ADDIS ABABA UNIVERSITY
ADDIS ABABA INSTITUTE OF TECHNOLOGY
SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

RC T-GIRDER BRIDGE DESIGN

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T-GIRDER BRIDGE DESIGN

Design Data and Specifications

i) Material Properties

Steel strength, f_y=	400	MPa
Concrete strength, f'_c =	28	MPa
Concrete density, y_c =	2400	kg/m^3
Bituminous density, y_b =	2250	kg/m^3
The modulus of elasticity of steel, E_s =	200	GPa

ii) Bridge Span and Support Dimensions

Clear span of the bridge,C_s =	17	m
Road way/ clear carriage width, R_w =	7.32	m
Bottom width of the concrete barrier/ post, B_c=	0.35	m
Curb width including B_c, (B_c+C_w) =	1.35	m
Curb depth C_d =	0.25	m
Bearing shelf width, W_{rs} =	0.5	m
Girder Spacing, G_s =	3	m
Diaphragm Spacing, D_s =	5	m
Number of Diaphragm/s=	4	
Skewness=	0	°
Concrete Barrier wall is used.	5	kN/m
Thickness of Asphalt Layer (Wearing Surface) =	100	mm
Concrete Cover for the slab (bottom) =	25	mm
Concrete Cover for the slab (top) =	60	mm
Concrete Cover for the girders =	50	mm

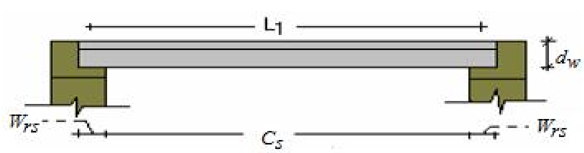
iii) Reinforcement Sizes

Diameter of main reinforcement for the girders =	32	mm
Diameter of main reinforcement for the slab =	16	mm
Diameter of distribution reinforcement =	12	mm
Diameter of temperature reinforcement =	12	mm
Diameter of shear reinforcement =	12	mm

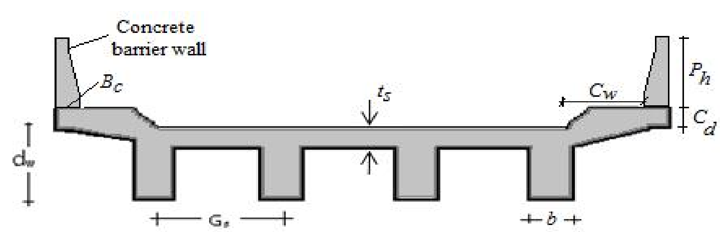
*** Design Method: LRFD**

*** Specifications:**

- AASHTO LRFD Bridge Design Specifications, 4th ed. 2007
- Ethiopian Roads Authority, ERA Bridge Design Manual, 2013



Longitudinal section of aT-Girder Bridge



Cross section of a RC T-Girder Bridge

T-Girder Bridge Design (17.5 m, c/c spacing of bridge)

1. Typical Section

1.1. Deck Thickness

Minimum recommended thickness of the slab, to control deflection is $t_{s,min}$=185 mm.(ERA BDM 2013, Article 5.4.1.1)
Deck slab thickness not less than the clear span between fillets, haunches, or webs divided by 20, unless transverse ribs at a spacing equal to the clear span are used [AASHTO Art. 5.14.1.5.1a]

$$D = \frac{(S+3000)}{30} \quad =200mm > 185mm \quad (Gs=3m)$$

Minimum recommended depth for continuous slabs, AASHTO (Table 2.5.2.6.3-1)

where: D is the thickness of the slab and S is c/c spacing of girders

Use ts= 220 mm

1.2. Web Thickness

Minimum thickness of the web, b_{min}=200 mm without prestressing duct. [AASHTO Art. 5.14.1.3.1c]
Assume 4 bars in one row are used:

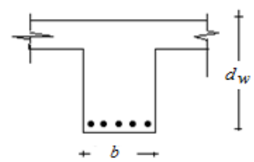
b= 4Φ+3(1.5Φ)+2*side cover =322mm [AASHTO Art. 5.10.3.1.1]

Use b= 360 mm

1.3. Girder Depth (AASHTO Table 2.5.2.6.3-1)

Structural depth, Dw = 0.07L for simple span T- beams (L=C_s +W_{rs})
dw=0.07*17500= 1225

Use dw= 1230 mm



b= 360 mm
dw = 1230 mm

1.4. Girder Spacing and overhang

Numbers of girders, N_g =INT(R_w/G_s)+1

N_g = 3 (Three girders with c/c spacing of 3 m are used.)

C_x=0.5(R_w-(N_g-1)G_s) = 0.66 m

B_c+C_x+C_w= 2.01 m (Overhang)

where:

B_c= Bottom width of the barrier (=0.35 m)
C_x= distance from the edge of the curb to the centerline of the exterior girder (=0.66 m)
C_w= Curb width (=1 m)

Number of lanes loaded = Int (7.32/3.6)=2

2. Deck Design

ts = 220 mm (deck thickness)

2.1. Weight of components

1) Slab (220mm thick)	=0.22*2400*9.81/1000	= 5.18 kN/m2
2) Wearing surface (100mm thick)	= 0.1*2250*9.81/1000	=2.21 kN/m2
3) Curb (250mm above slab)	=0.25*2400*9.81/1000	=5.89 kN/m2
4) Barriers		= 5 kN/m

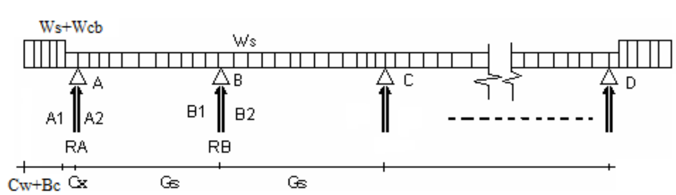
2.1. Dead load moments

An approximate method of analysis in which the deck is subdivided into strips perpendicular to the supporting components shall be considered acceptable for decks. (AASHTO, Article 4.6.2.1.1)

The strips shall be treated as continuous beams with span length equal to the c/c distance between girders. The girders are assumed rigid. For case in applying load factors, the bending moments will be determined for slab dead load, wearing surface and vehicle loads separately.

2.1.1 Slab dead load

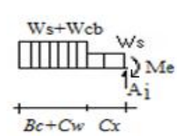
A one-meter strip width is taken for the analysis.



wt. of the curb, W_{cb}= 5.89
wt. of the slab, W_s= 5.18

$$M_e = \frac{W_s C_x^2}{2} + (W_s + W_{cb})(C_w + B_c)\left(\frac{C_w + B_c}{2} + C_x\right)$$

= 5.18*0.66^2/2+11.07*1.35*(1.35/2+0.66) =21.08 kN-m/m



Moment at supports

Using Influence segment coefficent method, moment at supports become, $M_b = \alpha_1 w l^2 + M_e \alpha_2$

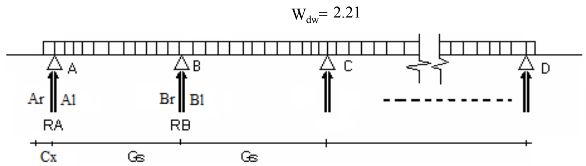
M_b=M_c = 5.18*(-0.125)*3^2+21.08*(-0.5)= -16.37 kN-m/m

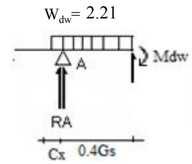
A_1= 18.363kN/m	B_1= 6.2kN/m	R_A= 27.703kN/m
A_2= 9.34kN/m	B_2= 7.77kN/m	R_B= 13.97kN/m

where:

- α_1 and α_2 are influence segment coefficients for moment (obtained from Tables)
- M_e = End moment at support A (exterior girder)
- M_b and M_c are support moments at supports B and C respectively
- B_1, B_2, A_1 and A_2 are shear forces at the left and right of supports B and A respectively.
- R_B and R_A are reactions at supports B and A respectively

2.1.2 Wearing Surface





M_{ew}=W_{dw}*C_x^2/2 =0.48 kN-m/m

Moment at supports

M_{bw}=M_{cw} = 2.21*(-0.125)*3^2+0.48*(-0.5) =-2.726 kN-m/m

A_r= 1.459kN/m	B_r= 4.064kN/m	R_A= 4.025kN/m
A_l= 2.566kN/m	B_l= 3.315kN/m	R_B= 7.379kN/m

M_{dw}= 0.5W_{dw}(C_x+0.4G_s)^2-R_A(0.4G_s), M_{dw} =1.007 kN-m/m

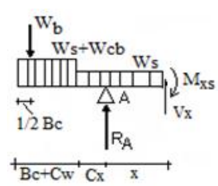
where

M_{dw}= Span moment due to wt. of the wearing surface (at 0.4*G_s from the ext. girder)
M_{ew} = end moment at support A due to wearing surface (exterior girder)
M_{bw} and M_{cw} are support moments at supports B and C due to wearing surface respectively
B_l, B_r are shear forces at the left and right of supports B respectively
R_A and R_B are reactions at supports A and B respectively

2.1.3 Span Moment due to slab dead load and barrier wteight (at 0.4Gs from the ext. girder)

R_A = 27.703+5 =32.703 kN/m

$$M_{xs} = W_s \frac{(C_x + x)}{2} + W_t\left(C_w + B_c\right)\left(\frac{C_w + B_c}{2} + C_x + x\right) + W_b\left(0.5B_c + C_w + x\right) - xR_A$$



M_{xs}= 22.776kN-m/m

where: $W_t = W_s + W_{cb}$ and $x = 0.4G_s$

x = location of maximum span moment at 0.4Gs

M_{xs}=Max span moment due to wt. of the slab, curbs and end barrier/posts (at 0.4Gs from the ext. girder)

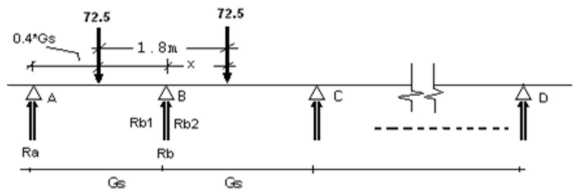
3. Vehicular Live Load

When decks are designed using the approximate strip method (Art. 4.6.2.1) and the strips are transverse, they shall be designed for the 145kN axle load (Art. 3.6.1.3.3). The design truck load shall be positioned transversely to produce maximum force effects.

3.1. Maximum Positive Live Load Moment

For repeating equal spans, the maximum positive bending moment occurs near the 0.4 points of the first interior span. The equivalent width of the strip over which the live load is applied is:

E=660+0.55G_s = 2310 mm (AASHTO, Table 4.6.2.1.3.1)



$0.4G_s$= 1.2 m

Location of wheels for maximum positive live load moment

Position of the second wheel = $0.4G_s+1.8$ = 3 m < 3 m (within the same span)

Joint	A	B		C		D
Member	AB	BA	BC	CB	CD	DC
DF	1.000	0.429	0.571	0.500	0.500	0.571
FEM	31.320	-20.880	0.000	0.000	0	0
CO	-31.320	-15.660				
Bal		15.660	20.880			
CO				10.440		
Bal				-5.220	-5.220	
CO			-2.610			
Bal		1.119	1.491			
CO				0.746		
Bal				-0.373	-0.373	
Final Moment	**0.000**	**-19.761**	**19.761**	**5.593**	**-5.593**	**0.000**

Using moment distribution method, the end moments become:

M_{BA}= $-M_{BC}$ -19.761 kN-m

Reaction due to the above loading

R_a= 36.913 kN　　　R_b= 112.81 kN

R_{b1}= 108.087 kN　　　M_{maxp} = 44.296 kN-m

R_{b2}= 4.723 kN

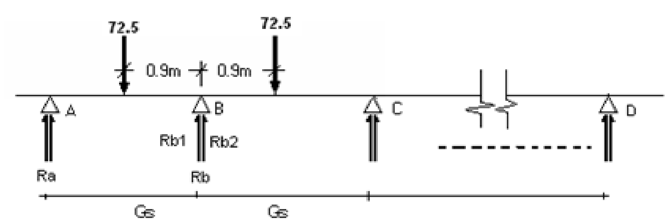
M_p= 1.2*44.296 kN-m/2.31m =23.011 kN-m/m

R_a= 1.2*36.913/2.31m =19.176 kN/m

R_b= 1.2*112.81 kN/2.31m =58.603 kN/m

3.2. Maximum Interior Negative live load moment

The critical placement of live load for maximum negative moment is at the first interior support.
The width of equivalent transverse strip is E=1220+0.25G_s (AASHTO, Table 4.6.2.1.3.1)



The equivalent width, E= 1800 mm

Joint	A	B		C		D
Member	AB	BA	BC	CB	CD	DC
DF	1.000	0.429	0.571	0.500	0.500	0.571
FEM	15.493	-24.445	24.445	-15.493	0	0
CO	-15.493	-7.746				
Bal		3.320	4.427			
CO				2.213		
Bal				6.640	6.640	
CO			3.320			
Bal		-1.423	-1.897			
CO				-0.949		
Bal				0.474	0.474	
Final Moment	**0.000**	**-30.294**	**30.294**	**-7.114**	**7.114**	**0.000**

Using moment distribution method, the end moments become:

M_{BA}= -M_{BC} -30.294 kN-m

R_a= 15.067 kN R_b= 111.799 kN

R_{b1}= 57.433 kN M_{maxN} = -30.294 kN-m

R_{b2}= 54.366 kN

M_{maxN} = 1.2*-30.294 kN-m/1.8m =-20.196 kN-m/m

R_b= 1.2*111.799 kN/1.8m =74.533 kN/m

4. Select Resistance Factors

Strength Limit States (RC)

Flexure & Torsion

⟶ Resistance Factor = 1

5. Select Load Modifiers

i) Strength Limit State

i) Ductility	0.95	[Art. 1.3.3]
ii) Continuous	0.95	[Art. 1.3.4]
iii) Importancy	1.05	[Art. 1.3.5]
⟶ Load Modifier =	0.95	

6. Select Applicable Load Combinations

i) Strength Limit State   U=0.95(1.25DC+1.50DW+1.75(LL+IM)+1.00(FR+TG)

7. Investigate Strength Limit State

Temperature gradient effect reduces gravity load effects.Because temperature gradient may not be there, assume TG=0

Thus, U=0.95(1.25DC+1.50DW+1.75(LL+IM))

8. Design Moment Computaions

M_{maxp}= 0.95*[1.25(Mxs)+1.5(Mdw)+1.75*1.33(Mp)]= 79.362 kN-m/m

$M_{sup\text{-}B}$= 0.95*[1.25(Mb+Mbb)+1.5(Mbw)+1.75*1.33(Mn)]= -73.442 kN-m/m

For reinforcement computation, negative moment may be taken at the face of the support. Thus, calculate moments at the face of support B (the width of the beam, b=360mm)

⟶ MDc=-15.055kN-m/m, MDw=-2.165kN-m/m and MLL=-13.672kN-m/m

M_{neg}= 0.95*[1.25*(-15.055)+1.5*(-2.165)+1.75*1.33*(-13.672)]-51.193 kN-m/m

9. Reinforcement

a) Positive Moment Reinforcement

dp=187 mm (effective depth,dp=220-16/2-25)

Mmaxp= 79.362 kN-m/m

$$\rho = \left(1-\sqrt{1-\frac{2M_u}{0.9bd^2\varphi f'_c}}\right)\frac{\varphi f'_c}{f_y} \qquad \rho_{min} = \frac{0.03 f_c'}{f_y} \qquad \varphi=0.85$$

Checking the adequacy of the section

The section is checked for the maximum design moment whether the initial depth under consideration is sufficed or not.

$$d_{ic}=\sqrt{\frac{M_{max}}{0.254bf_c'}}$$ =107.47mm The section is sufficed!

ρ =0.0067 (Using 16 mm diameter reinforcing bars)

ρ > ρ min Ok!

As=0.0067*1000*187 = 1248.95mm2

S =161mm ⟶ Use Φ 16 c/c 160 mm (As, provided= 1256 mm2)

b) Negative Moment Reinforcement

dn=152 mm (effective depth,dn=220-16/2-60)

M_{neg}= -51.193 kN-m/m

ρ =0.0065 (Using 16 mm diameter reinforcing bars)

ρ > ρ min Ok!

As=0.0065*1000*152 = 989.67mm2

S =203mm ⟶ Use Φ 16 c/c 200 mm (As, provided= 1004.8 mm2)

c) Distribution Reinforcement (AASHTO Article 5.14.4.1)

The amount of distribution reinforcement at the bottom of the slab to distribute the loads may be taken as a percentage of the primary reinforcement and a minimum spacing of 250mm.

P_e= min[67,3840/√Se]

P_e = Percentage of distribution reinforcement

S_e = Clear spacing of girders = G_s-b

Se = 3000-360 =2640

Thus P_e= min [67, 3840/sqrt (2640)] = 0.67

A_s= P_e*A_{sp} = 841.52mm2 **(Using 12 mm diameter reinforcing bars)**

S_{di} =min (a_{si}*1000/A_{ti}, 250] = Min (130,250)

S = 134.33 mm ⟶ Provide Φ 12 c/c 130 mm at bottom, longitudinal direction.

d) Shrinkage and Temperature Reinforcement (AASHTO Article 5.10.8)

Reinforcement for shrinkage and temperature reinforcement shall be provided near surfaces of concrete exposed to daily temperature changes. The steel should be distributed equally on both sides.

$A_{st} \geq 0.75A_g/f_y$

where: A_g is the gross concrete area

A_{st} = 0.75*1000*220/400=412.5 mm2/m

Top layer A_{st} = 1/2*412.5= 206.25 mm2/m

Spacing =min (a_{si}*1000/A_{st}, 250, 3t_s) = min (540, 450 ,660)

⟶ **Provide Φ12 c/c 450 mm at top, longitudinal direction.**

10. Investigation of Service Limit State

Actions to be considered at the service limit state shall be cracking, deformations, and concrete stresses, as specified in Articles 5.7.3.4, 5.7.3.6, and 5.9.4 respectively.

i) Durability

For durability, adequate cover shall be used (for bottom of cast in place slab, the cover is 25mm and 60mm cover at the top). Adequate concrete cover is provided here, thus there is no problem of durability.

a) Check positive moment reinforcement

The load factors used above in all dead and live loads are taken as unity.

Thus, Mp= 54.388 kN-m/m

Reinforcement :

$$A_s = \frac{M_p}{f_s j d_p}$$ Assume; j =0.875 and fs=0.6f_y

$$A_S = \frac{54.388E+06}{(0.6*400)*0.875*187} = 1384.98\text{mm2/m}$$ **Provide Φ 16 c/c 140 mm**

b) Check negaitive moment reinforcement

Mn = -35.404 kN-m/m

Reinforcement :

$$A_s = \frac{M_n}{f_s j d_n}$$ Assume; j =0.875 and fs=0.6f_y

$$A_S = \frac{35.404E+06}{(0.6*400)*0.875*152} = 1109.15\text{mm2/m}$$ **OK**

ii) Control of Cracking.

The cracking stress shall be taken as the modulus of rupture specified in AASHTO, Article 5.4.2.6.

Cracking may occur in the tension zone for RC members due to the low tensile strength of concrete. The cracks may be controlled by distributing steel reinforcements over the maximum tension zone in order limit the maximum allowable crack widths at the surface of the concrete for given types of environment.

The tensile stress in the mild steel reinforcement (f_s) at the service limit state doesn't exceed f_{sa} .

$$f_s \le f_{sa}$$

$$f_{sa} = \frac{Z}{(dc * A)^{\frac{1}{3}}} \le 0.6 f_y$$

$$Z = \begin{cases} 30,000 & \text{moderate exposure conditions} \\ 23,000 & \text{severe exposure conditions} \\ 17,500 & \text{buried structures} \end{cases}$$

Z=23000N/mm

where:

d_c= concrete cover + (diam. of bars/2)

clear cover to compute $d_c \le 50$mm

$A_c = 2 d_c S$

Z= Crack width parameter

A_c= area of concrete having the same centroid as the principal tensile reinforcement are bounded by the surfaces of the cross section and a line parallel the neutral axis divided by the number of bars (mm2), clear cover here also≤50mm.

S = spacing of bars.

f_r = modulus of rupture

$f_r = 0.63\sqrt{(f_c')}$ = 3.33 MPa → $0.8 f_r$ = 2.66 MPa

f_{cten}= tensile strength of the concrete

If $f_{cten} > 0.8 f_r$, the section has cracked (AASHTO, Article 5.7.3.4 and 5.4.2.6)

$$F_{cten} = \frac{6 M_{us}}{b D^2}$$

If $f_s > f_{sa}$, then the area of reinforcing bars has to be increased by reducing the spacing of bars.

a) Ppositive moment reinforcement

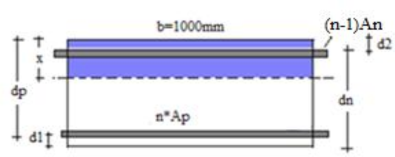
$$E_c = 0.043 \gamma_c \sqrt{f_c'}$$ Ec=26.752 GPa (Modulus Elasticity of concrete)

n = 7

$$\frac{1}{2} b x^2 + (n-1) A_n (x - d_2) = n A_p (d_p - x)$$

$$I_{cr} = \frac{b x^3}{3} + (n-1) A_n (x - d_2)^2 + n A_p (d_p - x)^2$$

$$fs = \frac{n M_{pu} (d_p - x)}{I_{cr}}$$



d1= 33 mm

d2= 68 mm

dp= 187 mm

dn= 152 mm

where:

x is the neutral axis depth from top fiber

M_{pu}= unfactored max positive moment

I_{cr} = moment of inertia of the composite transformed section

The equivalent concrete area, $n * A_p$ = 10048.010 mm^2

Ap =1435.43mm2, S=140mm dp=187mm, n=Es/Ec =7, d2=68 mm, An =1116.44mm2, Mpu=54.388 kN-m/m

Upon substitution, the corresponding values become:

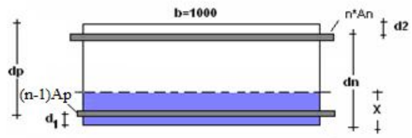
x=53.61 mm, Icr= 231.529E+06 mm4, fs=219.35MPa, fsa=341.86MPa

fs= 219.35 MPa < fsa (=240 MPa)

→ **No problem of cracking**

Provide Φ16 c/c 140 mm

b) Negative moment reinforcement



d1= 33 mm
d2= 68 mm
dp= 187 mm
dn= 152 mm

The equivalent concrete area, $n*A_n =$ **7764.050** **mm^2**

An =1116.44mm2, S=200mm dn=152mm, n=Es/Ec =, d1=33 mm, Ap =1435.43mm2, Mpn= -35.404 kN-m/m

$$\frac{1}{2}bx^2 + (n-1)A_p(x-d_1) = nA_n(d_n - x)$$ x= 40.27 mm

$$I_{cr} = \frac{bx^3}{3} + (n-1)A_p(x-d_1)^2 + nA_n(d_n-x)^2$$ = 119.784E+06 mm4

$$f_s = \frac{nM_{pn}(d_p - x)}{I_{cr}}$$ = 231.18 MPa

where:- M_{Pn}= unfactored negative moment

For top steel, dc=68 mm, A= 2*68*200 =27200mm2

$f_{sa}=23000/(d_c*A)^{1/3}$ **= 187.46 MPa**

→ **fs > fsa (There is a problem of Cracking)**

Increase the amount of reinforcing bars provided (reduce the c/c spacing b/n bars)

Try S = 140 mm As =1435.43 mm2

An =1435.43mm2, S=140mm dn=152mm, n=Es/Ec =7, d1=33 mm, Ap =1435.43mm2, Mpn= -35.404 kN-m/m

Upon substitution, the corresponding values become:

x=44.36 mm, Icr= 146.629E+06 mm4, fs=181.93MPa, and fsa=211.12MPa

fs= 181.93 MPa < fsa (=211.12 MPa)

→ **No problem of cracking**

Thus, Provide Φ16 c/c 140 mm

Investigation of Fatigue Limit State

Fatigue need not be investigated for concrete decks in multi-girder applications. [AASHTO Art 9.5.3]

T-Girder Bridge Design (17.5 m, c/c spacing of bridge)

1. Typical Section

1.1. Deck Thickness

Minimum recommended thickness of the slab, to control deflection is $t_{s,min}$=185 mm.(ERA BDM 2013, Article 5.4.1.1)
Deck slab thickness not less than the clear span between fillets, haunches, or webs divided by 20, unless transverse ribs at a spacing equal to the clear span are used [AASHTO Art. 5.14.1.5.1a]

$$D = \frac{(S+3000)}{30} = 200mm > 185mm \quad (Gs=3m)$$

Minimum recommended depth for continuous slabs, AASHTO (Table 2.5.2.6.3-1)

where: D is the thickness of the slab and S is c/c spacing of girders

Use ts= 220 mm

1.2. Web Thickness

Minimum thickness of the web, b_{min}=200 mm without prestressing duct. [AASHTO Art. 5.14.1.3.1c]
Assume 4 bars in one row are used:

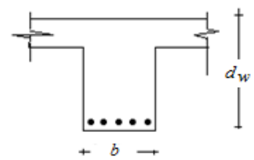
b= 4Φ+3(1.5Φ)+2*side cover =322mm [AASHTO Art. 5.10.3.1.1]

Use b= 360 mm

1.3. Girder Depth (AASHTO Table 2.5.2.6.3-1)

Structural depth, Dw = 0.07L for simple span T- beams ($L=C_s + W_{rs}$)
dw=0.07*17500= 1225

Use dw= 1230 mm



b= 360 mm
dw = 1230 mm

1.4. Girder Spacing and overhang

Numbers of girders, N_g =INT(R_w/G_s)+1

N_g = 3 (Three girders with c/c spacing of 3 m are used.)

$C_x=0.5(R_w-(N_g-1)G_s)$ = 0.66 m

$B_c+C_x+C_w$= 2.01 m (Overhang)

where:

B_c= Bottom width of the barrier (=0.35 m)
C_x= distance from the edge of the curb to the centerline of the exterior girder (=0.66 m)
C_w= Curb width (=1 m)

Number of lanes loaded = Int (7.32/3.6)=2

2. Deck Design

ts = 220 mm (deck thickness)

2.1. Weight of components

1) Slab (220mm thick)	=0.22*2400*9.81/1000	= 5.18 kN/m2
2) Wearing surface (100mm thick)	= 0.1*2250*9.81/1000	=2.21 kN/m2
3) Curb (250mm above slab)	=0.25*2400*9.81/1000	=5.89 kN/m2
4) Barriers		= 5 kN/m

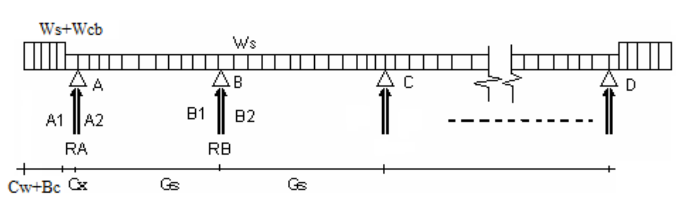
2.1. Dead load moments

An approximate method of analysis in which the deck is subdivided into strips perpendicular to the supporting components shall be considered acceptable for decks. (AASHTO, Article 4.6.2.1.1)

The strips shall be treated as continuous beams with span length equal to the c/c distance between girders. The girders are assumed rigid. For case in applying load factors, the bending moments will be determined for slab dead load, wearing surface and vehicle loads separately.

2.1.1 Slab dead load

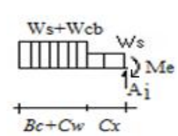
A one-meter strip width is taken for the analysis.



wt. of the curb, W_{cb}= 5.89
wt. of the slab, W_s= 5.18

$$M_e = \frac{W_s C_x^2}{2} + (W_s + W_{cb})(C_w + B_c)\left(\frac{C_w + B_c}{2} + C_x\right)$$

= 5.18*0.66^2/2+11.07*1.35*(1.35/2+0.66) =21.08 kN-m/m



Moment at supports

Using Influence segment coefficent method, moment at supports become, $M_b = \alpha_1 w l^2 + M_e \alpha_2$

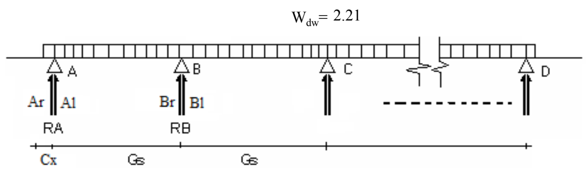
M_b=M_c = 5.18*(-0.125)*3^2+21.08*(-0.5)= -16.37 kN-m/m

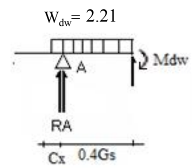
A_1= 18.363kN/m	B_1= 6.2kN/m	R_A= 27.703kN/m
A_2= 9.34kN/m	B_2= 7.77kN/m	R_B= 13.97kN/m

where:

- α_1 and α_2 are influence segment coefficients for moment (obtained from Tables)
- M_e = End moment at support A (exterior girder)
- M_b and M_c are support moments at supports B and C respectively
- B_1, B_2, A_1 and A_2 are shear forces at the left and right of supports B and A respectively.
- R_B and R_A are reactions at supports B and A respectively

2.1.2 Wearing Surface





M_{ew}=W_{dw}*C_x^2/2 =0.48 kN-m/m

Moment at supports

M_{bw}=M_{cw} = 2.21*(-0.125)*3^2+0.48*(-0.5) =-2.726 kN-m/m

A_r= 1.459kN/m	B_r= 4.064kN/m	R_A= 4.025kN/m
A_l= 2.566kN/m	B_l= 3.315kN/m	R_B= 7.379kN/m

M_{dw}= 0.5W_{dw}(C_x+0.4G_s)^2-R_A(0.4G_s), M_{dw} =1.007 kN-m/m

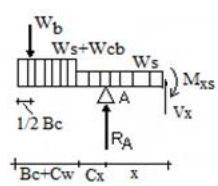
where

M_{dw}= Span moment due to wt. of the wearing surface (at 0.4*G_s from the ext. girder)
M_{ew} = end moment at support A due to wearing surface (exterior girder)
M_{bw} and M_{cw} are support moments at supports B and C due to wearing surface respectively
B_l, B_r are shear forces at the left and right of supports B respectively
R_A and R_B are reactions at supports A and B respectively

2.1.3 Span Moment due to slab dead load and barrier wteight (at 0.4Gs from the ext. girder)

R_A = 27.703+5 =32.703 kN/m

$$M_{xs} = W_s \frac{(C_x + x)}{2} + W_t\left(C_w + B_c\right)\left(\frac{C_w + B_c}{2} + C_x + x\right) + W_b\left(0.5B_c + C_w + x\right) - xR_A$$



M_{xs}= 22.776kN-m/m

where: $W_t = W_s + W_{cb}$ and $x = 0.4G_s$

x = location of maximum span moment at 0.4Gs

M_{xs}=Max span moment due to wt. of the slab, curbs and end barrier/posts (at 0.4Gs from the ext. girder)

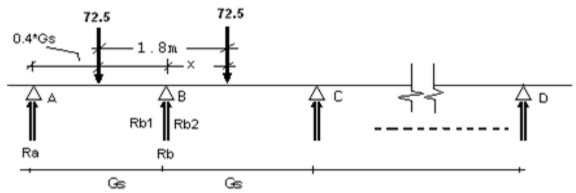
3. Vehicular Live Load

When decks are designed using the approximate strip method (Art. 4.6.2.1) and the strips are transverse, they shall be designed for the 145kN axle load (Art. 3.6.1.3.3). The design truck load shall be positioned transversely to produce maximum force effects.

3.1. Maximum Positive Live Load Moment

For repeating equal spans, the maximum positive bending moment occurs near the 0.4 points of the first interior span. The equivalent width of the strip over which the live load is applied is:

E=660+0.55G_s = 2310 mm (AASHTO, Table 4.6.2.1.3.1)



$0.4G_s$= 1.2 m

Location of wheels for maximum positive live load moment

Position of the second wheel = $0.4G_s$+1.8 = 3 m < 3 m (within the same span)

Joint	A	B		C		D
Member	AB	BA	BC	CB	CD	DC
DF	1.000	0.429	0.571	0.500	0.500	0.571
FEM	31.320	-20.880	0.000	0.000	0	0
CO	-31.320	-15.660				
Bal		15.660	20.880			
CO				10.440		
Bal				-5.220	-5.220	
CO			-2.610			
Bal		1.119	1.491			
CO				0.746		
Bal				-0.373	-0.373	
Final Moment	**0.000**	**-19.761**	**19.761**	**5.593**	**-5.593**	**0.000**

Using moment distribution method, the end moments become:

M_{BA}= $-M_{BC}$ -19.761 kN-m

Reaction due to the above loading

R_a= 36.913 kN　　　　R_b= 112.81 kN

R_{b1}= 108.087 kN　　　　M_{maxp} = 44.296 kN-m

R_{b2}= 4.723 kN

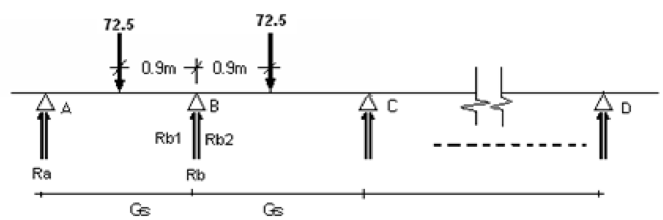
M_p= 1.2*44.296 kN-m/2.31m =23.011 kN-m/m

R_a= 1.2*36.913/2.31m =19.176 kN/m

R_b= 1.2*112.81 kN/2.31m =58.603 kN/m

3.2. Maximum Interior Negative live load moment

The critical placement of live load for maximum negative moment is at the first interior support.
The width of equivalent transverse strip is E=1220+0.25G_s (AASHTO, Table 4.6.2.1.3.1)



The equivalent width, E= 1800 mm

Joint	A	B		C		D
Member	AB	BA	BC	CB	CD	DC
DF	1.000	0.429	0.571	0.500	0.500	0.571
FEM	15.493	-24.445	24.445	-15.493	0	0
CO	-15.493	-7.746				
Bal		3.320	4.427			
CO				2.213		
Bal				6.640	6.640	
CO			3.320			
Bal		-1.423	-1.897			
CO				-0.949		
Bal				0.474	0.474	
Final Moment	**0.000**	**-30.294**	**30.294**	**-7.114**	**7.114**	**0.000**

Using moment distribution method, the end moments become:

M_{BA}= -M_{BC} -30.294 kN-m

R_a= 15.067 kN R_b= 111.799 kN

R_{b1}= 57.433 kN M_{maxN} = -30.294 kN-m

R_{b2}= 54.366 kN

M_{maxN} = 1.2*-30.294 kN-m/1.8m =-20.196 kN-m/m

R_b= 1.2*111.799 kN/1.8m =74.533 kN/m

4. Select Resistance Factors

Strength Limit States (RC)

Flexure & Torsion

→ Resistance Factor = 1

5. Select Load Modifiers

i) Strength Limit State

i) Ductility	0.95	[Art. 1.3.3]
ii) Continuous	0.95	[Art. 1.3.4]
iii) Importancy	1.05	[Art. 1.3.5]
→ Load Modifier =	0.95	

6. Select Applicable Load Combinations

i) Strength Limit State U=0.95(1.25DC+1.50DW+1.75(LL+IM)+1.00(FR+TG)

7. Investigate Strength Limit State

Temperature gradient effect reduces gravity load effects.Because temperature gradient may not be there, assume TG=0

Thus, U=0.95(1.25DC+1.50DW+1.75(LL+IM))

8. Design Moment Computaions

M_{maxp}= 0.95*[1.25(Mxs)+1.5(Mdw)+1.75*1.33(Mp)]= 79.362 kN-m/m

M_{sup-B}= 0.95*[1.25(Mb+Mbb)+1.5(Mbw)+1.75*1.33(Mn)]= -73.442 kN-m/m

For reinforcement computation, negative moment may be taken at the face of the support. Thus, calculate moments at the face of support B (the width of the beam, b=360mm)

→ MDc=-15.055kN-m/m, MDw=-2.165kN-m/m and MLL=-13.672kN-m/m

M_{neg}= 0.95*[1.25*(-15.055)+1.5*(-2.165)+1.75*1.33*(-13.672)]-51.193 kN-m/m

9. Reinforcement

a) Positive Moment Reinforcement

dp=187 mm (effective depth,dp=220-16/2-25)

Mmaxp= 79.362 kN-m/m

$$\rho = \left(1 - \sqrt{1 - \frac{2M_u}{0.9bd^2\varphi f'_c}}\right)\frac{\varphi f'_c}{f_y} \qquad \rho_{min} = \frac{0.03 f_c'}{f_y} \qquad \varphi=0.85$$

Checking the adequacy of the section

The section is checked for the maximum design moment whether the initial depth under consideration is sufficed or not.

$$d_{ic} = \sqrt{\frac{M_{max}}{0.254bf_c'}}$$ =107.47mm The section is sufficed!

ρ =0.0067 (Using 16 mm diameter reinforcing bars)

ρ > ρ min Ok!

As=0.0067*1000*187 = 1248.95mm2

S =161mm → Use Φ 16 c/c 160 mm (As, provided= 1256 mm2)

b) Negative Moment Reinforcement

dn=152 mm (effective depth,dn=220-16/2-60)

M_{neg}= -51.193 kN-m/m

ρ =0.0065 (Using 16 mm diameter reinforcing bars)

ρ > ρ min Ok!

As=0.0065*1000*152 = 989.67mm2

S =203mm ⟶ **Use Φ 16 c/c 200 mm** **(As, provided= 1004.8 mm2)**

c) Distribution Reinforcement (AASHTO Article 5.14.4.1)

The amount of distribution reinforcement at the bottom of the slab to distribute the loads may be taken as a percentage of the primary reinforcement and a minimum spacing of 250mm.

P_e= min[67,3840/√Se]

P_e = Percentage of distribution reinforcement

S_e = Clear spacing of girders = G_s-b

Se = 3000-360 =2640

Thus P_e= min [67, 3840/sqrt (2640)] = 0.67

A_s= P_e*A_{sp} = 841.52mm2 **(Using 12 mm diameter reinforcing bars)**

S_{di} =min (a_{si}*1000/A_{ti}, 250] = Min (130,250)

S = 134.33 mm ⟶ **Provide Φ 12 c/c 130 mm at bottom, longitudinal direction.**

d) Shrinkage and Temperature Reinforcement (AASHTO Article 5.10.8)

Reinforcement for shrinkage and temperature reinforcement shall be provided near surfaces of concrete exposed to daily temperature changes. The steel should be distributed equally on both sides.

$A_{st} \geq 0.75A_g/f_y$

where: A_g is the gross concrete area

A_{st} = 0.75*1000*220/400=412.5 mm2/m

Top layer A_{st} = 1/2*412.5= 206.25 mm2/m

Spacing =min (a_{si}*1000/A_{st}, 250, 3t_s) = min (540, 450 ,660)

⟶ **Provide Φ12 c/c 450 mm at top, longitudinal direction.**

10. Investigation of Service Limit State

Actions to be considered at the service limit state shall be cracking, deformations, and concrete stresses, as specified in Articles 5.7.3.4, 5.7.3.6, and 5.9.4 respectively.

i) Durability

For durability, adequate cover shall be used (for bottom of cast in place slab, the cover is 25mm and 60mm cover at the top). Adequate concrete cover is provided here, thus there is no problem of durability.

a) Check positive moment reinforcement

The load factors used above in all dead and live loads are taken as unity.

Thus, Mp= 54.388 kN-m/m

Reinforcement :

$$A_s = \frac{M_p}{f_s j d_p}$$ Assume; j =0.875 and fs=0.6f_y

$$A_S = \frac{54.388E+06}{(0.6*400)*0.875*187} = 1384.98\text{mm2/m}$$ **Provide Φ 16 c/c 140 mm**

b) Check negaitive moment reinforcement

Mn = -35.404 kN-m/m

Reinforcement :

$$A_s = \frac{M_n}{f_s j d_n}$$ Assume; j =0.875 and fs=0.6f_y

$$A_S = \frac{35.404E+06}{(0.6*400)*0.875*152} = 1109.15\text{mm2/m}$$ **OK**

ii) Control of Cracking.

The cracking stress shall be taken as the modulus of rupture specified in AASHTO, Article 5.4.2.6.

Cracking may occur in the tension zone for RC members due to the low tensile strength of concrete. The cracks may be controlled by distributing steel reinforcements over the maximum tension zone in order limit the maximum allowable crack widths at the surface of the concrete for given types of environment.

The tensile stress in the mild steel reinforcement (f_s) at the service limit state doesn't exceed f_{sa} .

$$f_s \le f_{sa}$$

$$f_{sa} = \frac{Z}{(dc * A)^{\frac{1}{3}}} \le 0.6 f_y$$

$$Z = \begin{cases} 30{,}000 & \text{moderate exposure conditions} \\ 23{,}000 & \text{severe exposure conditions} \\ 17{,}500 & \text{buried structures} \end{cases}$$

Z=23000N/mm

where:

d_c= concrete cover + (diam. of bars/2)

clear cover to compute $d_c \le 50$mm

$A_c = 2 d_c S$

Z= Crack width parameter

A_c= area of concrete having the same centroid as the principal tensile reinforcement are bounded by the surfaces of the cross section and a line parallel the neutral axis divided by the number of bars (mm2), clear cover here also≤50mm.

S = spacing of bars.

f_r = modulus of rupture

$f_r = 0.63\sqrt{(f_c')}$ = 3.33 MPa → $0.8 f_r$ = 2.66 MPa

f_{cten}= tensile strength of the concrete

If $f_{cten} > 0.8 f_r$, the section has cracked (AASHTO, Article 5.7.3.4 and 5.4.2.6)

$$F_{cten} = \frac{6 M_{us}}{b D^2}$$

If $f_s > f_{sa}$, then the area of reinforcing bars has to be increased by reducing the spacing of bars.

a) Ppositive moment reinforcement

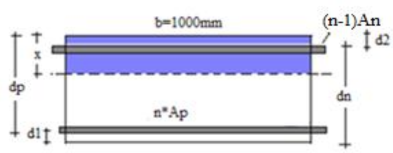
$$E_c = 0.043 \gamma_c \sqrt{f_c'}$$ Ec=26.752 GPa (Modulus Elasticity of concrete)

n = 7

$$\frac{1}{2} b x^2 + (n-1) A_n (x - d_2) = n A_p (d_p - x)$$

$$I_{cr} = \frac{b x^3}{3} + (n-1) A_n (x - d_2)^2 + n A_p (d_p - x)^2$$

$$fs = \frac{n M_{pu} (d_p - x)}{I_{cr}}$$



d1= 33 mm
d2= 68 mm
dp= 187 mm
dn= 152 mm

where:

x is the neutral axis depth from top fiber

M_{pu}= unfactored max positive moment

I_{cr} = moment of inertia of the composite transformed section

The equivalent concrete area, $n * A_p$ = 10048.010 mm^2

Ap =1435.43mm2, S=140mm dp=187mm, n=Es/Ec =7, d2=68 mm, An =1116.44mm2, Mpu=54.388 kN-m/m

Upon substitution, the corresponding values become:

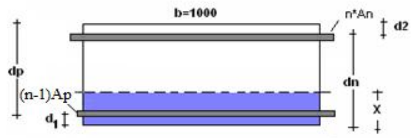
x=53.61 mm, Icr= 231.529E+06 mm4, fs=219.35MPa, fsa=341.86MPa

fs= 219.35 MPa < fsa (=240 MPa)

→ **No problem of cracking**

Provide Φ16 c/c 140 mm

b) Negative moment reinforcement



d1= 33 mm
d2= 68 mm
dp= 187 mm
dn= 152 mm

The equivalent concrete area, $n*A_n =$ **7764.050** mm^2

An =1116.44mm2, S=200mm dn=152mm, n=Es/Ec =, d1=33 mm, Ap =1435.43mm2, Mpn= -35.404 kN-m/m

$$\frac{1}{2}bx^2+(n-1)A_p(x-d_1)=nA_n(d_n-x)$$ x= 40.27 mm

$$I_{cr}=\frac{bx^3}{3}+(n-1)A_p(x-d_1)^2+nA_n(d_n-x)^2$$ = 119.784E+06 mm4

$$f_s=\frac{nM_{pn}(d_p-x)}{I_{cr}}$$ = 231.18 MPa

where:- M_{Pn}= unfactored negative moment

For top steel, dc=68 mm, A= 2*68*200 =27200mm2

$f_{sa}=23000/(d_c*A)^{1/3}$ **= 187.46 MPa**

→ **fs > fsa (There is a problem of Cracking)**

Increase the amount of reinforcing bars provided (reduce the c/c spacing b/n bars)

Try S = 140 mm As =1435.43 mm2

An =1435.43mm2, S=140mm dn=152mm, n=Es/Ec =7, d1=33 mm, Ap =1435.43mm2, Mpn= -35.404 kN-m/m

Upon substitution, the corresponding values become:

x=44.36 mm, Icr= 146.629E+06 mm4, fs=181.93MPa, and fsa=211.12MPa

fs= 181.93 MPa < fsa (=211.12 MPa)

→ **No problem of cracking**

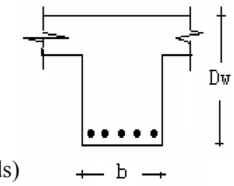
Thus, Provide Φ16 c/c 140 mm

Investigation of Fatigue Limit State

Fatigue need not be investigated for concrete decks in multi-girder applications. [AASHTO Art 9.5.3]

Design of Longitudinal Girders

1. Dead load effect due to web and diaphragm wt.



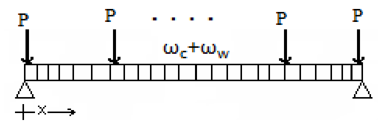
Structural depth, dw = 1230 mm
Web width, b= 360 mm
Own weight of web= b*(dw-ts)Yc=8.56kN/m. , Yc=2400 kg/m3
Assume the size of the diaphragm: 0.7m x 0.25m
4 diaphragms with c/c spacing of 5.8m is used. (two diaphragms are provided at the ends)
Diaphragm load on exterior girder =5.439 kN
Diaphragm load on interior girder = 10.878 kN

2. Slab reactions (dead loads & live loads) transferred to girders

Exterior Girder

Reactions, *DC:* $\omega_c = R_A$ = 27.703+8.56+5=41.263 kN/m (including web & barrier wt.)
DW: $\omega_w = R_A$ = 4.025 kN/m P= 5.439 kN

Moments, *DC:* M(x)= 361.051x-20.632x^2
DW: M(x)= 35.219x-2.013x^2



Interior Girder

Reactions, *DC:* $\omega_c = R_B$ = 13.97+8.56=22.53 kN/m (including web wt.)
DW: $\omega_w = R_B$ = 7.379 kN/m P= 10.878 kN

Moments, *DC:* M(x)= 197.138x-11.265x^2
DW: M(x)= 64.566x-3.69x^2

3. Distribution factors for Moments and Shears

The distribution factors for moment and shear for both external and internal girders are obtained from the empirical formula given in Table Art 4.6.2.2-1 of AASHTO 2007.

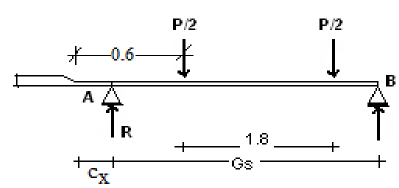
3.1.1 Distribution factors for Moment [Table 4.6.2.2.2b-1 and d-1]
a) Interior beams with concrete decks: [Table Art 4.6.2.2b and 2b-1]

Girder distribution factor with multiple presence factor included m_g is,
One Lane loaded:
$mg^{SI}_m = 0.06+(G_s/4.3)^{0.4}*(G_s/L)^{0.3}*(k_g/Lt_s^3)^{0.1}$
for preliminary design, $(k_g/Lt_s^3)=1$ and L is c/c of bridge length.

Two or more lanes loaded:
$mg^{ml}_m = 0.075+(G_s/2.9)^{0.6}*(G_s/L)^{0.2}*(k_g/Lt_s^3)^{0.1}$
Skew correction factor= $1-C_1(\tan\theta)^{1.5}$; $C_1=(G_s/L)^{0.5}*(k_g/Lt_s^3)^{0.25}$

mg^{sl}_m= 0.57 and mg^{ml}_m= 0.792
Skew correction factor =1
Thus, mg^{sl}_m= 0.570 and mg^{ml}_m= 0.792 **Use 0.792**

b) Exterior beams: [Table Art 4.6.2.2d and 2d-1]
One lane loaded: Lever Rule
$\sum M_B=0$, R=0.72P → mg^{se}_m= 1.2*0.72= 0.864



Two or more lanes loaded
$mg^{me}_m = e*mg^{ml}_m$
where, e=0.77+Cx/2.8 Cx= 0.66 m
e= 1.006 >1, thus use e=1.006 → mg^{me}_m= **0.797**

3.1.2 Distribution factors for Shear [Table 4.6.2.2.3a-1 and b-1]
a) Interior beams
mg^{sl}_v= 0.36+Gs/7.6= 0.755
mg^{ml}_v= 0.2+Gs/3.6-(Gs/L)^2= 1.004
Skew correction factor= $1+0.2(K_g/Lt_s^3)^{0.3}\tan\theta$
Skew correction factor =1 → mg^{sl}_v= 0.755 and mg^{ml}_v= 1.004 **Use 1.004**

b) Exterior beams

One lane loaded: Lever Rule

$mg^{se}_{v} = 1.2*0.72 = 0.864$

Two or more lanes loaded

$mg^{me}_{v} = e*mg^{sI}_{v}$ where, $e = 0.6 + Cx/3$ 0.82

$e = 0.82 \rightarrow mg^{me}_{v} = 0.823$ **Use 0.864**

0.864

Influence Lines for Bending Moment and Shear Force

4. Distributed Live Load Force Effects

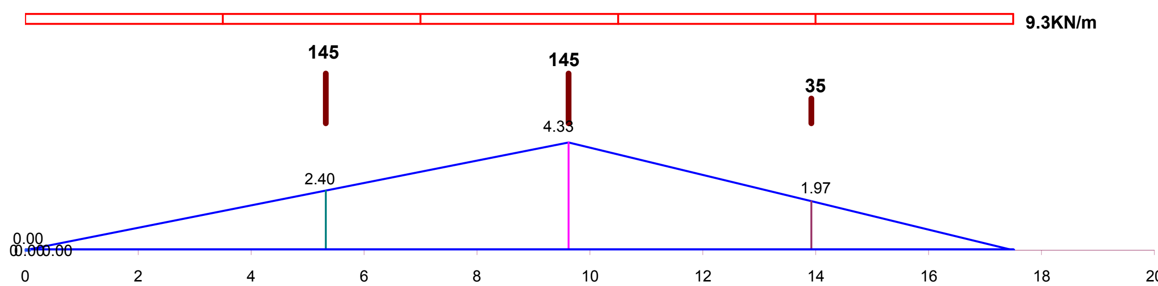
Single design vehicular load is considered.

i) Influence Line for Live Load Moment (Truck Load)

x is the position of the rear wheel.

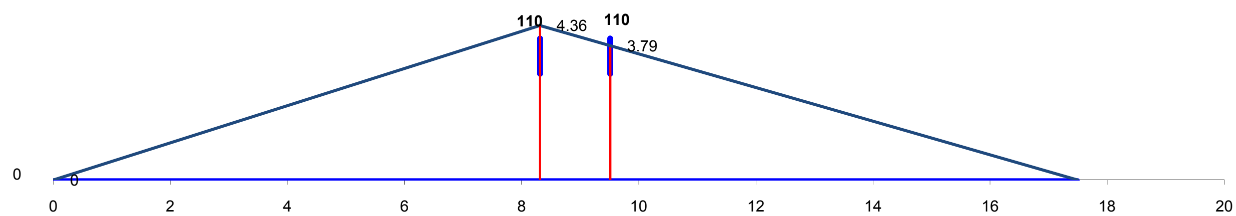
At x = 9.625 m, Mtr(x)=1044.306 kN-m /Max. effect/

NB: *x* is the position of the middle wheel measured from the left support.



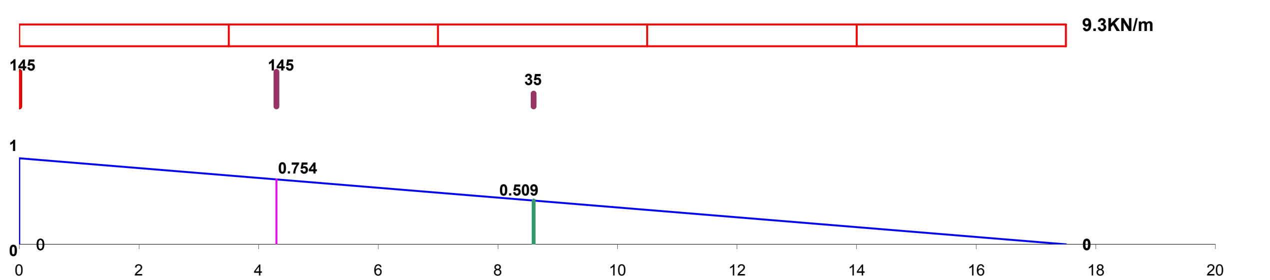
ii) Influence Line for live Load Moment (Tandem Load)

At x = 8.3125 m, Mtm(x) =897.394 kN-m /Max. effect/



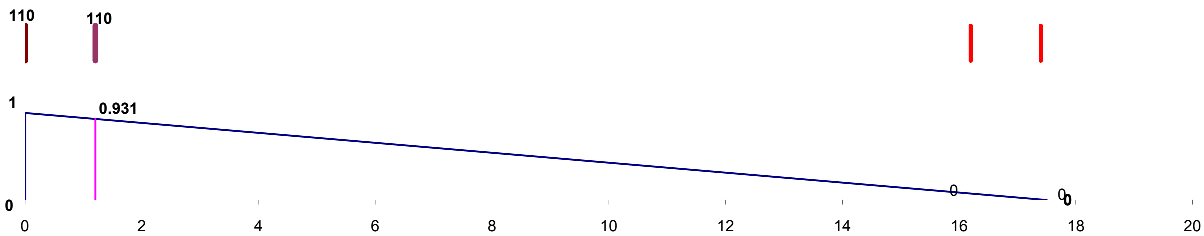
iii) Influence Line for Shear Force (Truck Load)

At x =0 m , the shear force due to truck load, Vtr(x) =272.171 kN



iv) Influence Line for Shear Force (Tandem Load)

At x =0 m , the shear force due to tandem load, Vtm(x) =212.457 kN



5. Select Resistance Factors

Strength Limit States (RC)

Flexure & Torsion

→ Resistance Factor = 1

6. Select Load Modifiers

i) Strength Limit State

i) Ductility	0.95	[Art. 1.3.3]
ii) Redundancy	1.05	[Art. 1.3.4]
iii) Importancy	1.05	[Art. 1.3.5]
→ Load Modifier =	1.05	

7. Select Applicable Load Combinations

i) Strength Limit State U=1.05*(1.25DC+1.50DW+1.75(LL+IM)+1.00FR+TG)

8. Investigate Strength Limit State

It is necessary to calculate moment and shear force at different locations to determine the bar cut off points and to calculate spacing of stirrups.

9. Design Shear and Moment Computaions

Distribution factors for moments and shear forces, mg is applied only on live load forces.

a) Interior Girder

i) Shear Force

X	Vtr	Vtm	Vln	Vdc	Vdw	VD	VDU
0	272.171	212.457	81.375	208.018	64.566	1192.653	627.545
1.75	239.671	190.457	65.914	168.591	51.651	1012.296	527.049
3.5	207.171	168.457	62.303	129.163	38.736	853.801	438.451
5.25	174.671	146.457	58.793	89.736	25.821	695.494	349.955
7	142.171	124.457	55.386	39.428	12.906	523.094	250.682
8.75	109.671	102.457	52.080	0.000	-0.009	365.162	162.390
9.625	94.871	91.457	48.876	-19.713	-6.467	286.891	118.142
10.5	80.371	80.457	45.773	-39.427	-12.924	209.756	74.385
11.375	65.871	69.457	42.773	-59.141	-19.382	141.184	34.156

VDmax=1.05*[mgv*1.75[1.33Max(Vtr,Vtm)+Vln]+(1.25Vdc+1.5Vdw)] **= 1192.653 kN**

ii) Moment

X	M_{tr}	M_{tm}	M_{ln}	M_{dc}	M_{dw}	MD	MDU
0.00	0.000	0.000	0.000	0.000	0.000	0.000	0.000
1.75	268.450	333.300	128.166	329.532	101.690	1424.311	796.703
3.50	473.900	589.600	227.850	590.067	180.779	2531.979	1418.266
5.25	712.775	768.900	299.053	781.603	237.266	3323.003	1864.688
7.00	930.700	871.200	341.775	885.101	271.152	3887.560	2164.053
8.75	1034.875	896.500	356.016	919.601	282.437	4172.975	2303.623
9.63	1044.306	880.275	352.455	910.977	279.604	4170.267	2296.816
10.50	1025.300	844.800	341.775	885.103	271.121	4070.616	2238.947
11.38	749.288	790.075	323.974	841.979	256.987	3510.561	1981.293

MDmax=1.05*[mgm*1.75[1.33Max(Mtr,Mtm)+Mln]+(1.25Mdc+1.5Mdw)] **= 4172.975 kN-m**

Vdc and Mdc include shear and moment due to weight of the deck slab, overhang, end barriers, and diaphragms.

b) Exterior Girder

i) Shear Force

X	Vtr	Vtm	Vln	Vdc	Vdw	VD	VDU
0	272.171	212.457	81.375	366.491	35.219	1240.372	707.174
1.75	239.671	190.457	65.914	294.279	28.174	1041.327	586.478
3.5	207.171	168.457	62.303	222.067	21.128	861.095	476.021
5.25	174.671	146.457	58.793	149.855	14.083	681.025	365.651
7	142.171	124.457	55.386	72.203	7.037	493.976	249.929
8.75	109.671	102.457	52.080	-0.009	-0.008	314.229	139.736
9.625	94.871	91.457	48.876	-36.115	-3.531	224.954	84.551
10.5	80.371	80.457	45.773	-72.221	-7.054	136.656	29.788
11.375	65.871	69.457	42.773	-108.327	-10.577	55.728	-21.937

VDmax=1.05*[mgv*1.75[1.33*Max(Vtr,Vtm)+Vln]+(1.25*Vdc+1.5*Vdw)] **= 1240.372 kN**

ii) Moment

X	M_{tr}	M_{tm}	M_{ln}	M_{dc}	M_{dw}	MD	MDU
0.00	0.000	0.000	0.000	0.000	0.000	0.000	0.000
1.75	268.450	333.300	128.166	578.174	55.468	1683.104	1001.430
3.50	473.900	589.600	227.850	1029.977	98.607	2989.238	1780.091
5.25	712.775	768.900	299.053	1355.408	129.416	3918.403	2335.983
7.00	930.700	871.200	341.775	1516.389	147.896	4536.510	2678.448
8.75	1034.875	896.500	356.016	1579.559	154.046	4852.870	2842.145
9.63	1044.306	880.275	352.455	1563.755	152.497	4842.844	2829.471
10.50	1025.300	844.800	341.775	1516.358	147.866	4720.681	2753.783
11.38	749.288	790.075	323.974	1437.368	140.153	4120.625	2465.418

MDmax=1.05*[mgm*1.75[1.33Max(Mtr,VMtm)+Mln]+(1.25Mdc+1.5Mdw)] **= 4852.87 kN-m**

Vdc and Mdc include shear and moment due to weight of the deck slab, overhang, end barriers, and diaphragms.

Checking the adequacy of the section

The section is checked for the maximum design moment whether the initial depth under consideration is sufficed or not.

Interior Girder, MD = 4172.975 kN-m

$$d_{ic}=\sqrt{\frac{M_{max}}{0.254bf_c'}} \quad =442.249 \text{ mm.}$$

Effective flange width, b_i

$b_i \leq$ { 1/4*Effective span =4.375 m; $12*t_{min}+b_w$ = 3 m; average spacing of adjacent beams= 3 m } → b_i= 3.000

Check depth: d = 442.249mm (The section is sufficed!)

→ 0.42*d = 185.745 mm < 220 mm (Rectangular)

Effective depth = 1072 mm (bars in three layers)

Exterior Girder, MD = 4852.87 kN-m

$$d_{ic}=\sqrt{\frac{M_{max}}{0.254bf_c'}} \quad = 466.908 \text{ mm.}$$

Effective flange width, b_e

b_e-0.5*b_i < { 1/8*Effective span =2.188 m; $6*t_{min}+b_w$ = 1.68 m; width of overhang= 1.63 m } → b_e= 3.130

Check depth: d = 466.908mm (The section is sufficed!)

→ 0.42*d = 196.101 mm < 220 mm (Rectangular)

Effective depth = 1072 mm

a_s= 803.84 Development length, lb =1215.29 mm

Reinforcement

a) Flexure

x (m)	Interior Girder				Exterior Girder			
	MD	A_S	No. of bars	Length	MD	A_S	No. of bars	Length
1.75	1424.31	6753.600	9 Φ32	19520	1683.104	7046.256	9 Φ32	19520
5.25	3323.00	8813.491	2 Φ32	9435	3918.403	10425.511	4 Φ32	9435
8.75	4172.98	11137.025	3 Φ32	2435	4852.870	12997.747	4 Φ32	2435

Crack control

Positive moment reinforcement- Interior Girder

M= 2303.623kN-m/m

Reinforcement :

$$A_s = \frac{M_p}{f_s j d_p}$$ Assume; j =0.875 and fs=0.6*f_y

$$A_S = \frac{2303623000}{(0.6*400)*0.875*1072} = 10232.87\text{mm2}$$ **OK!**

Provide additional 1 Φ 32 length =2435 mm,

Positive moment reinforcement- Exterior Girder

M= 2842.145kN-m/m

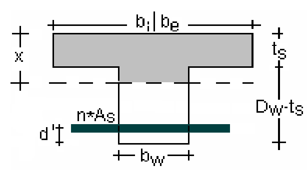
Reinforcement :

$$A_s = \frac{M_p}{f_s j d_p}$$ Assume; j =0.875 and fs=0.6*f_y

$$A_S = \frac{2842145000}{(0.6*400)*0.875*1072} = 12625.02\text{mm2}$$ **OK!**

Provide additional 1 Φ 32 length =2435 mm

Cracking Moment of Inertia



Interior Girder **13**

x_{int} = 211.48 mm

$I_{cr, int}$ = 67.187*E+9 mm4

Exterior Girder

x_{ext} = 222.273 mm

$I_{cr, ext}$ = 77.151*E+9 mm4

Int. girder fs=206.532MPa, (fsa=240Mpa), Provide additional 1 Φ 32 length =2435 mm,

Ext. girder fs=219.119MPa, (fsa=240Mpa), Provide additional 1 Φ 32 length =2435 mm

b) Shear Design

Nominal shear resistance

The section is check for maximum shear and thus shear reinforcements are designed.

$$Vn = \min. \begin{cases} Vc+Vs \\ 0.25*f'_c b_v d_v = 2.883 \text{ MPa} \end{cases}$$

$$V_c = 0.083\beta\sqrt{f'_c}\,b_v d_v$$

$Vs = A_v f_y d/S$

where: Vn is the nominal shear strength

Vc = shear strength provided by the shear reinforcement

bv = effective web width

dv = effective shear depth

S = spacing of shear reinforcement

Spacings

Determine V_U (the design shear force) at a distance dv from face of support.

$$S \le \frac{A_v f_y d_v \cot\theta}{V_s} \le \frac{A_v f_y}{0.083 b_v \sqrt{f'_c}}$$

$$for \quad V_u < 0.1 f'_c b_v d_v, \qquad S < 0.8 d_v < 600mm$$

$$for \quad V_u \ge 0.1 f'_c b_v d_v, \qquad S < 0.4 d_v < 300mm$$

Interior girder

dv =max { d-a/2 = 1143.9 mm; 0.9d= 1047.6 n; 0.72D = 885.6 mm } → dv = 1143.9 mm

$a = A_s f_y / 0.8 f_c' b_{eff}$
a = 40.2
d =1164 mm

Vs=(Vu/φ)-Vc, ϕ=0.9

Vn=2.883 MPa and f'c=28MPa → Vn/f'c = 0.103 **(<0.25) OK**

$$\varepsilon_x = \frac{M_D / d_v + 0.5 V_D \cot\theta}{E_s A_s} \le 0.002$$

From chart

x(m)	Vu	Mu	Assumed θ	εx (E-3)	θ	β	Vs	Spacing of bars
1.14	1074.762	931.011	36.50	1.065	36.26	2.042	824.86	Φ 12 c/c 150 mm
1.75	1012.296	1424.311	37.00	1.325	37.30	1.85	790.18	Φ 12 c/c 160 mm
3.50	853.801	2531.979	38.50	1.901	38.80	1.503	676.83	Φ 12 c/c 180 mm
5.25	695.494	3323.003	38.75	1.888	38.78	1.51	499.67	Φ 12 c/c 250 mm
7.00	523.094	3887.560	38.50	1.656	38.31	1.636	285.33	Φ 12 c/c 300 mm
8.75	365.162	4172.975	38.50	1.723	38.45	1.6	116.36	Φ 12 c/c 300 mm

Exterior girder

dv =max { d-a/2 = 1143.9 mm; 0.9d= 1047.6; 0.72D = 885.6 mm } → dv = 1143.9 mm

$a = A_s * f_y / 0.8 f_c' b_{eff}$
a = 40.2
d =1164 mm

→ Vn/f'c = 0.103 MPa

From chart

x(m)	Vu	Mu	Assumed θ	εx (E-3)	θ	β	Vs	Spacing of bars
1.14	1110.265	1100.173	36.68	1.18	36.72	1.957	879.68	Φ 12 c/c 160 mm
1.75	1041.327	1683.104	37.80	1.481	37.92	1.734	843.42	Φ 12 c/c 160 mm
3.50	861.095	2989.238	39.00	2	39.00	1.45	694.52	Φ 12 c/c 200 mm
5.25	681.025	3918.403	38.66	1.843	38.69	1.535	479.07	Φ 12 c/c 290 mm
7.00	493.976	4536.510	38.12	1.566	38.13	1.684	244.29	Φ 12 c/c 300 mm
8.75	314.229	4852.870	38.24	1.625	38.25	1.653	50.18	Φ 12 c/c 300 mm

x (m)	Interior Girder			Exterior Girder		
	VD	Vs	Spacing	VD	Vs	Spacing
1.14	1074.76	824.859	Φ 12 c/c 150 mm	1110.265	879.681	Φ 12 c/c 160 mm
1.75	1012.30	790.179	Φ 12 c/c 160 mm	1041.327	843.415	Φ 12 c/c 160 mm
3.50	853.80	676.832	Φ 12 c/c 180 mm	861.095	694.523	Φ 12 c/c 200 mm
5.25	695.49	499.669	Φ 12 c/c 250 mm	681.025	479.071	Φ 12 c/c 290 mm
7.00	523.09	285.325	Φ 12 c/c 300 mm	493.976	244.291	Φ 12 c/c 300 mm
8.75	365.16	116.356	Φ 12 c/c 300 mm	314.229	50.178	Φ 12 c/c 300 mm

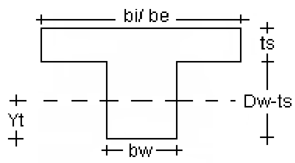
Deflection and camber, AASHTO Article 5.7.3.6.2

Deflection and camber calculations shall consider dead load, live load, erection loads, concrete creep and shrinkage. Immediate (Instantaneous) deflections may be computed taking the moment of inertia as either the effective moment of inertia I_e, or the gross moment of inertia Ig.

Unless a more exact deformation calculation is made, the long-term deflection due to creep and shrinkage may be taken as the immediate deflection multiplied by the following factor (AASHTO, Article 5.7.3.6.2).

4, if the instantaneous deflection is based on I_e.

$3.0 - 1.2\left(\frac{A'_s}{A_s}\right) \geq 1.6$ if the instantaneous deflection is based on I_g.



ts= 220 mm

Dw = 1230 mm

bw= 360 mm

bi = 3000 mm

be = 3130 mm

Dw-ts = 1010 mm

$$M_{cr} = f_r \frac{I_g}{y_r}$$

f_r = 0.63*√ (f_c') = 3.33 MPa

Interior Girder

$Y_{t, int}$= 901.54

$I_{g, int}$ = 1.222E+11 mm^4

$M_{cr, int}$ = 451.53 Mpa

Exterior Girder

$Y_{t, ext}$= 907.48

$I_{g, ext}$ = 1.237E+11 mm^4

$M_{cr, ext}$ = 453.87 Mpa

Total moment due to dead and live load (unfactored)

Interior Girder

MDU_{int}= 2303.623 kN-m

Exterior Girder

MDU_{ext}= 2842.145 kN-m

a) <u>Dead load deflection and camber</u>

The effective moment of inertia is calculated using the following equation:

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \leq I_g$$

Interior Girder

$I_{crt, int}$= 67.187*E+9 mm4 $I_{e, int}$= 67.601*E+9 mm4 (effective moment of inertia)

Exterior Girder

$I_{crt, ext}$= 77.151*E+9 mm4 $I_{e, ext}$= 77.341*E+9 mm4 (effective moment of inertia)

Maximum Dead load moment

Interior Girder, $M_{max\ DL}$ = 1202.038 kN-m

Exterior Girder, $M_{max\ DL}$ = 1733.605 kN-m

The maximum dead load deflection of the girders is obtained by integrating twice the DL moment equation. Thus, the maximum dead load moment at the mid span becomes:

$d_{dl, int}$ = 1/EIe*(32.86*x^3 - 0.94*x^4 -5031.13x) = 15.217 mm

$d_{dl, ext}$ = 1/EIe*(60.18*X^3 - 1.72*X^4 -9214.21X) = 24.355 mm

$d_{dl, int}$= 15.217 mm

$d_{dl, ext}$= 24.355 mm

Camber =d_{dl}+long term deflection, Long term deflection = 3*d_{dl}

Camber =d_{dl} (1+3)= 79.144 mm (average of interior and exterior girder)

b) Live Load Deflection (AASHTO, Article 2.5.2.6.2)

d_{max}=L_1/800 (L_1 is in mm); the permissible limit 21.875 mm

Where: L_1: span length of the bridge in mm

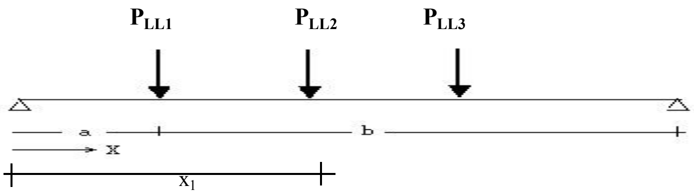
d_{max} : the permissible limit (max. deflection)

In the computation of live load deflection, design truck load alone or design lane load plus 25% of truck load is considered.

i) Deflection due to truck load

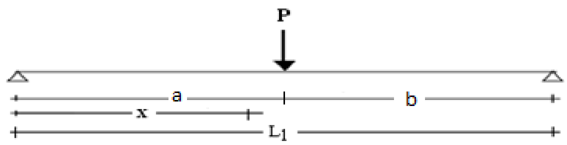
$P_{LL1} = P_{LL2} =$ 145 kN

$P_{LL3} =$ 35 kN



at x1=9.625 m, Mtr =1044.306 kN-m/m (Location of maximum truck load)

The maximum deflection of the bridge due to truck load occurs at a wheel load position where moment is a maximum. In general, the deflection at the point of maximum moment, X_1, due to each truck load at a distance '**a**' from the left support is given by:



$$d_{ki} = P_{LLi} b x \frac{\left(L_l^2 - b^2 - x^2\right)}{6 E_c I_c L_l} \quad for \;\; x \le a \qquad d_{ki} = P_{LLi} a \left(L_1 - x\right) \frac{\left(2L_1 x - a^2 - x^2\right)}{6 E_c I_c L_1} \quad for \;\; x \ge a$$

where:

d_{ki} = deflection due to each truck load.

a = location of the load to be considered

$b = L_1 - a$ $\quad x = L_1 - x_1$ (x is the point where deflection is computed, for the first load, $x = x_1$)

$I_{e,int} =$ 6.760E+10 and $I_{e,ext} =$ 7.734E+10

a) First Load, P_{LL1}

a =5.325 m, b =12.175 m, x =9.625 m

$d_{k1} =$ 0.00585m= 5.85mm

b) Second Load, P_{LL2}

a =9.625 m, b =7.875 m, x =9.625 m

$d_{k1} =$ 0.00877m= 8.77mm

c) Third Load, P_{LL3}

a =13.925 m, b =3.575 m, x =9.625 m

dtr3= 0.00127m= 1.27mm

Using the method of superposition, the total live load deflection due truck load is the sum of each deflections, d_{ki}'s.

$d_{kt} = d_{k1} + d_{k2} + d_{k3}$ = 5.85+ 8.77+ 1.27 = 15.89mm

Thus, Interior girder, $\sum d_{ki}$ = 15.89mm

Similarly for exterior girder,

$d_{k1} =$ 5.11 mm $\quad d_{k2} =$ 7.67 mm $\quad d_{k3} =$ 1.11 mm

→ $d_{kt} = d_{k1} + d_{k2} + d_{k3}$ = 13.89mm

ii) Deflection due to tandem load

The max. deflection of the interior girder due to tandem load occurred when a single concentrated tandem load is acting at the mid span.

$d_{tl} = P_{stl} * L_1^3 / 48 E_c I_c$

where: d_{tl} = deflection due to tandem load.

P_{stl} = Single concentrated tandem load = 110kN

d_{tl}= 2*110*17.5^3/(48*1808465.9) = 0.01358m = 13.58 mm

iii) Deflection due to lane load

W_{L1}=9.3

$d_{La} = 5*W_{L1}*L_1^4/(384EI_e) =$ 5*(9.3*2)*17.5^4/(384*1808465.9) = 0.01256m = 12.56 mm

Thus, the total live load deflection becomes:

$d_k = 0.25d_{kt}+d_{la}$

$d_{kt} =$ (0.25*15.89)+12.56 =16.533 mm (for 25% of truck load)

$d_{max}=m_g*d_{kt}=$ 0.667*16.533=11.03mm (< 21.875mm) **The deflection is within the limit.**

Investigation of Fatigue Limit State (AASHTO, Section 5.5.3)

Magnitude and Configuration of Fatigue Load

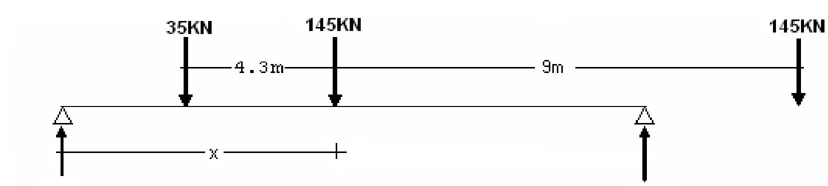
The fatigue load shall be one design truck or axles thereof specified in Article 3.6.1.2.2, but with a constant spacing of 9000mm between the 145 000-N axles. The dynamic load allowance specified in Article 3.6.2 shall be applied to the fatigue load.

U=0.75(LL+IM) ; F.S for LL is 0.75

U= Fatigue load shall be one design truck with 9m spacing.

Maximum moment results when the two front axles are on the span and the rear axle is out of the span.

M_{mf}= 0.75*1.15(M_{max})



Mmax occurs at x = 7 m and equals to 752.7KN-m

Mg^{SI}_m = 0.57/1.2= 0.475

M_{mf}= 0.75*1.5*Mg^{SI}_m*(M_{max}) = 402.224kN-m

a) Tensile live load stresses

for interior girder

d = 1072 mm, x = 211.48 mm, Icr=67.187*E+9 mm4

f_{smax}= 36.062 MPa

b) Reinforcing Bars

The stresses range in straight reinforcement bars resulting from fatigue load combination shall not exceed (AASHTO, Section 5.5.3.2).

If $f_{smax} < f_f$, then there is no problem of fatigue. Otherwise increase the area of reinforcing bars.

where:

f_f is the stress range.

f_{min} is the minimum live load stress resulting from fatigue load, combined with the more severe stress from permanent loads. For simplly supported slab bridge f_{min} is zero.

f_f=166-0.33f_{min} =166MPa

fs < ff (36.062 < 166 MPa) **OK!**

THE DESIGN IS COMPLETED!

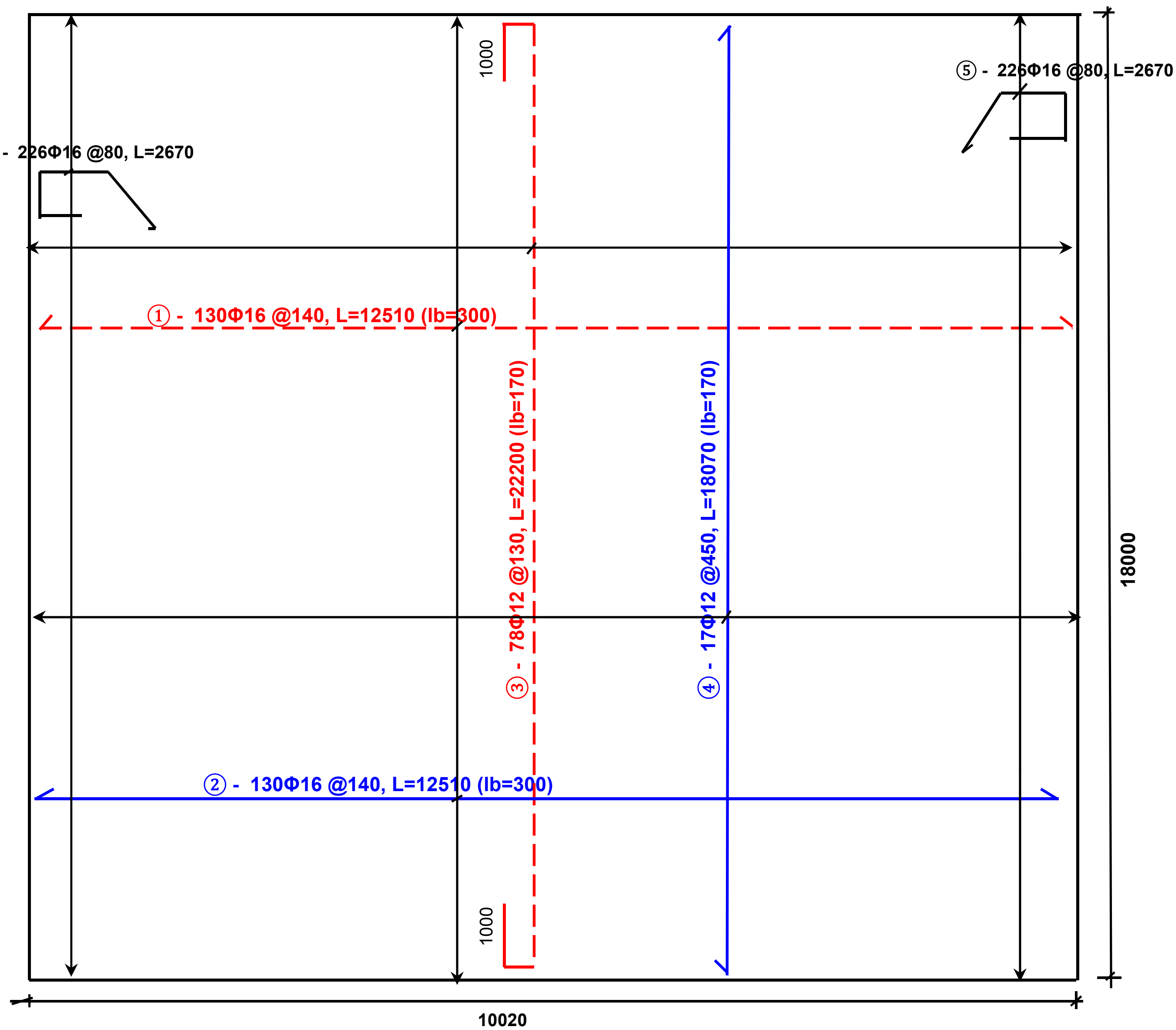


Fig 1. Plan

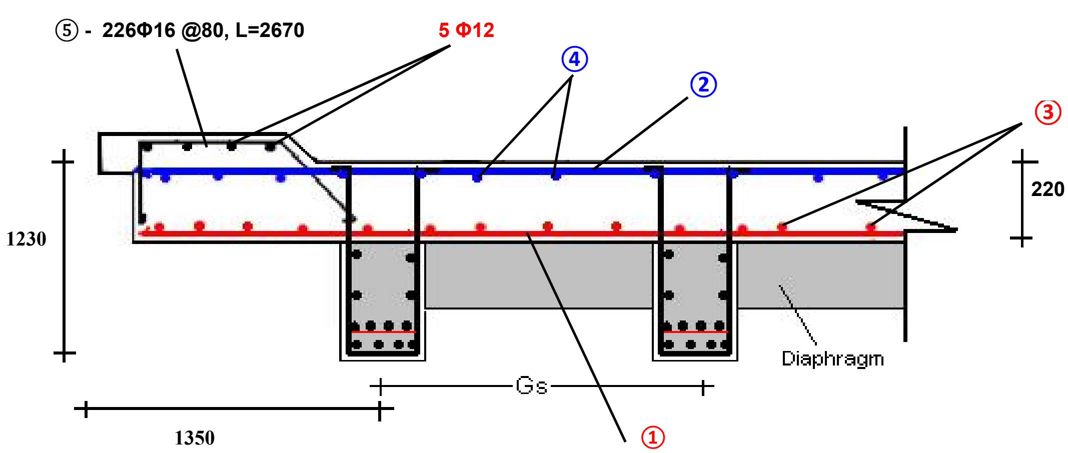
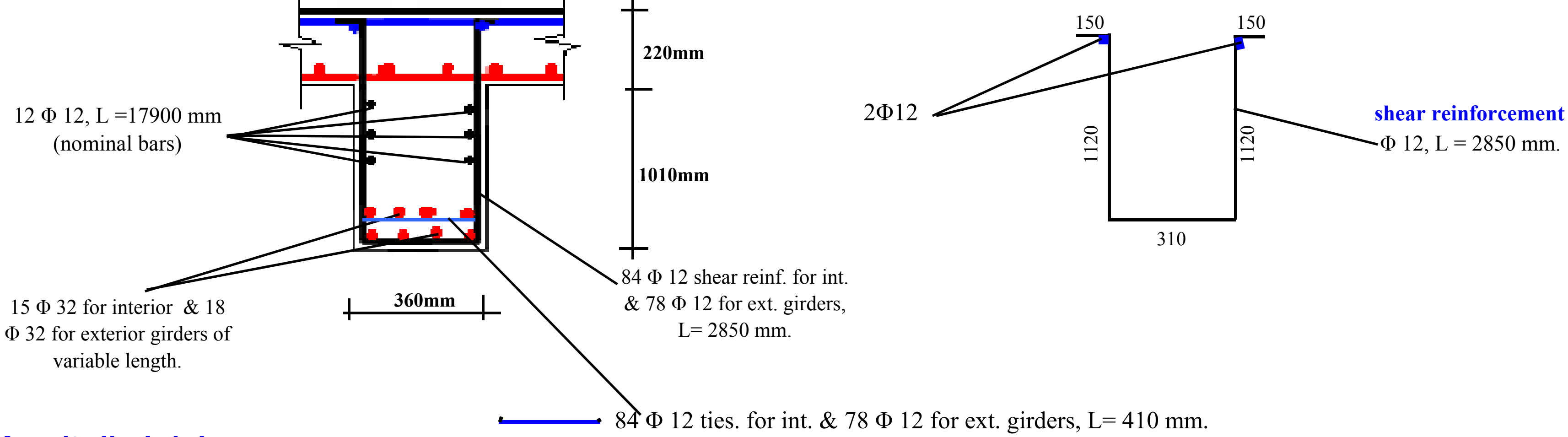


Fig 2. Bridge Section (deck slab reinforcements)

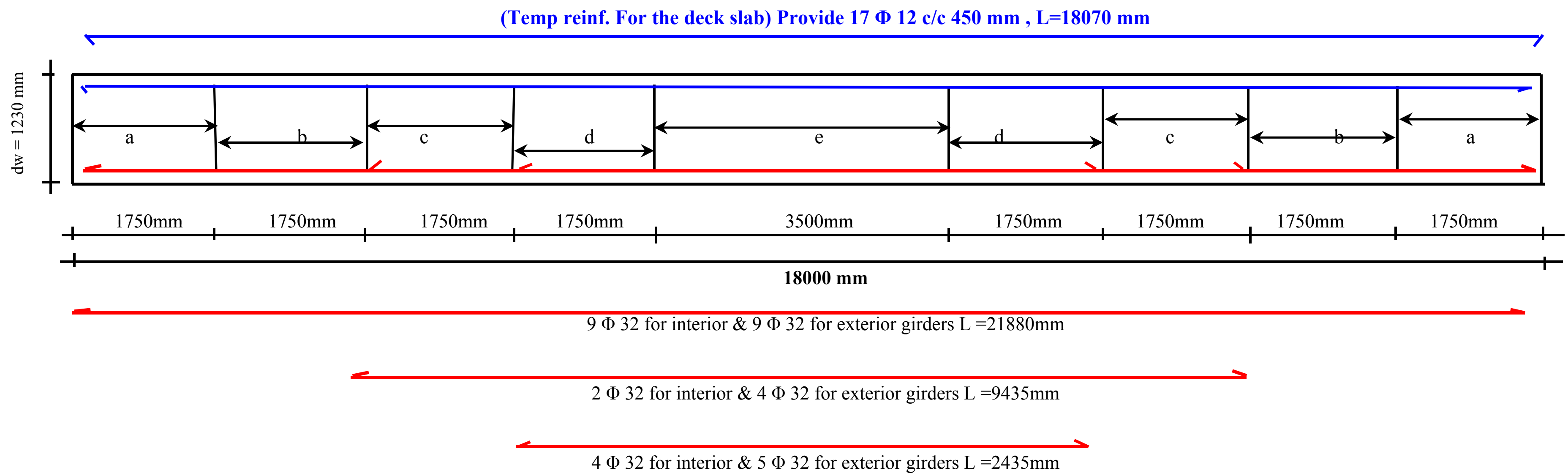
Designed by: Dr. Abrham G. 4/22/2020

Reinforcement details

Girder section



Longitudinal girder



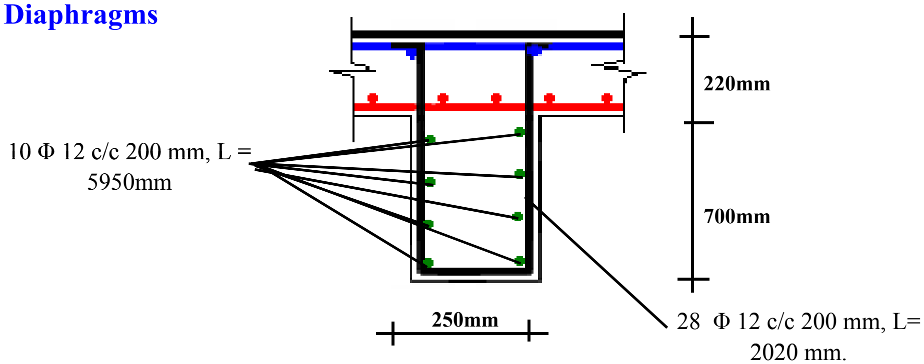
Shear reinforcements

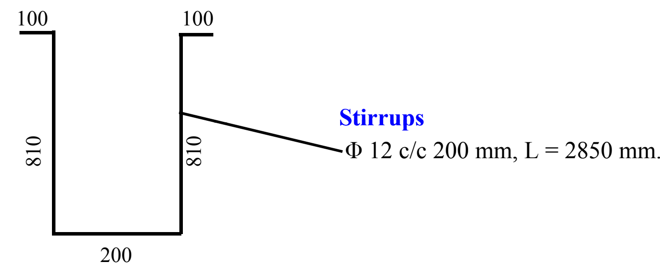
a = 12 Φ 12 c/c 150 mm for interior & 11 Φ 12 c/c 160 mm for exterior girders, L= 2850mm on both sides.
b = 10 Φ 12 c/c 180 mm for interior & 9 Φ 12 c/c 200 mm for exterior girders, L= 2850mm on both sides.
c = 8 Φ 12 c/c 250 mm for interior & 7 Φ 12 c/c 290 mm for exterior girders, L= 2850mm on both sides.
d = 6 Φ 12 c/c 300 mm for interior & 6 Φ 12 c/c 300 mm for exterior girders, L= 2850mm on both sides.
e = 12 Φ 12 c/c 300 mm for interior & 12 Φ 12 c/c 300 mm for exterior girders, L= 2850mm.

Designed by: Dr. Abrham G. 4/22/2020

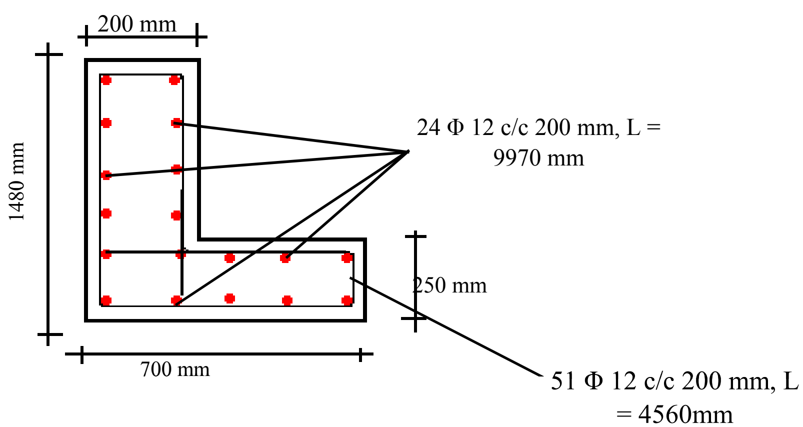
Reinforcement details

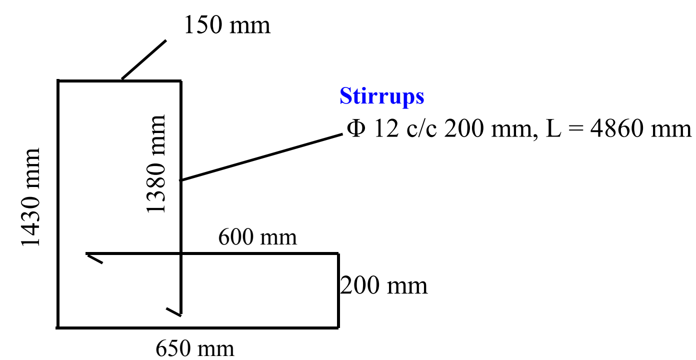
Diaphragms



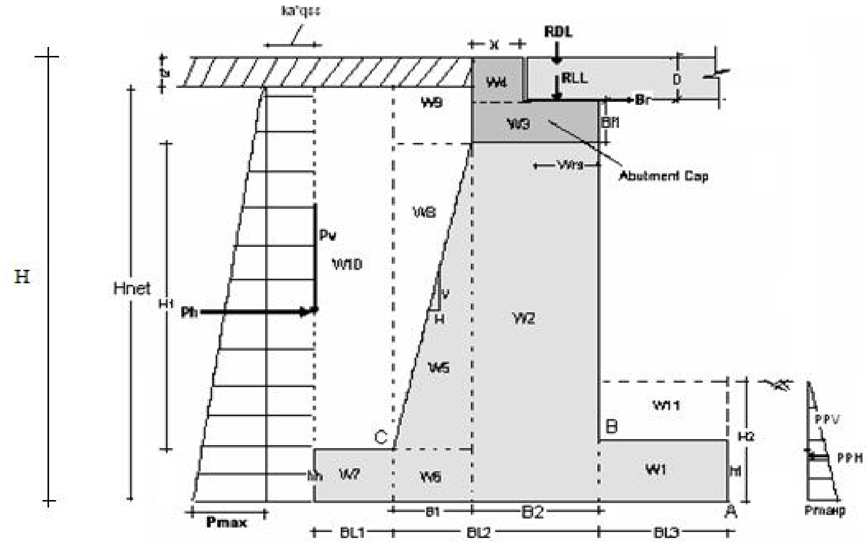


Beam seat





ABUTMENT DESIGN



Enter Preliminary Dimensions and Material Properties

Bottom width of the foundation on the left side, BL_1 = 1.75 m

Bottom width of the toe on the right side, BL_3 = 3.50 m

slope V:H = 2.5

Top width of the abutment cap, X= 0.20 m

Depth of the abutment cap, B_{fl}= 0.25 m

Depth of the base concrete, h_t= 0.50 m

Depth of the base concrete, h_h= 0.50 m

Total height of the abutment, H= 5.00 m

Allowable stress of the foundation Soil, s_{all} = 250 kPa

Friction angle of the backfill material, δ = 18 degrees

Density of the backfill material, γ_{bm} = 18.9 kN/m^3

Cohesion of the backfill material, C = 0

Unit weight of the abutment, γ_m = 24 kN/m^3 (Concrete)

Unit weight of the abutment cap (beam seat), γ_{ac} = 24 kN/m^3 (Stone masonry)

Soil thickness (surcharge), t_s = 0.22 m

Percentage of creep, shrinkage & temperature = 10 % of DL

Passive Pressure is considered.

Height, H_2 = 2.00 m

CALCULATIONS

1 DIMENSIONS

Net height of the abutment, H_{net}=(H-ts)= 4.78

H_1=H_{net}-hh-B_{fl} = 4.03m

B1=H1/Slope=4.03/2.5=1.61m

BL2=B1+Wrs+X = (1.61+0.5+0.2)=2.31 m

B2=Wrs+X = 0.5+0.2=0.7 m

B=BL1+BL2+BL3 = 1.75+2.31+3.5=7.56m

2 LOADS

i) Dead Load Reaction

Total DL reaction, Rdl= 1097.764kN (DL reaction from the deck slab and girders)

ii) Live Load Reactions

$R_{lli}=(max(V_{trl},V_{tml})+V_{ln})$

Vtr = 272.171kN, Vtm =212.457kN, Vln = 81.375kN

Total Live Load, Rlli = 965.888kN

iii) Wind Load on Live Loads

$WL=1.46*(H_1+1.8)$, 1.8m above roadway surface.

WL = 1.46*(250+1.8)=9.928kN

iv) Breaking Force, Brf

$B_{rf}=0.25*(2P_2+P_1)NL$, Braking force 25%, 1.8m above roadway surface.

=0.25*(2*145+35)*2= 162.5kN

v) Earthquake Force, EQ

EQ =48.775kN (This force is applied at the bearings)

vi) Creep, shrinkage & temperature...(10% of DL)

CR_SH_TU = 96.589 kN

vii) Lateral active earth pressure

$s_{ult} = 1.5S_{all}$ = 375kPa

$\phi=1.5\delta$= 27deg.

$\Theta=180-\tan^{-1}(H1/B1)$ = 111.8 deg.

$\eta=(1+Sqrt(Sin(\delta+\Theta))Sin\phi)/(Sin(\delta+\Theta)Sin\Theta)^2$ = 2.399

$Ka= (Sin(\phi+\Theta))^2/(\eta Sin^2\Theta(Sin(\Theta-\delta)))$ = 0.296

$\gamma= 180-(\delta+\Theta)$= 50.2deg.

$P_{max}=k_a\gamma_{bm}H_{net} -2C\sqrt{K_a}$

= 0.296*18.9*4.78-(2*0*√0.296) =26.741 kN/m2

$P_a=0.5P_{max}H_{net}$

= 0.5*26.741*4.78=63.911 kN/m

Pah=Pa*Sinγ=49.116 kN/m

Pav=Pa*Cosγ=40.893 kN/m

viii) Lateral passive earth pressure

kp = 1/ka = 3.377

$P_{maxp}=k_p\gamma_{bm}H_{net}+2C\sqrt{kp}$ = 127.7 kN/m2

$P_p=0.5P_{max}H_2$ = 127.7 kN/m

Pph=Pp*Sinγ = 98.1 kN/m

Ppv=Pp*Cosγ = 81.68 kN/m

ix) Dead load Surcharge

$q_{ss}= \gamma_{bm}*ts$

= 18.9*0.22 =4.158kPa

$Pressure =k_aq_{ss}$= 1.23 kN/m

$Qsh=k_aq_{ss}Sin(\gamma)$= 0.945 kN/m

$Qsv=k_aq_{ss}Cos(\gamma)$= 0.787 kN/m

x) Live load Surcharge

$Pressure=k_ah_{eq}\gamma_{bm}$= 4.872 kN/m

Q_{lh}=Pressure Sin(γ)= 3.685 kN/m

Q_{lv}=Pressure Cos(γ)= 3.188 kN/m

3 LOAD COMBINATIONS

Strength I:DC=1.25, EV=1.35, EH=1.5, LL=1.75, BR=1.75, LS=1.75, WS=0, WL=0, ES=1.5, CR_SH_TU=0.5

Strength Ia:DC=0.9, EV=1, EH=0.9, LL=1.75, BR=1.75, LS=1.75, WS=0, WL=0, ES=0.75, CR_SH_TU=0.5

Total Width of the abutment = 10.02m

		Unfactored Loads	Factored Forces (Strength I)	Factored Forces (Strength Ia)	Moment Arm from Pt. A	Unfactored Moments	Factored Moments (Strength I)	Factored Moments (Strength Ia)
Vertical Loads	W1	420.84	526.05	378.76	1.75	736.47	920.59	662.82
	W2	762.56	953.20	686.31	3.85	2935.86	3669.83	2642.28
	W3	42.08	52.61	37.88	3.85	162.02	202.53	145.82
	W4	59.16	73.95	53.24	4.10	242.55	303.19	218.29
	W5	683.36	854.20	615.02	4.74	3236.85	4046.06	2913.16
	W6	193.59	241.98	174.23	5.01	968.90	1211.12	872.01
	W7	210.42	263.03	189.38	6.39	1345.29	1681.61	1210.76
	W8	538.15	672.68	484.33	5.27	2837.82	3547.28	2554.04
	W9	384.17	480.22	345.75	5.01	1922.78	2403.48	1730.50
	W10	1584.15	1980.18	1425.73	6.39	10127.98	12659.97	9115.18
	W11	994.23	1242.79	894.81	1.75	1739.91	2174.89	1565.92
	RDL	1097.76	1372.21	987.99	3.75	4116.62	5145.77	3704.95
	RLL	965.89	1690.30	1690.30	3.75	3622.08	6338.64	6338.64
	Pav	195.47	263.88	195.47	7.56	1477.74	1994.95	1477.74
	Ppv	163.35	220.53	163.35	0.00	0.00	0.00	0.00
	Qsv	37.69	50.89	37.69	7.56	284.97	384.70	284.97
	Qlv	31.94	43.12	31.94	7.56	241.46	325.98	241.46
	TOTAL	**8,364.82**	**10,981.81**	**8,392.19**		**35,999.30**	**47,010.59**	**35,678.56**
Horizontal Loads	Brf	162.50	284.38	284.38	6.58	1069.25	1871.19	1871.19
	WL	9.93	0.00	0.00	6.58	65.33	0.00	0.00
	CR,SH,TU	96.59	48.29	48.29	4.78	461.70	230.85	230.85
	Pah	234.77	352.16	211.30	1.59	374.07	561.11	336.67
	Pph	196.20	294.30	176.58	-0.67	-130.80	-196.20	-117.72
	Qsh	45.27	67.91	40.75	2.39	108.21	162.31	97.38
	Qlh	36.92	55.38	33.23	2.39	88.24	132.35	79.41
	TOTAL	**830.96**	**1,102.42**	**794.52**		**2,209.14**	**2,761.61**	**2,497.78**
						∑M	**44,248.98**	**33,180.78**

Stability and Safety Criteria

1 Eccentricity $\% = (e_{max} - e) * 100 / e_{max}$

	V	∑M	$X_o = \sum M/V$	$e = B/2 - X_o$	$e_{max} = B/4$	Design Margin (%)	Remark
Strength I	10,981.810	44,248.98	4.029	-0.249	1.890	86.81	**OK!**
Strength Ia	8,392.185	33,180.78	3.954	-0.174	1.890	90.81	**OK!**

2 Sliding $\% = (\varphi_s * Fr - H) * 100 / \varphi_s * Fr$ $\varphi_s = 0.8$

	V	tanδ	Fr=V*tanδ	φ_s *Fr	HL	Design Margin (%)	Remark
Strength I	10,981.810	0.33	3569.738	2855.790	1102.423	61.40	**OK!**
Strength Ia	8,392.185	0.33	2727.957	2182.365	794.522	63.59	**OK!**

3 Bearing Capacity $\% = (\varphi_b * RI * q_{ult} - q_{max}) * 100 / (\varphi_b * RI * q_{ult})$ $\varphi_b = 0.5$

For bearing capacity criteria, $q_{ult} = 2*250 = 500$kPa

$q_{ult} = 500$ kPa $RI = (1-(H/V))^3$ $q_{max} = Vn/(2*Xo*B)$

$q_{ult} = 500$ kPa $RI = (1-(H/V))^3 = 0.728$ $q_{max} = Vn/(2*X_o*B)$

	H_n	V_n	H_n/V_n	$\varphi_b * RI * q_{ult}$	q_{max}	Design Margin (%)	Remark
Strength I	1,102.42	10,981.81	0.100	182.015	180.257	0.97	**OK!**
Strength Ia	794.522	8,392.19	0.095	185.505	140.382	24.32	**OK!**

Remark: The Preliminary Dimensions of the Abutment is adequate!

Design of Bearing (Using 60 durometer reinforced bearing)

Design Data:

Expandable span length of the bridge =17.5m

Dead Load reaction/girder= 396.27kN

Exterior girder=0.5L(41.263+4.025)=396.27kN

Interior girder=0.5L(22.53+7.379)=261.704kN

Live Load reaction (without inpact)/girder=mgv*[Max(Vtr,Vtm)+Vln]=354.961kN

Maximum temperature change =21 deg. C

- $\Delta_{shrnkage}$= Girder shortening due to concrete shirinkage =2 mm

Shear modulus of elastomer, G= 0.9 to 1.38 MPa (For this bearing deign, use G=1 MPa)

γ=Load factor for uniform temp. and etc = 1.2

Constant amplitude of fatigue treshold for Category A =165MPa

1. Temperature Movement

Thermal coefficient of normal concrete, α, is $10.8x10^{-6}/^{o}C$

$\Delta_{Temp}= \alpha \Delta T\ L=$ 0.0000108*21*17.5*1000=3.97mm

2. Girder Shortening

Δ_{sh}= 2mm

Bearing maximum longitudinal movement=$\gamma(\Delta_{sh}+\Delta_{Temp})$ = 1.2*(2+3.97)=7.164mm

3. Determination of the minimum bearing area

According to S14.7.5.3.2, the maximum compressive stress limit under service limit state for bearings fixed against shear deformations:

$\sigma_s \leq 1.66GS \leq 11MPa$

$\sigma_L \leq 0.66GS$

where:

σ_s = service average compressive stress due to the total load (MPa)

σ_L = service average compressive stress due to live load (MPa)

G = shear modulus of elastomer (MPa)

S = shape factor of the thickest layer of the bearing

To satisfy the 11MPa limit, the minimum bearing area, A_{req}, should satisfy:

$A_{req}>$ (396.27+354.961)*1000/11=68293.73 (mm^2)

For a first estimate,

-choose a width of W=(Width of girder bottom flange – 2(chamfer +edge clearance)

W=360-2(12.5+12.5)=310mm and L=320mm

⟶ Trial size of bearing is 310mm x 320mm, A=99200 >68293.73 (OK!)

The shape factor of a layer of an elastomeric bearing, S_i, is taken as the plan area of the layer divided by the area of perimeter free to bulge.

$S_i = LW/[2h_{ri}(L + W)]$

where:

L = length of a rectangular elastomeric bearing (parallel to the longitudinal bridge axis) (m

W = width of the bearing in the transverse direction (mm)

h_{ri} = thickness of ith elastomeric layer in elastomeric bearing (mm)

4.Compressive stress

Shape factor under total load, S_{TL},(S14.7.5.3.2-3)

$S_{TL} \geq \sigma_s/1.66G$

where:

$\sigma_s = P_{TL}/A_{req}$

P_{TL}= maximum bearing reaction under total load

$S_{TL} \geq$ (751.231*1000/99200)/(1.66*1)

$\geq$ 4.56

Shape factor under live load, S_{LL},

$S_{LL} \geq \sigma_L/0.66G$

where:

$\sigma_L = P_{LL}/A_{req}$

P_{LL}= maximum bearing live load reaction

$S_{LL} \geq (354.961*1000/99200)/(0.66*1)$

≥ 5.42

Thus, the minimum shape factor of any layer is 5.42

Notice that if holes are present in the elastomeric bearing their effect needs to be accounted for when calculating the shape factor because they reduce the loaded area and increase the area free to bulge.

5.Elastomer Thickness

Using the shape factors of S_{TL} and S_{LL} calculated above, determine the elastomer thickness.

$h_{ri(TL)} < (LW)/[2(S_{TL})(L + W)]$

$$h_{ri(TL)} < \frac{320*310}{2*4.56(320+310)}$$

< 17.27

$h_{ri(LL)} < (LW)/[2(S_{LL})(L + W)]$

$$h_{ri(IL)} < \frac{320*310}{2*5.42(320+310)}$$

< 14.53

⟶ Use an interior elastomer layer thickness of hri =14mm

The corresponding shape factor is:

$S = (LW)/[2(h_{ri})(L + W)] = 5.62$

Check compressive stresses

-average compressive stress due to toal load

$\sigma_s = P_{TL}/A = 751.231*1000/99200=7.57MPa$

i. σs=7.57MPa < 1.66GS(=9.33MPa) **Ok!**

ii. 1.66GS (=9.33MPa) < 11MPa **Ok!**

-average compressive stress due to live load

$\sigma_L = P_{LL}/A = 354.961*1000/99200=3.58MPa$

i. σs=3.58MPa < 0.66GS (=3.71MPa) **Ok!**

6. Compressive deflection

Deflections of elastometric bearings due to total load and live load alone will be considered separately

Instantaneous deflection is be taken as:

$\delta=\sum\varepsilon_i h_{ri}$ (S14.7.5.3.3-1)

where: ε_i = instantaneous compressive strain in ith elastomer layer of a laminated bearing

h_{ri} = thickness of i^{th} elastomeric layer in a laminated bearing

Values for ε_i are determined from test results or by analysis when considering long-term deflections.

Obtain ε_i from Fig. C14.7.5.3.3.1 of AASHTO.

For σs=7.57MPa and S=5.62, the value of εi= 0.0515

7. Shear deformation

The bearing is required to satisfy:

$h_{rt} \geq 2\,\Delta_s$ (S14.7.5.3.4-1)

$h_{rt} \geq 2*7.164=14.328mm$

where: h_{rt} =total elastomer thickness (sum of the thicknesses of all elastomer layers)

Δs =maximum shear deformation of the elastomer at the service limit state

8. Combined compression and rotation

Rectangular bearings are assumed to satisfy uplift requir ements if they satisfy:

$\sigma s > 1.0GS(\theta_s/n)(B/h_{ri})^2$ (S14.7.5.3.5-1)

where: n = number of interior layers of elastomer (interior layers are bonded on each face).

hri, thickness of ith elastomeric layer =14 mm

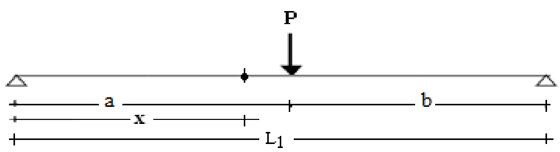
σs =maximum compressive stress in elastomer =7.57MPa

B = length of pad if rotation is about its transverse axis or width of pad if rotation is about i longitudinal axis. B=L=320mm

θs = maximum service rotation due to the total load (rads).

It will include the rotations due to live load and construction load (assume 0.005 rads)

The rotation at a point , x, due to each live load at a distance 'a' from the left support is given by:



$$EI\theta(x) = \frac{pbx^2}{2L} - \frac{pb}{6L}[L^2 - b^2]$$ (for x<a)

Thus, the rotation at the left support is obtained by setting x=0.

$$EI\theta(0) = -\frac{pb}{6L}[L^2 - b^2]$$

Girder	EI_{eff}	Location of Loads			Rotation, θ
		1^{st} (145kN)	2^{nd} (145kN)	3^{rd} (35kN)	θ
Interior	1.81E+15	b=7.875m	b=3.575m	b=-0.725m	-0.0023 rad
Exterior	2.07E+15	b=7.875m	b=3.575m	b=-0.725m	-0.002 rad

Rotation due to DL

$$\theta_{DC} = \frac{-wL^3}{24EI_e}$$

Girder	EI_{eff}	w	θ
Interior	1.81E+15	22.53KN/m	-0.0028 rad
Exterior	2.07E+15	41.263KN/m	-0.0045 rad

Long term rotation produced by creep effect should be considered in the calculation of camber.

$\theta_{dL}=3*\theta_{DC}= 3*0.0045=0.0135$ rad

$\theta_{design}=m_g*\theta_{LL}+=\theta_{dL}+\theta_{alw}= 0.667*(0.0023)+0.0135+0.005=0.0200341$ rad

8.1 Uplift requirement

To determine the number of interior layers of elastomer, n_u, for uplift

$n_u > 1.0GS(\theta_s)(B/h_{ri})^2/\sigma_s$

$$n_u > \frac{1.0*1*5.62*0.0200341(320/14)^{\wedge}2}{7.57}$$

> 7.77

8.2 Shear deformation requirement

To prevent excessive stress on the edges of the elastomer, rectangular bearings fixed against shear deformation must also satisfy:

$\sigma s < 1.875GS[1 - 0.2(\theta_s/n)(B/h_{ri})^2]$

The number of interior layers of elastomer, nc,required to limit compression along the edges.

$$n_c > \frac{-0.20(\theta s)(B/h_{ri})^2}{[\sigma_s/1.875GS-1]}$$

> 7.43

Use 8 interior layers, 14mm thick each

Exterior layers 9mm thick each (< 70% of the thickness of the interior layer)

9. Bearing maximum rotation

Instantaneous deflection is :

$\delta = \sum \varepsilon_i h_{ri}$ ($\varepsilon i = 0.0515$)

$\delta =$ $\delta = 9*0.0515*14 = 6.489$mm

Bearing rotational capacity:

$\theta_{capacity} = 2\delta/L = 0.041$ rad

$\theta_{design} = 0.0200341$ rad (< 0.041 rad) OK!

10. Stability of elastomeric bearings

The bearing pad should be designed to prevent instability at the service limit state by limiting the aver compressive stress to one half the estimated buckling stress.

For the bridge deck free to translate horizontally

$$\sigma_s \le \sigma_{cr} = \frac{G}{2A - B} \quad \text{where,} \quad A = \frac{1.92\frac{h_{rt}}{L}}{S\sqrt{1+\frac{2L}{W}}} \quad \text{and} \quad B = \frac{2.67}{S(S+2)\left(1+\frac{L}{4W}\right)}$$

hrt=8(14)+2(9)= 130mm

$$A = \frac{1.92*130/320}{5.62\text{Sqrt}(1+2*320/310)} = 0.079$$

$$B = \frac{2.67}{5.62(5.62+2)(1+320/4*310)} = 0.05$$

G/(2A-B) =9.26MPa (>7.57MPa) **The bearing is stable**

11. Bearing Steel Reinforcement

The reinforcement should sustain the tensile stresses induced by compression on the bearing. With the present load limitations, the minimum steel plate thickness practical for fabrication will usually provic adequate strength.

At the service limit state:

$h_s \ge 3h_{max}\sigma s/f_y$ (S14.7.5.3.7-1)

where:

h_{max} = thickness of thickest elastomeric layer in elastomeric bearing

σs= hmax=14 mm, σs=7.57MPa

Fy = yield strength of steel reinforcement = 400MPa

⟶ $h_{s(TL)} \ge 0.79$ mm

At the fatigue limit state:

$h_s \ge 2h_{max}\sigma_L/\Delta F_{TH}$ $\sigma_L = 3.58$MPa

ΔF_{TH} =constant amplitude fatigue treshold for category A

⟶ $h_{s(LL)} \ge 0.61$ mm

Use 9 steel reinforcement plates. hs= 1 mm thick each

If holes exist in the reinforcement, the minimum thickness is increased by a factor equal to twice the gross width divided by the net width. Holes in the reinforcement cause stress concentrations.

The total height of the bearing, hrt :

h_{rt} = cover layers + elastomer layers + shim thicknesses

=2(9)+8(14)+9(1)= 139mm (> $2\Delta = 2*7.164 = 14.328$mm) Ok!

Elastomeric Bearings (grade 60 Shore A Durometer hardness)

8 interior layers, 14mm thick each

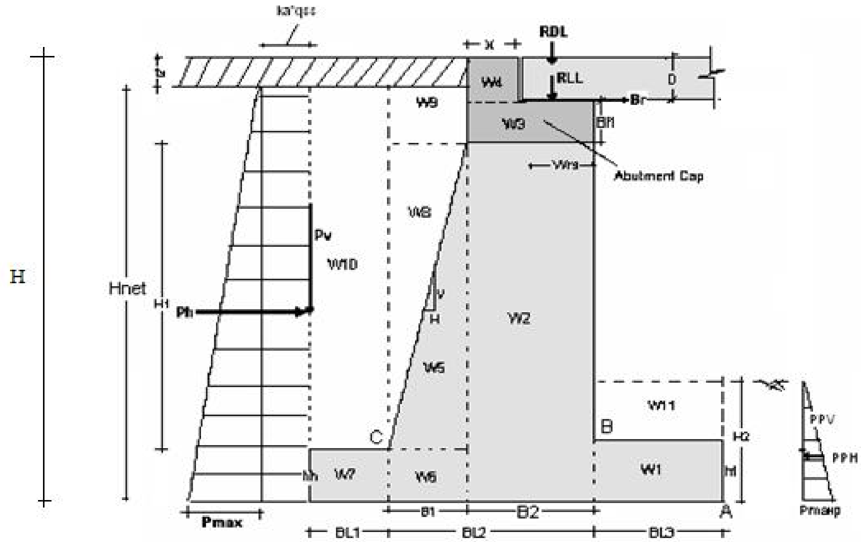
2 Exterior layers 9mm thick each

9 steel reinforcement plates. hs= 1 mm thick each

Total thickness of bearing is 139 mm

Bearing size: 310mm (transverse) x 320mm (longitudinal)

ABUTMENT DESIGN



Enter Preliminary Dimensions and Material Properties

Bottom width of the foundation on the left side, BL_1 = 1.75 m
Bottom width of the toe on the right side, BL_3 = 3.50 m
slope V:H = 2.5
Top width of the abutment cap, X= 0.20 m
Depth of the abutment cap, B_{fl}= 0.25 m
Depth of the base concrete, h_t= 0.50 m
Depth of the base concrete, h_h= 0.50 m
Total height of the abutment, H= 5.00 m
Allowable stress of the foundation Soil, s_{all} = 250 kPa
Friction angle of the backfill material, δ = 18 degrees
Density of the backfill material, γ_{bm} = 18.9 kN/m^3
Cohesion of the backfill material, C = 0
Unit weight of the abutment, γ_m = 24 kN/m^3 (Concrete)
Unit weight of the abutment cap (beam seat), γ_{ac} = 24 kN/m^3 (Stone masonry)
Soil thickness (surcharge), t_s = 0.22 m
Percentage of creep, shrinkage & temperature = 10 % of DL
Passive Pressure is considered.
Height, H_2 = 2.00 m

CALCULATIONS

1 DIMENSIONS

Net height of the abutment, H_{net}=(H-ts)= 4.78
$H_1=H_{net}-hh-B_{fl}$ = 4.03m
B1=H1/Slope=4.03/2.5=1.61m
BL2=B1+Wrs+X = (1.61+0.5+0.2)=2.31 m
B2=Wrs+X = 0.5+0.2=0.7 m
B=BL1+BL2+BL3 = 1.75+2.31+3.5=7.56m

2 LOADS

i) Dead Load Reaction

Total DL reaction, Rdl= 1097.764kN (DL reaction from the deck slab and girders)

ii) Live Load Reactions

R_{lli}=(max(V_{trl},V_{tml})+V_{ln})

Vtr = 272.171kN, Vtm =212.457kN, Vln = 81.375kN

Total Live Load, Rlli = 965.888kN

iii) Wind Load on Live Loads

WL=1.46*(H_1+1.8), 1.8m above roadway surface.

WL = 1.46*(250+1.8)=9.928kN

iv) Breaking Force, Brf

B_{rf}=0.25*(2P_2+P_1)NL, Braking force 25%, 1.8m above roadway surface.

=0.25*(2*145+35)*2= 162.5kN

v) Earthquake Force, EQ

EQ =48.775kN (This force is applied at the bearings)

vi) Creep, shrinkage & temperature...(10% of DL)

CR_SH_TU = 96.589 kN

vii) Lateral active earth pressure

$s_{ult} = 1.5S_{all}$ = 375kPa

$\phi=1.5\delta$= 27deg.

$\Theta=180-\tan^{-1}(H1/B1)$ = 111.8 deg.

$\eta=(1+Sqrt(Sin(\delta+\Theta))Sin\phi)/(Sin(\delta+\Theta)Sin\Theta)^2$ = 2.399

$Ka= (Sin(\phi+\Theta))^2/(\eta Sin^2\Theta(Sin(\Theta-\delta)))$ = 0.296

$\gamma= 180-(\delta+\Theta)$= 50.2deg.

$P_{max}=k_a\gamma_{bm}H_{net} -2C\sqrt{K_a}$

= 0.296*18.9*4.78-(2*0*√0.296) =26.741 kN/m2

$P_a=0.5P_{max}H_{net}$

= 0.5*26.741*4.78=63.911 kN/m

Pah=Pa*Sinγ=49.116 kN/m

Pav=Pa*Cosγ=40.893 kN/m

viii) Lateral passive earth pressure

kp = 1/ka = 3.377

$P_{maxp}=k_p\gamma_{bm}H_{net}+2C\sqrt{kp}$ = 127.7 kN/m2

$P_p=0.5P_{max}H_2$ = 127.7 kN/m

Pph=Pp*Sinγ = 98.1 kN/m

Ppv=Pp*Cosγ = 81.68 kN/m

ix) Dead load Surcharge

$q_{ss}= \gamma_{bm}$*ts

= 18.9*0.22 =4.158kPa

Pressure $=k_aq_{ss}$= 1.23 kN/m

$Qsh=k_aq_{ss}Sin(\gamma)$= 0.945 kN/m

$Qsv=k_aq_{ss}Cos(\gamma)$= 0.787 kN/m

x) Live load Surcharge

$Pressure=k_ah_{eq}\gamma_{bm}$= 4.872 kN/m

Q_{lh}=Pressure Sin(γ)= 3.685 kN/m

Q_{lv}=Pressure Cos(γ)= 3.188 kN/m

3 LOAD COMBINATIONS

Strength I:DC=1.25, EV=1.35, EH=1.5, LL=1.75, BR=1.75, LS=1.75, WS=0, WL=0, ES=1.5, CR_SH_TU=0.5

Strength Ia:DC=0.9, EV=1, EH=0.9, LL=1.75, BR=1.75, LS=1.75, WS=0, WL=0, ES=0.75, CR_SH_TU=0.5

Design of Barriers

Strength limit state and extreme event limit state are considered for the design of barriers. The design forces for a TL-4 barrier as per AASHTO: Table A13.2.1 is used.

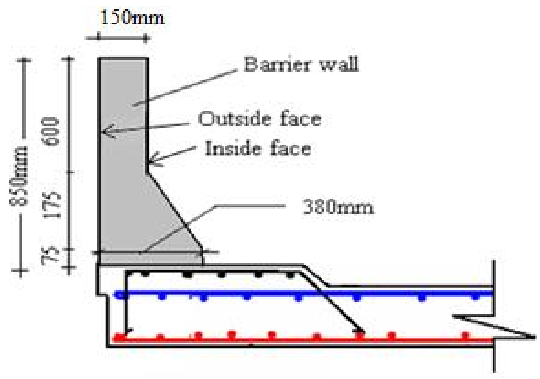


Fig: Cross section of a bridge

H= Barrier height= 0.85
top width= 150
cover= 25
bottom width= 350

Design forces for a TL-4 barrier (AASHTO: Table A13.2.1)

Direction	Force (KN)	Length(m)
Transverse (F_t)	240	1.07
Longituidinal (F_l)	80	1.07
Vertical (F_v)	80	5.5

$$F_t = R_w$$

$$R_w = \frac{2}{2I_c - L_t}\left(8M_b + 8M_w + \frac{M_c L_c^2}{H}\right) \quad \text{and} \quad L_c = \frac{l_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + \frac{8H(M_b + M_{wH})}{M_c}}$$

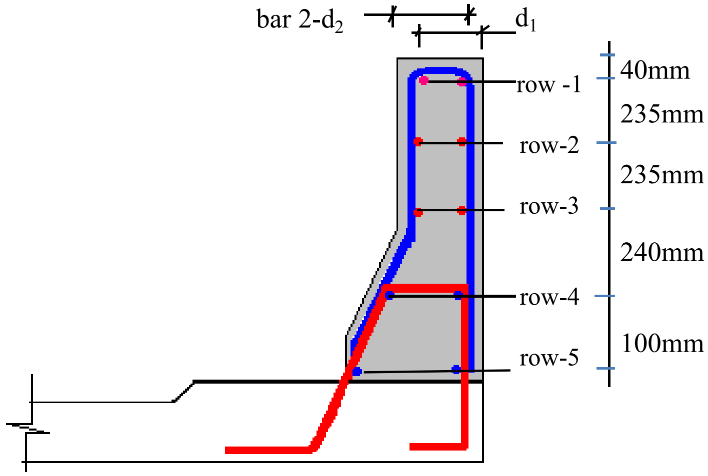
Where:

M_b - additional flexural resistance of beam in addition to M_w, if any, at top of wall (kN-m)
M_w - flexural resistance of the wall about its vertical axis (kN-m)
Mc - flexural resistance of cantilevered walls about an axis parallel to the longitudinal axis of the bridge (kN-m/m)
H - height of wall
L_t- longitudinal length of distribution of impact force, Ft (kN)
L_c - critical length of yield line failure pattern (m)

i) Flexural capacity of the wall about vertical axis, M_w

diam of horizontal bars= 10

Assume c/c spacing for horizontal bars (mm) 200



* bar 1 is on the inside face and bar 2 is on the outside face

* Provide 5φ10 horizontal bars, L= 18120mm on both faces (spacing of bars is as shown in the above diagram)

$$M_w = \varphi A_s f_y \left(d - \frac{a}{2} \right)$$

for extreme limit state φ =1

φ10, A=78.5mm2

$A_{s,tot}=n_b*A=$ 5*78.5=392.5mm2

$a=A_{s,tot} f_y/(0.85 f_c' b)$

$$a=\frac{392.5*400}{0.85*28*850}=7.76mm$$

Calculation of M_w

row	measured from the outside face	M_{w1}	measured from the inside face	M_{w2}
	d_1 (bar1)		d_2 (bar2)	
1	110	3.332	110	3.332
2	110	3.332	110	3.332
3	110	3.332	110	3.332
4	324	10.061	324	10.061
5	310	9.612	310	9.612
	Total	**29.669**		**29.669**

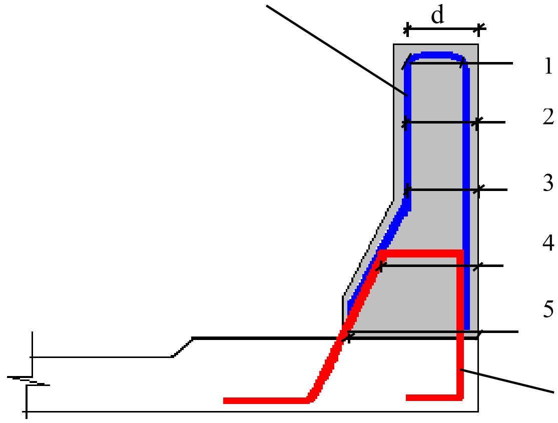
34.9 $M_{wi}=M_{w1}/H=$ 34.9kN-m/m $M_{wo}=M_{w2}/H=$ 34.9kN-m/m

Thus, $M_w= (2M_{wi}+M_{wo})/3$ =34.9kN-m/m

ii) Flexural capacity of the wall about horizontal axis, M_c

diam of vertical bars= 10

Assume c/c spacing for vertical bars (mm) 300 number of bars, n_b= 5



$$M_c = \varphi A_s f_y \left(d - \frac{a}{2} \right)$$

for φ=10, A=78.5mm2

$A_{s,tot}=n_b A$ =5*78.5mm2/m

=392.5mm2/m

$a=A_{s,tot} f_y/(0.85 f_c' b)$

a=6.6mm

Calculation of M_c

Location	d=measured from	d_{ave}	M_c	H_i
1	120	120	18.322	510
2	120			
3	120			
4	334	327.14	50.973	340
5	320			

Thus, $M_c= \sum(M_{ci}*H_i)/\sum H_i$

= 31.382kN-m

$$L_c = \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + \frac{8H(M_b + M_w H)}{M_c}}$$ =3429.457mm (M_b=0)

$$R_w = \frac{2}{2L_c - L_t}\left(8M_b + 8M_w + \frac{M_c L_c^2}{H}\right)$$ =246.479kN **(>240kN) OK!**