

SRM VALLIAMMAI ENGINEERING COLLEGE

SRM NAGAR, KATTANKULATHUR - 603203



ACADEMIC YEAR: 2026-2027

DEPARTMENT OF CIVIL ENGINEERING

ST 3165 – STRUCTURAL DESIGN AND DRAWING LABORATORY

Regulation 2023

I SEMESTER

LABORATORY MANUAL

Name : _____

Reg. No. : _____

Section : _____

Vision

To produce competent and quality engineers by imparting knowledge, excellence and global perspectives in Civil Engineering to our students and to make them ethically strong professional engineers to build our nation.

Mission

- To produce outstanding graduates with high technical knowledge to serve the nation.
- To impart value based education
- To provide solution to the challenges in the field of Civil Engineering.

PROGRAMME EDUCATIONAL OBJECTIVES (PEOs):

Graduates of the Programme ME Structural Engineering will

- | | |
|-------------|---|
| PEO1 | Gain knowledge and skills in structural engineering which will make them to have a career and professional accomplishment in the public or private sector organizations |
| PEO2 | Become consultants in Structural Engineering and solve complex real-life issues related to the analysis, design and maintenance of structures under various environmental conditions. |
| PEO3 | Contribute to the enhancement of knowledge in Structural Engineering by performing quality research in institutions of international repute or Research organizations or Academia. |
| PEO4 | Practice their profession with good communication, leadership, ethics and social responsibility and formulate solutions that are technically sound, economically feasible, and Socially acceptable. |
| PEO5 | Graduates will function in multi-disciplinary teams and adapt to evolving technologies through life-long learning and innovation |

PROGRAMME OUTCOMES (POs):

- | | |
|------------|--|
| PO1 | An ability to independently carry out research/investigation and development work to solve practical problems |
| PO2 | An ability to write and present a substantial technical report/document |
| PO3 | Students should be able to demonstrate a degree of mastery over the area as per the specialization of the program. The mastery should be at a level higher than the requirements in the appropriate bachelor's program |

PROGRAM SPECIFIC OUTCOMES (PSOs):

Graduates of the program M.E. Structural Engineering will be able to

PSO1	Knowledge of Structural Engineering discipline	Acquire in-depth knowledge of the Structural Engineering discipline, with an ability to evaluate, analyze and synthesize existing and new knowledge in structural design.
PSO2	Critical analysis of Structural Engineering issues and innovation	Critically analyze complex Structural Engineering problems, apply independent Judgment for Synthesizing Information And make innovative advances in a theoretical ,practical and policy context
PSO3	Conceptualization and evaluation of Engineering solutions to Structural Design issues	Conceptualize and solve Structural Engineering problems, evaluate potential solutions and arrive at technically feasible, economically viable and environmentally sound solutions with due consideration of health,safety,and socio-cultural factors

PREFACE

This instruction manual has been prepared by the Department of Civil Engineering to facilitate instruction doing practical classes and further to be used as a reference manual by the Civil Engineering students of this college. This manual covers explanation of experiments included in the syllabus according to Regulation 2023 of SRM VEC for the M.E Structural engineering degree course.

Mr. SATTAINATHAN SHARMA A
(Assistant Professor)

GENERAL INSTRUCTIONS

The following instructions should be strictly followed by students in the Lab:

- Students should wear lab coat in the lab.
- Students are advised to enter the lab WITH FORMAL SHOES ONLY.
- They are not supposed to move the systems and monitors.
- Students should bring necessary sheets for drawings.
- They should enter in the login name and password assigned to each student.
- Students are advised to complete their record work before the next class.
- Students are asked to logout from their area and switch off the computers before leaving the lab.
- Students can access the printers through lab technician.
- Students have free access to use the computers and software available in the lab.
- During the laboratory hours, accessing the internet is strictly prohibited.
- Computer games are strictly prohibited in the CAD lab.

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REG No: _____ Name: _____ Branch: _____

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COURSE OBJECTIVES:

- To make the student be familiar with the design and drawing of RC structural members.
- To make the student be familiar with the design and drawing of special RC structural members.
- To Understand and design and drawing of steel roof truss
- To understand and design and drawing of steel connection.
- To make the students to understand steel detailing RC members

LIST OF EXPERIMENTS:

1. Design and drawing of RC structural members.
2. Design and drawing of special RC Structural Members.
3. Design and drawing of RC T beam bridge
4. Design and drawing steel roof truss
5. Design and drawing of steel connections
6. Bar bending schedule

TOTAL: 45 PERIODS**COURSE OUTCOMES:**

1. Students will able to familiar with the design and drawing of RC structural members.
2. Students will able to familiar with the design and drawing of Special RC structural members.
3. Students will able to understanding design and drawing of steel roof truss
4. Students will able to understand design and drawing of steel connection.
5. Students will able to understand steel detailing RC members.

REFERENCES:

1. Krishnaraju N, Structural Design and Drawing, Universities Press, 2009
2. Punmia B. C, Ashok Kumar Jain and Arun Kumar Jain, Comprehensive Design of Steel Structures, Laxmi Publications Pvt, Ltd.,2003

CO	PO			PSO		
	1	2	3	1	2	3
1	3	2	2	3	3	3
2	3	1	-	2	1	1
3	2	-	2	3	2	3
4	3	1	2	3	2	2
5	3	-	1	2	1	1
Average	2.8	1.3	1.8	2.6	1.8	2.0

LIST OF EXPERIMENTS

1. Design and Detailing of One-way & Two Way Slab
2. Design and Detailing of Flat Slab
3. Design of Cantilever Retaining Wall
4. Design and Detailing of Counterfort Retaining Wall
5. Design and Detailing of RC Footings
6. Design and Detailing of RC Deep Beams
7. Design and Detailing of RC Poles
8. Design and Detailing of Stanchion
9. Analysis and Design of Tee-Beam Bridge
10. Analysis and Design of Steel Roof truss
11. Analysis and Design of Plate Girder
12. Analysis and Design of Pipe Culvert
13. Bar Bending Schedule of Slab
14. Bar Bending Schedule of Beam
15. Bar Bending Schedule of Column

Ex.No :

Design and Detailing of One-way Slab

Design a simply supported one –way slab over a clear span of 3.5 m. It carries a live load of 4 kN/m² and floor finish of 1.5 kN/m². The width of supporting wall is 230 mm. Adopt M20 concrete & Fe-415 steel.

Design Procedure

(Note: Use of IS 456:2000 and SP 16 is Permitted)

- Step : 1** Trail depth and effective span
- Step : 2** Load on slab
- Step : 3** Design bending moment and check for depth
- Step : 4** Area of Reinforcement
- Step : 5** Distribution Reinforcement
- Step : 6** Check for Shear
- Step : 7** Check for Deflection
- Step : 8** Check for Development length
- Step : 9** Check for cracking
- Step : 10** Reinforcement Detailing

(Using the above steps the design problem is done and necessary reinforcement diagram is draw in A3 Sheet)

Thus, One-way slab is designed and reinforcement Diagram is draw as per IS Codal provisions.

Ex.No :

Design and Detailing of Two-way Slab

Date :

Design a R.C Slab for a room measuring 6.5m X 5m. The slab is cast monolithically over the beams with corners held down. The width of the supporting beam is 230 mm. The slab carries superimposed load of 4.5kN/m². Use M-20 concrete and Fe-500 Steel.

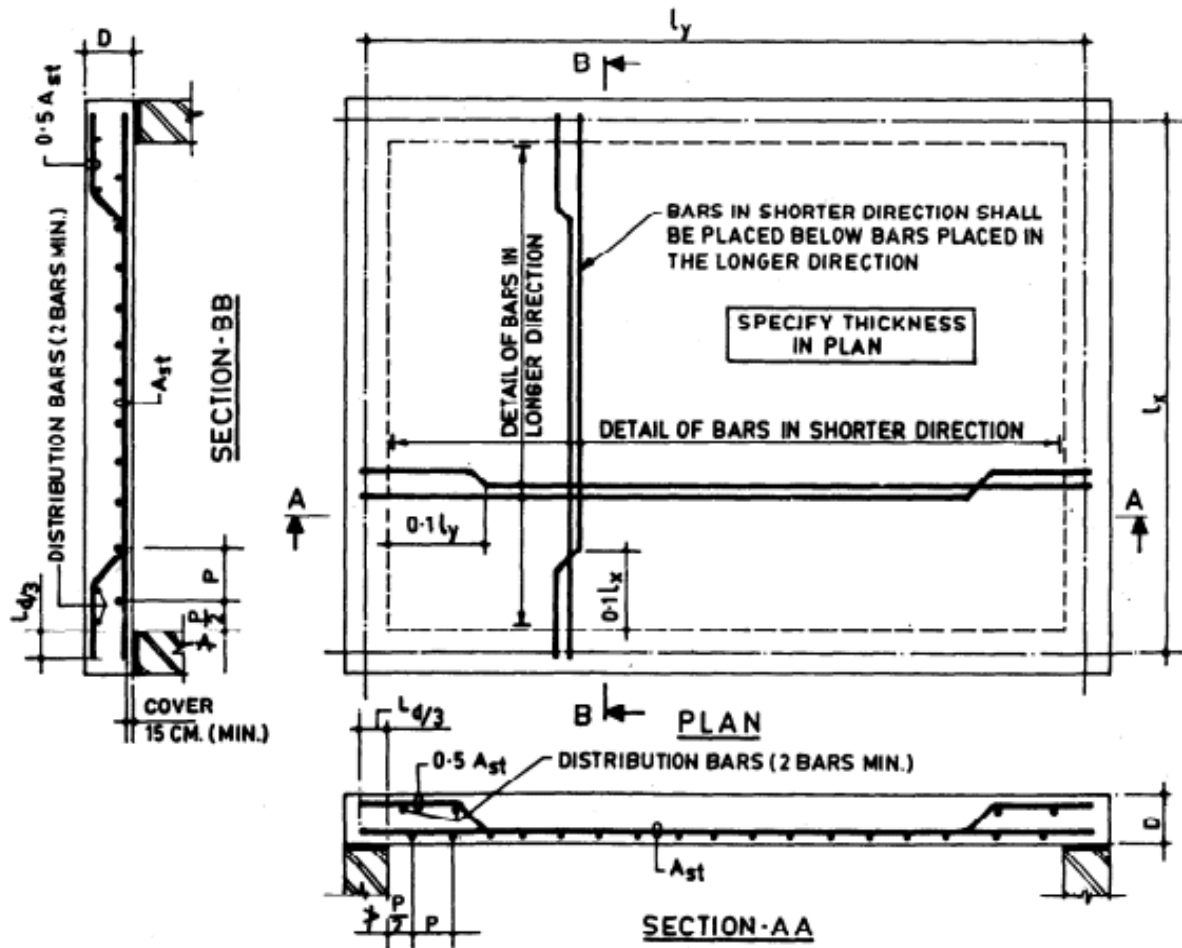
Design Procedure

(Note: Use of IS 456:2000 and SP 16 is Permitted)

- Step : 1** Trail depth and effective span
- Step : 2** Load on slab
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- Step : 7** Check for Deflection
- Step : 8** Check for Development length
- Step : 9** Check for cracking
- Step : 10** Reinforcement Detailing

(Using the above steps the design problem is done and necessary reinforcement diagram is draw in A3 Sheet)

Reinforcement Detailing



Result

Thus, Two-way slab is designed and reinforcement Diagram is draw as per IS Codal provisions.

Ex.No :

Design and Detailing of Flat Slab

Date :

Design an interior panel of a flat slab of size 5 m × 5 m without providing drop and column head. Size of columns is 500 × 500 mm and live load on the panel is 4 kN/m². Take floor finishing load as 1 kN/m². Use M20 concrete and Fe 415 steel.

Solution:

Thickness

Since drop is not provided and HYSD bars are used span to thickness ratio shall not exceed

$$\frac{1}{0.9 \times 32} = \frac{1}{28.8}$$

∴ Minimum thickness required

$$= \frac{\text{Span}}{28.8} = \frac{5000}{28.8} = 173.6 \text{ mm}$$

Let $d = 175 \text{ mm}$ and $D = 200 \text{ mm}$

Loads

$$\text{Self weight of slab} = 0.20 \times 25 = 5 \text{ kN/m}^2$$

$$\text{Finishing load} = 1 \text{ kN/m}^2$$

$$\text{Live load} = 4 \text{ kN/m}^2$$

$$\therefore \text{Total working load} = 10 \text{ kN/m}^2$$

$$\text{Factored load} = 1.5 \times 10 = 15 \text{ kN/m}^2$$

$$L_n = 5 - 0.5 = 4.5 \text{ m}$$

$$\therefore \text{Total design load in a panel } W = 15 L_2 L_n = 15 \times 5 \times 4.5 = 337.5 \text{ kN}$$

Moments

$$\text{Panel Moment } M_0 = \frac{WL_n}{8} = 337.5 \times \frac{4.5}{8} = 189.84 \text{ kNm}$$

$$\text{Panel -ve moment} = 0.65 \times 189.84 = 123.40 \text{ kNm}$$

$$\text{Panel +ve moment} = 0.35 \times 189.84 = 0.35 \times 189.84 = 66.44 \text{ kNm}$$

Distribution of moment into column strips and middle strip:

	Column Strip in kNm	Middle Strip in kNm
-ve moment	$0.75 \times 123.40 = 92.55$	30.85
+ve moment	$0.60 \times 66.44 = 39.86$	26.58

Checking the thickness selected:

Since Fe 415 steel is used,

$$M_{u \text{ lim}} = 0.138 f_{ck} b d^2$$

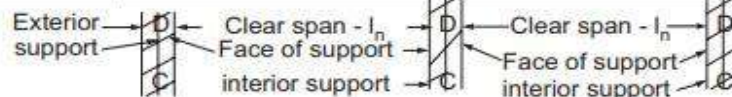
$$\text{Width of column strip} = 0.5 \times 5000 = 2500 \text{ mm}$$

$$\therefore M_{u \text{ lim}} = 0.138 \times 20 \times 2500 \times 175^2 = 211.3125 \times 10^6 \text{ Nmm}$$

$$= 211.3125 \text{ kNm}$$

Hence singly reinforced section can be designed i.e., thickness provided is satisfactory from the consideration of bending moment.

Strip	Type of bars	LOCATION	Minimum percentage of steel at section	WITHOUT DROP PANEL	WITH DROP PANEL
Column strip	Straight bars	TOP	50 Remainder		
		BOTTOM	50 Remainder		
	Bent bars*	TOP	50 Remainder		
		BOTTOM	50 Remainder		
Middle Strip	Straight bars	TOP	100		
		BOTTOM	50 Remainder		
	Bent bars*	TOP	50 Remainder		
		BOTTOM	50 Remainder		



Bar Length From Face of Support							
Minimum Length				Maximum Length			
Mark	a	b	c	d	e	f	g
Length	$0.14 l_n$	$0.20 l_n$	$0.22 l_n$	$0.30 l_n$	$0.33 l_n$	$0.20 l_n$	$0.24 l_n$

* Bent bars at exterior supports may be used if a general analysis is made.

Note. D is the diameter of the column and the dimension of the rectangular column in the direction under consideration.

Fig. Minimum bend joint locations and extensions for reinforcement in flat slabs

Check for Shear

The critical section for shear is at a distance $\frac{d}{2}$ from the column face. Hence periphery of critical section around a column is square of a size $= 500 + d = 500 + 175 = 675$ mm

Shear to be resisted by the critical section

$$V = 15 \times 5 \times 5 - 15 \times 0.675 \times 0.675 \\ = 368.166 \text{ kN}$$

$$\therefore \tau_v = \frac{368.166 \times 1000}{4 \times 675 \times 175} = 0.779 \text{ N/mm}^2$$

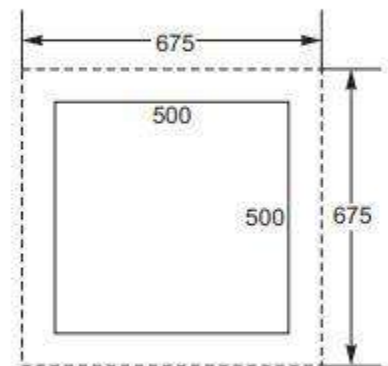
$$k_s = 1 + \beta_c \text{ subject to maximum of } 1.$$

$$\beta_c = \frac{L_1}{L_2} = \frac{5}{5} = 1$$

$$\therefore k_s = 1$$

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

safe in shear since $\tau_v < \tau_c$



Reinforcement

For -ve moment in column strip:

$$M_u = 92.55 \text{ kNm}$$

$$92.55 \times 10^6 = 0.87 f_y A_{st} d \left[1 - \frac{A_{st}}{bd} \frac{f_y}{f_{ck}} \right]$$

$$= 0.87 \times 415 \times A_{st} \times 175 \left[1 - \frac{A_{st}}{2500 \times 175} \times \frac{415}{20} \right]$$

$$i.e., \quad 1464.78 = A_{st} \left[1 - \frac{A_{st}}{21084.3} \right]$$

$$i.e., \quad A_{st}^2 - 21084.3 A_{st} + 1464.78 \times 21084.3 = 0$$

$$A_{st} = 1583.74 \text{ mm}^2$$

This is to be provided in a column strip of width 2500 mm. Hence using 12 mm bars, spacing required is given by

$$s = \frac{\pi/4 \times 12^2}{1583.74} \times 2500 = 178 \text{ mm}$$

Provide 12 mm bars at 175 mm c/c.

For +ve moment in column strip:

$$M_u = 39.86 \text{ kNm}$$

$$\therefore \quad 39.86 \times 10^6 = 0.87 \times 415 \times A_{st} \times 175 \left[1 - \frac{A_{st}}{2500 \times 175} \times \frac{415}{20} \right]$$

$$630.86 = A_{st} \left[1 - \frac{A_{st}}{21084.3} \right]$$

$$\text{or} \quad A_{st}^2 - 21084.3 A_{st} + 630.86 \times 21084.3 = 0$$

$$\therefore \quad A_{st} = 651 \text{ mm}^2$$

Using 10 mm bars, spacing required is

$$s = \frac{\pi/4 \times 10^2}{651} \times 2500 = 301.6 \text{ mm} < 2 \times \text{thickness of slab}$$

Hence provide 10 mm bars at 300 mm c/c.

Provide 10 mm diameter bars at 300 mm c/c in the middle strip to take up -ve and +ve moments. Since span is same in both directions, provide similar reinforcement in other direction also.

Reinforcement Details

It is as shown in Fig.

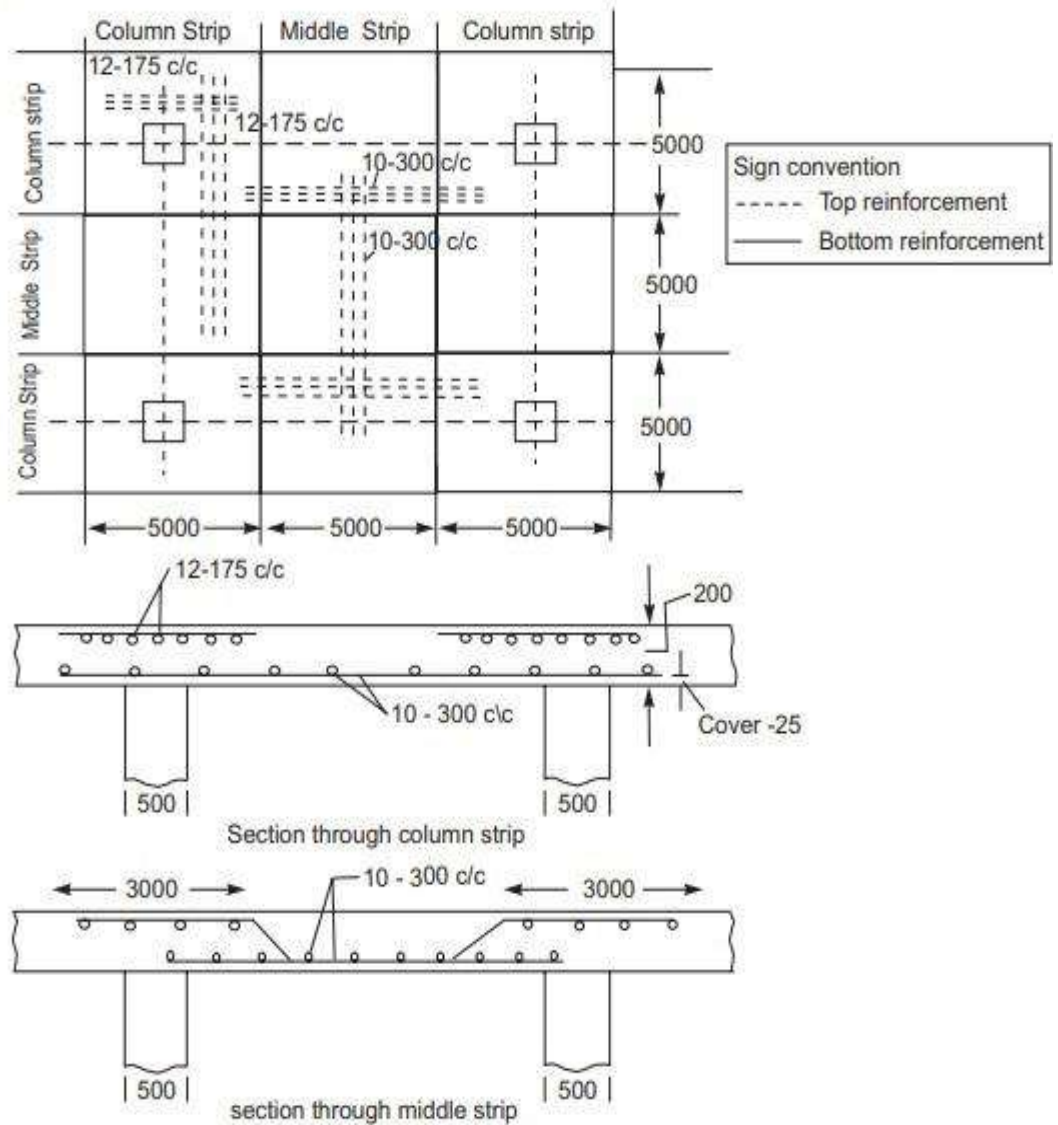


Fig. Reinforcement details [all dimension in mm units]

Result

Thus, Flat slab is designed and reinforcement Diagram is draw as per IS Codal provisions.

Ex.No :

Design and Detailing of Retaining Wall

Date :

Determine suitable dimensions of a cantilever retaining wall, which is required to support a bank of earth 4.0 m high above the ground level on the toe side of the wall. Consider the backfill surface to be inclined at an angle of 15° with the horizontal. Assume good soil for foundation at a depth of 1.25 m below the ground level with a safe bearing capacity of 160 kN/m^2 . Further assume the backfill to comprise granular soil with a unit weight of 16 kN/m^3 and an angle of shearing resistance of 30° . Assume the coefficient of friction between soil and concrete to be 0.5.

SOLUTION

1. Data given: $h = 4.0 + 1.25 = 5.25 \text{ m}$; $\mu = 0.5$

$$\theta = 15^\circ$$

$$\gamma_e = 16 \text{ kN/m}^3$$

$$\phi = 30^\circ$$

$$q_a = 160 \text{ kN/m}^2$$

• Earth pressure coefficients: $C_a = \left[\frac{\cos \theta - \sqrt{\cos^2 \theta - \cos^2 \phi}}{\cos \theta + \sqrt{\cos^2 \theta - \cos^2 \phi}} \right] \cos \theta = 0.373$

$$C_p = \frac{1 + \sin \theta}{1 - \sin \theta} = 3.0$$

2. Preliminary proportions:

- Thickness of footing base slab $\approx 0.08h = 0.08 \times 5.25 = 0.42 \text{ m}$ Assume a thickness of 420 mm.
- Assume a stem thickness of 450 mm at the base of the stem, tapering to a value of 150 mm at the top of the wall.
- For an economical proportioning of the length L of the base slab, it will be assumed that the vertical reaction R at the footing base is in line with the front face of the stem. For such a condition, (assuming the height above top of wall to be about 0.4 m), the length of the heel slab (inclusive of stem thickness):

$$X \approx \left(\sqrt{C_a/3} \right) h' = \sqrt{0.373/3} (5.25 + 0.4) \approx 2.0 \text{ m}$$

- Assuming a triangular base pressure distribution, $L = 1.5X = 3.0 \text{ m}$
- The preliminary proportions are shown in Fig.

3. Stability against overturning

- Force due to active pressure: $P_a = C_a \gamma_e h'^2 / 2$

where $h' = h + X \tan \theta$ [Fig. 14.28(a)]

$$= 5.25 + 2.0 \tan 15^\circ = 5.786 \text{ m}$$

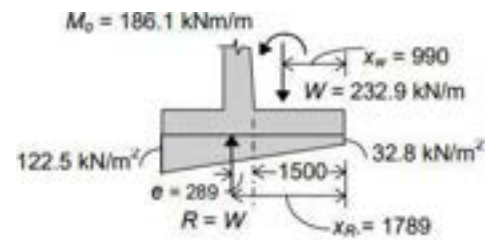
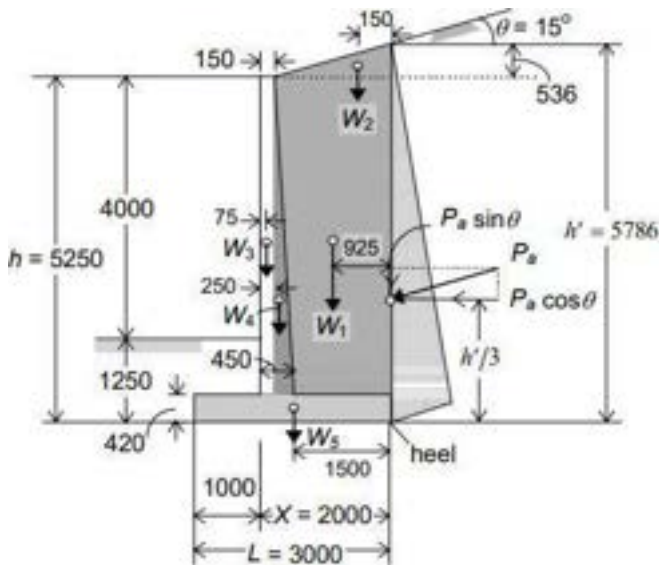
$$P_a = (0.373)(16)(5.786)^2 / 2 = 99.9 \text{ kN (per m length of wall)}$$

$$\Rightarrow P_a \cos \theta = 99.9 \cos 15^\circ = 96.5 \text{ kN}$$

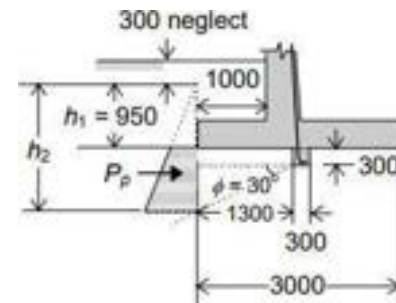
$$P_a \sin \theta = 99.9 \sin 15^\circ = 25.9 \text{ kN}$$

- Overturning moment $M_o = (P_a \cos \theta) h' / 3 = (96.5)(5.786/3) = 186.1 \text{ kNm}$
- Line of action of resultant of vertical forces [Fig. 14.28(a)] with respect to the heel can be located by applying statics, considering 1 m length of the wall:

force (kN)	distance from heel (m)	moment (kNm)
$W_1 = (16)(1.85)(5.25 - 0.42) = 143.0$	0.925	132.3
$W_2 = (16)(1.85)(0.5 \times 0.536) = 7.9$	0.617	4.9
$W_3 = (25)(0.15)(5.25 - 0.42) = 18.1$	1.925	34.8
$W_4 = (25 - 16)(4.83)(0.5 \times 0.30) = 6.5$	1.750	11.4
$W_5 = (25)(3.0)(0.42) = 31.5$	1.500	47.2
$P_a \sin \theta = 25.9$	0.000	0.0
$W = 232.9$		$M_W = 230.6$ kNm



(b) calculation of soil pressures



(c) design of shear key

Fig: Forces on wall (with preliminary proportions)

⇒ distance of resultant vertical force from

$$\text{heel } x_w = M_W / W = 230.6 / 232.9 = 0.990 \text{ m}$$

- Stabilising moment (about toe):

$$M_r = W(L - x_w)$$

$$= 232.9 \times (3.0 - 0.99)$$

$$= 468.1 \text{ kNm (per m length of wall)}$$

$$\Rightarrow (FS)_{\text{overturning}} = \frac{0.9 M_r}{M_o} = \frac{0.9 \times 468.1}{186.1} = 2.26 > 1.40$$

— OK

4. Soil Pressure at footing base:

- resultant vertical reaction $R = W = 232.9$ kN (per m length of wall)
- distance of R from heel: $L_R = (M_W + M_o) / R$
 $= (230.6 + 186.1) / 232.9 = 1.789 \text{ m}^\dagger$
- eccentricity $e = L_R - L/2 = 1.789 - 3.0/2 = 0.289 \text{ m}, < L/6 = 0.5$

- Hence, the resultant lies within the middle third of the base, which is desirable

$$\frac{6e}{L} = \frac{6 \times 0.289}{3.0} = 0.578$$

$$\Rightarrow q_{\max} = \frac{R}{L} \left(1 + \frac{6e}{L} \right) = \frac{232.9}{3.0} (1 + 0.578) = 122.5 \text{ kN/m}^2 < q_a \quad \text{--- OK}$$

$$\text{and } q_{\min} = \frac{232.9}{3.0} (1 - 0.578) = 32.8 \text{ kN/m}^2 \text{ [refer Fig. 14.28(b)]}$$

5. Stability against sliding

- Sliding force = $P_a \cos \theta = 96.5 \text{ kN}$
- Resisting force (ignoring passive pressure on the toe side) $F = \mu R$
 $= 0.5 \times 232.9 = 116.4 \text{ kN}$

$$\Rightarrow (FS)_{\text{sliding}} = \frac{0.9F}{P_a \cos \theta} = \frac{0.9 \times 116.4}{96.5} = 1.085 < 1.40$$

- Hence, a *shear key* may be provided to mobilise the balance force through passive resistance.
- Assume a shear key $300 \text{ mm} \times 300 \text{ mm}$, at a distance of 1300 mm from toe as shown in Fig. 14.28(c). Distance $h_2 = 0.950 + 300 + 1.300 \tan 30^\circ = 2.001 \text{ m}$

$$P_p = C_p \gamma_e (h_2^2 - h_1^2) / 2 = 3 \times 16 \times (2.001^2 - 0.95^2) / 2 = 74.44 \text{ kN}$$

$$(F.S)_{\text{sliding}} = \frac{0.9(116.4 + 74.44)}{96.5} = 1.78 > 1.4 \quad \text{--- OK}$$

6. Design of toe slab

- The loads considered for the design of the toe slab are as shown in fig. The net pressures, acting upward, are obtained by reducing the uniformly distributed self-weight of the toe slab from the gross pressures at the base. Self-weight loading = $25 \times 0.62 = 15.5 \text{ kN/m}^2$
- The net upward pressure varies from 126.9 kN/m^2 to 94.7 kN/m^2
- Assuming a clear cover of 75 mm and 16ϕ bars, $d = 620 - 75 - 8 = 537 \text{ mm}$
- Applying a load factor of 1.5 , the design shear force (at $d = 537 \text{ mm}$ from the front face of the stem) and the design moment at the face of the stem are given by:

$$V_u \approx 1.5(126.9 + 94.7)/2 \times (1.3 - 0.537) = 126.8 \text{ kN/m}$$

$$M_u = 1.5 \times [(94.7 \times 1.3^2/2) + (126.9 - 94.7) \times 0.5 \times 1.3^2 \times 2/3] = 147.2 \text{ kNm/m}$$

$$\bullet \text{ Nominal shear stress } \tau_v = \frac{V_u}{bd} = \frac{126.8 \times 10^3}{10^3 \times 537} = 0.236 \text{ MPa}$$

For a $\tau_c = 0.24 \text{ MPa}$, the required $p_t = 0.10$ with M 20 concrete

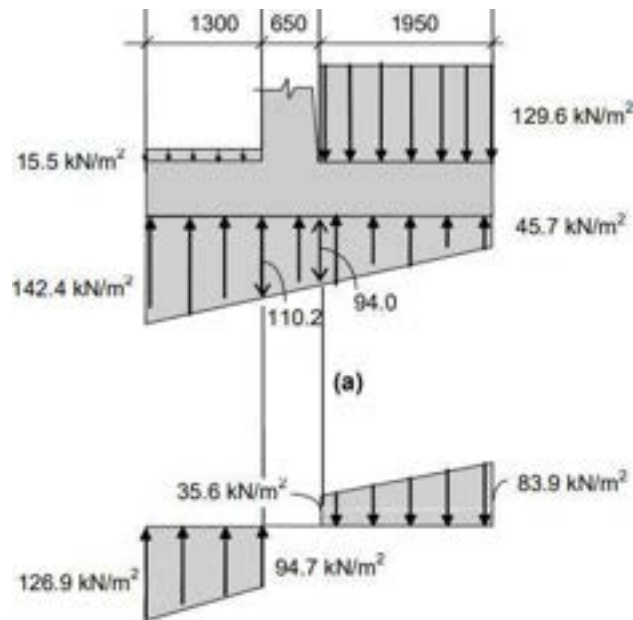


Fig: Net Soil pressures acting on base slab

- $$R = \frac{M_u}{bd^2} = \frac{147.2 \times 10^6}{10^3 \times 537^2} = 0.510 \text{ MPa}$$

$$\Rightarrow \frac{(p_t)_{reqd}}{100} = \frac{20}{2 \times 415} \left[1 - \sqrt{1 - 4.598 \times 0.510/20} \right] = 0.15 \times 10^{-2}, \text{ which is adequate for shear also}$$

$$\Rightarrow (A_{st})_{reqd} = (0.15 \times 10^{-2}) \times 10^3 \times 537 = 806 \text{ mm}^2/\text{m}$$

- Using 16 ϕ bars, spacing required = $201 \times 10^3/806 = 249 \text{ mm}$

Provide 16 ϕ bars @ 240 c/c at the bottom of the toe slab. The bars should extend by at least a distance $L_d = 47.0 \times 16 = 752 \text{ mm}$ beyond the front face of the stem, on both sides. As the toe slab length is only 1.3 m overall, no curtailment of bars is resorted to here.

7. Design of Heel Slab

- The loads considered for the design of the heel slab are as shown in Fig. The distributed loading acting downward on the heel slab is given by
 - Overburden + surcharge @ $16 \times (7.75 - 0.62) = 114.1 \text{ kN/m}_2$
 - Heel slab @ $25 \times 0.62 = 15.5 \text{ kN/m}^2 \Rightarrow w = 129.6 \text{ kN/m}^2$
- The net pressure acts downwards, varying between 35.6 kN/m_2 and 83.9 kN/m_2 as shown in Fig.
- Applying a load factor of 1.5, the design shear force and bending moment at the (rear) face of the stem are given by
 - $V_u = 1.5(35.6 + 83.9)/2 \times 1.95 = 174.8 \text{ kN/m}$
 - $M_u = 1.5 \times [(35.6 \times 1.952/2) + (83.9 - 35.6) \times 0.5 \times 1.952 \times 2/3] = 193.4 \text{ kNm/m}$
- Assuming a clear cover of 75 mm and 16 ϕ bars, $d = 620 - 75 - 8 = 537 \text{ mm}$
- Nominal shear stress

$$\tau_v = \frac{V_u}{bd} = \frac{174.8 \times 10^3}{10^3 \times 537} = 0.326 \text{ MPa}$$

Corresponding $\tau_c = 0.33$, with M 20 concrete; So $(p_t)_{\text{reqd.}} = 0.20$

$$\begin{aligned} \bullet R &= \frac{M_u}{bd^2} = \frac{193.4 \times 10^6}{10^3 \times 537^2} \\ &= 0.670 \text{ MPa} \\ \Rightarrow \frac{(p_t)_{\text{reqd.}}}{100} &= \frac{20}{2 \times 415} \left[1 - \sqrt{1 - 4.598 \times 0.670/20} \right] \\ &= 0.193 \times 10^{-2} \\ &< 0.20 \times 10^{-2} \text{ required for shear} \\ \Rightarrow (A_{st})_{\text{reqd.}} &= (0.20 \times 10^{-2}) \times 10^3 \times 537 \\ &= 1074 \text{ mm}^2/\text{m} \end{aligned}$$

- Using 16 ϕ bars, spacing required = $201 \times 103 / 1074 = 187 \text{ mm}$
Provide 16 ϕ bars @ 180 c/c at the top of the heel slab. The bars should extend by at least a distance $L_d = 47.0 \times 16 = 752 \text{ mm}$ beyond the rear face of the stem, on both sides. The bars may be curtailed part way to the heel; however, since the length is relatively short, this is not resorted to in this example.

8. Design of vertical stem

- Height of cantilever above base $h = 5.250 - 0.62 = 4.63 \text{ m}$
- Assuming a clear cover of 50 mm and 20 ϕ bars, d (at the base) = $650 - 50 - 10 = 590 \text{ mm}$
- Assuming a load factor of 1.5, maximum design moment

$$\begin{aligned} M_u &= 1.5 [C_u w_s h^2/2 + C_u \gamma_s h^3/6] \\ &= 1.5 \times (1/3) [40 \times 4.63^2/2 + 16 \times 4.63^3/6] \\ &= 346.7 \text{ kNm/m} \\ \Rightarrow R &= \frac{M_u}{bd^2} = \frac{346.7 \times 10^6}{10^3 \times 590^2} = 1.00 \text{ MPa} \\ \Rightarrow \frac{(p_t)_{\text{reqd.}}}{100} &= \frac{20}{2 \times 415} \left[1 - \sqrt{1 - 4.598 \times 1.00/20} \right] = 0.295 \times 10^{-2} \\ \Rightarrow (A_{st})_{\text{reqd.}} &= (0.295 \times 10^{-2}) \times 10^3 \times 590 = 1741 \text{ mm}^2/\text{m} \end{aligned}$$

- Using 16 ϕ bars, spacing required = $\frac{201 \times 10^3}{1741} = 115 \text{ mm}$

Provide 16 ϕ @ 110 c/c, bars extending into the 'shear key'. [This anchorage will be more than the minimum required: $L_d = 47.0 \times 16 = 752 \text{ mm}$]

9. Check for Shear:

Critical section is at $d = 0.59 \text{ m}$ above base, i.e., at $z_s = 4.63 - 0.59 = 4.04 \text{ m}$ below top edge. Shear force at critical section = $1.5 [C_u w_s z_s + C_u \gamma_s z_s^2/2]$

$$\begin{aligned} &= 1.5 \times (1/3) [40 \times 4.04 + 16 \times 4.04^2/2] \\ &= 146 \text{ kN/m} \end{aligned}$$

$$\tau_v = \frac{146 \times 10^3}{10^3 \times 590} = 0.248 \text{ MPa} < \tau_c \text{ for } p_t = 0.295$$

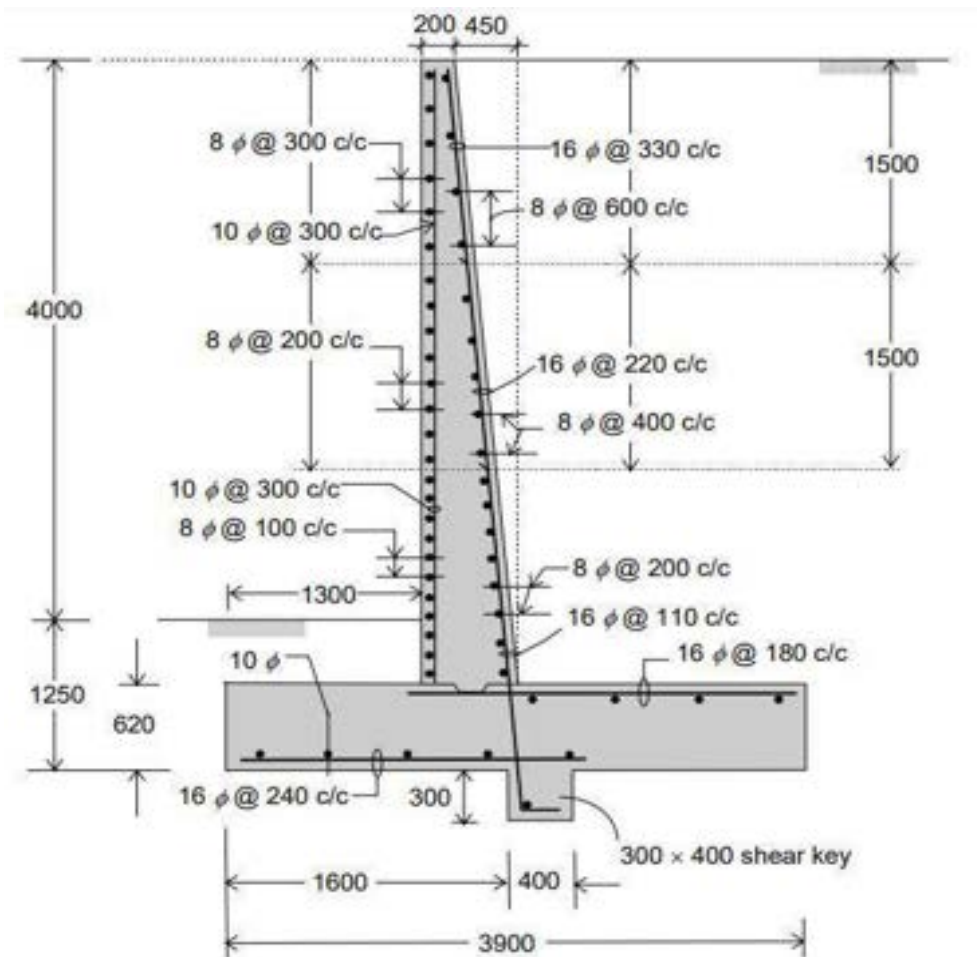
— OK

Note that since the shear stress is low and flexural reinforcement ratio also is low, the thickness of stem at base could be reduced for a more economical design.

- Curtailment of bars:
The curtailment of the bars may be done in two stages (at one-third and two-third heights of the stem above the base) as shown in Fig. below. It can be verified that the curtailment satisfies the Code requirements.
- Temperature and Shrinkage reinforcement Provide two-thirds of the (horizontal) bars near the front face (which is exposed to weather and the remaining one-third near the rear face. For the lowermost one-third height of the stem above base,

$$A_{st} = (0.0012 \times 103 \times 650) \times 2/3 = 520 \text{ mm}^2 / \text{m}$$
- Using 8 ϕ bars, spacing required = $50.3 \times 10^3 / 520 = 97 \text{ mm} \approx 100 \text{ mm}$. Provide 8 ϕ @ 100 c/c near front face and 8 ϕ @ 200 c/c near rear face in the lowermost one-third height of the wall; 8 ϕ @ 200 c/c near front face and 8 ϕ @ 400 c/c in the middle one-third height; and 8 ϕ @ 300 c/c near front face and 8 ϕ @ 600 c/c near the rear face in the top one-third height of the wall.
- Also provide nominal bars 10 ϕ bars @ 300 c/c vertically near the front face.

Reinforcement Detailing



Result

Thus, Cantilever Retaining Wall is designed and reinforcement Diagram is draw as per IS Codal provisions.

Ex.No :

Design and Detailing of Counterfort Retaining Wall

Date :

Design a suitable counterfort retaining wall to support a level backfill, 7.5 m high above the ground level on the toe side. Assume good soil for foundation at a depth of 1.5 m below the ground level with a safe bearing capacity of 170 kN/m^2 . Further assume the backfill to comprise granular soil with a unit weight of 16 kN/m^3 and an angle of shearing resistance of 30° . Assume the coefficient of friction between soil and concrete to be 0.5. Use M 25 and Fe 415 steel.

Solution:

1. Data given: $h = 7.5 + 1.5 = 9.0 \text{ m};$ $\mu = 0.5$
 $\theta = 0^\circ$ $\gamma_e = 16 \text{ kN/m}^3$
 $\phi = 30^\circ$ $q_a = 170 \text{ kN/m}^2$

• Earth pressure coefficients: $C_a = \frac{1 - \sin \theta}{1 + \sin \theta} = 0.333$
 $C_p = \frac{1 + \sin \theta}{1 - \sin \theta} = 3.0$

2. Preliminary proportions:

- The (triangular shaped) counterforts are provided on the rear (backfill) side of the wall, interconnecting the stem with the heel slab.
 - Spacing of counterforts $\approx h/3$ to $h/2 = 3.0 \text{ m}$ to 4.5 m
 - Assume the counterforts are placed with a clear spacing of 3.0 m .
 - Thickness of counterforts $\approx 0.05h = 0.05 \times 9.0 = 0.45 \text{ m}$. Assume a thickness of 500 mm .
- Thickness of heel slab $\approx 0.05h = 0.05 \times 9.0 = 0.45 \text{ m}$. Assume a thickness of 500 mm
- Assuming that the front buttresses are not provided, Thickness of toe slab $\approx 0.08h = 0.08 \times 9.0 = 0.72 \text{ m}$. Assume a thickness of 720 mm
- Thickness of stem slab $\approx 0.06h = 0.06 \times 9.0 = 0.54 \text{ m}$. Assume a stem thickness of 600 mm at the base of the stem, tapering to a value of 300 mm at the top of the wall.
- For an economical proportioning of the length L of the base slab, it is assumed that the vertical reaction R at the footing base is in line with the front face of the stem. For such a condition, (inclusive

$$X \approx \left(\sqrt{C_a/3} \right) h = \sqrt{0.333/3} (9.0) = 3.0 \text{ m}$$

of stem thickness)

- Assuming a triangular base pressure distribution, $L = 1.5X = 4.5 \text{ m}$
- The preliminary proportions are shown in Fig.

3. Stability against overturning

- Forces due to active pressure (per m length of wall)

$$P_a = C_a \gamma_e h^2/2 = (0.333)(16)(9.0)^2/2 = 216.0 \text{ kN}$$

• Overturning moment $M_o = P_a \times h/3$
 $\Rightarrow M_o = 216.0 \times (9.0/3)$
 $= 648.0 \text{ kNm (per m length of wall)}$

- Line of action of resultant of vertical forces with respect to the heel can be located by applying statics, considering 1 m length of the wall (the marginal additional weight due to counterfort is ignored)
 $\Rightarrow$ distance of resultant vertical force from heel $xW = MW / W = 864.8 / 506.9 = 1.706$ m

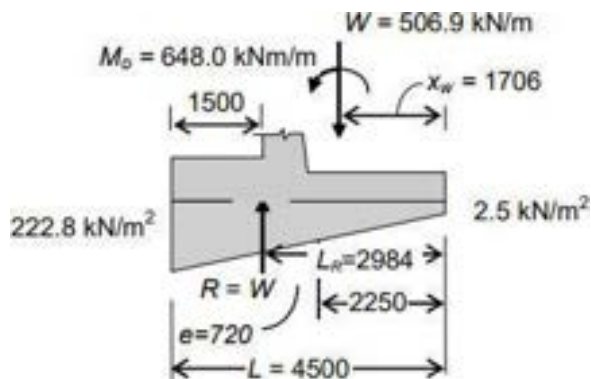
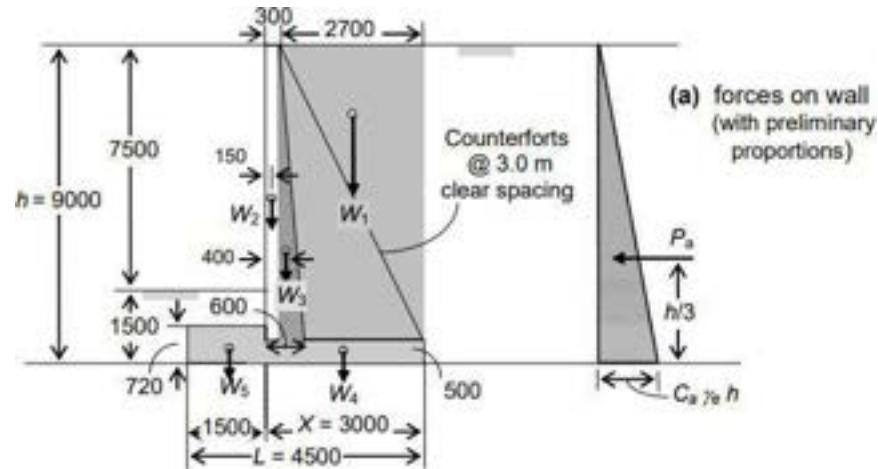


Fig: Cal. Of Soil Pressure

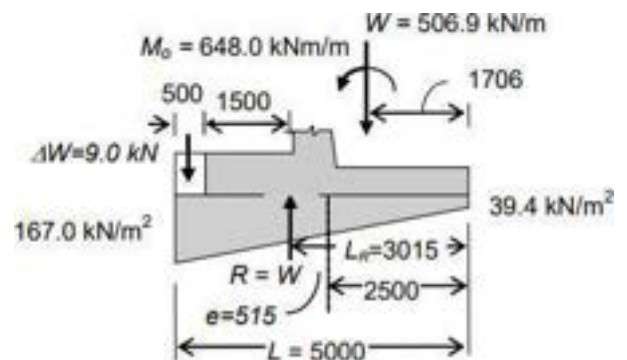


Fig: Revised design for safe soil pressures

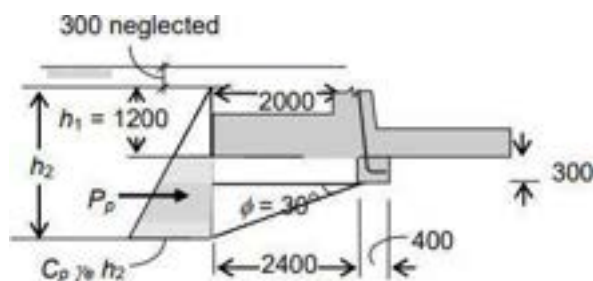


Fig: Design of Shear Key

force (kN)	distance from heel (m)	moment (kNm)
$W_1 = (16)(2.7)(9.0 - 0.5) = 367.20$	1.35	495.72
$W_2 = (25)(0.3)(9.0 - 0.5) = 63.75$	2.85	181.70
$W_3 = (25-16)(0.5)(0.3)(9.0 - 0.5) = 11.48$	2.60	29.85
$W_4 = (25)(3.0)(0.5) = 37.50$	1.50	56.30
$W_5 = (25)(1.5)(0.72) = 27.00$	3.75	101.25
$W = 506.9$		$M_W = 864.8$

- Stabilising moment (about toe):

$$M_r = W(L - x_W)$$

$$= 506.9 \times (4.5 - 1.706)$$

$$= 1416.4 \text{ kNm (per m length of wall)}$$

$$\Rightarrow (FS)_{\text{overturning}} = \frac{0.9M_r}{M_o} = \frac{0.9 \times 1416.4}{648.0} = 1.967 > 1.40 \quad \text{--- OK}$$

4. Soil Pressure at Footing Base:

- resultant vertical reaction $R = W = 506.9 \text{ kN}$ (per m length of wall)
- distance of R from heel: $L_R = (M_W + M_o)/R$
 $= (864.8 + 648.0) / 506.9 = 2.984 \text{ m}^\dagger$
- eccentricity $e = L_R - L/2 = 2.984 - 4.5/2 = 0.734 \text{ m}$ ($< L/6 = 0.75$)
indicating that the resultant lies well inside the middle third of the base.

$$\Rightarrow \frac{6e}{L} = \frac{6 \times 0.734}{4.5} = 0.978$$

$$\Rightarrow q_{\min} = \frac{R}{L} \left(1 - \frac{6e}{L}\right) = \frac{506.9}{4.5} (1 - 0.978) = 2.5 \text{ kN/m}^2 > 0 \quad \text{--- OK}$$

$$\Rightarrow q_{\max} = \frac{R}{L} \left(1 + \frac{6e}{L}\right) = \frac{506.9}{4.5} (1 + 0.978) = 222.8 \text{ kN/m}^2 > q_a = 170 \text{ kN/m}^2 \quad \text{--- UNSAFE.}$$

Hence, the length of the base slab needs to be suitably increased on the toe side – say, by 500 mm.

Let $L = 5.0 \text{ m}$. Additional weight due to 500 mm extension of toe slab $\Delta W = 25 \times 0.5 \times 0.72 = 9.0 \text{ kN}$

$$\Rightarrow R = W + \Delta W = 506.9 + 9.0 = 515.9 \text{ kN}$$

$$\text{Considering moments about the heel: } 515.9 L_R = 864.8 + (9.0)(5.0 - 0.25) + 648.0 \Rightarrow L_R = 3.015 \text{ m}$$

- Revised eccentricity $e = L_R - L/2 = 3.015 - 5.0/2 = 0.515 \text{ m } (< L/6 = 0.83)$

$$\Rightarrow \frac{6e}{L} = \frac{6 \times 0.515}{5.0} = 0.618$$

$$\Rightarrow q_{min} = \frac{R}{L} \left(1 - \frac{6e}{L} \right) = \frac{515.9}{5.0} (1 - 0.618) = 39.4 \text{ kN/m}^2 > 0 \quad \text{— OK}$$

$$\Rightarrow q_{max} = \frac{R}{L} \left(1 + \frac{6e}{L} \right) = \frac{515.9}{5.0} (1 + 0.618) = 167.0 \text{ kN/m}^2$$

$$< q_a = 170 \text{ kN/m}^2 \quad \text{— OK.}$$

5. Stability against sliding

- Sliding force $= P_a = 216.0 \text{ kN}$ (per m length of wall)
- Resisting force (ignoring passive pressure) $F = \mu R$
 $= 0.5 \times 515.9 = 257.9 \text{ kN} > P_a$

$$\bullet \quad (F.S.)_{sliding} = \frac{0.9F}{P_a} = \frac{0.9 \times 257.9}{216.0} = 1.075 < 1.4 \quad \text{— UNSAFE.}$$

- Hence, a *shear key* needs to be provided to generate the balance force through passive resistance.

$$\text{Required } P_p = 1.4 \times 216.0 - 0.9 \times 257.9 = 70.3 \text{ kN (per m length of wall)}$$

Providing a shear key $400 \text{ mm} \times 300 \text{ mm}$ at 2.4 m from toe,

$$h_2 = 1.2 + 0.3 + 2.4 \tan 30^\circ = 2.89 \text{ m} \quad P_p = 3 \times 16(2.89^2 - 1.2^2)/2 = 165.9 \text{ kN}$$

$$\Rightarrow (F.S.)_{sliding} = \frac{0.9 \times (257.9 + 165.9)}{216.0} = 1.766 > 1.4 \quad \text{— OK}$$

6. Design of toe slab

- The loads considered for the design of the toe slab are as shown in Fig.
- The net pressures, acting upward, are obtained by reducing the uniformly distributed self-weight of the toe slab from the gross pressures at the base. Self-weight loading $= 25 \times 0.72 = 18.0 \text{ kN/m}^2$

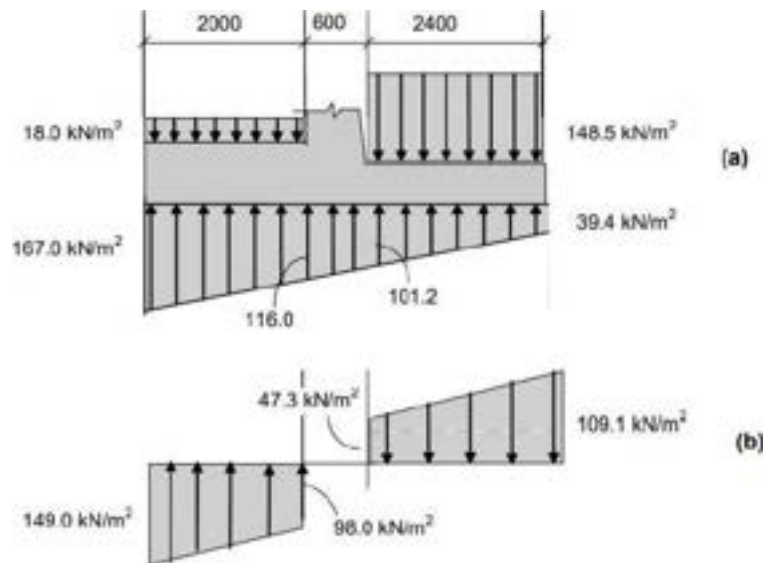


Fig: Net soil pressures acting on base slab

- The net upward pressure varies from 149.0 kN/m² to 97.9 kN/m², as shown above.
- Assuming a clear cover of 75 mm and 16 ϕ bars, $d = 720 - 75 - 8 = 637$ mm
- Applying a load factor of 1.5, the design shear force (at $d = 637$ mm from the front face of the stem) and the design moment at the face of the stem are given by:

$$V_u \approx 1.5 \times (149.0 + 97.9)/2 \times (2.0 - 0.637) = 252.4 \text{ kN/m}$$

$$M_u = 1.5 \times [(97.9 \times 2.0^2/2) + (149.0 - 97.9) \times 0.5 \times 2.0^2 \times 2/3] = 395.9 \text{ kNm/m}$$

- Nominal shear stress $\tau_v = \frac{V_u}{bd} = \frac{252.4 \times 10^3}{10^3 \times 637} = 0.396 \text{ MPa}$

For a $\tau_c = 0.396 \text{ MPa}$, the required $p_t = 0.32$ with M 25 concrete [refer Eq. 6.1]

- $R = \frac{M_u}{bd^2} = \frac{395.9 \times 10^6}{10^3 \times 637^2} = 0.976 \text{ MPa}$

$$\Rightarrow \frac{(p_t)_{reqd}}{100} = \frac{25}{2 \times 415} \left[1 - \sqrt{1 - 4.598 \times 0.976/25} \right] = 0.284 \times 10^{-2}$$

$< 0.32 \times 10^{-2}$ required for shear

$$\Rightarrow (A_{st})_{reqd} = (0.32 \times 10^{-2}) \times 10^3 \times 637 = 2039 \text{ mm}^2/\text{m}$$

- Using 16 ϕ bars, spacing required $= 201 \times 10^3 / 2039 = 98.6 \text{ mm}$
- Using 20 ϕ bars, spacing required $= 314 \times 10^3 / 2039 = 154 \text{ mm}$
- Provide 20 ϕ bars @ 150 c/c at the bottom of the toe slab.
- The bars should extend by at least a distance $L_d = 47.0 \times 20 = 940 \text{ mm}$ beyond the front face of the stem, on both sides.
- Distribution steel: Provide 10 ϕ bars @ 200 c/c for the transverse reinforcement.

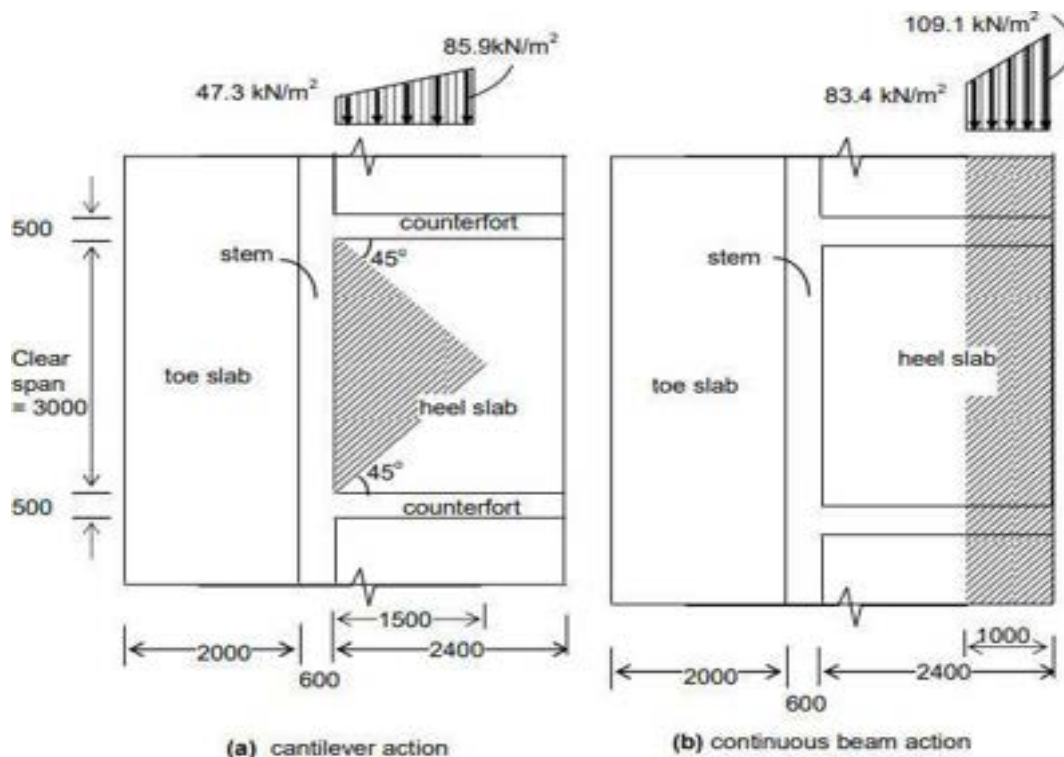
7. Design of heel slab

- The loads (net pressures) considered for the design of the heel slab are as shown in Fig. above.
- The distributed loading acting downward on the heel slab is given by
 - o overburden @ $16 \times (9.0 - 0.5) = 136.0 \text{ kN/m}^2$
 - o heel slab @ $25 \times 0.5 = 12.5 \text{ kN/m}^2 \Rightarrow w = 148.5 \text{ kN/m}^2$
- The net pressure acts downwards, varying between 47.3 kN/m^2 and 109.1 kN/m^2 as shown in Fig.
- The counterforts are provided at a clear spacing of 3.0 m throughout the length of the wall. Thus, each heel slab panel ($2.4 \text{ m} \times 3.0 \text{ m}$) may be considered to be fixed (continuous) at three edges (counterfort locations and junction with stem) and free at the fourth edge. The moment coefficients given in IS 456 do not cater to this set of boundary conditions, and reference needs to be made to other handbooks. Alternatively, we may apply the formulas obtained from yield line theory.
- A common simplified design practice is to assume that some tributary (triangular) portion of the net load acting on the heel slab is transmitted through cantilever action, while much of the load (particularly near the free edge) is transmitted in the perpendicular direction through continuous beam action. The reinforcements in the remaining regions are judiciously apportioned. This procedure is followed here.

Design of heel slab for continuous beam action

Assuming a clear cover of 75 mm and 16ϕ bars, $d = 500 - 75 - 8 = 417 \text{ mm}$. Consider a 1 m wide strip near the free edge of the heel. The intensity of pressure at a distance of 1 m from the free edge is 83.4 kN/m^2 .

Hence, the average loading on the strip may be taken as $(83.4 + 109.1)/2 = 96.25 \text{ kN/m}^2$. Applying a load factor of 1.5 , $w_u = 1.5 \times 96.25 = 144.4 \text{ kN/m}^2$. The effective span is given by $l = 3.0 + 0.417 = 3.417 \text{ m}$.



Max. negative moment occurring in the heel slab at the counterfort location is given by

$$M_{u,-ve} = w_u l^2 / 12 = 144.4 \times 3.417^2 / 12 = 140.5 \text{ kNm/m}$$

Max. mid-span moment may be taken as

$$M_{u,+ve} = w_u l^2 / 16 \approx 0.75 \times M_{u,-ve} = 105.4 \text{ kNm/m}$$

Design shear force

$$V_u = w_u \times (\text{clear span} / 2 - d) = 144.4 \times (3.0/2 - 0.417) = 156.4 \text{ kN/m}$$

Design of top reinforcement (for -ve moments) at the counterforts

- Nominal shear stress $\tau_v = \frac{V_u}{bd} = \frac{156.4 \times 10^3}{10^3 \times 417} = 0.375 \text{ MPa}$

For a $\tau_c = 0.375 \text{ MPa}$ with M 25 concrete [refer Eq. 6.1], the required $p_t = 0.28$.

- $R = \frac{M_u}{bd^2} = \frac{140.5 \times 10^6}{10^3 \times 417^2} = 0.808 \text{ MPa}$

$$\Rightarrow \frac{(p_t)_{reqd}}{100} = \frac{25}{2 \times 415} \left[1 - \sqrt{1 - 4.598 \times 0.808 / 25} \right] = 0.233 \times 10^{-2} < p_t = 0.28$$

required for shear (in the absence of stirrups).

$$\Rightarrow (A_{st})_{reqd} = (0.28 \times 10^{-2}) \times 10^3 \times 417 = 1168 \text{ mm}^2/\text{m} \text{ (required at 1m from the free edge)}$$

- Using 16 ϕ bars, spacing required = $201 \times 103 / 1168 = 172 \text{ mm}$
- Using 12 ϕ bars, spacing required = $113 \times 103 / 1168 = 96 \text{ mm}$
- Minimum reinforcement for temperature and shrinkage:
 - Min. $A_{st} = .012/100 \times 1000 \times 500 = 600 \text{ mm}^2/\text{m} < 1168 \text{ mm}^2/\text{m}$ OK.
- At a distance beyond 1m from the free edge, only minimum reinforcement need be provided:
 - Spacing of 12 ϕ bars required for min. reinf. = $113 \times 103 / 600 = 188 \text{ mm}$
- Provide 12 ϕ bars @ 180 c/c at the top of the heel slab throughout, and introduce additional 12 ϕ bars in between two adjacent bars at the counterforts near the free edge over a distance of approx. 1m;
 - i.e., Provide 5 additional 12 ϕ bars on top, extending 1m from either side of the face of the counterfort.

Design of bottom reinforcement (for +ve moment) at mid-span of heel slab

- $R \approx 0.75 \times 0.808 = 0.606 \text{ MPa}$

$$\Rightarrow \frac{(p_t)_{reqd}}{100} = \frac{25}{2 \times 415} \left[1 - \sqrt{1 - 4.598 \times 0.606 / 25} \right] = 0.173 \times 10^{-2}$$

$$\Rightarrow (A_{st})_{reqd} = (0.173 \times 10^{-2}) \times 10^3 \times 417 = 721 \text{ mm}^2/\text{m} > (A_{st})_{min} = 600 \text{ mm}^2/\text{m}$$

$$\text{Spacing of 12 } \phi \text{ bars required} = 113 \times 103 / 721 = 156 \text{ mm}$$

Provide **12 ϕ bars @ 150 c/c** at the bottom of the heel slab throughout.

Distribution steel:

Provide **10 ϕ bars @ 200 c/c** for the transverse reinforcement.

Design of heel slab for cantilever action

Consider the triangular loading on the heel slab to be carried by cantilever action with fixity at the face of the stem.

- The intensity of load at the face of the stem = 47.3 kN/m².
- The intensity of load at a distance of 1.5m from the face of the stem is 85.9 kN/m².
- Total B.M. due to loading on the triangular portion
$$= \left(\frac{1}{2} \times 3.0 \times 1.5 \right) \times \left[47.3 \times \frac{1.5}{3} + (85.9 - 47.3) \times \frac{1.5}{2 \times 3} \right] = 74.93 \text{ kNm}$$

This moment is distributed non-uniformly across the width of 3.0m. For design purposes, the max. moment intensity (in the middle region) may be taken as two times the average value.

$$\Rightarrow M_{max} = 2 \times (74.93 / 3.0) \\ = 49.95 \text{ kNm/m}$$

$$d = 417 - 12 = 405 \text{ mm}$$

Applying a load factor of 1.5,

$$R = \frac{M_u}{bd^2} = \frac{1.5 \times 49.95 \times 10^6}{1000 \times 405^2} = 0.457 \text{ MPa}$$

$$\Rightarrow \frac{(p_t)_{reqd}}{100} = \frac{25}{2 \times 415} \left[1 - \sqrt{1 - 4.598 \times 0.457 / 25} \right] = 0.130 \times 10^{-2}$$

$$\Rightarrow (A_{st})_{reqd} = (0.130 \times 10^{-2}) \times 10^3 \times 405 = 527 \text{ mm}^2/\text{m} < (A_{st})_{min} = 600 \text{ mm}^2/\text{m} \text{ (for temperature and shrinkage)}$$

Provide **12 ϕ bars @ 180 c/c** at the top of the heel slab throughout.

8. Design of vertical stem

The simplified analysis procedure adopted for the heel slab is used here for the vertical stem also. The cantilever action is limited to the bottom region only (triangular portion) with fixity at the junction of the stem with the base slab. Elsewhere, the stem is treated as a continuous beam spanning between the counterforts. The bending moments reduce along the height of the stem, owing to the reduction in the lateral pressures with increasing height.

Height of stem above base $h = 9.0 - 0.5 = 8.5 \text{ m}$.

Intensity of earth pressure at the base of the stem is

$$p_a = C_a \gamma_e h = (0.333)(16)(8.5) \\ = 45.33 \text{ kN/m}^2 \text{ (linearly varying to zero at the top)}$$

Applying a load factor of 1.5, $w_u = 1.5 \times 45.33 = 68.0 \text{ kN/m}^2$ at base.

Clear spacing between the counterforts = 3.0 m.

- **Design of stem for continuous beam action**

At base

Assuming a clear cover of 50 mm and 20 ϕ bars,

$$d = 600 - 50 - 10 = 540 \text{ mm and effective span, } l = 3.0 + 0.54 = 3.54 \text{ m}$$

Max. -ve moment occurring in the stem at the counterfort location is given by

$$M_{u,-ve} = w_u l^2 / 12 = 68.0 \times 3.54^2 / 12 = 71.0 \text{ kNm/m}$$

Max. mid-span moment may be taken as

$$M_{u,+ve} = w_u l^2 / 16 \approx 0.75 \times M_{u,-ve} = 53.3 \text{ kNm/m}$$

Design shear force

$$V_u = w_u \times (\text{clearspan}/2 - d) = 68.0 \times (3.0/2 - 0.54) = 65.3 \text{ kN/m}$$

Design of (rear face) reinforcement for -ve moments at the counterforts

$$\Rightarrow R = \frac{M_u}{bd^2} = \frac{71.0 \times 10^6}{10^3 \times 540^2} = 0.244 \text{ MPa}$$

$$\Rightarrow \frac{(p_t)_{reqd}}{100} = \frac{25}{2 \times 415} \left[1 - \sqrt{1 - 4.598 \times 0.244 / 25} \right] = 0.068 \times 10^{-2}$$

$$\Rightarrow (A_{st})_{reqd} = (0.068 \times 10^{-2}) \times 10^3 \times 540 = 369 \text{ mm}^2/\text{m}$$

$$\text{Min. } A_{st} = \frac{0.12}{100} (1000)(600) = 720 \text{ mm}^2/\text{m} > 369 \text{ mm}^2/\text{m}$$

Check for shear at base

$$\tau_v = \frac{65.3 \times 10^3}{10^3 \times 540} = 0.121 \text{ MPa} < \tau_c = 0.29 \text{ MPa (for minimum } p_t = 0.15) \text{ — OK}$$

(Evidently, it is possible to reduce the thickness of the stem, for economy).

Design of (front face) reinforcement for +ve moments in the mid-span of stem

The minimum reinforcement requirement will govern the design on both faces, since $M_{u,+ve} < M_{u,-ve}$.

Using 12 ϕ bars, spacing required = $113 \times 1000 / 720 = 156 \text{ mm}$

Provide **12 ϕ bars (horizontal) @ 150 c/c on both faces** of the stem (up to one-third height above base).

At one-third height above base

$d = 500 - 50 - 6 = 444 \text{ mm}$ and effective span $l = 3.444 \text{ m}$

$$M_{u,-ve} = w_u l^2 / 12 = (68.0 \times 2/3) \times (3.444)^2 / 12 = 44.81 \text{ kNm/m}$$

$$\Rightarrow R = \frac{M_u}{bd^2} = \frac{44.81 \times 10^6}{10^3 \times 444^2} = 0.227 \text{ MPa}$$

$$\Rightarrow \frac{(p_t)_{reqd}}{100} = \frac{25}{2 \times 415} \left[1 - \sqrt{1 - 4.598 \times 0.227 / 25} \right] = 0.064 \times 10^{-2}$$

$$\Rightarrow (A_{st})_{reqd} = (0.064 \times 10^{-2}) \times 10^3 \times 444 = 282 \text{ mm}^2/\text{m}$$

$$\text{Min. } A_{st} = \frac{0.12}{100} (1000)(500) = 600 \text{ mm}^2/\text{m} > 282 \text{ mm}^2/\text{m}$$

Using 12 ϕ bars, spacing required = $113 \times 1000 / 600 = 188 \text{ mm}$

Provide **12 ϕ bars (horizontal) @ 180 c/c on both faces** of the stem (in the middle one-third height).

At two-thirds height above base

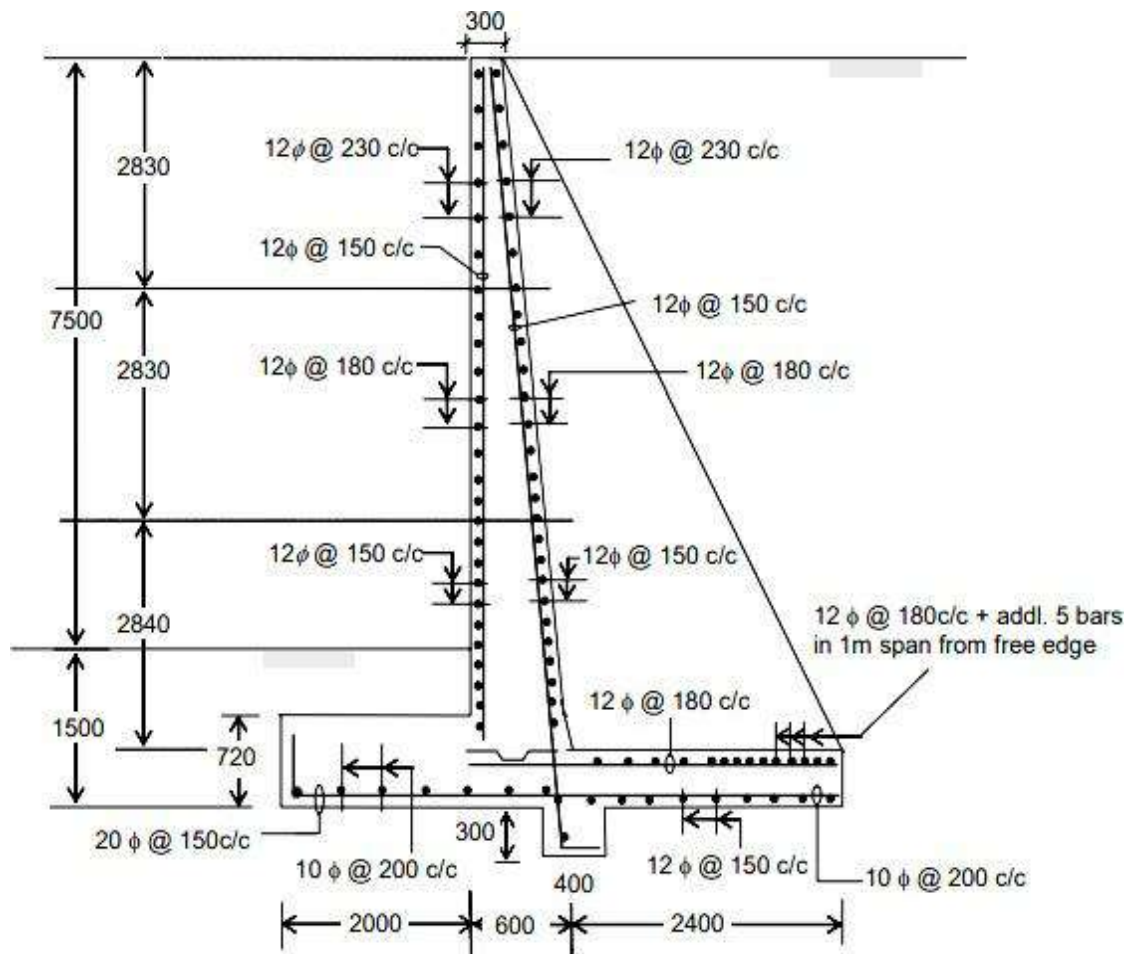
$$\text{Min. } A_{st} = \frac{0.12}{100} (1000)(400) = 480 \text{ mm}^2/\text{m}$$

Using 10 ϕ bars, spacing required = $78.5 \times 1000 / 480 = 163 \text{ mm}$

Using 12 ϕ bars, spacing required = $113 \times 1000 / 480 = 235 \text{ mm}$

Provide **12 ϕ bars (horizontal) @ 230 c/c on both faces** of the stem (in the upper one-third height).

Reinforcement Detailing



Result

Thus, Counterfort Retaining Wall is designed and reinforcement Diagram is draw as per IS Codal provisions.

Ex.No : **Design and Detailing of R.C. Footing**

Date :

Aim: To design a R.C footing (Types will vary) and draw the detailing diagram.

Design Procedure

(Note: Use of IS 456:2000 and SP 16 is Permitted)

Step 1 Find the Ultimate stresses of concrete and steel

Step 2 Compute the Loads acting on the footing by finding

- i) Total load from column
- ii) Self-weight of the footing
- iii) Total service load (sum of the i and ii)

Step 3 Design the Footing based on:

- i) Area of footing
- ii) Factored Bending moment
- iii) Soil pressure
- iv) Factored Shear force
- v) Design the Section based on the Design moments and Shear Forces

Step 4 Check for Shear (within permissible limit or not)

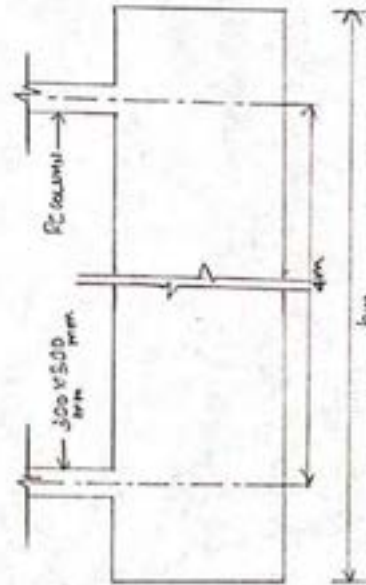
Step 5 Design of Strap beam based on:

- i) Factored load on beam
- ii) Factored Bending Moment and Shear Force acting on the beam
- iii) Live Load Bending moment and Shear Force

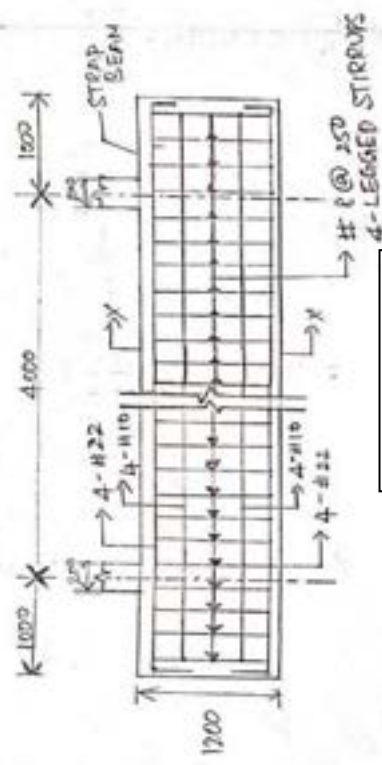
Note: Using the above steps the design problem is done and necessary reinforcement diagram is draw in A3 Sheet

Reinforcement Detailing *(Left Side of the Page)*

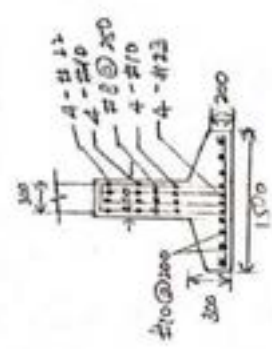
Result: Thus, R.C Footing is designed and reinforcement Diagram is draw as per IS Codal provisions.



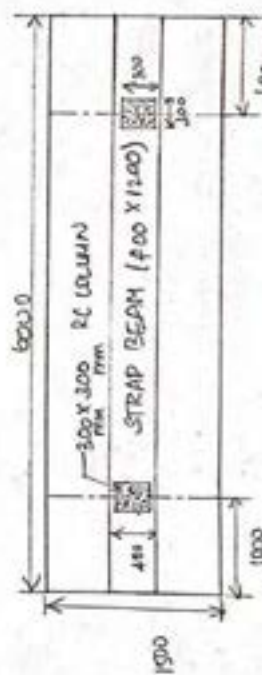
Combined Footing
(Symmetrically Loaded Columns)



Longitudinal Section



Section X-X



Plan
Reinforcement Details in Combined Footing
(Symmetrically Loaded Columns)

Ex.No : **Design and Detailing of Deep Beam**
Date :

Aim: To design a R.C Deep Beam and draw the detailing diagram

Design Procedure

(Note: Use of IS 456:2000 and SP 16 is Permitted)

Step 1 Design parameters based on:

- i) Effective span
- ii) Overall depth
- iii) Width of support
- iv) Total load on beam including self-weight
- v) Support to span ratio
- vi) Area of steel
- vii) Mid to span ratio
- viii) Maximum moment
- ix) Plastic moment

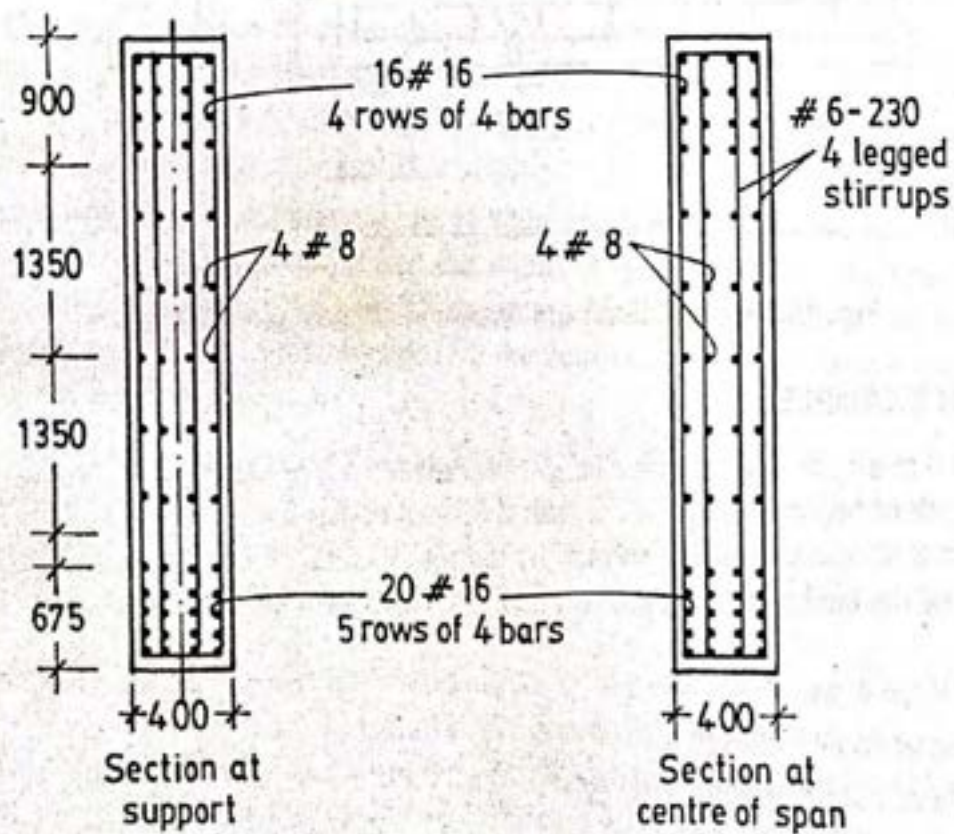
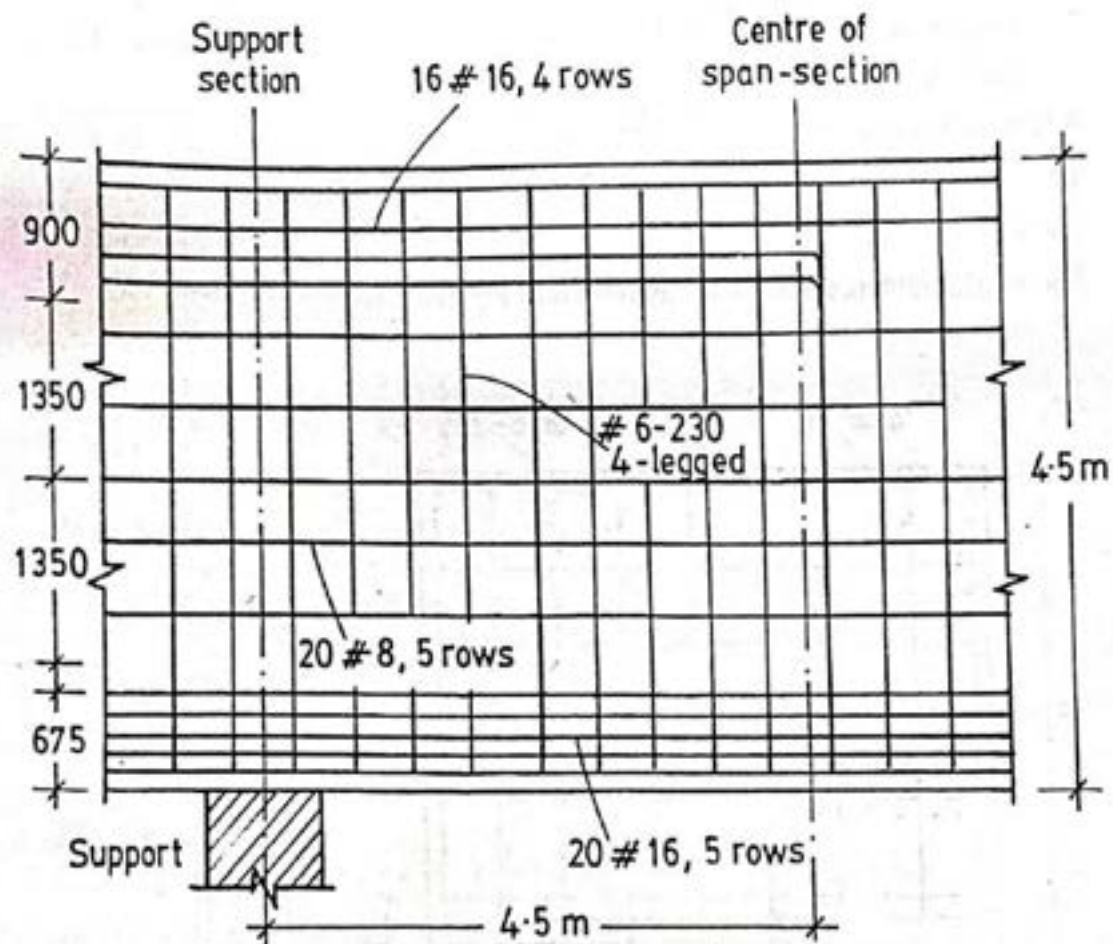
Step 2 Compute reinforcements based on:

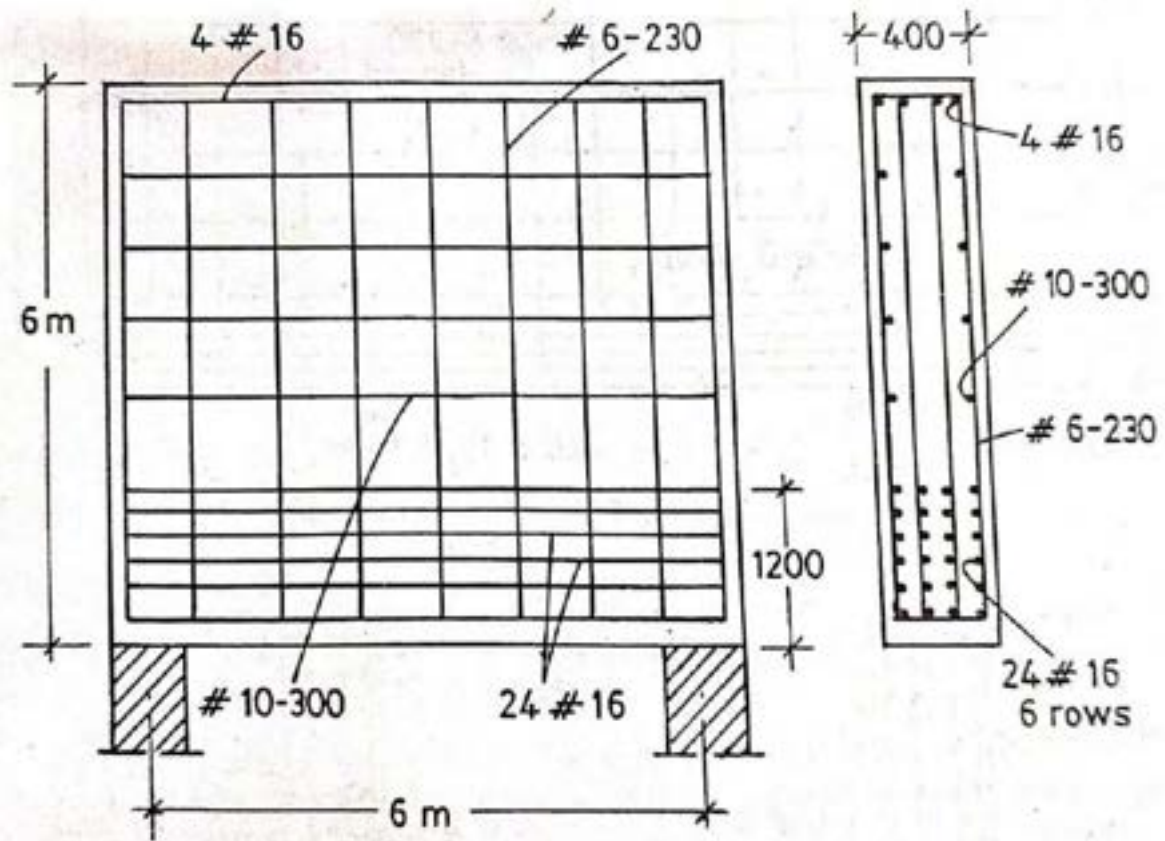
- i) Tension in reinforcement
- ii) Minimum horizontal reinforcement
- iii) Vertical reinforcement

Note: Using the above steps the design problem is done and necessary reinforcement diagram is draw in A3 Sheet

Reinforcement Detailing *(Left Side of the Page)*

Result: Thus, R.C Deep beam is designed and reinforcement Diagram is draw as per IS Codal provisions.





Design and Detailing of Poles

Date :

Aim: To design a R.C Poles and draw the detailing diagram

Design Procedure

(Note: Use of IS 456:2000 and SP 16 is Permitted)

Step 1 Calculation of Wind load

Step 2 Determination of Bending moments based on:

- i) Total transverse load
- ii) Ultimate load

Step 3 Design of reinforcements based on:

- i) Effective depth at ground level
- ii) Total service load
- iii) Ultimate moment

Step 4 Calculation of torsional moments based on:

- i) Bending moment at base

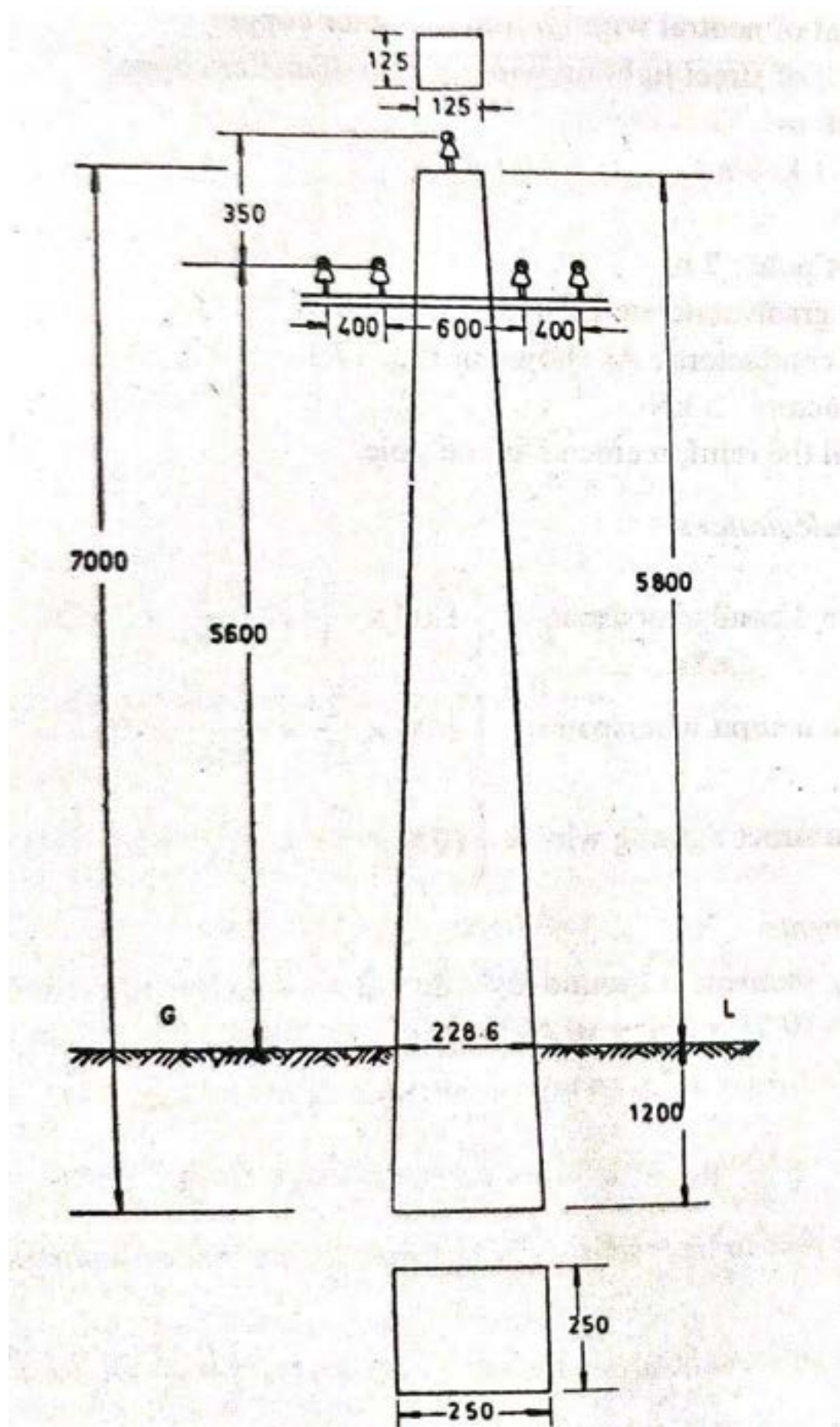
Step 5 Determination of transverse reinforcements based on:

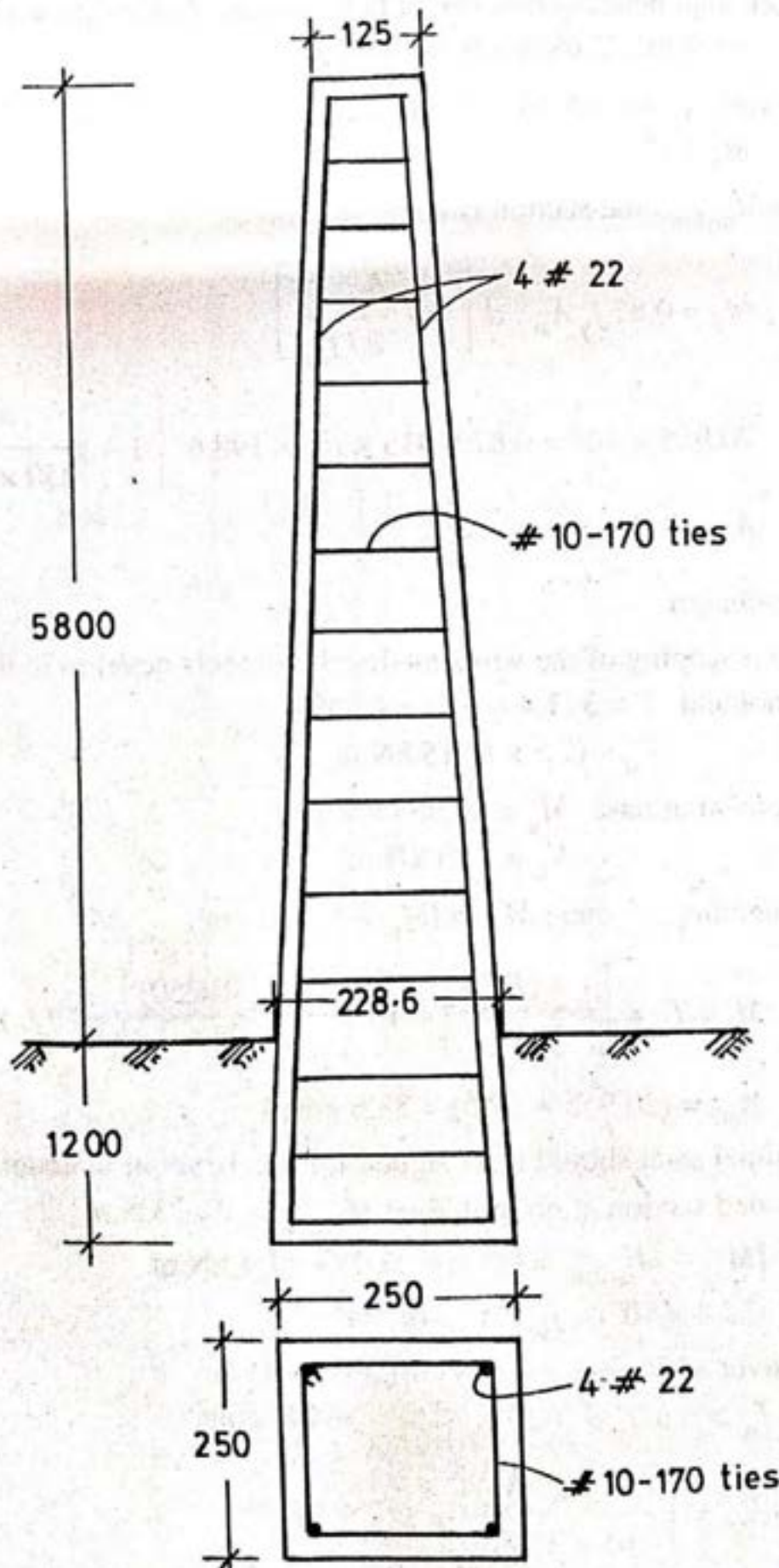
- i) Balance shear
- ii) Spacing

Note: Using the above steps the design problem is done and necessary reinforcement diagram is draw in A3 Sheet

Reinforcement Detailing (*Left Side of the Page*)

Result: Thus, R.C Poles is designed and reinforcement Diagram is draw as per IS Codal provisions.





Ex.No :

Design and Detailing of Stanchion

Date :

Aim: To design a Stanchion and draw the detailing diagram

Design Procedure

(Note: Use of IS 456:2000 and SP 16 is Permitted)

Step 1 Assume the Column Section

Step 2 Find the Safe load on the Column

Step 3 Determine the spacing of I-Sections

Step 4 Design the Single lacing between the I-Sections based on:

- i) Slenderness ratio of lacing bar
- ii) Angle of inclination
- iii) Transverse Shear
- iv) Force acting on lacing bar
- v) Safe allowable load
- vi) Rivet Value (Single shear and bearing)

Step 5 Design the Column base based on:

- i) Projection of the base plate from the edge of the column (both greater and smaller projection)
- ii) Pressure on base plate
- iii) Permissible bearing stress in base plate

Step 6 Assume appropriate cleat angle for connecting base plate and column

Step 7 Design the concrete bed block based on:

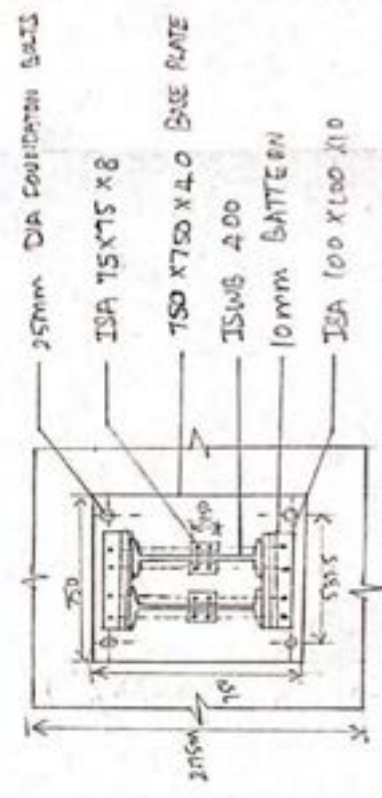
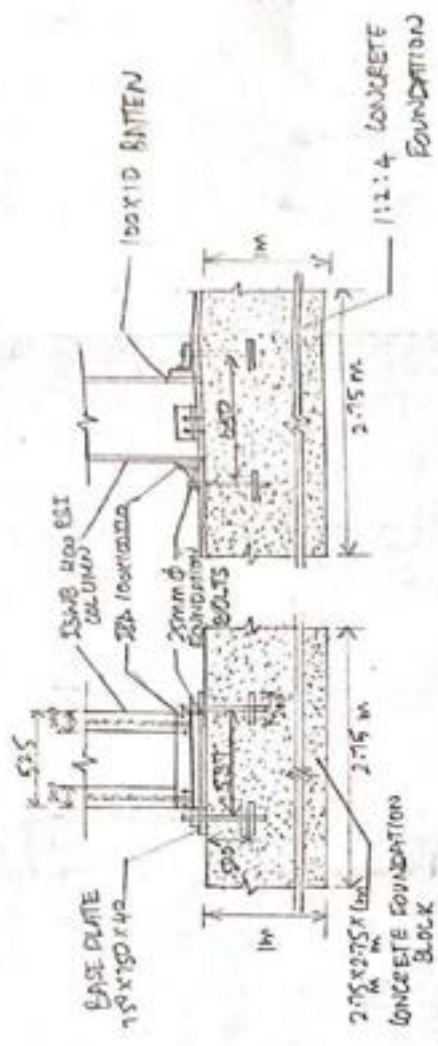
- i) Axial load on column

- ii) Self-weight of foundation
- iii) Angle of load dispersion (assume)
- iv) Force acting on lacing bar

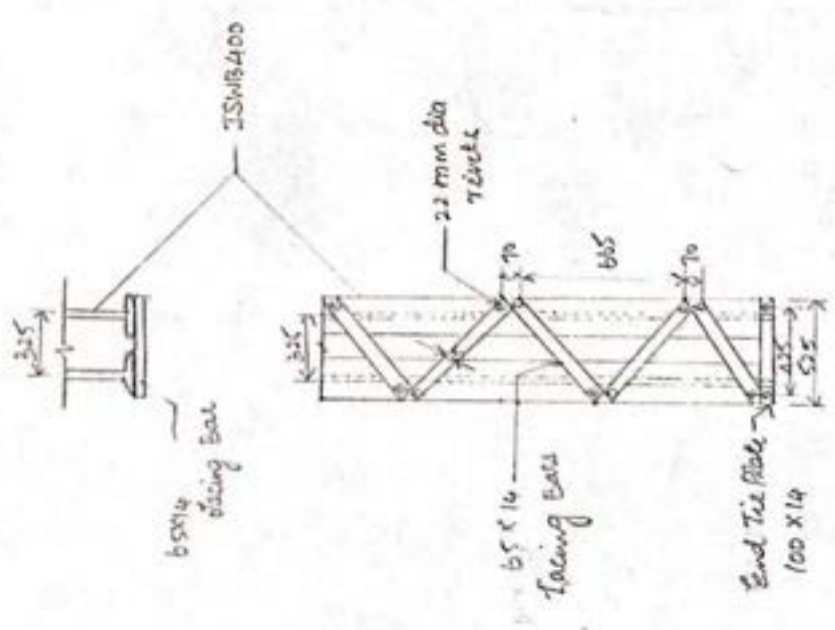
Note: Using the above steps the design problem is done and necessary reinforcement diagram is draw in A3 Sheet

Reinforcement Detailing *(Left Side of the Page)*

Result: Thus, Stanchion is designed and reinforcement Diagram is draw as per IS Codal provisions.



Built-Up Steel Column



Elevation and Plan of Single Laced Column

Ex.No :

Design and Detailing of Tee-Beam Bridge

Date :

Aim: To design a R.C Tee-Beam Bridge and draw the detailing diagram

Design Procedure

(Note: Use of IS 456:2000 and SP 16 is Permitted)

Step 1 Find the permissible stresses

Step 2 Determine the Cross Section of the deck

Step 3 Design the interior Slab Panels based on:

- i) Bending moments in the Slab
- ii) Shear forces acting on the slab
- iii) Dead load Shear Force and Bending moments
- iv) Use the above three to compute Design moments and Shear Forces
- v) Design the Section based on the Design moments and Shear Forces

Step 4 Check for Shear (within permissible limit or not)

Step 5 Design the Longitudinal Girders based on:

- i) Reaction factor for outer and inner girder
- ii) Dead Load from slab per girder
- iii) Live Load Bending moment and Shear Force
- iv) Dead load Bending moment and Shear Force
- v) Use the above two to compute Design moments and Shear Forces
- vi) Design the Section based on the Design moments and Shear Forces

Step 6 Design the Cross girders

Note: Using the above steps the design problem is done and necessary reinforcement diagram is draw in A3 Sheet

Reinforcement Detailing *(Left Side of the Page)*

Result: Thus, R.C Tee-Beam Bridge is designed and reinforcement Diagram is draw as per IS Codal provisions.



CROSS SECTION OF T-BEAM AND SLAB DECE

Ex.No :

Design and Detailing of Steel Roof Truss

Date :

Aim: To design a Steel Roof Truss and draw the detailing diagram

Design Procedure

(Note: Use of IS 456:2000 and SP 16 is Permitted)

Step 1 Derive the dimension of truss using the relation between central rise and span of truss

Step 2 Compute Loads acting on the Purlin

- i) Dead Load
- ii) Live Load
- iii) Wind Load

Step 3 Find the combinational loads acting on the Purlin

- i) Dead Load + Live Load
- ii) Dead Load + Wind Load

Step 4 Design the Purlin based on:

- i) Bending moment
- ii) Relation between width and length of purlin
- iii) Relation between depth and length of purlin

Step 5 Compute Loads acting on the Truss

- i) Dead Load
- ii) Live Load
- iii) Wind Load

Step 6 Design the Truss based on forces acting on each member of the Truss

Step 7 Design the Rivet joints based on forces acting on each member connected by the joints

Step 8 Design the Welded joints based on forces acting on each member connected by the joints

Step 9 Design the Shoe angle based on:

i) Reaction at support

Relation between reaction at support number of rivets

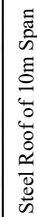
ii) required to connect the Shoe angle

Step 10 Design the Bearing plate

Note: Using the above steps the design problem is done and necessary reinforcement diagram is draw in A3 Sheet

Reinforcement Detailing *(Left Side of the Page)*

Result: Thus, Steel Roof Truss is designed and reinforcement Diagram is draw as per IS Codal provisions.



ALL DIMENSIONS ARE IN"MM"

Ex.No :

Design and Detailing of Plate Girder

Date :

Aim: To design a Plate Girder and draw the detailing diagram

Design Procedure

(Note: Use of IS 456:2000 and SP 16 is Permitted)

Step 1 Compute loads acting on the Girder

- i) Dead Load
- ii) Live Load

Step 2 Compute Bending and Shear Force

- i) For UDL effect
- ii) For Concentrated Load effect
- iii) Combined effect

Step 3 Assume the size of Plate Girder based on

- i) Length to depth ratio
- ii) Flange dimension
- iii) Web dimension

Step 4 Classify the sections (Flange and Web)

Step 5 Check the Serviceability of the girder

Step 6 Design the Web of girder based on:

- i) Critical Shear strength
- ii) Spacing of stiffeners
- iii) Slenderness parameters
- iv) Basic shear strength

Step 7 Check for web panel

Step 8 Design the Load Bearing Stiffener based on:

i) Bearing Load on Stiffener

ii) Load due to moment on Stiffener

Step 9 Check for Bearing of Load Bearing Stiffener

Step 10 Check for Outstand of Load Bearing Stiffener

Step 11 Check for Buckling of Load Bearing Stiffener

Step 12 Design the Intermediate Stiffener

Step 13 Check for Outstand of Intermediate Stiffener

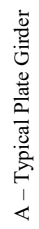
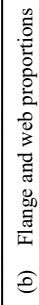
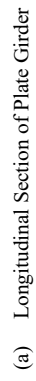
Step 14 Check for Buckling of Intermediate Stiffener

Step 15 Web Check between Stiffeners

Note: Using the above steps the design problem is done and necessary reinforcement diagram is draw in A3 Sheet

Reinforcement Detailing *(Left Side of the Page)*

Result: Thus, Plate Girder is designed and reinforcement Diagram is draw as per IS Codal provisions.



Ex.No :

Design and Detailing of Pipe Culvert

Date :

Aim: To design a Pipe Culvert and draw the detailing diagram

Design Procedure

(Note: Use of IS 456:2000 and SP 16 is Permitted)

- | | |
|--------|--|
| Step 1 | Compute the diameter of pipe culvert |
| Step 2 | Determine the load due to earth fill |
| Step 3 | To compute the load on pipe due to I.R.C. Class AA wheel loads |
| Step 4 | Check for safety factor |
| Step 5 | Determine reinforcements based on: |
| | i) Spiral reinforcement of hard drawn steel wire |
| | ii) Longitudinal reinforcement of mild steel |
| | iii) Weight of spiral reinforcement /meter length of pipe |
| | iv) Number of bars required |
| | v) Spacing |

Note: Using the above steps the design problem is done and necessary reinforcement diagram is draw in A3 Sheet

Reinforcement Detailing *(Left Side of the Page)*

Result: Thus, Pipe Culvert is designed and reinforcement Diagram is draw as per IS Codal provisions.

