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 overhang 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.

Let's have 5 stringers @ 4 spacings then

\[
0.85 + 45 = w_c + 380 \times 2
\]

\[
= 11240 + 380 \times 2
\]

\[
S = 2500 \text{ mm}
\]
**Depth of slab**

\[
h_{\text{min}} = \frac{s + 3000}{30} \geq 75 \text{ mm}
\]

\[
= \frac{2500 + 3000}{30}
\]

\[
= 184 \text{ mm}
\]

\[
\approx 190 \text{ mm}
\]

**Wearing surface**

\[
h_w = 75 \text{ mm}
\]

**Total depth of slab**

\[
h_{\text{slab}} = 190 + 75
\]

\[
= 265 \text{ mm}
\]
Clear cover

Minimum clear cover ontop

\[ = 60 \, mm \]

Clear cover at the bottom

\[ = 25 \, mm \]

Effective span of slab

\[ s_e = 2500 - \text{assumed bf of selected section} \, (10\% \, of \, the \, c.c \, span) \]

\[ s_e = 2500 - 0.01 \times 25000 \]

\[ = 2350 \, mm \]

\[ \frac{s_e}{h_{slab}} = \frac{2350}{190} \]

\[ = 12.36 \, (btn \, 12 & \, 18, OK) \]

Core depth

\[ = h_{slab} - 60 - 25 \]

\[ = 265 - 60 - 25 \]

\[ = 180 \, mm > 100 \, mm \, \, OK \]

Slab depth

\[ 190 \, mm > 175 \, mm, \, \, OK \]
Overhang

\[ = 40\% \text{ of } S \]
\[ = 40\% \times 2500 \]
\[ = 1000 \text{ mm} > 950 \text{ mm}, \quad \text{OK} \]

Bottom layer steel

Minimum steel

\[ A_{s,\text{min}} = 0.57 \text{ mm}^2/\text{mm} \]

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

\[ A_{s,\text{min}} = 1.2 \times 0.57 \]
\[ = 0.684 \text{ mm}^2/\text{mm} \]

Provide #15 @ 250 mm c/c

Top layer steel

\[ A_{s,\text{min}} = 0.38 \text{ mm}^2/\text{mm} \]

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

\[ A_{s,\text{min}} = 1.2 \times 0.38 \]
\[ = 0.456 \text{ mm}^2/\text{mm} \]

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**

\[ IM = 33\% \text{ for design truck and tendons} \]

\[ IM = 0 \text{ for lane loading} \]

AASHTO-LRFD Table 8.6.2.1-1

**Distribution factor for moment**
Lateral distribution of loads for moments

**Interior Girders**

One lane loaded

\[ s = 2500 \text{ mm} \]
\[ L_s = 1500 \text{ mm} \]

\[ g = 0.06 + \left( \frac{s}{4300} \right)^{0.4} \times \left( \frac{s}{L_s} \right)^{0.3} \times \left( \frac{\text{kg/lts}}{1} \right)^{3} \]

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

\[ = 0.53 \]

Two or more design lanes loaded

\[ g = 0.075 + \left( \frac{s}{2900} \right)^{0.6} \times \left( \frac{s}{L_s} \right)^{0.2} \times \left( \frac{\text{kg/lts}}{1} \right)^{0.1} \]

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

\[ = 0.714 \]

**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
\[ P = \text{axle load} \]

\[ M_c = 0 \]

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

\[ R = 0.648P \]

\[ g = 1.2 \times 0.648 \]

Two or more lanes loaded

\[ d_e = 1000 - w_b \]

\[ = 1000 - 380 \]

\[ = 620 \text{ mm} \]
\[ e = 0.77 + \frac{d_e}{2800} \geq 1 \]

\[ = 0.77 + \frac{620}{2800} \]

\[ = 0.99 \text{ say 1} \]

So, \( e = 1 \)

\[ g = e \times g_{\text{interioe}} \]

\[ = 1 \times 0.714 \]

\[ = 0.714 \]

**HL-93 Loading**

Design truck

\[ W_a = \sum_{\text{front axle}=35 \, kN}^{\text{rare axle}=145 \, kN} \]

Design tandem

\[ W_p = \sum_{\text{two axles at spacing of } 200 \, mm}^{\text{two axles at spacing of } 200 \, mm} \sum_{=110 \, kN} \]

Design lane load

\[ W_l = 9.3 \frac{kN}{m} \]

**Maximum central live load moments**

**For standard axle load**
Design truck

\[ \sum M_B = 0 \]

\[ R_A \times 15 = 35 \times 11.8 + 145 \times 7.5 + 145 \times 3.2 \]

\[ R_A = 131 \text{ kN} \]

\[ R_B = 194 \text{ kN} \]
\[ V_{\text{max}} = 194 \, kN \]

\[ M_{a,\text{max}} = 832 \, kN\text{m} \]
Design tandem

\[
\sum M_B = 0
\]

\[
R_A \times 15 = 110 \times 110 + 145 \times 6.3
\]

\[
R_A = 101.2 \text{ kN}
\]

\[
R_B = 119.21 \text{ kN}
\]
Design lane load

\[ WL = 9.3 \, \text{kN/m} \]

\[ A \]

\[ L_s = 15 \, \text{m} \]

\[ RA = 69.75 \, \text{kN} \]

\[ RB = \]

\[ M_l = \frac{wl \times l^2}{8} \]

\[ = \frac{9.3 \times 15^2}{8} \]

\[ = 261.56 \, kNm \]

Maximum live load and impact moment

For interior beams

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

\[ = 0.714 \left( 832 \times 1.2 \times 1.33 \times 262 \right) \]

\[ = 1135.16 \]

\[ \approx 1136 \, kNm \]
For exterior beams

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

\[ = 0.778(832 \times 1.2 \times 1.33 \times 262) \]

\[ = 1236.92 \]

\[ \approx 1237 \text{ kNm} \]

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

\[ g = 0.778 \text{ already calculated for moment giving lower value} \]

Two or more lanes loaded

\[ d_e = 1000 - 380 \]
\[ = 620 \text{ mm} \]

\[ e = 0.6 + \frac{d_e}{3000} \geq 1 \]
\[ = 0.6 + \frac{620}{3000} \]
\[ = 0.807 \]

\[ g = e \times g_{\text{interioe}} \]
\[ = 0.807 \times 0.84 \]
\[ = 0.678 \]
Maximum shear

1. For design Truck

\[ \sum MB = 0 \]

\[ RA \times 15 - 145 \times 15 - 145 \times 10.7 - 35 \times 6.4 = 0 \]

\[ RA = 263.36KN \]

\[ RB = 61.64KN \]

\[ Va = 264KN \]

For Design Tendam
\[ \sum MB = 0 \]

\[ RA \times 15 = 110 \times 15 \pm 110 \times 13.8 \]

\[ RA = 211.2\text{KN} \]

\[ RB = 8.8\text{KN} \]

\[ Vi = 213\text{KN} \]

**For Design Lane Load**

**Maximum Live Load and Impact Shear**
Interior Girder

\[ VLL \pm IM = g \left( V_{max} \times \left( 1 \pm \frac{IM}{100} \right) \pm VL \right) \]
\[ = 0.84(264 \times 1.33 \pm 70) \]
\[ = 354KN \]

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 × \( \frac{s}{g} \) × 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

\[ W_{dw} = \frac{0.075 \times 2250 \times 2.5 \times 9.81}{1000} \]
\[
\begin{align*}
M_{DW} &= W_{DW} \times \frac{L_s^2}{8} \\
M_{DW} &= 117KN/M \\
V_{DW} &= \frac{W_{DW} \times L_s}{2} \\
= 31KN
\end{align*}
\]

For Exterior Girders

Deck slab load \(=W_{Dc} = \frac{190}{1000} \times \frac{c}{2250} \times \frac{2250}{1000} \times \frac{9.8}{1000} \)

\(= 10KN/M \)

Barrier load 25 percent of load due to deck slab

\(= 2.5KN/M \)

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

\(= 3KN/M \)

Load due to Deck slab and barrier

\[
W_{DC} = 10 \pm 2.5
\]

\(= 12.5KN/M \)
\[
M_{DC} = \frac{W_{DC} \times Ls^2}{8}
\]

= 352 KN/M

\[
V_{DC} = \frac{W_{DC} \times Ls}{2}
\]

= 94 KN

Load due to 75mm wearing

\[W_{DW} = 3 \text{ KN/M}\]

\[M_{DW} = 84 \text{ KN/M}\]

\[V_{DW} = 23 \text{ KN}\]
FINAL ANALYSIS RESULTS OF SUPERSTRUCTURES

Maximum Live load and Impact moment

- On interior Girder/Beam

\[ M_{LL} + IM = 1136 \text{ KN-m} \]

- For Exterior Girder

\[ M_{LL} + IM = 1237 \text{ KN-m} \]

Maximum Live load Shear:

- On Interior Girder

\[ V_{LL} + IM = 354 \text{ KN} \]

- On exterior Girder

\[ V_{LL} + IM = 328 \text{ KN} \]

Maximum Dead load Moments

- On interior Girder:

Due to deck slab = \( M_{DC} = 362 \text{ KN-m} \)
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.

\[
RL_{EXT} = 3 \times 354 + 2 \times 328
\]

\[
= 1718 \, \text{KN}
\]

Reaction due to dead loads.

\[
RD_{EXT} = 3 \times (97 + 31) + 2 \times (94 + 23)
\]

\[
= 618 \, \text{KN}
\]

Factored Reaction = 1.2 (RD\text{\textsubscript{EXT}}) + 1.6 (RL\text{\textsubscript{EXT}})

\[
RU_{EXT} = 1.2 \times 618 + 1.6 \times 11718
\]

\[
= 3491 \, \text{KN}
\]

Total Reaction on Interior Support.
\[ RL_M = 2 \times RL_{EXT} \]
\[ = 2 \times 1718 \]
\[ = 3436 \text{ KN} \]

\[ RD_{IN} = 2 \times RD_{EXT} \]
\[ = 2 \times 618 \]
\[ = 1236 \text{ KN} \]

Factored Reaction on Interior Support.

\[ RU_{IN} = 1.2 \times (1236) + 1.6 \times (3436) \]
\[ = 6980.8 \]
\[ = 6981 \text{ KN} \]

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.

\[ P_D = P_B \frac{VDZ^2}{10000} \]  \( \text{(Table 3.8.1.2.1 – 1)} \)
$P_B = \text{base wind pressure, Table 3.8.1.2.1 – 1}$

For beams $P_B = 0.05 \text{ ksf } (2.4 \text{ KN/m}^2)$

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

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

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

$$P_D = 4 \text{ KN/m}^2$$