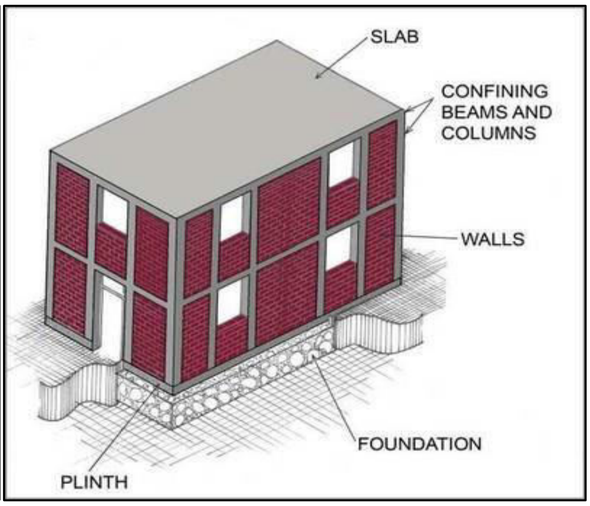
CE 316
Design of Concrete Structures Sessional



Department of Civil Engineering
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Preface

Design of Concrete Structure Sessional-I (CE316) manual contains the analysis and design of Slab bridge, Deck Girder Bridge and a Low-rise masonry building. This sessional is focused on bridge design and building design with an intention to make the students familiar with the difference between the design approach of these two very basic types of structures. For providing a complete guideline about the design procedure of a low-rise masonry building, detailed procedures are given on the design of slab, beam, stair and foundation. Also, different types of live load which are considered during the design of concrete structures are emphasized in this manual. AASHTO Code, Bangladesh National Building Code (BNBC 2020) and ACI Building Code are followed in this manual.

Sincere gratitude and reverences to the most respected faculty members Prof. Dr. Md. Mahmudur Rahman, Prof. Dr. Abdur Rouf and Mr. Md. Mashfiqul Islam for their precious time, valuable suggestions, and constructive advice in the entire process of updating the manual.

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Index

Design of Low Rise Masonry Building

Introduction

Load bearing construction is most appropriately used for buildings in which the floor area is subdivided into a relatively large number of rooms of small to medium size and in which the floor plan is repeated on each story throughout the height of the building. These considerations give ample opportunity for disposing load bearing walls, which are continuous from foundation to roof level and, because of the moderate floor spans, are not called upon to carry unduly heavy concentrations of vertical load. The types of buildings which are compatible with these requirements include flats, hostels, hotels and other residential buildings.

The basic advantage of masonry construction is that it is possible to use the same element to perform a variety of functions, which in a steel framed building, for example, have to be provided for separately, with consequent complication in detailed construction. Thus masonry may, simultaneously, provide structure, subdivision of space, thermal and acoustic insulation as well as fire and weather protection. As a material, it is relatively cheap but durable and produces external wall finishes of very acceptable appearance. Masonry construction is flexible in terms of building layout and can be constructed without very large capital expenditure on the part of the builder.

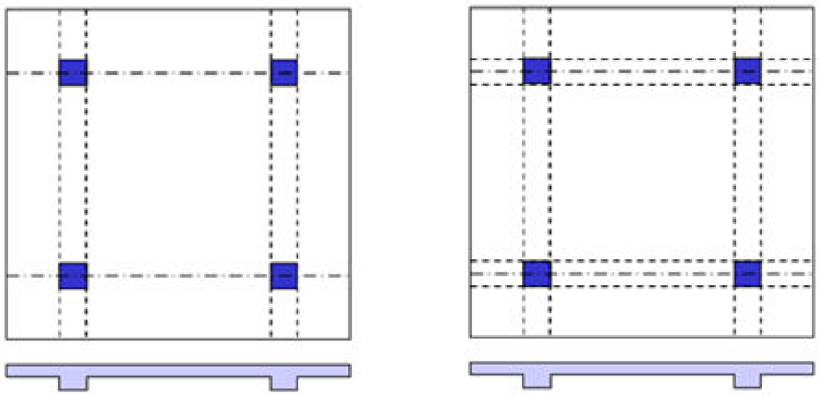
However, the quality of the masonry in a building depends on the materials used, and hence all masonry materials must conform to certain minimum standards. The basic components of masonry are block, brick and mortar, the latter being in itself a composite of cement, lime and sand and sometimes of other constituents.

1. Design of one way slab (USD)

One-way slabs are those slabs having supports along one way only (Figure 1(a)) or slabs having supports having on all sides but with an aspect ratio in plan of 2:1 or greater, in which bending is primarily about the long axis (Figure 2(a)). The slabs having supports on all sides acts as two way slab (Figure 1(b) and 2(b)) where there are bending about the both axes, but acts as one way slab where $L/B \geq 2$.

One way slab can be

1. Solid
2. Hollow or
3. Ribbed



(a) One-way slab (b) Two-way slab

Figure 1: One-way Slab and Two-way Slab

In figure 2(a) slab is supported on two opposite sides only. In this case the structural action of the slab is essentially one way.

In figure 2(b) there are beams on all four sides. Now if length to width ratio is 2 or greater, slab acts as one way slab even though supports are provided on all sides.

LOADING OF ONE WAY SLAB

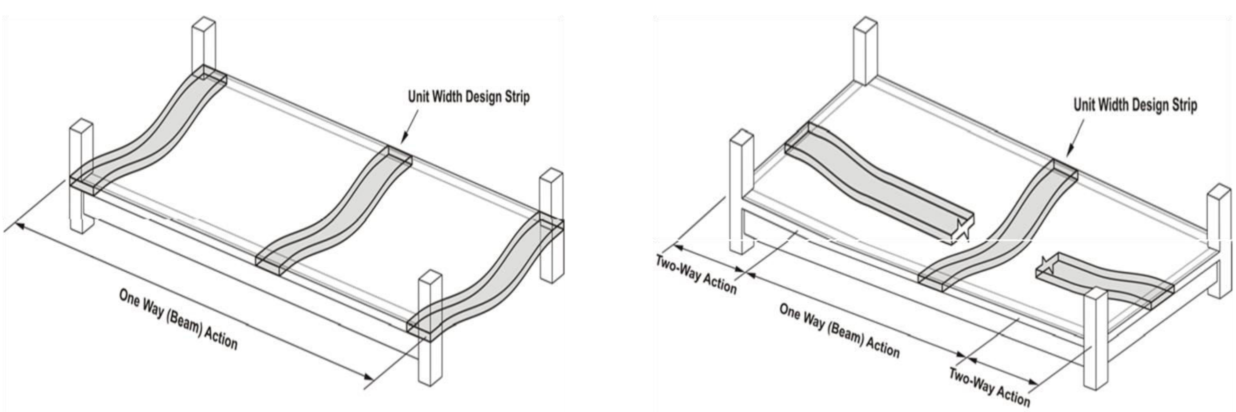


Figure 2: Slab (a) Supported on two opposite sides only and (b) Supported on all four sides

Figure 2(a) shows that when slabs are supported on two opposite sides only loads being carried by the slab in the direction perpendicular to the supporting beams.

Figure 2(b) shows that when supports are provided on all sides most of the load is carried in the short direction to the supporting beams and one way action is obtained.

1.1 SPECIFICATIONS

Minimum Slab Thickness

To control deflection, *ACI Code 9.5.2.1 specifies minimum thickness values for one-way* solid slabs.

Element	Simply Supported	One end continuous	Both ends continuous	Cantilever
One-way solid slabs	*l/20*	*l/24*	*l/28*	*l/10*

Here, l is the clear span

Multiplying factor = $0.4 + \frac{f_y}{100}$, f_y in ksi

If,

Thickness < 6 inch then upper rounding to neatest 0.25
Thickness ≥ 6 inch then upper rounding to neatest 0.50

Minimum Concrete Cover

According to ACI Code 7.7.1, the following minimum concrete cover is to be provided:

a. Concrete not exposed to weather or in contact with ground:

- Larger than Ø 36 mm bar -- 4 cm
- Ø 36 mm and smaller bars --- 2 cm

b. Concrete exposed to weather or in contact with ground:

- Ø 19 mm and larger bars-- 5 cm
- Ø 16 mm and smaller bars --- 4 cm

c. Concrete cast against and permanently exposed to earth ---------------------------------7.5 cm

Span Length

According to ACI code 8.7.1, if the slab rests freely on its supports, the span length may be taken equal to the clear span plus the depth of the slab but need not exceed the distance between centers of supports.

Bar Spacing

The spacing of the flexural bars should not exceed 3 times the thickness h or 18 inch according to ACI code 7.6.5.

The spacing of temperature and shrinkage reinforcement should not be placed farther apart than 5 times the slab thickness or 18 inch according to ACI code 7.12.2.

Generally, bar size should be selected so that actual spacing is not less than about 1.5 times the slab thickness.

Maximum Reinforcement Ratio

Reinforcement ratio: Reinforcement ratio is the ratio of reinforcement area to gross concrete area.

One-way solid slabs are designed as rectangular sections subjected to moment. Thus, the maximum reinforcement ratio corresponds to a net tensile stain in the reinforcement,(ϵ_t) of 0.004.

For temperature and shrinkage reinforcement ratio:

According to *ACI Code 7.12.2.1*

Slabs with Grade 40 or 50 deformed bars………………………………………… 0.0020

Slabs with Grade 60 deformed bars ……………………………………………… 0.0018

Slabs where reinforcement with yield strength

Exceeding 60000 psi..……………………………………………………… $\frac{0.0018\times 60000}{f_y}$

For flexural reinforcement:

According to *ACI Code 10.5.4,*

The minimum flexural reinforcement ratio is not to be less than the shrinkage reinforcement.

1.2 DESIGN EXAMPLE

Design the slab following the provisions of the ACI code.

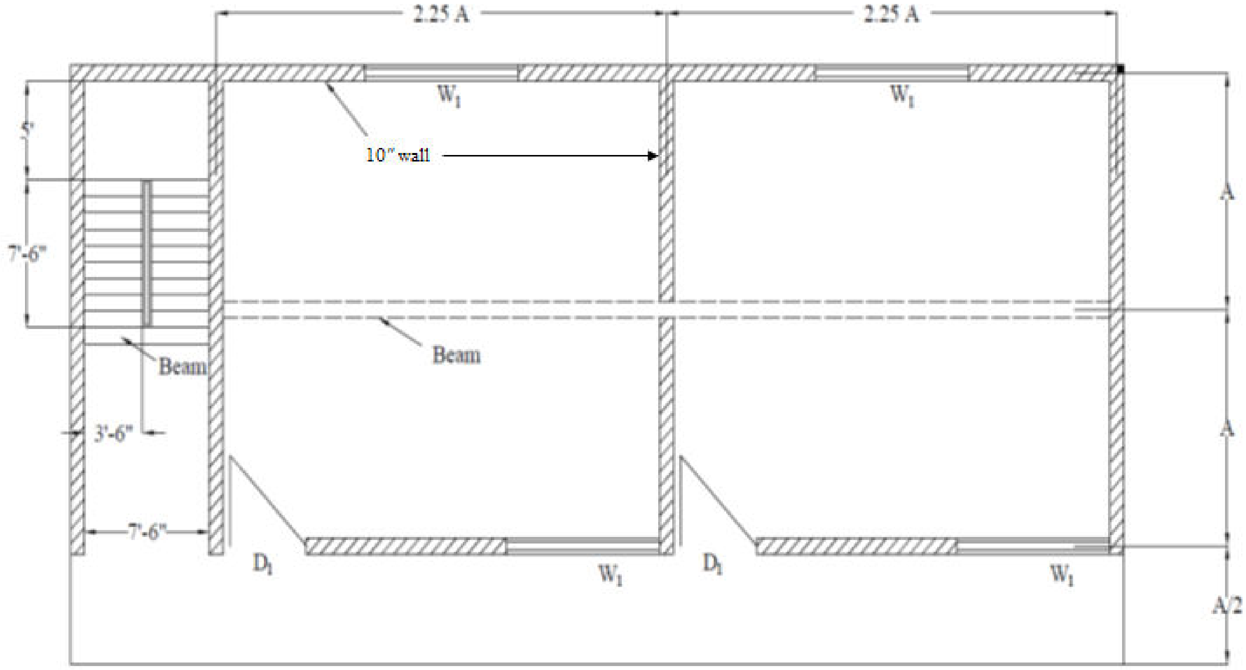


Figure 3: Typical Layout of Load Bearing Building

Design Data

Dimension, A	12 ft
Ultimate Strength of Concrete, f_c'	3 ksi
Yield Strength of Steel, f_y	40 ksi
Live Load on Floor	60 psf
Floor Finish (FF)	25 psf

THICKNESS ESTIMATION

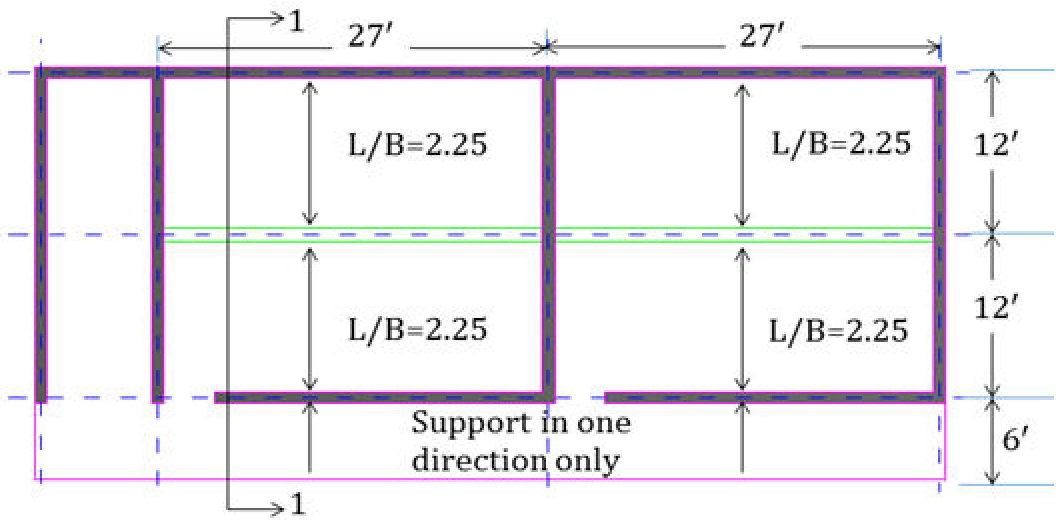


Figure 4: Load Transfer in Shorter Direction

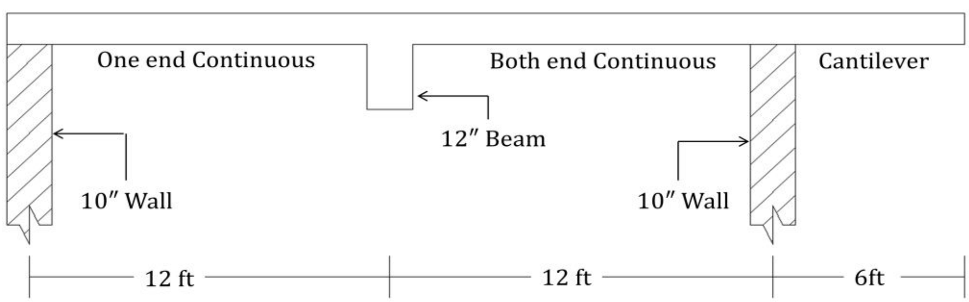


Figure 5: Section 1-1

- For one end continuous

$$l = 12 \times 12 - \frac{10}{2} - \frac{12}{2} = 133''$$

$$\text{Multiplying factor, MF} = 0.4 + \frac{40}{100} = 0.8$$

$$\text{Thickness} = \frac{l}{24} \times 0.8 = 4.43''$$

- For both end continuous

$$l = 12 \times 12 - \frac{10}{2} - \frac{12}{2} = 133''$$

$$\text{Multiplying factor, MF} = 0.4 + \frac{40}{100} = 0.8$$

$$\text{Thickness} = \frac{l}{28} \times 0.8 = 3.80''$$

- For Cantilever

$$l = 6 \times 12 - \frac{10}{2} = 67''$$

$$\text{Multiplying factor, MF} = 0.4 + \frac{40}{100} = 0.8$$

$$\text{Thickness} = \frac{l}{10} \times 0.8 = 5.36''$$

Among the thickness maximum thickness = 5.36″ Hence use 5.5″

DETERMINING LOADS

- Consider only a 1 ft width of beam.
- Floor Finish = 25 psf
- Live load = 60 psf
- Dead load = 150×5.50/12=68.75 psf (Unit wt. of Concrete = 150 pcf)
- Factored DL and LL, w_u =[(68.75+25)x1.2+60 x1.6] psf

  = 208.5 psf

  = 0.21 ksf

DETERMINING MAXIMUM MOMENTS

Moment coefficients at critical sections by ACI code:

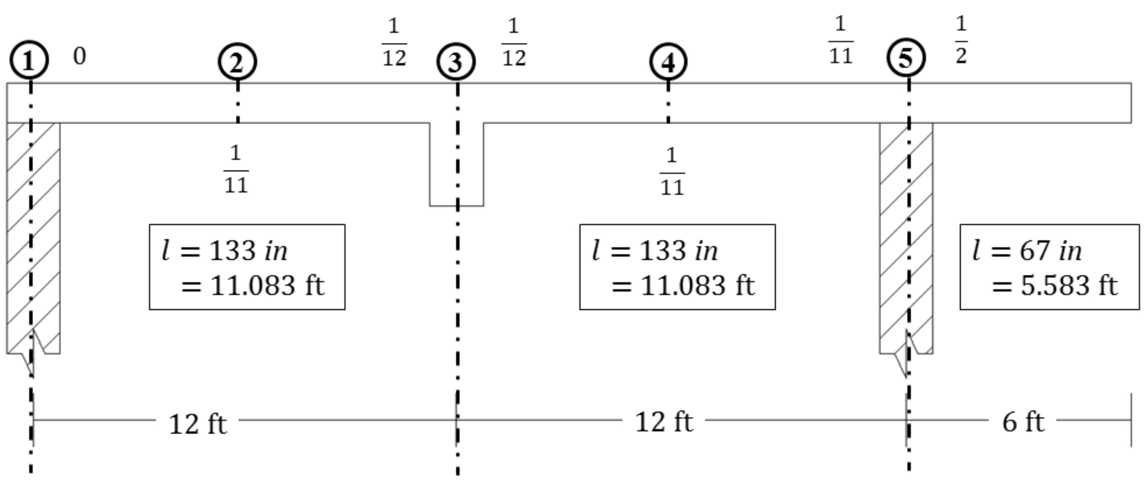


Figure 6: Moment Coefficient

- At Section 1 : $M_u = -0 \times w_u l^2 = -0\ k - ft/ft = -0\ k - in/ft$
- At Section 2 : $M_u = \frac{1}{11} \times w_u l^2 = 2.35\ k - ft/ft = 28.2\ k - in/ft$
- At Section 3 (Left) : $M_u = -\frac{1}{12} \times w_u l^2 = -2.15\ k - ft/ft = -25.8\ k - in/ft$

  At Section 3 (Right) : $M_u = -\frac{1}{12} \times w_u l^2 = -2.15\ k - ft/ft = -25.8\ k - in/ft$

  At Section 3 (Max.) : $M_u = -25.8\ k - in/ft$
- At Section 4 : $M_u = \frac{1}{11} \times w_u l^2 = 2.35\ k - ft/ft = 28.2\ k - in/ft$
- At Section 5 (Left) : $M_u = -\frac{1}{11} \times w_u l^2 = -2.35\ k - ft/ft = -28.2\ k - in/ft$

  At Section 5 (Right) : $M_u = -\frac{1}{2} \times w_u l^2 = -3.27\ k - ft/ft = -39.27\ k - in/ft$

  At Section 5 (Max.) : $M_u = -39.27\ k - in/ft$

Maximum Moment (Only Value) : $M_{max} = 39.27\ k - in/ft$ (For all Sections)

MINIMUM EFFECTIVE DEPTH

$$\rho = 0.85\beta_1 \frac{f_c'}{f_y} \frac{\epsilon_u}{\epsilon_u + \epsilon_t}$$

$$\rho = \rho_{0.005} = 0.85\beta_1 \frac{f_c'}{f_y} \frac{\epsilon_u}{\epsilon_u + 0.005}$$

$$\rho = \rho_{0.005} = 0.85 \times 0.85 \times \frac{3}{40} \times \frac{0.003}{0.003 + 0.005} = 0.0203$$

$$d_{req} = \sqrt{\frac{M_{max}}{\emptyset \rho f_y b \left(1 - 0.59\rho \frac{f_y}{f_c'}\right)}}$$

$$= \sqrt{\frac{39.27}{0.9 \times 0.0203 \times 40 \times 12 \times \left(1 - 0.59 \times 0.0203 \times \frac{40}{3}\right)}}$$

$$= 2.31\ in$$

CHECKING AVAILABILITY OF THICKNESS

As required effective depth, d_{req} is less than provided effective depth, $d_{prov.}$ of (5.50-1.00) = 4.50 in, the thickness of 5.50 in can be adopted.

MINIMUM REINFORCEMENT AREA

$$A_{s,min} = 0.002\,bt = (0.002 \times 12 \times 5.5)\ in^2/ft$$

$$= 0.13\ in^2/ft$$

Minimum reinforcement of 0.13 in^2/ft should be provided at every section.

REINFORCEMENT AREA DETERMINATION AT ALL SECTIONS

$$A_s = \frac{M_u}{\emptyset f_y (d - a/2)} \quad \& \qquad\qquad a = \frac{A_s f_y}{0.85 f_c' b}$$

- **At Section 1: (M_u = 0 k-in/ft)**

Let,

a = 1 in

$$A_s = \frac{M_u}{\emptyset f_y (d - a/2)}$$

$$= \frac{0}{0.9 \times 40(4.5 - 1/2)}$$

$$= 0\ in^2/ft$$

$and,$

$$a = \frac{A_s f_y}{0.85 f_c' b}$$

$$a = \frac{0 \times 40}{0.85 \times 3 \times 12}$$

$$= 0\ in$$

Again,

$$A_s = \frac{M_u}{\emptyset f_y (d - a/2)}$$

$$= \frac{0}{0.9 \times 40(4.5 - 0/2)}$$

$= 0\ in^2/ft$ (Same as before)(But $A_s < A_{s,min}$)

$= 0.013\ in^2/ft$

- **At Section 2: (M_u = 28.2 k-in/ft)**

Let,

a = 1 in

$$A_s = \frac{M_u}{\emptyset f_y(d - a/2)}$$

$$= \frac{28.2}{0.9 \times 40(4.5 - 1/2)}$$

$$= 0.2\ in^2/ft$$

$and,$

$$a = \frac{A_s f_y}{0.85 f_c' b}$$

$$a = \frac{0.2 \times 40}{0.85 \times 3 \times 12}$$

$$= 0.26\ in$$

Again,

$$A_s = \frac{M_u}{\emptyset f_y(d - a/2)}$$

$$= \frac{28.2}{0.9 \times 40(4.5 - 0.26/2)}$$

$$= 0.18\ in^2/ft$$

$and,$

$$a = \frac{A_s f_y}{0.85 f_c' b}$$

$$a = \frac{0.18 \times 40}{0.85 \times 3 \times 12}$$

$$= 0.24\ in$$

Again,

$$A_s = \frac{M_u}{\emptyset f_y(d - a/2)}$$

$$= \frac{28.2}{0.9 \times 40(4.5 - 0.24/2)}$$

$= 0.18\ in^2/ft$ (Same as before)(Here, A_s > $A_{s,min}$), OK

- **At Section 3: (M_u = 25.8 k-in/ft)**

Let,

$a = 1$ in

$$A_s = \frac{M_u}{\emptyset f_y(d - a/2)}$$

$$= \frac{25.8}{0.9 \times 40(4.5 - 1/2)}$$

$$= 0.18\ in^2/ft$$

$and,$

$$a = \frac{A_s f_y}{0.85 f_c' b}$$

$$a = \frac{0.18 \times 40}{0.85 \times 3 \times 12}$$

$$= 0.24\ in$$

Again,

$$A_s = \frac{M_u}{\emptyset f_y(d - a/2)}$$

$$= \frac{25.8}{0.9 \times 40(4.5 - 0.24/2)}$$

$$= 0.16\ in^2/ft$$

$and,$

$$a = \frac{A_s f_y}{0.85 f_c' b}$$

$$a = \frac{0.16 \times 40}{0.85 \times 3 \times 12}$$

$$= 0.21\ in$$

Again,

$$A_s = \frac{M_u}{\emptyset f_y(d - a/2)}$$

$$= \frac{25.8}{0.9 \times 40(4.5 - 0.21/2)}$$

$= 0.16\ in^2/ft$ (Same as before)(Here, $A_s > A_{s,min}$), OK

- **At Section 4: (M_u = 28.2 k-in/ft)**

Let,

$a = 1$ in

$$A_s = \frac{M_u}{\emptyset f_y(d - a/2)}$$

$$= \frac{28.2}{0.9 \times 40(4.5 - 1/2)}$$

$$= 0.2\ in^2/ft$$

$and,$

$$a = \frac{A_s f_y}{0.85 f_c' b}$$

$$a = \frac{0.2 \times 40}{0.85 \times 3 \times 12}$$

$$= 0.26\ in$$

Again,

$$A_s = \frac{M_u}{\emptyset f_y(d - a/2)}$$

$$= \frac{28.2}{0.9 \times 40(4.5 - 0.26/2)}$$

$$= 0.18\ in^2/ft$$

$and,$

$$a = \frac{A_s f_y}{0.85 f_c' b}$$

$$a = \frac{0.18 \times 40}{0.85 \times 3 \times 12}$$

$$= 0.24\ in$$

Again,

$$A_s = \frac{M_u}{\emptyset f_y(d - a/2)}$$

$$= \frac{28.2}{0.9 \times 40(4.5 - 0.24/2)}$$

$= 0.18\ in^2/ft$ (Same as before)(Here, $A_s > A_{s,min}$), OK

- **At Section 5: (M_u = 39.27 k-in/ft)**

Let,

a = 1 in

$$A_s = \frac{M_u}{\emptyset f_y(d - a/2)}$$

$$= \frac{39.27}{0.9 \times 40(4.5 - 1/2)}$$

$$= 0.27\ in^2/ft$$

and,

$$a = \frac{A_s f_y}{0.85 f_c' b}$$

$$a = \frac{0.27 \times 40}{0.85 \times 3 \times 12}$$

$$= 0.35\ in$$

Again,

$$A_s = \frac{M_u}{\emptyset f_y(d - a/2)}$$

$$= \frac{39.27}{0.9 \times 40(4.5 - 0.35/2)}$$

$$= 0.25\ in^2/ft$$

and,

$$a = \frac{A_s f_y}{0.85 f_c' b}$$

$$a = \frac{0.25 \times 40}{0.85 \times 3 \times 12}$$

$$= 0.33\ in$$

Again,

$$A_s = \frac{M_u}{\emptyset f_y(d - a/2)}$$

$$= \frac{39.27}{0.9 \times 40(4.5 - 0.33/2)}$$

$= 0.25\ in^2/ft$ (Same as before)(Here, $A_s > A_{s,min}$), OK

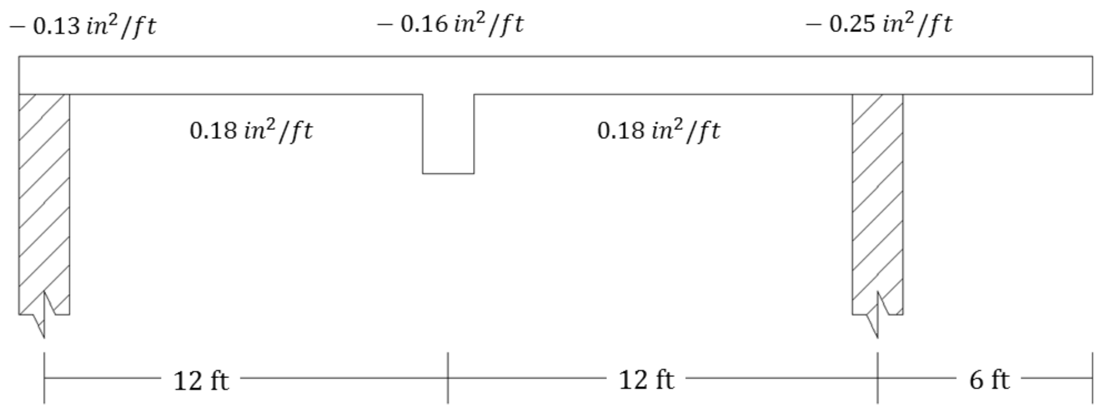


Figure 7: Reinforcement Area Required

REINFORCEMENT SPACING DETERMINATION

- **Short Direction:**

  $S_{min} = 1.5t = 1.5 \times 5.5 = 8.25"$
  $S_{max} = 3t\ or\ 18$=(3×5.5)" or 18" =16.5" or 18" = 16.5" ($Lower\ one$)

  $\boldsymbol{A_s = 0.18\ in^2/ft}$ **: (If #3 is provided) (At Section 2 & Section 4)**

  0.18 in^2 reinforcement required in 12 inch

  1 in^2 reinforcement required in $\frac{12}{0.18}$inch

  0.11 in^2 reinforcement required in $\frac{12\times0.11}{0.18}$inch

  $= 7.33" \approx 7"$ (0.5" $lower\ rounding$)

  But, $S < S_{min}$ (not Ok)

  $\boldsymbol{A_s = 0.18\ in^2/ft}$ **: (If #4 is provided) (At Section 2 & Section 4)**

  0.18 in^2 reinforcement required in 12 inch

  1 in^2 reinforcement required in $\frac{12}{0.18}$inch

  0.2 in^2 reinforcement required in $\frac{12\times0.2}{0.18}$inch

  $= 13.33" \approx 13"$ (0.5" $lower\ rounding$)

  Now, $S_{min} < S < S_{max}$ (Ok)

  $\therefore$ **#4 @13"** $\boldsymbol{c/c\ alt.ckd}$

  $\boldsymbol{A_s = 0.13\ in^2/ft}$ **: (At Section 1)**

  #4 @ 26" $c/c\ \ is\ already\ provided$
  In 26 inch already provided reinforcement area = 0.2 in^2

In 12 inch required reinforcement area = 0.13 in^2

In 1 inch required reinforcement area = $\frac{0.13 \times 26}{12}$ in^2

In 26 inch required reinforcement area = 0.28 in^2

Extra reinforcement required = (0.28-0.2) =0.08 in^2

No. of extra top bar required (if # 3 bar is used) = $\frac{0.08}{0.11}$= 0.73≈ 1

$$\mathbf{1\#3\ \textit{extra top between ckd. bar}}$$

$\boldsymbol{A_s = 0.16\ in^2/ft}$: **(At Section 3)**

$\#4\ @\ 26"\ c/c\ \ is\ already\ provided$
In 26 inch already provided reinforcement area = 0.2 in^2

In 12 inch required reinforcement area = 0.16 in^2

In 1 inch required reinforcement area = $\frac{0.16 \times 26}{12}$ in^2

In 26 inch required reinforcement area = 0.35 in^2

Extra reinforcement required = (0.35-0.2) =0.15 in^2

No. of extra top bar required (if # 4 bar is used) = $\frac{0.15}{0.2}$= 0.75≈ 1

$$\mathbf{1\#4\ \textit{extra top between ckd. bar}}$$

$\boldsymbol{A_s = 0.25\ in^2/ft}$: **(At Section 5)**

$\#4\ @\ 26"\ c/c\ \ is\ already\ provided$
In 26 inch already provided reinforcement area = 0.2 in^2

In 12 inch required reinforcement area = 0.25 in^2

In 1 inch required reinforcement area = $\frac{0.25 \times 26}{12}$ in^2

In 26 inch required reinforcement area = 0.54 in^2

Extra reinforcement required = (0.54-0.2) =0.34 in^2

No. of extra top bar required (if # 4 bar is used) = $\frac{0.34}{0.2}$= 1.70≈2

$$\mathbf{2\#4\ \textit{extra top between ckd. bar}}$$

- **Long Direction:**

  $S_{min} = 1.5t = 1.5 \times 5.5 = 8.25''$

  $S_{max} = 5t \; or \; 18 = (5\times5.5)'' \; or \; 18'' = 27.5'' \; or \; 18'' = 18'' \; (Lower \; one)$

  $A_{s,min} = 0.13 \; in^2/ft$

  $\boldsymbol{A_s = 0.13 \; in^2/ft}$ **:**

  0.13 in^2 reinforcement required in 12 inch

  1 in^2 reinforcement required in $\frac{12}{0.13}$inch

  0.11 in^2 reinforcement required in $\frac{12\times0.11}{0.13}$inch

  $= 10.15'' \approx 10'' \; (0.5'' \; lower \; rounding)$

  Now, $S_{min} < S < S_{max}$ (Ok)

  $\therefore \; \boldsymbol{\#3 \; @10'' \; c/c}$

DETAILING:

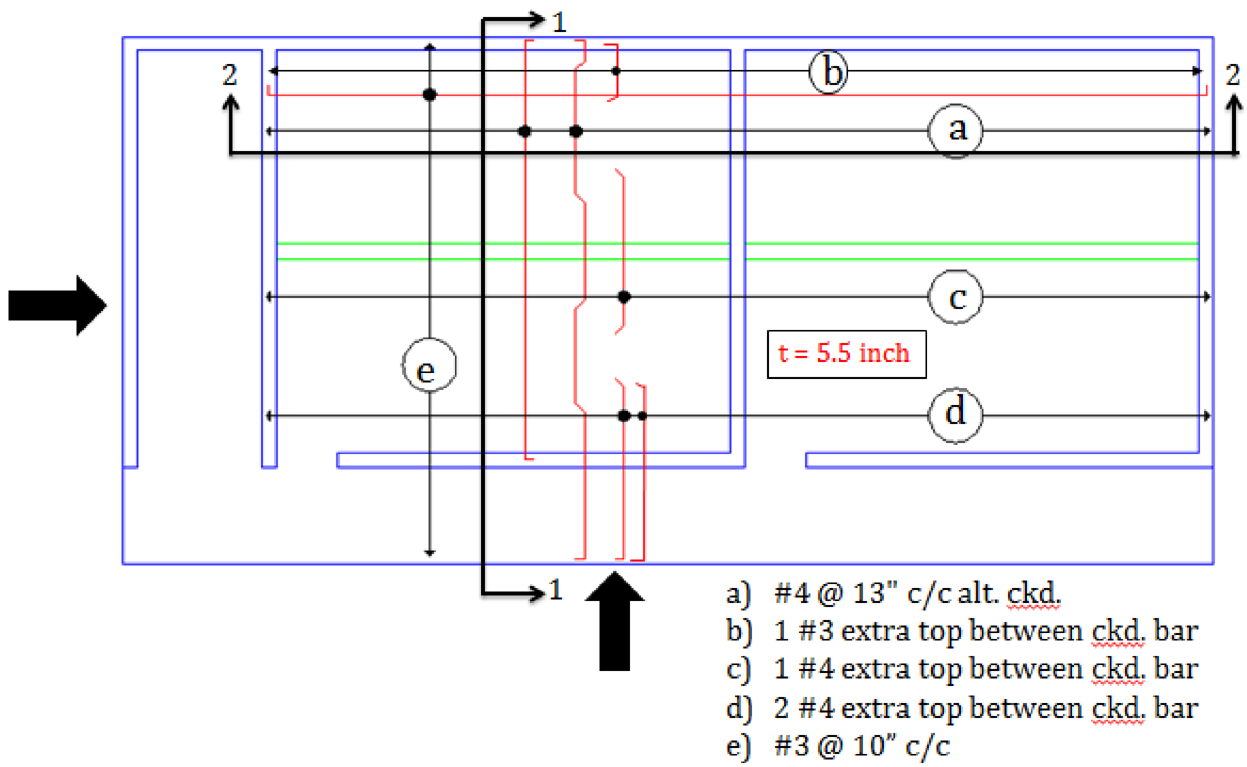


Figure 8: Detailing at Plan

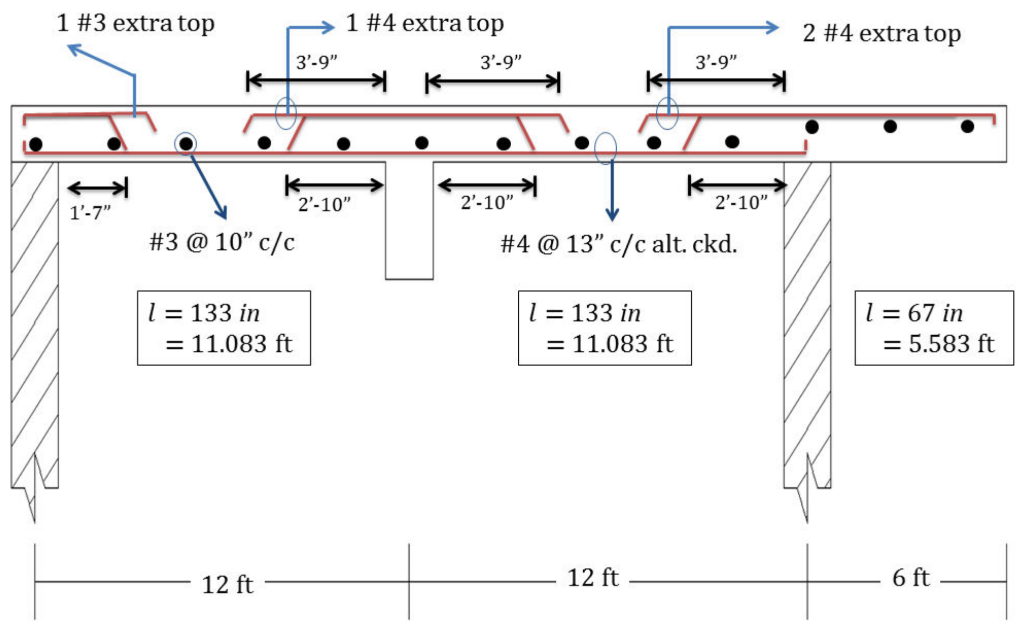


Figure 9: Detailing at Section 1-1

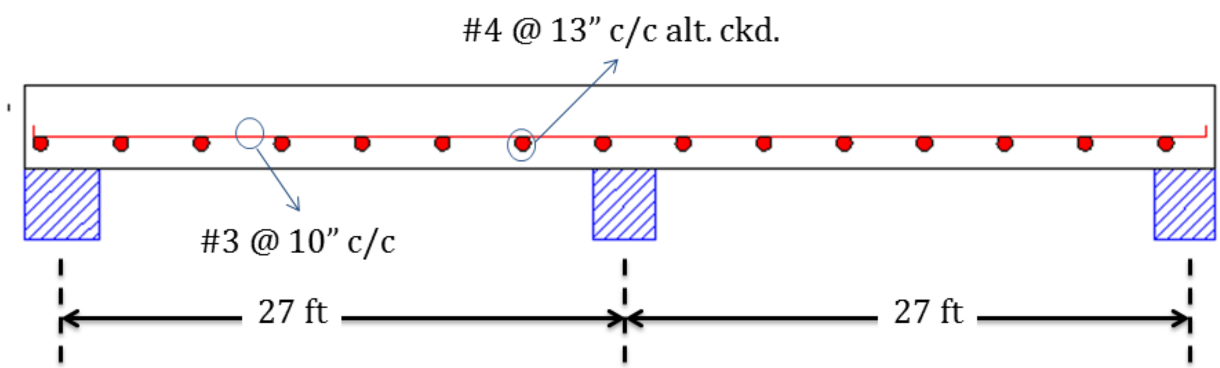


Figure 10: Detailing at Section 2-2

2. DESIGN OF BEAM (USD)

Introduction:

Ultimate Strength Design (USD)

- Based on the ultimate strength of the structure, assuming a failure condition either due to concrete crushing or by yielding of steel. Additional strength of steel due to strain hardening is not encountered in the analysis or design.
- Actual / working loads are multiplied by load factor to obtain the design loads.
- ACI codes emphasizes this method.

Assumptions:

There are five assumption that are made

1. Plane sections before bending remain plane after bending.

2. Strain in concrete is the same as in reinforcing bars at the same level, provided that the bond between the steel and concrete is sufficient to keep them acting together under the different load stages i.e., no slip can occur between the two materials.

3. The stress-strain curves for the steel and concrete are known.

4. The tensile strength of concrete may be neglected.

5. At ultimate strength, the maximum strain at the extreme compression fiber is assumed equal to 0.003

Design and Analysis

The main task of a structural engineer is the analysis and design of structures. The two approaches of design and analysis will be used.

Design of a section:

This implies that the external ultimate moment is known, and it is required to compute the dimensions of an adequate concrete section and the amount of steel reinforcement. Concrete strength and yield of steel used are given.

Analysis of a section:

This implies that the dimensions and steel used in the section (in addition to concrete and steel yield strengths) are given, and it is required to calculate the internal ultimate moment capacity of the section so that it can be compared with the applied external ultimate moment.

Beam Types

- **Singly reinforced section**
- **Doubly reinforced section**

Flexure Équations

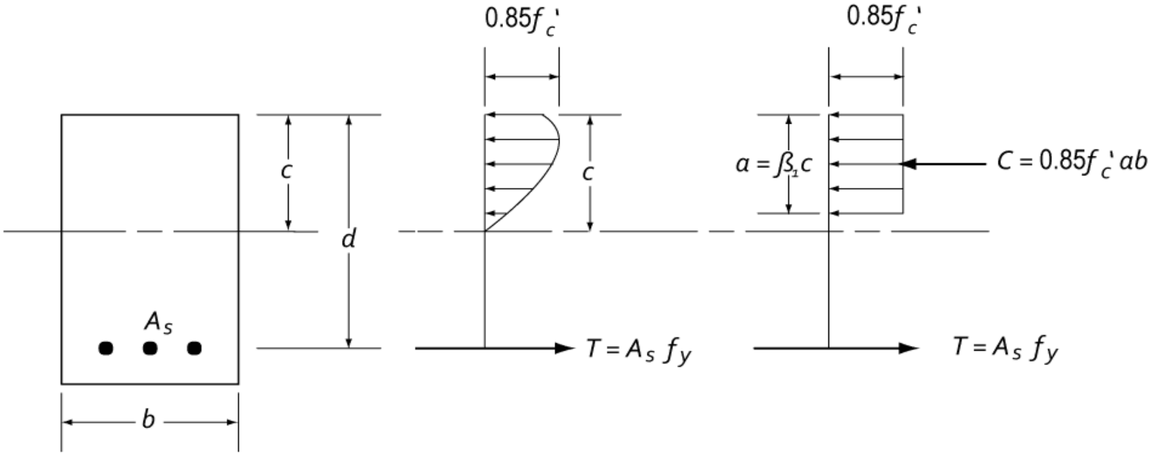


Figure 12: Diagrams for flexure equations

$$C = T$$

$$0.85 f_c' ab = A_s f_y$$

solving for a,

$$a = \frac{A_s f_y}{0.85 f_c' b} = \frac{\rho f_y d}{0.85 f_c'}$$

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

$$M_u = \phi M_n$$

$$M_u = \phi M_n = \phi A_s f_y \left(d - \frac{a}{2}\right) \qquad M_u = \phi A_s f_y d\left(1 - 0.59\frac{\rho f_y}{f_c'}\right)$$

$$\rho = \frac{A_s}{bd}$$

FAILURE MODES

- No Reinforcing
    - Brittle failure
- Reinforcing < balance
    - Steel yields before concrete fails
    - ductile failure
- Reinforcing = balance
    - Concrete fails just as steel yields
- Reinforcing > balance
    - Concrete fails before steel yields

$$\rho_{\min} = \frac{200}{f_y}$$

$$\rho_{\max} = 0.75\rho_{bal}$$

$$\rho_{bal} = \left(\frac{0.85\beta_1 f_c'}{f_y}\right)\left(\frac{87000}{87000 + f_y}\right)$$

$$\rho > \rho_{\max}$$

DESIGN EXAMPLE OF BEAMS

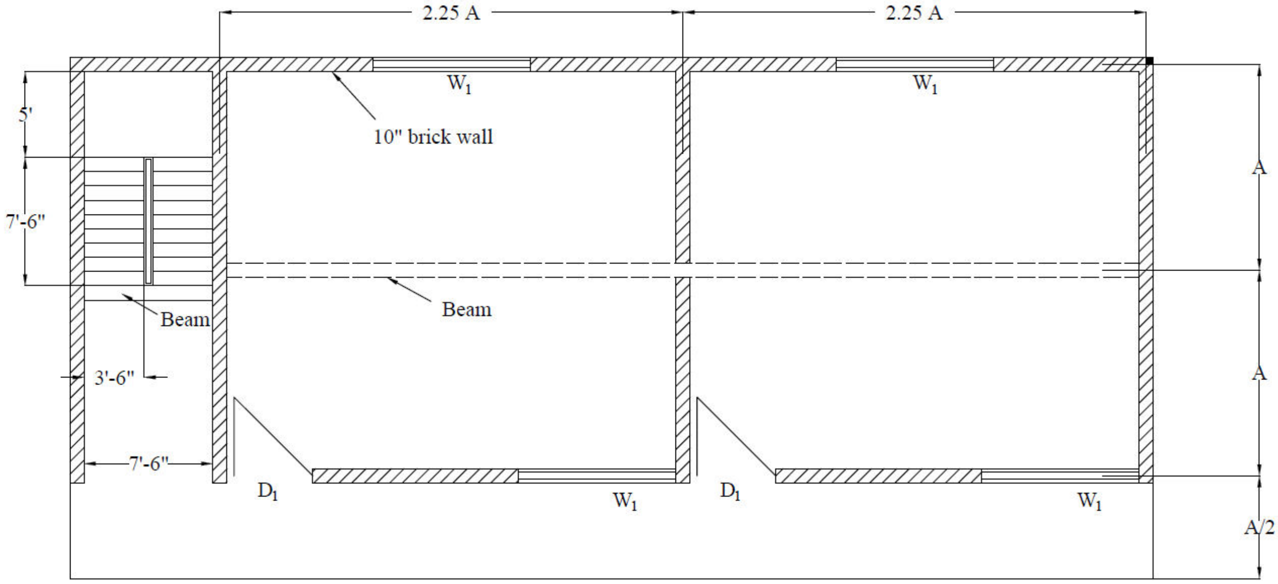


Figure 12: Beam Layout

Load Calculation for Beam Design

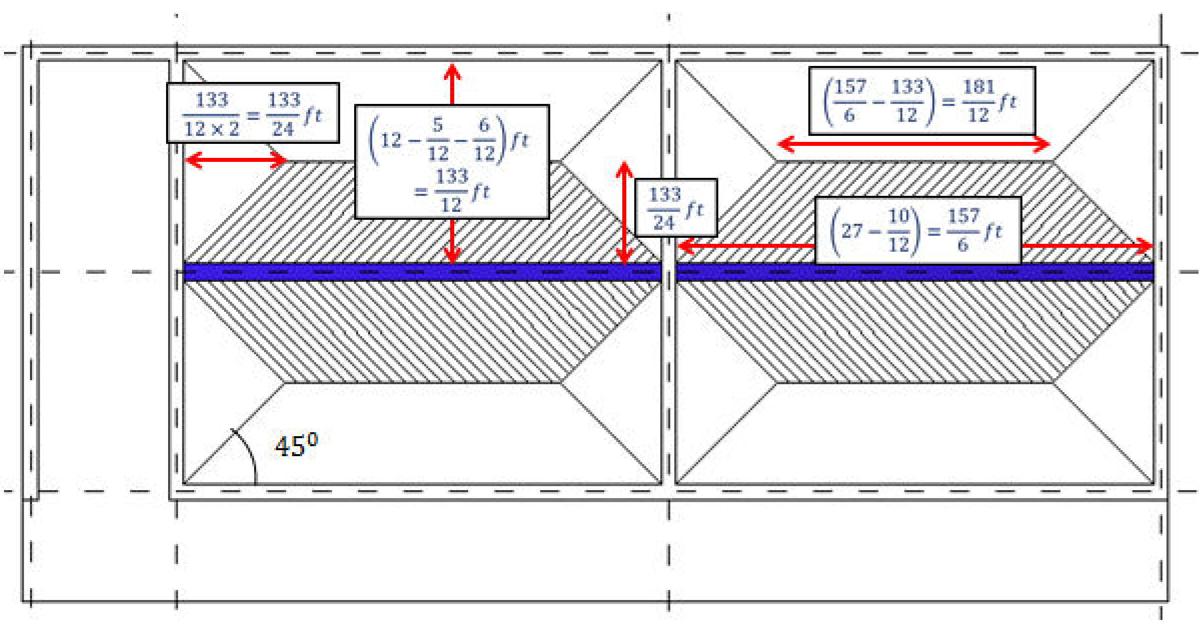


Figure 13: Determination of Contributing Area for Load Calculation

$$Controlling\ Area = 2 \times \frac{1}{2} \times \left(\frac{181}{12} + \frac{157}{6}\right) \times \frac{133}{24} = \frac{7315}{32}\ ft^2$$

Step 02 : Determination of Load

Load from Slab, $w_u = 0.21\ ksf$ (factored)

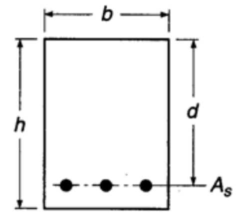
Load on Beam from Slab, $w_{u,slab} = (0.21\ ksf) \times ContributingArea/\text{Length}$

$$= (0.21\ ksf) \times \left(\frac{7315}{32} ft^2\right) \Big/ \left(\frac{157}{6} ft\right)$$

$$= 1.835\ kip/ft$$

$$\approx 1.84\ kip/ft\ (Factored)$$

Let, Beam Cross section = 12" × 24"(Assumption, h = 2b)



Self Weight of Beam, $w_{u,self\,wt.} = \left(\frac{12\times24}{144} \times 0.150\right) kip/ft$

$$= 0.3\ kip/ft(Unfactored)$$

Total Load on Beam, $w_u = (1.84 + 0.3 \times 1.2) kip/ft = \mathbf{2.2\ kip/ft}$

Bending Moment and Shear Force Diagram for Beam Design

- **Reactions at supports: (Assume the beam to have the following support conditions)**

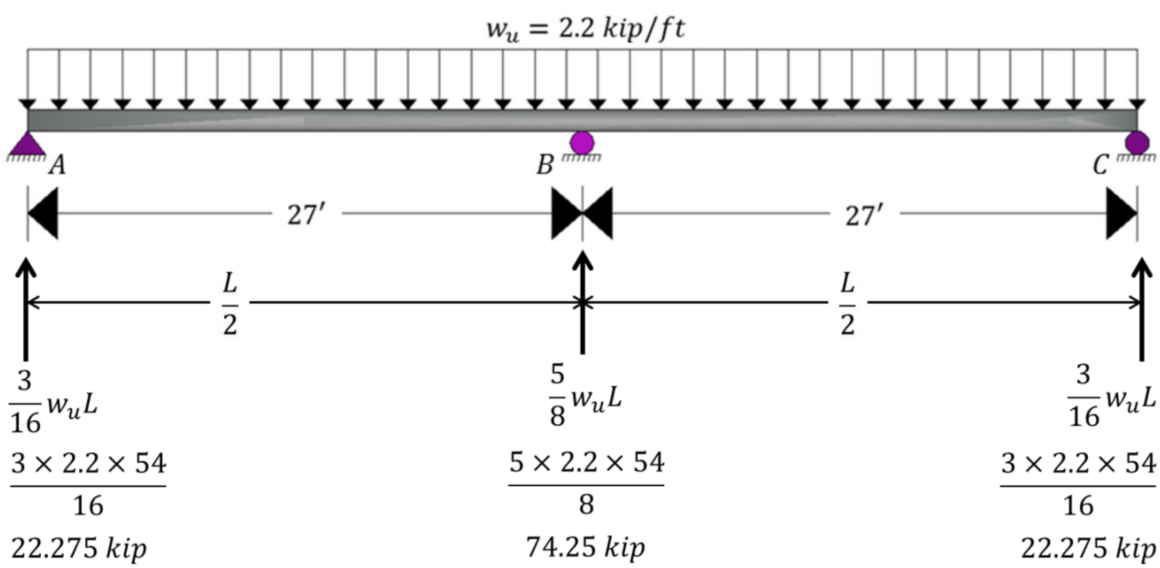


Figure 14: Determination Support Reactions

- **SFD and BMD**

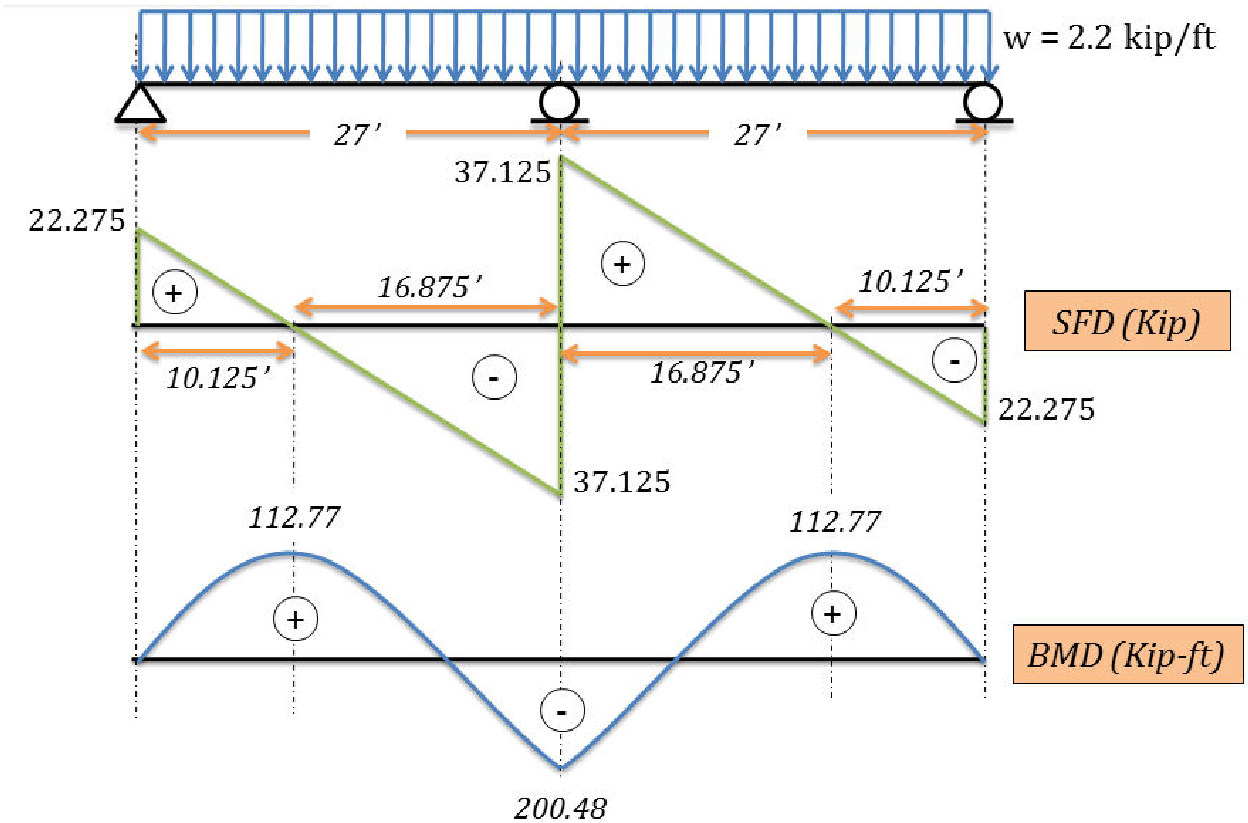


Figure 15: Shear Force Diagram and Bending Moment Diagram

"d" check for Maximum Moment

Maximum Moment, $M_{u,max}$ = 200.48 kip-ft = 2405.76 kip-in

$$\rho = \rho_{0.005} = 0.85\beta_1 \frac{f_c'}{f_y}\frac{\epsilon_u}{\epsilon_u + 0.005}$$
$$= 0.85 \times 0.85 \times \frac{3}{40} \times \frac{3}{8}$$
$$= 0.0203$$

$$\emptyset = 0.4833 + 83.33\epsilon_t$$
$$At,\ \epsilon_t = 0.005$$
$$\emptyset = 0.4833 + 83.33 \times 0.005$$
$$= 0.9$$

$$d^2 = \frac{M_{u,max}}{\emptyset \rho f_y b\left(1 - 0.59\rho\frac{f_y}{f_c'}\right)}$$

$$d_{req} = \sqrt{\frac{M_{u,max}}{\emptyset \rho f_y b\left(1 - 0.59\rho\frac{f_y}{f_c'}\right)}} = \sqrt{\frac{2405.76}{0.9 \times 0.0203 \times 40 \times 12\left(1 - 0.59 \times 0.0203 \times \frac{40}{3}\right)}}$$

$$d_{req} = 18.07\ inch$$

$$\boxed{d_{provided} = (h - 2.5)\ in = (24 - 2.5)\ in = 21.5"}$$

$$\boxed{d_{provided} > d_{required}} \longrightarrow \boxed{Section\ is\ ok}$$

FLEXURAL REINFORCEMENT AREA DETERMINATION:

$$A_{s,min} = max \begin{cases} \dfrac{3\sqrt{f_c'(psi)}}{f_y(psi)}bd = \dfrac{3\sqrt{3000}}{40000} \times 12 \times 21.5 = 1.06\ in^2 \\ \dfrac{200}{f_y(psi)}bd = \dfrac{200}{40000} \times 12 \times 21.5 = 1.29\ in^2 \end{cases} = 1.29\ in^2$$

$$A_s = \frac{M_u}{\phi f_y(d - a/2)} \quad \& \quad a = \frac{A_s f_y}{0.85 f_c' b}$$

Assume a, Get A_s; Check a with the newA_s; Continue this process until a converges

$\boxed{M_u = 112.77\ kip - ft:}$

$$A_s = \frac{112.77 \times 12}{0.9 \times 40 \times (21.5 - \frac{2.422}{2})} = 1.85\ in^2 (\text{Controls}) \qquad a = 2.422\ in$$

$\boxed{M_u = 200.48\ kip - ft:}$

$$A_s = \frac{200.48 \times 12}{0.9 \times 40 \times (21.5 - \frac{4.543}{2})} = 3.48\ in^2 (Controls) \qquad a = 4.543\ in$$

$$A_s = 1.85\ in^2\ (2\#6\ \&\ 1\#9)$$

$$A_s = 3.48\ in^2\ (2\#10\ \&\ 1\#9)$$

SHEAR REINFORCEMENT DETERMINATION

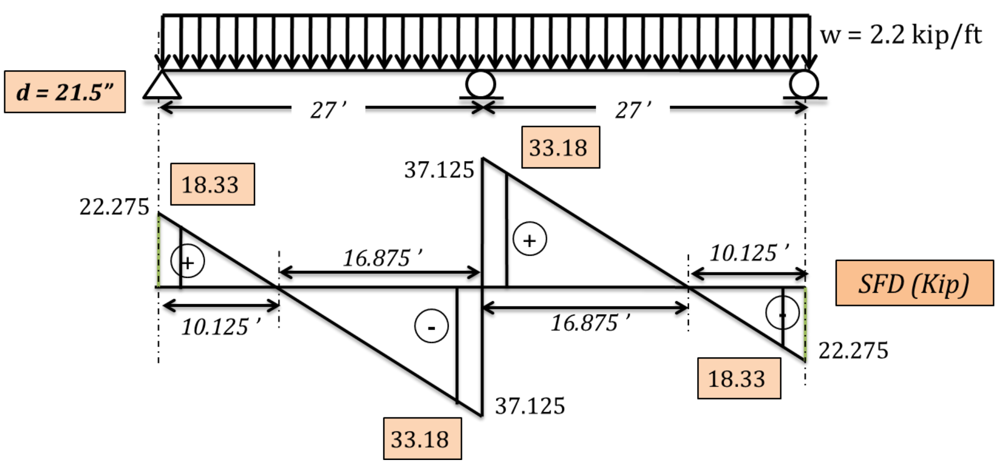


Figure 16: Shear Force Diagram of Beam

$$V_c = 2\lambda\sqrt{f_c'}\,bd(f_c'\,in\,psi)$$
$$= 2 \times 1 \times \sqrt{3000} \times 12 \times 21.5$$
$$= 28262.48\ lb$$
$$= 28.26\ kip$$

$$\boxed{\emptyset = 0.75}$$

$$\emptyset V_c = (0.75 \times 28.26)\ kip$$
$$= 21.20\ kip$$

$$\boxed{\frac{\emptyset V_c}{2} = \frac{21.2}{2} = 10.6\ kip}$$

if,

$V_u > \emptyset V_c$; *then reinforcements are required as per requirement (By formula)*

$\frac{\emptyset V_c}{2} \le V_u \le \emptyset V_c$; *then minimum reinforcements are required (maximum spacing)*

$V_u < \frac{\emptyset V_c}{2}$; *then no reinforcements are required*
but in construction minimum reinforcements are provided though

From Shear Force Diagram (SFD)

$V_u = 33.18\ kip$

$\emptyset V_c = 21.2\ kip$

$\frac{\emptyset V_c}{2} = 10.6\ kip$

$V_u > \emptyset V_c$

Reinforcements requirement are determined by formula

$V_u = 18.33\ kip$

$\emptyset V_c = 21.2\ kip$

$\frac{\emptyset V_c}{2} = 10.6\ kip$

$\frac{\emptyset V_c}{2} \le V_u \le \emptyset V_c$

Minimum reinforcements are required

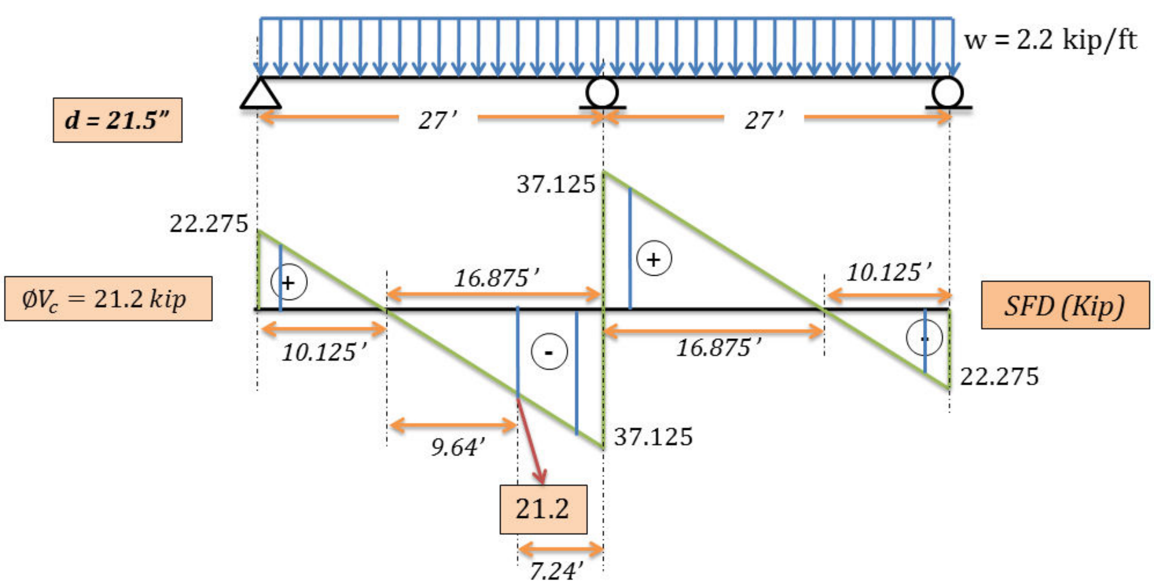


Figure 17: Distance determination where $V_u = \emptyset V_c$

For 7.24 ft Zone

$$if\ we\ use\ \#3\ bar \longrightarrow s = \frac{\emptyset A_v f_{yt} d}{V_u - \emptyset V_c} = \frac{0.75 \times 0.22 \times 40 \times 21.5}{33.18 - 21.2} = 11.85 \approx 11.5"$$

$$s > s_{max} \quad \therefore s = 10.5"$$

For Minimum Reinforcement Zone

$$if\ we\ use\ \#3\ bar$$

$$s_{max} = min \begin{cases} \dfrac{A_v f_{yt}}{0.75\sqrt{f_c} b} = \dfrac{0.22 \times 40000}{0.75 \times \sqrt{3000} \times 12} = 17.85" \\ \dfrac{A_v f_{yt}}{50b} = \dfrac{0.22 \times 40000}{50 \times 12} = 14.67" \\ \dfrac{d}{2} = \dfrac{21.5}{2} = 10.75" \\ 24" \end{cases} = 10.75 \approx 10.5"$$

Althrough 10.5" Spacing should be provided

$$Reinforcement\ providing\ start\ at\ \frac{10.5}{2} = 5"\ Spacing\ from\ the\ face\ of\ support$$

Detailing of Beam Reinforcement

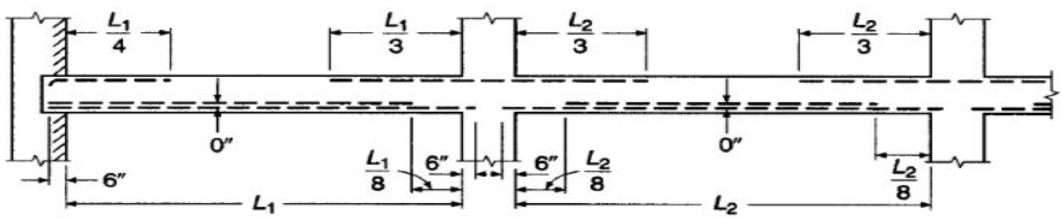


Figure 18: Beam Reinforcement Detailing Guideline

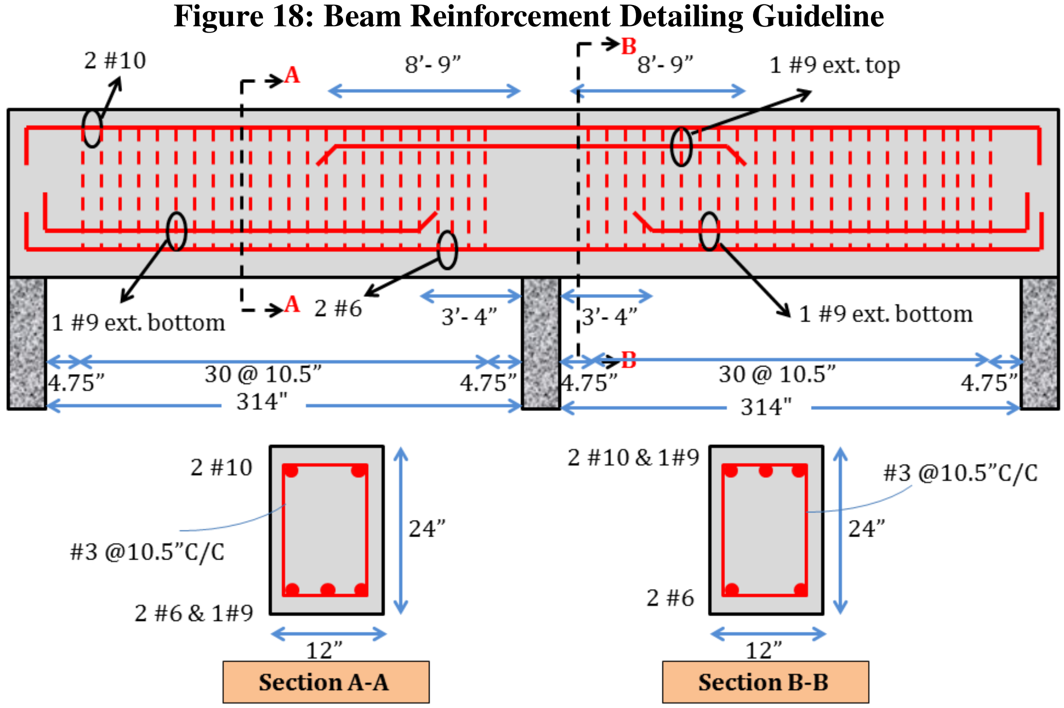


Figure 19: Beam Reinforcement Detailing According to the Design

3. DESIGN OF STAIR (USD)

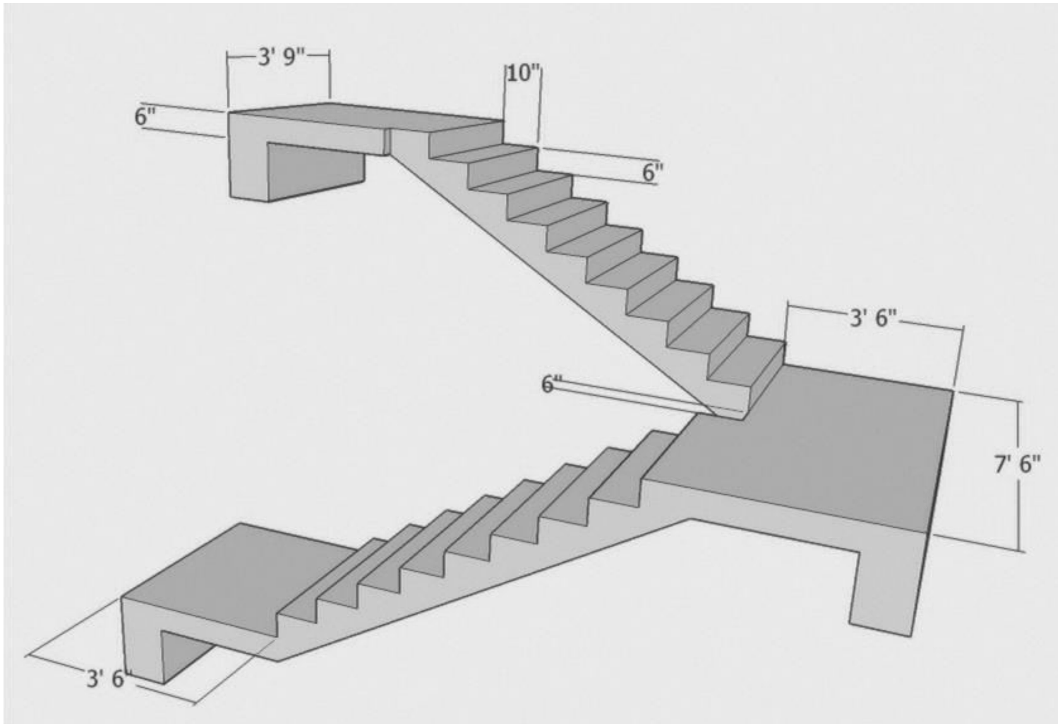


Figure 20: Stair

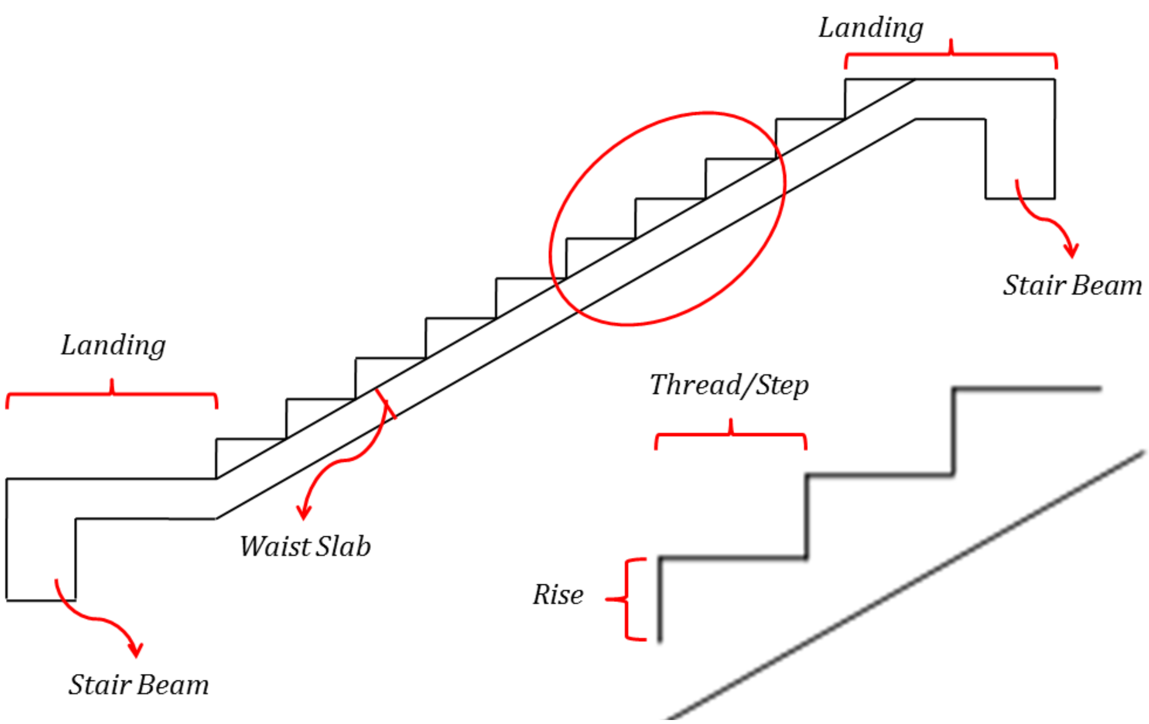
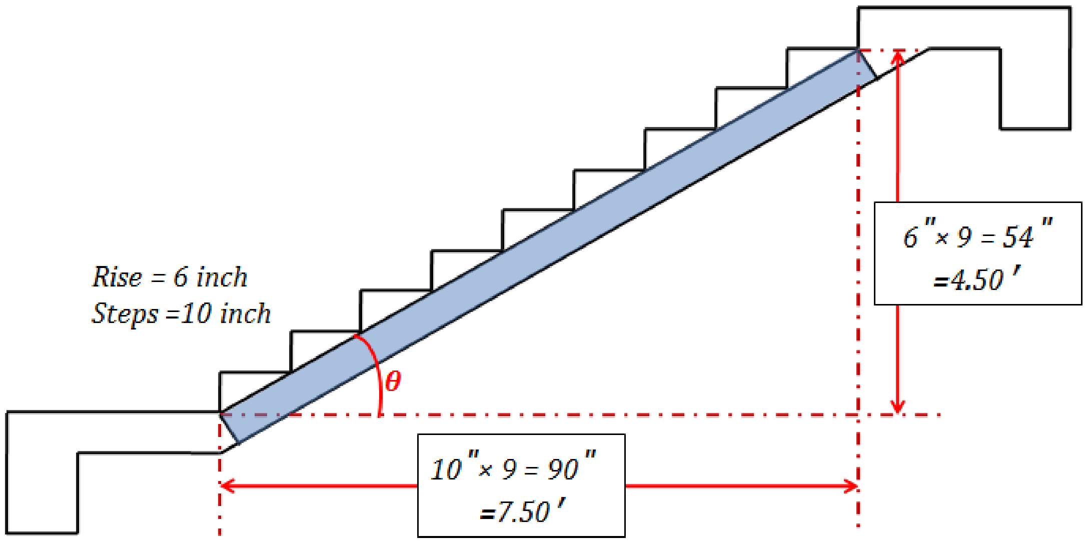


Figure 21: Components of Stair



$$\theta = \tan^{-1}\frac{4.5}{7.5} \text{ or } \tan^{-1}\frac{6}{10} = 30.964^\circ$$

Figure 22: Parameters of Stair

Step 01 : Determination of Load

Dead Load :

1. ***Waist Slab :*** *(Slab Thickness = 6 inch)*

$$\text{Waist Slab (Self Weight)} = \frac{\frac{6}{12}\times 150}{1000 \cos 30.964^0} = 0.088 \text{ Ksf}$$

2. ***Rises & Steps : (2.57″ equivalent thickness)***

$$\text{Rises \& Steps} = \frac{\frac{2.57}{12}\times 150}{1000 \cos 30.964^0} = 0.038 \text{ Ksf}$$

3. ***Railing :*** *(3 inch Thickness & 3 ft height)*

$$\text{Railing} = \frac{\frac{3}{12}\times 3 \times\sqrt{7.5^2+4.5^2}\times 150}{1000 \times 3.5 \times 7.5} = 0.038 \text{ Ksf}$$

4. ***Floor Finish :***

Floor Finish = 25 psf = 0.025 ksf

Live Load :

Live Load = 100 psf = 0.1 ksf

Total Load = 1.2DL+1.6LL
=1.2×(0.088+0.038+0.038+0.025)+1.6×0.1
=0.387 Ksf

Step 02 : Moment Calculation

Length, L = [(3.5×2)+7.5] ft = 14.5 ft

Positive Moment, $M(+ve) = \frac{wL^2}{14} = \frac{0.387 \times 14.5^2}{14} = 5.81 \text{ K} - \text{ft/ft}$

Negative Moment, $M(-ve) = \frac{wL^2}{9} = \frac{0.387 \times 14.5^2}{9} = 9.04 \text{ K} - \text{ft/ft}$

Step 03 : d check for maximum Moment

Maximum Moment, $M_{u,max}$ = 9.04 kip-ft/ft = 108.48 kip-in/ft

$$\rho = \rho_{0.005} = 0.85\beta_1 \frac{f_c'}{f_y} \frac{\epsilon_u}{\epsilon_u + 0.005} = 0.85 \times 0.85 \times \frac{3}{40} \times \frac{3}{8} = 0.0203$$

$$d_{req} = \sqrt{\frac{M_{u,max}}{\emptyset \rho f_y b \left(1 - 0.59\rho \frac{f_y}{f_c'}\right)}} = \sqrt{\frac{108.48}{0.9 \times 0.0203 \times 40 \times 12 \left(1 - 0.59 \times 0.0203 \times \frac{40}{3}\right)}}$$

$d_{req} = 3.84\ inch$ $\boxed{d_{provided} = (6-1)\ in = 5"}$ $\boxed{d_{provided} > d_{required}} \longrightarrow \boxed{ok}$

Step 04 : Reinforcement Area Determination

$\boxed{A_{s,min} = 0.002\ bt\ (As\ f_y = 40\ ksi) = 0.144\ in^2/ft}$ $\boxed{Here, b = 12\ in\ \&\ t = 6\ in}$

$$A_s = \frac{M_u}{\phi f_y (d - a/2)} \quad \& \quad a = \frac{A_s f_y}{0.85 f_c' b}$$

Load Transferring Direction :

$M(+ve) = 5.81\ kip - ft/ft \longrightarrow A_s(+ve) = 0.41\ in^2/ft\ (Controlled)$

If we use # 6 reinforcement then spacing, s $= \frac{12}{0.41} \times 0.44 = 12.88" \approx 12.5"$

$\boxed{\therefore \#6\ @\ 12.5"\ c/c\ alt.\ ckd}$

$M(-ve) = 9.04\ kip - ft/ft \longrightarrow A_s(-ve) = 0.66\ in^2/ft\ (Controlled)$

In 26" total requirement $= \frac{0.66}{12} \times 25 = 1.38\ in^2$ $\boxed{0.44 in^2\ already\ provided}$

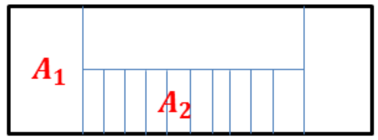
Extra top $= \frac{1.38 - 0.44}{0.44} = 2.14 \approx 3$

Other Direction :

$A_s = 0.144\ in^2/ft$

If we use # 3 reinforcement then spacing, s $= \frac{12}{0.144} \times 0.11 = 9.17" \approx 9"$

Step 05 : Stair Beam Design



$$Area = A_1 + A_2 = 7.5 \times 3.5 + 7.5 \times 3.5 = 52.5\,ft^2$$

Assume, Beam Size = 10”×12”

$d_{eff} = (12 - 2.5)\ inch = 9.5\ inch$

$$Self\ Weight = \frac{10}{12} \times \frac{12}{12} \times 0.150 = 0.125\ kip/ft (Unfactored)$$

$$Load\ on\ Stair\ beam\ from\ stair = \frac{Area \times UDL}{Length} = \frac{52.5 \times 0.387}{7.5} = 2.71\ kip/ft (factored)$$

$$Total\ Load\ on\ Beam, w = 2.71 + 0.125 \times 1.2 = 2.86\ kip/ft$$

$$\text{Positive Moment}, M(+ve) = \frac{wL^2}{14} = \frac{2.86 \times 7.5^2}{14} = 11.49\ kip - ft$$

$$\text{Negative Moment}, M(-ve) = \frac{wL^2}{16} = \frac{2.86 \times 7.5^2}{16} = 10.05\ kip - ft$$

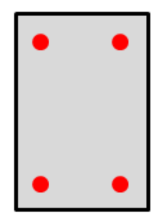
Maximum Moment, $M_{u,max}$ = 11.49 kip-ft = 137.88 kip-in

$$\rho = \rho_{0.005} = 0.85\beta_1 \frac{f_c'}{f_y} \frac{\epsilon_u}{\epsilon_u + 0.005} = 0.85 \times 0.85 \times \frac{3}{40} \times \frac{3}{8} = 0.0203$$

$$d_{req} = \sqrt{\frac{M_{u,max}}{\emptyset \rho f_y b \left(1 - 0.59\rho \frac{f_y}{f_c'}\right)}} = \sqrt{\frac{137.88}{0.9 \times 0.0203 \times 40 \times 10 \left(1 - 0.59 \times 0.0203 \times \frac{40}{3}\right)}}$$

$d_{req} = 4.74\ inch$ $\boxed{d_{provided} = (12 - 2.5)\ in = 9.5"}$ $\boxed{d_{provided} > d_{required}} \longrightarrow \boxed{ok}$

$$A_{s,min} = max \begin{cases} \frac{3\sqrt{f_c(psi)}}{f_y(psi)} bd = \frac{3\sqrt{3000}}{40000} \times 10 \times 12.5 = 0.51\ in^2 \\ \frac{200}{f_y(psi)} bd = \frac{200}{40000} \times 10 \times 12.5 = 0.625\ in^2 \end{cases} = 0.625\ in^2$$



$$\left.\begin{aligned} M(+ve) = 11.49\ kip - ft \longrightarrow A_s(+ve) = 0.42\ in^2 < A_{s,min} \\ M(-ve) = 10.05\ kip - ft \longrightarrow A_s(-ve) = 0.36\ in^2 < A_{s,min} \end{aligned}\right\} = 0.625\ in^2 (2\#6)$$

Step 06 : Detailing

$$\frac{L_o}{3} = 2.92 \approx 3'$$

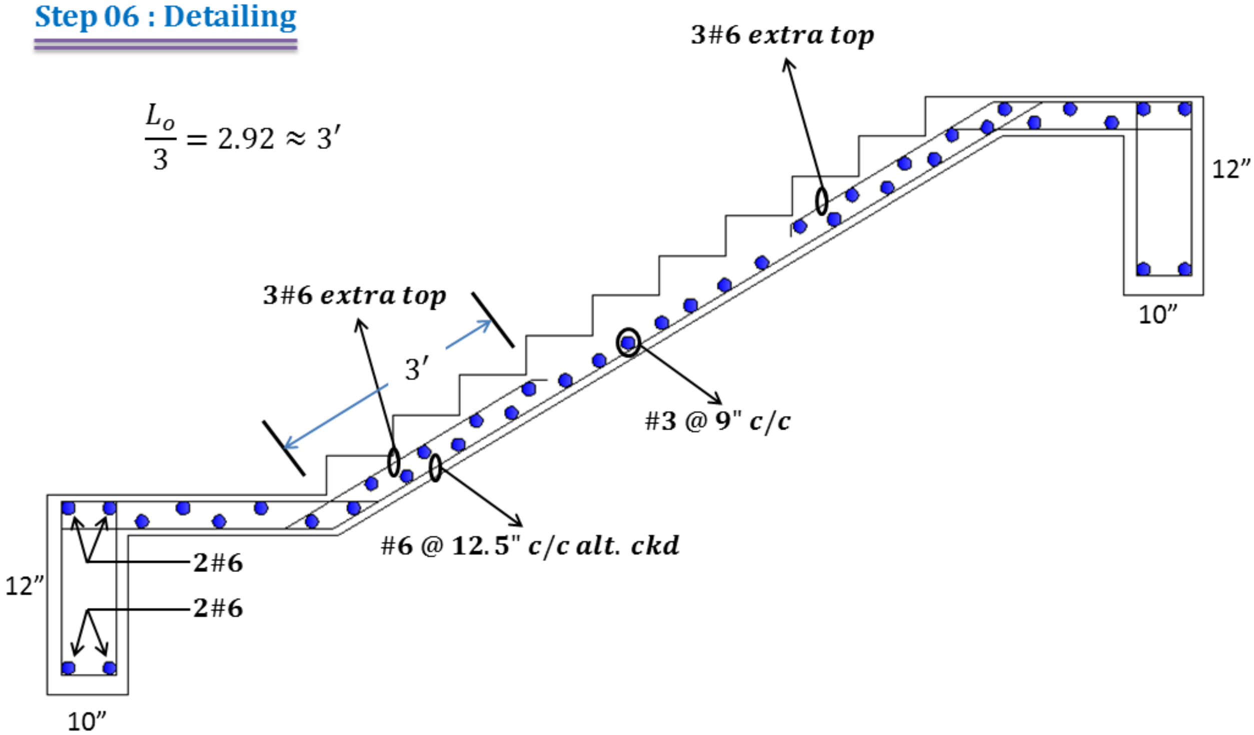


Figure 23: Reinforcement Details of stair.

4. DESIGN OF SUNSHADE (USD)

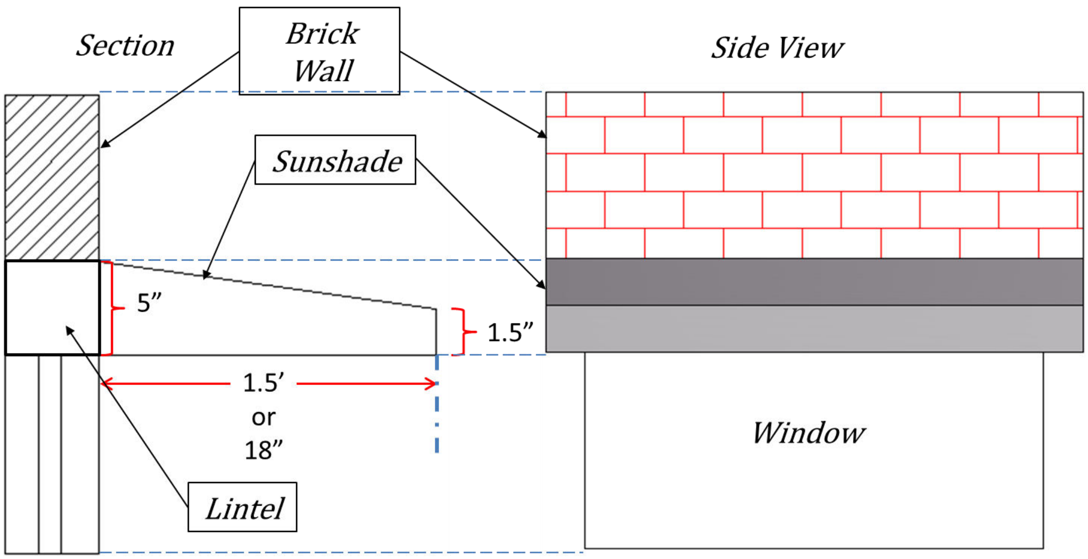


Figure 24: Section and Side View of a Wall with Window

Step 01 : Determination of Load

Dead Load :

1. ***Self Weight :***

$$Self\ Weight = \frac{1}{2} \times \frac{5+1.5}{12} \times 0.150 = 0.041\ ksf$$

2. ***Floor Finish :***

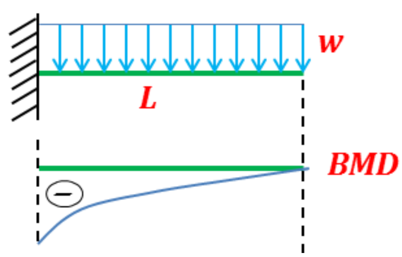
Floor Finish = 25 psf = 0.025 ksf

Live Load :

Live Load = 20 psf = 0.02 ksf

Total Load, w = 1.2DL+1.6LL
=1.2×(0.041+0.025)+1.6×0.02
=0.1112 Ksf

Step 02 : Determination of Moment :



$$M(-ve) = \frac{wL^2}{2} = \frac{0.1112 \times 1.5^2}{2} = 0.1251\ k\text{-}ft/ft$$

Step 03 : d check for maximum Moment

Maximum Moment, $M_{u,max}$ = 0.1251 kip-ft/ft = 1.5 kip-in/ft

$$\rho = \rho_{0.005} = 0.85\beta_1 \frac{f_c'}{f_y} \frac{\epsilon_u}{\epsilon_u + 0.005} = 0.85 \times 0.85 \times \frac{3}{40} \times \frac{3}{8} = 0.0203$$

$$d_{req} = \sqrt{\frac{M_{u,max}}{\phi \rho f_y b \left(1 - 0.59\rho \frac{f_y}{f_c'}\right)}} = \sqrt{\frac{1.5}{0.9 \times 0.0203 \times 40 \times 12 \left(1 - 0.59 \times 0.0203 \times \frac{40}{3}\right)}}$$

$$d_{req} = 0.204\ inch \qquad \boxed{d_{provided} = \left(\frac{5 + 1.5}{2} - 1\right) in = 2.25"}$$

$$\boxed{\boldsymbol{d_{provided} > d_{required}}} \longrightarrow \boxed{ok}$$

Step 04 : Reinforcement Area Determination

$$A_{s,min} = 0.002\ bt\ (As\ f_y = 40\ ksi) \qquad Here, b = 12\ in\ \ \&\ \ t = \frac{5 + 1.5}{2} = 3.25 in$$
$$= 0.078\ in^2/ft$$
$$M(-ve) = 0.1251\ kip - ft/ft \longrightarrow A_s(-ve) = 0.0186\ in^2/ft < A_{s,min} = 0.078\ in^2/ft$$

Step 05 : Reinforcement Spacing Determination

- $0.078\ in^2/ft\ (if\ we\ use\ \#3\ bar)$

$0.078\ in^2\ area\ required\ in\ 12 inch$

$1\ \ in^2\ area\ required\ in\ \frac{12}{0.078}\ inch$

$0.11\ \ in^2\ area\ required\ in\ \frac{12 \times 0.11}{0.078} inch$

$= 16.92\ inch$

$\approx 16.5\ inch (0.5"\ lower\ rounding)$

$\boxed{\boldsymbol{For\ Both\ Direction}}$

$\boxed{\therefore \#3\ @\ 16.5"\ c/c}$

Step 06 : Detailing

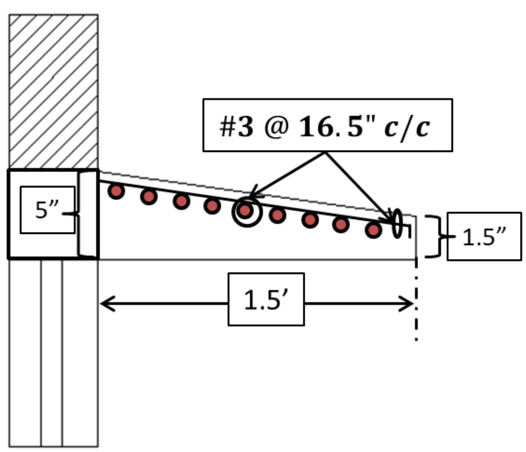


Figure 25: Detailing of Sunshade

5. DESIGN OF LINTEL (USD)

Assume, Lintel =10"x10"

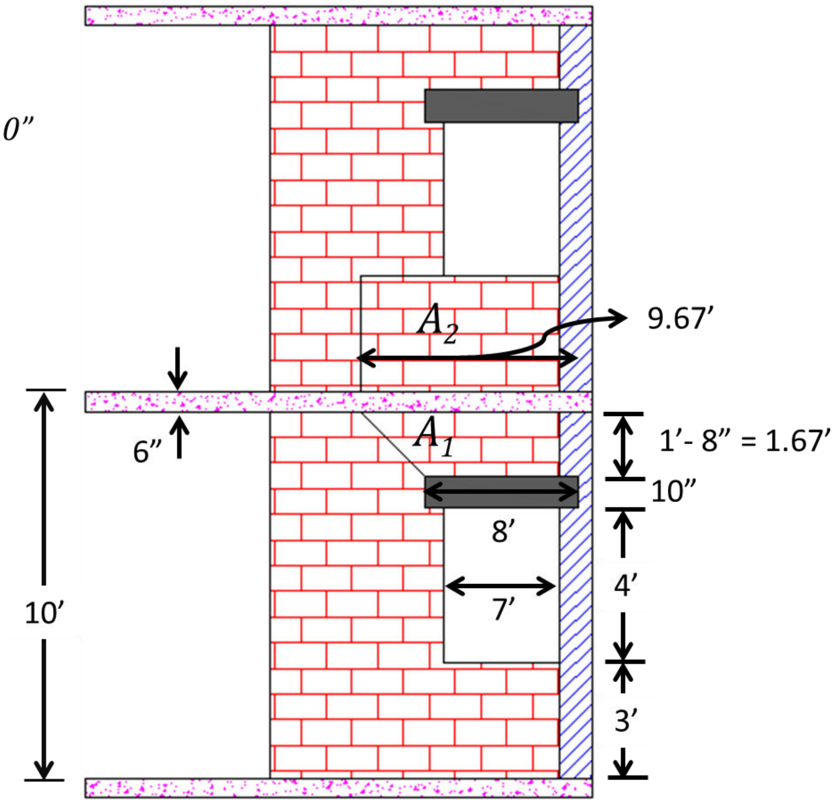


Figure 26: Information necessary to determine lintel load

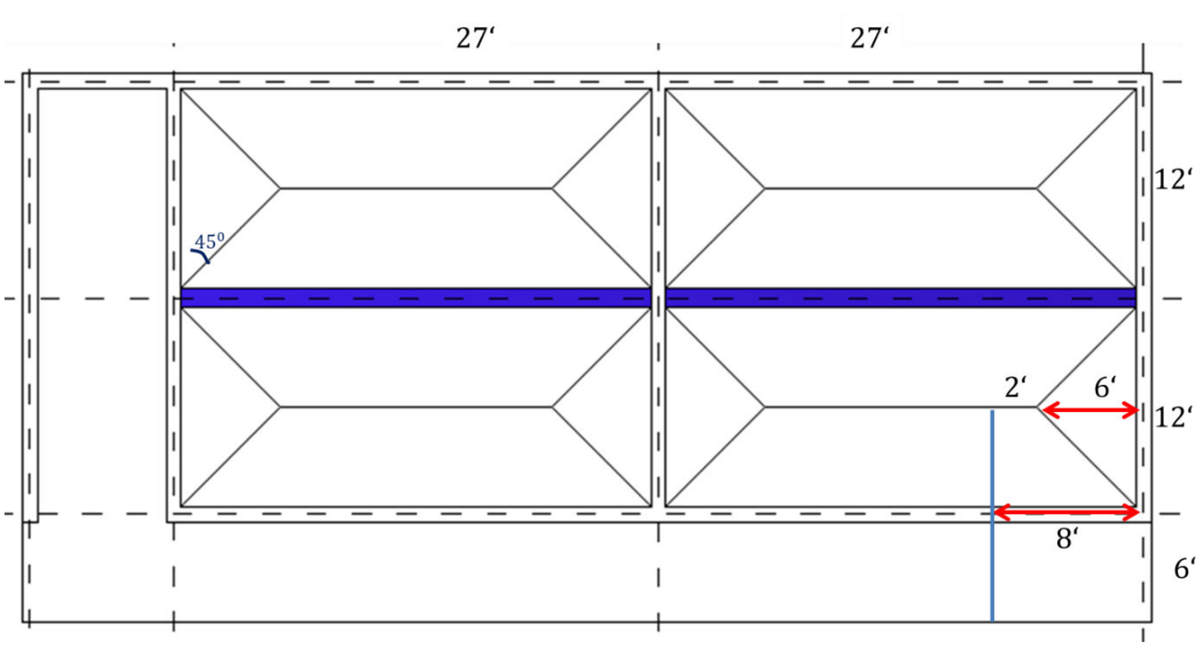


Figure 27: Loads come to Lintel from Slab

Step 01 : Determination of Load

All Dead Load ; No Live Load

1. ***From Brick wall :***

$$P_1 = \{Area\ of\ Trapezoid(A_1) + Area\ of\ Rectangle(A_2)\} \times \frac{10}{12} \times 0.120$$
$$= \left[\left\{\frac{1}{2} \times (8 + 9.67) \times 1.67 + 9.67 \times 3\right\} \times \frac{10}{12} \times 0.120\right]\ kip$$
$$= 4.38\ kip\ (Unfactored)$$

2. ***From Slab :***

$$P_2 = Floor\ udl\ (w_u) \times \{Area\ of\ Trapezoid(A_1) + Area\ of\ Rectangle(A_2)\}$$
$$= \left[0.21 \times \left\{\frac{1}{2} \times (8 + 2) \times 6 + 8 \times 6\right\}\right]\ kip$$
$$= 16.38\ kip\ (factored)$$

3. ***Self Weight of Lintel :***

$$P_3 = Volume\ (V) \times 0.150 = \frac{10\times10}{144} \times 8 \times 0.150 = 0.833\ kip\ (Unfactored)$$

$$\boxed{\textbf{Total Load (w)} = \frac{1.2\times(4.38+0.833)+16.38}{8} = 2.83\ kip/ft}$$

Step 02 : Moment Determination

Maximum Moment, $M_{u,max} = \frac{wL^2}{8} = \frac{2.83\times8^2}{8} = 22.64$ Kip-ft

Step 03 : d Check

$$\rho = \rho_{0.005} = 0.85\beta_1 \frac{f_c'}{f_y} \frac{\epsilon_u}{\epsilon_u + 0.005} = 0.85 \times 0.85 \times \frac{3}{40} \times \frac{3}{8} = 0.0203$$

$$d_{req} = \sqrt{\frac{M_{u,max}}{\emptyset \rho f_y b \left(1 - 0.59\rho \frac{f_y}{f_c'}\right)}} = \sqrt{\frac{22.64 \times 12}{0.9 \times 0.0203 \times 40 \times 10 \left(1 - 0.59 \times 0.0203 \times \frac{40}{3}\right)}}$$

$$\boxed{d_{provided} = (8 - 2)\ in = 6"}$$

$$d_{req} = 6\ inch$$

$$\boxed{d_{provided} = d_{required}} \longrightarrow \boxed{ok}$$

Step 04 : Reinforcement Area Determination

$$A_{s,min} = max \begin{cases} \frac{3\sqrt{f_c'\ (psi)}}{f_y(psi)} bd = \frac{3\sqrt{3000}}{40000} \times 10 \times 6 = 0.25\ in^2 \\ \frac{200}{f_y(psi)} bd = \frac{200}{40000} \times 10 \times 6 = 0.3\ in^2 \end{cases} = 0.3\ in^2$$

$M_{u,max}$ = 22.64 Kip-ft
A_s = 1.59 in^2

2#6 (bottom)
2#3 (top)

Step 05 : Detailing

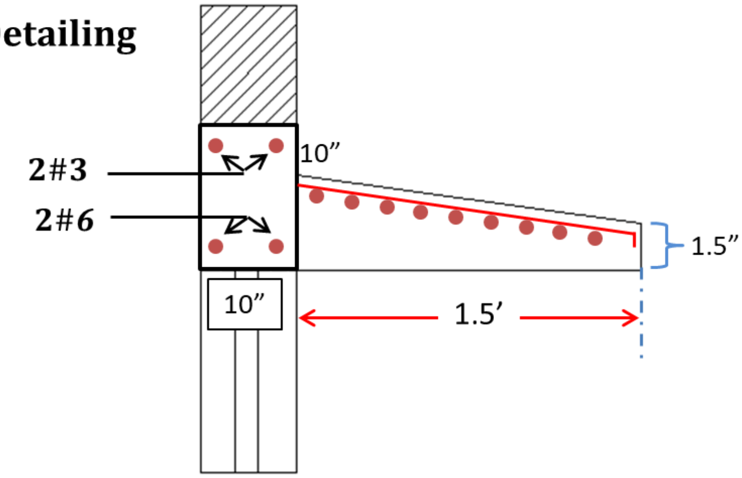


Figure 28: Detailing of Lintel

6. DESIGN OF BRICK FOUNDATION (WSD)

Design of brick foundation requires study and examination of multiple components, and it is the main structure of any building, since it is the base of it.

Foundation Depths

The required depth of the foundation will depend on several factors like:

1. Soil bearing capacity: How much load the existing soil can withstand.

2. Type of soil: This depth will vary depending on the type of soil beneath the structure.

3. Depth of frost penetration in case of fine sand and silt.

4. Height of ground water table: This will usually be reported on the soil study.

5. The minimum depth should not be less than 18 inches to allow removal of top soil and variations in ground level. However, depending on the structure, the engineer will select the best depth.

Foundation Material

The foundation is usually built in brick work, masonry or concrete under the base of a wall or column. This will enable to transfer the load to the soil in a uniform manner and allow the transition from the structure to the soil. It will depend on the recommendation by the structural engineer. For smaller and lightweight structures, the design will be different depending on the material and location of the structure.

Usually dry and uniform graded dense materials should have maximum shear resistance and maximum bearing capacity. In general submerged soil and clay have fewer bearing capacities, reducing the capacity to handle loads imposed by the structure.

Foundation Design Precaution

The foundation will be designed in a way that loads will be transferred uniformly to the contact surface. It should be designed to transmit the sum of dead load, live load and wind load to the ground. The net loading capacity coming into the soil should not exceed the bearing capacity of the soil. Settlements expected from the building will be design in such way that they will be controlled and uniform for the complete structure to avoid damages to the structure. Whole design of the foundation, super structure and characteristics of the ground should be studied to obtain benefits during construction work.

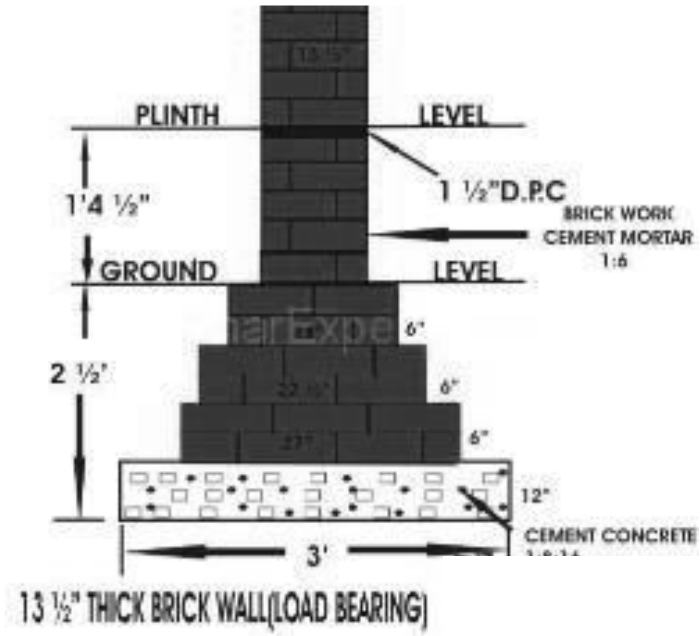


Figure 29: Typical Brick Foundation

Design Procedure

1. Find the total load on foundation. Suppose the load is 135k considering all load of superstructure on that particular foundation.

   Then the total load P is (135+Self weight of the foundation under G.L) k.

   Generally 10%-12% is considered as first assumption. For this example 10% load of superstructure is added as self-weight. So the total load P is (135+13.5) = 148.5 k

2. Find the soil bearing capacity. Suppose soil bearing capacity is 3 ksf

   Then the width of the foundation is B = $\frac{148.5\times12}{soil\ bearing\ capacity\times length\ of\ the\ foundation}$

   $= \frac{148.5\times12}{3\times22.5}$ where length of the foundation is 22.5 ft

   = 26.4 inch required width but in brick foundation width must be rounded to upper 5". So width for this foundation is 30".

3. Check the self-weight of the foundation whether it is less than the assumed value. If it is less than the design is ok.

DESIGN EXAMPLE OF FOUNDATION

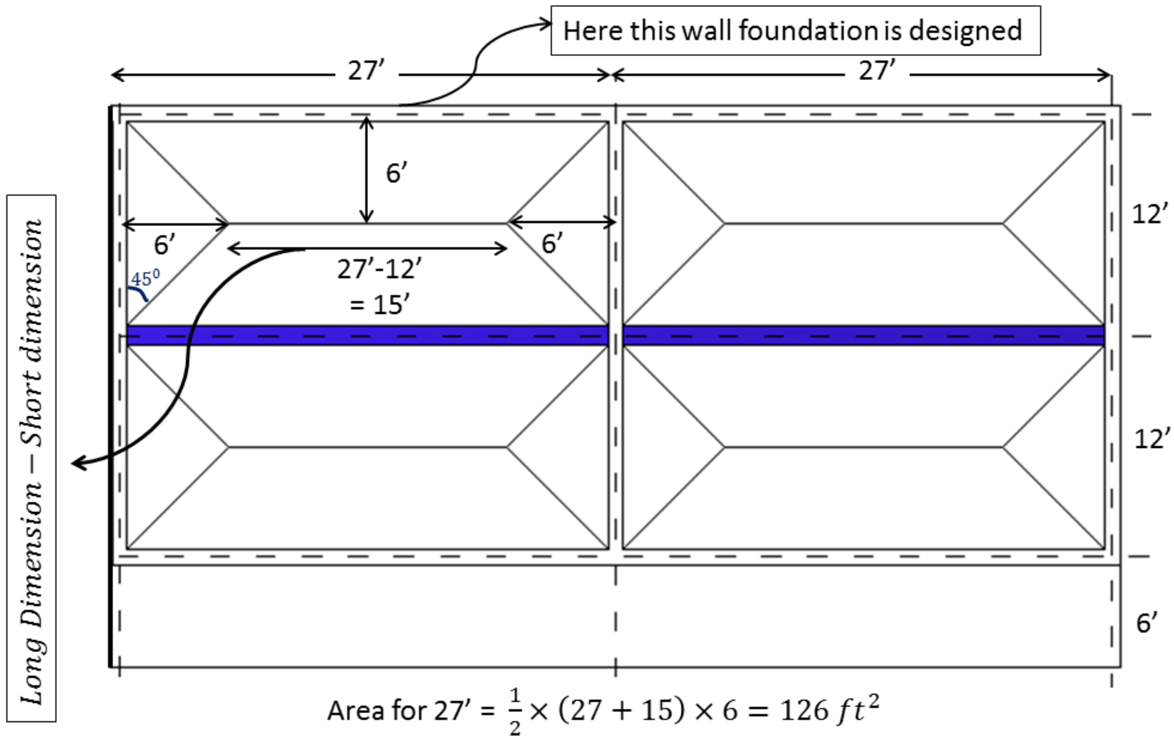


Figure 30: Contributing area of slab on brick wall

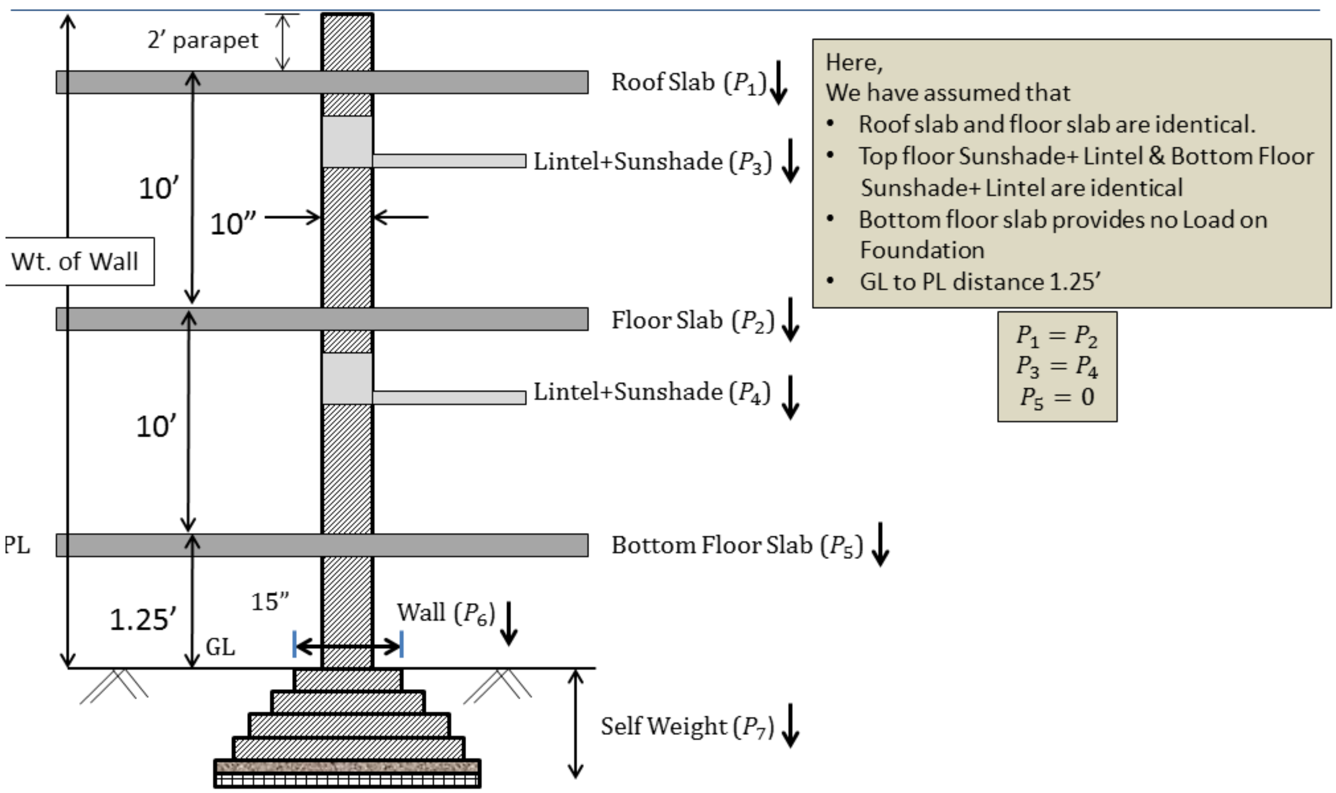


Figure 31: Loads on Foundation

Step 01 : Determination of Load

$$
\begin{aligned}
Unfactored\ w\ of\ Slab &= DL + LL \\
&= 93.75 + 60 \\
&= 153.75\ psf \\
&= 0.154\ ksf
\end{aligned}
$$

$$
\begin{aligned}
\boldsymbol{P_1} = \boldsymbol{P_2} &= Contributing\ Area\ \times Unfactored\ w\ of\ slab \\
&= (126 \times 0.154)\ kip \\
&= 19.404\ kip
\end{aligned}
$$

$$
\begin{aligned}
\boldsymbol{P_3} = \boldsymbol{P_4} &= Lintel\ Weight + Sunshade\ Weight \\
&= \left[\left(8 \times \frac{10}{12} \times \frac{10}{12} \times 0.150\right) + \left(\frac{5 + 1.5}{2 \times 12} \times 8 \times 1.5 \times 0.150\right)\right]\ kip \\
&= 1.321\ kip
\end{aligned}
$$

$$\boldsymbol{P_5} = 0\ kip$$

$$
\begin{aligned}
\boldsymbol{P_6} &= Weight\ of\ wall \\
&= \left[(1.25 + 10 + 10 + 2) \times \frac{10}{12} \times 27 \times 0.12\right]\ kip \\
&= 62.775\ kip
\end{aligned}
$$

$$\boldsymbol{Assume}, \boldsymbol{P_7} = 12\ \%\ of (\boldsymbol{P_1} + \boldsymbol{P_2} + \boldsymbol{P_3} + \boldsymbol{P_4} + \boldsymbol{P_5} + \boldsymbol{P_6}) = 12.507\ kip\ (\boldsymbol{Have\ to\ check})$$

$$Total\ Foundation\ Load = (19.404 \times 2 + 1.321 \times 2 + 0 + 62.775 + 12.507) = 116.732\ kip$$

Step 02 : Width Determination

Given,

Bearing Capacity of soil = 1.25 Tsf = 2.5 Ksf

$$Soil\ Bearing\ Capacity = \frac{Total\ Foundation\ Load}{Area\ at\ base\ of\ Foundation}$$

$$2.5\ ksf = \frac{116.732\ kip}{B(ft) \times 27ft}$$

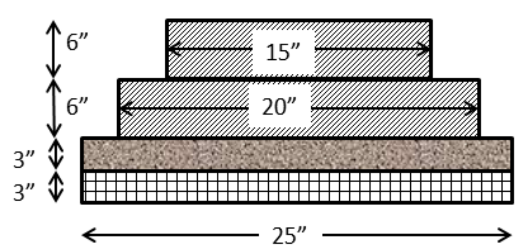
$$\mathrm{B}(ft) = 1.73$$

$$\mathrm{B} = 20.76 \approx 25"\ (5"\ upper\ rounding)$$

Step 03 : Self Weight Check

$$
\begin{aligned}
Self\ Weight = &\frac{15}{12} \times \frac{6}{12} \times 27 \times 0.12 + \frac{20}{12} \times \frac{6}{12} \times 27 \times 0.12 \\
&+ \frac{25}{12} \times \frac{3}{12} \times 27 \times 0.15 + \frac{25}{12} \times \frac{3}{12} \times 27 \times 0.12
\end{aligned}
$$

$$Self\ Weight = 8.522\ \mathrm{kip} < 12.507\ \mathrm{kip}\ (\mathbf{ok})$$



How to easily calculate the self weight (Method 1)

- **If Foundation Width, B = 40″ & Length, L = 25′**

Self Weight

$$= \frac{1}{12} \times \frac{6}{12} \times 25 \times 0.12 \times (15 + 20 + 25 + 30 + 35) + \frac{40}{12} \times \frac{3}{12} \times 25 \times (0.15 + 0.12)$$

$$= 21.25\ kip$$

- **If Foundation Width, B = 25″ & Length, L = 27′**

Self Weight

$$= \frac{1}{12} \times \frac{6}{12} \times 27 \times 0.12 \times (15 + 20) + \frac{25}{12} \times \frac{3}{12} \times 27 \times (0.15 + 0.12)$$

$$= 8.522\ kip$$

- **If Foundation Width, B = 45″ & Length, L = 20′**

Self Weight

$$= \frac{1}{12} \times \frac{6}{12} \times 20 \times 0.12 \times (15 + 20 + 25 + 30 + 35 + 40) + \frac{45}{12} \times \frac{3}{12} \times 20 \times (0.15 + 0.12)$$

$$= 21.5625\ kip$$

How to easily calculate the self weight (Method 2)

If Foundation Width, B(inch) & Length, L(ft) then Self Wt. $= \frac{4n^2+5n-24}{320} \times L$

Where, $n = \frac{B}{5}$

- **If Foundation Width, B = 40″ & Length, L = 25′ (n=40/5=8)**

Self Weight $= \frac{4n^2+5n-24}{320} \times L = \frac{4\times8^2+5\times8-24}{320} \times 25 = 21.25\ kip$

- **If Foundation Width, B = 25″ & Length, L = 27′ (n=25/5=5)**

Self Weight $= \frac{4n^2+5n-24}{320} \times L = \frac{4\times5^2+5\times5-24}{320} \times 27 = 8.522\ kip$

- **If Foundation Width, B = 45″ & Length, L = 20′ (n=45/5=9)**

Self Weight $= \frac{4n^2+5n-24}{320} \times L = \frac{4\times9^2+5\times9-24}{320} \times 20 = 21.5625\ kip$

Design of a slab bridge

Design of a slab bridge

Data and Specifications

Clear Span	15 ft
Clear width	26 ft
Live Loading	HS20
Wearing surface	30 psf
Concrete strength f_c'	3000 psi
Grade 40 reinforcement	

By AASHTO specifications, an allowable concrete stress of $f_c = 0.40\ f_c' = 1200$ psi

And an allowable steel stress of $f_s = 0.5\ f_y = 20{,}000$ psi will be used

Slab Design

Assume trial slab thickness , t = 12 in

The effective span of the slab,S = 15 + 1 = 16 ft [for simple span the span length shall be the distance of center to center of supports but shall not exceed clear span plus thickness of slab]

Total dead load = (t/12)*150 + wearing surface

= 180 psf

For a strip of 1 ft width W_{dl} = 180 lb/ft

Dead load moment, DLM = $W_{dl}\ S^2/8$

= 180*16*16/8

= 5760 ft-lb

Total load on each rear wheel , P_{20}= 16000 lb [for HS20 truck loading]

Width of slab over which wheel load is distributed,E = 4+0.06S [for main reinforcement parallel to traffic]

= 4+0.06*16

= 4.96 ft

The load on unit width of slab, P = P_{20}/E

=16000/4.96

=3230 lb

Live load moment, LLM = PS/4

=3230*16/4

= 12900 ft-lb

Impact coefficient, I = $\frac{50}{l+125} \leq 0.3$

Where l is the loaded length which is equal to effective span, S here

I = $\frac{50}{S+125}$

= 0.355>0.3

So I will be taken as 0.3

Impact moment, IM = Impact coefficient*Live load moment

= I*LLM

=0.3*12900

=3870 ft-lb

Total moment ,M = DLM+LLM+IM

=22,530 ft-lb

The design will be based on service load and the slab will be proportioned based on the cracked elastic section

$$k = \frac{n}{n + r}$$

Use n = 10 and r = fs/fc

k= 0.375

j = 1-k/3= 0.875

effective depth, $d_{reqd} = \sqrt{\frac{2M}{\text{fckjb}}} = 10.7$ in

$d_{provided}$= t-protective cover-half of main bar dia

= 12-1-0.5 [by providing #8 bar]

= 10.5

$d_{provided}$is sufficiently close to d_{reqd}

Reinforcement Calculation:

The required main reinforcement, $A_s = \frac{M}{f_s jd} = \frac{22530*12}{20000*0.875*10.5} = 1.47\ in^2/ft$
This is furnished by #8 bars 6 in on centers

Transverse reinforcement

For main reinforcement parallel to traffic transverse reinforcement,

$$A_{st} = \frac{100}{\sqrt{S}} \%\ of\ As\ (< 50\%\ of\ A_s)$$

$$= \frac{A_s}{\sqrt{S}}\ (< 0.5A_s)$$

$$= \frac{1.47}{\sqrt{16}}\ (< 0.5 * 1.47)$$

$$=0.37\ in^2/ft\ (< 0.735\ in^2/ft)$$

$$=0.37\ in^2/ft$$

#5 bars 10 in on centers are directly placed on top of the longitudinal reinforcement.

Curb design

To facilitate screeding off the slab the required curb will not be made monolithically.

The dead load carried by the edge beam $= \frac{22*24}{144} * 150 = 550\ plf$

Dead load moment $= 550 * \frac{16^2}{8} = 17600 ft - lb$

The specified live load moment $= 0.1 * P_{20} * S = 0.1 * 16000 * 16 = 25600\ ft - lb$

Total moment$= 17600 + 25600 = 43200\ ft - lb$

The resisting moment of the given section, $M_r = 0.5 f_c kjbd^2$

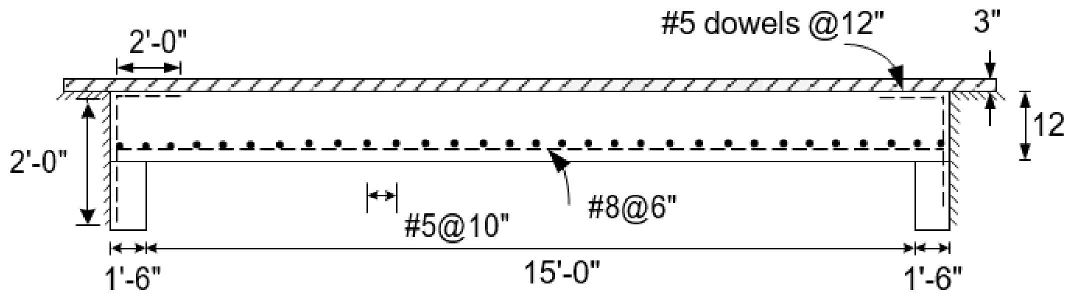
$$= 0.5 * 1200 ** 0.375 * 0.875 * 24 * \frac{10.5^2}{12}$$

$$= 43411\ ft - lb$$

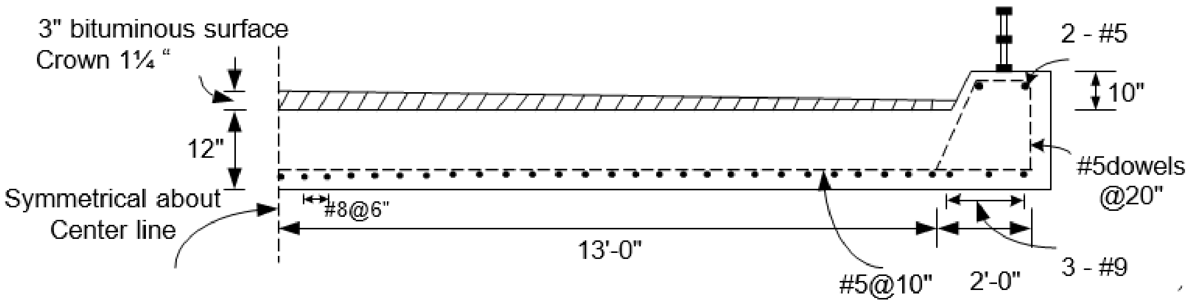
Which is adequate

The steel required, $A_s = \frac{43200*12}{20000*0.875*10.5} = 2.82\ in^2$

Which will be provided by by three No. 9 bar



Longitudinal Section



Tranverse Section

Design of a Deck Girder bridge

Design of a Deck Girder bridge

Data and Specifications

Clear Span 48 ft

Clear width 29 ft

Live Loading HS20

Concrete strength $f_c^{'}$ 3000 psi

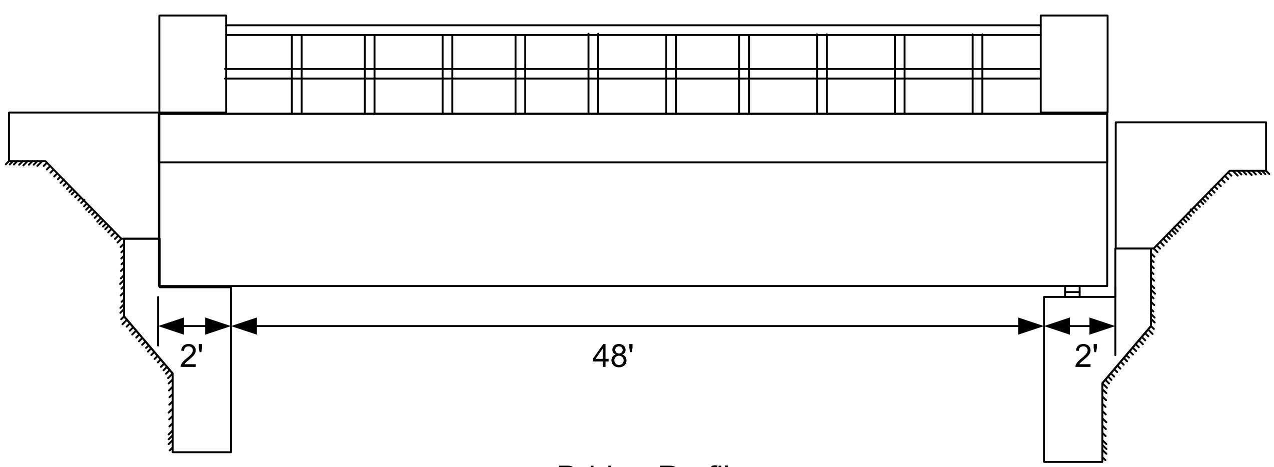
Future protective cover 15 pf

Grade 40 reinforcement

The bridge will consist of six girders

By AASHTO specifications, an allowable concrete stress of $f_c = 0.40\ f_c^{'} = 1200$ psi

And an allowable steel stress of $f_s = 0.5\ f_y = 20{,}000$ psi will be used



Bridge Profile

Slab Design

Span = clear distance between girders [The slab will be poured monolithically with the girder and fully continuous]

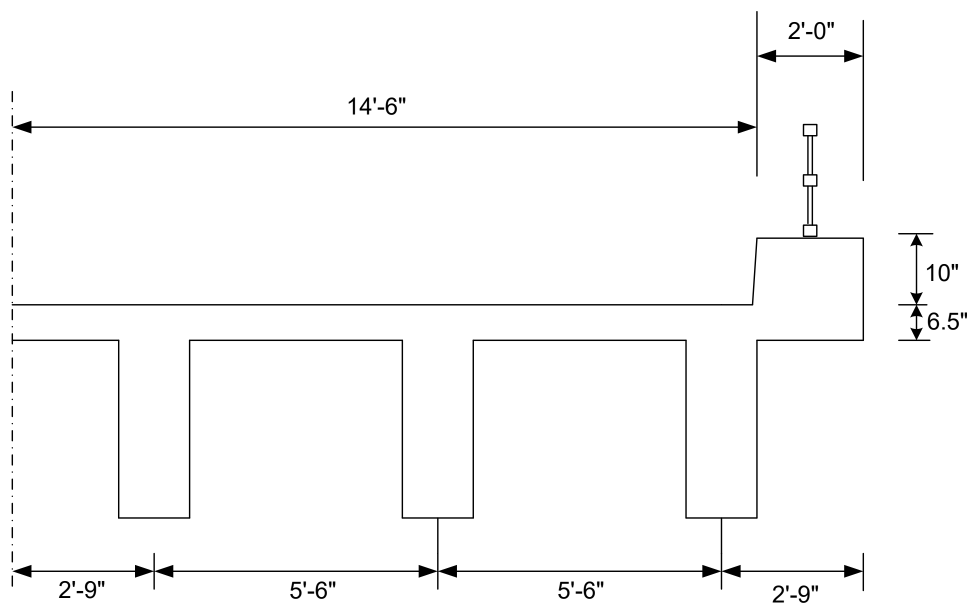
Assume the girder width = 14” and total slab thickness = 6” [including ¾ in wearing surface]

Clear span,S = 4 ft 4 in

Total dead load, W_{dl} = 6/12*150 + 15

= 90 psf

A coefficient of $\frac{1}{10}$ will be used for both positive and negative moment



Transverse Section

Dead load moment, DLM = W_{dl} $S^2/10$

$= \frac{1}{10} *90*4.33^2$

= 169 ft-lb

Total load on each rear wheel , P_{20}= 16000 lb [for HS20 truck loading]

Live load moment, LLM = $\frac{S+2}{32}*P_{20}$ [for main reinforcement perpendicular to traffic]

For slabs continuous over three or more supports a continuity factor of 0.8 shall be applied to the above formula

Live load moment, LLM = $\frac{S+2}{32}*P_{20}$

$$= 0.8*\frac{4.33+2}{32}*16000$$

$$= 2530 \text{ ft-lb}$$

Impact coefficient, I = $\frac{50}{l+125} \leq 0.3$

Where l is the loaded length which is equal to clear span, S here

$$I = \frac{50}{S+125}$$

$$= 0.387>0.3$$

So I will be taken as 0.3

Impact moment, IM = Impact coefficient*Live load moment

= I*LLM

=0.3*2530

=760 ft-lb

Total moment ,M = DLM+LLM+IM

=3459 ft-lb

The design will be based on service load and the slab will be proportioned based on the cracked elastic section

$$k = \frac{n}{n+r}$$

Use n = 10 and r = fs/fc

k= 0.375

j = 1-k/3= 0.875

effective depth, $d_{reqd} = \sqrt{\frac{2M}{\text{fckjb}}} = 4.19$ in

$d_{provided}$= t-protective cover-half of main bar dia

= 6-1-0.5*0.75-0.75 [by providing #6 bar, 1 in protective cover and ¾" wearing surface]

= 3.875 in

$d_{provided}$ is lower than d_{reqd}

So 6.5 in slab will be used

Reinforcement Calculation:

The required main reinforcement, $A_s = \frac{M}{f_s jd} = \frac{3459*12}{20000*0.875*4.37} = 0.54\ in^2/ft$

This is furnished by #6 bars 10 in on centers

Transverse reinforcement

For main reinforcement perpendicular to traffic transverse reinforcement

$A_{st} = \frac{220}{\sqrt{S}} \%\ of\ As\ (< 67\%\ of\ A_s)$

$= \frac{2.20 A_s}{\sqrt{S}}\ (< 0.67 A_s)$

$= \frac{2.20*0.54}{\sqrt{4.33}}\ (> 0.67 * 0.54)$

$=0.57\ in^2/ft\ (> 0.36\ in^2/ft)$

$=0.36\ in^2/ft$

#5 bars 10 in on centers are directly placed on top of the longitudinal reinforcement.

Interior Girder Design

Interior girders are T beams with a flange width equal to the center to center distance of girders

[based on ACI code]

Total moment calculation is done by following the steps listed below:

1. *Dead load moment calculation:*
   a. Calculate total load from slab and assumed 14"X30" stem (below the slab)in plf
   b. Calculate maximum dead load moment **DLM**$_{max}$[the girder will act like a simply supported beam with uniformly distributed load]
   c. Calculate DLM at one quarter and two quarter points of the span.

2. *Live load moment calculation:* For live load moment calculation both truck load moment and lane loading moment needed to be calculated. Based on this calculation the governing **Live Load Moment, LLM$_{max}$**has to be decided. Live load moment at one quarter and two quarter points of the span have to also be calculated.

3. *Impact moment calculation:*

Impact moment, IM$_{max}$ = Impact coefficient*Live load moment

Impact moment at one quarter and two quarter points of the span have also to be calculated.

Total Moment = **DLM**$_{max}$**+ LLM$_{max}$+ IM$_{max}$**
Total moment moment at one quarter and two quarter points of the span have also to be calculated.

Total shear calculation is done by following the steps listed below:

1. *Dead load shear calculation:* Calculate maximum dead load shear at the end of the beam and at atone quarter and two quarter points of the span of the beam.

2. *Live load shear calculation:* Calculate maximum live load shear for truck loading and lane loading at the end of the beam and atone quarter and two quarter points of the span of the beam. Based on the above calculation decide the governing Live load shear.

3. *Impact shear:*

Impact shear, IS = Impact coefficient*Live load shear

Impact shear at one quarter and two quarter points of the span have also to be calculated.

Total shear (63400 psi) at the end of the beam and at one quarter and two quarter points of the span have to be calculated.

Determination of cross section and steel area

According to AASHTO specification

Nominal shear stress at service load, $v = V/b_w d$

Where b_w = width of web.

In beam with web reinforcement the nominal shear stress, v may not exceed $4.95\sqrt{fc'}$

In sizing the web area maximum shear force of $2.95\sqrt{fc'} = 162$ psi will be used

$$b_w d = \frac{V}{v} = \frac{63{,}400}{162} = 391 \text{ in}^2$$

$asb_w = 14$ in, $d_{required} = 28$ in

if three rows of #11 bars are used with 2 in clear between rows and 2.5 in clear below the bottom row to allow for stirrups and concrete protection a total depth of 34.56 in (28+2+2.5+1.5*11/8) is obtained.

A total depth of 36 in will be used. The depth of the stem below the slab is then 36 - 6.5 = 29.5 in which is so close to the assumed value that the dead load moments need not be revised.

The next step is tensile steel area calculation and web reinforcement calculation.

The final step is bar cut off

Exterior Girder Design

The exterior girders are identical in cross section with the interior girders cause the raised curb section will be poured separately and cannot be counted in to participate in carrying loads.

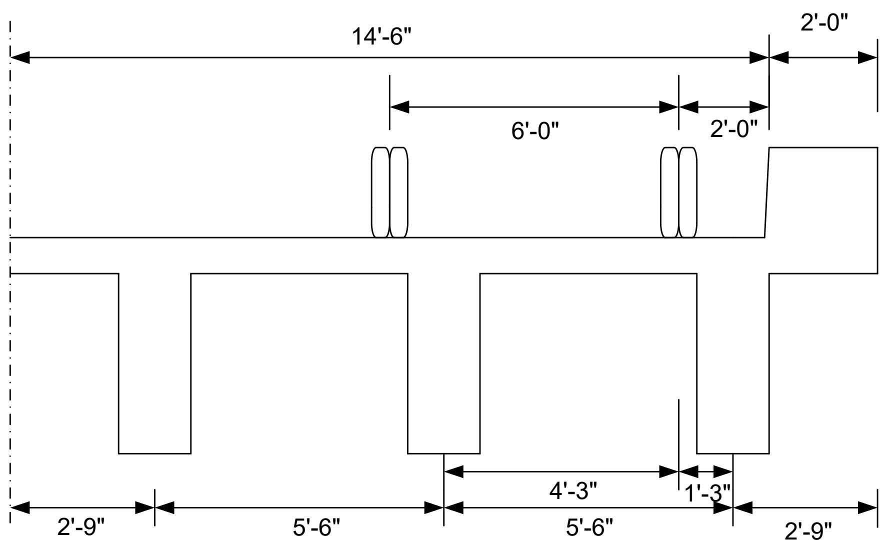
Moment Calculation:

In addition to the total dead load obtained for the interior girders the safety curbs will add some extra load (total 1215 plf).

By following the same procedure total moment and shear have to be calculated.

A portion of the wheel load which rests on the exterior slab panel is supported by the exterior girder.

That portion is obtained by placing the wheels as close to the curb as the clearance diagram will permit and treating the exterior slab panel as a simple beam. The portion is shown in the following fig and the proportion of the load is 4.25/5.5 = 0.773



Lateral Position of Wheel

The absolute maximum live load moment = $\frac{0.773}{1.1}$*346000 = 243000 ft-lb

The impact moment is 0.285*243000 = 69300 ft-lb and the total maximum moment is 692300 ft-lb.

Shear Calculation:

The maximum dead load shear is V_D = 1215*25 = 30400 lb

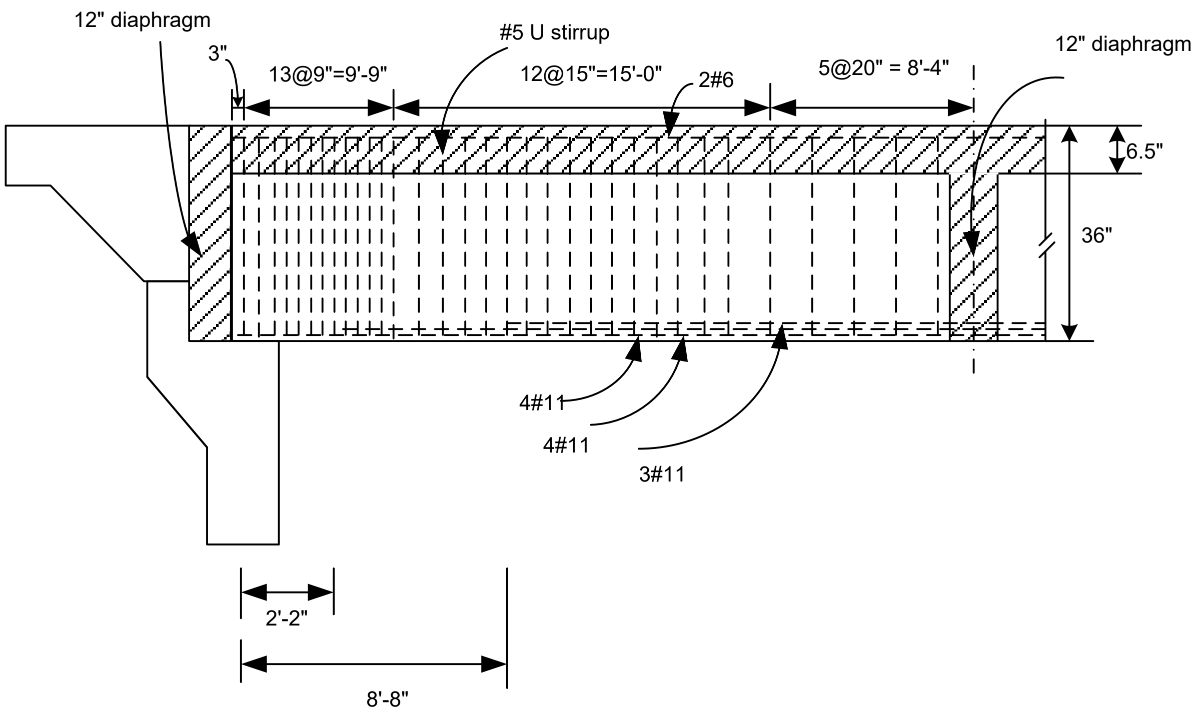
The maximum live load shear is proportional to the maximum live load shear in interior girder

$V_L = \frac{0.773}{1.1}$ *30600 = 21500 lb

The impact shear is 6100 lb. The total shear at the support is then 58000 lb. Shear at other points are found similarly.

Determination of cross section and reinforcement:

Once the shears and moments due to dead loads, live loads and impact are obtained the design of the exterior girders would follow along the lines of that of interior girders.



Details of Interior Girder

Diaphragms: A transverse will be built between the girders at either end of the bridge. The chief function of these diaphragm is to furnish lateral support to the girders, with some abutment details it also serve to prevent the backfill from spilling out onto the bridge seats. A similar diaphragm will be built between girders at midspan. Such intermediate diaphragms are required for all spans in excess of 40 ft and serve to ensure that all girders act together in resisting loads.

Fixed bearing is provided at one end and **expansion bearing** is provided at the other end.

Water proofing and drainage: all joints will be filled with a mastic compound to prevent water from seeping through the joints. Crowing is provided for quick drainage of the rain water.

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