FENTOM PARKWAY BRIDGE

ANALYSIS AND DESIGN

Total length of the bridge

$$L = 225\ m$$

Single span length

$$L_s = 15\ m$$

Curb to curb round width

$$w_c = 11.24\ m$$

Width of barrier

$$w_b = 380\ mm$$

Loading conditions

$$HL - 93$$

Wearing surface thickness

$$h_w = 75\ mm$$

Concrete compressive strength

$$f'_c = 30\ MPa$$

Steel yield strength

$$f_y = 420\ MPa$$

Structure steel

$$H36\ Grade$$

Arrangement of stringers (beams)

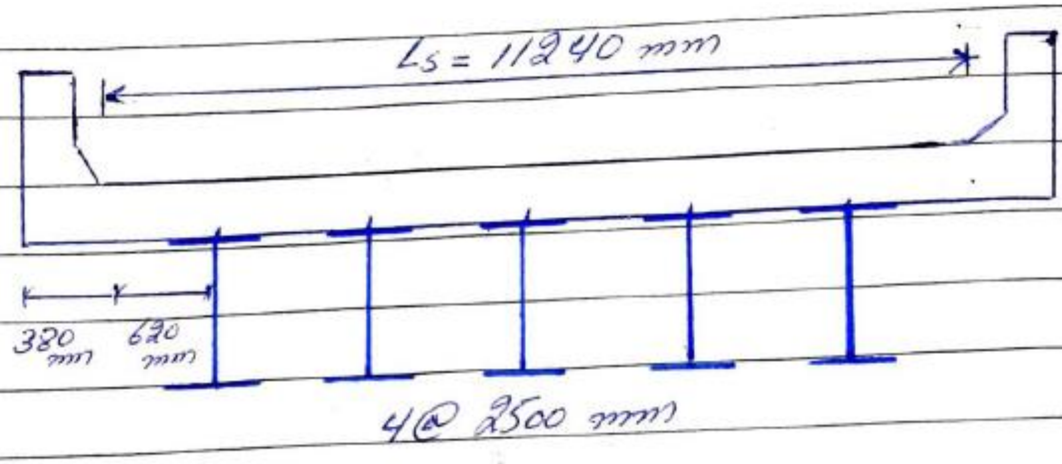
The over hang is generally kept at 35% to 40% of the inner spacings of beams and usual spacings of stringers (beams) is kept at 1.5 m to 3 m .

Lets have 5 stringers @ 4 spacings then

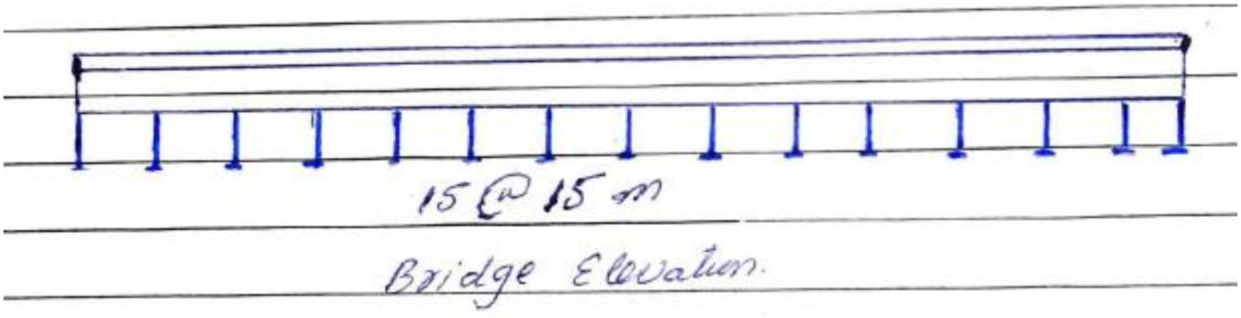
$$0.85 + 45 = w_c + 380 \times 2$$

$$= 11240 + 380 \times 2$$

$$S = 2500\ mm$$



Cross-section of bridge.



Depth of slab

$$h_{min} = \frac{s + 3000}{30} \geq 75\ mm$$

$$= \frac{2500 + 3000}{30}$$

$$= 184\ mm$$

$$\approx 190\ mm$$

Wearing surface

$$h_w = 75\ mm$$

Total depth of slab

$$h_{slab} = 190 + 75$$

$$= 265\ mm$$

Clear cover

Minimum clear cover ontop

$$= 60\ mm$$

Clear cover at the bottom

$$= 25\ mm$$

Effective span of slab

$$s_e = 2500 - assumed\ bf\ of\ selected\ section\ (10\%\ of\ the\ c.c\ span)$$

$$s_e = 2500 - 0.01 \times 25000$$

$$= 2350\ mm$$

$$\frac{s_e}{h_{slab}} = \frac{2350}{190}$$

$$= 12.36\ (btn\ 12\ and\ 18, OK)$$

Core depth

$$= h_{slab} - 60 - 25$$

$$= 265 - 60 - 25$$

$$= 180\ mm > 100mm \qquad OK$$

Slab depth

$$190\ mm > 175\ mm, \qquad OK$$

Overhang

$$= 40\%\ of\ S$$

$$= 40\% \times 2500$$

$$= 1000\ mm > 950\ mm, \qquad OK$$

Bottom layer steel

Minimum steel

$$A_{s,min} = 0.57\ mm^2/mm$$

It is empirically increased by 20% according to the expected increase in the live load.

$$A_{s,min} = 1.2 \times 0.57$$

$$= 0.684\ mm^2/mm$$

Provide #15 @ 250 mm c/c

Top layer steel

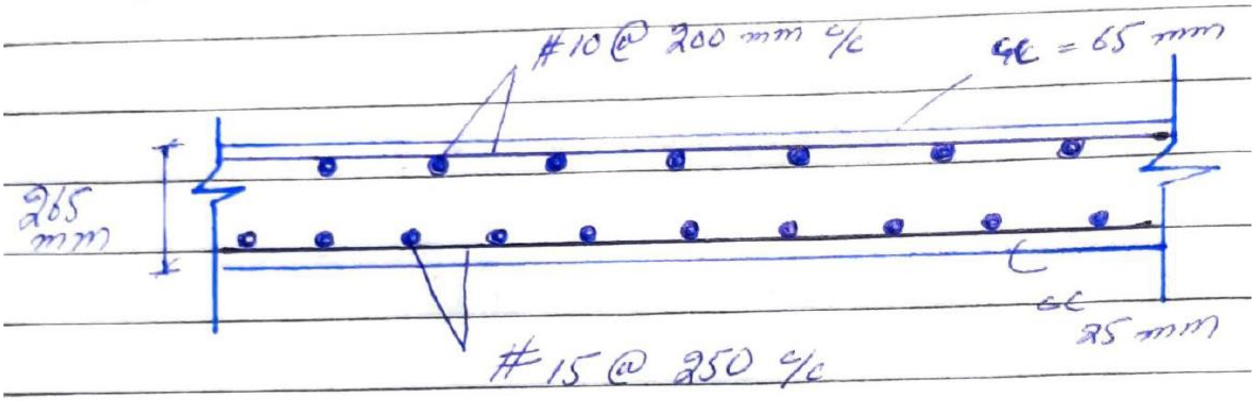
$$A_{s,min} = 0.38\ mm^2/mm$$

It is empirically increased by 20% according to the expected increase in the live load.

$$A_{s,min} = 1.2 \times 0.38$$

$$= 0.456\ mm^2/mm$$

Provide #10 @ 200 mm c/c



Deck slab reinforcement detail

No. of lanes

$$N_L = \frac{W_c}{3600}$$

$$= \frac{11240}{3600}$$

$$= 3$$

Multiple presence factor

For three loaded lanes, the multiple presence factor is 0.85

Dynamic load allowance

$$IM = 33\%\ for\ design\ truck\ and\ tendons$$

$$IM = 0\ for\ lane\ loading$$

AASHTO-LRFD Table 8.6.2.1-1

Distribution factor for moment

Lateral distribution of loads for moments

Interior Girders

One lane loaded

$$s = 2500\ mm$$

$$L_s = 1500\ mm$$

$$g = 0.06 + \left(\frac{s}{4300}\right)^{0.4} \times \left(\frac{s}{L_s}\right)^{0.3} \times (kg/lts)^3$$

$$= 0.06 + \left(\frac{2500}{4300}\right)^{0.4} \times \left(\frac{2500}{15000}\right)^{0.3} \times (1)^3$$

$$= 0.53$$

Two or more design lanes loaded

$$g = 0.075 + \left(\frac{s}{2900}\right)^{0.6} \times \left(\frac{s}{L_s}\right)^{0.2} \times (kg/lts)^{0.1}$$

$$= 0.075 + \left(\frac{2500}{2900}\right)^{0.6} \times \left(\frac{2500}{15000}\right)^{0.2} \times (1)^{0.1}$$

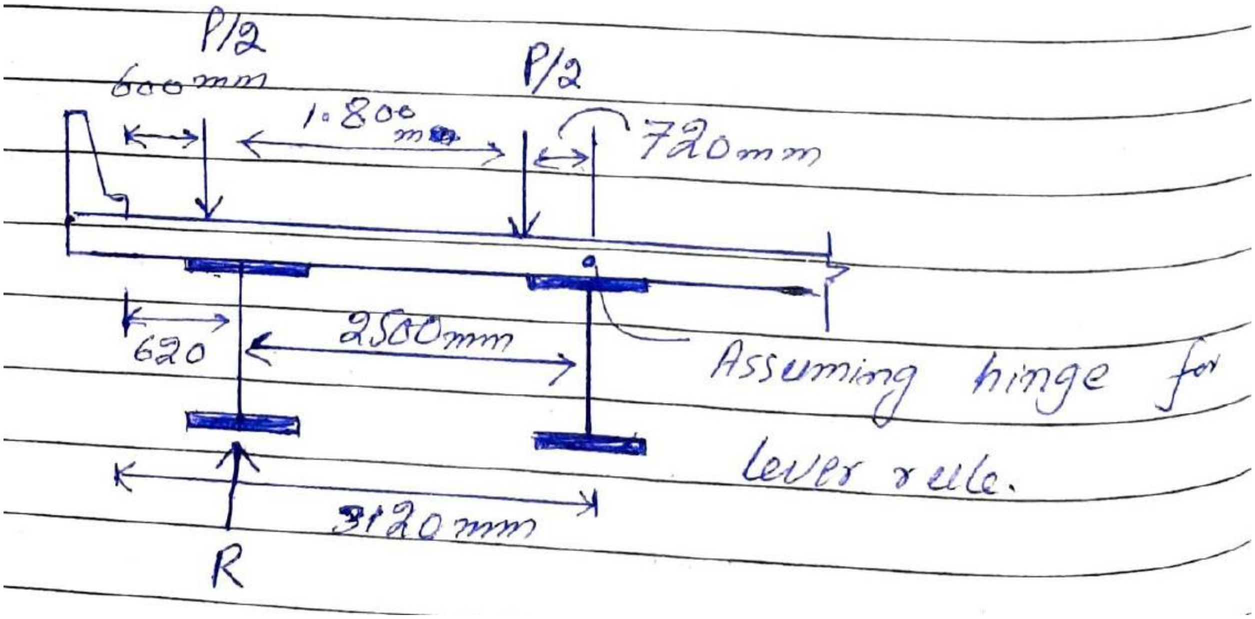
$$= 0.714$$

Exterior girders

One design lane loaded

The arrangement of loads for application of the lever arm rule to get contribution factor for the exterior girder increase of moment as shown

$$P = axle\ load$$



$$M_c = 0$$

$$R \times 2500 = \frac{P}{2} \times 720 + \frac{P}{2} \times 2520$$

$$R = 0.648P$$

$$g = 1.2 \times 0.648$$

Two or more lanes loaded

$$d_e = 1000 - w_b$$

$$= 1000 - 380$$

$$= 620\ mm$$

$$e = 0.77 + \frac{d_e}{2800} \geq 1$$

$$= 0.77 + \frac{620}{2800}$$

$$= 0.99 \; say \; 1$$

So, e=1

$$g = e \times g_{interioe}$$

$$= 1 \times 0.714$$

$$= 0.714$$

HL-93 Loading

Design truck

$$W_a = \sum_{rare\ axle=145\ kN}^{front\ axle=35\ kN}.$$

Design tandem

$$W_p = \sum_{=110\ kN}^{two\ axles\ at\ spacing\ of\ 200\ mm}.$$

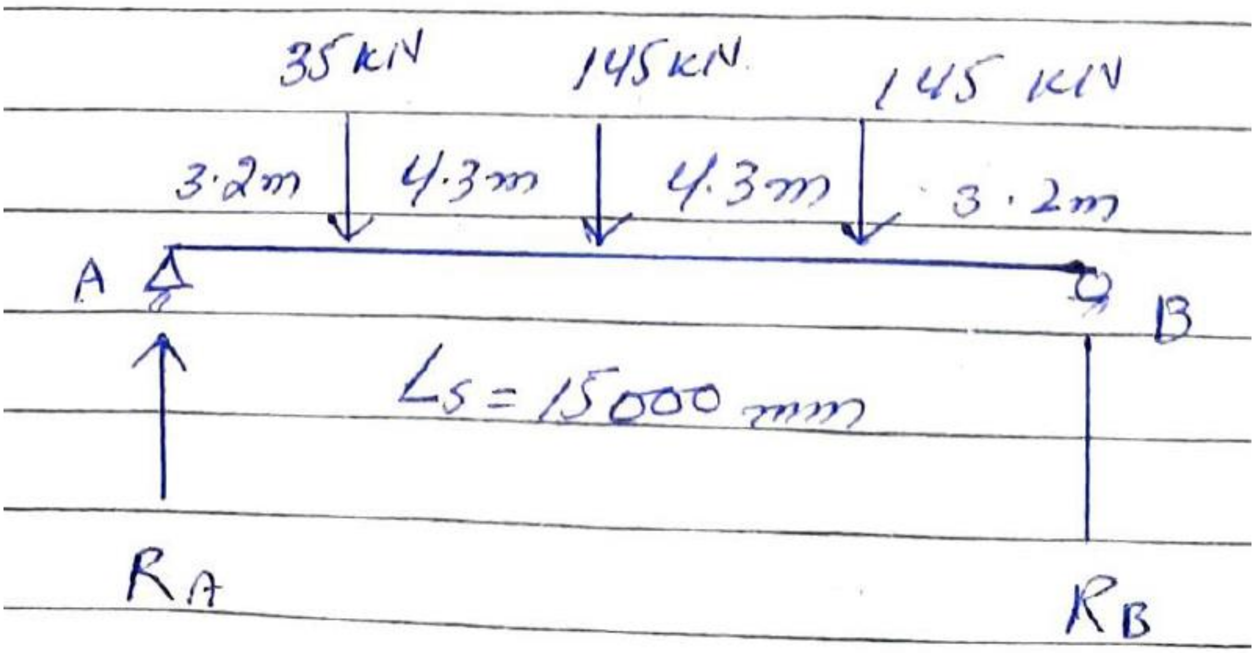
Design lane load

$$W_l = 9.3 \frac{kN}{m}$$

Maximum central live load moments

For standard axle load

Design truck

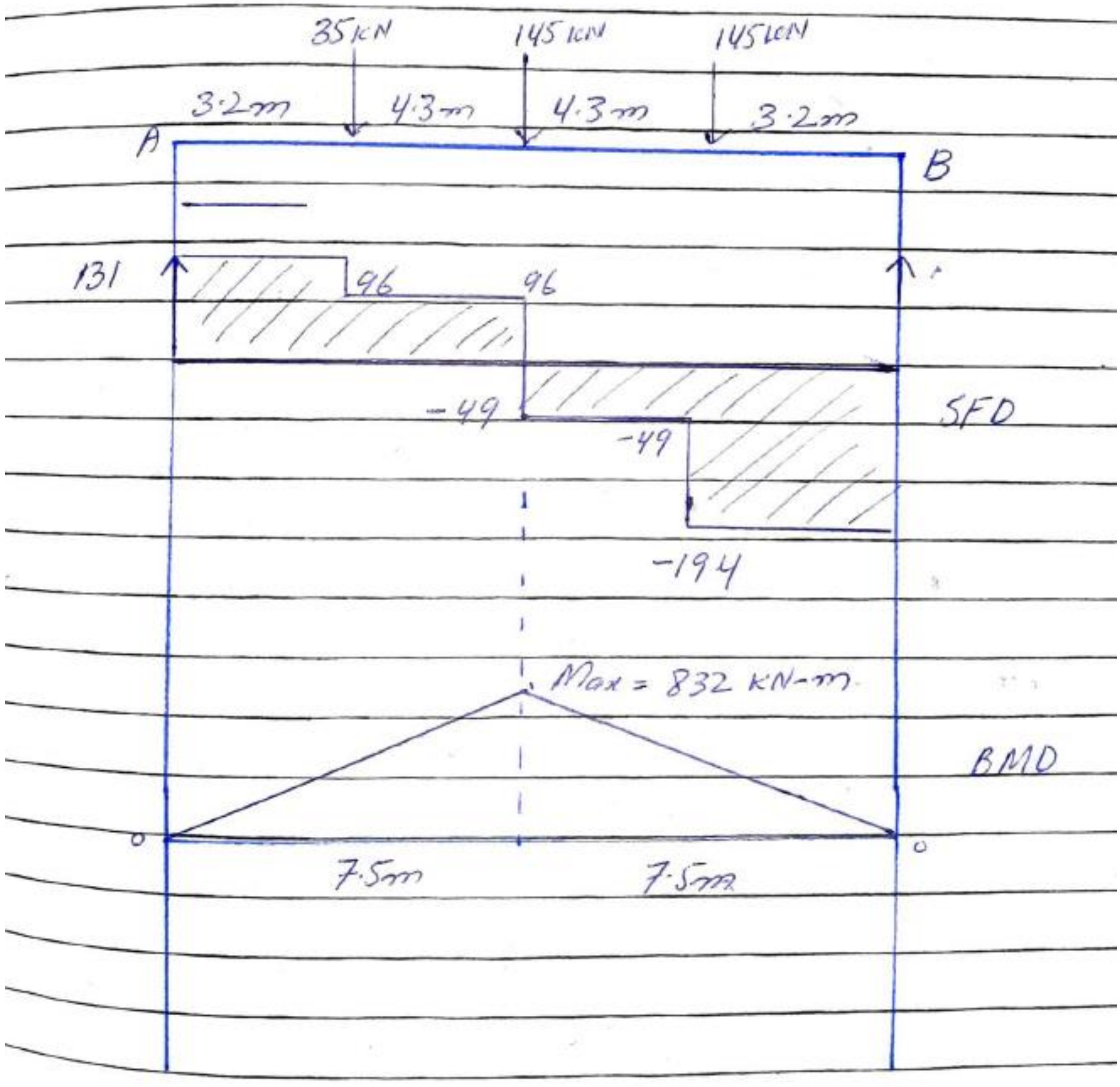


$$\sum M_B = 0$$

$$R_A \times 15 = 35 \times 11.8 + 145 \times 7.5 + 145 \times 3.2$$

$$R_A = 131\ kN$$

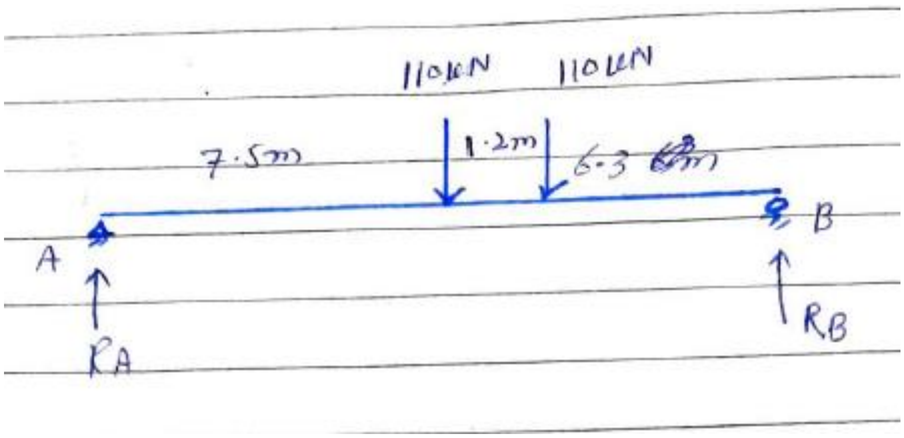
$$R_B = 194\ kN$$



$$V_{max} = 194\ kN$$

$$M_{a,max} = 832\ kNm$$

Design tandem

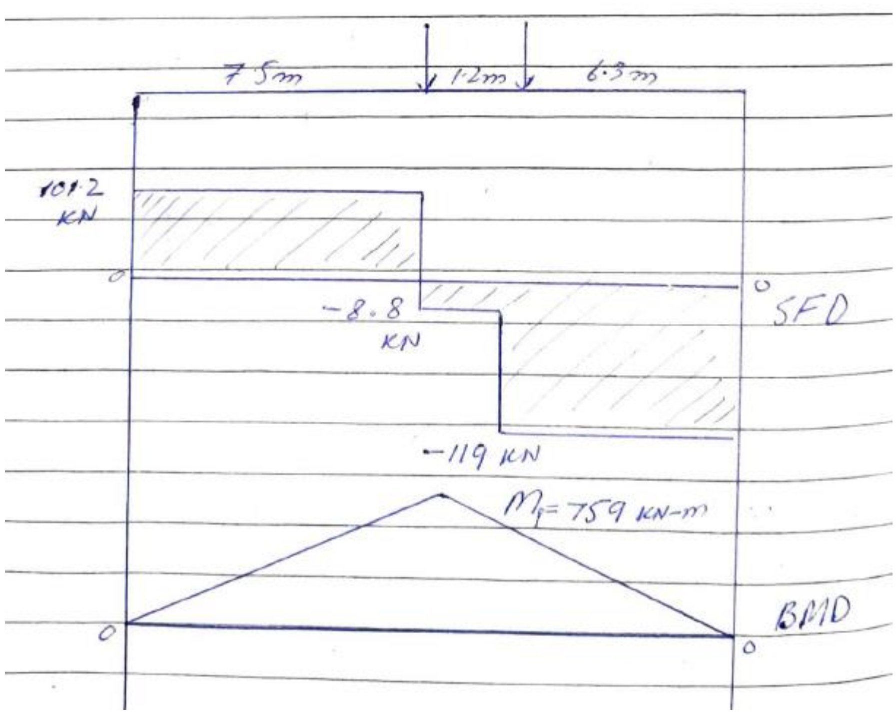


$$\sum M_B = 0$$

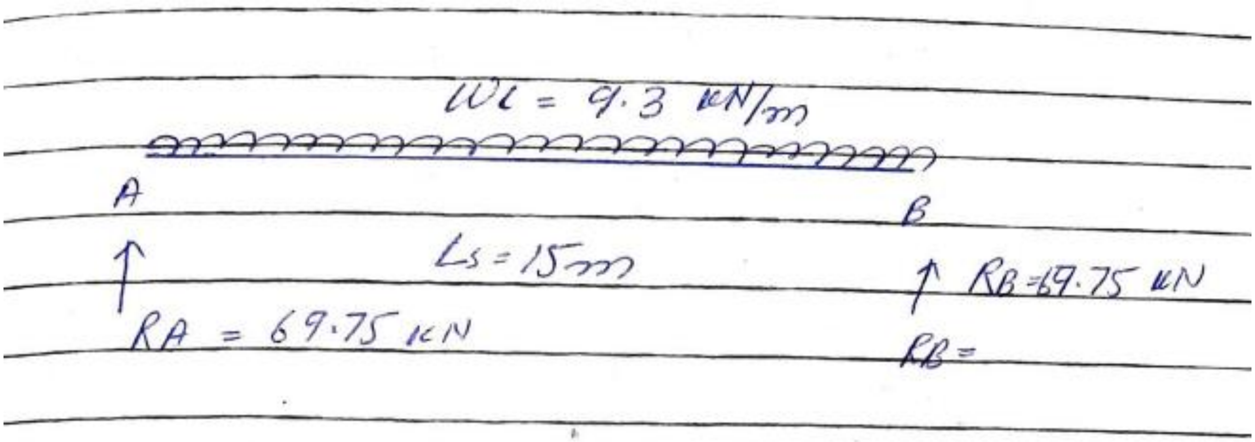
$$R_A \times 15 = 110 \times 110 - +145 \times 6.3$$

$$R_A = 101.2\ kN$$

$$R_B = 119.21\ kN$$



Design lane load



$$M_l = \frac{wl \times l_1^2}{8}$$

$$= \frac{9.3 \times 15^2}{8}$$

$$= 261.56\ kNm$$

Maximum live load and impact moment

For interior beams

$$M_{ll} + IM = g(larger\ of\ M_a and\ M_i) \times F \times \left(1 + \frac{IM}{100}\right) + ML$$

$$= 0.714(832 \times 1.2 \times 1.33 \times 262)$$

$$= 1135.16$$

$$\approx 1136\ kNm$$

For exterior beams

$$M_{ll} + IM = g(larger\ of\ M_a and\ M_i), \times F \times \left(1 + \frac{IM}{100}\right) + ML$$

$$= 0.778(832 \times 1.2 \times 1.33 \times 262)$$

$$= 1236.92$$

$$\approx 1237\ kNm$$

Lateral distribution factor for shear

Interior beams

One design lane loaded

$$g = 0.36 + \frac{s}{7600}$$

$$= 0.36 + \frac{2500}{7600}$$

$$= 0.689$$

Two or more design lanes loaded

$$g = 0.2 + \frac{s}{3600} + \left(\frac{s}{10700}\right)^2$$

$$= 0.2 + \frac{2500}{3600} + \left(\frac{2500}{10700}\right)^2$$

$$= 0.84$$

Exterior beams

One design lane loaded

$$g = 0.778 \; already \; calculated \; for \; moment \; giving \; lower \; value$$

Two or more lanes loaded

$$d_e = 1000 - 380$$

$$= 620 \; mm$$

$$e = 0.6 + \frac{d_e}{3000} \geq 1$$

$$= 0.6 + \frac{620}{3000}$$

$$= 0.807$$

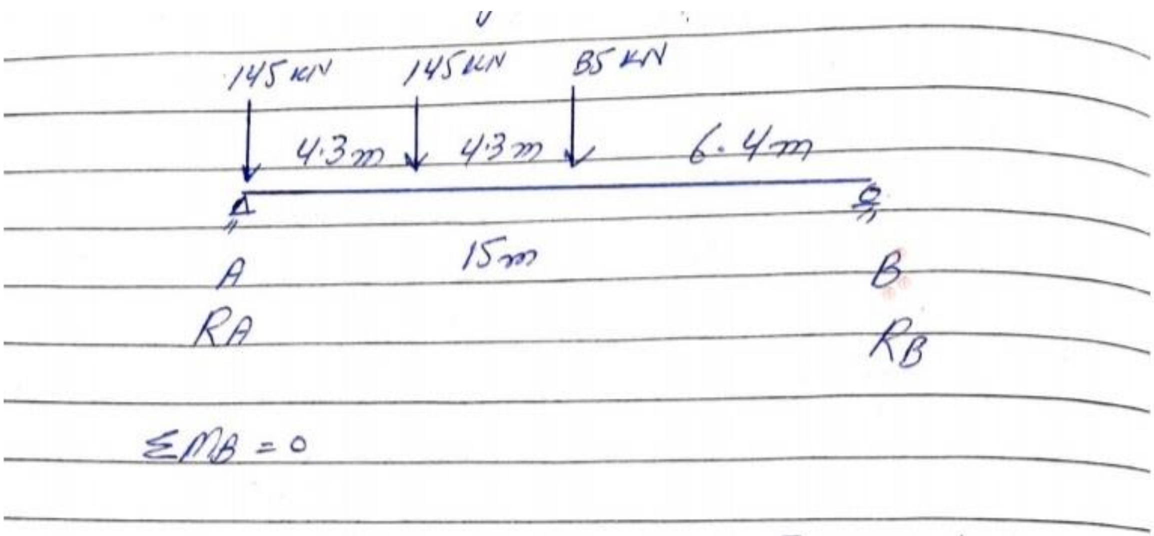
$$g = e \times g_{interioe}$$

$$= 0.807 \times 0.84$$

$$= 0.678$$

Maximum shear

1. **For design Truck**



$\sum MB$=O

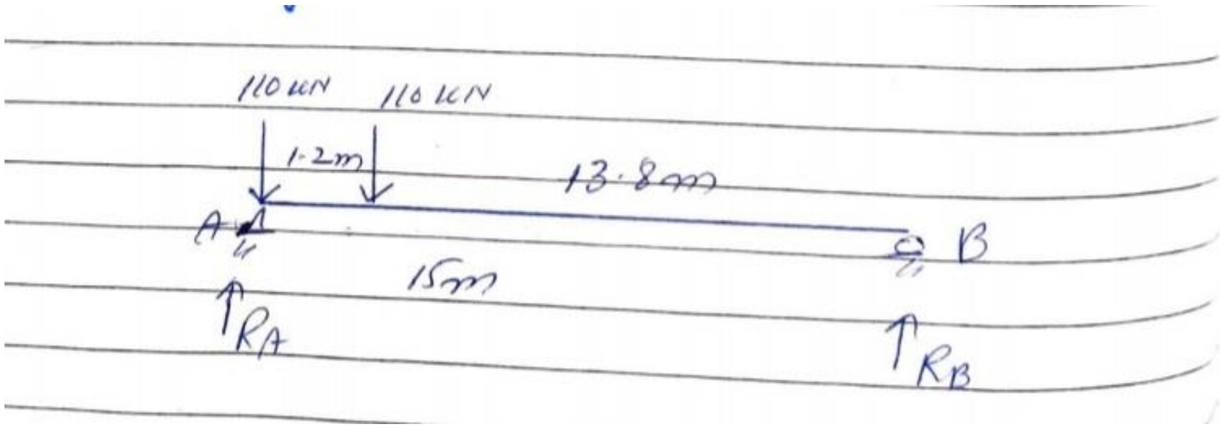
$$RA \times 15 - 145 \times 15 - 145 \times 10.7 - 35 \times 6.4 = 0$$

$$RA = 263.36KN$$

$$RB = 61.64KN$$

$$Va = 264KN$$

For Design Tendam



$\sum MB = 0$

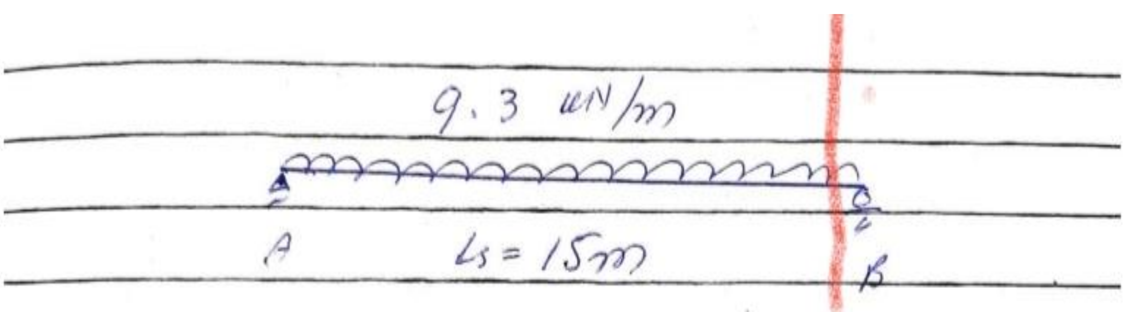
$$RA \times 15 = 110 \times 15 \pm 110 \times 13.8$$

$$RA = 211.2KN$$

$$RB = 8.8KN$$

Vi =213KN

For Design Lane Load



Maximum Live Load and Impact Shear

Interior Girder

$$VLL \pm IM = g\left(Vmax \times \left(1 \pm \frac{IM}{100}\right) \pm VL\right)$$

$$= 0.84(264 \times 1.33 \pm 70$$

$$= 354KN$$

Exterior girders

$$VLL \pm IM = 0.778(264 \times 1.33 \pm 70)$$

$$= 328KN$$

Dead Load Forces

For Interior Girders

Deck slab load=WDs× $c \times \frac{s}{g} \times 2$

$$= \frac{190}{1000} \times 2400 \times 2500 \times 9.81 \times 0.001$$

$$= 11.183KN/M$$

Assume Girder self-weight 15 percent of deck slab

$$= 1.677KN/M$$

WDc= $12.86KN/M$

MDc=$\frac{WDC \times LS^2}{8} = 362KN/M$

VDc= $\frac{Wdc \times Ls}{2}$ =97KN

Weight of wearing Course

$$W_{DW} = \frac{0.075 \times 2250 \times 2.5 \times 9.81}{1000}$$

$$= 4.13KN/M$$

$M_{DW} = W_{DW} \times \frac{LS^2}{8}$

M_{DW} = 117KN/M

$V_{DW} = \frac{WDW \times LS}{2}$

=31KN

For Exterior Girders

Deck slab load =WDc=$\frac{190}{1000} \times \partial c \times \frac{2250}{1000} \times \frac{9.8}{1000}$

=10KN/M

Barrier load 25percent of load due to deck slab

=2.5KN/M

Wearing Course load =$\frac{75}{1000} \times 2250 \times \frac{620}{1000} \times \frac{9.81}{1000}$

=3KN/M

Load due to Deck slab and barrier

W_{DC} = 10 ± 2.5

=12.5KN/M

$M_{DC} = \frac{WDC \times LS^2}{8}$

$=352KN/M$

$VDC = \frac{WDC \times Ls}{2}$

$=94KN$

Load due to 75mm wearing

$W_{DW} = 3KN/M$

$M_{DW} = 84KN/M$

$V_{DW} = 23KN$

FINAL ANALYSIS RESULTS OF SUPERSTRUCTURES

Maximum Live load and Impact moment

- On interior Girder/Beam

  MLL + IM = 1136 KN-m

- For Exterior Girder

  M_{LL} + IM = 1237 KN-m

Maximum Live load Shear:

- On Interior Girder

  V_{LL} + IM = 354 KN

- On exterior Girder

  V_{LL} + IM = 328 KN

Maximum Dead load Moments

- On interior Girder:

  Due to deck slab = M_{DC} =362 KN-m

Due to wearing course = M_{DW} =117 KN-m

Maximum dead load shear

- On interior Girder:

Due to deck slab = V_{DC} =97 KN

Due to wearing course = V_{DW} =31 KN

Maximum Dead load moment on Exterior Girder

Due to deck slab

M_{DC} = 352 KN-m

Due to 75mm wearing

M_{DW} = 84 KN-m

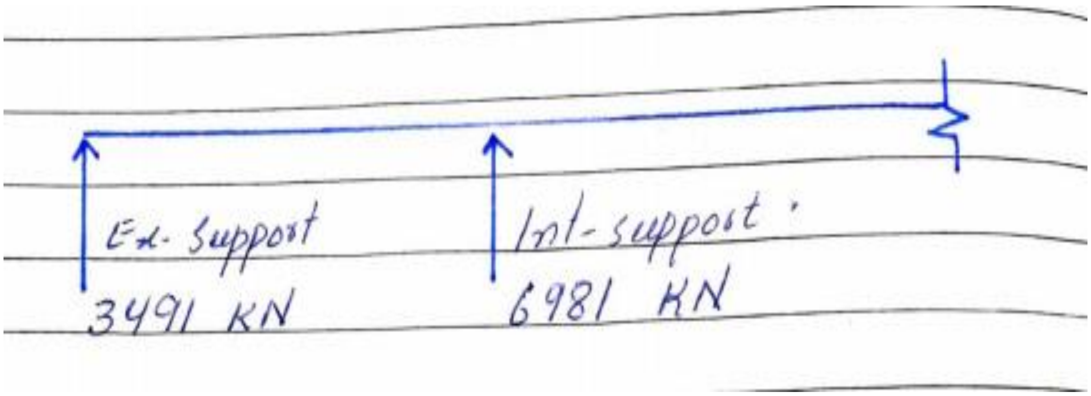
Maximum shear due to deck slab

V_{DC} = 94 KN

Due to 75mm wearing

V_{DW} = 23 KN

Total dead load and live load reaction at Exterior support and Interior supports.



Total Reaction on Exterior Support.

Reaction due to live load of three interior girder and two exterior girders.

$RL_{EXT} = 3 * 354 + 2*328$

$= 1718$ KN

Reaction due to dead loads.

$RD_{EXT} = 3 * (97 + 31) + 2 * (94 + 23)$

$= 618$ KN

Factored Reaction = $1.2 (RD_{EXT}) + 1.6 (RL_{EXT})$

$RU_{EXT} = 1.2 (618) + 1.6 (11718)$

$= 3491$ KN

Total Reaction on Interior Support.

$RL_M = 2 * RL_{EXT}$

$= 2 * 1718$

$= 3436$ KN

$RD_{IN} = 2 * RD_{EXT}$

$= 2 * 618$

$= 1236$ KN

Factored Reaction on Interior Support.

$RU_{IN} = 1.2\ (1236) + 1.6\ (3436)$

$= 6980.8$

$= 6981$ KN

Wind Load (AASHTO-LRFD) Bridge design specifications

Pressure bearing is assumed to be caused by base design wind velocity, $V_{B,}$ of 100 mph (45 m/s).

Wind load shall be uniformly distributed on area exposed to wind.

Wind pressure on structure.

$P_D = P_B \frac{VDZ^2}{10000}$ (Table 3.8.1.2.1 – 1)

P_B = base wind pressure, Table 3.8.1.2.1 – 1

For beams P_B = 0.05 ksf (2.4 KN/m^2)

V_{DZ} = design wind speed at elevation, Z (mph)

Assume V_{DZ} = 30 mph at z = 20 ft.

$$P_D = 0.05\left(\frac{130^2}{10{,}000}\right) = 0.0845\ ksf$$

$$P_D = 4\ KN/m^2$$