Heaven's Light is Our Guide

Rajshahi University of Engineering & Technology
Rajshahi

DEPARTMENT OF CIVIL ENGINEERING

Expt. No. 02

Name of Expt Design of slab bridge

SUBJECT : Reinforced concrete sessional-II	**SUBMITTED BY :**
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DATE OF SUB. :	**GROUP :** .... **ROLL NO** 1800147
	SESSION : 2018-2019

Chapter No - 02

Chapter Name: Design of slab bridge.

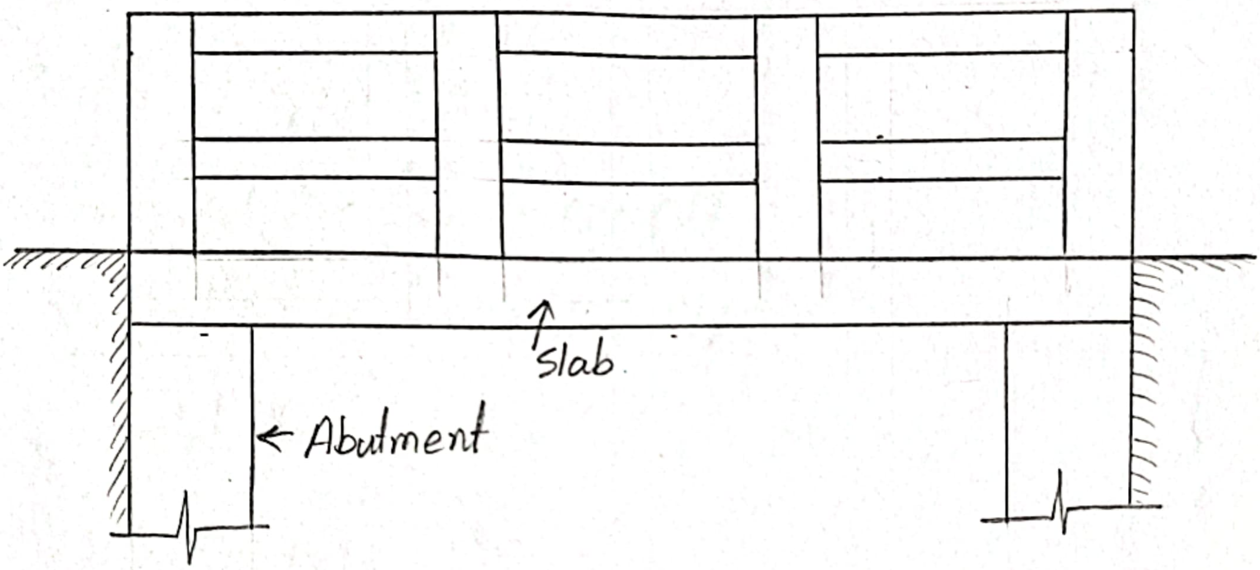


Fig. 2.1: Elevation of slab Bridge

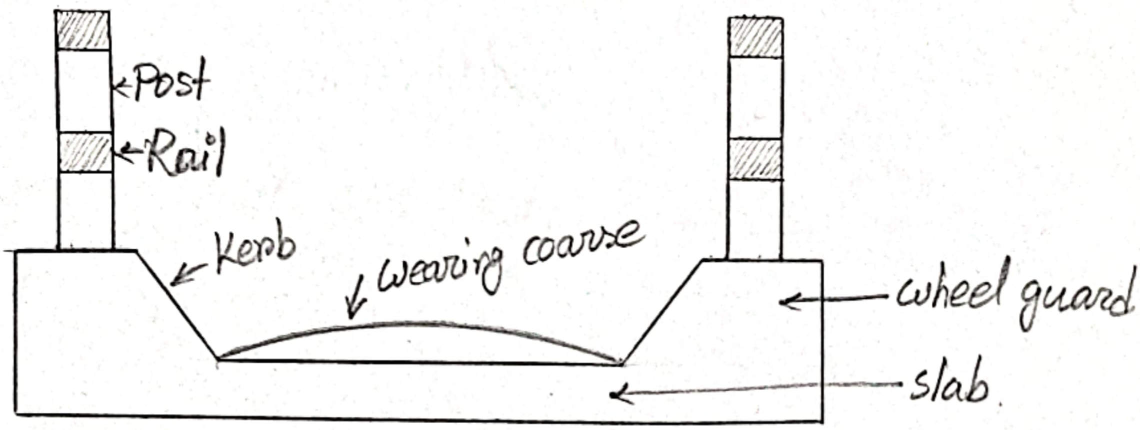


Fig. 2.2: Cross-section of slab Bridge

Specification:

i. length of clear span, $L = 8 + \frac{\text{last 2 digit of roll}}{4}$

$= 8 + \frac{47}{4}$

$= 19.75$ ft.

ii. Width of span = 30 ft.

iii. Horizontal live load = 200 plf

iv. Vertical live load = 150 plf.

v. Horizontal thrust due to wheel = 500 plf.

vi. $f_c' = 3500$ psi

vii. $f_y = 60000$ psi

viii. Live load H_{20}-S_{16} truck load.

2.1 : Design of rail and Post

Relevent Properties-

(i) $n = \frac{E_s}{E_c} = \frac{29 \times 10^6}{57000\sqrt{3500}} = 8.6 \approx 9$

(ii) $r = \frac{f_s}{f_c} = \frac{0.4 \times 60000}{0.45 \times 3500} = 15.24$.

(iii) $K = \frac{n}{n+r} = \frac{9}{9+15.24} = 0.37$

(iv) $j = 1 - \frac{K}{3} = 1 - \frac{0.37}{3} = 0.88$

(v) $R = \frac{1}{2} f_c j K = \frac{1}{2} \times 0.45 \times 3500 \times 0.37 \times 0.88 = 256.41$

Let,

(a) The spacing of rail post, $s = 6.5'$.

(b) Number of rail post $= \frac{\text{span length}}{\text{Spacing}} + 1 = \frac{19.75}{6.5} + 1$

$= 4.038 \simeq 4$.

(c) Actual spacing, $S_a = \frac{\text{span length}}{\text{Post number} - 1} = \frac{19.75}{4-1}$

$= 6.58'$.

2.1.1 Design of rail

Let us assume the section of railing $= 6'' \times 6''$

Load calculation:

(i) Self weight $= \frac{6 \times 6}{12 \times 12} \times 150 = 37.5$ Plf.

(ii) verticle load $= 150$ Plf

∴ Total load $W = 187.5$ Plf.

∴ vertical moment, $M = \frac{W \times S_a^2}{8}$

$= \frac{187.5 \times 6.58^2}{8}$

$= 1014.76$ lb-ft.

Depth check:

We know, $d_{req} = \sqrt{\frac{M}{Rb}}$

$= \sqrt{\frac{1014.76 \times 12}{256.41 \times 6}}$

$= 2.81''$

$d_{effective} = t - cc - \frac{\phi}{2}$

$= 6 - 1.5 - 1$

$= 3.5''$

Here $d_{effective} > d_{req}$ ∴ Design is ok.

Steel Calculation:

required steel area, $A_s = \frac{M}{f_s j d_{eff}}$

$$= \frac{1014.76 \times 12}{24000 \times 0.88 \times 3.5}$$

$$= 0.1647 \text{ in}^2$$

Minimum reinforcement, $A_{s\,min} = \frac{3\sqrt{f_c'}}{f_y} bd$

$$= \frac{3\sqrt{3500}}{60000} \times 6 \times 3.5$$

$$= 0.062 \text{ in}^2$$

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

$$= \frac{200}{60000} \times 6 \times 3.5$$

$$= 0.07 \text{ in}^2$$

$\therefore$ Required steel area, $A_s = 0.1647 \text{ in}^2$.

So we provide 4 #4 bar.

Working diagram:

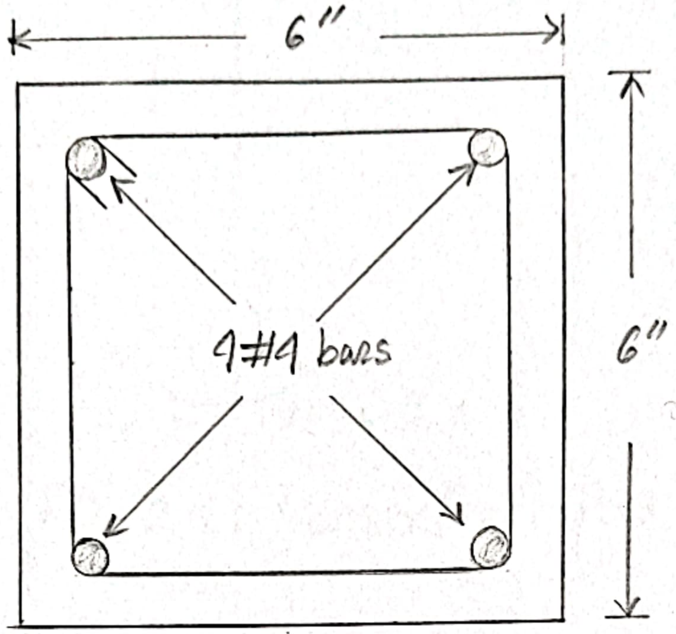


Fig. 2.3: Cross-section of Railing.

2.1.2 Design of Post:

Let us assume the section of the post = 8"×8"

Height of the post = 4'.

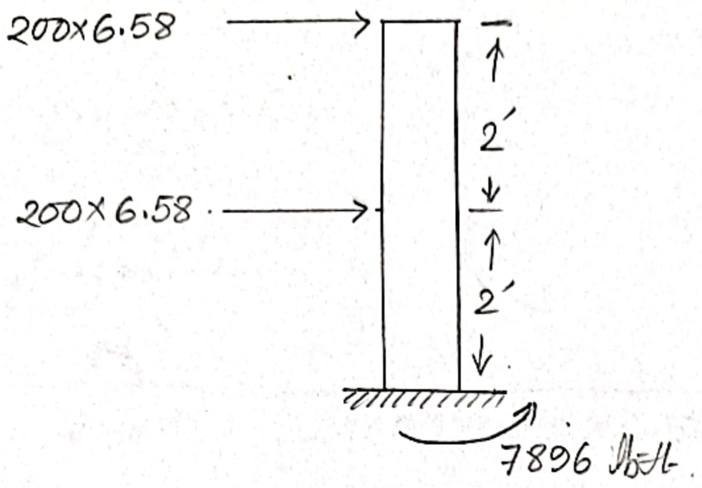


Fig. 2.4: Moment of Post.

Maximum horizontal moment at base, $M = (200 \times 6.58 \times 2) + (200 \times 6.58 \times 4)$

$= 7896$ lb-ft.

Depth Check:

$$d_{req} = \sqrt{\frac{M}{Rb}}$$

$$= \sqrt{\frac{7896 \times 12}{256.41 \times 8}}$$

$$= 6.8''$$

$t_{eff} = 6.8 + 1.5 = 8.3''$

$t_{assume} = 8''$

$t_{eff} \approx t_{assume}$

∴ Design is ok.

Steel calculation:

$$A_s = \frac{M}{f_s j d} = \frac{7896 \times 12}{24000 \times 0.88 \times 6.5}$$

$$= 0.69 \ in^2$$

We provide 4 #4 bars.

Shear check:

$$V_{dev} = (200 \times 6.58) + (200 \times 6.58)$$

$$= 2632 \ lb.$$

$$V_{allow} = 1.1 \sqrt{f_c'} \ bd$$

$$= 1.1 \sqrt{3500} \times 8 \times 6.5$$

$$= 3383.99 \ lb.$$

Here, $V_{allow} > V_{dev}$. ∴ Design is ok.

Stirrup calculation:

We provide #3 bars for stirrup.

Spacing, $S = \frac{A_v}{0.0015b} = \frac{2 \times 0.11}{0.0015 \times 8}$

$= 18.33$ in.

again $S = 3t = 3 \times 8$
$= 24$ in.

∴ Minimum spacing = 18.33 in.

Working Diagram:

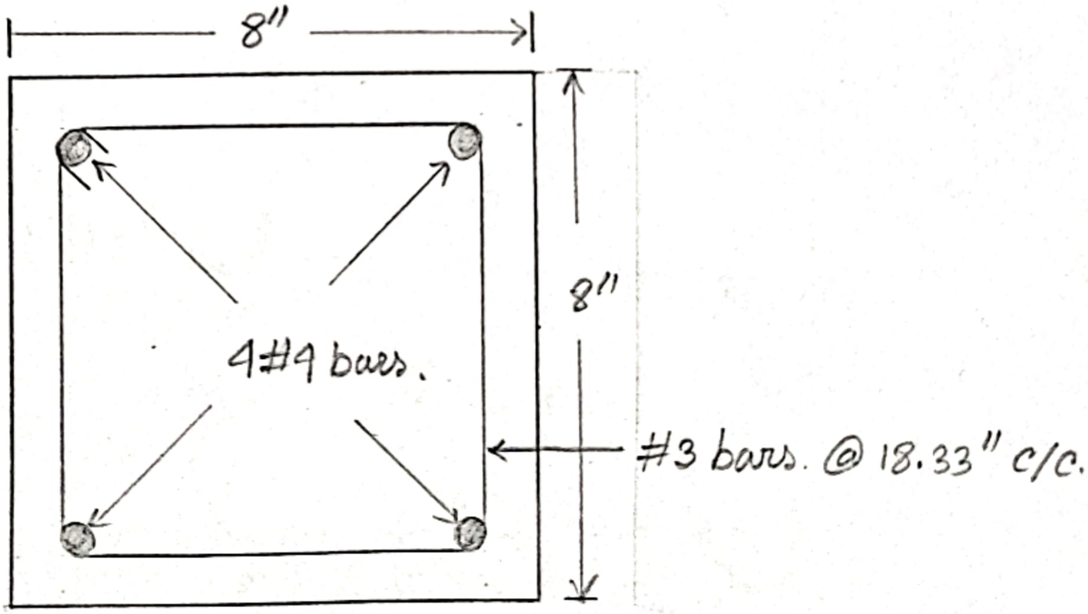
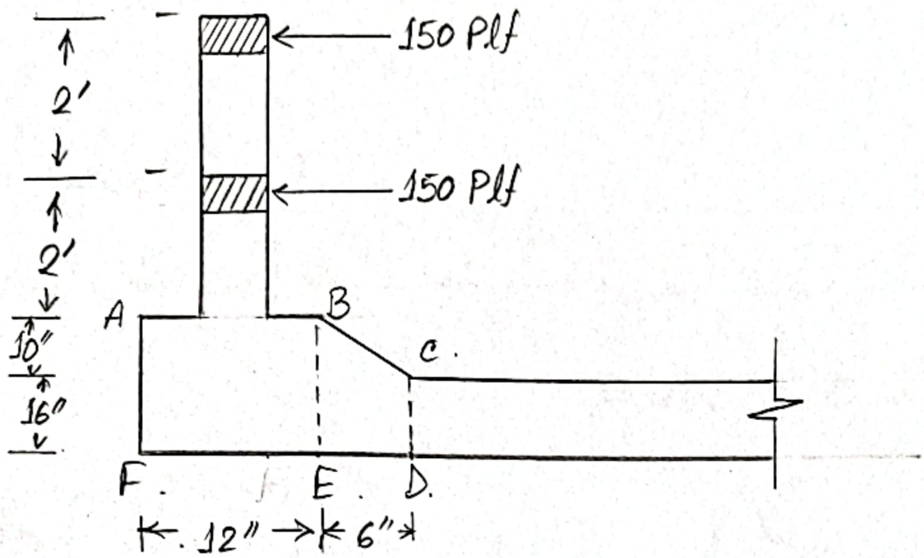


Fig.2.5: Cross section of Post

2.2 Design of Edge Beam:



Load calculation:

Dead load of ABEF Part $= \frac{26 \times 12}{12 \times 12} \times 150$

$= 325$ lb.

Dead load of BCDE Part $= \left\{ \left(\frac{\frac{26}{12} + \frac{16}{12}}{2} \right) \times \frac{6}{12} \right\} \times 150$

$= 131.25$ lb.

Dead load from rail and post $= \left\{ \frac{\frac{2 \times 6 \times 6}{144} \times 19.75 + \frac{8 \times 8}{144} \times 4 \times 4}{19.75} \right\} \times 150$

$= 129$ lb.

∴ Total dead load = 325 + 131.25 + 129
= 585.25 lb.

Moment Calculation:

Dead load Moment, $M_D = \frac{wL^2}{8} = \frac{585.25 \times (19.75)^2}{8}$
$= 28535.5$ lb-ft.

Live load Moment, $M_L = 0.1 \times H_{20-16} \times$ span length.
$= 0.1 \times 16000 \times 19.75$
$= 31600$ lb-ft.

∴ Total Moment, $M_T = 28535.5 + 31600$
$= 60135.5$ lb-ft.

Depth check:

$d_{req.} = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{60135.5 \times 12}{256.4 \times 12}}$
$= 15.3''$

$$d_{eff} = 26 - cc - \phi - \frac{6}{8\times2}$$

$$= 26 - 1.5 - \frac{4}{8} - \frac{6}{8\times2}$$

$$= 23.6''$$

$\because d_{eff} > d_{req}.$

$\therefore$ Design is ok.

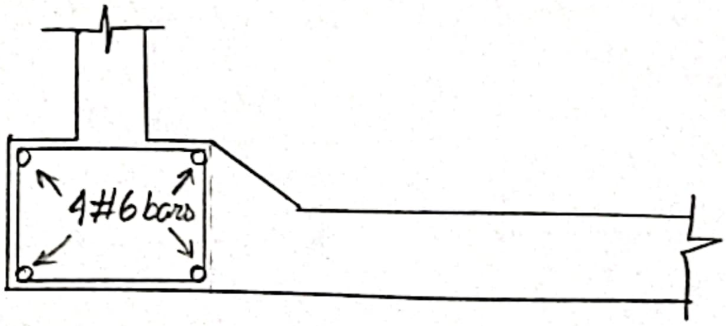
Steel calculation:

$$A_s = \frac{M}{f_s\, j\, d_{eff}} = \frac{60135.5\times12}{24000\times0.88\times23.6}$$

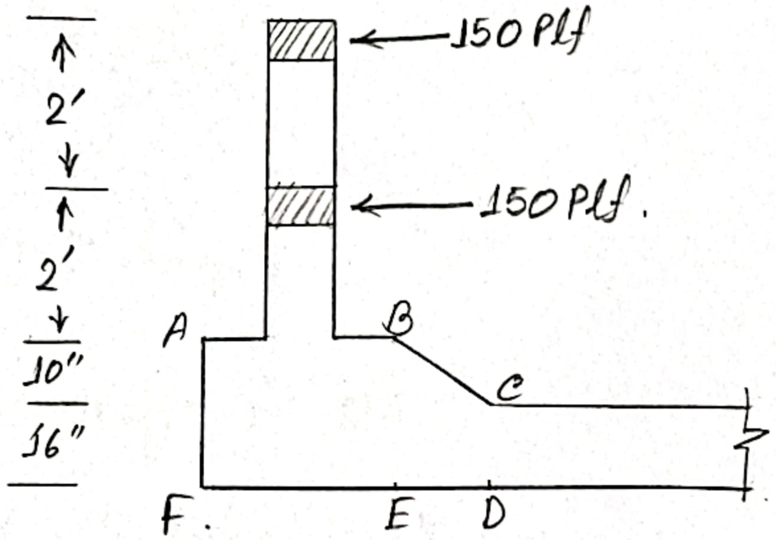
$$= 1.45\ in^2.$$

We provide 4 #6 bars.

Working Diagram:



2.3 Design of Kerb:



Critical Moment calculation,

$\Sigma M_c = 0$

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

$\Rightarrow M = 2634$ lb-ft.

depth check:

$d_{req} = \sqrt{\frac{2634 \times 12}{256.4 \times 12}}$

$= 3.2''$

$d_{effective} = 16 - 2.25$

$= 13.75''$

$\therefore d_{eff} > d_{req}$.

$\therefore$ The design is ok.

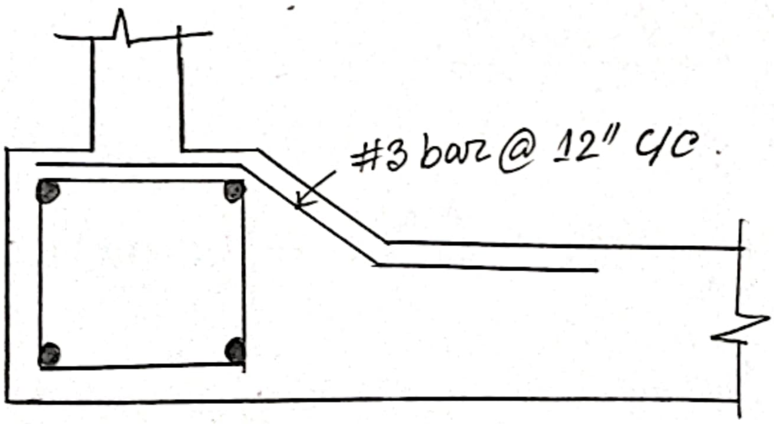
Steel calculation,

$$A_s = \frac{M}{f_s J d_{eff}} = \frac{2634 \times 12}{24000 \times 0.88 \times 13.75}$$

$$= 0.1 \text{ in}^2.$$

$\therefore$ Spacing is 12''.

Working diagram :



2.4 : Design of Slab-

(i) Span length = 19.75 ft.

(ii) Let us assume, thickness of slab = 16".

a. Dead load calculation :

i. self weight of slab $= \frac{16}{12} \times 150$

$= 200 \text{Psf}$.

ii. Weight of wearing coarse = 20 Psf.

∴ Total dead load = 200 + 20 = 220 Psf.

∴ Dead load shear $= \frac{wL}{2} = \frac{220 \times 1 \times 19.75}{2 \times 1000}$

$= 2.17 \text{ K}$

∴ Dead load Moment $= \frac{wL^2}{8} = \frac{220 \times 1 \times 19.75^2}{8 \times 1000}$

$= 10.75 \text{ K-ft}$.

Live load Analysis :

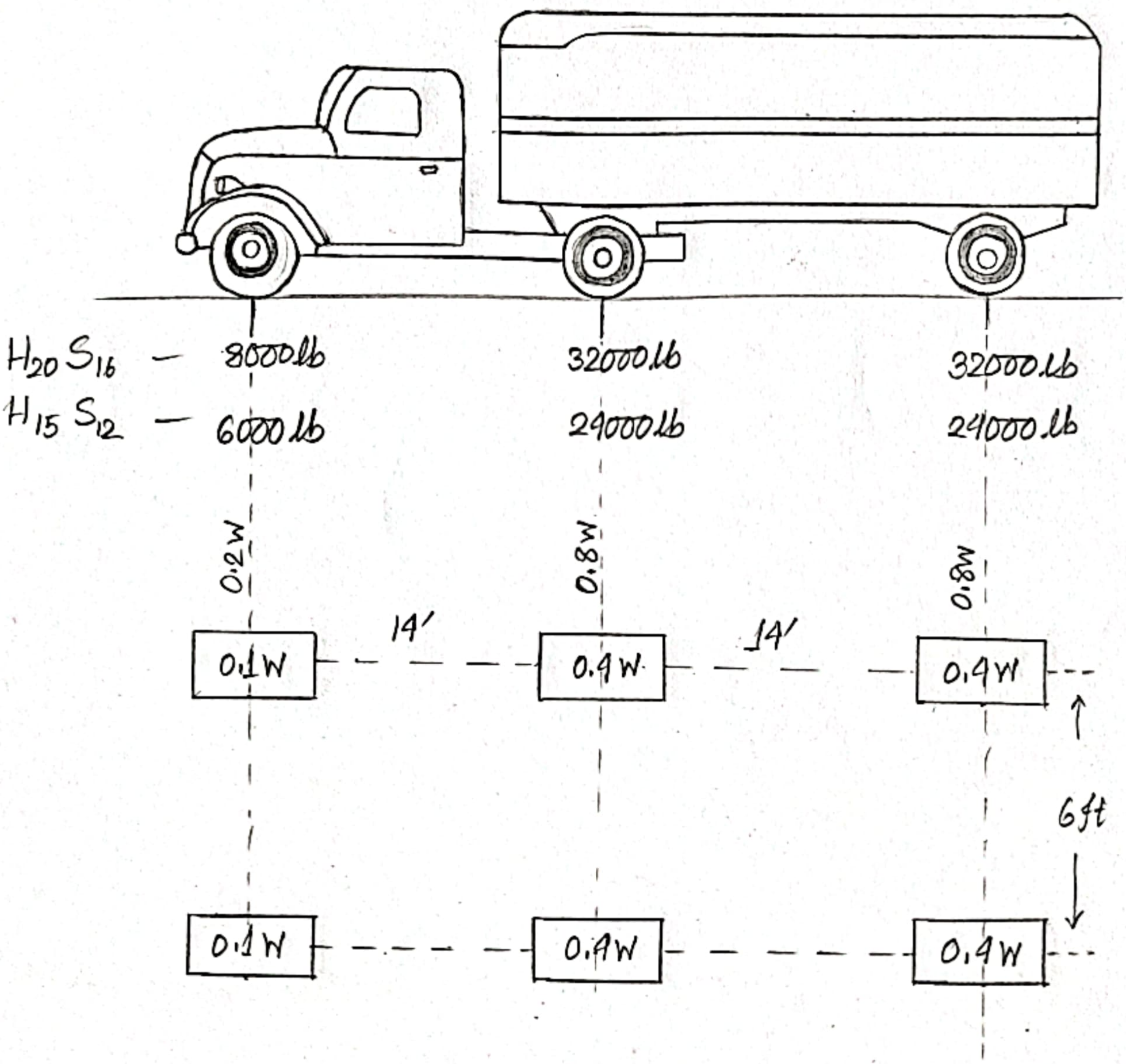


Fig. Load distribution of $H_{20}S_{16}$ and $H_{15}S_{12}$ truck.

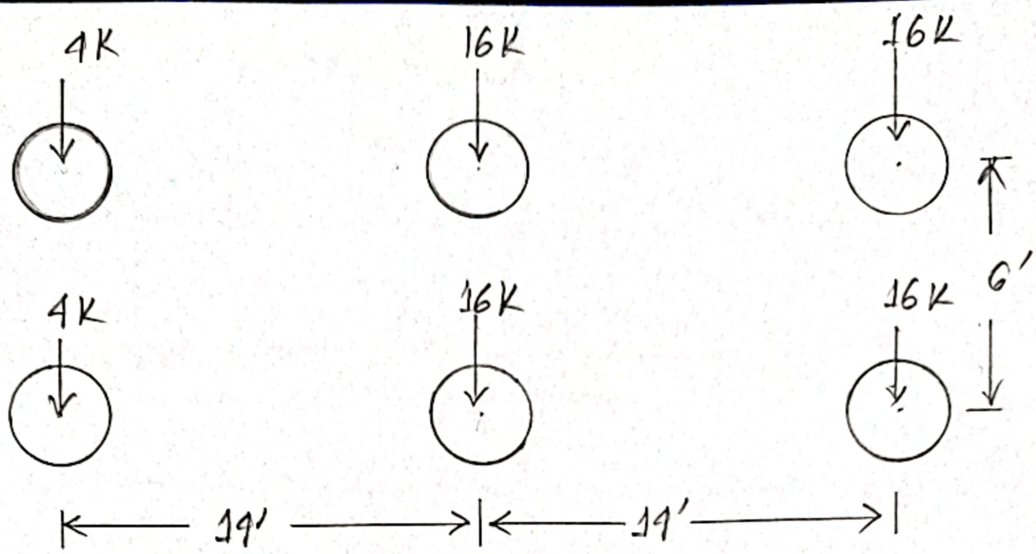
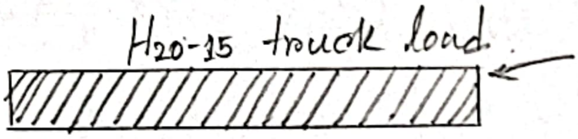


Fig:



Uniformly Distributed load = 640 lb/ft
Shear = 18000 lb.
Moment = 26000 lb.

Fig.

As our span length is 10.75 ft which is below 60 ft. Wheel load will be applicable here.

Moment calculation [Live load].

Case-I: Main reinforcement perpendicular to traffic

$$\text{Live load Moment} = \frac{S+2}{32} P_{20}$$

[Here
S = span length
$P_{20} = H_{20}$]

and S_{max} = Clear distance + truck upto slab.

$$\text{distribution reinforcement (\%)} = \frac{220}{\sqrt{S}} \% \quad [\text{max } 67\%].$$

Case-II: Main reinforcement parallel to the traffic.

Distribution of wheel load, $E = 4 + 0.06S \leq 7$

If $S \leq 50'$, then live load moment $= 900S$ lbft

If $50 < S < 100$, then live load moment $= 1000(1.35S - 20)$

$$\text{Distribution reinforcement (\%)} = \frac{100}{\sqrt{S}} \% \quad [\text{Max } 50\%].$$

☐ We will apply case-II in this design.

Main Reinforcement Parallel to the traffic.

Distribution wheel load, $E = 4 + 0.06 \times 19.75 \leq 7.$

$= 5.19 \leq 7.$

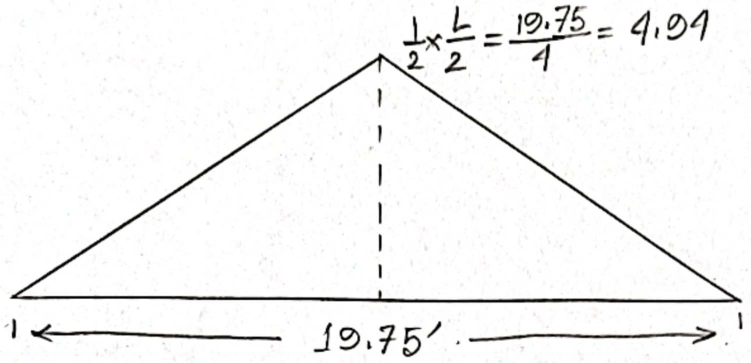


Fig.

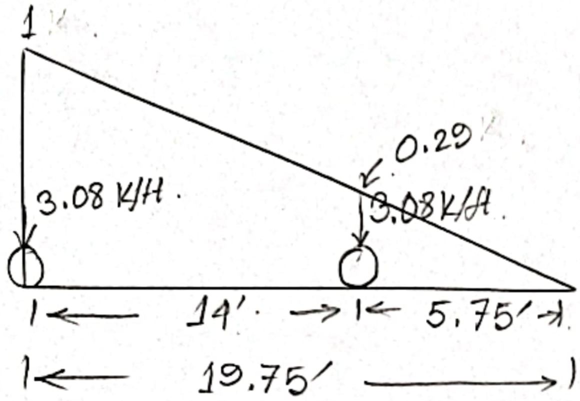
Per unit width of slab $= \frac{P}{E} = \frac{16}{5.19}$

$= 3.08$ K/ft.

Live Load moment $= \frac{P}{E} \times \frac{L}{4} = 3.08 \times 4.94$

$= 15.22$ K/ft

Calculation: of Shear:



Maximum live load shear = $(1 \times 3.08) + (0.29 \times 3.08)$

$= 3.97$ Kips.

Impact Load Calculation:

Impact coefficient $\Rightarrow$ $IC = \dfrac{50}{L+125} \leq 30\%$

$= \dfrac{50}{19.75+125}$

$= 34.5\% \approx 30\%$

$\therefore IC = 0.3$

∴ Impact Moment = LLM × IC = 15.22 × 0.3
= 4.57 K-ft.

∴ Impact shear = LLS × IC = 3.97 × 0.3
= 1.2 Kips.

Table: 2.1:

Dead load shear (K)	Live load shear (K)	Impact shear (K)	Design shear (K)
2.17	3.97	1.2	7.34

Table 2.2:

Dead Load Moment (K-ft)	Live Load Moment (K-ft)	Impact moment (K-ft)	Design moment (K-ft)
10.75	15.22	4.57	30.54

Depth check:

$$d_{req} = \sqrt{\frac{M}{R'b}} = \sqrt{\frac{30.54 \times 12000}{256.4 \times 12}}$$

$$= 10.91 \approx 11''$$

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

here $d_{eff} > d_{req}$

∴ The design is ok.

Reinforcement calculation:

$$A_s = \frac{M}{f_s j d_{eff}} = \frac{30.54 \times 12000}{24000 \times 0.88 \times 15}$$
$$= 1.17 \text{ in}^2.$$

We Provide #5 bar, spacing $= \frac{0.31 \times 12}{1.17} = 3.18'' \approx 3''$ c/c.

We provide #5 bar @ 3" c/c.

Distribution Reinforcement:

For main reinforcement parallel to traffic (%) $= \frac{100}{\sqrt{S}}$
$$= \frac{100}{\sqrt{19.75}}$$
$$= 22.5\%.$$

Now, distribution reinforcement $= 1.17 \times 0.225$
$= 0.27\ in^2$.

we use #4 bars, spacing $= \frac{0.2 \times 12}{0.27} = 8.88'' \approx 9''$ c/c.

We provide #4 bars @ 9" c/c.

Shear check:

Developed shear, $V_{dev} = 7.34$ kip $= 7340$ lb.

Allowable shear, $V_{all} = 1.1 \sqrt{f_c'}\ b\,d_{eff}$.

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

$= 11713.83$ lb.

here $V_{dev} < V_{all}$.

∴ Design is ok and No shear reinforcement is required.

Now, distribution reinforcement $= 1.17 \times 0.225$
$= 0.27\ in^2$.

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We provide #4 bars @ 9" c/c.

Shear check:

Developed shear, $V_{dev} = 7.34$ kip $= 7340$ lb.

Allowable shear, $V_{all} = 1.1 \sqrt{f_c'}\, b\, d_{eff}$.

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

$= 11713.83$ lb.

here $V_{dev} < V_{all}$.

∴ Design is ok and No shear reinforcement is required.

2.4. Design of Abutment wall.

Theory:

The forces acting on abutment walls are-

i. Lateral earth pressure.

ii. Surcharge

iii. Wheel load.

There are two types of failure mode in abutment wall. They are - i. External stability.

ii. Internal stability.

- External stability includes sliding force, total active pressure on abutment wall, resisting force, total friction force acting on concrete, soil at the base of the abutment.

- $\text{Factor of safety} = \frac{\text{resisting force}}{\text{sliding force}} > 1.5$

Resisting stress on wall.

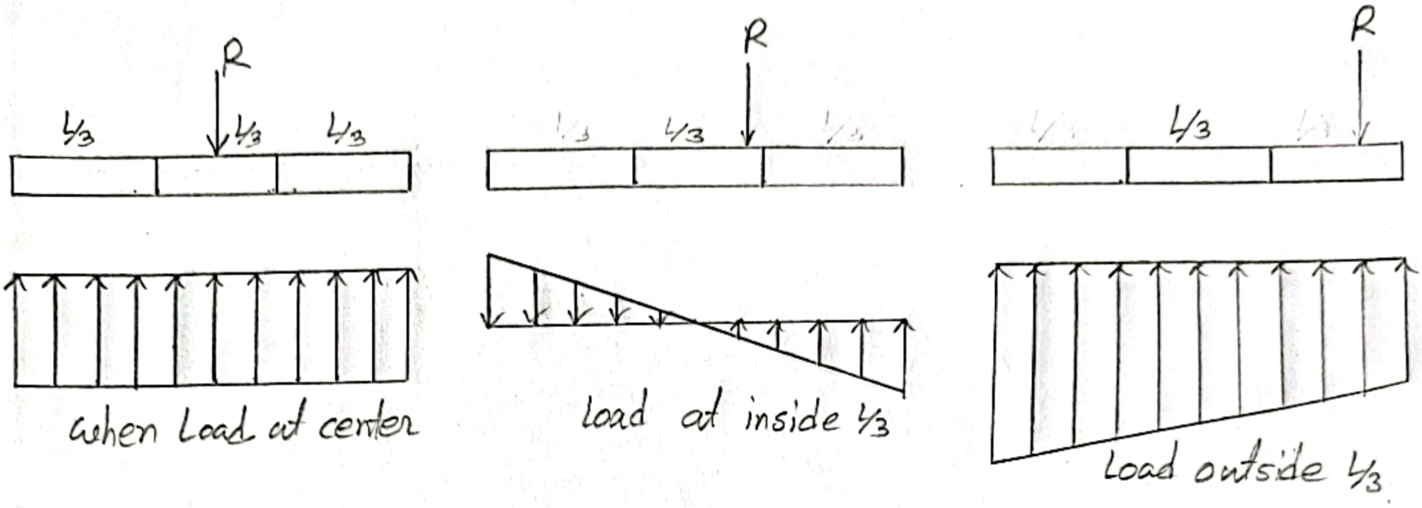


Figure. stress distribution at different situation.

$\therefore$ Resisting stress, $= \frac{P}{A} \pm \frac{MC}{I}$

Specifications :

1. Total height of the wall = 10 ft.
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, $f = 0.5$.

Calculation :

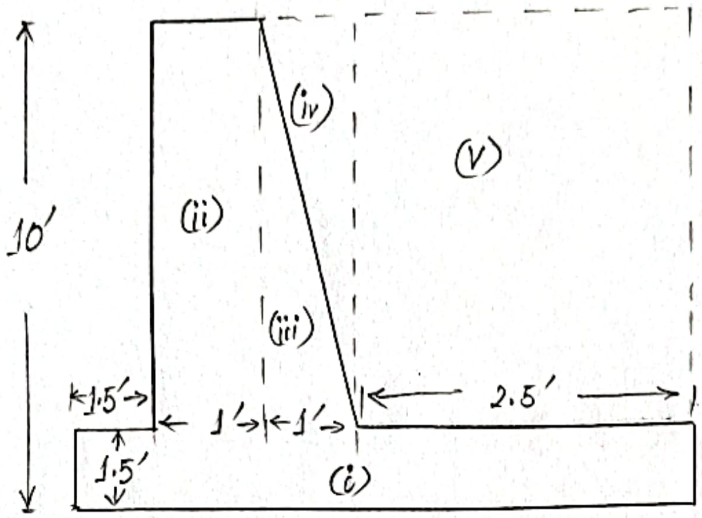


Figure: Abutment wall.

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

$$= 0.33$$

Force acting on wall $= K_a \gamma H = 0.33 \times 120 \times 10$

$$= 396 \text{ Psf.}$$

∴ Total active earth pressure, $P_a = \frac{1}{2} \times 396 \times 10$

$$= 1980 \text{ lb.}$$

Overturning Moment, $M_o = P_a \times \frac{H}{3} = 1980 \times \frac{10}{3}$

$= 6600$ lb-ft

Resisting Moment (M_R) calculation:

Case-I:

Section	Weight, W(lb)	Moment arm (ft)	Resisting Moment M_R (lb-ft)
i	$1.5 \times 6 \times 150 = 1350$	3	4050
ii	$1 \times 8.5 \times 150 = 1275$	$1.5 + 0.5 = 2$	2550
iii	$\frac{1}{2} \times 1 \times 8.5 \times 150 = 637.5$	$2.5 + \frac{1 \times 1}{3} = \frac{17}{6}$	1806.25
iv	$\frac{1}{2} \times 1 \times 8.5 \times 120 = 510$	$2.5 + \frac{1 \times 2}{3} = \frac{19}{6}$	1615
v	$2.5 \times 8.5 \times 120 = 2550$	$3.5 + \frac{2.5}{2} = 4.75$	12112.5
	$\Sigma W = 6322.5$		$\Sigma M_R = 22133.75$

Case-II:

Dead load	2420 (lb)	Moment arm = 2 ft	$M_R = 4840$ lb-ft
	$\Sigma W = 8742.5$ lb		$\Sigma M_R = 26973.75$ lb-ft

Case-III:

Live Load	3000 lb	Liver arm = 2 ft	$M_R = 6000$ lb-ft.
	$\Sigma W = 11742.5$ lb.		$\Sigma M_R = 32973.75$ lb-ft.

Stability Check:

For case I: No super structure is present.

Factor of safety against overturning moment, $= \frac{M_R}{M_o}$

$$= \frac{22133.75}{6600}$$

$$= 3.35 > 1.5$$

Design is ok.

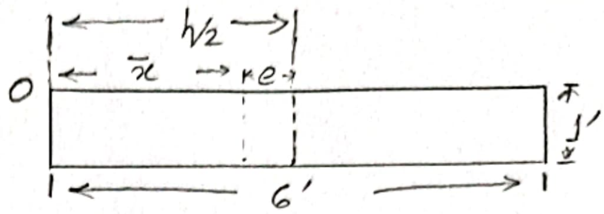
Factor of safety against sliding $= \frac{Wf}{P_a}$

$$= \frac{6322 \times 0.5}{1980}$$

$$= 1.6 > 1.5$$

∴ The design is ok.

Check for bearing capacity of soil :



Location of resultant from O

$$\bar{x} = \frac{M_R - M_O}{\Sigma W}$$

$$= \frac{22133.75 - 6600}{6322.5}$$

$$= 2.46 \text{ ft.}$$

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

$$= \frac{6}{2} - 2.46$$

$$= 0.54 \text{ ft.}$$

Now, Moment, $M = \Sigma W \times e$

$$= 6322.5 \times 0.54$$

$$= 3414.15 \text{ lb-ft.}$$

Moment of inertia, $I = \frac{bh^3}{12} = \frac{1 \times 63}{12}$

$$= 18 \text{ ft}^4.$$

$\therefore$ Stress, $\sigma_1 = \frac{P}{A} + \frac{MC}{I}$

$= \frac{6322.5}{6 \times 1} + \frac{3414.15 \times 3}{18}$

$= 1622.755$ Psf < 3000 Psf

and, $\sigma_2 = \frac{P}{A} - \frac{MC}{I}$

$= \frac{6322.5}{6 \times 1} - \frac{3414.15 \times 3}{18}$

$= 484.725$ Psf < 3000 Psf.

The design is ok.

Case II: Superstructure is present but no Live load

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

$= \frac{26973.75}{6600}$

$= 4.08 > 1.5$

Factor of safety against sliding $= \frac{\Sigma W f}{P_a} = \frac{8742.5 \times 0.5}{1980}$

$= 2.2 > 1.5$

∴ The design is ok.

Case-III: Superstructure is present with Live load

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

$= \frac{32973.75}{6600}$

$= 5 > 1.5$

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

$= \frac{11742.5 \times 0.5}{1980}$

$= 2.97 > 1.5$

The design is ok.

Chapter No- 3.1

Chapter Name : Design of a deck girder bridge.

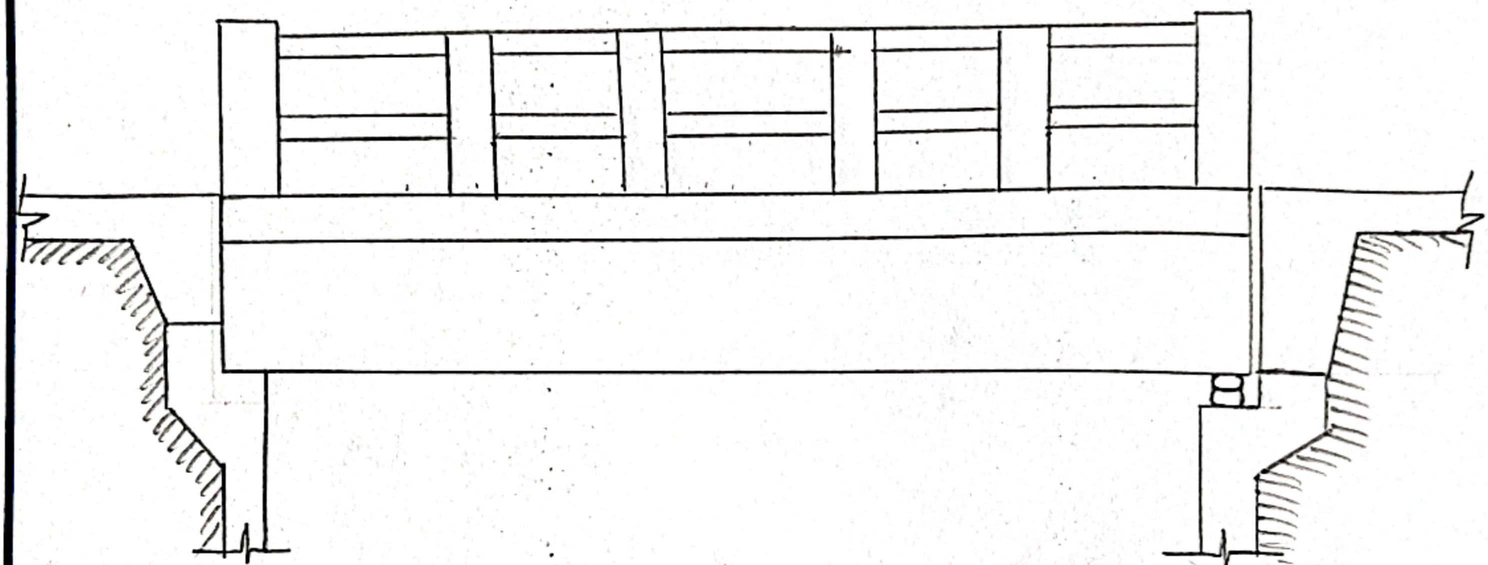


Fig. Elevation of deck girder bridge.

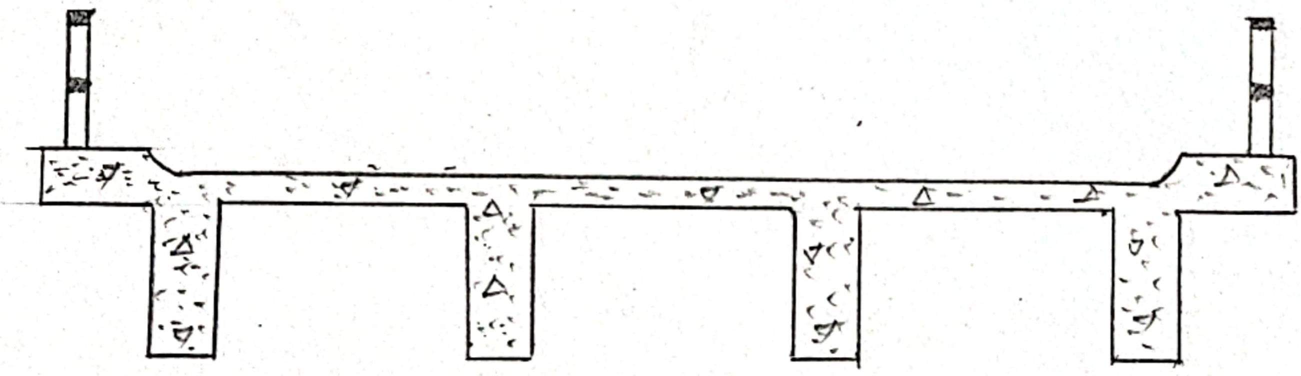


Fig. Cross section of deck girder bridge.

Specification:

① Length of the span, L = last 2 digit of roll + $\frac{50}{3}$

$= 47 + \frac{50}{3} = 63.67$ ft.

② Width of the bidge = Equivalent to 2 lane = 20 ft.

③ Number of girder = 4

④ Loading system = $H_{15}S_{12}$

⑤ Width of the girder maximum 15"

⑥ $f_c' = 3000$ Psi

⑦ $f_y = 50000$ Psi

Calculations:

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

$= 6.67$ ft.

(i) Relative properties:

$$n = \frac{E_s}{E_c} = \frac{29\times10^6}{57000\sqrt{3000}} = 9.$$

$$r = \frac{f_s}{f_c} = \frac{0.4\times50}{0.45\times3} = 14.81.$$

$$K = \frac{n}{n+r} = \frac{9}{9+14.81} = 0.38$$

$$j = 1-\frac{K}{3} = 1-\frac{0.38}{3} = 0.87.$$

$$\therefore R = \frac{1}{2} f_c\, j K = \frac{1}{2}\times0.45\times3000\times0.87\times0.38.$$

$$= 223.16.$$

(ii) Design of slab:

Weight of wearing coarse = 20 Psf

Assuming thickness of slab = 6″

Center to center spacing of girder = 6.67 ft.

$\therefore$ Length of clear span $= 6.67-\left(\frac{15}{12}\right) = 5.42$ ft.

(iii) Load calculation:

a. self weight of slab $= \frac{6}{12} \times 150 = 75$ Psf.

b. Weight of wearing coarse $= 20$ Psf.

∴ Total dead load $= 75 + 20 = 95$ Psf.

(iv) Moment calculation:

For continuity of girder,

Dead load moment $= \frac{1}{10} w.L^2$

$= \frac{1}{10} \times 95 \times 5.42^2$

$= 279.1$ lb-ft.

Live Load Moment for main reinforcement perpendicular to traffic,

Live Load moment $= \frac{S+2}{32} \times P_{15-12}$.

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

$= 2782.5$ lb-ft.

Impact Coefficient, $IC = \frac{50}{L + 125} \leq 0.3$

$= \frac{50}{5.42 + 125} = 0.38 \leq 0.3$

$\therefore$ Impact coefficient = 0.3

$\therefore$ Impact Moment = IC × Live Load Moment

= 0.3 × 2782.5

= 834.75 lb-ft.

$\therefore$ Design moment

= Dead load moment + Live load moment + Impact moment

= 279.1 + 2782.5 + 834.75

= 3896.32 lb-ft.

(V) Depth check:

$$d_{req} = \sqrt{\frac{M}{Rb}}$$

$$= \sqrt{\frac{3896.35 \times 12}{223.16 \times 12}}$$

$$= 4.18''$$

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

$d_{act} > d_{req}$.

∴ The design is ok.

(Vi) Reinforcement Calculation:

Main reinforcement, $A_s = \dfrac{M}{f_s j d_{act}}$

$$= \frac{3896.35 \times 12}{20000 \times 0.87 \times 5}$$

$$= 0.54 \text{ in}^2.$$

We provide #5 bar @ $\dfrac{0.31 \times 12}{0.54} \approx 7''$ c/c.

Distribution reinforcement perpendicular to traffic is

$= \frac{220}{\sqrt{S}}\ \%$ [max 67%] of main reinforcement.

$= \frac{220}{\sqrt{5.42}}\ \% = 94.9\%$ but max 67%.

$\therefore$ Distribution reinforcement $= 0.67 \times 0.54$

$= 0.36\ in^2$.

We provide #4 bar @ $\frac{0.2 \times 12}{0.36} = 6.67 \approx 6''$ c/c.

(VII) Shear Check:

Dead load shear $= \frac{wL}{2} = \frac{95 \times 5.42}{2} = 257.45$ lb.

Live load shear:

Distribution of wheel load, $E = 4 + 0.06 \times 5.42$

$= 4.3252$ [max=7].

$\therefore$ Live load shear $= \frac{P}{E} = \frac{12000}{4.3252} = 2774.44$ lb.

Impact shear $= 0.3 \times 2774.44$

$= 832.33$ lb.

$\therefore$ Total shear $= 257.45 + 2774.44 + 832.33$

$= 3864.22$ lb.

$\therefore$ developed shear ~~force~~ stress, $V_{der} = \frac{3864.22}{12 \times 5 \times 12}$

$= 5.36$ Psi.

Allowable shear ~~for~~stress, $V_{all} = 1.1\sqrt{f_c'}$

$= 1.1\sqrt{3000}$

$= 60.25$ Psi.

$V_{all} > V_{der}$.

$\therefore$ No shear reinforcement is necessary.

(VIII) Bond Check:

Developed bond, $U_{dev} = \frac{V_{max}}{\Sigma_0 \, j \, d_{act}}$.

$$= \frac{V_{max}}{\pi \times \frac{12}{Spacing} \times d_b \times 0.87 \times 5}.$$

$$= \frac{257.45}{\pi \times \frac{12}{7} \times \frac{5}{8} \times 0.87 \times 5}.$$

$$= 17.58 \text{ Psi}.$$

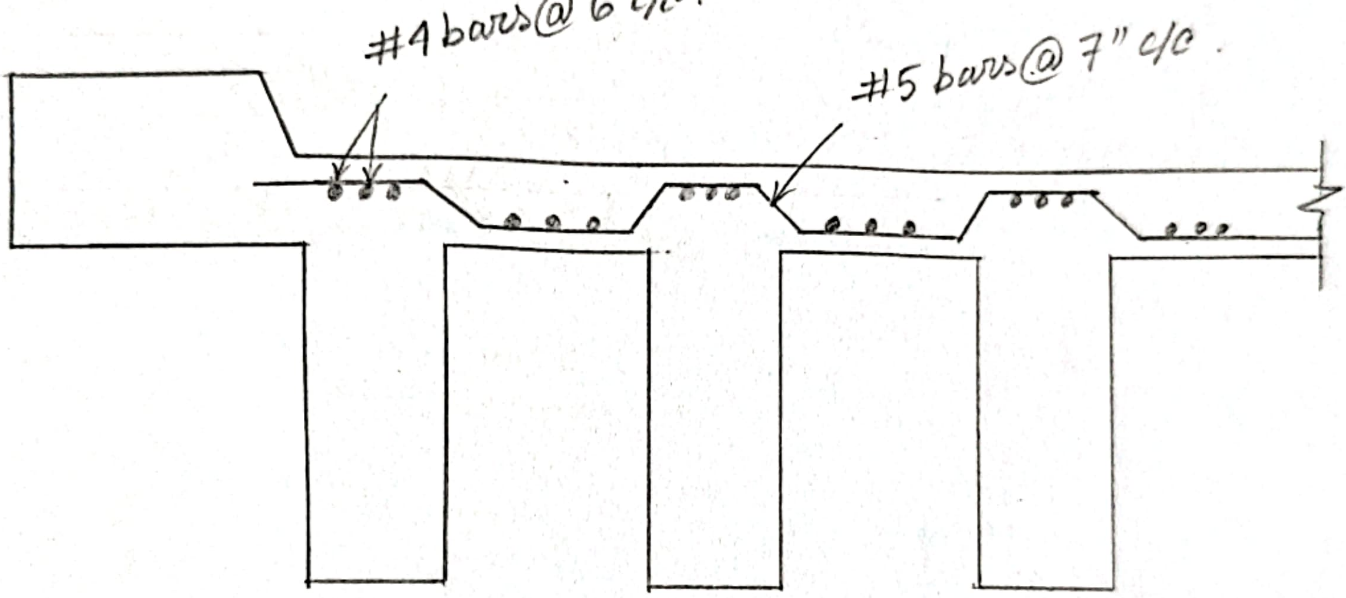
Allowable, $U_{all} = \frac{1.7\sqrt{f_c'}}{D} = \frac{1.7\sqrt{3000}}{5/8}$

$$= 148.98.$$

$\because U_{all} > U_{dev}$.

$\therefore$ The design is ok.

(IX) Working diagram:



Chapter No: 3.2

Chapter Name: Design of interior girder of a deck girder bridge.

Design Procedure:

The interior girders are T-beams with flange width equal to the center to center distance of girders.

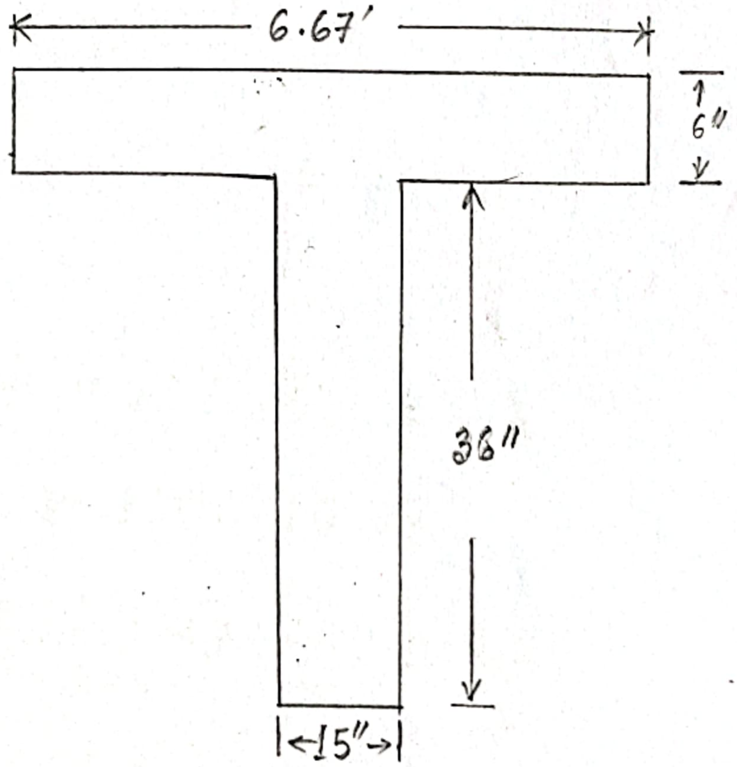


Fig. Interior girder with dimension.

(i) Load Calculation:

Let us assume slab thickness = 6".

Effective span length = 63.67 + 1 + 1

= 65.67 ft.

The weight slab per ft of beam = 95 × 6.67

= 633.65 Plf.

Self load of T-beam = $\frac{36 \times 15}{144} \times 150$

= 562.5 plf

∴ Total load = 633.65 + 562.5

= 1196.15 Plf

= 1.19 ≈ 1.2 Klf.

(ii) Dead Load shear and moment Calculation:

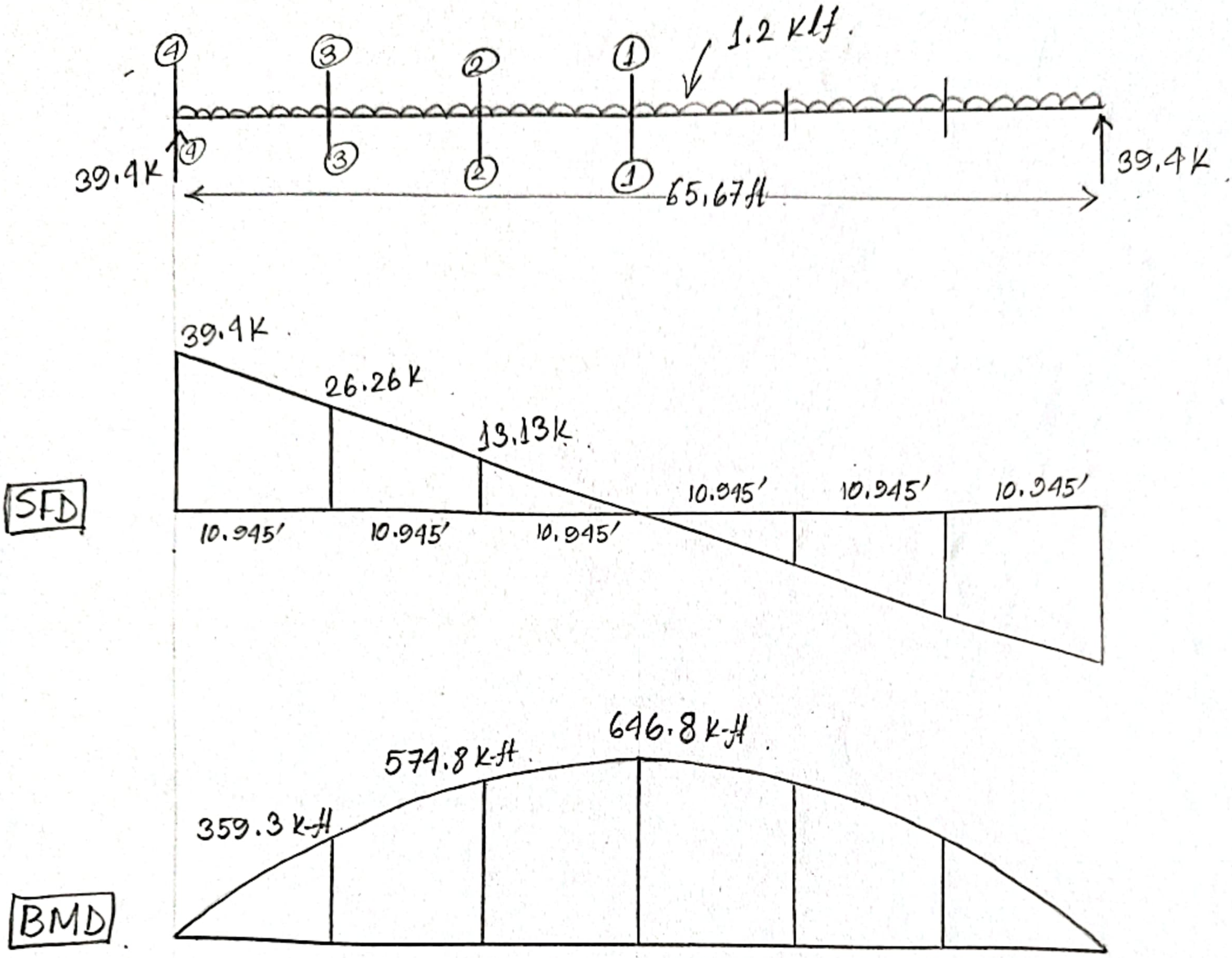


Fig. Dead load and shear force and bending moment diagram

Table: Dead Load shear and Moment:

Section	Shear force, V (K)	Bending Moment (k-ft)
1-1	0	646.8
2-2	13.13	574.8
3-3	26.26	359.3
4-4	39.4	0

(iii) Live Load Moment Calculation:

Effective wheel load - For one lane = $\frac{S}{6}$.

For two lane = $\frac{S}{5}$.

∴ For each interior girder must support = $\frac{6.67}{5}$

= 1.334 wheel load per wheel.

∴ The load from front wheel = 3×1.334 = 4 K.

The load from rare wheel = 12×1.334 = 16 K

Absolute maximum moment.

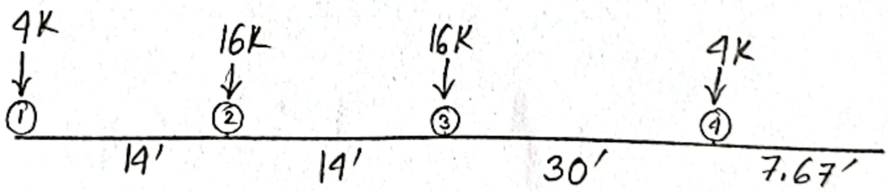


Fig. Arrangement of absolute maximum moment

Centroid from 4th wheel, $\bar{x} = \frac{(16\times30)+(16\times44)+(4\times58)}{4+16+16+4}$

$= 35.4$ ft.

Distance between 2nd wheel and $\bar{x}$,

$a = 44-\bar{x} = 44-35.4 = 8.6$ ft.

The position of wheel for maximum absolute moment from left corner $= \frac{L}{2} - \frac{a}{2} = \frac{65.67}{2} - \frac{8.6}{2}$

$= 28.535$ ft.

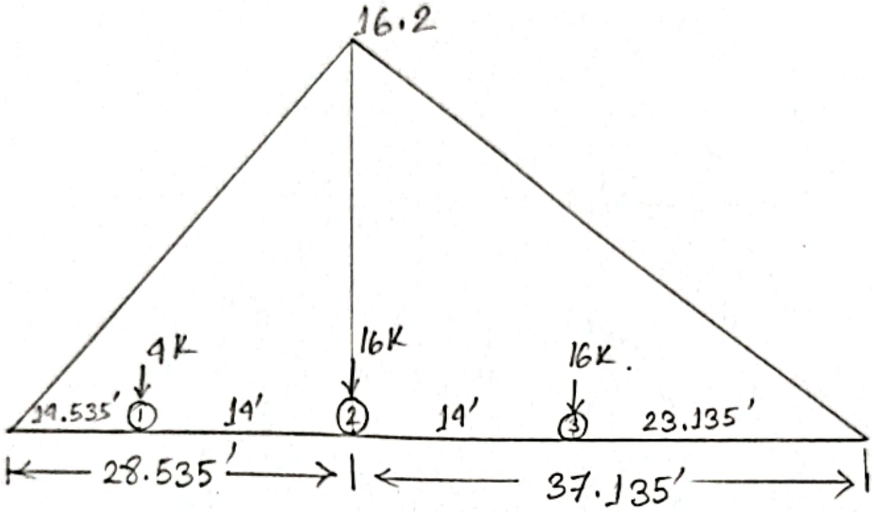


Fig. Absolute maximum moment diagram.

$$M_{abs} = \frac{16.2}{37.135}\left[(16 \times 23.135) + (16 \times 37.135)\right] + \frac{16.2}{28.535}(4 \times 14.535)$$

$= 420.68 + 33$

$= 453.68$ K-ft

≈ 453.7 K-ft.

Live load Moment at sections:

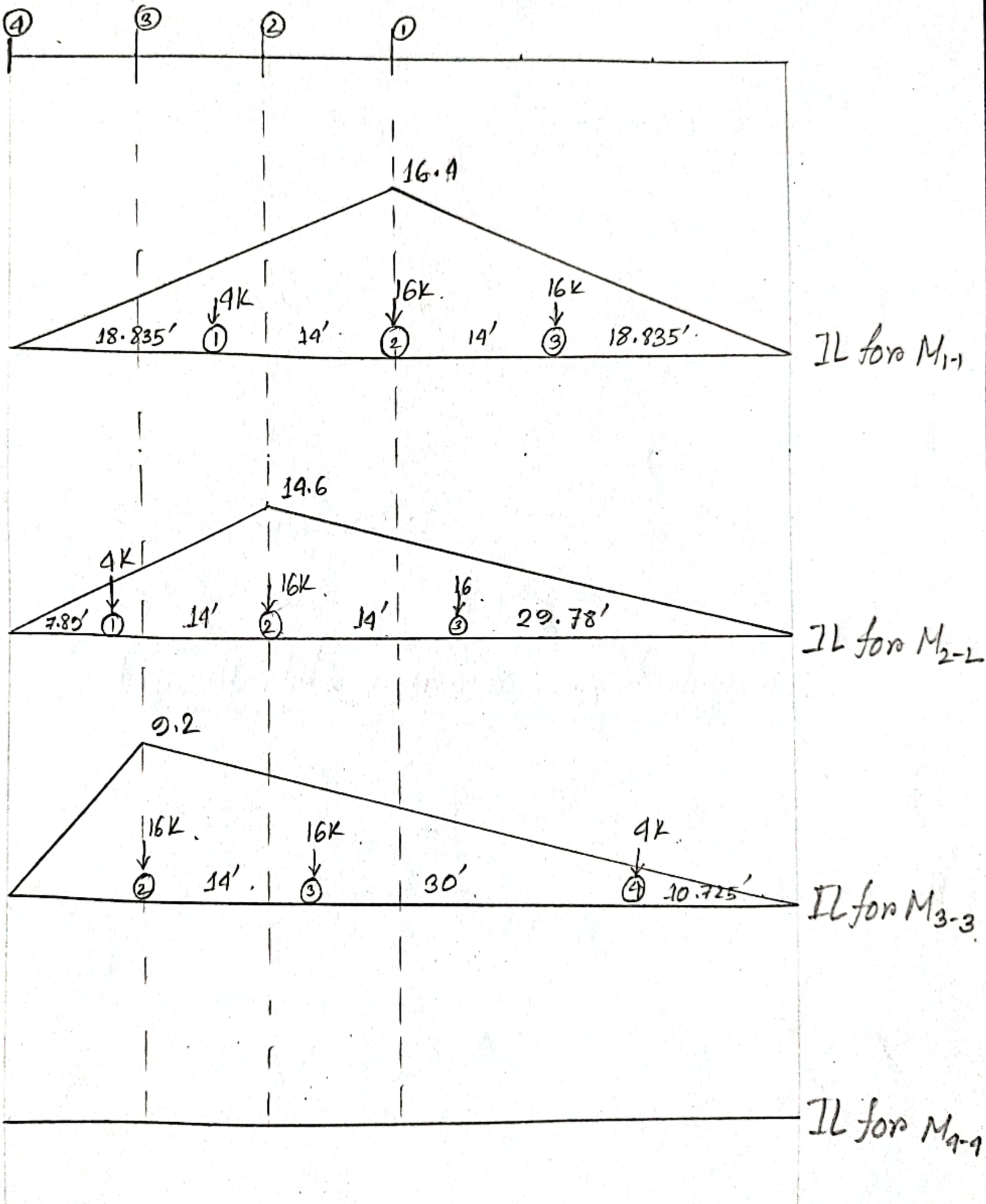


Fig. IL diagram for M_{1-1}, M_{2-2}, M_{3-3}, M_{4-4}

$$M_{1-1} = \frac{16.4}{32.835}\left[(16\times18.835)+(16\times32.835)\right]+\frac{16.4}{32.835}\times(4\times18.835)$$

$$= 450.5 \text{ k-ft.}$$

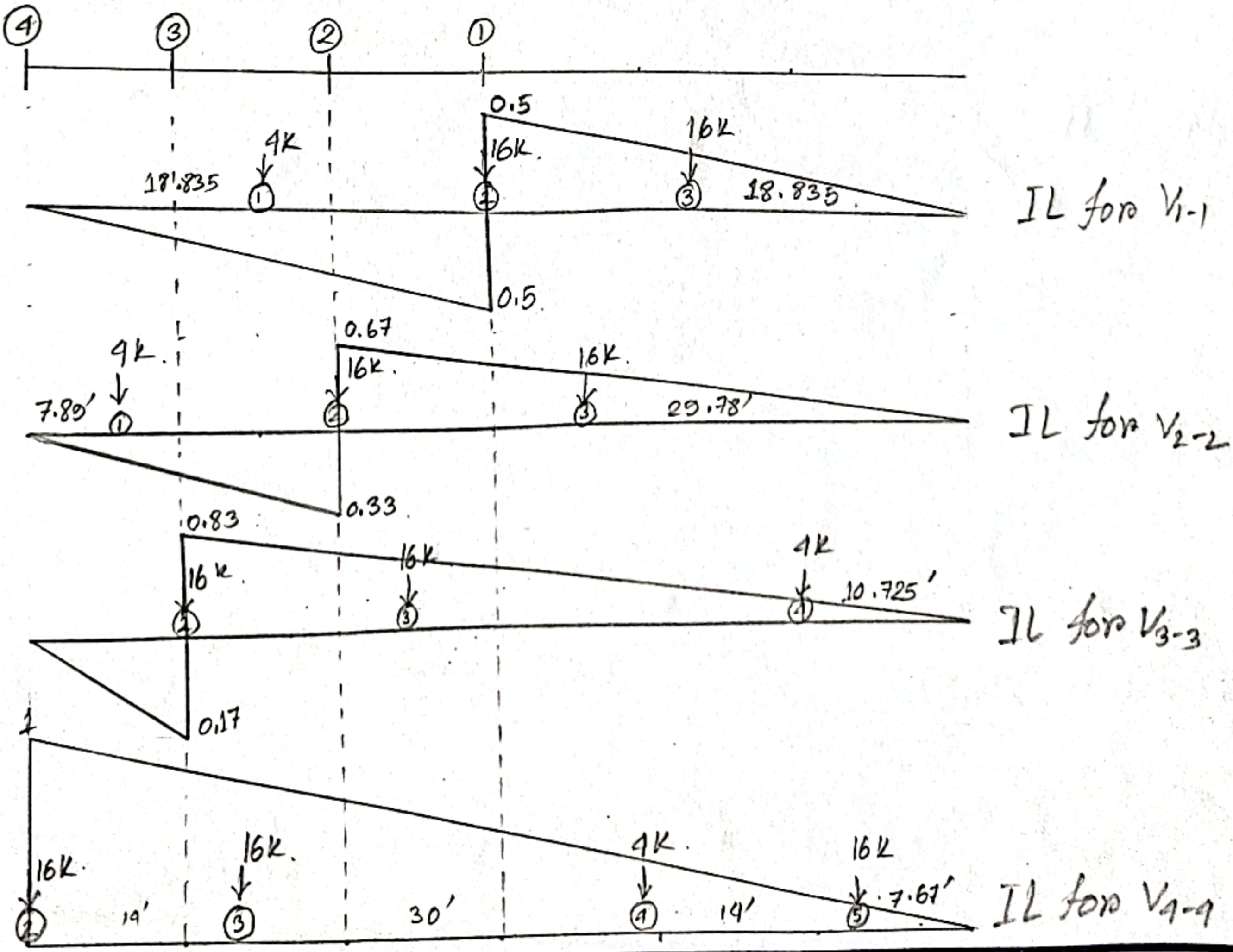
$$M_{2-2} = \frac{14.6}{43.78}\left[(16\times29.78)+(16\times43.78)\right]+\frac{14.6}{21.89}\times(4\times7.89)$$

$$= 413.5 \text{ k-ft.}$$

$$M_{3-3} = \frac{9.2}{51.725}\left[(16\times51.725)+(16\times40.725)+(4\times10.725)\right] = 263.9 \text{ k-ft.}$$

$$M_{4-4} = 0 \text{ k-ft.}$$

(iv) Live Load shear calculation:



$$V_{1-1} = \frac{0.5}{32.835}\left[(16\times32.835)+(16\times18.835)\right] = 12.6\ K.$$

$$V_{2-2} = \frac{0.67}{43.78}\left[(16\times29.78)+(16\times43.78)\right] = 18\ K.$$

$$V_{3-3} = \frac{0.83}{54.725}\left[(16\times54.725)+(16\times40.725)+(4\times10.725)\right] = 23.81\ K$$

$$V_{4-4} = \frac{1}{65.67}\left[(16\times7.67)+(4\times21.67)+(16\times51.67)+(16\times65.67)\right] = 31.78\ K$$

Table: Live Load Moment and shear

Section	Absolute maximum Live load moment (K-ft)	Live load moment (K-ft)	Live load shear (K)
1-1	453.7	450.5	12.6
2-2		413.5	18
3-3		263.9	23.81
4-4		0	31.78

(V) Impact moment and shear calculation.

$$\text{Impact Coefficient, } Ic = \frac{50}{65.67+125}$$

$$= 0.26.$$

Table: Impact Moment and shear

section	Impact coefficient, IC	Impact moment, LLM×IC (K-ft)	Impact shear LLS×IC (K-ft)
1-1	0.26	117.1	3.28
2-2		107.5	4.68
3-3		68.6	6.19
4-4		0	8.26
Abs.		118.	–

Table: Design shear

section	Dead shear (K)	Live shear (K)	Impact shear (K)	Total shear (K)	Design shear (K)
1-1	0	12.6	3.28	15.88	79.44
2-2	13.13	18	4.68	35.81	
3-3	26.26	23.81	6.19	56.26	
4-4	39.4	31.78	8.26	79.44	

Table: Design Moment

section	Dead Moment (K-ft)	Live Moment (K-ft)	Impact Moment (K-ft)	Total Moment (K-ft)	Design Moment (K-ft)
1-1	646.8	450.5	117.1	1214.4	1218.5
2-2	574.8	413.5	107.5	1095.8	
3-3	359.3	263.9	68.6	691.8	
4-4	0	0	0	0	
Abs	646.8	453.7	118	1218.5	

(vi) Depth check:

Design shear = 79.44 k

Maximum shearing stress in beam the web will take $= 0.06 f_c'$

$= 0.06 \times 3000$

$\therefore v = 180$ psi

The area to sustain this stress,

$$bd = \frac{V}{vj}$$

$$\Rightarrow d_{req} = \frac{V}{vjb} = \frac{79.44 \times 1000}{180 \times 0.87 \times 15}$$

$$= 33.8''$$

$\therefore d_{eff} = 36 + 6 - 2.5 - (2 \times 2) = 35.5'' > d_{req}$.

$\therefore$ The design is ok.

(vii) Reinforcement Calculation:

$$As_{Abs} = \frac{M_{Max}}{f_s(d - \frac{h_f}{2})} = \frac{1218.5 \times 12}{20(35.5 - \frac{6}{2})} = 22.5 \text{ in}^2.$$

We use 10 #14 bars in four layers $As'_{Abs} = 24 \text{ in}^2$.

spacing between top base = 14".
middle layer = 7"
bottom layer = 7"

$$As_{2-2} = \frac{1095.8 \times 12}{20\left(35.5 - \frac{6}{2}\right)} = 20.23 \, in^2.$$

We use 9#14 bars in three layers., $As'_{2-2} = 21.6 \, in^2$.

spacing between top base = 14".
middle layer = 7"
bottom layer = 7".

$$As_{3-3} = \frac{691.8 \times 12}{20\left(35.5 - \frac{6}{2}\right)} = 12.8 \, in^2.$$

We use 6#14 bars in 2 layers, $As'_{3-3} = 14.4 \, in^2$.

$As_{4-4} = 0 \, in^2$.

We use 2#14 bars in 1 layer, $As_{4-4} = 4.8 \, in^2$.

$$A_{s_{1-1}} = \frac{1214.4 \times 12}{20(35.5 - \frac{6}{2})} = 22.4 \ in^2.$$

We provide 10#14 bars in four layers $A_{s'_{1-1}} = 24 in^2$.

(viii) Check for T-beam:

Effective width of T-beam,

① $16h_f + b_w = 16 \times 6 + 15 = 111''$.

② $\frac{L}{4} = \frac{65.67 \times 12}{4} = 197.01''$

③ c/c distance $= 6.67 \times 12 = 80''$.

$\therefore$ Effective flange width $= 80''$.

$$\rho = \frac{A_s}{bd} = \frac{24}{80 \times 35.5} = 0.0085$$

$n = 9$.

$\therefore n\rho = 9 \times 0.0085 = 0.0765$.

$$K = \frac{n\rho + \frac{1}{2}\left(\frac{h_f}{d}\right)^2}{n\rho + \frac{1}{2}\left(\frac{h_f}{d}\right)} = \frac{0.0765 + \frac{1}{2}\left(\frac{6}{35.5}\right)^2}{0.0765 + \frac{1}{2}\left(\frac{6}{35.5}\right)}$$

$$= 0.56$$

$\therefore Kd = 0.56 \times 35.5 = 19.88 > h_f$.

$\therefore$ The section is T- beam (Confirmed).

(IX) Bond Check:

$$U_{dev} = \frac{V_{max}}{\Sigma_o\, j\, d} = \frac{79.44 \times 1000}{(4 \times 3.1416 \times \frac{14}{8}) \times 0.87 \times 35.5}$$

$$= 116.96 \text{ Psi}.$$

$U_{all} = 0.1 f_c' = 0.1 \times 3000 = 300$ Psi.

$\therefore U_{all} > U_{dev}$.

$\therefore$ Bond is checked.

(X) Web reinforcement Calculation:

Maximum stress of concrete; $v_c = 0.03 \times 3000 = 90$ Psi

shear stress at end, $v_d = \frac{79.44 \times 1000}{15 \times 35.5 \times 0.87} = 171.5$ Psi

$$\therefore S = \frac{A_v f_s}{(v_d - v_c) b_w} = \frac{2 \times 0.31 \times 20000}{(171.5 - 90) \times 15}$$

$$= 10.14'' \approx 10'' \text{ c/c.}$$

Distance upto which stirrup is required.

$$S = \frac{v_d - v_c}{v_d} \times \frac{L}{2} = \frac{171.5 - 90}{171.5} \times \frac{65.67}{2}$$

$$= 15.6 \text{ ft.}$$

maximum strirrup is required from, $\frac{S}{2} = \frac{10''}{2}$

$$= 5'' \text{ to } 15.6 \text{ ft.}$$

$$\therefore S_{max} = \frac{d_{eff}}{2} = \frac{35.5}{2}$$

$$= 17.75'' \text{ c/c.}$$

Distance upto which stirrup is required for S_{max}

$$= \frac{V_d - 0.5V_c}{V_d} \times \frac{L}{2}$$

$$= \frac{171.5 - (0.5 \times 90)}{171.5} \times \frac{65.67}{2}$$

$= 24.2 ft \approx 24 ft$

$\therefore$ Stirrup is required from 15.6 ft to 24 ft distance.

$\therefore$ No stirrup is required $= 32.835 - 24$

$= 8.835$ ft.

Chapter No- 3.3

Chapter Name: Design of exterior girder of a deck girder bridge.

Design Procedure:

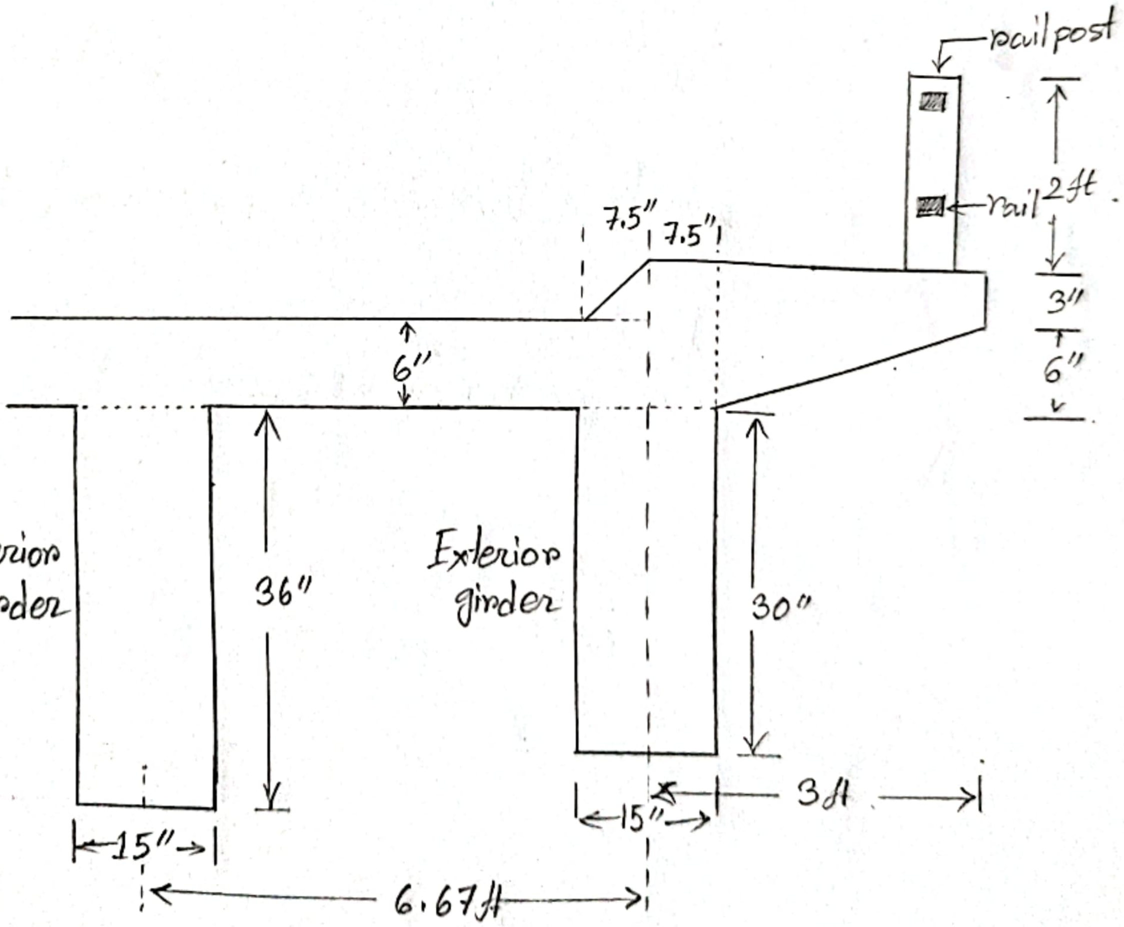


Fig.: Details of exterior girder.

(i) Load calculation:

Let exterior girder section is 15"x30"

Dead load from slab, exterior girder and Kerb section

$$= \left\{ \left(\frac{6}{12} \times \frac{6.67}{2} \right) + \left(\frac{30 \times 15}{12 \times 12} \right) + \left(\frac{1}{2} \times \frac{7.5}{12} \times \frac{7.5}{12} \right) + \frac{7.5 \times 9}{144} + \frac{1}{2} \times \left(\frac{3+9}{12} \right) \times \frac{28.5}{12} \right\} \times 150$$

$= 996.61$ Plf.

Dead load from railing and rail post $= 227.83$ Plf.

$\therefore$ Total load $= 996.61 + 227.83$

$= 1224.44$ Plf

$= 1.224$ Klf

≈ 1.23 Klf.

(ii) Dead load shear and moment calculation:

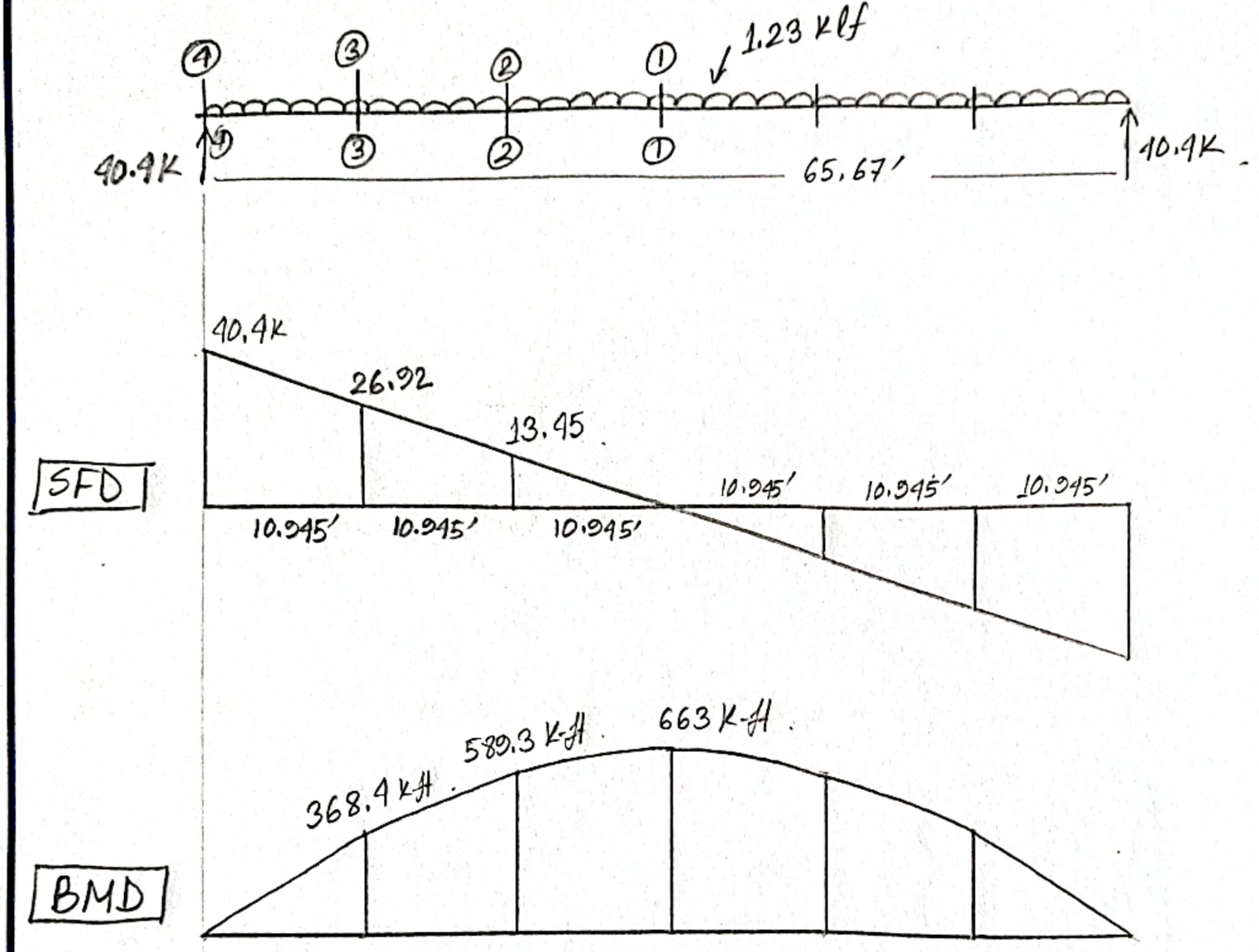


Fig. Dead load shear and bending moment diagram.

Table: Dead load shear and moment.

Section	Shear force, V(K)	Bending moment, M(K-ft)
1-1	0	663
2-2	13.45	589.3
3-3	26.92	368.4
4-4	40.4	0

(iii) Live load moment calculation:

Effective wheel load $= \dfrac{x/S}{\frac{S}{5}}$ [For 2 lane]

$= \dfrac{\frac{6.67-1.5}{6.67}}{1.334}$

$= 0.58$.

$\therefore$ The load from front wheel $= 3 \times 0.58$

$= 1.74$ Kips.

The load for rear wheel $= 12 \times 0.58$

$= 6.96$ Kips.

Absolute maximum moment:

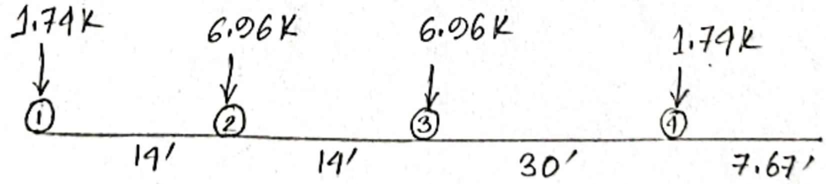


Fig. Arrangement for absolute moment.

Centroid from 4th wheen, $\bar{x} = \dfrac{(6.96\times30)+(6.96\times44)+(1.74\times58)}{1.74+1.74+6.96+6.96}$

$= 35.4$ ft.

$a = 44 - \bar{x} = 44 - 35.4 = 8.6$ ft

The position of 2nd wheel from right corner $= \frac{L}{2} + \frac{a}{2}$

$= \frac{65.67}{2} + \frac{8.6}{2} = 37.125'$

IL for Live load moment.

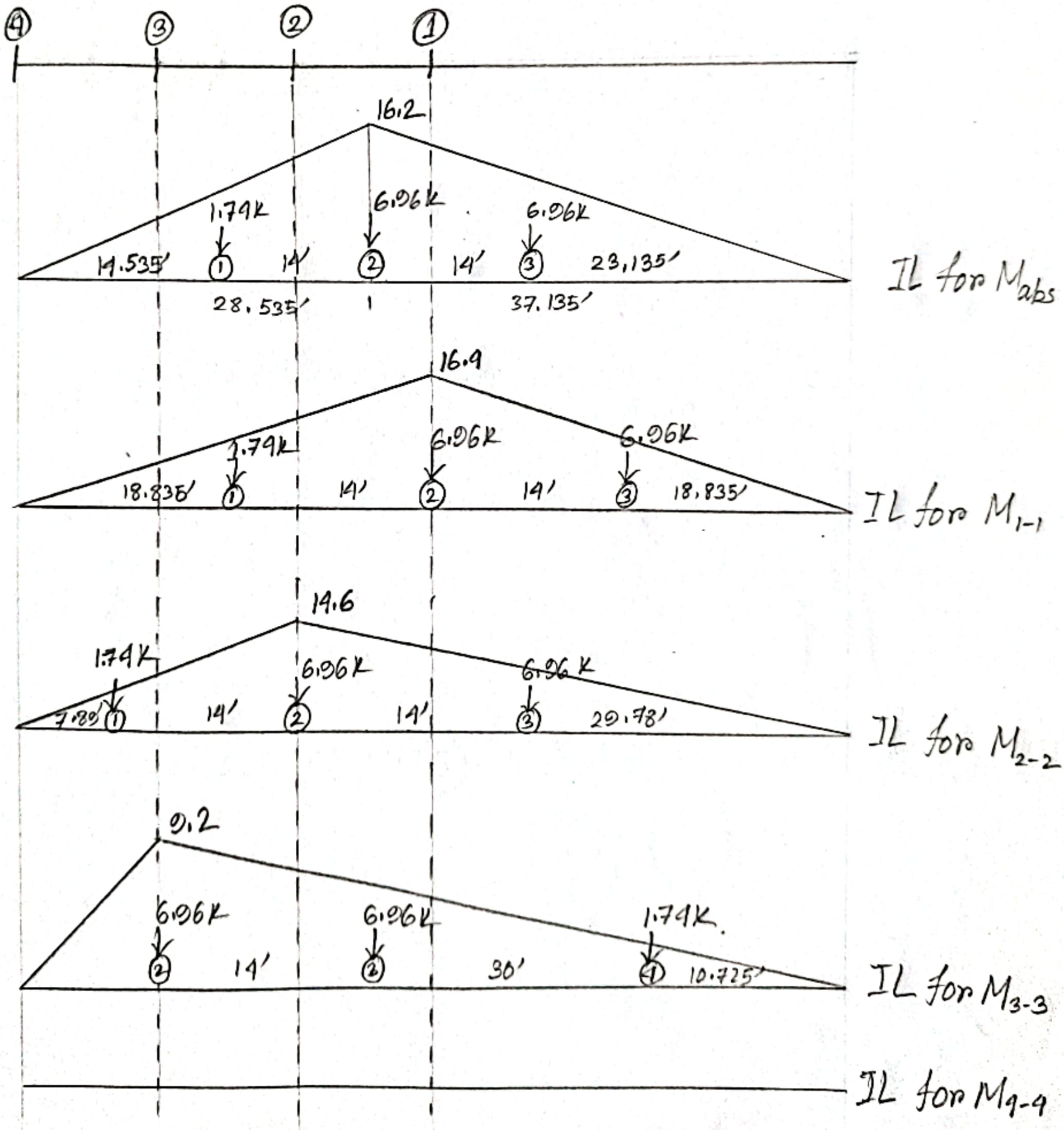


Fig. IL diagram for live load moment

$$M_{abs} = \frac{16.2}{37.135} \times \left[(23.135 \times 6.96) + (37.135 \times 6.96)\right] + \frac{16.2}{28.535}(1.74 \times 14.535)$$

$$= 183 + 14.36$$

$$= 197.4 \text{ K-ft.}$$

$$M_{1-1} = \frac{16.4}{32.835} \times \left[(6.96 \times 18.835) + (6.96 \times 32.835) + (1.74 \times 18.835)\right]$$

$$= 196 \text{ K-ft.}$$

$$M_{2-2} = \frac{14.6}{43.78} \times \left[(6.96 \times 29.78) + (6.96 \times 43.78)\right] + \frac{14.6}{21.89}(1.74 \times 7.89)$$

$$= 179.9 \text{ K-ft.}$$

$$M_{3-3} = \frac{9.2}{54.725} \times \left[(6.96 \times 54.725) + (6.96 \times 40.725) + (1.74 \times 10.725)\right]$$

$$= 114.8 \text{ K-ft.}$$

$$M_{4-4} = 0 \text{ K-ft.}$$

(iv) Live load shear Calculation :

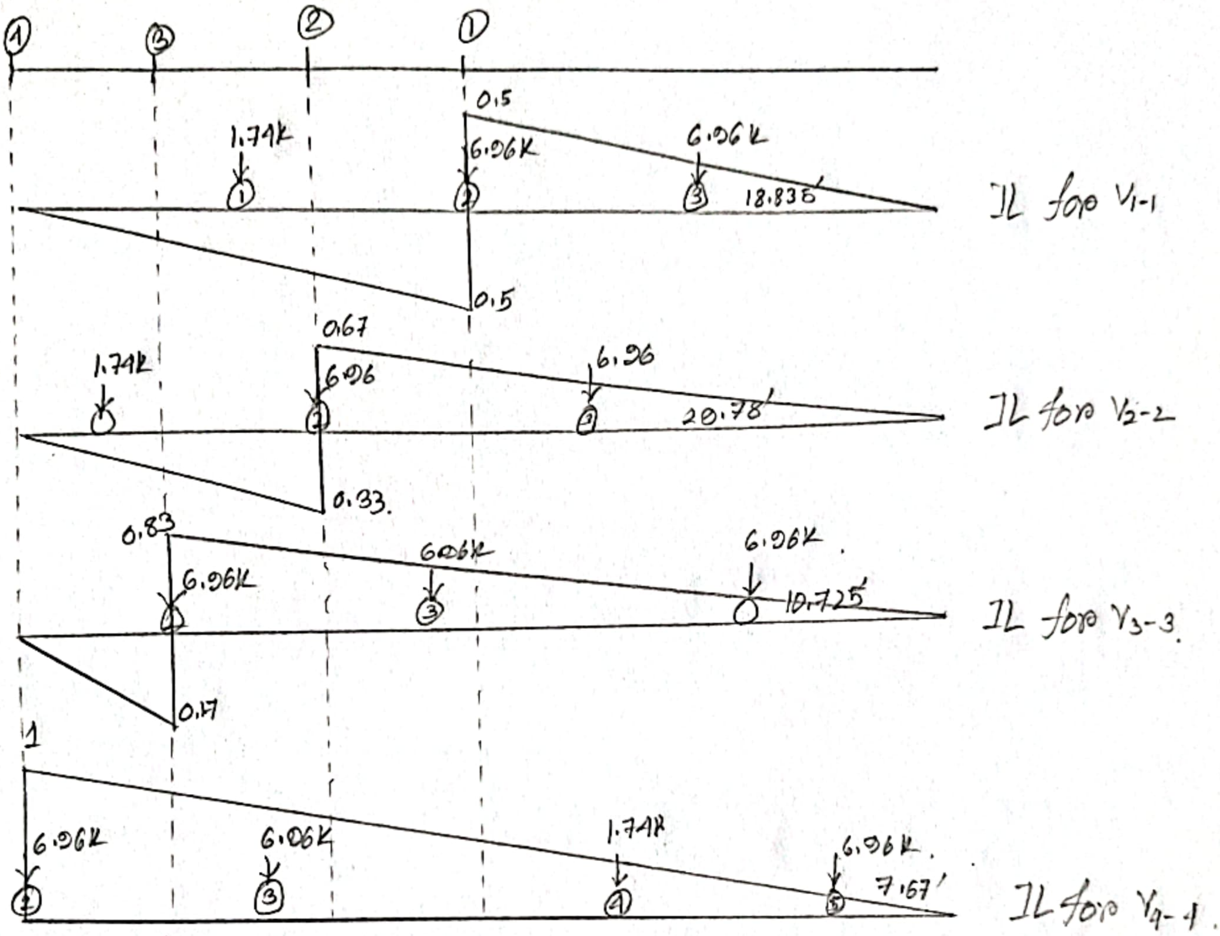


fig. IL for live load shear.

$$V_{1-1} = \frac{0.5}{32.835} \times \left[(6.96 \times 18.835) + (6.96 \times 32.835) \right] = 5.48\ K$$

$$V_{2-2} = \frac{0.67}{43.78} \times \left[(6.96 \times 29.78) + (6.96 \times 43.78) \right] = 7.85\ K.$$

$$V_{3-3} = \frac{0.83}{54.725} \times \left[(6.96 \times 54.725) + (6.96 \times 40.725) + (1.74 \times 10.725) \right]$$

$$= 10.36\ K.$$

$$V_{4-4} = \frac{1}{65.67}\left[(6.96\times7.67)+(1.74\times21.67)+(6.96\times51.67)+(6.96\times65.67)\right]$$

$$= 13.85\ K.$$

Table: Live Load Moment and shear

Section	Live load moment LLM (K-ft)	Live Load shear LLS (K)
1-1	196	5.48
2-2	179.9	7.85
3-3	114.8	10.36
4-4	0	13.85
Abs	197.4	-

(V) Impact moment and shear calculation.

$$\text{Impact coefficient, } IC = \frac{50}{65.67+125} = 0.26.$$

Table: Impact moment and shear:

	Impact coefficient (IC)	Section				
		1-1	2-2	3-3	4-4	Abs
Impact moment = LLM×IC (K-ft)	0.26	50.96	46.77	29.85	0	51.4
Impact shear = LLS×IC (K)		1.45	2.04	2.7	3.6	-

Table: Design shear.

section	Dead shear (K)	Live shear (K)	Impact shear (K)	Total shear (K)	Design shear (K)
1-1	0	5.48	1.45	6.93	
2-2	13.45	7.85	2.04	23.34	57.85
3-3	26.92	10.36	2.7	39.98	
4-4.	40.4	13.85	3.6	57.85	

Table: Design moment

Section.	Dead moment (K-ft)	Live moment (K-ft)	Impact moment (K-ft)	Total moment (K-ft)	Design moment. (K-ft)
1-1	663	196	50.96	909.96	
2-2	589.3	179.9	46.77	815.97	
3-3	368.4	114.8	29.85	513	~~911.8~~ 911.8
4-4	0	0	0	0	
Abs.	663.	197.4	51.4	911.8	

(vi) Depth check :

Maximum shear stress in the beam the web will take

$v = 0.06 f_c' = 0.06 \times 3000$

$= 180$ Psi

The area to sustain this stress

$$bd_{req} = \frac{V_{max}}{v j}$$

$$\Rightarrow d_{req} = \frac{V_{max}}{v j b} = \frac{57.85 \times 1000}{180 \times 0.87 \times 15}$$

$$= 24.65''$$

$$\therefore d_{eff} = 30 + 6 - 2.5 - \frac{5}{8} - \frac{14}{8} - \frac{2.5}{2}$$

$$= 29.785'' \approx 29.75''$$

$\therefore d_{eff} > d_{req}$.

$\therefore$ The design is adequate.

(Vii) Reinforcement Calculation:

$$As_{abs} = \frac{M_{max}}{f_s\left(d_{eff} - \frac{h}{2}\right)} = \frac{911.8 \times 12000}{20000\left(29.75 - \frac{6}{2}\right)} = 20.45\ in^2.$$

We provide 9#14 bars.

$$As_{1-1} = \frac{909.96 \times 12}{20\left(29.75 - \frac{6}{2}\right)} = 20.41\ in^2.$$

We provide 9# 14 bars

$$A_{s_{2-2}} = \frac{815.97 \times 12}{20\left(29.75 - \frac{6}{2}\right)} = 18.3 \ in^2$$

We provide 8#14 bars

$$A_{s_{3-3}} = \frac{513 \times 12}{20\left(29.75 - \frac{6}{2}\right)} = 11.5 \ in^2$$

We provide 6 #14 bars.

$$A_{s_{1-4}} = 0 \ in^2.$$

We provide 2#14 bars.

(viii) Check for L-beam:

Effective width, b -

① $6h_f + b_w = 6 \times 6 + 15 = 51''$

② $\frac{L}{12} + b_w = \frac{65.67}{12} + 15 = 80.67''$

③ $\frac{S}{2} + b_w = \frac{6.67 \times 12}{2} + 15 = 55''$

∴ Effective width, $b = 51''$

$$\rho = \frac{A_s}{bd} = \frac{20.45}{51 \times 29.75} = 0.0135$$

$n = 9$

$$\therefore n\rho = 9 \times 0.0135 = 0.1215.$$

$$K = \frac{n\rho + \frac{1}{2}\left(\frac{h_f}{d}\right)^2}{n\rho + \frac{1}{2}\left(\frac{h_f}{d}\right)} = \frac{0.1215 + \frac{1}{2}\left(\frac{6}{29.75}\right)^2}{0.1215 + \frac{1}{2}\left(\frac{6}{29.75}\right)}$$

$$= 0.65$$

$$\therefore Kd = 0.65 \times 29.75 = 19.34 > h_f.$$

∴ The section is L-beam (confirmed).

(ix) Bond check:

$$U_{dev} = \frac{V_{max}}{\Sigma_o j d} = \frac{57.85 \times 1000}{4 \times \pi \times \frac{14}{8} \times 0.87 \times 29.75}$$

$$= 101.65 \text{ Psi}$$

$U_{all} = 0.1 f_c' = 0.1 \times 3000 = 300$ Psi

$\therefore U_{all} > U_{dev}$.

$\therefore$ Bond check is okay.

(X) Web reinforcement:

Maximum stress of concrete, $V_c = 0.03 \times 3000 = 90$ Psi.

Shear at the end, $V_d = \dfrac{57.85 \times 1000}{15 \times 29.75 \times 0.87}$

$= 149$ Psi

$$S = \frac{A_v f_s}{(V_d - V_c) b_w} = \frac{2 \times 0.31 \times 20000}{(149 - 90) \times 15}$$

$= 14''$ c/c.

Distance upto which stirrup is required

$$S = \frac{V_d - V_c}{V_d} \times \frac{L}{2} = \frac{149 - 90}{149} \times \frac{65.67}{2}$$

$= 13$ ft.

Minimum stirrup required from $\frac{s}{2} = \frac{14''}{2}$

$= 7''$ to $13'$.

$S_{max} = \frac{d_{eff}}{2} = \frac{29.75}{2}$

$= 14.85''$ c/c.

Distance upto which stirrup is required for S_{max}

$= \frac{V_d - 0.5V_c}{V_d'} \times \frac{L}{2}$

$= \frac{149 - (0.5 \times 90)}{149} \times \frac{65.67}{2}$

$= 23$ ft.

∴ Stirrup is required from 13 ft to 23 ft distance

No stirrup is required $= 32.835 - 23$

$= 9.835$ ft.