ADDIS ABABA UNIVERSITY

ADDIS ABABA INSTITUTE OF TECHNOLOGY

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

DESIGN OF RC SLAB BRIDGE

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RC SLAB BRIDGE DESIGN

Design Data and Specifications

i) Material Properties

Steel strength, f_y=	400	MPa
Concrete strength, f'_c =	28	MPa
Concrete density, γ_c =	2400	kg/m^3
Bituminous density, γ_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 =	12	m
Road way /clear carriage width, R_w =	7.32	m
Additional curb width including bottom width of the concrete barrier/ posts, Cw=	1.25	m
Curb depth, C_d =	0.25	m
Bearing shelf width, W_{rs} =	0.5	m
Concrete posts and railings are used		
Railing depth, R_d =	0.3	m
Railing width, R_{ww} =	0.15	m
Post depth, P_d =	0.3	m
Post width, P_w =	0.3	m
Post height, P_h =	0.85	m
Post spacing, P_s =	2.32	m
Thickness of Asphalt Layer =	100	mm
Concrete cover for the slab =	35	mm

iii) Reinforcement Sizes

Diameter of main reinforcement =	32	mm
Diameter of distribution reinforcement =	16	mm
Diameter of temperature reinforcement =	12	mm

*** Design Method: LRFD**

*** Specifications:**

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

Design of RC Slab Bridge (12m of Clear Span)

1.Depth Determination

According to AASHTO (Table 2.5.2.6.3-1), minimum recommended depth for slabs with main reinforcement parallel to the traffic is:

D=1.2(S+3000)/30 where: S= Span: C_s+W_{rs} = 12.50 m (c/c spacing of the bridge)

D=1.2*(12500+3000)/30=620mm

Thus, Use D =620mm

2. Equivalent Strip widths, AASHTO Article 4.6.2.3

a) Interior Strip

i) One lane loaded: multiple presence factor included

E=min (18,000 , 250+0.42$(S*W)^{1/2}$) = 4704.773 mm ($W=R_w+2C_w$)

where: W is the edge-to-edge width of the bridge

ii) Multiple lanes loaded

Number of lanes loaded, NL= int (7.32/3.6)=2

E=2100+0.12Sqrt(S*W) =3429.511 mm <W/NL(9820/2=4910mm)

Use E = 3429.511 mm

Shear and moment per meter width of strip is critical for multiple lanes loaded because (4704.773 mm >3429.511 mm)

Equivalent Concentrated and distributed loads

Truck P_1'= 35kN/3429.511mm = 10.206kN/m

Truck P_2'= 145kN/3429.511mm = 42.28kN/m

Tandem P_3'= 110kN/3429.511mm = 32.075kN/m

Lane W'= 9.3kN/m/3429.511mm = 2.712kN/m2

b) Edge Strip

Longitudinal edge strip width for a line of wheels

E=distance from edge to face of barrier +300+1/2 strip width ≤ 1800mm

E =1250+300 +(1/2*3429.511) = 3264.756 mm

E=min (3264.756,1800)= 1800 mm

3. Influence Lines for shear force and bending moment

* Slab bridges shall be designed for all vehicular live loads specified in AASHTO Art. 3.6.1.2 including the lane load.

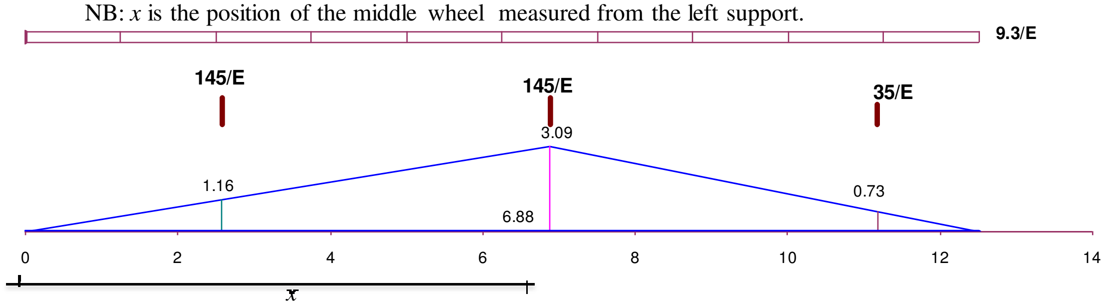
Live Load Force Effects

a) Interior Strip

i) Influence Line for Moment (Truck Load)

At position where x = 6.875 m, Mtr =187.233 kN-m/m (Max. effect of truck load)

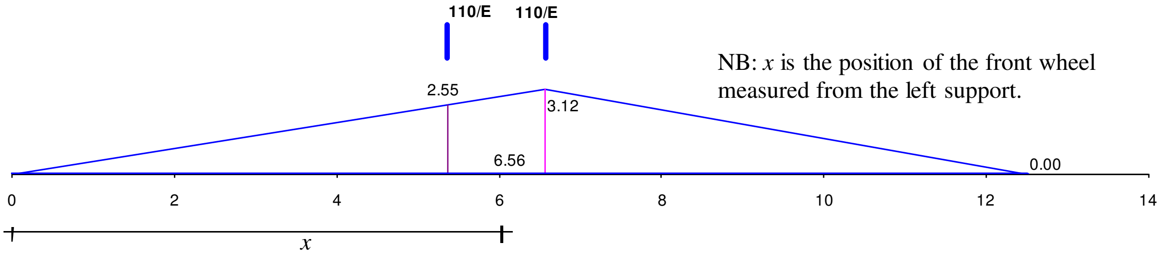
NB: *x* is the position of the middle wheel measured from the left support.



$M_{lane, max}$= 2.712*12.5^2/8 = 52.969kN-m/m

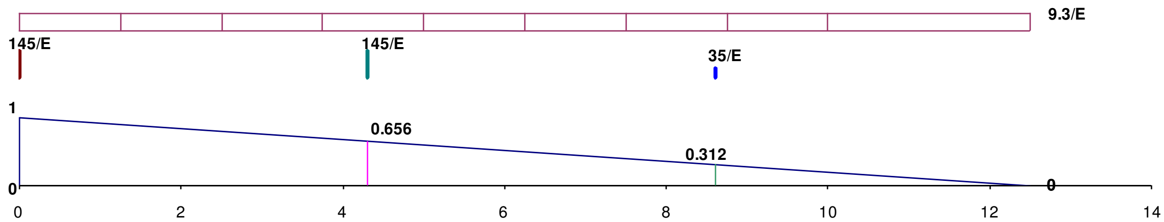
ii) Influence Line for Moment (Tandem Load)

At position where x = 6.5625 m, Mtm =181.685 kN-m/m (Max. effect of tandem load)



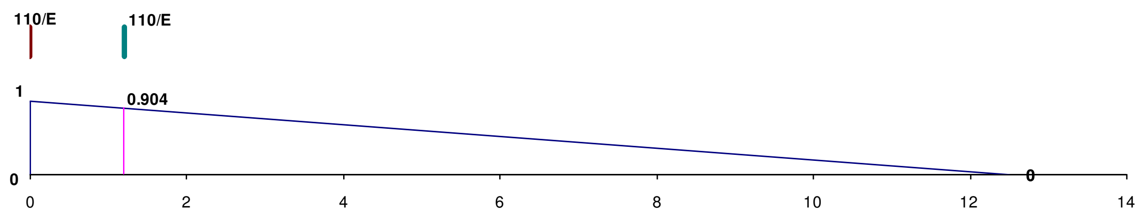
iii) Influence Line for Shear Force (Truck Load)

Max. shear force due to truck load occurs at position where x= 0m, Vtr =73.2 kN/m



iv) Influence Line for Shear Force (Tandem Load)

Max. shear force due to tandem load occurs at position where x =0m, Vtm =61.071 kN/m

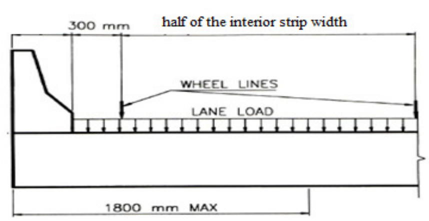


b) Edge strip

Half of the lane width is taken for design (multiple presence factor of 1.2 will be critical) since edge strip width is limitted to 1800mm for one-lane loaded.

V_{LL+IM} =1.2[IM*max(V_{tr},V_{tm})+V_{ln}]/2

M_{LL+IM} =1.2[IM*max(M_{tr},M_{tm})+M_{ln}]/2



Live load placement for shear and moment (edge strip)

4. Dead Load Force Effects

a) Interior Strip

Take 1m strip, unit wt. of concrete = 2400 kg/m3

d=569 mm (=620-32/2-35)

Wdc = 2400*9.81kN/m3/1000*0.62m = 14.597kN/m2 (dead load of the deck slab)

Wdw = 2250*9.81kN/m3/1000*0.1m = 2.207kN/m2 (dead load of the wearing surface)

b) Edge Strip

Dead Load Computations

Assume curb, railings & post loadings spread over edge strip width (1800mm)

$$\text{Curb} = \frac{(0.25*1.2+0.5*0.25*0.05)*(2400*9.81)/10}{1.8} + 14.597 = 18.603 \text{ kN/m2}$$

(wearing surface is not included)

$$\text{Railings} = \frac{0.15*0.3*(2400*9.81)/1000}{1.8} = 0.589 \text{ kN/m2}$$

Number of posts = Int. (clear span of bridge / post spacing)+1 = 6

$$\text{Posts} = \frac{(0.3*0.3*0.85)*6*(2400*9.81)/1000}{1.8*12.5} = 0.48 \text{ kN/m2}$$

Thus, the total edge load becomes, 18.603+0.589+0.48 = 19.672 kN/m2

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 (AASHTO, Table 3.4.1-1)

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

7.1. Investigate Strength Limit State

Design Shear and Moment Computaions

For simple slab bridges, temperature gradient effect reduces gravity load effects. Because temperature gradient may not be there, assume TG=0

Thus, U=1.05(1.25DC+1.50DW+1.75(LL+IM))

a) Interior Strip

i) Shear Force

x	V_{tr}	V_{tm}	V_{ln}	$V_{dl}=V_{dc}+V_{dw}$	V_D	V_{DU}
0	73.200	61.071	16.950	105.025	347.882	219.331

$VD_{max}=1.05(1.75[1.33Max(V_{tr},V_{tm})+V_{ln}]+1.25V_{dl})$ **= 347.882 kN/m**

ii) Moment

x	M_{tr}	M_{tm}	M_{ln}	$M_{dl}=M_{dc}+M_{dw}$	M_D	M_{DU}	ρ	A_s	S
6.25	183.299	181.224	52.969	328.203	973.616	624.959	0.00904	5143.63	150

$MD_{max}=1.05(1.75[1.33Max(M_{tr},M_{tm})+M_{ln}]+1.25M_{dl})$ **= 977.94 kN-m/m**

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

b) Edge Strip

i) Shear Force

V_{LL+IM} =1.2[IM*max(V_{tr},V_{tm})+V_{ln}]/2

x	V_{tr}	V_{tm}	V_{ln}	V_{dc}	V_D	V_{DU}
0	83.680	69.814	19.377	122.950	401.480	253.621

$VD_{max}=1.05(1.75[1.33Max(V_{tr},V_{tm})+V_{ln}]+1.25V_{dl})$ **= 401.48 kN/m**

ii) Moment

M_{LL+IM} =1.2[IM*max(M_{tr},M_{tm})+M_{ln}]/2

d_e= 569mm

x	M_{tr}	M_{tm}	M_{ln}	M_{dc}	M_D	M_{DU}	ρ	A_s	S
6.25	209.542	207.170	60.552	384.219	1127.646	723.462	0.01062	6044.35	130

$MD_{max}=1.05(1.75[1.33Max(M_{tr},M_{tm})+M_{ln}]+1.25M_{dl})$ **= 1132.482 kN-m/m**

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.

$$d_{ic} = \sqrt{\frac{M_{max}}{0.254bf_c'}} \quad =343.85mm$$

→ **dic < d used, The section is sufficed!**

Check for shear

$$Vcc = 0.14\sqrt{f_c'} + \frac{17.2\rho Vu}{Mu} < 0.29\sqrt{f_c'} \quad (=1.53MPa)$$

=0.14√(28)+17.2*0.00908*(219.331/626.38)/1000 = 0.741 ⟶ V_{cc} = 0.741MPa

$d_{iv}=V_{DU}/V_{cc}$= 219.331/0.741 =295.993mm The section is sufficed!

i) Flexural Reinforcement

Interior Strip **(Using 32 mm diameter reinforcing bars)**

As= 5168.53 mm2, → Use Φ 32 c/c 150 mm (As provided= 5358.93 mm2)

Edge Strip

As= 6073.11 mm2, → Use Φ 32 c/c 130 mm (As provided= 6183.38mm2)

ii) Shear Reinforcement, AASHTO Article 5.14.4.1

Slab bridges designed for moment in conformance with Article 4.6.2.3 may be considered satisfactory for shear.

iii) Distribution Reinforcement, AASHTO Article 5.14.4.1

The amount of bottom transverse reinforcement may be taken as a percentage of the main reinforcement required for positive moment and a minimum spacing of 250mm.

P_e= min[50,1750/√(L_1)]

P_e = Percentage of distribution reinforcement

Thus P_e= min [50, 1750/sqrt (12500)] = 0.157

a) Interior strip

A_{ti}= P_eA_{sti} = 809mm2 **(Using 16 mm diameter reinforcing bars)**

S_{di} =min(a_{si}*1000/A_{ti},250] = Min (240,250)

As= 809 mm2, S= 240 mm → **Use Φ 16 c/c 240 mm**

b) Edge strip

$A_{te}= P_eA_{ste}$ = 950.59mm2

S_{de} =min(a_{si}*1000/A_{te},250] = Min(210,250)

As= 950.59 mm2, S= 210 mm → **Use Φ 16 c/c 210 mm**

iv) Shrinkage and Temperature Reinforcement, AASHTO section 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*620/400=1162.5 mm2/m (in each direction, both faces)

Top layer A_{st} = 1/2*1162.5= 581.25 mm2/m

Spacing =min(a_{si}*1000/A_{st},250) = Min (190,250)

Use Φ12 mm rebars c/c 190mm, transverse (in each direction, both faces)

7.2. 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 35mm).
A 35mm concrete cover is provided here, thus there is no problem of durability.

ii) Adequacy of Reinforcement Bars

a) Moment Interior Strip

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

MDU= 626.38kN-m/m

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

$$A_S = \frac{626.38E+06 \text{ kN-m/m}}{(0.6*400)*0.875*569} = 5242.11\text{mm2/m}$$

(provide diam. 32 c/c 150mm)

b) Moment Edge Strip

MDU= 724.996kN-m/m

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

$$A_S = \frac{724.996E+06 \text{ kN-m/m}}{(0.6*400)*0.875*569} = 6067.42\text{mm2/m}$$

OK!

iii) 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} .

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

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

Z=crack width parameter=23,000N/mm

where:

d_c= concrete cover + (diam. of bars/2), measured from the extreme tension fiber

clear cover to compute $d_c \le 50$mm

$A_c = 2d_cS$

$f_r = 0.63\sqrt{(f_c')}$

$$f_{cten} = \frac{6M_{us}}{bD^2}$$; D is in mm.

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_{cten}= tensile strength of the concrete

If $f_{cten} > 0.8f_r$, the section has cracked (AASHTO, Article 5.7.3.4 and 5.4.2

MDu=626.38kN-m/m

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

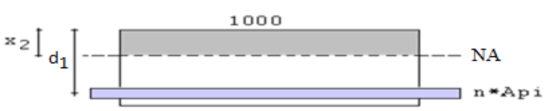
$$f_{cten} = \frac{6*626.38E+6 \text{ N-mm}}{1000*620\text{^}2} = 9.78 \text{ MPa}$$

⟶ **Section has cracked**

Steel stress for elastic cracked section for which moment of inertia of the composite transformed section, I_{cr}, should be used.

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

a) Interior Strip



Moment about the NA: $\frac{bx_2^{\ 2}}{2} = nA_{pi}(d_1 - x_2)$

$$I_{cr} = \frac{bx_2^{\ 3}}{3} + nA_{pi}(d_1 - x_2)^2$$

$$f_s = \frac{M_{Du}(d_1 - x_2)n}{I_{cr}}$$

$$f_{sa} = \frac{Z}{(2(D - d_1)^2 S_i)^{\frac{1}{3}}}$$

where:

x_2 is the neutral axis depth from top fiber

M_{Du}= unfactored max moment, interior strip.

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

The equivalent concrete area, nA_{pi} = 37512.510 mm^2

Api =5358.93mm2/m, Si=150mm d1=569mm, n=Es/Ec, MDu=626.38kNm/m

Substitution yields:

x2=172.479 mm ,Icr= 7.608 E+09 mm4, fs= 228.51 MPa, fsa= 240 MPa

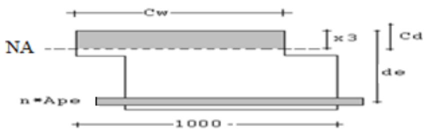
The above results show that :

fs (= 228.51 MPa) <fsa (=240 Mpa)

⟶ **No problem of cracking**

Provide 32 mm rebars spaced at 150 mm

b) Edge Strip



$$Moment\ about\ the\ NA:\ \frac{C_w x_3{}^2}{2} = nA_{pe}\left(d_e - x_3\right)$$

$$I_{cre} = \frac{1000\left(C_w - 0.05\right)x_3{}^3}{3} + n\,A_{pe}\left(d_e - x_3\right)^2$$

$$f_{se} = \frac{M_{Dse}\left(d_e - x_3\right)n}{I_{cre}}$$

$$f_{sae} = \frac{Z}{\left(2(D - d_e)^2 S_e\right)^{\frac{1}{3}}}$$

The equivalent concrete area is nApe= 43283.660 mm^2

Ape =6183.38mm2/m, Se=130mm de=569mm, n=Es/Ec, MDue=724.996kN-m/m

Substitution yields:

x3= 153.734 mm, Icre=8.372E+09mm4, = 251.71 MPa, fsa =240 MPa

fs (= 251.71 MPa,) > fsa (=240 MPa)

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

iii) Deformations

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 I_g.

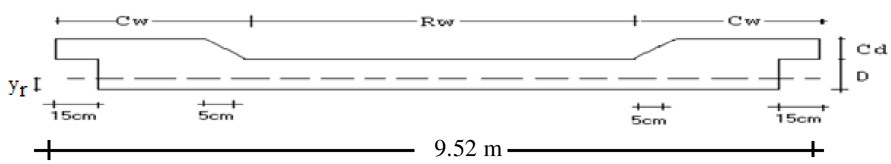
Unless a more exact deformation calculation is made, the long-term deflection due to creep and shrinkage may be taken as the immediate deflection multiplied by the following factor (AASHTO, Article 5.7.3.6.2).

4, if the instantaneous deflection is based on I_g.

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

Dead load camber

Total dead load of the bridge and the whole bridge cross section is considered.



C_w = 1.25 m
D = 0.62 m
R_w = 7.32 m
C_d = 0.25 m

Dead Loads of the whole bridge

Components	Weight	Width (m)		Total wt.
Wt. of slab	14.60	(R_w+2C_w-0.3)	9.52	138.963
Wearings wt.	2.21	R_w	7.32	16.155
Curb wt.	4.006	2C_w	2.5	10.015
Railing wt.	0.589	2E_e	3.6	2.120
Posts wt.	0.480	12 posts	12	5.76
			W_{dd} =	173.014

W_{dd} = 173.014 kN/m

The total, maximum and critical moments due to dead and live load of the whole slab bridge, respectively are given by the equations:

$M_{lld}=M_{md}+max(M_{trl},M_{tml})*E_{m1}*NL$; E_{m1} (interior strip width) is in m and NL is the number of design lanes.

$M_{md}= W_{dd}*L_1^2/8 =$ 173.014*12.5^2/8 =3379.18kN-m

$M_{lld}=$ 3379.18+1.33*187.23*3.4295*2=5087.18kN-m

$$y_r = \frac{(R_w+2C_w-0.3)D^2/2+2C_w*C_d(C_d/2+D)}{(R_w+2C_w-0.3)*D+(C_w*2C_d)}$$

$$y_r = \frac{9.52*0.62^2/2 + 1.25*0.25*2*(0.25/2+0.62)}{9.52*0.62+(1.25*0.25*2)} = 0.352m \quad (yr= 352mm)$$

Thus, the gross moment of inertia of the whole slab bridge becomes

Ig = 9.52*0.62^3/12 +(9.52*0.62*(0.62/2-0.352)^2)+2*1.25*0.25^3/12+(2*1.25*0.25*(0.745-0.352)^2)

$I_g=$ 2.99E+11 mm4

The critical moment of inertia is

$I_{cr} =(Icr)_{interior} *(W-2E_e)+2(I_{cr})_{edge}$ = (7.608E+9)*6.22 + 2*8.372E+9

$I_{cr} =$ 6.41E+10 mm^4

Where: W is the total width of the bridge

Ee is the width of the edge strip

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

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

Where:

M_{lld}= total moment due to dead and live load of the whole slab bridge

M_{md}= total dead load moment of the whole slab bridge

M_{cr}= critical moment of the whole slab bridge

$$M_{cr} = f_r \frac{I_g}{y_r} = 3.33*0.299E+12/352 = 2828.61kN\text{-}m$$

I_e= 1.04E+11 mm^4

Mcr < Mmd ⟶ **The Secton cracks under DL, Use Ie.**

$E_C = 0.043\gamma_c^{1.5}\sqrt{f_c'}$, Ec = 26.752 GPa (Modulus Elasticity of concrete)

γ_c : unit density of concrete (kg/m^3); refer to Table 3.5.1-1 or Article C5.4.2.4

f_c ' : Specified compressive strength of concrete (MPa)

Thus, the dead load deflection of the slab bridge is given by:

$$d_{dl} = \frac{5W_{dl}L_1^4}{384Ec*\ Ic} = 19.682\ mm$$

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

Camber $=d_{dl}(1+3)$= 78.728 mm Thus, provide a camber of 78.728 mm

b) Live Load Deflection (AASHTO, Article 2.5.2.6.2)

For vehicular load in general, $d_{max}=L_1/800$ = 12500/800 =15.625mm

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 (AASHTO, Article 3.6.1.3.2)

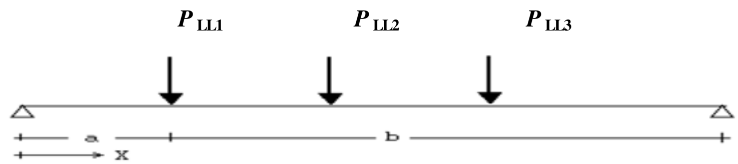
i) Deflection due to truck load

$P_{LL1}=P_{LL2}=1.33*145NL =$ 385.7kN

$P_{LL3}=1.33*35NL =$ 93.1kN

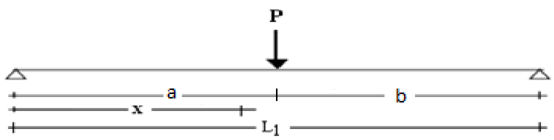
where:

P_{LL1} and P_{LL2} are truck loads.



At x1=6.875 m, Mtr =187.233 kN-m/m (Location of maximum truck load effect)

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



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

where:

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

a= location of the load to be considered, $b=L_1$-a and $x=L_1-X_1$

(EI)conc=26752*0.1E+12= 2.79E+15 (Nmm^2)

a) First Load, P_{LL1}

a=2.575m, b=9.925m, x=6.875m

d_{tr1}= 0.00314m= 3.14mm

b) Second Load, P_{LL2}

a=6.875m, b=5.625m, x=6.875m

d_{tr2}= 0.0055m= 5.5mm

c) Third Load, P_{LL3}

a=11.175m, b=1.325m, x=6.875m

dtr3= 0.00043m= 0.43mm

Using the method of superposition, the total live load deflection due truck load is the sum of each deflections, d_{ki}'s.
Thus, compare the value obtained with the permissible limit.

$d_{tr}=d_{tr1}+d_{tr2}+d_{tr3}$ = 3.14+ 5.5+0.43 = 9.07mm

ii) Deflection due to tandem load

The maximum deflection due to tandem load occurrs when a single concentrated tandem load is acting at the mid span.

$d_{tl}=P_{stl}*L_1^3/48E_cI_c$ (where P_{stl} is a concentrated load at the mid span)

where:

d_{tl} = deflection due to tandem load.

P_{stl} = Single concentrated factored tandem load = 1.33*110*2NL = 585.2kN

d_{tl}= 585.2*12.5^3/(48*2794377.51) = 0.00852m = 8.52 mm

iii) Deflection due to lane load

WL_1=9.3*NL

d_{La} = $5WL_1*L_1^4/(384EI_e)$ = 5*(9.3*2)*12.5^4/(384*2794377.51)

= 0.00212m =2.12 mm

d_{ll}=max(d_{tr}, (d_{la}+0.25d_{tr})) =9.07 mm

d_{max}= max(d_{ll},d_{tl}) =9.07 mm

Where:

d_{la} = deflection due to lane load.

WL_1= lane load

d_{ll} = total live load deflection

d_{max}= 9.07mm (< 15.625mm) **The deflection is within the limit.**

7.3 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.

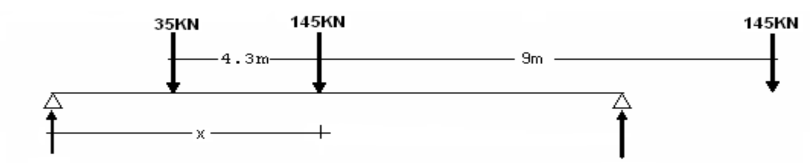
U=0.75(LL+IM) ; F.S for LL is 0.75

where:

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= 6.5625m and equals to 489.606 kNm

M_{mf}= 0.75*1.15(M_{max}) = 422.285kNm/lane

M_{mf}= the maximum moment for fatigue.

a) Tensile live load stresses

One lane loaded, Strip width is E_{m1} (for interior strip)

M_{llf}= M_{mf}/E_{m1} (E_{m1} is in meter)

= 422.285 /4.705 =89.75kNm/m

$$f_{s,max} = n\frac{M_{llf}(D-x_2)}{I_{cr}}$$ =7*89.75E+6*(620-172.479)/(7.61E+9) mm4 =36.953 MPa

f_{smax}= 36.953MPa (< 240MPa) **OK!**

where:

M_{llf} is the maximum moment per meter width for fatigue.

b) Reinforcing Bars

The stresses range in straight reinforcement bars resulting from fatigue load combination shall not exceed (AASHTO, Section 5.5

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 > 36.953MPa OK! ⟶ **No problem of fatigue.**

THE DESIGN IS COMPLETED!

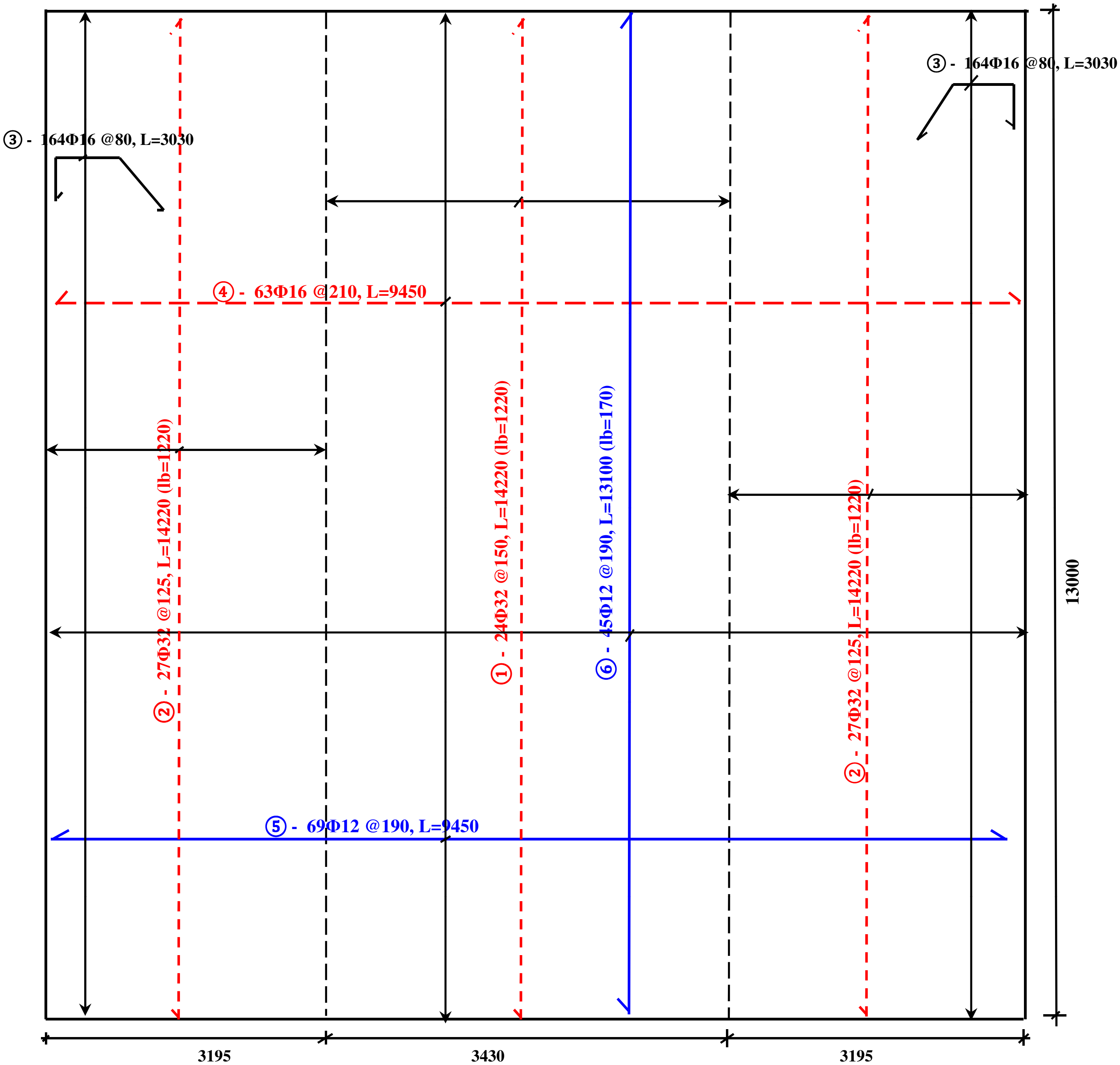


Fig 1. Bridge Plan

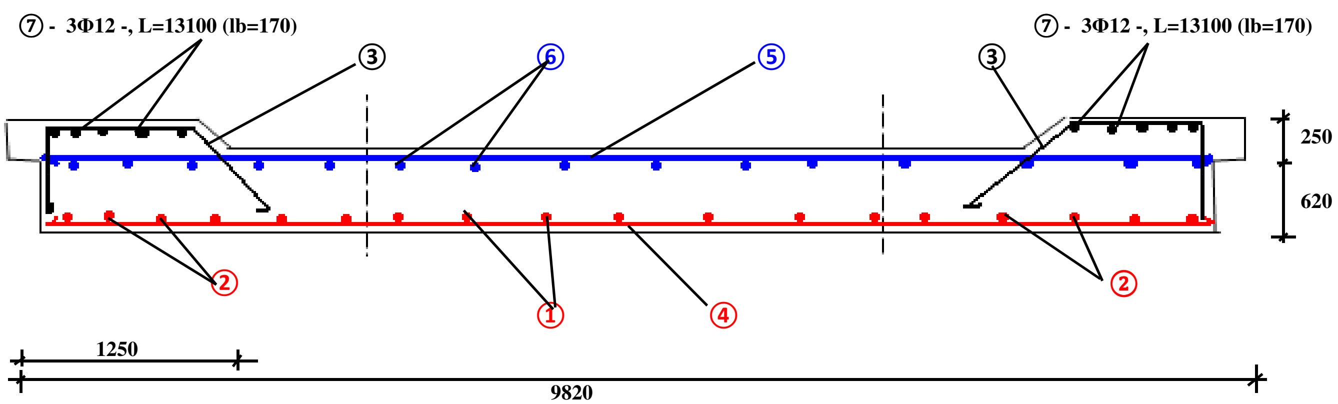


Fig 2. Bridge Cross Section

Design of Curbs

According to AASHTO Bridge Design Manual (Art. 2.7), curbs will be designed to resist a lateral force of 7.3 kN/m applied at the top of the curb or the railing load of 44.5kN whichever produces larger force effect.

$M_1 = 7.3C_d = 1.825$ kN-m/m

$M_2 = 44.5x/E$ $E = 0.833P_h + 1.143$

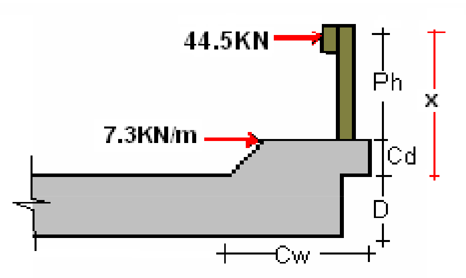
$M_2 = 26.45$kN-m/m $E = 1.851$ m

M2 will be taken for design.

⟶ $M_u = 1.3\max(M_1, M_2)$

Design Moment, M_u

⟶ $M_u = 34.385$ kN-m/m



b=1000mm

$d = C_w - 0.15 - cover - \Phi/2$ ⟶ d =1067mm

$\rho = 0.00008$ $\rho_{min} = 0.0021$ **(Using 16 mm diameter reinforcing bars)**

$As = \rho_{min} bd$

As= 2240.7 mm2, **Use Φ 16 c/c 80 mm**

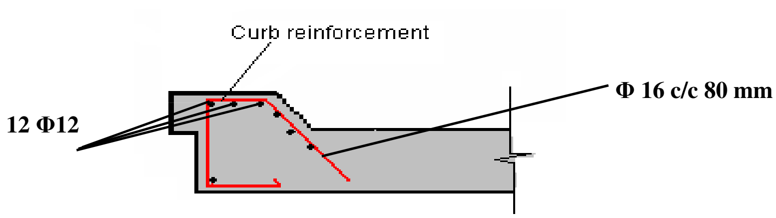
Temperature Reinforcement

$$A_{st} = \frac{0.75A_g}{f_y}$$

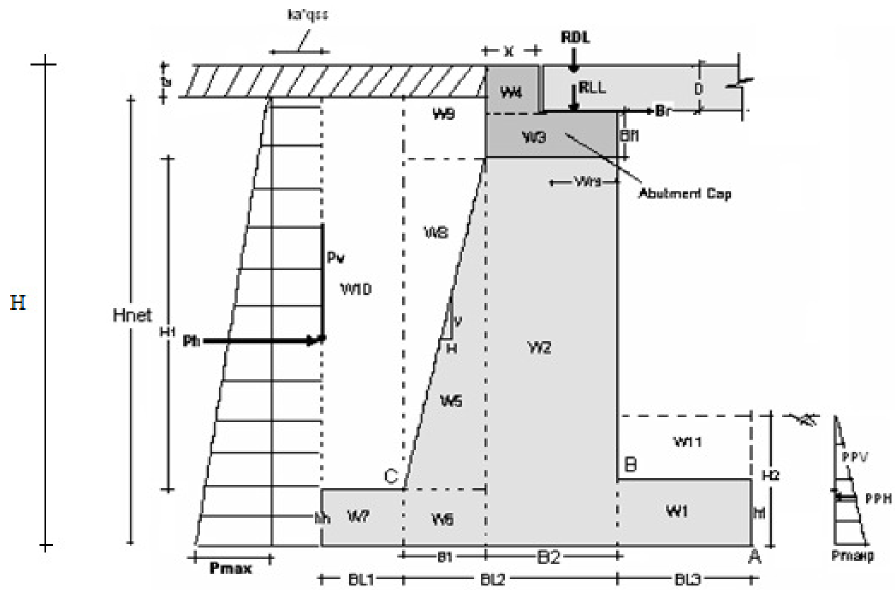
$A_{st} = 0.75Cw(D+Cd)/400 = 2039.0625$ mm2

Spacing =min(a_{st}*1000/Ast, 250) **= min(110, 250)**

⟶ **Use Φ12 mm rebars c/c 110mm**



ABUTMENT DESIGN



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 = 3.25 m

slope V:H = 2

Top width of the abutment cap, x = 0.20 m

Depth of the abutment cap, B_{f1}= 0.25 m

Depth of the base concrete, h_t= 0.50 m

Depth of the base concrete, h_h= 0.50 m

Top height of the abutment, H= 5.00 m

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

Friction angle of the backfill material, δ = 18 °

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 = 26 kN/m^3 (Stone masonry)

Unit weight of the abutment cap,γ_{ac} = 26 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)= 4.78m

H_1= 3.63m

B1=H1/Slope=3.63/2=1.82m

BL2=B1+Wrs+x = (1.82+0.5+0.2)=2.52 m

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

B=BL1+BL2+BL3 = 1.75+2.52+3.25 = 7.52m

2 LOADS

i) Dead Load Reaction

$Rdl=(W_{dc}+W_{dw})L_1/2$

$=173.014*12.5/2 =1081.338$ kN

ii) Live Load Reactions

$R_{lli}=2E_e*(max(V_{trel},V_{tmel})+V_{ln})+(R_{wt}-2E_e/1000)*(max(V_{trl},V_{tml})+V_{ln})$

$=2*1.8*(83.68+19.377)+(9.82-2*1.8)*(73.2+16.95)=931.738$N/m

iii) Wind Load on Live Loads

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

WL = 1.46*(5+1.8)=9.928kN

iv) Breaking Force, Brf

Brf=0.25*(2*P2+P1)*NL ,Braking force,25%, 1.8m above roadway surface.

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

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

CR_SH_TU = 108.134 kN

vi) Lateral active earth pressure

$q_{ult} = 1.5q_{all} =$ 375kPa

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

$\Theta=180-\tan^{-1}(H1/B1) =$ 116.65 deg.

$\eta=(1+Sqrt(Sin(\delta+\Theta))*Sin\phi)/(Sin(\delta+\Theta)*Sin\Theta)^2 =$ 2.426

$K_a= (Sin(\phi+\Theta))^2/(\eta*Sin^2\Theta*(Sin(\Theta-\delta))) =$ 0.276

$\gamma= 180-(\delta+\Theta)=$ 45.35deg.

$P_{max}=ka*\gamma_{bm}H_{net} -2C\sqrt{Ka}$

$= 0.276*18.9*4.78-(2*0*\sqrt{0.276}) =24.934$ kN/m2

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

$= 0.5*24.934*4.78=59.592$ kN/m

Pah=Pa Sinγ=42.408 kN/m

Pav=Pa Cosγ=41.866 kN/m

vii) Lateral passive earth pressure

Kp = 1/Ka =	3.62	
$P_{maxp}=kp*\gamma_{bm}H_{net} +2C\sqrt{Kp} =$	136.8	kN/m^2
$P_p=0.5* P_{maxp}H_2 =$	136.8	kN/m
Pph=Pp Sinγ=	97.38	kN/m
Ppv=Pp Cosγ=	96.13	kN/m

viii) Dead load Surcharge

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

=18.9*0.22 =4.158Kpa

$Pressure=k_a*q_{ss}=$	1.15	kN/m
$Qsh=k_a*q_{ss} Sin(\gamma)=$	0.818	kN/m
$Qsv=k_a*q_{ss} Cos(\gamma)=$	0.808	kN/m

xi) Live load Surcharge

$Pressure=k_a*h_{eq}*\gamma_{bm}=$	4.543	kN/m
Qlh=Pressure*Sin(γ)=	3.065	kN/m
Qlv=Pressure*Cos(γ)=	3.353	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 = 9.82m

		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	414.90	518.62	373.41	1.63	674.20	842.76	606.78
	W2	738.13	922.66	664.32	3.60	2657.27	3321.59	2391.54
	W3	44.68	55.85	40.21	3.60	160.85	201.06	144.77
	W4	31.66	39.57	28.49	3.85	121.89	152.36	109.70
	W5	727.23	909.03	654.51	4.56	3313.74	4142.17	2982.36
	W6	232.34	290.43	209.11	4.86	1129.18	1411.47	1016.26
	W7	223.41	279.26	201.06	6.35	1419.37	1774.21	1277.43
	W8	528.64	660.80	475.77	5.16	2729.54	3411.92	2456.58
	W9	219.56	274.45	197.61	4.86	1067.07	1333.84	960.37
	W10	1552.53	1940.66	1397.27	6.35	9863.72	12329.65	8877.35
	W11	904.79	1130.99	814.31	1.63	1470.28	1837.86	1323.26
	RDL	1081.34	1351.67	973.20	3.50	3784.68	4730.85	3406.21
	RLL	931.74	1630.54	1630.54	3.50	3261.08	5706.90	5706.90
	Pav	200.12	270.16	200.12	7.52	1504.90	2031.61	1504.90
	Ppv	192.27	259.56	192.27	0.00	0.00	0.00	0.00
	Qsv	37.92	51.20	37.92	7.52	285.19	385.00	285.19
	Qlv	32.93	44.46	32.93	7.52	247.64	334.32	247.64
	TOTAL	**8,094.18**	**10,629.91**	**8,123.06**		**33,690.61**	**43,947.57**	**33,297.24**
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	108.13	54.07	54.07	4.78	516.88	258.44	258.44
	Pah	202.71	304.07	182.44	1.59	322.98	484.48	290.69
	Pph	194.75	292.13	175.28	-0.67	-129.84	-194.75	-116.85
	Qsh	38.41	57.62	34.57	2.39	91.81	137.72	82.63
	Qlh	30.10	45.14	27.09	2.39	71.93	107.90	64.74
	TOTAL	**746.54**	**1,037.40**	**757.82**		**2,008.35**	**2,664.96**	**2,450.83**
						∑M	**41,282.61**	**30,846.41**

Stability and Safety Criteria

1 Eccentricity

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

	V	∑M	Xo=∑M/V	$e=B/2-X_o$	$e_{max}=B/4$	Design Margin (%)	Remark
Strength I	10,629.915	41,282.61	3.884	-0.124	1.880	93.42	**OK!**
Strength Ia	8,123.061	30,846.41	3.797	-0.037	1.880	98.01	**OK!**

2 Sliding

$\% = (\varphi_s * F_r - H)*100/\varphi_s * F_r$ $\varphi_s = 0.8$

	V	tanδ	Fr=V*tanδ	φ_s *Fr	HL	Design Margin (%)	Remark
Strength I	10,629.915	0.33	3455.351	2764.281	1037.405	62.47	**OK!**
Strength Ia	8,123.061	0.33	2640.475	2112.380	757.820	64.12	**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=500kPa

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

	Hn	Vn	Hn/Vn	$\varphi_b * RI * q_{ult}$	q_{max}	Design Margin (%)	Remark
Strength I	1,037.40	10,629.91	0.098	183.716	181.989	0.94	**OK!**
Strength Ia	757.820	8,123.06	0.093	186.355	142.229	23.68	**OK!**