# CE-4506
# Design of RC and Brick Masonry

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**University of Global Village (UGV), Barishal**

# Design of RC and Brick Masonry

| | |
|---|---|
| **COURSE CODE: CE-4506** | **CREDIT: 04** |
| **MID EXAMINATION: 2 HOURS** | **CIE MARKS: 120** |
| **SEMESTER END EXAMINATION: 3 HOURS** | **SEE MARKS: 80** |

**Course Learning Outcomes (CLOs):** After completing this course successfully, the students will be able to-

| | |
|---|---|
| **CLO 1** | **Apply** principles of structural design to analyze and proficiently design brick masonry elements, including walls and arches, ensuring compliance with relevant building codes and standards. |
| **CLO 2** | **Develop** comprehensive skills in foundation design, encompassing both shallow and deep foundation systems, to support various structures in the context of reinforced concrete and brick masonry applications. |
| **CLO 3** | **Integrate** knowledge of material properties and structural behavior to design efficient and cost-effective retaining walls, optimizing stability and load-bearing capacities. |
| **CLO 4** | **Apply** seismic-resistant design principles to ensure the resilience and safety of reinforced concrete and brick masonry structures in earthquake-prone regions. |
| **CLO 5** | **Demonstrate** proficiency in the design of combined footings, offering solutions for efficiently distributing loads and supporting multiple columns within the framework of reinforced concrete and brick masonry construction. |

| Sl. | Course Contents | Hours | CLOs |
|---|---|---|---|
| 01 | Retaining Wall: Definition, Classification, Common Terminology, Design of Gravity Retaining Wall and Cantilever Retaining Wall. | 10 | CLO1, CLO2 |
| 02 | Flat Slab: Definition, Types, Benefits, Punching Shear, Design of a Flat Slab.<br><br>Flat Plate: Definition, Classification, Benefits, Use, Methods of Design and Design of a Flat Plate Slab. | 10 | CLO1,CLO4 |
| 03 | Combined Footing: Definition, Types, Design of a Combined Footing.<br><br>Two-Way Slab: Definition, Condition, Difference between one way slab and two way slab, Design of a two way slab | 08 | CLO2,CLO3 |

**TEXT BOOKS:**


1. Design of RC and Brick Masonry Structures by Dr.P.Purushothamaraj and Dr.V.Ramaswamy.
2. Design of Concrete Structures by Arthur H. Nilson, David Darwin, Charles W. Dolan (Mc Graw Hill) – 13th edition.
3. Design of Reinforced Concrete by Jack C. McCormac, Russell H. Brown – 9th edition

| Week | Topic | Teaching Learning Strategy | Assessment Strategy | CLOs |
|---|---|---|---|---|
| 01 | Introduction to Retaining Wall | Lecture, Discussion | Mid, Final | CLO 1 |
| 02-03 | Design of Gravity Retaining Wall | Lecture, Discussion | Mid, Final | CLO 1 |
| 04-05 | Design of Cantilever Retaining Wall | Lecture, Presentation | Assignment, Mid, Final | CLO 3 |
| 06 | Introduction to Flat Slab | Lecture, Presentation | Assignment, Mid, Final | CLO 2 |
| 07-08 | Design of a Flat Slab | Lecture, Presentation | Assignment, Class Test, Mid, Final | CLO 4 |
| 09 | Introduction to Flat Plate Slab | Lecture, Presentation | Problem solving, Class Test, Final | CLO 4 |
| 10-11 | Design of a Flat Plate Slab | Lecture, Discussion | Mid, Final | CLO 2 |
| 12-13 | Introduction and Design of Combined Footing | Lecture, Presentation | Problem solving, Class Test, Final | CLO 3 |
| 14-15 | Design of two-way Slab | Lecture, Presentation | Assignment, Mid, Final | CLO 3 |
| 16-17 | Doubt Solving Class | Lecture, Discussion | Assignment, Final | |

**ASSESSMENT PATTERN**

**CIE-Continuous Internal Evaluation (120 Marks)**

<table>
<tr><th>Bloom’s Category Marks<br><br>(out of 120)</th><th>Tests<br><br>(60)</th><th>Assignments<br><br>(20)</th><th>Quizzes<br><br>(20)</th><th>External Participation in Curricular/Co-Curricular Activities (20)</th></tr>
<tr><td>Remember</td><td>10</td><td></td><td>10</td><td rowspan="6">Attendance<br><br>20</td></tr>
<tr><td>Understand</td><td>10</td><td></td><td>10</td></tr>
<tr><td>Apply</td><td>10</td><td></td><td></td></tr>
<tr><td>Analyze</td><td>10</td><td></td><td></td></tr>
<tr><td>Evaluate</td><td>10</td><td></td><td></td></tr>
<tr><td>Create</td><td>10</td><td>20</td><td></td></tr>
</table>

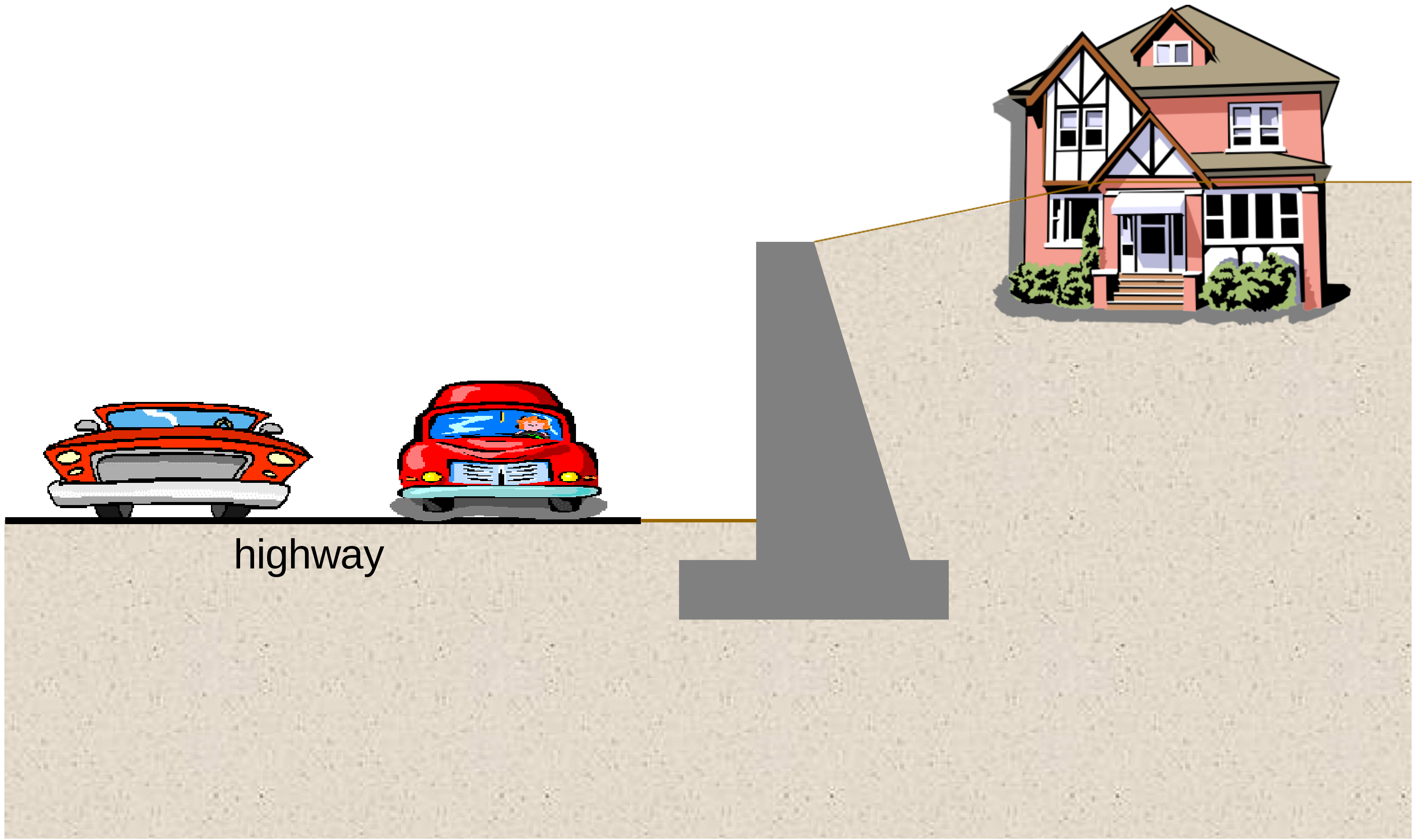
**SEE- Semester End Examination (80 Marks)**

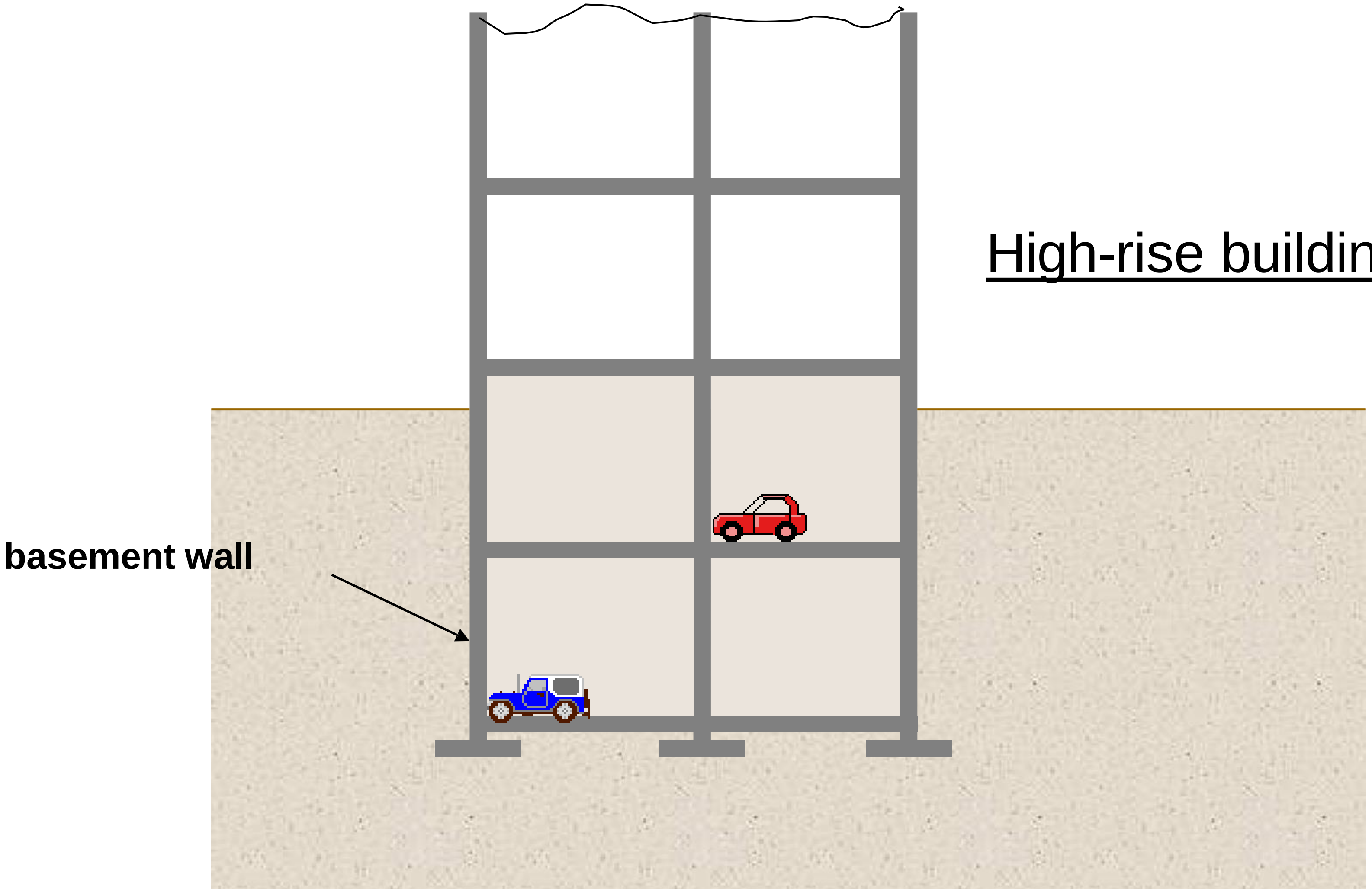
| Bloom’s Category | Tests |
|---|---|
| Remember | 10 |
| Understand | 15 |
| Apply | 15 |
| Analyze | 15 |
| Evaluate | 15 |
| Create | 10 |

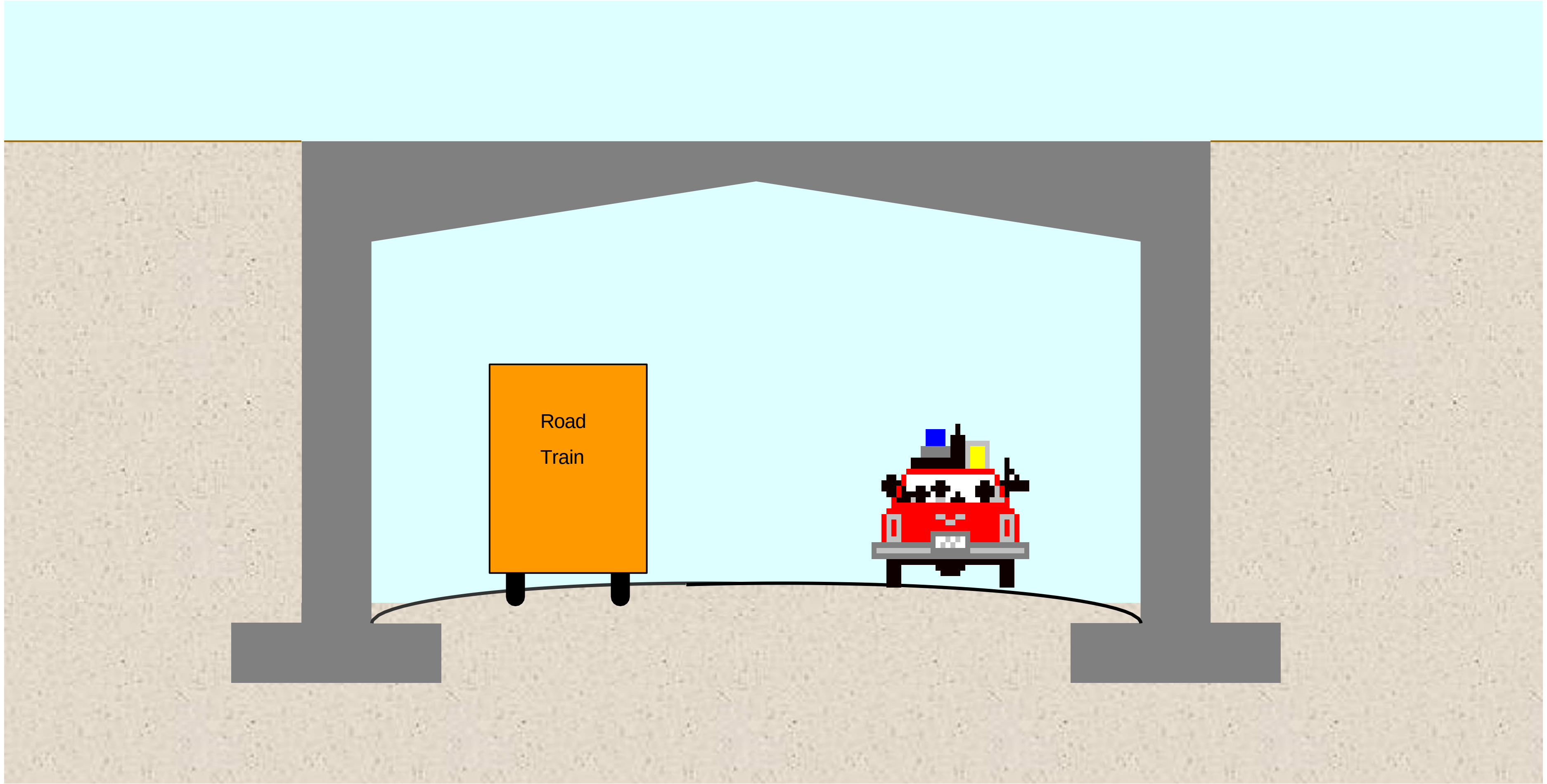
# Introduction to Retaining Wall

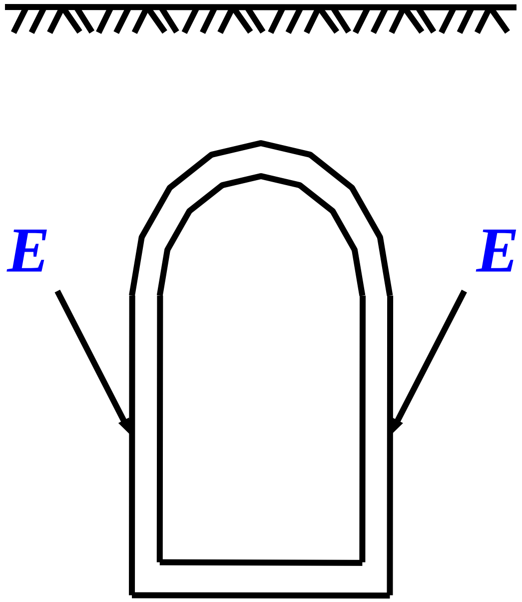
**(Week 01)**

# Retaining Wall

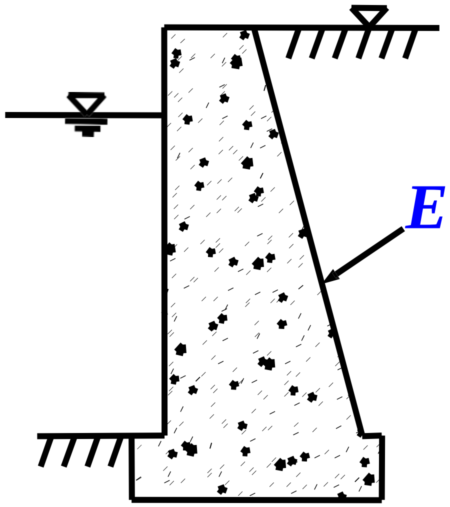

highway


High-rise building
basement wall

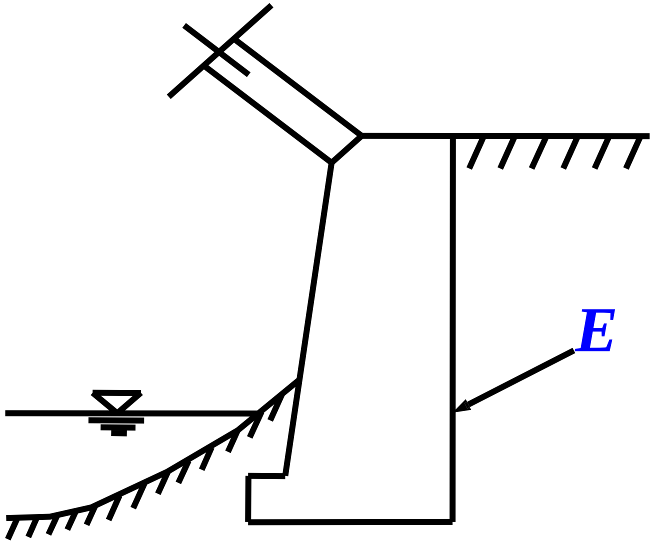

Road
Train



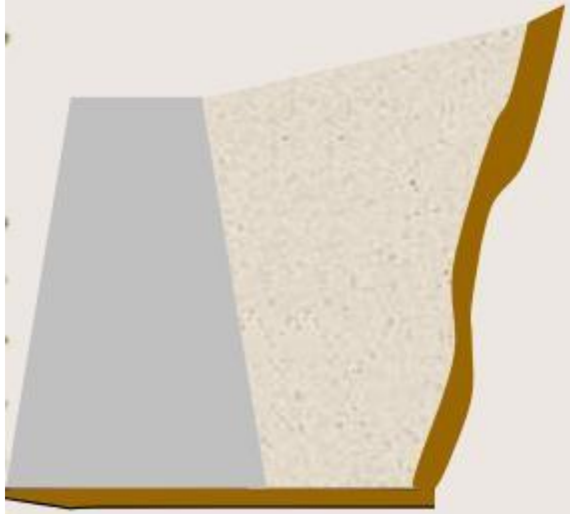

Tunnel



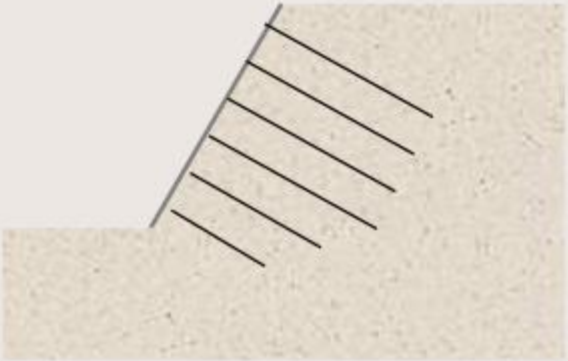

Dock



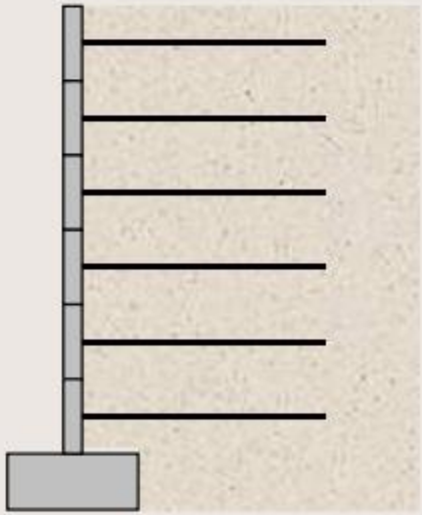

Abutment



Gravity Retaining wall

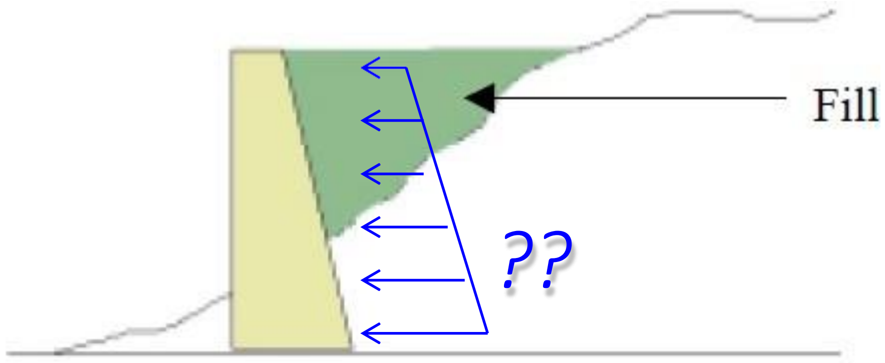


Soil nailing

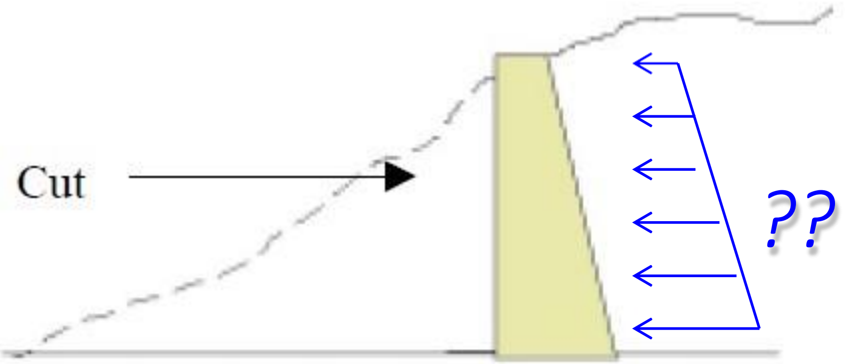


Reinforced earth wall

# Lateral Earth Pressure



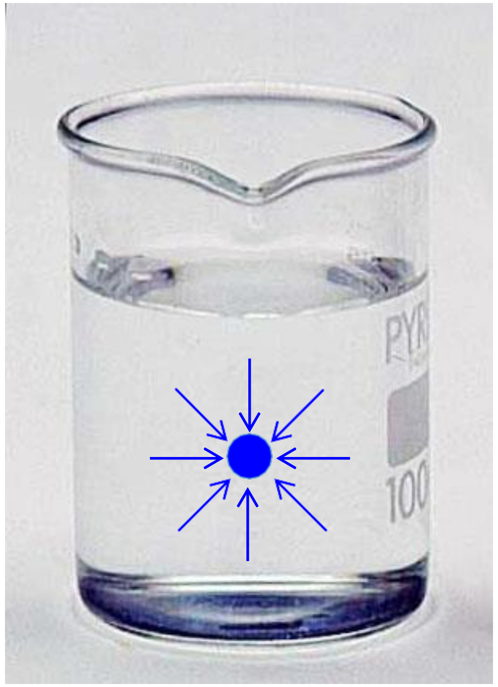

Retaining Wall to Support a Fill.




Retaining Wall to Support a Cut.

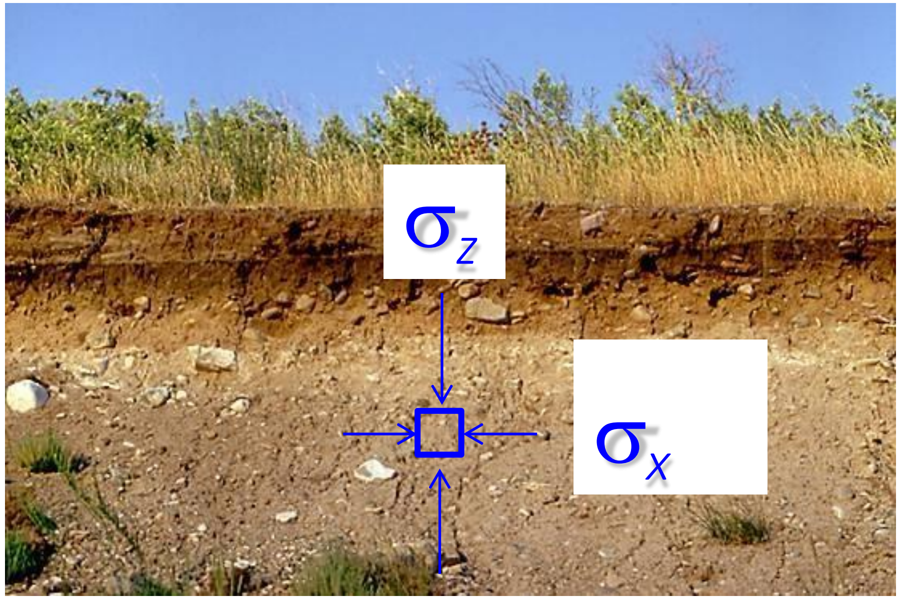
# Water Pressure and Soil Pressure

*Consider hydrostatic condition*




*Isotropic*

*Consider "at-rest" (geostatic) condition*



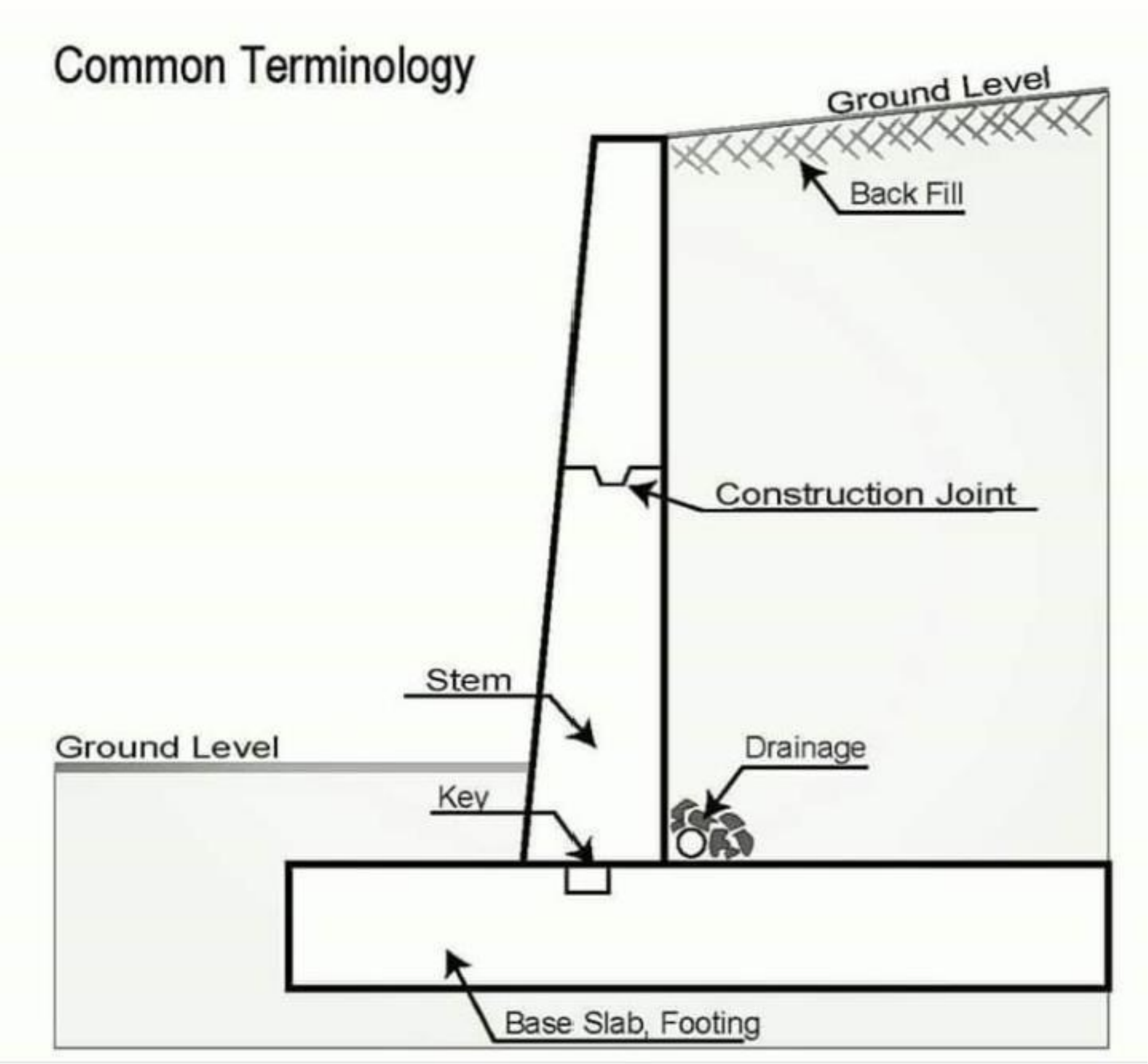

*Anisotropic*

$\sigma_z \neq \sigma_x \qquad \sigma_z > \sigma_x$







Common Terminology
Ground Level
Back Fill
Construction Joint
Stem
Ground Level
Drainage
Key
Base Slab, Footing

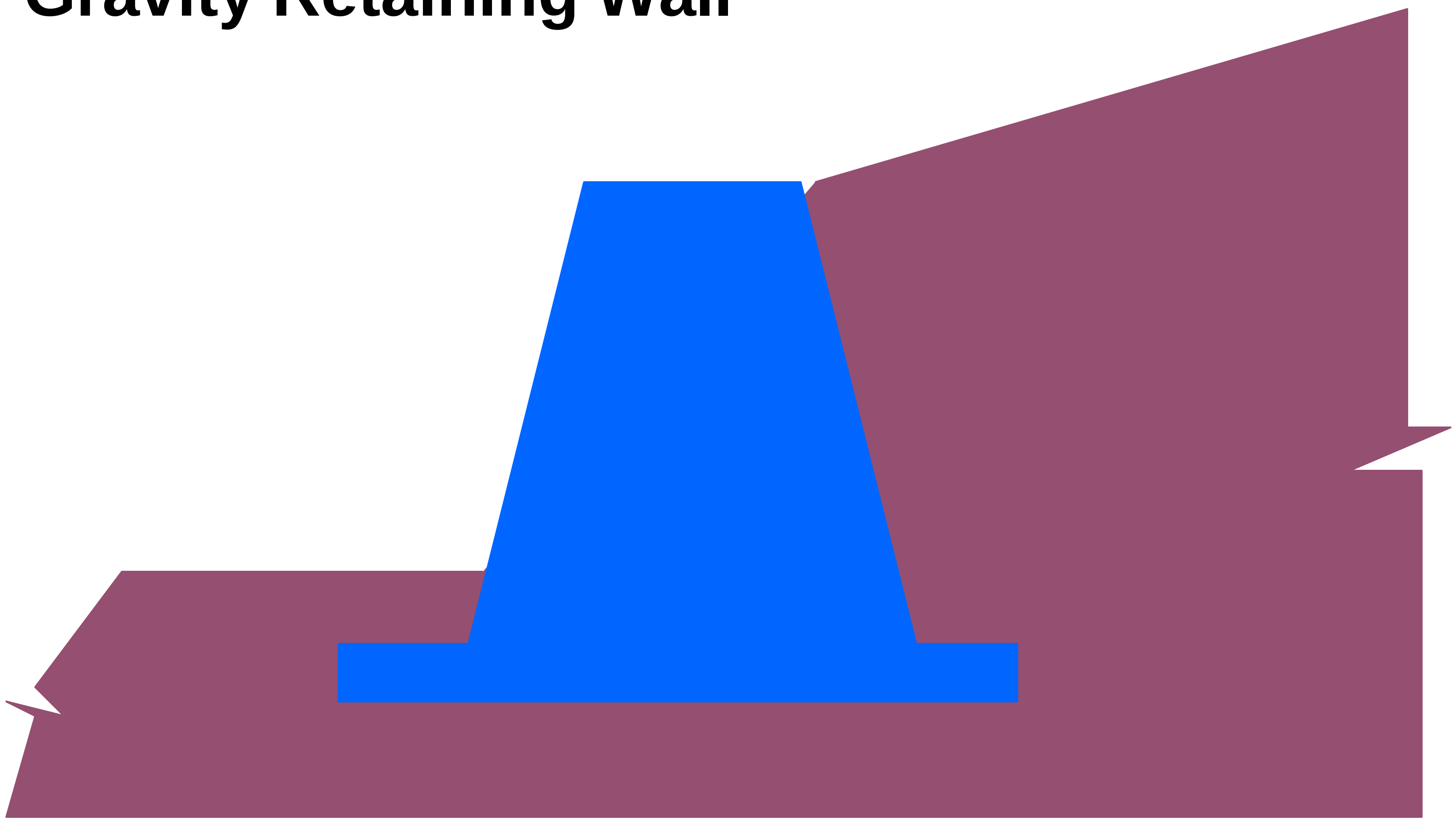
# Definition

- ❖Retaining wall is used to hold back masses of earth or other loose material where conditions make it impossible to let those masses assume their natural slope.
- ❖Two types-1) Gravity retaining wall

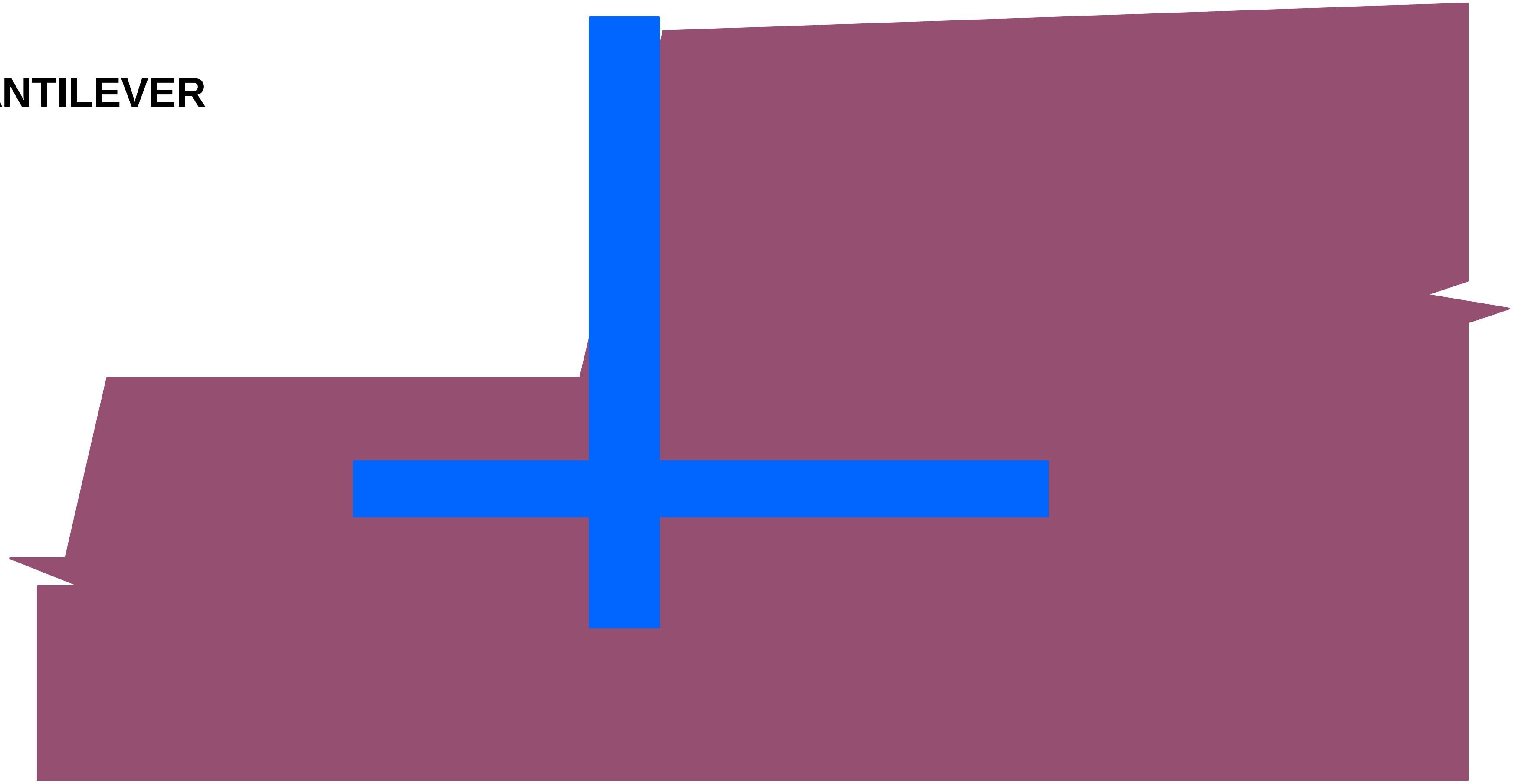
  2) Cantilever retaining wall.

  3) Counterfort retaining wall
- ❖ Loading-1) Self weight, 2) Structure & 3) Water

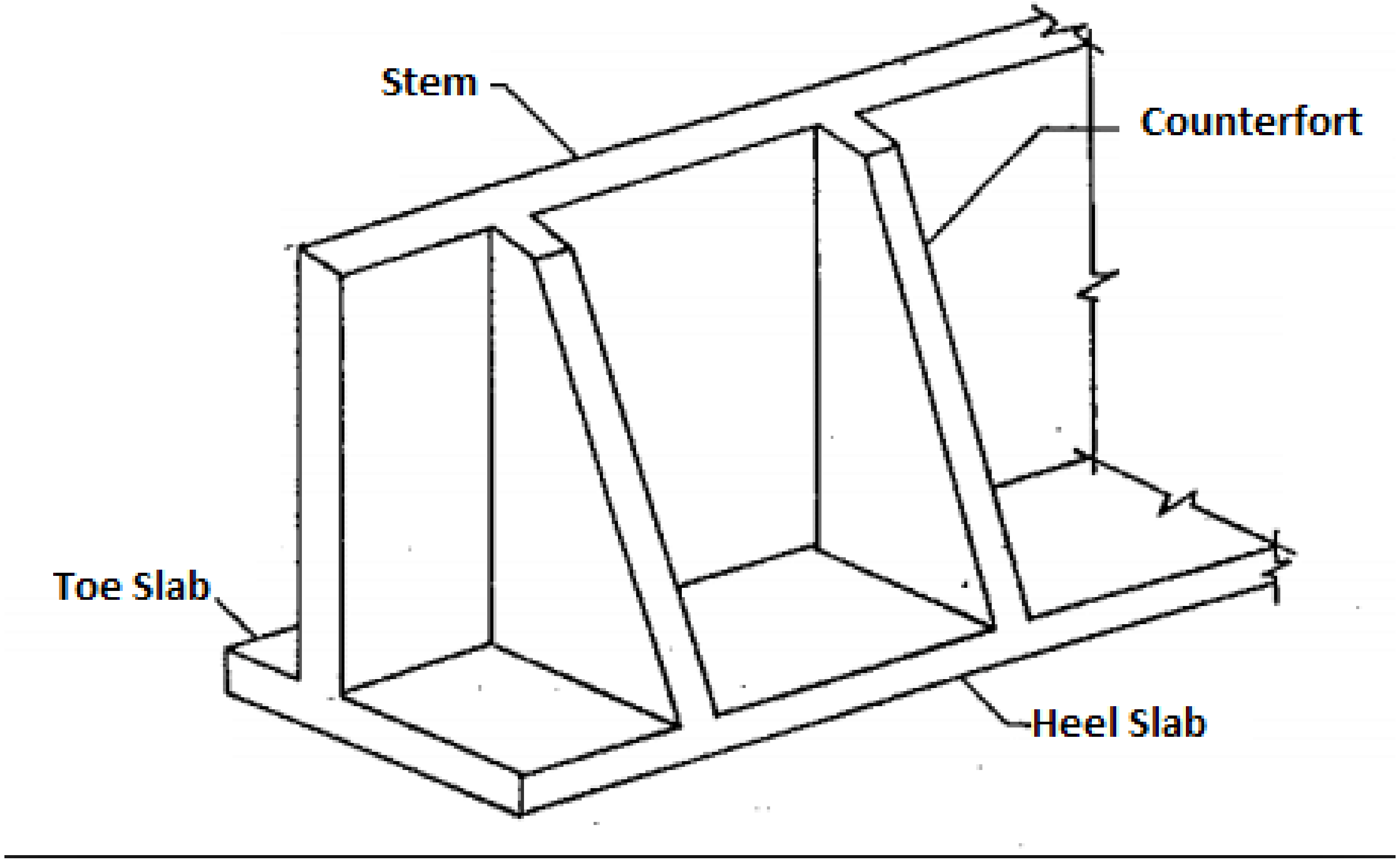
# Gravity Retaining Wall



**CANTILEVER**



# COUNTERFORT Retaining Wall



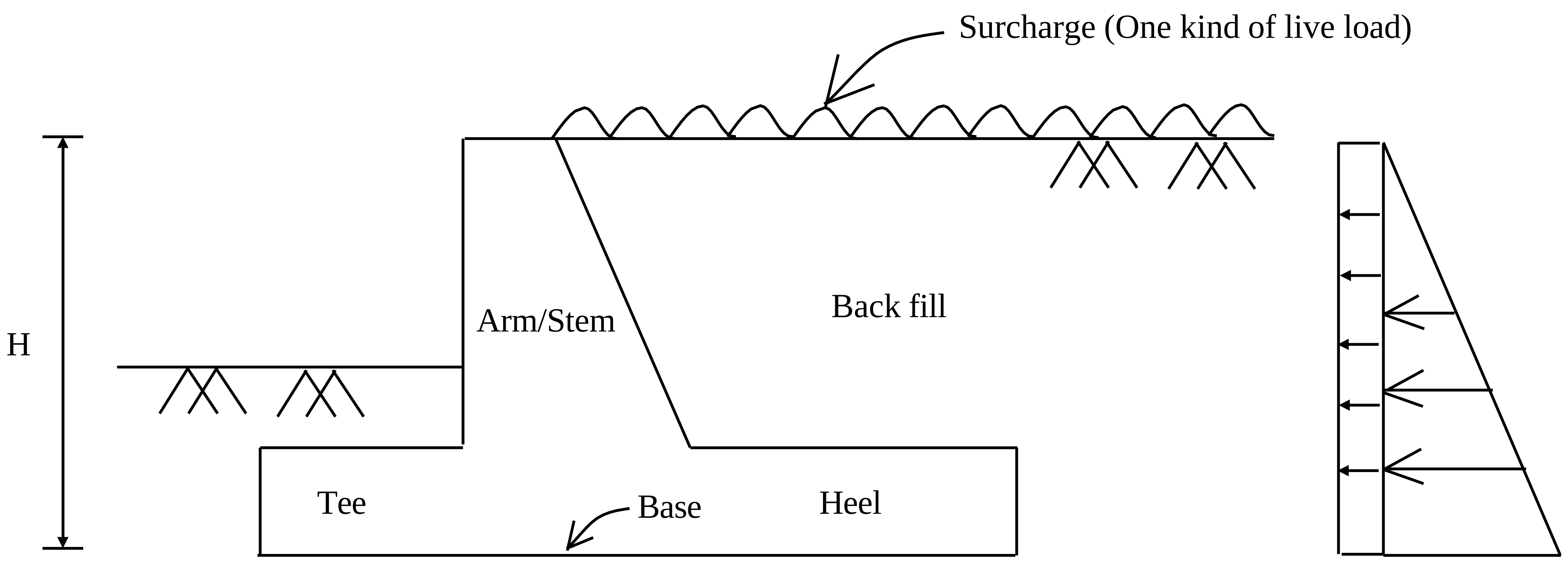



Fig.-Retaining Wall

# Mode of Failure

- Internal Stability
- External Stability
  - Sliding Failure
  - Overturning Failure
  - Base Failure

**Sliding Failure:**

Factor of safety against sliding = $\frac{Resisting\ force}{Sliding\ force} \geq 1.5$

**Overturning Failure:**

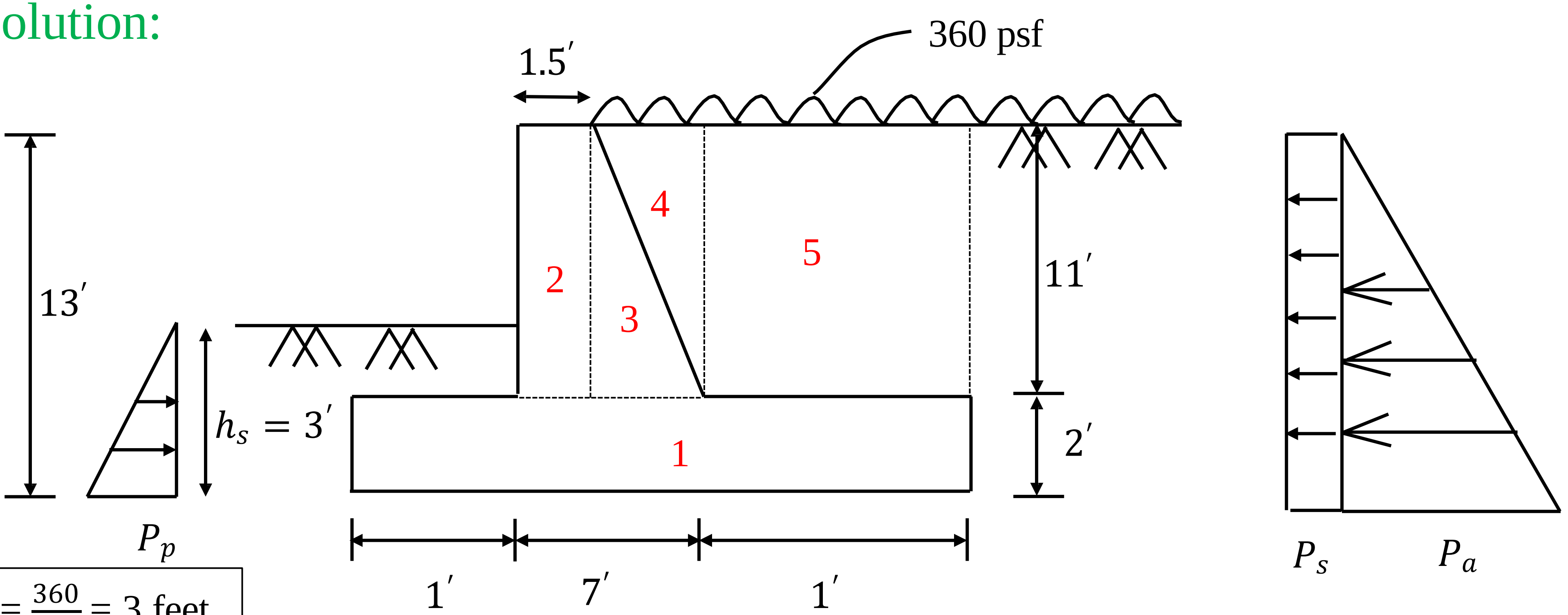
Factor of safety against overturning = $\frac{\text{Resisting}\ Moment}{\text{Overturning}\ Moment} \geq 2.5$

# Design of Gravity Retaining Wall

## (Week 02-03)

Problem-1: Design of a gravity retaining wall to retain a bank of height 10 ft with horizontal surface is subjected to a surcharge of 360 psf. Assume unit weight of soil, $\gamma_s =$ 120 psf. Angle of internal friction, $\varphi = 30°$. Base friction co-efficient, f = 0.5 and allowable bearing pressure, $q_{all}$ = 6000 psf. Unit weight of concrete, $\gamma_c$= 150 psf.

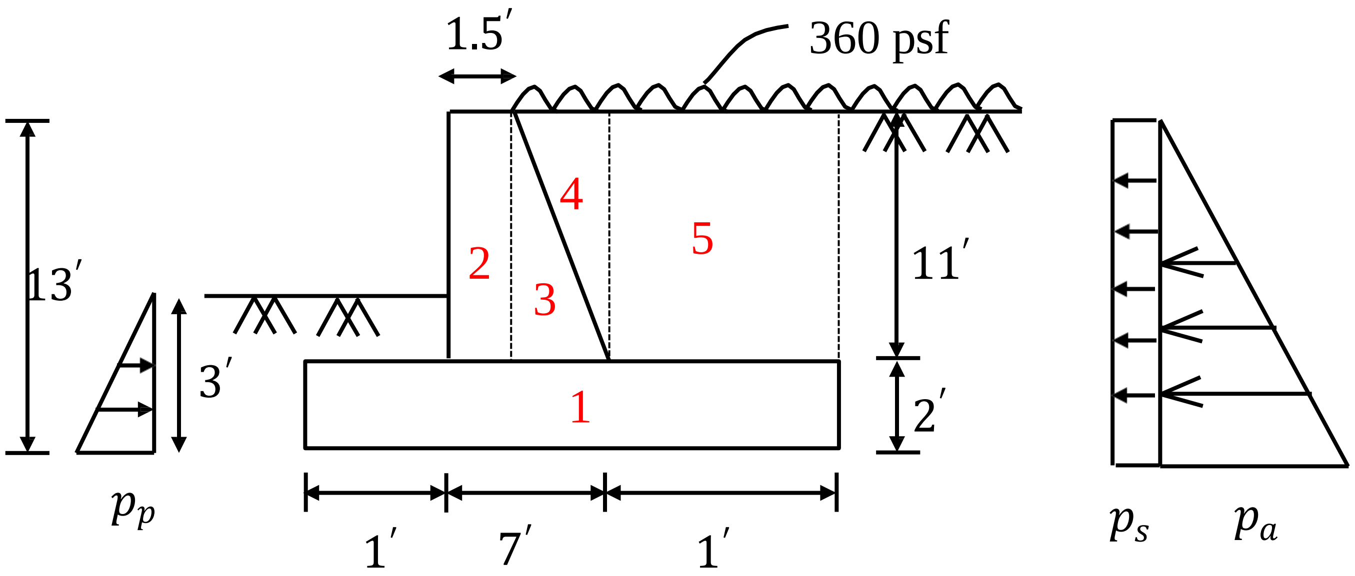
## Solution:




$h_s = \frac{360}{120} = 3$ feet

Overall height of the retaining wall = 10 + 3 = 13 feet

Base width = $\frac{2}{3}$ x Overall height = $\frac{2}{3}$ x 13 = 8.67 feet ≈ 9 feet




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

$k_p = 1/k_a = 3$

$p_s = k_a q$

$= \frac{1}{3}$ x 360

= 120 psf

$p_a = k_a \gamma H$

$= \frac{1}{3}$ x 120 x 13

= 520 psf

$p_p = k_p \gamma h_s$

= 3 x 120 x 3

= 1080 psf

Total active earth pressure , P = $\frac{1}{2}$ x $p_a$H + $p_s$H

= 0.5 x 520 x 13 + 120 x 13

= 4940 lb

Overturning moment , $M_o$ = $\frac{1}{2}$ x $p_a$H x $\frac{H}{3}$ +$p_s$H x $\frac{H}{2}$

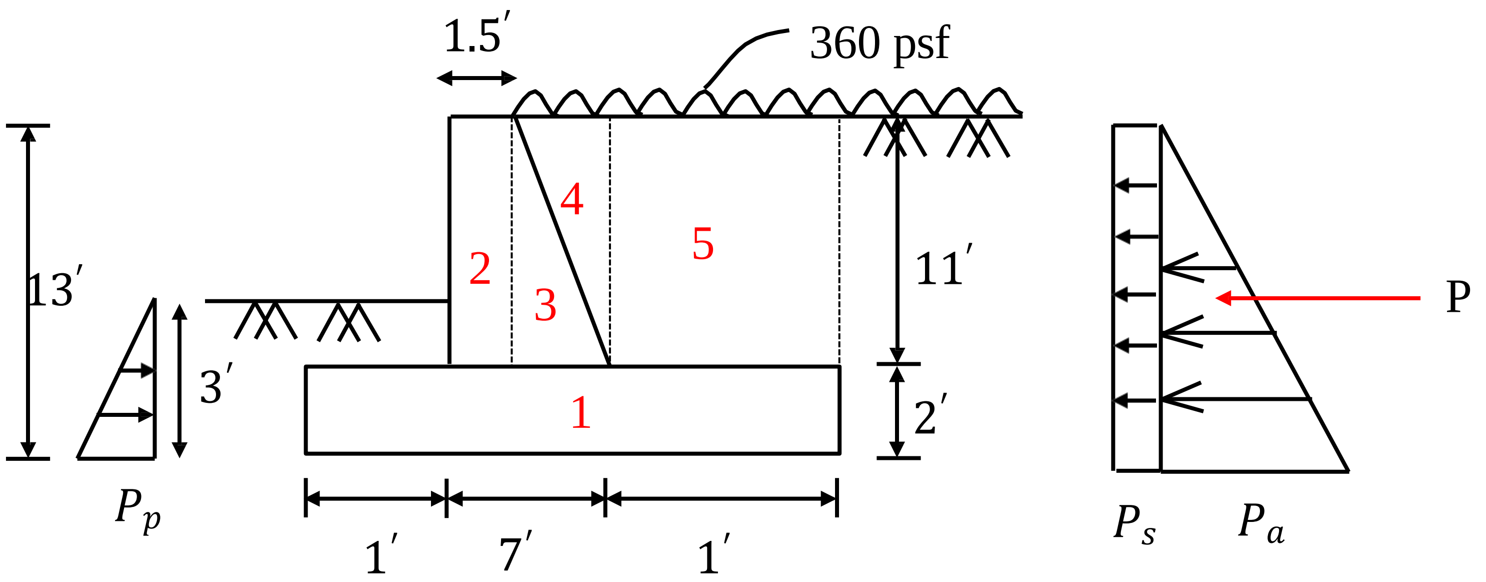
= 3380 x $\frac{13}{3}$ + 1560 x $\frac{13}{2}$

= 24787 lb-ft

Total passive earth pressure , $P_p$ = $\frac{1}{2}$ x $p_p h_s$

= 0.5 x 1080 x 3

= 1620 lb




| Section | W (lb) | Moment arm (ft) | Resisting moment, $M_R$ (lb-ft) |
|---|---|---|---|
| 1 | 9 x 2 x 150 = 2700 | 9/2 = 4.5 | 12150 |
| 2 | 1.5 x 11 x 150 = 2475 | 1 + 1.5/2 = 1.75 | 4331.25 |
| 3 | 0.5 x 5.5 x 11 x 150 = 4537.5 | 2.5 + 5.5/3 = 4.33 | 13662.5 |
| 4 | 0.5 x 5.5 x 11 x 120 = 3630 | 8 - 5.5/3 = 6.17 | 22385 |
| 5 | 1 x 11 x 120 = 1320 | 9-1/2 = 8.5 | 11220 |
| Total | 14662.5 | | 69748.75 |

## Check for Sliding Failure:

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

$$= \frac{w\ x\ f + \cancel{P_p}^{\,0}}{P}$$

$$= \frac{14662.5\ x\ 0.5}{4940}$$

$= 1.48 \approx 1.5$ (OK)

## Check for Overturning Failure:

Factor of safety against overturning = $\frac{\text{Resisting } Moment}{\text{Overturning } Moment}$

$$= \frac{M_R}{M_0}$$

$$= \frac{69748.75}{24786.67}$$

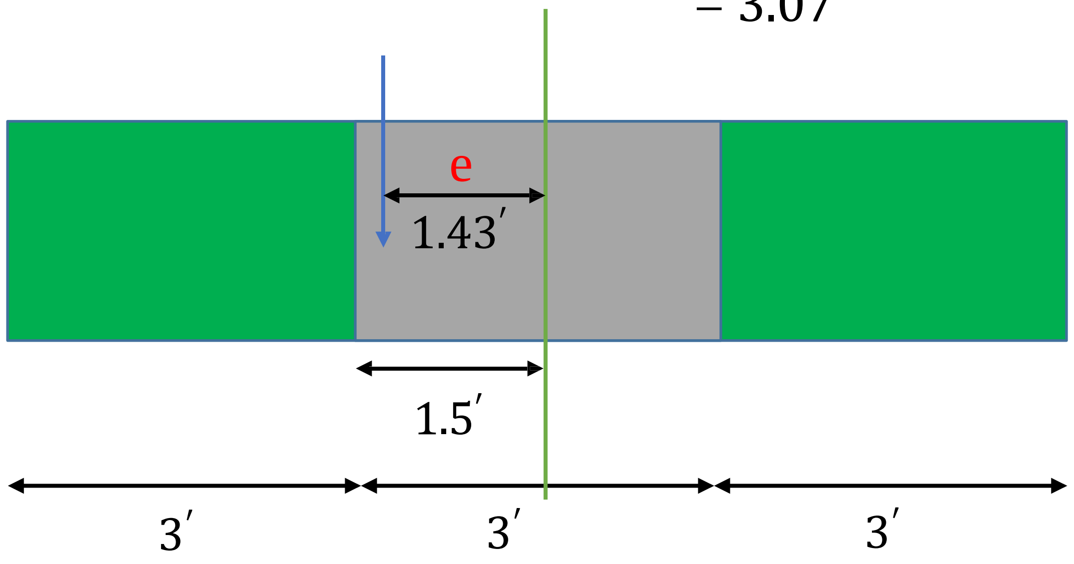
$= 2.81 > 2.5$ (OK)

## Check for base failure:

Resultant of total vertical load from left corner, a = $\dfrac{M_R - M_0}{W}$

$$= \frac{69748.75 - 24786.67}{14662.5}$$

$$= 3.07'$$




e = $\dfrac{B}{2}$ - a = $\dfrac{9}{2}$ - 3.07 = 1.43 feet

M = We

= 14662.5 x 1.43 = 20967.375 lb-ft

A = B x 1 = 9 $ft^2$

I = $\frac{1\ x\ 9^3}{12}$ = 60.75 $ft^4$

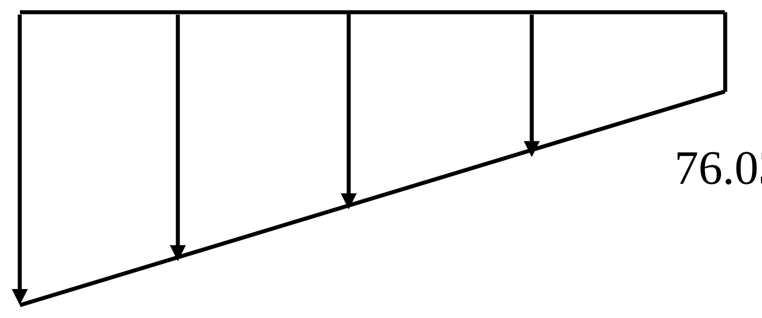
σ = $\frac{W}{A} \pm \frac{MC}{I}$

$\sigma_1 = \frac{W}{A} + \frac{MC}{I} = \frac{14662.5}{9} + \frac{20967.375\ x\ \frac{9}{2}}{60.75}$

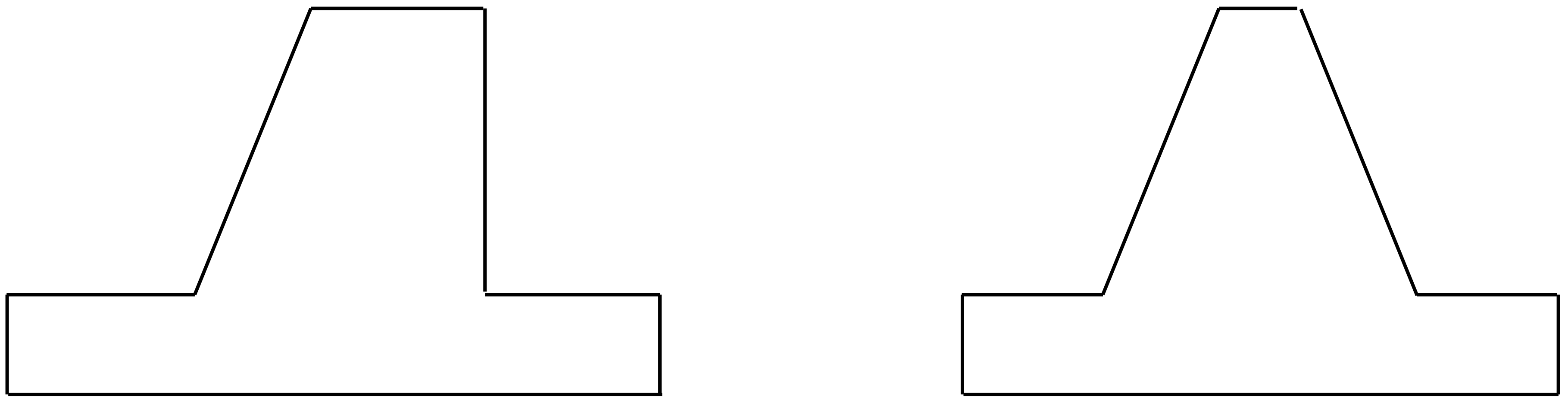
= 3182.31 psf (OK)

$\sigma_2 = \frac{W}{A} - \frac{MC}{I} = \frac{14662.5}{9} - \frac{20967.375\ x\ \frac{9}{2}}{60.75}$

= 76.03 psf (OK)



# Assignment-1

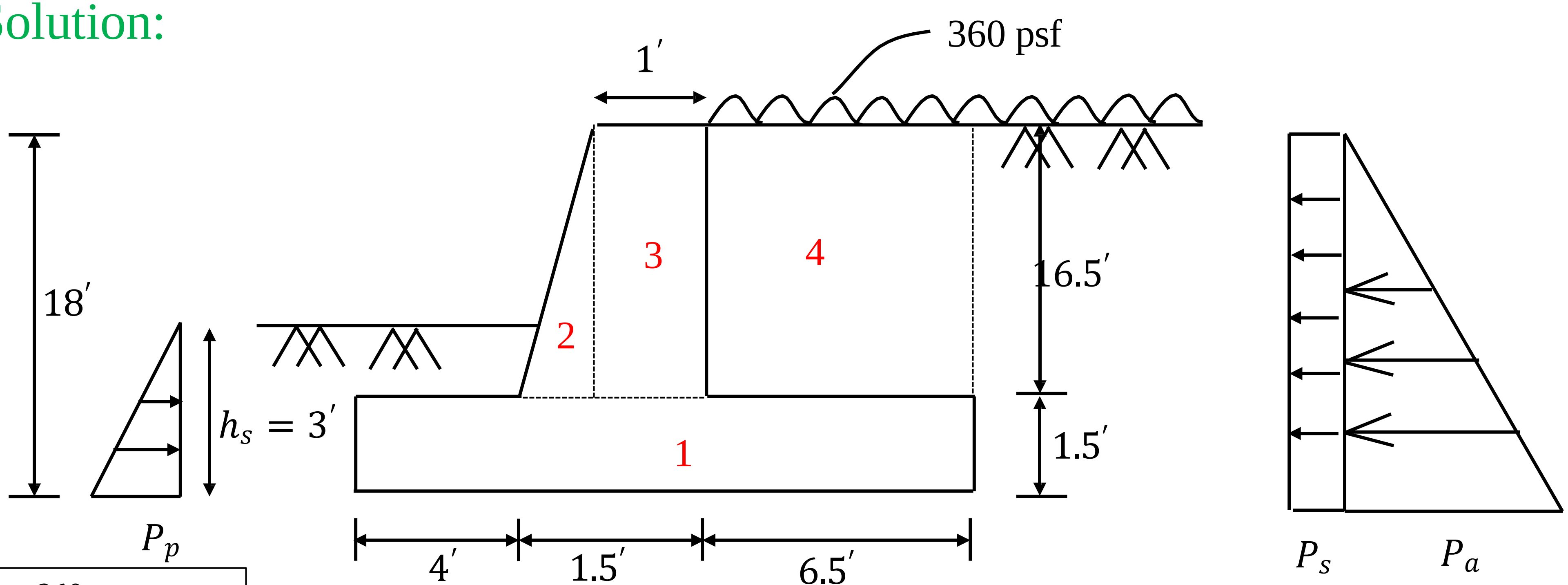


# Design of Cantilever Retaining Wall

## (Week 04-05)

Problem-2: Design of a cantilever retaining wall to retain a bank of earth which is 15 ft high and support a surcharge of 360 psf. Assume unit weight of soil, $\gamma_s =$ 120 psf. Angle of internal friction, $\varphi = 30°$. Base friction co-efficient, f = 0.5 and allowable bearing pressure, $q_{all}$ = 3 ksf. Unit weight of concrete, $\gamma_c$= 150 psf. $f_c{}'$ = 3 ksi & $f_s$ = 24 ksi.

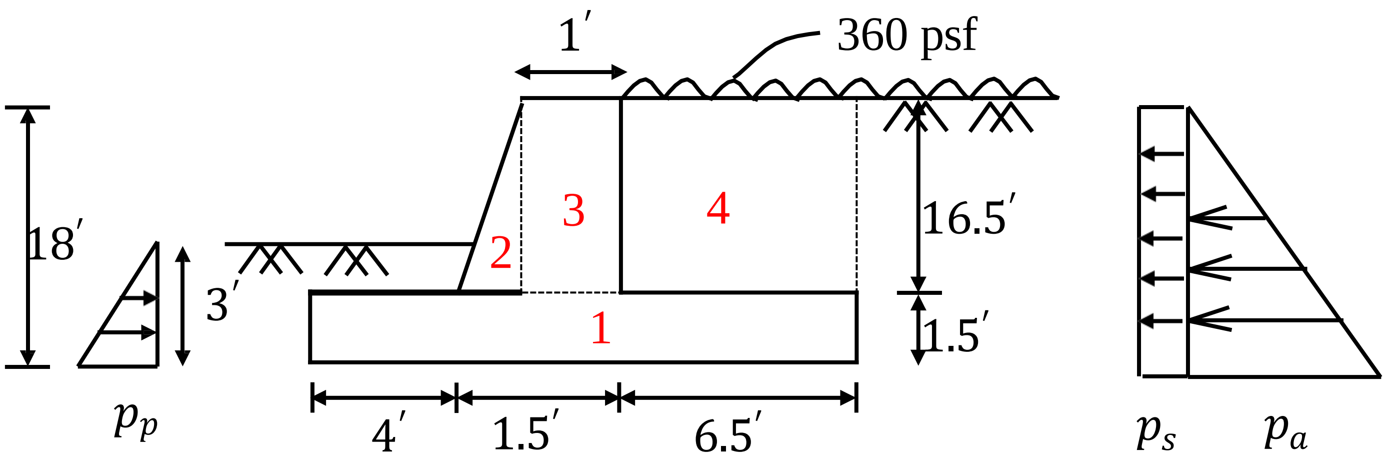
Solution:




$h_s = \frac{360}{120} = 3$ feet

Overall height of the retaining wall = 15 + 3 = 18 feet

Base width = $\frac{2}{3}$ x Overall height = $\frac{2}{3}$ x 18 = 12 feet




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

$k_p = 1/k_a = 3$

$p_s = k_a q$

$= \frac{1}{3}$ x 360

$= 120$ psf

$p_a = k_a \gamma H$

$= \frac{1}{3}$ x 120 x 18

$= 720$ psf

$p_p = k_p \gamma h_s$

$= 3$ x 120 x 3

$= 1080$ psf

Total active earth pressure , P = $\frac{1}{2}$ x $p_a$H + $p_s$H

= 0.5 x 720 x 18 + 120 x 18

= 8640 lb

Overturning moment , $M_o$ = $\frac{1}{2}$ x $p_a$H x $\frac{H}{3}$ +$p_s$H x $\frac{H}{2}$

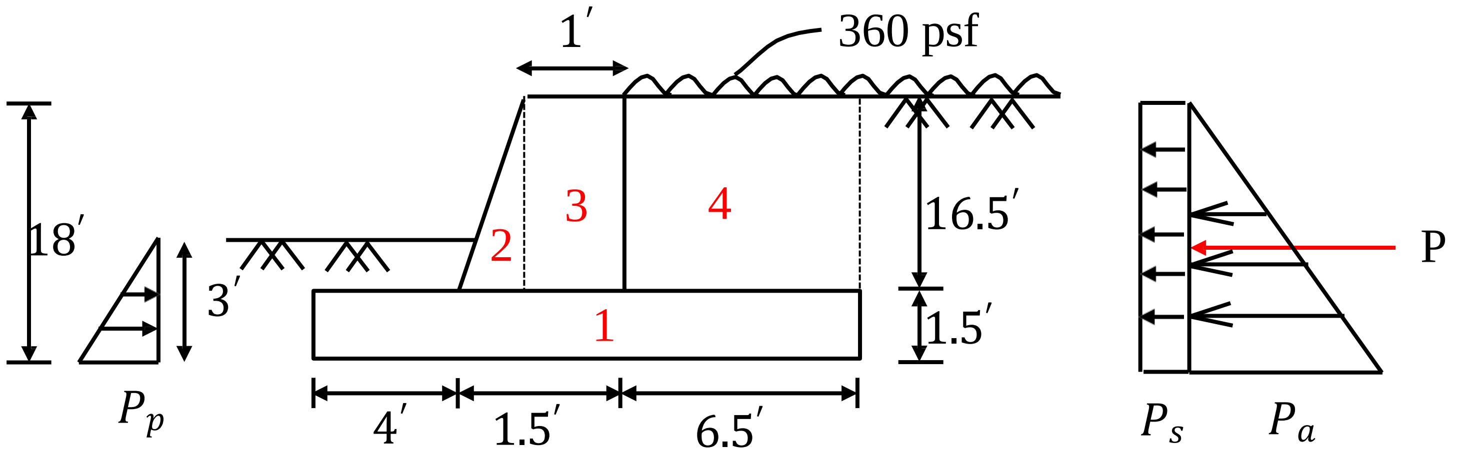
= 6480 x $\frac{18}{3}$ + 2160 x $\frac{18}{2}$

= 58320 lb-ft

Total passive earth pressure , $P_p$ = $\frac{1}{2}$ x $p_p h_s$

= 0.5 x 1080 x 3

= 1620 lb




| Section | W<br>(lb) | Moment arm<br>(ft) | Resisting moment, $M_R$<br>(lb-ft) |
|---|---|---|---|
| 1 | 12 x 1.5 x 150 = 2700 | 12/2 = 6 | 16200 |
| 2 | 0.5 x 0.5 x 16.5 x 150 = 618.75 | 4.5 - 0.5/3 = 4.33 | 2681.25 |
| 3 | 16.5 x 1 x 150 = 2475 | 4.5 + 1/2 = 5 | 12375 |
| 4 | 6.5 x 16.5 x 120 = 12870 | 12 - 6.5/2 = 8.75 | 112612.5 |
| Total | 18663.75 | | 143868.75 |

## Check for Sliding Failure:

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

$$= \frac{w\ x\ f + \cancel{P_p} \rightarrow 0}{P}$$

$$= \frac{18663.75\ x\ 0.5}{8640}$$

$= 1.08 < 1.5$ (Not OK)

## Check for Overturning Failure:

Factor of safety against overturning = $\frac{Overturning\ Moment}{Resisting\ Moment}$

$$= \frac{M_R}{M_0}$$

$$= \frac{143868.75}{58320}$$

$= 2.47 \approx 2.5$ (OK)

Considering passive earth pressure,

Check for Sliding Failure:

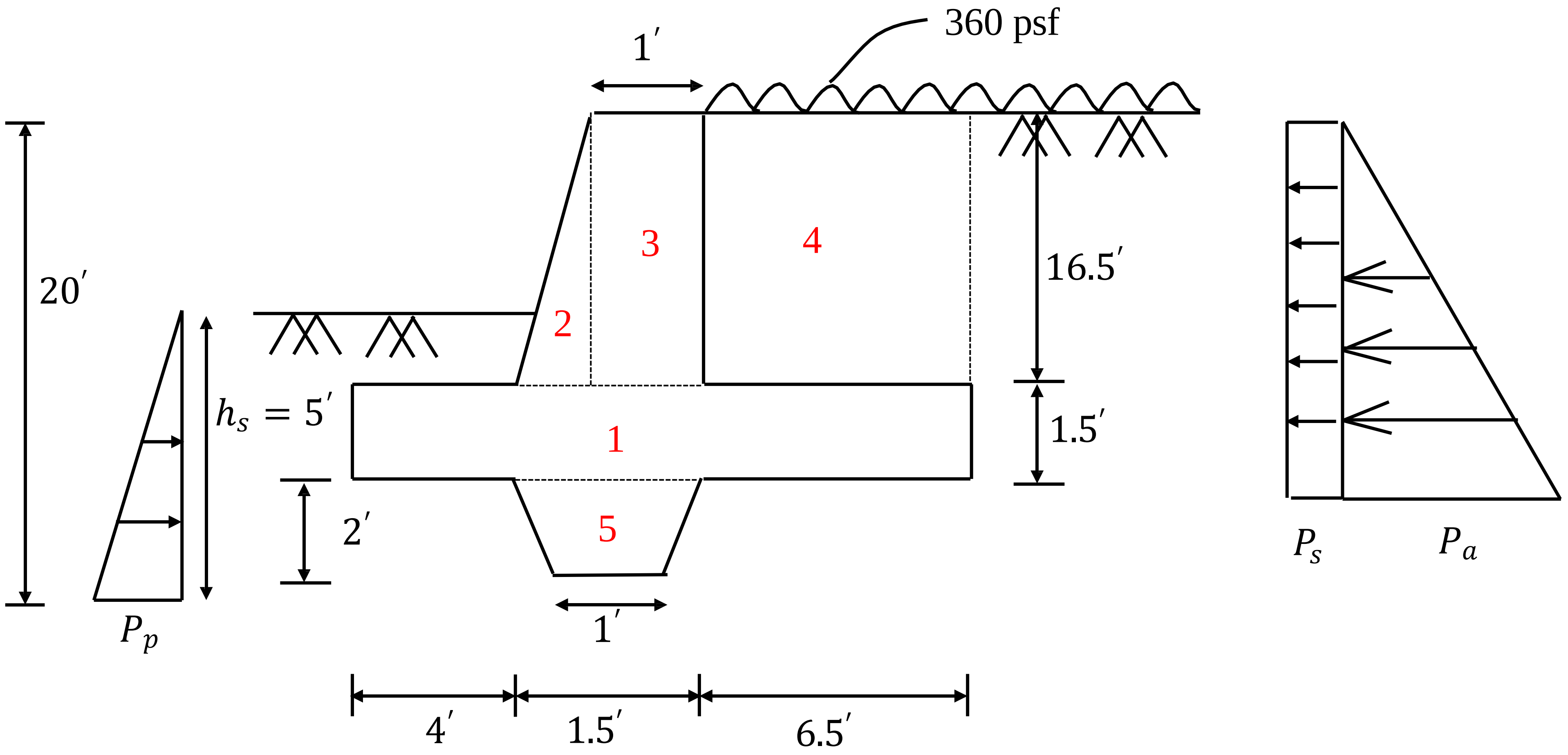
Factor of safety against sliding = $\frac{Resisting\ force}{Sliding\ force}$

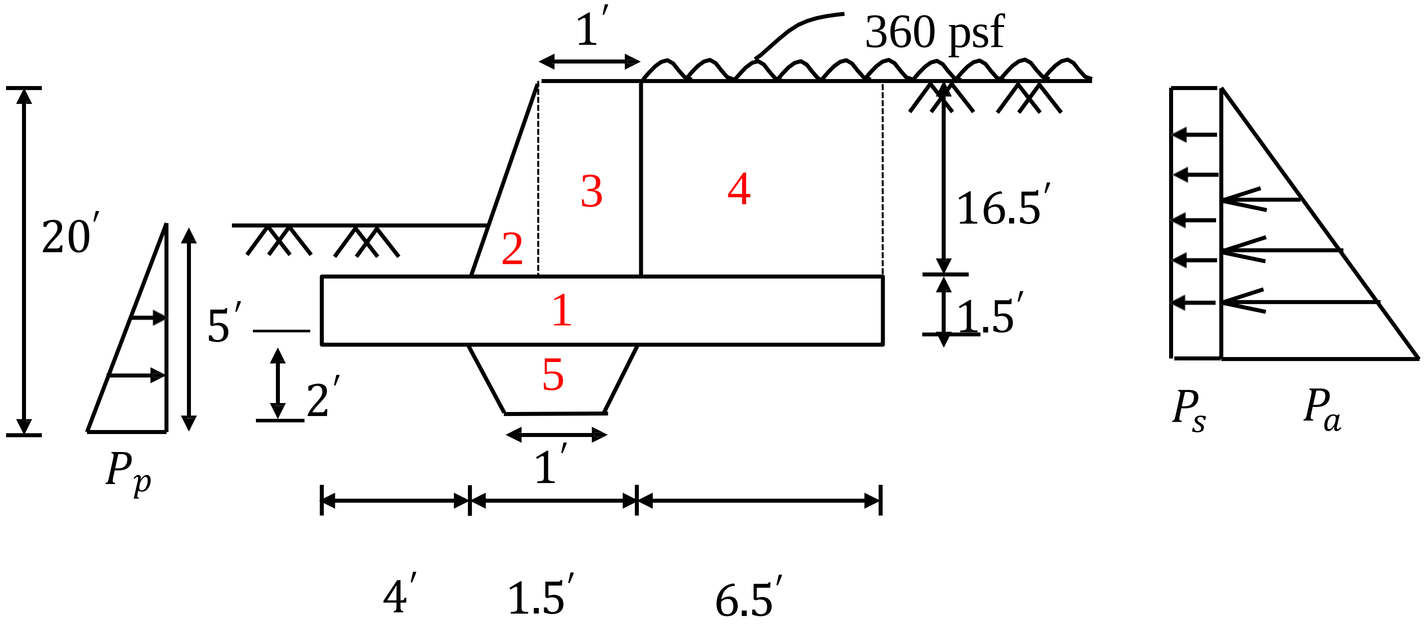
$$= \frac{w\ x\ f + P_p}{P}$$

$$= \frac{18663.75\ x\ 0.5+1620}{8640}$$

$$= 1.27 < 1.5\ (OK)$$

A key should be provided at the base.


360 psf
1′
20′
16.5′
1.5′
3
4
2
1
5
$h_s = 5'$
2′
1′
4′
1.5′
6.5′
$P_p$
$P_s$
$P_a$




| Section | W<br>(lb) | Moment arm<br>(ft) | Resisting moment, $M_R$<br>(lb-ft) |
|---|---|---|---|
| 1 | 12 x 1.5 x 150 = 2700 | 12/2 = 6 | 16200 |
| 2 | 0.5 x 0.5 x 16.5 x 150 = 618.75 | 4.5 - 0.5/3 = 4.33 | 2681.25 |
| 3 | 16.5 x 1 x 150 = 2475 | 4.5 + 1/2 = 5 | 12375 |
| 4 | 6.5 x 16.5 x 120 = 12870 | 12 - 6.5/2 = 8.75 | 112612.5 |
| 5 | 0.5 x (1 + 1.5) x 2 x 150 = 375 | 4 + 1.5/2 = 4.75 | 1781.25 |
| Total | 19038.75 | | 145650 |

## Check for Sliding Failure:

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

$$= \frac{w\ x\ f + P_p}{P}$$

$$= \frac{19308.75\ x\ 0.5+4500}{8640}$$

= 1.62 > 1.5 (OK)

$p_p = k_p \gamma h_s$
= 3 x 120 x 5
= 1800 psf

Total passive earth pressure ,
$P_p = \frac{1}{2}$ x $p_p h_s$
= 0.5 x 1800 x 5
= 4500 lb

## Check for Overturning Failure:

Factor of safety against overturning = $\frac{Overturning\ Moment}{Resisting\ Moment}$

$$= \frac{M_R}{M_0}$$

$$= \frac{145650}{58320}$$

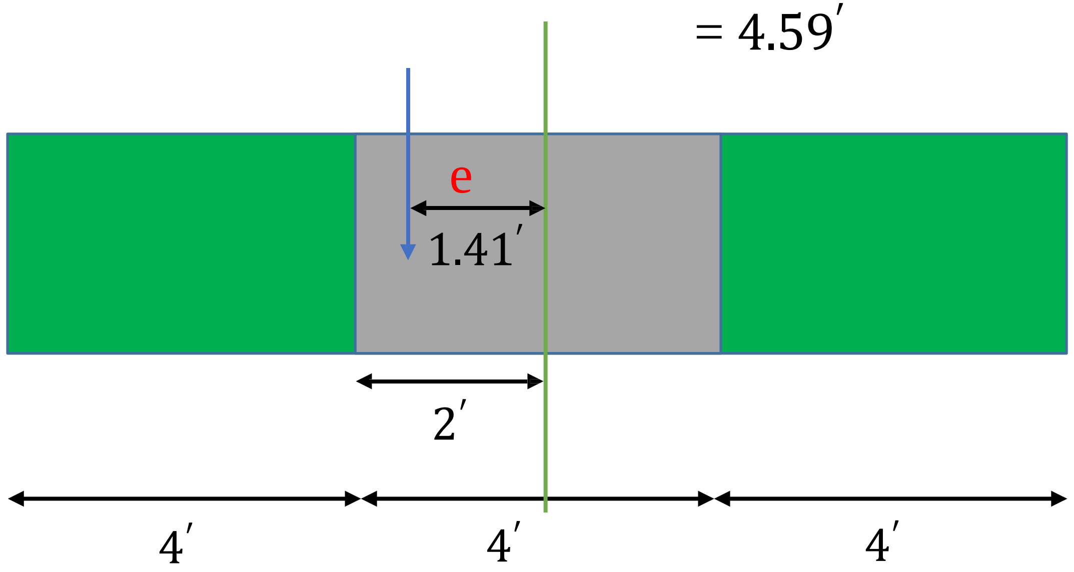
= 2.497 ≈ 2.5 (OK)

## Check for base failure:

Resultant of total vertical load from left corner, a = $\frac{M_R - M_0}{W}$

$$= \frac{145650 - 58320}{19038.75}$$

$$= 4.59'$$




e = $\frac{B}{2}$ - a = $\frac{12}{2}$ - 4.59 = 1.41 feet

M = We

= 19038.75 x 1.41 = 26844.64 lb-ft

A = B x 1 = 12 $ft^2$

I = $\frac{1\ x\ 12^3}{12}$ = 144 $ft^4$

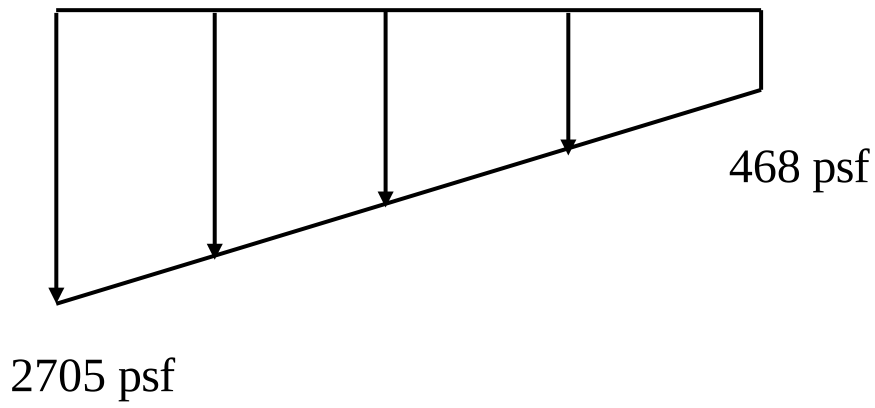
σ = $\frac{W}{A} \pm \frac{MC}{I}$

$\sigma_1 = \frac{W}{A} + \frac{MC}{I} = \frac{19038.75}{12} + \frac{26844.64 x \frac{12}{2}}{144}$

= 2705 psf (OK)

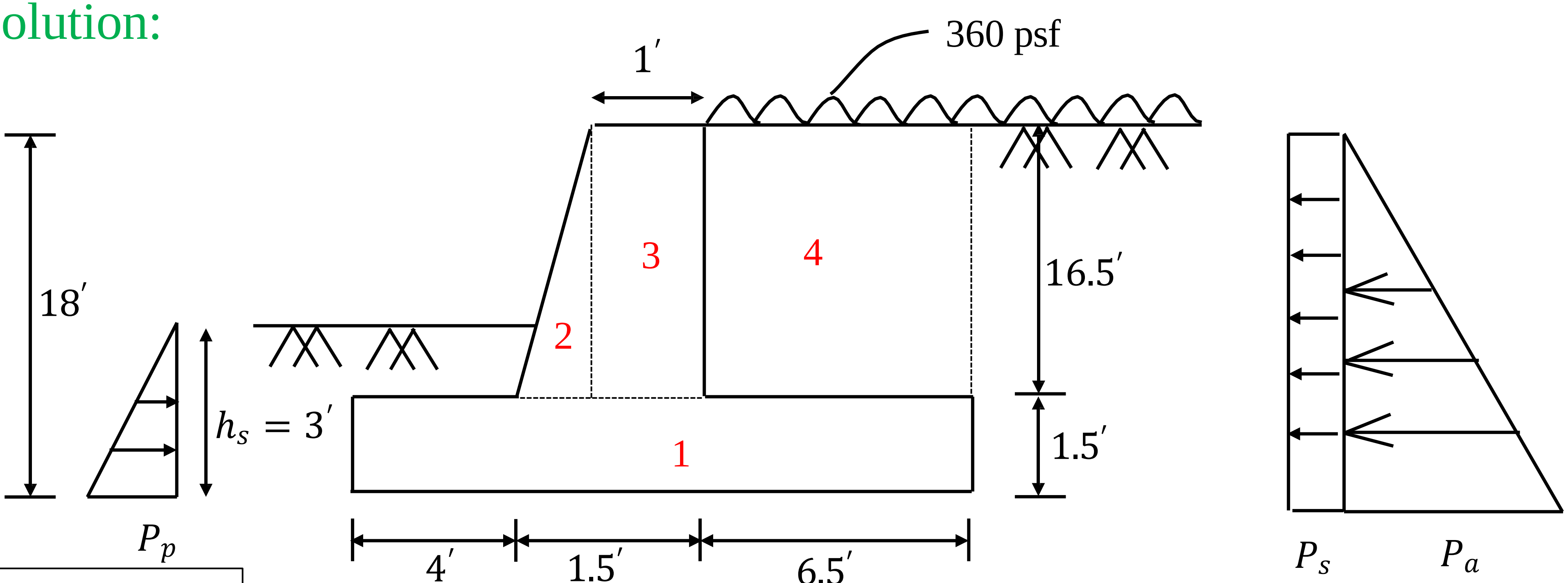
$\sigma_2 = \frac{W}{A} - \frac{MC}{I} = \frac{19038.75}{12} - \frac{26844.64 x \frac{12}{2}}{144}$

= 468 psf (OK)



Problem-2: Design of a cantilever retaining wall to retain a bank of earth which is 15 ft high and support a surcharge of 360 psf. Assume unit weight of soil, $\gamma_s =$ 120 psf. Angle of internal friction, $\varphi = 30°$. Base friction co-efficient, f = 0.5 and allowable bearing pressure, $q_{all}$ = 3 ksf. Unit weight of concrete, $\gamma_c$= 150 psf. $f_c{}'$ = 3 ksi & $f_s$ = 24 ksi.

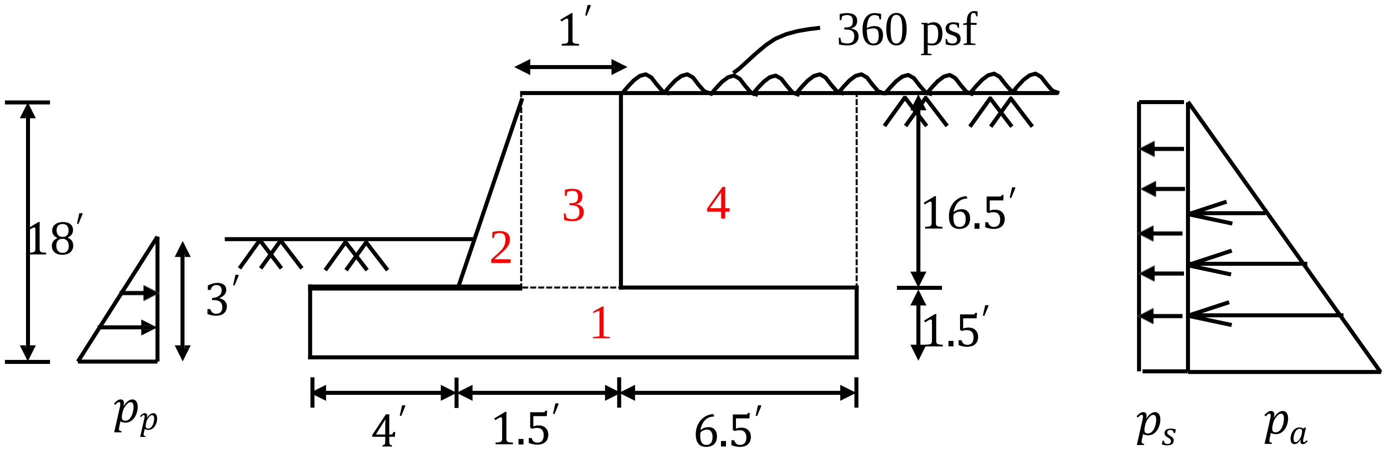
## Solution:




$h_s = \frac{360}{120} = 3$ feet

Overall height of the retaining wall = 15 + 3 = 18 feet

Base width = $\frac{2}{3}$ x Overall height = $\frac{2}{3}$ x 18 = 12 feet




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

$k_p = 1/k_a = 3$

$p_s = k_a q$

$= \frac{1}{3}$ x 360

= 120 psf

$p_a = k_a \gamma H$

$= \frac{1}{3}$ x 120 x 18

= 720 psf

$p_p = k_p \gamma h_s$

= 3 x 120 x 3

= 1080 psf

Total active earth pressure , P = $\frac{1}{2}$ x $p_a$H + $p_s$H

= 0.5 x 720 x 18 + 120 x 18

= 8640 lb

Overturning moment , $M_o = \frac{1}{2}$ x $p_a$H x $\frac{H}{3}$ +$p_s$H x $\frac{H}{2}$

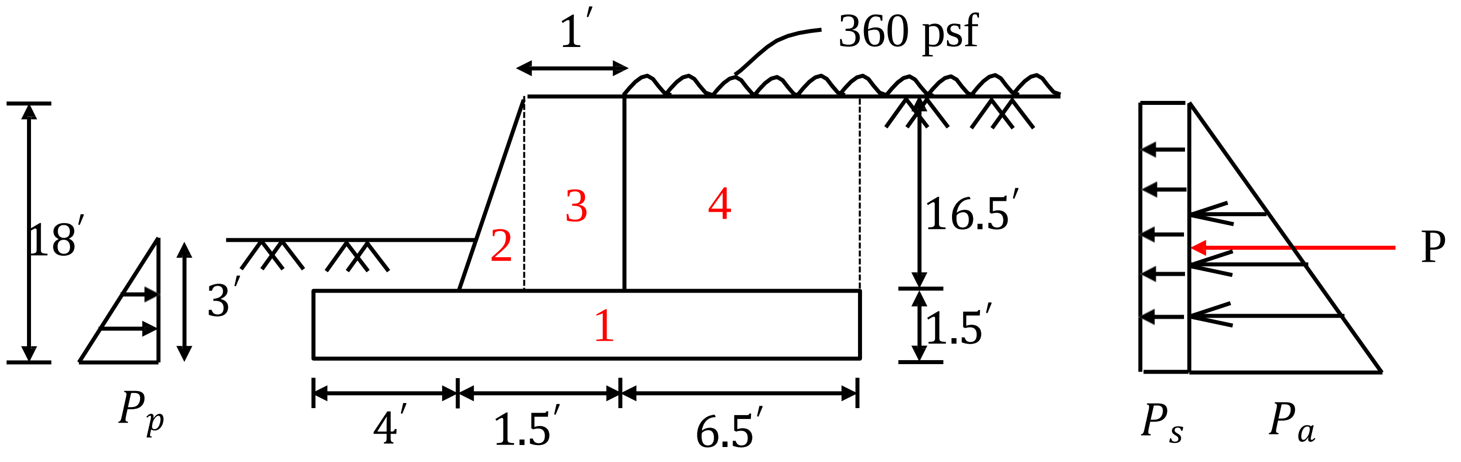
= 6480 x $\frac{18}{3}$ + 2160 x $\frac{18}{2}$

= 58320 lb-ft

Total passive earth pressure , $P_p = \frac{1}{2}$ x $p_p h_s$

= 0.5 x 1080 x 3

= 1620 lb




| Section | W<br>(lb) | Moment arm<br>(ft) | Resisting moment, $M_R$<br>(lb-ft) |
|---|---|---|---|
| 1 | 12 x 1.5 x 150 = 2700 | 12/2 = 6 | 16200 |
| 2 | 0.5 x 0.5 x 16.5 x 150 = 618.75 | 4.5 - 0.5/3 = 4.33 | 2681.25 |
| 3 | 16.5 x 1 x 150 = 2475 | 4.5 + 1/2 = 5 | 12375 |
| 4 | 6.5 x 16.5 x 120 = 12870 | 12 - 6.5/2 = 8.75 | 112612.5 |
| Total | 18663.75 | | 143868.75 |

## Check for Sliding Failure:

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

$$= \frac{w\ x\ f + \cancelto{0}{P_p}}{P}$$

$$= \frac{18663.75\ x\ 0.5}{8640}$$

$$= 1.08 < 1.5\ \text{(Not OK)}$$

## Check for Overturning Failure:

$$\text{Factor of safety against overturning} = \frac{Overturning\ Moment}{Resisting\ Moment}$$

$$= \frac{M_R}{M_0}$$

$$= \frac{143868.75}{58320}$$

$$= 2.47 \approx 2.5\ \text{(OK)}$$

Considering passive earth pressure,

Check for Sliding Failure:

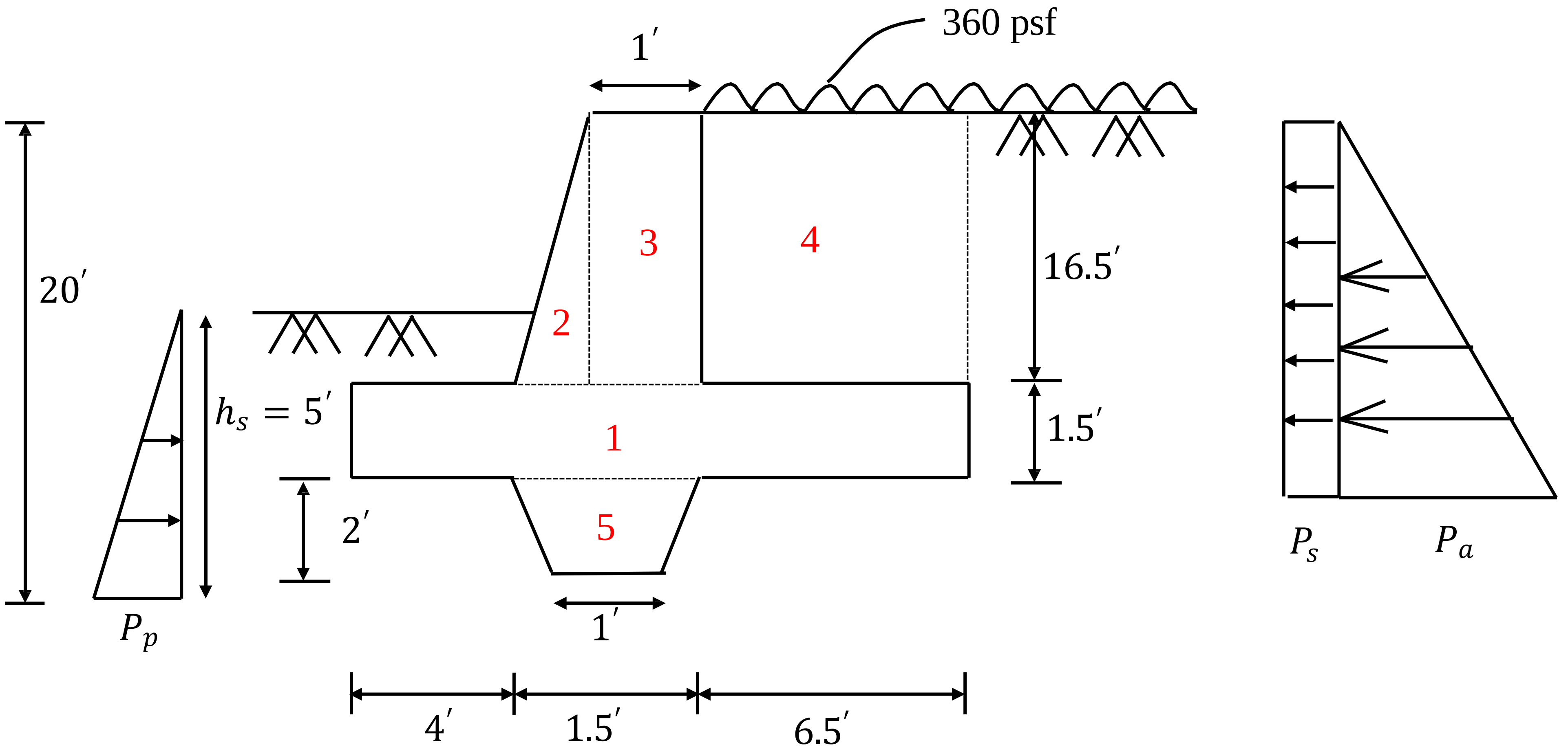
Factor of safety against sliding = $\frac{Resisting\ force}{Sliding\ force}$

$$= \frac{w\ x\ f + P_p}{P}$$

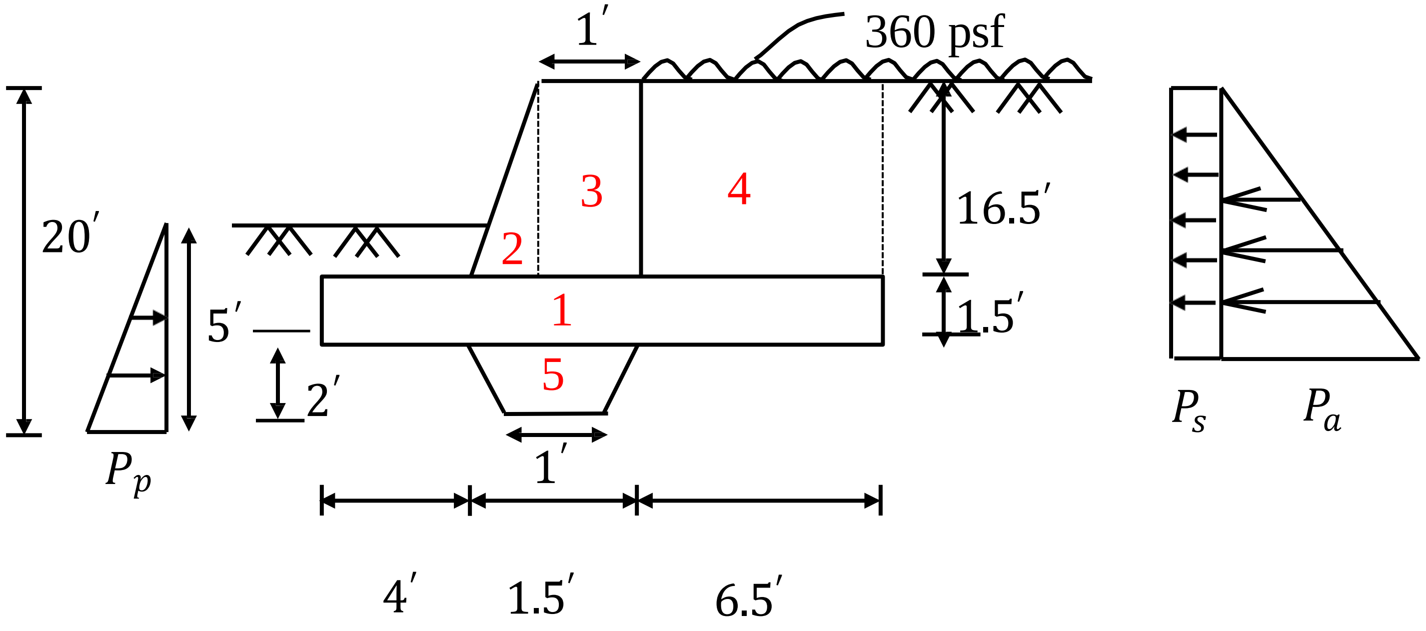
$$= \frac{18663.75\ x\ 0.5 + 1620}{8640}$$

$$= 1.27 < 1.5\ (\text{OK})$$

A key should be provided at the base.



360 psf
1′
20′
16.5′
1
2
3
4
5
$h_s = 5'$
1.5′
2′
1′
4′
1.5′
6.5′
$P_p$
$P_s$
$P_a$




| Section | W (lb) | Moment arm (ft) | Resisting moment, $M_R$ (lb-ft) |
|---|---|---|---|
| 1 | 12 x 1.5 x 150 = 2700 | 12/2 = 6 | 16200 |
| 2 | 0.5 x 0.5 x 16.5 x 150 = 618.75 | 4.5 - 0.5/3 = 4.33 | 2681.25 |
| 3 | 16.5 x 1 x 150 = 2475 | 4.5 + 1/2 = 5 | 12375 |
| 4 | 6.5 x 16.5 x 120 = 12870 | 12 - 6.5/2 = 8.75 | 112612.5 |
| 5 | 0.5 x (1 + 1.5) x 2 x 150 = 375 | 4 + 1.5/2 = 4.75 | 1781.25 |
| Total | 19038.75 | | 145650 |

## Check for Sliding Failure:

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

$$= \frac{w\ x\ f + P_p}{P}$$

$$= \frac{19308.75\ x\ 0.5+4500}{8640}$$

= 1.62 > 1.5 (OK)

$p_p = k_p \gamma h_s$

= 3 x 120 x 5

= 1800 psf

Total passive earth pressure ,

$P_p = \frac{1}{2}$ x $p_p h_s$

= 0.5 x 1800 x 5

= 4500 lb

## Check for Overturning Failure:

Factor of safety against overturning = $\frac{Overturning\ Moment}{Resisting\ Moment}$

$$= \frac{M_R}{M_0}$$

$$= \frac{145650}{58320}$$

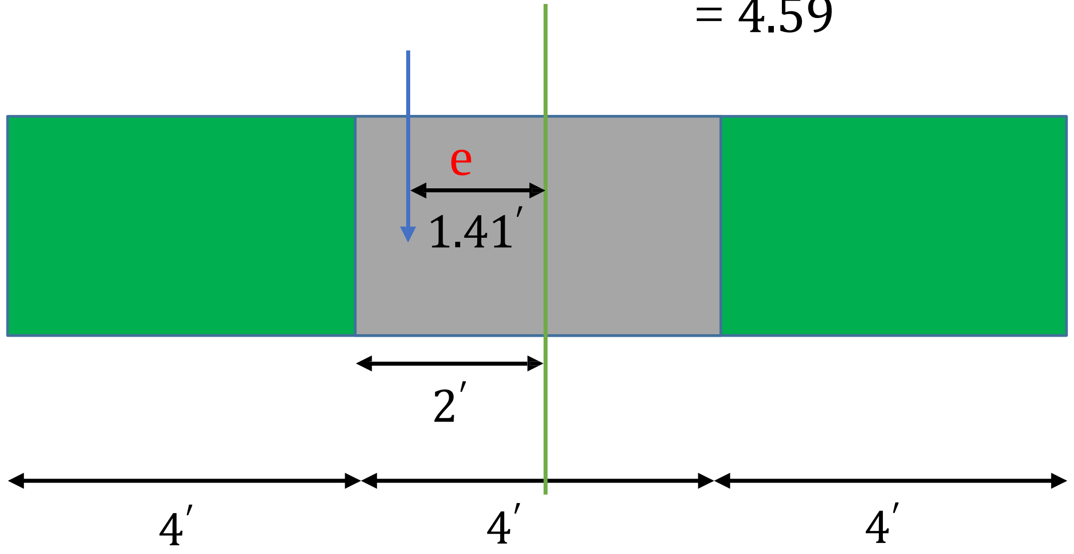
= 2.497 ≈ 2.5 (OK)

Check for base failure:

Resultant of total vertical load from left corner, a = $\frac{M_R - M_0}{W}$

$= \frac{145650 - 58320}{19038.75}$

$= 4.59'$




e = $\frac{B}{2}$ - a = $\frac{12}{2}$ - 4.59 = 1.41 feet

M = We

= 19038.75 x 1.41 = 26844.64 lb-ft

A = B x 1 = 12 $ft^2$

$$I = \frac{1\ x\ 12^3}{12} = 144\ ft^4$$

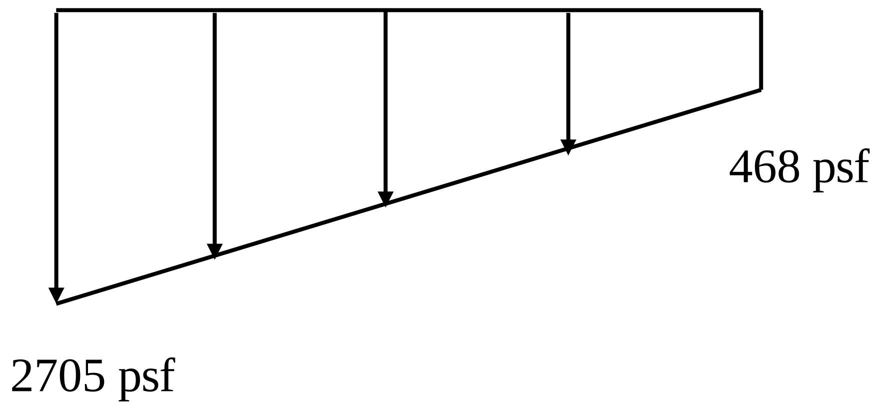
$$\sigma = \frac{W}{A} \pm \frac{MC}{I}$$

$$\sigma_1 = \frac{W}{A} + \frac{MC}{I} = \frac{19038.75}{12} + \frac{26844.64 x \frac{12}{2}}{144}$$

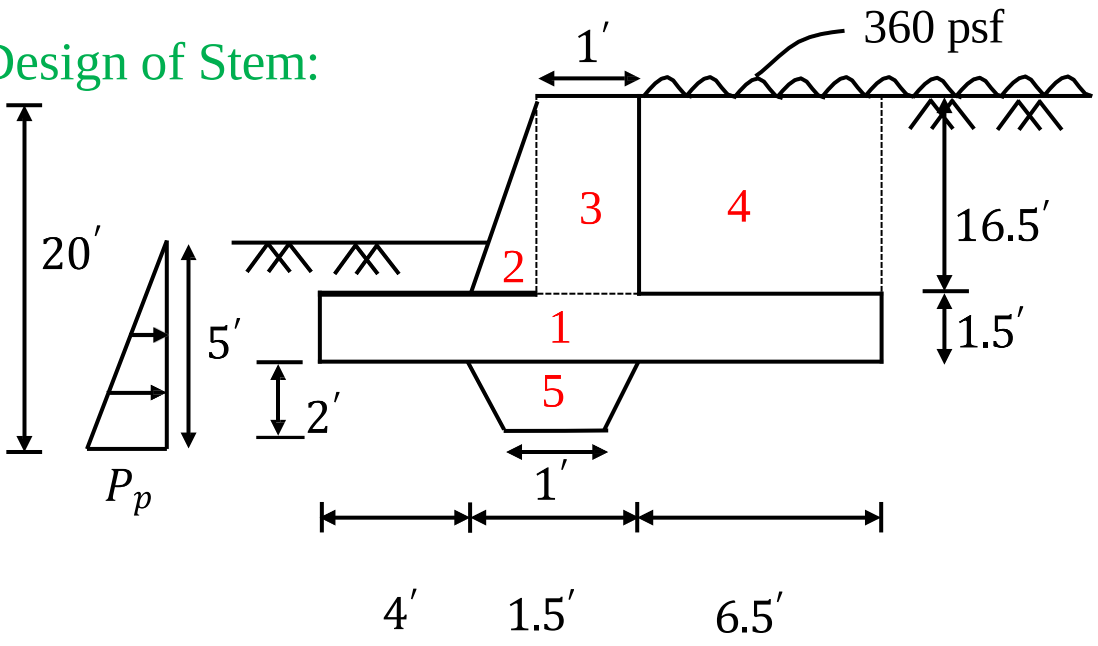
= 2705 psf (OK)

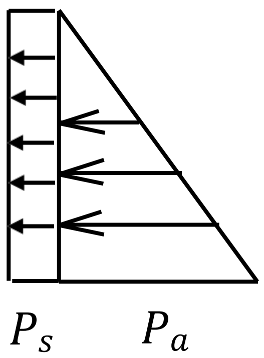
$$\sigma_2 = \frac{W}{A} - \frac{MC}{I} = \frac{19038.75}{12} - \frac{26844.64 x \frac{12}{2}}{144}$$

= 468 psf (OK)



## Design of Stem:




$$p_s = k_a q = \frac{1}{3} \text{ x } 360 = 120 \text{ psf}$$

$$p_a = k_a \gamma H = \frac{1}{3} \text{ x } 120 \text{ x } 18 = 720 \text{ psf}$$

$$\frac{720}{18} = \frac{x}{16.5}$$

$$x = \frac{720}{18} \text{ x } 16.5 = 660$$

$$n = \frac{E_s}{E_c} = \frac{29 \, x \, 10^6}{57000 \sqrt{f_c{}'}} = 9$$

$$r = \frac{f_s}{f_c} = \frac{0.4 \, f_y}{0.45 \, f_c{}'} = 17.78$$

$$k = \frac{n}{n+r} = 0.36$$

$$j = 1 - \frac{k}{3} = 0.88$$

$$R = \frac{f_c}{2} kj = \frac{0.45 \, x \, 3000}{2} \text{ x } 0.36 \text{ x } 0.88$$

$$= 199.5 \text{ psi}$$

$$V = 120 \text{ x } 16.5 + 0.5 \text{ x } 660 \text{ x } 16.5$$

$$= 7425 \text{ lb}$$

$$M = 120 \text{ x } 16.5 \text{ x } \frac{16.5}{2} + 0.5 \text{ x } 660 \text{ x } 16.5 \text{ x } \frac{16.5}{3}$$

$$= 46282.5 \text{ lb-ft}$$

$$d_{req} = \sqrt{\frac{M}{Rb}}$$

$$= \sqrt{\frac{46282.5 \text{ x } 12}{199.5 \ x \ 12}} = 15.23 > d_{eff} \text{ (OK)}$$

$$d_{eff} = \text{Thickness – c.c. - } \frac{1}{2} \text{ x bar dia}$$

$$= 18 – 2 - \frac{1}{2} \text{ x } \frac{8}{8} = 15.5 \text{ inch}$$

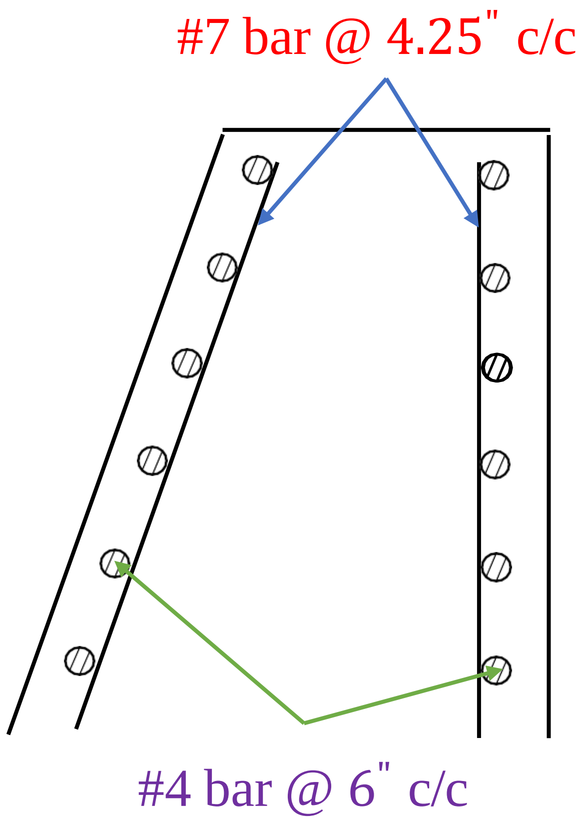
Main reinforcement, $A_s = \frac{M}{f_s jd} = \frac{46282.5 \text{ x } 12}{24000 \ x \ 0.88 \ x \ 15.5} = 1.7 \ in^2$

Using #7 bar, Spacing, S = $\frac{0.6 \ x \ 12}{1.7}$ = 4.25" c/c

Shrinkage reinforcement, $A_s$ = 0.0018bt = 0.0018 x 12 x 18

$= 0.4 \ in^2$

Using #4 bar, Spacing, S = $\frac{0.2 \ x \ 12}{0.4}$ = 6" c/c

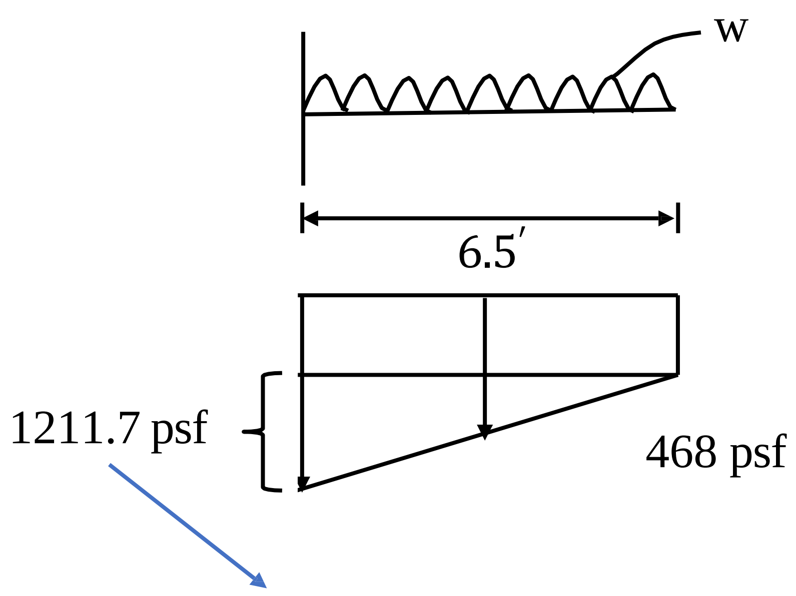


## Design of Heel:

Load calculation:

1. Self weight = 1.5 x 150 = 225 psf
2. Weight of soil = 16.5 x 120 = 1980 psf
3. Surcharge = 360 psf

Total load, w = 2565 psf




$$\frac{2237}{12} \text{ x } 6.5 = 1211.7$$

$$M = 2565 \text{ x } \frac{6.5^2}{2} - 468 \text{ x } \frac{6.5^2}{2} - 0.5 \text{ x } 1211.7 \text{ x } 6.5 \text{ x } \frac{6.5}{3}$$

$$= 35767 \text{ lb-ft}$$

$$d_{req} = \sqrt{\frac{M}{Rb}}$$

$$= \sqrt{\frac{35767 \text{x } 12}{199.5 \text{ x } 12}} = 13.39 \text{ in} < d_{eff} \text{ (OK)}$$

$$d_{eff} = \text{Thickness} - \text{c.c.} - \frac{1}{2} \text{ x bar diameter}$$

$$= 18 - 2 - \frac{1}{2} \text{ x } \frac{8}{8} = 15.5 \text{ inch}$$

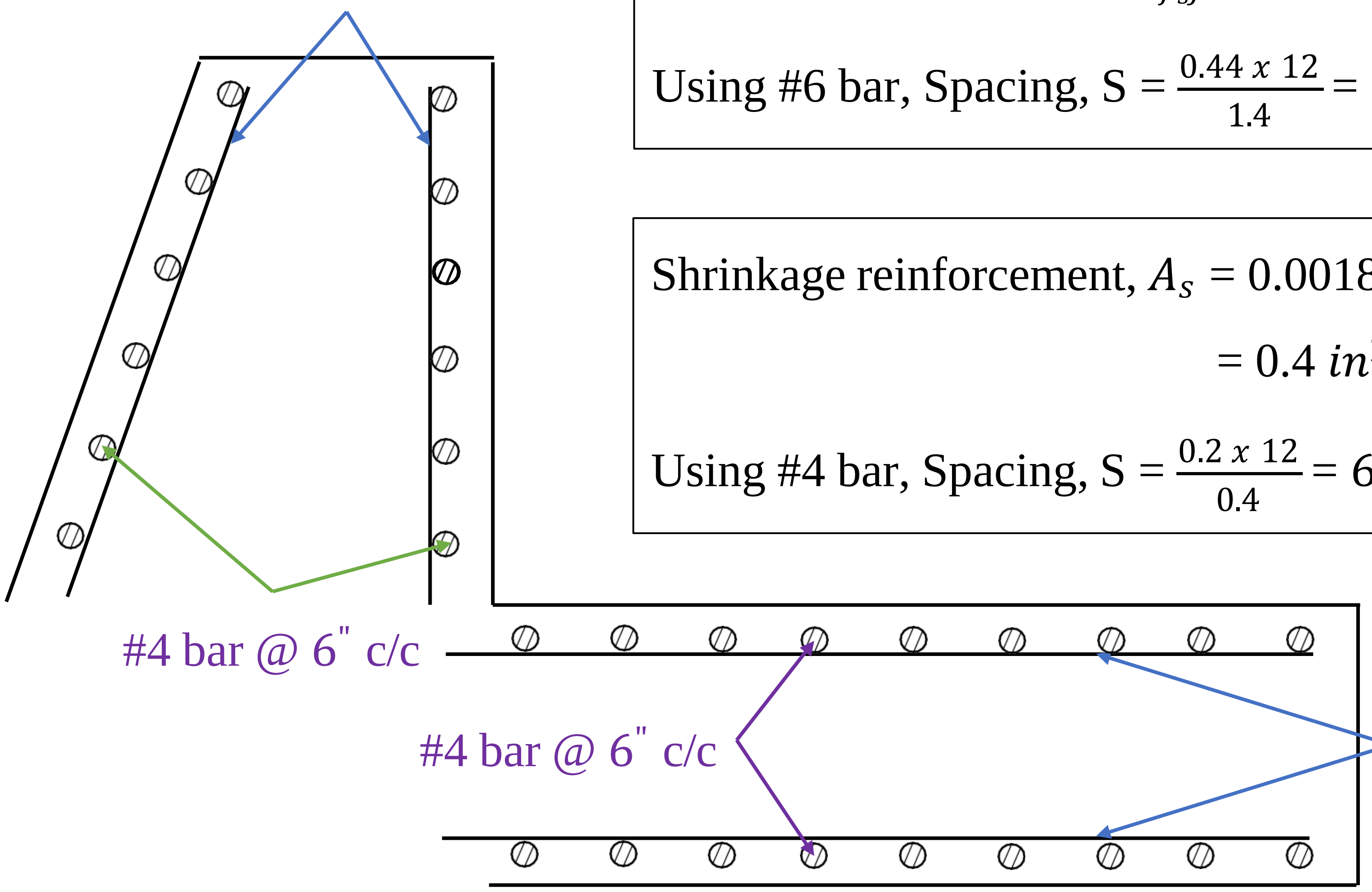
Main reinforcement, $A_s = \frac{M}{f_s j d} = \frac{35767\text{x } 12}{24000 \times 0.88 \times 15.5} = 1.4\ in^2$

Using #6 bar, Spacing, $S = \frac{0.44 \times 12}{1.4} = 3.75''$ c/c

Shrinkage reinforcement, $A_s$ = 0.0018bt = 0.0018 x 12 x 18

$= 0.4\ in^2$

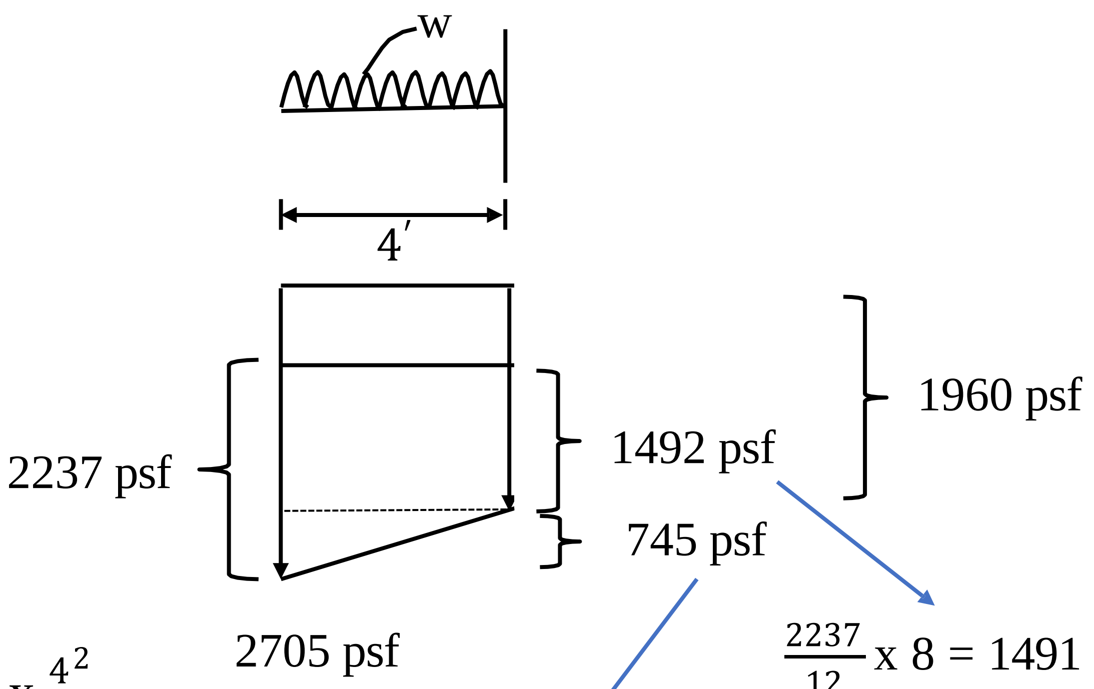
Using #4 bar, Spacing, $S = \frac{0.2 \times 12}{0.4} = 6''$ c/c



## Design of Heel:

Load calculation:

1. Self weight = 1.5 x 150 = 225 psf




$\frac{2237}{12}$ x 8 = 1491

2705 – 1960 = 745

$$M = 1959 \times \frac{4^2}{2} + 0.5 \times 745 \times 4 \times (\frac{2}{3} \times 4) - 225 \times \frac{4^2}{2}$$

$$= 17852.3 \text{ lb-ft}$$

$$d_{req} = \sqrt{\frac{M}{Rb}}$$

$$= \sqrt{\frac{17852.3 \times 12}{199.5 \times 12}} = 9.46 \text{ in} < d_{eff} \text{ (OK)}$$

$$d_{eff} = \text{Thickness} - \text{c.c.} - \frac{1}{2} \times \text{bar diameter}$$

$$= 18 - 2 - \frac{1}{2} \times \frac{8}{8} = 15.5 \text{ inch}$$

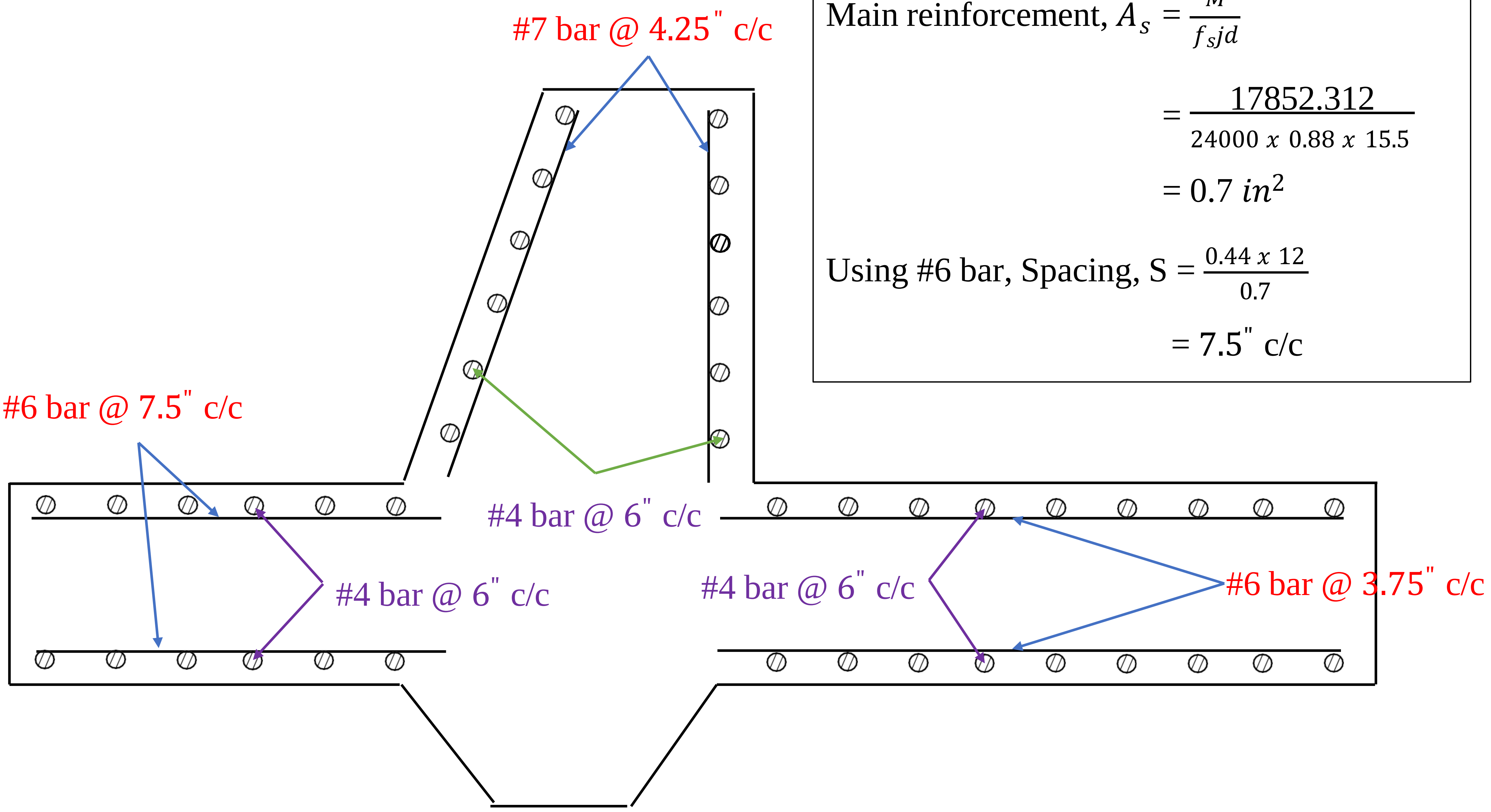
Main reinforcement, $A_s = \dfrac{M}{f_s j d}$

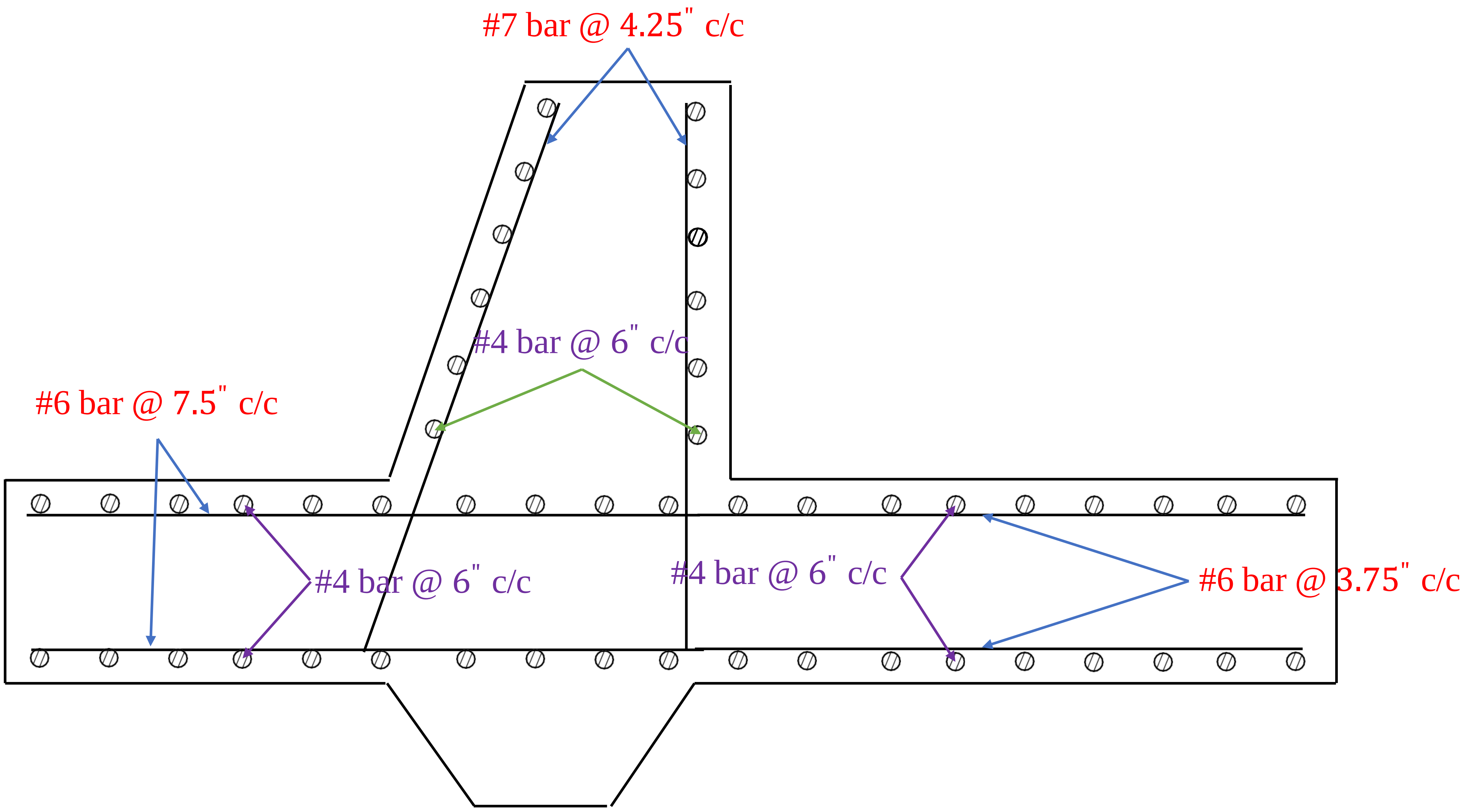
$$= \frac{17852.312}{24000 \ x \ 0.88 \ x \ 15.5}$$

$$= 0.7 \ in^2$$

Using #6 bar, Spacing, S = $\dfrac{0.44 \ x \ 12}{0.7}$

= 7.5" c/c

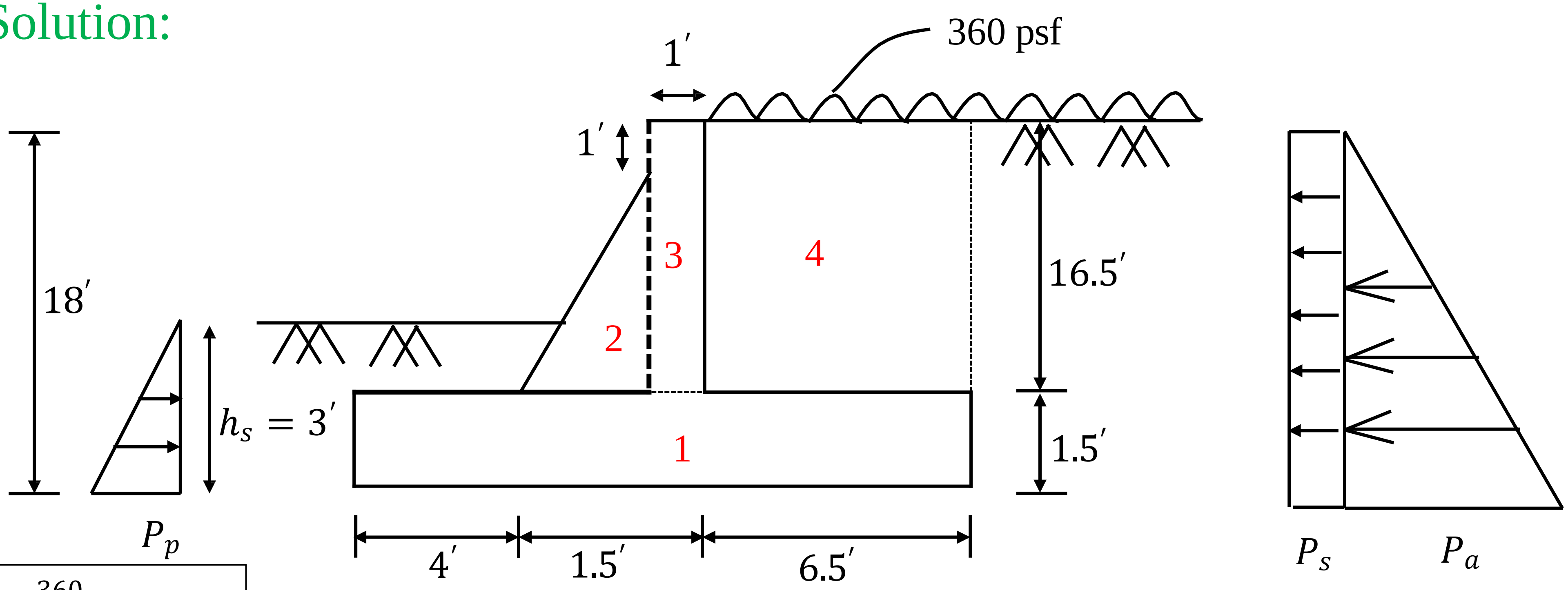




#7 bar @ 4.25" c/c
#4 bar @ 6" c/c
#6 bar @ 7.5" c/c
#4 bar @ 6" c/c
#4 bar @ 6" c/c
#6 bar @ 3.75" c/c

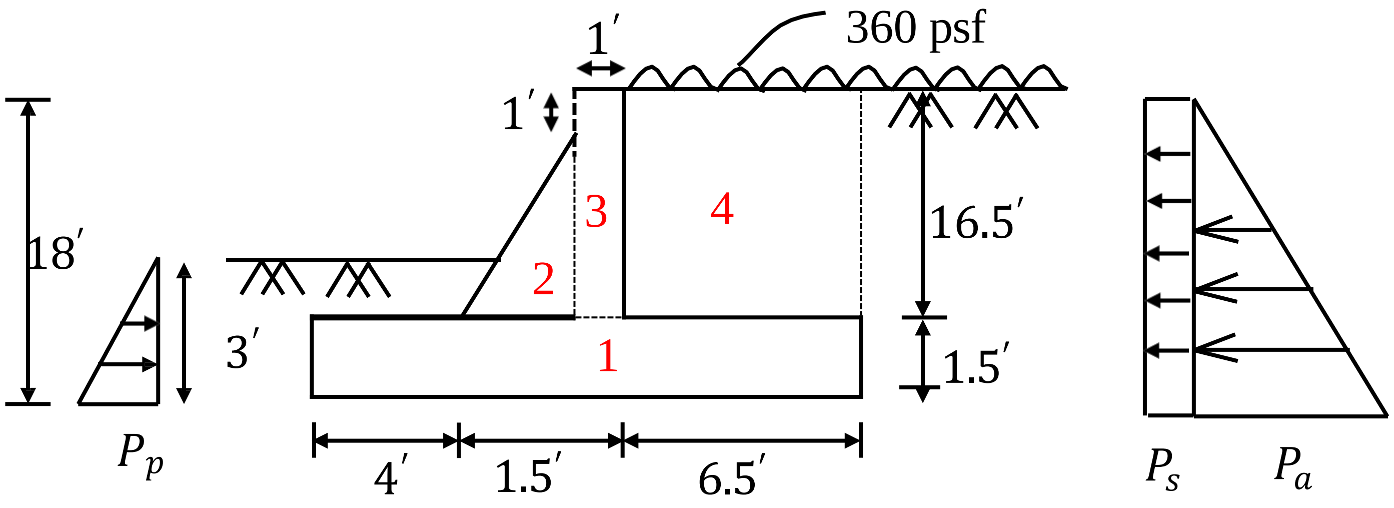
Problem-3: Check the safety factor of a cantilever retaining wall to retain a bank of earth which is 15 ft high and support a surcharge of 360 psf. Assume unit weight of soil, $\gamma_s =$ 120 psf. Angle of internal friction, φ = 30°. Base friction co-efficient, f = 0.5 and allowable bearing pressure, $q_{all}$ = 3 ksf. Unit weight of concrete, $\gamma_c$= 150 psf. $f_c{}'$ = 3 ksi & $f_s$ = 24 ksi.

## Solution:




$h_s = \frac{360}{120} = 3$ feet

Overall height of the retaining wall = 15 + 3 = 18 feet
Base width = $\frac{2}{3}$ x Overall height = $\frac{2}{3}$ x 18 = 12 feet




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

$k_p = 1/k_a = 3$

$p_s = k_a q$

$= \frac{1}{3}$ x 360

= 120 psf

$p_a = k_a \gamma H$

$= \frac{1}{3}$ x 120 x 18

= 720 psf

$p_p = k_p \gamma h_s$

= 3 x 120 x 3

= 1080 psf

Total active earth pressure , P = $\frac{1}{2}$ x $p_a$H + $p_s$H

= 0.5 x 720 x 18 + 120 x 18

= 8640 lb

Overturning moment , $M_o$ = $\frac{1}{2}$ x $p_a$H x $\frac{H}{3}$ +$p_s$H x $\frac{H}{2}$

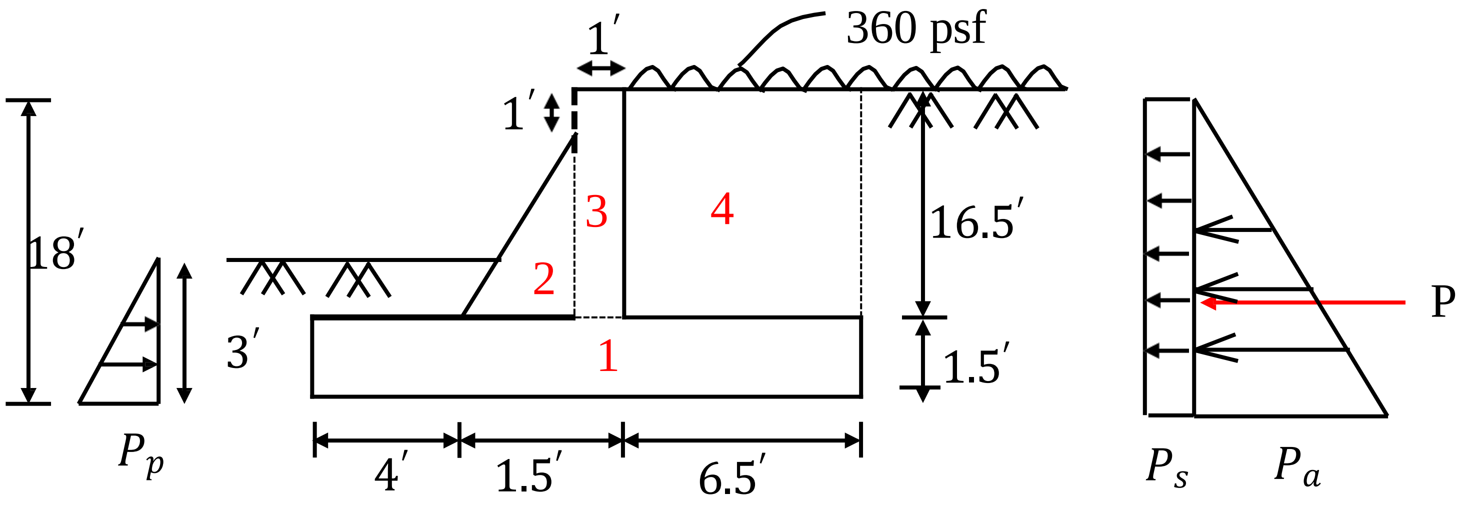
= 6480 x $\frac{18}{3}$ + 2160 x $\frac{18}{2}$

= 58320 lb-ft

Total passive earth pressure , $P_p$ = $\frac{1}{2}$ x $p_p h_s$

= 0.5 x 1080 x 3

= 1620 lb




| Section | W (lb) | Moment arm (ft) | Resisting moment, $M_R$ (lb-ft) |
|---|---|---|---|
| 1 | 12 x 1.5 x 150 = 2700 | 12/2 = 6 | 16200 |
| 2 | 0.5 x 0.5 x 15.5 x 150 = 581.25 | 4.5 - 0.5/3 = 4.33 | 2516.82 |
| 3 | 16.5 x 1 x 150 = 2475 | 4.5 + 1/2 = 5 | 12375 |
| 4 | 6.5 x 16.5 x 120 = 12870 | 12 - 6.5/2 = 8.75 | 112612.5 |
| Total | 18626.25 | | 143704.32 |

## Check for Sliding Failure:

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

$$= \frac{w\ x\ f + \cancel{P_p}^{\,0}}{P}$$

$$= \frac{18626.25 x\ 0.5}{8640}$$

$$= 1.07 < 1.5 \text{ (Not OK)}$$

## Check for Overturning Failure:

$$\text{Factor of safety against overturning} = \frac{Overturning\ Moment}{Resisting\ Moment}$$

$$= \frac{M_R}{M_0}$$

$$= \frac{143704.32}{58320}$$

$$= 2.46 \approx 2.5 \text{ (OK)}$$

Considering passive earth pressure,

Check for Sliding Failure:

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

$$= \frac{w\ x\ f + P_p}{P}$$

$$= \frac{18626.25 x\ 0.5 + 1620}{8640}$$

$$= 1.27 < 1.5\ (\text{Not OK})$$

A key should be provided at the base.

# Introduction to Flat Slab

## (Week 06)

# FLAT SLAB

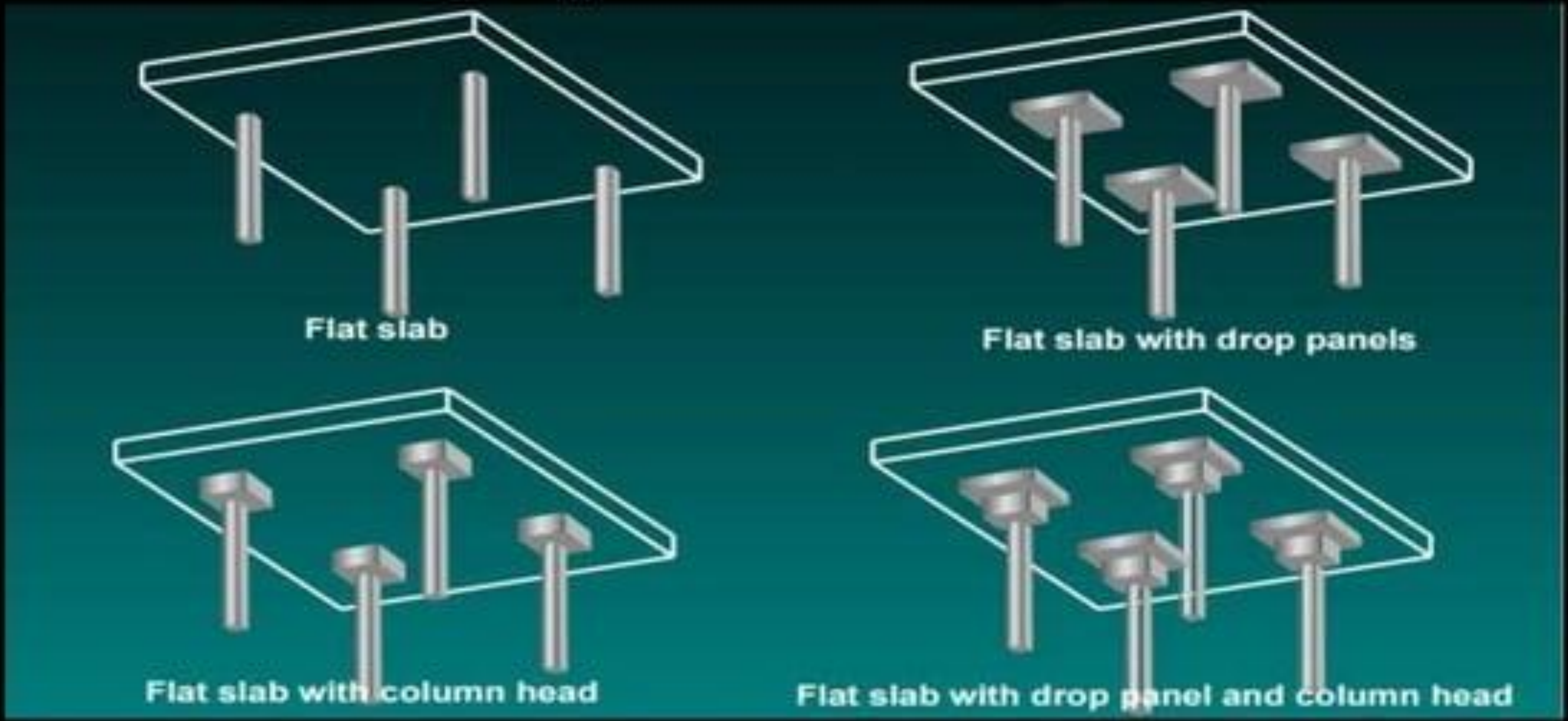
Flat slab are also known as beamless slab, is a type of slab in which the flooring slab is directly supported on columns without the agency of beam or girders.

For span from 5 to 9m thin flat slabs are the preferred solution for the construction of in- situ concrete frame building, where a square or near square grid is used.

They are provided in theatres, factories, and mills, shopping complex and other buildings.

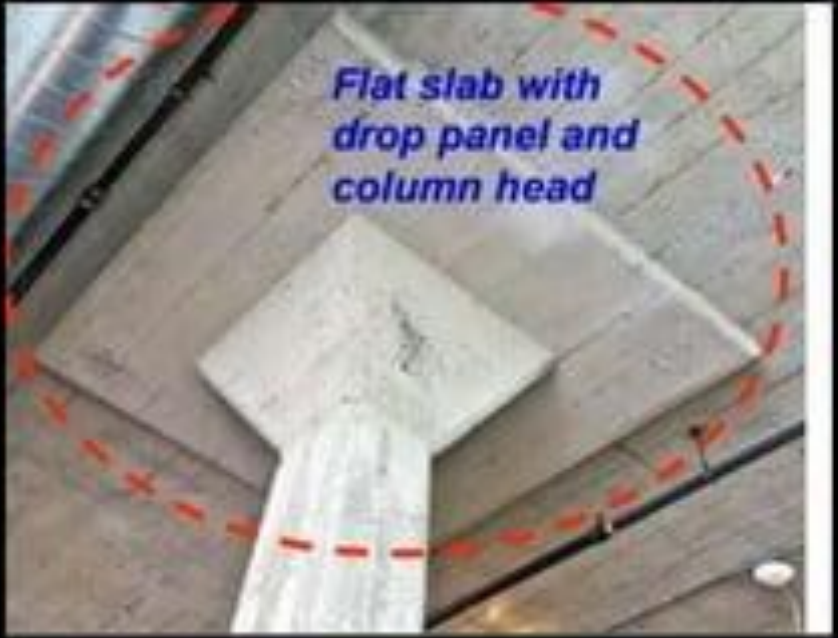
# Flat Slab

# Types of Flat slab




Flat Slab resting directly on columns


Flat Slab with drop panels


Flat slab with drop panel and column head


Flat slab with column head

# Uses of Components of Flat slab

Uses of column heads :

- increase shear strength of slab
- reduce the moment in the slab by reducing the clear or effective span

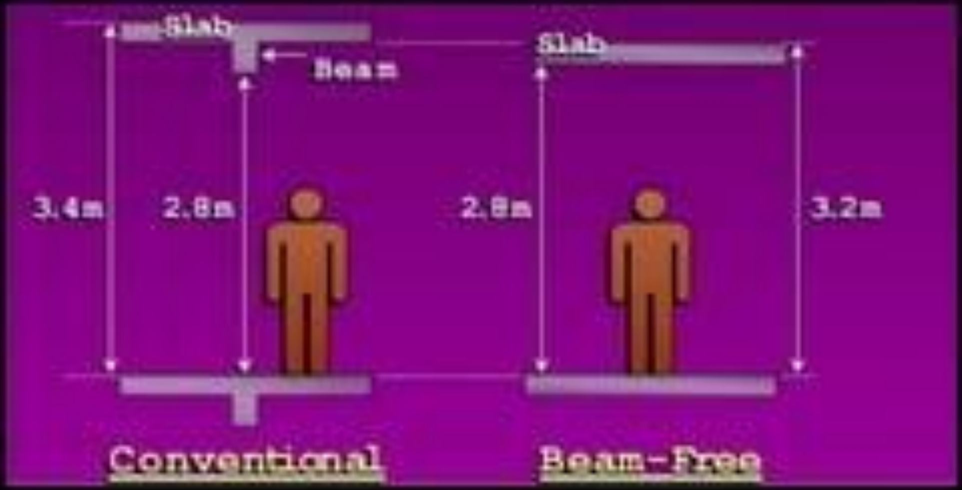
Flat slab with column head

Uses of drop panels :

- increase shear strength of slab
- increase negative moment capacity of slab
- stiffen the slab and hence reduce deflection

# Benefits of Flat Slab

- Flexibility in room layout
- Saving in building height
- Shorter construction time
- Prefabricated welded mesh




Building Height

Welded mesh

Form work

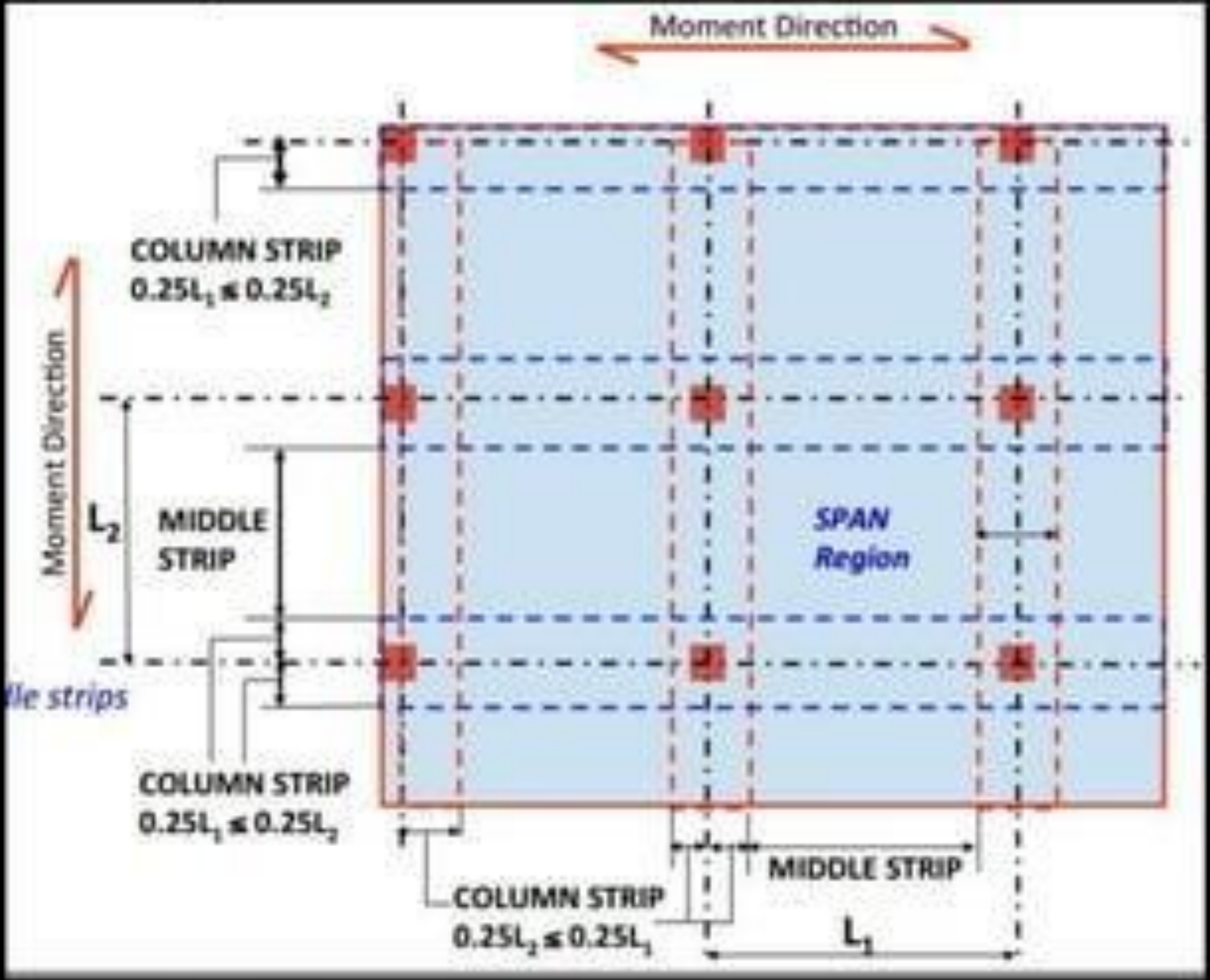
# PUNCHING SHEAR

- always a critical consideration in flat plate design around the columns
- instead of using thicker section, shear reinforcement in the form of shear heads, shear studs or stirrup cages may be embedded in the slab to enhance shear capacity at the edges of walls and columns

Shear Studs

# Column Strip and Middle strip

- Column Strip means a design strip having a width of 0.25L1 or 0.25L2,whichever is less.
- The remaining middle portion which is bound by the column strips is called middle strip.



General Design Considerations (CL 31.2)

1. Thickness of Flat Slab (31.2.1 )

- The thickness of the flat slab shall be generally controlled by considerations of span to effective depth ratios given in 23.2.

  = 40,if mild steel is used

  = 32, if HYSD bars are used

- For slabs with drops the span to effective depth ratios obtained in accordance with provisions in 23.2 shall be multiplied by 0.9.

  = 40x0.9 = 36,if mild steel is used

  = 32x0.9 = 28.8, if HYSD bars are used

- For this purpose, the longer span shall be considered.
- The minimum thickness of slab shall be 125

## 2. Drop (31.2.2 )

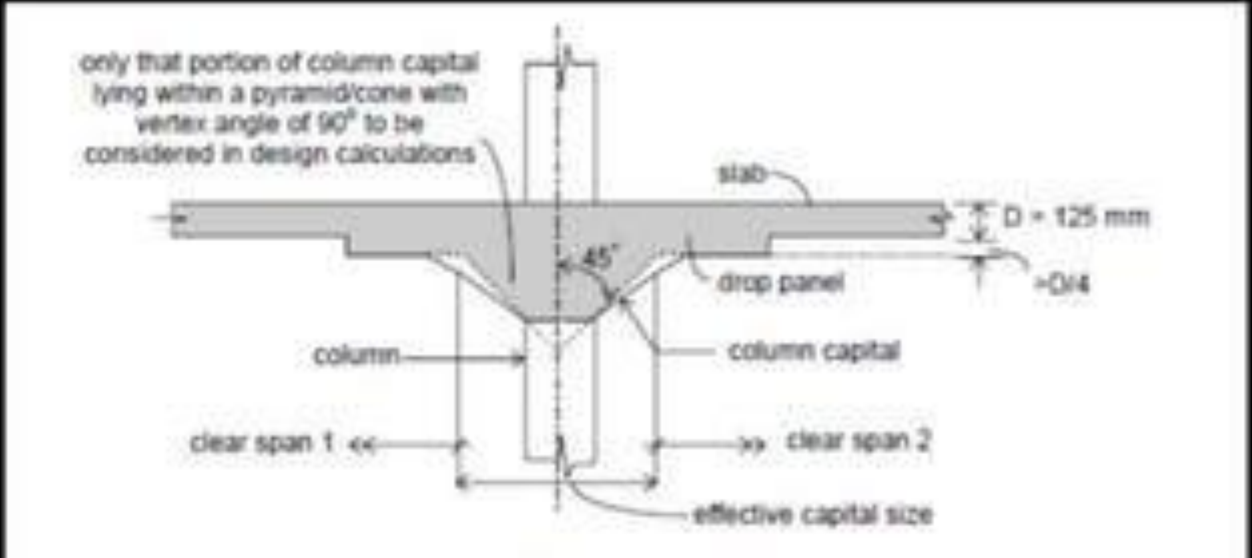
- The drops when provided shall be rectangular in plan, and have a length in each direction not less than one third of the panel length in that direction.
- For exterior panels, the width of drops at right angles to the non continuous edge and measured from the centre line of the columns shall be equal to one-half the width of drop for interior panels.
- Minimum thickness of Drop

  > ¼ of Slab thickness or

  > 100 mm

### 3. Column Heads (31.2.3)

- Where column heads are provided, that portion of a column head which lies within the largest right circular cone or pyramid that has a vertex angle of 90°and can be included entirely within the outlines of the column and the column head, shall be considered for design purposes.




Drop panel and column capital

Determination of Bending Moment (CL 31.3)

Methods of Analysis and Design (31.3.1.)

It shall be permissible to design the slab system by one of the following methods:

a) The direct design method as specified in 31.4
b) b) The equivalent frame method as specified in 31.5. In each case the applicable limitations given in 31.4 and 31.5 shall be met.

Direct Design Method (31.4)

A. Limitations : 31.4.1

Slab system designed by the direct design method shall fulfil the following conditions:

a) There shall be minimum of three continuous spans in each direction
b) The panels shall be rectangular, and the ratio of the longer span to the shorter span within a panel shall not be greater than 2.0
c) It shall be permissible to offset columns to a maximum of 10percent of the span in the direction of the offset not withstanding the provision in (b)
d) The successive span lengths in each direction shall not differ by more than one-third of the longer span.
e) The end spans may be shorter but not longer than the interior spans
f) The design live load shall not exceed three times the design dead load.

# Design of a Flat Slab

**(Week 07-08)**

**Example 1.2:** *Design an interior panel of a flat slab with panel size $6 \times 6$ m supported by columns of size $500 \times 500$ mm. Provide suitable drop. Take live load as 4 kN/m$^2$. Use M20 concrete and Fe 415 steel.*

**Solution :**

**Thickness :** Since Fe 415 steel is used and drop is provided, maximum span to thickness ratio permitted is 32

$$\therefore \text{ Thickness of flat slab } = \frac{6000}{32} = 187.5 \text{ mm}$$

Provide 190 mm thickness. Let the cover be 30 mm

$$\therefore \quad \text{Overall thickness} \quad D = 220 \text{ mm}$$

Let the drop be 50 mm. Hence at column head, $d$ = 240 mm and D = 270 mm

### Size of Drop

It should not be less than $\frac{1}{3} \times 6 \text{ m} = 2 \text{ m}$

Let us provide 3 m × 3 m drop so that the width of drop is equal to that of column head.

$\therefore$ Width of column strip = width of middle strip = 3000 mm.

### Loads

For the purpose of design let us take self-weight as that due to thickness at column strip

$$\begin{aligned}
\therefore \text{ Self-weight} &= 0.27 \times 1 \times 1 \times 25 = 6.75 \text{ kN/m}^2 \\
\text{Finishing load} &= 1.00 \text{ kN/m}^2 \\
\text{Live load} &= 4.00 \text{ kN/m}^2 \\
\hline
\text{Total load} &= 11.75 \text{ kN/m}^2 \\
\hline
\end{aligned}$$

$$\therefore \text{ Design (factored) load} = 1.5 \times 11.75 = 17.625 \text{ kN/m}^2$$

$$\text{Clear span} \quad L_n = 6 - 0.5 = 5.5 \text{ m}$$

$$\begin{aligned}
\therefore \text{ Design load} \quad W_0 &= W_u \times L_2 \times L_n \\
&= 17.625 \times 6 \times 5.5 \\
&= 581.625 \text{ kN}
\end{aligned}$$

**Design Total Moment**

Total moment

$$M_0 = \frac{W_0 L_n}{8} = \frac{581.625 \times 5.5}{8} = 400 \text{ kNm}$$

$\therefore$ Total negative moment $= 0.65 \times 400 = 260$ kNm

Total positive moment $= 0.35 \times 400 = 140$ kNm

The above moments are to be distributed into column strip and middle strip

| | *Column Strip* | *Middle Strip* |
|---|---|---|
| –ve moment | $0.75 \times 260 = 195$ kNm | $0.25 \times 260 = 65$ kNm |
| +ve moment | $0.6 \times 140 = 84$ kNm | $0.4 \times 140 = 56$ kNm |

Width of column strip = width of middle strip = 3000 mm

$$M_{u\,\lim} = 0.138\, f_{ck}\, b\, d^2 = 0.138 \times 20 \times 3000 \times 240^2 = 476.928 \times 10^6 \text{ Nmm}$$

$$= 476.928 \text{ kNm}$$

Thus $M_{u\,\lim} > M_u$. Hence thickness selected is sufficient.

**Check for Shear**

The critical section is at a distance

$$\frac{d}{2} = \frac{240}{2} = 120 \text{ mm from the face of column}$$

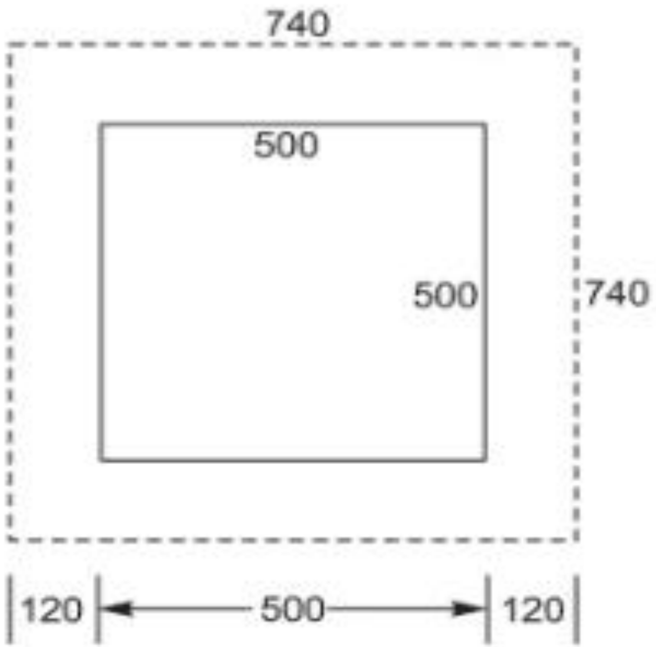
$\therefore$ It is a square of size $= 500 + 240 = 740$ mm

$$V = \text{Total load} - \text{load on } 0.740 \times 0.740 \text{ area}$$

$$= 17.625 \times 6 \times 6 - 17.625 \times 0.740 \times 0.740$$

$$= 624.849 \text{ kN}$$

$\therefore$ Nominal shear $= \tau_v = \dfrac{624.489 \times 1000}{4 \times 740 \times 240} = 0.880 \text{ N/mm}^2$




Shear strength $= k_s \, \tau_c$

where $k_s = 1 + \beta_c$ subject to maximum of 1

where $\beta_c = \dfrac{L_1}{L_2} = 1$

$\therefore$ $k_s = 1$

$$\tau_c = 0.25\sqrt{20} = 1.118 \text{ N/mm}^2$$

Design shear stress permitted

$$= 1.118 \text{ N/mm}^2 > \tau_v$$

Hence the slab is safe in shear without shear reinforcement also.

Shear strength may be checked at distance $\dfrac{d}{2}$ from drop. It is quite safe since drop size is large.

**Reinforcement**

($a$) For –ve moment in column strip

$$M_u = 195 \text{ kNm}$$

Thickness $\quad d = 240$ mm

$$\therefore \quad M_u = 0.87\, f_y\, A_{st}\, d\left[1 - \frac{A_{st}}{b \times d} \times \frac{f_y}{f_{ck}}\right]$$

$$195 \times 10^6 = 0.87 \times 415 \times A_{st} \times 240\left[1 - \frac{A_{st}}{3000 \times 240} \times \frac{415}{20}\right]$$

$$2250.38 = A_{st}\left[1 - \frac{A_{st}}{34698.8}\right]$$

$$A_{st}^{\,2} - 34698.8\, A_{st} + 2250.38 \times 34698.8 = 0$$

$$A_{st} = 2419 \text{ mm}^2 \text{ in } 3000 \text{ mm width}$$

Using 12 mm bars, spacing required is

$$s = \frac{\pi/4 \times 12^2}{2419} \times 3000 = 140.26 \text{ mm}$$

Provide 12 mm bars at 140 mm c/c

(*b*) For +ve moment in column strip

$$M_u = 84 \text{ kNm} = 84 \times 10^6 \text{ Nmm. Thickness } d = 190 \text{ mm}$$

$$84 \times 10^6 = 0.87 \times 415 \times A_{st} \times 190 \left[1 - \frac{A_{st}}{3000 \times 240} \times \frac{415}{20}\right]$$

$$1224.5 = A_{st}\left[1 - \frac{A_{st}}{27469.9}\right]$$

$$\therefore \qquad A_{st} = 1285 \text{ mm}^2$$

Using 10 mm bars

$$s = \frac{\pi/4 \times 10^2}{1285} \times 3000 = 183 \text{ mm}$$

Provide 10 mm bars at 180 mm c/c

(*c*) For –ve moment in middle strip:

$$M_u = 65 \text{ kNm}; \qquad \text{Thickness} = 190 \text{ mm}$$

$$65 \times 10^6 = 0.87 \times 415 \times A_{st} \times 190 \left[1 - \frac{A_{st}}{3000 \times 190} \times \frac{415}{20}\right]$$

$$947.5 = A_{st}\left[1 - \frac{A_{st}}{27469.9}\right]$$

$$A_{st}^2 - 27469.9\, A_{st} + 947.5 \times 27469.9 = 0$$

$$A_{st} = 983 \text{ mm}^2 \text{ in 3000 mm width}$$

Using 10 mm bars

$$s = \frac{\pi/4 \times 10^2}{983} \times 3000 = 239.7 \text{ mm}$$

Provide 10 mm bars at 230 mm c/c

(*d*) For +ve moment in middle strip

$$M_u = 56 \text{ kNm}; \qquad \text{Thickness} = 190 \text{ mm}$$

Provide 10 mm bars at 230 mm c/c in this portion also.

Since span is same in both direction, provide similar reinforcement in both directions. The details of reinforcement are shown in Fig. 1.11.

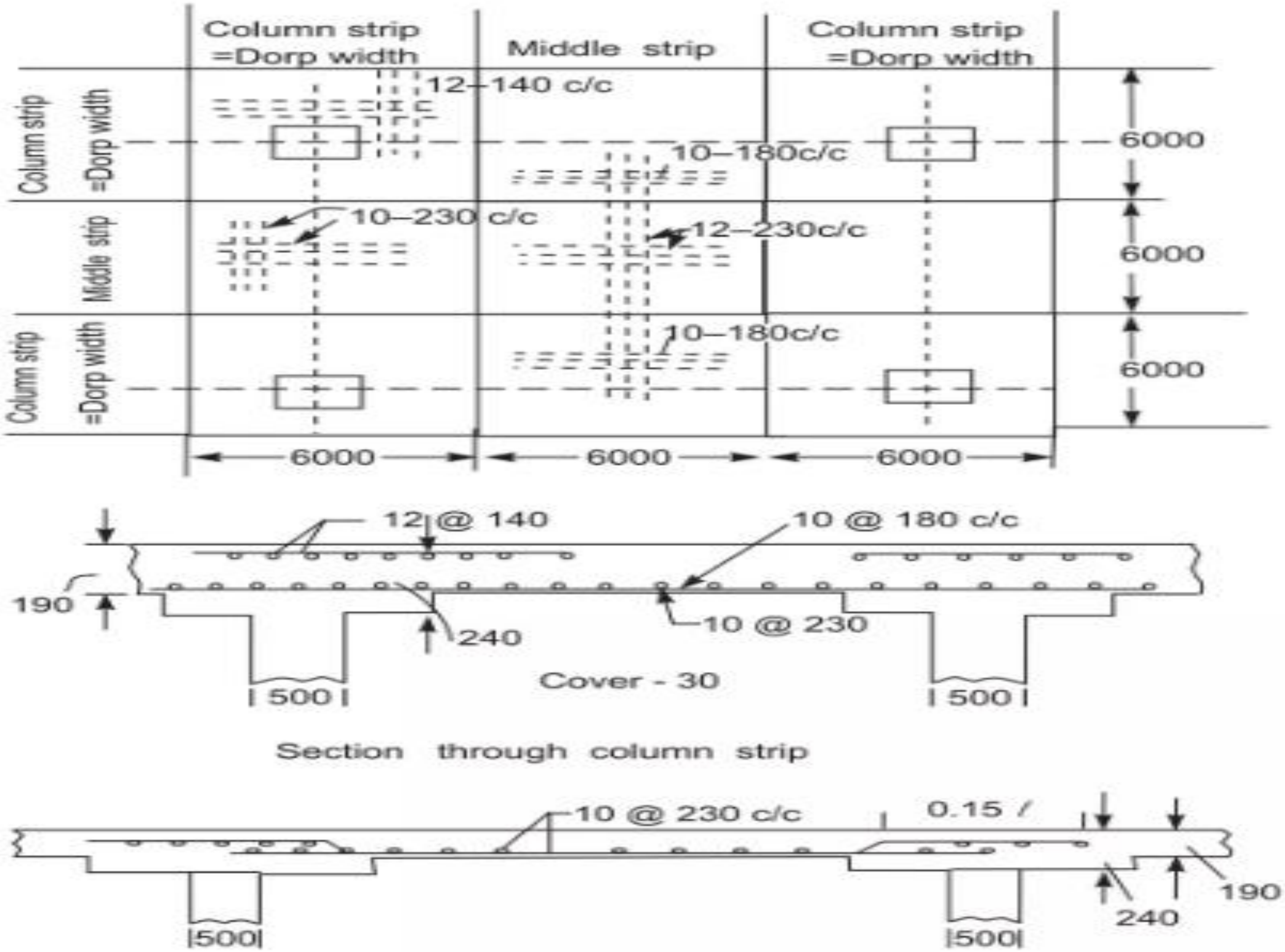


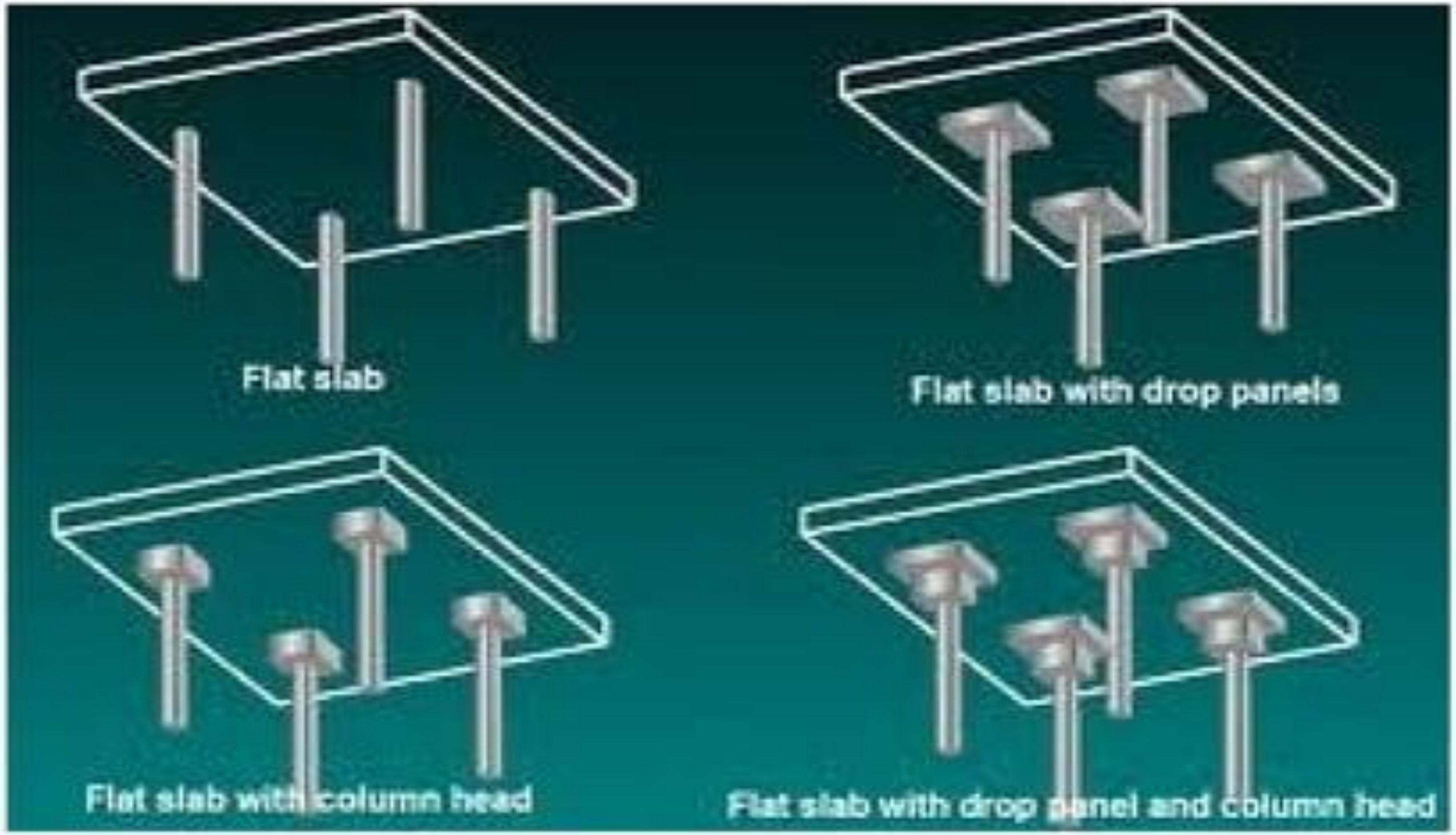

**Fig. 1.11** *Reinforcement details*

# Introduction to Flat Plate Slab

## (Week 09)

# FLAT PLATE

- *What is flat plate ?*
  - A reinforced concrete slab supported directly by concrete columns without the use of beams



Flat slab
Flat slab with drop panels
Flat slab with column head
Flat slab with drop panel and column head

# USES OF DIFFERENT TYPES OF FLAT PLATE

- Uses of column heads :

•increase shear strength of slab

•reduce the moment in the slab by reducing the clear or effective span

- Uses of drop panels :

•increase shear strength of slab

•increase negative moment capacity of slab

•stiffen the slab and hence reduce deflection

# BENEFITS

- Flexibility in room layout
- Saving in building height
- Shorter construction time
- Buildable score

## FLEXIBILITY IN ROOM LAYOUT

- allows Architect to introduce partition walls anywhere required
- allows owner to change the size of room layout
- allows choice of omitting false ceiling and finish soffit of slab with skim coating

# SAVING IN BUILDING HEIGHT

- Lower storey height will reduce building weight due to lower partitions and cladding to façade
- approx. saves 10% in vertical members
- reduce foundation load

# SHORTER CONSTRUCTION TIME

flat plate design will facilitate the use of
big table formwork to increase productivity

# BUILDABLE SCORE

- allows standardized structural members and prefabricated sections to be integrated into the design for ease of construction

- this process will make the structure more buildable, reduce the number of site workers and increase the productivity at site

- more tendency to achieve a higher Buildable score

*Design consideration*

# CRACK CONTROL

- advisable to perform crack width calculations based on spacing of reinforcement as detailed and the moment envelope obtained from structural analysis

- good detailing of reinforcement will
 –restrict the crack width to within acceptable tolerances as specified in the codes and
 –reduce future maintenance cost of the building

# PUNCHING SHEAR

- always a critical consideration in flat plate design around the columns
- instead of using thicker section, shear reinforcement in the form of shear heads, shear studs or stirrup cages may be embedded in the slab to enhance shear capacity at the edges of walls and columns

# CONSTRUCTION LOADS

- critical for fast track project where removal of forms at early strength is required
- possible to achieve 70% of specified concrete cube strength within a day or two by using high strength concrete
- alternatively use 2 sets of forms

# LATERAL STABILITY

- buildings with flat plate design is generally less rigid
- lateral stiffness depends largely on the configuration of lift core position, layout of walls and columns
- frame action is normally insufficient to resist lateral loads in high rise buildings, it needs to act in tendam with walls and lift cores to achieve the required stiffness

# LATERAL STABILITY

## MULTIPLE FUNCTION PERIMETER BEAMS

•adds lateral rigidity

•reduce slab deflection

# METHODS OF DESIGN

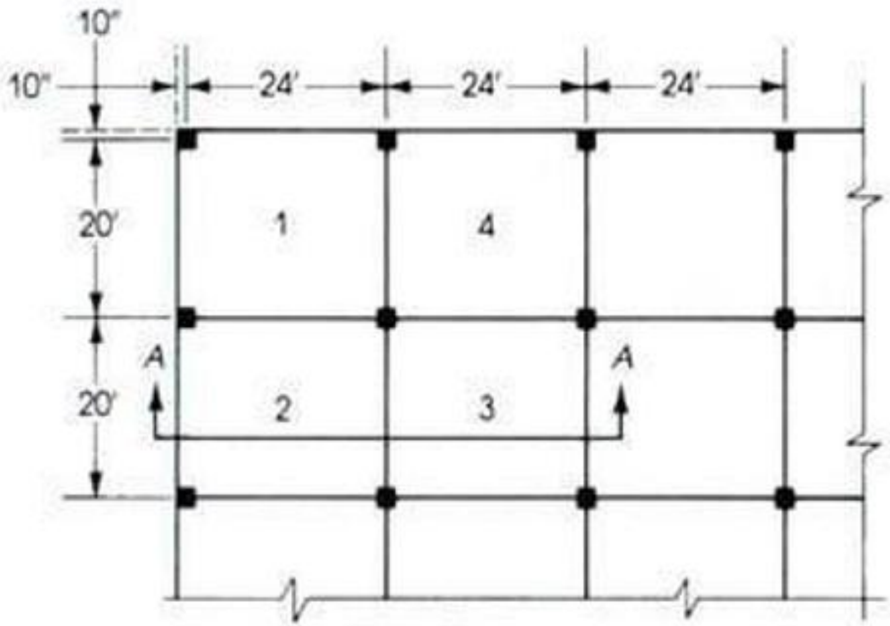
- the finite element analysis
- the simplified method
- the equivalent frame method

# Design of a Flat Plate Slab

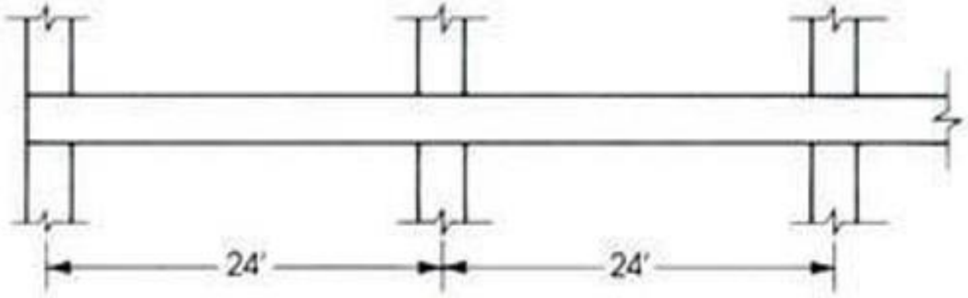
## (Week 10-11)

# Design Example 01

A flat plate floor system with panels 24 by 20 ft is supported on 20 in. square columns. Design the interior panel. Use $f'_c = 4$ ksi and $f_y = 60$ ksi and floor finish 24 psf and LL=80 psf.




All columns 20" × 20"




Section A-A

# Design Example 01

## Step 1: Calculation of Minimum Thickness 'h'

Slab thickness of an interior panel for $f_y$ = *60 ksi* and no edge beams,

$$h = \frac{l_n}{33} = \frac{24 x 12 - 20}{33} = 8.12'' \cong 9.0''$$

Effective depths, $d_s$ = 9.0-1.0 = 8.0 inch

$d_l$ = 9.0-1.5 = 7.5 inch

# Design Example 01

## Step-2: Calculation of Column and Middle Strips

Use the smaller of $l_1$ or $l_2$, so $l_2 = 20\ ft$

$$l = \frac{l_2}{4} = \frac{20\text{ ft}}{4} = 5\text{ ft}$$

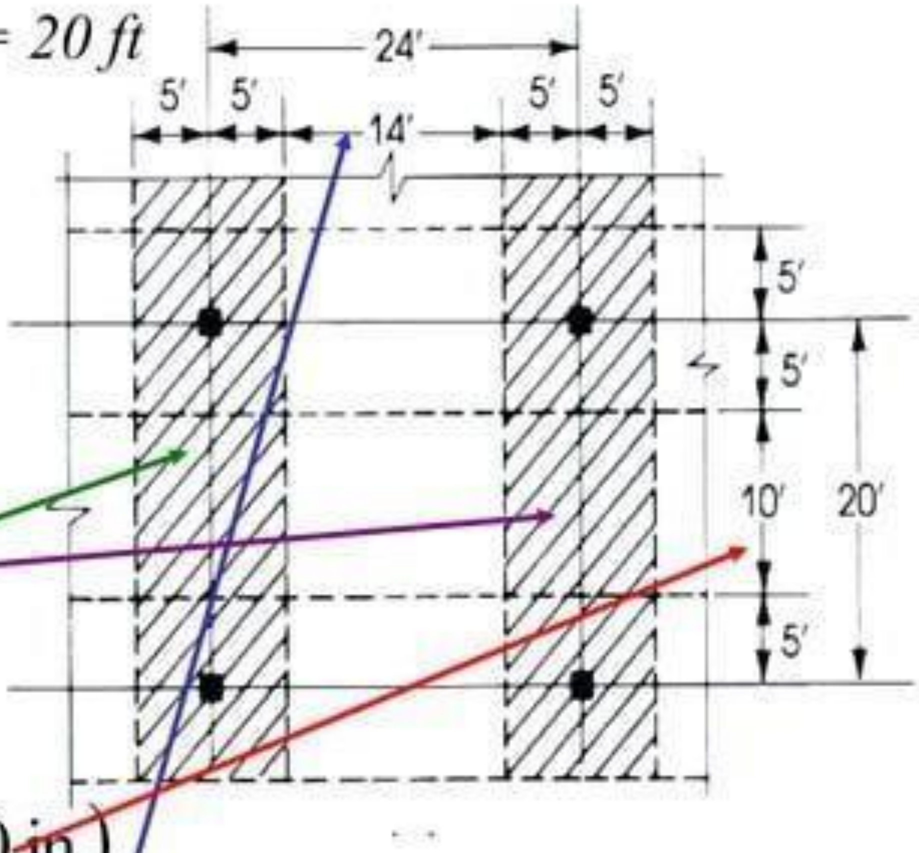
The **column strip width**,

$b = 2(5\text{ ft}) = 10\text{ ft} = 120\text{ in}$

The **middle strips** in both directions are

$$b_l = 20\text{ ft} - 2(5\text{ ft}) = 10\text{ ft} \quad (120\text{ in})$$

$$b_s = 24\text{ ft} - 2(5\text{ ft}) = 14\text{ ft} \quad (168\text{ in})$$

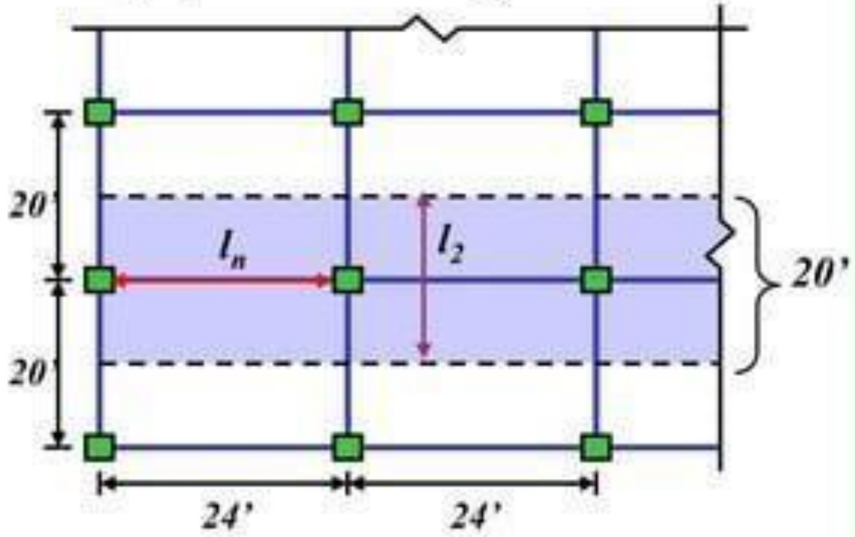


# Design Example 01

## Step-3: Calculation of Total Static Moment '$M_0$'

$$Self\ weight\ of\ slab = \frac{9}{12} x150 = 112.5\ psf$$

$$Total\ wt. = 1.2(112.5 + 24) + 1.6x80 = 292\ psf = 0.292\ ksf$$




*Moment $M_o$ for the long direction.*

$$l_n = 24\text{ ft} - 2\left(\frac{20\text{ in}}{2}\left(\frac{1\text{ ft}}{12\text{ in}}\right)\right) = 22.333\text{ ft}$$

$$M_{ol} = \frac{(wl_2)l_{n1}^2}{8} = \frac{((0.292\text{ k/ft}^2)(20\text{ ft}))(22.333\text{ ft})^2}{8} = 364.0\text{ k-ft}$$

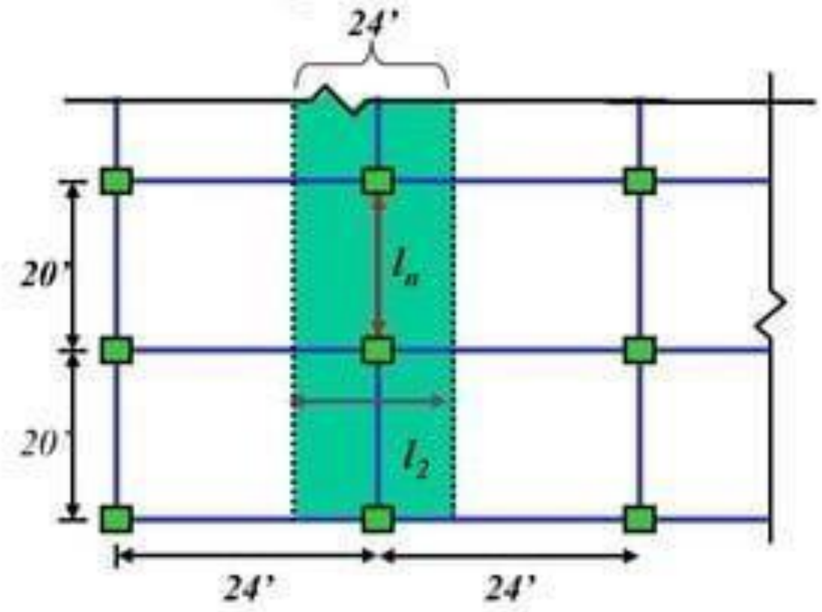
# Design Example 01

## Step-3: Calculation of Total Static Moment '$M_0$'

*Moment $M_o$ for the short direction*

$$l_n = 20\text{ ft} - 2\left(\frac{20\text{ in}}{2}\left(\frac{1\text{ ft}}{12\text{ in}}\right)\right) = 18.333\text{ ft}$$

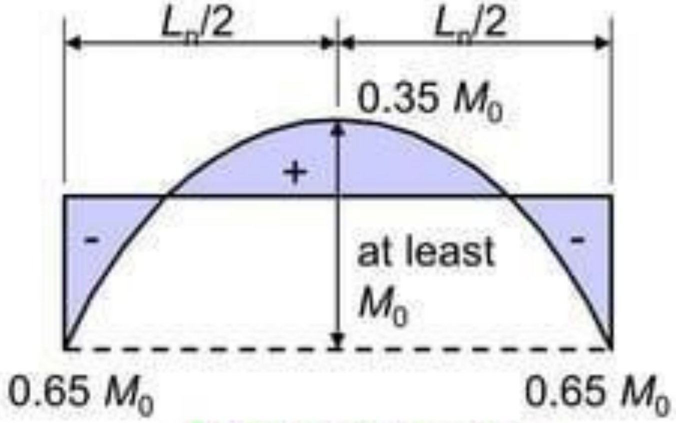
$$M_{o1} = \frac{(wl_2)l_{n1}^2}{8} = \frac{((0.292\text{ k/ft}^2)(24\text{ ft}))(18.333\text{ ft})^2}{8} = 294.3\text{ k-ft}$$



# Design Example 01

## Step-4: Longitudinal Distribution of '$M_0$'

For an Interior panel




For interior span

### Long Direction

-*ve* Moment at both supports $\longrightarrow$ $0.65(364\text{ k-ft}) = -236.6\text{ k-ft}$

+*ve* Moment at mid span $\longrightarrow$ $0.35(364\text{ k-ft}) = 127.4\text{ k-ft}$

### Short Direction

-*ve* Moment at both supports $\longrightarrow$ $0.65(294.3\text{ k-ft}) = -191.3\text{ k-ft}$

+*ve* Moment at mid span $\longrightarrow$ $0.35(294.3\text{ k-ft}) = 103\text{ k-ft}$

# Design Example 01

## Step-5: Transverse Distribution of '$M_0$'

### *Long Direction*

These distributions depend on the ratio of length $l_2/l_1$ *and* $\alpha_1\, l_2/l_1$

$$\alpha_1 = 0$$

$$\boxed{\frac{l_2}{l_1} = \frac{20}{24} = 0.8333}$$

$$\boxed{\alpha_1 \frac{l_2}{l_1} = 0}$$

# Design Example 01

## Step-5: Transverse Distribution of '$M_0$'

### *Long Direction*

Distribution % of $-ve$ and $+ve$ moments into column strips are as below:

**13.6.4.1** — Column strips shall be proportioned to resist the following portions in percent of interior negative factored moments:

| $\ell_2/\ell_1$ | 0.5 | 1.0 | 2.0 |
|---|---|---|---|
| $(\alpha_1\ell_2/\ell_1) = 0$ | 75 | 75 | 75 |
| $(\alpha_1\ell_2/\ell_1) \geq 1.0$ | 90 | 75 | 45 |

Linear interpolations shall be made between values shown.

**Factor = 0.75**

**13.6.4.4** — Column strips shall be proportioned to resist the following portions in percent of positive factored moments:

| $\ell_2/\ell_1$ | 0.5 | 1.0 | 2.0 |
|---|---|---|---|
| $(\alpha_1\ell_2/\ell_1) = 0$ | 60 | 60 | 60 |
| $(\alpha_1\ell_2/\ell_1) \geq 1.0$ | 90 | 75 | 45 |

Linear interpolations shall be made between values shown.

**Factor = 0.60**

# Design Example 01

## Step-5: Transverse Distribution of '$M_0$'

### *Long Direction*

#### ***Column Strip***

$$-ve\,moment = 0.75 x\, 236.6 = -177.45\, k-ft$$
$$+ve\,moment = 0.60 x\, 127.4 = +76.44\, k-ft$$

#### ***Middle Strip***

$$-ve\,moment = 0.25 x\, 236.6 = -59.15\, k-ft$$
$$+ve\,moment = 0.40 x\, 127.4 = +50.96\, k-ft$$

# Design Example 01

## Step-5: Transverse Distribution of '$M_0$'

### *Short Direction*

These distributions also depend on the ratio of length $l_2/l_1$ *and* $\alpha_1 l_2/l_1$

$$\alpha_1 = 0$$

$$\boxed{\frac{l_2}{l_1} = \frac{24}{20} = 1.222}$$

$$\boxed{\alpha_1 \frac{l_2}{l_1} = 0}$$

# Design Example 01

## Step-5: Transverse Distribution of '$M_0$'

### *Short Direction*

Distribution % of *–ve* and *+ve* moments into column strips are as below:

**13.6.4.1** — Column strips shall be proportioned to resist the following portions in percent of interior negative factored moments:

| $\ell_2/\ell_1$ | 0.5 | 1.0 | 2.0 |
|---|---|---|---|
| $(\alpha_1\ell_2/\ell_1) = 0$ | 75 | 75 | 75 |
| $(\alpha_1\ell_2/\ell_1) \geq 1.0$ | 90 | 75 | 45 |

Linear interpolations shall be made between values shown.

**Factor = 0.75**

**13.6.4.4** — Column strips shall be proportioned to resist the following portions in percent of positive factored moments:

| $\ell_2/\ell_1$ | 0.5 | 1.0 | 2.0 |
|---|---|---|---|
| $(\alpha_1\ell_2/\ell_1) = 0$ | 60 | 60 | 60 |
| $(\alpha_1\ell_2/\ell_1) \geq 1.0$ | 90 | 75 | 45 |

Linear interpolations shall be made between values shown.

**Factor = 0.60**

# Design Example 01

## Step-5: Transverse Distribution of '$M_0$'

### *Short Direction*

### Column Strip

$$-ve\,moment = 0.75\,x\,191.3 = -143.48\,k-ft$$
$$+ve\,moment = 0.60\,x\,103.0 = +61.80\,k-ft$$

### Middle Strip

$$-ve\,moment = 0.25\,x\,191.3 = -47.83\,k-ft$$
$$+ve\,moment = 0.40\,x\,103.0 = +41.20\,k-ft$$

# Design Example 01

## Step-6: Check for '$d$' and '$V_u$'

### '$d$' check:

$$In\ long\ direction\ max.\ moment = 177.45\,k-ft, b = 120'', d_p = 7.5''$$

$$Let, \rho = \rho_{max} = 0.0214$$

$$Now, M_u = \phi M_n$$

$$177.45x12 = 0.90x0.0214x60x120xd^2\left[1-0.59\frac{0.0214x60}{4}\right]$$

$$\therefore d = 4.35'' < 7.5''\ ok$$

$$In\ short\ direction\ max.\ moment = 143.48.6\,k-ft, b = 168'', d_p = 8.0''$$

$$\because 143.48x12 = 0.90x0.0214x60x168xd^2\left[1-0.59\frac{0.0214x60}{4}\right]$$

$$\therefore d = 3.39'' < 8.0''\ ok$$

# Design Example 01

## Step-6: Check for '$d$' and '$V_u$'

### '$V_u$' check:

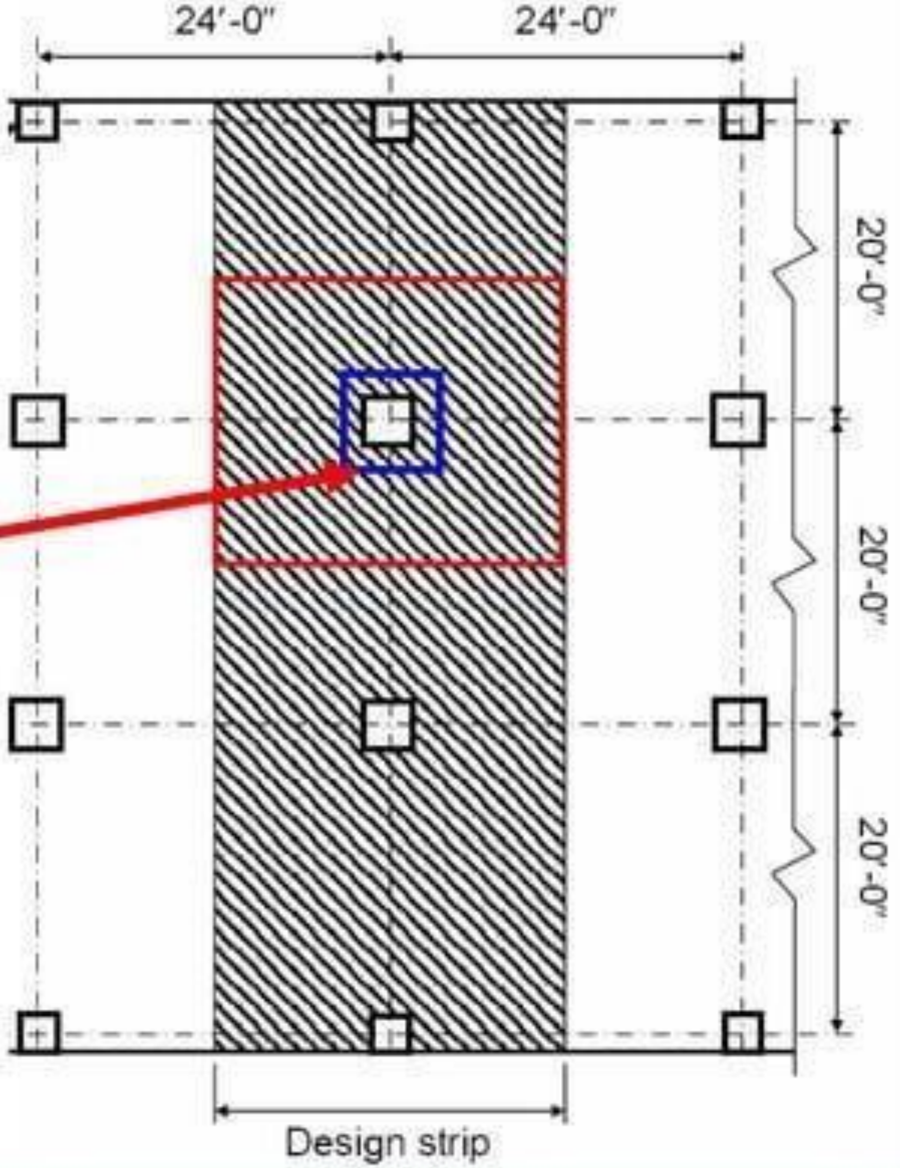
$$\text{Critical two-way shear occurs at distance of } \frac{d_1}{2} = 3.75'' \text{ from all faces of the column.}$$

$$\text{Shear perimeter, } b_0 = 2(c_1 + d) + 2(c_2 + d) = 2(20 + 3.75) + 2(20 + 3.75) = 96''$$

$$\text{Since } \beta_c = \frac{24}{20} = 1.2 < 2.0$$

shear strength of slab at critical section,

$$\phi V_c = \phi 4\sqrt{f_c'}\, b_0 d_s = 0.75 x 4 \sqrt{4000} x 96 x 8 = 145717.76\, lb = 145.72\, k$$



# Design Example 01

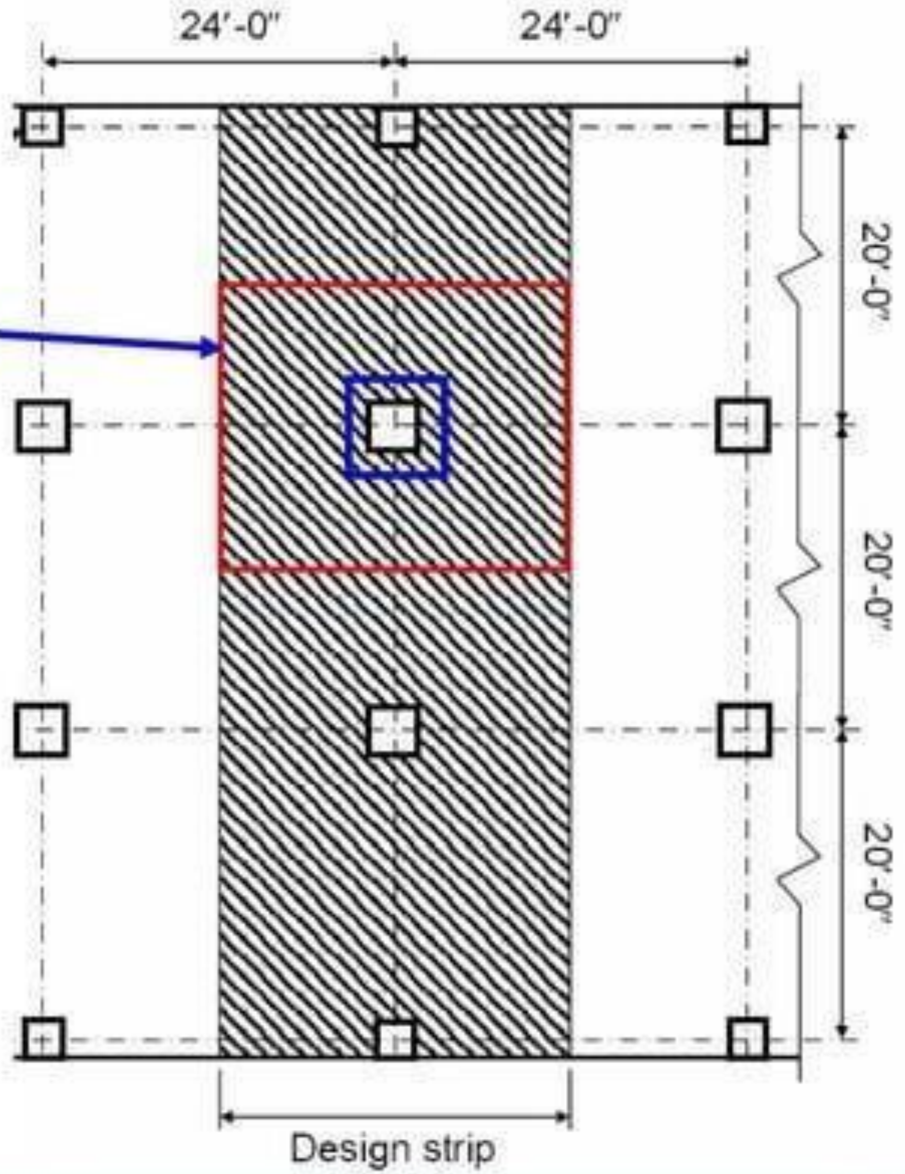
## Step-6: Check for '$d$' and '$V_u$'

### '$V_u$' check:

*Based on tributary area of the loaded floor, the factored shear is*

$$V_u = 0.292\left(20x24 - \frac{27.5x27.5}{12x12}\right) = 138.63\ k$$

*Since* $V_u < \varphi V_c$, *no shear reinforcement is reqd.*



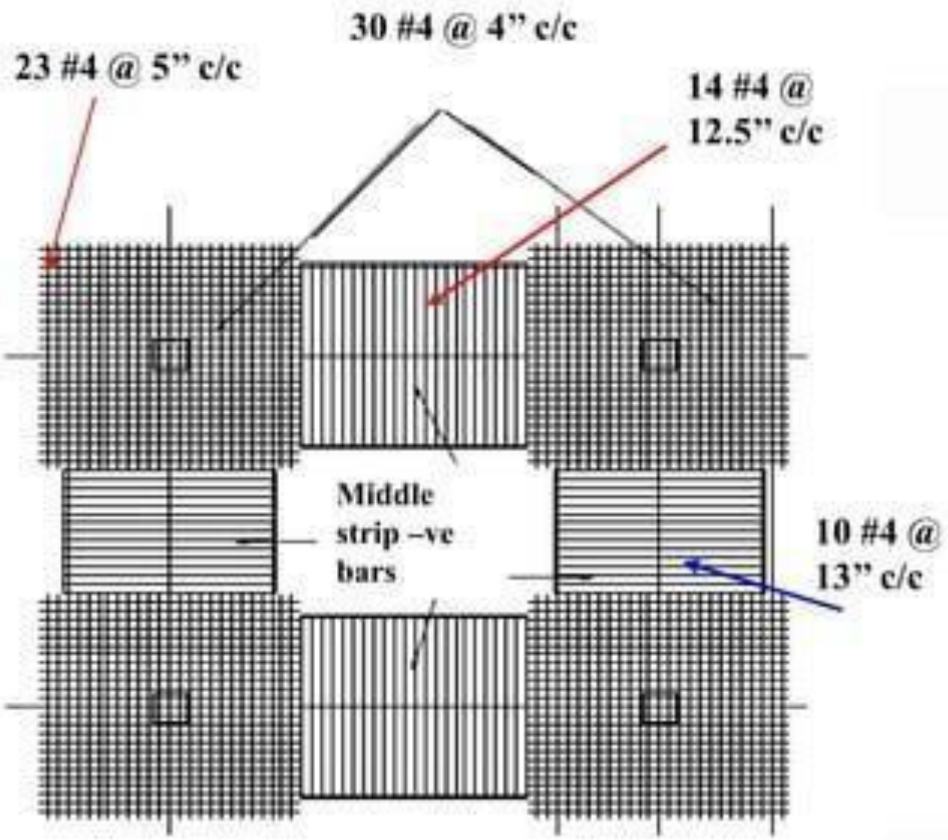
# Design Example 01

## Step-7: Calculation of '$A_s$'

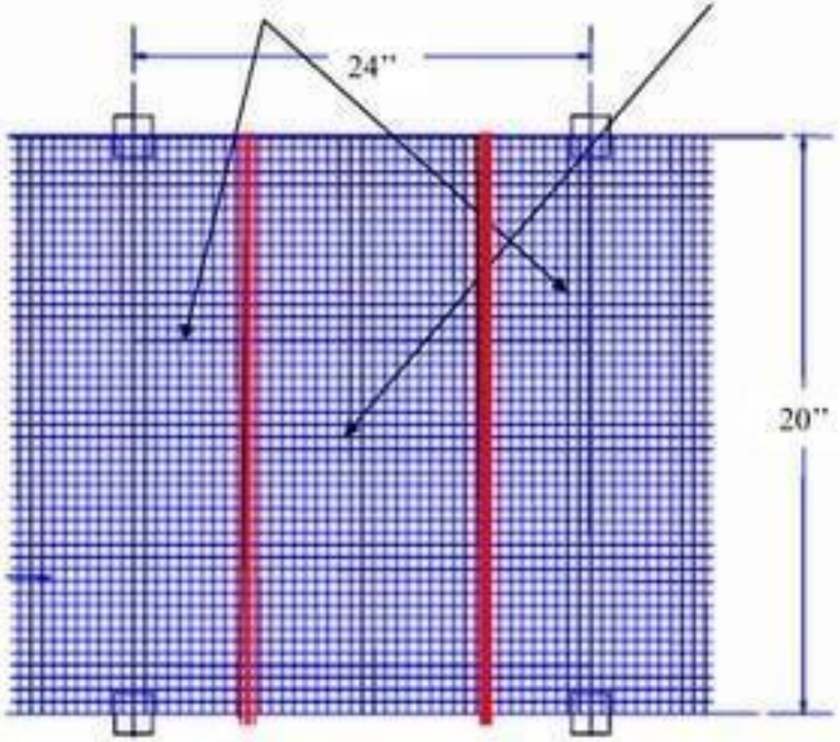
| Direction 1 | | Location 2 | $M_u$, ft-kips 3 | width b, in 4 | d, in 5 | Min$^m$ $A_s$ in$^2$ 6 | Reqd. $A_s$ in$^2$ 7 | No. of #4 bars 8 |
|---|---|---|---|---|---|---|---|---|
| Long direction | Column Strip (slab) | -ve | 177.45 | 120 | 7.5 | 1.95 | 5.92 | 30 @ 4.0"c/c |
| | | +ve | 76.44 | 120 | 7.5 | 1.95 | 2.55 | 13 @ 10.0" c/c |
| | Middle strip | -ve | 59.15 | 120 | 7.5 | 1.95 | 1.97 | 10 @ 13.0" c/c |
| | | +ve | 50.96 | 120 | 7.5 | 1.95 | 1.70 | 10 @ 13.0" c/c |
| Short direction | Column Strip (slab) | -ve | 143.48 | 120 | 8.0 | 1.95 | 4.48 | 23 @ 5.0"c/c |
| | | +ve | 61.80 | 120 | 8.0 | 1.95 | 1.93 | 13 @ 10.0"c/c |
| | Middle strip | -ve | 47.83 | 168 | 8.0 | 2.72 | 1.50 | 14 @ 12.5" c/c |
| | | +ve | 41.20 | 168 | 8.0 | 2.72 | 1.29 | 14 @ 12.5" c/c |

# Design Example 01

## Step-7: Calculation of ‘$A_s$’




Top bars in column & middle strips in both directions




Bottom bars in column & middle strips in both directions

# Design Example 01

## Step-7: Calculation of '$A_s$'

*Column #6 :*

$Minimum\ A_s = 0.0018bh,\ where\ b = width\ of\ column\ or\ middle\ strip$

*Column #7 :*

$Total\ required\ steel,\ A_s = \dfrac{M_u}{4d},only\ when\ f_y = 60\,ksi\ \&\ f_c' \le 4\,ksi$

$where\ M_u = Moment\ in\ ft-k\ and\ 'd'\ in\ inch$

*Column #8 :*

$No.of\ bars\ is\ calculated\ considering$

$1)\ maximum\ spacing\ '2h'\ or\ 18inch\ whichever\ is\ less.$

$2)\ minimum\ steel\ requirement$

$$Spacing,\ s = \frac{Width\ of\ strip}{n-1} \le '2h'\ or\ 18\,inch$$

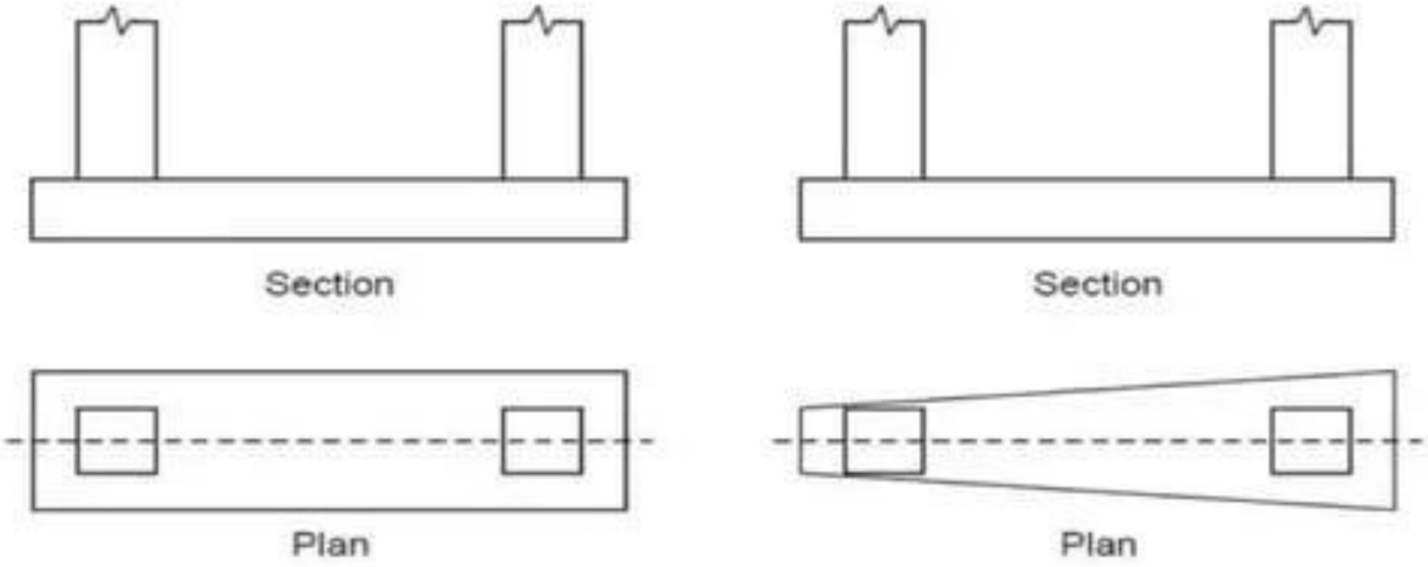
$$where\ 'n' = no.of\ total\ bars = \frac{Total\ required\ steel\ areas}{Area\ of\ one\ selected\ bar} = \frac{A_s}{A_b}$$

# Design of a Combined Footing

## (Week 12-13)

# What is combined footing?

- Whenever two or more columns in a straight line are carried on a single spread footing, it is called a combined footing.

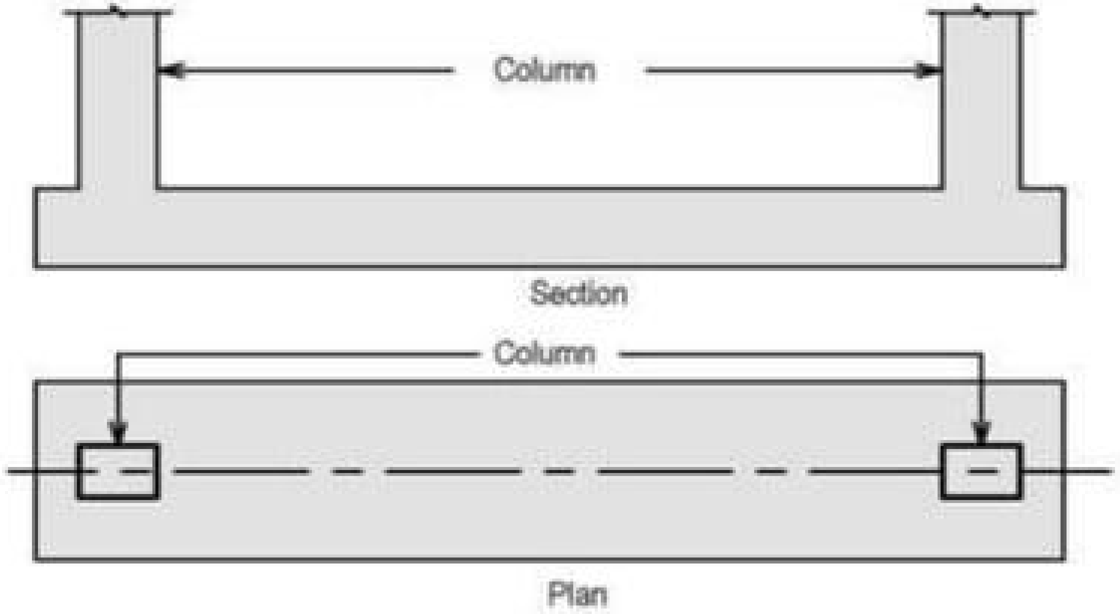


# How many types??

- The combined footing may be
  1. Rectangular
  2. Trapezoidal
  3. Tee-shaped in plan.
- The geometric proportions and shape are so fixed that the centeroid of the footing area coincides with the resultant of the column loads. This results in uniform pressure below the entire area of footing.

# Rectangular combined footing

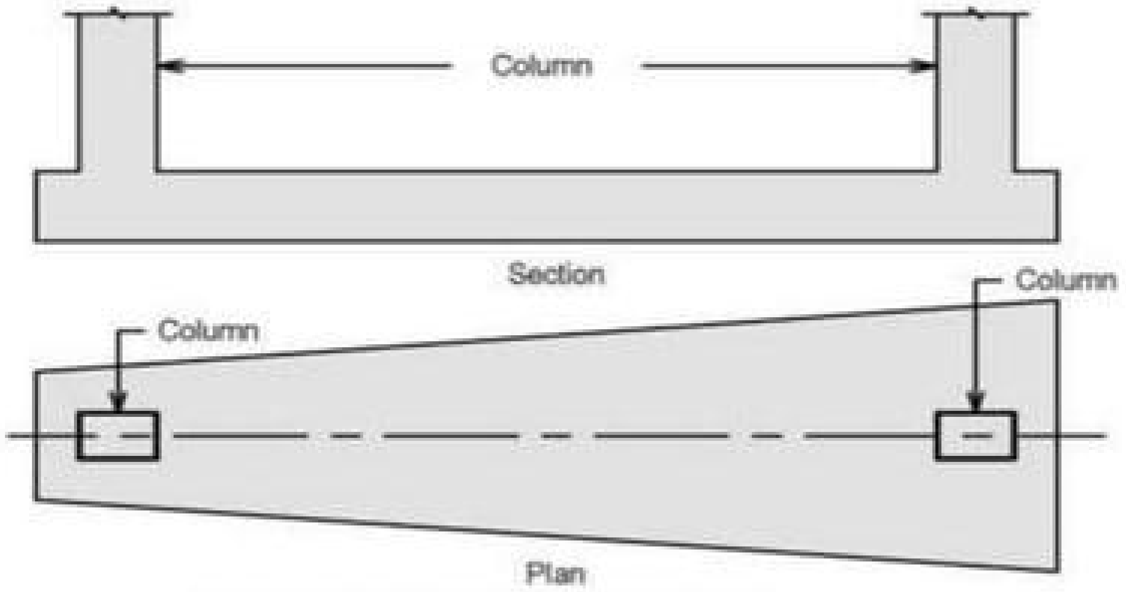
- Longitudinally, the footing acts as an upward loaded beam spanning between columns and cantilevering beyond. Using statics, the shear force and bending moment diagrams in the longitudinal direction are drawn. Moment is checked at the faces of the column. Shear force is critical at distance 'd' from the faces of columns or at the point of contra flexure. Two-way shear is checked under the heavier column.
- The footing is also subjected to transverse bending and this bending is spread over a transverse strip near the column.



(a) Rectangular combined Footing

# Trapezoidal footing

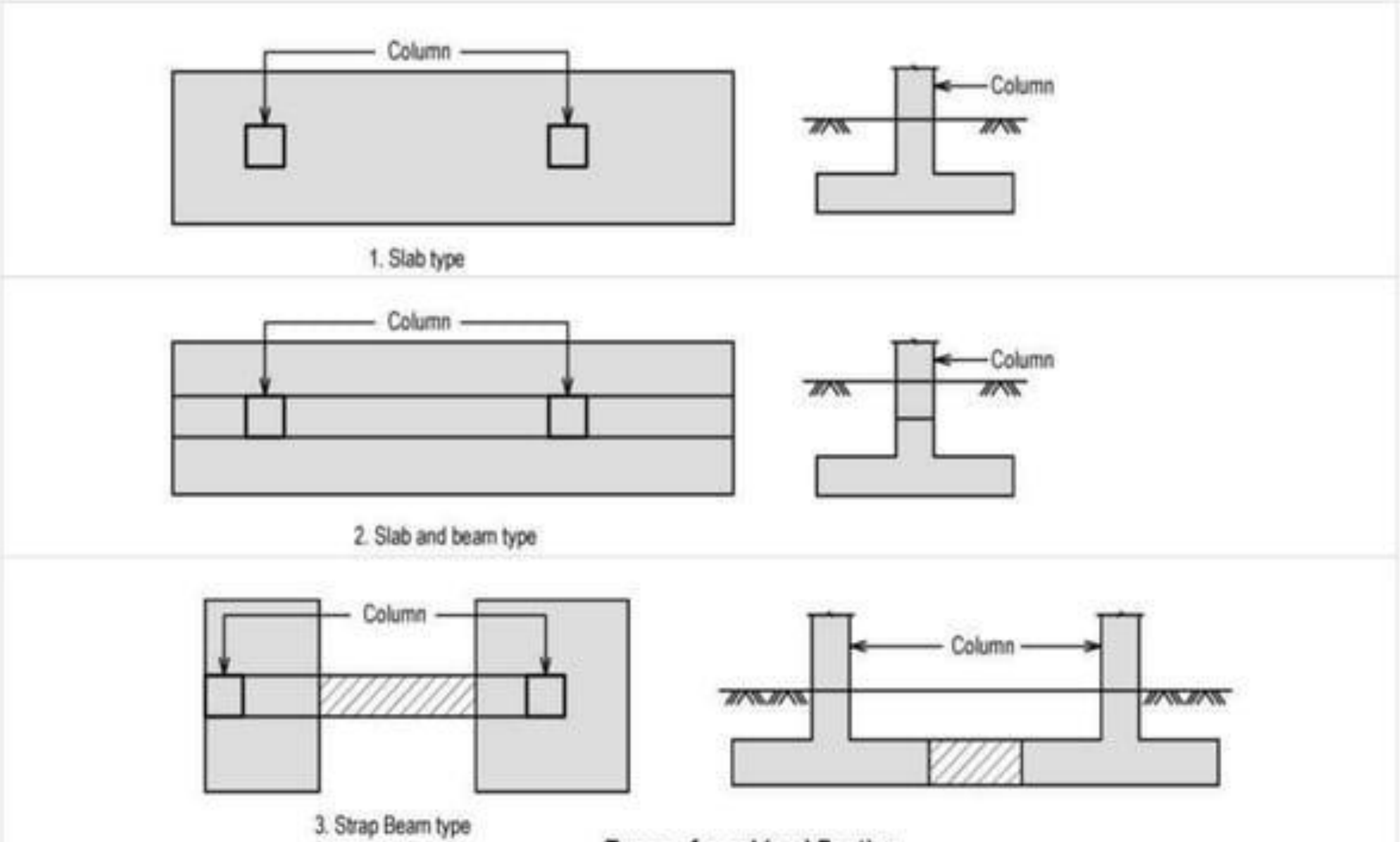
- Trapezoidal footing is provided when one column load is much more than the other. As a result, the both projections of footing beyond the faces of the columns will be restricted




**(b) Trapezoidal combined Footing**

There are three different types of combined footings:

- **Slab type combined footing** : It supports two or more column with bottom slab only.
- **Slab-beam type combined footing** : supports two or more column with bottom slab and beam.
- **Strap-beam type combined footing** : It is used when one column is located on a property line, resulting in is an eccentric load on a portion of the footing. Therefore provide a beam in the footing to the adjacent column footing to restrain the overturning effect.




**Types of combined Footing**




Two columns having cross-section of 500 × 500 mm and 600 × 600 mm are transmitting loads of 1250 kN and 1750 kN respectively. The c/c distance between the columns is 5 m and the bearing capacity of soil is 300 kN/m². Design a combined rectangular footing with beam joining the columns.

**Solution**

*Loads*

| | | |
|---|---|---|
| Super-imposed load | = 1250 + 1750 | = 3000 kN |
| Self-weight of footing (assuming 10% of superimposed load) | | = 300 kN |
| Total load | | = 3300 kN |

*Size of Footing*

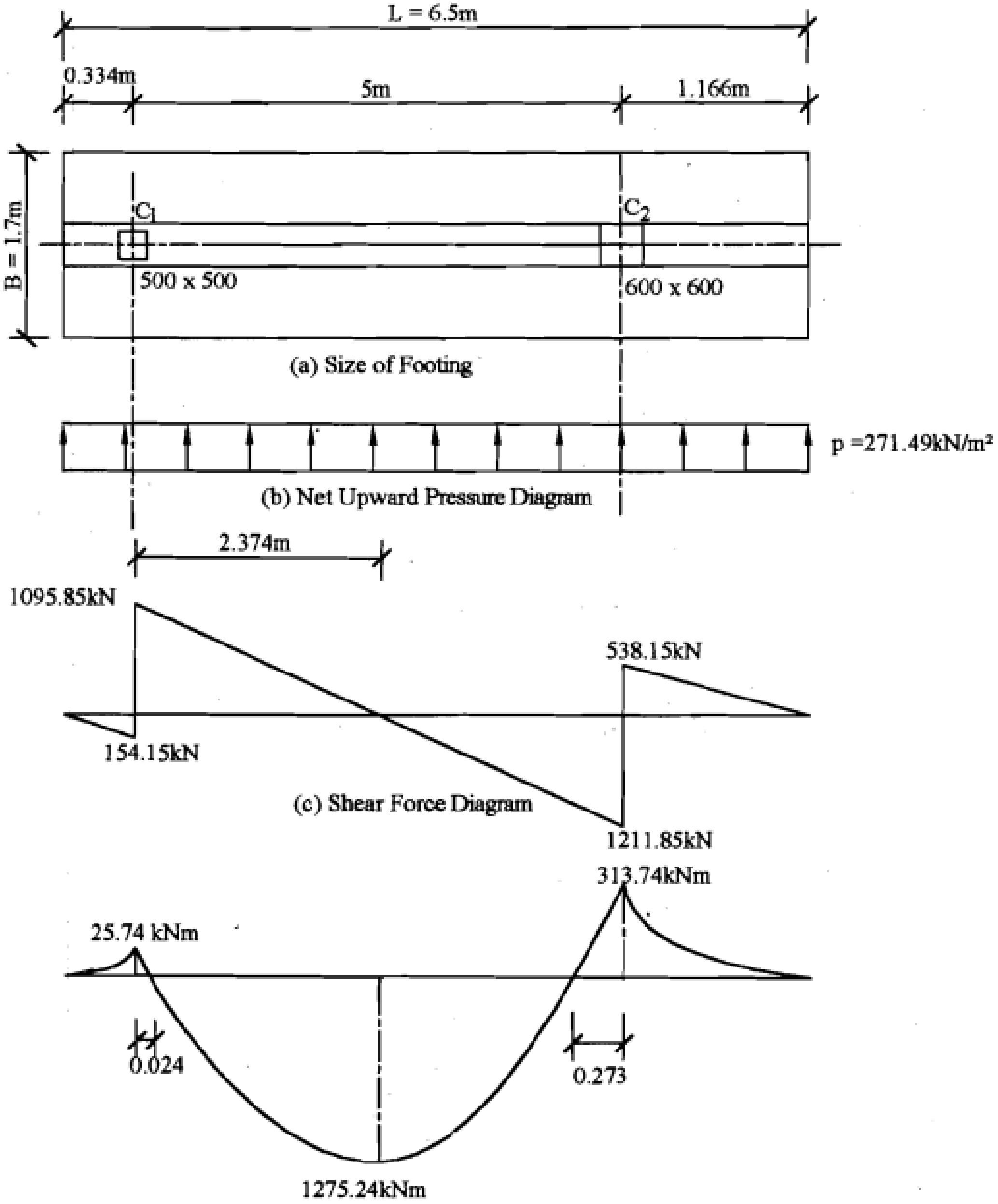
$$\text{Required area of footing} = \frac{3300}{300} = 11 \text{ m}^2$$

**Hence provided size of footing = 6.5 m × 1.7 m = 11.05 m² > 11 m²**

Let the C.G. of loads be at $\bar{x}$ from the centre of column $C_1$ (Figure 17.4(a)). Taking moment of superimposed loads about centre of column $C_1$,

$$-(1250 + 1750)\ \bar{x} + 1750 \times 5 = 0$$

or $\quad \bar{x} = 2.916$ m




(a) Size of Footing

(b) Net Upward Pressure Diagram

(c) Shear Force Diagram

(d) Bending Moment Diagram

For uniform soil pressure, C.G. of loads must coincide with C.G. of footing;

Projection of footing on L.H.S. of centre of column $C_1$

$$x_1 = \frac{L}{2} - 2.916 = 3.25 - 2.916 = 0.334 \text{ m}$$

Similarly, projection of footing on R.H.S. of centre of column $C_2$,

$$x_2 = \frac{L}{2} - (5 - 2.916) = 3.25 - 2.084 = 1.166 \text{ m}$$

Net upward pressure on footing

$$\frac{1250 + 1750}{6.5 \times 1.7} = 271.49 \text{ kN/m}^2 \quad \text{(Figure 17.4(b))}$$

The S.F.D. and B.M.D. have been drawn in Figure 17.4 (c & d).

*Design of Slab*

Let the width of the beam = 600 mm

$\therefore$ Projection of slab beyond the longitudinal face of beam = $\frac{1.7 - 0.6}{2} = 0.55$ m

$$\therefore M_{max} = 271.49 \times \frac{0.55^2}{2} = 41.06 \text{ kNm/m width}$$

$$\therefore d = \sqrt{\frac{M}{R_b b}} = \sqrt{\frac{41.06 \times 10^6}{0.874 \times 1000}} = 216.75 \text{ mm}$$

$$\therefore D = 216.75 + 40 + \frac{12}{2} = 262.75 \text{ mm}$$

**Provided $D$ = 350 mm**

$$d = 350 - 40 - 16\frac{12}{2} - = 288 \text{ mm}$$

$$A_{st} = \frac{M}{\sigma_{st} j_B d} = \frac{41.06 \times 10^6}{140 \times 0.865 \times 288} = 1177.29 \text{ mm}^2$$

$\therefore$ Provided $\phi$12@95 c/c (=1189.47 mm²/m)

***Check for shear***

S.F. at critical section (i.e. at $d$ from face of beam) = 271.49 (0.55 – 0.288)

= 71.13 kN/m width

$$\tau_v = \frac{71.13 \times 10^3}{1000 \times 288} = 0.247 \text{ N/mm}^2$$

$$p\% = \frac{1189.47}{1000 \times 288} \times 100 = 0.41\%$$

$$\tau_c = 0.22 + \frac{(0.29 - 0.22)}{0.25}(0.41 - 0.25) = 0.265 \text{ N/mm}^2$$

Permissible shear stress = $k\tau_c$ = 1 × 0.265 = 0.265 N/mm$^2$ > $\tau_v$

Hence no shear reinforcement is necessary.

**Check for developmet length**

$$L_d = \frac{\phi\sigma_s}{4\tau_{bd}} = \frac{12 \times 140}{4 \times 0.6} = 700 \text{ mm}$$

Straight length available beyond the face of the beam = 550 – 40 = 510 < $L_d$

With standard U-hook total lenth = 510 + 16 × 12 = 702 > 700 Hence O.K.

**Design of Beam**

**The beam will act as a T-beam in the span between points of contraflexture and as rectangular beam in the projected portion**

**Design of beam between $C_1$ & $C_2$**

$l_o$ for effective width calculation of *isolated* T-beam = 5 – (0.024 + 0.273) = 4.703 m

$$b_f = \frac{l_0}{\frac{l_0}{b}+4} + b_w = \frac{4.703}{\frac{4.703}{1.7}+4} + 0.6 = 1.295 \text{ m} < 1.7 \text{ m}$$

For balanced section

assuming $j_B = 0.9$

$$M = 0.45\, D_f\, d_f\, \sigma_{cbc} \left(\frac{2k_B d - D_f}{k_B}\right)$$

or $$1275.24 \times 10^6 = 0.45 \times 1295 \times 350 \times 5 \left(\frac{2 \times 0.404 \times d - 350}{0.404}\right)$$

or $$d = 1058.4 \text{ mm}$$

Hence provided $D$ = 1150 mm

$\therefore$ $$d = 1150 - 40 - 32 - \frac{32}{2} = 1062 \text{ (Assuming } \phi 32 \text{ reinforcement in two layers)}$$

**Areas of steel ($A_{st}$)**

i) **For critical B.M. in the span**

$M = \sigma_{st} \times A_{st} \times l_a$

$l_a \approx 0.9d = 0.9 \times 1062 = 955.8$ mm

$$\therefore\ A_{st} = \frac{M}{\sigma_{st} l_a} = \frac{1275.24 \times 10^6}{130 \times 955.8} = 10263.17 \text{ mm}^2$$

**Hence provided 13ϕ32 in two layers ($A_{st}$ = 10452 mm²)**

*Check for Development Length*

$$L_d \leq \frac{M_1}{V} + L_0$$

where $M_1 = \sigma_{st} A_{st} j_B d$

$= 140 \times n \times 804 \times 0.9 \times 1062$

$= n \times 107.58 \times 10^6$ Nmm

$V = 1085.86$ kN

$L_0$ = greater of 12ϕ (=12×32=384) or $d$ = 1062

Substituting all values in the above equation

$$L_d = 58.3 \times 32 \leq \left( \frac{n \times 107.58 \times 10^6}{1085.86 \times 10^3} + 1062 \right)$$

or $\quad n \geq 8.11$

**Hence all 13ϕ32 have been extended upto the edge of footing on R.H.S. and on L.H.S. all the bars may be bent at 90° to make up $L_0$ at the edge.**

ii) **For B.M. at the face of column on R.H.S. projection**

Load per m run = 271.49 × 1.7 = 461.53 kN/m

$$\therefore \quad M = 461.53 \times \frac{(1.166 - 0.3)^2}{2} = 173.064 \text{ kNm}$$

$$d = 1150 - 40 - \frac{32}{2} = 1094$$

$$\therefore \quad A_{st} = \frac{173.064 \times 10^6}{130 \times 0.865 \times 1094} = 1406.792 \text{ mm}^2$$

$$A_{st,min} = \frac{0.85bd}{f_y} = \frac{0.85 \times 600 \times 1094}{250} = 2231.76 \text{ mm}^2$$

**Hence provided 8ϕ20 ($A_{st}$ = 2513.27 mm$^2$)**

These bars may be bent at 90° at the edge to make up for development length

iii) **For B.M. at the face of column on L.H.S. projection**

$$M = 461.53 \times \frac{(0.334-0.25)^2}{2} = 1.628 \text{ kNm}$$

$A_{st,min}$ will only be sufficient.

**Hence provided 8ϕ20** and the bars may be bent at 90° at the edge to make up for development length

*Provision of Shear Reinforcement*

S.F. at *d* from interior face of R.H. column

$$V = \frac{1211.85}{(5-2.374)} \times (5-2.374-1.062) = 721.757 \text{ kN}$$

$$\tau_v = \frac{V}{bd} = \frac{721.757 \times 10^3}{600 \times 1062} = 1.133 < 1.6 \text{ N/mm}^2 \ (\tau_{max})$$

$$p\% = \frac{A_{st}}{bd} \times 100 = \frac{13 \times 804}{600 \times 1062} \times 100 = 1.64\%$$

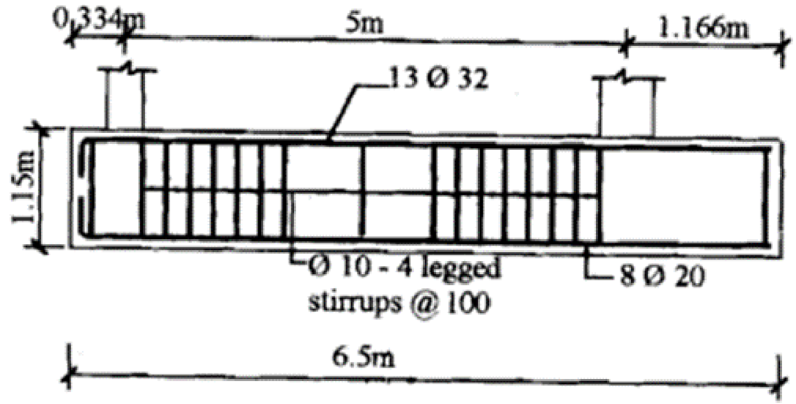
$$\tau_c = 0.42 + \frac{(0.44-0.42)}{0.25} \times (1.64-1.5) = 0.43 \text{ N/mm}^2$$

$$V_s = 721.757 - 0.431 \times 600 \times 1062 \times 10^{-3} = 447.124 \text{ kN}$$

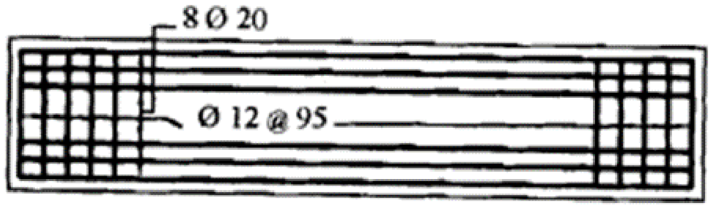
$$s_v = \frac{\sigma_{sv} A_{sv} d}{V_s} = \frac{140 \times 4 \times 78 \times 1062}{447.124 \times 10^3} = 103.75 \text{ mm}$$

**Hence provided ϕ10-4 legged stirrups @ 100 c/c**

The details of reinforcements have been shown in Figure 17.5.




(a) Detailing of Beam




(b) Detailing of Slab

**Figure 17.5: Details of the Designed Footing with Beam**

# Design of a Two-Way Slab

## (Week 14-15)

# One-Way Slab & Two-Way Slab


*Figure 10: Difference between One way slab & Two way slab*

*Source:* Google

# One-Way Slab & Two-Way Slab

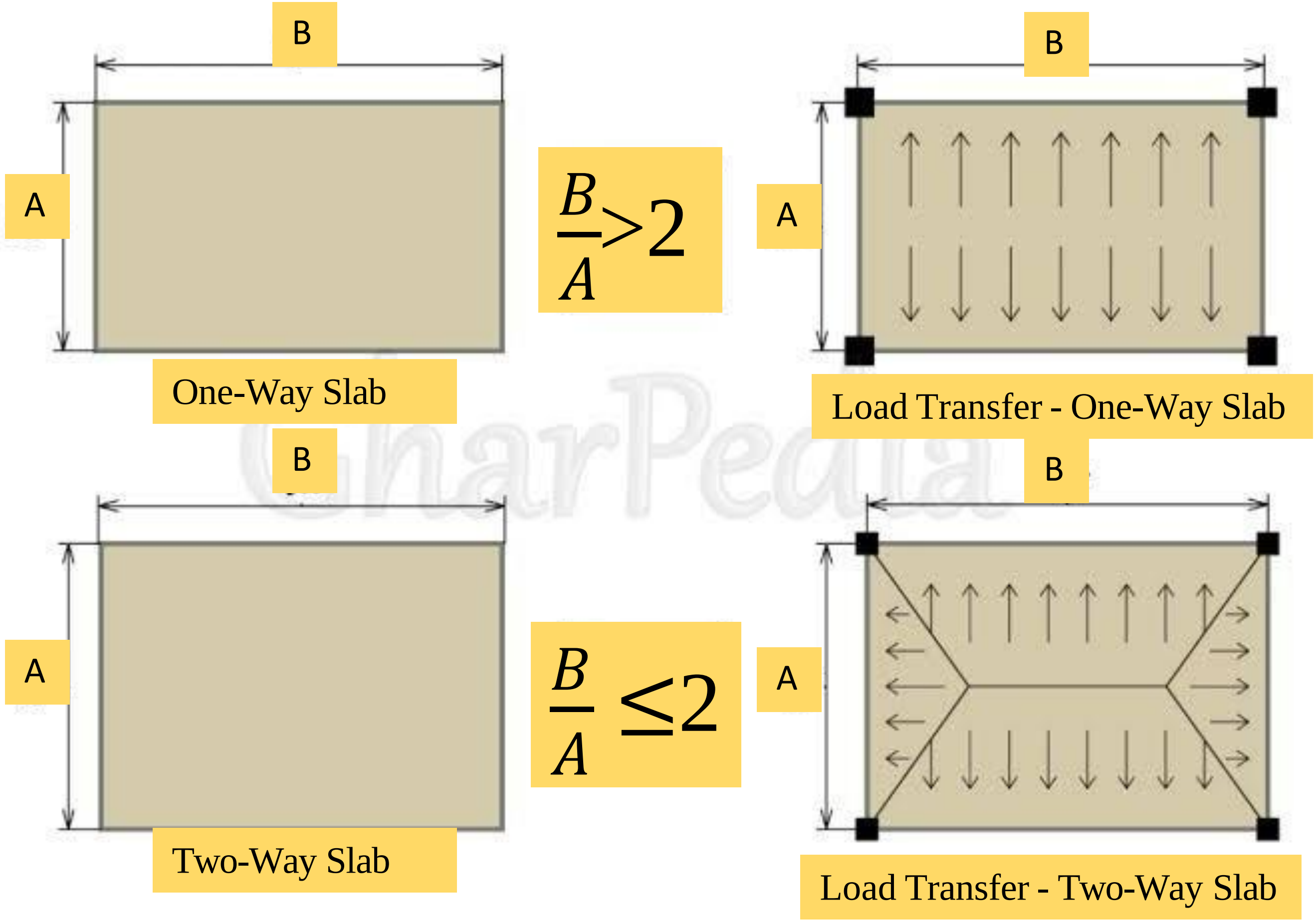
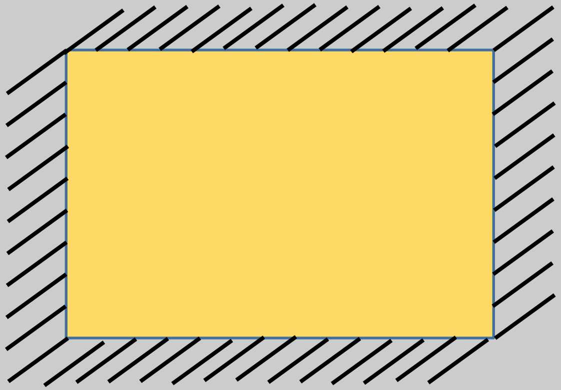
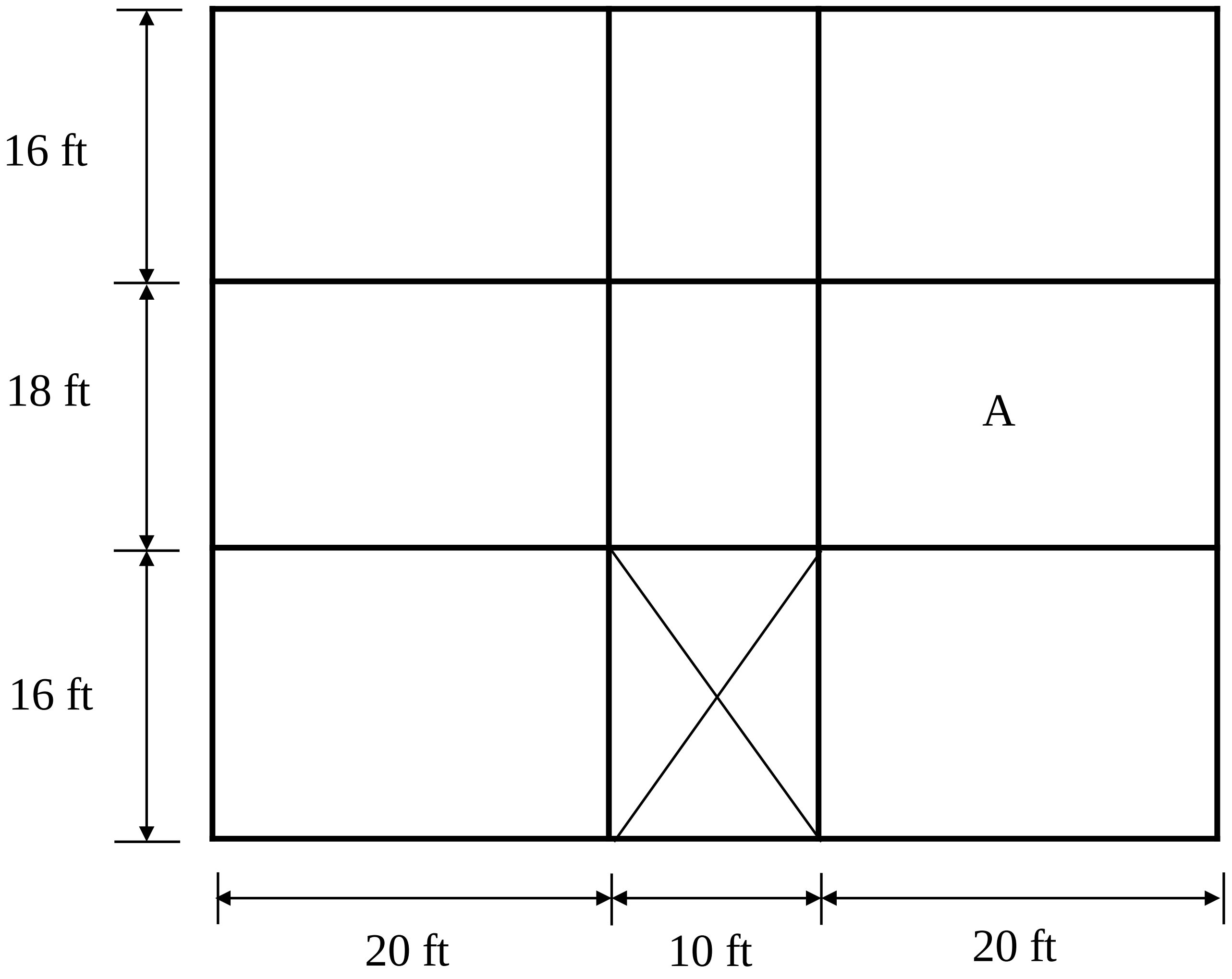



*Figure 11: Difference between One way slab & Two way slab*

# One-Way Slab & Two-Way Slab

| One-Way Slab | Two-Way Slab |
|---|---|
| 1. Load carries in one direction. | 1. Load caries in two directions. |
| 2. Deflection in one direction. | 2. Deflection in two directions. |
| 3. Supports in two opposite directions. | 3. At least two adjoining supports. |
| 4. $\frac{B}{A} > 2$ | 4. $\frac{B}{A} \leq 2$ |
| 5.  | 5.  |

Problem: Design the slab A. Partition wall, floor finishing and live load is 40 psf, 30 psf and 50 psf respectively.

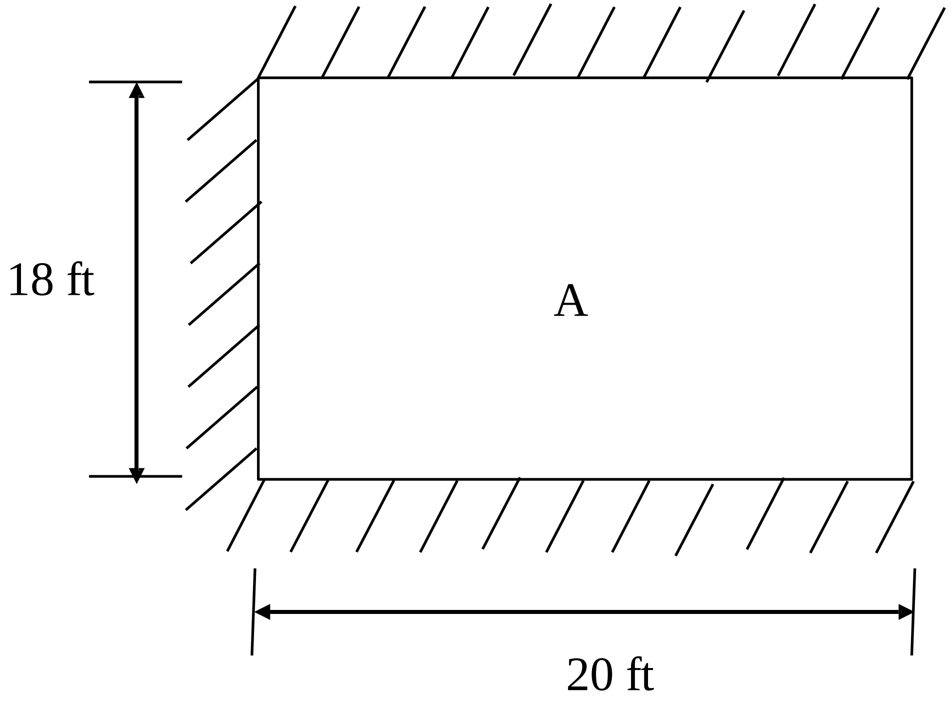



| Co-efficient | |
|---|---|
| $C_{A(neg)}$ | 0.068 |
| $C_{A(DL)}$ | 0.026 |
| $C_{A(LL)}$ | 0.036 |
| $C_{B(neg)}$ | 0.025 |
| $C_{B(DL)}$ | 0.015 |
| $C_{B(LL)}$ | 0.022 |

Type of Slab:

$$\frac{B}{A} = \frac{20}{18}$$

$= 1.11 < 2$ ( Two way slab confirmed)




Thickness Calculation:

$$t = \frac{2\,(L+B)}{180} \times 12$$

$$= \frac{2\,(20+18)}{180} \times 12$$

$$= 5.07 \approx 5.25 \text{ in}$$

## ***Load Calculation:***

Dead Load Calculations:

- ✓ Self-weight = $\frac{5.25}{12}$ x 150 = 65.63 psf
- ✓ Partition wall load = 40 psf
- ✓ Floor finishing load = 30 psf

Total dead load, $W_D$ = 65.63 + 40 + 30 = 130.63 psf

Live Load Calculation:

- ✓ Live load, $W_{LL}$ = 50 psf

Total load = $W_D$ + $W_{LL}$ = 130.63 + 50 = 180.63 psf

## *Moment Calculation:*

$$m = \frac{A}{B} = \frac{18}{20} = 0.9$$

Moment for shorter direction:

- Negative moment, $M_{A(neg)} = C_{A(neg)} \times W_T \times A^2$

$$= 0.068 \times 180.63 \times 18^2$$

$$= 3980 \text{ lb-ft}$$

- Positive moment, $M_{A(pos)} = C_{A(DL)} \times W_D \times A^2 + C_{A(LL)} \times W_{LL} \times A^2$

$$= 0.026 \times 130.63 \times 18^2 + 0.036 \times 50 \times 18^2$$

$$= 1684 \text{ lb-ft}$$

Moment for longer direction:

- Negative moment, $M_{B(neg)} = C_{B(neg)} \times W_T \times B^2$

$$= 0.025 \times 180.63 \times 20^2$$

$$= 1806 \text{ lb-ft}$$

- Positive moment, $M_{B(pos)} = C_{B(DL)} \times W_D \times B^2 + C_{B(LL)} \times W_{LL} \times B^2$

$$= 0.015 \times 130.63 \times 20^2 + 0.022 \times 50 \times 20^2$$

$$= 1224 \text{ lb-ft}$$

Moment for discontinuous edge:

- Negative moment, $M_{B(neg)} = M_{B(pos)}/3 = 408$ lb-ft

Depth check:

$$d = \sqrt{\frac{M_{max}}{Rb}} = \sqrt{\frac{3980 \text{ x } 12}{295 \text{ x } 12}} = 3.67''$$

$d_{eff} = 5.25\text{-}1 = 4.25'' > d$

Hence O.K.

## *Reinforcement Calculation:*

For Positive moment in shorter direction:

$$A_{s(pos)} = \frac{M}{f_{s} \text{ x } j \text{ x } d} = \frac{1684 \text{ x } 12}{24000 \text{ x } 0.875 \text{ x } 4.25} = 0.23 \text{ in}^2$$

But according to ACI code minimum steel required,

$$A_s = 0.0018bt$$

$$= 0.0018 \text{ x } 12 \text{ x } 5.25 = 0.11 \text{ in}^2$$

$$S_{max} = 5t = 5 \text{ x } 5.25 = 26.25''$$

Use #4 bar @ 10.25” c/c

For negative moment in shorter direction:

$$A_{s(neg)} = \frac{M}{f_{s \text{ x } j \text{ x } d}} = \frac{3980 \text{ x } 12}{24000 \text{ x } 0.875 \text{ x } 4.25} = 0.54 \text{ in}^2$$

Use #4 bar @ 4.25" c/c

For Positive moment in longer direction:

$$A_{s(pos)} = \frac{M}{f_{s \text{ x } j \text{ x } d}} = \frac{1224 \text{ x } 12}{24000 \text{ x } 0.875 \text{ x } 4.25} = 0.19 \text{ in}^2$$

Use #4 bar @ 10.5" c/c

For negative moment in longer direction:

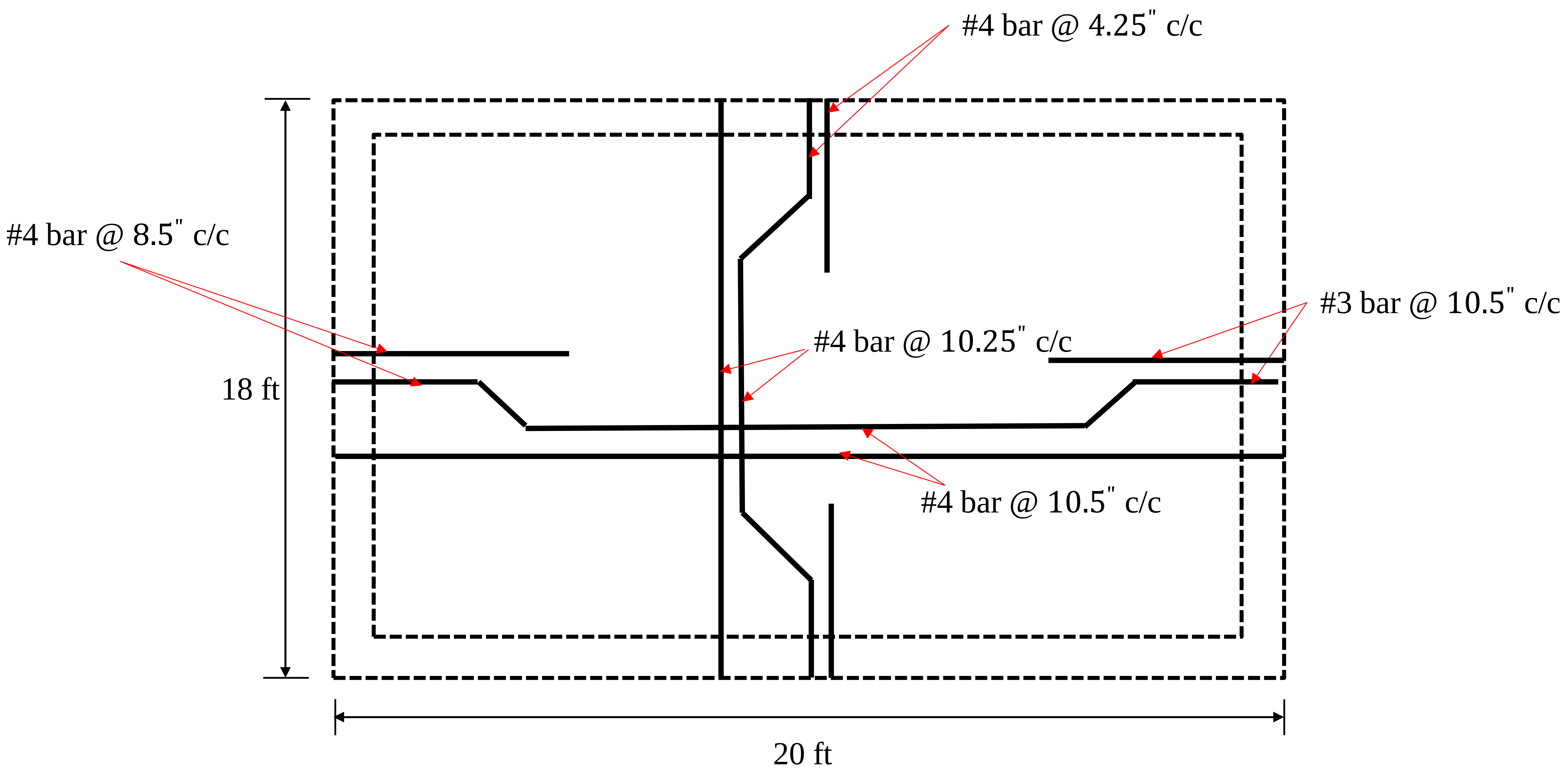
$$A_{s(neg)} = \frac{M}{f_{s\ x\ j\ x\ d}} = \frac{1806 \text{ x } 12}{24000 \text{ x } 0.875 \text{ x } 4.25} = 0.28 \text{ in}^2$$

Use #4 bar @ 8.5” c/c

For discontinuous edge:

$$A_{s(neg)} = \frac{M}{f_{s\ x\ j\ x\ d}} = \frac{408 \text{ x } 12}{24000 \text{ x } 0.875 \text{ x } 4.25} = 0.05 \text{ in}^2$$

Use #3 bars @ 10.5” c/c




Reinforcement Detail’s of Slab

Problem-4: An Engineer designed a slab using 60 grade steel to provide #4 bar @ $4.5''$c/c. If you want to provide 500W 10 mm bar then what would be the spacing?

Solution:

$$\text{Spacing} = \frac{0.2x\ 12}{A_{s1}} = 4.5$$

$$\text{or, } A_{s1} = 0.53\ in^2$$

$$A_{s1} f_1 = A_{s2} f_2$$

$$=> A_{s2} = \frac{0.53\ x\ 60}{72.5} = 0.44\ in^2$$

500W means $\frac{500 \text{ x } 145}{1000}$

= 72.5 grade steel

$$\text{Spacing} = \frac{\frac{\pi}{4}\ x\ 10^2 x 1000}{0.44\ x\ 25.4^2} = 276.67 \approx 250 \text{ mm c/c}$$


THANK YOU!