Techno India NJR Institute of Technology

Course File

WATER AND EARTH RETAINING STRUCTURES DESIGN
(6CE4-24)

For Techno India NJR Institute of Technology
पंकज पोरवाल
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(Principal)

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RAJASTHAN TECHNICAL UNIVERSITY, KOTA

Syllabus

3rd Year - VI Semester: B.Tech. (Civil Engineering)

6CE4-24: WATER AND EARTH RETAINING STRUCTURES DESIGN

Credit: 1 | **Max. Marks: 50(IA:30, ETE:20)**

0L+0T+2P | **End Term Exam: 2 Hours**

Assignments/ Exercises on the following topics:

SN	CONTENTS	Hours
1	**Continuous Beams:** Analysis andDesign of continuous beams using coefficients (IS Code), concept of moment redistribution	4
2	**Curved Beams:** Analysis and design of beams curved in plan.	4
3	**Circular Domes:** Analysisand design of Circular domes with u.d.l. & concentrated load at crown.	4
4	**Water Tanks and Towers:** Water Tanks and Water Towers-design of rectangular, circular and Intze type tanks, column brace type staging.	10
5	**Retaining walls:** Analysis and design of Cantilever Retaining Walls: Introduction to counterfort and buttress type retaining walls, their structural behaviour and stability analysis.	6
	TOTAL	28

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Course Overview :

Retaining structures are walls, dams, barriers, or bins that hold Earth materials or water in place or keep Earth materials or water from encroaching into an area. Retaining structures also are used to create stable surfaces for building pads, roads, bridge abutments, or wharves. Retaining structures can be used to limit the volume of excavations or to allow utilization of space near the boundary of a particular piece of land. Other structures that appear to be earth-retaining structures may have erosion protection as their primary purpose.

Retaining structures commonly are engineered features that are designed and constructed to hold soil or water in place. Structures that retain water are called dams, levees, or flood walls; structures that retain Earth are called earth-retaining structures or retaining walls

Course Outcomes:

CO.NO.	Cognitive Level	Course Outcome
1	**Comprehension**	Analyze the concepts of pre stressing in the design of beams.
2	**Application**	Design the torsion, continuous and curve beam
3	**Analysis**	Design of circular domes and water tanks
4	**Synthesis**	Analyze Yield line theory and design retaining wall
5	**Evaluation**	Design the culvert and bridge.

Prerequisites:

1. Fundamentals knowledge of Continuous Beam .
2. Fundamentals knowledge of Retaining wall.
3. Fundamentals knowledge of Circular Dome.

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Course Outcome Mapping with Program Outcome:

	PO1	PO2	PO3	PO4	PO5	PO6	PO7	PO8	PO9	PO10	PO11	PO12	PSO1	PSO2	PSO3
Course Outcome	3	3	3	3	2	2	2	1	1	1	2	3	2	1	1
	3	2	2	3	2	1	2	1	1	1	1	1	2	1	1
	3	2	2	2	2	1	1	1	2	1	1	2	2	2	1
	3	2	2	3	2	1	2	1	1	1	1	1	2	1	1
	3	2	2	2	2	1	1	1	2	1	1	2	2	1	1
CO355 (AVG)	3	2.2	2.2	2.6	2	1.2	1.6	1	1.4	1	1.2	1.8	2	1.2	1

Course Coverage Module Wise:

Lab No.	Experiments List According to RTU Syllabus
1	Continuous Beams: Analysis and Design of continuous beams using coefficients (IS Code).
2	Continuous Beams: Analysis and Design of continuous beams using coefficients (IS Code).
3	Continuous Beams: concept of moment redistribution..
4	Curved Beams: Analysis and design of beams curved in plan.
5	Circular Domes: Analysis and design of Circular domes with u.d.l. & concentrated load at crown.
6	Circular Domes: Analysis and design of Circular domes with u.d.l. & concentrated load at crown.
7	Water Tanks and Towers: Water Tanks and Water Towers-design of rectangular type tanks.
8	Water Tanks and Towers: Water Tanks and Water Towers-design of circular and Intze type tanks.
9	Water Tanks and Towers: Water Tanks and Water Towers-design of column brace type staging.
10	Retaining walls: Analysis and design of Cantilever Retaining Walls: Introduction to counter fort and buttress type retaining walls, their structural behaviour and stability analysis.
11	Retaining walls: Analysis and design of Cantilever Retaining Walls: Introduction to counter fort and buttress type retaining walls, their structural behaviour and stability analysis.

Faculty Lab Manual Link
https://drive.google.com/file/d/1Lkyw56KI9b6v1WucFMfwNbUHIuTh2quj/view?usp=sharing

Viva QUIZ Link

1. https://quizizz.com/admin/quiz/5d90cfc5ba34c6001ab29688/retaining-wall
2. https://www.brainkart.com/article/Important-Question-And-Answer--Civil---Retaining-Walls_3862/
3. https://www.objectivebooks.com/2017/03/rcc-structures-design-mcq-questions-set.html
4. https://engineeringinterviewquestions.com/mcqs-on-foundation-answers/

Assessment Methodology:

1. Practical exam on retaining structures.
2. Internal exams and Viva Conduct.
3. Final Exam (practical paper) at the end of the semester.

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EXPERIMENT NO. 4

Water Tanks and Towers:

Water Tanks and Water Towers-design of Rectangular, circular and Intze type tanks, column brace type staging

2.1. INTRODUCTION

Tanks are widely used for storing liquids like water, chemicals and petroleum etc. The tanks are nerally circular or rectangular in shape. They are broadly categorized into following three types:

1. Tanks resting on ground
2. Underground tanks
3. Elevated or overhead tanks.

The tanks resting on ground are supported on the ground directly. The sedimentation tanks, aeration anks, filteration tanks and clear water storage reservoirs are generally of this type while the septic ank, imhoff tank and simple water tanks collecting water from the mains are generally constructed as nderground tanks. Elevated or overhead water tanks, supported on staging, are commonly used in water distribution system. For constructing any type of liquid retaining structure, it is a must to ensure hat the concrete is dense and impervious. It is essential not only from the leakage point of view, but also affects the durability, cracking and resistance against chemical attack and corrosion.

The Indian Standard Code of practicle for design of liquid retaining concrete structures *i.e.*, **IS:3370** was first published in 1965. Presently, it is available in four parts as follows:

1. **IS 3370:2009 (Part 1): Code of Practice for Concrete Structures for Storage of Liquids:** General requirements.

2. **IS 3370:2009 (Part-2):** Code of Practice for Concrete Structures for Storage of Liquids: Reinforced Concrete Structures.
3. **IS 3370:1967 (Part-3):** Code of Practice for Concrete Structures for Storage of Liquids: Prestressed Concrete Structures.
4. **IS 3370:1967 (Part-4):** Design Tables for Design of Reinforced or Prestressed Concrete Structures for Storage of Liquids.

22.2. DESIGN PHILOSOPHY AND REQUIREMENTS

Design of liquid retaining structures is based upon the fact that the concrete should not crack and hence the tensile strength of concrete should be within permissible limits. In order to control cracking, various requirements regarding material, joints and reinforcement detailing are listed in IS 3370 (Part I): 2009, some of which are explained below:

1. Concrete mixes lower than M30 are not to be used for design of liquid retaining structures. The use of richer mixes results in less cracking.
2. The structure retaining the liquids should be designed as "subjected to Severe Exposure Conditions".
3. The cement content, not including fly ash and ground granulated blast furnace slag, should not exceed 400 kg/m^3 unless special consideration is taken for increased risk of cracking due to drying shrinkage etc.
4. Cracking can be controlled by using the plasticizers and by using minimum amount of cement content which will result in reduced water content per unit of concrete mix. The minimum cement content from durability criteria is 320 kg/m^3.
5. Cracking can also be controlled by reducing the steep changes in temperature and moisture content at early age of concrete. Curing should be done at least for a period 14 days.
6. Correct placing of reinforcement bars, use of deformed bars, bars closely spaced and use of small sized bars will also result in reduced cracking.
7. Crack width for reinforced concrete members in direct tension and flexural tension is considered satisfactory, if steel stress under service conditions does not exceed 130 N/mm^2 for high strength deformed bars.
8. The maximum calculated surface width of cracks for direct tension and flexure should not be more than 0.2 mm with specified cover.

22.3. METHODS OF DESIGN

The design of water tanks can be done by any of two methods given below:

(i) Limit state method of design.

(ii) Working stress method of design.

(i) Limit State Method of Design

In this method, all relevant limit states should be considered and satisfied with an adequate degree of safety and serviceability. The limit state of collapse and limit state of serviceability (Deflection and Cracking) should be followed as per IS 456:2000.

(ii) Working Stress Method of Design

The working stress method for design of water tanks is based on adequate resistance to cracking and strength.

The various assumptions of this method are as follows. (Refer Chapter 2)

(a) Plane sections remain plane before and after bending.

(b) Steel and concrete behave elastically and the modular ratio, m is given by:

$$m = \frac{280}{3\sigma_{cbc}}$$

(c) The tensile stress in concrete is limited to the values given in Table 22.1 for calculation of resistance to cracking.

(d) The tensile strength of concrete is ignored for all strength calculations.

Permissible Stresses in Concrete:

(1) Resistance to Cracking: The permissible tensile stress in concrete related to resistance to cracking is given in Table 1 of IS 3370 or Table 22.1 here.

TABLE 22.1. Permissible Concrete Stresses in Calculations Related to Resistance to Cracking

S. No.	Grade of Concrete	Direct tension (N/mm^2)	Flexural tension (N/mm^2)
1.	M25	1.3	1.8
2.	M30	1.5	2.0
2.	M35	1.6	2.2
3.	M40	1.8	2.4
4.	M45	2.0	2.6
5.	M50	2.1	2.8

The permissible shear stress values for concrete are given in Table 23 of IS 456:2000. These values can be exceeded provided the shear reinforcement is designed taking into account these exceeded values.

(2) For Strength Calculations: The permissible stresses in concrete for calculation of strength are given in Table 2 of IS 3370 or Table 22.2 here.

TABLE 22.2. Permissible Stresses in Concrete

S. No.	Grade of Concrete	Compressive Stress (N/mm^2) Bending (σ_{cbc})	Compressive Stress (N/mm^2) Direct (σ_{cc})	Average Bond Stress for Plain bars in tension (N/mm^2) τ_{bd}
1.	M25	8.5	6.0	0.9
2.	M30	10.0	8.0	1.0
2.	M35	11.5	9.0	1.1
3.	M40	[illegible]	10.0	1.2
4.	M45	14.5	11.0	1.3
5.	M50	[illegible]	12.0	1.4

Note:

1. **Bond stress in compression should be increased by 25%.**
2. **For deformed bars, the bond stress should be increased by 60%.**

Permissible Stresses in Steel

1. Resistance to Cracking: The permissible stresses in steel is limited by the fact that the permissible tensile stresses for resistance to cracking in concrete are not exceeded. In order to have the perfect bond between steel and concrete, the permissible stress in steel can be written as:

$$\sigma_{st} = m\sigma_{ct}$$

where σ_{st} = permissible tensile stress in steel

σ_{ct} = permissible tensile stress in concrete

m = modular ratio of steel and concrete.

2. Permissible Stresses for Strength Calculations: For the purpose of strength calculations in liquid retaining structures, the permissible stresses should be as listed in Table 4 of IS 3370 (Part 2) or Table 22.3.

TABLE 22.3. Permissible Stresses in Steel for Strength Calculations

S. No.	Type of Stress in Steel Reinforcement	Permissible Stresses (N/mm²)	
		Plain mild steel bars	High yield strength deformed bars
1.	Tensile stress in members under direct tension, bending and shear	115	130
2.	Compressive stress in columns subjected to direct load	125	140

22.4. IS CODE RECOMMENDATIONS REGARDING DETAILING IN WATER TANKS

1. The minimum reinforcement in walls, floors and roofs in each of two directions at right angles within each surface zone (Fig. 22.1 and 22.2) should not be less than 0.35% of the cross-section of surface zone for HYSD bars and 0.64% for mild steel bars.

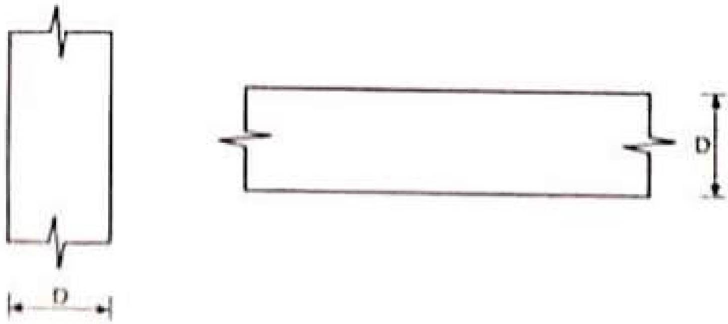

Fig. 22.1. Surfaces of walls and suspended slabs.

2. The minimum reinforcement can be further reduced to 0.24% for deformed bars and 0.40% for plain bars, for tanks, not having any dimension more than 15 m.
3. In tank walls and slabs, having thickness less than 200 mm, the reinforcement can be placed in one face only.

4. For ground/base slab, having thickness less than 300 mm, the reinforcement should be placed on one face, as near as possible to the upper surface consistent with the cover.
5. The spacing of reinforcing bars should not exceed 300 mm or thickness of the section, whichever is less.
6. Size of bars, distance between bars, laps and bends should be as per IS 456:2000.

Note:

1. For $D \geq 500$ mm *i.e.*, thickness of the member greater than or equal to 500 mm, each reinforcement face controls half of the total depth ($D/2$) of concrete.
2. For $D < 500$ *i.e.*, thickness of the member less than 500 mm, each reinforcement face controls 250 mm depth of concrete, ignoring any central core beyond the surface depth.

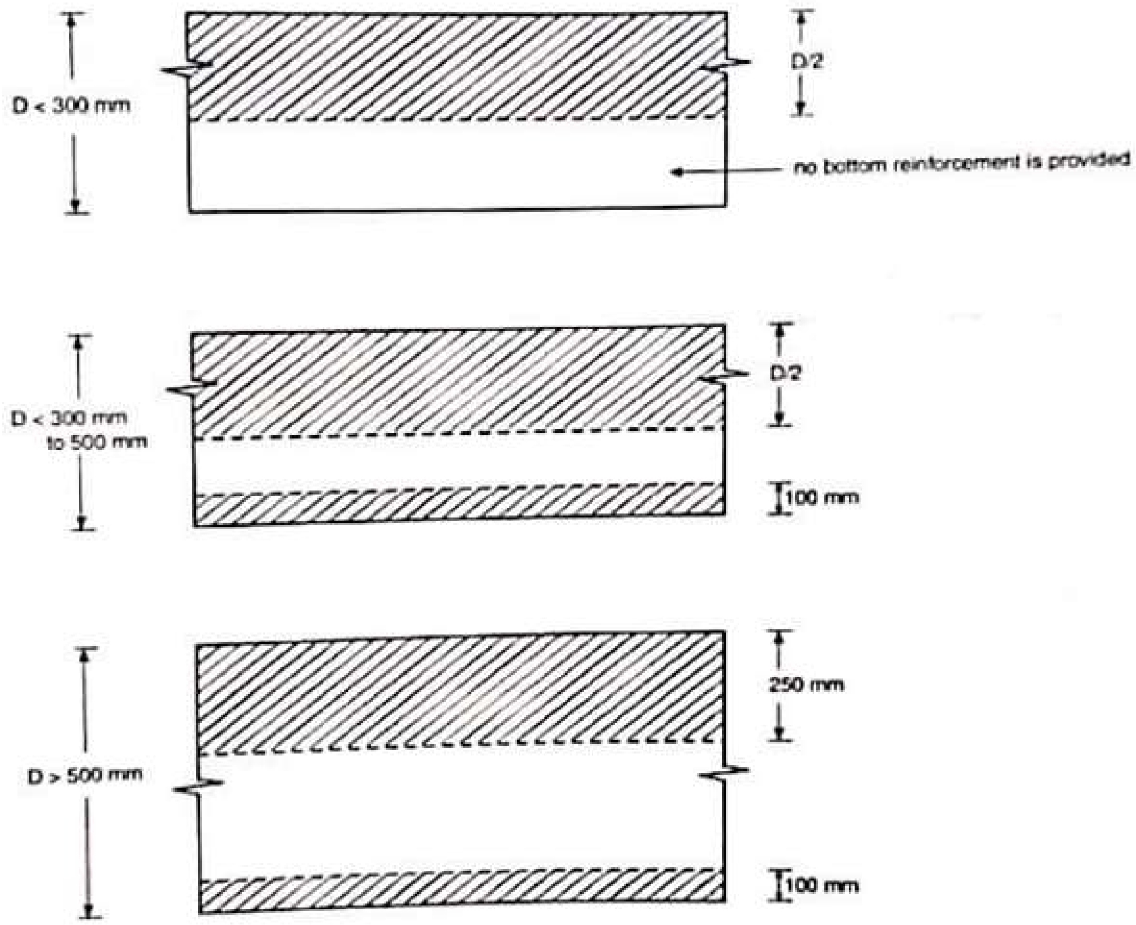

Fig. 22.2. Surface zones in ground/base slabs.

22.5. JOINTS IN WATER TANKS

The joins provided in water tanks are classified, as per the IS 3370 (Part 1): 2009, as given below:

(a) Movement Joints

In this type of joint, relative movement between the adjoining parts of a tank, such as wall and the floor slab, is permitted. These joints require use of special materials, in order to maintain water tightness thus accommodating the relative movement between the sides of the joint. There are three types of movement joints which are explained below:

(1) Contraction Joints

It is a movement joint, with deliberate discontinuity, without initial gap between the concrete on either side of the joint. The joint is designed to accommodate contraction of the concrete as shown in Fig. 22.3:

A contraction joint may be designed as complete or partial. In a complete contraction joint both concrete and steel are interrupted (discontinued) while in partial contraction joint only concrete is interrupted and the steel reinforcement is continuous. A complete contraction joint is not restrained to movement and is intended to accommodate only contraction of concrete while the partial contraction joint provide some restraint to movement in addition to accommodating some contraction of concrete. These joints are shown in Fig. 22.3(*a*) and (*b*).

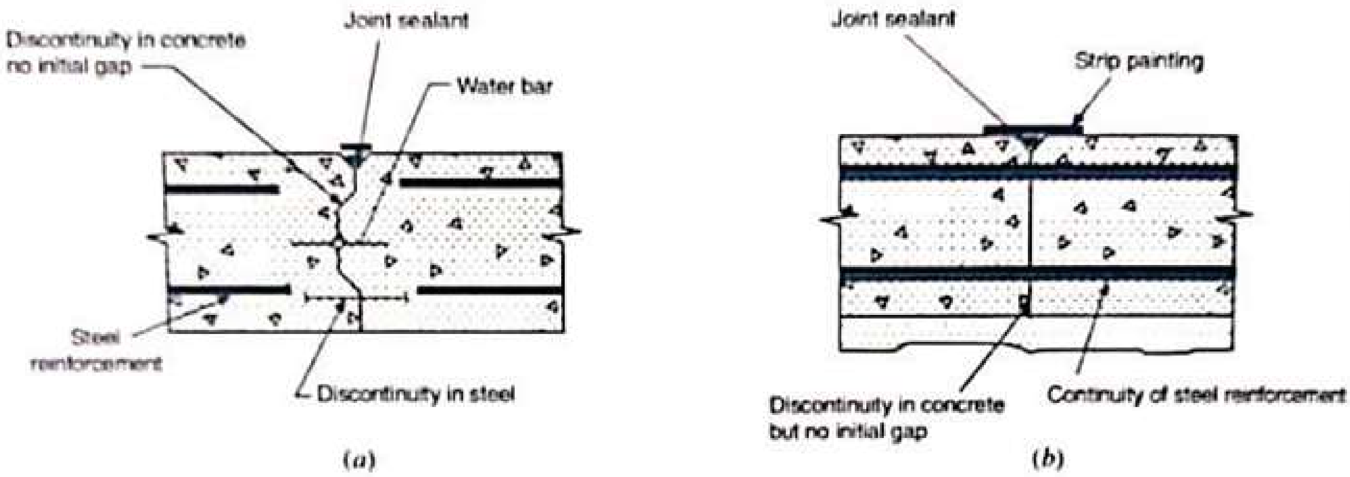

Fig. 22.3. Contraction joints.

(2) Expansion Joint

In this type of movement joint, complete discontinuity in both steel and concrete is provided to accommodate either expansion or contraction of the concrete. This joint has no restraint to movement. This type of joint requires an initial gap between the adjoining parts of a structure to accommodate expansion/contraction of the concrete as shown in Fig. 22.4.

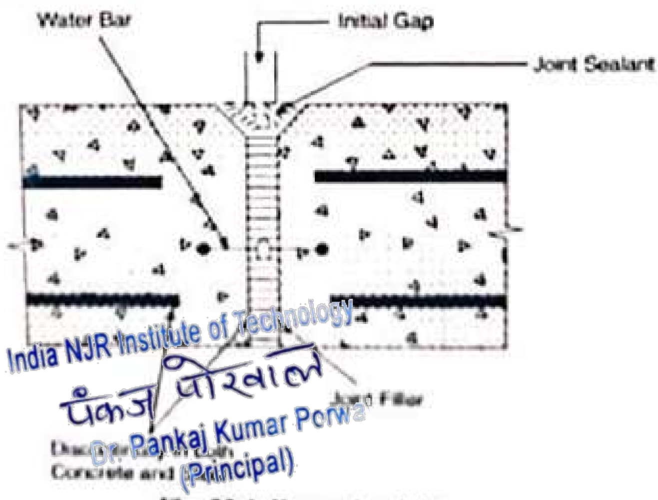

Fig. 22.4. Expansion joint

(3) Sliding Joint

A movement joint which allows the adjoining parts of a structure to slide relative to each other with minimum restraint is known as sliding joint. In this joint, complete discontinuity is provided in both steel and concrete and at the discontinuity special provision is made to facilitate the relative sliding movement. This type of joint is generally provided between the wall and the floor of the cylindrical tank and is shown in Fig. 22.5.

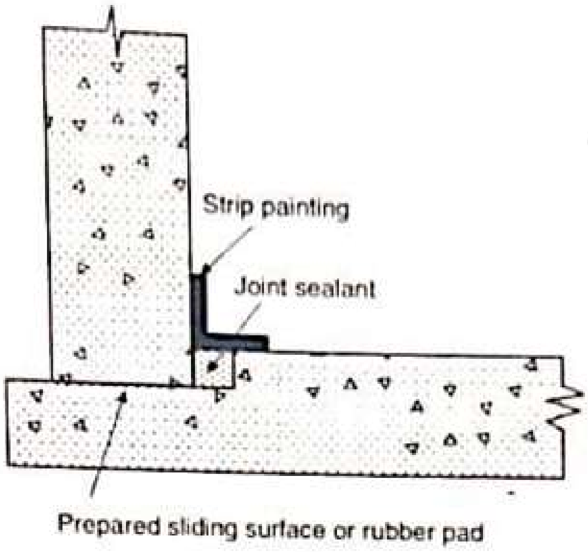

Fig. 22.5. Sliding joint

(b) Construction Joints

These joints are provided for convenience in construction. At these joints, special measures are incorporated to have subsequent continuity, without provision for further relative movement. These joints may be grouted and concrete at the joints should be bonded properly. The number of such joints should be kept as small as possible.

(c) Temporary Open Joints

A gap is sometimes left temporarily between the concrete of adjoining parts of a water tank which is filled with mortar or with suitable jointing material, after a suitable interval of time, before the structure is put to use. This type of joint is shown in Fig. 22.6(*a*) and (*b*). The width of the initial gap provided in the joint should be sufficient enough to allow the sides to be prepared before filling jointing material.

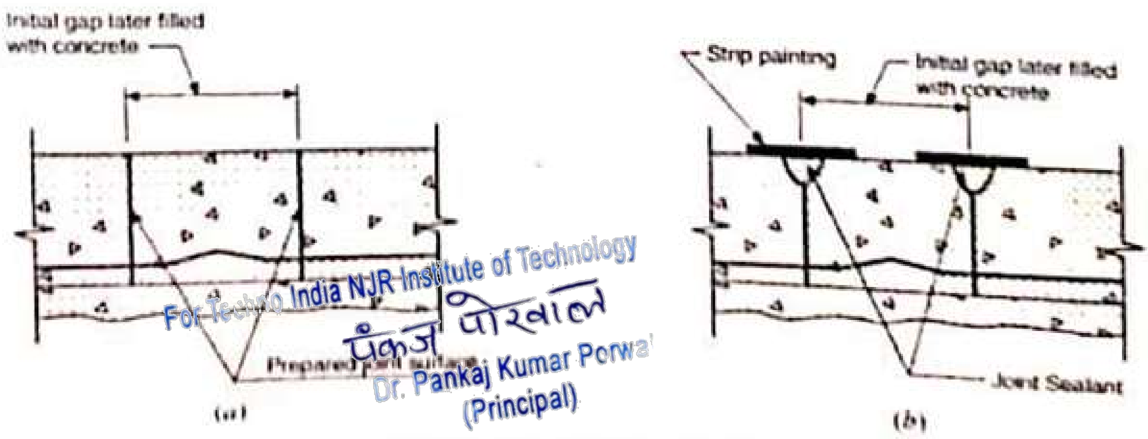

Fig. 22.6. Temporary open joints.

SOLVED EXAMPLES

Example 22.1. **Design a circular tank with a flexible base for a tank of 1,00,000 litre capacity. The depth of water in the tank is 5 m. Use M25 concrete and Fe 415 steel. Take unit weight of water as 9.8 kN/m^2.**

Solution. Given: Volume of water in tank = 1,00,000 l

$$= \frac{100000}{1000}\ m^3$$

Height of water in tank (H) = 5.0 m

Permissible tensile stress in steel = 130 N/mm^2 for HYSD bars [Table 22.3]

Permissible direct tensile stress in concrete = 1.3 N/mm^2 for M25 concrete [Table 22.1]

If D is the diameter of the tank then

$$\text{Volume of tank} = \frac{100000}{1000}$$

$$\frac{\pi}{4} . D^2 \times 5.0 = 100$$

$$D = 5.05\ m$$

Hence providing a diameter of 5.1 m.

- **Maximum hoop tension (T)**

$$T = \gamma H \frac{D}{2}$$

$$= 9.8 \times 5.0 \times \frac{5.1}{2}$$

$$T = 124.95 \text{ kN per m height of the wall}$$

- **Area of Steel**

$$A_{st} = \frac{T}{\sigma_{st}}$$

$$= \frac{124.95 \times 1000}{130}$$

$$A_{st} = 962\ mm^2$$

Using 12 mm diameter bars

$$\text{Spacing required} = \frac{113 \times 1000}{962}$$

$$= 117 \text{ mm}$$

Hence provide 12 mm diameter hoops (rings) @ 110 m c/c

$$A_{st\ provided} = 1027 \text{ mm}^2$$

At a distance 2.5 m from top $T = 62.5$ kN per m, and $A_{st\ reqd} = 481 \text{ mm}^2$, hence spacing can be doubled.

■ **Thickness of tank wall:** The thickness of the wall should be such that the tensile stress in concrete should not exceed the permissible value (σ_{ct})

$$\sigma_{ct} > \frac{T}{1000.t + (m-1)A_{st}}$$

$$1.3 > \frac{124.95 \times 1000}{1000.t + (11-1) \times 1027}$$

$$t > 85 \text{ mm}$$

Hence providing a thickness of 100 mm for tank wall

$$A_{st\ min} = 0.35\% \text{ of X-section area of surface zone}$$

$$= \frac{0.35}{100} \times \left(1000 \times \frac{100}{2}\right) \qquad [t < 300 \text{ mm, Art. 22.4}]$$

$$= 175 \text{ mm}^2 < 1027 \text{ mm}^2 \qquad \text{Hence OK}$$

The spacing of hoops > 300 mm or the thickness of section.

∴ Providing 12 mm diameter hoops @ 110 mm c/c along the height of the wall. The spacing is increased to 220 mm c/c at a distance 2.5 m from top.

■ **Distribution Reinforcement**

Distribution and temperature steel is provided @ 0.35%

$$= 175 \text{ mm}^2$$

Providing 8 mm diameter bars @ 250 mm c/c vertical steel

$$A_{st} = \frac{2 \times 50 \times 1000}{250}$$

$$= 200 \text{ mm}^2 > 175 \text{ mm}^2 \qquad \text{Hence OK}$$

■ **Design of Base/Floor Slab**

Since the tank floor is resting on the ground, the load gets directly transferred to the soil. Hence providing a minimum thickness of 150 mm and 0.35% minimum steel in each direction

$$= \frac{0.35}{100} \times 1000 \times \frac{150}{2}$$

[illegible] [Refer Art. 22.4]

Hence provide 8 mm diameter bars @ 180 mm c/c in both directions at top and bottom face of the floor slab.

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Scanned with CamScanner

The details of the reinforcement are shown in Fig. 22.8.

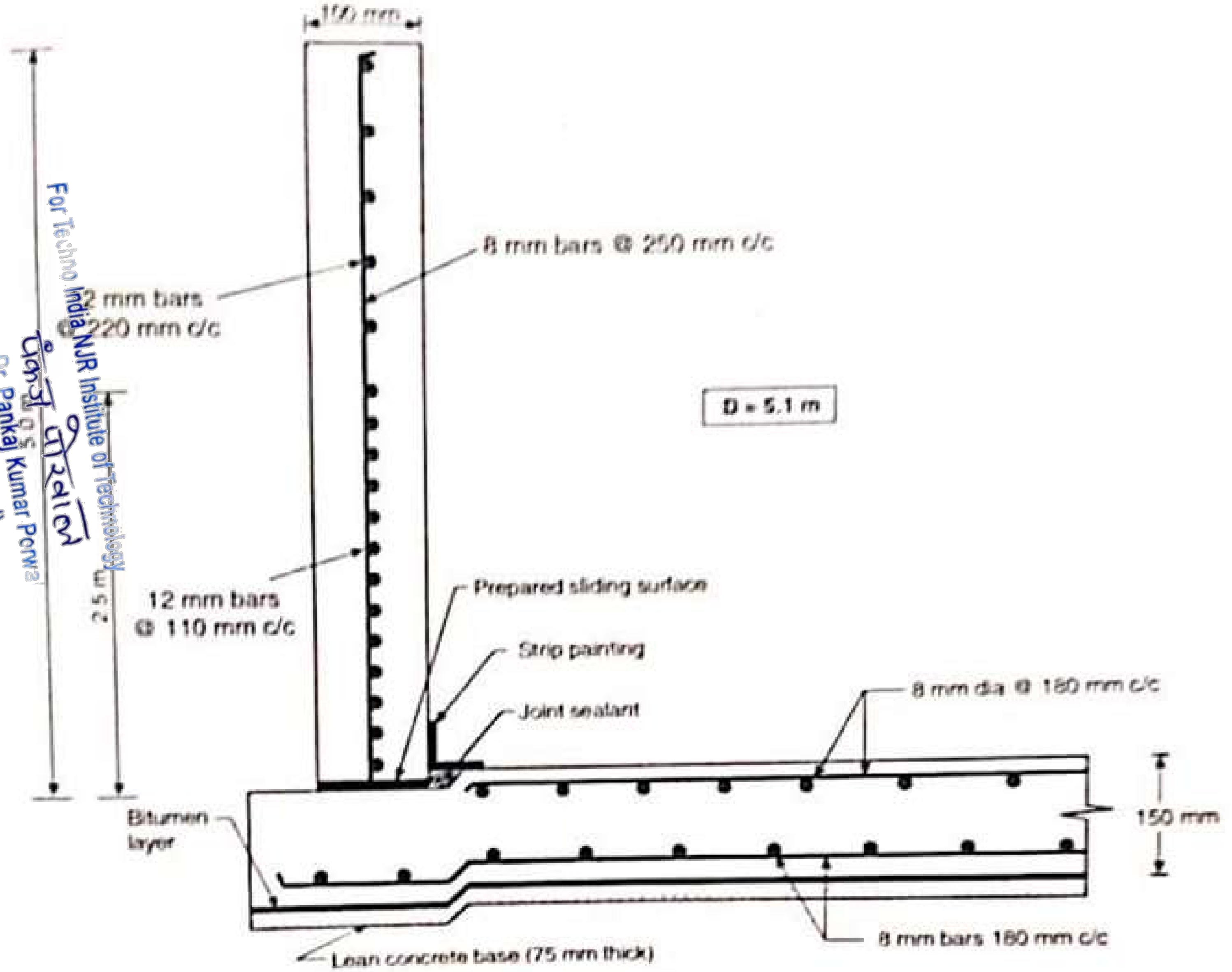

Fig. 22.8. Circular tank with flexible joint

EXPERIMENT NO. 5

Retaining walls:

Analysis and design of Cantilever Retaining Walls:

Following Examples will explain the method of designing a cantilever retaining

SOLVED EXAMPLES

Example 16.1. Design a cantilever retaining wall to retain horizontal earthen embankment of height 4 m above the ground level. The earthen backfill is having a density of 18 kN/m³ and angle of internal friction as 30°. The safe bearing capacity of the soil is 180 kN/m². The coefficient of friction between soil and concrete is assumed to be 0.45. Use M20 concrete and Fe 415 steel.

Solution. Given:

$f_{ck} = 20 \text{ N/mm}^2$

$f_y = 415 \text{ N/mm}^2$

$\phi = 30^\circ$

$\mu = 0.45$

$\gamma = 18 \text{ kN/m}^3$

Safe bearing capacity of soil $= q_0 = 180 \text{ kN/m}^2$

Height of earthen embankment $= 4.0 \text{ m}$

1 ■ **Coefficient of active earth pressure (K_a)**

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

$$K_a = \frac{1}{3}$$

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■ **Minimum depth of foundation (h_{min})**

$$h_{min} = \frac{q_0}{\gamma}\left(\frac{1-\sin\phi}{1+\sin\phi}\right)^2$$

$\frac{q_0}{\gamma} \times (K_a)^2$

$$= \frac{180}{18}\left(\frac{1}{3}\right)^2$$

$$h_{min} = 1.11 \text{ m say } 1.2 \text{ m}$$

∴ Providing the depth of foundation as 1.2 m

Total height of retaining wall = Depth of foundation + Height of embankment

$= 1.2 + 4.0$

Total height of retaining wall (H) $= 5.2$ m

■ **Preliminary dimensions of the retaining wall**

(1) **Base Width (b):** It varies from 0.4 H to 0.6 H

Assuming $b = 2.8$ m

Length of toe slab $= 0.3b$ to $0.4b$

$= 850$ mm (say)

(2) **Thickness of Base Slab**

Thickness of base slab is assumed to be $\frac{H}{10} = 500$ mm.

(3) **Thickness of vertical wall or Stem (Refer Fig. 16.10)**

Thickness of stem may be assumed as $\frac{H}{12}$ at base but here depth required from BM consideration is calculated.

Pressure at the base of the stem $= K_a \gamma h$ [$h = 5.2 - 0.5 = 4.7$ m]

$= \frac{1}{3} \times 18 \times 4.7$

$= 28.2$ kN/m²

Moment at the base of the stem $= \frac{1}{2}(K_a \gamma h) \cdot h \cdot \frac{h}{3}$

$= \frac{1}{2} \times (28.2) \times 4.7 \times \frac{4.7}{3}$

$= 103.83$ kNm

Ultimate moment at the base of the stem

$= 1.5 \times 103.83$

$= 155.7$[illegible] kNm

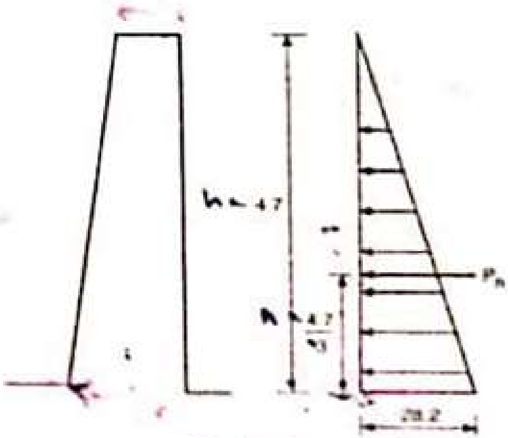

Fig. 16.10

Minimum depth required for a balance section is

$$d_{reqd} = \sqrt{\frac{M_u}{R_u \cdot b}}$$

$R_u = 2.76$, for M20 concrete and Fe 415 steel

$$d_{reqd} = \sqrt{\frac{155.74 \times 10^6}{2.76 \times 1000}}$$

$$= \sqrt{\frac{155.74 \times 10^6}{2.76 \times 1000}}$$

$$= 238 \text{ mm}$$

Assuming 60 mm cover,

$$\text{Total depth required} = 238 + 60$$

$$= 298 \text{ mm}$$

Hence taking $D = 350$ mm at base of stem and reducing it to 150 mm at top. Figure 16.11 shows the trial section.

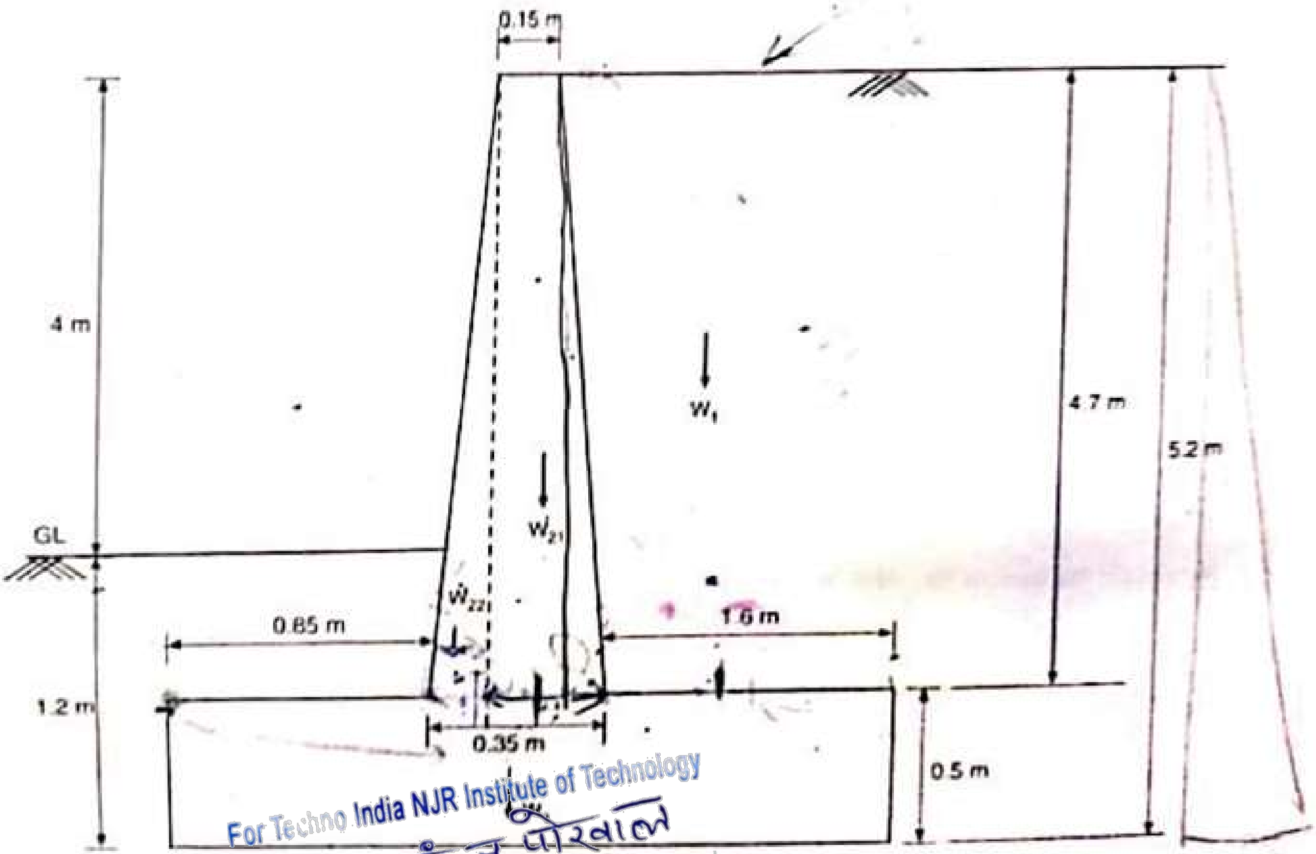

Fig. 16.11

■ **Forces Acting on the Retaining Wall:** Refer Fig. 16.11

Type of Force	Magnitude of Force (kN)	Position of force from toe end O (m)	Bending moment at toe end O (kNm)
(1) **Overturning force** $P_{ah} = \frac{1}{2}(K_a \gamma H).H$	$\frac{1}{2}\times\left(\frac{1}{3}\times 18\times 5.2\right)\times 5.2 = 81.12$	$\frac{H}{3} = \frac{5.2}{3} = 1.733$	$81.12 \times 1.733 = 140.61$ $\Sigma M_0 = 140.61$
(2) **Restoring forces** (a) Weight of backfill (W_1)	$1.6 \times 4.7 \times 18 = 135.36$	$2.8 - \frac{1.6}{2} = 2.0$	270.72
(b) Weight of stem (i) Weight of rectangular portion (W_{21})	$0.15 \times 4.7 \times 25 = 17.625$	$0.85 + 0.35 - \frac{0.15}{2} = 1.125$	19.828
(ii) Weight of triangular portion (W_{22})	$\frac{1}{2}\times 0.2\times 4.7\times 25 = 11.75$	$0.85 + \frac{2}{3}\times 0.2 = 0.983$	11.554
(c) Weight of base slab (W_3)	$0.5 \times 2.8 \times 25 = 35$	$\frac{2.8}{2} = 1.4$	49
	$\Sigma W = 199.735$		$\Sigma M_R = 351.1$

■ **Stability Checks**

(1) **Overturning**

$$\frac{0.9M_R}{M_0} = \frac{0.9 \times 351.10}{140.61}$$

$$= 2.2 > 1.4 \text{ hence o.k.}$$

(2) **Sliding**

$$\frac{0.9F_R}{F_S} \geq 1.4$$

$$F_R = \mu\Sigma W = 0.45 \times 199.735 = 89.88 \text{ kN}$$

$$F_S = P_{ah} = 81.12 \text{ kN}$$

$$\frac{0.9F_R}{F_S} = \frac{0.9 \times 89.46}{81.12} = 0.99 < 1.4$$

Hence, shear key is to be provided to increase the resistance against sliding.

(3) **Base Pressure**

Resultant moment at toe end O = $M_R - M_0$

= [illegible] − 140.61

= [illegible] kNm

The resultant vertical load $= \sum W = 199.73$ kN

It acts at a distance $\bar{x}$ from the toe end O (refer Fig. 16.12)

$$\bar{x} = \frac{210.49}{199.73} = 1.05 \text{ m}$$

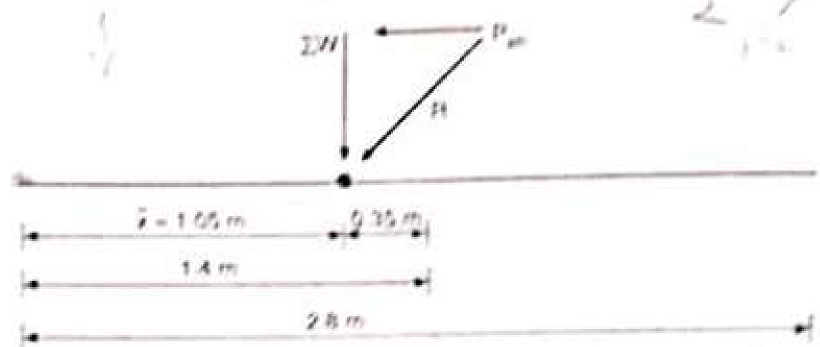

Fig. 16.12.

$$e = \frac{b}{2} - \bar{x} = 1.4 - 1.05$$

$$e = 0.35 \text{ m}$$

which lies in the middle third zone i.e., $\frac{b}{6}$ from centre (0.466 m). Hence OK.

■ **Maximum pressure at toe end O**

$$p_{max} = \frac{\sum W}{b}\left[1 + \frac{6e}{b}\right]$$

$$= \frac{199.73}{2.8}\left[1 + \frac{6 \times 0.35}{2.8}\right]$$

$$p_{max} = 124.83 \text{ kN/m}^2 < 180 \text{ kN/m}^2 \text{ (safe BC of soil)} \qquad \text{Hence OK}$$

■ **Minimum pressure at heel end $= p_{min}$**

$$p_{min} = \frac{\sum W}{b}\left[1 - \frac{6e}{b}\right]$$

$$= \frac{199.73}{2.8}\left[1 - \frac{6 \times 0.35}{2.8}\right]$$

$$= 17.83 \text{ kN/m}^2, \text{ which is positive.}$$

Hence OK, as no tension develops anywhere on the base slab

1. Design of Stem

The depth required for stem [illegible] checked while assuming the preliminary dimensions

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d = [illegible] mm

Maximum moment at base of stem = 155.73 kNm

Area of steel (A_{st}) in stem

$$M_u = 0.87 f_y A_{st} d \left[1 - \frac{f_y A_{st}}{f_{ck} bd}\right]$$

$$155.73 \times 10^6 = 0.87 \times 415 \times A_{st} \times 290 \left[1 - \frac{415 A_{st}}{20 \times 1000 \times 290}\right]$$

$$A_{st}^2 - 13979.23 A_{st} + 20794392.5 = 0$$

On solving the equation

$$A_{st\,(req)} = 1693 \text{ mm}^2$$

Using 16 mm diameter bars, $\quad A_\phi = 201 \text{ mm}^2$

$$\text{Spacing required} = \frac{201 \times 1000}{1693} = 118 \text{ mm}$$

Hence, provide 16 mm diameter, Fe 415 bars @ 100 mm c/c.

Distribution steel

Distribution steel is provided @ 0.12% of total x-sectional area

$$A_{st} = \frac{0.12}{100} \times 1000 \times \left(\frac{150 + 350}{2}\right) \qquad \left[\left(\frac{150+350}{2}\right) \text{is the average thickness of the stem}\right]$$

$$A_{st} = 300 \text{ mm}^2$$

Using 8 mm diameter bars, $\quad A_\phi = 50.3 \text{ mm}^2$

$$\text{Spacing required} = \frac{50.3 \times 1000}{300} = 167.5 \text{ mm}$$

Hence, provide 8 mm diameter Fe 415 bars @ 150 mm c/c, on the inner face of the stem as distribution steel.

Similarly provide 8 mm diameter Fe 415 bars @ 150 mm c/c in both directions at the outer face (front face) of the stem as temperature and shrinkage reinforcement since this face is exposed to weather.

Check for shear

The critical section for shear is at a distance d from base of stem *i.e.*, $h = 4.7 - 0.29 = 4.41$

$$\text{Shear force at this section of the stem} = \frac{1}{2}\left(\frac{1}{3} \times 18 \times 4.41\right) \times 4.41$$

$$= 58.3 \text{ kN}$$

$$V_u = 1.5 \times 58.3$$

$$V_u = 87.52 \text{ KN}$$

$$\text{Nominal shear stress} = \frac{V_u}{bd}$$

$$\tau_v = \frac{87.52 \times 1000}{\text{[illegible]}} = 0.30 \text{ N/mm}^2$$

For $\quad \frac{\text{[illegible]}}{1000 \times 290} = 0.69\%$

$\tau_c = 0.54 \text{ N/mm}^2$ (from Table 5.1)

$\tau_c = 0.54 \text{ N/mm}^2 > \tau_v$ hence OK.

Curtailment of tension reinforcement

As the stem of retaining wall behaves like a cantilever, the bending moment goes on reducing towards the top of the wall and becomes zero at the top. Therefore, tension reinforcement can be curtailed along the height of the stem.

Development length, L_d, for 16 mm diameter bars

$$L_d = \frac{0.87 f_y \phi}{4\tau_{bd}}$$

$$L_d = \frac{0.87 \times 415 \times 16}{4 \times 1.6 \times 1.2}$$

$$= 752 \text{ mm}$$

Therefore, no bar can be curtailed up to a distance of 752 mm from base of the stem. Curtailing bars at a distance 1000 mm from base of the stem *i.e.*,

$$4700 - 1000 = 3700 \text{ mm from top of the stem}$$

$$\text{Total depth at this section} = 150 + \frac{200 \times 3700}{4700}$$

$$= 307 \text{ mm}$$

$$\text{Effective depth at this section} = 307 - 60 = 247 \text{ mm}$$

Moment due to earth pressure at 3.7 m from top

$$= \frac{K_a \gamma h^3}{6}$$

$$= \frac{1}{6}\left[\frac{1}{3} \times 18 \times 3.7^3\right]$$

$$= 50.7 \text{ kNm}$$

$$M_u = 1.5 \times 50.7$$

$$M_u = 76 \text{ kNm}$$

Area of steel required for an ultimate bending moment of 76 kNm

$$76 \times 10^6 = 0.87 \times 415 \times A_{st} \times 247\left[1 - \frac{415 A_{st}}{20 \times 1000 \times 247}\right]$$

On solving, we get $A_{st\ reqd} = 924 \text{ mm}^2$

Using 16 mm diameter bars,

$$\text{Spacing required} = \frac{201 \times 1000}{924} = 217 \text{ mm}$$

Hence half of the bars can be curtailed but as per IS code, 12ϕ or d distance, whichever is more, is to be provided beyond the point of curtailment. Hence curtailment the bars at 1.3 m from base or 3.4 m from top of stem. Thus providing 16 mm diameter bars @ 200 mm c/c after a distance of 1.3 m from base of stream.

Similarly, one more curtailment can be done at 1.5 m from top of stem.

$$\text{Moment at this section} = \left(\frac{18 \times 1.5^3}{3 \times 6}\right)$$

$$= 3.375 \text{ kNm}$$

$$M_u = 1.5 \times 3.375$$
$$= 5.1 \text{ kNm}$$

$$\text{Depth at this section} = 150 + \frac{200}{4700} \times 3200$$
$$= 286 \text{ mm}$$
$$d = 286 - 60 = 226 \text{ mm}$$

$$5.1 \times 10^6 = 0.87 \times 415 \times A_{st} \times 226\left[1 - \frac{415\, A_{st}}{20 \times 1000 \times 226}\right]$$

$$A_{st\ reqd} = 65 \text{ mm}^2 < A_{st\ min} \text{ i.e., } 300 \text{ mm}^2$$

Hence curtailing another half of the bars at 1.5 m from top and providing 16 mm diameter bars @ 300 mm c/c.

2. Design of Heel Slab

The pressure distribution on heel slab is shown in Fig. 16.13.

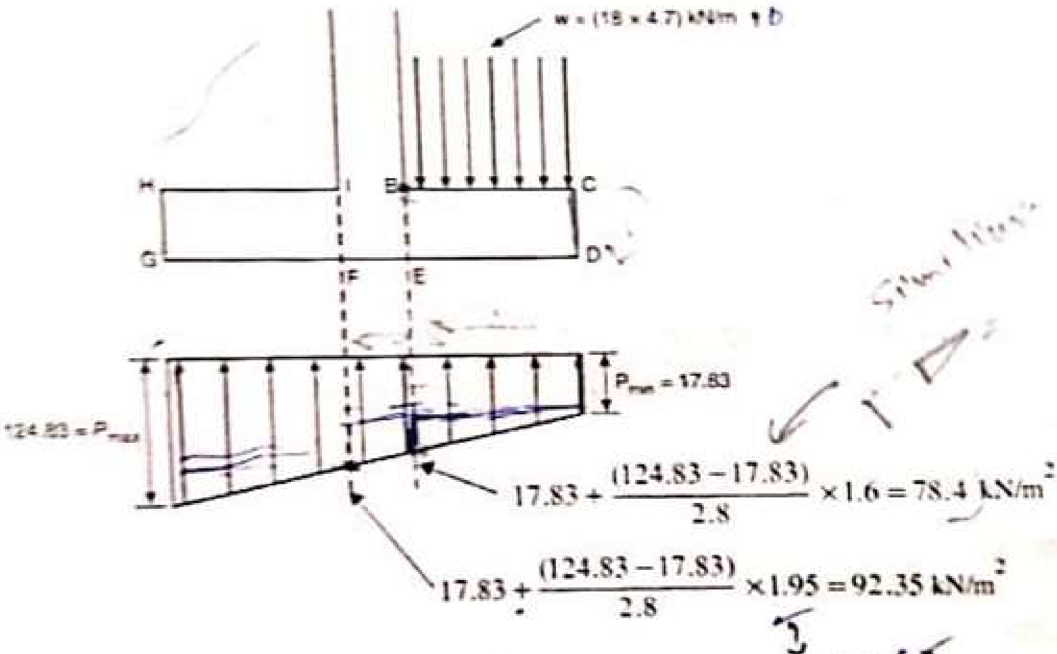


Fig. 16.13.

$$\text{Weight of earth supported on heel} = 18 \times 4.7 = 84.6 \text{ kN/m}$$
$$\text{Self weight of heel slab} = 0.5 \times 1.0 \times 25 = 12.5 \text{ kN/m}$$
$$\text{Total load} = 97.1 \text{ kN/m}$$

$$\text{Maximum bending moment at } B = \frac{97.1 \times 1.6^2}{2} - \frac{17.83 \times 1.6^2}{2} - \left\{\frac{1}{2}(78.4 - 17.83) \times 1.6 \times \frac{1.6}{3}\right\}$$
$$= [illegible]5.8$$
$$= 75.7 \text{ kNm}$$
$$M_u = 1.5 \times 75.7 = 113.6 \text{ kNm}$$
$$d_{reqd} = \sqrt{\frac{113.6 \times 10^6}{2.76 \times 1000}} = 202 \text{ mm} < 440 \text{ mm. Hence OK.}$$

Area of steel for heel slab

$$113.5 \times 10^6 = 0.87 \times 415 \times 440\left(1 - \frac{415 A_{st}}{20 \times 1000 \times 440}\right)$$

$$A_{st} = 741 \text{ mm}^2$$

$$\therefore \quad \text{Spacing of 12 mm bars} = \frac{113 \times 1000}{4741} = 152 \text{ mm}$$

$\therefore$ Provide 12 mm diameter bars @ 150 mm c/c at the top face of the heel slab *i.e.*, *BC*.

Distribution steel is provided @ 0.12% of sectional area in the other direction

$$\frac{0.12}{100} \times 1000 \times 500 = 600 \text{ mm}^2$$

Using 10 mm diameter bars, $\quad A_\phi = 78.5 \text{ mm}^2$, spacing required = 100 mm

Hence, provide same 10 mm dia bars @ 100 mm c/c in the other direction.

3. Design of Toe Slab

The weight of frontfill above the toe slab is neglected and maximum moment is calculated at the face of the stem.

$$\text{Maximum moment} = \frac{92.35 \times 0.85^2}{2} + \frac{1}{2}(124.83 - 92.35) \times 0.85 \times \frac{2}{3} \times 0.85$$

$$= 33.36 + 7.82 = 41.2 \text{ kNm}$$

$$M_u = 1.5 \times 41.2 = 61.8 \text{ kNm}$$

Area of steel for toe slab

$$61.80 \times 10^6 = 0.87 \times 415 \times A_{st} \times 440\left[1 - \frac{415 A_{st}}{20 \times 1000 \times 440}\right]$$

$$A_{st}^2 - 21209.88 A_{st} + 8251001.3 = 0$$

$$A_{st} = 396 \text{ mm}^2 < A_{st\,min}\ (600 \text{ mm}^2)$$

Hence providing minimum area of steel of 600 mm². Therefore provide 10 mm diameter bars @ 100 mm c/c in both directions.

4. Design of Shear key

As the wall is not safe in sliding, shear key is to be provided below the stem as shown in Fig. 16.14.

$$\text{Pressure at face of shear key} = 92.35 \text{ kN/m}$$

$$\text{Coefficient of passive earth pressure} = \frac{1 + \sin\phi}{1 - \sin\phi}$$

$$K_p = \frac{1.5}{0.5} = 3$$

$$\text{Let the depth of key} = a$$

$$\text{Resistance offered by shear key} = 3 \times 92.35 \times a$$

$$= 277.05a$$

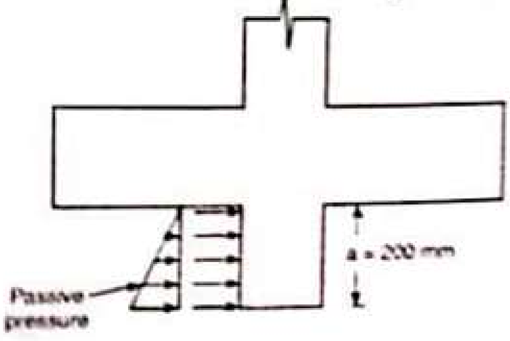


Fig. 16.14

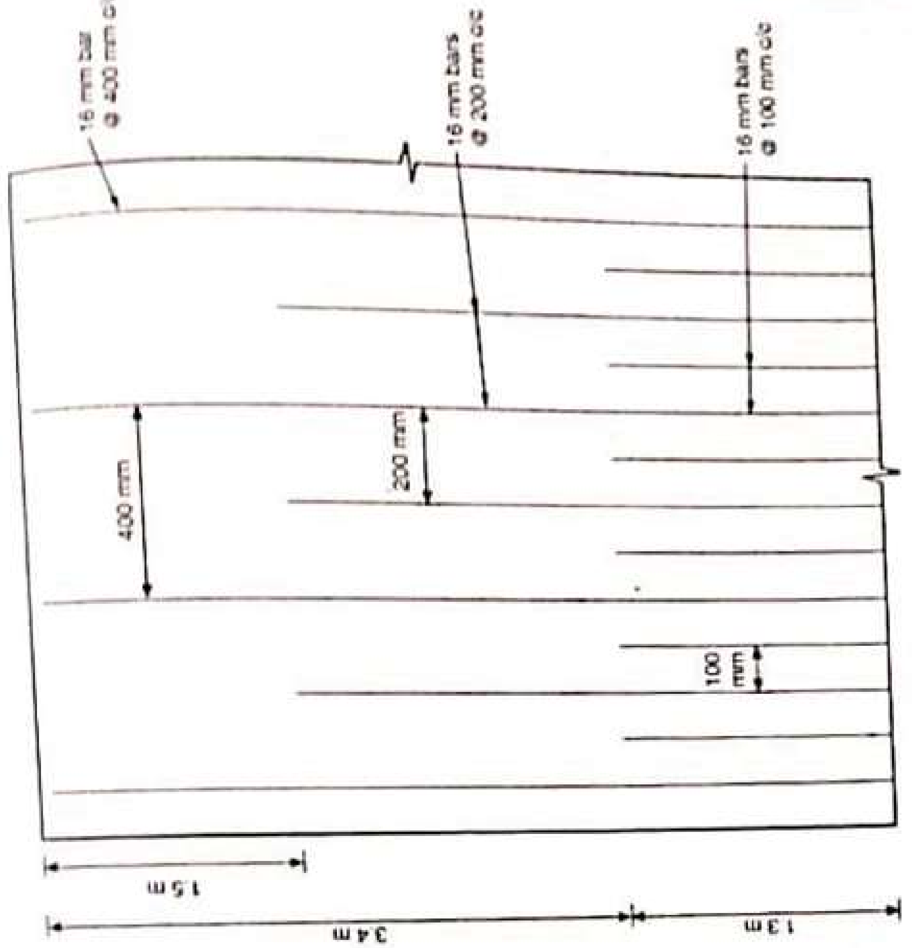
Factor of safety against sliding alongwith shear key

$$= \frac{0.9\mu \Sigma W + 277.05a}{[illegible]} = \frac{0.9 \times 89.88 + 277.05a}{81.12} = 1.4$$

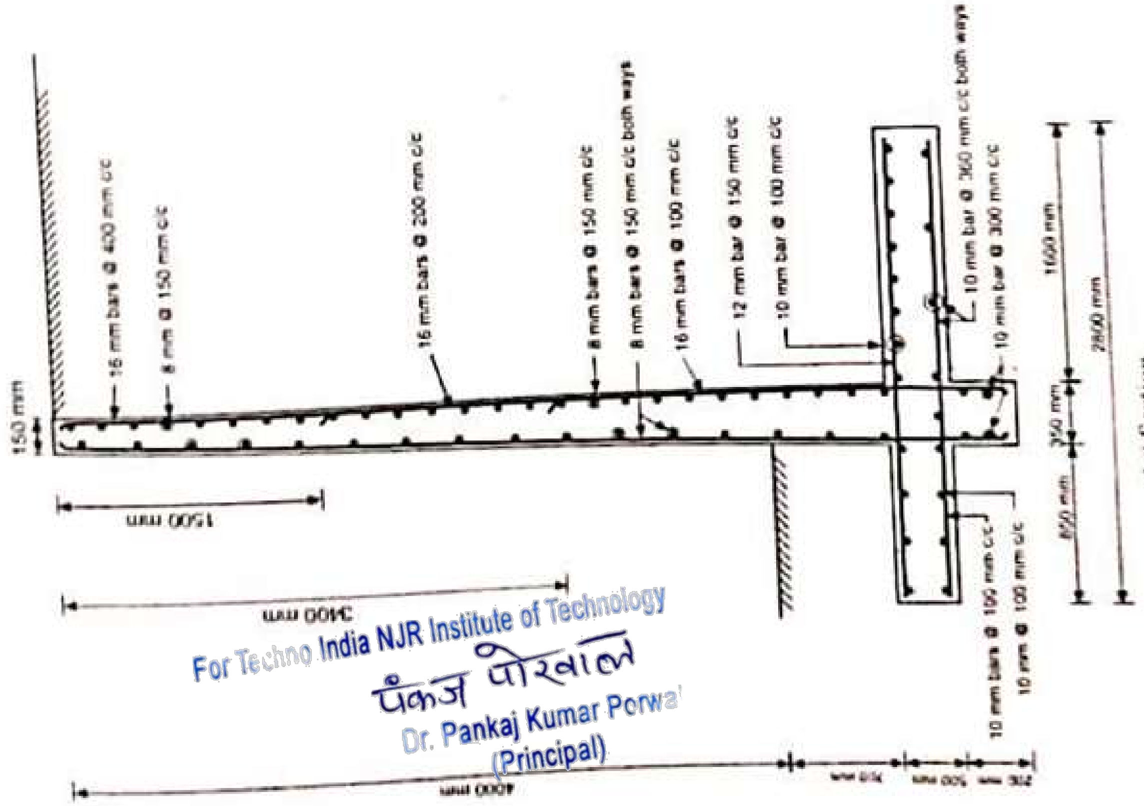
$\Rightarrow \quad a = 0.1$[illegible]

However, provide a 200 mm × 200 mm shear key.

The details of the reinforcement are shown in Fig. 16.15.



(b) L-Section of Stem



(a) Section

Fig. 16.15. Cantilever Retaining Wall