Heaven's Light is Our Guide

Rajshahi University of Engineering & Technology
Rajshahi

DEPARTMENT OF CIVIL ENGINEERING

Expt. No. 01

Name of Expt Introduction to Bridges

SUBJECT : Reinforced Concrete Sessional- II	**SUBMITTED BY :**
COURSE NO. : CE - 3218	**NAME :** Abu-Shad
DATE OF EXPT.:	**CLASS :** 3rd year even semester
DATE OF SUB. :	**GROUP :** .... **ROLL NO** 1800144
	SESSION : 2018-19

Chapter No : 01

Chapter Name: Introduction to Bridges.

Definition of Bridge:

A bridge is a structure built to span a physical structure obstacle (such as a body of water, valley, road or rail) without bloking the way underneath.

It is constructed for the purpose of providing passage over the obstacle, which is usually something that is otherwise difficult or impossible to cross.

There are many different designs of bridges, each serving a particular purpose and applicable to different situation.

Designs of bridges vary depending on factors such as:

1/ The function of the bridge.

2/ The nature of the terrain where the bridge is constructed & anchored &

3/ The material used to make it, &

4/ The funds available to build it.

Parts of a bridge:

There are different types of bridge. Different bridge types contain different parts. Following are the main parts of a bridge:

a) Guard rail.

b) Deck.

c) Abutment.

d) Pile.

e) Pier.

f) Girder.

g) Rail Track.

Guard Rail:

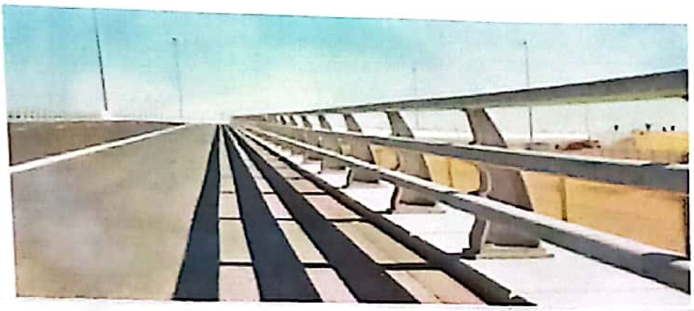


Fig: Guard Rail of a bridge

Curb:

Fig : Curb of a bridge

Abutment :

The support provided at the two ends of a bridge is known as an abutment.

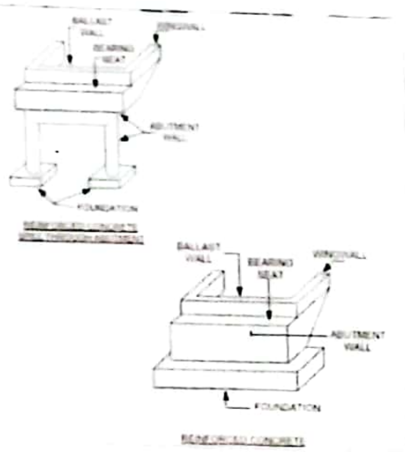
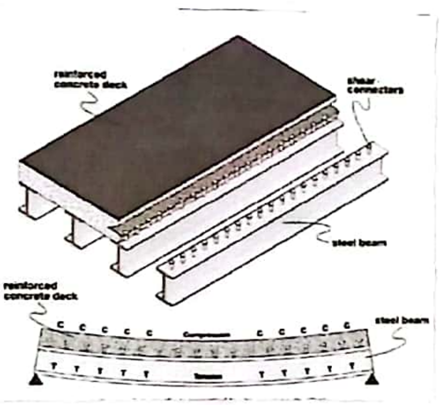
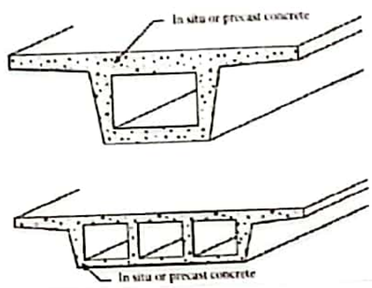


Fig : Abutment of bridges

Deck:

A deck is a fundamental part of any bridge to pass vehicle, goods, people etc, from one side to another.





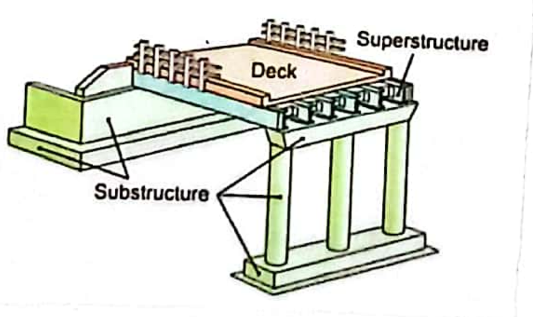
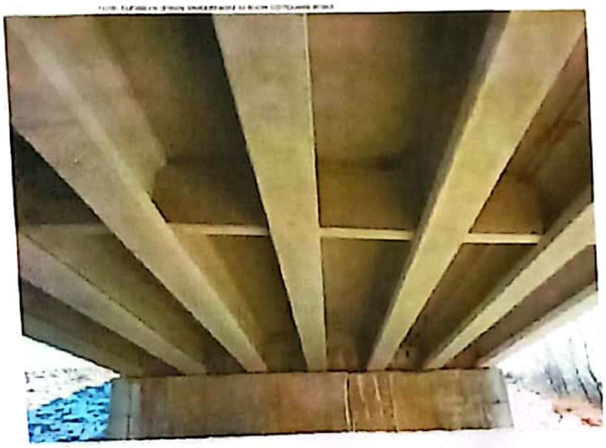


Fig : Deck of a bridge

Girder:

Just like the beam, girdere is used in the bridge. It can be two types I - joist and Box. This name has been given because of theire shape. I- joist girder type is commonly used in bridges. Box girder can be precast or cast in place and it is generally existing in prestressed condition.



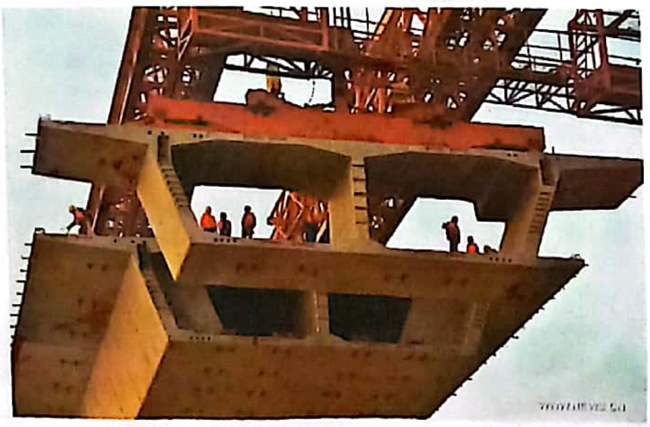


Fig : Girder of bridges.

Pier:

Pier is the compression member which stay above the pile and make the structure stable. Pier generally provides for span at intermediate points.

Piers perform two main function:

i) Transferring superstructure vertical loads to the foundation.

ii) Resisting to the horizontal forces acting on the bridges.

For bridge pier to pier, distance is the span. Water pressure is the extra pressure which acts onto the pier laterally.

Fig : Pier of Padma bridge.

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Pile :

For the bridge with pier, the pile is a fundamental component. Pile type foundation is generally needed when the upper soil layer is loose. Pile depth depends on the soil layer. To the find the hard soil layer which will make the structure stable, the pile is usually extended to some depth into the hard soil layer.

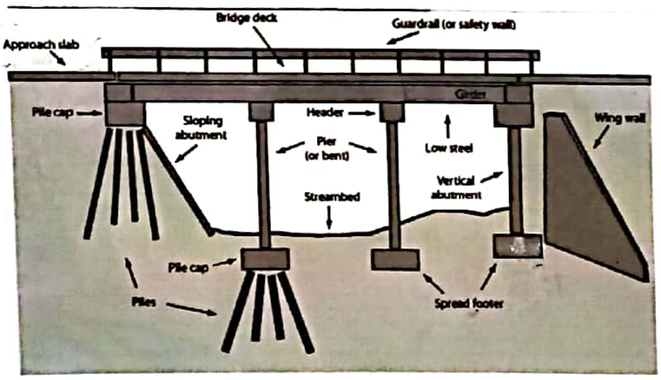


Fig : Pile of a bridge

Rail Track:

Normally Road traffic is the main vehicle onto the bridge but if the train needs to be passed through that bridge rail track is the extra component.

Fig : Rail track of Padma bridge

Types of Bridges:

Following are the main types of bridges,

1) Truss Bridge.
2) Arch Bridge.
3) Suspension Bridge.
4) Cable - stayed Bridge.
5) Slab Bridges.
6) Box Girder Bridge.
7) Fixed Bridges.
8) Movable Bridges.

Truss Bridge :

Bridges with truss are made by steel force member with only tension & compression. No bending moment is allowed in this structure. Most stable structural shape for truss is triangular.

Fig : Hardinge Bridge

Arch Bridges :

Arch bridge mainly exists in compression. Utilizes an areodynamic system with torsional rigidity.

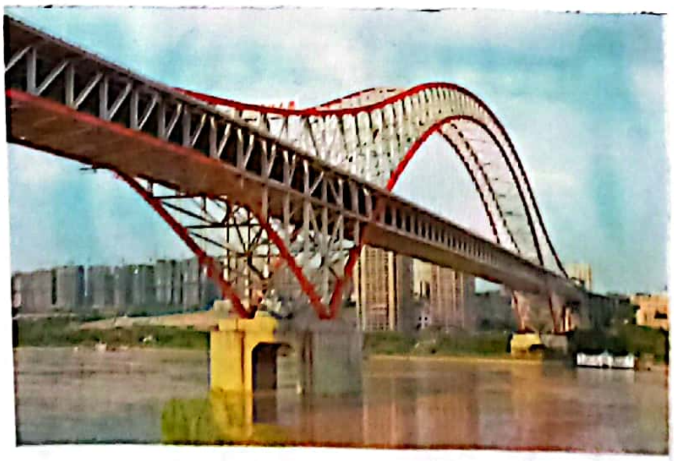


Fig : Arch bridge

Suspension Bridges:

In suspension bridges long spans can be provided which is essential in many situations. It gives freedom to the engineers to provide a long span with the help of a cable.

Following are the basic components of a suspension bridge structural system:

Stiffening Girder / Trusses: Longitudinal structures support and distribute moving vehicle loads. Secure aerodynamic stability of the structure.

Main Cables: Main cables are connected to girders through hangers rope. These hangers ropes transfers the loads from girders to the main cables. The main function of these main cables to carry these loads to the main towers.

Main Towers: Main cables are supported by these intermediate vertical structures and transfers the total load of the bridge to the foundation.

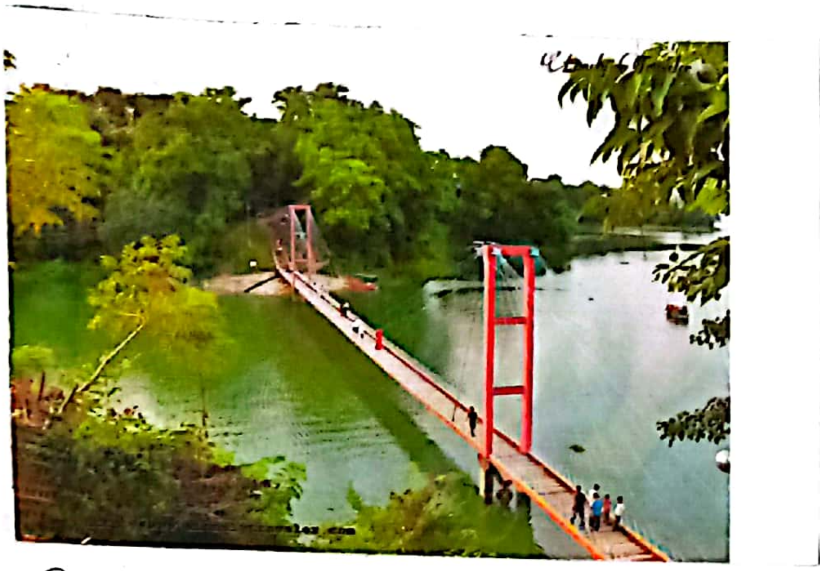


Figure : Hanging Bridge - Rangamati

Cable - stayed Bridges:

It has a lot of similarities to the suspension bridge. But there are few differences between a suspension bridge & cable stayed bridge. In this case, bridge mainly carries the vertical loads acting on the girder. The purpose of the stay cables is to provide intermediate support for the girders and it helps to span a long distance.

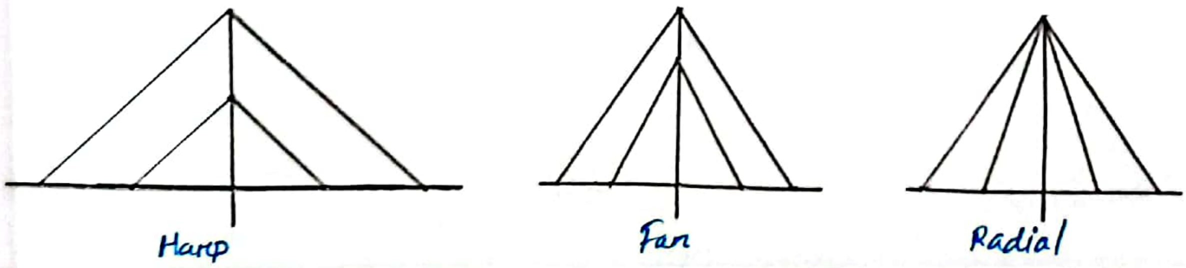


Figure: Shah Amanat Bridge - Chattogram.

Slab Bridges:

It is the most common type of bridges. Use in every place where the span is not so long.

Box Girder Bridges:

It's a box type girder different from normal I girder and it can easily resist more amount of torsion. This type of bridge contains top deck, vertical web, bottom slab. Box girder bridge can be subdivided into three basic categories:

1/ Single cell box.

2/ Multicell box.

3/ Box with struts supporting a cantilever deck.

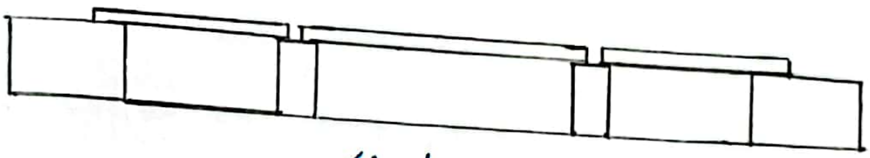
Fixed Bridges:

Majority of bridges are fixed, with no moveable parts to provide higher clearance for river/sea transport that is following below them. They are designed to stay where they are deemed unusble or demolished.

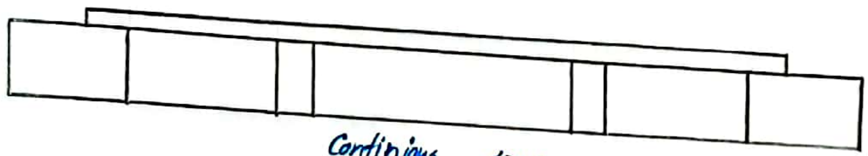
Movable Bridges:

A movable bridge is a bridge that has dynamic moving parts used to change the form of the bridge usually to allow passage for boats. There are many types of movable bridges, and they differ in the way they transform.

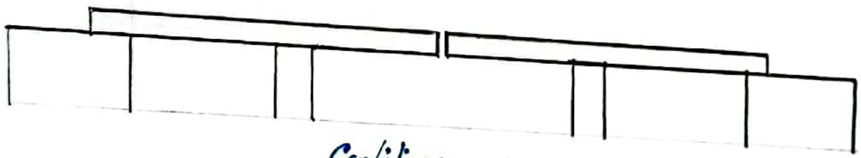
Span Type:



Simple span



Continious span



Cantilivere span

Site selection fore bridge:

The characteristics of an ideal site fore a bridge across a rivere are

a) A straight reach of the river.
b) Steady rivere flow without cross currents.
c) A narrow channel with firm banks.
d) Suitable high banks above high flood level on each side.
e) Rock or other hard in erodible strata close to the river bed level.
f) Absence of sharp curves in the approaches.
g) Absence of expensive rivere training works.
h) Avoidance of excessive underwatere construction.

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DEPARTMENT OF CIVIL ENGINEERING

Expt. No. 02

Name of Expt Design of Slab Bridge

SUBJECT : Reinforced Concrete Sessional-II	**SUBMITTED BY :**
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DATE OF EXPT.:	**CLASS :** 3rd year even semester
DATE OF SUB. :	**GROUP :** .... **ROLL NO** 1800144
	SESSION : 2018-19

Chapter No: 02

Name of the chapter: Design of Slab Bridge. (Design of railing & rail post)

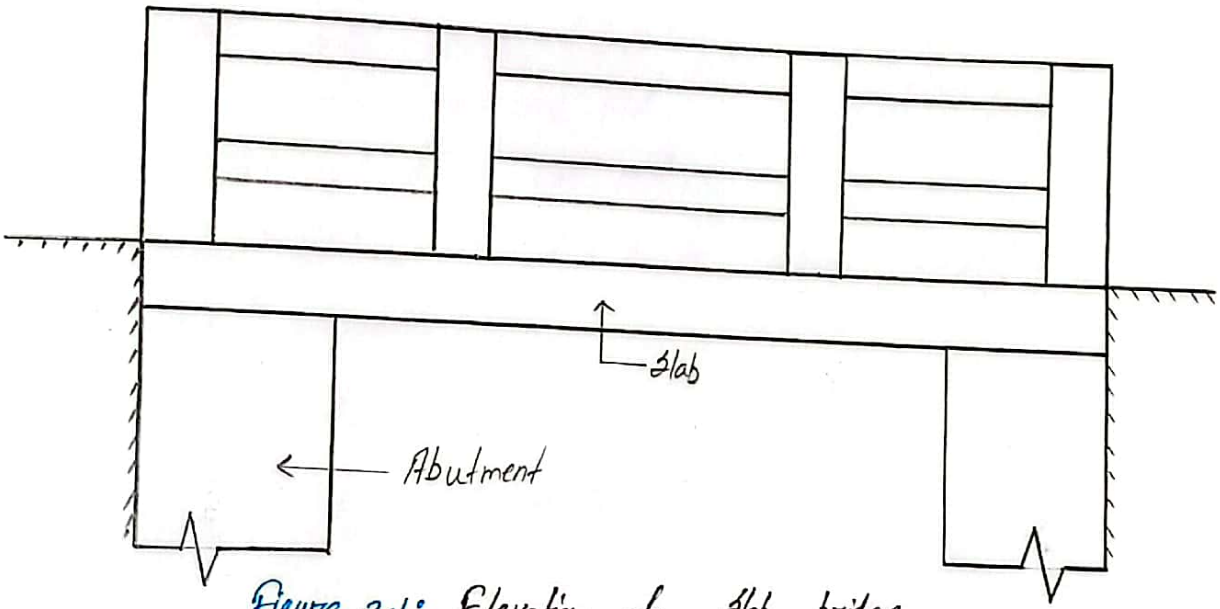


Figure 2.1: Elevation of slab bridge

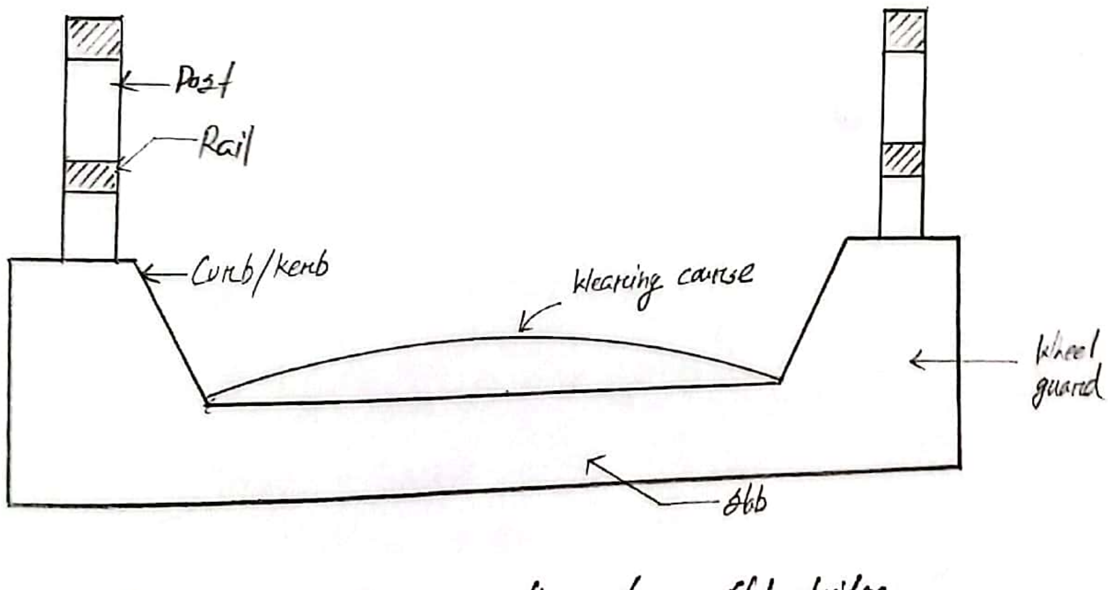


Figure: 2.2: Cross-section of slab bridge.

Design specification:

1) Length of clear span $= 8' + \frac{\text{last 2 digit of roll}}{4}$

$= 8' + \frac{44}{4} = 19'$

2) Width of the span = 30'
3) Horizontal load = 200 plf
4) Vertical live load = 150 plf
5) Horizontal thrust due to wheel = 500plf
6) Material properties :-

$f_c' = 3500$ psi

$f_y = 60000$ psi.

7) Loading system = H20S16 Truck load.

Design of Rail & Post:

Relevent properties :

1) $n = E_s/E_c = \frac{29 \times 10^6}{57000\sqrt{3500}} = 8.6 \approx 9$
2) $r = f_s/f_c = \frac{0.4 \times 60,000}{0.45 \times 3500} = 15.24$
3) $k = \frac{n}{n+r} = \frac{9}{9+15.24} = 0.37$
4) $j = 1 - k/3 = 1 - \frac{0.37}{3} = 0.88$
5) $R = \frac{1}{2} f_c jk = \frac{1}{2} \times 3500 \times 0.45 \times 0.88 \times 0.37 = 256.41$

Railing Post:

(i) Let the spacing of rail post, $s = 6.5'$

(ii) Number of rail post $= \frac{19'}{6.5'} + 1 = 3.92 \approx 4$

(iii) Actual spacing of rain post $s_a = \frac{19}{4-1} = 6.34'$

Assume the section of railing $= 6'' \times 6''$

Load calculation

(1) Self weight $= \frac{6'' \times 6''}{12'' \times 12''} \times 150 = 37.5$ plf

(2) Vertical load $= 150$ plf

$\therefore$ Total load $w_v = 187.5$ plf

So, vertical moment, $M_v = \frac{w_v \times s_a^2}{8}$

$= \frac{187.5 \times 6.3^2}{8}$

$= 942.08$ lb-ft

Depth check:

We know, $d = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{942.08 \times 12}{235.41 \times 6}}$

$= 2.71''$

$D_{effective} = t - LL - \phi/2 = 6 - 2 - 0.5 = 3.5$

$\therefore d_{req} < d_{eff}$ $\therefore$ OK design

Steel calculation:

Steel area, $A_s = \frac{M}{f_s j d}$

$= \frac{942.08 \times 12}{24000 \times 0.88 \times 3.5}$

$= 0.15 \, in^2$

Minimum reinforcement required

$A_{s(min)} = \frac{3\sqrt{f_c'}}{f_y} bd = \frac{3 \times \sqrt{3500} \times 12 \times 3.5}{60{,}000}$

$= 0.124 \, in^2$

$A_{s(min)} = \frac{200}{f_y} bd = \frac{200 \times 12 \times 3.5}{60{,}000}$

$= 0.14 \, in^2$

$\therefore$ Steel required $= 0.15 \, in^2$

So we will provide 4#5 bars

Working Diagram :

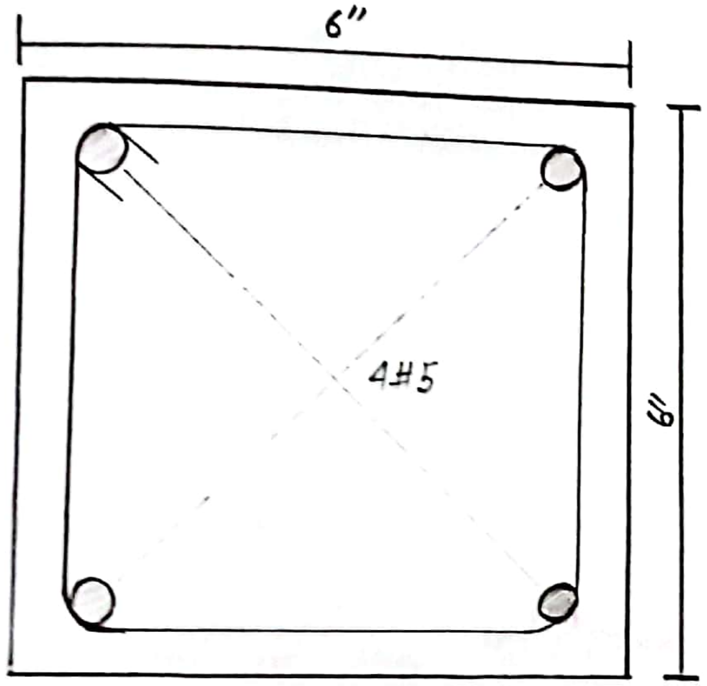
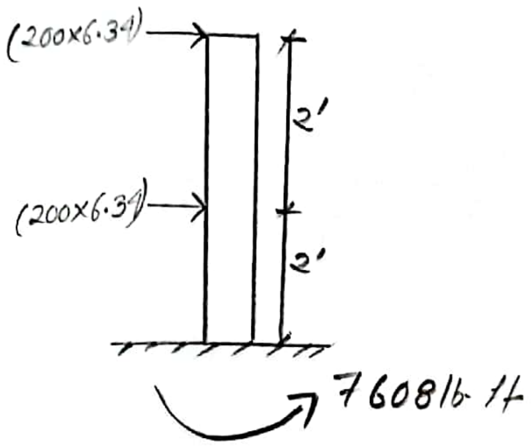


Figure 3 : Cross section of railing

Design of post:



Assume, the section of railpost = 8"×8"

Height of the rail post = 4'

Maximum Horizontal moment at base, $M_H = (200\times6.34\times4) + (200\times6.34\times2)$
$= 7608$ lb-ft

Depth check:

Moment, $M_H = 7608$ lb-ft

Now, $d_{req} = \sqrt{\frac{7608\times12}{256.41\times8}}$
$= 6.66''$

So $t_{eff} = 6.66 + 1.5 = 8.16''$

Our assumed section $t = 8''$

So it is relatively closed $t_{eff} \approx t$

So our assumed section for rail post $= (8'' \times 8'')$ is ok.

Steel calculation:

$$A_s = \frac{M}{f_s j d} = \frac{7608 \times 12}{24000 \times 0.88 \times 6.5} = 0.66 \text{ in}^2$$

Minimum reinforcement required,

$$A_{s(min)} = \frac{3\sqrt{f_c'}}{f_y} bd = \frac{3\sqrt{3500}}{60{,}000} \times 12 \times 6.5 = 0 \quad \text{in}^2$$

again $A_{s(min)} = \frac{200}{f_y} bd = \frac{200}{60000} \times 12 \times 6.5 = 0.26 \text{ in}^2$

So we will choose steel area 0.66 in^2

we can provide 4#5 bar

Working Diagram:

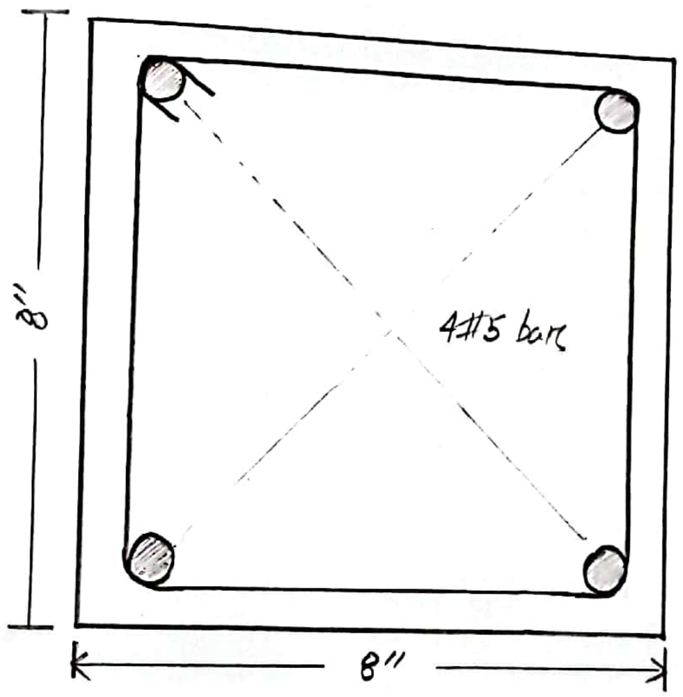
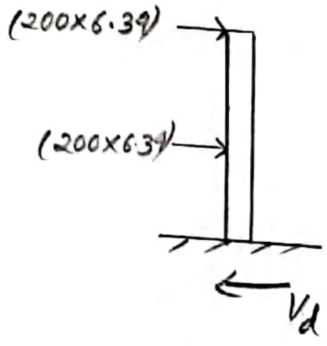


Figure 4: Cross section of rail post.

Shear Check:



Depeloped shear, $V_d = (200 \times 6.39) + (200 \times 6.39)$
$= 2536$ lb

Allowable shear $V_{all} = 1.1\sqrt{f_c'}\ bd = 1.1\sqrt{3500} \times \times 6.5$
$= 3384$ lb

$\therefore V_{all} > V_{dev}$, so no stirrup need. But we should provide.

Web reinforcement:

Spacing, $s = 3t = 3 \times 8 = 24''$

and $s_{rp} = \frac{A_V}{0.0015b} = \frac{2 \times .11}{0.0015 \times 8} = 18.33''$

So, #3 bar will be provided at rail post for stirrup at 18.33" spacing

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DEPARTMENT OF CIVIL ENGINEERING

Expt. No. ....2.2........

Name of Expt ......Edge Beam Design..

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SUBJECT : Reinforced Concrete Sessional-II	SUBMITTED BY :
COURSE NO. : CE - 3218	NAME : Abu-Shad
DATE OF EXPT.:........................	CLASS : 3rd year even semester
DATE OF SUB. :..........................	GROUP : 01 ROLL NO 1800144
	SESSION : 2018-19

Chapter No : 2.2

Chapter Name : Edge Beam Design:

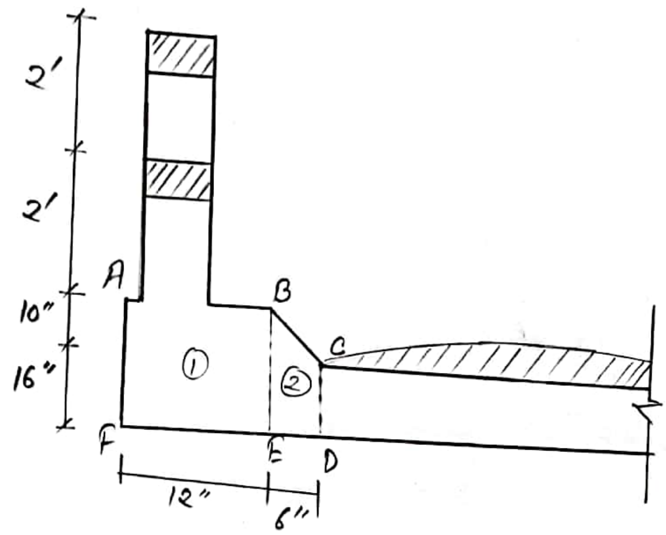


Figure : Edge Beam

Dead load of kerb:

1) Dead load of section (1)&(2) $= \left[\frac{(10+16)\times 1}{144} + \frac{(0.5\times 6\times 10)}{144} + \frac{6\times 16}{144}\right]\times 150$

$= 456.25$ plf

ii) Load coming from railing & rail post:

$$= \frac{\left[\frac{2\times6\times6}{144}\times19\right]+\left[\frac{8\times8}{144}\times4\times4\right]}{19}\times150$$

$= 131.14$ plf

$\therefore$ Total dead load $w_D = 587.4$ plf

Dead load moment $M_D = \frac{w_D \times L^2}{8} = \frac{587.4\times19^2}{8}$

$= 26505.97$ lb-ft

Live load moment $M_L = 0.1\times H_{20}S_{16}\times L = 0.1\times16000\times19$

$= 30,400$ lb-ft

$\therefore$ Total moment $= M_D + M_L = 56905.97$ lb-ft.

Depth check:

$$d_{req} = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{56905.97\times12}{256.41\times12}}$$

$= 14.9''$

Effective Depth $= 26 - cc - \phi/2 - \frac{\phi'}{4}$

$= 26 - 1.5 - 4/8 - 6/8\times2$

$= 23.63'' > d_{req}$ $\therefore$ OK

Reinforcement Calculation:

$$A_s = \frac{M}{f_s j d} = \frac{56905.94 \times 12}{24{,}000 \times 0.88 \times 23.63}$$

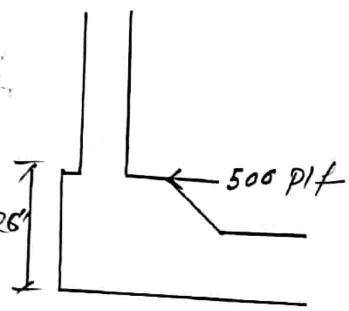
$$= 1.37 \text{ in}^2$$

Use 4# bar

Moment of critical section:

$$M_c = \left(k/_{thrust} \times \frac{26}{12}\right) + W_{H2} \times \left(2 + \frac{26}{12} + 4 + \right)$$

$$= 2633.33 \text{ lb-ft}$$



Depth check:

$$d_{req} = \sqrt{\frac{2633.33 \times 12}{256.4 \times 12}}$$

$$= 3.2''$$

$$d_{eff} = 16'' - 1.5'' - \frac{3}{8}'' - \frac{6''}{2 \times 8} = 13.75'' > d_{req}$$

∴ Depth is ok.

Reinforcement Calculation:

$$A_s = \frac{2633.39 \times 12}{24{,}000 \times 0.88 \times 13.75} = 0.1 \text{ in}^2$$

Use #3 bar at $= \frac{12 \times .11}{0.1} = 14$. c to c

Web reinforcement:

$S = 3t = 3 \times 26 = 78''$

$$S = \frac{A_v}{0.0015b} = \frac{2 \times .11}{0.0015 \times 18} = 8.15'' \approx 8'' \text{ c/c}$$

Reinforcement Details:

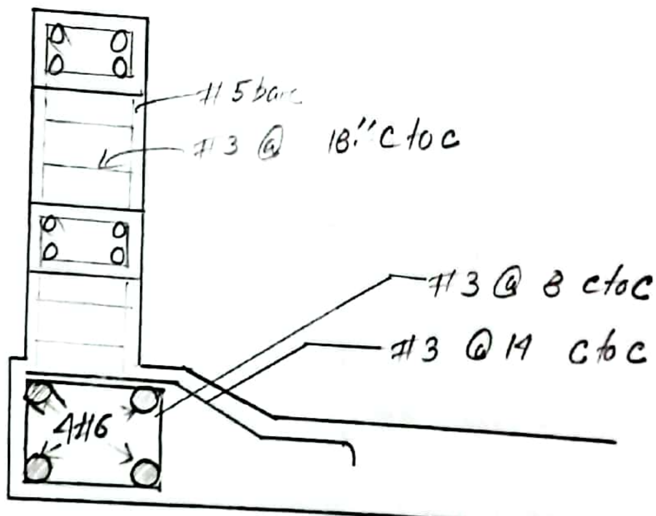


Figure: Reinforcement detail of edge beam.

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DEPARTMENT OF CIVIL ENGINEERING

Expt. No. 23

Name of Expt Design of Slab

SUBJECT : Reinforced Concrete Sessional-I	**SUBMITTED BY :**
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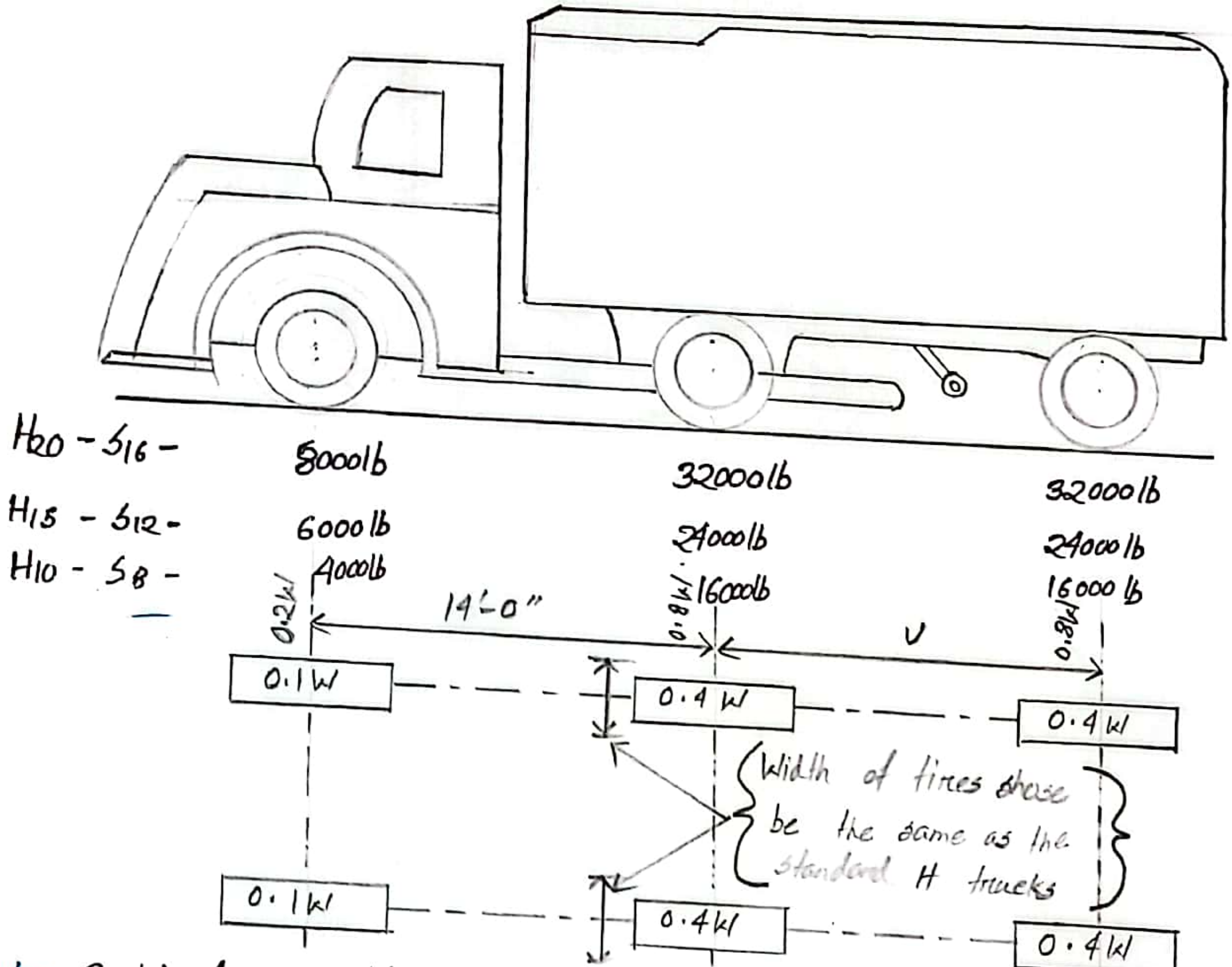
DATE OF EXPT.:	**CLASS :** 3rd year even semester
DATE OF SUB. :	**GROUP :** .... **ROLL NO** 1800149
	SESSION : 2018-19

Chapter No: 2.3

Chapter Name: Design of slab.



W = Combined weight on the first two axis which is the same as for the corresponding H truck.

V = V-ariable spacing - 14 feet to 30 feet inclusive spacing to be used in that which produces maximum stress.

Figure : Standard H-S truck.

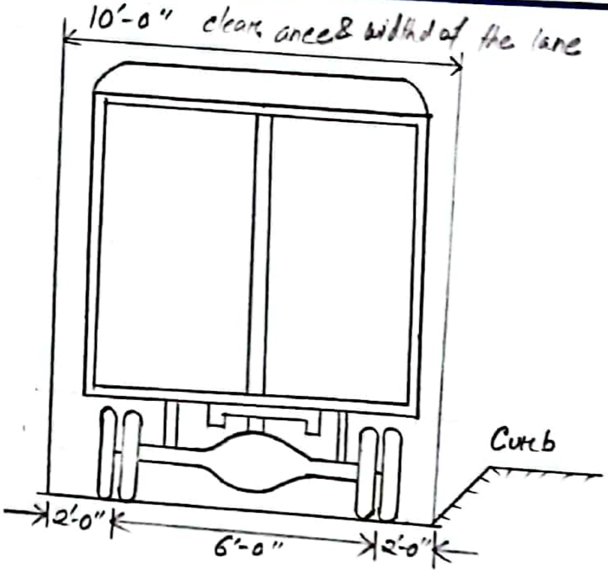
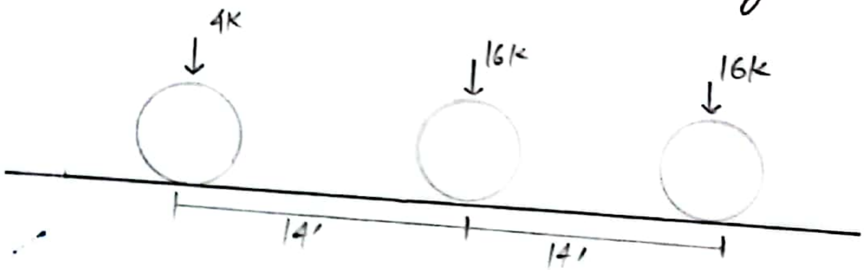
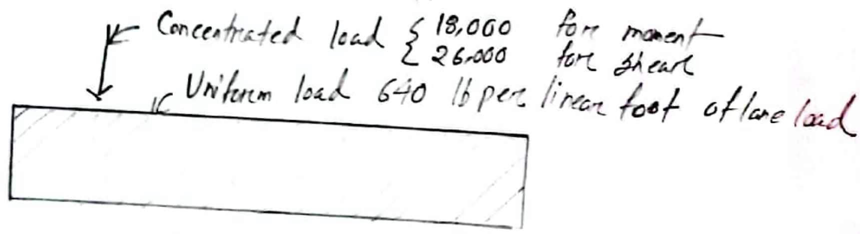


Figure : Standard H-S truck loading





H20 - S16 - 44 Loading

For H15 - S12 - 44 & H10 - S8 - 44 loading value is 75% & 50% of the H20 - S16 - 44 Loading.

Design of slab:

Design consideration

① Span length:

For simple span, S = C to C distance of support

S_{max} = clear distance + thickness of slab.

② Bending moment:

Case-I: Main reinforcement perpendicular to traffic,

Live load moment $= \frac{S+2}{32} P_{20}$; for H_{20}

$= \frac{S+2}{32} P_{15}$; for H_{15}

Case-II: Main reinforcement parallel to traffic

Distribution of wheel load, $E = 4 + 0.06S$ (ft) $\leq 7.0'$

Maximum live load moment per foot width of slab without impact,

① Span up to including 50; $S \leq 50'$

Live load moment $= 900S$ lb-ft

(ii) Span 50 to 100 ft; $50 < S < 100$

live load moment $= 1000\ (1.35S - 20)$ lb-ft

(3) Distribution reinforcement:

For main reinforcement parallel to traffic,

percentage $= \frac{100}{\sqrt{S}}$; maximum = 50%

For main reinforcement perpendicular to traffic,

percentage $= \frac{220}{\sqrt{S}}$ maximum = 67%

Span:

Effective span = 19 ft

Thickness of slab = 16" (Assume)

Load calculation:

Dead load:

(i) Self weight of slab = $16/12 \times 156$ = 200 psf.

(ii) Wearing course = 20 psf

∴ Total dead load = 220 psf

Analysis of bending moment and shear force:

Dead load moment: $M_D = \frac{Wl^2}{8} = \frac{220 \times 19^2}{8} = 9927.5$ lb-ft

Reaction = $\frac{Wl}{2}$ = 2090 K

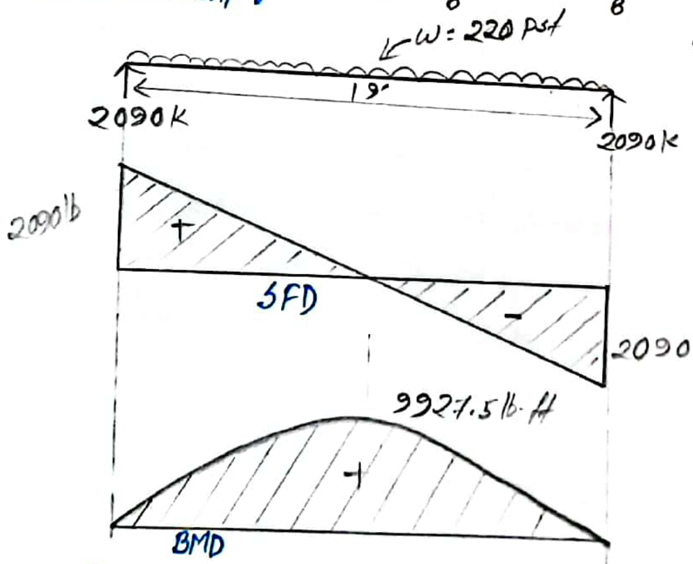


Figure: SFD & BMD

Live load moment:

Load on each rare wheel $= 16K$

Distribution of wheel loads, $E = 4 + 0.06S \leq 7.0'$

$= 5.14' \leq 7'$

Load on per unit width of slab $= P/E = 16/5.14 = 3.11 K/ft$

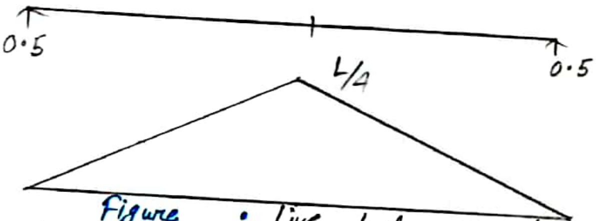


Figure : Live load moment

Live load moment $= P/E \times L/4 = 3.11 \times 19/4 = 14.79 K\text{-}ft$

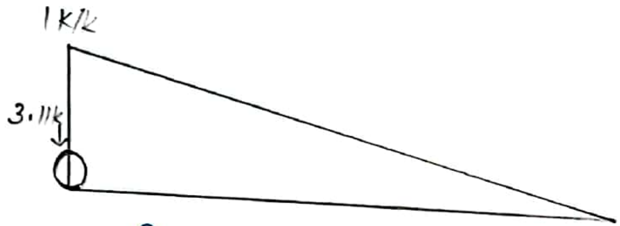


Figure : Live load shear

Live load shear $= 3.11 \times 1 = 3.11 K$

Impact analysis:

Impact co-efficient for shear, $I_c = \frac{50}{L+125} \leq 0.3$

$= \frac{50}{19+125} \leq 0.3$

= 0.3

Impact analysis:

Impact shear force = live load shear force × I_c
= 3.11 × 0.3 = 0.933 K

Impact moment = Live load moment × I_c
= 14.79 × 0.3
= 4.437 K-ft

Design Value for shear & moment:

Design shear = Dead load shear + Live load shear + Impact load shear
= 2.09 + 3.11 + 0.933
= 6.13 K

Design moment = Dead load moment + Live load moment + Impact load moment
= 9.93 + 14.79 + 4.43
= 29.15 K-ft

Depth check:

$$d_{req} = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{29.15 \times 12000}{256.14 \times 12}}$$

$$= 10.67''$$

$d_{eff} = 16'' - 1'' = 15'' > d_{req}$

$\therefore$ OK design

Reinforcement calculation:

$$A_s = \frac{M}{f_s j d} = \frac{29.15 \times 12000 \times 1}{24000 \times 0.88 \times 15}$$

$$= 1.1 \text{ in}^2$$

Use #5 bar @ 3.25'' c to c

Distribution reinforcement:

Main reinforcement is parallel to the traffic,

$$\text{percentage} = \frac{100}{\sqrt{S}} = \frac{100}{\sqrt{19}} = 22.94\%$$

$\therefore$ Distribution reinforcement $= 0.2294 \times 1.11$

$= 0.25 \text{ in}^2$

Use #4 @ 9.5'' c to c

Shear check:

Allowable shear $V_{all} = 1.1\sqrt{f_c'}\,bd = 1.1\sqrt{3500} \times 12 \times 15$

$= 11713.84 lb$
$= 11.71 K$

V_d = = Design shear = 6.13K

∴ No need of stirrup.

Bond check:

$$u_{dev} = \frac{V}{\Sigma_0 jd} = \frac{6.13 \times 10^3}{\pi \times 4/8 \times \frac{12}{3.25} \times 0.88 \times 15} = 80 \text{ psi}$$

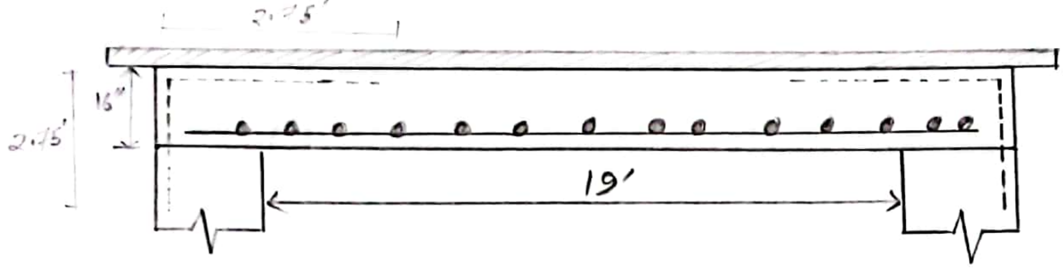
Allowable bond stress, $u_{all} = \frac{6.7\sqrt{f_c'}}{D} = \frac{6.7\sqrt{3500}}{4/8} \leq 560$

$= 792.75 \leq 560$

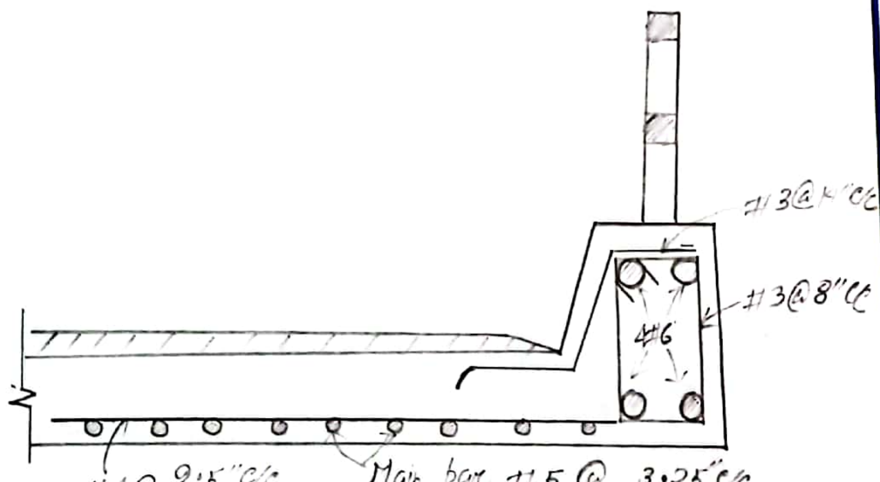
$= 560$

∴ $U_{dev} < U_{all}$ ∴ Design is ok.

Working Diagram:



(a) Longitudinal section.



(b) Transverse Section.

Figure : Details of Slab Bridge

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Rajshahi University of Engineering & Technology
Rajshahi

DEPARTMENT OF CIVIL ENGINEERING

Expt. No. 2.4

Name of Expt Design of Abutment Wall

SUBJECT : Reinforced Concrete Sessional-II	**SUBMITTED BY :**
COURSE NO. : CE-3218	**NAME :** Abu Shad
DATE OF EXPT.:	**CLASS :** 3rd year even semester
DATE OF SUB. :	**GROUP :** .... **ROLL NO** 1800144
	SESSION : 2018-19

Chapter No : 2.4
Name of the chapter : Design of Abutment Wall

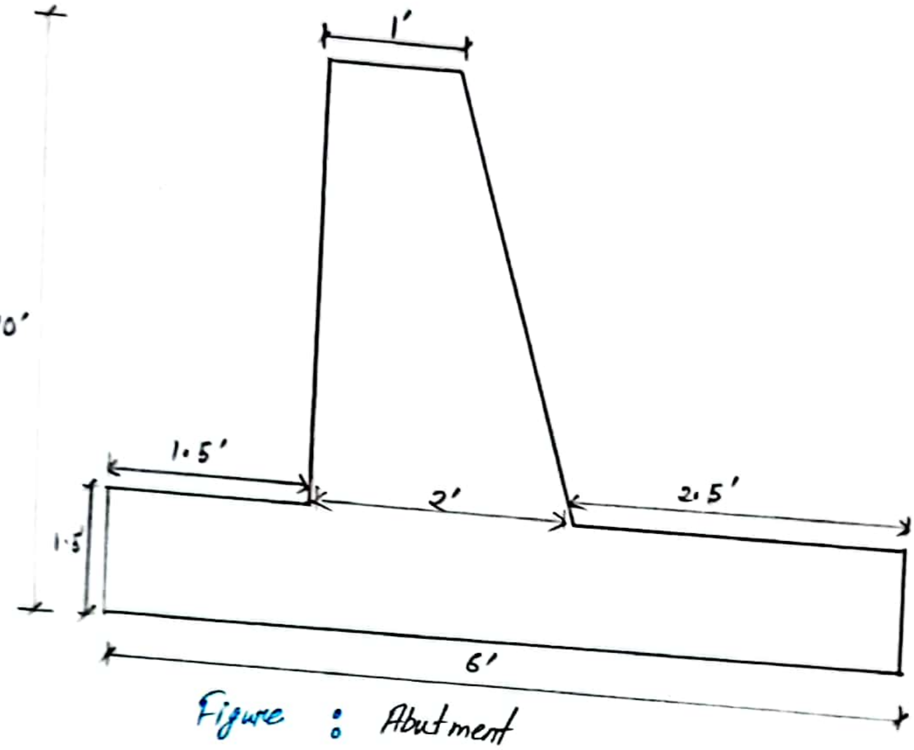


Figure : Abutment

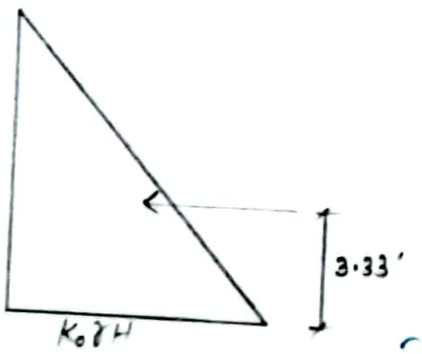


Figure : Pressure

Forces acting on structure of abutment:

i) Lateral earth pressure.
ii) Surcharge pressure.
iii) Wheel load.

Failure modes:

i) External stability.
ii) Internal stability.

(i) External stability:

(a) Sliding Failure:

Sliding force = Total active pressure acting on abutment
Resistance force = Total friction force acting between concrete and soil at the base of abutment.

$$\text{Factor of abutment} = \frac{\text{Resisting force}}{\text{Sliding force}}$$

[must be >1.5]

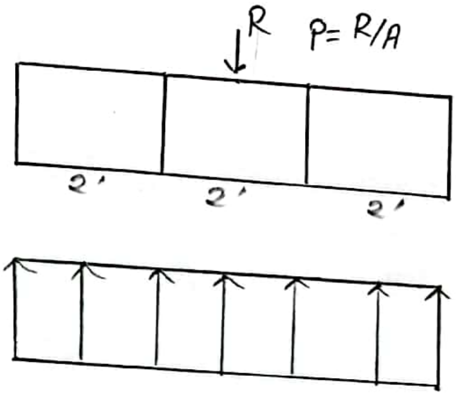
(b) Overturning failure :

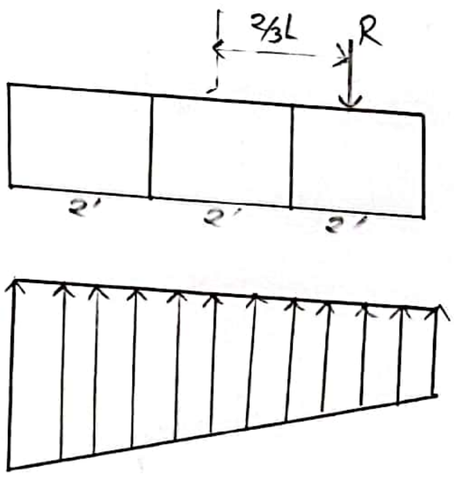
Overturning failure = overturning moment.

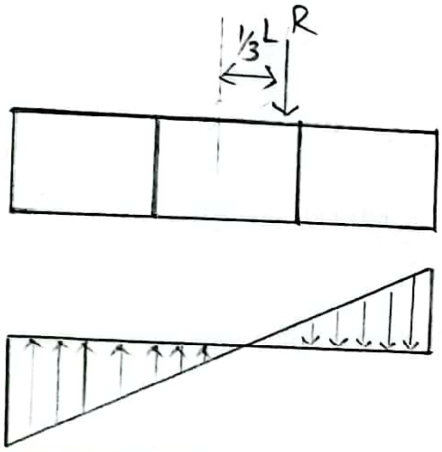
$$\text{Factor of safety} = \frac{\text{Resisting force}}{\text{Sliding force}}$$

(c) Bearing failure (Inadequacy in soil bearing capacity)

We have to consider a strip →







Retaining wall :

Types of retaining wall ->

(i) Gravity retaining wall.
(ii) Cantilever retaining wall.

Factors controlling height of abutment:

i) Height of flood level.
ii) Free board.
iii) Scour depth.

Specification :

(i) Height of the abutment = 10'.
(ii) Allowable soil pressure = 3 ksf.
(iii) Unit weight of soil = 120 psf.
(iv) Angle of internal friction, $\varphi = 30°$.
(v) Minimum factor of safety = 1.5.
(vi) Friction co-efficient f = 0.5.

$K_a = \frac{1-\sin\varphi}{1+\sin\varphi} = \frac{1-\sin 30^\circ}{1+\sin 30^\circ} = 0.33$

$K_p = \frac{1+\sin\varphi}{1-\sin\varphi} = 3$

$K_a \gamma H = 0.33 \times 120 \times 10 = 396$ psf.

Total active pressure $= \frac{1}{2} \times 396 \times 10 = 1980$ lb.

Overturning moment $= P_0 \times H/3 = 1980 = 6600$ lb-ft.

Considering 1' strip →

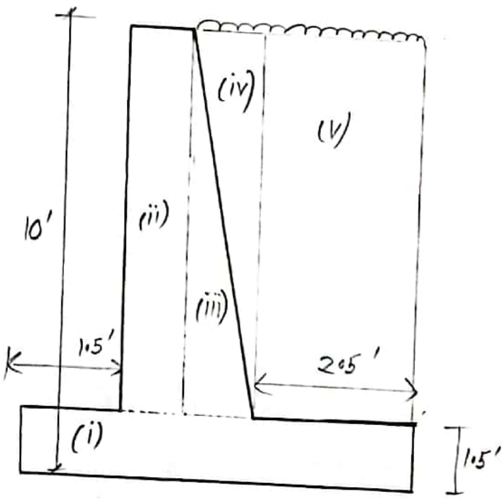


Figure : Section for calculating resisting moment...

Total weight and resisting moment

Case-I

Section	Weight (lb)	Moment arm (lb-ft)	Resisting moment (lb-ft)
i	1·5× 6×150	3	4050
ii	8·5×150×1	1·5+0·5	2550
iii	½ ×1 × 8·5×150	2·5+(1×⅓)	1804·125
iv	½ × 1×8·5×120	1·5 +1 +⅔	1615
v	2·5×8·5×120	3·5 + (2·5×½)	12,112·5
Total	= 6322·5		22131·62

Case-II

Dead Load	Load = 2420 lb Total weight = 8742·5 lb	Liver arm = 2′	moment = 4840 lb-ft Total moment = 26971·6 lb-ft

Case-III

Live Load	Load = 3000 Total weight = 11742·5	Liver arm = 2′	moment = 6000 Total moment = 32971·6 lb-ft

Stability Check:

Case - I : Super structure is absent.

Factor of safety against sliding $= \frac{W \times f}{P_a}$

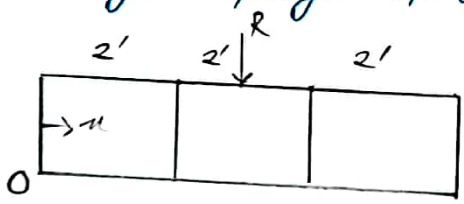
$= \frac{6322.5 \times .5}{1980}$

$= 1.6 > 1.5$ (ok)

Factor of safety against sliding of overturning

$= \frac{22131.62}{6600} = 3.3 > 1.5$ (ok)

Check for bearing capacity of soil:



location of resultant from O,

$\bar{x} = \frac{M_R - M_O}{W} = \frac{22131.6 - 66000}{6322.5} = 2.46'$

$e = b/2 = 6/2 = 3'$

$e = b/2 - x$

$= 3 - 2.46$

$= 0.54$

$M = W \times e$

$= 6322.5 \times 0.54$

$= 3414.15$ lb-ft

$\sigma = P/A \pm Mc/I$

$= \frac{6322.5}{6\times1} + \frac{3414.15\times3}{18}$

$= 1622.78$ psf < 3000 psf (allowable)

$I = \frac{bh^3}{12} = \frac{1\times6^3}{12} = 18$

or $\sigma = P/A - Mc/I$

$= \frac{6322.5}{6\times1} - \frac{3414.15\times3}{18}$

$= 484.72$ psf < 3000 psf (allowable)

Case - II

Super structure is present but no live load on super structure -

Factor of safety against overturning $= M_R/M_o = \frac{26971.6}{6600}$

$= 4.1 > 1.5$ (ok

Factor of safety against sliding $= \frac{W_f\times.5}{1980}$

$= \frac{8742.5\times.5}{1980}$

$= 2.2 > 1.5$

(ok)

Case -III

Superstructure is present with live load on the superstructure.

Factor of safety against overturning = M_r/M_o

$= \frac{32971.6}{6600} = 5 > 1.5$ (ok)

Factor of safety against sliding = $\frac{W_f \times .5}{1980}$

$= 2.97 > 1.5$ (ok)

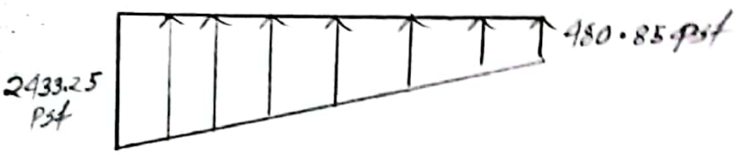
Case-II :

$$\bar{x} = \frac{M_R - M_o}{W} = \frac{26971.6 - 6600}{8742.5} = 2.33'$$

$\therefore e = b/2 - \bar{x} = 6/2 - 2.33 = 0.67'$

$\therefore M = We = 5857.5$ lb-ft

$$\therefore \sigma = \frac{8742.6}{6 \times 1} \pm \frac{5857.5 \times 3}{18} = (2433.25 \ \& \ 480.85) \text{ psf}$$



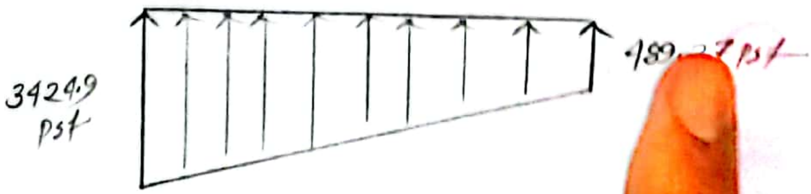
Case-III :

$$\bar{x} = \frac{M_R - M_o}{W} = \frac{32971.6 - 6600}{11742.5} = 2.25'$$

$\therefore e = b/2 - \bar{x} = 6/2 - 2.25 = 0.75'$

$\therefore M = We = 8806.87$ lb-ft

$$\therefore \sigma = \frac{11742.5}{6 \times 1} \pm \frac{8806.87 \times 3}{18} = (3424.9 \ \& \ 489.27) \text{ psf}$$



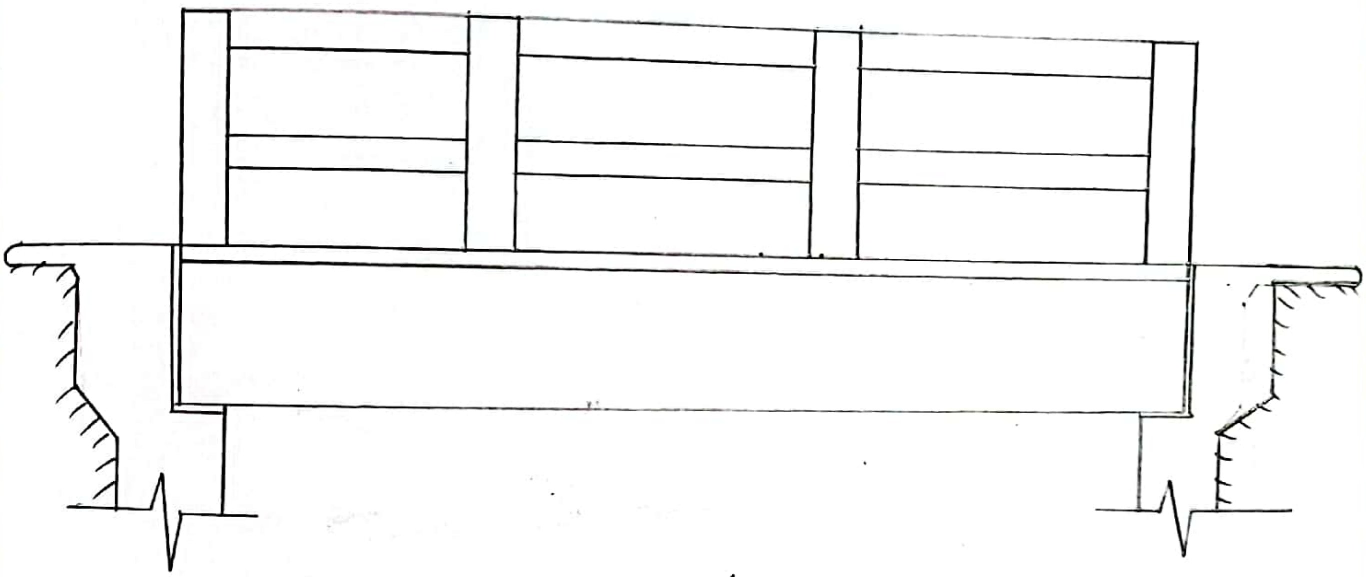


Figure: Bridge Profile

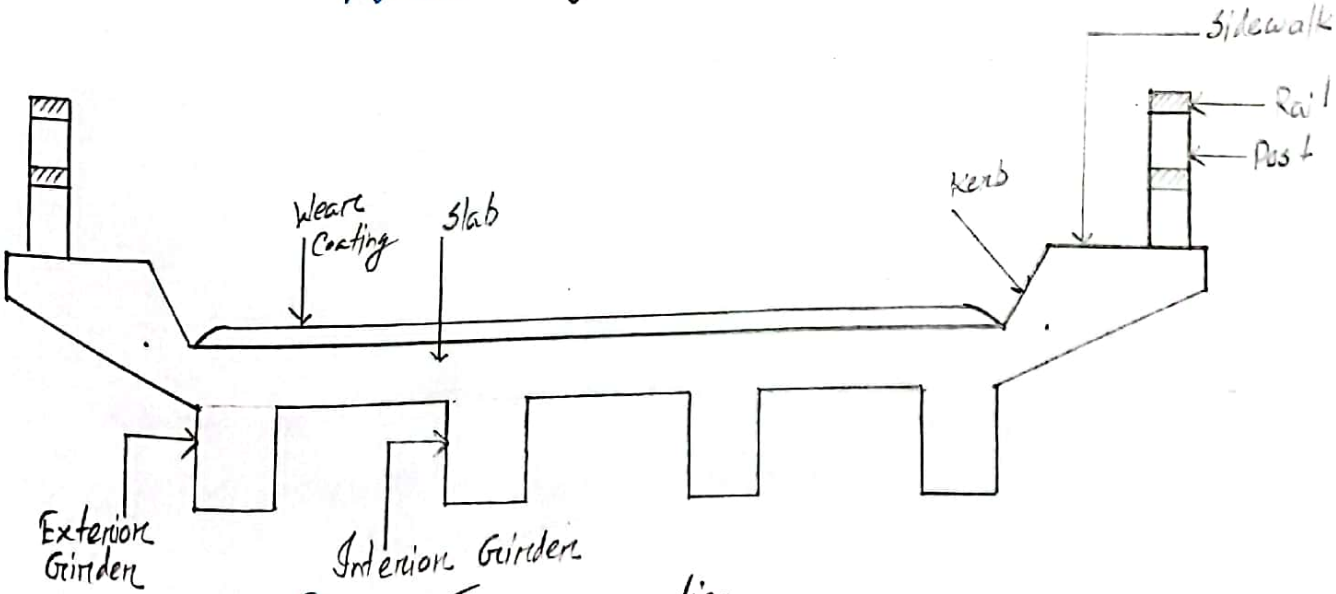


Figure: Transver section
Plate Girder Bridge

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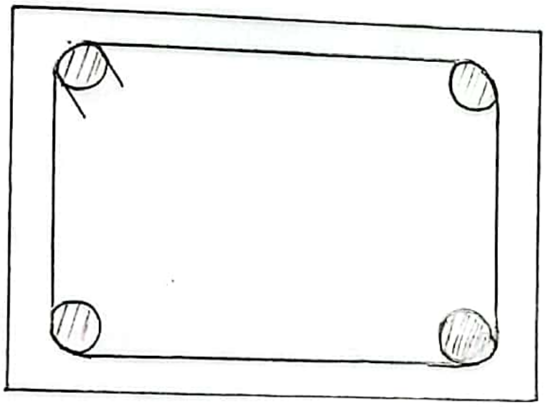
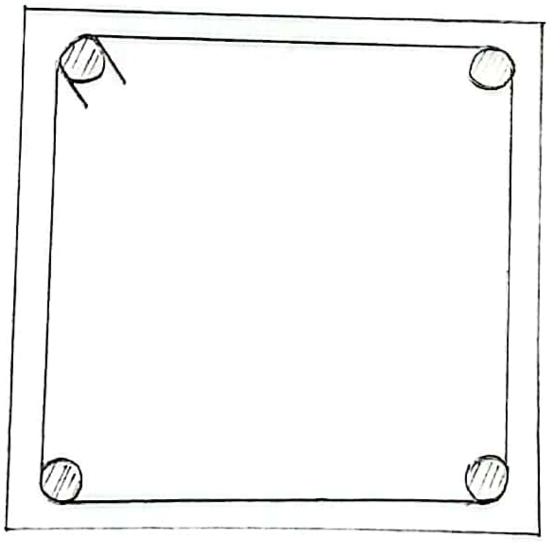
Reinforcement detail of Rail & Post:

Rail:



Description: (6×6)" with 4#5 bars

Post:



Description: (8×8)" with 4#5 bars

Design Specification:

(i) Clear span length = last 2 digit of roll + 50/3

$= 44 + 50/3$

$= 60.67'$

(ii) Width of the bridge = equivalent two lanes = 20'

(iii) No of girder = 4

(iv) Loading → HS 512 truck load.

(v) Material properties, $f_c' = 3000$ psi, $f_y = 50,000$ psi.

(vi) Width of girder = 15"

(vii) Weight of wearing course = 20 psf

Design Calculations:

Center to center spacing of girder $= \frac{20}{4-1} = 6.67'$

Clear spacing between two girder $= (6.67 - 15/12) = 5.42'$

Relevent Properties:

$n = 9$, $r = 14.81$,

$k = 0.38$, $j = 0.87$

$R = 223.16$ psi

Design for slab:

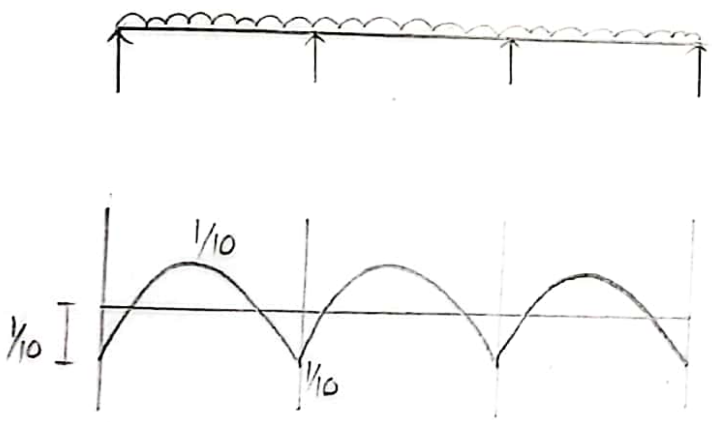
Assume the thickness of slab, t = 6″

Dead load moment:

Self weight = t/12 × 150 = 75 psf

weight of wearing course coat = 20 psf

∴ Total dead load, w = (75+20) = 95 psf



Dead load moment $M_o = \frac{WL^2}{10} = 1/10 \times 95 \times 5.42^2 = 279.07$ lb-ft

Live Load Moment:

For main reinforcement perpendicular to traffic:

$M_{LL} = \frac{S+2}{32} P_{15} = \frac{5.42+2}{32} \times 12000 = 2782.5$ lb-ft

Impact Moment:

$$I = \frac{50}{L + 125} \leq .3$$

$$= \frac{50}{5.42 + 125} = 0.38 \leq .3$$

$\therefore$ 30% of Live moment

$\therefore M_I = 0.3 \times M_{LL} = (0.3 \times 2782.5) = 834.75$ lb-ft

$\therefore$ Design moment:

$$M_D = M_D + M_{LL} + M_I$$
$$= 279.07 + 2782.5 + 834.75$$
$$= 3896.32 \text{ lb-ft}$$

Depth check:

$$d_{req} = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{3896.32 \times 12}{223.16 \times 12}} = 4.18''$$

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

$d_{act} > d_{req}$

So, that design is ok

Main reinforcement calculation:

$$A_s = \frac{M}{f_s j d} = \frac{3896.32 \times 12}{20{,}000 \times 0.875} = 0.53\ in^2$$

$$Spacing = \frac{0.31 \times 12}{0.53}$$

$$= 7.02 \simeq 7''\ c/c$$

Use #5 bar @ 7'' c/c

Distribution reinforcement calculation:

For main reinforcement perpendicular to traffic,

$$A_{st} = \frac{220}{\sqrt{S}} \leq 67\%$$ of main reinforcement.

$$= \frac{220}{\sqrt{5.42}} = 94.49 \leq 67\%$$

$\therefore A_{st} = 0.67 \times 0.53 = 0.36\ in^2$

Use # 4 @ $\frac{0.2 \times 12}{0.36} = 6.63 \simeq 6.5''\ c/c$

Shear check:

Dead load shear

$$V_{max} = \frac{wL}{2} = 257.45\ lb/ft$$

Live load on per unit width of slab $= P/E = \frac{12000}{4 + 0.06 \times L_L}$

$$= \frac{12000}{4 + 0.06 \times 60.67}$$

in 1ft $= 1570.64\ lb/ft$

$\therefore$ Impact shear $= 1570.64 \times 0.3 = 471.2\ lb/ft$

$\therefore V_d = \dfrac{257.45 + 1570.64 + 47.12}{bd \times (1 \times 12)} = \dfrac{2299.28}{12 \times 12 \times 5}$

cause in 1ft strip

$= 3.19$ psi

$V_{all} = 1.1\sqrt{f_c'} = 1.1\sqrt{3000} = 60.2$ psi

$\therefore V_d < V_{all}$

No need stirrup.

Bond check: $V_d = \dfrac{V_{max}}{\Sigma_o jd} = \dfrac{V_{max}}{\pi \times \frac{12}{spacing} \times d_b \times 0.87 \times 5}$

$= \dfrac{257.45}{\pi \times \frac{12}{7} \times \frac{5}{8} \times 0.87 \times 5}$

$= 17.58$ psi

$V_{all} = \dfrac{1.7\sqrt{f_c'}}{D} = \dfrac{1.7\sqrt{3000}}{5/8} = 148.98$

$\therefore V_{all} > V_d$ The design is ok

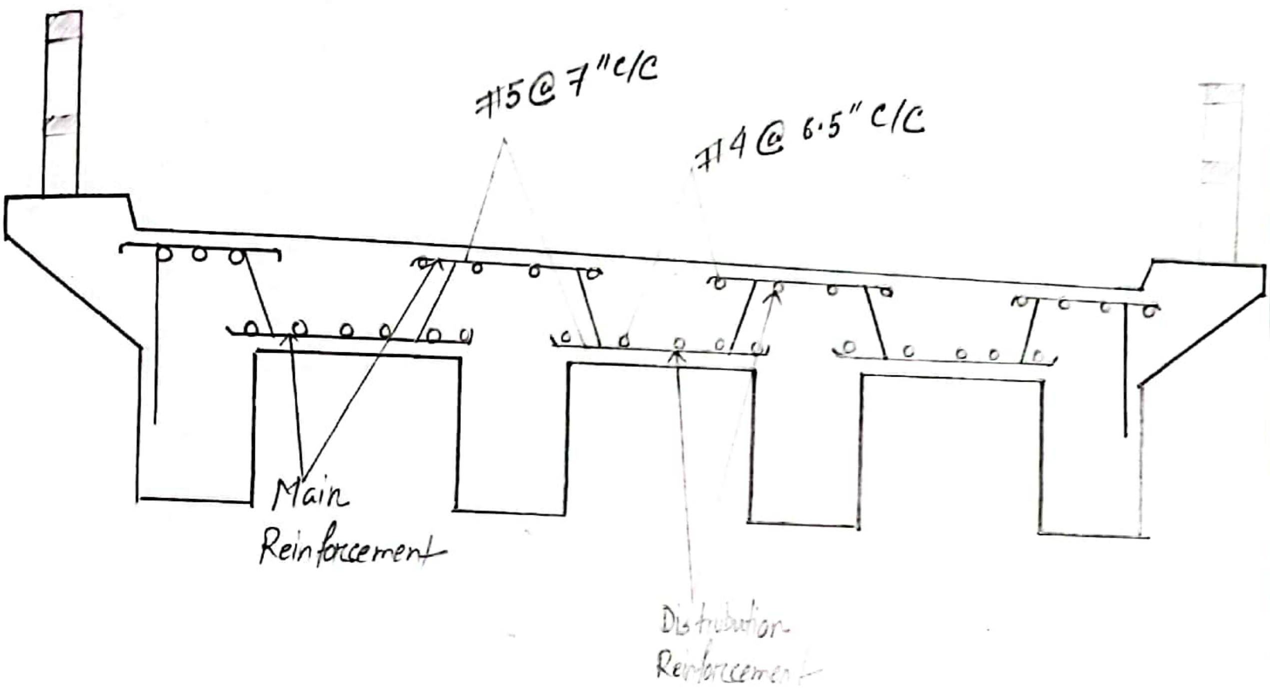
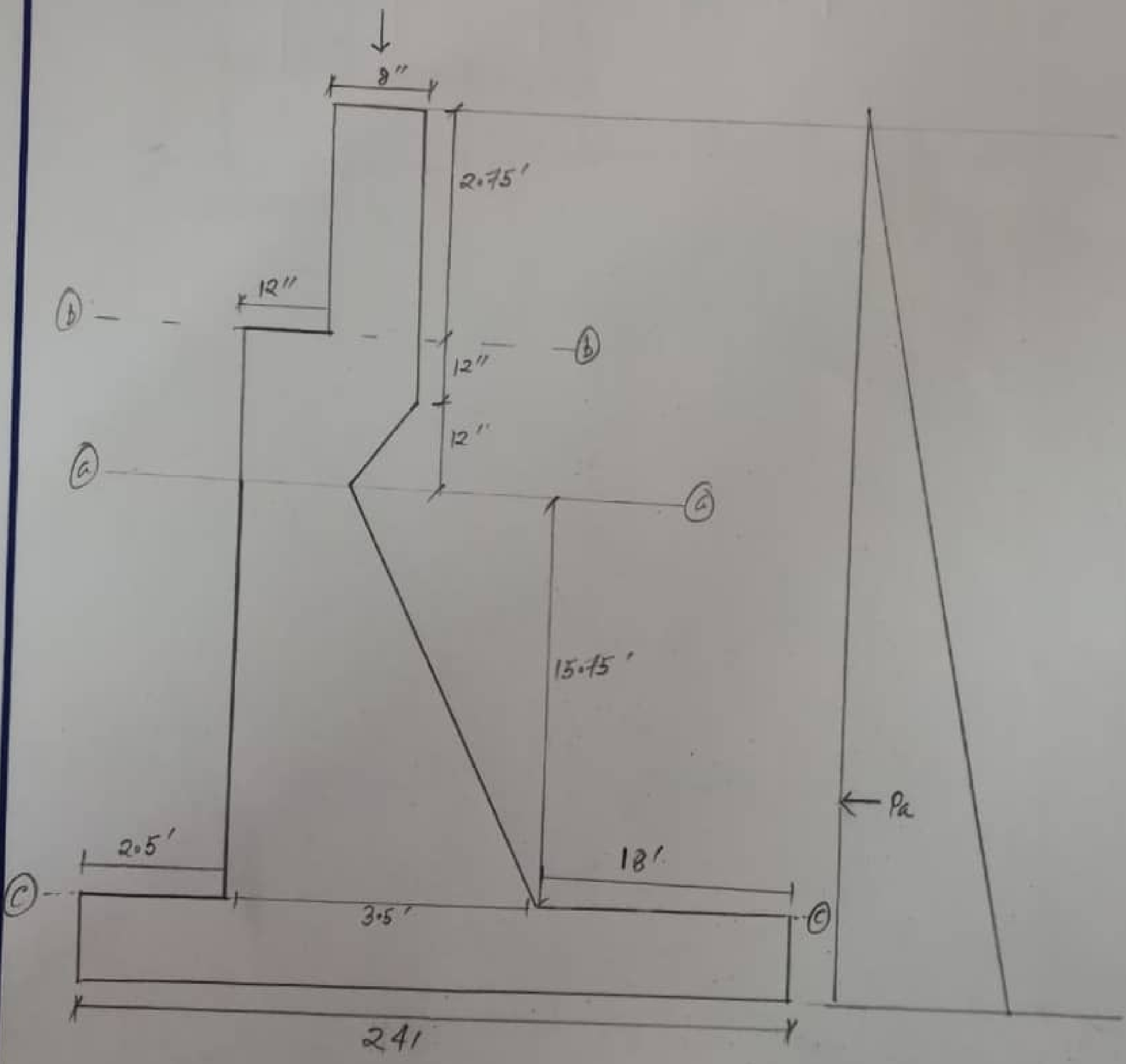
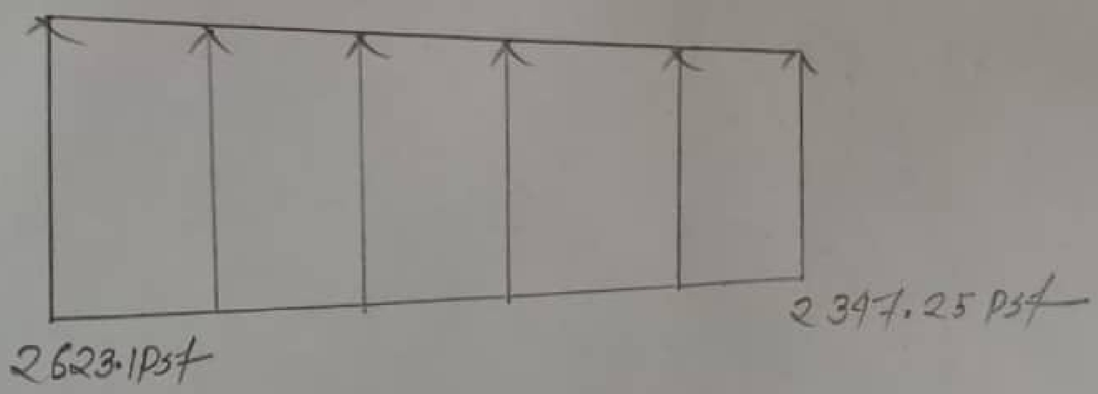


Figure : Typical reinforcement details of deck slab.

Design of Back fill:




2347.25 Psf
2623.1 Psf

Design of abutment for Deck Girder Bridge:

Design specification:

1/ Overall height of the abutment = 22'
2/ Bearing capacity of soil = 4 ksf
3/ Unit weight of soil = 120 pcf
4/ Angle of internal friction, $\phi = 30°$
5/ Co-efficient of friction, $f = 0.5$
6/ $f_c' = 3{,}000$ psi, $f_y = 50{,}000$ psi

Active earth pressure co-efficient, $k_a = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-\sin(30°)}{1+\sin(30°)}$
$= 1/3$

Passive earth pressure co-efficient, $k_p = 1/k_a = 3$

Now, $P_a = 1/2 \times K_a \gamma_s H \times H = 1/2 \times 1/3 \times 120 \times (22)^2 = 9670.32$ lb

$\therefore$ Active earth pressure, $P_a = 9670.32$ lb

$\therefore$ Overturning moment, $M_o = P_a \times H/3 = 9670.32 \times 22/3$
$= 70915.68$ lb-ft

Table : Calculation for total load, weight & resisting moment.

Section	Weight (lb)	Moment-arm (ft)	Moment (lb-ft)
1.	24×2×150 = 7200	24/2 = 12	86400
2.	15.25×1×150 = 2287.5	2.5 + 0.5 = 3	6862.5
3.	1×20/12 ×150 = 250	2.5 + 0.833 = 3.33	833.33
4.	1×1×150 = 150	2.5+0.5 = 3	450
5.	2.75 ×8/12 × 150 = 275	2.5+1 +0.33 = 3.833	1054.075
6.	1/2 ×8/12 ×1×120 = 40	2.5+1+ 2/3×8/12 = 3.94	157.77
7.	1/2×8/12 ×1×150 = 50	2.5+ 1+1/3×8/12 = 3.72	186.11
8.	1/2×2.5 ×15.25 ×120 = 2287.5	2.5+1 + 2/3×0.5 = 5.17	11818.75
9.	18×15.25×120 = 32940	15	494100
10.	4.75×19.833 ×120 = 11304.8	14.08	159171.72
11.	1/2 ×2.5 ×15.25 ×150 = 2859.375	4.333	12390.625
Total	W/f = 59644.175	ΣM_R	773424.88

Condition -1 : When superstructure is absent

$M_{a-a} = 1/2\, K_a \times \gamma \times h_a \times h_a \times h_a/3$

$= 1/2 \times 0.333 \times 120 \times \frac{(4.75)^3}{3} = 713.76$ lb-ft

$M_{b-b} = 1/2 \times K_a \times \gamma \times h_b \times h_b \times h_b/3 = 1/3 \times 0.333 \times 120 \times \frac{2.75^3}{3}$

$= 138.50$ lb-ft

Depth check:

For section a-a; $d = \sqrt{\frac{713.76 \times 12}{R \times b}}$

$= \sqrt{\frac{713.76 \times 12}{223.16 \times 12}}$

$= 1.8''$

$d_{eff} = t - 1 - \frac{5}{8 \times 2} = 12'' - 1'' - \frac{5''}{8 \times 2} = 10.69'' > d$

$\therefore d_{eff} > d$

$\therefore$ Check is okay.

For section (b-b)

$d = \sqrt{\frac{M_{b-b}}{Rb}} = \sqrt{\frac{138.50 \times 12}{223.16 \times 12}} = 0.8''$

$\therefore d_{eff} = 18 - 1 - \frac{5}{8 \times 2} = 16.69'' > d$

$\therefore$ Design is okay.

Stability Check:

Factor of safety (FS) against sliding $= \frac{W_f \times f}{P_a}$

$= \frac{59644.175 \times 0.5}{9670.32}$

$= 3.1 > 1.5$ (Okay)

Factor of safety against overturning $= \frac{M_R}{M_0} = \frac{773424.88}{76915.68}$

$= 10.9 > 1.5$ (Okay)

Check for soil pressure:

We know, $\sigma = W_f/A \pm Mc/I$

Here, $A = 24 \times 1 = 24 ft^2$; $I = \frac{1 \times 24^3}{12} = 1152 ft^4$; $c = B/2 = 24/2 = 12'$

$e = c - a = c - \frac{M_R - M_0}{W_f} = 0.222$

$\therefore M = W_f \times e = 59644.175 \times 0.222 = 13241$ lb-ft

$\therefore \sigma_1 = \frac{59644.175}{24} + \frac{13241 \times 12}{1152} = 2623.1$ psf < 4000 psf

$\sigma_2 = \frac{59644.175}{24} - \frac{13241 \times 12}{1152} = 2347.25$ psf < 4000 psf

Bond check:

$$U_d = \frac{V}{\Sigma jd} = \frac{66.37 \times 1000}{\pi \times 9/8 \times 12/6.6 \times 87 \times 40} = 3.37 \text{ ksi}$$

$V_{all} = 0.1 f_c' = 300$ ksi $> U_d$

∴ Bond check ok.

Design of base:

Moment with respect to base,

$M = 1/2 \times K_a \times \gamma_s \times \frac{H^3}{3} = 1/2 \times 0.333 \times 120 \times \frac{28^3}{3}$

$= 52800$ lb-ft

Depth check:

$d = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{52800 \times 12}{223 \times 12}} = 15.4''$

$d_{eff} = 168'' - 1.5' = 166.5'' > d$

Reinforcement calculation:

$A_s = \frac{52800 \times 12}{20{,}000 \times 0.87 \times 166.5} = 0.218 \text{ in}^2$

but $A_{s(min)} = 0.002bt = 0.002 \times 12 \times 168 = 4.032 \text{ in}^2$

$\therefore A_s = 4.032 \text{ in}^2$

(i) Spacing, $s = \frac{12 \times 0.6}{4.032} = 1.78''$

(ii) s_{max}, $s = 3t = 3 \times 168 = 504''$

(iii) $s_{max} = 18''$

$\therefore$ Using #7 bars @ 1.5" C/C

Reinforcement Calculation:

At section a-a, $A_{s(a-a)} = \frac{M_{a-a}}{f_s j d} = \frac{713.76 \times 12}{20,000 \times 0.87 \times 10.69}$

$= 0.05 \ in^2$

Minimum reinforce, $A_{smin} = 0.002bt$

$= 0.002 \times 12 \times 12$

$= 0.288 \ in^2$

Spacing, $s = \frac{12 \times 0.2}{0.288} = 8.33 \approx 8'' \ c/c$

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

$s_{max} = 18''$

∴ Using # 4 bar @ 8" c/c

For distribution reinforcement,

$A_{st} = 0.002bt = 0.288 \ in^2$

∴ Using # 4 bar @ 8" c/c

At section b-b,

$A_{s(b-b)} = \frac{M_{b-b}}{f_s j d} = \frac{138.50 \times 12}{20,000 \times 0.87 \times 16.69} = 0.006 \ in^2$

but, minimum steel $A_s = 0.002 \times bt = 0.002 \times 12 \times 18 = 0.432 \ in^2$

∴ Using # 4 bar @ spacing, $s = \frac{12 \times 0.2}{0.432} = 5.55 \approx 5''$ c/c

Distribution, $A_{st} = 0.432 \ in^2$

use # 4 @ 5" c/c

Design of Stem:

$M_{c-c} = 1/2 \times k_a \times \gamma \times h_c \times h_c \times h_c/3$

$= 1/2 \times 0.333 \times 120 \times (18)^3/3 = 38841.12$ lb-ft

$V_{cc} = 1/2 \times 0.333 \times 120 \times (18)^2 = 6473.52$ lb

Depth Check:

$d_{c-c} = \sqrt{\frac{M_{c-c}}{R \times b}} = \sqrt{\frac{38841.12 \times 12}{220.16 \times 12}} = 13.93"$

$\therefore d_{eff} = 42" - 1.5" = 40.5" > d_{c-c}$

$\therefore$ Check is okay.

Reinforcement Calculation:

At section c-c, $A_{s(c-c)} = \frac{M_{c-c}}{f_s j d} = \frac{38841.12 \times 12}{20{,}000 \times 0.87 \times 40.5} = 0.66 in^2$

$A_{s\,min} = 0.0018\, bt = 0.00 \quad \times 12 \times 42 =$ 1.008

Use #6 bar.

(i) Spacing, $s = \frac{12 \times 0.44}{1.008} = 5"$

(ii) s_{max}, $3t = 3 \times 42 = 126"$

(iii) $s_{max} = 18"$

$\therefore$ Using #6 bar @ 5" c/c.

Shear check:

$V_{all} = 1.1\sqrt{f_c'}\ bd = 1.1\sqrt{3000} \times 12 \times 40.5$

$= 29281.29 > V_{c-e}$

∴ Check is okay.

Development length: Here, $u_d = \frac{V}{\Sigma_o j d}$

$= \frac{6473.52}{\pi \times 6/8 \times 12/5 \times 0.87 \times 40.5}$

$= 32.5$ psi

∴ $L_d = \frac{f_s D}{4 u_d} = \frac{20,000 \times 6/8}{4 \times 32.5} = 115.38$

∴ Minimum development length, $L_{d\,min} = 12D = 12 \times 6/8$

$= 9''$

∴ Design is okay.