DESIGN AND CONSTRUCTION OF A BRIDGE IN JEONJU, SOUTH KOREA

(Capstone Project II)

Bachelor of Engineering

(Civil Engineering)

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Declaration

We hereby declare that this report entitled “Design and construction of a bridge in Jeonju, South Korea” is the result of our own project work except for quotations and citations, which have been duly acknowledged. We also declare that it has not been previously or concurrently submitted for any other degree at Nazarbayev University.

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Abstract

Designing a bridge is one of the complex engineering problems. In order to design a bridge, an engineer firstly needs to choose the type of bridge he is going to construct, and in the process of a design determine the most significant factors in the analysis and selection, and to develop a comprehensive understanding in designing a bridge. In this paper, our group has mainly focused on the structural part of the bridge such as analysis of the loads, and behavior of the bridge under stresses. It is also essential to understand material characteristics and behavior under the loads in order to maintain stability and duration. The specifications of the design criteria are met according to the AASHTO code.

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1. Introduction

The aim of the project is to construct a bridge with a span of 500 meters in Jeonju, South Korea, in a proposed location as can be seen in Figure 1.1.

Figure 1.1. Proposed location of the bridge.

For the 500 m span bridge, our group selected a beam bridge with a total of 10 spans: 8 middle spans with a length of 52 meters, and 2 edge spans with a length of 42 meters determined according to AASHTO code. The width of the bridge is 16.4 meters, and the assumed clearance between the river and superstructure is 9 meters. The superstructure of the bridge is supported by pier, which consists of a bent cap and a cylindrical column. There are 9 supports in total, and for the foundation it was chosen to be driven piles. The detailed analysis and selection of the bridge components and its dimensions can be found in the following sections.

2. Structural Analysis

2.1. Bridge architecture

The total length of the bridge is considered to be 500 m, and side spans are usually taken as 70-80% of the interior spans. Thus, considered 10 span bridge is selected:

- 0.8S * 2 + 8S = 500 m
- S_i = 52 m⟶ interior span
- S_s = 0.8 * 52 ~ 42⟶ side span

To reduce drop bent cap size and to eliminate expansion joints on the bearings the continuous span for the bridge is selected.

2.2. Girder spacing

Thumb rule in steel-concrete composite bridge design indicates that for spans less than 140 ft ~ 42 m the girder spacing is between 10 ft to 12 ft. For spans larger than that use 11ft to 14 ft . As previously stated S_i and S_s are greater than 42 m the girder spacing is selected to be S = 3.5m.

2.3. Section dimensions and selection

Using a thumb rule for continuous span minimum depth D = 0.027 * 52 m = 1.404m for the girder is presented. Initial trial depth D = 2 m is selected.

For web $D/t_w \leq 150 \longrightarrow t_w = 2000mm/150 = 13.3$ mm, where $t_w = 17$ mm is selected (AASHTO 6.10.2.1).

Maximum transported length of girders is 120ft ~ 36 m and weight is 180 kips ~ 8165 kg, thus 36 m should be chosen for $b_{fc} = 36000/85 = 423.5$ mm, where $b_{fc} = 425$ mm is selected.

Thickness of compression flange should be $t_{fc} = b_{fc}/18.3 = 23$ mm, where $t_{fc} = 28$ mm is selected.

Since compression flange and tension flanges are to be the same width $b_{ft} = 425$ mm.

As for tension flange thickness $t_{ft} = 1.5 * t_{fc} = 42$ mm, where $t_{ft} = 42$ mm is selected.

Compression flange and tension flanges have to meet following requirements by AASHTO 6.10.2.2:

$$b_{fc}/(2t_{fc}) = 7.6 \leq 12 \quad \text{O.K.}$$

$$b_{fc} = 425 \text{ mm} \geq D/6 = 333.3 \text{ mm} \quad \text{O.K.}$$

$$t_{fc} \geq 1.1t_w = 18.7 < 28\text{mm} \quad \text{O.K.}$$

$$0.1 < I_{yc} / I_{yt} = 0.67 < 10 \quad \text{O.K.}$$

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- $S_i = 52$ m$\longrightarrow$ interior span
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$$0.1 < I_{yc} / I_{yt} = 0.67 < 10 \qquad \text{O.K.}$$

2.6. Loads

2.6.1. Permanent Loads

Permanent loads – are loads that are constant or varying over a long period of time. Amongst them are structural components and attachments (DC), dead load of wearing surfaces and utilities (DW), down drag forces (DD), forces effects due to creep (CR) and etc. However in this particular report only deck slab, girder, wearing surface and barrier weights will be considered for the design of girders. The combination of loads in this reports are DC_1 - dead loads of deck slab (including haunch) + steel girders, DC_2 - dead loads of concrete barriers, DW – load of wearing surface.

2.6.2. Transient Loads

When accounting for calculation of live vehicular loads, AASHTO 1993 suggests using scenario where design lane load + truck load are used. Truck loads are usually used for design of regular situations on the highway. As specified in AASHTO LRFD Bridge Design Specifications

3.6.1.2.4 design lane load is 640 klf (9.3kN/m). For the calculation of design load, allowable factor for the live load is 1.35 as specified in AASHTO LRFD Bridge Design Specifications 3.4.1 – 1.

Loading scenario with design truck + lane load, where design truck front axle is 8 (35kN) kip, and rear axles are 32 kip (145kN). The distance between rear axles is from 14 ft (4.3m) to 30 ft (9 m), thus to account for heavier loading case, the distance in this design will be taken as 14 ft (4.3m) (American Association of State Highway and Transportation Officials, 2012).

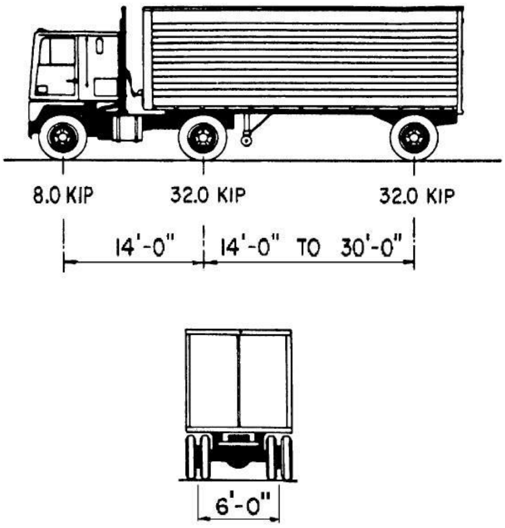


Figure 2.1. Design truck.

Permit truck is a very heavy vehicle that is an ultimate load to bear by the bridge. The configuration of such vehicle is shown below.

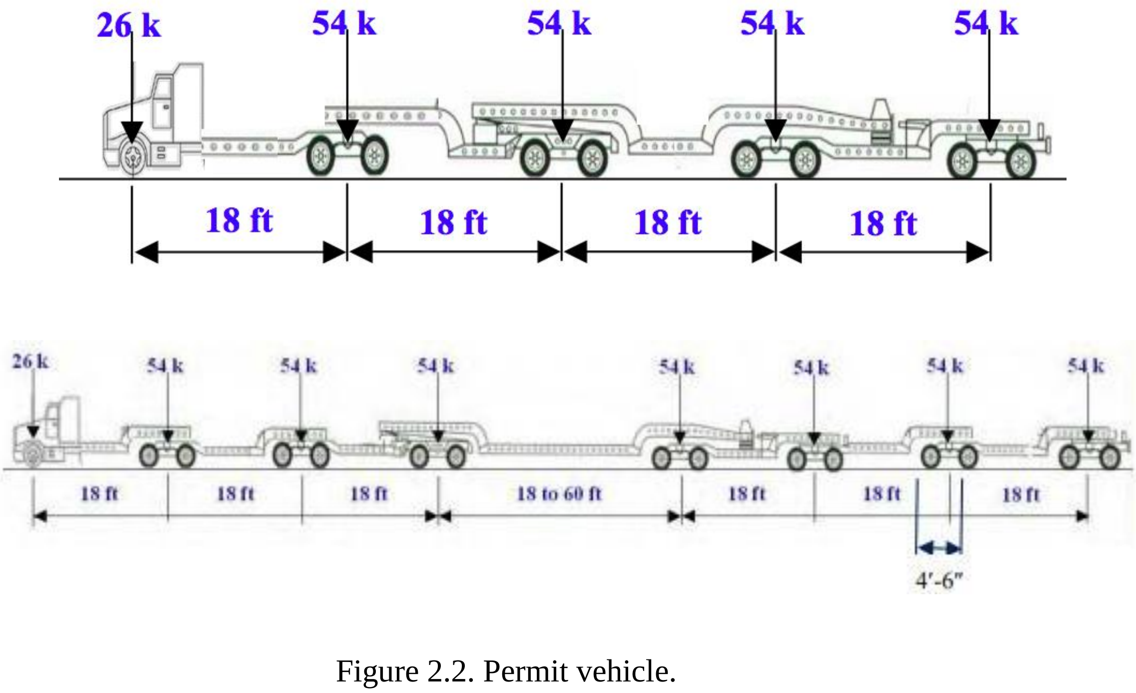


Figure 2.2. Permit vehicle.

2.6.3. Dead load and transient load analysis

Since the sections have been selected it is now possible to calculate the loads imposed on the structure, particularly girders. SAP 2000 have been used to obtain the results of unfactored dead and live load moments and shears. The results will be given in the appendix.

First of all, the model was constructed as a system of girders:

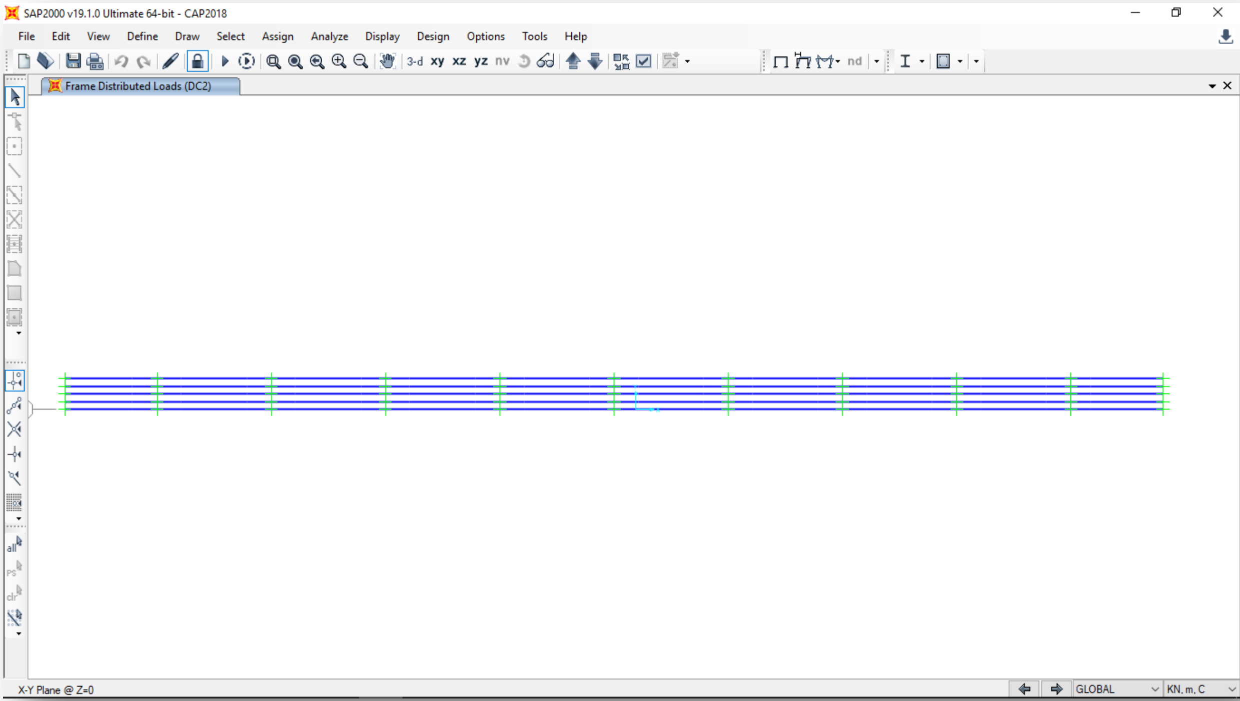


Figure 2.3. Top view of grid with girders.

The supports on the edges of the bridge has been chosen to be fixed and on the piers to be rollers.

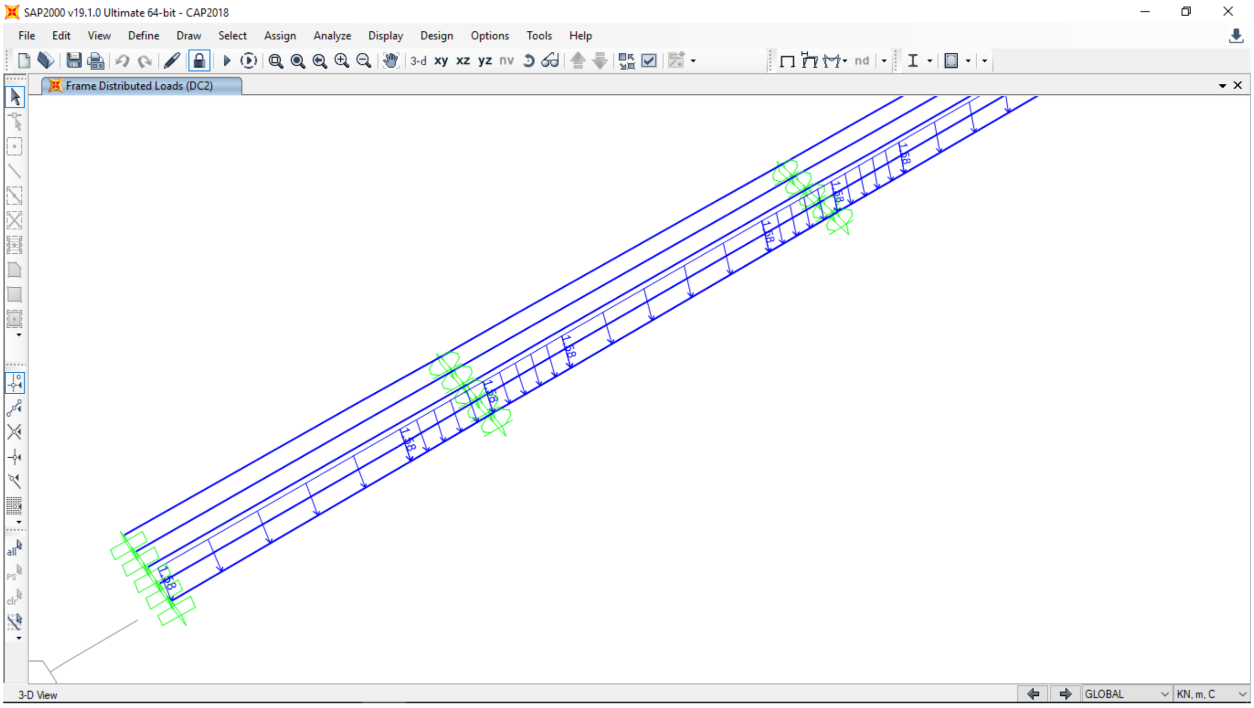


Figure 2.4. Isometric view of girders.

Next step is to define loads where DC_1 is the combination of slab weight and girder weight, DC_2 is the weight of barriers, DW is the load of wearing surface, LL(HL-93) weight of design truck, LL (P-15) weight of permit vehicle and LL(Lane Load) is the distributed lane load. The analysis has been conducted on two interior girders, as they account for main carriers of the load. After the combinations of loads have been defined the analysis is run and the results obtained as unfactored loads are given in the appendix of this report.

2.7. Vehicular dynamic load allowance (IM)

To account for bouncing effect while the vehicles are moving on the surface of the bridge the moments obtained through analysis in SAP 2000 should be multiplied by certain coefficients. For design truck (HL-93) the dynamic load allowance is 1.33 and for permit truck it is 1.25. The IM is not applicable for lane load. Notation for increased load is LL+IM.

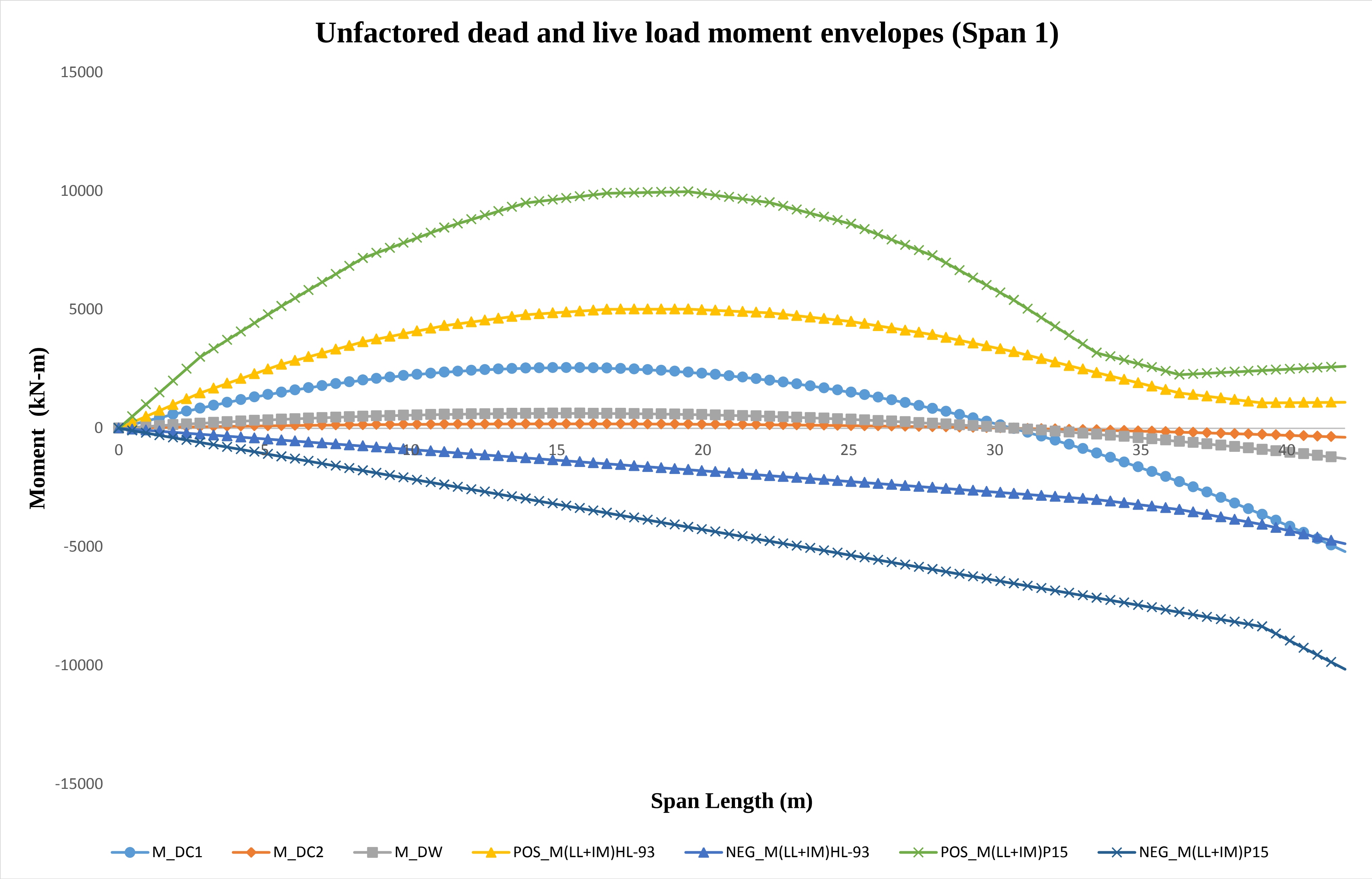


Figure 2.5. Unfactored dead and live load moment envelopes (Span 1).

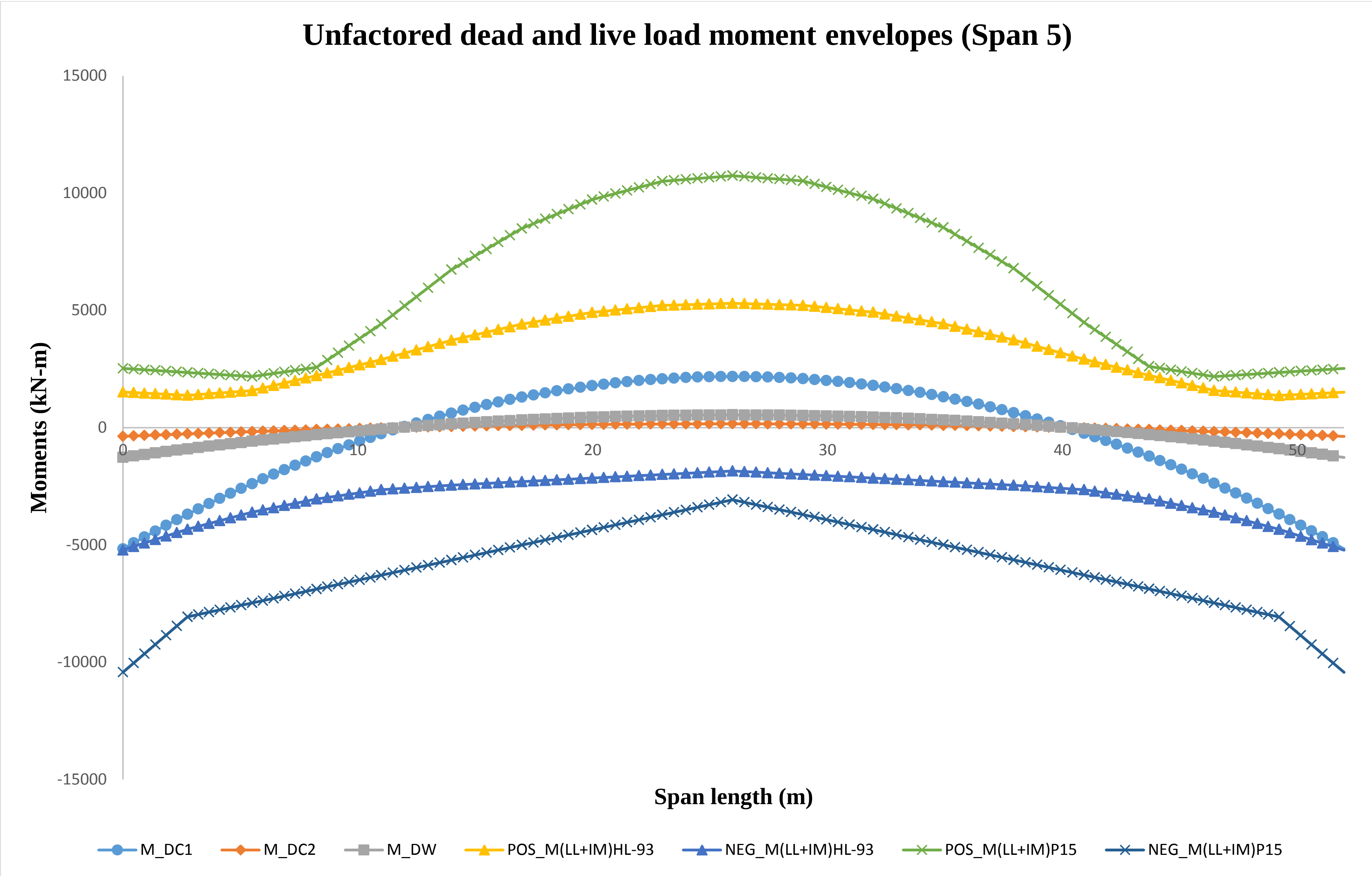


Figure 2.6. Unfactored dead and live load moment envelopes (Span 5).

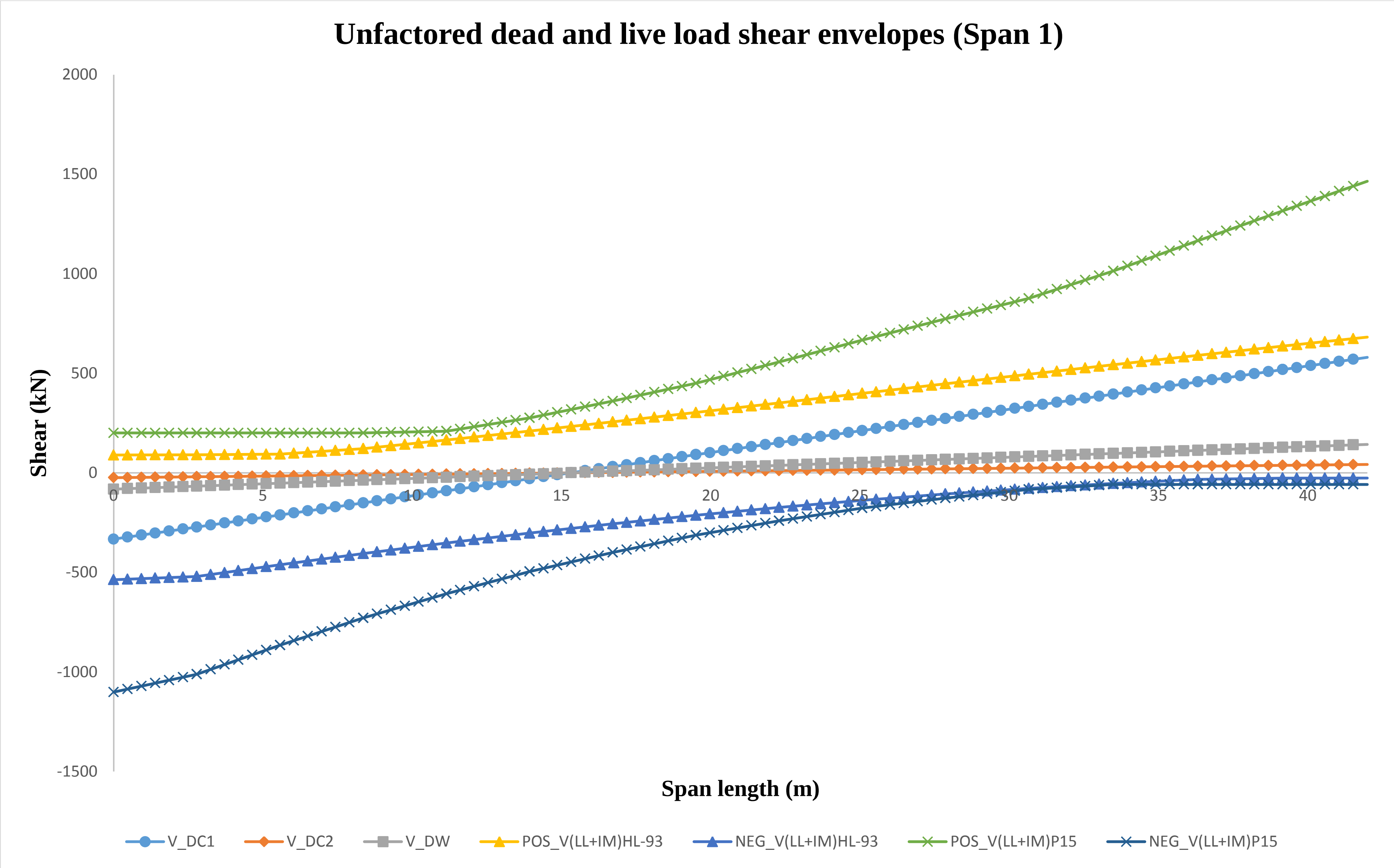


Figure 2.7. Unfactored dead and live load shear envelopes (Span 1).

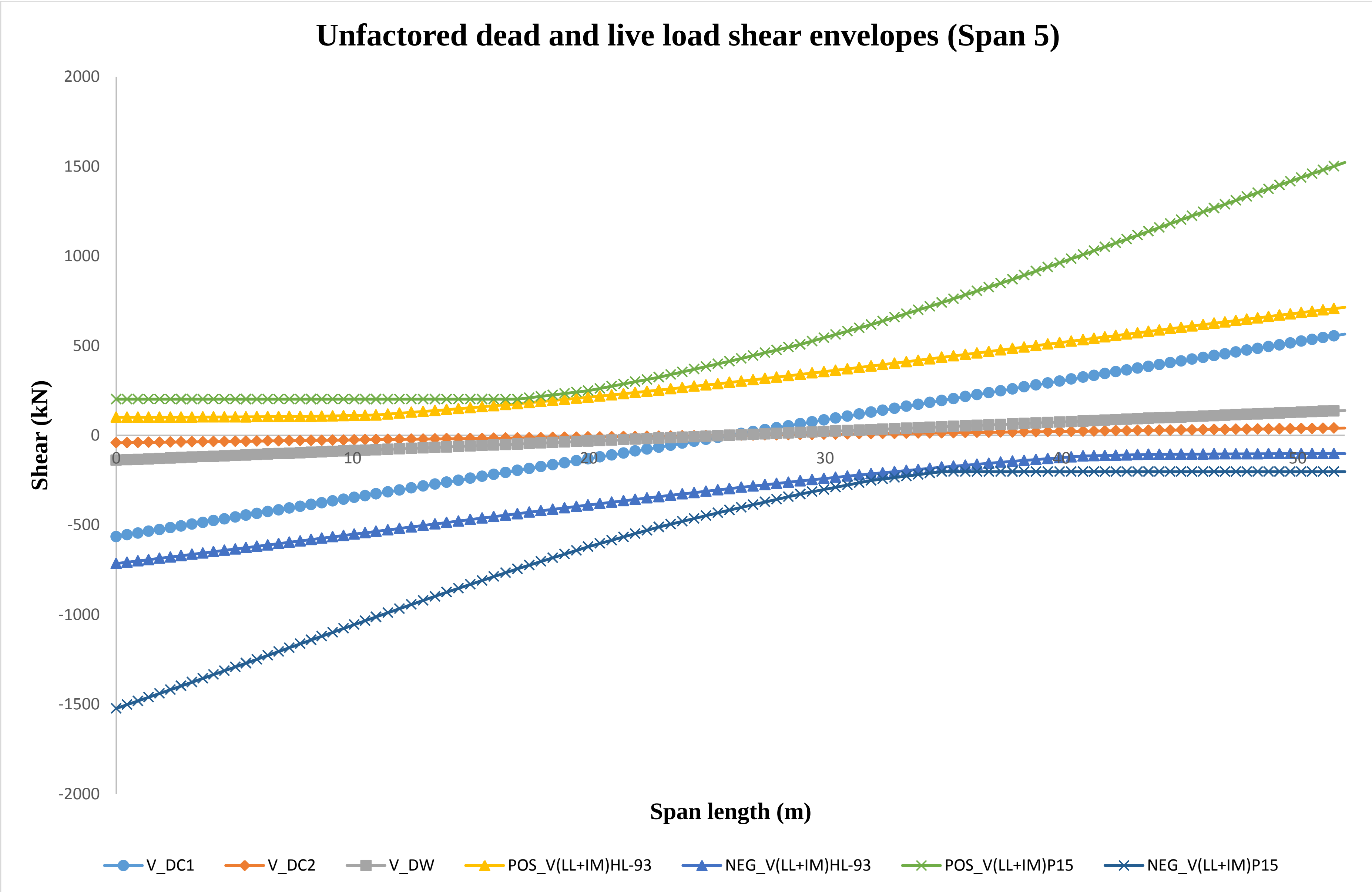


Figure 2.8. Unfactored dead and live load shear envelopes (Span 5).

3. Structural Design

3.1. Concrete deck

Concrete deck structure provides riding surface for vehicles on the bridge along with transferring the load directly to major load-carrying members, which in this case are plate I girders. This part of the report will deal with the analysis of concrete deck, its structural behavior and detailed design of reinforcement.

3.1.1. Design approach

3.1.1.1. Cast in place concrete deck

For the material of the deck cast in place method has been selected, due to concrete transportation inconvenience of precast structures.

3.1.1.2. Structural behavior

Behavior of concrete decks differs from those of concrete beams, in a sense that there isn't a pure flexure, instead it is term called internal arching. When the concrete begins to crack, flexure and membrane stresses resist the load imposed on the slab. Primarily due to live load, concrete cracks in three dimensions around the wheel contact area. The cracks appear at the bottom of the slab and neutral axis shift upward, thus allowing compressive stresses to prevent further crack propagation. The concrete portion above the neutral axis is in elastic state. This membrane compressive stress mainly resists the load. In order to fail the deflection should be large enough so that cone-shaped section can develop. Primarily, the slab fails under punching shear.

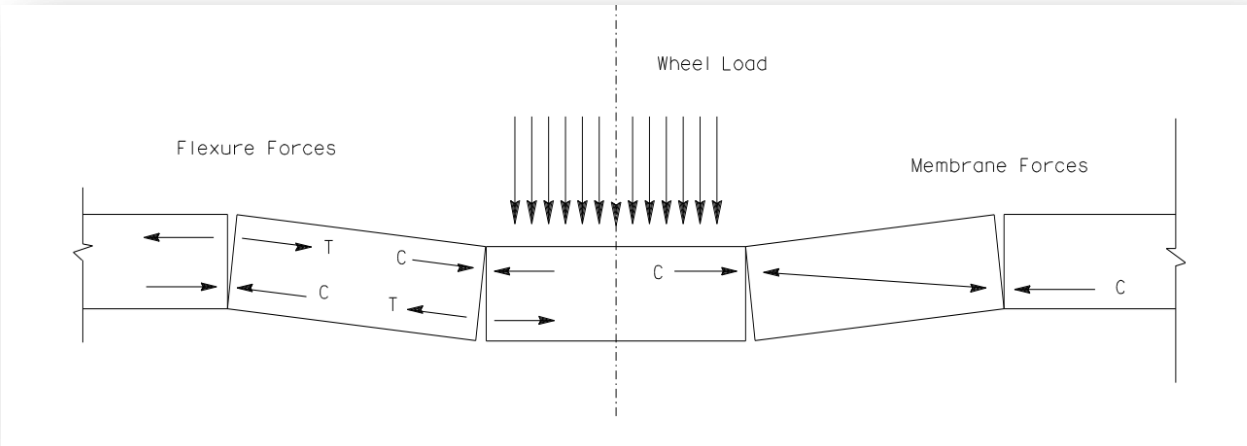


Figure 3.1. Membrane and flexure forces under wheel load.

3.1.1.3. Limit state

3.1.1.3.1. Service limit state

Service I limit state is the required condition for designing concrete decks, because it is used to control cracking and excessive deformations. California amendment (CA Article 9.5.2), deck slab are to be designed for exposure of Class 2, with $\gamma_e = 0.75$ (AASHTO Article 5.7.3.4).

3.1.1.3.2. Strength limit state

Strength I limit state is the consideration when designing the deck slab, as usual it is design for tension-controlled reinforced components. The resistance factor $\phi = 0.9$ (AASHTO Article 5.5.4.2). Strength II and permit vehicle loads are not considered in the design of deck slab.

3.1.1.3.3. Fatigue limit state

According to AASHTO Article 9.5.3 it is not required to investigate the deck slab for fatigue limit state in multi-girder systems.

3.1.2. Approximate method of analysis

During the analysis phase concrete deck is divided into transverse strips, supported by girders. For the simplification of analysis, girders are considered to be fixed supports. The width of the strip is determined according to AASHTO Article 4.6.2.1.3-1. However this table is valid only for interior strips; overhangs are designed separately.

3.1.2.1. Unfactored dead loads

In the design of the deck, for convenience of calculations dead loads are calculated for 1 ft wide section of the bridge, using any approved method of analysis. Thus the section is 1 ft wide concrete beam, with length 16.4 m and thickness t_{deck} = 7.87 in = 200 mm. The material has been assigned as follows:

Concrete:

$$E_c = 33{,}000 \cdot w_c \cdot \sqrt{f'c} = 3{,}457 \text{ ksi}$$

$$E_s = 29{,}000 \text{ ksi}$$

$$n = \frac{E_s}{E_c} = 8.39$$

Section:

$$b = 1 \text{ ft},\ t_{deck} = 200 \text{ mm} = 7.78 \text{ in},\ L = 16.4 \text{ m}$$

3.1.2.2. Snow load

AASHTO does not provide any exact requirements for snow loads, and if there is a need for snow load calculations refers to ASCE 7 code. Assuming bridge as a flat roof with parapets calculations have been conducted. Ground snow load in Jeonju, South Korea is $p_g = 0.5 kN/m^2$. From the ASCE 7 the coefficients have been calculated $C_e = 0.8$ (fully exposed, with surface roughness 0), $C_t = 1.2$ (unheated, open air), $I_s = 1.1$.

$$p_f = 0.7 \cdot C_e \cdot C_t \cdot I_s \cdot p_g = 0.7 \cdot 0.8 \cdot 1.2 \cdot 1.1 \cdot 0.5 = 0.3696 \text{ kN/m}^2$$

$$p_g > 0.96 \text{ kN/m}^2 \text{ then } p_f = p_m = I_s \cdot p_g = 0.55 \text{ kN/m}^2 = 11.49 \text{ lb/ft}^2$$

$$\gamma_{snow} = 0.13\ p_g + 14 = 14.065 \text{ lb/ft}^3$$

3.1.2.3. Snow drift

ASCE states that near parapets snow drift is calculate for upwind length of the parapet, thus l_u = 7 in. If l_u < 20 ft then l_u = 20 ft.

$$h_d = 0.43 \cdot \sqrt[3]{20} \cdot \sqrt[4]{11.49 + 10} - 1.5 = 1.013 \text{ ft}$$

$$h_c = 3.5 \text{ ft}, \ w = 4 \cdot h_d = 4 \cdot 1.013 = 4.052 \text{ ft}$$

$$p_d = h_d \cdot \gamma_{snow} = 1.013 \cdot 14.065 = 14.25 \text{ lb/ft}^2 = 0.68 \text{ kN/m}^2$$

Jeonju city itself is located at elevation around $H_{elevation}$ = 104.89 feet or 31.97 m. AASHTO Article 3.9.6 states that snow load and snow drift are not included in load combinations and need not be considered in the design stage, due to the location of the bridge being lower than 2000 ft elevation.

3.1.2.4. Imposed loads

Load of barrier = 266 plf = 0.3606 kN/m

$$\gamma_c = 0.145 \text{ kcf} = 2322.677 \text{ kg/m}^3$$

$$\gamma_w = 0.140 \text{ kcf} = 2242.585 \text{ kg/m}^3$$

$$\text{Deck} = 0.145 * \frac{7.87}{12} * 1 \text{ ft} = 0.095 \text{ kip/ft}$$

$$\text{Wearing surface (DW)} = 0.140 * \frac{2.95}{12} * 1 \text{ ft} = 0.0338 \frac{\text{kip}}{\text{ft}}$$

of load between the face of the barriers

$$\text{Barrier} = 266 * 1 \text{ ft} = 0.266 \frac{\text{kip}}{\text{ft}} \text{ at 5 in distance from the edge of the deck}$$

Snow load = 0.55 kN/m^2 • 1m = 0.55 kN/m between the faces of the barriers

Snow drift = 0.68 kN/m^2 • 1m = 0.68 kN/m from the face of the barrier for 1.2192 m

Since loads are symmetrical along the deck only first half of the deck slab has been displayed and the results obtained from the SAP2000 are as follows:

Table 3.1. Unfactored dead loads for transverse portion of the deck slab.

	M at 1.2 m (kNm)	M at 2.95 m (kNm)	M at 4.7 m (kNm)	M at 6.45 m (kNm)	M at 8.2 m (kNm)
Deck	-0.9982	0.8138	-1.5334	0.6346	-1.3567
Wearing surface	-1.2696	-0.4551	0.3595	0.0905	-0.1785
Barrier	-0.2249	0.3442	-0.5948	0.2203	-0.4727

The coordinates in the table represents moments at supports and at midspan of the transverse deck.

3.1.2.5. Unfactored live loads

In a case of a deck supported on 3 or more girders, Live Load moment can be obtained using AASHTO Appendix A4 T.A4-1. The table list positive and negative moments calculated using Equivalent Strip Method, to obtain other results interpolation may be used. The values in the table already include multiple presence factors and dynamic load allowance, and provided that the distance between centerline of exterior girders should be not less than 14 ft = 4.2672 m. To determine the distance from centerline of girders to the section of negative flexure AASHTO Article 4.6.2.1.6 states that for the deck supported on steel I girders, the distance is quarter of its flange width. In this particular case flange width is b_f = 425 mm, thus the design distance is 425 / 4 = 106.25 mm or 4.183075 in. Using interpolation method:

Design section = 106.25 mm or 4.183075 in

$+M_{LL}$ = 6.795512 kip-ft = 9.213477137701409 kN-m

$-M_{LL}$ = 6.439163175 kip-ft = - 8.730333004899602 kN-m

3.1.2.6. Factored design loads

Concrete deck slabs are to be investigated for Service I and Strength I load combinations. In it conservative to take minimum value of factors for positive values at supports and negative values at the midspan.

$$M_u = \eta * [\gamma_{DC} * M_{DC} + \gamma_{DW} * M_{DW} + m * \gamma_{LL}(M_{LL} + IM)]$$

$$\eta = 1$$

Table 3.2. Load factors.

	Load Factors				
	$\gamma_{DC,max}$	$\gamma_{DC,min}$	$\gamma_{DW,max}$	$\gamma_{DW,min}$	γ_{LL}
Strength I	1.25	0.9	1.5	0.65	1.75
Service I	1	1	1	1	1

Table 3.3. Factored Design Moments.

	Design Moments				
	M at 1.2 m (kNm)	M at 2.95 m (kNm)	M at 4.7 m (kNm)	M at 6.45 m (kNm)	M at 8.2 m (kNm)
Strength I	-18.71	17.27	-17.7	17.33	-17.832
Service I	-11.22	9.92	-10.49	10.16	-10.74

Design moments for Strength I Limit State:

Controlled positive moment $+M_u$ = 17.33 kNm = 12.781 kip-ft

Controlled negative moment – M_u = -18.71 kNm = -13.799 kip-ft

Design moments for Service I Limit State:

Controlled positive moment $+M_u$ = 10.16 kNm = 7.493 kip-ft

Controlled negative moment – M_u = -11.22kNm = -8.275 kip-ft

3.1.3. Transverse reinforcement design

3.1.3.1. Section design for Strength I limit state

Width of design section b = 1 ft = 12 in

Resistance factor ϕ_{str} = 0.9

3.1.3.2. Positive flexure design

Try #7 bar

Bar spacing s = 9 in

Bar diameter d_b = 0.875 in = 22.225 mm

Bar area A_s = 0.6 in^2 = 387 mm^2

Area of steel reinforcement per design length $A_s = b \bullet \frac{Ab}{s} = 12 \ast \frac{0.6}{9}$ = 0.8 in^2/ft

Cover $c_b = 1$ in

Effective depth $d_s = t_{deck} - c_b - \frac{db}{2} = 7.87 - 1 - \frac{0.875}{2} = 6.43$ in

Depth of equivalent stress block $a = \frac{A_s * f_y}{0.85 * f'_c * b} = \frac{0.8 * 60}{0.85 * 3.6 * 12} = 1.307$

Factored resistant moment $\phi M_n = \phi \bullet A_s f_y \bullet (d_s - a/2) = \frac{0.9 * 0.8 * 60 * (6.43 - 1.307/2)}{12} = 20.80$ kip-ft

Check $\phi M_n \geq +M_u \longrightarrow 20.80$ kip-ft ≥ 12.781 kip-ft OK

3.1.3.3. Negative flexure design

Try #7 bar

Bar spacing s = 9 in

Bar diameter $d_b = 0.875$ in = 22.225 mm

Bar area $A_s = 0.6\ in^2 = 387\ mm^2$

Area of steel reinforcement per design length $A_s = b * \frac{Ab}{s} = 12 * \frac{0.6}{9} = 0.8\ in^2/ft$

Cover $c_b = 1.5$ in

Effective depth $d_s = t_{deck} - c_b - \frac{db}{2} = 7.87 - 1.5 - \frac{0.875}{2} = 5.93$ in

Depth of equivalent stress block $a = \frac{A_s * f_y}{0.85 * f'_c * b} = \frac{0.8 * 60}{0.85 * 3.6 * 12} = 1.307$

Factored resistant moment $\phi Mn = \phi \bullet A_s f_y \bullet \left(d_s - \frac{a}{2}\right) = \frac{0.9 * 0.8 * 60 * \left(5.93 - \frac{1.307}{2}\right)}{12} = 18.99\ \text{kip} - \text{ft}$

Check $\phi M_n \geq -M_u \longrightarrow -18.99$ kip-ft ≥ -13.799 kip-ft OK

3.1.3.4. Minimum reinforcement

The amount of tensile reinforcement needed to sufficiently provide flexural resistance $M_r = \phi M_u$ should be equal or more than one of the following $1.33 \bullet (+M_u)$ or M_{cr}.

Cracking moment $M_{cr} = \gamma_3 \bullet \gamma_1 \bullet f_r \bullet S_c$

$\gamma_3 = 0.75$, $\gamma_1 = 1.6$

Modulus of rupture $f_r = 0.24 \bullet \sqrt{f'c} = 0.455$ ksi

Section modulus $S_c = b \bullet h^2/6 = 39.3701 \bullet 7.78^2 / 6 = 406.41\ in^3$

3.1.3.5. Positive flexure reinforcement check

$$+\phi M_n \begin{cases} 1.33 * (+M_u) = 1.33 * 12.781 = 16.99 \text{ kip} - \text{ft} \\ M_{cr} = \dfrac{(0.75 * 1.6 * 0.455 * 406.41)}{12} = 18.49 \text{ kip} - \text{ft} \end{cases}$$

22.571 kip-ft ≥ 18.49 kip-ft OK

$$-\phi M_n \begin{cases} 1.33 * (-M_u) = 1.33 * 13.799 = -18.35 \text{ kip} - \text{ft} \\ M_{cr} = \dfrac{(0.75 * 1.6 * 0.455 * 406.41)}{12} = -18.49 \text{ kip} - \text{ft} \end{cases}$$

- 23.56 kip-ft ≥ -18.49 kip-ft OK

3.1.3.6. Section design for Service I limit state

The spacing s closest to tension line should satisfy following equation:

$$s \leq \frac{700 * \gamma_e}{\beta_s * f_{ss}} - 2 * d_c$$

3.1.3.7. Cracking at the bottom of the deck

$$d_c = c_{bot} + \frac{d_b}{2} = 1 + \frac{0.875}{2} = 1.4375 \text{ in}$$

$$\beta_s = 1 + \frac{d_c}{0.7 * (t_{deck} - d_c)} = 1 + \frac{1.4375}{0.7 * (7.87 - 1.4375)} = 1.32$$

$$\rho = \frac{A_s}{b * d_s} = \frac{0.8}{12 * 6.43} = 0.01035$$

$$k = \sqrt{2 * n * \rho + (n * \rho)^2} - n * \rho = \sqrt{2 * 8.39 * 0.1035 + (8.39 * 0.1035)^2} - 8.39 * 0.1035 = 0.710$$

$$j = 1 - \frac{k}{3} = 1 - \frac{0.710}{3} = 0.763$$

$$f_{ss} = \frac{+M_{u,service}}{A_s * j * d_s} = \frac{7.493 * 12}{0.8 * 0.763 * 6.43} = 22.91 \text{ ksi}$$

$$s_{max} = \frac{700 * \gamma_e}{\beta_s * f_{ss}} - 2 * d_c = \frac{700 * 1.00}{1.32 * 22.91} - 2 * 1.4375 = 20.27 \text{ in}$$

Spacing used 9 in ≤ s_{max} = 20.27 in OK

3.1.3.8. Cracking at the top of the deck

$$d_c = c_{bot} + \frac{d_b}{2} = 1.5 + \frac{0.875}{2} = 1.9375 \text{ in}$$

$$\beta_s = 1 + \frac{d_c}{0.7 * (t_{deck} - d_c)} = 1 + \frac{1.9375}{0.7 * (7.87 - 1.9375)} = 1.467$$

$$\rho = \frac{A_s}{b * d_s} = \frac{1.029}{12 * 5.93} = 0.01446$$

$$k = \sqrt{2 * n * \rho + (n * \rho)^2} - n * \rho = \sqrt{2 * 8.39 * 0.01446 + (8.39 * 0.01446)^2} - 8.39 * 0.01446 = 0.386$$

$$j = 1 - \frac{k}{3} = 1 - \frac{0.386}{3} = 0.8713$$

$$f_{ss} = \frac{-M_{u,service}}{A_s * j * d_s} = \frac{-8.275 * 12}{1.029 * 0.8713 * 5.93} = 18.67 \text{ ksi}$$

$$s_{max} = \frac{700 * \gamma_e}{\beta_s * f_{ss}} - 2 * d_c = \frac{700 * 1.00}{1.467 * 18.67} - 2 * 1.9375 = 21.68 \text{ in}$$

Spacing used 9 in ≤ s_{max} = 21.68 in OK

3.1.4. Longitudinal reinforcement design

Reinforcement in longitudinal direction on each portion of the deck have to meet the requirements:

$$A_s \geq \frac{1.3 * b * t_{deck}}{2 * (b + t_{deck}) f_y}$$

$$0.11 \leq A_s \leq 0.60$$

For top reinforcement try #4 @6 in:

Bar diameter d_b = 0.500 in = 12.7 mm

Bar area A_s = 0.20 in^2 = 129 mm^2

$$A_{s,min} = \frac{1.3 * 12 * 7.87}{2 * (12 + 7.87) * 60} = 0.0515 \text{ in}^2/\text{ft}$$

$$0.0515 \frac{\text{in}^2}{\text{ft}} \leq 0.11 \text{ in}^2/\text{ft}$$

Thus 0.11 in^2/ft controls

$$A_s = \frac{b * A_b}{s} = \frac{12 * 0.2}{6} = 0.4 \text{ in}^2/\text{ft}$$

Check $A_s \geq A_{s,min} \longrightarrow 0.4 \frac{\text{in}^2}{\text{ft}} \geq 0.11 \text{in}^2/\text{ft}$ OK

$$S = S_{gdr} - t_{web} = 9.843 \text{ ft} - \frac{0.669 \text{ in}}{12 \frac{\text{in}}{\text{ft}}} = 9.79 \text{ ft}$$

AASHTO specifies that amount of secondary reinforcement should be a percentage of amount of primary reinforcement:

$$\frac{220}{\sqrt{S}} \leq 67\%$$

$$\frac{220}{\sqrt{9.79}} = 70.32\% \geq 67\%$$

thus use 67%

Amount of primary reinforcement for positive flexure $A_s = 0.8 \text{ in}^2/\text{ft}$

Amount of longitudinal reinforcement $A_s = 0.67 * 0.8 = 0.536 \text{ in}^2/\text{ft}$

For bottom reinforcement try #5 @ 8'' according to Colorado Department of transportation

Bar diameter d_b = 0.625 in = 15.875 mm

Bar area A_s = 0.31 in^2 = 200 mm^2

$$A_{s,min} = \frac{1.3 * 12 * 7.87}{2 * (12 + 7.87) * 60} = 0.0515\ in^2/ft$$

$$0.0515\ in^2/ft \leq 0.11$$

Thus 0.11 in^2/ft controls

$$A_s = \frac{b * A_b}{s} = \frac{12 * 0.31}{8} = 0.465\ in^2/ft$$

Check $A_s \geq A_{s,min} \longrightarrow 0.465\ \frac{in^2}{ft} \geq 0.11 in^2/ft$ OK

3.2. Drop bent cap

Combination of columns and bent cap beam, resolves the issue of transferring lateral loads such as wind, earthquake and vertical loads to the foundation. Bent cap itself support girders and transfers all the superstructure load to columns.

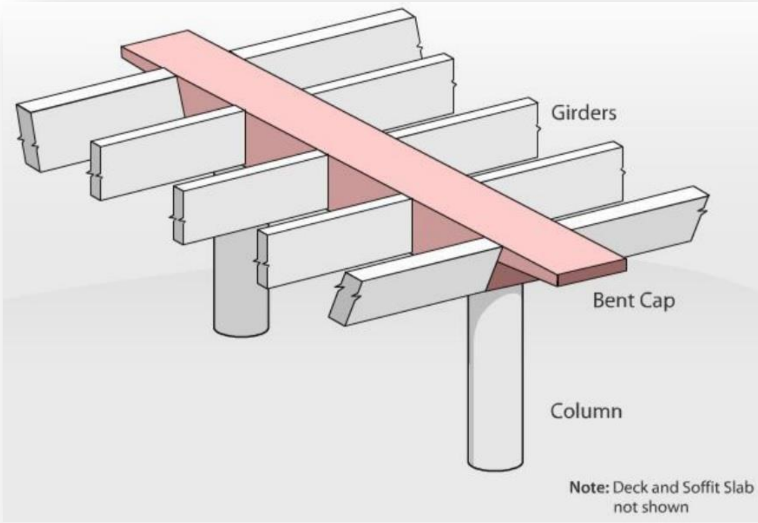


Figure 3.2. Concept of bent cap.

3.2.1. Type

The most common types of bent caps are integral, inverted tee and drop bent caps. For current project the most suitable option is drop bent cap since others are design for box girders.

3.2.2. Preliminary dimensions

AASHTO does not provide any minimum or maximum dimension for bent cap or columns, thus engineers generally rely on their past experience. The dimensions of bent cap, columns and layout of the structure will be shown in Figure X.

3.2.3. Flexural design

Drop bent cap is design for dead load, HL-93 truck, P-15 permit truck and fatigue truck loads.

Table 3.4. Bent cap reactions from live and dead loads.

Load and moment	Moment at midspan (kip-ft)	At faces of column (kip-ft)
DC	2317.9	-2234.14
DW	504.9	-474.6
HL-93 vehicle	771	-616.9
Permit Vehicle	3265.4	-2556.3
Fatigue Vehicle	658.3	-522.9

3.2.3.1. Factored positive moments

Strength I Limit State

$$M_u = 1.25 * M_{DC} + 1.5 * M_{DW} + 1.75 * \; M_{HL-93} = 1.25 * 2317.9 + 1.5 * 504.9 + 1.75 * 771$$
$$= 5003.9 \text{ kip} - \text{ft}$$

Strength II Limit State

$$M_u = 1.25 * M_{DC} + 1.5 * M_{DW} + 1.75 * \; M_{P-15} = 1.25 * 2317.9 + 1.5 * 504.9 + 1.35 * 3265.4$$
$$= 8063 \text{ kip} - \text{ft}$$

Service I Limit State

$$M_u = M_{DC} + M_{DW} + M_{HL-93} = 2317.9 + 504.9 + 771 = 3593.8 \text{ kip} - \text{ft}$$

3.2.3.2. Factored negative moments

Strength I Limit State

$$M_u = 1.25 * M_{DC} + 1.5 * M_{DW} + 1.75 * \; M_{HL-93}$$
$$= 1.25 * (-2234.14) + 1.5 * (-474.6) + 1.75 * (-616.9) = -4584.15 \text{ kip} - \text{ft}$$

Strength II Limit State

$$M_u = 1.25 * M_{DC} + 1.5 * M_{DW} + 1.75 * \; M_{HL-93}$$
$$= 1.25 * (-2234.14) + 1.5 * (-474.6) + 1.35 * (-2556.3) = -6955.6 \text{ kip} - \text{ft}$$

Service I Limit State

$$M_u = M_{DC} + M_{DW} + M_{HL-93} = (-2234.14) + (-474.6) + (-616.9) = -3325.6 \text{ kip} - \text{ft}$$

$$M_{cr} = \gamma_3 * \gamma_1 * f_r * S_c$$

$$f_r = 0.24 * \sqrt{f_c'} = 0.24 * \sqrt{4} = 0.48 \text{ ksi}$$

$$S_c = \frac{b * h^2}{6} = \frac{7.22 * 6.56^2}{6} = 51.78 \text{ ft}^3$$

$$M_{cr} = 0.75 * 1.6 * 0.48 * 51.78 * 12^2 = 4294 \text{ kip} - \text{ft}$$

$$M_{u,min} = \text{lesser of} \begin{cases} 1.0 * (M_{cr}) = 1.33 * 3593.8 = 4779.8 \text{ kip} - \text{ft} \\ 1.33 * M_u = 1.33 * 8063 = 10723.8 \text{ kip} - \text{ft} \end{cases}$$

Thus, controlling moment $M_u = 8063 \text{ kip} - \text{ft}$

3.2.3.3. Section design for Strength II limit state

Resistance factor $\phi_{str} = 0.9$

3.2.3.3.1. Positive flexure design

Try #14 bar

Bar spacing s = 7 in

Bar diameter d_b = 1.693 in = 43 mm

Bar area A_b = 2.25 in^2 = 1452 mm^2

Area of steel reinforcement $A_s = b \cdot \frac{Ab}{s} = 86.61 * \frac{2.25}{7} = 27.84\ \text{in}^2$

Cover c_b = 2.5 in

Effective depth $d_s = t_{cap} - c_b - \frac{db}{2} = 78.74 - 2.5 - \frac{1.693}{2} = 75.394$ in

Depth of equivalent stress block $a = \frac{A_s * f_y}{0.85 * f'_c * b} = \frac{27.84*60}{0.85*4*86.61} = 5.67$

Factored resistant moment $\phi M_n = \phi \cdot A_s f_y \cdot (d_s - a/2) = \frac{0.9*27.84*60*(75.394 - 5.67/2)}{12} = 9090.2\ \text{kip} - \text{ft}$

Check $\phi M_n \geq +M_u \longrightarrow 9090.2$ kip-ft $\geq$ 8063 kip-ft OK

3.2.3.3.2. Negative flexure design

$M_u = -6955.6\ \text{kip} - \text{ft controlls}$

Try #18 bar

Bar spacing s = 11 in

Bar diameter d_b = 2.257 in = 57.3 mm

Bar area A_b = 4 in^2 = 2581 mm^2

Area of steel reinforcement $A_s = b \cdot \frac{Ab}{s} = 86.61 * \frac{4}{11} = 31.49\ \text{in}^2$

Cover c_b = 3 in

Effective depth $d_s = t_{cap} - c_b - \frac{db}{2} = 78.74 - 3 - \frac{2.257}{2} = 74.61$ in

Depth of equivalent stress block $a = \frac{A_s * f_y}{0.85 * f'_c * b} = \frac{31.49*60}{0.85*4*86.61} = 6.42$

Factored resistant moment $\phi M_n = \phi \cdot A_s f_y \cdot (d_s - a/2) = \frac{0.9*31.49*60*(74.61 - 6.42/2)}{12} = 10117.7\ \text{kip} - \text{ft}$

Check $\phi M_n \geq -M_u \longrightarrow$ -10,117.7kip-ft ≥ -6995.6 kip-ft OK

3.2.3.4. Section design for Service I Limit State

In serviceability design stage permit loads are not considered.

3.2.3.4.1. Cracking at the bottom of the deck

$$n = \frac{E_s}{E_c} = \frac{29000}{3645} = 7.96$$

$$d_c = c_{bot} + \frac{d_b}{2} = 2.5 + \frac{1.693}{2} = 3.347 \text{ in}$$

$$\beta_s = 1 + \frac{d_c}{0.7 * (t_{deck} - d_c)} = 1 + \frac{3.347}{0.7 * (78.74 - 3.347)} = 1.063$$

$$\rho = \frac{A_s}{b * d_s} = \frac{27.84}{86.61 * 75.394} = 0.00426$$

$$k = \sqrt{2 * n * \rho + (n * \rho)^2} - n * \rho = \sqrt{2 * 7.96 * 0.00426 + (7.96 * 0.00426)^2} - 7.96 * 0.00426 = 0.229$$

$$j = 1 - \frac{k}{3} = 1 - \frac{0.229}{3} = 0.924$$

$$f_{ss} = \frac{+M_{u,service}}{A_s * j * d_s} = \frac{3593.8 * 12}{27.84 * 0.924 * 75.394} = 22.24 \text{ ksi}$$

$$s_{max} = \frac{700 * \gamma_e}{\beta_s * f_{ss}} - 2 * d_c = \frac{700 * 1.00}{1.063 * 22.24} - 2 * 3.347 = 22.91 \text{ in}$$

Spacing used 7 in ≤ s_{max} = 22.91 in OK

3.2.3.4.2. Cracking at the top of the deck

$$d_c = c_{bot} + \frac{d_b}{2} = 3 + \frac{2.257}{2} = 4.13 \text{ in}$$

$$\beta_s = 1 + \frac{d_c}{0.7 * (t_{deck} - d_c)} = 1 + \frac{4.13}{0.7 * (78.74 - 4.13)} = 1.079$$

$$\rho = \frac{A_s}{b * d_s} = \frac{31.49}{86.61 * 74.61} = 0.0049$$

$$k = \sqrt{2 * n * \rho + (n * \rho)^2} - n * \rho = \sqrt{2 * 7.96 * 0.0049 + (7.96 * 0.0049)^2} - 7.96 * 0.0049 = 0.243$$

$$j = 1 - \frac{k}{3} = 1 - \frac{0.243}{3} = 0.919$$

$$f_{ss} = \frac{-M_{u,service}}{A_s * j * d_s} = \frac{-3325.6 * 12}{31.49 * 0.919 * 74.61} = 18.48 \text{ ksi}$$

$$s_{max} = \frac{700 * \gamma_e}{\beta_s * f_{ss}} - 2 * d_c = \frac{700 * 1.00}{1.079 * 18.48} - 2 * 4.13 = 26.85 \text{ in}$$

Spacing used 11 in ≤ s_{max} = 26.85 in OK

3.2.3.5. Section design for Fatigue I Limit State

Table 3.5. Unfactored moments from HL-93 fatigue truck.

Load	Max and Min moments at midspan (kip-ft)		Max and Min moments at column faces (kip-ft)	
	+M	-M	+M	-M
DC	2317.9	NA	NA	-2234.14
DW	504.9	NA	NA	-474.6
Fatigue Vehicle	658.3	-76.2	56.8	-522.9

Factored loads for midspan:

$$M_{u,max} = +M_{DC} + (+M_{DW}) + 1.75 * (+M)_{fatigue} = 2317.9 + 504.9 + 1.75 * 658.3$$
$$= 3974.8 \text{ kip} - \text{ft}$$

$$M_{u,min} = +M_{DC} + (+M_{DW}) + 1.75 * (-M)_{fatigue} = 2317.9 + 504.9 + 1.75 * (-76.2)$$
$$= 2689.5 \text{ kip} - \text{ft}$$

Bottom steel reinforcement check:

$$f_{s,max} = \frac{+M_{fatigue}}{A_s * j * d_s} = \frac{3974.8 * 12}{27.84 * 0.924 * 75.394} = 24.6 \text{ ksi}$$

$$f_{s,min} = \frac{+M_{fatigue}}{A_s * j * d_s} = \frac{2689.5 * 12}{27.84 * 0.924 * 75.394} = 16.6 \text{ ksi}$$

$$\gamma * (\Delta f) = f_{s,max} - f_{s,min} = 24.6 - 16.6 = 8 \text{ ksi}$$

$$(\Delta F)_{TH} = 24 - 0.33 * f_{s,min} = 18.5\ ksi$$

$$\gamma * (\Delta f) < (\Delta F)_{TH} \longrightarrow 8 < 18.5 \text{ OK.}$$

Top steel reinforcement check:

$$M_{u,max} = -M_{DC} + (-M_{DW}) + 1.75 * (-M)_{fatigue} = -2234.14 - 474.6 - 1.75 * (-522.9)$$
$$= -3623.8 \text{ kip} - \text{ft}$$

$$M_{u,min} = -M_{DC} + (-M_{DW}) + 1.75 * (-M)_{fatigue} = -2234.14 - 474.6 + 1.75 * (56.8)$$
$$= -2609.34 \text{ kip} - \text{ft}$$

$$f_{s,max} = \frac{+M_{fatigue}}{A_s * j * d_s} = \frac{3623.8 * 12}{31.49 * 0.919 * 74.61} = 20.14 \text{ ksi}$$

$$f_{s,min} = \frac{+M_{fatigue}}{A_s * j * d_s} = \frac{2609.34 * 12}{31.49 * 0.919 * 74.61} = 14.5 \text{ ksi}$$

$$\gamma * (\Delta f) = f_{s,max} - f_{s,min} = 20.14 - 14.5 = 5.64 \text{ ksi}$$

$$(\Delta F)_{TH} = 24 - 0.33 * f_{s,min} = 19.215 \text{ ksi}$$

$$\gamma * (\Delta f) < (\Delta F)_{TH} \longrightarrow 5.64 < 19.215 \text{ OK.}$$

3.2.4. Shear design

Table 3.6. Drop bent cap reactions at column faces.

Loads	Shear demand	Associated Moments
DC	489.6	-2234.14
DW	103	-474.6
HL-93	146.8	68.9
Permit Vehicle	619	244

Strength I Limit State

$$V_u = 1.25 * V_{DC} + 1.5 * V_{DW} + 1.75 * V_{HL-93} = 1.25 * 489.6 + 1.5 * 103 + 1.75 * 146.8 = 1023.4 \text{ kip}$$

$$M_u = 1.25 * M_{DC} + 1.5 * M_{DW} + 1.75 * M_{HL-93}$$
$$= 1.25 * (-2234.14) + 1.5 * (-474.6) + 1.75 * 68.9 = 3384 \text{ kip} - \text{ft}$$

Strength II Limit State

$$V_u = 1.25 * V_{DC} + 1.5 * V_{DW} + 1.35 * V_{HL-93} = 1.25 * 489.6 + 1.5 * 103 + 1.35 * 619 = 1602.15 \text{ kip}$$

$$M_u = 1.25 * M_{DC} + 1.5 * M_{DW} + 1.75 * M_{HL-93}$$
$$= 1.25 * (-2234.14) + 1.5 * (-474.6) + 1.35 * 244 = 3175.8 \text{ kip} - \text{ft}$$

$$1602.15 \text{ kip} > 1023.4 \text{ kip} \longrightarrow \text{Strength II governs}$$

$$d_e = d_{cap} - 2.5 = 78.74 - 2.5 = 76.24 \text{ in}$$

$$a = \frac{(A'_s * f_y - A_s * f_y)}{0.85 * f'_c * b} = \frac{(31.49 * 60 - 27.84 * 60)}{0.85 * 4 * 86.61} = 0.744 \text{ in}$$

$$d_{v1} = d_e - \frac{a}{2} = 76.24 - \frac{0.744}{2} = 75.87 \text{ in}$$

$$d_v = \max\left(d_{v1}, 0.9 * d_e, 0.72 * d_{cap}\right) = 75.87 \text{ in}$$

$$v_u = \frac{V_u}{\phi * b * d_v} = \frac{1602.15}{0.9 * 86.61 * 75.87} = 0.271 \text{ ksi}$$

Shear stress factor:

$$v_u = \frac{v_u}{f_c} = \frac{0.271}{4} = 0.0678$$

Assuming 0.5 * cotθ = 1

$$\varepsilon_x = \frac{\frac{M_u}{d_v} + 0.5 * V_u * \cot\theta}{2 * E_s * A'_s} = \frac{\frac{3175.8 * 12}{75.87} + 1602.15}{2 * 29000 * 31.49} = 0.00115$$

According to AASHTO Table B5.2-1, from the ε_x*1000, and shear stress factor v_u, β = 2.23 and θ = 36.4.

Recalculate ε_x:

$$\varepsilon_x = \frac{\frac{M_u}{d_v} + 0.5 * V_u * \cot\theta}{2 * E_s * A'_s} = \frac{\frac{3175.8 * 12}{75.87} + 1602.15 * 0.5 * \cot(36.4)}{2 * 29000 * 31.49} = 0.00087$$

The AASHTO Table B5.2-1 suggests the same values β = 2.23 and θ = 36.4, thus convergence in calculations has been reached.

3.2.4.1. Shear reinforcement

Shear resistance from concrete portion:

$$V_c = 0.0316 * \beta * \sqrt{f_c} * b * d_v = 0.0316 * 2.23 * \sqrt{4} * 86.61 * 75.87 = 926.1 \text{ kips}$$

$$V_s = \frac{V_u}{\phi} - V_c = \frac{1717.1}{0.9} - 926.1 = 982 \text{ kips}$$

Needed shear stirrups:

$$\frac{A_v}{s} = \frac{V_u}{f_y * d_v * \cot\theta} = \frac{1717.1}{60 * 75.87 * \cot(36.4)} = 0.278\ \mathrm{in^2/in}$$

Check minimum required area

$$(\frac{A_v}{s})_{min} = 0.0316 * \sqrt{f_c}\frac{b}{f_y} = 0.0316 * \sqrt{4} * \frac{86.61}{60} = 0.0912\ \mathrm{in^2/in}$$

$$\text{Use} \quad \frac{A_v}{s} = 0.278\ \mathrm{in^2/in}$$

Spacing:

Try four legs of #7 bar $A_b = 0.6\ \mathrm{in^2}$, $A_v = 4 * 0.6 = 2.4\ \mathrm{in^2}$

$$\text{Spacing } s = \frac{2.4}{0.278} = 8.6\ \mathrm{in}$$

$$\text{Maximum spacing requirement } s_{max} \text{ for } v_u < 0.125 \longrightarrow \min(0.8 * d_v, 24\ \mathrm{in}), s_{max} = 24\ \mathrm{in}$$

Take spacing s = 8 in

Longitudinal reinforcement spacing

AASHTO 5.10.3.1.1 requires that distance between parallel bars be less than max (1.5 *d_b, 1.5* max aggregate size, 1.5 in)

$$s_{min} = 1.5 * 4 = 6\ \mathrm{in}$$

Current spacing between positive flexural reinforcement, taking into account clear cover and #14 longitudinal bars and #7 transverse reinforcement approximate outside diameters.

Side face reinforcement

AASHTO 5.7.3.4 requires that if sections are more than 3 in deep, then longitudinal skin reinforcement should be at both faces of the component at a distance $d_e/2$ near flexural reinforcement. Area of the skin reinforcement is calculated through following equation:

$$A_{sk} = 0.012 * (d_e - 30) = 0.012 * (76.24 - 30) = 0.55\ \mathrm{in^2/ft}$$

$$A_{sk,max} = \frac{\max[A'_s, A_s]}{4} = \frac{31.49}{4} = 7.87\ \mathrm{in^2}$$

$$s_{max} = \frac{76.24}{6} = 12.7 \text{ in or should be less than 12 in}$$

Try #7@9 in bars at each face

3.3. Column design

Columns in bridges are used for transferring vertical loads from the superstructure and resisting lateral loads such as seismic and wind loads.

3.3.1. Types of columns

Columns are divided according to 2 parameters, such as shape and height. Shapes are usually round, hollow, rectangular or hexagonal. Second parameter is height, which suggests that columns are tall or short. To determine the height parameter it is needed to know K – effective length factor, l_u – unsupported length of a compression member and r – radius of gyration. The height of columns is h = 12 m = 39.37 ft.

$$\text{Slenderness ratio} = \frac{K * l_u}{r}$$

$$r = 0.25 * 47.24 = 11.8$$

3.3.2. Loads

Columns are designed for dead loads (DC), wearing surface load (DW), permit truck vehicle load (P_{15}), wind load and seismic loads. These are included into Strength II Limit State.

3.3.3. Column reinforcement

Column diameter $D_c = 1.2 \text{ m} = 47.24 \text{ in}$

$$A_g = \frac{\pi * 47.24^2}{4} = 1753 \text{ in}^2$$

Table 3.7. Loads imposed on the column and their reactions at the column.

Load type	Moment (kip-ft)	Axial force (kips)
DC	701	768
DW	149	153
P-15	643	795
Wind load	11	0.76
Seismic load	158	9.14

AASHTO suggests that columns are to be designed for Strength II limit state, thus:

$$M_u = 1.25 * M_{DC} + 1.5 * M_{DW} + 1.75 * (LL + IM)M_{P-15}$$
$$= 1.25 * 701 + 1.5 * 149 + 1.35 * 1.35 * 643 = 2271.6 \text{ kip} - \text{ft}$$

$$P_u = 1.25 * P_{DC} + 1.5 * P_{DW} + 1.75 * (LL + IM)P_{P-15} = 1.25 * 768 + 1.5 * 153 + 1.35 * 1.35 * 795 = 2638.4 \text{ kips}$$

In order to properly design a column section it is needed to consider both flexure and axial loads in the design thus firstly finding the eccentricity. For circular columns ϕ = 0.75.

$$P_n = \frac{P_u}{\phi} = \frac{2638.4}{0.75} = 3517.9 \text{ kips}$$

$$M_n = \frac{2271.6 * 12}{0.75} = 36345.6 \text{ kip} - \text{in}$$

$$\text{eccentricity e} = \frac{M_n}{P_n} = \frac{36345.6}{3517.9} = 10.33 \text{ in}$$

$$\text{Cover c} = 2 \text{ in}$$

$$\frac{e}{h} = \frac{10.33}{47.24 - 2 * 2} = 0.24$$

$$K_n = \frac{P_n}{f_c * A_g} = \frac{3517.9}{4 * 1753} = 0.5$$

$$R_n = K_n * \frac{e}{h} = 0.5 * 0.24 = 0.12$$

$$\gamma = \frac{h'}{h} = \frac{47.24 - 2 * 2}{47.24} \approx 0.9$$

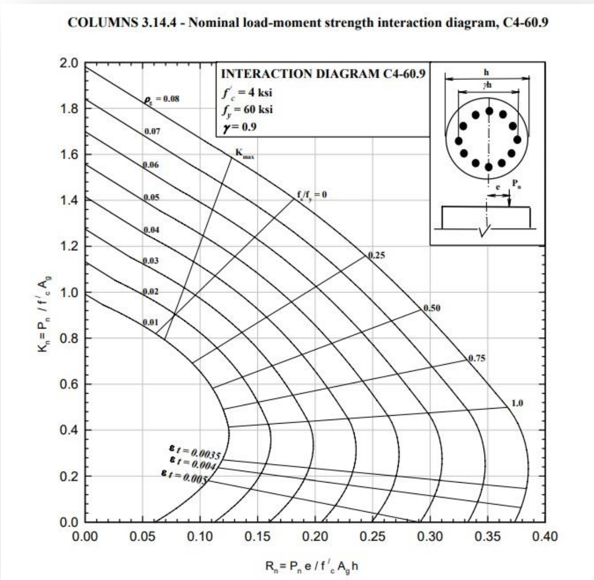


Figure 3.3. Nominal load – moment interaction diagram.

From figure 3.3. is it clear that needed reinforcement for current column design is ρ_g = 0.01*A_g = 0.01*1753 = 17.53 in^2.

Try #9 bars

$$d_b = 1.128 \text{ in} = 32.26 \text{ mm}$$

$$A_b = 1 \text{ in}^2 = 645 \text{ mm}^2$$

Choose 18-#9 bars

For spiral reinforcement appropriate procedure will be followed:

Try #4 bars

$$d_b = 0.5 \text{ in} = 9.525 \text{ mm}$$

$$A_b = 0.2 \text{ in}^2 = 129 \text{ mm}^2$$

$$D_c = 47.24 - 2 * 2 = 43.24 \text{ in}$$

$$A_c = \frac{\pi * D_c^2}{4} = 1468.5 \text{ in}^2$$

Ratio of spiral reinforcement

$$\rho_{s,min} = 0.45 * \left(\frac{A_g}{A_c} - 1\right) * \frac{f_c}{f_y} = 0.45 * \left(\frac{1753}{1468.5} - 1\right) * \frac{4}{60} = 0.0058$$

$$\text{Spacing } s_{max} = \frac{4*A_b*(D_c - d_b)}{\rho_{s,min}*D_c^2} = \frac{4*0.2*(43.24-0.5)}{0.0058*43.24^2} = 3.15 \text{ in}$$

Select spacing s = 3 in

3.3.4. Wind load on column

AASHTO Article 3.8.1.2 suggests that wind load should be calculated for design wind velocity of $V_B = 100$ mph, which then should be used to design bridge members at certain height. The velocity of wind V_{DZ} at specific height:

$$V_{DZ} = 2.5 * V_0 * \left(\frac{V_{30}}{V_B}\right) * \ln\left(\frac{Z}{Z_0}\right)$$

Suburban area is assumed since bridge is design for the outskirts of Jeonju, thus $V_0 = 10.9$ mph, $Z_0 = 3.28$ ft. V_{30} can be assumed to be $V_{30} = V_B = 100$ mph.

The height of the bridge Z = 53.4 ft.

$$V_{DZ} = 2.5 * \ 10.9 * \left(\frac{100}{100}\right) * \ln\left(\frac{53.4}{3.28}\right) = 76.03 \text{ mph}$$

$$P_B = 0.05 \text{ for superstructure}$$

$$P_B = 0.04 \text{ for columns}$$

$$P_D = \ P_B * \left(\frac{V_{DZ}}{V_B}\right)^2 = \ 0.05 * \left(\frac{76.03}{100}\right)^2 = 0.0289 \text{ ksf for superstructure}$$

$$P_D = \ P_B * \left(\frac{V_{DZ}}{V_B}\right)^2 = \ 0.04 * \left(\frac{76.03}{100}\right)^2 = 0.0231 \text{ ksf for columns}$$

$$P_D = 3.94 * 0.0231\text{ksf} = 0.0091 \text{ kip/ft of load}$$

Table 3.8. Wind load on structure.

Wind load on structure	
M_y (kip-ft)	-83
M_x (kip-ft)	52.9
P (kip)	0.76

3.3.5. Seismic load on column

Current load will be designed according to ASCE 7 10. First, mapped 5% damped spectral response acceleration parameters at 0.2 s and 1 s are S_s = 0.012g, S_1 = 0.0048g. Assuming site class D, F_a = 1.6 and F_v = 2.4 the equations yield:

$$S_{MS} = F_a * S_s = 1.6 * 0.012\text{g} = 0.0192\text{g}$$

$$S_{M1} = F_v * S_1 = 2.4 * 0.0048\text{g} = 0.01152\text{g}$$

$$S_{DS} = 2 * \frac{S_{MS}}{3} = 2 * \frac{0.0192}{3} = 0.0128\text{g}$$

$$S_{D1} = 2 * \frac{S_{M1}}{3} = 2 * \frac{0.01152}{3} = 0.00768\text{g}$$

From the map of long period transition T_L = 4s

Period limiting values $T_0 = 0.2 * \frac{S_{D1}}{S_{DS}} = 0.2 * \frac{0.00768}{0.0128} = 0.12\text{s}$

$$T_S = \frac{S_{D1}}{S_{DS}} = \frac{0.00768}{0.0128} = 0.6\text{s}$$

Approximate fundamental period: $T_a = C_t * h_n^x$, where for concrete moment resisting frames $C_t = 0.016$ and x = 0.9

$$T_a = 0.016 * 53.4^{0.9} = 0.574 \text{ s}$$

$$C_s = \frac{S_{DS} * I_e}{R} = \frac{0.0128 * 1.5}{3} = 0.0064$$

$$\text{since } T_a < T_L \longrightarrow 0.574\text{s} < 4\text{s then } C_s < \frac{S_{D1} * I_e}{T * R} = \frac{0.00768 * 1.5}{0.574 * 3} = 0.0067 \text{ OK}$$

$$\text{since } S_1 < 0.6\text{g} \longrightarrow 0.0048\text{g} < 0.6\text{g then } C_s > \max[\,0.044 * S_{D1} * I_e = 0.00084, 0.01], \text{since } 0.01 > 0.00084, \text{thus}$$

$$\text{thus } C_s = 0.01$$

The weight on top of the frame is calculated

$$w = (4163 + 18 + 683) * 2 = 9728 \text{ kN} = 2187 \text{ kips of weight at the base of the structure}$$

Thus base shear force will be $V = C_s * W = 0.01 * 2187 \text{ kips} = 21.87 \text{ kips}$

At the height of the column shear force will be the same, since the frame has only 1 story.

3.3.6. P-Δ Effect

Effect of load from seismic impact V_s = 21.87 kips = 97.3 kN

Effect of wind load V_w = 0.0091 kip/ft = 0.132 kN/m

In this design calculation frame combination of drop bent cap and columns will be assumed to be regular portal frame. The load on columns have already been calculated in the column design section. Total load on single column is $P_u = 2368.4$ kips = 10,535 kN.

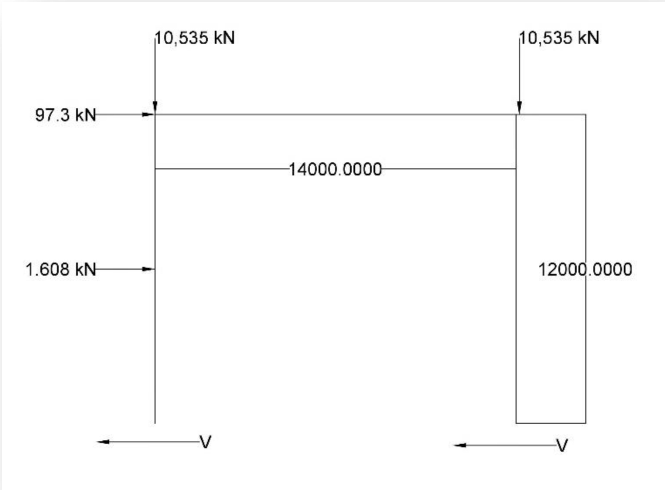


Figure 3.4. Lateral loads on frame.

In this case first it is needed to conduct shear racking drift and flexural drift calculations.

Section properties for the columns and bent cap.

$$I_c = \frac{1200^{\ 4} * \pi}{4} = 1.62 * 10^{12}\ mm^4, I_b = \frac{2200 * 2000^3}{12} = 1.47 * 10^{12}\ mm^4$$

$$E_c = 3654\ ksi = 25193\ MPa, \text{area of column}\ A_c = \frac{\pi * 1200^2}{4} = 1130973\ mm^2$$

3.3.6.1. Shear racking drift

$$97.3\ kN = V + V \longrightarrow V = 48.65\ kN$$

$$\delta^c = \frac{V * h^3}{24 * E_c} = \frac{2 * 48.65 * \ 10^3 * 12000^3}{24 * 25193 * 1.62 * 10^{12}} = 0.17\ mm$$

$$\delta^b = \frac{V * h^3}{24 * E_c} = \frac{2 * 48.65 * \ 10^3 * 12000^2 * 14000}{24 * 25193 * 1.47 * 10^{12}} = 0.11\ mm$$

$$\delta^t = \ \delta^c + \ \delta^b = 0.17 + 0.11 = 0.28\ mm$$

3.3.6.2. Flexural drift

$$\text{Total moment}\ M = 97.3 * 12 + 1.608 * 6 = 1177.2\ kNm$$

$$mL = 14 * 1 = 14\ m$$

$$N = \frac{M}{mL} = \frac{1177.2}{14} = 84.09\ kN$$

$$\varepsilon = \frac{N}{A_c * E_c} = \frac{84.09 * \ 10^3}{1130973 * 25193} = 2.95 * \ 10^{-6}$$

$$\phi = \frac{\varepsilon}{2mL} = \frac{2.95 * 10^{-6}}{2 * 14} = 1.054 * 10^{-7}$$

$$\theta = \frac{\varepsilon * h}{2 * m * L} = \frac{1.054 * 10^{-7} * 12000}{2 * 14} = 4.52 * \ 10^{-5} \text{ mm}$$

$$\text{Total drift } \Delta = 0.28 \text{ mm} + 4.52 * \ 10^{-5} \text{ mm} = 0.28 \text{ mm}$$

3.3.6.3. P-Δ drift

$$Q = \frac{P_u * \ \Delta}{V_{us} * L_c} = \frac{10535 * 10^3 * 0.28}{97.3 * \ 10^3 * 12000} = 2.53 * \ 10^{-3}$$

$$\beta = \frac{1}{1 - Q} = \frac{1}{1 - 2.523 * 10^{-3}} = 1.003$$

$$\Delta_t = \beta * \Delta = 1.003 * 0.28 = 0.281 \text{ mm}$$

3.4. Girders

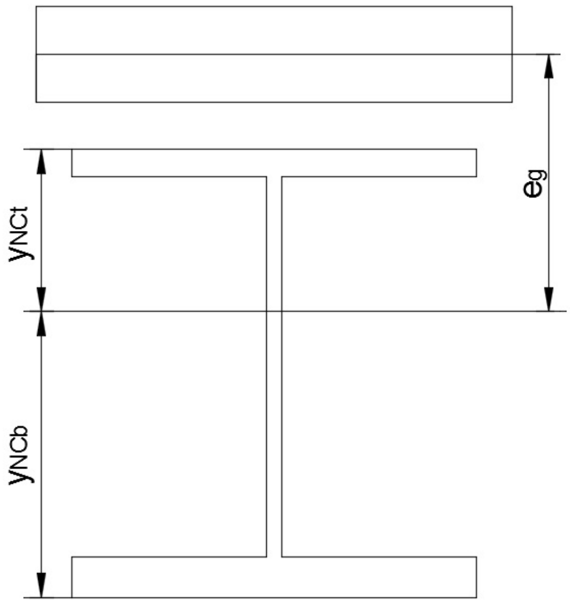
3.4.1. Calculate live load distribution factor

3.4.4.1. Check ranges of applicability of live load distribution factor

The live load distribution factor of a composite steel plate girder bridges is dependent on the girder spacing S, span length L, concrete slab depth t_s, longitudinal stiffness parameter K_g, and number of girders N_b. According to AASHTO Table 4.6.2.2.1-1, composite plate girder bridge is categorized as Type "a" structure. Thus, longitudinal stiffness parameter, K_g, is calculated in accordance with AASHTO 4.6.2.2.1-1.

Table 3.9. Preliminary section properties.

Component	**b or D (mm)**	**t (mm)**	**A_i (mm^2)**	**y_i (mm)**	**A_i*y_i (mm^3)**	**$y_i - y_{NCb}$ (mm)**	**$A_i (y_i - y_{NCb})^2$ (mm^4)**	**I_0 (mm^4)**
top flange 425x28	425	28	11900	2056	2.45E+07	1110.6	1.47E+10	7.77E+05
web 2000x17	2000	17	34000	1042	3.54E+07	96.6	3.17E+08	1.13E+10
bottom flange 425x42	425	42	17850	21	3.75E+05	-924.4	1.53E+10	6.25E+04
$\sum$			63750		6.03E+07		3.02E+10	1.13E+10



$y_{NCb} = \frac{\sum A_i y_i}{\sum A_i} = \frac{6.03*10^7}{63750} = 9.45*10^2$ mm

$y_{NCt} = 2070 - y_{NCb} = 1.1246*10^3$ mm

$I_{NC} = \sum I_0 + \sum A_i(y_i - y_{NCb})^2 = 4.16 * 10^{10}$mm^4

$e_g = y_{NCt} + 150 = 1274.6$ mm

$K_g = n(I_{NC} + Ae_g^2) = 8(4.16*10^{10} + 63750(1274.6)^2) = 1.16*10^{12}$mm^4

Composite steel plate girder bridge parameters are provided below in order to check the ranges of applicability of AASHTO Tables 4.6.2.2.2b-1 and 4.6.2.2.3a-1.

Girder spacing: 1.1m<S=3.5m<5m

Span length: 6.1m<L=(42m, 52m)<73.2m

Concrete deck: 112.5mm < t_s=200mm<300mm

Number of girders: N_b=5>4

Stiffness parameter: $3.9*10^9$mm^4< K_g=$1.16*10^{12}$mm^4<$2.73*10^{12}$mm^4

Bridge parameters meet the limitations specified in AASHTO Tables 4.6.2.2.2b-1 and 4.6.2.2.3a-1, hence, section Type “a” will be used.

3.4.4.2. Span length definition for use in live load distribution factor computation

Span length, L, shall be defined with the provision of AASHTO Table C4.6.2.2.1-1.

3.4.4.3. Live load distribution factor calculation

Live load distribution factors for Strength Limit State and Fatigue Limit State are calculated and listed in tables 3.4.2 and 3.4.3 in compliance with AASHTO Tables 4.6.2.2.2b-1 and 4.6.2.2.3a-1.

-One design lane loaded

$DF_m = 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12Lt_s^3}\right)^{0.1}$; $DF_v = 0.36 + \frac{S}{25}$

Two or more design lanes loaded

$$DF_m = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12Lt_S^3}\right)^{0.1}; \quad DF_v = 0.2 + \frac{S}{12} - \left(\frac{S}{35}\right)^2$$

$$K_g = 1.16*10^{12}\ mm^4$$

$$S = 3500mm$$

$$t_s = 200mm$$

Table 3.10.Live load distribution factors for interior girder for Strength Limit State.

		Moment DFm (Lane)		Shear DFv (Lane)	
Span	**Lane loaded**	**One**	**Two or more**	**One**	**Two or more**
1*	138	0.557	0.848	0.819	1.049
1&2**	154	0.535	0.822	0.819	1.049
2*	171	0.516	0.800	0.819	1.049
2&3**	171	0.516	0.800	0.819	1.049
3*	171	0.516	0.800	0.819	1.049
3&4**	171	0.516	0.800	0.819	1.049
4*	171	0.516	0.800	0.819	1.049
4&5**	171	0.516	0.800	0.819	1.049
5*	171	0.516	0.800	0.819	1.049
5&6**	171	0.516	0.800	0.819	1.049
6*	171	0.516	0.800	0.819	1.049
6&7**	171	0.516	0.800	0.819	1.049
7*	171	0.516	0.800	0.819	1.049
7&8**	171	0.516	0.800	0.819	1.049
8*	171	0.516	0.800	0.819	1.049
8&9**	171	0.516	0.800	0.819	1.049
9*	171	0.516	0.800	0.819	1.049
9&10**	154	0.535	0.822	0.819	1.049
10*	138	0.557	0.848	0.819	1.049

Note:

*The span length for which moment is being calculated for positive moment, negative moment – other than near interior supports of continuous spans, shear, and exterior reaction

**Average span length for negative moment-near interior supports of continuous spans from point of contra-flexure to point of contra-flexure under a uniform load on all spans, and interior reaction of continuous span.

Multiple presence factor have been included in the above live load DF.

Table 3.11. Live load distribution factors for interior girder for Fatigue Limit State.

		Moment DFm (Lane)	Shear DFv (Lane)
Span	**Lane Loaded**	**One**	**One**
1	138	0.464	0.683
1&2	154	0.446	0.683
2	171	0.430	0.683
2&3	171	0.430	0.683
3	171	0.430	0.683
3&4	171	0.430	0.683
4	171	0.430	0.683
4&5	171	0.430	0.683
5	171	0.430	0.683
5&6	171	0.430	0.683
6	171	0.430	0.683
6&7	171	0.430	0.683
7	171	0.430	0.683
7&8	171	0.430	0.683
8	171	0.430	0.683
8&9	171	0.430	0.683
9	171	0.430	0.683
9&10	154	0.446	0.683
10	138	0.464	0.683

Note:

*The span length for which moment is being calculated for positive moment, negative moment – other than near interior supports of continuous spans, shear, and exterior reaction

**Average span length for negative moment-near interior supports of continuous spans from point of contra-flexure to point of contra-flexure under a uniform load on all spans, and interior reaction of continuous span.

3.4.2. Load and resistance factors and load combinations

Composite plate girder bridge usually designed for the Strength Limit State, and checked for Fatigue Limit State, Service Limit State II, and Constructability.

3.4.2.1. Design equation

According to AASHTO LRFD Bridge Design Specification, the design equation shall be as follows:

$$\Sigma\eta_i\gamma_i Q_i \leq \phi R_n = R_r \quad \text{(AASHTO 1.3.2.1-1)}$$

$$\gamma_i - load\ factor$$

$$\phi - load\ factor$$

$$\eta_i - load\ modifier\ factor\ related\ to\ ductility, redundancy, and\ operational\ importance$$

$$\eta_i = \eta_D\eta_R\eta_I \geq 0.95 \text{ (AASHTO 1.3.2.1-2)}$$

where η_D, η_R, and η_I are ductility and redundancy and operational factors, respectively. According to CA 1.3.3, 1.3.4 and 1.3.5 $\eta_D = \eta_R = \eta_I = 1.0$, therefore, design equation will become:

$$\Sigma\gamma_i Q_i \leq \phi R_n = R_r$$

3.4.2.2. Applicable load factors and load combinations

DC = DC1 (weight of structural steel and components)+DC2(concrete deck slab)

DW – future wearing surface

(LL+IM) – unfactored force effect due to one design lane loaded

DF – distribution factor

1. Strength I: 1.25DC+1.5DW+1.75DF$(LL+IM)_{HL-93}$
2. Strength II: 1.25DC+1.5DW+1.35DF$(LL+IM)_{P15}$
3. Service II: 1.0DC+1.0DW+1.30DF$(LL+IM)_{HL-93}$
4. Fatigue I: 1.75DF$(LL+IM)_{HL-93}$
5. Fatigue II: 1.0DF$(LL+IM)_{P9}$

3.4.2.3. Applicable resistance factor

For Strength Limit State resistance factors are selected in compliance with AASHTO 6.5.4.2 and are listed in table 3.13.

Table 3.12. Strength Limit State resistance factors.

For flexure	ϕ_f = 1.0

For Shear	$\phi_v = 1.0$
For axial compression	$\phi_c = 0.9$
For tension, fracture in net section	$\phi_u = 0.8$
For tension, yielding in gross section	$\phi_y = 0.95$
For bearing on milled surfaces	$\phi_b = 1.0$
For bolts bearing on material	ϕ_{bb}=0.8
For shear connector	ϕ_{sc}=0.85
For block shear	ϕ_{bs}=0.8
For A325 bolts in shear	ϕ_s=0.8
For weld metal in fillet weld-shear in throat of weld metal	ϕ_{e2}=0.8

3.4.3. Design composite section in positive moment region at L = 16.7m of Span 1

3.4.3.1. Calculations of factored moments – Strength Limit State

Factored force effects for Strength Limit State I and Strength Limit State II are calculated and summarized in table 3.14.

Table 3.13. Factored moments at L=16.7m of Span 1.

Load type	**Unfactored Moment (kN-m)**	**Factored moment (kN-m)**
DC1	2525.96	M_{DC1} = 1.25(2525.96) = 3157.45 (applied to steel section alone)
DC2	183.58	M_{DC2} = 1.25(183.58) = 229.48 (applied to long-term composite section 3n = 24)
DW	621.62	M_{DW} = 1.5(621.62) = 932.43 (applied to long-term composite section 3n = 24)
(LL+IM)HL-93	5014	$M_{(LL+IM)HL\text{-}93}$ = 1.75*(0.848)*(5014) = 7440.8 (applied to short-term composite section n = 8)
(LL+IM)P15	9902.2	$M_{(LL+IM)P15}$ = 1.35(0.848)(9902.2) = 11336 (applied to short-term composite section n = 8)
Controlling DC+DW+(LL+IM)P15		M_u = 3157.45+229.48+932.43+11336 = 15655
Strength I: 1.25(DC)+1.5(DW)+1.75(DF)$(LL+IM)_{HL\text{-}93}$ Strength II: 1.25(DC)+1.5(DW)+1.35(DF)$(LL+IM)_{P15}$		

3.4.3.2. Elastic section properties

3.4.3.2.1. Effective flange width

CA 4.6.2.6 specifies that the effective width depends on the girder spacing to span length ratio. For end spans L=42m and S=3.5m

$\because$ S/L = 3.5/42 = 0.0833 < 0.32 (CA 4.6.2.6.1-2)

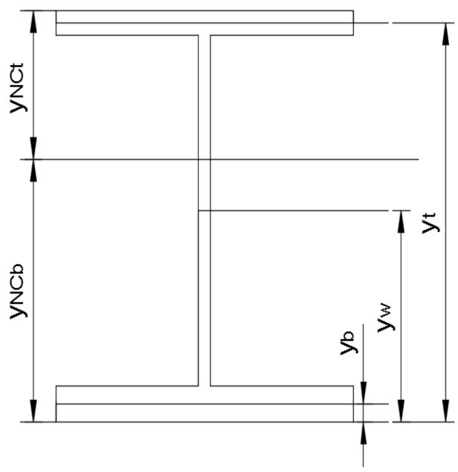
$\therefore$ b_{eff} = b = 3500 mm

3.4.3.2.2. Elastic section properties

Elastic section properties for steel section alone, the steel section and deck slab longitudinal reinforcement, the short-term composite section (n=8), and the long-term section (3n=24) are calculated and illustrated from table 3.15 to 3.18.

Table 3.14 Properties of steel section alone.

Component	**b or D (mm)**	**t (mm)**	**A_i (mm^2)**	**y_i (mm)**	**A_iy_i (mm^3)**	**$y_i - y_{NCb}$ (mm)**	**$A_i (y_i - y_{NCb})^2$ (mm^4)**	**I_0 (mm^4)**
top flange 425x28	425	28	11900	2056	2.45E+07	1110.6	1.47E+10	7.77E+05
web 2000x17	2000	17	34000	1042	3.54E+07	96.6	3.17E+08	1.13E+10
bottom flange 425x42	425	42	17850	21	3.75E+05	-924.4	1.53E+10	6.25E+04
$\sum$			63750		6.03E+07		3.02E+10	1.13E+10



$y_{NCb} = \frac{\sum A_i y_i}{\sum A_i} = \frac{6.03*10^7}{63750} = 9.45*10^2$ mm

$y_{NCt} = 2070 - y_{NCb} = 1.1246*10^3$ mm

$I_{NC} = \sum I_0 + \sum A_i(y_i - y_{NCb})^2 = 4.16 * 10^{10}$ mm^4

$S_{NCb} = \frac{I_{NC}}{y_{NCb}} = 4.4*10^7$ mm^3

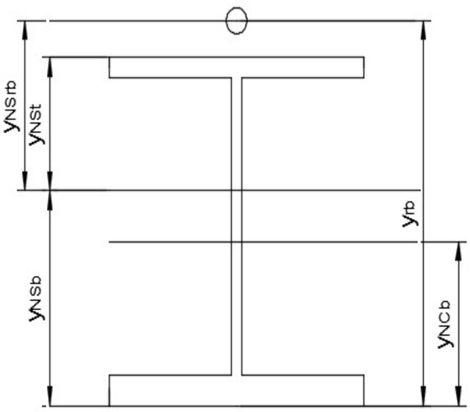
$S_{NCt} = \frac{I_{NC}}{y_{NCt}} = 3.7*10^7$ mm^3

Properties of steel section and deck slab longitudinal reinforcement are used in order to calculate stresses in negative moment region. Assume the total area of deck slab reinforcement is equal to 1% of concrete deck slab area, hence

$$A_s = 0.01(3500)(200) = 7000 \text{ mm}^2$$

Table 3.15. Properties of steel section and deck slab reinforcement.

Component	A_i (mm^2)	y_i (mm)	$A_i y_i$ (mm^3)	$y_i - y_{NSb}$ (mm)	$A_i (y_i - y_{NSb})^2$ (mm^4)	I_0 (mm^4)
Top Reinforcement	7000	2220	1.55E+07	1148.49	9.23E+09	0
Steel Section	63750	945.4	6.03E+07	-126.11	1.01E+09	4.16E+10
∑	70750		7.58E+07		1.02E+10	4.16E+10



$y_{NSb} = \frac{\sum A_i y_i}{\sum A_i} = \frac{7.58*10^7}{70750} = 1.072*10^3$ mm

$y_{NSt} = 2070 - y_{NSb} = 9.985*10^2$ mm

$y_{NSrb} = 2220 - y_{NSb} = 1.149*10^3$ mm

$I_{NS} = \sum I_0 + \sum A_i(y_i - y_{NSb})^2 = 5.18 * 10^{10}$ mm^4

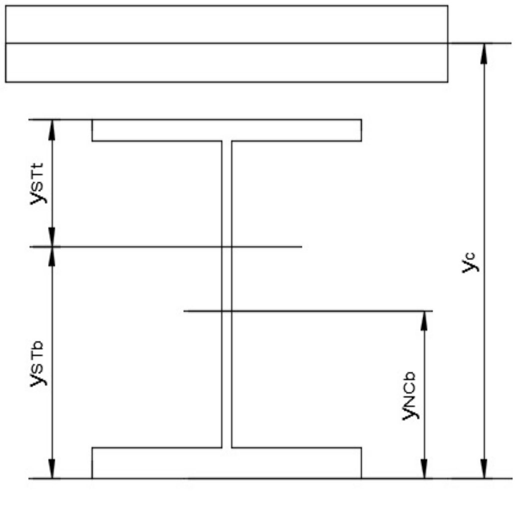
$S_{NSb} = \frac{I_{NS}}{y_{NSb}} = 4.84*10^7$ mm^3

$S_{NSt} = \frac{I_{NS}}{y_{NSt}} = 5.19*10^7$ mm^3

$S_{NSrb} = \frac{I_{NS}}{y_{NSrb}} = 4.51*10^7$ mm^3

Table 3.16. Properties of short-term section (n=8).

Component	A_i (mm^2)	y_i (mm)	$A_i y_i$ (mm^3)	$y_i - y_{STb}$ (mm)	$A_i (y_i - y_{STb})^2$ (mm^4)	I_0 (mm^4)
Steel Section	63750	945.4	6.03E+07	-737.37	3.47E+10	4.16E+10
Concrete Slab 3500/8x200	87500	2220	1.94E+08	537.23	2.53E+10	2.92E+08
∑	151250		2.55E+08		5.99E+10	4.19E+10



$y_{STb} = \frac{\sum A_i y_i}{\sum A_i} = \frac{2.55*10^8}{151250} = 1.68*10^3$ mm

$y_{STt} = 2070 - y_{STb} = 3.87*10^2$ mm

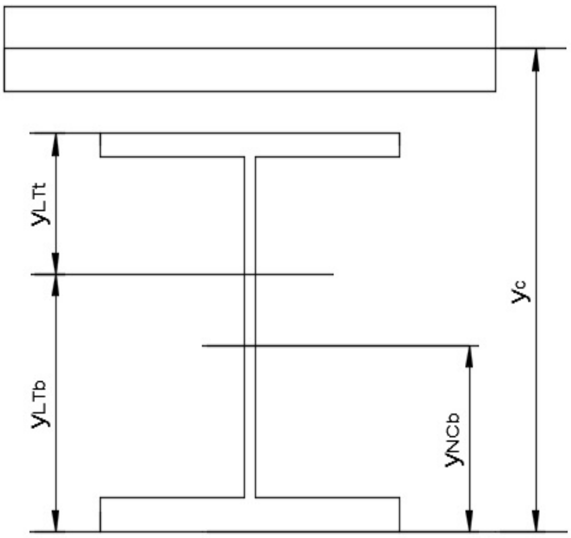
$I_{ST} = \sum I_0 + \sum A_i(y_i - y_{STb})^2 = 1.02 * 10^{11}$ mm^4

$S_{STb} = \frac{I_{ST}}{y_{STb}} = 6.05*10^7$ mm^3

$S_{STt} = \frac{I_{ST}}{y_{STt}} = 2.63*10^8$ mm^3

Table 3.17 Properties of long-term composite section (3n=24).

Component	A_i (mm^2)	y_i (mm)	A_iy_i (mm^3)	y_i-y_{LTb} (mm)	$A_i(y_i-y_{LTb})^2$ (mm^4)	I_0 (mm^4)
Steel Section	63750	945.4	6.03E+07	-400.10	1.02E+10	4.16E+10
Concrete Slab 3500/24x200	29167	2220	6.48E+07	874.50	2.23E+10	9.72E+07
$\sum$	92917		1.25E+08		3.25E+10	4.17E+10



$y_{LTb} = \frac{\sum A_i y_i}{\sum A_i} = \frac{1.25*10^8}{92917} = 1.35*10^3$ mm

$y_{STt} = 2070 - y_{NCb} = 7.25*10^2$ mm

$I_{LT} = \sum I_0 + \sum A_i(y_i - y_{LTb})^2 = 7.42 * 10^{10}$mm^4

$S_{LTb} = \frac{I_{LT}}{y_{LTb}} = 5.51*10^7$ mm^3

$S_{LTt} = \frac{I_{LT}}{y_{LTt}} = 1.02*10^8$ mm^3

Concrete haunch is excluded from elastic section properties calculations

3.4.3.3. Design for flexure – Strength Limit State

General requirement

In positive moment region the following equation has to be satisfied at the Strength Limit State (AASHTO 6.10.7.1.1-1):

$$M_u + \frac{1}{3} f_l S_{xt} \leq \phi_f M_n$$

$$f_l = 0$$

$$\therefore M_u \leq \phi_f M_n$$

Check section compactness

Specified minimum yield strength of flanges:

$$F_{yf} = 345 \text{ MPa} < 500 \text{ MPa (AASHTO 6.10.6.2.2)}$$

Web:

$$\frac{D}{t_w} = 117.7 < 150 \text{ (AASHTO 6.10.2.1.1-1)}$$

Section:

$$\frac{2D_{cp}}{t_w} \leq 3.76\sqrt{\frac{E}{F_{yc}}} \text{ (AASHTO 6.10.6.2.2-1)}$$

D_{cp} is depth of the web in compression at the plastic moment state.

Compressive force in concrete slab:

$$P_s = 0.85 f_c' b_{eff}\, t_s = 14875 \text{ kN}$$

Yield force in the top compression flange:

$$P_c = A_{fc}F_{yc} = 4105.5 \text{ kN}$$

Yield force in the web:

$$P_w = A_w F_{yw} = 11730 \text{ kN}$$

Yield force in the bottom tension flange:

$$P_t = A_{ft}F_{yt} = 6158.25 \text{ kN}$$

$$P_s + P_c = 18980.5 \text{ kN} > P_w + P_t = 17888.25 \text{ kN}$$

$$\therefore \text{ PNA is in the top compression flange} \Rightarrow D_{cp} = 0$$

$$\frac{2D_{cp}}{t_w} = 0 < 3.76\sqrt{\frac{E}{F_{yc}}} \text{ OK (AASHTO 6.10.6.2.2-1)}$$

3.4.3.3.1. Plastic moment, M_p
(AASHTO D6.1)

Determine location of Plastic Neutral Axis (PNA)

$$P_s + P_{c1} = P_{c2} + P_w + P_t$$

Where,

$$P_{c1} = \overline{y} b_{fc} F_{yc}$$

$$P_{c2} = (t_{fc} - \overline{y}) b_{fc} F_{yc}$$

$$\overline{y} = \frac{t_{fc}}{2}\left(\frac{P_w + P_t - P_s}{P_c} + 1\right) = 24.3\ mm < 28\ mm \text{ OK}$$

$$M_p = \sum M_{PNA} = P_s d_s + b_{cf} F_{yc} (\frac{\overline{y}^2 + (t_{fc} - \overline{y})^2}{2}) + P_t d_t + P_w d_w = 71128.7\ kN - m$$

3.4.3.3.2. Yield moment, M_y

$$M_y = M_{D1} + M_{D2} + M_{AD}$$

$$M_{D1} = M_{DC1} = 1.25(2547.7) = 3184.6 \text{ kN-m}$$

$$M_{D2} = M_{DC2} + M_{DW} = 1.25(185.2) + 1.5(626.9) = 1171.9 \text{ kN-m}$$

$$M_{AD} = S_{ST}(F_y - \frac{M_{D1}}{S_{NC}} - \frac{M_{D2}}{S_{LT}})$$

For the top flange:

$$M_{AD} = S_{STt}(F_y - \frac{M_{D1}}{S_{NCt}} - \frac{M_{D2}}{S_{LTt}}) = 2.63*10^8(345 - \frac{3184.6*10^6}{3.7*10^7} - \frac{1171.9*10^6}{1.02*10^8}) = 65077 \text{ kN-m}$$

For the bottom flange:

$$M_{AD} = S_{STb}(F_y - \frac{M_{D1}}{S_{NCb}} - \frac{M_{D2}}{S_{LTb}}) = 6.05*10^7(345 - \frac{3184.6*10^6}{4.4*10^7} - \frac{1171.9*10^6}{5.51*10^7}) = 15207 \text{ kN-m (control)}$$

$$\Rightarrow M_y = M_{D1} + M_{D2} + M_{AD} = 3184.6 + 1171.9 + 15207 = 19563.5 \text{ kN-m}$$

Flexural resistance:

$$M_n = min \begin{cases} M_p \ for \ D_p \leq 0.1 D_t \\ M_p \left(1 - \left(1 - \frac{M_y}{M_p}\right) \left(\frac{D_p/D_t - 0.1}{0.32} \right) \right) for \ D_p > 0.1 D_t \\ 1.3 R_h M_y \ for \ a \ continuous \ span \end{cases} \text{(AASHTO and CA 6.10.7.1.2-1,2,3)}$$

$R_h=1$ – hybrid factor (AASHTO 6.10.1.10.1)

$D_p \leq 0.42 D_t$ (AASHTO 6.10.7.3-1)

D_p = 274.3 mm

D_t = 2320 mm $\Rightarrow 0.42 D_t$ = 974.4 mm > D_p = 274.3 mm

$$M_n = 71128.7\left(1 - \left(1 - \frac{19563.5}{71128.7}\right)\left(\frac{274.3/_{2320} - 0.1}{0.32}\right)\right) = 68191 \text{ kN-m}$$

$$M_n = 1.3(1.0)(19563.5) = 25432.6 \text{ kN-m}$$

$$\Rightarrow M_n = 25432.6 \text{ kN-m}$$

Check design requirement:

$$M_u = 15655 \text{ kN-m} < \phi_f M_n = 25432.6 \text{ kN-m}$$

3.4.3.4. Fatigue moment ranges

Fatigue I:

$$M = \gamma(DF_m)(LL+IM)_{\text{HL-93}} = 1.75(0.464)(LL+IM)_{\text{HL-93}}$$

$$+M = 1.75(0.464)(3249.1) = 2638.3 \text{ kN-m}$$

$$-M = 1.75(0.464)(-843.2) = -684.7 \text{ kN-m}$$

Fatigue II:

$$M = \gamma(DF_m)(LL+IM)_{\text{P9}} = 1.0(0.464)(LL+IM)_{\text{P9}}$$

$$+M = 1.0(0.464)(9306.6) = 4318.3 \text{ kN-m}$$

$$-M = 1.0(0.464)(-3353.9) = -1556.2 \text{ kN-m}$$

3.4.3.4.1. Check typical girder details – fatigue limit states

Fatigue I – HL-93 truck for infinite life:

Flexural fatigue stress ranges at the bottom flange:

$$\gamma(\Delta f) = \left|\frac{+M}{S_{STb}}\right| + \left|\frac{-M}{S_{NCb}}\right| = \frac{2638.3*10^6}{6.05*10^7} + \frac{684.7*10^6}{4.4*10^7} = 59.2\ MPa \begin{array}{l} <82.7\ MPa\ OK\ for\ category\ C' \\ <110.3\ MPa\ OK\ for\ category\ B \end{array}$$

Flexural fatigue stress ranges at the top flange:

$$\gamma(\Delta f) = \left|\frac{+M}{S_{STt}}\right| + \left|\frac{-M}{S_{NCt}}\right| = \frac{2638.3*10^6}{2.63*10^8} + \frac{684.7*10^6}{3.7*10^7} = 28.5\ MPa \begin{array}{l} <68.9\ MPa\ OK\ for\ category\ C \\ <82.7\ MPa\ OK\ for\ category\ C' \\ <110.3\ MPa\ OK\ for\ category\ B \end{array}$$

Fatigue II – P-9 truck for finite life:

$$\gamma(\Delta f) = \left|\frac{+M}{S_{STb}}\right| + \left|\frac{-M}{S_{NCb}}\right| = \frac{4318.3*10^6}{6.05*10^7} + \frac{1556.2*10^6}{4.4*10^7} = 106.7\ MPa \begin{array}{l} <148.8\ MPa\ OK\ for\ category\ C' \\ <207.9\ MPa\ OK\ for\ category\ B \end{array}$$

$$\gamma(\Delta f) = \left|\frac{+M}{S_{STt}}\right| + \left|\frac{-M}{S_{NCt}}\right| = \frac{4318.3*10^6}{2.63*10^8} + \frac{1556.2*10^6}{3.7*10^7} = 58.5\ MPa \begin{array}{l} <148.8\ MPa\ OK\ for\ category\ C \\ <148.8\ MPa\ OK\ for\ category\ C' \\ <207.9\ MPa\ OK\ for\ category\ B \end{array}$$

3.4.3.5. Check requirements – Service Limit State

General requirements

Service Limit State II is to control the elastic and permanent deflections under the HL-93 design truck (AASHTO 6.10.4). According to AASHTO 2.5.2.6.2, the deflection Δ due to the live load may not exceed L/800

Illustrate calculations of factored moments – Service Limit State II

M_{DC1} = 2525.9 kN-m (applied to steel section alone)

$M_{DC2} + M_{DW}$ = 183.6 + 621.6 = 805.2 kN-m (applied to long-term composite section)

$M_{(LL+IM)HL\text{-}93}$ = 1.3(0.822)(5526.6) = 5905.7 kN-m (applied to short-term composite section)

Check flange stresses

For interior girder $f_l = 0$

$$f_f = \frac{M_{DC1}}{S_{NC}} + \frac{M_{DC2}+M_{DW}}{S_{LT}} + \frac{M_{(LL+IM)HL-93}}{S_{ST}} \leq 0.95R_hF_{yf} = 0.95(1.0)(345) = 327.75\ MPa$$

For the top flange:

$$f_f = \frac{2525.9*10^6}{3.7*10^7} + \frac{805.2*10^6}{1.02*10^8} + \frac{5905.7*10^6}{2.63*10^8} = 98.6\ MPa\ < 327.75\ MPa \text{ OK (AASHTO 6.10.4.2.2-1)}$$

For the bottom flange

$$f_f = \frac{2525.9*10^6}{4.4*10^7} + \frac{805.2*10^6}{5.51*10^7} + \frac{5905.7*10^6}{6.05*10^7} = 169.6\ MPa < 327.75\ MPa \text{ OK (AASHTO 6.10.4.2.2-2)}$$

For the compression flange

According to AASHTO 6.10.4.2.2, if for composite plate girder bridge AASHTO 6.10.2.1.1-1 is satisfied, i.e. $D/t_w \leq 150$, then AASHTO 6.10.4.2.2-4 is not required.

$$\frac{D}{t_w} = \frac{2000}{17} = 117.7 < 150 \text{ (AASHTO 6.10.2.1.1-1)}$$

3.4.3.6. Check requirements – Constructability (L_b = 8000mm)

M_u = 1.25(2547.7) = 3184.6 kN-m

$R_h = 1.0$, $R_b = 1.0$

Check compression flange

- Web compactness

$$\lambda_{rw} = 5.7\sqrt{\frac{E}{F_{yc}}} = 138.9$$

$$D_c = y_{NCt} - t_{fc} = 1096.6 \text{ mm}$$

$$\frac{2D_c}{t_w} = 129 < \lambda_{rw} = 138.9$$

Flexural resistance

- Local buckling resistance

$$\lambda_f = \frac{b_{fc}}{2t_{fc}} = 7.6 < \lambda_{pf} = 0.38\sqrt{\frac{E}{F_{yf}}} = 9.3$$

$$\Rightarrow F_{nc(FLB)} = R_h R_b F_{yc} = 345 \text{ MPa (AASHTO 6.10.8.2.2-1)}$$

- Lateral torsional buckling

$$r_t = \frac{b_{fc}}{25\sqrt{12(1+\frac{1}{3}\frac{D_c t_w}{b_{fc} t_{fc}})}} = 99.4\ mm \text{ (AASHTO 6.10.8.2.3-9)}$$

$$L_p = 1.0 r_t\sqrt{\frac{E}{F_{yc}}} = 2424\ mm$$

$$F_{yr} = \text{smaller}\begin{cases} 0.7F_{yc} = 241.5\ MPa \\ F_{yw} = 345\ MPa \end{cases} = 241.5 \text{ MPa} > 0.5F_{yc} = 172.5 \text{ MPa}$$

Use F_{yr} = 241.5 MPa

$$L_r = \pi r_t\sqrt{\frac{E}{F_{yr}}} = 9098\ mm \text{ (AASHTO 6.10.8.2.3-5)}$$

$$\because L_p = 2424 \text{ mm} < L_b = 8000 \text{ mm} < L_r = 9098 \text{ mm}$$

$$F_{nc(LTB)} = C_b(1-(1-\frac{F_{yr}}{R_h F_{yc}})(\frac{L_b - L_p}{L_r - L_p}))R_b R_h F_{yc} = 258.5 \text{ MPa} < R_h R_b F_{yc} = 345 \text{ MPa (AASHTO 6.10.8.2.3-2)}$$

$$\therefore F_{nc(LTB)} = 258.5 \text{ MPa}$$

$$f_{bu} = \frac{M_u}{S_{NCt}} = 86.1\ MPa < F_{nc(LTB)} = 258.5\ MPa \text{ OK (AASHTO 6.10.3.2.1-2)}$$

Calculate web bend – buckling resistance

$$k = 9(\frac{D}{D_c})^2 = 29.9 \text{ (AASHTO 6.10.1.9.1-2)}$$

$$F_{crw} = \frac{0.9Ek}{(\frac{D}{t_w})^2} = 398.6 \text{ MPa} > R_h F_{yc} = 1.0(345) = 345 \text{ MPa}$$

Use F_{crw} = 345 MPa (AASHTO 6.10.1.9.1-1)

f_{bu} = 86.1 MPa < $\phi_f F_{crw}$ = 345 MPa OK (AASHTO 6.10.3.2.1-3)

3.4.4. Design composite section in negative moment region at Bent 6

In negative moment region the girders are designed as non-composite section. According to AASHTO 6.10.10, when shear studs are provided in negative moment region the section is considered as composite section.

3.4.4.1. Calculations of factored moment and shears – Strength Limit State

Table 3.18 Factored moments at section Bent 6.

Load type	**Unfactored Moment (kN-m)**	**Factored moment (kN-m)**
DC1	-5196	M_{DC1} = 1.25(-5196) = -6495
DC2	-378	M_{DC2} = 1.25(-378) = -472.5
DW	-1279	M_{DW} = 1.5(-1279) = -1918.5
(LL+IM)HL-93	-4864	$M_{(LL+IM)HL\text{-}93}$ = 1.75*(0.822)*(-4864) = -6997
(LL+IM)P15	-10154	$M_{(LL+IM)P15}$ = 1.35(0.822)(-10154) = -11268
Controlling DC+DW+(LL+IM)P15		M_u = -6495-472.5-1918.5-11268 = -20154
Strength I: 1.25(DC)+1.5(DW)+1.75(DF)$(LL+IM)_{HL\text{-}93}$ Strength II: 1.25(DC)+1.5(DW)+1.35(DF)$(LL+IM)_{P15}$		

Table 3.19 Factored shears at section Bent 6.

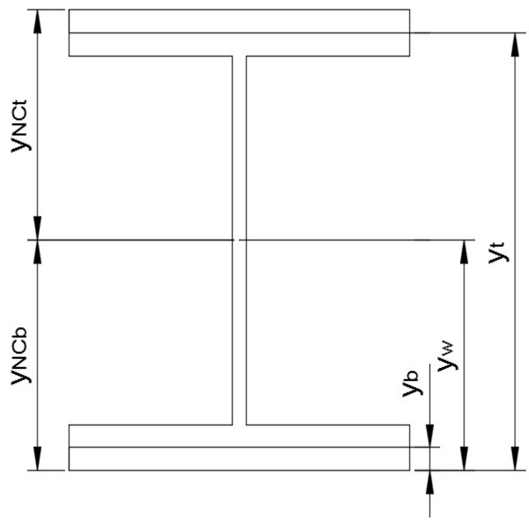
Load type	**Unfactored Moment (kN)**	**Factored moment (kN)**
DC1	-565.9	V_{DC1} = 1.25(-565.9) = -707.375
DC2	-41.1	V_{DC2} = 1.25(-41.1) = -51.375
DW	-139.3	V_{DW} = 1.5(-139.3) = -208.95
(LL+IM)HL-93	-704.7	$V_{(LL+IM)HL\text{-}93}$ = 1.75*(1.049)*(-704.7) = -1293.7
(LL+IM)P15	-1529.2	$V_{(LL+IM)P15}$ = 1.35(1.049)(-1529.2) = -2165.6
Controlling DC+DW+(LL+IM)P15		V_u = -707.375-51.375-208.95-2165.6= -3133.3

3.4.4.2. Elastic section properties

Elastic section properties are provided in order to calculate stresses, deflection, and camber in continuous beam bridge. Elastic section properties for steel section alone, the steel section and deck slab reinforcement, the short-term composite section, and the long-term composite section are calculated in tables 3.21-3.24.

Table 3.20 Properties of steel section alone.

Component	**b or D**	**t**	**A_i (mm^2)**	**y_i (mm)**	**A_iy_i (mm^3)**	**$y_i - y_{NCb}$ (mm)**	**$A_i(y_i - y_{NCb})^2$ (mm^4)**	**I_0 (mm^4)**
top flange 425x50	425	50	21250	2075	4.41E+07	1025	2.23E+10	4.43E+06
web 2000x17	2000	17	34000	1050	3.57E+07	0	0.00E+00	1.13E+10
bottom flange 425x50	425	50	21250	25	5.31E+05	-1025	2.23E+10	8.85E+04
∑			76500		8.03E+07		4.47E+10	1.13E+10



$$y_{NCb} = \frac{\sum A_i y_i}{\sum A_i} = \frac{8.03*10^7}{76500} = 1.05*10^3 \text{ mm}$$

$$y_{NCt} = 2100 - y_{NCb} = 1.05*10^3 \text{ mm}$$

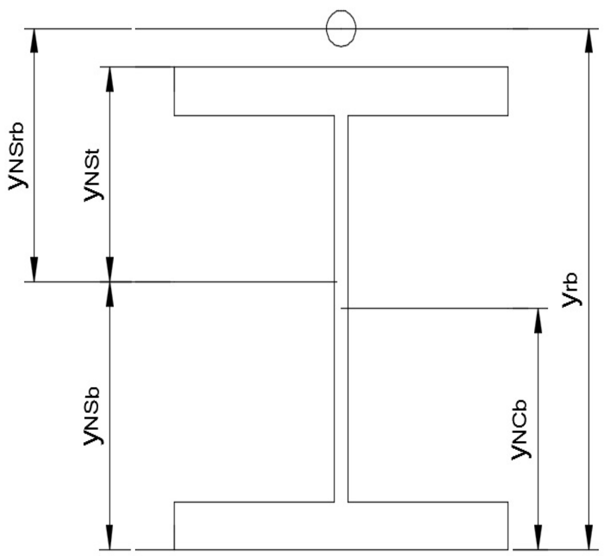
$$I_{NC} = \sum I_0 + \sum A_i(y_i - y_{NCb})^2 = 5.6 * 10^{10} \text{mm}^4$$

$$S_{NCb} = \frac{I_{NC}}{y_{NCb}} = 5.33*10^7 \text{ mm}^3$$

$$S_{NCt} = \frac{I_{NC}}{y_{NCt}} = 5.33*10^7 \text{ mm}^3$$

Table 3.21 Properties of steel section and deck slab reinforcement.

Component	**A_i (mm^2)**	**y_i (mm)**	**A_iy_i (mm^3)**	**y_i-y_{NSb} (mm)**	**$A_i(y_i$-$y_{NSb})^2$ (mm^4)**	**I_0 (mm^4)**
Top Reinforcement	7000	2250	1.58E+07	1099.40	8.46E+09	0
Steel Section	76500	1050	8.03E+07	-100.60	7.74E+08	5.60E+10
∑	83500		9.61E+07		9.23E+09	5.60E+10



$y_{NSb} = \frac{\sum A_i y_i}{\sum A_i} = \frac{9.61*10^7}{83500} = 1.151*10^3$ mm

$y_{NSt} = 2100 - y_{NSb} = 9.19*10^2$ mm

$y_{NSrb} = 2250 - y_{NSb} = 1.1*10^3$ mm

$I_{NS} = \sum I_0 + \sum A_i(y_i - y_{NSb})^2 = 6.52 * 10^{10}$ mm^4

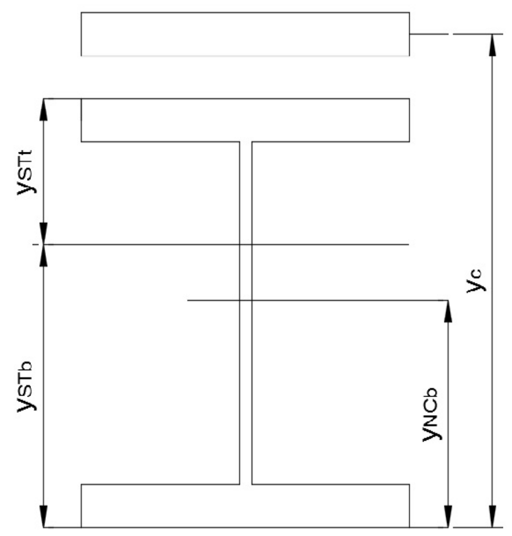
$S_{NSb} = \frac{I_{NS}}{y_{NSb}} = 5.67*10^7$ mm^3

$S_{NSt} = \frac{I_{NS}}{y_{NSt}} = 7.09*10^7$ mm^3

$S_{NSrb} = \frac{I_{NS}}{y_{NSrb}} = 5.93*10^7$ mm^3

Table 3.22 Properties of short-term composite section (n=8).

Component	**A_i (mm^2)**	**y_i (mm)**	**$A_i y_i$ (mm^3)**	**y_i-y_{STb} (mm)**	**$A_i(y_i-y_{STb})^2$ (mm^4)**	**I_0 (mm^4)**
Steel Section	76500	1050	8.03E+07	-640.24	3.14E+10	5.60E+10
Concrete Slab 3500/8x200	87500	2250	1.97E+08	559.76	2.74E+10	2.92E+08
$\sum$	164000		2.77E+08		5.88E+10	5.63E+10



$y_{STb} = \frac{\sum A_i y_i}{\sum A_i} = \frac{2.77*10^8}{164000} = 1.69*10^3$ mm

$y_{STt} = 2100 - y_{STb} = 3.8*10^2$ mm

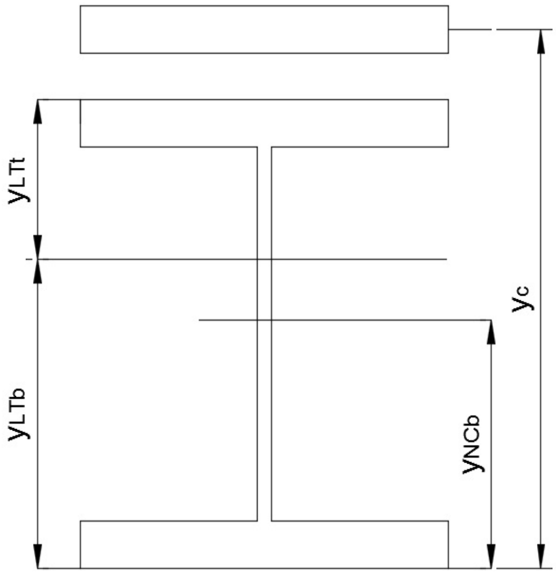
$I_{ST} = \sum I_0 + \sum A_i(y_i - y_{STb})^2 = 1.15 * 10^{11}$ mm^4

$S_{STb} = \frac{I_{ST}}{y_{STb}} = 6.81*10^7$ mm^3

$S_{STt} = \frac{I_{ST}}{y_{STt}} = 3.03*10^8$ mm^3

Table 3.23 Properties of long-term composite section (3n=24).

Component	**A_i (mm^2)**	**y_i (mm)**	**$A_i y_i$ (mm^3)**	**y_i-y_{LTb} (mm)**	**$A_i(y_i-y_{LTb})^2$ (mm^4)**	**I_0 (mm^4)**
Steel Section	76500	1050	8.03E+07	-331.23	8.39E+09	5.60E+10
Concrete Slab 3500/24x200	29167	2250	6.56E+07	868.77	2.20E+10	9.72E+07
$\sum$	105667		1.46E+08		3.04E+10	5.61E+10



$y_{LTb} = \frac{\sum A_i y_i}{\sum A_i} = \frac{1.46*10^8}{105667} = 1.38*10^3$ mm

$y_{STt} = 2100 - y_{NCb} = 6.89*10^2$ mm

$I_{LT} = \sum I_0 + \sum A_i(y_i - y_{LTb})^2 = 8.65 * 10^{10}$ mm^4

$S_{LTb} = \frac{I_{LT}}{y_{LTb}} = 6.26*10^7$ mm^3

$S_{LTt} = \frac{I_{LT}}{y_{LTt}} = 1.26*10^8$ mm^3

3.4.4.3. Design for flexure – Strength Limit State

General requirement

$$f_l = 0$$

$$M_u \leq \phi_f M_{nc}$$

$$M_u \leq \phi_f M_{nt}$$

Check section compactness

- Specified minimum yield strength of the flanges and web:

$$F_y \leq 500 \text{ MPa (AASHTO A6.1)}$$

- Web:

$$\frac{2D_c}{t_w} = 117.7 < \lambda_{rw} = 5.7\sqrt{\frac{E}{F_{yc}}} = 138.9 \text{ (AASHTO A6.1-1)}$$

- Flange ratio:

$$\frac{I_{yc}}{I_{yt}} = \frac{50*425^2}{50*425^2} = 1.0 > 0.3 \text{ (AASHTO A6.1-2)}$$

Flange strength reduction factor R_h and R_b

- R_h = 1.0 (AASHTO 6.10.1.10.1)
- R_b = 1.0 (AASHTO A6.10.1.10.2)

Flexural resistance based on compression flange

$$\because \lambda_f = \frac{b_{fc}}{2t_{fc}} = 4.25 < \lambda_{pf} = 0.38\sqrt{\frac{E}{F_{yf}}} = 9.3$$

$$M_{nc(FLB)} = R_{pc}M_{yc} \text{ (AASHTO A6.3.2-1)}$$

$$M_p = 2((425*50)(345)(1025)+(17*1000)(345)(500)) = 20894 \text{ kN-m}$$

$$M_{yc} = S_{NCt}F_{yc} = 18388.5 \text{ kN-m}$$

$$D_{cp} = D_c = 1000 \text{ mm (due to the symmetry)}$$

$$\lambda_{rw} = 5.7\sqrt{\frac{E}{F_{yc}}} = 138.9$$

$$\lambda_{pw(Dcp)} = \frac{\sqrt{\frac{E}{F_{yc}}}}{(0.54\frac{M_p}{R_h M_{yc}} - 0.09)} = 88.9 < \lambda_{rw}\left(\frac{D_{cp}}{D_p}\right) = 138.9$$

$$\because \frac{2D_{cp}}{t_w} = \frac{2000}{17} = 117.7 < \lambda_{pw(Dcp)} = 88.9 \text{ (AASHTO A6.2.1-1)}$$

∴ web is non-compact and web plastification factor is calculated as follows:

$$R_{pc} = \left(1 - \left(1 - \frac{R_h M_y}{M_p}\right)\left(\frac{\lambda_w - \lambda_{pw(Dc)}}{\lambda_{rw} - \lambda_{pw(Dc)}}\right)\right)\frac{M_p}{M_y} = 1.133 \text{ (AASHTO A6.2.2-1)}$$

$$M_{nc(FLB)} = R_{pc} M_{yc} = 20834.2 \text{ kN-m (AASHTO A6.3.2-1)}$$

Calculate lateral torsional buckling

L_b = 8000 mm

h = depth between centreline of flanges = 2050 mm

$$r_t = \frac{b_{fc}}{\sqrt{12\left(1 + \frac{1}{3}\frac{D_c t_w}{b_{fc} t_{fc}}\right)}} = 109\ mm$$

$$L_p = 1.0 r_t \sqrt{\frac{E}{F_{yc}}} = 2657\ mm$$

Ignoring rebar, $S_{xc} = S_{xt} = S_{NCb} = S_{NCt} = 5.33*10^7$ mm^3

$$F_{yr} = \text{smaller} \begin{cases} 0.7F_{yc} = 241.5\ MPa \\ R_h F_{yt} \frac{S_{xt}}{S_{xc}} = 345\ MPa \\ F_{yw} = 345\ MPa \end{cases} \Rightarrow F_{yr} = 241.5 \text{ MPa} > 0.5F_{yc} = 172.5 \text{ MPa}$$

Use F_{yr} = 241.5 MPa

$$J = \frac{Dt_w^3}{3} + \frac{b_{fc}t_{fc}^3}{3}\left(1 - 0.63\frac{t_{fc}}{b_{fc}}\right) + \frac{b_{ft}t_{ft}^3}{3}\left(1 - 0.63\frac{t_{ft}}{b_{ft}}\right) = 36.1 * 10^6 mm^4 \text{ (AASHTO A6.3.3-9)}$$

$$L_r = 1.95 r_t \frac{E}{F_{yr}} \sqrt{\frac{J}{S_{xc}h}} \sqrt{1 + \sqrt{1 + 6.67\left(\frac{F_{yr}S_{xc}h}{E*J}\right)^2}} = 10494.4\ mm \text{ (AASHTO A6.3.3-5)}$$

$$M_1 = -5084 + \left(\frac{2.4}{5.2}\right)(-9454+5084) = -7100 \text{ kN-m}$$

$$C_b = 1.75 - 1.05\left(\frac{M_1}{M_2}\right) + 0.3\left(\frac{M_1}{M_2}\right)^2 = 1.75 - 1.05\left(\frac{7100}{20156}\right) + 0.3\left(\frac{7100}{20156}\right)^2 = 1.42 < 2.3$$

(AASHTO A6.3.3-7)

$$\because L_p = 2657 \text{ mm} < L_b = 8000 \text{ mm} < L_r = 10494.4 \text{ mm}$$

$$M_{nc(LTB)} = C_b\left(1 - \left(1 - \frac{F_{yr}S_{xc}}{R_h M_{yc}}\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right)R_{pc}M_{yc} = 27687.8\ kN - m > R_{pc}M_{yc} = 20834.2\ kN - m \text{ (AASHTO A6.3.3-2)}$$

Use $M_{nc(LTB)}$ = 20834.2 kN-m

$$M_{nc} = \min\{M_{nc(FLB)}; M_{nc(LTB)}\} = 20834.2 \text{ kN-m}$$

Calculate flexural resistance – based on tension flange

$R_{pt} = R_{pc} = 1.133$, $M_{yt}=M_{yc}=20834.2$ kN-m (AASHTO A6.4-1) (due to the section symmetry)

Check design requirement

$$M_u = 20153 \text{ kN-m} < \phi_f M_{nc} = \phi_f M_{nt} = 20834.2 \text{ kN-m OK (AASHTO A6.1.1-1 \& A6.1.2-1)}$$

3.4.4.4. Design for shear – Strength Limit State

Select stiffener spacing

AASHTO C6.10.2.1.1 states that by limiting slenderness of transversely-stiffened webs to $D/t_w \leq 150$, the maximum transverse stiffener spacing up to 3D is permitted. For end panels adjacent to simple supports, stiffener spacing d_0 shall not exceed 1.5D (AASHTO 6.10.9.3.3) Try interior stiffener sapcing d_0 = 3000 mm < 3D = 6000 mm and end panel stiffener spacing d_0 = 2000 mm < 1.5D = 3000 mm.

Calculate shear resistance

Shear resistance for a stiffened interior web is as follows:

For d_0 = 3000 mm

$$k = 5 + \frac{5}{(\frac{2000}{2000})^2} = 10.0$$

$$\frac{D}{t_w} = \frac{2000}{17} = 117.7 > 1.4\sqrt{\frac{Ek}{F_{yw}}} = 108$$

$$\therefore C = \frac{1.57}{\left(\frac{D}{t_w}\right)^2}\left(\frac{Ek}{F_{yw}}\right) = 0.673$$

$$V_p = 0.58F_{yw}Dt_w = 6803.4\ kN$$

$$V_{cr} = CV_p = 4579\ kN$$

$$\frac{2Dt_w}{(b_{fc}t_{fc} + b_{ft}t_{ft})} = \frac{2*2000*17}{425*28+425*42} = 1.6 < 2.5$$

$$\therefore\ V_n = V_p\left(C + \frac{0.87(1-C)}{\sqrt{1+\left(\frac{d_0}{D}\right)^2}}\right) = 4997.6\ kN$$

Check design requirement

$$V_u = 3133.3\ kN < \phi_v V_n = 4997.6\ kN$$

Check transverse stiffener

- Projecting width

$$b_t = 155\, mm > 50 + \frac{D}{30} = 116.7\, mm$$

$$16t_p = 240\, mm > b_t = 155\, mm > \frac{b_f}{4} = 106.25\, mm$$

- Moment of inertia

$$I_{t1} = bt_w^3 J$$

$$I_{t2} = \frac{D^4 \rho_t^{1.3}}{40}\left(\frac{F_{yw}}{E}\right)^{1.5}$$

$$J = \frac{2.5}{\left(\frac{d_0}{D}\right)^2} - 2.0 \leq 0.5$$

ρ_t is the larger of F_{yw}/F_{crs} and 1.0

$$F_{crs} = \frac{0.31E}{\left(\frac{b_t}{t_p}\right)^2} \leq F_{ys}$$

F_{ys} is specified minimum yield strength of the stiffener

$$\because J = \frac{2.5}{\left(\frac{3000}{2000}\right)^2} - 2.0 = -0.89 \leq 0.5\ \because Use\, J = 0.5$$

b = smaller (d_0=3000mm and D=2000mm)=2000mm

$$\because F_{crs} = \frac{0.31 * 205000}{\left({}^{155}/_{15}\right)^2} = 595.2\, MPa > F_{ys} = 250\, MPa\ \Longrightarrow Use\, F_{crs} = 250\, MPa$$

$$\rho_t = larger\left(F_{yw}/F_{crs} = \frac{345}{250} = 1.38; 1.0\right) = 1.38$$

$$I_{t1} = 2000 * 17^3 * 0.5 = 4.913 * 10^6 mm^4$$

$$I_{t2} = 2000^4 * \frac{1.38^{1.3}}{40} \left(\frac{345}{205000}\right)^{1.5} = 41.98 * 10^6 mm^4$$

$$I_t = 2\left(\frac{155^3 * 15}{12} + 155 * 15\left(77.5 + \frac{17}{2}\right)^2\right) = 43.7 * 10^6 mm^4$$

$\therefore I_t = 43.7 * 10^6 mm^4 > I_{t2} = 41.98 * 10^6\ mm^4 > I_{t1} = 4.913 * 10^6 mm^4$ OK (AASHTO 6.10.11.1.3-1, 2)

3.4.4.5. Calculations of fatigue moments and shears

For bridge details, fatigue moment and shear ranges are shown below:

Fatigue I:

$+M = \gamma(DF_m)(LL+IM)_{HL} = (1.75)(0.464)(LL+IM)_{HL} = 0.812(LL+IM)_{HL}$

$-M = \gamma(DF_m)(LL+IM)_{HL} = (1.75)(0.446)(LL+IM)_{HL} = 0.7805(LL+IM)_{HL}$

$V = \gamma(DF_V)(LL+IM)_{HL} = (1.75)(0.683)(LL+IM)_{HL} = 1.19525(LL+IM)_{HL}$

+M = (0.812)(616.7) = 500.7604 kN-m

-M = (0.7805)(-2117.7) = -1652.865 kN-m

$\gamma(\Delta M)$ = 500.7604 + 1652.865 = 2153.63 kN-m

$\gamma(\Delta V)$ = 1.19525(416.9+61.4) = 571.7 kN

Fatigue II:

$+M = \gamma(DF_m)(LL+IM)_{P15} = (1.0)(0.464)(LL+IM)_{P15} = 0.464(LL+IM)_{P15}$

$-M = \gamma(DF_m)(LL+IM)_{P15} = (1.0)(0.446)(LL+IM)_{P15} = 0.446(LL+IM)_{P15}$

$V = \gamma(DF_V)(LL+IM)_{P15} = (1.0)(0.683)(LL+IM)_{P15} = 0.683(LL+IM)_{P15}$

+M = (0.464)(2453.1) = 1138.2 kN-m

-M = (0.446)(-9543) = -4256.2 kN-m

$\gamma(\Delta M)$ = 1138.2 + 4256.2 = 5394.4

$\gamma(\Delta V)$ = 0.683(204.3 + 1529.2) = 1184 kN

$V_u = V_{DC1} + V_{DC2} + V_{DW} + (1.75)(DF_v)(LL+IM)_{HL\text{-}93} = -1790.3$ kN

Check typical girder details and web – Fatigue Limit State

Fatigue I:

ADTT = 2500, N = (365)(75)n$(ADTT)_{SL}$ = (365)(75)(1.5)(0.8)(2500) = 0.82125*10^8 > N_{TH}

$(\Delta F_n) = (\Delta F_{TH})$ (AASHTO 6.6.1.2.5-1)

Fatigue II:

ADTT = 20, N = (365)(75)n$(ADTT)_{SL}$ = (365)(75)(1.2)(0.8)(20) = 525600 < N_{TH}

$(\Delta F_n) = \left(\frac{A}{N}\right)^{\frac{1}{3}}$ (AASHTO 6.6.1.2.5-2)

Table 3.24. Nominal Fatigue Resistance.

Detail Category	**Constant – A (x10^{11}) (MPa^3)**	**Fatigue I $(\Delta F_n) = (\Delta F_{TH})$ (MPa)**	**Fatigue II $(\Delta F_n) = \left(\frac{A}{N}\right)^{\frac{1}{3}}$ (MPa)**
B	39.3	110	195.5
C	14.4	69	140
C’	14.4	83	140
E	3.60	31	88

Fatigue I – HL-93 Truck for infinite life:

$$\gamma(\Delta f) = \frac{\gamma(\Delta M)}{S_{NC}} = \frac{2153.63 * 10^6}{5.33 * 10^7} = 40.4\ MPa \begin{array}{l} < 110\ MPa\ for\ category\ B\ (OK) \\ < 83\ for\ category\ C'(OK) \end{array}$$

Fatigue II – P9 truck for finite life:

$$\gamma(\Delta f) = \frac{\gamma(\Delta M)}{S_{NC}} = \frac{5394.4 * 10^6}{5.33 * 10^7} = 101.2\ MPa \begin{array}{l} < 195.5\ MPa\ for\ category\ B\ (OK) \\ < 140\ for\ category\ C'(OK) \end{array}$$

Fatigue I – HL-93 Truck for infinite life:

$$\gamma(\Delta f) = \frac{\gamma(\Delta M) C_{toe}}{I_{NC}} = \frac{2153.63 * 10^6 * 920}{5.6 * 10^{10}} = 35.4\ MPa < 83\ MPa\ for\ category\ C'$$

Fatigue II – P9 Truck for finite life:

$$\gamma(\Delta f) = \frac{\gamma(\Delta M) C_{toe}}{I_{NC}} = \frac{5394.4 * 10^6 * 920}{5.6 * 10^{10}} = 88.6\ MPa < 140\ MPa\ for\ category\ C'$$

Check special fatigue requirement for web:

This requirement is to ensure that significant elastic flexing of the web due t shear is not to occur and the member is able to sustain an infinite number of smaller loadings without fatigue cracking due to shear.

$V_{cr} = CV_p = 0.673 * 6803.4 = 4579\ kN > V_u = 1790.3\ kN$ OK(AASHTO 6.10.5.3-1)

3.4.4.6. Design flange-to-web welds

$$\tau_u = \frac{V_u Q}{I_{NC}} = \frac{3133.3 * 425 * 50 * 1025}{5.33 * 10^{10}} = 1.22 \frac{kN}{mm}$$

According to, AASHTO 6.13.3.4-1 → Use two fillet welds t_w = 10mm

Shear resistance of fillet welds (AASHTO 6.13.3.2.4b)

Use E70XX weld metal, F_{exx} = 482 MPa

$$R_r = 0.6\phi_{e2} F_{exx} = 0.6 * 0.8 * 482 = 231.4\ MPa$$

$$s_r = 2(0.707) t_w R_r = 2 * 0.707 * 10 * 231.4 = 3.27 \frac{kN}{mm} > s_u = 1.22 \frac{kN}{mm}$$

$$\therefore Use\ two\ flange - to - web\ welds\ t_w = 10\ mm$$

Shear resistance of the base metal of the web is

$R_r = 0.58\phi_v A_g F_y$ (AASHTO 6.13.5.3-1)

For web of t_w = 17mm, shear flow resistance is

$s_r = t_w (0.58)\phi_v F_y = 17 * 0.58 * 1.0 * 345 = 3.4 \frac{kN}{mm} < s_u = 1.22 \frac{kN}{mm}$ OK

3.4.4.7. Check requirements –Service Limit State

Moment at Service II

M_{DC1} = -5196.1 kN-m

M_{DC2} + M_{DW} = -377.6 + -1278.7 = -1656.3 kN-m

$M_{(LL+IM)HL\text{-}93}$ = 1.3*(0.822)*(-4864) = -5197.7 kN-m

Calculate web bend-buckling resistance

$$k = 9\left(\frac{D}{D_c}\right)^2 = 9\left(\frac{2000}{1000}\right)^2 = 36 \quad \text{(AASHTO 6.10.1.9.1-2)}$$

$$F_{crw} = \frac{0.9Ek}{\left(\frac{D}{t_w}\right)^2} = \frac{0.9*205000*36}{\left(\frac{2000}{17}\right)^2} = 479.9MPa > smaller\begin{Bmatrix} R_hF_{yc} = 1.0 * 345 = 345\ MPa \\ \frac{F_{yw}}{0.7} = \frac{345}{0.7} = 493MPa \end{Bmatrix} = 345MPa$$

(AASHTO 6.10.1.9.1-1)

Use F_{crw} = 345MPa

Check flange stress

- For both compression and tension flange

$$f_f = \frac{M_{DC1}+M_{DC2}+M_{DW}+M_{(LL+IM)HL}}{S_{NC}} = \frac{(5196.1+1656.3+5197.7)*10^6}{5.33*10^7} = 226.1MPa < 0.8R_hF_{yf} = 276MPa$$

OK (AASHTO 6.10.4.2.2-3)

- For compression flange

$$f_c = 226.1MPa < F_{crw} = 345MPa \quad \text{OK}$$

3.4.4.8. Check requirements – Constructability

Calculate factored moment and shear at Bent 2:

$$M_u = M_{DC1} = 1.25(-5196.1) = -6495.125kN - m$$

$$V_u = V_{DC1} = 1.25(-535.859) = -707.3kN$$

Check compression flange

- Check web compactness

$$\frac{2D_c}{t_w} = 117. < \lambda_{rw} = 5.7\sqrt{\frac{E}{F_{yc}}} = 138.9 \quad \text{(AASHTO 6.10.1.10.2-4)}$$

The web is non-compact and AASHTO equations 6.10.3.2.1-1 and 6.10.3.2.1-2 needs to be satisfied.

$R_h = 1.0;\ R_b = 1.0$

- Calculate flexural resistance

1) Local buckling resistance

$$\because\ \lambda_f = \frac{b_{fc}}{2t_{fc}} = \frac{425}{2*50} = 4.25 < \lambda_{pf} = 0.38\sqrt{\frac{E}{F_{yc}}} = 9.26$$

$$F_{nc(FLB)} = R_h R_b F_{yc} = 1.0*1.0*345 = 345MPa$$

2) Lateral torsional buckling

$r_t = \frac{b_{fc}}{\sqrt{12(1+\frac{1}{3}\frac{D_c t_w}{b_{fc}t_{fc}})}} = 109mm$ (AASHTO 6.10.8.2.3-9)

$L_p = 1.0r_t\sqrt{\frac{E}{F_{yc}}} = 1.0*109*\sqrt{\frac{205000}{345}} = 2657mm$ (AASHTO 6.10.8.2.3-4)

$F_{yr} = 241.5MPa$

$L_r = \pi r_t\sqrt{\frac{E}{F_{yr}}} = \pi*109\sqrt{\frac{205000}{241.5}} = 9977mm$ (AASHTO 6.10.8.2.3-5)

$\because L_p = 2657mm < L_b = 8000mm < L_r = 9977mm$

$F_{nc(LTB)} = C_b\left(1-\left(1-\frac{F_{yr}}{(R_hF_{yc})}\right)\left(\frac{L_b-L_p}{L_r-L_p}\right)\right)R_bR_hF_{yc} \le R_bR_hF_{yc}$ (AASHTO 6.10.8.2.3-2)

$F_{nc(LTB)} = 1.0\left(1-\left(1-\frac{241.5}{1.0*345}\right)\left(\frac{8000-2657}{9977-2657}\right)\right)1.0*1.0*345 = 269.5MPa \le R_bR_hF_{yc} = 345MPa$

Use $F_{nc(LTB)}$ = 269.5MPa

3) Nominal flexural resistance

$F_{nc} = \min\{345MPa; 269.5MPa\} = 269.5MPa$

$f_{bu} = \frac{M_u}{S_{NCb}} = \frac{6495.125*10^6}{5.33*10^7} = 121.9MPa < \phi_f F_{nc} = 269.5MPa$ (AASHTO 6.10.3.2.1-2)

- Web bend-buckling resistance

$k = 9\left(\frac{D}{D_c}\right)^2 = 36$ (AASHTO 6.10.1.9.1-2)

$F_{crw} = \frac{0.9Ek}{\left(\frac{D}{t_w}\right)^2} = 479.9MPa > smaller\begin{Bmatrix} R_h F_{yc} = 1.0 * 345 = 345\ MPa \\ \frac{F_{yw}}{0.7} = \frac{345}{0.7} = 493MPa \end{Bmatrix}$ (AASHTO 6.10.1.9.1-1)

Use $F_{crw} = 345MPa$

$f_{bu} = 121.9MPa < \phi_f F_{crw} = 345MPa$ OK (AASHTO 6.10.3.2.1-3)

Check tension flange

$f_{bu} = \frac{M_u}{S_{NCt}} = 121.9MPa < \phi_f F_{yt} = 345MPa$ OK (AASHTO 6.10.3.2.2-1)

Check for shear

$C = 0.673, V_p = 0.58F_{yw}Dt_w = 6803.4kN$

$V_{cr} = CV_p = 4578.7kN$

$V_u = 707.3kN < \phi_v F_{cr} = 4578.4kN$ OK (AASHTO 6.10.3.3-1)

3.4.4.9. Design shear connectors

The shear connectors are provided in the positive moment regions and usually designed for fatigue and checked for strength

Design for fatigue

The range of horizontal shear flow, V_s, is as follows:

$V_{sr} = \frac{V_f Q}{I_{ST}}$

V_f – factored fatigue vertical shear force range

I_{ST} – moment of inertia of the transformed short-term composite section

Q – first moment of area of transformed short-term area of the concrete deck about the neutral axis of the short-term composite section

I_{ST} = 1.02*$10^{11} mm^4$

$Q = \frac{A_c}{n}(y_c - y_{STb}) = 87500(2222 - 1682.8) = 47 * 10^6 mm^3$

$V_{sr} = \frac{V_f Q}{I_{ST}} = \frac{47*10^6}{1.02*10^{11}} V_f = 0.461 * 10^{-3} V_f$

Try d=22mm diameter, 3 per row

Fatigue I:

ADTT = 2500 ⇒ $(ADTT)_{SL}$ = p*ADTT = 0.8ADTT

$N = (365)(75)n(ADTT)_{SL} = 365 * 75 * 1.0 * 0.8 * 2500 = 5.475 * 10^7 > 5.97 * 10^6$

$Z_r = 38d^2 = 38 * 22^2 = 18.4kN$ (AASHTO 6.10.10.2-1)

Fatigue II:

ADTT=20

$N = (365)(75)n(ADTT)_{SL} = 365 * 75 * 1.0 * 0.8 * 20 = 438000 < 5.97 * 10^6$

$Z_r = \alpha d^2$ (AASHTO 6.10.10.2-3)

$\alpha = 34.5 - 4.28 \log N = 34.5 - 4.28 \log 438000 = 10.35 ksi = 71.4 MPa$ (AASHTO 6.10.10.2-2)

$Z_r = 71.4d^2 = 71.4 * 22^2 = 34.6kN$

Required pitch of shear connectors, p is obtained as:

$p = (nZ_r)/V_{sr}$ (AASHTO 6.10.10.1.2-1)

Table 3.25. Pitch of shear studs (Span 5).

L	Fatigue I -HL-93 Truck for infinite life			Fatigue II-P9 Truck for finite life		
	V_f (kN)	**V_{sr}=0,461*$10^{-3}V_f$**	**p (mm)**	**V_f**	**V_{sr}=0,461*$10^{-3}V_f$**	**p (mm)**
0	463,60	0,21	258,28	829,33	0,38	271,50
0,50	462,94	0,21	258,65	813,64	0,38	276,74
1,00	462,28	0,21	259,02	797,95	0,37	282,18
1,50	461,62	0,21	259,39	782,26	0,36	287,84
2,00	460,96	0,21	259,76	766,56	0,35	293,73
2,49	460,30	0,21	260,13	750,87	0,35	299,87

2,99	459,64	0,21	260,51	735,18	0,34	306,27
2,99	459,64	0,21	260,51	735,18	0,34	306,27
3,49	459,09	0,21	260,82	720,35	0,33	312,57
3,99	458,55	0,21	261,13	705,53	0,33	319,14
4,49	458,00	0,21	261,44	690,70	0,32	325,99
4,99	457,45	0,21	261,75	675,88	0,31	333,14
5,49	456,90	0,21	262,07	661,05	0,30	340,61
5,99	456,36	0,21	262,38	646,23	0,30	348,43
5,99	456,36	0,21	262,38	646,23	0,30	348,43
6,49	455,95	0,21	262,62	637,61	0,29	353,14
6,98	455,54	0,21	262,85	628,99	0,29	357,98
7,48	455,13	0,21	263,09	620,37	0,29	362,95
7,98	454,72	0,21	263,33	611,75	0,28	368,07
8,48	454,31	0,21	263,56	603,12	0,28	373,33
8,98	453,90	0,21	263,80	594,50	0,27	378,74
8,98	453,90	0,21	263,80	594,50	0,27	378,74
9,48	453,65	0,21	263,95	590,49	0,27	381,32
9,98	453,40	0,21	264,09	586,47	0,27	383,93
10,48	453,15	0,21	264,24	582,45	0,27	386,58
10,98	452,90	0,21	264,39	578,43	0,27	389,26
11,47	452,65	0,21	264,53	574,41	0,26	391,99
11,97	452,40	0,21	264,68	570,39	0,26	394,75
11,97	452,40	0,21	264,68	570,39	0,26	394,75
12,47	452,31	0,21	264,73	569,78	0,26	395,18
12,97	452,23	0,21	264,78	569,16	0,26	395,61
13,47	452,14	0,21	264,83	568,54	0,26	396,03
13,97	452,06	0,21	264,88	567,93	0,26	396,46
14,47	451,97	0,21	264,93	567,31	0,26	396,90
14,97	451,89	0,21	264,98	566,69	0,26	397,33
14,97	451,89	0,21	264,98	566,69	0,26	397,33

15,46	451,97	0,21	264,93	567,24	0,26	396,94
15,96	452,05	0,21	264,88	567,79	0,26	396,56
16,46	452,13	0,21	264,83	568,34	0,26	396,18
16,96	452,21	0,21	264,79	568,89	0,26	395,79
17,46	452,29	0,21	264,74	569,44	0,26	395,41
17,96	452,37	0,21	264,69	569,98	0,26	395,03
17,96	452,37	0,21	264,69	569,98	0,26	395,03
18,46	452,62	0,21	264,55	573,92	0,26	392,32
18,96	452,86	0,21	264,41	577,86	0,27	389,65
19,46	453,11	0,21	264,26	581,80	0,27	387,01
19,95	453,36	0,21	264,12	585,73	0,27	384,41
20,45	453,60	0,21	263,97	589,67	0,27	381,84
20,95	453,85	0,21	263,83	593,61	0,27	379,31
20,95	453,85	0,21	263,83	593,61	0,27	379,31
21,45	454,25	0,21	263,60	601,94	0,28	374,06
21,95	454,66	0,21	263,36	610,26	0,28	368,96
22,45	455,06	0,21	263,13	618,59	0,29	363,99
22,95	455,47	0,21	262,89	626,92	0,29	359,16
23,45	455,87	0,21	262,66	635,25	0,29	354,45
23,94	456,28	0,21	262,43	643,57	0,30	349,86
23,94	456,28	0,21	262,43	643,57	0,30	349,86
24,44	456,82	0,21	262,11	658,39	0,30	341,99
24,94	457,36	0,21	261,80	673,20	0,31	334,47
25,44	457,91	0,21	261,49	688,01	0,32	327,27
25,94	458,45	0,21	261,18	702,83	0,32	320,37
26,44	458,99	0,21	260,88	717,64	0,33	313,76
26,94	459,53	0,21	260,57	732,45	0,34	307,41
26,94	459,53	0,21	260,57	732,45	0,34	307,41
27,44	460,19	0,21	260,20	748,12	0,34	300,97
27,94	460,85	0,21	259,83	763,78	0,35	294,80

28,43	461,50	0,21	259,46	779,45	0,36	288,87
28,93	462,16	0,21	259,09	795,11	0,37	283,18
29,43	462,81	0,21	258,72	810,78	0,37	277,71
29,93	463,47	0,21	258,36	826,45	0,38	272,45

Select 3-22 mm diameter shear studs with F_u=420MPa (AASHTO 6.4.4) at spacing 250mm for the positive moment regions, and 500mm for the negative moment regions. Total number of shear connectors provided for girder in positive moment region is n=3*(119+1) =360.

Check for strength

The number of shear studs required between point of maximum flexural load and each point of contra flexure shall satisfy:

$$n = \frac{P}{Q_r} \qquad \text{(AASHTO 6.10.10.4.1-2)}$$

$$P = smaller \begin{Bmatrix} 0.85 f_c' b t_s = 0.85 * 25 * 3500 * 200 = 14875kN \\ A_s F_y = 63750 * 345 = 21993.75kN \end{Bmatrix} = 14875kN \quad \text{(AASHTO 6.10.10.4.2-2,3)}$$

The factored shear resistance of a single d=22mm shear stud is:

$$E_c = 33 w^{\frac{3}{2}} \sqrt{f_c'} = 3457ksi = 23835MPa \qquad \text{(AASHTO 5.4.2.4-1)}$$

$$Q_n = 0.5 A_{sc} \sqrt{f_c' E_c} = \frac{0.5\left((22^2)\pi\right)}{4} \sqrt{25 * 23835} = 146.7kN < A_{sc} F_u = 157.4kN \quad \text{(AASHTO 6.10.10.4.3-1)}$$

Use Q_n=146.7kN

$$n = \frac{360}{2} = 180 > \frac{P}{\phi_{sc} Q_n} = \frac{14875}{0.85 * 146.7} = 119.3 \qquad \text{OK}$$

Determine shear connectors at points of contra flexure

According to, AASHTO 6.10.10.3, for members which are non-composite for negative moment regions in the final condition, additional connectors shall be placed within a distance to one-third of the effective concrete deck width on each side of the point of dead load contra flexure.

$$n_{ac} = \frac{A_s f_{sr}}{Z_r} \qquad \text{(AASHTO 6.10.10.3-1)}$$

$$f_{sr} = \frac{\gamma(\Delta M)}{S_{NSrb}} = \frac{2153.63 * 10^6}{4.51 * 10^7} = 47.8MPa$$

$n_{ac} = \frac{7000*47.8}{18.4*10^3} = 18.2\, studs\ \Rightarrow provide\ 20\, studs$

3.4.5. Design bearing stiffeners at Bent 6

The bearing stiffeners consist of one or more plates welded to each side of the web and extend the full height of the web. The purpose of bearing stiffeners is to transmit the full bearing forces from factored loads. The bearing stiffeners shall be designed for axial resistance of a concentrically loaded column (AASHTO 6.10.11.2.4) and for bearing resistance (AASHTO 6.10.11.2.3).

3.4.5.1. Calculations of factored support forces at bent 6

Dead Load

R_{DC1} = 1.25(1130.514) = 1413.1 kN

R_{DC2} = 1.25(82.162) = 102.7 kN

R_{DW} = 1.5(278.208) = 417.3 kN

Live load

$R_{(LL+IM)HL\text{-}93}$ = 1.75*DF_v(LL+IM)HL-93 = 1.75*1.049(1027.93) = 1887 kN

$R_{(LL+IM)P15}$ = 1.35*DF_v(LL+IM)P15 = 1.35*1.049(2082.9) = 2949.7 kN

Controlling support force

R_u = 1413.1 + 102.7 + 417.3 + 2949.7 = 4882.8 kN

Select stiffeners

Assume $P_r = 0.85F_{ys}A_s = 0.85 * 25A_s = 212.5A_s$

$$A_s = \frac{R_u}{212.5} = \frac{4882.7 * 10^3}{212.5} = 22977.41mm^2$$

Try a pair of stiffeners, b_t*t_p=204mm*50mm

Check projecting width

$b_t = 204mm < 0.48t_p\sqrt{\frac{E}{F_{ys}}} = 687.3mm$ OK (AASHTO 6.10.11.2.2-1)

Check bearing resistance

Factored bearing resistance

$(R_{sb})_r = \phi_b (R_{sb})_n = (1.0)(1.4) A_{pn} F_{ys}$ (AASHTO 6.10.11.2.3-1 and 6.10.11.2.3-2)

Assume cope on bearing stiffener = 40mm

$A_{pn} = 2(204 - 40)(50) = 16400 mm^2$

$(R_{sb})_r = \phi_b (R_{sb})_n = 1.0 * 1.4 * 16400 * 250 = 5740 kN > R_u = 4882.8 kN$ OK

Check axial resistance

Stiffener area: A_{st} = 2*204*50 = 20400mm^2

Web area: $A_{web} = (18t_w + t_p)t_w = (18 * 17 + 50) * 17 = 6052 mm^2$

Total effective area: $A_s = A_{st} + A_{web} = 26452 mm^2$

$I_{x-x} = \frac{18*17*17^3 + 50(2*200+17)^3}{12} = 302.26 * 10^6 mm^4$

$r_s = \sqrt{\frac{I_{x-x}}{A_s}} = \sqrt{\frac{302.26*10^6}{26452}} = 106.89 mm$

K=0.75-effective length factor for the weld connection (AASHTO 6.10.11.2.4a)

l=D=2000mm – unbraced length for the bearing stiffener

$\frac{Kl}{r_s} = \frac{0.75*2000}{106.89} = 14.03 < 120$ OK (AASHTO 6.9.3)

$P_e = \frac{\pi^2 E}{\left(\frac{Kl}{r_s}\right)^2} A_s = \frac{\pi^2 * 205000}{(14.03)^2} * 26452 = 271892 kN$ (AASHTO 6.9.4.1.2-1)

Q=1.0 – according to AASHTO 6.9.4.1.1

$P_0 = Q F_{ys} A_s = 1.0 * 250 * 26452 = 6613 kN$

$\because \frac{P_e}{P_0} = \frac{271892}{6613} = 41.1 > 0.44$

$\because P_n = \left(0.658^{\frac{P_0}{P_e}}\right) P_0 = \left(0.658^{\frac{6613}{271892}}\right) * 6613 = 6546 kN$ (AASHTO 6.9.4.1.1-1)

$P_r = \phi_c P_n = 0.9 * 6546 = 5891.4 kN > R_u = 4882.8 kN$ OK

$\therefore$ Use two 50mm*204mm PL bearing stiffeners

3.4.5.2. Design bearing stiffener-to-web welds

Fillet weld are usually used for bearing stiffener-to-web connections. According to AASHTO Table 6.13.3.4-1, the minimum size of fillet weld for thicker plate thickness joined larger than 20mm is 8mm, but need not exceed the thickness of the thinner part joined. Try two fillet welds t_{weld} = 10mm on each stiffener.

$$R_r = 0.6\phi_{e2}F_{exx} \qquad \text{(AASHTO 6.13.3.2.4b-1)}$$

Weld metal is E70XX, F_{exx} = 500MPa

$$R_r = 0.6 * 0.8 * 500 = 240MPa$$

Total length of weld, allowing 50mm for clips at both the top and bottom of the stiffener, is:

L = 2000 – 2*50 = 1900mm

Total shear resistance of welds connecting the bearing stiffeners to the web:

$$V_r = 4(0.707)t_wLR_r = 4(0.707)(10)(1900)(231.84) = 12457kN > R_u = 4883kN \qquad \text{OK}$$

∴ Use two fillet welds t_w = 10mm on each stiffener.

3.4.6. Design intermediate cross frames

3.4.6.1. Calculate wind load

Design wind pressure is determined as:

$$P_D = P_B*(\frac{V_{DZ}}{V_B})^2 = P_B*\frac{V_{DZ}^2}{10{,}000} \qquad \text{(AASHTO 3.8.1.2.1-1)}$$

$$V_{DZ} = 2.5V_0(\frac{V_{30}}{V_B})\ln(\frac{Z}{Z_0}) \qquad \text{(AASHTO 3.8.1.1-1)}$$

Z = 30 ft., Z_0 = 8.2 ft., V_0 = 12 mph, $V_{30} = V_B$ = 100 mph

$$V_{DZ} = 2.5V_0(\frac{V_{30}}{V_B})\ln(\frac{Z}{Z_0}) = 2.5*12*(\frac{100}{100})\ln\left(\frac{33.45}{8.2}\right) = 42.2\ mph = 18.9 \text{ m/s}$$

$$P_D = P_B*(\frac{V_{DZ}}{V_B})^2 = P_B*\frac{V_{DZ}^2}{10{,}000} = 0.05*\frac{42.2^2}{10000} = 0.0089 \text{ ksf} = 0.43 \text{ kN/m}^2$$

d = 6.9 ft, depth of deck and barrier = 3.5 + 0.83 = 4.33 ft

$WS_{girder} = P_D(6.9+4.33)$ = 0.0089*11.23 = 1.46 kN/m = 0.1 kip/ft < 0.3 kip/ft

Use WS_{girder} = 0.3 kip/ft = 4.46 kN/m (AASHTO 3.8.1.2.1)

$WS_{bf} = \frac{WS_{girder}(\frac{d}{2})}{11.23} = \frac{0.3*(\frac{6.9}{2})}{11.23} = 1.362\frac{kN}{m} = 0.0922\ kip/ft$

WS_{tf} = 4.46 – 1.362 = 3.098 kN/m

Check flexural resistance of bottom flange

L_b = 8000mm

$M_{WS} = \frac{WS_{bf}L_b^2}{10} = \frac{1.362*8^2}{10} = 8.72kN - m$ (AASHTO C4.6.2.7.1-2)

$f_{l-ws} = \frac{M_{WS}}{\frac{t_f b_f^2}{6}} = \frac{8.72*10^6}{\frac{42*425^2}{6}} = 6.89MPa$

Strength III: 1.25DC+1.5DW+1.4WS

Strength IV: 1.25DC+1.5DW+1.35DF$(LL+IM)_{HL\text{-}93}$+0.4WS

Span 1, L=16.4m

M_{DC1}=3157.46kN-m, M_{DC2}=229.5kN-m, M_{DW}=932.4kN-m, $M_{(LL+IM)HL\text{-}93}$=5740kN-m

M_u=10059.36kN-m

The factored lateral bending stress in the bottom flange due to wind load is:

$f_l = 0.4f_{l-WS} = 0.4*6.89 = 2.756MPa < 0.6F_{yt} = 207MPa$ OK (AASHTO 6.10.1.6-1)

$M_u + \frac{1}{3}f_l S_{xt} \le \phi_f M_n$ (AASHTO 6.10.7.1.1-1)

$S_{xt} = \frac{M_{yt}}{F_{yt}} = \frac{19754*10^6}{345} = 57.26*10^6 mm^3$

$M_u + \frac{1}{3}f_l S_{xt} = 10059.36 + \frac{1}{3}*2.756*57.26*10^6 = 10111.96kN - m \le \phi_f M_n = 25680.2kN - m$

OK

Calculate forces acting on the cross frame

At strength limit state III, factored wind force acting on the bottom strut is:

$P_u = 1.4WS_{bf}L_b = 1.4 * 1.362 * 8 = 15.25kN$

Factored force acting on diagonals is:

$$P_u = \frac{15.25}{\cos\phi} = \frac{15.25}{\frac{1.75}{\sqrt{1.6^2+1.75^2}}} = 20.7kN$$

3.4.6.2. Design bottom strut

Select section

Try L150x150x12

$A_g = 3480mm^2$

$r_x=r_y=46mm$

$I_x = I_y = 747*10^4 mm^4$

$r_z = 29.5mm$

K=1.0 (AASHTO 4.6.2.5)

$L_z = 1750mm$

$\frac{KL_z}{r_z} = \frac{1.0*1750}{29.5} = 59.3 < 140$ OK

$\frac{KL_y}{r_y} = \frac{1.0*3500}{46} = 76 < 140$ OK

Check member strength

1) End connections are to a single leg of the angle, and are welded or use minimum two-bolt connections.
2) Angles are loaded at the ends in compression through the same leg.
3) There are no intermediate transverse loads.

Determine effective slenderness ratio

For equal-leg angles that are individual members, $\frac{L}{r_x} = \frac{3500}{46} = 76 < 80$

$\left(\frac{KL}{r}\right)_{eff} = 72 + \frac{0.75L}{r_x} = 72 + 0.75 * 76 = 129$ (AASHTO 6.9.4.4-1)

Determine slender element reduction factor, Q

$$\because \frac{b}{t} = \frac{150}{12} = 12.5 < k\sqrt{\frac{E}{F_y}} = 0.45\sqrt{\frac{205000}{250}} = 12.9 \qquad \text{(AASHTO 6.9.4.2.1-1)}$$

$$\therefore Q = 1.0$$

Determine nominal axial compression strength

$$P_e = \frac{\pi^2 E}{\left(\frac{KL}{r_s}\right)^2} A_g = \frac{\pi^2 * 205000}{(129)^2} * 3480 = 423.1kN \qquad \text{(AASHTO 6.9.4.1.2-1)}$$

$$P_0 = QF_{ys}A_s = 1.0 * 250 * 3480 = 870kN$$

$$\because \frac{P_e}{P_0} = \frac{423.1}{870} = 0.486 > 0.44$$

$$\because P_n = \left(0.658^{\frac{P_0}{P_e}}\right) P_0 = \left(0.658^{\frac{870}{423.1}}\right) * 870 = 367.9kN \qquad \text{(AASHTO 6.9.4.1-1)}$$

Check compressive strength

$$P_u = 15.25kN < \phi_c P_n = 0.9 * 367.9 = 331kN$$

OK

3.4.6.3. Design diagonal

Select section

Try L100x100x8

A_g = 1550mm^2

$$r_{min} = 19.6mm$$

$$L = \sqrt{1600^2 + 1750^2} = 2371mm$$

Check limiting effective slenderness ratio

$$\left(\frac{KL}{r_z}\right) = 1.0 * \frac{2371}{19.6} = 121 < 140 \qquad \text{OK (AASHTO 6.9.3)}$$

Determine slender element reduction factor, Q

$$\because \frac{b}{t} = \frac{100}{8} = 12.5 < k\sqrt{\frac{E}{F_y}} = 0.45\sqrt{\frac{205000}{250}} = 12.9 \quad \text{(AASHTO 6.9.4.2.1-1)}$$

$$\therefore Q = 1.0$$

Determine nominal axial compression strength

$$P_e = \frac{\pi^2 E}{\left(\frac{KL}{r_s}\right)^2} A_g = \frac{\pi^2 * 205000}{(121)^2} * 1550 = 214.2kN \quad \text{(AASHTO 6.9.4.1.2-1)}$$

$$P_0 = QF_{ys}A_s = 1.0 * 250 * 1550 = 387.5kN$$

$$\because \frac{P_e}{P_0} = \frac{214.2}{387.5} = 0.55 > 0.44$$

$$\because P_n = \left(0.658^{\frac{P_0}{P_e}}\right) P_0 = \left(0.658^{\frac{387.5}{214.2}}\right) * 387.5 = 181.7kN \quad \text{(AASHTO 6.9.4.1-1)}$$

Check compressive strength

$$P_u = 20.7kN < \phi_c P_n = 0.9 * 181.7 = 163.5kN$$

OK

3.4.6.4. Design top strut

Since the force in the top strut is assumed zero, we select an angle L150x150x12 to provide lateral stability to the top flange during construction and to design for 2% of the flange yielding strength.

$A_g = 3480mm^2$

$r_x = r_y = 46mm$

$I_x = I_y = 747*10^4 mm^4$

$r_z = 29.5mm$

K=1.0 (AASHTO 4.6.2.5)

$L_z = 1750mm$

$$\frac{KL_z}{r_z} = \frac{1.0*1750}{29.5} = 59.3 < 140 \quad \text{OK}$$

$$\frac{KL_y}{r_y} = \frac{1.0*3500}{46} = 76 < 140 \quad \text{OK}$$

Determine effective slenderness ratio

For equal-leg angles that are individual members, $\frac{L}{r_x} = \frac{3500}{46} = 76 < 80$

$$\left(\frac{KL}{r}\right)_{eff} = 72 + \frac{0.75L}{r_x} = 72 + 0.75 * 76 = 129 \qquad \text{(AASHTO 6.9.4.4-1)}$$

Determine slender element reduction factor, Q

$$\because \frac{b}{t} = \frac{150}{12} = 12.5 < k\sqrt{\frac{E}{F_y}} = 0.45\sqrt{\frac{205000}{250}} = 12.9 \qquad \text{(AASHTO 6.9.4.2.1-1)}$$

$$\therefore Q = 1.0$$

Determine nominal axial compression strength

$$P_e = \frac{\pi^2 E}{\left(\frac{KL}{r_s}\right)^2} A_g = \frac{\pi^2 * 205000}{(129)^2} * 3480 = 423.1kN \qquad \text{(AASHTO 6.9.4.1.2-1)}$$

$$P_0 = QF_{ys}A_s = 1.0 * 250 * 3480 = 870kN$$

$$\because \frac{P_e}{P_0} = \frac{423.1}{870} = 0.486 > 0.44$$

$$\because P_n = \left(0.658^{\frac{P_0}{P_e}}\right) P_0 = \left(0.658^{\frac{870}{423.1}}\right) * 870 = 367.9kN \qquad \text{(AASHTO 6.9.4.1-1)}$$

3.4.6.5. Design end connections

Determine design load

AASHTO 6.13.1 requires that cross frame shall be designed for the calculated member forces. Thus, the connection of bottom strut is designed for P_u=15.25kN, and connection of diagonals are designed for P_u=20.7kN

Determine number of bolts

- Select bolts

Try A325 high-strength 16mm diameter bolts with threads excluded from shear plane, with bolt spacing = 48mm, and edge distance = 31.25mm. (AASHTO 6.13.2.6.1 and 6.13.2.6.6-1)

- Determine nominal resistance per bolt

$R_n = 0.48A_bF_{ub}N_s$ (AASHTO 6.13.2.7-1)

$A_b = 8^2\pi = 201mm^2$

$F_{ub} = 800MPa$

$N_s = 1$ (For single shear)

$R_n = 0.48 * 201 * 800 * 1 = 77.2kN$ (AASHTO 6.13.2.7-1)

Calculate the design bearing strength for each bolt on stiffener material.

$L_c = 31.25 - \frac{16}{2} = 23.25mm < 2d = 32$

Stiffener material A709 Grade 36, F_u = 400MPa

$R_n = 1.2L_ctF_u = 1.2 * 23.25 * 10 * 400 = 111.6kN$ (AASHTO 6.13.2.9-2)

It is clear that shear controls and nominal shear resistance per bolt is 77.2kN.

- Determine number of bolts required

$N = \frac{P_u}{\phi_s R_v} = \frac{15.25}{0.8*77.2} = 0.25$

∴ Use 2 bolts (AASHTO 6.9.4.4)

4. Geotechnical Design

4.1. Design of the foundation

Foundation is an essential supporting part of the structure in any construction process which derives its capacity to withstand the weight of the superstructure and loads coming from it from the ground strata. It is of major importance to define criteria for the geotechnical design of the foundation. In general, they must ensure safety against structural failure. So, there are two main considerations in foundation design. First, bearing capacity of soil under the foundation so that it can resist vertical and lateral loads, and second, meeting allowable settlement caused by working loads of the superstructure.

Hence, in order to maintain stability for the loads, it is essential to distinguish between types of foundations. In general, they are classified as deep foundations and shallow foundations. Shallow foundations support the bridge that depend on base of the footings, where deep foundations withstand the superstructure load by deriving their support capacity from 2 parts, which are base and skin friction, or either one or both. Deep foundations in comparison to shallow foundations occupy relatively smaller ground surface area, and mainly, the choice of deep foundations over shallow foundations is in their capacity to reach deeper soil layers, and hence withstand larger loads. Taking into account the soil profile of the site, which is water table above the ground surface, liquefaction and scouring potential, and all the loads from the superstructure, it is necessary to transmit the superstructure loads into the deeper layers of the ground, such as competent soil or bedrock. Thus, deep foundations are chosen for the design.

With the existence of the river passing through the location for the construction, it is required to place a sheet pile for the construction process. For the construction of 500 m span bridge the soil profile differs along the bridge length. Mainly as, the water table is above the ground surface and water table is below the ground surface since the width of the river is not directly proportional to the bridge length. For the design method calculations which are listed in Section 3.5, the point BH-12 is chosen, as constructing at river is a bit more challenging than constructing at dry surface.

4.2. Foundation selection

In general, deep foundations mainly selected for the design are classified as driven piles and drilled shafts. They can be grouped within a cap footing, or act as a single slender structure. There are several factors that affect the choice of pile types, such as noise, vibration, settlement, drivability, constructability, as well as cost-efficiency.

Firstly, in a design procedure it should be noted that choosing the right foundation type should be based on the steps:

- Site evaluation and soil profile information based on field and laboratory tests;
- Evaluation of construction procedure conditions;
- Selection of appropriate type of deep foundations;
- Identify design elements for the foundation type chosen;
- Perform design calculations;
- Check with code requirements;

4.3. Site description

The main task of this project is to build a bridge in Jeonju, South Korea, on proposed location illustrated in Figure 3.3.1. General soil profile of this area can be found in Appendix – A, Table A-1.

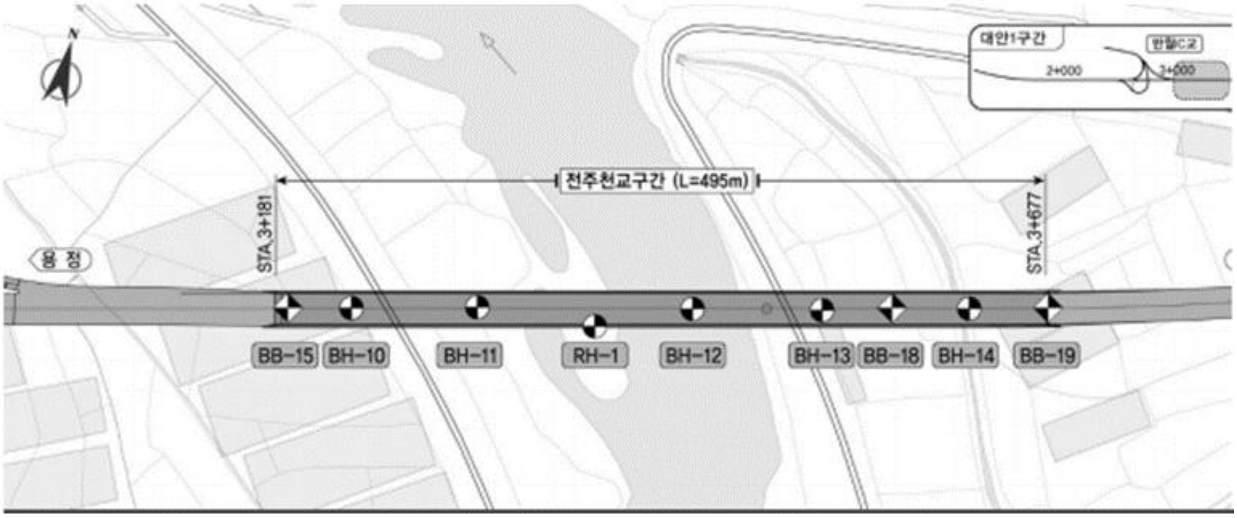


Figure 4.1. Map view of bridge location.

As it can be seen from Figure 4.1, points BB-15, BH-10, BH-11, RH-1, BH-12, BH-13, BB-18, BH-14, BB-19 were pointed out. For these sections only, there are detailed characteristics of soil profile that were available for research. The soil profile data was recorded in Appendix – A, Table A-2. For the design of foundation, section BH-12 needs to be analyzed and considered. The reason is that this section represents the soil profile under the water of 1.6 m the river. The characteristics of each layer for this section are listed further in Table 4.1.

Table 4.1. Geological strata of the section.

Soil Layer	Characteristics	Classification
Sedimentary rock	• Found at depth 0.0 - 8.2m • Composed of silty sand and mixed with gravel • Wet condition, light brown to dark brown color	Silty gravel
Weathered rock	• Found at depth 2.3-19.5m • Granite is weathered and decomposed into silty sand due to its impact • The color may vary from tan to dark gray	Silty sand
Soft stone	• Found at depth 6.7-24.0m • The color of granite may vary from gray to dark gray color • Normal weathering, there are minor cracks	Bed rock
Carcass	• Found at depth starting from 9.5m • Carcass of granite • Slightly weathered or fresh	

For the convenience of calculation, the soil layers were classified, and all the numerical characteristics were determined through thorough analysis. The properties of soil used for design are summarized in Table 4.2 and the soil profile is illustrated in Figure 4.2.

Table 4.2. Mechanical-physical properties of the soil.

Layer	Depth	Layer	Saturated unit	Effective	Modulus of	Density,

	(m)	thickness, h(m)	weight, γ_{sat} (kN/m^3)	frictional angle, ϕ'	Elasticity, E (MPa)	ρ (kg/ m$^{3)}$
Water	1.6	1.6	9.8	-	-	1000
Silty gravel	4.3	2.7	20.75	40	20	1974
Silty sand	18.6	14.3	21.2	38	30	1972
Bed rock	19.6	-	26	-	50000	-

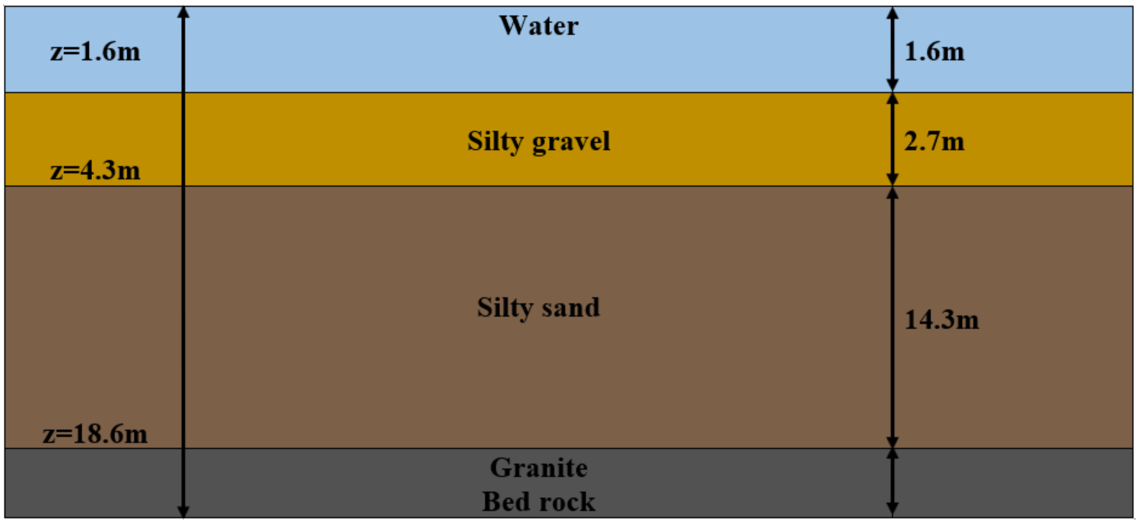


Figure 4.2. Soil profile of the site.

In addition to language barrier and unfamiliarity with the Korean data storage sources, Jeonju is considerably small city. Given current limitations, the information obtained and provided is the only one that can be found and used for design. In reality, soil profile can differ slightly from place to place and depth of river may vary from location to another. However, since there is no access for field and laboratory tests on the given region, there is no results of SPT and CPT. Thus, the soil profile is considered to be uniform.

4.4.Choice of deep foundation type (precast reinforced concrete piles)

As it was mentioned earlier, the most preferred foundation piles for the design is driven piles and drilled shafts. Piles that are embedded into the ground by driving is classified as driven piles, and they can be of various types: precast reinforced concrete, precast prestressed concrete, cast in place concrete using steel shell, steel pipes, and timber; while drilled shaft, also called cast in drilled hole (CIDH) piles, are placed in the ground, firstly by drilling, then using casing and casting concrete in-situ, and they can be either reinforced or prestressed. The choice between mentioned piles depends on various issues, such as noise, liquefaction, scouring, settlement, drivability, and availability. Timber piles cannot resist large structural loads, and steel pipes are costly. Thus, for a beam bridge crossing the river, the considering driven piles and drilled shafts, preferable pile type is driven piles based on advantages and disadvantages listed below.

Table 4.3. Advantages and disadvantages of driven piles and drilled shafts.

	Advantages	Disadvantages
Driven piles	• Resisting capacity is high; • Anti-corrosive methods can be easily accommodated; • Possibility of hard driving;	• Splicing is difficult without sufficient length specified; • If spliced rate of breakage increases;
Drilled shafts	• Easy accommodation of length; • Strong in bearing and resistance; • Construction method is not limited to one;	• A thorough inspection is needed; • Quality depends on construction process; • Boulders can adversely and seriously affect quality; • Most of the resistance comes from end bearing resistance, which will require tip displacement;

4.5. Design calculation methods

Static analysis is conducted to find sum of soil and rock resistance at toe of the pile and along the pile shaft. To design the foundation, 4 general types of static analysis were performed. Further properties of a pile and pile group were determined:

1. Ultimate axial compression capacity, i.e. bearing capacity;
2. Ultimate uplift capacity;
3. Ultimate lateral resistance;
4. Settlement.

According to the soil profile of the area, static analysis will be performed for cohesionless soil. Resistance of the shaft occurred from its surface area (R_s), and toe resistance (R_t) will sum up to find ultimate pile capacity:

$Q_u=R_s+R_t$

This load transfer is illustrated in Figure 4.3 for a single pile in a pile group.

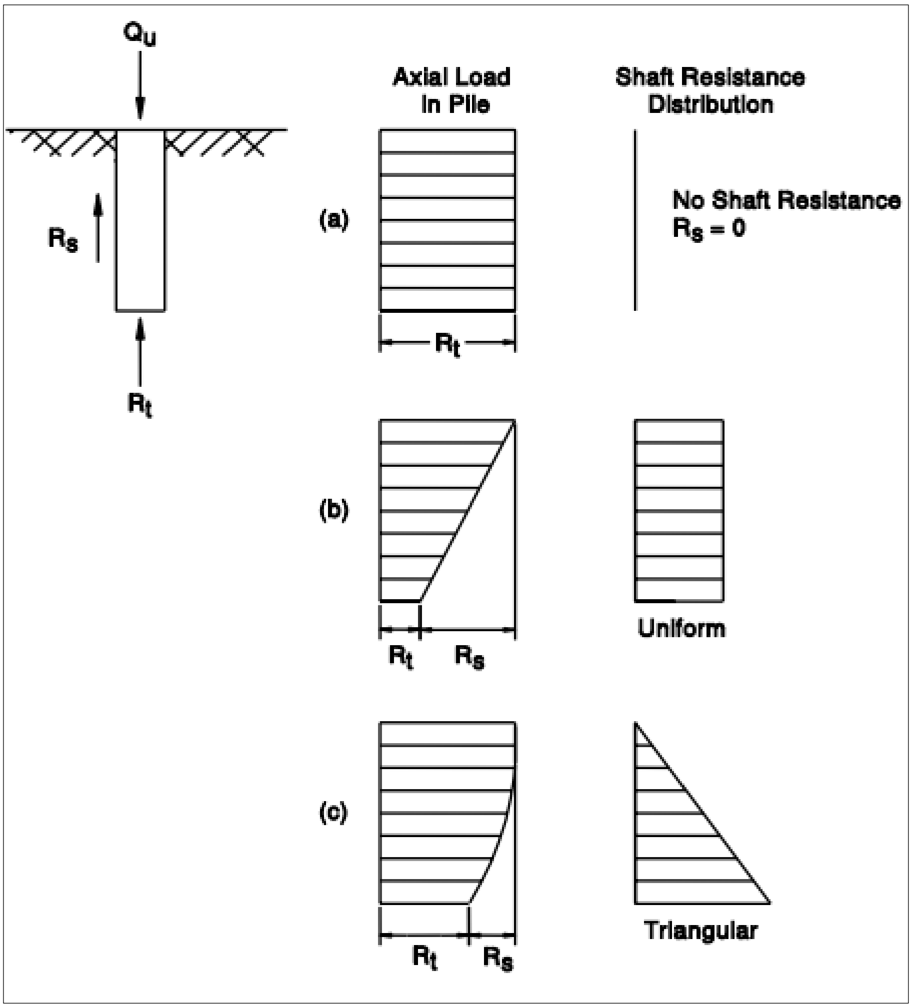


Figure 4.3. Profile of the load transfer.

In Figure 4.3 for (a) shaft resistance was not considered. For (b) shaft resistance was considered uniform, which is typical for cohesive soil, which is not applicable for this project. For (c) shaft resistance was considered triangular, which is characteristic for cohesionless soils.

After finding ultimate pile capacity, allowable soil resistance can be found by dividing it by safety factor. The value of factor of safety is determined from the Table 4.4, which was provided by AASHTO code.

Table 4.4. Factor of safety.

Construction Control Method	Factor of Safety
Static load test (ASTM D-1143) with wave equation analysis	2.00
Dynamic testing (ASTM D-4945) with wave equation analysis	2.25
Indicator piles with wave equation analysis	2.50
Wave equation analysis	2.75
Gates dynamic formula	3.50

4.5.1. Ultimate axial compression capacity

Ultimate axial compression capacity is calculated to determine the performance of the foundation in the long term and to determine the soil layer resistance to loads. To calculate ultimate capacity of the piles, 3 methods provided by AASHTO design code: Meyerhof, Nordlund and Effective Stress methods will be used. Meyerhof method is based on SPT. However, since there is no opportunity to conduct Standard Penetration Test on Jeonju soil, standard values specified in AASHTO design code will be used. Norlund and Effective stress tests are calculated also using standard values. For the bridge design all the methods will be calculated, and values will be compared and analyzed.

4.5.1.1. Meyerhof SPT method of static axial pile capacity calculation

Meyerhof uses correlation between SPT results and static pile load test for cohesionless soils. The detailed method to calculate ultimate and allowable deign load is written below.

$Q_a=\frac{Q_u}{FS}$;

$Q_u=R_s+R_t$;

- $R_s=R_{s1}+R_{s2}$

    $R_{s1}=f_{s1}A_{s1}$

    $R_{s2}=f_{s2}A_{s2}$

        $A_s=2\pi R*h$, for the circular piles

        $f_s=2\overline{N'} \leq 100$kPa

- $R_t=q_tA_t$

    $A_t=\pi R^2$

    $q_t=\frac{40\bar{N}'_B D_B}{b} \leq 400\bar{N}'_B$

Q_a - allowable design load (kN);

FS - factor of safety;

Q_u - ultimate pile capacity (kN);

R_{s1}, R_{s2} - ultimate shaft resistance for each layer;

A_{s1}, A_{s2} - pile shaft exterior surface area;

h - height of the pile section at a specific layer;

f_{s1}, f_{s2} - unit shaft resistance (kPa);

$\overline{N'}$ - average corrected SPT N' value for each soil, calculated using values from the Table A-3 in Appendix - A. All the corrected SPT N' values for the specific layer are determined by corresponding depth values defined in the table;

R_t - ultimate toe resistance (kN);

A_t-cross sectional area of the pile (pile toe area);

q_t - unit toe resistance over the pile toe area;

$\overline{N}'_B$ - the average corrected SPT N' value for the bearing stratum. According to AASHTO code, the effect of overlying stratum is irrelevant since the pile is embedded to more than 10 times pile diameters. Corrected SPT N' values for the bearing stratum are taken from the Table A-3 in Appendix - A. The depth is taken for the bearing stratum considering pile extended to 3 more pile diameters below the pile toe;

D_B - pile embedded length into the bearing stratum.

4.5.1.2. Nordlund Method of static axial pile capacity calculation

Nordlund method in order to calculate shaft resistance, considers the shape of the pile taper and its soil displacement. Also, differences in coefficient of soil-pile friction is considered in this method. Detailed calculation formulas and needed figures and tables are described below.

$Q_a=\frac{Q_u}{FS}$

$Q_u=R_s+R_t$

- $R_s=R_{s1} + R_{s2}$

$R_{s1}=K_\delta C_f p_{d1} sin\delta C_d D$

$R_{s2}=K_\delta C_f p_{d2} sin\delta C_d D$

For the pile end which is located at second layer of soil:

$p_{d1}=\gamma_1 \frac{h_1}{2} - \gamma_w \frac{h_1}{2}$

$p_{d2}=[\gamma_1 h_1 + \gamma_2 \frac{l-h_1}{2}]$ -[$\gamma_w (h_1 + \frac{l-h_1}{2})$]

For circular piles $C_d=2\pi R$

δ=$\frac{\delta}{\Phi}$

- R_t= $\alpha_t N'_q A_t p_t \leq q_L A_t$

  $A_t = \pi R^2$

  p_t=$[\gamma_1 h_1 + \gamma_2(l - h_1)] - [\gamma_w(h_1 + h_2)]$

R_s - ultimate shaft resistance;

p_{d1} and p_{d1} - average effective overburden pressure at the midpoint of each soil layer, (kPa);

γ_w - unit weight of water;

h_w - depth of water;

γ_1- unit weight of first layer of soil;

h_1- depth of first layer of soil;

γ_2- unit weight of second layer of soil;

l - length of pile;

K_δ - coefficient of lateral earth pressure for each Φ angle, determined from Appendix - A. Table A-4;

C_f - correction factor, determined from Appendix - F. Figure F-1;

C_d - perimeter of pile;

δ - friction angle between pile and soil;

$\frac{\delta}{\Phi}$ - ratio found from Appendix - A. Figure A-2;

R_t - ultimate toe resistance (kN);

α_t - coefficient based on pile length to diameter ratio, determined from Appendix - A. Figure A-3;

N'_q - bearing capacity factor estimated from Appendix - A. Figure A-4;

A_t-cross sectional area of the pile;

p_t - effective overburden pressure at pile toe;

q_L - limiting unit toe resistance determined from Appendix - A. Figure A-5.

4.5.1.3. Effective Stress Method of static axial pile capacity calculation

Effective stress method can be applied for cohesionless soils and was used to model long term drained shear strength condition. The calculation method with all necessary formulas and components is described below.

$Q_a=\frac{Q_u}{FS}$

$Q_u=R_s+R_t$

- $R_s=R_{s1}+R_{s2}$

    $R_{s1}=f_{s1}A_{s1}$

    $R_{s2}=f_{s2}A_{s2}$

    $f_s=\beta p_0$

    $p_{o1}=\gamma_1\frac{h_1}{2}-\gamma_w\frac{h_1}{2}$

    $p_{o2}=[\gamma_1 h_1+\gamma_2\frac{l-h_1}{2}]-[\gamma_w(h_1+\frac{l-h_1}{2})]$

- $R_t=q_t A_t$

    $A_t=\pi R^2$

    $q_t=N_t p_t$

A_{s1}, A_{s2} are determined as in Section 3.5.1.1

β is beta shaft resistance coefficient, determined for each soil layer from Appendix - A. Table A-5 or Appendix - A. Figure A-6

p_o-effective overburden pressure at midpoint of layer

R_t -ultimate toe resistance (kN)

p_t is calculated as described in Section 3.5.1.2.

N_t is toe bearing capacity coefficient estimated from the Appendix - A. Table A-5 or Appendix - A. Figure A-7.

4.5.2. Ultimate uplift capacity

Ultimate uplift capacity of a pile of a group is needed to determine the resistance of soil against uplift or tensile loading. Often it is important for minimum pile penetration determination. AASHTO design code limit the uplift capacity to the least of the following Figure 4.4:

1. The design uplift capacity of a single pile times the number of piles in a pile group. The design uplift capacity of a single pile is specified as 1/3 the ultimate shaft resistance calculated in a static analysis method, or ½ the failure load determined from an uplift load test.

2. ⅔ the effective weight of the pile group and the soil contained within a block defined by the perimeter of the pile group and the embedded length of the piles.

3. ½ the effective weight of the pile group and the soil contained within a block defined by the perimeter of the pile group and the embedded pile length plus ½ the total soil shear resistance on the peripheral surface of the pile group.

Figure 4.4. AASHTO design code limit of uplift capacity.

For the pile group, uplift capacity calculation will be based on AASHTO design code. As shown in Figure 4.4, uplift capacity value will be limited to one of the followings:

- Uplift capacity of a single pile= $\frac{1}{2}$ * Ultimate Shaft Resistance

  Group uplift capacity = Uplift capacity of a single pile * Number of piles in a group

- Effective weight of a pile group = n*A*h*$(\gamma_c - \gamma_w)$

  Effective weight of soil = (Layer 1+Layer 2) (Gross Area of Pile Group – Sum of Piles Areas)

  Group uplift capacity= $\frac{2}{3}$ (Effective weight of a pile group+ Effective weight of soil)

  n – number of piles in a group

  A – area of the pile

  H – depth of the pile

  γ_c - unit weight of concrete

- $R_s = p_d \tan\phi$

  p_d - the effective overburden stress at depth d

  ϕ - the friction angle of the soil

  Group uplift capacity = $\frac{1}{2}(R_{s1}+R_{s2})$

4.5.3. Ultimate lateral resistance

Ultimate lateral resistance static analysis of a pile group is soil-structure interaction analysis method. It considers soil strength, deformation behavior and structural properties of pile. According to AASHTO design code, there are two approaches to obtain lateral pile capacity lateral load tests or analytical methods. Since our project is based only on “theory, empirical data and rational consideration of various site parameters", analytical method will be used. There are two approaches mentioned in AASHTO design code: Broms' and Reese's. Broms' method in based only on hand calculation and it is very straightforward, while Reese's method is based on computer solution. Thus, Broms' method will be used to calculate lateral pile capacity. In AASHTO design code specification this method is stated to be suitable for purely cohesive of purely cohesionless soils, which is appropriate for this project.

Total Pile Group Lateral Load Capacity = n*Q_m

Q_m=reduction factor*Q_m

$Q_m = \frac{Q_u}{2.5}$

$Q_u/K_p b^3 \gamma$ value is taken from Appendix - A. Figure A-9

$y(EI)^{3/5} K_h^{2/5} / Q_a D$ value is taken from Appendix - A. Figure A-8

$K_p = \tan^2 (45 + \phi/2)$ (value is needed for Appendix - A. Figure A-9 to find $Q_u/K_p b^3 \gamma$)

ηD value is needed to determine pile length

$\eta D > 4.0$ - long pile

$\eta D < 2.0$ -short pile

$2.0 < \eta D < 4.0$ – intermediate pile

$\eta = \sqrt[5]{K_h/EI}$ for cohesionless soil.

$K_h = \frac{1}{2} K_h$(from table) for medium to dense soil.

$K_h = \frac{1}{4} K_h$(from table) for loose soil.

n-number of piles in a pile group

reduction factor determined from Appendix - A. Table A-7

Q_m – maximum allowable working load for a single pile

ϕ – average angle of internal friction

γ - average effective soil unit weight over embedded length of pile

K_h-coefficient of horizontal subgrade reaction within the critical depth. For cohesionless soils it is determined from

M_y - the resisting moment of the pile

4.5.4. Settlement of Pile groups in cohesionless soils

Settlement of a pile group analysis is conducted in order to estimate foundation deformation under the axial loads. Settlement of pile groups in cohesionless soils can be calculated Meyerhof's method. He suggested estimating settlement as follows:

$s = \frac{0.96 p_f \sqrt{B} I_f}{\overline{N\prime}}$ in SI units; $s = \frac{4 p_f \sqrt{B} I_f}{\overline{N\prime}}$ in US units

$I_f = 1- [D / 8B] \geq 0.5$

$\Delta = \frac{Q_a L}{AE}$

$\Delta = \Delta_1 + \Delta_2$

Δ -Elastic compression

s - Estimated total settlement

p_f - Design foundation pressure (Group design load divided by group area)

B - Width of pile group

N' - Average corrected SPT N' value within a depth B below pile toe.

D - Pile embedment depth

I_f - Influence factor for group embedment

L - Length of pile

A - Pile cross sectional area

E - Modulus of elasticity of pile material

Qa - Design axial load in pile

4.6. Design calculation

As it was written in the previous sections, there are 3 methods used for the calculation of the bearing capacity. The calculation is done using Excel, and the datasheet is presented below. Factor of safety for the ultimate bearing capacity of single pile in static analysis (FHWA Section 9.6).

4.6.1. Static axial pile capacity calculation

Table 4.5. Stresses at various pile lengths.

Stresses			
Pile Length	p_t	u	p_0
8	184.07	94.08	89.99
10	226.47	113.7	112.79
12	268.87	133.3	135.59
14	311.27	152.9	158.39
16	332.47	162.7	169.79

Table 4.6. Meyerhof method calculation results.

	pile length	f_s	A_s	R_s	q_t	A_t	R_t	Q_u	$Q_{allowable}$
MEYERHOF	8	70	10.73	751.4	11476.38	0.29	3349.5	4100.904	2050.452
	10	70	14.56	1019	13200	0.29	3852.6	4872.083	2436.041

	12	64	18.39	1177	13200	0.29	3852.6	5029.833	2514.917
	14	62	22.22	1378	13200	0.29	3852.6	5230.519	2615.259
	16	66	24.14	1593	13200	0.29	3852.6	5445.815	2722.907

Table 4.7. Nordlund method calculation results.

	pile length	б/ф	ф	б	$K_б$	C_f	p_d	C_d	R_s
NORDLUND	8	0.8	38	30.4	1.2	1.08	59.78	1.915115	397.9075
	10	0.8	38	30.4	1.2	1.08	71.18	1.915115	652.5761
	12	0.8	38	30.4	1.2	1.08	82.58	1.915115	964.5132
	14	0.8	38	30.4	1.2	1.08	93.98	1.915115	1333.719
	16	0.8	38	30.4	1.2	1.08	99.68	1.915115	1539.797

	pile length	D/b	α_t	N_q	p_0	R_t	Q_u	$Q_{allowable}$
NORDLUND	8	13.12	0.6	235	89.99	3703	4101.2	2050.622
	10	16.4	0.6	235	112.79	4642	5294.2	2647.098
	12	19.69	0.6	235	135.59	5580	6544.4	3272.208
	14	22.97	0.6	235	158.39	6518	7851.9	3925.952
	16	26.25	0.6	235	169.79	6987	8527.1	4263.562

Table 4.8. Effective stress method calculation results.

	pile length	B	f_s	R_s	ф'	N_t	p_t	q_t	R_t	Q_u	$Q_{allowable}$
EFFECTIVE	8	0.57	51.29	550.6	38	95	184.07	8549.05	2495	3046	1522.87
	10	0.57	64.29	936.3	38	95	226.47	10715.05	3127	4064	2031.83
	12	0.57	77.29	1422	38	95	268.87	12881.05	3760	5181	2590.568
	14	0.57	90.28	2006	38	95	311.27	15047.05	4392	6398	3199.083
	16	0.57	96.78	2336	38	95	332.47	16130.05	4708	7044	3522.007

Table 4.9. Summarized results of static axial pile capacity calculated by 3 methods.

	Meyerhof		Nordlund		Effective stress	
Pile Length	Qu	Qall	Qu	Qall	Qu	Qall
8	4100.904	2050.452	4101.244	2050.622	3045.74	1522.87
10	4872.083	2436.041	5294.195	2647.098	4063.66	2031.83
12	5029.833	2514.917	6544.415	3272.208	5181.135	2590.568
14	5230.519	2615.259	7851.903	3925.952	6398.166	3199.083
16	5445.815	2722.907	8527.123	4263.562	7044.015	3522.007

As can be seen from the values obtained above, each method shows different but approximately close allowable bearing capacity for a single pile. Allowable bearing capacity is derived from dividing ultimate bearing capacity by a Factor of Safety = 2 for a static analysis. Results from each method differ from one another, because in each method they use different input for obtaining bearing capacity.

4.7. Special Design Considerations (Scouring)

Soil is subjected to scour whenever there is a stream of flowing water. There are various types of scour, but in general, scour is an erosion of soil materials. The erosion rate depends on material type. If there is a flooding event, loose granular soils may be eroded in a several hours, while cohesive soils can be eroded in a relatively smaller rate. Since the ultimate capacity of a driven pile depends on resistive capacity of the soil, the scour can detrimentally affect the pile capacity that is why it should be considered during the design.

According to FHWA (2006), there are several issues to be considered:

- If the foundation is supported by piles which are subjected to scour, it may be required to reevaluate the design by changing pile dimensions to meet desired requirements;
- Additional lateral restraint should be considered due to increase in unsupported pile length after scour. Since the part of soil is eroded pile length which is above the soil surface may be subjected to buckling. In this case, the maximum supportable pile load may be controlled by buckling, and not by unit stress. The unbraced length must include the effect of the lateral soil resistance for the embedded length of the pile. Per AASHTO Article 10.7.3.13.4 (2012) for preliminary design, the depth to fixity below the ground may be taken as:
    - $1.8*[E_p*I_w/n_h]^{0.25}$

  Where:

  E_p = modulus of elasticity (ksi) (11.11 ksi, Appendix)

  I_w = weak axis moment of inertia for pile (ft^4) (0.7854 ft^4)

  n_h = rate of increase of soil modulus with depth for sands (1.39 ksi/ft).

  Then, the depth to fixity will be:

  $1.8*[11.11*0.7854/1.39]^{0.25}$ = 2.85 ft or 0.868 meters

Per AASHTO Article 10.7.3.13.4 (2012), in stability evaluation the effective length of the pile shall be equal to the laterally unsupported length and embedded depth to fixity.

- Placing the top of the footing or pile cap below the streambed a depth equal to the estimated long term degradation and contraction scour depth will minimize obstruction to flood flows and resulting scour. Footing may be constructed even at lower depth so that piles will not be subjected to scouring. However, if to place pile cap above mudline, the design of foundation will be more cost effective.

According to Papanicolaou et al., (2013), the measured soil depth of gravel bed for different methods were 0.8 m, 3.8 m, and 1.94 m. Thus, in this case, the silty gravel layer of 2.7 m has an approximated scour depth of 1.2 m using standard deviation. So, the bearing capacity results before and after the scouring of 1.2 m for Nordlund method of gravel layer is listed in the table below.

Table 4.10. Norlund method (scouring).

	Nordlund (before scour)			Nordlund (after scour)		
Pile length	p_d	Q_u	Q_{all}	p_d	Q_u	Q_{all}
8	46.635	4101.244	2050.622	34.59	2896.936	1448.468
10	58.035	5294.195	2647.098	45.99	4026.616	2013.308
12	69.435	6544.415	3272.208	57.39	5213.565	2606.782
14	80.83	7851.903	3925.952	68.79	6457.781	3228.891
16	86.53	8527.123	4263.562	74.49	7101.365	3550.683

Table 4.11. Effective stress method (scouring).

Effective stress method				
	Before scour		After scour	
Pile length	Q_u	Q_{all}	Q_u	Q_{all}
8	3046	1522.87	2193.177	1096.588
10	4064	2031.83	3156.101	1578.051
12	5181	2590.568	4218.581	2109.29
14	6398	3199.083	5380.616	2690.308
16	7044	3522.007	5998.966	2999.483

Allowable bearing capacity values for the nordlund method and effective stress method before and after scour differ as the bearing capacity depends on the skin resistance and toe resistance of the pile. So, when the depth of the first layer shifted from 2.7 to 0.4 because of the scouring the pile resistance, which in turn depends on the effective stress at the mid-depth of the layer, decreased. Nevertheless, in the table Q_{all} calculated is for 14 m length piles, and according to the design criteria 4 piles per pile cap still meets the requirements.

4.7.1. Ultimate uplift capacity calculation

Table 4.12. DATA for estimating Design Uplift Capacity.

DATA for estimating Design Uplift Capacity						
Length (m)	Diameter (m)	Area (m^2)	#piles	y_c (kN/m^3)	$y_{(w)}$ (kN/m^3)	Eff.weight (kN/m^3)
14	0.6096	0.291864	4	24	9.8	232.089862
Water (kN/m^2)	Layer 1 (kN/m^2)	Layer 2 (kN/m^2)	Gross Area of pile group (m^2)		Pile area (m^2)	effective weight of soil (kN)
38.22	46.52	303.16	5.945795		1.167454	1791.232622
p_{d1} (kN/m^2)	p_{d2} (kN/m^2)	ϕ_1	ϕ_2	R_{s1} (kN)	R_{s2} (kN)	Total soil shear resistance (kN)
42.37	149.34	40	38	819.235	15293.2	11612.45

Per AASHTO Code (2012), there are 3 criterions listed for design uplift capacity. Using the data obtained from Tables and Figures in the Appendix A (described in Section 3.5), the calculated uplift capacity for each of the 3 criterions is listed in the table 4.13 below. In 1st criteria the design uplift capacity of a single pile is specified as one-third the ultimate shaft resistance calculated in a static analysis method or one-third of the ultimate shaft resistance. 2nd criteria for two-thirds of the effective weight of the pile group and the soil contained within a block defined by the perimeter of the pile group and the embedded length of the piles. 3rd criteria for one-half the effective weight of the pile group and the embedded pile length plus one-half the total soil shear resistance on the peripheral surface of the pile group.

Table 4.13. Estimation of uplift capacities based on criteria.

Criterion 1	Criterion 2	Criterion 3
uplift capacity	uplift capacity	
967.25	2023.32	
design group uplift capacity	design group uplift capacity	design group uplift capacity
3869.00	2579.33	9067.89

Per AASHTO specifications (2012), pile group uplift capacity is up to 2616 kN, and the least uplift capacity of the pile group should be chosen for the design. That is, 2579 kN. Which is:

2579 kN < 2616 kN OK

4.7.2. Lateral pile capacity analysis of a pile subjected to scouring

Data needed for the calculation of lateral pile capacity by Broms' method is on the table 4.14 below. Some data were derived from the listed in the Appendix (described in Section 4.5.3).

Table 4.14. Broms' method calculation results.

Pile Parameters							
E	I (m4)	S (m3)	f'c (Mpa)	D	b	e©	My
27800	0.006779	0.0222	24.82	14	0.6096	0	551.004

Broms' method	4d	5d	K_h (kN/m^3)	adjust K_h	η (m^{-1})	η *D	$\phi_{average}$
	3.048	3.81	5429	2714.5	0.491932	5.903184	35.36
	K_p	avrg $y_{(eff)}$	$\frac{My}{y*b^4*Kp}$	$\frac{Qu}{y*b^3*Kp}$	Q_u (kN)	Q_m	y
	2.7	15.90215	92.92986	50	486.3227	180.613	2.77766

Under lateral loading the permissible horizontal deflection is limited to 10 mm.

So, in this case the critical depth is chosen to be 3.81 m. The maximum allowable working load for a single pile, Q_m, is derived from the ultimate load, and Q_u is the ultimate failure load for a single pile. The deflection of pile in cohesionless soil is 2.78 mm. Thus, the working load for the single pile meets the requirement of maximum horizontal deflection of 10 mm. So then:

Deflection of pile	<	10 mm	
2.78 mm	<	10 mm	OK
Working load (Q_m)	<	Failure load (Q_u)	
180.613 kN	<	486.32 kN	OK

$$\text{Total Pile group lateral load capacity} = 180.613 * 4 = 722.45 \text{ kN}$$

4.7.3. Settlement calculation

Table 4.15. Data for estimating SETTLEMENT by Meyerhof method based on SPT data.

Data for estimating SETTLEMENT by Meyerhof method based on SPT data

Load (kN)	length	width	p_f (kPa)	SPT N'	depth l (m)	A (m^2)	E (kPa)
4982	2.1336	2.1336	1094.4	34	14	0.29	2.8E+07

In calculation of settlement group load (4982 kN) is divided by group area (2.1336*2.1336 m^2). Next step, is corrected SPT N' value for the depth below pile toe. From the data above, we calculate I_f which is:

$I_f = 1 – [12/8*2.1336] = 0.29696 < 0.5$

As influence factor is smaller than required, next step is to calculate estimated total pile group settlement due to soil compression.

$$S = \frac{0.96*1094.4*\sqrt{2.1336}*0.29696}{34} = 8.115 \text{ mm}$$

Then, elastic compression of each pile is needed to be calculated for each pile segment, which are:

$$\Delta_1 = \frac{1358*5.3}{0.29*27800000} = 0.00089 \text{ m or } 0.89 \text{ mm}$$

$$\Delta_2 = \frac{692*2.7}{0.29*27800000} = 0.00023 \text{ m or } 0.23 \text{ mm}$$

So, the Total pile group settlement is:

$$\Delta = 8.115 + 0.89 + 0.23 = 9.23 \text{ mm} < 25 \text{ mm} \quad \text{OK}$$

It is less than the maximum allowable settlement of the pile group. So, the pile cap dimensions and number of piles chosen for the design is preferable.

The settlement was also analyzed using the PLAXIS 2D software. This is needed in order to check the deformation under the pile toe and effects of stress of the surrounding soil. There are as mentioned earlier 2 layers of soil, and in the software there were two main stages: construction and dynamic loading. As it can be seen from the Figures 4.5, 4.6 below the analysis shows the soil-pile interaction, and total displacement was calculated to be 18.07 mm, which is smaller than the allowable settlement of 25 mm.

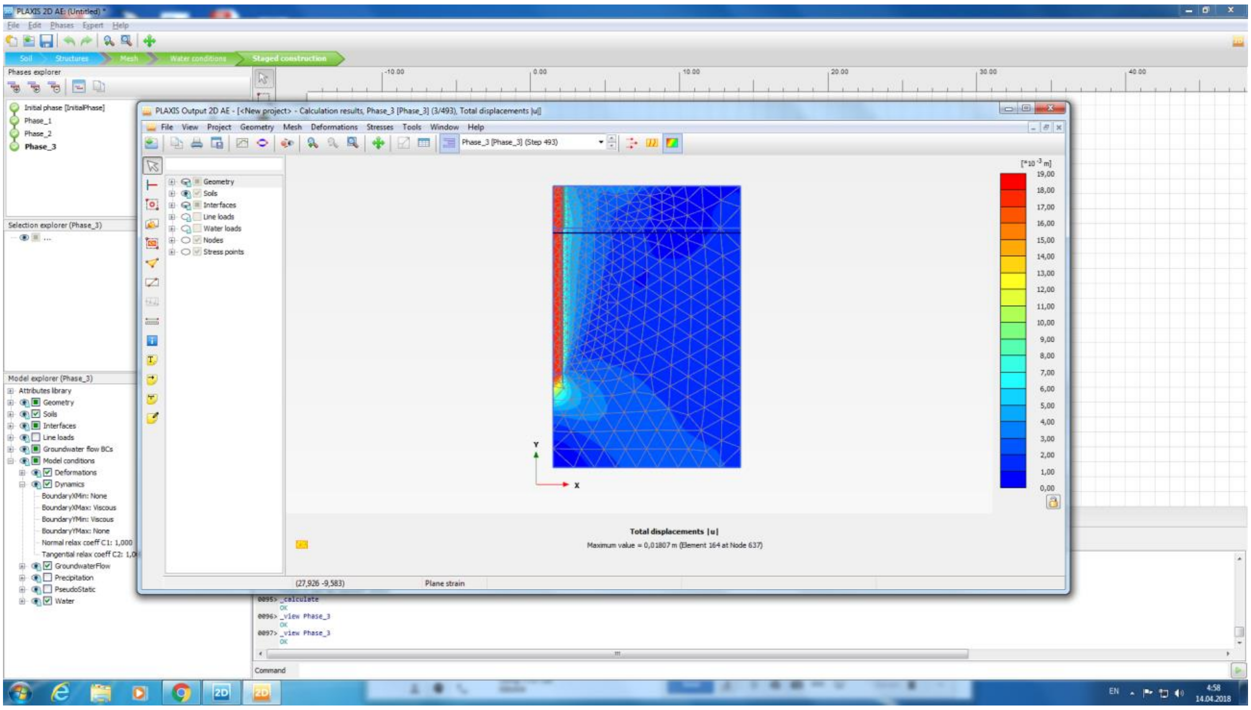


Figure 4.5. Plaxis simulation settlement of a single pile.

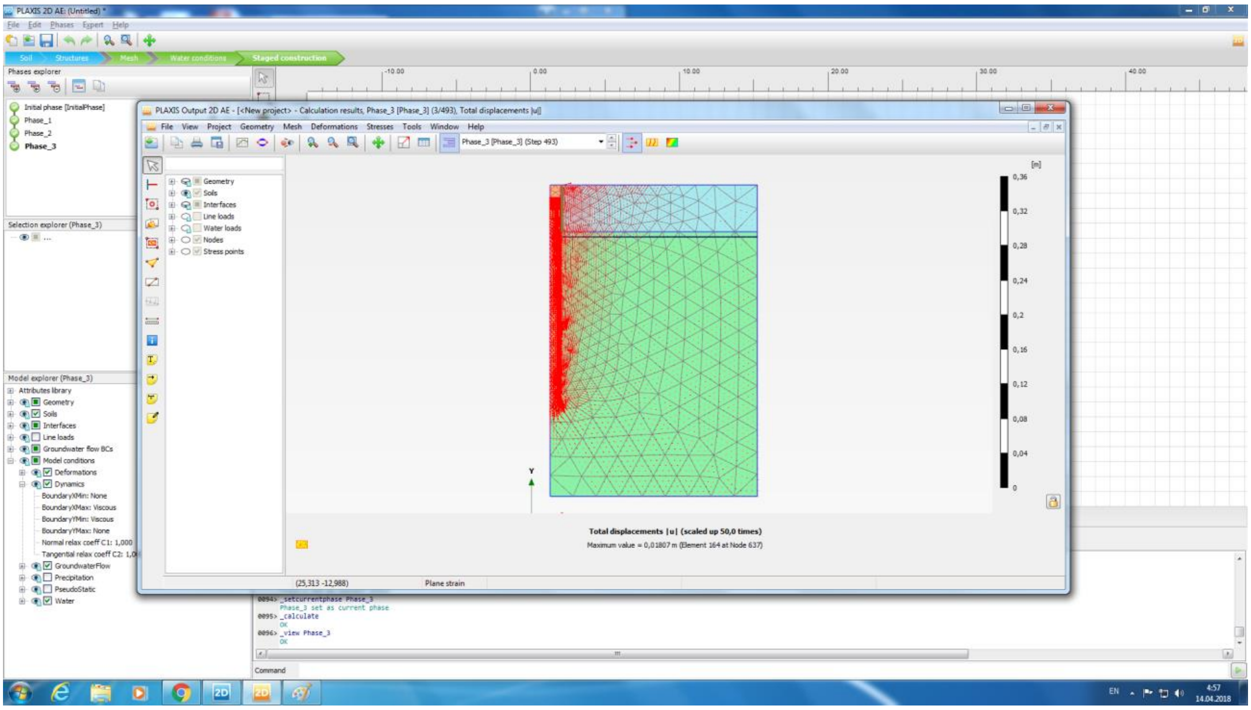


Figure 4.6. Analysis of the settlement using PLAXIS 2D software.

4.8. Design of piles and preliminary design of pile caps

4.8.1. Pile requirements

According to AASHTO (2012, Article 10.7.1.1), minimum center-to-center pile spacing is 30 in or 2.5 pile diameter, whichever is greater, and the nearest pile side and pile cap edge distance is to be no less than 9 in or 0.5 pile diameter, whichever is greater. The projection of pile top into the pile cap shall be minimum 12 in, but if pile is projected into the cap with the help of bras or strands, then the projection should be minimum of 6 in.

In accordance to the AASHTO specifications (Article 5.13.4.4.3) the longitudinal reinforcement of the pile should consist of minimum 4 bars spaced uniformly around the perimeter of the pile. The area of reinforcing steel shall not be less than 1.5% of the gross concrete cross-sectional area. The full length of longitudinal steel shall be enclosed with spiral reinforcement or equivalent hoops. In accordance with AASHTO Article 5.13.4.6.3e (2012), for precast piles the reinforcement should not exceed the pitch of 9 in, and should be minimum of 4 inches.

According to AASHTO Articles 4.5.7.3 and 4.5.11 (2012), design stresses and driving stresses should be as (see Appendix A).

Design stress: $0.33\ f'_c - 0.27\ f_{pe}$ = 0.33*34.5 – 0.27*5 = 10.035 MPa

Driving stress (compression limit): should be smaller than 0.85*34.5-5 = 24.325 MPa

Driving stress (tension limit): should be smaller than 5 MPa for severe corrosive environments.

4.8.2. Layout of pile groups (FHWA Section 10.5)

Number of piles is derived from the ratio of largest axial load of the superstructure to the allowable axial load on a single pile, which is:

$$\frac{Fz}{Qa} = n$$

Where:

F_z = largest axial load of the superstructure

Q_a = allowable axial load on a single pile

n = number of piles in group;

Choosing the minimum allowable bearing capacity amongst the methods mentioned earlier after scouring will be of effective stress method. In which, pile number per cap will be:

n = 9830 kN / 2690 kN = 3.65 (round-up) = 4 piles

Thus, for each pile cap, it is preferable to design 4 piles under each footing.

Then the trial configuration to be checked for single pile axial capacity adequacy under the combined superstructure axial loads and moments. The maximum single pile axial loads may be derived from:

$$q_s = \frac{F_z + W_c + W_s}{n} \pm \frac{M_x * y}{\sum y^2} \pm \frac{M_y * x}{\sum x^2}$$

where: F_z = factored, axial load of the superstructure acting upon the pile cap

W_c	=	estimated weight of pile cap
W_s	=	estimated weight of soil above pile cap, if applicable
N	=	number of piles in the group
M_x	=	factored, moment about the x axis acting on the pile cap
M_y	=	factored, moment about the y axis acting on the pile cap
x	=	distance along x-axis from the center of the column to each pile center
y	=	distance along y-axis from the center of the column to each pile center

Then,

q_s = (4915+275.24)/4 + 1864.4*39.6/(4*39.6)+(627.3*39.6/(4*39.6) =

q_s = 1920.5 kN

All the values above are calculated using excel sheet and loads from the structural part. Considering the lowest allowable bearing capacity and factored axial loads of the superstructure after scouring, the required allowable bearing capacity is when the pile length reaches 14 m, that is:

2690 kN > 1920.5 kN OK

1. Determined pile cap dimensions in accordance to AASHTO Specifications are as follows:

   Depth = 1.524 m; Width and Length (square cap) = 2.7432 m.

   Determine effective depth to concrete reinforcement, d.

   d = D – pile embedment – clear space – distance to center of steel (1.5*d_b)

   d = 2.7432 – 0.3048 – 0.150 – 0.2286 = 2.0598 m

   Determine critical punching shear perimeter, b_0, around the column.

   b_0 = (1.2 + 2.0598) = 3.2598 m

2. Check punching shear at d/2 from column

Per AASHTO Article 8.15.5.6 (2012) and Article 8.15.5.6.3 (2012) the followings are:

$v \leq v_c$

v = (n_o*Q_A)/(b_o*d)

v = (4*2690)/(1.6298*3.2598) = 2025.3 kN

$v_c = (0.8 + 2/\beta_c)*(f'_c)^{0.5}$

v_c = 16450 kN

2025.3 kN < 16450 kN OK

3. Check beam shear, AASHTO Article 8.15.5.6.1 (2012):

   Beam shear D = c/2 + d/2 (AASHTO Article 4.4.11.3.2)

$D=3.2598/2 + 1.2/2 = 2.23$ m

$v \leq v_c$ (AASHTO Article 8.15.5.6)

Shear load and stress:

$V = n_c * Q_A$

$v = n_cQ_A/wd = 4*2690/2.7432*1.2 = 3268.7$ kN

$v_c = 0.95\ f'_c{}^{0.5} = 0.95 * 5.874 = 5580$ kN

3268.7 kN < 5580 kN OK

4. Check Bending

$M_u = \sum Q_A * z$ (z – distance from the edge of the column to each pile)

Area of steel required (AASHTO Article 8.16.3.2):

$M_u = \phi M_n = \phi[A_sf_y\ (d - a/2)]$

$M_u = 0.9 * [0.073\ m^2 * 413685\ kN/m^2 * (1.2 - 0.375/2)] = 27513$ kN-m

$a = A_sf_y / 0.85\ f'_c\ b = 0.073*413685 / 0.85 * 34500 * 2.7432 = 0.375$

Failure in the concrete is assumed to take place at 0.003 strain. The reinforcement ratio, A_s/bd must be less than p_b.

$p_b = [0.8*0.85*f'_c]/f_y * [87000/(87000+f_y)] = 9.854 * 10^{-3}$

$A_s/bd = 8.16 * 10^{-3}$

8.16 E-3 < 9.854 E-3 OK

Table 4.16. Maximum allowable stresses for precast, prestressed, concrete piles.

TABLE 10-4 MAXIMUM ALLOWABLE STRESSES FOR PRECAST, PRESTRESSED, CONCRETE PILES	
	AASHTO (2002) Standard Specification Articles 4.5.7.3 and 4.5.11
Design Stresses	$0.33\ f'_c - 0.27\ f_{pe}$ (on gross concrete area) f'_c minimum of 34.5 MPa (5.0 ksi) f_{pe} generally > 5 MPa (0.7 ksi)
Driving Stresses	Compression Limit $< 0.85\ f'_c - f_{pe}$ (on gross concrete area) Tension Limit (1) $< 0.25\ (f'_c)^{1/2} + f_{pe}$ (on gross concrete area) **SI Units** * $< 3\ (f'_c)^{1/2} + f_{pe}$ (on gross concrete area) **US Units** * Tension Limit (2) $< f_{pe}$ (on gross concrete area) (1) - Normal Environments (2) - Severe Corrosive Environments * **Note:** f'_c and f_{pe} must be in MPa for SI Unit equation and in psi for US Unit equation.

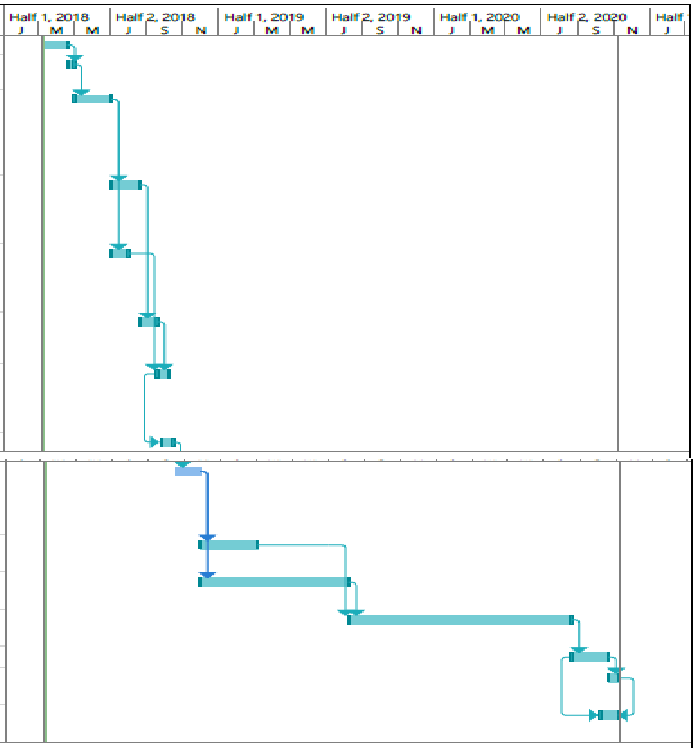
5. Construction Management

For identifying the project's scope and execution time, distinguishing the materials and structures used, calculating the budget of the project the construction management is prepared. In this section the project scheduling, cost analysis, risk management, construction technologies and methods are included.

5.1. Construction Scheduling

The project scheduling and its execution have a significant role in successful construction completion. It includes the clarification of a construction processes and their sequence, as well as the time required for finishing. This schedule is presented in the form of Gantt chart. The concise Gantt chart is presented in the Figure 5.1 below.

ID	Task Mode	Task Name	Duration	Start	Finish
1		site survey	30 days	Fri 09.03.18	Thu 19.04.18
2		rough architectural design	7 days	Fri 20.04.18	Mon 30.04.18
3		calculations of superstructure components' and abutments dimensions	45 days	Tue 01.05.18	Mon 02.07.18
4		calculation of substructure components' dimensions	35 days	Tue 03.07.18	Mon 20.08.18
5		designing of superstructure and abutment components	21 days	Tue 03.07.18	Tue 31.07.18
6		designing of substructure components	21 days	Tue 21.08.18	Tue 18.09.18
7		defining amount of construction materials, needed equipment and labor	14 days	Wed 19.09.18	Mon 08.10.18
8		cost analysis	14 days	Wed 26.09.1	Mon 15.10.18
9		site preparation (conducting site facilities, traffic management)	30 days	Tue 16.10.18	Mon 26.11.18
10		construction of abutments	70 days	Tue 27.11.18	Mon 04.03.19
11		construction of substructure	180 days	Tue 27.11.18	Mon 05.08.19
12		construction of superstructure	270 days	Tue 06.08.19	Mon 17.08.20
13		construction of road	45 days	Tue 18.08.20	Mon 19.10.20
14		commissioning of bridge	14 days	Tue 20.10.20	Fri 06.11.20
15		preparation for opening	25 days	Mon 05.10.20	Fri 06.11.20



Project: Project management
Date: Fri 09.03.18

Task
Split
Milestone
Summary
Project Summary
Inactive Task
Inactive Milestone
Inactive Summary
Manual Task
Duration-only
Manual Summary Rollup
Manual Summary
Start-only
Finish-only
External Tasks
External Milestone
Deadline
Progress
Manual Progress

Page 2

Figure 5.1. Gantt Chart.

5.2. Cost estimation

Estimating the cost of a project is also important part for managing. During the project development the project cost is analyzed and categorized into a preliminary, detailed and final estimation. The preliminary cost estimate means the approximate calculations in the beginning of the project. The detailed and final costs are estimated after the parameters of structures, materials volume and etc. are determined and it gives more accurate numbers (Kerznger, 2013).

For this paper, project cost estimation includes mainly three types of cost: material cost, equipment and labor cost.

Table 5.1. Cost estimation for bridge construction.

Description	**Calculations**	**Number of units**	**Unit cost**	**Material cost**	**Equipment and labor cost**	**Total cost**
Barrier	395.85 kg/m * 500 m	500 m	$3.77/m	$1,885	*2.5	$47,12.5
Wearing surface (asphalt)	0.075 m * 15.892 m * 500 m = 595.95 m3	595.95 m3	$141.73/m3	$84,465	*2.5	$211,162.5
Deck slab	0.2 m * 16.4 m * 500 m = 1640 m3	160 m3	$1438.75/m3	$230,200	*2.5	$575,500
Haunch	0.05 m * 0.425 m * 500 m = 10.625 m3	# of haunch per girder = 5	$38/m3	$1947.5	*2.5	$4868.75
Girder	positive moment region: 31 m * 500 kg/m = 15500 kg 30 m * 500 kg/m = 15000 kg negative moment region: 22 m * 600 kg/m = 13200 kg	positive moment region: 10 40 negative moment region: 45	$2.54/kg	positive moment region: $1,917,700 negative moment region: $1,508,760	*2.5	$4,794,250 $3,771,900
Bent cap	2m * 2.2 m * 14.8 m = 65.56 m3 65.56 m3 * 2402.77 kg/m3 =	# total of bent caps = 9	$76/m3	$44,843	*2.5	$112,107.5

	157525.6 kg					
Column	12 m* $(1.2\ m)^2 * \pi / 4$ = 13.7 m3 13.7 m3 * 2402.77 kg/m3 = 32918.95 kg	# of columns per line= 2 # of lines = 9	$1307.95/ m3	$161,270.2	*2.5	$403,175.5
Pile cap	2.4384 m * 2.4384 m * 1.524 m = 9.06 m3	# of caps per line = 2 total # of caps = 18	$76/m3	$12,394	*2.5	$30,985
Pile	12 m* $(0.6096\ m)^2 * \pi / 4$ = 3.5 m3	# of piles per cap = 4 total # of piles = 72	$76/m3	$19,152	*2.5	$47,880
			Total cost:	$3,982,616.7	*2.5	**$9,956,541.8**

Total cost of project is **$9,956,541.8.**

In order to calculate the total cost of the project, it was considered that the material cost of project takes approximately 40% and rest of the cost goes to the equipment and labor cost. Thus, as it was mentioned in the Table 5.1, the material cost of each section is multiplied by 2.5 factor.

5.3. Risk Management

Risk management is one of the significant parts of construction work. As construction is considered with the human work, risk related to their life are considered more seriously. Thus, the project manager should prepare the risk assessment which includes risk identification, risk value and risk mitigation plan. The risk are identified on the previous experience of construction projects, it is existing risk, and could be only for certain type of construction, where the risks should be predicted. The risk value is found as a function of probability and impact, which is,

*Risk value = Probability value * Impact value*

However, for this paper, it was decided to prepare the list of risks and its mitigation plan because all risks should be taken into account even it has low of high probability and light or sever impact.

Federal Highway Administration's 'Safety and Health on Bridge Repair, Renovation and Demolition Projects' Report categorized 24 sections of hazards with protection from it, but the 12 sections were chosen related to bridge construction. The following table shows these hazards with brief mitigation steps.

Table 5.2. The risk mitigation plan.

RISK RELATED TO:	**RISK MITIGATION**
Personal health and safety	Provide all workers with personal protective equipment including head, hearing, eye and face, foot, and protection.
Fall protection	Conduct trainings for employees. Set the safety nets beyond the edge. Provide with personal fall arrest system and guardrail systems.
Electricity	Give instructions to employees on electrical equipment is used. Install electrical equipment safely according to their voltage. Put the warning and danger signs around electrical equipment. Use the interrupters of ground fault circuit.
Motor vehicles and mechanized equipment	Park all vehicles at special places. Inspect machinery and equipment to verify their safe operating condition. Use the lights, reflectors to notify the location of vehicle or equipment at night. Provide with trainings of how the machinery/ equipment is used.
Hand and power tools	Avoid of using the broken, burned, defective tools, and report about it. Select properly equipment or tool according to

	its area of using. Do not use the power/electric tool in wet or damp conditions. Operate electric tool within design limitations. Wear rubber gloves and safety footwear during working with electrical tools.
Fire protection and prevention	Access to all firefighting equipment always. Do not smoke nearby by flammable and combustible material. Locate equipment with a internal combustion engine-power away from flammable materials.
Sanitation	Supply working places with sanitary conditions. Provide with an adequate number of hand washing facilities, toilets.
Welding and cutting	Use only qualified welders for welding, cutting or heating. Inspect for proper ventilation and no fire hazards before starting to weld or cut. Do not weld or cut sparks.
Floor and wall opening	Cover all floor openings. Close the platforms and ladderway floor openings using railings with toe bards. Do not use equipment and store material to close to floor opening.
Excavations	Do not work in excavation where water is stored. No workers underneath the loads handled by digging and/or lifting equipment. Set the ladders, stairways for excavations lower than 1.2 m.
Diving safety	Supply employees with all diving equipment. Provide training on diving. Record any injury during and after diving.
Work over water	Keep the emergency rescue boat nearby the working area. Protect the workers with fall arrest system. Inspect personal protective equipment from falling and drowning before operations start and etc.

5.4. Construction technologies and methods

In this part the construction technologies and methods of main parts of bridge will be described. It includes sheet pile, pile foundation with pile cap, column, bent cap, girder, deck installations, and asphalt pavement. Also, before starting the work construction site should be prepared.

5.4.1 Sheet Pile Installation

For the installation of sheet piles the vibration, driving are the most common methods (Evans et al., 2012). Its installation equipment are vibratory or driving hammers. In bridge construction sheet piling is executed by a vibratory pile driving hammer. The first step for starting installation is setting out the correct position of sheet pile. The vibratory hammer lifts up a sheet pile and places it vertically to the group in order to drive into the ground. After, the next pile is interlocked with the adjacent one and then driven into. The process is repeated until all sheet piles' installation is competed (Installation of Retaining Wall – Sheet Pile Wall, n.d.).

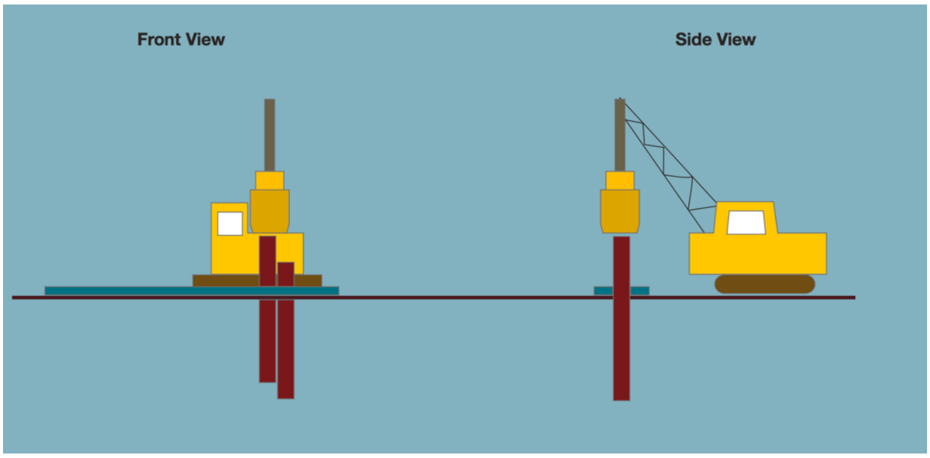


Figure 5.2. Installation of sheet pile (Installation of Retaining Wall – Sheet Pile Wall, n.d.).

5.4.2 Dewatering

Foundation is placed on dry surface for long-term service. Thus, for the execution of the construction work there is a need to remove the surface water, in this case is water from river. This process is called dewatering and its completion, the construction area is dry and solidified, and the discharge of sediment should be minimized (Minnesota Department of Transportation, 2008).

5.4.3 Pile driving

The method that is going to be used for pile driving in bridge construction is drop hammer method. The drop hammer has a heavy ram which is positioned between the leads. The drop method means a raising of ram to a certain height and then releasing it to hit the pile head (Jamal, 2017). The energy generated during the falling of ham causes the installation of pile into the soil. The two types of drop hammers are commonly used: single-acting stream and double-acting pile hammer (Pile Foundation Design, 2003). For this project, the double-acting pile is preferable because it is suitable for site's soil profile.

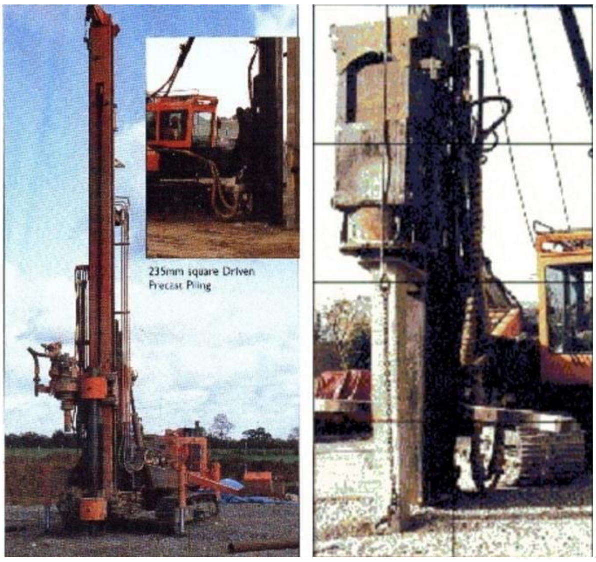


Figure 5.3. Pile driving using hammer (Pile Foundation Design, 2003).

5.4.4 Material durability

Concrete structures are to be designed in such a way that reinforcing steel or prestressing steel should be provided with a protection from the corrosion throughout the structure life. For durability, the considerations of the design are quality of the concrete, coatings that are protective, minimum cover, size of the reinforcement and distribution, details, and width of the crack. Critical factors that affect the concrete durability are (AASHTO LRFD, 2012 Section 5.12.1):

- Reinforcement to be adequately covered;
- Combination of the nonreactive cement and aggregate;
- Thoroughly consolidating the concrete;
- Sufficient cement content;
- Low water-to-cement ratio;
- Curing thoroughly with water;

Concrete cover for steel should not be less than the specifications in Table #. It is necessary in order to prevent from splitting caused by stresses, and providing the tolerance for placement. Galvanizing and epoxy coating of the steel is necessary to protect from the corrosion type of chloride-induced (AASHTO LRFD, 2012 Section 5.12.3).

Table 5.3. Concrete cover for steel (in) (AASHTO LRFD, 2012, Table 5.12.3-1).

Situation	Cover (in.)
Direct exposure to salt water	4.0
Cast against earth	3.0
Coastal	3.0
Exposure to deicing salts	2.5
Deck surfaces subject to tire stud or chain wear	2.5
Exterior other than above	2.0
Interior other than above • Up to No. 11 bar • No. 14 and No. 18 bars	 1.5 2.0
Bottom of cast-in-place slabs • Up to No. 11 bar • No. 14 and No. 18 bars	 1.0 2.0
Precast soffit form panels	0.8
Precast reinforced piles • Noncorrosive environments • Corrosive environments	 2.0 3.0
Precast prestressed piles	2.0
Cast-in-place piles • Noncorrosive environments • Corrosive environments - General - Protected • Shells • Auger-cast, tremie concrete, or slurry construction	 2.0 3.0 3.0 2.0 3.0

Firstly, in order to ensure the durability of the concrete, it is essential to focus on how the components of the concrete is mixed, and w/c ratio as well as admixture, and also curing greatly affects the concrete strength. Concrete consists of the aggregate, cement, admixtures, and water. Engineer should firstly need to understand how each component affects the concrete as can be seen from the Table 5.4.

Table 5.4. Concrete mix characteristics by class (AASHTO LRFD, 2012, Table C5.4.2.1-1).

Class of Concrete	Minimum Cement Content	Maximum *W/C* Ratio	Air Content Range	Coarse Aggregate Per AASHTO M 43 (ASTM D448)	28-Day Compressive Strength
	pcy	lbs. Per lbs.	%	Square Size of Openings (in.)	ksi
A A(AE)	611 611	0.49 0.45	— 6.0 ± 1.5	1.0 to No. 4 1.0 to No. 4	4.0 4.0
B	517	0.58	—	2.0 to No. 3 and No. 3 to No. 4	2.4
B(AE)	517	0.55	5.0 ± 1.5	2.0 to No. 3 and No. 3 to No. 4	2.4
C C(AE)	658 658	0.49 0.45	— 7.0 ± 1.5	0.5 to No. 4 0.5 to No. 4	4.0 4.0
P P(HPC)	564	0.49	As specified elsewhere	1.0 to No. 4 or 0.75 to No. 4	As specified elsewhere
S	658	0.58	—	1.0 to No. 4	—
Lightweight	564	As specified in the contract documents			

The most important factor affecting the strength of the concrete is the water-to-cement ratio. Briefly, the lower is the ratio, the higher is the strength of the concrete, as can be seen from the Figure 5.4. Basically the w/c ratio is chosen from 0.4 to 0.6 (Kosmatka et al., 2002, p.8).

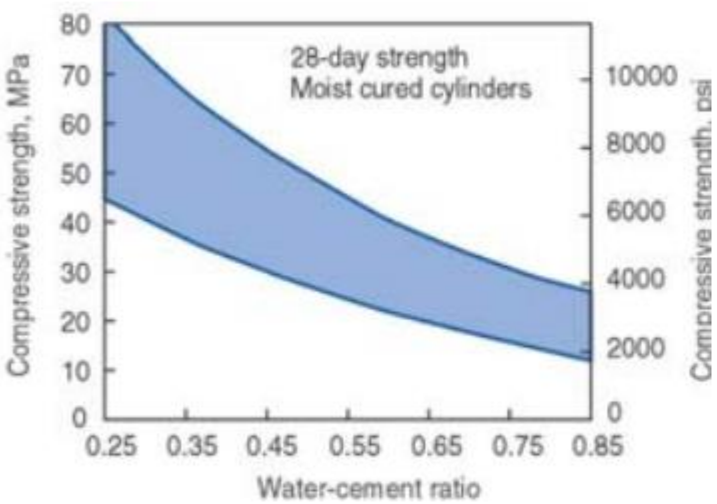


Figure 5.4. w/c ratio and compressive strength (Kosmatka et al., 2002, p.8).

It is essential also to define materials for mixing Portland cement. Since there is not enough information on the availability of raw materials in Jeonju, there are different types of mixing as can be seen in the Figure #, and the as a result there are 6 types of Portland cement listed in the Figure # by representing characteristics and applications of each type (Kosmatka et al., 2002, p.25).

Calcium	Iron	Silica	Alumina	Sulfate
Alkali waste	Blast-furnace flue dust	Calcium silicate	Aluminum-ore refuse*	Anhydrite
Aragonite*	Clay*	Cement rock	Bauxite	Calcium sulfate
Calcite*	Iron ore*	Clay*	Cement rock	Gypsum*
Cement-kiln dust	Mill scale*	Fly ash	Clay*	
Cement rock	Ore washings	Fuller's earth	Copper slag	
Chalk	Pyrite cinders	Limestone	Fly ash*	
Clay	Shale	Loess	Fuller's earth	
Fuller's earth		Marl*	Granodiorite	
Limestone*		Ore washings	Limestone	
Marble		Quartzite	Loess	
Marl*		Rice-hull ash	Ore washings	
Seashells		Sand*	Shale*	
Shale*		Sandstone	Slag	
Slag		Shale*	Staurolite	
		Slag		
		Traprock		

Note: Many industrial byproducts have potential as raw materials for the manufacture of portland cement.
*Most common sources.

Figure 5.5. Raw materials for Portland cement (Kosmatka et al., 2002).

	Classification	Characteristics	Applications
Type I	General purpose	Fairly high C_3S content for good early strength development	General construction (most buildings, bridges, pavements, precast units, etc)
Type II	Moderate sulfate resistance	Low C_3A content (<8%)	Structures exposed to soil or water containing sulfate ions
Type III	High early strength	Ground more finely, may have slightly more C_3S	Rapid construction, cold weather concreting
Type IV	Low heat of hydration (slow reacting)	Low content of C_3S (<50%) and C_3A	Massive structures such as dams. Now rare.
Type V	High sulfate resistance	Very low C_3A content (<5%)	Structures exposed to high levels of sulfate ions
White	White color	No C_4AF, low MgO	Decorative (otherwise has properties similar to Type I)

Figure 5.6. Types of Portland cement (Monograph, n.d.).

Admixtures are also used in increasing the workability and strength of the concrete. Air-entraining admixture is the chemical type in which the air-entrainment increases the strength of the concrete. Basically, it has a beneficial impact on the workability in concrete with low water-to-cement ratio without increasing the permeability of the concrete. Super Plasticizer is also a chemical admixture which is more effective in reducing the water, which in turn may influence fluidity and reduce water-to-cement ratio.

The next factor affecting the concrete strength is the curing. Curing is the suiting the temperature, and time for the concrete to acquire strength. From the figure # below, it can be seen that 28 days moist-cured is enough for concrete acquire 90% of its highest compressive strength, and cast/curing temperature ratio of the concrete shows that at low temperature casting, and warm curing shows the highest compressive strength acquired (Kosmatka et al., 2002, p.6.).

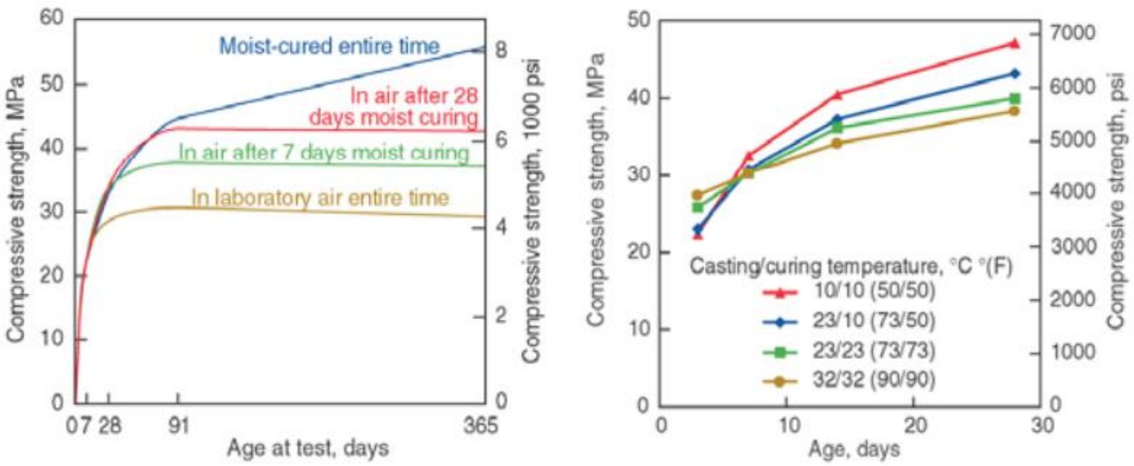


Figure 5.7 Concrete curing (Kosmatka et al., 2002).

The most destructive effect on concrete durability has the freezing and thawing creating a random crack. This happens when the concrete is saturated, then the pores will be filled with water, and when the water freezes, concrete volume increases, and this leads to the crack on the surface of the concrete. The figure 5.8 below shows the effect of freeze-thaw on the concrete which was subjected to 150 cycles (Kosmatka et al., 2002, p.12).

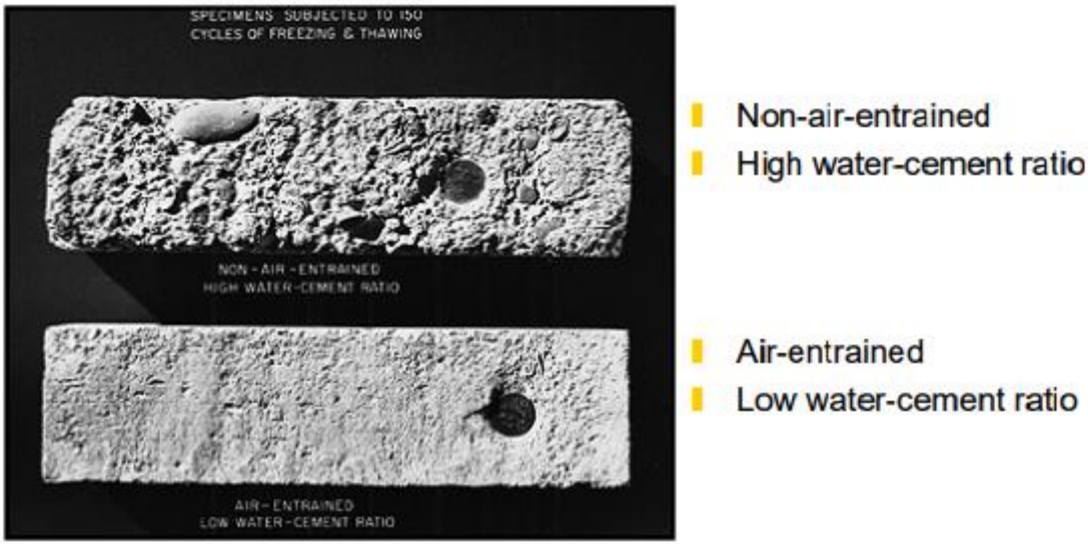


Figure 5.8. Freeze-thaw effect on concrete (Kosmatka et al., 2002).

Non-air entrained and high w/c ratio concrete has a high possibility that lowers the durability of the concrete. Thus, possible solution for resisting freeze and thaw is to use air-entrained concrete with a low w/c ratio, in which the creation of air bubbles relieves the pressure via ice-forming thus lowers the chance of crack to happen (Nawy, 2008).

Groundwaters and seawaters often contain sulfates, which can attack the concrete by forming a gypsum, when penetration of sulfate ions take place in concrete. This leads to the volume expansion, which is in turn causes concrete crack. This process can be prolonged until the concrete is completely deteriorated as in the Figure # (Nawy, 2008).

Figure 5.9. Sulfate attack on concrete.

One of the methods to prevent the concrete destruction caused by sulfate attack is (Nawy, 2008):

- W/C ratio is the most significant part of a concrete mixture, and the ratio should be no more than in the Table 5.5;
- Portland cement with <70% slag content;
- Portland cement with C_3A <3 wt%;

Table 5.5. Concrete requirements exposed to sulfate (Nawy, 2008).

Sulfate Exposure	Water-Soluble Sulfate (SO_4) in Soil (% by Mass)	Sulfate (SO_4) in Water (ppm)	Cement Type[a]	Maximum Water/ Cementing Material Ratio (by Mass)
Negligible	0.00–0.10	0–150	Use any cement	—
Moderate[b]	0.10–0.20	150–1500	II, IP (MS), IS (<70) (MS), MS	0.50
Severe	0.20–2.00	1500–10,000	V, HS	0.45
Very severe	Over 2.00	Over 10,000	V, HS	0.40

[a] Cement Type II and Type V are specified in ASTM C 150, and the remaining types are specified in ASTM C 595 or ASTM C 1157. Pozzolans or slags determined by test to improve sulfate resistance may also be used.
[b] Seawater.

6. Conclusion

While designing bridge in Jeonju, mainly structural component such as girders, deck slab, diaphragms, reinforced cast-in-place concrete deck, drop bent cap, reinforced concrete columns and foundation design has been performed. Geotechnical design of the bridge substructure and soil was performed and bearing capacity of soil and pile foundation has been calculated, which in turn met the imposed load demand. Cost analysis and scheduling yield acceptable results, which mean that not only the bridge construction but the whole project is viable.

7. Reference

American Association of State Highway and Transportation Officials (2012). *AASHTO LRFD Bridge Design Specifications*. American Association of State Highway and Transportation Officials.

Ames Construction, Inc., Minnesota Department of Transportation, & WSB & Associates. (2008). *Minnesota Field Guide for Winter Bridge Construction.* Assessed April 10, 2018

Caltrans, (2012). Bridge Memo to Designers: Guidelines for Identification of Steel Bridge Members, California Department of Transportation, Sacramento, CA.

Caltrans. 2013. *Seismic Design Criteria v1.7*. Retrieved from http://www.dot.ca.gov/hq/esc/earthquake_engineering/sdc/documents/Seismic-Design-Criteria-(SDC-1.7-Full-Version,-OEE-Release).pdf

Caltrans. 2014. Memo to designers 3-1. Retrieved from http://www.dot.ca.gov/des/techpubs/manuals/bridge-memo-to-designer/page/section-3/3-1.pdf

Caltrans. 2015. *Concrete Bent Caps.* Retrieved from http://www.dot.ca.gov/des/techpubs/manuals/bridge-design-practice/page/bdp-12.pdf

Caltrans. 2015. *Deep Foundations.* Retrieved from http://www.dot.ca.gov/des/techpubs/manuals/bridge-design-practice/page/bdp-16.pdf

Caltrans. 2015. Standard Plan B2-5. Retrieved from http://www.dot.ca.gov/hq/esc/oe/project_plans/highway_plans/stdplans_US-customary-units_15/viewable_pdf/b02-05.pdf

Chen W., Duan L. 2000. *Bridge Engineering Handbook.* CRC Press LLC.

Evans, R., White, D., & Klaiber, W. (2012). *Modified Sheet Pile Abutments for Low-Volume Road Bridges* (Tech. No. IHRB Project TR-568).

FHWA-NHI-05-042. 2006. *Design and Construction of Driven Pile Foundation - Volume I.* National Highway Institute. Retrieved from http://apps.itd.idaho.gov/apps/manuals/Materials/Materials%20References/FHWA-NHI-05-042-Volume%20I.pdf

Installation of Retaining Wall – Sheet Pile Wall. (n.d.). Retrieved April 10, 2018, from https://www.lta.gov.sg/content/dam/ltaweb/corp/PublicTransport/files/SPW Final (inside).pdf

Jamal, H. (2017, June 09). Haseeb Jamal. Retrieved April 11, 2018, from https://www.aboutcivil.org/pile-driving-equipment-hammers.html

Kerzner, H., 2013. Project management: a systems approach to planning, scheduling, and controlling. John Wiley & Sons.

Kosmatka S.H., Kerkhoff, B., and Panarese, W.C. 2002. *Design and Control of Concrete Mixtures 14th Edition.* Portland Cement Association. Retrieved from https://www.researchgate.net/profile/Steven_Kosmatka/publication/284663491_Design_and_Control_of_Concrete_Mixtures/links/5655d8f908aefe619b1c5f2b/Design-and-Control-of-Concrete-Mixtures.pdf

Nawy, E.G. 2008. *Concrete Construction Engineering Handbook 2nd Edition.* CRC Press. New Jersey. Retrieved from http://ebooks.narotama.ac.id/files/Concrete%20Construction%20Engineering%20(2nd%20Edition)/Cover%20&%20Table%20of%20Contents%20-%20Concrete%20Construction%20Engineering%20(2nd%20Edition).pdf

Papanicolaou, A.N., Moustakidis, I., Tsakiris, A., Wilson, C. Abban, B. 2013. *Prediction of Scour Depth in Gravel Bed Rivers using Radio Frequency IDs.* WA-RD 821.1. Retrieved from https://www.wsdot.wa.gov/research/reports/fullreports/821.1.pdf

Pier Details. 2016. Accessed in March 9, 2018. Retrieved from http://www.virginiadot.org/business/resources/bridge/Manuals/Part2/Chapter15.pdf

Pile Foundation Design-Chapter 8. (2003, May 05). Retrieved April 11, 2018, from https://archive.is/20030505003515/http://www.sbe.napier.ac.uk/projects/piledesign/guide/chapter8.htm

Safety and Health on Bridge Repair, Renovation and Demolition Projects. (n.d.). Retrieved April 10, 2018, from https://www.fhwa.dot.gov/publications/research/infrastructure/structures/98180/ch3/ch3.cfm

Sheet Pile Wall Construction (n.d.). Retrieved April 10, 2018, from http://www.railsystem.net/sheet-pile-wall-construction/

United States of America, Virginia Department of Transportation. (2016, December 28). *Parapets/Rails/Medians/Sidewalks*.

Appendix A

Table A-1. Soil profile and its characteristics.

Soil layer		Distribution depth, (m)	Characteristics	TCR / RQD (%)	Relative density
Fill		0.0～2.3	· Clay silt composed of silt sand · Wet condition, yellowish brown to dark brown color · Mixed small amounts of gravel and tuff	6/30～40/30	Loose to compact
Sedimentary rock	Sandy soil	0.0～8.2	· composed of silty sand and mixed with gravel · Wet condition, light brown to dark brown color	4/30～40/30	Very loose ~ dense
	Gravel	0.5～6.9	· Silty quality gravel ~ sandy gravel (ø1 ~ 10cm) · Wet condition, light brown to dark brown color	12/30～50/11	Usually dense ~ very dense
Weathered soil		3.6～18.0	·Composed of silt vellum or neutral sand · Wet state, grayish brown to dark gray color	29/30～50/10	Usually dense ~ very dense
Weathered rock		2.3～19.5	· Granite is weathered and decomposed into silty sand due to its impact · Change the color from tan to dark gray	50/10～50/2	Very dense
Soft stone		6.7～24.0	· The soft color of granite, dark gray to dark gray color · Normal weathering, weak to moderately strong, very severe crack to medium crack · Crushing development near RH-1 of 19.0 ~ 20.8m	55/0～87/62 -	-
Carcass		9.5 or less	· Carcass of granite · Slightly weathered or fresh	96/70～100/96	-

Table A-2. Soil profile of each section for the given area.

Section	Groundwater level (m)	Layered depth (m)						Total (m)
		Fill	Sedimentary rock	Weathered soil	Weathered rock	Soft stone	Carcass	
BH-10	3.8	2.3	5.9	5.3	7.0	-	-	20.5
BH-11	3.6	0.5	6.4	11.1	1.5	3.5	2.0	25.0

RH-1	1.8	0.5	3.8	1.7	0.7	17.3	6.0	30.0
BH-12	**Depth 1.6**	**-**	**2.7**	**-**	**14.3**	**1.0**	**6.0**	**24.0**
BH-13	2.3	0.5	4.4	0.6	1.6	2.4	3.8	13.3
BH-14	2.3	0.5	3.1	9.4	3.3	3.2	-	19.5

Table A-3. Field SPT N values with depth.

Depth (m)	p_o (kPa)	Field SPT N value	Correction Factor	Corrected SPT N' (Field SPT N x Correction Factor)
2.0	10.2	7	1.75	12
3.5	17.9	7	1.65	12
5.0	25.5	9	1.43	13
6.5	42.6	85	1.28	109
8.0	59.7	96	1.16	111
9.5	76.0	31	1.09	34
11.0	90.7	35	1.04	36
12.5	105.4	32	0.98	31
14.0	120.1	33	0.95	31
15.5	134.8	38	0.90	34
17.0	149.5	34	0.87	30
18.5	164.2	39	0.83	32
20.0	178.9	41	0.81	33

Table A-4. K_δ values based on displaced volume

ϕ	Displaced Volume -V, m^3/m (ft^3/ft)									
	0.093 (1.0)	0.186 (2.0)	0.279 (3.0)	0.372 (4.0)	0.465 (5.0)	0.558 (6.0)	0.651 (7.0)	0.744 (8.0)	0.837 (9.0)	0.930 (10.0)
25	0.85	0.90	0.92	0.94	0.95	0.97	0.98	0.99	0.99	1.00
26	0.91	0.96	1.00	1.02	1.04	1.05	1.06	1.07	1.08	1.09
27	0.97	1.03	1.07	1.10	1.12	1.13	1.15	1.16	1.17	1.18
28	1.03	1.10	1.14	1.17	1.20	1.22	1.23	1.25	1.26	1.27
29	1.09	1.17	1.22	1.25	1.28	1.30	1.32	1.33	1.35	1.36
30	1.15	1.24	1.29	1.33	1.36	1.38	1.40	1.42	1.44	1.45
31	1.27	1.38	1.44	1.49	1.52	1.55	1.57	1.60	1.61	1.63
32	1.39	1.52	1.59	1.64	1.68	1.72	1.74	1.77	1.79	1.81
33	1.51	1.65	1.74	1.80	1.85	1.88	1.92	1.94	1.97	1.99
34	1.63	1.79	1.89	1.96	2.01	2.05	2.09	2.12	2.15	2.17
35	1.75	1.93	2.04	2.11	2.17	2.22	2.26	2.29	2.32	2.35
36	2.00	2.22	2.35	2.45	2.52	2.58	2.63	2.67	2.71	2.74
37	2.25	2.51	2.67	2.78	2.87	2.93	2.99	3.04	3.09	3.13
38	2.50	2.81	2.99	3.11	3.21	3.29	3.36	3.42	3.47	3.52
39	2.75	3.10	3.30	3.45	3.56	3.65	3.73	3.80	3.86	3.91
40	3.00	3.39	3.62	3.78	3.91	4.01	4.10	4.17	4.24	4.30

To use this table, value of V and ω is needed

ω is pile taper angle, taken to be equal to 0^o

V is volume of displaced soil, V = A_t*1.0 m/m

Table A-5.

TABLE 9-6 APPROXIMATE RANGE OF β AND N_t COEFFICIENTS (Fellenius, 1991)			
Soil Type	ϕ'	Β	N_t
Clay	25 - 30	0.23 - 0.40	3 - 30
Silt	28 - 34	0.27 - 0.50	20 - 40
Sand	32 - 40	0.30 - 0.60	30 - 150
Gravel	35 - 45	0.35 - 0.80	60 - 300

Table A-6.

TABLE 9-14 VALUES OF K_h FOR COHESIONLESS SOILS		
Soil Density	K_h, in kN/m^3 (lbs/in^3)	
	Above Ground Water	Below Ground Water
Loose	1900 (7)	1086 (4)
Medium	8143 (30)	5429 (20)
Dense	17644 (65)	10857 (40)

Table A-7.

z	Reduction Factor
8b	1.0
6b	0.8
4b	0.5
3b	0.4

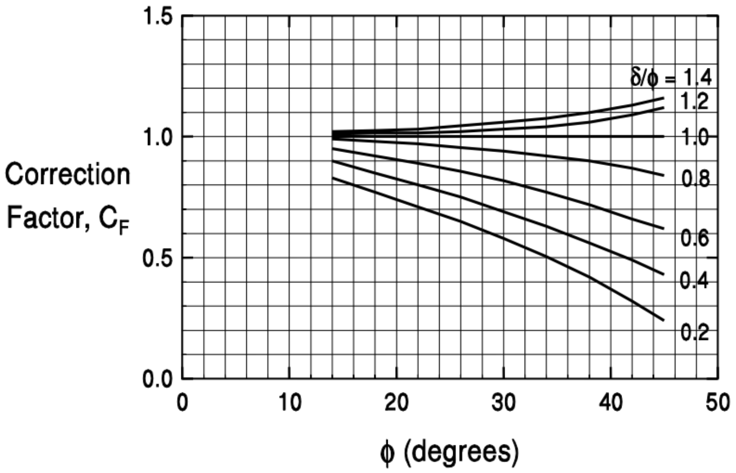


Figure A-1. C_f correction factors based on $\frac{\delta}{\Phi}$ ratio.

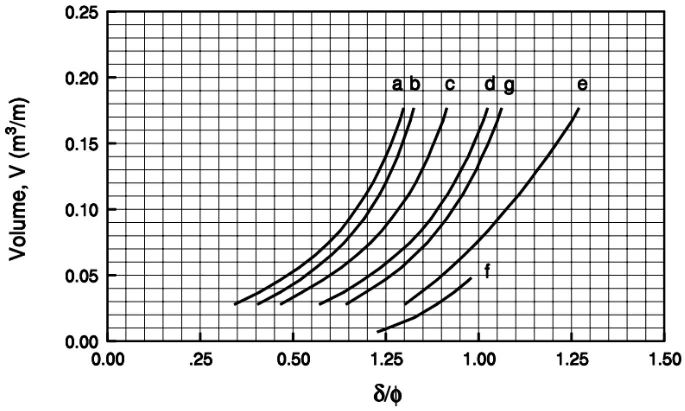


Figure A-2. δ/Φ value based on displaced volume.

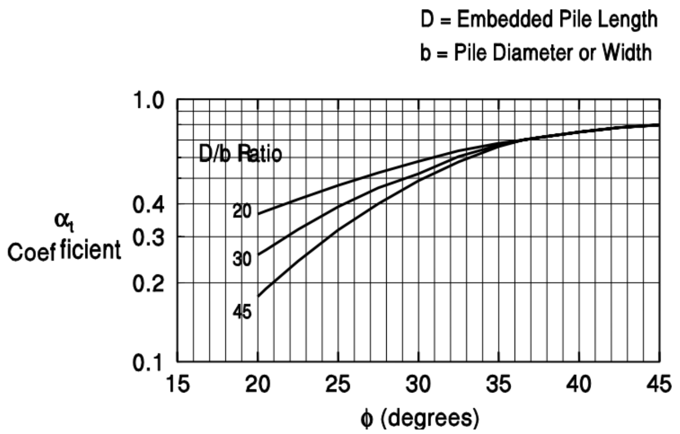


Figure A-3. α_t coefficient based on length to diameter ratio.

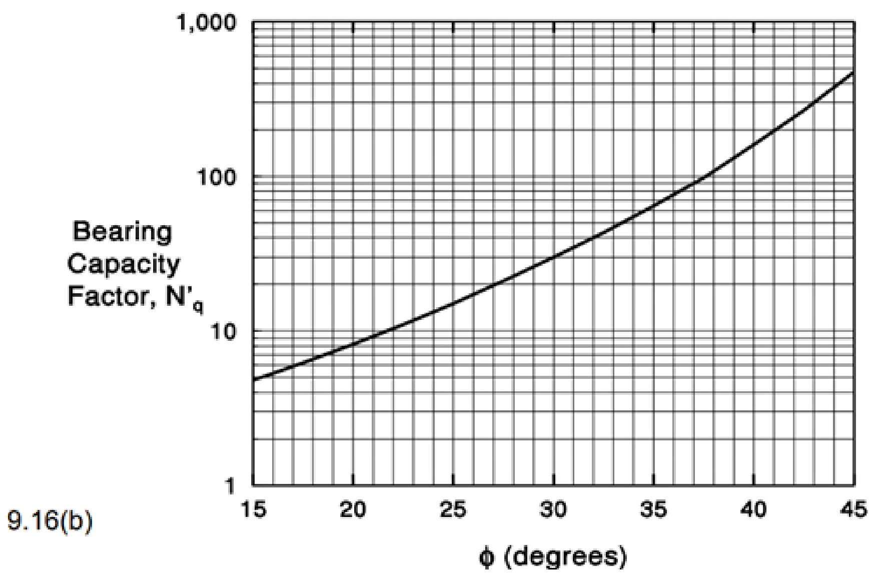


Figure A-4. N'_q factor estimation graph.

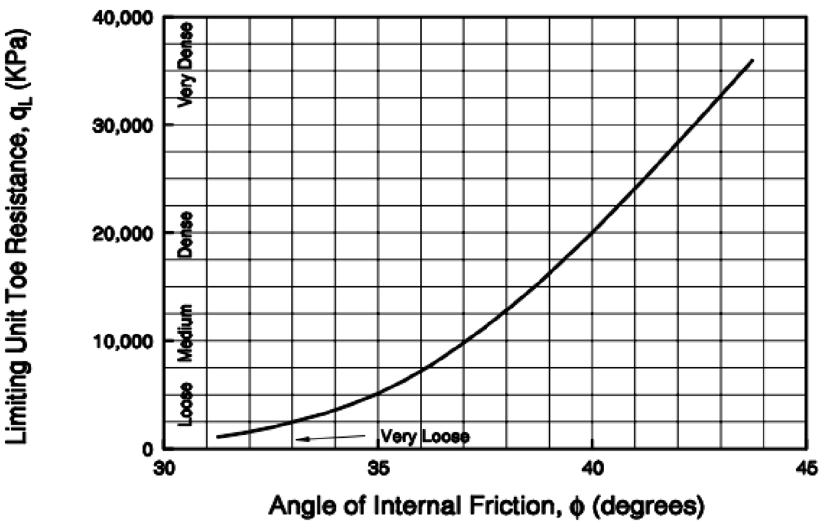


Figure A-5. Limiting unit toe resistance graph.

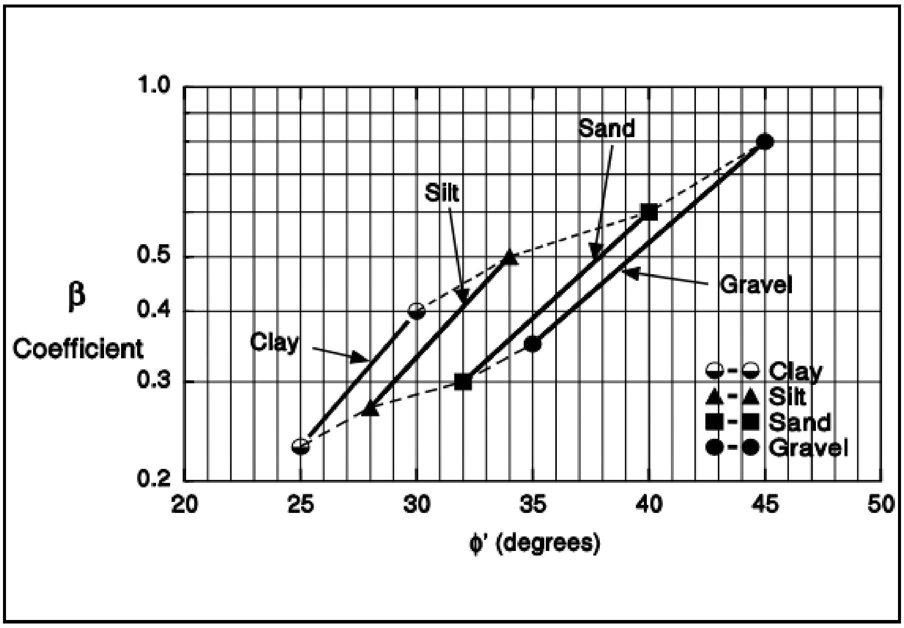
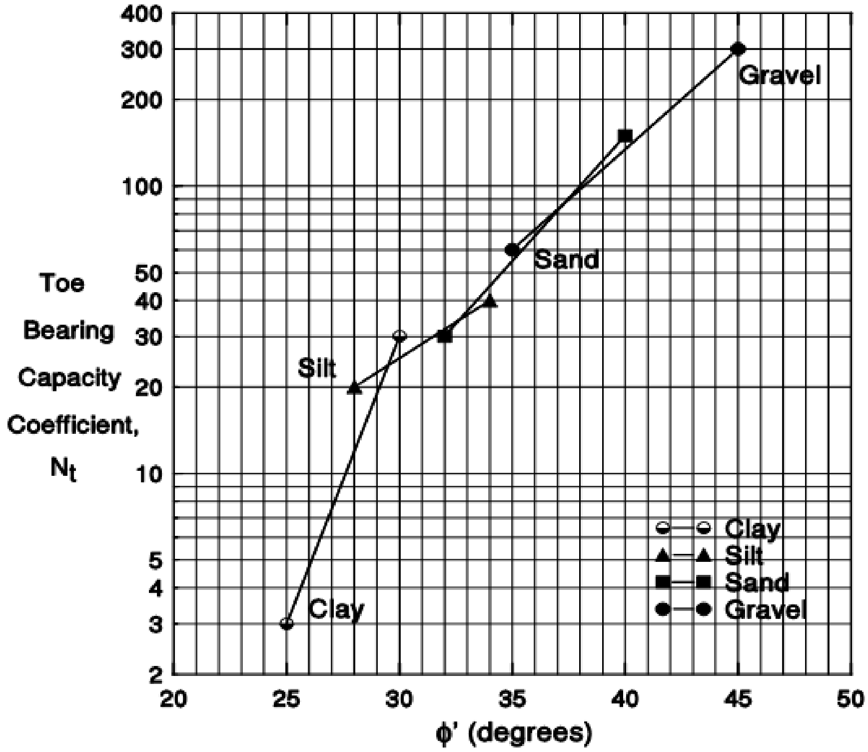


Figure A-6



400
300
200
100
50
40
30
20
10
5
4
3
2
Toe
Bearing
Capacity
Coefficient,
N_t
Gravel
Sand
Silt
Clay
Clay
Silt
Sand
Gravel
20
25
30
35
40
45
50
ϕ' (degrees)

Figure A-7

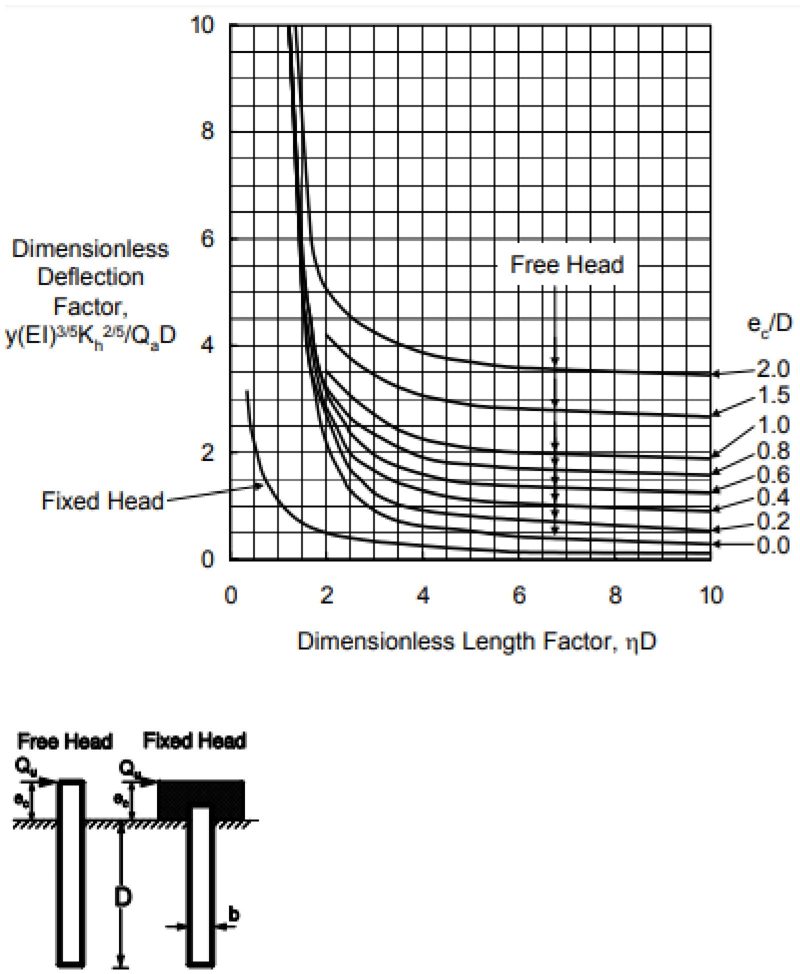


Figure A-8

Dimensionless length factor = ηD

If:

ηD>4.0 - long pile

ηD<2.0 -short pile

2.0< ηD<4.0 – intermediate pile

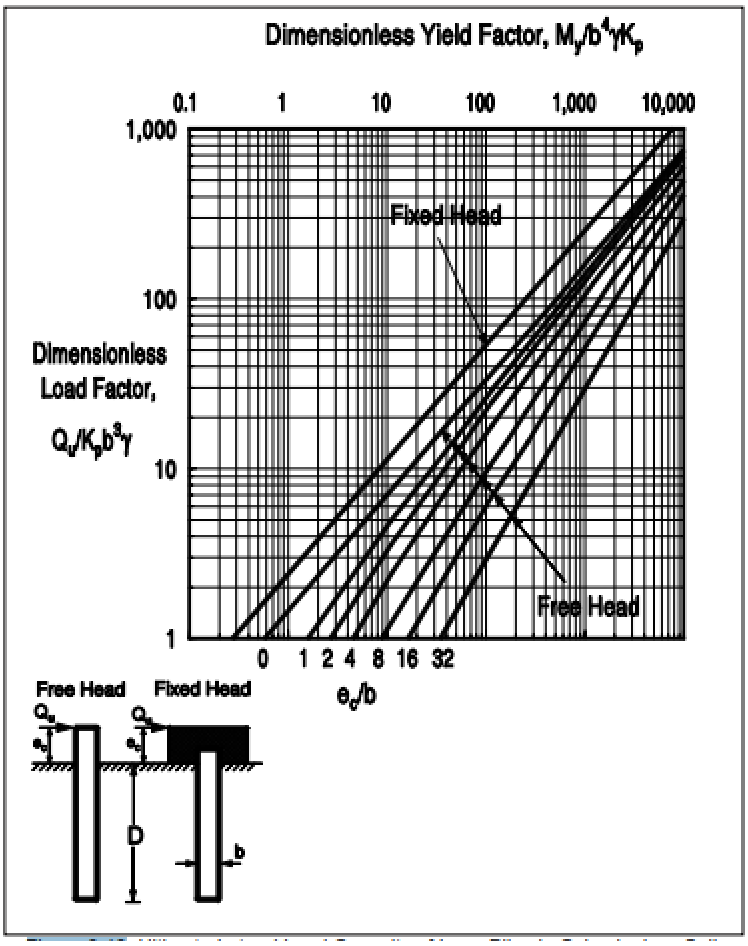


Figure F-9.

$\frac{M_y}{b^4\gamma K_p}$ value is found to use this figure

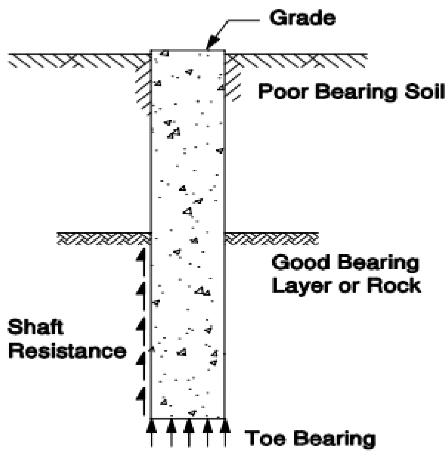
TABLE 8-1 TECHNICAL SUMMARY OF PILES* (CONTINUED)		
PILE TYPE	DRILLED SHAFTS	TYPICAL ILLUSTRATION
TYPICAL LENGTHS	5 m to 65 m or more (15 – 200 ft)	
MATERIAL SPECIFICATIONS	ACI 318 - for concrete. ASTM A82, A615, A722, and A884 for reinforcing steel.	
MAXIMUM STRESSES	33% of 28-day strength of concrete.	
TYPICAL AXIAL DESIGN LOADS	1,500 kN - 20,000 kN (330 – 4500 kips) or more.	
DISADVANTAGES	• Requires relatively more extensive inspection. • Construction procedures are critical to quality. • Boulders can be a serious problem, especially in small diameter shafts. • Mobilization of end bearing on a long shaft can require substantial displacement of shaft head.	
ADVANTAGES	• Length variations easily accommodated. • High bearing capacity and bending resistance. • Availability of several construction methods. • Can be continued above ground as a column.	
REMARKS	• No driving observations (blow count) available to aid in assessing capacity. • Not recommended in soft clays and loose sands.	

Figure A-10.

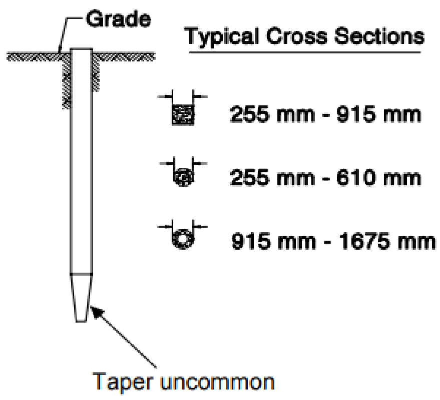
TABLE 8-1 TECHNICAL SUMMARY OF PILES* (CONTINUED)		
PILE TYPE	PRESTRESSED/PRECAST CONCRETE	TYPICAL ILLUSTRATION
TYPICAL LENGTHS	15 m - 40 m (50 – 130 ft) for prestressed. 10 m - 15 m (30 - 50 ft) for reinforced.	
MATERIAL SPECIFICATIONS	ACI 318 - for concrete. ASTM - A82, A615, A722, and A884 - for reinforcing steel. ASTM - A416, A421, and A882 - for prestressing.	
MAXIMUM STRESSES	See Chapter 10.	Note: Reinforcing may be Prestressed
TYPICAL AXIAL DESIGN LOADS	400 kN - 4,500 kN (90 – 1000 kips) for prestressed. 400 kN - 1,000 kN (90 – 225 kips) for reinforced.	
DISADVANTAGES	• Unless prestressed, vulnerable to handling damage. • Relatively high breakage rate, especially when piles are to be spliced. • Considerable displacement. • Difficult to splice when insufficient length ordered.	
ADVANTAGES	• High load capacities. • Corrosion resistance obtainable. • Hard driving possible.	
REMARKS	• Cylinder piles are well suited for bending resistance.	

Figure A-11.