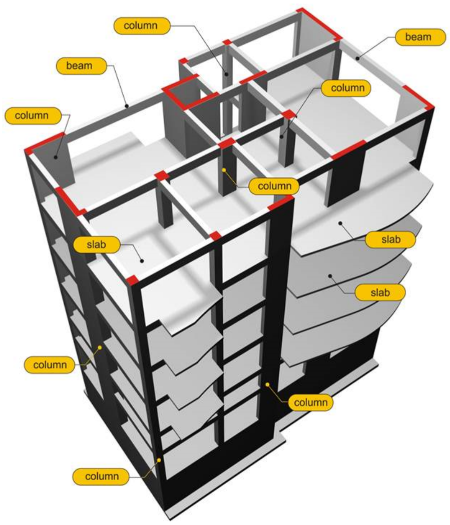
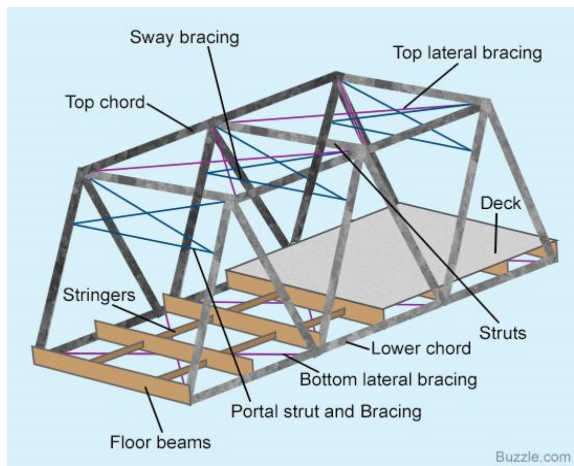
CE 412
Structural Analysis and Design Sessional-II





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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 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 of Dr. Khan Mahmud Amanat, and Mr. Ruhul Amin, 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.)

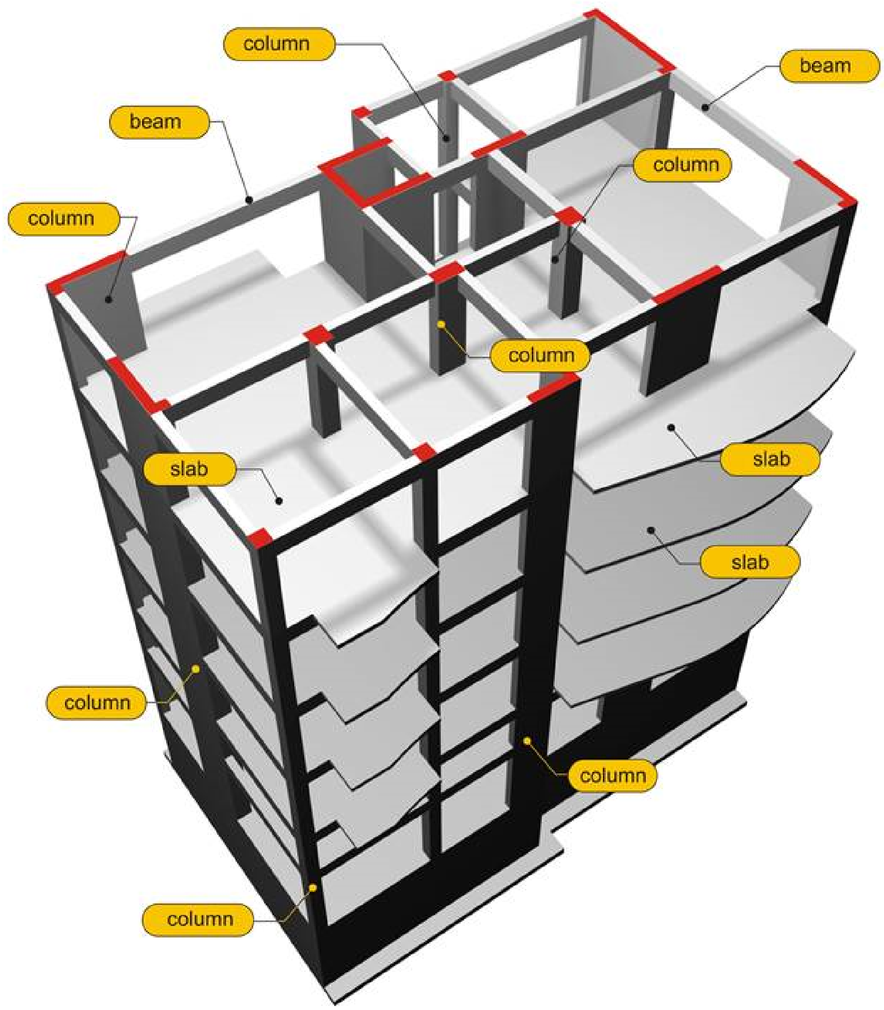


Figure 1: Super structural elements

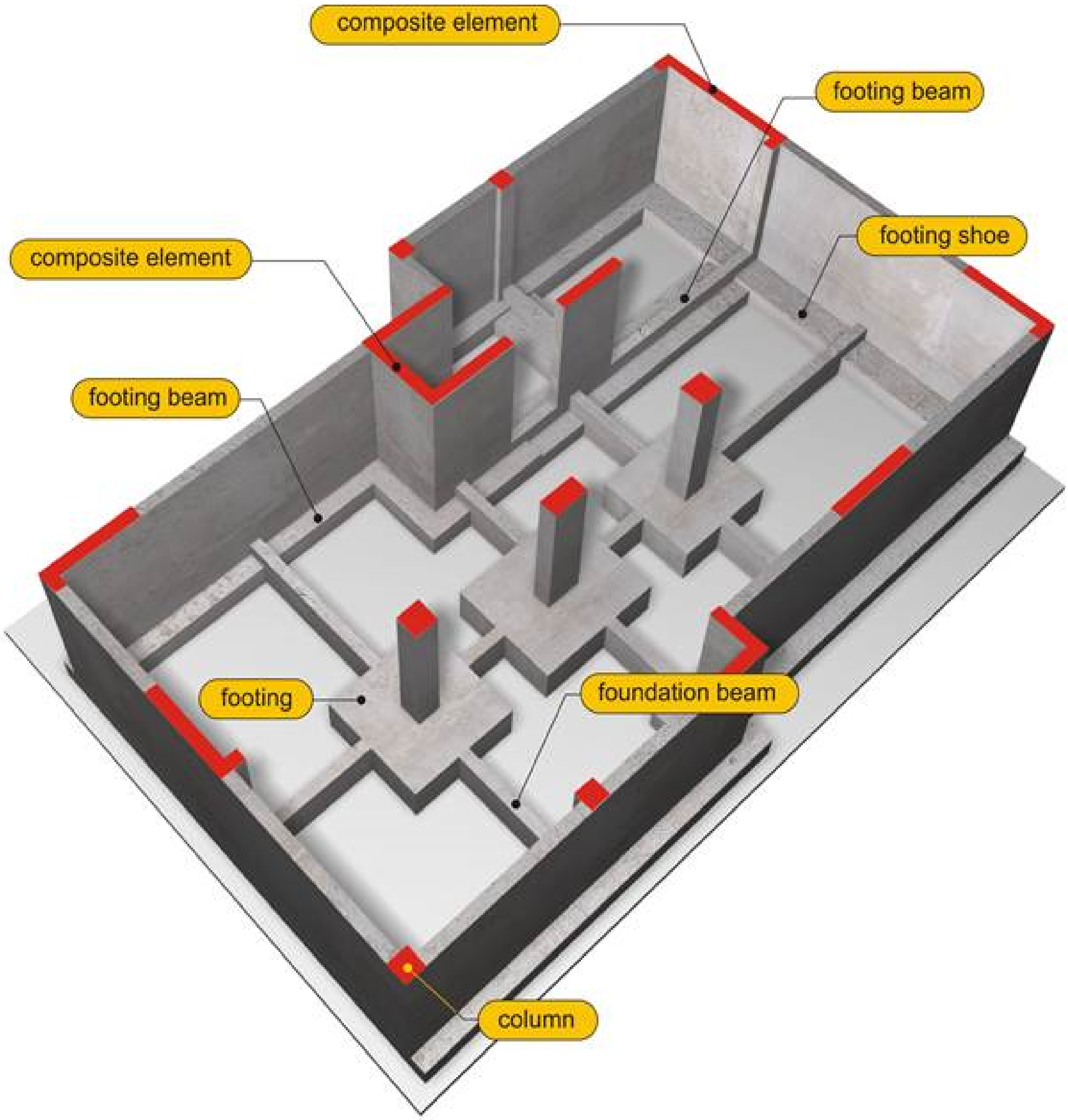


Figure 2: Foundation elements

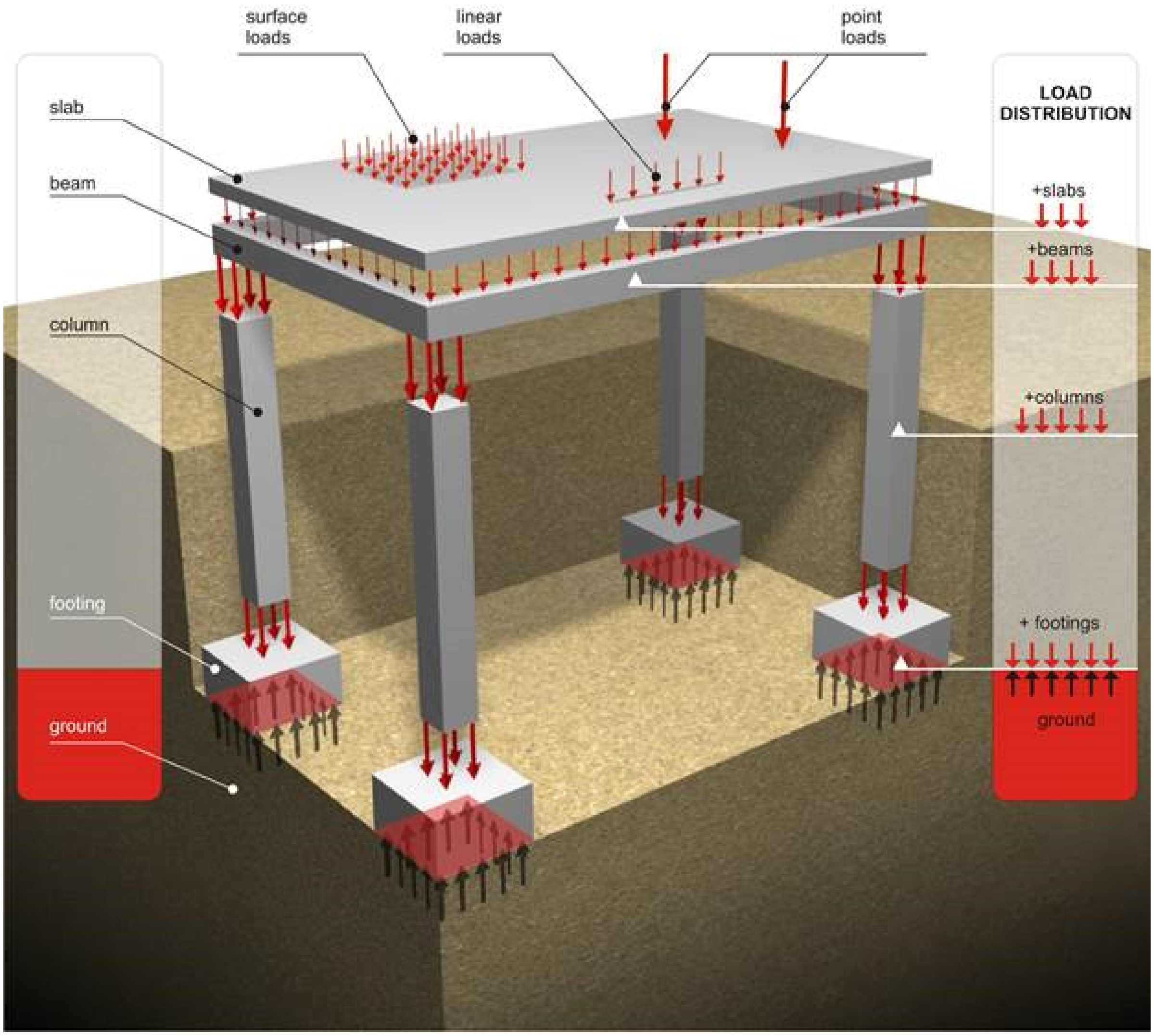


Figure 3: Gravity load distribution

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 according to BNBC in the context of Bangladesh,

I. DL+LL

II. 1.4DL+1.7LL

III. $0.75[1.4DL+1.7LL\pm1.7\{1.1(EQ_x \text{ or } EQ_y)\}] \sim 1.05DL+1.275LL\pm1.4(EQ_x \text{ or } EQ_y)$

IV. $0.75\{1.4DL+1.7LL\pm1.7(W_x \text{ or } W_y)\} \sim 1.05DL+1.275LL\pm1.275(W_x \text{ or } W_y)$

- Design the structural elements separately by considering their integrity and construction feasibility of that design.

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 peripherial 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

W.S.D Method

f'_c = Cylindrical strength of concrete

$f_c = 0.45\, f'_c$

f_y = Yield strength of reinforcement

$E_c = 33 \times w^{1.5} \times \sqrt{f'c}$

$n = \frac{Es}{Ec} = \frac{29x10^6}{145^{15}x33\sqrt{f'c}}$

$k = \frac{n}{(n+r)}$

j = 1- k/3

$R = \frac{1}{2} fc \times k \times j$

v_c =Allowable shear stress without web reinforcement $=1.1\sqrt{f'c}$

v = Allowable shear stress without web reinforcement $=5\sqrt{f'c}$

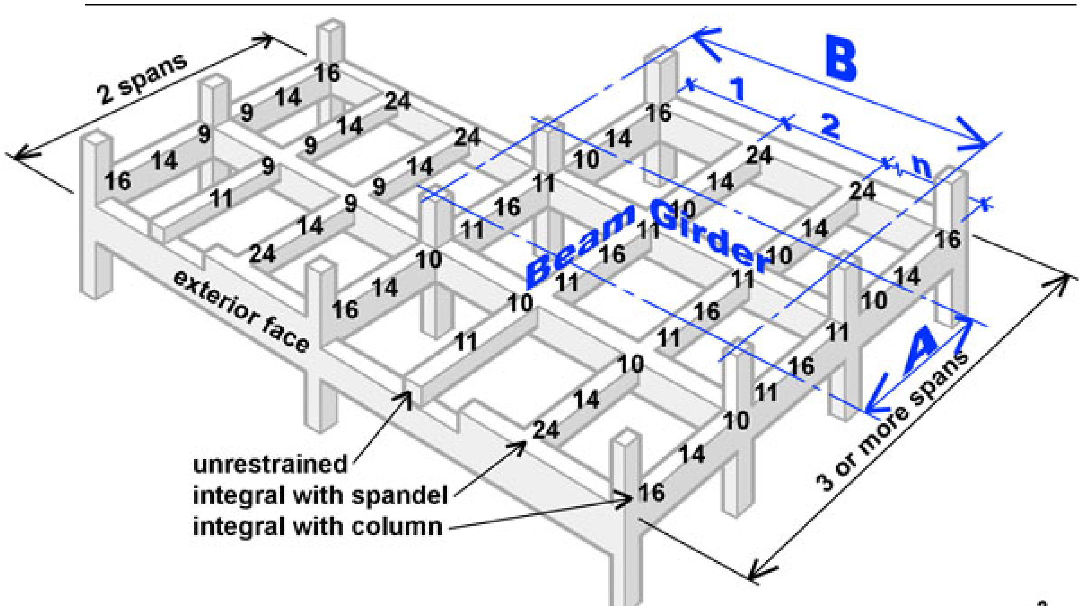
V_c =Allowable peripherial shear stress in slab and footing without web reinforcement $=2\sqrt{f'c}$

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

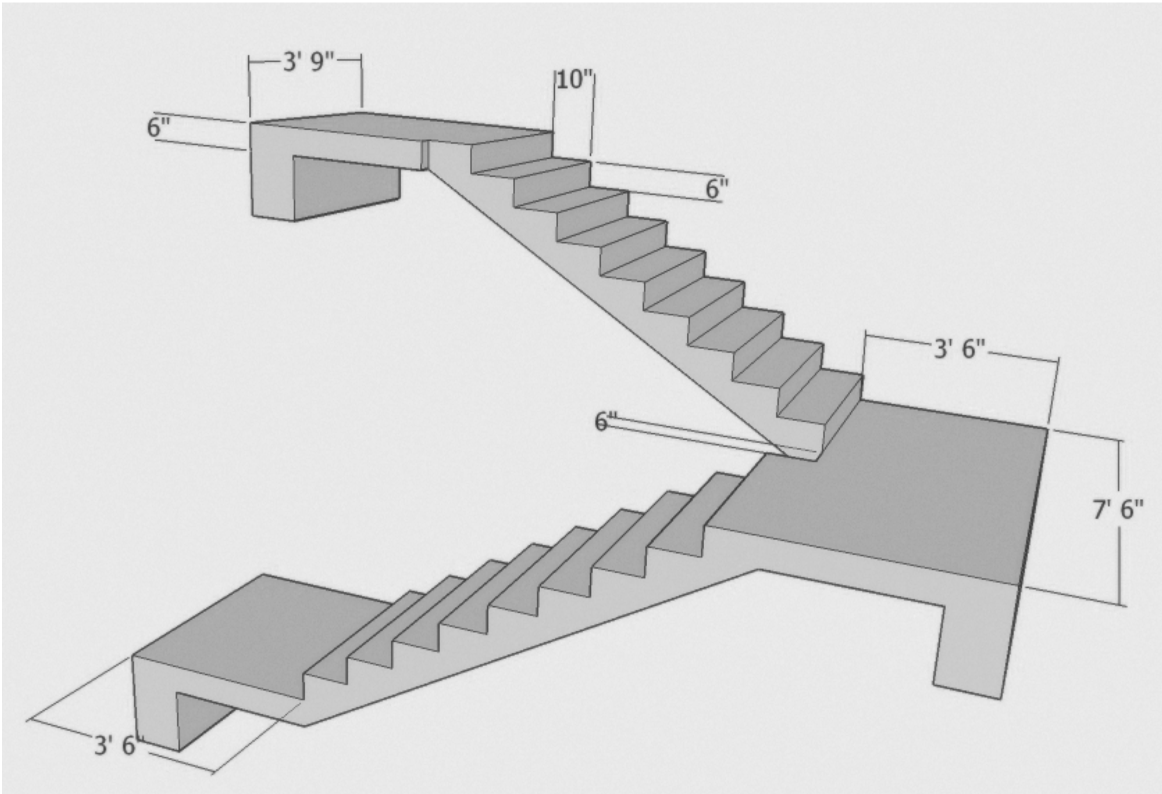


Figure 5: Typical stair

a) Assumptions and considerations

f_y= 60000 psi
f'_c= 3000 psi

Thickness of waist and landing slab = 6"

Live Load=82 psf = 0.082 ksf (BNBC)

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)

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

Total load, W= (0.082*1.7) + [1.4*(0.1 +0.025)] =0.31 ksf

c) Moment and reinforcement calculation

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

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

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	$\frac{0.0018 \times 60{,}000}{f_y}$

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

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

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

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

Again,

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

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

The distance between two cranked rod 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, 10"x12"

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

So, self-weight = (0.83*1*150)/1000 = 0.12k/ft

Load on Stair beam = $\frac{0.31*14.5*3.5}{7.5}$ + (0.42*9*0.12 + 0.12)*1.4 = 2.9 k/ft

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

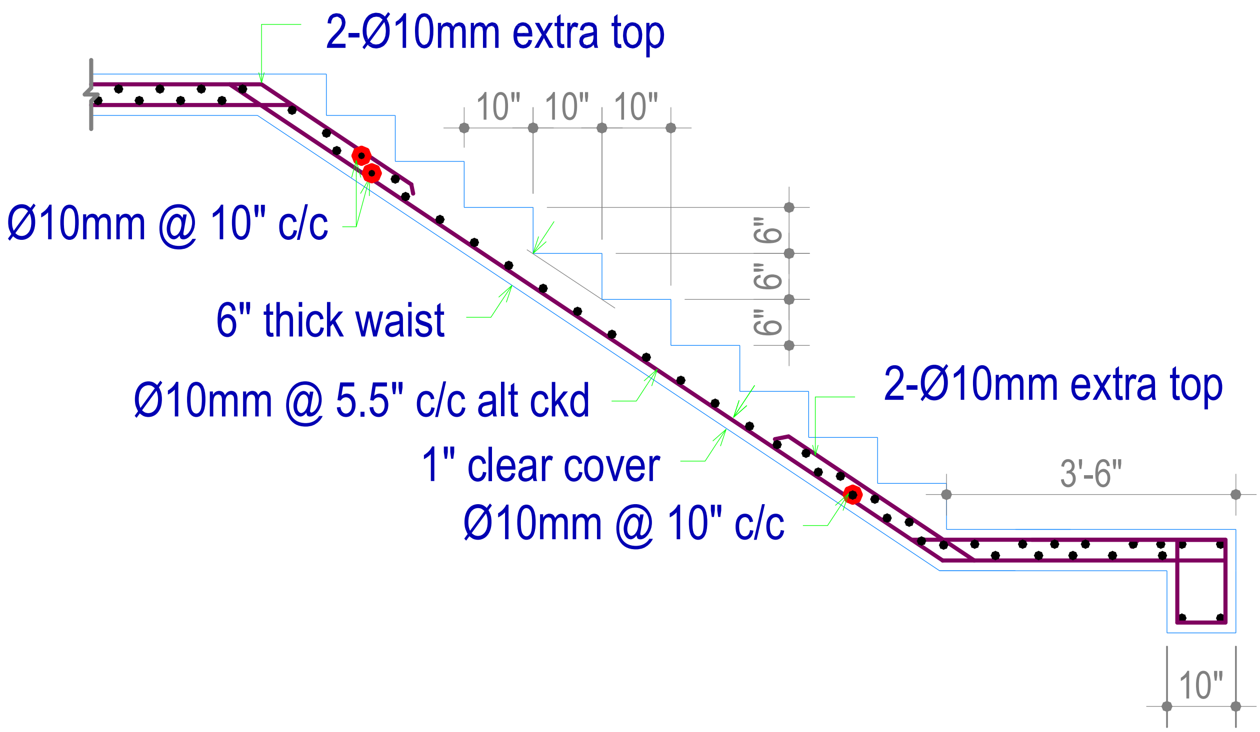


Figure 6: Reinforcement details of stair

1.4 Design of OWR

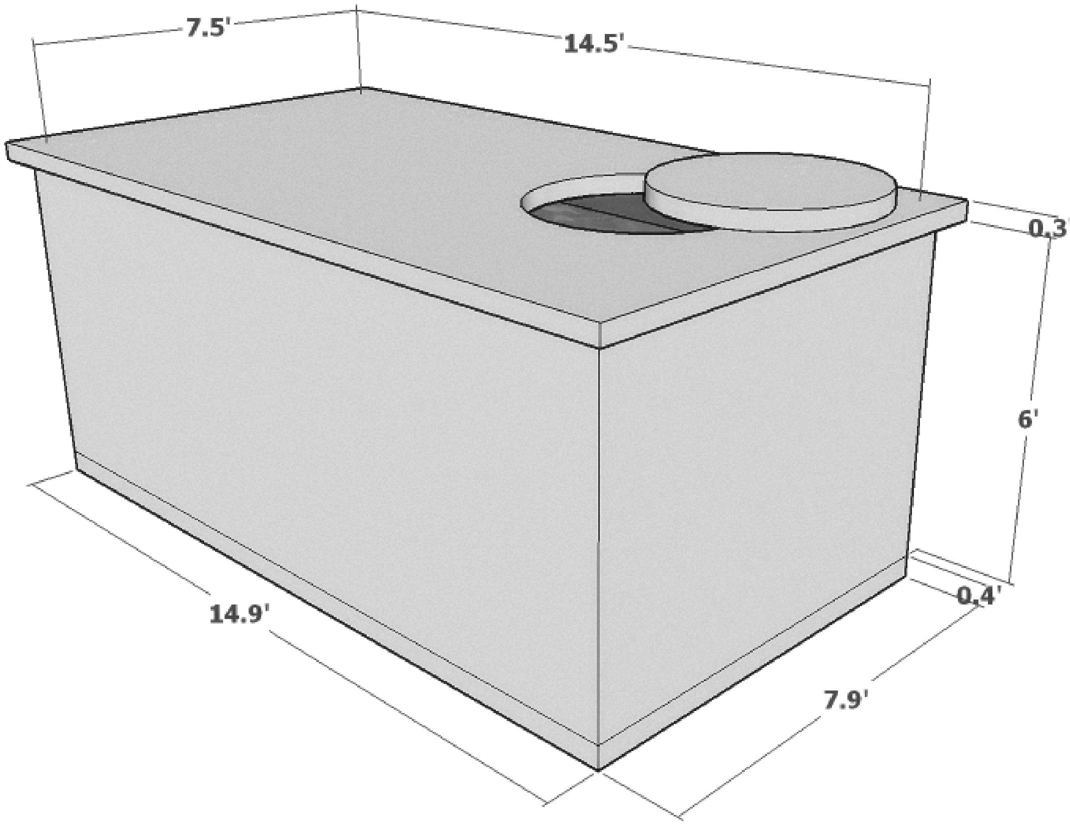


Figure 7: Roof top water reservoir (Overhead water reservoir)

a) Assumptions and considerations

$f'c$= 3000 psi

fy= 60000 psi

6^{th} floor building of 2 units & 5 members in each unit.
Water consuming 210 per capita per day (BNBC 1993)

b) Water reservoir size calculation

Total members= 6*2*5= 60 persons.
Total water consuming= 60*210 = 12600 litters for a full day.

$$= \frac{12600}{1000} m^3 = 12.6*3.28^3 = 445ft^3$$

Inner length & width of Reservoir are,
Length =14.5 ft and width = 7.5 ft (From plan)

so, Height= $\frac{445}{7.5*14.5}$ = 4.09 ft+1 ft = 5.09 ft~ 6 ft ; [where, free Board= 1 ft]
Height= 6 ft

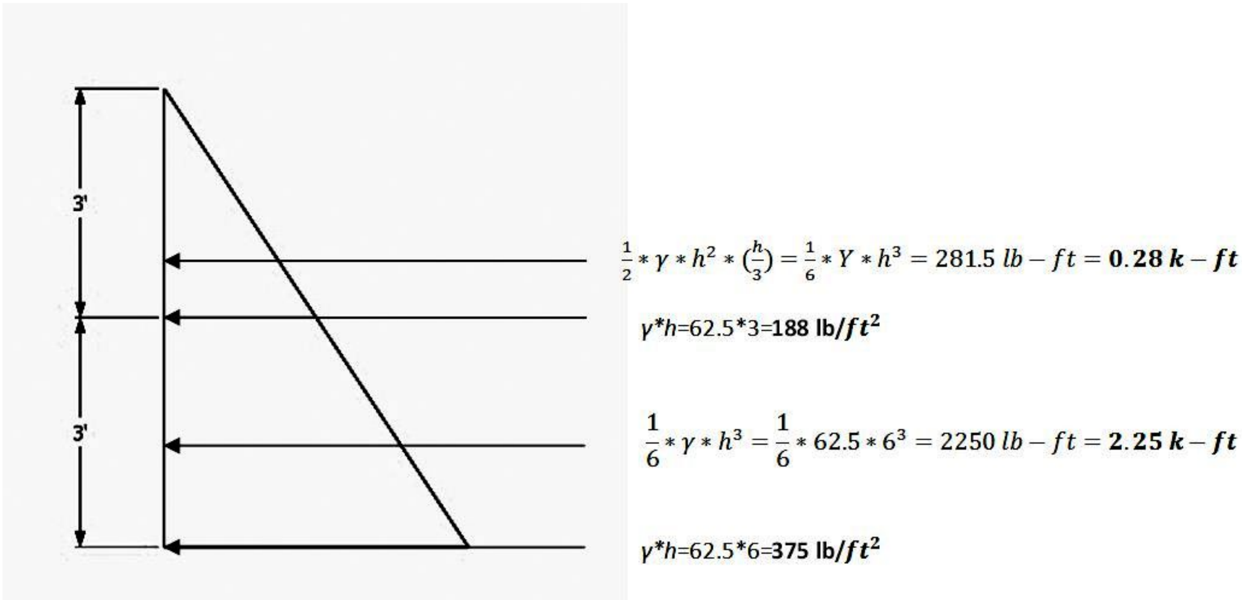


Figure 8: Pressure distribution on reservoir wall

<u>c) Vertical Reinforcement of wall</u>

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

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

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

$$d^2 = \frac{2.25*12}{0.9*0.0135*60*12*\left(1-0.59*\frac{0.0135*60}{3}\right)} = \frac{27}{7.59} = 3.56\ in^2$$

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

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

$$\mathbf{As} = \frac{M*12}{\phi * fy * \left(d - \frac{a}{2}\right)} = \frac{2.25*12}{0.9*60*\left(4 - \frac{0.25}{2}\right)} = 0.13 in^2/ft \text{ (controlled)}$$

$$a = \frac{As*fy}{.85*f'c*b} = \frac{0.13*60}{.85*3*12} = 0.26 \text{(ok)}$$

Now, $\frac{0.11*12}{0.13} = 10.15"$;

use, ϕ10mm@10" c/c.

d) Horizontal reinforcement of wall

Force $= \gamma * h * (\frac{14.5}{2} + \frac{14.5}{2}) = 62.5 * 6 * (\frac{14.5}{2} + \frac{14.5}{2}) = 5438$ lb

Again,

$\frac{\text{force}}{\text{stress}} = \frac{5438}{f_y} = \frac{5438}{60000} = 0.09\ in^2/ft$

As_{min} controls.

Now, $\frac{0.11*12}{0.12} = 11"$;

Use ϕ10 @ 11" c/c

e) Design of bottom slab

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

Simply supported	$l/20$
One end continuous	$l/24$
Both ends continuous	$l/28$
Cantilever	$l/10$

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

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

$$\frac{5 w_A l_A^4}{384\ EI} = \frac{5 w_B l_B^4}{384\ EI}$$

$$w_A\, l_A^4 = w_B\, l_B^4$$

$$w_A = w_B \left(\frac{l_B}{l_A}\right)^4$$

$w_A = 15.63 * w_B$

$w_A + w_B = 56.25$ psf

w_B = 3.38 psf

w_A = 52.87 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*1.7) + [1.4*(0.05287 +0.025)] =0.75 ksf

<u>Moment for short direction</u>

$M^{+} = \frac{wL^2}{14} = \frac{0.75*7.5^2}{14}$ = 3 k-ft/ft

$M^{-} = \frac{wL^2}{24} = \frac{0.75*7.5^2}{24}$ = 1.75 k-ft/ft

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

$$d^2 = \frac{3*12}{0.9*0.015*60*12*\left(1-0.59*\frac{0.015*60}{3}\right)} = 4.5$$

d = 2.12" < provided, 3.5" (ok)

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

$$+As = \frac{M*12}{\phi*fy*\left(d-\frac{a}{2}\right)} = \frac{3*12}{0.9*60*\left(3.5-\frac{0.4}{2}\right)} = 0.2\ in.^2$$

$$a = \frac{As*fy}{.85*f'c*b} = \frac{0.2*60}{.85*3*12} = 0.39\ (ok)$$

Now, $\frac{0.11*12}{0.2} = 6.6"$;

Use ϕ10mm @ 6.5" c/c alt. ckd and 1-ϕ10mm as extra top.

f) Top slab

For 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}.

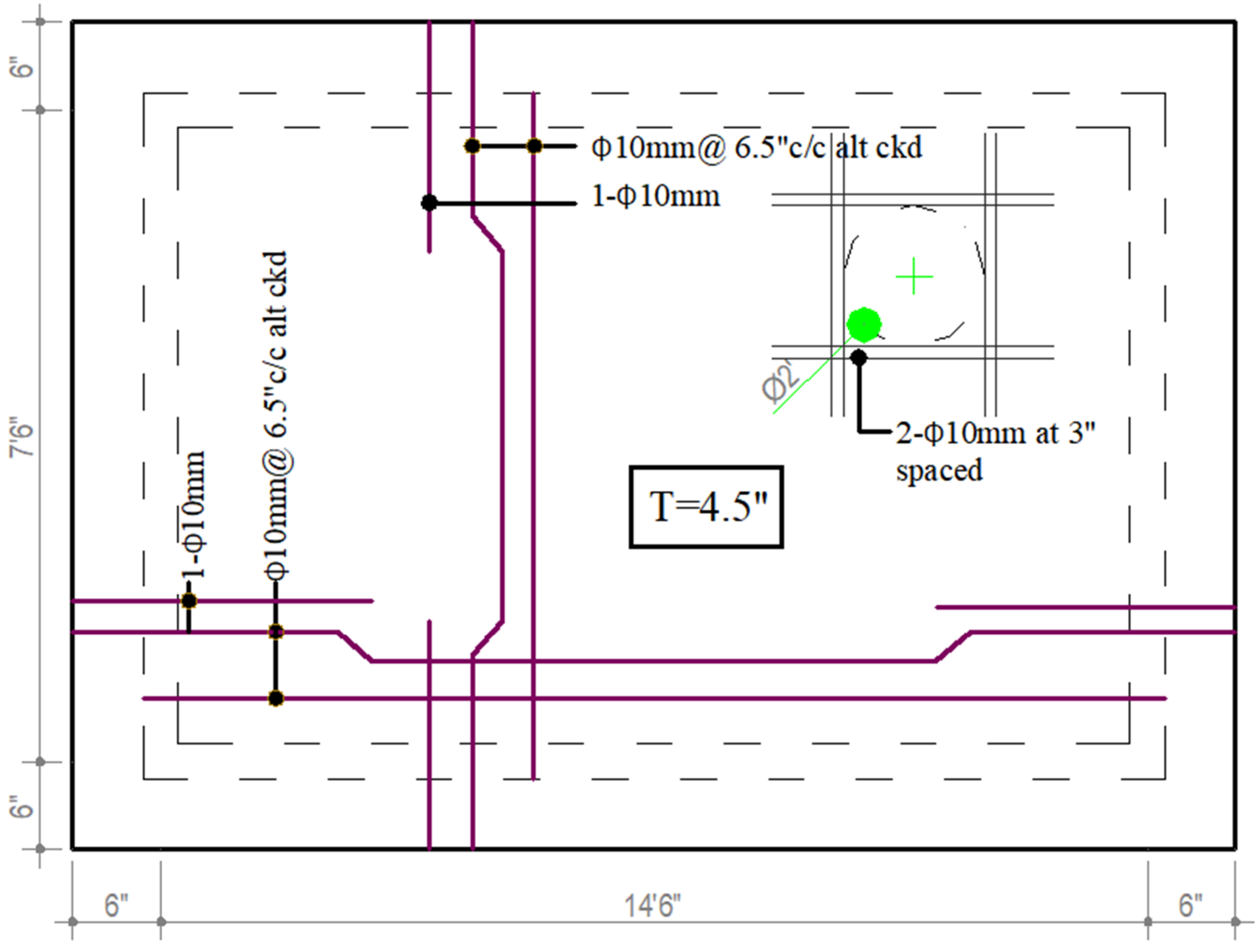


Figure 9: Reinforcement details of top slab overhead water reservoir

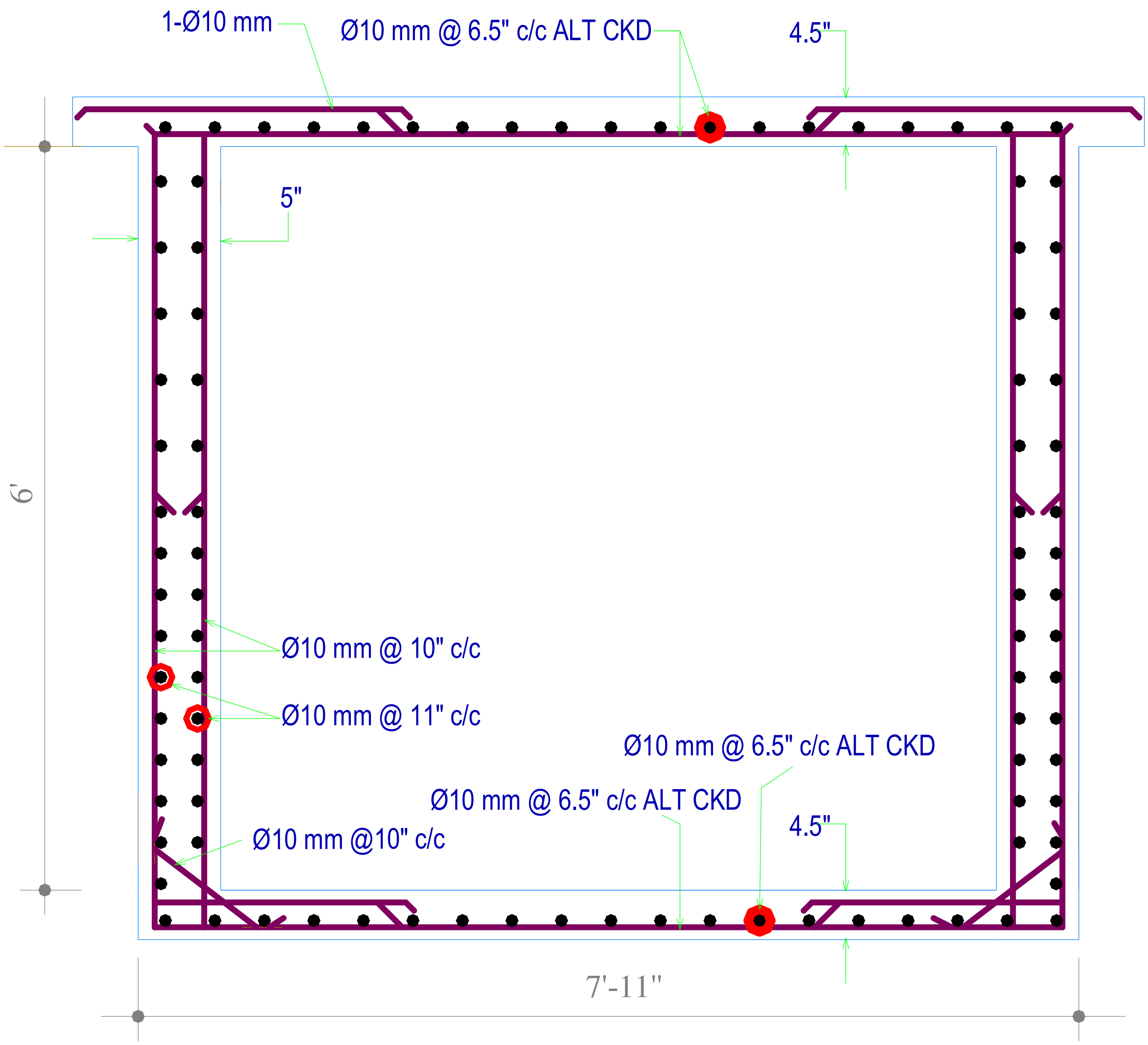


Figure 10: Reinforcement details of roof top water reservoir (elevation view)

g) Load on beam

Here, Load from Bottom Slab = 0.75 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

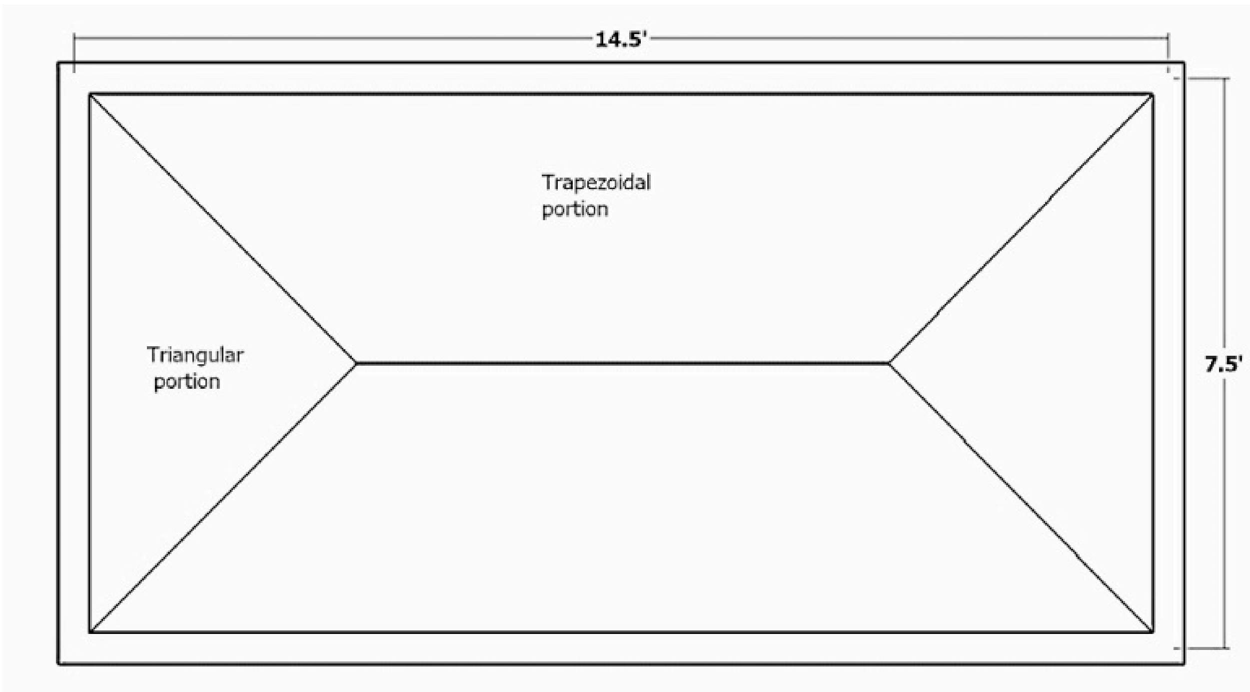


Figure 11: Load distribution of slab

Trapezoidal portion,

$$= \frac{\frac{1}{2} * (14.5 + 7) * 3.75 * (0.75 + .056 * 1.4)}{14.5} + 0.12 * 1.4 + (0.42 * 6 * 0.15) * 1.4 = 3 \text{ k/ft}$$

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

1.5 Lateral Loads Calculation of Residential Building

a) Earthquake Load Calculation:

From BNBC (2006)

Seismic Zone-coefficient, Z=0.15 [Dhaka]

Structural Importance Coefficient, I=1

Response Modification Coefficient, R=8

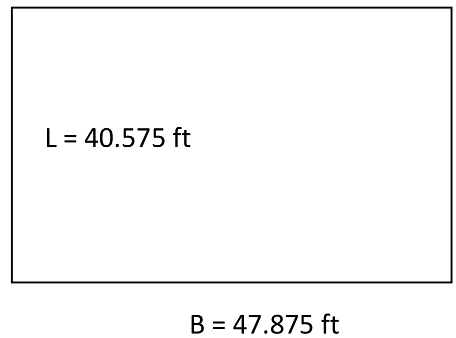


Figure 12: Plan of the building

Now,

Numerical Co-efficient,

$C=\frac{1.25*S}{T^{\frac{2}{3}}} = \frac{1.25*1.5}{(0.61)^{\frac{2}{3}}} = 2.6 < 2.75$

$s_2=1.5$

Assume,

Height of structure from base = 56 ft
Dead load on each floor = 175 kip

$T = C_t * (h_n)^{3/4} = 0.073*(\frac{56}{3.28})^{3/4} = 0.61<0.7;$

$W= DL* Area* Storied= 175*47.875*40.575*5*\frac{1}{1000}=1699.7\ k\sim1700\ k$

$$V = \frac{Z*I*C*W}{R} = \frac{0.15*1*2.6*1700}{8} = 82.9\ \text{kip}$$

$W_i= DL*Area*\frac{1}{1000} = 175*47.875*40.575*\frac{1}{1000} = 339.9\ k \sim 340\ k$

$\Sigma W_i * h_i = 340*(6+16+26+36+46+56) = 63240$

Here, $F_t=0$ as, $T<0.7$

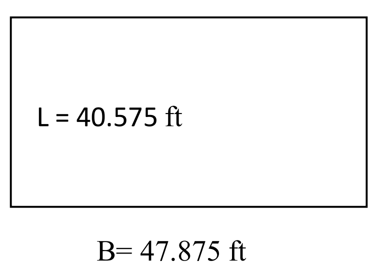
Load on each floor, $F_x = \frac{(V-F_t)*W_i*h_x}{\Sigma W_i*h_i}$

$F_x = \frac{(82.9-0)*340*h_x}{53040} = 0.446 * h_x$

Table 4: Equivalent earthquake forces at different levels.

Floor	**h_x**	**Force, $F_x = 0.53 * h_x$**
Grade beam	6 ft	2.676
Ground floor	16 ft	7.136
1^{st}	26 ft	11.596
2^{nd}	36 ft	16.056
3^{rd}	46 ft	20.516
4^{th}	56 ft	24.976
	Total =	82.956

b) Wind Load Calculation



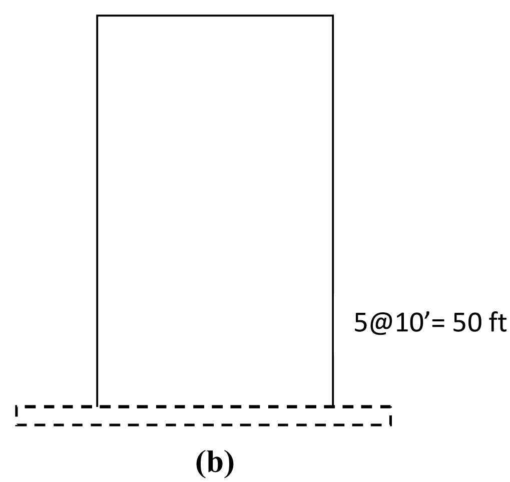


Figure 13: a) Plan b) Elevation of the building

Here,
Gust Co-efficient, $C_G = 1.43$
$C_c = 47.2 * 10^{-6}$

B= 47.875ft
L= 40.575ft
Heght, h= 50ft

Now,
Important Co − efficient, $C_I = 1.00$
Combined height & $exposure\ Co - efficient, C_z$ = Table 6.2.10 (BNBC 2006)

Wind Velocity= 210 $\frac{km}{hr}$ (Dhaka)

$q_z = C_C * C_I * C_z * {V_b}^2 = 2.08 * C_z$

$C_z = 0.1879*z^{0.4435} \geq 0.368$ (z = ht. above ground in meter)

Table 5: Overall pressure coefficients, C_p for rectangular building with flat roof. (Ref: BNBC 2006)

h/B	L/B					
	0.1	0.5	0.65	1.0	2.0	≥ 3.0
≤ 0.5	1.40	1.45	1.55	1.40	1.15	1.10
10.0	1.55	1.85	2.00	1.70	1.30	1.15
20.0	1.80	2.25	2.55	2.00	1.40	1.20
≥ 40.0	1.95	2.50	2.80	2.20	1.60	1.25
Note:(1) These coefficients are to be used with Method-2 given in Sec 2.4.6.6a(ii). Use $\bar{C}_p = \pm 0.7$ for roof in all cases. (2) Linear interpolation may be made for intermediate values of $\bar{h}/B$ and L/B.						

Here,

$$\frac{L}{B} = 0.85;$$
$$\frac{h}{B} = 1.04;$$
$$\therefore\ C_p = 1.49$$

$P_z = C_G * C_p * q_z = 4.43 * C_z$

Table 6: Equivalent wind forces at different floor levels.

Height (m)	C_z	q_z	$p_z = (\frac{kN}{m^2})$	$F_z = P_z*A$ (kN)	F_z (kN)	F (Kip)
3.048	0.368	0.76544	1.63	1.63*3.0478*14.6	73.54	16.42
6.096	0.415	0.8632	1.84	1.84*3.0478*14.6	81.88	18.28
9.144	0.498	1.03584	2.21	2.21*3.0478*14.6	98.35	21.95
12.192	0.57	1.1856	2.53	2.53*3.0478*14.6	112.6	25.13
15.24	0.63	1.3104	2.79	$2.79*\frac{3.0478}{2}*14.6$	62.08	13.86

1.6 Design of Floor Slabs

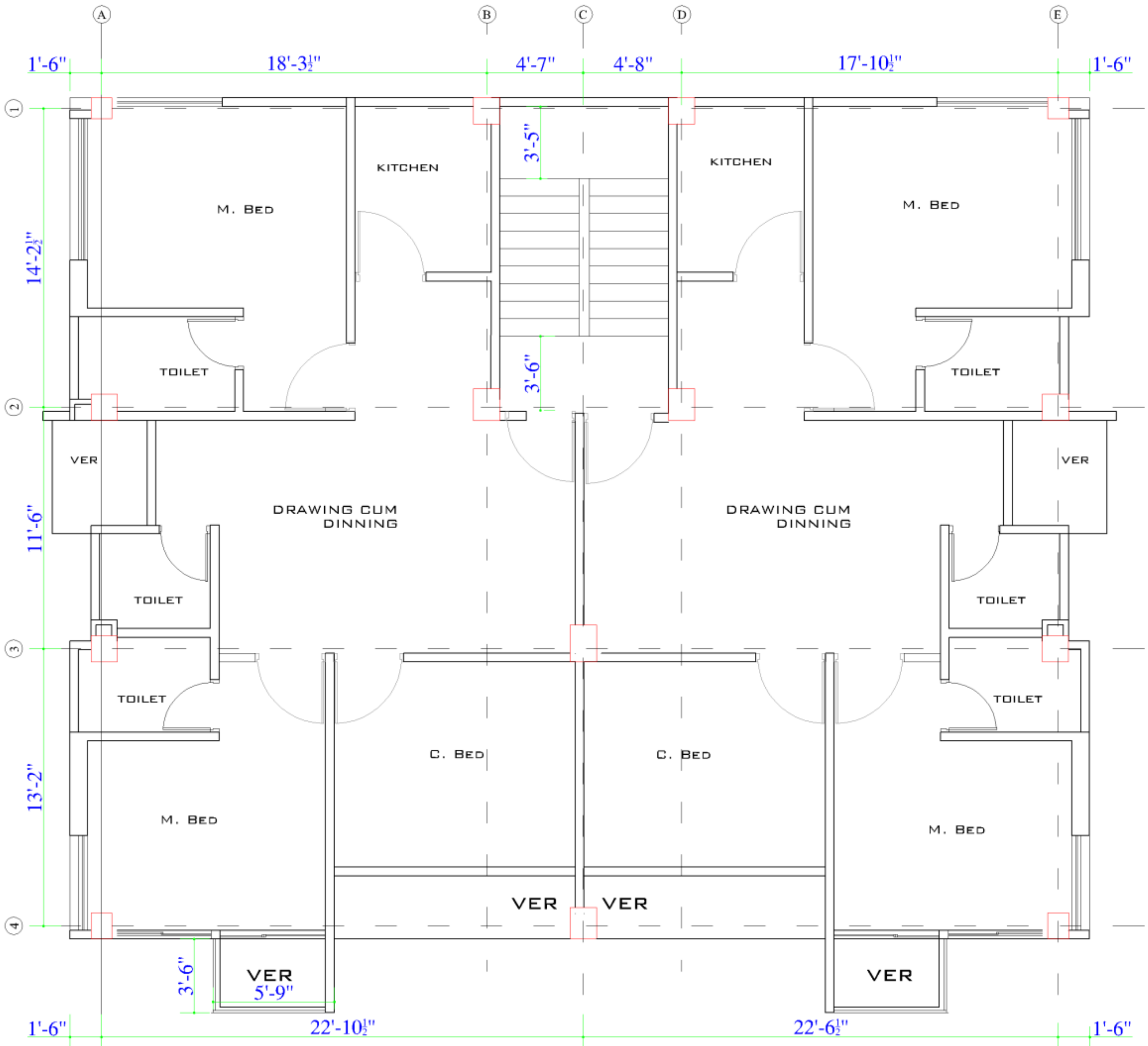


Figure 14: Typical floor plan

a) Assumptions and considerations

f'c=3000 psi
fy= 60000 psi

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

Considering the largest two panels of 22'-10"x13'-2" and 22'-10"x11'-6

So, β = $\frac{22.83}{13.17} = 1.73$

Thickness, t = 5.8 in. ≈ 5.5 in.

b) Load calculation

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

Floor finish = 30 psf
Partition wall = 40 psf
Live Load = 40 psf (BNBC)

$W_{DL} = 69 + 30 + 40 = 139\ psf * 1.2 = 167\ psf$
$W_{LL} = 40 = 40\ psf * 1.6 = 64\ psf$
Total, W = (167+64) = 231psf

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 2006)

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= $\{C_{A(DL)} * W_{(DL)} * A^2\} + \{C_{A(LL)} * W_{(LL)} * A^2\} = 2.22\ k - ft/ft$

short distance A, -M = $\{-C_A * W * A^2\}$ = 3.57 k-ft/ft

For, case 9

Short distance A, +M= $\{C_{A(DL)} * W_{(DL)} * A^2\} + \{C_{A(LL)} * W_{(LL)} * A^2\}$= 1.41 k-ft/ft

short distance A, -M = $\{-C_A * W * A^2\}$ = 2.69 k-ft/ft

So, in short direction –M = $3.5\ k - ft/ft$ and +M = $2.22\ k - ft/ft$

$As_{min} = 0.002 * b * t = 0.002 * 12 * 5.5$= 0.132 in^2/ft

$+As = \frac{M*12}{\phi*fy*\left(d-\frac{a}{2}\right)} = \frac{2.22*12}{0.9*60*\left(4.5-\frac{0.2}{2}\right)}$ = 0.11 $in.^2/ft$

$a = \frac{As*fy}{.85*f'c*b} = \frac{0.11*60}{.85*3*12} = 0.2$**(ok)**

Now,$\frac{0.11*12}{0.13} = 10.15$"; use, ϕ10mm@10" c/c alt. ckd.

Again,

$-As = \frac{M*12}{\phi*fy*\left(d-\frac{a}{2}\right)} = \frac{3.5*12}{0.9*60*\left(4.5-\frac{0.3}{2}\right)}$= 0.18 $in.^2/ft$ (controlled)

$a = \frac{As*fy}{.85*f'c*b} = \frac{0.5*60}{.85*3*12} = 0.3$ **(ok)**

The distance between two cranked rods is 20".

So, Required reinforcement = 0.18 - $\frac{0.11*12}{20} = 0.114$ in^2/ft

The extra negative reinforcement required, 20 / ($\frac{0.11*12}{0.114}$) = 20/11.58 = 1.73 ~ 2 So, use 2-Ø10mm 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 ϕ10mm@10" c/c alt. ckd and 2-Ø10*mm* as extra top.

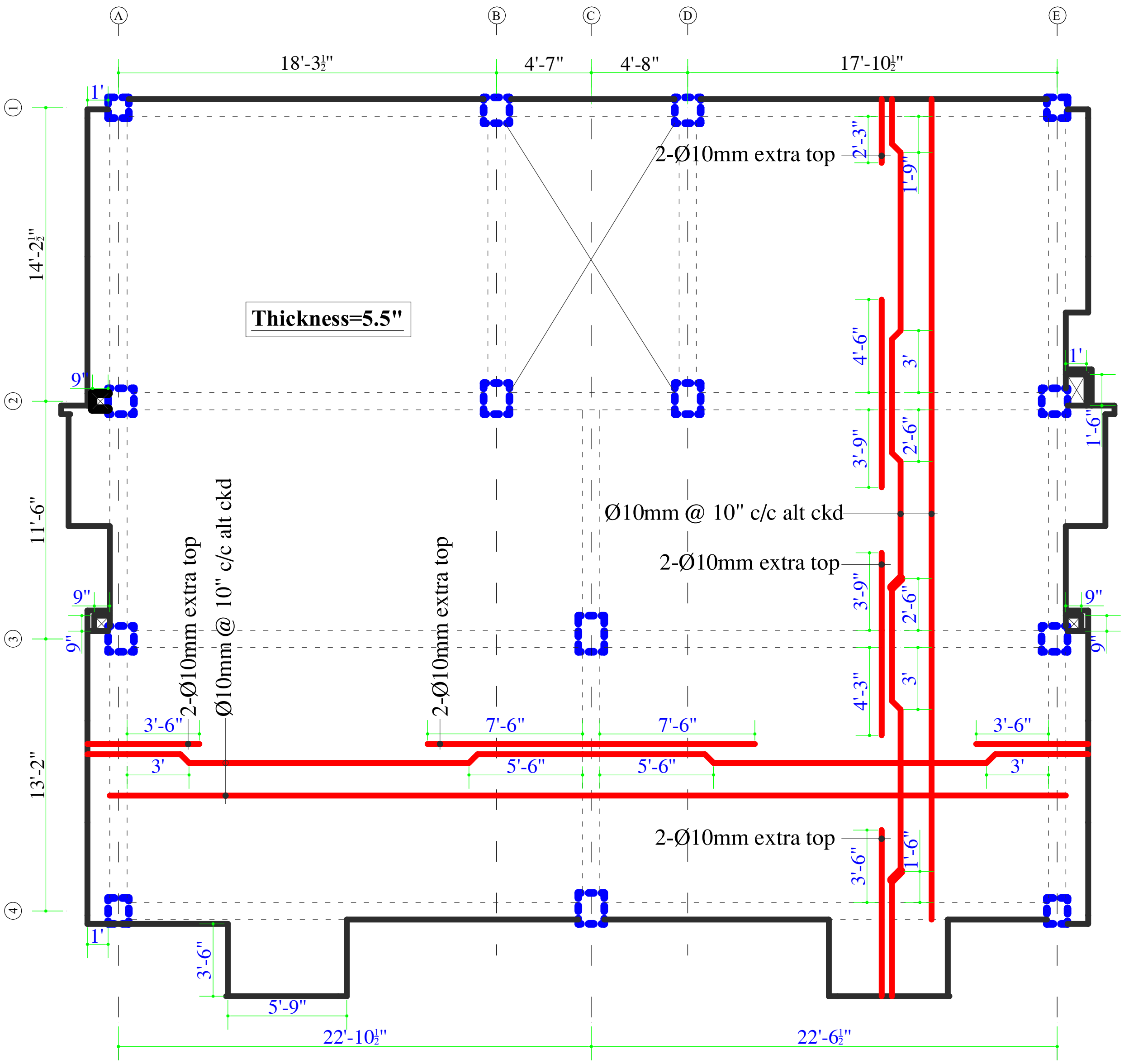


Figure 15: Reinforcement Details of Slab

1.7 Design of Floor Beams

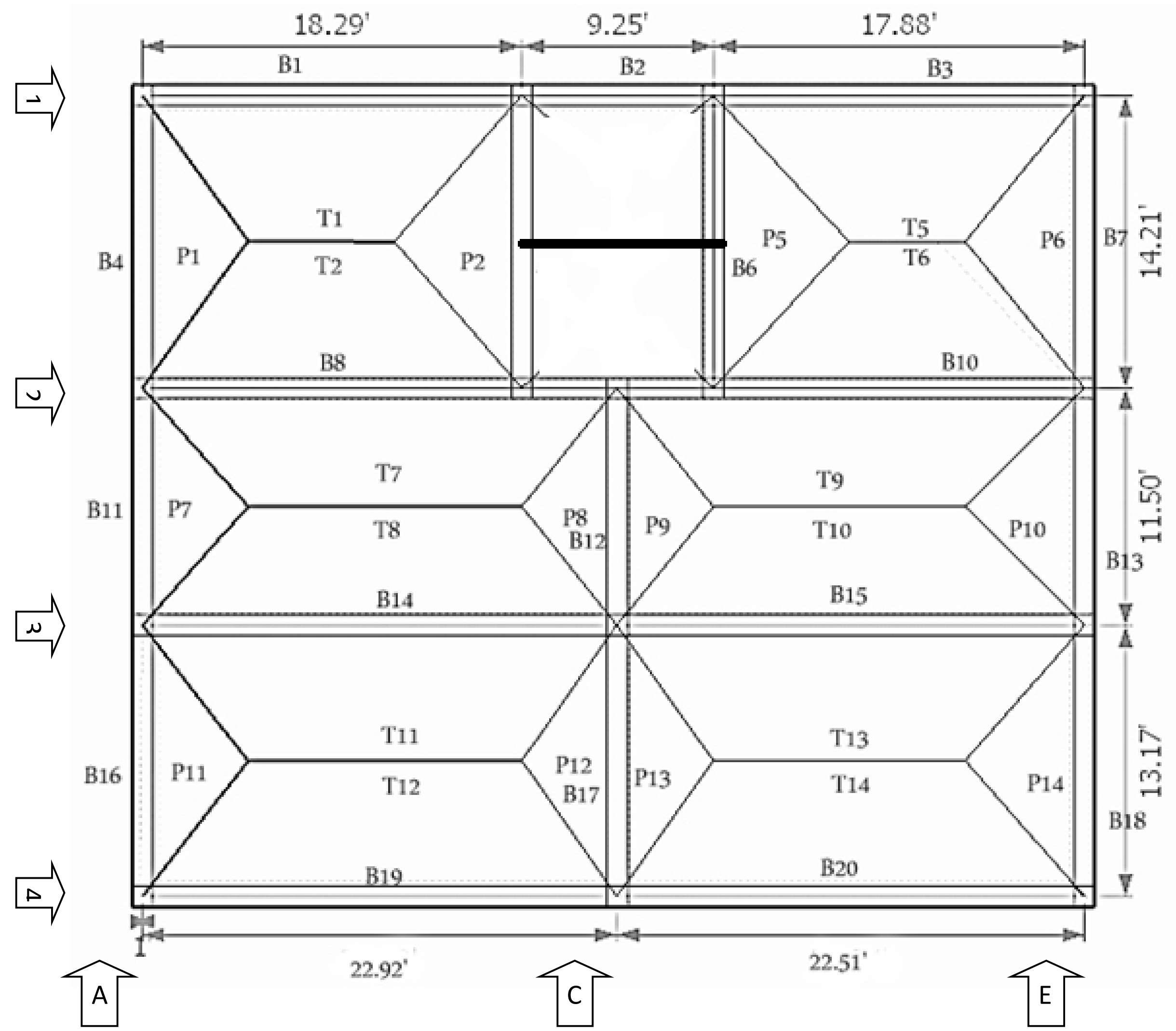


Figure 16: Beam layout

a) Assumptions and considerations

Load on slab, W = 231 psf
f_c'=3000 psi
fy = 60000 psi

b) Load calculation

Beam in-between A and B grid on grid 2

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 = $\frac{0.231*98.73}{22.92} + \frac{0.231*107.56}{22.92}$ = 2.08 kip/ft

Partition wall on beam = 0.42* 9* 120 = 0.45 * 1.2 = 0.54 k/ft

Total load = 0.27 + 2.08 +0.54 = 2.89 kip/ft

<u>c) Moment and reinforcement</u>

At grid 3-A joint

- $M_u = \frac{wl^2}{16} = \frac{2.89*22.92^2}{16}$ = 94.9 kip-ft = 1138.64 kip-in

At grid 3-C joint

- $M_u = \frac{wl^2}{9} = \frac{2.89*22.92^2}{9}$ = 168.7 kip-ft = 2024.26 kip-in

At mid span

+ $M_u = \frac{wl^2}{14} = \frac{2.89*22.92^2}{14}$ = 108.44 kip-ft = 1301.3 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.

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

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

$$\emptyset M_n = 0.9 * 1497.54 = 1347.8\ k - in > M_u = \ 1301.3\ kip - in$$

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

$$\emptyset M_n = 0.9 * 1497.54 = 1347.8k - in < M_u = \ 2024.26\ kip - in$$

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, $a \ = 5''$

$$-A_s = \frac{{M_u}/{\emptyset}}{f_y(d - {a}/{2})} = \frac{{1138.64}/{0.9}}{60(13.5 - {5}/{2})} = 1.92in^2$$

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

$$-A_s = \frac{{1138.64}/{0.9}}{60(13.5 - {3.76}/{2})} = 1.81in^2$$

$$a \ = \frac{1.81 * 60}{0.85 * 3 * 12} = 3.55''$$

$$-A_s = 1.79in^2$$

For midspan,

Assume, $a \ = 3''$

$$+A_s = \frac{{M_u}/{\emptyset}}{f_y(d - {a}/{2})} = \frac{{1301.3}/{0.9}}{60(13.5 - {3}/{2})} = 2.00in^2$$

$$a \ = \frac{A_s f_y}{0.85 f_c' b} = \frac{2 * 60}{0.85 * 3 * 12} = 3.92''$$

$$+A_s = \frac{{1301.3}/{0.9}}{60(13.5 - {3.92}/{2})} = 2.09in^2$$

$$a = \frac{2.09 * 60}{0.85 * 3 * 12} = 4.1''$$

$+A_s = 2.1\ in^2$

For grid 3-C joint, Remaining moment, $M_1 = \frac{2024.26}{0.9} - 1497.54 = 751.64\ kip - in$

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, $-A_s' = \frac{751.64}{43.5\ (13.5-2.5)} = 1.57\ in^2$

Total area of tensile reinforcement at 60 ksi, $A_s = 2.2 + 1.57 * \frac{43.5}{60} = 3.34\ in^2$

The summaries of reinforcement are as follows,

At mid span, $+A_s = 2.1$ in^2
Grid 3-A joint, $-A_s = 1.79$ in^2
Grid 3-C joint, $-A_s = 3.34$ in^2 (tension) and compressive reinforcement, 1.57 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 $\frac{2.1}{3} = 0.7 in^2$ all through as positive reinforcement but it is less than compressive reinforcement 1.57 in^2.

Provide $\frac{3.56}{3} = 1.19 in^2$ all through as negative reinforcement. For a beam having width of 12", it is difficult to place more than three reinforcement in a row and more than five reinforcement in a face.

d) Shear design

$V_u = 0.5WL = 0.5 * 2.89 * 22.92 = 33.12\ k$
$\emptyset * V_a = 2 * \emptyset * \sqrt{f_c'} b * d = 2 * 0.75 * \sqrt{3000} * 12 * 13.5 = 13.3\ kip$

Use Ø10mm as shear reinforcement.

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

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

$s_{max} = 24''$

$s = \frac{\emptyset A_v f_y d}{V_u - \emptyset V_a} = \frac{0.75*2*.121*60*13.5}{33.12-13.3} = 6.67''$

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

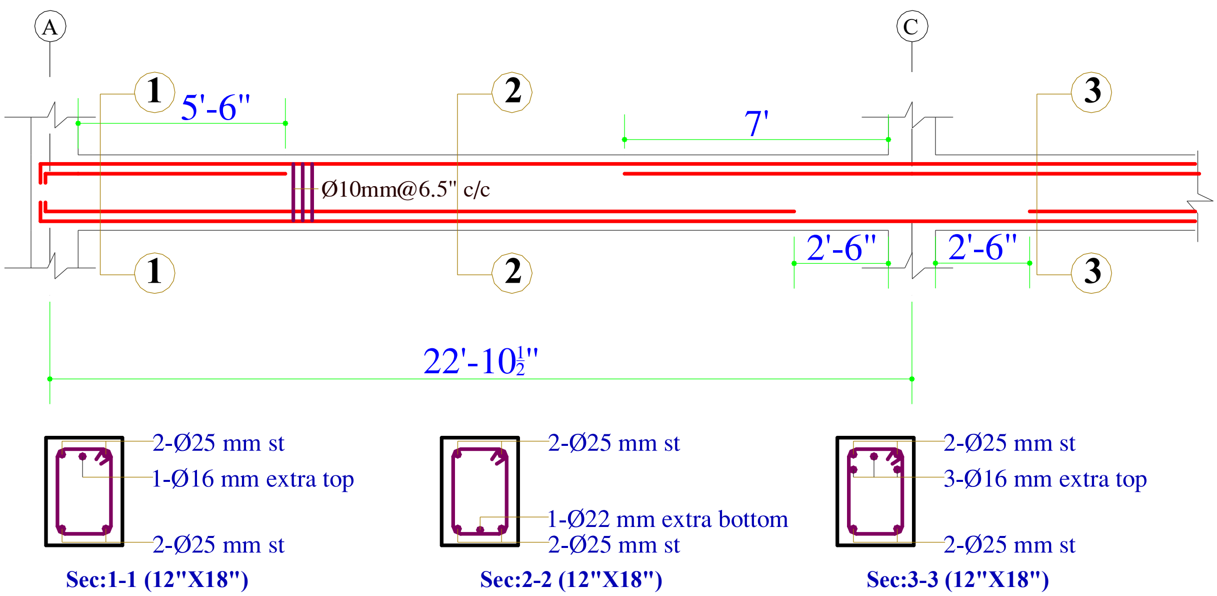


Figure 17: Reinforcement detail of beam

1.8 Design of Column

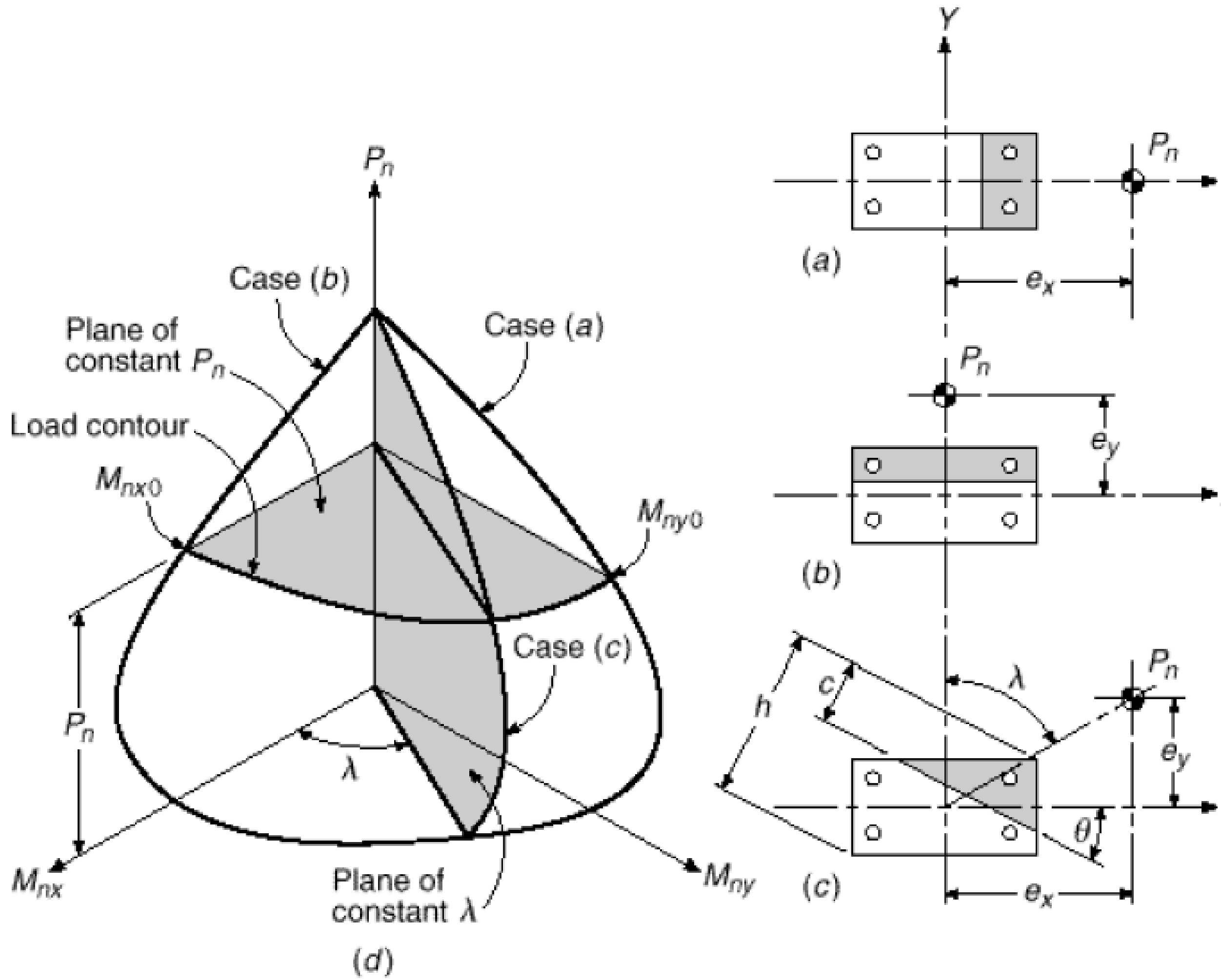


Figure 18: Interaction diagram for compression plus biaxial bending a) uniaxial bending about Y axis; b) uniaxial bending about X axis; c) biaxial bending about diagonal axis; d) interaction surface. (Ref: ACI Code, Design of Concrete Structure, 13th edition, Chap-8, P-274)

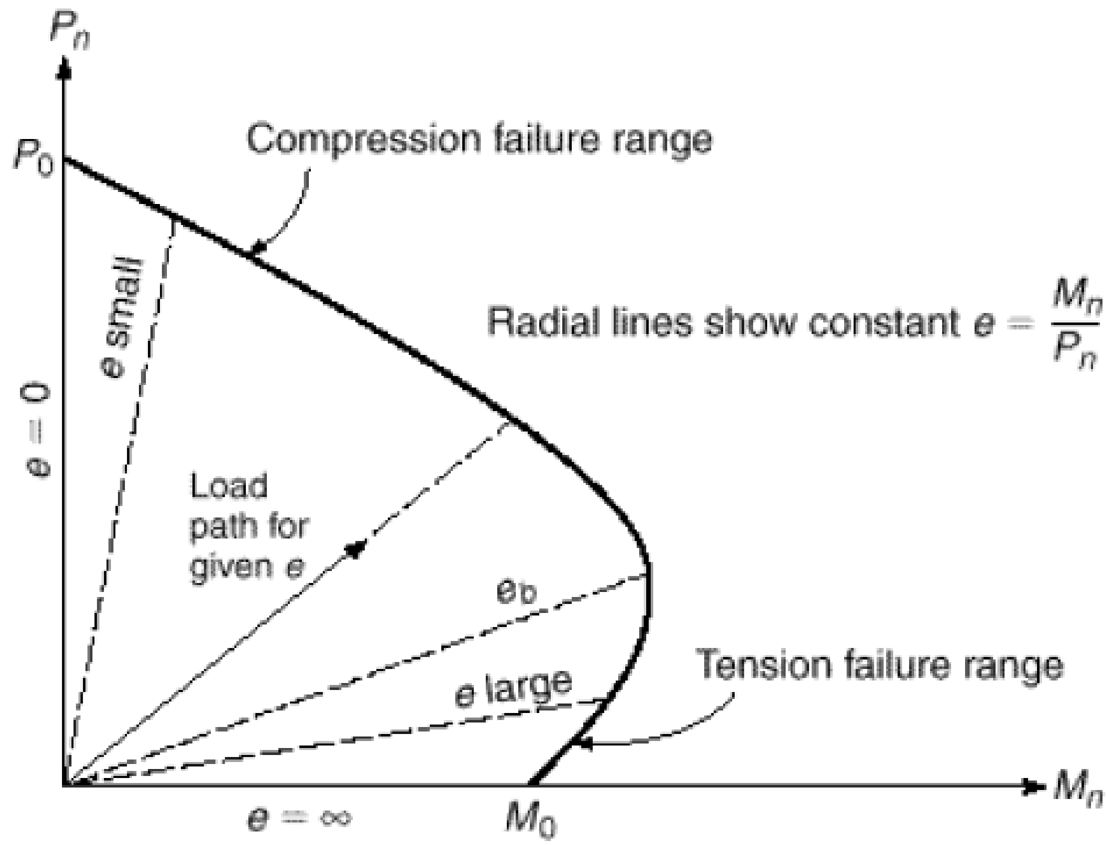


Figure 19: Interaction diagram for nominal column strength in combined bending and axial load. (Ref: ACI Code, Design of Concrete Structure, 13th edition, Chap-8, P-260)

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, $\rho_g = 2\%$

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>

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

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

$e_x/h = 2.6/18 = 0.14$

From graph, $K\eta = 0.79$

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

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, $K\eta = 0.85$

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

$P_x = 918$ k

For P_o, $K\eta = (1.1+1.12)/2 = 1.11$

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

$P_o = 1200$ k

$$\frac{1}{P_n} = \frac{1}{P_x} + \frac{1}{P_y} - \frac{1}{P_o}$$

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

ϕ P_n = 0.65*700 k =455k < 554 k (not ok)

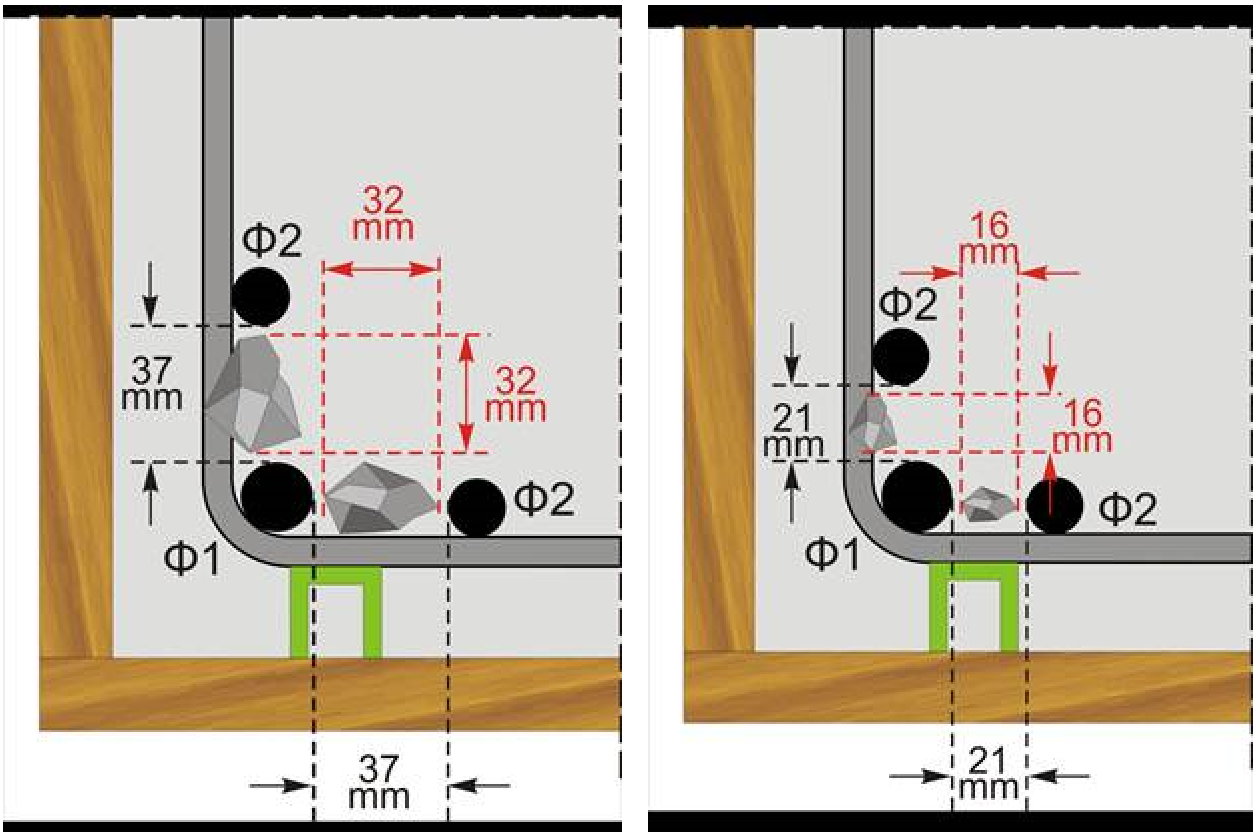


Figure 20: Minimum spacing between reinforcement bars

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

Ø10mmbars 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.

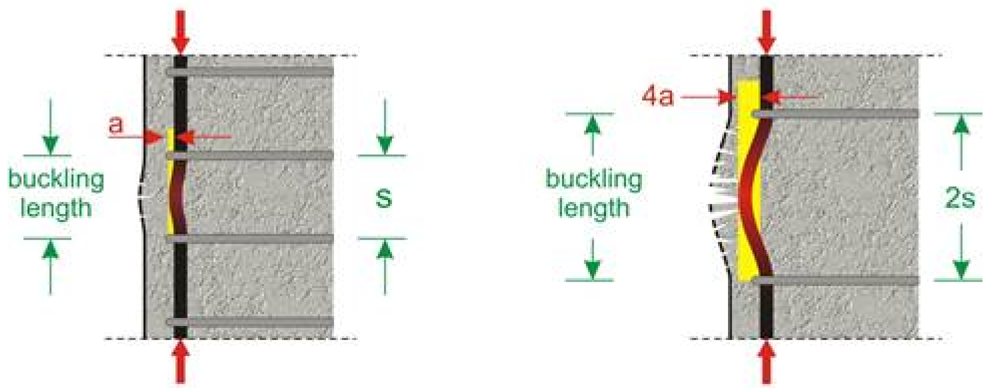


Figure 21: Failure mechanism of a column

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.

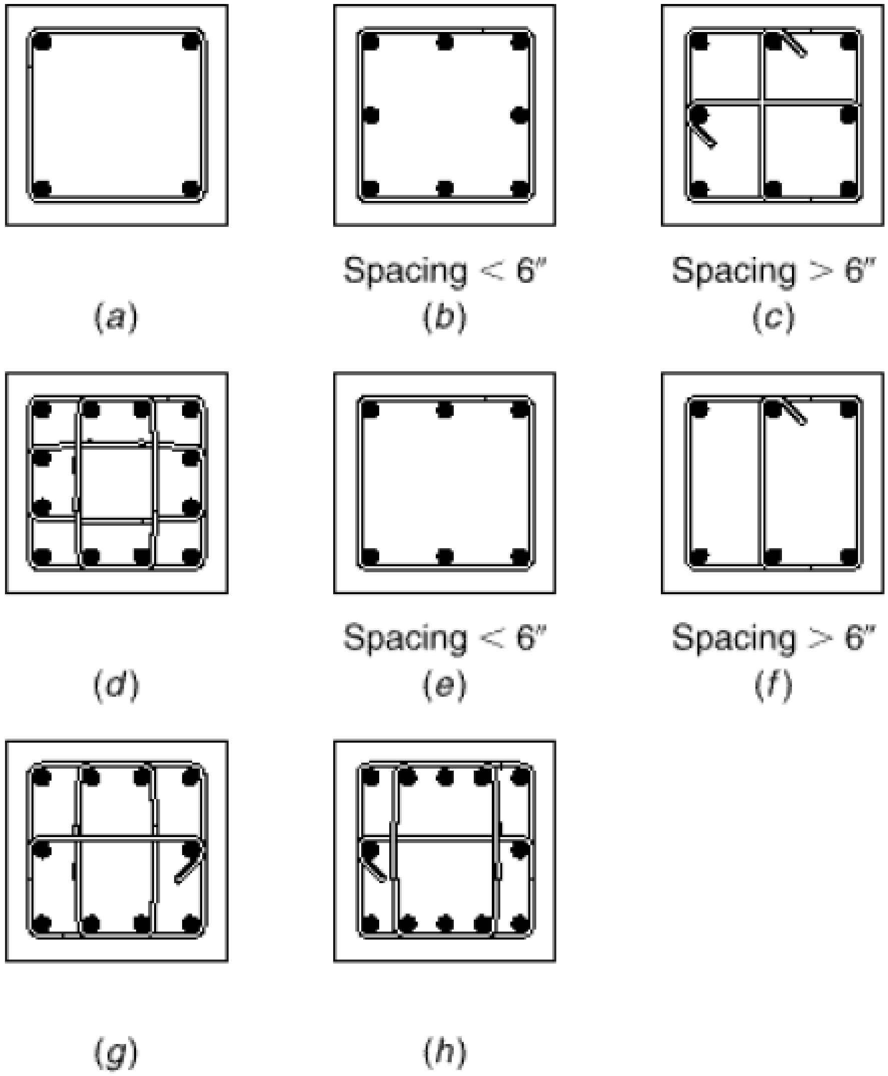


Figure 22: Tie arrangement of rectangular column ((Ref: ACI Code, Design of Concrete Structure, 13th edition, Chap-8, P-254)

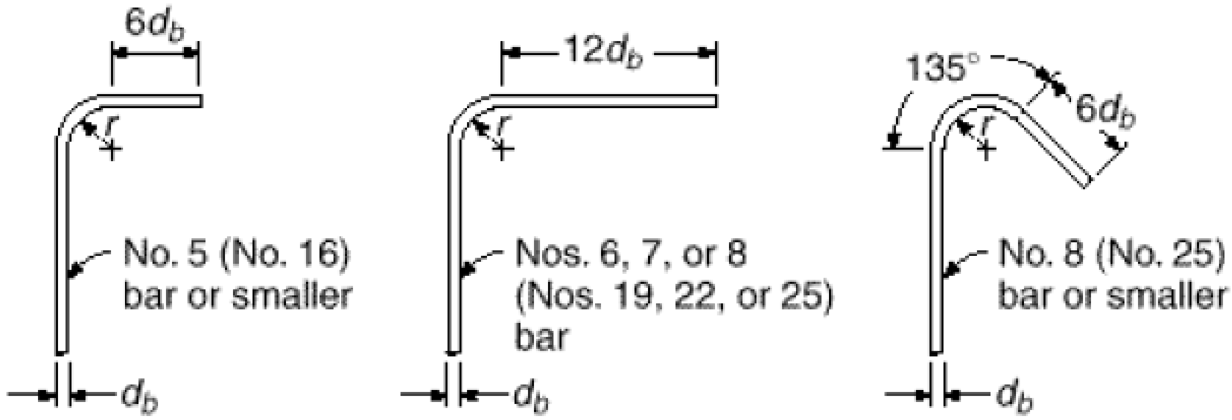


Figure 23: Standard bar hook for tie and stirrup. (Ref: ACI Code, Design of Concrete Structure, 13th edition, Chap-5, P-177)

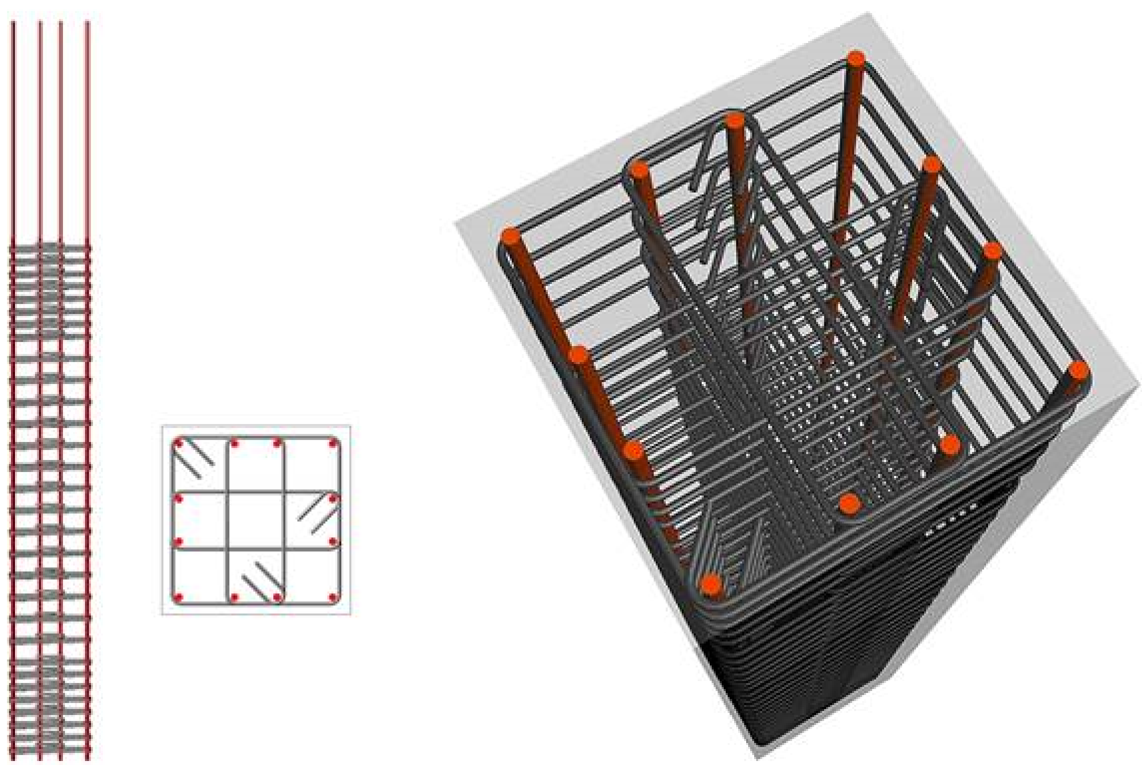


Figure 24: Typical column detail

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 (15th edition).
- Design of RCC Members by WSD and USD Methods, Public Works Department (PWD), 1997.
- Treasure of RCC Designs by Sushil Kumar (16th edition).
- 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***

Fig.2: Arch Bridge - ***Sydney Harbour Bridge, Australia***

Fig. 3:Truss Bridge – ***Ikitsuki Bridge, Nagasaki, Japan***

Fig.4: Cable-stayed Bridge – ***Rion Antirion Bridge, Greece***

Fig. 5: Suspension Bridge – ***Akashi Kaikyo Bridge, Japan***

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

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.

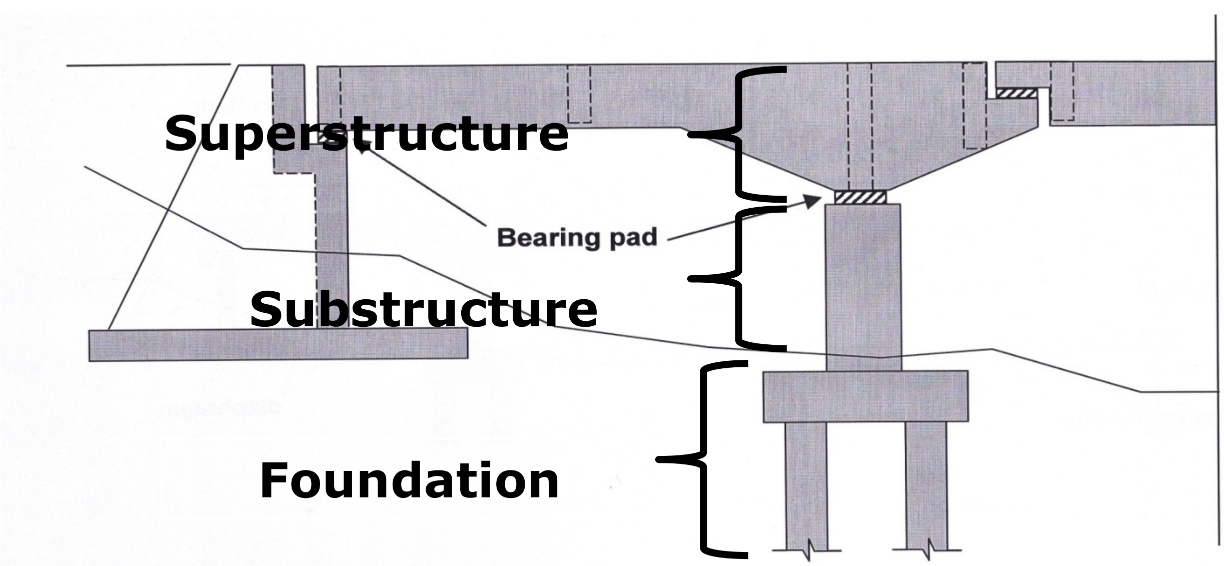


Fig. 7: Different parts of a Bridge

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*)

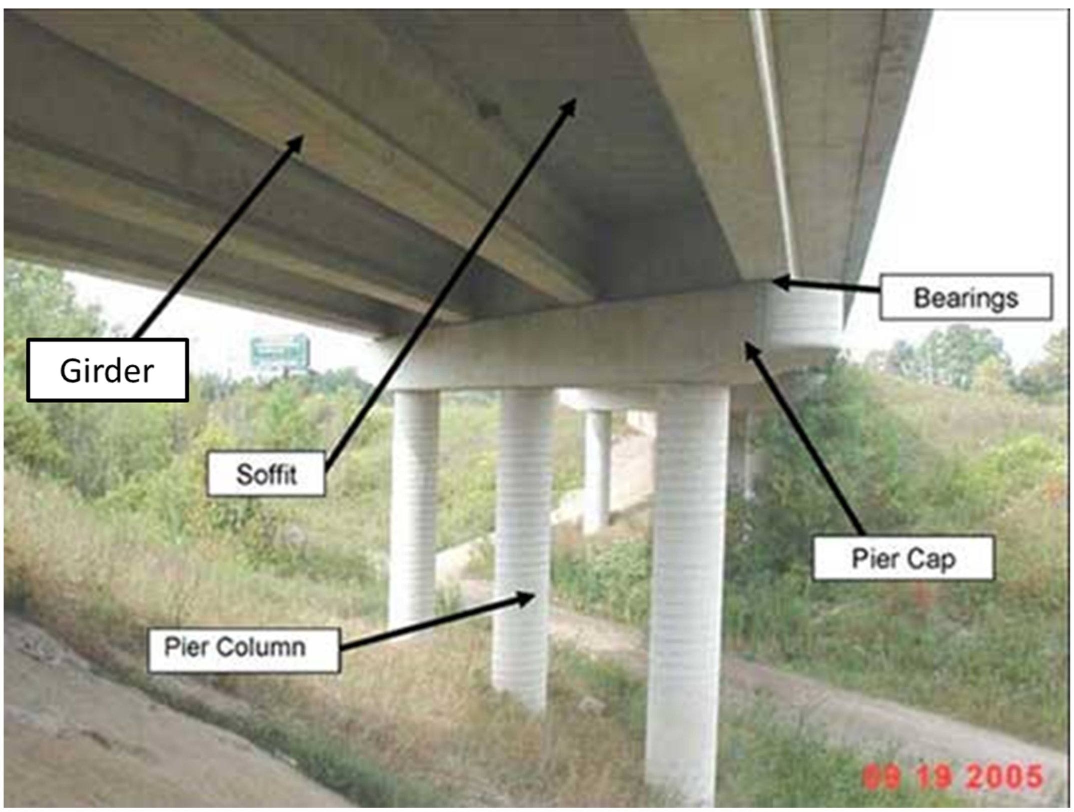


Fig. 8: Different components of a Bridge

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.

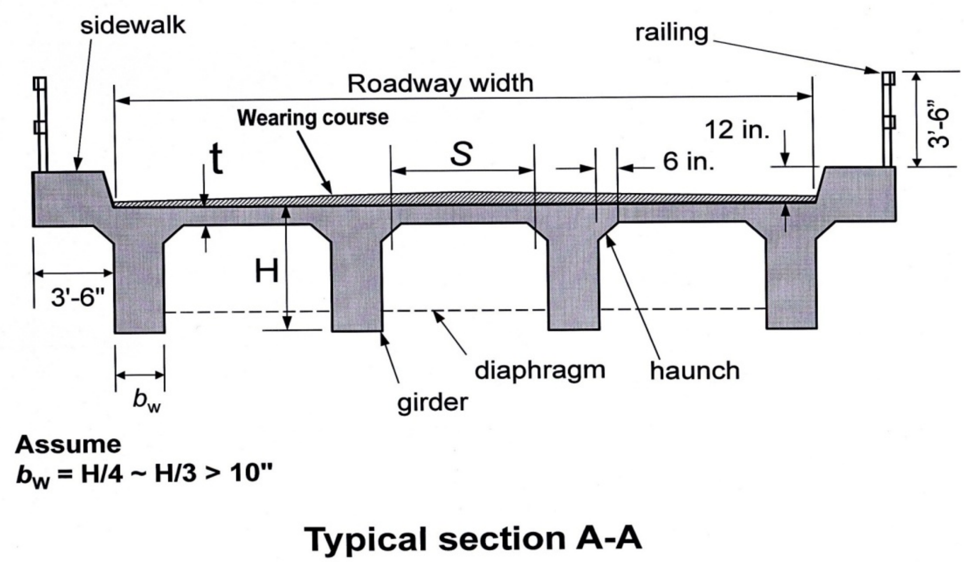


Fig. 10: Transverse section

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.

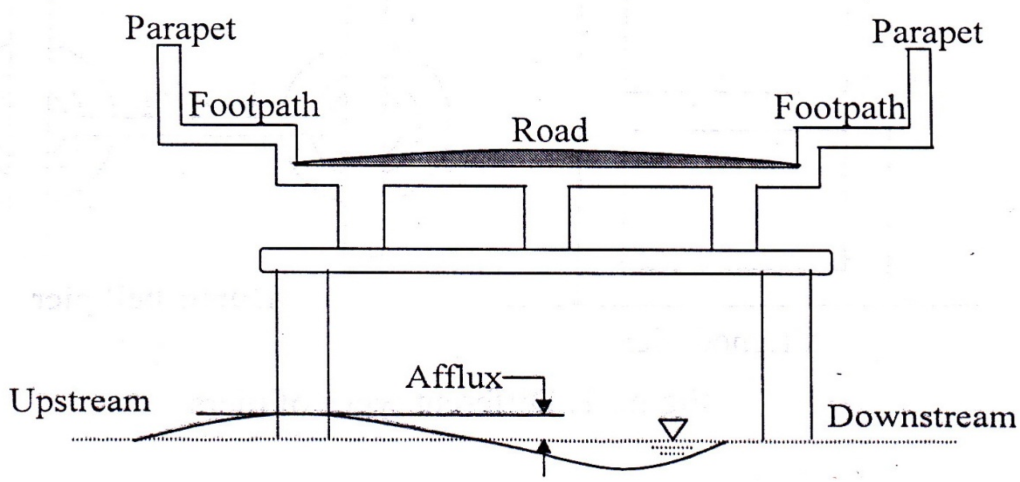


Fig. 11: Afflux

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 upto the bridge.

Approach Slab

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

One end rests on the backfill of the abutment andextends 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.

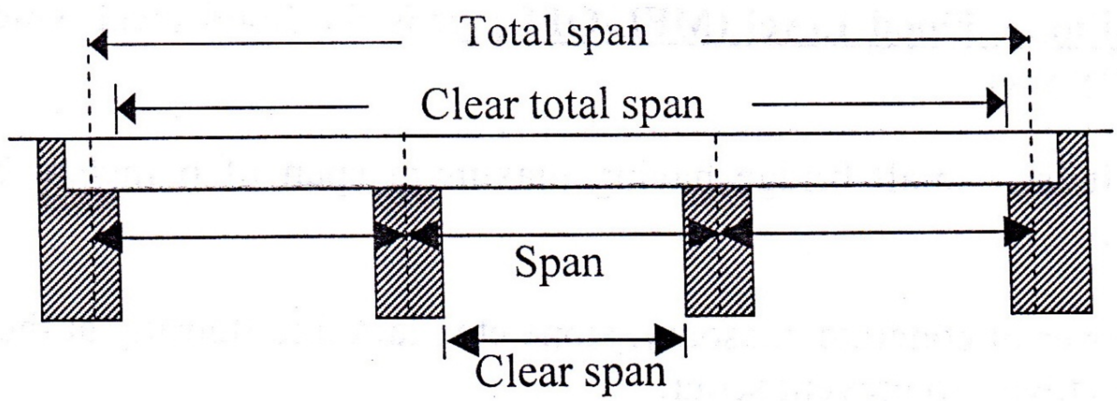


Fig. 12: Total span, total clear span, span and clear span

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

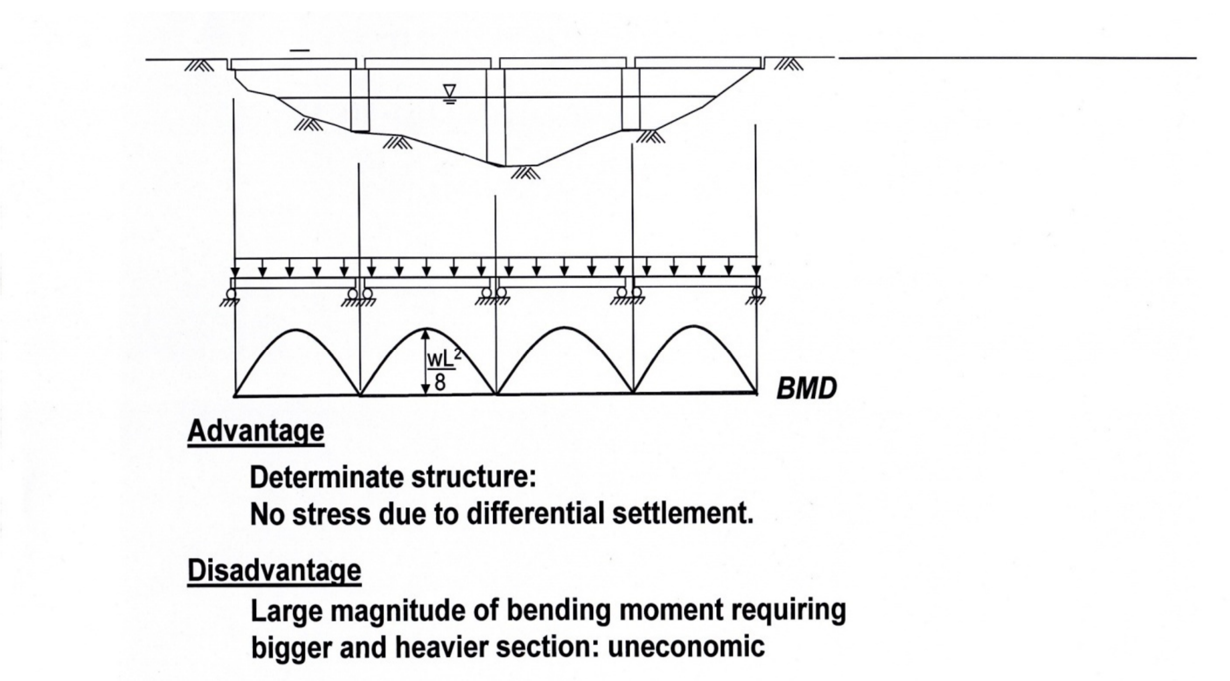
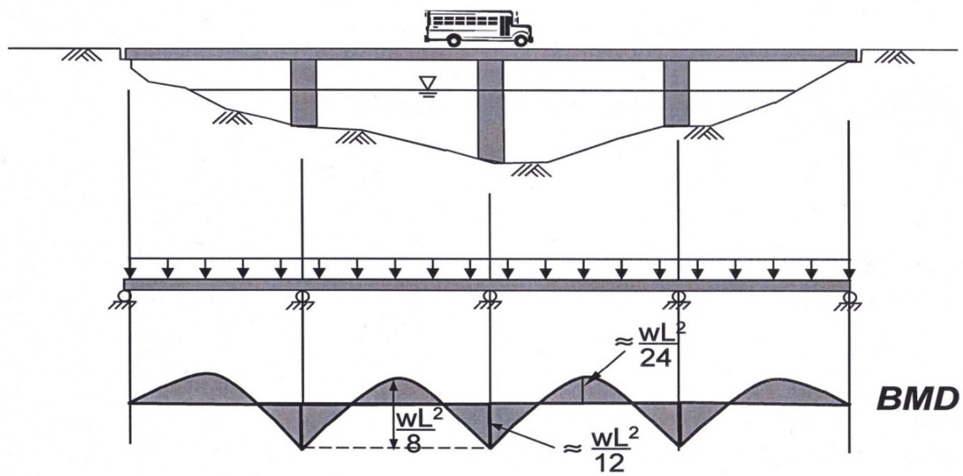


Fig. 13: A bridge having simply supported span

b) Continuous span bridge



Advantage

**Magnitude of maximum moment reduced:
Resulting in economic section**

Disadvantage

Large bending moment due to uneven/differential settlement

Fig. 14: A bridge having continuous span

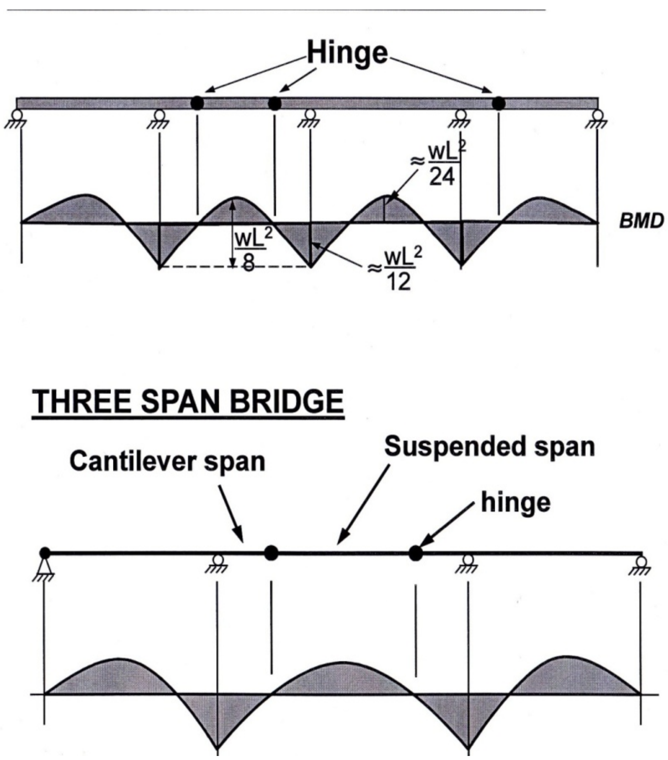
c) What is a Balanced Cantilever Bridge?

- 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



Hinges render the structure determinate:
Thus the problem of large stress due to settlement is eliminated.

Bending moment diagram of indeterminate structure is retained:
Thus the design section becomes economic

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**

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: Googlemap)

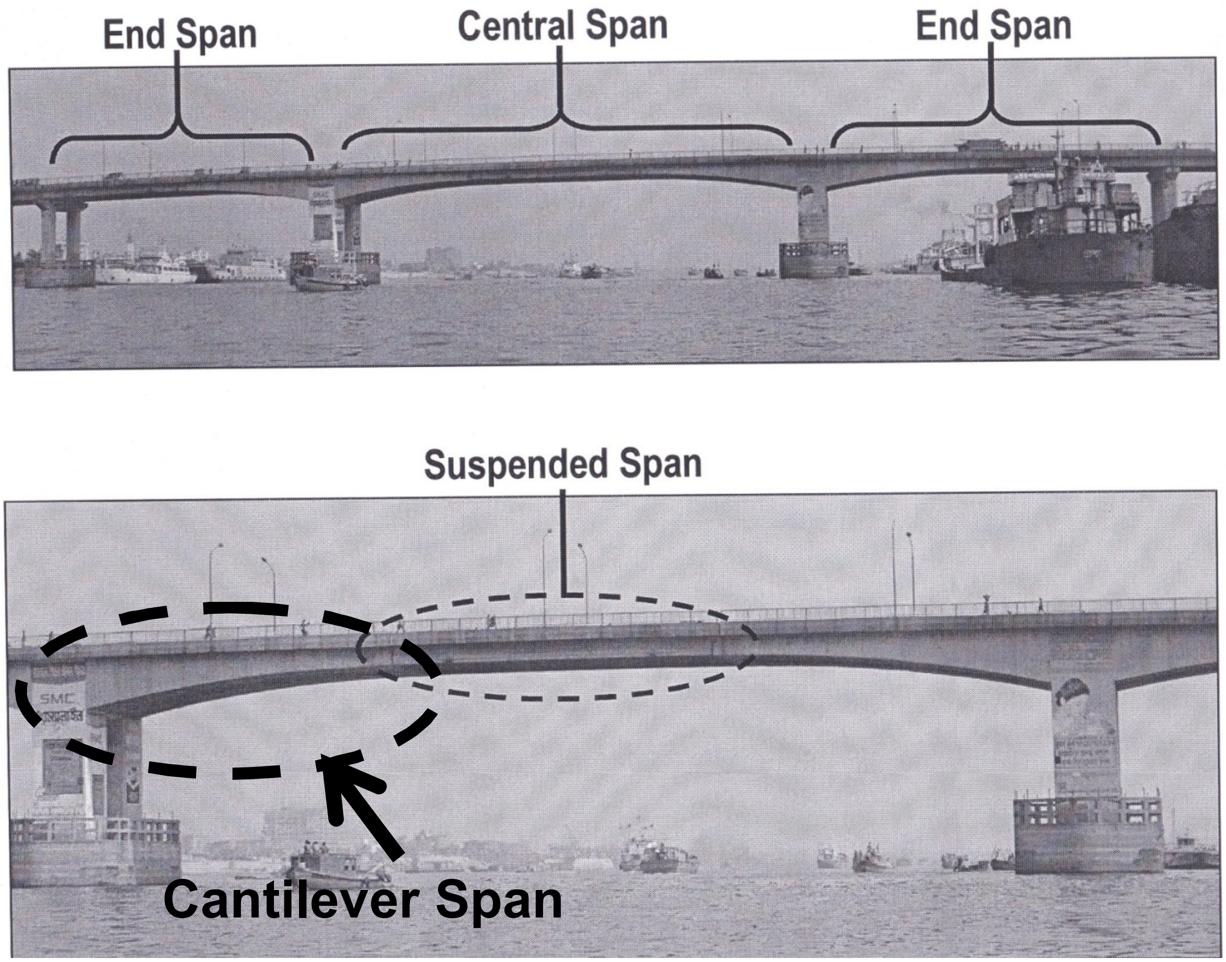


Fig.18: Spans of **Bangladesh China Friendship Bridge**

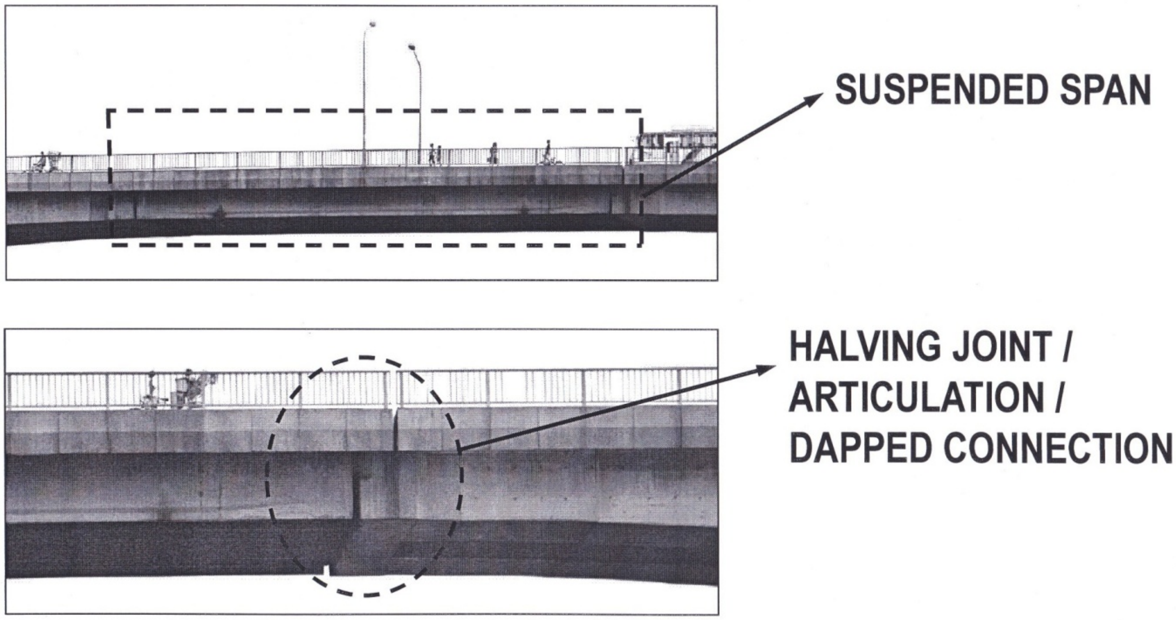


Fig.19: Articulation/ Halving joint

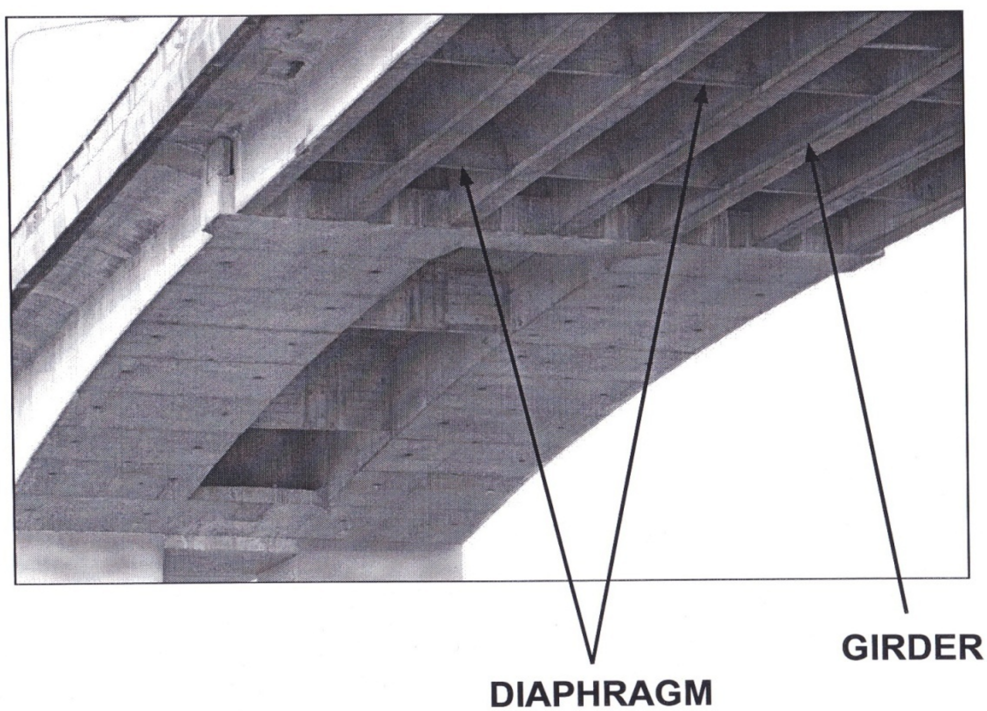


Fig.20: A back view showing diaphragm/cross girder and longitudinal girder

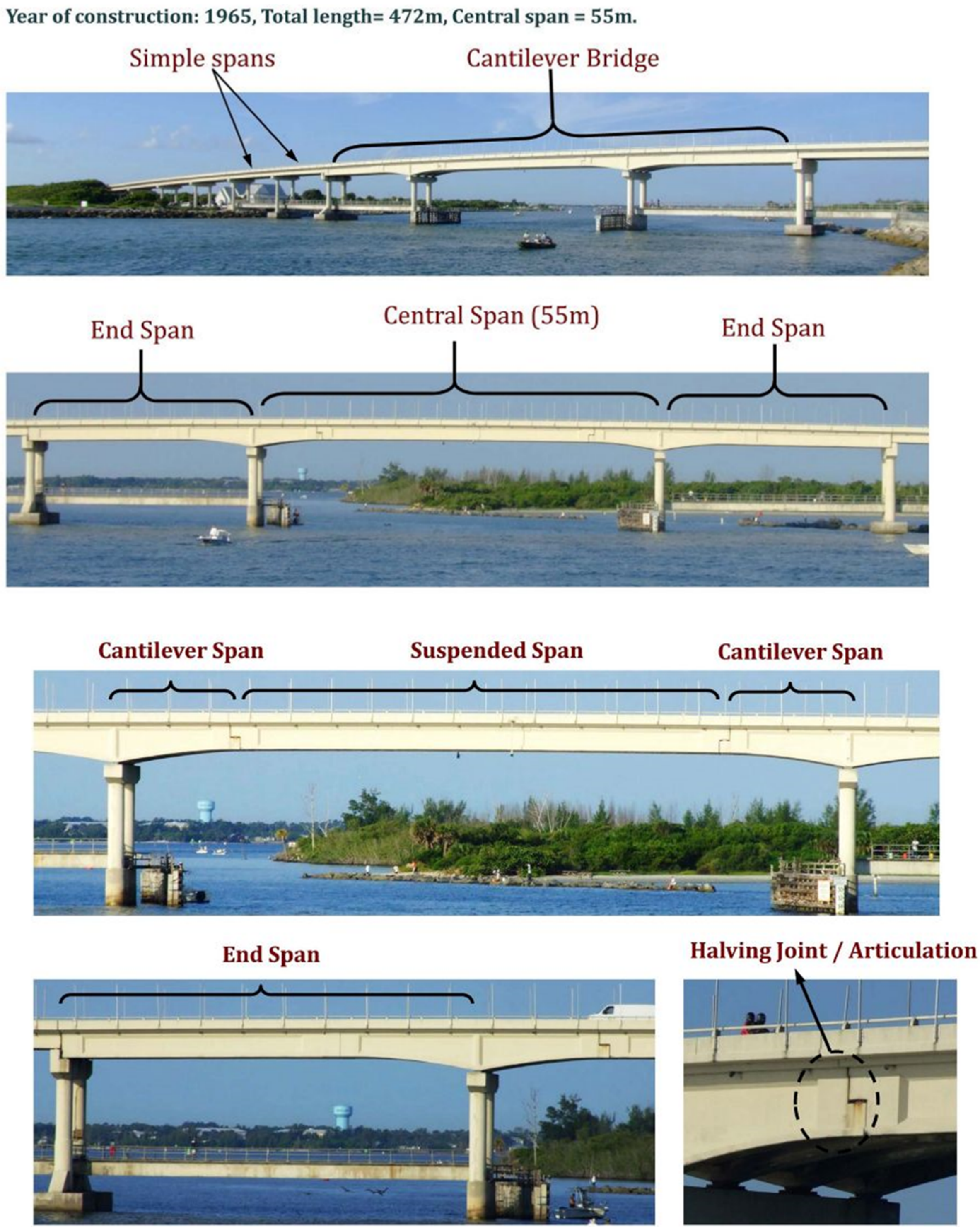


Fig. 21: Sebastian Intel Bridge, Florida, USA

Support Details

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

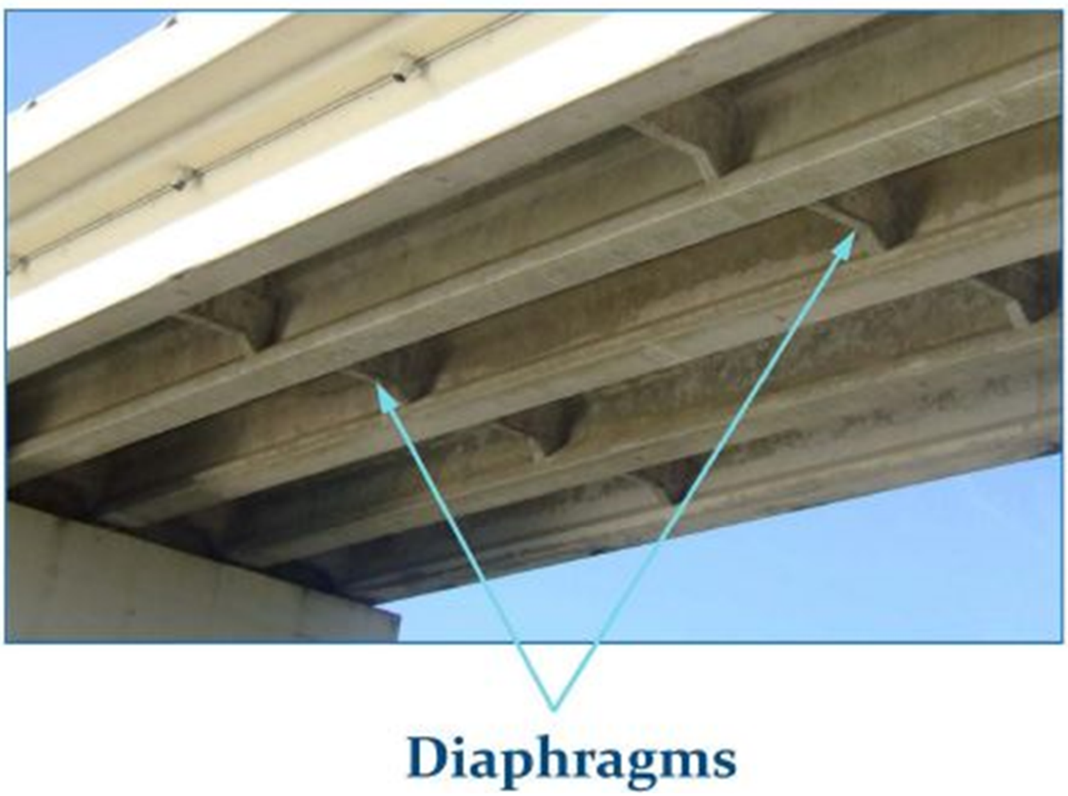


Fig. 23: Diaphram or cross girder of Sebastian Intel Bridge, Florida, USA

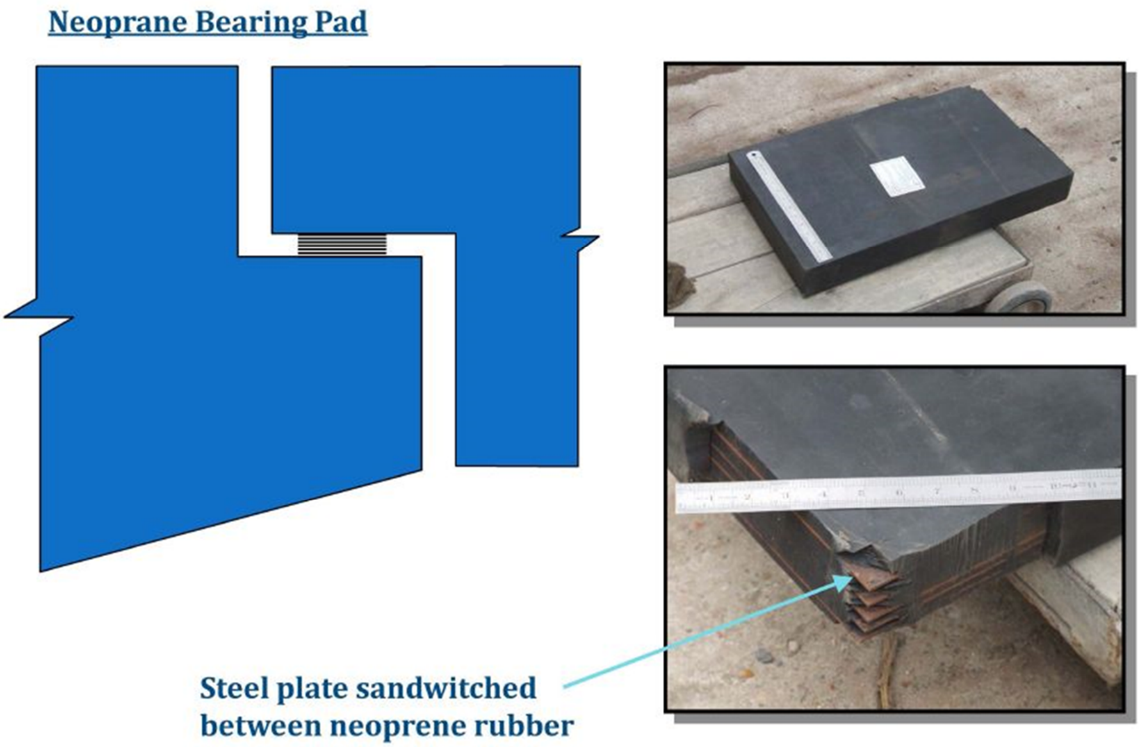


Fig. 24: Bearing Pad of Sebastian Intel Bridge, Florida, USA

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

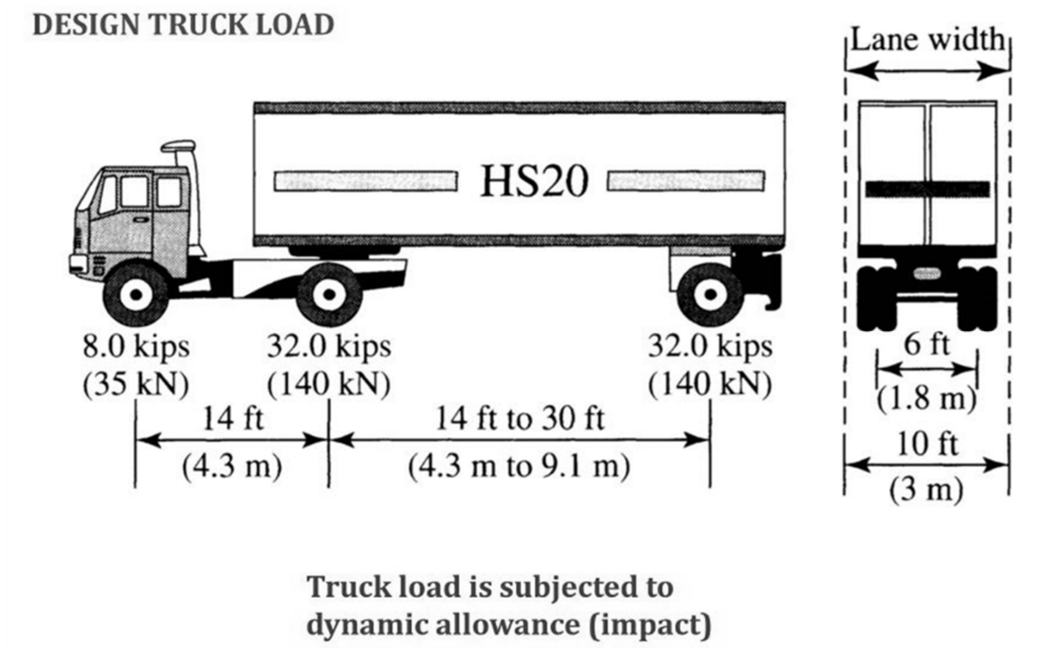
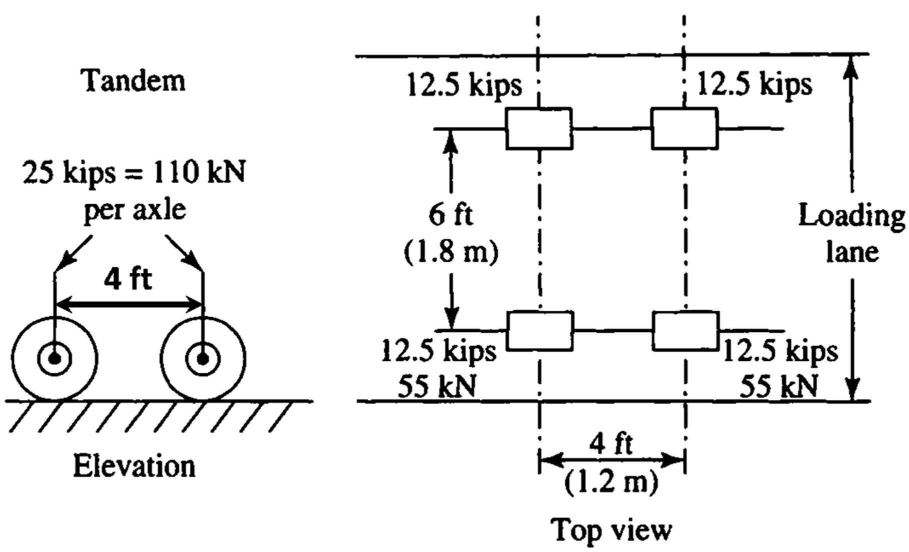


Fig.25: Design Truck load (HS20-516)

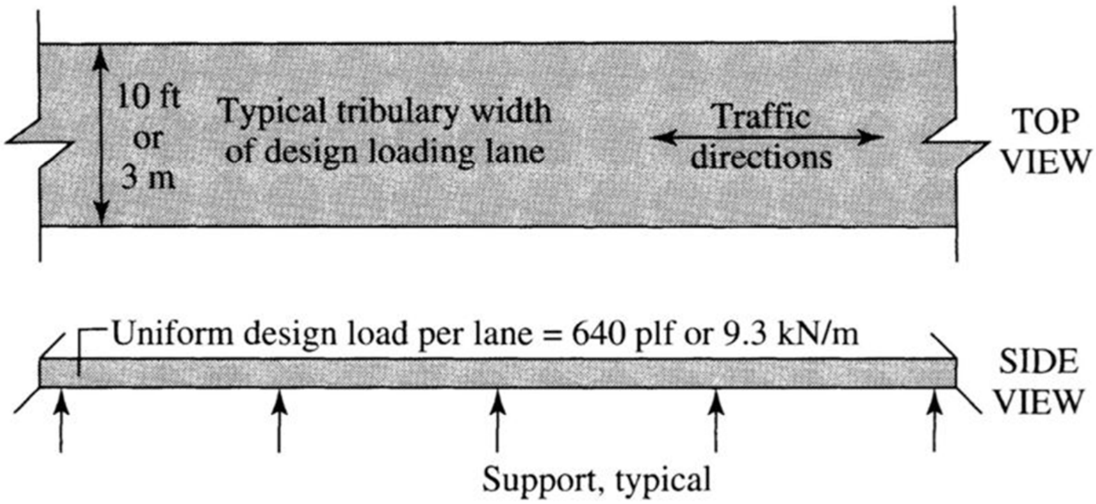
DESIGN TANDEM LOAD



TANDEM load is subjected to dynamic allowance (impact)

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**

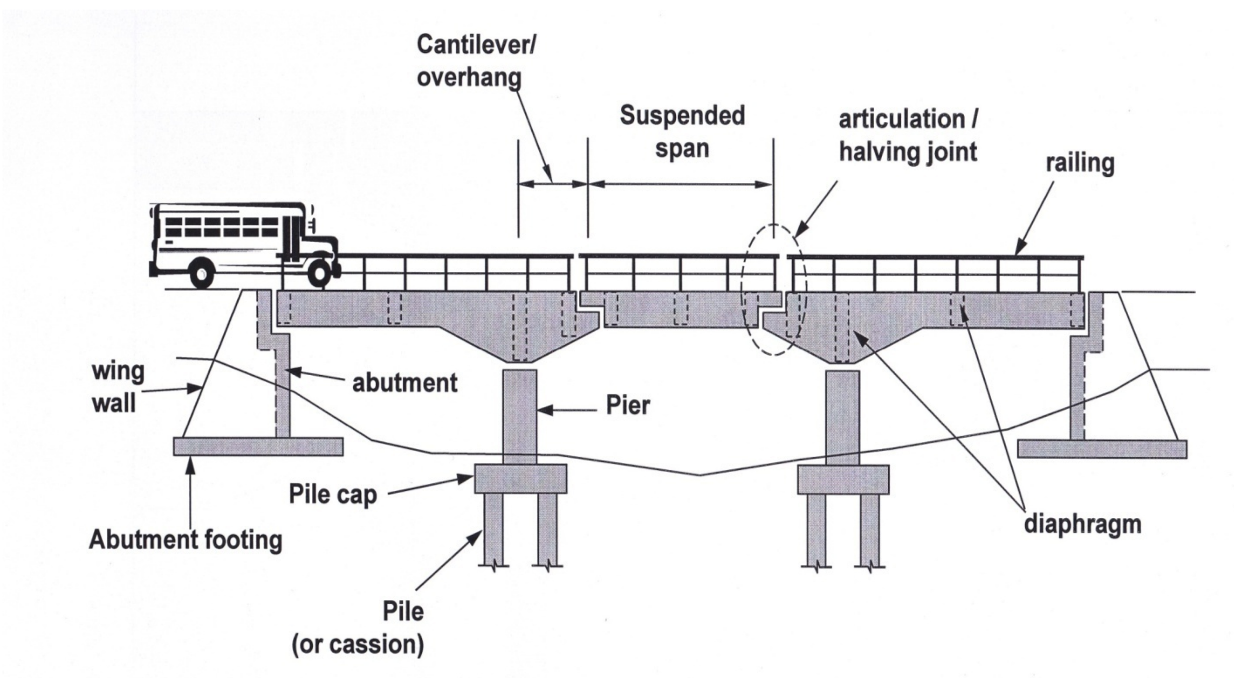
IM **= Impact effect of vehicle or tandem load**

PL **= Pedestrian load**

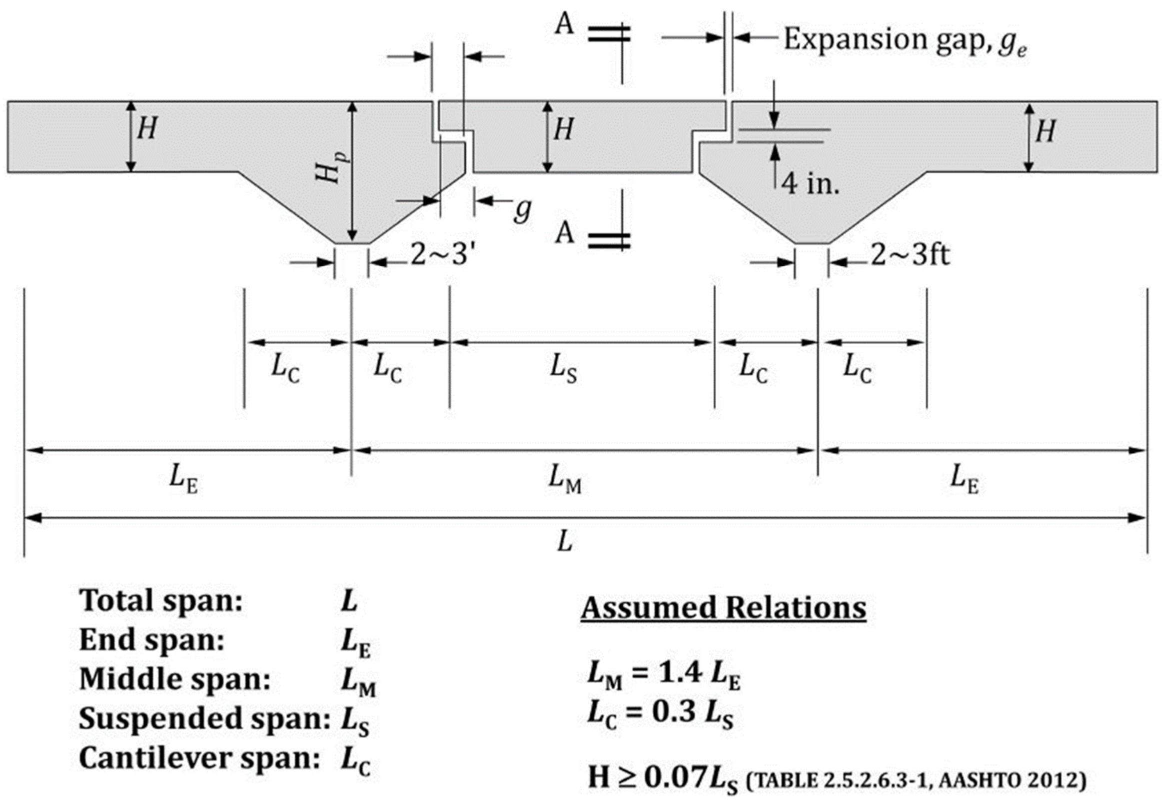
$$\gamma_p(DC) + \gamma_p(DW) + 1.75(LL)(1+IM/100)_{Truck/Tandem} + 1.75(LL)_{Lane} + 1.75(PL)$$

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

2.7 DESIGN OF DIFFERENT COMPONENTS



(a)



(b)

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

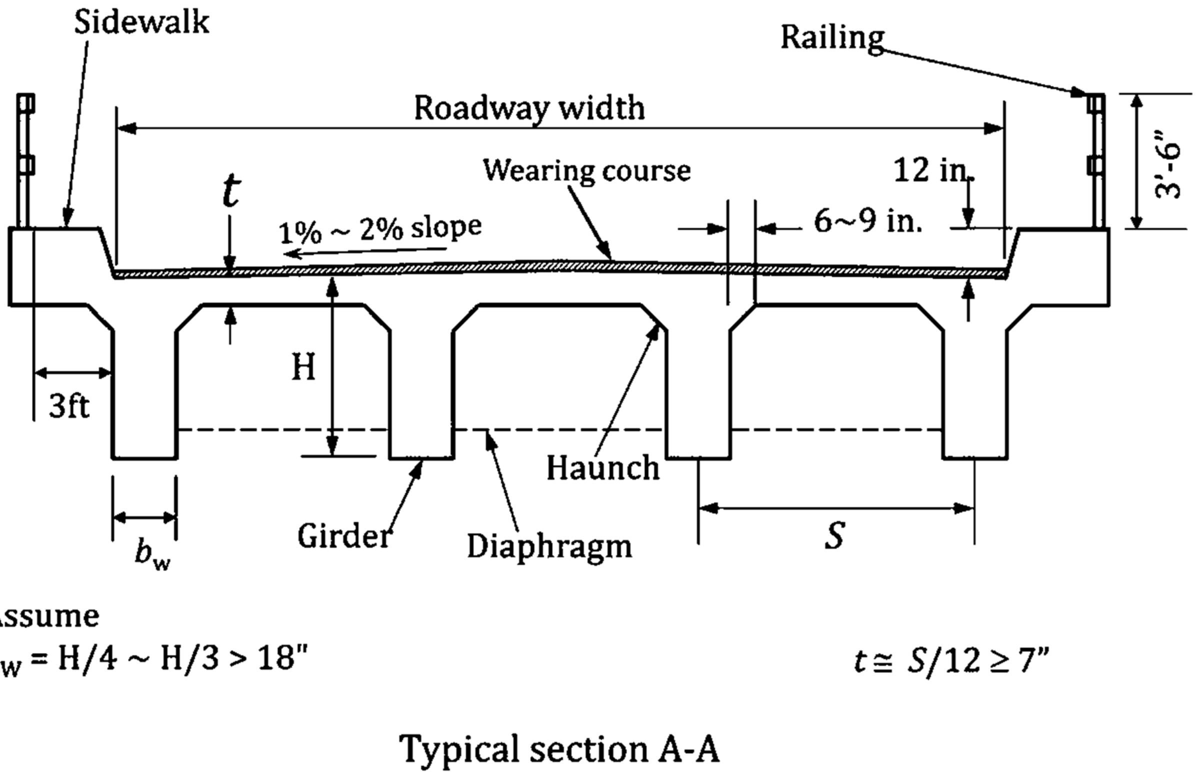


Fig. 29: Transverse section

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 Sl	Total Span, L ft	f_c' (ksi)	f_y (ksi)	Student Sl	Total Span, L ft	f_c' (ksi)	f_y (ksi)	Student Sl	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

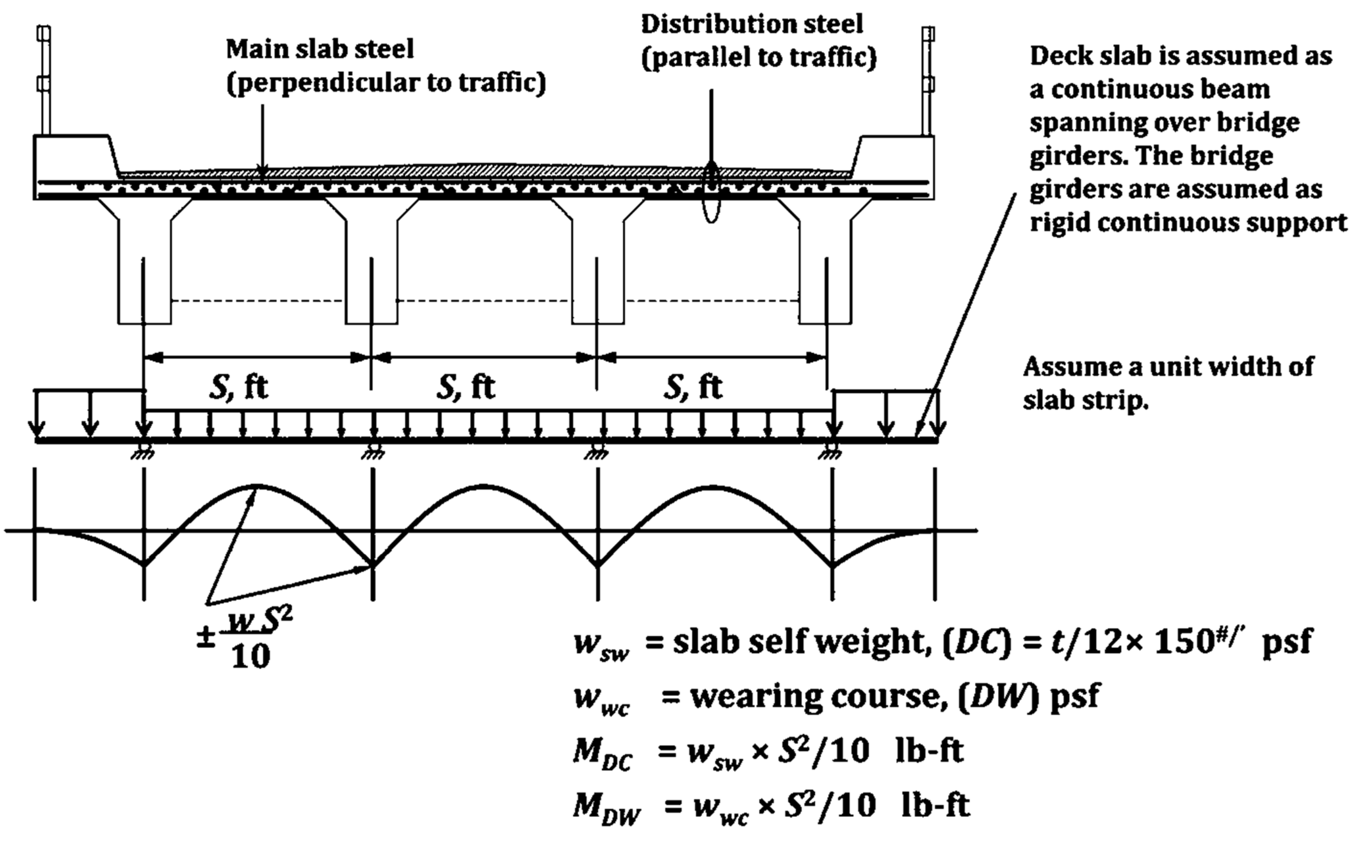
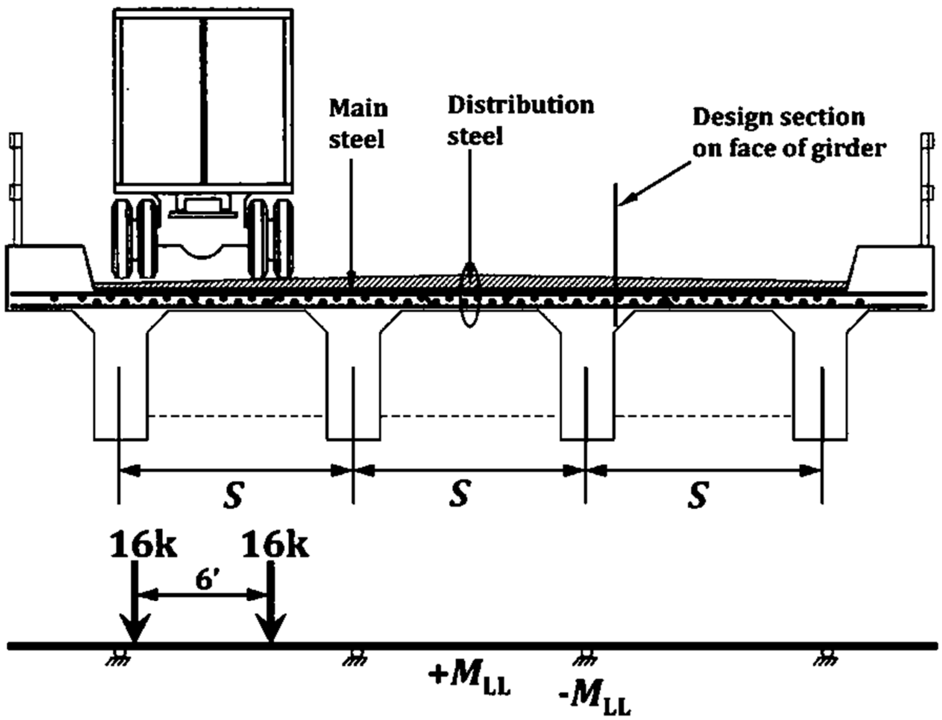


Fig. 30: Dead load on deck slab

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…

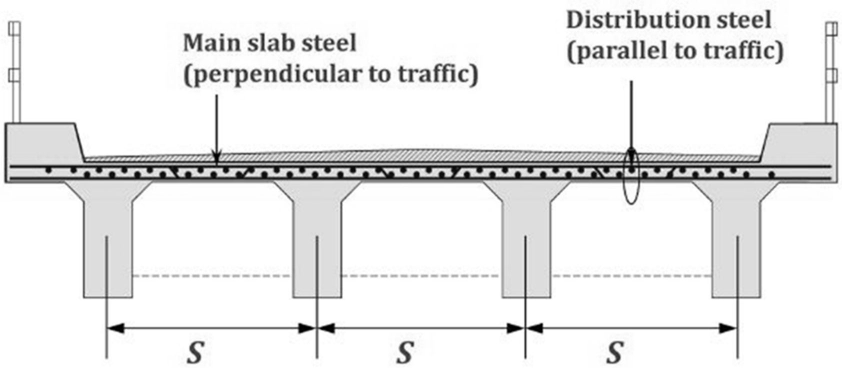
- **Multiple presence factors and the dynamic load allowance are included in the tabulated values.**
- **The moments are applicable for decks supported on at least three girders and having a width of not less than 14.0 ft between the centerlines of the exterior girders.**
- **For each combination of 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

S		Positive Moment	Negative Moment						
			Distance from CL of Girder to Design Section for Negative 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

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

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

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

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

Reinforcement Design of Deck

<u>Resistance factor φ</u>

Moment 0.90

<u>Deck Slab</u>

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

Assume $jd \approx 0.95d$

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

Determine, $a = \frac{A_s f_y}{0.85 f'_c b}$

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

Determine, β_1

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

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

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

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

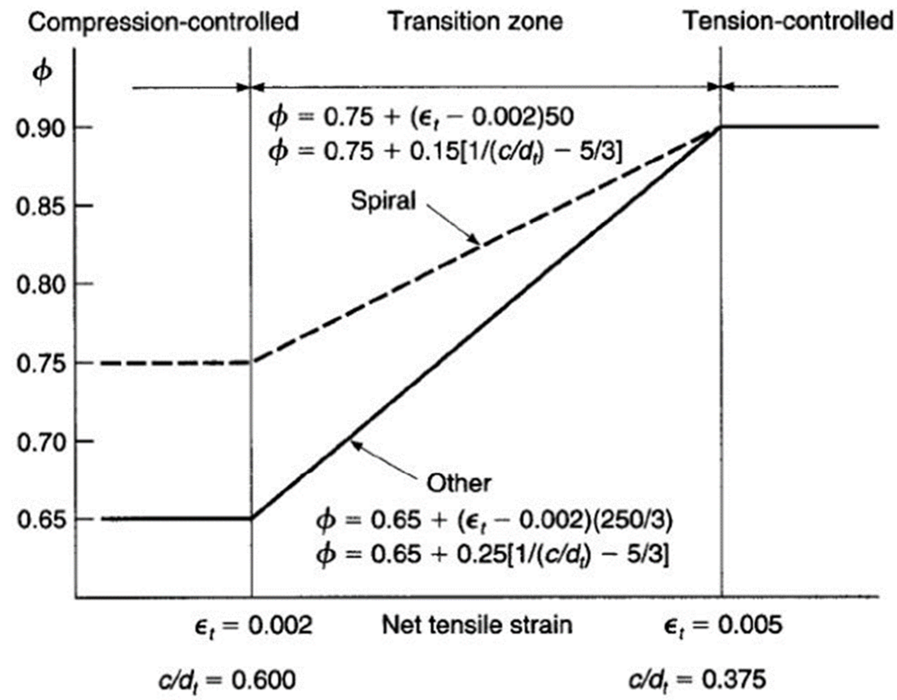
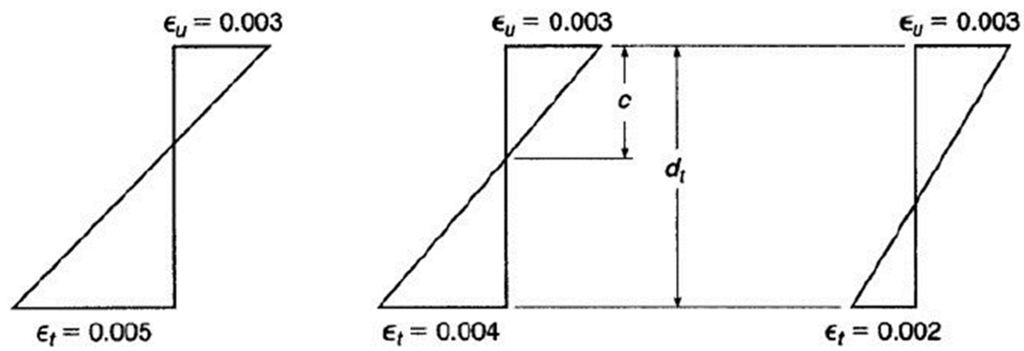


FIGURE 3.10
Net tensile strain and c/d_t ratios.



$$\frac{c}{d_t} = \frac{0.003}{0.003 + 0.005} = 0.375$$

(*a*)
Tension-controlled member

$$\frac{c}{d_t} = \frac{0.003}{0.003 + 0.004} = 0.429$$

(*b*)
Minimum net tensile strain for flexural member

$$\frac{c}{d_t} = \frac{0.003}{0.003 + 0.002} = 0.600$$

(*c*)
Compression-controlled member

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:

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

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

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.)

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

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

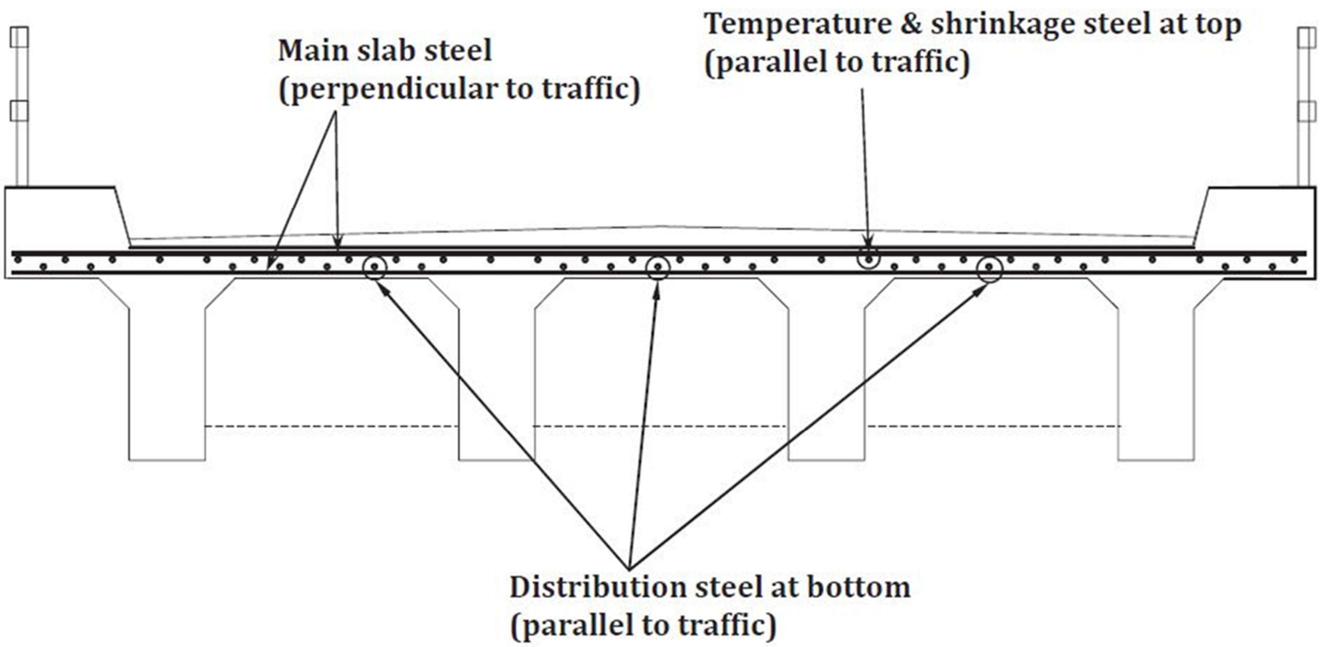


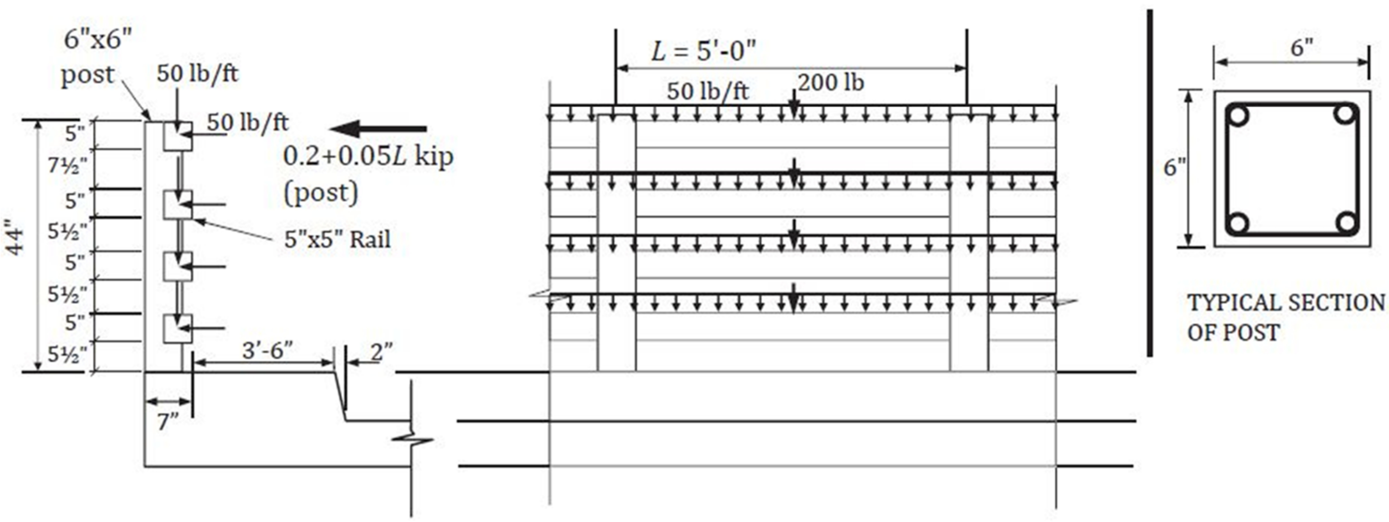
Fig.32: Reinforcement detailing of Slab

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

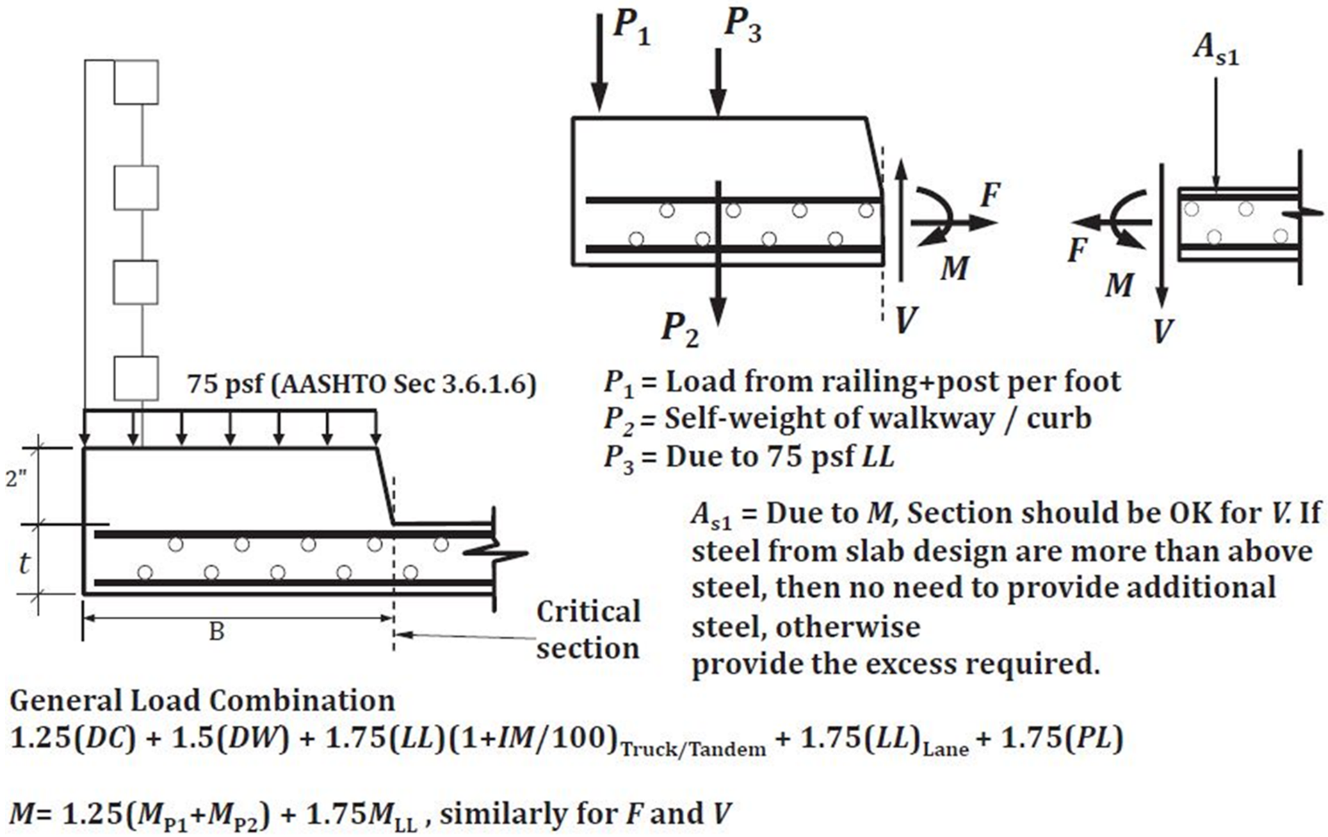
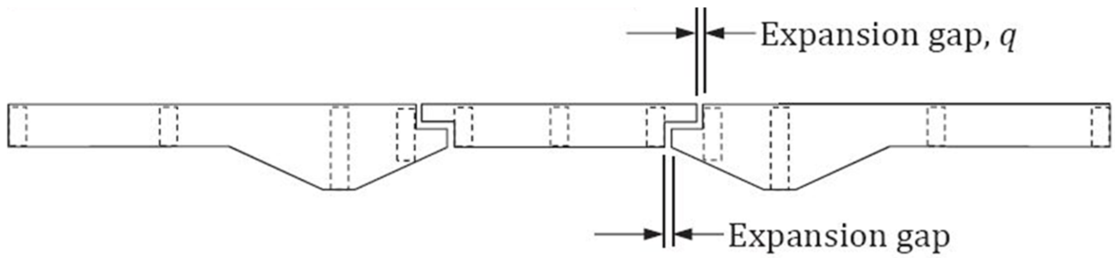


Fig. 34: Loads on Curb or sidewalk

- 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

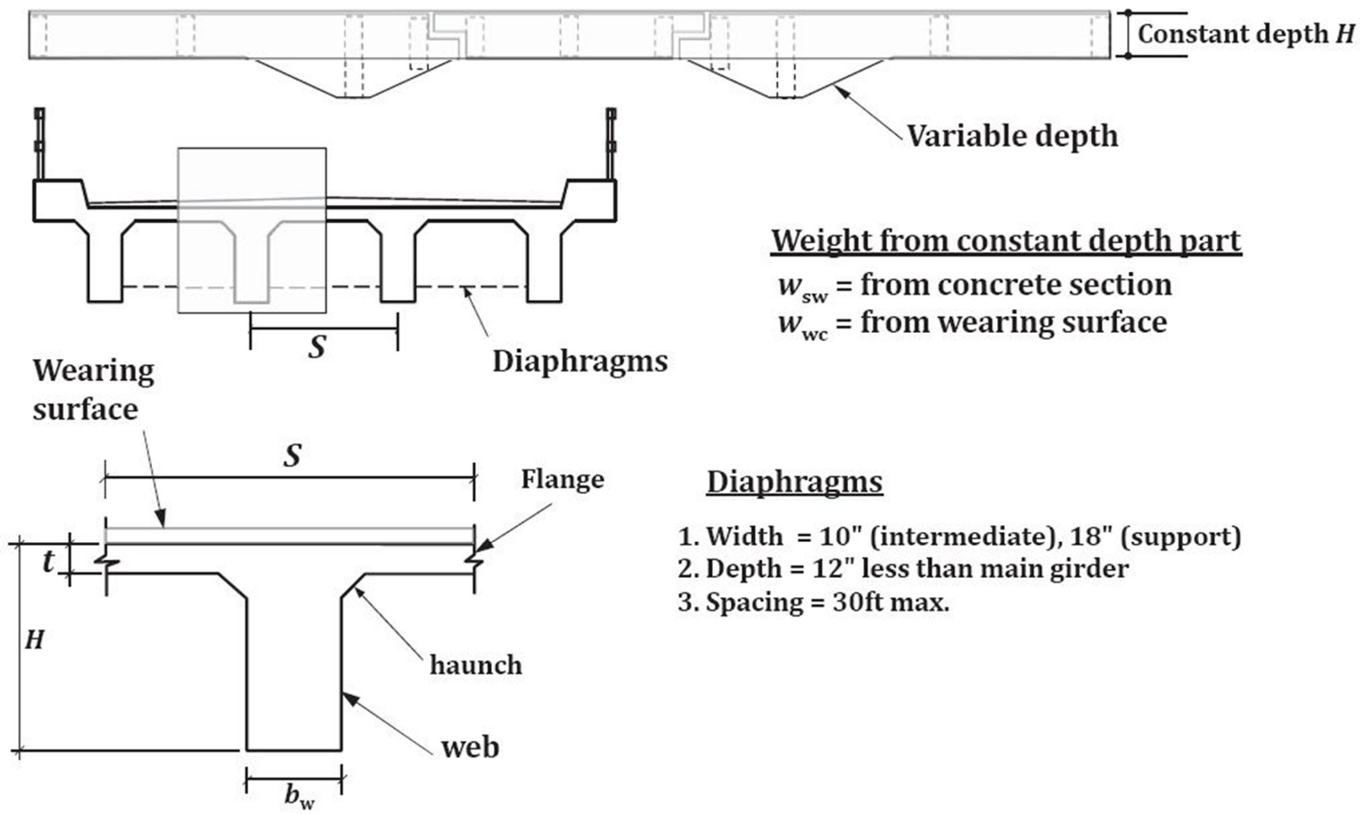


Fig. 35: Different dimensions of longitudinal girder

Dead load Analysis of Interior Girder

- Determine Dead load coming from self weight, wearing surface (DW).

- Determine self weight of cross girder/diaphram.

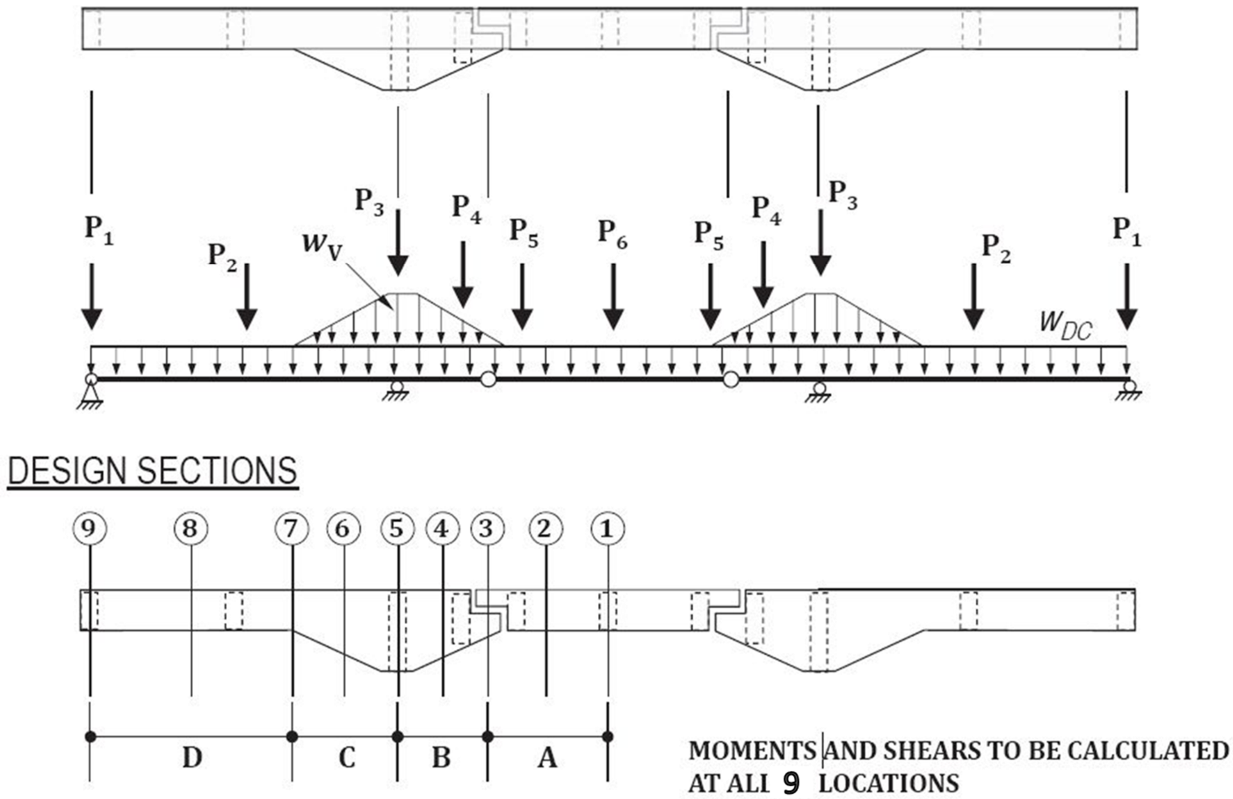


Fig. 36: Design sections of Interior Girder

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			

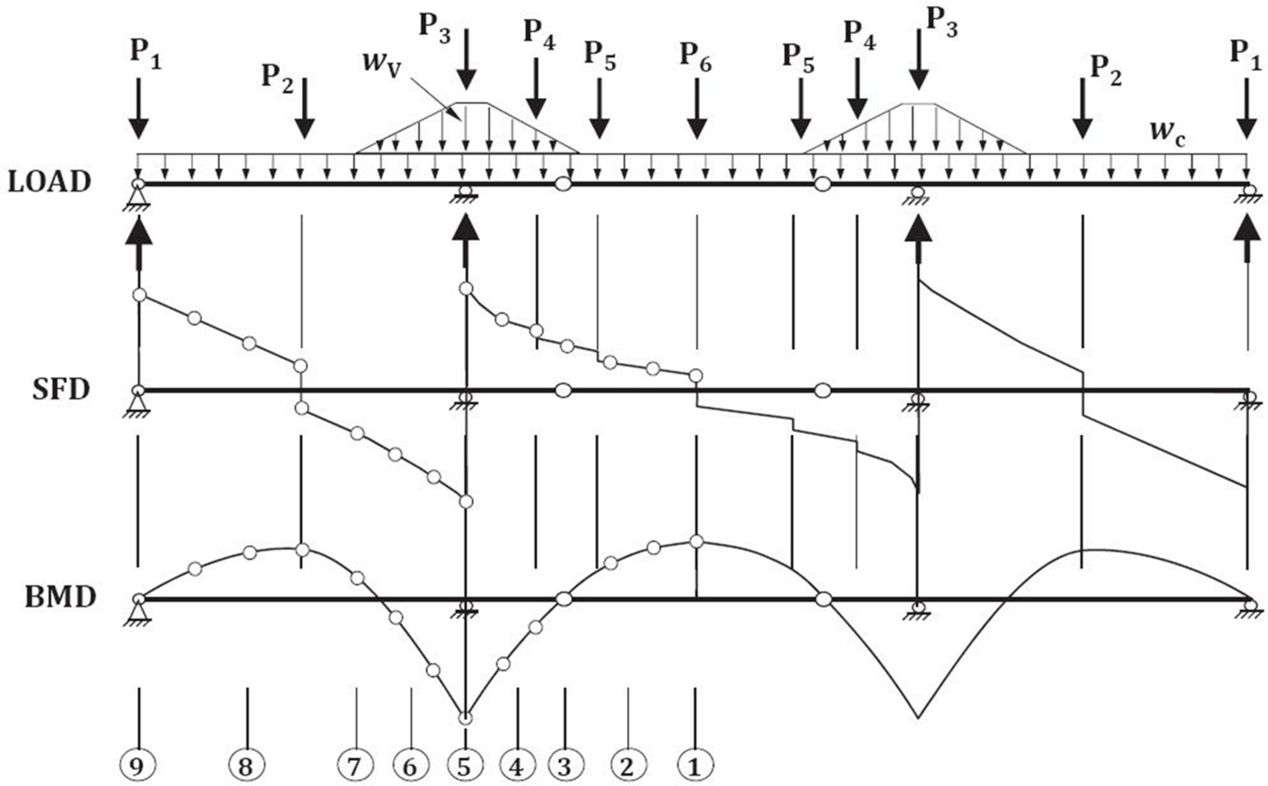


Fig. 37: SFD and BMD of interior girder due to DC dead load

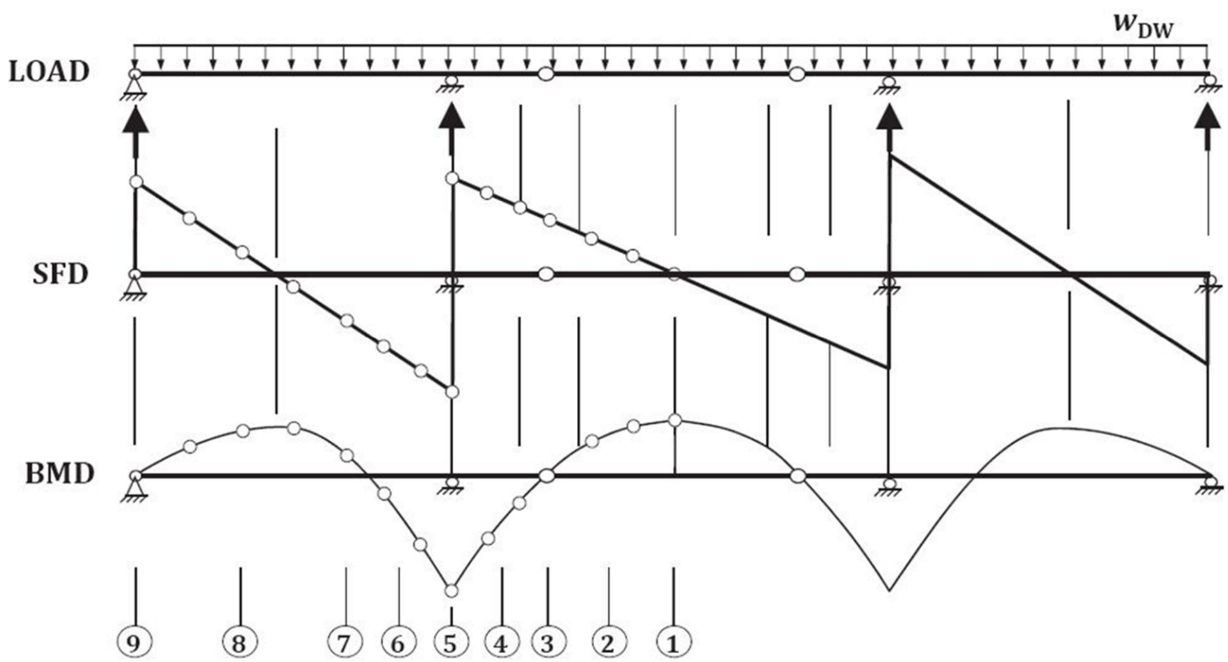


Fig. 38: SFD and BMD of interior girder due to DW dead load (wearing course)

Live load analysis of Interior Girder

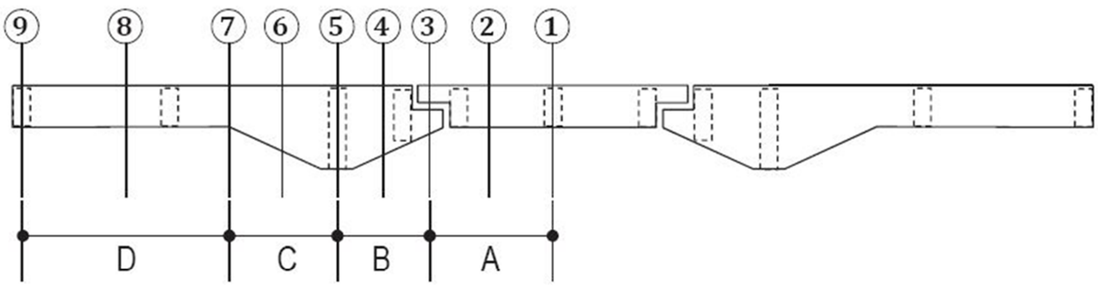
Influence Line (IL)

- 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.....

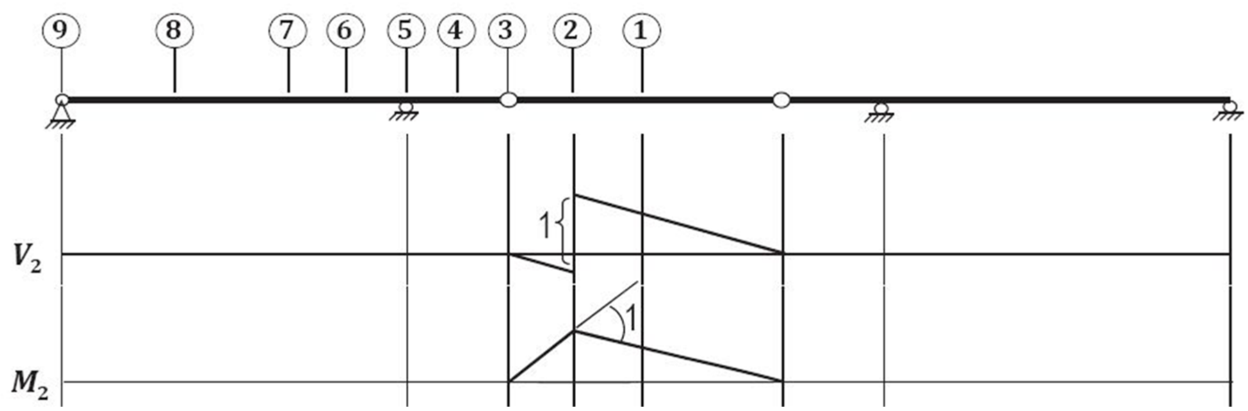


Fig. 39: IL diagram for shear and moment at section 2

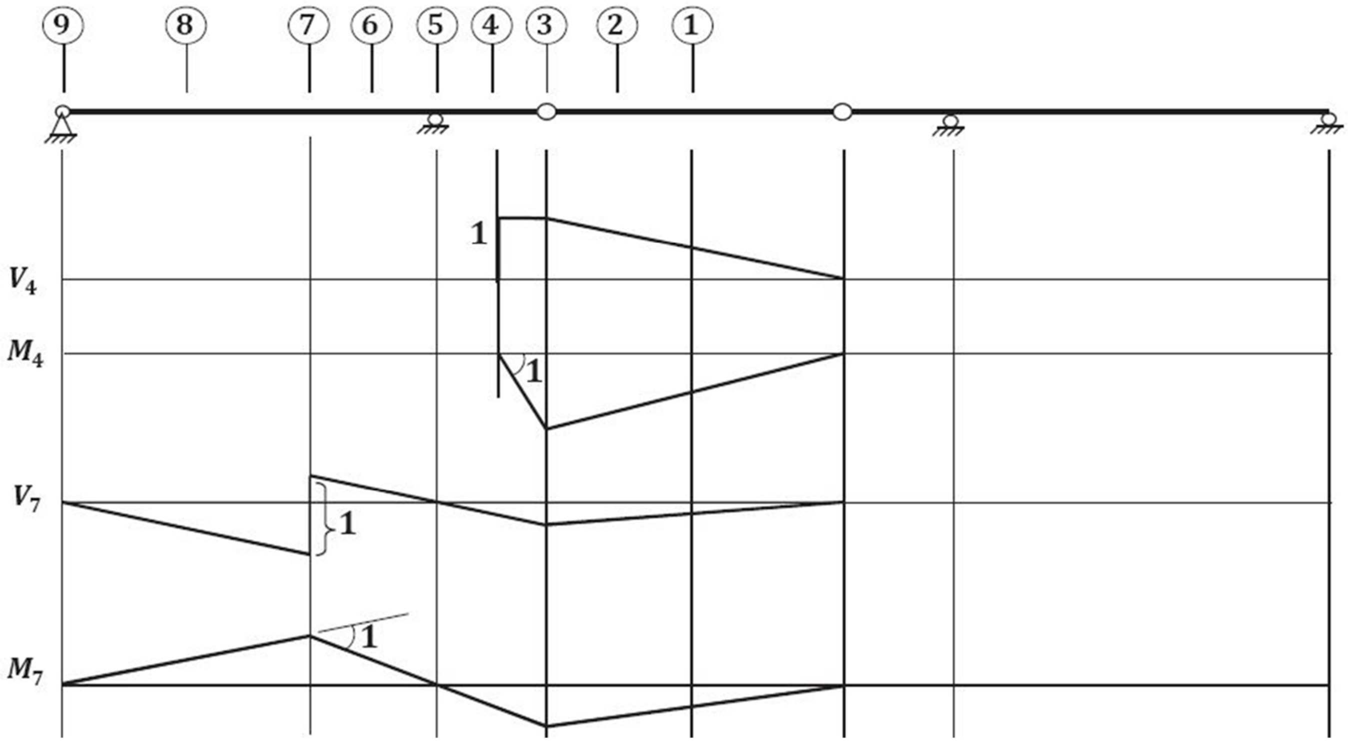
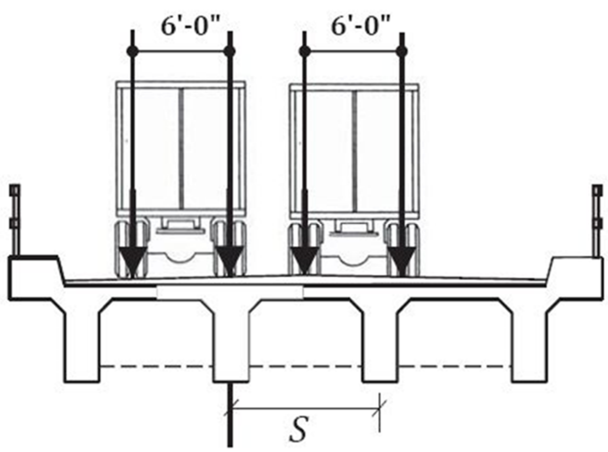


Fig. 40: IL diagram for shear and moment at section 4 and 7

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

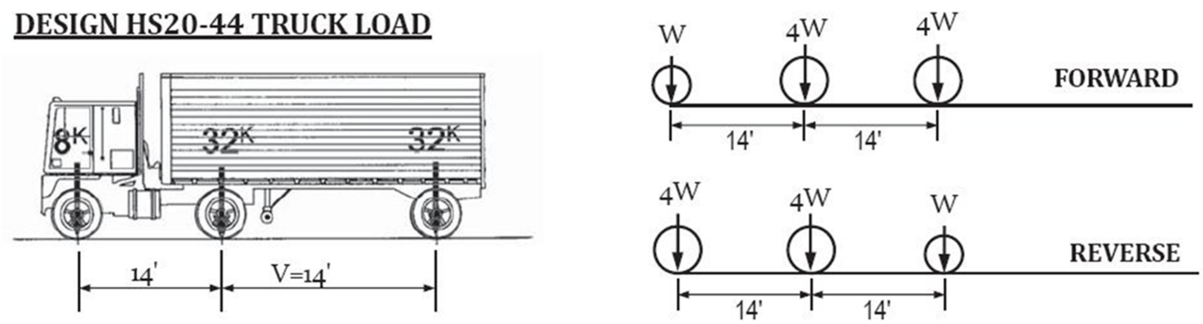
Force Effect	L (ft)
Positive Moment	The length of the span for which moment is being calculated
Negative Moment—Near interior supports of continuous spans from point of contraflexure to point of contraflexure under a uniform load on all spans	The average length of the two adjacent spans
Negative Moment—Other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

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

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

DESIGN HS20-44 TRUCK LOAD



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

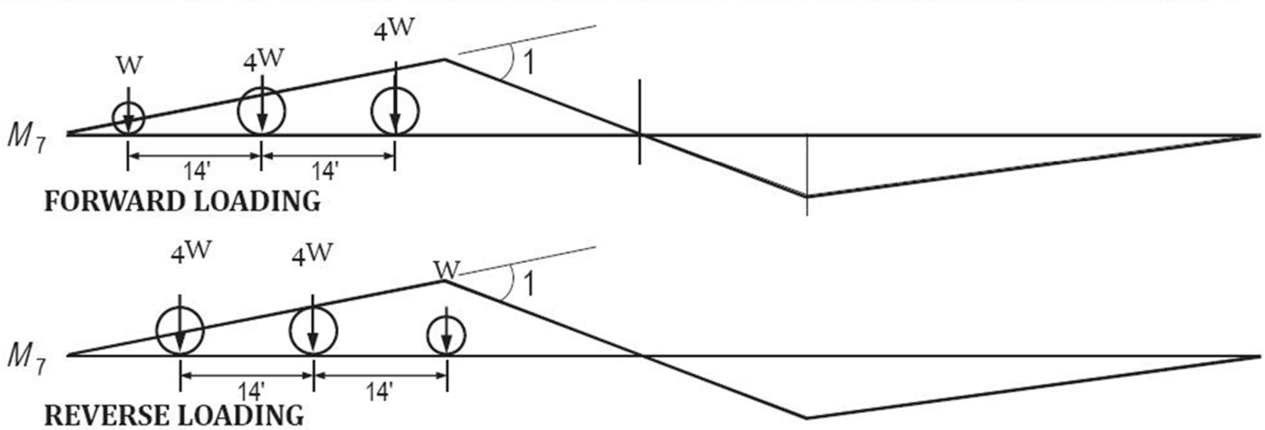


Fig. 41: Maximum positive moment at section 7 for forward and backward truck wheel load

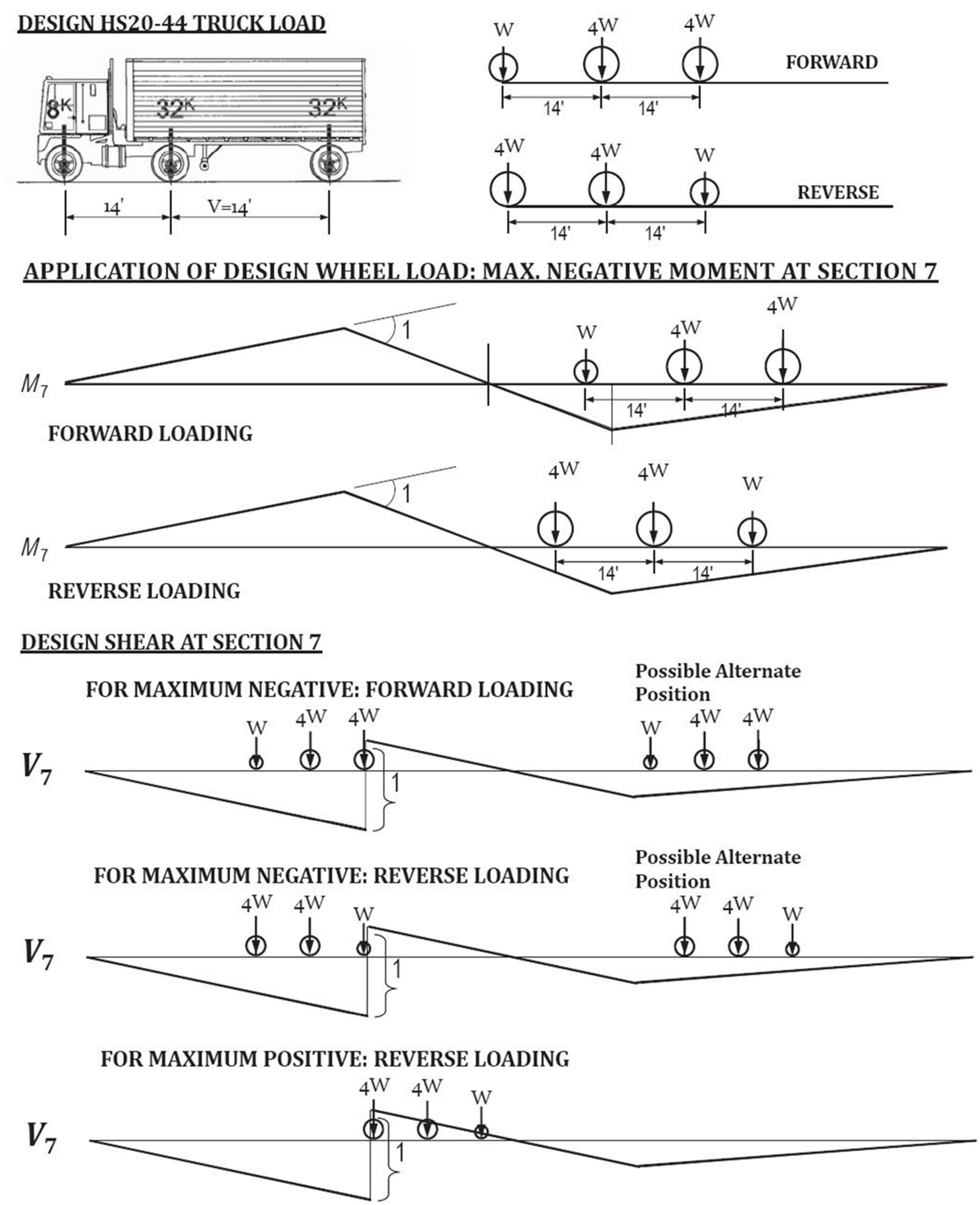
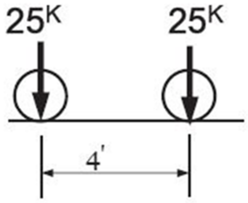
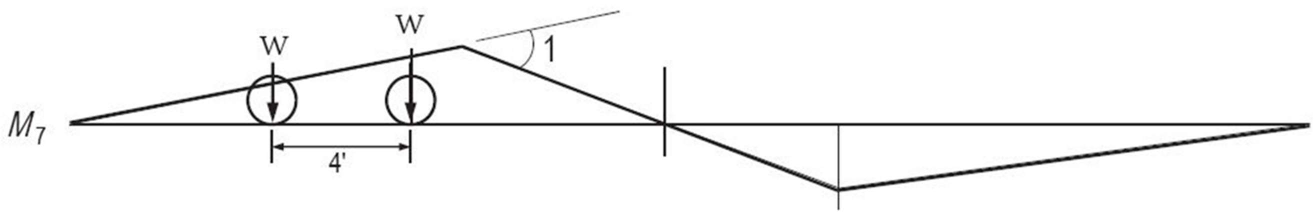


Fig. 42: Maximum +- moment, shear at section 7 for forward and backward truck wheel load

DESIGN TANDEM LOAD



APPLICATION OF DESIGN TANDEM LOAD



FOR MAXIMUM NEGATIVE

FOR MAXIMUM POSITIVE

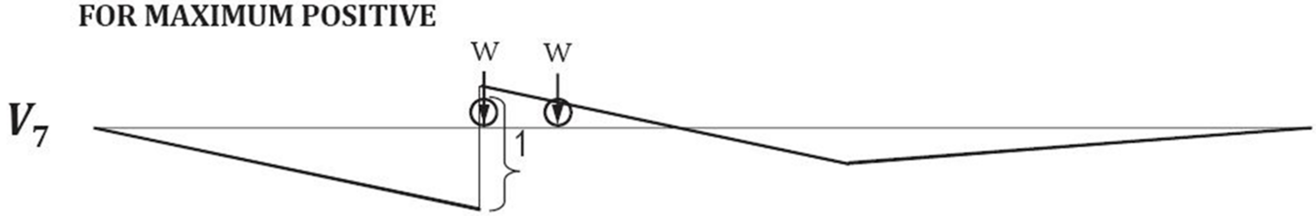


Fig. 43: Maximum positive & negative moment, shear at section 7 for tandem load

EQUIVALENT LANE LOAD

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

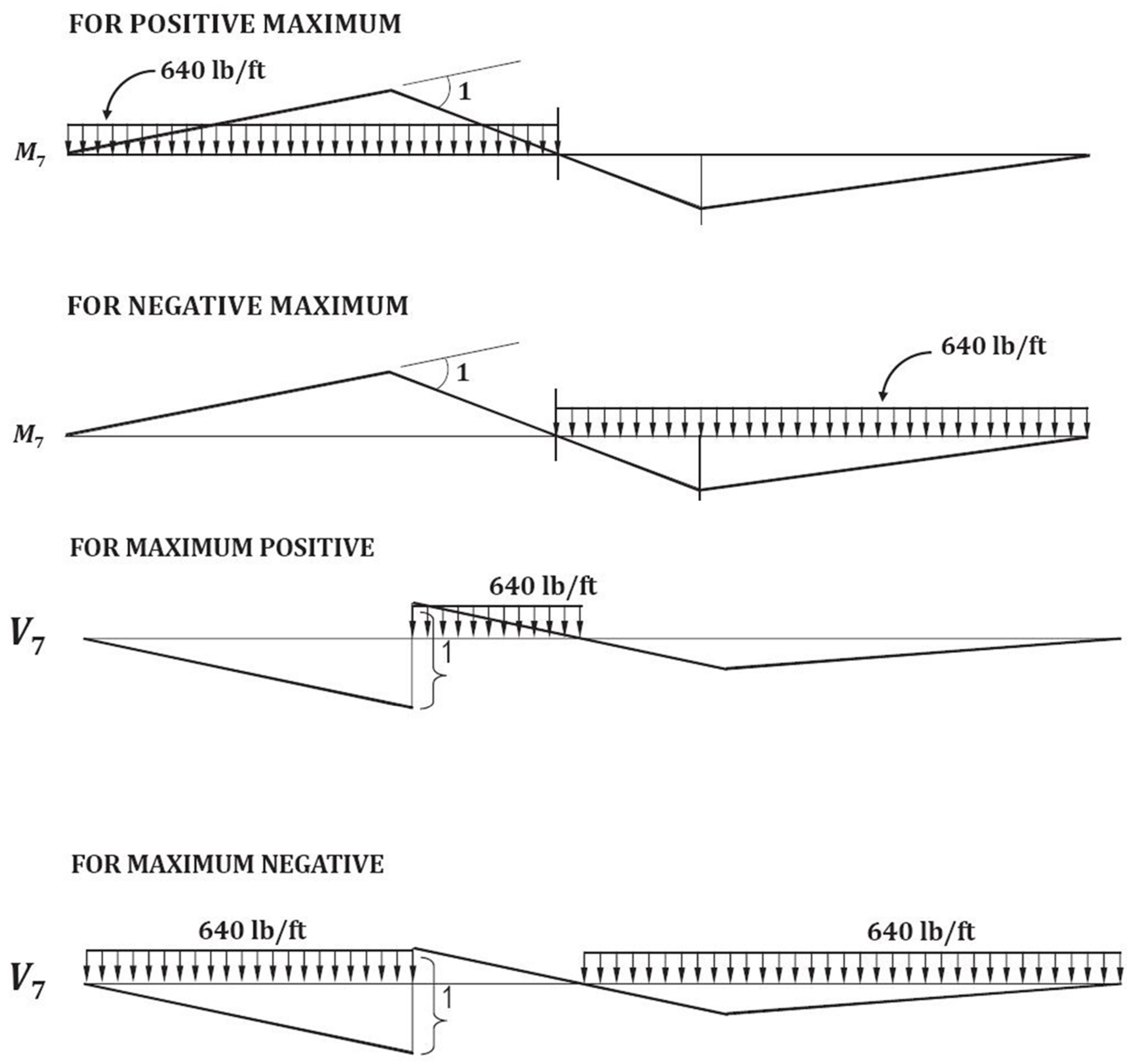


Fig. 44: Maximum positive, negative moment, shear at section 7 for equivalent lane load

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

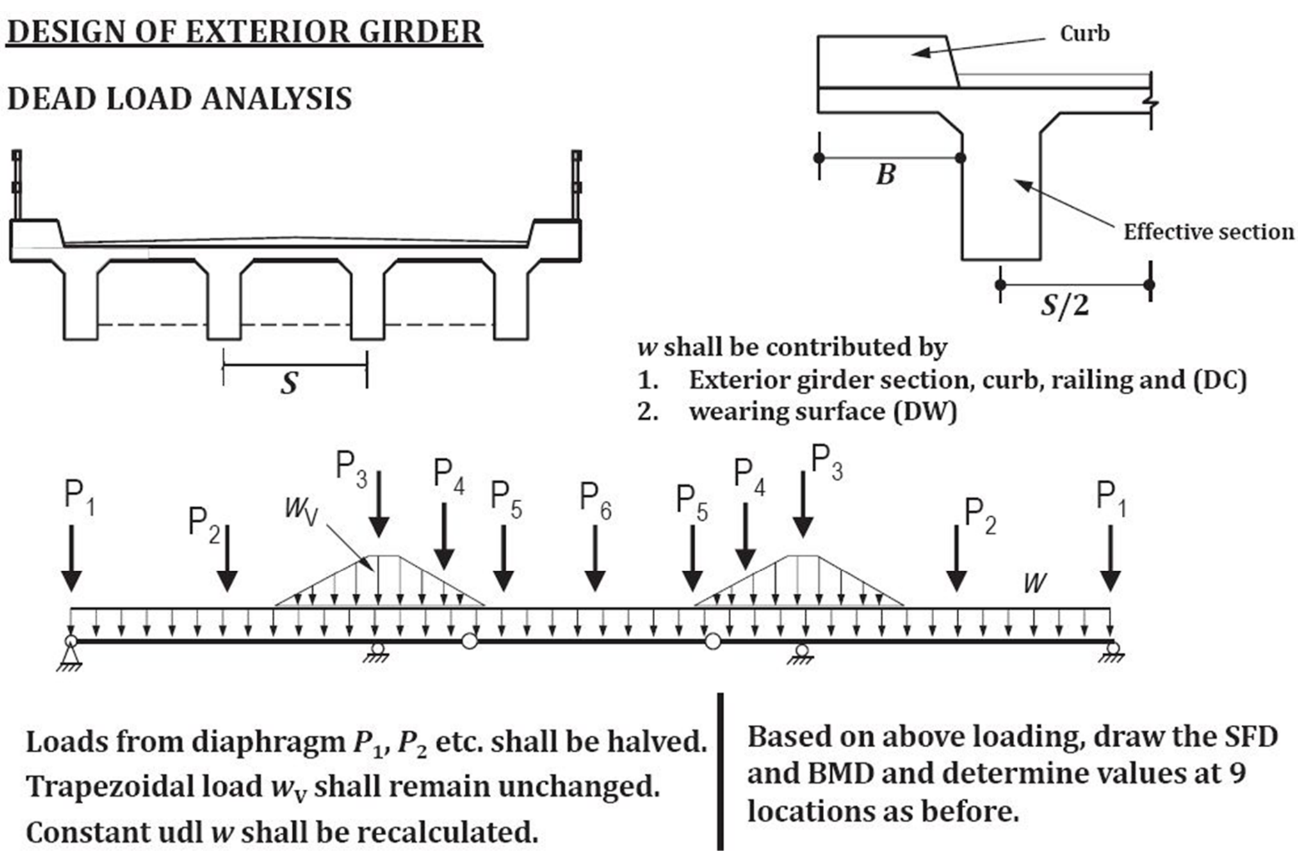


Fig. 45: Dead load on Exterior girder

DESIGN OF EXTERIOR GIRDER

LIVE LOAD ANALYSIS

Distribution factor for moment

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

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

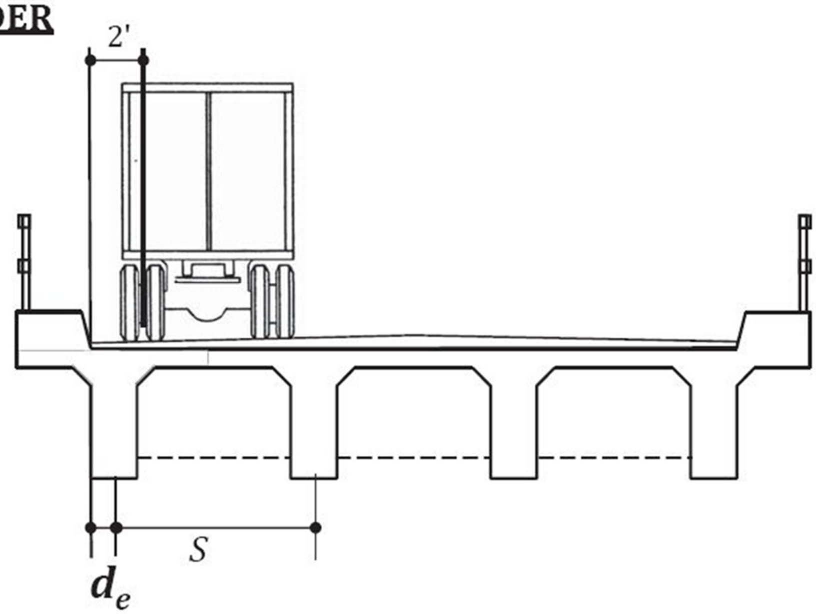
$$-1.0 \le d_e \le 5.5$$

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, 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															

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]

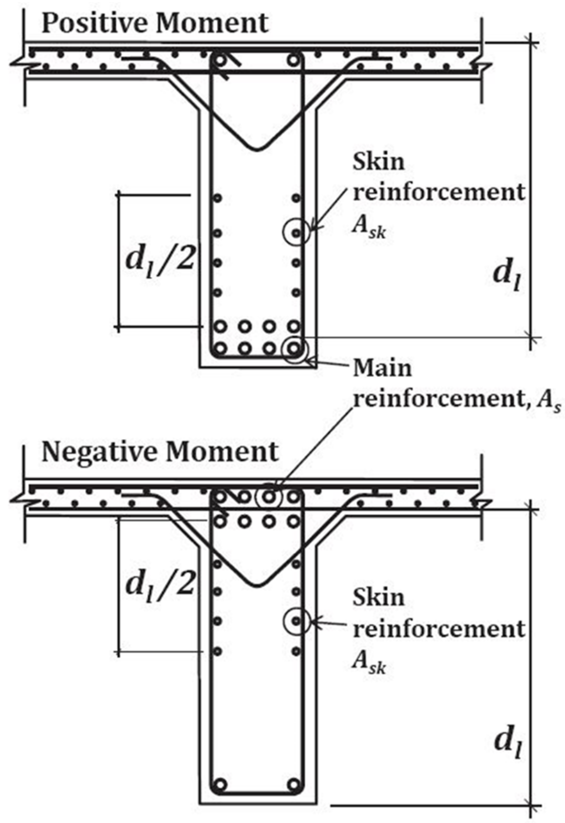
If d_ℓ of non-prestressed or partially prestressed concrete members exceeds 3.0 ft, longitudinal skin reinforcement shall be uniformly distributed along both side faces of the component for a distance d_ℓ /2 (inch) nearest the flexural tension reinforcement.

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

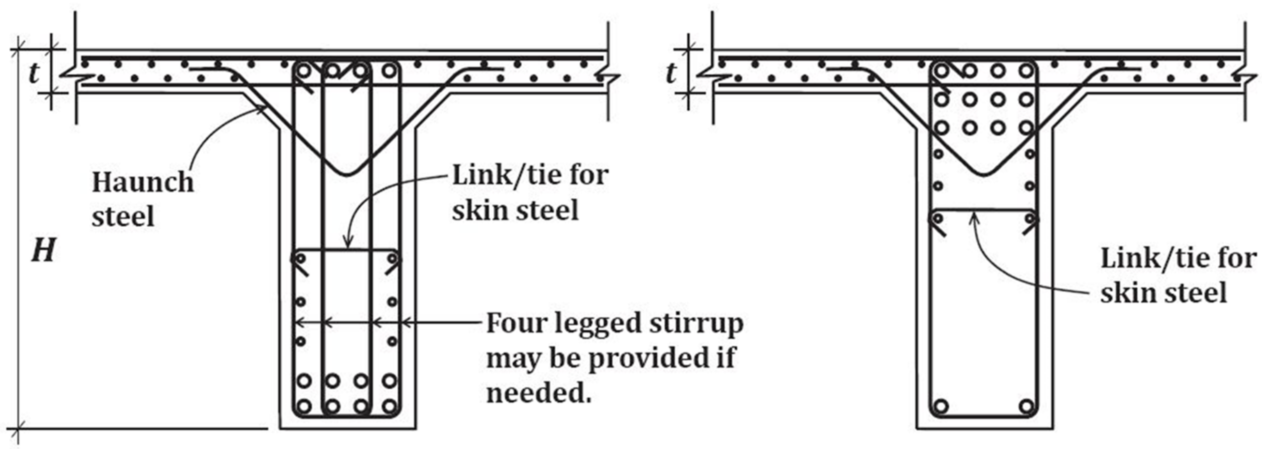
$$A_{sk} \geq 0.012(d_l - 30) \leq \frac{A_s}{4}$$

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_ℓ/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

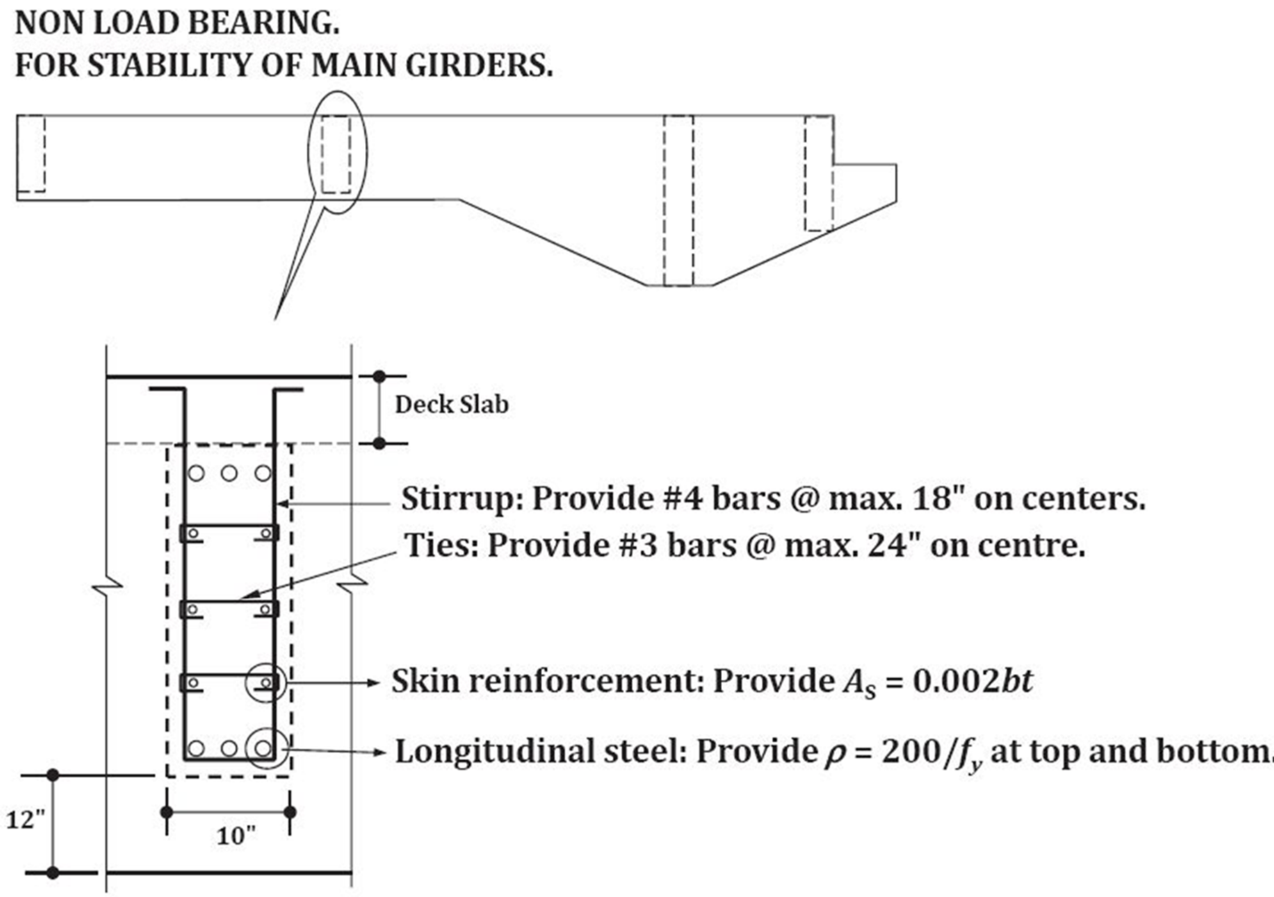


Fig.48 : Reinforcement detailing of cross girder

g) DESIGN OF ARTICULATION

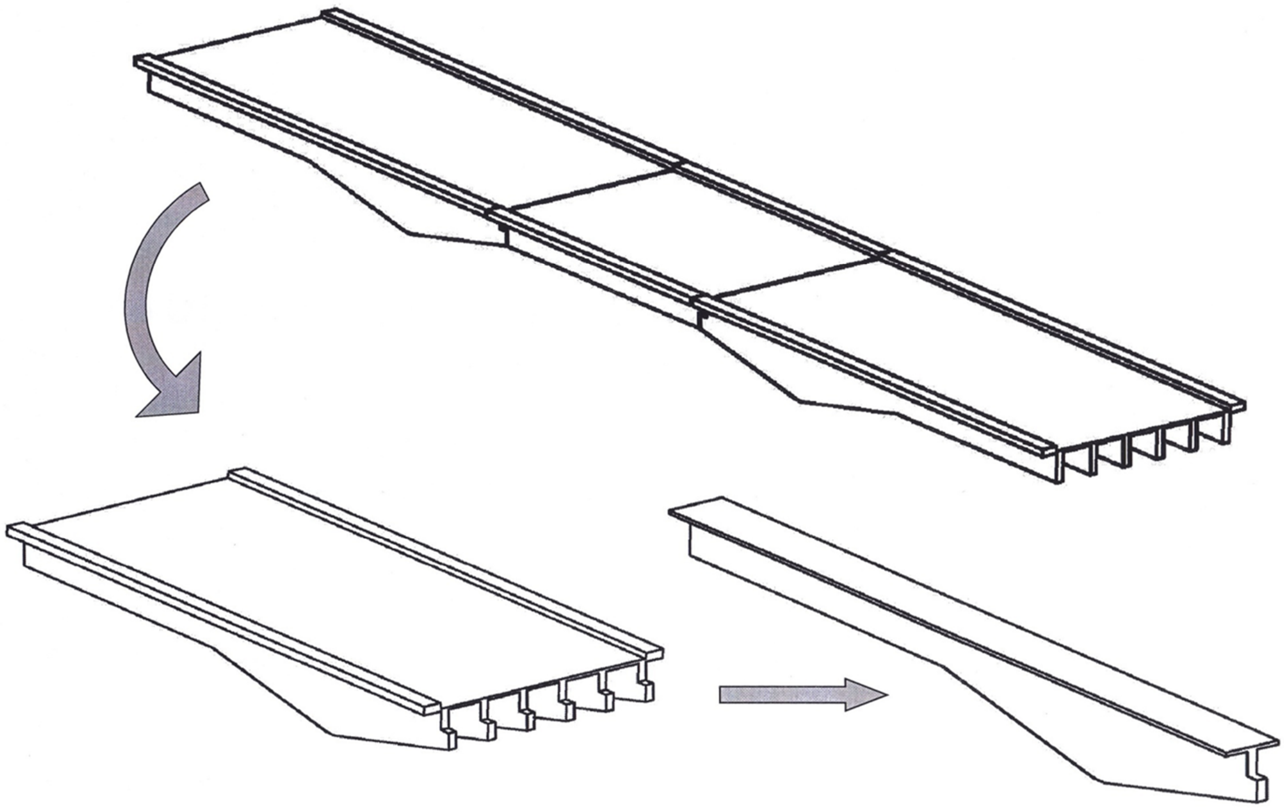


Fig.49 : Articulation or halving joint

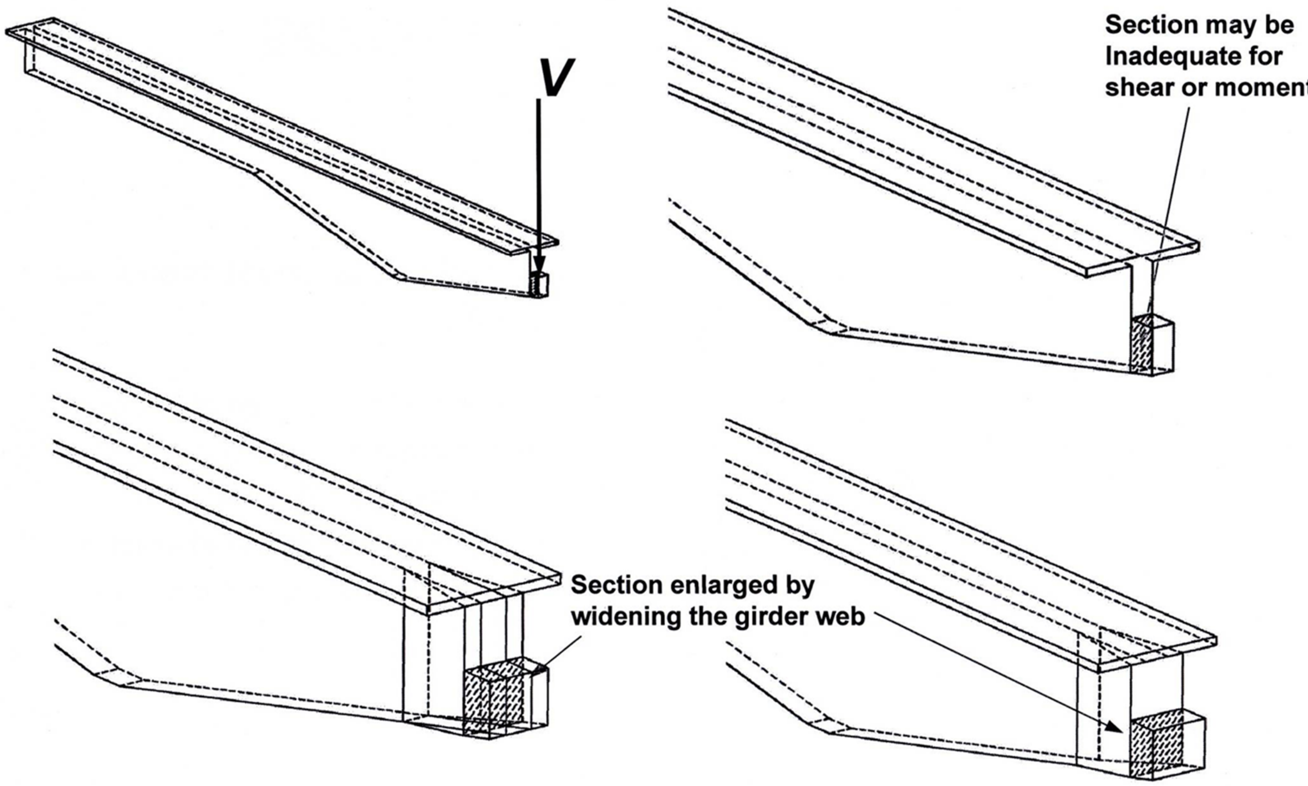


Fig.50 : Widening of girder near articulation location

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.

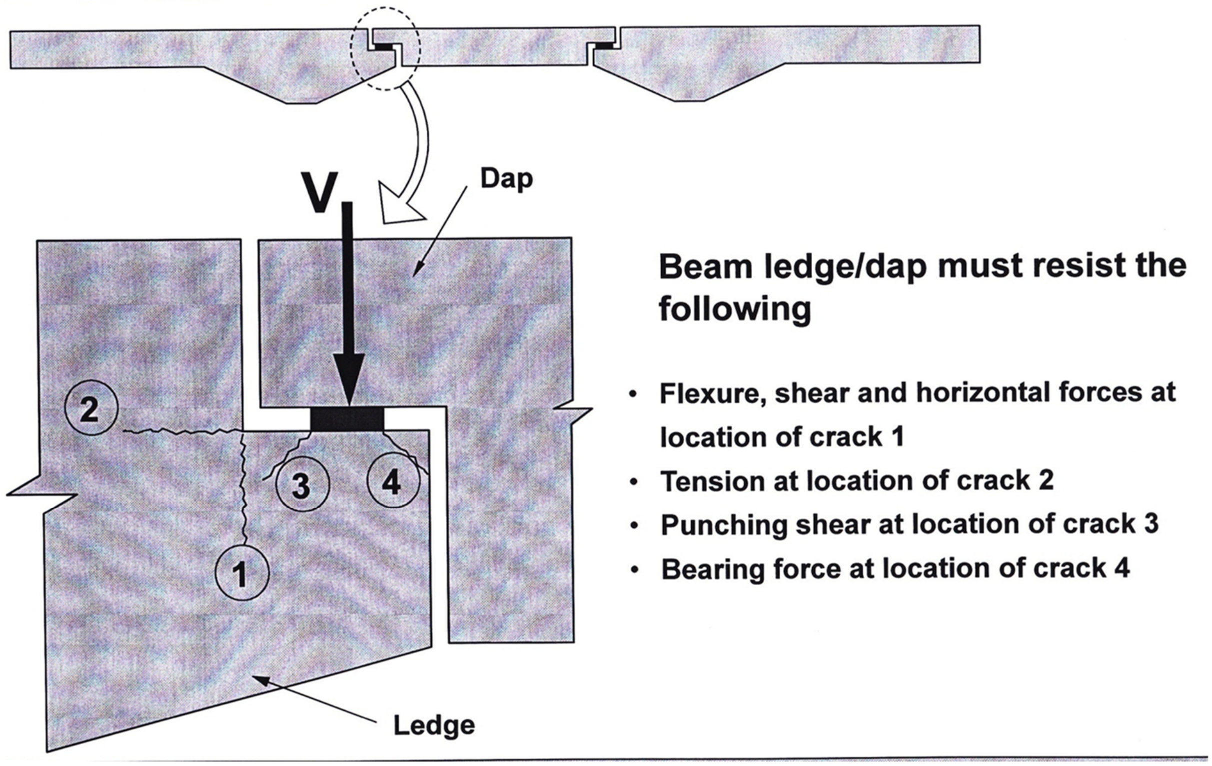


Fig.51 : Cracks at articulation

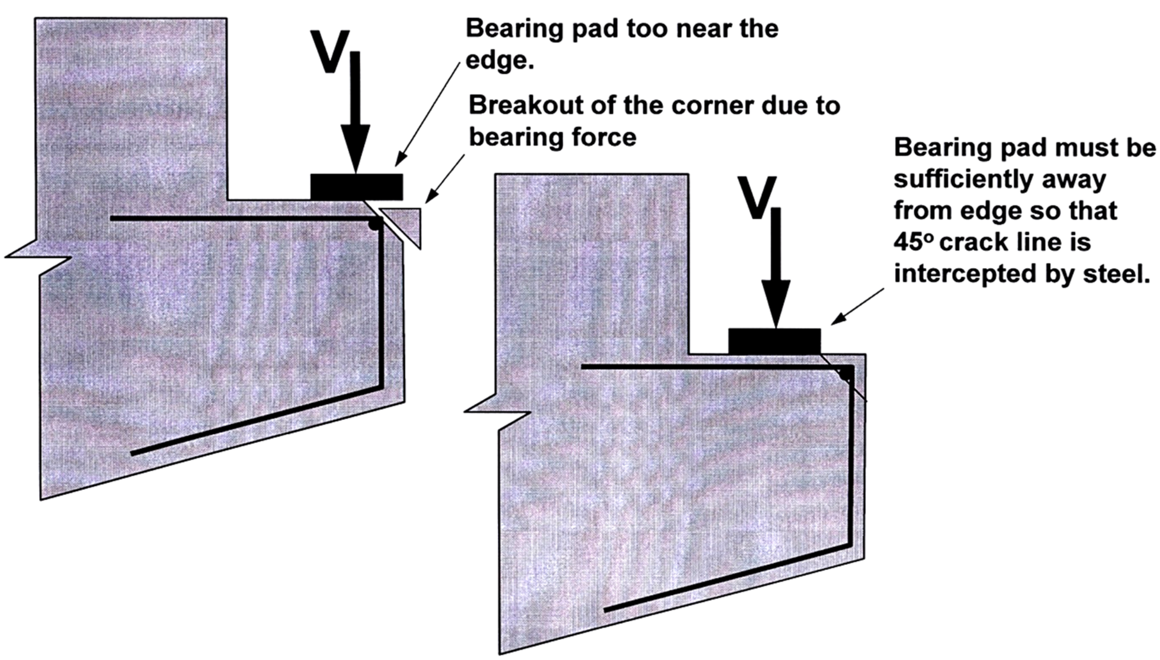


Fig.52 : Clearance requirement around bearing pad near articulation

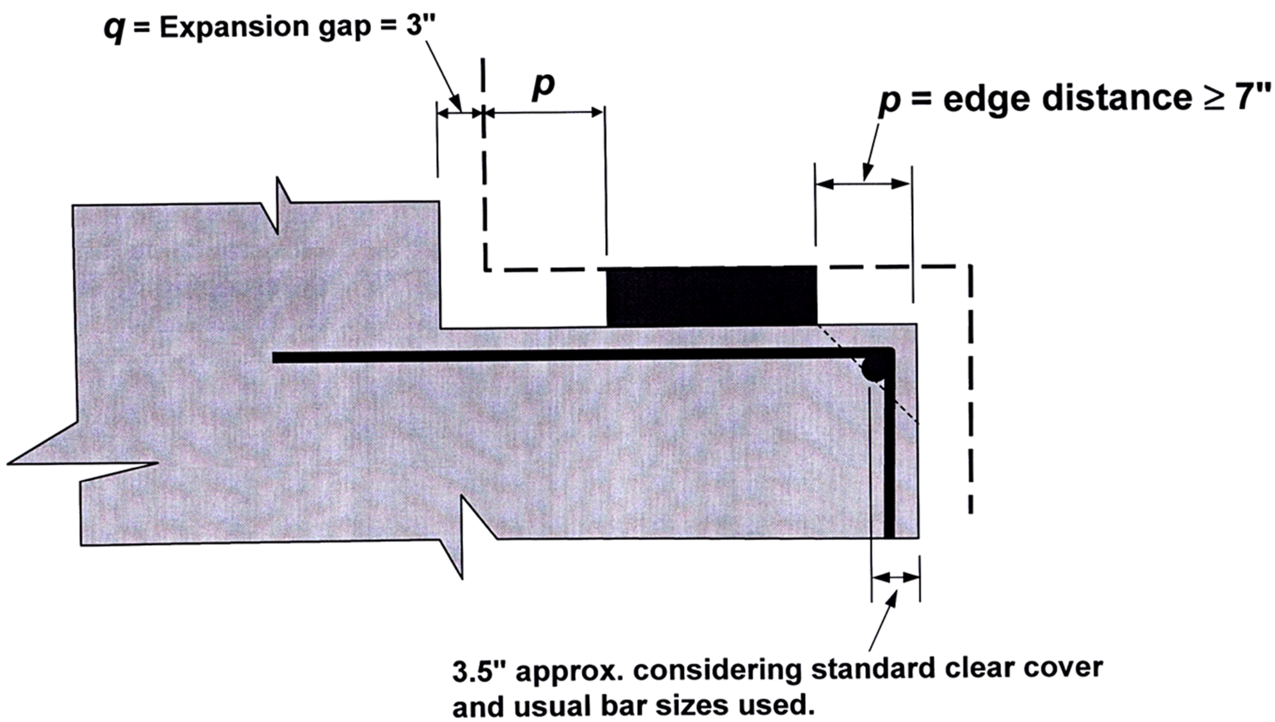
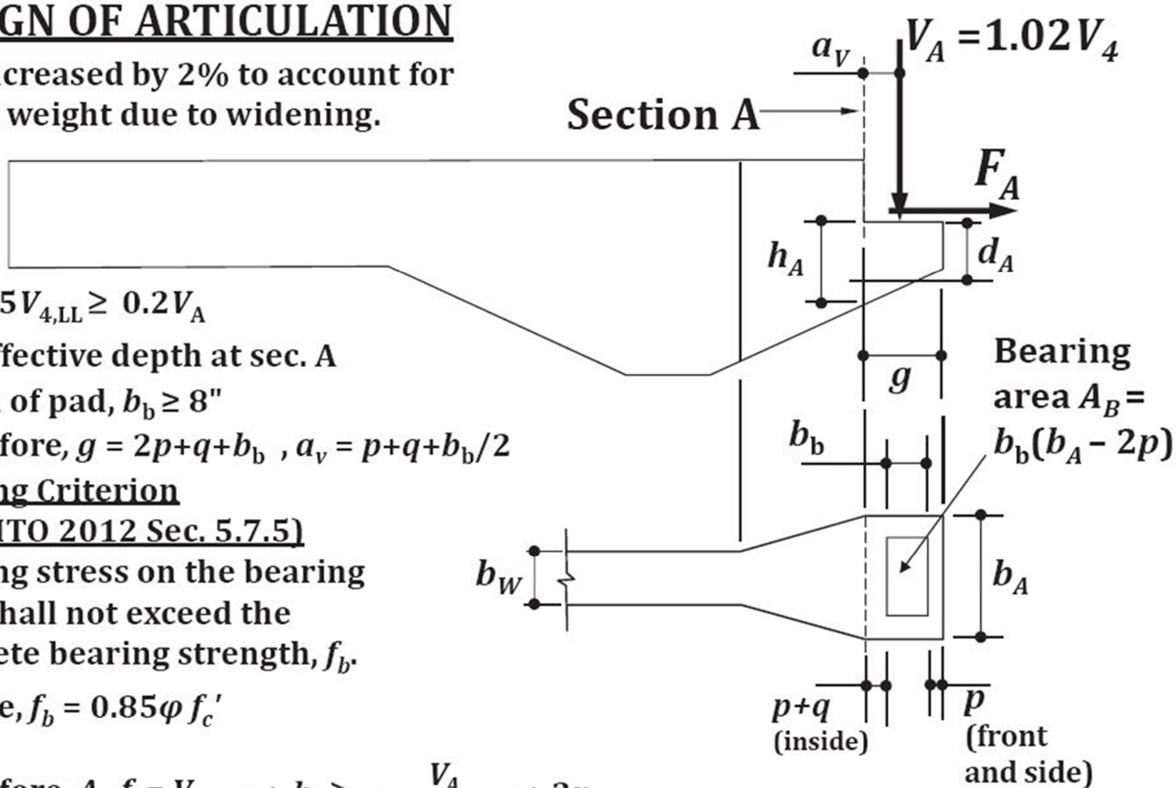


Fig. 53: Expansion gap and edge distance around bearing pad

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

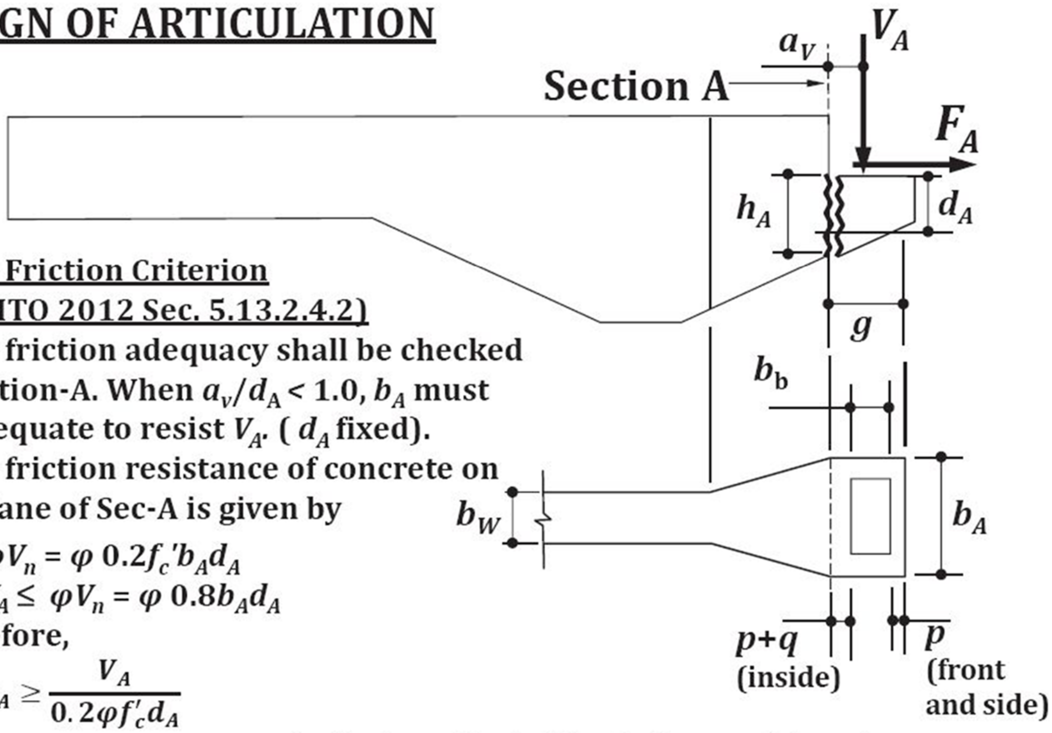
Bearing Criterion
(AASHTO 2012 Sec. 5.7.5)

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



Shear Friction Criterion
(AASHTO 2012 Sec. 5.13.2.4.2)

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

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

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

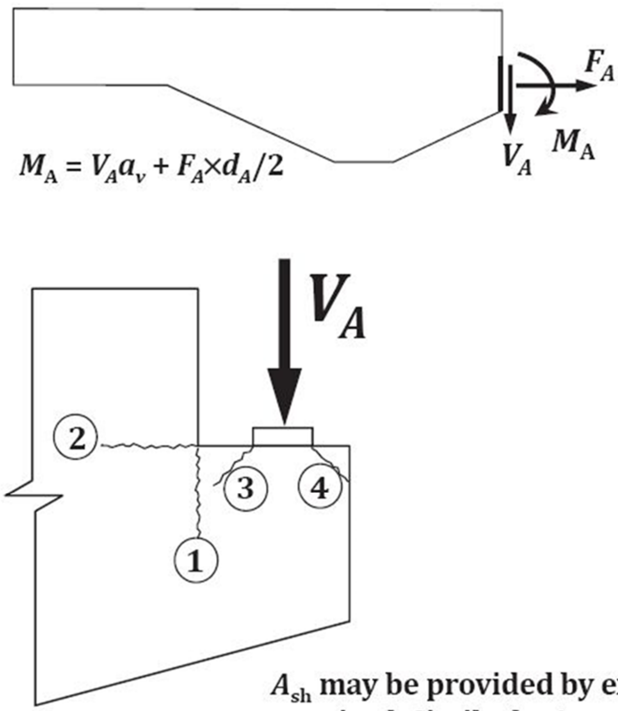
Therefore,

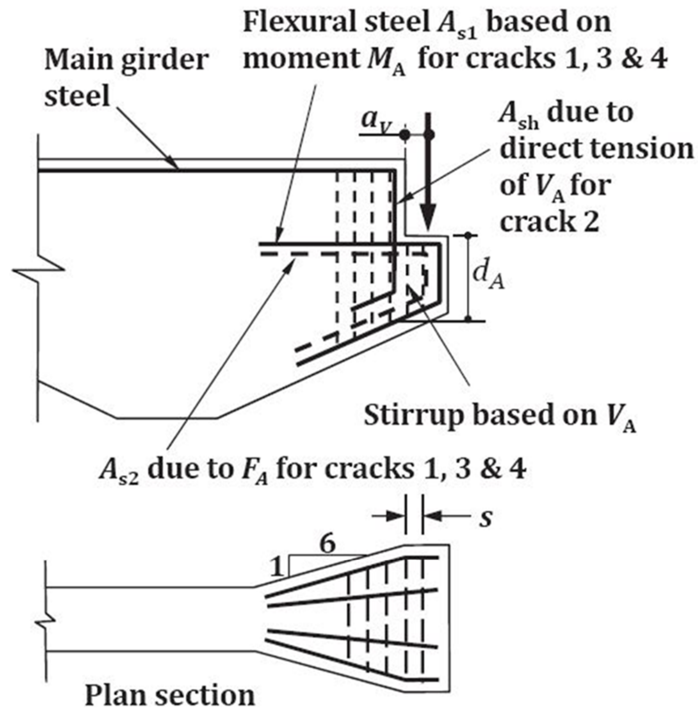
$$b_A \geq \frac{V_A}{0.2\varphi f_c' d_A}$$

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

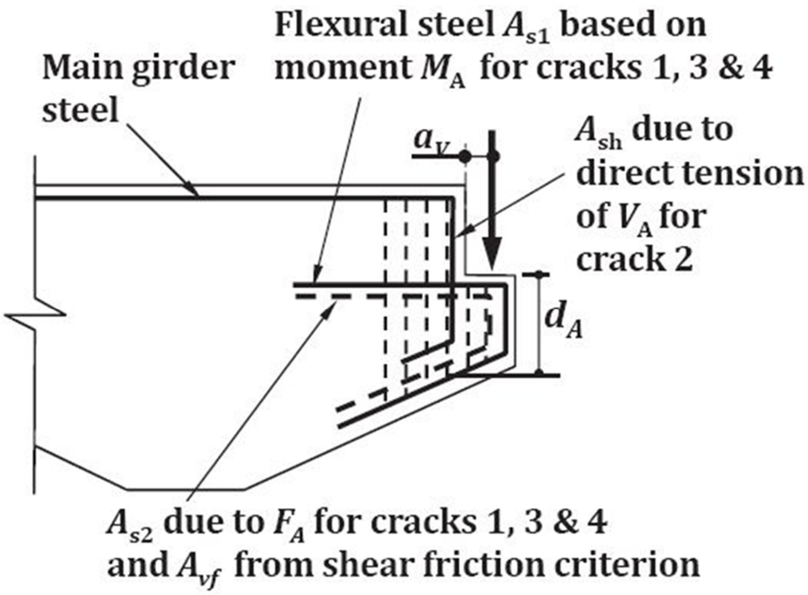
Final value of b_A shall be the larger of the value obtained from bearing and shear friction criteria.

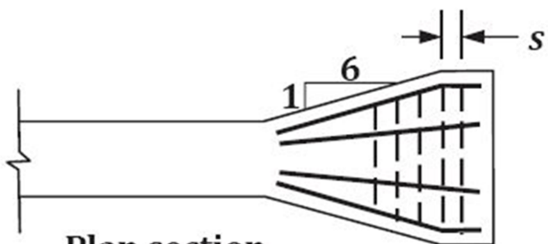
DESIGN OF ARTICULATION





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.





Plan section

For flexural steel A_{s1}

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

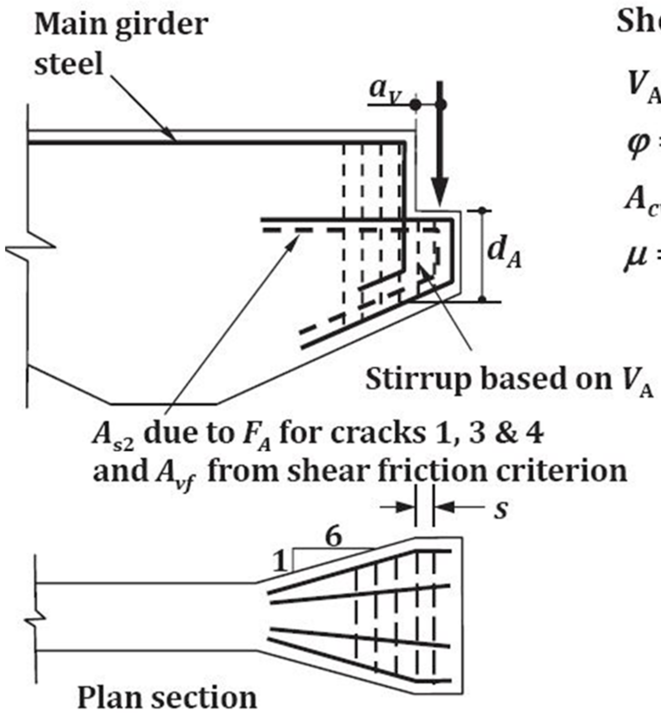
$$a = \frac{A_{s1} f_y}{0.85 f'_c b_A}, \ \varphi = 0.9$$

For steel A_{sh} and A_{s2}

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

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

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]

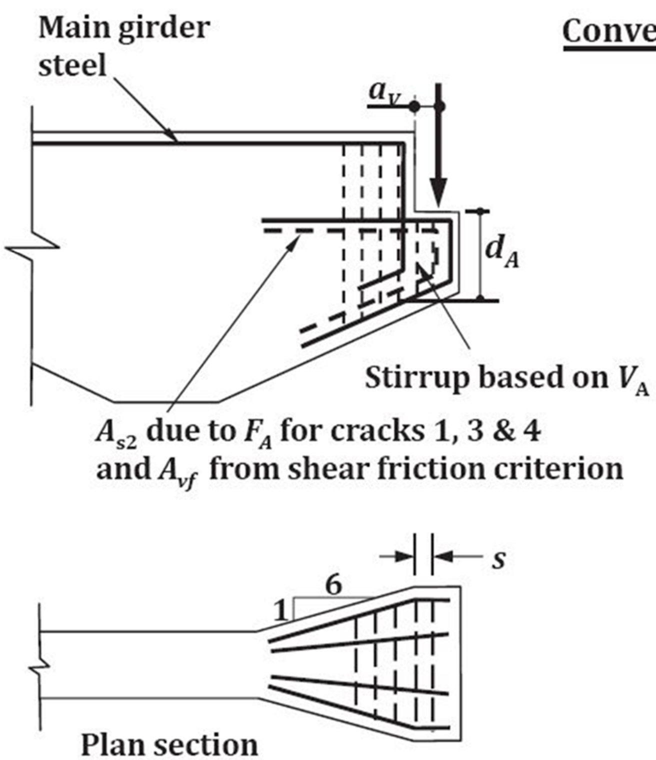
$A_{cv} = b_A \times d_A$ = shear area (in^2)

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

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

$A_{vf} \geq 0.05A_{cv}/f_y$ [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

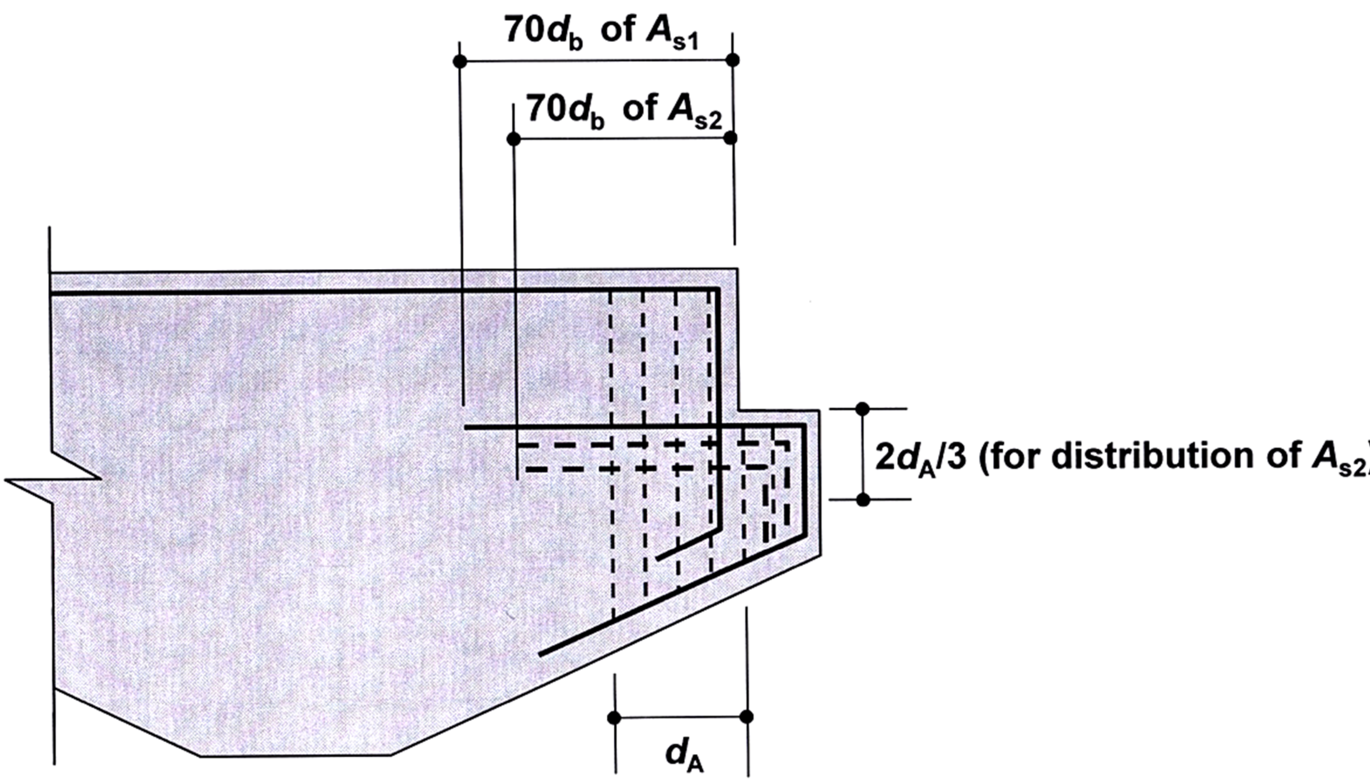


Fig.54 : Reinforcement detailing of articulation

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

4. Other online resources.