Heaven’s Light is Our Guide

Rajshahi University of Engineering & Technology

Department of Civil Engineering

COURSE NO: CE-3218

COURSE TITLE: Reinforced Concrete Sessional-II

Submitted By	
Name: S.M.ABDULLAH AL AHAD Roll: 1600098 Section: B Session: 2016-17	Date: 28/11/2021

INDEX

Heaven's Light is Our Guide

Rajshahi University of Engineering & Technology
Rajshahi

DEPARTMENT OF CIVIL ENGINEERING

Expt. No. 01

Name of Expt Design of a slab bridge

SUBJECT : Reinforced concrete sessional II	**SUBMITTED BY :**
COURSE NO. : CE 3218	**NAME :** S.M. Abdullah Al Ahad
DATE OF EXPT. : 29/12/19	**CLASS :** 3rd year even semester
DATE OF SUB. : 07/01/20	**GROUP :** .... **ROLL NO** 1600098
	SESSION : 2016-17

Experiment no : 01

Name of the Experiment : Design of a slab bridge.

Introduction :

Bridge : A bridge is a structure built to span a physical obstacle, such as body of water, valley or road, without closing the way underneath. It is constructed for the purpose of providing passage over the obstacle. Usually something that can be detrimental to cross otherwise. There are many different designs that each serve a particular purpose and apply to different situations. Designs of bridges vary depending on the function of the bridge. Nature of terrain where the bridge is constructed and anchored materials used and funds available to build it.

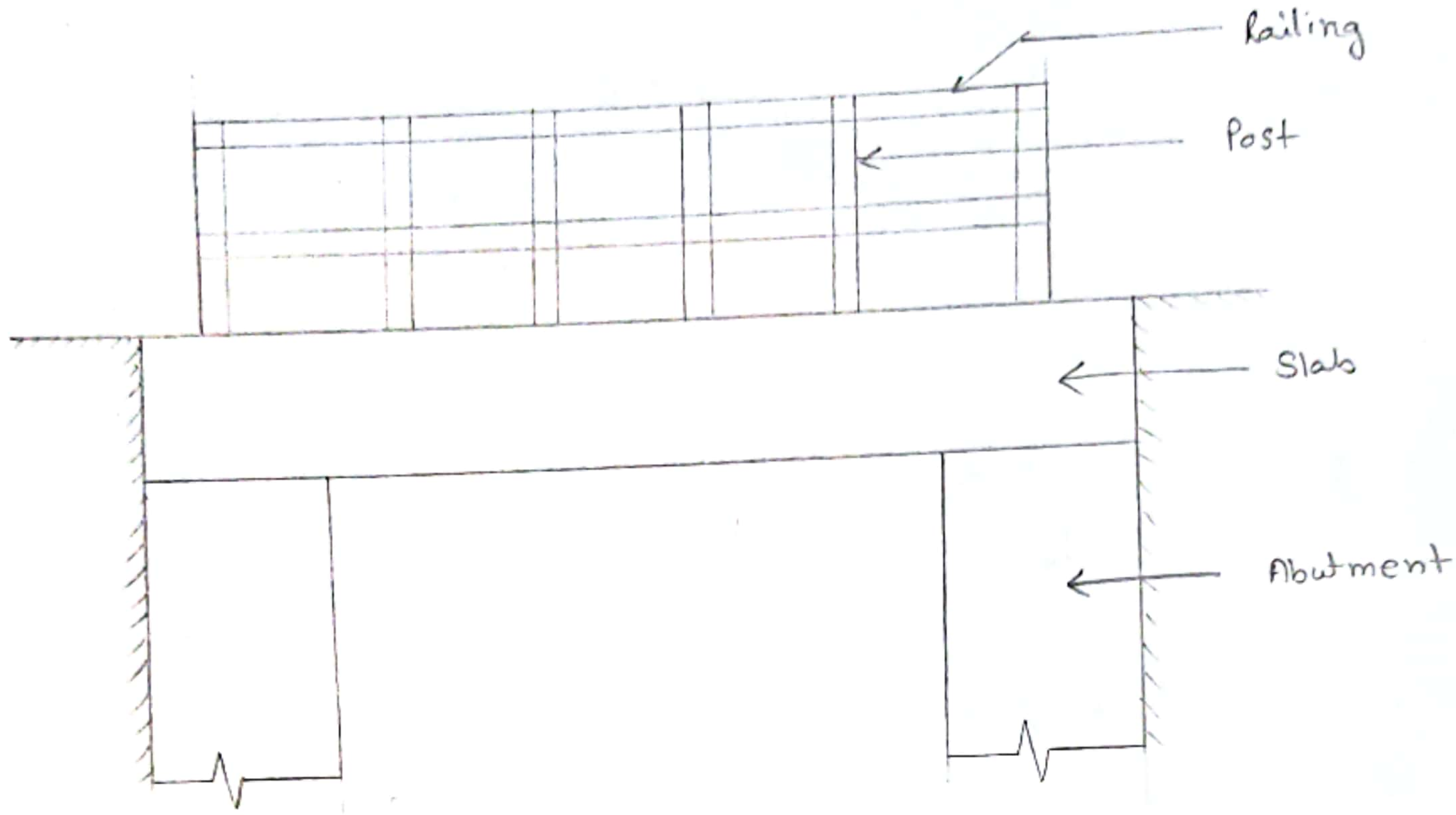


Fig. 1.1 : Elevation of a slab bridge.

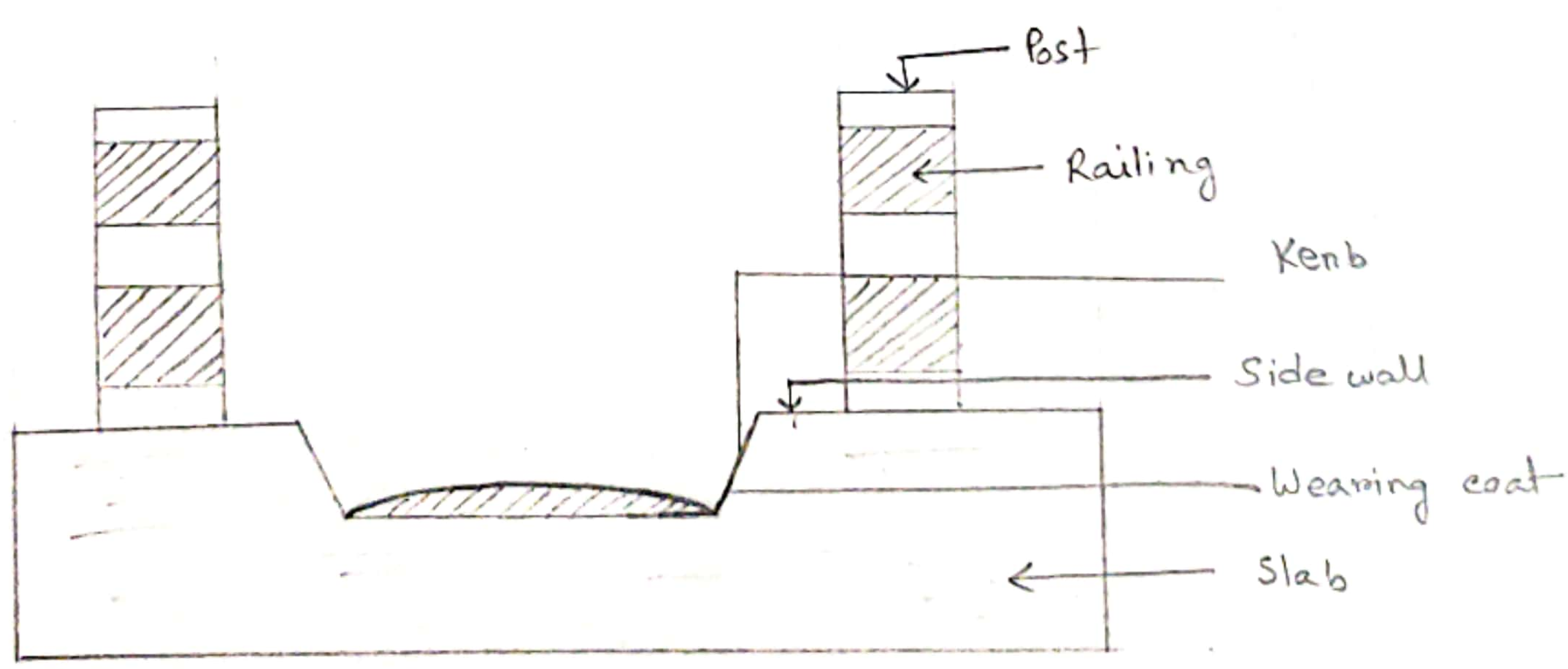


Fig. 1.2 : Cross section of slab bridge.

Specification :-

I) Span $= 13 + (98 \times .1) = 22.8$ ft.

II) Width of the bridge = 3 lanes = 30'

III) Loading system = $H_{20}S_{16}$ (even roll)

iv) Material properties :-

$f_c' = 3500$ psi. $= 3.5$ ksi

$f_y = 60,000$ psi $= 60$ ksi

f_c according to AASHTO $= 0.4 f_c'$

$\therefore f_c = 0.4 f_c'$

$= 0.4 \times 3.5$ Ksi $= 1.4$ Ksi $= 1400$ psi

v) Thickness of this slab = 12"

VI) Effective span, L = 22.8 ft

Relevant properties :-

1) $n = \frac{E_s}{E_c} = \frac{29 \times 10^6}{57000\sqrt{3500}} = 8.59 \approx 9$

2) $r = \frac{f_s}{f_c} = \frac{0.4 \times 60,000}{0.4 \times 3500} = 17.14$

3) $K = \frac{n}{n+r} = 0.344$

4) $j = 1 - K/3 = 0.8852 = 0.889$

5) $R = \frac{1}{2} f_c jk = \frac{1}{2} \times 1400 \times .344 \times .889 = 214.07$

Load calculation :-

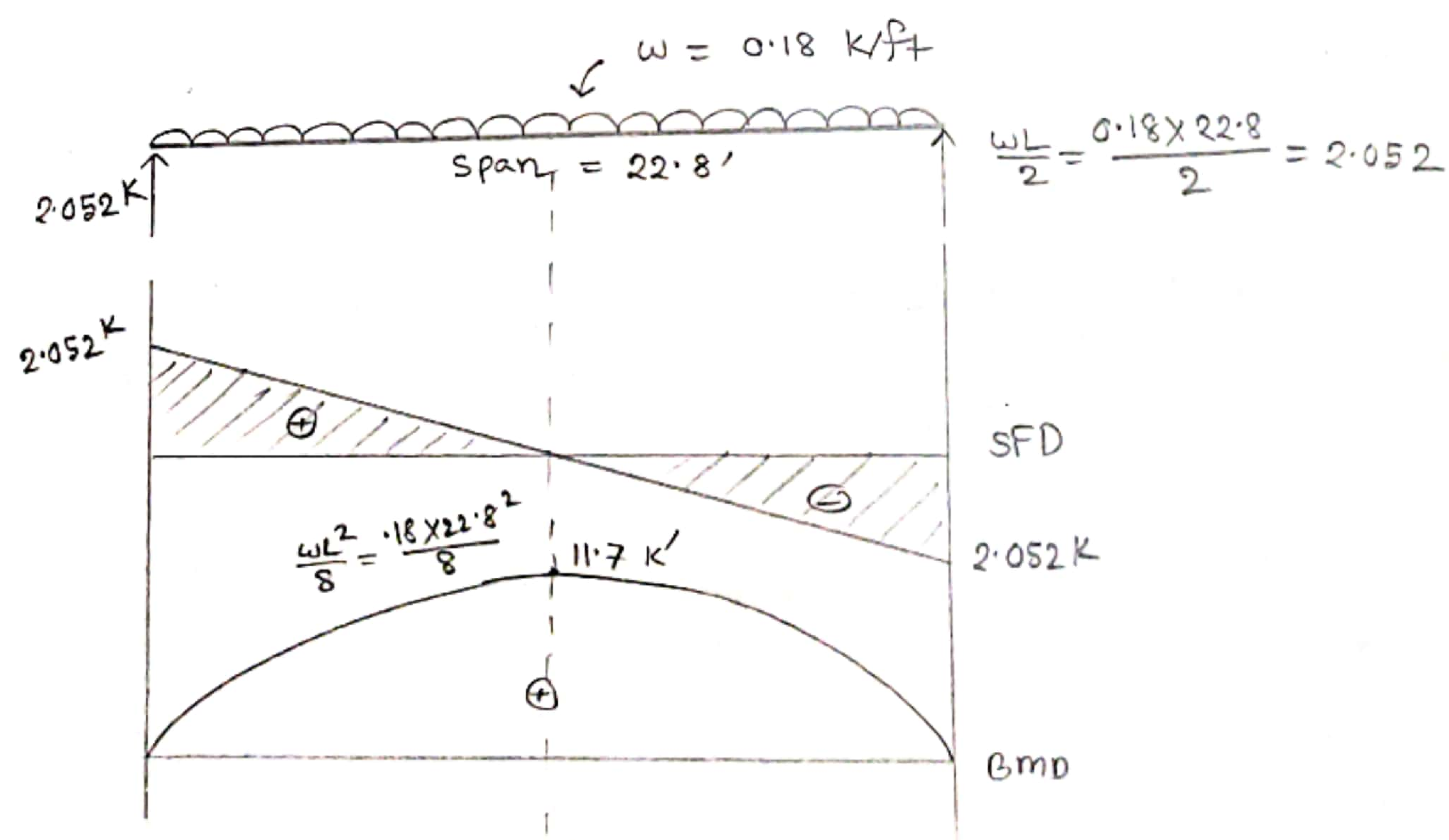
Dead load calculation :-

1) Self weight of slab $= \frac{12}{12} \times 150 = 150$ psf

11) Self weight of wearing coat $= 30$ psf

Total DL $= 180$ psf

Dead load shear and moment :-



Vmax for dead load = 2.052 K

Mmax for dead load = 11.7 K-ft.

Live load analysis :-

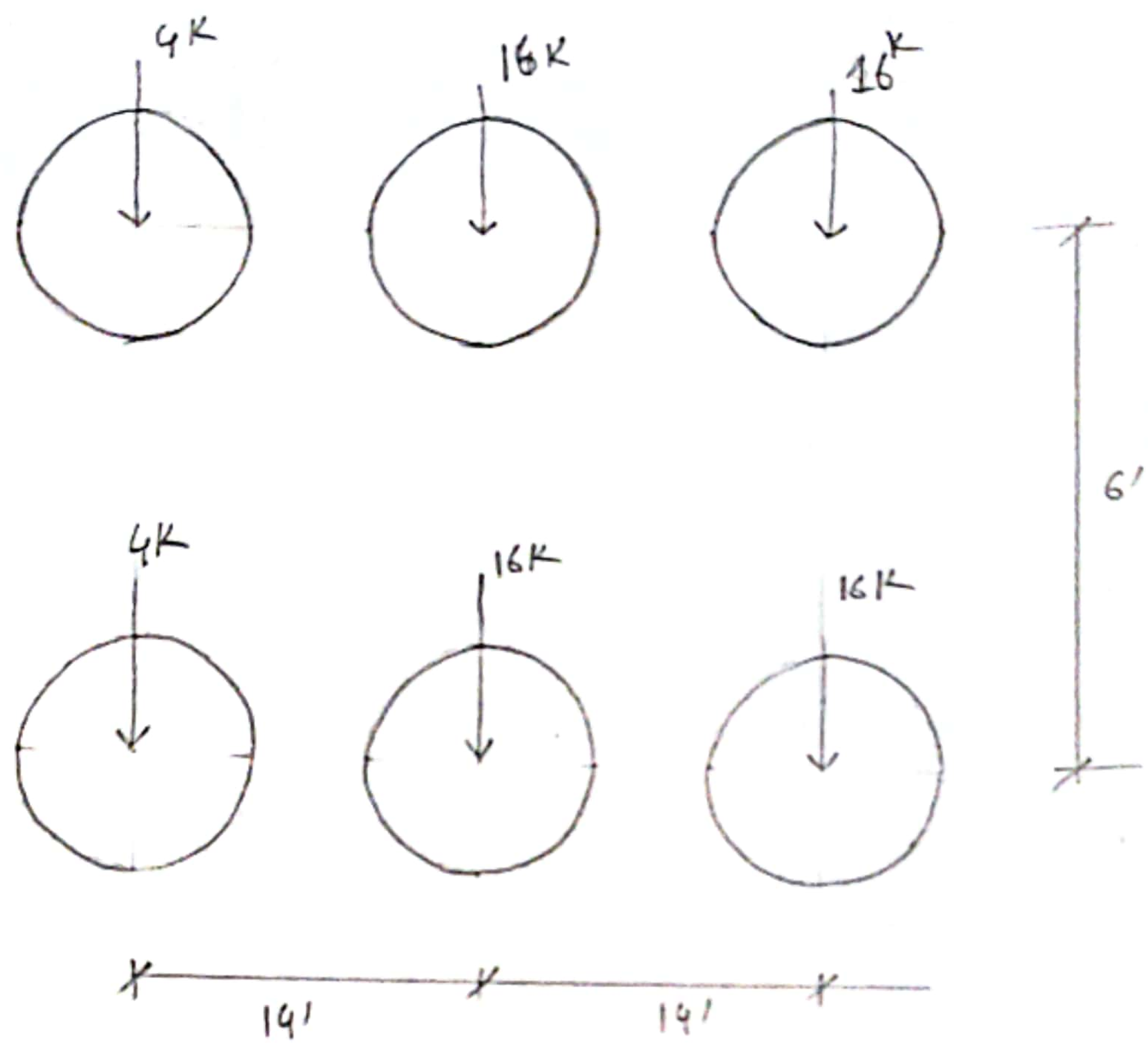


Figure : H_{20} S_{16} Truck load.

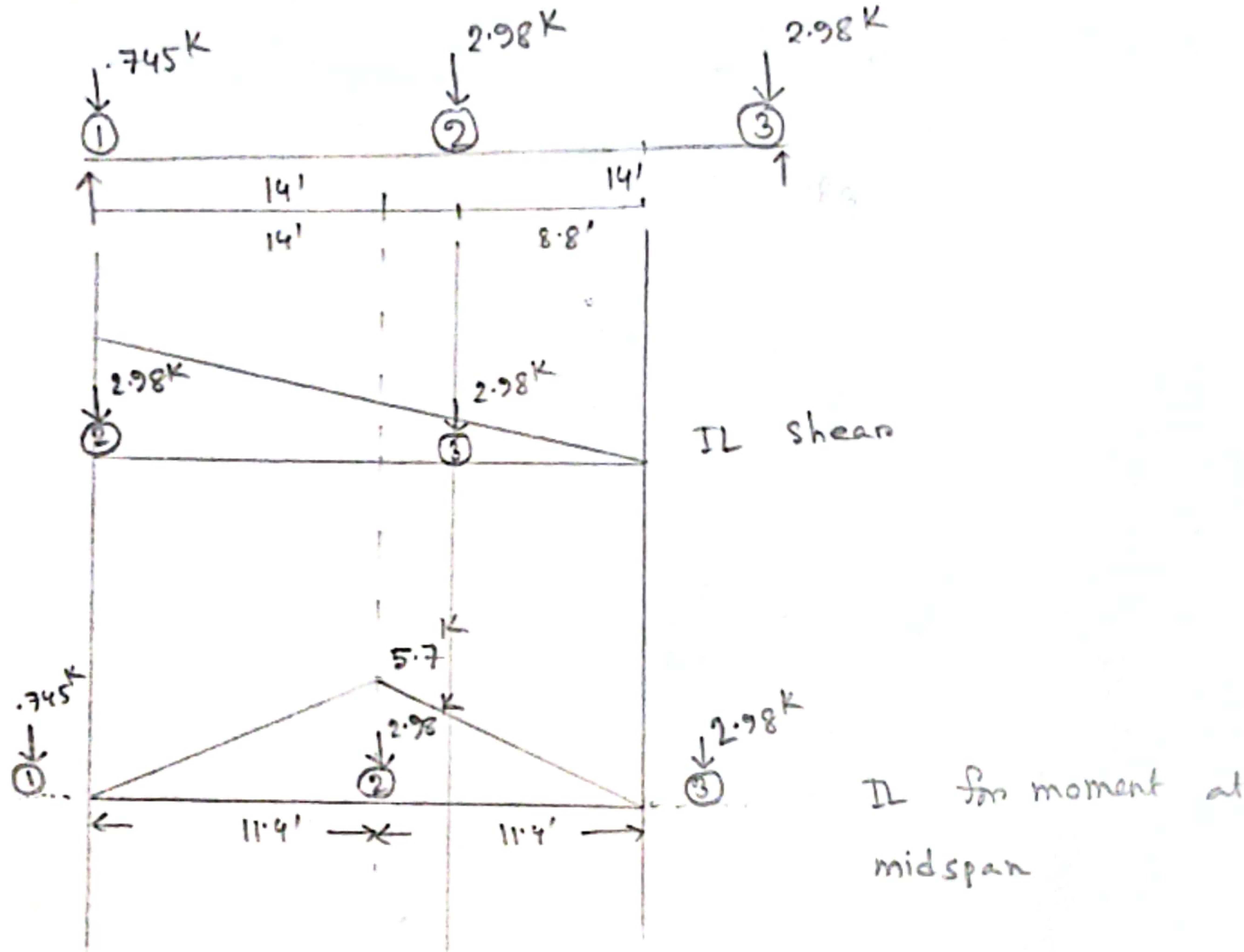
Effective width over which wheel load should be applied,

$$E = 4 + 0.06\,S \quad , \; S = \text{span length}$$
$$= 4 + 0.06 \times 22.8$$
$$= 5.37' \qquad [E \ngtr 7']$$

Effective wheel load, $P_e = \dfrac{P_{20}}{E} = \dfrac{16}{5.37} = 2.98$ K

Effective wheel load, $P_e = \dfrac{4}{5.37} = .745$ K

Live load shear and moment:-



$$V_{max} = \frac{1}{22.8}\left[2.98 \times 8.8 + 2.98 \times 22.8\right] = 4.13\ K$$

$$M_{max} = \frac{5.7}{11.4}\left[2.98 \times 11.4\right] = 16.986\ K'$$

Since, main reinforcement is parallel to traffic,

M_{max} on LL moment $= 900\,S$ lb-ft

[condition: Span up to and including 50′ for H20 S16 loading]

$\therefore$ LL moment $= 900\,S$

$= 900 \times 22.8$ lb-ft

$= 20,520$ lb-ft

$= 20.5$ K-ft

Impact shear and moment :-

Impact co-efficient $I.C = \dfrac{50}{L+125} \leq 0.3$

$L =$ loaded length

$I.C = \dfrac{50}{22.8+125} = 0.34 \approx 0.3$

Impact shear $= (LLS \times IC) = 4.13 \times 0.3 = 1.24K$

Impact moment $= (LLM \times IC) = 16.986 \times .3 = 5.096\,K'$

Impact moment (main reinforcement is parallel to traffic)

$= 20.5 \times 0.3$

$= 6.15\,K'$

Table for design shear and moment :-

Table for design shear:-

Table : 1.1

Dead load shear (Kip)	Live load shear (Kip)	Impact shear (Kip)	Design shear
2.052 K	4.13 K	1.24 K	7.42 K

Table for design moment:-

Table : 1.2

Dead load moment (K')	live load moment (K')	Impact moment (K')	Design moment (K')
11.7	16.986	5.096	33.78 K'

Depth Check:-

$$d_{req} = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{33.78 \times 12000}{214.07 \times 12}} = 12.56''$$

$$d_{eff} = t - 1 = 12 - 1 = 11' < d_{req}$$

$\therefore$ Design is not okay.

If we consider minimum thickness, $t_{min} = 15''$

$$d_{eff} = t_{min} - 1 = 15 - 1 = 14'' > d_{req}$$

Hence the design is OK.

2nd trial :-

Load calculation :-

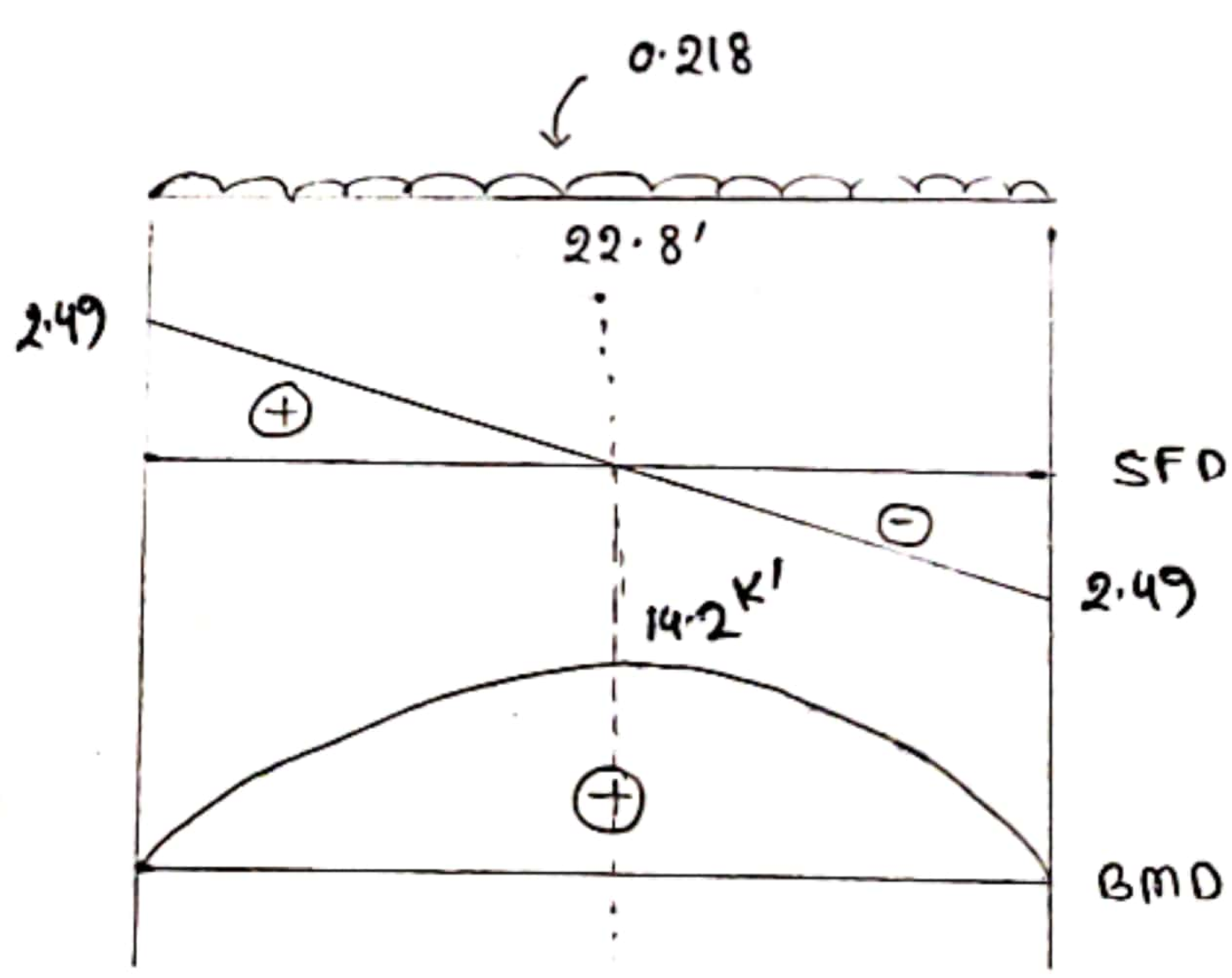
D.L calculation :-

1) Self weight for slab = $\frac{15}{12} \times 150$ = 187.5 psf

11) Self weight for wearing coat = $\frac{3}{12} \times 120$ = 30 psf

Total dead load = 217.5 psf = 0.218

Dead load shear and moment :-



Vmax for D.L = 2.49 K

Mmax for D.L = 14.2 K'

Table for design shear :-

Table : 1·3 :-

Dead load shear	Live load shear	Impact shear (Kip)	Design shear (Kip)
2·49 K	4·13 K	1·24 K	7·86

Table for design moment :-

Table 1·4 :-

Dead load moment (Kip)	live load moment	Impact moment	Design shear moment
14·2	16·986	5·096	36·3

Depth Check :-

$$d_{req} = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{36\cdot3 \times 12000}{214\cdot07 \times 12}} = 13\cdot02''$$

$$d_{eff} = t - 1 = 15 - 1 = 14'' > d_{req} \quad [14'' > 13\cdot02'']$$

∴ The design is OK.

Reinforcement Calculation:-

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

$$= \frac{36.3 \times 12 \times 1000}{24000 \times 0.889 \times \cancel{29}\ 14}$$

$$= 1.46 \text{ in}^2$$

Use # 5 bars, spacing, $S = \frac{0.31 \times 12}{1.46} = 2.55'' \text{ c/c} \approx 3'' \text{ c/c}$

$S_{max} = 3t = 3 \times 15 = 45''$

Therefore, use # 5 bars @ 3" c/c

Distribution reinforcement :-

For main reinforcement parallel to traffic :-

$$\text{Percentage} = \frac{100}{\sqrt{S}} = \frac{100}{\sqrt{22.8}} = 20.94\%$$

$\therefore$ Distribution reinforcement $= (1.46 \times 20.94\%) \text{ in}^2$

$$= 0.306 \text{ in}^2$$

Use # 4 bars ; $S = \frac{0.2 \times 12}{0.306} = 7.8'' \approx 8'' \text{ c/c}$

Use # 4 bars @ S = 8" c/c.

Shear Check:-

Developed shear, $V_d = \frac{1.15\,V}{bd} = \frac{1.15 \times 7860}{12 \times 14} = 53.8\text{ K}$

$= 53.8\text{ K}$

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

$= 1.1\sqrt{3500}$

$= 65.07 > 53.8\ (V_d)$

$\therefore V_{all} > V_d$

Thus, No Stirrup is required.

Reinforcement calculation:-

$$As = \frac{M}{f_s jd}$$

$$= \frac{144.89 \times 12 \times 1000}{24000 \times 0.889 \times 29}$$

$$= 2.81 \ in^2$$

Use # 9 bar @ $= \frac{1 \times 12}{2.81} = 4.27'' \ c/c$

Distribution reinforcement:-

Since, main reinforcement is parallel to traffic.

percentage $= \frac{100}{\sqrt{S}} = \frac{100}{\sqrt{22.8}} = 20.94\%$

$\therefore$ Distribution reinforcement $= (2.81 \times 20.94\%) \ in^2$

$= 0.588 \ in^2$

Use # 6 bar @ $= \frac{0.44 \times 12}{0.588} = 8.98'' \ c/c.$

Heaven's Light is Our Guide

Rajshahi University of Engineering & Technology
Rajshahi

DEPARTMENT OF CIVIL ENGINEERING

Expt. No. 02

Name of Expt Design of Edge Beam

SUBJECT : Reinforced Concrete Sessional – II	**SUBMITTED BY :**
COURSE NO. : CE – 3218	**NAME :** S.M. Abdullah Al Ahad
DATE OF EXPT.:	**CLASS :** 3rd year even semester
DATE OF SUB. :	**GROUP :** – **ROLL NO** 1600098
	SESSION : 2016-17

Design of edge beam :-

Edge beam shall be provided for all slabs having main reinforcement parallel to traffic. The beam may consist of a slab section additionally reinforced, A beam integral with and deeper than the slab or an integral reinforced section of slab and kerb. It shall be designed to resist a live load moment of 0.10 PS where P is the wheel loads in pounds (P20 or P15) and S is the span length in feet. The moment may be reduced by 20% incase of continuous span.

Assume edge beam section = 12" X 24"

Assume section of post = 8" X 8"

height of the post = 3.5 ft

c/c spacing of post = 6 ft.

Section of rail = 6" X 6"

Dead load analysis:-

Load Calculation:-

i) Self weight of edge beam $= \frac{12 \times 24}{12 \times 12} \times 150 = 300$ plf.

ii) weight of post $= \frac{8 \times 8}{12 \times 12} \times 3.5 \times 150 \times \frac{1}{\text{c/c spacing of post}}$

$= \frac{8 \times 8}{12 \times 12} \times 3.5 \times 150 \times 1/6$

$= 38.88$ lb/ft

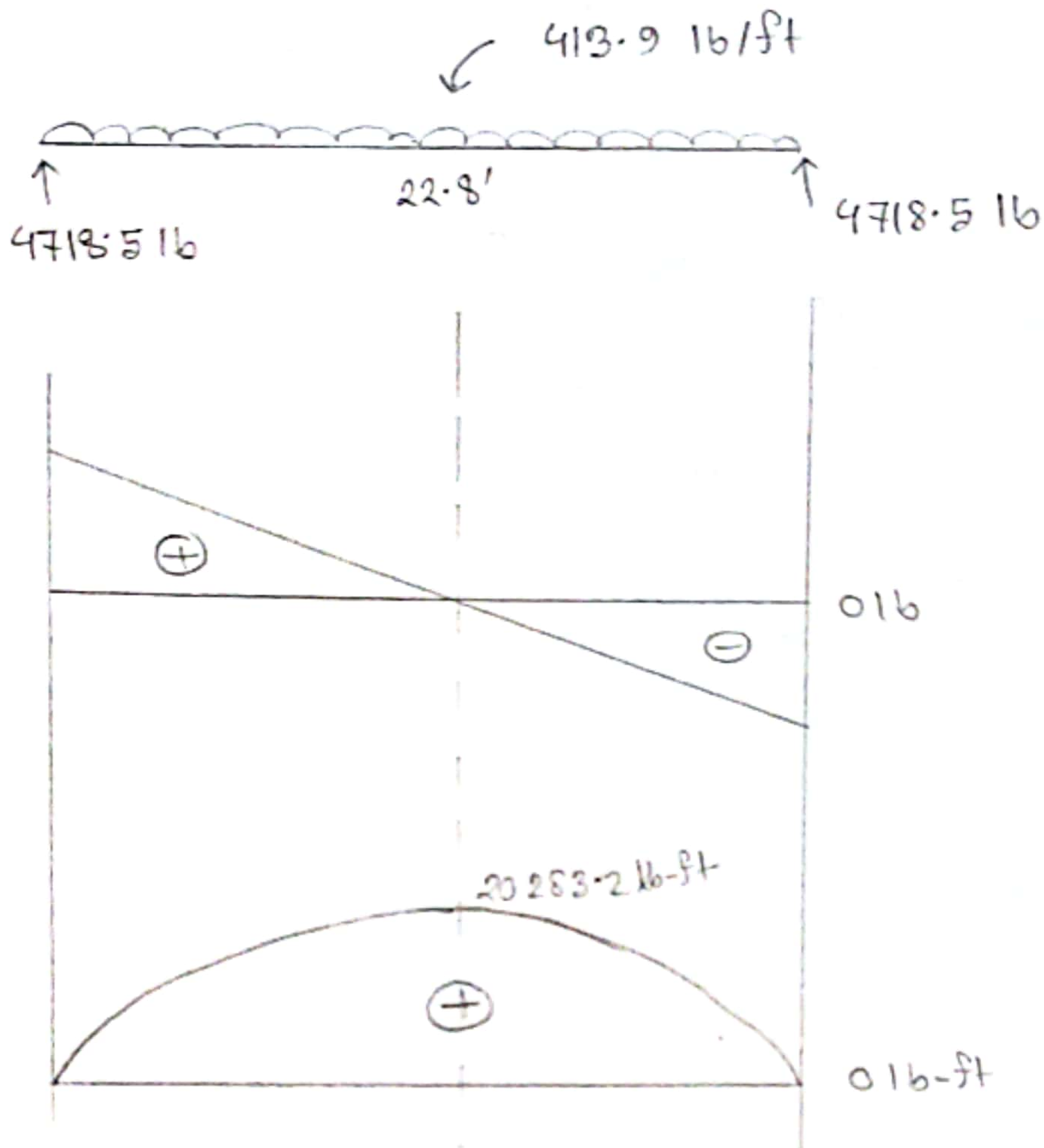
iii) Weight of rail $= \frac{6 \times 6}{12 \times 12} \times 150 \times 2 = 75$ lb/ft

$\therefore$ total dead load $= 300 + 38.88 + 75$

$= 413.9$ lb/ft

Reaction $R_A = R_B = \frac{413.9 \times 22.8}{2} = 4718.5$ lb.

Moment $= \frac{wL^2}{8} = \frac{413.9 \times 22.8^2}{8} = 26895.22$ lb-ft.



$V_{DL} = 4718.5$ lb

$M_{DL} = 20283.2$ lb-ft

Live load analysis:-

Live load moment $M_{LL} = 0.10\ PS$

$= 0.10 \times 16000 \times 22.8'$

$= 36480$ lb-ft

Live load shear, $V_{LL} = 0.3\ P$

$= 0.3 \times 16000$

$= 4800$ lb.

Design shear = Dead load shear + live load shear

$= 4718.5 + 4800$

$= 9518.5$ lb.

Design moment $= M_{DL} + M_{LL} = (20283.2 + 36480)$

$= 56763.2$ lb-ft

Depth Check:

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

$$= \sqrt{\frac{56.76 \times 12 \times 1000}{214.07 \times 12}} = 16.28''$$

$d_{eff} = t - c.c - \phi_s - \phi/2$

$d_{act} = 24'' - 3''$

$= 21'' > d_{req}$

Reinforcement Calculation:

$$A_s = \frac{M}{f_s j d} = \frac{56.76 \times 12000}{24000 \times 21 \times .889} = 1.52 \text{ in}^2$$

Provide 2 # 8 bar $= 2 \times .79 = 1.571$ in^2

Shear Check :-

$$V_d = \frac{1.15\, wL}{2}$$

$$= \frac{1.15 \times .414 \times 1000 \times 22.8}{2}$$

$$= 5427.54 \text{ lb}$$

$\therefore V_{allowed} = 1.1\sqrt{f_c'}\,bd = 1.1 \times \sqrt{4000} \times 12 \times 21$

$= 17531.67$ lb.

$\therefore V_{allowed} > V_d$; Hence No stirrup is required.

Working diagram :-

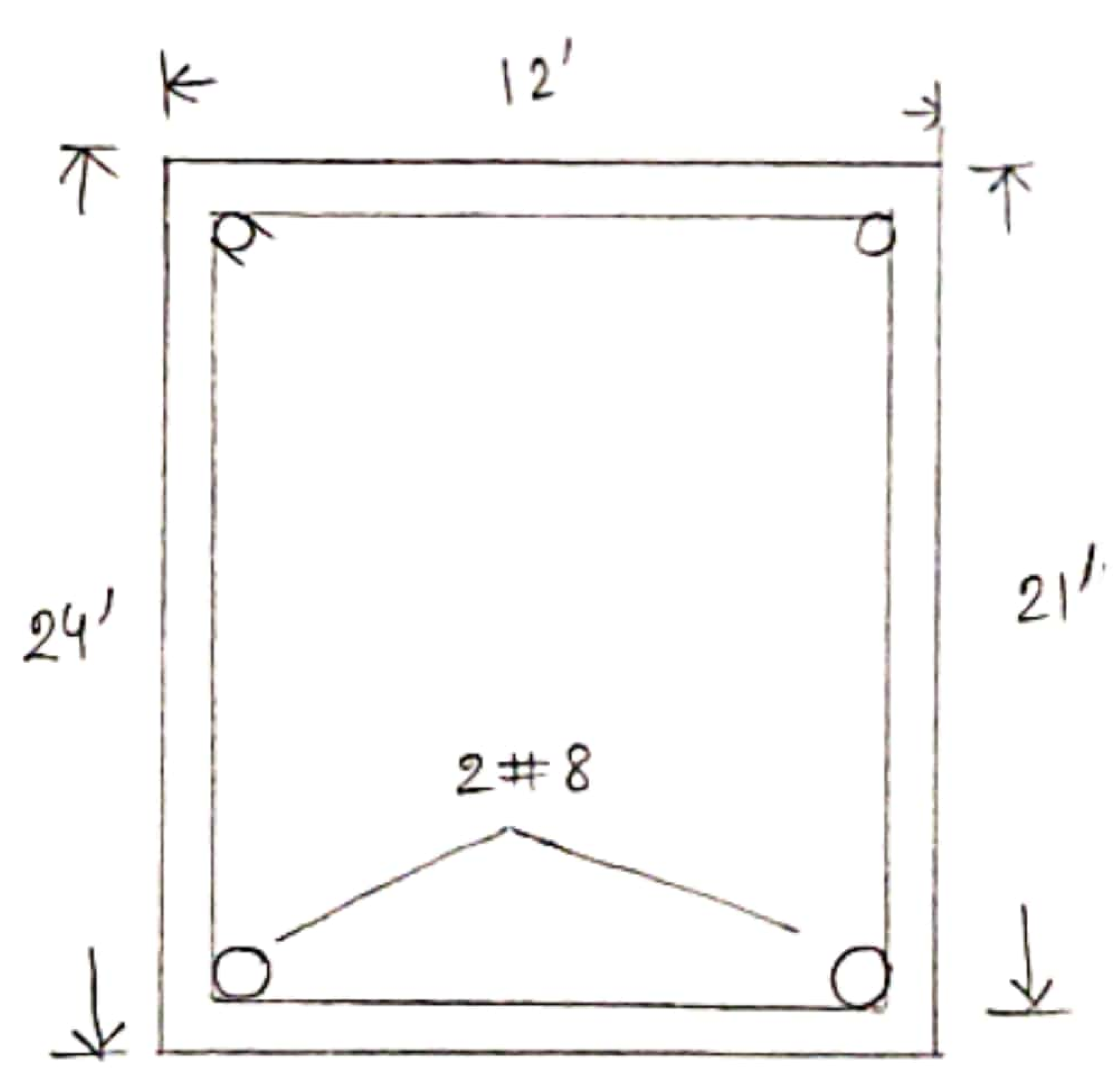


Fig: Cross section of edge beam.

Heaven's Light is Our Guide

Rajshahi University of Engineering & Technology
Rajshahi

DEPARTMENT OF
CIVIL ENGINEERING

Expt. No. 03

Name of Expt Design of a Deck Girder Bridge

SUBJECT : Reinforced Concrete Sessional -II	**SUBMITTED BY :**
COURSE NO. : CE- 3218	**NAME :** S.M. Abdullah Al Ahad
DATE OF EXPT.:	**CLASS :** 3rd year even semester
DATE OF SUB. :	**GROUP :** - **ROLL NO** 1600098
	SESSION : 2016-17

Design of a Deck Girder Bridge:-

Specification:

1) Span of bridge $= 35' + \frac{98}{4} = 59.5'$
2) Width of bridge $= 30' = 3$ lane
3) Loading system $= H_{15} S_{12}$
4) Maximum number of girder $= 5$
5) Thickness of girder (maximum) $= 15''$
6) $f_c' = 3000$ psi
7) $f_y = 50{,}000$ psi.

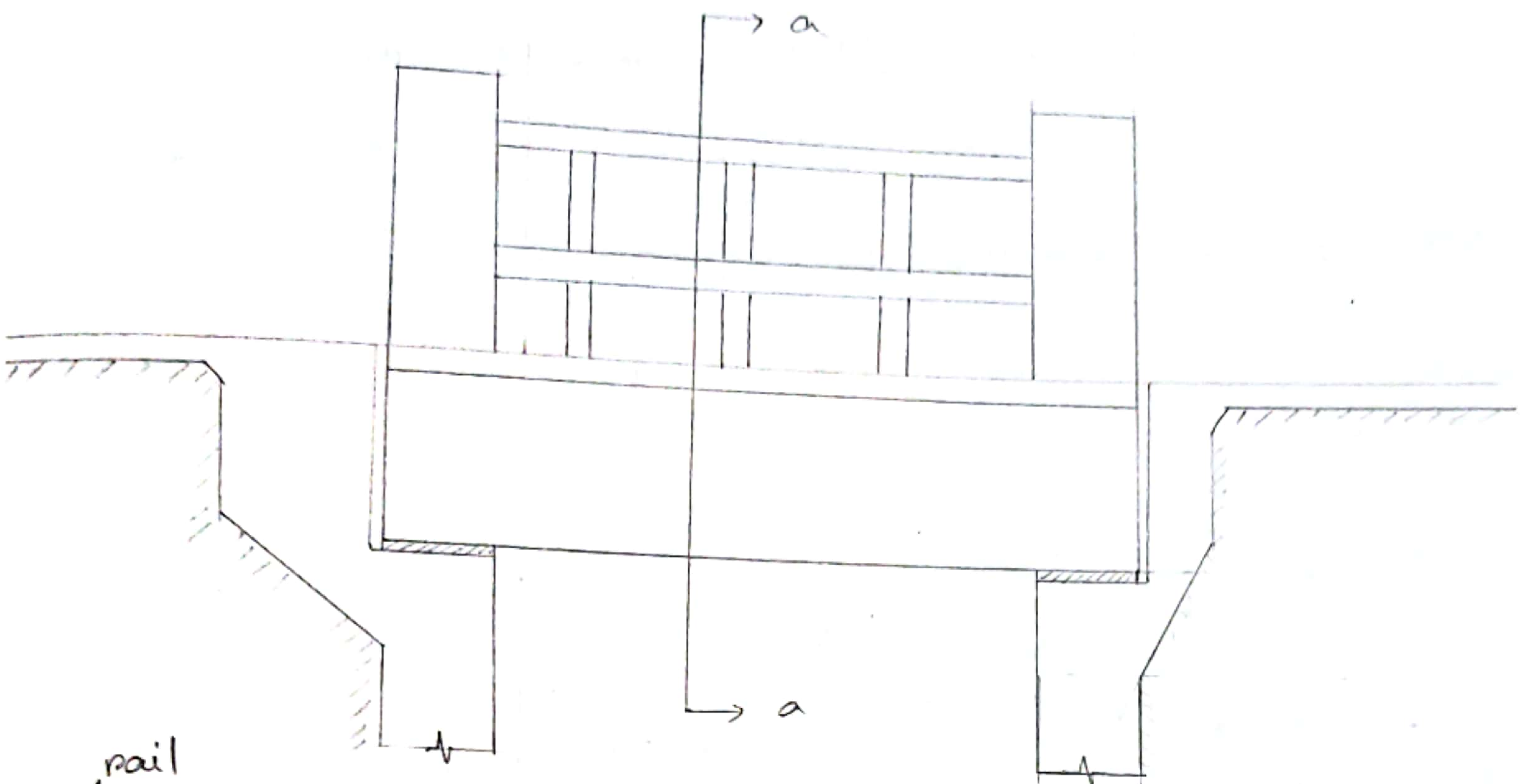


Fig: Elevation of a deck Girder Bridge.

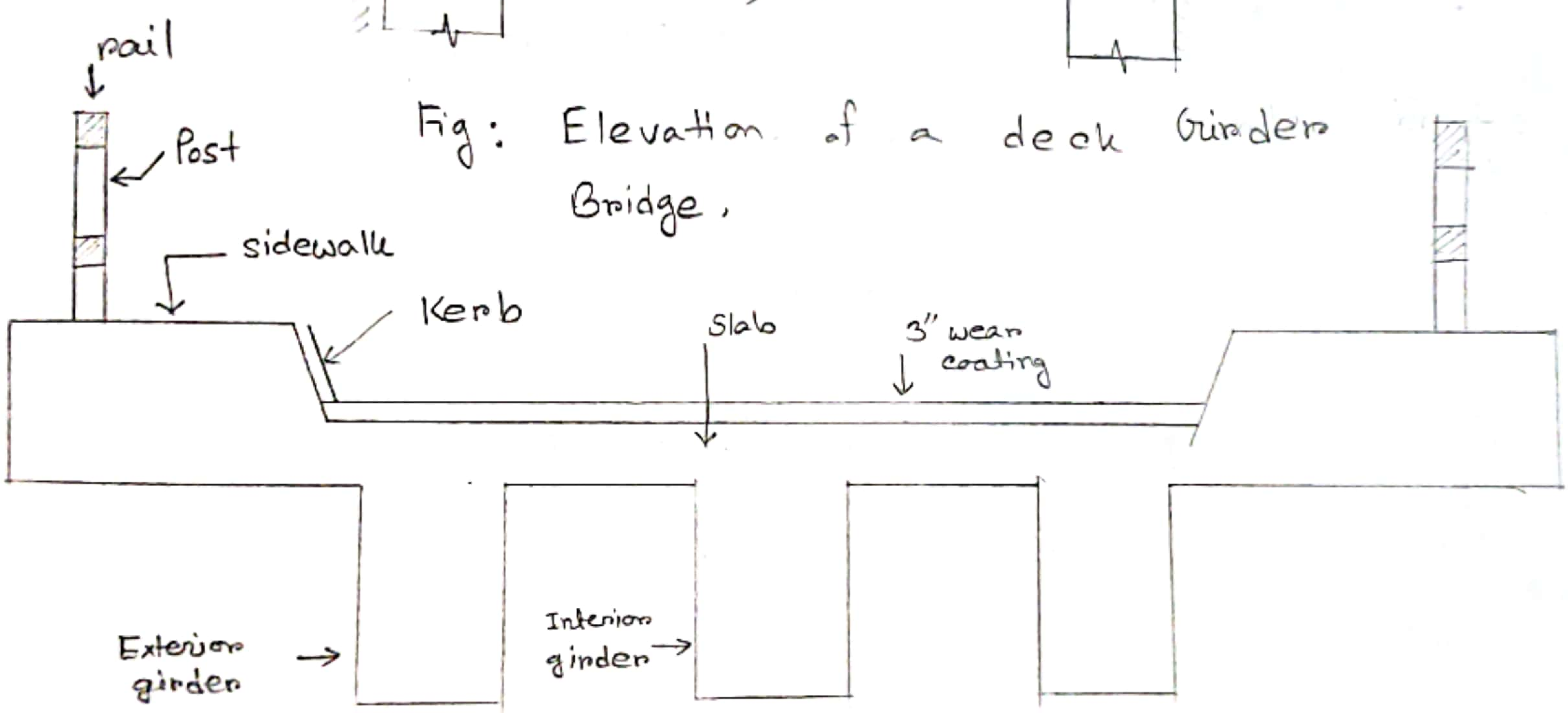


Fig: Cross section of a Deck girder Bridge

Calculations:-

1) span of the bridge $= 35' + \frac{98}{4} = 59.5'$

2) width of the bridge $= 30'$

3) Centre to Centre spacing of girder $= \frac{30}{5-1} = 7.5'$

4) Material properties,

$$n = \frac{E_s}{E_c} = \frac{29000,000}{57000\sqrt{3000}} = 9$$

$$p = \frac{f_s}{f_c} = \frac{50 \times .4}{3 \times 0.40} = 16.67$$

$$k = \frac{n}{n+p} = \frac{9}{9+16.67} = 0.38$$

$$j = 1 - \frac{0.38}{3} = 0.874$$

$$R = \frac{1}{2} f_c jk = \frac{1}{2} \times .45 \times 3000 \times 0.874 \times .38$$

$$= 185 \text{ psi}.$$

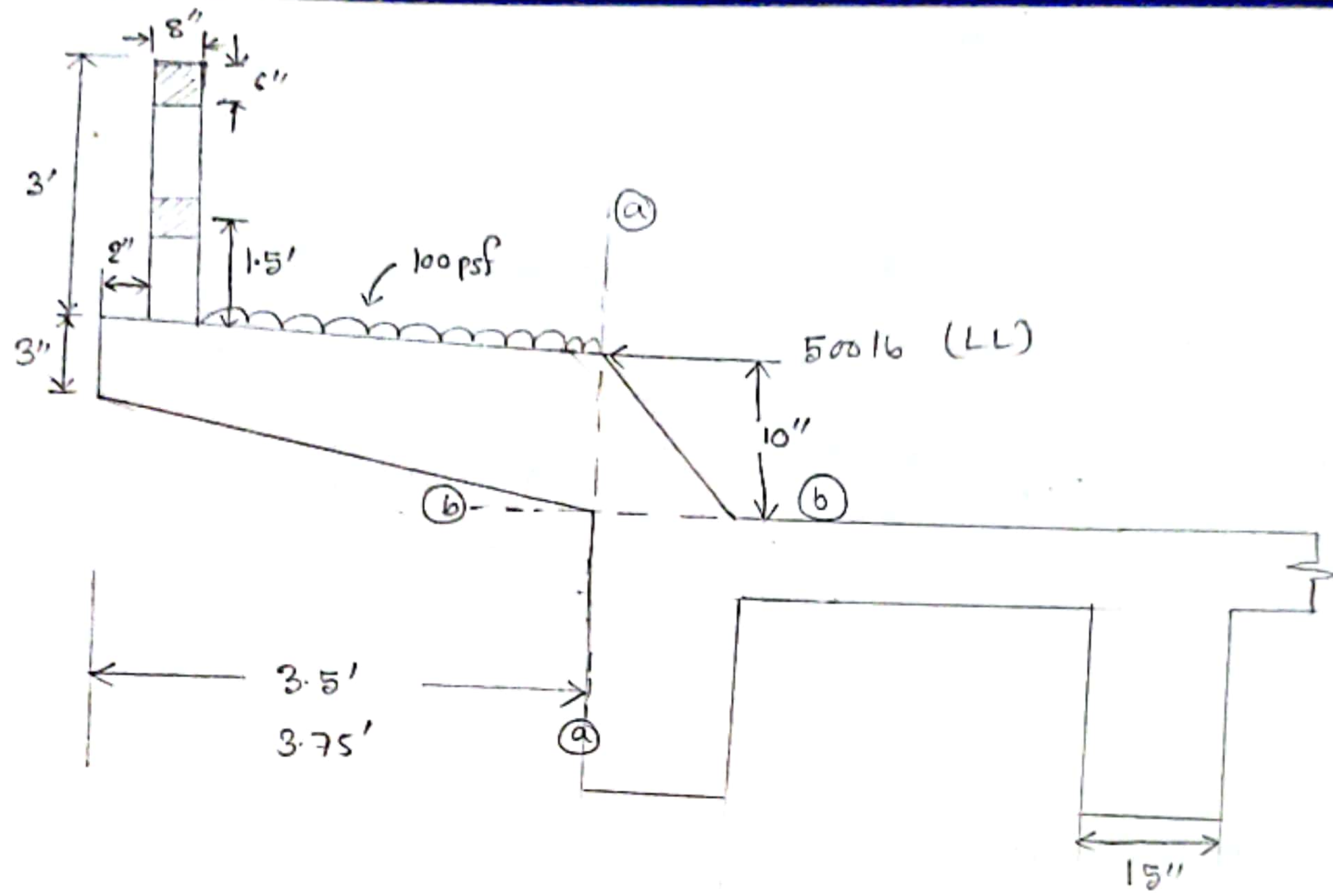


Fig: Cross section of a sidewalk and kerb.

Design of sidewalk:

Let us assume live load on sidewalk = 80 psf

Height of the post = 3'

Spacing of the post = 5.9"

Cross section = 8" x 8"

∴ Vertical load on post per foot of side walk

$$= \frac{8 \times 8}{144} \times 150 \times \frac{3}{5.9}$$

$$= 33.89 \ \text{lb/ft}$$

Weight of rail $= \frac{6 \times 6}{144} \times 150 \times 2 = 75$ psf

Let;

Vertical live load on post = 100 psf

Total dead load = 33.89 + 75 = 108.89 plf

Total load = dead load + live load

= 108.89 + 100

= 208.89 plf

Moment calculation:-

Moment arm for rail and post from the section (a)-(a)

$= 3.5' - \left(\frac{8''}{2 \times 12} + \frac{2'}{12}\right) = 3'$

Now; horizontal load for upper rail = 150 lb/ft

horizontal load for lower rail = 300 lb/ft

Moment arm for upper rail from section (b)-(b)

$= 3' + \frac{10''}{12} - \frac{3}{12}$

$= 3'7''$

$= 3.58'$

Moment arm for lower rail from sec (b) – (b)

$$= \frac{10}{12} + 1.5'$$

$$= 2.33'$$

Now; Self weight of side walls $= \frac{3}{12} \times 3.5 \times 150 + \frac{1}{2} \times 3.5 \times \frac{7}{12} \times 150$

$$= 284.375 \text{ plf}$$

Moment arm for rectangular portion from section (a) – (a)

$$= \frac{3.5}{2} = 1.75'$$

moment arm for triangular portion from section (b) - (b)

$$= \frac{3.5}{3} = 1.67'$$

Live load calculation:

Total live load $= 80 \times \left(3.5 - 8/12 - 2/12\right)$

$$= 80 \times 2.67$$

$$= 213.3 \text{ lb/ft}$$

Moment arm from section ⓐ - ⓐ $= \frac{3.5 - 8/12 - 2/12}{2} = 1.33'$

Moment calculation:

Total vertical load = 208.89 lb/ft

Total moment about (a) - (a)

$M_{a-a} = 208.89 \times 3 + (3/12 \times 3.5 \times 150) \times 1.75 + (1/2 \times 3.5 \times 7/12 \times 150) \times 1.67 + (213.3 \times 1.33)$

$= 1395.77$ lb/ft

Total moment about (b) - (b) $= 190 \times 3.58 + 300 \times 2.38 + 500 \times 10/12$

$= 1652.67$ lb-ft

∴ Total moment $= (1395.77 + 1652.67)$ lb-ft

$= 3048.44$ lb-ft.

Depth Check :-

$$d_{req} = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{3048.44 \times 12}{184.8 \times 12}} = 4.06''$$

$$d_{actual} = 10'' - (\text{clear cover} + \tfrac{1}{2}\text{dia})$$

$$= 10 - (1.5 + \tfrac{1}{2} \times 4/8)$$

$$= 8.25 > d_{req}$$

So, design is ok.

Reinforcement calculation:

$$A_s = \frac{m}{f_s j d} = \frac{3048.44 \times 12}{20{,}000 \times 0.88 \times 8.25} = 0.25 \text{ in}^2$$

Provide # 3 bars @ $\frac{0.11 \times 12}{.25} = 5.28 \approx 5''$ c/c

Distribution & temperature reinforcement

$$A_{s\,min} = 0.0020\, bt$$

$$= 0.0020 \times 12 \times 10$$

$$= 0.24 \text{ in}^2$$

Use # 3 bar @ $\frac{0.11 \times 12}{.24} = 5.5''$ c/c

Shear check :-

$V_{allowable} = 1.1\sqrt{f_c'} = 1.1\sqrt{3000} = 60.25$ psi

shear about section (a) - (a) :—

V_{a-a} = total vertical load = 208.89 + 284.375 + 213.3 + 100 × (3.5 − 8/12 − 2/12)

= 973.23 lb.

$\therefore v_{a-a} = 1.15 \dfrac{V_{a-a}}{bd} = 1.15 \times \dfrac{973.23}{12 \times 8.25} = 11.3$ psi

$v_{a-a} < v_{all}$ (ok)

Shear about section (b) - (b) :—

$V_{b-b} = 150 + 300 + 500 = 950$ lb

$v_{b-b} = 1.15 \times \dfrac{950}{12 \times 8.25} = 11.03$

$v_{b-b} < v_{all}$ (ok)

bond check :-

$$\text{Allowable bond stress} = \frac{3.4\sqrt{f_c'}}{D}$$

$$= \frac{3.4 \times \sqrt{3000}}{3/12}$$

$$= 744.9 \text{ psi}$$

$$V_{available} = \frac{V_{max}}{\Sigma o \, jd}$$

$$= \frac{950}{\pi \times 3/12 \times \frac{12}{5} \times .88 \times 8.25}$$

$$= 69.42 \text{ psi}$$

u available < u allowable .

[OK]

Heaven's Light is Our Guide

Rajshahi University of Engineering & Technology
Rajshahi

DEPARTMENT OF CIVIL ENGINEERING

Expt. No. 04

Name of Expt .... Design of Abutment ....

....

SUBJECT : Reinforced Concrete Sessional II	SUBMITTED BY :
COURSE NO. : CE 3218	NAME : S.M. Abdullah Al Ahad
DATE OF EXPT.:	CLASS : 3rd year even Semester.
DATE OF SUB. :	GROUP : – ROLL NO 1600098
	SESSION : 2016-17.

Design of Abutment

Specifications:

1. Overall height of abutment = 12 ft.
2. Unit weight of soil = 120 pcf
3. Angle of internal friction, $\phi = 30^\circ$
4. Friction co-efficient = 0.5
5. Bearing capacity of soil, $P_a = 4$ ksf.

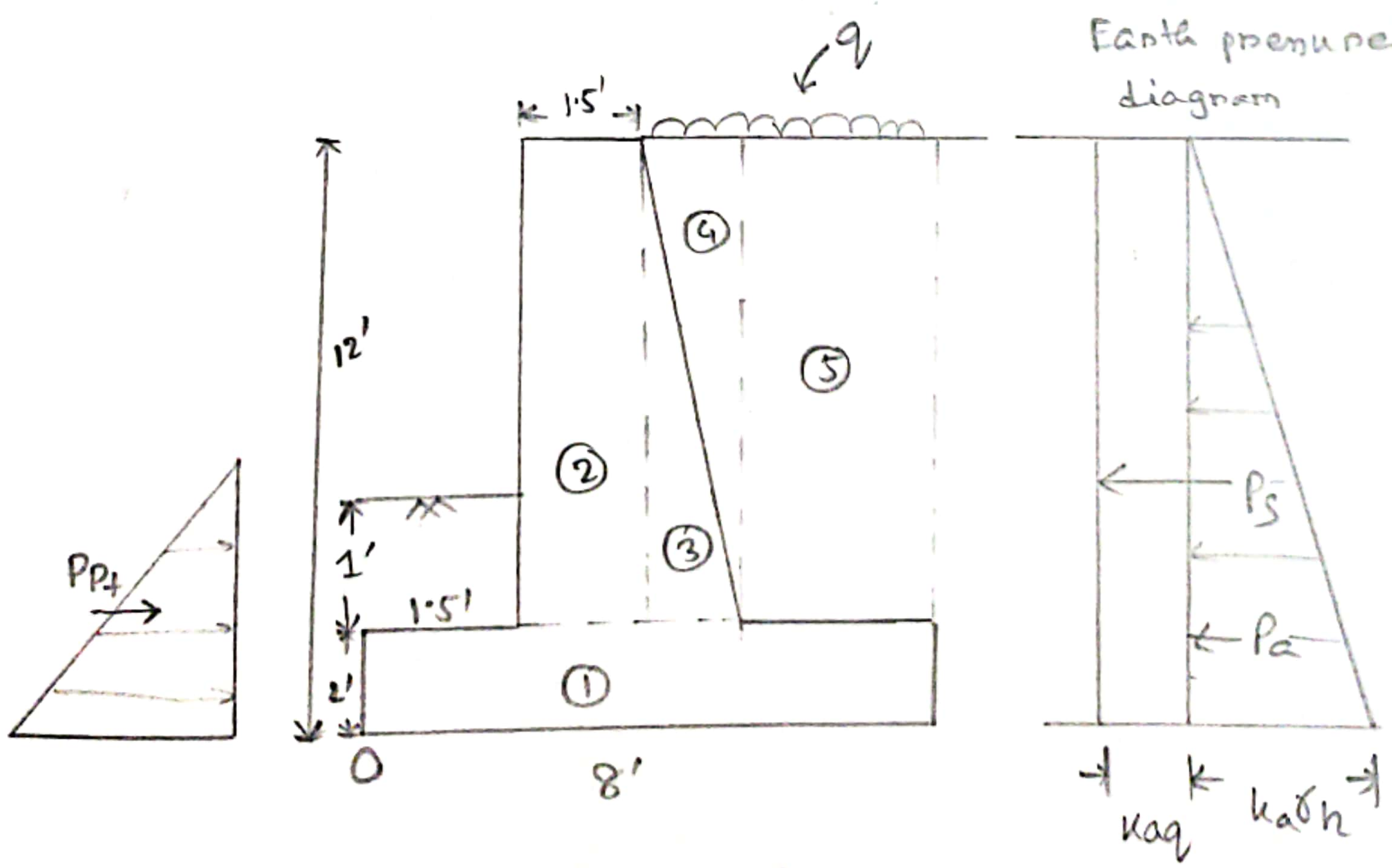


Fig: Abutment of a slab bridge

Calculation:-

$$K_a = \frac{1-\sin\phi}{1+\sin\phi} = 0.33$$

$$K_p = 1/K_a = 3$$

$$P_a = \frac{1}{2} K_a \gamma h^2 = \frac{1}{2} \times .33 \times 120 \times (12)^2$$
$$= 2851.2 \text{ lb.}$$

Active earth pressure due to surcharge

$$P_s = K_a \times q_s \times h$$
$$= .33 \times 300 \times 12$$
$$= 1188 \text{ lb.}$$

Total active pressure (Sliding force)

$$H_a = P_a + P_s$$
$$= (2851.2 + 1188) \text{ lb.}$$
$$= 4040 \text{ lb.}$$

Over turning moment, $M_o = (P_a \times h/3) + (P_s \times h/2)$

$= (2851.2 \times 12/3) + (1188 \times 12/2)$

$= 18532$ lb-ft.

Calculation of resisting moment :-

Section	Weight (lb)	Moment Arm	Moment, M_R (lb-ft)
I	$8 \times 2 \times 150 = 2400$	$8/2 = 4$	9600
II	$1.5 \times 10 \times 150 = 2250$	$1.5 + \frac{1.5}{2} = 2.25$	5062.5
III	$\frac{1}{2} \times 2.5 \times 10 \times 150 = 1875$	$1.5 + 1.5 + \frac{2.5}{3} = 3.83$	7181.25
IV	$\frac{1}{2} \times 2.5 \times 10 \times 120 = 1500$	$1.5 \times 1.5 + \frac{2 \times 2.5}{3} = 4.67$	7005
V	$2.5 \times 10 \times 120 = 3000$	$1.5 + 4 + \frac{2.5}{2} = 6.75$	20250
	$\Sigma W = 11025$ lb		$\Sigma M_R = 49098.75$ lb-ft

Check for sliding:-

Factor of safety $= \frac{W \times f}{P_a} = \frac{11025 \times 0.5}{2851.2} = 1.93 > 1.5$

Design is OK.

☐ Check against over turning:

$$\text{Factor of safety} = \frac{M_R}{M_o}$$

$$= \frac{49098}{11404.8}$$

$$= 4.3 > 1.5$$

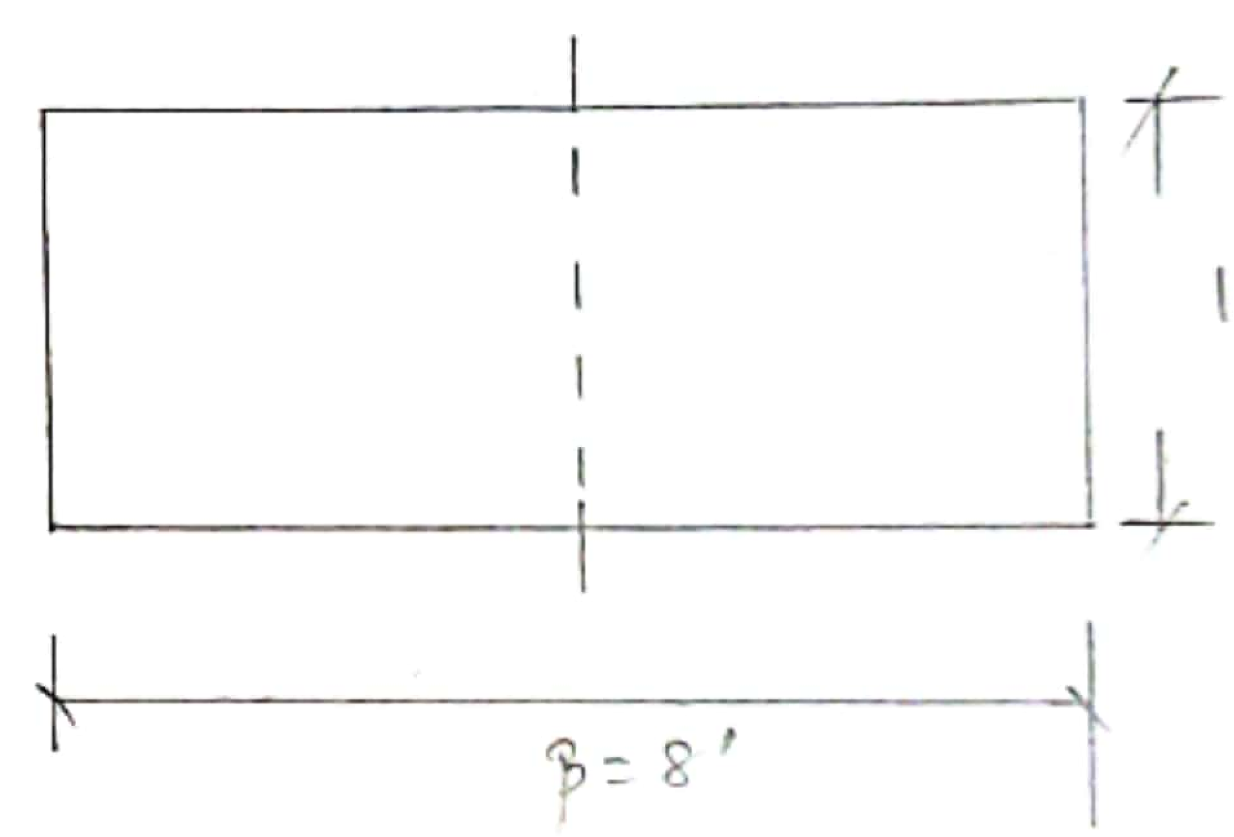
Design is okay.

☐ Check for bearing capacity:

Here; $A = 8 \times 1 = 8 \ ft^2$

$$I = \frac{1 \times 8^3}{12} = 42.67 \ ft^2$$

$$c = B/2 = 8/2 = 4 \ ft.$$



location of resultant force

from O; $a = \frac{M_R - M_o}{\omega}$

$$= \frac{37693.2}{11025}$$

$$= 3.41$$

eccentricity, $e = B/2 - a = 4 - 3.41$

$$= 0.59 \ ft$$

Now; $M = \omega \times e = 11025 \times 0.59 = 6504.75$ lb-ft.

Now; $\sigma_1 = \frac{w}{A} + \frac{me}{I}$

$= \frac{11025}{8} + \frac{6504.75 \times 4}{42.67}$

$= 1378.125 + 609.77$

$= 1987.89 < 4000$ psf.

& $\sigma_2 = \frac{w}{A} - \frac{me}{I} = \frac{11025}{8} - \frac{6504.75 \times 4}{42.67}$

$= 1378.125 - 609.77$

$= 768.35 < 4000$ psf

$\therefore$ Design is okay.

Heaven's Light is Our Guide

Rajshahi University of Engineering & Technology
Rajshahi

DEPARTMENT OF CIVIL ENGINEERING

Expt. No. 05

Name of Expt: Design of slab of a deck girder bridge

SUBJECT : Reinforced Concrete Sessional II.	SUBMITTED BY :
COURSE NO. : CE-3128	NAME : S.M. Abdullah Al Ahad
DATE OF EXPT.:	CLASS : 3rd year even semester.
DATE OF SUB. :	GROUP : – ROLL NO 1600398
	SESSION : 2016-17

Design of slab of a deck girder bridge :-

we consider the slab perpendicular to the traffic

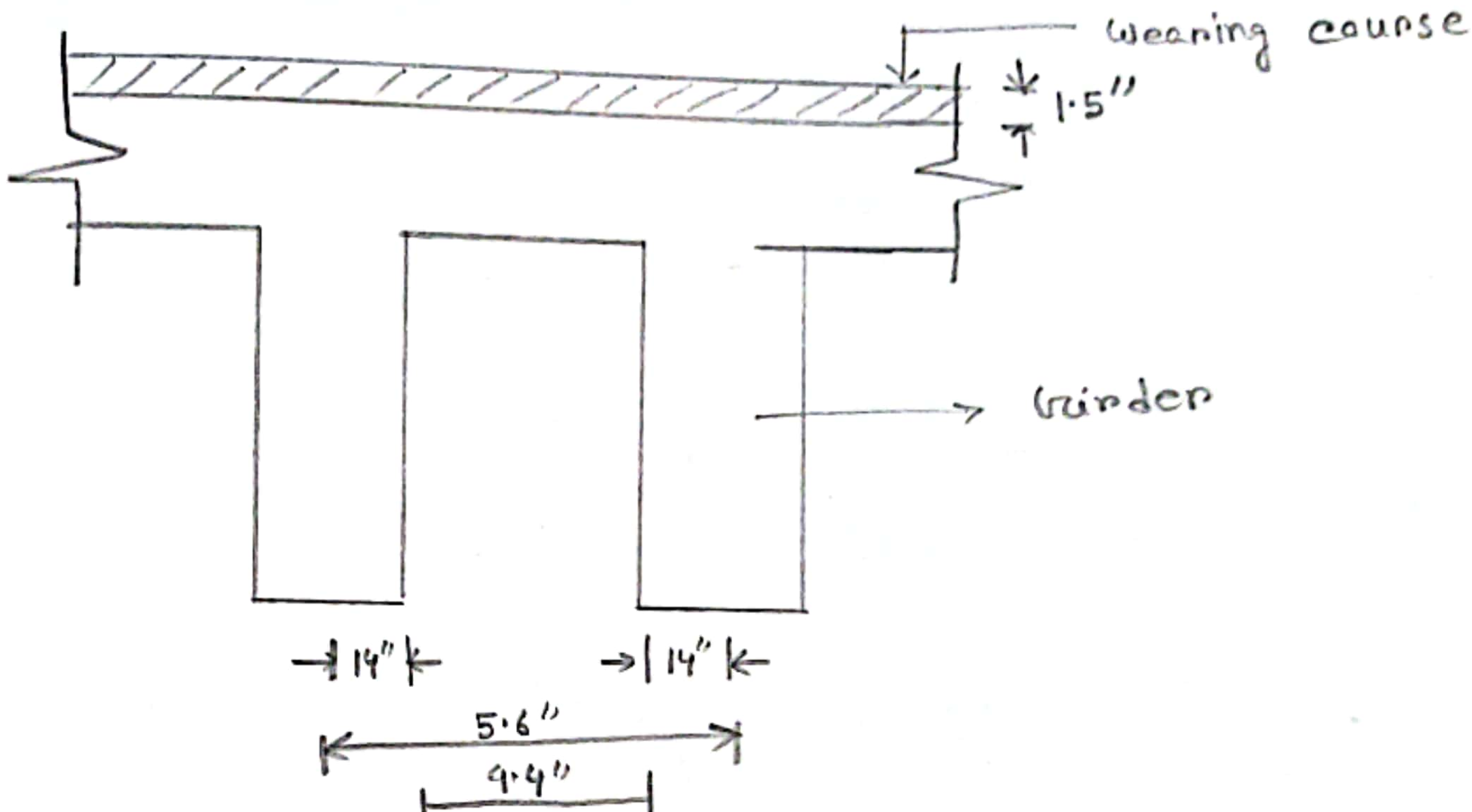


Fig: Cross section of a portion of a bridge

(i) Load calculation:-

(a) Dead load for self weight $= \frac{t}{12} \times 150 = (\frac{6}{12} \times 150)$ psf $= 75$ psf

(b) Dead load for wearing coarse $= 15$ psf

$\therefore$ Total dead load $= (75+15) = 90$ psf

ii) Moment calculation:-

a) DL moment $= \frac{wL^2}{10} = \frac{90 \times 4.33^2}{10} = 168.74$ lb-ft.

b) LL moment (for main reinforcement perpendicular to the traffic)

$M_{LL} = 0.8 \times \frac{(S+2)}{32} \times P_{15} = \frac{.8 \times (4.33+2) \times 12,000}{32}$

$= 1899$ lb-ft.

(c) Impact co-efficient, I.C $= \frac{50}{L+125} \leq 0.3$

$$\frac{50}{4.33+125} = \frac{50}{4.33+125} = 0.386 \sim 0.3$$

Impact moment, $M_I = M_{LL} \times I = 1899 \times 0.3$ lb-ft
$= 570$ lb-ft

Design moment:-

Design moment $= M_{DL} + M_{LL} + M_I = 168.74 + 1899 + 570$
$= 2637.44$ lb-ft

Depth check:-

$$d_{req} = \sqrt{M/Rb} = \sqrt{\frac{2637.44 \times 12}{223.16 \times 12}} = 3.43''$$

here; $n=9$, $r = 14.81$, $K = 0.38$, $J = 0.87$, $R = 223.16$

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

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

So, design is ok.

Main Reinforcement Calculation:-

$$A_s = \frac{M}{f_s j d} = \frac{2637.44 \times 12}{20,000 \times 0.87 \times 5} = 0.363 \text{ inch}^2$$

$$\text{Spacing} = \frac{0.20 \times 12}{0.363} = 6.61'' \sim 6.5''$$

Use # 4 bar @ 6.5" c/c.

Distribution reinforcement Calculation :-

For main reinforcement perpendicular to traffic.

$$A_{st} = \frac{220}{\sqrt{S}} < 67\% \text{ of main reinforcement. (min)}$$

$$= \frac{220}{\sqrt{4.33}} = 105.72 \sim 67\%$$

$$= 0.67 \times 0.363 \text{ in}^2 = 0.243 \text{ in}^2$$

Use # 3 bars at $\frac{0.11 \times 12}{0.243} = 5.42'' \sim 5''$ c/c

Shear check :-

$$V_{max} = 1.15 \frac{wL}{2} = 1.15 \times \frac{90 \times 4.33}{2} = 224.1 \text{ lb}$$

$$V_d = \frac{V_{max}}{bd} = \frac{224.1}{12 \times 5} = 3.73 \text{ psi}$$

$$V_{all} = 1.1\sqrt{f_c'} = 60.25 \text{ psi.}$$

$\therefore$ $V_d < V_{all}$. So, No need of stirrup. (OK)

Bond check :-

$$V_d = \frac{V_{max}}{\Sigma_o jd} = \frac{224.1}{3.14 \times 4/8 \times 12/7 \times .87 \times 5} = 19.13 \text{ psi.}$$

$$V_{all} = \frac{1.7\sqrt{f_c'}}{D} = \frac{1.7\sqrt{3000}}{4/8} = 186.225 \text{ psi.}$$

$\therefore V_{all} > V_d$. [OK]

Development length :-

$$L_d = \frac{f_s D}{4 v_d} = \frac{20,000 \times 4/8}{4 \times 19.13} = 130.7''$$

Heaven's Light is Our Guide

Rajshahi University of Engineering & Technology
Rajshahi

DEPARTMENT OF CIVIL ENGINEERING

Expt. No. 06

Name of Expt Design of interior girder of a deck girder bridge

SUBJECT : Reinforced Concrete Sessional II

COURSE NO. : CE-3218

DATE OF EXPT.:

DATE OF SUB. :

SUBMITTED BY :

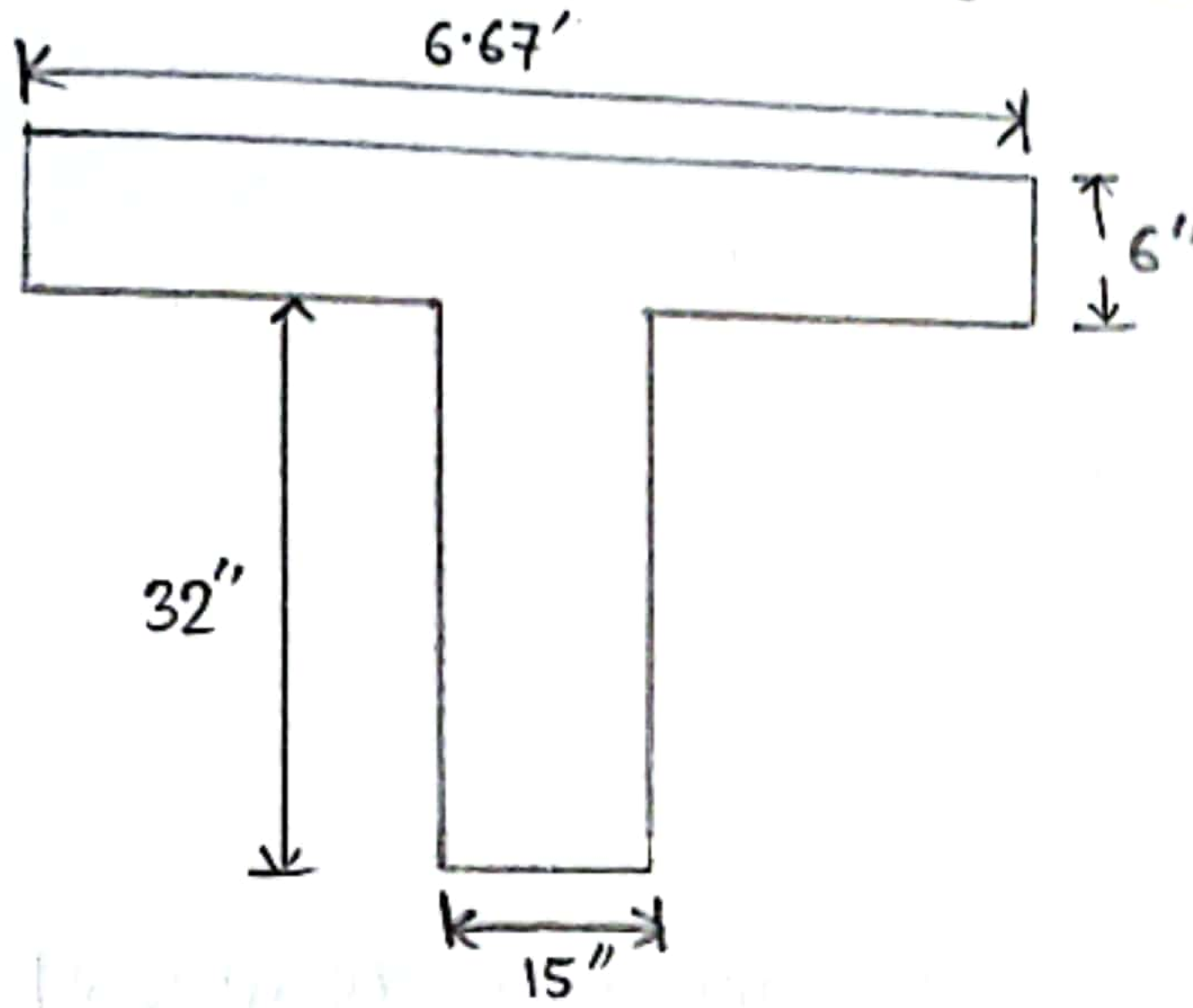
NAME : S.M. Abdullah Al Ahad

CLASS : 3rd year even semester

GROUP : .... ROLL NO 1600098

SESSION : 2016-17

Design of interior girder of a deck girder bridge:-



Let, slab thickness = 6"

Effective span length $= 35 + \frac{98}{4} + 1 + 1 = 61.5'$

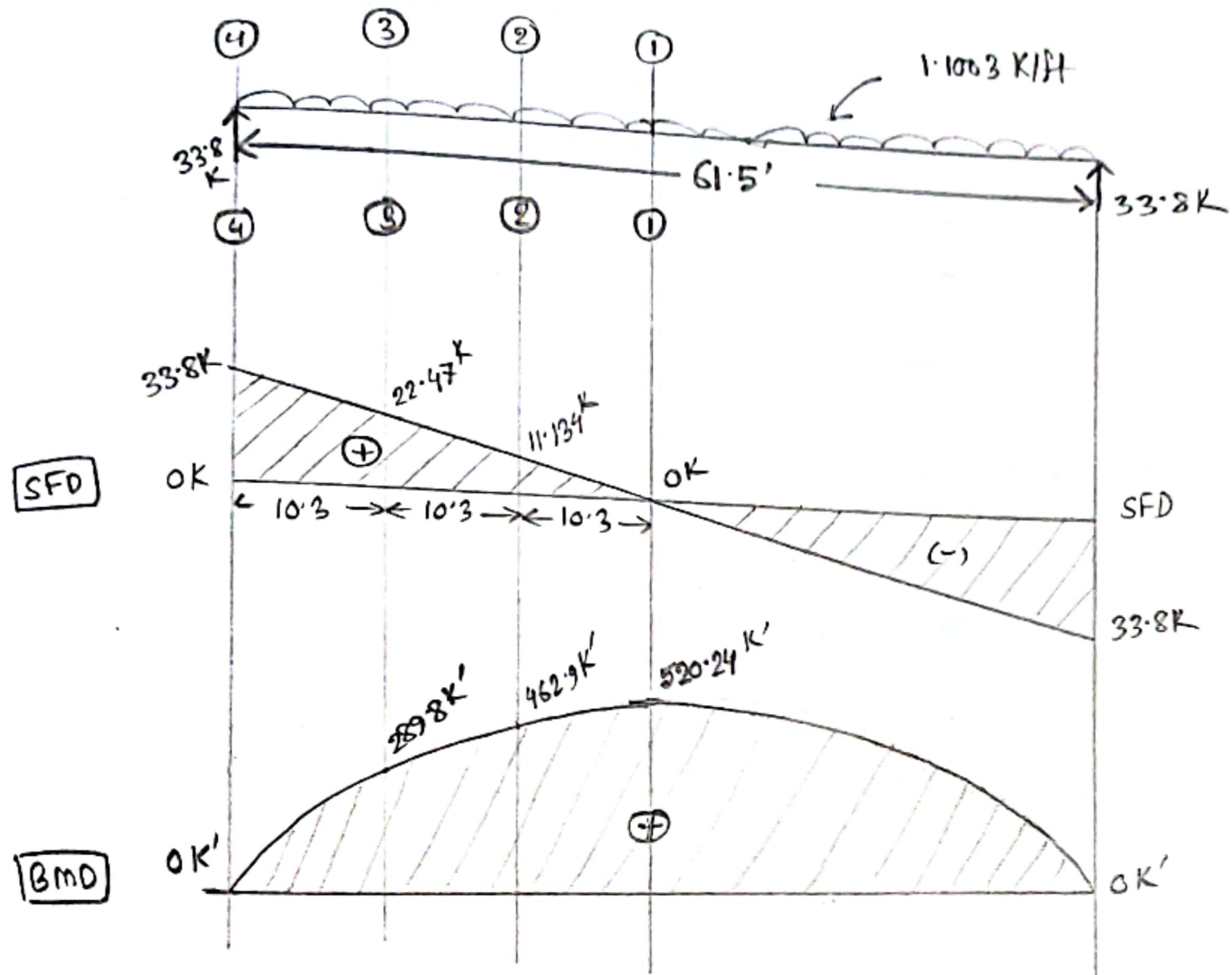
The cross section below the slab = 15" X 32"

i) Dead load for slab or weight of slab = 90×6.67 = 600.3 plf

ii) Dead load for girder or weight of girder = $\frac{15 \times 32}{144} \times 150$ = 500 plf

$\therefore$ Total load = (600.3 + 500) = 1100.3 plf = 1.1003 K/ft

Dead load shear and moment diagram:-



Dead load shear:-

$V_{1-1} = 0K$

$V_{2-2} = 11.134^K$

$V_{3-3} = 22.47K$

$V_{4-4} = 33.8K$ (maximum)

Dead load moment:-

$m_{1-1} = 520.24K'$ (max)

$m_{2-2} = 462.9K'$

$m_{3-3} = 289.8K'$

$m_{4-4} = 0K'$

Live load moment:- Effective wheel load formula : One traffic lane $S/6$, two or more $= S/5$.

According to AASHTO, $S = 5.00$ [if lane is greater than 2]

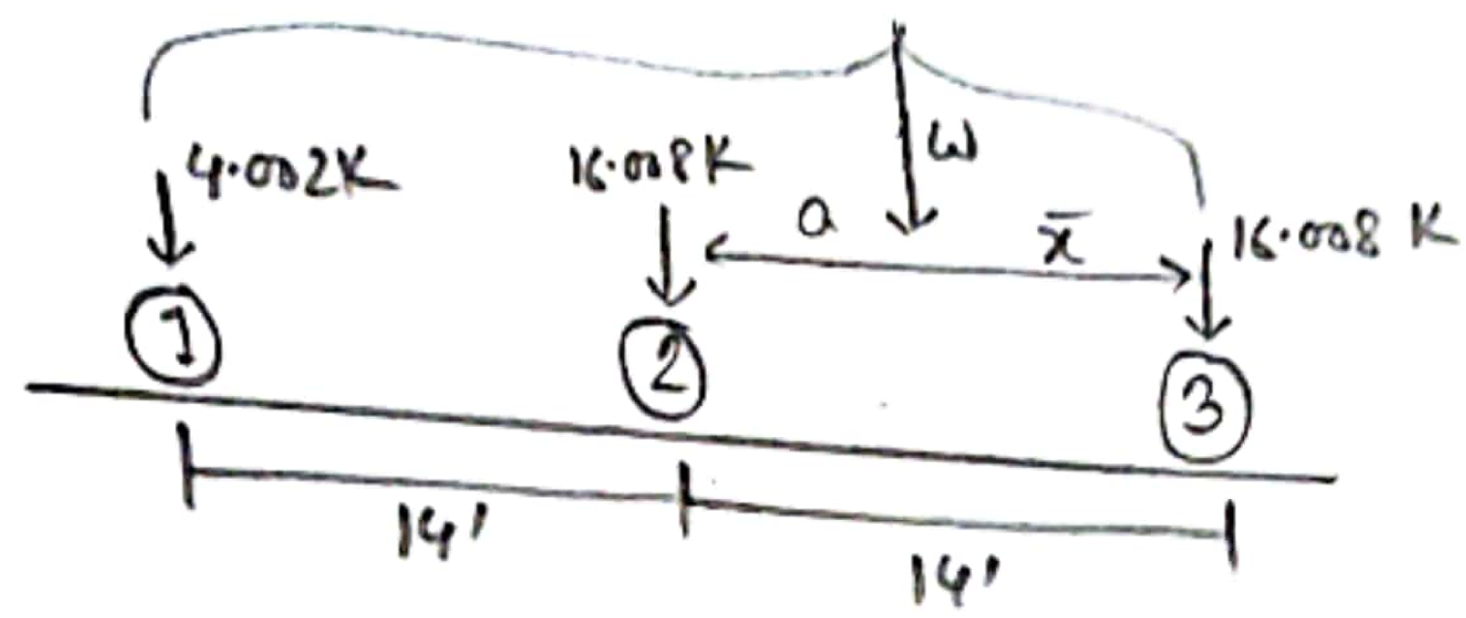
$\therefore$ Each interior girder must support $\frac{6.67}{5.00} = 1.334$ wheel loads per wheel

The load from front wheel $= 3 \times \frac{6.67}{5} = 4.002K$

The load from rare wheel $= 12 \times \frac{6.67}{5} = 16.008K$

Absolute moment (max) :-

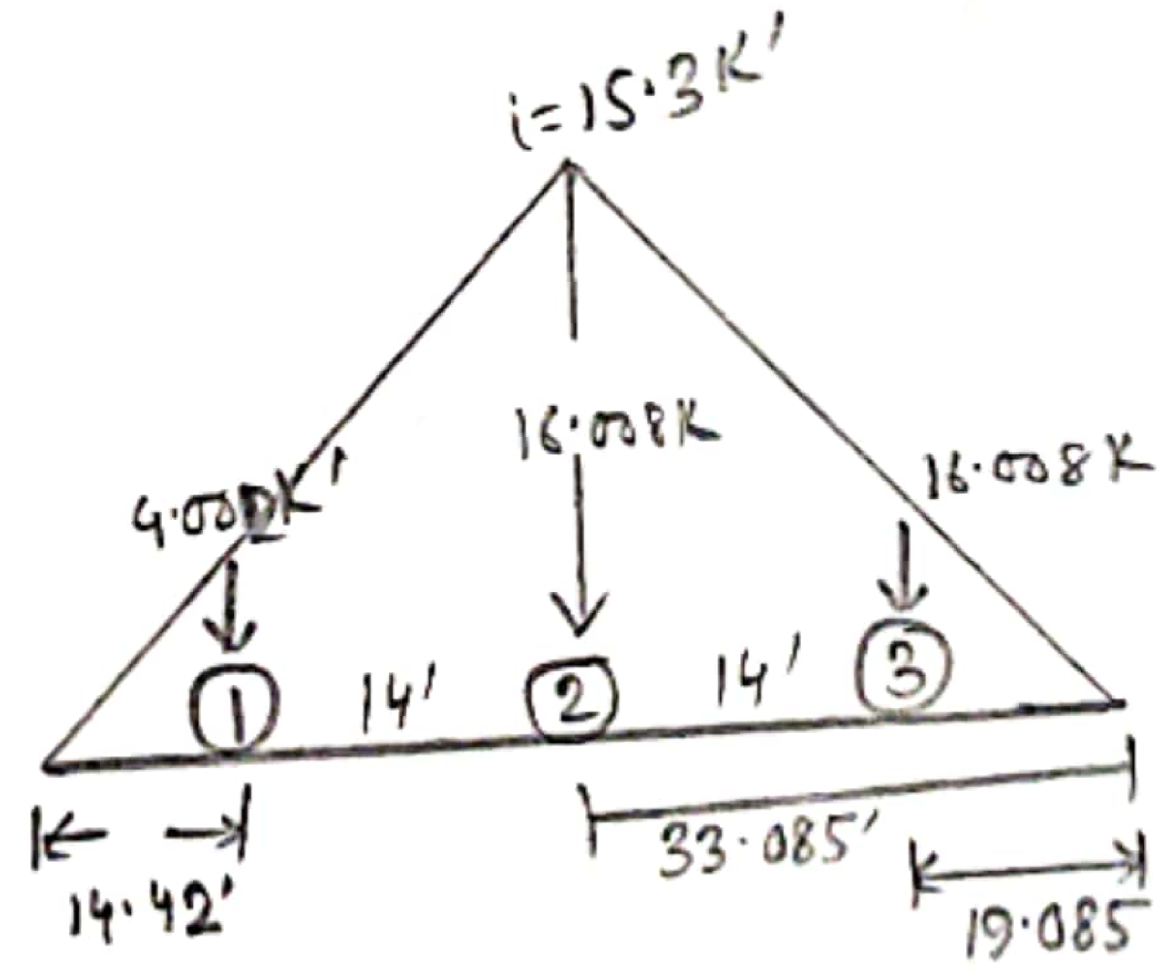
$$\bar{x} = \frac{16.008 \times 14 + 4.002 \times 28}{36.018} = 9.33'$$



Centre of the gravity of the wheel from the right support is 9.33'.

Distance between wheel 2 and the centre of the gravity of the loads $a = 14 - \bar{x} = 4.67'$

The positive of wheel for maximum absolute moment from right corner $= \frac{L}{2} + \frac{a}{2} = \frac{61.5}{2} + \frac{4.67}{2} = 33.085'$

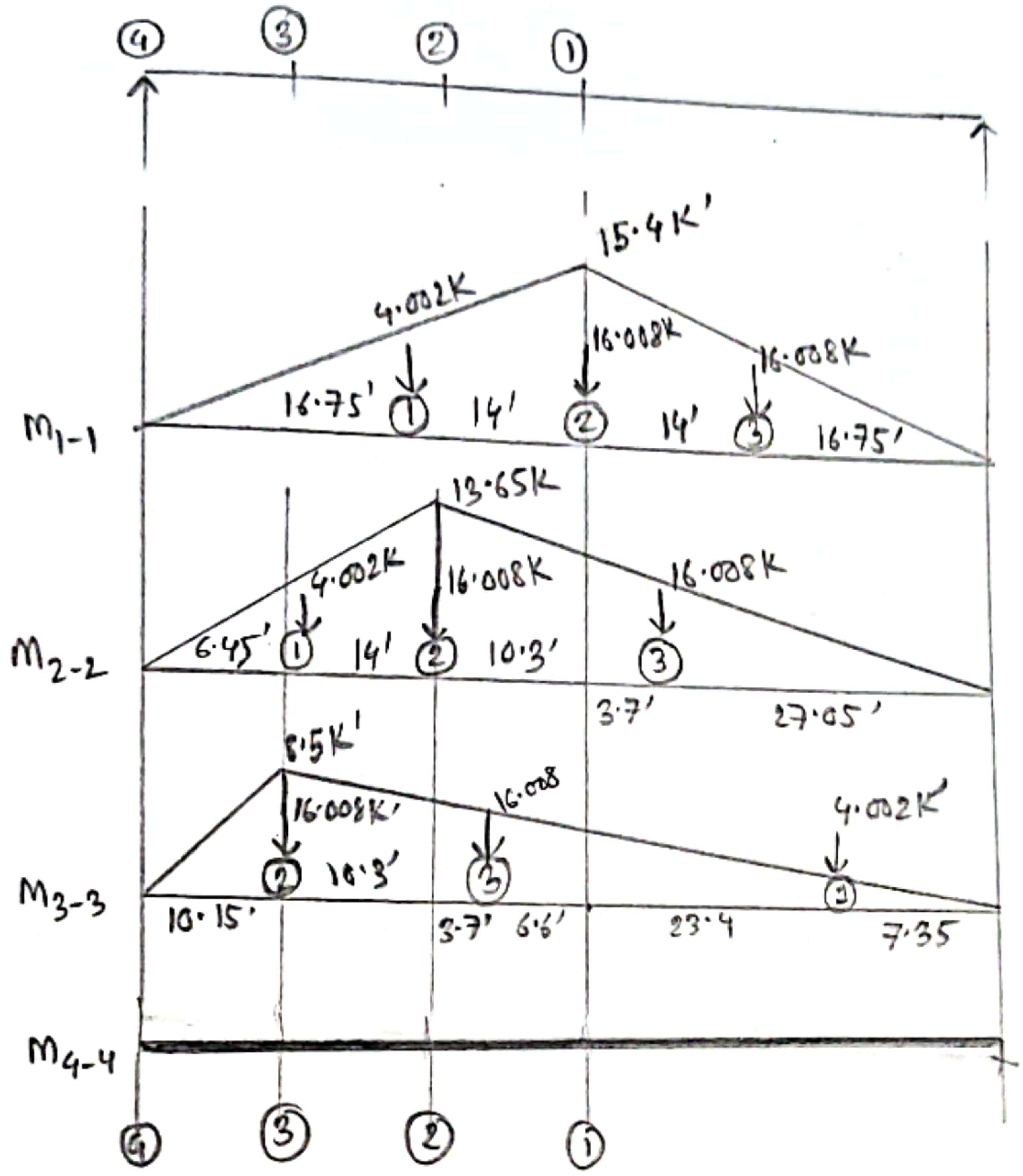


$$M_{abs} = \frac{15.3}{33.085}\left[(16.008 \times 19.085) + (16.008 \times 33.085)\right] + \frac{15.3}{28.42}\left[4.002 \times 14.42\right]$$

$$= 0.46 \times 835.14 + 31.1$$

$$= 415.3 \text{ Kft.}$$

live load moment :-



$$M_{1-1} = \frac{15.4}{30.75}\left[(16.008 \times 16.75) + (16.008 \times 30.75)\right] + \frac{15.4}{30.75}\left[4.002 \times 16.75\right]$$

$$= 414.21\ K'$$

$$M_{2-2} = \frac{13.65}{41.05}\left[(16.008 \times 27.05) + (16.008 \times 41.05)\right] + \frac{13.65}{20.45}\left[4.002 \times 6.45\right]$$

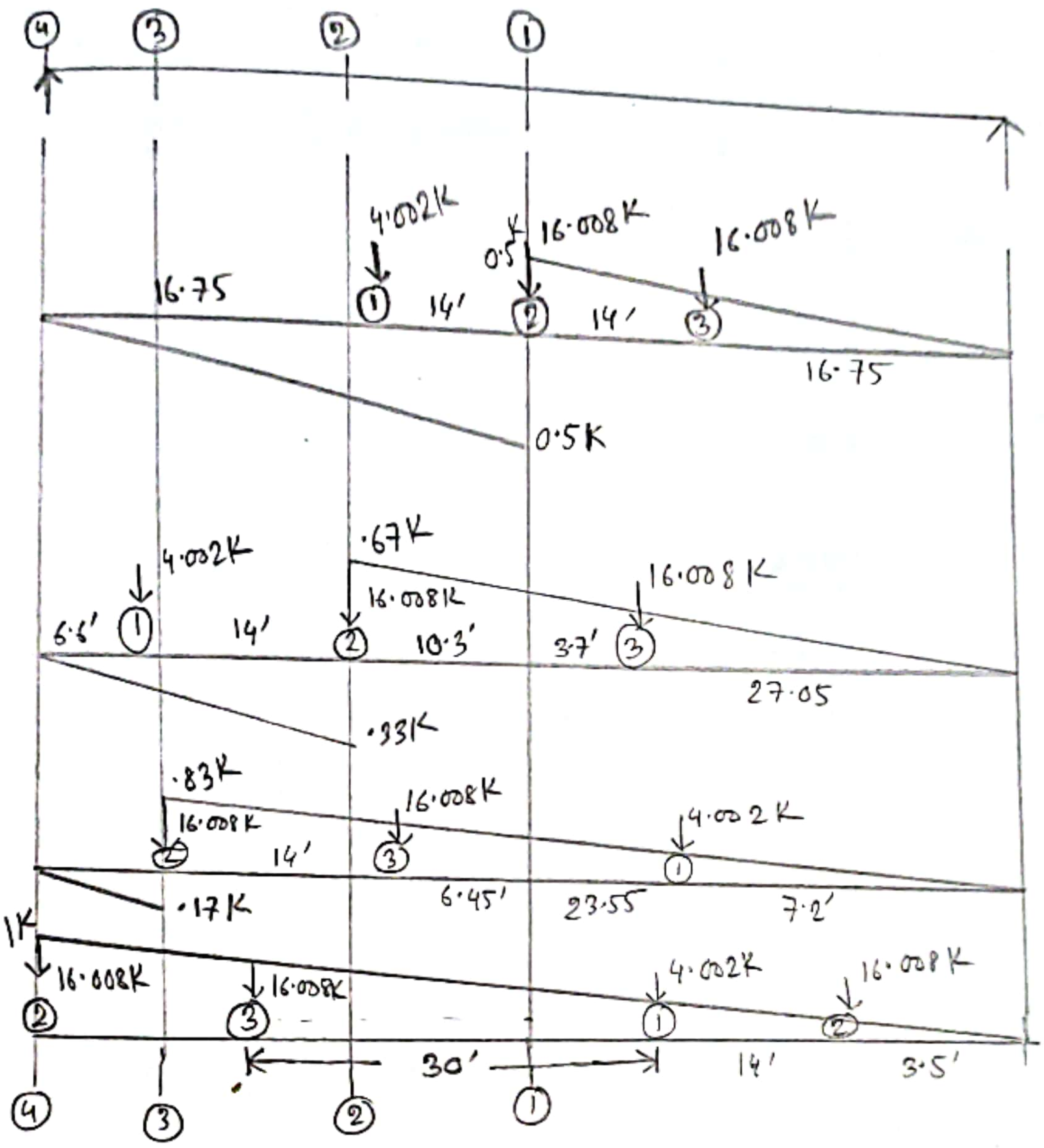
$$= 379.73\ K'$$

$$M_{3-3} = \frac{8.5}{51.35}\left[(4.002 \times 7.35) + (16.008 \times 37.35) + (16.008 \times 51.35)\right] + \frac{8.5}{10.15}\left[0\right]$$

$$= 239.91\ K'$$

$M_{4-4} = 0\ K'$

live load Shear: -



$$V_{1-1} = \frac{0.5}{30.75}\left[(16.008 \times 16.75) + (16.008 \times 30.75)\right] = 12.364\text{ K}$$

$$V_{2-2} = \frac{0.67}{41.05}\left[(16.008 \times 27.05) + (16.008 \times 41.05)\right] = 17.8\text{ K}$$

$$V_{3-3} = \frac{.83}{51.2}\left[(4.002 \times 7.2) + (16.008 \times 37.2) + (16.008 \times 51.2)\right] = 23.41\text{ K}$$

$$V_{4-4} = \frac{1}{61.5}\left[(16.008 \times 3.5) + (4.002 \times 17.5) + (16.008 \times 47.5) + (16.008 \times 61.5)\right]$$

$$= 30.42\text{ K}$$

Impact shear :-

$V_{1-1} = 12.4 \times .27 = 3.35$ K

$V_{2-2} = 17.8 \times .27 = 4.81$ K

$V_{3-3} = 23.41 \times .27 = 6.32$ K

$V_{4-4} = 30.42 \times .27 = 8.21$ K

[$\because$ Impact co efficient, $I_c = \frac{50}{61.5+125} = 0.27$]

Impact moment :-

$M_{1-1} = 414.21 \times .27 = 111.84$ K'

$M_{2-2} = 379.73 \times .27 = 102.53$ K'

$M_{3-3} = 239.91 \times .27 = 64.78$ K'

$M_{4-4} = 0 \times .27 = 0$ K'

$M_{abs} = 415.3 \times .27 = 112.13$ K'

Table for design shear

Section	Dead load Shear (K)	live load shear (K)	Impact shear (K)	Total shear (K)	Design shear (K)
1-1	0	12.364	3.35	15.714	72.43
2-2	11.134	17.8	4.81	33.74	
3-3	22.47	23.41	6.32	52.2	
4-4	33.8	30.42	8.21	72.43	

Table for Design moment

Section	Dead load moment (K')	liveload moment (K')	Impact moment (K')	Total moment (K')	Design moment (K')
1-1	520.24	414.21	111.84	1046.29	1047.67
2-2	462.9	379.73	102.53	945.16	
3-3	289.8	239.91	64.78	594.49	
4-4	0	0	0	0	
absolute	~~415.3~~ 520.243	415.3	112.13	1047.67	

Depth check:-

Design shear = 72.43 K

Allowable shear stress, $v = 0.06 \times f_c'$

$= 0.06 \times 3000 = 180$ psi

Now; $bd = \frac{V}{vj} = \frac{72.43 \times 1000}{180 \times .87} = 462.52$

$d_{req} = \frac{462.52}{15} = 30.83$ $[\because b = 15'']$

$d_{eff} = (32+6) - \{(2 \times 2) + 2.5\} = 31.5'' > d_{req}$

$\therefore$ Design is ok.

Reinforcement calculation:-

$As_1 = \frac{M_1}{f_s(d - h_f/2)} = \frac{1046.29 \times 12}{20(31.5-3)} = 22.03$ in^2

Use 10 # 14 bars in four layers As = 23.52 in^2

Spacing between top base = 7''

middle layer = 7''

bottom layer = 7''

$As_2 = \frac{945.16 \times 12}{20(31.5-3)} = 19.9$ in^2

Use 9 # 14 bars in three layer As = 21.17 in^2

Spacing between top base = 14''

middle layer = 7''

bottom layer = 7''

$$As_3 = \frac{594.49 \times 12}{20(31.5-3)} = 12.52 \text{ in}^2$$

Use 6 # 14 bars in two layers; $As = 14.12 \text{ in}^2$

$As_4 = 0 \text{ in}^2$

Use 2 # 14 bars in single layer.

$$As_{avs} = \frac{1047.67 \times 12}{20(31.5-3)} = 22.1 \text{ in}^2$$

Use 10 # 14 bars in four layers $As = 23.52 \text{ in}^2$

Check for T beam:-

Effective width of T beam

1. $16h_f + b_w =$ ~~111"~~ $16 \times 6 + 15 = 111''$
2. $L/4 = \frac{61.5}{4} \times 12 = 184.5''$
3. $c/c = 6.67 \times 12 = 80''$

Effective flange width = 80"

$$\rho = \frac{As}{bd} = \frac{23.52}{80 \times 31.5} = 0.0093 \quad , \quad n = \frac{29 \times 10^6}{57000\sqrt{3000}} = 9$$

$n\rho = 0.084$

$$K = \frac{\rho n + \frac{1}{2}(h_f/d)^2}{\rho n + \frac{1}{2}(h_f/d)} = 0.57.$$

$Kd = 17.95 > h_f$

∴ T beam is confirmed.

Bond check:-

$$V_d = \frac{V}{\Sigma_o jd} = \frac{72.43 \times 1000}{22 \times 0.87 \times 31.5} = 120.13 \text{ psi}$$

$$\Sigma_o = n\pi d = 4 \times 3.14 \times \frac{14}{8} = 21.99 \approx 22$$

$$V_{all} = 0.1 f_c' = 0.1 \times 3000 = 300 \text{ psi} > V_d$$

$\therefore$ Bond check is ok

Web reinforcement:-

Concrete carries unit shear $= 0.03 f_c' = 90$ psi

stress develop at end $V_{end} = \frac{72.43 \times 1000}{15 \times 31.5 \times .87} = 176.2$ psi

$$S_{min} = \frac{A_v f_s}{(V_{end} - V_e) b_w} = \frac{2 \times .31 \times 20{,}000}{(176.2 - 90) 15} = 9.6'' \approx 10'' \text{ c/c}$$

$$S_{max} = d/2 = 15.75''$$

$$V_c' = \frac{A_v f_s}{S_{max} \times b_w} = \frac{0.31 \times 2 \times 20{,}000}{15.75 \times 15} = 52.49 \text{ psi}$$

$$V_{max} = (52.49 + 90) = 142.49 \text{ psi}$$

Horizontal distance for $(V_e + V_c')$ for mid point $= \frac{142.49 \times 30.75}{176.2} = 24.9$

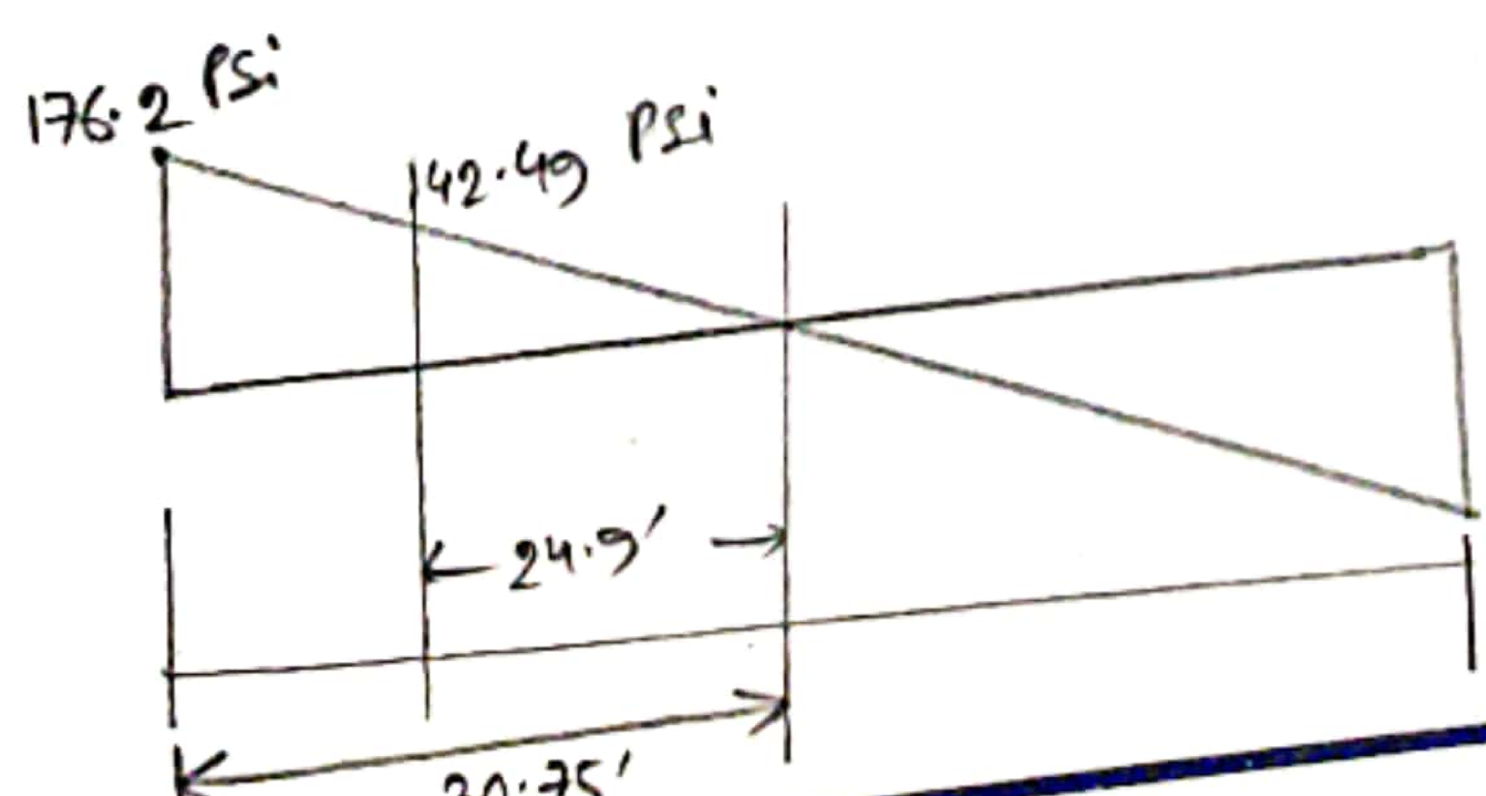
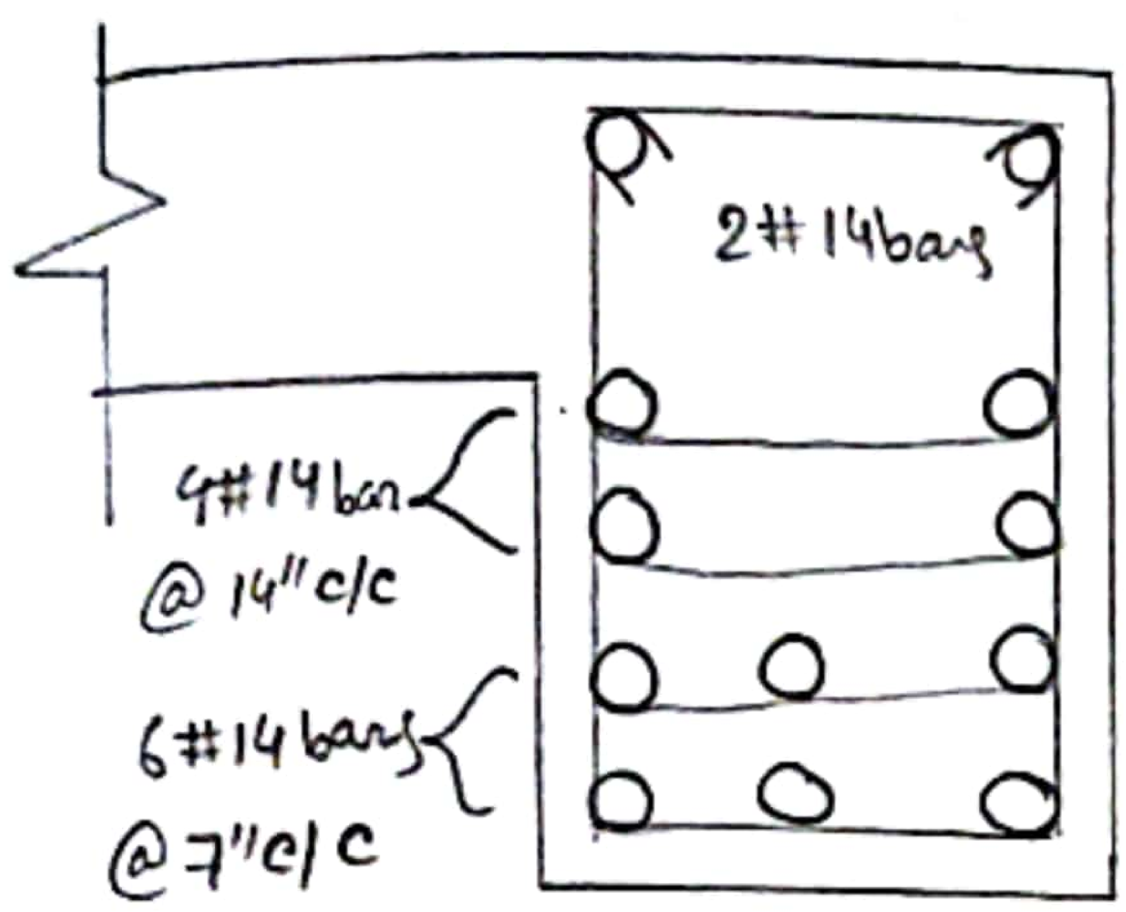
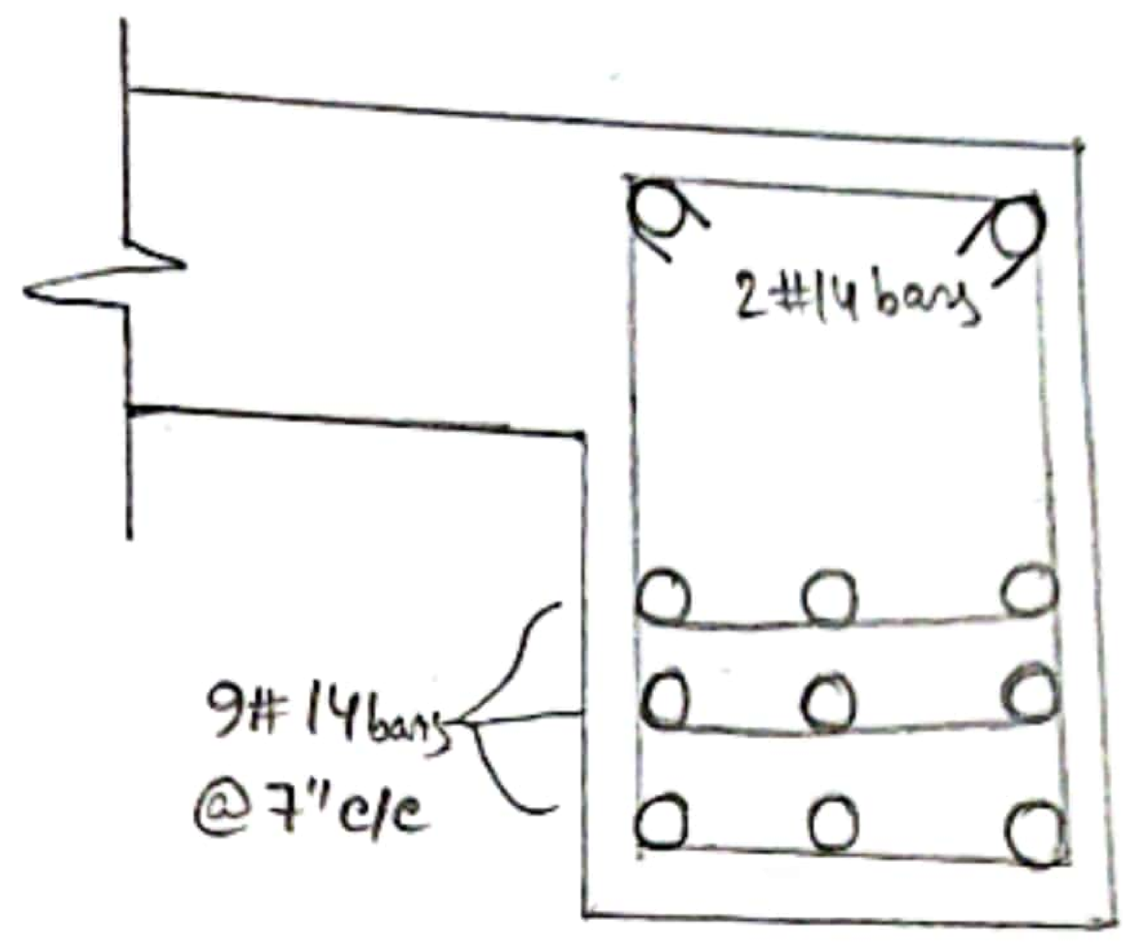


Fig: Web reinforcement.

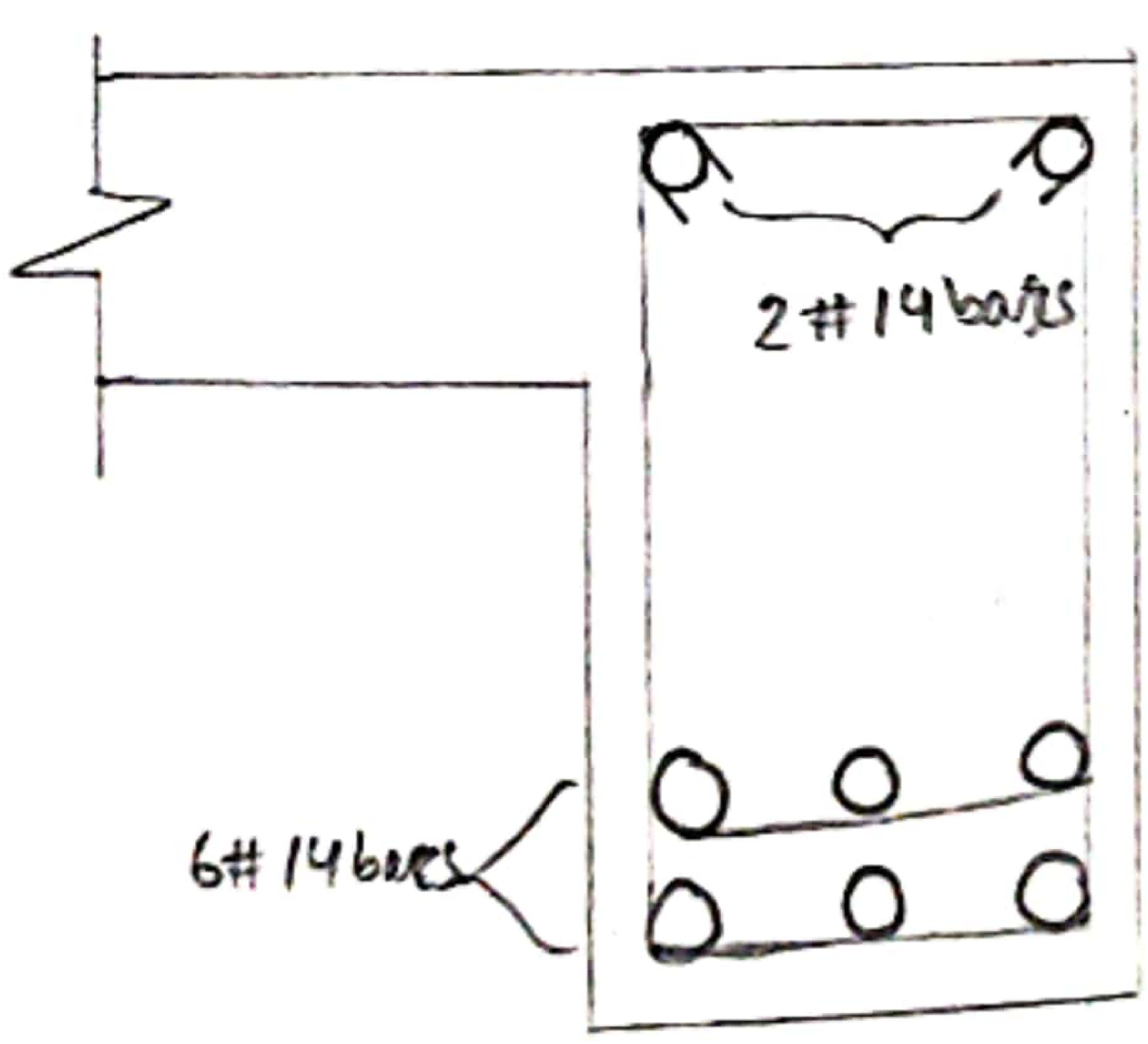
Working diagram:-



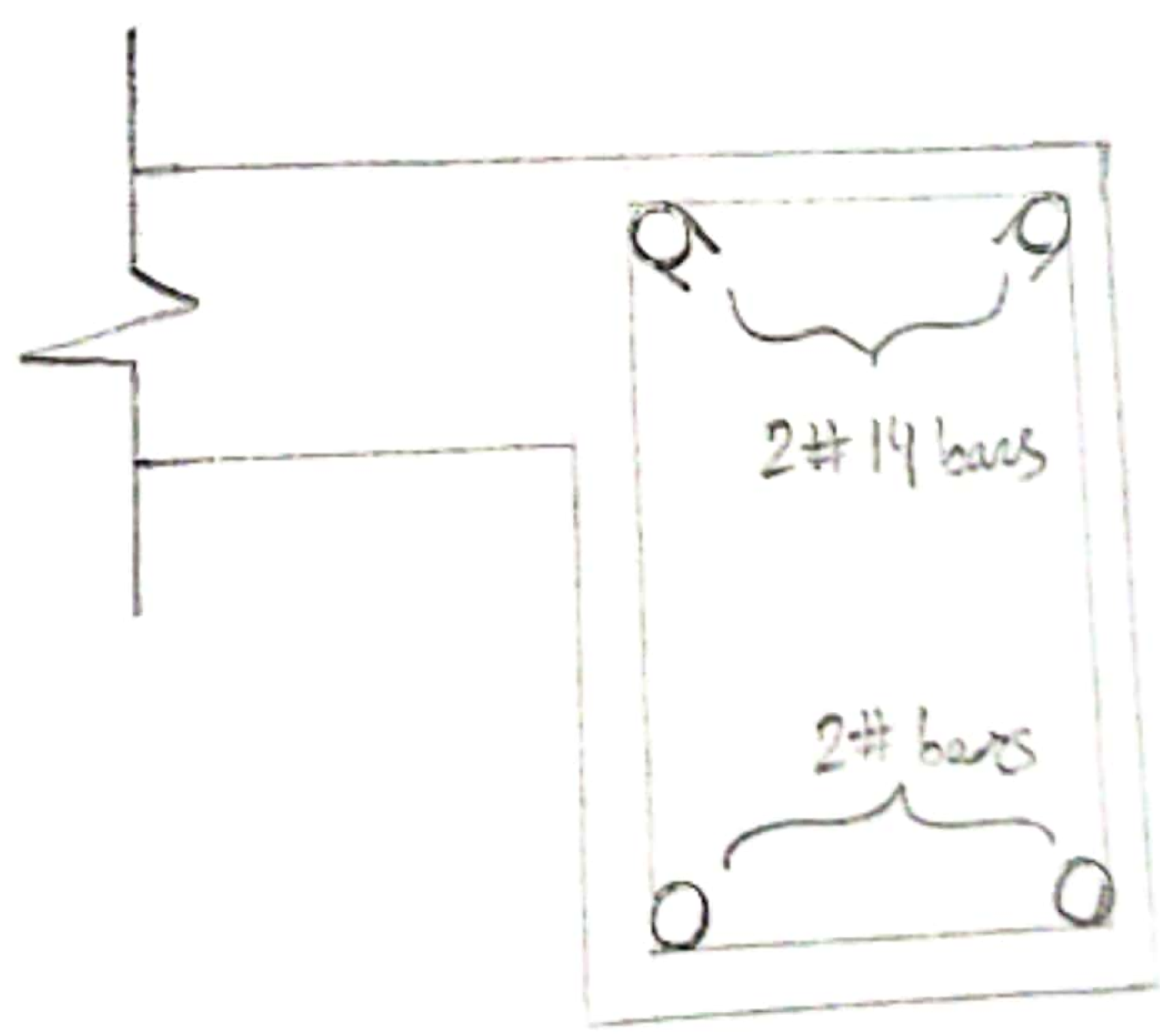
Section 1-1



section 2-2



section 3-3



section 4-4

Heaven's Light is Our Guide

Rajshahi University of Engineering & Technology
Rajshahi

DEPARTMENT OF CIVIL ENGINEERING

Expt. No. 07

Name of Expt Design of Exterior girder of a deck girder bridge

SUBJECT : Reinforced Concrete Sessional-I	SUBMITTED BY :
COURSE NO. : CE-3218	NAME : S.M. Abdullah Al Ahad
DATE OF EXPT.:	CLASS : 3rd year even semester
DATE OF SUB. :	GROUP : - ROLL NO 1600098
	SESSION : 2016-17

Design of exterior girder of a deck girder bridge :-

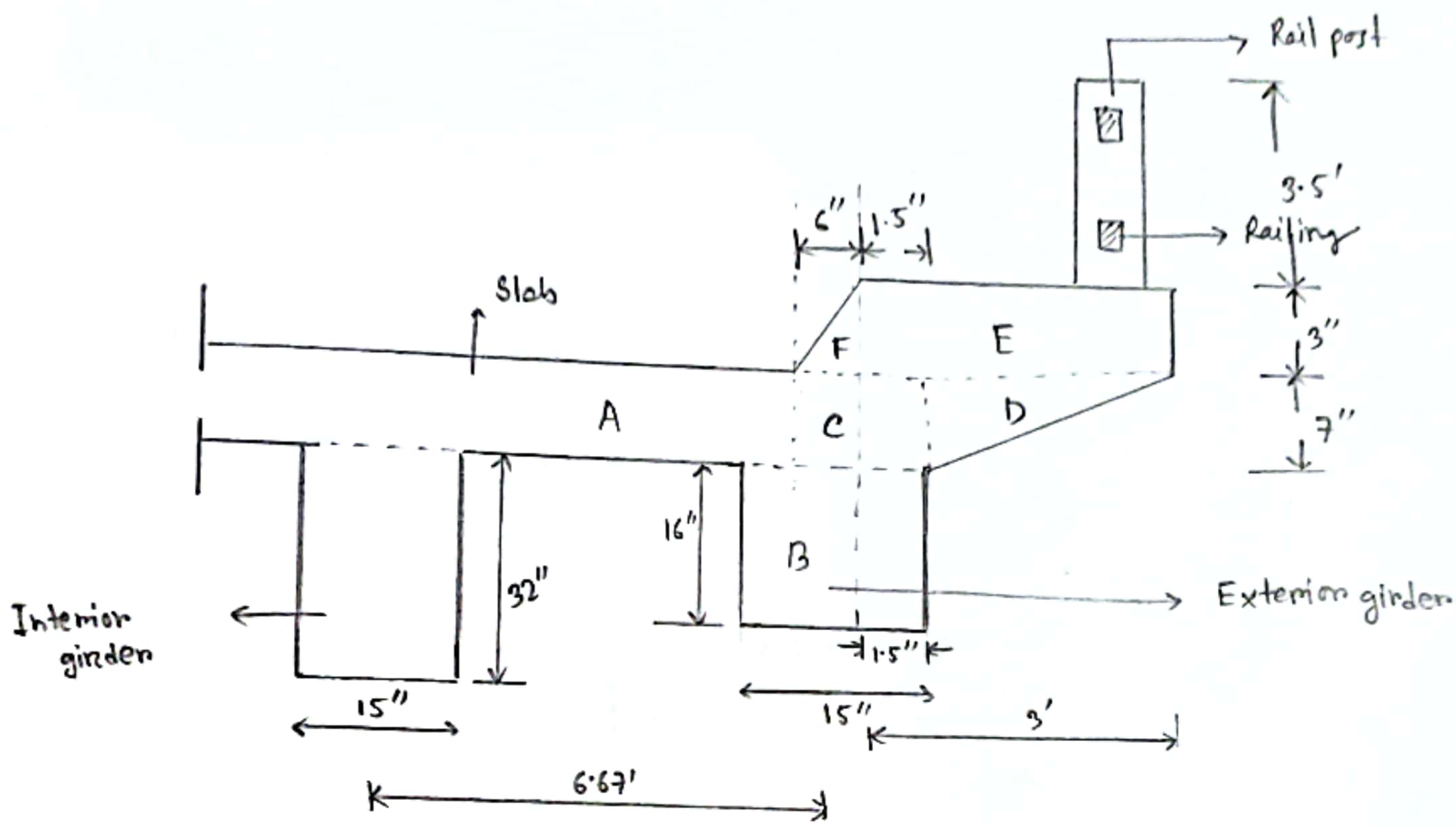


Fig: Typical details of exterior girder.

Dead load shear and moment :-

(i) load calculation from slab girder, kerb :

let, exterior girder section is 15"X16"

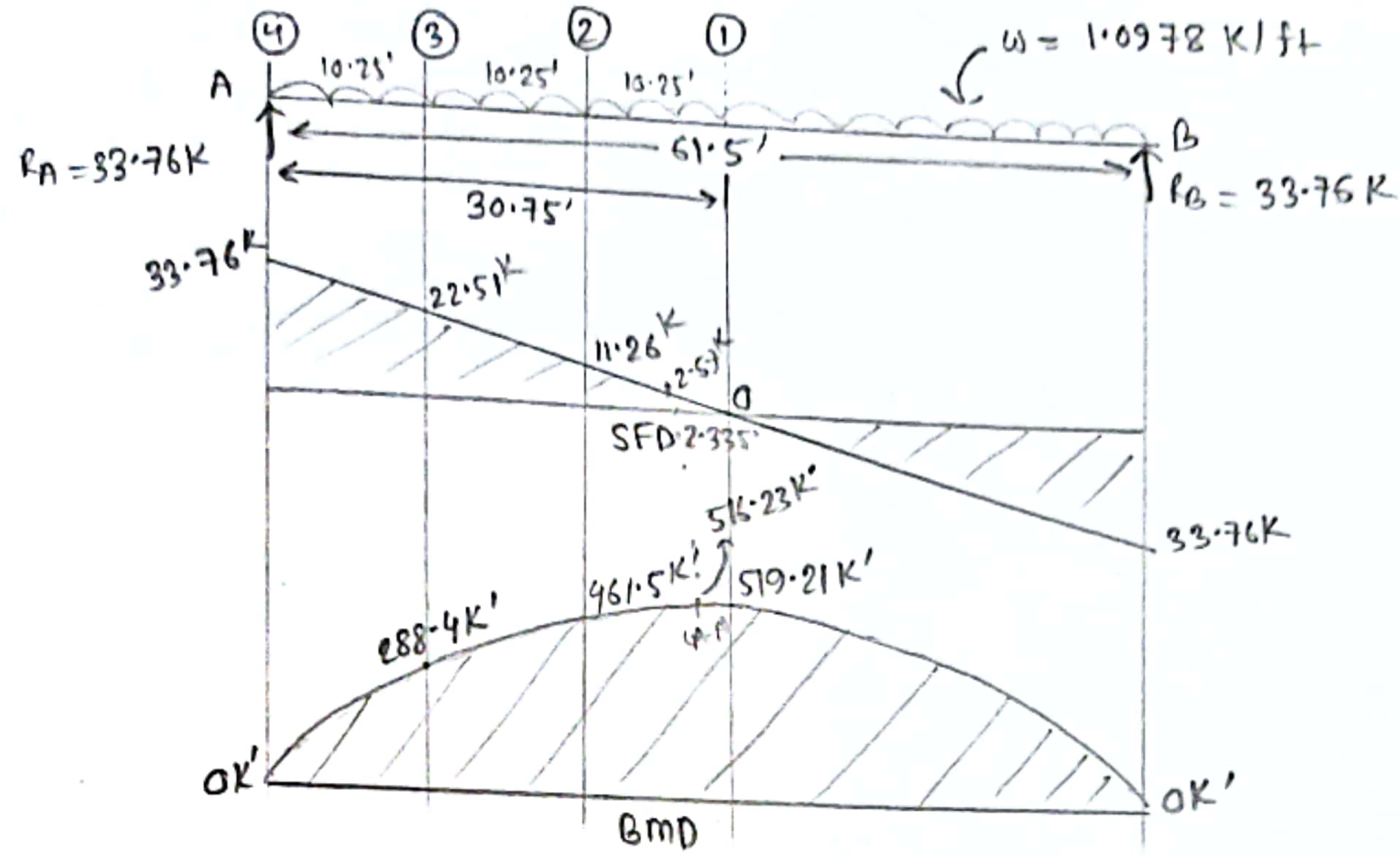
$$\text{Dead load} = \left\{\frac{7}{12}\times\frac{6.67}{2} + \frac{15\times16}{144} + \frac{7\times7.5}{144} + \left(\frac{1}{2}\times\frac{34.5\times7}{144}\right) + \left(\frac{3\times3}{12}\right) + \left(\frac{1}{2}\times\frac{6\times3}{144}\right)\right\}\times150$$

$$= 844.16 \text{ plf}$$

Dead load from railing = 37.5 plf

Dead load from rail post = 178.65 plf

$\therefore$ Total dead load, $\omega = 844.16 + (37.5\times2) + (178.65) = 1097.81$ plf



$Sf_{1-1} = 0K$

$Sf_{2-2} = 11{\cdot}26K$

$Sf_{3-3} = 22{\cdot}51\,K$

$Sf_{4-4} = 33{\cdot}76\,K$ (maximum)

$BM_{1-1} = 519{\cdot}21\,K'$

$BM_{2-2} = 461{\cdot}5K'$

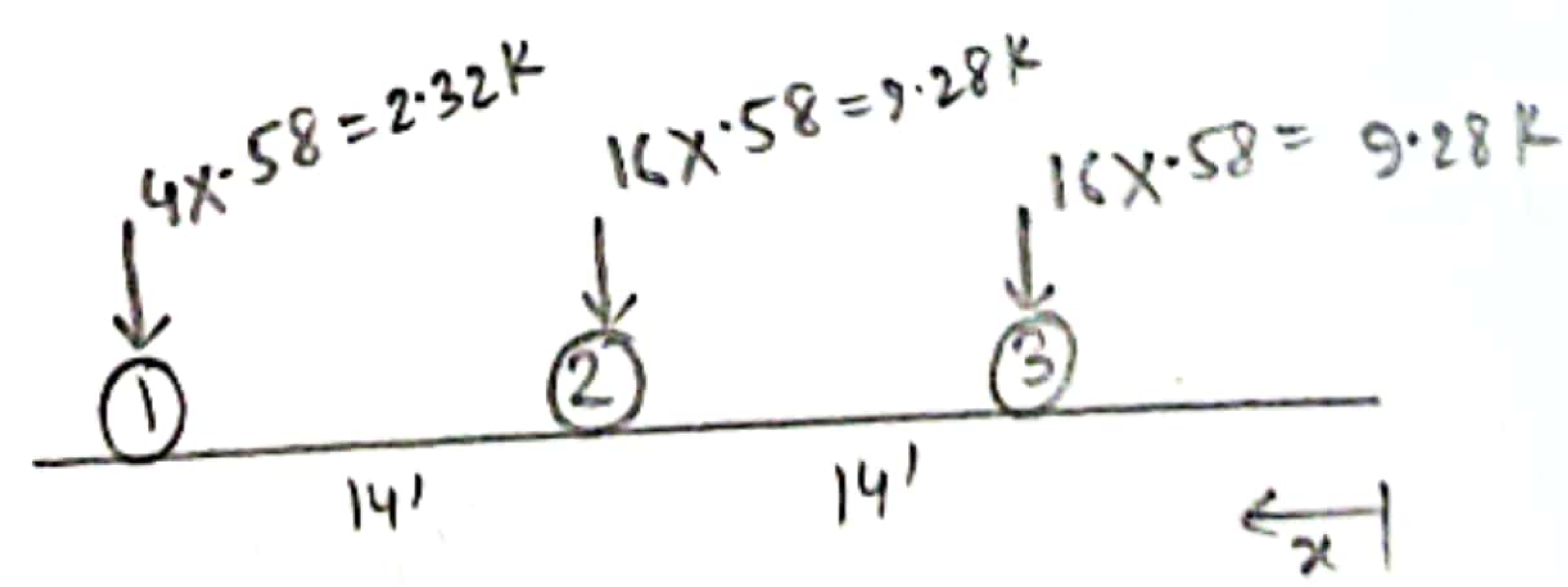
$BM_{3-3} = 288{\cdot}4K'$

$BM_{4-4} = 0K'$

$BM_{Ⓐ-Ⓐ} = 516{\cdot}23K'$

Live load shear and moment;-

Effective wheel $= \left[\dfrac{6{\cdot}67 - 1{\cdot}5}{6{\cdot}67}\right] / 1{\cdot}33 = 0{\cdot}58$ (By effective no of wheels)



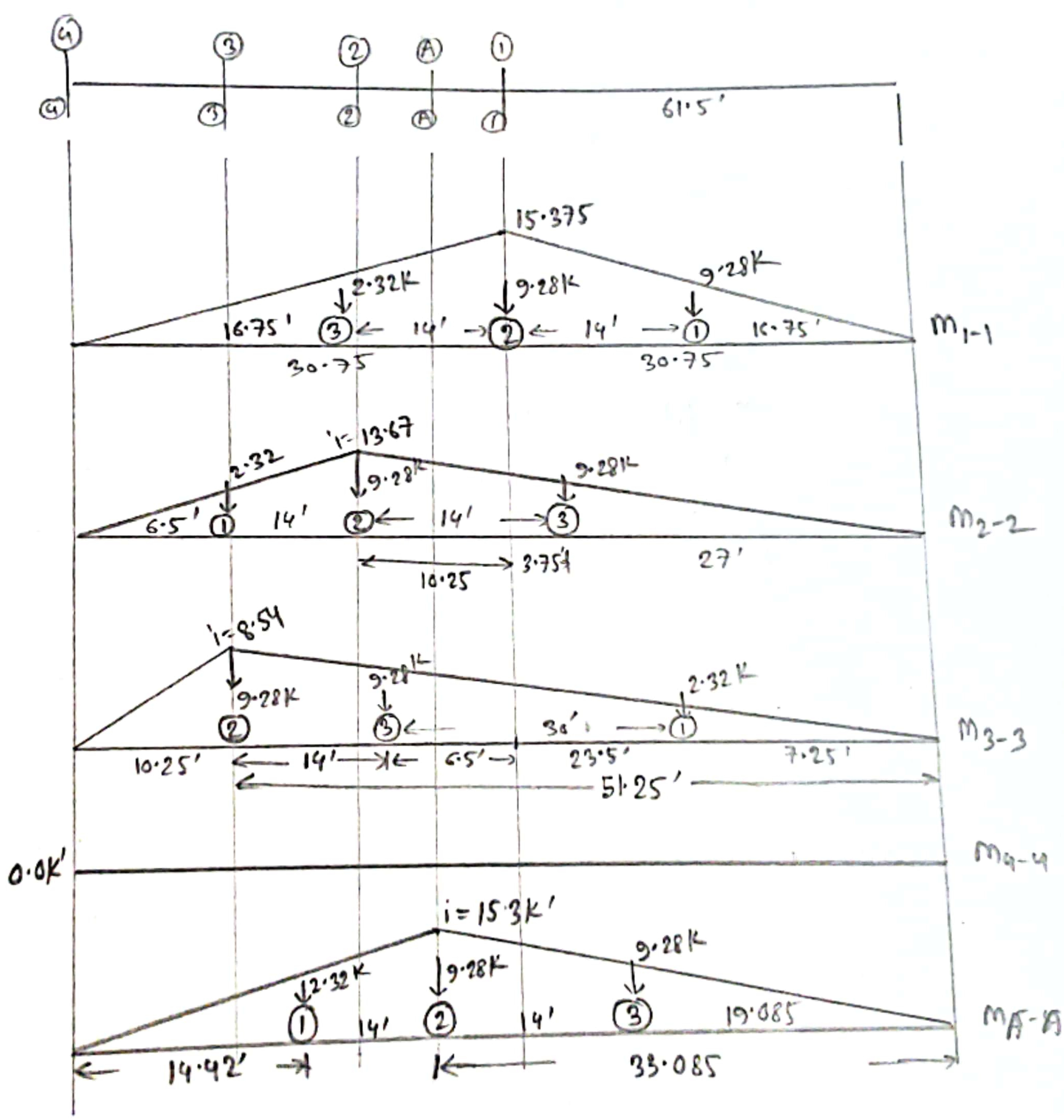
Taking moment at right side;

$$\bar{x} = \frac{9{\cdot}28\times14 + 2{\cdot}32\times28}{2{\cdot}32+9{\cdot}28+9{\cdot}28} = 9{\cdot}33'$$

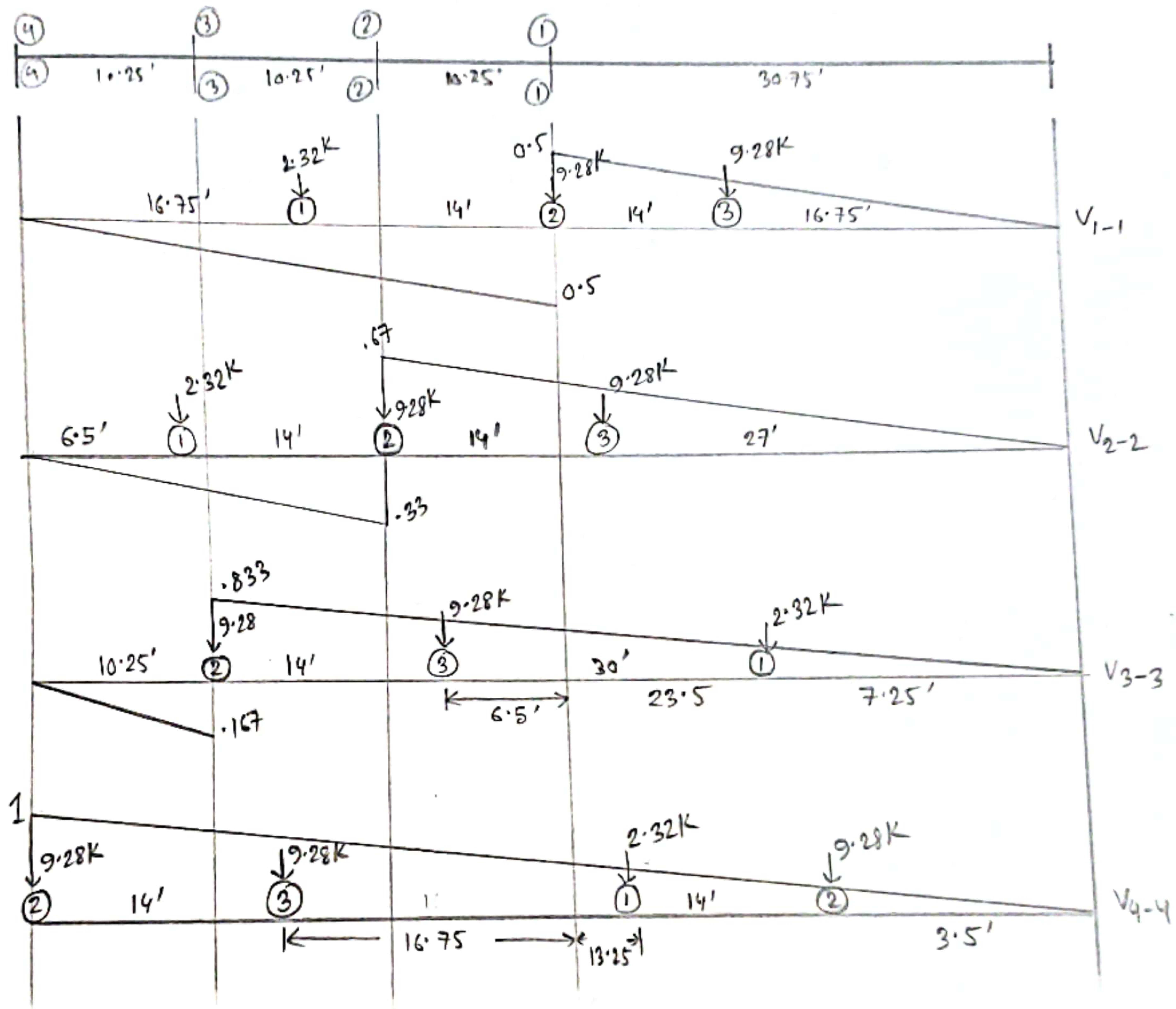
$a = 14 - \bar{x} = 14 - 9.33' = 4.67'$

The position of wheel for abs. max moment from right corner $= \frac{L}{2} + \frac{a}{2} = \frac{61.5}{2} + \frac{4.67}{2} = 33.085'$

IL for Live load moment :-



IL for L.L. shear :-



$$V_{1-1} = \frac{.5}{30.75}\left[9.28 \times 16.75 + 9.28 \times 30.75\right] = 7.17\ K$$

$$V_{2-2} = \frac{.67}{41}\left[9.28 \times 27 + 9.28 \times 41\right] = 10.31\ K$$

$$V_{3-3} = \frac{.833}{51.25}\left[2.32 \times 7.25 + 9.28 \times 37.25 + 9.28 \times 51.25\right] = 13.6\ K$$

$$V_{4-4} = \frac{1}{61.5}\left[9.28 \times 3.5 + 2.32 \times 17.5 + 9.28 \times 47.5 + 9.28 \times 61.5\right]$$

$$= 17.64\ K$$

Moment calculation:-

$$M_{1-1} = \frac{15.375}{30.75}\left[9.28\times16.75 + 9.28\times30.75\right] + \frac{15.375}{30.75}\left[2.32\times16.75\right]$$

$$= 239.83\ K'$$

$$M_{2-2} = \frac{13.67}{41}\left[9.28\times27 + 9.28\times41\right] + \frac{13.67}{20.5}\left[2.32\times6.5\right]$$

$$= 220.454\ K'$$

$$M_{3-3} = \frac{8.54}{51.25}\left[2.32\times7.25 + 9.28\times37.25 + 9.28\times51.25\right]$$

$$= 139.66\ K'$$

$$M_{4-4} = 0\ K'$$

$$M_{A-A} = \frac{15.3}{33.085}\left[9.28\times19.085 + 9.28\times33.085\right] + \frac{15.3}{28.42}\times\left[2.32\times14.42\right]$$

$$= 241.9\ K'$$

Impact shear and moment:-

$$\text{Impact factor} = \frac{50}{125+61.5} = 0.27 < 30\%.$$

Impact moment:-

$$M_{1-1} = 239.83\times.27 = 64.75\ K'$$

$$M_{2-2} = 220.45\times.27 = 59.52\ K'$$

$$M_{3-3} = 139.66\times.27 = 37.71\ K'$$

$$M_{4-4} = 0\times.27 = 0\ K'$$

$$M_{A-A} = 241.9\times.27 = 65.31\ K'$$

Impact shear :-

$V_{1-1} = 7.17 \times .27 = 1.94\ K$

$V_{2-2} = 10.31 \times .27 = 2.8\ K$

$V_{3-3} = 13.6 \times .27 = 3.67\ K$

$V_{4-4} = 17.64 \times .27 = 4.763\ K$

Design chart for shear

Sector	Dead load shear (K)	Live load shear (K)	Impact shear (K)	Total shear (K)	Design shear (K)
1-1	0	7.17	1.94	9.11	56.163
2-2	11.26	10.31	2.8	24.37	
3-3	22.51	13.6	3.67	39.78	
4-4	33.76	17.64	4.763	56.163	

Design Chart for moment

Sector	Dead load moment (K)	Live load moment (K')	Impact moment (K')	Total moment K'	Design moment K'
1-1	519.21	239.83	64.75	823.8	823.44
2-2	461.5	220.454	59.52	741.47	
3-3	288.4	139.66	37.71	465.77	
4-4	0	0	0	0	
A-A	516.23	241.9	65.31	823.44	

Determination of cross section & steel area:-

Maximum shear stress in the beam by AASHTO system:-

$v = 0.06\,f_c' = 0.06 \times 3000 = 180$ psi

$V_{all} = 1.1\sqrt{f_c'} = 1.1 \times \sqrt{3000} = 65.08$ psi.

$b'd_{req} = \dfrac{V_{max}}{v \times j} = \dfrac{56.163 \times 1000}{180 \times .8} = 352.56\,K'$

Here; $b'd = 352.56'$ $\because b' = 15''$

$\Rightarrow d = 23.5''$

$\therefore t = 23.5'' + 1'' + \frac{11}{8}'' + \frac{.31''}{2} = 27.26''$

$d_{eff} = 27.26'' - 2'' = 25''$

Tensile steel required; $A_s = \dfrac{823.8 \times 12000}{24000 \times (25 - 7/2)} = 19.2\ in^2$
(14 #11 bar)

Effective width, b1 –

(i) $b = \dfrac{L}{12} + b_w = \dfrac{61.5 \times 12}{12} + 15 = 76.5''$

(ii) $b = 6h_f + b_w = 6 \times 7 + 15 = 57''$

(iii) $b = \dfrac{S}{2} + b_w = \dfrac{6.67 \times 12}{2} + 15 = 55.02'' \approx 55''$

Effective, $b = 55''$

$\rho = \frac{A_s}{bd} = \frac{19.2}{55 \times 25} = 0.014$; $n = \frac{E_s}{E_c} = 9.28 \approx 9$

$$K = \frac{n\rho + \frac{1}{2}(t/d)^2}{n\rho + \frac{1}{2}(t/d)} = \frac{9 \times 0.014 + 1/2 (7/25)^2}{9 \times 0.014 + 1/2 (7/25)} = 0.62$$

$Kd = 0.62 \times 25 = 15.5 > h_f$

$\therefore$ $Kd > h_f$.

So; L beam is confirmed.

Bond Check :-

$$U_d = \frac{V}{\Sigma_o jd} = \frac{56.163 \times 1000}{17.28 \times .885 \times 25} = 146.9 \text{ psi}$$

$\Sigma_o = n \pi d$
$= 4 \times \pi \times \frac{11}{8}$
$= 17.28$

$U_{allow} = 0.10 f_c' = 0.1 \times 3000 = 300$ psi

$U_{allow} > U_d$.

So; Bond check is okay.

Heaven's Light is Our Guide

Rajshahi University of Engineering & Technology
Rajshahi

DEPARTMENT OF CIVIL ENGINEERING

Expt. No. ....08....

Name of Expt ....Design of Abutment....

......

	SUBMITTED BY :
SUBJECT : Reinforced concrete sessional-II.	**NAME :** S.M. Abdullah Al Ahad
COURSE NO. : CE-3218	**CLASS :** 3rd year even Semester
DATE OF EXPT.:	**GROUP :** ...... **ROLL NO** 1600098
DATE OF SUB. :	**SESSION :** 2016-17

Design of abutment for Deck girder bridge:-

Design specification:-

1) Overall height of the abutment = 20'
2) Bearing capacity of soil = 4 ksf
3) Unit weight of soil, γ = 120 pcf
4) Angle of inertia friction, $\phi = 30°$
5) Co efficient of friction, $f = 0.5$
6) $f_c' = 3000$ psi & $f_y = 60,000$ psi.

Proportioning of dimensions:-

1) Width of abutment = (width of roadway) + (2X width of sidewalk)

$$= 30 + 2 \times 5 = 40 \text{ ft}.$$

Effective width of abutment $= 40 + 2.75 + 2.75 = 45'$

2) Width of base $= \frac{2}{3} \times 20 = 13.33$ ft ≈ 14 ft.

3) Active earth pressure co-efficient, $K_a = \dfrac{1 - \sin\phi}{1 + \sin\phi} = 0.333$

Passive earth pressure co-efficient, $K_p = \dfrac{1}{K_a} = \dfrac{1}{.33} = 3$

Now;

$$P_a = \frac{1}{2} \times K_a \times \gamma_s H \times H = \frac{1}{2} \times .333 \times 120 \times (20)^2 = 7992 \text{ lb}$$

$\therefore$ Active pressure = 7992 lb.

$\therefore$ Over turning moment, $M_o = P_a \times H/3 = 7992 \times \frac{20}{3}$

$= 53280$ lb-ft.

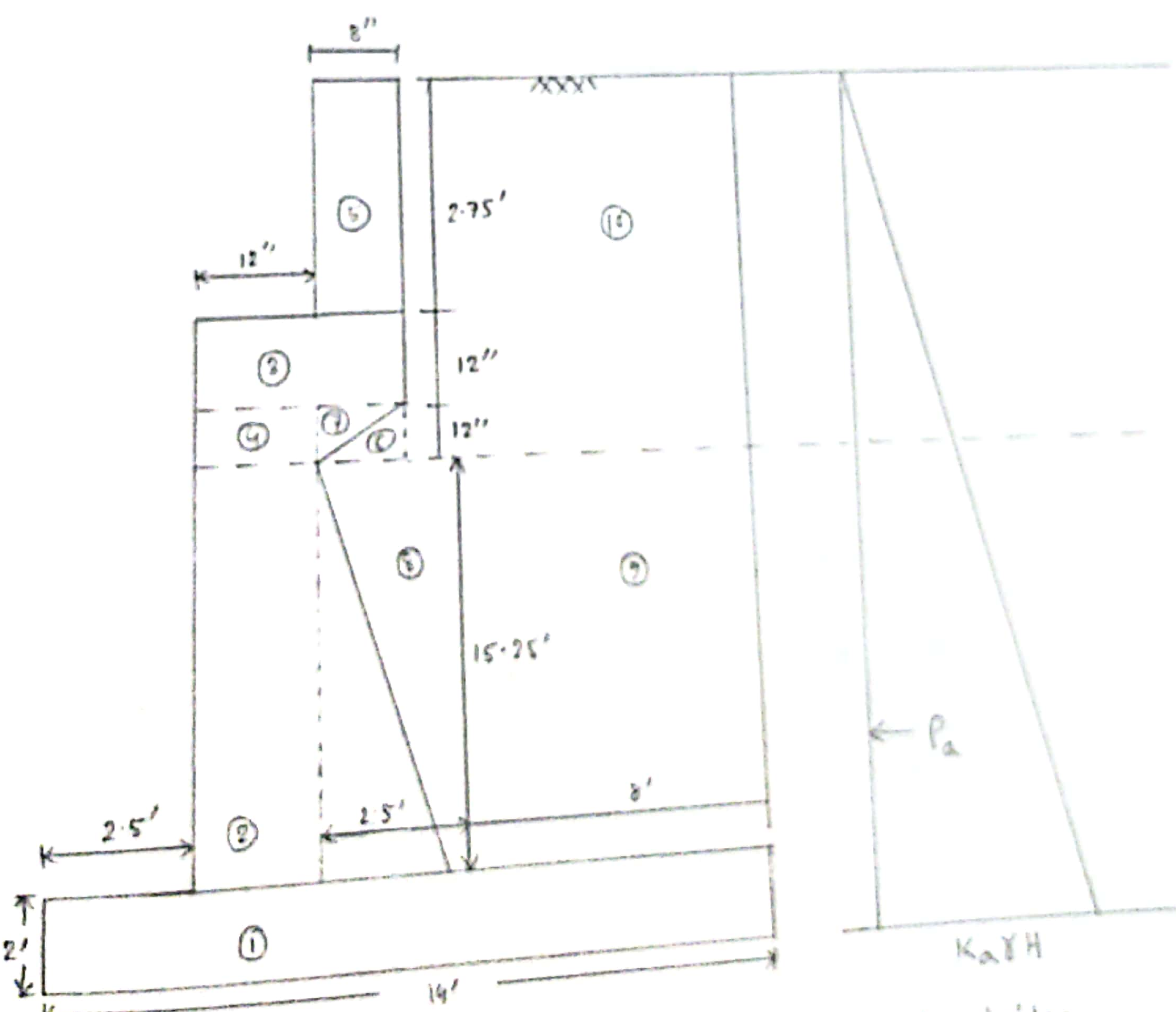


Fig: Details of abutment of deck girder bridge

Load coming from super structure:-

Total Dead load = [No. of interior girder X DL reaction of interior girder + 2 X DL reaction of exterior girder]

$= 4 \times 33.8 + 2 \times 33.76$

$= 202.7$ K.

Total live load = [No. of interior girder X LL reaction of interior girder + 2 X LL reaction of exterior girder]

$= 4 \times 30.92 + 2 \times 17.64$

$= 156.96$ K.

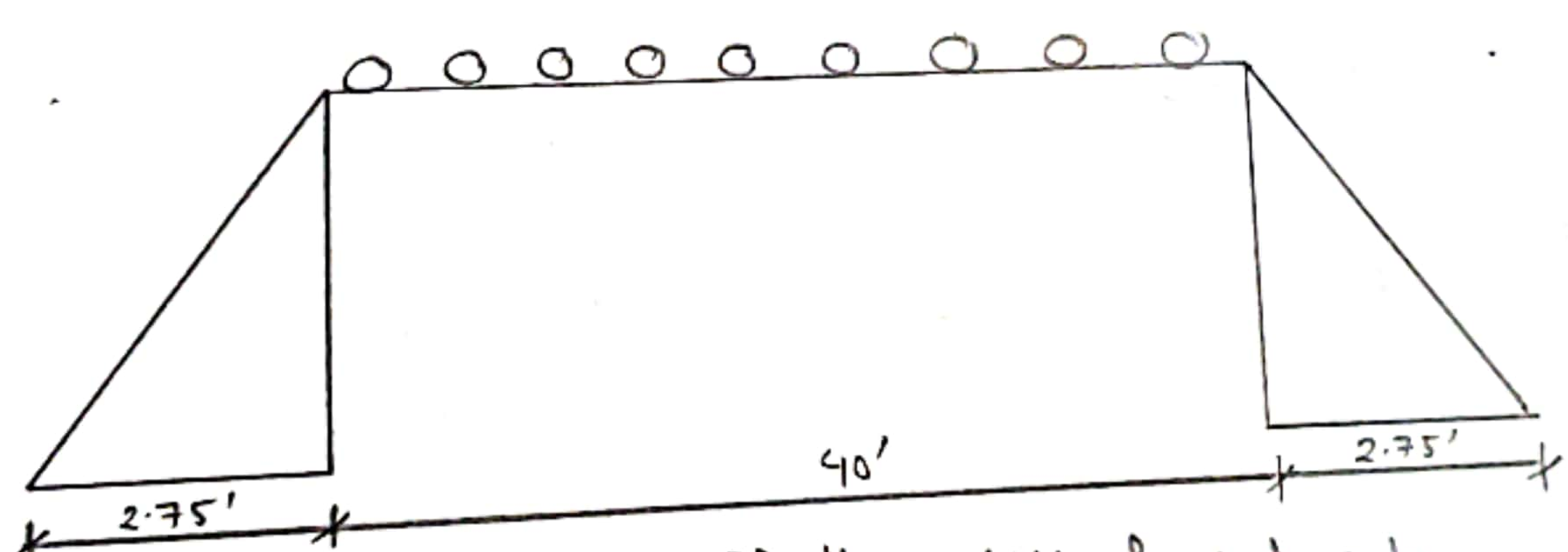


fig: Effective width of abutment

∴ Effective width of abutment $= 40' + 2.75' + 2.75' = 45'$

∴ Dead load per foot of abutment $= \frac{202.7}{45} = 4504.4$ lb.

∴ Live load per foot of abutment $= \frac{156.96}{45} = 3488$ lb.

Table 01: Calculation for Total load, weight & resisting moment

Section	Weight (lb)	Moment Arm (ft)	Moment (lb-ft)
1	14×2×150 = 4200	14/2 = 7	29400
2	15.25×1×150 = 2287.5	2.5 + .5 = 3	6862.5
3	12/12 × 20/12 × 150 = 250	2.5 + .833 = 3.33	833.33
4	1×1×150 = 150	2.5 + .5 = 3	450
5	2.75 × 8/12 × 150 = 275	2.5 + 1 + .33 = 3.833	1054.075
6	1/2 × 8/12 × 1 × 120 = 40	2.5 + 1 + 2/3 × 8/12 = 3.94	157.77
7	1/2 × 8/12 × 1 × 150 = 50	2.5 + 1 + 1/3 × 8/12 = 3.72	186.11
8	1/2 × 2.5 × 15.25 × 120 = 2287.5	2.5 + 1 + 2/3 × 2.5 = 5.167	11818.75
9	8 × 15.25 × 120 = 14,640	2.5 + 3.5 + 4 = 10	146400
10	4.75 × 9.833 × 120 = 5605	2.5 + 12/12 + 8/12 + 9.833/2 = 9.08	50911.15
11	1/2 × 2.5 × 15.25 × 150 = 2859.375	2.5 + 1 + 1/3 × 2.5 = 4.333	12390.625
Case I: total	W_1 = 32644.375	ΣMR_1 =	260464.31
Case II: DL	(+) 4504.4	2.5 + 0.5 = 3	(+) 13513.2
Subtotal	W_2 = 37148.775	ΣMR_2 =	273977.51
Case III LL	+ 3488	3	+ 10464
Subtotal	W_3 = 40636.775	ΣMR_3 =	284441.51

Stability Check:-

Case 1:- Super structure is absent.

Factor of safety (FS) against sliding $= \frac{W_1 \times f}{P_a} = \frac{32644.375 \times .5}{7992}$

$= 2.04 > 1.5$ (OK)

FS against overturning $= \frac{260464.31}{53280}$

$= 4.88 > 1.5$ (ok).

Check for soil pressure:-

We know,

$$\sigma = \frac{W}{A} \pm \frac{Mc}{I}$$

$A = 14 \times 1 = 14 \text{ ft}^2$, $I = \frac{1 \times 14^3}{12} = 228.67 \text{ ft}^4$

$c = \frac{B}{2} = \frac{14}{2} = 7$

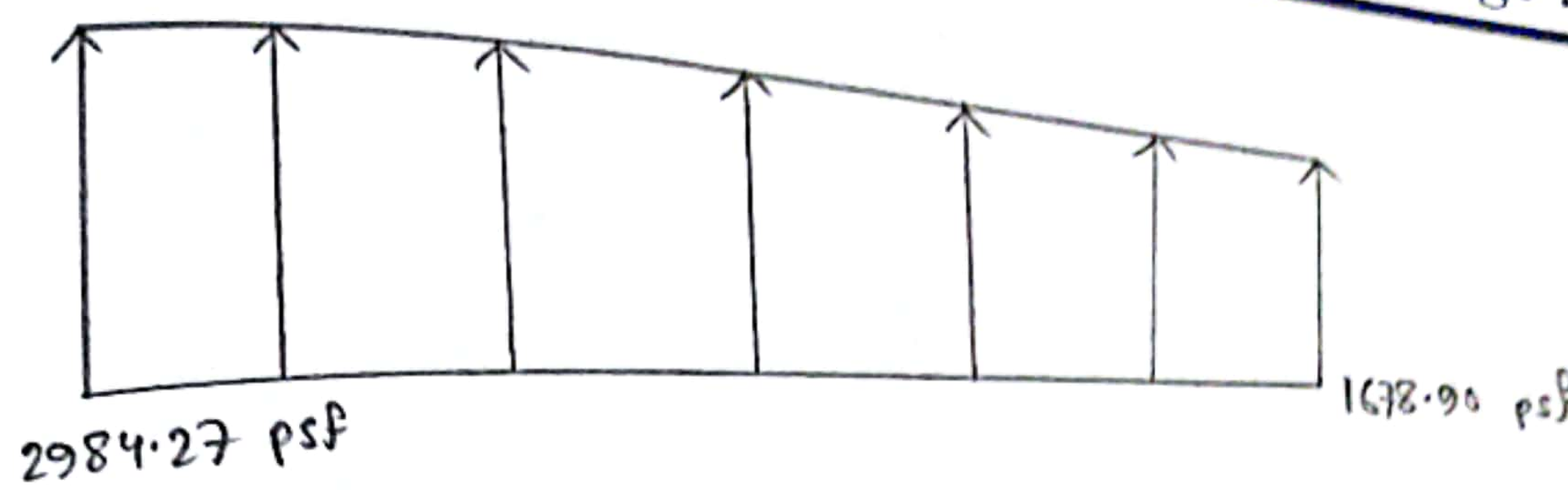
$e = c - a = c - \frac{M_{R1} - M_o}{W_1} = 7 - \frac{260464.31 - 53280}{32644.375} = 0.65$

$M = W_1 \times e = 32644.375 \times e = 21326.315$ lb-ft

$\sigma_1 = \frac{32644.375}{14} + \frac{21326.315 \times 7}{228.67} = 2984.27 \text{ psf} < 4000$

$\sigma_2 = \frac{32644.375}{14} - \frac{21326.315 \times 7}{228.67} = 1678.90 < 4000 \text{ psf}$

[OK]



Case II :-
Super structure is present but, no live load on superstructure

i) FS against sliding $= \frac{W_2 \times f}{P_a} = \frac{37148.775 \times 0.5}{7992} = 2.32 > 1.5$

ii) FS against overturning $= \frac{M_{R_2}}{M_o} = \frac{273977.51}{53280} = 5.14 > 1.5$

$\therefore$ Design is OK.

Check against soil pressure:-

Here; $A = 14\ in^2$, $I = 288.67\ in^4$, $c = 7'$

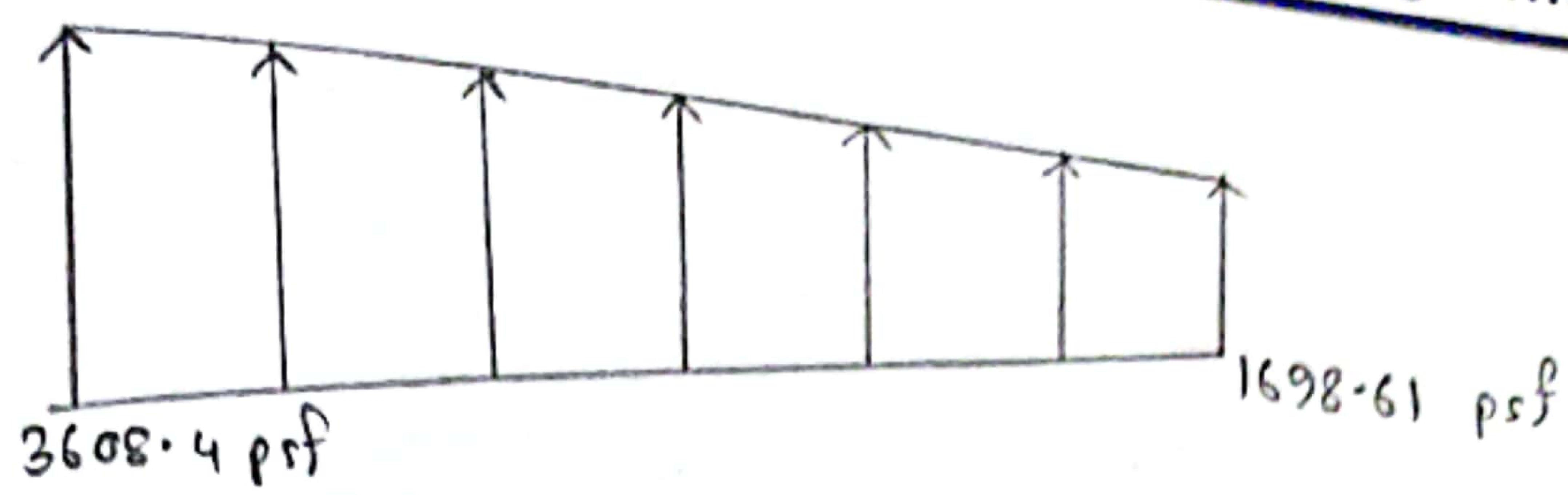
$\therefore e = c - a = c - \frac{M_{R_2} - M_o}{W_2} = 7 - \frac{273977.51 - 53280}{37148.775} = 1.06$

$\therefore M = W_2 \times e = 37148.775 \times 1.06 = 39377.7$ lb-ft

$\sigma_1 = \frac{37148.775}{14} + \frac{39377.7 \times 7}{288.67} = 3608.4 < 4000$ psf

$\sigma_2 = \frac{37148.775}{14} - \frac{39377.7 \times 7}{288.67} = 1698.61 < 4000$ psf

$\therefore$ Design is OK.



Case III :-

Line load present on the super structure.

i) FS against sliding $= \dfrac{W_3 \times f}{P_a} = \dfrac{40636.775 \times .5}{7992} = 2.54 > 1.5$

ii) FS against overturning $= \dfrac{M_{R_3}}{M_o} = \dfrac{284441.51}{53280} = 5.34 > 1.5$

Check against soil pressure :-

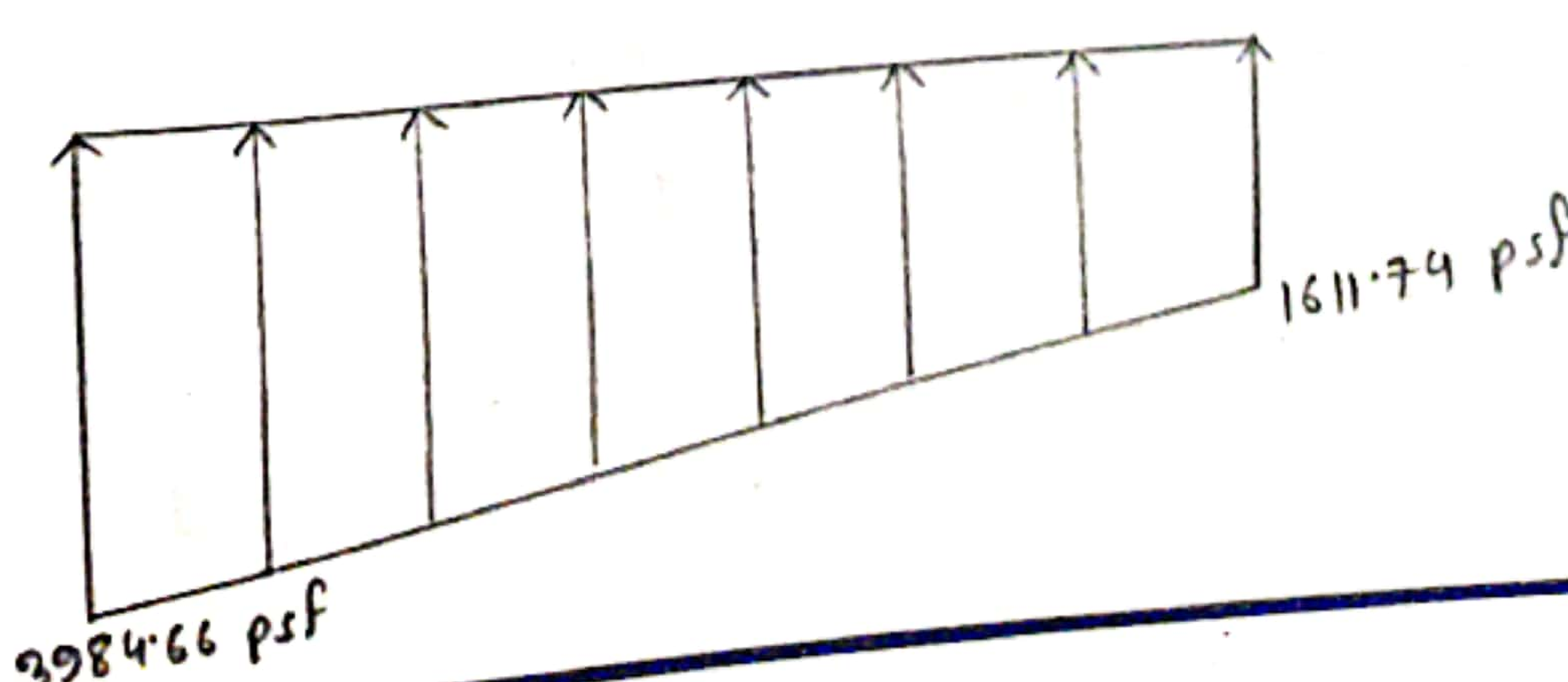
$$e = c - a = 7 - \frac{284441.51 - 53280}{40636.775} = 1.31$$

$$M = W_3 \times e = 53234.2 \text{ lb-ft}$$

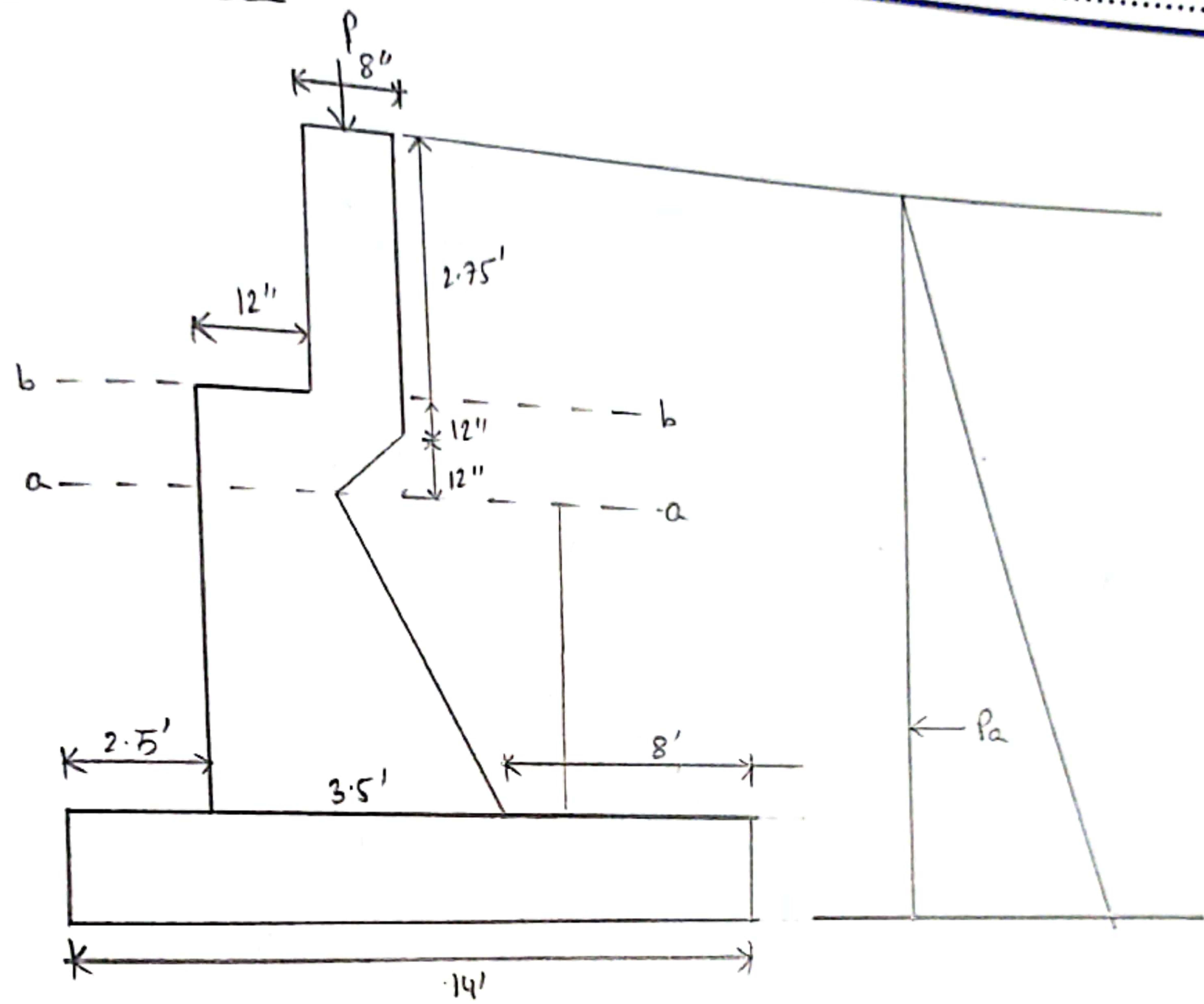
$$\sigma_1 = \frac{40636.775}{14} + \frac{53234.2 \times 7}{288.67} = 3984.66 \text{ psf} < 4000 \text{ psf}$$

$$\sigma_2 = \frac{40636.775}{14} - \frac{53234.2 \times 7}{288.67} = 1611.74 < 4000 \text{ psf}$$

∴ Design is ok.



Design of backwall :-



Condition I :-

When super structure is absent.

$$M_{a-a} = \frac{1}{2} \times k_a \times \gamma \times h_a \times h_a \times h_a/3$$

$$= \frac{1}{2} \times .33 \times 120 \times \frac{4.75^3}{3} = 713.76 \text{ lb-ft}$$

$$M_{b-b} = \frac{1}{2} \times k_a \times \gamma \times h_b \times h_b \times h_b/3$$

$$= \frac{1}{2} \times .333 \times 120 \times \frac{2.75^3}{3}$$

$$= 138.50 \text{ lb-ft.}$$

Condition I

Depth Check:-

for section a-a, $d = \sqrt{\dfrac{713.76 \times 12}{\frac{1}{2} \times 1350 \times .88 \times .336 \times 12}} = 1.89''$

$d_{eff} = t - 1 - \dfrac{5}{8 \times 12} = 12 - 1 - \dfrac{5}{8 \times 12} = 10.69'' > d$

$\therefore d_{eff} > d$ [Check is okay]

for section ⓑ-ⓑ

$d = \sqrt{\dfrac{M_{bb}}{R_b}} = \sqrt{\dfrac{138.50 \times 12}{.5 \times 1350 \times .88 \times .336 \times 12}} = 1''$

$\therefore d_{eff} = 18 - 1 - \dfrac{5}{8 \times 12} = 16.69'' > d$

∴ Design is okay.

Reinforcement calculation:-

At section ⓐ-ⓐ; $A_{s\,a-a} = \dfrac{M_{a-a}}{f_s j d} = \dfrac{713.76 \times 12}{24000 \times .88 \times 10.69}$

$= 0.03 \text{ in}^2$

minimum reinforcement, $A_{s\,min} = 0.0018\, bt = 0.0018 \times 12 \times 12$

$= 0.26 \text{ in}^2.$

$\therefore A_s = 0.26 \text{ in}^2$

Spacing, $S = \dfrac{12 \times .20}{.26} = 9.23 \approx 9''$ c/c

$S_{max} = 3t = 3 \times 8 = 24''$

$S_{max} = 18''$

Using #4 bars @ 9" c/c.