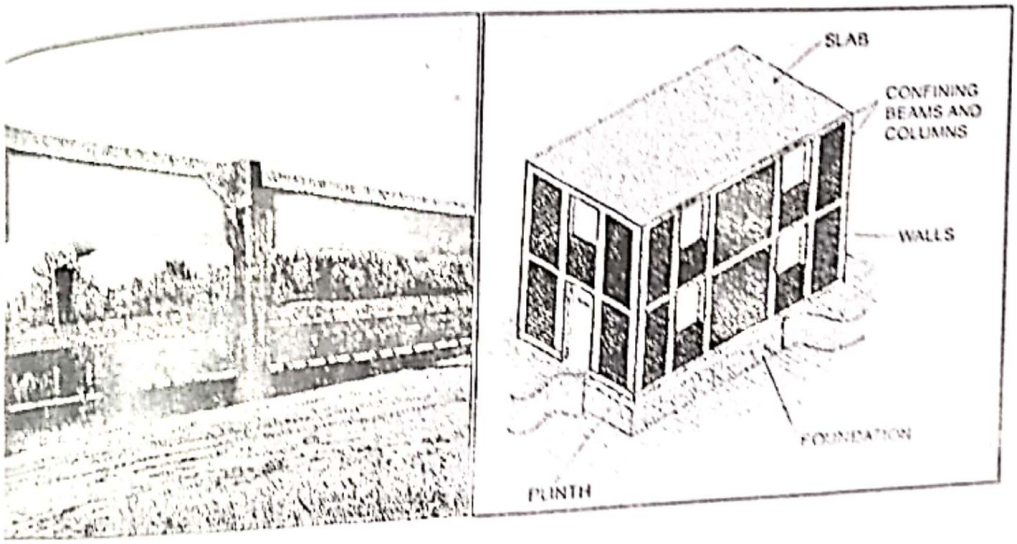
Design of a Low -Rise Masonry Building

CE 316
Design of Concrete Structures Sessional



Department of Civil Engineering
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Version 2, November, 2017

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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 itself a composite of cement, lime and sand and sometimes of other constituents.

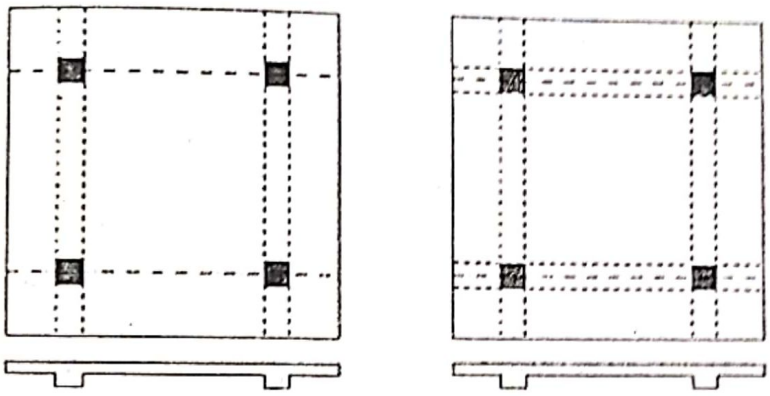
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Design of one way slab (USD)

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

way slab can be

lid
llow or
bbed



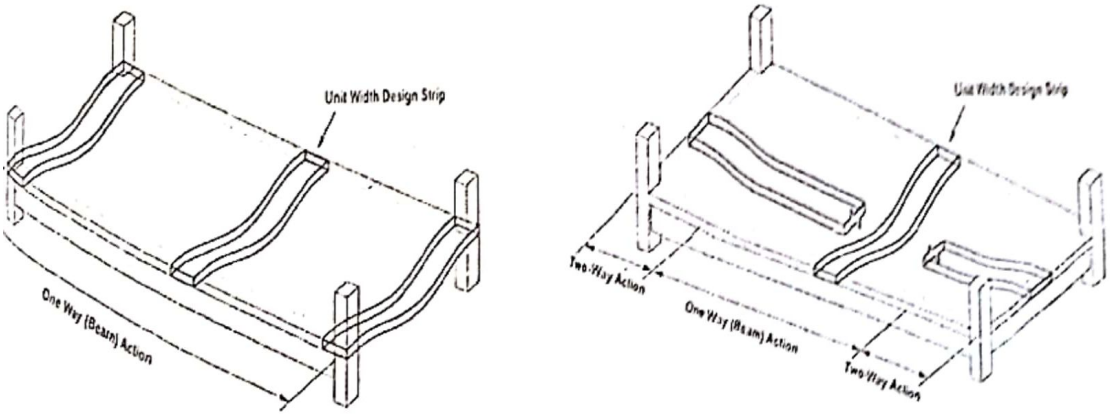
(a) One-way slab (b) Two-way slab

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

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

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

DING OF ONE WAY SLAB



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

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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.

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Spacing

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

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

rally, bar size should be selected so that actual spacing is not less than about 1.5 times the hickness.

imum Reinforcement Ratio

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

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

emperature and shrinkage reinforcement ratio:

rding to *ACI Code 7.12.2.1*

with Grade 40 or 50 deformed bars..... 0.0020

with Grade 60 deformed bars 0.0018

where reinforcement with yield strength

ding 60000 psi..... $\frac{0.0018 \times 60000}{f_y}$

lexural reinforcement:

ding to *ACI Code 10.5.4,*

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

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1.2 DESIGN EXAMPLE

Design the slab following the provisions of the ACI code.

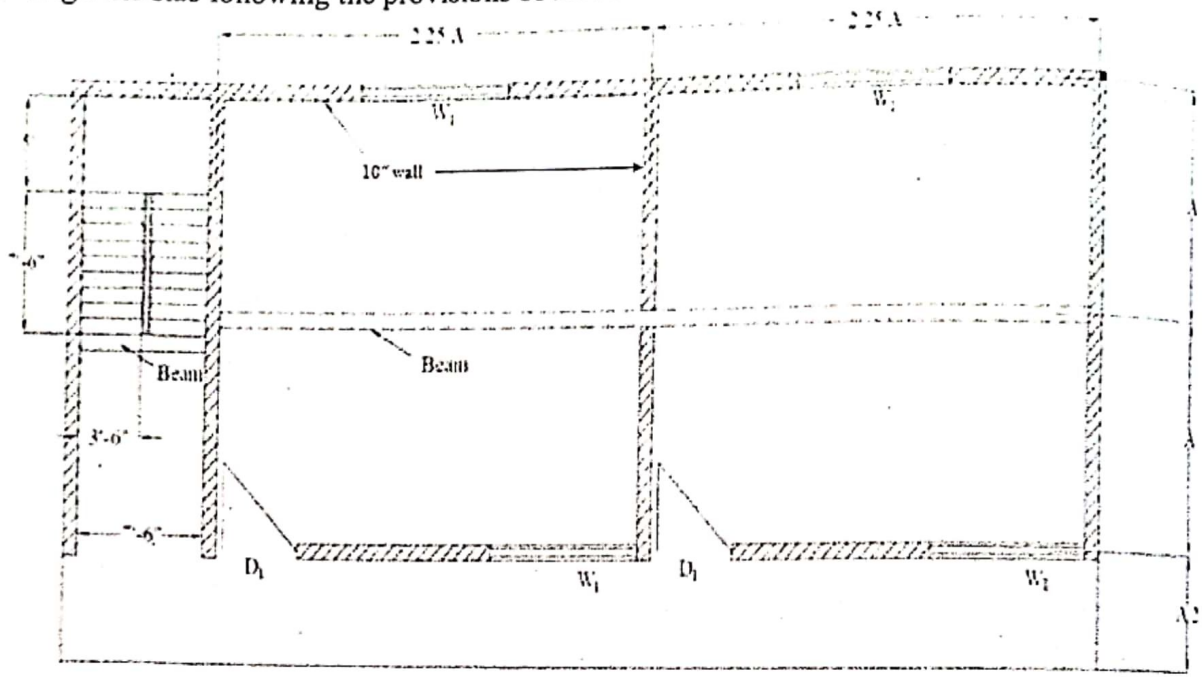


Figure 3: Typical Layout of Load Bearing Building

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

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CKNESS ESTIMATION

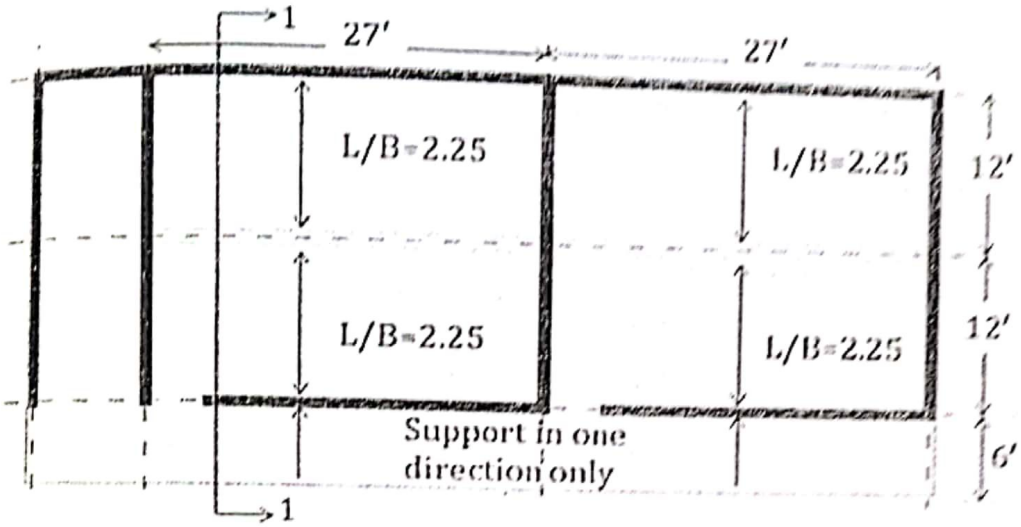


Figure 4: Load Transfer in Shorter Direction

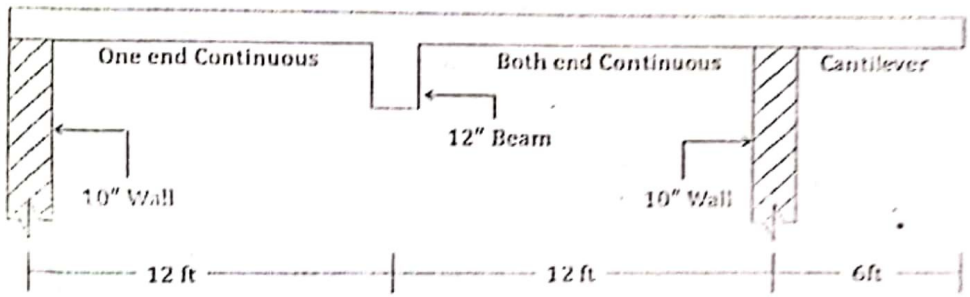


Figure 5: Section 1-1

one end continuous

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

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

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

both end continuous

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

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

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

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➢ 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:

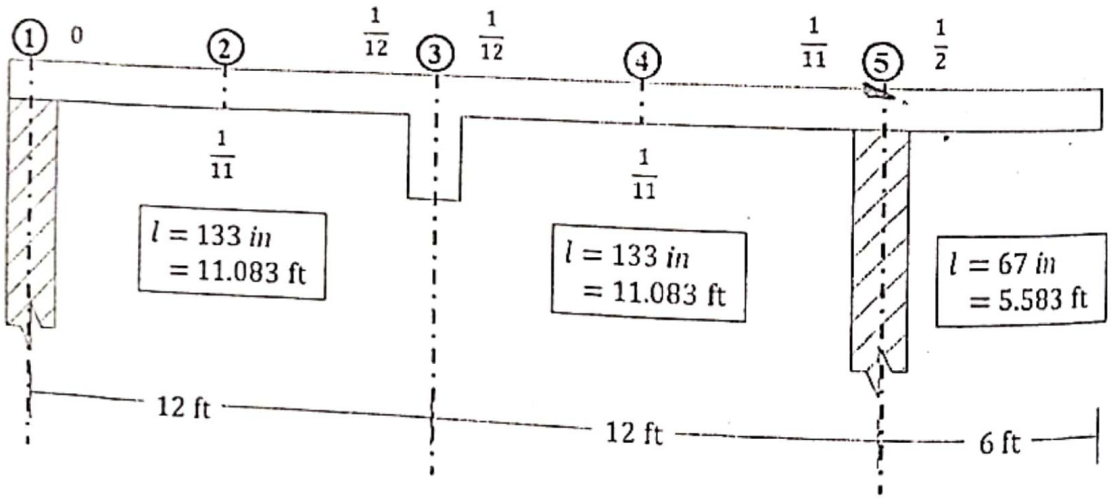


Figure 6: Moment Coefficient

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- 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.) : $M_u = -39.27\ k - in/ft$

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

ɱUM EFFECTIVE DEPTH

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

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

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

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

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

$= 2.31\ in$

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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

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

➢ **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 $A_s < A_{s,min}$)
$$= 0.013\ in^2/ft$$

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

= 1 in

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

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

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

nd,

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

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

$$= 0.26\ in$$

gain,

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

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

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

nd,

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

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

$$= 0.24\ in$$

gain,

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

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

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

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➢ **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

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

= 1 in

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

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

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

nd,

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

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

$$= 0.26\ in$$

gain,

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

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

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

l,

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

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

= 0.24 *in*

in,

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

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

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

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➢ **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

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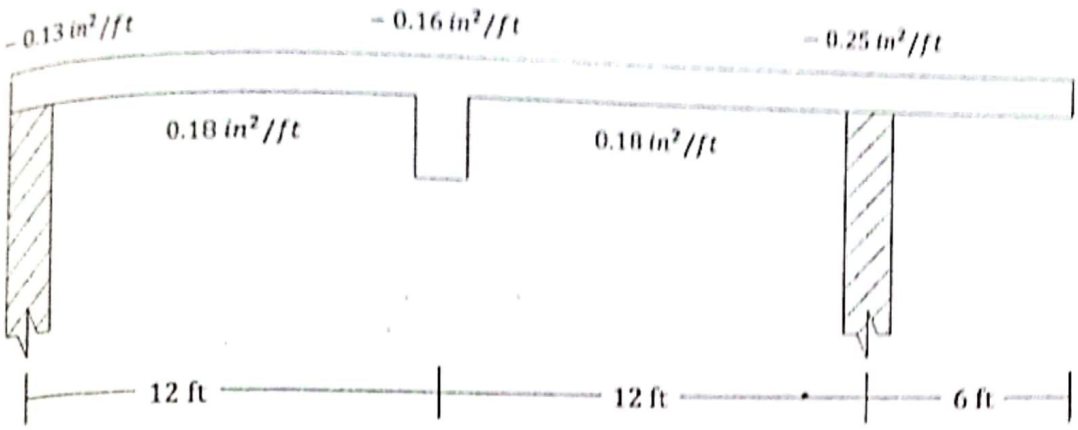


Figure 7: Reinforcement Area Required

FORCEMENT SPACING DETERMINATION

ort Direction:

$_n = 1.5t = 1.5 \times 5.5 = 8.25"$

$_{tx} = 3t \text{ or } 18 = (3 \times 5.5)" \text{ or } 18" = 16.5" \text{ or } 18" = 16.5"$ *(Lower one)*

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

0.18 in² reinforcement required in 12 inch

1 in² reinforcement required in $\frac{12}{0.18}$ inch

0.11 in² reinforcement required in $\frac{12 \times 0.11}{0.18}$ inch

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

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

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

).18 in² reinforcement required in 12 inch

1 in² reinforcement required in $\frac{12}{0.18}$ inch

).2 in² reinforcement required in $\frac{12 \times 0.2}{0.18}$ inch

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

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

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

$= 0.13\ in^2/ft$: (At Section 1)

#4 @ 26" c/c is already provided

In 26 inch already provided reinforcement area = 0.2 in²

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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) = $\frac{0.08}{0.11} = 0.73 \approx 1$

1#3 extra top between ckd. bar

$A_s = 0.16\ in^2/ft$: (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) = $\frac{0.15}{0.2} = 0.75 \approx 1$

1#4 extra top between ckd. bar

$A_s = 0.25\ in^2/ft$: (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) = $\frac{0.34}{0.2} = 1.70 \approx 2$

2#4 extra top between ckd. bar

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Long Direction:

$S_{min} = 1.5t = 1.5 \times 5.5 = 8.25"$

$S_{max} = 5t \text{ or } 18 = (5 \times 5.5)" \text{ or } 18" = 27.5" \text{ or } 18" = 18"$ (*Lower one*)

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

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

0.13 in² reinforcement required in 12 inch

1 in² reinforcement required in $\frac{12}{0.13}$ inch

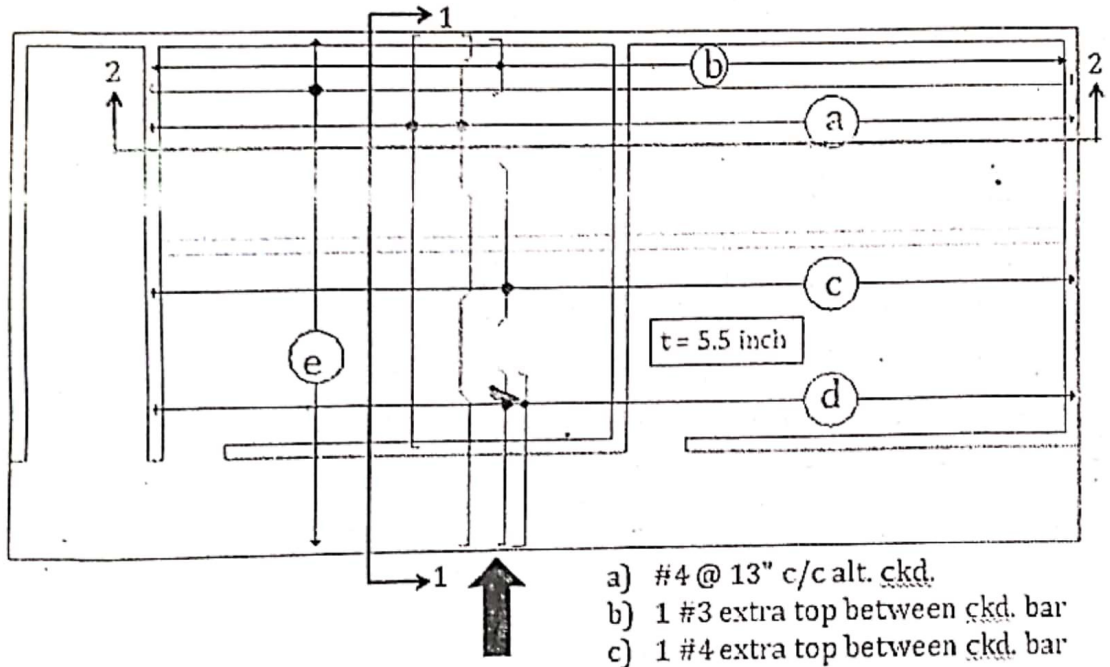
0.11 in² reinforcement required in $\frac{12 \times 0.11}{0.13}$ inch

$= 10.15" \approx 10"$ (0.5" *lower rounding*)

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

$\therefore$ **#3 @10" c/c**

AILING:



a) #4 @ 13" c/c alt. ckd.
b) 1 #3 extra top between ckd. bar
c) 1 #4 extra top between ckd. bar
d) 2 #4 extra top between ckd. bar
e) #3 @ 10" c/c

Figure 8: Detailing at Plan

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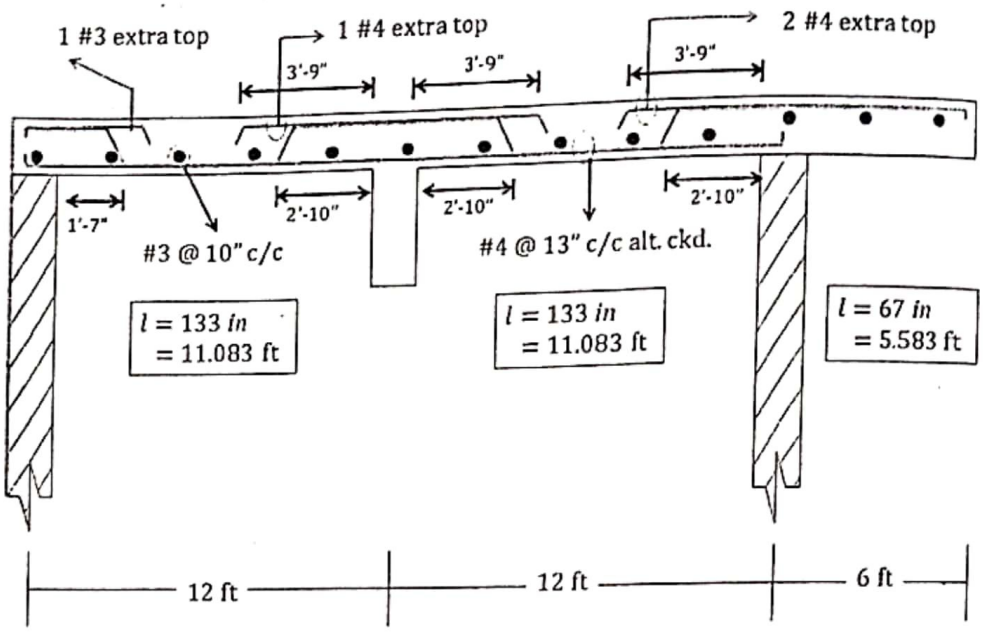


Figure 9: Detailing at Section 1-1

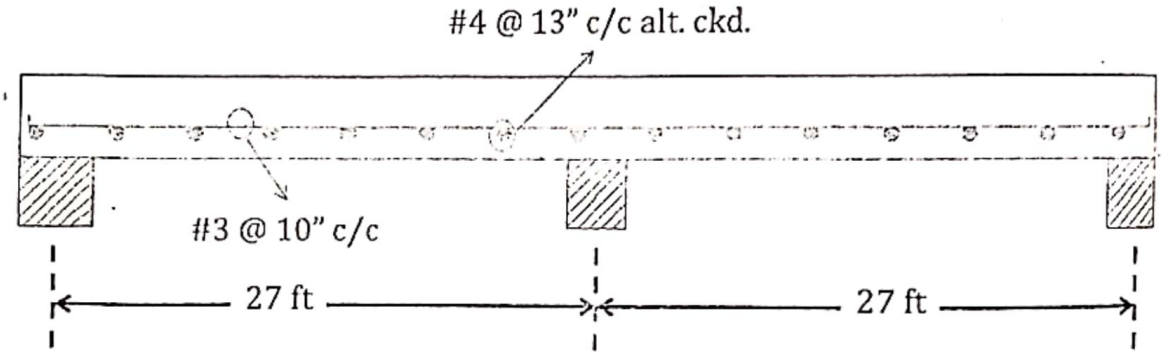


Figure 10: Detailing at Section 2-2

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ESIGN OF BEAM (USD)

duction:

nate 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.

mptions:

e are five assumption that are made

lane sections before bending remain plane after bending.

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

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

he tensile strength of concrete may be neglected.

t ultimate strength, the maximum strain at the extreme compression fiber is assumed equal to 03

sign and Analysis

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

sign of a section:

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

alysis of a section:

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

am Types

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

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Flexure Equations

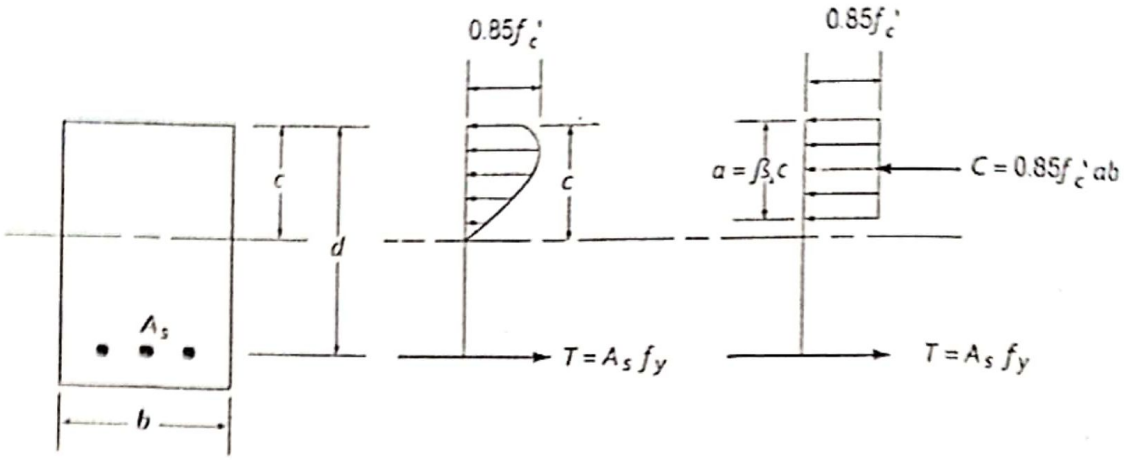


Figure 12: Diagrams for flexure equations

$$C = T$$

$$0.85f_c'ab = A_s f_y$$

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

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

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

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

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

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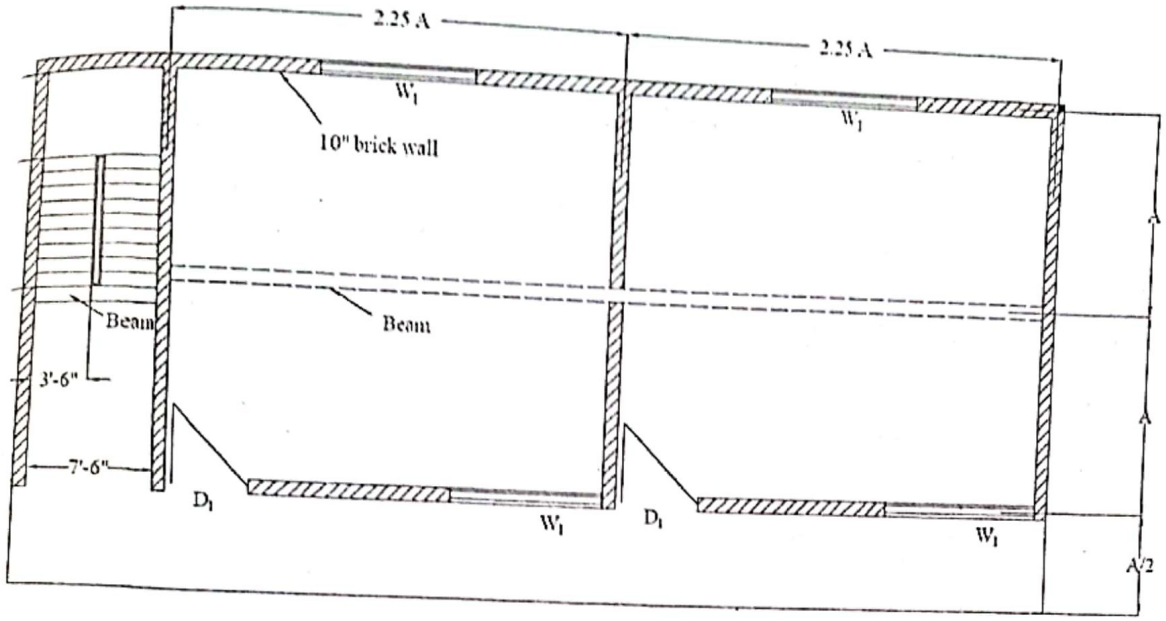


Figure 12: Beam Layout

d Calculation for Beam Design

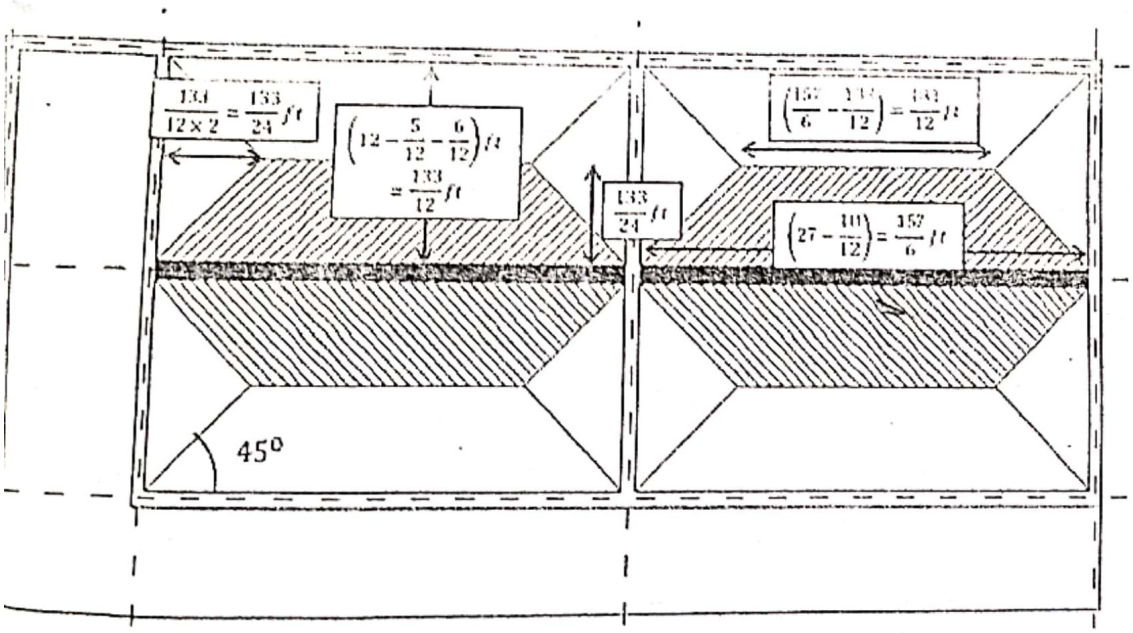


Figure 13: Determination of Contributing Area for Load Calculation

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

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Step 02 : Determination of Load

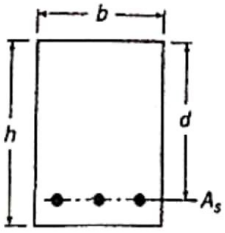
Load from Slab, $w_u = 0.21\ ksf$ (factored)

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

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

$$= 1.835\ kip/ft$$

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



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

Self Weight of Beam, $w_{u,self\ wt.} = \left(\frac{12 \times 24}{144} \times 0.150\right) kip/ft$

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

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)

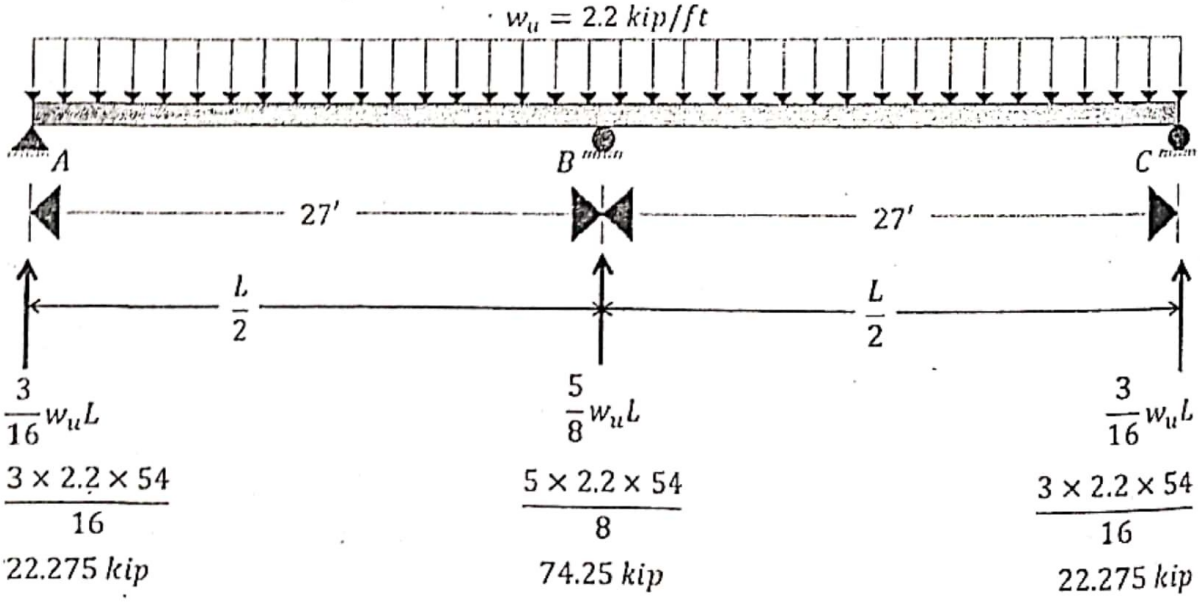


Figure 14: Determination Support Reactions

- **SFD and BMD**

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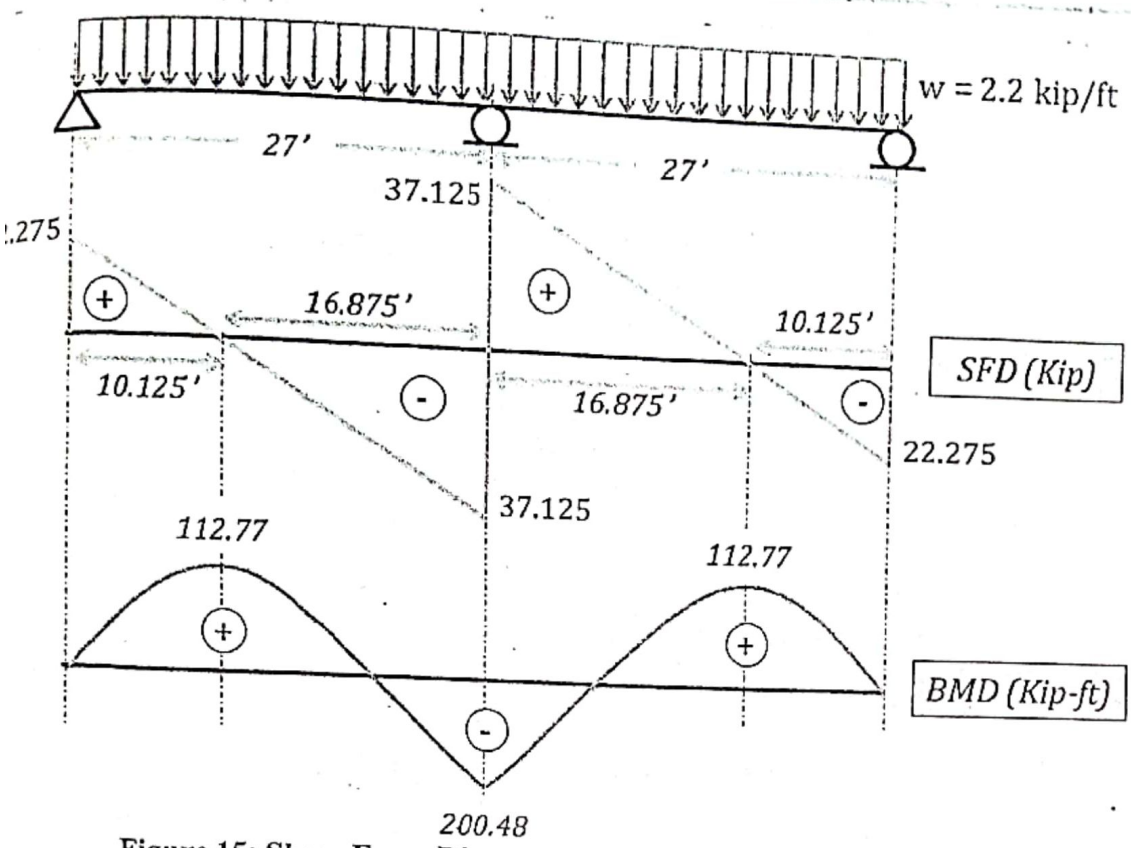


Figure 15: Shear Force Diagram and Bending Moment Diagram

heck for Maximum Moment

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

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

$$= 0.85 \times 0.85 \times \frac{3}{40} \times \frac{3}{8}$$

$$= 0.0203$$

$$\emptyset = 0.4833 + 83.33\epsilon_t$$

$$At, \epsilon_t = 0.005$$

$$\emptyset = 0.4833 + 83.33 \times 0.005$$

$$= 0.9$$

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

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

d_{req} = 18.07 *inch*

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

$d_{provided} > d_{required}$ ⟶ *Section is ok*

URAL REINFORCEMENT AREA DETERMINATION:

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

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

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

$M_u = 112.77\ kip - ft$:

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

$M_u = 200.48\ kip - ft$:

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

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

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

SHEAR REINFORCEMENT DETERMINATION

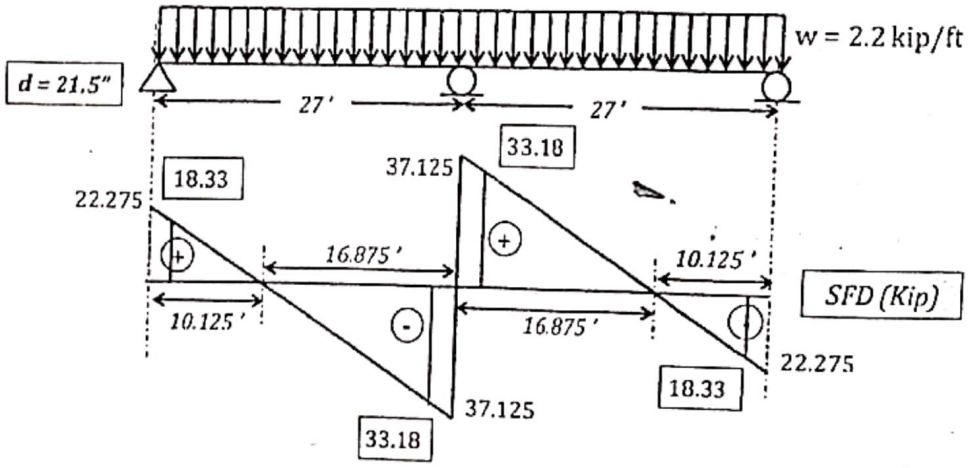


Figure 16: Shear Force Diagram of Beam

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$V_c = 2\lambda\sqrt{f_c'}\, bd (f_c' \text{ in psi})$

$= 2 \times 1 \times \sqrt{3000} \times 12 \times 21.5$

$= 28262.48\ lb$

$= 28.26\ kip$

$\emptyset = 0.75$

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

$= 21.20\ kip$

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

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

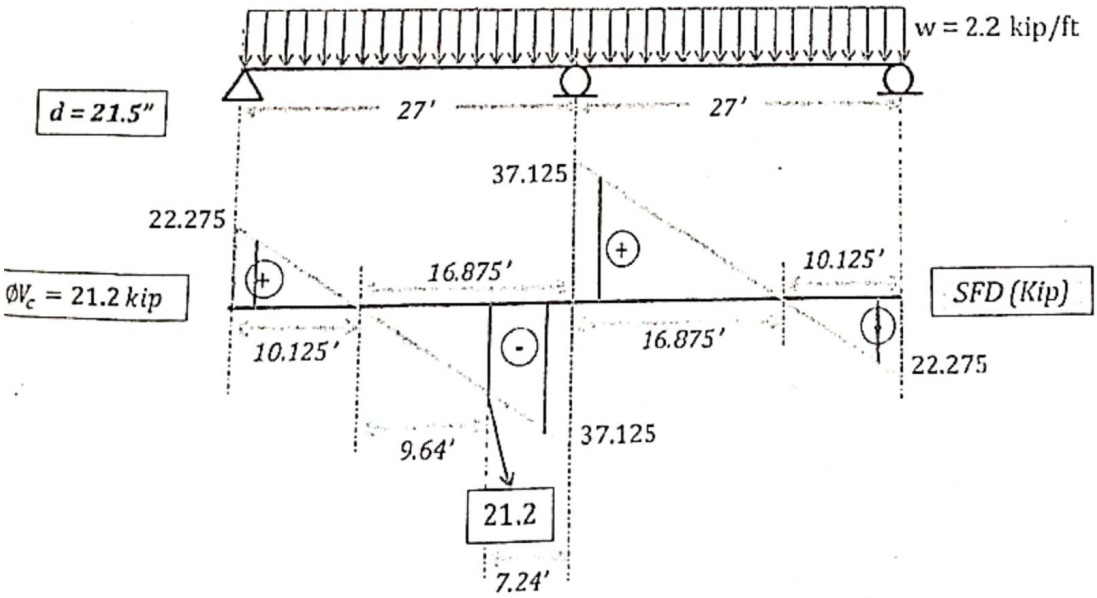


Figure 17: Distance determination where $V_u = \emptyset V_c$

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For 7.24 ft Zone

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

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

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

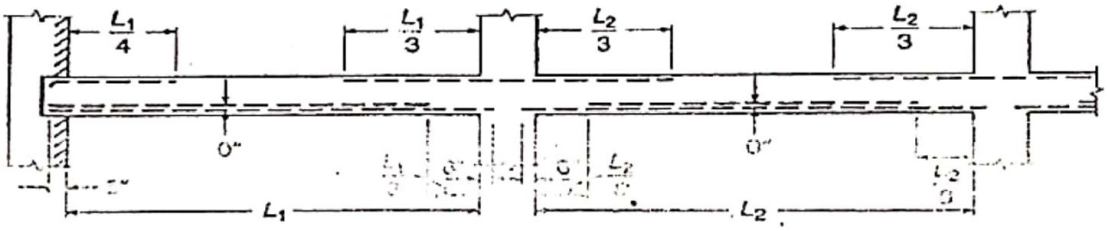


Figure 18: Beam Reinforcement Detailing Guideline

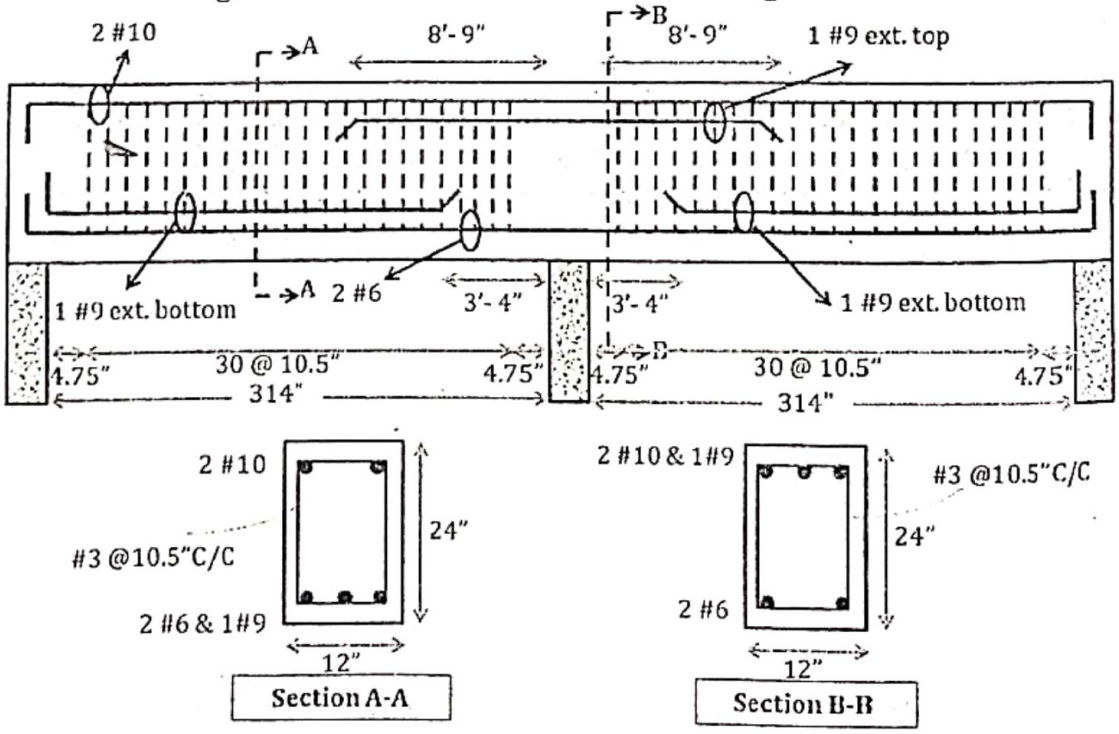


Figure 19: Beam Reinforcement Detailing According to the Design

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DESIGN OF STAIR (USD)

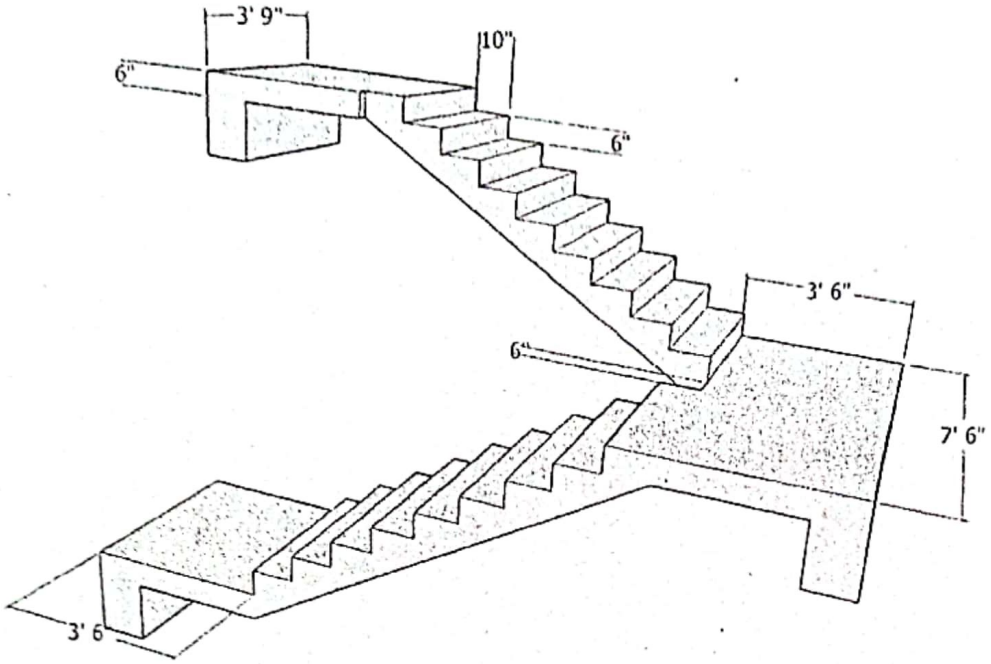


Figure 20: Stair

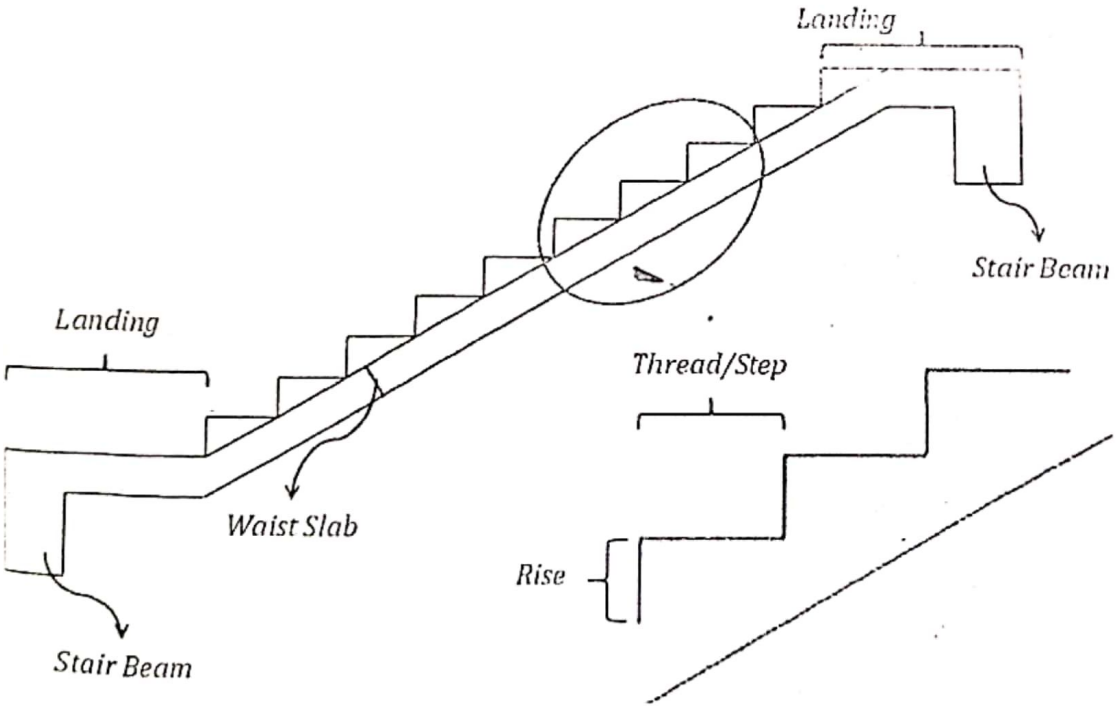
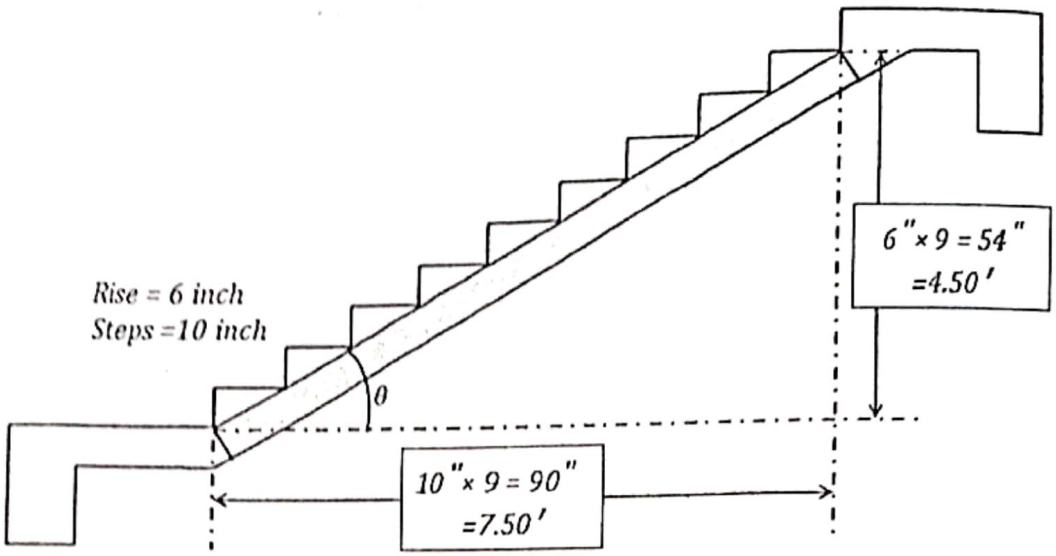


Figure 21: Components of Stair

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

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

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

tep 03 : d check for maximum Moment

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

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

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

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

ep 04 : Reinforcement Area Determination

$A_{s,min} = 0.002\ bt\ (As\ f_y = 40\ ksi)$
$= 0.144\ in^2/ft$

$\boxed{Here, b = 12\ in\ \&\ t = 6\ in}$

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

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

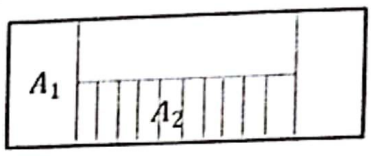
er Direction :

$A_s = 0.144\ in^2/ft$

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

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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"

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

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

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

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

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

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

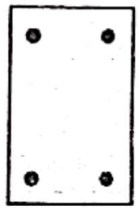
Maximum Moment, $M_{u,max}$ = 11.49 kip-ft = 137.88 kip-in

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

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

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

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



$M(+ve) = 11.49\ kip - ft \longrightarrow A_s(+ve) = 0.42\ in^2 < A_{s,min}$

$M(-ve) = 10.05\ kip - ft \longrightarrow A_s(-ve) = 0.36\ in^2 < A_{s,min}$

$= 0.625\ in^2 (2\#6)$

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

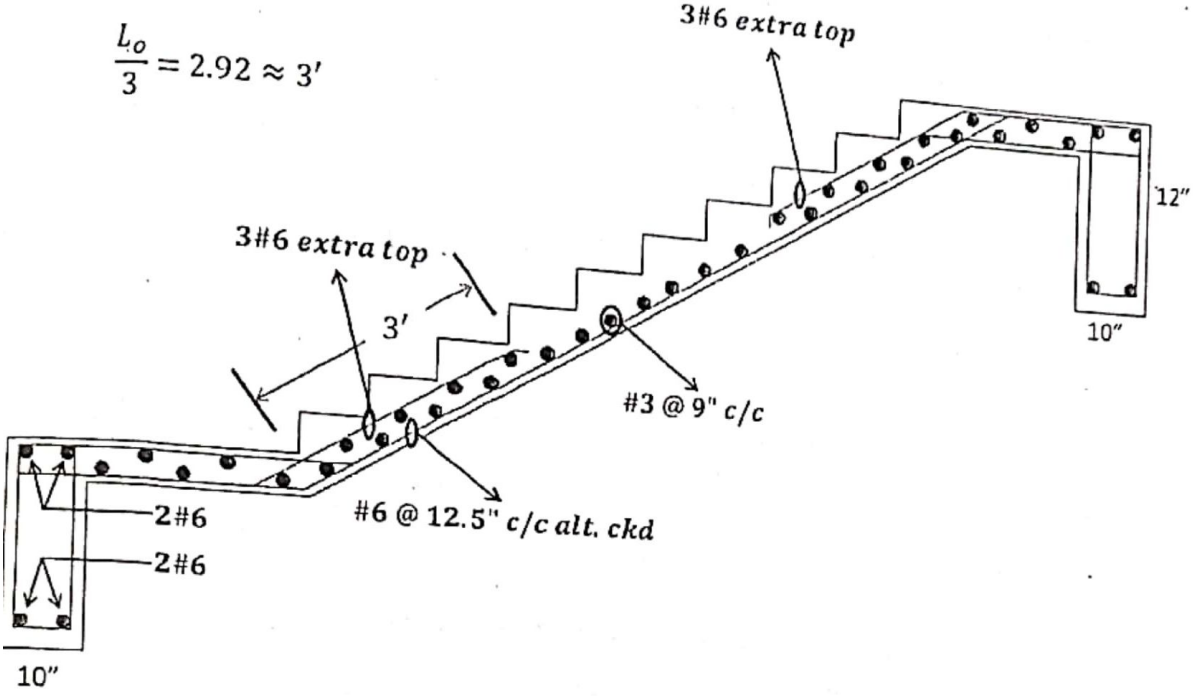


Figure 23: Reinforcement Details of stair.

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4. DESIGN OF SUNSHADE (USD)

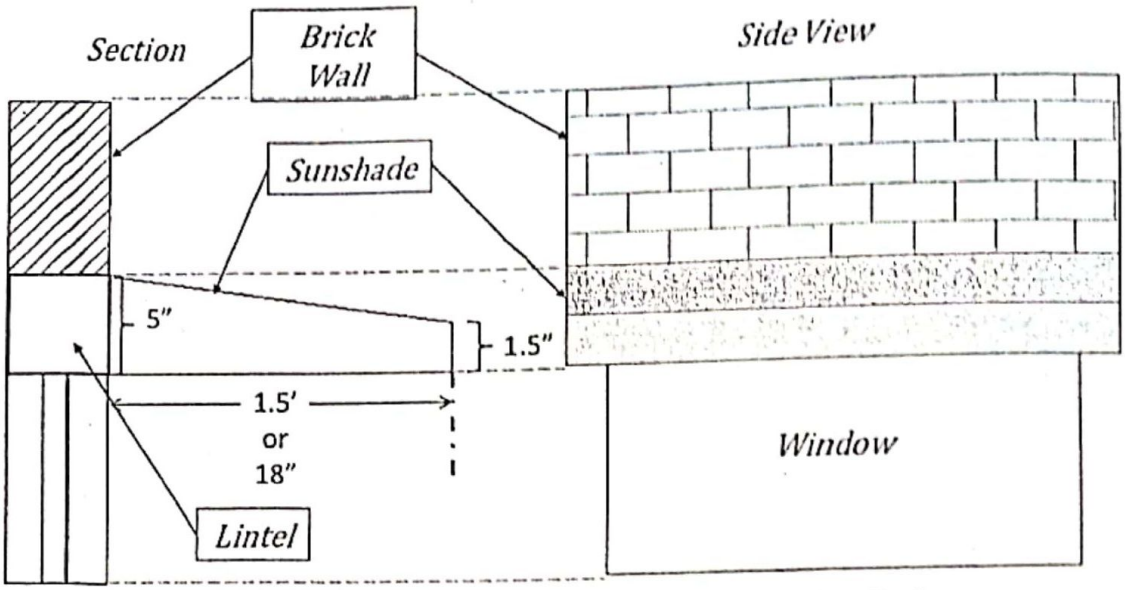


Figure 24: Section and Side View of a Wall with Window

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 :*

$$Floor\ Finish = 25\ psf = 0.025\ ksf$$

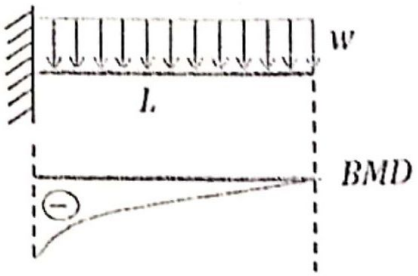
Live Load :

$$Live\ Load = 20\ psf = 0.02\ ksf$$

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:

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



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tep 03 : d check for maximum Moment

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

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

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

$d_{req} = 0.204\ inch$

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

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

tep 04 : Reinforcement Area Determination

$A_{s,min} = 0.002\ bt\ (As\ f_y = 40\ ksi) = 0.078\ in^2/ft$ $\quad$ Here, $b = 12\ in$ & $t = \frac{5 + 1.5}{2} = 3.25in$

$M(-ve) = 0.1251\ kip - ft/ft \longrightarrow A_s(-ve) = 0.0186\ in^2/ft < A_{s,min} = 0.078\ in^2/ft$

tep 05 : Reinforcement Spacing Determination

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

For Both Direction

0.078 in^2 area required in 12inch

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

$= 16.92\ inch$

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

$\therefore$ #3 @ 16.5" c/c

ep 06 : Detailing

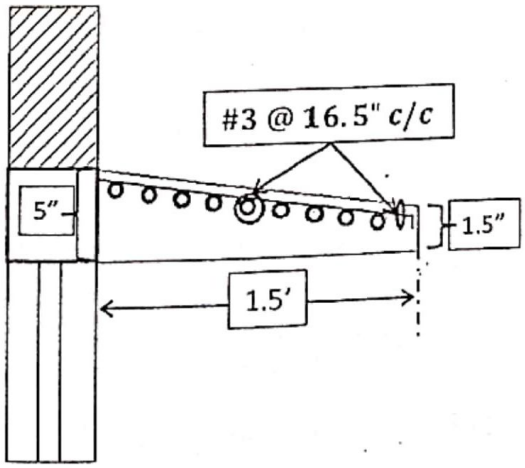


Figure 25: Detailing of Sunshade

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5. DESIGN OF LINTEL (USD)

Assume, Lintel =10"x10"

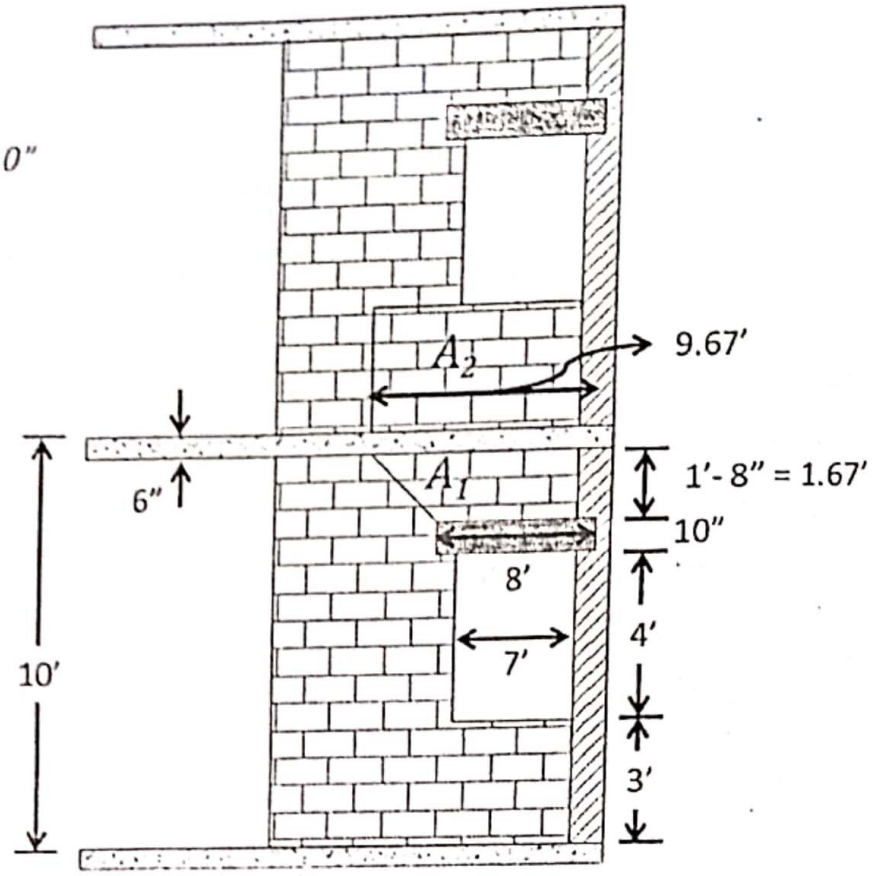


Figure 26: Information necessary to determine lintel load

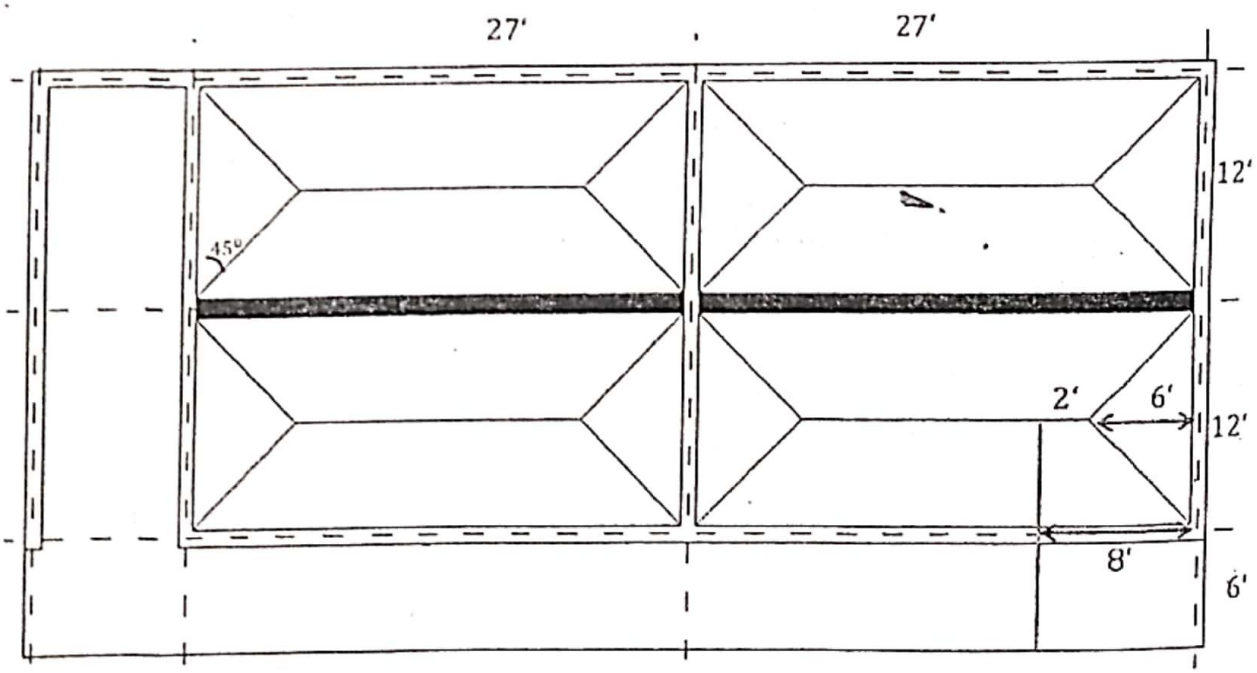


Figure 27: Loads come to Lintel from Slab

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tep 01 : Determination of Load

From Brick wall :

All Dead Load ; No Live Load

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

From Slab :

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

Self Weight of Lintel :

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

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

ep 02 : Moment Determination

aximum Moment, $M_{u,max} = \frac{wl^2}{8} = \frac{2.83 \times 8^2}{8} = 22.64$ Kip-ft

p 03 : d Check

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

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

$d_{req} = 6\ inch$

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

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

p 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)

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Step 05 : Detailing

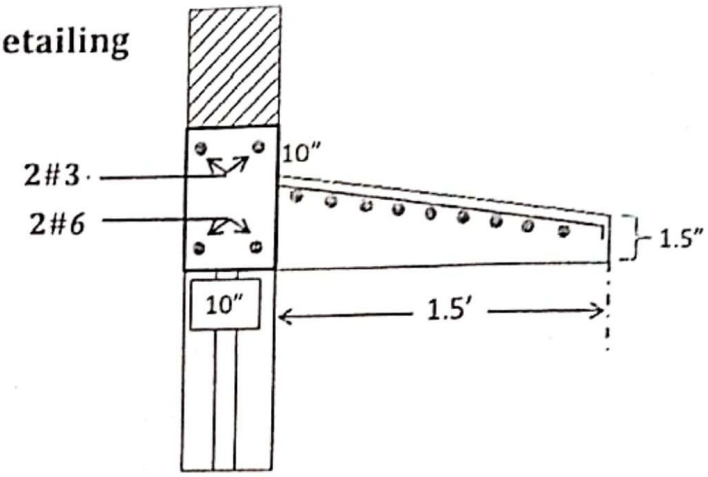


Figure 28: Detailing of Lintel

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DESIGN OF BRICK FOUNDATION (WSD)

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

oundation Depths

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

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

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

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

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

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

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.

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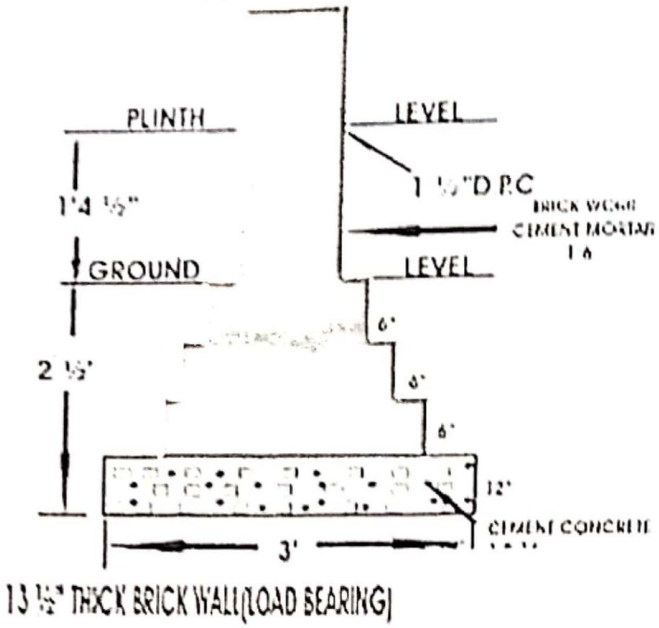


Figure 29: Typical Brick Foundation

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.

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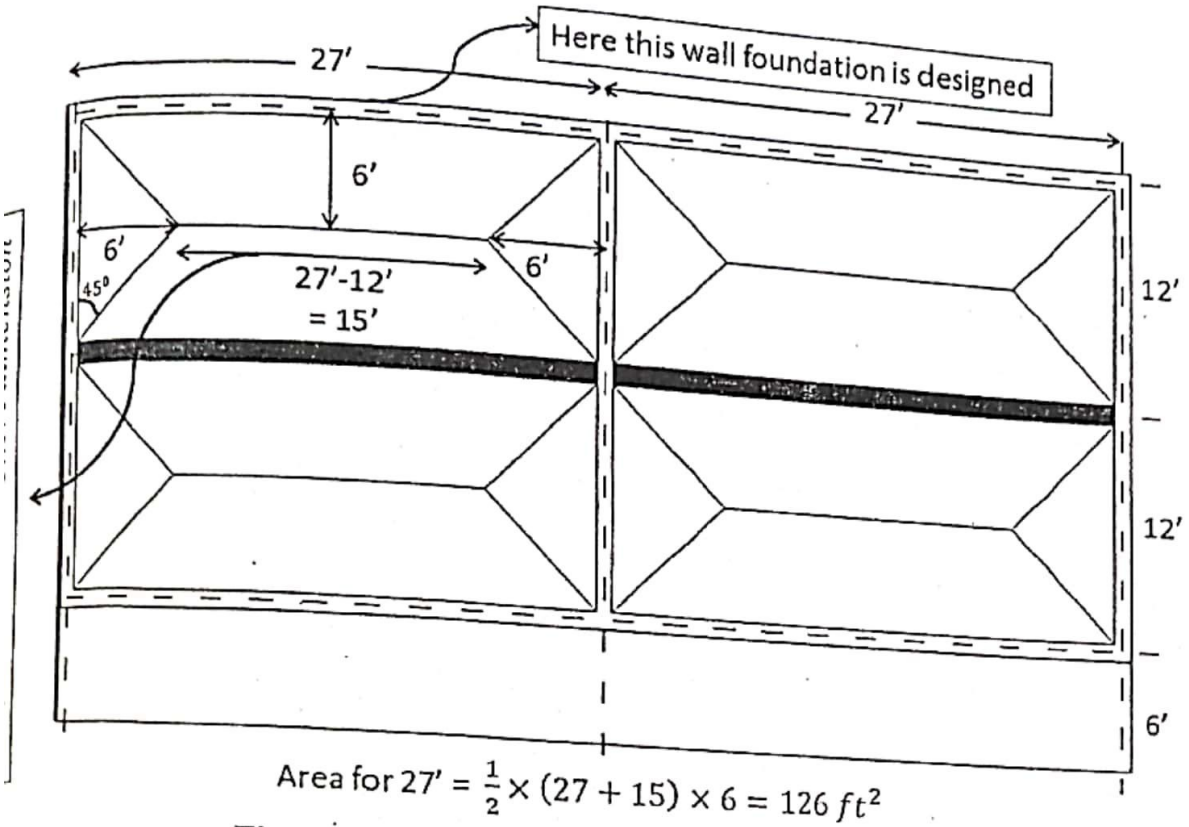


Figure 30: Contributing area of slab on brick wall

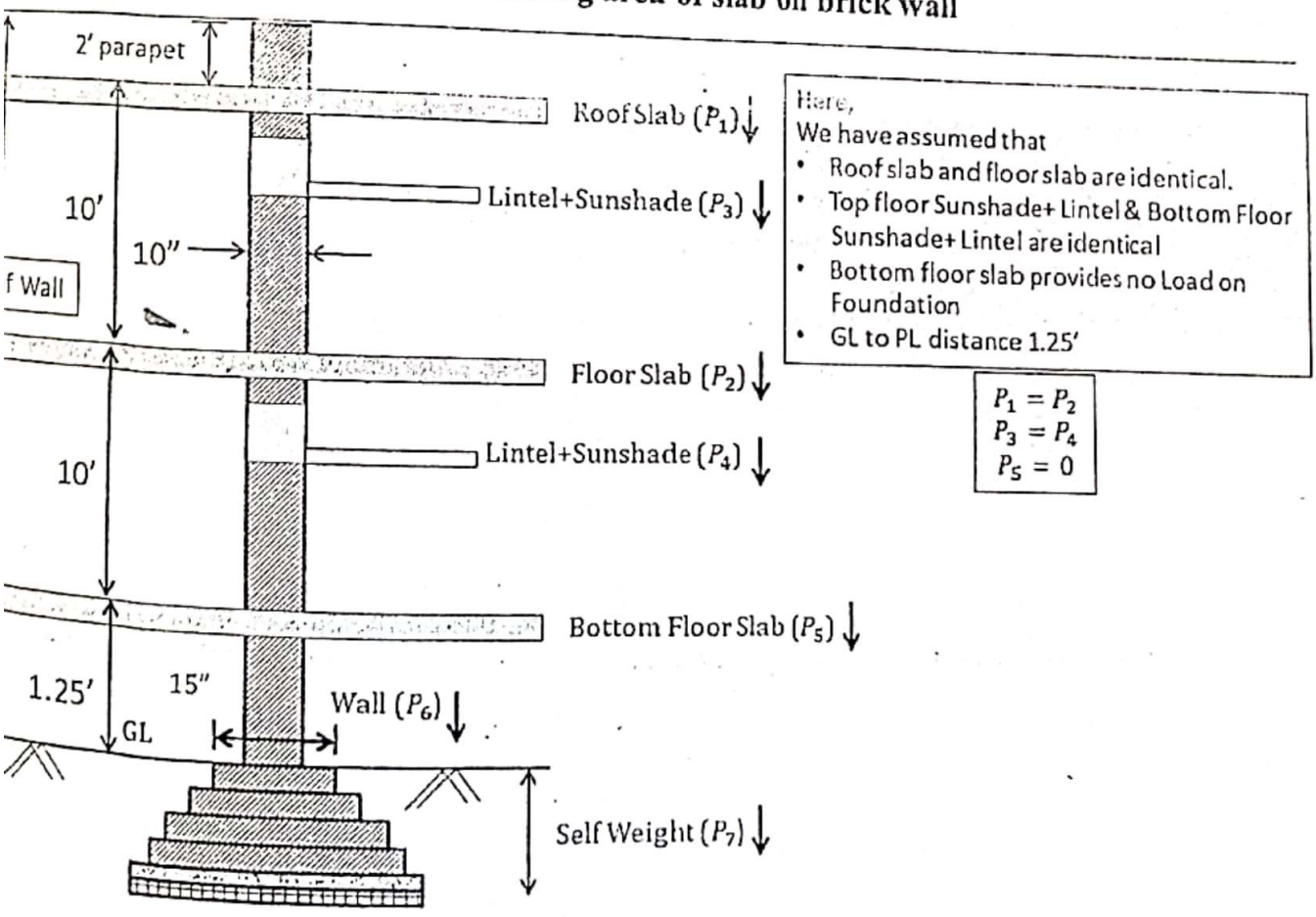


Figure 31: Loads on Foundation

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Step 01 : Determination of Load

$$\text{Unfactored w of Slab} = DL + LL$$
$$= 93.75 + 60$$
$$= 153.75\ psf$$
$$= 0.154\ ksf$$

$$P_1 = P_2 = \text{Contributing Area} \times \text{Unfactored w of slab}$$
$$= (126 \times 0.154)\ kip$$
$$= 19.404\ kip$$

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

$$P_5 = 0\ kip$$

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

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

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

Step 02 : Width Determination

Given,

Bearing Capacity of soil = 1.25 Tsf = 2.5 Ksf

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

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

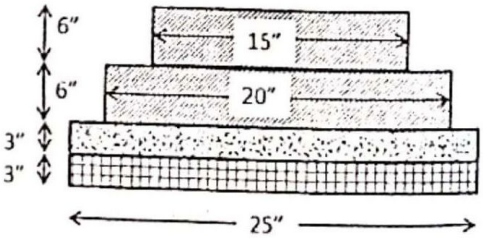
$$B(ft) = 1.73$$

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

Step 03 : Self Weight Check

$$\text{Self Weight} = \frac{15}{12} \times \frac{6}{12} \times 27 \times 0.12 + \frac{20}{12} \times \frac{6}{12} \times 27 \times 0.12$$
$$+ \frac{25}{12} \times \frac{3}{12} \times 27 \times 0.15 + \frac{25}{12} \times \frac{3}{12} \times 27 \times 0.12$$

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



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

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