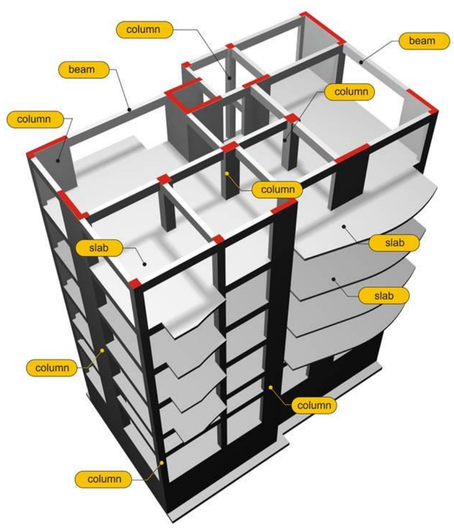
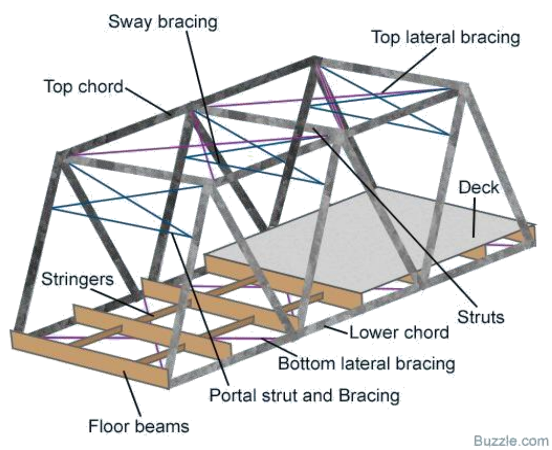
CE 412
Structural Analysis and Design Sessional-II





Department of Civil Engineering
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Preface

This lab handout is intended to give an overview of a Multi storied Building and a Balanced Cantilever Bridge structural analysis and design. It concentrates on the gravity loading only. This handout provides a basic guideline for analysis, design and detailing works as well as reviewing a standard code of practice. To provide the undergraduate students with a well-organized, user-friendly, and easy-to-follow resource, this handout is divided into two major parts. The first part mainly focuses on the structural analysis and design of Reinforced concrete (RC) Multistoried Building that includes design of Slab, Beam, Column, Stair, Water reservoir and Lateral load analysis. The other part deals with the Balanced Cantilever Bridge including an introduction to Bridge Engineering, details about Balanced Cantilever Bridge, design of Deck Slab, design of Railing, Post and Curb/Sidewalk, design of Interior Girder considering dead and live loads only, design of Exterior Girder considering dead and live loads only, design of Diaphragm or Cross Girder and Design of Articulation. Handouts from Prof. Dr. Khan Mahmud Amanat, and Mr. Ruhul Amin, faculty members of BUET, were helpful as well as suggestions from some faculty members of the Department of Civil Engineering, AUST.

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Part 1

Structural Analysis and Design of the Multistoried RC Building

Part 2

Preliminary Design of the Superstructure of a Balanced Cantilever Bridge for Gravity loading

Part 1: Structural Analysis and Design of the Multistoried RC Building

1.1 Introduction

Generally, the design of any structure (building, bridge etc.) can be dividing in two segments,

- Foundation design (footing, basement, retaining wall, abutment, underground water reservoir etc.)
- Design of superstructure (beam, column, slab, girder, stair etc.)

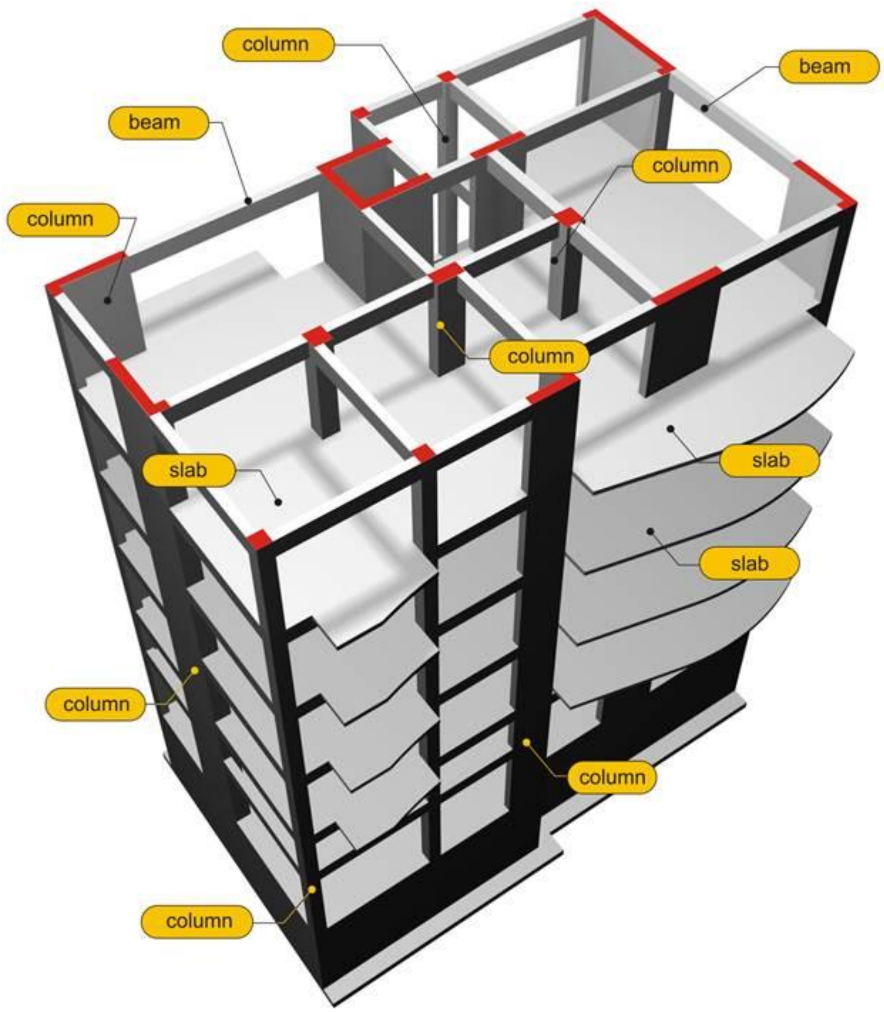


Figure 1: Super structural elements

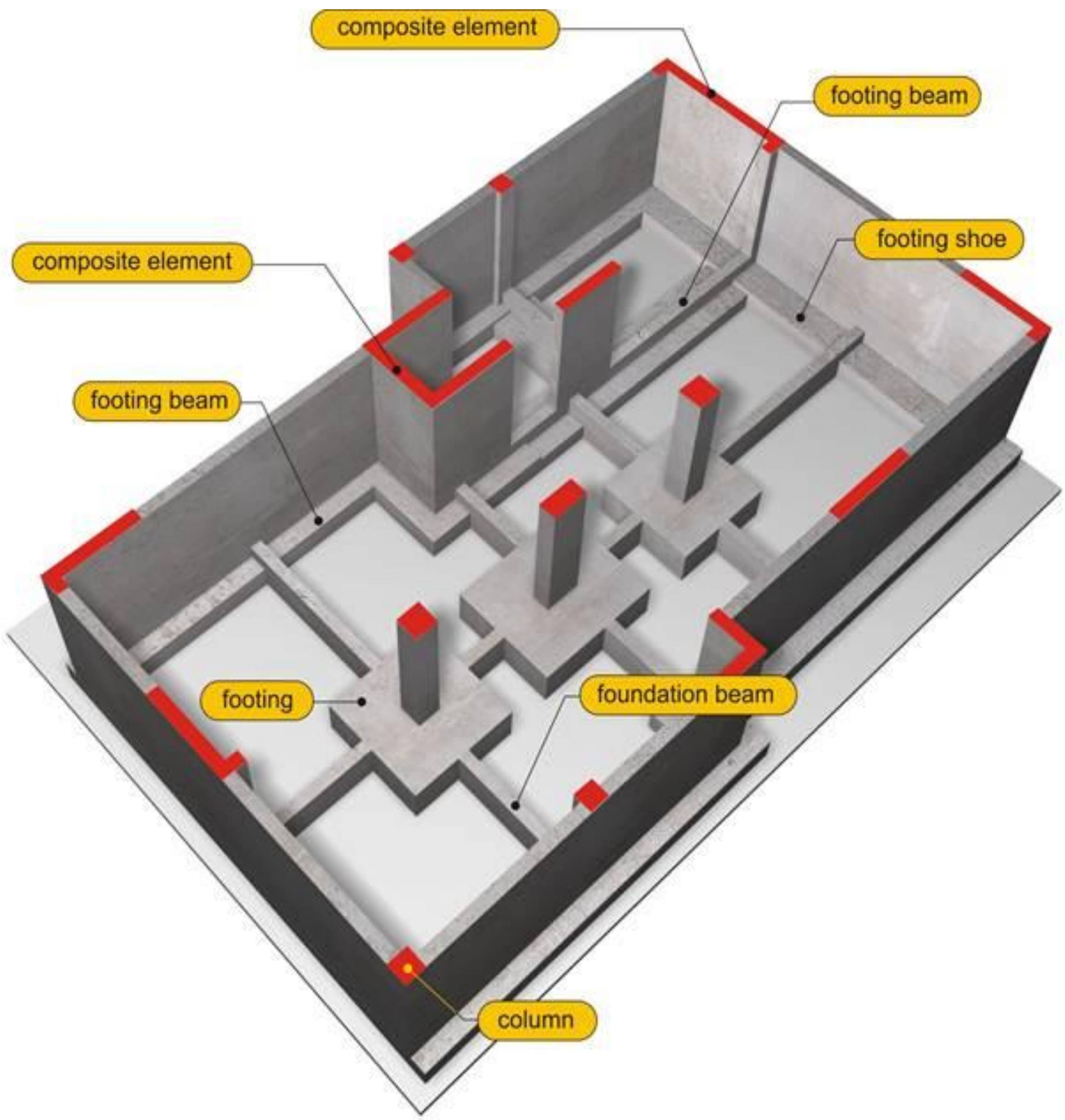


Figure 2: Foundation elements

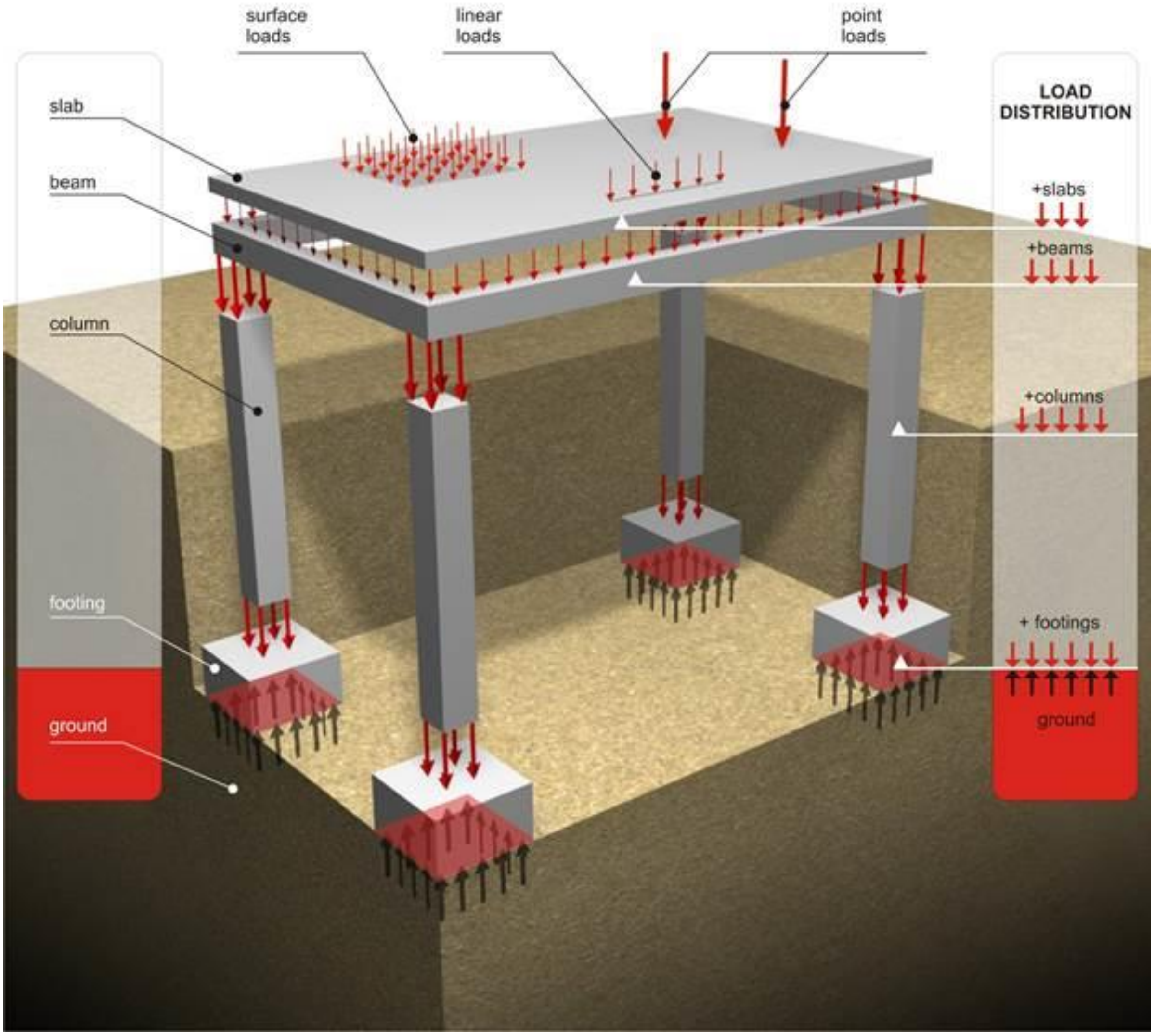


Figure 3: Gravity load distribution

Steps of design

- Specify the type of structural system like RCC or Steel or Composite, beam supported or flat plate or braced etc.
- Specify the loads based on the type of services, like residential or commercial or institutional etc. from codes and judgments.
- Prepare a preliminary model of the structure based on preliminary calculations and judgment.
- Analyze the model for desired load combinations for Strength Design Methods according to BNBC 2020 in the context of Bangladesh,

1. $1.4(D + F)$
2. $1.2(D + F + T) + 1.6(L + H) + 0.5(L_r \; or \; R)$
3. $1.2D + 1.6(L_r \text{ or } R) + (L \text{ or } 0.8W)$
4. $1.2D + 1.6W + L + 0.5(L_r \text{ or } R)$
5. $1.2D + 1.0E + 1.0L$
6. $0.9D + 1.6W + 1.6H$
7. $0.9D + 1.0E + 1.6H$

1.2 Notations

U.S.D Method

f'_c = Cylindrical strength of concrete

f_y = Yield strength of reinforcement

V_c = Allowable shear force without web reinforcement = $2\lambda\sqrt{f'c}\,b_w d$

V = Allowable shear force with web reinforcement = $8\lambda\sqrt{f'c}\,b_w d$

V = Allowable peripheral shear force in slab and footing without web reinforcement =$4\lambda\sqrt{f'c}\,b_w d$

Strength reduction factors:

Flexure, without axial load = 0.90

Axial compression and axial compression with flexure:

Members with spiral Reinforcement = 0.75

Other reinforcement = 0.65

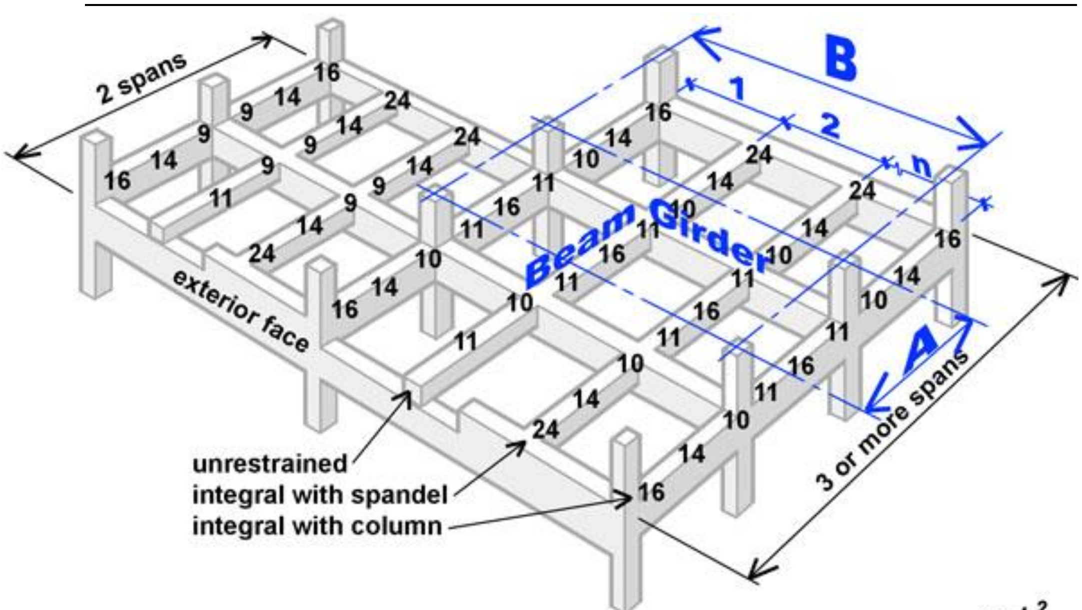
Shear and torsion = 0.75

Bearing on concrete = 0.75

Table 1: Moment and shear values using ACI coefficients. (Ref: ACI Code, Design of Concrete Structure, 15th edition, Chap-11, P-363)

Positive moment	
End spans	
If discontinuous end is unrestrained	$\frac{1}{11} w_u l_n^2$
If discontinuous end is integral with the support	$\frac{1}{14} w_u l_n^2$
Interior spans	$\frac{1}{16} w_u l_n^2$
Negative moment at exterior face of first interior support	
Two spans	$\frac{1}{9} w_u l_n^2$
More than two spans	$\frac{1}{10} w_u l_n^2$
Negative moment at other faces of interior supports	$\frac{1}{11} w_u l_n^2$
Negative moment at face of all supports for (1) slabs with spans not exceeding 10 ft and (2) beams and girders where ratio of sum of column stiffness to beam stiffness exceeds 8 at each end of the span	$\frac{1}{12} w_u l_n^2$
Negative moment at interior faces of exterior supports for members built integrally with their supports	
Where the support is a spandrel beam or girder	$\frac{1}{24} w_u l_n^2$
Where the support is a column	$\frac{1}{16} w_u l_n^2$
Shear in end members at first interior support	$1.15 \frac{w_u l_n}{2}$
Shear at all other supports	$\frac{w_u l_n}{2}$

† w_u = total factored load per unit length of beam or per unit area of slab.
l_n = clear span for positive moment and shear and the average of the two adjacent clear spans for negative moment.



Numbers on beams refer to moment value coefficient, *x*, in the equation: $M_u = \frac{w_u l_n^2}{x}$

The number, **n**, refers to the *number of spaces* into which the girder span is subdivided by beams.

Figure 4: Moment coefficients for beam.

1.3 Design of Stair

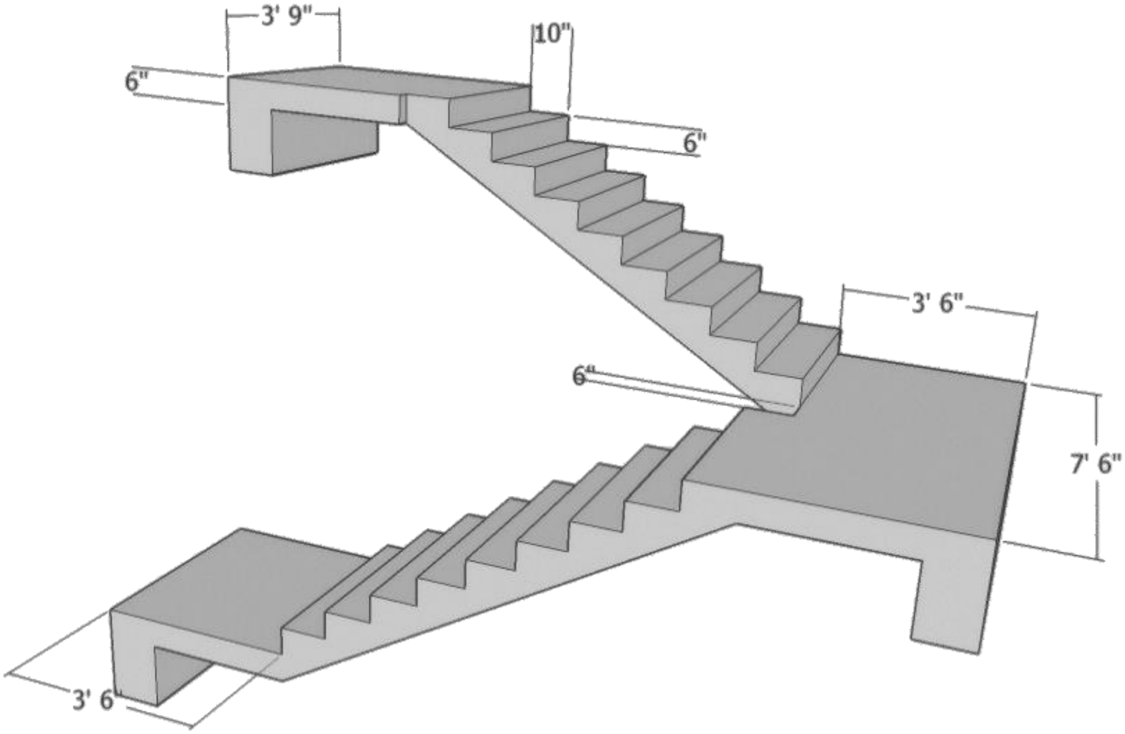


Figure 5: Typical stair

a) Assumptions and considerations

f_y= 60000 psi
f'_c= 3000 psi

Thickness of waist and landing slab = 6"

Live Load=0.1 ksf (BNBC 2020)

Floor Finish= 25 psf = 0.025 ksf

b) Load calculation

Rises & Steps= $\left(\frac{\frac{1}{2}*\frac{6}{12}*\frac{10}{12}*3.5*9*150}{1000}\right)$= 0.98 k

Waist slab =$\left(\frac{\sqrt{7.5^2+4.5^2}*\frac{6}{12}*3.5*150}{1000}\right)$=$\left(\frac{8.75*\frac{6}{12}*3.5*150}{1000}\right)$ =2.3 k

Total Dead Load = Landing slab + (Rises & Steps+ Waist)

$$=\frac{\left(\frac{\frac{6}{12}*150}{1000}\right)+\left(\frac{0.98+2.3}{3.5*7.5}\right)}{2} = 0.1 \text{ ksf}$$

Total load, W= (0.1*1.6) + [1.2*(0.1 +0.025)] =0.31 ksf

c) Moment and reinforcement calculation

$M^{+} = \frac{WL^2}{14} = \frac{0.31 *(2*3.5+7.5)^2}{14} = 4.7$ k-ft/ft

$M^{-} = \frac{WL^2}{9} = \frac{0.31 *14.5^2}{9} = 7.24$ k-ft/ft

d = (t-1) = (6-1) = 5"

$\rho_{0.005} = 0.85 * \beta_1 * \frac{f'c}{fy} * \frac{0.003}{0.003+\epsilon_t} = 0.85 * 0.85 * \frac{3000}{60000} * \frac{0.003}{0.003+0.005} = 0.0135$

$M_u = \phi * \rho_{0.005} * fy * b * d^2 * \left(1 - 0.59 * \frac{\rho_{0.005} * fy}{f'c}\right)$

$d^2 = \frac{7.24*12}{0.9*0.0135*60*12*\left(1-0.59*\frac{0.0135*60}{3}\right)} = \frac{86.9}{8} = 11.28 \text{ in}^2$

d = 3.36"< provided, 5" (ok)

Table 2: Minimum ratios of temperature and shrinkage reinforcement in slabs based on gross concrete area. (Ref: ACI Code, Design of Concrete Structure, 15th edition, Chap-12, P-385)

Slabs where Grade 40 or 50 deformed bars are used	0.0020
Slabs where Grade 60 deformed bars or welded wire fabric (smooth or deformed) are used	0.0018
Slabs where reinforcement with yield strength exceeding 60,000 psi measured at yield strain of 0.35 percent is used	$\frac{0.0018 \times 60{,}000}{f_y}$

$As_{min} = 0.0018 * b * t = 0.0018 * 12 * 6 = 0.129 \text{in.}^2$

$+As = \frac{M*12}{\phi*fy*\left(d-\frac{a}{2}\right)} = \frac{4.7*12}{0.9*60*\left(5-\frac{0.5}{2}\right)} = 0.23 \text{in.}^2/\text{ft}$ (controlled)

$a = \frac{As*fy}{.85*f'c*b} = \frac{0.23*60}{0.85*3*12} = 0.48$ (ok)

Now, $\frac{0.11*12}{0.23} = 5.74$"; use Ø10mm@5.5" c/c alt ckd

Again,

$$\mathbf{-As} = \frac{\mathbf{M*12}}{\phi * \boldsymbol{fy} * \left(\mathbf{d} - \frac{\mathbf{a}}{\mathbf{2}}\right)} = \frac{7.24*12}{0.9*60*\left(5-\frac{0.7}{2}\right)} = 0.34\text{in.}^2/\text{ft}$$ **(controlled)**

$$\mathbf{a} = \frac{\mathbf{As*fy}}{\mathbf{0.85}*\boldsymbol{f'c}*\mathbf{b}} = \frac{0.34*60}{.85*3*12} = 0.68''$$ **(ok)**

The distance between two cranked bars is 11".

So, Required reinforcement = 0.34 - $\frac{0.11*12}{11}$ = 0.22in.^{2}/ft

The extra negative reinforcement required, 11 / ($\frac{0.11*12}{0.22}$) = 11/6 = 1.83 So, use 2-Ø10mm as extra top.

For shrinkage, $As_{min} = 0.0018 * 12 * 6 = 0.129\text{in.}^2$

Now, $\frac{0.11*12}{0.129} = 10.23''$; useØ10mm@10" c/c

d) Stair Beam

Assume beam size, 12"x 12"

d = (t-2.5) = (12-2.5) = 9.5"

So, self-weight = (1*1*150)/1000 = 0.15 k/ft

Load on Stair beam = $\frac{0.31*14.5*3.5}{7.5}$ + (0.42*9*0.12 + 0.15) *1.2 = 2.82 k/ft

The stair beam will be designed as described in the floor beam design segment.

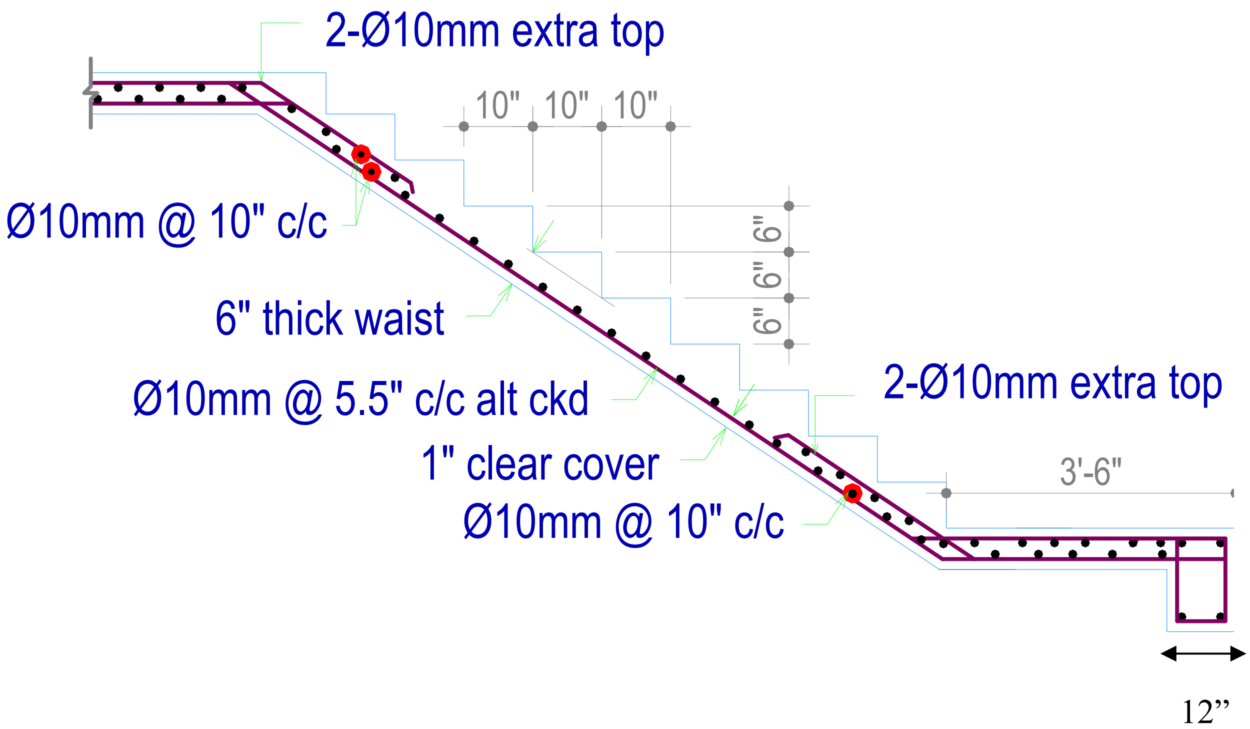


Figure 6: Reinforcement details of stair

1.4 Design of Overhead Water tank (OWR)

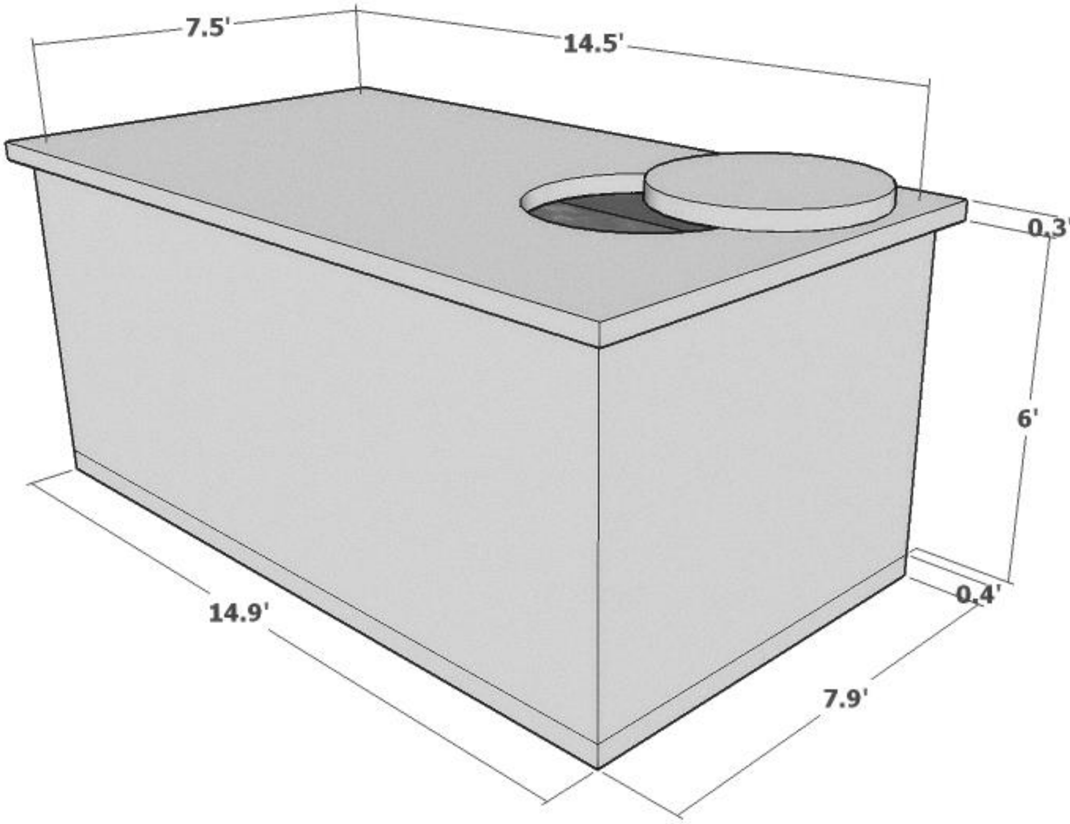


Figure 7: Roof top water reservoir (Overhead water reservoir)

a) Assumptions and considerations

$f'c$= 3000 psi
fy= 60000 psi

6th floor building of 2 units & **6** members in each unit.
Water consumption for **big multi-family apartment/flat** in **city corporation area** considering **full facility** = 200 liter per capita per day (Part VIII, Table 8.5.1 (a), BNBC 2020: Page 4815)

b) Water reservoir size calculation

Total members= 6×2×6= 72 persons.
Total water consuming= 72×200 = 14400 litters for a full day.

$$= \frac{14400}{1000}\,\text{m}^3 = 14.4 \times 3.28^3 = 508.14 \text{ ft}^3$$

Inner length & width of Reservoir are,
Length =14.5 ft and width = 7.5 ft (From plan)
so, Height of the Reservoir =$\frac{508.14}{7.5\times14.5}$ = 4.67 ft + 1 ft = 5.67 ft~ 6 ft; [where, free Board= 1 ft]
Height = 6 ft

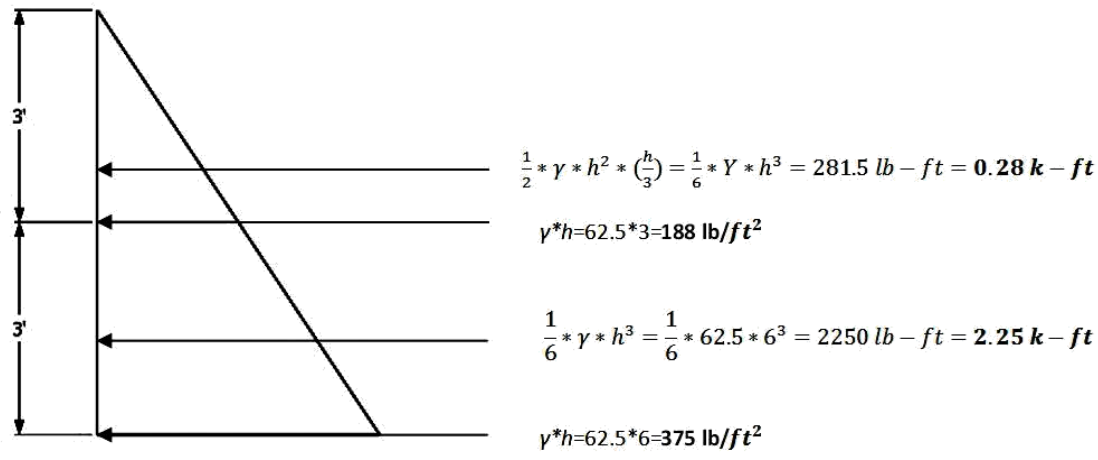


Figure 8: Pressure distribution on reservoir wall

c) Vertical Reinforcement of wall

Let wall thickness = 5"
so, Effective depth, d = 5-1 = 4"

$$\rho_{0.005} = 0.85 \times \beta_1 \times \frac{f_c{}'}{f_y} \times \frac{0.003}{0.003+\epsilon_t} = 0.85 \times 0.85 \times \frac{3000}{60000} \times \frac{0.003}{0.003+0.005} = 0.0135$$

$$M_u = \phi \times \rho_{0.005} \times f_y \times b \times d^2 \times \left(1 - 0.59 \times \frac{\rho_{0.005} \times f_y}{f_c{}'}\right)$$

$$d^2 = \frac{2.25\times12}{0.9\times0.0135\times60\times12\times\left(1-0.59\times\frac{0.0135\times60}{3}\right)} = \frac{27}{7.59} = 3.56\ in^2$$

d = 1.92" < provided, 4" (ok)

$$As_{min} = 0.0018 \times b \times t = 0.0018 \times 12 \times 5 = 0.12\ in^2/ft$$

$$\boldsymbol{A_s} = \frac{M\times12}{\varphi\times f_y\times\left(d-\frac{a}{2}\right)} = \frac{2.25\times12}{0.9\times60\times\left(4-\frac{0.25}{2}\right)} = 0.13\ in^2/ft \text{ (controlled)}$$

$$a = \frac{A_s\times f_y}{0.85\times f_c'\times b} = \frac{0.13\times60}{0.85\times3\times12} = 0.26 \text{ (ok)}$$

Now, Spacing = $\frac{0.11\times12}{0.13}$ = 10.15"; **Use Ø10 mm @10" c/c.**

d) Horizontal reinforcement of wall

Force = $\gamma \times h \times (\frac{14.5}{2}+\frac{14.5}{2})$ = $62.5 \times 6\times (\frac{14.5}{2}+\frac{14.5}{2})$ = 5437.5 lb/ft

Again, $\frac{\text{force}}{\text{stress}} = \frac{5437.5}{f_y} = \frac{5437.5}{60000}$ = 0.091 in^2/ft

As_{min} controls.

Now, spacing = $\frac{0.11\times12}{0.12}$ = 11"; **Use Ø10 mm @ 11" c/c**

e) Design of bottom slab

Table 3: Minimum thickness of non-prestressed one-way slabs. (Ref: ACI Code, Design of Concrete Structure, 15th edition, Chap-12, P-384)

Element	Simply Supported	One end continuous	Both ends continuous	Cantilever
One-way solid slabs	*l/20*	*l/24*	*l/28*	*l/10*

Here, l is the clear span

Multiplying factor = $0.4+\frac{f_y}{100}$, f_y in ksi

If,

Thickness < 6 inch then upper rounding to neatest 0.25
Thickness ≥ 6 inch then upper rounding to neatest 0.50

Thickness = $\frac{7.5}{20} \times 12$ = 4.5 in

Self-weight of slab = (4.5/12) × 150 = 56.25 psf

$$\frac{5wl_A^4}{384\ \text{EI}} = \frac{5\text{w}_\text{B}l_B^4}{384\ \text{EI}}$$

$$w_Al_A^4 = \text{w}_\text{B}l_B^4$$

$$\text{w}_\text{A} = \text{w}_\text{B}\left(\frac{l_B}{l_A}\right)^4$$

$$w_A = w_B \times \left(\frac{14.5}{7.5}\right)^4 = 13.97\ w_B$$

$\text{w}_\text{A} + \text{w}_\text{B}$ = 56.25 psf

w_B = 3.76 psf

w_A = 52.49 psf

Floor Finish = 25 psf = 0.025 ksf

As the slab is one-way slab, design only for short direction.

Total load, w = [0.0625 × 6 × 1.6] + [1.2 × (0.05249 + 0.025)]= 0.693 ksf

<u>Moment for short direction</u>

$M^{+} = \frac{wL^2}{14} = \frac{0.69 \times 7.5^2}{14} = 2.77$ k-ft/ft

$M^{-} = \frac{wL^2}{24} = \frac{0.69 \times 7.5^2}{24} = 1.62$ k-ft/ft

$$M_u = \phi \times \rho_{0.005} \times f_y \times b \times d^2 \times \left(1 - 0.59 \times \frac{\rho_{0.005} \times f_y}{f_c{}'}\right)$$

$$d^2 = \frac{2.77 \times 12}{0.9 \times 0.0135 \times 60 \times 12 \times \left(1 - 0.59 \times \frac{0.0135 \times 60}{3}\right)} = 4.52$$

d = 2.13" < provided, 3.5" (ok)

As_{min} = 0.0018 × b × t = 0.0018×12×4.5 = 0.1 in^2/ft

$$+A_s = \frac{M \times 12}{\phi \times f_y \times \left(d - \frac{a}{2}\right)} = \frac{2.77 \times 12}{0.9 \times 60 \times \left(3.5 - \frac{0.36}{2}\right)} = 0.185\ in^2/ft$$

$$a = \frac{A_s \times f_y}{0.85 \times f_c' \times b} = \frac{0.185 \times 60}{0.85 \times 3 \times 12} = 0.36\ \text{(ok)}$$

Now, $\frac{0.11 \times 12}{0.185} = 7.135"$;

Similarly Calculate the negative reinforcement required for the negative moment of 1.62 k-ft/ft.

Use Ø10 mm @ 7" c/c alt. ckd and 1- Ø10 mm as extra top.

Draw the reinforcement detailing of bottom slab (follow figure 9).

f) Top slab

For the top slab there is no water load and some live load which is negligible. As the bottom slab is controlled by 4.5" thickness, top slab will be governed by a thickness of 4.5" and $As_{min.}$

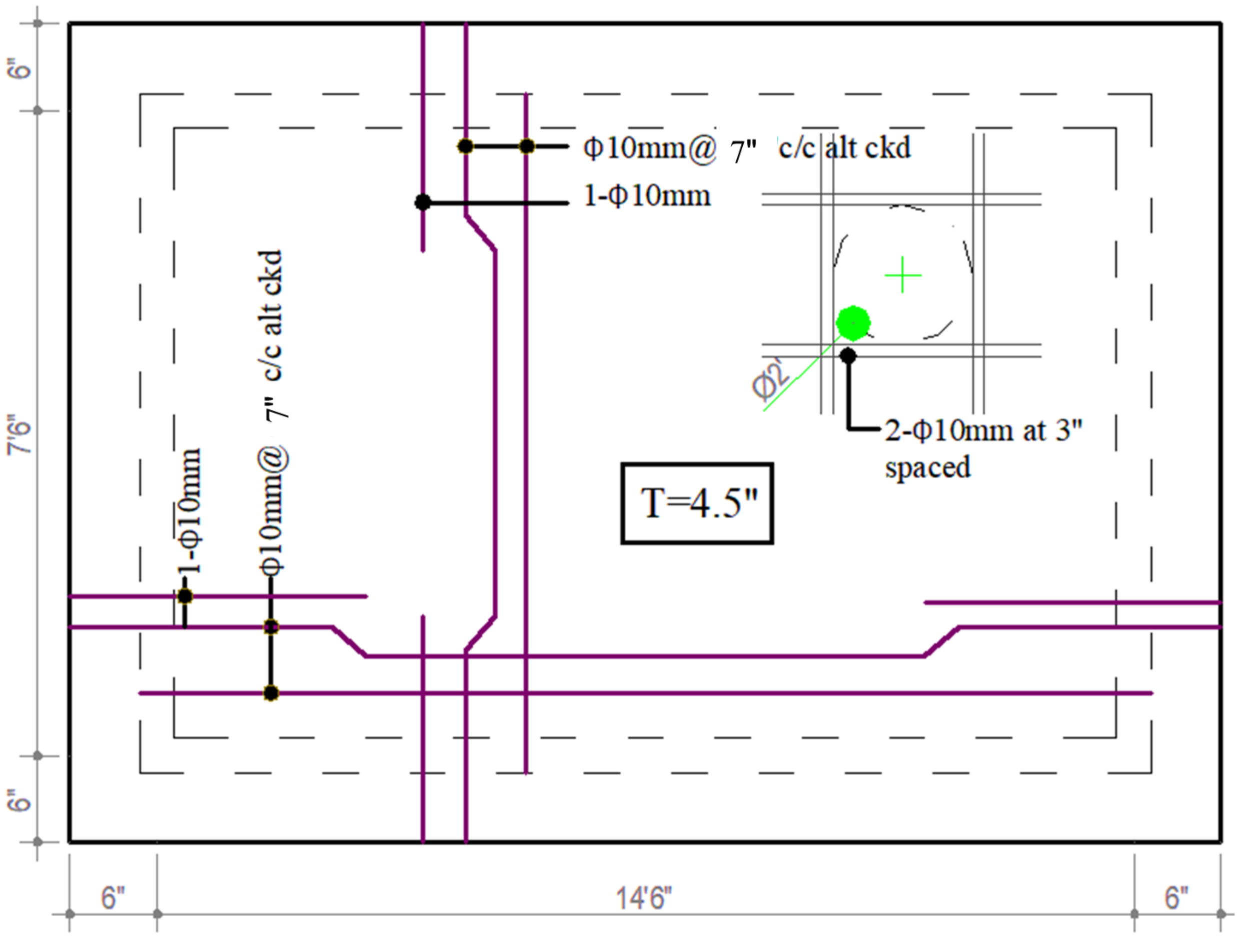


Figure 9: Reinforcement details of top slab overhead water reservoir

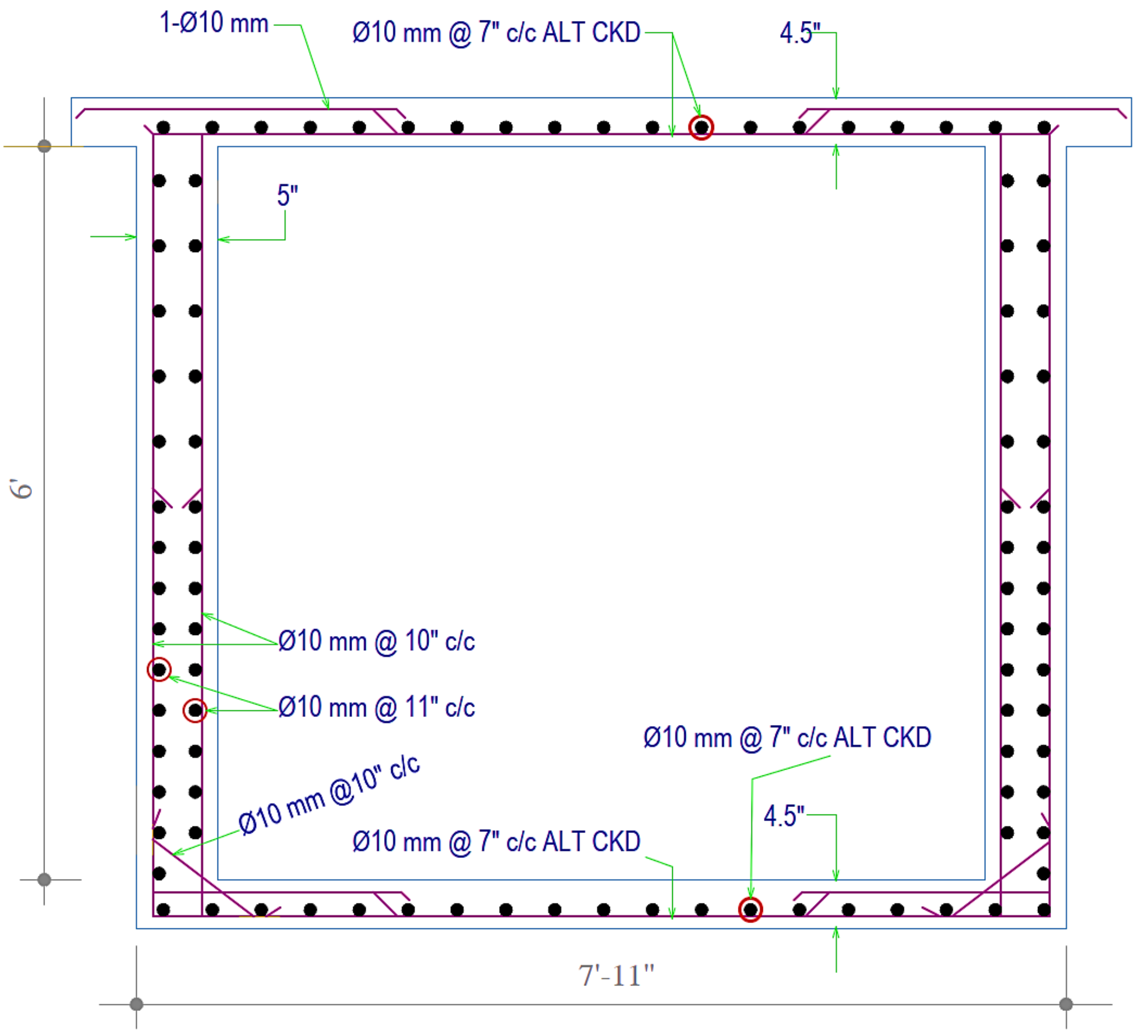


Figure 10: Reinforcement details of roof top water reservoir (elevation view)

g) Load on beam

Here, Load from Bottom Slab = 0.69 ksf

Beam Thickness, **t** = 12 in

Effective Depth, **d** = (12-2.5) = 9.5 in

Self-weight = 0.83× 1×150 = 0.12 k/ft

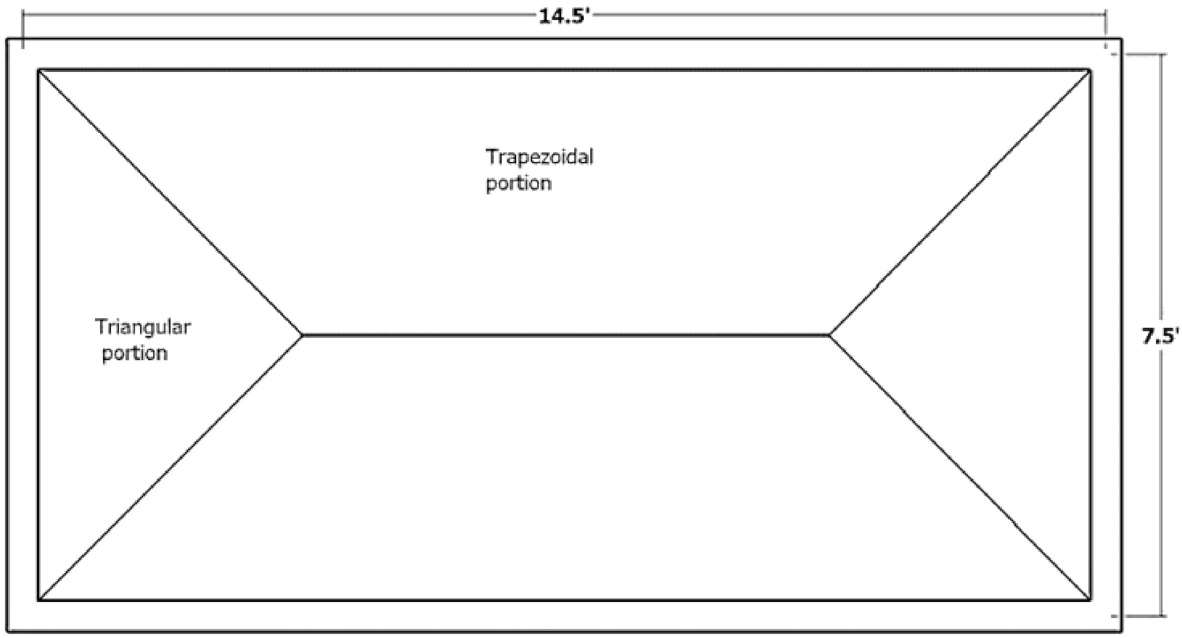


Figure 11: Load distribution of slab

Trapezoidal portion,

$$= \frac{\frac{1}{2}\times(14.5+7)\times3.75\times(0.69+0.05625\times1.2)}{14.5} + 0.12 \times 1.2 + (0.42 \times 6 \times 0.15) \times 1.2 = 2.63 \text{ k/ft}$$

The beam will be designed as discussed in the floor beam design segment.

1.5 Design of Floor Slabs

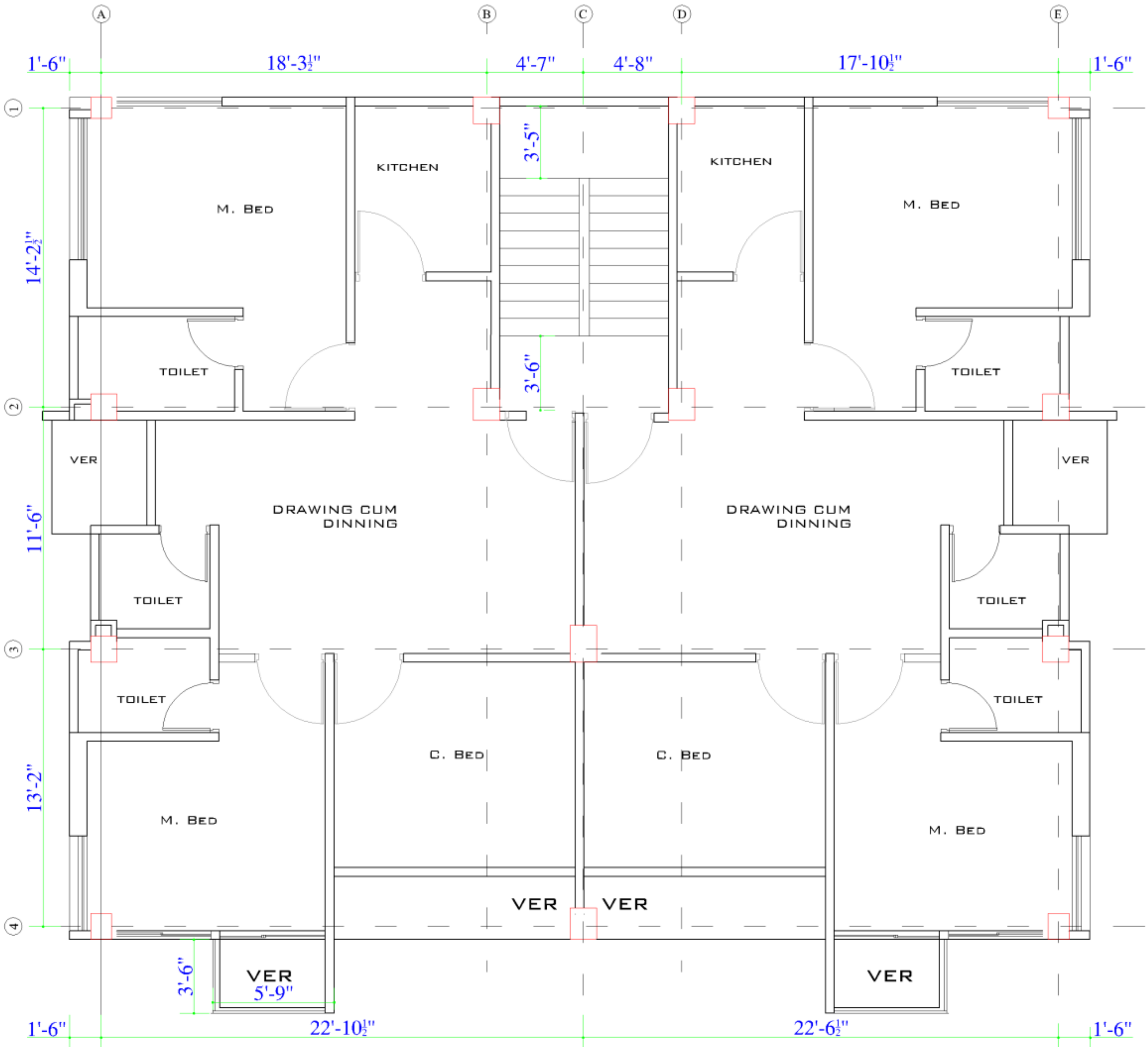


Figure 14: Typical floor plan

a) Assumptions and considerations

f'_c=3000 psi
f_y= 60000 psi

Thickness, t = $\frac{longlength\,(0.8+\frac{fy}{200000})}{36+9\beta}$ and t = Periphery / 180

Considering the largest two panels of 22'-10" ×13'-2" and 22'-10" ×11'-6

So, $\beta = \frac{22.83}{13.17} = 1.73$

Thickness, t = 4.8 in. ≈5.5 in. (considering the serviceability of the residential building)

b) Load calculation

Self-weight of slab = $\frac{5.5}{12} \times 150 = 69$ psf

Floor finish = 30 psf

Partition wall = 40 psf

Live Load = 42 psf (BNBC 2020, Table 6.2.3, Residential, All other areas except stairs and balconies)

$W_{DL} = 1.2(69 + 30 + 40) = 1.2 \times 139\, psf = 166.8\, psf$

$W_{LL} = 42\, psf \times 1.6 = 67.2\, psf$

Total, W = (166.8+67.2) = 234 psf

m = $\frac{13.17}{22.83}$ = 0.58 ~ 0.6 and case 4

m = $\frac{11.5}{22.83}$ = 0.5and case 9

Table 7: Moment coefficients for two-way slabs. (Ref: BNBC 2020)

Conditions	Case 4	Case 9
$-C_A$	0.089	0.088
$-C_B$	0.011	0.003
$+C_{A(DL)}$	0.053	0.038
$+C_{B(DL)}$	0.007	0.002
$+C_{A(LL)}$	0.067	0.067
$+C_{B(LL)}$	0.009	0.004

From judgment it can be said that the slab will be critical in short direction only.

c) Moment and reinforcement calculation

For, case 4

Short distance A, +M= $\{C_{A(DL)} * W_{(DL)} * A^2\} + \{C_{A(LL)} * W_{(LL)} * A^2\}$ = 2.31 $k - ft/ft$

short distance A, -M = $\{-C_A * W * A^2\}$ = 3.61 $k - ft/ft$

For, case 9

Short distance A, +M= $\{C_{A(DL)} * W_{(DL)} * A^2\} + \{C_{A(LL)} * W_{(LL)} * A^2\}$ = 1.43 $k - ft/ft$

short distance A, -M = $\{-C_A * W * A^2\}$ = 2.72 $k - ft/ft$

So, in short direction –M = 3.61 $k - ft/ft$ and +M = 2.31 $k - ft/ft$

$\boldsymbol{As_{min}} = 0.0018 * b * t = 0.0018 * 12 * 5.5 = 0.119\ in^2/ft$

$+As = \frac{M*12}{\varphi*fy*(d-\frac{a}{2})} = \frac{2.31*12}{0.9*60*(4.5-\frac{0.23}{2})} = 0.12\ in^2/ft\ (controlled)$

$a = \frac{As*fy}{0.85*f'_c*b} = \frac{0.12*60}{0.85*3*12} = 0.23\ in$

Now,$\frac{0.11*12}{0.12} = 10.15$"; use, ϕ 10 mm@ 10" c/c alt. ckd.

Again,

$-As = \frac{M*12}{\varphi*fy*(d-\frac{a}{2})} = \frac{3.61*12}{0.9*60*(4.5-\frac{0.36}{2})} = 0.186\ in^2/ft\ (controlled)$

$a = \frac{As*fy}{0.85*f'_c*b} = \frac{0.186*60}{0.85*3*12} = 0.36\ in$

The distance between two cranked rods is 20".

So, Required reinforcement = 0.186 - $\frac{0.11*12}{20} = 0.12$ in^2/ft

The extra negative reinforcement required, 20 / ($\frac{0.11*12}{0.12}$) = 1.81 ~ 2 So, use 2-Ø10 mm as extra top.

By observing the moment coefficients, it can be said that all the reinforcement in long direction will be controlled by As_{min}

So, the reinforcement will be ϕ 10 mm@ 10" c/c alt. ckd and 2-Ø10 mm as extra top.

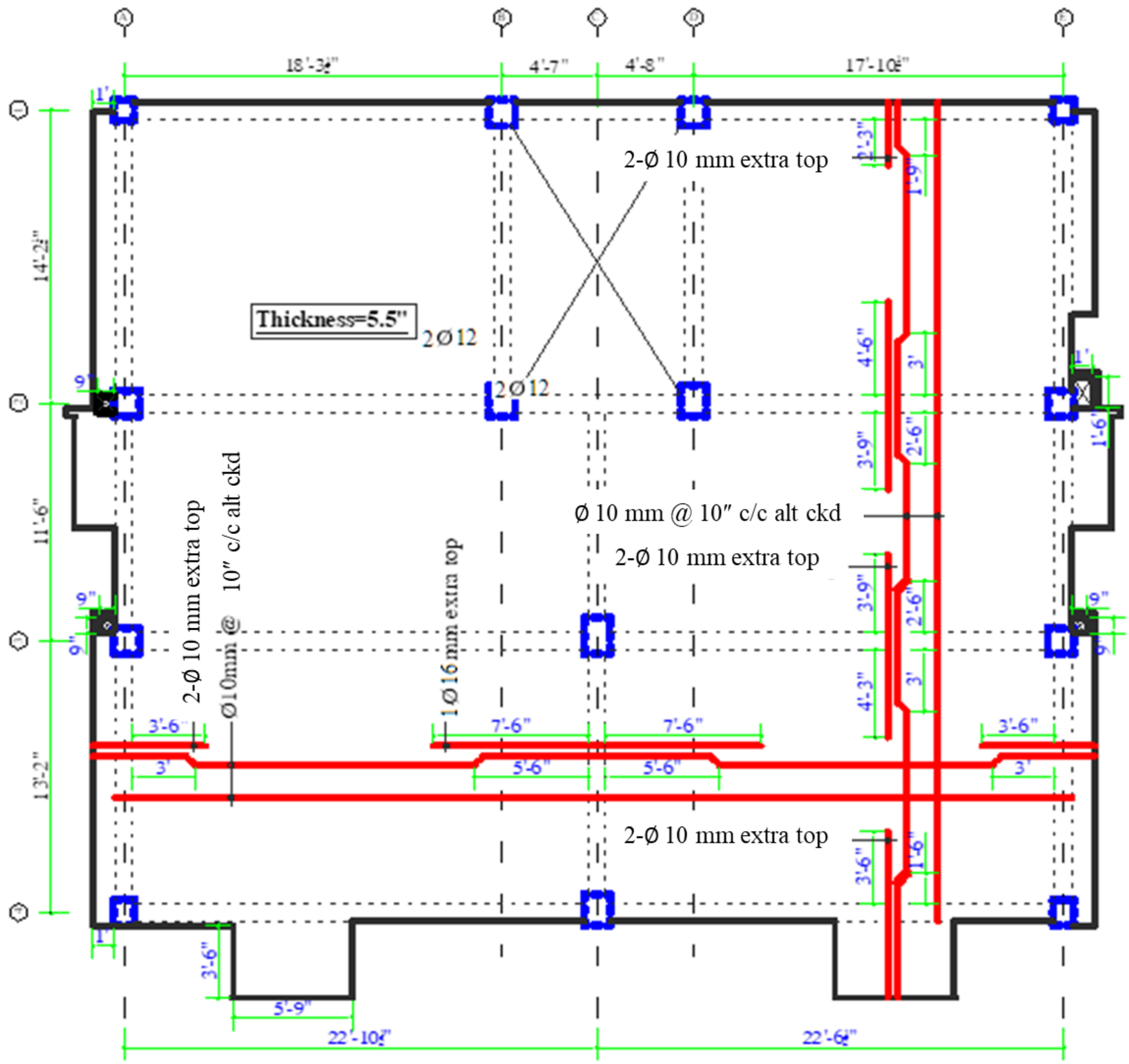


Figure 15: Typical Reinforcement Details of Slab

****Note:** Students have to draw the reinforcement detailing as per their calculated values.

1.6 Lateral Loads Calculation of Residential Building

Calculation of Seismic Load

Steps to be followed to calculate seismic load:

Step 01: Calculation of Site Classification

Step 02: Calculation of structure (building) natural period (Page 3208)

Step 03: Calculation of zone coefficient and importance factor

Step 04: Calculation of seismic design category and response reduction factor

Step 05: Determination of Soil factor (S) and other parameters (T_A, T_B etc.)

Step 06: Calculation of normalized acceleration response spectrum

Step 07: Calculation of design spectral acceleration or lateral seismic force coefficient

Step 08: Calculation of total seismic load

Step 09: Calculation of seismic design base shear

Step 10: Calculation of vertical distribution of lateral force

Site Classification:

$$\bar{V}_s = \sum_{i=1}^{n} d_i \Big/ \sum_{i=1}^{n} \frac{d_i}{V_{si}} \qquad (6.2.31)$$

$$\bar{N} = \sum_{i=1}^{n} d_i \Big/ \sum_{i=1}^{n} \frac{d_i}{N_i} \qquad (6.2.32)$$

$$\bar{S}_u = \sum_{i=1}^{k} d_{ci} \Big/ \sum_{i=1}^{k} \frac{d_{ci}}{S_{ui}} \qquad (6.2.33)$$

(BNBC 2020, Part 6 Chapter 2 (Page 3189)

n = Number of soil layers in upper 30 m

d_i = Thickness of layer i

V_{si} = Shear wave velocity of layer i

N_i = Field (uncorrected) Standard Penetration Value for layer i

k = Number of cohesive soil layers in upper 30 m

d_{ci} = Thickness of cohesive layer i

s_{ui} = Undrained shear strength of cohesive layer i

(BNBC 2020, Part 6 Chapter 2 (Page 3190 to 3191)

Table 6.2.13: Site Classification Based on Soil Properties

Site Class	Description of soil profile up to 30 meters depth	Average Soil Properties in top 30 meters		
		Shear wave velocity, $\overline{V}_s$ (m/s)	SPT Value, $\overline{N}$ (blows/30cm)	Undrained shear strength, $\overline{S}_u$ (kPa)
SA	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	> 800	--	--
SB	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of metres in thickness, characterised by a gradual increase of mechanical properties with depth.	360 – 800	> 50	> 250
SC	Deep deposits of dense or medium dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres.	180 – 360	15 - 50	70 - 250
SD	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	< 180	< 15	< 70

Site Class	Description of soil profile up to 30 meters depth	Average Soil Properties in top 30 meters		
		Shear wave velocity, $\bar{V}_s$ (m/s)	SPT Value, $\bar{N}$ (blows/30cm)	Undrained shear strength, $\bar{S}_u$ (kPa)
SE	A soil profile consisting of a surface alluvium layer with V_s values of type SC or SD and thickness varying between about 5 m and 20 m, underlain by stiffer material with V_s > 800 m/s.	--	--	--
S_1	Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index (PI > 40) and high water content	< 100 (indicative)	--	10 - 20
S_2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types SA to SE or S_1	--	--	--

Natural Period, $T = C_t(h_n)^m$ **(Eqn. 6.2.38)**

h_n = Height of building in metres from foundation or from top of rigid basement. This excludes the basement storeys, where basement walls are connected with the ground floor deck or fitted between the building columns. But it includes the basement storeys, when they are not so connected.

Table 6.2.20: Values for Coefficients to Estimate Approximate Period

	h_n in meter		h_n in feet	
Structure type	C_t	m	C_t	m
Concrete moment-resisting frames	0.0466	0.9	0.016	0.9
Steel moment-resisting frames	0.0724	0.8	0.028	0.8
Eccentrically braced steel frame	0.0731	0.75	0.03	0.75
All other structural systems	0.0488	0.75	0.02	0.75

Table 6.2.16: Site Dependent Soil Factor and Other Parameters Defining Elastic Response Spectrum

Soil Type	S	T_B(s)	T_C(s)	T_D(s)
SA	1	0.15	0.4	2
SB	1.2	0.15	0.5	2
SC	1.15	0.2	0.6	2
SD	1.35	0.2	0.8	2
SE	1.4	0.15	0.5	2

T = Structure (building) period as defined in Sec 2.5.7.2

T_B = Lower limit of the period of the constant spectral acceleration branch given in Table 6.2.16 as a function of site class.

T_C = Upper limit of the period of the constant spectral acceleration branch given in Table 6.2.16 as a function of site class

T_D = Lower limit of the period of the constant spectral displacement branch given in Table 6.2.16 as a function of site class

$$C_s = S\left(1 + \frac{T}{T_B}(2.5\eta - 1)\right) \quad \text{for} \quad 0 \le T \le T_B \qquad (6.2.35a)$$

$$C_s = 2.5 S\eta \quad \text{for} \quad T_B \le T \le T_C \qquad (6.2.35b)$$

$$C_s = 2.5 S\eta\left(\frac{T_C}{T}\right) \quad \text{for} \quad T_C \le T \le T_D \qquad (6.2.35c)$$

$$C_s = 2.5 S\eta\left(\frac{T_C T_D}{T^2}\right) \quad \text{for} \quad T_D \le T \le 4\ sec \qquad (6.2.35d)$$

$$\eta = \sqrt{10/(5+\xi)} \ge 0.55 \qquad (6.2.36)$$

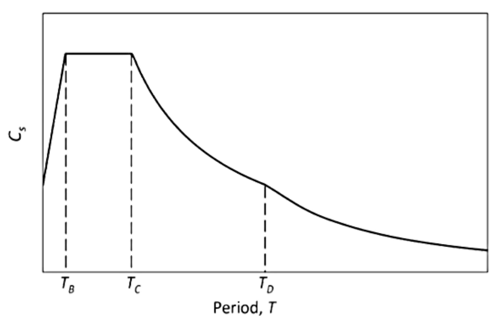


Figure 6.2.25 Typical shape of the elastic response spectrum coefficient C_s

η = Damping correction factor as a function of damping with a reference value of η = 1 for 5% viscous damping

ξ = viscous damping ratio of the structure, expressed as a percentage of critical damping.

Table 6.2.19: Response Reduction Factor, Deflection Amplification Factor and Height Limitations for Different Structural Systems

Seismic Force–Resisting System	Response Reduction Factor, R	System Overstrength Factor, Ω_o	Deflection Amplification Factor, C_d	Seismic Design Category B	Seismic Design Category C	Seismic Design Category D
				Height limit (m)		
A. BEARING WALL SYSTEMS (no frame)						
1. Special reinforced concrete shear walls	5	2.5	5	NL	NL	50
2. Ordinary reinforced concrete shear walls	4	2.5	4	NL	NL	NP
3. Ordinary reinforced masonry shear walls	2	2.5	1.75	NL	50	NP
4. Ordinary plain masonry shear walls	1.5	2.5	1.25	18	NP	NP
B. BUILDING FRAME SYSTEMS (with bracing or shear wall)						
5. Special reinforced concrete shear walls	6	2.5	5	NL	NL	50
6. Ordinary reinforced concrete shear walls	5	2.5	4.25	NL	NL	NP
7. Ordinary reinforced masonry shear walls	2	2.5	2	NL	50	NP
8. Ordinary plain masonry shear walls	1.5	2.5	1.25	18	NP	NP

Seismic Force–Resisting System	Response Reduction Factor, R	System Overstrength Factor, Ω_o	Deflection Amplification Factor, C_d	Seismic Design Category B	Seismic Design Category C	Seismic Design Category D
				Height limit (m)		
C. MOMENT RESISTING FRAME SYSTEMS (no shear wall)						
4. Special reinforced concrete moment frames	8	3	5.5	NL	NL	NL
5. Intermediate reinforced concrete moment frames	5	3	4.5	NL	NL	NP
6. Ordinary reinforced concrete moment frames	3	3	2.5	NL	NP	NP
D. DUAL SYSTEMS: SPECIAL MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES (with bracing or shear wall)						
3. Special reinforced concrete shear walls	7	2.5	5.5	NL	NL	NL
4. Ordinary reinforced concrete shear walls	6	2.5	5	NL	NL	NP

E. DUAL SYSTEMS: INTERMEDIATE MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES (with bracing or shear wall)						
3. Ordinary reinforced masonry shear walls	3	3	3	NL	50	NP
4. Ordinary reinforced concrete shear walls	5.5	2.5	4.5	NL	NL	NP
F. DUAL SHEAR WALL-FRAME SYSTEM: ORDINARY REINFORCED CONCRETE MOMENT FRAMES AND ORDINARY REINFORCED CONCRETE SHEAR WALLS	4.5	2.5	4	NL	NP	NP

Table 6.2.14: Description of Seismic Zones

Seismic Zone	Location	Seismic Intensity	Seismic Zone Coefficient, Z
1	Southwestern part including Barisal, Khulna, Jessore, Rajshahi	Low	0.12
2	Lower Central and Northwestern part including Noakhali, Dhaka, Pabna, Dinajpur, as well as Southwestern corner including Sundarbans	Moderate	0.20
3	Upper Central and Northwestern part including Brahmanbaria, Sirajganj, Rangpur	Severe	0.28
4	Northeastern part including Sylhet, Mymensingh, Kurigram	Very Severe	0.36

Table 6.2.15: Seismic Zone Coefficient Z for Some Important Towns of Bangladesh

Town	Z	Town	Z	Town	Z
Bagerhat	0.12	Jamalpur	0.36	Patuakhali	0.12
Bandarban	0.28	Jessore	0.12	Pirojpur	0.12
Barguna	0.12	Jhalokati	0.12	Rajbari	0.20
Barisal	0.12	Jhenaidah	0.12	Rajshahi	0.12
Bhola	0.12	Khagrachari	0.28	Rangamati	0.28
Bogra	0.28	Khulna	0.12	Rangpur	0.28
Brahmanbaria	0.28	Kishoreganj	0.36	Satkhira	0.12
Chandpur	0.20	Kurigram	0.36	Shariatpur	0.20
Chapainababganj	0.12	Kushtia	0.20	Sherpur	0.36
Chittagong	0.28	Lakshmipur	0.20	Sirajganj	0.28
Chuadanga	0.12	Lalmanirhat	0.28	Srimangal	0.36

Town	Z	Town	Z	Town	Z
Comilla	0.20	Madaripur	0.20	Sunamganj	0.36
Cox's Bazar	0.28	Magura	0.12	Sylhet	0.36
Dhaka	0.20	Manikganj	0.20	Tangail	0.28
Dinajpur	0.20	Maulvibazar	0.36	Thakurgaon	0.20
Faridpur	0.20	Meherpur	0.12	Natore	0.20
Feni	0.20	Mongla	0.12	Naogaon	0.20
Gaibandha	0.28	Munshiganj	0.20	Netrakona	0.36
Gazipur	0.20	Mymensingh	0.36	Nilphamari	0.12
Gopalganj	0.12	Narail	0.12	Noakhali	0.20
Habiganj	0.36	Narayanganj	0.20	Pabna	0.20
Jaipurhat	0.20	Narsingdi	0.28	Panchagarh	0.20

Occupancy Category (Summarized)

Nature of Occupancy	**Occupancy Category**
Buildings and other structures that represent a low hazard to human life in the event of failure	I
All buildings and other structures except those listed in Occupancy Categories I, III, and IV	II
Buildings and other structures that represent a substantial hazard to human life in the event of failure, including, but not limited to: • Where more than 300 people congregate in one area • Daycare facilities with a capacity greater than 150 • School facilities with a capacity greater than 250 • Colleges or adult education facilities having more than 500 students. • Health care facilities with a capacity of 50 or more resident patients but nor surgery facility. • Jails and detention facilities	III
Buildings and other structures designated as essential facilities, including, but not limited to: • Hospitals and other health care facilities having surgery or emergency treatment facilities • Fire, rescue, ambulance, and police stations and emergency vehicle garages • Designated earthquake, hurricane, or other emergency shelters	IV

Nature of Occupancy	**Occupancy Category**
• Designated emergency preparedness, communication, and operation centers and other facilities required for emergency response • Power generating stations and other public utility facilities required in an emergency • Ancillary structures (including, but not limited to, communication towers, fuel storage tanks, cooling towers, electrical substation structures, fire water storage tanks or other structures housing or supporting water, or other fire-suppression material or equipment) required for operation of Occupancy Category IV structures during an emergency	

Table 6.2.17: Importance Factors for Buildings and Structures for Earthquake design

Occupancy Category	Importance factor, I
I, II	1
III	1.25
IV	1.5


Bangladesh
Seismic Zones
INDIA
INDIA
INDIA
Panchagarh
Thakurgaon
Nilphamari
Kurigram
Dinajpur
Rangpur
Gaibandha
Joypurhat
Naogaon
Bogra
Sherpur
Jamalpur
Netrakona
Sunamganj
Sylhet
Nawabganj
Mymensingh
Z = 0.28
Z = 0.36
Rajshahi
Maulvibazar
Natore
Sirajganj
Tangail
Kishorgonj
Habiganj
Pabna
Gazipur
Narsingdi
Brahmanbaria
Meherpur
Kushtia
Manikganj
Dhaka
Narayanganj
Munshiganj
Chuadanga
Rajbari
Z = 0.20
Jhenaidah
Magura
Faridpur
Madaripur
Comilla
Khagrachhari
Narail
Gopalganj
Jessore
Feni
Z = 0.12
Barisal
Lakshmipur
Rangamati
Noakhali
Satkhira
Pirojpur
Jhalokati
Chittagong
Chittagong
Khulna
Patuakhali
Bhola
Z = 0.28
Bagerhat
Barguna
Z = 0.20
Bandarban
Bay of Bengal
Cox's Bazar

Table 6.2.18: Seismic Design Category of Buildings

Site Class	Occupancy Category I, II and III				Occupancy Category IV			
	Zone 1	Zone 2	Zone 3	Zone 4	Zone 1	Zone 2	Zone 3	Zone 4
SA	B	C	C	D	C	D	D	D
SB	B	C	D	D	C	D	D	D
SC	B	C	D	D	C	D	D	D
SD	C	D	D	D	D	D	D	D
SE, S1, S2	D	D	D	D	D	D	D	D

Buildings shall be assigned a seismic design category among B, C or D based on seismic zone, local site conditions and importance class of building, as given in Table 6.2.18. Seismic design category D has the most stringent seismic design detailing, while seismic design category B has the least seismic design detailing requirements.

Spectral Acceleration, $S_a = \frac{2}{3}\frac{ZI}{R}C_s$ Equation 6.2.34

S_a = Design spectral acceleration (in units of g) which shall not be less than ***0.67βZIS***

Note: The minimum value of S_a ***should not be less than 0.044*** $S_{DS}I$***. The values of*** S_{DS} ***are provided in Table 6.C.4 of Appendix C.***

β = Coefficient used to calculate lower bound for S_a. Recommended value is 0.11

C_s = Normalized acceleration response spectrum, which is a function of structure (building) period and soil type (site class)

Table 6.C.4: Spectral Response Acceleration Parameter S_{DS} for Different Seismic Zone and Soil Type

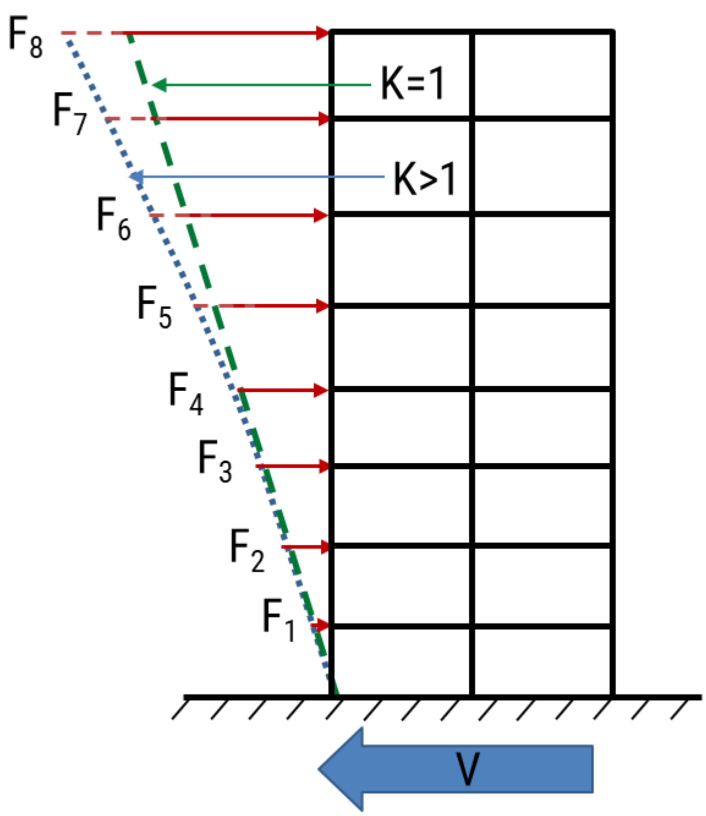
Site Class	Zone 1	Zone 2	Zone 3	Zone 4
SA	0.2	0.333	0.466	0.6
SB	0.24	0.4	0.56	0.72
SC	0.23	0.383	0.536	0.69
SD	0.27	0.45	0.63	0.81
SE, S1, S2	0.28	0.466	0.653	0.84

Design Base Shear, $V = S_a W$ Equation 6.2.37

W = Total seismic weight of the building defined in Sec 2.5.7.3

Vertical distribution of lateral forces, $F_x = V \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$ Equation 6.2.41

F_x = Part of base shear induced at level x.
w_i and w_x = Part of the total effective seismic weight of the structure (W) assigned to level i or x
h_i and h_x = Height from the base to level i or x
n = Number of stories
k = 1 for structure period ≤ 0.5
= 2 for structure period ≥ 2.5s
= linear interpolation between 1 and 2 for other periods.



Design Example:

Calculate the vertical distribution of earthquake forces on a 45 m high 15-storied residential building with story height 3 m each and located in Dhaka town. The concrete building has a rectangular plan 30 m × 60 m and the basic structural system is developed. Dead load including partitions = 9 kN/m^2 (each floor), live load = 3 kN/m^2 (each floor) and viscous damping ratio of the structure = 5.

Depth (m)	SPT-N	Depth (m)	SPT-N	Depth (m)	SPT-N
0	0	10.5	16	21	50
1.5	5	12	29	22.5	50
3	7	13.5	32	24	50
4.5	10	15	38	25.5	50
6	12	16.5	46	27	50
7.5	13	18	50	28.5	50
9	10	19.5	50	30	50

Solution:

Step 01: Calculation of Site Classification

Depth (m)	SPT-N	d (m)	d/SPT-N
0	0	0	-
1.5	5	1.5	0.3
3	7	1.5	0.214286
4.5	10	1.5	0.15
6	12	1.5	0.125
7.5	13	1.5	0.115385
9	10	1.5	0.15
10.5	16	1.5	0.09375
12	29	1.5	0.051724
13.5	32	1.5	0.046875
15	38	1.5	0.039474
16.5	46	1.5	0.032609
18	50	1.5	0.03
19.5	50	1.5	0.03
21	50	1.5	0.03
22.5	50	1.5	0.03
24	50	1.5	0.03
25.5	50	1.5	0.03
27	50	1.5	0.03
28.5	50	1.5	0.03
30	50	1.5	0.03
Total		30	1.589102

$SPT - N(avg) = \frac{\Sigma d}{\Sigma(\frac{d}{SPT-N})} = \frac{30}{1.59} \quad = 18.88$

From Table 6.2.13, we get Site Classification = SC

Step 02: Calculation of structure (building) natural period (Page 3208)

From Table 6.2.20 (Page 3209) we get, C_t = 0.0466 and m = 0.9

From given data, $h_n \; = \; 45 \; m$

$T \; = \; C_t \; (h_n)^m = 0.0466(45)^{0.9} \quad = 1.433 \; sec$

Step 03: Calculation of zone coefficient and importance factor:

From Table 6.2.15 or Figure 6.2.24, we get for Dhaka.

Z = Zone - II = 0.2 **(Page 3196)**

From Table 6.1.1 & Table 6.2.17, we get-

Importance factor, I = 1 (for Occupancy Category - II) (**Page 3061) (Page 3197)**

Step 04: Calculation of seismic design category and response reduction factor:

From Table 6.2.18, we get, Seismic design category = C **(Page 3198)**

From 8.3.2 (Provisions) we get, IMRF (Intermediate Moment Resisting Frame) **(Page 3669)**

From Table 6.2.19, we get, R = 5 **(Page 3202)**

$\frac{I}{R} = \frac{1}{5} = 0.2 \; (< 1)(ok)$

(Page 3193)

Step 05: Determination of Soil factor (S) and other parameters (T_A, T_B etc.)

From Table 6.2.16, we get-

S = 1.15

T_B= 0.2 sec

T_C= 0.6 sec

T_D= 2 sec

Step 06: Calculation of normalized acceleration response spectrum

$\boldsymbol{\eta = \sqrt{\frac{10}{5+\xi}} = \sqrt{\frac{10}{5+5}} = 1(> 0.55), OK}$ **(Page 3194)**

$\boldsymbol{C_s \; = \; 2.5 \, S\eta \left(\frac{T_C}{T}\right) = \; 2.5 \; \times \; 1.15 \; \times \; 1 \; \left(\frac{0.6}{1.433}\right) = 1.204}$ **(Page 3193)**

Step 07: Calculation of design spectral acceleration or lateral seismic force coefficient

$S_a \; = \frac{2}{3}\frac{ZI}{R}C_s$ (Page 3193)

$S_a \; = \frac{2}{3}\frac{0.2\times1}{5} \times 1.204 = 0.03211 \; (> \mathbf{0.01695})(OK)$

$0.67\beta ZIC \; = 0.67 \times 0.11 \times 0.2 \times 1 \times 1.15$
$= \mathbf{0.01695}$ (Page 3193)
$0.044 \; S_{DS}I \; = 0.044 \times 0.383 \times 1$
$= 0.01685$ (Page 3208)

Step 08: Calculation of total seismic load

$DL + 0.25LL = 9 + 0.25 \times 3 = 9.75\ kN/m^2$ (Page 3209)

Seismic Load on each story, $w_i = 9.75 \times (30 \times 60)$

$$= 17550\ kN\ (i = 1{\sim}15)$$

$$w_1 = w_2 = w_3 = \ \ldots\ldots\ldots\ldots\ldots\ldots = w_{15} = 17550\ kN$$

$$Total\ seismic\ Load,\ W = 17550 \times 15 = 263250\ kN$$

Step 09: Calculation of seismic design base shear

$V = S_a W = 0.03211 \times 263250 = 8452.96\ kN$ (Page 3207)

Step 10: Calculation of vertical distribution of lateral force

T (sec)	≤0.5	≥2.5
k	1	2

$$T = 1.433\ sec$$
$$k = 0.5T + 0.75$$
$$= 1.4665$$

$$F_x = V\frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} = V\frac{w_x h_x^k}{w_1 h_1^k + w_2 h_2^k + w_3 h_3^k + \ldots\ldots\ldots. + w_n h_n^k}$$

$$= \frac{17550 h_x^k}{17550(h_1^k + h_2^k + \ldots\ldots\ldots + h_n^k)} \times V$$

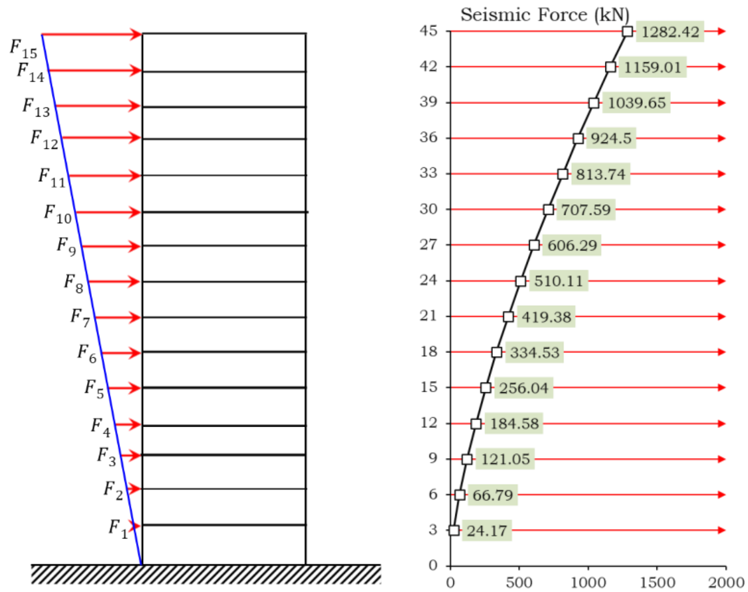
$$= \frac{h_x^k}{3^k + 6^k + \ldots\ldots. + 45^k} \times 8452.96$$

$$= \frac{h_x^k}{3^k(1^k + 2^k + \ldots\ldots. + 15^k)} \times 8452.96$$

$$= \frac{h_x^{1.46665}}{1750.906} \times 8452.96 = 4.828\ h_x^{1.4665}$$

F_x	h_x (m)	$4.828\ h_x^{1.4665}$ (kN)
F_1	3	24.18
F_2	6	66.82
F_3	9	121.1
F_4	12	184.7
F_5	15	256.2
F_6	18	334.7
F_7	21	419.6
F_8	24	510.3

F_x	h_x (m)	$4.828\ h_x^{1.4665}$ (kN)
F_9	27	606.5
F_{10}	30	707.9
F_{11}	33	814.1
F_{12}	36	924.9
F_{13}	39	1040
F_{14}	42	1159
F_{15}	45	1283



Step 10: Calculation of vertical distribution of lateral force (alternate way)

Story Level, i	wx (kN)	Cumulative floor height, hx (m)	$w_x \times h_x^k$	$F_x = V \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} (kN)$
1	17550	3	87902.2182	24.17
2	17550	6	242926.959	66.79
3	17550	9	440273.502	121.05
4	17550	12	671354.02	184.58
5	17550	15	931268.862	256.04
6	17550	18	1216741.8	334.53
7	17550	21	1525384.46	419.38
8	17550	24	1855356.94	510.11
9	17550	27	2205186.18	606.29
10	17550	30	2573658.75	707.59
11	17550	33	2959752.87	813.74
12	17550	36	3362593.02	924.5
13	17550	39	3781418.23	1039.65
14	17550	42	4215559.24	1159.01
15	17550	45	4664421.58	1282.42

$\sum_{i=1}^{n} w_i h_i^k$ =30733798.6

Wind Load Calculation (According to BNBC 2020)

Following the published documents by Ministry of Housing and Public Works,

Government of the People's Republic of Bangladesh

(SRO No. 55-Law/2020, Date: 05-11-1426/18-02-2020)

Method 02 – Analytical Procedure (BNBC 2020.Art 2.4.1- Page 547)

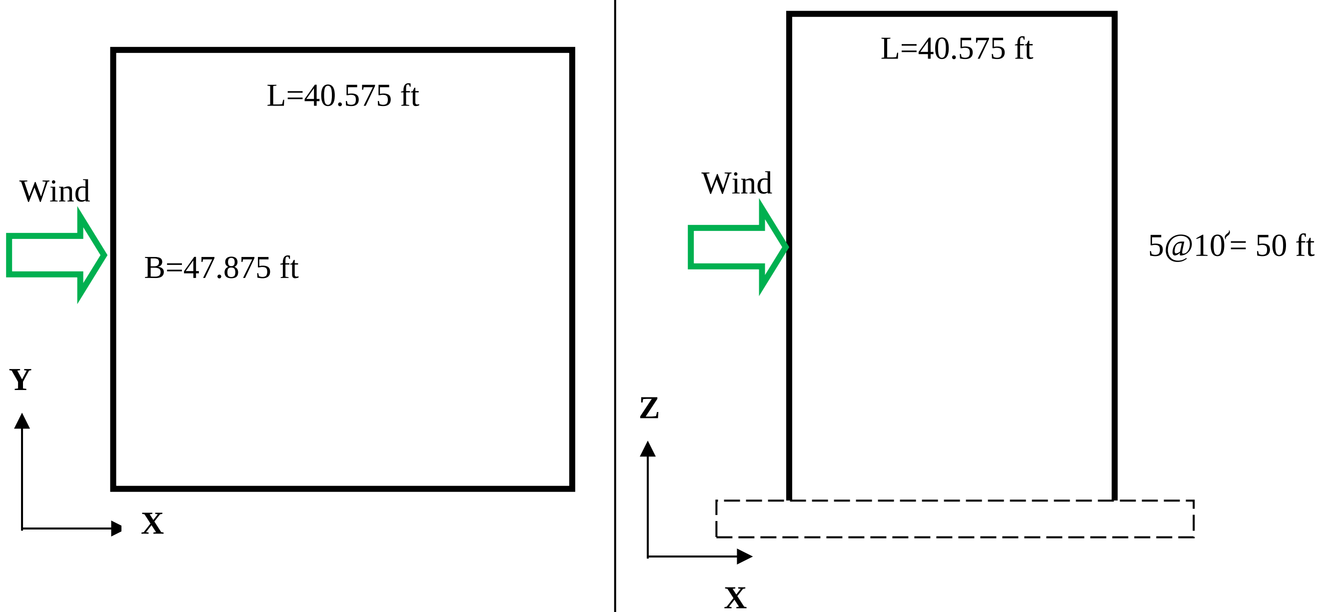


Figure 1(a): Plan View of Building

Figure 1(b): Elevation View of Building

For this analysis,

- ✓ L= 40.575 ft = 12.367 m
- ✓ B= 47.875 ft = 14.592 m
- ✓ H= 50ft = 15.24 m
- ✓ Location Dhaka
- ✓ Wind - X direction

Velocity Pressure Calculation:

$q_z = 0.000613 * K_z * K_{zt} * K_d * V^2 * I$ (kN/m^2) ………….(Eq 6.2.17) (Page- 559)

Where,

q_z = Velocity Pressure (kN/m^2)

K_z = Velocity Pressure Exposure Coefficient ..……….. (Table 6.2.11) (Page- 605)

- Details of Exposure Categories - Art 2.4.6 (Page 554).
  For Dhaka (Urban Area) consider Exposure A.
- Cases (Page 606). Consider Case 1 here because of low rise building (If height of the building is less than 75 ft then it is called low rise building according to International Building Code (IBC)-2015).

2.4.6.2 Surface roughness categories

A ground surface roughness within each 45o sector shall be determined for a distance upwind of the site as defined in Sec 2.4.6.3 from the categories defined in the following text, for the purpose of assigning an exposure category as defined in Sec 2.4.6.3.

Surface Roughness A: Urban and suburban areas, wooded areas, or other terrain with numerous closely spaced obstructions having the size of single-family dwellings or larger.

Surface Roughness B: Open terrain with scattered obstructions having heights generally less than 9.1 m. This category includes flat open country, grasslands, and all water surfaces in cyclone prone regions.

Surface Roughness C: Flat, unobstructed areas and water surfaces outside cyclone prone regions. This category includes smooth mud flats and salt flats.

2.4.6.3 Exposure categories

Exposure A: Exposure A shall apply where the ground surface roughness condition, as defined by Surface Roughness A, prevails in the upwind direction for a distance of at least 792 m or 20 times the height of the building, whichever is greater.

Exception: For buildings whose mean roof height is less than or equal to 9.1 m, the upwind distance may be reduced to 457 m.

Exposure B: Exposure B shall apply for all cases where Exposures A or C do not apply.

Exposure C: Exposure C shall apply where the ground surface roughness, as defined by Surface Roughness C, prevails in the upwind direction for a distance greater than 1,524 m or 20 times the building height, whichever is greater. Exposure C shall extend into downwind areas of Surface Roughness A or B for a distance of 200 m or 20 times the height of the building, whichever is greater. For a site located in the transition zone between exposure categories, the category resulting in the largest wind forces shall be used.

Exception: An intermediate exposure between the preceding categories is permitted in a transition zone provided that it is determined by a rational analysis method defined in the recognized literature.

Case Determination-

Notes:

1. Case 1:
   (a) All components and cladding.
   (b) Main wind force resisting system in low-rise buildings designed using Figure 6.2.10.

   Case 2:
   (a) All main wind force resisting systems in buildings except those in low-rise buildings designed using Figure 6.2.10.
   (b) All main wind force resisting systems in other structures.

Table 6.2.11: Velocity Pressure Exposure Coefficients, K_h and K_z

Height above ground level, z	Exposure (Note 1)			
	A		B	C
(m)	Case 1	Case 2	Case 1 & 2	Case 1 & 2
0-4.6	0.70	0.57	0.85	1.03
6.1	0.70	0.62	0.90	1.08
7.6	0.70	0.66	0.94	1.12
9.1	0.70	0.70	0.98	1.16
12.2	0.76	0.76	1.04	1.22
15.2	0.81	0.81	1.09	1.27
18	0.85	0.85	1.13	1.31
21.3	0.89	0.89	1.17	1.34
24.4	0.93	0.93	1.21	1.38
27.41	0.96	0.96	1.24	1.40
30.5	0.99	0.99	1.26	1.43
36.6	1.04	1.04	1.31	1.48
42.7	1.09	1.09	1.36	1.52
48.8	1.13	1.13	1.39	1.55
54.9	1.17	1.17	1.43	1.58
61.0	1.20	1.20	1.46	1.61
76.2	1.28	1.28	1.53	1.68
91.4	1.35	1.35	1.59	1.73
106.7	1.41	1.41	1.64	1.78
121.9	1.47	1.47	1.69	1.82
137.2	1.52	1.52	1.73	1.86
152.4	1.56	1.56	1.77	1.89

K_{zt} = Topographic Factor .... (Art 2.4.7) (Page- 556)

- For flat or even surface like Dhaka consider

  $K_{zt} = 1$ .... (Art 2.4.7.2) (Page- 556).

- For other cases $K_{zt} = (1 + K_1 * K_2 * K_3)^2$

  Here, the value of K_1, K_2 & K_3 may determine from Figure 6.2.4 (page - 572 & 573).

Equation:

$K_{zt} = (1 + K_1\ K_2\ K_3)^2$; K_1 determined from Table below; $K_2 = \left(1 - \frac{|x|}{\mu L_h}\right)$; $K_3 = e^{-\gamma z/L_h}$

Parameters for Speed-Up Over Hills and Escarpments						
Hill Shape	***K₁/(H/Lₕ)***			***γ***	***μ***	
	Exposure A	**Exposure B**	**Exposure C**		**Upwind of crest**	**Downwind of Crest**
2-dimensional ridges (or valleys with negative H in $K_1/(H/L_h)$)	1.30	1.45	1.55	3	1.5	1.5
2-dimensional escarpments	0.75	0.85	0.95	2.5	1.5	4
3-dimensional axisym. Hill	0.95	1.05	1.15	4	1.5	1.5

K_d = Wind Directionality Factor ………… (Table 6.2.12) (Page -606)

- Building Main Wind Force resisting system =0.85.

Table 6.2.12: Wind Directionality Factor, K_d

Structure Type	Directionality Factor K_d *	Structure Type	Directionality Factor K_d *
Buildings		Solid Signs	0.85
Main Wind Force Resisting System	0.85	Open Signs and Lattice Framework	0.85
Components and Cladding	0.85	Trussed Towers	
Arched Roofs	0.85	Triangular, square, rectangular	0.85
Chimneys, Tanks, and Similar Structures		All other cross section	0.95
Square	0.90		
Hexagonal	0.95		
Round	0.95		

* Directionality Factor K_d has been calibrated with combinations of loads specified in Sec 2.7. This factor shall only be applied when used in conjunction with load combinations specified in Sections 2.7.2 and 2.7.3.

V = Basic Wind Speed (m/s) …………(Table 6.2.8) (Page – 603)

- For Dhaka city V= 65.7 m/s

Table 6.2.8: Basic Wind Speeds, *V*, for Selected Locations in Bangladesh

Location	Basic Wind Speed (m/s)	Location	Basic Wind Speed (m/s)
Angarpota	47.8	Lalmonirhat	63.7
Bagerhat	77.5	Madaripur	68.1
Bandarban	62.5	Magura	65.0
Barguna	80.0	Manikganj	58.2
Barisal	78.7	Meherpur	58.2
Bhola	69.5	Maheshkhali	80.0
Bogra	61.9	Moulvibazar	53.0
Brahmanbaria	56.7	Munshiganj	57.1
Chandpur	50.6	Mymensingh	67.4
Chapai Nawabganj	41.4	Naogaon	55.2
Chittagong	80.0	Narail	68.6
Chuadanga	61.9	Narayanganj	61.1
Comilla	61.4	Narsinghdi	59.7
Cox's Bazar	80.0	Natore	61.9
Dahagram	47.8	Netrokona	65.6
Dhaka	65.7	Nilphamari	44.7
Dinajpur	41.4	Noakhali	57.1
Faridpur	63.1	Pabna	63.1
Feni	64.1	Panchagarh	41.4
Gaibandha	65.6	Patuakhali	80.0
Gazipur	66.5	Pirojpur	80.0
Gopalganj	74.5	Rajbari	59.1
Habiganj	54.2	Rajshahi	49.2
Hatiya	80.0	Rangamati	56.7
Ishurdi	69.5	Rangpur	65.3
Joypurhat	56.7	Satkhira	57.6
Jamalpur	56.7	Shariatpur	61.9
Jessore	64.1	Sherpur	62.5
Jhalakati	80.0	Sirajganj	50.6
Jhenaidah	65.0	Srimangal	50.6
Khagrachhari	56.7	St. Martin's Island	80.0
Khulna	73.3	Sunamganj	61.1
Kutubdia	80.0	Sylhet	61.1
Kishoreganj	64.7	Sandwip	80.0
Kurigram	65.6	Tangail	50.6
Kushtia	66.9	Teknaf	80.0
Lakshmipur	51.2	Thakurgaon	41.4

I= Importance Factor …………..(Table 6.2.9) (Page -604)

- ❖ For our calculation Occupancy Category is II for residential building and V>44 m/s, use I= 1.

Table 6.1.1: Occupancy Category of Buildings and other Structures for Flood, Surge, Wind and Earthquake Loads.

Nature of Occupancy	Occupancy Category
Buildings and other structures that represent a low hazard to human life in the event of failure, including, but not limited to: • Agricultural facilities • Certain temporary facilities • Minor storage facilities	I
All buildings and other structures except those listed in Occupancy Categories I, III and IV	II
Buildings and other structures that represent a substantial hazard to human life in the event of failure, including, but not limited to: • Buildings and other structures where more than 300 people congregate in one area • Buildings and other structures with day care facilities with a capacity greater than 150 • Buildings and other structures with elementary school or secondary school facilities with a capacity greater than 250 • Buildings and other structures with a capacity greater than 500 for colleges or adult education facilities • Healthcare facilities with a capacity of 50 or more resident patients, but not having surgery or emergency Treatment facilities • Jails and detention facilities	III
Buildings and other structures designated as essential facilities, including, but not limited to: • Hospitals and other healthcare facilities having surgery or emergency treatment facilities • Fire, rescue, ambulance, and police stations and emergency vehicle garages • Designated earthquake, hurricane, or other emergency shelters • Designated emergency preparedness, communication, and operation centers and other facilities required for emergency response • Power generating stations and other public utility facilities required in an emergency	IV

Table 6.2.9: Importance Factor, I (Wind Loads)

Occupancy Category[1] or Importance Class	Non-Cyclone Prone Regions and Cyclone Prone Regions with V = 38-44 m/s	Cyclone Prone Regions with V > 44 m/s
I	0.87	0.77
II	1.0	1.00
III	1.15	1.15
IV	1.15	1.15

[1] The building and structure classification categories are listed in Table 6.1.1

Now,

$q_z = 0.000613 * K_z * K_{zt} * K_d * V^2 * I$

$= 0.000613 * K_z * 1 * 0.85 * 65.7^2 * 1$

$\boxed{q_z = 2.2491 * K_z}$ ……………………………………………….. **(I)**

Design Wind Pressure Calculation:

$P = q * G * C_p - q_i * (GC_{pi})$ (kN/m^2) …………(Eq 6.2.19) (Page- 560)

Where,

$q = q_z =$ Velocity Pressure (kN/m^2)

G = Gust Effect Factor …………..(Art 2.4.8) (Page- 556)

- For a **Rigid Structure** Gust Effect Factor should be taken as 0.85.

A building or other structure whose fundamental frequency (F= $\frac{1}{T}$) is greater than or equal to 1 Hz then this building is known as Rigid Structure.

Building Base Period T= $C_t * (h_n)^m$ .... (Eq 6.2.38) (Page- 630)

Here,

h_n = Total Height of Building = 15.24 m

Take C_t and m from Table 6.2.20 (Page -631)

Table 6.2.20: Values for Coefficients to Estimate Approximate Period

Structure type	C_t	m	
Concrete moment-resisting frames	0.0466	0.9	Note: Consider moment resisting frames as frames which resist 100% of seismic force and are not enclosed or adjoined by components that are more rigid and will prevent the frames from deflecting under seismic forces.
Steel moment-resisting frames	0.0724	0.8	
Eccentrically braced steel frame	0.0731	0.75	
All other structural systems	0.0488	0.75	

We consider a Concrete Moment resisting frame,

value of Ct =0.0466 & m = 0.9

T= $C_t * (h_n)^m$

= $0.0466 * (15.24)^{0.9}$

=0.5408 Sec

Fundamental Frequency F= $\frac{1}{T} = \frac{1}{0.5408}$ =1.84896 Hz > 1

So, the structure is rigid.

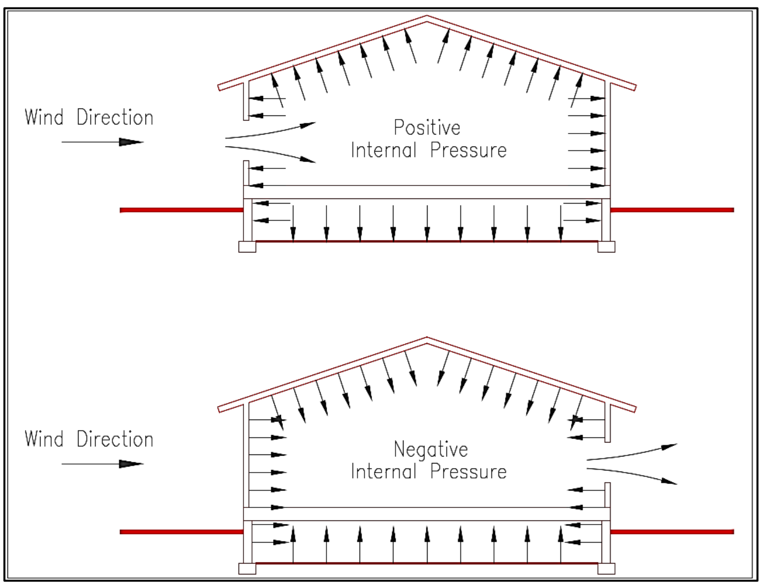
Cp = External Pressure Coefficient ....(Figure 6.2.6) (Page – 574)

- For Windward Side Cp = 0.8
- For Leeward Side, $\frac{\mathbf{L}}{\mathbf{B}} = \frac{12.367}{14.592}$ = 0.8475,

  So, Cp = -0.5

Wall Pressure Coefficients, C_p			
Surface	L/B	C_p	Use With
Windward Wall	All values	0.8	q_z
Leeward Wall	0-1	-0.5	q_h
	2	-0.3	
	≥ 4	-0.2	
Side Wall	All values	-0.7	q_h

q_i = Internal Pressure = q_z



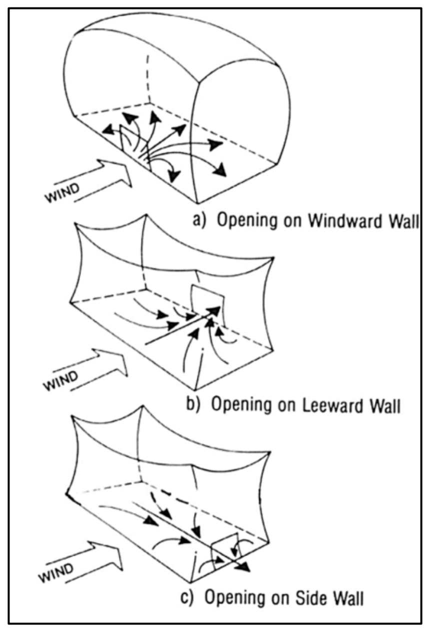


Figure: Internal Pressure

GCpi = Internal Pressure Coefficient …………..(Figure 6.2.5) (Page- 573)

For Enclosed Member

The value of GCpi is -0.18 for Windward side & +0.18 for Leeward side.

Enclosed, Partially Enclosed, and Open Buildings: Walls & Roofs

Enclosure Classification	GC_{pi}	Notes:
Open Building	0.00	1. Plus and minus signs signify pressures acting toward and away from the internal surfaces, respectively.
Partially Enclosed Building	+0.55 -0.55	2. Values of GC_{pi} shall be used with q_z or q_h as specified in Sec 2.4.11.
Enclosed Building	+0.18 -0.18	3. Two cases shall be considered to determine the critical load requirements for the appropriate condition: (i) a positive value of GC_{pi} applied to all internal surfaces (ii) a negative value of GC_{pi} applied to all internal surfaces.

DESIGN WIND FORCE

Design Wind Pressure for Windward Side, $P_W = q*G*Cp - q_i * (GCpi)$

$$= q_z*0.85 *0.8 - q_h*(\pm 0.18)$$
$$= (2.2491*K_z) *0.85*0.8 - (2.2491*K_h)*(\pm 0.18)$$
$$= 1.5294*K_z - 2.2491*.81*(\pm 0.18)$$
$$= 1.5294*K_z - (\pm\ 0.3279)\ kN/m^2$$

Design Wind Pressure for Leeward Side, $P_L = q*G*Cp - q_i*(GCpi)$

$$= q_h*0.85 *(-0.5) - q_h*(\pm 0.18)$$
$$= (2.2491*K_h) *0.85*(-0.5) - (2.2491*K_h) *(\pm 0.18)$$
$$= (2.2491*0.81) *0.85*(-0.5) - 2.2491*.81*(\pm 0.18)$$
$$= -0.7743 - (\pm\ 0.3279)\ kN/m^2$$

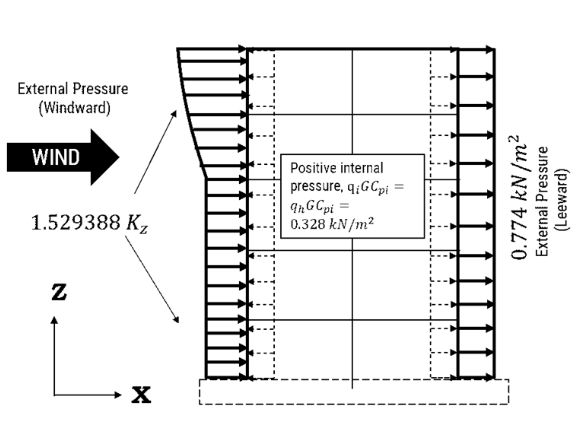


Fig.: Pressure distribution considering positive internal pressure

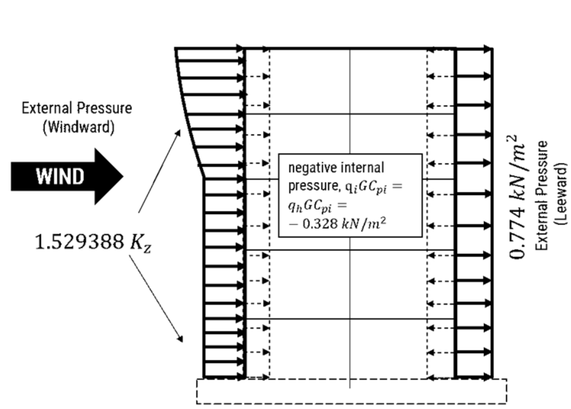


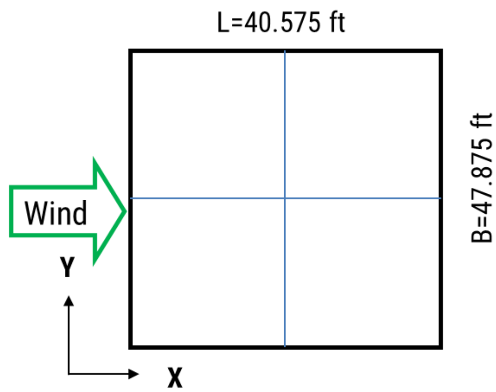
Fig.: Pressure distribution considering negative internal pressure

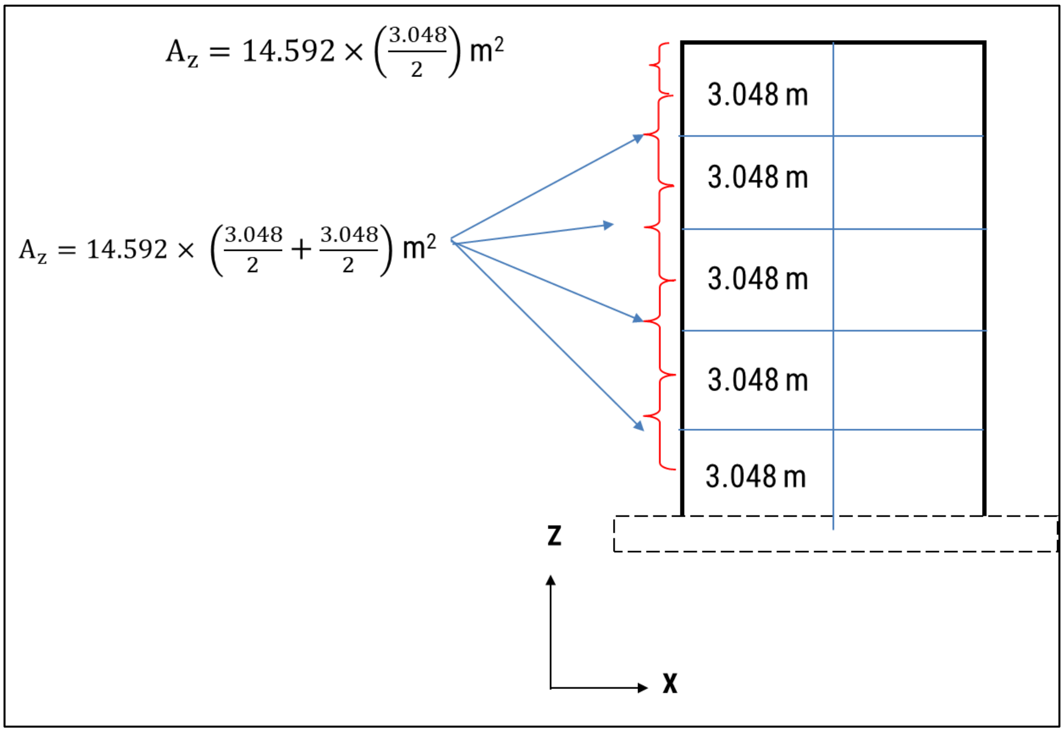
Windward Direction

Height above ground level, z(m)	K_z	Design wind pressure Pw = 1.5294*K_z – (± 0.3279) kN/m^2		Contributing Area, A_z (m^2)	Design Wind Force $F_w = P_W*A_z$ (kN)	
		Considering (+0.3279) in above equation	Considering (–0.3279) in above equation		Considering (+0.3279)	Considering (–0.3279)
3.0488	0.7	0.7427	1.3985	44.4909	33.0434	62.2205
6.0976	0.7	0.7427	1.3985	44.4909	33.0434	62.2205
9.1463	0.7	0.7427	1.3985	44.4909	33.0434	62.2205
12.1951	0.76	0.8344	1.4903	44.4909	37.1232	66.3048
15.2439	0.81	0.9109	1.5667	22.2454	20.2633	34.8519

Leeward Direction

Height above ground level, z(m)	Design wind pressure P_L = – 0.7743 – (± 0.3279) kN/m^2		Contributing Area, A_z (m^2)	Design Wind Force $F_L = P_L*A_z$ (kN)	
	Considering (+0.3279) in above equation	Considering (–0.3279) in above equation		Considering (+0.3279)	Considering (–0.3279)
3.0488	-1.1022	-0.4464	44.4909	-49.0379	-19.8607
6.0976			44.4909	-49.0379	-19.8607
9.1463			44.4909	-49.0379	-19.8607
12.1951			44.4909	-49.0379	-19.8607
15.2439			22.2454	-24.5189	-9.9304


L=40.575 ft
B=47.875 ft
Wind
Y
X


$A_z = 14.592 \times \left(\frac{3.048}{2}\right) m^2$
$A_z = 14.592 \times \left(\frac{3.048}{2} + \frac{3.048}{2}\right) m^2$
3.048 m
3.048 m
3.048 m
3.048 m
3.048 m
Z
X

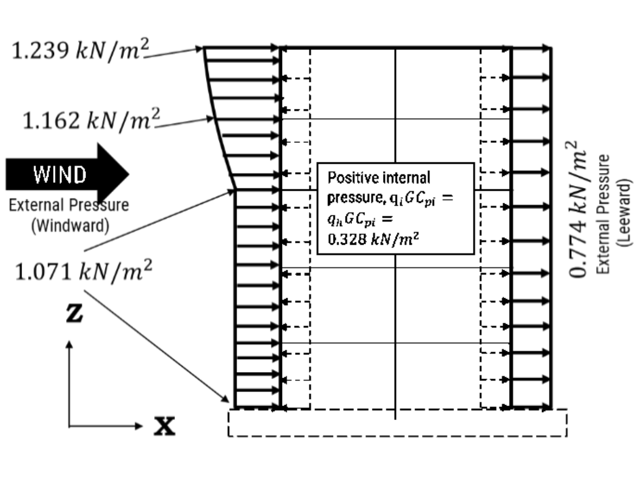


Fig.: Pressure distribution considering positive internal pressure

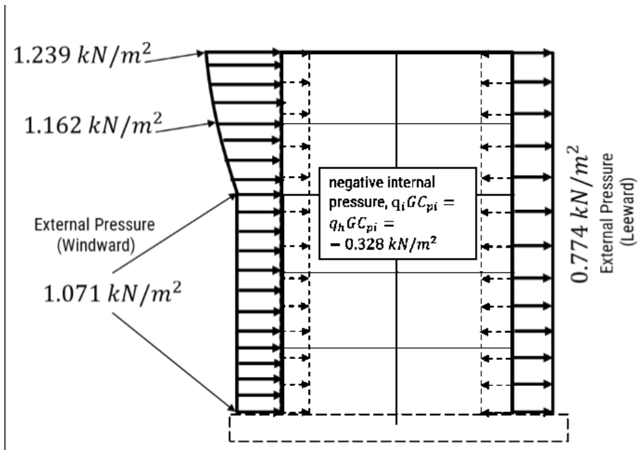


Fig.: Pressure distribution considering negative internal pressure

N.B.: Similarly calculate the wind forces in y direction.

1.7 Design of Floor Beams

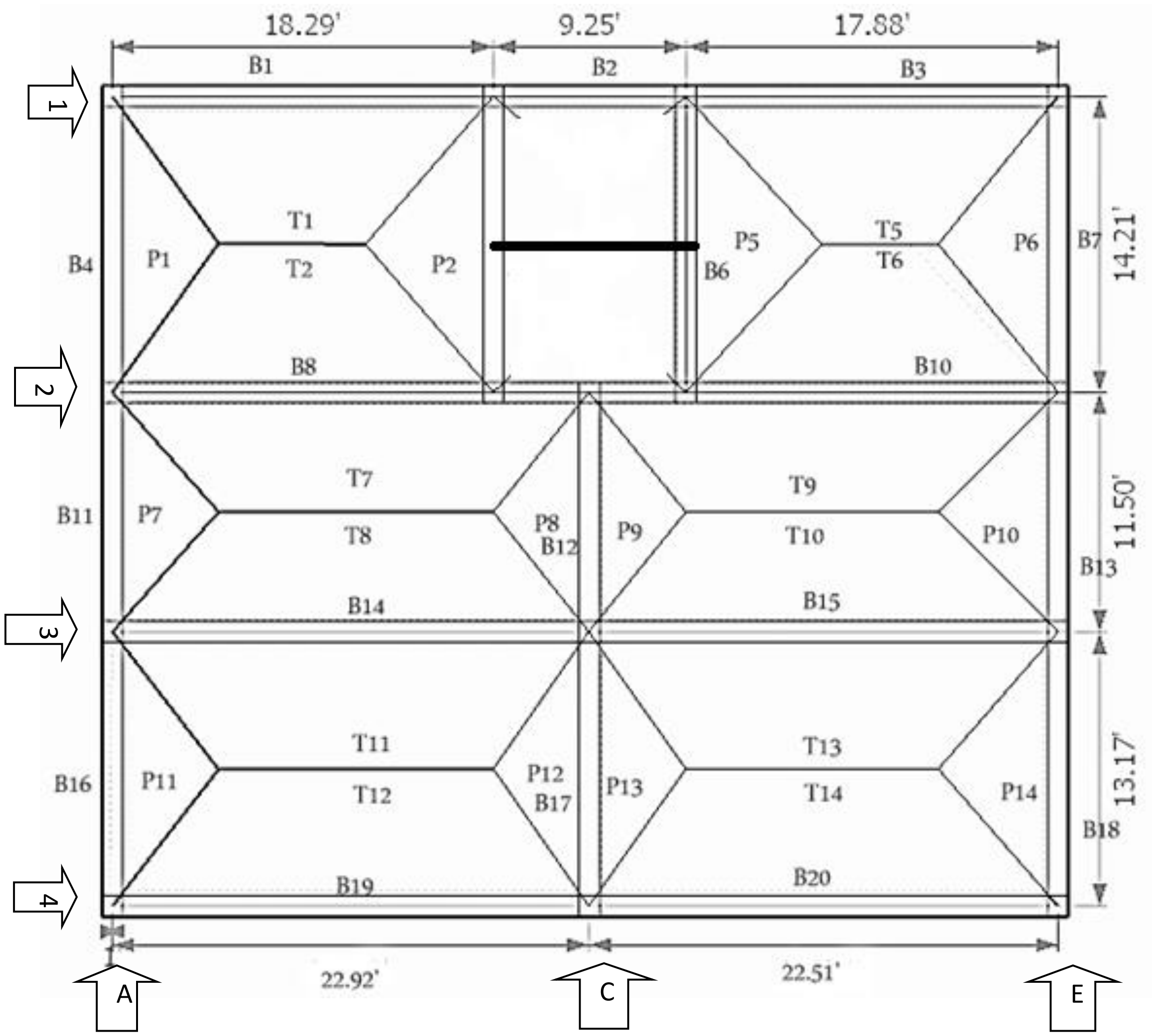


Figure 16: Beam layout

a) Assumptions and considerations

Load on slab, W = 234 psf
f_c'=3000 psi
fy = 60000 psi

b) Load calculation

Beam in-between A and C grid on grid 3

Trapezoidal panel:

$T_8 = \frac{1}{2}$ *(22.92 +11.42) *5.75 = 98.73 $ft^2 \approx T_{10}$

$T_{11} = \frac{1}{2}$ *(22.92 +9.75) *6.585 = 107.56$ft^2 \approx T_{13}$

Assuming, a beam of width 12″ and height 18″

Self-weight = $\frac{12*18}{144} * \frac{150}{1000}$ = 0.225 kip/ft *1.2 = 0.27 kip/ft

Load from Slab = $\frac{0.234*98.73}{22.92} + \frac{0.234*107.56}{22.92}$ = 2.11 kip/ft

Partition wall on beam = 0.42* 8.5* 120 = 0.43 * 1.2 = 0.52 k/ft

Total load = 0.27 + 2.11 +0.52 = 2.90 kip/ft

c) Moment and reinforcement

At grid 3-A joint

- $M_u = \frac{wl^2}{16} = \frac{2.90*22.92^2}{16}$ = 95.22 kip-ft = 1142.64 kip-in

At grid 3-C joint

- $M_u = \frac{wl^2}{9} = \frac{2.90*22.92^2}{9}$ = 169.27 kip-ft = 2031.3 kip-in

At mid span

+ $M_u = \frac{wl^2}{14} = \frac{2.90*22.92^2}{14}$ = 108.82 kip-ft = 1305.84 kip-in

Here, d = 18 - 2.5 – 2 = 13.5"

From table A.4 [Tension controlled], [Ref: Nilson pg:745]

$\rho_{.005}$= 0.0135 and φ=0.9

A_s= $\rho_{.005}$*b*d = 0.0135*12*13.5 = 2.2 in^2

$a = \frac{A_s f_y}{0.85 f_c' b} = \frac{2.2*60}{0.85*3*12} = 4.31''$

$a < h_f$, Rectangular beam analysis.

$$\therefore c = \frac{a}{\beta_1} = \frac{4.31}{0.85} = 5.07''$$

$$M_n = A_s f_y \left(d - \frac{a}{2}\right) = 2.2 * 60 * \left(13.5 - \frac{4.31}{2}\right) = 1497.54\ kip - in$$

$$\emptyset M_n = 0.9 * 1497.54 = 1347.8\ k - in > M_u = \ 1305.84\ kip - in$$

The beam will be designed as singly reinforcement for midspan and grid 3-A joint.

$$\emptyset M_n = 0.9 * 1497.54 = 1347.8 k - in < M_u = \ 2031.3\ \ kip - in$$

The beam will be designed as doubly reinforcement for grid 3-C joint. Compression reinforcement is required as well as tension reinforcement.

For grid 3-A joint,

Assume, $a\ = 5''$

$$-A_s = \frac{M_u/\emptyset}{f_y(d - a/2)} = \frac{1142.64\ /0.9}{60(13.5 - 5/2)} = 1.92 in^2$$

$$a\ = \frac{A_s f_y}{0.85 f_c' b} = \frac{1.92 * 60}{0.85 * 3 * 12} = 3.76''$$

$$-A_s = \frac{1142.64\ /0.9}{60(13.5 - 3.76/2)} = 1.82 in^2$$

$$a\ = \frac{1.82 * 60}{0.85 * 3 * 12} = 3.56''$$

$$-A_s = 1.81 in^2$$

For midspan,

Assume, $a\ = 3''$

$$+A_s = \frac{M_u/\emptyset}{f_y(d - a/2)} = \frac{1305.84\ /0.9}{60(13.5 - 3/2)} = 2.02 in^2$$

$$a\ = \frac{A_s f_y}{0.85 f_c' b} = \frac{2.02 * 60}{0.85 * 3 * 12} = 3.96''$$

$$+A_s = \frac{1305.84/0.9}{60(13.5 - 3.96/2)} = 2.10 in^2$$

$$a = \frac{2.10 * 60}{0.85 * 3 * 12} = 4.13''$$

$+A_s = 2.11\ in^2$

For grid 3-C joint, Remaining moment, $M_1 = \frac{2031.3}{0.9} - 1497.54 = 759.46\ kip - in$

Considering that the compression steel will not yield i.e., $\rho < \bar{\rho}_{cy}$

Using the strain distribution,

$$\epsilon_s' = \epsilon_u \frac{c - d'}{c} = 0.003 * \frac{5.07 - 2.5}{5.07} = 0.0015$$

$f_s' = \epsilon_s' E_s = 0.0015 * 29000 = 43.5\ ksi$

Compression reinforcement for grid 3-C joint, $-A_s' = \frac{759.46}{43.5\ (13.5-2.5)} = 1.59\ in^2$

Total area of tensile reinforcement at 60 ksi, $A_s = 2.2 + 1.59 * \frac{43.5}{60} = 3.35\ in^2$

Now, $\bar{\rho}_{cy} = 0.85\beta \frac{f_c'}{f_y} \frac{d'}{d} \frac{\epsilon_u}{\epsilon_u - \epsilon_y} + \rho' = 0.85\beta \frac{f_c'}{f_y} \frac{d'}{d} \frac{\epsilon_u}{\epsilon_u - \epsilon_y} + \frac{A_s'}{\mathrm{bd}}$

$= 0.85 * 0.85 * \frac{3}{60} * \frac{2.5}{13.5} * \frac{0.003}{0.003-0.00207} + \frac{1.59}{12*13.5}$; $[For\ 60\ grade\ steel\ \epsilon_y = 0.00207]$

$= 0.0313$

And $\rho = \frac{\mathrm{As}}{\mathrm{bd}} = \frac{3.35}{12*13.5} = 0.0207 < \bar{\rho}_{cy}$, so the compression steel will not yield.

The summaries of reinforcement are as follows,

At mid span, $+A_s = 2.11$ in^2
Grid 3-A joint, $-A_s = 1.81$ in^2
Grid 3-C joint, $-A_s = 3.35$ in^2 (tension) and compressive reinforcement, 1.59 in^2

Now, for structural integrity minimum 1/3 reinforcement need to be provided all through the beam and compressive reinforcement at Grid 3-C.

Provide $\frac{2.11}{3}$ =0.70 in^2 all through as positive reinforcement but it is less than compressive reinforcement 1.59 in^2.

Provide $\frac{3.35}{3}$ =1.12 in^2 all through as negative reinforcement. For a beam having 12" width, it is difficult to place more than 3 reinforcements in a row and more than 5 reinforcements in a face.

d) Shear design

$V_u = 0.5WL = 0.5 * 2.90 * 22.92 = 33.23\ k$

$\emptyset * V_c = 2 * \emptyset * \sqrt{f_c'}b * d = 2 * 0.75 * \sqrt{3000} * 12 * 13.5 = 13.3\ kip$

[In S.I. unit, $V_c = 0.17\lambda\sqrt{f_c'}bd$; BNBC 2020, where f_c' in MPa and b, d in mm]

Use Ø10mm as shear reinforcement.

$S_{max} = \frac{A_v f_y}{50 b_w} = \frac{2*0.121*60000}{50*12} = 24''$

$S_{max} = \frac{13.5}{2} = 6.5''(govern)$

$S_{max} = 24''$

$s = \frac{\emptyset A_v f_y d}{V_u - \emptyset V_c} = \frac{0.75*2*.121*60*13.5}{33.23-13.3} = 7.38''$

So, provide Ø10mm @ 6.5" c/c all through the beam.

Symmetric beam, so providing same reinforcement in B14 and B15. Design the beams for the load combinations as mentioned in BNBC using Approximate method for gravity load and Portal method for lateral load.

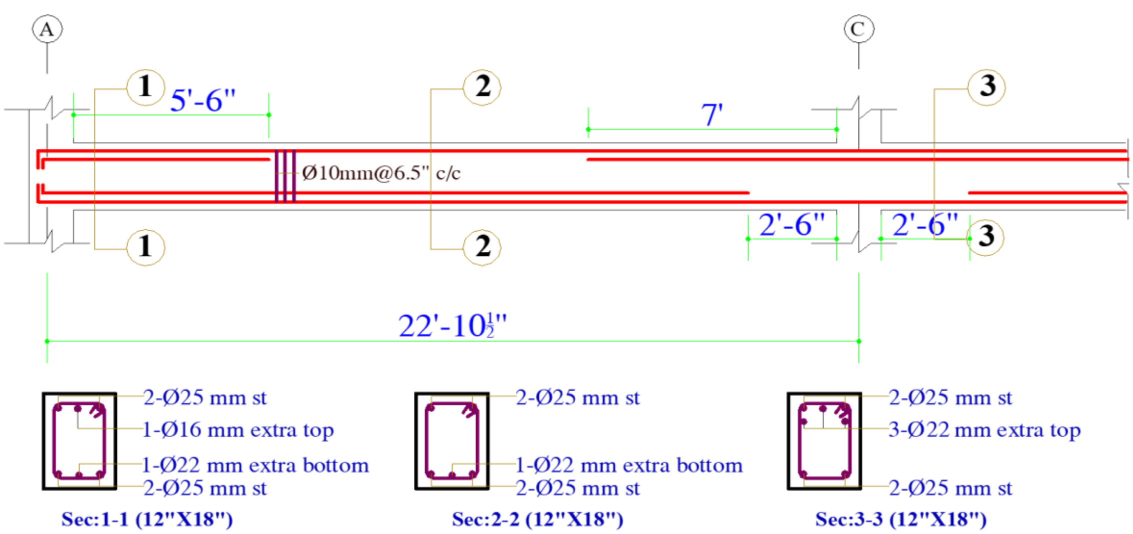


Figure 17: Reinforcement detail of beam

1.8 Design of Column

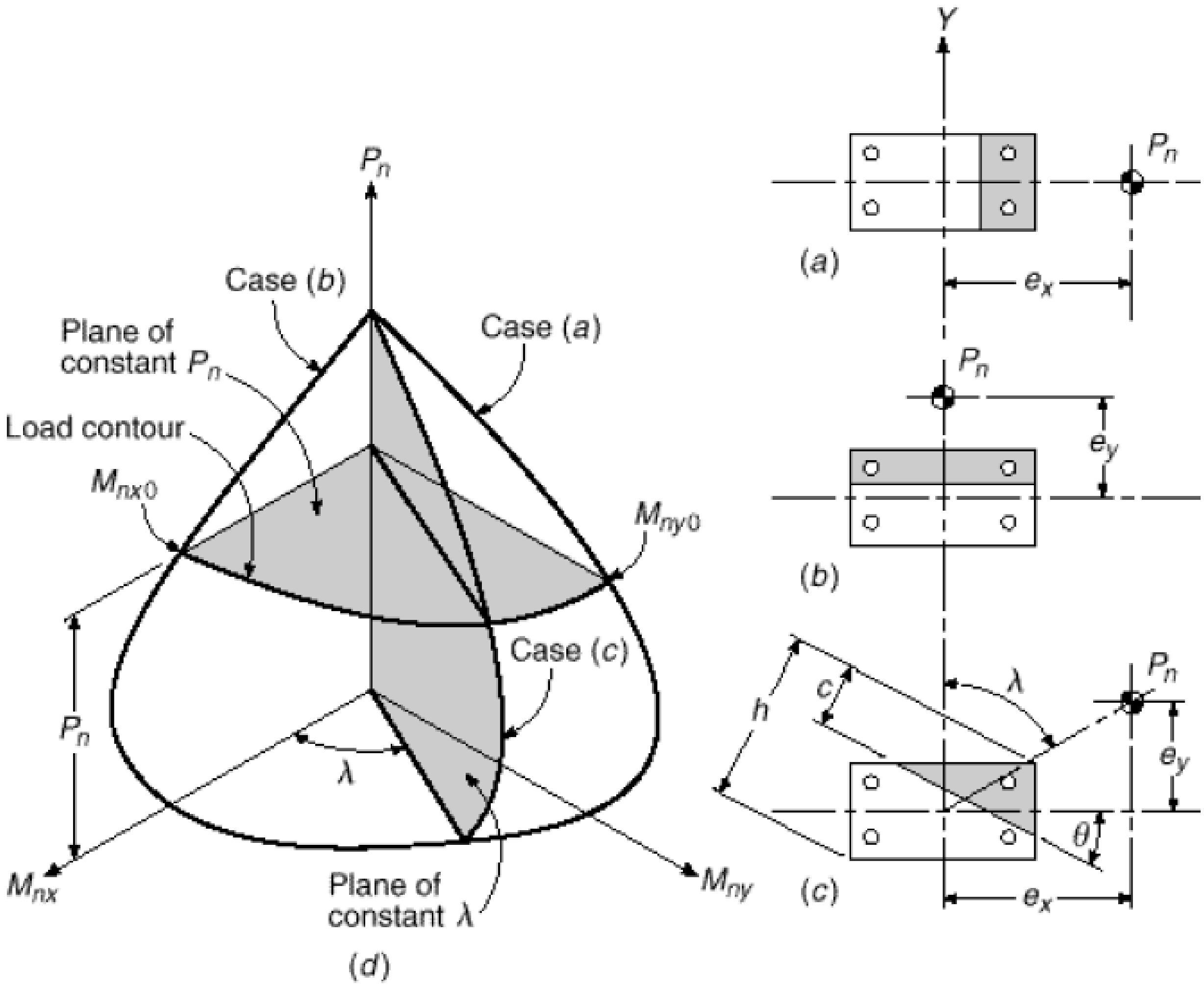


Figure 18: Interaction diagram for compression plus biaxial bending a) uniaxial bending about Y axis; b) uniaxial bending about X axis; c) biaxial bending about diagonal axis; d) interaction surface. (Ref: ACI Code, Design of Concrete Structure, 13th edition, Chap-8, P-274)

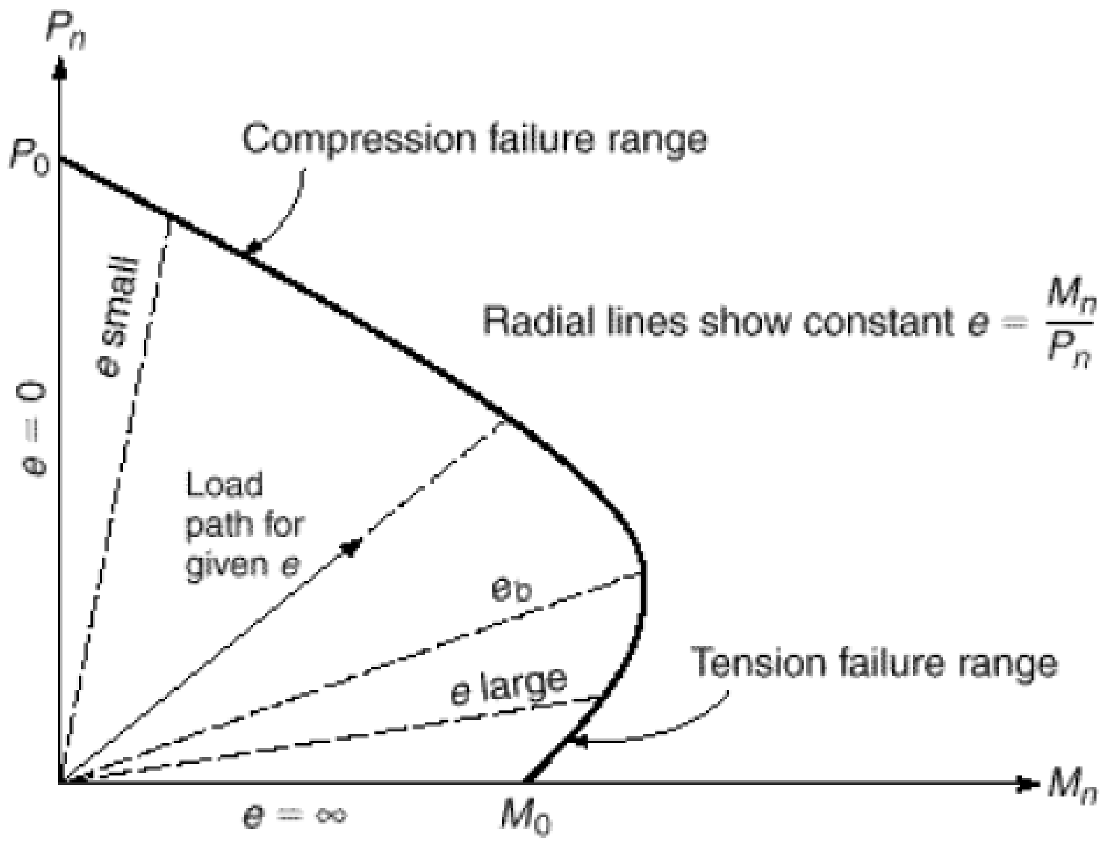


Figure 19: Interaction diagram for nominal column strength in combined bending and axial load. (Ref: ACI Code, Design of Concrete Structure, 13th edition, Chap-8, P-260)

a) Assumptions and considerations

f_y= 60000 psi
f'_c=4000 psi

For a column,

P = 554 K

M_x = 85 K-ft

M_y = 120K-ft

For, tied column, due to accidental eccentricity strength reduction factor α = 0.8 and

Based on importance strength reduction factor ϕ = 0.65, (ACI Code, Design of Concrete Structure, 13th edition, Chap-8, P-252) let, ρ_g = 2%

Now, $\phi P_n = \alpha\phi[0.85 f'_c*Ag + \rho_g *A_g*fy]$

554 = 0.65*0.8[0.85*4*Ag+0.02*Ag*60]

Ag = 232 in^2

Let, 18"x15"

<u>For My or dimension parallel to X axis,</u>

$\gamma = d_x/D_x$ = (18-2.5*2)/18 = 0.72~ 0.7

Eccentricity $e_x = M_y/P$ = 120/554 = 0.21' = 2.6"

e_x/h = 2.6/18 = 0.14

From graph, Kη = 0.79

$$\frac{Py}{f'cAg} = 0.79$$

Py = 853 k

<u>For Mx or dimension parallel to Y axis,</u>

$\gamma = d_y/D_y = 0.67 \approx 0.6$

e_y = 85/554 = 0.15' = 1.8"

e_y/h = 1.8/15 = 0.12

From graph, Kη = 0.85

$$\frac{Px}{f'cAg} = 0.85$$

P_x = 918 k

For P_o, Kη = (1.1+1.12)/2 = 1.11

$$\frac{Po}{f'cAg} = 1.11$$

P_o = 1200 k

$$\frac{1}{Pn} = \frac{1}{Px} + \frac{1}{Py} - \frac{1}{Po}$$

$$= \frac{1}{918} + \frac{1}{853} - \frac{1}{1200}$$

$\phi P_n = 0.65*700\ k = 455k < 554\ k$ (not ok)

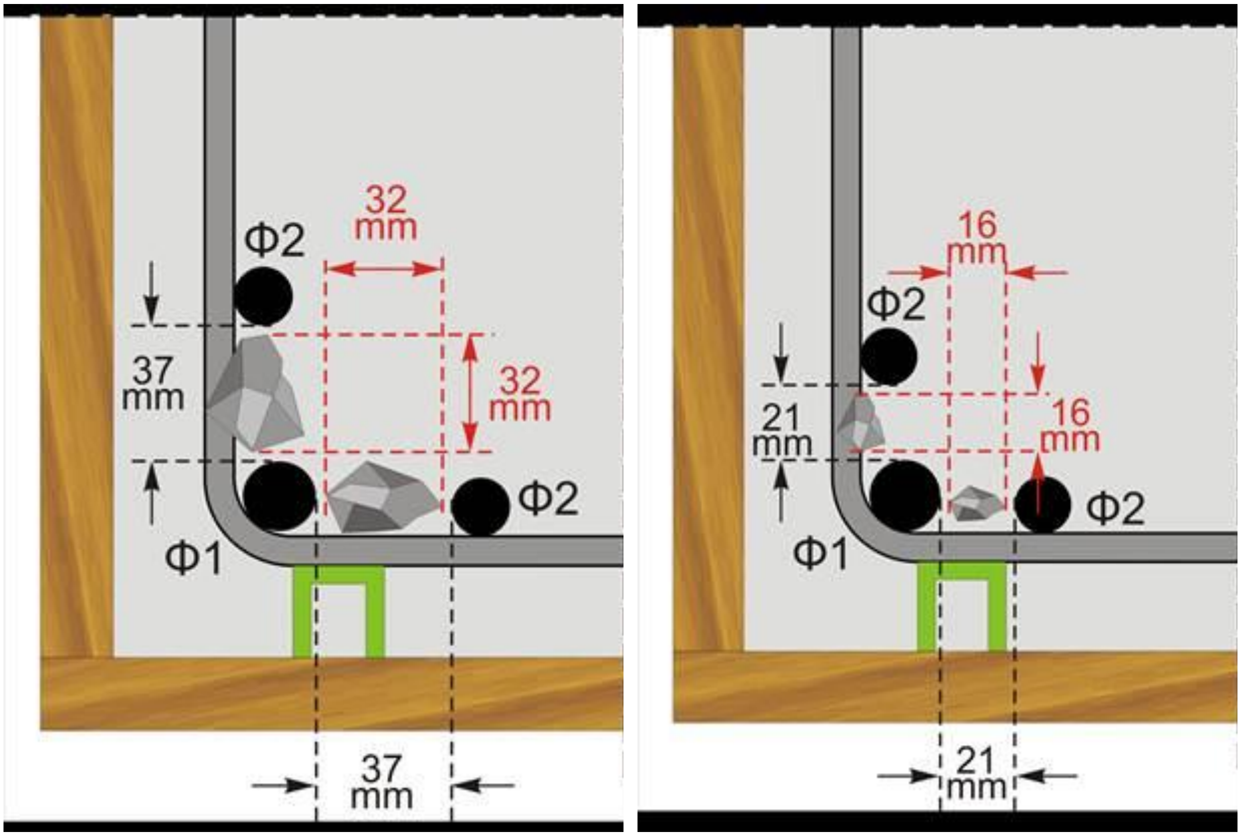


Figure 20: Minimum spacing between reinforcement bars

The distance between reinforcement bars must be such to allow the largest expected concrete size gravel to pass between them. In order to have properly anchored reinforcement, it is mandatory for rebars to be surrounded by concrete.

The minimum spacing between two reinforcement bars should be at least equal to the maximum coarse aggregate dimension plus a margin of 5 mm.

b) Tie bar

$\varnothing 10mm$bars are used.

Longitudinal Spacing

16 d_b of main bar = 16*20/25.4 = 12"

48 d_b of tie bar = 48*10/25.4 = 18"

Least dimension = 15"

So, spacing at top and bottom 12/2 = 6" c/c and at middle span 12" c/c.

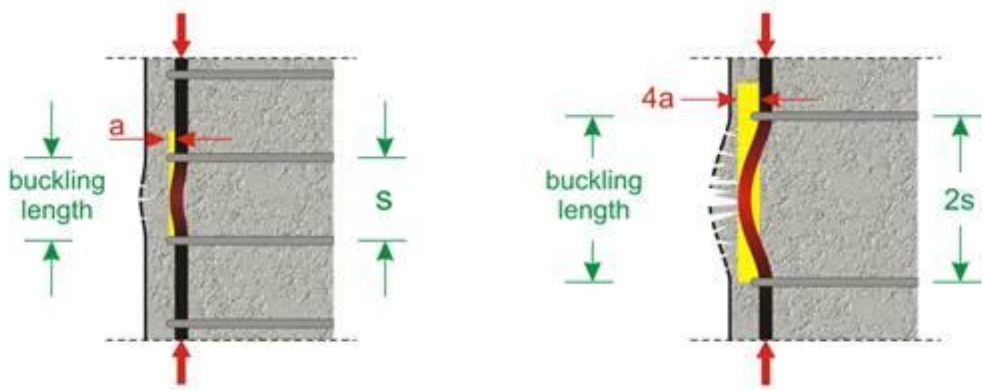


Figure 21: Failure mechanism of a column

A column with 10% fewer rebars has around 10% lower capacity strength. However, if we remove even a single intermediate stirrup, the capacity strength of that same column will be lowered even by 50%. This happens because the stirrup's removal doubles the buckling length of the rebars previously enclosed by it.

Cross sectional Spacing

the reinforcement at a distance greater than 6" from the outer most bar should be under a lateral tie and

Alternate bar should be under lateral tie.

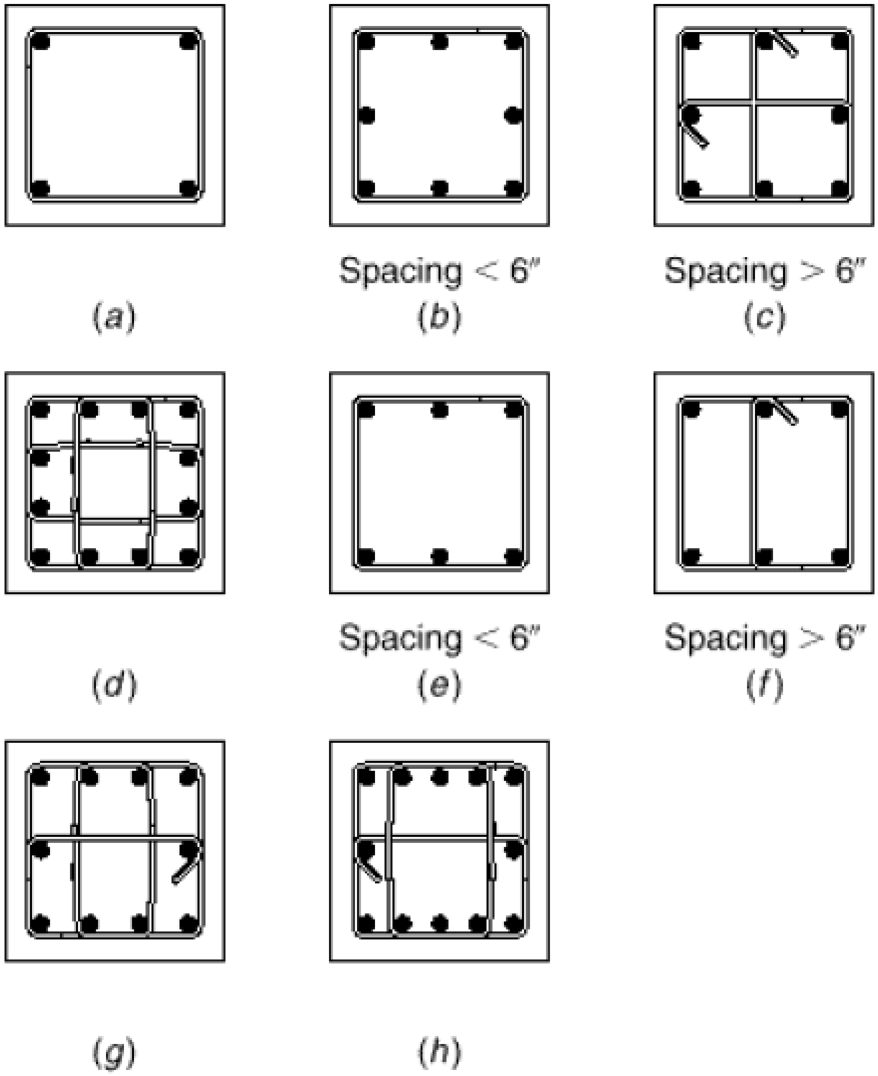


Figure 22: Tie arrangement of rectangular column ((Ref: ACI Code, Design of Concrete Structure, 13th edition, Chap-8, P-254)

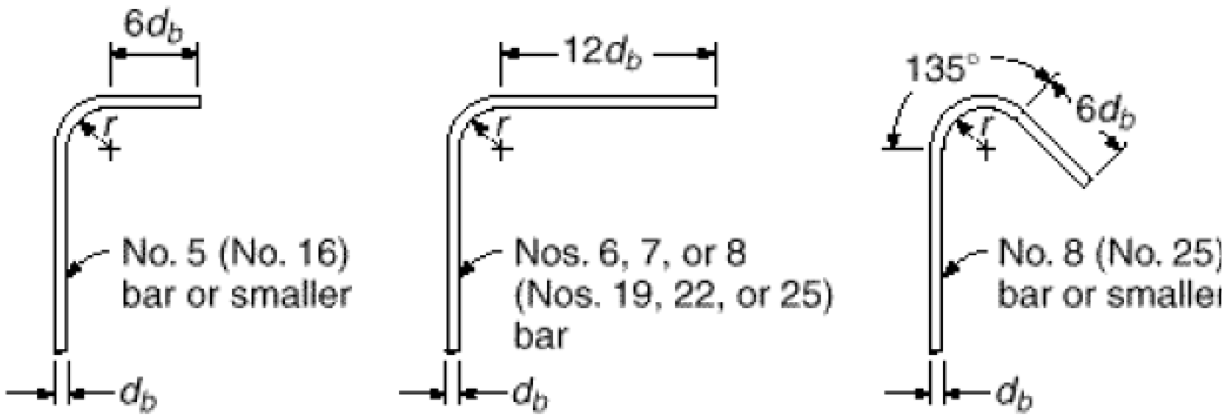


Figure 23: Standard bar hook for tie and stirrup. (Ref: ACI Code, Design of Concrete Structure, 13th edition, Chap-5, P-177)

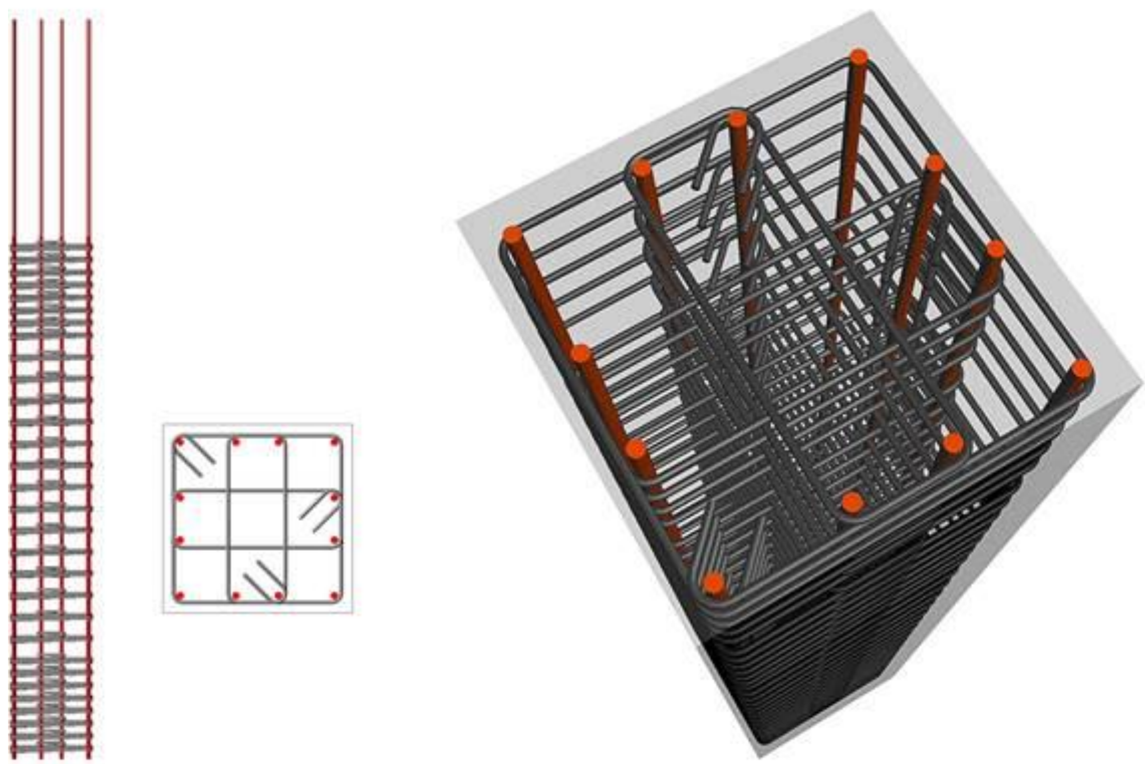


Figure 24: Typical column detail

References

- ACI code 318-14, American Concrete Institute, 2014.
- Bangladesh National Building Code (BNBC), 2006.
- Concrete Technology by Neville.
- Design of Concrete Structure by David Darwin, Charles W. Dolan and Arthur H. Nilson (15^{th} edition).
- Design of RCC Members by WSD and USD Methods, Public Works Department (PWD), 1997.
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- www.buildinghow.com

Part 2: Preliminary Design of the Superstructure of a Balanced Cantilever Bridge for Gravity loading

2.1 LECTURE PLAN

Lecture 1

Introduction to Bridge Engineering
About Balanced Cantilever Bridge

Lecture 2

Design of Deck Slab, Railing, Post and Sidewalk
Design of Interior Girder

(Dead load Calculation, Shear force diagram, Bending Moment Diagram for dead load)

Lecture 3 & 4

Design of Interior Girder, Exterior Girder

(SFD & BMD for live load including truck load, tandem load and Lane load at different sections, Corresponding Impact shear & moment, Design of reinforcement for shear & moment)

Lecture 5

Design of Cross Girder/ Diaphram and Articulation

2.2 SUBMISSION GUIDELINE OF BRIDGE DESIGN

The Design Report shall explain the details of the design process. It shall include the following items:

Design Specification, Standards followed in Analysis & Design
Loads and Load Combinations
Design of Slab
Design of Railing, Post and Sidewalk
Design of Interior Girder
Design of Exterior Girder
Design of Diaphrams or Cross Girders
Design of Articulation

[Note: Appropriate hand sketches showing the details of reinforcements must accompany all design calculations.]

2.3 INTRODUCTION TO BRIDGE ENGINEERING

a) What is a Bridge?

A Bridge is a structure providing passage over an obstacle without closing the way beneath.

The required passage may be for a road, a railway, pedestrians, a canal or a pipeline.

b) Requirements of an Ideal Bridge

Economical

Serves the intended functions with safety and convenience

Aesthetic elegant look

c) Selection of Bridge Site

A straight reach of the river

Steady river flow without serious whirls and cross currents

A narrow channel with firm banks

Suitable high banks above high flood level on each side

Rock or other hard strata close to the river bed level

Absence of sharp curves in the approaches

Avoidance of excessive underwater construction

Avoidance of expensive river training work

Proximity to a direct alignment of the connected road

d) Choice of a type of a Bridge

Channel Section

Sub-soil condition

Grades and Alignment

Hydraulic Data

Weather

Navigation requirements

Economic and Strategic considerations

Labour availability

Materials of Construction available

Period of Construction

Type of loading

Erection Facilities

e) Types of Bridge (based on action)

Slab Bridge

Deck-girder Bridge

Balanced- Cantilever Bridge

Suspension Bridge

Cable-stayed Bridge

Fig. 1: Deck-girder Bridge – ***Niteroi Bridge, Rio De Janeiro, Brazil*** (Source :online)

Fig.2: Arch Bridge - ***Sydney Harbour Bridge, Australia*** (Source :online)

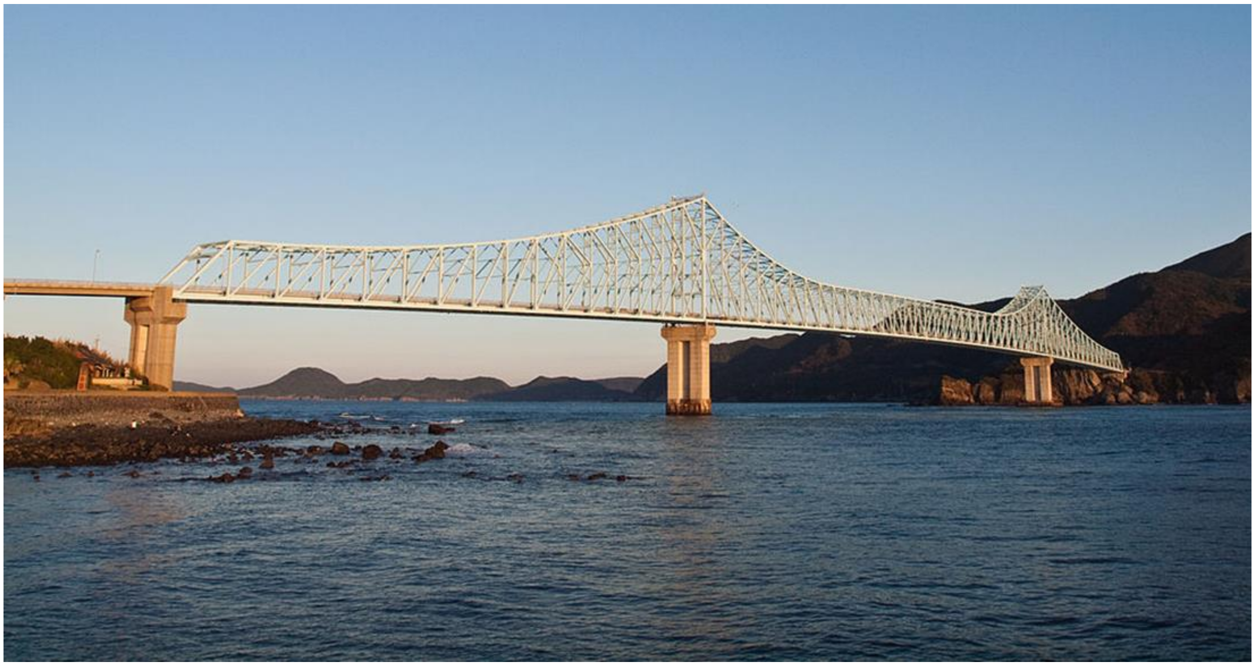


Fig. 3:Truss Bridge – ***Ikitsuki Bridge, Nagasaki, Japan*** (Source :online)

Fig.4: Cable-stayed Bridge – ***Rion Antirion Bridge, Greece*** (Source :online)

Fig. 5: Suspension Bridge – ***Akashi Kaikyo Bridge, Japan*** (Source :online)

Fig.6: Swing Bridge- (***Bridge Across Shatt-al-arab, Iraq***) (Source :online)

f) Types of Bridge (based on type of Support)

Simply-Supported Bridge

Continuous Bridge

Fixed Bridge

Cantilever Bridge

g) Types of Bridge (based on material)

Concrete/ R.C.C Bridge

Steel Bridge

Stone Bridge

Timber Bridge

Composite Bridge

Table 1: Classification of Bridge (based on span length)

Main Span Length	Type of Bridge
0-10m	Beam/ Girder R.C.C Bridge
10-50m	Precast Concrete (PCC) I- Girder Bridge
50-100m	Prestressed (PSC) concrete Box-Girder Bridge
100-200m	Composite Bridge (Steel Girder & Steel-Concrete Composite Slab)
>200m	PSC Extradose Bridge
1000-1500m (1-1.5km)	Cable-Stayed Bridge
>1500m (1.5km)	Suspension Bridge

h) Different Parts of a Bridge

Foundation: The portion below the bed level of a river.

Substructure: The parts below the bearings level and above the foundation.

Superstructure: Components above the level of bearings.

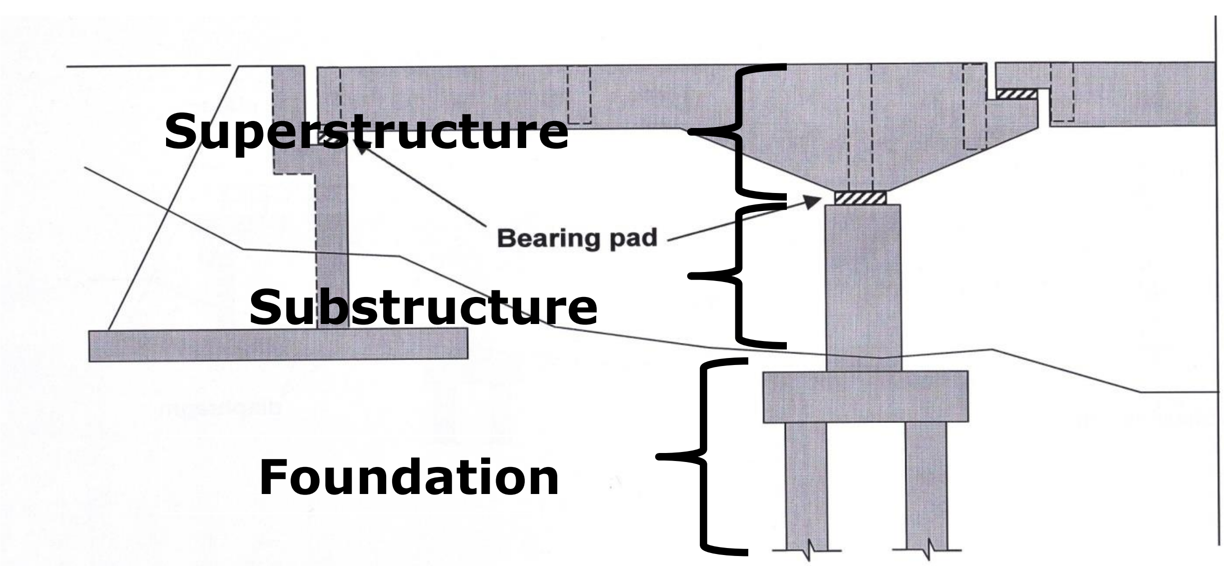


Fig. 7: Different parts of a Bridge

i) Components of a Bridge

Deck Slab

Girder

Diaphram or Cross Girder

Bearings for the decking

Abutment, Wingwall

Pier, Viaduct

Foundation (i.e.Pile)

Handrail, Curb/ Sidewalk

Approach to the Bridge *(to connect the bridge proper to the roads on either side*)

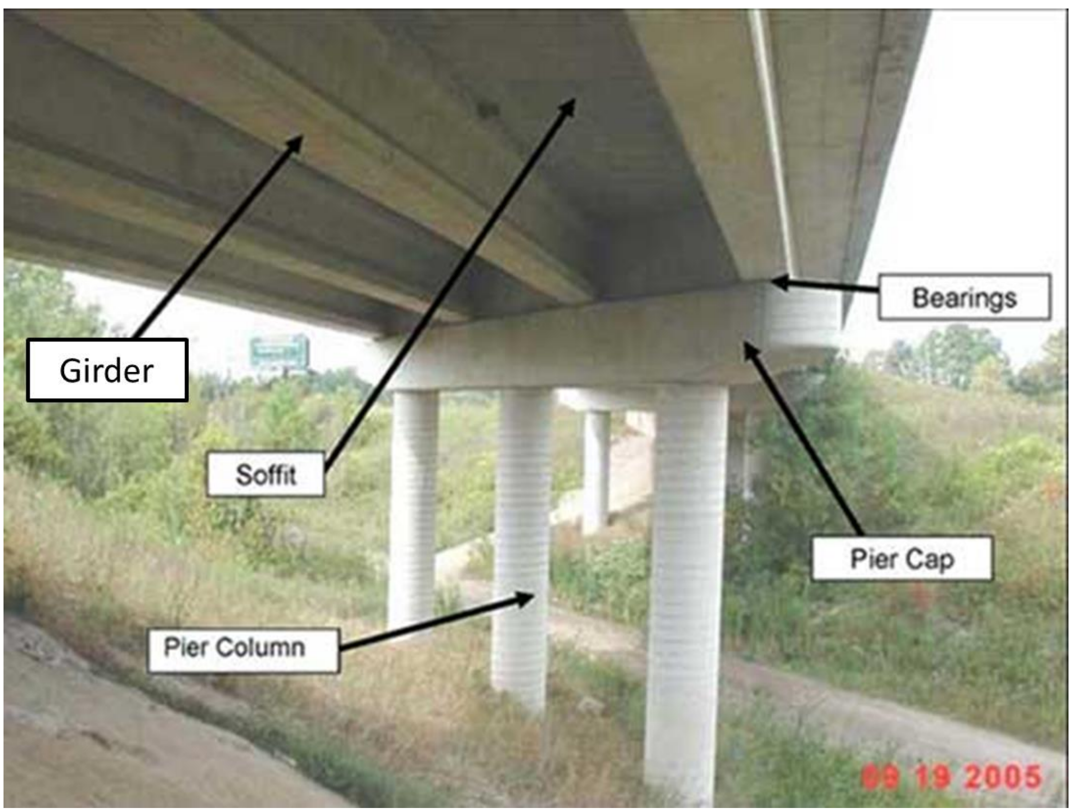


Fig. 8: Different components of a Bridge

j) BRIDGE TERMINOLOGY

Abutment

The end supports of the superstructure of a bridge.

Supports the bridge deck at the ends.

Retains the approach road embankment.

Wing walls

The walls constructed on both sides of the abutments.

Anchor the bridge to its approach road.

Support the embankments of approach road.

Protect the embankments from the wave action of running water.

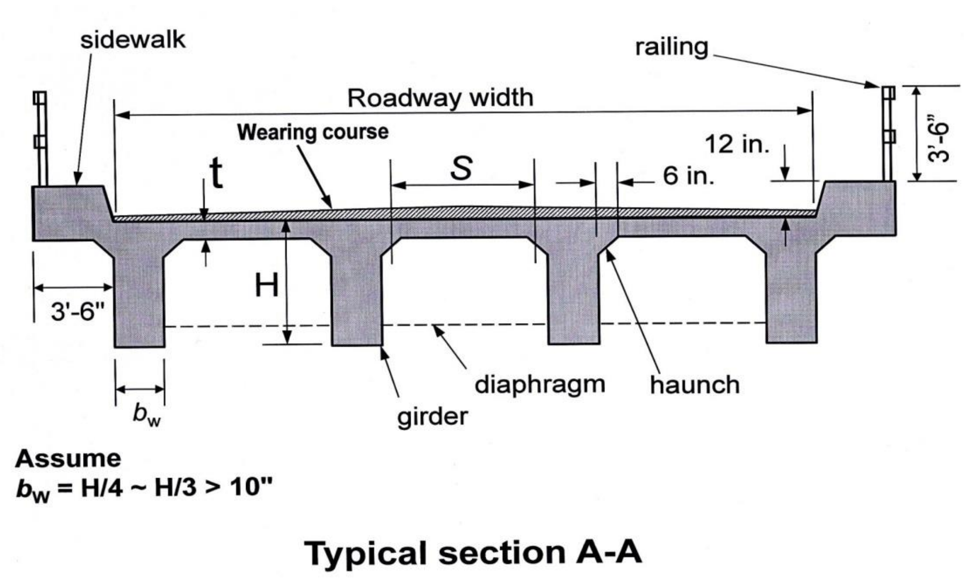


Fig. 10: Transverse section

Curb/ Sidewalk

Raised portion of a roadway slab on both sides.

Provided to check the vehicle to fall out the bridge.

Width of 60cm & Height of 22.5 cm are adopted.

Roadside slope is kept as 1 in 8 upto 20cm & top portion is curved.

Footpath

The passage where only pedestrians are allowed to walk.

Width may be taken as 1.5 to 2.2 metre.

Handrail

Protective measures adopted to prevent the falling to river of the bridge users.

Pier

Intermediate supports of the superstructure of a bridge.

Transfer load from the superstructure to the sub-soil through the foundation.

Obstruct the flow of water on the upstream.

Facilitate a long bridge to be converted into segments.

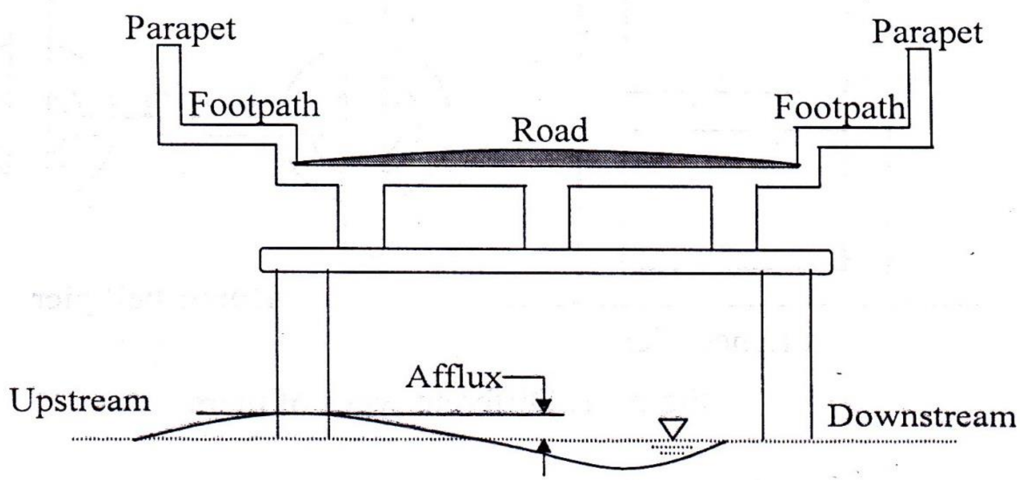


Fig. 11: Afflux

Afflux

The rise in water level of the river near bridge due to obstruction created by obstruction of piers.

Afflux = Difference of levels of downstream and upstream water surface of bridge.

Freeboard

The difference between the high flood level and the level of the crown of the road at its lowest point.

Approaches/ Embankments

The structures that carry the road or railway track up to the bridge.

Approach Slab

The slab provided to join the approach road with the bridge.

One end rests on the backfill of the abutment and extends into the approach at least by 3.5m.

Backfill

Materials used to fill the space at the back of the bridge.

They are the broken stone, gravel, sand etc. and should be clean.

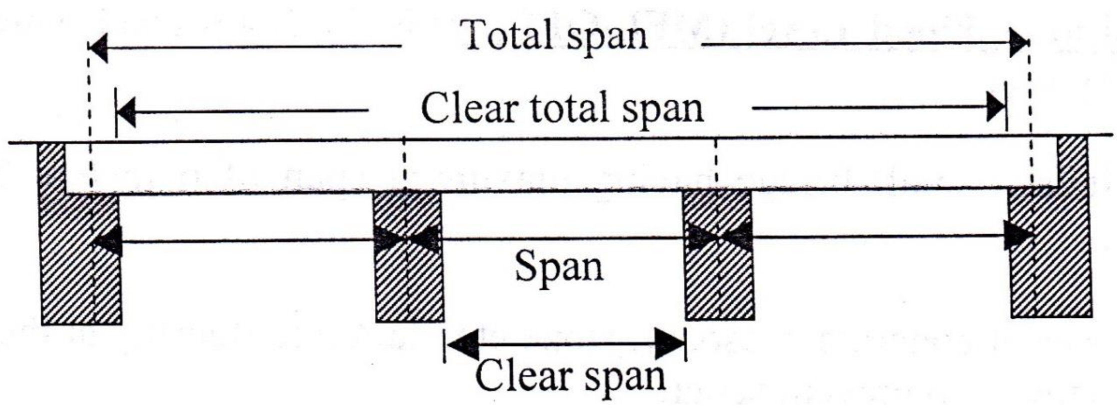


Fig. 12: Total span, total clear span, span and clear span

Total Span & Total Clear Span

The centre to centre distance between the end supports of a bridge is termed as total span. Clear distance between the end supports is termed as total clear span.

Span & Clear Span

The centre to centre distance between any two adjacent supports is termed as span. Clear distance between any two adjacent supports is termed as clear span.

Headroom

The distance between the highest point of the vehicle using that bridge and the lowest point of any protruding member of the bridge.

High Flood Level (HFL)

The highest water level ever recorded during a flood in a river or stream.

Low Flood Level (LFL)

The lowest water level in a river or stream during dry weather

Mean or Ordinary Flood Level (MFL)

The flood level that normally occurs every year.

k) Softwares for Bridge Design

SAP 2000

CSiBridge

ADAPT ABI 2012

Structural Bridge Design

CRSI (Slab Bridge Designer)

ANSYS Civil FEM Bridge

MIDAS

2.4 ABOUT BALANCED CANTILEVER BRIDGE

a) Multiple simply supported span bridge

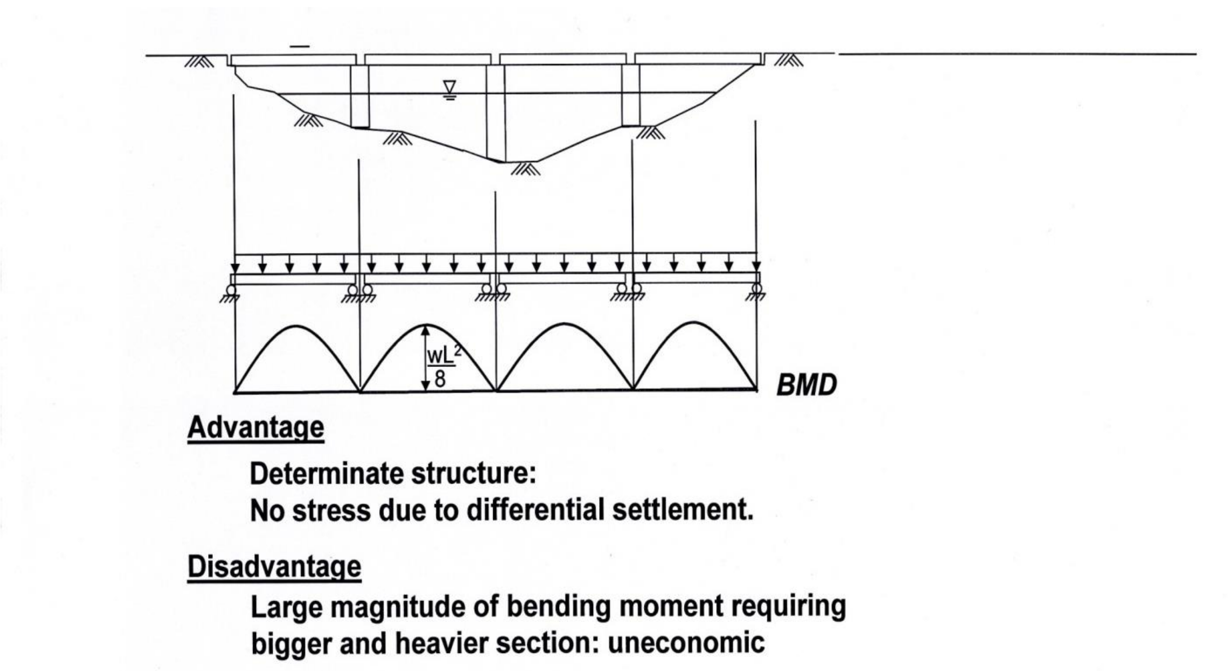


Fig. 13: A bridge having simply supported span

b) Continuous span bridge

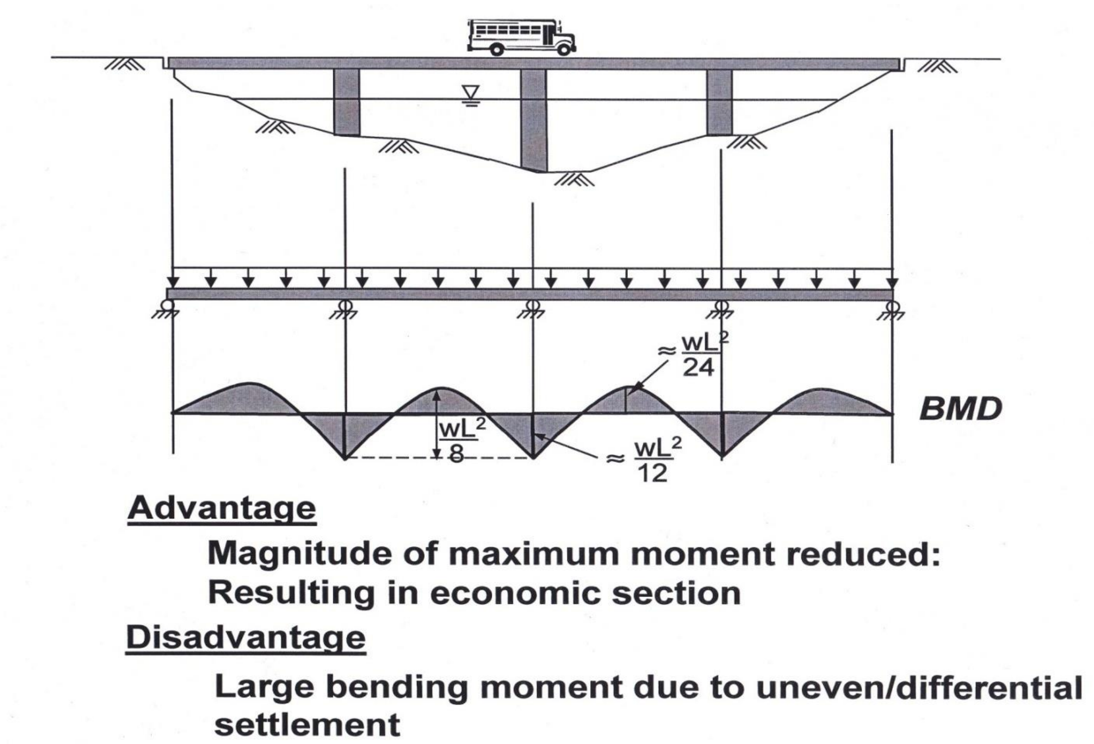


Fig. 14: A bridge having continuous span

c) What is a Balanced Cantilever Bridge?

- A cantilever bridge is a bridge built using cantilevers, structures that project horizontally into space, supported on only one end.
- The suspended span is designed as a simply supported span with supports at the articulations.
- A simple cantilever span is formed by two cantilever arms extending from opposite sides of an obstacle to be crossed.

d) Developing the idea of Cantilever form

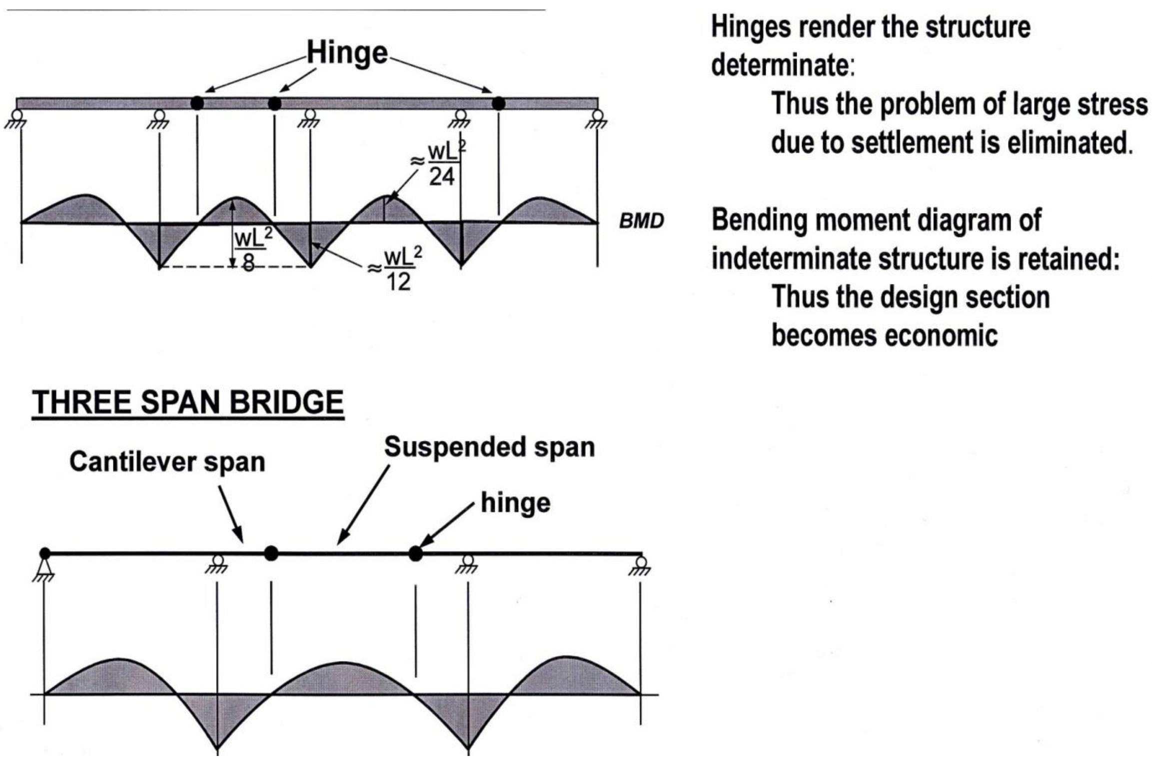


Fig. 15: A bridge having intermediate hinges

e) Advantages of Balanced Cantilever Bridge

Being a Determinate Structure.

The problem of large stress due to differential support settlement is eliminated due to the internal hinges.

The design section becomes economic.

Less concrete, steel are required for cantilever design.

f) Disadvantages of Balanced Cantilever Bridge

Requires a little more skill on the part of the designer.

Requires more elaborate detailing of the reinforcements.

Articulations are very congested with steel and anchorages.

2.5 DETAILS OF SOME EXISTING BRIDGES

a) World's largest Cantilever Bridge- Quebec Bridge, CANADA

Fig.16: **Quebec bridge, CANADA** (Source: online)

Total length: 987 m (3,239 ft)

Width: 29 m (94 ft) wide

Longest span : 549 m (1,800 ft)

Opened: December 3, 1919

Carries: 3 lanes of roadway

1 rail line

1 pedestrian walkway

Crosses: St. Lawrence River

b) Bangladesh China Friendship Bridge

- **Bridge Type :** Pre-stressed concrete box girder
- **Length :** 151 m (over river *Dhaleswari on Dhaka-Munshigonj road*)
- **Width :** 10 m (carriage way - 7.5 m & sidewalk - 2x1.25 m)
- **No. of Lanes :** 2 Lanes
- **No. of Span:** 37 nos.
- **No. of Abutment:** 2 nos.
- **No. of Piers:** 38 nos.
- **Type of Foundation :** Pile foundation

Fig.17: **Bangladesh China Friendship Bridge or Mukterpur Bridge, Bangladesh**

(Source: online)

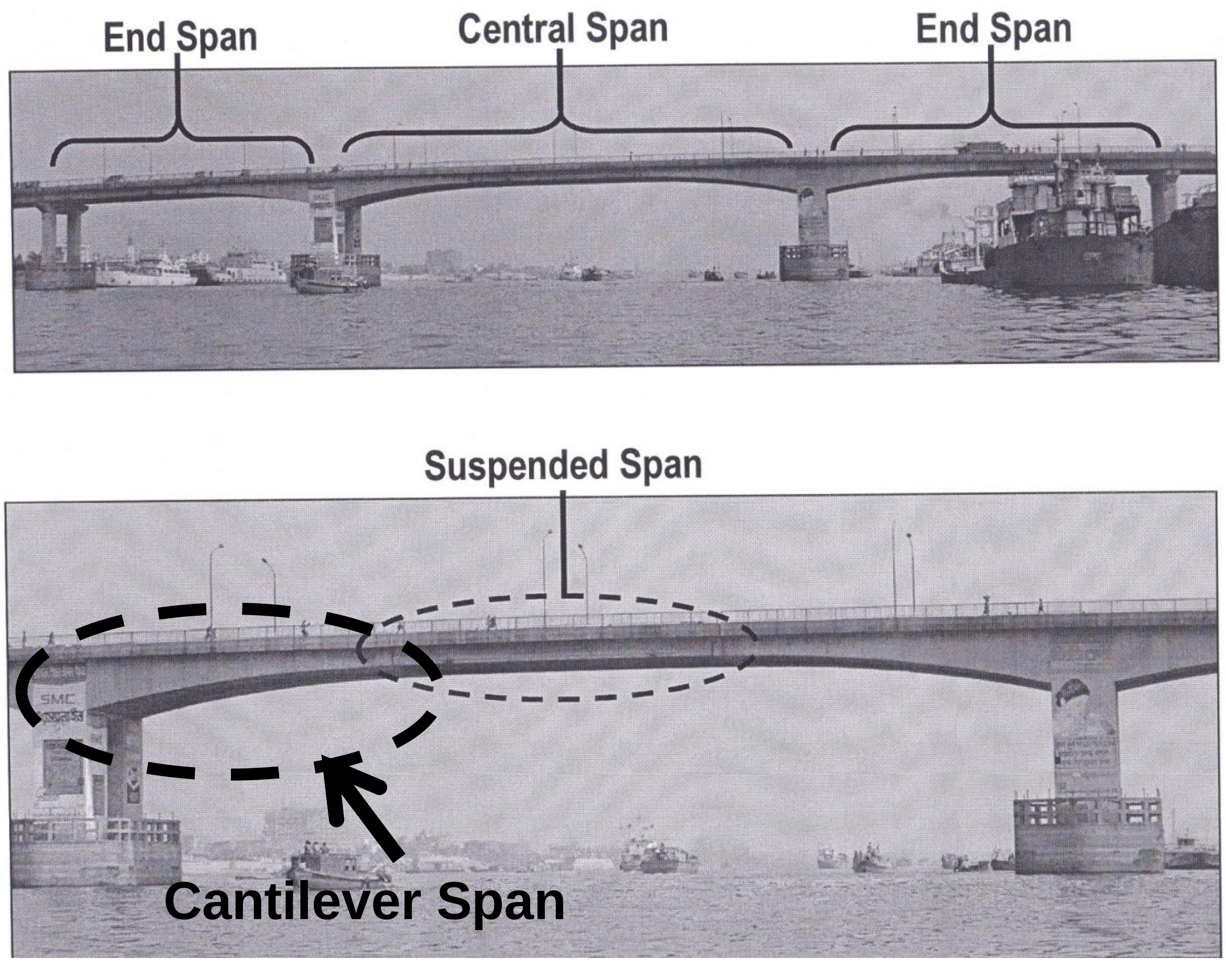


Fig.18: Spans of **Bangladesh China Friendship Bridge**

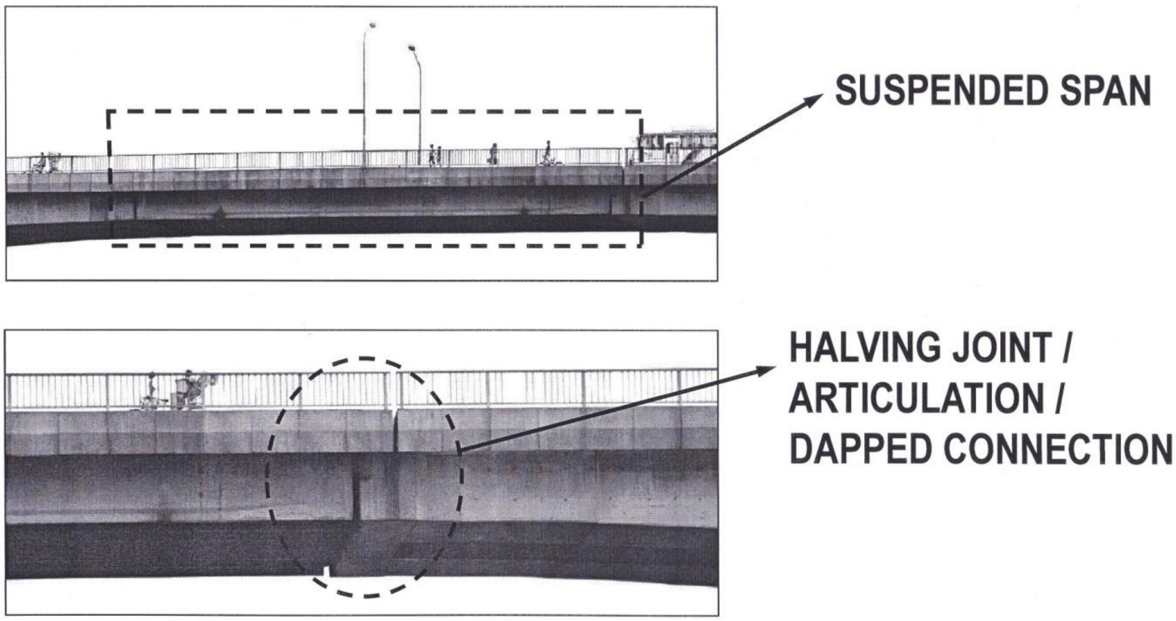


Fig.19: Articulation/ Halving joint

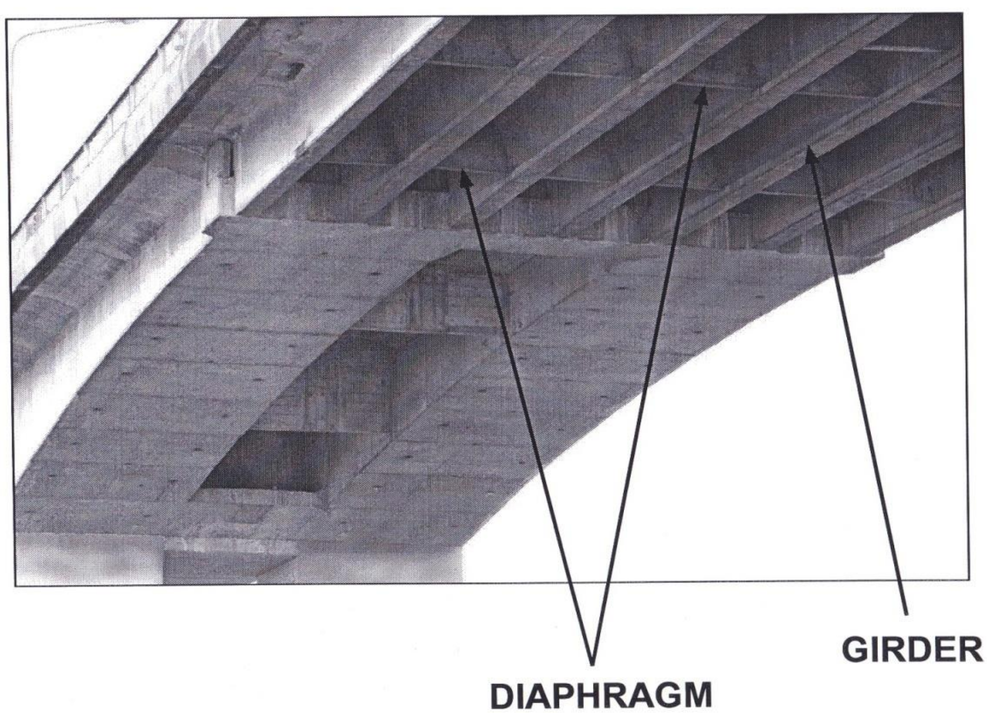


Fig.20: A back view showing diaphragm/cross girder and longitudinal girder

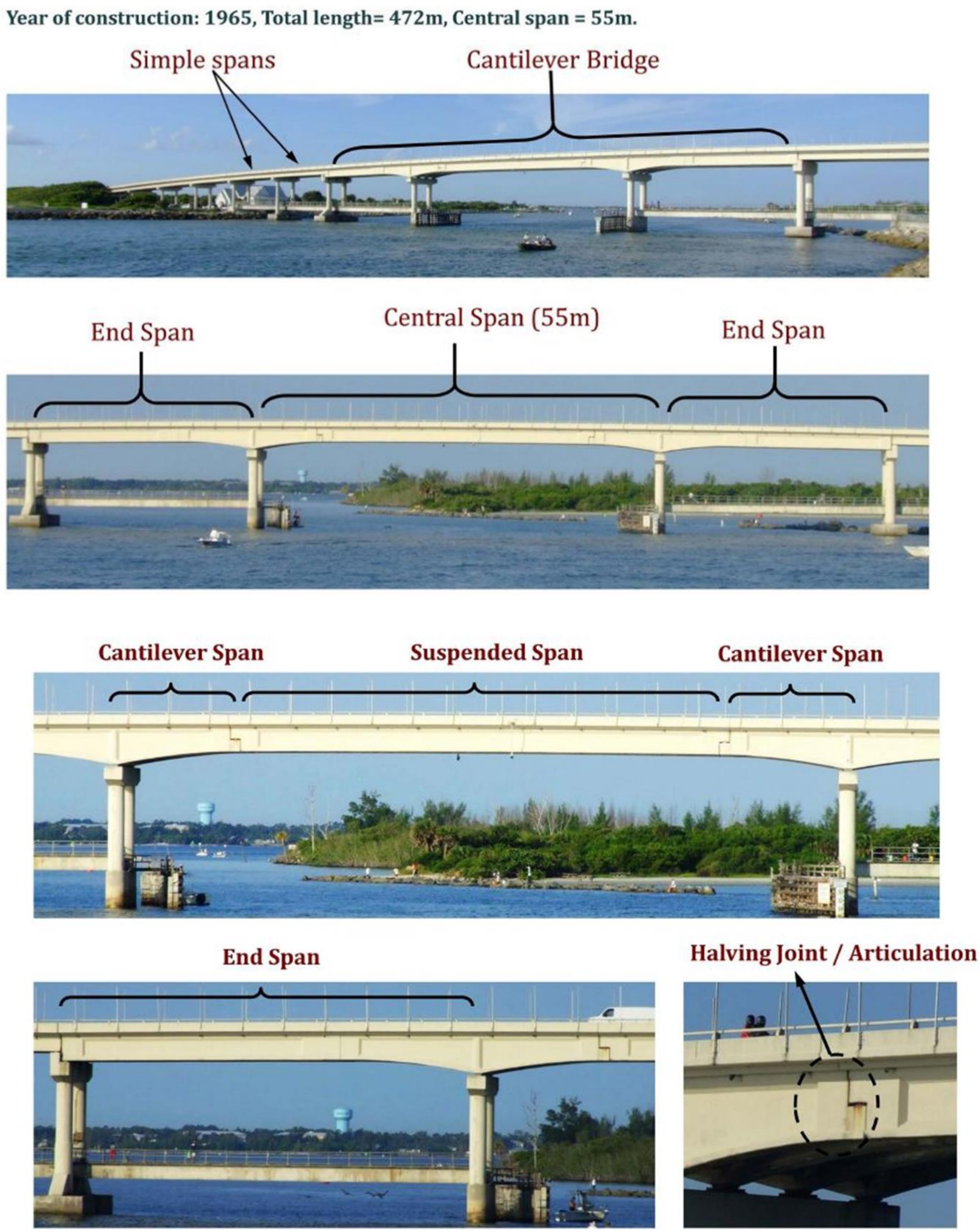


Fig. 21: Sebastian Intel Bridge, Florida, USA

Support Details

Fig. 22: Support details of Sebastian Intel Bridge, Florida, USA

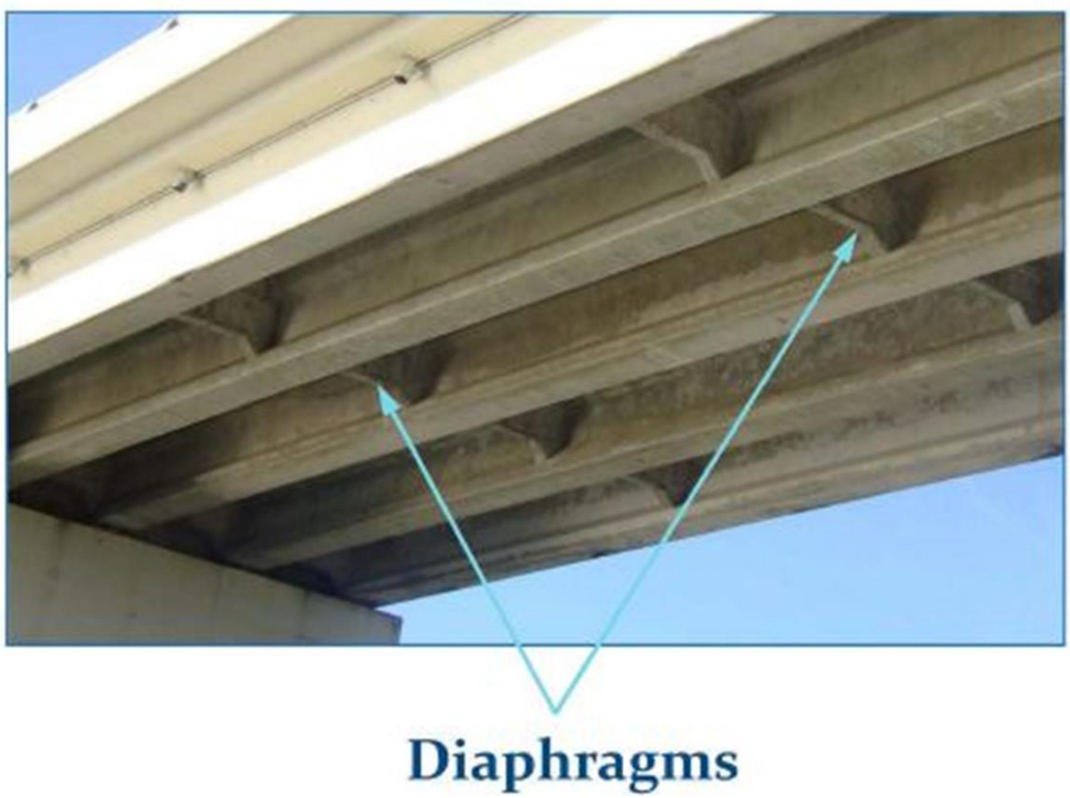


Fig. 23: Diaphram or cross girder of Sebastian Intel Bridge, Florida, USA

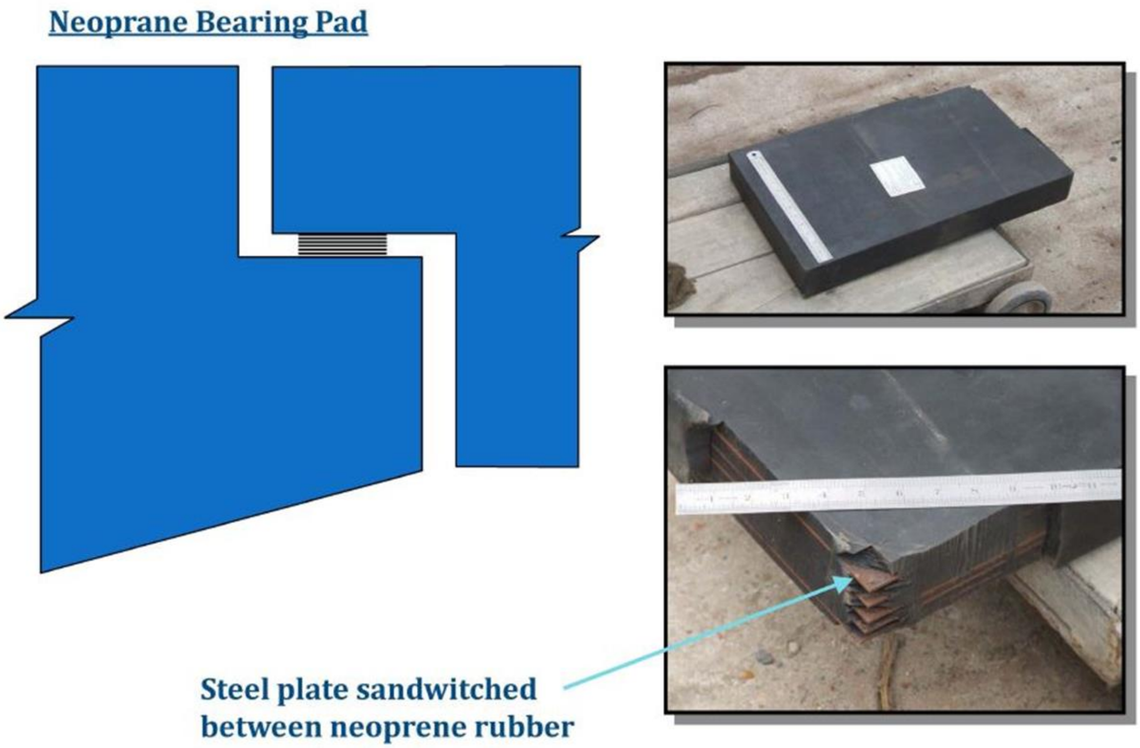


Fig. 24: Bearing Pad of Sebastian Intel Bridge, Florida, USA

2.6 LOADS ON BRIDGE

Dead load

Live load (i.e. Vehicles and Pedestrians)

Dynamic or Impact effect of live load

Wind loading

Seismic Forces

Buoyancy

Water current forces

Thermal Forces

Erection Forces

Earth Pressure

Centrifugal Forces (for curved deck)

Longitudinal Forces (for stopping vehicle)

Ice loading

Loads on Bridge (AASHTO 2012, Sec. 3.3.2)

The following permanent and transient loads and forces are considered to act on a bridge structure:

CR = force effects due to creep
DD = downdrag force
DC = dead load of structural components and nonstructural attachments
DW = dead load of wearing surfaces and utilities
EH = horizontal earth pressure load
EL = miscellaneous locked-in force effects resulting from the construction process, including jacking apart of cantilevers in segmental construction
ES = earth surcharge load
EV = vertical pressure from dead load of earth fill
PS = secondary forces from post-tensioning
SH = force effects due to shrinkage

BL = blast loading
BR = vehicular braking force
CE = vehicular centrifugal force
CT = vehicular collision force
CV = vessel collision force
EQ = earthquake load
FR = friction load
IC = ice load
IM = vehicular dynamic load allowance
LL = vehicular live load
LS = live load surcharge
PL = pedestrian live load
SE = force effect due to settlement
TG = force effect due to temperature gradient
TU = force effect due to uniform temperature
WA = water load and stream pressure
WL = wind on live load
WS = wind load on structure

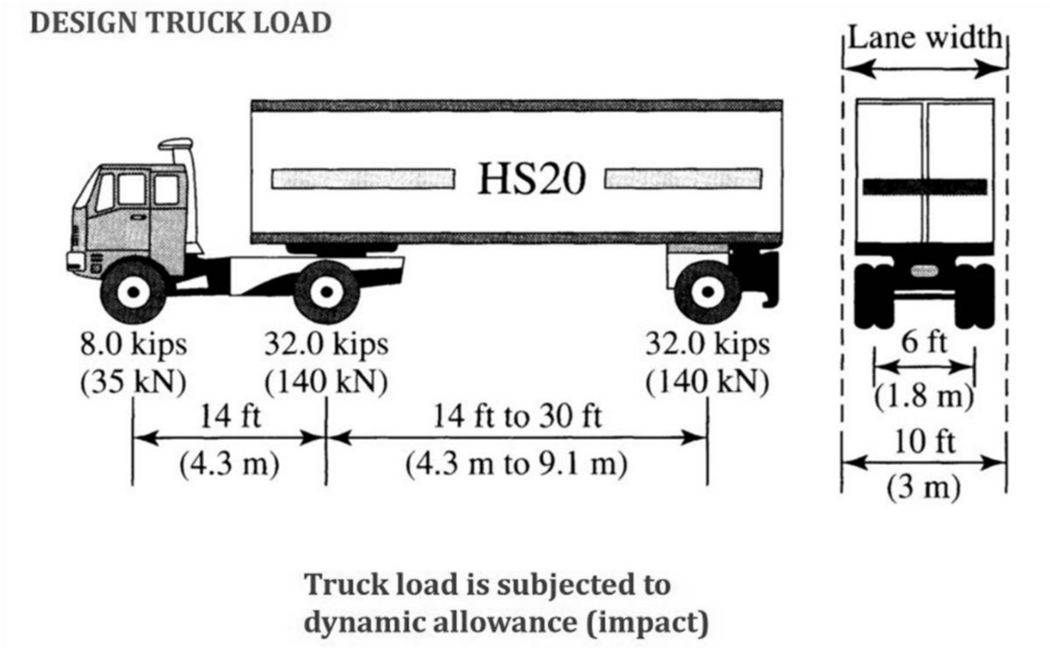


Fig.25: Design Truck load (HS20-516)

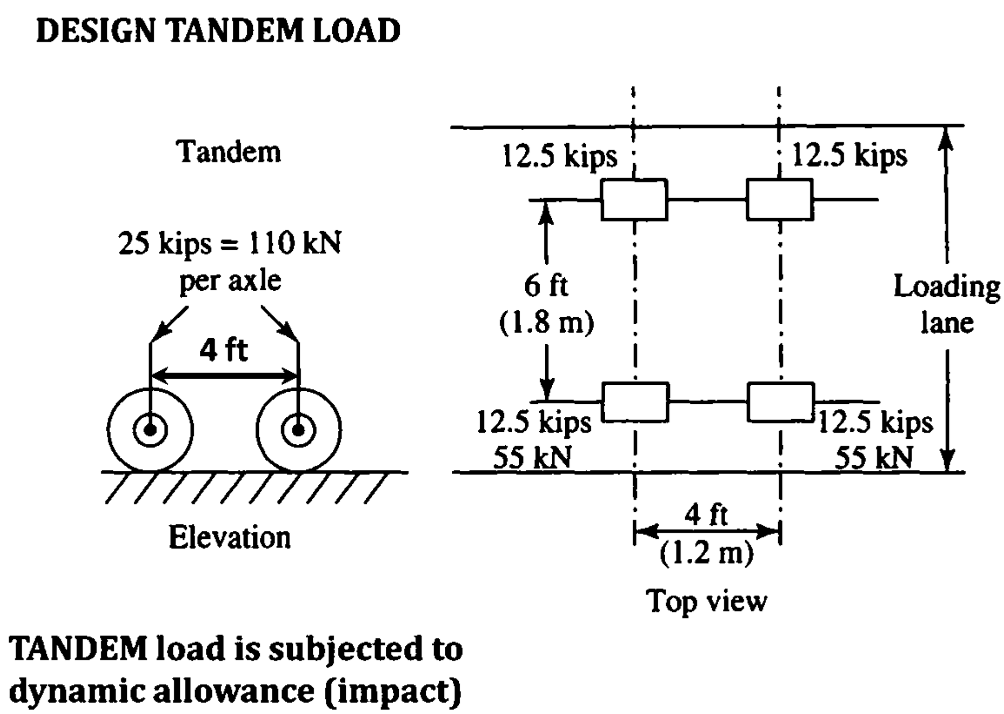
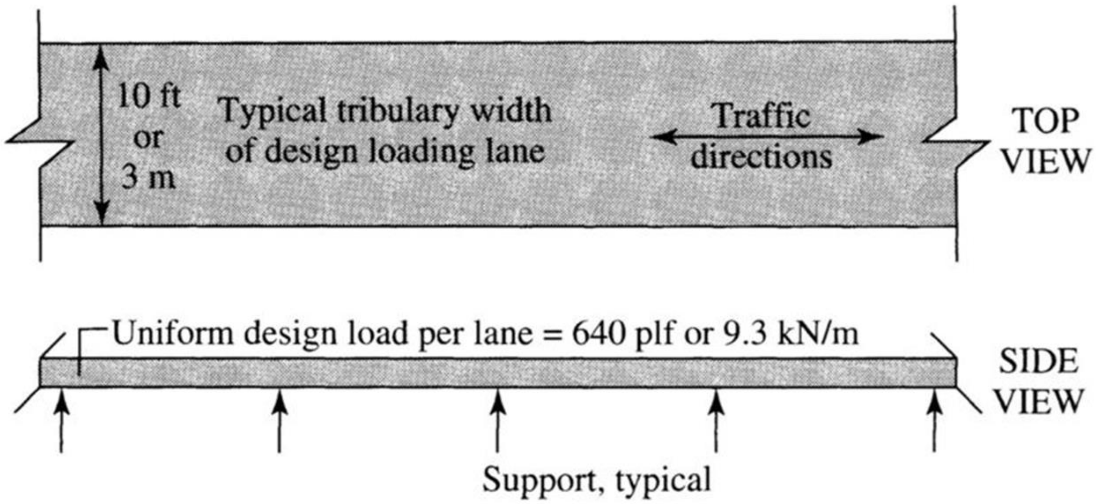


Fig.26: Design Tandem load

DESIGN LANE LOAD



LANE load is NOT subjected to dynamic allowance (impact)

Fig.27: Design Lane load

1. **Standard lane width: 12 ft, Load occupies 10 ft width across lane.**
2. **Fractional lanes not permitted.**
3. **For total bridge load: lane loads may be reduced as follows:**

1 or 2 lane bridge:	**No reduction**
3 lanes:	**90 percent**
4 or more lanes:	**75 percent**

DESIGN VEHICULAR LIVE LOAD

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

- **Design truck or design tandem, and**
- **Design lane load.**

Each design lane under consideration shall be occupied by either the design truck or tandem, coincident with the lane load, where applicable. The loads shall be assumed to occupy 10.0 ft transversely within a design lane.

Dynamic Effect of Live Load (for Truck or Tandem)
IMPACT ALLOWANCE

- ❑ **The term impact as ordinarily used in structural design refers to the dynamic effect of a suddenly applied load.**
- ❑ **In the building of a structure, the materials are added slowly; people entering a building are also considered a gradual loading. Dead loads are static loads; i.e., they have no effect other than weight.**
- ❑ **Live loads may be either static or they may have a dynamic effect. Any live load that can have a dynamic effect should be increased by an impact factor. While a dynamic analysis of a structure could be made, such a procedure is unnecessary in ordinary design. Thus, empirical formulas and impact factors are usually used.**
- ❑ **For highway bridge design, impact is always to be considered. AASHTO prescribes empirically that the static effect of live load be multiplied by a factor**

$$(1 + IM/100)$$

to take into account the dynamic effect of live load.

LIMIT STATES:

Strength I—Basic load combination relating to the normal vehicular use of the bridge without wind.

Strength II—Load combination relating to the use of the bridge by Owner-specified special design vehicles, evaluation permit vehicles, or both without wind.

Strength III—Load combination relating to the bridge exposed to wind velocity exceeding 55 mph.

Strength IV—Load combination relating to very high dead load to live load force effect ratios.

Strength V—Load combination relating to normal vehicular use of the bridge with wind of 55 mph velocity.

Extreme Event I—Load combination including earthquake. The load factor for live load γ_{EQ}, shall be determined on a project-specific basis.

Extreme Event II—Load combination relating to ice load, collision by vessels and vehicles, check floods, and certain hydraulic events with a reduced live load other than that which is part of the vehicular collision load, *CT*. The cases of check floods shall not be combined with *BL*, *CV*, *CT*, or *IC*.

Fatigue I—Fatigue and fracture load combination related to infinite load-induced fatigue life.

Fatigue II—Fatigue and fracture load combination related to finite load-induced fatigue life.

For the present case

DC **= Self weight of structural components**

DW **= Weight of wearing course**

LL **= Lane load with vehicle or tandem**

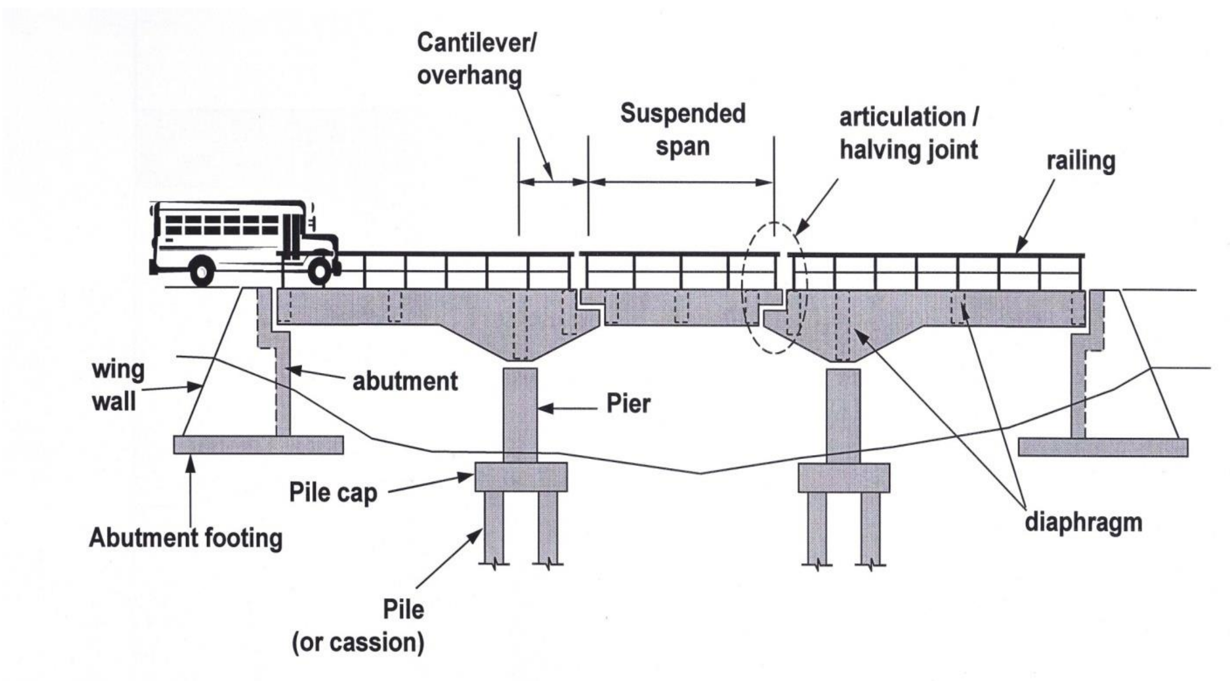
IM **= Impact effect of vehicle or tandem load**

PL **= Pedestrian load**

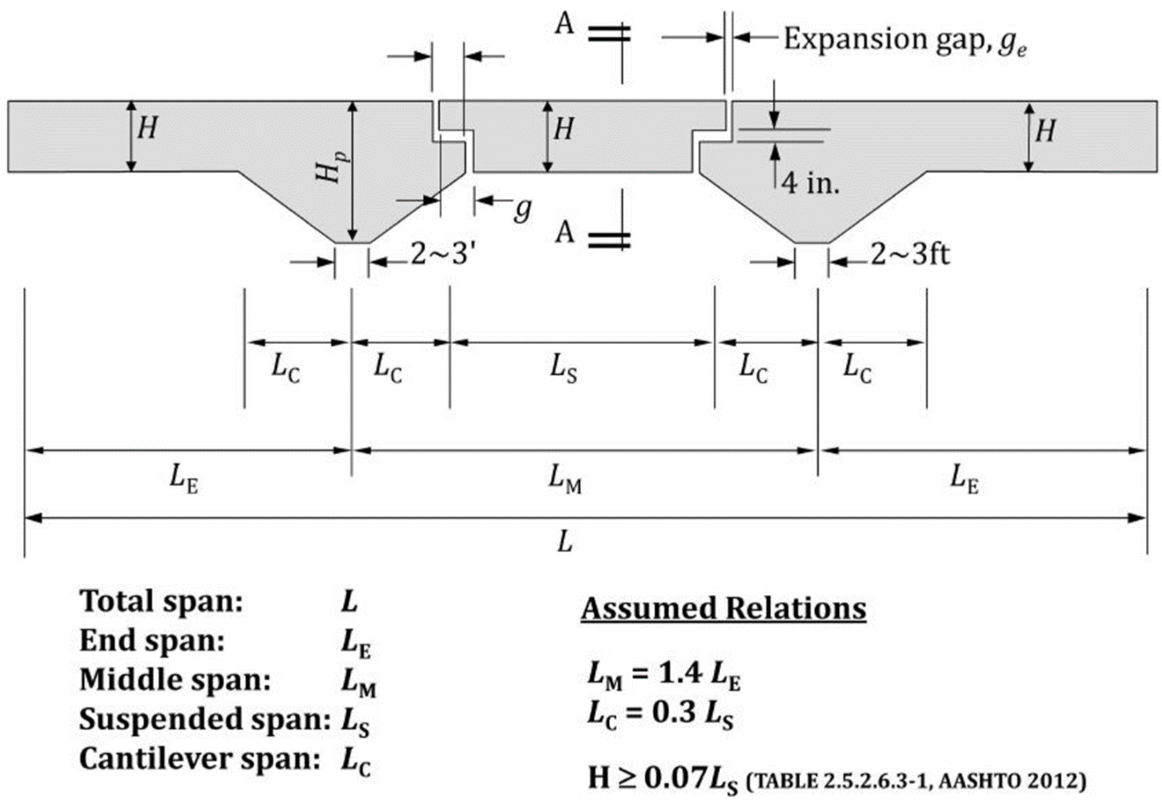
$$\gamma_p(DC) + \gamma_p(DW) + 1.75(LL)(1+IM/100)_{\text{Truck/Tandem}} + 1.75(LL)_{\text{Lane}} + 1.75(PL)$$

$$= 1.25(DC) + 1.5(DW) + 1.75(LL)(1+IM/100)_{\text{Truck/Tandem}} + 1.75(LL)_{\text{Lane}} + 1.75(PL)$$

2.7 DESIGN OF DIFFERENT COMPONENTS



(a)



(b)

Fig.28: Longitudinal profile a three spanned balanced cantilever bridge

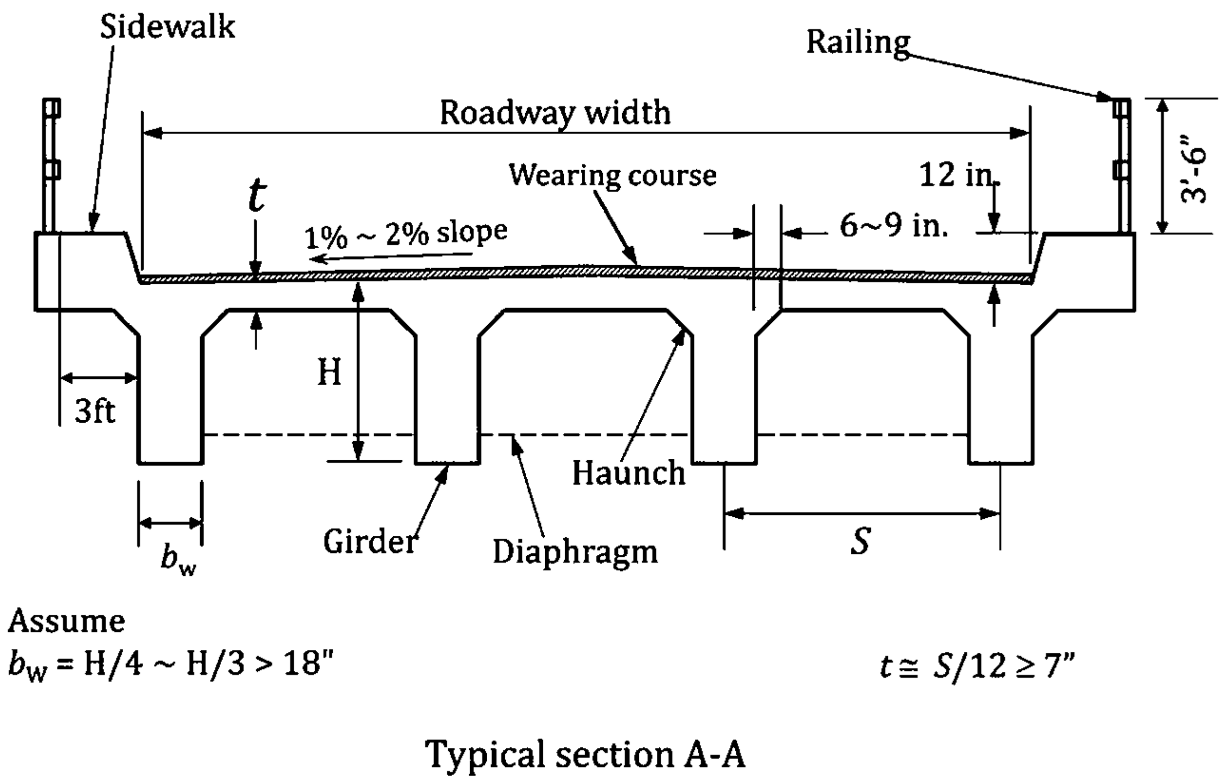


Fig. 29: Transverse section

Design Data for Students:

COMMON DATA
Wearing course, w_{wc} = 30 psf
Width of side walk = 3'-6"

DESIGN CODE
AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS, 6TH ED. 2012

Lane width

Sec-A	Sec-B	Sec-C
14'	13'	12'

Number of lanes = 2
Concrete clear cover = Beam 1.5", Slab: 1.0"

Girder depth at pier

H_p = 2.0H for L < 350',
= 1.5H for $L \geq$ 350'

PER STUDENT DATA

Student SI	Total Span, L ft	f_c' (ksi)	f_y (ksi)	Student SI	Total Span, L ft	f_c' (ksi)	f_y (ksi)	Student SI	Total Span, L ft	f_c' (ksi)	f_y (ksi)
1	250	4	72	24	319	5	72	47	388	4	60
2	253	4	72	25	322	5	72	48	391	4	60
3	256	4	60	26	325	5	72	49	394	4	60
4	259	4	60	27	328	5	72	50	397	4	60
5	262	4	60	28	331	5	72	51	400	4	60
6	265	4	60	29	334	5	72	52	403	5	72
7	268	4	60	30	337	5	72	53	406	5	72
8	271	4	60	31	340	5	72	54	409	5	72
9	274	4	60	32	343	5	72	55	412	5	72
10	277	4	60	33	346	5	72	56	415	5	72
11	280	4	60	34	349	5	72	57	418	5	72
12	283	4	60	35	352	4	60	58	421	5	72
13	286	4	60	36	355	4	60	59	424	5	72
14	289	4	60	37	358	4	60	60	427	5	72
15	292	4	60	38	361	4	60	61	430	5	72
16	295	4	60	39	364	4	60	62	433	5	72
17	298	4	60	40	367	4	60	63	436	5	72
18	301	5	72	41	370	4	60	64	439	5	72
19	304	5	72	42	373	4	60	65	442	5	72
20	307	5	72	43	376	4	60	66	445	5	72
21	310	5	72	44	379	4	60	67	448	5	72
22	313	5	72	45	382	4	60	68	451	5	72
23	316	5	72	46	385	4	60	69	454	5	72

Instructions for Students

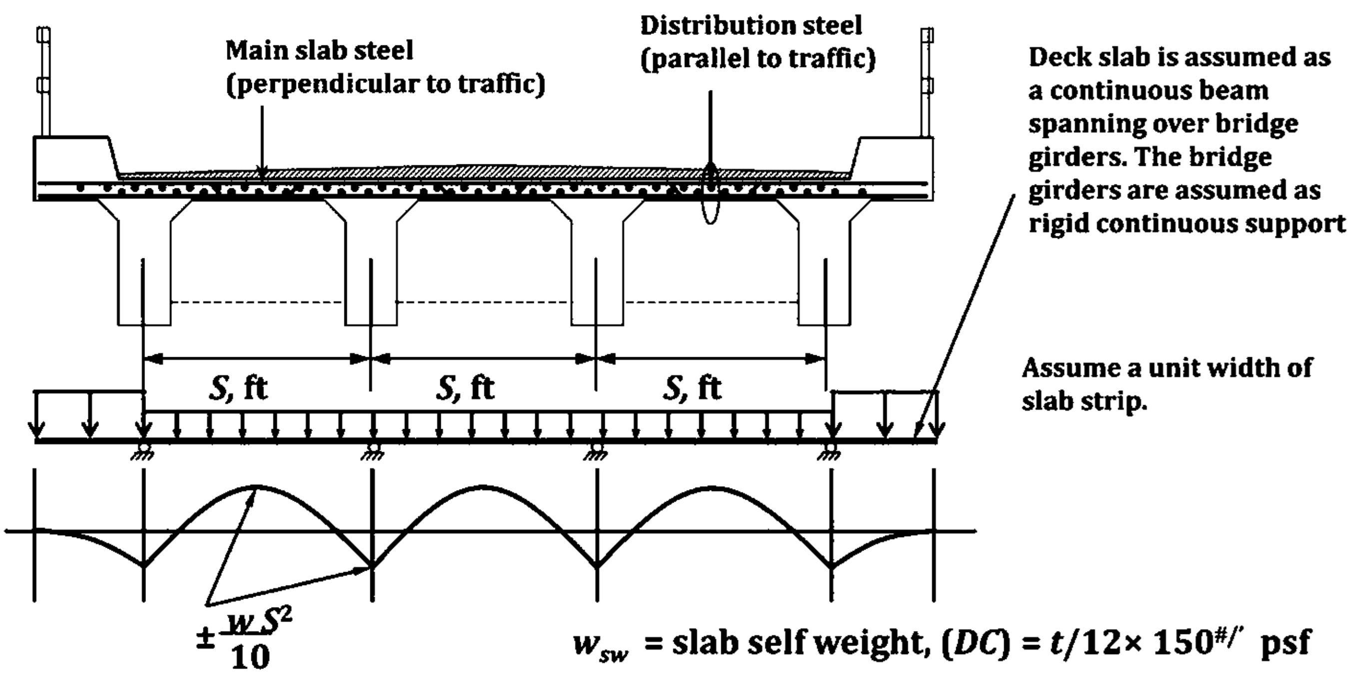
Follow the serial number of the students given in the previous table as starting from the smallest to upper student number for each section which will be provided in the class.

Draw SFD, BMD of interior girder due to dead load and also verify those results using software.

Draw influence line diagram for shear and moment at the assigned sections and also verify them using software for at least three sections.

a) DESIGN OF DECK SLAB

Design for Dead Load



w_{sw} = slab self weight, $(DC) = t/12 \times 150^{\#/'}$ psf

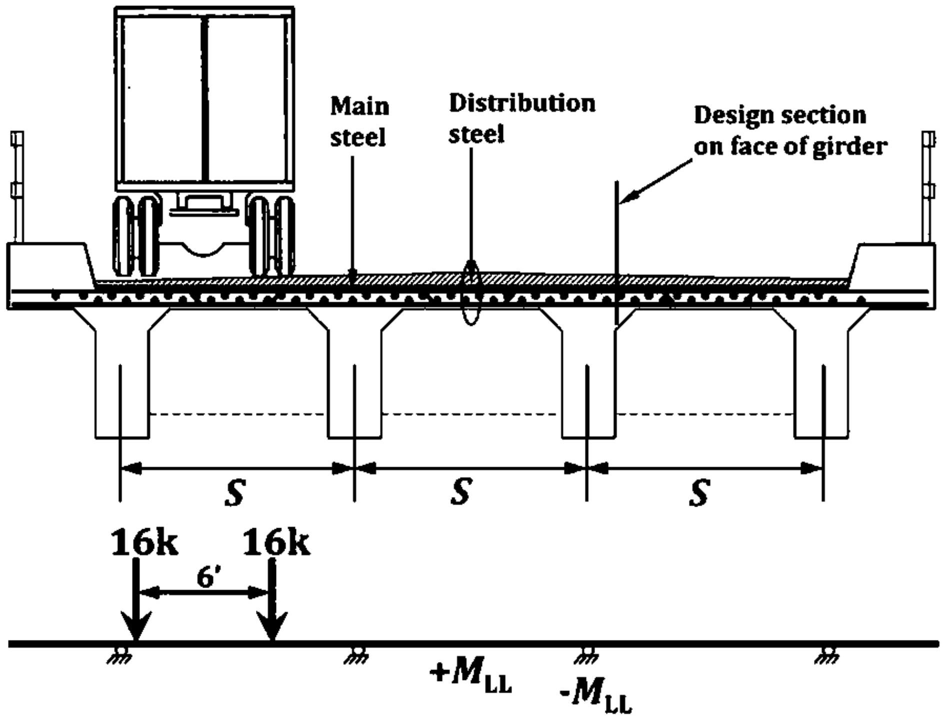
w_{wc} = wearing course, (DW) psf

$M_{DC} = w_{sw} \times S^2/10$ lb-ft

$M_{DW} = w_{wc} \times S^2/10$ lb-ft

Fig. 30: Dead load on deck slab

Deign for Vehicular Live load



Detailed analysis can be performed based on influence line to determine the maximum effect.
Alternatively, Table A4-1 in Appendix A4 of AASHTO 2012 can be used.

Fig. 31: Vehicular live load on deck slab

Table A4-1 in Appendix A4 of AASHTO 2012, page 4-98
Important Assumptions…

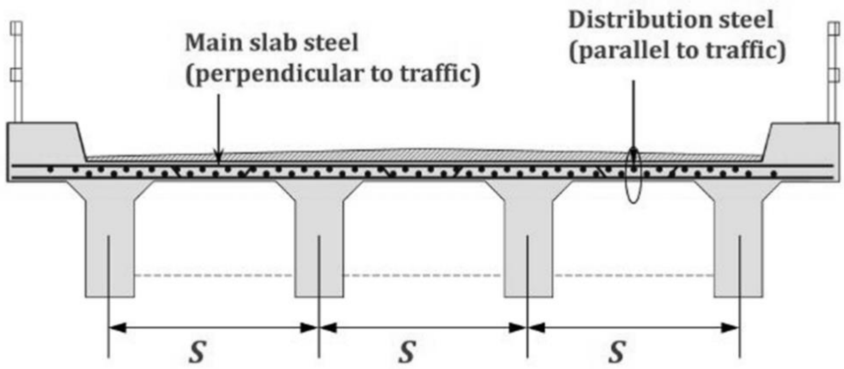
- **Multiple presence factors and the dynamic load allowance are included in the tabulated values.**
- **The moments are applicable for decks supported on at least three girders and having a width of not less than 14.0 ft between the centerlines of the exterior girders.**
- **For each combination of girder spacing and number of girders, the following two cases of overhang width were considered:**
  - **Minimum total overhang width of 21.0 in. measured from the center of the exterior girder, and**
  - **Maximum total overhang width equal to the smaller of 0.625 times the girder spacing and 6.0 ft. A railing system width of 21.0 in. was used to determine the clear overhang width. For other widths of railing systems, the difference in the moments in the interior regions of the deck is expected to be within the acceptable limits for practical design. The moments do not apply to the deck overhangs and the adjacent regions of the deck that need to be designed taking into account the provisions of Article A13.4.1.**

DECK SLAB DESIGN: VEHICLE LOAD

4-98 AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS

Table A4-1—Maximum Live Load Moments per Unit Width, kip-ft/ft

		Negative Moment						
		Distance from CL of Girder to Design Section for Negative Moment						
S	Positive Moment	0.0 in.	3 in.	6 in.	9 in.	12 in.	18 in.	24 in.
7' –0"	5.21	5.98	5.17	4.36	3.56	2.84	1.63	1.37
7' –3"	5.32	6.13	5.31	4.49	3.68	2.96	1.65	1.51
7' –6"	5.44	6.26	5.43	4.61	3.78	3.15	1.88	1.72
7' –9"	5.56	6.38	5.54	4.71	3.88	3.30	2.21	1.94
8' –0"	5.69	6.48	5.65	4.81	3.98	3.43	2.49	2.16
8' –3"	5.83	6.58	5.74	4.90	4.06	3.53	2.74	2.37
8' –6"	5.99	6.66	5.82	4.98	4.14	3.61	2.96	2.58
8' –9"	6.14	6.74	5.90	5.06	4.22	3.67	3.15	2.79
9' –0"	6.29	6.81	5.97	5.13	4.28	3.71	3.31	3.00
9' –3"	6.44	6.87	6.03	5.19	4.40	3.82	3.47	3.20
9' –6"	6.59	7.15	6.31	5.46	4.66	4.04	3.68	3.39
9' –9"	6.74	7.51	6.65	5.80	4.94	4.21	3.89	3.58
10' –0"	6.89	7.85	6.99	6.13	5.26	4.41	4.09	3.77
10' –3"	7.03	8.19	7.32	6.45	5.58	4.71	4.29	3.96
10' –6"	7.17	8.52	7.64	6.77	5.89	5.02	4.48	4.15
10' –9"	7.32	8.83	7.95	7.08	6.20	5.32	4.68	4.34



General Load Combination
$1.25(DC) + 1.5(DW) + 1.75(LL)(1+IM/100)_{Truck/Tandem} + 1.75(LL)_{Lane} + 1.75(PL)$

Design slab moment, $M = 1.25\ M_{DC} + 1.5\ M_{DW} + 1.75\ M_{LL}$ [$\rightarrow M_{STRENGTH}$]

Where M_{LL} is the live load slab moment from Table A4-1 which includes the impact effect.

$M_{SERVICE} = M_{DC} + M_{DW} + M_{LL}$ [required for crack control calculations]

Reinforcement Design of Deck

Resistance factor φ
Moment 0.90

Deck Slab

$$A_s \geq \frac{M_u}{\phi f_y\left(d-\frac{a}{2}\right)} \approx \frac{M_u}{\phi f_y(jd)}$$

Assume $jd \approx 0.95d$

Check $A_s \geq A_{s,\min} = \dfrac{200}{f_y} bd$

Determine, $a = \dfrac{A_s f_y}{0.85 f_c' b}$

Revise, $A_s = \dfrac{M_u}{\phi f_y\left(d-\frac{a}{2}\right)}$

Determine, β_1

Depth of neutral axis, $c = a/\beta_1$

Check $c < {}^{3}/_{8}\, d$ (tension controlled)

Finally, $\phi M_n = \phi A_s f_y\left(d-\dfrac{a}{2}\right) \geq M_u$

FIGURE 3.9
Variation of strength reduction factor with net tensile strain in the steel.

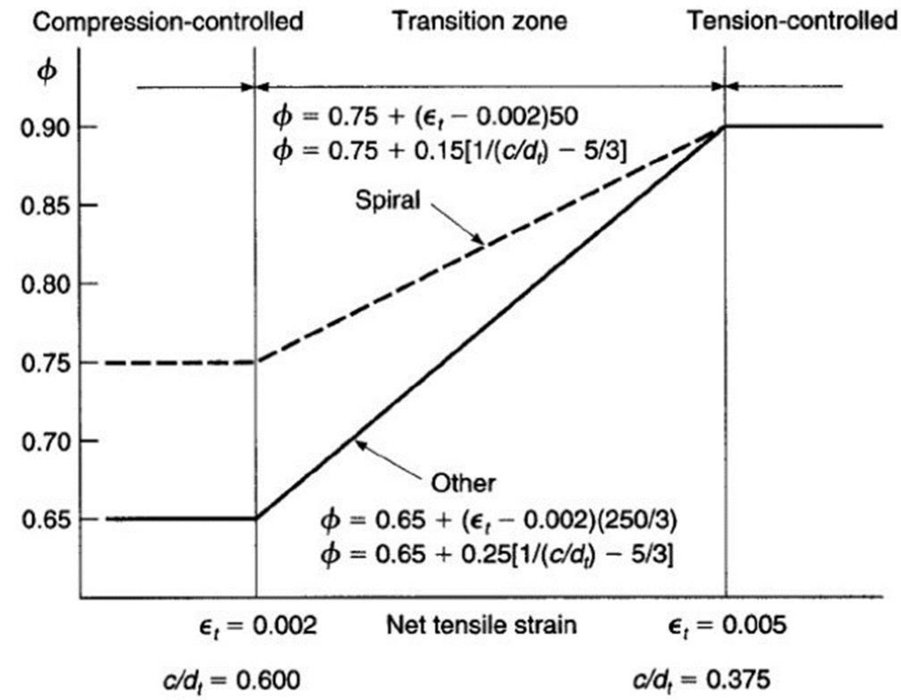
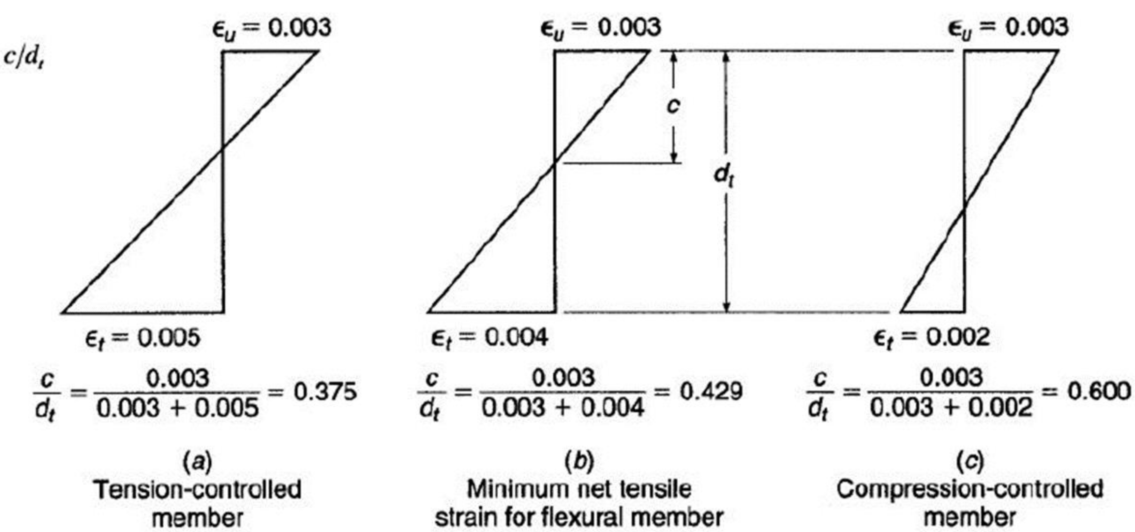


FIGURE 3.10
Net tensile strain and c/d_t ratios.



Control of Cracking by Distribution of Reinforcement
(Sec. 5.7.3.4 AASHTO 2012)

The spacing s of mild steel reinforcement in the layer closest to the tension face shall satisfy the following:

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

$$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)}$$

Actual spacing of steel shall not be more than s calculated above.

γ_e = exposure factor
= $\boxed{\text{1.00 for Class 1 exposure condition}}$
= 0.75 for Class 2 exposure condition

d_c = thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto (in.)

f_{ss} = tensile stress in steel reinforcement at the service limit state (ksi)

h = overall thickness or depth of the component (in.)

Assume $f_{ss} = f_y \times (M_{SERVICE}/M_{STRENGTH})$

Shrinkage & Temperature Reinforcement of Deck (AASHTO 2012, Art 5.10.8)

Reinforcement for shrinkage and temperature stresses shall be provided near surfaces of concrete exposed to daily temperature changes and in structural mass concrete. Temperature and shrinkage reinforcement is provided to ensure that the total reinforcement on exposed surfaces is not less than that specified herein.

For bars or welded wire fabric, the area of reinforcement per foot, on each face and in each direction, shall satisfy:

$$A_s \geq \frac{1.30bh}{2(b+h)f_y} \quad (5.10.8\text{-}1)$$

$$0.11 \leq A_s \leq 0.60 \quad (5.10.8\text{-}2)$$

A_s = area of reinforcement in each direction and each face (in.^{2}/ft)

b = least width of component section (in.)

h = least thickness of component section (in.)

f_y = specified yield strength of reinforcing bars $\leq$75 ksi

Where the least dimension varies along the length of wall, footing, or other component, multiple sections should be examined to represent the average condition at each section.

Spacing shall not exceed:

- ❑ 3.0 times the component thickness, or 18.0 in.
- ❑ 12.0 in. for walls and footings greater than 18.0 in. thick
- ❑ 12.0 in. for other components greater than 36.0 in. thick

For components 6.0 in. or less in thickness the minimum steel specified may be placed in a single layer. Shrinkage and temperature steel shall not be required for:

- ❑ End face of walls 18 in. or less in thickness.
- ❑ Side faces of buried footings 36 in. or less in thickness
- ❑ Faces of all other components, with smaller dimension less than or equal to 18.0 in.

Distribution Reinforcement of Deck
(AASHTO 2012, Art 5.10.8)

Reinforcement shall be placed in the secondary direction in the bottom of slabs as a percentage of the primary reinforcement for positive moment as follows.

For primary reinforcement parallel to traffic:

$100/\sqrt{S_c} \leq 50\%$

For primary reinforcement perpendicular to traffic:

$220/\sqrt{S_c} \leq 67\%$

where:

S_c = the effective span length of slab taken as equal to the effective length specified in Article 9.7.2.3 (ft) = clear distance between the girders.

Reinforcement Detailing of Deck Slab

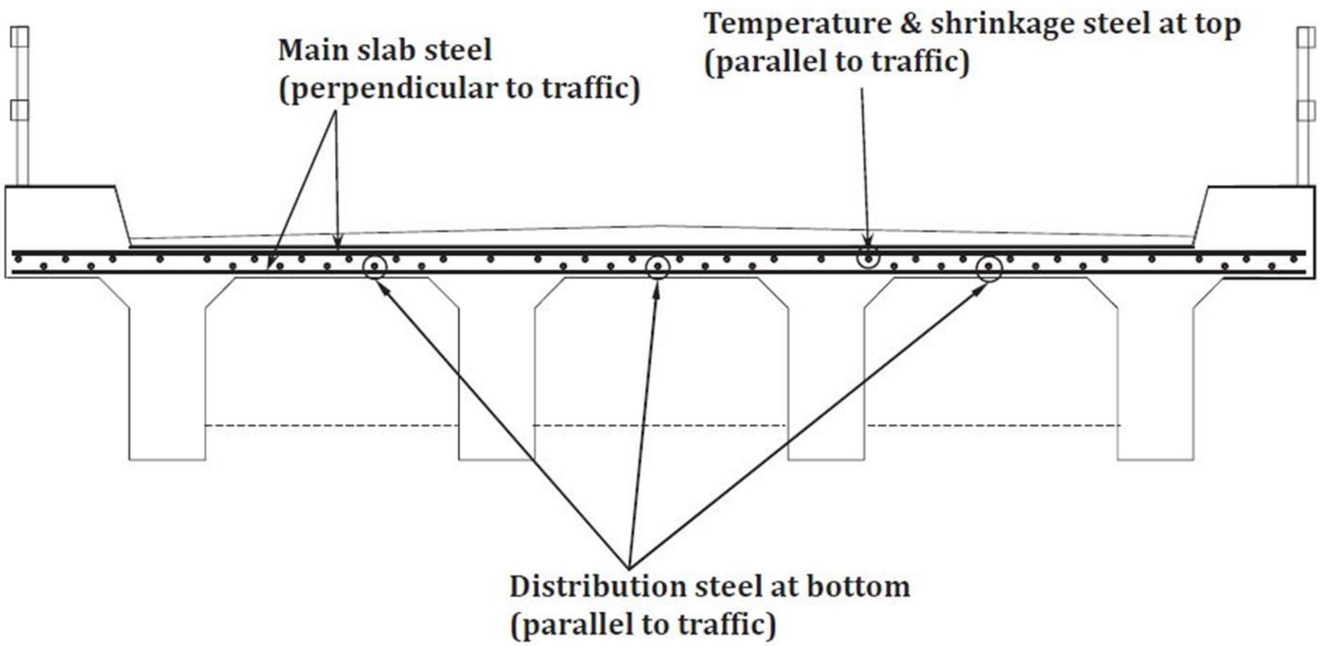


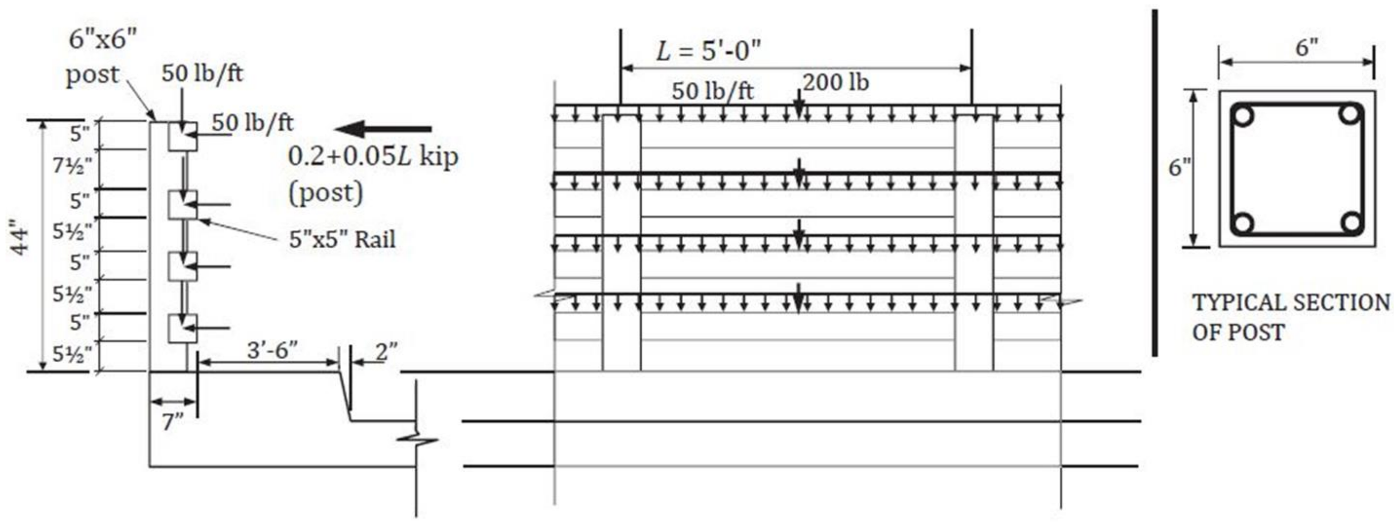
Fig.32: Reinforcement detailing of Slab

b) DEIGN OF RAILING

Minimum height of rail post : 42 inch [Sec. 13.8.2]

Opening between rails shall be less than 6 inch for portion 27 inch vertically from walkway surface.

Opening between rails shall be less than 8 inch for portion above 27 inch from walkway surface.



Each railing shall be designed for 50 lb/ft uniformly distributed load acting simultaneously in both vertical and horizontal direction.

Fig.33 : Side view and elevation view of railing and post

- Each railing shall be designed for 50 lb/ft uniformly distributed live load acting simultaneously in both vertical and horizontal direction.
- Opening between rails < 6 inch for portion 27 in. vertically from walkway surface.
- Opening between rails < 8 inch for portion above 27 in. from walkway surface.

Design Steps:

- Assume, 5in. x 5in. Railing
- Consider Live load on each railing = 50lb/ft
- Determine Dead load per unit length
- Determine total load w_T per unit length
- Determine Maximum Moment = $1/10\ w_T\, l^2$
- Determine steel Area A_s .

c) DESIGN OF CURB / SIDEWALK

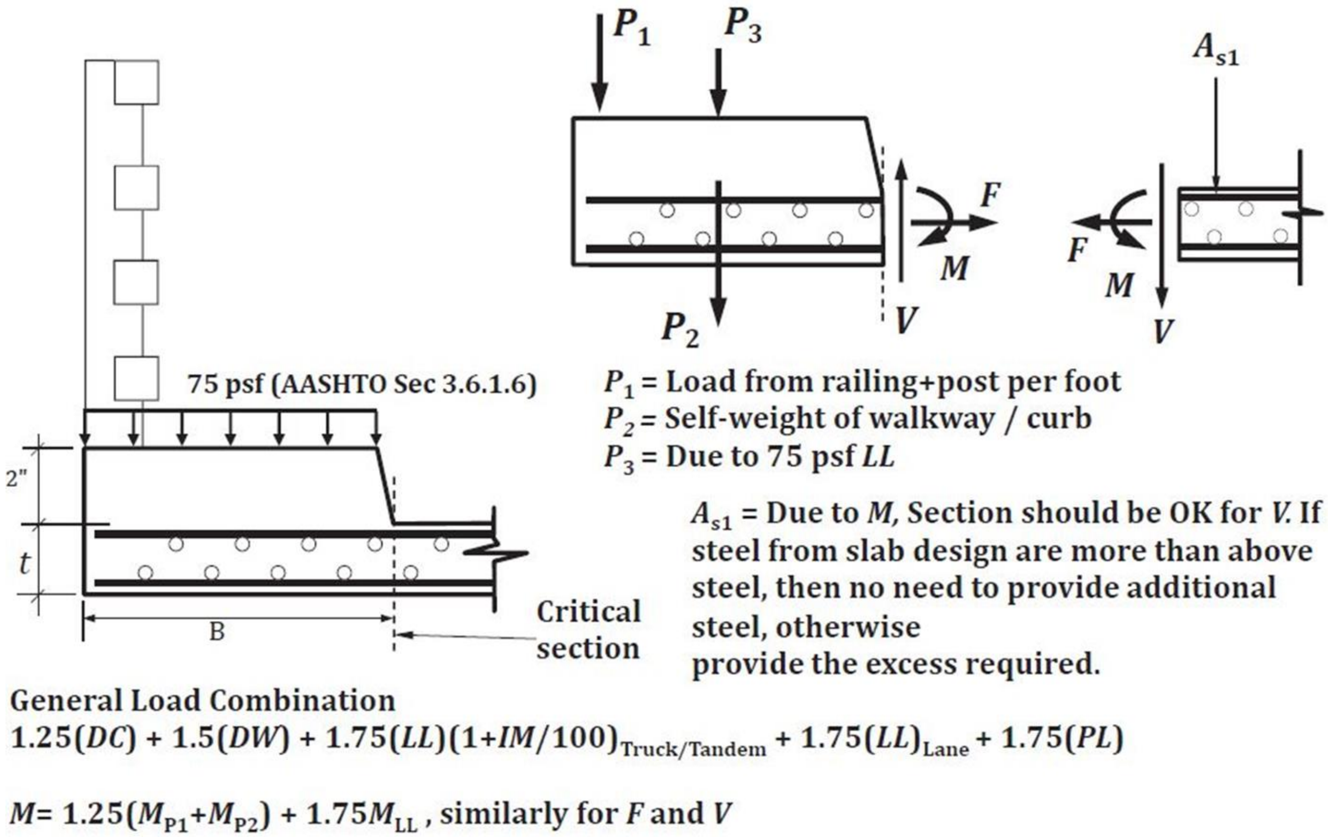
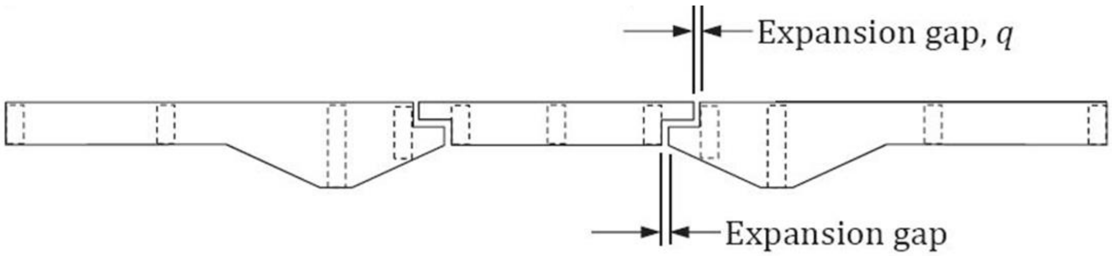


Fig. 34: Loads on Curb or sidewalk

- Determine $P_1, P_2, P_3, P_4.$
- Determine bending moment M at critical section
- Determine steel area, A_{s1} due to M

Expansion gap Determination



Expansion gap is required to accommodate the thermal expansion-contraction. In Bangladesh seasonal temperature varies between 5 °C to 40 °C. For the purpose of design we take Δt = 40 °C.
Thermal expansion co-efficient of concrete α_c = 0.00001 / °C.
Therefore, maximum expansion/contraction shall be
$\Delta L = \alpha_c(\Delta t)L$ where L is the length under consideration.
Total expansion may be divided at the two expansion gaps at the ends of the suspended span. Also we shall maintain a minimum of 1 inch gap in the event of extreme condition.

Thus, if L is the total span of the bridge and we confine the expansion/contraction only at the ends of suspended span then

$q = \alpha_c \Delta t\ (L/2) + 1 = 0.00001 \times 40 \times L/2$ (inch) + 1 [rounded to higher ½ inch]

d) DESIGN OF INTERIOR GIRDER

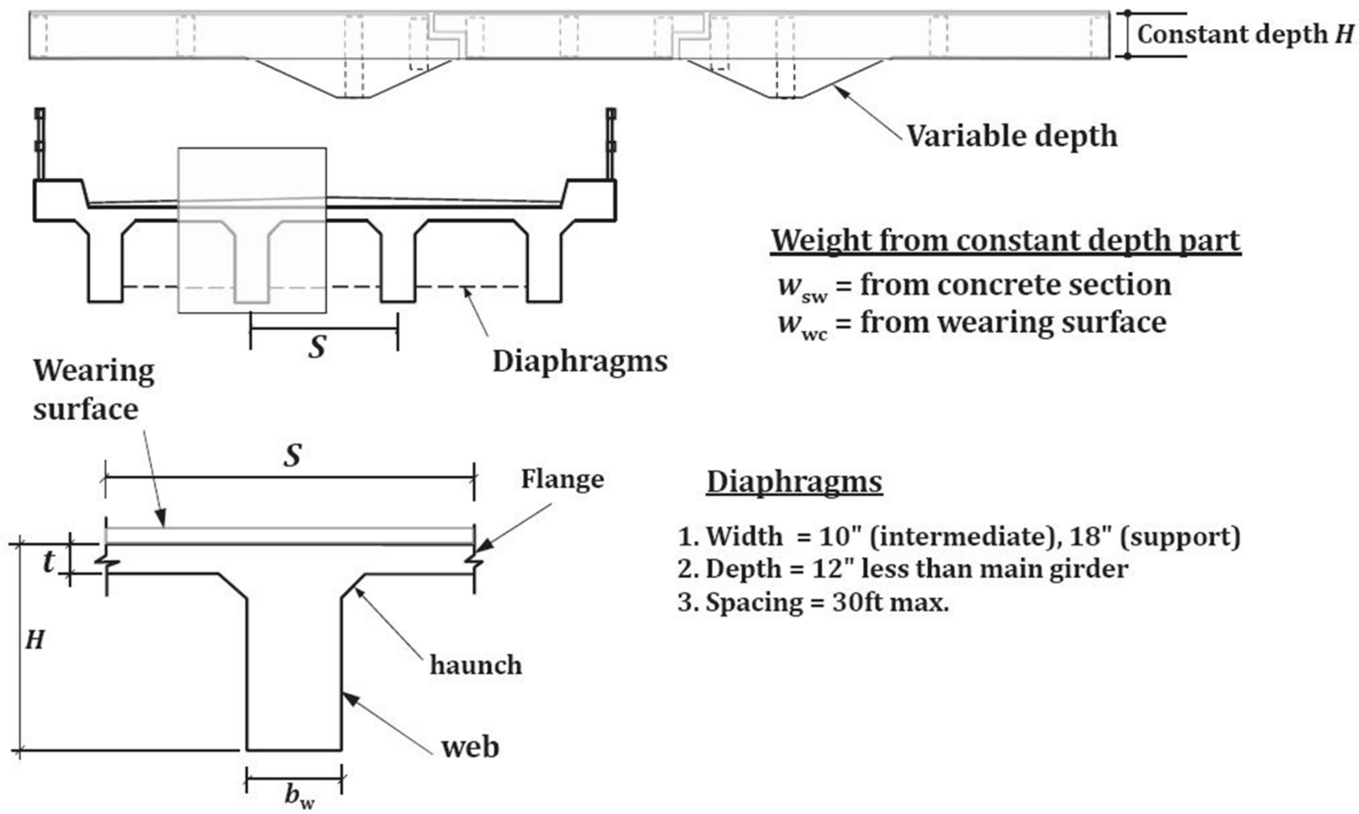


Fig. 35: Different dimensions of longitudinal girder

Dead load Analysis of Interior Girder

- Determine Dead load coming from self weight, wearing surface (DW).
- Determine self weight of cross girder/diaphram.

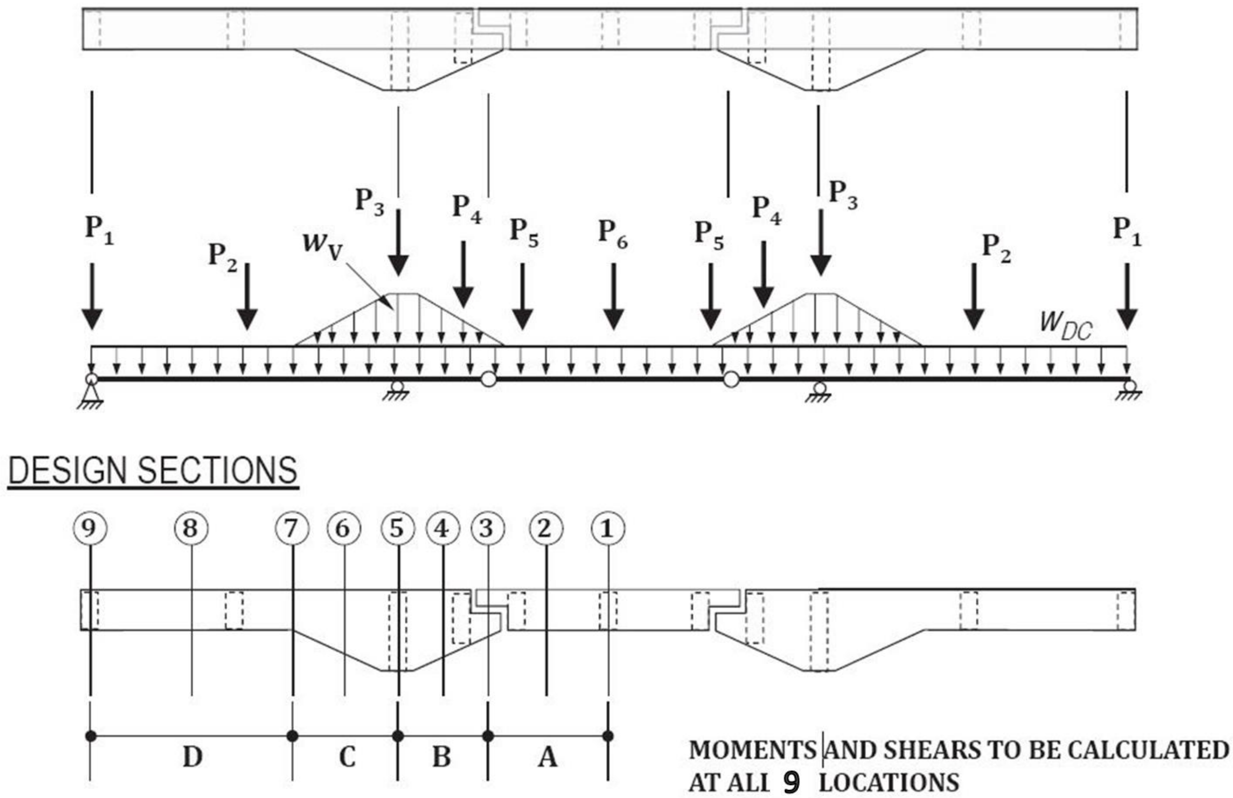


Fig. 36: Design sections of Interior Girder

Table 2: Determining concentrated load of cross girder/diaphragm on main girder

Load of Diaphram	**Depth of Cross girder (in.)**	**Width of Girder, b_d (inch)**	**Load (lb)**
P1			
P2			
P3			
P4			
P5			
P6			

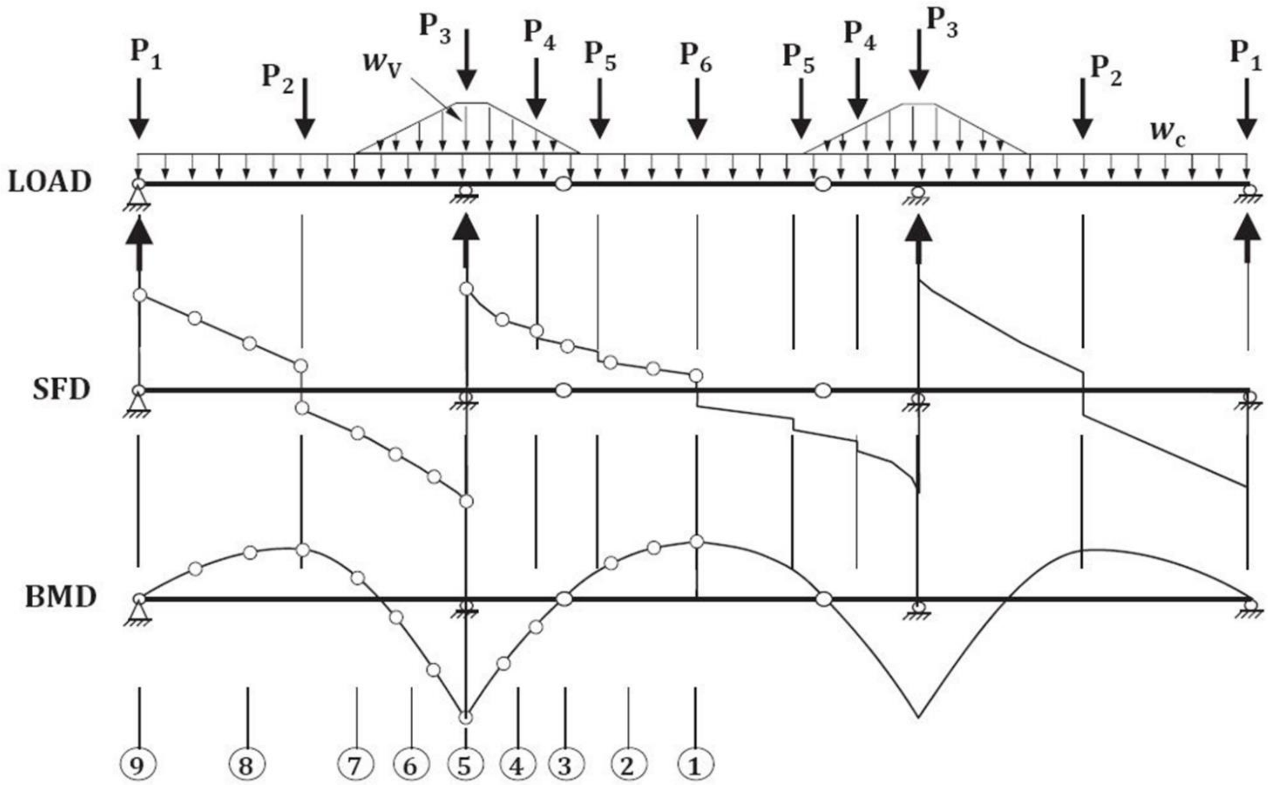


Fig. 37: SFD and BMD of interior girder due to DC dead load

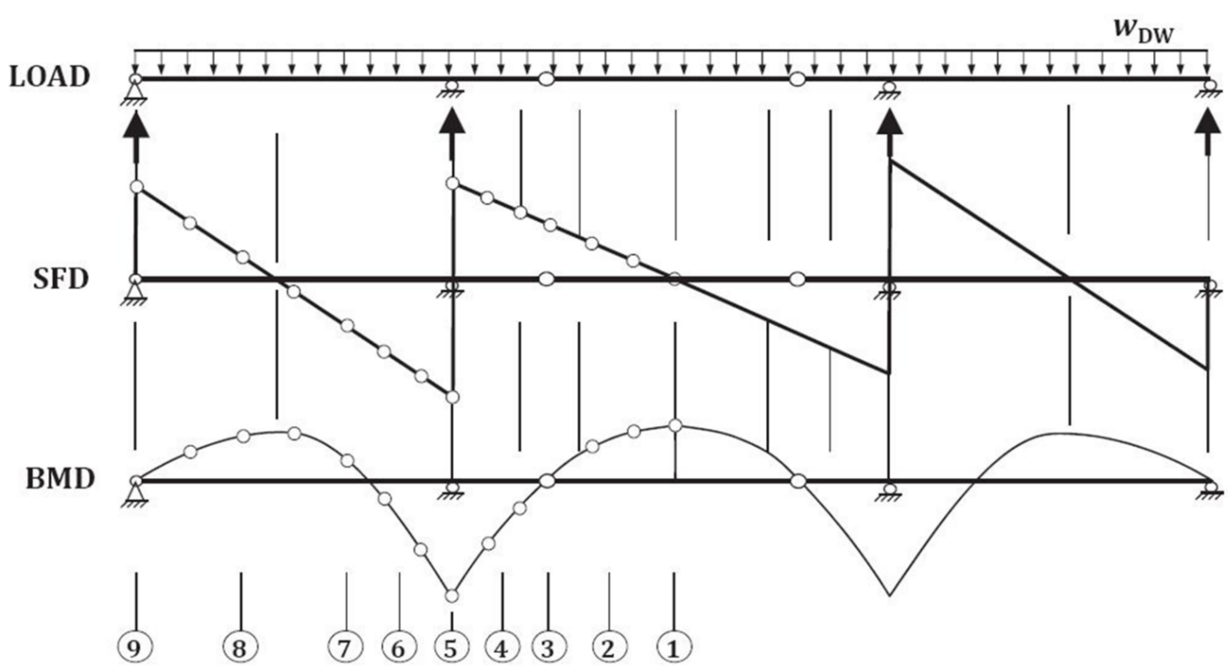


Fig. 38: SFD and BMD of interior girder due to DW dead load (wearing course)

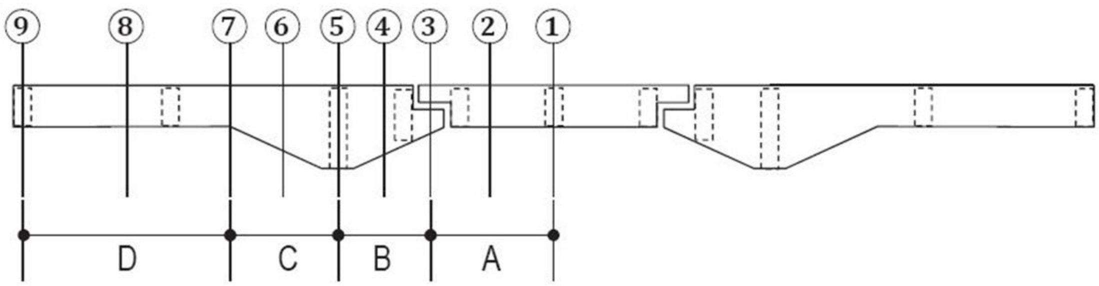
Live load analysis of Interior Girder

Influence Line (IL)

- IL is a digram showing the variation in shear, moment, reaction, stress in a structure due to a unit load moving across the structure.
- ***Miller Breslay's Principle***

"The ordinates of IL for any stress element (such as axial force, shear force, bending moment or reaction) of any structure are proportional to those of the deflection curve which is obtained by removing the restrain corresponding to that element from structure & introducing in its place, a corresponding deformation into the primary structure which remains."

; IL



I.L. FOR SHEAR AND MOMENT AT ALL 9 DESIGN SECTIONS NEED TO BE DRAWN

A FEW ARE SHOWN BELOW.....

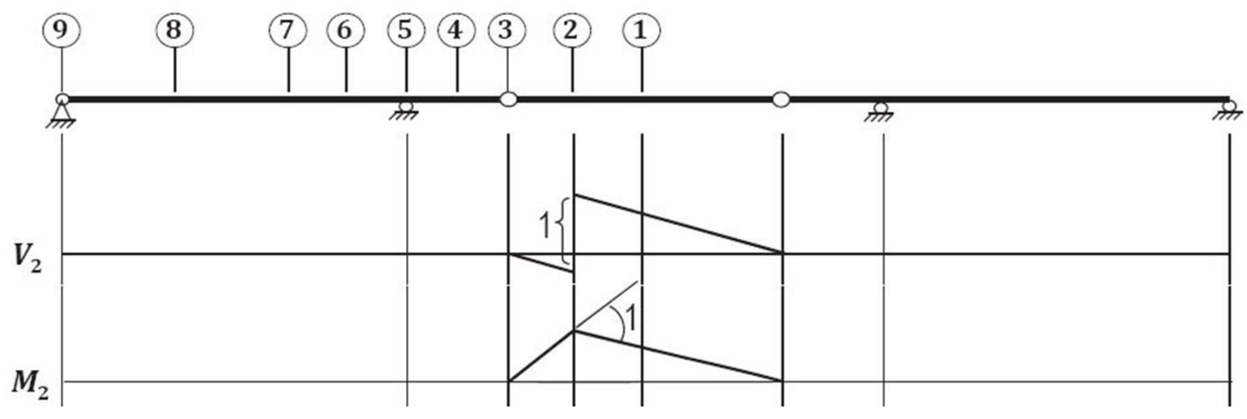


Fig. 39: IL diagram for shear and moment at section 2

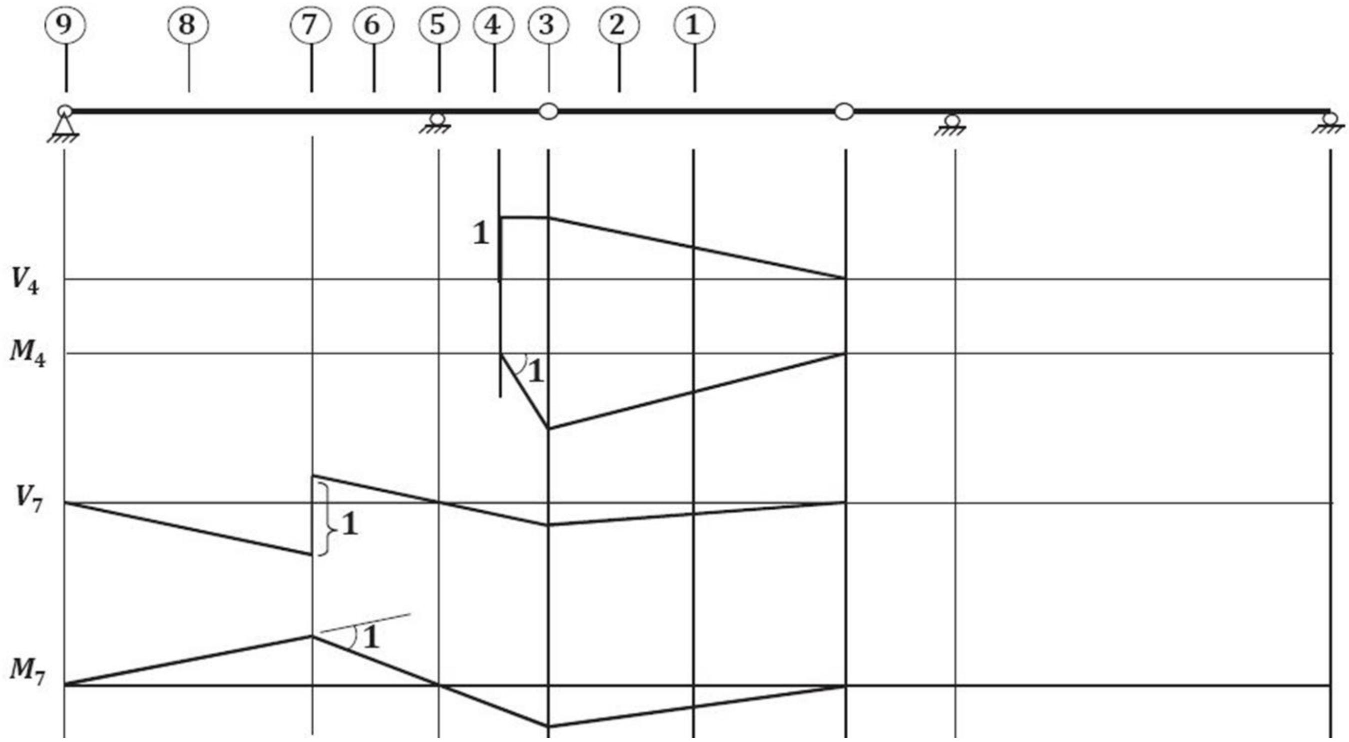
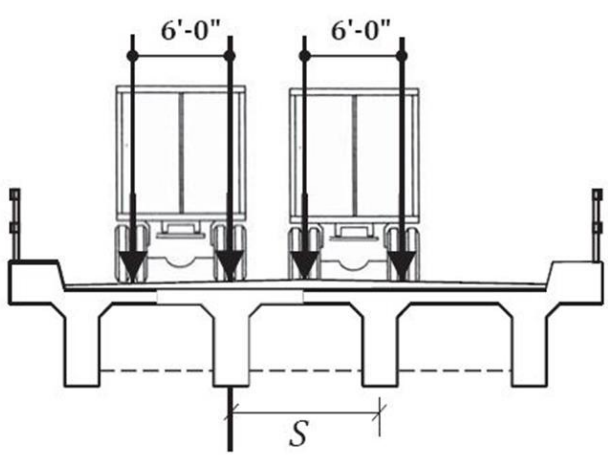


Fig. 40: IL diagram for shear and moment at section 4 and 7

LIVE LOAD MULTIPLIER



Truck wheel on one side may act directly on an interior girder. The other wheel shall be a distance apart from the girder. Thus full vehicle axle load may not act on one girder. This is considered using a distribution factor. (AASHTO Table 4.6.2.2.2b-1 and 4.6.2.2.3a-1)

INTERIOR GIRDER: Two or more lanes are loaded

Distribution factor for moment, $$\alpha_{i,m} = 0.075+\left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0\,Lt_s^3}\right)^{0.1}$$

Distribution factor for shear, $$\alpha_{i,v} = 0.2+\frac{S}{12}-\left(\frac{S}{35}\right)^{2.0}$$

L for Use in Live Load Distribution Factor Equations

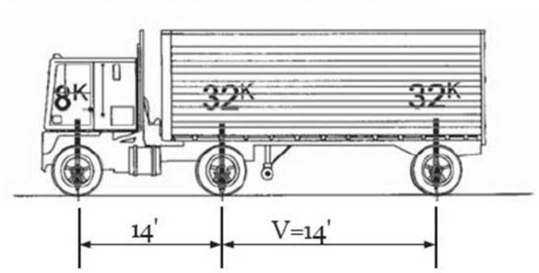
Force Effect	L (ft)
Positive Moment	The length of the span for which moment is being calculated
Negative Moment—Near interior supports of continuous spans from point of contraflexure to point of contraflexure under a uniform load on all spans	The average length of the two adjacent spans
Negative Moment—Other than near interior supports of continuous spans	The length of the span for which moment is being calculated
Shear	The length of the span for which shear is being calculated
Exterior Reaction	The length of the exterior span
Interior Reaction of Continuous Span	The average length of the two adjacent spans

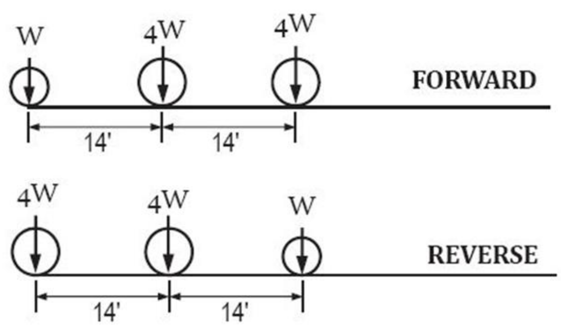
Range of Applicability

$$3.5 \le S \le 16.0$$
$$4.5 \le t_s \le 12.0$$
$$20 \le L \le 240$$
$$N_b \ge 4$$
$$10{,}000 \le K_g \le 7{,}000{,}000$$

$$\left(\frac{K_g}{12.0\,Lt_s^3}\right)^{0.1} = \mathbf{1.05}$$

DESIGN HS20-44 TRUCK LOAD





APPLICATION OF DESIGN WHEEL LOAD: MAX. POSITIVE MOMENT AT SECTION 7

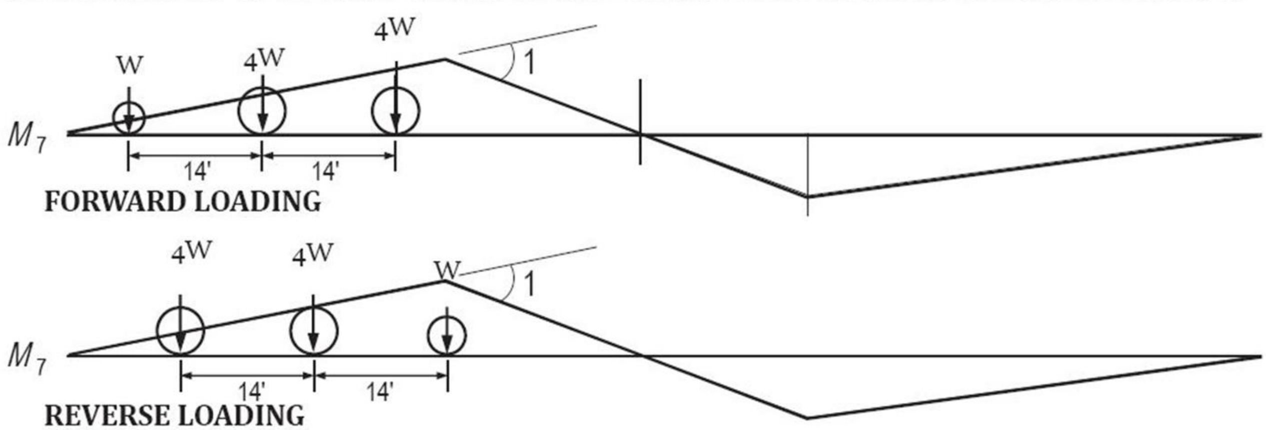


Fig. 41: Maximum positive moment at section 7 for forward and backward truck wheel load

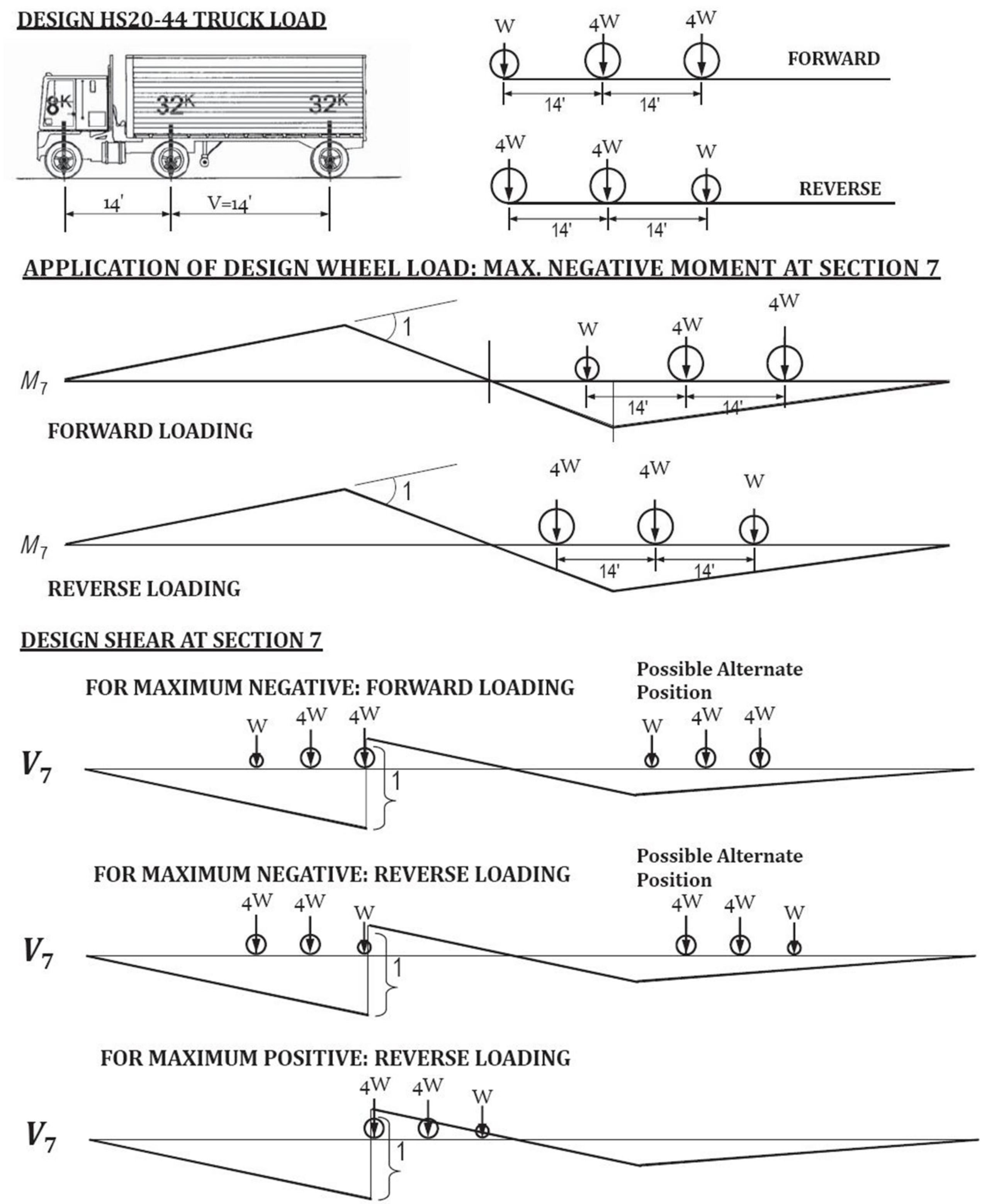
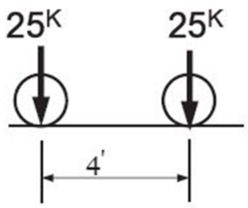
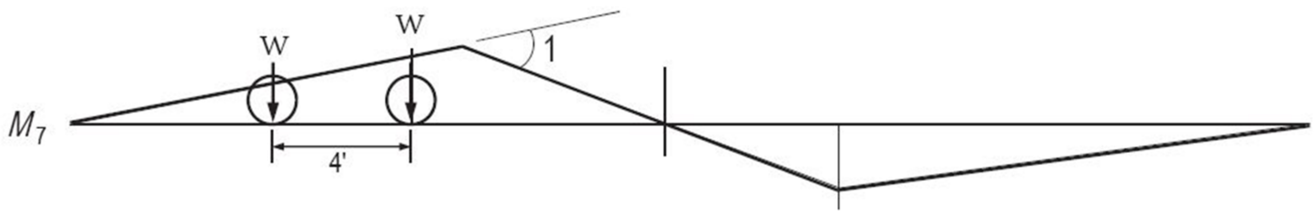


Fig. 42: Maximum +- moment, shear at section 7 for forward and backward truck wheel load

DESIGN TANDEM LOAD



APPLICATION OF DESIGN TANDEM LOAD



FOR MAXIMUM NEGATIVE

FOR MAXIMUM POSITIVE

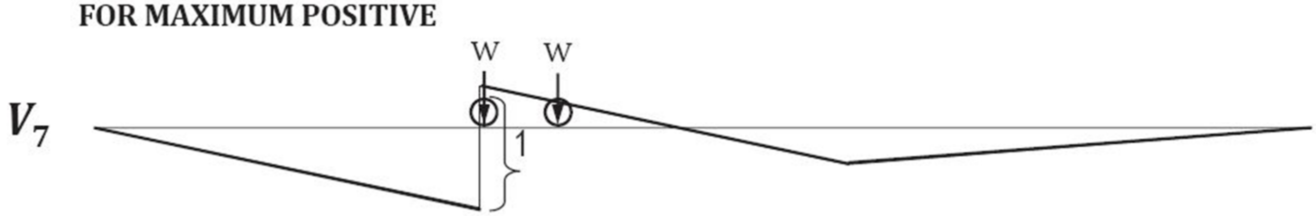


Fig. 43: Maximum positive & negative moment, shear at section 7 for tandem load

EQUIVALENT LANE LOAD

Equivalent lane load must be used in addition to design wheel load to represent truck train.

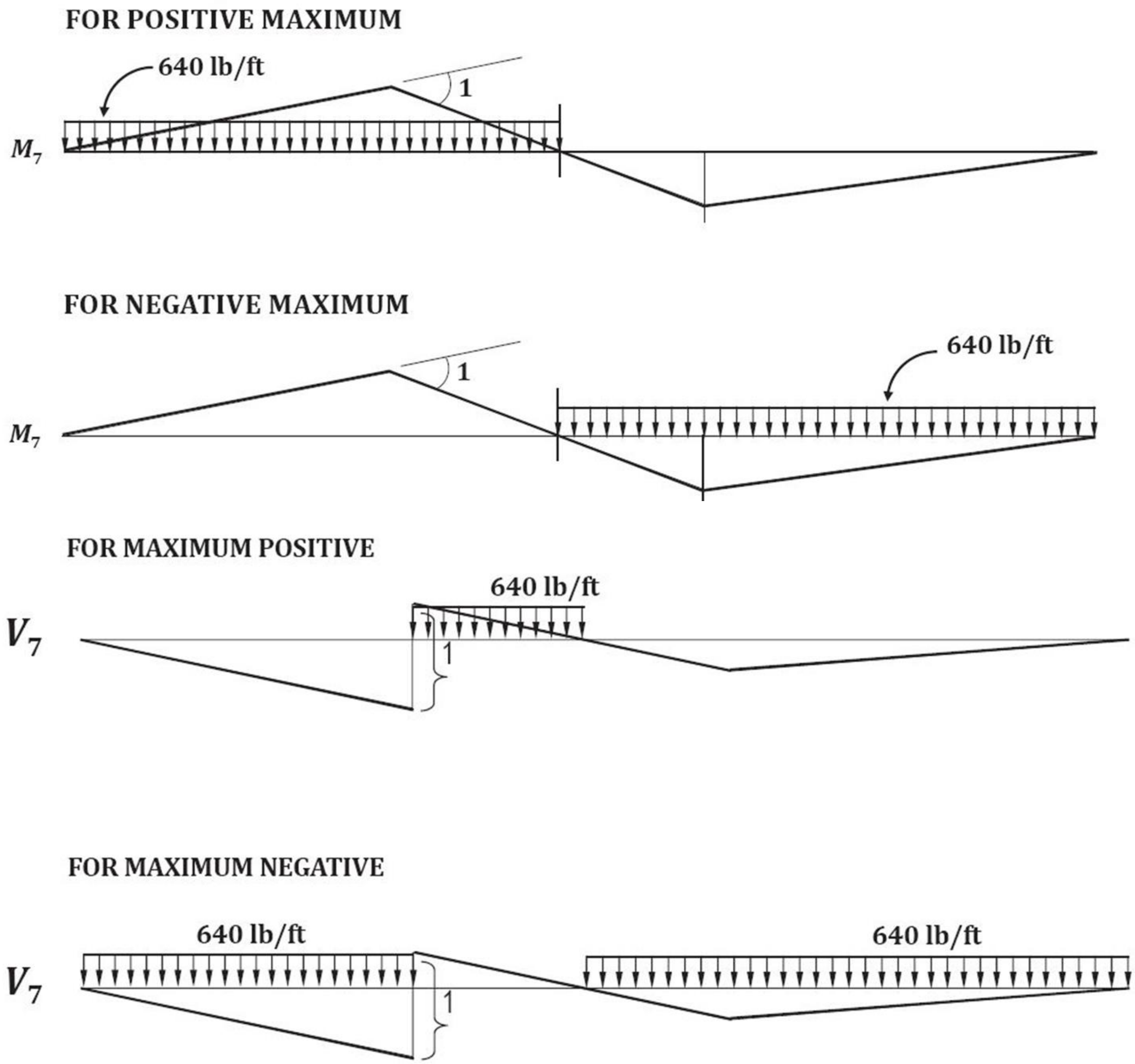


Fig. 44: Maximum positive, negative moment, shear at section 7 for equivalent lane load

Table 3:

COMBINATION OF MOMENT: INTERIOR GIRDER

	Self weight Moment (DC)	Wearing Course Moment (DW)	$\alpha_{i,m}$	Truck Load Moment (Positive)	Tandem Load Moment (Positive)	Lane Load Moment (Positive)	(1 + IM/100)	Combined Positive Moment (Truck), 1.25a+1.5b+1.75cgd+1.75f	Combined Positive Moment (Tandem), 1.25a+1.5b+1.75cge+1.75f	Truck Load Moment (Negative)	Tandem Load Moment (Negative)	Lane Load Moment (negative)	Combined Negative Moment (Truck), 1.25a+1.5b+1.75cgj + 1.75l	Combined Negative Moment (Tandem), 1.25a+1.5b+1.75cgk + 1.75l	Design Positive Moment (Max of h, i)	Design Negative Moment (Max of m, n)
Factor	1.25	1.5		1.75	1.75	1.75										
Loc	a	b	c	d	e	f	g	h	i	j	k	l	m	n		
1																
2																
4																
5																
6																
7																
8																

COMBINATION OF SHEAR: INTERIOR GIRDER

	Self weight Shear (DC)	Wearing Course Shear (DW)	$\alpha_{i,v}$	Truck Load Shear (Positive)	Tandem Load Shear (Positive)	Lane Load Shear (Positive)	(1 + IM/100)	Combined Positive Shear (Truck), 1.25a+1.5b+1.75cgd+1.75f	Combined Positive Shear (Tandem), 1.25a+1.5b+1.75cge+1.75f	Truck Load Shear (Negative)	Tandem Load Shear (Negative)	Lane Load Shear (negative)	Combined Negative Shear (Truck), 1.25a+1.5b+1.75cgj + 1.75l	Combined Negative Shear (Tandem), 1.25a+1.5b+1.75cgk + 1.75l	Design Shear (Abs max of h, I, m, n)
Factor	1.25	1.5		1.75	1.75	1.75									
Loc	a	b	c	d	e	f	g	h	i	j	k	l	m	n	
1															
2															
3															
4															
5R															
5L															
6															
7															
8															
9															

Flexural Reinforcement Design of Interior Girder

- Determine Effective width $b_{eff.}$ for Interior Girder.
- Consider the Design moment for each section.
- Determine steel area A_s for maximum design moment.
- Bar Cut-off will be done where required.

e) DESIGN OF EXTERIOR GIRDER

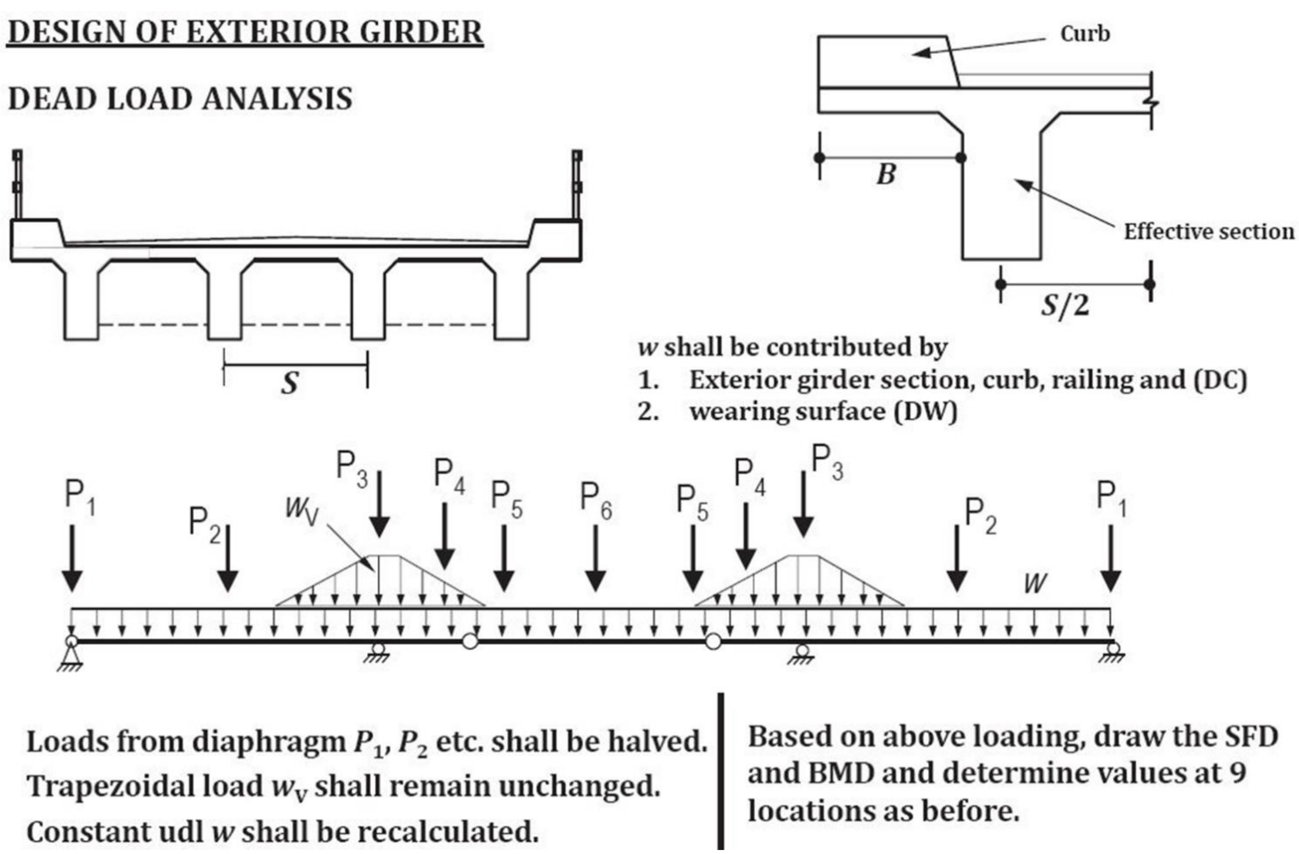


Fig. 45: Dead load on Exterior girder

DESIGN OF EXTERIOR GIRDER

LIVE LOAD ANALYSIS

Distribution factor for moment

$$\alpha_{e,m} = e\alpha_{i,m}$$

$$e = 0.77 + \frac{d_e}{9.1}$$

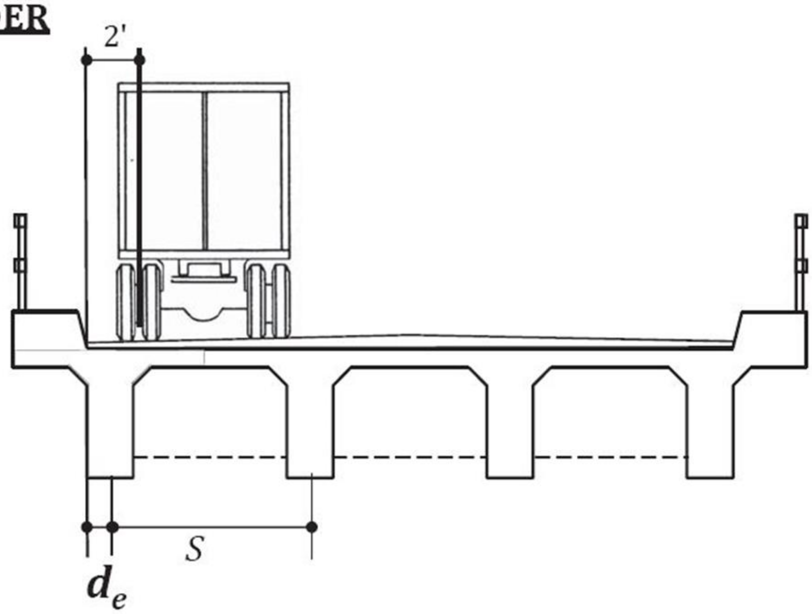
$$-1.0 \leq d_e \leq 5.5$$

Distribution factor for shear

$$\alpha_{e,v} = e\alpha_{i,v}$$

$$e = 0.6 + \frac{d_e}{10}$$

Here d_e is in feet.



Now prepare the load combination tables for shear and moment. Dead load values shall be recalculated based on revised loading (DC and DW). Live load values may be directly copied from previous load combination tables and combinations may be performed with α values for exterior girder.

Fig. 46: Live load on Exterior girder

Table 4:

COMBINATION OF SHEAR: EXTERIOR GIRDER

	Self weight Shear (DC)	Wearing Course Shear (DW)	$\alpha_{i,v}$	Truck Load Shear (Positive)	Tandem Load Shear (Positive)	Lane Load Shear (Positive)	(1 + IM/100)	Combined Positive Shear (Truck), 1.25a+1.5b+1.75cgd+1.75f	Combined Positive Shear (Tandem), 1.25a+1.5b+1.75cge+1.75f	Truck Load Shear (Negative)	Tandem Load Shear (Negative)	Lane Load Shear (negative)	Combined Negative Shear (Truck), 1.25a+1.5b+1.75cg j + 1.75l	Combined Negative Shear (Tandem), 1.25a+1.5b+1.75cgk + 1.75l	Design Shear (Abs max of h, l, m, n)
Factor	1.25	1.5		1.75	1.75	1.75									
Loc	a	b	c	d	e	f	g	h	i	j	k	l	m	n	
1															
2															
3															
4															
5R															
5L															
6															
7															
8															
9															

COMBINATION OF MOMENT: EXTERIOR GIRDER

	Self weight Moment (DC)	Wearing Course Moment (DW)	$\alpha_{i,m}$	Truck Load Moment (Positive)	Tandem Load Moment (Positive)	Lane Load Moment (Positive)	(1 + IM/100)	Combined Positive Moment (Truck), 1.25a+1.5b+1.75cgd+1.75f	Combined Positive Moment (Tandem), 1.25a+1.5b+1.75cge+1.75f	Truck Load Moment (Negative)	Tandem Load Moment (Negative)	Lane Load Moment (negative)	Combined Negative Moment (Truck), 1.25a+1.5b+1.75cg j + 1.75l	Combined Negative Moment (Tandem), 1.25a+1.5b+1.75cgk + 1.75l	Design Positive Moment (Max of h, i)	Design Negative Moment (Max of m, n)
Factor	1.25	1.5		1.75	1.75	1.75										
Loc	a	b	c	d	e	f	g	h	i	j	k	l	m	n		
1																
2																
4																
5																
6																
7																
8																

REINFORCEMENT DESIGN OF T-GIRDERS (AASHTO 2012 Section 5)

Resistance factor φ [Sec. 5.5.4.2.1]

Moment 0.90

Shear 0.90

Positive Steel (T-section, bottom steel) [Sec. 5.7.3.2.1]

$$A_s \geq \frac{M_u}{\phi f_y\left(d - \frac{a}{2}\right)} \approx \frac{M_u}{\phi f_y (jd)}$$

Assume $jd \approx 0.95d$

Check $A_s \geq A_{s,\min} = \frac{200}{f_y} b_w d$

where, f_y is in psi

Effective flange width, b_e of T-Girder

b_e = Spacing of girders = S

Check, $a = \frac{A_s f_y}{0.85 f_c' b_e}$

Revise, $A_s = \frac{M_u}{\phi f_y\left(d - \frac{a}{2}\right)}$

Revise, $a = \frac{A_s f_y}{0.85 f_c' b_e}$

Determine, β_1

Depth of neutral axis, $c = a/\beta_1$

Check $c < {}^3/_8\, d$ (tension controlled)

Finally, $\phi M_n = \phi A_s f_y\left(d - \frac{a}{2}\right) \geq M_u$

Negative Steel (Rectangular Section, top steel)

Design procedure same as before except that use beam web width b_w instead of b_e.

REINFORCEMENT DESIGN OF T-GIRDERS (AASHTO 2012 Section 5)

Design for Shear (Sec. 5.8.3.3)

Shear reinforcement required when $V_u > 0.5\phi\, V_c$ (V_u and V_c are in kip, Sec. 5.8.2.4)

If $V_u > 0.25\phi\, f_c' b_w d$ then section has to be revised. (f_c' in ksi, b_w and d are in inch)

Nominal shear resistance, $V_n = V_c + V_s$

where $V_c = 0.0316\beta(\sqrt{f_c'})b_w d$, where $\beta = 2.0$ (Eq. 5.8.3.3-3)

Stirrup spacing, $s = \frac{\phi A_v f_y d}{V_u - \phi V_c}$

Minimum transverse reinforcement (Eq. 5.8.2.5-1), $A_v \geq 0.0316\sqrt{f_c'}\frac{b_w s}{f_y}$

where, A_v in in^2, f_c' in ksi, b_w is beam web width in inch, s is stirrup spacing in inch, f_y in ksi.

Shear stress in concrete $v_u = V_u/(\phi\, b_w d)$

Maximum stirrup spacing [Sec 5.8.2.7]:

$s_{\max} = 0.8d \leq 24''$ when $v_u < 0.125 f_c'$

$s_{\max} = 0.4d \leq 12''$ when $v_u \geq 0.125 f_c'$

SKIN REINFORCEMENT [Sec. 5.7.3.4]

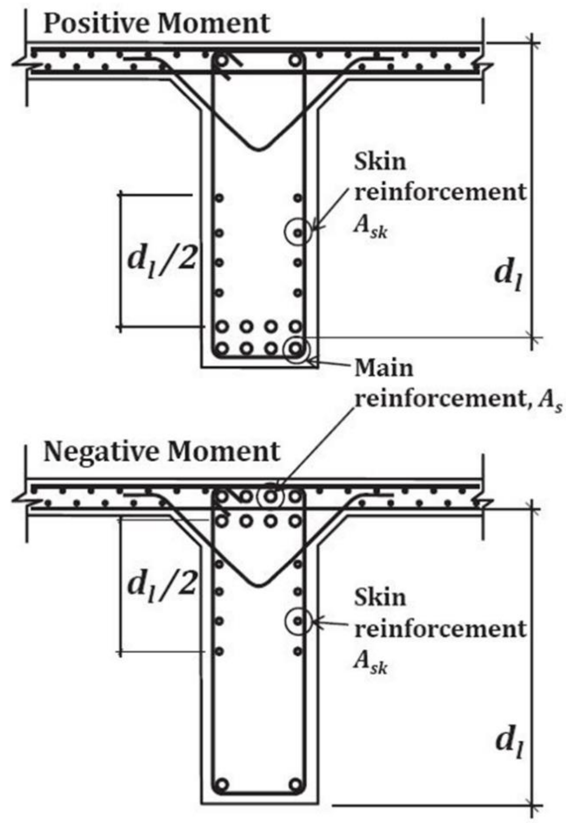
If d_ℓ of non-prestressed or partially prestressed concrete members exceeds 3.0 ft, longitudinal skin reinforcement shall be uniformly distributed along both side faces of the component for a distance d_ℓ /2 (inch) nearest the flexural tension reinforcement.

The area of skin reinforcement A_{sk} in in²/ft of height on each side face shall satisfy (Eq. 5.7.3.4-2):

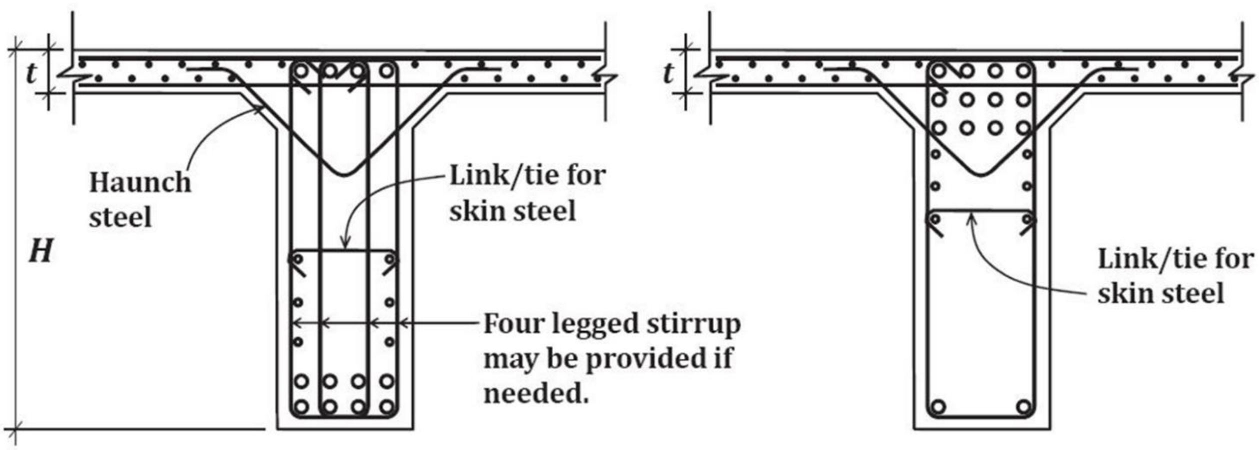
$$A_{sk} \geq 0.012(d_l - 30) \leq \frac{A_s}{4}$$

However, the total area of longitudinal skin reinforcement (per face) need not exceed one-fourth of the required flexural tensile reinforcement A_s.

The maximum spacing of the skin reinforcement shall not exceed either $d_\ell/6$ or 12.0 in.



REINFORCEMENT DETAILING OF T-GIRDERS



Haunch Steel
Provide #3 or #4 bar @ 6" ~ 9" c/c along the length of the girder

Link/Tie for skin reinforcement
Provide #3 or #4 bars. Vertical and longitudinal spacing may not exceed 24".

Fig.47: Reinforcement detailing of main girder

f) DESIGN OF CROSS GIRDER/ DIAPHRAM

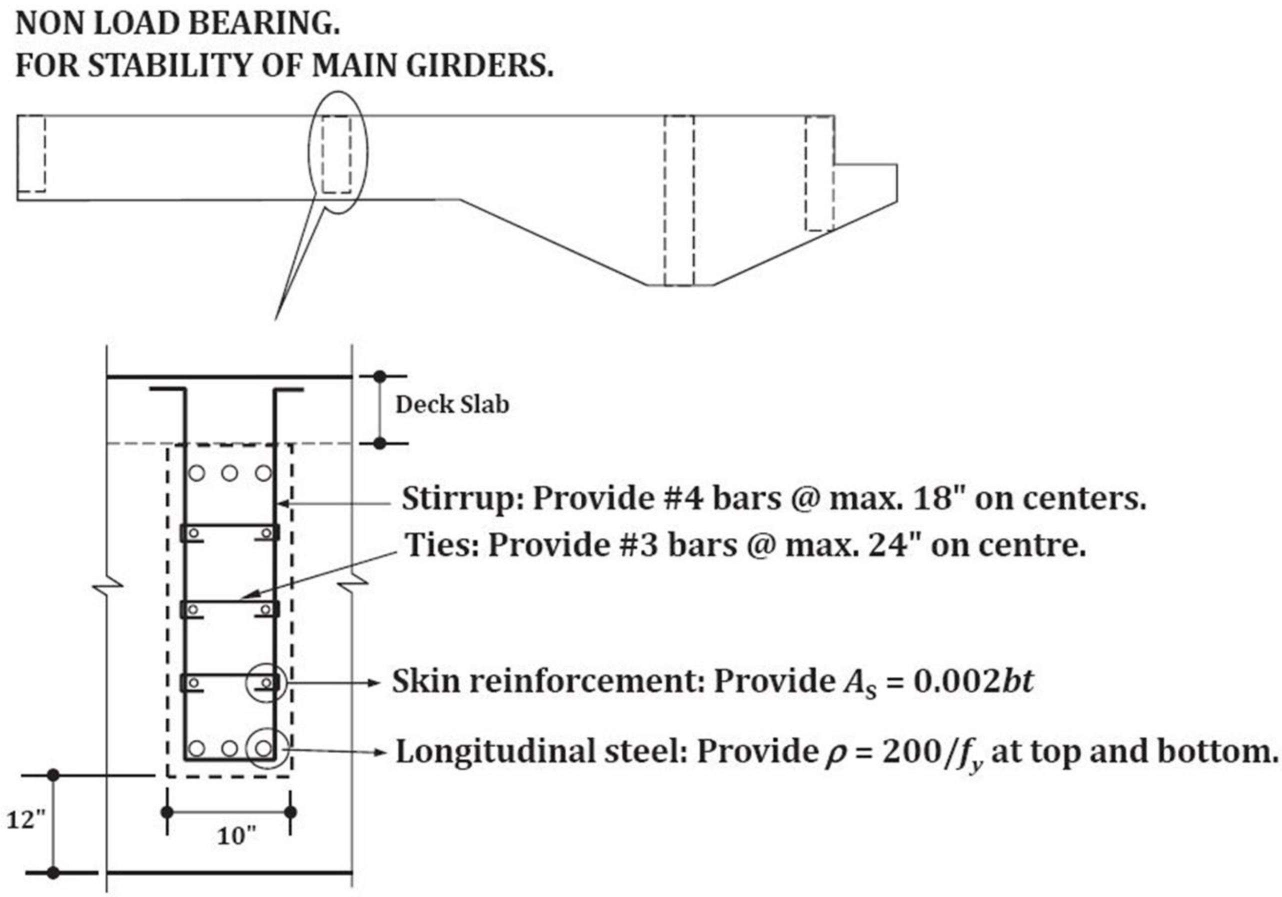


Fig.48 : Reinforcement detailing of cross girder

g) DESIGN OF ARTICULATION

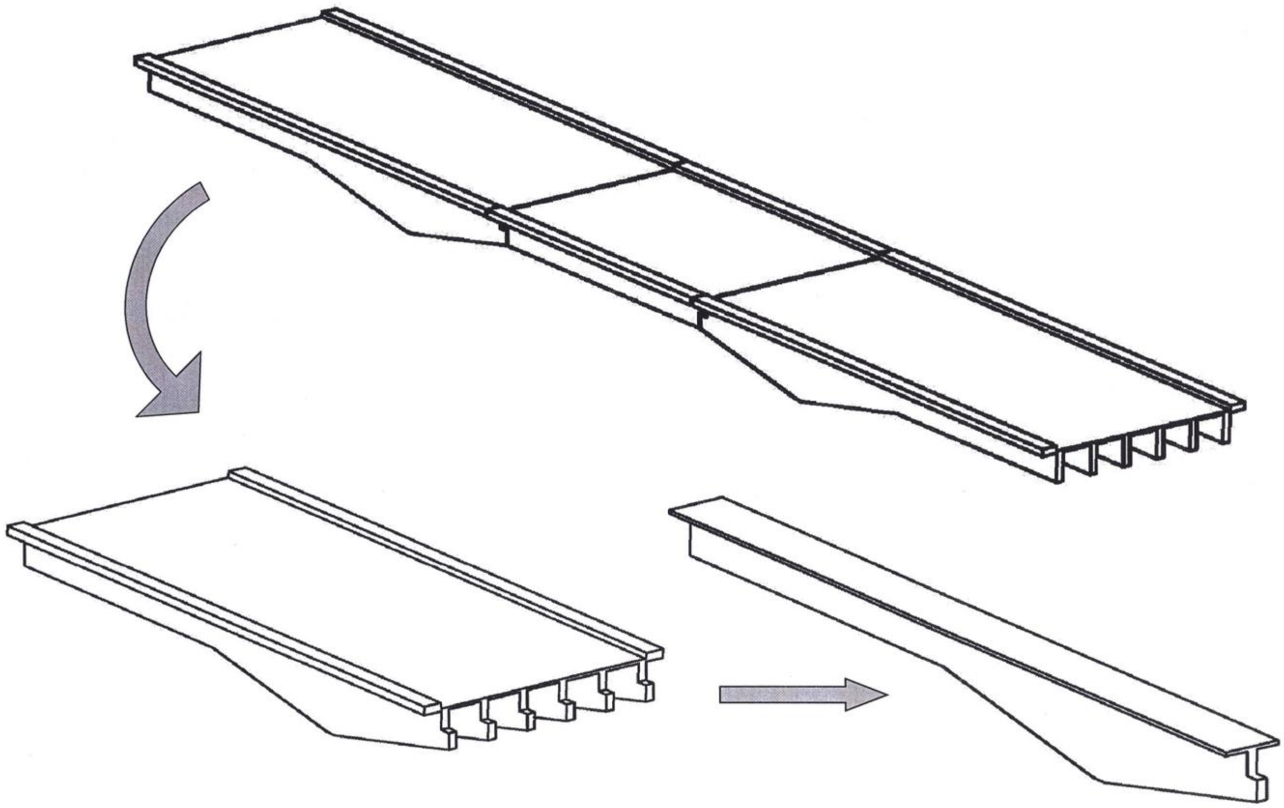


Fig.49 : Articulation or halving joint

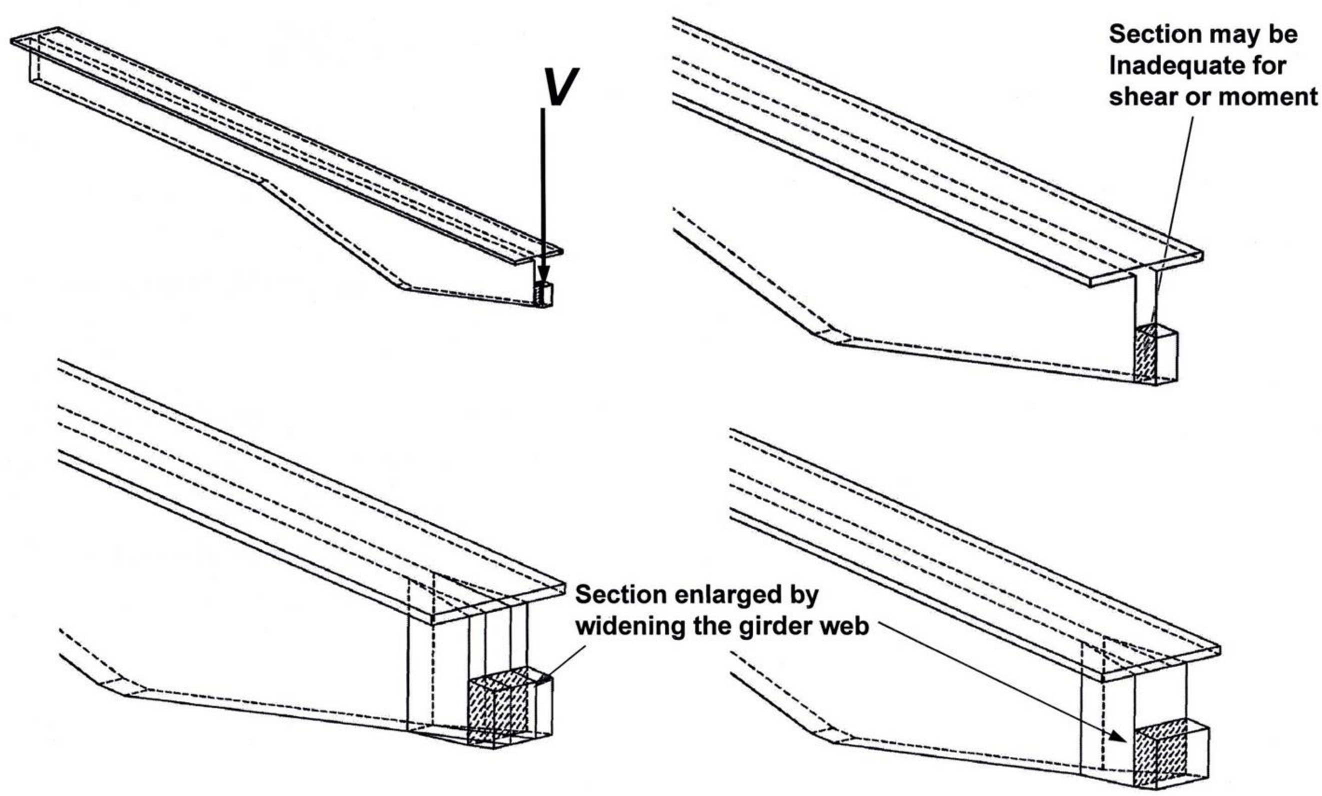


Fig.50: Widening of girder near articulation location

What is Articulation

- The connection between the suspended span and the edge of the cantilever is called ‘Articulation’.
- The bearings at articulations can be in the form of sliding plates, roller-rocker arrangement or elastomeric pads.

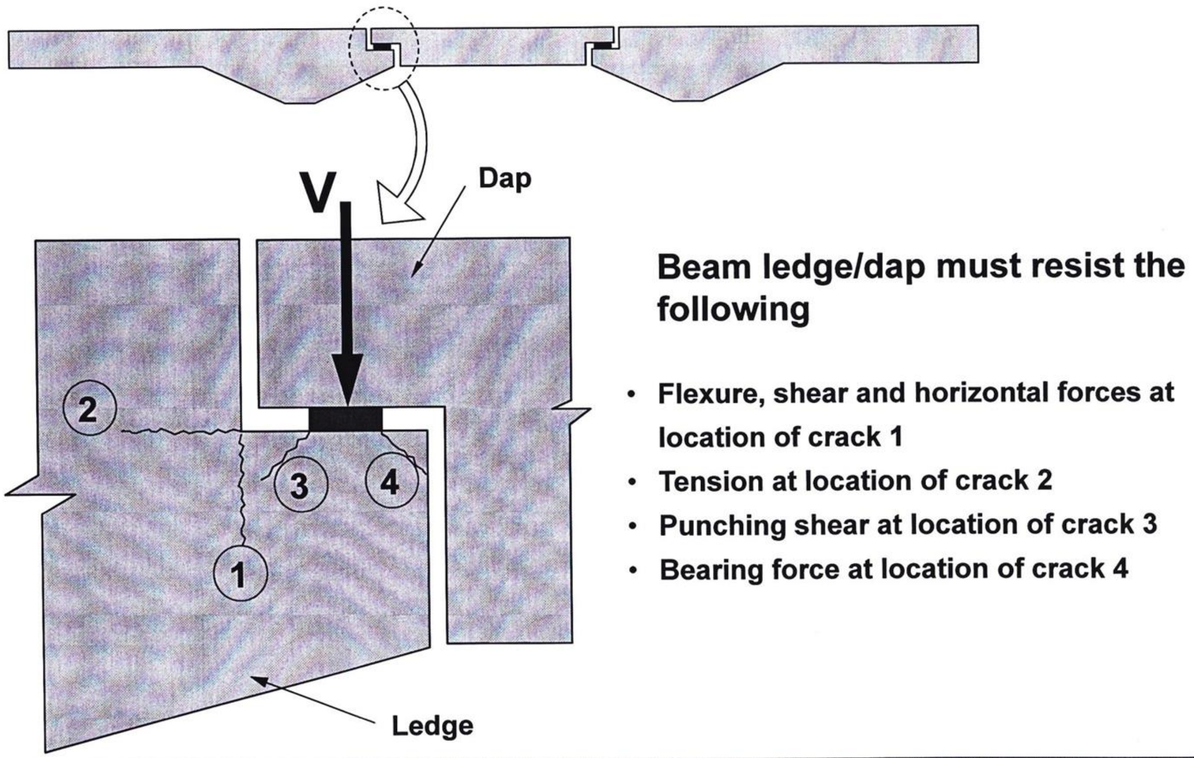


Fig.51 : Cracks at articulation

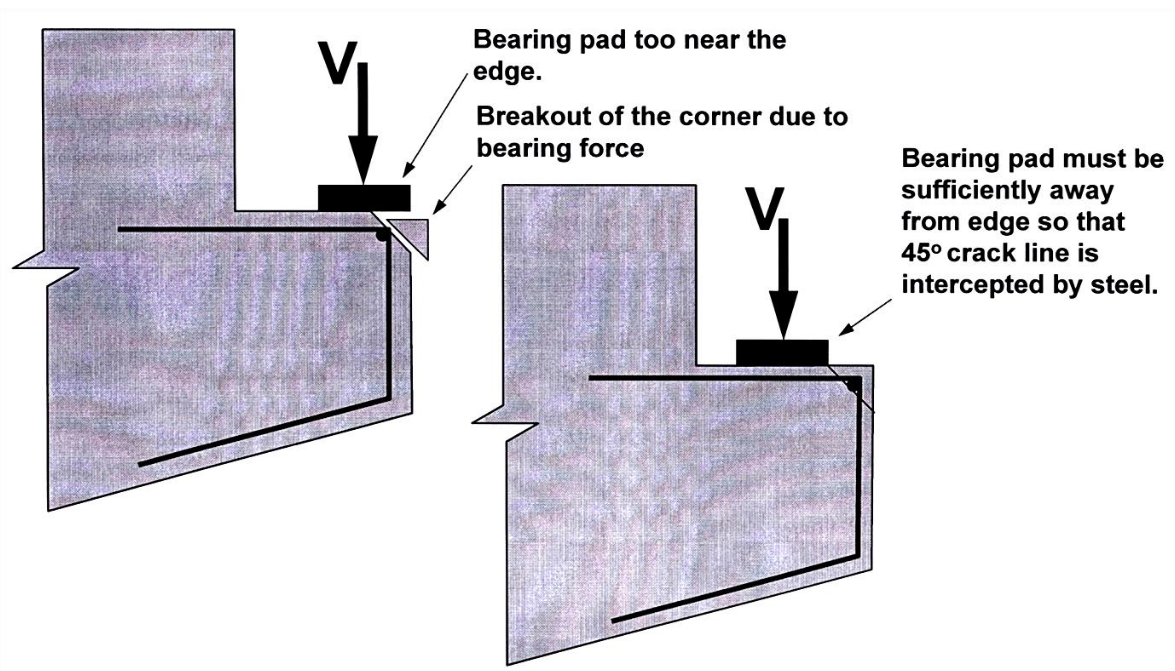


Fig.52 : Clearance requirement around bearing pad near articulation

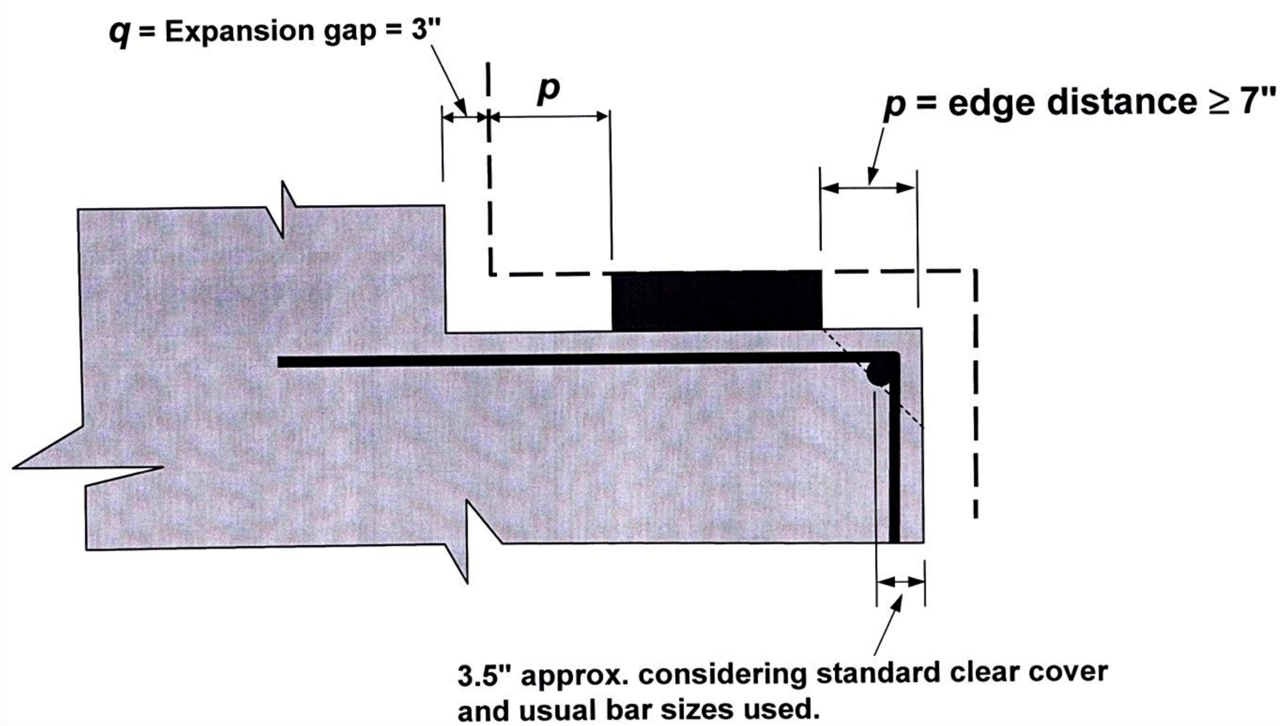
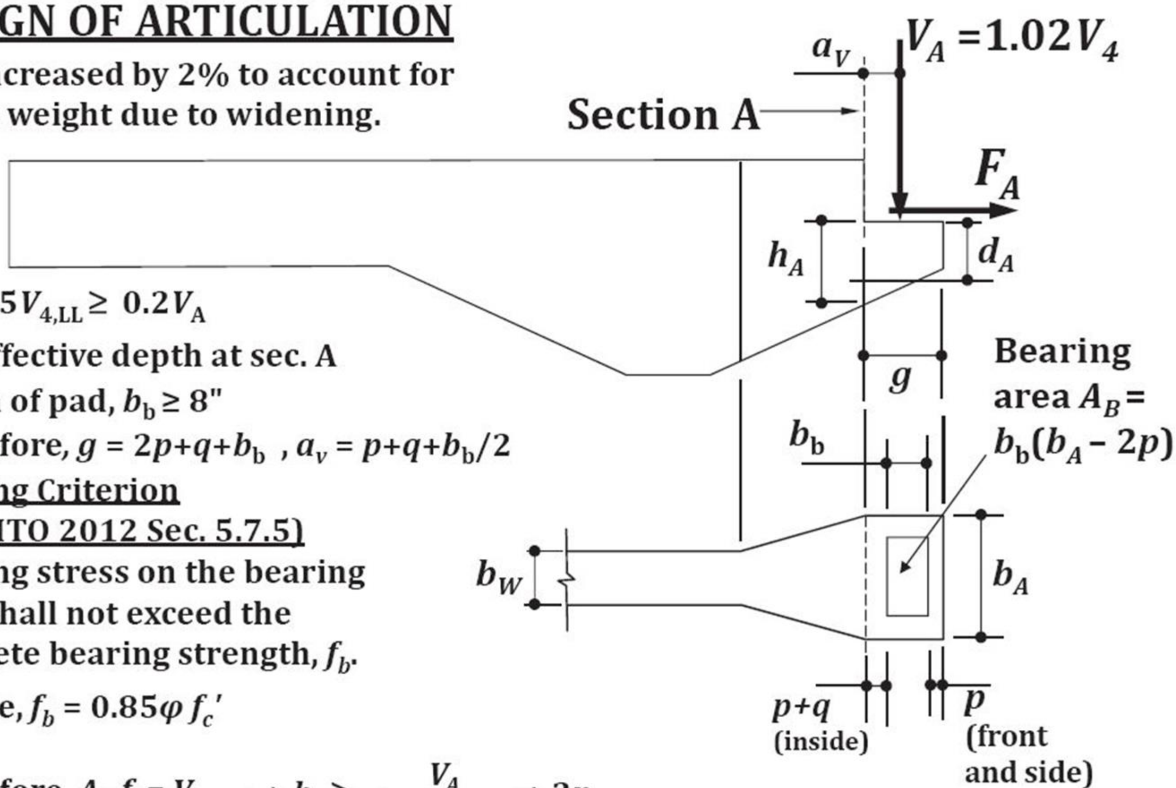


Fig. 53: Expansion gap and edge distance around bearing pad

DESIGN OF ARTICULATION

V_4 is increased by 2% to account for excess weight due to widening.



$F_A = 0.5V_{4,LL} \geq 0.2V_A$

d_A = effective depth at sec. A

Width of pad, $b_b \geq 8"$

Therefore, $g = 2p+q+b_b$, $a_v = p+q+b_b/2$

<u>Bearing Criterion</u>

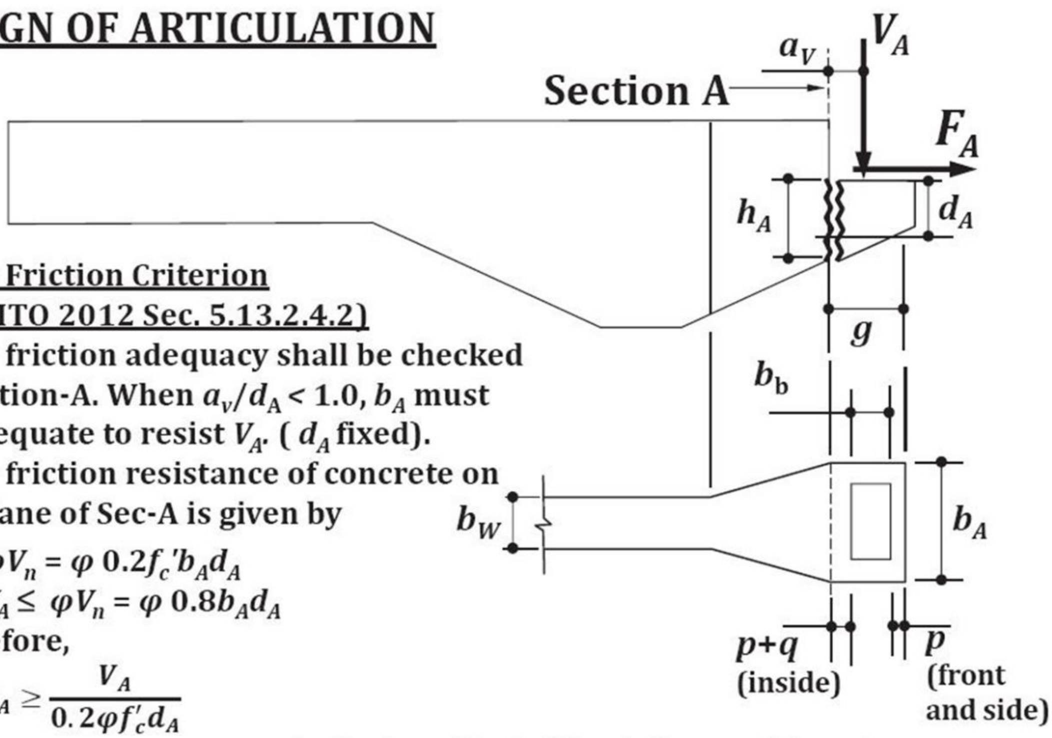
<u>(AASHTO 2012 Sec. 5.7.5)</u>

Bearing stress on the bearing area shall not exceed the concrete bearing strength, f_b.

Where, $f_b = 0.85\varphi f_c'$

Therefore, $A_B f_b = V_A \quad \Rightarrow b_A \geq \dfrac{V_A}{b_b f_b} + 2p$

DESIGN OF ARTICULATION



<u>Shear Friction Criterion</u>

<u>(AASHTO 2012 Sec. 5.13.2.4.2)</u>

Shear friction adequacy shall be checked at Section-A. When $a_v/d_A < 1.0$, b_A must be adequate to resist V_A. (d_A fixed).

Shear friction resistance of concrete on the plane of Sec-A is given by

$V_A \leq \varphi V_n = \varphi\, 0.2f_c' b_A d_A$

And $V_A \leq \varphi V_n = \varphi\, 0.8 b_A d_A$

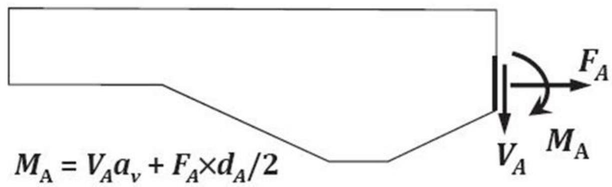
Therefore,

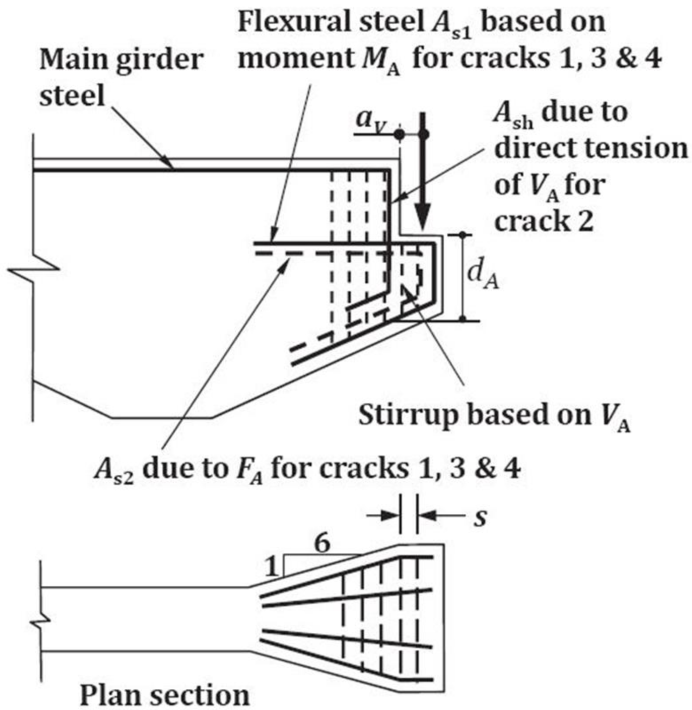
$$b_A \geq \frac{V_A}{0.2\varphi f_c' d_A}$$

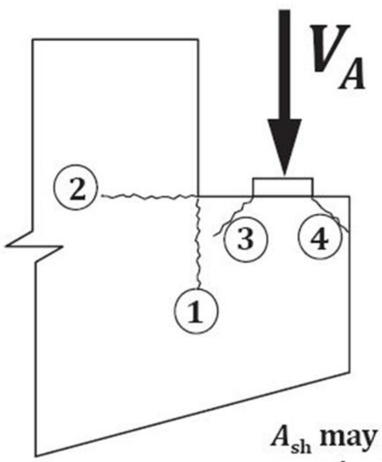
$$\text{or, } b_A \geq \frac{V_A}{\varphi\, 0.8 dA}$$

Final value of b_A shall be the larger of the value obtained from bearing and shear friction criteria.

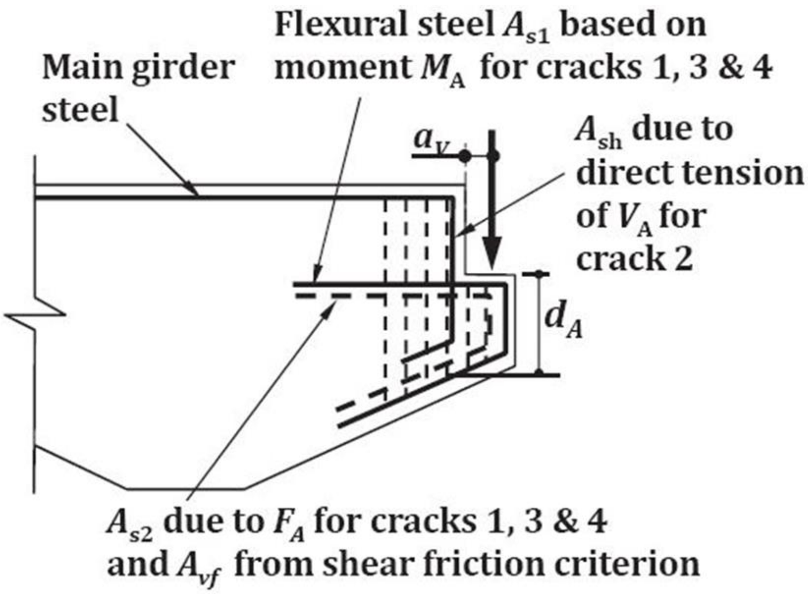
DESIGN OF ARTICULATION

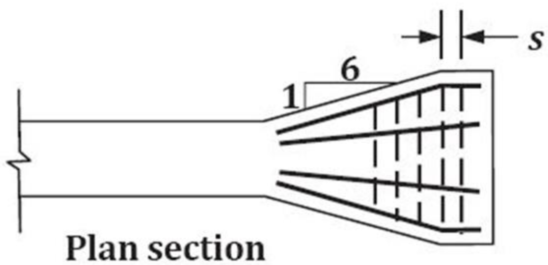






A_{sh} may be provided by extending and bending the main top bar as required. Similarly, A_{s1} and A_{s2} may be provided by extending and bending the girder bottom steel.





For flexural steel A_{s1}

$$A_{s1} = \frac{M_A}{\varphi f_y (d_A - a/2)}$$

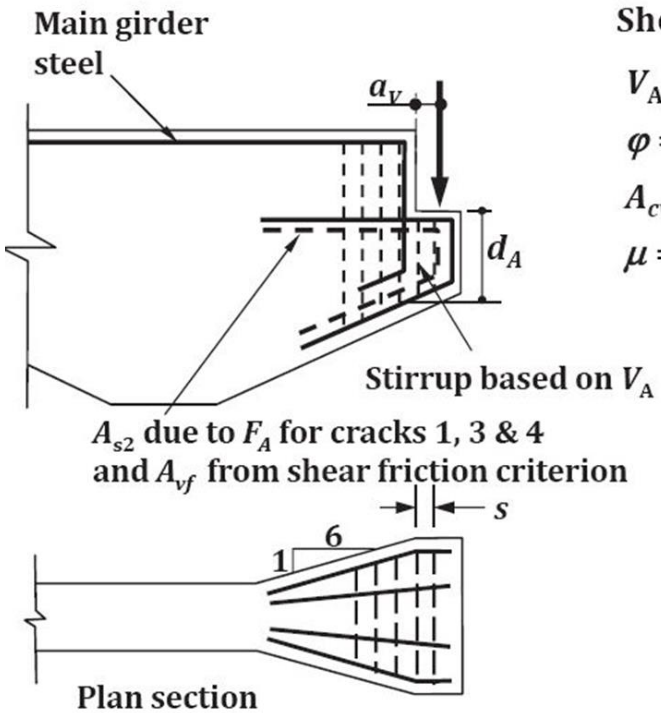
$$a = \frac{A_{s1} f_y}{0.85 f'_c b_A}, \quad \varphi = 0.9$$

For steel A_{sh} and A_{s2}

$$A_{sh} = \frac{V_A}{\varphi f_y}, \quad \varphi = 0.85$$

$$A_{s2} = \frac{F_A}{\varphi f_y}$$

DESIGN OF ARTICULATION



SHEAR FRICTION REINFORCEMENT [Sec 5.8.4]

Shear friction criterion: $a_v/d_A < 1.0$

$V_A = \varphi\{c\,A_{cv} + \mu\,A_{vf}\,f_y\}$, V_A in lb [Eq. 5.8.4.1-3]

$\varphi = 0.9$ for shear, $c = 0.0$ psi [Sec.5.8.4.3]

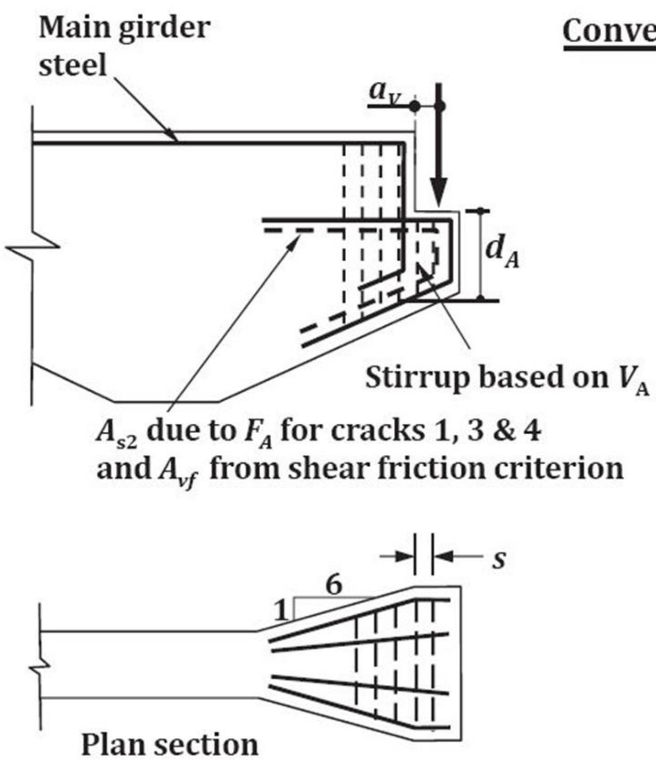
$A_{cv} = b_A \times d_A$ = shear area (in^2)

$\mu = 1.4$ = friction factor, f_y in psi

A_{vf} = shear steel crossing the shear plane, in^2

$A_{vf} \geq 0.05 A_{cv}/f_y$ [Eq. 5.8.4.4-1]

A_{vf} may be merged with A_{s2} and A_{s1}



Conventional flexural shear criterion: $a_v/d_A > 1.0$

$$s = \frac{\varphi A_v f_v d_A}{V_A - \varphi V_c}$$

$V_c = 0.0316\beta(\sqrt{f_c'})\,b_A d_A$

Check s_{max} < 12" or $d_A/2$

Though conventional flexural shear steel is required only when $a_V/d_A > 1$, we shall, nevertheless, provide such steel even when $a_V/d_A < 1$.

Design Steps

1. Determine flexural steel area A_{s1} based on moment M_A.
2. Determine steel area A_{s2} based on F_A.
3. Determine steel area A_{sh} based on V_A
4. Determine required spacing s for stirrup
5. Check spacing of stirrup with maximum spacing

Detailing of Articulation

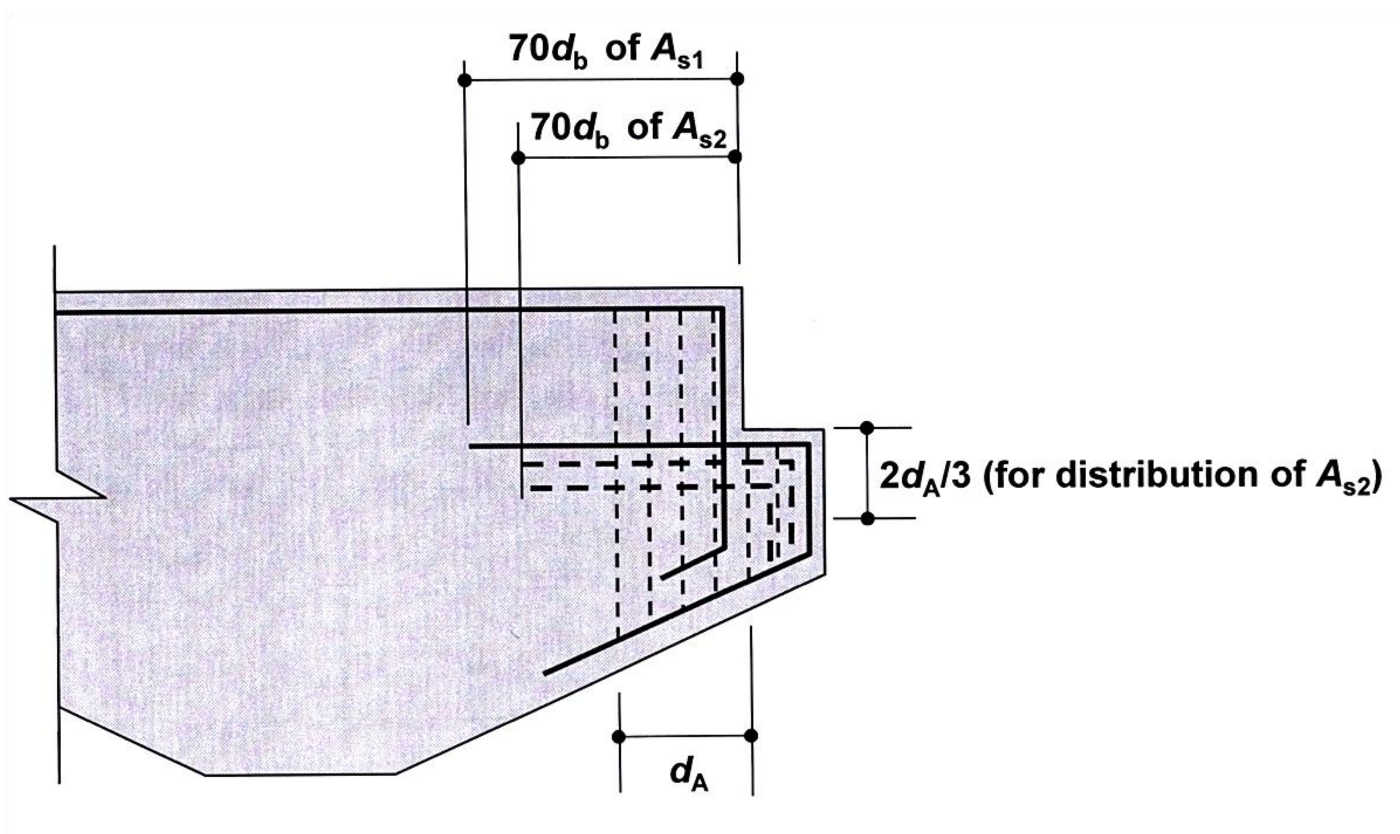


Fig.54 : Reinforcement detailing of articulation

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3. AASHTO LRFD Bridge design Specifications, 6th edition, 2012, US.

4. Other online resources.