DESIG OF CONCRETE STRUCTURE

G.C.BEHERA

A: Programme B.Tech

Course Code	**PCI4I103**
Course Name	DEISGN OF CONCRETE STRUCTURE
Semester	5th
Course Type	Theory
Contact Hour	40
IA Marks	50
University Exam Marks	100

B: Course Prerequisites

Code	Course Name	Description	Semester
PCME4202	Mechanics of Solid	Mod-II- Shear Force and Bending Moment for Simple Beams	3rd
PCME4202	Mechanics of Solid	Mod-II- Deflection of Beams	3rd

C: Course Objective

- To state the four objectives of the design of reinforced concrete structures and name the three methods of design of concrete structure and identify the best method of design,
- To state the basis of the analysis of the structure,
- Name the different loads, forces and effects to be considered in the design,
- State the basis of determining the combination of different loads acting on the structure.
- To gain basic knowledge to analyze the beams, slabs, stair cases and Columns .

D: Course Syllabus:

- **Design of Concrete Structures (3-0-0)**
- **Module I (10 Classes)**
- Properties of concrete and reinforcing steel, philosophy, concept and methods of reinforced concrete design, introduction to limit state method, limit state of collapse and limit state of serviceability, application of limit state method to rectangular beams for flexure, shear, bond and torsion
- **Module II (8 Classes)**
- Design of doubly reinforced beams, design of T and L beams, design of one way and two way slabs, design of staircases.
- **Module III (8 Classes)**
- Design of short and long columns with axial and eccentric loadings, Design of isolated and combined column footings
- **Module IV (8 Classes)**
- Retaining walls, various forces acting on retaining wall, stability requirement, design of cantilever and counterfort retaining walls,
- **Module V (6 Classes)**
- Design of water tanks, design requirements, design of tanks on ground, under ground and elevated water tanks.

BOOKS AND DIGITAL LEARNING SOURCE

Books:

1. Design of Reinforced Concrete Structue by N. Subramanian, Oxford University Press
2. Limit State Design by A.K.Jain, Neemchand& Bros
3. Reinforced Concrete Design by S U Pillai & D. Menon, McGraw Hill
4. Design of concrete structures by J.N.Bandyopadhyay, PHI
5. Limit State Design of Reinforced Concrete -P.C Verghese
6. Reinforced Concrete Design by S.N.Sinha, McGraw Hill
7. RCC Design-B.C.Punmia, A.K.Jain and A.K.Jain-Laxmi Publications

Digital Learning Resources:

Course Name	**Design of Reinforced Concrete Structures 12 weeks**
Course Link	(https://nptel.ac.in/courses/105/105/105105105/)
Course Instructor	PROF. NIRJHAR DHANG, ,IIT Kharagpur

E: Course Outcome

- Students who successfully complete this course will be able to
  - Identify and compute the main mechanical properties of concrete and steel.
  - Identify and calculate the design loads and distribution.
  - Apply the strength method to design R.C. structural members.
  - Analyze and design R.C. beams and slabs for flexure and shear.
  - Analyze and design short and slender R.C. columns and their footings.
  - Analyze and design retaining structures.

F: Programme Outcomes Addressed to the students

PO-1 Engineering Knowledge	**CO-1**
PO-2- Problem Analysis	CO 4,5&6,
PO-3- Design / development of Solution	CO 4,5 & 6,
PO-4- Conduct Analysis on complex analysis	CO-2, CO-3
PO-5 Modern Tool Usages	
PO-6 Engineer and Society	CO-3
PO-7- Environment and Sustainability	CO-1
PO-8- Ethics	
PO-9- Individual and Team work	2,3,4,5 & 6
PO-10-Communication	
PO-11- Project Mang. And Finance	
Po-12- Life Long Learning	CO-2

G. Gaps in the Syllabus (if Any) to meet Industry Requirement

Gap-1- Complete Design of a structure not included in the syllabus.
Gap-2-Not more emphasis is given on field calculations.

H: Topics beyond Syllabus/ Advanced Topics

Design of Lintels, Chhaza
A complete design methodology.

I: Assessment Methodologies:

S.N	Description	Type
1	Assignment	Direct
2	Internal Exam	Direct
3	University Exam	Direct
4	Tutorial	Direct
5	Presentation	Direct
	Student Feedback	Indirect
6	Employer feedback	Indirect

J: Course Plan:

S.N	Day	Chapter/ Module	Topics To be Covered	Topics Covered
01		I	Properties of concrete ,comp, tensile and other properties	
02		I	Objective of design , concept and methods of reinforced concrete design	
03		I	Introduction to limit state method, Limit state of collapse , Limit state of serviceability	
04		I	Analysis of Limit state method to rectangular beams for flexure, Determination of NA axis, MOR, Percentage of steel	
05			MOR of under reinforced, balance and over reinforced beam	
06		I	Application of Limit state method to rectangular beams for shear	
07		I	Shear design of beams	
08		I	Application of Limit state method to rectangular beams for bond, bond length calculation, Hooks	
09		I	Design of a complete rectangular Simply supported beam, effective length calculations, design parameters	
10		I	Application of Limit state method to rectangular beams for torsion.	

J: Course Plan:

S.N	Day	Chapter/ Module	Topics To be Covered	Topics Covered
11		II	Analysis of doubly reinforced beams	11
12		II	Design of doubly reinforced beams	12
13		II	Analysis of Flanged section.	13
14		II	Design of T beam, L beam	14
15		II	Analysis of one way slabs	15
16		II	Design of two way slab	16
17		II	Analysis of two way slabs	17
18		II	Calculation of loads of staircases.	18

J: Course Plan:

S.N	Day	Chapter/ Module	Topics To be Covered	Topics Covered
19		III	Analysis of short with axial loading,	19
20		III	Design of short with axial loading,	20
21		III	Analysis of short columns with eccentric loading,	21
22		III	Analysis of long columns	22
23		III	Analysis of long columns	23
24		III	Design of isolated column footing.	24
25		III	Problems on combined footing	25
26		III	Problems on combined footing	26

J: Course Plan:

S. N	Day	Chapter/ Module	Topics To be Covered	Topics Covered
27-28		IV	Retaining Walls and forces acting , Stability	27-28
29-30		IV	Design of Cantilever type retaining wall	29-30
31-32		IV	Design of Cantilever type retaining wall	31-32
33-34		IV	Design of Counterfort retaining wall	33-34
35-36		V	Design of ground water tank	35-36
37-38		V	Design of Ground water tank	37-38
39-40		V	Design of Elevated water tank	39-40

MODULE-1 CLASS -1

- Concrete highest consumed material.
- Italian architect Ponti once remarked that concrete liberated us from the rectangle
- Due to its flexibility in form and superiority in performance, it has replaced, to a large extent, the earlier materials like stone, timber and steel.
- Further, architect's scope and imaginations have widened to a great extent due to its mouldability and monolithicity. Thus, it has helped the architects and engineers to build several attractive shell forms and other curved structures. However, its role in several straight line structural forms like multistoried frames, bridges, foundations etc. is enormous.

Objectives of the Design of Reinforced Concrete Structures

- **The structures so designed should have an acceptable probability of performing satisfactorily during their intended life.**
- **The designed structure should sustain all loads and deform within limits for construction and use.**
- **The designed structures should be durable.**
- **The designed structures should adequately resist to the effects of misuse and fire.**

How to fulfil the objectives?

To fulfil objective it is necessary

- To understand the strength and deformation characteristics of the materials used in the design as also their deterioration under hostile exposure.

What are the materials : Concrete, Reinforcement

What is concrete

Why reinforcement

What is reinforcement

CLASS-1.1
Properties of concrete ,comp, tensile and other properties

G.C.Behera

Comp. Strength of Concrete

- The strength and deformation characteristics of concrete thus depend on the grade and type of cement, aggregates, admixtures, environmental conditions and curing. The increase of strength with its age during curing is considered to be marginal after 28 days. Blended cements (like fly ash cement) have slower rate of strength gain than ordinary Portland cement as recognized by code, Depending on several factors during its preparation, placement and curing, concrete has a wide range of compressive strength and the material is graded on the basis of its compressive strength on 28^{th} day also known as "characteristic strength" as defined below while discussing various strength and deformation properties.

Characteristic strength

- Characteristic strength is defined as the strength below which not more than five per cent of the test results are expected to fall. Concrete is graded on the basis of its characteristic compressive strength of 150 mm size cube at 28 days and expressed in N/mm^2.
- The grades are designated by one letter M (for mix) and a number from 10 to 80 indicating the characteristic compressive strength (f_{ck}) *in N/mm^2. As per IS 456 (Table 2), concrete has three groups as*
- *(i) ordinary concrete (M 10 to M 20),*
- *(ii) standard concrete (M 25 to M 55) and*
- *(iii) high strength concrete (M 60 to M 80).*

The size of specimen for determining characteristic strength may be different in different countries.

Procedure for Compressive strength

- IS 516-1959
- Temp-$(27\pm3)^0$ C
- Mixing by Machine power loaded
- Add all the mixing water into the drum before solid material.
- solid materials: the skip shall be loaded with about half of coarse aggregate, then with the fine aggregate, then with the cement and then with the remaining coarse aggregate on top.

  Mixing by drum hand operated
- materials in a similar manner, and the water shall be added immediately before the rotation of the drum is started.
- The period of mixing shall be not less than 2 minutes after all the materials are in the drum, and shall continue till the resulting concrete is uniform in appearance. When using
- pan mixers, the concrete shall be heaped together before sampling.

Hand Mixing --

The concrete batch shall be mixed on a watertight, non-absorbent platform with a shovel, trowel or similar suitable implement, using the following procedure:

- a) The cement and fine aggregate shall be mixed dry until the mixture

  is thoroughly blended and is uniform in colour,
- b) The coarse aggregate shall then be added and mixed with the cement and fine aggregate until the coarse aggregate is uniformly distributed throughout the batch, and
- c) The water shall then be added and the entire batch mixed until the concrete appears to be homogeneous and has the desired consistency. If repeated mixing is necessary, because of the addition of water in increments while adjusting the consistency, the batch shall be discarded and a fresh batch made without interrupting the mixing to make trial consistency tests.

- Test specimens cubical in shape shall be 15 X 15 X 15 *cm. If the largest nominal size of the aggregate does not* exceed 2 cn, 10 cm cubes may be used as an alternative.

  Cylindrical test

- specimens shall have a length equal to twice the diameter. They shall be 15 cm in diameter and 30 cm long.
- *Tamping Bar - The tamping bar shall be a steel bar 16 mm* diameter, 0·6 m long and bullet pointed at the lower end.

MOULD

- The mould shall be of metal, preferably steel or cast iron, and stout enough to prevent distortion. It shall be constructed in such a manner as to facilitate the removal of the moulded specimen without damage.
- The height of the mould and the distance between opposite faces shall be the specified size + 0·2 mm.
- The angle between adjacent internal faces and between internal faces and top and bottom planes of the mould shall be 90° +0-5°. The interior faces of the mould shall be plane surfaces with a permissible variation of 0·03 mm.
- *Cylinders -- The cylindrical mould shall be of metal which shall* be not less than 3 mm thick. Each mould shall be capable of being opened longitudinally to facilitate removal of the specimen and shall be provided with a means of keeping it closed while in use. The ends shall not depart from a plane surface, perpendicular to the axis of the mould, by more than 0·05 mm. When assembled ready for use, the mean internal diameter of the mould shall be 15·0 cm ±0·2 mm and in no direction shall the internal diameter be less than 14·95 cm or more than 15·05 cm. The height shall be 30-0 cm ±0-1 cm.

COMPACTION

- *Compacting by Hand – In uniform manner* For cubical specimens, in no case shall the concrete be subjected to less than 35 strokes per layer for 15 cm cubes or 25 strokes per layer for 10 cm cubes. For cylindrical specimens, the number of strokes shall not be less than thirty per layer. The strokes shall penetrate into the underlying layer and the bottom layer shall be rodded throughout its depth. Where voids arc left by the tamping bar, the sides of the mould shall be tapped to close the voids.

Compaction by vibration

- *When compacting by vibration, each* layer shall be vibrated by means of an electric or pneumatic hammer or vibrator or by means of a suitable vibrating table until the specified condition is attained.
- Capping Specimens - The ends of all cylindrical test specimens that are not plane within 0·05 mm shall be capped.

Curing -

- The test specimen- shall be stored in a place, free from vibration, in moist air of at least 90 percent relative humidity and at a temperature of 27° ± 2°e for 24 hours ± l hour from the time of addition of water to the dry ingredients.
- After this period, the specimens shall be marked and removed from the moulds and, unless required for test within 24 hours, immediately submerged in clean, fresh water or saturated lime solution and kept there until taken out just prior to test.
- The water or solution in which the specimens are submerged shall be renewed every seven days and shall be maintained at a temperature of 27° ± 2°C. The specimens shall not be alk.weo to become dry at any time until they have been tested.

TESTING

- The testing machine CTM OR UTM
- The permissible error shall be not greater than ± 2 percent of the maximum load.
- specimens, the most usual being 7 and 28 days. Ages of 13 weeks and one year are recommended if tests at greater ages are required. Where it may be necessary to obtain the early strengths. tests may be made at the ages of 24 hours ± 1/2 hour and 72 hours ± 2 hours.
- The ages shall be calculated from the time of the addition of water to the dry ingredients.
- Number of Specimen - At least three specimens, preferably from different batches, shall be made for testing at each selected age.

- In the case of cubes, the specimen shall be placed In the machine In such a manner that the load shall be applied to opposite sides of the cubes as cast, not to the top and bottom.
- The load shall be applied without shock and Increased continuously at a rate of approximately 140 kg/sqcm/min until the resistance of the specimen to the Increasing load breaks down and no greater load can be sustained.
- The measured compressive strength of the specimen shall be calculated by dividing the maximum load applied to the specimen during the test by the cross-sectional area.
- Average of three values shall be taken as the representative of the batch provided the Individual variation is not more than ± 15 percent of the average. Otherwise repeat tests.

(b) Other strengths of concrete

- In addition to its good compressive strength, concrete has flexural and splitting tensile strengths too. The flexural and splitting tensile strengths are obtained as described in IS 516 and IS 5816, respectively. However, the following expression gives an estimation of flexural strength (f_{cr}) *of concrete from its characteristic compressive strength (cl. 6.2.2)*

$$f_{cr} = 0.7\sqrt{f_{ck}} \quad {}^{N}/_{mm^2}$$

- The standard size shall be 15 x IS x 70 cm or Alternatively, if the largest nominal size of the aggregate docs not exceed 19 mm, specimens 10 x 10 x 5O cm may be used.
- *Tamping Bar - The tamping bar shall be a steel bar weighing* 2 kg, 40 cm long, and shall have a ramming face 25 mm square.

- The bed of the testing machine shall be provided with two steel rollers, 38 mm in diameter, on which the specimen is to be supported, and these
- rollers shall be so mounted that the distance from centre to centre is 60 cm for 15·0 cm specimens or 40 cm for 10·0 cm specimens, The load shall be applied through two similar roller", mounted at the third points of the supporting span, that is, spaced at 20 or 13·3 cm centre to centre.

The load shall be applied

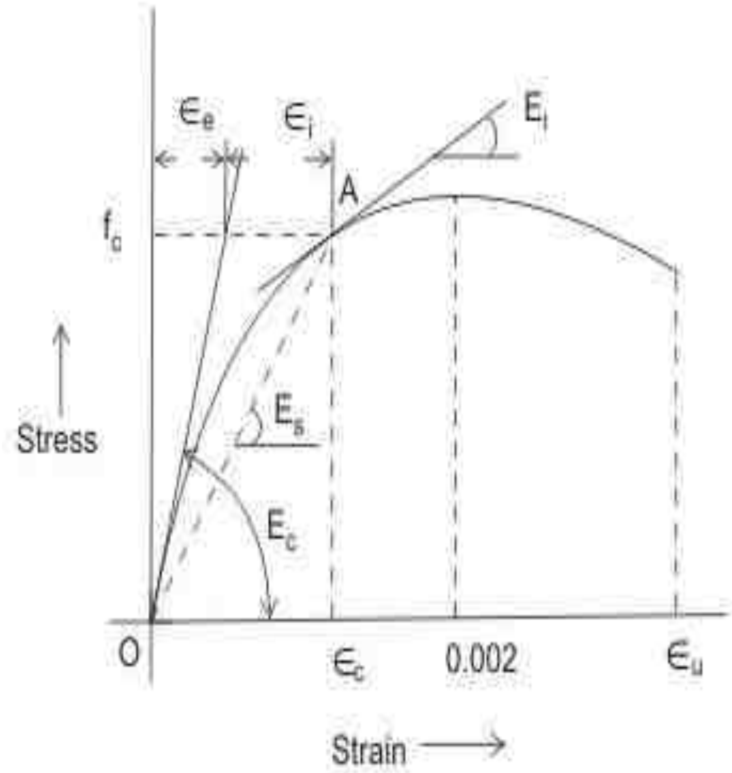
- without shock and increasing continuously at a rate such that the extreme fibre stress increases at approximately 7 kg/sq cn.rmm. that is, at a rate of loading of 400 kg/min for the 15·0 cm specimen- and at a rate of 180 kg/min for the 10·0 cm specimens.

- when a is greater than 20 cm for 15·0 cm specimen, or greater than 13.3 cm for a 10·0 cm specimen, $f_{cr} = \frac{PL}{bd^2}$
- when a is less than 20 cm but greater than17 cm for 15·0 cm specimen, or less than 13.3 cm but greater than for a 10·0 cm specimen

$$f_{cr} = \frac{3Pa}{bd^2}$$

Elastic Modulus

- E_c = *initial tangent modulus at the origin, also known as short term static modulus*
- E_s = *secant modulus at A*
- E_t = *tangent modulus at A*
- ε_e = *elastic strain at A*
- ε_i = *inelastic strain at A*



(d) Shrinkage of concrete

- Shrinkage is the time dependent deformation, generally compressive in nature. The constituents of concrete, size of the member and environmental conditions are the factors on which the total shrinkage of concrete depends. However, the total shrinkage of concrete is most influenced by the total amount of water present in the concrete at the time of mixing for a given humidity and temperature. The cement content, however, influences the total shrinkage of concrete to a lesser extent. The approximate value of the total shrinkage strain for design is taken as 0.0003 in the absence of test data.

CREEP

- Creep is another time dependent deformation of concrete by which it continues to deform, usually under compressive stress. The creep strains recover partly when the stresses are released. creep recovery in two parts. The elastic recovery is immediate and the creep recovery is slow in nature.

The creep of concrete is influenced by

- Properties of concrete
- Water/cement ratio
- Humidity and temperature of curing
- Humidity during the period of use
- Age of concrete at first loading
- Magnitude of stress and its duration
- Surface-volume ratio of the member

Workability and Durability of Concrete

- **Workability**
- It is the property which determines the ease and homogeneity with which concrete can be mixed, placed, compacted and finished. A workable concrete will not have any segregation or bleeding. Segregation causes large voids and hence concrete becomes less durable. Bleeding results in several small pores on the surface due to excess water coming up. Bleeding also makes concrete less durable. The degree of workability of concrete is classified from very low to very high with the corresponding value of slump in mm

Durability of concrete

- A durable concrete performs satisfactorily in the working environment during its anticipated exposure conditions during service. The durable concrete should have low permeability with adequate cement content, sufficient low free water/cement ratio and ensured complete compaction of concrete by adequate curing. For more information, please refer to cl. 8 of IS 456.

Design mix and nominal mix concrete

- In design mix, the proportions of cement, aggregates (sand and gravel), water and mineral admixtures, if any, are actually designed, while in nominal mix, the proportions are nominally adopted. The design mix concrete is preferred to the nominal mix as the former results in the grade of concrete having the specified workability and characteristic strength (vide cl. 9 of IS 456).

Batching

- Mass and volume are the two types of batching for measuring cement, sand, coarse aggregates, admixtures and water. Coarse aggregates may be gravel, grade stone chips or other man made aggregates. The quantities of cement, sand, coarse aggregates and solid admixtures shall be measured by mass. Liquid admixtures and water are measured either by volume or by mass (cl. 10 of IS 456).

Properties of Steel

- Steel is used as the reinforcing material in concrete to make it good in tension. Steel as such is good in tension as well as in compression. Unlike concrete, steel reinforcement rods are produced in steel plants. Moreover, the reinforcing bars or rods are commercially available in some specific diameters. Normally, steel bars up to 12 mm in diameter are designated as bars which can be coiled for transportation. Bars more than 12 mm in diameter are termed as rods and they are transported in standard lengths.
- Like concrete, steel also has several types or grades. The four types of steel used in concrete structures as specified in cl. 5.6 of IS 456 are given below:

(i) Mild steel and medium tensile steel bars conforming to IS 432 (Part 1)

(ii) High yield strength deformed (HYSD) steel bars conforming to IS 1786

(iii) Hard-drawn steel wire fabric conforming to IS 1566

(iv) Structural steel conforming to Grade A of IS 2062.

Mild steel bars had been progressively replaced by HYSD bars and subsequently TMT bars are promoted in our country. The implications of adopting different kinds of blended cement and reinforcing steel should be examined before adopting.

- Until the relevant Indian Standard specification for reinforcing steel are modified to include the concept of characteristic strength, the characteristic value shall be assumed as the minimum yield stress or 0.2% proof stress specified in the relevant Indian Standard specification. The characteristic strength of steel designated by symbol fy (N/mm^2)
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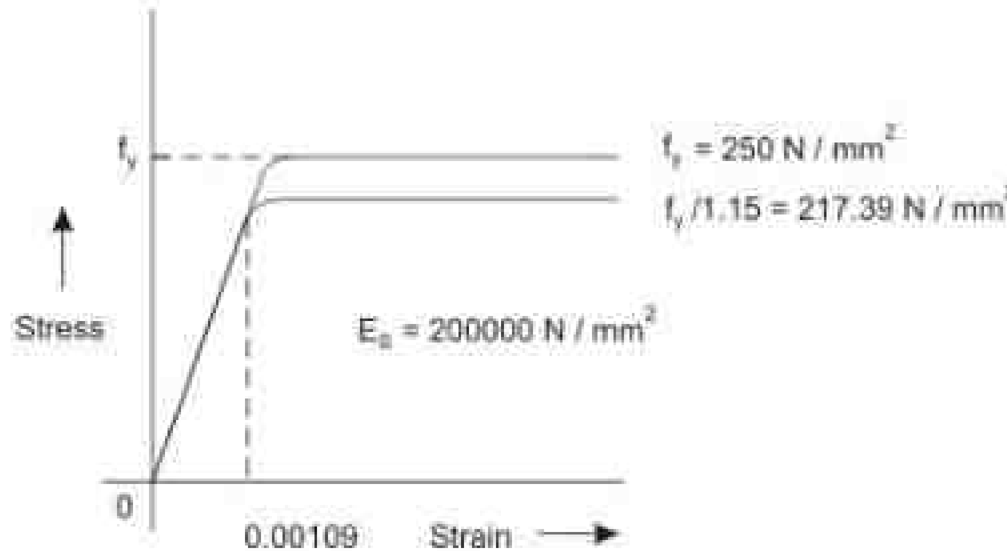
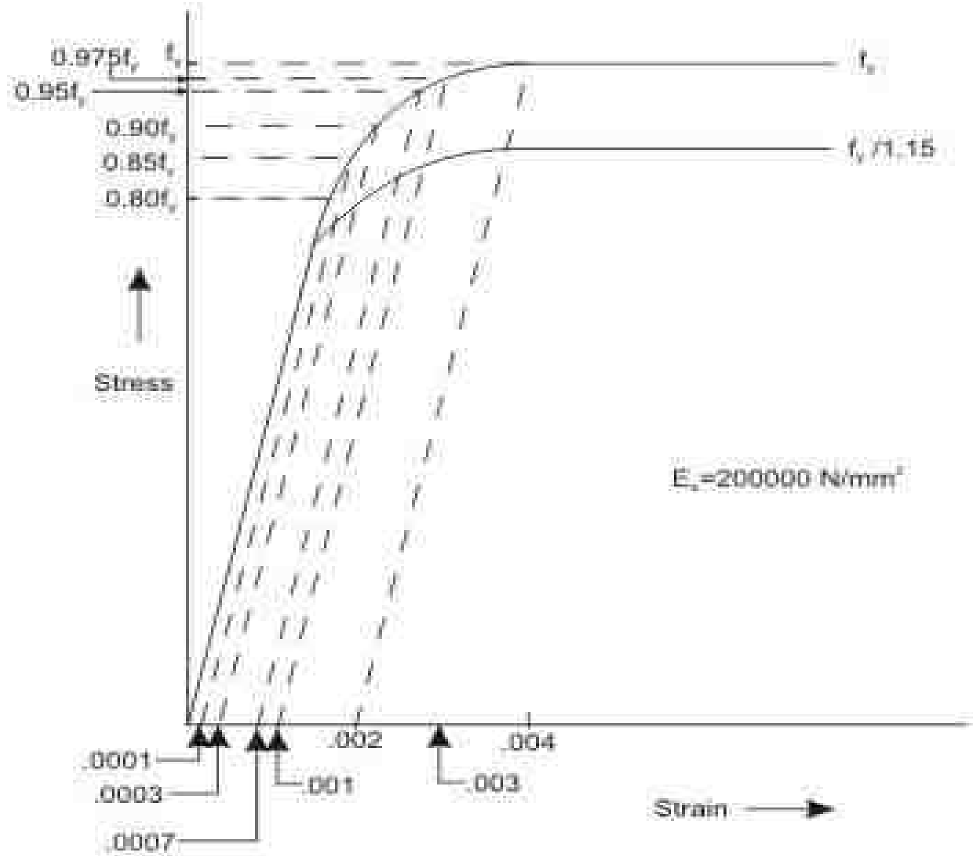


Stress - Strain curve for mild steel (idealised) (Fe 250) with definite yield point.



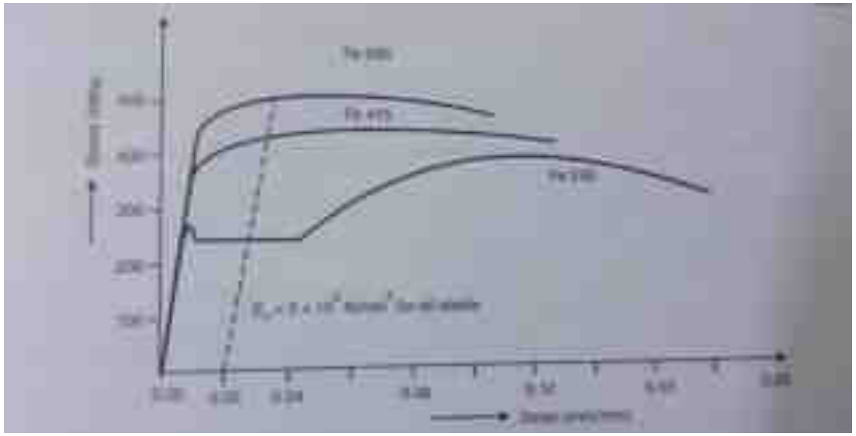
Stress-Strain Curve for cold worked deformed bar

- Figures show the representative stress-strain curves for steel having definite yield point and not having definite yield point, respectively. The characteristic yield strength f_y *of steel is assumed as the minimum yield stress or 0.2 per cent of proof stress for steel having no definite yield point. The modulus of elasticity of steel is taken to be 200000 N/mm^2.*
- For mild steel (Fig. Slide-26), the stress is proportional to the strain up to the yield point. Thereafter, post yield strain increases faster while the stress is assumed to remain at constant value of f_y.
- For cold-worked bars (Slide 27), the stress is proportional to the strain up to a stress of 0.8 f_y. *Thereafter, the inelastic curve is defined as given below:*

- For cold-worked bars (slide-27), the stress is proportional to the strain up to a stress of 0.8 f_y. *Thereafter, the inelastic curve is defined as given in fig:*
- Linear interpolation is to be done for intermediate values. The two grades of cold-worked bars used as steel reinforcement are Fe 415 and Fe 500 with the values of f^y *as 415 N/mm² and 500 N/mm², respectively.*

Stress	Inelastic strain
0.80 f_y	Nil
0.85 f_y	0.0001
0.90 f_y	0.0003
0.95 f_y	0.0007
0.975 f_y	0.0010
1.00 f_y	0.0020

MILD STEEL



DESIGN PRINCIPLE

DESIGN PRINCIPLE

CLASS- 2

Objective of design , concept and methods of reinforced concrete design

Objectives of the Design of Reinforced Concrete Structures

- **The structures so designed should have an acceptable probability of performing satisfactorily during their intended life.**
- **The designed structure should sustain all loads and deform within limits for construction and use.**
- **The designed structures should be durable.**
- **The designed structures should adequately resist to the effects of misuse and fire.**

How to fulfil the objectives?

To fulfil objective it is necessary

- To understand the strength and deformation characteristics of the materials used in the design as also their deterioration under hostile exposure.

What are the materials : Concrete, Reinforcement

What is concrete

Why reinforcement

What is reinforcement

In any method of design, the following are the common steps to be followed:

(i) To assess the dead loads and other external loads and forces likely to be applied on the structure,

(ii) To determine the design loads from different combinations of loads,

(iii) To estimate structural responses (bending moment, shear force, axial thrust etc.) due to the design loads,

(iv) To determine the cross-sectional areas of concrete sections and amounts of reinforcement needed.

Method of Design

- Three methods of design are accepted in cl. 18.2 of IS 456:2000 (Indian Standard Plain and Reinforced Concrete - Code of Practice, published by the Bureau of Indian Standards, New Delhi). They are as follows:
- LIMIT STATE METHOD
- WORKKING STATE METHOD
- METHOD BASED ON EXPERIMENTAL RESULT

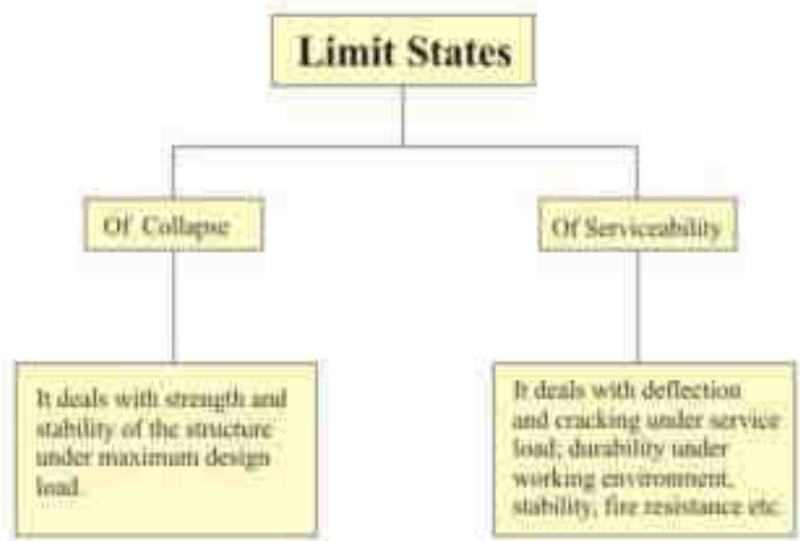
- Many of the above steps have lot of uncertainties. Estimation of loads and evaluation of material properties are to name a few. Hence, some suitable factors of safety should be taken into consideration depending on the degrees of such uncertainties.
- Limit state method is one of the three methods of design as per IS 456:2000. The code has put more emphasis on this method by presenting it in a full section (Section 5), while accommodating the working stress method in Annex B of the code (IS 456). Considering rapid development in concrete technology and simultaneous development in handling problems of uncertainties, the limit state method is a superior method where certain aspects of reality can be explained in a better manner.

LIMIT STATE

- Limit states are the acceptable limits for the safety and serviceability requirements of the structure before failure occurs. The design of structures by this method will thus ensure that they will not reach limit states and will not become unfit for the use for which they are intended. It is worth mentioning that structures will not just fail or collapse by violating (exceeding) the limit states. Failure, therefore, implies that clearly defined limit states of structural usefulness has been exceeded.
- Limit state of collapse was found / detailed in several countries in continent fifty years ago. In 1960 Soviet Code recognized three limit states: (i) deformation, (ii) cracking and (iii) collapse.

(i) Limit state of collapse deals with the strength and stability of structures subjected to the maximum design loads out of the possible combinations of several types of loads. Therefore, this limit state ensures that neither any part nor the whole structure should collapse or become unstable under any combination of expected overloads.

(ii) Limit state of serviceability deals with deflection and cracking of structures under service loads, durability under working environment during their anticipated exposure conditions during service, stability of structures as a whole, fire resistance etc.



The two main limit states

LIMIT STATE METHOD

- The term “Limit states” is of continental origin where there are two limit states - serviceability / collapse. For reasons not very clear, in English literature limit state of collapse is termed as limit state.
- limit state method of design has been found to be the best for the design of reinforced concrete members. More details of this method are explained in Module 3 (Lesson 4). However, because of its superiority to other two methods (see sections 2.3.2 and 2.3.3 of Lesson 3), IS 456:2000 has been thoroughly updated in its fourth revision in 2000 taking into consideration the rapid development in the field of concrete technology and incorporating important aspects like durability etc. This standard has put greater emphasis to limit state method of design by presenting it in a full section (section 5), while the working stress method has been given in Annex B of the same standard. Accordingly, structures or structural elements shall normally be designed by limit state method.

Design Loads

- The design loads are determined separately for the two methods of design as mentioned below after determining the combination of different loads.
- **In the limit state method, the design load is the characteristic load with appropriate partial safety factor (vide sec. 2.3.2.3 for partial safety factors).**
- **In the working stress method, the design load is the characteristic load only.**

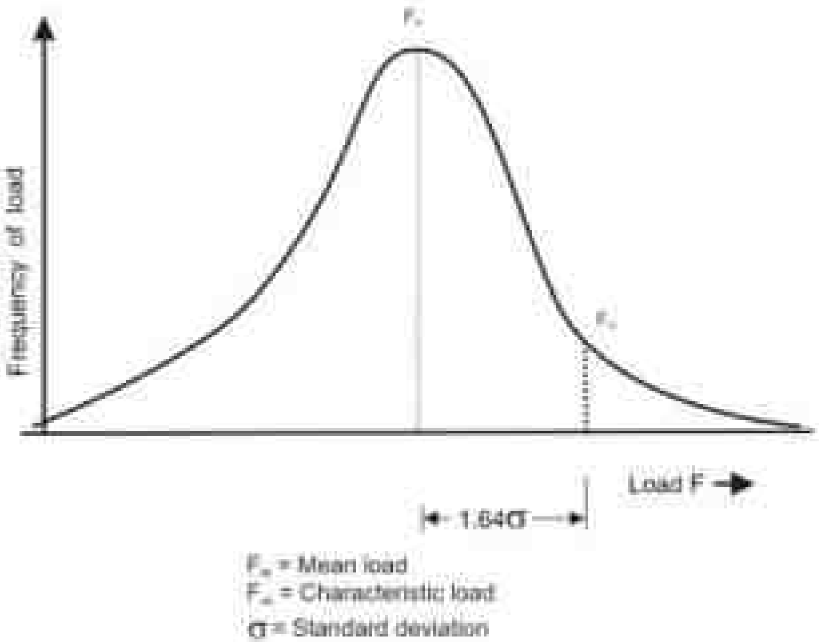
LOAD COMBINATION

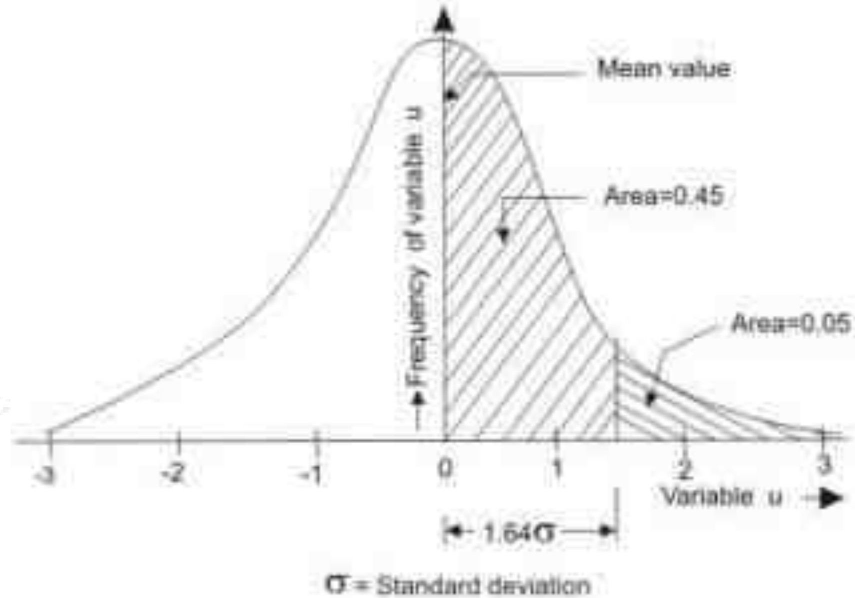
Load combinations	Limit state of collapse			Limit state of serviceability (for short term effects only)		
	DL	*IL*	*WL*	*DL*	*IL*	*WL*
DL + *IL*	1.5		1.0	1.0	1.0	-
DL + *WL*	1.5 or 0.9[1)]	-	1.5	1.0	-	1.0
DL + *IL* + *WL*	1.2			1.0	0.8	0.8

CHARACTERISTIC LOADS

- The characteristic values of loads are based on statistical data. It is assumed that in ninety-five per cent cases the characteristic loads will not be exceeded during the life of the structures.
- However, structures are subjected to overloading also. Hence, structures should be designed with loads obtained by multiplying the characteristic loads with suitable factors of safety depending on the nature of loads or their combinations, and the limit state being considered. These factors of safety for loads are termed as partial safety factors *(γ_f) for loads. Thus, the design loads are calculated as*
- *Design load= Characteristic Load X Partial Load factor*

CHARACTERISTIC LOADS





Characteristic load = Average/mean load + *K (σ=standard deviation for load)*

The value of *K is assumed such that the actual load does not exceed the characteristic load during the life of the structure in 95 per cent of the cases.*

In absence of any data, **loads** given in various standards shall be assumed as the **characteristic loads**.

DESIGN LOAD

Values of Partial load factors

Load Combination	**LIMIT State of Collapse**			**LIMIT State of Serviceability**		
	DL	LL	WL	DL	LL	WL
DL+LL	1.5	1.5		1.0	1.0	
DL+WL	1.5or 0.9[1]		1.5	1.0		1.0
DL+LL+WL	1.2	1.2	1.2	1.0	0.8	0.8

1 While considering earthquake effects, substitute *EL* for *WL*.

2 For the limit states of serviceability, the values of γ_f given in this table are applicable for short term effects. While assessing the long term effects due to creep the dead load and that part of the live load likely to be permanent may only be considered.

1) This value is to be considered when stability against overturning or stress reversal is critical.

Design load= Characteristic Load X Partial Load factor

DIFFERENT LOADS

- Loads
- The following are the different types of loads and forces acting on the structure. As mentioned earlier, their values have been assumed based on earlier data and experiences. It is worth mentioning that their assumed values as stipulated in IS 875 have been used successfully.
- **Dead loads**
- These are the self weight of the structure to be designed. Needless to mention that the dimensions of the cross section are to be assumed initially which enable to estimate the dead loads from the known unit weights of the materials of the structure. The accuracy of the estimation thus depends on the assumed values of the initial dimensions of the cross section. The values of unit weights of the materials are specified in Part 1 of IS 875.
- **Imposed loads**
- They are also known as live loads and consist of all loads other than the dead loads of the structure. The values of the imposed loads depend on the functional requirement of the structure. Residential buildings will have comparatively lower values of the imposed loads than those of school or office buildings. The standard values are stipulated in Part 2 of IS 875.
- **Wind loads**
- These loads depend on the velocity of the wind at the location of the structure, permeability of the structure, height of the structure etc. They may be horizontal or inclined forces depending on the angle of inclination of the roof for pitched roof structures. They can even be suction type of forces depending on the angle of inclination of the roof or geometry of the buildings. Wind loads are specified in Part 3 of IS 875.
-

DIFFERENT LOADS

- **Snow loads**
- These are important loads for structures located in areas having snow fall, which gets accumulated in different parts of the structure depending on projections, height, slope etc. of the structure . The standard values of snow loads are specified in Part 4 of IS 875.
- **Earthquake forces**
- Earthquake generates waves which move from the origin of its location (epicenter) with velocities depending on the intensity and magnitude of the earthquake. The impact of earthquake on structures depends on the stiffness of the structure, stiffness of the soil media, height and location of the structure etc.. Accordingly, the country has been divided into several zones depending on the magnitude of the earthquake. The earthquake forces are prescribed in IS 1893. Designers have adopted equivalent static load approach or spectral method.
- **Shrinkage, creep and temperature effects**
- Shrinkage, creep and temperature (high or low) may produce stresses and cause deformations like other loads and forces . Hence, these are also considered as loads which are time dependent. The safety and serviceability of structures are to be checked following the stipulations of cls. 6.2.4, 5 and 6 of IS 456:2000 and Part 5 of IS 875.
- **Other forces and effects**
- It is difficult to prepare an exhaustive list of loads, forces and effects coming onto the structures and affecting the safety and serviceability of them. However, IS 456:2000 stipulates the following forces and effects to be taken into account in case they are liable to affect materially the safety and serviceability of the structures. The relevant codes as mentioned therein are also indicated below:

CHARACTERISTIC STRENGTH

- Characteristic strength of a material as obtained from the statistical approach is the strength of that material below which not more than five per cent of the test results are expected to fall .However, such characteristic strengths may differ from sample to sample also. Accordingly, the design strength is calculated dividing the characteristic strength further by the partial safety factor for the material γ_m, *where* γ_m *depends on the material and the limit state being considered. Thus, K is 1.65 for 5% probability*

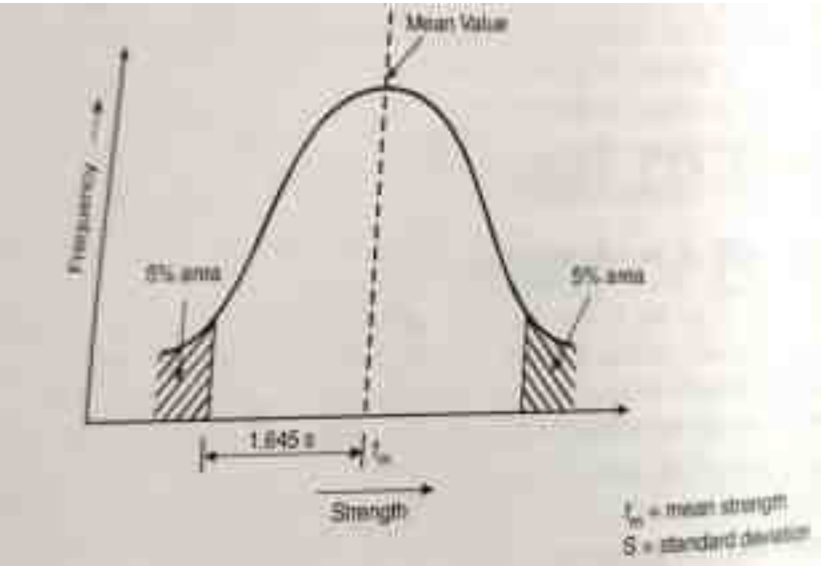
  Characteristic strength= Average/mean strength -*K (standard deviation for strength)*

- Standard Deviation= $$S_d = \sqrt{\frac{\sum \delta^2}{n-1}}$$

  Where, δ=deviation of the individual test strength from the average or mean strength of n samples.
  n= number of test results.
  IS 456:2000 has recommended minimum value of n=30.

Design strength of material= Characteristic strength /Partial safety factor for material

CHARACTERISTIC STRENGTH



Characteristic Strength= mean strength- k.S=f_m-kS
S=standard deviation, k=1.65 for 5% probability

Characteristic strengths may differ from sample to sample also. Accordingly, the design strength is calculated dividing the characteristic strength further by the partial safety factor for the material.

Clause 36.4.2 of IS 456 states that γ_m *for concrete and steel should be taken as 1.5 and 1.15, respectively when assessing the strength of the structures or structural members employing limit state of collapse.*

WHY LIMIT STATE

- Concept of separate partial safety factors of loads of different combinations in the two limit state methods.

- (ii) Concept of separate partial safety factors of materials depending on their quality control during preparation. Thus, γ^m *for concrete is 1.5 and the same for steel is 1.15. This is more logical than one arbitrary value in the name of safety factor.*

- (iii) A structure designed by employing limit state method of collapse and checked for other limit states will ensure the strength and stability requirements at the collapse under the design loads and also deflection and cracking at the limit state of serviceability. This will help to achieve the structure with acceptable probabilities that the structure will not become unfit for the use for which it is intended.
- (iv) The stress block represents in a more realistic manner when the structure is at the collapsing stage (limit state of collapse) subjected to design loads.

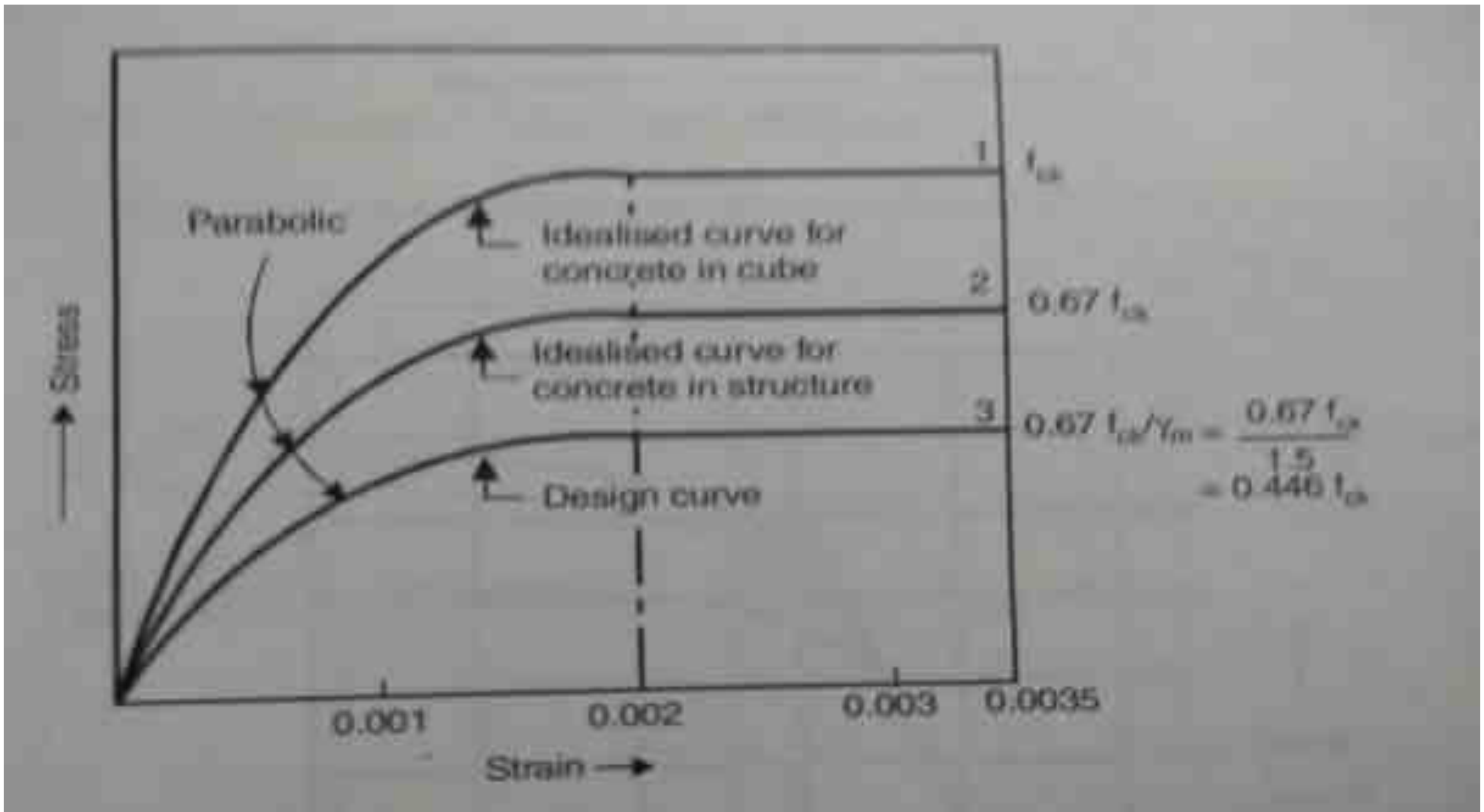
Working stress method

- This method of design, considered as the method of earlier times, has several limitations. However, in situations where limit state method cannot be conveniently applied, working stress method can be employed as an alternative. It is expected that in the near future the working stress method will be completely replaced by the limit state method. Presently, this method is put in Annex B of IS 456:2000.

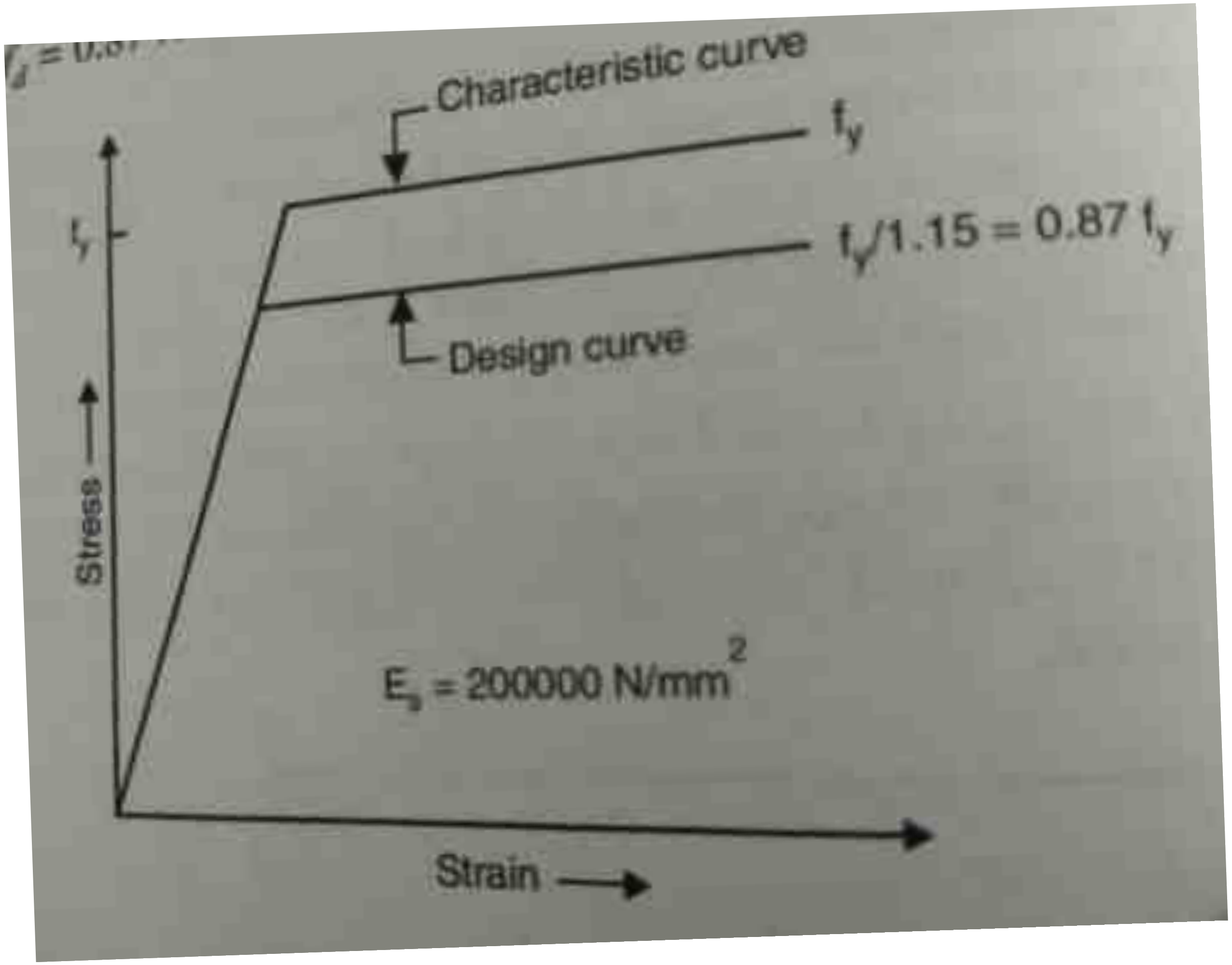
METHOD BASED ON EXPERIMENTAL APPROACH

- The designer may perform experimental investigations on models or full size structures or elements and accordingly design the structures or elements and also it should satisfy design objectives. Moreover, the engineer-in-charge has to approve the experimental details and the analysis connected therewith.
- Though the choice of the method of design is still left to the designer as per cl. 18.2 of IS 456:2000, the superiority of the limit state method is evident from the emphasis given to this method by presenting it in a full section (Section 5), while accommodating the working stress method in Annex B of IS 456:2000, from its earlier place of section 6 in IS 456:1978. It is expected that a gradual change over to the limit state method of design will take place in the near future after overcoming the inconveniences of adopting this method in some situations.

STRESS STRAIN DIGRAM FOR CONCRETE

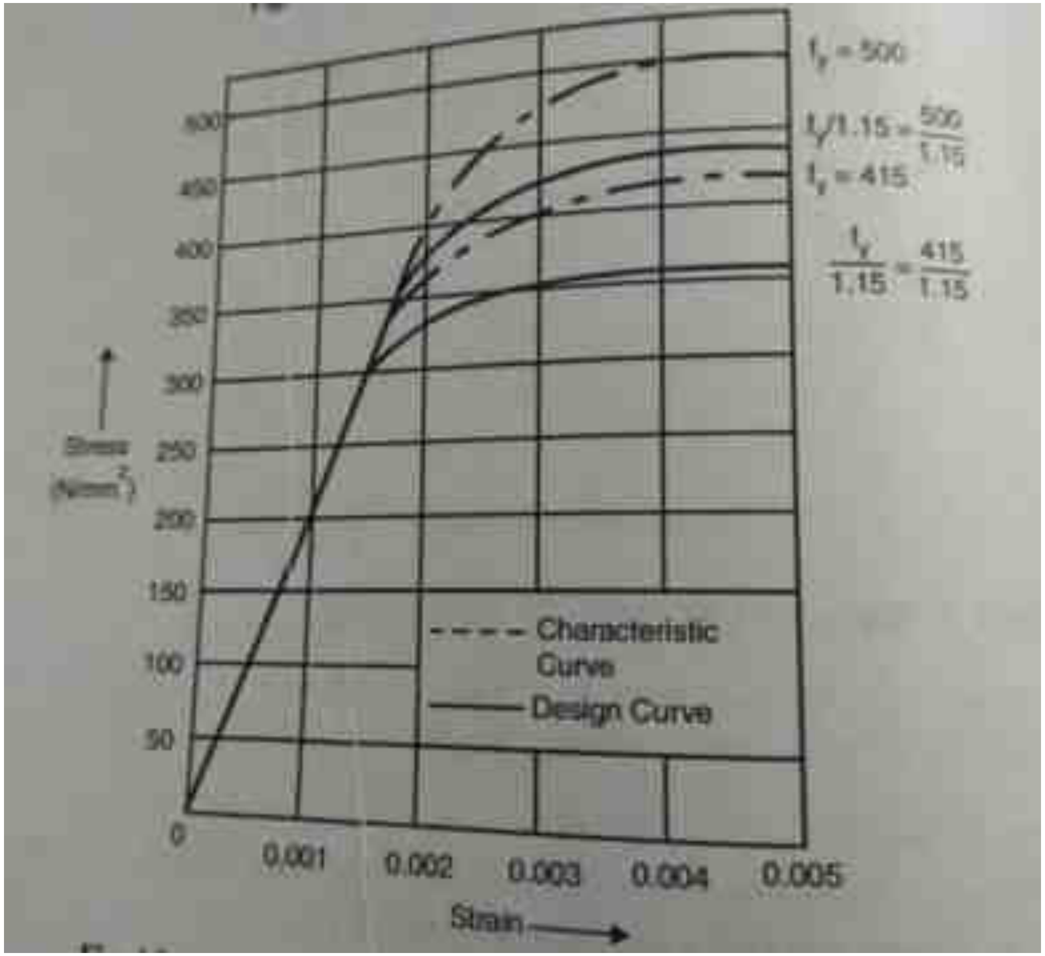


MILD STEEL



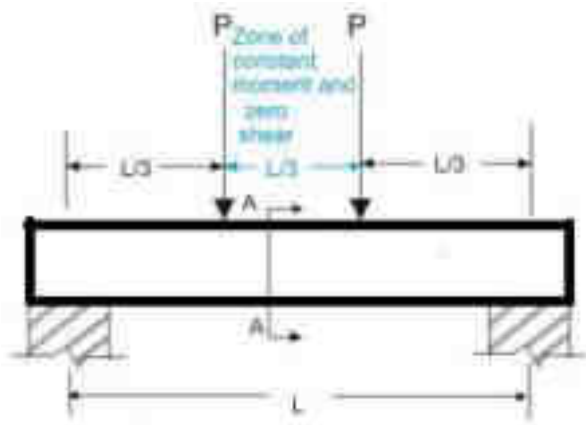
Characteristic curve
f_y
$f_y/1.15 = 0.87\ f_y$
Design curve
f_y
Stress
$E_s = 200000\ N/mm^2$
Strain

HYSD BARS

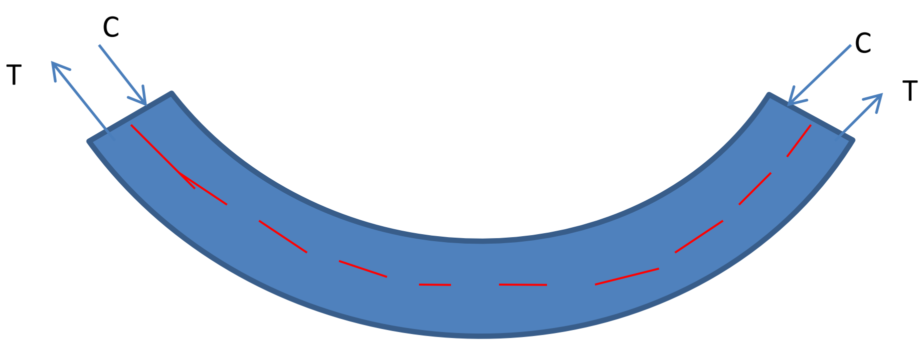


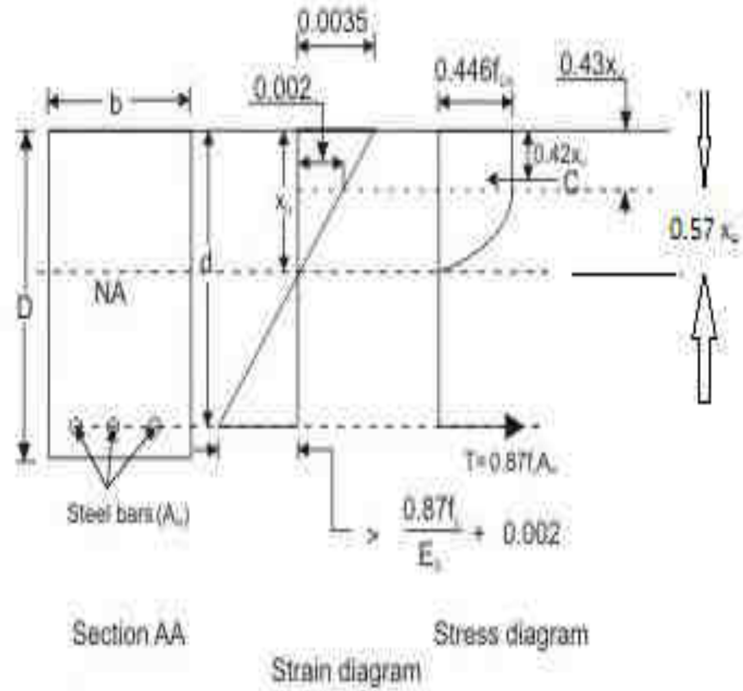
$f_y = 500$
$f_y/1.15 = \frac{500}{1.15}$
$f_y = 415$
$\frac{f_y}{1.15} = \frac{415}{1.15}$
Stress (N/mm²)
500
450
400
350
300
250
200
150
100
50
0
0.001
0.002
0.003
0.004
0.005
Strain
Characteristic Curve
Design Curve

FLEXURAL STRENGTH OF BEAMS



Rectangular Beam under Flexure





Rectangular beam under flexure when $x_u < x_{u,max}$

ASSUMPTIONS

The following are the assumptions of the design of flexural members employing limit state of collapse:

(i) Plane sections normal to the axis remain plane after bending.

This assumption ensures that the cross-section of the member does not warp due to the loads applied. It further means that the strain at any point on the cross-section is directly proportional to its distance from the neutral axis.

(ii) The maximum strain in concrete at the outer most compression fibre is taken as 0.0035 in bending .

This is a clearly defined limiting strain of concrete in bending compression beyond which the concrete will be taken as reaching the state of collapse. It is very clear that the specified limiting strain of 0.0035 does not depend on the strength of concrete.

(iii) The acceptable stress-strain curve of concrete is assumed to be parabolic .

The maximum compressive stress-strain curve in the structure is obtained by reducing the values of the top parabolic curve (Figs. 21 of IS 456:2000) in two stages. First, dividing by 1.5 due to size effect and secondly, again dividing by 1.5 considering the partial safety factor of the material. The middle and bottom curves (Fig. 21 of IS 456:2000) represent these stages. Thus, the maximum compressive stress in bending is limited to the constant value of 0.446 f_{ck} *for the strain ranging from 0.002 to 0.0035 (Figs. 21 and 22 of IS 456:2000).*

ASSUMPTIONS

(iv) The tensile strength of concrete is ignored.

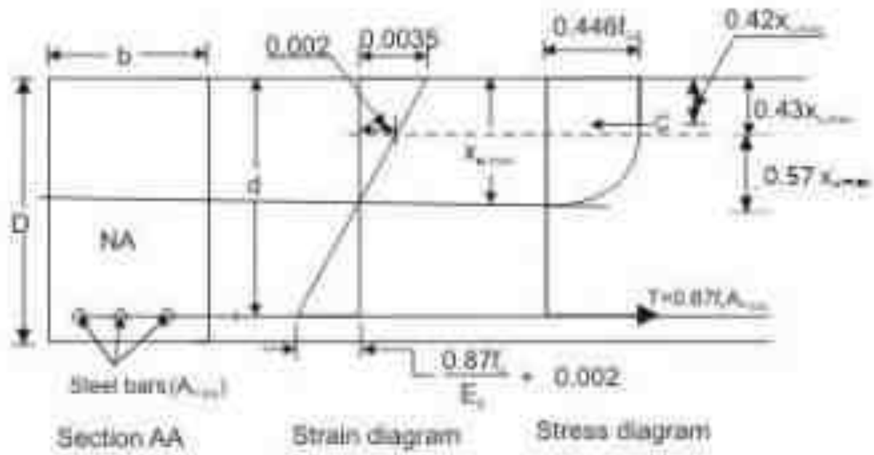
Concrete has some tensile strength (very small but not zero). Yet, this tensile strength is ignored and the steel reinforcement is assumed to resist the tensile stress. However, the tensile strength of concrete is taken into account to check the deflection and crack widths in the limit state of serviceability.

(v) The design stresses of the reinforcement are derived from the representative stress-strain curves as shown in Figs. 23A and B of IS 456:2000, for the type of steel used using the partial safety factor γ_m *as 1.15.*

In the reinforced concrete structures, two types of steel are used: one with definite yield point (Figs. 23B of IS 456:2000) and the other where the yield points are not definite (cold work deformed bars). The representative stress-strain diagram (Fig. 23A of IS 456:2000) defines the points between 0.8 f_y *and 1.0* f_y *in case of cold work deformed bars where the curve is inelastic.*

(vi) The maximum strain in the tension reinforcement in the section at failure shall not be less than $f_y/(1.15\ E_s) + 0.002$,

This assumption ensures ductile failure in which the tensile reinforcement undergoes a certain degree of inelastic deformation before concrete fails in compression.



Rectangular beam under flexure when $x_u = x_{u,max}$

BEAM SECTIONS

A_{st} = area of tension steel

b = width of the beam

C = total compressive force of concrete

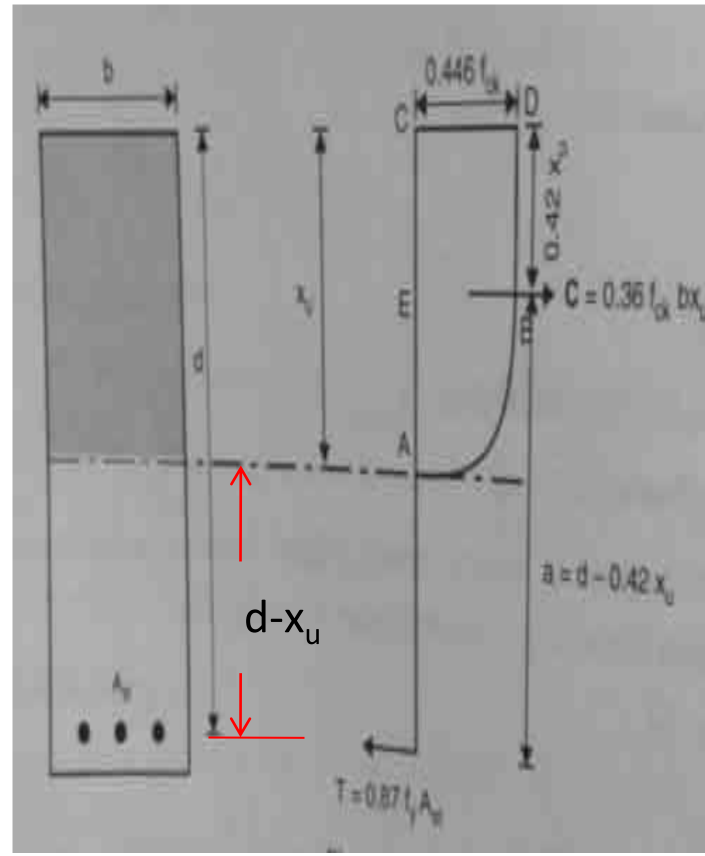
D=Overall depth

d = effective depth of the beam

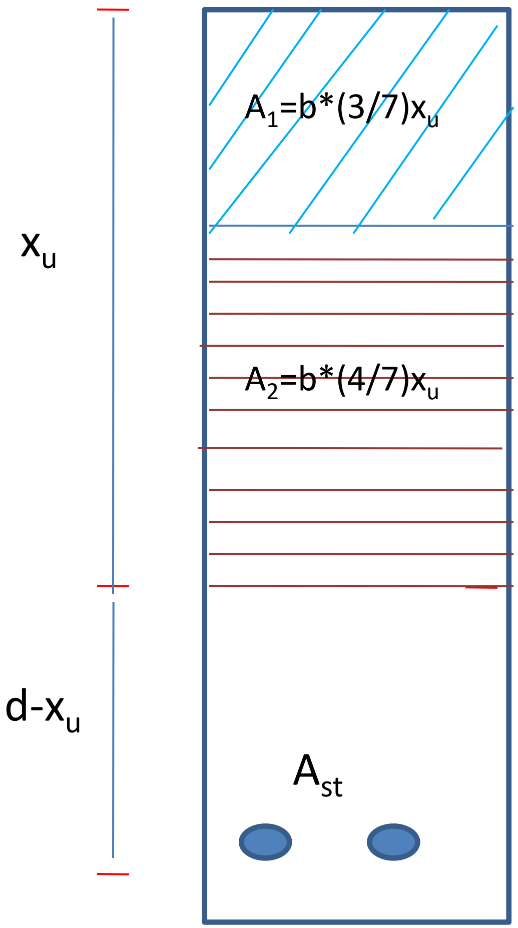
L = centre to centre distance between supports

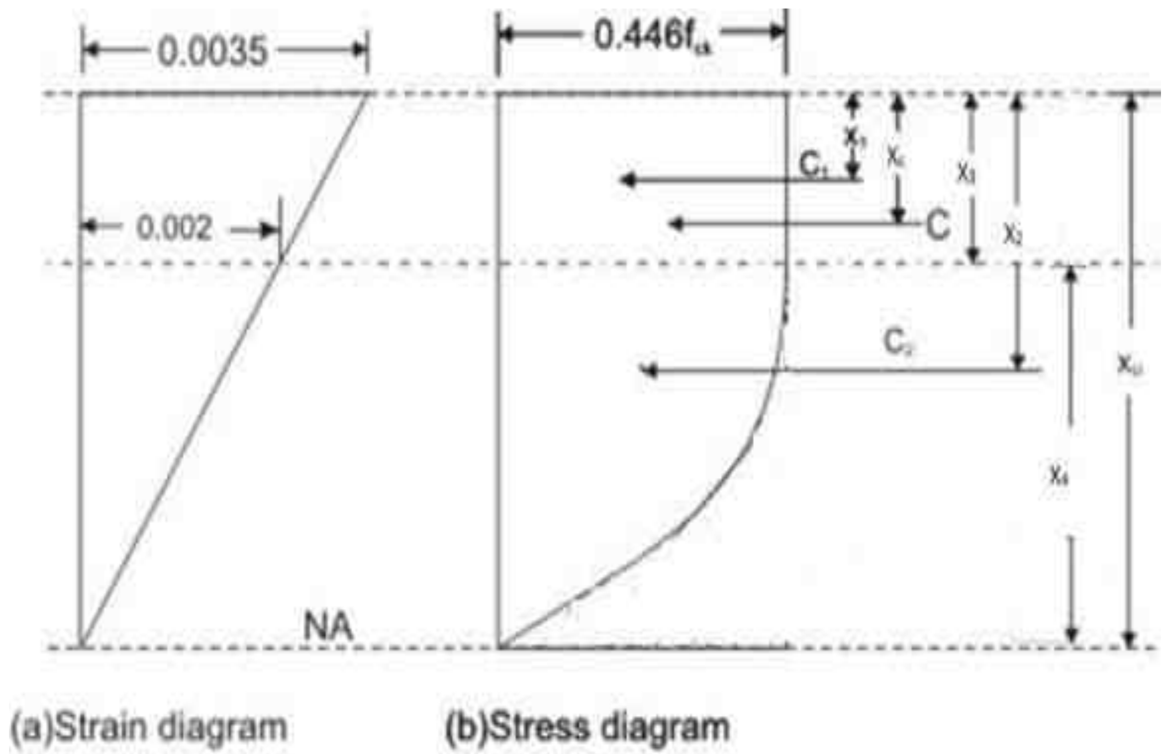
T = total tensile force of steel

x_u = depth of neutral axis from the most compressed fibre



SECTION ABOVE NEUTRAL AXIS





Stress Strain Diagram above the Neutral Axis

EQULIBRIUM EQUATIONS

The cross-sections of the beam under the applied loads as shown in Figure has three types of combinations of shear forces and bending moments:

(i) only shear force is there at the support and bending moment is zero,

(ii) both bending moment (increasing gradually) and shear force (constant = *P) are there between the support and the loading point and*

(iii) a constant moment (= PL/3) is there in the middle third zone i.e. between the two loads where the shear force is zero.

Since the beam is in static equilibrium, any cross-section of the beam is also in static equilibrium. Considering the cross-section in the middle zone ,

The three equations of equilibrium are the following

(i) Equilibrium of horizontal forces: $\Sigma H = 0$ *gives* $T = C$

(ii) Equilibrium of vertical shear forces: $\Sigma V = 0$

This equation gives an identity 0 = 0 as there is no shear in the middle third zone of the beam.

(iii) Equilibrium of moments: $\Sigma M = 0$,

This equation shows that the applied moment at the section is fully resisted by moment of the resisting couple $T a = C a$ *, where a is the operating lever arm between T and C (Figs. 3.4.19 and 20).*

CALCULATION OF FORCES

- *C = Total compressive force of concrete = $C_1 + C_2$*
- *C_1 = Compressive force of concrete due to the constant stress of 0.446 f_{ck} and up to a depth of x_3 from the top fibre*
- *C_2 = Compressive force of concrete due to the convex parabolic stress block of values ranging from zero at the neutral axis to 0.446 f_{ck} at a distance of x_4*
- *x_1 = Distance of the line of action of C_1 from the most compressed fibre*
- *x_2 = Distance of the line of action of C_2 from the most compressed fibre*
- *x_3 = Distance of the fibre from the most compressed fibre to where the strain = 0.002 and stress = 0.446 f_{ck}*
- *x_4 = Distance from N.A to the point where strain is 0.002 in compression zone*
- *x_u = Distance of the neutral axis from the most compressed fibre .*

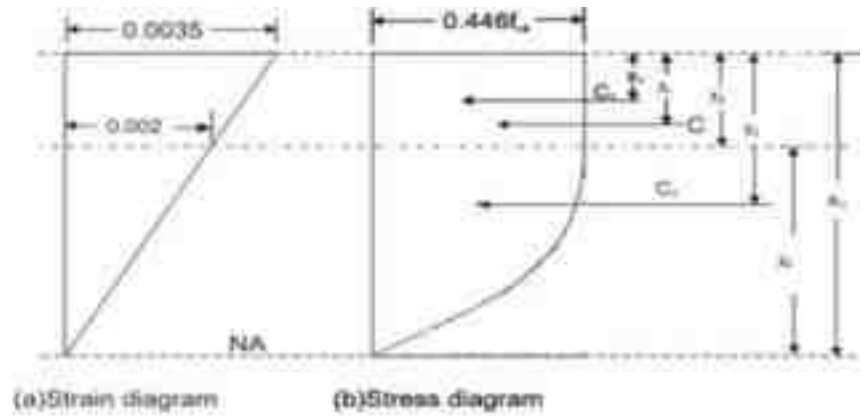
CALCULATION OF COMPRESSIVE FORCE

- $x_3+x_4=x_u$

$$\frac{x_4}{x_u} = \frac{0.002}{0.0035} = \frac{4}{7}$$

- $x_4=(4/7)x_u$

  $x_3=(3/7)x_u$



Stress Strain Diagram above the Neutral Axis

$$C_1 = 0.446 f_{ck} * b * x_3$$

$$C_1 = 0.446 f_{ck} * b * (\frac{3}{7}) x_u$$

$$C_1 = 0.191 f_{ck} * b * x_u$$

$$x_1 = 0.5\, x_3 = 0.5 * \left(\frac{3}{7}\right) x_u = \frac{3}{14} x_u$$

$$C_2 = \frac{2}{3} * 0.446 f_{ck} * b * x_4$$

$$C_2 = \frac{2}{3} * 0.446 f_{ck} * b * \frac{4}{7} x_u$$

$$C_2 = \frac{2}{3} * 0.446 f_{ck} * b * \frac{4}{7} x_u$$

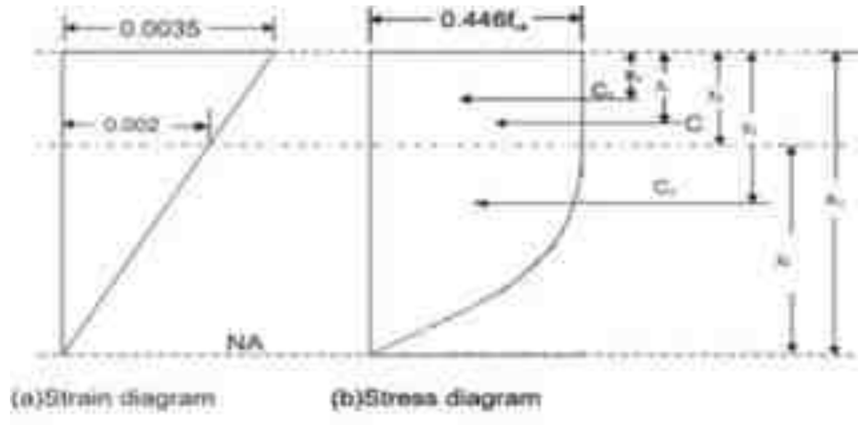
$$C_2 = 0.169 f_{ck} * b * x_u$$

CALCULATION OF x_c

$$C = C_1 + C_2 = 0.36 f_{ck} b x_u$$

$$x_2 = x_3 + \frac{3}{8}\,x_4 = \left(\frac{3}{7}\right) x_u + \frac{3}{8}\frac{4}{7} x_u = 0.642\, x_u$$

$$x_c = \frac{C_1 x_1 + C_2 x_2}{C_1 + C_2 = C} = 0.415 x_u = 0.42 x_u$$



Stress Strain Diagram above the Neutral Axis

Equating Sum of All horizontal forces equal to zero

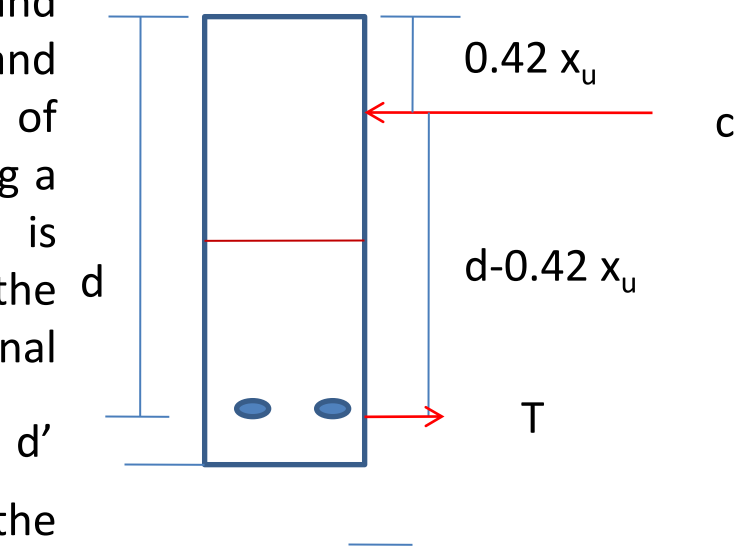
C=T

$$C = 0.36 f_{ck} * b * x_u = T = 0.87 * f_y * A_{st}$$

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

CALCULATION OF MOR

As on a particular section Compressive force C and Tension T are created and both are equal in magnitude and opposite in diction and their line of action not in same point , it is creating a couple . This couple or moment is Moment of Resistance(MOR) of the section which resists the external moment.



If this MOR is greater or equal to the external moment, the section is safe

MOR=(C or T) * Lever arm

Lever arm= Distance between C and T=z

z-=d-0.42x_u

$$MOR = (C = T) * z = C * z = 0.36 f_{ck} b x_u * (d - 0.42 x_u)$$

$$MOR = (C = T) * z = T * z = 0.87 f_{yk} * A_{st} * (d - 0.42 x_u)$$

OVER ,BALANCE AND UNDER REINFORCED SECTIONS

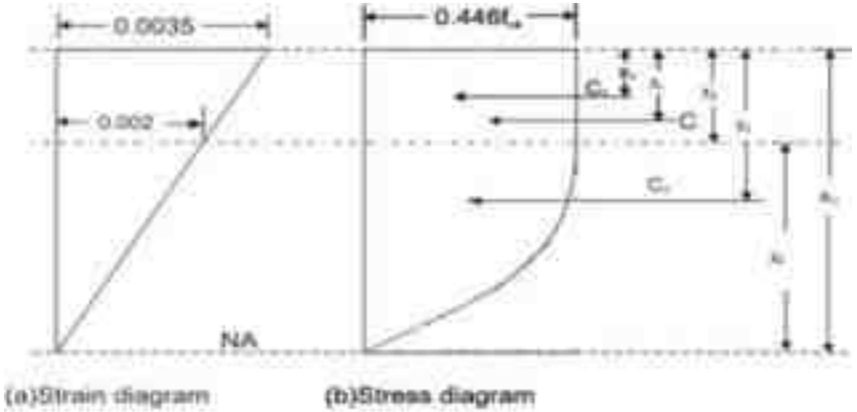
G.C. BEHERA

CALCULATION OF x_c

$$C = C_1 + C_2 = 0.36 f_{ck} b x_u$$

$$x_2 = x_3 + \frac{3}{8} x_4 = \left(\frac{3}{7}\right) x_u + \frac{3}{8}\frac{4}{7} x_u = 0.642\, x_u$$

$$x_c = \frac{C_1 x_1 + C_2 x_2}{C_1 + C_2 = C} = 0.415 x_u = 0.42 x_u$$



Stress Strain Diagram above the Neutral Axis

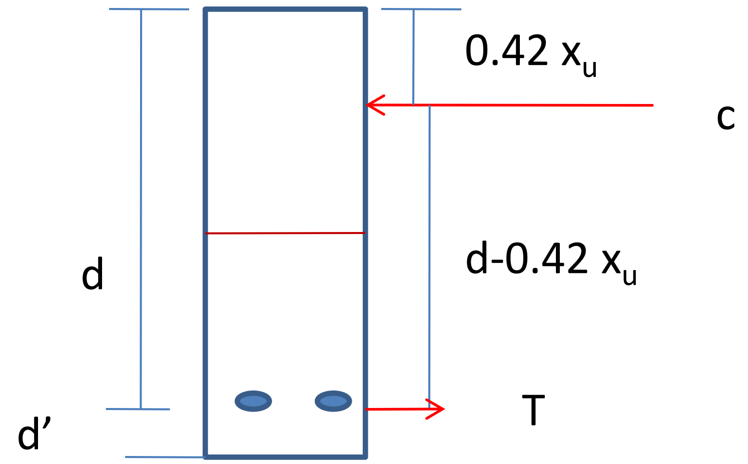
Equating Sum of All horizontal forces equal to zero
C=T

$$C = 0.36 f_{ck} * b * x_u = T = 0.87 * f_y * A_{st}$$

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

CALCULATION OF MOR AND PERCENTAGE OF STEEL

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



$$MOR = (C = T) * z = C * z = 0.36 f_{ck} b x_u * (d - 0.42 x_u)$$

$$MOR = 0.36 f_{ck} bd\, x_u (1 - 0.42 \frac{x_u}{d})$$

$$MOR = 0.36 f_{ck} b (x_u/d) * d * d[1 - 0.42 * (x_u/d)]$$

$$MOR = 0.36 f_{ck} (x_u/d) * [1 - 0.42 * (x_u/d)] bd^2$$

$$MOR = (C = T) * z = T * z = 0.87 f_y * A_{st} * (d - 0.42 x_u)$$

PECENTAGE OF STEEL

$$MOR = T * z = 0.87 f_y * A_{st} * (d - 0.42 x_u)$$

$$= 0.87 f_y * A_{st} * \left(d - 0.42 * \frac{0.87 * f_y * A_{st}}{0.36 f_{ck} * b} \right)$$

$$= 0.87 f_y * A_{st} * \left(d - \frac{1.015 * f_y * A_{st}}{f_{ck} * b} \right)$$

$$= 0.87 f_y * A_{st} * \left(d - \frac{f_y * A_{st}}{f_{ck} * b} \right)$$

$$= 0.87 f_y * A_{st} * d \left(1 - \frac{f_y * A_{st}}{f_{ck} * b * d} \right)$$

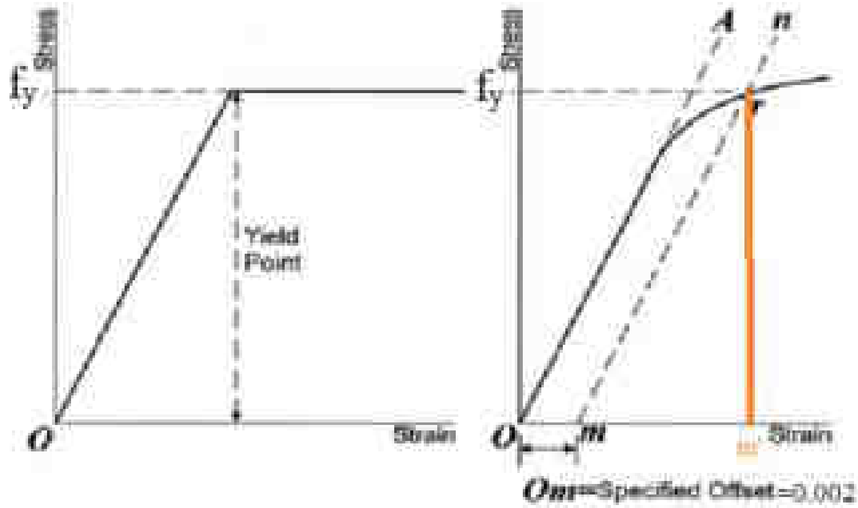
$$C = 0.36 f_{ck} * b * x_u = T = 0.87 * f_y * A_{st}$$

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

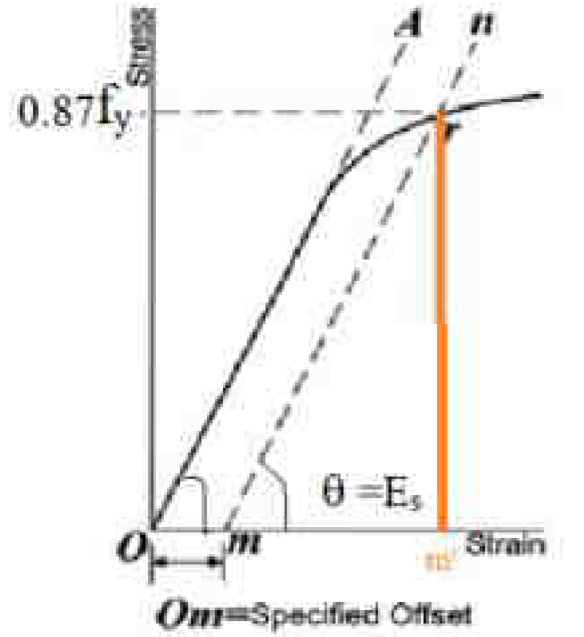
$$\frac{A_{st}}{b * d} * 100 = \frac{0.36 f_{ck} x_u}{0.87 f_y \, d} * 100$$

$$p_t = \frac{0.36 f_{ck} x_u}{0.87 f_y \, d} * 100$$

ACTUAL STRESS STRAIN AND DESIGN STRESS STRAIN CURVE



(a) Yield Point Method (b) Offset Method



(b) Offset Method

ACTUAL STRESS STRAIN

$$tan\theta = E_s = \frac{rm'}{mm'} = \frac{0.87f_y}{mm'}$$

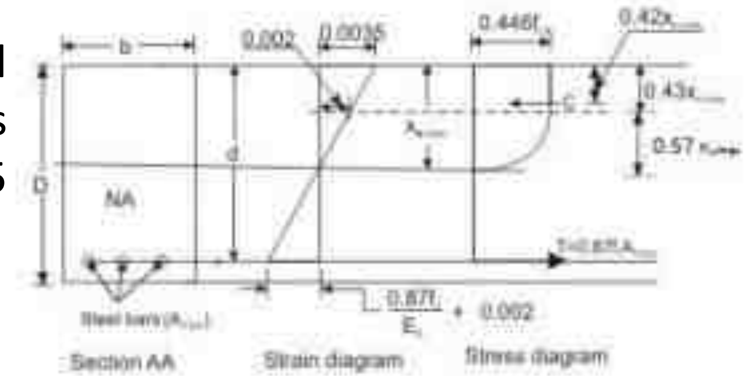
$$mm' = \frac{0.87f_y}{E_s}$$

$$Minimum\ \ strain\ at\ yielding = om' = om + mm'$$

$$= 0.002 + \frac{0.87 * f_y}{E_s}$$

BALANCE SECTION

When maximum stresses in concrete and steel reach simultaneously, the section is known as balance section. The strain in concrete is 0.0035 while strain in steel is $(0.87f_y/E_s)+0.002$
Taking $E_s=2.0*10^5\ N/mm^2$



Rectangular beam under flexure when $x_u=x_{u,max}$

$$\frac{x_{umax}}{d-x_{umax}}=\frac{0.0035}{\frac{0.87*f_y}{E_s}+0.002}$$

$$\frac{x_{umax}}{d-x_{umax}+x_{umax}}=\frac{0.0035}{\frac{0.87*f_y}{E_s}+0.002+0.0035}$$

$$\frac{x_{umax}}{d}=\frac{0.0035}{\frac{0.87*f_y}{E_s}+0.0055}$$

$$\frac{x_{umax}}{d}=0.53\ for\ Fe250$$

$$\frac{x_{umax}}{d}=0.48\ \ for\ Fe415$$

$$\frac{x_{umax}}{d}=0.46\ \ for\ Fe500$$

X_{umax} also known as x_{ulimit}

Independent of grade of concrete

BALANCE SECTION (MOR)

- For a balance section, x_u actual Neutral axis is equal to critical neutral axis x_{umax} or x_{ulimit}.

$$MOR = 0.36 f_{ck}(x_u/d) * [1 - 0.42 * (x_u/d)]bd^2$$

- $$MOR = M_{ulim} = 0.36 f_{ck}(x_{umax}/d) * [1 - 0.42 * (x_{umax}/d)]bd^2$$

$$p_t = \frac{0.36 f_{ck} x_u}{0.87 f_y\, d} * 100 \qquad p_{tlim} = \frac{0.36 f_{ck}}{0.87 f_y}\left(\frac{x_{ulim}}{d}\right) * 100$$

$$MOR = M_{ulim} = 0.148 * f_{ck} * bd^2 \; for\; Fe250$$

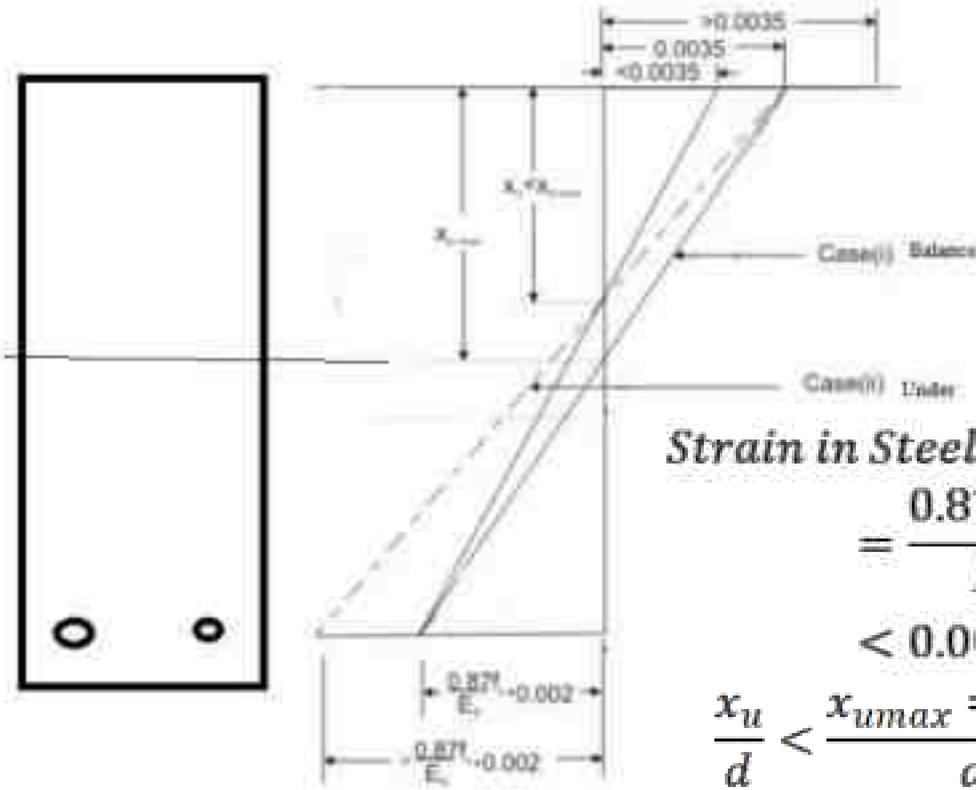
$$MOR = M_{ulim} = 0.138 * f_{ck} * bd^2 \; for\; Fe415$$

$$MOR = M_{ulim} = 0.133 * f_{ck} * bd^2 \; for\; Fe500$$

BALANCE SECTION (MOR)

f_{ck}	Values of p_{tlim}			Values of M_{ulim}		
	Fe250	Fe415	Fe500	Fe250	Fe415	Fe500
20	1.76	0.96	0.75	2.96 bd^2	2.76 bd^2	2.66bd^2
25	2.20	1.20	0.94	3.70 bd^2	3.45 bd^2	3.33 bd^2
30	2.64	1.43	1.13	4.44 bd^2	4.14 bd^2	3.99 bd^2
35	3.08	1.67	1.32	5.21 bd^2	4.83 bd^2	4.65 bd^2
40	3.52	1.91	1.51	5.92 bd^2	5.52 bd^2	5.32 bd^2

UNDER REINFORCED SECTION



$$Strain\ in\ Steel\ reaches\ its\ yield\ value\ first = \frac{0.87 * f_y}{E_s} + 0.002, but\ strain\ in\ concrete < 0.0035$$

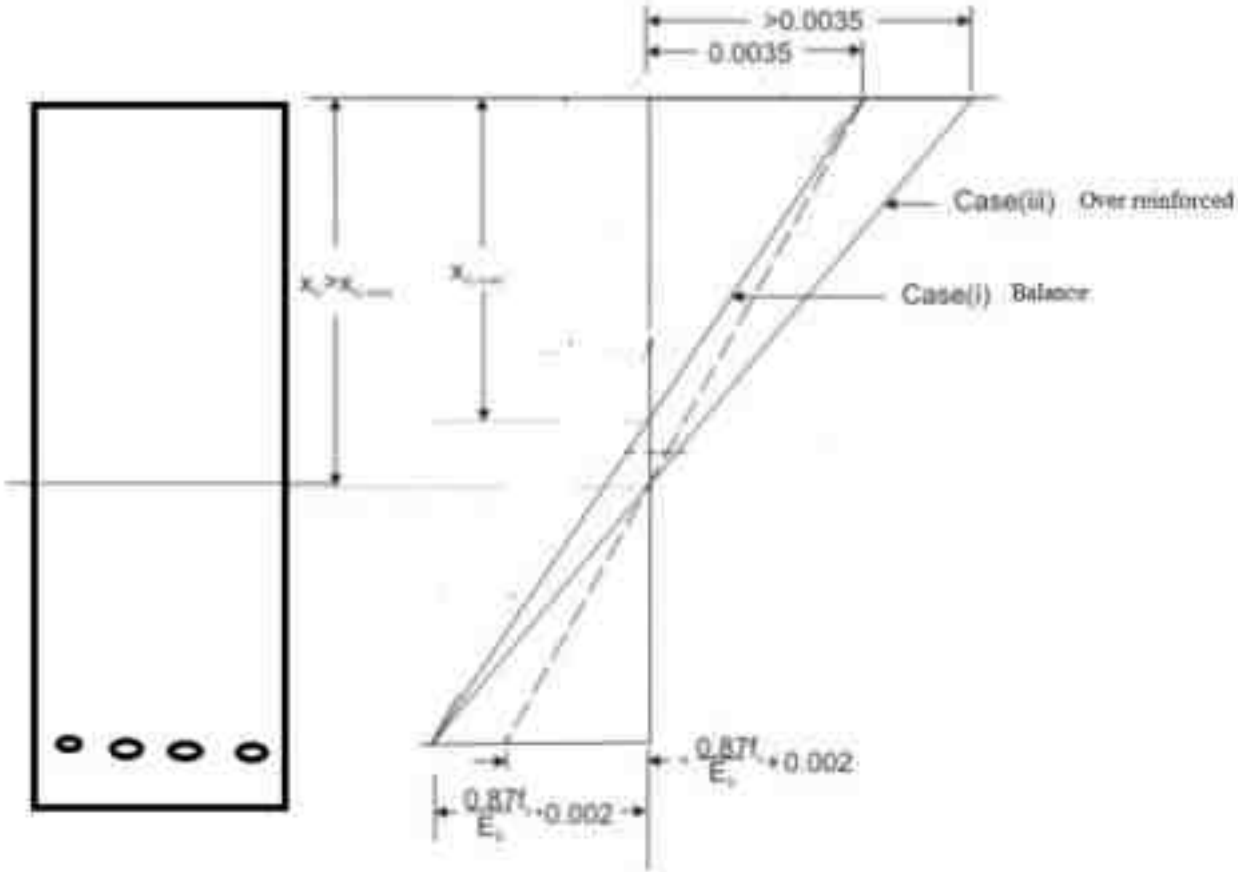
$$\frac{x_u}{d} < \frac{x_{umax} = x_{ulim}}{d} \qquad p < p_{lim}$$

Section fails in ductile manner

$$MOR = T * z = 0.87 f_y * A_{st} * (d - 0.42 x_u) =$$

$$= 0.87 f_y * A_{st} * d \left(1 - \frac{f_y * A_{st}}{f_{ck} * b * d}\right)$$

OVER EINFORCED SECTION



Strain in concrete reaches its ultimate value first i.e. 0.0035

$$\text{strain in steel is less than yield strain} = \frac{0.87 * f_y}{E_s} + 0.002$$

OVER EINFORCED SECTION

- For over reinforced case, there are two possibilities.

  Either strain in concrete is more than 0.0035 (The maximum strain in concrete at the outer most compression fibre is taken as 0.0035 in bending (which is not possible as there is fracture in concrete at strain 0.0035)

  or

  strain in steel less than yield strain which is against assumption The maximum strain in the tension reinforcement in the section at failure shall not be less than $f_y/(1.15\ E_s) + 0.002$, This assumption ensures ductile failure in which the tensile reinforcement undergoes a certain degree of inelastic deformation before concrete fails in compression.

So, over reinforced section in limit state method is not allowed.

$$\frac{x_u}{d} > \frac{x_{umax} = x_{ulim}}{d} \qquad p > p_{lim}$$

Failure is sudden, failure due to concrete.
Brittle failure

Moment of Resistance= MOR is calculated by putting x_u=x_{umax}

$$MOR = M_{ulim}$$
$$= 0.36 f_{ck}(x_{umax}/d) * [1 - 0.42 * (x_{umax}/d)]bd^2$$

OVER ,BALANCE AND UNDER REINFORCED SECTIONS PROBLEMS

G.C. BEHERA

TYPES OF PROBLEMS

- TYPE-1- TO DETERMINE MOR OF A GIVEN SECTION
- It may be required to estimate the moment of resistance.

  GIVEN

- Section, b, D, effective cover,
- A_{st},
- Grade of Concrete and Grade of Steel

SOLUTION FOR TYPE-1

- STEPS

1. Find b, D, d, A_{st}, f_{ck}, f_y
2. Calculate actual NA depth x_u

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

3. Calculate $X_{umax} = X_{ulimit}$

$$\frac{x_{umax}}{d} = \frac{0.0035}{\frac{0.87 * f_y}{E_s} + 0.0055}$$

SOLUTION FOR TYPE-1

- STEPS

4. A) If $x_u < x_{ulimit}$

   Section is under-reinforced.

   MOR is calculated taking tensile force into consideration.

$$MOR = T * z = 0.87 f_y * A_{st} * (d - 0.42 x_u)$$

$$= 0.87 f_y * A_{st} * \left(d - 0.42 * \frac{0.87 * f_y * A_{st}}{0.36 f_{ck} * b}\right)$$

$$= 0.87 f_y * A_{st} * \left(d - \frac{1.015 * f_y * A_{st}}{f_{ck} * b}\right)$$

$$= 0.87 f_y * A_{st} * \left(d - \frac{f_y * A_{st}}{f_{ck} * b}\right)$$

$$= 0.87 f_y * A_{st} * d \left(1 - \frac{f_y * A_{st}}{f_{ck} * b * d}\right)$$

SOLUTION FOR TYPE-1

- STEPS

4. B) If $x_u = x_{ulimit}$

   Section is Balance section.

   MOR is calculated taking tensile force or compression force into consideration.

   $M_u = M_{ulimit}$

$$MOR = M_{ulim}$$
$$= 0.36 f_{ck}(x_{umax}/d) * [1 - 0.42 * (x_{umax}/d)] bd^2$$

SOLUTION FOR TYPE-1

- STEPS

4. C) If $x_u > x_{ulimit}$

   Section is over reinforced.

   MOR is calculated taking compression force into consideration.

   MOR is equal to M_{ulimit}

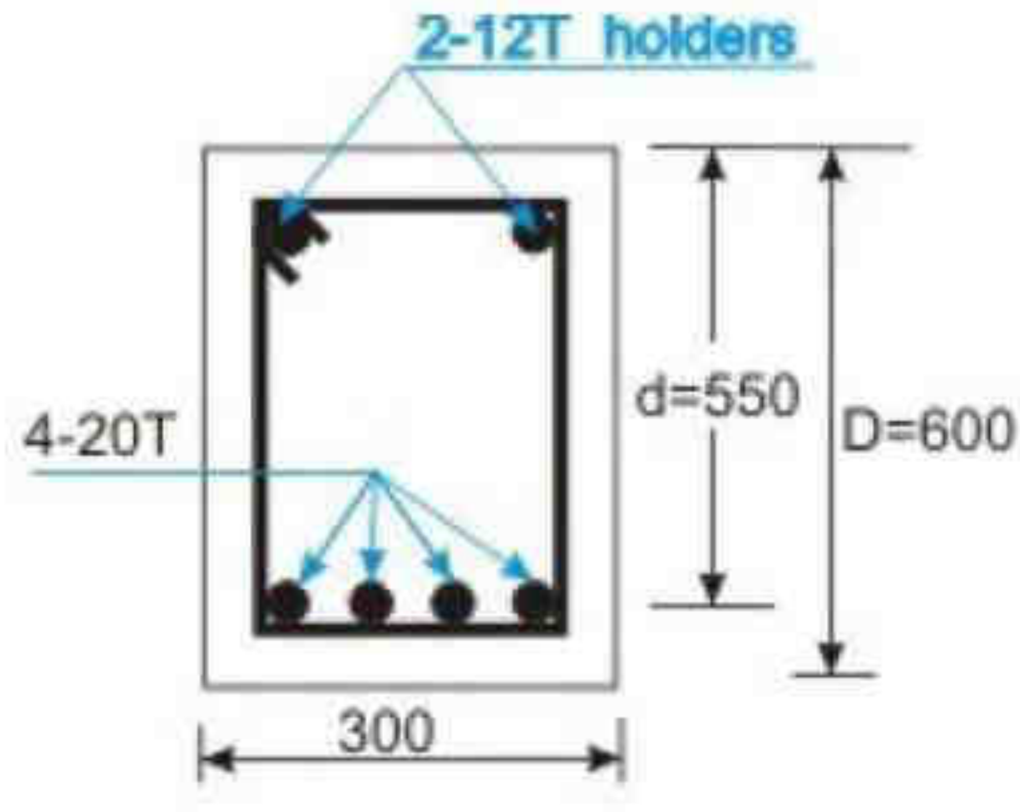
   Put $x_u = x_{ulimit}$

   $M_u = M_{ulimit}$

$$MOR = M_{ulim}$$
$$= 0.36 f_{ck}(x_{umax}/d) * [1 - 0.42 * (x_{umax}/d)] bd^2$$

PROBLEM-TYPE-1

Determine the service imposed loads of two simply supported beam of same effective span of 8 m and same cross-sectional dimensions, but having two different amounts of reinforcement. Both the beams are made of M 20 and Fe 415.



SOLUTION

Given data:

$b = 300\ mm$,

$d = 550\ mm$,

$D = 600\ mm$,

$A_{st} = 4*(\pi/4)*(20)^2 = 1256\ mm^2$ (4-20 T),

$L_{eff} = 8\ m$

$f_{ck} = 20\ MPa$

$f_y = 415\ MPa$

and boundary condition = simply supported

SOLUTION

Step 1: Calculate actual NA depth x_u

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

X_u= 209.9439 mm

Step 2:Calculate x_{ulim}

$$\frac{x_{umax}}{d} = \frac{0.0035}{\frac{0.87 * f_y}{E_s} + 0.0055}$$

For Fe 415 x_{umax}=x_{ulim}=0.48d= 264 mm

Step 3: x_u<x_{ulim}

Section is under reinforced

$$MOR = T * z = 0.87 f_y * A_{st} * (d - 0.42 x_u)$$

MOR=209427197 Nmm=209.43 kNm

If load is w,

factor load=wu,

then external factor BM=M_u=wu*$(l_{eff})^2$

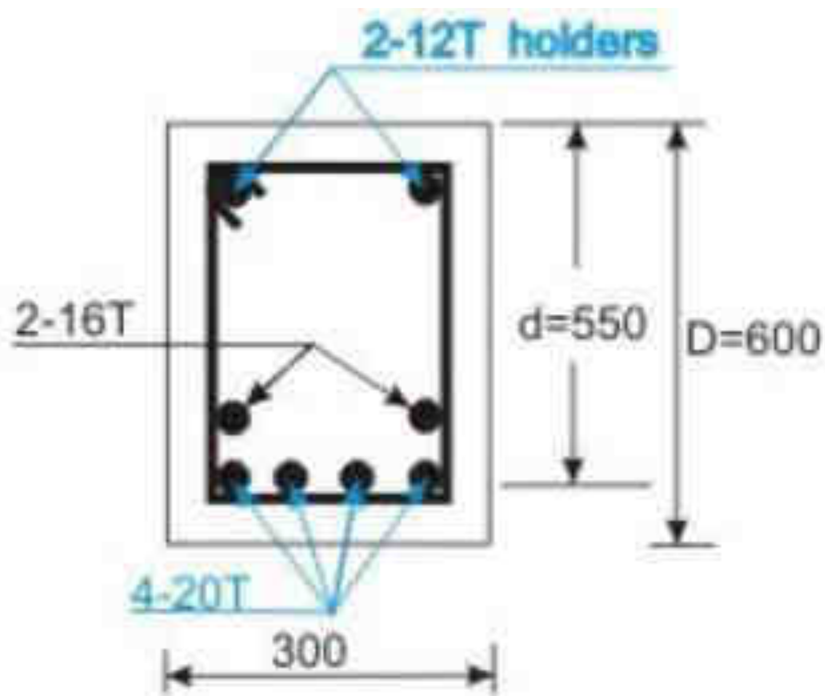
Mu= MOR= 209.43 kNm

SOLUTION

- Factor BM=M_u=209.43=[w_u*$(l_{eff})^2$]/8
- w_u=26.1784 kN/m
- w= 26.1784/1.5 = 17.452266 kN/m
- w=DL+LL
- DL=0.3*0.6*1*25=4.5 kN/m
- LL= 17.452266 -4.5=12.952266 kN/m
- Maximum Service LL or Imposed Load that can be given=12.952266 kN/m

PROBLEM-TYPE-1

Determine the service imposed loads of two simply supported beam of same effective span of 8 m and same cross-sectional dimensions, but having two different amounts of reinforcement. Both the beams are made of M 20 and Fe 415.



SOLUTION

Given data:

b = 300 mm,

d = 550 mm,

D = 600 mm,

$A_{st} = 4*(\pi/4)*(20)^2+2*(\pi/4)*(16)^2=1658\ mm^2$ *(4-20 T, 2-16 T),*

$L_{eff} = 8\ m$

f_{ck}*=20 Mpa*

f_y*=415 MPa*

and boundary condition = simply supported

SOLUTION

Step 1: Calculate actual NA depth x_u

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

X_u= 277.1393 mm

Step 2:Calculate x_{ulim}

$$\frac{x_{umax}}{d} = \frac{0.0035}{\frac{0.87 * f_y}{E_s} + 0.0055}$$

For Fe 415 x_{umax}=x_{ulim}=0.48d= 264 mm

Step 3: x_u> x_{ulim}

Section is over reinforced

$$MOR = M_{ulim}$$
$$= 0.36 f_{ck}(x_{umax}/d) * [1 - 0.42 * (x_{umax}/d)]bd^2$$

MOR= 250403789 Nmm=250.40 kNm

If load is w,

factor load=wu,

then external factor BM=M_u=(1/8)*wu*$(l_{eff})^2$

Mu= MOR= 250.40 kNm

SOLUTION

- Factor BM=M_u=250.40=$w_u*(l_{eff})^2$
- w_u=31.3 kN/m
- w= 31.3/1.5 = 20.866667 kN/m
- W=DL+LL
- DL=0.3*0.6*1*25=4.5 kN/m
- LL= 20.866667 -4.5=16.366667 kN/m
- Maximum Service LL or Imposed Load that can be given= 16.366667 kN/m

SOLUTION

- Factor BM=M_u=250.40=w_u*$(l_{eff})^2$
- w_u=31.3 kN/m
- w= 31.3/1.5 = 20.866667   kN/m
- w=DL+LL
- DL=0.3*0.6*1*25=4.5 kN/m
- LL= 20.866667 -4.5=16.366667   kN/m
- Maximum Service LL or Imposed Load that can be given= 16.366667 kN/m

SOLUTION

- Factor BM=M_u= As per steel

$$M_u = 0.87\, f_y\, A_{st}\, (d - 0.42\, x_{u,\,max})$$

BM=M_u=262.87 kNm

This is not true. All the steel in tension zone may not be yielded

TYPES OF PROBLEMS-II

- To Compute Amount of Steel for a given load
- Given

  Section (b,D, d)

  Grade of Concrete and Steel

  Service load,

  Length of Beam and End Condition

TYPES OF PROBLEMS-II

- Solution:
- Step-1

  Compute Mu

  Find Load w, Calculate w_u

  Find M_u from beam length and end condition
- Step-2

  Compute Mulimit $MOR = M_{ulim}$

$$= 0.36 f_{ck}(x_{umax}/d) * [1 - 0.42 * (x_{umax}/d)]bd^2$$

  Step-3 A)

  If $M_u < M_{ulimit}$

  Design it as under reinforced

  M_u=MOR $MOR = T * z = 0.87 f_y * A_{st} * (d - 0.42 x_u) =$

$$= 0.87 f_y * A_{st} * d\left(1 - \frac{f_y * A_{st}}{f_{ck} * b * d}\right)$$

In the equation Mu known, only unknown is A_{st}In the equation Mu
As it a quadratic equation, solution will provide you two values
Find out the feasible one.

TYPES OF PROBLEMS-II

Step-3 B)

If $M_u = M_{ulimit}$

Design it as Balance section.

M_u=MOR

$$MOR = T * z = 0.87f_y * A_{st} * (d - 0.42x_u) =$$

$$= 0.87f_y * A_{st} * d\left(1 - \frac{f_y * A_{st}}{f_{ck} * b * d}\right)$$

In the equation Mu known, only unknown is A_{st}In the equation Mu
As it a quadratic equation, solution will provide you two values
Find out the feasible one.

OR find $_{xulimit}$

$$MOR = M_u = T * z = 0.87f_y * A_{st} * (d - 0.42x_{ulim})$$

In the equation Mu known, only unknown is A_{st}

TYPES OF PROBLEMS-II

Step-3 C)

If $M_u > M_{ulimit}$

Design it as over reinforced

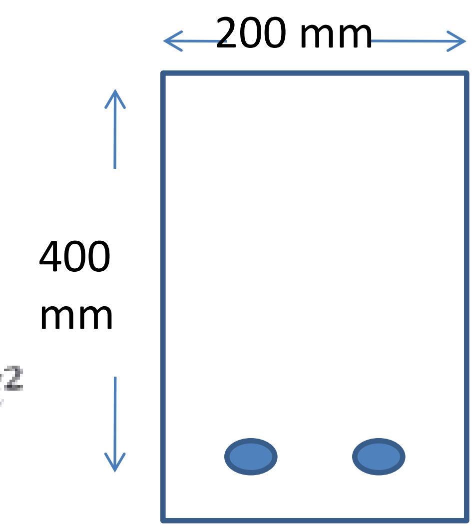
This section is not allowed in Limit state method.

Redesign the section.

Increase the section or make it doubly reinforced section

TYPES OF PROBLEMS-II

Problem: A rectangular beam 200 mm wide and 400 mm deep upto centre of reinforcement. Design the beam if it has to resist a moment 25 kNm. Use M20 and fe415



Solution:

Step-1 Computation of M_u

M=25 kNm

M_u= Factor BM=1.5*25= 37.5 kNm

Step-2 Computation of M_{ulimit}

$$M_{ulim} = 0.36 f_{ck} (x_{umax}/d) * [1 - 0.42 * (x_{umax}/d)] b d^2$$

M_{ulimit}=.36*20*.48*(1-0.42*.48)*200*400^2=88.30 kNm

As $M_u < M_{ulimit}$

Design it as under reinforced

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

TYPES OF PROBLEMS-II

M_u= Factor BM=1.5*25= 37.5 kNm

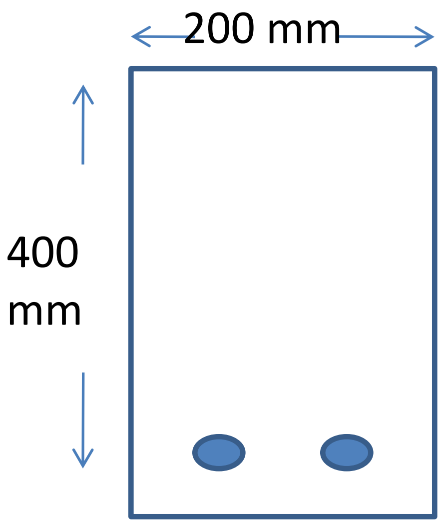
$$M_u = 0.87 f_y * A_{st} * d\left(1 - \frac{f_y * A_{st}}{f_{ck} * b * d}\right)$$

$$37.5 * 10^6 = 0.87 * 415 * A_{st} * 400 * \left(1 - \frac{415 * A_{st}}{20 * 200 * 400}\right)$$

A_{st}= 279.99= 280 mm^2
A_{st}= 3575.4048 mm^2 Discard this result

ASSIGNMENT –POBLEM TYPE-II

Problem: A rectangular beam 200 mm wide and 400 mm deep upto centre of reinforcement. Design the beam if it has to resist a moment 60 kNm. Use M25 and fe500



DESIGN OF BEAMS

G.C. BEHERA

EFFECTIVE LENGTH

22.2 Effective Span

Unless otherwise specified, the effective span of a member shall be as follows:

a) ***Simply Supported Beam or Slab*—The effective span of a member that is not built integrally with its supports shall be taken as clear span plus the effective depth of slab or beam or centre to centre of supports, whichever is less.**

- Clear distance between walls = 6m
- Thickness of one wall =300 mm
- Thickness of another wall =400 mm
- Effective depth of beam= 450 mm
- Simply supported Beam
- Find effective length of beam?
- L_{eff}=Clear span+deff=6m+.45m=6.45 m
- L_{eff}=C/C bet supports=6m+.3/2+0.4/2=6.35 m

EFFECTIVE LENGTH

- ***B)Continuous Beam or Slab -*** *In the case of* continuous beam or slab, if the width of the support is less than l/12 of the clear span, the effective span shall be as in 22.2 (a). If the supports are wider than l/12 of the clear span or 600 mm whichever is less, the effective span shall be taken as under:
- 1) For end span with one end fixed and the other continuous or for intermediate spans, the effective span shall be the clear span between supports;
- 2) For end span with one end free and the other continuous, the effective span shall be equal to the clear span plus half the effective depth of the beam or slab or the clear span plus half the width of the discontinuous support, whichever is less;
- 3) In the case of spans with roller or rocket bearings, the effective span shall always be the distance between the centres of bearings

c) **Cantilever**-The effective length of a cantilever shall betaken as its length to the face of the support plus half the effective depth except where it forms the end of a continuous beam where the length to the centre of support shall be taken.

- D) **F*rames-*** *In the analysis of a continuous frame,* centre to centre distance shall be used.

SHEAR AND BM COEFFICIENTS

Table 12 Bending Moment Coefficients
(*Clause* 22.5.1)

Type of Load	Span Moments		Support Moments	
	Near Middle of End Span	At Middle of Interior Span	At Support Next to the End Support	At Other Interior Supports
(1)	(2)	(3)	(4)	(5)
Dead load and imposed load (fixed)	$+\frac{1}{12}$	$+\frac{1}{16}$	$-\frac{1}{10}$	$-\frac{1}{12}$
Imposed load (not fixed)	$+\frac{1}{10}$	$+\frac{1}{12}$	$-\frac{1}{9}$	$-\frac{1}{9}$

NOTE — For obtaining the bending moment, the coefficient shall be multiplied by the total design load and effective span.

Table 13 Shear for Coefficients
(*Clauses* 22.5.1 *and* 22.5.2)

Type of Load	At End Support	At Support Next to the End Support		At All Other Interior Supports
		Outer Side	Inner Side	
(1)	(2)	(3)	(4)	(5)
Dead load and imposed load (fixed)	0.4	0.6	0.55	0.5
Imposed load (not fixed)	0.45	0.6	0.6	0.6

NOTE — For obtaining the shear force, the coefficient shall be multiplied by the total design load.

23.2.1 The vertical deflection limits may generally be assumed to be satisfied provided that the span to depth ratios are not greater than the values obtained as below:

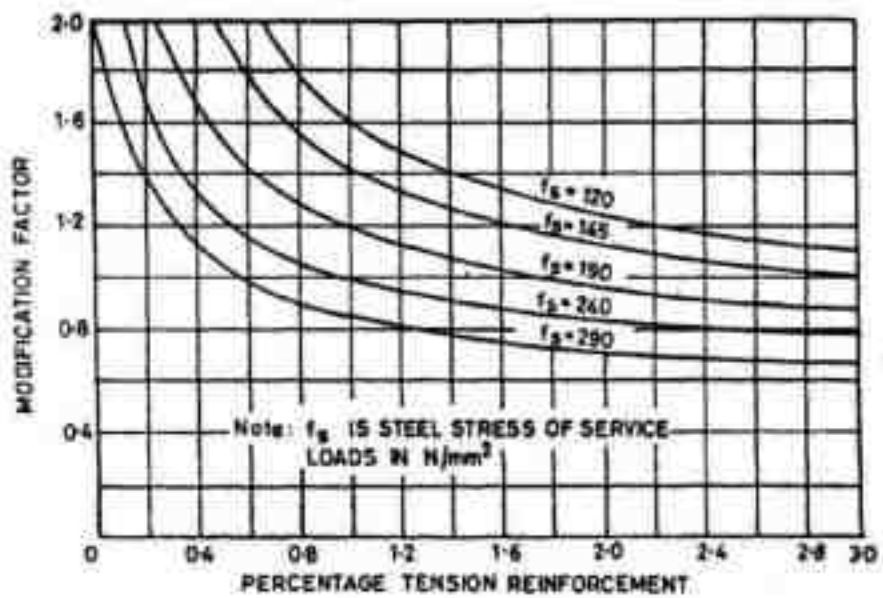
a) Basic values of span to effective depth ratios for spans up to 10 m:

Cantilever	7
Simply supported	20
Continuous	26

b) For spans above 10 m, the values in (a) may be multiplied by 10/span in metres, except for cantilever in which case deflection calculations should be made.

c) Depending on the area and the stress of steel for tension reinforcement, the values in (a) or (b) shall be modified by multiplying with the modification factor obtained as per Fig. 4.

d) Depending on the area of compression reinforcement, the value of span to depth ratio be further modified by multiplying with the modification factor obtained as per Fig. 5.



$$f_s = 0.58\, f_y \frac{\text{Area of cross-section of steel required}}{\text{Area of cross-section of steel provided}}$$

Fig. 4 Modification Factor for Tension Reinforcement

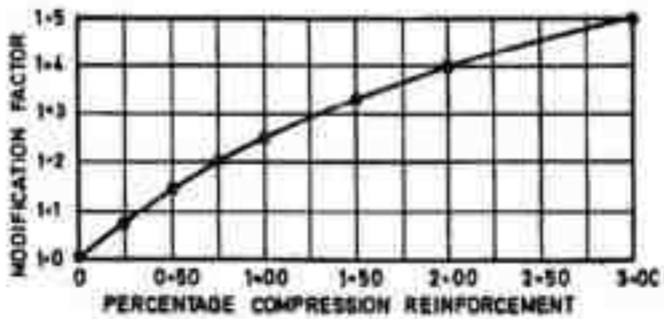
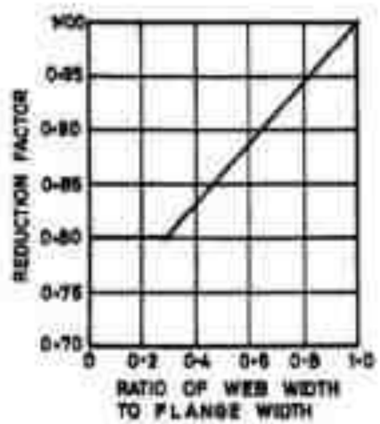


FIG. 5 MODIFICATION FACTOR FOR COMPRESSION REINFORCEMENT



26.3.3 *Maximum Distance Between Bars in Tension*

Unless the calculation of crack widths shows that a greater spacing is acceptable, the following rules shall be applied to flexural members in normal internal or external conditions of exposure.

a) *Beams* — The horizontal distance between parallel reinforcement bars, or groups, near the tension face of a beam shall not be greater than the value given in Table 15 depending on the amount of redistribution carried out in analysis and the characteristic strength of the reinforcement.

b) *Slabs*

   1) The horizontal distance between parallel main reinforcement bars shall not be more than three times the effective depth of solid slab or 300 mm whichever is smaller.

   2) The horizontal distance between parallel reinforcement bars provided against shrinkage and temperature shall not be more than five times the effective depth of a solid slab or 450 mm whichever is smaller.

26.4.1 *Nominal Cover*

Nominal cover is the design depth of concrete cover to all steel reinforcements, including links. It is the dimension used in design and indicated in the drawings. It shall be not less than the diameter of the bar.

26.4.2 *Nominal Cover to Meet Durability Requirement*

Minimum values for the nominal cover of normal-weight aggregate concrete which should be provided to all reinforcement, including links depending on the condition of exposure described in **8.2.3** shall be as given in Table 16.

26.4.2.1 However for a longitudinal reinforcing bar in a column nominal cover shall in any case not be less than 40 mm, or less than the diameter of such bar. In the case of columns of minimum dimension of 200 mm or under, whose reinforcing bars do not exceed 12 mm, a nominal cover of 25 mm may be used.

26.4.2.2 For footings minimum cover shall be 50 mm.

26.4.3 *Nominal Cover to Meet Specified Period of Fire Resistance*

Minimum values of nominal cover of normal-weight aggregate concrete to be provided to all reinforcement including links to meet specified period of fire resistance shall be given in Table 16A.

TENSILE REINFORCEMENT

Minimum reinforcement not less than

given by the following:

$$\frac{A_s}{bd} = \frac{0.85}{f_y}$$

where

A_s = minimum area of tension reinforcement,

b = breadth of beam or the breadth of the web of T-beam,

d = effective depth, and

f_y = characteristic strength of reinforcement in N/mm².

b) *Maximum reinforcement*—The maximum area of tension reinforcement shall not exceed 0.04 bD.

26.5.1.2 *Compression reinforcement*

The maximum area of compression reinforcement shall not exceed 0.04 bD. Compression reinforcement in beams shall be enclosed by stirrups for effective lateral restraint. The arrangement of stirrups shall be as specified in **26.5.3.2**.

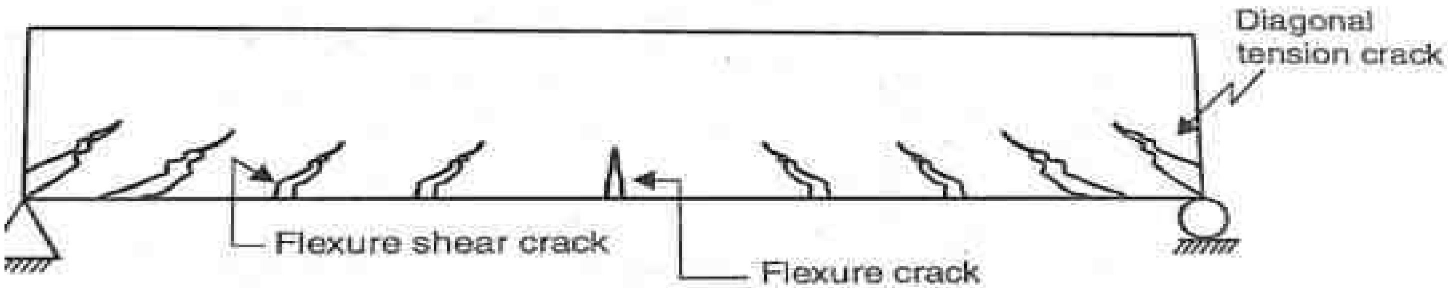
DESIGN FOR SHEAR

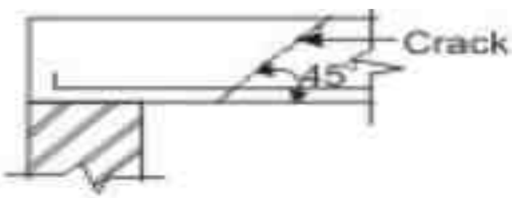
G.C. BEHERA

FAILURE OF BEAMS

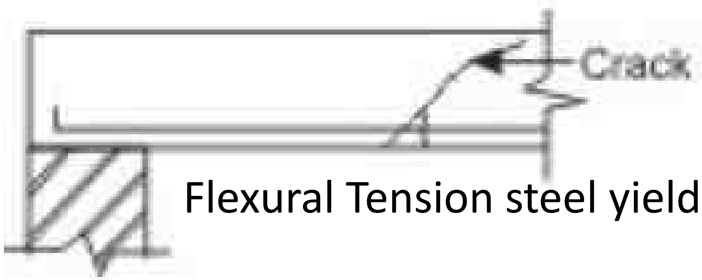
- This lesson explains the three failure modes due to shear force in beams and defines different shear stresses needed to design the beams for shear. The critical sections for shear and the minimum shear reinforcement to be provided in beams are mentioned as per IS 456. The design of shear reinforcement will be illustrated through several numerical problems including the curtailment of tension reinforcement in flexural members.
- Bending in reinforced concrete beams is usually accompanied by shear, the exact analysis of which is very complex. However, experimental studies confirmed the following three different modes of failure due to possible combinations of shear force and bending moment at a given section

Failure modes

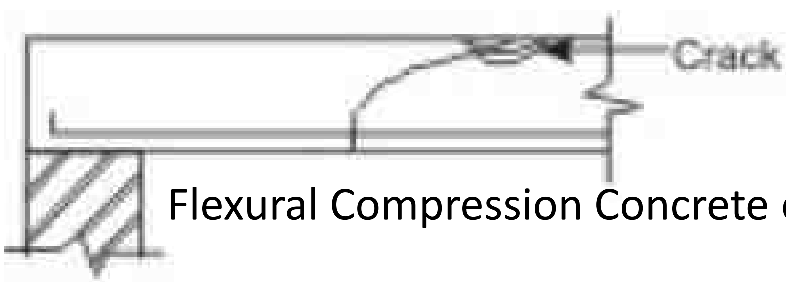




Web shear progresses along dotted dotted lines

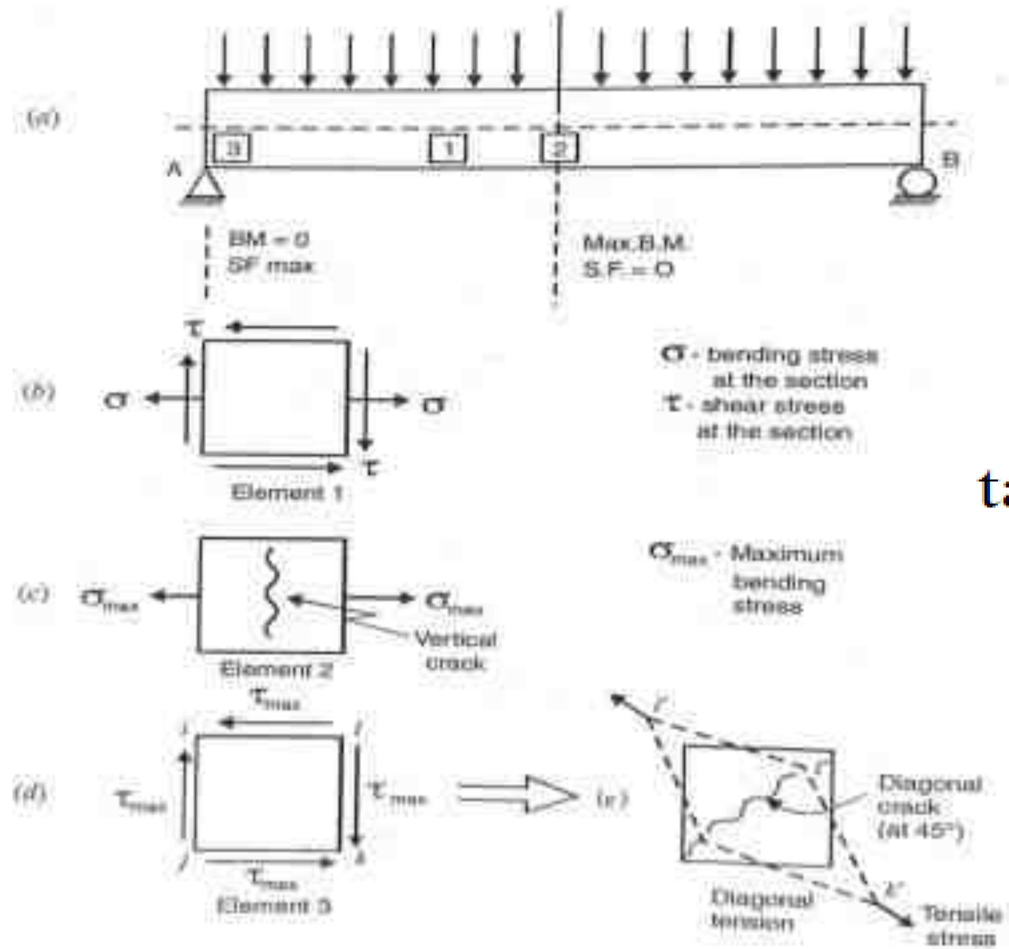


Flexural Tension steel yields



Flexural Compression Concrete crushes in compression

SHEAR STRESS VARIATION

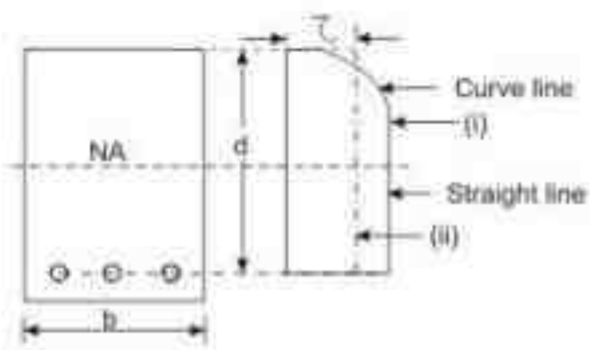


$$\tan 2\theta_P = \frac{-2\tau_{xy}}{\sigma_x - \sigma_y}$$

Diagonal ik subjected to tension, crack along JL

SHEAR STRESS CALCULATION

- The distribution of shear stress in reinforced concrete rectangular, *T and L-beams of uniform and varying depths depends on the distribution of the normal stress. However, for the sake of simplicity the nominal shear stress* τ_v *is considered which is calculated as follows (IS 456, cls. 40.1 and 40.1.1):*



Note:

(i) Actual distribution
(ii) Average distribution

(b) T-beam

Note:

(i) Actual distribution
(ii) Average distribution

SHEAR STRESS CALCULATION

In a beam of uniform depth $\tau_{vu} = V_u / bd$

For varying Depth

$$\tau_{vu} = \frac{V_u \pm \frac{M_u}{d} \tan\beta}{bd}$$

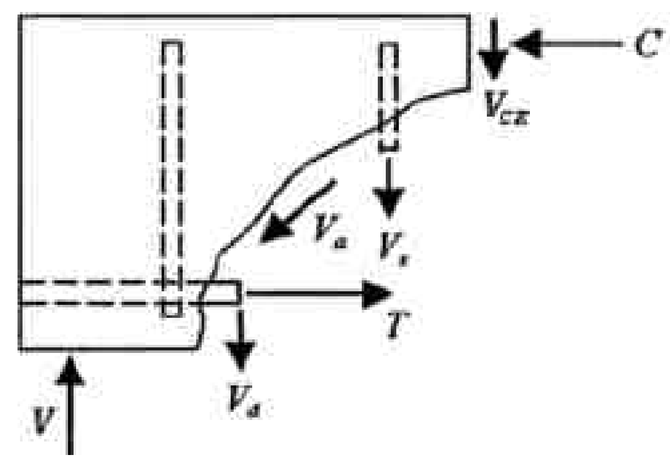
- where *Vu* = Factored shear force due to design loads,
- *b* = breadth of rectangular beams and breadth of the web b_w for flanged beams, and
- *d* = effective depth.
- M_u = Factored bending moment at the section, and
- β = angle between the top and the bottom edges.
- τ_{vu} is the nominal shear stress
- The positive sign is applicable when the bending moment *M_u decreases numerically in the same direction as the effective depth increases, and the negative sign is applicable when the bending moment M_u increases numerically in the same direction as the effective depth increases.*

Design Shear Strength of Reinforced Concrete Beams

- Recent laboratory experiments confirmed that reinforced concrete in beams has shear strength even without any shear reinforcement. This shear strength (τ_c) *depends on the grade of concrete and the percentage of tension steel in beams. On the other hand, the shear strength of reinforced concrete with the reinforcement is restricted to some maximum value* τ_{cmax} *depending on the grade of concrete. These minimum and maximum shear strengths of reinforced concrete (IS 456, cls. 40.2.1 and 40.2.3, respectively) are given below:*

SHEAR RESISTING MECHANISM

- Freebody diagram of a segment of reinforced concrete beam separated by a diagonal tension crack.
- The components of shear transfer mechanism are
- a) Shear resistance by uncracked concrete in compression Vcz
- b) Vertical component of aggregate interlock across crack surface V_{ay}
- c) Dowel force in tension reinforcement V_d
- d) Shear resistance by stirrups V_s



Mechanism of Shear Transfer in Cracked concrete Beam

Shear strength of Reinforced Concrete

Table 19 of IS 456 stipulates the design shear strength of concrete τ^c *for different grades of concrete with a wide range of percentages of positive tensile steel reinforcement. It is worth mentioning that the reinforced concrete beams must be provided with the minimum shear reinforcement as per cl. 40.3 even when* τ_v *is less than* τ_c *given in Table.*

Design shear strength of concrete, τ_c *in N/mm*^{2}

(100 A_s/b d)	Grade of concrete				
	M 20	M 25	M 30	M 35	M40 and above
≤ 0.15	0.28	0.29	0.29	0.29	0.30
0.25	0.36	0.36	0.37	0.37	0.38
0.50	0.48	0.49	0.50	0.50	0.51
0.75	0.56	0.57	0.59	0.59	0.60
1.00	0.62	0.64	0.66	0.67	0.68
1.25	0.67	0.70	0.71	0.73	0.74
1.50	0.72	0.74	0.76	0.78	0.79
1.75	0.75	0.78	0.80	0.82	0.84
2.00	0.79	0.82	0.84	0.86	0.88
2.25	0.81	0.85	0.88	0.90	0.92
2.50	0.82	0.88	0.91	0.93	0.95
2.75	0.82	0.90	0.94	0.96	0.98
≥ 3.00	0.82	0.92	0.96	0.99	1.01

A_s *is the area of longitudinal tension reinforcement which continues at least one effective depth beyond the section considered except at support where the full area of tension reinforcement may be used provided the detailing is as per IS 456, cls. 26.2.2 and 26.2.3.*

Maximum shear stress τ_{cmax} *with shear reinforcement (cls. 40.2.3, 40.5.1 and 41.3.1)*

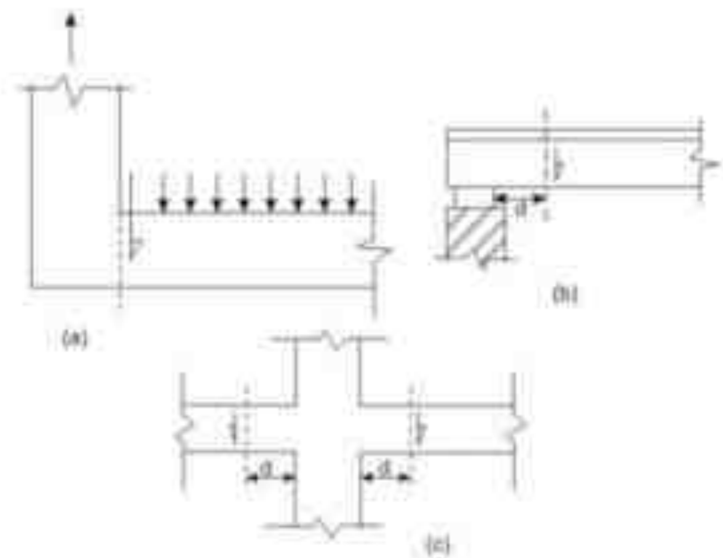
Table 20 of IS 456 stipulates the maximum shear stress of reinforced concrete in beams τ_{cmax} *as given below in Table. Under no circumstances, the nominal shear stress in beams* τ_v *shall exceed* τ_{cmax} *given in Table below for different grades of concrete.*

Maximum shear stress, τ_{cmax} *in N/mm²*

Grade of concrete	M 20	M 25	M 30	M 35	M 40 and above
τ_{cmax}, N/mm²	2.8	3.1	3.5	3.7	4.0

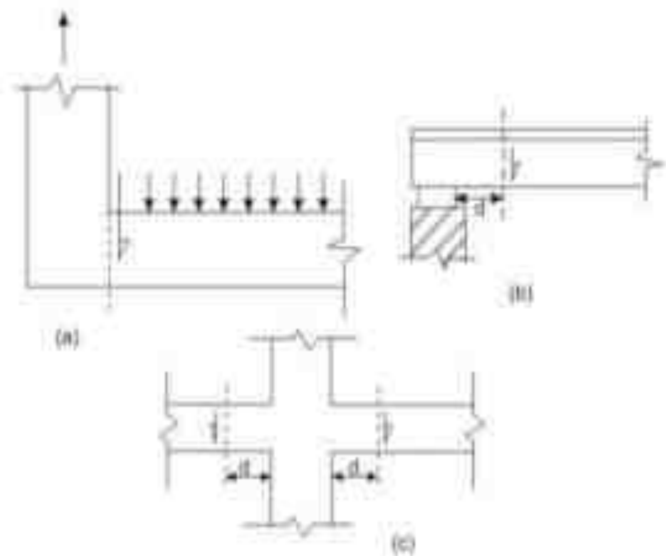
Critical Section for Shear

- Clauses 22.6.2 and 22.6.2.1 stipulate the critical section for shear and are as follows:
- For beams generally subjected to uniformly distributed loads or where the principal load is located further than 2*d from the face of the support, where d is the effective depth of the beam, the critical sections depend on the conditions of supports as shown in Figs. are mentioned below.*
- (i) When the reaction in the direction of the applied shear introduces tension (a) into the end region of the member, the shear force is to be computed at the face of the support of the member at that section.



Critical Section for Shear

- (ii) When the reaction in the direction of the applied shear introduces compression into the end region of the member (Figs. b and c), the shear force computed at a distance *d from the face of the support is to be used for the design of sections located at a distance less than d from the face of the support. The enhanced shear strength of sections close to supports, however, may be considered as discussed in the following section.*

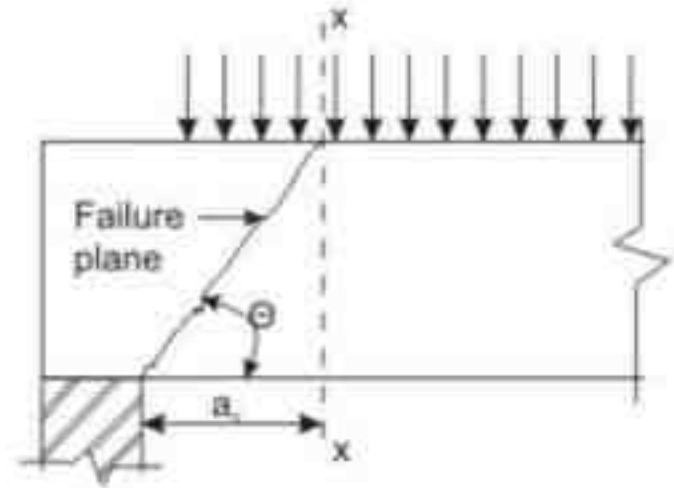


Enhanced Shear Strength of Sections Close to supports

- 40.5.1 General

Shear failure at sections of beams and cantilevers without shear reinforcement will normally occur on plane inclined at an angle 30” to the horizontal. If the angle of failure plane is forced to be inclined more steeply than this [because the section considered (X - X) in Fig. is close to a support or for other reasons], the shear force equired to produce failure is increased.

The enhancement of shear strength may be taken into account in the design of sections near a support by increasing design shear strength of concrete to 2d τ_c /a_v, provided that design shear stress at the face of the support remains less than the values given in Table 20.



Account may be taken of the enhancement in any situation where the section considered is closer to the face of a support or concentrated load than twice the effective depth, *d. To be effective, tension* reinforcement should extend on each side of the point where it is intersected by a possible failure plane for a distance at least equal to the effective depth, or be provided with an equivdent anchorage.

Minimum Shear Reinforcement (cls. 40.3, 26.5.1.5 and 26.5.1.6 of IS 456)

Minimum shear reinforcement has to be provided even when τ^v *is less than* τ_c *given in Table 6.1 as recommended in cl. 40.3 of IS 456. The amount of minimum shear reinforcement, as given in cl. 26.5.1.6, is given below.*

- The minimum shear reinforcement in the form of stirrups shall be provided such that:

$$\frac{A_{sv}}{bs_v} \geq \frac{0.4}{0.87 f_y}$$

- where *Asv* = total cross-sectional area of stirrup legs effective in shear,
- s_v = stirrup spacing along the length of the member,
- *b* = breadth of the beam or breadth of the web of the web of flanged beam *bw*, and
- f_y = characteristic strength of the stirrup reinforcement in N/mm^2 which shall not be taken greater than 415 N/mm^2.

- The minimum shear reinforcement is provided for the following:
- (i) Any sudden failure of beams is prevented if concrete cover bursts and the bond to the tension steel is lost.
- (ii) Brittle shear failure is arrested which would have occurred without shear reinforcement.
- (iii) Tension failure is prevented which would have occurred due to shrinkage, thermal stresses and internal cracking in beams.
- (iv) To hold the reinforcement in place when concrete is poured.
- (v) Section becomes effective with the tie effect of the compression steel.

Further, cl. 26.5.1.5 of IS 456 stipulates that the maximum spacing of shear reinforcement measured along the axis of the member shall not be more than 0.75 *d for vertical stirrups and d for inclined stirrups at 45°, where d is the effective depth of the section. However, the spacing shall not exceed 300 mm in any case.*

Design of Shear Reinforcement (cl. 40.4 of IS 456)

- When τ_v is more than τ_c , shear reinforcement shall be provided in any of the three following forms:
- (a) Vertical stirrups,
- (b) Bent-up bars along with stirrups, and
- (c) Inclined stirrups.
- In the case of bent-up bars, it is to be seen that the contribution towards shear resistance of bent-up bars should not be more than fifty per cent of that of the total shear reinforcement.
- The amount of shear reinforcement to be provided is determined to carry a shear force V_{us} equal to
- $V_{us}=V_u-V_c=V_u-\tau_c\, bd$
- where *b* is the breadth of rectangular beams or b_w in the case of flanged beams.

The strengths of shear reinforcement *Vus* for the three types of shear reinforcement are as follows

SHEAR REINFORCEMENT

- (a) Vertical stirrups: $V_{us} = \frac{d}{s_v} A_{sv} 0.87 f_y$
- (b) For inclined stirrups or a series of bars bent-up at different cross-sections:

$$V_{us} = \frac{d}{s_v} A_{sv} 0.87 f_y (sin\alpha + cos\alpha)$$

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

$$V_{us} = A_{sb} 0.87 f_y \sin\alpha$$

- where A_{sv} = *total cross-sectional area of stirrup legs or bent-up bars within a distance* s_v,
- s_v = *spacing of stirrups or bent-up bars along the length of the member,*
- τ_v = *nominal shear stress,*
- τ_c = *design shear strength of concrete,*

SHEAR REINFORCEMENT

- *b = breadth of the member which for the flanged beams shall be taken as the breadth of the web* b_w*,*
- f_y *= characteristic strength of the stirrup or bent-up reinforcement which shall not be taken greater than 415 N/mm*^{2}*,*
- α = angle between the inclined stirrup or bent-up bar and the axis of the member, not less than 45^o, and *d = effective depth.*

The following two points are to be noted:

- (i) The total shear resistance shall be computed as the sum of the resistance for the various types separately where more than one type of shear reinforcement is used.
- (ii) The area of stirrups shall not be less than the minimum specified in cl. 26.5.1.6.

Shear Reinforcement for Sections Close to Supports

As stipulated in cl. 40.5.2 of IS 456, the total area of the required shear reinforcement A_s *is obtained from:*

$$A_s = a_v b (\tau_v - 2d \tau_c / a_v) / 0.87 f_y$$

$$\text{and} \qquad \geq 0.4 a_v b / 0.87 f_y$$

This reinforcement should be provided within the middle three quarters of a^v, *where* a_v *is less than d, horizontal shear reinforcement will be effective than vertical.*

Alternatively, one simplified method has been recommended in cl. 40.5.3 of IS 456 and the same is given below.

The following method is for beams carrying generally uniform load or where the principal load is located further than 2*d from the face of support. The shear stress is calculated at a section a distance d from the face of support. The value of* τ_c *is calculated in accordance with Table 6.1 and appropriate shear reinforcement is provided at sections closer to the support. No further check for shear at such sections is required.*

Curtailment of Tension Reinforcement in Flexural Members (cl. 26.2.3.2 of IS 456)

Curtailment of tension reinforcement is done to provide the required reduced area of steel with the reduction of the bending moment. However, shear force increases with the reduction of bending moment. Therefore, it is necessary to satisfy any one of following three conditions while terminating the flexural reinforcement in tension zone:

- (i) The shear stress τ_v *at the cut-off point should not exceed two-thirds of the permitted value which includes the shear strength of the web reinforcement. Accordingly,*

$$\tau_v \leq (2/3)\,(\tau_c + V_{us}/b\,d)$$

or $$V_{us} \geq (1.5\,\tau_v - \tau_c)\,b\,d$$

Curtailment of Tension Reinforcement in Flexural Members (cl. 26.2.3.2 of IS 456)

- (ii) For each of the terminated bars, additional stirrup area should be provided over a distance of three-fourth of effective depth from the cut-off point. The additional stirrup area shall not be less than 0.4 $b\ s/f_y$, *where* b *is the breadth of rectangular beams and is replaced by* b_w, *the breadth of the web for flanged beams,*
- s *= spacing of additional stirrups and*
- f_y *is the characteristic strength of stirrup reinforcement in N/mm*^{2}. *The value of* s *shall not exceed* $d/(8\ \beta_b)$,

  where β_b *is the ratio of area of bars cut-off to the total area of bars at that section, and* d *is the effective depth.*

Curtailment of Tension Reinforcement in Flexural Members (cl. 26.2.3.2 of IS 456)

- (iii) For bars of diameters 36 mm and smaller, the continuing bars provide double the area required for flexure at the cut-off point. The shear stress should not exceed three-fourths that permitted.

$$\tau_v \leq (3/4)(\tau_c + V_{us}/b\,d)$$

$$\text{or} \quad V_{us} \geq (1.33\,\tau_v - \tau_c)\,b\,d$$

BOND, ANCHORAGE

G.C. BEHERA

BOND

- 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. The length of a member required to develop the full bond is called the anchorage length.
- The bond is measured by bond stress. The local bond stress varies along a member with the variation of bending moment. The average value throughout its anchorage length is designated as the average bond stress. In our calculation, the average bond stress will be used.

BOND

- **The term bond** refers to the adhesion between the concrete and the steel which resists the slipping of steel bar from concrete. It is this bond which is responsible for transfer to stresses from steel to concrete and thereby providing composite action of steel and concrete in R.C.C. The bond develops due to setting of concrete on drying which results in gripping of the steel bars.
- The bond resistance in reinforced concrete is obtained by following mechanism:

➢ Chemical adhesion: it is due to gum like property of the substances, formed after setting of concrete.

➢ Frictional resistance: it is due to adhesion between steel and concrete.

➢ Gripping action: it is due to gripping of steel by the concrete on drying.

➢ Mechanical interlock: It is provided by the corrugations or ribs present on the surface of the deformed bars.

BOND

- The bond is assumed to be perfect in the design of reinforced concrete. The bond between steel and concrete can be increased by the following methods:

➢ Using twisted or deformed bar.

➢ Using rich mix of concrete.

➢ Adequate compaction and curing of concrete for proper setting.

➢ Providing hooks at the end of reinforcing bars.

BOND

- The concept of development length and anchorage of the steel reinforcement has replaced the earlier practice of checking and satisfying the permissible flexural bond stress. It is observed that the flexural bond stress does not provide an appropriate method of ensuring safety against bond failure. The development length criteria gives a better estimate of the strength of the bond. In simply supported beams, the critical section exists at the point of the maximum stress or at a section where bars are curtailed. In continuous beam, in addition to points of maximum stress and curtailment, points of contraflexure should be checked for bond.
- There are two types of bond failure:

➢ Anchorage bond failure

➢ Flexural bond failure

BOND ANCHORAGE

- Thus, a tensile member has to be anchored properly by providing additional length on either side of the point of maximum tension, which is known as 'Development length in tension'. Similarly, for compression members also, we have 'Development length L_d *in compression'.*
- Deformed bars are known to be superior to the smooth mild steel bars due to the presence of ribs. In such a case, it is needed to check for the sufficient development length L_d *only rather than checking both for the local bond stress and development length as required for the smooth mild steel bars. Accordingly, IS 456, cl. 26.2 stipulates the requirements of proper anchorage of reinforcement in terms of development length* L^d *only employing design bond stress* τ_{bd}.

Design Bond Stress τ_{bd}

- The design bond stress τ_{bd} *is defined as the shear force per unit nominal surface area of reinforcing bar. The stress is acting on the interface between bars and surrounding concrete and along the direction parallel to the bars.*

- This concept of design bond stress finally results in additional length of a bar of specified diameter to be provided beyond a given critical section. Though, the overall bond failure may be avoided by this provision of additional development length L_d*, slippage of a bar may not always result in overall failure of a beam. It is, thus, desirable to provide end anchorages also to maintain the integrity of the structure and thereby, to enable it carrying the loads. Clause 26.2 of IS 456 stipulates, "The calculated tension or compression in any bar at any section shall be developed on each side of the section by an appropriate development length or end anchorage or by a combination thereof."*

Design bond stress – values

- The local bond stress varies along the length of the reinforcement while the average bond stress gives the average value throughout its development length.
- This average bond stress is still used in the working stress method and IS 456 has mentioned about it in cl. B-2.1.2. However, in the limit state method of design, the average bond stress has been designated as design bond stress τ_{bd} *and the values are given in cl. 26.2.1.1. The same is given below as a ready reference.*
-

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

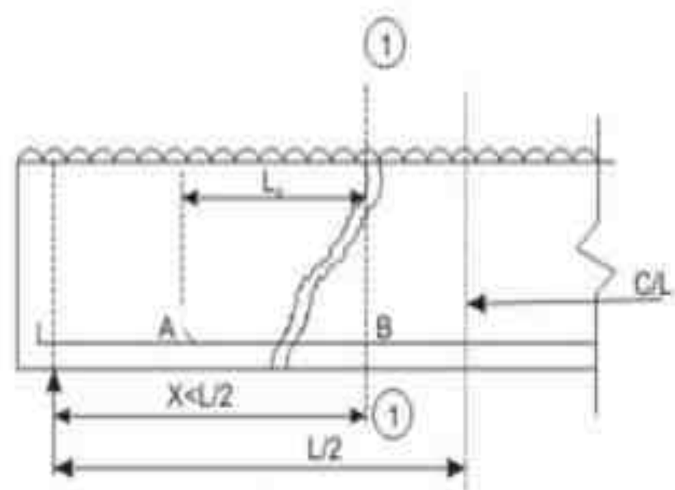
For deformed bars conforming to IS 1786, these values shall be increased by 60 per cent. For bars in compression, the values of bond stress in tension shall be increased by 25 per cent.

DESIGN BOND STRESS

Grade of concrete	**τ_{bd} for plain bars (N/mm^2)**	**τ_{bd} for deformed bars (N/mm^2)**
M20	1.2	1.92
M25	1.4	2.24
M30	1.5	2.4
M35	1.7	2.72
M40 and above	1.9	3.04

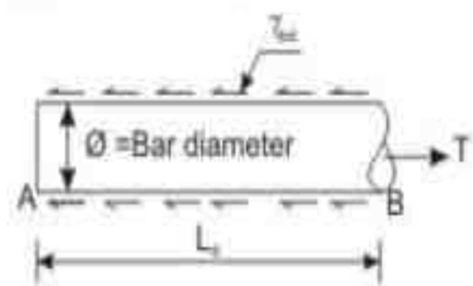
ACNHORAGE BOND (DEVELOPMENT LENGTH)

Figure shows a simply supported beam subjected to uniformly distributed load. Because of the maximum moment, the A_{st} *required is the maximum at x = L/2. For any section 1-1 at a distance x < L/2, some of the* tensile bars can be curtailed. Let us then assume that section 1-1 is the theoretical cut-off point of one bar. However, it is necessary to extend the bar for a length L_d *as explained earlier. Let us derive the expression to determine* L_d *of this bar.*



(a): Beam showing L_d of a bar

The free body diagram of the segment AB of the bar. At B, the tensile force *T trying to pull out the bar is of the value* $T = (\pi/4)*(\phi)^2*\sigma_S$ *, where* ϕ *is the nominal diameter of the bar and* σ_s *is the tensile stress in bar at the section considered at design loads. It is necessary to have the resistance force to be developed by* τ_{bd} *for the length* L_d *to overcome the tensile force.*



(b): Free body diagram of segment AB

ACNHORAGE BOND (DEVELOPMENT LENGTH)

$T = (\pi/4)*(\phi)^2*\sigma_S$

σ_S is the stress in steel and maximum is $0.87f_y$

$T = (\pi/4)*(\phi)^2*0.87f_y$

It is necessary to have the resistance force to be developed by τ_{bd} *for the length L_d to overcome the tensile force.*

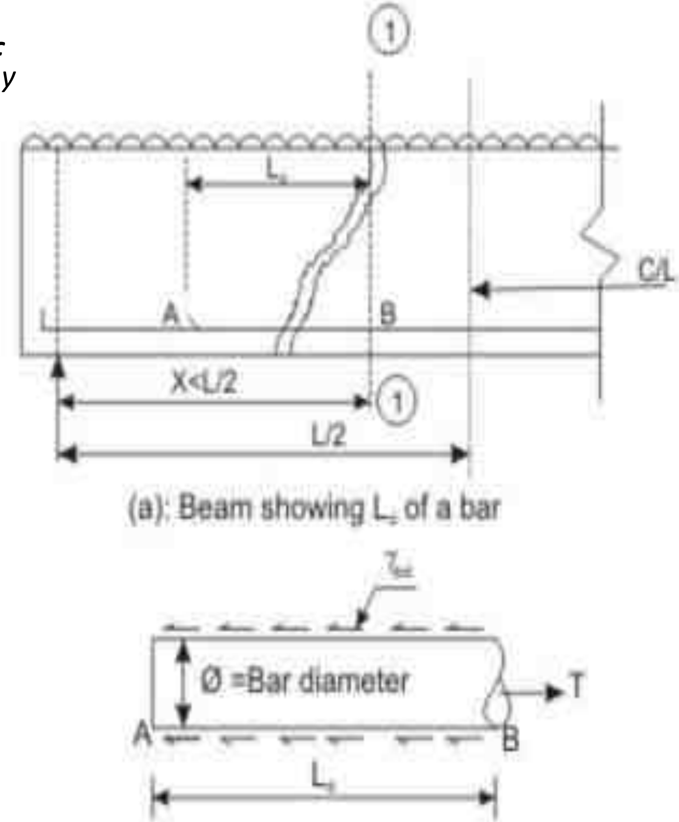
The resistance force is developed on the periphry of the rod .

Area of this bond stress acting over a length of L_d= $\pi * \phi *L_d$

Bond force over this length Ld= $\pi * \phi *L_d *\tau_{bd}$ Bond force must be greater than tension T

$$\pi * \varphi * L_d * \tau_{bd} \geq \frac{\pi}{4} * \varphi^2 * 0.87 f_y$$

$$L_d \geq \frac{0.87 f_y * \varphi}{4 * \tau_{bd}}$$

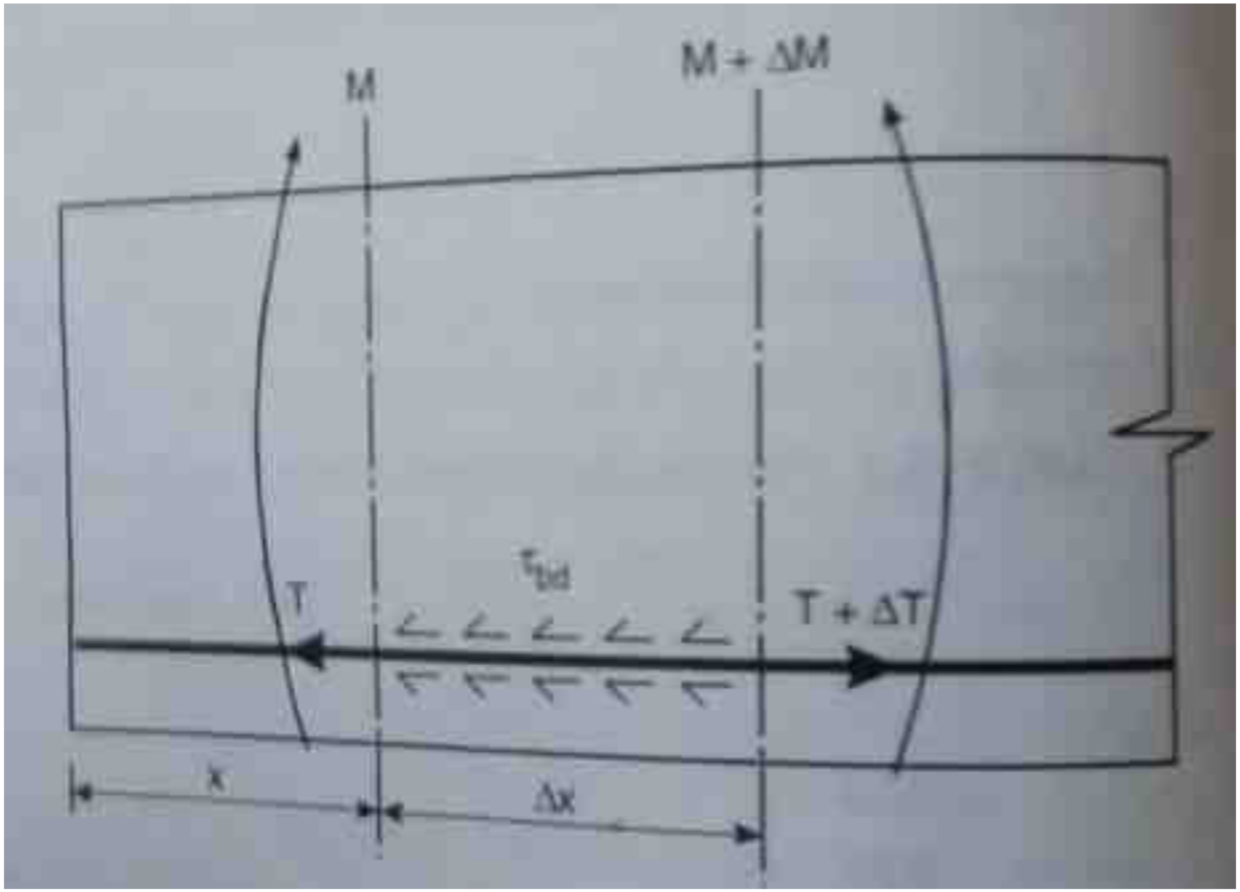


(a): Beam showing L_d of a bar

(b): Free body diagram of segment AB

FLEXURAL BOND

- Flexural bond is also known as local bond.
- Local bond at a point is the rate of change of tension in Steel at a given location.
- In a simple beam, at the critical section i.e. at the face of support, at the points of inflection and at the points of high shear force, high bond stress ay develop due to the large variations in bending moment. These bond stresses are called flexural bond stresses and should be checked carefully at all critical sections.
- Consider a beam subjected to flexural loading. Consider two sections at a distance of χ *and* $\chi+\Delta\chi$ along the length of the beam subjected to a moment M and $M + \Delta M$ respectively.
- Let T and T+ ΔT *is the tension developed ther*


M
M + ΔM
τbd
T
T + ΔT
x
Δx

FLEXURAL BOND

- It z is the lever arm, T=M/z
- T+ *ΔT =(M + ΔM)/z*
- *ΔT= ΔM /z*
- *ΔT must be registered by bond stress.*
- $\Delta T = \pi * \phi * \Delta x * \tau_{bd} = \Delta M / z$

$$\tau_{bd} = \frac{\Delta M}{z} * \left(\frac{1}{\pi * \varphi * \Delta x}\right) \qquad \tau_{bd} = \frac{\Delta M}{\Delta x} * \left(\frac{1}{\pi * \varphi * z}\right)$$

$$\tau_{bd} = \left(\frac{V}{\pi * \varphi * z}\right) \qquad \tau_{bd} = \frac{V}{(\sum O) z}$$

$\sum O$ *represents total perimeter*

- From anchorage bond we have

$$L_d \geq \frac{0.87 f_y * \varphi}{4 * \tau_{bd}} \qquad \tau_{bd} = \left(\frac{0.87 * f_y * A_{st}}{(\sum O) * L_d}\right) \qquad \tau_{bd} = \frac{V}{(\sum O) z}$$

- Equating both we will get

$$L_d = \left(\frac{0.87 * f_y * A_{st}}{V}\right) z = \frac{M_1}{V} \qquad \tau_{bd} = \left(\frac{0.87 * f_y * A_{st}}{(\sum O) * L_d}\right) = \frac{V}{(\sum O) z}$$

- Where M_1 is the moment of Resistance considering all bars there
- As per IS 456 $$L_d \leq \frac{M_1}{V} + L_0$$
- L_0 is the sum of anchorage beyond the centre of support and equivalent anchorage value of any hook or any mechanical anchorage at the support.
- This additional length is provided for safety.
- L_o is limited to the effective depth of the member or 12 ϕ which ever is greater.
- L_d is called as the development length. It is the minimum length of the bar which must be embedded in the concrete beyond any section to develop its full strength. This is also called as anchorage length in case of axial tension or axial compression and development length in case of flexural tension or flexural compression.

- To get the L_d value satisfactory,
  - Decrease bar dia,
  - Increase L_0
  - By reducing bent up bars

Checking of L_d value

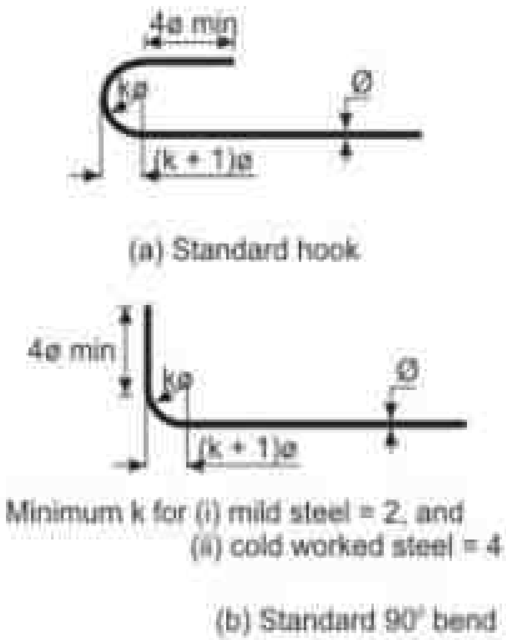
- At simple supports
- At Cantilever Support
- At point of Contra flexure
- At bar cut off points

ANCHORING OF BARS

- 25.2.2 ANCHORING REINFORCING BARS 25.2.2.1 Any deficiency in the required development length can be made up by anchoring the reinforcing bars suitably. Deformed bars have superior bond properties owing to mechanical bearing and, therefore, provisiabsolutely essential. However, plain bars should preferably end in hooks, as there may be some uncertainty regarding the full mobilization of bond strength through adhesion and friction. on of hooks is not

Anchoring Reinforcing Bars

(a) Bars in tension (cl. 26.2.2.1 of IS 456)



The salient points are:

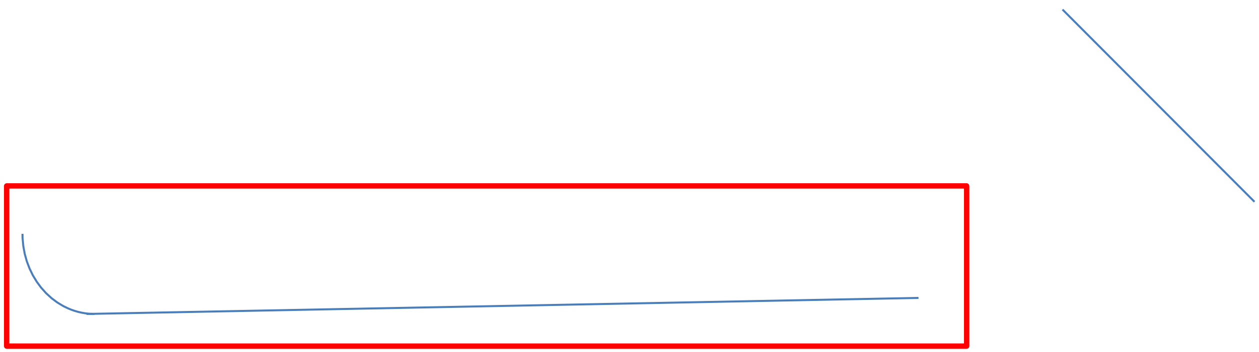
• Deformed bars may not need end anchorages if the development length requirement is satisfied.

• Hooks should normally be provided for plain bars in tension.

• Standard hooks and bends should be as per IS 2502 or as given in Table 67 of SP-16, which are shown in Figs.

• The anchorage value of standard bend shall be considered as 4 times the diameter of the bar for each 45^{o} bend subject to a maximum value of 16 times the diameter of the bar.

• The anchorage value of standard U-type hook shall be 16 times the diameter of the bar.



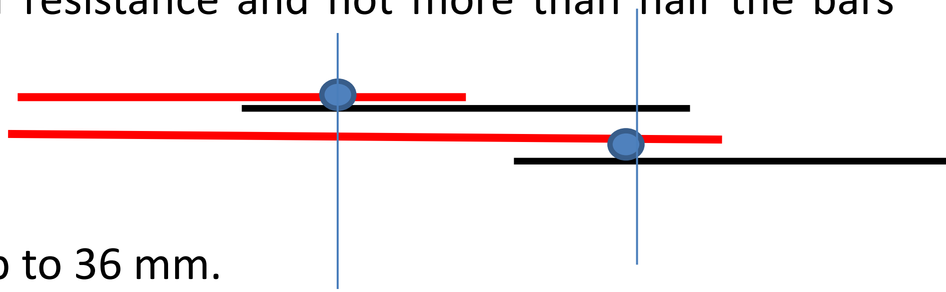
BARS IN COMPRESSION

- The anchorage length of straight compression bars shall be equal to its development length as mentioned.

- The development length shall include the projected length of hooks, bends and straight lengths beyond bends, if provided.

Reinforcement Splicing (cl. 26.2.5 of IS 456)

- Reinforcement is needed to be joined to make it longer by overlapping sufficient length or by welding to develop its full design bond stress. They should be away from the sections of maximum stress and be staggered. IS 456 (cl. 26.2.5) recommends that splices in flexural members should not be at sections where the bending moment is more than 50 per cent of the moment of resistance and not more than half the bars shall be spliced at a section.



- **(a) Lap Splices (cl. 26.2.5.1 of IS 456)**
- The following are the salient points:
- • They should be used for bar diameters up to 36 mm.
- • They should be considered as staggered if the centre to centre distance of the splices is at least 1.3 times the lap length calculated as mentioned below.
- • The lap length including anchorage value of hooks for bars in flexural tension shall be *L_d or 30φ, whichever is greater. The same for direct tension shall be $2L_d$ or 30φ, whichever is greater.*
- • The lap length in compression shall be equal to *L_d in compression but not less than 24φ.*
- • The lap length shall be calculated on the basis of diameter of the smaller bar when bars of two different diameters are to be spliced.
- • Lap splices of bundled bars shall be made by splicing one bar at a time and all such individual splices within a bundle shall be staggered.

- **(b) Strength of Welds (cl. 26.2.5.2 of IS 456)**
- The strength of welded splices and mechanical connections shall be taken as 100 per cent of the design strength of joined bars for compression splices.
- For tension splices, such strength of welded bars shall be taken as 80 per cent of the design strength of welded bars. However, it can go even up to 100 per cent if welding is strictly supervised and if at any cross-section of the member not more than 20 per cent of the tensile reinforcement is welded. For mechanical connection of tension splice, 100 per cent of design strength of mechanical connection shall be taken.

26.2.3.1 For curtailment, reinforcement shall extend beyond the point at which it is no longer required to resist flexure for a distance equal to the effective depth of the member or 12 times the bar diameter, whichever is greater except at simple support or end of cantilever. In addition **26.2.3.2** to **26.2.3.5** shall also be satisfied.

NOTE—A point at which reinforcement is no longer required to resist flexure is where the resistance moment of the section, considering only the continuing bars, is equal to the design moment.

26.2.3.2 Flexural reinforcement shall not be terminated in a tension zone unless any one of the following conditions is satisfied:

a) The shear at the cut-off point does not exceed two-thirds that permitted, including the shear strength of web reinforcement provided.

b) Stirrup area in excess of that required for shear and torsion is provided along each terminated bar over a distance from the cut-off point equal to three-fourths the effective depth of the member. The excess stirrup area shall be not less than 0.4 bs/f_y, where b is the breadth of beam, s is the spacing and f_y is the characteristic strength of reinforcement in N/mm^2. The resulting spacing shall not exceed $d/8\ \beta_b$ where β_b is the ratio of the area of bars cut-off to the total area of bars at the section, and d is the effective depth.

c) For 36 mm and smaller bars, the continuing bars provide double the area required for flexure at the cut-off point and the shear does not exceed three-fourths that permitted.

26.2.3.3 *Positive moment reinforcement*

a) At least one-third the positive moment reinforcement in simple members and one-fourth the positive moment reinforcement in continuous members shall extend along the same face of the member into the support, to a length equal to $L_d/3$.

b) When a flexural member is part of the primary lateral load resisting system, the positive reinforcement required to be extended into the support as described in (a) shall be anchored to develop its design stress in tension at the face of the support.

c) At simple supports and at points of inflection, positive moment tension reinforcement shall be limited to a diameter such that L_d computed for f_d by **26.2.1** does not exceed

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

where

M_1 = moment of resistance of the section assuming all reinforcement at the section to be stressed to f_d;

f_d = $0.87\, f_y$ in the case of limit state design and the permissible stress σ_{st} in the case of working stress design;

V = shear force at the section due to design loads;

L_0 = sum of the anchorage beyond the centre of the support and the equivalent anchorage value of any hook or mechanical anchorage at simple support; and at a point of inflection, L_0 is limited to the effective depth of the members or 12ϕ, whichever is greater; and

ϕ = diameter of bar.

The value of M_1/V in the above expression may be increased by 30 percent when the ends of the reinforcement are confined by a compressive reaction.

26.2.3.4 *Negative moment reinforcement*

At least one-third of the total reinforcement provided for negative moment at the support shall extend beyond the point of inflection for a distance not less than the effective depth of the member of 12ϕ or one-sixteenth of the clear span whichever is greater.

26.2.3.5 *Curtailment of bundled bars*

Bars in a bundle shall terminate at different points spaced apart by not less than 40 times the bar diameter except for bundles stopping at a support.

26.3.2 *Minimum Distance Between Individual Bars*

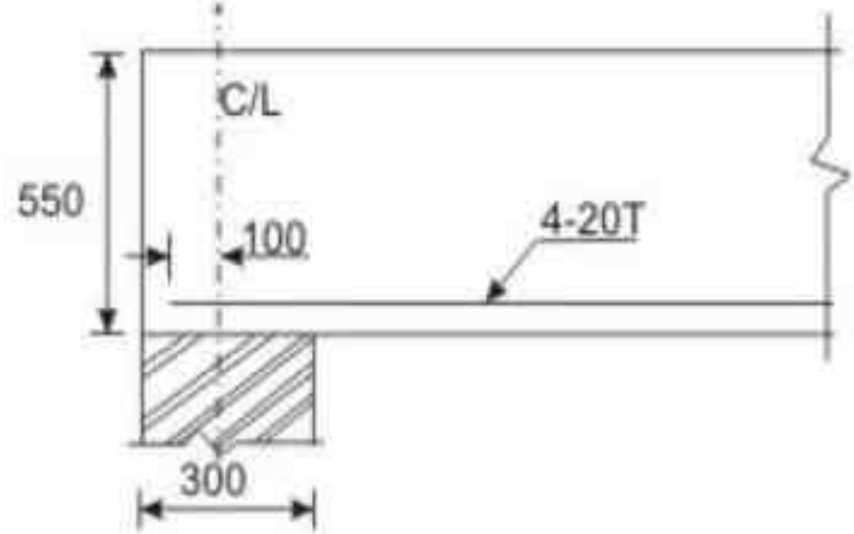
The following shall apply for spacing of bars:

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

   1) The diameter of the bar if the diameters are equal,
   2) The diameter of the larger bar if the diameters are unequal, and
   3) 5 mm more than the nominal maximum size of coarse aggregate.

   NOTE—This does not preclude the use of larger size of aggregates beyond the congested reinforcement in the same member; the size of aggregates may be reduced around congested reinforcement to comply with this provision.

PROBLEM



Determine the anchorage length of 4-20T reinforcing bars going into the support of the simply supported beam shown in Fig. The factored shear force V_u = *280 kN, width of the column support = 300 mm. Use M 20 concrete and Fe 415 steel.*

SOLUTION

- τ_{bd} *for M 20 and Fe 415 (with 60% increased) = 1.6(1.2) = 1.92 N/mm²*
- L_d*=47.01* ϕ

$$L_d \geq \frac{0.87 f_y * \varphi}{4 * \tau_{bd}}$$

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

$$(L_d)_{\text{when } \sigma_s = f_d} \leq \frac{M_1}{V} + L_o$$

Here, to find M_1, we need x_u

$$x_u = \frac{0.87\, f_y\, A_{st}}{0.36\, f_{ck}\, b} = \frac{0.87(415)\,(1256)}{0.36(20)\,(300)} = 209.94 \text{ mm}$$

$x_{u,max}$ = 0.48(500) = 240 mm

Since $x_u < x_{u,max}$; $M_1 = 0.87\, f_y\, A_{st}\,(d - 0.42\, x_u)$

or M_1 = 0.87(415) (1256) {500 – 0.42(209.94)} = 187.754 kNm

and V = 280 kN

$$L_d \leq 1.3 \left(\frac{M_1}{V}\right) + L_o$$

$$47.01\phi \leq 1.3 \left(\frac{M_1}{V}\right) + L_o$$

or $\quad 47.01\phi \leq 1.3 \left\{\frac{187.754(10^6)}{280(10^3)}\right\}$; if L_o is assumed as zero.

or $\quad \phi \leq 18.54$ mm

Therefore, 20 mm diameter bar does not allow $L_o = 0$.

Determination of L_o:

$$1.3 \left(\frac{M_1}{V}\right) + L_o \geq 47.01\phi$$

Minimum $L_o = 47.01\phi - 1.3 \left(\frac{M_1}{V}\right) = 47.01(20) - 1.3\left(\frac{187754}{280}\right) = 68.485$ mm

So the anchorage is 100 mm beyond centre of support as shown in Fig.

DESIGN OF BEAMS

G.C. BEHERA

EFFECTIVE LENGTH

22.2 Effective Span

Unless otherwise specified, the effective span of a member shall be as follows:

a) ***Simply Supported Beam or Slab*—The effective span of a member that is not built integrally with its supports shall be taken as clear span plus the effective depth of slab or beam or centre to centre of supports, whichever is less.**

- Clear distance between walls = 6m
- Thickness of one wall =300 mm
- Thickness of another wall =400 mm
- Effective depth of beam= 450 mm
- Simply supported Beam
- Find effective length of beam?
- L_{eff}=Clear span+deff=6m+.45m=6.45 m
- L_{eff}=C/C bet supports=6m+.3/2+0.4/2=6.35 m

EFFECTIVE LENGTH

- ***B)Continuous Beam or Slab -*** *In the case of* continuous beam or slab, if the width of the support is less than l/12 of the clear span, the effective span shall be as in 22.2 (a). If the supports are wider than l/12 of the clear span or 600 mm whichever is less, the effective span shall be taken as under:
- 1) For end span with one end fixed and the other continuous or for intermediate spans, the effective span shall be the clear span between supports;
- 2) For end span with one end free and the other continuous, the effective span shall be equal to the clear span plus half the effective depth of the beam or slab or the clear span plus half the width of the discontinuous support, whichever is less;
- 3) In the case of spans with roller or rocket bearings, the effective span shall always be the distance between the centres of bearings

c) **Cantilever**-The effective length of a cantilever shall betaken as its length to the face of the support plus half the effective depth except where it forms the end of a continuous beam where the length to the centre of support shall be taken.

- D) **F*rames-*** *In the analysis of a continuous frame,* centre to centre distance shall be used.

SHEAR AND BM COEFFICIENTS

Table 12 Bending Moment Coefficients
(*Clause* 22.5.1)

Type of Load	Span Moments		Support Moments	
	Near Middle of End Span	At Middle of Interior Span	At Support Next to the End Support	At Other Interior Supports
(1)	(2)	(3)	(4)	(5)
Dead load and imposed load (fixed)	$+\frac{1}{12}$	$+\frac{1}{16}$	$-\frac{1}{10}$	$-\frac{1}{12}$
Imposed load (not fixed)	$+\frac{1}{10}$	$+\frac{1}{12}$	$-\frac{1}{9}$	$-\frac{1}{9}$

NOTE — For obtaining the bending moment, the coefficient shall be multiplied by the total design load and effective span.

Table 13 Shear for Coefficients
(*Clauses* 22.5.1 *and* 22.5.2)

Type of Load	At End Support	At Support Next to the End Support		At All Other Interior Supports
		Outer Side	Inner Side	
(1)	(2)	(3)	(4)	(5)
Dead load and imposed load (fixed)	0.4	0.6	0.55	0.5
Imposed load (not fixed)	0.45	0.6	0.6	0.6

NOTE — For obtaining the shear force, the coefficient shall be multiplied by the total design load.

23.2.1 The vertical deflection limits may generally be assumed to be satisfied provided that the span to depth ratios are not greater than the values obtained as below:

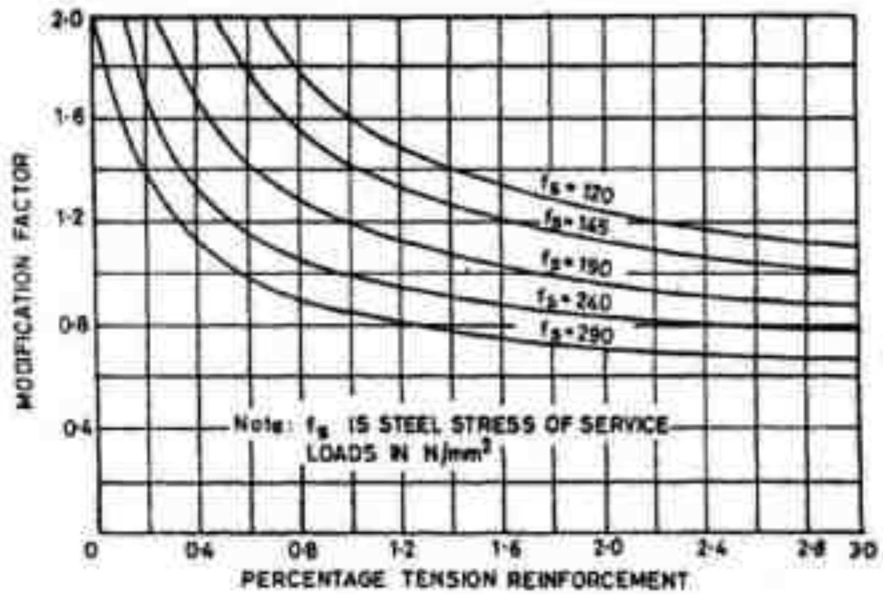
a) Basic values of span to effective depth ratios for spans up to 10 m:

Cantilever	7
Simply supported	20
Continuous	26

b) For spans above 10 m, the values in (a) may be multiplied by 10/span in metres, except for cantilever in which case deflection calculations should be made.

c) Depending on the area and the stress of steel for tension reinforcement, the values in (a) or (b) shall be modified by multiplying with the modification factor obtained as per Fig. 4.

d) Depending on the area of compression reinforcement, the value of span to depth ratio be further modified by multiplying with the modification factor obtained as per Fig. 5.



$$f_s = 0.58\, f_y \frac{\text{Area of cross-section of steel required}}{\text{Area of cross-section of steel provided}}$$

FIG. 4 MODIFICATION FACTOR FOR TENSION REINFORCEMENT

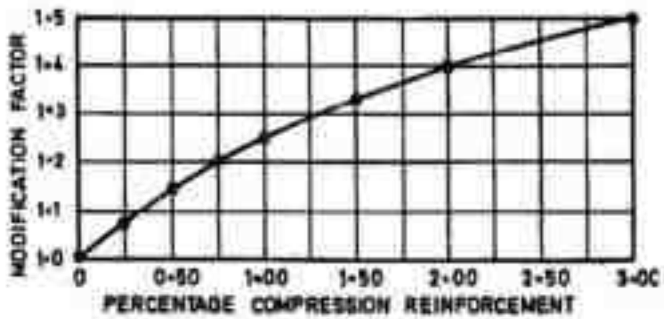
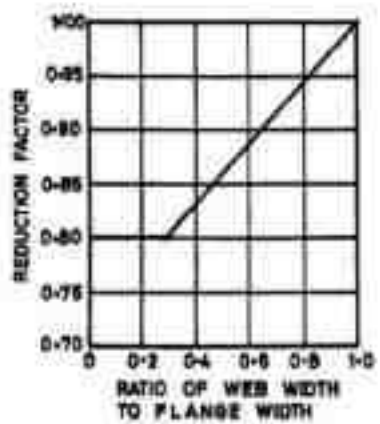


FIG. 5 MODIFICATION FACTOR FOR COMPRESSION REINFORCEMENT



26.3.3 *Maximum Distance Between Bars in Tension*

Unless the calculation of crack widths shows that a greater spacing is acceptable, the following rules shall be applied to flexural members in normal internal or external conditions of exposure.

a) *Beams* — The horizontal distance between parallel reinforcement bars, or groups, near the tension face of a beam shall not be greater than the value given in Table 15 depending on the amount of redistribution carried out in analysis and the characteristic strength of the reinforcement.

b) *Slabs*

1) The horizontal distance between parallel main reinforcement bars shall not be more than three times the effective depth of solid slab or 300 mm whichever is smaller.

2) The horizontal distance between parallel reinforcement bars provided against shrinkage and temperature shall not be more than five times the effective depth of a solid slab or 450 mm whichever is smaller.

26.4.1 *Nominal Cover*

Nominal cover is the design depth of concrete cover to all steel reinforcements, including links. It is the dimension used in design and indicated in the drawings. It shall be not less than the diameter of the bar.

26.4.2 *Nominal Cover to Meet Durability Requirement*

Minimum values for the nominal cover of normal-weight aggregate concrete which should be provided to all reinforcement, including links depending on the condition of exposure described in **8.2.3** shall be as given in Table 16.

26.4.2.1 However for a longitudinal reinforcing bar in a column nominal cover shall in any case not be less than 40 mm, or less than the diameter of such bar. In the case of columns of minimum dimension of 200 mm or under, whose reinforcing bars do not exceed 12 mm, a nominal cover of 25 mm may be used.

26.4.2.2 For footings minimum cover shall be 50 mm.

26.4.3 *Nominal Cover to Meet Specified Period of Fire Resistance*

Minimum values of nominal cover of normal-weight aggregate concrete to be provided to all reinforcement including links to meet specified period of fire resistance shall be given in Table 16A.

Table 16 Nominal Cover to Meet Durability Requirements

(*Clause* 26.4.2)

Exposure	Nominal Concrete Cover in mm not Less Than
Mild	20
Moderate	30
Severe	45
Very severe	50
Extreme	75

NOTES

1 For main reinforcement up to 12 mm diameter bar for mild exposure the nominal cover may be reduced by 5 mm.

2 Unless specified otherwise, actual concrete cover should not deviate from the required nominal cover by $^{+10}_{0}$ mm

3 For exposure condition 'severe' and 'very severe', reduction of 5 mm may be made, where concrete grade is M35 and above.

26.5.1.3 *Side face reinforcement*

Where the depth of the web in a beam exceeds 750 mm, side face reinforcement shall be provided along the two faces. The total area of such reinforcement shall be not less than 0.1 percent of the web area and shall be distributed equally on two faces at a spacing not exceeding 300 mm or web thickness whichever is less.

26.5.1.4 *Transverse reinforcement in beams for shear and torsion*

The transverse reinforcement in beams shall be taken around the outer-most tension and compression bars. In T-beams and I-beams, such reinforcement shall pass around longitudinal bars located close to the outer face of the flange.

26.5.1.5 *Maximum spacing of shear reinforcement*

The maximum spacing of shear reinforcement measured along the axis of the member shall not exceed 0.75 d for vertical stirrups and d for inclined stirrups at 45°, where d is the effective depth of the section under consideration. In no case shall the spacing exceed 300 mm.

26.5.1.6 *Minimum shear reinforcement*

Minimum shear reinforcement in the form of stirrups shall be provided such that:

$$\frac{A_{sv}}{b s_v} \geq \frac{0.4}{0.87 f_y}$$

where

- A_{sv} = total cross-sectional area of stirrup legs effective in shear,
- s_v = stirrup spacing along the length of the member,
- b = breadth of the beam or breadth of the web of flanged beam, and
- f_y = characteristic strength of the stirrup reinforcement in N/mm^2 which shall not be taken greater than 415 N/mm^2.

Where the maximum shear stress calculated is less than half the permissible value and in members of minor structural importance such as lintels, this provision need not be complied with.

26.3.2 *Minimum Distance Between Individual Bars*

The following shall apply for spacing of bars:

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

   1) The diameter of the bar if the diameters are equal,

   2) The diameter of the larger bar if the diameters are unequal, and

   3) 5 mm more than the nominal maximum size of coarse aggregate.

   NOTE—This does not preclude the use of larger size of aggregates beyond the congested reinforcement in the same member; the size of aggregates may be reduced around congested reinforcement to comply with this provision.

b) Greater horizontal distance than the minimum specified in (a) 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.

c) Where there are two or more rows of bars, the bars shall be vertically in line and the minimum vertical distance between the bars shall be 15 mm, two-thirds the nominal maximum size of aggregate or the maximum size of bars, whichever is greater.

26.3.3 *Maximum Distance Between Bars in Tension*

Unless the calculation of crack widths shows that a greater spacing is acceptable, the following rules shall be applied to flexural members in normal internal or external conditions of exposure.

a) *Beams* — The horizontal distance between parallel reinforcement bars, or groups, near the tension face of a beam shall not be greater than the value given in Table 15 depending on the amount of redistribution carried out in analysis and the characteristic strength of the reinforcement.

b) *Slabs*

1) The horizontal distance between parallel main reinforcement bars shall not be more than three times the effective depth of solid slab or 300 mm whichever is smaller.

2) The horizontal distance between parallel reinforcement bars provided against shrinkage and temperature shall not be more than five times the effective depth of a solid slab or 450 mm whichever is smaller.

TENSILE REINFORCEMENT

Minimum reinforcement not less than

given by the following:

$$\frac{A_s}{bd} = \frac{0.85}{f_y}$$

where

A_s = minimum area of tension reinforcement,

b = breadth of beam or the breadth of the web of T-beam,

d = effective depth, and

f_y = characteristic strength of reinforcement in N/mm^2.

b) *Maximum reinforcement*—The maximum area of tension reinforcement shall not exceed 0.04 bD.

26.5.1.2 *Compression reinforcement*

The maximum area of compression reinforcement shall not exceed 0.04 bD. Compression reinforcement in beams shall be enclosed by stirrups for effective lateral restraint. The arrangement of stirrups shall be as specified in **26.5.3.2**.

PROBLEM

- Design a RC rectangular beam for a simply supported beam supported on two masonry walls 230 mm thick having centre to centre distance 6 mt. The beam is carrying imposed laod 15 kN/m. Design the beam with M20 and Fe 415.
- **SOLUTION:**
- *Step-1: Assumption of Dimensions*:
- Take a trial depth=L/10 to L/15 for simply supported case. (L/d ratio 20).

  Here let us take L/d=12 d=6m/12=500 mm, Take D=500 mm, $d_{effassumed}$ =450 mm

  Assume b=D/2=250 mm

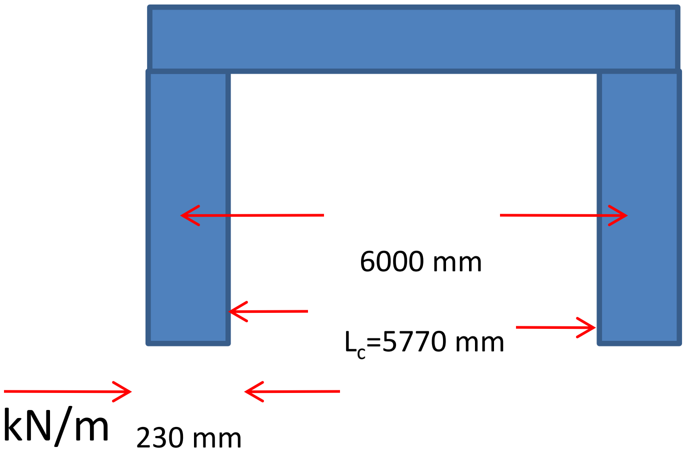
Step-2: Calculation of Effective length:

Clear Span=L_c=6 mt-0.23m=5.77 m

a) Leff=c/c distance of supports=6.0 m

b) Leff=L_c+d_{eff}= 5.77m+0.45m=6.22 m

which ever is smaller= 6.0 mt.



Step-3: Calculation load and moment:

DL of the beam=1m*0.5 m*0.25m*25=3.125 kN/m

Imposed Laod=15 kN/m

Total Load=DL+LL=3.125+15=18.125 kN/m

Factored Load=wu=18.125*1.5=27.1875=27.2 kN/m

Factored Moment=$w_u l_{eff}^2/8$= 122.36 kNm

PROBLEM

Step-3: Calculation load and moment:

Maximum SF will be calculated at a distance d_{eff} from Face of the support.

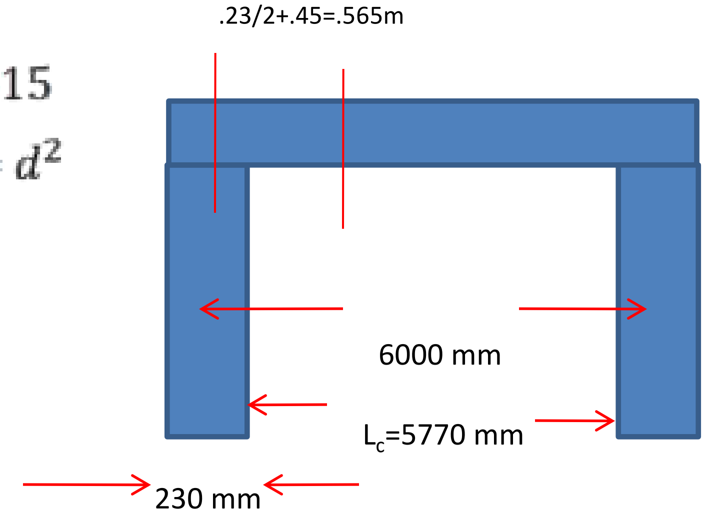
Vu= $w_u l_{eff}/2 - w_u * 565 = 66.20$ kN

Step-4: Depth Calculation:

To resist a factored bending moment 122.36 kNm making a balance section with breadth 250 mm, we required a depth

$$MOR = M_{ulim} = 0.138 * f_{ck} * bd^2 \ for\ Fe415$$

$$MOR = 122.364 * 10^6 = 0.138 * 20 * 250 * d^2$$



$d_{required}$=421.11 mm

$d_{assumed} > d_{required}$

Now we will take d=450 mm , b=250mm

Step-5: Area of Main steel:

As depth required is 421.11 mm, we have taken 450 mm, it will be designed as under-reinforced section.

$$MOR = 122.364 * 10^6$$

$$= 0.87 * 415 * A_{st} * 450\left(1 - \frac{415 * A_{st}}{20 * 250 * 450}\right)$$

A_{st}=903.80 mm^2 or 4517.885 mm^2

$$A_{st(\min)} = \frac{0.85 * bd}{f_y} = \frac{0.85 * 250 * 450}{415} = 230.42\ mm^2$$

$$A_{st(\max)} = 0.04\ bD = 0.04 * 250 * 500 = 5000\ mm^2$$

So, take A_{st} =3 nos. of 20 mm dia. Bars, $A_{st(provided)}$= 942 mm^2

Step-5: Check for Shear:

$$\tau_{vu} = \frac{V_u}{bd} = \frac{66.2 * 10^3}{250 * 450} = 0.59\ N/\ mm^2$$

$$\tau_{vu} < (\tau_{cmax} = 2.8\ N/mm^2)$$

Calculation of τ_c $\quad p_t = \frac{A_{st(provided)}}{b * d} * 100 = \frac{942 * 100}{250 * 450} = 0.837$

SOLUTION

p_t	τ_c
0.75	0.56
1.00	0.62
0.837	

Calculation of shear resistance of concrete τ_c
By interpolation

$$\tau_c = 0.56 + \frac{0.62 - 0.56}{1.0 - 0.75} * (0.837 - 0.75) = 0.58\ N/\ mm^2$$

$$\tau_{vu} > \tau_c, so\ provide\ shear\ reinforcement$$

So, provide shear reinforcement

$$V_c = Shear\ force\ \ taken\ by\ Concrete = \tau_c * b * d$$
$$= 0.58 * 250 * 450 = 65.34\ kN$$
$$V_{su} = Shear\ force\ \ to\ be\ taken\ by\ stirrup\ and\ bent\ up$$
$$= V_u - V_c = 66.2 - 65.34 = 0.85\ kN$$

As V_{su} is less , let us take 2 legged 8 mm dia. Bars as stirrup

$$s_v = \frac{0.87 * f_y * A_{sv} * d}{V_{su}} = \frac{0.87 * 415 * 100.54 * 450}{850}$$
$$= 19217.3\ mm$$

- Maximum spacing as per minimum reinforcement

$$s_v = \frac{0.87 * f_y * A_{sv}}{0.4 * b} = \frac{0.87 * 415 * 100.54}{0.4 * 250} = 364\,mm$$

- Spacing should be least of the following
- a) 0.75d=0.75*450=337.5 mm
- b) 300 mm
- So, provide 2 legged 8 mm dia. Stirrups 300 mm c/c throughout the length of the beam.

Step-6: Check for development length:

$$M_1 = 0.87 f_y * A_{st} * d\left(1 - \frac{f_y * A_{st}}{f_{ck} * b * d}\right) = 126.46\,kNm$$

Vu= $w_u l_{eff}/2 - w_u * 565 = 66.20$ kN

$$L_d = \frac{0.87 f_y * \varphi}{4 * \tau_{bd}} = \frac{0.87 * 415 * 20}{4 * 1.6 * 1.2} = 1193.94\,mm$$

Taking no anchorage length L_0=0

$$\{L_d = 1193.94\} \leq [\left(\frac{M_1}{V_u} + L_0\right) = (\frac{126.45 * 10^6}{66200} + 0)$$

$$= 1910.23\,mm]$$ Condition is satisfied

- *Step-7: Check for deflection*:

$$\frac{l_{eff}}{d_{eff}} < K * m_l * m_t * m_c * m_f$$

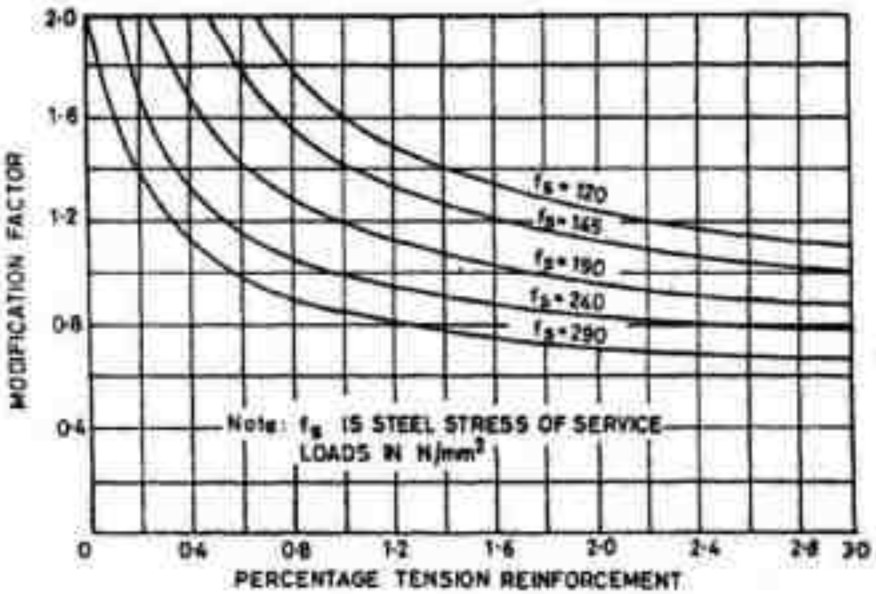
- For simply supported beam less than 10 m, k=20
- m_l= modification factor for length if l> 10 m
- m_t= modification factor for tensile reinforcement
- m_c= modification factor for compressive reinforcement
- m_f= modification factor for flanged beam

Here as length is < 10 m, do not consider $m_{l,}$ it is not a flanged section and also there is no compressive reinforcement.

Calculation of m_t: p_t=0.837

$f_s=0.58*f_y*(A_{st\ required}/ A_{st\ provided})=0.58*415*(903.8/ 942)=230.93$

P_t=0.837, f_s=230.93



$$f_s = 0.58 f_y \frac{\text{Area of cross-section of steel required}}{\text{Area of cross-section of steel provided}}$$

FIG. 4 MODIFICATION FACTOR FOR TENSION REINFORCEMENT

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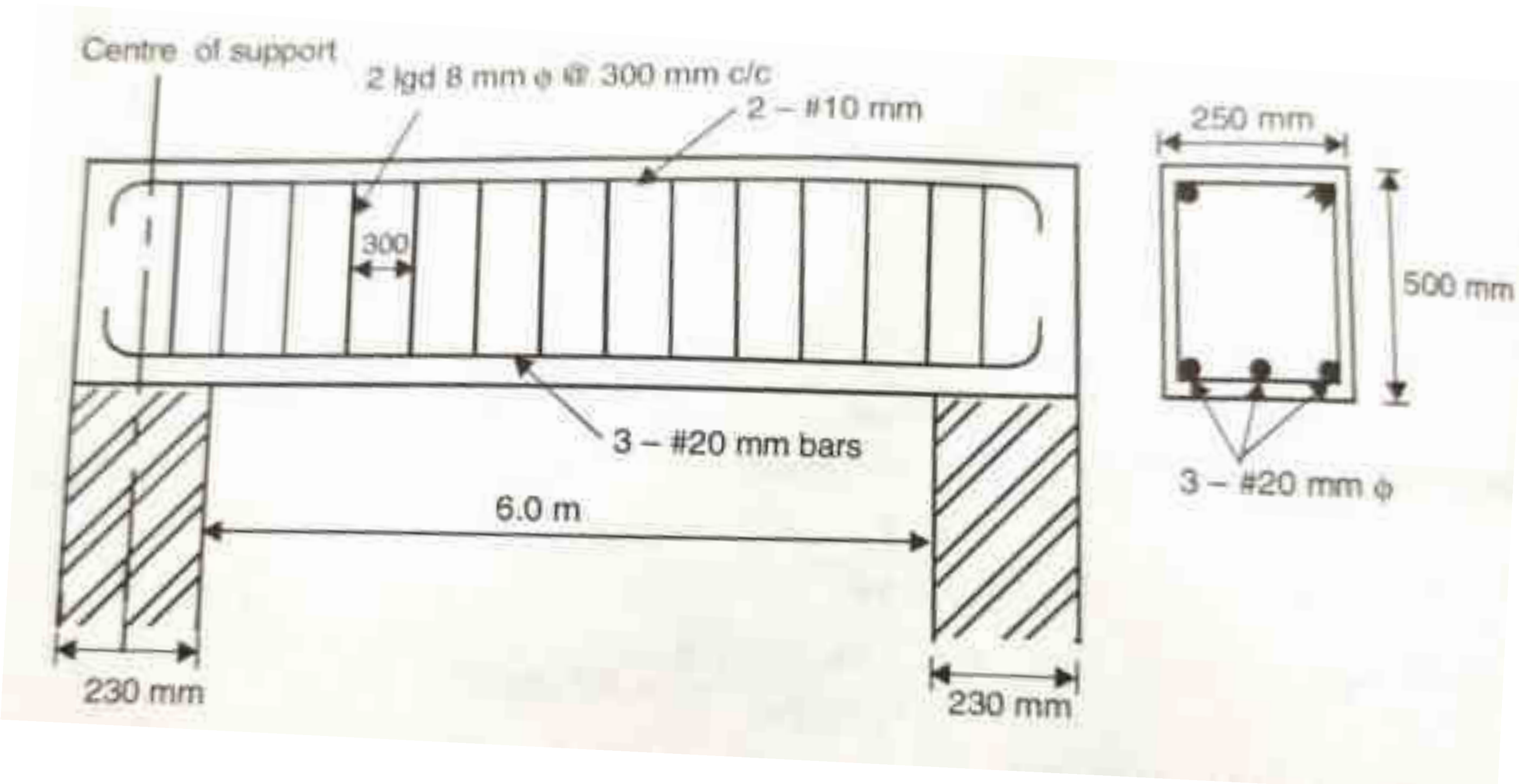
m_t=1.05,

l_{eff}/d_{eff}=6000/450=13.33

$$\left(\frac{l_{eff}}{d_{eff}} = 13.33\right) < K * m_t = 20 * 1.05 = 21, \qquad so, OK$$

Longitudinal and cross section of Beam



CLASS-10 TORSION

G.C. BEHERA

TORSION

- 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. Space frames, inverted *L*-beams as in supporting sunshades and canopies, beams curved in plan, edge beams of slabs are some of the examples where torsional moments are also present.
- Skew bending theory, space-truss analogy and Nadai's sand heap theory are some of the theories developed to understand the behaviour of reinforced concrete under torsion combined with bending moment and shear.

TORSION

- These torsional moments are of two types:
- (i) Primary or equilibrium torsion, and
- (ii) Secondary or compatibility torsion.
- The primary torsion is required for the basic static equilibrium of most of the statically determinate structures. Accordingly, this torsional moment must be considered in the design as it is a major component.
- The secondary torsion is required to satisfy the compatibility condition between members. However, statically indeterminate structures may have any of the two types of torsions.
- Minor torsional effects may be ignored in statically indeterminate structures due to the advantage of having more than one load path for the distribution of loads to maintain the equilibrium. This may produce minor cracks without causing failure. However, torsional moments should be taken into account in the statically indeterminate structures if they are of equilibrium type and where the torsional stiffness of the members has been considered in the structural analysis. It is worth mentioning that torsion must be considered in structures subjected to unsymmetrical loadings about axes.

TORSION

- Clause 41 of IS 456 stipulates the above stating that, "In structures, where torsion is required to maintain equilibrium, members shall be designed for torsion in accordance with 41.2, 41.3 and 41.4. However, for such indeterminate structures where torsion can be eliminated by releasing redundant restraints, no specific design for torsion is necessary, provided torsional stiffness is neglected in the calculation of internal forces. Adequate control of any torsional cracking is provided by the shear reinforcement as per cl. 40".

ANALYSIS FOR TORSION

- The exact analysis of reinforced concrete members subjected to torsional moments combined with bending moments and shear forces is beyond the scope here. However, the codal provisions of designing such members are discussed below.
- Approach of Design for Combined Bending, Shear and Torsion as per IS 456
- As per the stipulations of IS 456, the longitudinal and transverse reinforcements are determined taking into account the combined effects of bending moment, shear force and torsional moment.
- Two empirical relations of equivalent shear and equivalent bending moment are given.
- These fictitious shear force and bending moment, designated as equivalent shear and equivalent bending moment, are separate functions of actual shear and torsion, and actual actual bending moment and torsion, respectively.

ANALYSIS FOR TORSION

- a) The equivalent shear, a function of the actual shear and torsional moment is determined from the following empirical relation:
- $V_e = V_u + 1.6(T_u/b)$
- where V_e = equivalent shear,
- V_u = actual shear,
- T_u = actual torsional moment,
- b = breadth of beam

$$\tau_{ve} = \frac{V_e}{bd} \qquad \tau_{ve} \leq \tau_{cmax}$$

ANALYSIS FOR TORSION

- If $\tau_{ve} \leq \tau_c$, then provide the minimum stirrup.
- If $\tau_{ve} > \tau_c$, then provide both longitudinal and transverse steel.
- Cl. 41.4.2 *Longitudinal Reinforcement*
- The longitudinal reinforcement shall be designed to resist an equivalent bending moment, M_{e1}, given by

$$M_{e1} = M_u + M_t$$

- Mu is the factored BM ,
- M_t is the equivalent moment due to torsion. $M_t = T_u(\frac{1 + D/b}{1.7})$
- T_u is the factored Torsion, D is the overall depth, b is the width of the beam.

- Cl.41.4.2.1
- *If the numerical value of* M_t*, as defined in 41.4.2 exceeds the numerical value of the moment Mu, longitudinal reinforcement shall be provided on the flexural compression face, such that the beam can also withstand an equivalent* M_{e2} *given by*
- $M_{e2} = M_t - M_u$*, the moment* M_{e2}*, being taken as acting in the opposite sense to the moment* M_u*.*

DESIGN FOR TORSION

- Cl.41.4.3 *Transverse Reinforcement*
- Two legged closed hoops enclosing the corner longitudinal bars shall have an area of cross-section A_{sv}

$$A_{sv} = \frac{T_u s_v}{b_1 d_1 (0.87 f_y)} + \frac{V_u s_v}{2.5 d_1 (0.87 f_y)}$$

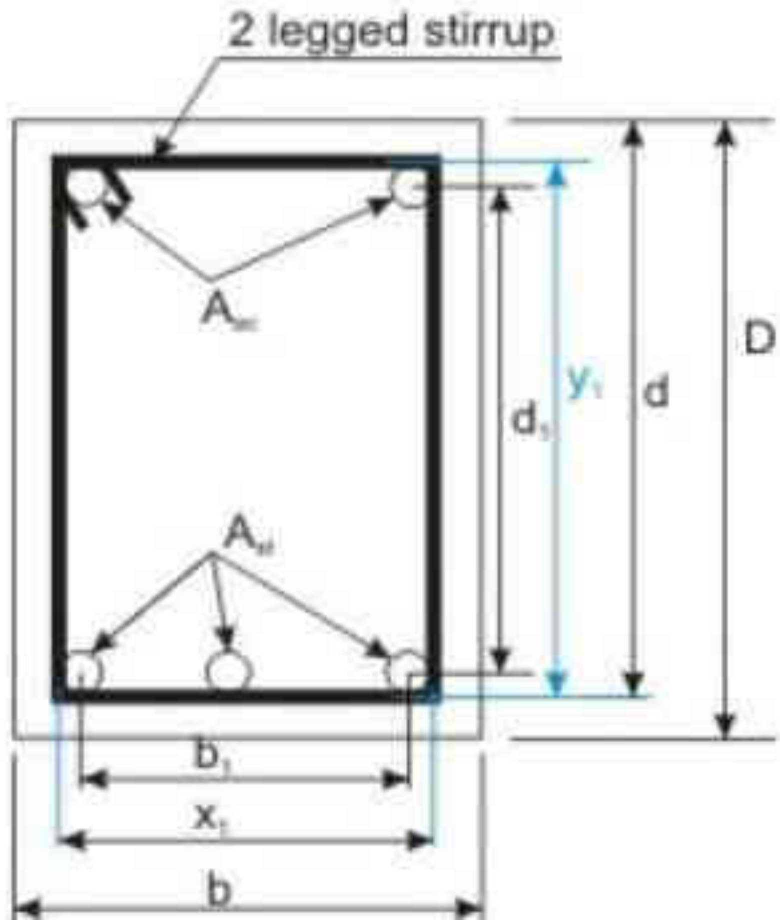
- *How ever total transverse reinforcement should not be less than*

$$A_{sv} \geq (\tau_{ve} - \tau_c)\, b\, s_v / (0.87 f_y)$$

- T_u=torsional moment
- V_u= *Factored SF*
- S_v=spacing of the stirrup reinforcement,
- b_1=centre-to-centre distance between corner bars in the direction of the width,
- d_1=centre-to-centre distance between comer in the direction of the depth
- b=breadth of the member,
- f_y=characteristic strength of the stirrup reinforcement

The transverse reinforcement shall consist of rectangular close stirrups placed perpendicular to the axis of the member. The spacing of stirrups shall not be more than the least of x_1, $(x_1 + y_1)/4$ *and 300 mm, where* x_1 *and* y_1 *are the short and long dimensions of the stirrups*

Longitudinal reinforcements should be placed as close as possible to the corners of the cross-section.



PROBLEM

Determine the reinforcement required of a ring beam of b = 400 mm, d = 650 mm, D = 700 mm and subjected to factored Mu = 200 kNm, factored Tu = 50 kNm and factored Vu = 100 kN. Use M 20 and Fe 415 for the design.

Step 1: Check for the depth of the beam

Calculation of equivalent shear force due to torsion moment

V_{et}=1.6(T_u/b)=1.6*(50/.04)=200 kN

V_e=V_u+V_{et}=100+200=300kN

Equivalent shear stress=τ_{ve}=V_e/bd=300*1000/(400*600)=1.154 N/mm^2

τ_{cmax} for M20 concrete=Table 20 IS-456 2.8N/mm^2

τ_{cmax}> τ_{ve}, No redesign is required.

- Step 2: Longitudinal tension reinforcement
- Calculation of Bending Moment
- M_t=Equivalent bending moment due to torsion= $(T_u/1.7)\ \{1 + (D/b)\} = (50/1.7)\ \{1 + (700/400)\} = 80.88$ kNm
- $M_{e1} = M_u + M_t = 200 + 80.88 = 280.88$ kNm

As $M_u > M_t$, M_{e2} not required.

$M_{e1}/bd^2 = (280.88)(10^6)/(400)(650)(650) = 1.66\ N/mm^2$

From Table 2 of SP-16, corresponding to

$Mu/bd^2 = 1.66\ N/mm^2$, $p_t = 0.5156$.

$A_{st} = p_t*b*d/100 = 0.5156(400)(650)/100 = 1340.56\ mm^2$. Provide 2-25T and 2-16T = 981 + 402 = 1383 mm^2 .

p_t provided=$A_{st}*100/bd=1383*100/(400*650)= 0.532$

$A_{stmin}=0.85*bd/fy=0.85*400*650/415=532.53$ mm2

$A_{stmax}=0.04bD=0.04*400*700=11200$ mm2

$(A_{stmin}=532.53)<(A_{stprovided}=1383)<(A_{stmax}=11200)$, So,ok

Step 3: Longitudinal compressive reinforcement

As $M_t<M_u$, No compressive reinforcement is required.

Step 4: Longitudinal side face reinforcement

IS-456-CL-26.5.1.3 Side face reinforcement Where the depth of the web in a beam exceeds 750 mm, side face reinforcement shall be provided along the two faces. The total area of such reinforcement shall be not less than 0.1 percent of the web area and shall be distributed equally on two faces at a spacing not exceeding 300 mm or web thickness whichever is less.

Side face reinforcement shall be provided as the depth of the beam exceeds 450 mm.

Area of side face reinforcement=0.1*bd/100=0.1*400*650/100=260 mm^2,

Providing 2-10 mm diameter bars (area = 157 mm2) on each face the total area for two faces will be 157*2=314 mm^2. Hence o.k.

Step 5: TRANSVERSE REINFORCEMENT

Calculation of τ_c

For pt=0.532,

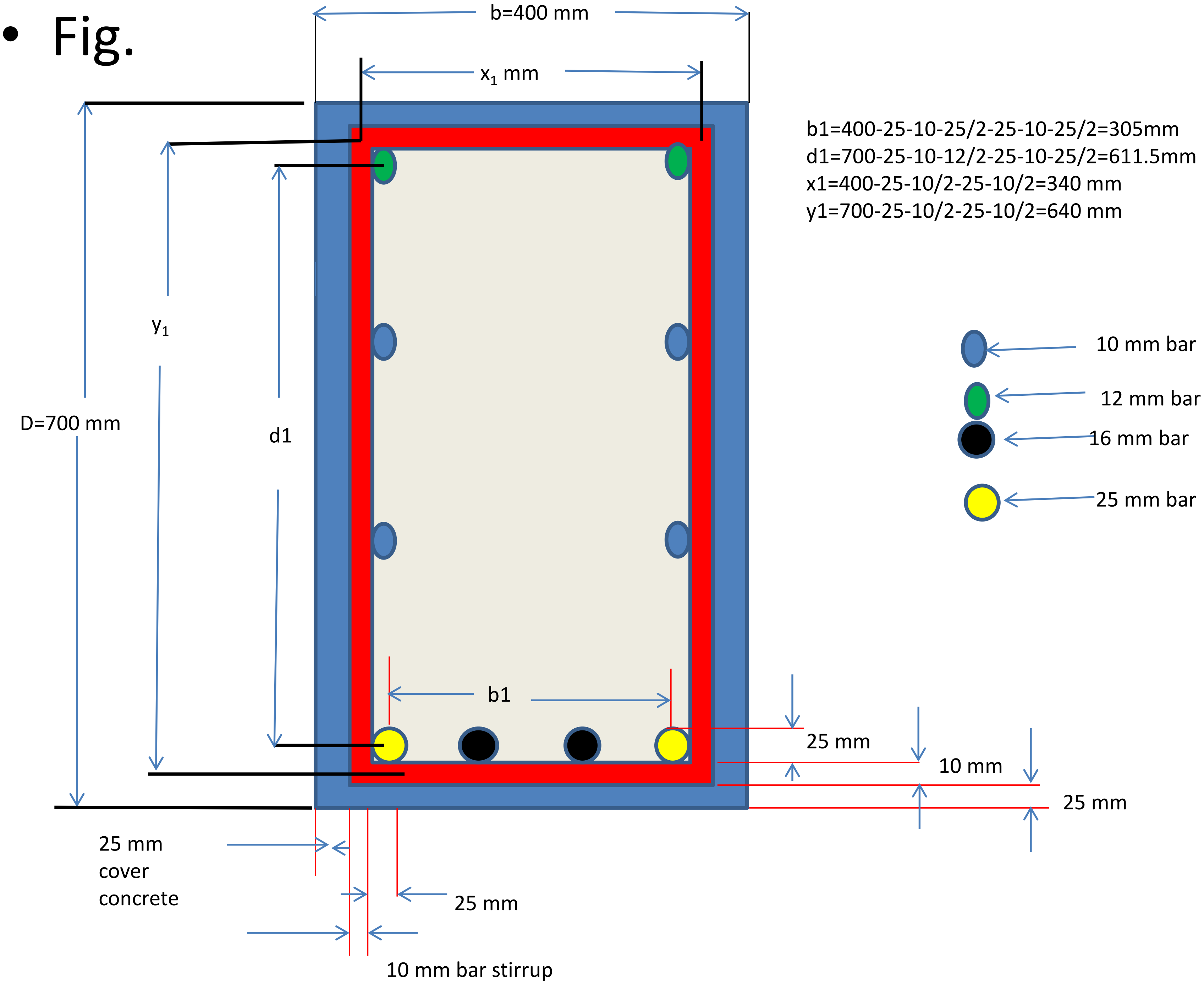
τ_c =0.48+(0.56-0.48)*(0.532-0.5)/(0.75-0.5)=0.488 N/mm2

Table 19 Design Shear Strength of Concrete, τ_c, N/mm²

(*Clauses* 40.2.1, 40.2.2, 40.3, 40.4, 40.5.3, 41.3.2, 41.3.3 *and* 41.4.3)

$100 \frac{A_s}{bd}$	Concrete Grade					
	M 15	M 20	M 25	M 30	M 35	M 40 and above
(1)	(2)	(3)	(4)	(5)	(6)	(7)
≤ 0.15	0.28	0.28	0.29	0.29	0.29	0.30
0.25	0.35	0.36	0.36	0.37	0.37	0.38
0.50	0.46	0.48	0.49	0.50	0.50	0.51
0.75	0.54	0.56	0.57	0.59	0.59	0.60
1.00	0.60	0.62	0.64	0.66	0.67	0.68
1.25	0.64	0.67	0.70	0.71	0.73	0.74
1.50	0.68	0.72	0.74	0.76	0.78	0.79
1.75	0.71	0.75	0.78	0.80	0.82	0.84
2.00	0.71	0.79	0.82	0.84	0.86	0.88
2.25	0.71	0.81	0.85	0.88	0.90	0.92
2.50	0.71	0.82	0.88	0.91	0.93	0.95
2.75	0.71	0.82	0.90	0.94	0.96	0.98
3.00 and above	0.71	0.82	0.92	0.96	0.99	1.01

• Fig.


b=400 mm
x1 mm
b1=400-25-10-25/2-25-10-25/2=305mm
d1=700-25-10-12/2-25-10-25/2=611.5mm
x1=400-25-10/2-25-10/2=340 mm
y1=700-25-10/2-25-10/2=640 mm
y1
D=700 mm
d1
10 mm bar
12 mm bar
16 mm bar
25 mm bar
b1
25 mm
10 mm
25 mm
25 mm
cover
concrete
25 mm
10 mm bar stirrup

Using 2 legged 10 mm dia bars

$$A_{sv} = \frac{T_u s_v}{b_1 d_1 (0.87 f_y)} + \frac{V_u s_v}{2.5 d_1 (0.87 f_y)}$$

$$157 = \frac{50 * 10^6 * s_v}{305 * 611.5 * \ (0.87 * 415)} + \frac{100 * 10^3 * s_v}{2.5 * 611.5(0.87 * 415)}$$

s_v=169.97 mm
Provide 2 legged 10 mm dia bars 150 mmc/c

Again $A_{sv} \geq (\tau_{ve} - \tau_c) b * s_v / (0.87 * f_y)$

$A_{sv} \geq [(\tau_{ve} - \tau_c) b * s_v / (0.87 * f_y) = \{(1.154-0.488)*400*150/(0.87*415)\} = 110.67\ mm^2]$

$[A_{sv}=157] \geq 110.67\ mm^2$
Design is ok.
Spacing can not be greater than x1=340 mm
Spacing can not be greater than(x1+y1)/4=(340+640)/4=245 mm
Spacing can not be greater than 300 mm
Stirrup spacing provided 150 mm satisfies all these three conditions.