Chapter No: 2.2

Chapter Name: Design of edge beam

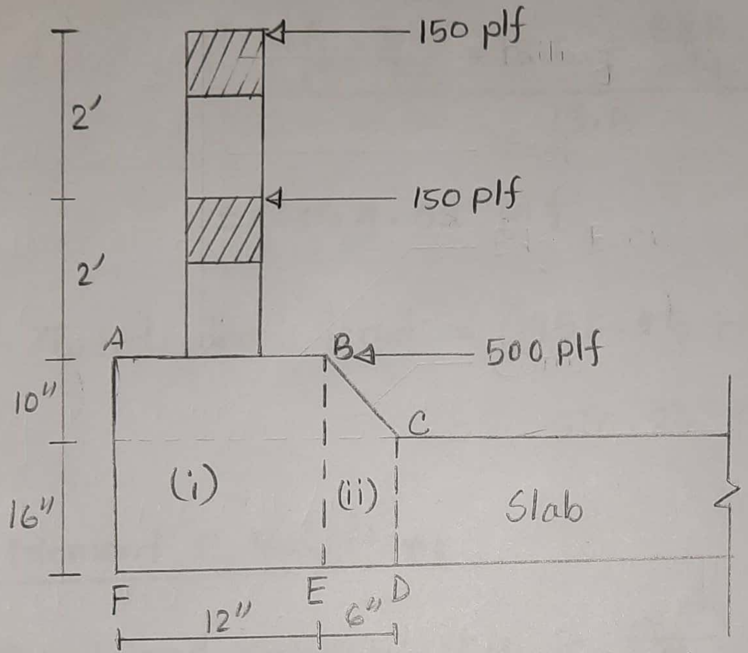

Fig 3.1 : Cross section of edge beam

Design of edge beam :

Load Calculation:

1) Dead load of section(i) and(ii) $= \left\{ \frac{10+16}{12} \times \frac{12}{12} + \frac{1}{2} \times \frac{16+26}{12} \times \frac{6}{12} \right\} \times 150$

$= 456.25$ plf

Section of railing is 6″ × 6″

Dead load of railing and post

$$= \frac{\left[2 \times \frac{6 \times 6}{144} \times 13.5 + \frac{8 \times 8}{144} \times 4 \times 4\right] \times 150}{13.5}$$

$$= 154.01 \text{ plf}$$

Total dead load $= (456.25 + 154.01)$ plf

$$= 610.26 \text{ plf}$$

Moment Calculation:

i) Dead load moment, $M_D = \frac{wl^2}{8} = \frac{610.26 \times (13.5)^2}{8}$

$$= 13902.54 \text{ lb-ft}$$

ii) Live load moment, $M_L = 0.1 \times H_{20}S_{16} \times L$

$$= 0.1 \times 16000 \times 13.5$$

$$= 21600 \text{ lb-ft}$$

Total moment = Dead load moment + live load moment

$$= 13902.54 + 21600 = 35502.54 \text{ lb-ft}$$

Depth Check:

$$d_{req} = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{35502.54 \times 12}{256.41 \times 12}} = 11.77''$$

$$d_{eff} = \left[26'' - 1.5'' - \frac{4}{8}'' - \frac{6}{8\times 2}''\right]$$

$$= 23.63''$$

$d_{req} < d_{eff}$

$\therefore$ The design is okay.

Steel Calculation:

$$A_s = \frac{M}{f_s j d}$$

$$= \frac{35502.54 \times 12}{24000 \times 0.88 \times 23.63}$$

$$= 0.85 \text{ in}^2$$

providing
5#4 bars

Critical section moment:

$$M = \frac{500 \times 26}{12} + 150 \times \left(2 + \frac{26}{12}\right) + 150 \times \left(4 + \frac{26}{12}\right)$$

$$= 2633.33 \text{ lb-ft}$$

Depth Check: $d = \sqrt{\frac{M}{Rb}}$

$$d_{check} = \sqrt{\frac{2633.33 \times 12}{256.41 \times 12}}$$

$$= 3.2''$$

$$d_{eff} = 16 - 2.25$$

$$= 13.75$$

$\therefore d_{check} < d_{eff}$ $\therefore$ The design is okay.

Steel Calculation:

$$A_s = \frac{M}{f_s j d} = \frac{2634 \times 12}{24000 \times 0.88 \times 13.75}$$

$$= 0.11 \text{ in}^2$$

Proving #3 barr @ 12'' c/c

Working diagram:

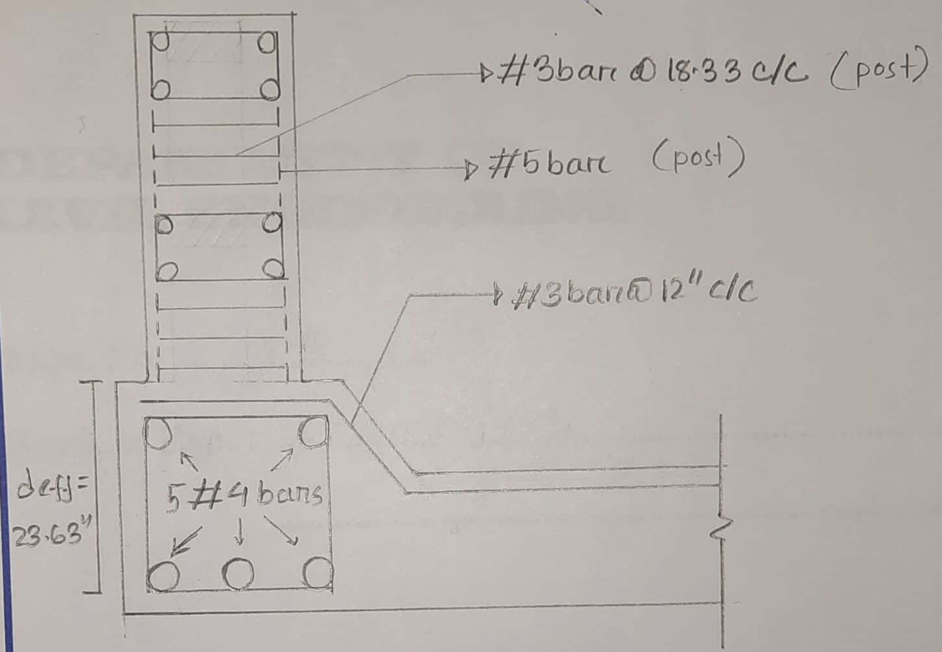

Fig 3.3 : Reinforcement details of edge beam.

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DEPARTMENT OF CIVIL ENGINEERING

Expt. No. ....2·3.........

Name of Expt ....Design of slab..

....

	SUBMITTED BY :
SUBJECT :	NAME : ....
COURSE NO. :..............................	CLASS :..................................
DATE OF EXPT.:..........................	GROUP :........ ROLL NO ....
DATE OF SUB. :..........................	SESSION : ....

Chapter No: 2.3

Chapter Name: Design of slab

Design of slab:

Span length = 13.5′

Let us assume, thickness of slab, t = 16″

i) Load Calculation:

i) Self-weight of slab = $\frac{16}{12} \times 150$ = 200 psf

ii) Wearing course = 20 psf

∴ Total load = (200+20) = 220 psf = 0.22 ksf

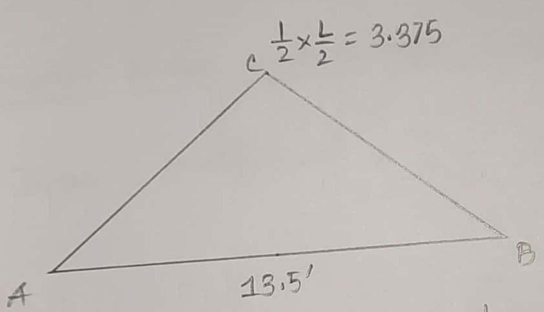
Distribution wheel load, E = 4 + 0.06 S

= 4 + 0.06 × 13.5

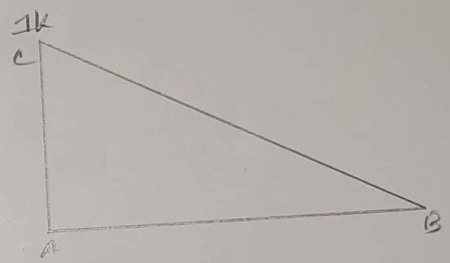
= 4.81 ≤ 7

11) Moment Calculation & Shear calculation:

Live load moment:



IL of maximum moment



IL of maximum shear

Load per unit width of slab $= \frac{P}{E} = \frac{16}{4.81} = 3.33k$

Live load moment (maximum) $= \frac{P}{E} \times \frac{L}{4} = 3.33 \times \frac{13.5}{4}$

$= 11.24 \text{ k-ft}$

Maximum shear $= 3.33 \times 1 = 3.33$ kip

Dead load Moment: $= \frac{wl^2}{8} = \frac{0.22 \times (13.5)^2}{8} = 5.012$ k-ft

Dead load shear $= \frac{wl}{2} = \frac{0.22 \times 13.5}{2}$

$= 1.485$ kips

Impact Shear and moment Calculation:

Impact coefficient, $I = \frac{50}{l+125} \leq 0.03$

$= \frac{50}{13.5+125}$

$= 0.36$

$\therefore I = 0.3$

Impact moment $= 0.3 \times$ live load moment

$= 0.3 \times 11.24$

$= 3.372$ k-ft

Impact shear $= 0.3 \times$ live load shear

$= 0.3 \times 3.33$

$= 0.999$ kip

∴ Design moment = Dead load moment + live load moment + Impact moment

$= 5.012 + 11.24 + 3.372$

$= 19.62$ k-ft

∴ Design shear = Dead load shear + live load shear + Impact shear

$= 1.485 + 3.33 + 0.999$

$= 5.814$ k

Depth check:

$$d = \sqrt{\frac{M}{Rb}}$$

$$= \sqrt{\frac{19.62 \times 12000}{256.41 \times 12}}$$

$$= 8.75'' \simeq 9''$$

$d_{eff} = 16'' - 1'' = 15''$

∴ $d_{req} < d_{eff}$ ∴ The design is okay.

Steel Calculation:

$$A_s = \frac{M}{f_s j d}$$

$$= \frac{19.62 \times 12000}{24000 \times 0.88 \times 15}$$

$$= 0.74 \text{ in}^2$$

3 #5 bar are used.

$\therefore$ proving #5 bar @ 5" c/c

Distribution reinforcement:

$$A_s = \frac{100}{\sqrt{S}}$$

$$= \frac{100}{\sqrt{13.5}}$$

$$= 27.22\ \%$$

$$A_s = \frac{27.22}{100} \times 0.74$$

$$= 0.20 \text{ in}^2$$

providing #4 bar @ 12" c/c

Steel Calculation:

$$A_s = \frac{M}{f_s j d}$$

$$= \frac{19.62 \times 12000}{24000 \times 0.88 \times 15}$$

$$= 0.74 \text{ in}^2$$

3 #5 bar are used.

∴ proving #5 bar @ 5" c/c

Distribution reinforcement:

$$A_s = \frac{100}{\sqrt{S}}$$

$$= \frac{100}{\sqrt{13.5}}$$

$$= 27.22\ \%$$

$$A_s = \frac{27.22}{100} \times 0.74$$

$$= 0.20 \text{ in}^2$$

providing #4 bar @ 12" c/c

Shear Check:

Developed shear force, $V_{dev} = 5.814k = 5814$ lb

Allowable shear force, $V_{all} = 1.1\sqrt{f_c'}\,bd$

$= 1.1\sqrt{3500} \times 12 \times 15$

$= 11713.84$ lb

$\because V_{dev} < V_{all}$ $\therefore$ The design is okay.

Working diagram:

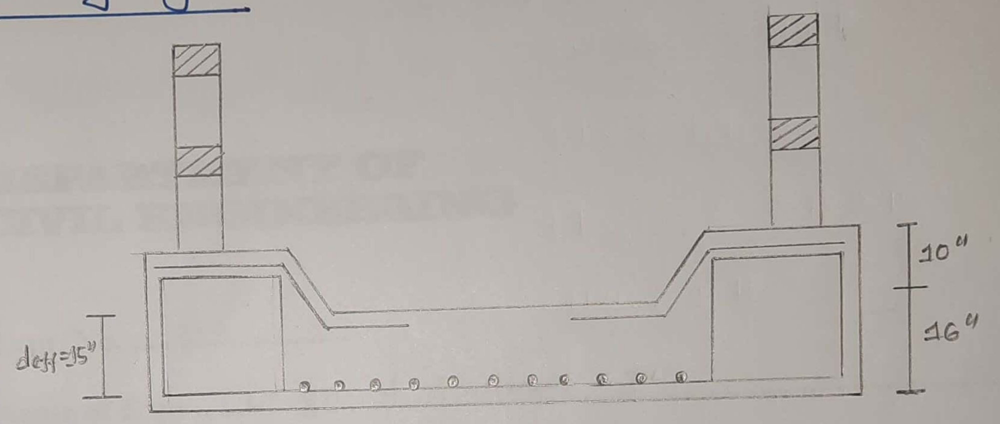


Fig: Reinforcement details of slab

Chapter No: 2.4

Chapter Name: Design of Abutment Wall

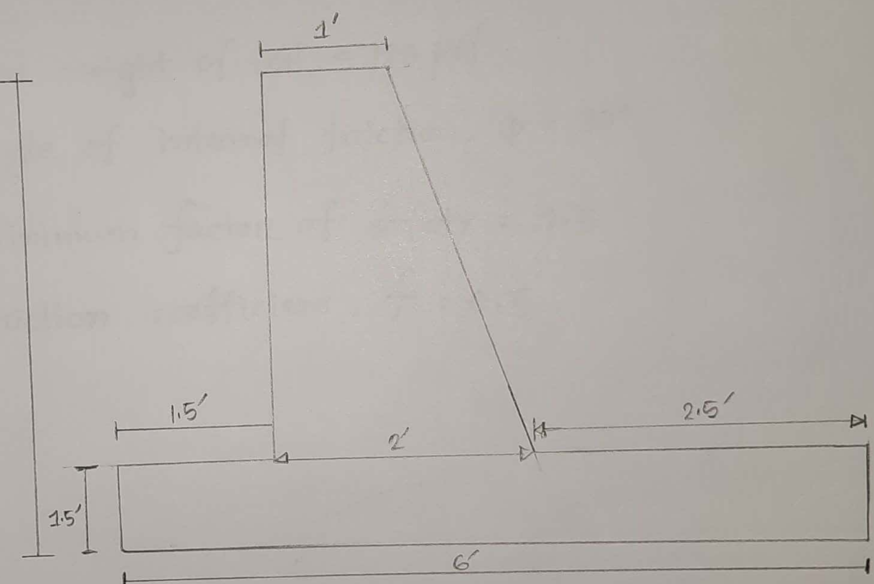


Figure: Abutment

Specifications:

1. Height of abutment = 10′
2. Allowable soil pressure = 3 ksf
3. Unit weight of soil = 120 psf
4. Angle of internal friction, $\phi = 30°$
5. Minimum factor of safety = 1.5
6. Friction coefficient, $f = 0.5$

Coefficient of active earth pressure:

$$K_A = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-\sin 30^\circ}{1+\sin 30^\circ} = \frac{1}{3} = 0.333$$

Coefficient of passive earth pressure:

$$K_P = \frac{1+\sin\phi}{1-\sin\phi} = \frac{1+\sin 30^\circ}{1-\sin 30^\circ} = 3$$

Total Active earth pressure $= k_a \gamma H = \frac{1}{3} \times 120 \times 10$

$= 400$ psf

Force in active earth pressure, $P_a = \frac{1}{2} \times 400 \times 10$

$= 2000$ lb

Overturning moment, $M_o = P_a \times \frac{L}{3}$

$= 2000 \times \frac{10}{3}$

$= 6666.67$ lb-ft

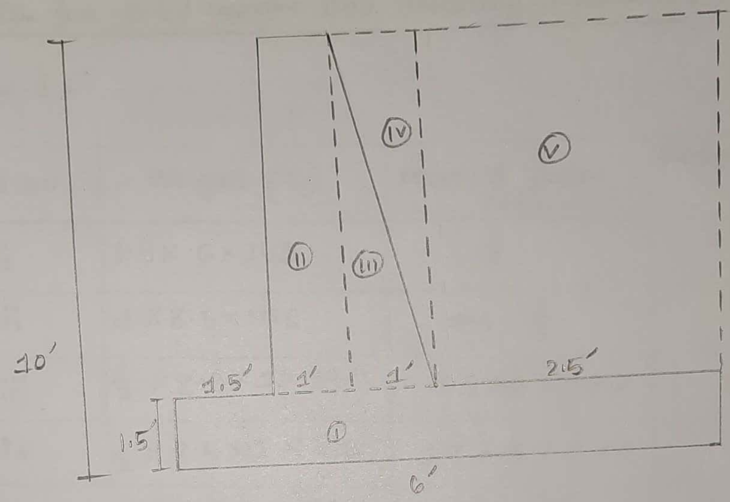


Figure: Section for calculating resisting moment

Table for total weight and resisting moment :

Case-I :

Section	Weight (lb)	Moment arm (ft)	Resisting moment (lb-ft)
i	$1.5 \times 6 \times 150$	3	4050
ii	$1 \times 8.5 \times 150$	$1.5 + \frac{1}{2}$	2550
iii	$\frac{1}{2} \times 8.5 \times 1 \times 150$	$1.5 + 1 + \frac{1}{3} \times 1$	1806.25
iv	$\frac{1}{2} \times 8.5 \times 1 \times 120$	$1.5 + 1 + \frac{2}{3} \times 1$	1615
v	$2.5 \times 8.5 \times 120$	$1.5 + 2 + \frac{2.5}{2}$	12112.5
Total	= 6322.5		22133.75

Case II :

Loads	Weight (lb)	Moment arm (ft)	Resisting moment (lb-ft)
Self weight + Soil weight	6322.5		22133.75
Dead load from super structure	2420	2	4840
Total	8742.5		26973.75

Case III :

Loads	Weight (lb)	Moment arm (ft)	Moment (lb-ft)
Self weight + Soil weight + Dead load from Super structure	8742.5		26973.75
Live load	3000	2	6000
Total	11742.5		32973.75

Stability Check:

Case-1: Superstructure is absent

$$\text{Overturning safety} = \frac{\text{Resisting moment}}{\text{Overturning moment}}$$

$$= \frac{M_R}{M_O}$$

$$= \frac{22133.75}{6666.67} = 3.32 > 1.5$$

∴ The design is okay.

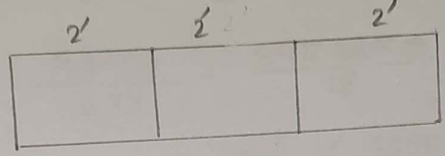
$$\text{Factor of safety against sliding} = \frac{W \times f}{P_a}$$

$$= \frac{6322.5 \times 0.5}{2000}$$

$$= 1.58 > 1.5$$

∴ the design is okay.

Check for bearing capacity of soil:



Location of resultant force, $\bar{x} = \frac{M_R - M_O}{w}$

$= \frac{22133.75 - 6666.67}{6322.5}$

$= 2.45$

Eccentricity, $e = \frac{b}{2} - \bar{x}$

$= 3 - 2.45 = 0.55'$

$\therefore M = w \times e = 6322.5 \times 0.55 = 3477.38$ lb-ft

Moment of inertia, $I = \frac{bh^3}{12} = \frac{1.0 \times 6^3}{12}$

$= 18$

Soil pressure,

$\therefore \sigma = \frac{P}{A} \pm \frac{MC}{I} = \frac{6322.5}{6 \times 1} + \frac{3477.38 \times 3}{18}$

$= 1633.31$ psf < 3000 psf

$$\text{or } \sigma = \frac{P}{A} - \frac{MC}{I}$$

$$= \frac{6322.5}{6\times1} - \frac{3477.38\times3}{18}$$

$$= 474.19 \text{ psf} < 3000 \text{ psf}$$

$\therefore$ the design is okay.

Case-II: Superstructure is present but no live load on superstructure

Factor of safety against overturning $= \frac{M_R}{M_O} = \frac{26973.75}{6666.67}$

$= 4.05 > 1.5$

Factor of safety against Sliding $= \frac{W \times f}{P_a} = \frac{8742.5\times0.5}{2000}$

$= 2.19 < 1.5$

$\therefore$ the design is okay.

Check for bearing capacity of soil:

Location of resultant force, $\bar{x} = \dfrac{M_R - M_O}{W}$

$$= \frac{26973.75 - 6666.67}{8742.5}$$

$$= 2.32'$$

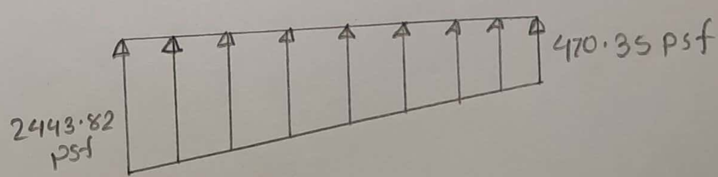
$e = \dfrac{b}{2} - \bar{x} = 3' - 2.32' = 0.68'$

$M = w \times e = 8742.5 \times 0.68 = 5920.42$ lb-ft

$$\sigma = \frac{P}{A} \pm \frac{MC}{I} = \frac{8742.5}{6 \times 1} \pm \frac{5920.42 \times 3}{18}$$

$$= (2443.82 \text{ or } 470.35) \text{ psf} < 3000 \text{ psf}$$

$\therefore$ The design is ok



Case-III: Superstructure is present with Dead load and live load.

Factor of safety against overturning $= \frac{M_R}{M_O}$

$= \frac{32973.75}{6666.67}$

$= 4.95 > 1.5$

Factor of safety against Sliding $= \frac{W \times f}{P_a}$

$= \frac{11742.5 \times 0.5}{2000}$

$= 2.94 > 1.5$

∴ the design is okay.

Check for bearing capacity of soil:

Location of resultant force, $\bar{x} = \frac{M_R - M_O}{W}$

$= \frac{32973.75 - 6666.67}{11742.5}$

$= 2.24$

eccentricity, $e = \frac{b}{2} - \bar{x}$

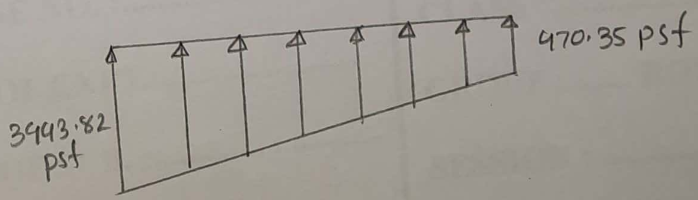
$= 3 - 2.24 = 0.76'$

$\therefore M = w \times e = 11742.5 \times 0.76 = 8920.42$ lb-ft

$\therefore \sigma = \frac{P}{A} \pm \frac{MC}{I} = \frac{11742.5}{6 \times 1} \pm \frac{8920.42 \times 3}{18}$

$= (3443.82$ or $470.35)$ psf

< 3000 psf



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Expt. No. ....

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DATE OF SUB. :..........................	SESSION : ....

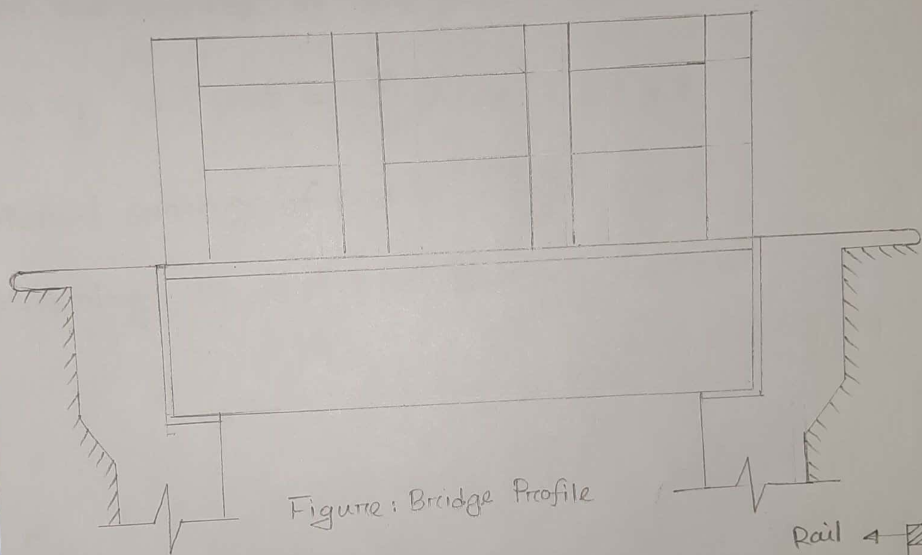

Figure : Bridge Profile

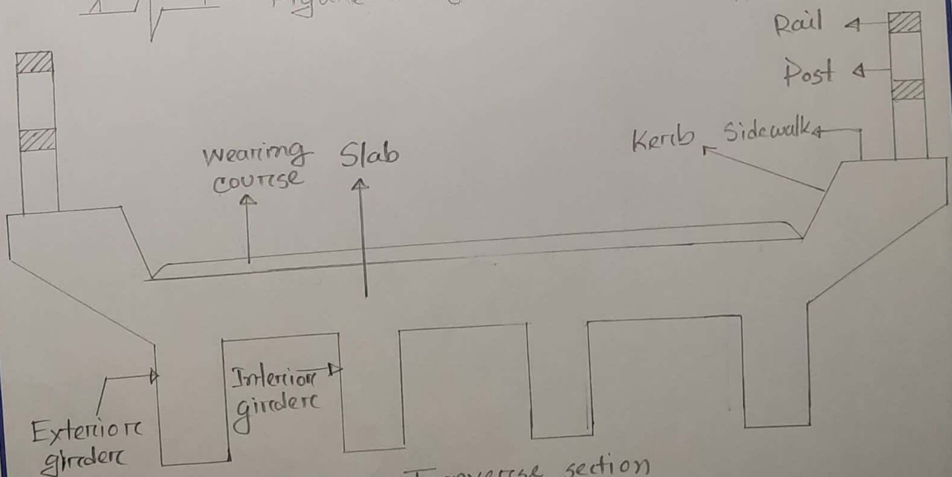

Figure : Tranverse section
Deck girder Bridge

Design of Railing:

Let the spacing of rail posts = 6.5′

No of rail post = $\frac{38.67}{6.5} + 1 = 6.9 \simeq 7$

Actual spacing of post = $\frac{38.67}{7-1} = 6.4'$

Assuming the section of railing = 6″×6″

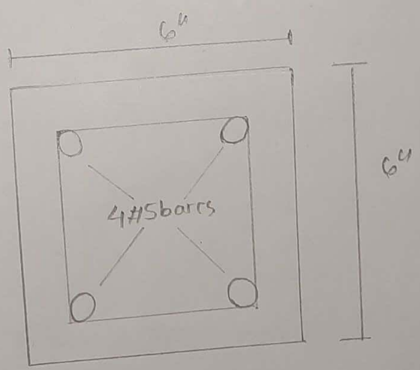

Figure: Cross section of railing

Design of rail post:

Assuming the section of rail post = 8" × 8"

Height of the post = 4'

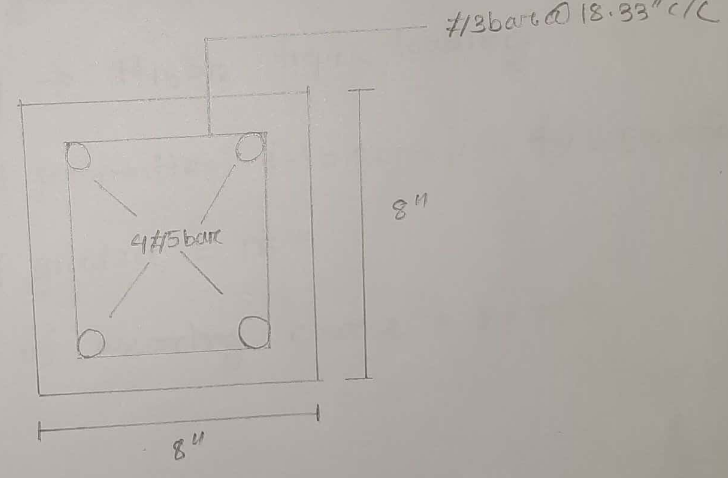

Figure: Cross section of post

Design Specifications:

i) Clear span length = last 2 digit of roll + $\frac{50}{3}$ = ~~38~~.

= $22 + 50/3 = 38.67'$

ii) Width of the bridge = equivalent two lanes = 20'

iii) No of girder = 4

iv) Loading → $H_{15}S_{12}$ truck loading

v) Material properties, $f_c' = 3000$ psi, $f_y = 50{,}000$ psi

vi) Width of girder = 15"

vii) Weight of wearing course = 20 psf

Design Calculations:

Center to center spacing of girder $= \frac{20}{4-1} = 6.67'$

Clear spacing between two girders $= \left(6.67 - \frac{15}{12}\right)$
$= 5.42'$

Relevent Properties:

$n = 9$, $\pi = 14.81$, $K = 0.38$, $j = 0.87$, $R = 223.16$ psi

Slab design:

Assuming the thickness of slab $= 6''$

Load Calculation:

Self weight of slab $= \frac{t}{12} \times 150 = \frac{6}{12} \times 150 = 75$ psf

Weight of wearing course $= 20$ psf

Total Dead load $= 95$ psf

Dead load moment Calculation:

For continuity of slab,

Dead load moment, $M_o = \frac{WL^2}{10}$

$= \frac{1}{10} \times 95 \times (\cancel{38.67})^2 (5.42)^2$

$= 279.08$ lb-ft

Live load moment Calculation:

For main reinforcement perpendicular to traffic

$= \frac{S+2}{32} \times P_{15}$

$= \frac{5.42+2}{32} \times 12000$

$= 2782.5$ lb-ft

Impact moment Calculation:

Impact factor, $I = \frac{50}{L+125} = \frac{50}{5.42+125} = 0.38$ should be <.30

$\therefore I = 0.3$

Impact moment $=$ live moment $\times 0.3\cancel{8}$

$= 2782.5 \times 0.3\cancel{8} = 834.75$ lb-ft

$\therefore$ Design moment, $M_D = M_0 + M_{LL} + M_I$

$= 279.08 + 2782.5 + 834.75$

$= 3896.33$ lb-ft

Depth Check:

$$d_{req} = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{3896.3 \times 12}{223.16 \times 12}} = 4.18''$$

$d_{actual} = 6'' - 1 = 5''$

$d_{actual} > d_{req}$

$\therefore$ the design is okay.

Main reinforcement Calculation:

$$A_s = \frac{M}{f_s j d} = \frac{3896.3 \times 12}{20000 \times 0.87 \times 5} = 0.53 \text{ in}^2$$

$$\text{spacing} = \frac{0.31 \times 12}{0.53} = 7.02'' \simeq 7'' \text{ c/c}$$

using #5 bar @ 7" c/c

Distribution reinforcement: For main reinforcement perpendicular to traffic

$$A_{st} = \frac{220}{\sqrt{5}} \leq 67\% \text{ of main reinforcement}$$

$$= \frac{220}{\sqrt{5.42}} = 94.5 \leq 67\%$$

$$A_{st} = 0.67 \times 0.53 = 0.36 \text{ in}^2$$

using #4 bar @ $\frac{0.20 \times 12}{0.36} = 6.67 \simeq 6.5''$ c/c

Shear Check:

Dead load shear:

Developed shear, $V_{max} = \frac{WL}{2} = \frac{95 \times 5.42}{2} = 257.45$ lb/ft

Live load shear:

Live load on per unit width of slab $= \frac{P}{E}$

$$= \frac{12000}{4 + 0.06 \times 38.67}$$

$$= 1898.67 \text{ lb/ft}$$

Impact shear $= 1898.67 \times 0.3 = 569.60$ lb/ft

Shear Stress:

$$u_b = \frac{V_{max}}{bd} = \frac{257.45 + 1898.67 + 569.60}{12 \times 5 \times 12}$$

$$= 3.79 \text{ psi}$$

$$u_{all} = 1.1\sqrt{f_c'}$$

$$= 1.1\sqrt{3000} = 60.25 \text{ psi}$$

$\therefore u_d < u_{all}$

$\therefore$ the design is okay.

Bond Check:

$$u_{dev} = \frac{V_{max}}{\Sigma_0 jd} = \frac{V_{max}}{\frac{b}{spacing} \times \pi \times D \times j \times d}$$

$$= \frac{1898.67 + 257.45 + 569.60}{\frac{12}{7} \times \pi \times \frac{5}{8} \times j \times 5} = \cancel{17.58 \text{ psi}} = 186.16 \text{ psi}$$

$$u_{all} = \frac{1.7\sqrt{f_c'}}{D}$$

$$= \frac{1.7 \times \sqrt{3000}}{5/8} = 148.98 \text{ psi}$$

$\therefore u_{dev} < u_{all}$ $\therefore$ The design is okay.

Working diagram:

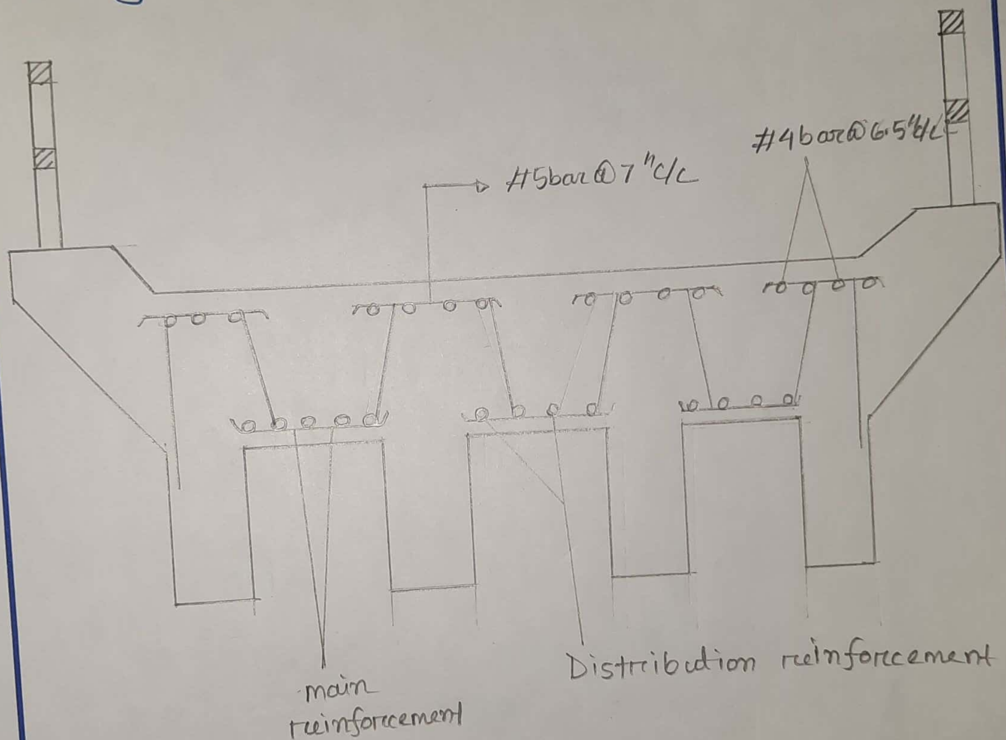

Figure: Typical reinforcement details of deckslab

Chapter No: 3

Chapter Name: Design of a Deck girder Bridge (Slab design)

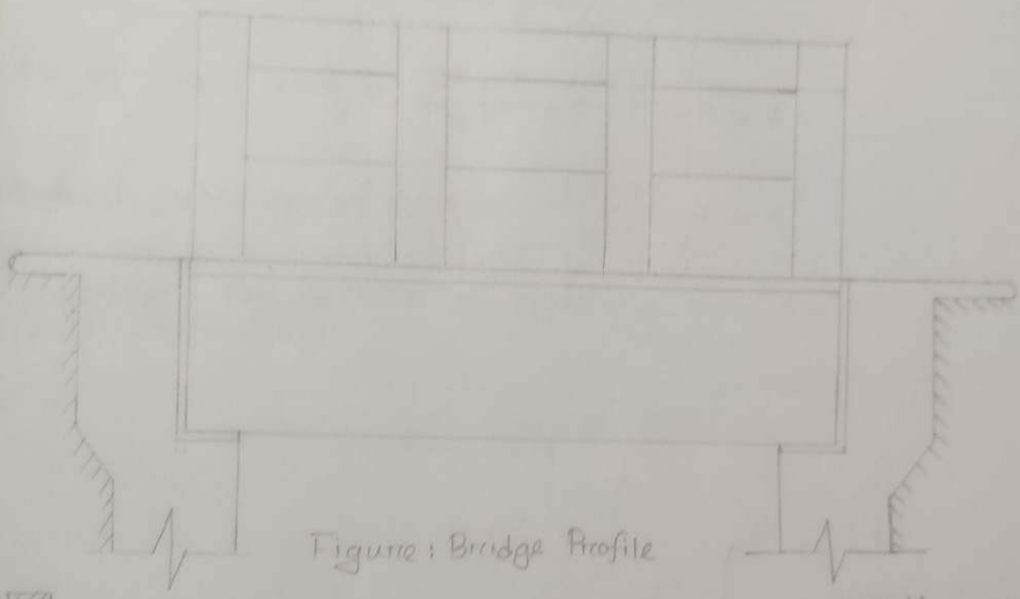

Figure: Bridge Profile

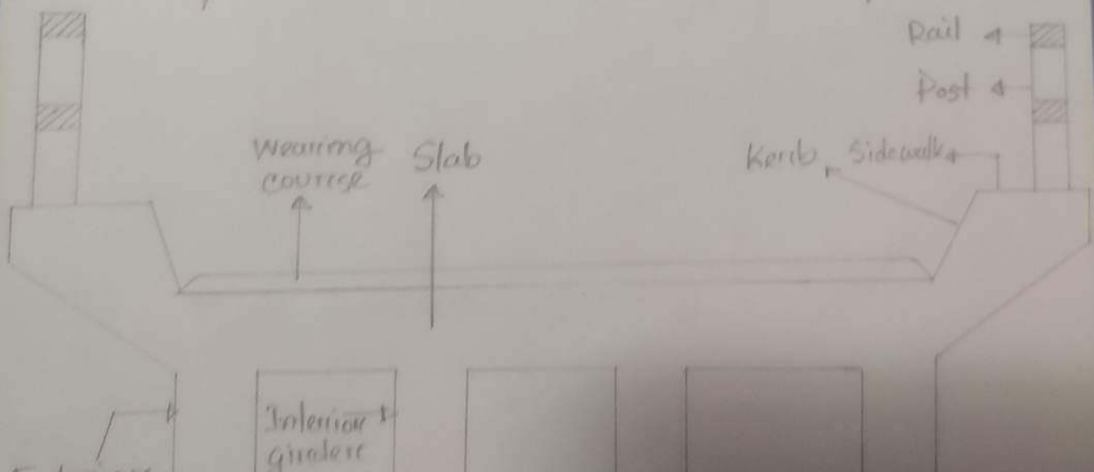

Figure: Tranverse section
Deck girder Bridge

Design of Railing:

Let the spacing of rail posts = 6.5'

No of rail post = $\frac{38.67}{6.5} + 1 = 6.9 \simeq 7$

Actual spacing of post = $\frac{38.67}{7-1} = 6.4'$

Assuming the section of railing = 6"×6"

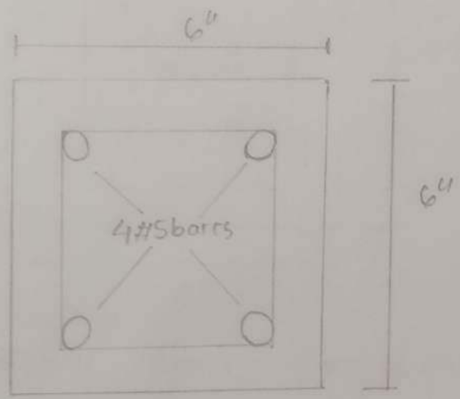

Figure: Cross section of railing

Design of rail post:

Assuming the section of rail post = 8" × 8"

Height of the post = 4'

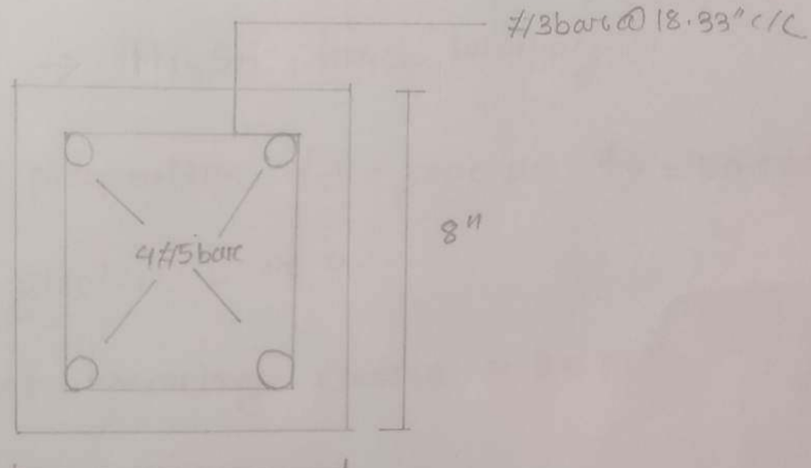

Figure: Cross section of post

Design Specifications:

i) Clear span length = last 2 digit of roll + $\frac{50}{3}$ = ~~38.~~

$= 22 + 50/3 = 38.67'$

ii) Width of the bridge = equivalent two lanes = 20'

iii) No of girder = 4

iv) Loading → $H_{15}S_{12}$ truck loading

v) Material properties, $f_c' = 3000$ psi, $f_y = 50{,}000$ psi

vi) width of girder = 15"

vii) Weight of wearing course = 20 psf

Design Calculations:

Center to center spacing of girder $= \frac{20}{4-1} = 6.67'$

Clear spacing between two girders $= \left(6.67 - \frac{15}{12}\right)$
$= 5.42'$

Relevent Properties:

$n = 9$, $r = 14.81$, $K = 0.38$, $j = 0.87$, $R = 223.16$ psi

Slab design:

Assuming the thickness of slab $= 6''$

Load Calculation:

Self weight of slab $= \frac{t}{12} \times 150 = \frac{6}{12} \times 150 = 75$ psf

weight of wearing course $= 20$ psf

Total Dead load $= 95$ psf

Dead load moment Calculation:

For continuity of slab,

Dedd load moment, $M_o = \frac{WL^2}{10}$

$= \frac{1}{10} \times 95 \times \cancel{(38.67)^2} (5.42)^2$

$= 279.08$ lb-ft

Live load moment Calculation:

For main rainforcement perpendicular to traffic

$= \frac{S+2}{32} \times P_{15}$

$= \frac{5.42+2}{32} \times 12000$

$= 2782.5$ lb-ft

Impact moment Calculation:

Impact factor, $I = \frac{50}{L+125} = \frac{50}{5.42+125} = 0.38$ should be <.30

$\therefore I = 0.3$

Impact moment = live moment $\times$ 0.38

$= 2782.5 \times 0.38 = 834.75$ lb-ft

∴ Design moment, $M_D = M_0 + M_{LL} + M_I$

$= 279.08 + 2782.5 + 834.75$

$= 3896.33$ lb-ft

Depth Check:

$$d_{req} = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{3896.3 \times 12}{223.16 \times 12}} = 4.18''$$

$d_{actual} = 6'' - 1 = 5''$

$d_{actual} > d_{req}$

∴ the design is okay.

Main reinforcement Calculation:

$$A_s = \frac{M}{f_s j d} = \frac{3896.3 \times 12}{20000 \times 0.87 \times 5} = 0.53 \text{ in}^2$$

$$\text{spacing} = \frac{0.31 \times 12}{0.53} = 7.02'' \simeq 7'' \text{ c/c}$$

using #5 bar @ 7" c/c

Distribution reinforcement: For main reinforcement perpendicular to traffic

$$A_{st} = \frac{220}{\sqrt{5}} \leq 67\% \text{ of main reinforcement}$$

$$= \frac{220}{\sqrt{5.42}} = 94.5 \leq 67\%$$

$$A_{st} = 0.67 \times 0.53 = 0.36 \text{ in}^2$$

using #4 bar @ $\frac{0.20 \times 12}{0.36} = 6.67 \simeq 6.5''$ c/c

Shear Check:

Dead load shear:

Developed shear, $V_{max} = \frac{WL}{2} = \frac{95 \times 5.42}{2} = 257.45$ lb/ft

Live load shear:

Live load on per unit width of slab $= \frac{P}{E}$

$$= \frac{12000}{4 + 0.06 \times 38.67}$$

$$= 1898.67 \text{ lb/ft}$$

Impact shear $= 1898.67 \times 0.3 = 569.60$ lb/ft

Shear Stress:

$$u_D = \frac{V_{max}}{bd} = \frac{257.45 + 1898.67 + 569.60}{12 \times 5 \times 12}$$

$$= 3.79 \text{ psi}$$

$$u_{all} = 1.1 \sqrt{f_c'}$$

$$= 1.1 \sqrt{3000} = 60.25 \text{ psi}$$

$u_d < u_{all}$

$\therefore$ the design is okay.

Bond Check:

$$u_{dev} = \frac{V_{max}}{\Sigma_0 jd} = \frac{V_{max}}{\frac{b}{spacing} \times \pi \times D \times j \times d}$$

$$= \frac{1898.67 + 257.45 + 569.60}{\frac{12}{7} \times \pi \times \frac{5}{8} \times j \times 5} = \cancel{17.58 \text{ psi}} = 186.16 \text{ psi}$$

$$u_{all} = \frac{1.7\sqrt{f_c'}}{D}$$

$$= \frac{1.7 \times \sqrt{3000}}{5/8} = 148.98 \text{ psi}$$

$\therefore u_{dev} < u_{all}$ $\therefore$ The design is okay.

Working diagram:

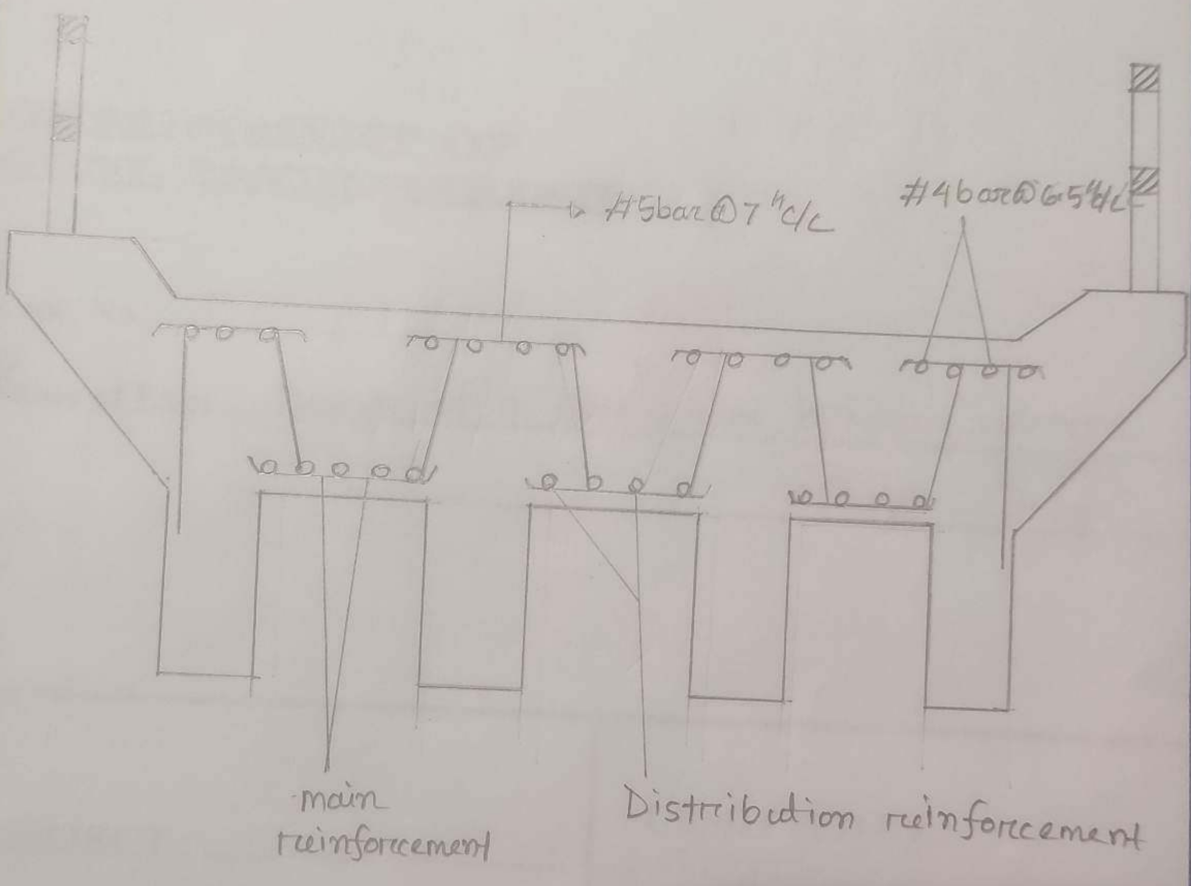

Figure: Typical reinforcement details of deckslab

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DEPARTMENT OF CIVIL ENGINEERING

Expt. No. ....

Name of Expt .... Design of a deck girder bridge (interior girder design)

SUBJECT :	SUBMITTED BY :
COURSE NO. :	NAME : ....
DATE OF EXPT.:	CLASS : ....
DATE OF SUB. :	GROUP :........ ROLL NO ....
	SESSION : ....

Chapter No: 3.2

Chapter Name: Design of deck girder bridge (Interior girder design)

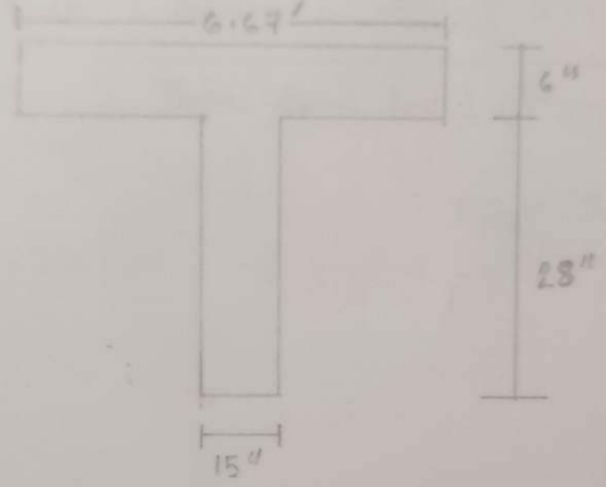


Fig: Typical Details of Interior girder

Effective span length = 38.67' + 2' = 40.67'

Thickness of flange = 6"

Height of the web = 28"

Width of the web = 15"

Load Calculation:

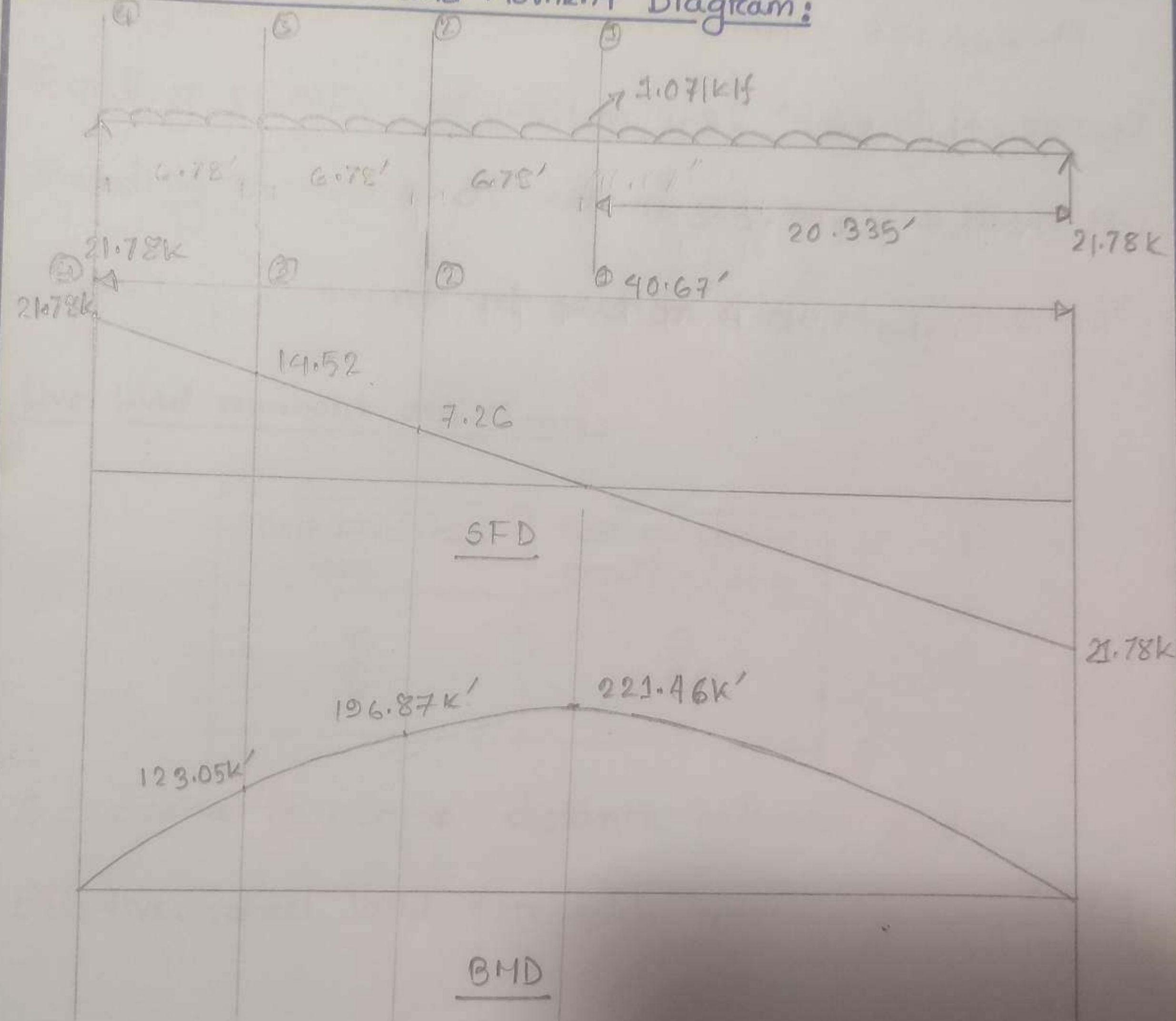
i) Weight of slab = $95 \times 6.67 = 633.65$ plf

ii) Self weight of T beam = $\frac{28 \times 15}{12 \times 12} \times 150 = 437.5$ plf

Total dead load, $W_D = 1071.15$ plf

$\therefore W_D = 1.07115$ klf

Dead Load Shear and Moment Diagram:



Shear force at section 1-1, $SF_{1-1} = 0k$

Shear force at section 2-2, $SF_{2-2} = 7.26k$

Shear force at section 3-3, $SF_{3-3} = 14.52k$

Shear force at section 4-4, $SF_{4-4} = 21.78k$

Bending moment at section 1-1, $M_{1-1} = 221.46 k\text{-}ft$

Bending moment at section 2-2, $M_{2-2} = 196.87 k\text{-}ft$

Bending moment at section 3-3, $M_{3-3} = 123.05 k\text{-}ft$

Bending moment at section 4-4, $M_{4-4} = 0 k\text{-}ft$

Live load moment and shear:

One traffic lane	two or more traffic lane
$\frac{S}{6}$	$\frac{S}{5}$

S = centre to centre distance between girder

Effective wheel load for each interior girder $= \frac{6.67}{5}$

$= 1.33k$

The load from front wheel $= 3 \times 1.33 = 4k$

The load from rare wheel $= 12 \times 1.33 = 15.96k$

Absolute Maximum Moment:

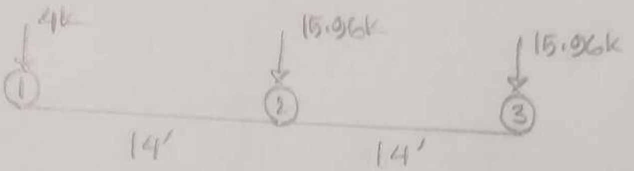


Fig : Determination of centre of gravity of wheel

$$\bar{x} = \frac{15.96\times14 + 4\times28}{4+15.96\times2} \simeq 9.34'$$

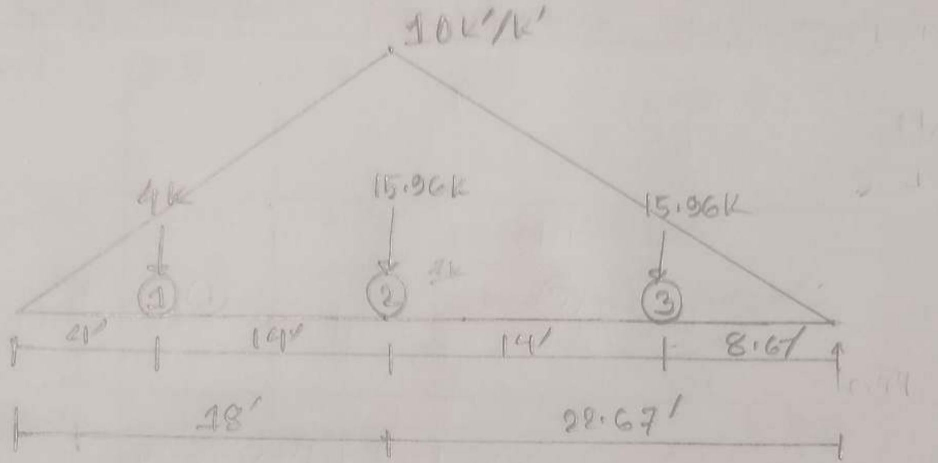
So, maximum absolute moment will occur due to wheel (2)

$$a = 14 - \bar{x} = 14 - 9.34 = 4.66'$$

Location of wheel $= \frac{L}{2} + \frac{a}{2} = \frac{40.67}{2} + \frac{4.66}{2}$

$= 22.67' \cancel{\simeq 23'}$

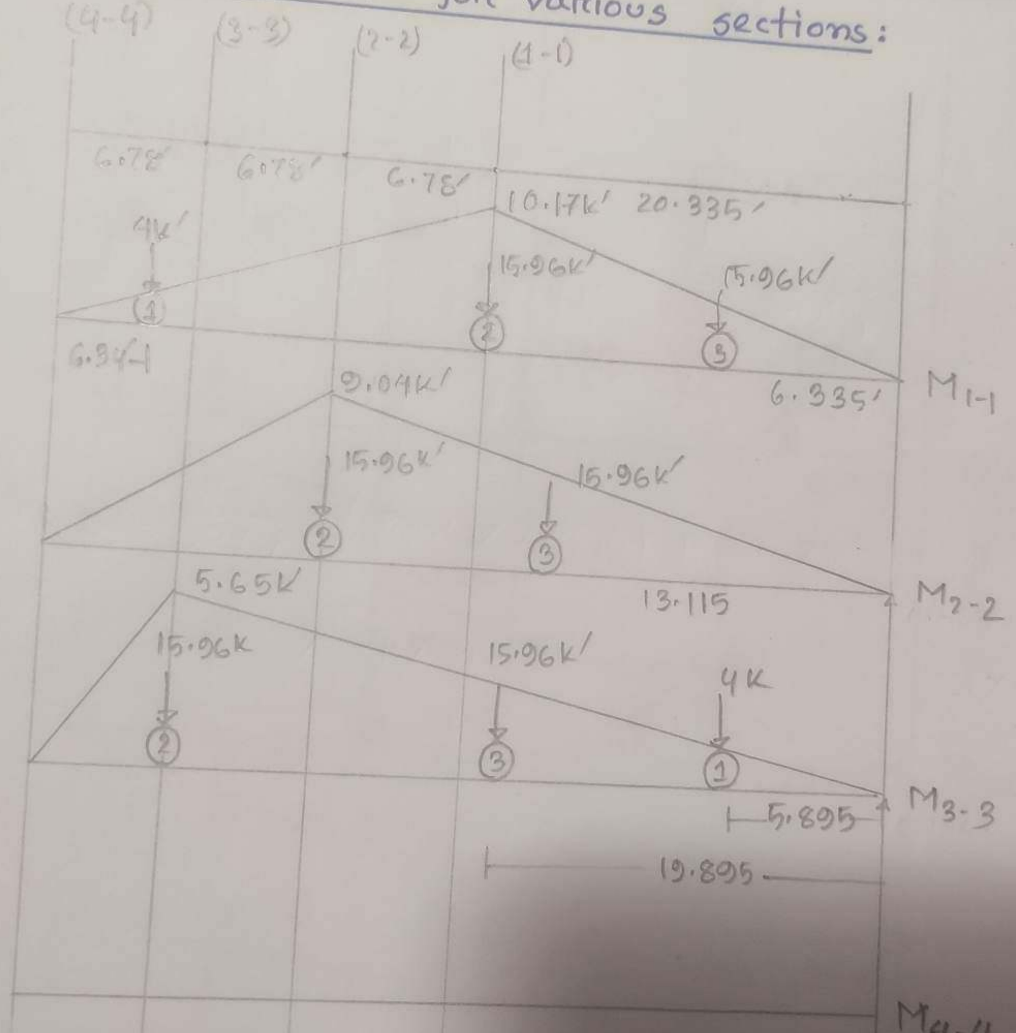
from the right support



$$\therefore M_{abs} = \frac{10}{22.67} \times (15.96 \times 22.67 + 15.96 \times 8.67) + \frac{10}{18} \times 4 \times 4$$

$$= 229.53 \text{ K}'$$

Live load moment for various sections:



$$M_{1-1} = \frac{10.17}{20.335}(15.96 \times 20.335 + 15.96 \times 6.335') + \frac{10.17}{20.335} \times 4 \times 6.34$$

$$= 225.56 \text{ k-ft}$$

$$M_{2-2} = \frac{9.04}{27.115}(15.96 \times 27.115 + 15.96 \times 13.115) = 214.06 \text{ k-ft}$$

$$M_{3-3} = \frac{5.65}{33.895}(15.96 \times 33.895 + 15.96 \times 19.895 + 4 \times 5.895)$$

$$= 147.033 \text{ k-ft}, \quad M_{4-4} = 0 \text{ k-ft}$$

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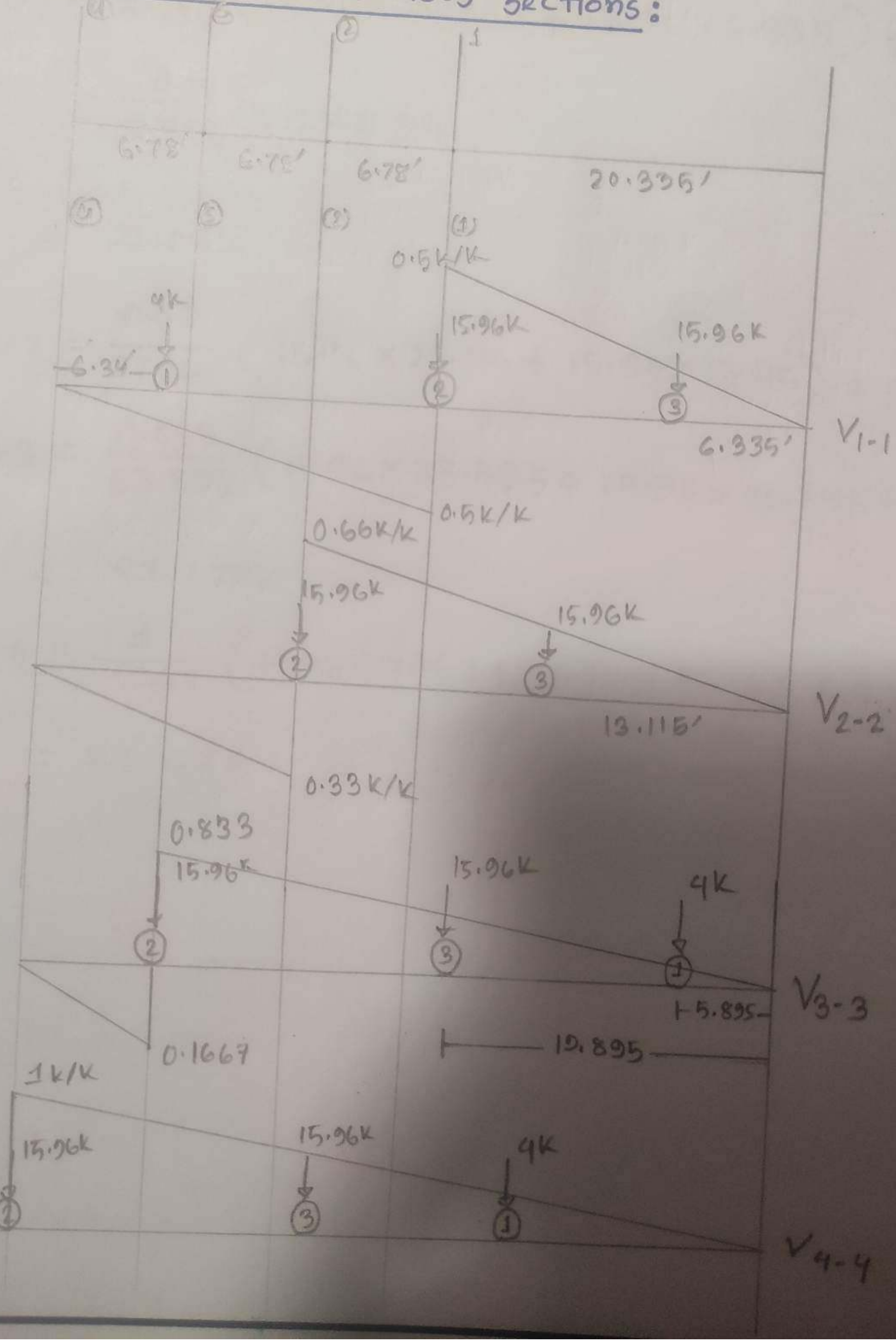
Live load Sheare of various sections:



$$V_{1-1} = \frac{0.5}{20.335}(15.96 \times 20.335 + 15.96 \times 6.335) + \frac{0.5}{20.335} \times 4 \times 6.34$$

$$= 11.09\text{k}$$

$$V_{2-2} = \frac{0.66}{27.115}(15.96 \times 27.115 + 15.96 \times 13.115) + = 15.63\text{k}$$

$$V_{3-3} = \frac{0.833}{33.895}(15.96 \times 33.895 + 15.96 \times 19.895 + 4 \times 5.895)$$

$$= 21.678\text{k}$$

$$V_{4-4} = \frac{1}{40.67}(15.96 \times 40.67 + 15.96 \times 26.67 + 4 \times 12.67)$$

$$= 27.67\text{k}$$

Impact Shear and Moment:

Impact coefficient $= \frac{50}{125+40.67} = 0.302$

$\therefore$ Impact moment for Section 1-1 $= 225.56 \times 0.302 = 68.12$ k'

Impact moment for section 2-2 $= 214.06 \times 0.302 = 64.65$ k'

Impact moment for section 3-3 $= 147.033 \times 0.302$

$= 44.40$ k'

Impact moment for section 4-4 $= 0$ k'

Impact absolute moment $= 229.53 \times 0.302 = 69.32$ k'

Impact shear for section 1-1 $= 11.09 \times 0.302 = 3.35$ k

Impact shear for section 2-2 $= 15.63 \times 0.302 = 4.72$ k

Impact shear for section 3-3 $= 21.678 \times 0.302 = 6.55$ k

Impact shear for section 4-4 $= 27.67 \times 0.302 = 8.36$ k

Table: : Moment Chart

Section	Dead load Moment (k')	Live Load Moment (k')	Impact load moment (k')	Total moment (k')	Design Moment (k')
1-1	221.46	225.56	68.12	515.14	
2-2	196.87	214.06	64.65	475.58	515.14
3-3	123.05	147.033	44.46	314.48	
4-4	0	0	0	0	

Table: : Shear Chart

Section	Dead load Shear (k)	Live load Shear (k)	Impact load Shear (k)	Total shear (k)	Design Shear
1-1	0	11.09	3.35	14.44	
2-2	7.26	15.63	4.72	27.61	57.81
3-3	14.52	21.678	6.55	42.75	
4-4	21.78	27.67	8.36	57.81	

Depth Check:

Maximum shear stress in the girder $= 0.06 f_c'$

$= 0.06 \times 3000 = 180$ psi

Now, $V_{max} = vjbd \therefore bd = \frac{V_{max}}{vj}$

$= \frac{57.81 \times 1000}{180 \times 0.87}$

$15 \times d = 369.16$

$\therefore d_{req} = 24.61''$

$\therefore d_{eff} = 34 - 2.5 - \frac{5}{8} - \frac{1}{2} \times \frac{11}{8}$

$= \cancel{28.875''}\ 30.1875''$

~~dactual~~ $d_{eff} > d_{req}$

$\therefore$ the design is okay.

Reinforcement Calculation:

Section 1-1:

$$A_{s_1} = \frac{M_1}{f_s\left(d - \frac{h_f}{2}\right)} = \frac{515.14 \times 12}{20\left(30.1875 - \frac{6}{2}\right)} = 11.37 \text{ in}^2$$

used 8 #11 bars in three layers.

Section 2-2:

$$A_{s_2} = \frac{M_2}{f_s\left(d - \frac{h_f}{2}\right)} = \frac{475.58 \times 12}{20\left(30.1875 - \frac{6}{2}\right)} = 10.5 \text{ in}^2$$

used 8 #11 bars

Section 3-3:

$$A_{s_3} = \frac{M_3}{f_s\left(d - \frac{h_f}{2}\right)} = \frac{314.48 \times 12}{20\left(30.1875 - \frac{6}{2}\right)} = 6.94 \text{ in}^2$$

used 5 #11 bars

Check for T beam:

Effective Flange width of T beam

i) $16h_f + b_w = 16 \times 6 + 15 = 111''$

ii) $L/4 = \frac{40.67 \times 12}{4} = 122.01''$

iii) c/c spacing between girder $= 6.67' \times 12$
$= 80.04''$

$\therefore$ Effective flange width, $b_f = 80.04''$

$$\rho = \frac{A_s}{bd} = \frac{11.37}{80.04 \times 30.1875} = 0.0047,$$

$$n = \frac{29 \times 10^6}{57000\sqrt{3000}} = 9.29$$

$\rho n = 0.044$

$$k = \frac{\rho n + \frac{1}{2}\left(\frac{h_f}{d}\right)^2}{\rho n + \frac{1}{2}\left(\frac{h_f}{d}\right)} = \frac{0.044 + \frac{1}{2} \times \left(\frac{6}{30.1875}\right)^2}{0.044 + \frac{1}{2} \times \left(\frac{6}{30.1875}\right)^2}$$

$= 0.44$

$kd = 13.42 > h_f$

$\therefore$ T beam is confirmed.

Web reinforcement:

Concrete resists a unit shear of $0.03f_c' = 0.03 \times 3000$
$= 90psi$

Shear developed at the end, $v_d = \frac{V}{b_w j d}$

$= \frac{57.81 \times 1000}{15 \times 0.87 \times 30.1875}$

$= 146.75$ psi

$\therefore v_d > 90psi$

$\therefore$ the stirrups will be provided.

Spacing, $s = \frac{A_v f_s}{(v_d - v_c) b_w} = \frac{2 \times 0.31 \times 0.4 \times 50000}{(146.75 - 90) \times 15}$

$= 14.57$ in

~~Second stirrup~~ ~~= s =~~

First stirrup will be placed from support

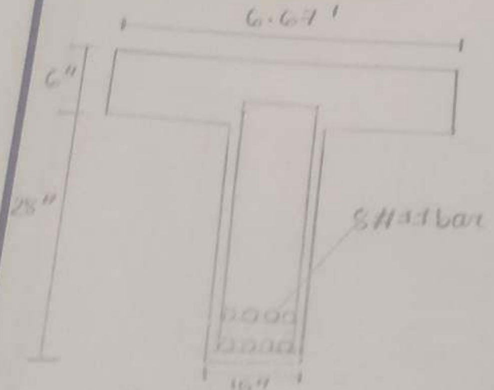
$= \frac{s}{2} = \frac{14.57}{2}$

$= 7.29''$

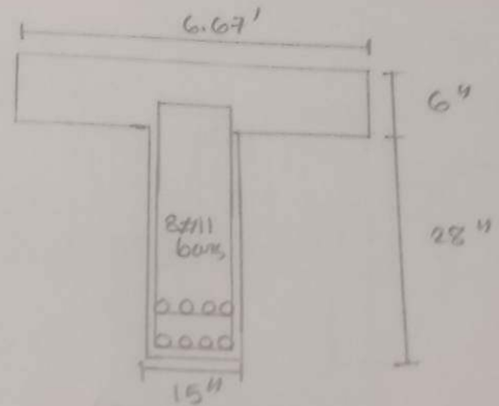
Second stirrup $= s = 14.57$ in

Third stirrup $= \frac{d}{2} = \frac{30.1875}{2} = 15.09''$

Working diagram:



Section 1-1



Section 2-2

Section 3-3

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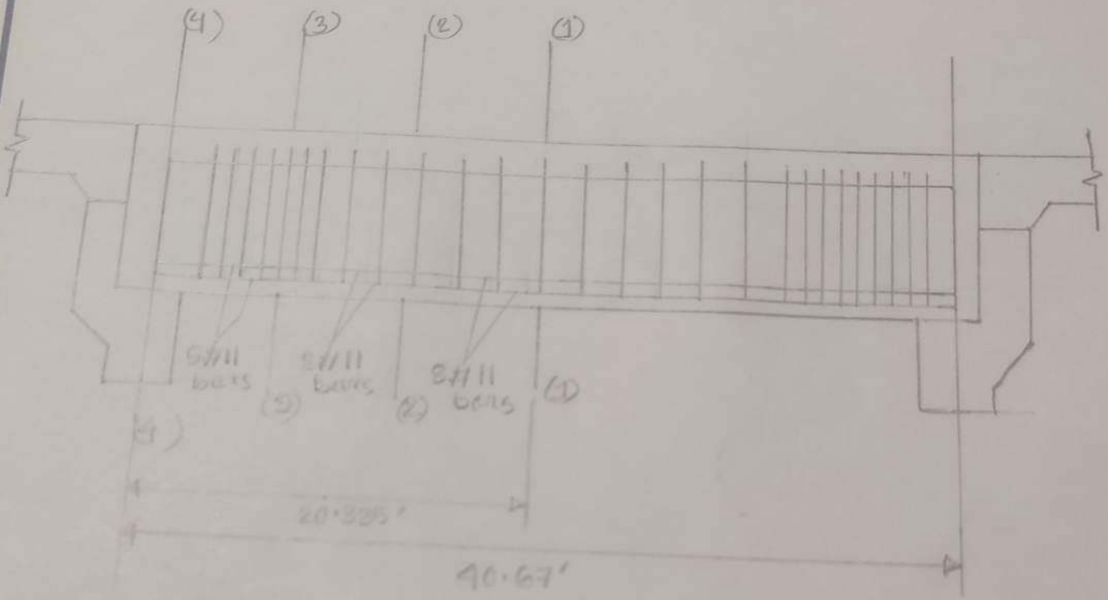


Fig: Reinforcement details of interior girder
(Longitudinal section)

Chapter No: 3.3

Chapter Name: Design of External Girder

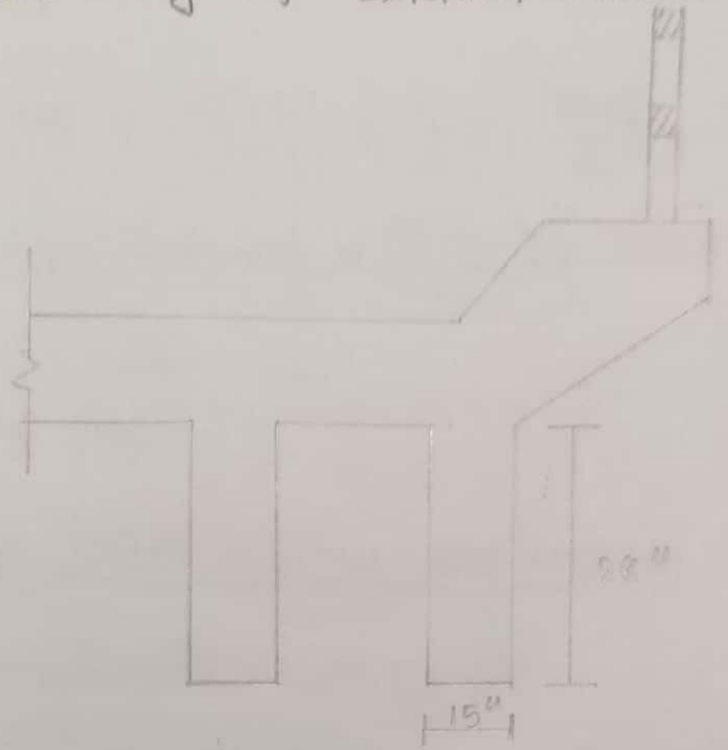


Fig: Cross section of girder

Load Calculation:

Load coming from slab, girder and sidewalk

$$= \left\{ \frac{6}{12} \times 3.335 + \frac{15 \times 28}{144} + \frac{1}{2} \times \frac{(13+6)}{12} \times \frac{7.5}{12} + 3.75 \times \frac{1}{2} \times \left(\frac{3+13}{12}\right) \right\} \times 150$$

$= 1136.84$ plf

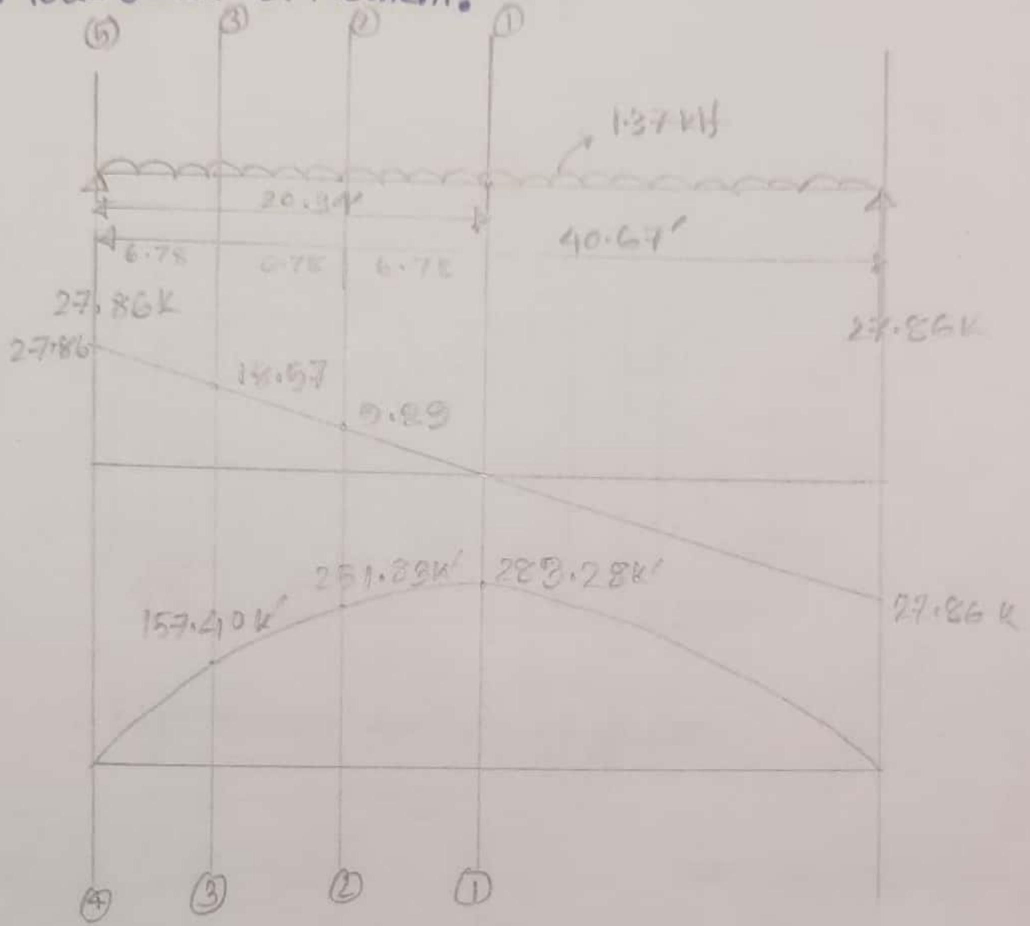
Load coming from railing and rail post:

Load from sidewalk $= 227.85$ plf

$\therefore$ Total load $= 1136.84 + 227.85 = \cancel{1757.66 \text{ plf}}\ 1364.69$ plf

$= 1.36$ klf

Dead load shear & Moment:



Bending moment of section 1-1 = 283.28k'

Bending moment of section 2-2 = 251.83k'

Bending moment of section 3-3 = 157.40k'

Bending moment of section 4-4 = 0k'

Shear at section (1-1) = 0k

Shear at section (2-2) = 9.29 k

Shear at section (3-3) = 18.57k

Shear at section (4-4) = 27.86k

Live load sheare and moment:

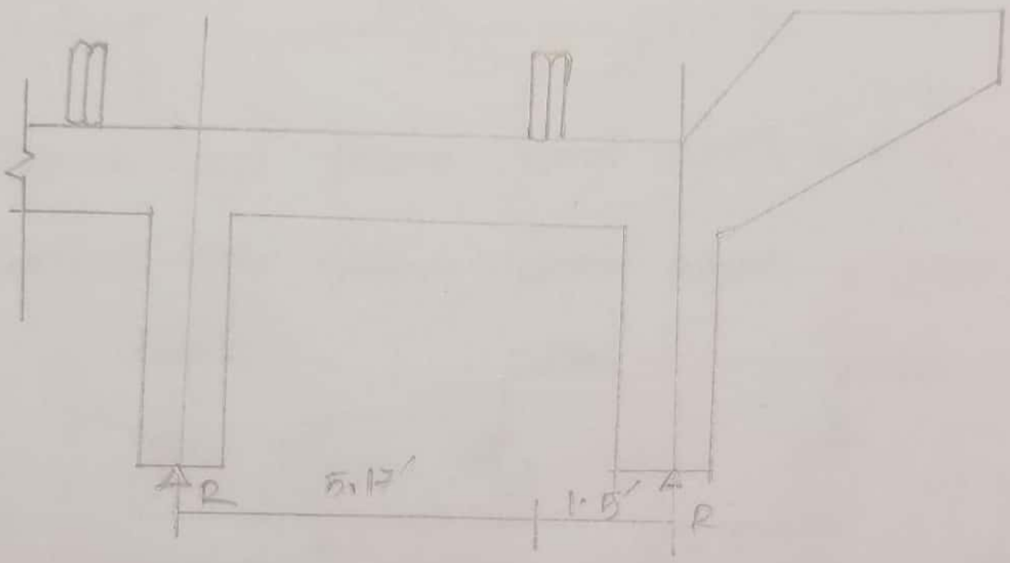


Fig: Cross section of girder

Effective portion of wheel load on exterior girder

$$R = \left(\frac{\frac{5.17}{6.67}}{1.33} \right) = 0.59$$

Effective load from rare wheel $= 0.59 \times 12 = 7.08k$

Effective load from front wheel $= 0.59 \times 3 = 1.77k$

$$\bar{x} = \frac{7.08 \times 14 + 1.77 \times 28}{1.77 + 7.08 \times 2} = 9.33$$

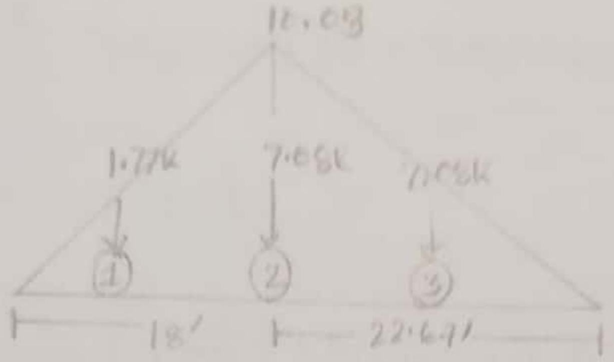
Maximum absolute moment will occur due to wheel(2)

$$a = 14 - 9.33 = 4.67' \therefore \frac{a}{2} = 2.33'$$

position of wheel(2) from right support $= \frac{x}{2} + \frac{a}{2}$

$$= \frac{40.67}{2} + \frac{\cancel{2.33}\ 4.67}{2}$$

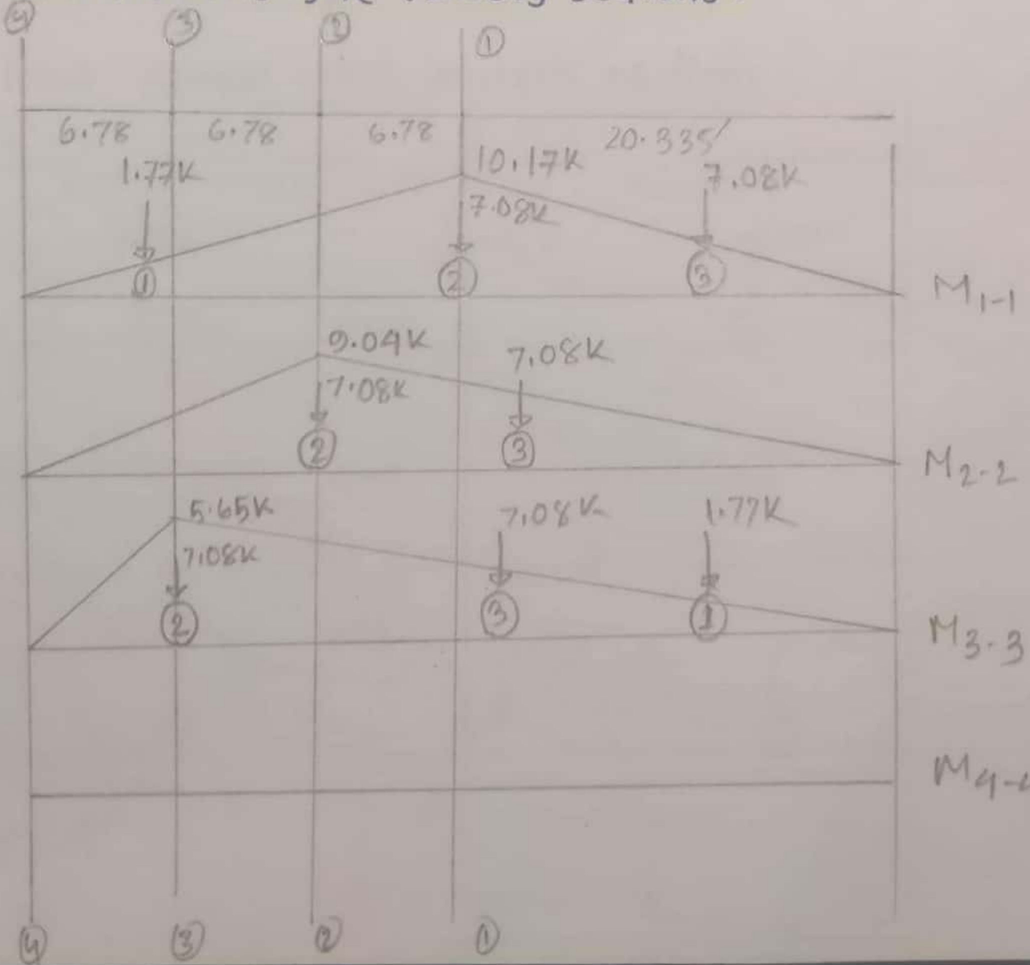
$$= \cancel{21.5}\ 22.67'$$



$$M_{abs} = \frac{10.03}{22.67}\left(7.08 \times 22.67 + 7.08 \times 8.67\right) + \frac{10.03}{18} \times (1.77 \times 4)$$

$$= 102.12 \text{ k-ft}$$

Live load moments for various sections:



$$M_{1-1} = \frac{10.17}{20.335}(7.08\times 20.335 + 7.08\times 6.335) + \frac{10.17}{20.335}\times 1.77\times 6.78$$

$$= 100.44 \text{ k-ft}$$

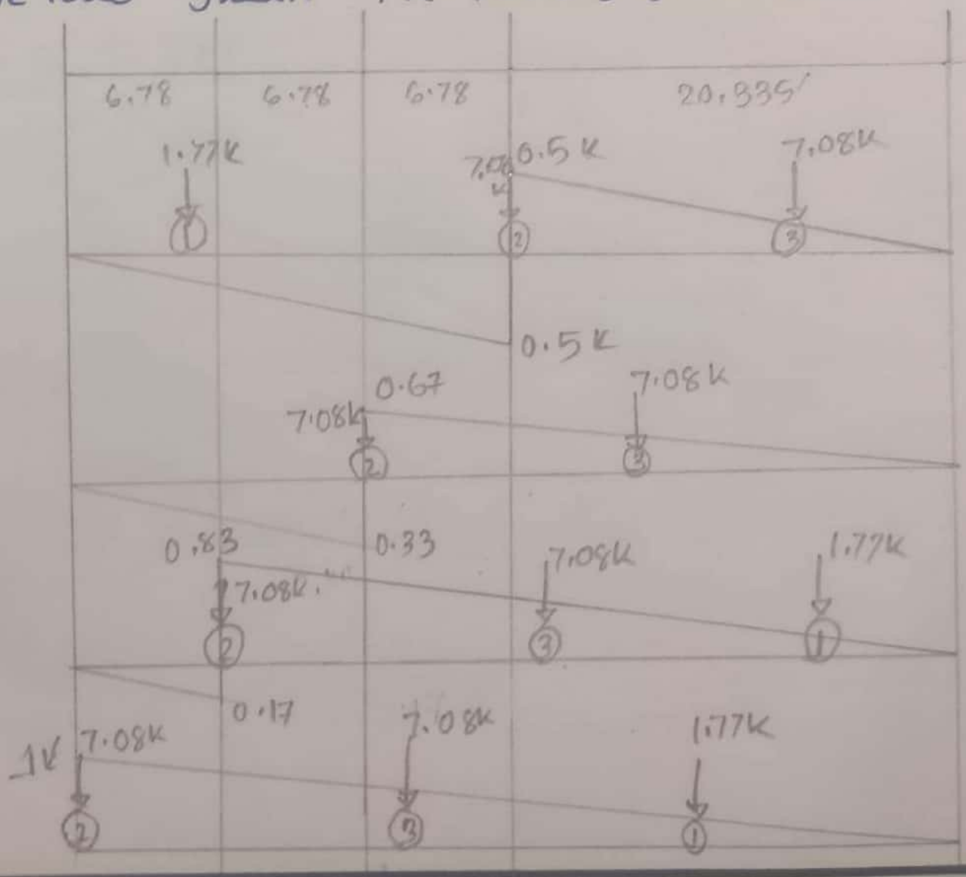
$$M_{2-2} = \frac{9.04}{27.12}(7.08\times 27.12 + 7.08\times 13.115) = 94.95\text{k-ft}$$

$$M_{3-3} = \frac{5.65}{33.90}(7.08\times 33.90 + 7.08\times 19.9 + 1.77\times 5.9)$$

$$= 65.22 \text{ k-ft}$$

$$M_{4-4} = 0$$

Live load Shear for various section:



$$V_{1-1} = \frac{0.5}{20.335}(7.08 \times 20.335 + 7.08 \times 6.34) - \frac{0.5}{20.335} \times 1.77 \times 6.335$$

$$= 4.37$$

$$V_{2-2} = \frac{0.67}{20.335}(7.08 \times \overset{27.12}{\cancel{20.335}} + 7.08 \times 13.12) = 9.39k$$

$$V_{3-3} = \frac{0.83}{33.895}(7.08 \times 33.895 + 7.08 \times 19.895 + 1.77 \times 5.895)$$

$$= 9.58k$$

$$V_{4-4} = \frac{1}{40.67}(7.08 \times 40.67 + 7.08 \times 26.67 + 1.77 \times 12.67)$$

$$= 12.27k$$

Impact shear and moment:

Impact coefficient $= \dfrac{50}{125 + 40.67} = 0.302$ [illegible]

$= 0.30$

Impact moment for sec ①-① $= 0.30 \times 100.44 = 30.132$

Impact moment for sec 2-2 $= 0.30 \times 94.95 = 28.49$

Impact moment for sec 3-3 $= 0.30 \times 65.22 = 19.57$

Impact moment for sec 4-4 $= 0.30 \times 0 = 0$

Impact absolute moment $= 0.30 \times 102.12 = 30.64$

Impact shear for sec (1-1) $= 0.30 \times 4.37 = 1.311$

Impact shear for sec 2-2 $= 0.30 \times 9.39 = 2.82$

Impact shear for sec (3-3) $= 0.30 \times \cancel{12.27}\ 9.58 = \cancel{3.68}\ 2.87$

Impact shear for sec (4-4) $= 0.30 \times 12.27 = 3.68$

Table: Moment Chart

Section	Dead load moment k-ft	Live load moment k-ft	Impact load moment k-ft	Total moment k-ft	Design moment k-ft
1-1	283.28	100.44	30.132	413.85	
2-2	251.83	94.95	28.49	375.27	
3-3	157.40	65.22	19.57	242.19	413.85
4-4	0k'	0k	0	0	
Absolute	279.54	102.12	30.64	412.3	

Table: Shear Chart

Section	Dead load shear (K)	live load shear (K)	Impact load shear (K)	Total shear (K)	Design shear (K)
1-1	0	4.37	1.31	5.68	
2-2	9.29	9.39	2.82	21.5	
3-3	18.57	9.58	2.87	31.83	43.81
4-4	27.86	12.27	3.68	43.81	

Determination of cross section: (Depth check)

Maximum shear stress in girder $= 0.06 f_c' = 0.06 \times 3000$
$= 180$ psi $= v$

$$bd = \frac{V}{vj} = \frac{43.81 \times 1000}{180 \times 0.87} = 279.76$$

$$\therefore d_{req} = \frac{279.76}{15} = 18.65''$$

$$d_{eff} = (28+6) - 2.5 - \frac{5}{8} - \frac{1.14}{2 \times 8} - 2 = \cancel{26.13}'' \; 28.1875''$$

$\therefore$ the design is okay.

Reinforcement Calculation:

$$A_{s1} = \frac{M_1}{f_s(d - \frac{h_f}{2})} = \frac{413.85 \times 12}{20(28.1875 - \frac{6}{2})} = 9.93 \text{ in}^2$$
using ~~7-#11 bar~~

$$A_{s2} = \frac{M_2}{f_s(d - \frac{h_f}{2})} = \frac{375.27 \times 12}{20(28.1875 - \frac{6}{2})} = 9.006 \text{ in}^2$$
using 7#11 bar

$$A_{s3} = \frac{M_3}{f_s(d - \frac{h_f}{2})} = \frac{242.19 \times 12}{20(28.1875 - \frac{6}{2})} = 5.81 \text{ in}^2$$
using 4#11 bar

$$A_{s4} = \frac{M_4}{f_s(d - \frac{h_f}{2})} = \frac{0}{f_s(d - \frac{h_f}{2})} = 0$$

$$A_s(abs) = \frac{M_{abs}}{f_s(d - \frac{h_f}{2})} = \frac{412.3 \times 12}{20\,(28.1875 - \frac{6}{2})} = 9.89$$

using 7 #11 bar

Confirmation of L Beam:

Effective width of compression flange, $b = 6h_f + b_w$

$= 6 \times 6 + 15$

$= 51''$

$\therefore b = \frac{L}{12} + b_w = \frac{40.67 \times 12}{12} + 15$

$= 55.67''$

$\therefore b = 51''$

$\therefore b = \frac{S}{2} + b_w = \frac{6.67 \times 12}{2} + 15$

$= 55.02$

$$\therefore \rho = \frac{A_s}{bd} = \frac{\cancel{10}8.26}{51 \times 28.1875} = 0.007 \quad [n = 9]$$

$$\rho n = 0.007 \times 9 = 0.063$$

$$k = \frac{n\rho + \frac{1}{2}(t/d)^2}{n\rho + (t/d)^2} = \cancel{0.25}\ 0.31 \quad \therefore kd = 8.68'' > h_f$$

$\therefore$ L-beam is confirmed section

$$f_c = \frac{M}{(1 - \frac{h_f}{2kd})\,btjd} = \frac{413.85 \times 12}{(1 - \frac{6}{2 \times 8.68}) \times 51 \times 6 \times 0.87 \times 28.1875}$$

$= 1.36\ ksi > 1.35\ ksi$

Web reinforcement:

Maximum stress carried by concrete, $V_c = 0.03f_c'$

$= 0.03 \times 3000$

$= 90psi = 0.09 ksi$

Shear developed at the end, $V_d = \frac{V}{b_w j d} = \frac{43.81 \times 1000}{15 \times 0.87 \times 28.1875}$

$= 128.47 psi$

$V_d > V_c$

∴ Stirrups will be provided.

Spacing, $S = \frac{A_v f_s}{(V_d - V_c) b_w} = \frac{2 \times 0.31 \times 20000}{(128.47 - 90) \times 15}$

$S_{max} = \frac{28.1875}{2} = 14.09 in = 11.5'' c/c$

$= 14.09 in$

Sheare carried by steel for maximum spacing

$V' = \frac{A_v f_s}{S_{max} \times b_w} = \frac{2 \times 0.31 \times 0.4 \times 50000}{13.07 \times 15} = 63.25 psi$

Distance upto which stirrup is required for

$S_{max} = \frac{V_d - 0.5V_c}{V_d} \times \frac{L}{2} = \frac{128.47 - 0.5 \times 90}{128.47} \times \frac{40.67}{2}$

$= 13.21''$ (from midspan

Distance upto which stirrup is required,

$$S = \frac{V_d - V_c}{V_d} \times \frac{L}{2} = \frac{128.47 - 90}{128.47} \times \frac{40.67}{2}$$

$$= 6.09'$$

Minimum stirrup required from $\frac{S}{2} = \frac{6.67}{2}$

$$= 3.34'$$

to $6.09'$

WORKING DIAGRAM OF E

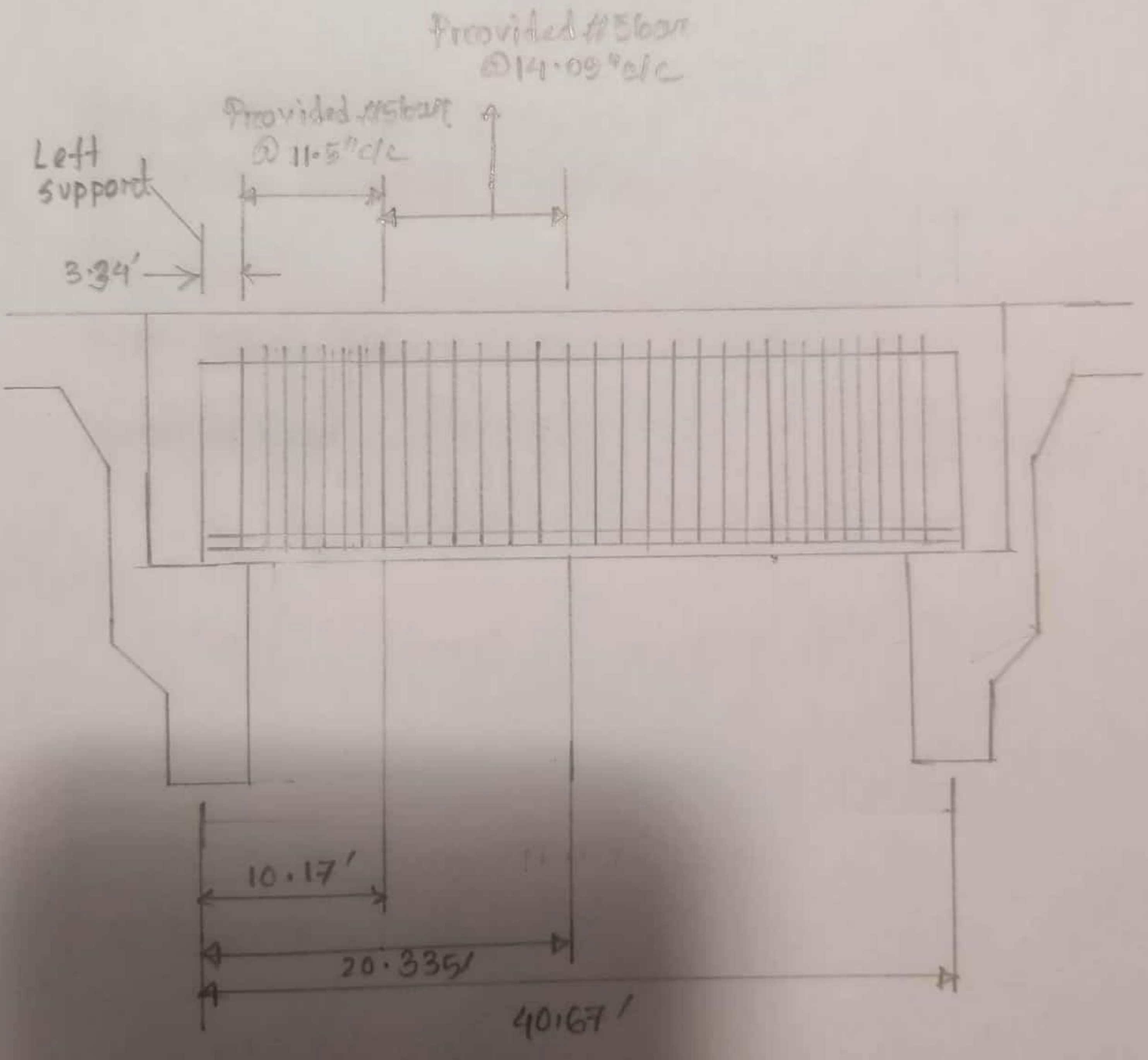
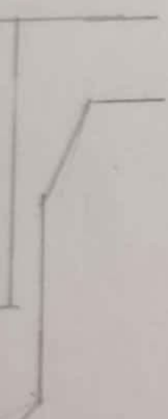


Fig : Reinforcement Details of Exterior
Girder (Longitudinal section)

OF EXTERIOR GIRDER



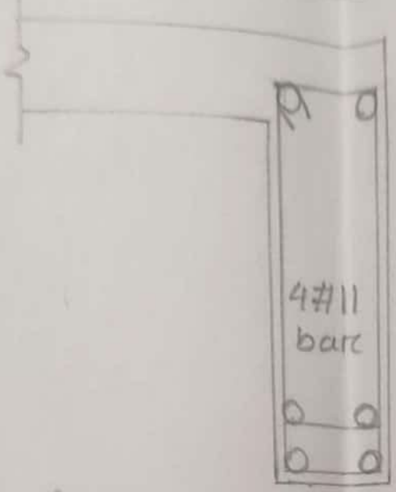


Fig: Reinforcement details in transverse section of exterior gireder at section 3-3

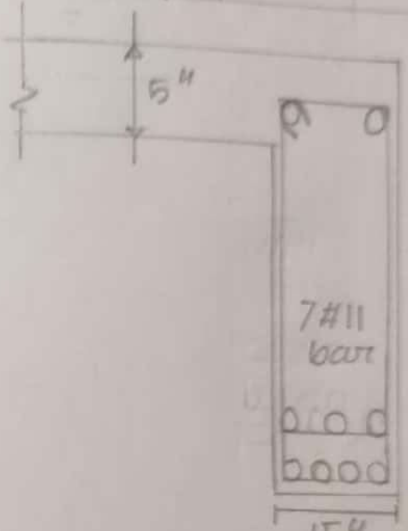


Fig: Reinforcement details in transverse section of Exterior girder at section 1-1

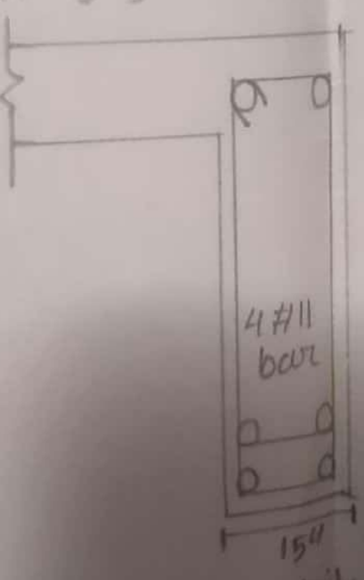


Fig: Reinforcement details at section 4-4

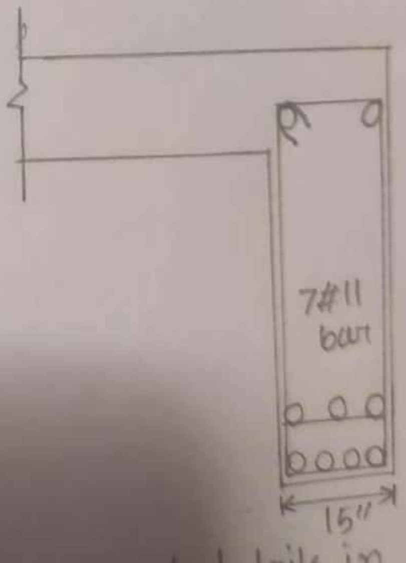


Fig: Reinforcement details in transverse section of Exterior girder at section 2-2

WORKING DIAGRAM OF EXTERIOR GIRDER

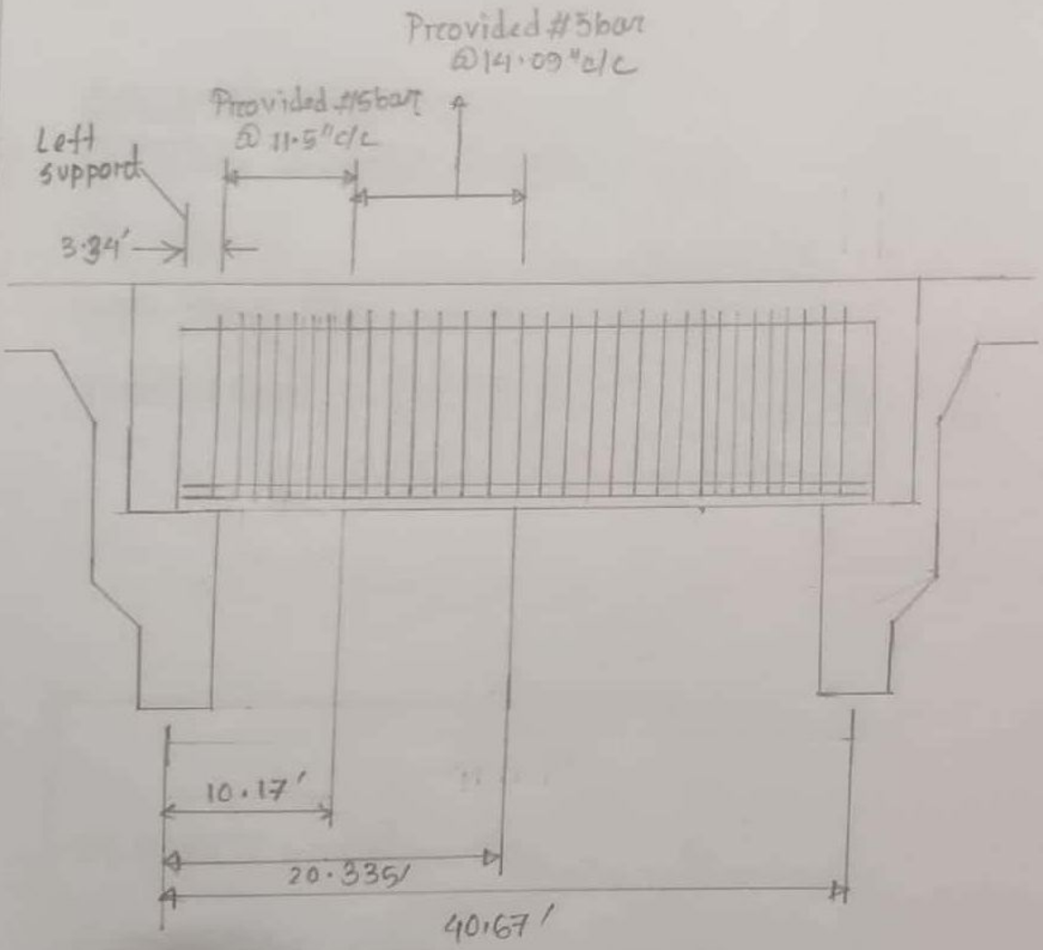


Fig : Reinforcement Details of Exteriore Girdere (Longitudinal section)

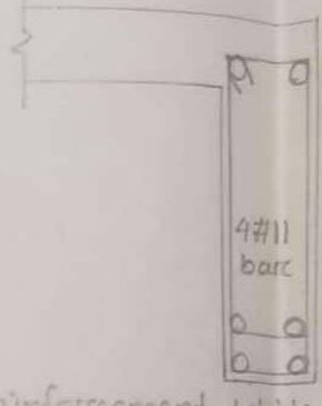


Fig: Reinforcement details in transverse section of exteriore girder at section 3-3

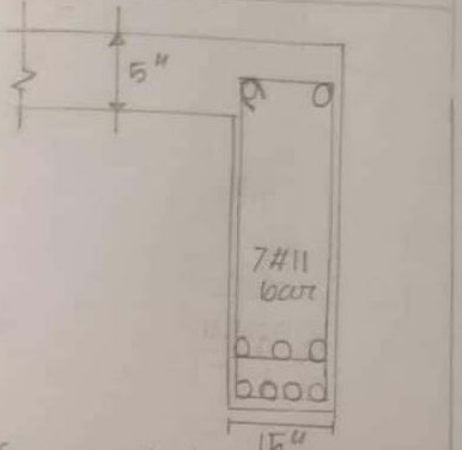


Fig: Reinforcement details in transverse section of Exteriore girdere at section 1-1

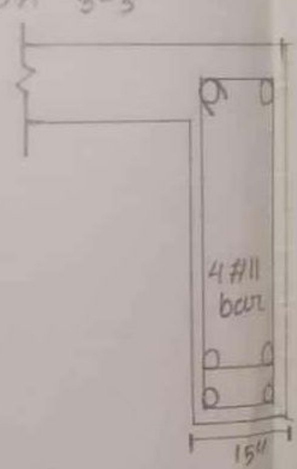


Fig: Reinforcement details at section 4-4

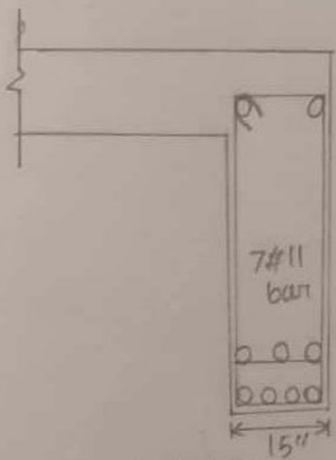


Fig : Reinforcement details in transverse section of Exteriore girdere at section 2-2