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Discover, Learn, and Innovate in Civil Engineering

UNIT – I

Part - A (2 marks)

1. Write a short note on limit state of durability. [N/D-15]

The acceptable limit for safety and serviceability requirements before failure occurs is called a limit state. The aim of design is to achieve acceptable probabilities that the structure will not become unfit for the use for which it is intended, that is, that it will not reach a limit state.

2. What is partial safety factor? [N/D-15]

Factors of safety (FoS), also known as (and used interchangeably with) safety factor (SF), is a term describing the load carrying capacity of a system beyond the expected or actual loads. Essentially, the factor of safety is how much stronger the system is than it usually needs to be for an intended load.

3. Write any two assumptions are made in elastic theory methods. [M/J-16]

The following are the assumptions made in working stress method:

a) At any cross-section, plane sections before bending remain plain after bending
b) All tensile stresses are taken up by reinforcement and none by concrete, except as otherwise specifically permitted
c) The stress-strain relationship of steel and concrete, under working loads, is a straight line
d) The modular ratio *m* has the value -280/3

4. What is the formula used to find the critical neutral axis in working stress method? [M/J-16], [M/J-11]

$$n_c = 1/ (1+ (\sigma_{st}/ m . \sigma_{bc})) d$$

where,

σ_{bc} is permissible stress in concrete.

σ_{st} is permissible stress in steel

m is modular ratio

5. Write any two advantages of limit state over other methods. [N/D-15], [N/D-11]

The advantages of limit state method over the other methods are the following:

a) In the limit state method of analysis, the principles of both elastic as well as plastic theories used and hence suitable for concrete structures

b) The structure designed by limit state method is safe and serviceable under design loads and at the same time it is ensured that the structure does not collapse even under the worst possible loading conditions

c) The process of stress redistribution, moment redistribution etc., are considered in the analysis and more realistic factor of safety values are used in the design

d) Hence the design by limit state method is found to be more economical

e) The overall sizes of flexural members (depth requirements) arrived by limit state method are less and hence they provide better appearance to the structure

6. What is the formula used to find the actual neural axis in working stress methods. [N/D-16]

$$0.5\, b\, n_a^2 = m\, A_{st}\, (d - n_a)$$

where,

b = width of beam

d = effective depth of beam

n_a = actual Neutral axis

7. Define collapse load. [M/J-13]

The load that causes the(n +1) th hinge to form a mechanism is called collapse load where n is the degree of statically indeterminacy. Once the structure becomes a mechanism.

8. Enlist different factors that are influencing the durability of concrete as per BIS? [M/J-13]

Factors affecting durability of concrete. Concrete durability has been defined by the American concrete institute as its resistance to weathering action, chemical attack, abrasion and other degradation processes. Durability is the ability to last a long time without significant deterioration.

9. What are the assumptions made in the working stress method? [N/D-12], [M/J-11]

a) At any cross-section, plane sections before bending remain plain after bending.

b) All tensile stresses are taken up by reinforcement and none by concrete, except as otherwise specifically permitted.

c) The stress-strain relationship of steel and concrete, under working loads, is a Straight line. d) The modular ratio m has the value -280/3σbc.

10. What is meant by balanced section? [N/D-

12]

When the maximum stress in steel and concrete simultaneously reach their allowable values, the section is said to be balanced section. In this section the actual neutral axis depth is equal to the critical neutral axis.

11. Define characteristic strength in limit state method. [M/J-12]

The term 'characteristic strength' means that value of the strength of the material below which not more than 5 percent of the test results are expected to fall.

12. Define: Limit state. [M/J-12]

The acceptable limit for the safety and serviceability requirements before failure occurs is called a 'limit state'. The aim of design is to achieve acceptable probabilities that the structure will not become unfit for the use for which it is intended, that is, that it will not reach a limit state.

Part - B (16 marks)

1. Explain the codal recommendations for limit states design? State their significance. (16) [N/D-15], [N/D-16], [N/D-12], [M/J-11]

Limit state design (LSD), also known as load and resistance factor design (LRFD), refers to a design method used in structural engineering. A limit state is a condition of a structure beyond which it no longer fulfills the relevant design criteria.[1] The condition may refer to a degree of loading or other actions on the structure, while the criteria refer to structural integrity, fitness for use, durability or other design requirements. A structure designed by LSD is proportioned to sustain all actions likely to occur during its design life, and to remain fit for use, with an appropriate level of reliability for each limit state. Building codes based on LSD implicitly define the appropriate levels of reliability by their prescriptions.

Limit state design requires the structure to satisfy two principal criteria:

1. Ultimate Limit state (ULS)
2. Serviceability limit state (SLS).

Any design process involves a number of assumptions. The loads to which a structure will be subjected must be estimated, sizes of members to check must be chosen and design criteria must be selected. All engineering design criteria have a common goal: that of ensuring a safe structure and ensuring the functionality of the structure.

1. Ultimate Limit State

A clear distinction is made between the Ultimate State (US) and the Ultimate Limit State (ULS). The US is a physical situation that involves either excessive deformations leading and approaching collapse of the component under consideration or the structure as a whole, as relevant, or deformations exceeding pre agreed values. It involves of course considerable inelastic (plastic) behavior of the structural scheme and residual deformations. While the ULS is not a physical situation but rather an agreed computational condition that must be fulfilled, among other additional criteria, in order to comply with the engineering demands for strength and stability under design loads. The ULS condition is computationally checked at a certain point along the behavior function of the structural scheme, located at the upper part of its elastic zone at approximately 15% lower than the elastic limit. That means that the ULS is a purely elastic condition, located on the behavior function far below the real Ultimate point, which is located deeply within the plastic zone. The rationale for choosing the ULS at the upper part of the elastic zone is that as long as the ULS design criteria is fulfilled, the structure will behave in the same way under repetitive loadings, and as long as it keeps this way, it proves that the level of safety and reliability assumed as the basis for this design is properly maintained and justified, (following the probabilistic safety approach). A structure is deemed to satisfy the ultimate limit state criterion if all factored bending, shear and tensile or compressive stresses are below the factored resistances calculated for the section under consideration. The factored stresses referred to are found by applying Magnification Factors to the loads on the section. Reduction Factors are applied to determine the various factored resistances of the section.

2. Serviceability Limit State

1) limit state of deflection.

2) limit state of cracking.

3) limit state of vibration.

In addition to the ULS check mentioned above, a Service Limit State (SLS) computational check must be performed. As for the ULS, here also the SLS is not a physical situation but rather a computational check. The aim is to prove that under the action of Characteristic design loads (un-factored), and/or whilst applying certain (un-factored) magnitudes of imposed deformations, settlements, or vibrations, or temperature gradients etc. the structural behavior complies with, and does not exceed, the SLS design criteria values, specified in the relevant standard in force. These criteria involve various stress limits, deformation limits (deflections, rotations and curvature), flexibility (or rigidity) limits, dynamic behavior limits, as well as crack control requirements (crack width) and other arrangements concerned with the durability of the structure and its level of everyday service level and human comfort achieved, and its abilities to fulfill its everyday functions. In view of non-structural issues it might also involve limits applied to acoustics and heat transmission that might also affect the structural design. To satisfy the serviceability limit state criterion, a structure must remain functional for its intended use subject to routine (read: everyday) loading, and as such the structure must not cause occupant discomfort under routine conditions. This calculation check is performed at a point located at the lower half of the elastic zone, where characteristic (un-factored) actions are applied and the structural behavior is purely elastic.

Some of important limit states of failure or collapse are listed below:

a) Failure of one or more critical sections in flexure,shear,torsion or due to their combinations
b) Failure due to fatigue under repeated load
c) Failure due to bond and anchorage failure of reinforcement
d) Failure due to elastic instability of structural members
e) Failure due to impact, earthquake, fire or frost
f) Failure due to destructive effects of chemicals, corrosion of reinforcements

Limit state design (LSD) refers to a design method used in structural engineering. A limit state is a condition of a structure beyond which it no longer fulfills the relevant design criteria. The condition may refer to a degree of loading or other actions on the structure, while the criteria refer to structural integrity, fitness for use, durability or other design requirements. A structure designed by LSD is proportioned to sustain all actions likely to occur during its design life, and to remain fit for use, with an appropriate level of reliability for each limit state. Building codes based on LSD implicitly define the appropriate levels of reliability by their prescriptions.

2. Design a rectangular section for a simply supported reinforced concrete beam of effective span of 5 m carrying a concentrated load of 40 kN at its mid span. The concrete to be used is of grade M 20 and the reinforcement consists of Fe 415 steel bars.

(i) Self weight of beam is ignored.
(ii) Self weight of beam is considered; Use working stress method. (16) [N/D-15],

[N/D-16]

Soln: Permissible stresses:

$\sigma_{cbc} = 7\ N/mm^2$

$\sigma_{st} = 230\ N/mm^2$

$m = 13.33 \quad \left(\frac{280}{3\sigma_{cbc}}\right)$

Cross Sectional Dimensions:

$M_R = Q.b.d^2$

$Q = 0.5\ \sigma_{cbc}\ k.\ J.$

$k = \frac{280}{280 + 3\sigma_{st}} = \frac{280}{280 + 3 \times 230} = 0.29$

$J = 1 - k/3 = 1 - \frac{0.29}{3} = 0.9$

$Q = 0.5 \times 7 \times 0.29 \times 0.9 = 0.91$

$\therefore d = \sqrt{\frac{M}{Qb}} = \sqrt{\frac{65 \times 10^6}{0.91}}$

$65 \times 10^6 = 0.91 \times 0.5\ d \times d^2$

$65 \times 10^6 = 0.455\ d^3$

$\therefore d = 522.8\ mm \simeq 530\ mm$

$\therefore b = d/2 = \frac{522.8}{2} = 261.4\ mm \simeq 265\ mm$

Cover = 40 mm.

$\therefore$ Overall depth (D) = (530 + 40) = 570 mm

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∴ Adopt the section 265 mm by 570 mm.

Reinforcements:

Area of Reinforcement at Tension

$$A_{st} = \frac{M_r}{\sigma_{st}\, j\, d} = \frac{65 \times 10^6}{230 \times 0.9 \times 530} = 593\ mm^2$$

∴ use 3 no. of 16 mm ϕ of bar

3. A Reinforced concrete rectangular beam is supported on two wall 750 mm thick, spaced at a clear distance of 6 m. The beam carries a super imposed load of 3 kN/m. Design the beam in working stress method. M20 grade concrete Fe 250 bars. Draw reinforcement details. (16)
[M/J- 16], [N/D-12]

Given data:-

Moment $M = 60 kNm$

Width $b = 300mm$

Effective depth $d = 500mm$

M_{20}, $\sigma_{cbc} = 7 N/mm^2$

Fe_{415} $\sigma_{st} = 230 N/mm^2$

Solution:-

Balanced section Moment

$$M_{Bal.} = \frac{\sigma_{cbc}}{2} \times bd^2 k_{bal}\left(1 - \frac{k_{bal}}{3}\right)$$

$$\text{Modular ratio, } m = \frac{280}{3cbc} = \frac{280}{3 \times 7}$$

$$= 13.33$$

$$k_{Bal} = \frac{1}{\left(\frac{\sigma_{st}}{m\,\sigma_{cbc}}\right) + 1}$$

$$= \frac{1}{\left(\frac{230}{13.33 \times 7}\right) + 1} = 0.289$$

$$M_{Bal} = 7/2 \times 300 \times 500^2 \times 0.289 \left(1 - \frac{0.\ldots}{}\right.$$

$$M_{Bal} = 68.55 \times 10^6 \, N/mm$$

$$M < M_{Bal}$$

It is singly reinforced section.

$$M = A_{st} \, \sigma_{st} \left(d - x/3\right)$$

$$x = kd = 0.289 \times 500 = 144.50 \, mm$$

$$60 \times 10^6 = A_{st} \times 230 \times \left(500 - \frac{144.5}{3}\right)$$

$$A_{st} = 577.36 \, mm^2$$

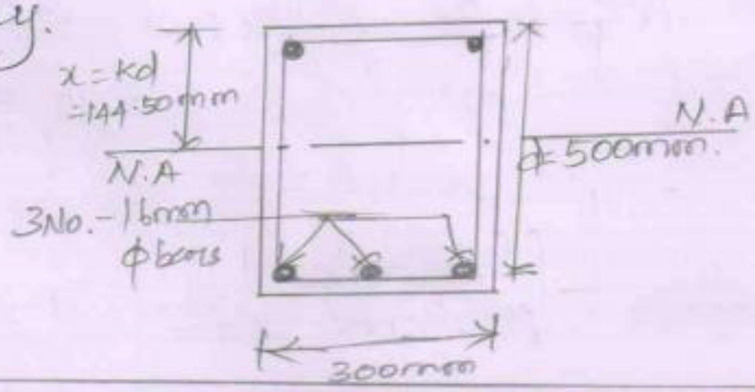
Provide 16mm ϕ bars.

$$\text{No. of bars required} = \frac{577.36}{201} = 2.87$$

$$= 3 \, Nos.$$

Provide 3 No. of 16mmϕ bars at tension zone only.

cross section of Beam



4. Design one way simply supported slab on a clear span of 4m, the width of the supports being 300mm. The dead load on the slab may be taken as 1000N/m^2 excluding its self weight. The live load on the slab is 2000 N/m^2. Use M20 grade concrete and Fe415 grade steel. Adopt working stress method. (16) [M/J-16]

Solution:-

load calculation:-

Consider thickness of slab = 125mm

Selfweight of slab = 0.125×25 = 3.125 kN/m²

live load = 2.5 kN/m²

Floor finish = 1.0 kN/m²

Total load = 6.625 kN/m²

Max. B.M for Cantilever slab = $\frac{Wl_e^2}{2}$

Moment at support = $\frac{6.625 \times 1.5^2}{2}$

= 7.45 kNm

Thickness of slab calculations:-

Moment = Qbd^2

$Q = 0.91$ and $j = 0.90$ for M_{20} & Fe_{415}

$7.45 \times 10^6 = 0.91 \times 1000 \times d^2$

$d_{required} = 90.48$ mm

Provide 8mm diameter bar & clear cover for slab = 15mm

Effective depth provided = D − Clear cover − $\frac{\text{bardia}}{2}$

= 125 − 15 − 8/2

= 106mm

$d_p > d_{req}$ Hence safe.

Area of main steel calculation

$$A_{st} = \frac{M}{\sigma_{st} j d}$$

$$= \frac{7.45 \times 10^6}{230 \times 0.90 \times 106} = 339.53 mm^2$$

$$Spacing = \frac{50}{339.53} \times 1000$$

$$= 147.26 mm$$

Provide 8mm diameter bar at 145mm c/c.

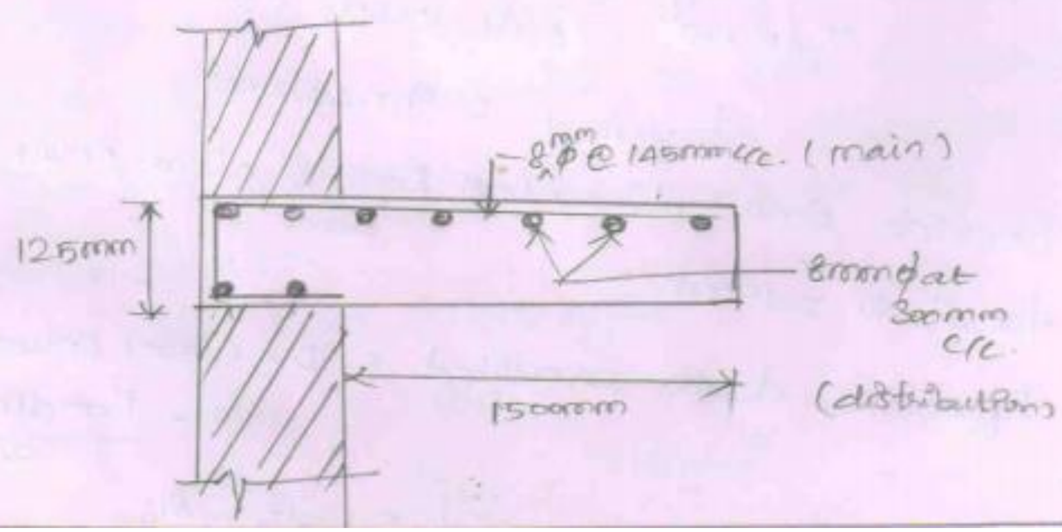
Area of distribution steel calculation

$$A_{st\ distribution} = \frac{0.12}{100} \times b \times D$$

$$= \frac{0.12}{100} \times 1000 \times 125 = 150 mm^2$$

$$Spacing = \frac{50}{150} \times 1000 = 333.33 mm$$

Provide 8mm diameter bar at 300mm c/c.



5. Explain the concept of elastic method and ultimate load method And write the advantages of limit state method over other methods. [M/J-13], [M/J-11]

Three Major Design Philosophies

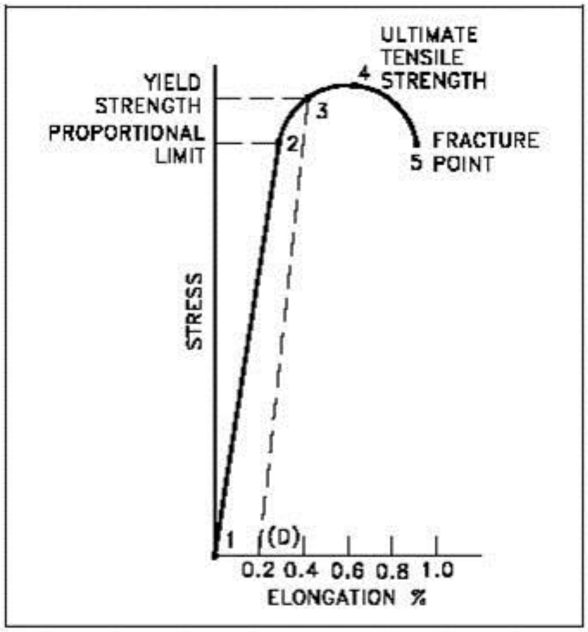
A design philosophy is a set of assumptions and procedures which are used to meet the conditions of serviceability, safety, economy and functionality of the structure. Several design philosophies have been introduced from different parts of the world. Some of the design philosophies that has been used by engineers are

1. Working Stress Method(WSM)/ Allowable Stress Design (ASD)
2. Ultimate Load Method (ULM)
3. Limit State Method(LSM)

The earliest one is working stress method, which was introduced in 20th century. This is based on linear elastic theory. This method was used in IS 456 till revision IS 456:2000. In 1950s Ultimate Load method was introduced which is based on the ultimate load which can be carried out by material. Its acceptance can be seen in ACI code in 1956 and British codes in 1957 and Indian codes in 1964. The most recently accepted code of practice is based on Limit State method. This is used in IS 456 from revision IS 456:2000, British code CP 110(1973) (now BS 8110(1997)) and ACI 318-71 (now ACI 318-95).

Working Stress Method/ Allowable Stress Design:

Working stress method is used for the design of Reinforced concrete, Steel and Timber structures. The main assumption in the WSM is that the behaviour of structural material is restricted with in linear-elastic region and the safety of it is ensured by restricting the stresses coming on the members by working loads. Thus the allowable stresses will come in the linear portion (i.e., initial phase) of the stress-strain curve. Thus a factor of safety was introduced to the design



When we consider the effect of creep, shrinkage, stress concentrations and others secondary effects the assumption of material behavior in the elastic range will not hold. These will lead to increase of stresses into the inelastic range. WSM cannot account for loads acting simultaneously, but has different degrees of uncertainty. It cannot account for the loads having counteracting effects, such as dead load and wind load. The above will lead to non-conservative design. Working Stress method will lead to large FOS and over-sized sections, thus reducing the design economy.

WSM is still being using in special structures such as water tanks, chimneys in India. Elastic regions holds good in serviceability checks such as crack width, deflection etc.

Ultimate Load Method:

This is also known as load factor method or ultimate strength method. In this we make use of the nonlinear region of stress strain curves of steel and concrete. The safety is ensured by introducing load factor.

"Load factor is the ratio of ultimate strength to the service loads"

The ULM makes it possible to consider the effects of different loads acting simultaneously thus solving the shortcomings of WSM. As the ultimate strength of the material is considered we will get much slender sections for columns and beams compared to WSM method. But the serviceability criteria is not met because of large deflections and cracks in the sections. The fall-back in the method was that even though the nonlinear stress strain behaviour of was considered sections but the nonlinear analysis of the structural was not carried out for the load effects. Thus the stress distribution at ultimate load was just the magnification of service load by load factor following the linear elastic theory.

This philosophy is an advancement over the traditional design philosophies. It considers the safety at the ultimate load and serviceability at the working load, sort of extension of the WSM and ULM.

"Limit state is the state of impending failure, beyond which a structure ceases to perform its intended function satisfactorily, in terms of either safety or serviceability."

There are 2 types of limit states

1. Ultimate Limit State: It considers strength, overturning, fatigue, sliding etc.
2. Serviceability Limit State: It considers crack width, deflection, vibration etc.

It uses multiple safety factors for the required safety and serviceability at the ultimate load and working load respectively by considering all limit states. These are called "partial safety factors".

Partial safety factor for materials:

The strength of concrete in actual structure is taken as (0.67 *characteristic strength), i.e. 0.67 $f_{ck.}$ The partial safety factor (for ultimate limit state) for concrete is 1.5 and that for steel is 1.15. The value is higher for concrete as it is expected to have more variability compared to steel. The partial safety factor (for serviceability limit state) for concrete and that for steel is taken as 1.0. This is taken as unity as we are interested in estimating the actual deflections are crack width during service loads.

Partial safety factor for loads:

Various load combinations is specified in IS 456 are

For Ultimate limit states

- UL = 1.5(DL+LL)
- UL = 1.5(DL+QL) OR 0.9DL+1.5QL
- UL = 1.2(DL+LL+QL)

The load factor of 1.2 is considered for the combination of three because the probability of the three loads reaching its peak together are less.

For serviceability limit states

- SL = 1.0 (DL+LL)
- SL = 1.0 (DL+QL)
- SL = 1.0DL+ 0.8LL+ 0.8 QL

The load factor is taken as 0.8 in the third case as the probability of wind load or earthquake load acting with the peak of live load is less. For all cases the safety factor is taken as 1.0 as we are considering the serviceability of structure here.

The advantages of limit state method over the other methods are the following:

a. In the limit state method of analysis, the principles of both elastic as well as plastic theories used and hence suitable for concrete structures
b. The structure designed by limit state method is safe and serviceable under design loads and at the same time it is ensured that the structure does not collapse even under the worst possible loading conditions
c. The process of stress redistribution, moment redistribution etc., are considered in the analysis and more realistic factor of safety values are used in the design
d. Hence the design by limit state method is found to be more economical
e. The overall sizes of flexural members (depth requirements) arrived by limit state method are less and hence they provide better appearance to the structure
f. Because of the modified assumptions regarding the maximum compressive strains in concrete and steel, the design of compressive reinforcement for double reinforced beams and eccentrically loaded columns by limit state method gives realistic values which is not so in other methods.

6. Design a R.C beam to carry a load of 6 kN/m inclusive of its own weight on an effect span of 6m keep the breath to be 2/3 rd of the effective depth .the permissible stressed in the concrete and steel are not to exceed 5N/mm2 and 140 N/mm2.takem=18.
[N/D-15], [M/J-12]

Step 1: Design constants. Modular ratio:

m =18.

Coefficient n σbc.m/(σbc.m + σst) 0.39

Lever arm Coefficient, j=1-(n/3)

= 0.87

Moment of resistance Coefficient

Q σbc/2. n. j

= 0.84

Step 2: Moment on the beam:

M = (w.l2)/8

= (6x62)/8

= 27kNm

M = Qbd^2 d2

= M/Qb

= (27x106)/ (0.84x2/3xd)

d = 245mm.

Step 3: Balanced Moment:

Mbal = Qbd^2

= 0.84x245x3652

= 27.41kNm. > M.

It can be designed as singly reinforced section.

Step 4: Area of steel:

Ast = Mbal / (σst.j.d)

=616.72mm^2

Use 20mm dia bars

Ast = $\pi/4\ (20^2)$

= 314.15mm^2

No. of bars

= Ast/ast

= 616.72/314.15

= 1.96

say 2nos.

Provide 2#20mm dia bars at the tension side

7. Differentiate between working stress method and limit state method. [M/J-13], [N/D-12]

a) In the limit state method of analysis, the principles of both elastic as well as plastic theories used and hence suitable for concrete structures.
b) The structure designed by limit state method is safe and serviceable under design loads and at the same time it is ensured that the structure does not collapse even under the worst possible loading conditions.
c) The process of stress redistribution, moment redistribution etc., are considered in the analysis and more realistic factor of safety values are used in the design. Hence the design by limit state method is found to be more economical.
d) The overall sizes of flexural members (depth requirements) arrived by limit state method are less and hence they provide better appearance to the structure.
e) Because of the modified assumptions regarding the maximum compressive strains in concrete and steel, the design of compressive reinforcement for double reinforced beams and eccentrically loaded columns by limit state method gives realistic valued which is not so in other methods.

UNIT – II

Part - A (2 marks)

1. Differentiate between under reinforced and over reinforce section [N/D-15], [M/J-12]

A beam reaches its permissible stress in steel under the working moment before concrete reaches its stress is called as Under reinforced section. A beam reaches its permissible stress in concrete under the working moment before steel reaches its stress is called over reinforced section.

2. Enumerate balanced section? [N/D-15], [N/D-12]

When the maximum stress in steel and concrete simultaneously reach their allowable values, the section is said to be balanced section. In this section the actual neutral axis depth is equal to the critical neutral axis.

3. Write any two guidelines to select the cross sectional dimensions of reinforced concrete beams. [M/J-16]

The deflection of a structure or part there of shall not adversely affect the appearance or efficiency of the structure or finishes or partitions. The deflection shall generally be limited to the following:

a) The final deflection due to all loads including the effects of temperature, creep and shrinkage and measured from the as-cast level of the, supports of floors, roofs and all other horizontal members, should not normally exceed span/250
b) The deflection including the effects of temperature, creep and shrinkage occurring after erection of partitions and the application of finishes should not normally exceed span/350 or 20 mm, whichever is less.

4. Enumerate the advantages of flanged beams. [M/J-16]

- Since the beam is casted monolithically with the slab, the flange also takes up the compressive stresses which mean it will be more effective in resisting the sagging moment acting on the beam.
- Better head room, this is direct outcome of the first point since the depth of the beam can be considerably reduced.

- For larger spans, t beams are usually preferred rather than rectangular beam as the deflection is reduced to a good extent.

5. On what circumstances doubly reinforced beams are to be adopted? [N/D-16], [M/J-12]

Situations in which doubly reinforced sections preferred are:

a) when the members are subjected to alternate external loads and the bending moment in the section reverses
b) when the members are subjected to loading eccentric on either side of axis
c) when overall size of beam section is limited
d) when beam section is continuous over several supports.

6. Write any two general features of two way slab? [N/D-16]

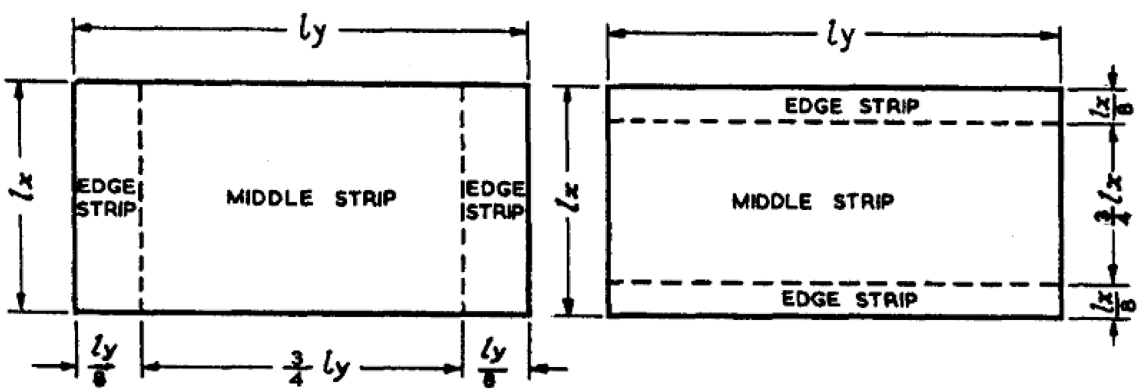
Two way slab is supported by beams in all four sides.

The ratio of longer span panel (L) to shorter span panel (B) is less than 2.

Thus, L/B < 2.

Main reinforcement is provided in both the directions for two way slabs.

7. Sketch the edge and middle strip of one way slab? [M/J-13]



25A For Span l_x 25B For Span l_y

Division of Slab into Middle and Edge Strips

8. What is doubly reinforced beam? [M/J-13], [M/J-11]

The section reinforced in both tension and compression is known as doubly reinforced beams. The doubly reinforced beams are adopted when the balanced moment is smaller than the actual moment.

9. Explain the check for deflection control in the design of slabs? [N/D-12]

The deflection of a structure or part thereof shall not adversely affect the appearance or efficiency of the structure or finishes or partitions. The deflection shall generally be limited to the following:

a) The final deflection due to all loads including the effects of temperature, creep and shrinkage measured from the as-cast level of the , supports of floors, roofs and all other horizontal members, should not normally exceed span/250.

b) The deflection including the effects of temperature, creep and shrinkage occurring after erection of partitions and the application of finishes should not normally exceed span/350 or 20 mm whichever is less.

10. When do you do for doubly reinforced beams? [N/D-12], [M/J-13]

The section reinforced in both tension and compression zone is known as doubly reinforced section. The doubly reinforced beams are adopt when the balanced moment is smaller than the actual moment.

Part - B (16 marks)

1. On what circumstances doubly reinforced beams are to be adopted? [N/D-16], [M/J-11]

Situations in which doubly reinforced sections preferred are:

1. when the members are subjected to alternate external loads and the bending moment in the section reverses
2. when the members are subjected to loading eccentric on either side of axis
3. when overall size of beam section is limited
4. when beam section is continuous over several supports.

The section reinforced in both tension and compression is known as doubly reinforced beams. The doubly reinforced beams are adopted when the balanced moment is smaller than the actual moment.

2. Write any two general features of two way slab [N/D-16], [N/D-12], [M/J-13]

One way slab is supported by beams on only 2 sides.

The ratio of longer span panel (L) to shorter span panel (B) is equal to or greater than 2. Thus, L/B ≥ 2

Main reinforcement is provided in only one direction for one way slabs.

Two way slab is supported by beams in all four sides.

The ratio of longer span panel (L) to shorter span panel (B) is less than 2.

Thus, L/B < 2.

Main reinforcement is provided in both the directions for two way slabs.

Moment developed in slab are influenced by following factors:

a) Short and long span length L_X & L_Y
b) Type of supporting edges such as free, fixed and continous
c) Magnitude and type of load on slab such as concentrated, uniformly distributed, etc.,

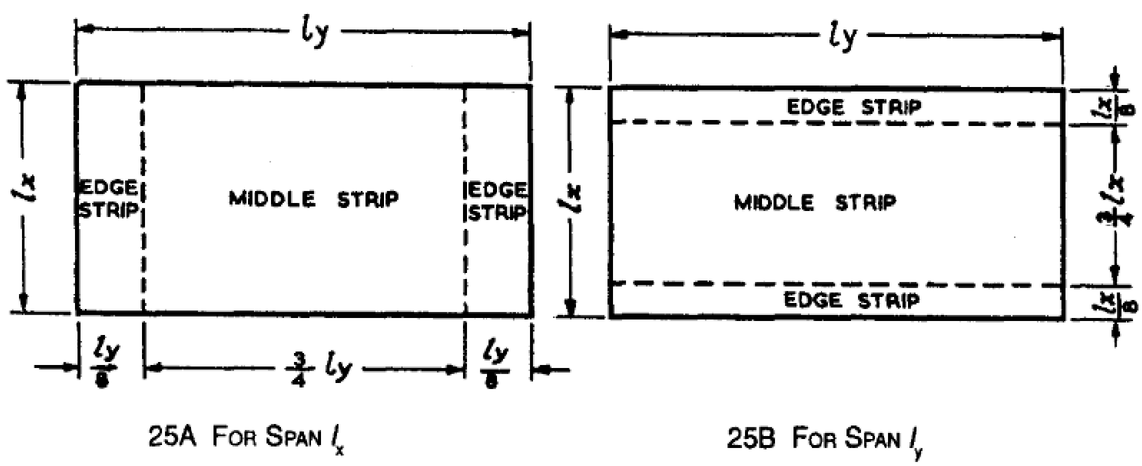


Fig. 25 Division of Slab into Middle and Edge Strips

The depth of slab chosen from deflection requirements will be usually greater than the depth required for balanced design. Hence, the area of steel required will be less than the balanced amount. So, the slab is designed as under reinforced section.

The deflection of a structure or part there of shall not adversely affect the appearance or efficiency of the structure or finishes or partitions. The deflection shall generally be limited to the following:

1. The final deflection due to all loads including the effects of temperature, creep and shrinkage and measured from the as-cast level of the, supports of floors, roofs and all other horizontal members, should not normally exceed span/250
2. The deflection including the effects of temperature, creep and shrinkage occurring after erection of partitions and the application of finishes should not normally exceed span/350 or 20 mm, whichever is less.

3. A T-beam slab floor of an office comprises of a slab 150 mm thickness spanning between ribs spaced at 3 m centres. The effective span of the beam is 8 m. Live load on floor is 4 kN/m^2. Using M20 grade and Fe 415 HYSD bars. Design one of the intermediate tee beams. Use limit state method. (16) [M/J-16], [N/D-16]

Given data :-

width of flange b_f = 1500mm

Thickness of flange D_f = 100mm

width of beam b or b_w = 250mm

ultimate moment M_u = 150 kNm

Solution :-

(1) limiting moment of resistance ($M_{u\,limit}$) Calculation.

If $M_u < M_{u\,lim}$ T-beam can be designed as singly reinforced section.

$$M_{u\,lim} = 0.36 f_{ck} b_f x_{u\,lim} (d - 0.42 x_{u\,lim})$$

Consider $x_{u\,lim} = D_f$ = 100mm → (1)

i.e.,) limiting neutral axis coincide bottom of flange.

$x_{u\,lim} = 0.48\,d$ → (2)

$D_f = 0.48d$

$d = \frac{D_f}{0.48} = \frac{100}{0.48} = 208.33mm$

$M_{ulim} = 0.36 \times 20 \times 1500 \times 100\,(208.33 - 0.42 \times 100)$

$= 179.64 \times 10^6$ Nmm $= 179.64$ kNm

$M_u < M_{ulim}$ Beam can be designed as singly reinforced beam.

Area of steel calculation:-

$$A_{st} = \frac{M_u}{0.87 f_y (d - 0.42 x_u)}$$

Provide 20mmϕ bar

$D = d +$ clear cover $+ \phi/2 = 208.33 + 25 + \frac{20}{2}$

$= 243.33$ mm

Provide $D = 300$

$d = 300 - 25 - 20/2 = 265mm$

Assume neutral axis lies within the flange portion (ie) $x_u < D_f$

$M_u = 0.36 f_{ck} b_f x_u (d - 0.42 x_u)$

$(150 \times 10^6) = 0.36 \times 20 \times 1500 \times x_u (265 - 0.42 x_u)$

$150 \times 10^6 = 2862000 x_u - 4536 x_u^2$

$x_u = 57.67mm < D_f$

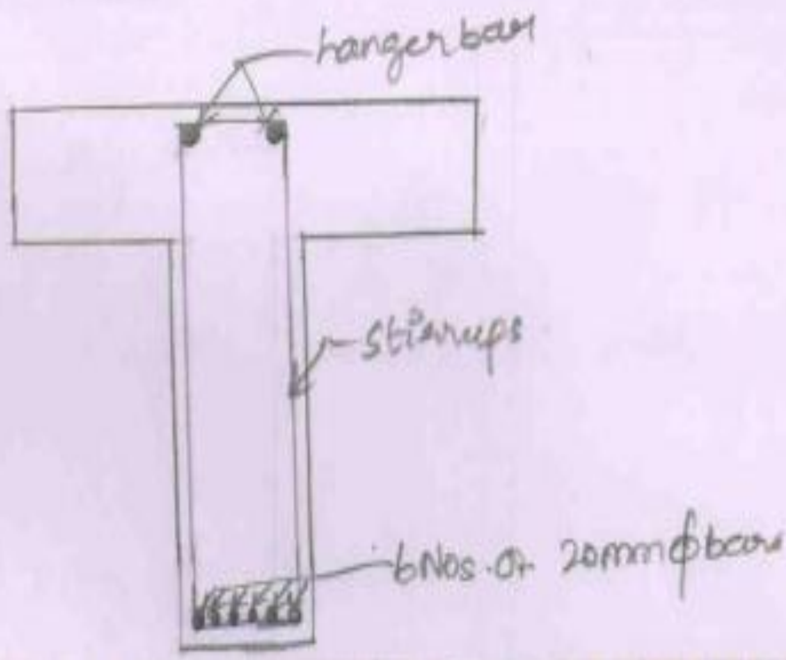
$$A_{st} = \frac{M_u}{0.87 f_y (d - 0.42 x_u)}$$

$$= \frac{150 \times 10^6}{0.85 \times 415 \times (265 - 0.42 \times 57.69)}$$

$$= 1733.91 mm^2$$

$$\text{No. of bars} = \frac{1733.91}{314} = 5.52 \approx 6 \text{ Nos}$$

Provide 6 Nos of 20mm ϕ bars in tension zone



4. Design a two way slab for an office floor size 35 mx4.5 m discontinuous and simply supported edges on all the sides with the corners prevented from lifting and supporting a service live load of 4.4 kN/m^2. Adopt M 20 grade and Fe 415 HYSD bars. (16)
[M/J-16], [N/D-12]

Given:

Size = 7mx5m

Width of Supports = 230 mm

Edge condition = interior Live load = 4kN/m 2

Floor finish = 1kN/m 2

Consider M 20 grade concrete and Fe 415 grade steel.

Step 1:

Type of Slab :

ly/lx = 7/5 = 1.4>2.

It has to be designed as two way slab.

Step 2:

Effective depth calculation:

For Economic consideration adopt shorter span to design the slab.

d = span/(basic value x modification factor)

= 5000/(20x0.95) = 270mm

D = 270 + 20 + 10/2

= 295mm

Step 3:

Effective Span:

For shorter span:

Le = clear span + effective depth = 5000 + 270 = 5.27m (or)

Le =c/c distance b/w supports = 5000 + 2(230/2)

= 5.23m Adopt effective span = 5.23m least value.

For longer span: Le = clear span + effective depth

= 7000 + 270 = 7.27m (or) Le =c/c distance b/w supports

= 7000 + 2(230/2) = 7.23m

Adopt effective span = 7.23m least value.

Step 4:

Load calculation:

Live load = 4kN/m2

Dead load = 1x1x0.27x25 = 6.75kN/m 2

Floor Finish = 1kN/m 2

Total load = 11.75kN/m 2

Factored load = 11.75 x 1.5 = 17.625kN/m 2

Step 5:

Moment calculation:

Mx = αx . w .

lx = 0.103x17.625x5.23 = 9.49kNm

My = αy . w . lx = 0.048 x17.625x5.23 = 4.425kNm

Step 6:

Check for effective depth:

M = Qbd2

d 2 = M/Qb = 9.49/2.76x1 = 149.39mm

say 150mm.

For design consideration adopt d = 150mm.

Step 7:

Area of Steel. For longer span:

Mu = 0.87 fy Ast d (1- (fy ast)/(fck b d)) 4.425x106 = 087x415xAstx150(1-(415 Ast)/(20x1000x150))

Ast = 180mm 2 Use 10mm dia bars

Spacing ,S = ast/Astx1000 = (78.53/300)1000 = 261mm

say 260mmc/c

Provide 10mm dia @260mm c/c.

For shorter span:

Mu = 0.87 fy Ast d (1- (fy ast)/(fck b d))

9.49x106 = 087x415xAstx150(1-(415 Ast)/(20x1000x150))

Ast = 200mm 2 Use 10mm dia bars Spacing ,

S = ast/Astx1000 = (78.53/300)1000 = 281mm say 300mmc/c

Provide 10mm dia @300mm c/c.

6..Design a slab over a room 5m x 7m as per I.S code. The slab is supported on masonry walls all round with adequate restraint and the corners are held down. The live load on the corners is held down. The live load on the slab is 330N/m^2. The slab has a bearing of 150 mm on the supporting walls. (16) [N/D-16], [N/D-13]

Step 1:

Type of Slab:

ly/lx = 4/1 = 4>2.

It has to be designed as one way slab.

Step 2:

Effective depth calculation:

d = span/(basic value x modification factor)

= 4000/(20x0.95) = 270mm

D = 270 + 20 + 10/2 = 295mm

Step 3:

Effective Span:

Le = clear span + effective depth = 4000 + 270 = 4.27m (or)

Le =c/c distance b/w supports = 4000 + 2(230/2) = 4.23m

Adopt effective span = 4.23m

least value.

Step 4:

Load calculation:

Live load = 4kN/m2

Dead load = 1x1x0.27x25 = 6.75kN/m 2

Floor Finish = 1kN/m 2

Total load = 11.75kN/m 2

Factored load = 11.75 x 1.5 = 17.625kN/m 2

Step 5:

Moment calculation:

M = wl2 /8 = (17.625x4.23 2)/8

= 60.26kNm

Step 6:

Check for effective depth:

M = Qbd2

d 2 = M/Qb = 60.26/2.76x1 = 149.39mm

say 150mm.

For design consideration

adopt d = 150mm.

Step 7:

Area of Steel:

Mu = 0.87 fy Ast d (1- (fy ast)/(fck b d)) 60.26x106 = 087x415xAstx150(1-(415 Ast)/(20x1000x150))

Ast = 300mm 2 Use 10mm dia bars Spacing ,

S = ast/Astx1000

= (78.53/300)1000

= 261mm say 260mmc/c

Provide 10mm dia @260mm c/c

7. Design a one way slab with a clear span of 5m, simply supported on 230mm thick masonry walls and subjected to a live load of 4kN/m² and a surface finish of 1kN/mm².Assume Fe 415 steel. Assume that the slab is subjected to moderate exposure conditions. [N/D-12], [M/J-12]

Step 1: Type of Slab:

ly/lx = 5/1 = 5>2.

It has to be designed as one way slab.

Step 2:Effective depth calculation:

d = span/(basic value x modification factor)

= 5000/(20x0.95)

= 270mm

D = 270 + 20 + 10/2

= 295mm

Step 3: Effective Span:

Le = clear span + effective depth

= 5000 + 270

= 5.27m (or)

e =c/c distance b/w supports

= 5000 + 2(230/2)

= 5.23m

Adopt effective span = 5.23m

least value.

Step 4: load calculation:

Live load = $4kN/m^2$

Dead load = 1x1x0.27x25

= $6.75kN/m^2$

Floor Finish = $1kN/m^2$

Total load = $11.75kN/m^2$

Factored load = 11.75 x 1.5

= $17.625kN/m^2$

Step 5: Moment calculation:

M = $wl^2/8$

= (17.625x5.232)/8

= 60.26kNm

Step 6: Check for effective depth:

M = Qbd^2 d2

= M/Qb

= 60.26/2.76x1

= 149.39mm say 150mm.

For design consideration adopt d = 150mm.

Step 7: Area of Steel:

Mu = 0.87 fy Ast d (1- (fy ast)/(fck b d)) 60.26x106

= 087x415xAstx150(1-(415 Ast)/(20x1000x150))

Ast = 300mm²

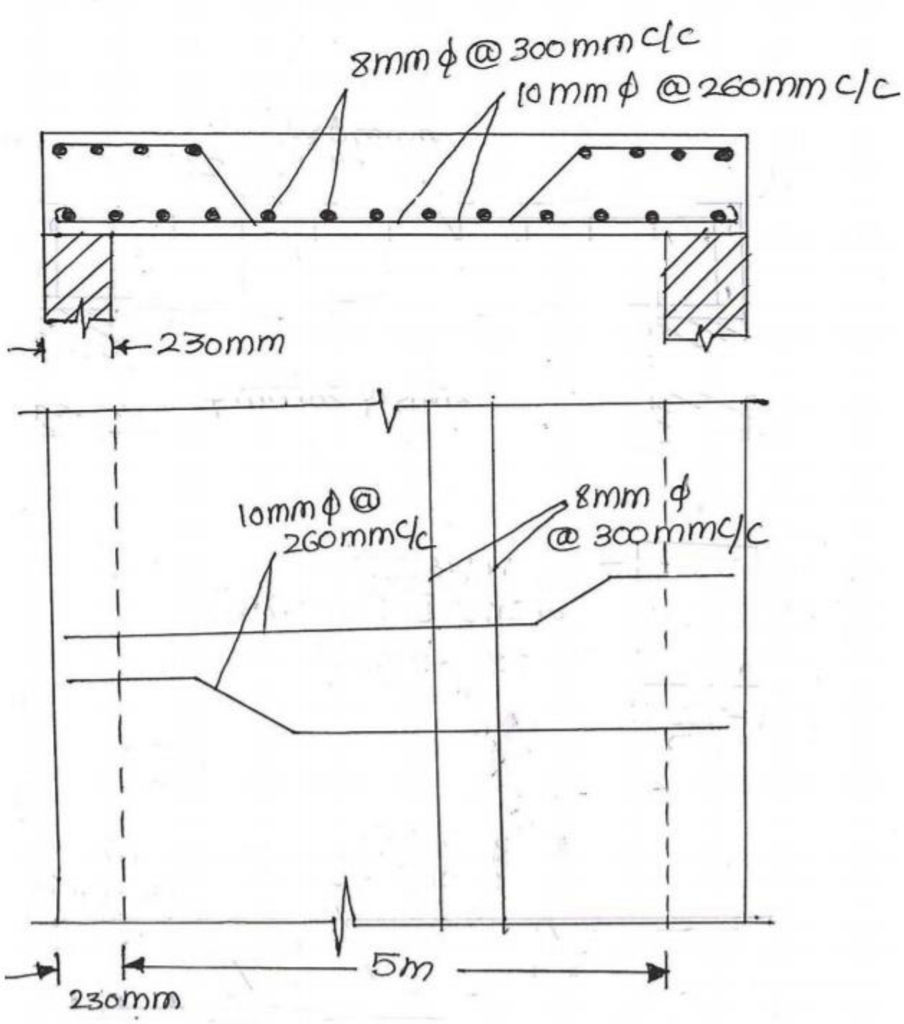
Use 10mm dia bars

Spacing ,S = ast/Astx1000

= (78.53/300)1000

= 261mm

say 260mmc/c Provide 10mm dia @260mm c/c.



8. Design a rectangular beam of cross section 230 x 600 mm and of effective span 6m.imposed load on the beam is 40 kN/m. Use M20 concrete and Fe415 steel. [M/J-16], [N/D-12]

Step 1: Size of the beam:

b = 230mm &

D = 600mm

d = 600-25-20/2

=565mm

Step 2: load calculation:

Live load = 40kN/m²

Dead load = 1x.23x.565x25

= 3.245kN/m^2

Total load = 43.24kN/m^2

Factored load = 43.24 x 1.5

= 64.86kN/m^2

Step 3: Moment Calculation:

M = wl2/8

= (64.86x62)/8

= 291.9kNm

Step 4: Check for effective depth:

Mbal = Qbd2

= 2.97x230x5652

= 218kNm < M

Hence it can be designed as Doubly reinforced beam section.

Step 5: Area of Steel:

Ast = Ast1 + Ast2

Mu = 0.87 fy Ast d (1- (fy ast)/(fck b d)) 218x106

= 087x415xAstx565 (1-(415 Ast)/ (20x230x565))

Ast = 1365mm^2

Use 20mm dia bars,

ast = π/4 (202) = 314.15mm^2

No. of bars = Ast/ast

= 1365/314.15 = 4.47

say 5nos.

Ast2 = (M-Mbal)/(0.87fy(d-d1))

= (291x106-218x106)/(361x(565-20))

=371.03mm^2

Use 20mm dia bars,

ast = π/4 (202)

= $314.15 mm^2$

No. of bars = Ast/ast

= 371.03/314.15

= 1.8

say 2nos.

Step 6: Area of Compression steel:

Asc = (M-Mbal) / (fsc.(d-d1))

= (291x106-218x106) / (351.8x(470-30))

= $1580.65 mm^2$

Use 20mm dia bars,

ast = $\pi/4\ (20^2)$

= $314.15 mm^2$

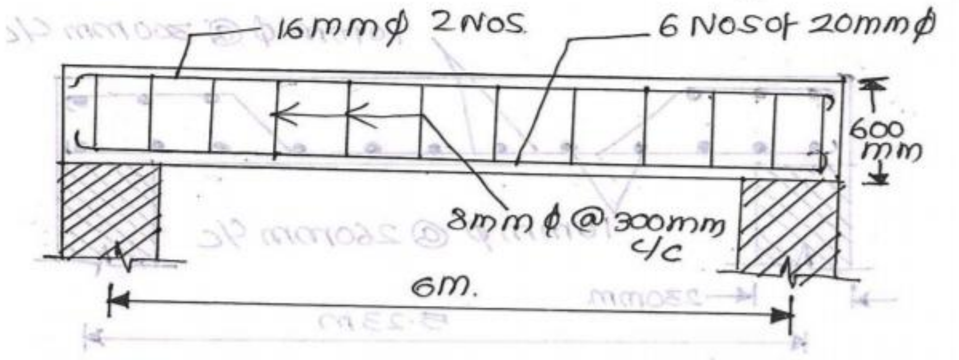
No. of bars = Ast/ast

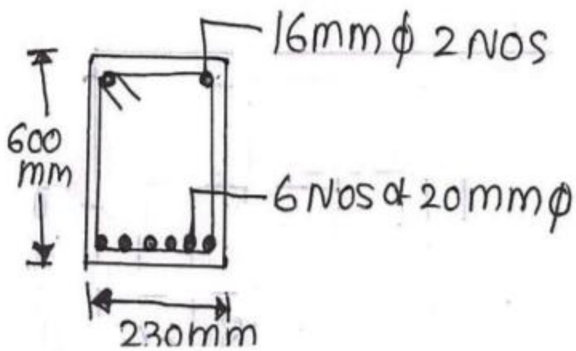
= 1580.65/314.15

= 5.5

say 6nos.

Provide 6#20mm dia bars as compression reinforcement.


16mm ϕ 2 NOS
6 NOS of 20mm ϕ
600 mm
8mm ϕ @ 300mm c/c
6m.


16mm ϕ 2 NOS
600 mm
6 NOS of 20mm ϕ
230mm

UNIT III

Part - A (2 marks)

1. What is the formula used to find the spacing of inclined stirrups? [N/D-15]

a) For vertical stirrups:

$$V_{us} = \frac{0.87 f_y A_{sv} d}{s_v}$$

b) For inclined stirrups or a series of bars bent-up at different cross-sections:

$$V_{us} = \frac{0.8.7 f_y A_{sv} d}{s_v} (\sin\alpha + \cos\alpha)$$

c) For single bar or single group of parallel bars, all bent-up at the same cross-section:

2. What are the functions of longitudinal reinforcement with respect torsion? [N/D-15]

- To assist concrete, in resting compression, so as to reduce the overall size, of the column.
- To resist any tension that might develop due to bending caused by transverse load, eccentric load or the moments.
- To reduce the effect of creep and shrinkage due to sustained loading.
- To prevent or delay sudden brittle collapse.
- To impart necessary ductility to the column.
- To hold the transverse reinforcement.

3. What is the importance of anchorage value of bends. [M/J-16]

The bond between steel and concrete is very important and essential so that they can act together without any slip in a loaded structure. With the perfect bond between them, the plane section of a beam remains plane even after bending.

4. Define shear friction. [M/J-16]

The shear friction analogy is a design. It is a valuable and simple tool which can be used to estimate the maximum shear force transmitted across a cracked plane in a reinforced concrete member.

5. What is the important mechanism of shear resistance in beams with web reinforcement? [N/D-16]

In reinforced concrete building construction, stirrups are most commonly used as shear reinforcement, for their simplicity in fabrication and installation. Stirrups are spaced closely at the high shear region. Congestion near the support of the reinforced concrete beams due to the presence of the closely spaced stirrups increase the cost and time required for installation.

6. Define flexural bond and anchorage bond. [N/D-16]

Flexure bond: It arises in flexural members on account of shear or a variation in bending moment, which in turn
causes a variation in axial tension along length of reinforcing bar.

Development bond: It arises over the length of anchorage provided for a bar or near the end of reinforcing bar. This bond resists the pulling out of bar if it is in tension or conversely, the pushing in of the bar if it is in compression.

7. Write down the effect of torsion in RC beams? [M/J-13]

Generally beams are provided with main reinforcement on the tension side for flexure and transverse reinforcement for shear and torsion. On several situations beams and slabs are subjected to torsion in addition to bending moment and shear force. Loads acting normal to the plane of bending will cause bending moment and shear force. However, loads away from the plane of bending will induce torsional moment along with bending moment and shear.

8. Write about local bond and anchorage length? [M/J-13]

Local bond length is provided for overlapping two rebars in order to safely transfer the load from one bar to another bar.

Anchorage Length is provided to transfer the load from steel to concrete. Development Length is also known as anchorage length. Development length is the length of the bar required to transfer stress from steel to concrete.

7. Write about anchorage bars in tension. [M/J-12]

Deformed bars may be used without end anchorages provided the development length required is satisfied. Hook should normally be provided for plain bars in tension. The anchorage value of a band shall be taken as 4times the diameter of the bar for each 450 bend subjected to a maximum of 16 times the diameter of the bar. The anchorage value of a standard U-type hook shall be equal to 16 times the diameter of the bar.

8. What are the various types of shear reinforcement? [M/J-12]

   a) Vertical stirrups
   b) Bent up bars with stirrups
   c) Inclined stirrups

9. What are the types of shear failure in reinforced concrete beam? [N/D-13]

   a) Shear tension
   b) Flexure shear
   c) Shear compression
   d) Shear bond

10. Define bond stress. [N/D-12], [M/J-13]

    The tangential or shear stress developed along the contact surface of the reinforcing bar and the surrounding concrete if generally termed as bond stress and is expressed in terms of the tangential force per unit nominal surface of the reinforcing bar.

Part - B (16 marks)

1. Design a shear of rectangular reinforced concrete beam section to carry a factored bending Moment of 220 kNm, factored shear force of 120 kN, and a factored torsional moment of 75 kNm. Use M20 grade concrete and Fe415 steel. (16) [N/D-15], [N/D-16], [N/D-12],

Given:

$b = 350mm$

$D = 750mm$

$T_u = 140 KNm$

$M_u = 200 KNm$

$V_u = 110 KN$

$f_{ck} = 25 N/mm^2$

$f_y = 415 N/mm^2$

Take $d' = 50mm$; $b' = 25mm$

$d = 750 - 50$

$d = 700mm$

$d_1 = 750 - 50 - 50 = 650mm$

$b_1 = 350 - 25 - 25 = 300mm$.

To find:

Reinforcements

Solution:

Equivalent B.M:

$M_{e1} = M_u + M_t$ (Pg - 75)

$$= (200 \times 10^6) + T_u \left[\frac{1 + (D/b)}{1.7} \right]$$

$$= (200 \times 10^6) + (140 \times 10^6) \left[\frac{1 + 750/350}{1.7} \right]$$

$M_{e1} = 458.82\,KNm$

Longitudinal Reinforcement : ⇒ Under-reinforced

$M_u = 0.87 f_y A_{st} d \left[1 - \frac{A_{st} f_y}{f_{ck} b d}\right]$ (pg-96)

$458.82 \times 10^6 = 0.87 \times 415 \times A_{st} \times 700 \left[1 - \frac{415 A_{st}}{25 \times 350 \times 700}\right]$

$458.82 \times 10^6 = 252.75 \times 10^3 A_{st} - 17.125 A_{st}^2$.

$A_{st} = 2119.75\,mm^2$

Assume $\phi = 25\,mm$

No. of bars $= \frac{2119.75}{\frac{\pi}{4} \times 25^2} = 4.318$

$\simeq 5$ Nos

Area of 5 Nos $= 5 \times \frac{\pi}{4} \times 25^2 = 2454.36\,mm^2$

Transverse reinforcement :

$A_{sv} = \frac{T_u S_v}{b_1 d_1 (0.87 f_y)} + \frac{V_u S_v}{2.5 d_1 (0.87 f_y)}$; $\frac{(\tau_{ve} - \tau_c) b \cdot S_v}{0.87 f_y}$

A_{sv} = provide 2 legged of 8mm ϕ stirrup

$A_{sv} = 2 \times \frac{\pi}{4} \times 8^2 = 100.53\,mm^2$

$100.53 = \frac{140 \times 10^6 \times S_v}{300 \times 650 \times 0.87 \times 415} + \frac{110 \times 10^3 \times S_v}{2.5 \times 650 \times 0.87 \times 415}$; $\frac{(\tau_{ve} - \tau_c) b \cdot S_v}{0.87 f_y} = A_{sv}$

$100.53 = 1.988 S_v + 0.187 S_v$

$100.53 = 2.175 S_v$

$S_v = 46.22\,mm$

$S_v \simeq 50\,mm$

From (Tab - 19)

$$\tau_c = 0.64 \, N/mm^2$$

$$V_e = V_u + 1.6 \frac{T_u}{b}$$

$$= (110 \times 10^3) + 1.6 \times \frac{140 \times 10^6}{350}$$

$$V_e = 750 \, kN$$

$$\tau_{ve} = \frac{V_e}{bd} = \frac{750 \times 10^3}{350 \times 700} = 3.06 \, N/mm^2$$

$$\tau_{ve} = 3.06 \, N/mm^2$$

$$A_{sv} = \frac{(\tau_{ve} - \tau_c) b \cdot s_v}{0.87 f_y}$$

$$100.53 = \frac{(3.06 - 0.64) \times 350 \times s_v}{0.87 \times 415}$$

$$100.53 = 2.34 s_v$$

$$s_v = 42.96 mm$$

Total transverse reinforcement shall ~~not~~ be lesser ~~than~~ of 42.96mm & 50mm

Hence provide 2 legged ~~of~~ 8mm ϕ stirrups @ a spacing of 40mm c/c.

2. A Simple supported RC beam size 300x500mm effective is reinforced with 4 bars of 16 mm diameter HYSD steel of grade Fe415. Determine the anchorage length of the bars at the simply supported end if it is subjected to a factored force of 350 kN at the centre of 300 mm wide masonry support. The concrete mix of grade M20 is to be used. Draw the reinforcement details.(16) [N/D-15], [N/D-12], [M/J-13]

Given:

$b = 250 mm$

$D = 500 mm$

$M_u = 40 KNm$

$V_u = 40 KN$

$T_u = 30 KNm$

$d' = 50mm$; $b' = 25 mm$

$\therefore d = 500 - 50$

$d = 450mm$

$d_1 = 500 - 50 - 50 = 400mm$

$b_1 = 250 - 25 - 25 = 200mm$.

To find:

Reinforcements.

Solution:

Equivalent B.M:

$M_{e1} = M_u + M_t$

$= (40 \times 10^6) + T_u \left[\frac{1 + (D/b)}{1.7} \right]$

$= (40 \times 10^6) + (30 \times 10^6) \left[\frac{1 + 500/250}{1.7} \right]$

$M_{e1} = 92.94 KNm$

Longitudinal reinforcement ⇒ Under-reinforced:

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

$$92.94\times10^6 = 0.87\times415\times A_{st}\times450\times\left[1-\frac{415\ A_{st}}{20\times250\times450}\right]$$

$$92.94\times10^6 = 162.47\times10^3 A_{st} - 29.96 A_{st}^2$$

$$\boxed{A_{st} = 649.94\ mm^2}$$

Provide $\phi = 16mm$

$$\text{No. of bars} = \frac{649.94}{\frac{\pi}{4}\times16^2} = 3.23 \simeq 4\text{ Nos}$$

$$\text{Area of 4 Nos} = 4\times\frac{\pi}{4}\times16^2 = \underline{804.24\ mm^2}$$

Transverse reinforcement:

$$A_{sv} = \frac{T_u S_v}{b_1 d_1 (0.87 f_y)} + \frac{V_u S_v}{2.5 d_1 (0.87 f_y)} ; \frac{(\tau_{ve}-\tau_c)\cdot b\cdot S_v}{0.87 f_y}$$

$$\frac{\pi}{4}\times8^2\times2 = \frac{30\times10^6\times S_v}{200\times400\times0.87\times415} + \frac{40\times10^3\times S_v}{2.5\times400\times0.87\times415}$$

$$100.53 = 1.038 S_v + 0.110 S_v$$

$$100.53 = 1.148 S_v$$

$$S_v = 87.56mm$$

$$\boxed{S_v \simeq 90mm}$$

$$A_{sv} = \frac{(\tau_{ve}-\tau_c)\cdot b\cdot S_v}{0.87 f_y}$$

$$\tau_{ve} = \frac{V_e}{bd} = \frac{V_u + 1.6\frac{T_u}{b}}{bd} = \frac{40\times10^3 + 1.6\times\frac{30\times10^6}{250}}{250\times450}$$

$$\boxed{\tau_{ve} = 2.06\ N/mm^2}$$

$$p = \frac{100 A_{st}}{bd} = \frac{100\times804.24}{250\times450} = 0.714\%$$
$$\therefore \boxed{\tau_c = 0.548\ N/mm^2}$$

$$100.53 = \frac{(2.06-0.548)\times250\times S_v}{0.87\times415}$$

$S_v = 96$ mm.

$S_v \simeq 100mm$

Take the lesser value of 90mm, Hence provide 2 legged of 8mm ϕ stirrup @ 90mm c/c.

3. Explain the terms Diagonal tension and bond stress with reference to R.C beams. (16) [M/J-16], [N/D-13]

Beams are designed on the basis of limit state of collapse in flexure and checked for other limit states of shear, torsion and serviceability. To ensure safety the resistance to bending, shear, torsion and axial loads at every section should be greater than the appropriate values at that produced by the probable most unfavourable combination of loads on the structure using the appropriate safety factors.

The following general specifications and practical requirements are necessary for designing the reinforced cement concrete beams.

a. Selection of grade of concrete Apart from strength and deflection, durability shall also be considered to select the grade of concrete to be used. Table 5 of IS 456:2000 shall be referred for the grade of concrete to be used. In this table the grade of concrete to be used is recommended based on the different environmental exposure conditions.
b. Selection of grade of steel Normally Fe 250, Fe 415 and Fe 500 are used. In earthquake zones and other places where there are possibilities of vibration, impact, blast etc, Fe 250 (mild steel) is preferred as it is more ductile.
c. Size of the beam, The size of the beam shall be fixed based on the architectural requirements, placing of reinforcement, economy of the formwork, deflection, design moments and shear.
d. In addition, the depth of the beam depends on the clear height below the beam and the width depends on the thickness of the wall to be constructed below the beam. The width of the beam is usually equal to the width of the wall so that there is no projection or offset at the common surface of contact between the beam and the wall. The commonly used widths of the beam are 115 mm, 150 mm, 200 mm, 230 mm, 250 mm, 300 mm.
e. Cover to the reinforcement Cover is the certain thickness of concrete provided all round the steel bars to give adequate protection to steel against fire, corrosion and other harmful elements present in the atmosphere. It is measured as distance from the outer concrete surface to the nearest surface of steel.

The amount of cover to be provided depends on the condition of exposure and shall be as given in the Table 16 of IS 456:2000. The cover shall not be less than the diameter of the bar.

f. Spacing of the bars The details of spacing of bars to be provided in beams are given in clause 26.3.2 of IS 456. As per this clause the following shall be considered for spacing of bars.

The horizontal distance between two parallel main bars shall usually be not less than the greatest of the following

1) Diameter of the bar if the diameters are equal
2) The diameter of the larger bar if the diameters are unequal iii. 5mm more than the nominal maximum size of coarse aggregate Greater horizontal spacing than the minimum specified above should be provided wherever possible. However when needle vibrators are used, the horizontal distance between bars of a group may be reduced to two thirds the nominal maximum size of the coarse aggregate, provided that sufficient space is left between groups of bars to enable the vibrator to be immersed. The minimum vertical distance between the bars shall be of the greatest of the following
    - 15 mm
    - Maximum size of aggregate iii. Maximum size of bars Maximum distance between bars in tension in beams: The maximum distance between parallel reinforcement bars shall not be greater than the values given in table 15 of IS 456:2000.

4. A beam of rectangular section is reinforced with 6 nos of 18 mm diameter bars in tension and is supported on an effective span of 5 m, the beam being 300 mm wide and 700 mm deep. The beam carries a uniformly distributed load of 42 kN/m. Design the shear reinforcement considering no bars are bent up for shear. Assume σ_{sv} = 230 N/mm^2, τ_c = 0.30 N/mm^2 and f_y = 415 N/mm^2 (16) [M/J-16], [M/J-14]

Given:

$b = 200mm$

$D = 350\ mm$

$V_u = 80KN$

$M_u = 25KNm$

$T_u = 6KNm$

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

$f_y = 250N/mm^2$

$d' = 50mm$; $b' = 25mm$

$d = 350 - 50 = 300mm$

$\boxed{d = 300mm}$

$d_1 = 350 - 50 - 50 = 250mm$

$b_1 = 200 - 25 - 25 = 150mm$.

To find:

Torsional reinforcement

Solution:

Equivalent B.M:

$M_{e1} = M_u + M_t$

$= (25\times10^6) + T_u \left[\frac{\quad}{1.7} \right]$

$= (25\times10^6) + (6\times10^6)\left[\frac{1+350/200}{1.7}\right]$

$M_{e1} = 34.70\ KNm$

Longitudinal reinforcement → under-reinforced:

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

$34.70\times10^6 = 0.87\times415\times A_{st}\times300\left[1 - \frac{415 A_{st}}{20\times200\times300}\right]$

$34.70\times10^6 = 108315 A_{st} - 37.45 A_{st}^2$

$A_{st} = 366.90 mm^2$

Provide $\phi = 12mm$

$No.\ of\ bars = \frac{366.90}{\frac{\pi}{4}\times12^2} = 3.24 \simeq 4 Nos.$

$Area\ of\ 4 Nos = 4\times\frac{\pi}{4}\times12^2 = 452.38 mm^2$

Transverse reinforcement:

$A_{sv} = \frac{T_u S_v}{b_1 d_1 (0.87 f_y)} + \frac{V_u S_v}{2.5 d_1 (0.87 f_y)} ; \frac{(\tau_{ve} - \tau_c)\cdot b\cdot S_v}{0.87 f_y}$

$2\times\frac{\pi}{4}\times8^2 = \frac{6\times10^6\times S_v}{150\times250\times0.87\times415} + \frac{80\times10^3\times S_v}{2.5\times250\times0.87\times415}$

$100.53 = 0.443 S_v + 0.354 S_v$

$S_v = 126.13 mm$

$S_v \simeq 130mm$

$A_{sv} = \frac{(\tau_{ve} - \tau_c)\, b\cdot S_v}{0.87 f_y}$

6×10^6

$\tau_{ve} = 2.13 \, N/mm^2$

$$p = \frac{100 A_{st}}{bd} = \frac{100 \times 452.38}{200 \times 300} = 0.753\%$$

From (Tab-19)

$\tau_c = 0.56 \, N/mm^2$

$$A_{sv} = \frac{(\tau_{ve} - \tau_c) \, b \cdot S_v}{0.87 f_y}$$

$$100.53 = \frac{(2.13 - 0.56) \times 200 \times S_v}{0.87 \times 415}$$

$100.53 = 0.869 S_v$

$S_v = 115.68 \, mm$

$S_v \simeq 120 \, mm$

Take $S_v = 120 \, mm$

Hence provide 2 legged of 8mm ϕ stirrup @ 120mm spacing c/c.

6. A Simple supported RC beam size 300x500mm effective is reinforced with 4 bars of 16 mm diameter HYSD steel of grade Fe415. Determine the anchorage length of the bars at the simply supported end if it is subjected to a factored force of 350 kN at the centre of 300 mm wide masonry support. The concrete mix of grade M20 is to be used. Draw the reinforcement details. (16) [N/D-16], [M/J-14]

Given:

$b = 200mm$

$D = 350 mm$

$V_u = 80 KN$

$M_u = 25 KNm$

$T_u = 6 KNm$

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

$f_y = 250 N/mm^2$

$d' = 50 mm$; $b' = 25 mm$

$d = 350 - 50 = 300mm$

$d = 300 mm$

$d_1 = 350 - 50 - 50 = 250 mm$

$b_1 = 200 - 25 - 25 = 150 mm$.

To find:

Torsional reinforcement

Solution:

Equivalent B.M:

$M_{e1} = M_u + M_t$

$= (25\times10^6) + T_u \left[\frac{\quad}{1.7}\right]$

$= (25\times10^6) + (6\times10^6)\left[\frac{1+350/200}{1.7}\right]$

$M_{e1} = 34.70\ kNm$

Longitudinal reinforcement → under-reinforced:

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

$34.70\times10^6 = 0.87\times415\times A_{st}\times300\left[1-\frac{415 A_{st}}{20\times200\times300}\right]$

$34.70\times10^6 = 108315 A_{st} - 37.45 A_{st}^2$

$A_{st} = 366.90 mm^2$

Provide $\phi = 12mm$

$No.\ of\ bars = \frac{366.90}{\frac{\pi}{4}\times12^2} = 3.24 \simeq 4 Nos.$

$Area\ of\ 4 Nos = 4\times\frac{\pi}{4}\times12^2 = 452.38\ mm^2$

Transverse reinforcement:

$A_{sv} = \frac{T_u S_v}{b_1 d_1 (0.87 f_y)} + \frac{V_u S_v}{2.5 d_1 (0.87 f_y)} ; \frac{(\tau_{ve}-\tau_c)\cdot b\cdot S_v}{0.87 f_y}$

$2\times\frac{\pi}{4}\times8^2 = \frac{6\times10^6\times S_v}{150\times250\times0.87\times415} + \frac{80\times10^3\times S_v}{2.5\times250\times0.87\times415}$

$100.53 = 0.443 S_v + 0.354 S_v$

$S_v = 126.13 mm$

$S_v \simeq 130mm$

$A_{sv} = \frac{(\tau_{ve}-\tau_c)\, b\cdot S_v}{0.87 f_y}$

$\tau_{ve} = \frac{V_e}{bd} = \frac{V_u + 1.6\frac{T_u}{b}}{bd} = \frac{80\times10^3 + 1.6\times\frac{6\times10^6}{200}}{200\times300}$

$\tau_{ve} = 2.13\ N/mm^2$

$p = \frac{100\ A_{st}}{bd} = \frac{100 \times 452.38}{200 \times 300} = 0.753\%$.

From (Tab - 19)

$\tau_c = 0.56\ N/mm^2$

$A_{sv} = \frac{(\tau_{ve} - \tau_c)\ b \cdot S_v}{0.87\ f_y}$

$100.53 = \frac{(2.13 - 0.56) \times 200 \times S_v}{0.87 \times 415}$

$100.53 = 0.869\ S_v$

$S_v = 115.68\ mm$

$S_v \simeq 120\ mm$

Take $S_v = 120\ mm$

Hence provide 2 legged of 8mm ϕ stirrup @ 120mm spacing c/c.

7. Describe the procedure for design of shear reinforcement. [M/J-12], [M/J- 11], [N/D-13]

The nominal shear stress value is determined using the formula

shear force / (bxd) in KN/mm².

The design shear strength of concrete is determined by using the tables in IS456-2000.

Compare the nominal shear strength and design shear strength, the nominal shear stress value is less then design shear strength the minimum shear reinforcement is provided in the form of vertical stirrups, in the case IS456-2000-26.5.1.6 is referred.

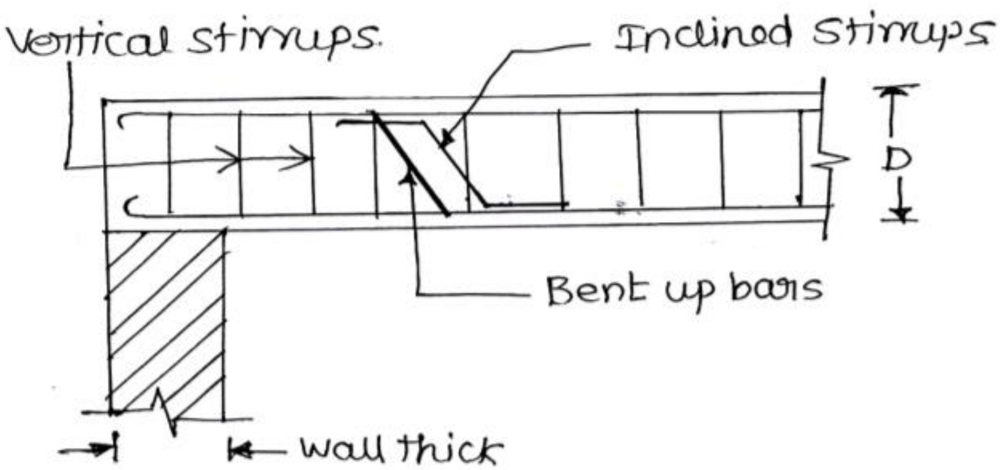
The nominal shear stress value is exceeds the design shear strength, shear reinforcement shall be provided in any of the following forms.

a) Vertical stirrups

b) Bent up bars along with stirrups,

c) Inclined stirrups.

Where bent up bars are provided their contribution towards shear resistance shall not be more than half that of the total shear reinforcement. Shear reinforcement shall be provided to carry a shear equal to the strength of shear reinforcement.



UNIT IV

Part - A (2 marks)

1. What is meant by braced column? [N/D-15]

A column may be considered braced in a given plane if lateral stability to the structure as a whole is provided by walls or bracing or buttressing designed to resist all lateral forces in that plane.

2. How the compression failures occur in columns? [N/D-15]

The following assumptions are made for column failing under pure compression:

i. The maximum compressive strain in concrete in axial compression is 0.002
ii. Plane sections remain plane in compression
iii. The design stress strain curve for steel in compression is taken to be the same as in tension

3. Write any two salient assumptions are made in the limit state design of columns. [M/J-16]

The following assumptions are made for column failing under pure compression:

i. The maximum compressive strain in concrete in axial compression is 0.002
ii. Plane sections remain plane in compression
iii. The design stress strain curve for steel in compression is taken to be the same as in tension

4. What are the important limitations of slender columns? [M/J-16]

A short concrete column is one having a ratio of unsupported length to least dimension of the cross section equal to or less than 10. If the ratio is greater than 10, it is considered a long column (sometimes referred to as a slender column).

5. Write any two reinforcement provision in columns. [N/D-16]

As per IS 456-2000 a reinforced concrete column shall have longitudinal steel reinforcement and cross sectional area of such reinforcement shall not be less than 0.8% nor more than 6% of cross sectional area of column required to transmit all the loading.

The effective length of a column depends upon unsupported length and boundary conditions at end of columns. The effective length L_{ef} can be expressed in the form:

$L_{ef} = kL$

where L = Unsupported length or clear height of column

k = Effective length ratio

6. What is the salient condition for minimum eccentricity of column? [N/D-16]

All axially loaded columns should be designed considering the minimum eccentricity

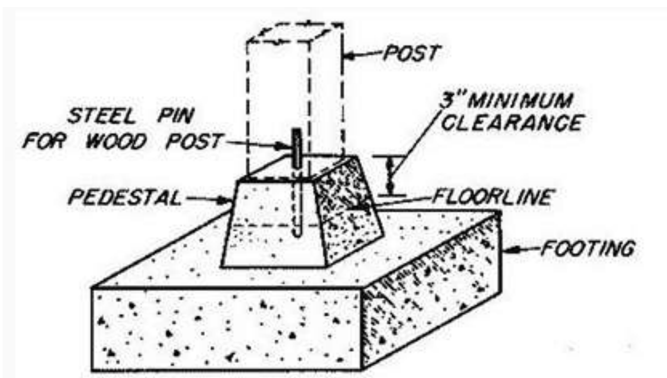
ex min ≥ greater of)l/500 + D/30) or 20 mm

ey min ≥ greater of)l/500 + b/30) or 20 mm

where l, D and b are the unsupported length, larger lateral dimension and least lateral dimension, respectively.

7. What is pedestal? [M/J-13]

A concrete pedestal is a compression element provided to carry the loads from supported elements like columns, statues etc. to footing below the ground. It is generally provided below the metal columns.



8. Write down the expression for minimum eccentricity [M/J-13]

Consider a short column subjected to an eccentric load P with an eccentricity e form the axis.

Maximum stress = Direct Stress + Bending stress

$$f_c = \frac{P}{A} + \frac{M}{Z} \qquad Z = \frac{I}{y}$$

$$= \frac{P}{A} + \frac{p.e.y_c}{Ak^2} \qquad I = Ak^2$$

$$k = \sqrt{\frac{I}{A}}$$

where

A	=	Sectional are of the column
Z	=	Sectional modulus of the column
y_c	=	Distance of extreme fibre from N.A
k	=	Least radius of gyration.

10. Write about percentage of reinforcement for columns [M/J-12] [N/D-13]

As per is 456 a reinforced concrete column shall have longitudinal steel reinforcement and the cross-sectional area of such reinforcement shall be not be less than 0.8% nor more than 6% of the cross-sectional area of the column required to transmit all the loading.

11. What is the loading the condition for short column? [M/J-13]

Short axially loaded members in axial compression

Short axially loaded column with minimum eccentricity

12. What are the modes of failure of a column? [N/D-16], [M/J-13]

- Compression failure
- Tension failure

Part - B (16 marks)

1. Design a column having an effective length of 4.75 m to support factored load of 1600kN. Consider the reinforcement ratio ρ to be in the range 1.5 to 2.0 percent and the effective cover to longitudinal steel of 55mm. The materials to be used are M25 grade of concrete and HYSD steel bars of grade Fe415. (16) [N/D-15]

Let us assume 1.0% steel (1 to 2%)

Say ASC = 1.0%

Ag =1/100

Ag = 0.01Ag

fck = 20 MPa,

fy= 415 MPa,

P = 980 kN

1. Area of concrete

Ac = Ag – Asc

= Ag -0.01Ag = 0.99 Ag

2. Ultimate load carried by the column

Pu = 0.4 fck Ac + 0.67 fy Asc 980 x 1.5 x 1000

= 0.4x20x 0.99 Ag + 0.67x415 x 0.01Ag

= 7.92 Ag + 2.78 Ag =10.7Ag

Ag = 137383 mm2

Let us design a square column:

B = D = √ Ag

=370.6 mm

say 375 x 375 mm

This is ok.

However this size cannot take the minimum eccentricity of 20 mm as

emin/D = 20/375

=0.053 > 0.05.

To restrict the eccentricity to 20 mm,

The required size is 400x 400 mm.

Area of steel required is

Ag = 1373.8 mm2 .

Provide 4 bar of 22 mm diameter.

Steel provided is 380 x 4 = 1520 mm2

Actual percentage of steel

= 100Asc/ bD

= 100x1520 /400x400

= 0.95 % which is more than 0.8% and less than 6% and therefore ok.

3. Design of Transverse steel:

Diameter of tie = ¼ diameter of main steel

= 22/4

=5.5mm or 6 mm,

whichever is greater.

Provide 6 mm.

Spacing: < 300 mm, < 16 x22 = 352mm, < LLD = 400mm.

Say 300mm c/c

4. Design of circular column:

Here Ag = 137383 mm2 π x D2 /4

D= 418.2 mm say 420 mm.

This satisfy the minimum eccentricity of 20m

Also provide 7 bars of 16 mm,

7 x 201 = 1407 mm2

5. Design of Transverse steel:

Dia of tie = ¼ dia of main steel

= 16/4 = 4 mm

or 6 mm,

whichever is greater.

Provide 6 mm.

Spacing: < 300 mm, < 16 x16 = 256 mm, < LLD = 420mm.

Say 250 mm c/c.

2. A braced reinforced concrete column of circular cross-section of 500mm diameter is to support a factored axial load of 2250 kN along with a factored moment of 160 kNm. The unsupported length of the column is 6.3m effective length of 5.5m. Design the column when it is to be provided with: Lateral ties and Spiral reinforcement. The M25 grade of concrete and HYSD steel bars of grade Fe415. (16)
[N/D-15], [M/J-12]

Given:

fck = 20 MPa,

fy= 415 MPa,

Pu = 2500kN

Let us assume 1.0% steel (1 to 2%)

Say ASC = 1.0%

Ag =1/100 Ag

= 0.01Ag

1. Area of concrete

Ac = Ag – Asc

= Ag -0.01Ag

= 0.99 Ag

2. Ultimate load carried by the column

Pu = 0.4 fck Ac + 0.67 fy Asc 2500 x 1000

= 0.4x20x 0.99 Ag + 0.67x415 x 0.01Ag

= 7.92 Ag + 2.78 Ag

=10.7Ag

Ag = 233645 mm2

π x D2 /4 = Ag,

D = 545.4 mm

say 550 mm.

Area of steel = 2336 mm2 ,

Also provide 8 bars of 20 mm,

= 6 x 314

= 2512 mm2

3. Check for shortness:

Ends are hinged

lex = ley = l = 3000 mm

lex /D= 3000/550 < 12,

ley /b = 3000/425 < 12,

Column is short

4. Check for minimum eccentricity:

Here, emin,

x = emin,

y = lux/500 + D/30 = 3000/500 + 550/30

= 24.22mm or 20mm

whichever is greater.

emin = 24.22 mm < 0.05D = 0.05 x 550 =27.5 mm.

5. Design of Transverse steel:

Diameter of tie = ¼ dia of main steel

= 20/4 = 5 mm or 6 mm,

whichever is greater.

Provide 6 mm or 8 mm.

Spacing: < 300 mm, < 16 x20 = 320 mm, < LLD = 550mm.

Say 300 mm c/c

Similarly square column can be designed.

If the size of the column provided is less than that provided above, then the minimum eccentricity criteria are not satisfied. Then emin is more and the column is to be designed as uni axial bending case or bi axial bending case as the case may be. This situation arises when more steel is provided (say 2% in this case)

3. Design the reinforcements in a circular column of diameter 300 mm to support a service axial load of 800 kN. The column has unsupported length of 3 m and is braced against side sway. The column is reinforced with helical ties. The material to be used is M 25 grade of concrete and HYSD steel bars of grade Fe 415. (16) [M/J-16], [M/J-12], [N/D-13]

Given

Diameter of column = 500 mm

Grade of concrete M20

Characteristic strength 250 N/mm2

Factored load 1600 kN

Factored moment 125 kN.m

1. Lateral reinforcement :

(a) Hoop reinforcement

(b) Helical reinforcement (Assume moment due to minimum eccentricity to be less than the actual moment). Assuming 25 mm bars with 40 mm cover,

d1 = 40 + 12.5 = 52.5 mm

d1 /D – 52.5/50 = 0.105

Charts for d'/D = 0.10 will be used.

(a) Column with hoop reinforcement Pu/fck D

D = (1600 x 1000)/ (20 x 500 x 500)

= 0.32 Mu/fck D x D2 =125 x 106 /(20 x 500 x 5002)

= 0.05 Referring to Chart 52,

for fy = 250 N/mm2

p/fck = 0.87

Percentage of reinforcement, p = 0.87 x 20

= 1.74 % As

= 1.74 x (π x 5002 /4)/100

= 3416 mm2

(b) Column with Helical Reinforcement

According to 38.4 of the Code, the strength of a compression member with helical reinforcement is 1.05 times the strength of a similar member with lateral ties.

Therefore, the, given load and moment should be divided by 1.05 before referring to the chart.

Pu/fck D D = (1600/1.05 x 1000)/ (20 x 500 x 500)

= 0.31 Mu/fck D x D2

=125/1.05 x 106 /(20 x 500 x 5002)

= 0.048

Hence, From Chart 52, for fy = 250 N/mm2 ,

p/fck = 0.078 p

= 0.078 x 20

= 1.56 % As

= 1.56 x(π x 500 x 500/4)/100

= 3063 cm2

According to 38.4.1 of the Code the ratio of the volume of helical reinforcement to the volume of the core shall not be less than 0.36 (Ag/Ac - 1) x fck /fy

where Ag is the gross area of the section

Ac is the area of the core measured to the outside diameter of the helix.

Assuming 8 mm dia bars for the helix,

Core diameter = 500 - 2 (40 - 8) = 436 mm

Ag/AC = 500/436

Ag/Ac - 1) x fck /fy

= 0.36(0.315) 20/250

=0.0091

Volume of helical reinforcement / Volume of core

= Ash π x 428 /(π/4 x 4362) sh 0.09 Ash / sh

where,

Ash is the area of the bar forming the helix

sh is the pitch of the helix.

In order to satisfy the coda requirement,

0.09 Ash / sh ≥ 0.0091

For 8 mm dia bar, sh ≤ 0.09 x 50 / 0.0091

= 49.7 mm.

Thus provide 48 mm pitch.

4. Design the reinforcements in as short column 400mmx400mm at the corner of a multistoreyed building to support an axial factored load of 1500 kN, together with biaxial moments of 50 kNm acting in

perpendicular planes. Adopt M20 grade of concrete and steel grade Fe415 HYSD bars. (16)
[M/J-16], [N/D-13]

Reinforcement is distributed equally on four sides.

As a first trial assume the reinforcement percentage,

p = 1.2% p/fck

= 1.2/15 = 0.08

Uniaxial moment capacity of the section about xx-axis :

d1 /D = 52.5 /600 = 0.088

Chart for d'/D = 0.1 will be used.

Pu/fck b D = (1600 x 1000)/ (15 x 400 x 600) = 0.444

Referring to chart 44

Mu/fck b x D2 = 0.09 Mux1

= 0.09 x 15 x 400 x 6002)

= 194.4 kN.m

Uni-axial moment capacity of the section about yy axis :

d1 /D = 52.5 /400 = 0.131 Chart for d1 /D =0.15 will be used.

Referring to Chart 45,

Mu/fck b x D2 = 0.083 Mux1

= 0.083 x 15 x 600 x 4002)

= 119.52 kN.m

Calculation of Puz :

Referring to Chart 63 corresponding to

p = 1.2,

fy = 415 and

fck = 15,

Puz/Ag = 10.3

Puz = 10.3 x 400 x 600

= 2472 kN

Mux/Mux1 = 120/194.4

=0.62 Muy/Muy1

=90/119.52

= 0.75

Pu /Puz =1600/2472

= 0.65

Referring to Chart 64,

The permissible value of Mux/Mux1 corresponding to Muy/Muy1 and Pu /Puz is equal to 0.58

The actual value of 0.62 is only slightly higher than the value read from the Chart.

This can be made up by slight increase in reinforcement.

Using Boris load contour equation as per IS:456-2000

Pu /Puz = 0.65 thus,

αn = 1 + [(2-1) / (0.8 - 0.2)] (0.65-0.2)

= 1.75 [0.62]1.75 + [0.75]1.75

= 1.04

slightly greater than 1 and slightly unsafe.

This can be made up by slight increase in reinforcement say 1.3%

Thus provide As = 1.3x400x600/100

= 3120 mm2

Provide 1.3 % of steel

p/fck = 1.3/15

= 0.086

d1 /D = 52.5 /600

= 0.088

= 0.1

From chart 44

Mu/fck b x D2 = 0.095

Mux1 = 0.095 x 15 x 400 x 6002)

= 205.2 kN.m

Referring to Chart 45,

Mu/fck b x D2 = 0.085

Mux1 = 0.085 x 15 x 600 x 4002)

= 122.4 kN.m

Chart 63 : Puz/Ag = 10.4

Puz = 10.4 x 400 x 600 = 2496 kN

Mux/Mux1 = 120/205.2 =0.585

Muy/Muy1=90/122.4 = 0.735

Pu /Puz =1600/2496 = 0.641

Referring to Chart 64, the permissible value of Mux/Mux1 corresponding to

Muy/Muy1 and Pu /Puz is equal to 0.60

Hence the section is O.K.

Using Boris load contour equation as per IS:456-2000

Pu /Puz = 0.641

Thus, αn = 1 + [(2-1) / (0.8 - 0.2)] (0.641-0.2)

= 1.735 [120/205.2]1.735 + [90/122.4]1.735

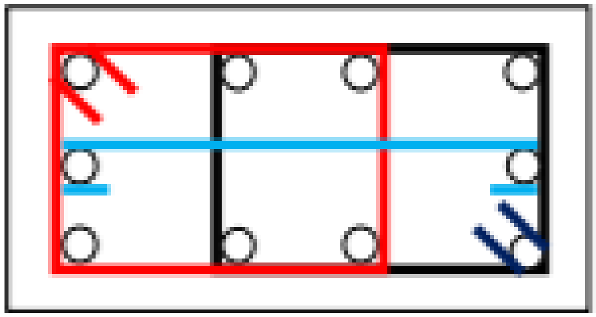
= 0.981 ≤ 1 Thus OK

As = 3120 mm2 .

Provide 10 bars of 20 mm dia. Steel provided is 314 x 10 = 3140 mm2

Design of transverse steel:

Provide 8 mm dia stirrups at 300 mm c/c as shown satisfying the requirements of IS: 456-2000



5. Consider the reinforcement ratio p to be in the range 1.5 to 2.0 percent and the effective cover to longitudinal steel of 55mm. The materials to be used are M25 grade of concrete and HYSD steel bars of grade Fe415. (16)

[M/J-16], [N/D-14]

Solution:

Given:

$D_x = 500$ mm,

$b = 300$ mm,

$A_s = 2946\ mm^2$

$M_{ux} = 125$ kNm,

$M_{uy} = 75$ kNm,

$f_{ck} = 25$ MPa,

$f_y = 415$ MPa

Applied eccentricities $e_x = M_{ux} / P_u$

$= 125 \times 10^3 / 1400$

$= 89.3$ mm

$e_x / D_x = 0.179$

$e_y = M_{uy} / P_u$

$= 75 \times 10^3 / 1400$

$= 53.6$ mm

$e_y / D_y = 0.179$

These eccentricities for the short column are clearly not less than the minimum eccentricities specified by the Code.

Uniaxial moment capacities:

Mux1 , Muy1, corresponding to

P u = 1400 kN,

M ux1 = 187 kNm

M uy1 = 110 kNm

Calculation of Puz :

Referring to Chart 63 corresponding to

p = 1.2,

fy = 415 and

fck = 15,

Puz/Ag = 10.3

Puz = 10.3 x 400 x 600

= 2472 kN

Mux/Mux1 = 120/194.4

=0.62 Muy/Muy1

=90/119.52

= 0.75

Pu /Puz =1600/2472

= 0.65

Referring to Chart 64,

The permissible value of Mux/Mux1 corresponding to Muy/Muy1 and Pu /Puz is equal to 0.58

The actual value of 0.62 is only slightly higher than the value read from the Chart.

This can be made up by slight increase in reinforcement.

Using Boris load contour equation as per IS:456-2000

Pu /Puz = 0.65 thus,

αn = 1 + [(2-1) / (0.8 - 0.2)] (0.65-0.2)

= 1.75 [0.62]1.75 + [0.75]1.75

= 1.04

slightly greater than 1 and slightly unsafe.

This can be made up by slight increase in reinforcement say 1.3%

Thus provide As = 1.3x400x600/100

= 3120 mm2

Provide 1.3 % of steel

p/fck = 1.3/15

= 0.086

d1 /D = 52.5 /600

= 0.088

= 0.1

From chart 44

Mu/fck b x D2 = 0.095

Mux1 = 0.095 x 15 x 400 x 6002)

= 205.2 kN.m

Referring to Chart 45,

Mu/fck b x D2 = 0.085

Mux1 = 0.085 x 15 x 600 x 4002)

= 122.4 kN.m

Chart 63 : Puz/Ag = 10.4

Puz = 10.4 x 400 x 600 = 2496 kN

Mux/Mux1 = 120/205.2 =0.585

Muy/Muy1=90/122.4 = 0.735

Pu /Puz =1600/2496 = 0.641

Referring to Chart 64, the permissible value of Mux/Mux1 corresponding to Muy/Muy1 and Pu /Puz is equal to 0.60

Values of P uz and α n P uz = 0.45fck A g + (0.75f y – 0.45fck)Asc

= (0.45 × 25 × 300 × 500) + (0.75 × 415 – 0.45 × 25)×2946

= (1687500 + 883800)N = 2571 kN ⇒ P u /P uz

= 1400/2571

= 0.545 (which lies between 0.2 and 0.8)

$\alpha_n = 1.575$

Check safety under biaxial bending $[125/187]^{1.575} + [75/110]^{1}$

$= 0.530 + 0.547$

$= 1.077 > 1.0$

Hence, almost ok.

6. A braced reinforced concrete column of circular cross-section of 500mm diameter is to support a factored axial load of 2300 kN along with a factored moment of 165 kNm. The unsupported length of the column is 6.3m effective length of 5.5m. Design the column when it is to be provided with:

(i) Lateral ties and

(ii) Spiral reinforcement. The M25 grade of concrete and HYSD steel bars of grade Fe415.

[M/J-16], [M/J-15]

Given data:-

Unsupported length of column $l_0 = 3m$

For the condition the column is effectively held in position at both ends but not restrained against rotation (Table 8.1) } $L_{ef} = 1.0$ $L_0 = 3.0m$

Diameter of column $D = 400mm$

$f_{ck} = 25 N/mm^2$ $f_y = 415 N/mm^2$

Axial working load $= 1200$ kN.

Factored load $= 1.5 \times 1200$

$= 1800$ KN.

Step ①:- Slenderness ratio

$$L_{ef}/D = \frac{3000}{4000} = 7.5$$

As the slenderness ratio ≤ 12 the column is designed as a short column

Step ②:- Minimum Eccentricity:-

$$e_{min} = \frac{L}{500} + D/30$$

$$= \frac{3000}{500} + \frac{400}{30}$$

$$= 19.33mm < 20mm$$

and $0.05D = 0.05 \times 400 = 20mm$

Hence minimum eccentricity is acceptable.

Step ③:- Main reinforcement:-

we know

$$(P_u)_{helical} = 1.05\ (P_u)_{tie}$$

$$(P_u)_{helical} = 1.05\left[0.4 f_{ck} A_g + (0.67 f_y - 0.4 f_{ck}) \times A_{sc}\right]$$

$$\frac{1800\times10^3}{1.05} = \left[\left[\frac{0.4\times25\times\pi\times400^2}{4}\right] - \left[(0.67\times415) - 0.4\times25\right) A_{sc}\right]$$

$$1714.3\times10^6 = 1256000 + 268.05\ A_{sc}$$

$$A_{sc} = \frac{458000}{268.05} = 1708.6\ mm^2$$

A_{sc} (minimum) = 0.8% of cross section.

$$= \frac{0.008\times\pi\times400^2}{4}$$

$$= 1004.8\ mm^2$$

Hence the calculated Asc (1708.6 mm²) is more than the minimum

Provide 6 bars of 20mm ϕ (Asc = 1884 mm²)

Step (4) :- Helical Reinforcement :-

Adopting a clear cover to 50mm over spirals.

Core diameter = [400 − (2×50)] = 300mm

Assuming 8mm dia bars to be used, the

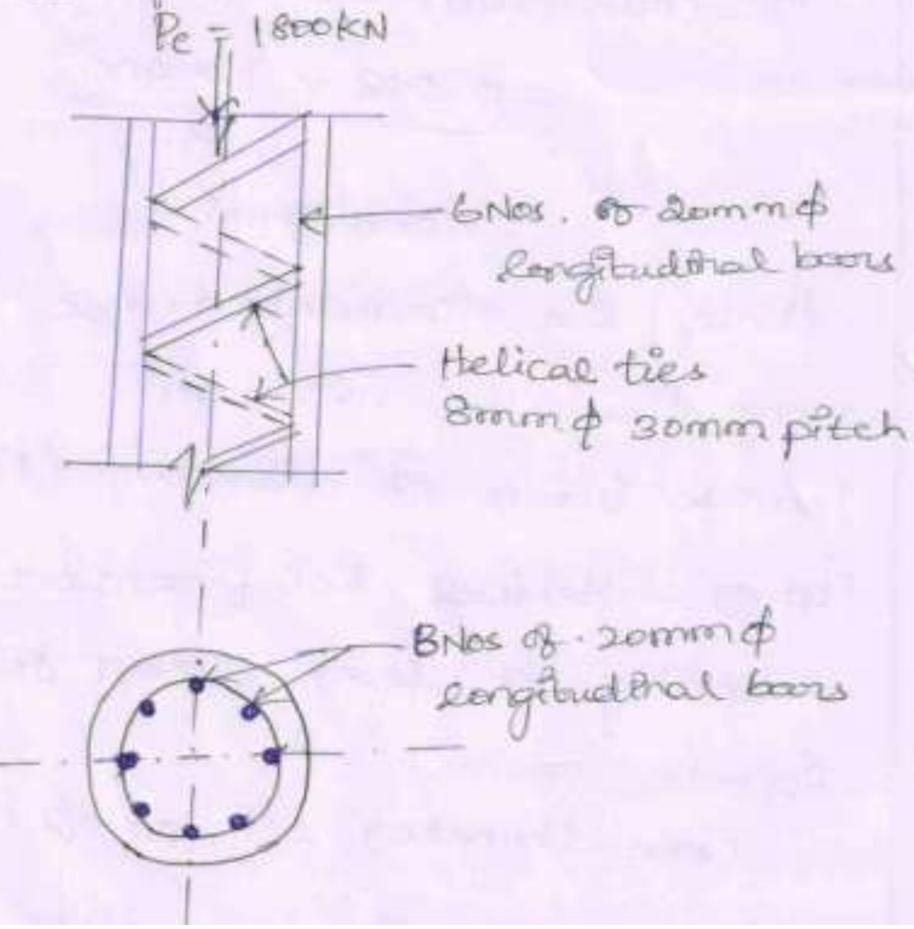
$$S = \frac{11.1 \times a \times D_K f_y}{(D^2 - D_K^2) f_{ck}}$$

$$a = \frac{\pi \times 8^2}{4} = 50.24\ mm^2$$

$$S = \frac{11.1 \times 50.24 \times 300 \times 415}{(400^2 - 300^2) \times 25}$$

$$= 39.8 cm.$$

Hence provide 8mm diameter helical Spiral at a pitch of 30mm.



8. Design the reinforcements in as column 530mmx450mm at the corner of a multistoreyed building to support an axial factored load of 1500 kN, together with biaxial moments of 50 kNm acting in perpendicular planes. Adopt M20 grade of concrete and steel grade Fe415 HYSD bars. (16)

[M/J-12] [N/D-13]

Characteristic strength of steel, $f_y = 500 \text{ N/mm}^2$

Effective length of column, $L_{ex} = L_{ey} = 6.6 \text{ m}$

Unsupported length, $L_0 = 7 \text{ m}$

Factored load, $P_u = 1600 \text{ kN}$

Factored moment about major axis, $M_{ux} = 45$ kN.m at top and 30 kN.m at bottom

Factored moment about minor axis, $M_{uy} = 30$ kN.m at top and 20 kN.m at bottom

(ii) Slenderness Ratio

$$\left.\begin{array}{l} \dfrac{L_{ex}}{D} = \dfrac{6600}{530} = 12.45 \\ \dfrac{L_{ey}}{b} = \dfrac{6600}{450} = 14.67 \end{array}\right\} > 12$$

Hence the column is slender about both axes. From Table 1 of SP-16 (Table 8.2) corresponding slenderness ratios

$$\frac{e_x}{D} = 0.078 \quad \left(\text{for } \frac{L_{ex}}{D} = 12.45\right)$$

$$\frac{e_y}{b} = 0.108 \quad \left(\text{for } \frac{L_{ey}}{b} = 14.67\right)$$

(iii) Additional Moments

$$M_{ax} = P_u e_x = 1600 \times 0.078 \times 0.53 = 66.14 \text{ kN.m}$$

$$M_{ay} = P_u e_y = 1600 \times 0.108 \times 0.45 = 77.6 \text{ kN.m}$$

(iv) Reinforcement

Assume 3.5% steel at the first trial with reinforcement equally distributed on all four faces.

$$A_g = 530 \times 450 = 23.85 \times 10^4 \text{ mm}^2$$

From Chart 63 of SP16 (Fig.8.21) corresponding to the percentage of reinforcement (3.5%) and the characteristic strength of reinforcement ($f_y = 500 \text{ N/mm}^2$) and $f_{ck} = 25 \text{ N/mm}^2$

$$\left(\frac{P_{uz}}{A_g}\right) = 24.0 \text{ N/mm}^2$$

$$\therefore \quad P_{uz} = \frac{24.0 \times 23.85 \times 10^4}{10^3} = 5724 \text{ kN}$$

(v) Computation of P_b

Assuming 25 mm dia bars with $\frac{d'}{D} = \frac{d'}{b} = 0.10$ from Table 60 of SP.16 (Table 8.3) $k_1 = 0.207$, $k_2 = 0.425$

$$P_{bx} = P_{by} = \left[k_1 + k_2 \frac{p}{f_{ck}} \right] f_{ck}\, bD$$

$$= \left(0.207 + 0.425 \times \frac{3.5}{25} \right) 25 \times 450 \times 530 \times 10^{-3}$$

$$= 1589 \text{ kN}$$

(vi) Reduction Factors k_x and k_y

$$k_x = k_y = \left(\frac{P_{uz} - P_u}{P_{uz} - P_{bx}} \right) = \left(\frac{5724 - 1600}{5724 - 1589} \right)$$

$$= 0.997$$

Hence modified additional moments are

$$M_{ux} = 66.14 \times 0.997 = 65.94 \text{ kN.m}$$

$$M_{uy} = 77.6 \times 0.997 = 77.37 \text{ kN.m}$$

Additional moments due to slenderness effects should be added to the initial moments after modifying the initial moment as per Clause 39.7.1 of IS: 456-2000.

$$M_{ux} = (0.6 \times 45 - 0.4 \times 30) = 15 \text{ kN.m}$$

$$M_{uy} = (0.6 \times 35 - 0.4 \times 20) = 13 \text{ kN.m}$$

Above actual moments should be compared with those calculated from minimum eccentricity consideration and the greater value is taken as the initial moment for adding the additional moments.

$$e_x = \left[\frac{L}{500} + \frac{D}{30} \right] = \frac{7700}{500} + \frac{530}{30} = 33.07 \text{ mm}$$

$$e_y = \left[\frac{L}{500} + \frac{b}{30} \right] = \frac{7700}{500} + \frac{450}{30} = 33.40 \text{ mm}$$

Both e_x and e_y are greater than 20 mm

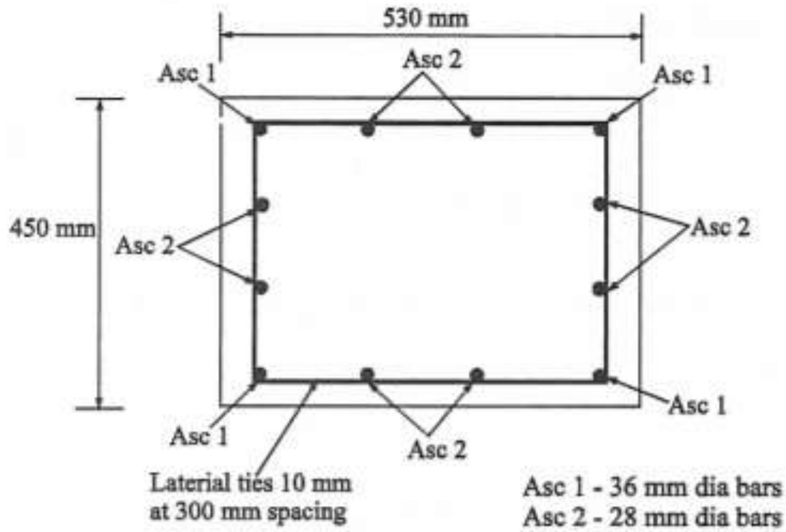
Moments due to minimum eccentricity.

$$M_{ux} = 1600 \times \frac{33.07}{1000} = 52.91 \text{ kN.m}$$

$$M_{uy} = 1600 \times \frac{30.40}{1000} = 48.64 \text{ kN.m}$$

$$A_{sc} = \frac{3.5 \times 450 \times 530}{100} = 8347.5 \text{ mm}^2$$

Provide 4 bars of 36 mm dia at each corners and 8 bars of 28 mm dia with 2 bars on each face.



CE 6505 DESIGN OF REINDORCED CONCRETE ELEMENTS

UNIT V

Part - A (2 marks)

1. What is meant by proportioning of footing? [N/D-15]

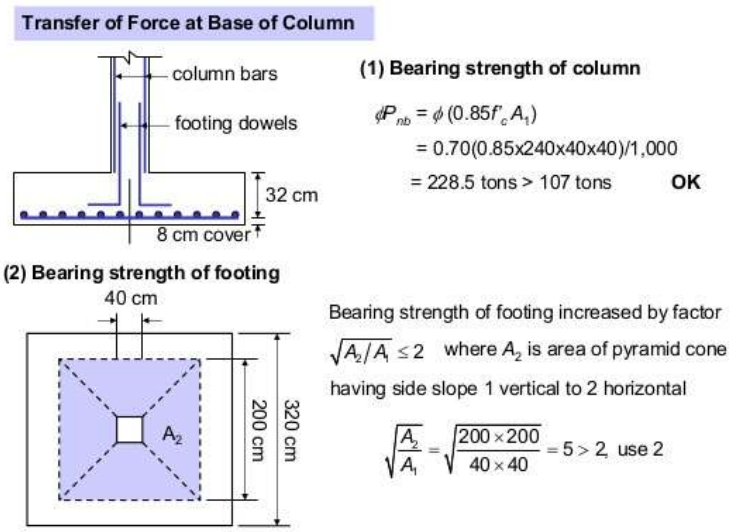
The pressure on the soil from each square foot of the footings should be the same, where the soil is uniform, and at no place must the bearing power of the soil be exceeded. To secure the most satisfactory results, therefore, the footings must be proportioned to properly distribute the weight they are to carry over sufficient areas of ground, to secure uniform settlement in each case. If these conditions were always properly considered, there would be few cracks in the mason work, as such cracks are caused usually by unequal settlement. A uniform settlement even of an inch or more would in most buildings pass unnoticed.

2. On which circumstances combined rectangular footings are suitable? [N/D-15]

Combined footings are provided when two or more columns are located close to each other or they are heavily loaded or rest on soil with low safe bearing capacity, resulting in an overlap of areas.

3. Why the dowel bars are provided in footing? [M/J-16]

When complete column bars are not erected at the beggining then you can place dowel bars and tie column rods after footing



4. What is the necessity of providing combined footings? [M/J-16]

Combined footings are used when:

1) there are two isolated footings overlapping (when columns are too close to each other, like within 2m)

2) soil bearing capacity is inconsistent and low within an area

3) the footing is extending beyond your property.

5. Define punching shear. [N/D-16]

The shear failure of column footing occurs either similar to that of footing for wall due to punching of column through the slab known as Punching shear. It occurs at a distance of half the effective depth of footing from the face of column.

6. Enumerate proportioning of footings. [N/D-16]

The shear failure of column footing occurs either similar to that of footing for wall due to formation of diagonal tension cracks on an approximate 45˚ plane known as one way shear. The shear failure of column footing occurs either similar to that of footing for wall due to punching of column through the slab known as two way shear.

7. State the rankine's equation to determine the minimum depth of foundation? [M/J-13]

Minimum depth of foundation is calculated from Rankine formula

$$D_{min} = \frac{q}{\gamma}\left[\frac{1 - sin\emptyset}{1 + sin\emptyset}\right]^2$$

8. When is the combined footing provided? [M/J-13]

Combined footings are provided only when it is absolutely necessary, as

- When two columns are close together, causing overlap of adjacent isolated footings
- Where soil bearing capacity is low, causing overlap of adjacent isolated footings
- Proximity of building line or existing building or sewer, adjacent to a building column.

9. What are the advantages of providing a pedestal?

The advantages of providing pedestal are:

(i) For providing pedestal, the cantilevering projection of footing is reduced, thus reducing bending moment and shear for the footing

(ii) Width for resisting the bending moment is reduced

(iii) Larger perimeter is provided to resist two way shear

10. What are the causes for failure of footing? [N/D-12]

The common causes for failure of footing are:

(i) Unequal settlement of sub soil

(ii) Shrinkage of soil below the foundation due to withdrawal of moisture

(iii) Lateral pressure causing over turning of structure

(iv) Lateral movement of soil close to the structure.

11. Why transverse reinforcement is necessary in a column? [M/J-13]

Transverse reinforcement is provided to impart effective lateral support against buckling to every longitudinal bar. It is either in the form of circular rings or polygonal link(lateral ties) with internal angles not exceeding 135˚.

12. What is meant by uniaxially and biaxially eccentrically loaded columns? [N/D-12]

Uniaxially eccentrically loaded columns: If the moments act about only one axis, they are called as uniaxially eccentrically loaded columns.

Biaxially eccentrically loaded columns: If the moments act about both the axis, they are called as biaxially eccentrically loaded columns.

13. List out the specifications for spacing of transverse links. [M/J-12]

Spacing of transverse links shall not exceed the least of the following:

(a) The least lateral dimensions of the column

(b) Sixteen times the diameter of smallest longitudinal reinforcing rod in column

(c) Forty-times the diameter of transverse reinforcement

14. What are the specifications for diameter of transverse links? [N/D-11]

Specifications for diameter of transverse links are the following:

The diameter of the transverse links shall not be less than

(i) One-fourth diameter of the largest longitudinal bar

(ii) 5 mm

15. List out the IS recommendations regarding longitudinal reinforcements. [M/J-12]

The following are the IS recommendations regarding longitudinal reinforcements:

a) The minimum number of longitudinal bars provided in a column shall be four in rectangular columns and six in circular column

b)The bars shall be not be less than 12 mm in diameter

c) Spacing of longitudinal bars measured along the periphery of column shall not exceed 300 mm

Part - B (16 marks)

1. A 230mm thick masonry wall is to provided with a reinforced concrete footing on a site having soil with SBC, unit weight and angle of repose of 125 kN/m^2, 17.5 kN/m^3 and 30^0 respectively. The M20 grade of concrete and HYSD steel bars of grade Fe 415. Design the footing when the wall supports at service state: a load of 150 kN/m length. (16) [N/D-15], [M/J-12]

Given data:-

P_u = 1500kN, f_{ck} = 20 N/mm^2

b = 300mm f_y = 415 N/mm^2

D = 500mm

q_s = 200kN/m^2

Factored SBC = 1.5 × 200

= 300kN/m^2

step①:- size of footing:-

Load on column = 1500kN

Selfweight of the footing (assumed) } 10% = 150kN

Total Factored load W_u = 1650kN

Footing area = $\frac{1650}{700}$ = 5.5m^2

= 6.0m^2

Proportion the footing area in the same proportion as the sides of the column

Hence (3x) × (5x) = 6

Short side of footing = 3×0.63 = 1.89 m

long side of footing = 5×0.63 = 3.25 m

Adopt a rectangular footing of size 2m by 3m.

Factored soil pressure at the base is given as

$$q_u = \frac{1650}{2\times3} = 275\ kN/m^2 < 300 kN/m^2$$

Hence the footing area is adequate

Step ②:- Factored Bending moment:-

Cantilever projection from the short face of the column } = 0.5 (3 − 0.5) = 1.25 m

cantilever projection from the long face of the column } = 0.5 (2 − 0.3) = 0.85 m

Bending moment at short side face of the column } = $0.5 q_u L_y^2$

$= 0.5 \times 275 \times 1.25^2$

$= 214.8\ kNm.$

Bending moment at long side face of the column } = $0.5\, q_u L_x^2$

$= 0.5 \times 275 \times 0.85^2 = 99.3\ kNm.$

Step (3): –

Depth footing

a) Depth based on moment consideration.

$$M_u = 0.138 f_{ck}\, b d^2$$

$$d = \sqrt{\frac{M_u}{0.138 \times f_{ck} \times b}} = \sqrt{\frac{214.8 \times 10^6}{0.138 \times 20 \times 1000}}$$

$$= 279\,mm$$

b) Depth based on shear stress.

Shear force per metre (longer direction)

$$V_{uL} = 275 \left[\frac{3000}{2} - \frac{500}{2} - d\right] N$$

$$V_{uL} = 275\,[1250 - d]\,N$$

Assuming the shear strength of τ_c $= 0.36\,N/mm^2$ for M_{20} grade concrete with nominal percentage of steel $p_t = 0.25$.

$$\tau_c = \frac{V_{uL}}{bd}$$

$$0.36 = \frac{2.75\,(1250 - d)}{10^3 \times d}$$

$$360d = 275 \times 1250 - 275d.$$

$$d = \frac{275 \times 1250}{(360 + 275)} = 541\,mm$$

Adopt effective depth of 550 mm and overall depth of 600 mm

Step (4) Reinforcement details :-

a) Longer direction

$A_{st}(l) = ??$

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

$$A_{st}(l) = 1125.5 \text{ mm}^2$$

Adopt 16mm diameter bars at 160mm c/c.

b) Shorter direction :-

$$99.3 \times 10^6 = 0.87 \times 415 \times A_{st} \times 550 \left[1 - \frac{415 A_{st}}{1000 \times 550 \times 20}\right]$$

$$A_{st} = 510 \text{mm}^2$$

Provide 12mm ϕ bars at 200mm c/c.

c) Central Band :-

Ratio of long to short side

$$\beta = \frac{L_y}{L_x} = 3/2 = 1.5$$

Reinforcement in central band width of 2m $= \frac{2}{\beta + 1} A_{st}$

$$= \frac{2}{1.5+1} \times 2 \times 565 = 904 \text{ mm}^2$$

Minimum reinforcement

$$= 0.0012 \times 1000 \times 600 = 1440 \text{mm}^2 > 904 \text{mm}^2$$

Provide 12mm ϕ bars at 150mm c/c.

1. A Rectangular column 600x400 mm carries a load of 800kN. Design a rectangular footing to support the column. The safe bearing capacity of the soil is 200 kN/m^2. Use M20 grade concrete. (16) [N/D-15]

Safe bearing Capacity = 150 kN/m^2

Characteristic load P = 250 kN/m

Thickness of wall = 345 mm

Depth of footing below G.L = 1.2 m

Grade of Concrete M_{20} f_{ck} = 20 N/mm^2

Grade of Steel Fe_{415} f_y = 415 N/mm^2

Solution:-

a) Width of footing Calculation:-

Assume Self weight of footing and weight of backfill = 10% of load, P

$= 10/100 \times 250 = 25$ kN/m

Total load = 250 + 25 = 275 kN/m

considering 1m width of footing along the wall,

Required length of footing = $\frac{\text{Total load}}{\text{SBC}}$

$= \frac{275}{150}$

$= 1.83\,m$

Provide 1.85m length footing.

b) Bending moment Calculation:—

Projection form face of wall

$x = \frac{\text{length of footing} - \text{Wall thickness}}{2}$

$x = \frac{1.85 - 0.345}{2} = 0.7525m$

p per m run $= \frac{250}{185 \times 1} = 135\,kN/m^2$

Bending moment at face of wall $= \frac{px^2}{2}$

$= \frac{135 \times 0.7525^2}{2}$

$= 38.22\,kNm$

Factored B.M = 57.33 kNm

c) Thickness of footing Calculation:-

Required effective depth d_{req}

$$= \sqrt{\frac{M_u}{0.138 \times f_{ck} \times b}}$$ For Fe415 steel

$$= \sqrt{\frac{57.33 \times 10^6}{0.138 \times 20 \times 1000}}$$

$$= 144.13 mm$$

Provide 16mm diameter main reinforcement

$$D_{req} = d_{req} + \text{Clear cover} + \phi/2$$

$$= 144.13 + 50 + 16/2$$

$$= 202.13 mm$$

Consider $D_{provided} = 2 \times D_{req}$. to avoid failure of footing in punching shear.

$$D_p = 2 \times 202.13 = 404.26 mm$$

Provide 405mm overall depth.

Effective depth provided $d_p = 405 - 50 - \frac{16}{2}$

$$= 347 mm$$

From Table 2, SP16

For $f_{ck} = 20\ N/mm^2$, & $f_y = 415\ N/mm^2$

$$P_t = \frac{0.128 + 0.143}{2} = 0.136\%$$

$$A_{st} = p_t\, bd = \frac{0.136}{100} \times 1000 \times 347$$

$$= 471.92\ mm^2$$

$$\text{Spacing} = \frac{\text{Area of one bar}}{A_{st}} \times 1000$$

$$= \frac{201}{471.92} \times 1000 = 425.92\ mm\ c/c.$$

(16mm ϕ) used.

$$\text{Spacing} = \frac{113}{471.92} \times 1000 = 239.45\ mm\ c/c$$

(12mm ϕ bars used.)

Provide 12mm ϕ bars at 230mm c/c.

Area of distribution steel $A_{st\,dist} = 0.12\%$ of $b \times D$

(minimum Ast).

$$A_{st(dist)} = \frac{0.12}{100} \times 1000 \times 405$$

$$= 486\ mm^2.$$

Provide 12mm ϕ bars.

Provide 12mm ϕ bars at 230 mm c/c.

e) Check for one way shear or vertical Shear:-

$$A_{st\,prov.} = \frac{\text{Area of one bar}}{\text{Spacing provided}} \times 1000$$

$$= \frac{113}{230} \times 1000 = 491.3\ mm^2$$

$$P_t = \frac{100 A_{st}}{bd} = \frac{100 \times 491.3}{1000 \times 349}$$

$$d = 405 - 50 - 12/2 = 349mm$$

$$P_t = 0.14\%$$

From Table 19, IS 456 - 2000,

$$\tau_c = 0.28\ N/mm^2$$

$$V_{uc} = \tau_c \times b \times d$$

$$= 0.28 \times 1000 \times 349 = 97720 N$$

Shear resisted by concrete $V_{uc} = 97.72 kN$

One way shear is critical at section 'd' from face of the wall.

$$V_u = 1.5 \times (p \times \text{Shaded area}).$$

$$V_u = 1.5 \times (135 \times 0.403 \times 1m$$

$$V_u = 81.61\ kN < 97.72\ kN$$

$$V_u < V_{uc}$$

Hence safe in one way shear.

2. Design a reinforced concrete footing for a rectangular column of section 300mm x500mm supporting an azial factored load 1500 kN. The safe bearing capacity of the soil at site is 185 kN/m². Adopt M 20 grade of concrete and HYSD steel bars of grade Fe 415. (16) [M/J-16]

Given data:—

Size of column = 400mm x 400mm.

Spacing of columns = 5m

Working load on each column = 500kN

Bearing capacity of soil q_s = 200kN/m²

f_{ck} = 20 N/mm² & f_y = 415 N/mm²

Step (1):- Loads on footing:—

Factored load on each column

= 1.5 x 500 = 750 kN

Total load on columns = 2x750

= 1500kN

Selfweight (10% assumed = 150 kN

Total factored load = 1650kN.

Step(2):- Size of footing :-

Factored Bearing Capacity $= 1.5 \times 200$
$= 300 kN/m^2$

Area of footing $= \frac{1650}{300} = 5.5 m^2$

Spacing between columns = 5m.

Adopt a footing size of 7m by 1.5m ($A = 10.5 m^2$) giving projection from column about 500mm

A combined footing with a strap beam is assumed.

Adopt width of strap beam $= 400 + 50 + 50$
$= 500mm$

A combined footing with a strap beam is assumed.

Adopt width of strap beam = 500mm.

Step(3):- Design of Footing:-

soil pressure $q_u = \frac{1650}{7 \times 1.5}$

$= 157 kN/m^2$

$< 300 kN/m^2$

Cantilever projection of footing

$= 0.5 (1.5 - 0.50) = 0.50 m$

Ultimate design moment $= 0.5 q_u L^2$

$M_u = 0.5 \times 157 \times 0.50^2$

$= 19.6$ kNm

Effective depth of footing $d = \sqrt{\frac{M}{0.138 f_{ck} b}}$

$= 84.3$ mm

Depth based on shear consideration will be nearly double than that required for moment.

Hence adopt effective depth $d = 250$ mm

over all depth $D = 250 + 50$

$= 300$ mm

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

$$19.6 \times 10^6 = 0.87 \times 415 \times A_{st} \times 250 \left[1 - \frac{A_{st} \times 415}{1000 \times 250 \times 20}\right]$$

$A_{st} = 222$ mm^2.

Minimum reinforcement $= \frac{0.12}{100} \times 1000 \times 300$

$= 360$ mm^2

Adopt 10mm diameter bars at 200 mm centres ($A_{st} = 393$ mm^2) as main reinforcement

Step (4) Check for Shear Stress:–

Shear stress at a distance equal to the effective depth is

$$V_u = (0.50 - 0.25)\ 157$$

$$= 39.7\ KN$$

$$\tau_v = \frac{V_u}{bd} = \frac{39.3 \times 10^3}{1000 \times 250} = 0.158\ N/mm$$

$$P = \frac{100 \times A_{st}}{bd} = \frac{100 \times 393}{1000 \times 250}$$

$$= 0.157$$

From table 19 of IS 456 - 2000

$$\tau_c = 0.28\ N/mm^2$$

∴ Permissible shear stress $= k\tau_c$

$$= 1 \times 0.28$$

$$= 0.28\ N/mm^2$$

Since $k\tau_c > \tau_v$ shear stresses are within permissible limits.

Step (5):– Design of strap Beam:–

Factored load on the beam $W_u = 1.5 \times 157$

$$= 236\ kN/m$$

Neglecting the small cantilever projection of beam.

$$M = \frac{W_u L^2}{8} = \frac{236 \times 5^2}{8}$$

$$V_u = \frac{W_u L}{2} = \frac{236 \times 5}{2} = \frac{236 \times 5}{2}$$

$$= 590\ kN$$

Depth of strap beam is computed based on the moment which will be less than shear consideration.

Assuming $\tau_c = 1.2\ N/mm^2$

$$\tau_c = \frac{V_u}{bd}$$

$$d = \frac{V_u}{b\,\tau_c} = \frac{500 \times 10^3}{500 \times 1.2} = 983\ mm$$

$$= 1000\ mm$$

Effective depth $d = 1000\ mm$

overall depth $D = 1150\ mm$

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

$$738 \times 10^6 = 0.87 \times 415 \times A_{st} \times 1000 \times \left[1 - \frac{A_{st} \times 415}{500 \times 1000 \times 20}\right]$$

$$A_{st} = 2257\ mm^2$$

Provide 5 bars of 25 mm diameter

Shear stress $\tau_v = \frac{V_u}{bd}$

$$= \frac{590 \times 10^3}{500 \times 1000} = 1.18\ N/mm^2$$

$$P = \frac{100 A_{st}}{bd} = \frac{100 \times 2453}{500 \times 1000}$$

From Table 19 of IS 456 - 2000, the Permissible shear stress is

$$\tau_c = 0.46 \, N/mm^2 < \tau_v$$

Hence shear reinforcement are required to resist the balanced shear force which is calculated as

$$V_u = \left[590 - \frac{(0.46 \times 500 \times 1000)}{1000}\right]$$

$$= 360 \, kN$$

Using 10mm ϕ 4 legged stirrups the spacing is

$$S_v = \frac{0.87 \times f_y \times A_{st} \times d}{V_u}$$

$$= \frac{0.87 \times 415 \times 4 \times \left(\frac{\pi \times 10^2}{4}\right) \times 1900}{360 \times 10^3}$$

$$= 316 mm$$

Adopt 10mm ϕ 4 legged stirrups at 300mm in the strap beam.

Side face reinforcement of 0.1% of web area as specified in the IS 456-2000 Code is provided.

3. Design a combined column footing with a strap beam for two reinforced concrete 300mm x300mm size spaced 4m apart and each supporting a factored axial load of 750kN. Assume the ultimate bearing capacity of soil at site as 225 kN/m². Adopt M20 grade of concrete and steel grade Fe415 HYSD bars. (16) [M/J-16], [N/D-11]

Step 1: Size of footing

Load on column = 800 kN

Extra load at 10% of load due to self weight of soil = 80 kN

Hence, total load, P = 880 kN

Let us provide a square isolated footing, where L=B

Equating the maximum pressure of the footing to SBC of soil,

$$\frac{P}{A} + \frac{M}{Z} = SBC$$

i.e., $\frac{880}{B^2} + \frac{40 \times 6}{B^3} = 250$

On solving the above equation, and taking the least and feasible value, B = 2 m

Hence, provide a square footing of size 2 m x 2 m

The maximum and minimum soil pressures are given by

$$p_{max} = \frac{800}{2^2} + \frac{40 \times 6}{2^3} = 230\frac{kN}{m^2} < 250\frac{kN}{m^2} \; O.K.$$

$$p_{min} = \frac{800}{2^2} - \frac{40 \times 6}{2^3} = 170\frac{kN}{m^2} > Zero \quad O.K.$$

Hence, factored upward pressures of soil are,

$p_{u,max}$ = 345 kN/m² and $p_{u,min}$ = 255 kN/m²

Further, average pressure at the center of the footing is given by

$p_{u,avg}$ = 300 kN/m^2

and, factored load, P_u = 900 kN, factored uniaxial moment, M_u = 60 kN-m

Step 2: Two way shear

Assume an uniform overall thickness of footing, D = 450 mm

Assuming 16 mm diameter bars for main steel, effective thickness of footing 'd' is

d = 450 – 50 – 16 – 8 = 376 mm

The critical section for the two way shear or punching shear occurs at a distance of d/2 from the face of the column (Fig. 9), where a and b are the dimensions of the column.

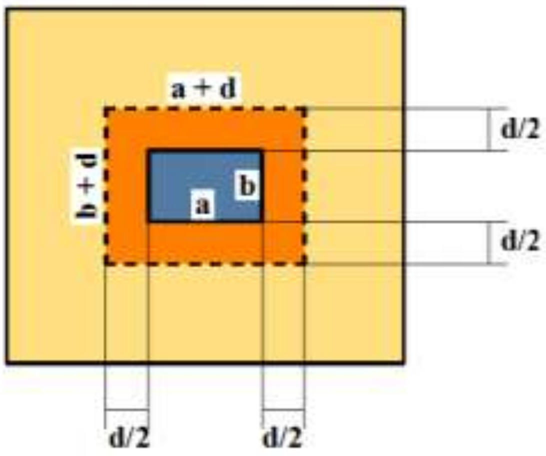


Fig. 9 Critical section in two way shear

Hence, punching area of footing = $(a + d)^2 = (0.30 + 0.376)^2 = 0.457$ m^2

where a = b = side of column

Punching shear force = Factored load – (Factored average pressure x punching area of footing)

= 1200 – (300 x 0.457)

= 1062.9 kN

Perimeter along the critical section = 4 (a+d) = 4 (300+ 376)

= 2704 mm

Therefore, nominal shear stress in punching or punching shear stress ζ_V is computed as

$$\zeta_v = \frac{\textit{Punching shear force}}{\textit{perimeter x effective thickness}}$$

$$= \frac{1062.9 \times 1000}{2704 \, x \, 376} = 1.05 \, N/mm^2$$

Allowable shear stress = $k_S \cdot \zeta_C$

where $\zeta_C = 0.25\sqrt{f_{ck}} = 0.25\sqrt{25} = 1.25 \, N/mm^2$

and, $k_s = (0.5 + \beta_C) = \left(0.5 + \frac{0.30}{0.30}\right) = 1.0$; Hence, adopt k_s=1

Thus, Allowable shear stress = $k_S \cdot \zeta_C$ = 1 x 1.25 = 1.25 N/mm^2

Since the punching shear stress (1.05 N/mm^2) is less than the allowable shear stress (1.25 N/mm^2), the assumed thickness is sufficient to resist the punching shear force.

Hence, the assumed thickness of footing D = 450 mm is sufficient.

The effective depth for the lower layer of reinforcement, , d = 450 – 50 – 8 = 392 mm, and the effective depth for the upper layer of reinforcement, d = d = 450 – 50 – 16 – 8 = 376 mm.

Step 3: Design for flexure

The critical section for flexure occurs at the face of the column (Fig. 10).

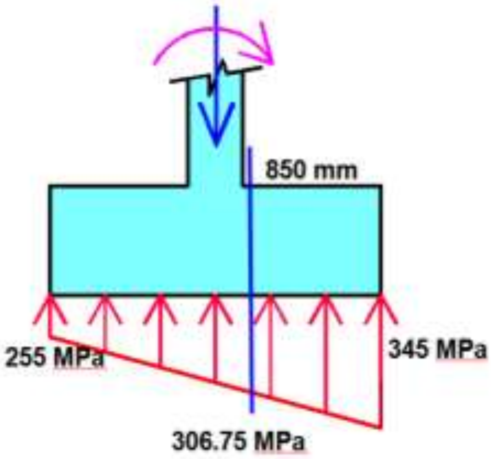


Fig. 10 Critical section for flexure

The projection of footing beyond the column face is treated as a cantilever slab subjected to factored upward pressure of soil.

Factored maximum upward pressure of soil, $p_{u,max}$ = 345 *kN/m²*

Factored upward pressure of soil at critical section, p_u = 306.75 *kN/m²*

Projection of footing beyond the column face, l = (2000 – 300)/2 = 850 mm

Bending moment at the critical section in the footing is

$$\mathbf{M_u = [Total\ force]\ x\ [Distance\ of\ CG\ from\ critical\ section]}$$

$$\mathbf{M_u} = \left[\left(\frac{345+306.75}{2}\right)0.85\right]\ \mathrm{x}\ \left[\left(\frac{2\ \mathrm{x}\ 345+306.75}{345+306.75}\right)\mathrm{x}\ \frac{0.85}{3}\right]$$

M_u = 119.11 kN-m/ m width of footing

The area of steel A_{st} can be determined using the following moment of resistance relation for under reinforced condition given in Annex G – 1.1 b of IS 456 :2000.

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

Considering 1m width of footing,

$$119.11\ \mathrm{x}\ 10^6 = 0.87\ \mathrm{x}\ 415\ \mathrm{x}\ A_{st}\ \mathrm{x}\ 376\left[1-\frac{415\ \mathrm{x}\ A_{st}}{1000\ \mathrm{x}\ 376\ \mathrm{x}\ 25}\right]$$

Solving the quadratic equation,

A_{st} = 914.30 mm² and 21,735.76 mm²

Selecting the least and feasible value, A_{st} = 914.30 mm²

The corresponding value of p_t = 0.24 %

Hence from flexure criterion, p_t = 0.24 %

Step 4: One way shear

The critical section for one way shear occurs at a distance of 'd' from the face of the column (Fig. 11).

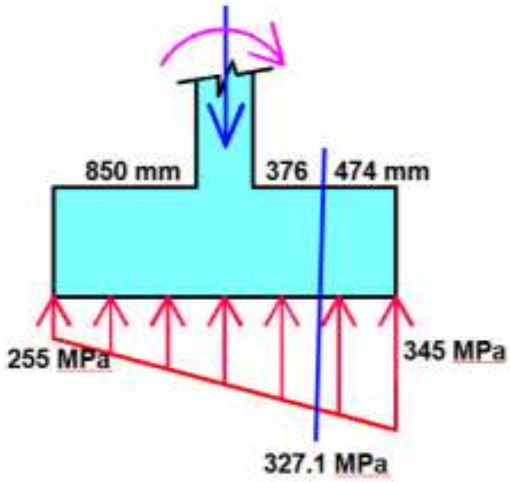


Fig. 11 Critical section for one way shear

Factored maximum upward pressure of soil, $p_{u,max}$ = 345 kN/m^2

Factored upward pressure of soil at critical section, p_u = 327.1 kN/m^2

For the cantilever slab, total Shear Force along critical sectionconsidering the entire width B is

$$V_u = [\text{Total force}]\ x\ [(l - d) \times B]$$

$$V_u = \left[\frac{345 + 327.1}{2}\right]\ x\ [(0.85 - 0.376) \times 2]$$

V_u = 318.58 kN

The nominal shear stress is given by

$$\zeta_V = \frac{V_u}{B\,d} = \frac{318.58 \times 1000}{2000 \times 376} = 0.42\ N/mm^2$$

From 19 of IS 456 :2000, find the p_t required to have a minimum design shear strength $\zeta_C = \zeta_V$ = 0.42 N/mm^2 with f_{ck} = 25 N/mm^2.

For p_t = 0.365 % the design shear strength ζ_C is 0.42 N/mm^2 = ζ_V = 0.42 N/mm^2.

Hence from one way shear criterion, p_t = 0.365 %

Comparing p_t from flexure and one way shear criterion, provide p_t = 0.365 % (larger of the two values)

Hence, $A_{st} = \frac{p_t}{100}\ b\ d = \frac{0.365}{100}\ 1000 \times 376 = 1372.4\ mm^2$

Provide ϕ 16 mm dia bars at 140 mm c/c.

Therefore, A_{st} provided = 1436 mm^2 > A_{st} required (1372.4 mm^2). Hence O.K.

Step 5: Check for development length

Sufficient development length should be available for the reinforcement from the critical section. Here, the critical section considered for L_d is that of flexure.

The development length for 16 mm dia bars is given by

$L_d = 47\,\phi = 47 \times 16 = 752$ mm.

Providing 60 mm side cover, the total length available from the critical section is

$\frac{1}{2}(L - a) - 60 = \frac{1}{2}(2000 - 300) - 60 = 790 \text{ mm} > L_d$ Hence O.K.

Step 6: Check for bearing stress

The load is assumed to disperse from the base of column to the base of footing at rate of 2H : 1V.

Hence, the side of the area of dispersion at the bottom of footing = 300 + 2 (2 x 450) = 2100 mm.

Since this is lesser than the side of the footing (i.e., 2000 mm),

$A_1 = 2 \times 2 = 4\ m^2$

The dimension of the column is 300 mm x 300 mm.

Hence, $A_2 = 0.30 \times 0.30 = 0.09\ m^2$

$$\sqrt{\frac{A_1}{A_2}} = \sqrt{\frac{4}{0.09}} = 6.67 > 2$$

Hence, Limit the value of $\sqrt{\frac{A_1}{A_2}} = 2$

∴ Permissible bearing stress = $0.45\, f_{ck}\sqrt{\frac{A_1}{A_2}}$

$= 0.45 \times 25 \times 2 = 22.5\ N/mm^2$

Actual bearing stress = $\frac{\text{Factored load}}{\text{Area at column base}} = \frac{1200 \times 1000}{300 \times 300} = 13.33\ N/mm^2$

Since the Actual bearing stress (13.33 N/mm^2) is less than the Permissible bearing stress (22.5 N/mm^2), the design for bearing stress is satisfactory.

Appropriate detailing should be shown both in plan and elevation for the footing as per the recommendations given in SP 34.

4. A 230mm thick masonry wall is to provided with a reinforced concrete footing on a site having soil with SBC, unit weight and angle of repose of 130 kN/m^2, 17.5 kN/m^3 and 30^0 respectively. The M20 grade of concrete and HYSD steel bars of grade Fe 415. Design the footing when the wall supports at service state: a load of 150 kN/m length. (16) [N/D-16], [M/J-14], [N/D-13]

Solution :

Step 1: Size of the footing:

Assuming the weight of combined footing and backfill as 15 per cent of the total loads of the columns, we have the required base area, considering

qc = 200

Area of the base = (800 + 1200)(1.15)/200

= 11.5 m2 .

It is necessary that the resultant of the loads of two columns and the centroid of the footing coincide so that a uniform distribution of soil pressure is obtained.

Thus, the distance of the centroid of the footing y from column C1 is:

y = 800(0) + 1200(3)/2000 = 1.8 m

Since y is greater than half the c/c distance of column, a rectangular footing has to be designed.

Let us provide 4 m x 3 m coinciding the centroid of the footing and the resultant line of action of the two loads, i.e. at a distance of 2 m from the left edge.

Step 2: Thickness of footing slab based on one way shear:

Considering the footing as a wide beam of B = 3 m in the longitudinal direction, the uniformly distributed factored load = (800 + 1200)(1.5)/4 = 750 kN/m.

The critical section of one-way shear is sec.11 (at point K) at a distance of d + 250 mm from G (the location of column C2).

The one-way shear force is Shear force = (1600 – d – 250)1200/1600

= (1012.5 – 0.75d) kN

Assuming p = 0.15 per cent reinforcement in the footing slab,

the shear strength of M 20 concrete = 0.28 N/mm2 .

Hence, the shear strength of section 11 = (3000)d(0.28)(10-3) kN.

From the condition that shear strength shear force,

we have ≥ (3000)d(0.28)(10-3) 1012.5 – 0.75d,

which gives d 636.79 mm.

Provide d = 650 mm and

The total depth = 650 + 50 + 16 + 8 = 724 mm (assuming cover = 50 mm and the diameter of bars = 16 mm).

Step 3: Checking for two-way shear

(i) Around column C2

The effective depth along 4.0 m is 650 + 16 = 666 mm.

The critical section for the two-way shear around column C2 is at a distance of 666/2 = 333 mm from the face of the column and marked by 2222.

The two-way punching shear force, considering the soil pressure = 750/3 = 250 kN/m2 ,

is Vu = 1800 – (1.166)(0.966)(250)

= 1518.411 kN

As per cl.31.6.3.1 of IS 456,

here ks = 0.5 + (500/300) but >/ 1.0;

so, ks = 1.0.

Therefore,

shear strength of concrete = 0.25(20)1/2 (2){(300 + 666) + (500 + 666)}(666) = 3174.92 kN > 1518.411 kN. Hence, o.k.

(ii) Around column C1

The effective depth of footing is 666 mm.

The critical section is marked by 3333.

The two-way punching shear = 1200 – (1.066)(0.733)(250) = 1004.65 kN.

The resistance to two-way shear = 0.25(20)1/2 (2){(1066 + 733)}(666)

= 2679.1 kN > 1004.65 kN.

Hence, o.k.

Thus, the depth of the footing is governed by one-way shear.

Step 4: Gross bearing capacity

Assuming unit weights of concrete and soil as 25 kN/m3 and 18 kN/m3 , respectively,

The gross bearing capacity under service load is determined below.

(i) Due to two loads: (800 + 1200)/(3)(4) = 166.67 kN/m2

(ii) Due to weight of the footing: With a total depth of the footing = 724 mm,

the pressure = 0.724(25) = 18.1 kN/m2 .

(iii) Due to backfill of 1500 – 724 = 776 mm,

(iv) the pressure = 0.776(18) = 13.968 kN/m2 .

(v) The total pressure = 166.67 + 18.1 + 13.968 = 198.738 kN/m2 < 200 kN/m2 .

Hence, o.k.

Step 5: Bending moments (longitudinal direction)

Maximum positive moment shows the maximum positive bending moment = 720 kNm at a distance of 1.4 m from the column C1 (at point J).

With effective depth d = 666 mm,

we have M/Bd2 = 720(106)/(3000)(666)(666) = 0.541 N/mm2

Table 2 of SP-16 gives

p = 0.1553 per cent.

Ast = 0.1553(3000)(666)/100 = 3102.894 mm

Provide 16 bars of 16 mm diameter (area = 3217 mm2),

spacing = (3000 – 50 – 16)/15

= 195.6 mm c/c, say 190 mm c/c.

Development length of 16 mm bars = 47.01(16)

= 752.16 mm

Length available = 1600 – 50 – 16

= 1534 mm > 752.16 mm

Hence, o.k.

Maximum negative moment shows the maximum negative moment = 240 kNm at a distance of 800 mm from the right edge.

With the effective depth = 666 mm,

We have M/Bd2 = 240(106)/(3000)(666)(666)

= 0.018 N/mm2

It is very nominal.

So, provide 0.15 per cent steel, which gives Ast = 0.15(3000)(666)/100

= 2997 mm2 .

Provide 27 bars of 12 mm (area = 3053 mm2) at spacing = (3000 – 50 – 12)/26 = 113 mm c/c; say 110 mm c/c.

Development length = 47.01(12) = 564 mm Length available = 800 – 50 – 12 = 738 mm > 564 mm

Hence, o.k.

Step 6: Design of column strip as transverse beam

The two column strips under columns C1 and C2

Distribution reinforcement Nominal distribution reinforcement shall be provided at top and bottom where the main reinforcement bars are not provided.

The amount @ 0.12 per cent comes to 0.12(1000)(652)/100

= 782.4 mm2 /metre.

Provide 12 mm diameter bars @ 140 mm c/c (area = 808 mm2 /m).

5. A Rectangular column 550x350 mm carries a load of 775kN. Design a rectangular footing to support the column. The safe bearing capacity of the soil is 210 kN/m^2. Use M15 grade concrete. (16) [N/D-16], [M/J-12]

Given

Pu = 1620 kN and Mu = 170 kNm.

Step 1:

The footing should be symmetric with respect to the column as the moment is reversible.

Assuming the weights of footing and backfill as 15 per cent of Pu,

the eccentricity of load Pu at the base is 6 3 e = Mu/P(1.15)

= 170(10)/1620(1.15)(10) = 91.25 mm.

This eccentricity may be taken as < L/6 of the footing.

The factored bearing pressure is 200(1.5) = 300 kN/m2 .

For the footing of length L and width B,

we, therefore, have: 2 Pu/BL + 6M/BL 300 ≤ or, 1620(1.15)/BL + 6(170)/BL2 ≤ 300

or,

BL2 – 6.21L – 3.4 ≤ 0

For the economic proportion, let us keep equal projection beyond the face of the column in the two directions.

This gives (L – 0.45)/2 = (B – 0.3)/2

or, B = L – 0.15

we have (L – 0.15) L 2 – 6.212 – 3.4 ≤ 0 or L 3 – 0.15 L 2 – 6.21 L – 3.4 ≤ 0

We have L = 2.8 m and B = 2.65 m.

Let us provide L = 2.85 m and B = 2.70 m.

We get the maximum and minimum pressures as 1620(1.15)/(2.85)(2.70) 170(6)/(2.7)(2.85)(2.85)

= 242.105 ± 46.51

= 288.615 kN/m2 and 195.595 kN/m2 , respectively.

Both the values are less than 300 kN/m2 .

Hence, o.k.

Step 2: Thickness of footing slab based on one-way shear

The critical section is at a distance d from the face of the column.

The average soil pressure at sec.11 is {288.615 – (288.615 – 195.595)(1200 – d)/2850}

= 249.449 + 0.0326d.

The one-way shear force at sec.11 = (2.7)(1.2 – 0.001d((249.449 + 0.0326d) kN.

Assuming 0.15 per cent reinforcement in the footing slab, the shear strength of M 25 concrete = 0.29 N/mm2 .

Hence, the shear strength of the section = 2700(d)(0.29)(103) kN.

From the condition that shear strength has to be shear force, we have ≥ 2700(d)(0.29)(10-3) = (2.7)(1.2 – 0.001d)(249.449 + 0.0326d)

This gives, d 2 + 15347.51534d – 9182171.779 = 0

Solving, we get d = 576.6198. Let us assume d = 600 mm

Step 3: Checking for two-way shear

At the critical section, the shear resistance is obtained cl.31.6.31 of IS 456, which gives

cт = (0.5 + 450/300)(0.25)(25)1/2 but the multiplying factor (0.5 + 450/300) >/ 1.0.

So, we have cт = 0.25(25)1/2 = 1.25 N/mm2 .

Hence, the shear resistance

= (1.25)(2){(300 + 600) + (450 + 600)}(600)

= 2925 kN.

Actual shear force is determined on the basis of average soil pressure at the centre line of the cross-section which is (195.595 + 288.615)/2 = 242.105 kN/m2.

So, the actual shear force,

Vu = (242.105){(2.7)(2.85) – (0.3 + 0.6)(0.45 + 0.6)}

= 1634.209 kN < shear resistance (= 2925 kN).

Hence, the depth of the footing is governed by one-way shear.

With effective depth = 600 mm,

the total depth of footing = 600 + 50 (cover) + 16 (bar dia) + 8 (half bar dia) = 674 mm.

Step 4: Gross bearing capacity:

Assuming the unit weights of concrete and soil as 25 kN/m3 and 18 kN/m3 , respectively,

we have the bearing pressure for

(i) Pu = 1620 kN,

(ii) Mu = 170 kNm and

(iii) self weight of footing and backfill soil.

Due to Pu = 1620 kN:

pressure = 1620/(2.7)(2.85) = 210.53 kN/m 2

Due to Mu = 170 kNm: pressure

= ± 170(6)/(2.7)(2.85)(2.85)

= 46.51 kN/m

Self weight of footing of depth 674 mm and soil of (1000 – 674) = 326 mm:

pressure = 0.674(25) + 0.326(18)

= 22.718 kN/m2

Thus, the maximum and minimum pressures are = 210.53 + 22.718 46.51

= 279.758 kN/m±2 and 186.738 kN/m2 < 300 kN/m2 .

Hence, o.k.

Step 5: Bending moment

In the long direction (along the length = 2850 mm)

Bending moment at the face of column (is determined where the soil pressure

= 288.615 – (288.615 – 195.595)(1200)/2850 = 249.45 kN/m2 .

So, the bending moment = 249.45(2.7)(1.2)(0.6) + (288.615 – 249.45)(2.7)(1.2)(2)/(2)(3) = 527.23 kNm.

M/Bd2 = 527.23(106)/(2700)(616)(616) = 0.515 N/mm2 < 3.45 N/mm2 for M 25 concrete.

Table 3 of SP-16 gives p = 0.1462 < 0.15 per cent as required for one-way shear.

2 Thus, Ast = 0.15(2700)(616)/100 = 2494.8 mm .

Provide 13 bars of 16 mm diameter (area = 2613 mm2), spacing = (2700 – 100 – 16)/12 = 215.33 mm, say 210 mm c/c.

(ii) In the short direction (B = 2700 mm)

The average pressure on soil between the edge and centre of the footing

= (288.615 + 242.105)/2 = 265.36 kN/m2 .

The bending moment is determined with this pressure as an approximation.

Bending moment = (265.36)(1.2)(0.6)(2.85) kNm

= 544.519 kNm M/Ld2 = 544.519(106)/(2850)(600)(600)

= 0.531

Table 3 of SP-16 gives p = 0.15068,

which gives area of steel = 0.15068(2850)(600)/100 = 2576.628 mm2 .

Provide 13 bars of 16 mm diameter (area = 2613 mm2) @ 210 mm c/c; i.e. the same arrangement in both directions.

Step 6: Development length

Development length of 16 mm diameter bars (M 25 concrete)

= 0.87(415)(16)/4(1.6)(1.4)

= 644.73 mm.

Length available = 1200 – 50 – 8 = 1142 mm > 644.73 mm.

Hence, o.k.

Step 7: Transfer of force at the base of the column:

Since the column is having moment along with the axial force, some of the bars are in tension.

The transfer of tensile force is not possible through the column footing interface.

The required development length of 20 mm bars = 0.87(415)/4(1.4) (1.6) = 805.92 mm.

Length available = 600 mm < 805.92 mm.

The bars shall be given 90o bend and then shall be extended by 200 mm horizontally to give a total length of 600 + 8(20) (bend value) + 200

= 960 mm > 805.92 mm.

6. Design a reinforced concrete footing an azial factored load 2000 kN. The safe bearing capacity of the soil at site is 150 kN/m2. Adopt M 20 grade of concrete and HYSD steel bars of grade Fe 415. (16) [N/D-13] [M/J-12]

(i) Given Data

$$P_u = 2000 \text{ kN}; \qquad f_{ck} = 20 \text{ N/mm}^2$$

$$b = 400 \text{ mm}; \qquad f_y = 415 \text{ N/mm}^2$$

$$D = 600 \text{ mm}$$

$$q_s = 150 \text{ kN/m}^2$$

$$q_u = 1.5 \times 150 \text{ kN/m}^2$$

(ii) Size of Footing

Load on column = 2000 kN

Assume self weight is ignored

Total factored load, $W_u = 2000$ kN

$$\text{Footing area} = \frac{2000}{1.5 \times 150} = 8.9 \text{ m}^2 \approx 10 \text{ m}^2$$

Footing is proportioned approximately in the same proportion as that of the column sides.

Hence $4x \times 6x = 10$

$$x = 0.65$$

Short side of footing $= 4x = 4 \times 0.65 = 2.63 \text{ m} \approx 2.5 \text{ m}$

Long side of footing $= 6x = 5 \times 0.65 = 3.94 \text{ m} \approx 4.0 \text{ m}$

Factored soil pressure,

$$q_u = \frac{2000}{2.5 \times 4.0} = 200 \text{ kN/m}^2 < 1.5 \times 150 = 225 \text{ kN/m}^2$$

Hence the footing area is adequate since the soil pressure developed at the base is less than the factored bearing capacity of the soil.

(iii) Factored Bending Moment

$$\left.\begin{array}{l}\text{Cantilever projection from the}\\ \text{short side face of the footing}\end{array}\right\} = 0.5\,(4 - 0.6) = 1.70 \text{ m}$$

$$\left.\begin{array}{l}\text{Cantilever projection from the}\\ \text{long side face of the footing}\end{array}\right\} = 0.5\,(2.5 - 0.4) = 1.05 \text{ m}$$

$$\left.\begin{array}{l}\text{Bending moment at short}\\ \text{side face of the column}\end{array}\right\} = \frac{q_u \times L_y^2}{2} = \frac{200 \times 1.7^2}{2} = 289 \text{ kN.m}$$

$$\left.\begin{array}{l}\text{Bending moment at long}\\ \text{side face of the column}\end{array}\right\} = \frac{q_u \times L_x^2}{2} = \frac{200 \times 1.05^2}{2} = 110.25 \text{ kN.m}$$

(iv) Depth of Footing

(a) From moment consideration

$$M_u = 0.138\, f_{ck}\, b\, d^2$$

$$\therefore \qquad d = \sqrt{\frac{M_u}{f_{ck}\, b}} = \sqrt{\frac{289 \times 10^6}{0.138 \times 20 \times 10^3}} = 323.6 \text{ m}$$

(b) From shear stress consideration

For one-way shear the critical section is located at a distance d from the face of t

$$\left.\begin{array}{l}\text{Shear force per metre width}\\ \text{(longer direction)}\end{array}\right\} V_{uL} = q_u\left(\frac{L_y}{2} - \frac{600}{2} - d\right) N$$

$$V_{uL} = 200\left(\frac{4000}{2} - \frac{600}{2} - d\right) N = 200\,(1700 - d)\, N$$

Assuming shear strength $\tau_c = 0.36 \text{ N/mm}^2$ for M20 concrete with nominal of steel, $p = 0.25$

$$\tau_c = \frac{V_{uL}}{b\, d}$$

$$0.36 = \frac{200\,(1700 - d)}{1000 \times d}$$

$$360\, d = 140000 - 200\, d$$

$$d = 250 \text{ mm}$$

Adopt effective depth as 350 mm and overall depth as 400 mm.

(v) Reinforcement

(a) Longer Direction

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

$$289 \times 10^6 = 0.87 \times 415 \times A_{st} \times 350\left[1 - \frac{A_{st} \times 415}{10^3 \times 350 \times 20}\right]$$

$$289 \times 10^6 = 126367.5\, A_{st} - 7.49\, A_{st}^2$$

$$A_{st}^2 - 16871.5\, A_{st} + 38584779.7 = 0$$

$$A_{st} = \frac{+\,16871.5 \pm \sqrt{(-\,16871.5)^2 - 4 \times 1 \times 38584779.7}}{2}$$

$$(A_{st})_l = \frac{16871.5 \pm 11415.3^2}{2} = 2728 \text{ mm}^2$$

$$\left.\begin{array}{l}\text{No. of bars per metre length}\\ \text{of 20 mm dia. bars}\end{array}\right\} = \frac{2728}{\pi\,\frac{20^2}{4}}$$

$$= 8.7$$

$$\text{Spacing } = \frac{1000}{8.7} = 114.9 \text{ mm} = 100 \text{ mm}$$

Adopt 20 mm diameter bars at 100 mm centres $((A_{st})_l = 314 \times 10 = 3140 \text{ mm}^2)$

(b) Shorter Direction

$$110 \times 10^6 = 0.87 \times 415 \times A_{st} \times 350\left[1 - \frac{415\, A_{st}}{10^3 \times 350 \times 20}\right]$$

$$A_{st}^2 - 16871.5\, A_{st} + 14686248.3 = 0$$

$$A_{st} = \frac{+16871.5 \pm \sqrt{(-16871.5)^2 - 4 \times 14686248.3}}{2}$$

$$(A_{st})_s = \frac{+16871.5 \pm 15030.1^2}{2} = 920.7 \text{ mm}^2$$

Provide 16 mm dia bars at 200 mm centres.

(c) Central Band

Central band width = Width of footing = 2.5 m.

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

$$\beta = \frac{4}{2.5} = 1.6$$

$$\therefore \left.\begin{array}{l}\text{Reinforcement in the central}\\ \text{band of 2.5 m}\end{array}\right\} = \left(\frac{2}{1.6+1}\right) 920.7 \times 2.5$$

$$(A_{st})_{cb} = 1770.6 \text{ mm}^2$$

Minimum reinforcement $= 0.0012 \times 1000 \times 400$

$$= 960 \text{ mm}^2 < 1770.6 \text{ mm}^2$$

Hence provide 16 mm dia bars at 110 mm centres $((A_{st})_{cb} = 1809 \text{ mm}^2)$

Critical section for one-way shear is located at a distance d from the face of the column

$$\left.\begin{array}{l}\text{Ultimate shear force per}\\ \text{metre width in the longer direction}\end{array}\right\} V_{uL} = 200\,(1700 - 350)/10^3$$

$$V_{uL} = 270 \text{ kN}$$

$$\frac{100\,(A_{st})_l}{b\,d} = \frac{100 \times 3140}{10^3 \times 350} = 0.90$$

From Table 19 of IS456-2000 the permissible stress in concrete is got as

$$k_s\,\tau_c = 1 \times 0.596 = 0.596 \text{ N/mm}^2$$

Nominal shear stress $\tau_v = \dfrac{V_u}{b\,d} = \dfrac{270 \times 10^3}{10^3 \times 350} = 0.77 \text{ N/mm}^2$

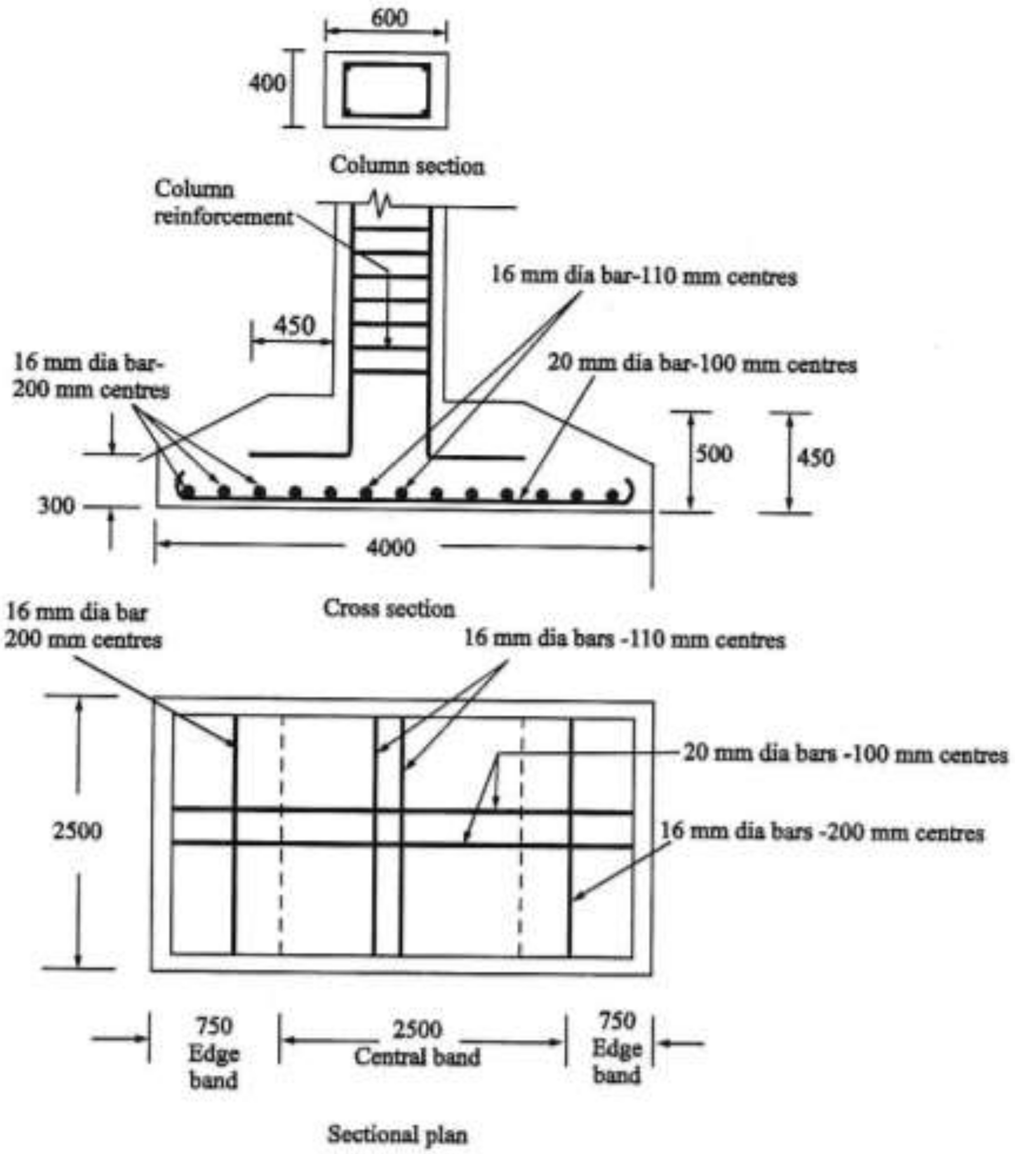
Making $k_s\,\tau_c = \tau_v = 0.596$ and the depth may be worked out as

$$\tau_v = 0.596 = \frac{270 \times 10^3}{10^3 \times d}$$

$\therefore \quad d = 453$ mm

Adopt a revised effective depth of 450 mm and overall depth of 500 mm.

(vi) Details of Reinforcement.



Details of reinforcement of the rectangular footing

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