EXAM ACHARYA
EDUCATION REDEFINED

GPSC - CIVIL

Reinforced Cement Concrete

Education's purpose is to replace an empty mind with an open one.

Malcolm Forbes

The content of this book covers all PSC exam syllabus such as MPSC, RPSC, UPPSC, MPPSC, OPSC etc.

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CHAPTER - 1

INTRODUCTION

PLAIN AND REINFORCED CONCRETE

Plain Concrete

It is mixture of sand, gravel, cement, and water which results in a solid mass.

Concrete is strong in compression but weak in tension. Its tensile strength is approx., one tenth of compressive strength. Plain concrete is mostly used in mass concrete work. (As in dams)

Reinforced Concrete

- It is a concrete with reinforcement embedded in it. The embedded reinforcement makes it capable of resisting tension also.
- Steel bars embedded in the tension zone concrete, relieves concrete of any tension and takes all tension without separating from concrete.
- The bond between steel and surrounding concrete ensures strain compatibility i.e., the strain at any point in the steel is equal to that in the adjoining concrete.
- Reinforcing steel imparts ductility to concrete which is otherwise brittle material.
- Here ductility means large deflection owing to yielding of steel, thereby giving ample warning of impending collapse.
- Tensile stress in concrete arises on account of direct tension, flexural tension, diagonal tension (due to shear), temperature and shrinkage effect, restraint to deformation.

Table 2: Minimum grade of concrete

Exposure	Minimum grade of concrete
Mild	M 20
Moderate	M 25
Severe	M 30
Very Severe	M 35
Extreme	M 40

- M 15 or lower grade concrete are used for PCC.
- Minimum grade of concrete for RCC is M20.
- M20 to M40 grade are used for RCC.
- M40 or higher-grade concrete is used for pre-stressed concrete.
- What is M20?

M 20 is the grade of concrete

where,

M=Mix

20= 20 N/mm^2 characteristic compressive strength at 28 days.

Note

➢ Minimum grade f concrete
- RCC-M20
- Pre-tensioning- M 40
- Post-Tensioning M 30
- Coastal area - M 30

150 × 150 × 150 mm size concrete cubes are tested after 28 days of curing. If compressive strength of concrete is more than 20 N/mm^2, the grade of concrete is referred to as M 20.

For M 20 concrete, f_{ck}= 20 N/mm^2

For M 25 concrete, f_{ck}= 25 N/mm^2

These bars do not show definite yield point. So, the yield point is taken as 0.2% proof stress, which is determined from the stress-strain curve as follows:

i. Draw a straight line parallel to the initial straight portion of the stress-strain curve, corresponding to strain value of 0.002 (0.2%).

ii. The point where this line curs the stress-Strain curve is taken as the yield stress or 0.2% proof stress.

Advantages of HYSD Bars

- HYSD bars have higher yield strength compared to mild steel bars.
- Because of the yield strength, the mass of the steel required is reduced, which leads to overall economy.
- The HYSD bars have better bond with concrete.

Disadvantages

- It is less ductile than mild steel.
- The low percentage of longitudinal steel reduces the capacity of the section in shear.
- With higher stress level, the deflection and cracking of the member are larger.

iii. TMT bars

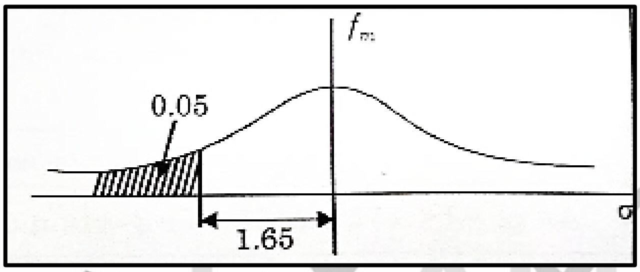
TMT= Thermo Mechanically Treated bars

TMT bars are manufactured by subjecting the hot rolled M.S. bars to a controlled cooling process which converts the outer surface of the bar into hardened structure. This process increases the yield strength of the bar without losing its yielding property.

In Addition, the bond strength is also increased due to this treatment. TMT bars have high strength combined with high elongation. This increases ductility and weldability of the bars.

- Additional samples may be required for various purposes such as to determine the strength of concrete at 7 days or at the time of striking the form work, or to determine the duration of curing, or to check the testing error. Additional specimen may also be required for testing samples cured by accelerated methods
- To report strength of cube, we take average of three specimens of a sample.
- Individual variation should not be more than ± 15% of average. If variation is more, test results of the sample are invalid.

CHARACTERISTIC STRENGTH (f_{ck}) AND GRADE OF CONCRETE



It is that strength below which not more than 5% of test results are expected to fall

$$(f_{ck} = f_m - 1.65\sigma)$$

- Concrete is designated by characteristic cube strength of concrete at 28 days.
- As cement hydrates, it gains strength over a long period. Hence we need to specify the strength after some particular time.

FLEXURAL STRENGTH OF CONCRETE (MODULUS OF RUPTURE)

Tensile strength of concrete in flexure is called flexural strength.

Splitting tensile strength $f_{ct} = \frac{2P}{\pi d L}$

- F_{ct}= splitting tensile strength = 0.66 fcr modulus of rupture
- Direct tesile strength = (0.5- 0.625) f_{cr}
- Modulus of elasticity of concrete, Ec = 5000 f_{ck} ……. Short term modulus of elasticity

➢ **Long term modulus of elasticity (Ece):**

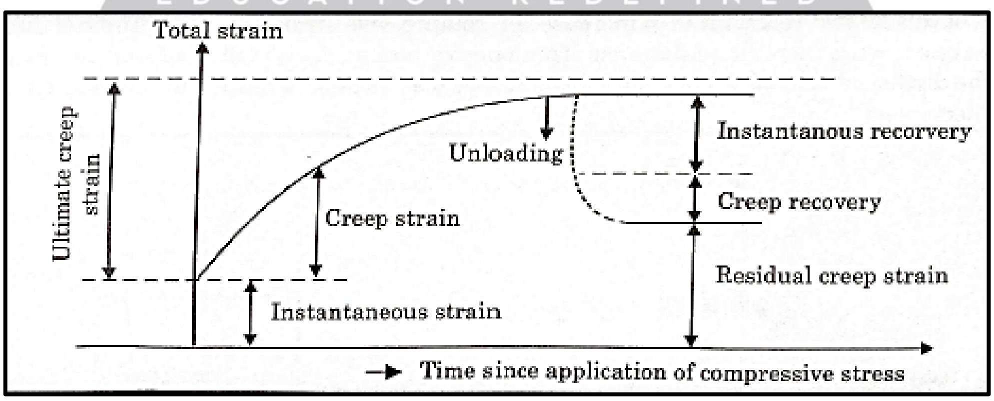
$$E_{ce} = \frac{Ec}{1+\theta}$$

Where,

θ= creep coefficient

CREEP IN CONCRETE

- When concrete is subjected to sustained compressive loading, its deformation keep on increasing with time, even though the stress level is not altered.
- Time dependent component of total strain is called creep.



Creep increases when:

(a)Cement content is high (b) W/c ratio is high (c) Aggregate content is low (d) Air entertainment is high (e) relative humidity is low (f) Temperature (causing moisture

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Building Material and Construction

Dream is not that which you see while sleeping it is something that does not let you sleep.

A.P.J. Abdul Kalam

The content of this book covers all PSC exam syllabus such as MPSC, RPSC, UPPSC, MPPSC, OPSC etc.

SHRINKAGE IN CONCRETE

- Concrete undergoes volume changes as it changes phase from plastic to solid and this process is called shrinkage.
- Shrinkage is usually expressed as a linear strain (mm/mm).
- Shrinkage and temperature effects are similar.
- Shrinkage and creep are not independent phenomena. But for convenience it is normal practice to treat effects as separate, independent and additive.
- Unlike creep, shrinkage, strains are independent of the stress condition of concrete.
- Shrinkage is reversible to a great extent.
- The total shrinkage of concrete depends upon the constituents of concrete, size of member and environmental conditions etc.
- For a given humidity and temperature, the total shrinkage of concrete is mostly influenced by the total amount of water present in the concrete at the time of mixing and to a lesser extent by the cement content.
- Shrinkage has detrimental effects on the serviceability and durability of concrete. Shrinkage has been divided into five types as per different mechanism.

  a) Chemical Shrinkage

  b) Autogenous Shrinkage

  c) Plastic Shrinkage

  d) Drying shrinkage

  e) Carbonation Shrinkage

TEMPERATURE EFFECTS

- Concrete expand with rise in temperature and contracts with fall in temperature.
- The effect of thermal contraction and shrinkage are similar.

1. For main reinforcement up to 12 mm diameter bar for mild exposure the nominal cover may be reduced by 5 mm.
2. Unless specified otherwise, actual concrete cover should not deviate from he required nominal cover by +10 mm.
3. For exposure condition 'severe' and 'very severe', reduction of 5 mm may be made, where concrete grade is M35 and above.

DESIGN METHODS

1) Working stress method
2) Ultimate load Method
3) Limit state method

WORKING STRESS METHOD

- This was the traditional method of design
- Material is assumed to behave in a linear elastic manner.
- Stresses within the material is not allowed to exceed the permissible stress.

$$\text{Permissible stress} = \frac{\textit{Strength of material}}{\textit{Factor of safety}}$$

In RCC design, permissible stress in tension in tension steel: $\sigma_{st} = 0.55\ f_y$ (approx.)

$$\Rightarrow \text{F.O.S} = 1.8$$

Permissible compressive stress in concrete in bending: $\sigma_{cbc} = 0.33\ f_{ck}$ (approx.)

$$\Rightarrow \text{F.O.S} = 3$$

Deficiency in WSM

- It may not be possible to keep the stress within permissible stress. This is because of
  - ✓ Long term effect of shrinkage and creep

- This is clearly in error because significant inelastic behaviour and redistribution of stress resultant takes place as loading is increased from service loads to ultimate loads.

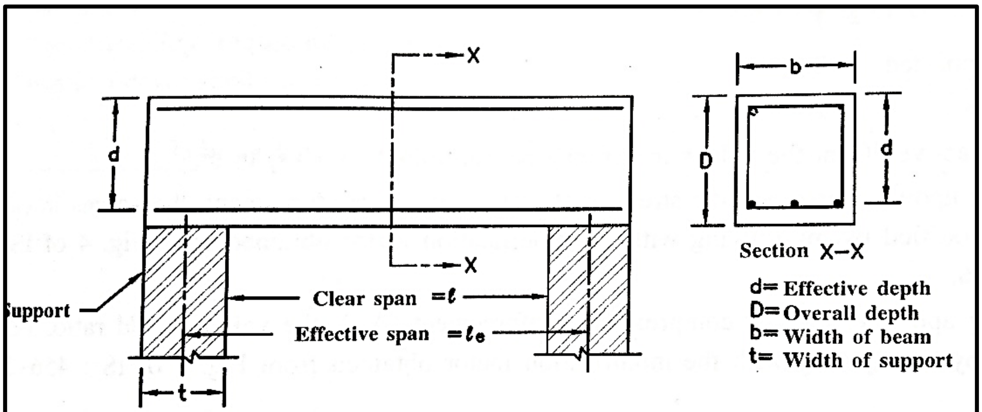
LIMIT STATE METHOD OF DESIGN: (INTRODUCED IN 1970)

- We know that there is uncertainty in loading and the material properties. Besides uncertainty also occur due to uncertainty in dimension of member, uncertainty due to simplifying assumption used in analysis & design, etc.
- To account for these uncertainties, F.O.S. was used in WSM and load factors are used in Ultimate load method. However there was no theoretical justification for use of the F.O.S. and load factors.
- To overcome this reliability based analysis was performed and factors of safety were established both for lading and material properties. These factors of safety are called partial safety factor.
- Analysis based on the above concept was called limit state method.
- LSM provides adequate safety at ultimate load and adequate serviceability at service loads by considering all possible limit states.
- In this method of design, actual stresses developed at collapse differ considerable from the theoretical values.
- Selection of various partial safety factors are sound probalistic basis.
- Limit state is state in which the structure become unfit for use.

There are two types of limit states.

a) **Limit state of serviceability**: Satisfactory performance under service load. Like discomfort caused by excessive deflection, crack width, vibration, leakage, loss of durability etc.

b) **Limit state of collapse:** Adequate margin of safety for normal over loads. These include limit state of strength, overturning, sliding, buckling, fatigue etc.



Simply Supported Beam

b) Continuous Beam or slab

i. If, width of support < $\frac{1}{12}$×clean span

Effective span (le) is taken as per (a) above.

ii. If, width of support > $\frac{1}{12}$ ×Clear span

Or

Width of support > 600 mm

Effective span (le) is taken as under:

1) For end span with one end free and other continuous or for intermediate span:

Effective span (l_e)= clear span

2) For end span with one end free and other continuous

i. Clear span + $\frac{d}{2}$

ii. Clear span + $\frac{1}{2}$ × width of continuous support

Whichever is smaller.

d) Depending upon the area of compression reinforcement (A_{sc}), the value of L/d ratio, be further modified by multiplying with the modification factor obtained from Fig. 5 of IS: 456-2000, P. 39.

e) For T beam and L beam, the values of (a) or (b) be modified as Fig. 6 of IS: 456-2000, P. 39 and P_t for use in Fig. 4 and 5 of code, should be based on area equal to b_f d

Cover to reinforcement

The main purpose of providing cover to steel reinforcement is

- To protect the reinforcement against corrosion
- To provide cover against fire
- To develop the sufficient bond strength along the surface of the steel bar

Cover to reinforcement can be indicated by two ways:

i. Nominal cover (clear cover)
ii. Effective cover

i. **Nominal cover (clear cover):** the thickness of concrete from the surface of reinforcement bar to the nearest edge of concrete is called nominal cover (clear cover)

ii. **Effective cover:** The thickness of concrete from the centre of bar to the nearest edge of concrete is called effective cover.

e = Effective cover

e’ = Clear cover

d = Effective depth

D = Overall depth

 Effective cover = clear cover + $\frac{dia.of\ bar}{2d}$

Notes
1. For main reinforcement upto 12 mm dia. Bar for mild exposure the nominal cover may be reduced by 5 mm.
2. Unless specified otherwise, actual concrete cover should not deviate from the required nominal cover by +10 mm
3. For exposure condition, 'severe' and 'very severe' reduction of 5 mm may be made, where concrete grade is M-35 and above.

iii. For longitudinal bars in column,

– Nominal cover 40 mm

– Nominal Cover diameter of bar

For slabs = 20 mm

Or beam = 20 mm

iv. For footings,

– Nominal cover 50 mm

v. For fire resistance, nominal cover shall not be less than values given in IS: 456-2000, p. 47, table-16A

Qu5 Separation of coarse aggregates from mortar during transportation is known as

a) Bleeding

b) Creeping

c) Segregation

d) Shrinkage

Qu6 The value of ultimate creep coefficient for concrete

a) Increases with age of loading

b) Decreases with age of loading

c) Remains constant

d) Is taken as 0.0003

Qu7 Shrinkage in concrete can be reduced by using

a) Low water cement ratio

b) Less cement in the concrete

c) Proper concrete mix

d) All the above

TEST YOUR SELF

Qu8 The leaching action in concrete is the example of

a) Decomposition

b) Creeping

c) Crystallization

d) Chemical reaction

CHAPTER – 2

IS: 456-2000 CRITERIA

MINIMUM HORIZONTAL DISTANCE BETWEEN BARS: (IS: 456-2000, P. 45)

It shall not be less than the greatest of the following:

i. Diameter of bar, if diameters are equal.
ii. Dia. of larger bar, if diameter is unequal.
iii. Maxi. Size of aggregate + 5 mm

MINIMUM VERTICAL DISTANCE BETWEEN BARS

Shall Not Be Less Than Greater of

i. 15 mm
ii. dia. Of bar
iii. 2/3 × size of aggregate

SIDE FACE REINFORCEMENT: (IS: 456-2000, P.47)

- If the depth of web of the beam exceeds 750 mm side face reinforcement is required.
- Total area of side face reinforcement shall not be less than 0.1% of the web area.
- It shall be equally distributed on two faces.
- Spacing shall not exceed 300 mm or width of web.

DEEP BEAM: (IS: 456-2000, P.51)

$$\frac{l}{d} \ngtr 2 \ldots\ldots \text{ for simply supported beam}$$

TEST RESULT

The test result shall be the avenger of the strength of the three speτcimens.

- The individual variation should not be more than ± 15%
- Mean of the group of 4-non overlapping consecutive test results

  $\geq f_{ck} + 0.825$ S

  $\geq f_{ck} + 3$ N/mm^2

  For M 15 and above

  For M 20, M 25 …… S = 4 N/mm^2

- Individual test result (IS: 456-2000, P.29, 30)

  $\geq f_{ck} - 3$ N/mm^2

  $\frac{l_e}{D} \leq 12$ ………. Short column (IS: 456-2000, P.41)

  $\frac{l_e}{D} > 12$ ………. long column

TWO WAY SLAB ANALYSIS

Elastic method

- Based on Pigeaud's theory
- Wester guard's theory

Limit state method

- Johansen's yield line theory
- Rankine Grassof's theory

B.M. Coefficients for Two-way simply supported slab- Rankine Grashoff theory

B.M. Coefficients for Two-way restrained slab- Johansen's yield line theory

D ≥ 300 mm, k=1.0

BUNDLED BARS (IS: 456-2000, P.43)

The development length for bundle bars shall be increased as under: (IS: 456-2000, P.43)

Two bars in contact- 10 %

Three bars in contact – 20%

Four bars in contact – 33%

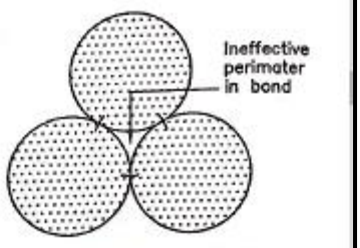


Fig. 4.4 (A) Bundled bars

SPLICING OF BARS (IS: 456-2000, P.45)

- Splicing of bars shall be avoided at a section where in B.M. is more than 50% of M.R.
- Not more than 50% of bars should be spliced at a section.
- Bars greater than 36mm ϕ should not be lap spliced. They should be welded.
- For bars in flexural tension, Lap length> L_d or 30 ϕ
- For bars in direct tension, Lap length> 2 L_d or 30 ϕ
- For bars in compression, Lap length> L_d or 24 ϕ

Permissible shear stress (for footings)

$$\tau_c' = k_s . \tau_c$$

$$\tau_c = 0.25\sqrt{f_{ck}}$$

$$k_s = 0.5 + \beta_c \ldots\ldots \not> 1$$

$$\beta_c = \frac{\text{short side of column}}{\text{long side of column}}$$

if $\tau_v > 1.5\,\tau_c$ ….. footing shall be redesigned.

Permissible limits of solids in water for concrete

Sulphate (SO3)……….. 400 mg/l

Chloride (Cl))……….. 500 mg/l……. For RCC

……….. 2000 mg/l ……. For PCC

Organic matter……….. 200 mg/l

Inorganic matter= 3000 mg/l (IS: 456-2000, P.15)

CONTROL OF DEFLECTION (IS: 456-2000, P.37)

The final deflection due to all loads of horizontal member should not exceed *l*/250.

The deflection including the effects of temperature, creep, shrinkage after erection of partitions are finishes should not exceed l/350 or 20 mm whichever is less.

For spans up to 10 m: l/d ratio are

Cantilever = 7

Simply supported = 20

Continuous = 26

For span > 10 m, above values are multiplied by 10/span.

For Bent Up Bars

$$V_{us} = 0.87\, f_y\, A_{sv}.\sin\alpha$$

- Shear resistance of bent up bars should not exceed 50% of V_{us}
- Maximum strain in concrete in compression in flexure = 0.0035
- Maximum strain in concrete in axial compression = 0.002
- Maximum strain in compression steel of doubly RC beam= 0.0035 $(1 - \frac{d'}{x_u})$
- Tensile strength of concrete is measured by split cylinder test (compressive load on cylinder along diameter)
- Flexural strength of concrete (modulus of rupture) is measured by beam test.

FLAT SLABS (IS: 456-2000, P.53)

- Slab directly resting on columns without beams.
- The minimum thickness of flat slab is **125 mm**.
- Drops are provided to resist **shear**.
- Column strip width

= 2× (0.25 l_1)

or

= 2× (0.25 l_2)

l_1= span in the working direction

Whichever is small.

- For column head, right circular cone has vertex angle of 90°.
- In interior span, total design moment (M_0) shall be distributed as:
- Negative design moment= 0.65
- Positive design moment= 0.35

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Construction, Planning and Management

"All Birds find shelter during a rain. But Eagle avoids rain by flying above the Clouds."

A.P.J. Abdul Kalam

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CLEAR YOUR CONCEPT

Qu1 I.S has specified the full-length strength of concrete after

a) 7days

b) 14days

c) 21days

d) 28days

Qu2 The minimum grade of reinforced concrete in sea water as per IS 456 : 2000 is

a) M15

b) M20

c) M30

d) M40

Qu3 The characteristic strength of concrete is defined as that strength below which not more than ________ of the test results are expected to fall

a) 10percent

b) 5percent

c) 15percent

d) 20percent

Qu4 Additional cover thickness in reinforced cement concrete member totally immersed in sea water is

a) 25mm

b) 30mm

c) 35mm

d) 40mm

CHAPTER – 3

DESIGN OF BEAM

WHAT IS LIMIT STATE METHOD?

- The acceptable limit for the safety and serviceability requirement of a structure or structural element before failure occurs is called limit state.
- In this, structure is so designed that it carry the load with sufficient degree of safety and serviceability and structure will not become unfit for use for which it is to be designed.

Two main limit state are

Limit state of collapse

To satisfy this limit state, the strength must be adquate to carry the loads.

Limit state of collapse includes limit state of

- Flexure
- Compression
- Torsion
- Shear

Limit state of serviceability

To satisfy this limit state, deflection, cracking and vibration must not be excessive.

- Excessive deflection can reduce the efficiency of the structure and must be avoided.
- Cracking causes ingress of water.

CHARACTERISTIC STRENGTH OF MATERIALS

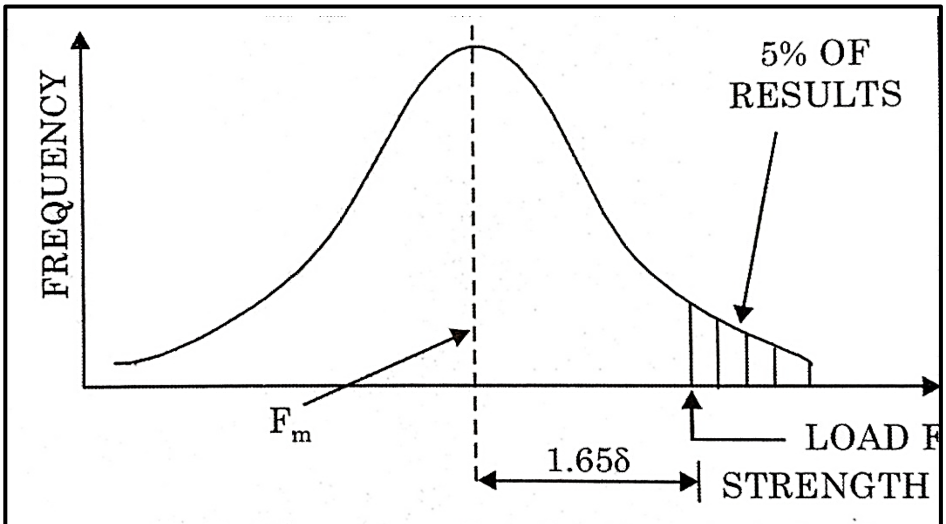
The strength of material below which not more than 5% of the test results are expected to fall is known as the characteristic strength of the material and denoted by f_{ck} for concrete.

Note

1. In the designation of concrete mix, letter M refers to the mix and the number to the specified characteristic compressive strength of 15cm cube at 28 days, expressed in N/mm^2
2. M5 and M7.5 grades of concrete may be used for lean concrete bases and simple foundation for masonry walls. These mixes need not be designed.
3. Grade of concrete lower than M20 shall not be used in reinforced concrete (as per IS 456: 2000)
4. For sea water grade of concrete lower than M30 shall not be used in reinforced concrete. (as per IS 456: 2000)

CHARACTERISTIC LOAD

Value of load which has a 95% probability of not being exceed during the life of the structure is known as characteristic load and is denoted by F



Definition curve for characteristic load

$F = F_m + 1.65\ \delta$

F_m = mean value of load

(F_m - 1.65 δ) and (F_m + 1.65 δ) are two important limit within which "probability of lying test result" is maximum. These limit is called confidence limit.

PARTIAL SAFETY FACTOR

In limit state method of design two factors of safety are used, one to account for uncertainty in material property and other for uncertainty in loading. Hence the factors are called partial factor of safety.

γ_{ms}= partial safety factor for material strength.

Table: value of partial safety factor for loads under various load combination

Load Combination	Limit State of Collapse			Limit States of Serviceability		
	DL	IL	WL	DL	IL	WL
(1)	(2)	(3)	(4)	(5)	(6)	(7)
DL + IL	1.5			1.0	1.0	–
DL + WL	1.5 or 0.9*	–	1.5	1.0	–	1.0
DL + IL + WL	1.2			1.0	0.8	0.8

Notes

- This value is to be considered when stability against overturning or stress reversal is critical. Design values are obtained when partial safety factors are applied to characteristic load and materials.
- While considering earthquake effects, substitute EL for WL.
- For the limit states of serviceability, the value of γ_f given in this table are applicable for short term effects. While assessing the long term effects due to creep the dead load and that part of the live load likely to be permanent may only be considered.

The design load F_d is given by

$F_d = F\,\gamma_f$

F= characteristic load

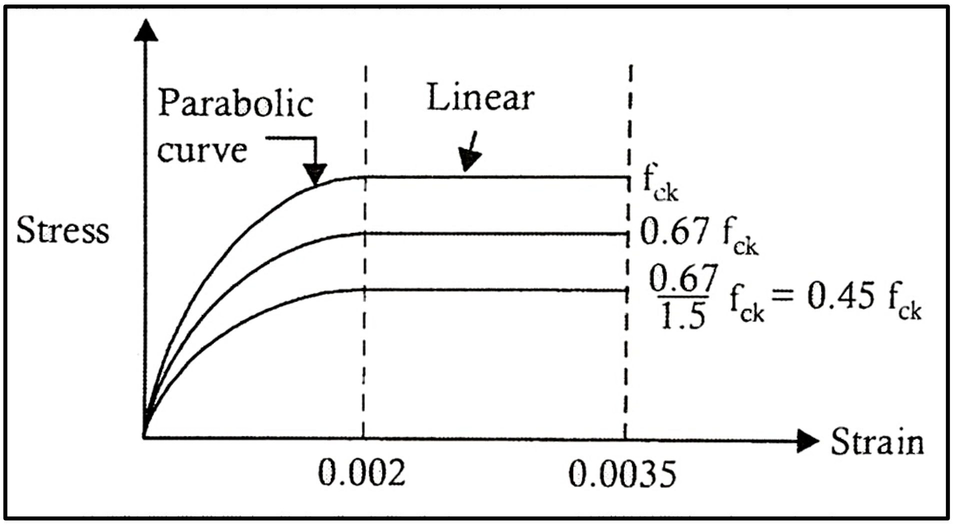
γ_f= partial safety of factor for load

Characteristic loads	Indian Standard specification
Dead loads	IS:875 (Part1): 1987
Imposed (live) loads	IS:875 (Part2): 1987
Wind loads	IS:875 (Part3): 1987
Seismic loads	IS:1893(Part1): 2002

ASSUMPTION IN LIMIT STATE OF COLLAPSE: FLEXURE

1. Plane section before bending remains plane even after the bending.

This assumption means that strain at any point on the cross-section is directly proportional to its distance from its neutral axis, it means strain diagram is linear.



Note

➢ In limit state method of design, the failure criterion for reinforced concrete beam and column is maximum principal strain theory

- For design purpose, the compressive strength of concrete shall be assumed to be 0.67 times the characteristic strength and partial factor of safety γ_{ms} = 1.5 shall be applied in addition to this.

- It may be noted that for the design of flexural members, the characteristic strength of concrete is taken as 0.67 fck. This is on account for the fact that in actual structure, size of concrete member may be more than the cube size tested in laboratory. Larger size leads to more variability in strength. Hence, strength of the concrete member will be lesser than that in cube. However beyond 450 mm size cube the variation of strength is not much.

- The variation of strain-stress curve shall be parabolic up to 0.002 strain and, thereafter, the stress remains constant up to the maximum permissible strain of 0.0035. At limit state, this is an idealised curve for concrete in compression and is valid for all grades of concrete irrespective of percentage pf tensile reinforcement.

Note

➢ Compared to working stress method, limit state method takes concrete to a higher stress level by taking into account the non-linear stress-strain diagram.

$= \frac{2}{3} \times 0.45\, f_{ck}.\, \frac{4}{7}\, x_u\, b$

$C_2 = 0.171\, f_{ck}\, x_u\, b$

This will act at a distant

$y_2 = (\frac{3}{7}x_u + \frac{3}{8} \times \frac{4}{7}x_u) = (\frac{3}{7}x_u \times \frac{12}{56}x_u) = \frac{224x_u + 12x_u}{56} = \frac{36}{56}x_u$

$y_2 = \frac{9}{14}x_u$ from top

This combined compressive force will act a distance from top.

C.G. location of total force from top ($\bar{y}$)$= \frac{c_1 y_1 + c_2 y_2}{c_1 + c_2}$ $\frac{0.193\, f_{ck} x_u\, b \times \frac{3}{14}x_u + 0.171\, f_{ck} x_u\, b\, .\frac{9}{14}x_u}{0.193\, f_{ck} x_u\, b + 0.171\, f_{ck} x_u\, b}$

$\bar{y} = 0.42$ xu

 Lever arm distant between centroid of compressive force to centroid of tensile force (z) is given by

$z = d - \bar{y}$

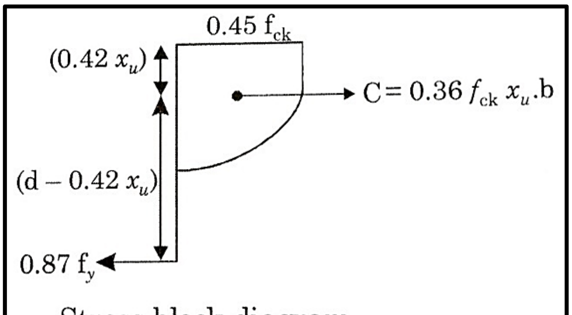
$z = d - 0.42\, x_u$

Total compressive force

$C = C_1 + C_2$

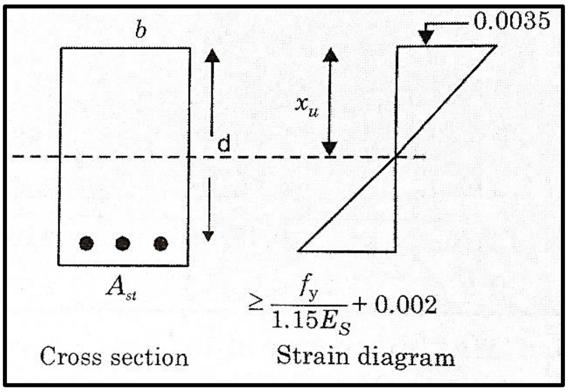
$= 0.193\, f_{ck} x_u\, b + 0.171\, f_{ck} x_u\, b$

$= 0.36\, f_{ck} x_u\, b$



Stress block diagram

LIMITING DEPTH OF NEUTRAL AXIS



From the recommendation that, $\epsilon_{st} \geq \frac{0.87f_y}{E_s}$ +0.0020

And strain in concrete at collapse, should be 0.0035,

$$\frac{0.0035\ (d-x_u)}{x_u} \geq \frac{0.87f_y}{E_s} + 0.0020$$

$$\frac{d}{x_u} - 1 \geq \frac{\frac{0.87f_y}{E_s} + 0.0020}{0.0035}$$

$$\frac{d}{x_u} \geq \frac{\frac{0.87f_y}{E_s} + 0.0055}{0.0035}$$

$$\frac{x_u}{d} \leq \frac{0.0035}{\frac{0.87f_y}{E_s} + 0.0055}$$

Thus limiting value of neutral axis depth is given by

$$\left(\frac{x_u}{d}\right)_{lim} = \frac{0.0035}{0.0055 + \frac{0.87f_y}{E_s}}$$

Table: Value of $x_{u\ lim}/d$ for various grades of steel.

	Fe 250	**Fe 415**	**Fe 500**
$\frac{x_{u\ lim}}{d}$	0.5313	0.4791	0.4560
As per IS code	0.53	0.48	0.46

b) To determine steel area in tension when concrete cross-section and applied and applied moment are known

c) To design a cross-section for a bending moment.

a) To determine moment of resistance (M_u) when cross-section of a beam is known

- First of all the depth of neutral axis (x_u) is calculated assuming that, the strain is outermost compression fibre of concrete has reached a value of 0.0035 and strain in tensile steel is not less than $0.002+\frac{f_y}{1.15E_s}$

- In other words, the maximum stress in concrete at the top most fibre is 0.45 f_{ck} and the stress in tensile steel is $\frac{f_y}{1.15}$ = 0.87 f_y

- From equilibrium of horizontal forces on a section

  Total compressive force (C) = Total tensile force (T)

$$0.36\, f_{ck}\, x_u\, b = \frac{f_y}{1.15} A_{st} = 0.87\, f_y\, A_{st}$$

$$x_u = \frac{0.87\, f_y\, A_{st}}{0.36\, f_{ck}\, b} \text{ …….. (1)}$$

- For Analysis purposes, it is convenient to express x_u in non-dimensional form as follows:

$$\frac{x_u}{d} = \frac{0.87\, f_y\, A_{st}}{0.36\, f_{ck}\, bd} \text{ …….. (2)}$$

If $\frac{x_u}{d} < \frac{x_{u\,lim}}{d}$, section is under reinforced

If $\frac{x_u}{d} = \frac{x_{u\,lim}}{d}$, section is balanced

If $\frac{x_u}{d} > \frac{x_{u\,lim}}{d}$ section is over reinforced.

$$M_{u\,lim} = 0.87\, f_y A_{st} \times d\,(1- \frac{0.42 \times 0.87\, f_y A_{st}}{0.36\, f_{ck}\, b})$$

$$M_{u,\,lim} = 0.87\, f_y A_{st}\, d\,(1- \frac{f_y A_{st}}{f_{ck}\, bd})$$

Thus limiting moment of resistance ($M_{u\ lim}$) of a given singly reinforced section is corresponding to

$$A_{st} = A_{st\ lim}\ \&\ x_u = x_{u\ lim}$$

Limiting percentage tensile reinforcement

$$A_{st\ lim} = \frac{0.36\, f_{ck}\, b\, x_{u,lim}}{0.87\, f_y})$$

$$\frac{A_{st}}{bd} \times 100 = P_{st\ lim} = \%\ \text{limiting tensile reinforcement}$$

$$P_{st\ lim} = \frac{0.36\, f_{ck}}{0.87\, f_y}\frac{x_{u\ lim}}{d} \times 100 = 41.38\, \frac{f_{ck}}{f_y}\,(\frac{x_{u,lim}}{d})$$

$$P_{t\ lim} = 41.38 \frac{f_{ck}}{f_y}\,(\frac{x_{u,lim}}{d})$$

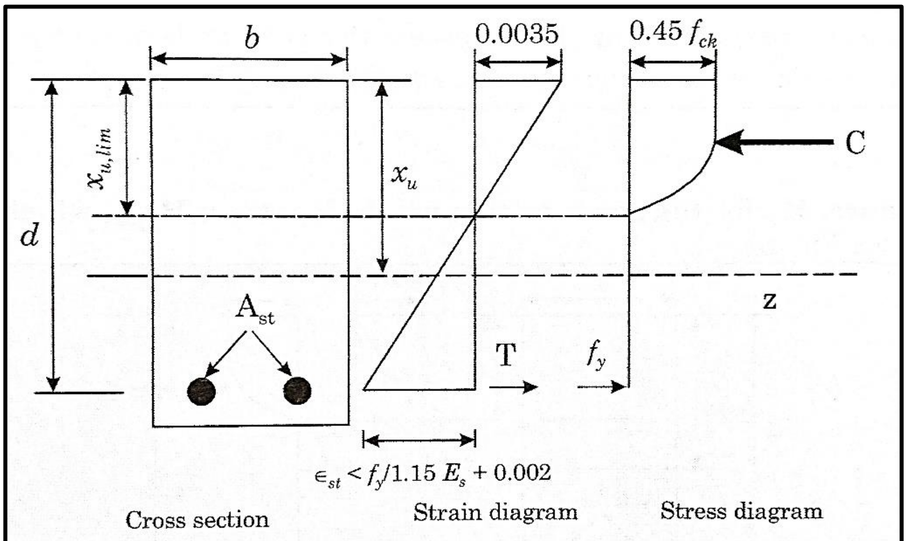
- If $P_t < P_{t\ lim}$, section is under reinforced
- If $P_t > P_{t\ lim}$, section is over reinforced
- If $P = P_{t\ lim}$, section is balanced
- Value of $P_{t\ lim}$ and $\frac{M_{u\ lim}}{bd^2}$ for singly reinforced rectangular beam section for various grades of steel and concrete is given as under.

	$\frac{x_{u\ lim}}{d}$	$R_u = \frac{M_{u\ lim}}{bd^2}$	$P_{t\ lim}$
Fe 250	0.53	0.148 f_{ck}	0.088 f_{ck}
Fe 415	0.48	0.138 f_{ck}	0.0479 f_{ck}
Fe 500	0.46	0.133 f_{ck}	0.038 f_{ck}

- As the code says $x_u \le x_{u\ lim}$, hence $P_t \le P_{t\ lim}$ and $M_{uR} \le M_{uR\ lim}$

$$M_u = 0.87\, f_y A_{st}\, d \left(1 - \frac{f_y A_{st}}{f_{ck}\, bd}\right)$$

iii) If ($\frac{x_u}{d} > \frac{x_{u\,lim}}{d}$) i.e. the section is over-reinforced. The section may be changed as it violates the assumption that the maximum strain in tensile reinforcement shall not be less than $\frac{fy}{1.15E_s} + 0.002$



Strain-stress diagrams for cross-section when $x_u/d > x_{u,\,\mathrm{lim}}/d$.

As per recommendation of IS code x_u should not be more than $x_{u\,lim}$ hence depth of N.A. is limiting to $x_{u\,lim.}$ This automatically restricts the use of over reinforced section and moment of resistance is thus taken as max of $M_{u\,lim}$

To determine steel area in Tension when concrete cross-section & applied moment are known

$$M_u = M_{u\,applied}$$

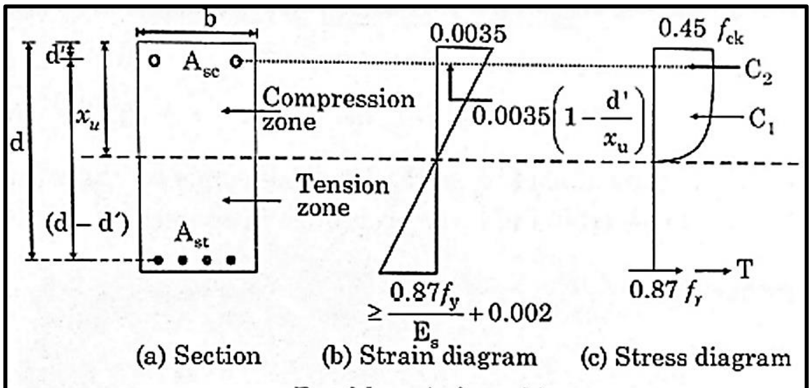
$$M_u = 0.87\, f_y A_{st}\, d \left(1 - \frac{f_y A_{st}}{f_{ck}\, bd}\right)$$

From this relationship A_{st} is calculated.

DOUBLY REINFORCED BEAMS

- If section dimensions are given and Mu > Mu lim provision of singly reinforced section will make the beam over reinforced.
- Hence, section dimension needs to be modified or higher grade of steel/concrete should be used.
- However, if section dimension cannot be modified (preferably depth) we need to provide steel on compression side also so that the neutral axis does not shift downwards even by providing steel in tension greater than the $A_{st\ lim}$. Thus, we will have to provide doubly reinforced section.
- Doubly reinforced section is also used in situation where reversal of moments is likely to occur (example: multistorey frame subjected to lateral loads).
- Other advantage of using compression reinforcement is reduction in long term deflection due to shrinkage and creep.
- While providing compression reinforced, it should be ensured that the compression reinforcements are enclosed by closed stirrups in order to prevent their possible buckling and to provide some ductility by confinement of concrete.

ANALYSIS OF DOUBLY REINFORCED SECTION



Doubly reinforced beam

$\frac{d'}{d}$ Should be normally between 0.05 – 0.2

- Moment of resistance of the section is

$$M_{uR} = 0.36\, f_{ck}\, b\, x_u\, (d - 0.42x_u) + A_{sc}\, (f_{sc} - f_{cc})\, (d - d')$$

- As f_{sc} and f_{cc} depends on the value of 'x_u' and x_u itself depends on the value of f_{sc} and f_{cc} we cannot have a direct solution. Hence trial and error procedure is adopted

Trial and Error Procedure

- Assume some value of x_u,
- Find strain in compression steel, 0.0035 ($\frac{x_u - d'}{x_u}$) and hence find f_{sc} and f_{cc} from the stress-strain curve of steel and concrete respectively for the assumed x_u.
- In equation

$$0.36\, f_{ck}\, b\, x_u + A_{sc}\, (f_{sc} - f_{cc}) = 0.87\, f_y\, A_{st}$$

L.H.S. R.H.S.

If LHS = RHS, the value of x assumed is correct otherwise repeat with different value of x_u till we get LHS=RHS. i.e., when we get correct value of x_u.

- Check if $x_u \leq x_{u\,lim}$, if yes, moment of resistance can be calculated from

$$M_{uR} = 0.36\, f_{ck}\, b\, x_u\, (d - 0.42\, x_u) + A_{sc}\, (f_{sc} - f_{cc})\, (d - d') \; \ldots.. \; (A)$$

- If $x_u > x_{u\,lim}$, Moment of resistance is found out by replacing x_u by $x_{u\,lim}$ in the above equation.

DESIGN OF DOUBLY REINFORCED RECTANGULAR SECTION

A doubly reinforced section may be designed by assuming it to be consisting of two beams (A) and (B)

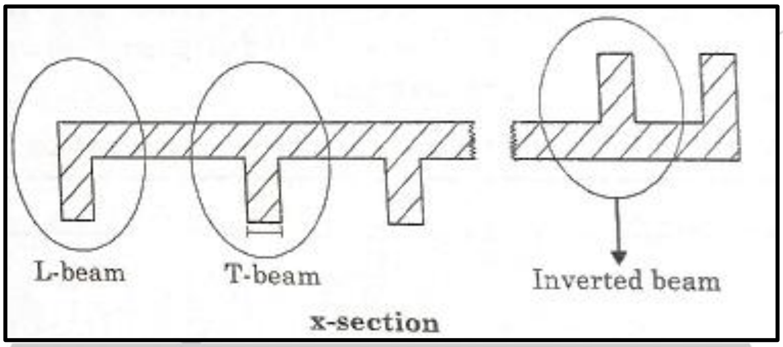
$$A_{st2} = \frac{f_{sc} A_{sc}}{0.87 f_y} \quad \ldots\text{(G)}$$

f_{sc} is found out from stress strain curve of steel with strain equal to 0.0035 $\left(\frac{x_{u\,lim} - d'}{x_{u\,lim}}\right)$

FLANGED BEAM

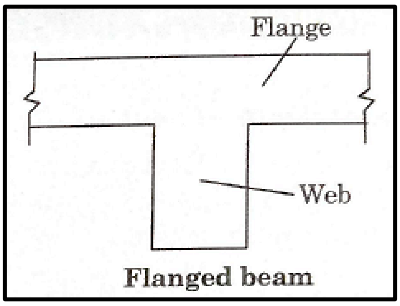
Introduction

- In monolithic construction, slab and beams are cast together. If slab in such cases is in compression zone they become effective (partially or wholly) in adding significantly to the area of concrete in compression in beam. However if flanges (slab) are located in tension zone, concrete in the flange (slab) becomes ineffective in cracked section analysis.



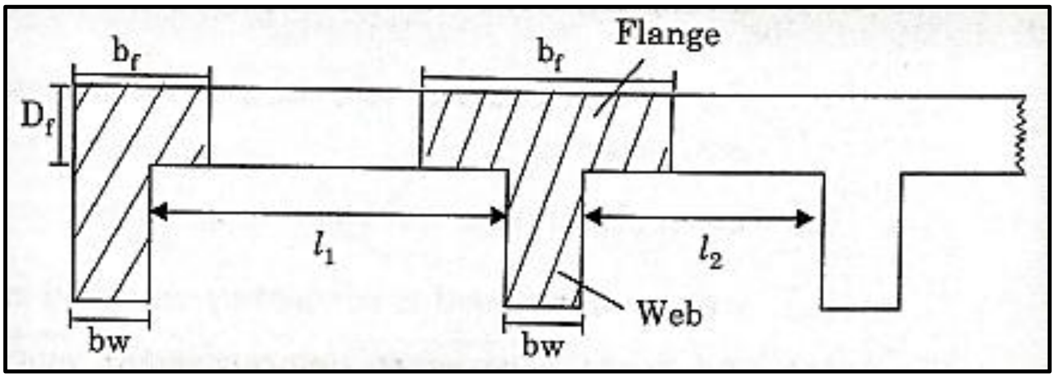
- In support region of a continuous beam, B.M is negative (-ve) i.e., hogging and slab therefore is in tension. Thus beam in that region even if cast monolithically, is designed as rectangular beam.

- Away from the support region, the slab will be in compression (BM being sagging). Hence in this region, beam is designed as flanged beam in which part of the slab is taken as a part of beam.



IS CODE SPECIFICATIONS

Effective Flange width for T and L beams



L - Beam

$$b_f = b_w + 3D_f + \frac{l_0}{12}$$

$$b_f \ngtr b_w + \frac{l_1}{2}$$

T - Beam

$$b_f = b_w + 6D_f + \frac{l_0}{6}$$

$$b_f \ngtr b_w + \frac{l_1 + l_2}{2}$$

Where,

b_f = effective width of the flange,

b_w = breadth of the web,

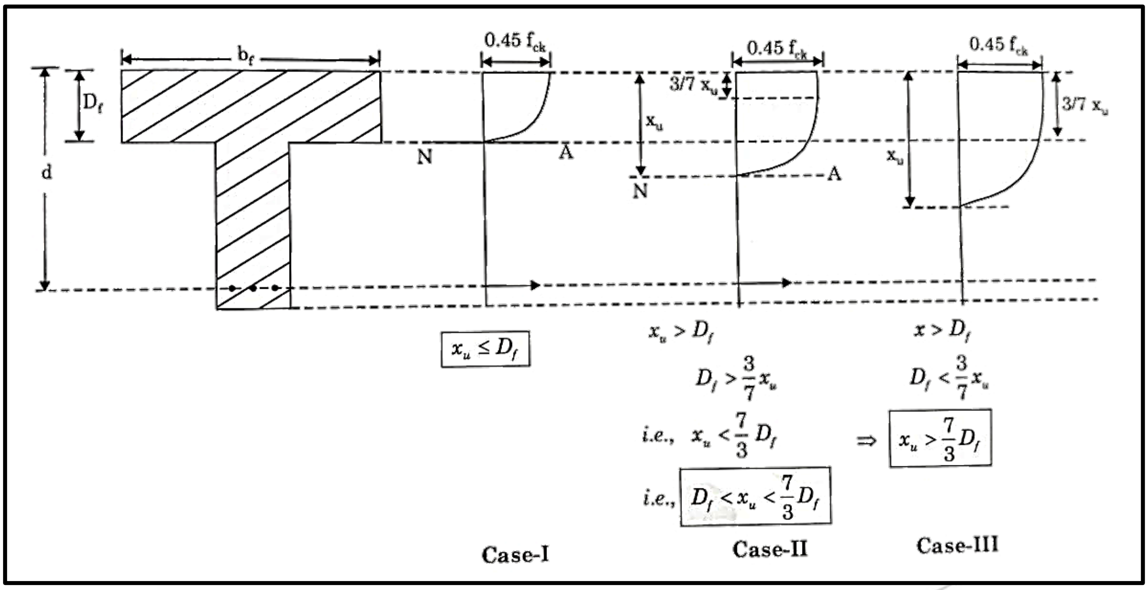
D_f = thickness of the flange,

l_0 = distance between point of zero moment in the beam

l_0 = effective span of beam for simply supported (SS) beam and

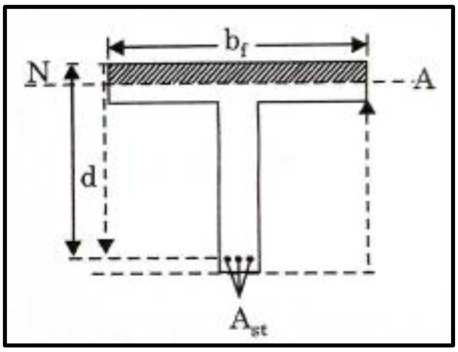
= 0.7 x (effective span of beam) for continuous beam

- If $x_u \le D_f$, the beam can be thought of as a rectangular section of width b_f.
- The stress distribution for various value of x_u is as shown in figure below.



Case 1: Neutral Axis lies in Flange

- When $x \le D_f$ Neutral axis lies in flange
- Thus portion below N.A. is not effective in tension
- Hence analysis and design will be as per a rectangular beam of width 'b_f' and depth'd'.
- To check whether the neutral axis lies in flange or not, we will have to compare flange force and tension as below.



- To start with let us assume that neutral axis lies at the bottom of flange.

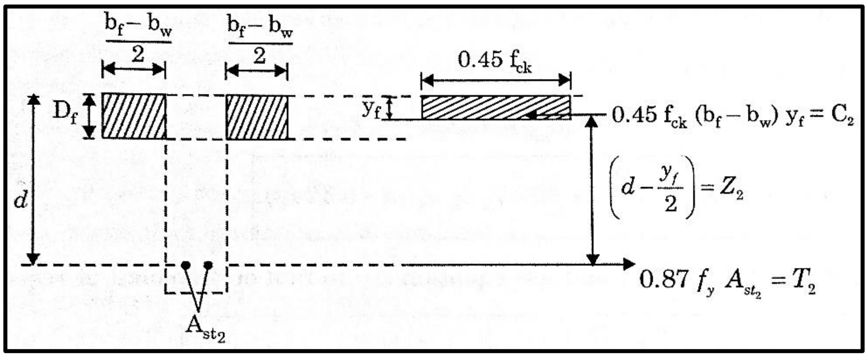
GPSC - CIVIL

Design of Steel Structures

"Shoot for the Moon. Even if you miss, you will land among the Stars."

Les Brown

The content of this book covers all PSC exam syllabus such as MPSC, RPSC, UPPSC, MPPSC, OPSC etc.



- With the above stress distribution, if we find the neutral axis of the flanged beam we have

$$C_1 + C_2 = T_1 + T_2$$

$$0.36\ f_{ck}\ b_w\ x_u + 0.45\ f_{ck}\ (b_f - b_w)\ y_f = 0.87\ f_y\ (A_{st1} + A_{st2})$$

$$0.36\ f_{ck}\ b_w\ x_u + 0.45\ f_{ck}\ (b_f - b_w)\ (0.65\ D_f + 0.15\ x_u) = 0.87\ f_y\ A_{st} \quad \text{..... (A)}$$

- Find the value of x_u from equation (A) above and check it satisfies.

Condition (α): $D_f < x_u < \frac{7}{3} D_f$

Condition (β): $x_u < x_{u\ lim}$

If both (α) and (β) are satisfied, moment of the section is given by

$$M_{uR} = C_1 Z_1 + C_2 Z_2$$

$$M_{uR} = 0.36\ f_{ck}\ b_w\ x_u\ (d - 0.42 x_u) + 0.45\ f_{ck}\ (b_f - b_w) y_f\ (d - \frac{y_f}{2}) \text{... (B)}$$

- If (α) is satisfied and (β) not satisfied.

Take $x_u = x_{u\ lim}$ and use equation (B) to find out moment of resistance.

$$\mathbf{M_{u\ lim} = 0.36\ f_{ck}\ b_w\ x_{u\ lim}\ (d - 0.42\ x_{u\ lim}) + 0.45\ f_{ck}\ (b_f - b_w)\ (0.65\ D_f + 0.15\ x_{u\ lim})\ (d - \frac{0.65\ D_f\ 0.15\ x_{u\ lim}}{2})} \text{ ....(B)}$$

$$\frac{D_f}{d} > 0.227$$

With Fe 415 grade steel

$$0.48d < \frac{7}{3}D_f$$

$$\frac{D_f}{d} > \frac{0.48 \times 3}{7}$$

$$\frac{D_f}{d} > 0.2057$$

With Fe 500 grade steel

$$0.46d < \frac{7}{3}D_f$$

$$\frac{D_f}{d} > 0.197$$

- On this basis of above analysis, it can be concluded that for $\frac{D_f}{d} > 0.2$, $x_{u\,lim} < \frac{7}{3}D_f$

Thus, IS code recommends that limiting MOR for $\frac{D_f}{d} > 0.2$ should be calculated as

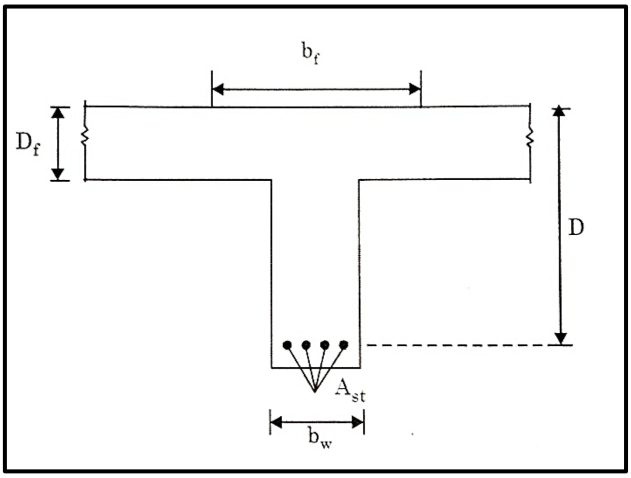
$$M_{u\,lim} = 0.36\, f_{ck}\, b_w\, x_{u\,lim}\, (d - 0.42\, x_u) + 0.45\, f_{ck}\, (b_f - b_w)\, y_f\, (d - \frac{y_f}{2})$$

$$\text{Where } y_f = 0.15\, x_{u\,lim} + 0.65\, D_f$$

- Similarly, it can be shown that for $\frac{D_f}{d} < 0.2$, $x_{u\,lim} > \frac{7}{3}$ Df, for all grades of steel.

  Hence IS code recommends that for $\frac{D_f}{d} < 0.2$, limiting moment of resistance should be calculated as

$$M_{uR\,lim} = 0.45\, f_{ck}\, (b_f - b_w)\, D_f\, (d - \frac{D_f}{2}) + 0.36\, f_{ck}\, b_w\, x_{u\,lim}\, (d - 0.42\, x_{u\,lim})$$



- To ascertain the zone in which the neutral axis lies, we need to calculate MOR at x= D_f and at x= $\frac{7}{3}D_f$

  At x_u= D_f, $M_{uR\,(x=Df)} = 0.36\, f_{ck}\, b_f\, D_f\, (d\text{-}0.42D_f)$

  If $M_{u\,(\text{factored moment})} < M_{uR\,(x=Df)}$ neutral axis will be in flange.

- Hence design will be like a singly reinforced rectangular section of width of b_f provided that $x_u < x_{u\,lim}$ and A_{st} will be calculated as

$$A_{st} = 0.87\, f_y\, A_{st}\, (d\text{-}\ \frac{f_y\, A_{st}}{f_{ck} b_f}) \ \ldots.(A)$$

  Find A_{st} from (A)

- At x_u= $\frac{7}{3}D_f$

$$M_{uR\,(x=\frac{7}{3}D_f)} = 0.45\, f_{ck}\, (b_f - b_w)\, D_f\ \ (d - \frac{D_f}{2}) + 0.36\, f_{ck}\, b_w\, (\frac{7}{3}D_f)\, (d - 0.42 \times \frac{7}{3}D_f)$$

$$T = C_1 + C_2$$

$$0.87\ f_y A_{st} = 0.45\ f_{ck} (b_f - b_w) D_f + 0.36\ f_{ck}\ b_w\ x_u \ \ldots \qquad (D)$$

Find A_{st} from this equation (D)

- In all the above cases if x_u obtained is greater than $x_{u\ lim}$, i.e if M_u (factored moment) is more than $M_{u\ lim}$, either depth of the section can be increased or doubly reinforced section should be chosen.
- However, doubly reinforced section is rarely needed in case of flanged section.

Qu5 In a singly reinforced beam, if the concrete is stressed to its allowable limit earlier then steel, the section is said to be

a) Economical section

b) Over reinforced section

c) Balanced section

d) Under reinforced section

TEST YOUR SELF

Qu6 The maximum area of tension reinforcement in beams shall not exceed _________

a) 2%

b) 4%

c) 0.15%

d) 1.5%

Qu7 The width of the flange of a T- beam should be less than

a) One-third of the effective span of the T-beam

b) Distance between the centers of T-beam

c) Breadth of the rib plus twelve times the thickness of the slab

d) Least of the above

Answer

1-(c), 2-(a), 3-(d), 4-(c), 5-(b), 6-(b),7-(d)

As shown in fig. A bar of diameter is embedded in concrete. To find L_d, pull out test is performed.

Let, P = Pull put force

 Bond resistance of concrete = strength of bar

$$\tau_{bd} \times (\pi \times \phi \times L_d) = \sigma_s \times \frac{\pi}{4} \times \phi^2$$

$$\mathbf{L_d = \frac{\phi\, \sigma_s}{4.\tau_{bd}}}$$

BOND STRESS (τ_{bd})

The shear force acting per unit surface area of the bar, in the direction of force is known as bond stress

Table: Design bond stress (τ_{bd}) for plain bars in tension (Ref. IS: 456-2000, P. 43)

Grade of concrete	M 20	M 25	M 30	M 35	M 40 and above
Design bond stress τ_{bd} , N/mm^2	1.2	1.4	1.5	1.7	1.9

- For deformed bars, increase the value by 60%
- For compression, increase the value by 25%
- If development length criteria is not satisfied, provide smaller diameter bars, more in number.

TYPES OF STIRRUPS

Vertical Stirrups

Normally, vertical stirrups are provided 2-legged. But, for heavy shear force 4 legged or 6 legged stirrups may be used.

$$S_V = \frac{A_{SV}\,(0.87\,f_y)}{0.4\,b}$$

Where,

A_{sv} = c/s area of stirrup legs

s_V = spacing of stirrups

B = breadth of the beam

f_y = characteristics Strength of stirrups

$$\tau_{max} = 0.625\,\sqrt{f_{ck}}\ \mathrm{N/mm^2}$$

(b) If $\tau_v > \tau_c$

Shear reinforcement is to be designed.

i) Calculate S.F. in shear reinforcement.

$$V_{us} = V_u - \tau_c.\,bd$$

(IS: 456-2000, P73)

V_{us} = Strength of shear reinforcement

If bent up bars are provided find V_{us1},

$$V_{us1} = 0.87\,f_y.\,A_{sv}.\sin\alpha$$

V_{us1}= strength of bent up bars

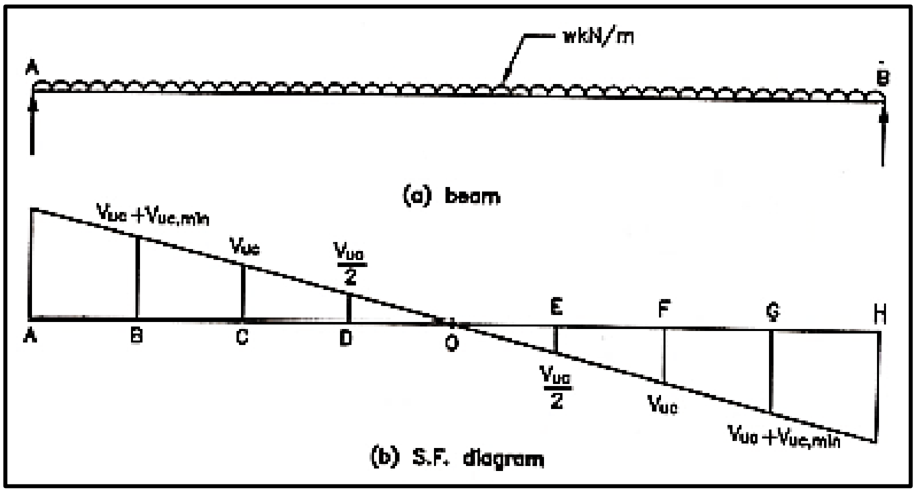
α = angle of bent up bar (45°)

A_{sv} = area of bent up bars

f_y = Characteristics strength of bent up bars

$$\textbf{Net } V_{us} = V_{us} - V_{us1}$$

$$V_{us1} \ngtr 0.5\,V_{us}$$



In the central region C-D-O-E-F where S.F. is less than V_{uc}, theoretically no shear reinforcement is required. However, minimum shear reinforcement is provided in this region, as per code. The length C-D-O-E-F of the beam may be called as the zone of minimum shear reinforcement.

Let,

$V_{us,min}$ = Shear capacity of minimum shear reinforcement

As per IS: 456-2000, Cl. 26.5.1.6, where the maximum shear stress calculated is less than half the permissible value (i.e. $\tau_v \leq \frac{\tau_c}{2}$), the criteria of minimum shear reinforcement need not be compiled. Thus, in region D-O-E where $\tau_v \leq \frac{\tau_c}{2}$, the spacing of shear reinforcement may be increased than that required for minimum shear reinforcement. However, in practice, in region D-O-E also, minimum shear reinforcement is provided

At any section just left of C or just right of F, the shear stress $\tau_v > \tau_c$ and shear reinforcement shall be designed.

But, the shear taken by stirrups $V_{us} = V_u - \tau_c.bd$ is small and hence we get designed shear reinforcement as minimum shear reinforcement. Thus, in region B-O-G minimum shear reinforcement may be sufficient.

In region AB or GH, the designed shear reinforcement more than minimum is required. In this region,

$$V_u > V_{uc} + V_{us,\,min}$$

CLEAR YOUR CONCEPT

Qu1 The length of the straight portion of a bar beyond the end of the hook should be at least

a) Twice the diameter

b) Thrice the diameter

c) Four times the diameter

d) Seven times the diameter

Qu2 If a beam fails in bond, then its bond strength can be increased most economically by

a) Increasing the depth of beam

b) Using thinner bars but more in number

c) Using thicker bars but less in number

d) None of the above

Qu3 Spacing of stirrups provided in a simply supported concrete beam rectangular cross section carrying uniformly distributed loads

a) Kept constant throughout the length of the beam

b) Decreased towards the center of the beam

c) Increased towards the end supports

d) Increased towards the mid span

Qu4 Shear reinforcement is provided in the form of

a) Vertical bars

b) Inclined bars

c) Combination of verticals and inclined

d) Any one of the above

CHAPTER – 5

SLABS

TYPES OF SLABS

A slab is a plate element having depth (D), very small as compared to its length and width. Slab is used as floor or roof in buildings, carry uniformly distributed load.

Slab may be - simply supported

- continuous

- cantilever

Types of slabs based on support conditions are:

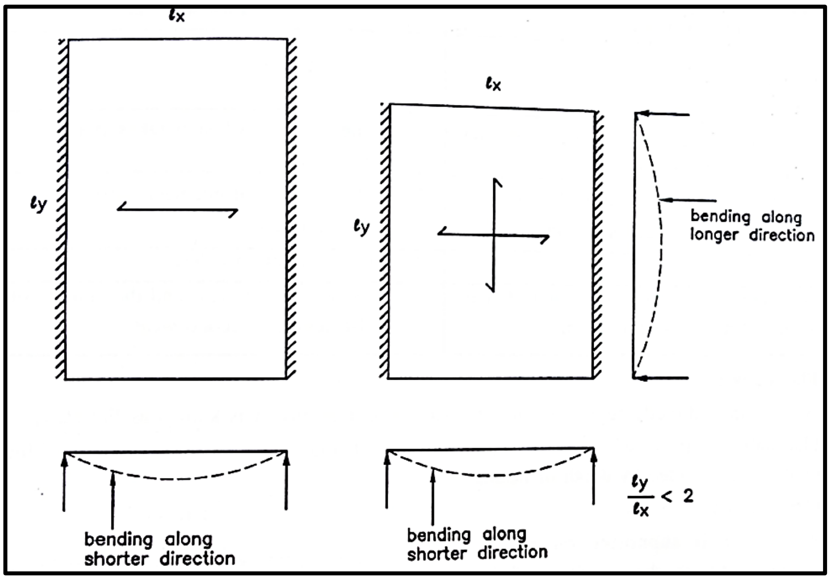
(a) One way spanning slab

(b) Two way spanning slab

(c) Flat slab resting directly on columns without beams.

(d) Grid slabs or waffle slabs

(e) Circular slabs and other shapes.



ONE WAY SLAB	TWO WAY SLAB
1. $\frac{l_y}{l_x} \geq 2$	1. $\frac{l_y}{l_x} < 2$
2. The bending of slab takes place in shorter direction (x-direction) only.	2. The bending of slab takes place in both the directions.
3. Main steel reinforcement is provided along l_x. Along l_y distribution steel is provided.	3. Main steel reinforcement is provided along both the directions.
4. Depth required is more.	4. Depth required is less.
5. As thickness is more and the amount of steel is also more, it is less economical.	5. As thickness is less and the amount of steel is also less, it is economical.

FLAT SLABS

- When slab is directly supported on columns, without beams, it is known as flat slab.
- Flat slabs are provided to increase the floor height and to permit large amount of light which might be obstructed by depth of beams.

GRID SLABS

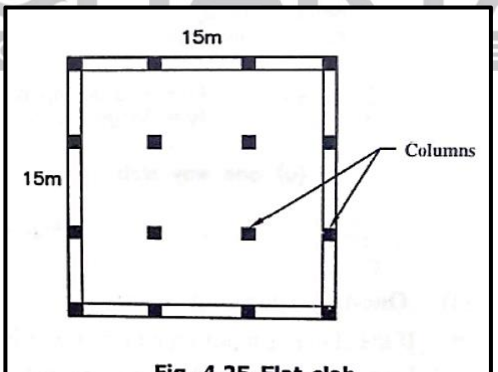


Fig. 4.25 Flat slab

- When slab is supported on beams with columns only on the periphery of the hall, the slab is called grid slab.
- Sometimes, in large halls, public places, marriage halls, auditoriums, etc. a large column free area is required. In these cases, large deep beams may be permitted but the columns are permitted only on periphery.

Assume % steel 0.3 to 0.6%

$$\mathbf{f_s = 0.58\ f_y} \times \frac{A_{st}\ required}{A_{st}\ required}$$

f_s = stress in steel at service loads

Initially assume that A_{st} required = A_{st} provided

$\therefore f_y = 250\ N/mm^2$ $\quad f_s = 0.58 \times 250 = 145\ N/mm^2$

$f_y = 415\ N/mm^2$ $\quad f_s = 0.58 \times 415 = 240\ N/mm^2$

$f_y = 500\ N/mm^2$ $\quad f_s = 0.58 \times 500 = 290\ N/mm^2$

(b) Effective Span

(IS : 456-2000, P. 34, Cl. 22.2(a))

(i) Clear span + d

(ii) c/c of supports

Whichever is smaller

(c) Reinforcement Requirements

(i) Minimum Reinforcement IS : 456-2000, P. 48, Cl. 26.5.2.1

For Fe-250 $\quad p_t$ = 0.15% of total c/s area (b × D)

For $\left.\begin{matrix} Fe\ 415 \\ Fe\ 500 \end{matrix}\right\}$ $\quad p_t$ = 0.12% of total c/s area

(Ii) Maximum Diameter

Maximum diameter of bar should not be more than $\frac{1}{8}$ × D.

where D = overall depth of slab.

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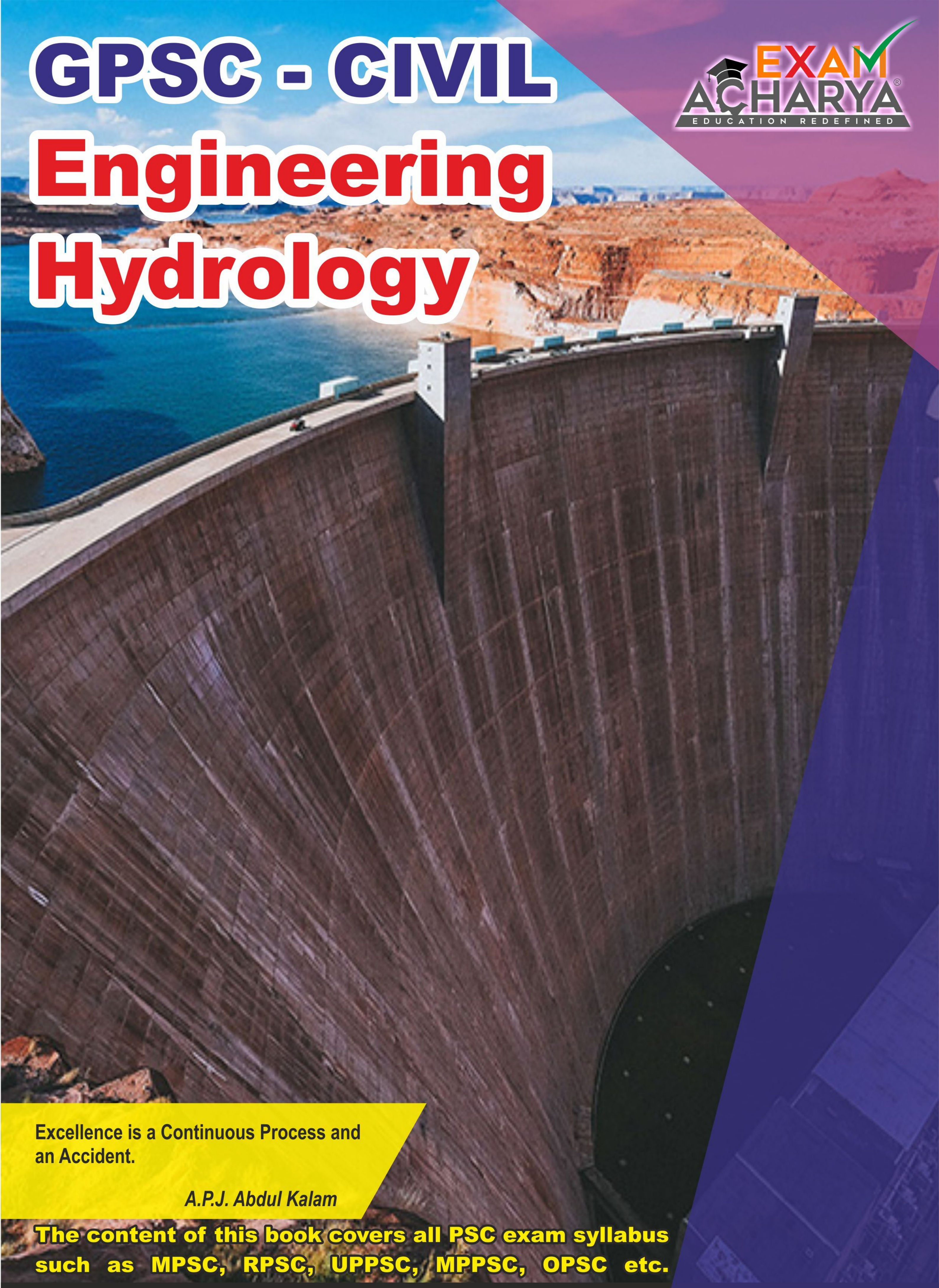
Engineering Hydrology

EXAM ACHARYA
EDUCATION REDEFINED



Excellence is a Continuous Process and an Accident.

A.P.J. Abdul Kalam

The content of this book covers all PSC exam syllabus such as MPSC, RPSC, UPPSC, MPPSC, OPSC etc.

If Actual $\frac{L}{D}$ < allowable $\frac{L}{D}$

$\therefore$ O.K.

Check for Development Length (L_d)

IS: 456-2000, P. 44, Cl. 26.2.3.3 (c)

$$L_d \text{ should be} \le 1.3\ \frac{M_1}{V} + L_o$$

where, $L_d = \frac{\emptyset.\sigma_s}{4.\tau_{bd}}$

$\sigma_s = 0.87\ f_y$ IS : 456-2000, P. 42 or SP. 16, P. 184.

50% steel is bent up near support. Therefore, find M.R. for 50% steel only.

$\therefore$ M_1 = M.R. for 50% steel at support

V = shear force at support

L_o = sum of anchorage beyond centre of support

(i) d

(ii) 12 ϕ

take L_o as the smaller of two values.

Hint:

For Main Steel

If A_{st} > 300 mm^2, use 10 ϕ bars

If A_{st} < 300 mm^2, use 8 ϕ bars

For Distribution Steel

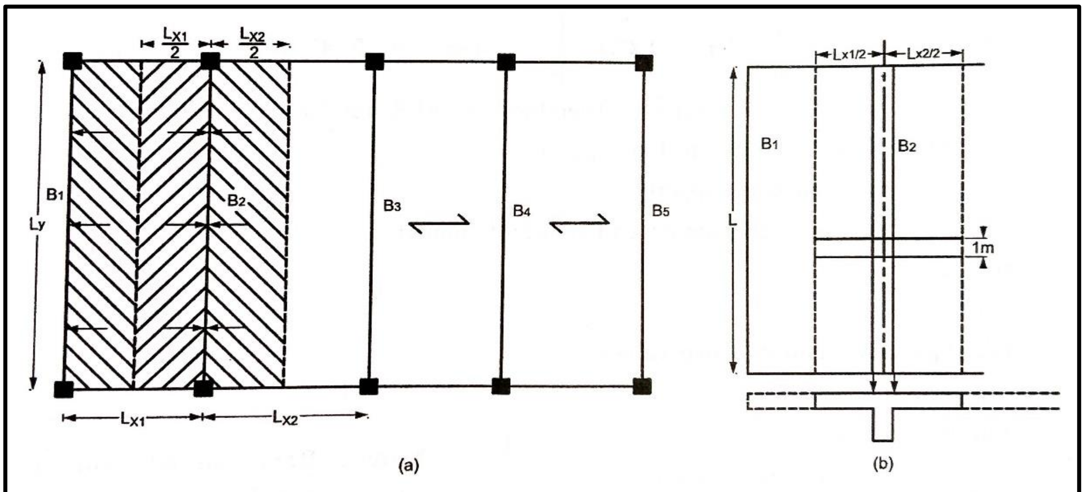
If A_{st} > 200 mm^2, use 8 ϕ bars

If A_{st} < 200 mm^2, use 6 ϕ bars

Fig. shows the area of the slab transferring the load to the beam.

Load on beam $B_1 = w. \frac{L_{x1}}{2}$ kN/m

Load on beam $B_2 = w. \left(\frac{L_{x1}}{2} + \frac{L_{x2}}{2}\right)$ ... if spans on either side of beam are unequal



Load on beam supporting one-way slab

Load on beam $B_2 = w. \left(\frac{L_x}{2} + \frac{L_x}{2}\right) = w. L_x$ …. if $L_{x1} = L_{x2} = L_x$

where, w = load on slab per m^2.

For obtaining ultimate load on beam multiply equivalent u.d.I.by load factor of 1.5.

<u>Load on Beam Supporting Cantilever Slab</u>

Load on beam per m = $w. L_x$ kN/m

In addition to this the beam is subjected to torsion of $\frac{w.L_x^2}{2}$.

For obtaining ultimate load on beam multiply w by load factor 1.5.

These trapezoidal and triangular loads are converted into equivalent uniformly distributed loads by using equivalence factors as given below:

Equivalent u.d.l. on beam 'A' for triangular load

$$w_e = \frac{w.L_x}{3} \quad \text{... for B.M.}$$

where,

w_e = Equivalent u.d.l. (kN/m)

w = Uniformly distributed load on slab (kN/m^2)

$$w_e = \frac{w.L_x}{4} \text{ ... for shear}$$

Equivalent u.d.l. on beam 'B' for trapezoidal load

$$w_e = \frac{w.L_x}{2}\left[1 - \frac{1}{3\beta^2}\right] \text{... For B.M.}$$

Where,

w = Intensity of load on slab

L_x = Short span of slab

L_y = Long span of slab

$$\beta = \frac{L_y}{L_x}$$

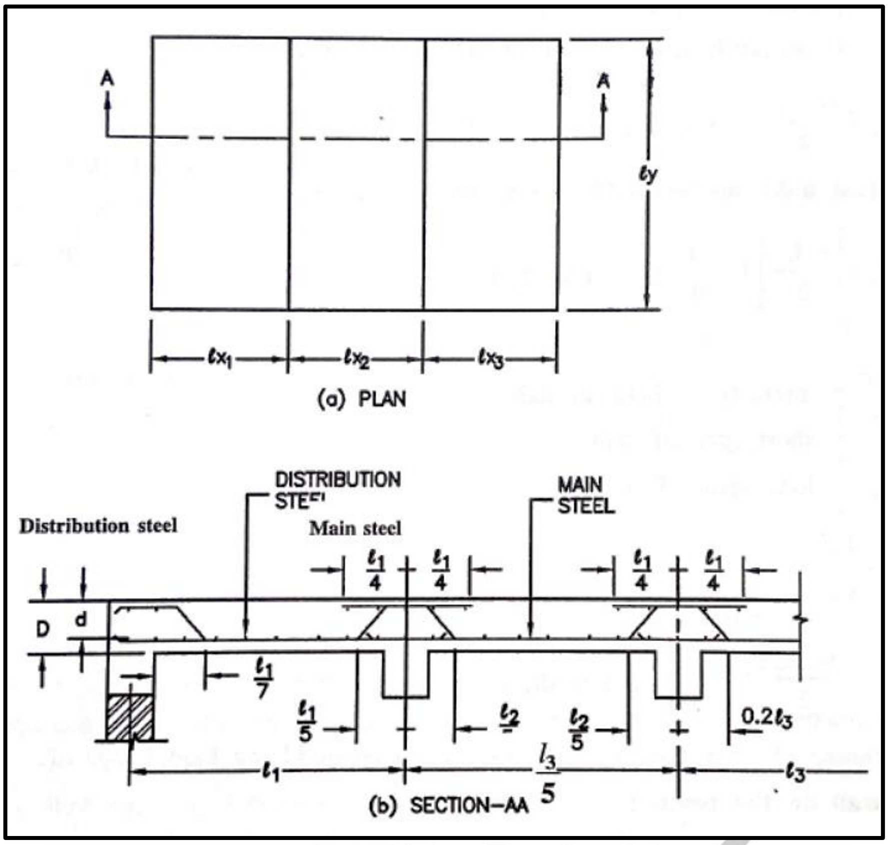
$$w_e = \frac{w.L_x}{2}\left[1 - \frac{1}{2\beta}\right] \text{ ... For shear}$$

For obtained ultimate load multiply the equivalent u.d.l. by load factor of 1.5.

Weight of Wall on the Beam

Load on beam due to wall = $(t_w \times H_w) \times \gamma$ kN/m

where,



Bending moment coefficients (Table 12 of IS 456)

Type of load	Span moments		Support moments	
	Near middle of end span	**At middle of interior span**	**At support next to the end support**	**At other interior supports**
Dead load and imposed load (fixed)	$+\frac{1}{12}$	$+\frac{1}{16}$	$-\frac{1}{10}$	$-\frac{1}{12}$
Imposed load (not fixed)	$+\frac{1}{10}$	$+\frac{1}{12}$	$-\frac{1}{9}$	$-\frac{1}{9}$

Note

- For obtaining bending moment, the coefficients shall be multiplied by the total design load and effective span.

As per IS: 456 - 2000, P.90, CI.D-2

$$M_x = \alpha_x . w . l_x^2$$

$$M_y = \alpha_y . w l_x^2$$

Where,

w = Total design load per unit area

α_x, α_y = Coefficients given in table-27 of IS : 456-2000.

M_x = Moment on strip of unit width spanning along l_x

M_y = Moment on strip of unit width spanning along l_y

l_x = Shorter span

l_y = Longer span

Control of Deflection

As per IS: 456 - 2000, P. 37, Cl. 23.2.1, deflection check for two-way slab is made in a similar manner as for one way slab.

However, when short span is less than 3.5 m, calculation is made as under.

- For two-way slab, for finding $\frac{l}{d}$ ratio, consider l_x.
- If short span (l_x) < 3.5 m

Live load < 3 kN/m^2

for mild steel, $\frac{l}{d}$ ratio is taken as

Simply supported slab = 35

continuous slab = 40

For Fe - 415 steel, above values of $\frac{l}{d}$ are multiplied by 0.8 (IS: 456 - 2000, P. 39)

- Rules for the spacing of reinforcement are similar to that of one-way slabs.

(1) Slabs are considered as divided in each direction into middle strips and edge strips. Fig.

(2) The maximum moments calculated (M_x and M_y) apply only to middle strips.

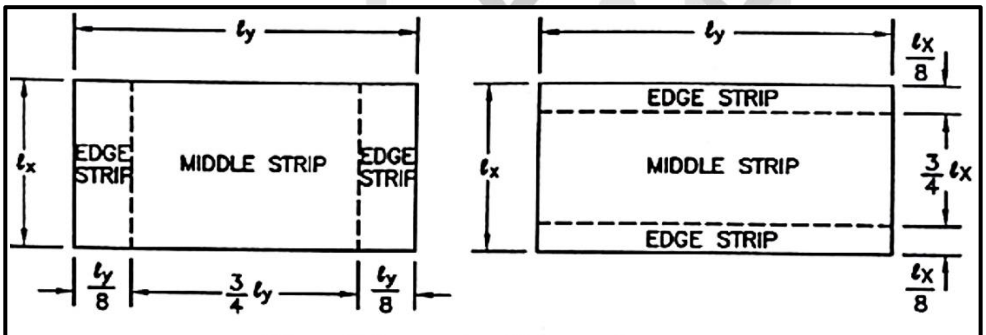
(3) Tension reinforcement provided at mid span shall extend in the lower part of the slab to within 0.25 l or (l/4) of a continuous edge or 0.15 l or (l/7) of a discontinuous edge.

(4) Over the continuous edge of a middle strip, the tension reinforcement shall extend in the upper part of the slab at a distance of 0.15 l or (1/7) from the support, and atleast 50 percent shall extend to a distance 0.3 l.

(5) Provide minimum reinforcement in the edge strip as per IS : 456-2000, P-48, Cl. 26.5.2.1.

For Fe – 250, 0.15% of total c/S area.

For Fe – 415 or Fe-500, 0.12% of total c/S area.



Edge Strips and middle strip

- B.M. coefficients for two way simply supported slab

  - based on Rankine Grassof theory

- B.M. coefficients for two way restrained slab

  - based on Johansen's yield line theory

- B.M. coefficients for continuous slab

  - based on Johansen's yield line theory.

WORKING STRESS METHOD

Modular Ratio

$$\mathbf{m} = \frac{E_s}{E_c} = \frac{280}{3\sigma_{cbc}}$$

Where σ_{cbc} = Permissible compressive stress due to bending in concrete

= 7 N/mm^2 …. for M 20 IS: 456-2000, P.81

= 8.5 N/mm^2 …. for M 25

σ_{st} = Permissible stress in steel

= 130 N/mm^2

$$\mathbf{k} = \frac{m.\sigma_{cbc}}{m.\sigma_{cbc} + \sigma_{st}}$$

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

$$\mathbf{Q} = \frac{1}{2}\sigma_{cbc}\,.\,\mathbf{k}.\,\mathbf{j}$$

$$\mathbf{M} = \mathbf{Q}.\,\mathbf{bd}^2$$

Tensile stress in steel = $\frac{F_t}{A_c + m.A_{st}}$

Maximum tensile stress in M 20 = 2.8 N/mm^2

CLEAR YOUR CONCEPT

Qu1 For slabs spanning in two directions, for calculating the span to effective depth ratios

a) Shorter span should be considered

b) Longer span should be considered

c) Average value of shorter and longer span should be considered

d) Both the spans should be considered in their respective directions

Qu2 The clear distance between the lateral restraints for a simply supported or continuous beam to ensure lateral stability not exceed

a) $60b^2$ or $250b^2/d$ whichever is more

b) $60b^2$ or $250d^2/b$ whichever is less

c) $60b$ or $250b^2/d$ whichever is more

d) $60b$ or $250b^2/d$ whichever is less

Qu3 Minimum spacing between horizontal parallel reinforcements of different sizes should not be less than

a) One diameter of thinner bar

b) One diameter of thicker b ar

c) Sum of the diameter of thinner and thicker bars

d) Twice the diameter of thinner bar

Qu4 The effective span of a simply supported slab is

a) Distance between the centers of the bearings

b) Clear distance between the inner faces of the walls plus twice the thickness of the wall

c) Clear span plus effective depth of the slab

d) None of these

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Transportation Engineering

END is not the end if fact E.N.D. means
" Effort Never dies"

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Qu8 The horizontal distance between parallel main reinforcements in RC slab shall not be more than

a) 4times effective depth of slab

b) 5times effective depth of slab

c) 3times effective depth of slab

d) 2times effective depth of slab

Answer

1-(a), 2-(d), 3-(b), 4-(a), 5-(b), 6-(b),7-(c), 8-(c)

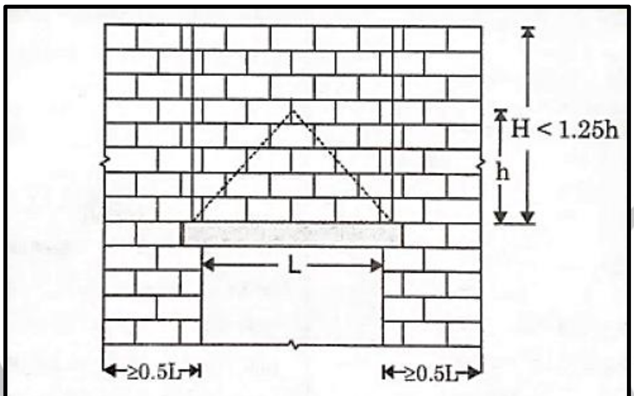
Case-II

For H < 1.25 h and length of side ≥ 0.5 L, full rectangular load is assumed to be carried by the lintel. That is arch action does not take place.

In the above case the load that is to be considered for design is the vertical load of the triangle of masonary.

Case-III

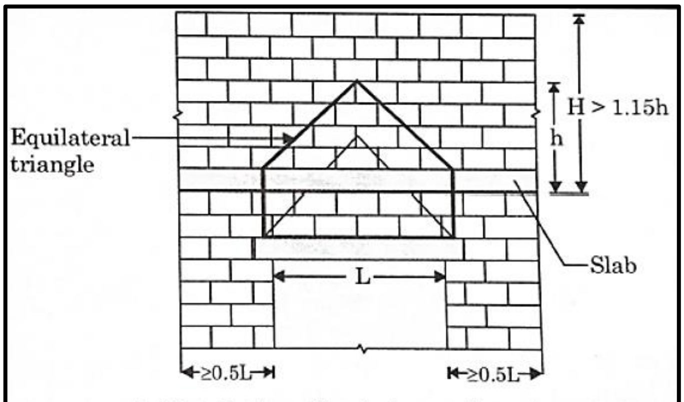
For H < 1.25 h and length of side ≥ 0.5 L, full rectangular load is assumed to be carried by the lintel. That is arch action does not take place.



(b) Rectangular distribution of load when H < 1.25h and walls on either side of opening ≥ 0.5L

Case-IV

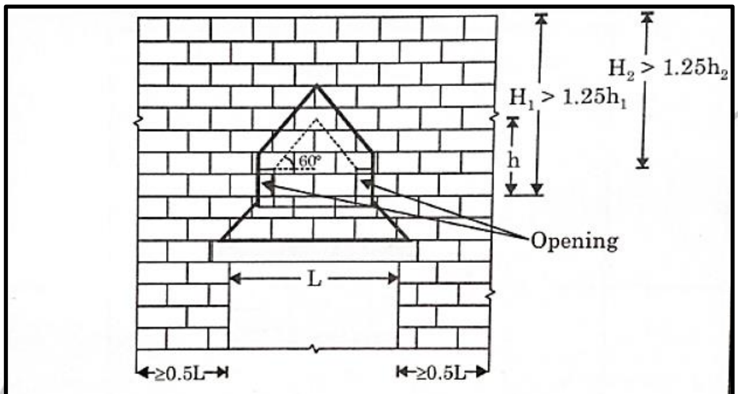
If H > 1.25h, but length of the side wall on one side is less than 0.5 times the length of lintel, vertical load of the square shape masonry above lintel is carried by the lintel.



(e) Distribution of load when wall carries a slab

Case-VII :

If an opening is encountered in the triangle masonry portion, the load transfer is given as



(e) Distribution of load when wall has opening above the lintel

IS 9893: 1981 gives the specification for lintels. Lintels are designed similar to beams as per the provisions of IS 456; 2000.

CHAPTER – 7

FOOTINGS

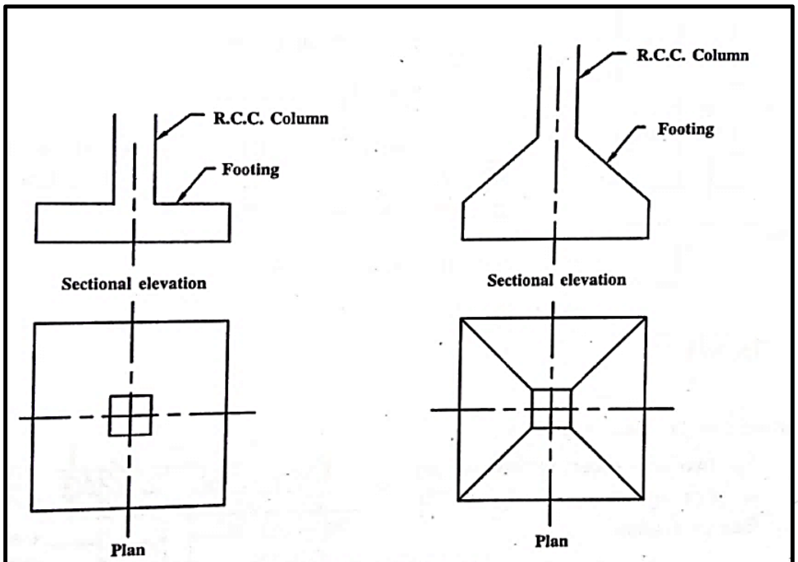
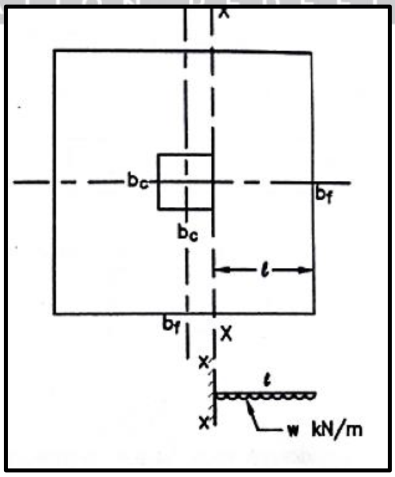


Fig. 4.19 Isolated Footings

CRITICAL SECTIONS

For isolated column footing, three sections are important.



Where,

V_u = S.F. on hatched area

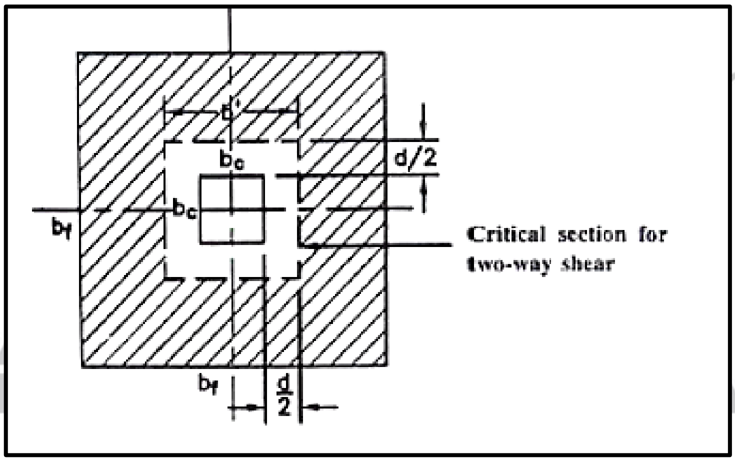
= p × b_f

If depth of footing (D), is less than 300 mm, the design shear strength of concrete is taken as k. τ_c, where value of k is taken from IS: 456 - 2000, P.72.

If $\tau_v < \tau_c$ ... safe in one way shear.

Critical Section for Two-Way Shear

- For two-way shear, critical section is taken at distance 0.5 d from the face of column.



d = effective depth of footing

For slopped footing

d = effective depth of footing at column face

V_u = S. F. on area outside the critical section

$$\mathbf{V_u = p \times \text{hatched area}}$$

Where

p = net upward pressure on hatched area = b_f^2 - b'^{2}

∴ Nominal shear stress (τ_v)

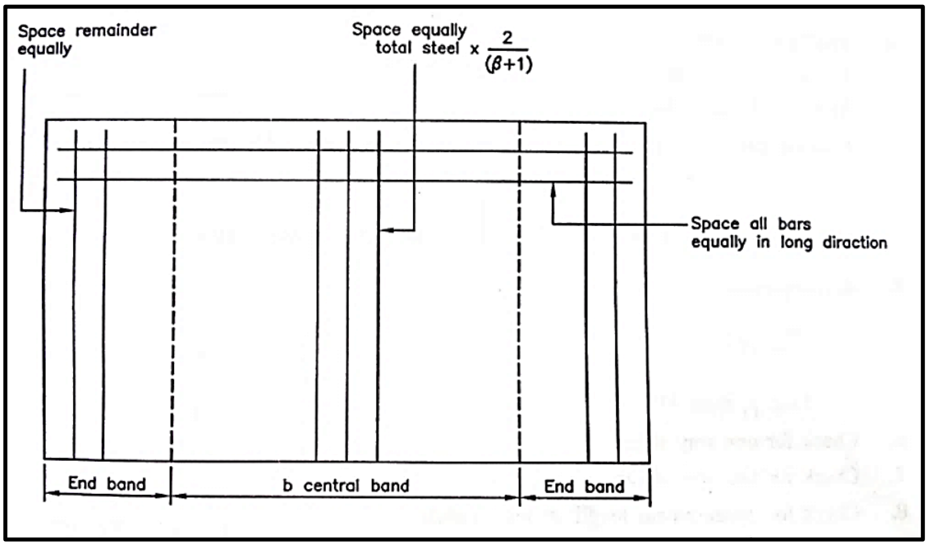
For reinforcement in the short direction a central band equal to width of footing shall be worked along the length of the footing and portion of the reinforcement determined in accordance with the equation given below shall be uniformly distributed across the central band :

$$\frac{\textit{Reinforcement in central band width}}{\textit{Total reinforcement in short direction}} = \frac{2}{\beta+1}$$

Where,

$$\beta = \frac{\textit{long side of footing}}{\textit{short side of footing}}$$

The remainder of the reinforcement shall be uniformly distributed in the outer portions of the footing.



Spacing of Bars (IS:456-2000, P.46, Table-15)

Redistribution of moment is not done in footing. Therefore, the clear distance between two bars shall not exceed the followings.

f_y N/mm^2	Clear distance between bars (mm)
250 N/mm^2	300 mm
415 N/mm^2	180 mm
500 N/mm^2	150 mm

take clear cover = 50 mm

Reinforcement

$$\frac{M_u}{bd^2} = Q$$

Find p_t from SP - 16

Check For One Way Shear

Check For Two Way Shear

Check For Development Length & Load Transfer

Qu5 When RCC footing is not to extend in the plot of the neighbouring house, the type of footing preferred is

a) Cellular flat not footing

b) Inverted flat not footing

c) Strap footing

d) Both (a) and (b)

TEST YOUR SELF

Qu6 Minimum clear cover (in mm) to the main steel bars in slab, beam, column, and footing respectively are

a) 10, 15, 20, 25

b) 15, 25, 40, 40

c) 20, 20, 40, 50

d) 20, 35, 40, 75

Qu7 An RC square footing of side length 2m and uniform effective depth 200mm is provided for a 300mm X 300mm column. The line of action of the vertical compressive load passes through the centroid of the footing as well as of the column. If the magnitude of the load is 320 KN, the nominal transverse (one way) shear stress in the footing is

a) 0.26 N/mm^2

b) 0.30 N/mm^2

c) 0.34 N/mm^2

d) 0.75 N/mm^2

Answer

1-(b), 2-(c), 3-(c), 4-(c), 5-(c), 6-(c), 7-(a)

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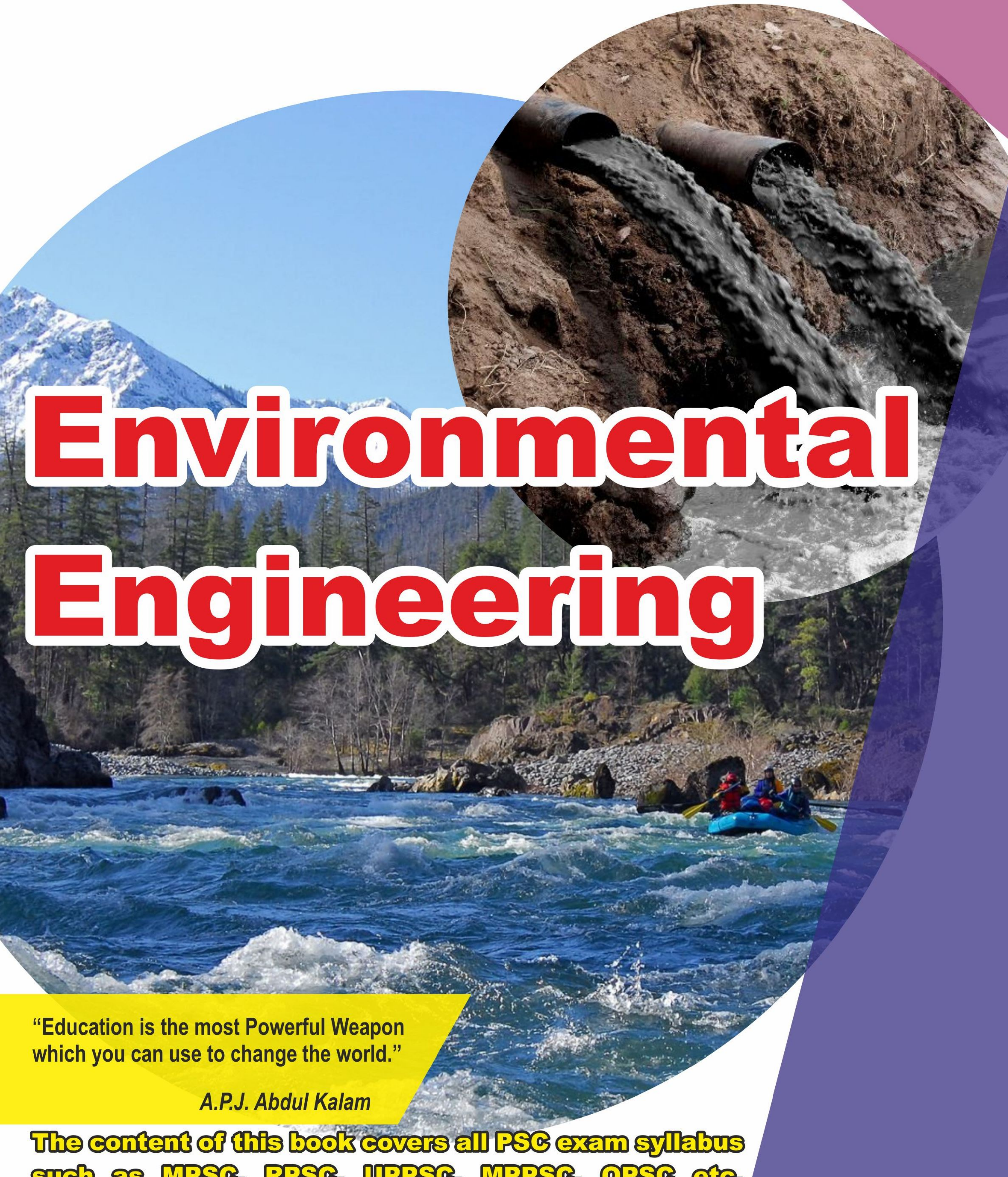

Environmental Engineering

"Education is the most Powerful Weapon which you can use to change the world."

A.P.J. Abdul Kalam

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Cantilever Retaining Wall

A cantilever retaining wall resists the earth pressure, horizontal and any other, by the cantilever bending action. Adequate reinforcement is provided to avoid development of tensile stresses beyond permissible limit in concrete. A cantilever can be further classified as a T shape cantilever wall and L shape cantilever wall. It is the most common type of retaining structure and is generally economical for heights upto about 8.0 m.

Counterfort Retaining Wall

It is a modified cantilever wall where the vertical stem as well as heel slab are strengthened by providing counterforts. This results in vertical stem and heel slab acting as a continuous slab. It is economical for heights above 7.0 m (approx.)

Buttress Wall

It is a modification over counterfort retaining wall where the counterfort is provided on the opposite side of the backfill. Although buttresses are structurally more efficient (and more economical) than counterforts. Buttress reduces the clearance in front of the wall and hence is not commonly used.

EARTH PRESSURE THEORY

Earth pressure theories are used to obtain the lateral earth pressure exerted by the backfill. These theories pertain to some of the oldest of researches done in the field of civil engineering. Various theories exist however Rankine's and Coulomb's earth pressure theory are most commonly studied for engineering application.

Rankine's Earth Pressure Theory

Rankine's earth pressure theory is applied to uniform cohesionless soil initially. Later its scope has been widened. The basic assumptions made are

1. The soil mass is semi-infinite, homogeneous dry and cohesionless.

Passive Earth Pressure

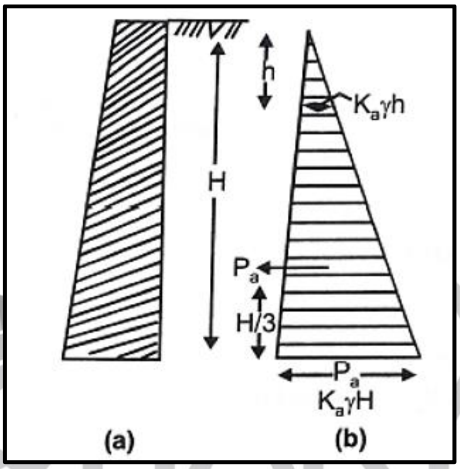
These are generated when the wall is pushed, pressed against the backfill that is the wall yields towards the backfill.

$$P_p = K_p \gamma H$$

K_p is the passive earth pressure coefficient

$$K_p = \frac{1+\sin\emptyset}{1-\sin\emptyset} = \frac{1}{K_a}$$

K_a and K_p are the ratio of lateral earth pressure to the vertical over burden.



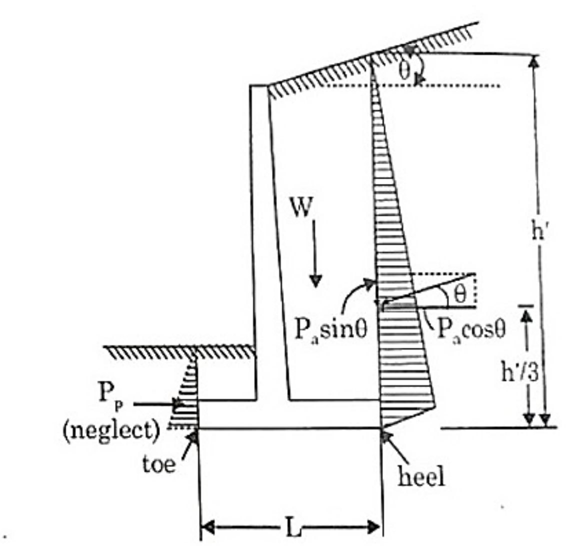
<u>Dry or moist backfill no surcharge</u>

Active earth pressure at bottom = $p_a = k_a \gamma H$

Total active earth pressure P_a **(per unit length) =** $\frac{1}{2} K_a \gamma H^2$

Backfill with Sloping Surface

$$K_a = \left[\frac{\cos\theta - \sqrt{\cos^2\theta - \cos^2\emptyset}}{\cos\theta + \sqrt{\cos^2\theta - \cos^2\emptyset}}\right]\cos\theta$$



Forces acting on a cantilever retaining wall

θ = angle of inclination of backfill

∅ = angle of repose of soil

- The direction of active pressure, P_a is parallel to the surface of the backfill.
- Maximum value of pressure (at heel) is $K_a \gamma h'$

h' = height of backfill measured vertically above the heel.

So total force on the wall on unit length of wall

$$p_a = \frac{K_a \gamma (h`)^2}{2}$$

P_a acts at a height $\frac{h`}{3}$ above the heel of an inclination of θ with the horizontal.

The passive pressure developed on the toe side is generally neglected.

Sliding

The horizontal (lateral) earth pressure causes the retaining wall to slide, if it has not developed sufficient friction.

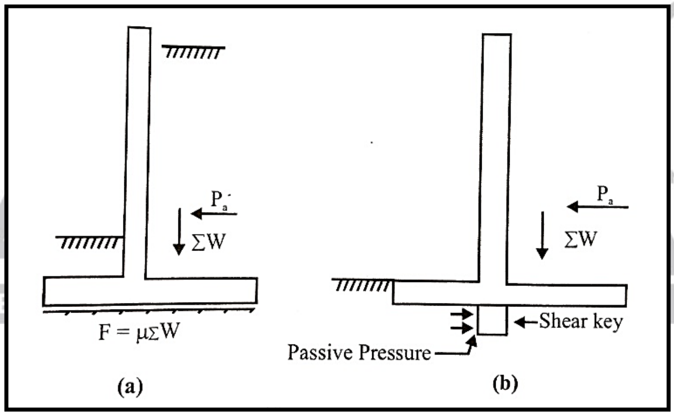
$$\textbf{Resisting Force} = F_R = (\sum w) \times \mu$$

$$\textbf{Factor of safety against sliding} = F_s = \frac{\mu(\sum w)}{H}$$

Here H = P_a

F_S must not be less then 1.5. If F□ < 1.5, a shear key is to be provided to resist sliding.

F_S must be greater than 1.0 when the earthquake forces are acting.



Tension Failure

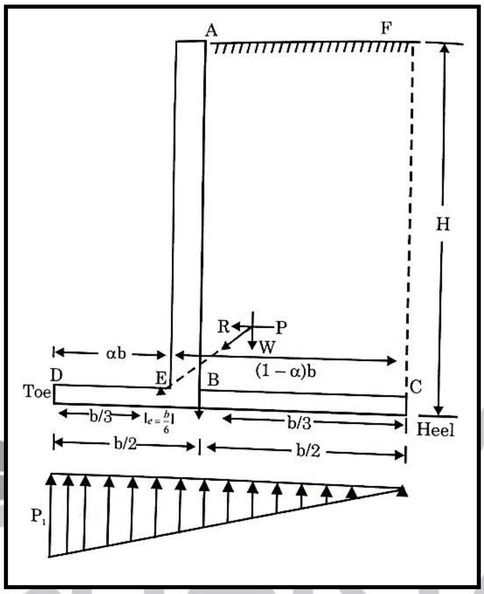
As concrete is weak in tension, retaining wall fails if excessive tension develops.

Let M = $\sum w_i \bar{x}_i$ - p_a H/3 be the net moment at base, now the point of application of resultant is given by

$$\bar{x} = \frac{\sum M}{\sum W}, \textbf{ eccentricity} = e = \frac{b}{2} - \bar{x}$$

2. The dimension of the retaining wall is determined so that the wall is safe against overturning and sliding. It is also to be ensured that the maximum and minimum stresses are within their permissible limits.

Let the width of toe slab be 'α b' and that of heel slab be (1-α)b where b is the total slab width that is to be found, assuming the unit weight of retaining wall and soil be 1. 1γ (where γ is the unit weight of soil alone). Neglecting the weight of toe slab, total weight W is given by



$$\mathbf{W = (1- \alpha)\, b \times H \times (1.1\gamma) = 1.1\gamma bH\, (1- \alpha)}$$

the point of application of this is mid-point of heel slab that is at a distance of $\frac{(1-\alpha)}{2}$ b from point C.

The horizontal pressure acting on the wall is given by P = $K_a\gamma\,\frac{H^2}{2}$ now, in order to avoid tension (p_2= 0) development at heel the resultant should pass through the outer third point. Hence moment about this point must be zero.

$$W\,[\frac{2b}{3} - \frac{1}{2}(1 - \alpha)\, b] - P\,\frac{H}{3} = 0$$

this gives b ≅ 0.95H $\sqrt{\frac{K_a}{(1-\alpha)(1+3\alpha)}}$, this gives the width of the wall once the 'α' is known.

CLEAR YOUR CONCEPT

Qu1 A cantilever retaining wall should not be used for heights more than

a) 4m

b) 6m

c) 8m

d) 10m

Qu2 The minimum thickness of reinforced concrete wall should be

a) 7.5cm

b) 10cm

c) 15cm

d) 12.5cm

Qu3 Which of the following RC retaining walls is suitable for height beyond 7m?

a) L-shaped wall

b) T- shaped wall

c) Counterfort type

d) All of the above

Qu4 For the design of retaining walls, the minimum factor of safety against overturning is taken as

a) 1.5

b) 2.0

c) 2.5

d) 3.0

CHAPTER – 9

COLUMN

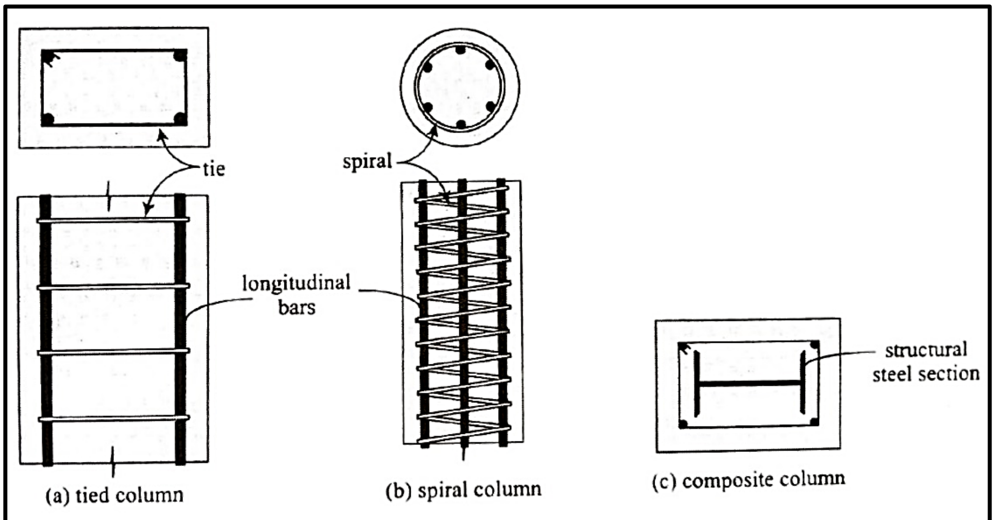
INTRODUCTION

- A 'compression member' is a structural element which is subjected predominately to axial compressive forces. Compression members are most commonly encountered in reinforced concrete buildings as columns (and sometimes as reinforced concrete walls).
- The column is representative of all types of compression members, and hence, sometimes, the terms column and compression member are used interchangeably.
- The code (Cl. 25.1.1) defines the column as a compression member, the effective length of which exceed three times the least lateral dimension.
- The term pedestal is used to describe a vertical compression member whose effective length is less than three times its least lateral dimension (Cl. 26.5.3.1) of the code.

Assumptions

The following assumptions are made for the limit state of collapse in compression:

1. Plane sections normal to the axis remain plane after bending.
2. The relationship between stress-strain distribution in concrete is assumed to be parabolic. The maximum compressive stress is equal to 0.67 $f_{ck}/1.5$ or 0.45 f_{ck}.
3. The tensile strength of concrete is ignored.
4. The stresses in reinforcement are derived from the representative stress-strain curve for the type of steel used.

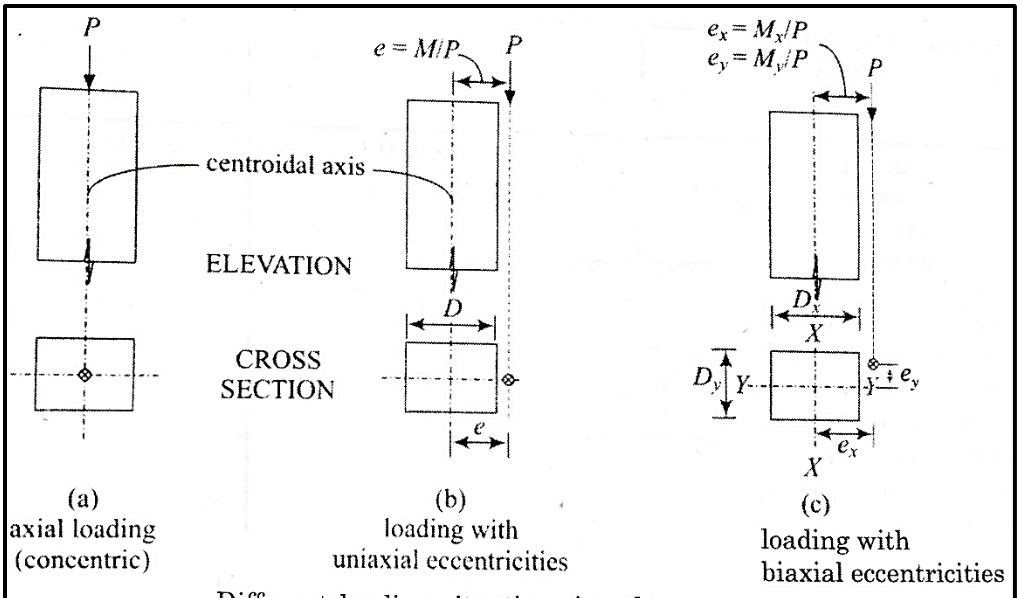


Types of columns—tied, spiral and composite.

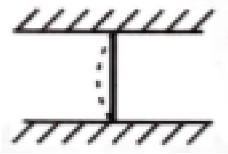
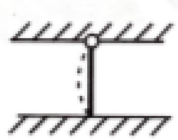
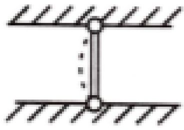
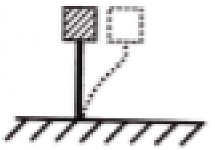
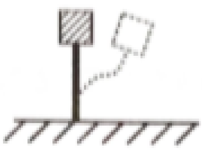
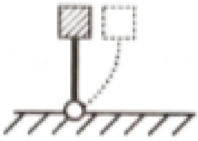
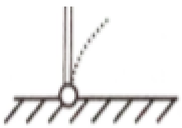
CLASSIFICATION OF COLUMNS BASED ON TYPE OF LOADING

Columns may be classified into the following three types, based on the nature of loading.

1. Columns with axial loading (applied concentrically)
2. Columns with uniaxial eccentric loading.
3. Columns with biaxial eccentric loading.



Different loading situations in columns.

Degree of end restraint of compression member	**Symbol**	**Theoretical value of effective length**	**Recommended value of effective length**
Effectively held in position and restrained		0.50 l	0.65 l
Effectively held in position at both ends, rotation at one end		0.70 l	0.80 l
Effectively held in position at both ends, but not restrained against rotation.		1.00 l	1.00 l
Effectively held in position and restrained against rotation at one end, and at the other restrained against rotation but not held in position.		1.00 l	1.2 l
Effectively held in position and restrained against rotation in one end, and at the other partially restrained against rotation but not held in position		-	1.5 l
Effectively held in position at one end but not restrained against rotation, and at the other end restrained against rotation but not held in position.		2.00 l	2.00 l
Effectively held in position and restrained against rotation at one end but not held in position nor restrained against rotation at the other end.		2.00 l	2.00 l

From above table it can be concluded that effective length of a column depends upon end condition height of column and does not depends upon span of beam it supports

Note

- l is the unsupported length of compression member. It is the clear distance between the floor. For a portal truss column fixed at the base, the point of contraflexure is assumed at a distance midway between the base and the foot of the knee-brace.

GPSC - CIVIL

Water Resource Engineering

EXAM ACHARYA
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"Don't Fear for Facing Failure in the First Attempt, Because even the Successful Maths Start with 'Zero' only."

A.P.J. Abdul Kalam

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CODE REQUIREMENTS ON REINFORCEMENT AND DETAILING

Longitudinal Reinforcement

Minimum Reinforcement

The longitudinal bars must, in general, have a cross-sectional area not less than 0.8% of the gross area of the column section. Such a minimum limit is specified by the Code:

- To ensure nominal flexural resistance under unforeseen eccentricities in loading
- To prevent the yielding of the bars due to creep and shrinkage effects, which result in a transfer of load from the concrete to the steel.

- In any column that has a larger cross-sectional area than that required to support the load, the minimum percentage of steel shall be based upon the area of concrete required to resist the direct stress and not upon the actual area.
- However, in the case of pedestals which are designed as plain concrete columns, the minimum requirement of longitudinal bars may be taken as 0.15 per cent of the gross area of cross-section.
- In the case of reinforcement concrete walls, the Code (Cl.32.5) has introduced detailed provisions regarding minimum reinforcement requirements for vertical (and horizontal) steel as given below
  - (i) The vertical reinforcement should not be less than 0.15% of the gross area in general.
  - (ii) This may be reduced to 0.12% if welded wire fabric or deformed bars (Fe 415/Fe 500 grade steel) is used, provided the bar diameter does not exceed 16 mm.
  - (iii) This reinforcement should be placed in two layers if the wall is more than 200 mm thick.

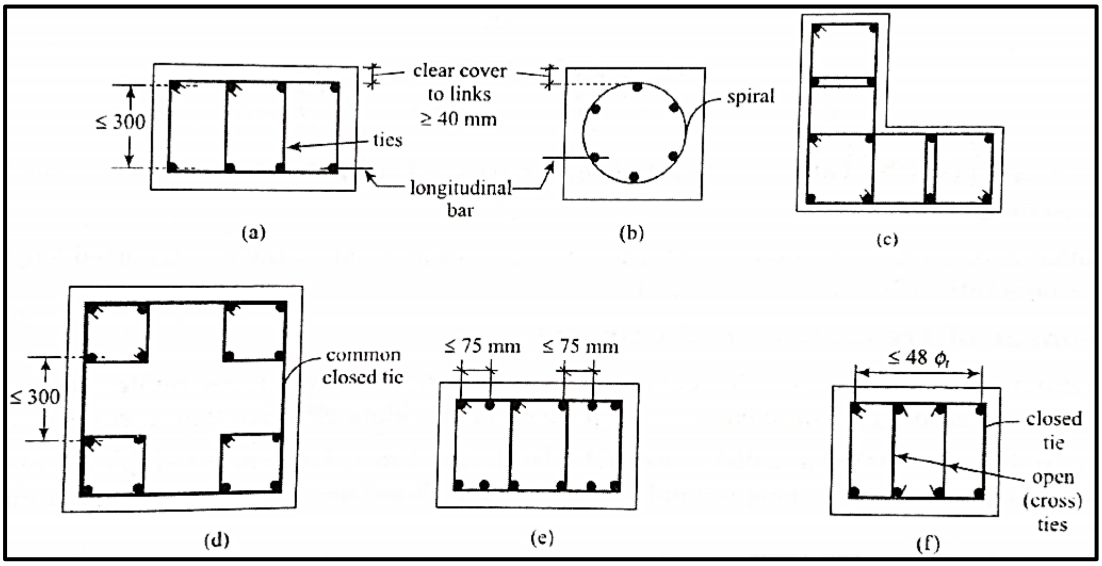


Fig. Some code recommendations for detailing in columns

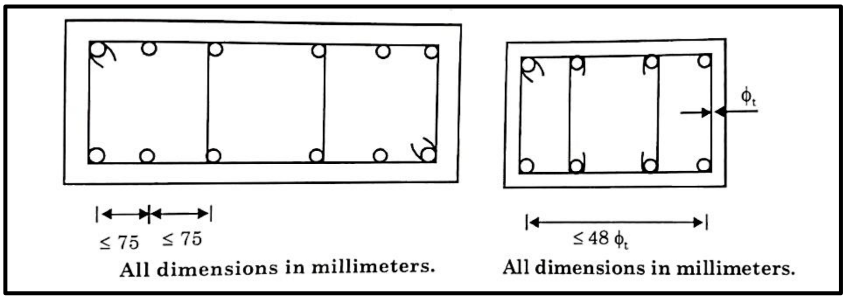
Cover to Reinforcement

A minimum clear cover of 40 mm or bar diameter (whichever is greater), to the column bars is recommended by the Code (Cl. 26.4.2.1) for columns in general a reduced clear cover of 25 mm is permitted in small-sized columns (D $\leq$ 200mm and whose reinforcing bars do not exceed 12mm) and a minimum clear cover of 15 mm (or bar diameter, whichever is greater) is specified for walls. However, in aggressive environments, it is desirable, in the interest of durability, to provide increased cover but preferably not greater than 75 mm.

TRANSVERSE REINFORCEMENT (Cl. 26.5.3.2 of the Code)

All longitudinal reinforcement in a compression member must be enclosed within transverse reinforcement, comprising either lateral ties (with internal angles not exceeding 135^0) or spirals. This is required:

- To prevent the premature buckling of individual bars
- To confine the concrete in the 'core', thus improving ductility and strength
- To hold the longitudinal bars in position during construction
- To provide resistance against shear and torsion, if required.



Spirals

Spiral reinforcement provides confinement to the concrete core from all direction and hence the core is under triaxial compression which enhances significantly the ductility of the column at ultimate loads. The diameter and pitch of the spiral may be computed as in the case of ties — except when the column is designed to carry a 5 percent overload (as permitted by the Code), in which case

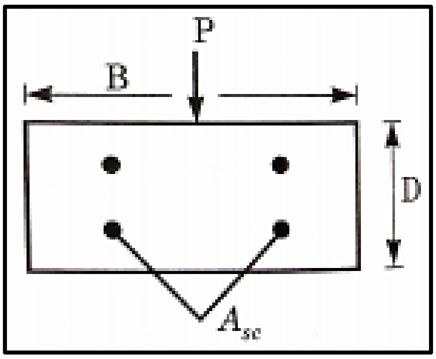
$$\text{pitch } S_t < \begin{cases} 75\ mm \\ core\ diameter\ /\ 6 \end{cases} \qquad \text{and} \qquad S_t > \begin{cases} 25\ mm \\ 3\emptyset_t \end{cases}$$

The ends of the spiral should be anchored properly by providing one and a half extra turns.

Helically reinforced columns are very much suitable for earthquake resistant structures

DESIGN OF SHORT COLUMNS UNDER AXIAL COMPRESSION

If load P is applied on a column of width B and depth D



For pure axial loading condition, the design strength of a short column is

$$\boldsymbol{P_u = 0.45 f_{ck} A_g + (f_{sc} - 0.45 f_{ck})\, A_{sc}}$$

$$f_{sc} = 0.87 f_y \text{ for Fe 250}$$

$$0.790\ f_y \text{ for Fe 415}$$

$$0.746\ f_y \text{ for Fe 500}$$

The design stress in concrete is $\frac{0.67\, f_{ck}}{1.5} = 0.45\ f_{ck}$

and design stress in steel is 0.87 f_y in case of Fe 250 under 'pure' axial loading conditions, the design strength of a short column is

$$P_u = 0.45 f_{ck} A_c + 0.87\ f_y\ A_{sc} \quad \ldots\text{(iii)}$$

However, the code requires all columns to be designed for minimum eccentricities in loading hence equation (iii) cannot be applied directly. Nevertheless, where the calculated minimum eccentricity (in any plane) does not exceeds 0.05 time the lateral dimension (in the plane considered), the code (Cl. 39.3) permits the use of the following simplified formula, obtained by reducing the P_u by 10%

Now, $$\boldsymbol{P_u = 0.4 f_{ck} A_c + 0.67\ f_y\ A_{sc}}$$

The permissible load for column is increased by 1.05 times if helical reinforcement is provided because it provides very good confinement to core and enhances the ductility significantly

$$\boldsymbol{P_{u_{helical}} = 1.05(0.4\ f_{ck}\ A_c + 0.67\ f_y\ A_{sc})}$$

$$A_c = \frac{\pi}{4} {D_c}^2$$

Core volume V_c for unit length (for 1 m) of column

$$\boldsymbol{V_c = 1000 \times A_c}$$

Now,

$$D_h = D_c - \phi_h$$

V_h= (No. of turns) X length in one turn × c/s area of helical bar

$$= \frac{1000}{p} \pi D_h \times \frac{\pi}{4} \phi_h^2$$

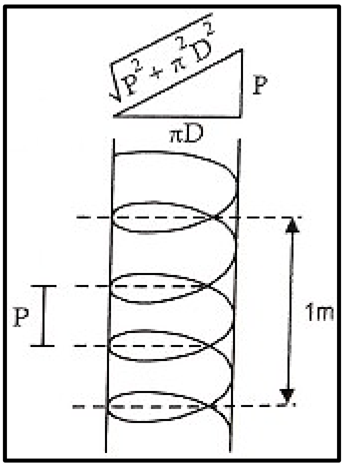
$$\boldsymbol{\frac{1000}{p} \pi D_h \times \frac{\pi}{4} \phi_h^2}$$

Where,

P= pitch of helix

D_h = diameter of helix

ϕ_h= diameter of helical reinforcement



- On the other hand, if the eccentricity is relatively large, the flexural behaviour predominates, and the consequent failure is termed tension failure.

- In fact, depending on the exact magnitude of the loading eccentricity e, it is possible to predict whether a 'compression failure' or a 'tension failure' will take place.

Balanced Failure

- In between 'compression failure and 'tension failure', there exists a critical failure condition, termed 'balanced failure'.

- This failure condition refers to that ultimate limit state wherein the yielding of the outermost row of longitudinal steel on the tension side and the attainment of the maximum compressive strain in concrete ε_{cu} = 0.0035 at the highly compressed edge of the column occur simultaneously.

- In other words, both crushing of concrete (in the highly compressed edge) and yielding of steel (in the outermost tension steel) occur simultaneously.

- In this context, for design purpose, the 'yield strain' ε_y is defined simply as that corresponding to the conventional definition of 'yield point' in the design stress-strain curve for steel.

$$\varepsilon_y = \begin{cases} 0.87\, f_y E_s & for\ Fe\ 250 \\ 0.87 f_y / E_s\ + 0.002 & for\ Fe415\ /\ Fe\ 500 \end{cases} \quad \text{....(i)}$$

This implies

$$\varepsilon_y = \begin{cases} 0.0010875 & for\ Fe\ 250 \\ 0.0038052 & for\ Fe\ 415 \\ 0.004175 & for\ Fe\ 500 \end{cases} \quad \text{....(ii)}$$

- It may be noted that the Code (Cl 39.7 1.1), in connection with slender column design, refers to an axial load, P_b (denoting approximately the balanced condition), corresponding to ε = 0.002 in the outermost layer of tension steel, for all grades of steel.

Interaction Curve

- The 'interaction curve' is a complete graphical representation of the design strength of a uniaxially eccentrically loaded column of given proportions.

- Each point on the curve corresponds to the design strength values of P_u and M_u associated with a specific eccentricity (e) of loading.

- That is to say, if load 'P' is applied on a short column with an eccentricity 'e' and if this load is gradually increased till the ultimate limit state is reached and that ultimate load at failure is given by $P = P_u$ and the corresponding moment by $M = M_u = P_u.e$, then the co-ordinates (M_u, P_u) form a unique point on the interaction diagram.

- The interaction curves define the different (M_u, P_u) combinations for all possible eccentricities of loading $0 \leq e < \infty$.

- For design purposes. the calculations of M_u and P_u are based on the design stress-strain curves (including partial safety factors) and the resulting interaction curve is referred to as the 'design interaction curve' (which is different from the characteristic interaction curve).

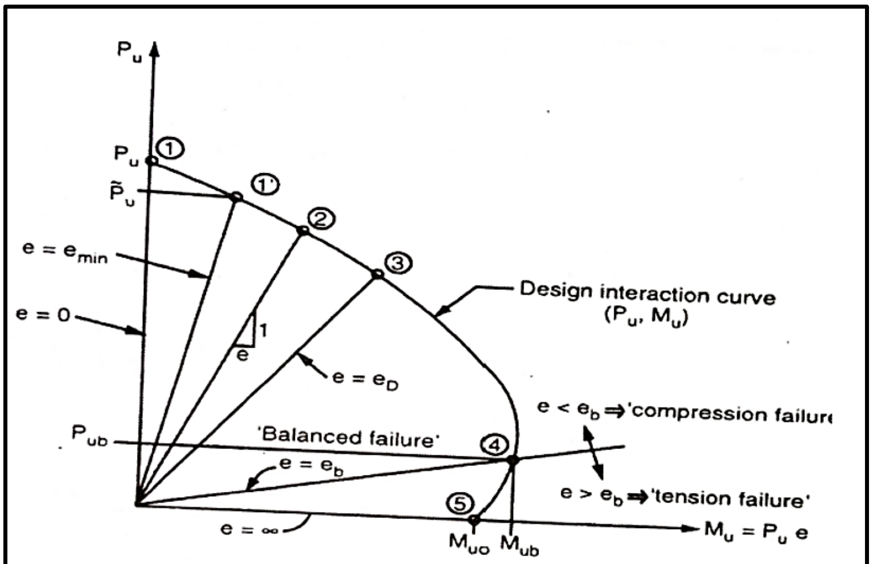
- Using the design interaction curve for a given column section, it is possible to make a quick judgement as to whether or not the section is safe under a specified factored load effect combination (P_u, M_u).

- If the point given by the co-ordinates (M_u, P_u) falls within the design interaction curve, the column is 'safe', otherwise it is not.

(c) The point (3) corresponds to the condition $e = e_D$ (i.e. $x_u = D$). For $e < e_D$, the entire section is in under compression and neutral axis is located outside the section ($x_u > D$) with $0.002 < \varepsilon_u < 0.0035$.

For $e > e_D$, the neutral axis is located within the section ($x_u < D$) and $\varepsilon_u = 0.0035$ at the "highly compressed edge".

Point (2) represents a general case, with the neutral axis outside the section ($e < e_D$).



Typical $P_u - M_u$ interaction diagram

(d) The point (4) corresponds to the balanced failure condition, with $e = e_b$, and $x_u = x_{ulim}$. The design strength values for this 'balanced failure' condition are denoted as p_{ub} and M_{ub}.

For $P_u < P_{ub}$ (i.e. $e > e_b$), the mode of failure is called tension failure.

(e) The point (5) corresponds to a 'pure' bending condition ($e = \infty$, $P_u = 0$). The resulting ultimate moment of resistance is denoted by M_{u0} and the corresponding neutral axis depth takes on a minimum value $x_{u\ min}$. This case is the same as the doubly reinforced section of beam.

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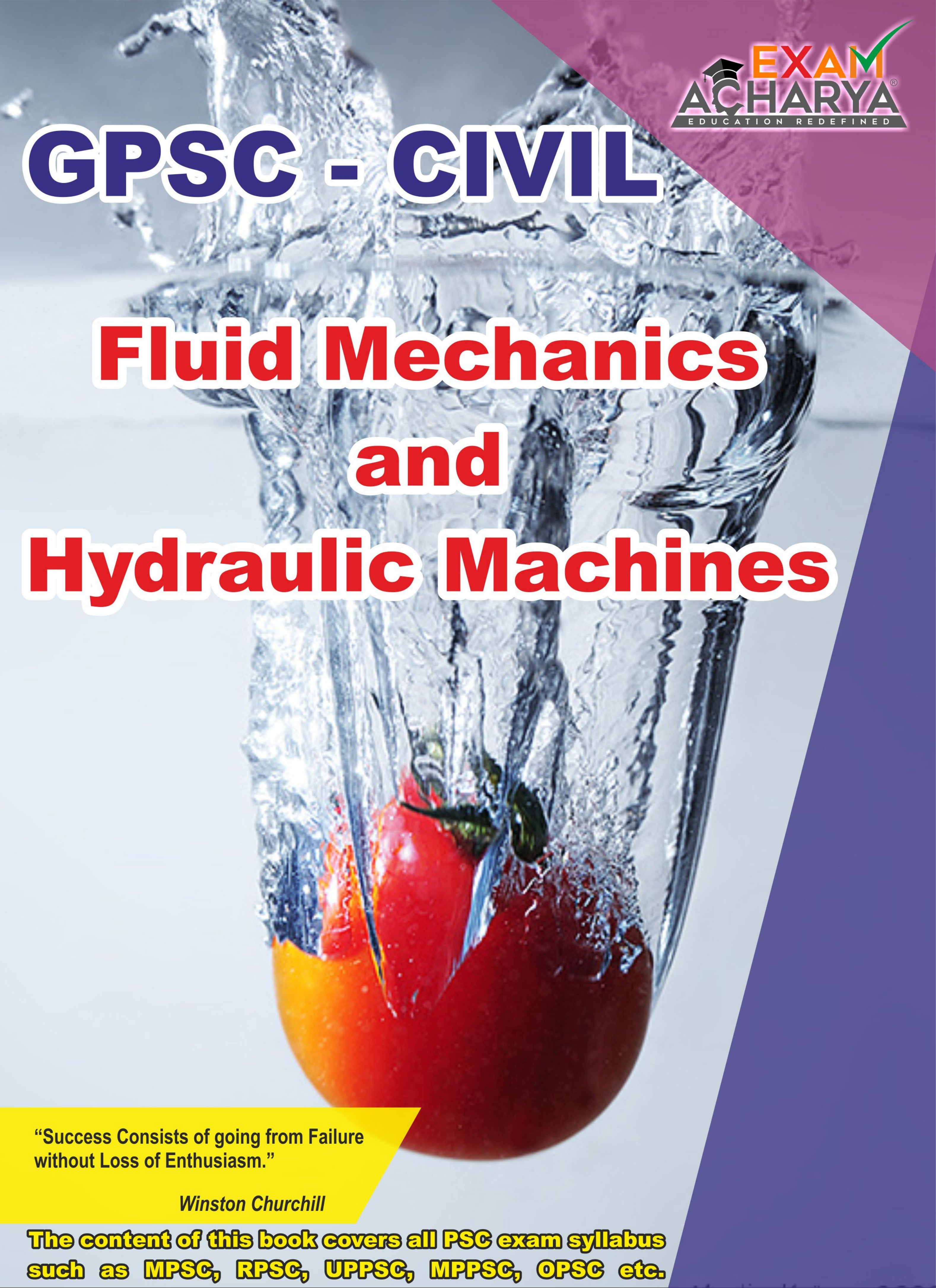
Fluid Mechanics and Hydraulic Machines



"Success Consists of going from Failure without Loss of Enthusiasm."

Winston Churchill

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For values of $\frac{P_u}{P_{uz}}$ = 0.2 to 0.8. the values of α_n vary linearly from 1.0 to 2.0. For values less than 0.2, α_n is 1.0, for values greater than 0.8 α_n is 2.0 as shown in Fig.

$$\text{Accordingly, } \alpha_n = \begin{cases} 1.0 \ \ for \ \frac{P_u}{P_{uz}} < 0.2 \\ 2.0 \ \ for \ \frac{P_u}{P_{uz}} > 0.8 \\ 0.667 + 1.667 \ \frac{P_u}{P_{uz}} \ otherwise \end{cases}$$

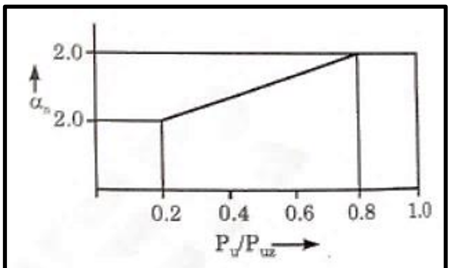


Fig. 8.7

SLENDER COLUMNS

An essential step in the design of a column is to determine whether the proposed dimensions will make it a short column or a slender column. A short compression member, whose l/D ratio is less than 12, is not in danger of buckling prior to achieving its ultimate strength based on the properties of the cross-section. Moreover, the lateral deflection of short compression members subjected to bending moments are small, thus, contributing little secondary bending moment P-Δ as as shown in Fig. These buckling and additional deflection effects are more pronounced in slender compression members and reduce their ultimate strength as compared to that of a short column having the same cross-section and amount of steel.

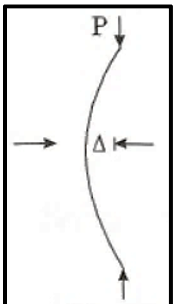


Fig. 8.8

These expressions are applicable to a balance design of a slender column subjected to uniaxial bending as well as biaxial bending. As the axial load increases from zero, the tensile stress in the steel decreases to zero and changes to a compressive stress. As this occurs, the curvature and deflection decreases. Clause 39.7.1.1 of the code permits a reduction in the additional moments by a factor k given by:

$$\mathbf{K = \frac{P_z - P}{P_z - P_b} \leq 1}$$

P_z = Capacity of cross section under pure axial load

P_z = $0.45\ f_{ck}\ A_c + 0.75\ f_y\ A_{sc}$

P_b = Balance axial load corresponding to the condition of maximum compressive strain of 0.0035 in concrete and tensile strain of 0.002 in the outermost layer of tension steel.

The value of P_b will depend on arrangement of reinforcement and the cover ratio d'/D, in addition to the grades of concrete and steel. The values of P_b can be determined using the following equations:

For rectangular section $P_b = \left(q_1 + \frac{q_2 p}{\sigma_{ck}}\right) f_{ck} bD$

For circular sections $P_b = \left(q_1 + \frac{q_2 p}{\sigma_{ck}}\right) f_{ck} D^2$

q_1 and q_2 are particular coefficients.

Qu5 Lap length compression shall not be less than

a) 15ϕ

b) 20ϕ

c) 24ϕ

d) 30ϕ

Qu6 For walls, columns and vertical faces of all the structural members, the form work is generally removed after

a) 24 to 48 hours

b) 3days

c) 7days

d) 14days

TEST YOUR SELF

Qu7 According to IS: 456 – 2000, the maximum reinforcement in a column is

a) 4%

b) 2%

c) 6%

d) 8%

Qu8 Maximum spacing of longitudinal bars measured along the periphery of the RC column shall not exceed

a) 200mm

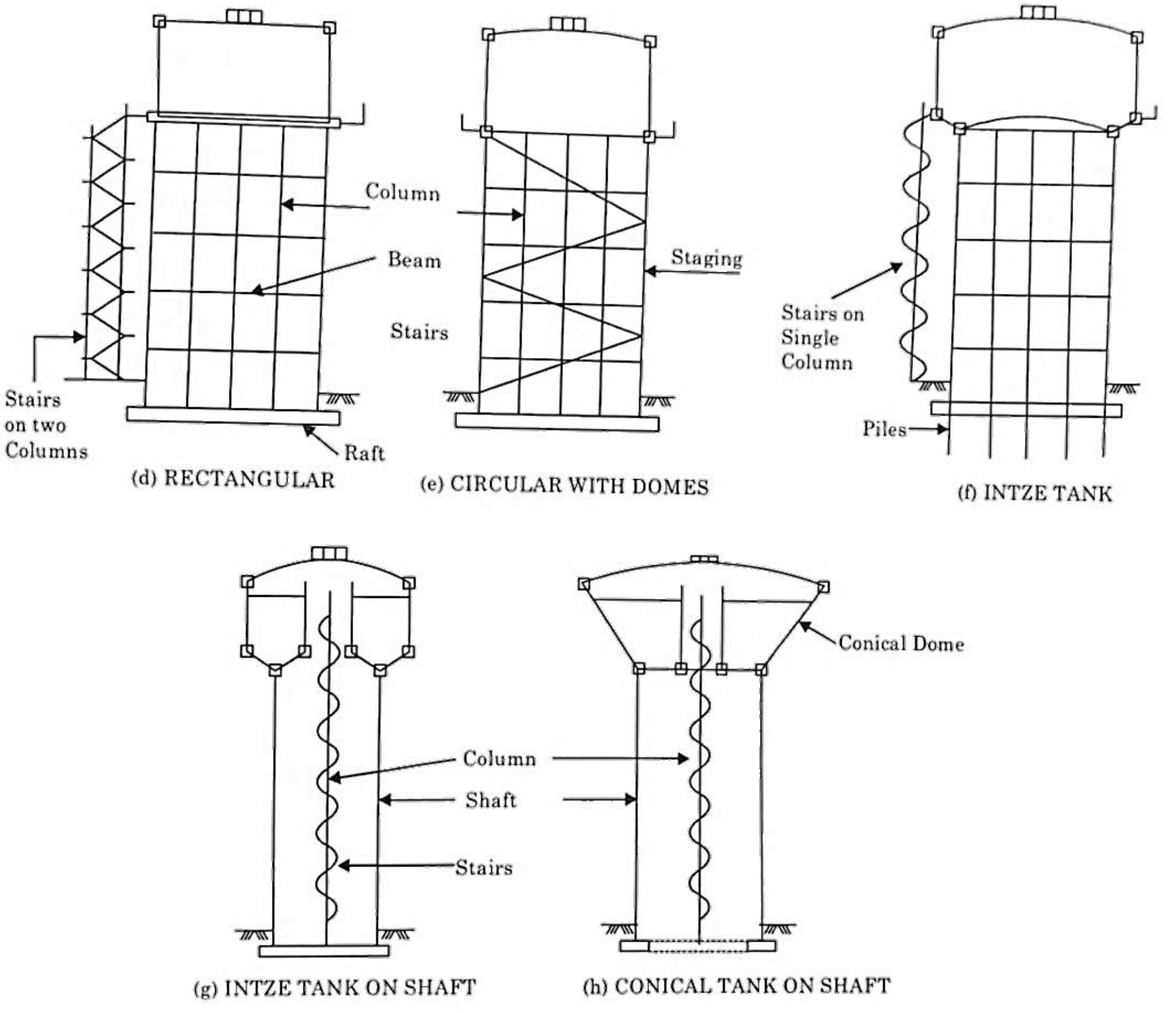
b) 250mm

c) 300mm

d) 20 times diameter of longitudinal bar

Answer

1-(a), 2-(d), 3-(c), 4-(c), 5-(c), 6-(a), 7-(c), 8-(c)



Typical liquid retaining structure

- The design of structure for liquid storage (water tanks) is based upon four codes namely.

IS 3370(Part 1): 2009 General Requirements

Part 2: 2009 Reinforced concrete structure.

Part 3: Prestressed concrete structure.

Part 4 (1967): Design tables for moments, shear and tension.

Note

- IS 3370 Part 2:2009 acknowledges both WSM and LSM method and recommends design based on anyone of them.

Permissible Stresses on Concrete

Resistance to Cracking

- For calculations relating to the resistance to cracking, the permissible concrete stresses shall conform to the values specified in Table 1. Although cracks may develop in practice. compliance, with assumption stated ensures that these cracks are not excessive.

Table-1: Permissible Concrete Stresses in Calculations Relating to Resistance to Cracking

Sr. No.	Grade of Concrete	Permissible Concrete Stress, N/mm²	
		Direct Tension	Tension due to bending
(i)	M25	1.3	1.8
(ii)	M30	1.5	2.0
(iii)	M35	1.6	2.2
(iv)	M40	1.8	2.4

Strength Calculation

- In strength calculations, the permissible concrete stresses shall be in accordance with Table 2 and Table 3.

Table 2: Permissible Stresses in Concrete

Sr. No.	Grade of Concrete	Permissible Concrete Stress in Compression (N/mm²)		Permissible Stress in Bond (Average) for Plain Bars in Tension (N/mm²)
		Bending σ_{cbc}	Direct σ_{cc}	τ_{bd}
(i)	M25	8.5	6.0	0.9
(ii)	M30	10.0	8.0	1.0
(iii)	M35	11.5	9.0	1.1
(iv)	M40	13.0	10.0	1.2

in steel shall be equal to the product of modular ratio of steel and concrete, and the corresponding permissible tensile stress in concrete.

Strength Calculations

Table 4: Permissible Strength in Steel Reinforcement for Strength

Type of Stress in Steel Reinforcement	**Permissible Stresses, N/mm^2**	
	Plain Round Mild Steel Bars	**High Strength Deformed bars**
Tensile stress in members under direct tension, bending and shear	115	130

ANALYSIS OF MEMBER

The members of water tank can be subjected to axial tensions, bending, or combined axial tension and bending. The analysis for each cases is carried out below:

Members Subjected to Axial Tension only

- Such members shall satisfy the following conditions:

  (i) There should be sufficient reinforcement to resist all the tensile forces (considering zero stress in concrete).

  (ii) The calculated tensile stress in concrete should not exceed the permissible stress.

$$\mathbf{T} = \boldsymbol{\sigma_{st}} \times \mathbf{A_t} \qquad \ldots(1)$$

Where,

T = Force of tension

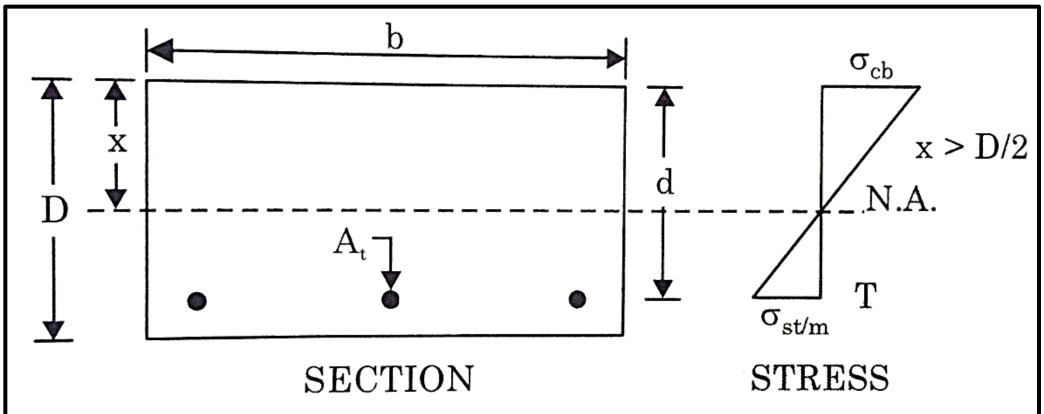
σ_{st} = Permissible stress in tension in steel

A_t = Area of reinforcement

Uncracked Section

Let the depth of neutral axis from the extreme compression edge is x.

Taking moment of area about the neutral axis



Concrete uncracked in tension

$$bx\frac{x}{2} = b(D-x)\frac{(D-x)}{2} + (m-1)\,A_t\,(d-x) \qquad \text{....(5)}$$

For an assumed value of area of tension steel near about the minimum steel, depth of neutral axis can be calculated. The moment of inertia of the equivalent section is given as :

$$I = \frac{bD^3}{12} + bD(x-0.5D)^2 + (m-1)\,A_t\,(d-x)^2 \qquad \text{...(6)}$$

Where, d = effective depth of the section

The tensile stress in concrete can be computed using Eq. 4, where y = d - x. The tensile stress in concrete surrounding the steel will he σ_{st}/m.

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Surveying

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The best Brains of the Nation may be found on the last Benches of the Classroom.

A.P.J. Abdul Kalam

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then, $N = N_b$

Where,

N_b = coefficient of the neutral axis for a balanced section

N = coefficient of the neutral axis

σ'_{st} = permissible stress in steel in bending tension

σ'_{cbc} = permissible stress in concrete in bending compression

Force of compression, $$\mathbf{C = \frac{1}{2}\sigma_{cbc}\, bNd} \quad \text{...(11)}$$

Force of tension $$\mathbf{T = \sigma_{st} A_t} \quad \text{...(12)}$$

Moment of resistance with respect to concrete

$$\mathbf{MOR = \left(\frac{1}{2}\sigma_{cbc}\, bNd\right) \times jd} \quad \text{...(13)}$$

$$\mathbf{MOR = \frac{1}{2}\sigma_{cbc}\, Njbd^2 = Kbd^2} \quad \text{...(14)}$$

Where, $$\mathbf{K = \frac{1}{2}\sigma_{cbc}\, Nj} \quad \text{...(15)}$$

j = coefficient of lever arm

$$\mathbf{j = 1 - \frac{N}{3}} \quad \text{...(16)}$$

Moment of resistance with respect to steel

$$\mathbf{M = (\sigma_{st} A_t)\, jd} \quad \text{...(17)}$$

or, $$\mathbf{A_t = \frac{M}{\sigma_{st} jd}} \quad \text{...(18)}$$

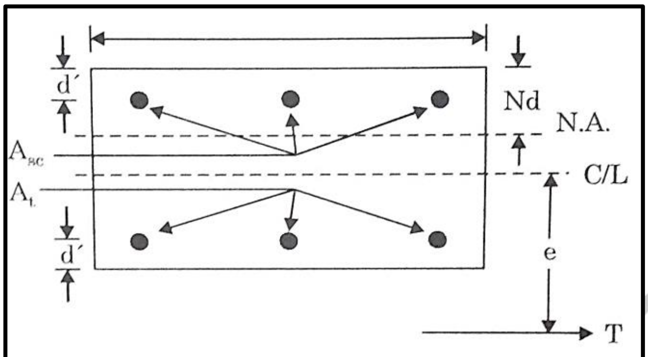
Eq. 18 gives the quantity of steel required to resist the entire force of tension. Here, the neutral axis is usually above the mid depth of the section,

$$\sigma_{st1} = \frac{(d_2 - e)T}{A_{t1}(d_1 + d_2)} \qquad \ldots(20)$$

and
$$\sigma_{st1}A_{t1} + \sigma_{st2}A_{t2} = T \qquad \ldots(21)$$

(ii) When eccentricity is large, that is tensile force is small (e = M/T).

Here, the line of action of the force lies outside the section. The direct and bending stresses are equally dominant. Hence, the neutral axis can be located by trial and error.



Section under large eccentricity

Force of compression in concrete = $\frac{1}{2}\sigma_{cbc}\, bNd$

Force of compression in steel = (m -1)A_{sc} $\left(\frac{Nd-d'}{Nd}\right)\sigma_{cbc}$

Force of tension in steel = $\sigma_{st}A_t$

∴ Equilibrium of forces gives:

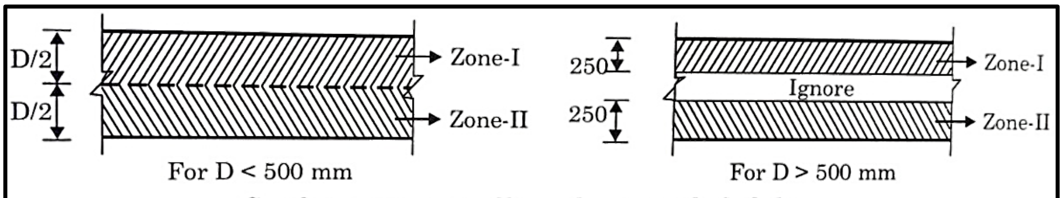
$$\sigma_{st}A_t - \frac{1}{2}\sigma_{cbc}\,bNd - (m-1)\,A_{sc}\left(\frac{Nd-d'}{Nd}\right)\sigma_{cbc} = T \qquad \ldots(22)$$

For moment equilibrium,

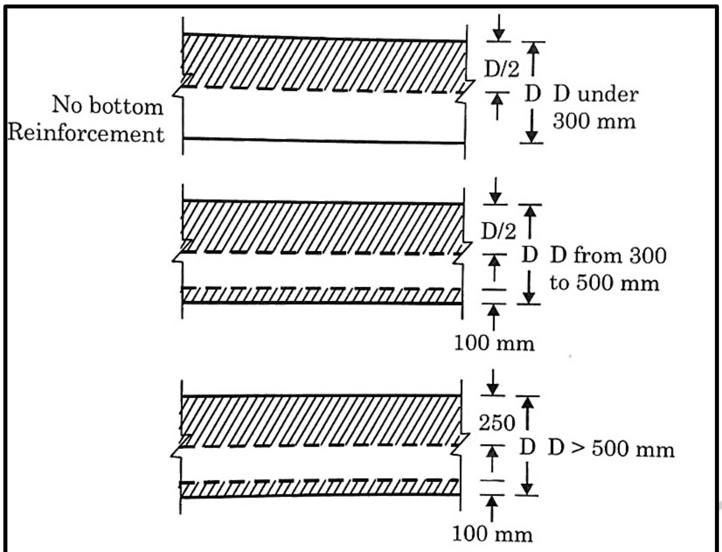
Taking moment of all the forces about the tension steel,

$$\frac{1}{2}\sigma_{cbc}\,bNd\left(d - \frac{Nd}{3}\right) + (m-1)A_{sc}\left(\frac{Nd-d'}{Nd}\right)\sigma_{cbc}\,(d - d') = T\left(e - \frac{D}{2} + d'\right)$$

…(23)



Surface zones - walls and suspended slabs



Surface zones in grounded slabs

- The minimum reinforcement may be reduced to 0.24% for HYSD bar for tanks having any dimension of the container not more than 15 m and for mild steel it can be reduced to 0.4%
- In walls of less than 200 mm thickness, the calculated amount of reinforcement may be placed in one face.
- In ground slabs less than 300 mm thickness, the calculated reinforcement should be placed in one face as near as possible to the upper surface with appropriate cover.
- When reinforcement is placed in two layers, the two layers of reinforcing steel should be placed on near each face of the section to make up the minimum reinforcement.
- For liquid faces of parts of members either in contact with the liquid or enclosing the space above the liquid, the minimum clear cover to all reinforcement should be 45 mm. The cover should be determined based on durability criteria.

- Risk of cracking shall be minimised by reducing the restrains on the free expansion or contraction of the structure.
- Small size of bars should be used for reinforcement.

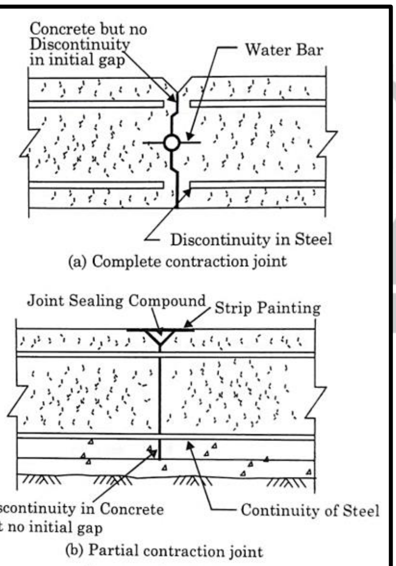
JOINTS

Joints are categorized as follows:

Movement Joints

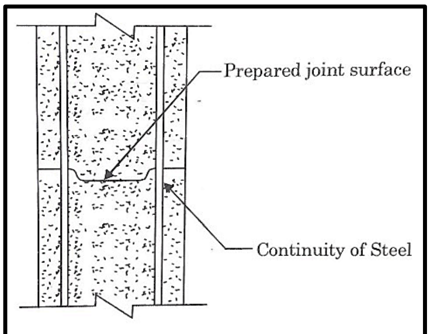
There are three types of movement joints:

(a) Contraction Joint is a movement joint with a deliberate discontinuity but no initial gap between the concrete on either side of the joint. The purpose of this joint is to accommodate contraction of concrete.



Contraction joints

(b) Expansion Joint is a movement joint with complete discontinuity in both reinforcement and concrete and intended to accommodate either expansion or contraction of the structure. An expansion type water bar should be provided either centrally in a wall or on the soffit of a floor.



Construction joint

DESIGN AND DETAILING OF JOINTS

Design of a movement joint should aim at achieving the following desirable properties for effective functioning:

(a) The joint should accommodate repeated movement of the structure without loss of water tightness.

(b) The design should provide for exclusion of grit and debris which would prevent the closing of the joint.

(c) The material used in the construction of movement joints should have the following properties:

(i) It should not suffer permanent distortion or extrusion and should not be displaced by fluid pressure.

(ii) It should not slump unduly in hot weather or become brittle in cold weather.

(iii) It should be insoluble and durable and should not be affected by exposure to light or by evaporative of solvent or plasticisers.

(iv) In special cases, the materials should be nontoxic, taintless or resistant or chemical and biological action as may be specified.

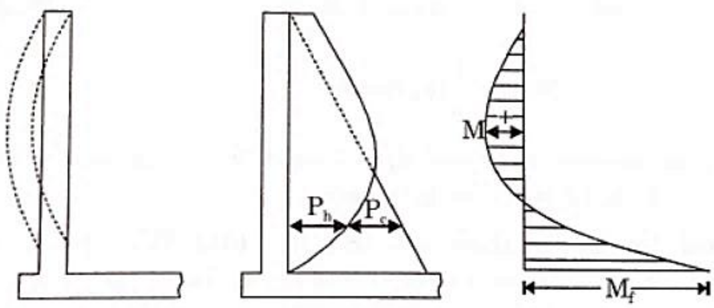
Floor Slab

For floor slab resting on the ground, a minimum thickness of 200 mm is to be provided.

Circular Tank with Rigid Joint Between Floor and Wall

In case of rigid joint no relative movement at the base is possible. This will change the behaviour of the tank wall under hydrostatic load. The upper portion of the wall will have hoop tension whereas the lower part will behave like a cantilever fixed at the base.

Shallow tanks with low hoop stresses essentially behave as cantilevers.



(a) Deformation (b) Load distribution (c) B.M.D.

Analysis of such a tank can be done by using simple bending theory and solving the differential equation by applying the correct boundary conditions.

However, this is a tedious process. There are other simplified methods which are usually used for analysis:

(i) Reissner's method

(ii) Carpenter's method

(iii) Approximate method

(iv) IS code method

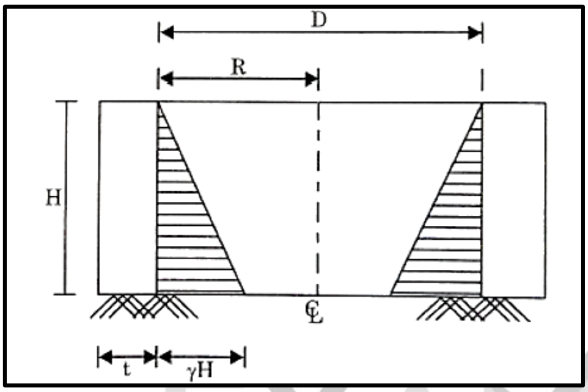
Reissener's method and carpenter's method make use of the tables prepared for analysis. Approximate method is used if any tables are not available.

IS Code Method

Indian standard code IS: 3370 (part-IV) — 1967, gives the coefficients for force responses (namely hoop tension, moment and shear force) for various types of support conditions, using which the tank can be designed.

Design tension = coefficient × $\frac{\gamma_w HD}{2}$ per metre

Some coefficients are as provided below for illustration:



T = coefficients × γHR …………..(KN/m)

Table: Tension in circular ring wall, fixed based, free top and subject to triangular load

Coefficients at points										
$\frac{H^2}{Dt}$	**0.0H**	**0.1H**	**0.2H**	**0.3H**	**0.4H**	**0.5H**	**0.6H**	**0.7H**	**0.8H**	**0.9H**
1.2	+0.283	+0.271	+0.254	+0.234	+0.209	+0.180	+0.142	+0.099	+0.054	+0.016
1.6	+0.265	+0.268	+0.268	+0.266	+0.250	+0.226	+0.185	+0.134	+0.075	+0.023
2.0	+0.234	+0.251	+0.273	+0.285	+0.285	+0.274	+0.232	+0.172	+0.104	+0.031

Note
- γ = Unit weight of the liquid
- Positive sign indicates tension on the outside

Accordingly, we have

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Geo-technical and Foundation Engineering

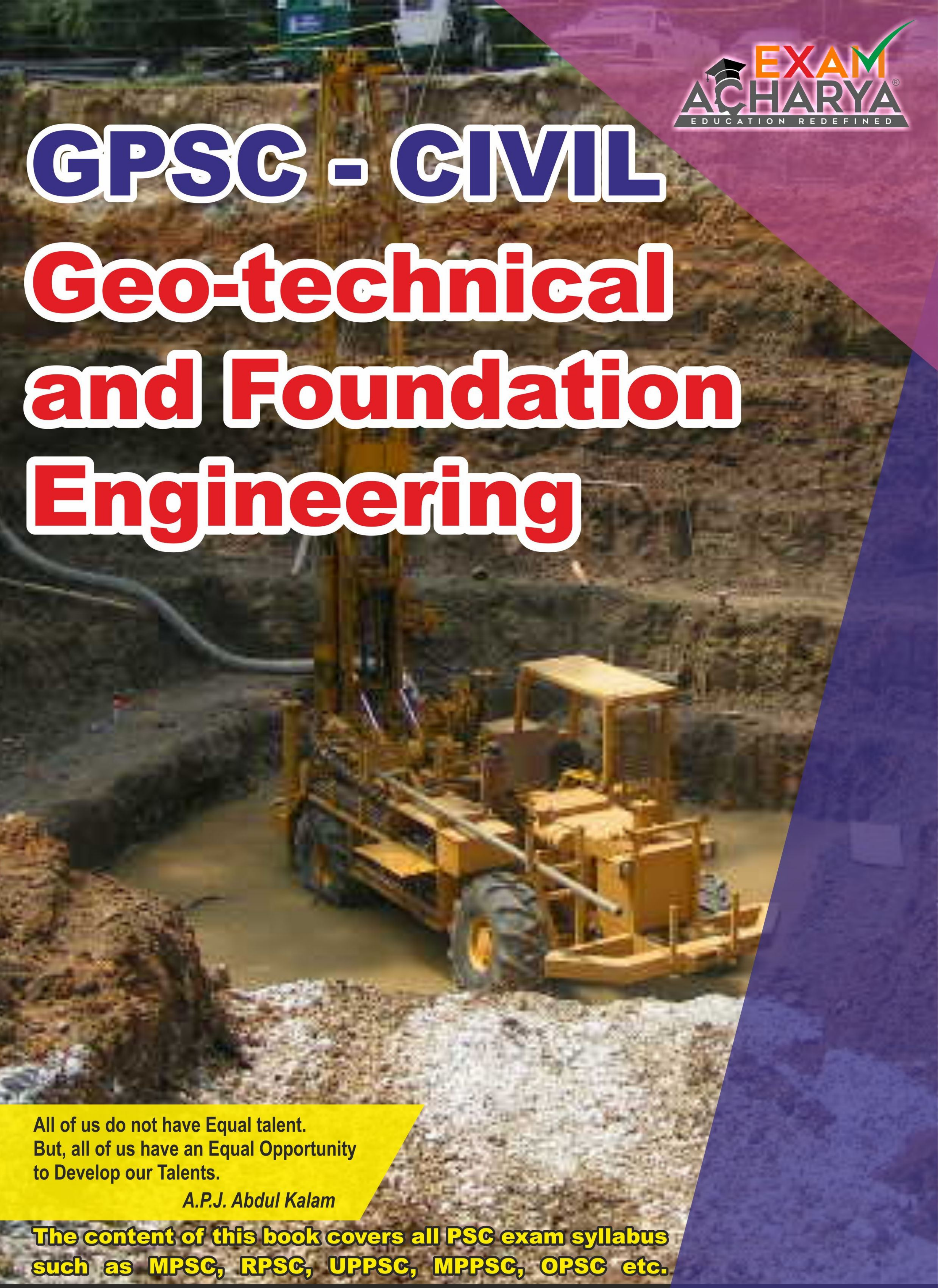

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Table : Shear at base of cylindrical wall

Coefficients at points				
$\frac{H^2}{Dt}$	**Triangular load fixed base**	**Rectangular load fixed base**	**Triangular or rectangular load hinged base**	**Moment at edge**
1.2	+0.339	+0.460	+0.220	-2.00
1.6	+0.317	+0.407	+0.204	-2.28
2.0	+0.299	+0.370	+0.189	-2.57

Note
- γ = Unit weight of the liquid
- Positive sign indicates shear acting inward.

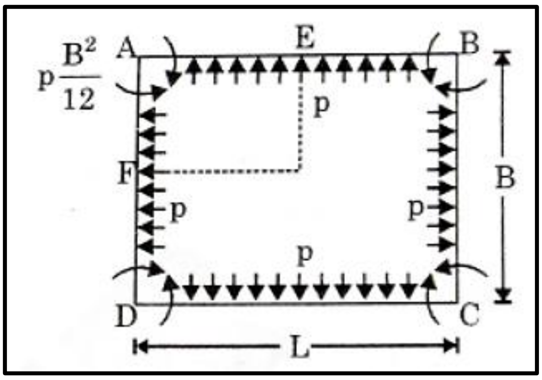
Rectangular Water Tanks

The main forces acting on a rectangular water tanks are hoop tension and bending moment. Bending moment mainly governs the thickness of the member and reinforcement is governed by hoop tension.

Let H be the height and L be the length of the rectangular/square water tank.

The edge where two adjacent walls of tank meet (at a right angle usually) can be treated as a rigid support for analysis. The top edge of the walls which support the roof slab (cover to the tank) are treated as a hinge support and the bottom edge where the walls connected (built in) to the bottom (base slab) slab are treated as fixed support for analysis and design.

Approximate Method



Direct tension along long wall = $P_L = \gamma_w$ (H-h) $\frac{B}{2}$

Direct tension along short wall = $P_B = \gamma_w$(H - h)$\frac{L}{2}$

The bottom 'h' portion of the wall will bend as vertical cantilever. The moment at the base of the cantilever is given by $\frac{\gamma_w}{6}(Hh^2)$, causing tension on the water face.

L/B Greater Than 2

In this case the long walls are assumed to bend as vertical cantilevers fixed at the base, subjected to triangularly distributed load. The short walls are subjected to bending in a horizontal plane, supported on the two long side walls, for the portion from top to a height h $\left(\text{h} = \frac{H}{4} \text{ or } 1, \text{which ever is more}\right)$. The bottom 'h' portion of the short walls is assumed to act as a vertical cantilever.

IS Code Method

Based on exact analysis, IS:3370 (Part IV) contains tables to find out moment axial tension and shear in rectangular tanks.

TEST YOUR SELF

Qu5 The minimum HYSD reinforcement in walls of a water tank (size: 5m x 3m x 2m) for each surface zone shall not be less than

a) 0.35%

b) 0.60%

c) 0.24%

d) 0.40%

Qu6 For a wall of thickness 600mm of a water tank, the thickness of individual surface zone for calculations of minimum reinforcement

a) 250mm

b) 300mm

c) 500mm

d) 600mm

Answer

1-(a), 2-(d), 3-(a), 4-(d), 5-(c), 6-(a)

- **Waist** - The thickness of the waist-slab on which steps are made is known as waist. The depth (thickness) of the waist is the minimum thickness perpendicular to the soffit of the staircase (cl. 33.3 of IS:456-2000). The steps of the staircase resting on waist-slab can be made of bricks or concrete
- **Stringer** - These are the sloping members of the stairs, use to support the end of steps.
- **Winders** - These are the steps used for changing the direction of stairs. These are usually triangular in plan.
- **Pitch or slope** - Vertical angle made by line of noising with horizontal. Pitch should not be more than 38°.

GENERAL GUIDELINES

The following are some of the general guidelines for planning a staircase:

- The respective dimensions of tread and riser for all the parallel steps should be the same in consecutive floor of a building.
- The minimum vertical headroom above any step should be 2 m.
- The pitch should not be more than 38°.

Proportioning of Stair Case

The following components of staircase are to be proportioned before designing a stair case.

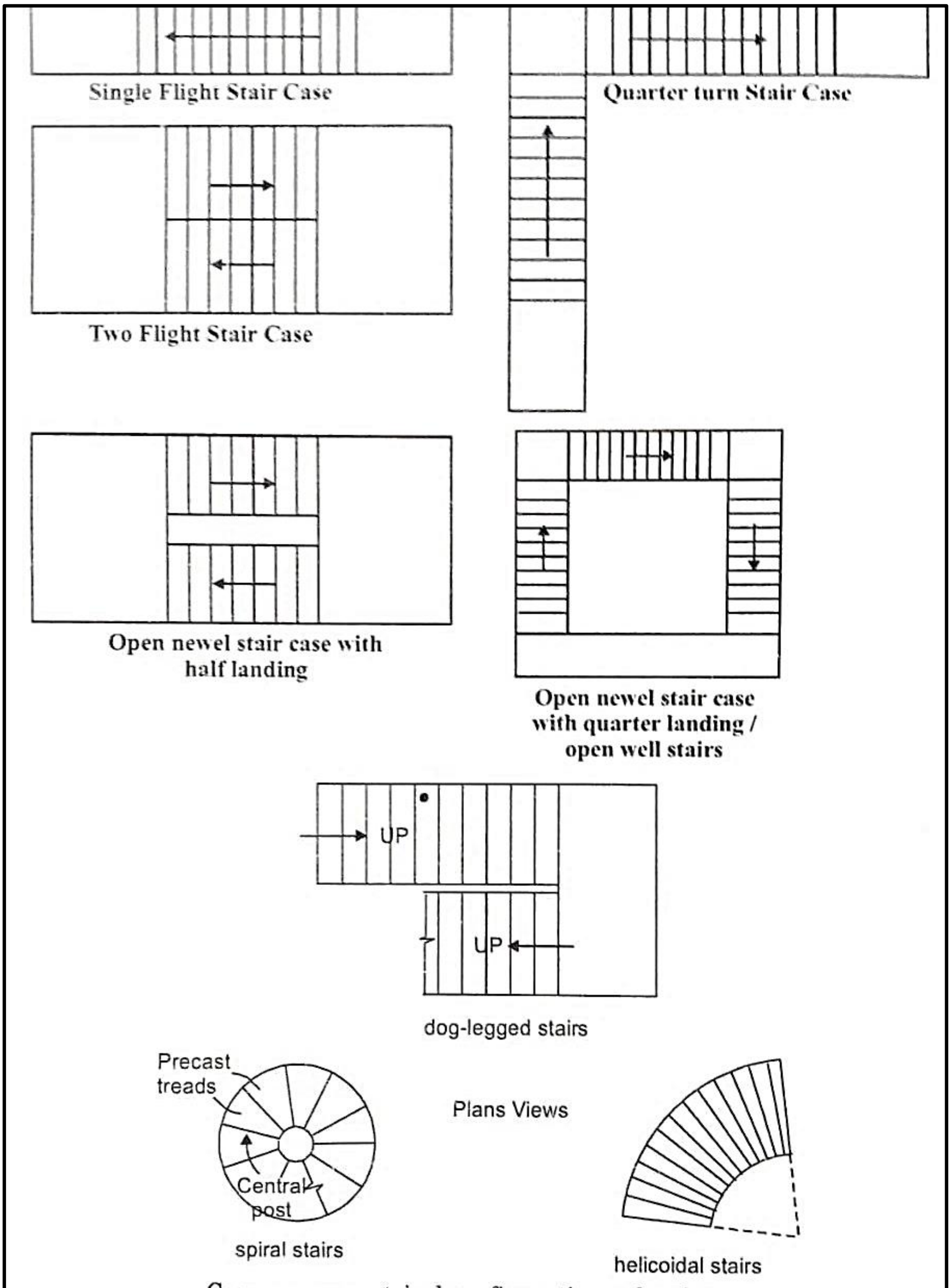
Width

The width of a step in a staircase (width of stair case) is proportioned in accordance with the recommendations gives below.

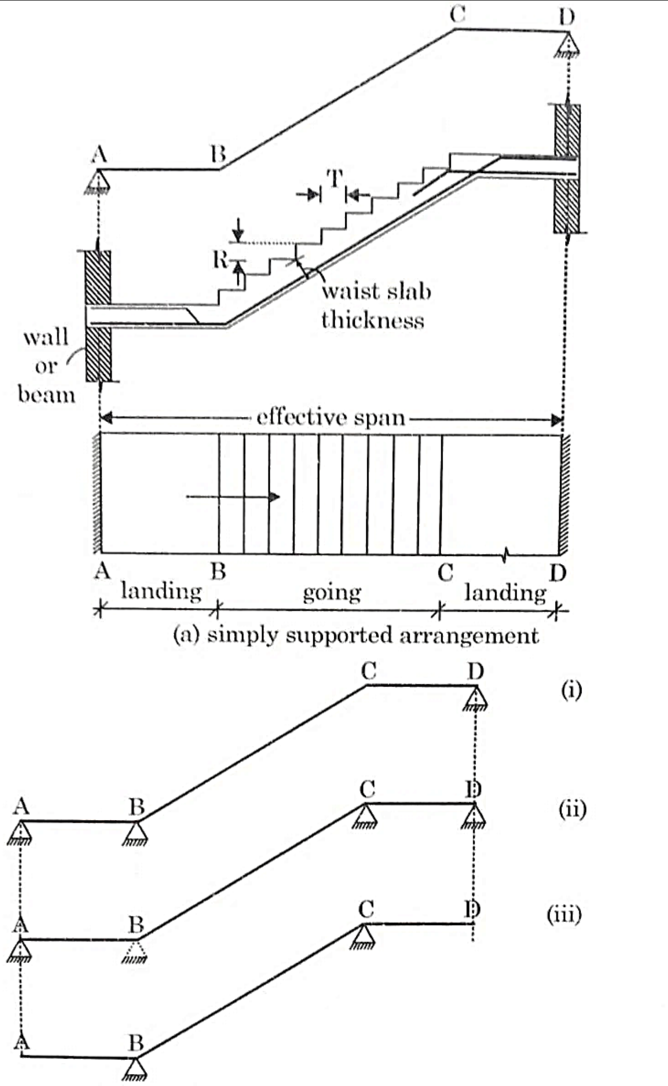
Type of staircase	**Width of staircase**
Service stair for maintenance, catwalks and two floor residential buildings	1-1.2 m

Based on Geometrical Configuration

A wide variety of staircases are there in practice. However, some of the common types are



Common geometrical configurations of stairs



(a) simply supported arrangement

(b) alternative support arrangement

Typical examples of stair slabs spanning longitudinally

However, in certain situations, beam or wall supports may not be available parallel to the riser at the landing. In this case the flight is supported between landings which span transversely, parallel to the risers. In such case the effective span shall be considered as stated below.

(iii)The horizontal distance equal to the going of the stairs plus at each end either half the width of the landing or one meter, whichever is smaller when the stair slab is spanning on to the edge of a landing slab which spans parallel with the risers.

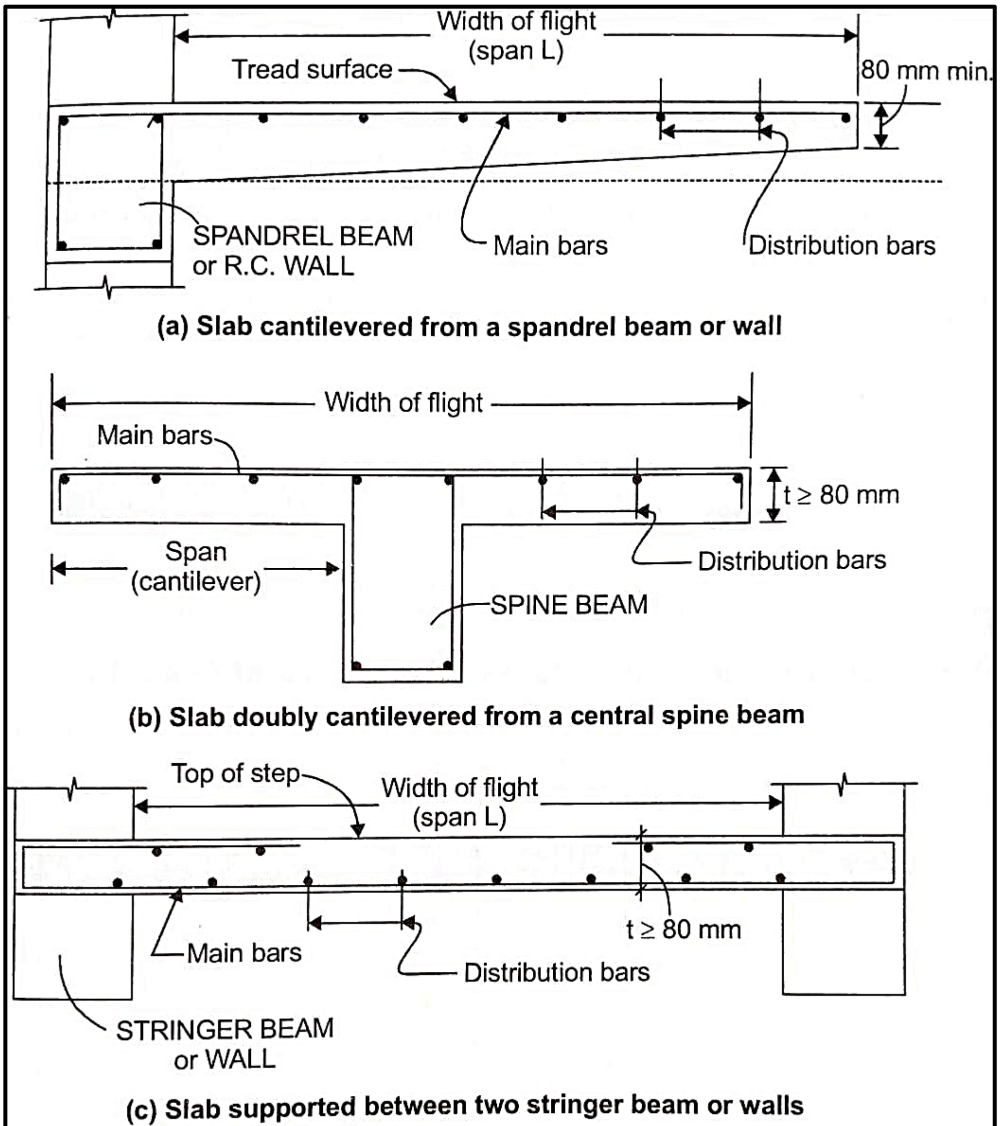
Stair Slab Spanning Transversely

Here, either the waist slabs or the slab components of isolated tread- slab and tread riser units are supported on their sides or are cantilevers along the width direction from a central beam (Spine beam). The slabs thus bend in a transverse vertical plane. The following are the different arrangements:

(i) Cantilever slabs from a spandrel beam or wall.

(ii) Doubly cantilever slabs from a central beam (Spine beam)

(iii) Slab supported between two stringer beams or walls.



Stairs slab spanning transversely

- The above loads should be multiplied by the appropriate load factors in order to get the factored loads.

Distribution of Gravity Loads in Special Cases (Clause 33.2 of IS 456)

- In the case of stairs with open wells, where spans partly crossing at right angles occur, the load on the areas common to any two such span may be taken as one half in each direction as shown in figure below

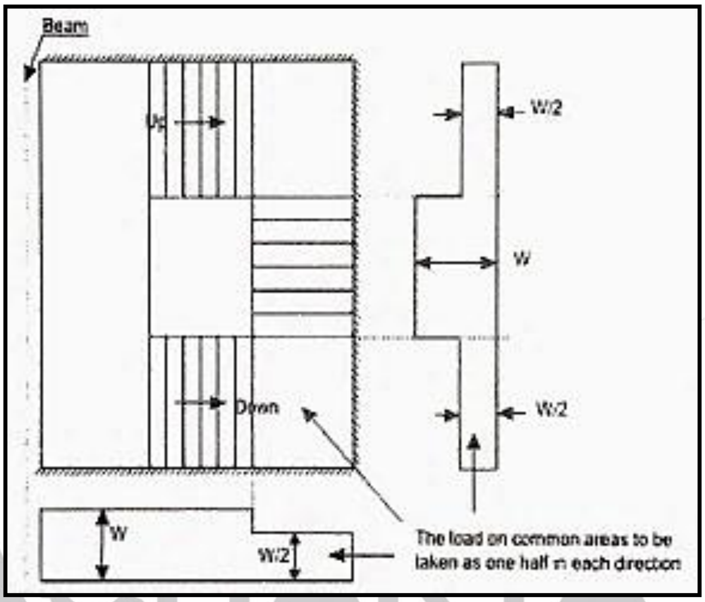


Fig. Loading on open-well staircases

- When a longitudinally spanning flight (or landing) is embedded at least 110 mm into a side wall, the longitudinally acting component of the gravity load can be assumed to act on a reduced width of flight.
- A reduction of 150 mm from the face of the side wall is permitted in the above condition.
- Furthermore, the effective width of the section can be increased by 75 mm (into the support) for purposes of design.

So, if the width of the flight be W (in mm) then the load may be assumed to act over a reduced width (W+150) mm and the effective width resisting flexure may be taken as (W + 75) mm.

Load Effects in Waist Slabs

In this type the longitudinal axis of the flight is inclined to horizontal

- The steps are generally non structural elements
- The waist slab is designed to resist the load effects
- The load is resolved in two components i.e. tangential component and the normal load components
- Waist slab can be spanned transversely or longitudinal depending upon the support conditions.

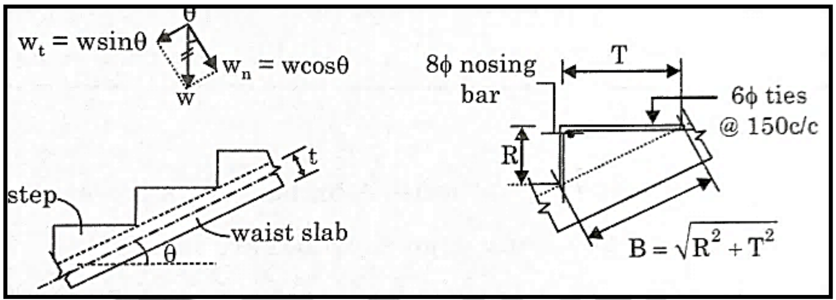


Fig. Waist slab-steps arrangement

Waist Slab Spanning Transversely

Here the normal load component causes the waist slab to bend in transverse plane and reinforcement is provided accordingly at top or bottom depending on the support condition (i.e. cantilever or simply supported)

Distribution reinforcement is provided in the longitudinal direction.

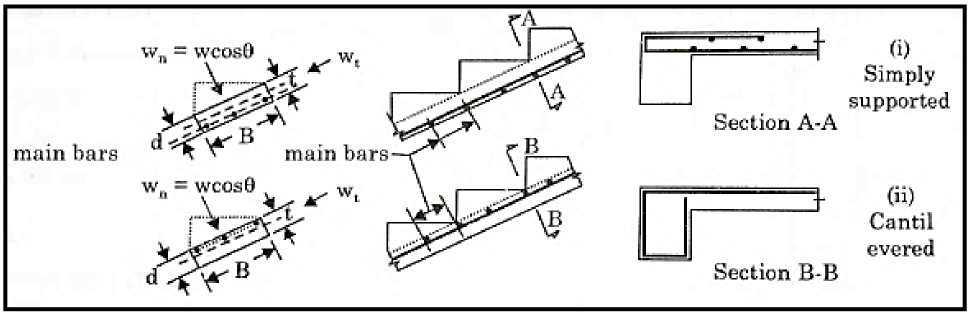


Fig. Load effects and detailing in waist slabs (transversely spanning waist slabs)

EXAM ACHARYA
EDUCATION REDEFINED

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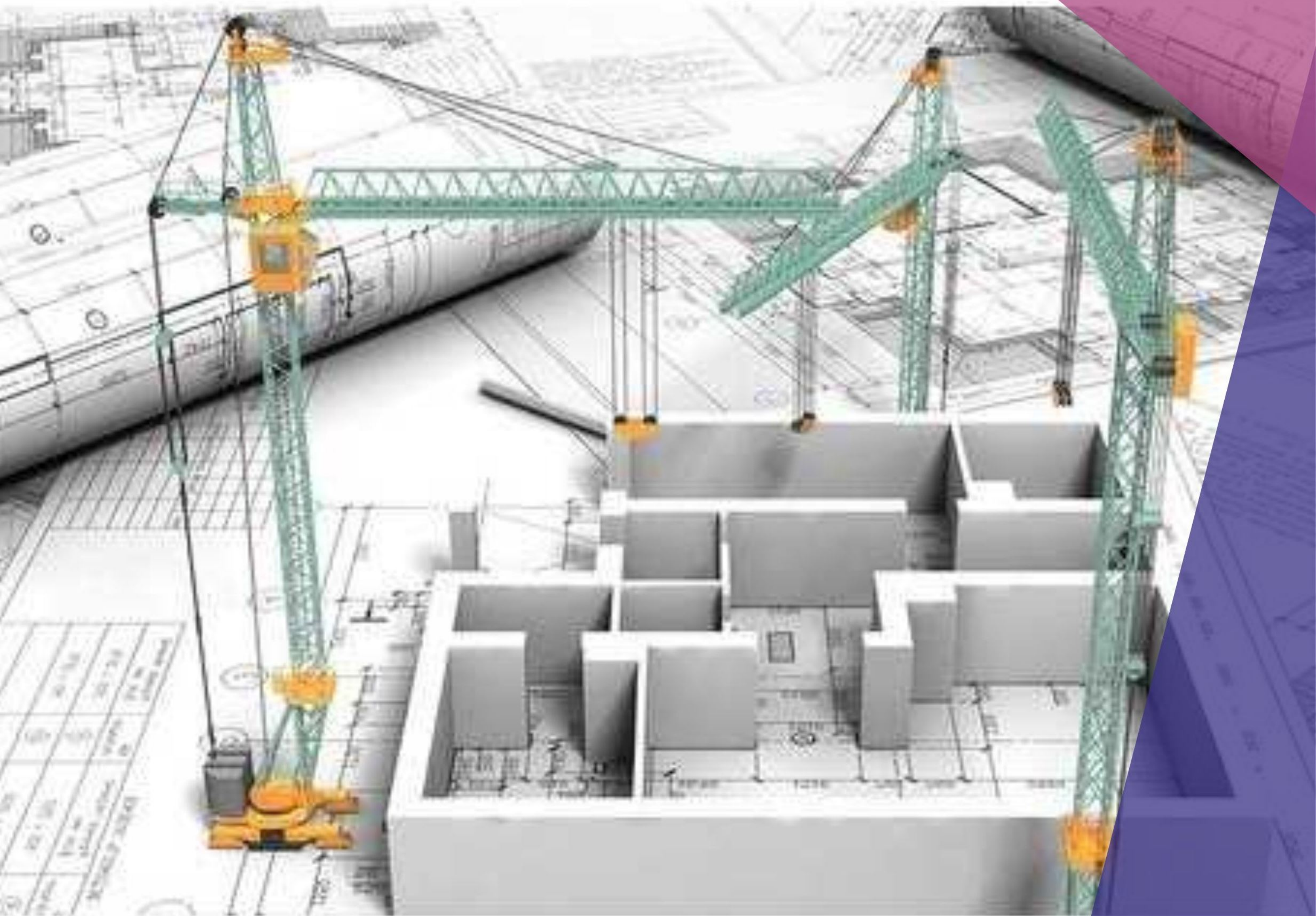

Structural Analysis

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Tread-Riser Units Spanning Transversely

- It is assumed that each tread-riser unit (riser slab + one-half of each tread slab on either side) behave independently as a beam with a Z-section
- These are basically one way slabs spanning transversely and is to be designed for uniformly distributed gravity loads.

This 'tread-riser' unit behaves like flanged beam which is transversely loaded

The overall depth of the beam is given by (R+t),

Note
- So for simply supported stairs, the top flange of this Z-section (i.e., half the tread at the top) will be in compression, while the other half of the same tread forming the lower flange of the tread-riser unit above will be in tension.
- Thus one-half of every tread is in compression and the other in tension

- In most cases of tread-riser units spanning transversely, the bending moments are low, and generally a nominal slab thickness of 100 mm is sufficient

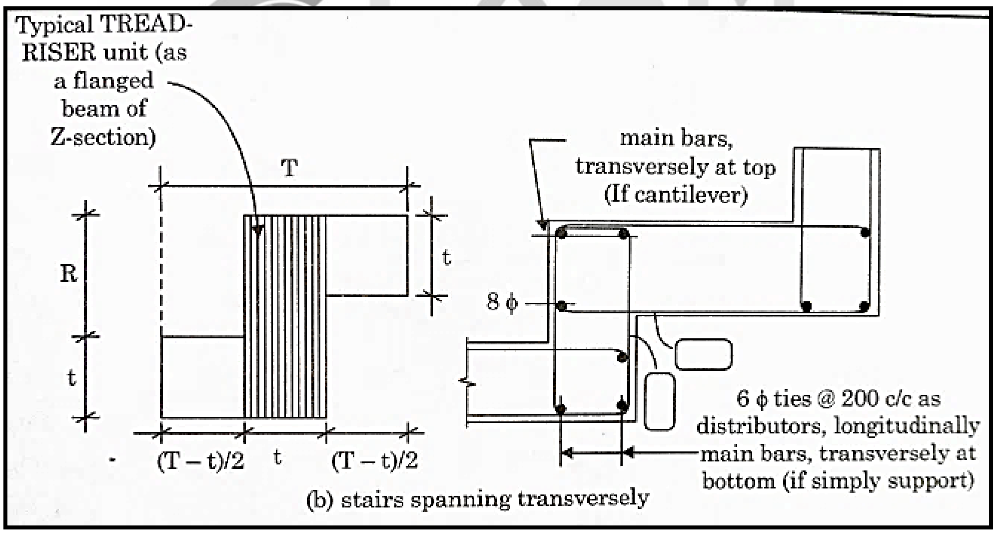


Fig. Load Effects and detailing in tread-riser units (transversely)

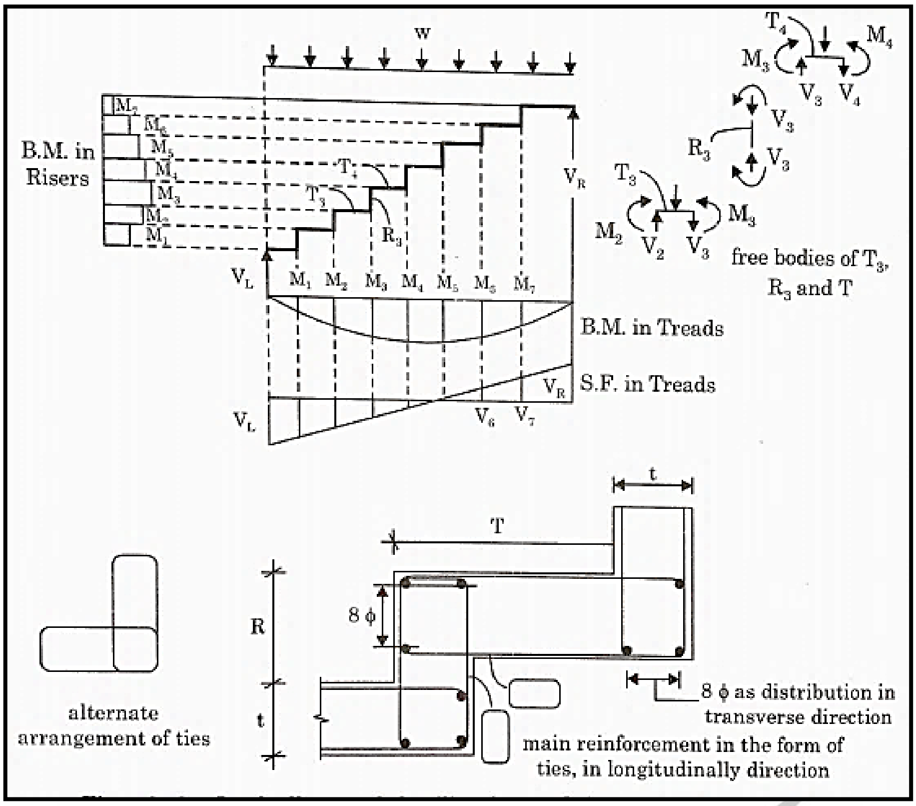


Fig. Load Effects and detailing in tread-riser units (longitudinally)

Qu5 The type of stairs in which the direction of flight is changed through 180° by introduction of landings is called

a) Quarter-turn stairs

b) Bifurcated stairs

c) Dog-legged stairs

d) Straight stairs

<u>TEST YOUR SELF:</u>

Qu6 In case of stairs (Rise X Tread) should be between in cm^2

a) 200 and 250

b) 250 and 350

c) 400 and 500

d) 600 and 800

Qu7 In which of the following arrangements of stairs case it is assumed that 50% length of tread slab is in compression and the remaining in tension

a) Tread-riser units spanning transversely

b) Tread-riser units spanning longitudinally

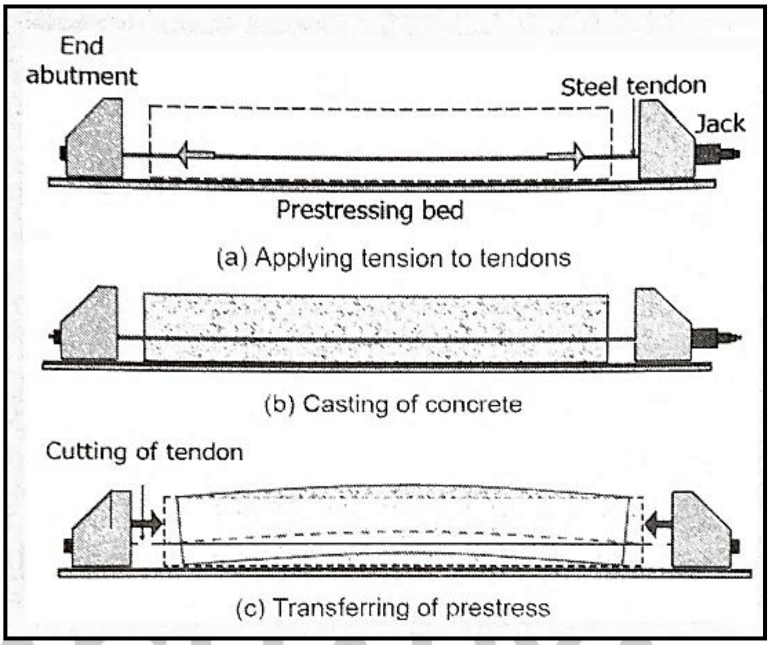
c) Waist slab spanning transversely

d) Waist slab spanning longitudinally

Answer

1-(b), 2-(d), 3-(d), 4-(a), 5-(c), 6-(c), 7-(a)

- Once the concrete attains the desired strength for prestressing, the tendons are cut loose from the abutments.
- The prestress is transferred to the concrete from the tendons, due to the bond between them.
- The stages of pre tensioning are shown schematically in the following figures.



POST TENSIONING

- In post-tensioning systems, the ducts for the tendons (or strands) are placed along with the reinforcement before the casting of concrete.
- The tendons are placed in the ducts after the casting of concrete.
- The duct prevents contact between concrete and the tendons during the tensioning operation.
- After anchoring a tendon at one end, the tension is applied at the other end by a jack. The tensioning of tendons and pre-compression of concrete occur simultaneously. Finally, the jacking end is anchored.

(Force in cable initially = P = prestressing force = $\frac{E_s\Delta}{l_s}\cdot A_s$)

(Loss of strain in cable due to shortening of concrete over a period of time = $\frac{\delta}{l_s}$)

Loss of stress in cable = $E_s\cdot\frac{\delta}{l_s}$

Loss of prestressing force = $E_s\frac{\delta}{l_s}\cdot A_s$

Normally due to creep and shrinkage in long term the strain lost is approximately 0.0008

$$\text{Stress lost} = 0.0008 \times 2 \times 10^5 = 160\ \text{N/mm}^2$$

If we use Fe-250 or Fe-415, all of the initial stress in it will be lost in due course. Hence there would not be any pressuressing force remaining in concrete, thus the beam will fail.

Hence, we use high strength steel such that the initial prestress in it would be 1200 - 2000 N/mm² in which the loss would be around 200 N/mm².

Need for High Strength Concrete in Pre-stress Concrete

- High strength concrete offers high resistance to tension, shear, bond and bearing.
- In case of pre-tensioned members, tensile stress in steel of very high magnitude should be transferred to concrete as prestress through bonding between steel and surrounding concrete.
- In post-tensioned members transfer of stress is through bearing at end sector. Hence concrete of appreciable bond and bearing strength is quite essential for pre-stressed concrete.
- In addition to above, use of high strength concrete has following advantages:
  1) High strength concrete is less liable to shrinkage cracks and has higher modulus of elasticity and smaller ultimate creep strain. As a result, loss of prestress in steel is reduced.

LIMITATIONS OF PRESTRESSING

Although prestressing has advantages, some aspects can be considered as limitations, like

- Prestressing needs skilled technology. Hence, it is not as common as reinforced concrete.
- The use of high strength materials is costly.
- There is additional cost in auxiliary equipment's.
- There is need for quality control and inspection.

TYPES OF PRESTRESSING

Prestressing of concrete can be classified in several ways. The following are the various classifications.

Source of Prestressing Force

This classification is based on the method by which the prestressing force is generated. There are four sources of prestressing force: Mechanical, hydraulic, electrical, thermal and chemical.

Hydraulic Prestressing

This is the simplest type of prestressing, producing large prestressing forces. The hydraulic jack used for the tensioning of tendons, comprises of calibrated pressure gauges which directly indicate the magnitude of force developed during the tensioning.

Mechanical Prestressing

In this type of prestressing, the devices include weights with or without lever transmission, geared transmission in conjunction with pulley blocks, screw jacks with or without gear drives and wire-winding machines. This type of prestressing is adopted for mass scale production.

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Solid Mechanics

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Internal Prestressing

- When the prestressing is achieved by elements located inside the concrete member (commonly, by embedded tendons), it is called internal prestressing.
- Most of the applications of prestressing are internal prestressing, concrete is cast around the ducts for placing the tendons.

Pre Tensioning or Post Tensioning

Pre Tensioning

The tension is applied to the tendons before casting of the concrete. The pre-compression is transmitted from steel to concrete through bond over the transmission length near the ends.

Post Tensioning

The tension is applied to the tendons (located in a duct) after hardening of the concrete. The pre-compression is transmitted from steel to concrete by the anchorage device (at the ends blocks).

Linear and Circular Prestressing

Linear Prestressing

When the prestressed members are straight or flat, in the direction of prestressing, the prestressing is called linear prestressing. For example, prestressing of beams, piles, poles and slabs. The profile of the prestressing tendon may be curved.

Circular Prestressing

When the prestressed members are curved in the direction of prestressing, the prestressing is called circular prestressing. For example, circumferential prestressing of tanks, silos, pipes and similar structures.

STAGES AND DEVICES OF PRETENSIONING AND POST TENSIONING AND THEIR ADVANTAGES AND DISADVANTAGES

Stages of Pre-tensioning

The various stages of the pre-tensioning operation are summarised as follows.

1. Anchoring of tendons against the end abutments
2. Placing of jacks
3. Applying tension to the tendons
4. Casting of concrete
5. Cutting of the tendons.

During the cutting of the tendons, the prestress is transferred to the concrete with elastic shortening and camber of the member.

Advantages of Pre-Tensioning

The relative advantages of pre-tensioning as compared to post-tensioning are as follows:

- Pre-tensioning is suitable for precast members produced in bulk.
- In pre-tensioning large anchorage device is not present.
- Prestress load is transferred in a single-stage process

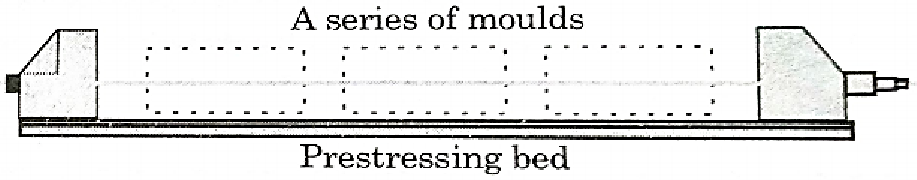
Disadvantages of Pre-Tensioning

The relative disadvantages are as follows.

- A prestressing bed is required for the pre-tensioning operation.
- There is a waiting period in the prestressing bed, before the concrete attains sufficient strength.

HOYER SYSTEM OF PRETENSIONING

Hoyer system is generally used for mass production (like Railway sleepers, poles etc) The end abutments are kept sufficient distance apart, and several members are cast in a single line. The shuttering is provided at the sides and between the members.

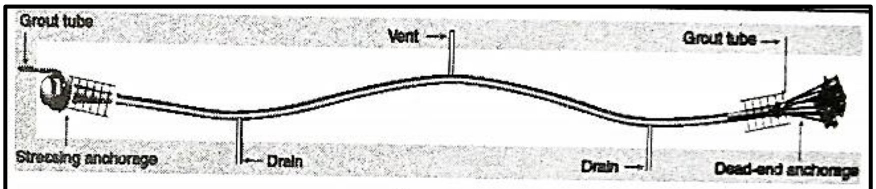


Schematic representation of Hoyer system

This system is also called the Long Line Method. The following figure is a schematic representation of the Hoyer system

Stages of Post tensioning

- Unlike pre-tensioning, the tendons are pulled with the reaction acting against the hardened concrete. If the ducts are filled with grout, then it is known as bonded post-tensioning.
- The grout is a neat cement paste or a sand-cement mortar containing suitable admixture.
- In unbounded post-tensioning, as the name suggests, the ducts are never grouted and the tendon is held in tension solely by the end anchorages.
- The following sketch shows a schematic representation of a grouted post-tensioned member. The profile of the duct depends on the support conditions.
- For a simply supported member, the duct has a sagging profile between the ends. For a continuous member, the duct sags in the span and hogs over the support.



Post-tensioning

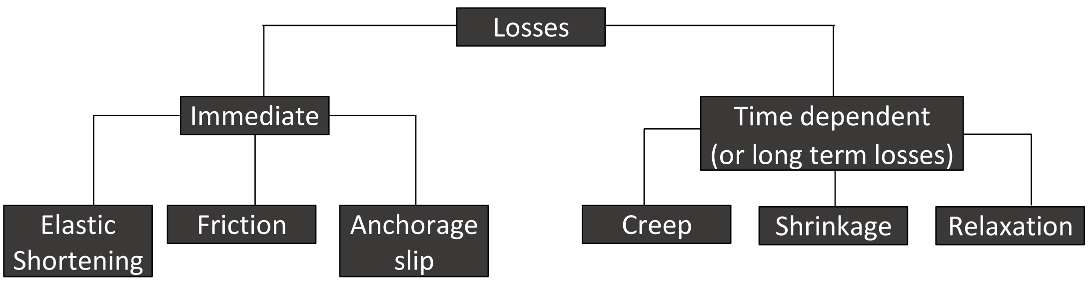
The various stages of the post-tensioning operation are summarised as follows.

Chapter – 13

LOSSES IN PRESTRESS

LOSSES IN PRESTRESS

- In prestressed concrete applications, the most important variable is the prestressing force. In the early days, it was observed that the prestressing force does not stay constant, but reduces with time.
- Even during prestressing of the tendons and the transfer of prestress to the concrete member, there is a drop of the prestressing force from the recorded value in the jack gauge.
- The various reductions of the prestressing force are termed as the losses in prestress.
- The losses are broadly classified into two groups, immediate and time-dependent.
- The immediate losses occur during prestressing of the tendons and the transfer of prestress to the concrete member.
- The time-dependent losses occur during the service life of the prestressed member.
- The losses due to elastic shortening of the member, friction at the tendon-concrete interface and slip of the anchorage are the immediate losses.
- The losses due to the shrinkage and creep of the concrete and relaxation of the steel are the time dependent losses.



The various losses in prestress are shown in the following chart.

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Contact:

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Test Series Available..

Total weekly test : 35

Total mid subject test : 16

Total full length test : 13

Mock test : 16

Total test : 80

Post-tensioned Members

If there is only one tendon, there is no loss because the applied prestress is recorded after the elastic shortening of the member. For more than one tendon, if the tendons are stretched sequentially, there is loss in a tendon during subsequent stretching of the other tendons.

Elastic Shortening Loss Calculation

The elastic shortening loss is quantified by the drop in prestress (Δf_s) in a tendon due to the change in strain in the tendon ($\Delta \varepsilon_s$)

- It is assumed that the change in strain in the tendon is equal to the strain in concrete (ε_c) at the level of the tendon due to the prestressing force. This assumption is called strain compatibility between concrete and steel.
- The strain in concrete at the level of the tendon is calculated from the stress in concrete (f_c) at the same level due to the prestressing force.
- A linear elastic relationship is used to calculate the strain from the stress.

The quantification of the losses is explained below.

$$\Delta f_s = E_s \Delta \varepsilon_s = E_s \varepsilon_c$$

$$= E_s \left(\frac{f_c}{E_c}\right)$$

$$\boxed{\Delta \boldsymbol{f}_s = \boldsymbol{m f}_c}$$

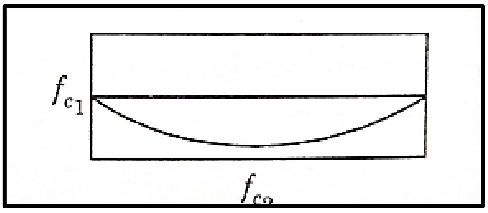
Where,

m = Modular ratio

- In case of pre-tensioning with axial pre-stressing we will have

$$f_{c_{avg}} = \frac{f_{c_1} + f_{c_2}}{2}$$

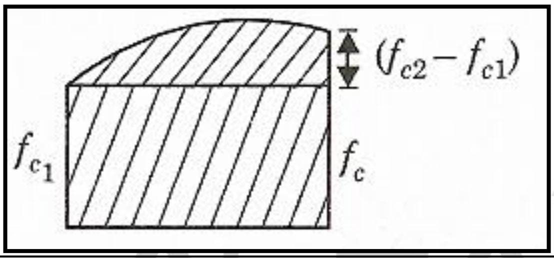
For Parabolic Cable Profile



$$f_{c_{avg}} = f_{c_1} + \frac{2}{3}(f_{c_2} - f_{c_1})$$

Note

$$f_{c_1} + \frac{\frac{2}{3}(f_{c_2} - f_{c_1}) \times l}{l}$$



$$\textbf{Prestressing loss} = \mathbf{m} \times f_{c\,avg}$$

Note

1. While calculating f_c we use the initial prestressing force (without loss) and the area of the section will be taken as b×d
2. The same concept of averaging will be used in case of post tensioning when all bars are not stretched simultaneously.

Elastic Shortening for Post Tensioned Member

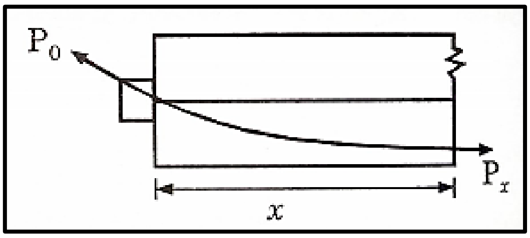
- In post tensioned member elastic shortening loss will be zero if all bars are stressed simultaneously and anchored. This is because initial prestressing force is measured only after tensioning and hence elastic shortening would have already occurred before the bar is anchored.
- However, if the bars are successively tensioned and anchored then losses will occur as described below.

P_0 = Prestressing force at jacking end.

k = Coefficient called wobble correction factor.

μ = Coefficient of friction in curve

α = Cumulative angle in radian through which the tangent to the cable profile has turned between any two point under consideration.



- For small values of $\mu\alpha + kx$, the above expression can be simplified by the Taylor series expansion.

$$P_x = P_0 (1 - \mu\alpha - kx)$$

Thus, for a tendon with single curvature, the variation of the prestressing force is linear with the distance from the stretching end. The following figure shows the variation of prestressing force.

- In the absence of test data, IS:1343 - 1980 provides guidelines for the values of μ and k.

Table: Values of coefficient of friction

Type of interface	μ
For steel moving on smooth concrete	0.55
For steel moving on steel fixed to duct	0.30
For steel moving on lead	0.25

- μ can be reduced by using lubrication like

  (i) Grease (ii) Graphite (iii) Paraffin oil

Calculation of ‘q’ for parabolic cable profile

Jacking at one end

$$x = L$$

$$y = \frac{4e}{L^2}x(L - x)$$

$$\frac{\mathrm{dy}}{\mathrm{dx}} = \frac{4e}{L^2}(L - 2x)$$

$$\frac{\mathrm{dy}}{\mathrm{dx}} = -\frac{4e}{L}$$

$$\boldsymbol{\theta = -\frac{4e}{L}}$$

$$\boldsymbol{\alpha = 2\theta = \frac{8e}{L}}$$

Jacking from both ends

$$\mathrm{x} = \frac{L}{2}$$

$$\frac{\mathrm{dy}}{\mathrm{dx}} = \frac{4h}{L}\left(L - 2 \times \frac{L}{2}\right) = 0$$

$$\boldsymbol{\theta = 0}$$

$$\alpha = \theta - (-0)$$

$$\boldsymbol{\alpha = \theta = \frac{4e}{L}}$$

Combined effect of friction and wobble effect leads to

$$P_x = P_0 e^{(\mu\alpha + kx)}$$

After expansion we have,

$$\mathrm{P}_x = \mathrm{P}_0(1 - \mu\alpha - kx)$$

$$\mathrm{P}_x = \mathrm{P}_0 - \mathrm{P}_0(\mu\alpha + kx)$$

$$\mathrm{P}_0(\mu\alpha + kx) = \mathrm{P}_0 - \mathrm{P}_x = \text{loss in prestress}$$

- Since the loss of stress is caused by a definite total amount of shortening, the percentage loss is higher for short members than for long ones.
- With the long-line pre tensioning system, the slip at the anchorage is normally very small in comparison with the length of the tensioned wire and hence is generally ignored.
- While prestressing a short member, due care should be taken to allow for the loss of stress due to anchorage slip, which forms a major portion of the total loss.

LOSS OF PRESTRESS DUE TO CREEP OF CONCRETE

Creep is the property of concrete by which it continues to deform with time under sustained loading.

Creep coefficient is defined as $\phi = \frac{\text{creep strain}}{\text{elastic strain}} = \frac{\varepsilon_{cp}}{\varepsilon_c}$

$$\varepsilon_{cp} = \phi \varepsilon_c \qquad \ldots (i)$$

$$\varepsilon_c = \frac{f_c}{E_c}$$

$$\varepsilon_{cp} = \phi \frac{f_c}{E_c}$$

Loss of stress = $\varepsilon_{cp} \times E_s$

$$= \phi . \frac{f_c}{E_c} \times E_s$$

$$\text{m} = \frac{E_s}{E_c}$$

$$\textbf{Loss of stress} = \mathbf{m}\boldsymbol{\phi f_c}$$

- Note that elastic shortening loss multiplied by creep co-efficient is equal to loss due to creep.

- The value may be increased by 50 per cent in dry atmospheric conditions, subject to a maximum value of $3x10^{-4}$ units.

The loss of stress in steel due to the shrinkage of concrete is estimated as,

$$\textbf{Loss of stress} = \varepsilon_{cs} \times E_s$$

Where, E_s = modulus of elasticity of steel.

LOSS OF PRESTRESS DUE TO RELAXATION OF STEEL

- Relaxation of steel is defined as the decrease in stress with time under constant strain.
- Due to the relaxation of steel, the prestress in the tendon is reduced with time.
- The relaxation depends on the type of steel, initial prestress (f_{pi}) and the temperature.
- The magnitude of loss of prestress due to relaxation of steel is in the range of 2 to 5% generally.
- To calculate the drop (or loss) in prestress (Δf_p), the recommendations of IS:1343 - 1980 can be followed in absence of test data.

Table: Relaxation losses for prestressing steel at 1000 H at 27 C (as per IS 1343 - 1980)

Initial Stress	Relaxation Loss N/mm²
$0.5f_p$	0
$0.6f_p$	35
$0.7f_p$	70
$0.8f_p$	90

Note
- f_p is the characteristic strength of prestressing steel

CLEAR YOUR CONCEPT

Qu1 The phenomena of development of internal tensile stresses in a concrete member by means of tensioning devices are called as ____________

a) Pre-tensioning

b) Post-tensioning

c) Prestressing of concrete

d) Thermoelectric prestressing

Qu2 In reinforced concrete members the prestress is commonly introduced by ___________

a) Tensioning the steel reinforcement

b) Tendons

c) Shortening of concrete

d) Rings

Qu3 Which of the following basic concept is involved in the analysis of prestressed concrete members?

a) Combined and bending stresses

b) Principal stresses

c) Shear stresses

d) Overhead stresses

Qu4 The prestressing of concrete member is carried out to reduce ___________

a) Compressive stresses

b) Tensile stresses

c) Bending stresses

d) Shear force