.99	1201.11	70.62
	18	11.40	-196.50	6938.12	1201.11	70.62
	19	11.40	-196.50	6282.25	1201.11	70.62
	20	11.40	-196.50	5626.39	1201.11	70.62
Extreme Event I (with EQlong)	1	114.05	-133.50	7101.29	833.20	706.53
	2	114.05	-133.50	6775.36	833.20	706.53
	3	114.05	-133.50	6449.43	833.20	706.53
	4	114.05	-133.50	6123.50	833.20	706.53
	5	114.05	-133.50	5797.57	833.20	706.53
	6	114.05	-133.50	7416.50	833.20	706.53
	7	114.05	-133.50	7090.57	833.20	706.53
	8	114.05	-133.50	6764.64	833.20	706.53
	9	114.05	-133.50	6438.71	833.20	706.53
	10	114.05	-133.50	6112.79	833.20	706.53
	11	114.05	-133.50	7731.71	833.20	706.53
	12	114.05	-133.50	7405.79	833.20	706.53
	13	114.05	-133.50	7079.86	833.20	706.53
	14	114.05	-133.50	6753.93	833.20	706.53
	15	114.05	-133.50	6428.00	833.20	706.53
	16	114.05	-133.50	8046.93	833.20	706.53
	17	114.05	-133.50	7721.00	833.20	706.53
	18	114.05	-133.50	7395.07	833.20	706.53
	19	114.05	-133.50	7069.14	833.20	706.53
	20	114.05	-133.50	6743.22	833.20	706.53

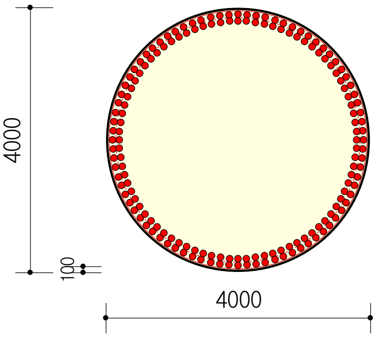
2.10.3. CHECK FOR PIER SHAFT

Item	Sub-item	Mark	Unit	Value
• Factored Axial force		Nu	Kn	50033.28
• Factored Plexural moment		Mux	Kn.m	751.79
• Factored Plexural moment		Muy	Kn.m	41340.60
• Diameter of Pier shaft		D	m	4.00
• Section area		Ag	m2	12.57
• Moment of inertia of concrete section		Ic	m4	12.57
• Cover thickness		a	m	0.075
• Reinf. Diameter		Ds	mm	32.00
• Number of rebar		n_s	nos	168.00
• Rebar area		As	mm2	135113.62
Check minimum reinforcement				
• Minimum rebar area required (0.135*fy/f'c)*Ag		As req	mm2	127234.50
• Check condition As > (0.135*fy/f'c)*Ag				**OK**
Check maximum reinforcement				
• Maximum rebar area 0.08*Ag		As max	mm2	1005309.6
• Check condition As < 0.08*Ag				**OK**
Check ratio spiral or Tier (5.7.4.6)				
• Distance to outside of Spairal or Ties to concrete face			mm	66.00
• Effect diamete		Deff	m	3.87
• Area of core measured to the outside diameter of the spiral			m2	11.75
• Ratio spiral Rebar required		ρsa		0.00234
Required Area of Spiral Rebar	space		mm	200
	Effective length			3.87
	layer			2
	Area			226.6
	Requaired Dhs			17.0
Actuaral	Effective length	d	m	3.868
	Diameter	Dhr	mm	18
	Area of Rebar	Ah	mm2	254.5
	layer	Nl	nos	2
	Total area of spiral	Ac	m2	508.938
	space	s	mm	200
	Ratio spiral Rebar	ρs	-	0.0026315
• Check condition		$\rho_s > \rho_{sa}$		OK
Check Crack (At Service State)				
• Modulus of rupture of concrete		fr = 0.63*sqrt(f'c)	Mpa	3.45
• Stress of concrete at tension fiber		σ'_r	Mpa	1.70
• If f'r > 0.8fr require check crack		$\sigma'_r > 0.8*\sigma_r$	Mpa	No check
• Center of newtral axial		x	mm	3.24
• Maximum stress of Compression fiber of concrete		σ_c	Mpa	8.70
• Maximum stress of Compression Rebar		σ_{rc}	Mpa	-127.10
• Maximum stress of Tension Rebar		σ_{rt}	Mpa	26.90
• Check			σ_{rt} <0.6.fy	OK

MIDAS/Set **Column Design [Body pier D4.0m-P15]**

	Company	Nipponkoei	**Project Name**	DN Qn Expressway
	Designer	vinh	**File Name**	D:\...\bodypier-D4.0m-P15.BOI

1. Geometry and Materials



Design Code : ACI318M-02

Stress Profile : Equivalent Stress Block

Material Data : f_c' = 30 MPa (β_1 = 0.850)

f_y = 400, f_{ys} = 400 MPa

Section Dim. : Φ4000 mm

Effective Len. : KL_u = 19000 mm

Steel Distribut.: 84 - D32 (d_c = 100 mm)

: 84 - P32 (d_c = 200 mm)

Total Steel Area A_{st} = 131896 mm^2 (ρ_{st} = 0.0107)

2. Member Force and Moment

Unit : kN, kN-m

L.C.	P_u	M_{ux}	M_{uy}	R_{atioV}	V_{ux}	V_{uy}	R_{atioH}	Remark
1	55478.8	751.8	6112.9	0.288	95.8	214.0	0.008	
2	47908.0	751.8	32680.5	0.317	95.8	1579.8	0.053	
3	53748.3	751.8	17060.0	0.279	95.8	736.8	0.025	
4	49976.0	35576.1	42828.3	0.463	3339.8	5442.2	0.214	
5	50033.3	751.8	41340.6	0.377	95.8	1952.1	0.065	
6	50352.5	7517.9	17388.9	0.266	957.7	1206.1	0.052	
7	42300.1	751.8	13638.4	0.219	95.8	599.0	0.020	
8	42307.1	751.8	13634.6	0.219	95.8	599.0	0.020	
9	34426.0	0.0	89145.0	0.651	0.0	0.0	0.020	

3. Magnified Moment

KL_u/r_x = 19000/1000= 19.00 < 34-12(M_1/M_2) = 22.00

δ_x = 1.000

KL_u/r_y = 19000/1000= 19.00 < 34-12(M_1/M_2) = 22.00

δ_y = 1.000

4. Design Force and Moment

Design Load Combination No : 9

P_u = 34426.0 kN

M_{ux} = 0.0, M_{uy} = 89145.0 kN-m

5. Check Axial and Moment Capacity

Rotation Angle and Depth to the Neutral Axis θ = 0.00°, c = 1345 mm

Strength Reduction Factor Φ = 0.9000

Maximum Axial Load $\Phi P_{n(max)}$ = 192777.8 kN

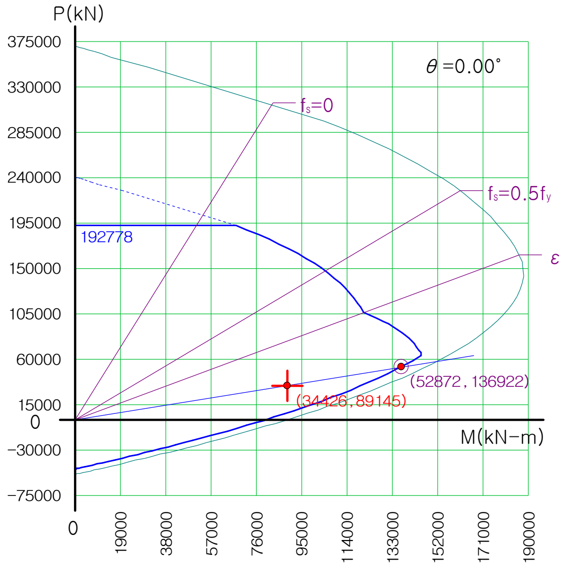
Design Axial Load Strength ΦP_n = 52871.5 kN

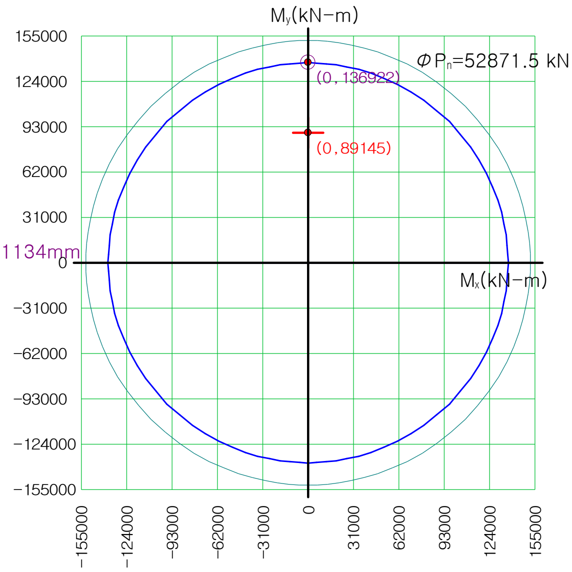
Design Moment Strength ΦM_{nx} = N.A

ΦM_{ny} = 136922.5 kN-m

Strength Ratio : Applied/Design = 0.651 < 1.000 .... O.K

Company	Nipponkoei	Project Name	DN Qn Expressway
Designer	vinh	File Name	D:\...\bodypier-D4.0m-P15.BOI





6. Check Shear Capacity

Design Load Combination No : 4

Strength Reduction Factor Φ= 0.750

Design Force Vu = 6385.3 kN (Pu = 49976.0 kN)

Required Hoop Spacing : D18 @ 508 mm

Provided Hoop Spacing : D18 @ 200 mm (Tie)

$\Phi V_c + \Phi V_s$ = 27949.3 + 1912.3 = 29861.5 kN > V_u = 6385.3 kN .... O.K

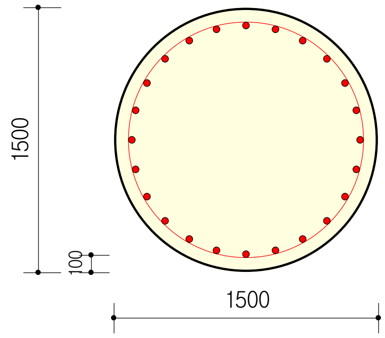
2.10.4. CHECK FOR PILE

Item		Mark	Unit	Value
• Factored Axial force		Nu	Kn	6214.84
• Factored Plexural moment		Mux	Kn.m	373.44
• Factored Plexural moment		Muy	Kn.m	70.62
• Diameter of Pier shaft		D	m	1.50
• Section area		Ag	m2	1.77
• Moment of inertia of concrete section		Ic	m4	0.25
• Cover thickness		a	m	0.075
• Reinf. Diameter		Ds	mm	D32
• Number of rebar		n_s	nos	24
• Rebar area		As	mm2	19301.95
Check minimum reinforcement				
• Minimum rebar area required (0.135*fy/f'c)*Ag		As req	mm2	17892.35
• Check condition As > (0.135*fy/f'c)*Ag				**OK**
Check maximum reinforcement				
• Maximum rebar area 0.08*Ag		As max	mm2	141371.7
• Check condition As < 0.08*Ag				**OK**
Check ratio spiral or Tier (5.7.4.6)				
• Distance to outside of Spairal or Ties to concrete face			mm	68.00
• Effect diamete		Deff	m	1.36
• Area of core measured to the outside diameter of the spiral			m2	1.46
• Ratio spiral Rebar required		ρsa		0.00707
Required Area of Spiral Rebar	space		mm	75
	Effective length			1.36
	layer			1
	Area			180.7
	Requaired Dhs			15.2
Actuaral	Effective length	d	m	1.364
	Diameter	Dhr	mm	16
	Area of Rebar	Ah	mm2	201.1
	layer	Nl	nos	1
	Total area of spiral	Ac	m2	201.062
	space	s	mm	75
	Ratio spiral Rebar	ρs	-	0.0078617
• Check condition			$\rho_s > \rho_{sa}$	OK

Company	Nipponkoei	**Project Name**	DN Qn Expressway
Designer	vinh	**File Name**	D:\...\Pile D1.5m - Pier P15.BOI

1. Geometry and Materials

Design Code : ACI318M-02
Stress Profile : Equivalent Stress Block
Material Data : f_c' = 30 MPa (β_1 = 0.850)
f_y = 400, f_{ys} = 400 MPa
Section Dim. : Φ1500 mm
Effective Len. : KL_u = 15000 mm
Steel Distribut.: 24 - D32 (d_c = 100 mm)
Total Steel Area A_{st} = 19061 mm^2 (ρ_{st} = 0.0108)



2. Member Force and Moment

Unit : kN, kN-m

L.C.	P_u	M_{ux}	M_{uy}	R_{atioV}	V_{ux}	V_{uy}	R_{atioH}	Remark
1	7648.3	138.0	71.0	0.282	11.0	23.0	0.005	
2	7774.1	974.0	71.0	0.312	11.0	159.0	0.033	
3	7827.8	457.0	71.0	0.288	11.0	75.0	0.016	
4	6214.8	373.0	71.0	0.229	11.0	61.0	0.013	
5	6215.4	373.0	71.0	0.229	11.0	61.0	0.013	
6	8804.2	1723.0	1022.0	0.454	168.0	273.0	0.067	
7	8249.9	1201.0	71.0	0.349	11.0	197.0	0.041	
8	8046.9	833.0	707.0	0.321	114.0	134.0	0.037	

3. Magnified Moment

KL_u/r_x = 15000/375 = 40.00 > 34-12(M_1/M_2) = 22.00
δ_x = MAX[1.00/(1-P_u/0.75/52256), 1.0] = 1.290

KL_u/r_y = 15000/375 = 40.00 > 34-12(M_1/M_2) = 22.00
δ_y = MAX[1.00/(1-P_u/0.75/52256), 1.0] = 1.290

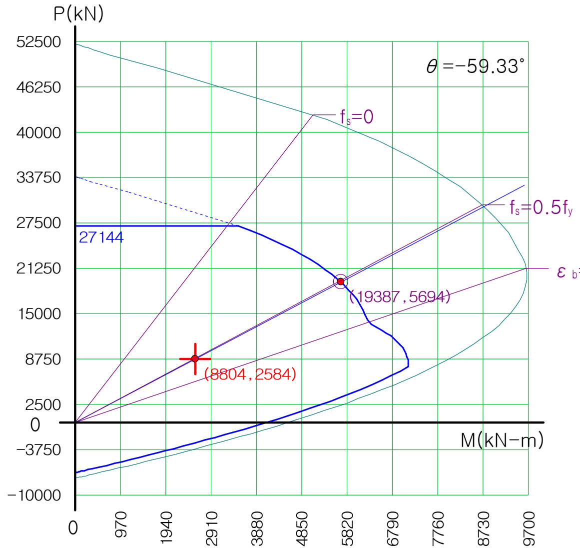
4. Design Force and Moment

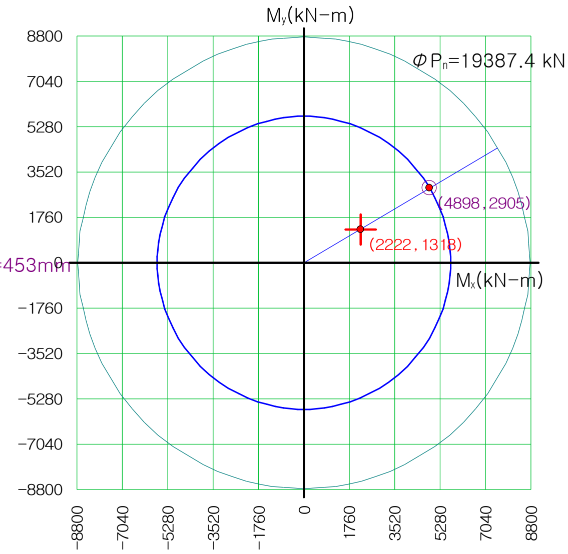
Design Load Combination No : 6
P_u = 8804.2 kN
M_{ux} = 1723.0, M_{uy} = 1022.0 kN-m
$\delta_x M_{ux}$= δ_x*M_{ux} = 2222.2 kN-m
$\delta_y M_{uy}$= δ_y*M_{uy}, = 1318.1 kN-m

5. Check Axial and Moment Capacity

Rotation Angle and Depth to the Neutral Axis θ = -59.33°, c = 1060 mm
Strength Reduction Factor Φ = 0.6500
Maximum Axial Load $\Phi P_{n(max)}$ = 27144.3 kN
Design Axial Load Strength ΦP_n = 19387.4 kN
Design Moment Strength ΦM_{nx} = 4897.5 kN-m
ΦM_{ny} = 2904.8 kN-m
Strength Ratio : Applied/Design = 0.454 < 1.000 .... O.K

Company	Nipponkoei	Project Name	DN Qn Expressway
Designer	vinh	File Name	D:\...\Pile D1.5m - Pier P15.BOI





6. Check Shear Capacity

Design Load Combination No : 6

Strength Reduction Factor Φ= 0.750

Design Force Vu = 320.6 kN (Pu = 8804.2 kN)

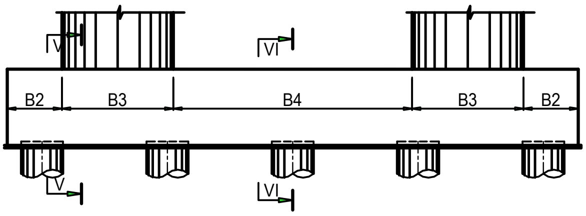
Required Hoop Spacing : D16 @ 508 mm

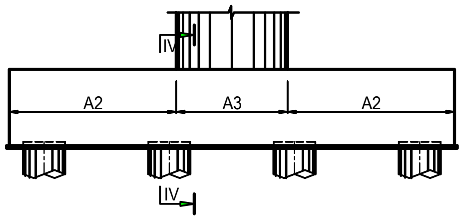
Provided Hoop Spacing : D16 @ 150 mm (Tie)

$\Phi V_c + \Phi V_s$ = 3870.3 + 924.5 = 4794.8 kN > V_u = 320.6 kN .... O.K

2.10.5. CHECK FOR PILE CAP

2.10.5.1. The Force to section IV-IV, section V-V, section VI-VI





At section IV - IV

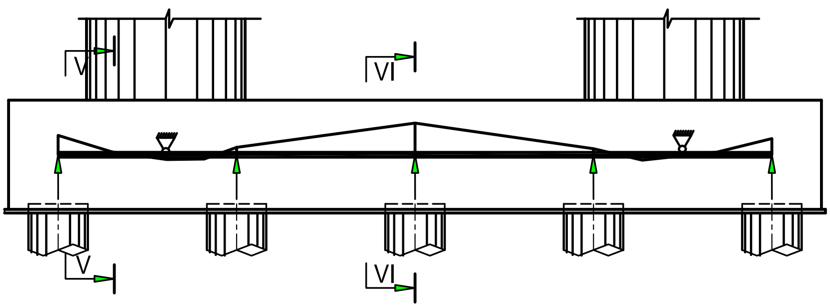
COMBINATION	Longitudinal direction		
	Q (kN)	M(kN.m)	V (kN.m)
Strength I	74669.2	236122.7	102.6
Strength II	67098.2	212145.3	102.6
Strength III	72938.7	230642.2	102.6
Service I	57815.1	182746.0	102.6
Service III	57822.2	182768.2	125.4
Extreme I (CV)	74724.0	230043.8	1846.9
Extreme II (EQ trans)	69223.6	218876.7	125.4
Extreme II (EQ long)	72374.6	225664.3	1254.55

At section V - V

COMBINATION	Transverse direction		
	Q (kN)	M(kN.m)	V (kN.m)
Strength I	30404.2	34387.1	-45.4
Strength II	30907.4	34956.3	-318.5
Strength III	31122.3	35199.3	-149.9
Service I	24670.3	27902.1	-122.4
Service III	24672.7	27904.8	-122.4
Extreme I (CV)	31692.8	35844.6	-545.5
Extreme II (EQ trans)	32810.34	37108.5	-393.00
Extreme II (EQ long)	30296.43	34265.3	-267.00

At section VI - VI

COMBINATION	Transverse direction		
	Q (kN)	M(kN.m)	V (kN.m)
Strength I	24453.6	35484.0	-45.4
Strength II	23698.0	31765.0	-318.5
Strength III	24609.6	34635.0	-149.9
Service I	19505.6	27412.4	-122.4
Service III	19510.1	27419.0	-122.4
Extreme I (CV)	23724.5	32778.5	-545.5
Extreme II (EQ trans)	25000.20	32769.90	-393.0
Extreme II (EQ long)	23525.30	32978.70	-267.0



2.10.5.2. Ultimate load check, shear capacity check and crack control

*** Check flexure momen capacity of pile cap:**

Item			Section IV-IV (Bottom bar)	Section V-V (Bottom bar)	Section VI-VI (Upper bar)	Unit
• Factored Plexural moment		M_u	236122.66	35199.31	35484.00	kN.m
• Factored Shear force		V_u	74669.15	31122.29	24609.60	kN
• Hight of Section		h	3000	3000	3000	mm
• Width of section		b	20500	16000	16000	mm
• Section area		A_c	61500000	48000000	48000000	mm^2
• Moment of inertia of concrete section		I_g	4.6E+13	3.6E+13	3.6E+13	mm^4
• Tension reinforcement:	Distance from tension reinf. to extreme compression fiber	d_c	186	214	154	mm
	Reinf. Diameter	∅	36	28	36	mm
	Space	@	150	150	150	mm
	Number of bar	n	270	210	105	bar
	Total area of reinf.	A_s	275234	129554	107216	mm^2
• comp. reinforcement:	Distance from compressive reinf. to extreme Tension fiber		100	154	150	mm
	Diameter		36	36	28	mm
	Reinf. Space		150	150	150	mm
	Number of bar		135	105	210	bar
	Total area of reinf.	A'_s	137753	107216	129554	mm^2
Check Flexural Moment						
• Resistance factor		Φ	0.90	0.90	0.90	
• The corresponding effective		d_e	2814	2786	2846	mm
• Stress block factor		β_1	0.84	0.84	0.84	
• Depth of the equivalent stress block = c*β1		a	210.60	127.01	105.11	mm
• Distance from extreme compression fiber to the neutral axis		c	252.01	151.98	125.78	mm
• The nominal flexural resistance:		M_n	298210	141084	119801	kN.m
• Factored flexural resistance		$M_r = \Phi.M_n$	268389	126976	107821	kN.m
• Check condition		$M_r > M_u$	**O.K**	**O.K**	**O.K**	
Mimimum Reinforcement						
• Ratio of tension steel to gross area		$\rho = A_s/(b.d)$	0.48	0.29	0.24	%
• Check		$\rho > 0.03 \cdot f'_c/f'_y$	**O.K**	**O.K**	**O.K**	0.23
• Cracking moment		1.2Mcr	127329.06	99378.78	99378.78	Kn.m
• Check		Mr> min(1.2Mcr, 1.33Mu)	**O.K**	**O.K**	**O.K**	
Maximum Reinforcement						
• Obligation Condition		c/d_e	0.09	0.05	0.04	
• Check		$c/d_e < 0.42$	**O.K**	**O.K**	**O.K**	
Check shear resistance						
• Factored Shear force		V_u	74669.15	31122.29	24609.60	kN
• Resistance factor		Φ	0.90	0.90	0.90	
• The effective shear deepth		d_v	2709	2722	2793	mm
• Effective width		b_v	20500	16000	16000	mm
• Angle of inclination of diagonal compressive stress		θ	43	38	38	degree
• Angle of inclination of transerverse reinf. To longitudinal axis		α	90	90	90	degree
• Factor indicating ability of diagonally cracked concrete to transmit tension		β	1.75	1.95	1.95	
• Value		$0.1 \cdot f'_c \cdot b_v \cdot d_v$	166585	130680	134085	kN
• Max spacing of transverse reinforcement		s_{max}	600	600	600	mm
• Spacing of stirrup		s	300	450	450	mm
• Diameter of transverse reinforcement		∅	D 36	D 36	D 28	
• Number of transverse reinf. within distance s		n	2	2	2	bar
• Total area of transverse reinf.		A_v	2036	2036	1232	mm^2
• Diameter of stirrup		∅	D 20	D 20	D 20	mm
• Number of stirrup within distance s		n	69	35	35	bar
• Total area of stirrup		A_v	21781.71	10960.67	10960.67	
• Assume		θ	43.00	41.00	41.00	degree
• Strain in tensile reinforcement		ex	2.31E-03	1.19E-03	1.25E-03	
If ex<0, multiple with reduce factor		Fc	-	-	-	
• Ratio of shear stress and f'c		V/f'c	0.05	0.03	0.02	
• β final			1.75	1.95	1.95	
• θ final			43.00	38.00	38.00	
• The shear resistance of concrete:		V_c	44176.49	38615.36	39621.70	kN

• The shear resistance of stirrup	V_s	40168.85	18466.26	18947.51	kN
• Value	$0.25*f'_c \cdot b_v \cdot d_v$	416462.28	326699.16	335213.16	kN
• The nominal shear resistance:	V_n	84345.34	57081.62	58569.21	kN
• The factored shear resistance	V_r	75910.81	51373.46	52712.29	kN
• Check	$V_r > V_u$	**O.K**	**O.K**	**O.K**	
•Requiring transverse reinforcement	$V_u > 0.5 \cdot \Phi \cdot V_c$	**Need**	**Need**	**Need**	
•Check minimum transverse reinforcement	$A_v > 0.083 \cdot (f'_c{}^{0.5}) \cdot b_v \cdot s/f_y$	**O.K**	**O.K**	**O.K**	
Check Flexural and shear resistance at Extreme state					
• Factored Plexural moment	M_u	230043.84	37108.50	32978.70	kN.m
• Factored Shear force	V_u	74724.03	32810.34	25000.20	kN
• Resistance factor	Φ	1.00	1.00	1.00	
• The nominal flexural resistance:	M_n	298210	141084	119801	kN.m
• Factored flexural resistance	$M_r = \Phi.M_n$	298210	141084	119801	kN.m
• The nominal flexural resistance:	V_n	84345	57082	58569	Kn
• Factored flexural resistance	$V_r = \Phi.V_n$	84345	57082	58569	Kn
• Check condition	$M_r > M_u$	**O.K**	**O.K**	**O.K**	
	$V_r > V_u$	**O.K**	**O.K**	**O.K**	
Check crack					
Interior force combination	**Service III**				
• Factored moment	M_u	1.83E+05	2.79E+04	2.74E+04	kN.m
• Modulus of rupture of concrete	fr = 0.63*sqrt(f'c)	3.45	3.45	3.45	MPa
• Distance from extreme tension fiber to the neutral axis	$y_t = h-c$	2748	2848	2874	mm
• Stress of concrete at tension fiber	$f'r = Ms*y_t/I_g$	11	2	2	MPa
• Check	f'r >	0.8*fr	0.8*fr	0.8*fr	
		check crack	No check	No check	
• Crack width parameter	Z	= 23000	= 23000	= 23000	N/mm
• Ratio of reinf. Modulus with concrete modulus	$n = E_s/E_c$	= 7.00	= 7.00	= 7.00	
• The distance from extreme fiber to the neutral axis	c	= 827.31	= 621.51	= 565.75	mm
• Effective moment of inertia	J	1.15E+13	5.53E+12	4.87E+12	mm^4
• Arm	de-c	= 1986.69	= 2164.49	= 2280.25	mm
• Tension stress in reinforcement	fs =n*Ms*(de-c)/J	= 221.53	= 76.47	= 89.90	MPa
• Area of concrete having the same centroid as the principal tensile reinforcement divided by number of bars	A	= 15163	= 15209	= 15190	mm2
• Tension stress in reinforcement with service state	$f_{sa} = Z/(d_c*A)^{1/3}$	= 227.67	= 227.43	= 232.18	Mpa
• Check condition	$f_s < f_{sa}$	**O.K**	**O.K**	**O.K**	
• Check condition	$f_s < 0.6*f_y$	**O.K**	**O.K**	**O.K**	

2.11 PIER P16
2.11 TRỤ CẦU P16

PIER P16 - CALCULATION SHEET

CONTENT:

2.11.1. GENERAL DATA

CALCULATION PROCEDURE & STANDARD:

- Bridge Design Standard 22 TCN - 272 - 05

2.11.1.1. Design live load

Design vehicle load		HL93	22TCN 272 - 05
Number of lane		6	(lane)
Pedestrian		0.00	KG/m^2

2.11.1.2. Bridge width

Width carriageway	$B =$	12.00	(m)
Width of median guardrail	$B_{pc} =$	0.50	(m)
Width parapet	$B_{lc} =$	0.50	(m)
Bridge width	$B =$	13.00	(m)

2.11.1.3. Superstructure:

Span-arrangement		Continuous box girder 65+5@100+65m	
Height of box girder at pier section	$H =$	6.00	(m)
Height of box girder at mid-span section	$h =$	2.50	(m)
Pavement thickness	$d_{BTN} =$	0.084	(m)

2.11.1.4. Material property:

Concrete

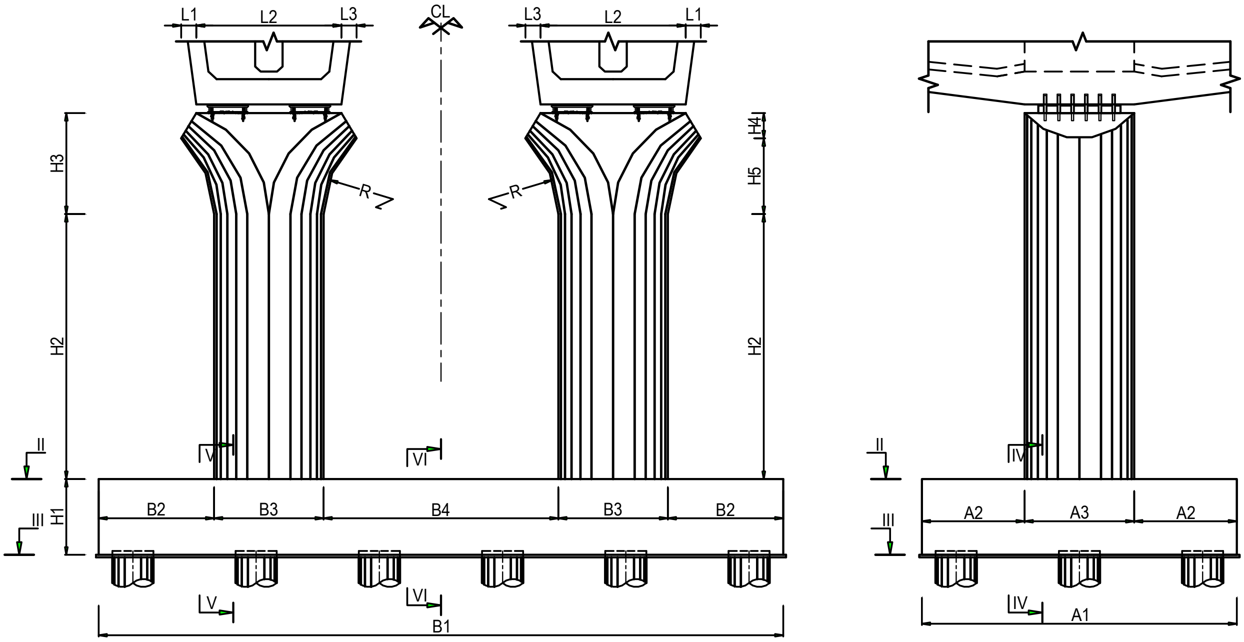
Compressive strength of cylindrical at 28 day	$f'c =$	30.00	MPa
Concrete density	$g =$	24.50	KN/m^3
Elastic modulus	$Ec =$	29440	MPa
Tension strength of concrete	$fr =$	3.45	MPa

Steel

Steel modulus	$Es =$	200000	MPa
Yeild strength of steel bar	$fy =$	400.00	MPa
Calculation unit:		KN, m, KN.m, Mpa, KN/m^2	

2.11.1.5. THE PIER GEOMETRIC

GENERAL DIMENSIONS



vertical			Horizontal			Thickness		
Remark	Value	Unit	Remark	Value	Unit	Remark	Value	Unit
a_1 =	11.50	(m)	L_1 =	0.55	(m)	h_1 =	3.00	(m)
a_2 =	3.75	(m)	L_2 =	5.30	(m)	h_2 =	3.500	(m)
a_3 =	4.00	(m)	L_3 =	0.55	(m)	h_3 =	4.000	(m)
			B_1 =	25.00	(m)	h_4 =	1.000	(m)
			B_2 =	3.965	(m)	h_5 =	3.000	(m)
			B_3 =	4.00	(m)			
			B_4 =	9.070	(m)			
			R =	4.34	(m)			

The design elevation:

Proposed height	EL_{mc} =	16.750	(m)
Elevation of top of pier cap	EL_{xm} =	10.545	(m)
Hight water level (H1%)	EL_{MNTK} =	9.200	(m)
Daily water level (H5%)	EL_{MNTB} =	4.070	(m)
Ground elevation	EL_{TN} =	2.240	(m)
Daily water level (H5%)	EL_{TT} =	4.070	(m)
Top of Pile cap (Section II - II)	$EL_{MC\ II-II}$ =	2.645	(m)
Bottom of Pile cap (Section III - III)	$EL_{MCIII-III}$ =	-0.355	(m)

2.11.2. CALCULATED PIER FORCE AND PILE FORCE

2.11.2.1. Loading combination at bottom of pier shaft

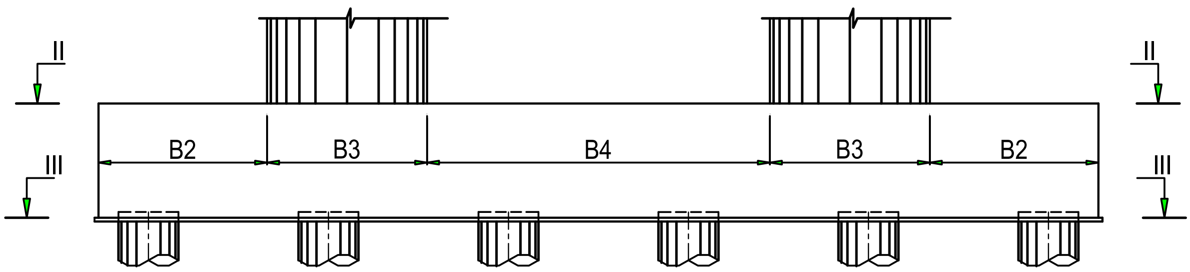
(Result from RM2010 SPACE-FRAME program)

Loading combination in accordance with 22 TCN 272-05, table 3.4.1.1 was mentioned in appendix of this report

LOADING COMBINATION AT TOP OF PILE CAP DUE TO SUPERSTRUCTURE + VESSEL COLLISION AND SEFTWEIGHT						
Limited state	Vertical axis	Longitudinal axis		Horizontal axis		Remarks
	N_x (kN)	H_y (kN)	M_z (kNm)	H_z (kN)	M_y (kNm)	
STRENGTH I	49582.00	38.00	117.00	201.00	7750.00	
STRENGTH II	42178.00	38.00	117.00	1964.00	23708.00	
STRENGTH III	47890.00	38.00	117.00	898.00	15069.00	
SERVICE I	37614.00	38.00	117.00	924.00	13967.00	
SERVICE III	37654.00	38.00	117.00	925.00	13980.00	
EXTREME EVENT (CV)	44019.00	38.00	117.00	1056.00	15129.00	
EXTREME EVENT (EQ trans)	44059.00	38.00	117.00	2466.00	30575.00	
EXTREME EVENT (EQ long)	44244.00	378.00	1172.00	1094.00	12552.00	

2.11.2.2. Loading combination at bottom of pile cap

LOADING COMBINATION AT TOP OF PILE CAP DUE TO SUPERSTRUCTURE + VESSEL						
Limited state	Vertical axis	Longitudinal axis		Horizontal axis		Remarks
	N_x (kN)	H_y (kN)	M_z (kNm)	H_z (kN)	M_y (kNm)	
STRENGTH I	136914	112	516	427	15828	
STRENGTH II	122106	112	516	3902	59631	
STRENGTH III	133529	112	516	1821	34856	
SERVICE I	105629	112	516	1874	32995	
SERVICE III	105707	112	516	1877	33029	
EXTREME EVENT (CV)	125787	56	258	1069	18250	
EXTREME EVENT (EQ trans)	125868	112	516	4959	75727	
EXTREME EVENT (EQ long)	126237	1122	5163	2445	31791	



2.11.2.3. Loading combination at top of piling

2.11.2.3.1. Piling material:

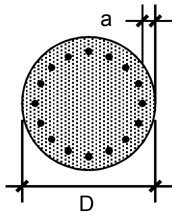
Concrete 30 Mpa

E_c (Mpa)	294401
γ_c (KN/m^3)	24.5

Steel bar

Type	CB-400-T
E_s (Mpa)	200000

2.11.2.3.2. Piling dimension



+ Diameter **D** = 1.50 m

a = 0.100 m

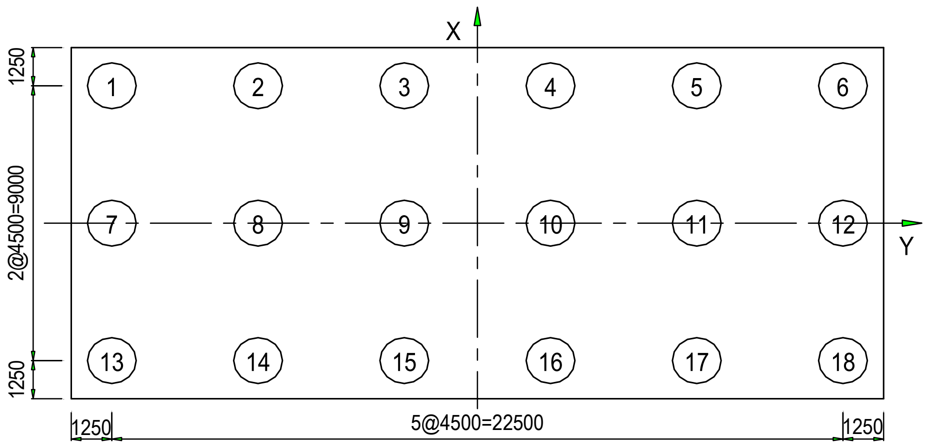
+ Length **L** = 60.00 m

2.11.2.3.3. Maximum Internal force and displacement at top piling

Maximum Internal force and diplacements *(Result follow Piling software)*

Combination	N	Hx	My	x	y	z
	(KN)	(KN)	(KN.m)	(m)	(m)	(rad)
Strength I	7810.48	6.22	25.76	-	-	-
Strength II	7615.27	6.22	25.76	-	-	-
Strength III	7889.31	6.22	25.76	-	-	-
Service I	6322.51	6.22	25.76	**0.000**	**-0.002**	**0.007**
Service III	6327.35	6.22	25.76	**0.000**	**-0.002**	**0.007**
Extreme I (CV)	7240.04	3.11	12.88	-	-	-
Extreme II (EQ trans)	8044.03	6.22	25.76	-	-	-
Extreme II (EQ long)	7646.41	62.33	258.08	-	-	-

- Check displacement of top pile not exceed 38mm (10.7.2.7) **OK**



Arrangement of pile

2.11.2.3.4. Internal force for each pile

Combination	Pile Number	Hx (KN)	Hy (KN)	N (KN.m)	Mx (m)	My (m)
Strength I	1	6.22	-23.72	7774.19	97.18	25.76
	2	6.22	-23.72	7699.79	97.18	25.76
	3	6.22	-23.72	7625.39	97.18	25.76
	4	6.22	-23.72	7550.99	97.18	25.76
	5	6.22	-23.72	7476.59	97.18	25.76
	6	6.22	-23.72	7402.19	97.18	25.76
	7	6.22	-23.72	7792.34	97.18	25.76
	8	6.22	-23.72	7717.93	97.18	25.76
	9	6.22	-23.72	7643.53	97.18	25.76
	10	6.22	-23.72	7569.13	97.18	25.76
	11	6.22	-23.72	7494.73	97.18	25.76
	12	6.22	-23.72	7420.33	97.18	25.76
	13	6.22	-23.72	7810.48	97.18	25.76
	14	6.22	-23.72	7736.08	97.18	25.76
	15	6.22	-23.72	7661.68	97.18	25.76
	16	6.22	-23.72	7587.28	97.18	25.76
	17	6.22	-23.72	7512.87	97.18	25.76
	18	6.22	-23.72	7438.47	97.18	25.76
Strength II	1	6.22	-216.78	7578.99	957.83	25.76
	2	6.22	-216.78	7253.60	957.83	25.76
	3	6.22	-216.78	6928.22	957.83	25.76
	4	6.22	-216.78	6602.83	957.83	25.76
	5	6.22	-216.78	6277.45	957.83	25.76
	6	6.22	-216.78	5952.06	957.83	25.76
	7	6.22	-216.78	7597.13	957.83	25.76
	8	6.22	-216.78	7271.74	957.83	25.76
	9	6.22	-216.78	6946.36	957.83	25.76
	10	6.22	-216.78	6620.98	957.83	25.76
	11	6.22	-216.78	6295.59	957.83	25.76
	12	6.22	-216.78	5970.21	957.83	25.76
	13	6.22	-216.78	7615.27	957.83	25.76
	14	6.22	-216.78	7289.89	957.83	25.76
	15	6.22	-216.78	6964.50	957.83	25.76
	16	6.22	-216.78	6639.12	957.83	25.76
	17	6.22	-216.78	6313.73	957.83	25.76
	18	6.22	-216.78	5988.35	957.83	25.76
Strength III	1	6.22	-101.17	7853.03	441.23	25.76
	2	6.22	-101.17	7671.87	441.23	25.76
	3	6.22	-101.17	7490.71	441.23	25.76
	4	6.22	-101.17	7309.56	441.23	25.76
	5	6.22	-101.17	7128.40	441.23	25.76
	6	6.22	-101.17	6947.25	441.23	25.76
	7	6.22	-101.17	7871.17	441.23	25.76
	8	6.22	-101.17	7690.01	441.23	25.76
	9	6.22	-101.17	7508.86	441.23	25.76
	10	6.22	-101.17	7327.70	441.23	25.76
	11	6.22	-101.17	7146.54	441.23	25.76
	12	6.22	-101.17	6965.39	441.23	25.76
	13	6.22	-101.17	7889.31	441.23	25.76
	14	6.22	-101.17	7708.16	441.23	25.76
	15	6.22	-101.17	7527.00	441.23	25.76
	16	6.22	-101.17	7345.84	441.23	25.76
	17	6.22	-101.17	7164.69	441.23	25.76
	18	6.22	-101.17	6983.53	441.23	25.76

- Result for internal force at top pile

Combination	Pile Number	**Hx** (KN)	**Hy** (KN)	**N** (KN.m)	**Mx** (m)	**My** (m)
Service I	1	6.22	-104.11	6286.23	456.44	25.76
	2	6.22	-104.11	6111.79	456.44	25.76
	3	6.22	-104.11	5937.35	456.44	25.76
	4	6.22	-104.11	5762.92	456.44	25.76
	5	6.22	-104.11	5588.48	456.44	25.76
	6	6.22	-104.11	5414.04	456.44	25.76
	7	6.22	-104.11	6304.37	456.44	25.76
	8	6.22	-104.11	6129.93	456.44	25.76
	9	6.22	-104.11	5955.50	456.44	25.76
	10	6.22	-104.11	5781.06	456.44	25.76
	11	6.22	-104.11	5606.62	456.44	25.76
	12	6.22	-104.11	5432.18	456.44	25.76
	13	6.22	-104.11	6322.51	456.44	25.76
	14	6.22	-104.11	6148.08	456.44	25.76
	15	6.22	-104.11	5973.64	456.44	25.76
	16	6.22	-104.11	5799.20	456.44	25.76
	17	6.22	-104.11	5624.76	456.44	25.76
	18	6.22	-104.11	5450.33	456.44	25.76
Service III	1	6.22	-104.28	6291.07	457.18	25.76
	2	6.22	-104.28	6116.43	457.18	25.76
	3	6.22	-104.28	5941.79	457.18	25.76
	4	6.22	-104.28	5767.15	457.18	25.76
	5	6.22	-104.28	5592.51	457.18	25.76
	6	6.22	-104.28	5417.87	457.18	25.76
	7	6.22	-104.28	6309.21	457.18	25.76
	8	6.22	-104.28	6134.57	457.18	25.76
	9	6.22	-104.28	5959.93	457.18	25.76
	10	6.22	-104.28	5785.29	457.18	25.76
	11	6.22	-104.28	5610.65	457.18	25.76
	12	6.22	-104.28	5436.02	457.18	25.76
	13	6.22	-104.28	6327.35	457.18	25.76
	14	6.22	-104.28	6152.71	457.18	25.76
	15	6.22	-104.28	5978.07	457.18	25.76
	16	6.22	-104.28	5803.43	457.18	25.76
	17	6.22	-104.28	5628.80	457.18	25.76
	18	6.22	-104.28	5454.16	457.18	25.76
Extreme Event I (with CV)	1	3.11	-59.39	7221.90	260.84	12.88
	2	3.11	-59.39	7124.78	260.84	12.88
	3	3.11	-59.39	7027.66	260.84	12.88
	4	3.11	-59.39	6930.54	260.84	12.88
	5	3.11	-59.39	6833.41	260.84	12.88
	6	3.11	-59.39	6736.29	260.84	12.88
	7	3.11	-59.39	7230.97	260.84	12.88
	8	3.11	-59.39	7133.85	260.84	12.88
	9	3.11	-59.39	7036.73	260.84	12.88
	10	3.11	-59.39	6939.61	260.84	12.88
	11	3.11	-59.39	6842.48	260.84	12.88
	12	3.11	-59.39	6745.36	260.84	12.88
	13	3.11	-59.39	7240.04	260.84	12.88
	14	3.11	-59.39	7142.92	260.84	12.88
	15	3.11	-59.39	7045.80	260.84	12.88
	16	3.11	-59.39	6948.68	260.84	12.88
	17	3.11	-59.39	6851.55	260.84	12.88
	18	3.11	-59.39	6754.43	260.84	12.88

- Result for internal force at top pile

Combination	Pile Number	**Hx** (KN)	**Hy** (KN)	**N** (KN.m)	**Mx** (m)	**My** (m)
Extreme Event I (with EQtran)	1	6.22	-275.50	8007.74	1217.34	25.76
	2	6.22	-275.50	7594.46	1217.34	25.76
	3	6.22	-275.50	7181.17	1217.34	25.76
	4	6.22	-275.50	6767.88	1217.34	25.76
	5	6.22	-275.50	6354.59	1217.34	25.76
	6	6.22	-275.50	5941.31	1217.34	25.76
	7	6.22	-275.50	8025.89	1217.34	25.76
	8	6.22	-275.50	7612.60	1217.34	25.76
	9	6.22	-275.50	7199.31	1217.34	25.76
	10	6.22	-275.50	6786.02	1217.34	25.76
	11	6.22	-275.50	6372.74	1217.34	25.76
	12	6.22	-275.50	5959.45	1217.34	25.76
	13	6.22	-275.50	8044.03	1217.34	25.76
	14	6.22	-275.50	7630.74	1217.34	25.76
	15	6.22	-275.50	7217.45	1217.34	25.76
	16	6.22	-275.50	6804.17	1217.34	25.76
	17	6.22	-275.50	6390.88	1217.34	25.76
	18	6.22	-275.50	5977.59	1217.34	25.76
Extreme Event I (with EQlong)	1	62.33	-135.83	7283.14	604.76	258.08
	2	62.33	-135.83	7102.49	604.76	258.08
	3	62.33	-135.83	6921.85	604.76	258.08
	4	62.33	-135.83	6741.21	604.76	258.08
	5	62.33	-135.83	6560.57	604.76	258.08
	6	62.33	-135.83	6379.93	604.76	258.08
	7	62.33	-135.83	7464.77	604.76	258.08
	8	62.33	-135.83	7284.13	604.76	258.08
	9	62.33	-135.83	7103.49	604.76	258.08
	10	62.33	-135.83	6922.85	604.76	258.08
	11	62.33	-135.83	6742.20	604.76	258.08
	12	62.33	-135.83	6561.56	604.76	258.08
	13	62.33	-135.83	7646.41	604.76	258.08
	14	62.33	-135.83	7465.77	604.76	258.08
	15	62.33	-135.83	7285.12	604.76	258.08
	16	62.33	-135.83	7104.48	604.76	258.08
	17	62.33	-135.83	6923.84	604.76	258.08
	18	62.33	-135.83	6743.20	604.76	258.08

2.11.3. CHECK FOR PIER SHAFT

Item		Mark	Unit	Value
• Factored Axial force		Nu	Kn	37654.00
• Factored Plexural moment		Mux	Kn.m	117.00
• Factored Plexural moment		Muy	Kn.m	13980.00
• Diameter of Pier shaft		D	m	4.00
• Section area		Ag	m2	12.57
• Moment of inertia of concrete section		Ic	m4	12.57
• Cover thickness		a	m	0.075
• Reinf. Diameter		Ds	mm	32.00
• Number of rebar		n_s	nos	168.00
• Rebar area		As	mm2	135113.62
Check minimum reinforcement				
• Minimum rebar area required (0.135*f'c/fy)*Ag		As req	mm2	127234.50
• Check condition As > (0.135*f'c/fy)*Ag				**OK**
Check maximum reinforcement				
• Maximum rebar area 0.08*Ag		As max	mm2	1005309.6
• Check condition As < 0.08*Ag				**OK**
Check ratio spiral or Tier (5.7.4.6)				
• Distance to outside of Spairal or Ties to concrete face			mm	66.00
• Effect diamete		Deff	m	3.87
• Area of core measured to the outside diameter of the spiral			m2	11.75
• Ratio spiral Rebar required		ρsa		0.00234
Required Area of Spiral Rebar	space		mm	200
	Effective length			3.87
	layer			2
	Area			226.6
	Requaired Dhs			17.0
Actuaral	Effective length	d	m	3.868
	Diameter	Dhr	mm	18
	Area of Rebar	Ah	mm2	254.5
	layer	Nl	nos	2
	Total area of spiral	Ac	m2	508.938
	space	s	mm	200
	Ratio spiral Rebar	ρs	-	0.0026315
• Check condition		$\rho_s > \rho_{sa}$		OK
Check Crack (At Service State)				
• Modulus of rupture of concrete		fr = 0.63*sqrt(f'c)	Mpa	3.45
• Stress of concrete at tension fiber		σ'_r	Mpa	1.75
• If f'r > 0.8fr require check crack		$\sigma'_r > 0.8*\sigma_r$	Mpa	No check
• Center of newtral axial		x	mm	3.025
• Maximum stress of Compression fiber of concrete		σ_c	Mpa	7.96
• Maximum stress of Compression Rebar		σ_{rc}	Mpa	-124.3
• Maximum stress of Tension Rebar		σ_{rt}	Mpa	31.4
• Check			σ_{rt} <0.6.fy	OK

2.11.4. CHECK FOR PILE

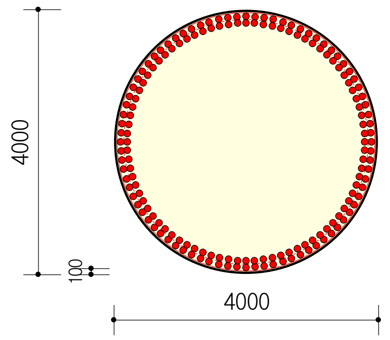
Item		Mark	Unit	Value
• Factored Axial force		Nu	Kn	6291.07
• Factored Plexural moment		Mux	Kn.m	457.18
• Factored Plexural moment		Muy	Kn.m	25.76
• Diameter of Pier shaft		D	m	1.50
• Section area		Ag	m2	1.77
• Moment of inertia of concrete section		Ic	m4	0.25
• Cover thickness		a	m	0.075
• Reinf. Diameter		Ds	mm	D32
• Number of rebar		n_s	nos	24
• Rebar area		As	mm2	19301.95
Check minimum reinforcement				
• Minimum rebar area required (0.135*f'c/fy)*Ag		As req	mm2	17892.35
• Check condition As > (0.135*f'c/fy)*Ag				**OK**
Check maximum reinforcement				
• Maximum rebar area 0.08*Ag		As max	mm2	141371.7
• Check condition As < 0.08*Ag				**OK**
Check ratio spiral or Tier (5.7.4.6)				
• Distance to outside of Spairal or Ties to concrete face			mm	68.00
• Effect diamete		Deff	m	1.36
• Area of core measured to the outside diameter of the spiral			m2	1.46
• Ratio spiral Rebar required		ρsa		0.00707
Required Area of Spiral Rebar	space		mm	75
	Effective length			1.36
	layer			1
	Area			180.7
	Requaired Dhs			15.2
Actuaral	Effective length	d	m	1.364
	Diameter	Dhr	mm	16
	Area of Rebar	Ah	mm2	201.1
	layer	Nl	nos	1
	Total area of spiral	Ac	m2	201.062
	space	s	mm	75
	Ratio spiral Rebar	ρs	-	0.0078617
• Check condition		$\rho_s > \rho_{sa}$		OK

MIDAS/Set **Column Design [Pier Shaft D4.0m-P16]**

	Company	Nipponkoei	Project Name	DN Qn Expressway
	Designer	vinh	File Name	D:\...\bodypier-D4.0m-P16.BOI

1. Geometry and Materials

Design Code : ACI318M-02
Stress Profile : Equivalent Stress Block
Material Data : f_c' = 30 MPa (β_1 = 0.850)
f_y = 400, f_{ys} = 400 MPa
Section Dim. : Φ4000 mm
Effective Len. : KL_u = 28000 mm
Steel Distribut.: 84 - D32 (d_c = 100 mm)
: 84 - D32 (d_c = 200 mm)
Total Steel Area A_{st} = 131896 mm^2 (ρ_{st} = 0.0106)



2. Member Force and Moment

Unit : kN, kN-m

L.C.	P_u	M_{ux}	M_{uy}	R_{atioV}	V_{ux}	V_{uy}	R_{atioH}	Remark
1	49582.0	117.0	7750.0	0.265	38.0	201.0	0.007	
2	42178.0	117.0	23708.0	0.277	38.0	1964.0	0.064	
3	47890.0	117.0	15069.0	0.261	38.0	898.0	0.029	
4	37614.0	117.0	13967.0	0.212	38.0	924.0	0.030	
5	37654.0	117.0	13980.0	0.212	38.0	925.0	0.030	
6	44019.0	117.0	15129.0	0.244	38.0	1056.0	0.035	
7	44059.0	117.0	30575.0	0.325	38.0	2466.0	0.081	
8	44244.0	1172.0	12552.0	0.237	378.0	1094.0	0.038	

3. Magnified Moment

KL_u/r_x = 7500/1000 = 7.50 < 34-12(M_1/M_2) = 22.00
δ_x = 1.000

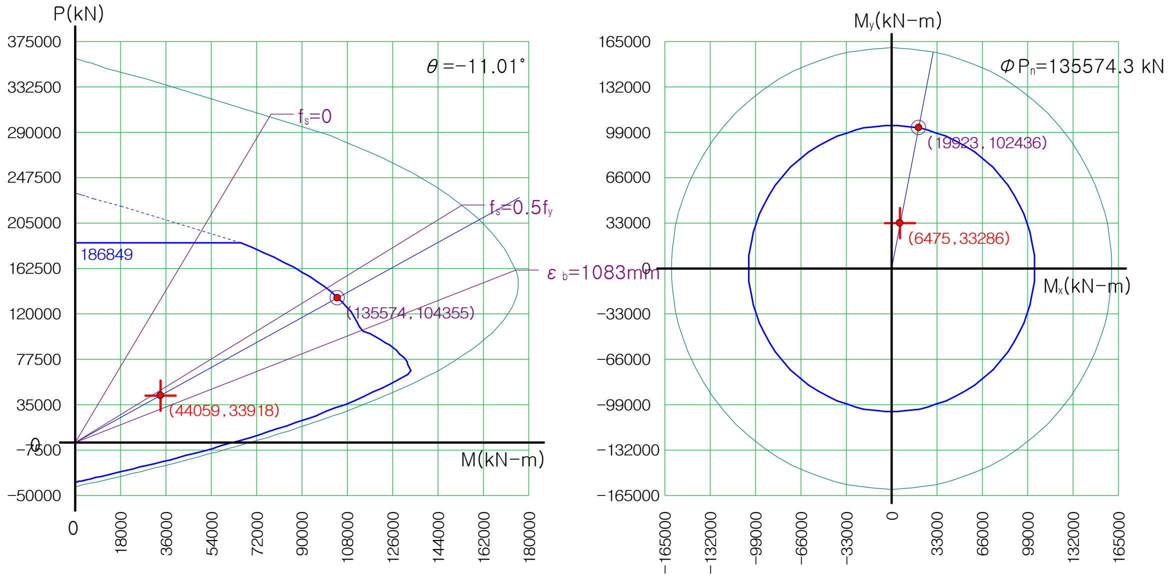
KL_u/r_y = 7500/1000 = 7.50 < 34-12(M_1/M_2) = 22.00
δ_y = 1.000

4. Design Force and Moment

Design Load Combination No : 7
P_u = 44059.0 kN
M_{ux} = 117.0, M_{uy} = 30575.0 kN-m
$\delta_x M_{ux}$= δ_x*MAX[M_{ux}, $P_u e_{min}$] = 6475.4 kN-m
$\delta_y M_{uy}$= δ_y*M_{uy}, = 33286.3 kN-m

5. Check Axial and Moment Capacity

Rotation Angle and Depth to the Neutral Axis θ = -11.01°, c = 2816 mm
Strength Reduction Factor Φ = 0.6500
Maximum Axial Load $\Phi P_{n(max)}$ = 186848.8 kN
Design Axial Load Strength ΦP_n = 135574.3 kN
Design Moment Strength ΦM_{nx} = 19922.6 kN-m
ΦM_{ny} = 102435.7 kN-m
Strength Ratio : Applied/Design = 0.325 < 1.000 .... O.K



6. Check Shear Capacity

Design Load Combination No : 7

Strength Reduction Factor Φ= 0.750

Design Force Vu = 2466.3 kN (Pu = 44059.0 kN)

Required Hoop Spacing :D18 @ 448 mm

Provided Hoop Spacing :D18 @ 200 mm (Tie)

$\Phi V_c + \Phi V_s$ = 27857.9 + 2734.6 = 30592.4 kN > V_u = 2466.3 kN .... O.K

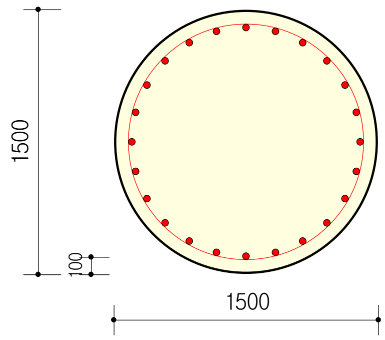
2.11.4. CHECK FOR PILE

Item		Mark	Unit	Value
• Factored Axial force		Nu	Kn	6291.07
• Factored Plexural moment		Mux	Kn.m	457.18
• Factored Plexural moment		Muy	Kn.m	25.76
• Diameter of Pier shaft		D	m	1.50
• Section area		Ag	m2	1.77
• Moment of inertia of concrete section		Ic	m4	0.25
• Cover thickness		a	m	0.075
• Reinf. Diameter		Ds	mm	D32
• Number of rebar		n_s	nos	24
• Rebar area		As	mm2	19301.95
Check minimum reinforcement				
• Minimum rebar area required (0.135*f'c/fy)*Ag		As req	mm2	17892.35
• Check condition As > (0.135*f'c/fy)*Ag				**OK**
Check maximum reinforcement				
• Maximum rebar area 0.08*Ag		As max	mm2	141371.7
• Check condition As < 0.08*Ag				**OK**
Check ratio spiral or Tier (5.7.4.6)				
• Distance to outside of Spairal or Ties to concrete face			mm	68.00
• Effect diamete		Deff	m	1.36
• Area of core measured to the outside diameter of the spiral			m2	1.46
• Ratio spiral Rebar required		ρsa		0.00707
Required Area of Spiral Rebar	space		mm	75
	Effective length			1.36
	layer			1
	Area			180.7
	Requaired Dhs			15.2
Actuaral	Effective length	d	m	1.364
	Diameter	Dhr	mm	16
	Area of Rebar	Ah	mm2	201.1
	layer	Nl	nos	1
	Total area of spiral	Ac	m2	201.062
	space	s	mm	75
	Ratio spiral Rebar	ρs	-	0.0078617
• Check condition		$\rho_s > \rho_{sa}$		OK

	Company	Nipponkoei	Project Name	DN Qn Expressway
	Designer	vinh	File Name	D:\...\Pile D1.5m - Pier P16.B01

1. Geometry and Materials

Design Code : ACI318M-02
Stress Profile : Equivalent Stress Block
Material Data : f_c' = 30 MPa (β_1 = 0.850)
f_y = 400, f_{ys} = 400 MPa
Section Dim. : Φ1500 mm
Effective Len. : KL_u = 15000 mm
Steel Distribut.: 24 - D32 (d_c = 100 mm)
Total Steel Area A_{st} = 19061 mm^2 (ρ_{st} = 0.0108)



2. Member Force and Moment

Unit : kN, kN-m

L.C.	P_u	M_{ux}	M_{uy}	R_{atioV}	V_{ux}	V_{uy}	R_{atioH}	Remark
1	7810.5	97.0	26.0	0.288	6.0	24.0	0.005	
2	7615.3	958.0	26.0	0.331	6.0	217.0	0.045	
3	7889.3	441.0	26.0	0.291	6.0	101.0	0.021	
4	6322.5	456.0	26.0	0.234	6.0	104.0	0.022	
5	6327.4	457.0	26.0	0.234	6.0	104.0	0.022	
6	7240.0	261.0	13.0	0.267	3.0	59.0	0.012	
7	8044.0	1217.0	26.0	0.383	6.0	276.0	0.058	
8	7646.4	605.0	258.0	0.294	62.0	136.0	0.031	

3. Magnified Moment

KL_u/r_x = 15000/375 = 40.00 > 34-12(M_1/M_2) = 22.00
δ_x = MAX[1.00/(1-P_u/0.75/29394), 1.0] = 1.575

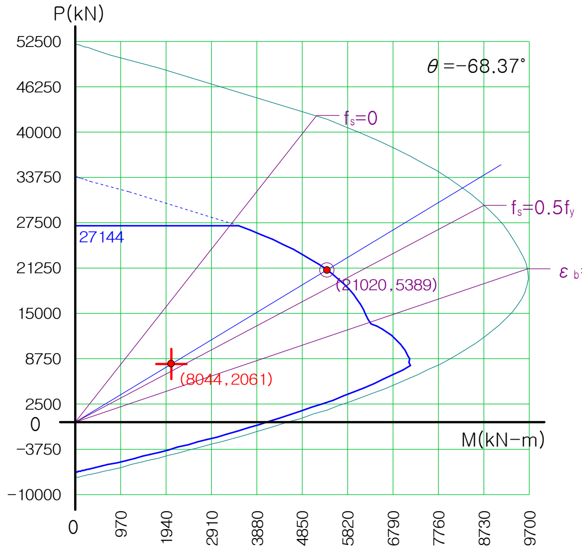
KL_u/r_y = 15000/375 = 40.00 > 34-12(M_1/M_2) = 22.00
δ_y = MAX[1.00/(1-P_u/0.75/29394), 1.0] = 1.575

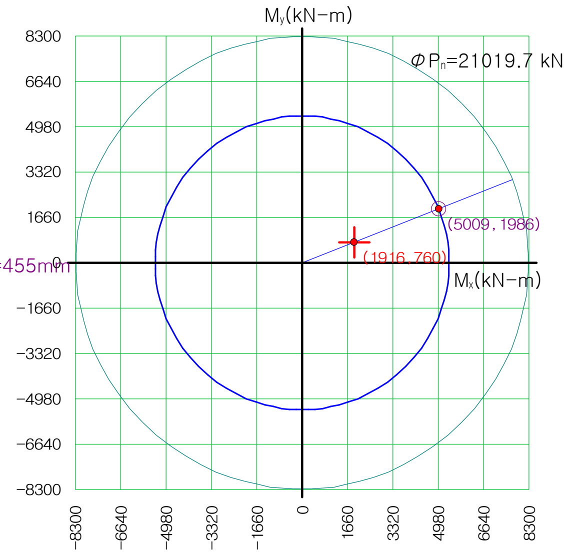
4. Design Force and Moment

Design Load Combination No : 7
P_u = 8044.0 kN
M_{ux} = 1217.0, M_{uy} = 26.0 kN-m
$\delta_x M_{ux}$= δ_x*M_{ux} = 1916.2 kN-m
$\delta_y M_{uy}$= δ_y*MAX[M_{uy}, $P_u e_{min}$] = 759.9 kN-m

5. Check Axial and Moment Capacity

Rotation Angle and Depth to the Neutral Axis θ = -68.37°, c = 1123 mm
Strength Reduction Factor Φ = 0.6500
Maximum Axial Load $\Phi P_{n(max)}$ = 27144.3 kN
Design Axial Load Strength ΦP_n = 21019.7 kN
Design Moment Strength ΦM_{nx} = 5009.4 kN-m
ΦM_{ny} = 1986.3 kN-m
Strength Ratio : Applied/Design = 0.383 < 1.000 .... O.K





6. Check Shear Capacity

Design Load Combination No : 7

Strength Reduction Factor Φ= 0.750

Design Force Vu = 276.1 kN (Pu = 8044.0 kN)

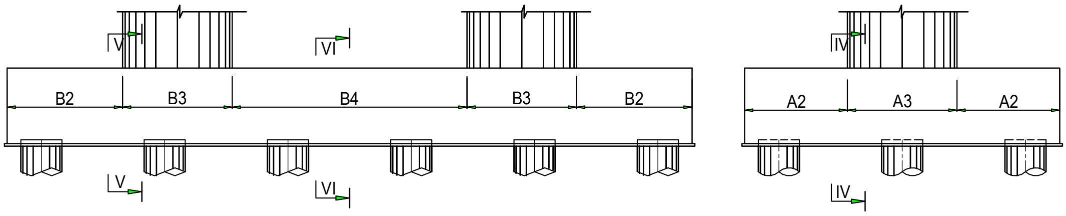
Required Hoop Spacing : D16 @ 508 mm

Provided Hoop Spacing : D16 @ 150 mm (Tie)

$\Phi V_c + \Phi V_s$ = 3858.8 + 924.5 = 4783.3 kN > V_u = 276.1 kN .... O.K

2.11.5. CHECK FOR PILE CAP

2.11.5.1. The Force to section IV-IV, section V-V, section VI-VI



At section IV - IV

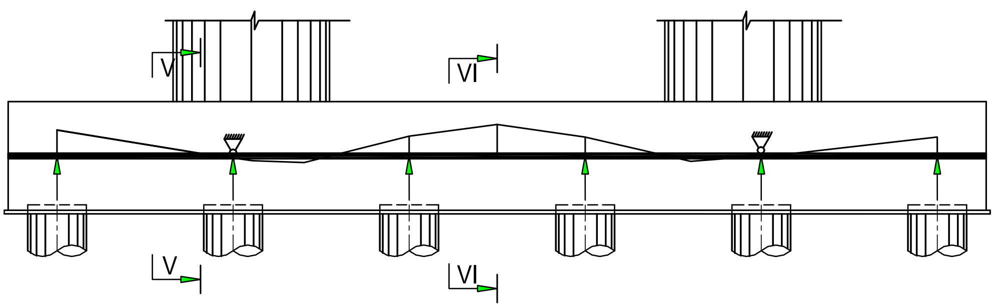
COMBINATION	Longitudinal direction		
	Q (kN)	M(kN.m)	V (kN.m)
Strength I	45746.9	144880.3	56.0
Strength II	40810.9	129248.0	56.0
Strength III	44618.5	141306.9	56.0
Service I	35318.5	111853.7	56.0
Service III	11160.3	35344.5	56.0
Extreme I (CV)	41983.4	132961.5	28.0
Extreme II (EQ trans)	42064.9	133219.4	56.0
Extreme II (EQ long)	43168.8	136715.6	561.00

At section V - V

COMBINATION	Transverse direction		
	Q (kN)	M(kN.m)	V (kN.m)
Strength I	23377.0	84881.9	-47.4
Strength II	22791.4	82755.5	-433.6
Strength III	23613.5	85740.6	-202.3
Service I	18913.1	68673.5	-208.2
Service III	18927.6	68726.2	-208.6
Extreme I (CV)	14364.8	52158.7	-118.8
Extreme II (EQ trans)	24077.66	87426.0	-551.00
Extreme II (EQ long)	22394.31	81313.8	-271.67

At section VI - VI

COMBINATION	Transverse direction		
	Q (kN)	M(kN.m)	V (kN.m)
Strength I	28513.0	33124.0	-47.4
Strength II	25125.0	28152.0	-433.6
Strength III	26352.0	22536.0	-202.3
Service I	22536.0	23542.0	-208.2
Service III	22152.0	23524.0	-208.6
Extreme I (CV)	15623.0	18659.0	-118.8
Extreme II (EQ trans)	27956.00	29521.00	-551.0
Extreme II (EQ long)	34698.00	29653.00	-271.7



2.11.5.2. Ultimate load check, shear capacity check and crack control

*** Check flexure momen capacity of pile cap:**

Item		Symbol	Section IV-IV (Bottom bar)	Section V-V (Bottom bar)	Section VI-VI (Upper bar)	Unit
• Factored Plexural moment		M_u	144880.27	85740.64	33124.00	kN.m
• Factored Shear force		V_u	45746.85	23613.51	28513.00	kN
• Hight of Section		h	3000	3000	3000	mm
• Width of section		b	25000	11500	11500	mm
• Section area		A_c	75000000	34500000	34500000	mm^2
• Moment of inertia of concrete section		I_g	5.6E+13	2.6E+13	2.6E+13	mm^4
• Tension reinforcement:	Distance from tension reinf. to extreme compression fiber	d_c	228	260	146	mm
	Reinf. Diameter	∅	28	32	36	mm
	Space	@	150	150	150	mm
	Number of bar	n	330	150	75	bar
	Total area of reinf.	A_s	203445	120959	76680	mm^2
• comp. reinforcement:	Distance from compressive reinf. to extreme Tension fiber		100	146	152	mm
	Diameter		28	36	32	mm
	Reinf. Space		150	150	150	mm
	Number of bar		165	75	150	bar
	Total area of reinf.	A'_s	101804	76680	120959	mm^2
Check Flexural Moment at Strength state						
• Resistance factor		Φ	0.90	0.90	0.90	
• The corresponding effective		d_e	2772	2740	2854	mm
• Stress block factor		β_1	0.84	0.84	0.84	
• Depth of the equivalent stress block = c*β1		a	127.65	164.99	104.59	mm
• Distance from extreme compression fiber to the neutral axis		c	152.75	197.42	125.15	mm
• The nominal flexural resistance:		M_n	220385	128579	85934	kN.m
• Factored flexural resistance		$M_r = \Phi.M_n$	198347	115722	77340	kN.m
• Check condition		$M_r > M_u$	O.K	O.K	O.K	
Mimimum Reinforcement						
• Ratio of tension steel to gross area		$\rho = A_s/(b.d)$	0.29	0.38	0.23	%
• Check		$\rho > 0.03 \cdot f'_c/f'_y$	O.K	O.K	O.K	0.23
• Cracking moment		1.2Mcr	155279.35	71428.50	71428.50	Kn.m
• Check		Mr> min(1.2Mcr, 1.33Mu)	O.K	O.K	O.K	
Maximum Reinforcement						
• Obligation Condition		c/d_e	0.06	0.07	0.04	
• Check		$c/d_e < 0.42$	O.K	O.K	O.K	
Check shear resistance						
• Factored Shear force		V_u	45746.85	23613.51	28513.00	kN
• Resistance factor		Φ	0.90	0.90	0.90	
• The effective shear deepth		d_v	2708	2658	2802	mm
• Effective width		b_v	25000	11500	11500	mm
• Angle of inclination of diagonal compressive stress		θ	43	43	43	degree
• Angle of inclination of transerverse reinf. To longitudinal axis		α	90	90	90	degree
• Factor indicating ability of diagonally cracked concrete to transmit tension		β	1.95	1.95	1.95	
• Value		$0.1 \cdot f'_c \cdot b_v \cdot d_v$	203113	91684	96659	kN
• Max spacing of transverse reinforcement		s_{max}	600	600	600	mm
• Spacing of stirrup		s	450	450	450	mm
• Diameter of transverse reinforcement		∅	D 28	D 32	D 36	
• Number of transverse reinf. within distance s		n	6	6	3	bar
• Total area of transverse reinf.		A_v	3695	4825	3054	mm^2
• Diameter of stirrup		∅	D 20	D 20	D 20	mm
• Number of stirrup within distance s		n	57	27	27	bar
• Total area of stirrup		A_v	17767.45	8342.67	8342.67	
• Assume		θ	43.00	43.00	43.00	degree
• Strain in tensile reinforcement		ex	1.92E-03	1.86E-03	1.77E-03	
If ex<0, multiple with reduce factor		Fc	-	-	-	
• Ratio of shear stress and f'c		V/f'c	0.03	0.03	0.03	
• β final			1.95	1.95	1.95	
• θ final			43.00	43.00	43.00	
• The shear resistance of concrete:		V_c	60019.16	27092.26	28562.31	kN

• The shear resistance of stirrup	V_s	21839.74	10062.94	10608.97	kN
• Value	$0.25*f'_c \cdot b_v \cdot d_v$	507782.68	229209.77	241646.91	kN
• The nominal shear resistance:	V_n	81858.90	37155.20	39171.28	kN
• The factored shear resistance	V_r	73673.01	33439.68	35254.15	kN
• Check	**$V_r > V_u$**	**O.K**	**O.K**	**O.K**	
•Requiring transverse reinforcement	**$V_u > 0.5 \cdot \Phi \cdot V_c$**	**Need**	**Need**	**Need**	
•Check minimum transverse reinforcement	**$A_v > 0.083 \cdot (f'_c{}^{0.5}) \cdot b_v \cdot s/f_y$**	**O.K**	**O.K**	**O.K**	
Check Flexural and shear resistance at Extreme state					
• Factored Plexural moment	M_u	136715.64	87425.97	29653.00	kN.m
• Factored Shear force	V_u	43168.82	24077.66	34698.00	kN
• Resistance factor	Φ	1.00	1.00	1.00	
• The nominal flexural resistance:	M_n	220385	128579	85934	kN.m
• Factored flexural resistance	$M_r = \Phi.M_n$	220385	128579	85934	kN.m
• The nominal flexural resistance:	V_n	81859	37155	39171	Kn
• Factored flexural resistance	$V_r = \Phi.V_n$	81859	37155	39171	Kn
• Check condition	**$M_r > M_u$**	**O.K**	**O.K**	**O.K**	
	$V_r > V_u$	**O.K**	**O.K**	**O.K**	
Check crack					
Interior force combination	**Service I**				
• Factored moment	M_u	1.12E+05	6.87E+04	2.35E+04	kN.m
• Modulus of rupture of concrete	fr = 0.63*sqrt(f'c)	3.45	3.45	3.45	MPa
• Distance from extreme tension fiber to the neutral axis	y_t = h-c	2847	2803	2875	mm
• Stress of concrete at tension fiber	$f'r = Ms*y_t/I_g$	5.66	7.44	2.62	MPa
• Check	f'r >	0.8*fr	0.8*fr	0.8*fr	
		check crack	check crack	No check	
• Crack width parameter	Z	= 23000	= 23000	= 23000	N/mm
• Ratio of reinf. Modulus with concrete modulus	$n = E_s/E_c$	= 7.00	= 7.00	= 7.00	
• The distance from extreme fiber to the neutral axis	c	= 621.81	= 713.08	= 564.94	mm
• Effective moment of inertia	J	8.59E+12	4.87E+12	3.50E+12	mm^4
• Arm	de-c	= 2150.19	= 2026.92	= 2289.06	mm
• Tension stress in reinforcement	fs =n*Ms*(de-c)/J	= 196.04	= 200.29	= 107.66	MPa
• Area of concrete having the same centroid as the principal tensile reinforcement divided by number of bars	A	= 15133	= 15293	= 15265	mm2
• Tension stress in reinforcement with service state	$f_{sa} = Z/(d_c*A)^{1/3}$	= 232.47	= 229.29	= 227.15	Mpa
• Check condition	**$f_s < f_{sa}$**	**O.K**	**O.K**	**O.K**	
• Check condition	**$f_s < 0.6*f_y$**	**O.K**	**O.K**	**O.K**	

2.12 PILE CAPACITY
2.12 SỨC CHỊU TẢI CỌC KHOAN NHỒI

1. Soil Condition

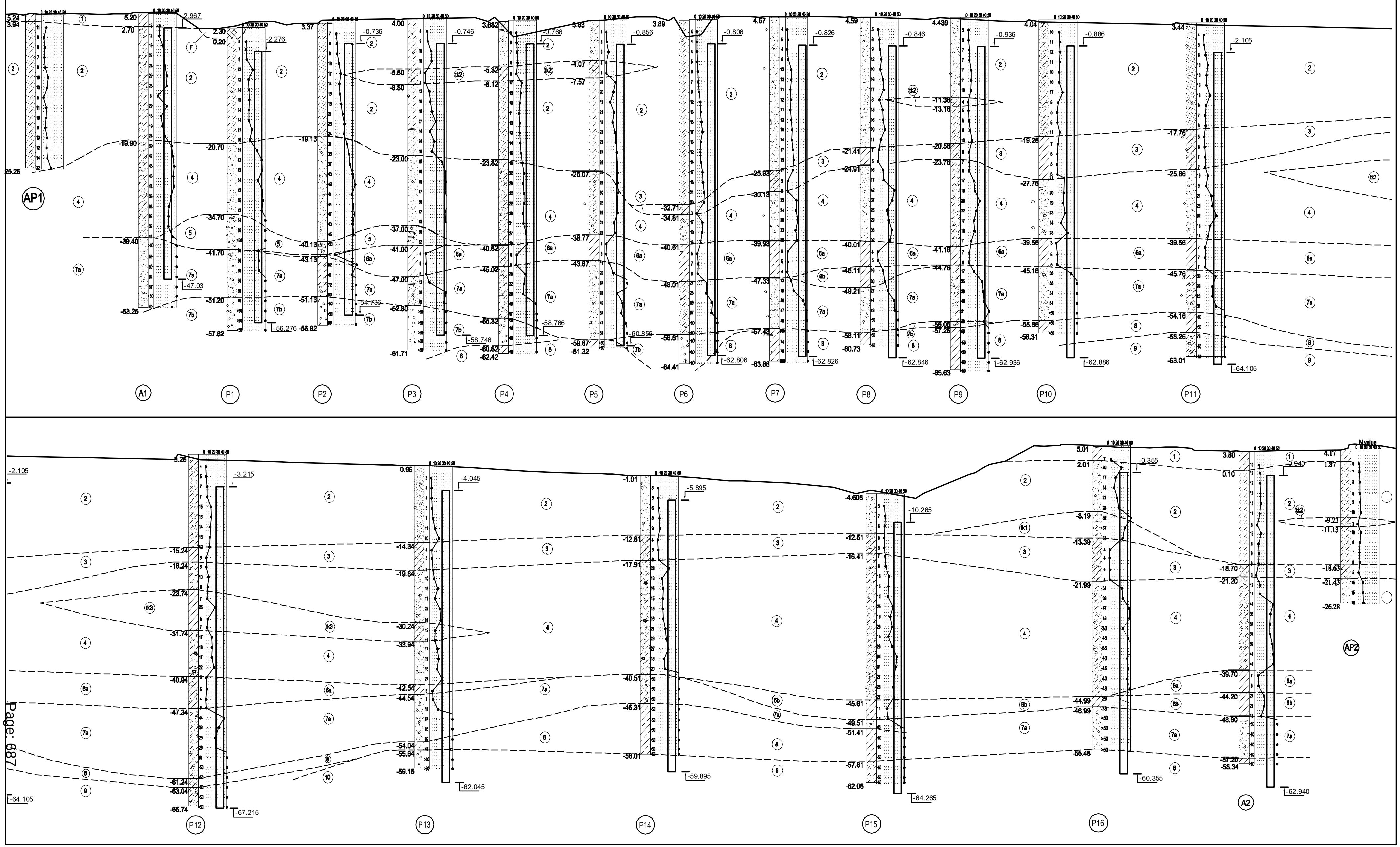
1.1. General:

The soil profile of Ky Lam bridge is shown in figures bellow.

Symbol Ký hiệu	Layer - Lense Lớp - Thấu kính	Geology Mô tả địa chất	Thickness Bề dày (m)	SPT value Giá trị SPT (blow/30cm)	Average Internal Friction angle (^{0})	Average Dry weight (g/cm^3)	Average Unit weight (g/cm3)
	1	High plasticity clay (CH) Đất sét rất dẻo (CH)	From 2.50 to 3.70 Average: 3.1	From 7 to 10 Average: 9	$15^{0}28'$	1.467	2.689
	2	Clayey sand (SC) Cát lẫn sét (SC)	From 7.90 to 36.60 Average: 22.3	from 4 to 28 Average: 16	$28^{0}11'$	1.440	2.648
	Tk1	Poorly graded sand with clay (SP-SC) Cát cấp phối kém- cát lẫn sét (SP-SC)	5.2	from 33 to 57 Average: 45			2.650
	Tk2a	Low plasticity clay (CL) Đất sét ít dẻo (CL)	1.4	from 5 to 7 Average: 6	$6^{0}10'$	1.148	2.692
	Tk2	High plasticity silt (MH) Đất bụi rất dẻo (MH)	From 1.80 to 3.50 Average: 2.65	from 4 to 5 Average: 5	$6^{0}10'$	1.148	2.692
	3	High plasticity silt (MH) Đất bụi rất dẻo (MH)	from 2.10 to 8.50 Average: 5.30	from 3 to 7 Average: 5	$6^{0}02'$	1.174	2.656
	4	Poorly graded sand with clay (SP-SC) Cát cấp phối kém- cát lẫn sét (SP-SC)	from 5.80 to 29.20 Average: 17.50	from 8 to 49 Average: 29			2.652
	Tk3	High plasticity clay (CH) Đất sét rất dẻo (CH)	from 3.70 to 8.00 Average: 5.85	from 7 to 25 Average: 16	$6^{0}05'$	1.157	2.691
	5	Poorly graded sand (SP) Cát cấp phối kém (SP)	from 2.50 to 7.00 Average: 4.75	>50			2.654
	6a	High plasticity clay (CH) Đất sét rất dẻo (CH)	from 0.50 to 7.40 Average: 3.95	from 4 to 8 Average: 6	$13^{0}12'$	1.312	2.691
	6b	High plasticity clay (CH) Đất sét rất dẻo (CH)	from 2.00 to 4.60 Average: 3.30	from 8 to 26 Average: 17	$20^{0}04'$	1.522	2.692
	7a	Poorly graded sand (SP) Cát cấp phối kém (SP)	from 1.90 to 13.85 Average: 7.88	from 11 to >50	$32^{0}41'$		2.657
	7b	Well graded gravel(GW) Sỏi cấp phối tốt (GW)	from 1.20 to 9.0 Average: 5.10	> 50			2.66
	8	Low plasticity clay (CL) Đất sét ít dẻo (CL)	from 1.60 to 9.00 Average: 5.30	> 50	$29^{0}09'$	1.837	2.691
	9	Clayey sand (SC) Cát lẫn sét (SC)	4	> 50	$29^{0}33'$	1.901	2.673
	10	Poorly graded gravel (GP) Sỏi cấp phối xấu (GP)	3.5	> 50			2.675

Table 1.1 Definition of Soil layer of Ky Lam Bridge

SOIL PROFILE DESCRIPTION - MIÊU TẢ CÁC LỚP ĐỊA CHẤT



1.2. Estimation of Resistance of Bored Shaft

Resistance factor of a bored shaft is estimated in accordance with the AASHTO LRFD 4th, some values are not specified in the Vietnamese Standard. The following equations and factors shall be used.

Table.3.2.3. Bearing Capacity Method & Factors

	Cohesive Soil			Cohesionless Soil with gravelly sands**		
	Methods	Resistance factors	Other Factors	Methods	Resistance factors	Other Factors
Shaft Resistance	α-Method	0.45	Group Effects	Reese & O'Neill (1999)	0.55	Group Effects
Tip Resistance	Reese & O'Neill (1999)	0.4	Group Effects *Large Diameter Factor	Reese & O'Neill (1999)	0.5	Group Effects *Large Diameter Factor
Shaft Resistance in IGMs*	Reese & O'Neill (1999)	0.6	Group Effects	Reese & O'Neill (1999)	0.6	Group Effects
Tip Resistance in IGMs*	Reese & O'Neill (1999)	0.55	Group Effects *Large Diameter Factor	Reese & O'Neill (1999)	0.55	Group Effects *Large Diameter Factor

- IGMs are defined by Onei'l and Reese (1999) as follow:
  + Cohesive IGM – clay shales or mudstones with an S_u of to 0.25 to 2.5 (Mpa).
  + Cohesionless IGM – granular tills or granular residual with SPT value N_{60} >50 blows/30cm.
- Soil layer 5 and 7b are gravel layer which have SPT value more than 50 blow. Therefore, they consided as IGM layer.

1.3. Estimation of Settlement of pile group:

The Pile foundation input to bearing layer are layer 7a, 7b, 8, 9, 10 in medium dense to very dense. Sub layer 7a, 7b, 10 are sand and gravel, and layer 8, 9 are clayed sand with heigh grain ratio of sand. Therefore, it can estimated settlement of pile group by SPT value.

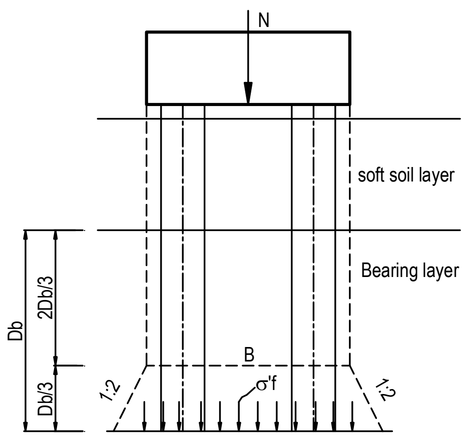


Fig. 1.3.1 Location of equivalent footing

1.4. *Desin Principle*

1.4.1. General

The flow chart for design flow of pile foundation is, as follows:

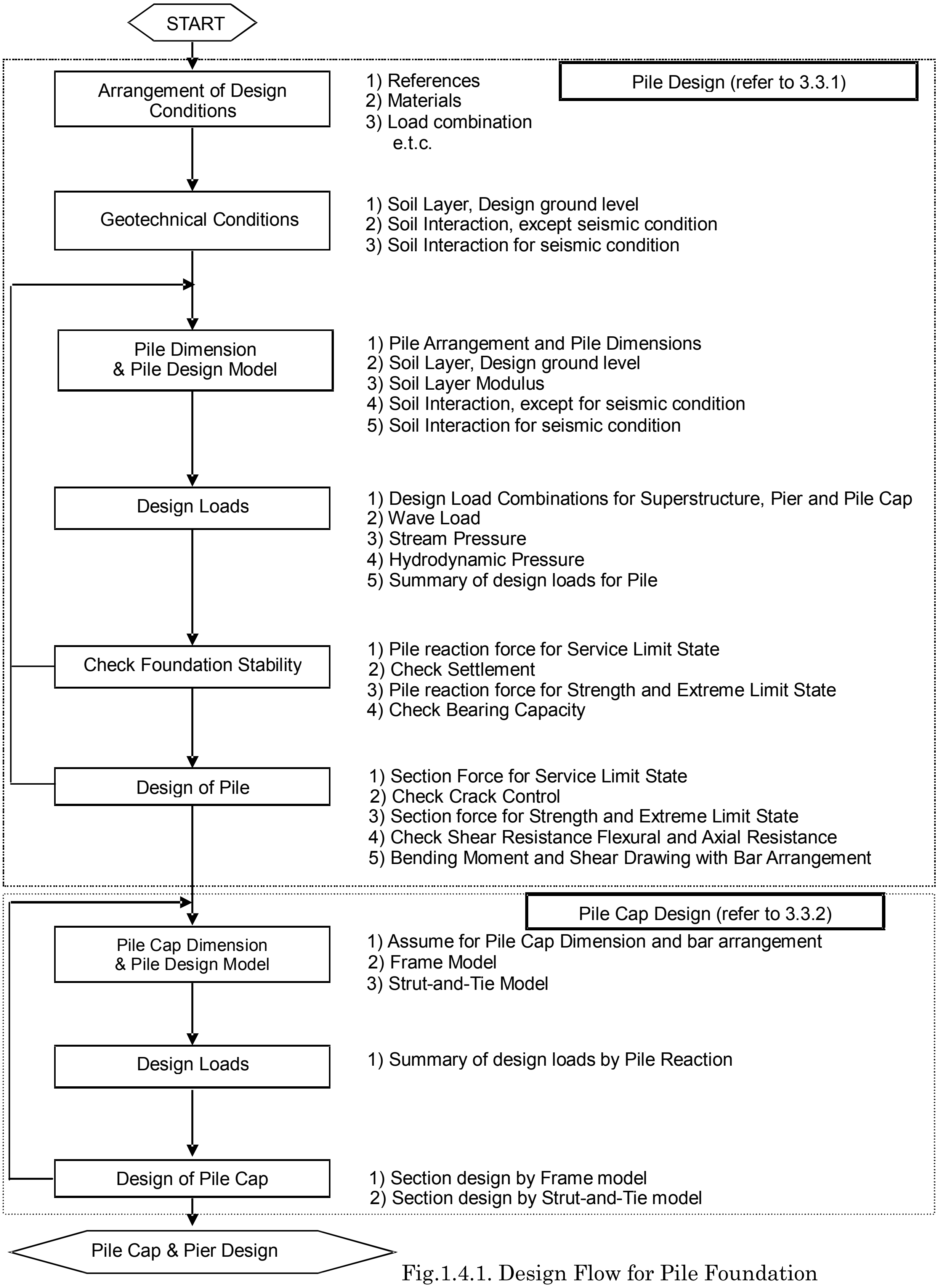


Fig.1.4.1. Design Flow for Pile Foundation

NUMBER & LENGTH OF PILE OF KY LAM BRIDGE - SỐ LƯỢNG VÀ CHIỀU DÀI CỌC KHOAN NHỒI CẦU KỲ LAM

Abutment / Pier	Super-T girder span / Nhịp dầm Super-T											Box girder span / Nhịp dầm hộp						
	A1	P1	P2	P3	P4	P5	P6	P7	P8	P9	P10	P11	P12	P13	P14	P15	P16	A2
Bottom Footing elevation Cao độ đáy bệ cọc	2.967	-2.276	-0.736	-0.746	-0.766	-0.856	-0.806	-0.826	-0.846	-0.936	-0.886	-2.105	-3.215	-4.045	-5.895	-10.265	-0.355	-0.940
Ground elevation after scour Cao độ sau xói	2.97	-0.85	0.04	1.15	1.95	-2.89	1.54	1.74	1.55	0.97	-0.88	-3.22	-4.40	-5.97	-8.56	-12.15	1.05	-0.94
Pile Number / số lượng cọc	12	8	8	8	8	8	8	8	8	8	10	18	18	22	22	20	18	18
Pile length/ chiều dài cọc	50	54	54	58	58	58	62	62	62	62	62	62	64	60	56	54	60	62
Pile tip elevation \ CĐ đáy cọc	-47.033	-56.276	-54.736	-58.746	-58.766	-58.856	-62.806	-62.826	-62.846	-62.936	-62.886	-64.105	-67.215	-64.045	-61.895	-64.265	-60.355	-62.940
Bearing layer elevation cao độ lớp chịu lực	-39.4	-41.7	-43.13	-47	-50.32	-47.87	-53.11	-51.33	-54.91	-56.06	-49.16	-54.16	-56.34	-48.54	-46.31	-51.41	-46.99	-48.8
Input on bearing layer Chiều dài ngàm	8	15	12	12	8	11	10	11	8	7	14	10	11	16	16	13	13	14
Loading design / Tải trọng thiết kế	6974	8262	8262	8306	8262	8262	8262	8306	8262	8262	7386	8023	8369	8775	8917	7828	7889	8576
Pile Capacity / Sức chịu tải của cọc	7541	8754	8483	8457	8574	8463	8720	8809	8964	8950	7781	8711	8765	9075	9505	8717	8955	8889

PILE CAPACITY AND SETTLEMENT OF GROUP PILE

1. DATA

- Code of boreholes		AB1
- Pile diameter :		1500 mm
- Elevation of the underground water	EL_3 =	1.00 m
- Ground elevation after scour	EL_4 =	2.967 m
- Base bottom elevation	EL_1 =	2.967 m
- Expected pipe tip elevation	EL_2 =	-47.033 m
- Pipe length	L =	50.0 m
- Perimeter of cross section	P =	4.71 m
- Area of the pile cross section	A_b =	1.77 m2
- Concrete weight	γ_c =	24.50 kN/m3
Soil layer at Tip of pile:		1

('1' =cohesionless soil, '2' = cohesive soil, '3' = intermediate material , '4' = rock)

* Reference document

+ Standard 22 TCN 272-05
+ Standard AASHTO LRFD 2007

Legends

* Because the pile foundation of the bridge crosses the soil area with strong weather rock and IGM, so 22TCN272-05 can not meet requirement to base for the calculation of the pipe load bearing capacity, then AASHTO 2007 is referred to calculate the process items which standard 22TCN272-05 has not yet mentioned

2. CALCULATION FOR THE BEARING FORCE OF BORED PILES

2.1. SIDEWALL FRICTION q_s:

2.1.1. For cohesionless soil

+ Using Method β : O'Neil and Reese (1999)

$q_s = \beta\sigma'_v \leq 0.19$MPa; with $0.25 < \beta < 1.2$

+ For sandy soil:

$\beta = 1.5 - 7.7\text{x}10^{-3}(z)^{0.5}$ for $N_{60} \geq 15$

$\beta = N_{60}/15[1.5 - 7.7\text{x}10^{-3}(z)^{0.5}]$ for $N_{60} < 15$

+ For Gravelly sand and gravels and SPT values are more than 50 blows/30cm:

$\beta = 2\text{-}0.00082\,(z)^{0.5}$ with $0.25 < \beta < 1.8$

With : + z : The depth of the subsoil layer at the middle location of the soil layer

+ N : The average number of hammers SPT (not yet calibrated) along the pile body (hammer/300)

2.1.2. As for conhesive soil, side wall friction is calculated according to the method α

$q_s = \alpha S_u$ (10.8.3.3.1-1)

In which:

+ The cohesive factor a is calculated according to the method API (10.7.3.3.2a-1)
+ S_u : medium non- drain shear resistance strengthen

* Calculation method S_u:

$S_u = 0.06*N_{60}$ (bar) = $0.006*N_{60}$ (MPa) (Terzaghi & Peck)

* Calculation method a: (10.8.3.5.1b AASHTO LRFD 2007)

$\alpha = 0.55$ with $S_u/0.101 \leq 1.5$

$\alpha = 0.55 - 0.1(S_u/0.101 - 1.5)$ with $1.5 \leq S_u/0.101 \leq 2.5$

*** Calculation method of effective vertical stress s'$_v$ at the center of each soil layer:**

$\sigma'_z = \sigma_{vz} - u_z$

At the soil layer over the underground water level : $\sigma_v = \gamma.z_i$; $u_z = 0$

At the soil layer under the ground water level : $\sigma_v = \gamma_{sat}.z_i$; $u_z = z_i$

Suppose: clay layer and water- penetration weathered rock layer

Soil layer placed under the ground water level and obtain the penetration, use the saturated weight

In which

+ σ_v : total stress at the depth z (kN/m^2)
+ σ'_v : effective stress at the depth z (kN/m^2)
+ u : Pore water pressure (kN/m^2)
+ z_i : Depth to the middle point of the i layer counted from the surfac (m)
+ l_i : thickness of the i soil layer (m)

+ γ : Natural density of the soil layer above the ground water level (kN/m^3)

+ γ_{sat}: the saturated desnsity of the soil layer under the ground wate (kN/m^3)

$$\gamma_{sat} = \gamma_d(1 - \gamma_w/\gamma_s) + \gamma_w$$

+ γ_d : Dry density (kN/m^3)

+ γ_s : Unit weight (kN/m^3)

+ γ_w : water density γ_w = 10 (kN/m^3)

+ With: Energy efficiency E_h = 50

Name of layer	Layer thickness	Elevation of the layer bottom	depth	Unit weight	Dry density	saturated density	Total stress	Pore water pressure	Effective stress
	li		z_i	γ_s	γ_d	γ_{sat}	σ_{vz}	u	σ'_z
	(m)	(m)	(m)	(kN/m^3)	(kN/m^3)	(kN/m^3)	(kN/m^2)	(kN/m^2)	(kN/m^2)
		2.97							
2	1.50	1.47	0.75	0.00	0.00	0.00	0.00	0.00	0.00
2	17.37	-15.90	10.18	26.48	14.40	18.96	193.10	101.84	91.26
2	4.00	-19.90	20.87	26.48	14.50	19.02	396.98	208.67	188.31
4	6.00	-25.90	25.87	26.52	14.50	19.03	492.31	258.67	233.64
4	13.50	-39.40	35.62	26.52	14.50	19.03	677.88	356.17	321.71
7a	4.50	-43.90	44.62	26.57	16.10	20.04	894.15	446.17	447.98
7a	3.13	-47.03	48.43	26.57	16.10	20.04	970.63	484.34	486.30
	1.50	-48.53							

Name of layer	Soil type	σ'_z (N/mm^2)	SPT N (Blow/30cm)	N_{60} = N*E_h/60	S_u (N/mm^2)	α	β	q_s (N/mm^2)	Resistance factor φs	value $\varphi_s l_i q_s$ (N/mm)
2	1	0.000	0	-	-	-			0.55	0.00
2	1	0.091	21	17	-	-	0.723	0.066	0.55	630.24
2	1	0.188	23	19	-	-	0.388	0.073	0.55	160.62
4	1	0.234	42	35	-	-	0.262	0.061	0.55	201.69
4	1	0.322	37	31	-	-	0.250	0.080	0.55	597.17
7a	1	0.448	65	54	-	-	0.250	0.112	0.55	277.19
7a	1	0.486	65	54	-	-	0.250	0.122	0.55	209.49
	Total									2076.40

Total resistance of the side wall: $Q_s = \varphi_s\phi_i A_s = \Sigma\varphi_s\phi_i l_i P$ = **9784.8** (Kn)

φ_s: Shaft resistance factor of the pile in the ground (according to the table 10.5.5.2.4-1 AASHTO 2007)

2.2. PILE TIP RESISTANCE q_p:

Soil layer at Tip of pile: 1 Sand layer

2.2.1. Pile tip resistance: Method Reese and O'Neill (1999)

- Nominal Tip resistance follow:

$q_p = 0.057N$ with $N \leq 50$

$q_p = 0.59\sigma'_v((N60(Pa/\sigma'_v))$^0.8 with N > 50

In which

N = 65 (hammer /300mm)-Number of hammer SPT-N has not been calibrated

D = 1500 (mm) Diameter of bore pile

D_p = 1500 (mm) diameter of bore pipe

σ'_v = 0.49 (Mpa) effective vertical prestress

=> Nominal tip resistance q_p = **2.302** (Mpa)

- As for the bottom diameter bigger than 1270mm, q_p has to reduce as follows:

$q_{pr} = 1270*q_p/D_p$ = **1.949** (Mpa)

- Tip resistance of pile:

$Q_b = \phi * q_p * A_b$ = 1721.77 (kN)

With tip resistance factor : ϕ = **0.5** (Table 10.5.5.2.4-1 AASHTO2007)

2.3. Pile capacity:

- Design resistance of the single pile :

$Q_T = (Q_s+Q_P) =$ 11506.6 kN

- Resistance of the resisting pile according to the pile group:

$Q_R = \eta.Q_T =$ **8821.69** kN

In which

+ $\eta =$ 0.77 Effective factor due to the pile working by group (AASHTO 2007 - 10.8.3.6.3)

+ d = 4.5 (m) The distance from the center to center of pile.

2.4. CONCLUSION

- Final pile capacity: $Q_R =$ 8821.69 kN
- Seftweight of pile: W = 1281.18 kN
- Maximum internal force applied to pile: $P_{max} =$ 6974 kN
- Check pile capacity : $P_{max} + W \leq Q_R$

⇔ 8255.18 < 8821.69 **OK**

3. THE SETTLEMENT OF PILE GROUP:

- The Settlement of pile groups in cohesionless soils using SPT value:

$$\rho = \frac{30qI\sqrt{B}}{N_{60}} \quad \text{(mm)} \quad (10.7.2.3.3\text{-}1)$$

In which: - q: net foundation pressure applid at $2D_b/3$, this pressure is equal to the applied load at the top of group divided by the area of the equivalent footing and does not include the weight of the piles or soil between the piers (Mpa).

- B : Width or smallest dimensions of pile group (mm).
- I : Influence factor of the effective group embedment (dim)

$I = 1 - 0{,}125D'/B$

- D' : Effective depth taken as $2D_b/3$, (mm)
- D_b : depth of embedment of piles in layer that provides support, (mm)

Nmax	Equivalent area	B	D_b	D'	I	N_{60}	q	ρ	Check
(Kn)	(mm^2)	(mm)	(mm)	(mm)	(dim)	blow	(Mpa)	(mm)	$\rho < 25.4$
41136	2.88E+08	7000	7633	5088.67	0.91	54	0.14	6.03	Ok

PILE CAPACITY AND SETTLEMENT OF GROUP PILE

1. DATA

- Code of boreholes		**P1**
- Pile diameter :		1500 mm
- Elevation of the underground water	EL_3 =	1.00 m
- Ground elevation after scour	EL_4 =	-0.850 m
- Base bottom elevation	EL_1 =	-2.276 m
- Expected pipe tip elevation	EL_2 =	-56.276 m
- Pipe length	L =	54.0 m
- Perimeter of cross section	P =	4.71 m
- Area of the pile cross section	A_b =	1.77 m2
- Concrete weight	γ_c =	24.50 kN/m3
Soil layer at Tip of pile:		3

('1' =cohesionless soil, '2' = cohesive soil, '3' = intermediate material , '4' = rock)

* Reference document

+ Standard 22 TCN 272-05
+ Standard AASHTO LRFD 2007

Legends

* Because the pile foundation of the bridge crosses the soil area with strong weather rock and IGM, so 22TCN272-05 can not meet requirement to base for the calculation of the pipe load bearing capacity, then AASHTO 2007 is referred to calculate the process items which standard 22TCN272-05 has not yet mentioned

2. CALCULATION FOR THE BEARING FORCE OF BORED PILES

2.1. SIDEWALL FRICTION q_s:

2.1.1. For cohesionless soil

+ Using Method β : O'Neil and Reese (1999)

$q_s = \beta\sigma'_v \leq 0.19$MPa; with $0.25 < \beta < 1.2$

+ For sandy soil:

$\beta = 1.5 - 7.7x10^{-3}(z)^{0.5}$ for $N_{60} \geq 15$

$\beta = N_{60}/15[1.5 - 7.7x10^{-3}(z)^{0.5}]$ for $N_{60} < 15$

+ For Gravelly sand and gravels and SPT values are more than 50 blows/30cm:

$\beta = 2\text{-}0.00082\ (z)^{0.5}$ with $0.25 < \beta < 1.8$

With : + z : The depth of the subsoil layer at the middle location of the soil layer

+ N : The average number of hammers SPT (not yet calibrated) along the pile body (hammer/300)

2.1.2. As for conhesive soil, side wall friction is calculated according to the method α

$q_s = \alpha S_u$ (10.8.3.3.1-1)

In which:

+ The cohesive factor a is calculated according to the method API (10.7.3.3.2a-1)
+ S_u : medium non- drain shear resistance strengthen

* Calculation method S_u**:**

$S_u = 0.06*N_{60}$ (bar) = $0.006*N_{60}$ (MPa) (Terzaghi & Peck)

* Calculation method a**:** (10.8.3.5.1b AASHTO LRFD 2007)

$\alpha = 0.55$ with $S_u/0.101 \leq 1.5$

$\alpha = 0.55 - 0.1(S_u/0.101 - 1.5)$ with $1.5 \leq S_u/0.101 \leq 2.5$

*** Calculation method of effective vertical stress s'$_v$ at the center of each soil layer:**

$\sigma'_z = \sigma_{vz} - u_z$

At the soil layer over the underground water level : $\sigma_v = \gamma.z_i$; $u_z = 0$

At the soil layer under the ground water level : $\sigma_v = \gamma_{sat}.z_i$; $u_z = z_i$

Suppose: clay layer and water- penetration weathered rock layer

Soil layer placed under the ground water level and obtain the penetration, use the saturated weight

In which

+ σ_v : total stress at the depth z (kN/m^2)
+ σ'_v : effective stress at the depth z (kN/m^2)
+ u : Pore water pressure (kN/m^2)
+ z_i : Depth to the middle point of the i layer counted from the surfac (m)
+ l_i : thickness of the i soil layer (m)

+ γ : Natural density of the soil layer above the ground water level (kN/m^3)
+ γ_{sat}: the saturated desnsity of the soil layer under the ground wate (kN/m^3)

$$\gamma_{sat} = \gamma_d(1- \gamma_w/\gamma_s) + \gamma_w$$

+ γ_d : Dry density (kN/m^3)
+ γ_s : Unit weight (kN/m^3)
+ γ_w : water density γ_w = 10 (kN/m^3)
+ With: Energy efficiency E_h = 50

Name of layer	Layer thickness	Elevation of the layer bottom	depth	Unit weight	Dry density	saturated density	Total stress	Pore water pressure	Effective stress
	li		z_i	γ_s	γ_d	γ_{sat}	σ_{vz}	u	σ'_z
	(m)	(m)	(m)	(kN/m^3)	(kN/m^3)	(kN/m^3)	(kN/m^2)	(kN/m^2)	(kN/m^2)
		-2.28							
2	1.50	-3.78	0.75	0.00	0.00	0.00	0.00	0.00	0.00
2	16.92	-20.70	9.96	26.47	14.40	18.96	188.88	99.62	89.26
4	14.00	-34.70	25.42	26.53	14.50	19.03	483.93	254.24	229.69
5	7.00	-41.70	35.92	26.54	14.50	19.04	683.87	359.24	324.63
7a	5.50	-47.20	42.17	26.51	14.50	19.03	802.59	421.74	380.85
7a	4.00	-51.20	46.92	26.52	16.10	20.03	939.85	469.24	470.61
7b	5.08	-56.28	51.46	26.54	16.10	20.03	1030.97	514.62	516.35
	1.50	-57.78							

Name of layer	Soil type	σ'_z (N/mm^2)	SPT N (Blow/30cm)	N_{60} = N*E_h/60	S_u (N/mm^2)	α	β	q_s (N/mm^2)	Resistance factor φs	value $\varphi_s l_i q_s$ (N/mm)
2	1	0.000	0	-	-	-			0.55	0.00
2	1	0.089	15	12	-	-	0.707	0.063	0.55	587.47
4	1	0.230	40	33	-	-	0.272	0.063	0.55	481.50
5	3	0.325	53	44	-	-	0.250	0.081	0.60	340.86
7a	1	0.381	35	29	-	-	0.250	0.095	0.55	288.02
7a	1	0.471	58	48	-	-	0.250	0.118	0.55	258.83
7b	3	0.516	75	63	-	-	0.250	0.129	0.60	393.15
	Total									2349.83

Total resistance of the side wall: $Q_s = \varphi_s \phi_i A_s = \Sigma\varphi_s \phi_i l_i P$ = **11073.3** (Kn)

φ_s: Shaft resistance factor of the pile in the ground (according to the table 10.5.5.2.4-1 AASHTO 2007)

2.2. PILE TIP RESISTANCE q_p:

Soil layer at Tip of pile: 3 IGM

2.2.1. Pile tip resistance: Method Reese and O'Neill (1999)

- Nominal Tip resistance follow:

$q_p = 0.057N$ with $N \leq 50$

$q_p = 0.59\sigma'_v((N60(Pa/\sigma'_v))$^0.8 with N > 50

In which

N = 75 (hammer /300mm)-Number of hammer SPT-N has not been calibrated

D = 1500 (mm) Diameter of bore pile

D_p = 1500 (mm) diameter of bore pipe

σ'_v = 0.52 (Mpa) effective vertical prestress

=> Nominal tip resistance q_p = **2.612** (Mpa)

- As for the bottom diameter bigger than 1270mm, q_p has to reduce as follows:

$q_{pr} = 1270*q_p/D_p$ = **2.211** (Mpa)

- Tip resistance of pile:

$Q_b = \phi * q_p * A_b$ = 2149.29 (kN)

With tip resistance factor : ϕ = **0.55** (Table 10.5.5.2.4-1 AASHTO2007)

2.3. Pile capacity:

- Design resistance of the single pile :

Q_T = (Q_s+Q_P) = 13222.6 kN

- Resistance of the resisting pile according to the pile group:

Q_R = $\eta.Q_T$ = **10137.31** kN

In which

+ η = 0.77 Effective factor due to the pile working by group (AASHTO 2007 - 10.8.3.6.3)

+ d = 4.5 (m) The distance from the center to center of pile.

2.4. CONCLUSION

- Final pile capacity: Q_R = 10137.31 kN
- Seftweight of pile: W = 1383.68 kN
- Maximum internal force applied to pile: P_{max} = 8262 kN
- Check pile capacity : P_{max} + W ≤ Q_R

⇔ 9645.68 < 10137.31 **OK**

3. THE SETTLEMENT OF PILE GROUP:

- The Settlement of pile groups in cohesionless soils using SPT value:

$$\rho = \frac{30qI\sqrt{B}}{N_{60}}$$ (mm) (10.7.2.3.3-1)

In which: - q: net foundation pressure applid at $2D_b/3$, this pressure is equal to the applied load at the top of group divided by the area of the equivalent footing and does not include the weight of the piles or soil between the piers (Mpa).

- B : Width or smallest dimensions of pile group (mm).
- I : Influence factor of the effective group embedment (dim)

I = 1 - 0,125D'/B

- D' : Effective depth taken as $2D_b/3$, (mm)
- D_b : depth of embedment of piles in layer that provides support, (mm)

Nmax	Equivalent area	B	D_b	D'	I	N_{60}	q	ρ	Check
(Kn)	(mm^2)	(mm)	(mm)	(mm)	(dim)	blow	(Mpa)	(mm)	ρ <25.4
38264	2.52E+08	7000	9076	6050.67	0.89	63	0.15	5.44	Ok

PILE CAPACITY AND SETTLEMENT OF GROUP PILE

1. DATA

- Code of boreholes		P2	
- Pile diameter :		1500	mm
- Elevation of the underground water	EL_3 =	1.00	m
- Ground elevation after scour	EL_4 =	0.040	m
- Base bottom elevation	EL_1 =	-0.736	m
- Expected pipe tip elevation	EL_2 =	-54.736	m
- Pipe length	L =	54.0	m
- Perimeter of cross section	P =	4.71	m
- Area of the pile cross section	A_b =	1.77	m2
- Concrete weight	γ_c =	24.50	kN/m3
Soil layer at Tip of pile:		3	

('1' =cohesionless soil, '2' = cohesive soil, '3' = intermediate material , '4' = rock)

* Reference document

+ Standard 22 TCN 272-05

+ Standard AASHTO LRFD 2007

Legends

* Because the pile foundation of the bridge crosses the soil area with strong weather rock and IGM, so 22TCN272-05 can not meet requirement to base for the calculation of the pipe load bearing capacity, then AASHTO 2007 is referred to calculate the process items which standard 22TCN272-05 has not yet mentioned

2. CALCULATION FOR THE BEARING FORCE OF BORED PILES

2.1. SIDEWALL FRICTION q_s:

2.1.1. For cohesionless soil

+ Using Method β : O'Neil and Reese (1999)

$q_s = \beta\sigma'_v \leq 0.19$MPa; with $0.25 < \beta < 1.2$

+ For sandy soil:

$\beta = 1.5 - 7.7\text{x}10^{-3}(z)^{0.5}$ for $N_{60} \geq 15$

$\beta = N_{60}/15[1.5 - 7.7\text{x}10^{-3}(z)^{0.5}]$ for $N_{60} < 15$

+ For Gravelly sand and gravels and SPT values are more than 50 blows/30cm:

$\beta = 2\text{-}0.00082\ (z)^{0.5}$ with $0.25 < \beta < 1.8$

With : + z : The depth of the subsoil layer at the middle location of the soil layer

+ N : The average number of hammers SPT (not yet calibrated) along the pile body (hammer/300)

2.1.2. As for conhesive soil, side wall friction is calculated according to the method α

$q_s = \alpha S_u$ (10.8.3.3.1-1)

In which:

+ The cohesive factor a is calculated according to the method API (10.7.3.3.2a-1)

+ S_u : medium non- drain shear resistance strengthen

* Calculation method S_u**:**

$S_u = 0.06*N_{60}$ (bar) = $0.006*N_{60}$ (MPa) (Terzaghi & Peck)

* Calculation method a**:** (10.8.3.5.1b AASHTO LRFD 2007)

$\alpha = 0.55$ with $S_u/0.101 \leq 1.5$

$\alpha = 0.55 - 0.1(S_u/0.101 - 1.5)$ with $1.5 \leq S_u/0.101 \leq 2.5$

*** Calculation method of effective vertical stress s'$_v$ at the center of each soil layer:**

$\sigma'_z = \sigma_{vz} - u_z$

At the soil layer over the underground water level : $\sigma_v = \gamma.z_i$; $u_z = 0$

At the soil layer under the ground water level : $\sigma_v = \gamma_{sat}.z_i$; $u_z = z_i$

Suppose: clay layer and water- penetration weathered rock layer

Soil layer placed under the ground water level and obtain the penetration, use the saturated weight

In which

+ σ_v : total stress at the depth z (kN/m^2)

+ σ'_v : effective stress at the depth z (kN/m^2)

+ u : Pore water pressure (kN/m^2)

+ z_i : Depth to the middle point of the i layer counted from the surfac (m)

+ l_i : thickness of the i soil layer (m)

+ γ : Natural density of the soil layer above the ground water level (kN/m³)
+ γ_{sat}: the saturated desnsity of the soil layer under the ground wate (kN/m³)

$$\gamma_{sat} = \gamma_d(1- \gamma_w/\gamma_s) + \gamma_w$$

+ γ_d : Dry density (kN/m³)
+ γ_s : Unit weight (kN/m³)
+ γ_w : water density γ_w = 10 (kN/m³)
+ With: Energy efficiency E_h = 50

Name of layer	Layer thickness	Elevation of the layer bottom	depth	Unit weight	Dry density	saturated density	Total stress	Pore water pressure	Effective stress
	li		z_i	γ_s	γ_d	γ_{sat}	σ_{vz}	u	σ'_z
	(m)	(m)	(m)	(kN/m³)	(kN/m³)	(kN/m³)	(kN/m²)	(kN/m²)	(kN/m²)
		-0.74							
2	1.50	-2.24	0.75	0.00	0.00	0.00	0.00	0.00	0.00
2	16.89	-19.13	9.95	26.46	14.40	18.96	188.57	99.47	89.10
4	21.00	-40.13	28.89	26.60	14.50	19.05	550.40	288.94	261.46
5	2.50	-42.63	40.64	26.69	14.50	19.07	774.97	406.44	368.53
6a	0.50	-43.13	42.14	26.91	13.12	18.24	768.90	421.44	347.46
7a	4.00	-47.13	44.39	26.52	14.50	19.03	844.93	443.94	400.99
7a	4.00	-51.13	48.39	26.52	16.10	20.03	969.29	483.94	485.35
7b	3.61	-54.74	52.20	26.69	16.10	20.07	1047.48	521.97	525.51
	1.50	-56.24							

Name of layer	Soil type	σ'_z (N/mm²)	SPT N (Blow/30cm)	N_{60} = N*E_h/60	S_u (N/mm²)	α	β	q_s (N/mm²)	Resistance factor φs	value $\varphi_s l_i q_s$ (N/mm)
2	1	0.000	0	-	-	-			0.55	0.00
2	1	0.089	16	13	-	-	0.732	0.065	0.55	606.08
4	1	0.261	42	35	-	-	0.250	0.065	0.55	754.96
5	3	0.369	50	42	-	-	0.250	0.092	0.60	138.20
6a	2	0.347	4	3	0.020	0.55	0.250	0.011	0.45	2.48
7a	1	0.401	46	38	-	-	0.250	0.100	0.55	220.54
7a	1	0.485	75	63	-	-	0.250	0.121	0.55	266.94
7b	3	0.526	75	63	-	-	0.250	0.131	0.60	284.25
	Total									2273.44

Total resistance of the side wall: $Q_s = \varphi_s\phi_i A_s = \Sigma\varphi_s\phi_i l_i P$ = **10713.3** (Kn)

φ_s: Shaft resistance factor of the pile in the ground (according to the table 10.5.5.2.4-1 AASHTO 2007)

2.2. PILE TIP RESISTANCE q_p:

Soil layer at Tip of pile: 3 IGM

2.2.1. Pile tip resistance: Method Reese and O'Neill (1999)

- Nominal Tip resistance follow:

$q_p = 0.057N$ with $N \le 50$

$q_p = 0.59\sigma'_v((N60(Pa/\sigma'_v))\text{^}0.8$ with N > 50

In which

N = 75 (hammer /300mm)-Number of hammer SPT-N has not been calibrated

D = 1500 (mm) Diameter of bore pile

D_p = 1500 (mm) diameter of bore pipe

σ'_v = 0.53 (Mpa) effective vertical prestress

=> Nominal tip resistance q_p = **2.621** (Mpa)

- As for the bottom diameter bigger than 1270mm, q_p has to reduce as follows:

$q_{pr} = 1270*q_p/D_p$ = **2.219** (Mpa)

- Tip resistance of pile:

$Q_b = \phi*q_p*A_b$ = 2156.86 (kN)

With tip resistance factor : ϕ = **0.55** (Table 10.5.5.2.4-1 AASHTO2007)

2.3. Pile capacity:

- Design resistance of the single pile :

Q_T = (Q_s+Q_P) = 12870.2 kN

- Resistance of the resisting pile according to the pile group:

Q_R = $\eta.Q_T$ = **9867.15** kN

In which

+ η = 0.77 Effective factor due to the pile working by group (AASHTO 2007 - 10.8.3.6.3)

+ d = 4.5 (m) The distance from the center to center of pile.

2.4. CONCLUSION

- Final pile capacity: Q_R = 9867.15 kN
- Seftweight of pile: W = 1383.68 kN
- Maximum internal force applied to pile: P_{max} = 8262 kN
- Check pile capacity : P_{max} + W ≤ Q_R

⇔ 9645.68 < 9867.15 **OK**

3. THE SETTLEMENT OF PILE GROUP:

- The Settlement of pile groups in cohesionless soils using SPT value:

$$\rho = \frac{30qI\sqrt{B}}{N_{60}} \quad \text{(mm)} \quad (10.7.2.3.3\text{-}1)$$

In which: - q: net foundation pressure applid at $2D_b/3$, this pressure is equal to the applied load at the top of group divided by the area of the equivalent footing and does not include the weight of the piles or soil between the piers (Mpa).

- B : Width or smallest dimensions of pile group (mm).
- I : Influence factor of the effective group embedment (dim)

I = 1 - 0,125D'/B

- D' : Effective depth taken as $2D_b/3$, (mm)
- D_b : depth of embedment of piles in layer that provides support, (mm)

Nmax	Equivalent area	B	D_b	D'	I	N_{60}	q	ρ	Check
(Kn)	(mm^2)	(mm)	(mm)	(mm)	(dim)	blow	(Mpa)	(mm)	ρ <25.4
38264	2.33E+08	7000	7606	5070.67	0.91	63	0.16	6.01	Ok

PILE CAPACITY AND SETTLEMENT OF GROUP PILE

1. DATA

- Code of boreholes | P3
- Pile diameter : 1500 mm
- Elevation of the underground water $EL_3 =$ 1.00 m
- Ground elevation after scour $EL_4 =$ 1.150 m
- Base bottom elevation $EL_1 =$ -0.746 m
- Expected pipe tip elevation $EL_2 =$ -58.746 m
- Pipe length L = 58.0 m
- Perimeter of cross section P = 4.71 m
- Area of the pile cross section $A_b =$ 1.77 m2
- Concrete weight $\gamma_c =$ 24.50 kN/m3

Soil layer at Tip of pile: 3

('1' =cohesionless soil, '2' = cohesive soil, '3' = intermediate material , '4' = rock)

* Reference document
  + Standard 22 TCN 272-05
  + Standard AASHTO LRFD 2007

Legends

* Because the pile foundation of the bridge crosses the soil area with strong weather rock and IGM, so 22TCN272-05 can not meet requirement to base for the calculation of the pipe load bearing capacity, then AASHTO 2007 is referred to calculate the process items which standard 22TCN272-05 has not yet mentioned

2. CALCULATION FOR THE BEARING FORCE OF BORED PILES

2.1. SIDEWALL FRICTION q_s:

2.1.1. For cohesionless soil

+ Using Method β : O'Neil and Reese (1999)

$q_s = \beta\sigma'_v \leq 0.19$MPa; with $0.25 < \beta < 1.2$

+ For sandy soil:

$\beta = 1.5 - 7.7\text{x}10^{-3}(z)^{0.5}$ for $N_{60} \geq 15$

$\beta = N_{60}/15[1.5 - 7.7\text{x}10^{-3}(z)^{0.5}]$ for $N_{60} < 15$

+ For Gravelly sand and gravels and SPT values are more than 50 blows/30cm:

$\beta = 2\text{-}0.00082\ (z)^{0.5}$ with $0.25 < \beta < 1.8$

With : + z : The depth of the subsoil layer at the middle location of the soil layer

+ N : The average number of hammers SPT (not yet calibrated) along the pile body (hammer/300)

2.1.2. As for conhesive soil, side wall friction is calculated according to the method α

$q_s = \alpha S_u$ (10.8.3.3.1-1)

In which:

+ The cohesive factor a is calculated according to the method API (10.7.3.3.2a-1)

+ S_u : medium non- drain shear resistance strengthen

* Calculation method **S_u:**

$S_u = 0.06{*}N_{60}$ (bar) $= 0.006{*}N_{60}$ (MPa) (Terzaghi & Peck)

* Calculation method a**:** (10.8.3.5.1b AASHTO LRFD 2007)

$\alpha = 0.55$ with $S_u/0.101 \leq 1.5$

$\alpha = 0.55 - 0.1(S_u/0.101 - 1.5)$ with $1.5 \leq S_u/0.101 \leq 2.5$

*** Calculation method of effective vertical stress s'$_v$ at the center of each soil layer:**

$\sigma'_z = \sigma_{vz} - u_z$

At the soil layer over the underground water level : $\sigma_v = \gamma.z_i$; $u_z = 0$

At the soil layer under the ground water level : $\sigma_v = \gamma_{sat}.z_i$; $u_z = z_i$

Suppose: clay layer and water- penetration weathered rock layer

Soil layer placed under the ground water level and obtain the penetration, use the saturated weight

In which

+ σ_v : total stress at the depth z (kN/m^2)

+ σ'_v : effective stress at the depth z (kN/m^2)

+ u : Pore water pressure (kN/m^2)

+ z_i : Depth to the middle point of the i layer counted from the surfac (m)

+ l_i : thickness of the i soil layer (m)

+ γ : Natural density of the soil layer above the ground water level (kN/m^3)

+ γ_{sat}: the saturated desnsity of the soil layer under the ground wate (kN/m³)

$$\gamma_{sat} = \gamma_d(1-\gamma_w/\gamma_s) + \gamma_w$$

+ γ_d : Dry density (kN/m³)

+ γ_s : Unit weight (kN/m³)

+ γ_w : water density γ_w = 10 (kN/m³)

+ With: Energy efficiency E_h = 50

Name of layer	Layer thickness	Elevation of the layer bottom	depth	Unit weight	Dry density	saturated density	Total stress	Pore water pressure	Effective stress
	li		z_i	γ_s	γ_d	γ_{sat}	σ_{vz}	u	σ'_z
	(m)	(m)	(m)	(kN/m³)	(kN/m³)	(kN/m³)	(kN/m²)	(kN/m²)	(kN/m²)
		-0.75							
2	1.50	-2.25	0.75	0.00	0.00	0.00	0.00	0.00	0.00
2	3.55	-5.80	3.28	26.46	14.40	18.96	62.12	32.77	29.35
tk2	3.00	-8.80	6.55	26.92	11.48	17.22	112.83	65.54	47.29
2	14.20	-23.00	15.15	26.46	14.50	19.02	288.23	151.54	136.69
4	14.00	-37.00	29.25	26.60	14.50	19.05	557.26	292.54	264.72
5	4.00	-41.00	38.25	26.69	14.50	19.07	729.40	382.54	346.86
6a	6.00	-47.00	43.25	26.91	13.12	18.24	789.15	432.54	356.61
7a	2.00	-49.00	47.25	26.52	14.50	19.03	899.36	472.54	426.82
7a	3.80	-52.80	50.15	26.52	16.10	20.03	1004.54	501.54	503.00
7b	5.95	-58.75	55.03	26.69	16.10	20.07	1104.27	550.27	554.00
	1.50	-60.25							

Name of layer	Soil type	σ'_z (N/mm²)	SPT N (Blow/30cm)	N_{60} = N*E_h/60	S_u (N/mm²)	α	β	q_s (N/mm²)	Resistance factor φs	value $\varphi_s l_i q_s$ (N/mm)
2	1	0.000	0	-	-	-			0.55	0.00
2	1	0.029	13	11	-	-	0.900	0.026	0.55	51.66
tk2	1	0.047	4	3	-	-	0.250	0.012	0.55	19.51
2	1	0.137	19	15	-	-	0.552	0.075	0.55	589.41
4	1	0.265	48	40	-	-	0.250	0.066	0.55	509.58
5	3	0.347	64	53	-	-	0.250	0.087	0.60	208.12
6a	2	0.357	5	4	0.025	0.55	0.250	0.014	0.45	37.13
7a	1	0.427	31	26	-	-	0.250	0.107	0.55	117.38
7a	1	0.503	47	39	-	-	0.250	0.126	0.55	262.82
7b	3	0.554	75	63	-	-	0.250	0.138	0.60	494.11
	Total									2289.70

Total resistance of the side wall: $Q_s = \varphi_s \phi_i A_s = \Sigma\varphi_s\phi_i l_i P$ = **10790.0** (Kn)

φ_s: Shaft resistance factor of the pile in the ground (according to the table 10.5.5.2.4-1 AASHTO 2007)

2.2. PILE TIP RESISTANCE q_p:

Soil layer at Tip of pile: 3 IGM

2.2.1. Pile tip resistance: Method Reese and O'Neill (1999)

- Nominal Tip resistance follow:

$q_p = 0.057N$ with $N \leq 50$

$q_p = 0.59\sigma'_v((N60(Pa/\sigma'_v))$^0.8 with N > 50

In which

N = 75 (hammer /300mm)-Number of hammer SPT-N has not been calibrated

D = 1500 (mm) Diameter of bore pile

D_p = 1500 (mm) diameter of bore pipe

σ'_v = 0.55 (Mpa) effective vertical prestress

=> Nominal tip resistance q_p = **2.649** (Mpa)

- As for the bottom diameter bigger than 1270mm, q_p has to reduce as follows:

$q_{pr} = 1270*q_p/D_p$ = **2.243** (Mpa)

- **Tip resistance of pile:**

$Q_b = \phi * q_p * A_b =$ 2179.75 (kN)

With tip resistance factor : $\phi =$ **0.55** (Table 10.5.5.2.4-1 AASHTO2007)

2.3. Pile capacity:

- Design resistance of the single pile :

$Q_T = (Q_s + Q_P) =$ 12969.7 kN

- Resistance of the resisting pile according to the pile group:

$Q_R = \eta . Q_T =$ **9943.45** kN

In which

+ $\eta =$ 0.77 Effective factor due to the pile working by group (AASHTO 2007 - 10.8.3.6.3)

+ d = 4.5 (m) The distance from the center to center of pile.

2.4. CONCLUSION

- Final pile capacity: $Q_R =$ 9943.45 kN
- Seftweight of pile: W = 1486.17 kN
- Maximum internal force applied to pile: $P_{max} =$ 8306 kN
- Check pile capacity : $P_{max} + W \leq Q_R$

$\Leftrightarrow$ 9792.17 < 9943.45 **OK**

3. THE SETTLEMENT OF PILE GROUP:

- The Settlement of pile groups in cohesionless soils using SPT value:

$$\rho = \frac{30qI\sqrt{B}}{N_{60}}$$ (mm) (10.7.2.3.3-1)

In which: - q: net foundation pressure applid at $2D_b/3$, this pressure is equal to the applied load at the top of group divided by the area of the equivalent footing and does not include the weight of the piles or soil between the piers (Mpa).

- B : Width or smallest dimensions of pile group (mm).
- I : Influence factor of the effective group embedment (dim)

I = 1 - 0,125D'/B

- D' : Effective depth taken as $2D_b/3$, (mm)
- D_b : depth of embedment of piles in layer that provides support, (mm)

Nmax	Equivalent area	B	D_b	D'	I	N_{60}	q	ρ	Check
(Kn)	(mm^2)	(mm)	(mm)	(mm)	(dim)	blow	(Mpa)	(mm)	ρ <25.4
37981	2.61E+08	7000	9746	6497.33	0.88	63	0.15	5.17	Ok

PILE CAPACITY AND SETTLEMENT OF GROUP PILE

1. DATA

- Code of boreholes **P4**
- Pile diameter : 1500 mm
- Elevation of the underground water $EL_3 =$ 1.00 m
- Ground elevation after scour $EL_4 =$ 1.950 m
- Base bottom elevation $EL_1 =$ -0.766 m
- Expected pipe tip elevation $EL_2 =$ -58.766 m
- Pipe length L = 58.0 m
- Perimeter of cross section P = 4.71 m
- Area of the pile cross section $A_b =$ 1.77 m2
- Concrete weight $\gamma_c =$ 24.50 kN/m3

Soil layer at Tip of pile: 3

('1' =cohesionless soil, '2' = cohesive soil, '3' = intermediate material , '4' = rock)

* Reference document
  + Standard 22 TCN 272-05
  + Standard AASHTO LRFD 2007

Legends

* Because the pile foundation of the bridge crosses the soil area with strong weather rock and IGM, so 22TCN272-05 can not meet requirement to base for the calculation of the pipe load bearing capacity, then AASHTO 2007 is referred to calculate the process items which standard 22TCN272-05 has not yet mentioned

2. CALCULATION FOR THE BEARING FORCE OF BORED PILES

2.1. SIDEWALL FRICTION q_s:

2.1.1. For cohesionless soil

+ Using Method β : O'Neil and Reese (1999)

$q_s = \beta\sigma'_v \le 0.19$MPa; with $0.25 < \beta < 1.2$

+ For sandy soil:

$\beta = 1.5 - 7.7\times10^{-3}(z)^{0.5}$ for $N_{60} \ge 15$

$\beta = N_{60}/15[1.5 - 7.7\times10^{-3}(z)^{0.5}]$ for $N_{60} < 15$

+ For Gravelly sand and gravels and SPT values are more than 50 blows/30cm:

$\beta = 2-0.00082\ (z)^{0.5}$ with $0.25 < \beta < 1.8$

With : + z : The depth of the subsoil layer at the middle location of the soil layer
+ N : The average number of hammers SPT (not yet calibrated) along the pile body (hammer/300)

2.1.2. As for conhesive soil, side wall friction is calculated according to the method α

$q_s = \alpha S_u$ (10.8.3.3.1-1)

In which:

+ The cohesive factor a is calculated according to the method API (10.7.3.3.2a-1)
+ S_u : medium non- drain shear resistance strengthen

* Calculation method **S_u:**

$S_u = 0.06*N_{60}$ (bar) $= 0.006*N_{60}$ (MPa) (Terzaghi & Peck)

* Calculation method a**:** (10.8.3.5.1b AASHTO LRFD 2007)

$\alpha = 0.55$ with $S_u/0.101 \le 1.5$

$\alpha = 0.55 - 0.1(S_u/0.101 - 1.5)$ with $1.5 \le S_u/0.101 \le 2.5$

*** Calculation method of effective vertical stress s'$_v$ at the center of each soil layer:**

$\sigma'_z = \sigma_{vz} - u_z$

At the soil layer over the underground water level : $\sigma_v = \gamma.z_i$; $u_z = 0$

At the soil layer under the ground water level : $\sigma_v = \gamma_{sat}.z_i$; $u_z = z_i$

Suppose: clay layer and water- penetration weathered rock layer

Soil layer placed under the ground water level and obtain the penetration, use the saturated weight

In which

+ σ_v : total stress at the depth z (kN/m^2)
+ σ'_v : effective stress at the depth z (kN/m^2)
+ u : Pore water pressure (kN/m^2)
+ z_i : Depth to the middle point of the i layer counted from the surfac (m)
+ l_i : thickness of the i soil layer (m)
+ γ : Natural density of the soil layer above the ground water level (kN/m^3)
+ γ_{sat}: the saturated desnsity of the soil layer under the ground wate (kN/m^3)

$\gamma_{sat} = \gamma_d(1 - \gamma_w/\gamma_s) + \gamma_w$

+ γ_d : Dry density (kN/m³)
+ γ_s : Unit weight (kN/m³)
+ γ_w : water density γ_w = 10 (kN/m³)
+ With: Energy efficiency E_h = 50

Name of layer	Layer thickness	Elevation of the layer bottom	depth	Unit weight	Dry density	saturated density	Total stress	Pore water pressure	Effective stress
	li		z_i	γ_s	γ_d	γ_{sat}	σ_{vz}	u	σ'_z
	(m)	(m)	(m)	(kN/m³)	(kN/m³)	(kN/m³)	(kN/m²)	(kN/m²)	(kN/m²)
		-0.77							
2	1.50	-2.27	0.75	0.00	0.00	0.00	0.00	0.00	0.00
2	3.05	-5.32	3.03	26.46	14.40	18.96	57.39	30.27	27.12
tk2	2.80	-8.12	5.95	26.92	11.48	17.22	102.50	59.54	42.96
2	15.70	-23.82	15.20	26.46	14.50	19.02	289.18	152.04	137.14
4	8.00	-31.82	27.05	26.60	14.50	19.05	515.35	270.54	244.81
4	9.00	-40.82	35.55	26.69	14.50	19.07	677.92	355.54	322.38
6a	4.20	-45.02	42.15	26.91	13.12	18.24	769.08	421.54	347.54
7a	4.00	-49.02	46.25	26.52	14.50	19.03	880.33	462.54	417.79
7a	6.30	-55.32	51.40	26.52	16.10	20.03	1029.58	514.04	515.54
7b	3.45	-58.77	56.28	26.69	16.10	20.07	1129.35	562.77	566.58
	1.50	-60.27							

Name of layer	Soil type	σ'_z (N/mm²)	SPT N (Blow/30cm)	N_{60} = N*E_h/60	S_u (N/mm²)	α	β	q_s (N/mm²)	Resistance factor φs	value $\varphi_s l_i q_s$ (N/mm)
2	1	0.000	0	-	-	-			0.55	0.00
2	1	0.027	7	6	-	-	0.502	0.014	0.55	22.88
tk2	1	0.043	5	4	-	-	0.302	0.013	0.55	19.98
2	1	0.137	14	12	-	-	0.514	0.070	0.55	608.51
4	1	0.245	21	17	-	-	0.250	0.061	0.55	269.29
4	1	0.322	24	20	-	-	0.250	0.081	0.55	398.94
6a	2	0.348	6	5	0.030	0.55	0.250	0.017	0.45	31.19
7a	1	0.418	22	18	-	-	0.250	0.104	0.55	229.78
7a	1	0.516	52	43	-	-	0.250	0.129	0.55	446.58
7b	3	0.567	75	63	-	-	0.250	0.142	0.60	292.87
	Total									2320.01

Total resistance of the side wall: $Q_s = \varphi_s \phi_i A_s = \Sigma \varphi_s \phi_i l_i P$ = **10932.8** (Kn)

φ_s: Shaft resistance factor of the pile in the ground (according to the table 10.5.5.2.4-1 AASHTO 2007)

2.2. PILE TIP RESISTANCE q_p:

Soil layer at Tip of pile: 3 IGM

2.2.1. Pile tip resistance: Method Reese and O'Neill (1999)

- Nominal Tip resistance follow:

q_p = 0.057N with N ≤ 50

$q_p = 0.59\sigma'_v((N60(Pa/\sigma'_v))$^0.8 with N > 50

In which

N = 75 (hammer /300mm)-Number of hammer SPT-N has not been calibrated

D = 1500 (mm) Diameter of bore pile

D_p = 1500 (mm) diameter of bore pipe

σ'_v = 0.57 (Mpa) effective vertical prestress

=> Nominal tip resistance q_p = **2.661** (Mpa)

- As for the bottom diameter bigger than 1270mm, q_p has to reduce as follows:

$q_{pr} = 1270 * q_p / D_p$ = **2.253** (Mpa)

- Tip resistance of pile:

$Q_b = \phi * q_p * A_b$ = 2189.57 (kN)

With tip resistance factor : ϕ = **0.55** (Table 10.5.5.2.4-1 AASHTO2007)

2.3. Pile capacity:

- Design resistance of the single pile :

Q_T = (Q_s+Q_P) = 13122.4 kN

- Resistance of the resisting pile according to the pile group:

Q_R = $\eta.Q_T$ = **10060.48** kN

In which

+ η = 0.77 Effective factor due to the pile working by group (AASHTO 2007 - 10.8.3.6.3)

+ d = 4.5 (m) The distance from the center to center of pile.

2.4. CONCLUSION

- Final pile capacity: Q_R = 10060.48 kN
- Seftweight of pile: W = 1486.17 kN
- Maximum internal force applied to pile: P_{max} = 8262 kN
- Check pile capacity : P_{max} + W ≤ Q_R

⇔ 9748.17 < 10060.48 **OK**

3. THE SETTLEMENT OF PILE GROUP:

- The Settlement of pile groups in cohesionless soils using SPT value:

$$\rho = \frac{30qI\sqrt{B}}{N_{60}} \quad \text{(mm)} \quad (10.7.2.3.3\text{-}1)$$

In which: - q: net foundation pressure applid at $2D_b/3$, this pressure is equal to the applied load at the top of group divided by the area of the equivalent footing and does not include the weight of the piles or soil between the piers (Mpa).

- B : Width or smallest dimensions of pile group (mm).
- I : Influence factor of the effective group embedment (dim)

I = 1 - 0,125D'/B

- D' : Effective depth taken as $2D_b/3$, (mm)
- D_b : depth of embedment of piles in layer that provides support, (mm)

Nmax	Equivalent area	B	D_b	D'	I	N_{60}	q	ρ	Check
(Kn)	(mm^2)	(mm)	(mm)	(mm)	(dim)	blow	(Mpa)	(mm)	$\rho < 25.4$
38264	2.61E+08	7000	9746	6497.33	0.88	63	0.15	5.21	Ok

PILE CAPACITY AND SETTLEMENT OF GROUP PILE

1. DATA

- Code of boreholes		**P5**	
- Pile diameter :		1500	mm
- Elevation of the underground water	EL_3 =	1.00	m
- Ground elevation after scour	EL_4 =	-2.890	m
- Base bottom elevation	EL_1 =	-0.856	m
- Expected pipe tip elevation	EL_2 =	-58.856	m
- Pipe length	L =	58.0	m
- Perimeter of cross section	P =	4.71	m
- Area of the pile cross section	A_b =	1.77	m2
- Concrete weight	γ_c =	24.50	kN/m3
Soil layer at Tip of pile:		2	

('1' =cohesionless soil, '2' = cohesive soil, '3' = intermediate material , '4' = rock)

* Reference document

+ Standard 22 TCN 272-05

+ Standard AASHTO LRFD 2007

Legends

* Because the pile foundation of the bridge crosses the soil area with strong weather rock and IGM, so 22TCN272-05 can not meet requirement to base for the calculation of the pipe load bearing capacity, then AASHTO 2007 is referred to calculate the process items which standard 22TCN272-05 has not yet mentioned

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+ For sandy soil:

$\beta = 1.5 - 7.7\times10^{-3}(z)^{0.5}$ for $N_{60} \geq 15$

$\beta = N_{60}/15[1.5 - 7.7\times10^{-3}(z)^{0.5}]$ for $N_{60} < 15$

+ For Gravelly sand and gravels and SPT values are more than 50 blows/30cm:

$\beta = 2-0.00082\ (z)^{0.5}$ with $0.25 < \beta < 1.8$

With : + z : The depth of the subsoil layer at the middle location of the soil layer

+ N : The average number of hammers SPT (not yet calibrated) along the pile body (hammer/300)

2.1.2. As for conhesive soil, side wall friction is calculated according to the method α

$q_s = \alpha S_u$ (10.8.3.3.1-1)

In which:

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+ S_u : medium non- drain shear resistance strengthen

* Calculation method **S_u:**

$S_u = 0.06*N_{60}$ (bar) $= 0.006*N_{60}$ (MPa) (Terzaghi & Peck)

* Calculation method **a:** (10.8.3.5.1b AASHTO LRFD 2007)

$\alpha = 0.55$ with $S_u/0.101 \leq 1.5$

$\alpha = 0.55 - 0.1(S_u/0.101 - 1.5)$ with $1.5 \leq S_u/0.101 \leq 2.5$

*** Calculation method of effective vertical stress s'_v at the center of each soil layer:**

$\sigma'_z = \sigma_{vz} - u_z$

At the soil layer over the underground water level : $\sigma_v = \gamma.z_i$; $u_z = 0$

At the soil layer under the ground water level : $\sigma_v = \gamma_{sat}.z_i$; $u_z = z_i$

Suppose: clay layer and water- penetration weathered rock layer

Soil layer placed under the ground water level and obtain the penetration, use the saturated weight

In which

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+ σ'_v : effective stress at the depth z (kN/m²)

+ u : Pore water pressure (kN/m²)

+ z_i : Depth to the middle point of the i layer counted from the surfac (m)

+ l_i : thickness of the i soil layer (m)

+ γ : Natural density of the soil layer above the ground water level (kN/m³)

+ γ_{sat}: the saturated desnsity of the soil layer under the ground wate (kN/m³)

$$\gamma_{sat} = \gamma_d(1 - \gamma_w/\gamma_s) + \gamma_w$$

+ γ_d : Dry density (kN/m³)

+ γ_s : Unit weight (kN/m³)

+ γ_w : water density γ_w = 10 (kN/m³)

+ With: Energy efficiency E_h = 50

Name of layer	Layer thickness	Elevation of the layer bottom	depth	Unit weight	Dry density	saturated density	Total stress	Pore water pressure	Effective stress
	li		z_i	γ_s	γ_d	γ_{sat}	σ_{vz}	u	σ'_z
	(m)	(m)	(m)	(kN/m³)	(kN/m³)	(kN/m³)	(kN/m²)	(kN/m²)	(kN/m²)
		-0.86							
2	2.03	-2.89	1.02	0.00	0.00	0.00	0.00	0.00	0.00
2	1.18	-4.07	2.62	26.46	14.40	18.96	49.75	26.24	23.51
tk2	3.50	-7.57	4.96	26.92	11.48	17.22	85.46	49.64	35.82
2	18.50	-26.07	15.96	26.46	14.40	18.96	302.64	159.64	143.00
4	4.00	-30.07	27.21	26.60	14.50	19.05	518.40	272.14	246.26
4	8.70	-38.77	33.56	26.69	14.50	19.07	639.97	335.64	304.33
6a	5.10	-43.87	40.46	26.91	13.12	18.24	738.25	404.64	333.61
7a	4.00	-47.87	45.01	26.52	14.50	19.03	856.73	450.14	406.59
7a	11.80	-59.67	52.91	26.52	16.10	20.03	1059.82	529.14	530.68
8	-0.81	-58.86	58.41	26.91	18.37	21.54	1258.29	584.07	674.22
	1.50	-60.36							

Name of layer	Soil type	σ'_z (N/mm²)	SPT N (Blow/30cm)	N_{60} = N*E_h/60	S_u (N/mm²)	α	β	q_s (N/mm²)	Resistance factor φs	value $\varphi_s l_i q_s$ (N/mm)
2	1	0.000	0	-	-	-			0.55	0.00
2	1	0.024	6	5	-	-	0.442	0.010	0.55	6.75
tk2	1	0.036	4	3	-	-	0.255	0.009	0.55	17.60
2	1	0.143	22	18	-	-	0.527	0.075	0.55	766.98
4	1	0.246	29	24	-	-	0.250	0.062	0.55	135.44
4	1	0.304	25	21	-	-	0.250	0.076	0.55	364.06
6a	2	0.334	4	3	0.020	0.55	0.250	0.011	0.45	25.25
7a	1	0.407	15	13	-	-	0.250	0.102	0.55	223.62
7a	1	0.531	43	36	-	-	0.250	0.133	0.55	861.03
8	2	0.674	75	63	0.375	0.55	0.250	0.206	0.45	-75.55
	Total									2325.18

Total resistance of the side wall: $Q_s = \varphi_s \phi_i A_s = \Sigma \varphi_s \phi_i l_i P$ = **10957.1** (Kn)

φ_s: Shaft resistance factor of the pile in the ground (according to the table 10.5.5.2.4-1 AASHTO 2007)

2.2. PILE TIP RESISTANCE q_p:

Soil layer at Tip of pile: 2 Clay layer

2.2.1. Pile tip resistance in the clay soil

Fomular: **q_p = NcSu <=4** (CT: 10.8.3.3.2-1)

q_p = NcSu = 3.38

In which Nc = 6[1+0.2(Z/D)] <= 9

Nc = 6[1+0.2(Z/D)] = 52.4

Reduce:

If Su <0.024 Mpa Nc should be multiplied by 0,67

In which

D = 1500 (mm) - Diameter of pile

Z = 58000 (mm) - Penetration of shaft

Su = 0.375 (Mpa)

Nc = 9

=> q_p **= 3.375** (Mpa)

As for the bottom diameter bigger than 1270mm, qp has to reduce as follows:

$q_{pr} = 1270*q_p/D_p$

pile tip resistance property in the sandy soil as follows

Reduction q_{pr} **= 2.858** (Mpa)

- Tip resistance of pile:

$Q_b = \phi*q_p*A_b =$ 2019.85 (kN)

With tip resistance factor : $\phi =$ **0.4** (Table 10.5.5.2.4-1 AASHTO2007)

2.3. Pile capacity:

- Design resistance of the single pile :

$Q_T = (Q_s+Q_P) =$ 12977.0 kN

- Resistance of the resisting pile according to the pile group:

$Q_R = \eta.Q_T =$ **9949.02** kN

In which

+ η = 0.77 Effective factor due to the pile working by group (AASHTO 2007 - 10.8.3.6.3)

+ d = 4.5 (m) The distance from the center to center of pile.

2.4. CONCLUSION

- Final pile capacity: Q_R = 9949.02 kN
- Seftweight of pile: W = 1486.17 kN
- Maximum internal force applied to pile: P_{max} = 8262 kN
- Check pile capacity : $P_{max} + W \leq Q_R$

⇔ 9748.17 < 9949.02 **OK**

3. THE SETTLEMENT OF PILE GROUP:

- The Settlement of pile groups in cohesionless soils using SPT value:

$$\rho = \frac{30qI\sqrt{B}}{N_{60}}$$ (mm) (10.7.2.3.3-1)

In which: - q: net foundation pressure applid at $2D_b/3$, this pressure is equal to the applied load at the top of group divided by the area of the equivalent footing and does not include the weight of the piles or soil between the piers (Mpa).

- B : Width or smallest dimensions of pile group (mm).
- I : Influence factor of the effective group embedment (dim)

$I = 1 - 0,125D'/B$

- D' : Effective depth taken as $2D_b/3$, (mm)
- D_b : depth of embedment of piles in layer that provides support, (mm)

Nmax	Equivalent area	B	D_b	D'	I	N_{60}	q	ρ	Check
(Kn)	(mm^2)	(mm)	(mm)	(mm)	(dim)	blow	(Mpa)	(mm)	$\rho < 25.4$
38264	143500000	7000	10986	7324.00	0.87	63	0.27	9.31	Ok

PILE CAPACITY AND SETTLEMENT OF GROUP PILE

1. DATA

- Code of boreholes		P6	
- Pile diameter :		1500	mm
- Elevation of the underground water	EL_3 =	1.00	m
- Ground elevation after scour	EL_4 =	1.540	m
- Base bottom elevation	EL_1 =	-0.806	m
- Expected pipe tip elevation	EL_2 =	-62.806	m
- Pipe length	L =	62.0	m
- Perimeter of cross section	P =	4.71	m
- Area of the pile cross section	A_b =	1.77	m2
- Concrete weight	γ_c =	24.50	kN/m3
Soil layer at Tip of pile:		3	

('1' =cohesionless soil, '2' = cohesive soil, '3' = intermediate material , '4' = rock)

* Reference document

+ Standard 22 TCN 272-05
+ Standard AASHTO LRFD 2007

Legends

* Because the pile foundation of the bridge crosses the soil area with strong weather rock and IGM, so 22TCN272-05 can not meet requirement to base for the calculation of the pipe load bearing capacity, then AASHTO 2007 is referred to calculate the process items which standard 22TCN272-05 has not yet mentioned

2. CALCULATION FOR THE BEARING FORCE OF BORED PILES

2.1. SIDEWALL FRICTION q_s:

2.1.1. For cohesionless soil

+ Using Method β : O'Neil and Reese (1999)

$q_s = \beta\sigma'_v \leq 0.19$MPa; with $0.25 < \beta < 1.2$

+ For sandy soil:

$\beta = 1.5 - 7.7x10^{-3}(z)^{0.5}$ for $N_{60} \geq 15$

$\beta = N_{60}/15[1.5 - 7.7x10^{-3}(z)^{0.5}]$ for $N_{60} < 15$

+ For Gravelly sand and gravels and SPT values are more than 50 blows/30cm:

$\beta = 2-0.00082\ (z)^{0.5}$ with $0.25 < \beta < 1.8$

With : + z : The depth of the subsoil layer at the middle location of the soil layer

+ N : The average number of hammers SPT (not yet calibrated) along the pile body (hammer/300)

2.1.2. As for conhesive soil, side wall friction is calculated according to the method α

$q_s = \alpha S_u$ (10.8.3.3.1-1)

In which:

+ The cohesive factor a is calculated according to the method API (10.7.3.3.2a-1)
+ S_u : medium non- drain shear resistance strengthen

* Calculation method S_u**:**

$S_u = 0.06*N_{60}$ (bar) = $0.006*N_{60}$ (MPa) (Terzaghi & Peck)

* Calculation method a**:** (10.8.3.5.1b AASHTO LRFD 2007)

$\alpha = 0.55$ with $S_u/0.101 \leq 1.5$

$\alpha = 0.55 - 0.1(S_u/0.101 - 1.5)$ with $1.5 \leq S_u/0.101 \leq 2.5$

*** Calculation method of effective vertical stress s'_v at the center of each soil layer:**

$\sigma'_z = \sigma_{vz} - u_z$

At the soil layer over the underground water level : $\sigma_v = \gamma.z_i$; $u_z = 0$

At the soil layer under the ground water level : $\sigma_v = \gamma_{sat}.z_i$; $u_z = z_i$

Suppose: clay layer and water- penetration weathered rock layer

Soil layer placed under the ground water level and obtain the penetration, use the saturated weight

In which

+ σ_v : total stress at the depth z (kN/m^2)
+ σ'_v : effective stress at the depth z (kN/m^2)
+ u : Pore water pressure (kN/m^2)
+ z_i : Depth to the middle point of the i layer counted from the surfac (m)
+ l_i : thickness of the i soil layer (m)
+ γ : Natural density of the soil layer above the ground water level (kN/m^3)

+ γ_{sat}: the saturated desnsity of the soil layer under the ground wate (kN/m^3)

$$\gamma_{sat} = \gamma_d(1 - \gamma_w/\gamma_s) + \gamma_w$$

+ γ_d : Dry density (kN/m^3)

+ γ_s : Unit weight (kN/m^3)

+ γ_w : water density $\gamma_w =$ 10 (kN/m^3)

+ With: Energy efficiency E_h = 50

Name of layer	Layer thickness	Elevation of the layer bottom	depth	Unit weight	Dry density	saturated density	Total stress	Pore water pressure	Effective stress
	l_i		z_i	γ_s	γ_d	γ_{sat}	σ_{vz}	u	σ'_z
	(m)	(m)	(m)	(kN/m^3)	(kN/m^3)	(kN/m^3)	(kN/m^2)	(kN/m^2)	(kN/m^2)
		-0.81							
2	1.50	-2.31	0.75	0.00	0.00	0.00	0.00	0.00	0.00
2	4.00	-6.31	3.50	26.46	14.40	18.96	66.39	35.02	31.37
2	4.00	-10.31	7.50	26.92	11.48	17.22	129.19	75.04	54.15
2	22.40	-32.71	20.70	26.46	14.40	18.96	392.50	207.04	185.46
3	2.10	-34.81	32.95	26.56	11.74	17.32	570.76	329.54	241.22
4	5.80	-40.61	36.90	26.69	14.50	19.07	703.66	369.04	334.62
6a	7.40	-48.01	43.50	26.91	13.12	18.24	793.71	435.04	358.67
7a	4.00	-52.01	49.20	26.52	14.50	19.03	936.47	492.04	444.43
7a	6.60	-58.61	54.50	26.52	16.10	20.03	1091.67	545.04	546.63
7b	4.20	-62.81	59.90	26.69	16.10	20.07	1202.10	599.02	603.08
	1.50	-64.31							

Name of layer	Soil type	σ'_z (N/mm^2)	SPT N (Blow/30cm)	$N_{60} = N^*E_h/60$	S_u (N/mm^2)	α	β	q_s (N/mm^2)	Resistance factor φs	value $\varphi_s l_i q_s$ (N/mm)
2	1	0.000	0	-	-	-			0.55	0.00
2	1	0.031	4	3	-	-	0.278	0.009	0.55	19.24
2	1	0.054	6	5	-	-	0.305	0.017	0.55	36.38
2	1	0.185	15	12	-	-	0.384	0.071	0.55	877.43
3	2	0.241	3	3	0.015	0.55	0.250	0.008	0.45	7.80
4	1	0.335	15	13	-	-	0.250	0.084	0.55	266.86
6a	2	0.359	6	5	0.030	0.55	0.250	0.017	0.45	54.95
7a	1	0.444	26	22	-	-	0.250	0.111	0.55	244.44
7a	1	0.547	44	37	-	-	0.250	0.137	0.55	496.06
7b	3	0.603	75	63	-	-	0.250	0.151	0.60	379.58
	Total									2382.73

Total resistance of the side wall: $Q_s = \varphi_s\phi_i A_s = \Sigma\varphi_s\phi_i l_i P =$ **11228.4** (Kn)

φ_s: Shaft resistance factor of the pile in the ground (according to the table 10.5.5.2.4-1 AASHTO 2007)

2.2. PILE TIP RESISTANCE q_p:

Soil layer at Tip of pile: 3 IGM

2.2.1. Pile tip resistance: Method Reese and O'Neill (1999)

- Nominal Tip resistance follow:

$q_p = 0.057N$ with $N \leq 50$

$q_p = 0.59\sigma'_v((N60(Pa/\sigma'_v))$^0.8 with N > 50

In which

N = 75 (hammer /300mm)-Number of hammer SPT-N has not been calibrated

D = 1500 (mm) Diameter of bore pile

D_p = 1500 (mm) diameter of bore pipe

σ'_v = 0.60 (Mpa) effective vertical prestress

=> Nominal tip resistance q_p = **2.694** (Mpa)

- As for the bottom diameter bigger than 1270mm, q_p has to reduce as follows:

$q_{pr} = 1270^*q_p/D_p$ = **2.281** (Mpa)

- **Tip resistance of pile:**

$Q_b = \phi * q_p * A_b =$ 2217.08 (kN)

With tip resistance factor : $\phi =$ **0.55** (Table 10.5.5.2.4-1 AASHTO2007)

2.3. Pile capacity:

- Design resistance of the single pile :

$Q_T = (Q_s + Q_P) =$ 13445.4 kN

- Resistance of the resisting pile according to the pile group:

$Q_R = \eta . Q_T =$ **10308.17** kN

In which

+ $\eta =$ 0.77 Effective factor due to the pile working by group (AASHTO 2007 - 10.8.3.6.3)

+ d = 4.5 (m) The distance from the center to center of pile.

2.4. CONCLUSION

- Final pile capacity: $Q_R =$ 10308.17 kN
- Seftweight of pile: W = 1588.66 kN
- Maximum internal force applied to pile: $P_{max} =$ 8262 kN
- Check pile capacity : $P_{max} + W \leq Q_R$

$\Leftrightarrow$ 9850.66 < 10308.17 **OK**

3. THE SETTLEMENT OF PILE GROUP:

- The Settlement of pile groups in cohesionless soils using SPT value:

$$\rho = \frac{30qI\sqrt{B}}{N_{60}}$$ (mm) (10.7.2.3.3-1)

In which: - q: net foundation pressure applid at $2D_b/3$, this pressure is equal to the applied load at the top of group divided by the area of the equivalent footing and does not include the weight of the piles or soil between the piers (Mpa).

- B : Width or smallest dimensions of pile group (mm).
- I : Influence factor of the effective group embedment (dim)

  I = 1 - 0,125D'/B

- D' : Effective depth taken as $2D_b/3$, (mm)
- D_b : depth of embedment of piles in layer that provides support, (mm)

Nmax	Equivalent area	B	D_b	D'	I	N_{60}	q	ρ	Check
(Kn)	(mm^2)	(mm)	(mm)	(mm)	(dim)	blow	(Mpa)	(mm)	ρ <25.4
38264	2.75E+08	7000	10796	7197.33	0.87	63	0.14	4.87	Ok

PILE CAPACITY AND SETTLEMENT OF GROUP PILE

1. DATA

- Code of boreholes		P7	
- Pile diameter :		1500	mm
- Elevation of the underground water	EL_3 =	1.00	m
- Ground elevation after scour	EL_4 =	1.740	m
- Base bottom elevation	EL_1 =	-0.826	m
- Expected pipe tip elevation	EL_2 =	-62.826	m
- Pipe length	L =	62.0	m
- Perimeter of cross section	P =	4.71	m
- Area of the pile cross section	A_b =	1.77	m2
- Concrete weight	γ_c =	24.50	kN/m3
Soil layer at Tip of pile:		2	

('1' =cohesionless soil, '2' = cohesive soil, '3' = intermediate material , '4' = rock)

* Reference document

+ Standard 22 TCN 272-05
+ Standard AASHTO LRFD 2007

Legends

* Because the pile foundation of the bridge crosses the soil area with strong weather rock and IGM, so 22TCN272-05 can not meet requirement to base for the calculation of the pipe load bearing capacity, then AASHTO 2007 is referred to calculate the process items which standard 22TCN272-05 has not yet mentioned

2. CALCULATION FOR THE BEARING FORCE OF BORED PILES

2.1. SIDEWALL FRICTION q_s:

2.1.1. For cohesionless soil

+ Using Method β : O'Neil and Reese (1999)

$q_s = \beta\sigma'_v \le 0.19$MPa; with $0.25 < \beta < 1.2$

+ For sandy soil:

$\beta = 1.5 - 7.7\text{x}10^{-3}(z)^{0.5}$ for $N_{60} \ge 15$

$\beta = N_{60}/15[1.5 - 7.7\text{x}10^{-3}(z)^{0.5}]$ for $N_{60} < 15$

+ For Gravelly sand and gravels and SPT values are more than 50 blows/30cm:

$\beta = 2-0.00082\ (z)^{0.5}$ with $0.25 < \beta < 1.8$

With : + z : The depth of the subsoil layer at the middle location of the soil layer

+ N : The average number of hammers SPT (not yet calibrated) along the pile body (hammer/300)

2.1.2. As for conhesive soil, side wall friction is calculated according to the method α

$q_s = \alpha S_u$ (10.8.3.3.1-1)

In which:

+ The cohesive factor a is calculated according to the method API (10.7.3.3.2a-1)
+ S_u : medium non- drain shear resistance strengthen

* Calculation method S_u**:**

$S_u = 0.06*N_{60}$ (bar) $= 0.006*N_{60}$ (MPa) (Terzaghi & Peck)

* Calculation method a**:** (10.8.3.5.1b AASHTO LRFD 2007)

$\alpha = 0.55$ with $S_u/0.101 \le 1.5$

$\alpha = 0.55 - 0.1(S_u/0.101 - 1.5)$ with $1.5 \le S_u/0.101 \le 2.5$

*** Calculation method of effective vertical stress s'$_v$ at the center of each soil layer:**

$\sigma'_z = \sigma_{vz} - u_z$

At the soil layer over the underground water level : $\sigma_v = \gamma.z_i$; $u_z = 0$

At the soil layer under the ground water level : $\sigma_v = \gamma_{sat}.z_i$; $u_z = z_i$

Suppose: clay layer and water- penetration weathered rock layer

Soil layer placed under the ground water level and obtain the penetration, use the saturated weight

In which

+ σ_v : total stress at the depth z (kN/m^2)
+ σ'_v : effective stress at the depth z (kN/m^2)
+ u : Pore water pressure (kN/m^2)
+ z_i : Depth to the middle point of the i layer counted from the surfac (m)
+ l_i : thickness of the i soil layer (m)
+ γ : Natural density of the soil layer above the ground water level (kN/m^3)

+ γ_{sat}: the saturated desnsity of the soil layer under the ground wate (kN/m^3)

$$\gamma_{sat} = \gamma_d(1 - \gamma_w/\gamma_s) + \gamma_w$$

+ γ_d : Dry density (kN/m^3)

+ γ_s : Unit weight (kN/m^3)

+ γ_w : water density γ_w = 10 (kN/m^3)

+ With: Energy efficiency E_h = 50

Name of layer	Layer thickness	Elevation of the layer bottom	depth	Unit weight	Dry density	saturated density	Total stress	Pore water pressure	Effective stress
	li		z_i	γ_s	γ_d	γ_{sat}	σ_{vz}	u	σ'_z
	(m)	(m)	(m)	(kN/m^3)	(kN/m^3)	(kN/m^3)	(kN/m^2)	(kN/m^2)	(kN/m^2)
		-0.83							
2	1.50	-2.33	0.75	0.00	0.00	0.00	0.00	0.00	0.00
2	4.00	-6.33	3.50	26.46	14.40	18.96	66.39	35.02	31.37
2	4.00	-10.33	7.50	26.92	14.40	19.05	142.96	75.04	67.92
2	15.60	-25.93	17.30	26.46	14.40	18.96	328.05	173.04	155.01
3	4.20	-30.13	27.20	26.56	11.74	17.32	471.17	272.04	199.13
4	9.80	-39.93	34.20	26.52	14.50	19.03	650.99	342.04	308.95
6a	7.40	-47.33	42.80	26.91	13.12	18.24	780.94	428.04	352.90
7a	6.00	-53.33	49.50	26.52	14.50	19.03	942.18	495.04	447.14
7a	4.10	-57.43	54.55	26.52	16.10	20.03	1092.67	545.54	547.13
8	5.40	-62.83	59.30	26.69	16.10	20.07	1190.06	593.02	597.04
	1.50	-64.33							

Name of layer	Soil type	σ'_z (N/mm^2)	SPT N (Blow/30cm)	$N_{60} = N^*E_h/60$	S_u (N/mm^2)	α	β	q_s (N/mm^2)	Resistance factor φs	value $\varphi_s l_i q_s$ (N/mm)
2	1	0.000	0	-	-	-			0.55	0.00
2	1	0.031	5	4	-	-	0.313	0.010	0.55	21.64
2	1	0.068	5	4	-	-	0.278	0.019	0.55	41.49
2	1	0.155	14	12	-	-	0.448	0.069	0.55	596.00
3	2	0.199	5	4	0.025	0.55	0.250	0.014	0.45	25.99
4	1	0.309	25	21	-	-	0.250	0.077	0.55	416.30
6a	2	0.353	5	4	0.025	0.55	0.250	0.014	0.45	45.79
7a	1	0.447	29	24	-	-	0.250	0.112	0.55	368.89
7a	1	0.547	47	39	-	-	0.250	0.137	0.55	308.44
8	2	0.597	85	71	0.425	0.55	0.250	0.234	0.45	567.59
	Total									2392.14

Total resistance of the side wall: $Q_s = \varphi_s \phi_i A_s = \Sigma \varphi_s \phi_i l_i P$ = **11272.7** (Kn)

φ_s: Shaft resistance factor of the pile in the ground (according to the table 10.5.5.2.4-1 AASHTO 2007)

2.2. PILE TIP RESISTANCE q_p:

Soil layer at Tip of pile: 2 Clay layer

2.2.1. Pile resistance in the clay soil

Fomular: q_p **= NcSu <=4** (CT: 10.8.3.3.2-1)

q_p **= NcSu** = 3.83

In which Nc = 6[1+0.2(Z/D)] <= 9

Nc = 6[1+0.2(Z/D)] = = 55.6

Reduce:

If Su <0.024 Mpa Nc should be multiplied by 0,67

In which

D = 1500 (mm) - Diameter of pile

Z = 62000 (mm) - Penetration of shaft

Su = 0.425 (Mpa)

Nc = 9

Vậy q_p **= 3.825** (Mpa)

As for the bottom diameter bigger than 1270mm, qp has to reduce as follows:

$q_{pr} = 1270*q_p/D_p$

pile tip resistance property in the sandy soil as follows

Reduction q_{pr} **= 3.239** (Mpa)

- Tip resistance of pile:

$Q_b = \phi * q_p * A_b =$ 2289.16 (kN)

With tip resistance factor : $\phi =$ **0.4** (Table 10.5.5.2.4-1 AASHTO2007)

2.3. Pile capacity:

- Design resistance of the single pile :

$Q_T = (Q_s + Q_P) =$ 13561.8 kN

- Resistance of the resisting pile according to the pile group:

$Q_R = \eta . Q_T =$ **10397.42** kN

In which

+ $\eta =$ 0.77 Effective factor due to the pile working by group (AASHTO 2007 - 10.8.3.6.3)

+ d = 4.5 (m) The distance from the center to center of pile.

2.4. CONCLUSION

- Final pile capacity: $Q_R =$ 10397.42 kN
- Seftweight of pile: W = 1588.66 kN
- Maximum internal force applied to pile: $P_{max} =$ 8306 kN
- Check pile capacity : $P_{max} + W \leq Q_R$

$\Leftrightarrow$ 9894.66 < 10397.42 **OK**

3. THE SETTLEMENT OF PILE GROUP:

- The Settlement of pile groups in cohesionless soils using SPT value:

$$\rho = \frac{30qI\sqrt{B}}{N_{60}} \quad \text{(mm)} \quad (10.7.2.3.3\text{-}1)$$

In which: - q: net foundation pressure applid at $2D_b/3$, this pressure is equal to the applied load at the top of group divided by the area of the equivalent footing and does not include the weight of the piles or soil between the piers (Mpa).

- B : Width or smallest dimensions of pile group (mm).
- I : Influence factor of the effective group embedment (dim)

$I = 1 - 0,125D'/B$

- D' : Effective depth taken as $2D_b/3$, (mm)
- D_b : depth of embedment of piles in layer that provides support, (mm)

Nmax	Equivalent area	B	D_b	D'	I	N_{60}	q	ρ	Check
(Kn)	(mm^2)	(mm)	(mm)	(mm)	(dim)	blow	(Mpa)	(mm)	ρ <25.4
37981	2.57E+08	7000	9496	6330.67	0.89	71	0.15	4.64	Ok

PILE CAPACITY AND SETTLEMENT OF GROUP PILE

1. DATA

- Code of boreholes		P8	
- Pile diameter :		1500	mm
- Elevation of the underground water	EL_3 =	1.00	m
- Ground elevation after scour	EL_4 =	0.970	m
- Base bottom elevation	EL_1 =	-0.846	m
- Expected pipe tip elevation	EL_2 =	-62.846	m
- Pipe length	L =	62.0	m
- Perimeter of cross section	P =	4.71	m
- Area of the pile cross section	A_b =	1.77	m2
- Concrete weight	γ_c =	24.50	kN/m3
Soil layer at Tip of pile:		2	

('1' =cohesionless soil, '2' = cohesive soil, '3' = intermediate material , '4' = rock)

* Reference document

+ Standard 22 TCN 272-05
+ Standard AASHTO LRFD 2007

Legends

* Because the pile foundation of the bridge crosses the soil area with strong weather rock and IGM, so 22TCN272-05 can not meet requirement to base for the calculation of the pipe load bearing capacity, then AASHTO 2007 is referred to calculate the process items which standard 22TCN272-05 has not yet mentioned

2. CALCULATION FOR THE BEARING FORCE OF BORED PILES

2.1. SIDEWALL FRICTION q_s:

2.1.1. For cohesionless soil

+ Using Method β : O'Neil and Reese (1999)

$q_s = \beta\sigma'_v \le 0.19$MPa; with $0.25 < \beta < 1.2$

+ For sandy soil:

$\beta = 1.5 - 7.7x10^{-3}(z)^{0.5}$ for $N_{60} \ge 15$

$\beta = N_{60}/15[1.5 - 7.7x10^{-3}(z)^{0.5}]$ for $N_{60} < 15$

+ For Gravelly sand and gravels and SPT values are more than 50 blows/30cm:

$\beta = 2-0.00082\ (z)^{0.5}$ with $0.25 < \beta < 1.8$

With : + z : The depth of the subsoil layer at the middle location of the soil layer

+ N : The average number of hammers SPT (not yet calibrated) along the pile body (hammer/300)

2.1.2. As for conhesive soil, side wall friction is calculated according to the method α

$q_s = \alpha S_u$ (10.8.3.3.1-1)

In which:

+ The cohesive factor a is calculated according to the method API (10.7.3.3.2a-1)
+ S_u : medium non- drain shear resistance strengthen

* Calculation method **S_u:**

$S_u = 0.06*N_{60}$ (bar) $= 0.006*N_{60}$ (MPa) (Terzaghi & Peck)

* Calculation method **a:** (10.8.3.5.1b AASHTO LRFD 2007)

$\alpha = 0.55$ with $S_u/0.101 \le 1.5$

$\alpha = 0.55 - 0.1(S_u/0.101 - 1.5)$ with $1.5 \le S_u/0.101 \le 2.5$

*** Calculation method of effective vertical stress s'$_v$ at the center of each soil layer:**

$\sigma'_z = \sigma_{vz} - u_z$

At the soil layer over the underground water level : $\sigma_v = \gamma.z_i$; $u_z = 0$

At the soil layer under the ground water level : $\sigma_v = \gamma_{sat}.z_i$; $u_z = z_i$

Suppose: clay layer and water- penetration weathered rock layer

Soil layer placed under the ground water level and obtain the penetration, use the saturated weight

In which

+ σ_v : total stress at the depth z	(kN/m^2)
+ σ'_v : effective stress at the depth z	(kN/m^2)
+ u : Pore water pressure	(kN/m^2)
+ z_i : Depth to the middle point of the i layer counted from the surfac	(m)
+ l_i : thickness of the i soil layer	(m)
+ γ : Natural density of the soil layer above the ground water level	(kN/m^3)
+ γ_{sat}: the saturated desnsity of the soil layer under the ground wate	(kN/m^3)

$$\gamma_{sat} = \gamma_d(1-\gamma_w/\gamma_s) + \gamma_w$$

+ γ_d : Dry density (kN/m³)
+ γ_s : Unit weight (kN/m³)
+ γ_w : water density $\gamma_w =$ 10 (kN/m³)
+ With: Energy efficiency E_h = 50

Name of layer	Layer thickness	Elevation of the layer bottom	depth	Unit weight	Dry density	saturated density	Total stress	Pore water pressure	Effective stress
	li		z_i	γ_s	γ_d	γ_{sat}	σ_{vz}	u	σ'_z
	(m)	(m)	(m)	(kN/m³)	(kN/m³)	(kN/m³)	(kN/m²)	(kN/m²)	(kN/m²)
		-0.85							
2	1.50	-2.35	0.75	0.00	0.00	0.00	0.00	0.00	0.00
2	4.00	-6.35	3.50	26.46	14.40	18.96	66.39	35.02	31.37
2	15.06	-21.41	13.03	26.92	14.40	19.05	248.31	130.34	117.97
3	3.50	-24.91	22.31	26.56	11.74	17.32	386.47	223.14	163.33
4	15.10	-40.01	31.61	26.56	14.50	19.04	601.95	316.14	285.81
6a	5.10	-45.11	41.71	26.91	13.12	18.24	761.05	417.14	343.91
6b	4.10	-49.21	46.31	26.92	15.22	19.57	906.19	463.14	443.05
7a	4.00	-53.21	50.36	26.57	14.50	19.04	959.07	503.64	455.43
7a	4.90	-58.11	54.81	26.57	16.10	20.04	1098.50	548.14	550.36
8	4.74	-62.85	59.63	26.69	16.10	20.07	1196.68	596.32	600.36
	1.50	-64.35							

Name of layer	Soil type	σ'_z (N/mm²)	SPT N (Blow/30cm)	$N_{60} = N{*}E_h/60$	S_u (N/mm²)	α	β	q_s (N/mm²)	Resistance factor φs	value $\varphi_s l_i q_s$ (N/mm)
2	1	0.000	0	-	-	-			0.55	0.00
2	1	0.031	7	5	-	-	0.453	0.014	0.55	31.26
2	1	0.118	13	11	-	-	0.555	0.065	0.55	542.00
3	2	0.163	6	5	0.030	0.55	0.250	0.017	0.45	25.99
4	1	0.286	29	24	-	-	0.250	0.071	0.55	593.42
6a	2	0.344	5	4	0.025	0.55	0.250	0.014	0.45	31.56
6b	2	0.443	18	15	0.090	0.55	0.250	0.050	0.45	91.33
7a	1	0.455	34	28	-	-	0.250	0.114	0.55	250.49
7a	1	0.550	46	38	-	-	0.250	0.138	0.55	370.81
8	2	0.600	85	71	0.425	0.55	0.250	0.234	0.45	498.17
	Total									2435.01

Total resistance of the side wall: $Q_s = \varphi_s\phi_i A_s = \Sigma\varphi_s\phi_i l_i P =$ **11474.7** (Kn)

φ_s: Shaft resistance factor of the pile in the ground (according to the table 10.5.5.2.4-1 AASHTO 2007)

2.2. PILE TIP RESISTANCE q_p:

Soil layer at Tip of pile: 2 Clay layer

2.2.1. Pile resistance in the clay soil

Fomular: q_p = NcSu <=4 (CT: 10.8.3.3.2-1)

q_p = NcSu = 3.83

In which Nc = 6[1+0.2(Z/D)] <= 9

Nc = 6[1+0.2(Z/D)] = = 55.6

Reduce:

If Su <0.024 Mpa Nc should be multiplied by 0,67

In which

D = 1500 (mm) - Diameter of pile

Z = 62000 (mm) - Penetration of shaft

Su = 0.425 (Mpa)

Nc = 9

=> q_p **= 3.825** (Mpa)

- As for the bottom diameter bigger than 1270mm, q_p has to reduce as follows:

$q_{pr} = 1270{*}q_p/D_p$ **= 3.239** (Mpa)

- Tip resistance of pile:

$Q_b = \phi * q_p * A_b =$ 2289.16 (kN)

With tip resistance factor : $\phi =$ **0.4** (Table 10.5.5.2.4-1 AASHTO2007)

2.3. Pile capacity:

- Design resistance of the single pile :

$Q_T = (Q_s + Q_P) =$ 13763.9 kN

- Resistance of the resisting pile according to the pile group:

$Q_R = \eta . Q_T =$ **10552.31** kN

In which

+ $\eta =$ 0.77 Effective factor due to the pile working by group (AASHTO 2007 - 10.8.3.6.3)

+ d = 4.5 (m) The distance from the center to center of pile.

2.4. CONCLUSION

- Final pile capacity: $Q_R =$ 10552.31 kN
- Seftweight of pile: W = 1588.66 kN
- Maximum internal force applied to pile: $P_{max} =$ 8306 kN
- Check pile capacity : $P_{max} + W \leq Q_R$

⇔ 9894.66 < 10552.31 **OK**

3. THE SETTLEMENT OF PILE GROUP:

- The Settlement of pile groups in cohesionless soils using SPT value:

$$\rho = \frac{30qI\sqrt{B}}{N_{60}} \quad \text{(mm)} \quad (10.7.2.3.3\text{-}1)$$

In which: - q: net foundation pressure applid at $2D_b/3$, this pressure is equal to the applied load at the top of group divided by the area of the equivalent footing and does not include the weight of the piles or soil between the piers (Mpa).

- B : Width or smallest dimensions of pile group (mm).
- I : Influence factor of the effective group embedment (dim)

$I = 1 - 0{,}125D'/B$

- D' : Effective depth taken as $2D_b/3$, (mm)
- D_b : depth of embedment of piles in layer that provides support, (mm)

Nmax	Equivalent area	B	D_b	D'	I	N_{60}	q	ρ	Check
(Kn)	(mm^2)	(mm)	(mm)	(mm)	(dim)	blow	(Mpa)	(mm)	$\rho < 25.4$
38264	2.59E+08	7000	9636	6424.00	0.89	71	0.15	4.63	Ok

PILE CAPACITY AND SETTLEMENT OF GROUP PILE

1. DATA

- Code of boreholes		**P9**	
- Pile diameter :		1500	mm
- Elevation of the underground water	EL_3 =	1.00	m
- Ground elevation after scour	EL_4 =	0.970	m
- Base bottom elevation	EL_1 =	-0.936	m
- Expected pipe tip elevation	EL_2 =	-62.936	m
- Pipe length	L =	62.0	m
- Perimeter of cross section	P =	4.71	m
- Area of the pile cross section	A_b =	1.77	m2
- Concrete weight	γ_c =	24.50	kN/m3
Soil layer at Tip of pile:		2	

('1' =cohesionless soil, '2' = cohesive soil, '3' = intermediate material , '4' = rock)

* Reference document

+ Standard 22 TCN 272-05
+ Standard AASHTO LRFD 2007

Legends

* Because the pile foundation of the bridge crosses the soil area with strong weather rock and IGM, so 22TCN272-05 can not meet requirement to base for the calculation of the pipe load bearing capacity, then AASHTO 2007 is referred to calculate the process items which standard 22TCN272-05 has not yet mentioned

2. CALCULATION FOR THE BEARING FORCE OF BORED PILES

2.1. SIDEWALL FRICTION q_s:

2.1.1. For cohesionless soil

+ Using Method β : O'Neil and Reese (1999)

$q_s = \beta\sigma'_v \leq 0.19$MPa; with $0.25 < \beta < 1.2$

+ For sandy soil:

$\beta = 1.5 - 7.7\times10^{-3}(z)^{0.5}$ for $N_{60} \geq 15$

$\beta = N_{60}/15[1.5 - 7.7\times10^{-3}(z)^{0.5}]$ for $N_{60} < 15$

+ For Gravelly sand and gravels and SPT values are more than 50 blows/30cm:

$\beta = 2-0.00082\ (z)^{0.5}$ with $0.25 < \beta < 1.8$

With : + z : The depth of the subsoil layer at the middle location of the soil layer

+ N : The average number of hammers SPT (not yet calibrated) along the pile body (hammer/300)

2.1.2. As for conhesive soil, side wall friction is calculated according to the method α

$q_s = \alpha S_u$ (10.8.3.3.1-1)

In which:

+ The cohesive factor a is calculated according to the method API (10.7.3.3.2a-1)
+ S_u : medium non- drain shear resistance strengthen

* Calculation method S_u**:**

$S_u = 0.06*N_{60}$ (bar) $= 0.006*N_{60}$ (MPa) (Terzaghi & Peck)

* Calculation method a**:** (10.8.3.5.1b AASHTO LRFD 2007)

$\alpha = 0.55$ with $S_u/0.101 \leq 1.5$

$\alpha = 0.55 - 0.1(S_u/0.101 - 1.5)$ with $1.5 \leq S_u/0.101 \leq 2.5$

*** Calculation method of effective vertical stress s'$_v$ at the center of each soil layer:**

$\sigma'_z = \sigma_{vz} - u_z$

At the soil layer over the underground water level : $\sigma_v = \gamma.z_i$; $u_z = 0$

At the soil layer under the ground water level : $\sigma_v = \gamma_{sat}.z_i$; $u_z = z_i$

Suppose: clay layer and water- penetration weathered rock layer

Soil layer placed under the ground water level and obtain the penetration, use the saturated weight

In which

+ σ_v : total stress at the depth z (kN/m^2)
+ σ'_v : effective stress at the depth z (kN/m^2)
+ u : Pore water pressure (kN/m^2)
+ z_i : Depth to the middle point of the i layer counted from the surfac (m)
+ l_i : thickness of the i soil layer (m)
+ γ : Natural density of the soil layer above the ground water level (kN/m^3)

+ γ_{sat}: the saturated desnsity of the soil layer under the ground wate (kN/m^3)

$\gamma_{sat} = \gamma_d(1 - \gamma_w/\gamma_s) + \gamma_w$

+ γ_d : Dry density (kN/m^3)

+ γ_s : Unit weight (kN/m^3)

+ γ_w : water density γ_w = 10 (kN/m^3)

+ With: Energy efficiency E_h = 50

Name of layer	Layer thickness	Elevation of the layer bottom	depth	Unit weight	Dry density	saturated density	Total stress	Pore water pressure	Effective stress
	li		z_i	γ_s	γ_d	γ_{sat}	σ_{vz}	u	σ'_z
	(m)	(m)	(m)	(kN/m^3)	(kN/m^3)	(kN/m^3)	(kN/m^2)	(kN/m^2)	(kN/m^2)
		-0.94							
2	1.50	-2.44	0.75	0.00	0.00	0.00	0.00	0.00	0.00
2	8.92	-11.36	5.96	26.46	14.40	18.96	113.03	59.62	53.41
tk2	1.80	-13.16	11.32	26.92	11.48	17.22	194.95	113.24	81.71
2	7.40	-20.56	15.92	26.56	14.40	18.98	302.21	159.24	142.97
3	3.20	-23.76	21.22	26.56	11.74	17.32	367.60	212.24	155.36
4	17.40	-41.16	31.52	26.52	14.50	19.03	599.98	315.24	284.74
6a	3.60	-44.76	42.02	26.91	13.12	18.24	766.71	420.24	346.47
7a	11.30	-56.06	49.47	26.57	14.50	19.04	942.12	494.74	447.38
7b	1.20	-57.26	55.72	26.60	16.10	20.05	1117.12	557.24	559.88
8	5.68	-62.94	59.16	26.91	18.37	21.54	1274.56	591.62	682.94
	1.50	-64.44							

Name of layer	Soil type	σ'_z (N/mm^2)	SPT N (Blow/30cm)	N_{60} = N*E_h/60	S_u (N/mm^2)	α	β	q_s (N/mm^2)	Resistance factor φs	value $\varphi_s l_i q_s$ (N/mm)
2	1	0.000	0	-	-	-			0.55	0.00
2	1	0.053	9	7	-	-	0.513	0.027	0.55	134.50
tk2	2	0.082	4	3	0.020	0.55	0.250	0.011	0.45	8.91
2	1	0.143	8	7	-	-	0.291	0.042	0.55	169.09
3	2	0.155	6	5	0.030	0.55	0.250	0.017	0.45	23.76
4	1	0.285	16	14	-	-	0.250	0.071	0.55	681.24
6a	2	0.346	5	4	0.023	0.55	0.250	0.013	0.45	20.79
7a	1	0.447	36	30	-	-	0.250	0.112	0.55	695.12
7b	3	0.560	60	50	-	-	0.250	0.140	0.60	100.78
8	2	0.683	85	71	0.425	0.55	0.250	0.234	0.45	597.04
	Total									2431.22

Total resistance of the side wall: $Q_s = \varphi_s \phi_i A_s = \Sigma \varphi_s \phi_i l_i P$ = **11456.8** (Kn)

φ_s: Shaft resistance factor of the pile in the ground (according to the table 10.5.5.2.4-1 AASHTO 2007)

2.2. PILE TIP RESISTANCE q_p:

Soil layer at Tip of pile: 2 Clay layer

2.2.1. Pile resistance in the clay soil

Fomular: **q_p = NcSu <=4** (CT: 10.8.3.3.2-1)

q_p = NcSu = 3.83

In which Nc = 6[1+0.2(Z/D)] <= 9

Nc = 6[1+0.2(Z/D)] = = 55.6

Reduce:

If Su <0.024 Mpa Nc should be multiplied by 0,67

In which

D = 1500 (mm) - Diameter of pile

Z = 62000 (mm) - Penetration of shaft

Su = 0.425 (Mpa)

Nc = 9

=> q_p **= 3.825** (Mpa)

As for the bottom diameter bigger than 1270mm, qp has to reduce as follows:

$q_{pr} = 1270*q_p/D_p$ **= 3.239** (Mpa)

- Tip resistance of pile:

$Q_b = \phi * q_p * A_b =$ 2289.16 (kN)

Pile tip resistance factor: **0.4** (Table 10.5.5.2.4-1 AASHTO2007)

2.3. Pile capacity:

- Design resistance of the single pile :

$Q_T = (Q_s + Q_P) =$ 13746.0 kN

- Resistance of the resisting pile according to the pile group:

$Q_R = \eta . Q_T =$ **10538.61** kN

In which

+ η = 0.77 Effective factor due to the pile working by group (AASHTO 2007 - 10.8.3.6.3)

+ d = 4.5 (m) The distance from the center to center of pile.

2.4. CONCLUSION

- Final pile capacity: Q_R = 10538.61 kN
- Seftweight of pile: W = 1588.66 kN
- Maximum internal force applied to pile: P_{max} = 8306 kN
- Check pile capacity : $P_{max} + W \leq Q_R$

$\Leftrightarrow$ 9894.66 < 10538.61 **OK**

3. THE SETTLEMENT OF PILE GROUP:

- The Settlement of pile groups in cohesionless soils using SPT value:

$$\rho = \frac{30qI\sqrt{B}}{N_{60}} \quad \text{(mm)} \quad (10.7.2.3.3\text{-}1)$$

In which: - q: net foundation pressure applid at $2D_b/3$, this pressure is equal to the applied load at the top of group divided by the area of the equivalent footing and does not include the weight of the piles or soil between the piers (Mpa).

- B : Width or smallest dimensions of pile group (mm).
- I : Influence factor of the effective group embedment (dim)

$I = 1 - 0,125D'/B$

- D' : Effective depth taken as $2D_b/3$, (mm)
- D_b : depth of embedment of piles in layer that provides support, (mm)

Nmax	Equivalent area	B	D_b	D'	I	N_{60}	q	ρ	Check
(Kn)	(mm^2)	(mm)	(mm)	(mm)	(dim)	blow	(Mpa)	(mm)	ρ <25.4
38264	2.23E+08	7000	6876	4584.00	0.92	71	0.17	5.58	Ok

PILE CAPACITY AND SETTLEMENT OF GROUP PILE

1. DATA

- Code of boreholes **P10**
- Pile diameter : 1500 mm
- Elevation of the underground water EL_3 = 1.00 m
- Ground elevation after scour EL_4 = -0.880 m
- Base bottom elevation EL_1 = -0.886 m
- Expected pipe tip elevation EL_2 = -62.886 m
- Pipe length L = 62.0 m
- Perimeter of cross section P = 4.71 m
- Area of the pile cross section A_b = 1.77 m2
- Concrete weight γ_c = 24.50 kN/m3

Soil layer at Tip of pile: 2

('1' =cohesionless soil, '2' = cohesive soil, '3' = intermediate material , '4' = rock)

* Reference document

+ Standard 22 TCN 272-05
+ Standard AASHTO LRFD 2007

Legends

* Because the pile foundation of the bridge crosses the soil area with strong weather rock and IGM, so 22TCN272-05 can not meet requirement to base for the calculation of the pipe load bearing capacity, then AASHTO 2007 is referred to calculate the process items which standard 22TCN272-05 has not yet mentioned

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$q_s = \beta\sigma'_v \le 0.19$MPa; with $0.25 < \beta < 1.2$

+ For sandy soil:

$\beta = 1.5 - 7.7\times10^{-3}(z)^{0.5}$ for $N_{60} \ge 15$

$\beta = N_{60}/15[1.5 - 7.7\times10^{-3}(z)^{0.5}]$ for $N_{60} < 15$

+ For Gravelly sand and gravels and SPT values are more than 50 blows/30cm:

$\beta = 2-0.00082\,(z)^{0.5}$ with $0.25 < \beta < 1.8$

With : + z : The depth of the subsoil layer at the middle location of the soil layer

+ N : The average number of hammers SPT (not yet calibrated) along the pile body (hammer/300)

2.1.2. As for conhesive soil, side wall friction is calculated according to the method α

$q_s = \alpha S_u$ (10.8.3.3.1-1)

In which:

+ The cohesive factor a is calculated according to the method API (10.7.3.3.2a-1)
+ S_u : medium non- drain shear resistance strengthen

* Calculation method S_u**:**

$S_u = 0.06*N_{60}$ (bar) $= 0.006*N_{60}$ (MPa) (Terzaghi & Peck)

* Calculation method a**:** (10.8.3.5.1b AASHTO LRFD 2007)

$\alpha = 0.55$ with $S_u/0.101 \le 1.5$

$\alpha = 0.55 - 0.1(S_u/0.101 - 1.5)$ with $1.5 \le S_u/0.101 \le 2.5$

*** Calculation method of effective vertical stress s'$_v$ at the center of each soil layer:**

$\sigma'_z = \sigma_{vz} - u_z$

At the soil layer over the underground water level : $\sigma_v = \gamma.z_i$; $u_z = 0$

At the soil layer under the ground water level : $\sigma_v = \gamma_{sat}.z_i$; $u_z = z_i$

Suppose: clay layer and water- penetration weathered rock layer

Soil layer placed under the ground water level and obtain the penetration, use the saturated weight

In which

+ σ_v : total stress at the depth z (kN/m^2)
+ σ'_v : effective stress at the depth z (kN/m^2)
+ u : Pore water pressure (kN/m^2)
+ z_i : Depth to the middle point of the i layer counted from the surfac (m)
+ l_i : thickness of the i soil layer (m)
+ γ : Natural density of the soil layer above the ground water level (kN/m^3)

+ γ_{sat}: the saturated desnsity of the soil layer under the ground wate (kN/m^3)

$$\gamma_{sat} = \gamma_d(1-\gamma_w/\gamma_s)+\gamma_w$$

+ γ_d : Dry density (kN/m^3)

+ γ_s : Unit weight (kN/m^3)

+ γ_w : water density γ_w = 10 (kN/m^3)

+ With: Energy efficiency E_h = 50

Name of layer	Layer thickness	Elevation of the layer bottom	depth	Unit weight	Dry density	saturated density	Total stress	Pore water pressure	Effective stress
	li		z_i	γ_s	γ_d	γ_{sat}	σ_{vz}	u	σ'_z
	(m)	(m)	(m)	(kN/m^3)	(kN/m^3)	(kN/m^3)	(kN/m^2)	(kN/m^2)	(kN/m^2)
		-0.89							
2	1.50	-2.39	0.75	0.00	0.00	0.00	0.00	0.00	0.00
2	9.00	-11.39	6.00	26.46	14.40	18.96	113.78	60.02	53.76
2	7.87	-19.26	14.44	26.56	14.40	18.98	274.03	144.39	129.64
3	8.50	-27.76	22.62	26.56	11.74	17.32	391.84	226.24	165.60
4	11.80	-39.56	32.77	26.52	14.50	19.03	623.77	327.74	296.03
6a	5.60	-45.16	41.47	26.91	13.12	18.24	756.67	414.74	341.93
7a	4.00	-49.16	46.27	26.57	14.50	19.04	881.18	462.74	418.44
7a	6.70	-55.86	51.62	26.60	14.50	19.05	983.38	516.24	467.14
8	7.03	-62.89	58.49	26.60	18.37	21.46	1255.36	584.87	670.49
	1.50	-64.39							

Name of layer	Soil type	σ'_z (N/mm^2)	SPT N (Blow/30cm)	N_{60} = $N*E_h/60$	S_u (N/mm^2)	α	β	q_s (N/mm^2)	Resistance factor φs	value $\varphi_s l_i q_s$ (N/mm)
2	1	0.000	0	-	-	-			0.55	0.00
2	1	0.054	11	9	-	-	0.650	0.035	0.55	173.20
2	1	0.130	10	8	-	-	0.364	0.047	0.55	204.26
3	2	0.166	6	5	0.030	0.55	0.250	0.017	0.45	63.11
4	1	0.296	21	17	-	-	0.250	0.074	0.55	480.31
6a	2	0.342	5	4	0.023	0.55	0.250	0.013	0.45	32.34
7a	1	0.418	45	38	-	-	0.250	0.105	0.55	230.14
7a	1	0.467	65	54	-	-	0.250	0.117	0.55	430.35
8	2	0.670	68	57	0.340	0.55	0.250	0.187	0.45	591.24
	Total									2204.95

Total resistance of the side wall: $Q_s = \varphi_s\phi_i A_s = \Sigma\varphi_s\phi_i l_i P$ = **10390.6** (Kn)

φ_s: Shaft resistance factor of the pile in the ground (according to the table 10.5.5.2.4-1 AASHTO 2007)

2.2. PILE TIP RESISTANCE q_p:

Soil layer at Tip of pile: 2 Clay layer

2.2.1. Pile resistance in the clay soil

Fomular: $q_p = NcSu <= 4$ (CT: 10.8.3.3.2-1)

$q_p = NcSu$ = 3.06

In which Nc = 6[1+0.2(Z/D)] <= 9

Nc = 6[1+0.2(Z/D)] = = 55.6

Reduce:

If Su <0.024 Mpa Nc should be multiplied by 0,67

In which

D = 1500 (mm) - Diameter of pile

Z = 62000 (mm) - Penetration of shaft

Su = 0.340 (Mpa)

Nc = 9

Vậy q_p **= 3.06** (Mpa)

As for the bottom diameter bigger than 1270mm, qp has to reduce as follows:

$q_{pr} = 1270*q_p/D_p$ **= 2.591** (Mpa)

- Tip resistance of pile:

$Q_b = \phi * q_p * A_b =$ 1831.33 (kN)

With tip resistance factor : $\phi =$ **0.4** (Table 10.5.5.2.4-1 AASHTO2007)

2.2.1. Pile tip resistance: Method Reese and O'Neill (1999)

- Nominal Tip resistance follow:

$q_p = 0.057N$ with $N \leq 50$

$q_p = 0.59\sigma'_v((N60(Pa/\sigma'_v))$^0.8 with $N > 50$

In which

N = 68 (hammer /300mm)-Number of hammer SPT-N has not been calibrated

D = 1500 (mm) Diameter of bore pile

D_p = 1500 (mm) diameter of bore pipe

σ'_v = 0.67 (Mpa) effective vertical prestress

=> Nominal tip resistance $q_p =$ **2.544** (Mpa)

- As for the bottom diameter bigger than 1270mm, q_p has to reduce as follows:

$q_{pr} = 1270 * q_p / D_p =$ **2.154** (Mpa)

- Tip resistance of pile:

$Q_b = \phi * q_p * A_b =$ 1522.79 (kN)

With tip resistance factor : $\phi =$ **0.4** (Table 10.5.5.2.4-1 AASHTO2007)

2.3. Pile capacity:

- Design resistance of the single pile :

$Q_T = (Q_s + Q_P) =$ 12221.9 kN

- Resistance of the resisting pile according to the pile group:

$Q_R = \eta . Q_T =$ **9370.12** kN

In which

+ η = 0.77 Effective factor due to the pile working by group (AASHTO 2007 - 10.8.3.6.3)

+ d = 4.5 (m) The distance from the center to center of pile.

2.4. CONCLUSION

- Final pile capacity: $Q_R =$ 9370.12 kN
- Seftweight of pile: $W =$ 1588.66 kN
- Maximum internal force applied to pile: $P_{max} =$ 7386 kN
- Check pile capacity : $P_{max} + W \leq Q_R$

$\Leftrightarrow$ 8974.66 < 9370.12 **OK**

3. THE SETTLEMENT OF PILE GROUP:

- The Settlement of pile groups using SPT value:

$$\rho = \frac{30qI\sqrt{B}}{N_{60}}$$ (mm) (10.7.2.3.3-1)

In which: - q: net foundation pressure applid at $2D_b/3$, this pressure is equal to the applied load at the top of group divided by the area of the equivalent footing and does not include the weight of the piles or soil between the piers (Mpa).

- B : Width or smallest dimensions of pile group (mm).
- I : Influence factor of the effective group embedment (dim)

$I = 1 - 0{,}125D'/B$

- D' : Effective depth taken as $2D_b/3$, (mm)
- D_b : depth of embedment of piles in layer that provides support, (mm)

Nmax	Equivalent area	B	D_b	D'	I	N_{60}	q	ρ	Check
(Kn)	(mm^2)	(mm)	(mm)	(mm)	(dim)	blow	(Mpa)	(mm)	$\rho < 25.4$
42078	3.17E+08	7000	13726	9150.67	0.84	57	0.13	4.92	Ok

PILE CAPACITY AND SETTLEMENT OF GROUP PILE

1. DATA

- Code of boreholes		P11	
- Pile diameter :		1500	mm
- Elevation of the underground water	EL_3 =	1.00	m
- Ground elevation after scour	EL_4 =	-3.220	m
- Base bottom elevation	EL_1 =	-2.105	m
- Expected pipe tip elevation	EL_2 =	-64.105	m
- Pipe length	L =	62.0	m
- Perimeter of cross section	P =	4.71	m
- Area of the pile cross section	A_b =	1.77	m2
- Concrete weight	γ_c =	24.50	kN/m3
Soil layer at Tip of pile:		2	

('1' =cohesionless soil, '2' = cohesive soil, '3' = intermediate material , '4' = rock)

* Reference document

+ Standard 22 TCN 272-05
+ Standard AASHTO LRFD 2007

Legends

* Because the pile foundation of the bridge crosses the soil area with strong weather rock and IGM, so 22TCN272-05 can not meet requirement to base for the calculation of the pipe load bearing capacity, then AASHTO 2007 is referred to calculate the process items which standard 22TCN272-05 has not yet mentioned

2. CALCULATION FOR THE BEARING FORCE OF BORED PILES

2.1. SIDEWALL FRICTION q_s:

2.1.1. For cohesionless soil

+ Using Method β : O'Neil and Reese (1999)

$q_s = \beta\sigma'_v \leq 0.19$MPa; with $0.25 < \beta < 1.2$

+ For sandy soil:

$\beta = 1.5 - 7.7\text{x}10^{-3}(z)^{0.5}$ for $N_{60} \geq 15$

$\beta = N_{60}/15[1.5 - 7.7\text{x}10^{-3}(z)^{0.5}]$ for $N_{60} < 15$

+ For Gravelly sand and gravels and SPT values are more than 50 blows/30cm:

$\beta = 2-0.00082\ (z)^{0.5}$ with $0.25 < \beta < 1.8$

With : + z : The depth of the subsoil layer at the middle location of the soil layer

+ N : The average number of hammers SPT (not yet calibrated) along the pile body (hammer/300)

2.1.2. As for conhesive soil, side wall friction is calculated according to the method α

$q_s = \alpha S_u$ (10.8.3.3.1-1)

In which:

+ The cohesive factor a is calculated according to the method API (10.7.3.3.2a-1)
+ S_u : medium non- drain shear resistance strengthen

* Calculation method **S_u:**

$S_u = 0.06{*}N_{60}$ (bar) = $0.006{*}N_{60}$ (MPa) (Terzaghi & Peck)

* Calculation method a**:** (10.8.3.5.1b AASHTO LRFD 2007)

$\alpha = 0.55$ with $S_u/0.101 \leq 1.5$

$\alpha = 0.55 - 0.1(S_u/0.101 - 1.5)$ with $1.5 \leq S_u/0.101 \leq 2.5$

*** Calculation method of effective vertical stress s'$_v$ at the center of each soil layer:**

$\sigma'_z = \sigma_{vz} - u_z$

At the soil layer over the underground water level : $\sigma_v = \gamma.z_i$; $u_z = 0$

At the soil layer under the ground water level : $\sigma_v = \gamma_{sat}.z_i$; $u_z = z_i$

Suppose: clay layer and water- penetration weathered rock layer

Soil layer placed under the ground water level and obtain the penetration, use the saturated weight

In which

+ σ_v : total stress at the depth z (kN/m^2)
+ σ'_v : effective stress at the depth z (kN/m^2)
+ u : Pore water pressure (kN/m^2)
+ z_i : Depth to the middle point of the i layer counted from the surfac (m)
+ l_i : thickness of the i soil layer (m)

+ γ : Natural density of the soil layer above the ground water level (kN/m^3)
+ γ_{sat}: the saturated desnsity of the soil layer under the ground wate (kN/m^3)

$$\gamma_{sat} = \gamma_d(1- \gamma_w/\gamma_s) + \gamma_w$$

+ γ_d : Dry density (kN/m^3)
+ γ_s : Unit weight (kN/m^3)
+ γ_w : water density γ_w = 10 (kN/m^3)
+ With: Energy efficiency E_h = 50

Name of layer	Layer thickness	Elevation of the layer bottom	depth	Unit weight	Dry density	saturated density	Total stress	Pore water pressure	Effective stress
	li		z_i	γ_s	γ_d	γ_{sat}	σ_{vz}	u	σ'_z
	(m)	(m)	(m)	(kN/m^3)	(kN/m^3)	(kN/m^3)	(kN/m^2)	(kN/m^2)	(kN/m^2)
		-2.11							
2	1.50	-3.61	0.75	0.00	0.00	0.00	0.00	0.00	0.00
2	8.01	-11.61	5.50	26.46	14.40	18.96	104.32	55.03	49.29
2	6.15	-17.76	12.58	26.56	14.40	18.98	238.75	125.80	112.95
3	8.10	-25.86	19.71	26.56	11.74	17.32	341.29	197.05	144.24
4	13.70	-39.56	30.61	26.52	14.50	19.03	582.49	306.05	276.44
6a	6.20	-45.76	40.56	26.91	13.12	18.24	739.91	405.55	334.36
7a	8.40	-54.16	47.86	26.57	14.50	19.04	911.29	478.55	432.74
8	4.10	-58.26	54.11	26.91	18.37	21.54	1165.61	541.05	624.56
9	5.85	-64.11	59.08	26.73	19.01	21.90	1293.69	590.78	702.91
	1.50	-65.61							

Name of layer	Soil type	σ'_z (N/mm^2)	SPT N (Blow/30cm)	N_{60} = N*E_h/60	S_u (N/mm^2)	α	β	q_s (N/mm^2)	Resistance factor φs	value $\varphi_s l_i q_s$ (N/mm)
2	1	0.000	0	-	-	-			0.55	0.00
2	1	0.049	8	6	-	-	0.464	0.023	0.55	100.78
2	1	0.113	10	9	-	-	0.438	0.050	0.55	167.48
3	2	0.144	6	5	0.031	0.55	0.250	0.017	0.45	62.65
4	1	0.276	19	15	-	-	0.250	0.069	0.55	520.74
6a	2	0.334	7	6	0.035	0.55	0.250	0.019	0.45	53.71
7a	1	0.433	26	22	-	-	0.250	0.108	0.55	499.81
8	2	0.625	68	57	0.340	0.55	0.250	0.187	0.45	345.02
9	2	0.703	85	71	0.425	0.55	0.250	0.234	0.45	614.82
	Total									2365.01

Total resistance of the side wall: $Q_s = \varphi_s \phi_i A_s = \Sigma \varphi_s \phi_i l_i P$ = **11144.8** (Kn)

φ_s: Shaft resistance factor of the pile in the ground (according to the table 10.5.5.2.4-1 AASHTO 2007)

2.2. PILE TIP RESISTANCE q_p:

Soil layer at Tip of pile: 2 Clay layer

2.2.1. Pile tip resistance: Method Reese and O'Neill (1999)

- Nominal Tip resistance follow:

q_p = 0.057N with N $\leq$ 50

q_p = 0.59σ'_v((N60(Pa/σ'_v))^0.8 with N > 50

In which

N = 85 (hammer /300mm)-Number of hammer SPT-N has not been calibrated

D = 1500 (mm) Diameter of bore pile

D_p = 1500 (mm) diameter of bore pipe

σ'_v = 0.70 (Mpa) effective vertical prestress

=> Nominal tip resistance q_p = **3.071** (Mpa)

- As for the bottom diameter bigger than 1270mm, q_p has to reduce as follows:

q_{pr} = 1270*q_p/D_p = **2.600** (Mpa)

- Tip resistance of pile:

$Q_b = \phi * q_p * A_b =$ 1837.68 (kN)

With tip resistance factor : $\phi =$ **0.4** (Table 10.5.5.2.4-1 AASHTO2007)

2.3. Pile capacity:

- Design resistance of the single pile :

$Q_T = (Q_s + Q_P) =$ 13434.0 kN

- Resistance of the resisting pile according to the pile group:

$Q_R = \eta . Q_T =$ **10299.41** kN

In which

+ $\eta =$ 0.77 Effective factor due to the pile working by group (AASHTO 2007 - 10.8.3.6.3)

+ d = 4.5 (m) The distance from the center to center of pile.

2.4. CONCLUSION

- Final pile capacity: $Q_R =$ 10299.41 kN
- Seftweight of pile: W = 1588.66 kN
- Maximum internal force applied to pile: $P_{max} =$ 8022.62 kN
- Check pile capacity : $P_{max} + W \leq Q_R$

⇔ 9611.28 < 10299.41 **OK**

3. THE SETTLEMENT OF PILE GROUP:

- The Settlement of pile groups in cohesionless soils using SPT value:

$$\rho = \frac{30qI\sqrt{B}}{N_{60}} \quad \text{(mm)} \quad (10.7.2.3.3\text{-}1)$$

In which: - q: net foundation pressure applid at $2D_b/3$, this pressure is equal to the applied load at the top of group divided by the area of the equivalent footing and does not include the weight of the piles or soil between the piers (Mpa).

- B : Width or smallest dimensions of pile group (mm).
- I : Influence factor of the effective group embedment (dim)

I = 1 - 0,125D'/B

- D' : Effective depth taken as $2D_b/3$, (mm)
- D_b : depth of embedment of piles in layer that provides support, (mm)

Nmax	Equivalent area	B	D_b	D'	I	N_{60}	q	ρ	Check
(Kn)	(mm^2)	(mm)	(mm)	(mm)	(dim)	blow	(Mpa)	(mm)	$\rho < 25.4$
129940	4.42E+08	11500	9945	6630.00	0.93	71	0.29	12.39	Ok

PILE CAPACITY AND SETTLEMENT OF GROUP PILE

1. DATA

- Code of boreholes		P12	
- Pile diameter :		1500	mm
- Elevation of the underground water	EL_3 =	1.00	m
- Ground elevation after scour	EL_4 =	-4.400	m
- Base bottom elevation	EL_1 =	-3.215	m
- Expected pipe tip elevation	EL_2 =	-67.215	m
- Pipe length	L =	64.0	m
- Perimeter of cross section	P =	4.71	m
- Area of the pile cross section	A_b =	1.77	m2
- Concrete weight	γ_c =	24.50	kN/m3
Soil layer at Tip of pile:		2	

('1' =cohesionless soil, '2' = cohesive soil, '3' = intermediate material , '4' = rock)

* Reference document

+ Standard 22 TCN 272-05
+ Standard AASHTO LRFD 2007

Legends

* Because the pile foundation of the bridge crosses the soil area with strong weather rock and IGM, so 22TCN272-05 can not meet requirement to base for the calculation of the pipe load bearing capacity, then AASHTO 2007 is referred to calculate the process items which standard 22TCN272-05 has not yet mentioned

2. CALCULATION FOR THE BEARING FORCE OF BORED PILES

2.1. SIDEWALL FRICTION q_s:

2.1.1. For cohesionless soil

+ Using Method β : O'Neil and Reese (1999)

$q_s = \beta\sigma'_v \leq 0.19$MPa; with $0.25 < \beta < 1.2$

+ For sandy soil:

$\beta = 1.5 - 7.7\text{x}10^{-3}(z)^{0.5}$ for $N_{60} \geq 15$

$\beta = N_{60}/15[1.5 - 7.7\text{x}10^{-3}(z)^{0.5}]$ for $N_{60} < 15$

+ For Gravelly sand and gravels and SPT values are more than 50 blows/30cm:

$\beta = 2\text{-}0.00082\ (z)^{0.5}$ with $0.25 < \beta < 1.8$

With : + z : The depth of the subsoil layer at the middle location of the soil layer

+ N : The average number of hammers SPT (not yet calibrated) along the pile body (hammer/300)

2.1.2. As for conhesive soil, side wall friction is calculated according to the method α

$q_s = \alpha S_u$ (10.8.3.3.1-1)

In which:

+ The cohesive factor a is calculated according to the method API (10.7.3.3.2a-1)
+ $S_{u\,:}$ medium non- drain shear resistance strengthen

* Calculation method S_u**:**

$S_u = 0.06*N_{60}$ (bar) $= 0.006*N_{60}$ (MPa) (Terzaghi & Peck)

* Calculation method a**:** (10.8.3.5.1b AASHTO LRFD 2007)

$\alpha = 0.55$ with $S_u/0.101 \leq 1.5$

$\alpha = 0.55 - 0.1(S_u/0.101 - 1.5)$ with $1.5 \leq S_u/0.101 \leq 2.5$

*** Calculation method of effective vertical stress s'$_v$ at the center of each soil layer:**

$\sigma'_z = \sigma_{vz} - u_z$

At the soil layer over the underground water level : $\sigma_v = \gamma.z_i$; $u_z = 0$

At the soil layer under the ground water level : $\sigma_v = \gamma_{sat}.z_i$; $u_z = z_i$

Suppose: clay layer and water- penetration weathered rock layer

Soil layer placed under the ground water level and obtain the penetration, use the saturated weight

In which

+ σ_v : total stress at the depth z (kN/m^2)
+ σ'_v : effective stress at the depth z (kN/m^2)
+ u : Pore water pressure (kN/m^2)
+ z_i : Depth to the middle point of the i layer counted from the surfac (m)
+ l_i : thickness of the i soil layer (m)
+ γ : Natural density of the soil layer above the ground water level (kN/m^3)

+ γ_{sat}: the saturated desnsity of the soil layer under the ground wate (kN/m^3)

$$\gamma_{sat} = \gamma_d(1 - \gamma_w/\gamma_s) + \gamma_w$$

+ γ_d : Dry density (kN/m^3)

+ γ_s : Unit weight (kN/m^3)

+ γ_w : water density γ_w = 10 (kN/m^3)

+ With: Energy efficiency E_h = 50

Name of layer	Layer thickness	Elevation of the layer bottom	depth	Unit weight	Dry density	saturated density	Total stress	Pore water pressure	Effective stress
	li		z_i	γ_s	γ_d	γ_{sat}	σ_{vz}	u	σ'_z
	(m)	(m)	(m)	(kN/m^3)	(kN/m^3)	(kN/m^3)	(kN/m^2)	(kN/m^2)	(kN/m^2)
		-3.22							
2	1.50	-4.72	0.75	0.00	0.00	0.00	0.00	0.00	0.00
2	10.53	-15.24	6.76	26.46	14.40	18.96	128.20	67.63	60.58
3	3.00	-18.24	13.53	26.56	14.40	18.98	256.68	135.25	121.43
4	5.50	-23.74	17.78	26.52	14.40	18.97	337.19	177.75	159.44
tk3	8.00	-31.74	24.53	26.91	11.57	17.27	423.56	245.25	178.31
4	9.20	-40.94	33.13	26.52	14.50	19.03	630.45	331.25	299.20
6a	6.40	-47.34	40.93	26.91	13.12	18.24	746.66	409.25	337.41
7a	13.90	-61.24	51.08	26.60	14.50	19.05	972.92	510.75	462.17
8	1.80	-63.04	58.93	26.91	18.37	21.54	1269.45	589.25	680.20
9	4.18	-67.22	61.91	26.73	19.01	21.90	1355.77	619.13	736.64
	1.50	-68.72							

Name of layer	Soil type	σ'_z (N/mm^2)	SPT N (Blow/30cm)	$N_{60} = N*E_h/60$	S_u (N/mm^2)	α	β	q_s (N/mm^2)	Resistance factor φs	value $\varphi_s l_i q_s$ (N/mm)
2	1	0.000	0	-	-	-			0.55	0.00
2	1	0.061	10	9	-	-	0.597	0.036	0.55	209.39
3	2	0.121	5	4	0.025	0.55	0.250	0.014	0.45	18.56
4	1	0.159	8	6	-	-	0.250	0.040	0.55	120.58
tk3	2	0.178	12	10	0.059	0.55	0.250	0.032	0.45	116.33
4	1	0.299	19	15	-	-	0.250	0.075	0.55	378.49
6a	2	0.337	5	4	0.025	0.55	0.250	0.014	0.45	39.60
7a	1	0.462	43	36	-	-	0.250	0.116	0.55	883.32
8	2	0.680	85	71	0.425	0.55	0.250	0.234	0.45	189.34
9	2	0.737	85	71	0.425	0.55	0.250	0.234	0.45	439.16
	Total									2394.77

Total resistance of the side wall: $Q_s = \varphi_s \phi_i A_s = \Sigma \varphi_s \phi_i l_i P$ = **11285.1** (Kn)

φ_s: Shaft resistance factor of the pile in the ground (according to the table 10.5.5.2.4-1 AASHTO 2007)

2.2. PILE TIP RESISTANCE q_p:

Soil layer at Tip of pile: 2 Clay layer

2.2.1. Pile tip resistance: Method Reese and O'Neill (1999)

- Nominal Tip resistance follow:

$q_p = 0.057N$ with $N \le 50$

$q_p = 0.59\sigma'_v((N60(Pa/\sigma'_v))$^0.8 with N > 50

In which

N = 85 (hammer /300mm)-Number of hammer SPT-N has not been calibrated

D = 1500 (mm) Diameter of bore pile

D_p = 1500 (mm) diameter of bore pipe

σ'_v = 0.74 (Mpa) effective vertical prestress

=> Nominal tip resistance q_p = **3.100** (Mpa)

- As for the bottom diameter bigger than 1270mm, q_p has to reduce as follows:

$q_{pr} = 1270*q_p/D_p$ = **2.624** (Mpa)

- Tip resistance of pile:

$Q_b = \phi * q_p * A_b =$ 1854.99 (kN)

With tip resistance factor : $\phi =$ **0.4** (Table 10.5.5.2.4-1 AASHTO2007)

2.3. Pile capacity:

- Design resistance of the single pile :

$Q_T = (Q_s + Q_P) =$ 13574.2 kN

- Resistance of the resisting pile according to the pile group:

$Q_R = \eta . Q_T =$ **10406.91** kN

In which

+ $\eta =$ 0.77 Effective factor due to the pile working by group (AASHTO 2007 - 10.8.3.6.3)

+ d = 4.5 (m) The distance from the center to center of pile.

2.4. CONCLUSION

- Final pile capacity: $Q_R =$ 10406.91 kN
- Seftweight of pile: W = 1639.91 kN
- Maximum internal force applied to pile: $P_{max} =$ 8369 kN
- Check pile capacity : $P_{max} + W \leq Q_R$

$\Leftrightarrow$ 10008.91 < 10406.91 **OK**

3. THE SETTLEMENT OF PILE GROUP:

- The Settlement of pile groups in cohesionless soils using SPT value:

$$\rho = \frac{30qI\sqrt{B}}{N_{60}} \quad \text{(mm)} \quad (10.7.2.3.3\text{-}1)$$

In which: - q: net foundation pressure applid at $2D_b/3$, this pressure is equal to the applied load at the top of group divided by the area of the equivalent footing and does not include the weight of the piles or soil between the piers (Mpa).

- B : Width or smallest dimensions of pile group (mm).
- I : Influence factor of the effective group embedment (dim)

I = 1 - 0,125D'/B

- D' : Effective depth taken as $2D_b/3$, (mm)
- D_b : depth of embedment of piles in layer that provides support, (mm)

Nmax	Equivalent area	B	D_b	D'	I	N_{60}	q	ρ	Check
(Kn)	(mm^2)	(mm)	(mm)	(mm)	(dim)	blow	(Mpa)	(mm)	ρ <25.4
133412	3.77E+08	11500	5975	3983.33	0.96	71	0.35	15.39	Ok

PILE CAPACITY AND SETTLEMENT OF GROUP PILE

1. DATA

- Code of boreholes		**P13**	
- Pile diameter :		1500	mm
- Elevation of the underground water	EL_3 =	1.00	m
- Ground elevation after scour	EL_4 =	-5.970	m
- Base bottom elevation	EL_1 =	-4.045	m
- Expected pipe tip elevation	EL_2 =	-64.045	m
- Pipe length	L =	60.0	m
- Perimeter of cross section	P =	4.71	m
- Area of the pile cross section	A_b =	1.77	m2
- Concrete weight	γ_c =	24.50	kN/m3
Soil layer at Tip of pile:		3	

('1' =cohesionless soil, '2' = cohesive soil, '3' = intermediate material , '4' = rock)

* Reference document

+ Standard 22 TCN 272-05

+ Standard AASHTO LRFD 2007

Legends

* Because the pile foundation of the bridge crosses the soil area with strong weather rock and IGM, so 22TCN272-05 can not meet requirement to base for the calculation of the pipe load bearing capacity, then AASHTO 2007 is referred to calculate the process items which standard 22TCN272-05 has not yet mentioned

2. CALCULATION FOR THE BEARING FORCE OF BORED PILES

2.1. SIDEWALL FRICTION q_s:

2.1.1. For cohesionless soil

+ Using Method β : O'Neil and Reese (1999)

$q_s = \beta\sigma'_v \le 0.19$MPa; with $0.25 < \beta < 1.2$

+ For sandy soil:

$\beta = 1.5 - 7.7\text{x}10^{-3}(z)^{0.5}$ for $N_{60} \ge 15$

$\beta = N_{60}/15[1.5 - 7.7\text{x}10^{-3}(z)^{0.5}]$ for $N_{60} < 15$

+ For Gravelly sand and gravels and SPT values are more than 50 blows/30cm:

$\beta = 2-0.00082\ (z)^{0.5}$ with $0.25 < \beta < 1.8$

With : + z : The depth of the subsoil layer at the middle location of the soil layer

+ N : The average number of hammers SPT (not yet calibrated) along the pile body (hammer/300)

2.1.2. As for conhesive soil, side wall friction is calculated according to the method α

$q_s = \alpha S_u$ (10.8.3.3.1-1)

In which:

+ The cohesive factor a is calculated according to the method API (10.7.3.3.2a-1)

+ S_u : medium non- drain shear resistance strengthen

* Calculation method **S_u:**

$S_u = 0.06{*}N_{60}$ (bar) $= 0.006{*}N_{60}$ (MPa) (Terzaghi & Peck)

* Calculation method **a:** (10.8.3.5.1b AASHTO LRFD 2007)

$\alpha = 0.55$ with $S_u/0.101 \le 1.5$

$\alpha = 0.55 - 0.1(S_u/0.101 - 1.5)$ with $1.5 \le S_u/0.101 \le 2.5$

*** Calculation method of effective vertical stress s'$_v$ at the center of each soil layer:**

$\sigma'_z = \sigma_{vz} - u_z$

At the soil layer over the underground water level : $\sigma_v = \gamma.z_i$; $u_z = 0$

At the soil layer under the ground water level : $\sigma_v = \gamma_{sat}.z_i$; $u_z = z_i$

Suppose: clay layer and water- penetration weathered rock layer

Soil layer placed under the ground water level and obtain the penetration, use the saturated weight

In which

+ σ_v : total stress at the depth z (kN/m^2)

+ σ'_v : effective stress at the depth z (kN/m^2)

+ u : Pore water pressure (kN/m^2)

+ z_i : Depth to the middle point of the i layer counted from the surfac (m)

+ l_i : thickness of the i soil layer (m)

+ γ : Natural density of the soil layer above the ground water level (kN/m^3)

+ γ_{sat}: the saturated desnsity of the soil layer under the ground wate (kN/m^3)

$\gamma_{sat} = \gamma_d(1- \gamma_w/\gamma_s) + \gamma_w$

+ γ_d : Dry density (kN/m^3)

+ γ_s : Unit weight (kN/m^3)

+ γ_w : water density γ_w = 10 (kN/m^3)

+ With: Energy efficiency E_h = 50

Name of layer	Layer thickness	Elevation of the layer bottom	depth	Unit weight	Dry density	saturated density	Total stress	Pore water pressure	Effective stress
	li		z_i	γ_s	γ_d	γ_{sat}	σ_{vz}	u	σ'_z
	(m)	(m)	(m)	(kN/m^3)	(kN/m^3)	(kN/m^3)	(kN/m^2)	(kN/m^2)	(kN/m^2)
		-4.05							
2	1.93	-5.97	0.96	0.00	0.00	0.00	0.00	0.00	0.00
2	8.37	-14.34	6.11	26.46	14.40	18.96	115.83	61.10	54.73
3	5.50	-19.84	13.05	26.56	14.40	18.98	247.57	130.45	117.12
4	10.40	-30.24	21.00	26.52	14.40	18.97	398.28	209.95	188.33
tk3	3.70	-33.94	28.05	26.91	11.57	17.27	484.35	280.45	203.90
4	8.60	-42.54	34.20	26.52	14.50	19.03	650.81	341.95	308.86
6a	2.00	-44.54	39.50	26.91	13.12	18.24	720.57	394.95	325.62
7a	9.50	-54.04	45.25	26.57	14.50	19.04	861.59	452.45	409.14
8	1.60	-55.64	50.80	26.91	18.37	21.54	1094.30	507.95	586.35
10	8.41	-64.05	55.80	26.73	19.01	21.90	1221.86	557.98	663.89
	1.50	-65.55							

Name of layer	Soil type	σ'_z (N/mm^2)	SPT N (Blow/30cm)	$N_{60} = N^*E_h/60$	S_u (N/mm^2)	α	β	q_s (N/mm^2)	Resistance factor φs	value $\varphi_s l_i q_s$ (N/mm)
2	1	0.000	0	-	-	-			0.55	0.00
2	1	0.055	8	6	-	-	0.462	0.025	0.55	116.38
3	2	0.117	5	4	0.027	0.55	0.250	0.015	0.45	36.30
4	1	0.188	17	14	-	-	0.384	0.072	0.55	413.98
tk3	2	0.204	12	10	0.058	0.55	0.250	0.032	0.45	52.66
4	1	0.309	20	16	-	-	0.250	0.077	0.55	365.23
6a	2	0.326	6	5	0.030	0.55	0.250	0.017	0.45	14.85
7a	1	0.409	61	51	-	-	0.250	0.102	0.55	534.44
8	2	0.586	44	37	0.220	0.55	0.250	0.121	0.45	87.12
10	3	0.664	75	63	-	-	0.250	0.166	0.60	836.99
	Total									2457.94

Total resistance of the side wall: $Q_s = \varphi_s \phi_i A_s = \Sigma\varphi_s\phi_i l_i P$ = **11582.8** (Kn)

φ_s: Shaft resistance factor of the pile in the ground (according to the table 10.5.5.2.4-1 AASHTO 2007)

2.2. PILE TIP RESISTANCE q_p:

Soil layer at Tip of pile: 3 IGM

2.2.1. Pile tip resistance: Method Reese and O'Neill (1999)

- Nominal Tip resistance follow:

$q_p = 0.057N$ with $N \leq 50$

$q_p = 0.59\sigma'_v((N60(Pa/\sigma'_v))$^0.8 with N > 50

In which

N = 75 (hammer /300mm)-Number of hammer SPT-N has not been calibrated

D = 1500 (mm) Diameter of bore pile

D_p = 1500 (mm) diameter of bore pipe

σ'_v = 0.66 (Mpa) effective vertical prestress

=> Nominal tip resistance q_p = **2.746** (Mpa)

- As for the bottom diameter bigger than 1270mm, q_p has to reduce as follows:

$q_{pr} = 1270^*q_p/D_p$ = **2.325** (Mpa)

- Tip resistance of pile:

$Q_b = \phi * q_p * A_b =$ 2260.08 (kN)

With tip resistance factor : $\phi =$ **0.55** (Table 10.5.5.2.4-1 AASHTO2007)

2.3. Pile capacity:

- Design resistance of the single pile :

$Q_T = (Q_s + Q_P) =$ 13842.9 kN

- Resistance of the resisting pile according to the pile group:

$Q_R = \eta . Q_T =$ **10612.87** kN

In which

+ η = 0.77 Effective factor due to the pile working by group (AASHTO 2007 - 10.8.3.6.3)

+ d = 4.5 (m) The distance from the center to center of pile.

2.4. CONCLUSION

- Final pile capacity: Q_R = 10612.87 kN
- Seftweight of pile: W = 1537.42 kN
- Maximum internal force applied to pile: P_{max} = 8774.57 kN
- Check pile capacity : $P_{max} + W \leq Q_R$

⇔ 10311.99 < 10612.87 **OK**

3. THE SETTLEMENT OF PILE GROUP:

- The Settlement of pile groups in cohesionless soils using SPT value:

$$\rho = \frac{30qI\sqrt{B}}{N_{60}} \quad \text{(mm)} \quad (10.7.2.3.3\text{-}1)$$

In which: - q: net foundation pressure applid at $2D_b/3$, this pressure is equal to the applied load at the top of group divided by the area of the equivalent footing and does not include the weight of the piles or soil between the piers (Mpa).

- B : Width or smallest dimensions of pile group (mm).
- I : Influence factor of the effective group embedment (dim)

I = 1 - 0,125D'/B

- D' : Effective depth taken as $2D_b/3$, (mm)
- D_b : depth of embedment of piles in layer that provides support, (mm)

Nmax	Equivalent area	B	D_b	D'	I	N_{60}	q	ρ	Check
(Kn)	(mm^2)	(mm)	(mm)	(mm)	(dim)	blow	(Mpa)	(mm)	ρ <25.4
134101	5.73E+08	16000	10005	6670.00	0.95	63	0.23	13.48	Ok

PILE CAPACITY AND SETTLEMENT OF GROUP PILE

1. DATA

- Code of boreholes		P14	
- Pile diameter :		1500	mm
- Elevation of the underground water	EL_3 =	1.00	m
- Ground elevation after scour	EL_4 =	-8.560	m
- Base bottom elevation	EL_1 =	-5.895	m
- Expected pipe tip elevation	EL_2 =	-61.895	m
- Pipe length	L =	56.0	m
- Perimeter of cross section	P =	4.71	m
- Area of the pile cross section	A_b =	1.77	m2
- Concrete weight	γ_c =	24.50	kN/m3
Soil layer at Tip of pile:		2	

('1' =cohesionless soil, '2' = cohesive soil, '3' = intermediate material , '4' = rock)

* Reference document

+ Standard 22 TCN 272-05
+ Standard AASHTO LRFD 2007

Legends

* Because the pile foundation of the bridge crosses the soil area with strong weather rock and IGM, so 22TCN272-05 can not meet requirement to base for the calculation of the pipe load bearing capacity, then AASHTO 2007 is referred to calculate the process items which standard 22TCN272-05 has not yet mentioned

2. CALCULATION FOR THE BEARING FORCE OF BORED PILES

2.1. SIDEWALL FRICTION q_s:

2.1.1. For cohesionless soil

+ Using Method β : O'Neil and Reese (1999)

$q_s = \beta\sigma'_v \le 0.19$MPa; with $0.25 < \beta < 1.2$

+ For sandy soil:

$\beta = 1.5 - 7.7\text{x}10^{-3}(z)^{0.5}$ for $N_{60} \ge 15$

$\beta = N_{60}/15[1.5 - 7.7\text{x}10^{-3}(z)^{0.5}]$ for $N_{60} < 15$

+ For Gravelly sand and gravels and SPT values are more than 50 blows/30cm:

$\beta = 2-0.00082\ (z)^{0.5}$ with $0.25 < \beta < 1.8$

With : + z : The depth of the subsoil layer at the middle location of the soil layer

+ N : The average number of hammers SPT (not yet calibrated) along the pile body (hammer/300)

2.1.2. As for conhesive soil, side wall friction is calculated according to the method α

$q_s = \alpha S_u$ (10.8.3.3.1-1)

In which:

+ The cohesive factor a is calculated according to the method API (10.7.3.3.2a-1)
+ $S_{u:}$ medium non- drain shear resistance strengthen

* Calculation method S_u:

$S_u = 0.06{*}N_{60}$ (bar) = $0.006{*}N_{60}$ (MPa) (Terzaghi & Peck)

* Calculation method a: (10.8.3.5.1b AASHTO LRFD 2007)

$\alpha = 0.55$ with $S_u/0.101 \le 1.5$

$\alpha = 0.55 - 0.1(S_u/0.101 - 1.5)$ with $1.5 \le S_u/0.101 \le 2.5$

*** Calculation method of effective vertical stress s'$_v$ at the center of each soil layer:**

$\sigma'_z = \sigma_{vz} - u_z$

At the soil layer over the underground water level : $\sigma_v = \gamma.z_i$; $u_z = 0$

At the soil layer under the ground water level : $\sigma_v = \gamma_{sat}.z_i$; $u_z = z_i$

Suppose: clay layer and water- penetration weathered rock layer

Soil layer placed under the ground water level and obtain the penetration, use the saturated weight

In which

+ σ_v : total stress at the depth z (kN/m^2)
+ σ'_v : effective stress at the depth z (kN/m^2)
+ u : Pore water pressure (kN/m^2)
+ z_i : Depth to the middle point of the i layer counted from the surfac (m)
+ l_i : thickness of the i soil layer (m)
+ γ : Natural density of the soil layer above the ground water level (kN/m^3)

+ γ_{sat}: the saturated desnsity of the soil layer under the ground wate (kN/m^3)

$$\gamma_{sat} = \gamma_d(1-\gamma_w/\gamma_s)+\gamma_w$$

+ γ_d : Dry density (kN/m^3)

+ γ_s : Unit weight (kN/m^3)

+ γ_w : water density $\gamma_w =$ 10 (kN/m^3)

+ With: Energy efficiency E_h = 50

Name of layer	Layer thickness	Elevation of the layer bottom	depth	Unit weight	Dry density	saturated density	Total stress	Pore water pressure	Effective stress
	li		z_i	γ_s	γ_d	γ_{sat}	σ_{vz}	u	σ'_z
	(m)	(m)	(m)	(kN/m^3)	(kN/m^3)	(kN/m^3)	(kN/m^2)	(kN/m^2)	(kN/m^2)
		-5.90							
2	2.67	-8.56	1.33	0.00	0.00	0.00	0.00	0.00	0.00
2	4.25	-12.81	4.79	26.46	14.40	18.96	90.81	47.90	42.91
3	5.10	-17.91	9.47	26.56	11.74	17.32	163.93	94.65	69.28
4	4.00	-21.91	14.02	26.52	14.40	18.97	265.87	140.15	125.72
4	14.00	-35.91	23.02	26.91	14.40	19.05	438.41	230.15	208.26
4	4.60	-40.51	32.32	26.52	14.50	19.03	615.03	323.15	291.88
7a	2.00	-42.51	35.62	26.91	14.50	19.11	680.66	356.15	324.51
7a	3.80	-46.31	38.52	26.57	14.50	19.04	733.43	385.15	348.28
8	9.69	-56.00	45.26	26.91	18.37	21.54	975.06	452.60	522.46
8	5.90	-61.90	53.05	26.91	18.37	21.54	1142.94	530.53	612.41
	1.50	-63.40							

Name of layer	Soil type	σ'_z (N/mm^2)	SPT N (Blow/30cm)	$N_{60} = N*E_h/60$	S_u (N/mm^2)	α	β	q_s (N/mm^2)	Resistance factor φs	value $\varphi_s l_i q_s$ (N/mm)
2	1	0.000	0	-	-	-			0.55	0.00
2	1	0.043	8	6	-	-	0.494	0.021	0.55	49.58
3	2	0.069	5	4	0.025	0.55	0.250	0.014	0.45	31.56
4	1	0.126	21	17	-	-	0.588	0.074	0.55	162.75
4	1	0.208	19	16	-	-	0.332	0.069	0.55	532.16
4	1	0.292	19	16	-	-	0.250	0.073	0.55	184.62
7a	1	0.325	75	63	-	-	0.250	0.081	0.55	89.24
7a	1	0.348	75	63	-	-	0.250	0.087	0.55	181.98
8	2	0.522	85	71	0.425	0.55	0.250	0.215	0.45	797.07
8	2	0.612	85	71	0.425	0.55	0.250	0.215	0.45	513.43
	Total									2542.37

Total resistance of the side wall: $Q_s = \varphi_s \phi_i A_s = \Sigma\varphi_s\phi_i l_i P =$ **11980.6** (Kn)

φ_s: Shaft resistance factor of the pile in the ground (according to the table 10.5.5.2.4-1 AASHTO 2007)

2.2. PILE TIP RESISTANCE q_p:

Soil layer at Tip of pile: 2 Clay layer

2.2.1. Pile resistance in the clay soil

Fomular: $q_p = NcSu <= 4$ (CT: 10.8.3.3.2-1)

$q_p = NcSu$ = 3.83

In which Nc = 6[1+0.2(Z/D)] <= 9

Nc = 6[1+0.2(Z/D)] = = 50.8

Reduce:

If Su <0.024 Mpa Nc should be multiplied by 0,67

In which

D = 1500 (mm) - Diameter of pile

Z = 56000 (mm) - Penetration of shaft

Su = 0.425 (Mpa)

Nc = 9

Vậy q_p **= 3.825** (Mpa)

As for the bottom diameter bigger than 1270mm, qp has to reduce as follows:

$q_{pr} = 1270*q_p/D_p$ **= 3.239** (Mpa)

- Tip resistance of pile:

$Q_b = \phi * q_p * A_b =$ 2289.16 (kN)

Pile tip resistance factor: **0.4** (Table 10.5.5.2.4-1 AASHTO2007)

2.3. Pile capacity:

- Design resistance of the single pile :

$Q_T = (Q_s + Q_P) =$ 14269.8 kN

- Resistance of the resisting pile according to the pile group:

$Q_R = \eta . Q_T =$ **10940.17** kN

In which

+ η = 0.77 Effective factor due to the pile working by group (AASHTO 2007 - 10.8.3.6.3)

+ d = 4.5 (m) The distance from the center to center of pile.

2.4. CONCLUSION

- Final pile capacity: Q_R = 10940.17 kN
- Seftweight of pile: W = 1434.92 kN
- Maximum internal force applied to pile: P_{max} = 8916.855 kN
- Check pile capacity : $P_{max} + W \leq Q_R$

$\Leftrightarrow$ 10351.78 < 10940.17 **OK**

3. THE SETTLEMENT OF PILE GROUP:

- The Settlement of pile groups in cohesionless soils using SPT value:

$$\rho = \frac{30qI\sqrt{B}}{N_{60}} \quad \text{(mm)} \quad (10.7.2.3.3\text{-}1)$$

In which: - q: net foundation pressure applid at $2D_b/3$, this pressure is equal to the applied load at the top of group divided by the area of the equivalent footing and does not include the weight of the piles or soil between the piers (Mpa).

- B : Width or smallest dimensions of pile group (mm).
- I : Influence factor of the effective group embedment (dim)

$I = 1 - 0,125D'/B$

- D' : Effective depth taken as $2D_b/3$, (mm)
- D_b : depth of embedment of piles in layer that provides support, (mm)

Nmax	Equivalent area	B	D_b	D'	I	N_{60}	q	ρ	Check
(Kn)	(mm^2)	(mm)	(mm)	(mm)	(dim)	blow	(Mpa)	(mm)	ρ <25.4
136504	6.82E+08	16000	15585	10390.00	0.92	71	0.20	9.85	Ok

PILE CAPACITY AND SETTLEMENT OF GROUP PILE

1. DATA

- Code of boreholes		**P15**	
- Pile diameter :		1500	mm
- Elevation of the underground water	EL_3 =	1.00	m
- Ground elevation after scour	EL_4 =	-12.150	m
- Base bottom elevation	EL_1 =	-10.265	m
- Expected pipe tip elevation	EL_2 =	-64.265	m
- Pipe length	L =	54.0	m
- Perimeter of cross section	P =	4.71	m
- Area of the pile cross section	A_b =	1.77	m2
- Concrete weight	γ_c =	24.50	kN/m3
Soil layer at Tip of pile:		2	

('1' =cohesionless soil, '2' = cohesive soil, '3' = intermediate material , '4' = rock)

* Reference document

+ Standard 22 TCN 272-05

+ Standard AASHTO LRFD 2007

Legends

* Because the pile foundation of the bridge crosses the soil area with strong weather rock and IGM, so 22TCN272-05 can not meet requirement to base for the calculation of the pipe load bearing capacity, then AASHTO 2007 is referred to calculate the process items which standard 22TCN272-05 has not yet mentioned

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2.1. SIDEWALL FRICTION q_s:

2.1.1. For cohesionless soil

+ Using Method β : O'Neil and Reese (1999)

$q_s = \beta\sigma'_v \le 0.19$MPa; with $0.25 < \beta < 1.2$

+ For sandy soil:

$\beta = 1.5 - 7.7\times10^{-3}(z)^{0.5}$ for $N_{60} \ge 15$

$\beta = N_{60}/15[1.5 - 7.7\times10^{-3}(z)^{0.5}]$ for $N_{60} < 15$

+ For Gravelly sand and gravels and SPT values are more than 50 blows/30cm:

$\beta = 2-0.00082\ (z)^{0.5}$ with $0.25 < \beta < 1.8$

With : + z : The depth of the subsoil layer at the middle location of the soil layer

+ N : The average number of hammers SPT (not yet calibrated) along the pile body (hammer/300)

2.1.2. As for conhesive soil, side wall friction is calculated according to the method α

$q_s = \alpha S_u$ (10.8.3.3.1-1)

In which:

+ The cohesive factor a is calculated according to the method API (10.7.3.3.2a-1)

+ S_u : medium non- drain shear resistance strengthen

* Calculation method S_u**:**

$S_u = 0.06*N_{60}$ (bar) = $0.006*N_{60}$ (MPa) (Terzaghi & Peck)

* Calculation method a**:** (10.8.3.5.1b AASHTO LRFD 2007)

$\alpha = 0.55$ with $S_u/0.101 \le 1.5$

$\alpha = 0.55 - 0.1(S_u/0.101 - 1.5)$ with $1.5 \le S_u/0.101 \le 2.5$

*** Calculation method of effective vertical stress s'$_v$ at the center of each soil layer:**

$\sigma'_z = \sigma_{vz} - u_z$

At the soil layer over the underground water level : $\sigma_v = \gamma.z_i$; $u_z = 0$

At the soil layer under the ground water level : $\sigma_v = \gamma_{sat}.z_i$; $u_z = z_i$

Suppose: clay layer and water- penetration weathered rock layer

Soil layer placed under the ground water level and obtain the penetration, use the saturated weight

In which

+ σ_v : total stress at the depth z	(kN/m^2)
+ σ'_v : effective stress at the depth z	(kN/m^2)
+ u : Pore water pressure	(kN/m^2)
+ z_i : Depth to the middle point of the i layer counted from the surfac	(m)
+ l_i : thickness of the i soil layer	(m)
+ γ : Natural density of the soil layer above the ground water level	(kN/m^3)

+ γ_{sat}: the saturated desnsity of the soil layer under the ground wate (kN/m^3)

$\gamma_{sat} = \gamma_d(1 - \gamma_w/\gamma_s) + \gamma_w$

+ γ_d : Dry density (kN/m^3)

+ γ_s : Unit weight (kN/m^3)

+ γ_w : water density γ_w = 10 (kN/m^3)

+ With: Energy efficiency E_h = 50

Name of layer	Layer thickness	Elevation of the layer bottom	depth	Unit weight	Dry density	saturated density	Total stress	Pore water pressure	Effective stress
	li		z_i	γ_s	γ_d	γ_{sat}	σ_{vz}	u	σ'_z
	(m)	(m)	(m)	(kN/m^3)	(kN/m^3)	(kN/m^3)	(kN/m^2)	(kN/m^2)	(kN/m^2)
		-10.27							
2	1.89	-12.15	0.94	0.00	0.00	0.00	0.00	0.00	0.00
2	0.36	-12.51	2.07	26.46	14.40	18.96	39.15	20.65	18.50
3	3.90	-16.41	4.20	26.56	11.48	17.16	71.98	41.95	30.03
4	5.50	-21.91	8.90	26.52	14.40	18.97	168.74	88.95	79.79
4	14.00	-35.91	18.65	26.91	14.40	19.05	355.17	186.45	168.72
4	9.70	-45.61	30.50	26.52	14.50	19.03	580.39	304.95	275.44
6b	3.90	-49.51	37.30	26.92	15.22	19.57	729.72	372.95	356.77
7a	1.90	-51.41	40.20	26.57	14.50	19.04	765.42	401.95	363.47
8	6.40	-57.81	44.35	26.91	18.37	21.54	955.35	443.45	511.90
9	6.46	-64.27	50.77	26.73	19.01	21.90	1111.82	507.73	604.10
	1.50	-65.77							

Name of layer	Soil type	σ'_z (N/mm^2)	SPT N (Blow/30cm)	N_{60} = N*E_h/60	S_u (N/mm^2)	α	β	q_s (N/mm^2)	Resistance factor φs	value $\varphi_s l_i q_s$ (N/mm)
2	1	0.000	0	-	-	-			0.55	0.00
2	1	0.018	6	5	-	-	0.460	0.009	0.55	1.68
3	2	0.030	5	4	0.025	0.55	0.250	0.014	0.45	24.13
4	1	0.080	10	9	-	-	0.533	0.043	0.55	128.66
4	1	0.169	22	18	-	-	0.449	0.076	0.55	582.77
4	1	0.275	26	22	-	-	0.250	0.069	0.55	367.37
6b	2	0.357	11	9	0.055	0.55	0.250	0.030	0.45	53.09
7a	1	0.363	43	36	-	-	0.250	0.091	0.55	94.96
8	2	0.512	85	71	0.425	0.55	0.250	0.215	0.45	526.44
9	2	0.604	85	71	0.425	0.55	0.250	0.215	0.45	530.97
	Total									2310.07

Total resistance of the side wall: $Q_s = \varphi_s \phi_i A_s = \Sigma \varphi_s \phi_i l_i P$ = **10886.0** (Kn)

φ_s: Shaft resistance factor of the pile in the ground (according to the table 10.5.5.2.4-1 AASHTO 2007)

2.2. PILE TIP RESISTANCE q_p:

Soil layer at Tip of pile: 2 Clay layer

2.2.1. Pile resistance in the clay soil

Fomular: **q_p = NcSu <=4** (10.8.3.3.2-1)

q_p = NcSu = 3.83

In which Nc = 6[1+0.2(Z/D)] <= 9

Nc = 6[1+0.2(Z/D)] = = 49.2

Reduce:

If Su <0.024 Mpa Nc should be multiplied by 0,67

In which

D = 1500 (mm) - Diameter of pile

Z = 54000 (mm) - Penetration of shaft

Su = 0.425 (Mpa)

Nc = 9

=> q_p **= 3.825** (Mpa)

As for the bottom diameter bigger than 1270mm, qp has to reduce as follows:

$q_{pr} = 1270*q_p/D_p$ **= 3.239** (Mpa)

Pile tip resistance

$Q_b = \phi * q_p * A_b =$ 2289.16 (kN)

With tip resistance factor : $\phi =$ **0.4** (Table 10.5.5.2.4-1 AASHTO2007)

2.3. Pile capacity:

- Design resistance of the single pile :

$Q_T = (Q_s + Q_P) =$ 13175.1 kN

- Resistance of the resisting pile according to the pile group:

$Q_R = \eta . Q_T =$ **10100.92** kN

In which

+ $\eta =$ 0.77 Effective factor due to the pile working by group (AASHTO 2007 - 10.8.3.6.3)

+ d = 4.5 (m) The distance from the center to center of pile.

2.4. CONCLUSION

- Final pile capacity: $Q_R =$ 10100.92 kN
- Seftweight of pile: W = 1383.68 kN
- Maximum internal force applied to pile: $P_{max} =$ 7828 kN
- Check pile capacity : $P_{max} + W \leq Q_R$

$\Leftrightarrow$ 9211.68 < 10100.92 **OK**

3. THE SETTLEMENT OF PILE GROUP:

- The Settlement of pile groups in cohesionless soils using SPT value:

$$\rho = \frac{30 q I \sqrt{B}}{N_{60}}$$ (mm) (10.7.2.3.3-1)

In which: - q: net foundation pressure applid at $2D_b/3$, this pressure is equal to the applied load at the top of group divided by the area of the equivalent footing and does not include the weight of the piles or soil between the piers (Mpa).

- B : Width or smallest dimensions of pile group (mm).
- I : Influence factor of the effective group embedment (dim)

$I = 1 - 0{,}125D'/B$

- D' : Effective depth taken as $2D_b/3$, (mm)
- D_b : depth of embedment of piles in layer that provides support, (mm)

Nmax	Equivalent area	B	D_b	D'	I	N_{60}	q	ρ	Check
(Kn)	(mm^2)	(mm)	(mm)	(mm)	(dim)	blow	(Mpa)	(mm)	ρ <25.4
137817	5.33E+08	16000	12855	8570.00	0.93	71	0.26	12.92	Ok

PILE CAPACITY AND SETTLEMENT OF GROUP PILE

1. DATA

- Code of boreholes		**P16**	
- Pile diameter :		1500	mm
- Elevation of the underground water	EL_3 =	1.00	m
- Ground elevation after scour	EL_4 =	1.050	m
- Base bottom elevation	EL_1 =	-0.355	m
- Expected pipe tip elevation	EL_2 =	-60.355	m
- Pipe length	L =	60.0	m
- Perimeter of cross section	P =	4.71	m
- Area of the pile cross section	A_b =	1.77	m2
- Concrete weight	γ_c =	24.50	kN/m3
Soil layer at Tip of pile:		2	

('1' =cohesionless soil, '2' = cohesive soil, '3' = intermediate material , '4' = rock)

* Reference document

+ Standard 22 TCN 272-05
+ Standard AASHTO LRFD 2007

Legends

* Because the pile foundation of the bridge crosses the soil area with strong weather rock and IGM, so 22TCN272-05 can not meet requirement to base for the calculation of the pipe load bearing capacity, then AASHTO 2007 is referred to calculate the process items which standard 22TCN272-05 has not yet mentioned

2. CALCULATION FOR THE BEARING FORCE OF BORED PILES

2.1. SIDEWALL FRICTION q_s:

2.1.1. For cohesionless soil

+ Using Method β : O'Neil and Reese (1999)

$$q_s = \beta\sigma'_v \leq 0.19MPa; \qquad \text{with } 0.25 < \beta < 1.2$$

+ For sandy soil:

$$\beta = 1.5 - 7.7x10^{-3}(z)^{0.5} \qquad \text{for } N_{60} \geq 15$$

$$\beta = N_{60}/15[1.5 - 7.7x10^{-3}(z)^{0.5}] \qquad \text{for } N_{60} < 15$$

+ For Gravelly sand and gravels and SPT values are more than 50 blows/30cm:

$$\beta = 2\text{-}0.00082\,(z)^{0.5} \qquad \text{with } 0.25 < \beta < 1.8$$

With : + z : The depth of the subsoil layer at the middle location of the soil layer

+ N : The average number of hammers SPT (not yet calibrated) along the pile body (hammer/300)

2.1.2. As for conhesive soil, side wall friction is calculated according to the method α

$$q_s = \alpha S_u \qquad (10.8.3.3.1\text{-}1)$$

In which:

+ The cohesive factor a is calculated according to the method API (10.7.3.3.2a-1)
+ S_u : medium non- drain shear resistance strengthen

* Calculation method **S_u:**

$$S_u = 0.06{*}N_{60}\,(bar) = 0.006{*}N_{60}\,(MPa) \qquad \text{(Terzaghi \& Peck)}$$

* Calculation method a**:** (10.8.3.5.1b AASHTO LRFD 2007)

$$\alpha = 0.55 \qquad \text{with } S_u/0.101 \leq 1.5$$

$$\alpha = 0.55 - 0.1(S_u/0.101 - 1.5) \qquad \text{with } 1.5 \leq S_u/0.101 \leq 2.5$$

*** Calculation method of effective vertical stress s'$_v$ at the center of each soil layer:**

$$\sigma'_z = \sigma_{vz} - u_z$$

At the soil layer over the underground water level : $\sigma_v = \gamma.z_i$; $u_z = 0$

At the soil layer under the ground water level : $\sigma_v = \gamma_{sat}.z_i$; $u_z = z_i$

Suppose: clay layer and water- penetration weathered rock layer

Soil layer placed under the ground water level and obtain the penetration, use the saturated weight

In which

+ σ_v : total stress at the depth z	(kN/m^2)
+ σ'_v : effective stress at the depth z	(kN/m^2)
+ u : Pore water pressure	(kN/m^2)
+ z_i : Depth to the middle point of the i layer counted from the surfac	(m)
+ l_i : thickness of the i soil layer	(m)
+ γ : Natural density of the soil layer above the ground water level	(kN/m^3)
+ γ_{sat}: the saturated desnsity of the soil layer under the ground wate	(kN/m^3)

$\gamma_{sat} = \gamma_d(1-\gamma_w/\gamma_s) + \gamma_w$

+ γ_d : Dry density (kN/m³)
+ γ_s : Unit weight (kN/m³)
+ γ_w : water density $\gamma_w =$ 10 (kN/m³)
+ With: Energy efficiency E_h = 50

Name of layer	Layer thickness	Elevation of the layer bottom	depth	Unit weight	Dry density	saturated density	Total stress	Pore water pressure	Effective stress
	li		z_i	γ_s	γ_d	γ_{sat}	σ_{vz}	u	σ'_z
	(m)	(m)	(m)	(kN/m³)	(kN/m³)	(kN/m³)	(kN/m²)	(kN/m²)	(kN/m²)
		-0.36							
2	1.50	-1.86	0.75	0.00	0.00	0.00	0.00	0.00	0.00
2	6.34	-8.19	4.67	26.46	14.40	18.96	88.49	46.68	41.81
tk1	5.20	-13.39	10.44	26.56	14.40	18.98	198.04	104.35	93.69
3	8.60	-21.99	17.34	26.56	11.74	17.32	300.24	173.35	126.89
4	23.00	-44.99	33.14	26.91	14.50	19.11	633.27	331.35	301.92
6b	2.00	-46.99	45.64	26.52	14.50	19.03	868.54	456.35	412.19
7a	8.49	-55.48	50.88	26.91	13.12	18.24	928.28	508.80	419.48
8	4.88	-60.36	57.56	26.91	18.37	21.54	1240.10	575.63	664.48
	1.50	-61.86							

Name of layer	Soil type	σ'_z (N/mm²)	SPT N (Blow/30cm)	N_{60} = N*E_h/60	S_u (N/mm²)	α	β	q_s (N/mm²)	Resistance factor φs	value $\varphi_s l_i q_s$ (N/mm)
2	1	0.000	0	-	-	-			0.55	0.00
2	1	0.042	19	16	-	-	0.974	0.041	0.55	141.88
tk1	1	0.094	41	34	-	-	0.713	0.067	0.55	191.16
3	2	0.127	6	5	0.030	0.55	0.250	0.017	0.45	63.86
4	1	0.302	41	35	-	-	0.250	0.075	0.55	954.81
6b	2	0.412	26	22	0.130	0.55	0.250	0.072	0.45	64.35
7a	1	0.419	75	63	-	-	0.250	0.105	0.55	489.69
8	2	0.664	85	71	0.425	0.55	0.250	0.234	0.45	512.79
	Total									2418.54

Total resistance of the side wall: $Q_s = \varphi_s \phi_i A_s = \Sigma \varphi_s \phi_i l_i P =$ **11397.1** (Kn)

φ_s: Shaft resistance factor of the pile in the ground (according to the table 10.5.5.2.4-1 AASHTO 2007)

2.2. PILE TIP RESISTANCE q_p:

Soil layer at Tip of pile: 2 Clay layer

2.2.1. Pile resistance in the clay soil

Fomular: **q_p = NcSu <=4** (CT: 10.8.3.3.2-1)

q_p = NcSu = 3.83

In which Nc = 6[1+0.2(Z/D)] <= 9

Nc = 6[1+0.2(Z/D)] = = 54

Reduce:

If Su <0.024 Mpa Nc should be multiplied by 0,67

In which

D = 1500 (mm) - Diameter of pile

Z = 60000 (mm) - Penetration of shaft

Su = 0.425 (Mpa)

Nc = 9

Vậy q_p **= 3.825** (Mpa)

As for the bottom diameter bigger than 1270mm, qp has to reduce as follows:

$q_{pr} = 1270*q_p/D_p$ **= 3.239** (Mpa)

- Tip resistance of pile:

$Q_b = \phi * q_p * A_b =$ 2289.16 (kN)

With tip resistance factor : $\phi =$ **0.4** (Table 10.5.5.2.4-1 AASHTO2007)

2.3. Pile capacity:

- Design resistance of the single pile :

$Q_T = (Q_s+Q_P) =$ 13686.2 kN

- Resistance of the resisting pile according to the pile group:

$Q_R = \eta.Q_T =$ **10492.79** kN

In which

+ η = 0.77 Effective factor due to the pile working by group (AASHTO 2007 - 10.8.3.6.3)

+ d = 4.5 (m) The distance from the center to center of pile.

2.4. CONCLUSION

- Final pile capacity: Q_R = 10492.79 kN
- Seftweight of pile: W = 1537.42 kN
- Maximum internal force applied to pile: P_{max} = 7889.31 kN
- Check pile capacity : $P_{max} + W \le Q_R$

$\Leftrightarrow$ 9426.73 < 10492.79 **OK**

3. THE SETTLEMENT OF PILE GROUP:

- The Settlement of pile groups in cohesionless soils using SPT value:

$$\rho = \frac{30qI\sqrt{B}}{N_{60}} \quad \text{(mm)} \quad (10.7.2.3.3\text{-}1)$$

In which: - q: net foundation pressure applid at $2D_b/3$, this pressure is equal to the applied load at the top of group divided by the area of the equivalent footing and does not include the weight of the piles or soil between the piers (Mpa).

- B : Width or smallest dimensions of pile group (mm).
- I : Influence factor of the effective group embedment (dim)

I = 1 - 0,125D'/B

- D' : Effective depth taken as $2D_b/3$, (mm)
- D_b : depth of embedment of piles in layer that provides support, (mm)

Nmax	Equivalent area	B	D_b	D'	I	N_{60}	q	ρ	Check
(Kn)	(mm^2)	(mm)	(mm)	(mm)	(dim)	blow	(Mpa)	(mm)	ρ <25.4
126237	5.02E+08	11500	13365	8910.00	0.90	71	0.25	10.32	Ok

PILE CAPACITY AND SETTLEMENT OF GROUP PILE

1. DATA

- Code of boreholes		**AB2**	
- Pile diameter :		1500	mm
- Elevation of the underground water	EL_3 =	1.00	m
- Ground elevation after scour	EL_4 =	-0.940	m
- Base bottom elevation	EL_1 =	-0.940	m
- Expected pipe tip elevation	EL_2 =	-62.940	m
- Pipe length	L =	62.0	m
- Perimeter of cross section	P =	4.71	m
- Area of the pile cross section	A_b =	1.77	m2
- Concrete weight	γ_c =	24.50	kN/m3
Soil layer at Tip of pile:		2	

('1' =cohesionless soil, '2' = cohesive soil, '3' = intermediate material , '4' = rock)

* Reference document

+ Standard 22 TCN 272-05
+ Standard AASHTO LRFD 2007

Legends

* Because the pile foundation of the bridge crosses the soil area with strong weather rock and IGM, so 22TCN272-05 can not meet requirement to base for the calculation of the pipe load bearing capacity, then AASHTO 2007 is referred to calculate the process items which standard 22TCN272-05 has not yet mentioned

2. CALCULATION FOR THE BEARING FORCE OF BORED PILES

2.1. SIDEWALL FRICTION q_s:

2.1.1. For cohesionless soil

+ Using Method β : O'Neil and Reese (1999)

$q_s = \beta\sigma'_v \leq 0.19$MPa; with $0.25 < \beta < 1.2$

+ For sandy soil:

$\beta = 1.5 - 7.7x10^{-3}(z)^{0.5}$ for $N_{60} \geq 15$

$\beta = N_{60}/15[1.5 - 7.7x10^{-3}(z)^{0.5}]$ for $N_{60} < 15$

+ For Gravelly sand and gravels and SPT values are more than 50 blows/30cm:

$\beta = 2-0.00082\ (z)^{0.5}$ with $0.25 < \beta < 1.8$

With : + z : The depth of the subsoil layer at the middle location of the soil layer

+ N : The average number of hammers SPT (not yet calibrated) along the pile body (hammer/300)

2.1.2. As for conhesive soil, side wall friction is calculated according to the method α

$q_s = \alpha S_u$ (10.8.3.3.1-1)

In which:

+ The cohesive factor a is calculated according to the method API (10.7.3.3.2a-1)
+ S_u : medium non- drain shear resistance strengthen

* Calculation method **S_u:**

$S_u = 0.06*N_{60}$ (bar) $= 0.006*N_{60}$ (MPa) (Terzaghi & Peck)

* Calculation method a**:** (10.8.3.5.1b AASHTO LRFD 2007)

$\alpha = 0.55$ with $S_u/0.101 \leq 1.5$

$\alpha = 0.55 - 0.1(S_u/0.101 - 1.5)$ with $1.5 \leq S_u/0.101 \leq 2.5$

*** Calculation method of effective vertical stress s'$_v$ at the center of each soil layer:**

$\sigma'_z = \sigma_{vz} - u_z$

At the soil layer over the underground water level : $\sigma_v = \gamma.z_i$; $u_z = 0$

At the soil layer under the ground water level : $\sigma_v = \gamma_{sat}.z_i$; $u_z = z_i$

Suppose: clay layer and water- penetration weathered rock layer

Soil layer placed under the ground water level and obtain the penetration, use the saturated weight

In which

+ σ_v : total stress at the depth z	(kN/m^2)
+ σ'_v : effective stress at the depth z	(kN/m^2)
+ u : Pore water pressure	(kN/m^2)
+ z_i : Depth to the middle point of the i layer counted from the surfac	(m)
+ l_i : thickness of the i soil layer	(m)

+ γ : Natural density of the soil layer above the ground water level (kN/m^3)
+ γ_{sat}: the saturated desnsity of the soil layer under the ground wate (kN/m^3)

$$\gamma_{sat} = \gamma_d(1 - \gamma_w/\gamma_s) + \gamma_w$$

+ γ_d : Dry density (kN/m^3)
+ γ_s : Unit weight (kN/m^3)
+ γ_w : water density γ_w = 10 (kN/m^3)
+ With: Energy efficiency E_h = 50

Name of layer	Layer thickness	Elevation of the layer bottom	depth	Unit weight	Dry density	saturated density	Total stress	Pore water pressure	Effective stress
	li		z_i	γ_s	γ_d	γ_{sat}	σ_{vz}	u	σ'_z
	(m)	(m)	(m)	(kN/m^3)	(kN/m^3)	(kN/m^3)	(kN/m^2)	(kN/m^2)	(kN/m^2)
		-0.94							
2	1.50	-2.44	0.75	0.00	0.00	0.00	0.00	0.00	0.00
2	16.26	-18.70	9.63	26.46	14.40	18.96	182.56	96.30	86.26
3	2.50	-21.20	19.01	26.56	11.74	17.32	329.25	190.10	139.15
4	18.50	-39.70	29.51	26.56	14.50	19.04	561.89	295.10	266.79
6a	4.50	-44.20	41.01	26.91	13.12	18.24	748.21	410.10	338.11
6b	4.60	-48.80	45.56	26.92	15.22	19.57	891.44	455.60	435.84
7a	8.40	-57.20	52.06	26.57	14.50	19.04	991.36	520.60	470.76
8	5.74	-62.94	59.13	26.91	18.37	21.54	1273.87	591.30	682.57
	1.50	-64.44							

Name of layer	Soil type	σ'_z (N/mm^2)	SPT N (Blow/30cm)	N_{60} = N*E_h/60	S_u (N/mm^2)	α	β	q_s (N/mm^2)	Resistance factor φs	value $\varphi_s l_i q_s$ (N/mm)
2	1	0.000	0	-	-	-			0.55	0.00
2	1	0.086	11	9	-	-	0.546	0.047	0.55	421.12
3	2	0.139	6	5	0.028	0.55	0.250	0.015	0.45	17.02
4	1	0.267	32	27	-	-	0.250	0.067	0.55	678.65
6a	2	0.338	8	6	0.038	0.55	0.250	0.021	0.45	41.77
6b	2	0.436	19	16	0.095	0.55	0.250	0.052	0.45	108.16
7a	1	0.471	75	63	-	-	0.250	0.118	0.55	543.73
8	2	0.683	85	71	0.425	0.55	0.250	0.234	0.45	603.78
	Total									2414.22

Total resistance of the side wall: $Q_s = \varphi_s \phi_i A_s = \Sigma \varphi_s \phi_i l_i P$ = **11376.7** (Kn)

φ_s: Shaft resistance factor of the pile in the ground (according to the table 10.5.5.2.4-1 AASHTO 2007)

2.2. PILE TIP RESISTANCE q_p:

Soil layer at Tip of pile: 2 Clay layer

2.2.1. Pile resistance in the clay soil

Fomular: q_p = NcSu <=4 (CT: 10.8.3.3.2-1)

q_p = NcSu = 3.83

In which Nc = 6[1+0.2(Z/D)] <= 9

Nc = 6[1+0.2(Z/D)] = = 55.6

Reduce:

If Su <0.024 Mpa Nc should be multiplied by 0,67

In which

D = 1500 (mm) - Diameter of pile

Z = 62000 (mm) - Penetration of shaft

Su = 0.425 (Mpa)

Nc = 9

=> q_p **= 3.825** (Mpa)

As for the bottom diameter bigger than 1270mm, qp has to reduce as follows:

q_{pr} = 1270*q_p/D_p **= 3.239** (Mpa)

- **Tip resistance of pile:**

$Q_b = \phi * q_p * A_b =$ 2289.16 (kN)

With tip resistance factor : $\phi =$ **0.4** (Table 10.5.5.2.4-1 AASHTO2007)

2.3. Pile capacity:

- Design resistance of the single pile :

$Q_T = (Q_s + Q_P) =$ 13665.9 kN

- Resistance of the resisting pile according to the pile group:

$Q_R = \eta . Q_T =$ **10477.18** kN

In which

+ $\eta =$ 0.77 Effective factor due to the pile working by group (AASHTO 2007 - 10.8.3.6.3)

+ d = 4.5 (m) The distance from the center to center of pile.

2.4. CONCLUSION

- Final pile capacity: $Q_R =$ 10477.18 kN
- Seftweight of pile: W = 1588.66 kN
- Maximum internal force applied to pile: $P_{max} =$ 8576 kN
- Check pile capacity : $P_{max} + W \le Q_R$

⇔ 10164.66 < 10477.18 **OK**

3. THE SETTLEMENT OF PILE GROUP:

- The Settlement of pile groups in cohesionless soils using SPT value:

$$\rho = \frac{30qI\sqrt{B}}{N_{60}}$$ (mm) (10.7.2.3.3-1)

In which: - q: net foundation pressure applid at $2D_b/3$, this pressure is equal to the applied load at the top of group divided by the area of the equivalent footing and does not include the weight of the piles or soil between the piers (Mpa).

- B : Width or smallest dimensions of pile group (mm).
- I : Influence factor of the effective group embedment (dim)

I = 1 - 0,125D'/B

- D' : Effective depth taken as $2D_b/3$, (mm)
- D_b : depth of embedment of piles in layer that provides support, (mm)

Nmax	Equivalent area	B	D_b	D'	I	N_{60}	q	ρ	Check
(Kn)	(mm^2)	(mm)	(mm)	(mm)	(dim)	blow	(Mpa)	(mm)	$\rho < 25.4$
83116	3.87E+08	11500	5740	3826.67	0.96	71	0.21	9.36	Ok

2.13 APROACH SLAB
2.13 BẢN QUÁ ĐỘ

1. Design Condition

1. 1. General Condition

Table 1. Design Condition

Item	Symbol	Unit	Design Value	Remark
Length of Slab	L	m	6.000	
Width of Slab	B	m	11.720	
Length of Span	Ls	m	4.200	
Thickness of Slab	ts	m	0.300	
Live Load	-	-	HL-93	
Impact	im	-	0.25	

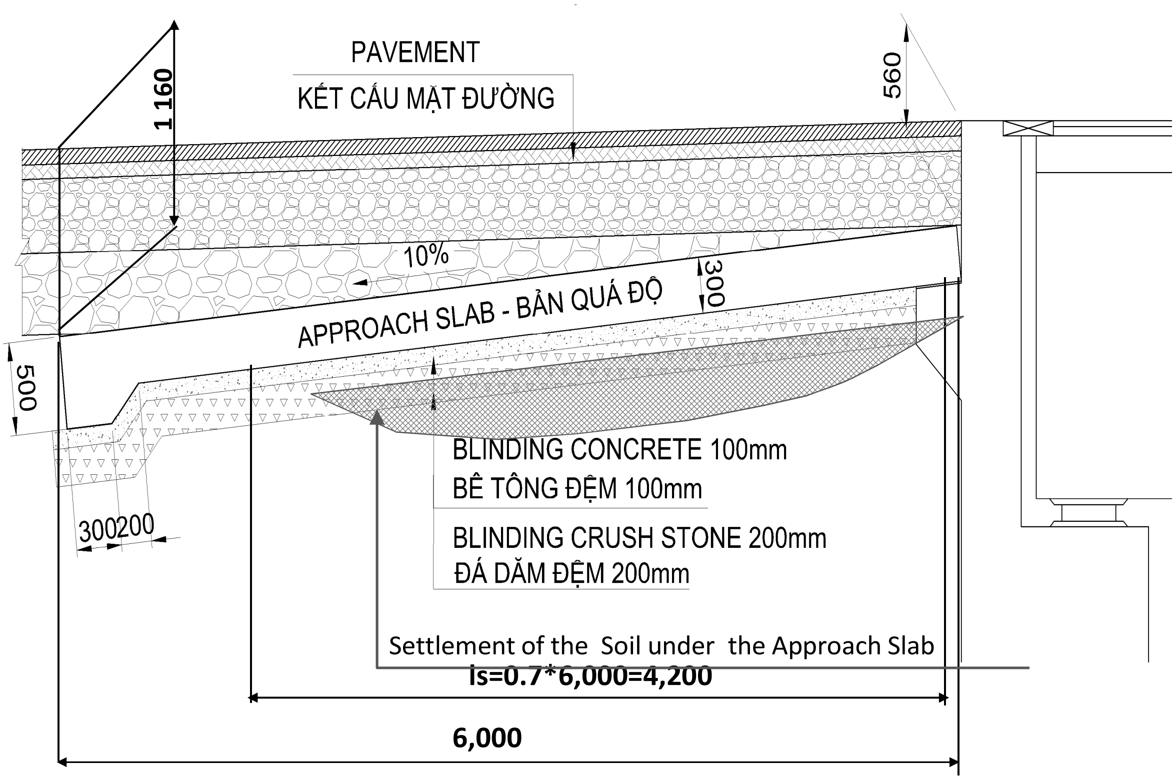


Figure Structural Drawing

1. 2. Design Basic

Apporach Slab shall be satisfied Eq.1 for each limit state. For Service limit states, resistance factors ϕ shall be taken as 1.0

- Load modifier factor ηi

$$\Sigma \eta i \cdot \gamma i \cdot Qi \leqq \phi Rn = Rn \qquad \textbf{Eq.1}$$

$\eta_D =$ 1.00 (for ductility)

$\eta_R =$ 1.00 (for redundancy)

$\eta_I =$ 1.00 (for oprational importance)

- Rsistance Factor φ for the Strength Limit Strength

Table Resistance Factor φ for Reincrced Concrete

Condition	Resistance Factor φ
Flexure and Tension	0.9
Shear and Torsion	0.9
Bearing and S&T Model	0.7
For Anchorage Zone(Compression)	0.8

- Crack Control for the Service Limit States

The tensil stress fr in the mild steel reinforcement at service limit state fra doas not exceed:

$$f_r = f_{ra} = \min\left(\frac{Z}{(d_c A)^{(1/3)}}, 0.6 \cdot f_{ry}\right)$$

fr: tensile stress in reinforcement

fra limit stress of reinforcement

Z: crack widthapameter(N/mm), Z= 17,500

dc: depth of concrete measured from exreme tension fiber to center of bar(mm),dc≦50mm

A: Area of concrete having same centroid as principle tensile reinforcement and bouded by the surface of the cross-section and astraight line parralled to the neutral axis;divided by number of barss, for caluculation purpose,the thickness of cover used to compute A shall not be taken to be greater than 50mm.

1. 3. Material

(1) Concrete

Table Property of Concrte

Item	Symbol	Unit	Desidn Value	Remark
Compressive Strength at 28 days	f'c	Mpa	25	
Modulas of Elastity	Ec	Mpa	25,300	
Poisson's Ratio	ν	-	0.2	
Tensile Bending Strength	fctr	Mpa	3.15	0.63*√ f'c
Limit Strein	εcfu	-	0.003	
Liner Thermal Coefficient	εt	-	0.0000108	

(1) Reincorcing Bar

Table Property of Reinforcing Bar

Item	Symbol	Unit	Desidn Value	Remark
Yield Strength	fry	Mpa	400	CB400-V
Tensile Strength	fru	Mpa	570	
Modulus of Elasticit	Er	Mpa	200,000	
Limit of elongation	εru	%	10.0	Engineering Judge

1. 3. Load condition

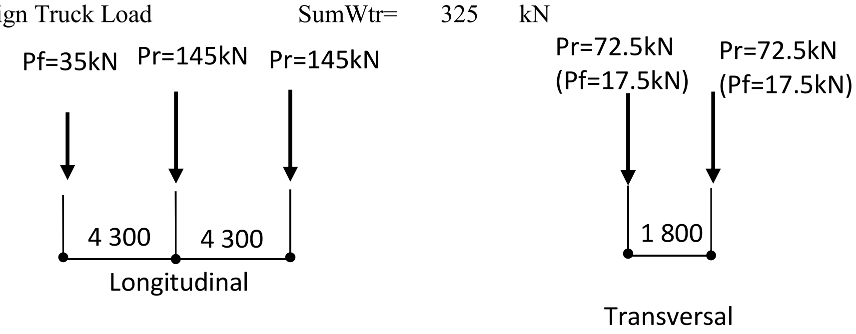
(1) Dead Load

Table Unit Weight

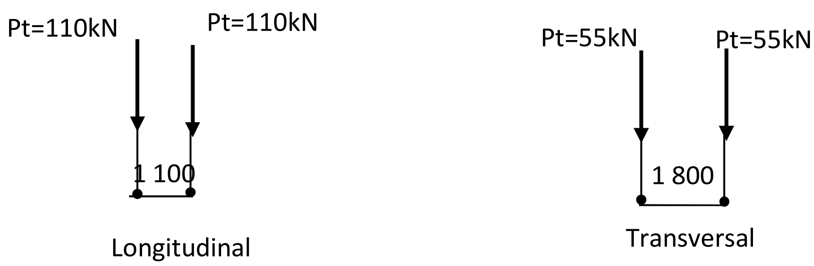
Item	Symbol	Unit	Unit Weight	Remark
Asphalt Concrete	γp	kN/m3	22.0	DW
Aggregate Base	γs	kN/m3	19.0	EV
Reinforcing Concrete	γpc	kN/m3	24.5	DC

(2) Live Load LL

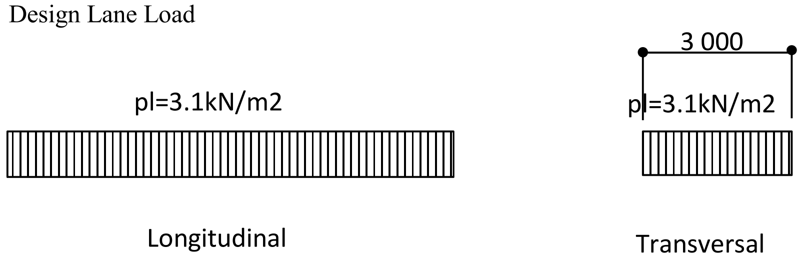
Design Truck Load SumWtr= 325 kN



Tandem Load SumWtt= 220 kN



Design Lane Load



1. 4. Load Combination

	DC	DW	EV	LL,IM
Strenght	1.25	1.5	1.35	1.75
Service	1.00	1.00	1.00	1.00

DC: Self Weight of Approach Slab
DW: Asphalt Concrete
EV: Aggregate Base
LL: Live Load
IM: Impact

2. Verification of Approach Slab

2. 1. Caluculation of Internal Force

2. 1. 1. Load

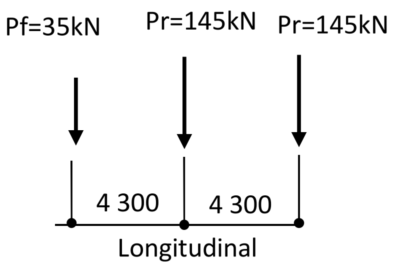
Dead Load

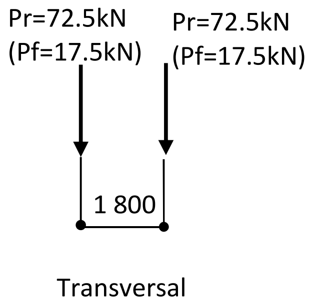
Unit Load

	Symbol	h	b	γ	w
		m	m	kN/m3	kN/m
Pavement	wdw	0.080	1.000	23.0	1.840
Soil	wes	0.780	1.000	20.0	15.600
Concrete	wdc	0.300	1.000	24.5	7.350
Sum	w				24.790

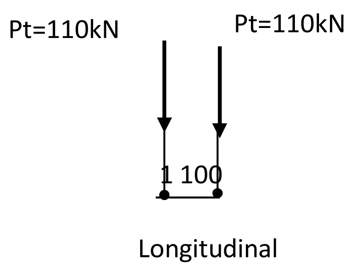
Live Load Effective width is considered.

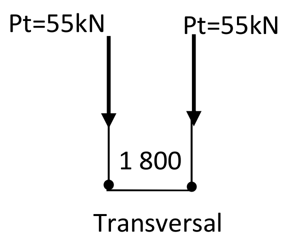
Design Truck Load SumWtr= 325 kN



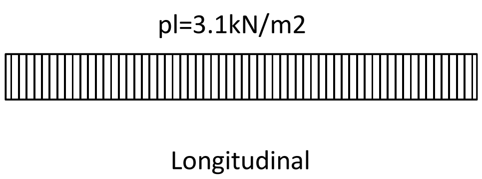


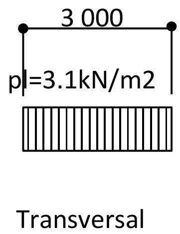
Tandem Load SumWtt= 220 kN





Design Lane Load





2. 1. 2. Internal Force

-Dead Load

Bending Moment at Span Center

Mc=1/8*w*l^2

	Symbol	w	l	km	Ms	γi	Mu
		m	m	-	kNm/m	-	kNm/m
Pavement	wdw	1.840	**4.200**	1.000	4.06	1.5	6.09
Soil	wes	15.600	4.200	1.000	34.40	1.5	51.60
Concrete	wdc	7.350	4.200	1.000	16.21	1.3	20.26
Sum	w						77.94

Shear Force at Support

	Symbol	w	l	km	Ss	γi	Su
		m	m	-	kN/m	-	kN/m
Pavement	wdw	1.840	**4.200**	1.000	3.86	1.5	5.80
Soil	wes	15.600	4.200	1.000	32.76	1.5	49.14
Concrete	wdc	7.350	4.200	1.000	15.44	1.3	19.29
Sum	w						74.23

- Live load

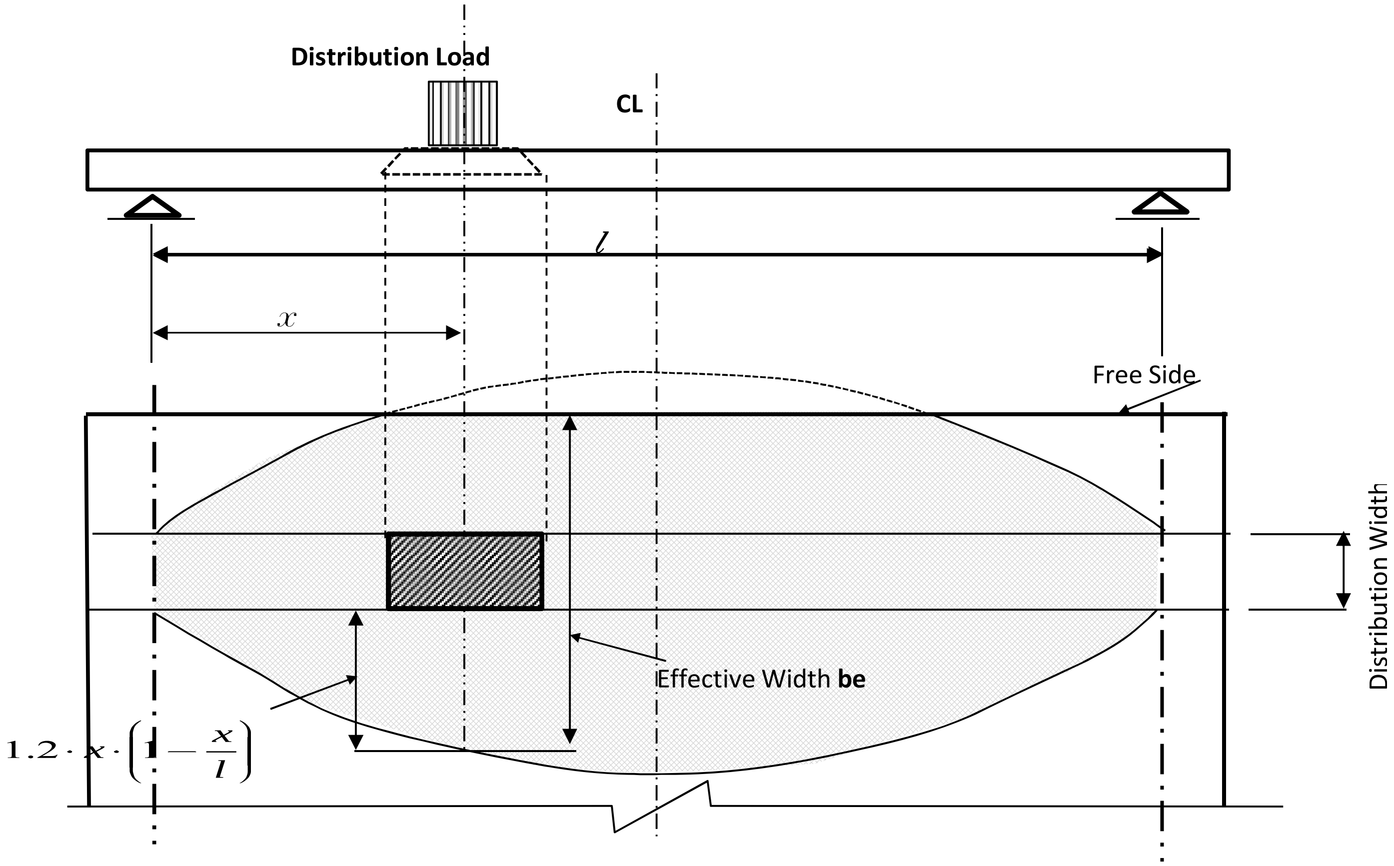
Effective Width of Wheel Load from JR(Japanese Reilway Campany) Standard

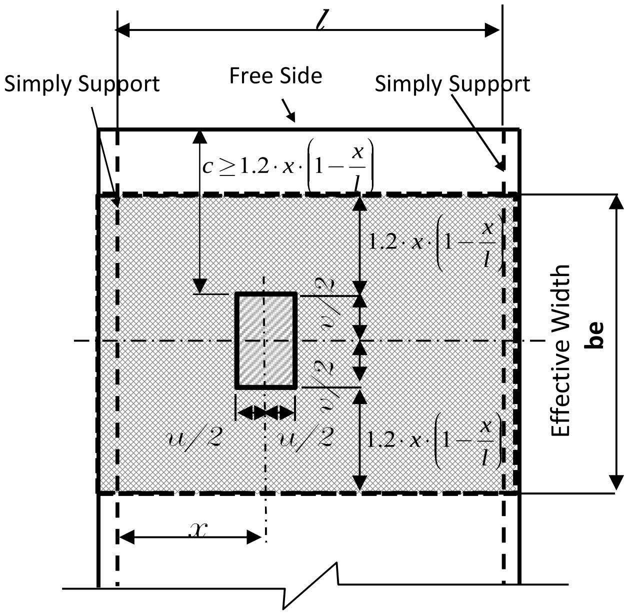
a) If c ≥ 1.2 x*(1-x/l) Then

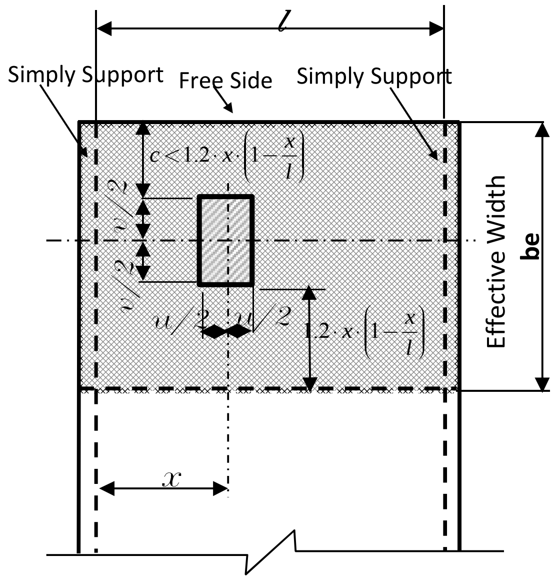
be=v+2.4*x*(1-x/l)

b) If c < 1.2 x*(1-x/l) Then

be=v+c+2.4*x*(1-x/l)



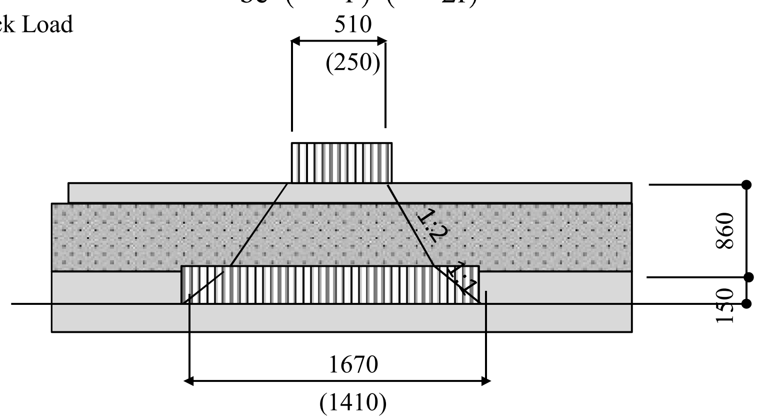


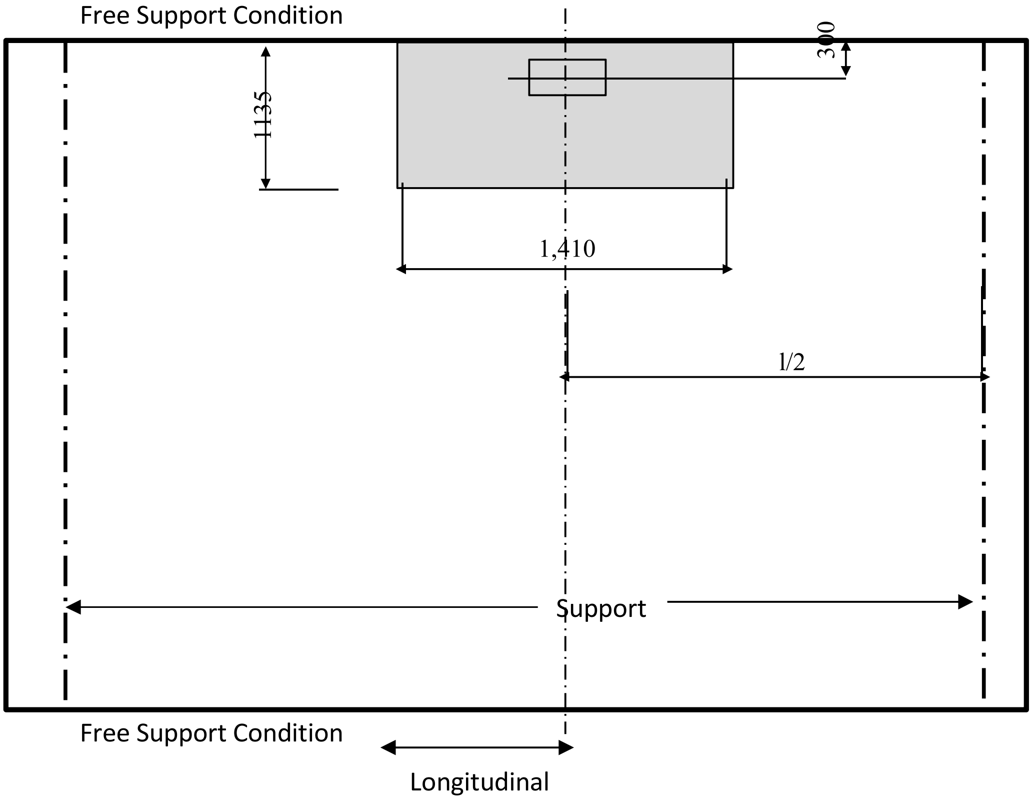


Bending Moment

$$M\max = \frac{P \cdot x}{be}\left(1-\frac{x}{l}\right)\cdot\left(1-\frac{u}{2l}\right)$$

'-Truck Load

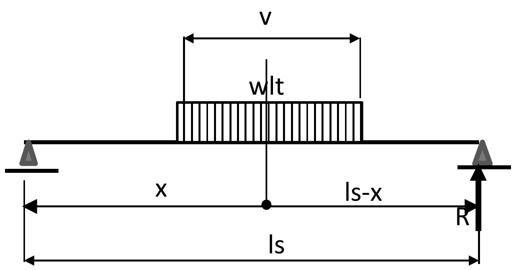




$$R = \frac{(v/2+x)}{2*l} wlt*(v/2+x)$$

$$R = \frac{x}{ls} wlt*v$$

$$R = \frac{l-(v/2+l-x)/2}{l} wlt*(v/2+l-x)$$

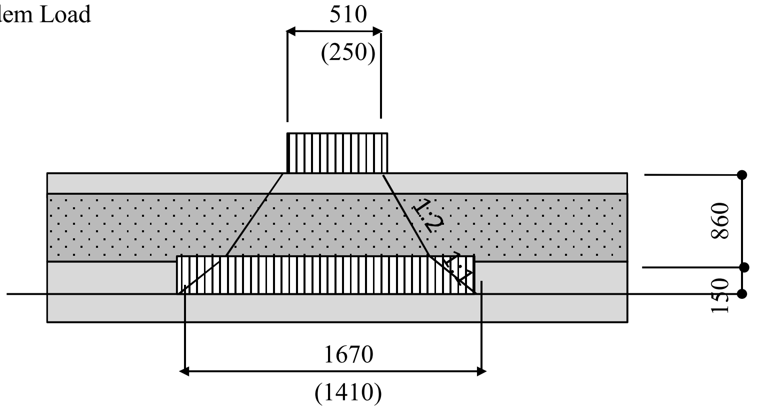


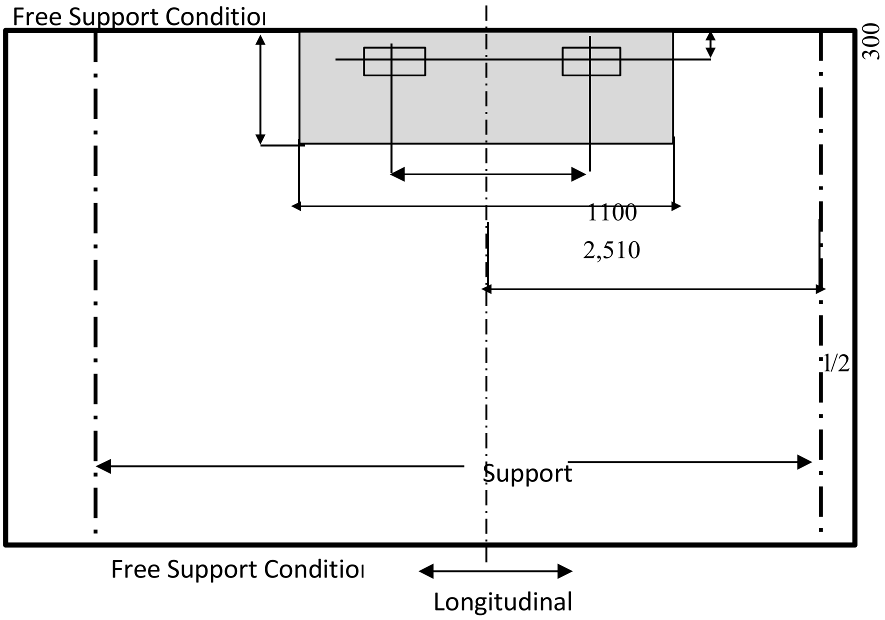
Moment of Truck Load

L=1.2x(1-x/l)

NO.	x	l	L	c	v	u	be	Pl	wlt	Mto	S2
	m	m	m	m	m	m	m	kN	kN/m2	kNm/m	kN/m
1	0.000	4.200	0.000	0.000	1.135	1.410	1.135	72.5	45.303	0.00	1.737
2	0.420	4.200	0.454	0.000	1.135	1.410	2.042	72.5	25.178	11.17	2.923
3	0.568	4.200	0.589	0.000	1.135	1.410	2.313	72.5	22.231	12.80	3.409
4	0.840	4.200	0.806	0.000	1.135	1.410	2.748	72.5	18.713	14.75	4.248
5	1.260	4.200	1.058	0.000	1.135	1.410	3.252	72.5	15.812	16.36	5.384
6	1.680	4.200	1.210	0.000	1.135	1.410	3.554	72.5	14.467	17.11	6.568
7	2.100	4.200	1.260	0.000	1.135	1.410	3.655	72.5	14.068	17.33	7.984
8	2.520	4.200	1.210	0.000	1.135	1.410	3.554	72.5	14.467	17.11	9.852
9	2.940	4.200	1.058	0.000	1.135	1.410	3.252	72.5	15.812	16.36	12.563
10	3.360	4.200	0.806	0.000	1.135	1.410	2.748	72.5	18.713	14.75	16.991
11	3.633	4.200	0.589	0.000	1.135	1.410	2.313	72.5	22.231	12.80	21.822
11	3.780	4.200	0.454	0.000	1.135	1.410	2.042	72.5	25.178	11.17	21.940
12	4.200	4.200	0.000	0.000	1.135	1.410	1.135	72.5	45.303	0.00	23.972

'-Tandem Load





Moment of Tandem Load

L=1.2x(1-x/l)

NO.	x	l	L	c	v	u	be	Pl	wpl	Mto	S2
	m	m	m	m	m	m	m	kN	kN/m2	kNm/m	kN/m
1	0.000	4.200	0.000	0.000	1.135	2.510	1.135	110.0	38.612	0.00	1.480
2	0.420	4.200	0.454	0.000	1.135	2.510	2.042	110.0	21.460	14.28	2.491
3	0.568	4.200	0.589	0.000	1.135	2.510	2.313	110.0	18.947	16.37	2.906
4	0.840	4.200	0.806	0.000	1.135	2.510	2.748	110.0	15.949	18.86	3.620
5	1.260	4.200	1.058	0.000	1.135	2.510	3.252	110.0	13.477	20.92	4.589
6	1.680	4.200	1.210	0.000	1.135	2.510	3.554	110.0	12.330	21.87	5.598
7	2.100	4.200	1.260	0.000	1.135	2.510	3.655	110.0	11.990	22.16	6.805
8	2.520	4.200	1.210	0.000	1.135	2.510	3.554	110.0	12.330	21.87	8.397
9	2.940	4.200	1.058	0.000	1.135	2.510	3.252	110.0	13.477	20.92	10.708
10	3.360	4.200	0.806	0.000	1.135	2.510	2.748	110.0	15.949	18.86	14.482
11	3.633	4.200	0.589	0.000	1.135	2.510	2.313	110.0	18.947	16.37	18.600
12	3.780	4.200	0.454	0.000	1.135	2.510	2.042	110.0	21.460	14.28	18.700
13	4.200	4.200	0.000	0.000	1.135	2.510	1.135	110.0	38.612	0.00	20.432

Moment of Line Load

$$M = \frac{1}{2} w_l \cdot \frac{x}{l}\left(1 - \frac{x}{l}\right)$$

NO.	x	l	pl	be	wl	Mlo	S2
	m	m	kN/m2	m	kN/m2	kNm/m	kN/m
1	0.000	4.200	3.100	1.000	3.100	0.00	6.51
2	0.420	4.200	3.100	1.000	3.100	2.46	6.51
3	0.568	4.200	3.100	1.000	3.100	3.20	6.51
4	0.840	4.200	3.100	1.000	3.100	4.37	6.51
5	1.260	4.200	3.100	1.000	3.100	5.74	6.51
6	1.680	4.200	3.100	1.000	3.100	6.56	6.51
7	2.100	4.200	3.100	1.000	3.100	6.84	6.51
8	2.520	4.200	3.100	1.000	3.100	6.56	6.51
9	2.940	4.200	3.100	1.000	3.100	5.74	6.51
10	3.360	4.200	3.100	1.000	3.100	4.37	6.51
11	3.633	4.200	3.100	1.000	3.100	3.20	6.51
12	3.780	4.200	3.100	1.000	3.100	2.46	6.51
13	4.200	4.200	3.100	1.000	3.100	0.00	6.51

Live Load Combination

Bending Moment

MAX(Truck ,Tandem)Load + Line Load km*(MLL+IM)

NO.	x	Mto	Mlo	MLL	IM	km	Mls	γi	Mu
	m	kNm/m	kNm/m	kNm/m	-	-	-	-	kNm/m
1	0.000	0.00	0.00	0.00	0.25	1.2	0.00	1.75	0.00
2	0.420	14.28	2.46	16.74	0.25	1.2	25.11	1.75	43.94
3	0.568	16.37	3.20	19.56	0.25	1.2	29.34	1.75	51.35
4	0.840	18.86	4.37	23.24	0.25	1.2	34.86	1.75	61.00
5	1.260	20.92	5.74	26.66	0.25	1.2	39.99	1.75	69.99
6	1.680	21.87	6.56	28.44	0.25	1.2	42.66	1.75	74.65
7	2.100	22.16	6.84	28.99	0.25	1.2	43.49	1.75	76.11
8	2.520	21.87	6.56	28.44	0.25	1.2	42.66	1.75	74.65
9	2.940	20.92	5.74	26.66	0.25	1.2	39.99	1.75	69.99
10	3.360	18.86	4.37	23.24	0.25	1.2	34.86	1.75	61.00
11	3.633	16.37	3.20	19.56	0.25	1.2	29.34	1.75	51.35
12	3.780	14.28	2.46	16.74	0.25	1.2	25.11	1.75	43.94
13	4.200	0.00	0.00	0.00	0.25	1.2	0.00	1.75	0.00

Momnet of Each Limit State Service Strength

	Symbol	M	γi	Mu
		kNm/m	-	kNm/m
Pavement	Mdw	4.06	1.50	6.09
Aggregate Base	Mev	34.40	1.35	46.44
Concrete	Mdc	16.21	1.25	20.26
Live Load	ML+IM	43.49	1.75	76.11
Sum	M	98.15		148.89

Shear force

MAX(Truck ,Tandem)Load + Line Load km*(MLL+IM)

NO.	x	St,t	Sl	Sll	Im	km	Ss	γi	Su
	m	kNm/m	kNm/m	kNm/m	-	-	-	-	kN/m
1	0.000	1.74	6.51	8.25	0.25	1.2	12.37	1.75	21.65
2	0.420	2.92	6.51	9.43	0.25	1.2	14.15	1.75	24.76
3	0.568	3.41	6.51	9.92	0.25	1.2	14.88	1.75	26.04
4	0.840	4.25	6.51	10.76	0.25	1.2	16.14	1.75	28.24
5	1.260	5.38	6.51	11.89	0.25	1.2	17.84	1.75	31.22
6	1.680	6.57	6.51	13.08	0.25	1.2	19.62	1.75	34.33
7	2.100	7.98	6.51	14.49	0.25	1.2	21.74	1.75	38.05
8	2.520	9.85	6.51	16.36	0.25	1.2	24.54	1.75	42.95
9	2.940	12.56	6.51	19.07	0.25	1.2	28.61	1.75	50.07
10	3.360	16.99	6.51	23.50	0.25	1.2	35.25	1.75	61.69
11	3.633	21.82	6.51	28.33	0.25	1.2	42.50	1.75	74.37
12	3.780	21.94	6.51	28.45	0.25	1.2	42.68	1.75	74.68
13	4.200	23.97	6.51	30.48	0.25	1.2	45.72	1.75	80.02

Shear Force of Each Limit State Service Strength

	Symbol	S	γi	Su
		kN/m	-	kN/m
Pavement	Sdw	3.86	1.50	5.80
Aggregate Base	Sev	32.76	1.35	44.23
Concrete	Sdc	15.44	1.25	19.29
Live Load	SL+IM	45.72	1.75	80.02
Sum	S	97.78		149.33

Verification of Span Center for Bending Moment and Minimum Reinforcement

LRFD 5.7.3 5.7.3.2

Item				Symbol	Unit	Value	Remark
Matrial	Conrete			f'c	Mpa	**25**	
	Reinfocing Bar			fry	fry	**400**	
Sectional Property	Concrete Section		Width	b	m	**1.000**	
			Thickness	h	m	**0.300**	
	Reinforcement	Layer 1	Diameter	Dr1	mm	**22**	
			Space	@	mm	**125**	
			Number	Nr1	mm	8.000	
			Aria	Ar1	mm2	3041.1	
			Cover	dr1	m	**0.215**	
		Layer 2	Diameter	Dr2	mm	**16**	
			Space	@	mm	**125**	
			Number	Nr2	mm	8.000	
			Aria	Ar2	mm2	1608.5	
			Cover	dr1	m	**0.085**	
Strength Limit State	Stress Block Factor			β1	-	0.85	
	Tension of Rebar			Tr1	kN	1216.4	
	Compresion of Rebar			Cr1	kN	0.0	
	Compresion of Concrete			Cco	kN/m	18062.5	
	Nutral Axis			cuy	m	0.067345	0
	Depth of Equivalent Stress Block			a	m	0.057244	
	Flexural Resistance of Rebar1			Mn1	kNm	226.7	
	Flexural Resistance of Rebar2			Mn2	kNm	0.0	
	Flexural Resistance of Concrete			Mn3	kNm	0.0	
	Nominal Flexural Resitance			Mn	kNm	226.7	
	Resistance Factor			φm	-	0.9	
	Factored Moment			Mr	kNm	204.0	
	Moment of SLS			Mu	kNm	148.9	
	Safty Factor(Mr/Mu)			F	-	1.370	
	Jagje Ha≥1 OK			-	-	OK	
Minimum Reinforcement	Section Modulus of Compsite Section			Sc	m3	0.01500	
	Section Modulus of Mono or Noncompsite Section			Snc	m3	0.01500	
	Modulus of Rupture			fctr	Mpa	3.15	
	Compression stress in Concrete due to effective prestress			fcpe	Mpa	0	
	Total Unfactored Dead Lod Momemt			Mhc	kNm	54.66	
	Cracking Moment	$Mcr1=S_c(f_r-f_{cpe})-M_{dch}(S_c/S_{hc}-1)$		Mcr1	kNm	47.25	
		Mcr2=Sc*fr		Mcr2	kNm	47.25	
		Max(Mcr1,Mcr2)		Mcr	kNm	47.25	
	Factored Moment			Mu	kNm	148.9	
	Factored Flexural Resistance			Mr	kNm	204.0	
	1.2Times the Craking moment 1.2Mcr			1.2Mcr	kNm	56.7	
	1.33 Times the Factored Moment 1.33Mu			1.33Mu	kNm	198.0	
	Judgement		Mr≧1.2Mcr-->OK	-	-	OK	
			or Mr≧1.3Mu-->OK	-	-	OK	
			Total Judge	-	-	OK	

Control of Cracking by Distribution of Reinforcement LRFD 5.7.3.3.2

Item				Symbol	Unit	Value	Remark
Matrial		Conrete		f'c	Mpa	25	
		Reinfocing Bar		fry	fry	400	
Sectional Property	Concrete Section		Width	b	m	1.000	
			Thickness	h	m	0.300	
	Reinforcement	Layer 1	Diameter	Dr1	mm	**22**	**0**
			Space	@	mm	**125**	
			Number	Nr1	mm	8.000	
			Aria	Ar1	mm2	3041.1	
			Cover	dr1	m	0.215	
		Layer 2	Diameter	Dr2	mm	**16**	**0**
			Space	@	mm	**125**	
			Number	Nr2	mm	8.0000	
			Aria	Ar2	mm2	1608.5	
			Cover	dr1	m	0.085	
Check At Service Stage LRFD 5.7.3.3.2	Bending Moment			Ms	kNm/m	98.15	
	Neutral Axis			cy	m	0.09282	
	Stress	Concrete		σcu	Mpa	11.21	
		Reinforcemnt		σr	Mpa	177.11	
	Cruck Contorol	Cover		dc	mm	**50**	
		Effective depth		d	mm	215	
		Widyh		b	mm	1000	
		(1+dc/(0.7d))		β	-	1.332226	
		Number of Rebar		Nr	nos	8.0000	
		Cruck Parameter		Zo	N/mm	17,500	
				Z	N/mm	17,500	
		Area of Concrete / Nr		A	mm2	12500	
		Limit of Rebar Stress1		fra1	Mpa	204.7	
		Yeild Stress of Rebar		fry	Mpa	400	
		Limit of Rebar Stress2		fra2	Mpa	240	
		Limit of Rebar Stress		fra	Mpa	204.7	
		Jagje fra≥σr -> OK		-	-	OK	

Verification of Shear LRFD 5.14.5.3

The Design Method is appleid the design for shear in slabs of box culverts specified in Article 5.14.5.3.

$$V_c = \left(0.178\sqrt{f_c^{'}} + 32\frac{A_s}{bd_c}\frac{V_u d_c}{M_u}\right)bd_e$$

but Vc shall nt exceed 0.332√ fc'bde

As; area of reinforcing steeel(mm2)

de; effective depth from extreme compression fiber to the centroid of the tensile force in the tensile reinforcement(m)

Vu; shear from factored loads(kN)

Mu; moment from factored loads(kNm)

b; design width usually (m)

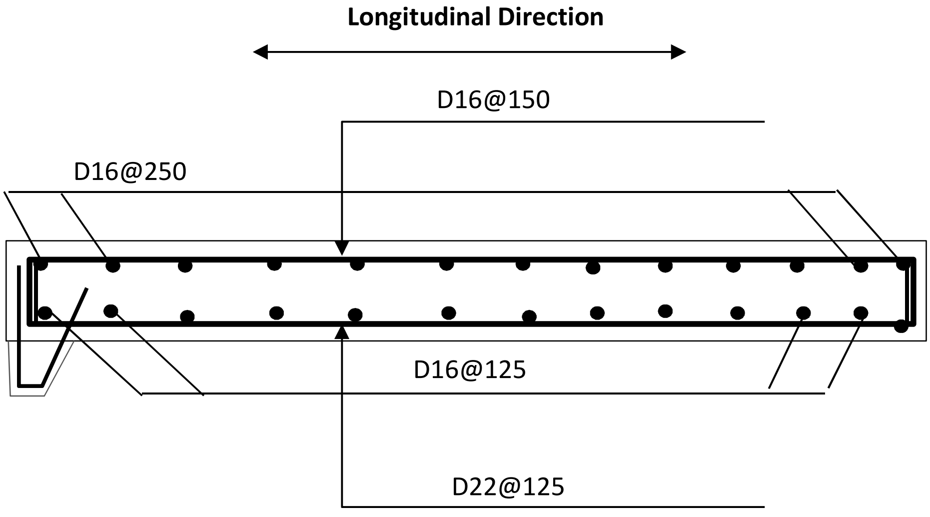
Vudc/Mu shall not be taken to be grater than 1.0.;

LRFD 5.14.5.3

Item		Symbol	Unit	Value	Remark
Internal Force	Factored Shear Force	Su	kN	148.89	
	Factored Moment	Mu	kNm	0	
Matrial	Conrete	f'c	Mpa	**25**	
	Reinfocing Bar	fry	Mpa	**400**	
Concrete Section	Width	b	m	**1.000**	
	Thickness	h	m	**0.300**	
Reinforcement	Diameter	Dr1	mm	**22**	**150**
	Number	Nr1	mm	6.6667	
	Aria	As	mm2	2534.2	
	Effective depth	de	m	0.215	
Resitance Shear Force	0.178Sqrt(f'c)	τc1	Mpa	0.89	
	k1=Vude/Mu	k1	-	1.000	
	ps=As/bde	ps	-	0.0118	
	32*ps*k1	τc2	Mpa	0.377	
	τc=τc1+τc2	τc	Mpa	1.27	
	Vc1=τc*bde	Vc1	lkN	272.4	
	Vc2=0.332Sqrt(f'c)bde	Vc2	kN	356.9	
	Vc=Min(Vc1,Vc2)	Vc	kN	272.4	
	Resitance Factor	φ	-	0.9	
	Factored Shear Resictance	Vn	kN	245.2	
Judgement	Vn≧Su--->OK	-	-	OK	

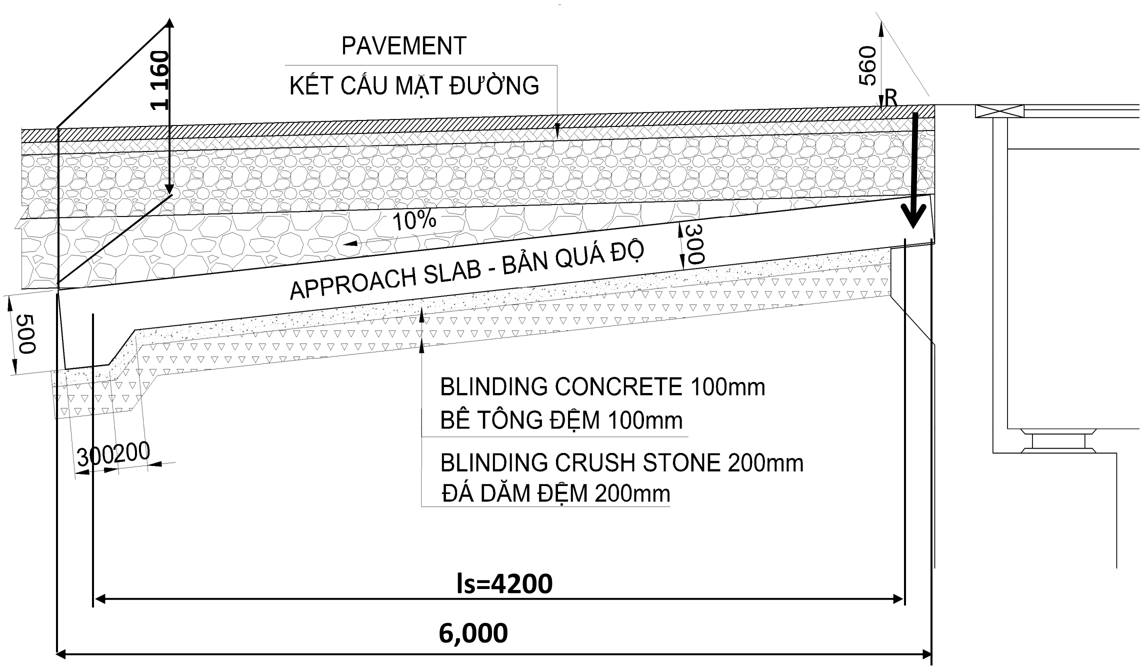
Arrangement of reinnforcing steel bar

	Diamiter	Space	Area	Main bar tomain Bar
Main Rebar	22	125	3041.1	1
Compression Side	16	125	1608.5	1/1.9
Distributed Rebar	16	125	1608.5	1/1.9
Othe Rbar	16	250	804.2	1/3.8



Caluculationof Braket for Approach Slab

Reaction of the Bracket



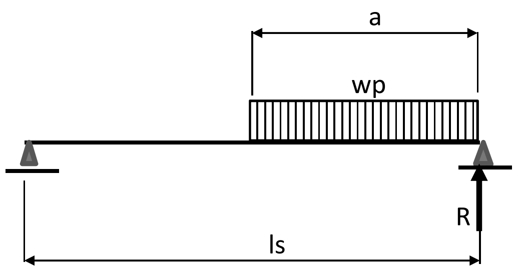
Dead Load R=05*l*w

	Symbol	w	l	km	Ss	γi	Su
		m	m	-	kNm/m	-	kNm/m
Pavement	wdw	1.840	4.200	1.000	3.86	1.5	5.80
Aggregate Base	wes	15.600	4.200	1.000	32.76	1.5	49.14
Concrete	wdc	7.350	4.200	1.000	15.44	1.3	19.29
Sum							74.23

Live Load

Truck and Tandem Lload

$$R = w \cdot a \cdot \left(1 - \frac{a}{2 \cdot 1}\right)$$



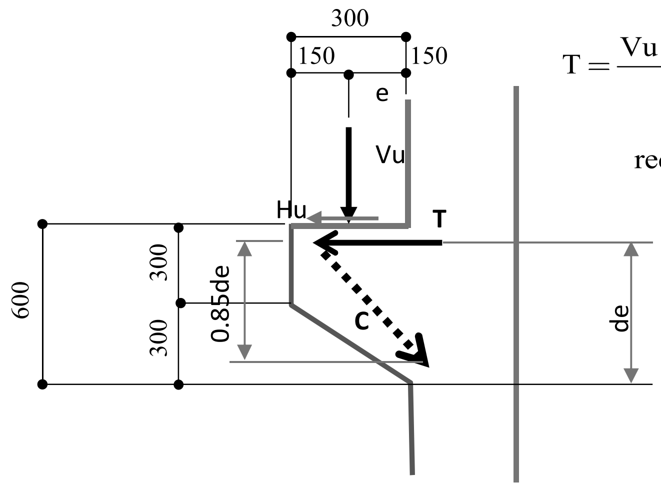
	w	ls	a	So	IM	km	Ss	γi	Su
	kN/m	m	m	kN	-	-	kN	-	kN
Truck Load	45.303	4.200	1.410	53.15	0.25	1.20	79.73	1.75	139.5
Tandem Load	38.612	4.200	2.510	67.96	0.25	1.20	101.94	1.75	178.4
Line Load	3.100	4.200	4.200	6.51	0.25	1.20	9.77	1.75	17.1
Truck +Line Load							89.50		156.6
Tandem +Line Load							111.70		195.5

Reaction of Each Limit State

	Symbol	Ss	γi	Su
		kN/m	-	kN/m
Pavement	Rpv	3.86	1.50	5.80
Aggregate Base	Rso	32.76	1.35	44.23
Concrete	Rdc	15.44	1.25	19.29
Live Load	Rl+IM	111.70	1.75	195.48
Sum		163.76		264.79

Caluculation of Bracket and Parapet

For Bracket Method of caluculation is used Strut and Tie model.



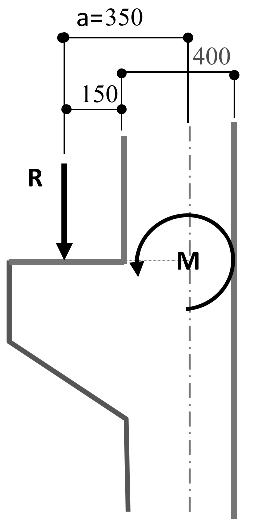
$$T = \frac{Vu \cdot e + Hu \cdot (h - de)}{0.85 \cdot de} + Hu$$

$$reqAr = \frac{T}{fry}$$

Restance of Shear Friction

			Symbol	Unit	Value	Remark
Deminsion	Hight of Bracket		h	m	0.6	
	Width		b	m	1	
	Effective Depth		de	m	0.53	
Yield Strength of Rebar			fry	Mpa	400	
Working Force	Vertical Force		Vu	kN	264.79	
	Horizontal Force		Hu	kN	0	
	Position of Force		e	m	0.15	
	Working Moment		M	kNm	39.72	
Check of Reinforcement	Arem Length		a	m	0.4505	
	Tension		Tu	kN	88.17	
	Requared Rebar		reqAr	mm2	220.4	
	Resistance	Diamiter	D	mm	16	
		Space(ctc)	@	mm	150	
		Number	Nr	nos	6.666667	
		Area	Ar	mm2	1340.4	
		Nominal	Tn	kN	536.2	
		R-Factor	φ	-	0.8	
		Factored	Tr	kN	428.9	
	Judge Tr≥Tur->OK			-	OK	
Check of Friction Resistance						

For Parapet



Bending Moment

	Symbol	Rs	a	Ms	γi	Mu
		kN/m	m	kN/m	-	kN/m
Pavement	Rpv	3.86	**0.350**	1.35	1.50	2.03
Soil	Rso	32.76	**0.350**	11.47	1.35	15.48
Concrete	Rdc	15.44	**0.350**	5.40	1.25	6.75
Live Load	Rl+IM	111.70	**0.350**	39.10	1.75	68.42
Sum		163.76		57.32		92.68

Verification of Cruck Control and Bending Resistance

Item				Symbol	Unit	Value	Remark
Martial		Concrete		f'c	Mpa	25	
		Reinforcing Bar		fry	fry	400	
Sectional Property	Concrete Section		Width	b	m	1.000	
			Thickness	h	m	0.400	
	Reinforcement	Layer 1	Diameter	Dr1	mm	16	
			Space	@	mm	150	
			Number	Nr1	mm	6.6667	
			Aria	Ar1	mm2	1340.4	
			Cover	dr1	m	0.330	
Check At Service Stage	Bending Moment			Ms	kNm/m	57.32	
	Neutral Axis			cy	m	0.088197	
	Stress	Concrete		σcu	Mpa	4.3	
		Reinforcement		σr	Mpa	142.2	
	Crack Control	Cover		dc	mm	50	
		Effective depth		d	mm	330	
		Width		b	mm	1000	
		(1+dc/(0.7d))		β	-	1.21645	
		Number of Rebar		Nr	nos	6.6667	
		Crack Parameter		Z	N/mm	22606.76	
		Area of Concrete / Nr		A	mm2	15000	
		Limit of Rebar Stress1		fra1	Mpa	248.8	
		Yield Stress of Rebar		fry	Mpa	400	
		Limit of Rebar Stress2		fra2	Mpa	240	
		Limit of Rebar Stress		fra	Mpa	240.0	
		Jangle fra≥σr -> OK		-	-	OK	
Strength Limit State	Stress Block Factor			β1	-	0.85	
	Tension of Rebar			Tr1	kN	536.2	
	Compression of Rebar			Cr1	kN	0.0	
	Compression of Concrete			Cco	kN/m	18062.5	
	Neutral Axis			cuy	m	0.029684	0
	Depth of Equivalent Stress Block			a	m	0.025231	
	Flexural Resistance of Rebar1			Mn1	kNm	170.2	
	Flexural Resistance of Rebar2			Mn2	kNm	0.0	
	Flexural Resistance of Concrete			Mn3	kNm	0.0	
	Nominal Flexural Resistance			Mn	kNm	170.2	
	Resistance Factor			φm	-	0.9	
	Factored Moment			Mr	kNm	153.2	
	Moment of SLS			Mu	kNm	92.7	
	Safety Factor(Mr/Mu)			F	-	1.7	
	Judge Ha≥1 OK			-	-	OK	

3. DRAINAGE DESIGN
3. THIẾT KẾ THOÁT NƯỚC MẶT CẦU

1. DRAINAGE DESIGN FOR MAJOR RIVER BRIDGES

1.1 INTRODUCTION

Deck drain gutter and grate inlet type of catch pit will be used to capture pavement surface runoff occurred during a storm runoff event and divert it down via grate inlets installed at interval along the gutter on both edges of the major river bridge deck. From the inlet, the runoff will be discharged into a vertical connecting pipe and further discharged into a vertical leg of a T-shape or an elbow pipe fitting and further drained into the collecting pipeline which is normally laid longitudinally under the bridge deck at the same designed slope as the bridge deck slope or steeper slopes according to the hydraulic calculation for the appropriate pipe sizes. Many inlets will be connected to the same collecting pipeline depending on the inlet spacing and inlet numbers required to drain the pavement surface runoff effectively. The end of the longitudinal pipeline will be connected with the vertical pipe by using a T-shape or an elbow pipe fitting. The vertical pipe will be laid along the exposed vertical wall of the bridge abutment, each of which is located on the river bank (on both river banks), before all of the collected and accumulated runoff is drained through the vertical and/or connecting inclined pipe with its outlet or down-spout at the end to the at-grade catch basin, specially designed to prevent the drained water to discharged directly into the nearby major river. This catch basin is named as “Ecosystem Infiltration Basin” or “EIB”.

The specific reason on this matter is that: water from the gutter inlet of the major river bridge is not allowed to drain directly into the river since the spill of fuel and chemicals into the main river by the traffic accident must be avoided. This requirement was previously mentioned in Section 5.4.2.3 of the Supplemental EIA Report and was approved by MONRE in Decision No. 2046/DQ-BTNMT, dated October 29, 2010.

Besides, there is a suggestion that the proposed drainage outlet should be avoided to drain directly on the cross roads passing under the bridge. As for the canal or the main channel of river, the proposed drainage outlet should also be avoided to drain directly on it as much as applicable. And in some conditions, installation of the inlets or catch pits on the bridge deck surface is not required. For example in the case that it is planned to install the expansion joint on the bridge or flyover, the catch pits may be installed at least in front of it only. But in the case that the portal rigid frame is planned to be used as the bridge structure, the catch pit is not required on the bridge deck surface, the deck drain should be joined to the road side ditches behind the abutments.

About the function of the EIB, the surface runoff draining out into the EIB will be stored for a period of time in the EIB, then will infiltrate gradually by gravity through the filter material layers provided at the bottom of the EIB. If the capacity of the EIB is not sufficient to store the drained runoff, which is not frequently occurred because its capacity is proposed to be about 50 cubic meters that are the maximum permissible volume of the fuel or chemicals to be loaded on one truck, it will be overflown through the spill-outlet of the EIB into the provided at-grade ditch crossing under the bridge. Then it will be further conveyed to the nearby road side ditch or the existing public waterways.

For all of the major river bridges (4 bridges) in this Project, the catch pit with grating at the inlet shall be a pre-fabricated Cast Iron type in accordance with ASTM A48 or JIS G5501 Standards. Such catch pit with grate inlet shall be strong enough to bear the design loads specified for the bridge (22 TCN 272-2005: Vietnamese Specification for Bridge Design). The round-shaped grate inlet is common in Vietnam. However, in consideration of the characteristics of heavy rain in the Project area, the rectangular-shaped type will be adopted to catch the surface water effectively. From the calculation, a 400 x 300 mm grate inlet for the catch pit is selected to be used for the major river bridge in this Project.

The type of pipes to be used for such deck drainage purpose shall be a Polyvinyl Chloride (PVC) pipe in accordance with TCVN 6151-1996 or ASTM A53 Standards. The vertical pipe or T-shape fitting shall be fitted to the circular-shape outlet of the catch pit. The minimum diameter of the collecting pipe and down spout is calculated based on the pipe slope and the accumulated amount of runoff in the pipe from upstream to downstream sides. A nominal diameter of the collecting pipe and its vertical or inclined down-spout is determined to be between 200-300 mm (for Chiem Son and Tra Bong bridges), 200-400 mm (for Tra Khuc bridge) and 300-400 mm (for Ky Lam bridge) with the minimum inlet spacing (or interval) of 10 m for all four (4) bridges.

In summary, for drainage design of the major river bridges, a 400 x 300 mm grate inlet for the Cast Iron catch pit (470 mm total depth) shall be used to capture the pavement surface runoff. The vertical PVC pipe with the appropriate diameter shall be used to extend the length of the catch pit down-spout end to the T-shape or elbow pipe fitting, connecting with the longitudinal collecting pipe. The end of the longitudinal pipeline will be connected with the vertical pipe by using a T-shape or an elbow pipe fitting. The size of the vertical and/or inclined PVC pipe extending to the down-spout shall be the same as that at the end of the longitudinal collecting pipe. The PVC pipe of 200-400 mm Nominal Diameter (DN) with allowable Nominal Pressure (PN) of 6.6 bar (or 0.66 Mpa) or PN6 class, for each bridge as mentioned above, shall be used according to the calculated size needed for each portion of the collecting pipeline.

1.2 DESIGN CRITERIA AND CONCEPTS

(i) Frequency of Rainfall and Rainfall Duration:

+ The maximum rainfall intensity of frequency 10 years (probability = 10%) at a rainfall duration of 15 minute is adopted for design of the bridge deck drainage for the major river bridges in this Project. It is 156 mm/hr from the Danang Rainfall Intensity-Duration-Frequency (IDF) curves.

(ii) Peak Flow Calculation:

One of the most commonly used equations for the calculation of peak flow from small areas (not more than 20 sq.km) is the Rational formula as given below.

$$Q = (CIA)/K_u$$

where:

Q = Peak flow rate in cu.m/sec

C = Dimensionless runoff coefficient

I = Rainfall intensity (for a short period duration) in mm/hr

A = Drainage area in hectares (ha)

K_u = Unit conversion factor equals to 360

The runoff coefficient C is a function of the ground cover and a host of other hydrologic abstractions. It relates the estimated peak discharge to a theoretical maximum of 100% runoff. Typical values for C are given in Table below. If the basin contains varying amounts of different land cover or other abstractions, a composite coefficient can be calculated through areal weighing.

Table 1.2-1: Runoff Coefficients for Rational Formula

Type of Drainage Area	Runoff Coefficient, C*
Residential:	
Single-family areas	0.30 - 0.50
Multi-units, detached	0.40 - 0.60
Multi-units, attached	0.60 - 0.75
Suburban	0.25 - 0.40
Apartment dwelling areas	0.50 - 0.70
Industrial:	
Light areas	0.50 - 0.80
Heavy areas	0.60 - 0.90
Parks, cemeteries	0.10 - 0.25
Playgrounds	0.20 - 0.40
Railroad yard areas	0.20 - 0.40
Unimproved areas	0.10 - 0.30
Lawns:	
Sandy soil, flat, 2%	0.05 - 0.10
Sandy soil, average, 2 - 7%	0.10 - 0.15
Sandy soil, steep, 7%	0.15 - 0.20
Heavy soil, flat, 2%	0.13 - 0.17
Heavy soil, average, 2 - 7%	0.18 - 0.22

Heavy soil, steep, 7%	0.25 - 0.35
Streets:	
Asphaltic	0.70 - 0.95
Concrete	0.80 - 0.95
Brick	0.70 - 0.85

Rainfall intensity, duration, and frequency curves are needed data for the Rational method calculation. Rainfall IDF (Intensity-Duration-Frequency) curves at Danang previously prepared by our Hydrology study team are adopted in the calculation of the peak flows from small drainage areas. The result of rainfall IDF curve analysis and its values estimated for further uses are shown in the Tables and Figure below.

Table 1.2-2: Rainfall Intensity for Short Duration at Danang

Duration	Rainfall Intensity (mm/hr)							
	50% P	**33.3% P**	**20% P**	**10% P**	**5% P**	**4% P**	**2% P**	**1% P**
5	142	159	176	198	217	224	242	260
10	122	138	154	174	192	198	215	232
15	109	123	138	156	173	178	194	210
30	83	94	107	121	134	138	151	163
45	69	78	88	99	110	114	124	134
60	59	67	75	85	94	97	105	114

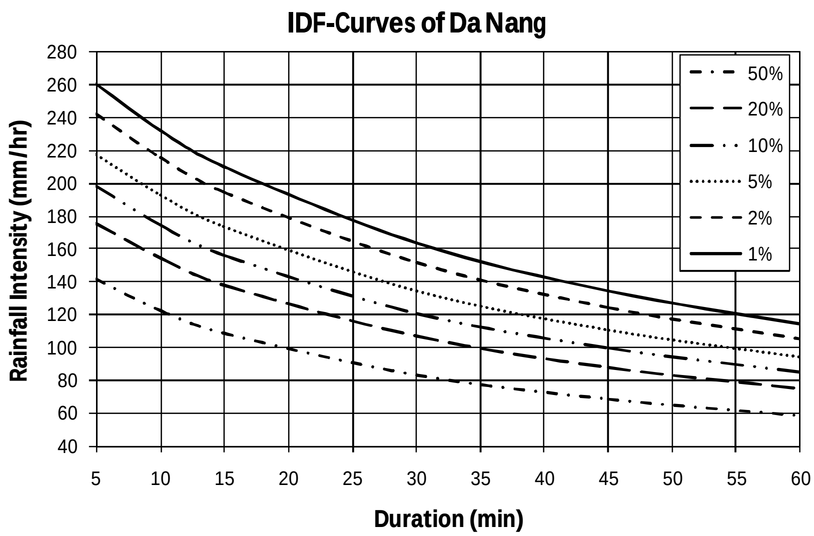


Figure 1.2-1: Rainfall IDF-Curves of Danang

Based on the Tables and Figure shown above, short duration rainfall intensities of 5, 10, 15, 30, 45 and 60 min can be estimated for 1%, 2%, 4% and 5% probability levels from the IDF relations. For instance, 5-min design rainfall intensities are: 260 mm/hr (1%P), 242 mm/hr (2%P), 224 mm/hr (4%P) and 217 mm/hr (5%P). Similarly, 60-min design rainfall intensities are: 114 mm/hr (1%P), 105 mm/hr (2%P), 97 mm/hr (4%P) and 94 mm/hr (5%P).

In case the drainage area is larger than 20 sq.km, the Rational method should not be adopted because there will be higher errors in the calculation result. Most hydrologists are likely to use the other methods, e.g. Unit Hydrograph, Regional Flood Curves, etc., for the estimation of the peak flow rate for a specific region or drainage area.

For the Rational formula, a value of ‘C’ of 0.95 is adopted in this case for the calculation of the deck drainage based on the type of the pavement and the deck surface, a maximum value for the range is adopted for surface runoff estimation on the safe side.

As for the drainage area, ‘A’, in this formula, it is the result of different drainage widths on the cross-slope pavement surface multiplied by the trial value of water intercepting inlet interval (or spacing) along the gutter closed to the bridge parapet, to obtain the drainage area for each inlet.

The inundated width is the allowable spread of water on the pavement when the cross-sectional area of the gutter-flow along the edge of the bridge parapet is considered. The design speed is important to the selection of the design criteria on this matter. At speeds greater than 75 km/hr, it has been shown that water on the pavement can cause hydroplaning which a risk of accident from uncontrollable driving will be higher.

Therefore, the spread of water is not allowed on the traffic lanes of the main road (thru-way) of expressway or the traffic lanes of the main bridge portion (in this case) which the maximum allowable speed is 120 km/hr, the spread will be allowed only on the emergency lane closing to the parapet, i.e. the maximum spread (inundated width) is 3.25 m for the main bridge portion of the major river bridge. This matter is specified in a Vietnamese Standard: 22 TCN 273-2001, "Standard for Designing Highway (junctions)".

(iii) Calculation and Design of Drainage Pipe and Inlet Spacing:

The required opening area for a design discharge of the drainage structure/pipe can be calculated by using a hydraulic formula as shown below:

$$A = Q/V$$

Where
A = Minimum opening area of the structure/pipe in sq.m
Q = Design discharge or flow rate in cu.m
V = Allowable velocity of flow in m/sec

To obtain the required opening area for a known design discharge of the drainage structure/pipe or channel by this formula, it needs to assume the allowable velocity of flow to be not more than some values, e.g. a value of 2.5 m/sec for the concrete or asphaltic surface of the pavement on the bridge deck or a value of 3.0 m/sec for the PVC drainage pipe. For safety purposes, the maximum design flow in the longitudinal pipe shall be a value which is not more than 90% of the full-pipe capacity. In that case, the pipe flow will be likely an open-channel flow.

The alternative approach to determine the flow velocity of an open channel or structure can be carried out by using the Manning equation as provided below.

$$V = \frac{1}{n} R^{2/3} S^{1/2}$$

Where V = Velocity of flow in m/sec
n = Manning roughness coefficient
R = Hydraulic radius in m (= A/P, and P = Wetted Perimeter in m)
S = Slope of the channel

The Manning Roughness Coefficient (n) varies with the type of materials used for the drainage structure or ditch or channel. The values shown in the following Table are recommended to be adopted for calculation of its flow capacity.

Table 1.2-3: Manning Roughness Coefficient for Different Materials

Type of Material	Manning Roughness Coefficient (n)
Concrete Pipe or Box Culvert (pre-fab.)	0.013
Concrete Pipe or Box Culvert or Ditch	0.015
Rock-cut Ditch (smooth /uniform)	0.033
Mortared Stone Ditch	0.040
Earth Ditch (straight /uniform)	0.035

The Manning 'n' adopted for calculation of the surface runoff in this case is 0.013 which is suitable for the smooth surfaces of the surface water open channel flow or gutter flow closed to the parapet.

1.3 DRAINAGE CALCULATION METHOD

(i) Deck Drainage:

+ ***On-Deck Drainage***:

The major river bridge: use rainfall intensity of frequency 10 years (probability = 10%) and rainfall duration equal to 15 minutes, which can obtain the rainfall intensity from the Danang IDF to be 156 mm/hr.

Determining the inlet overflow capacity by using overflow velocity equaled to the Critical Velocity (V_c) which can be determined by this expression:

$$V_c = \sqrt{2gh}$$

Where g = acceleration due to gravity
h = average height of water when the width of ponding is equaled to the maximum allowable spread of water from the inside edge of the parapet as mentioned above

Then calculate inlet capacity by this expression

$$Q = VA$$

Where Q = Inlet capacity
$V = V_c$ = Critical Velocity
A = L.Yc = Overflow area
L = overflow length
Yc = Critical overflow depth, simplified to be 2/3 h

The applied size of catch pit (with grate inlet) shall have the capacity twice of the calculated size due to clogging. Therefore, the catch pit size for the major river bridge is 400 x 300 x 470 (depth) mm, spacing at a minimum acceptable value of 10 m. interval.

+ **Collecting Pipeline**:

Collecting Pipe: use PVC pipe in accordance with TCVN 6151-1996 or ASTM A53 Standards. The following requirements are also adopted:

- the minimum pipe nominal diameter (DN) is 200 mm. (300 mm for Ky Lam bridge)
- The minimum longitudinal slope of pipe is 2% (preferable).

Full pipe flow velocity is calculated by using Manning formula from the pipe slope and pipe size selected by trial. The designed pipe size and slope shall be able to accommodate the accumulated runoff at all locations along the pipeline. However, such accumulated runoff at each location should not be larger than 90% of the pipe full flow capacity at the location.

Summary of Designed Collecting Pipe:

Pipe material: PVC (conforming to TCVN 6151-1996, ASTM A53 Standards)
Nominal Pressure: PN6 (= 0.66 MPa)
Nominal Diameter : DN 200 (outside diameter = 225.3 mm) or DN 300 for Ky Lam bridge

+ **Summary of the calculation:**

Summary of the calculation includes surface runoff, spacing of inlets (inlet interval), sizes of pipe, etc. are shown in Appendix ….. .

DETAILED DESIGN FOR DA NANG - QUANG NGAI EXPRESSWAY DEVELOPMENT PROJECT

CACULATING HYDRAILIC OF KYLAM BRIDGE DRAINAGE

P%= 10% T(min)= 15

Inlet name	Distance	Accumulated distance	Area: Surface	Area: Accumulated area	Runoff coefficient	Rainfall intensity	Runoff flow	Longitudinal grade of bridge (i1)	Longitudinal grade of invert of pipe (i2)	Capacity flow (at 90%pipe)	Proposed pipe size	Conclusion	Remark	H: height of vertical pipe (varied height)
	(m2)	(m)	(m2)	(m2)		(mm/hr)	(m3/s)			(m3/s)	(mm)			(m)
From: PEAK VERTICAL TO A1 ATBUMENT														
Peak		0												
I1	10	10	125	125	0.95	156	0.005	0.0027	0.0030	0.0612	D300	OK	i1<i2	0.473
I2	10	20	125	250	0.95	156	0.010	0.0027	0.0030	0.0612	D300	OK	i1<i2	0.476
I3	10	30	125	375	0.95	156	0.015	0.0027	0.0030	0.0612	D300	OK	i1<i2	0.480
I4	10	40	125	500	0.95	156	0.021	0.0027	0.0030	0.0612	D300	OK	i1<i2	0.483
I5	10	50	125	625	0.95	156	0.026	0.0027	0.0030	0.0612	D300	OK	i1<i2	0.486
I6	10	60	125	750	0.95	156	0.031	0.0027	0.0030	0.0612	D300	OK	i1<i2	0.489
I7	10	70	125	875	0.95	156	0.036	0.0027	0.0030	0.0612	D300	OK	i1<i2	0.492
I8	10	80	125	1000	0.95	156	0.041	0.0027	0.0030	0.0612	D300	OK	i1<i2	0.496
I9	10	90	125	1125	0.95	156	0.046	0.0027	0.0030	0.0612	D300	OK	i1<i2	0.499
I10	10	100	125	1250	0.95	156	0.051	0.0027	0.0030	0.0612	D300	OK	i1<i2	0.502
I11	10	110	125	1375	0.95	156	0.057	0.0089	0.0089	0.1054	D300	OK	i1<i2	0.503
I12	10	120	125	1500	0.95	156	0.062	0.0089	0.0089	0.1054	D300	OK	i1<i2	0.503
I13	10	130	125	1625	0.95	156	0.067	0.0089	0.0089	0.1054	D300	OK	i1<i2	0.504
I14	10	140	125	1750	0.95	156	0.072	0.0089	0.0089	0.1054	D300	OK	i1<i2	0.504
I15	10	150	125	1875	0.95	156	0.077	0.0089	0.0089	0.1054	D300	OK	i1<i2	0.505
I16	10	160	125	2000	0.95	156	0.082	0.0089	0.0089	0.1054	D300	OK	i1=i2	0.505
I17	10	170	125	2125	0.95	156	0.087	0.0089	0.0089	0.1054	D300	OK	i1=i2	0.506
I18	10	180	125	2250	0.95	156	0.093	0.0089	0.0089	0.1054	D300	OK	i1=i2	0.506
I19	10	190	125	2375	0.95	156	0.098	0.0089	0.0089	0.1054	D300	OK	i1=i2	0.507
I20	10	200	125	2500	0.95	156	0.103	0.0089	0.0089	0.1054	D300	OK	i1=i2	0.507
I21	10	210	125	2625	0.95	156	0.108	0.0120	0.0120	0.1223	D300	OK	i1=i2	0.507
I22	10	220	125	2750	0.95	156	0.113	0.0120	0.0120	0.1223	D300	OK	i1=i2	0.507
I23	10	230	125	2875	0.95	156	0.118	0.0120	0.0120	0.1223	D300	OK	i1=i2	0.507
I24	10	240	125	3000	0.95	156	0.124	0.0120	0.0120	0.2630	D400	OK	i1=i2	0.507
I25	10	250	125	3125	0.95	156	0.129	0.0120	0.0120	0.2630	D400	OK	i1=i2	0.507
I26	10	260	125	3250	0.95	156	0.134	0.0120	0.0120	0.2630	D400	OK	i1=i2	0.507
I27	10	270	125	3375	0.95	156	0.139	0.0120	0.0120	0.2630	D400	OK	i1=i2	0.507
I28	10	280	125	3500	0.95	156	0.144	0.0120	0.0120	0.2630	D400	OK	i1=i2	0.507
I29	10	290	125	3625	0.95	156	0.149	0.0120	0.0120	0.2630	D400	OK	i1=i2	0.507
I30	10	300	125	3750	0.95	156	0.154	0.0120	0.0120	0.2630	D400	OK	i1=i2	0.507
I31	10	310	125	3875	0.95	156	0.160	0.0120	0.0120	0.2630	D400	OK	i1=i2	0.507
I32	10	320	125	4000	0.95	156	0.165	0.0120	0.0120	0.2630	D400	OK	i1=i2	0.507
I33	10	330	125	4125	0.95	156	0.170	0.0120	0.0120	0.2630	D400	OK	i1=i2	0.507
I34	10	340	125	4250	0.95	156	0.175	0.0120	0.0120	0.2630	D400	OK	i1=i2	0.507
I35	10	350	125	4375	0.95	156	0.180	0.0120	0.0120	0.2630	D400	OK	i1=i2	0.507
I36	10	360	125	4500	0.95	156	0.185	0.0120	0.0120	0.2630	D400	OK	i1=i2	0.507
I37	10	370	125	4625	0.95	156	0.190	0.0120	0.0120	0.2630	D400	OK	i1=i2	0.507
I38	10	380	125	4750	0.95	156	0.196	0.0120	0.0120	0.2630	D400	OK	i1=i2	0.507
I39	10	390	125	4875	0.95	156	0.201	0.0120	0.0120	0.2630	D400	OK	i1=i2	0.507
I40	10	400	125	5000	0.95	156	0.206	0.0120	0.0120	0.2630	D400	OK	i1=i2	0.507
I41	10	410	125	5125	0.95	156	0.211	0.0120	0.0120	0.2630	D400	OK	i1=i2	0.507
I42	10	420	125	5250	0.95	156	0.216	0.0120	0.0120	0.2630	D400	OK	i1=i2	0.507
I43	10	430	125	5375	0.95	156	0.221	0.0120	0.0120	0.2630	D400	OK	i1<i2	0.507
I44	10	440	125	5500	0.95	156	0.226	0.0120	0.0120	0.2630	D400	OK	i1<i2	0.507

DETAILED DESIGN FOR DA NANG - QUANG NGAI EXPRESSWAY DEVELOPMENT PROJECT

CACULATING HYDRAILIC OF KYLAM BRIDGE DRAINAGE

P%= 10% T(min)= 15

Inlet name	Distance	Accumulated distance	Area		Runoff coefficient	Rainfall intensity	Runoff flow	Longitudinal grade of bridge (i1)	Longitudinal grade of invert of pipe (i2)	Capacity flow (at 90%pipe)	Proposed pipe size	Conclusion	Remark	H: height of vertical pipe (varied height)
			Surface	Accumulated area										
	(m2)	(m)	(m2)	(m2)		(mm/hr)	(m3/s)			(m3/s)	(mm)			(m)
I45	10	450	125	5625	0.95	156	0.232	0.0120	0.0120	0.2630	D400	OK	i1<i2	0.507
I46	10	460	125	5750	0.95	156	0.237	0.0120	0.0120	0.2630	D400	OK	i1<i2	0.507
I47	10	470	125	5875	0.95	156	0.242	0.0120	0.0120	0.2630	D400	OK	i1<i2	0.507
I48	10	480	125	6000	0.95	156	0.247	0.0120	0.0150	0.2940	D400	OK	i1<i2	0.537
I49	10	490	125	6125	0.95	156	0.252	0.0120	0.0150	0.2940	D400	OK	i1<i2	0.567
I50	10	500	125	6250	0.95	156	0.257	0.0120	0.0150	0.2940	D400	OK	i1<i2	0.597
I51	10	510	125	6375	0.95	156	0.262	0.0120	0.0150	0.2940	D400	OK	i1<i2	0.627
I52	10	520	125	6500	0.95	156	0.268	0.0120	0.0150	0.2940	D400	OK	i1<i2	0.657
I53	10	530	125	6625	0.95	156	0.273	0.0120	0.0150	0.2940	D400	OK	i1<i2	0.687
I54	10	540	125	6750	0.95	156	0.278	0.0120	0.0150	0.2940	D400	OK	i1=i2	0.717
I55	10	550	125	6875	0.95	156	0.283	0.0120	0.0160	0.3037	D400	OK	i1=i2	0.757
I56	10	560	125	7000	0.95	156	0.288	0.0120	0.0160	0.3037	D400	OK	i1=i2	0.797
I57	10	570	125	7125	0.95	156	0.293	0.0120	0.0160	0.3037	D400	OK	i1=i2	0.837
I58	10	580	125	7250	0.95	156	0.298	0.0120	0.0160	0.3037	D400	OK	i1=i2	0.877
I59	10	590	125	7375	0.95	156	0.304	0.0120	0.0160	0.3037	D400	OK	i1=i2	0.917
I60	10	600	125	7500	0.95	156	0.309	0.0120	0.0190	0.3309	D400	OK	i1=i2	0.987
I61	10	610	125	7625	0.95	156	0.314	0.0120	0.0190	0.3309	D400	OK	i1=i2	1.057
I62	10	620	125	7750	0.95	156	0.319	0.0120	0.0190	0.3309	D400	OK	i1=i2	1.127
I63	10	630	125	7875	0.95	156	0.324	0.0120	0.0190	0.3309	D400	OK	i1=i2	1.197
I64	10	640	125	8000	0.95	156	0.329	0.0120	0.0190	0.3309	D400	OK	i1=i2	1.267
I65	10	650	125	8125	0.95	156	0.334	0.0120	0.0204	0.3429	D400	OK	i1=i2	1.351
I66	12.6	662.6	157.5	8282.5	0.95	156	0.341	0.0120	0.0204	0.3429	D400	OK	i1=i2	1.457
From: PEAK VERTICAL TO A2 ATBUMENT														
Peak														
I'1	10	10	125	125	0.95	156	0.005	0.0035	0.0035	0.0661	D300	OK	i1=i2	0.470
I'2	10	20	125	250	0.95	156	0.010	0.0035	0.0035	0.0661	D300	OK	i1=i2	0.470
I'3	10	30	125	375	0.95	156	0.015	0.0035	0.0035	0.0661	D300	OK	i1=i2	0.470
I'4	10	40	125	500	0.95	156	0.021	0.0035	0.0035	0.0661	D300	OK	i1=i2	0.470
I'5	10	50	125	625	0.95	156	0.026	0.0035	0.0035	0.0661	D300	OK	i1=i2	0.470
I'6	10	60	125	750	0.95	156	0.031	0.0035	0.0035	0.0661	D300	OK	i1<i2	0.470
I'7	10	70	125	875	0.95	156	0.036	0.0035	0.0035	0.0661	D300	OK	i1<i2	0.470
I'8	10	80	125	1000	0.95	156	0.041	0.0035	0.0035	0.0661	D300	OK	i1<i2	0.470
I'9	10	90	125	1125	0.95	156	0.046	0.0035	0.0035	0.0661	D300	OK	i1<i2	0.470
I'10	10	100	125	1250	0.95	156	0.051	0.0035	0.0035	0.0661	D300	OK	i1<i2	0.470
I'11	10	110	125	1375	0.95	156	0.057	0.0097	0.0097	0.1100	D300	OK	i1<i2	0.470
I'12	10	120	125	1500	0.95	156	0.062	0.0097	0.0097	0.1100	D300	OK	i1<i2	0.470
I'13	10	130	125	1625	0.95	156	0.067	0.0097	0.0097	0.1100	D300	OK	i1<i2	0.471
I'14	10	140	125	1750	0.95	156	0.072	0.0097	0.0097	0.1100	D300	OK	i1<i2	0.471
I'15	10	150	125	1875	0.95	156	0.077	0.0097	0.0097	0.1100	D300	OK	i1<i2	0.471
I'16	10	160	125	2000	0.95	156	0.082	0.0097	0.0097	0.1100	D300	OK	i1=i2	0.471
I'17	10	170	125	2125	0.95	156	0.087	0.0097	0.0097	0.1100	D300	OK	i1=i2	0.471
I'18	10	180	125	2250	0.95	156	0.093	0.0097	0.0159	0.1408	D300	OK	i1=i2	0.534
I'19	10	190	125	2375	0.95	156	0.098	0.0097	0.0159	0.1408	D300	OK	i1=i2	0.596
I'20	10	200	125	2500	0.95	156	0.103	0.0097	0.0159	0.1408	D300	OK	i1=i2	0.658
I'21	10	210	125	2625	0.95	156	0.108	0.0159	0.0159	0.1408	D300	OK	i1=i2	0.659
I'22	10	220	125	2750	0.95	156	0.113	0.0159	0.0159	0.1408	D300	OK	i1=i2	0.659
I'23	10	230	125	2875	0.95	156	0.118	0.0159	0.0159	0.3027	D400	OK	i1=i2	0.660

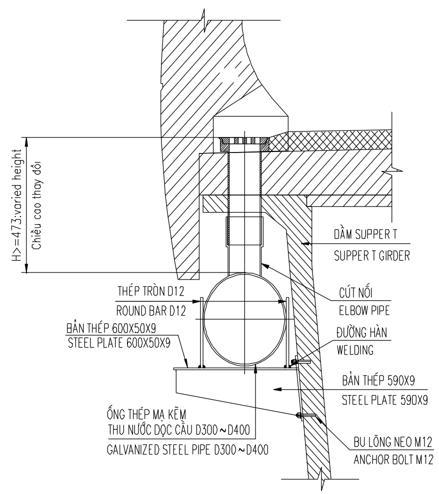
DETAILED DESIGN FOR DA NANG - QUANG NGAI EXPRESSWAY DEVELOPMENT PROJECT

CACULATING HYDRAILIC OF KYLAM BRIDGE DRAINAGE

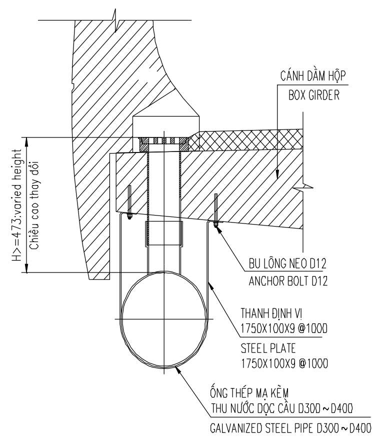
P%= 10% T(min)= 15

Inlet name	Distance	Accumulated distance	Area: Surface	Area: Accumulated area	Runoff coefficient	Rainfall intensity	Runoff flow	Longitudinal grade of bridge (i1)	Longitudinal grade of invert of pipe (i2)	Capacity flow (at 90%pipe)	Proposed pipe size	Conclusion	Remark	H: height of vertical pipe (varied height)
	(m2)	(m)	(m2)	(m2)		(mm/hr)	(m3/s)			(m3/s)	(mm)			(m)
I'24	10	240	125	3000	0.95	156	0.124	0.0159	0.0159	0.3027	D400	OK	i1=i2	0.660
I'25	10	250	125	3125	0.95	156	0.129	0.0159	0.0159	0.3027	D400	OK	i1=i2	0.661
I'26	10	260	125	3250	0.95	156	0.134	0.0159	0.0159	0.3027	D400	OK	i1=i2	0.661
I'27	10	270	125	3375	0.95	156	0.139	0.0159	0.0159	0.3027	D400	OK	i1=i2	0.662
I'28	10	280	125	3500	0.95	156	0.144	0.0159	0.0159	0.3027	D400	OK	i1=i2	0.662
I'29	10	290	125	3625	0.95	156	0.149	0.0159	0.0159	0.3027	D400	OK	i1=i2	0.663
I'30	10	300	125	3750	0.95	156	0.154	0.0159	0.0159	0.3027	D400	OK	i1=i2	0.663
I'31	10	310	125	3875	0.95	156	0.160	0.0190	0.0190	0.3309	D400	OK	i1=i2	0.663
I'32	10	320	125	4000	0.95	156	0.165	0.0190	0.0190	0.3309	D400	OK	i1=i2	0.663
I'33	10	330	125	4125	0.95	156	0.170	0.0190	0.0190	0.3309	D400	OK	i1=i2	0.663
I'34	10	340	125	4250	0.95	156	0.175	0.0190	0.0190	0.3309	D400	OK	i1=i2	0.663
I'35	12	352	150	4400	0.95	156	0.181	0.0190	0.0190	0.3309	D400	OK	i1=i2	0.663
I'36	12	364	150	4550	0.95	156	0.187	0.0190	0.0190	0.3309	D400	OK	i1=i2	0.663

TẠI NHỊP SUPPER "T"/ AT SUPPER "T" SPAN



TẠI DẦM HỘP/ AT BOX GIRDER



4. TEMPORARY BRIDGE
4. THIẾT KẾ CẦU TẠM

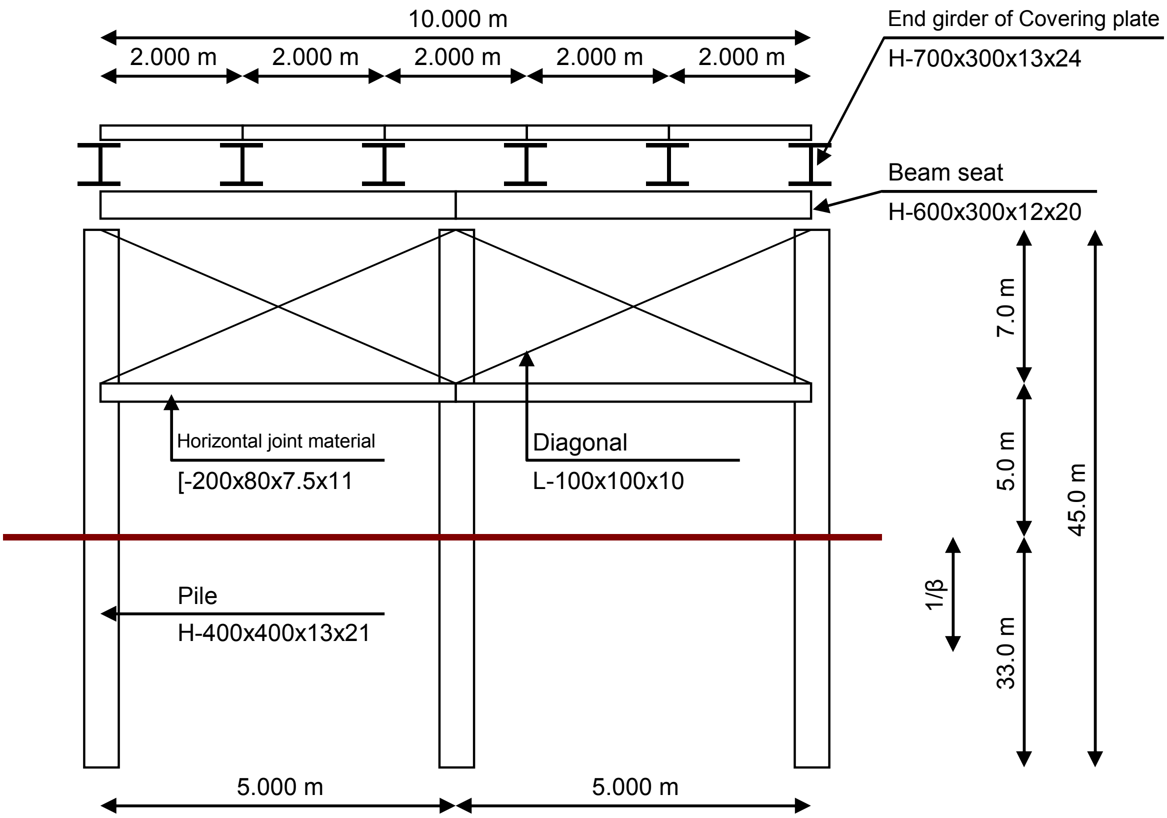
Calculation of Temporary Bridge (End girder of the Covering plate is parallel to running of the vehicle)

1. Design condition

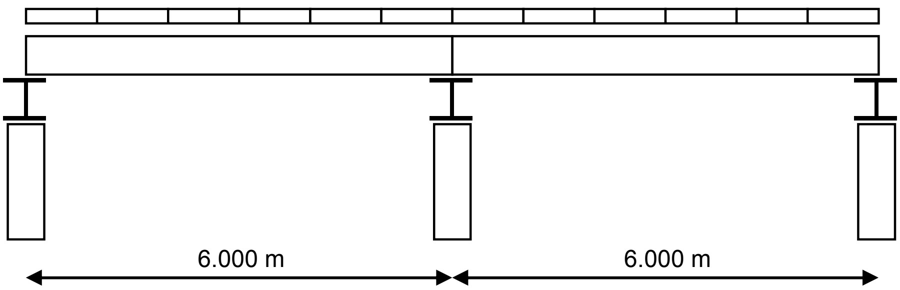
1.1. Form of Temporary Bridge

(1) Transverse width	L1=	10.0	m	
(2) Span of longitudinal direction	L2=	6.0	m	(Maximum)

Transverse direction Bridge



Bridge axis direction



1.2. Superstructure

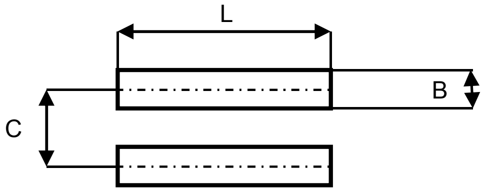
1.2.1. Loading condition

(1) The Live load of cars T-25 (the Live load of B)

(2) The Live load of Heavy machines (Crawler crane)

1) Weight of vehicle	W=	480	kN	(50t Crawler crane)
2) Lifting load	T=	150	kN	(Assumes the pile driving of Riverse circulation)
3) Total		630	kN	

4) Crawler dimensions



L=	4.270	m
C=	3.800	m
B=	0.800	m

5) Work assignment rate

･Said lifting work assignment rate	:	0.75
･Diagonal lifting work assignment rate	:	0.70
･Diagonal lifting work Ground rate	:	0.90

6) When the crawler crane work, the concept of load distribution state

☐ Calculated as a distributed load

■ Calculated as a line load

7) Impact coefficient i = 0.3

1.2.2. Covering Plate

1) Covering Plate type 1000*2000

2) Weight 2.00 kN/m2

1.2.3. End girder of Covering plate

Materials H-700x300x13x24

1.2.4. Beam seat

Materials H-600x300x12x20

Computation span of Beam seat 5.00 m

1.3. Substructure

1.3.1. Materials

1) Direction perpendicular to the number of columns Pile

n= 3 columns

2) Pile H-400x400x13x21

3) Diagonal L-100x100x10

4) Horizontal joint material [-200x80x7.5x11

1.3.2. Installation methods (for Pile) Vibration Method

2. Calculation of Superstructure

2.1. Calculation of End girder of the Covering plate

2.1.1. Section force of the Dead load

(1) The Dead load

Covering Plate	2.00 kN/m2	*	2.00 m			=	4.00 kN/m
End girder	182 kN/m	*	0.009807 SI	*	1.00 m	=	1.78 kN/m
							5.78 kN/m

(2) Calculation of the section force

1) Bending moment

$$M= \frac{w\lambda^2}{8} = \frac{5.78 \quad * \quad 6.0^{\ 2}}{8} = 26 \text{ kN}$$

2) Shear force

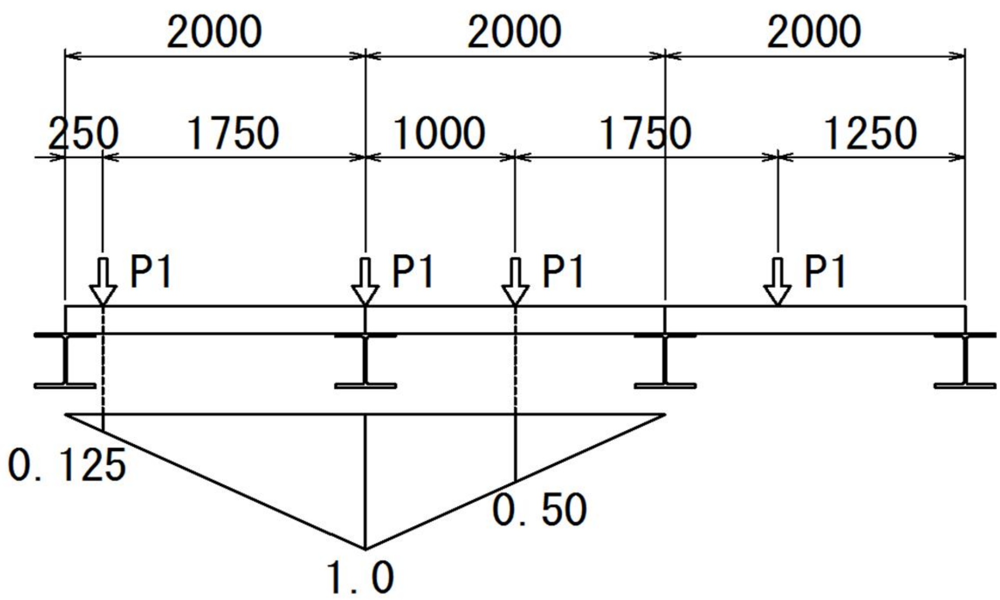
$$S= \frac{w\lambda}{2} = \frac{5.78 \quad * \quad 6.0}{2} = 17.4 \text{ kN}$$

2.1.2. Section force of the Live load T

(1) The Live load "T-25"

Maximum reaction force that is the Live load T for End girder is calculated by the influence line.
(Loading two to the Live load T.)

The Live load T P1= 100 kN



Σ= 0.125 + 1.000 + 0.50 = 1.625

/ Maximum reaction force "P" by the Live load is to consider the impact.

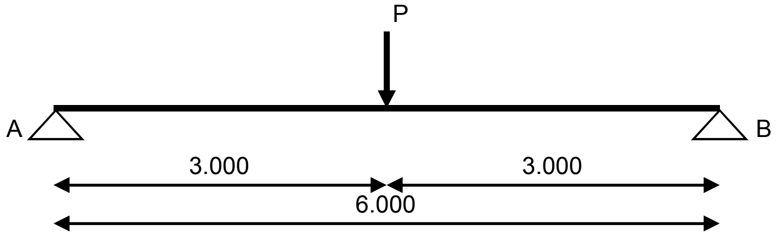
P = (1+i)*P1*Σy = (1+0.3)*100*1.625 = 211.3 kN

/ For consider the effects (L<4.0m) of load taken by the Live load "B", multiplying the section force coefficients are shown below.

$$\alpha = \frac{L}{32} + \frac{7}{8} = \frac{6}{32} + \frac{7}{8} = 1.063$$

(2) Calculation of the section force

1) Bending moment

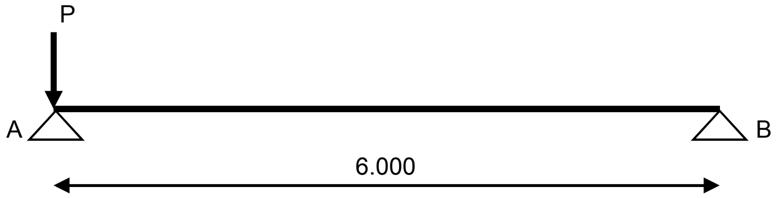


RA = 1/2*P = 1/2 * 211.3 = 105.7 kN

M = RA * 3.00 * α

= 105.7 * 3.00 * 1.063 = 337.1 kNm

2) Shear force



S = RA * α = 105.7 * 1.063 = 112.4 kN

2.1.3 Section force of Heavy machine work load

When the Heavy machine to work, be considered as a line load that are own weight and lifting load. And to consider each type that are Said lifting work and Diagonal lifting work.

(1) Ground pressure of Side lifting work

$$q1 = \frac{0.75 * w}{L*B} * 1.3 = \frac{0.75 \quad * 630}{4.27 \quad * 0.80} * 1.3 = 179.82 \quad kN/m2$$

$$q2 = \frac{0.25 * w}{L*B} * 1.3 = \frac{0.25 \quad * 630}{4.27 \quad * 0.80} * 1.3 = 59.94 \quad kN/m2$$

(2) Ground pressure of Diagonal lifting work

$$q1 = \frac{0.7 * w}{0.9 *L \quad *B*1/2} * 1.3 = \frac{0.7 \quad * 630}{0.90* \quad 4.27* \quad 0.80/2} * 1.3 = 373.0 \quad kN/m2$$

$$q2 = \frac{0.3 * w}{0.9 *L \quad *B*1/2} * 1.3 = \frac{0.3 \quad * 630}{0.90* \quad 4.27* \quad 0.80/2} * 1.3 = 159.8 \quad kN/m2$$

2.1.3.1. Side lifting work (When the parallel End girder and Heavy machine)

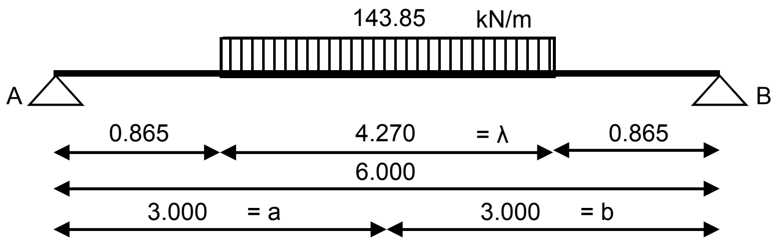
(1) The load when working

$$q = \frac{0.75 * w}{L} * 1.3 = \frac{0.75 \quad * 630}{4.270} * 1.3 = 143.85 \quad kN/m$$

(2) The load when working

To be loaded for the maximum section force, and to calculate the section force.

1) Bending moment



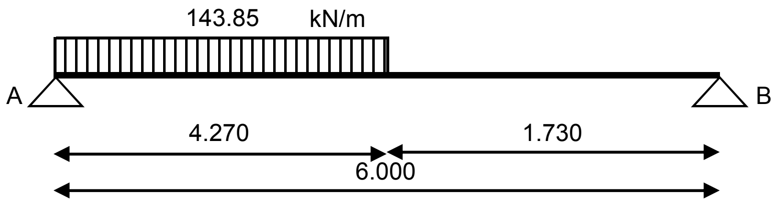
$$RA = 1/2*q*L = 1/2 \quad * \quad 143.9 \quad * \quad 4.27 = 307.1 \quad kN$$

$$M_{max} = \frac{ab\lambda}{l^2} q(l-\lambda/2)$$

$$= \frac{3.00 \quad * \quad 3.00 \quad * \quad 4.27 \quad * \quad 143.9 \quad * \quad (6.00 \quad -4.27 \quad / 2)}{6.000^{2}}$$

$$= 593.5 \quad kNm$$

2) Shear force



$$S = RA = \frac{143.9 \quad * \quad 4.27 \quad * (\quad 6.00 \quad -4.27 \quad / 2)}{6.00} = 395.7 \quad kN$$

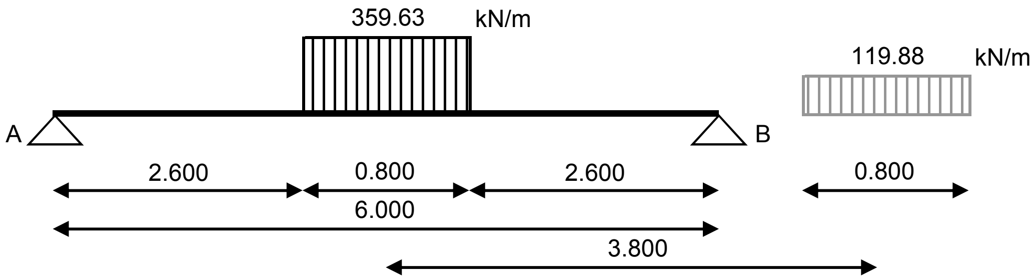
2.1.3.2. Side lifting work (When the orthogonal End girder and Heavy machine)

(1) The load when working

q1' =	1/2*q1	*	4.00	=	1/2 *	179.8	*	4.00	=	359.63	kN/m
q2' =	1/2*q2	*	4.00	=	1/2 *	59.94	*	4.00	=	119.88	kN/m

(2) The load when working

1) Bending moment



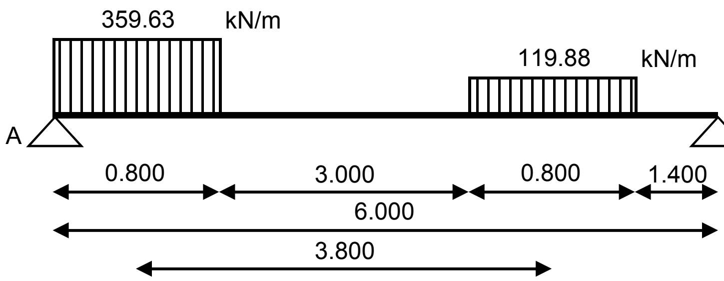
RA = 1/2*q*L = 1/2 * 359.6 * 0.80 = 143.9 kN

$$M_{max} = \frac{ab\lambda}{l^2} q(l-\lambda/2)$$

$$= \frac{3.00 * 3.00 * 0.80 * 359.6 * (6.00 - 0.80 / 2)}{6.000^2}$$

= 402.8 kNm

2) Shear force



$$S = RA = \frac{359.6 * 0.80 * 5.60 + 119.9 * 0.80 * 1.80}{6.00}$$

= 297.3 kN

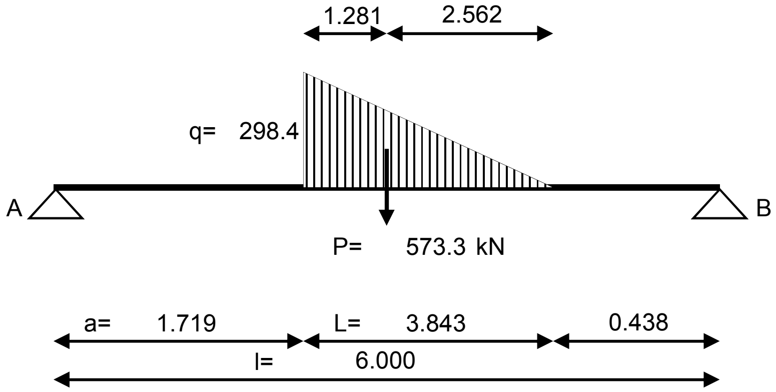
2.1.3.3. Diagonal lifting work (When the parallel End girder and Heavy machine)

(1) The load when working

$$q = \frac{0.7 * w}{0.9 * L * 1/2} * 1.3 = \frac{0.7 * 630}{0.9 * 4.270 * 1/2} * 1.3 = 298.36 \text{ kN/m}$$

(2) The load when working

1) Bending moment



RA = RB = 573.3 * (2.562 + 0.438) / 6.00 = 286.7 kN

Shear force is zero position (x) from the position of the load change 0.

x = √(2*S*L/q) = √(2 * 286.7 * 3.843 / 298.4)

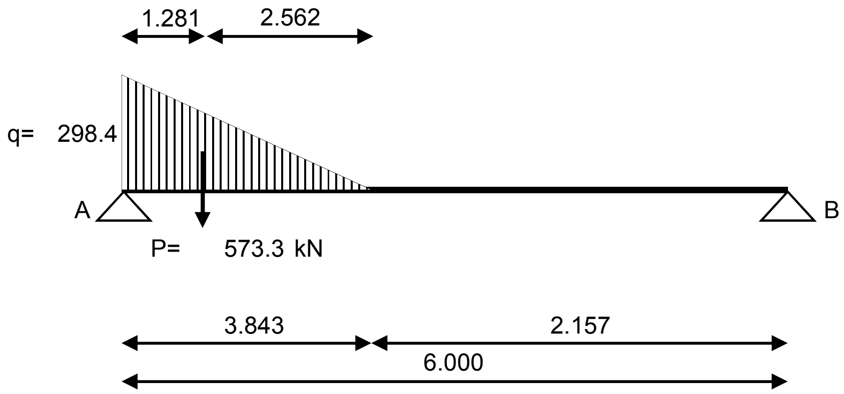
= 2.717 m

M = RB*(l-L-a+x)-q*x^3/6L

286.7 * (6.00 -3.84 -1.72 +2.72)

-298.4 * 2.72^3 / (6 * 3.843) = 644.8 kNm

2) Shear force



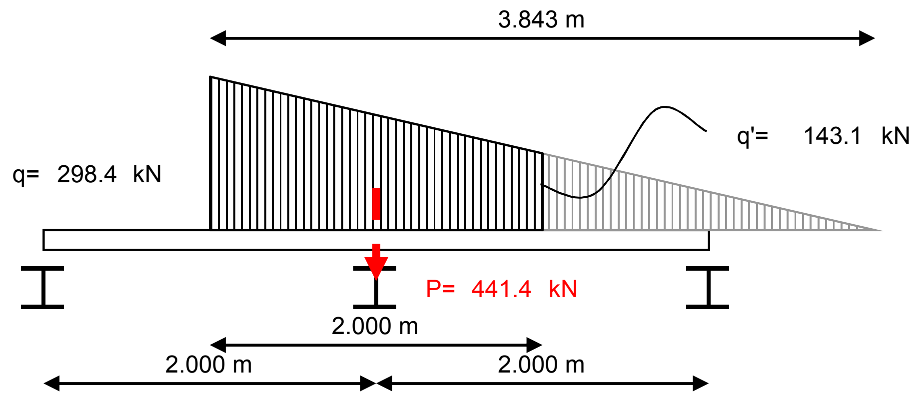
S = RA = 573.3 * 4.719 / 6.00 = 450.9 kN

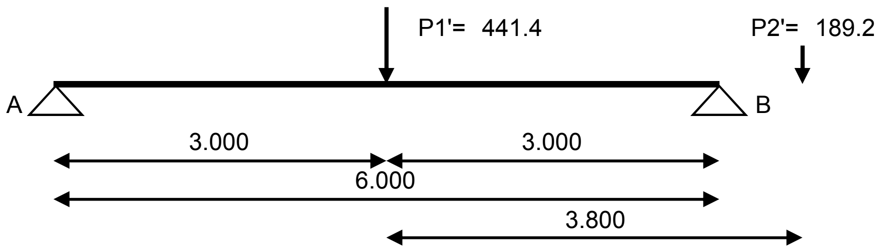
2.1.3.4. Diagonal lifting work (When the orthogonal End girder and Heavy machine)

(1) The load when working

1) Bending moment

· Calculation of indirect load beam

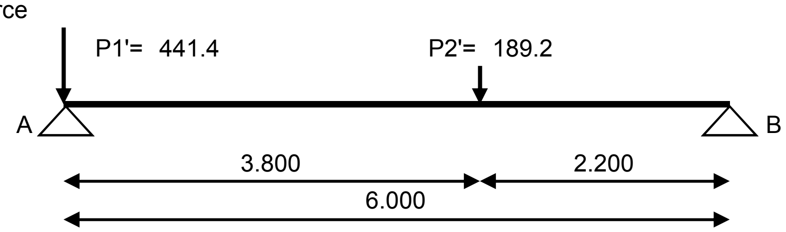




$$RA = 441.4 \ * \ 3.000 \ / \ 6.00 = 220.7 \text{ kN}$$

$$M = 220.7 \ * \ 3.00 = 662.2 \text{ kNm}$$

2) Shear force



$$S = RA = \frac{441.4 \ * \ 6.00 + 189.2 \ * \ 2.20}{6.00}$$

$$= 510.8 \text{ kN}$$

2.1.4. Aggregate section force

	Bending moment (kN·m)				Shear force (kN)			
	Dead load	Live load	Heavy machine	Total	Dead load	Live load	Heavy machine	Total
The Live load "T-25"	26	337.1		363.1	17.4	112.4		129.8
Side lifting work (1)	26		593.5	619.5	17.4		395.7	413.1
Side lifting work (2)	26		402.8	428.8	17.4		297.3	314.7
Diagonal lifting work (1)	26		644.8	670.8	17.4		450.9	468.3
Diagonal lifting work (2)	26		662.2	**688.2**	17.4		510.8	**528.2**

In the design calculation is used to force the maximum section force

M_{MAX}= 688.2 kN·m

S_{MAX}= 528.2 kN

2.1.5. Calculation of stress intensity

(1) Specifications of the steel used

End girder of Covering plate H-700x300x13x24

Modulus of section Z= 5640 cm3

Moment of ineria I = 197000 cm4

(2) Allowable bending unit stress

·Distance between the fixed flange L = λ = 600 cm

·Flange width B = 30 cm

$$\frac{\lambda}{b} = \frac{600}{30} = 20.0$$

$4.5 < \lambda/b \leq 30$ → $\sigma bca=[140-2.4(\lambda/b-4.5)]*1.5$(N/mm2)

σ_{bca}= 154.2 N/mm2

(3) Flexural stress

$$\sigma_{bc} = \frac{M_{MAX}}{Z} = \frac{688.2 \quad *10^{6}}{5640 \quad *10^{3}} = 122 \text{ N/mm2} \leq 154.2 \text{ N/mm2}$$

OK

(4) Shear force

$$\tau = \frac{S_{MAX}}{Aw} = \frac{528.2 \quad *10^{3}}{8476} = 62.32 \text{ N/mm2} \leq 120 \text{ N/mm2}$$

OK

Aw : Web area

Aw = tw*(h-2t) = 13 * (700- 2*24) = 8476 mm2

2.1.6. Calculation of the deflection by heavy machinery

(1) Calculation of uniform load

・Dead load does not take into account the impact

$$M_{MAX} = \frac{M_{MAX}}{1+i} = \frac{688.2}{1.3} = 529.4 \text{ kN·m}$$

・Equivalent uniform load

$$wo = \frac{8*M_{MAX}}{L^2} = \frac{8 * 529.4}{6.00^2} = 117.6 \text{ kN/m}$$

(2) Calculation of the deflection

$$\delta = \frac{5*wo*\lambda^4}{384EI} = \frac{5 * 117.6 * 6000^4}{384*2.0*10^5* 1.97E+09} = 5.0\text{mm} \leq 25\text{mm}$$

OK

$$\frac{\sigma_{MAX}}{L} = \frac{5.0}{6000} = \frac{1}{1191} < \frac{1}{400}$$ OK

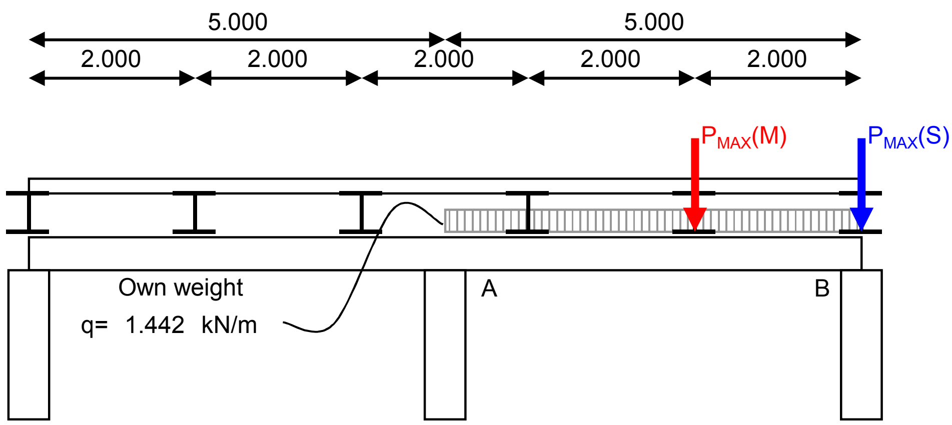
2.2. Calculation of Beam seat

(1) Specifications of the steel used

Beam seat	H-600x300x12x20		
Own weight	w=	1.442	kN/m
Modulus of section	Z=	3890	cm3
Moment of ineria	I =	114000	cm4

(2) Calculation of the section force

P_{MAX}= 528.2 kN



1) Bending moment

·Bending moment by End girder

RA1= 528.2 * 2.00 m / 5.00 m = 211.3 kN

M1= 211.3 * 3.00 = 633.8 kN·m

·Bending moment ownself

$$M2 = \frac{ql^2}{2}\left(\frac{x}{l} - \frac{x^2}{l^2}\right)$$

$$= \frac{1.442 \quad {*5.00}^2}{2}\left(\frac{3.00}{5.00} - \frac{9.00}{25.00}\right) = 4.3 \text{ kN·m}$$

·Bending moment total

M1 + M2 = 633.8 + 4.3 = 638.1 kN·m

2) Shear force

·Shear force by End girder

P = S1 = = 528.2 kN

·Shear force ownself

S2 = 1/2ql= 1/2 * 1.442 * 5.00 = 3.6 kN

·Shear force total

S1 + S2 = 528.2 + 3.6 = 531.8 kN

(3) Allowable bending unit stress

・Distance between the fixed flange L = λ = 500 cm

・Flange width B = 30 cm

$$\frac{\lambda}{b} = \frac{500}{30} = 16.7$$

$4.5 < \lambda/b \leq 30$ → $\sigma bca=[140-2.4(\lambda/b-4.5)]*1.5(N/mm2)$

$\sigma_{bca}=$ 166.2 N/mm2

(4) Flexural stress

$$\sigma_{bc} = \frac{M_{MAX}}{Z} = \frac{638.1 \quad *10^{6}}{3890 \quad *10^{3}} = 164 \text{ N/mm2} \leq 166.2 \text{ N/mm2}$$

OK

(5) Shear force

$$\tau = \frac{S_{MAX}}{Aw} = \frac{531.8 \quad *10^{3}}{6720} = 79.14 \text{ N/mm2} \leq 120 \text{ N/mm2}$$

OK

Aw : Web area

Aw = tw*(h-2t) = 12 * (600- 2*20) = 6720 mm2

3. Calculation of Substructure

3.1. Calculation of weight

3.1.1. Section force of the Dead load

covering plate	2.000	kN/m2	*	2.00	m *	6.00m	=	24.00	kN
end girder of covering plate	1.785	kN/m	*	6.00	m		=	10.71	kN
beam seat	1.442	kN/m	*	5.00	m		=	7.21	kN
diagonal	0.146	kN/m	*	3.14	m *	2 本	=	0.92	kN
horizontal joint material	0.241	kN/m	*	5.00	m *	2 本	=	2.41	kN
H-pile	1.687	kN/m	*	45.0	m		=	75.90	kN
							N1=	121.15	kN

3.1.2. Vertical force by live load

N2= 528.2 kN

3.1.3. Total of Vertical force

N1 + N2 = 121.2 + 528.2 = 649.4 kN

3.1.4. Calculation of allowable bearing capacity

Standard of bearing pile	H-400x400	A=	0.160	m2	9
Penetration length of the pile	33.000	U=	1.600	m	

Layer	Soil	Thickness	Depth	N	N'	N"	α	qd	qd·A	β	fi	li	U·fi·li
The ground surface			0.000										
2a	sand	7.900	- -7.900	8	8	8	1.0	1600	-	0.9	14.400	- 7.900	- 182.016
tk2	clay	3.900	- -11.800	5	5	5	1.0	1000	-	0.9	4.500	- 3.900	- 28.080
2b	sand	29.200	-33.000 -41.000	18	18	18	1.0	3600	3.392	0.9	32.400	21.200 -	1,099.008 -
4c	clay	3.900	- -44.900	11	11	11	1.0	2200	-	0.9	9.900	- -	- -
5b	sand	1.900	- -46.800	60	40	50	1.0	8000	-	0.9	90.000	- -	- -
6	clay	6.400	- -53.200	60	40	15	1.0	8000	-	0.9	13.500	- -	- -
7a	sand	10.000	- -63.200	60	40	50	1.0	8000	-	0.9	90.000	- -	- -
								qd·A	3.392			ΣU·fi·li	1,309.104

Ra=1/2(qd·A+ΣU·fi·l = 656.2

3.1.5. Verification of bearing capacity

Ra= 656.2 kN ≥ 649.4 kN OK

3.2. Examination for horizontal load

Horizontal load of the pile is to calculate the weak axis direction of H-beam.

3.2.1. Horizontal force by the load T

Horizontal force by the load T is considered to be driving three vehicles on a temporary bridge.

H1 = [All live load] * 0.1

[All live load] = 3 * 2 * 100 + (L/32 + 7/8)

= 601.1 kN (L= 6.00 m)

H = 601.1 * 0.1 = 60.1 kN

Horizontal force of per a pile (N = 3)

H1 = H / N = 60.11 / 3 = 20.0 kN

3.2.2. Horizontal force by Heavy machine

Horizontal force by Heavy machine is considered to oneself.

H2' = [Live load of Heavy machine] * 0.15

= 630 * 0.15 = 94.5 kN

Horizontal force of per a pile (N = 3)

H2 = H / N = 94.5 / 3 = 31.5 kN

3.2.3. Maximum horizontal force of per a pile

Select the maximum value of the H1 and H2

H0 = 31.5 kN

3.2.4. Bending moment of pile

To calculate the bending moment of pile head and the underground section.

(1) Characteristic value of pile

$$\beta = \sqrt[4]{\frac{kH \cdot D}{4 \cdot E \cdot I}} = \sqrt[4]{\frac{29536 \quad * \quad 0.40}{4 * \quad 2.0E+08 \quad * \quad 2.240E-04}}$$

$$= 0.507 \ m^{-1}$$

(2) Calculation of horizontal subgrade reaction

E0 = 20000 kN/m3 (Measured value)

α = 1

D = 0.40 m

E = 2.0E+08 kN/m2

I = 2.240E-04 m4

kH0 = 1/0.3*α*E0 = 66666.67 kN/m3

$kH = kH0*(BH/0.3)\text{^}(-3/4) = 6.67E+04 \quad * (\quad 0.888 \ / \ 0.3 \ )^{-3/4}$

= 29536 kN/m3

$$BH = \sqrt{\frac{D}{\beta}} = \sqrt{\frac{0.40}{0.507}} = 0.888 \ m$$

(3) Calculation of bending moment

β·λ = 0.507 * 33.0 = 16.72 ≥ 2.5 λ : Penetration length of the pile

Calculation is performed as a pile of semi-infinite length.

·The bending moment of pile head

$$M0 = \frac{1+\beta h}{2\beta} \cdot H0 = \frac{1 + \quad 0.507 \quad * 5.0}{2 * 0.507} * 31.5 = 109.8 \ kN \cdot m$$

M0 : The bending moment of pile head

H0 : Maximum horizontal force of per a pile

h : Protrusion length of the pile 5.0 m

·The bending moment of the underground section.

$$Mm = \frac{H0}{2\beta} \cdot \sqrt{1+(\beta \cdot h)^2} \quad \exp\left(-\tan^{-1} \frac{1}{\beta h} \right)$$

$$= \frac{31.5}{2 * 0.507} \sqrt{1 + (\quad 0.507 \quad * 5.0 \quad)^2} *\exp\left(-\tan^{-1} \frac{1}{0.507 \quad * 5.0} \right)$$

$$= 58.1 \ kN \cdot m$$

·Select the maximum value of the M0 and Mm

M = 109.8 kN·m

3.2.5. Calculation of stress intensity

H-pile H-400x400x13x21

(1) Degree of bending compressive stress around the strong axis

$$\sigma_{bcy} = \frac{M}{Zx} = \frac{0.0 \quad *10^{6}}{3330 \quad *10^{3}} = 0.0 \text{ N/mm2}$$

(2) Degree of bending compressive stress around the weak axis

$$\sigma_{bcz} = \frac{M}{Zy} = \frac{109.8 \quad *10^{6}}{1120 \quad *10^{3}} = 98.1 \text{ N/mm2}$$

(3) Axial compressive stress

$$\sigma_{c} = \frac{N}{A} = \frac{649.4 \quad *10^{3}}{218.7 \quad *10^{2}} = 29.7 \text{ N/mm2}$$

(4) Calculation of stress intensity for buckling

Calculation of Length for buckling

$$L = H3+h3+1/\beta = 5.0 + 0.2 + 1 / 0.507 = 7.173 \text{ m}$$

$$= 7173 \text{ mm}$$

1) Stress intensity of buckling around weak axis

$$\sigma_{eaz} = \frac{1,200,000}{(L/rx)^2} = \frac{1,200,000}{(7173 \quad /101)^{2}} = 237.9 \text{ N/mm2}$$

2) Stress intensity of buckling around strong axis

$$\sigma_{eay} = \frac{1,200,000}{(L/ry)^2} = \frac{1,200,000}{(7173 \quad /175)^{2}} = 714.2 \text{ N/mm2}$$

3) Calculation of stress intensity allowable axial compression around weak axis

$$\frac{L}{rz} = \frac{7173}{101} = 71.02 \qquad 18 < L / r \leq 92$$

$$\sigma_{caz} = [140-0.82(L/r-18)]*1.5$$

$$= [140- \ 0.82*(\ 71.02 \ -18)] *1.5 = 144.8 \text{ N/mm2}$$

4) Calculation of stress intensity allowable axial compression around strong axis

$$\frac{L}{b} = \frac{7173}{400} = 17.93 \qquad 4.5 < L / r \leq 30$$

$$\sigma_{bagy} = [140-2.4(L/b-4.5)]*1.5$$

$$= [140- \ 2.4*(\ 17.93 \ -4.5)] *1.5 = 161.6 \text{ N/mm2}$$

(5) Verification of the member at the same time undergo axial compression and bending moment

・Verification formula 1

$$\frac{\sigma_c}{\sigma_{caz}} + \frac{\sigma_{bcy}}{\sigma_{bagy}(1-\sigma_c/\sigma_{eay})} + \frac{\sigma_{bcz}}{\sigma_{bao}(1-\sigma_c/\sigma_{eaz})}$$

$$= \frac{29.7}{144.8} + \frac{0}{161.6 \;\; * \, (1 - \;\; 29.7 \, / \;\; 714.2)} + \frac{98.1}{210 \;\; * \, (1 - \;\; 29.7 \, / \;\; 237.9)}$$

$$= 0.739 \quad \leq \quad 1 \qquad \text{OK}$$

・Verification formula 2

$$\sigma_c + \frac{\sigma_{bcy}}{(1-\sigma_c/\sigma_{eay})} + \frac{\sigma_{bcz}}{(1-\sigma_c/\sigma_{eaz})}$$

$$= 29.7 + \frac{0}{(1 - \;\; 29.7 \, / \;\; 714.2)} + \frac{98.1}{(1 - \;\; 29.7 \, / \;\; 237.9)}$$

$$= 141.7 \quad \leq \quad 210 \quad \text{N/mm2=δcaλ} \qquad \text{OK}$$

5. DESIGN PAVEMENT OF MAINROAD AND FRONTAGE ROAD

5. THIẾT KẾ KẾT CẤU MẶT ĐƯỜNG CHÍNH VÀ ĐƯỜNG GOM ĐẦU CẦU

PHỤ LỤC - BẢNG TÍNH KẾT CẤU MẶT ĐƯỜNG CAO TỐC

CÁC THÔNG SỐ TÍNH TOÁN

Tải trọng trục, P = 120 kN   Đường kính vệt bánh xe, D = 36 cm   Áp lực tính toán lên mặt đường, p= 0.60 Mpa

Lớp	Vật liệu	Ei (Mpa)			Chiều dày	Rku	C	j
		Tính trượt	Tính độ võng	Tính kéo uốn	(cm)	(Mpa)	(Mpa)	(độ)
5	Bê tông nhựa chặt loại 1 lớp trên	250	420	1800	5	2.60		
4	Bê tông nhựa chặt loại 1 lớp dưới	300	350	1600	7	2.2		
3	Đá dăm đen		350	1000	10	1.6		
2	CPĐD móng trên loại I (móng trên)		300		30			
1	CPĐD móng trên loại II (móng dưới)		250		36			
0	Nền đất K98		45				0.032	26
					88.00			

1. KIỂM TRA KẾT CẤU THEO TIÊU CHUẨN ĐỘ VÕNG ĐÀN HỒI

Công thức tính:

$$E_{ch} \geq K_{cd}^{dv} \times E_{yc} \quad = \quad \mathbf{220} \text{ Mpa}$$

Trong đú:

+ Eyc được xác định theo số trục xe 100 kN:
  được xác định theo lưu lượng xe: = **200** Mpa (Trị số theo lưu lượng xe dự báo)
+ K_{cd}^{dv} được xác định theo bảng 3-2: = **1.10** (ứng với độ tin cậy 0,9 theo bảng 3-3)

+ Xác định Mô đuyn đàn hồi chung trên mặt kết cấu mặt đường bằng cách quy đổi các lớp kết cấu áo đường hai lớp một lần lượt từ dưới lên:

$$\frac{E_2, h_2}{E_1, h_1} \rightarrow E_{tb}, h_{tb}$$

Với: $$E_{tb} = E_1 . \left[\frac{\left(1 + k.t^{1/3}\right)}{\left(1 + k\right)} \right]^3 \qquad k = \frac{h_1}{h_2} \qquad t = \frac{E_2}{E_1}$$

$$E_{ch} = 1{,}05 E_0 / \left[\left(\frac{\left(1 - E_0 / E_1\right)}{\sqrt{1 + 4(H/D)^2 (E_0/E_1)^{-0.67}}} \right) + (E_0/E_1) \right] Mpa$$

		Eyc = **200** Mpa		
Bê tông nhựa chặt loại 1 lớp trên	E5 =	420	Mpa	5 cm
Bê tông nhựa chặt loại 1 lớp dưới	E4 =	350	Mpa	7 cm
Đá dăm đen	E3 =	350	Mpa	10 cm
CPĐD móng trên loại I (móng trên)	E2 =	300	Mpa	30 cm
CPĐD móng trên loại II (móng dưới)	E1 =	250	Mpa	36 cm
	Eo ≥	45 Mpa		

Lop	Ei (Mpa)	t	hi (cm)	k	H' (cm)	Etb (Mpa)	D (cm)	H/D	β	Etbdc (Mpa)	Eo/Etbdc	Ech/Etbdc *Hình 3-1*	Ech (Mpa)
0	45												
1	250		36			250							
2	300	1.20	30	0.83	66	272							
3	350	1.29	10	0.15	76	282							
4	350	1.24	7	0.09	83	287							
5	420	1.46	5	0.06	88	294	36	2.44	1.24	364	0.12		**223**

\+ Tính toán kiểm tra đối chiếu bằng cách dùng biểu đồ H3-1 cho kết quả Ech= 221 Mpa (sai số 5%) là có thể chấp nhận được.

\+ Mô đun đàn hồi chung: Ech = **223** Mpa > **220** Mpa

==> Kết cấu đủ khả năng chịu tải theo tiêu chuẩn độ võng đàn hồi

<u>2. KIỂM TRA KẾT CẤU THEO TIÊU CHUẨN CHỊU KÉO UỐN</u>

Công thức kiểm tra:

$$\delta_{ku} \leq \frac{R_{tt}^{ku}}{K_{cd}^{ku}}$$

Trong đú:

\+ K_{cd}^{dv} được xác định theo bảng 3-7 = **1.10** (ứng với độ tin cậy 0,90 theo bảng 3-3)

\+ Xác định Mô đun đàn hồi trên mặt các lớp vật liệu

Lớp	Ei (Mpa)	t (E_2/E_1)	hi cm	k (h_2/h_1)	H (cm)	Etbi (Mpa)	D cm	H/D	β Bảng 3.6	Etbdc (Mpa)	Eo/Etbdc	Echm/Etbdc *Hõnh 3-1*	Echm (Mpa)
0	45												
1	250		36		36								
2	300	1.20	30	1.20	66	277	36	1.83	1.20	331	0.14	<u>0.66</u>	**219**
3	1000	3.62	10	0.15	76	368	36	2.11	1.22	448	0.10		**238**
4	1600	4.35	7	0.09	83	430	36	2.31	1.23	529	0.08		**276**

\+ Xác định ứng suất kéo uốn δ_{ku} lớn nhất phát sinh ở đáy lớp vật liệu liền khối theo công thức:

$$\delta_{ku} = \overline{\delta_{ku}} \times p \times k_b$$

Với: p = 0.60 Mpa

K_b = 0.85

\+ Trong đó $\overline{\delta_{ku}}$ được xác định theo Toán đồ Hình 3-5

2.1 Với lớp bê tông nhựa lớp trên

Ei (Mpa)	Echm (Mpa)	Ei/Echm	H/D	$\overline{\delta_{ku}}$	δ_{ku} (Mpa)
1800	276	6.53	0.14	1.70	**0.867**

+ Xác định cường độ chịu kéo uốn của vật liệu liền khối theo công thức:

$$R_{tt}^{ku} = k_1 \times k_2 \times R_{ku}$$

Trong đú k_1= $11.11 / N_e^{0.22}$ = 0.399 Với Ne = 3.7x10^6 (trục/làn)

k_2 = 1.0

-> R_{tt}^{ku} =1.037 Mpa

-> $\frac{R_{tt}^{ku}}{K_{cd}^{ku}}$ = 0.943 Mpa

So sánh δ_{ku} = 0.867 Mpa < 0.943 Mpa

==> Lớp bê tông nhựa lớp trên đảm bảo điều kiện chịu kéo khi uốn

2.2 Với lớp bê tông nhựa lớp dưới

+ Xác định Môđun đàn hồi chung của 2 lớp bê tông nhựa: $H=h_1 + h_2$ = 12 cm $Ec = \frac{\sum E_i \times h_i}{H}$ = 1683 Mpa

Ec (Mpa)	Echm (Mpa)	Ec/Ecm	H/D	$\overline{\delta_{ku}}$	δ_{ku} (Mpa)
1683	238	7.07	0.33	1.58	**0.806**

+ Xác định cường độ chịu kéo uốn của vật liệu liền khối theo công thức sau:

$$R_{tt}^{ku} = k_1 \times k_2 \times R_{ku}$$

Trong đú k_1= $11.11 / N_e^{0.22}$ = 0.399 Với Ne = 3.7x10^6 (trục/làn)

$k_2 =$ 1.0 $R_{tt}^{ku} =$ 0.889 Mpa $\dfrac{R_{tt}^{ku}}{K_{cd}^{ku}} =$ 0.81 Mpa

So sỏnh δ_{ku} = 0.806 Mpa < 0.808 Mpa

==> Lớp bê tông nhựa lớp dưới đảm bảo điều chịu chịu kéo uốn

2.2 Với lớp đá dăm đen

+ Xác định Môđun đàn hồi chung của 2 lớp bê tông nhựa: $H=h_1+h_2+h_3=$ 22 cm $Ec = \dfrac{\sum E_i \times h_i}{H} =$ 1373 Mpa

Ec (Mpa)	Echm (Mpa)	Ec/Ecm	H/D	$\overline{\delta_{ku}}$	δ_{ku} (Mpa)
1373	276	4.98	0.61	0.95	**0.485**

+ Xác định cường độ chịu kéo uốn của vật liệu liền khối theo công thức sau:

$$R_{tt}^{ku} = k_1 \times k_2 \times R_{ku}$$

Trong đú $k_1 =$ $11.11 / N_e^{0.22}$ = 0.399 Với Ne = 3.7x10^6 (trục/làn)

$k_2 =$ 1.0 $R_{tt}^{ku} =$ 0.638 Mpa $\dfrac{R_{tt}^{ku}}{K_{cd}^{ku}} =$ 0.58 Mpa

So sỏnh δ_{ku} = 0.485 Mpa < 0.580 Mpa

==> Lớp cấp phối gia cố xi măng đảm bảo điều chịu chịu kéo uốn

<u>3. KIỂM TRA KẾT CẤU THEO TIÊU CHUẨN CHỊU CẮT TRƯỢT</u>

Công thức tính $$T_{ax} + T_{av} \le \frac{C_{tt}}{K_{cd}^{tr}}$$

Trong đó:

+ K_{cd}^{tr} được xác định theo bảng 3-7 = **0.94** (ứng với độ tin cậy 0,90 theo bảng 3-3)

+ Tính Etb của hệ kết cấu

Lớp	Etr (Mpa)	t	hi (cm)	k	H' (cm)	Etb' (Mpa)	D (cm)	H/D	β	Etbdc (Mpa)
0	45									
1	250		36		36					
2	300	1.20	30	1.20	66	277				
3	300	1.08	10	0.15	76	303				
4	300	0.99	7	0.09	83	303	36			
5	250	0.83	5	0.06	88	300	36	2.44	1.24	372

+ Xác định ứng suất cắt: $T = Tax + Tav$

Etbdc (Mpa)	Eo (Mpa)	Ei/Echm	H/D	T ax/p H3-3	T ax (Mpa)	T av H3-4	T (Mpa)
372	45	8.26	2.67	0.012	0.0072	-0.0027	0.005

+ Xác định Ctt

$$C_{tt} = C \times k_1 \times k_2 \times k_3 =$$ 0.019 Mpa

Trong đó:

k1 = 0.6

k2 = 0.65

k3 = 1.5

==> $\dfrac{C_{tt}}{K_{cd}^{tr}} =$ 0.020

T = T ax + T av = 0.005 Mpa < 0.020 Mpa

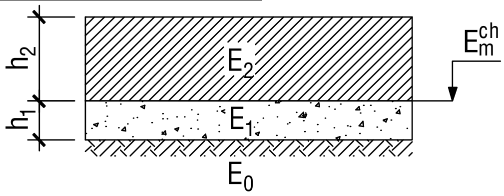
==> Nền đất bảo đảm điều kiện chống trượt

TÍNH TOÁN CƯỜNG ĐỘ MẶT ĐƯỜNG GOM - KẾT CẤU BTXM ĐỔ TẠI CHỖ

Công trình: Thiết kế kỹ thuật, đường cao tốc Đà Nẵng - Quảng Ngãi

Gói thầu 3A, Km16+880 - Km18+100

1/ Kết cấu và tham số tính toán :



Bản BTXM M300 dày 16 cm.
Cấp phối đá dăm loại 1: dày 12 cm.
Nền đất lẫn sỏi sạn K95

- **Mặt đường**	Bản BTXM không cốt thép, đổ tại chỗ, Mác BT:		300	
	Kích thước tấm :	L_b =	450	cm
		B_b =	350	cm
	Cường độ kéo khi uốn	R_{ku} =	40	daN/cm^2
	Cường độ chịu nén :	R_n =	300	daN/cm^2
	Moduyn đàn hồi :	E_b =	315000	daN/cm^2
- **Móng :**	Cấp phối đá dăm loại 1:	h_1 =	12	cm
	Moduyn đàn hồi :	E_1 =	3000	daN/cm^2
- **Nền đất** : Đất đồi sỏi sạn				
	Moduyn đàn hồi :	Eo =	450	daN/cm^2
- **Tải trọng :**		Xe 8T		
+ Tải trọng trục :		Pt =	5600	daN
+ Tải trọng bánh xe :		Pbx	2800	daN
+ Hệ số xung kích :		$1+\mu$	1.2	
+ Tải trọng tính toán :		P_{tt} =	3360	daN/cm^2
- Đường kính vệt bánh xe tính toán :		D =	33	cm
- Bán kính vệt bánh xe :		R =	16.5	cm
- Hệ số triết giảm cường độ :		n =	0.5	

2/ Tính chiều dày tấm bê tông:

- Giả định chiều dày tấm BTXM	h =	16	cm
D_m = D +h =		49	cm
- Mô duyn đàn hồi tương đương trên mặt lớp móng :			
h_m/D =		0.245	
E_0/E_1 =		0.15	
- Tra toán đồ hình 3-1 Tiêu chuẩn 22 TCN 211-06			
$E_{ch}{}^m / E_1$ =		0.203	
$E_{ch}{}^m$ =		609.9	daN/cm^2
- Xác định các hệ số theo vị trí đặt tải : $\alpha_1, \alpha_2, \alpha_3$	theo :		
h/R =		0.97	
$E_b / E_{ch}{}^m$ =		516	
- Từ cặp trị số trên tra bảng 4.1, 4.2, 4.3 :	α_1 =	1.088	
	α_2 =	1.560	
	α_3 =	1.624	
=>Tính chiều dày cho trường hợp tải trọng tác dụng ở góc tấm:	$\alpha 3$ =	1.624	

- Chiều dày tấm BTXM :

$$h_{tt} = \sqrt{\frac{\alpha.P_{II}}{[\sigma]}} = \quad 16.518 \quad cm$$

Trong đó : $[\sigma] =$ $n*R_{ku}$ Cường độ kéo uốn cho phép của bê tông.

- Sai số abs(h_{tt} - h)/h_{tt}*100 = 3.14% <= 5% - Đạt yêu cầu

Vậy chọn chiều dày bản BTXM h = 16 cm

3/ Kiểm toán tấm bê tông xi măng chịu tác dụng đồng thời của tải trọng và nhiệt độ :

- Chênh lệch nhiệt độ giữa mặt trên vàmặt dưới tấm BTXM tính theo công thức :

$\Delta\tau =$ 0.84*h = 13.44 ^{o}C

- Hệ số Poisson của BT : $\mu =$ 0.15
- Bán kính độ cứng của tấm BT :

$$l = 0,6h\sqrt[3]{\frac{E}{E^{m}_{cm}}} = 77.023$$

- Xác định tỷ số

Lb/l = 5.84

Bb/l = 4.54

- Dựa vào 2 tỷ số trên tra toán đồ hình 4.3 được :

Cx = 0.877

Cy = 0.572

Et = 0.6xEb = 189000 daN/cm2

$\alpha =$ 0.00001 1/^{o}C

- Ứng suất uốn vồng theo hướng dọc ở giữa tấm :

$$\sigma_t = \frac{E_t}{2(1-\mu^2)}(C_x + \mu.C_y).\Delta t = \quad 12.509 \quad daN/cm^2$$

- Ứng suất uốn vồng theo hướng ngang ở giữa tấm :

$$\sigma_n = \frac{E_t\alpha}{2(1-\mu^2)}(C_y + \mu.C_x).\Delta t = \quad 9.146 \quad daN/cm^2$$

- Ứng suất uốn vồng theo hướng dọc ở cạnh tấm :

$$\sigma_c = C_x.\Delta t\frac{E_t}{2(1-\mu^2)} = \quad 11.394 \quad daN/cm^2$$

- Ứng suất do tải trọng gây ra ở giữa tấm :

$$\delta_1 = \frac{\alpha_1 \times P_{tt}}{h^2} = \quad 14.284 \quad daN/cm2$$

- Ứng suất do tải trọng gây ra ở cạnh tấm :

$$\delta_2 = \frac{\alpha_2 \times P_{tt}}{h^2} = \quad 20.471 \quad daN/cm2$$

* Ứng suất tổng cộng do tải trọng và nhiệt độ cùng tác dụng gây ra ở mặt cắt giữa tấm theo hướng dọc :

$$\delta_I = \delta_t + \delta_1 = \quad 26.793 \quad daN/cm2$$

* Ứng suất tổng cộng do tải trọng và nhiệt độ cùng tác dụng gây ra ở cạnh tấm :

$$\delta_{II} = \delta_2 + \delta_c = \quad 31.865 \text{ daN/cm2}$$

Kiểm toán theo DeltaII

Trong trường hợp này :

$[\delta\ ]$ = 0.9R_{ku} = 36 daN/cm2

0.875 : hệ số triết giảm cường độ dưới tác dụng đồng thời của hoạt tải và ứng suất nhiệt

Ta có : δ_{II} = 31.865 daN/cm2 < 36 daN/cm2

Tấm BT làm việc an toàn dưới tác dụng tổng hợp của tải trọng và nhiệt độ

4/ Kiểm tra chiều dày lớp móng:

- Kiểm tra chiều dày lớp móng dưới mặt đường BTXM bảo đảm đất nền không bị biến dạng dẻo :

$$\tau_{am} + \tau_{ab} < K' K_1 C \leq [\tau_a]$$

Trong đó :

τ_{am} :		Ứng suất cắt hoạt động lớn nhất do hoạt tải gây ra.
τ_{ab} :		Ứng suất do tỉnh tải gây ra.
K' =	1.0	Khi lưu lượng xe nhỏ hơn 1000 xe/ngày đêm
K_1 =	0.65	Khi tấm BT liên kết bằng thanh truyền lực
C =	0.38	Lực dính tiêu chuẩn đất nền daN/cm2

- Vậy $[\tau_a]$ = K'.K.C = 0.25 daN/cm2
- Tính τ_{am} và τ_{ab}

với l = 77

φ = 15 ^{o}

Z = h1 = 12 cm

Chiều sâu từ mặt tiếp xúc :

h + z = 28 cm

Và P_{tt}/l^2 = 0.566 cm

Theo toán đồ hình 4.6 : τ_{am} = 0.077 daN/cm2

Theo toán đồ hình 4.7 : τ_{ab} = 0.005 daN/cm2

$\tau_{am} + \tau_{ab}$ = 0.082 daN/cm2

$$\tau_{am} + \tau_{ab} < [\tau_a]$$

Chiều dày lớp móng đã chọn đảm bảo không phát sinh biến dạng dẻo trong đất nền

6. APPENDIX: THE ESTIMATION OF SCOUR / PHỤ LỤC: TÍNH TOÁN XÓI

TABLE 1. DISTRIBUTION OF VELOCITY IN THE CROSS SECTION AT KY LAM BRIDGE

(Analysis condition: According to design frequency P1%)

$H_{1\%}$ = 9.20 m　　$Q_{L.chủ}$ = 9357 (m^3/s)

$Q_{1\%}$ = 10953 (m^3/s)　　$Q_{Bãi.1}$ = 1226 (m^3/s)　　$Q_{Bãi.2}$ = 370 (m^3/s)

Part	Pier Ab.	∇_{ground} (m)	h_i (m)	Δl_i (m)	$\Sigma\Delta l_i$ (m)	ω_t (m^2)	$\Sigma\omega_t$ (m^2)	$h^{5/3}$ ($m^{5/3}$)	Ai	$\alpha_{Velocity}$	q_i (m^3/sm)	Q_i (m^3/s)	ΣQ_i (m^3/s)	$V_{bq\ th.tr}$ (m/s)	Remarks
Plain on left side		5.22	3.98		0.00		0.00								Htb
		5.23	3.97	0.29	0.29	1.14	1.14	9.95	1.43		2.55	0.37	0.37	0.64	
	A1	5.39	3.81	4.31	4.60	16.78	17.93	9.29	41.52		2.38	10.64	11.01	0.63	
		5.53	3.67	9.78	14.38	36.57	54.50	8.73	88.14		2.24	22.59	33.60	0.61	
		5.48	3.72	5.25	19.63	19.40	73.90	8.93	46.36		2.29	11.88	45.49	0.62	
		2.32	6.88	7.41	27.04	39.25	113.15	24.89	125.23		6.38	32.10	77.59	0.93	
		2.40	6.80	1.73	28.76	11.80	124.95	24.41	42.53		6.26	10.90	88.49	0.92	
	P1	2.57	6.63	4.06	32.82	27.28	152.23	23.40	97.11		6.00	24.89	113.38	0.90	
		2.83	6.37	6.29	39.12	40.90	193.13	21.89	142.47		5.61	36.52	149.90	0.88	
		3.63	5.57	4.03	43.14	24.04	217.17	17.50	79.31		4.49	20.33	170.23	0.81	
		3.68	5.52	7.91	51.05	43.86	261.03	17.24	137.40		4.42	35.22	205.45	0.80	
		3.04	6.16	3.88	54.94	22.68	283.70	20.70	73.66		5.31	18.88	224.33	0.86	
		3.25	5.95	2.59	57.52	15.67	299.38	19.54	52.08		5.01	13.35	237.68	0.84	
	P2	3.32	5.88	4.06	61.59	24.03	323.41	19.16	78.60		4.91	20.15	257.83	0.84	
		3.38	5.82	3.27	64.86	19.14	342.55	18.83	62.14		4.83	15.93	273.76	0.83	
		3.91	5.29	1.37	66.23	7.59	350.14	16.06	23.83		4.12	6.11	279.86	0.78	
		4.02	5.18	5.68	71.91	29.74	379.88	15.51	89.66		3.98	22.98	302.85	0.77	
		4.18	5.02	14.38	86.29	73.34	453.22	14.72	217.34		3.77	55.71	358.56	0.75	
	P3	4.23	4.97	4.06	90.35	20.29	473.51	14.47	59.30		3.71	15.20	373.76	0.75	
		4.34	4.86	10.32	100.67	50.72	524.23	13.94	146.62		3.57	37.58	411.34	0.74	
		4.50	4.70	14.38	115.05	68.74	592.97	13.19	195.09		3.38	50.01	461.35	0.72	
	P4	4.55	4.65	4.06	119.11	18.99	611.96	12.95	53.10		3.32	13.61	474.96	0.71	
		4.57	4.63	1.98	121.09	9.18	621.14	12.86	25.52		3.30	6.54	481.50	0.71	
		0.66	8.54	6.18	127.27	40.72	661.86	35.68	150.09		9.15	38.47	519.97	1.07	
		0.83	8.37	2.16	129.43	18.24	680.10	34.50	75.70		8.84	19.40	539.38	1.06	
		0.91	8.29	14.38	143.81	119.79	799.89	33.96	492.27		8.70	126.18	665.56	1.05	
	P5	0.84	8.36	4.06	147.87	33.82	833.71	34.44	138.93		8.83	35.61	701.17	1.06	
		0.79	8.41	10.32	158.19	86.52	920.23	34.78	357.09		8.91	91.53	792.71	1.06	
		0.63	8.57	5.32	163.51	45.18	965.41	35.89	188.01		9.20	48.19	840.90	1.07	
		4.61	4.59	2.80	166.32	18.45	983.86	12.68	68.10		3.25	17.46	858.36	0.71	
		4.59	4.61	6.26	172.57	28.78	1012.64	12.77	79.59		3.27	20.40	878.76	0.71	
	P6	4.55	4.65	4.06	176.64	18.81	1031.45	12.95	52.25		3.32	13.39	892.15	0.71	
		4.54	4.66	1.62	178.25	7.53	1038.98	13.00	21.00		3.33	5.38	897.53	0.72	

BẢNG 1. PHÂN PHỐI LƯU TỐC DÒNG CHẢY TẠI MẶT CẮT NGANG CẦU KỲ LAM

(Điều kiện tính toán: ứng với tần suất P1%)

$H_{1\%}$ = 9.20 m $Q_{L.chủ}$ = 9357 (m^3/s)

$Q_{1\%}$ = 10953 (m^3/s) $Q_{Bãi.1}$ = 1226 (m^3/s) $Q_{Bãi.2}$ = 370 (m^3/s)

Bộ phận	Trụ Mố	$\nabla_{Tự\ nhiên}$ (m)	h_i (m)	Δl_i (m)	$\Sigma\Delta l_i$ (m)	ω_t (m^2)	$\Sigma\omega_t$ (m^2)	$h^{5/3}$ ($m^{5/3}$)	Ai	$\alpha_{Velocity}$	q_i (m^3/sm)	Q_i (m^3/s)	ΣQ_i (m^3/s)	$V_{bq\ th.tr}$ (m/s)	Ghi chú
Bãi trái		5.22	3.98		0.00		0.00								Htb
		5.23	3.97	0.29	0.29	1.14	1.14	9.95	1.43		2.55	0.37	0.37	0.64	
	A1	5.39	3.81	4.31	4.60	16.78	17.93	9.29	41.52		2.38	10.64	11.01	0.63	
		5.53	3.67	9.78	14.38	36.57	54.50	8.73	88.14		2.24	22.59	33.60	0.61	
		5.48	3.72	5.25	19.63	19.40	73.90	8.93	46.36		2.29	11.88	45.49	0.62	
		2.32	6.88	7.41	27.04	39.25	113.15	24.89	125.23		6.38	32.10	77.59	0.93	
		2.40	6.80	1.73	28.76	11.80	124.95	24.41	42.53		6.26	10.90	88.49	0.92	
	P1	2.57	6.63	4.06	32.82	27.28	152.23	23.40	97.11		6.00	24.89	113.38	0.90	
		2.83	6.37	6.29	39.12	40.90	193.13	21.89	142.47		5.61	36.52	149.90	0.88	
		3.63	5.57	4.03	43.14	24.04	217.17	17.50	79.31		4.49	20.33	170.23	0.81	
		3.68	5.52	7.91	51.05	43.86	261.03	17.24	137.40		4.42	35.22	205.45	0.80	
		3.04	6.16	3.88	54.94	22.68	283.70	20.70	73.66		5.31	18.88	224.33	0.86	
		3.25	5.95	2.59	57.52	15.67	299.38	19.54	52.08		5.01	13.35	237.68	0.84	
	P2	3.32	5.88	4.06	61.59	24.03	323.41	19.16	78.60		4.91	20.15	257.83	0.84	
		3.38	5.82	3.27	64.86	19.14	342.55	18.83	62.14		4.83	15.93	273.76	0.83	
		3.91	5.29	1.37	66.23	7.59	350.14	16.06	23.83		4.12	6.11	279.86	0.78	
		4.02	5.18	5.68	71.91	29.74	379.88	15.51	89.66		3.98	22.98	302.85	0.77	
		4.18	5.02	14.38	86.29	73.34	453.22	14.72	217.34		3.77	55.71	358.56	0.75	
	P3	4.23	4.97	4.06	90.35	20.29	473.51	14.47	59.30		3.71	15.20	373.76	0.75	
		4.34	4.86	10.32	100.67	50.72	524.23	13.94	146.62		3.57	37.58	411.34	0.74	
		4.50	4.70	14.38	115.05	68.74	592.97	13.19	195.09		3.38	50.01	461.35	0.72	
	P4	4.55	4.65	4.06	119.11	18.99	611.96	12.95	53.10		3.32	13.61	474.96	0.71	
		4.57	4.63	1.98	121.09	9.18	621.14	12.86	25.52		3.30	6.54	481.50	0.71	
		0.66	8.54	6.18	127.27	40.72	661.86	35.68	150.09		9.15	38.47	519.97	1.07	
		0.83	8.37	2.16	129.43	18.24	680.10	34.50	75.70		8.84	19.40	539.38	1.06	
		0.91	8.29	14.38	143.81	119.79	799.89	33.96	492.27		8.70	126.18	665.56	1.05	
	P5	0.84	8.36	4.06	147.87	33.82	833.71	34.44	138.93		8.83	35.61	701.17	1.06	
		0.79	8.41	10.32	158.19	86.52	920.23	34.78	357.09		8.91	91.53	792.71	1.06	
		0.63	8.57	5.32	163.51	45.18	965.41	35.89	188.01		9.20	48.19	840.90	1.07	
		4.61	4.59	2.80	166.32	18.45	983.86	12.68	68.10		3.25	17.46	858.36	0.71	
		4.59	4.61	6.26	172.57	28.78	1012.64	12.77	79.59		3.27	20.40	878.76	0.71	
	P6	4.55	4.65	4.06	176.64	18.81	1031.45	12.95	52.25		3.32	13.39	892.15	0.71	
		4.54	4.66	1.62	178.25	7.53	1038.98	13.00	21.00		3.33	5.38	897.53	0.72	

Part	Pier Ab.	∇_{ground} (m)	h_i (m)	Δl_i (m)	$\Sigma\Delta l_i$ (m)	ω_t (m^2)	$\Sigma\omega_t$ (m^2)	$h^{5/3}$ ($m^{5/3}$)	Ai	$\alpha_{Velocity}$	q_i (m^3/sm)	Q_i (m^3/s)	ΣQ_i (m^3/s)	$V_{bq\ th.tr}$ (m/s)	Remarks
		0.67	8.53	3.81	182.07	25.13	1064.11	35.61	92.63		9.13	23.74	921.28	1.07	
		1.06	8.14	4.89	186.95	40.75	1104.87	32.94	167.59		8.44	42.96	964.23	1.04	
		4.63	4.57	4.03	190.98	25.59	1130.46	12.59	91.65		3.23	23.49	987.73	0.71	
		4.69	4.51	10.35	201.34	47.01	1177.47	12.31	128.89		3.16	33.04	1020.77	0.70	
	P7	4.71	4.49	4.06	205.40	18.28	1195.75	12.22	49.83		3.13	12.77	1033.54	0.70	
		4.71	4.49	10.32	215.72	46.33	1242.08	12.22	126.09		3.13	32.32	1065.86	0.70	
		4.61	4.59	14.38	230.10	65.29	1307.37	12.68	179.02		3.25	45.89	1111.75	0.71	
	P8	4.56	4.64	3.34	233.44	15.43	1322.80	12.91	42.77		3.31	10.96	1122.72	0.71	
		4.42	4.78	10.32	243.76	48.60	1371.40	13.56	136.57		3.48	35.01	1157.72	0.73	
		4.15	5.05	14.38	258.14	70.68	1442.08	14.86	204.41		3.81	52.40	1210.12	0.75	
	P9	4.08	5.12	4.06	262.20	20.66	1462.74	15.21	61.09		3.90	15.66	1225.78	0.76	
				262.20		**1462.74**			**4782.03**	**0.2563**		**1225.78**		**0.84**	5.58
Main channal		4.08	5.12		0.00		0.00								
		3.89	5.31	10.32	10.32	53.81	53.81	16.16	83.38		10.40	53.66	53.7	1.96	
		3.73	5.47	14.38	24.70	77.51	131.33	16.98	238.32		10.93	153.36	207.0	2.00	
	P10	3.69	5.51	3.52	28.22	19.34	150.67	17.19	60.20		11.06	38.74	245.7	2.01	
		3.56	5.64	10.86	39.08	60.53	211.20	17.87	190.33		11.50	122.48	368.2	2.04	
		3.37	5.83	14.38	53.46	82.48	293.68	18.88	264.29		12.15	170.07	538.3	2.08	
		3.20	6.00	14.38	67.84	85.06	378.74	19.81	278.25		12.75	179.05	717.3	2.12	
	P11	3.09	6.11	7.19	75.03	43.54	422.28	20.42	144.64		13.14	93.08	810.4	2.15	
		3.03	6.17	7.19	82.22	44.15	466.43	20.76	148.04		13.36	95.26	905.7	2.16	
		2.87	6.33	14.38	96.61	89.88	556.31	21.66	305.00		13.94	196.26	1101.9	2.20	
		2.69	6.51	14.38	110.99	92.33	648.64	22.70	318.95		14.61	205.25	1307.2	2.24	
		2.52	6.68	14.38	125.37	94.84	743.48	23.69	333.57		15.25	214.65	1521.8	2.28	
		2.35	6.85	14.38	139.75	97.29	840.77	24.71	348.02		15.90	223.95	1745.8	2.32	
	P12	2.24	6.96	7.19	146.94	49.65	890.42	25.37	180.04		16.33	115.86	1861.7	2.35	
		2.17	7.03	7.19	154.13	50.30	940.72	25.80	183.97		16.60	118.38	1980.0	2.36	
		2.00	7.20	14.38	168.51	102.32	1043.04	26.85	378.54		17.28	243.59	2223.6	2.40	
		1.67	7.53	14.38	182.89	105.92	1148.96	28.93	401.05		18.62	258.07	2481.7	2.47	
		1.40	7.80	14.38	197.27	110.23	1259.19	30.68	428.60		19.74	275.80	2757.5	2.53	
		1.17	8.03	14.38	211.65	113.83	1373.02	32.20	452.12		20.72	290.94	3048.4	2.58	
	P13	1.08	8.12	7.19	218.85	58.06	1431.08	32.80	233.71		21.11	150.39	3198.8	2.60	
		0.99	8.21	7.19	226.04	58.71	1489.79	33.41	238.06		21.50	153.19	3352.0	2.62	
		0.55	8.65	14.38	240.42	121.23	1611.02	36.45	502.34		23.45	323.25	3675.3	2.71	
		-0.41	9.61	14.38	254.80	131.30	1742.32	43.44	574.43		27.95	369.64	4044.9	2.91	
		-0.70	9.90	14.38	269.18	140.29	1882.61	45.64	640.55		29.37	412.19	4457.1	2.97	

Bộ phận	Trụ Mố	$\nabla_{Tự\ nhiên}$ (m)	h_i (m)	Δl_i (m)	$\Sigma\Delta l_i$ (m)	ω_t (m^2)	$\Sigma\omega_t$ (m^2)	$h^{5/3}$ ($m^{5/3}$)	Ai	$\alpha_{Velocity}$	q_i (m^3/sm)	Q_i (m^3/s)	ΣQ_i (m^3/s)	$V_{bq\ th.tr}$ (m/s)	Ghi chú
		0.67	8.53	3.81	182.07	25.13	1064.11	35.61	92.63		9.13	23.74	921.28	1.07	
		1.06	8.14	4.89	186.95	40.75	1104.87	32.94	167.59		8.44	42.96	964.23	1.04	
		4.63	4.57	4.03	190.98	25.59	1130.46	12.59	91.65		3.23	23.49	987.73	0.71	
		4.69	4.51	10.35	201.34	47.01	1177.47	12.31	128.89		3.16	33.04	1020.77	0.70	
	P7	4.71	4.49	4.06	205.40	18.28	1195.75	12.22	49.83		3.13	12.77	1033.54	0.70	
		4.71	4.49	10.32	215.72	46.33	1242.08	12.22	126.09		3.13	32.32	1065.86	0.70	
		4.61	4.59	14.38	230.10	65.29	1307.37	12.68	179.02		3.25	45.89	1111.75	0.71	
	P8	4.56	4.64	3.34	233.44	15.43	1322.80	12.91	42.77		3.31	10.96	1122.72	0.71	
		4.42	4.78	10.32	243.76	48.60	1371.40	13.56	136.57		3.48	35.01	1157.72	0.73	
		4.15	5.05	14.38	258.14	70.68	1442.08	14.86	204.41		3.81	52.40	1210.12	0.75	
	P9	4.08	5.12	4.06	262.20	20.66	1462.74	15.21	61.09		3.90	15.66	1225.78	0.76	
				262.20		**1462.74**			**4782.03**	**0.2563**		**1225.78**		**0.84**	5.58
Lòng chủ		4.08	5.12		0.00		0.00								
		3.89	5.31	10.32	10.32	53.81	53.81	16.16	83.38		10.40	53.66	53.7	1.96	
		3.73	5.47	14.38	24.70	77.51	131.33	16.98	238.32		10.93	153.36	207.0	2.00	
	P10	3.69	5.51	3.52	28.22	19.34	150.67	17.19	60.20		11.06	38.74	245.7	2.01	
		3.56	5.64	10.86	39.08	60.53	211.20	17.87	190.33		11.50	122.48	368.2	2.04	
		3.37	5.83	14.38	53.46	82.48	293.68	18.88	264.29		12.15	170.07	538.3	2.08	
		3.20	6.00	14.38	67.84	85.06	378.74	19.81	278.25		12.75	179.05	717.3	2.12	
	P11	3.09	6.11	7.19	75.03	43.54	422.28	20.42	144.64		13.14	93.08	810.4	2.15	
		3.03	6.17	7.19	82.22	44.15	466.43	20.76	148.04		13.36	95.26	905.7	2.16	
		2.87	6.33	14.38	96.61	89.88	556.31	21.66	305.00		13.94	196.26	1101.9	2.20	
		2.69	6.51	14.38	110.99	92.33	648.64	22.70	318.95		14.61	205.25	1307.2	2.24	
		2.52	6.68	14.38	125.37	94.84	743.48	23.69	333.57		15.25	214.65	1521.8	2.28	
		2.35	6.85	14.38	139.75	97.29	840.77	24.71	348.02		15.90	223.95	1745.8	2.32	
	P12	2.24	6.96	7.19	146.94	49.65	890.42	25.37	180.04		16.33	115.86	1861.7	2.35	
		2.17	7.03	7.19	154.13	50.30	940.72	25.80	183.97		16.60	118.38	1980.0	2.36	
		2.00	7.20	14.38	168.51	102.32	1043.04	26.85	378.54		17.28	243.59	2223.6	2.40	
		1.67	7.53	14.38	182.89	105.92	1148.96	28.93	401.05		18.62	258.07	2481.7	2.47	
		1.40	7.80	14.38	197.27	110.23	1259.19	30.68	428.60		19.74	275.80	2757.5	2.53	
		1.17	8.03	14.38	211.65	113.83	1373.02	32.20	452.12		20.72	290.94	3048.4	2.58	
	P13	1.08	8.12	7.19	218.85	58.06	1431.08	32.80	233.71		21.11	150.39	3198.8	2.60	
		0.99	8.21	7.19	226.04	58.71	1489.79	33.41	238.06		21.50	153.19	3352.0	2.62	
		0.55	8.65	14.38	240.42	121.23	1611.02	36.45	502.34		23.45	323.25	3675.3	2.71	
		-0.41	9.61	14.38	254.80	131.30	1742.32	43.44	574.43		27.95	369.64	4044.9	2.91	
		-0.70	9.90	14.38	269.18	140.29	1882.61	45.64	640.55		29.37	412.19	4457.1	2.97	

Part	Pier Ab.	∇_{ground} (m)	h_i (m)	Δl_i (m)	$\Sigma\Delta l_i$ (m)	ω_t (m^2)	$\Sigma\omega_t$ (m^2)	$h^{5/3}$ ($m^{5/3}$)	Ai	$\alpha_{Velocity}$	q_i (m^3/sm)	Q_i (m^3/s)	ΣQ_i (m^3/s)	$V_{bq\ th.tr}$ (m/s)	Remarks
	P14	-0.88	10.08	7.19	276.37	71.83	1954.45	47.04	333.21		30.27	214.42	4671.5	3.00	
		-0.94	10.14	7.19	283.56	72.70	2027.14	47.50	339.89		30.57	218.72	4890.2	3.01	
		-1.48	10.68	14.38	297.94	149.71	2176.85	51.79	714.00		33.33	459.46	5349.7	3.12	
		-2.05	11.25	14.38	312.32	157.69	2334.54	56.48	778.57		36.35	501.01	5850.7	3.23	
		-2.49	11.69	14.38	326.70	164.95	2499.49	60.21	839.10		38.75	539.96	6390.7	3.31	
		-3.34	12.54	14.38	341.08	174.23	2673.72	67.68	919.65		43.55	591.79	6982.5	3.47	
	P15	-3.68	12.88	7.19	348.28	91.39	2765.11	70.77	497.78		45.54	320.32	7302.8	3.54	
		-4.21	13.41	7.19	355.47	94.52	2859.63	75.69	526.57		48.71	338.84	7641.6	3.63	
		-5.54	14.74	11.86	367.33	166.99	3026.62	88.61	974.67		57.02	627.20	8268.8	3.87	
		-4.47	13.67	2.52	369.85	35.75	3062.37	78.15	209.85		50.29	135.04	8403.9	3.68	
		-0.80	10.00	14.38	384.23	170.20	3232.57	46.42	895.71		29.87	576.38	8980.2	2.99	
		-0.41	9.61	1.94	386.17	19.04	3251.61	43.44	87.22		27.95	56.13	9036.4	2.91	
		1.22	7.98	4.53	390.70	39.84	3291.45	31.87	170.57		20.51	109.76	9146.1	2.57	
		1.93	7.27	5.18	395.88	39.48	3330.93	27.28	153.11		17.56	98.53	9244.7	2.41	
		4.94	4.26	9.06	404.94	52.23	3383.16	11.19	174.30		7.20	112.16	9356.8	1.69	
				404.94		**3383.16**			**14540.61**	**0.643**		**9356.82**		**2.77**	8.35
Plain on right side		4.94	4.26												
		5.01	4.19	8.05	8.05	34.03	34.03	10.89	43.85		3.01	12.12	12.1	0.72	
	P16	4.97	4.23	7.19	15.24	30.27	64.30	11.06	78.93		3.06	21.82	33.9	0.72	
		4.91	4.29	7.19	22.43	30.63	94.93	11.33	80.50		3.13	22.25	56.2	0.73	
		4.55	4.65	14.38	36.82	64.28	159.21	12.95	174.59		3.58	48.27	104.5	0.77	
		4.05	5.15	14.38	51.20	70.47	229.68	15.36	203.58		4.25	56.28	160.7	0.82	
	A2	3.89	5.31	10.93	62.13	57.16	286.84	16.16	172.25		4.47	47.62	208.4	0.84	
		3.87	5.33	3.45	65.58	18.36	305.21	16.26	55.96		4.50	15.47	223.8	0.84	
		3.91	5.29	3.74	69.32	19.85	325.06	16.06	60.43		4.44	16.71	240.5	0.84	
		4.04	5.16	10.71	80.03	55.98	381.04	15.41	168.57		4.26	46.60	287.1	0.83	
		4.15	5.05	14.38	94.41	73.42	454.46	14.86	217.68		4.11	60.18	347.3	0.81	
		4.44	4.76	5.54	99.95	27.16	481.61	13.47	78.44		3.72	21.68	369.0	0.78	
		5.20	4.00	0.43	100.38	1.89	483.50	10.08	5.08		2.79	1.40	370.4	0.70	
				100.38		**483.50**			**1339.86**	**0.276**		**370.40**		**0.77**	4.82

Bộ phận	Trụ Mố	$\nabla_{Tự\ nhiên}$ (m)	h_i (m)	Δl_i (m)	$\Sigma\Delta l_i$ (m)	ω_t (m^2)	$\Sigma\omega_t$ (m^2)	$h^{5/3}$ ($m^{5/3}$)	Ai	$\alpha_{Velocity}$	q_i (m^3/sm)	Q_i (m^3/s)	ΣQ_i (m^3/s)	$V_{bq\ th.tr}$ (m/s)	Ghi chú
	P14	-0.88	10.08	7.19	276.37	71.83	1954.45	47.04	333.21		30.27	214.42	4671.5	3.00	
		-0.94	10.14	7.19	283.56	72.70	2027.14	47.50	339.89		30.57	218.72	4890.2	3.01	
		-1.48	10.68	14.38	297.94	149.71	2176.85	51.79	714.00		33.33	459.46	5349.7	3.12	
		-2.05	11.25	14.38	312.32	157.69	2334.54	56.48	778.57		36.35	501.01	5850.7	3.23	
		-2.49	11.69	14.38	326.70	164.95	2499.49	60.21	839.10		38.75	539.96	6390.7	3.31	
		-3.34	12.54	14.38	341.08	174.23	2673.72	67.68	919.65		43.55	591.79	6982.5	3.47	
	P15	-3.68	12.88	7.19	348.28	91.39	2765.11	70.77	497.78		45.54	320.32	7302.8	3.54	
		-4.21	13.41	7.19	355.47	94.52	2859.63	75.69	526.57		48.71	338.84	7641.6	3.63	
		-5.54	14.74	11.86	367.33	166.99	3026.62	88.61	974.67		57.02	627.20	8268.8	3.87	
		-4.47	13.67	2.52	369.85	35.75	3062.37	78.15	209.85		50.29	135.04	8403.9	3.68	
		-0.80	10.00	14.38	384.23	170.20	3232.57	46.42	895.71		29.87	576.38	8980.2	2.99	
		-0.41	9.61	1.94	386.17	19.04	3251.61	43.44	87.22		27.95	56.13	9036.4	2.91	
		1.22	7.98	4.53	390.70	39.84	3291.45	31.87	170.57		20.51	109.76	9146.1	2.57	
		1.93	7.27	5.18	395.88	39.48	3330.93	27.28	153.11		17.56	98.53	9244.7	2.41	
		4.94	4.26	9.06	404.94	52.23	3383.16	11.19	174.30		7.20	112.16	9356.8	1.69	
				404.94		**3383.16**			**14540.61**	**0.643**		**9356.82**		**2.77**	8.35
Bãi phải		4.94	4.26												
		5.01	4.19	8.05	8.05	34.03	34.03	10.89	43.85		3.01	12.12	12.1	0.72	
	P16	4.97	4.23	7.19	15.24	30.27	64.30	11.06	78.93		3.06	21.82	33.9	0.72	
		4.91	4.29	7.19	22.43	30.63	94.93	11.33	80.50		3.13	22.25	56.2	0.73	
		4.55	4.65	14.38	36.82	64.28	159.21	12.95	174.59		3.58	48.27	104.5	0.77	
		4.05	5.15	14.38	51.20	70.47	229.68	15.36	203.58		4.25	56.28	160.7	0.82	
	A2	3.89	5.31	10.93	62.13	57.16	286.84	16.16	172.25		4.47	47.62	208.4	0.84	
		3.87	5.33	3.45	65.58	18.36	305.21	16.26	55.96		4.50	15.47	223.8	0.84	
		3.91	5.29	3.74	69.32	19.85	325.06	16.06	60.43		4.44	16.71	240.5	0.84	
		4.04	5.16	10.71	80.03	55.98	381.04	15.41	168.57		4.26	46.60	287.1	0.83	
		4.15	5.05	14.38	94.41	73.42	454.46	14.86	217.68		4.11	60.18	347.3	0.81	
		4.44	4.76	5.54	99.95	27.16	481.61	13.47	78.44		3.72	21.68	369.0	0.78	
		5.20	4.00	0.43	100.38	1.89	483.50	10.08	5.08		2.79	1.40	370.4	0.70	
				100.38		**483.50**			**1339.86**	**0.276**		**370.40**		**0.77**	4.82

TABLE 2. ESTIMATION OF AVERAGE SCOUR DEPTH IN THE SECTION UNDER BRIDGE, FOR MAIN CHANNAL.

(By Laursen formula- Recommended in Hydraulic Engineering Center User's Manual HEC No.18, 2001)

H_{TT} = **9.20** m

N_o.	y_1	D_{50}	V_c	V	V_c/V	Type	Exponet for mode of bed material transport					Discharge, Bottom width in the main channel at approach section and contracted section.				D_m	Depth after scour	y_o	Δy_{xch}	Conclution
							S_1	V_*	ω	$\frac{V_*}{\omega}$	K_1	Q_1	W_1	Q_2	W_2		y_2			
	(m)	(mm)	(m/s)	(m/s)			(m/m)	(m/s)	(m/s)			(m^3/s)	(m)	(m^3/s)	(m)	(mm)	(m)	(m)	(m)	
Main channal	8.35	0.230	0.54	2.77	0.20	Live-bed scour	0.00023	0.14	0.022	6.2	0.69	9357.0	404.9	9357.0	379.9	0.29	8.73	8.35	0.38	Scouring under bridge

In which

- Δy_{xch}- Average depth of contraction scour, m:

  $\Delta y_{xch} = y_2 - y_o$.

- **y_2**- Average depth after scour in the contractioned section , m:

  $y_2 = y_1[Q_2/Q_1]^{6/7} * [W_1/W_2]^{k1}$ (with $V_c < V$, live-bed scour)

  $y_2 = [0.025 * Q^2 / D_m^{2/3} W^2]^{3/7}$ (with $V_c > V$, clear-water scour)

- y_1- Average depth in the main channel at approach section, m;
- **y_0**- Average depth in the main channel at th contracted section before scour,m;
- Q_1- Flow in the main channel at the approach section, m^3/s;
- Q_2- Flow in the main channel ats the contracted section, m^3/s.
- W_1- Bottom width in the main channel at the approach section, m;
- W_2- Bottom width of the main channel ats the contracted section less piers widths, m;
- k_1- Exponent for mode of the bed material transport, $k_1 = f(V_* / \omega)$.
- D_{50}- Average median diameter of the bed material particle size for which 50% is finer by weight, mm or m;
- V_c- Critical velocity for bed material, $V_c = f(y, D_{50})$, m/s.
- V- Average velocity in the main channel, m/s.
- D_m- Diameter of smallest sediment particles being swept away by the water, $D_m = 1.25D_{50}$, m or mm;
- $V_* = (gy_1S_1)^{1/2}$- Velocity when particles at a pier begin to move, m/s;
  - g- Acceleration due to gravity, m/s^2;
  - S_1- Slop of energy in the river section;
- ω- Sedimentaion rate sinks of sediment particle diameter D_{50}, m/s.

BẢNG 2. DỰ BÁO XÓI THU HẸP TRUNG BÌNH DƯỚI CẦU TẠI MẶT CẮT TỰ NHIÊN, KHU VỰC LÒNG CHỦ.

(Theo phương trình của Laursen - trong hướng dẫn thuỷ lực công trình HEC No.18, 2001)

H_{TT} = **9.20** m

N_o.	y_1	D_{50}	V_c	V	V_c/V	Thuộc loại	Tìm số mũ K_1					Lưu lượng, bề rộng mặt cắt ở thượng lưu cầu và mặt cắt bị thu hẹp.				D_m	Chiều sâu sau xói thu hẹp	y_o	Δy_{xch}	Kết luận
							S_1	V_*	ω	$\frac{V_*}{\omega}$	K_1	Q_1	W_1	Q_2	W_2		y_2			
	(m)	(mm)	(m/s)	(m/s)			(m/m)	(m/s)	(m/s)			(m^3/s)	(m)	(m^3/s)	(m)	(mm)	(m)	(m)	(m)	
Lòng chủ	8.35	0.230	0.54	2.77	0.20	Xói nước đục	0.000230	0.14	0.022	6.2	0.69	9357.0	404.9	9357.0	379.9	0.29	8.73	8.35	0.38	có xói thu hẹp do cầu

Trong đó:

- Δy_{xch}- chiều sâu trung bình của xói chung, m:

  $\Delta y_{xch} = y_2 - y_o$.
- **y_2**- chiều sâu trung bình sau xói chung ở mặt cắt bị thu hẹp, m:

  $y_2 = y_1[Q_2/Q_1]^{6/7} * [W_1/W_2]^{k1}$ (khi $V_c < V$, xói nước đục)

  $y_2 = [0.025 * Q^2 / D_m^{2/3} W^2]^{3/7}$ (khi $V_c > V$, xói nước trong)
- y_1- chiều sâu trung bình của dòng chảy ở thượng lưu, m;
- **y_0**- chiều sâu hiện tại ở mặt cắt bị thu hẹp trước xói chung.
- Q_1- lưu lượng ở khu vực dòng chảy thượng lưu có vận chuyển bùn cát, m^3/s;
- Q_2- lưu lượng ở khu vực dòng chảy bị thu hẹp, m^3/s.
- W_1- bề rộng đáy ở khu vực dòng chảy thượng lưu, m;
- W_2- bề rộng đáy ở khu vực dòng chảy bị thu hẹp, trừ đi bề rộng trụ, m;
- k_1- số mũ xét đến dạng vận chuyển bùn cát, $k_1 = f(V_* / \omega)$.
- D_{50}- đường kính trung bình của hạt bùn cát đáy,

  mà đường kính hạt dưới 50% là nhỏ hơn, mm hoặc m;
- V_c- Tốc độ tới hạn của hạt bùn cát, $V_c = f(y, D_{50})$, m/s.
- V- Tốc độ trung bình của dòng chảy, m/s.
- D_m- Đường kính của hạt bùn cát nhỏ nhất không bị

  dòng nước cuốn đi, $D_m = 1.25D_{50}$, m hoặc mm;
- $V_* = (gy_1S_1)^{1/2}$- tốc độ khởi động của hạt bùn cát ở đoạn thượng lưu, m/s;
  - g- gia tốc trọng trường, m/s^2;
  - S_1- độ dốc đường năng lượng ở đoạn sông;
- ω- tốc độ lắng chìm của hạt bùn cát có đường kính D_{50}, m/s.

TABLE 3. ESTIMATION OF LOCAL SCOUR DEPTH AT PIERS AND ABUTMENTS

(Recommended in Hydraulic Engineering Center User's Manual HEC No.18, 2001)

H_{TT} = 9.20 m

No.	∇_{sxc}	y_1	y_f	V_1	k_s	V_f	Fr_f	a or "a"	Coefficient K_1		Coefficient K_2			Coefficient K_3		Coefficient K_4		Δy_{xcb}	∇_{sx}
Trụ	(m)	(m)	(m)	m/s	(mm)	(m/s)		(m)	Pier shape	K_1	θ (^{0})	L (m)	K_2	Đáy sông là	K_3	D_{50} (mm)	K_4	(m)	(m)
T1	2.19	7.01	7.01	0.90	0.760	0.90	0.11	2.50	Circle	1.0	0	2.50	1.0	Small dunes	1.1	0.230	1.0	3.04	-0.85
T2	2.94	6.26	6.26	0.84	0.760	0.84	0.11	2.50	Circle	1.0	0	2.50	1.0	Small dunes	1.1	0.230	1.0	2.90	0.04
T3	3.85	5.35	5.35	0.75	0.760	0.75	0.10	2.50	Circle	1.0	0	2.50	1.0	Small dunes	1.1	0.230	1.0	2.70	1.15
T4	4.55	4.65	4.65	0.71	0.760	0.71	0.11	2.50	Circle	1.0	0	2.50	1.0	Small dunes	1.1	0.230	1.0	2.60	1.95
T5	0.46	8.74	8.74	1.06	0.760	1.06	0.11	2.50	Circle	1.0	0	2.50	1.0	Small dunes	1.1	0.230	1.0	3.35	-2.89
T6	4.17	5.03	5.03	0.71	0.760	0.71	0.10	2.50	Circle	1.0	0	2.50	1.0	Small dunes	1.1	0.230	1.0	2.63	1.54
T7	4.33	4.87	4.87	0.70	0.760	0.70	0.10	2.50	Circle	1.0	0	2.50	1.0	Small dunes	1.1	0.230	1.0	2.59	1.74
T8	4.18	5.02	5.02	0.71	0.760	0.71	0.10	2.50	Circle	1.0	0	2.50	1.0	Small dunes	1.1	0.230	1.0	2.63	1.55
T9	3.70	5.50	5.50	0.76	0.760	0.76	0.10	2.50	Circle	1.0	0	2.50	1.0	Small dunes	1.1	0.230	1.0	2.73	0.97
T10	3.31	5.89	5.89	2.01	0.760	2.01	0.26	2.50	Circle	1.0	0	2.50	1.0	Small dunes	1.1	0.230	1.0	4.19	-0.88
T11	2.71	6.49	6.49	2.15	0.760	2.15	0.27	4.00	Circle	1.0	0	4.00	1.0	Small dunes	1.1	0.230	1.0	5.93	-3.22
T12	1.86	7.34	7.34	2.35	0.760	2.35	0.28	4.00	Circle	1.0	0	4.00	1.0	Small dunes	1.1	0.230	1.0	6.26	-4.40
T13	0.70	8.50	8.50	2.60	0.760	2.60	0.28	4.00	Circle	1.0	0	4.00	1.0	Small dunes	1.1	0.230	1.0	6.67	-5.97
T14	-1.26	10.46	10.46	3.00	0.760	3.00	0.30	4.00	Circle	1.0	0	4.00	1.0	Small dunes	1.1	0.230	1.0	7.30	-8.56
T15	-4.06	13.26	13.26	3.54	0.760	3.54	0.31	4.00	Circle	1.0	0	4.00	1.0	Small dunes	1.1	0.890	1.0	8.09	-12.15
T16	4.59	4.61	4.61	0.72	0.760	0.72	0.11	4.00	Circle	1.0	0	4.00	1.0	Small dunes	1.1	0.230	1.0	3.54	1.05

ESTIAMTION OF LOCAL SCOUR DEPTH AT PIERS, ELEVATION AFTER CONSTRUCTION SCOUR

Local scour depth is estimated by fomula as follow:

$$\Delta y_{xcb} = 2.0\, K_1 K_2 K_3 K_4\, a^{0.65}\, y_1^{0.35}\, Fr_1^{0.43}$$

In which:

Δy_{xcb} = Local scour depth, m

K_1 : Correction factor for pier nose shape.

K_2 : Correction factor for angle θ between pier and direction of flow

Fr_1 : Froude number just upstream of the pire.

$Fr_1 = V_1 / (g.y_1)^{0.5}$

g : Acceleration due to gravity (9,81 m/s^2)

BẢNG 3. DỰ BÁO XÓI CỤC BỘ TẠI TRỤ VÀ MỐ CẦU

(Theo hướng dẫn thuỷ lực công trình HEC No.18, 1995)

H_{TT} = 9.20 m

No.	∇_{sxc}	y_1	y_f	V_1	k_s	V_f	Fr_f	a hoặc "a"	Xác định K_1		Xác định K_2			Xác định K_3		Xác định K_4		Δy_{xcb}	∇_{sx}
Trụ	(m)	(m)	(m)	m/s	(mm)	(m/s)		(m)	H. D. mũi trụ	K_1	θ (^{0})	L (m)	K_2	Đáy sông là	K_3	D_{50} (mm)	K_4	(m)	(m)
T1	2.19	7.01	7.01	0.90	0.760	0.90	0.11	2.50	Tròn	1.0	0	2.50	1.0	đụn cát nhỏ	1.1	0.230	1.0	3.04	-0.85
T2	2.94	6.26	6.26	0.84	0.760	0.84	0.11	2.50	Tròn	1.0	0	2.50	1.0	đụn cát nhỏ	1.1	0.230	1.0	2.90	0.04
T3	3.85	5.35	5.35	0.75	0.760	0.75	0.10	2.50	Tròn	1.0	0	2.50	1.0	đụn cát nhỏ	1.1	0.230	1.0	2.70	1.15
T4	4.55	4.65	4.65	0.71	0.760	0.71	0.11	2.50	Tròn	1.0	0	2.50	1.0	đụn cát nhỏ	1.1	0.230	1.0	2.60	1.95
T5	0.46	8.74	8.74	1.06	0.760	1.06	0.11	2.50	Tròn	1.0	0	2.50	1.0	đụn cát nhỏ	1.1	0.230	1.0	3.35	-2.89
T6	4.17	5.03	5.03	0.71	0.760	0.71	0.10	2.50	Tròn	1.0	0	2.50	1.0	đụn cát nhỏ	1.1	0.230	1.0	2.63	1.54
T7	4.33	4.87	4.87	0.70	0.760	0.70	0.10	2.50	Tròn	1.0	0	2.50	1.0	đụn cát nhỏ	1.1	0.230	1.0	2.59	1.74
T8	4.18	5.02	5.02	0.71	0.760	0.71	0.10	2.50	Tròn	1.0	0	2.50	1.0	đụn cát nhỏ	1.1	0.230	1.0	2.63	1.55
T9	3.70	5.50	5.50	0.76	0.760	0.76	0.10	2.50	Tròn	1.0	0	2.50	1.0	đụn cát nhỏ	1.1	0.230	1.0	2.73	0.97
T10	3.31	5.89	5.89	2.01	0.760	2.01	0.26	2.50	Tròn	1.0	0	2.50	1.0	đụn cát nhỏ	1.1	0.230	1.0	4.19	-0.88
T11	2.71	6.49	6.49	2.15	0.760	2.15	0.27	4.00	Tròn	1.0	0	4.00	1.0	đụn cát nhỏ	1.1	0.230	1.0	5.93	-3.22
T12	1.86	7.34	7.34	2.35	0.760	2.35	0.28	4.00	Tròn	1.0	0	4.00	1.0	đụn cát nhỏ	1.1	0.230	1.0	6.26	-4.40
T13	0.70	8.50	8.50	2.60	0.760	2.60	0.28	4.00	Tròn	1.0	0	4.00	1.0	đụn cát nhỏ	1.1	0.230	1.0	6.67	-5.97
T14	-1.26	10.46	10.46	3.00	0.760	3.00	0.30	4.00	Tròn	1.0	0	4.00	1.0	đụn cát nhỏ	1.1	0.230	1.0	7.30	-8.56
T15	-4.06	13.26	13.26	3.54	0.760	3.54	0.31	4.00	Tròn	1.0	0	4.00	1.0	đụn cát nhỏ	1.1	0.890	1.0	8.09	-12.15
T16	4.59	4.61	4.61	0.72	0.760	0.72	0.11	4.00	Tròn	1.0	0	4.00	1.0	đụn cát nhỏ	1.1	0.230	1.0	3.54	1.05

TÍNH XÓI CỤC BỘ CHO TRỤ CẦU, CAO ĐỘ SAU XÓI CHUNG

Xói cục bộ trụ cầu được dự báo theo công thức sau:

$$\Delta y_{xcb} = 2.0\, K_1 K_2 K_3 K_4\, a^{0.65}\, y_f^{0.35}\, Fr_1^{0.43}$$

Trong đó:

Δy_{xcb} = chiều sâu hố xói cục bộ, m

K_1 : hệ số hiệu chỉnh cho hình dạng mũi trụ.

K_2 : hệ số hiệu chỉnh cho góc chéo θ giữa phương trục dọc trụ và phương dòng chảy

Fr_1 : hệ số Froude.

$Fr_1 = V_f / (g.y_1)^{0.5}$

g : gia tốc trọng trường (9,81 m/s^2)

K_3 :Correction factor for bed condition.

K_4 : Correction factor for armoring of bed material (considered only D_{50} = > 60 mm)

a : Pier width, m

V_1 : Velocity in the main channel, m/s

V_1 : Velocity in the main channel, m/s

y_1 : Flow depth just upstream of pier, m

K_3 : hệ số hiệu chỉnh cho tình trạng đáy sông.

K_4 : hệ số hiệu chỉnh xét tới lớp phủ của vật liệu đáy (chỉ xét khi D_{50} = > 60 mm)

a : bề rộng trung bình của bệ hoặc tổng bề rộng cọc, m

V_1 : tốc độ trung bình dòng chảy trước trụ, m/s

V_1 : Tốc độ trung bình ở thủy trực của dòng chảy tiến vào trụ, m/s

y_1 : Chiều sâu dòng chảy ở thượng lưu trụ, m

y_1 : Chiều sâu dòng chảy ở thượng lưu trụ, m