CHAPTER-1 -

INTRODUCTION TO DESIGN AND DETAILING

Objectives of Design and Detailing

Every structure must be **designed** to satisfy three basic requirements;

1) **Stability** to prevent over turning, sliding or buckling of the structure, or parts of it, under the action of loads;
2) **Strengths** to resist safely the stresses inducedby the loads in the various structural members.
3) **Serviceability** to ensure satisfactory performance underservice load condition. Serviceabilityincludestwo parameters i.e deflection and cracking. The deflection should be limited to ensure the better appearance of the structure and to prevent cracking. The cracking of the reinforced concrete should not be excessive to ensure better appearance and also to prevent the access of water from cracks which may corrode there in for cement.

The rearetwootherconsiderationsthata sensibledesigneroughttobearinmind, viz.**economy**and **aesthetics**

A good structural design often involving elaborate computations is a worthwhile exercise if only it isfollowed by good detailing and construction practices. In normal design practices it is often seen that analysisofstructuresforstressresultantsanddesignofindividualmembers(criticalsectionsofbeams,slabsandcolumns) for maximum load effects(bending moments, shear, torsion and axial forces) are done regularly withinsufficientattentiongiventosupposedlylesserimportantaspectse.g.termination, extendingandbendingofbars, anchorage and development, stirrup anchorage, splices, construction details at joints or connections (slab-beam, beam-column etc.), provision of continuity and discontinuity at connection of members , constructionsequencingandreinforcementplacement,deflectioncalculationsandcont rol,crackcontrol,covertoreinforcement,creep and shrinkage etc.

The factors as enumerated above are very critical from the point of view of a successful structure andneedstobefairly assessedwithsufficientaccuracy andspeltoutindetailthroughvariousdrawingsandspecificationsbythe designerso thatthe constructionofthestructure canbehandled bythe siteengineer.

.

DifferentMethodsofDesign

Over the years, various design philosophies have evolved in different parts of the world, with regard toreinforcedconcretedesign.Adesignphilosophyisbuiltuponafewfundamentalassumptionsandisreflectiveofa wayofthinking.

Working Stress Method:

The earliest codified design philosophy is that of **working stress method** of design (WSM). Close to ahundredyearsold,thistraditionalmethodofdesign,basedonlinearelastictheoryisstillsurvivinginanumber of countries. In WSM it is assumed that structural material e.g. concrete and steel behave in linearly elasticmanner and adequate safety can be ensured by restricting the stresses in the material induced by working loads(service loads) on the structure. As the specified permissible (allowable) stresses are kept well below thematerialstrength,theassumptionoflinearelasticbehaviorconsideredjustifiable.Theratioof thestrengthofthematerialtothepermissiblestressisoftenreferredtoasthefactorofsafety.Whil eapplyingWSMthestressesunderappliedloadsareanalysedby‘simplebendingtheory’wherest raincompatibilityisassumed(dueto bondbetween concrete andsteel).

UltimateLoadMethod:

With the growing realization of the shortcomings of WSM in reinforced concrete design, and withincreased understanding of the behavior of reinforced concrete at *ultimate loads*, the ultimate load method ofdesign (ULM) evolved in the 1950s and became an alternative to WSM. This methodissometimes alsoreferredto asthe*load factormethod*orthe*ultimate strengthmethod.*

In thismethod, the stress condition at the state ofimpending collapse of the structure is analysed, andthe nonlinear stress-strain curve of concrete and steel are made use of the concept of ‘modular ratio’ and itsassociated problems are avoided. The safety measure in thedesign is introduced by an appropriate choice oftheloadfactor, definedasthe ratioofthe ultimateload(designload)totheworkingload. Thismethod

AdvantagesOfReinforcedConcrete

Thefollowingare majoradvantagesofreinforcedcementconcrete (RCC)

- ReinforcedCementConcretehasgoodcompressivestress(becauseofconcrete).
- RCCalsohashightensilestress(becauseofsteel).
- Ithasgoodresistancetodamagebyfireandweathering(becauseofconcrete).
- RCC protectssteel barsfrombucklingandtwistingatthehightemperature.

- RCCpreventssteelfromrusting.
- ReinforcedConcreteisdurable.
- Themonolithiccharacterofreinforcedconcretegivesitmorerigidity.
- Maintenancecost ofRCCispracticallynil.

It is possible to produce steel whoseyield strength is 3 to 4 time more that of ordinary reinforced steel andto produceconcrete 4 to 5 time stronger in compression than the ordinary concrete. This may high strength material offer manyadvantagesincludingsmallermembercross-sections,reducedeadloadandlongerspan generally results in more slender sections, and often more economical design of beams and columns (comparedtoWSM), particularlywhenhigh strength reinforcing steeland concrete areused.

LimitStateMethod:

The philosophy of the limit state method of design (LSM) represents a definite advancement over thetraditional WSM (based on service load conditions alone) and ULM (based on ultimate load conditions alone).LSM aims for a comprehensive and rational solution to the design problem, by considering safety at ultimateloadsandserviceabilityatworkingloads.TheLSMusesamultiplesafetyfactorformatwhichattemptstoprovide adequate safety at ultimate loads as well as adequate serviceability at service loads by considering allpossible'limitstates'.

CHAPTER-2

WORKING STRESS METHOD OF DESIGN

GeneralConcept

Workingstressmethodisbasedonthebehaviorofasectionundertheloadexpected tobeencounteredby it during its service period. The strength of concrete in the tension zone ofthememberis neglectedalthough the concrete does have some strength for direct tension and flexural tension (tension due to bending).The material both concrete and steel, are assumed to behave perfectly elastically, i.e., stress is proportional tostrain.Thedistributionofstrainacross asectionisassumedtobe linear. Thesectionthatare plane before bending remain plane after bending.Thus,thestrain,hencestressatanypointisproportionaltothedistanceofthe point from the neutralaxis. Withthisa triangular stress distributioninconcrete is obtained, ranging from zero at neutral axis to amaximum at the compressive face of the section. It is further assumed in this method that the reisperfect bond between the steel and the surrounding concrete, the strain sinbothmaterialsatth at point are same and hence the ratio of stresses in steel and concrete will be the same as the ratio of elastic moduli of steel and concrete. This ratio being known as 'modular ratio', the method is also called 'Modular Ratio Method'.

In this method, external forces and moments are assumed to be resisted by the internal compressiveforcesdevelopedinconcreteandtensileresistiveforcesinsteelandtheinter nalresistivecoupleduetotheabovetwoforces,inconcreteactingthroughthecentroidoft riangulardistributionofthecompressivestressesand in steel acting at the centroid of tensile reinforcement. The distance between the lines of action of resultantresistiveforcesis known as'Leverarm'.

Moments and forces acting on the structure are computed from the service loads. The section of the component memberis proportioned to resist these moments and forces such that the maximum stresses developed in materials are restricted to a fraction of their true strengths. The factors of safety used in getting maximum permissible stressesare asfollows:

Material *Factor ofSafety*

Forconcrete3.0

Fo

rSt

eel

1.7

8

Assumptionsof WSM

TheanalysisanddesignofaRCCmemberarebasedonthefollowingassumptions.

(i) Concreteis assumedtobehomogeneous.

(ii) Atanycrosssection,planesectionsbeforebendingremainplaneafterbending.

(iii) Thestress-strainrelationshipforconcreteisastraightline,underworking loads.

(iv) Thestress-strainrelationshipforsteelisastraightline,underworkingloads.

(v) Concreteareaontensionsideisassumedtobeineffective.

(vi) Alltensilestressesaretakenupbyreinforcementsandnonebyconcreteex ceptwhenspeciallypermitted.

(vii) Thesteelareaisassumedtobeconcentratedatthecentroidofthesteel.

(viii) Themodularratiohasthevalue280/3σ_{cbc}whereσ_{cbc}ispermissiblestressi ncompressionduetobendinginconcreteinN/mm^{2}asspecifiedincode(IS: 456-2000)

MomentofResistance

(a) *For Balanced section*: When the maximum stresses in steel and concrete simultaneously reach theirallowable values, the section is said to be a ‘Balanced Section’. The moment of resistance shall beprovided by the couple developed by compressive force acting at the centroid of stress diagram on thearea of concrete in compression and tensile forceb acting at the centroid of reinforcement multipliedbythedistancebetweentheseforces.Thisdistanceisknownas‘leverarm ’.

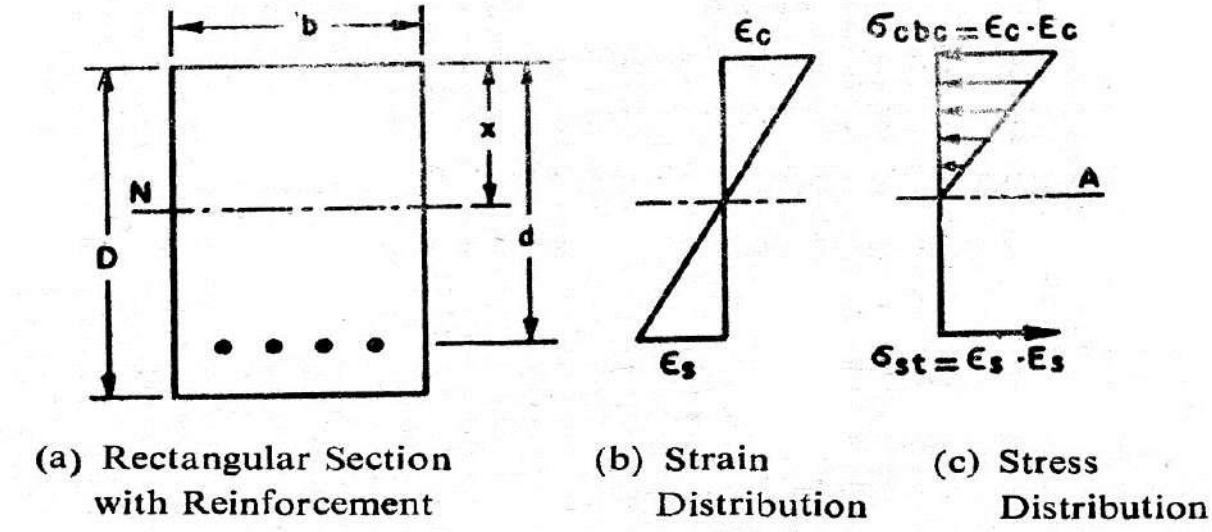

b
x
N
D
d
ϵ_c
ϵ_s
$\sigma_{cbc} = \epsilon_c \cdot E_c$
A
$\sigma_{st} = \epsilon_s \cdot E_s$
(a) Rectangular Section with Reinforcement
(b) Strain Distribution
(c) Stress Distribution

Fig.2.1(a-c)

Let in Fig.2.1(a-c): b = width of section

D = overall depth of section

d = effective depth of section (distance from extreme compression fiber to the centroid of steel area,

A_s = area of tensile steel

ϵ_c = Maximum strain in concrete,

ϵ_s = maximum strain at the centroid of the steel,

σ_{cbc} = maximum compressive stress in concrete in bending

σ_{st} = Stress in steel

E_s/E_c = ratio of Yong's modulus of elasticity of steel to concrete

= modular ratio 'm'

Since the strains in concrete and steel are proportional to their distances from the neutral axis,

$$\frac{\epsilon_c}{\epsilon_s} = \frac{x}{d-x} \text{ or } \frac{d-x}{x} = \frac{\epsilon_s}{\epsilon_c}$$

$$\frac{d}{x} - 1 = \frac{\sigma_{st}}{E_s} \cdot \frac{E_c}{\sigma_{cbc}} = \frac{\sigma_{st}}{\sigma_{cbc}} \cdot \frac{1}{m}$$

Or $\frac{d}{x} = 1 + \frac{\sigma_{st}}{m.\sigma_{cbc}}$ or $x = \frac{1}{1 + \frac{\sigma_{st}}{m.\sigma_{cbc}}} .d = k.d$

Where k = neutral axis constant = $\frac{1}{\ldots}$

Total compressive force = b.s.σcbc

2

Total tensile forces = σct

$d - \frac{x}{\ldots} = d - \frac{k.d}{\ldots} = d\left(1 - \frac{k}{\ldots}\right) = j.d$

Z = Lever arm = As

3

Where j is called the lever arm constant.

Moment of resistance $= MR = \frac{b.x}{2}.\sigma_{cbc}.jd = \frac{b.kd}{2}.\sigma_{cbc}.jd = \frac{1}{2}.\sigma_{cbc}.k.j.b.d^2 = Q.b.d^2$

Where Q is called moment of resistance constant and is equal to $\frac{1}{2}.k.j.\sigma_{cbc}$

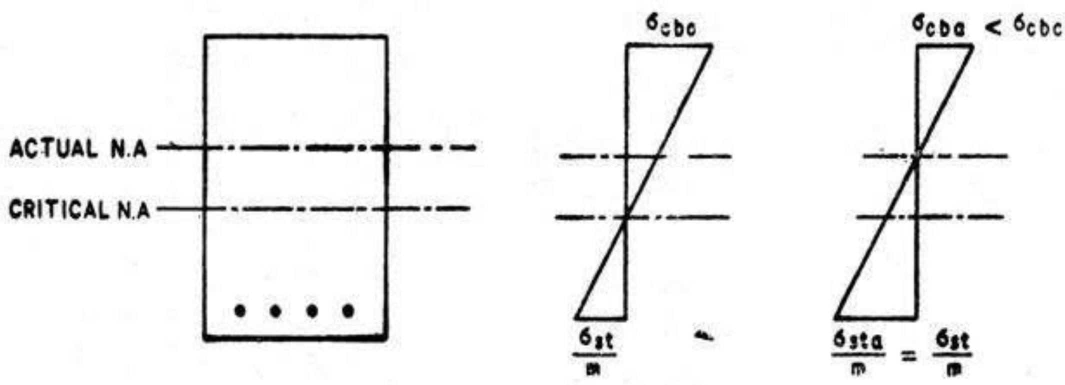
(b) Under reinforcedsection

When the percentage of steel in a section is less than that required for a balanced section, thesection is called 'Under-reinforced section.' In this case (Fig.2.2) concrete stress does not reach itsmaximum allowable value while the stress in steel reaches its maximum permissible value. The positionof the neutral axis will shift upwards, i.e., the neutral axis depth will be smaller than that in the balancedsectionasshowninFigure2.2.Themomentofresistanceofsuchasection willbegovernedbyallowable tensilestressinsteel.

Momentofresistance= $\sigma_{st}.A_s.\left(d-\frac{x'}{3}\right)=\sigma_{st}A_s.jd$ where $j=1-\frac{k'}{3}$

Since $p=\frac{A_s\times 100}{b.d}$

Momentofresistance $=\sigma_{st}.\frac{p.b.d}{100}.j'd=\frac{\sigma_{st}.p.j'}{100}.b.d^2=Q'.b.d^2$ where $Q'=\frac{\sigma_{st}.p.j'}{100}$



(a) Rectangular Section with Reinforcement (b) Strain Distribution (c) Stress Distribution

Fig.2.2(a-c)

(c)**Over reinforcedsection**:

When the percentage of steel in a section is more than that required for a balanced section, thesection is called ‘Over-reinforced section’. In this case (Fig.2.3) the stress in concrete reachesitsmaximum allowable value earlier than that in steel. As the percentage steel is more, the position of theneutral axis will shift towards steel from the critical or balanced neutral axis position.Thus the neutralaxisdepthwillbegreaterthanthatincaseofbalancedsection.

Moment of resistance of such a section will be governed by compressive stress in concrete,

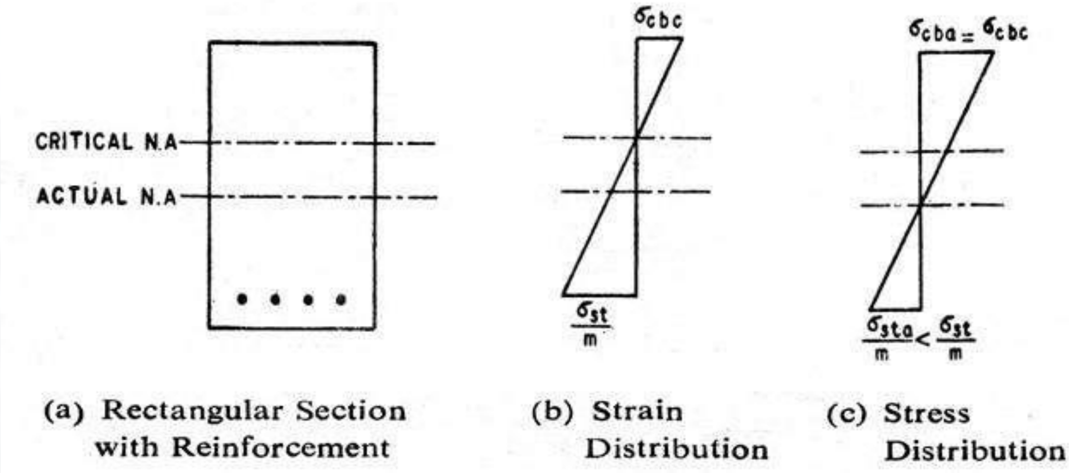


Fig.2.3(a-c)

$$\text{Moment of resistance} = \frac{1}{2}\,b.x.\sigma_{cbc}\left(d - \frac{x}{3}\right) = \frac{\sigma_{cbc}}{2}.b.x.d.\left(1 - \frac{k'}{3}\right)$$

$$= \frac{\sigma_{cbc}}{2}.b.x.d.j' = \frac{1}{2}.\sigma_{cbc}.k'.j'.b.d^2 = Q'.b.d^2 \quad \text{where } Q' = \frac{\sigma_{cbc}.k'.j'}{2} = Constant$$

Basic concept of design of single reinforced members

The following types of problems can be encountered in the design of reinforced concrete members.

(A) Determination of Area of Tensile Reinforcement

The section, bending moment to be resisted and the maximum stresses in steel and concrete are given.

Steps to be followed:

(i) Determine *k, j, Q* (or *Q′*) for the given stress.

(ii) Find the critical moment of resistance, $M = Q.b.d^2$ from the dimensions of the beam

(iii) Compare the bending moment to be resisted with M, the critical moment of resistance

(a) If B.M. is less than M, design the section as under reinforced.

$$M = \sigma_{st}.A_s.\left(d - \frac{x}{3}\right)$$

To find A_s in terms of x, take moments of areas about $N.A.$

$$b.x.\frac{x}{2} = m.A_s.(d - x)$$

$$A_s = \frac{b.x^2}{2.m.(d - x)} \qquad M = \frac{\sigma_{st}.b.x^2}{2.m.(d - x)}\left(d - \frac{x}{3}\right) = B.M. \text{ to be resisted}$$

Solvefor'x',andthen$A_s$canbecalculated.

(b) If$B.M.$ismorethan$M$,designthesectionasover-reinforced.
$M = c_{bc}.b.x.\left(d - \frac{x}{3}\right) = B.M.$toberesisted. Determine'$x$'.Then$A_s$ canbeobtainedbytaking

2 3

momentsofareas(compressiveand tensile)aboutusingthefollowing expression.

$$A_s = \frac{b.x^2}{2.m.(d - x)}$$

(B) DesignofSectionforaGivenloading

Designthesectionasbalancedsectionforthegivenloading.

Stepstobefollowed:

(i) Findthemaximumbendingmoment($B.M.$)duetogivenloading.

(ii) Computetheconstantsk,j,Qforthebalancedsectionforknownstresses.

(iii) Fixthedepthtobreadthratioofthebeamsectionas2to4.

(iv) From$M=Q.b.d^2$,find'd'andthen'b'fromdepthtobreadthratio.

(v) Obtainoveralldepth'D'byaddingconcretecoverto 'd'the effectivedepth.

(vi) Calculate$A_s$fromtherelation

$$A_s = \frac{B.M.}{\sigma_{st}.j.d}$$

(C) To DeterminetheLoadcarryingCapacityofagivenBeam

Thedimensionsofthebeamsection,thematerialstressesandareaofreinforcing steelaregiven.

Stepstobefollowed:

(i) Findthepositionoftheneutralaxisfromsectionandreinforcementgiven.

(ii) Findthepositionofthecritical$N.A.$fromknownpermissiblestressesofconcreteandsteel.

$$x = \frac{1}{\dots}.d$$

st $m.$ cbc

(iii) Check if (i) > (ii) - the section is over-reinforced

(i) < (ii) - the section is under-reinforced

(iv) Calculate M from relation

$M = b.x.\frac{1}{2}.\sigma_{cbc}.\left(d - \frac{x}{3}\right)$ for over-reinforced section and $M = \sigma_{st}.A_s.\left(d - \frac{x}{3}\right)$ for under-reinforced section.

(v) If the effective span and the support conditions of the beam are known, the load carrying capacity can be computed.

(D) To Check The Stresses Developed In Concrete And Steel

The section, reinforcement and bending moment are given.

Steps to be followed:

(i) Find the position of *N.A.* using the following relation.

$$b.\frac{x^2}{2} = m.A_s.(d - x)$$

(ii) Determine lever arm, $z = d - \frac{x}{3}$

(iii) $B.M. = \sigma_{st}.A_s.z$ is used to find out the actual stress in steel σ_{sa}.

(iv) To compute the actual stress in concrete σ_{cba}, use the following relation.

$$BM = \frac{\sigma_{cba}}{2}.b.x.z$$

Doubly Reinforced Beam Sections by Working Stress Method

Very frequently it becomes essential for a section to carry bending moment more than it can resist as a balanced section. Such a situation is encountered when the dimensions of the cross section are limited because of structural, head room or architectural reasons. Although a balanced section is the most economical section but because of limitations of size, section has to be sometimes over-reinforced by providing extra reinforcement on tension face than that required for a balanced section and also some reinforcement on compression face. Such sections reinforced both in tension and compression are also known as "Doubly Reinforced Sections". In some loading cases reversal of stresses in the section take place (this happens whenwindblowsinopposite

directionsatdifferenttimings),thereinforcementisrequired onbothfaces.

MOMENTOFRESISTANCEOFDOUBLYREINFORCEDSECTIONS

Consider a rectangular section reinforced on tension as well as compression faces as shown in Fig.2.4 (a-c)Let*b* = width ofsection,

d=effectivedepthofsection,

D= overalldepth ofsection,

d'=covertocentreofcompressivesteel,

M=Bendingmomentortotalmomentofresistance,

Mbal=Momentofresistanceofabalancedsectionwithtensionreinforcement,

Ast=Totalareaoftensilesteel,

Ast1=Areaoftensile

steelrequiredtodevelop*MbalAst2*=Are

aoftensilesteelrequiredtodevelop*M2A*

sc= Areaofcompressionsteel,

σst=Stressinsteel,and

σsc=Stressincompressivesteel

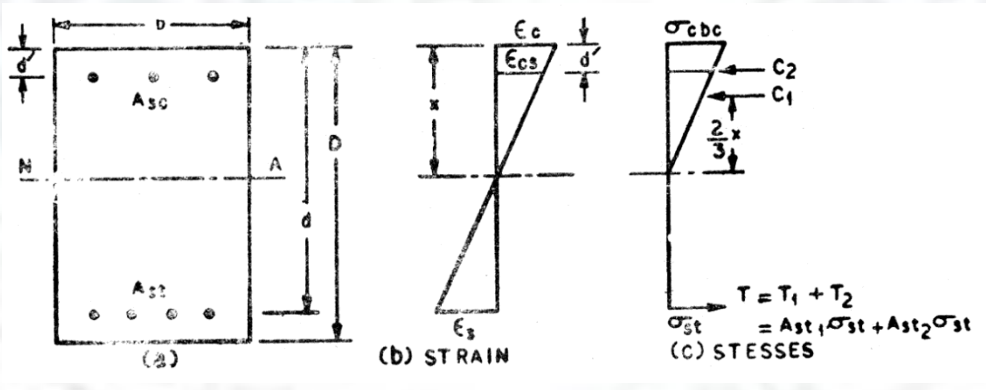


Fig.2.4

(a-c)Sincestrains are proportionaltothe distance

from*N.A.*,

$$\frac{Strainintopfibre\ \ ofconcrete}{ainin\ Compression\ \ Steel} = \frac{xStr}{x - d'}$$

$$\frac{\varepsilon_{cbc}}{\varepsilon_{sc}} = \frac{x}{x - d'}$$

$$\frac{\sigma_{cbc}}{E_c} \cdot \frac{E_s}{\sigma_{sc}} = \frac{x}{x - d'}$$

$$\sigma_{sc} = \sigma_{cbc} \cdot \frac{x - d'}{x} \cdot m$$

Since $\sigma_{cbc} \cdot \frac{x - d'}{x}$ isthestress inconcrete atthe levelofcompressionsteel,itcanbedenotedas

σ'_{cbc}

$\therefore \sigma_{sc} = m.\sigma'_{cbc}$

As per the provisions ofIS:456-2000 Code , the permissible compressive stress inbars , in a beam orslabwhencompressiveresistanceoftheconcreteistakenintoaccount,canbetakenas1.5mtimesthecompressive stressinsurroundingconcrete (1.5m σ’cbc) or permissible stress insteel incompression(σsc)whicheverisless.

$\sigma_{sc} = 1.5m\sigma'_{cbc}$

Totalequivalentconcreteareresistingcompression($x . b - A_{sc}$)+1.5mA_{sc}=$x.b$+(1.5m-1)A_{sc}

Taking moment about centre of tensile steelMoment of resistance

$M = C_1.(d-x/3)+C_2(d-d')$Where$C_1$= totalcompressive forcein concrete,

C_2=totalcompressiveforceincompressionsteel,

$$M = b.x\frac{\sigma_{cbc}}{2}.(d - \frac{x}{3}) + (1.5m - 1)A_{sc}\sigma_{cbc}\frac{x-d'}{x}.(d - d') = Q.b.d^2 + (1.5m-1)A_{sc}\sigma_{cbc}\frac{x-d'}{x}.(d - d')$$

$= M_1 + M_2$

WhereM_1= Momentof resistance ofthe

balanced section $\therefore M_{bal} M_2 =$ *Moment of resistance of the compression steel*

Area of tension steel $A_{st1} = \dfrac{M_1}{\sigma_{st}.j.d}$

Area of tension steel equivalent to compression steel $A_{st2} = \dfrac{M_2}{\sigma_{st}(d - d')}$

Thus the total tensile steel A_{st} shall be :

$\therefore \quad A_{st} = A_{st1} + A_{st2}$

The area of compression steel can be obtained as

$(1.5m - 1)A_{sc}(x - d') = mA_{st2}.(d - x)$

DesignConceptofT-Beam

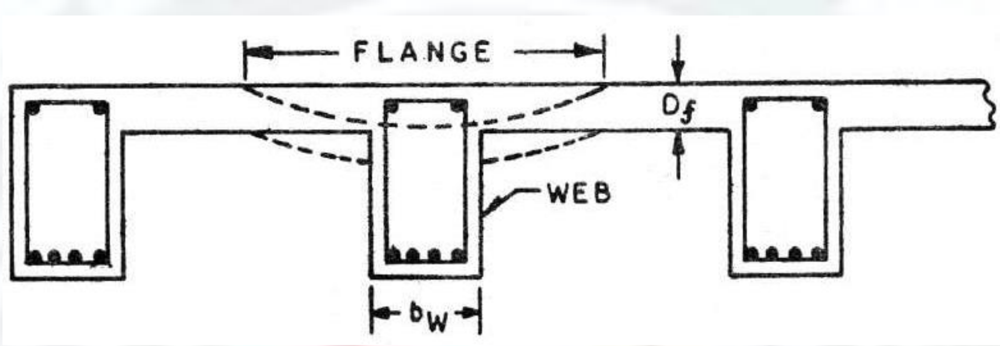


Fig.2.5

FlangedbeamsectionscompriseT-beamsandL-beamswheretheslabsandbeamsarecastmonolithically having no distinction between beams and slabs. Consequently the beams andslabs are soclosely tied that when the beam deflects under applied loads it drags along with it a portion of the slab also asshowninFig.2.5.thisportionoftheslabassistsinresistingtheeffectsoftheloadsandiscalledthe‘flange’ofthe T-beams. For design of such beams, the profile is similar to a T-section forintermediate beams. Theportionofthebeambelowtheslabiscalled‘web’or‘Rib’.Aslabwhichisassumedtoactasflangeofa T-beamshallsatisfythe following conditions:

(a) The slab shall be cast integrally with the web or the the web and the slab shall be effectively bondedtogetherin anyothermanner;and
(b) If the main reinforcement ofthe slab is parallel to the beam, transverse reinforcement shall beprovidedas shown in Fig.2.6, such reinforcement shall not be less than 60% of the mainreinforcementat mid-spanoftheslab.

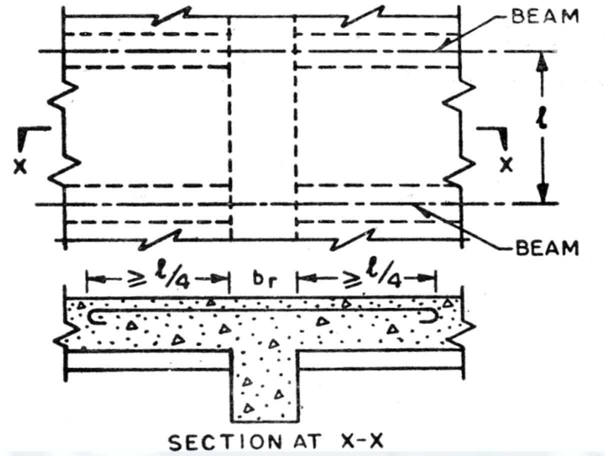


Fig.2.6

CHAPTER-
3LIMITSTATEMETHO
D

SAFETYANDSERVICEABILITYREQUIREMENTS

In the method of design based on limit state concept, the structure shall be designed to withstand safelyall loads liable to act on it throughout its life; it shall also satisfy the serviceability requirements, such aslimitations on deflection and cracking. The acceptable limit for the safetyandserviceability requirementsbefore failure occurs is called a 'limit state'. The aim of design is to achieve acceptable probabilities that thestructurewillnotbecome unfitforthe useforwhichitis intendedthatitwillnotreachalimitstate.

All relevant limit states shall be considered in design toensure an adequate degree of safety andserviceability. In general, the structure shall be designed on the basis of the most critical limit state and shall becheckedforotherlimitstates.

For ensuring the above objective, the design should be based on characteristic values for materialstrengths and applied loads, which take into account the variations in the material strengthsand in the loads tobe supported. The characteristic values should be based on statistical data if available; where such data are notavailable they should be based on experience. The 'design values' are derived from the characteristic valuesthrough the use of partial safety factors, one for material strengths and the other for loads. In the absence ofspecial considerations these factors should have the values given in 36 according to the material, the type ofloadingand the limitstate beingconsidered.

LimitStateofCollapse

The limit state of collapse of the structure or part of the structure could be assessed from rupture of one or more critical sections and from buckling due to elastic or plastic in stability (including the effects of swaywhereappropriate) oroverturning. The resistance to bending, shear, torsion and axial load satevery section shall not be less than the appropriate value at that section produced by the probable most unfavourable combination of loads on the structure using the appropriate partial safety factors.

LimitStateDesign

Forensuringthedesignobjectives,thedesignshouldbebasedoncharacteristicvaluesf ormaterialstrengths and applied loads (actions), which take into account the probability of variations in the materialstrengths and in the loads to be supported. The characteristic values should be based on statistical data, ifavailable. Where such data is not available, they should be based on experience. The design values are derived from the characteristic values through the use of partial safety factors, both formaterial strength sand for loads. In the absence of special considerations, the sefactors should have the values given in this section accordingto the material,the type of load and the limit state being considered. The reliability of design is ensured by requiring that Design Action Design Strength

Limit states are the states beyond which the structure no longer satisfies the performance requirementsspecified.The limitstatesare classified as

a) Limit state of strength

b) Limit state of serviceability

a) The limit state of strength are those associated withfailures (or imminent failure), under the action ofprobable and most unfavorable combination of loads on the structure using the appropriate partial safety factors, which may endanger he safety of life and property. The limit state of strength includes:

a) Loss of equilibrium of the structureasa whole or any of its parts or components.

b) Loss of stability of the structure (including the effect of sway where appropriate and overturning) or any of its parts including supports and foundations.

c) Failure by excessive deformation, rupture of the structure or any of its parts or components.

d) Fracture due to fatigue.

e) Brittle fracture.

b) The limit state of serviceability include

a) Deformation and deflections, which may adversely affect the appearance or, effective, use of thestructure or may cause improper functioning of equipment or services or may cause damages tofinishesand non-structuralmembers.

b) Vibrations in the structure or any of its components causing discomfort to people, damages to thestructure,itscontentsorwhichmaylimititsfunctionaleffectiveness.Specialconsiderationshallbe given to floor vibration systems susceptible to vibration, such as large open floor areas free ofpartitions toensurethatsuchvibrationsisacceptablefortheintendeduseandoccupancy.

c) Repairabledamageduetofatigue.

d) Corrosionanddurability.

LimitStatesofServiceability

To satisfy the limit state of serviceability the deflection and cracking in the structure shall not beexcessive.This limitstate corresponds to deflection andcracking.

Deflection

The deflection of a structure or part shall not adversely affect the appearance or efficiency of thestructureorfinishes orpartitions.

Cracking

Cracking of concrete should not adversely affect the appearance or durability ofthe structure; theacceptablelimitsof crackingwouldvary withthetypeof structureandenvironment.The actualwidthofcracks will vary between the wide limits and predictions of absolute maximum width are not possible. The surface width of cracks should not exceed0.3mm.

In members where cracking in the tensile zone is harmful either because they are exposed to the effects of the weather or continuously exposed to moisture or in contact soil or ground water, an upper limit of 0.2 mm is suggested for the maximum width of cracks. For particularly aggressive environment, such as the 'severe' category, the assessed surface width of cracks should notin general, exceed 0.1 mm.

CHARACTERISTIC AND DESIGN VALUES AND PARTIAL SAFETY FACTORS

1. **Characteristic Strength of Materials**

Characteristic strength means that value of the strength of the material below which not more than 5 percent of the test results are expected to fall and is denoted by ƒ. The characteristics trength of concrete (f_{ck}) is as per the mix of concrete. The characteristic strength of steel (f_y) is the minimum stressor 0.2percent ofproofstress.

2. **Characteristic Loads**

Characteristic load means that value of load which has a 95 percent probability of not being exceeded during the life of the structure. Since dataarenotavailabletoexpressloadsinstatisticalterms,forthepurposeofthisstandard,deadloadsgiveninIS875(Partl),imposedloadsgiveninIS875(Part2),windloadsgiveninIS875(Part3),snowloadasgiveninIS875(Part**4)**andseismicforcesgiveninIS1893-2002(part-I)shallbeassumed asthe characteristic loads.

Design

ValuesMat

erials

Thedesignstrengthofthematerialsƒdisgivenby

$$f_d = \frac{f}{\gamma_m}$$

d

L

o

a

.

The design load, F,is givenby

$$f_d = \frac{F}{\gamma_f}$$

Where,F=characteristicload

and γ_f=partialsafetyfactorappropriatetothenatureofloadingandthelimitstate beingconsidered.

ConsequencesofAttainingLimitState

Wheretheconsequencesofastructure attainingalimitstateareofaseriousnature suchashugelossoflife

anddisruptionoftheeconomy,highervaluesf or γ_f and γ_m than those given under36.4.1 and 36.4.2maybe applied.

PartialSafetyFactors:

1. **PartialSafetyFactor γ_f forLoads**

Sr. No.	LoadCombination	UltimateLimitState	ServiceabilityLimitState
1	DL+ LL	1.5(DL +LL)	DL+ LL
2	DL + WL i) DL contribute tostability ii) DL assistsoverturning	0.9 DL+ 1.5WL 1.5 (DL+WL)	DL + WL DL+ WL
3	DL + LL + WL	1.2(DL + LL+ WL)	DL + 0.8 LL + 0.8 WL

2. **PartialSafetyFactor γ_m for MaterialStrength**

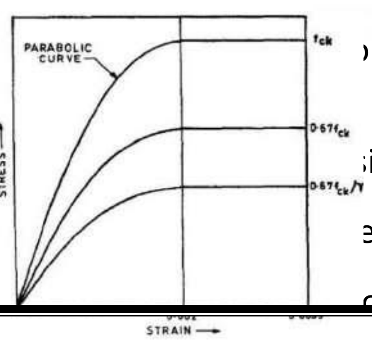
Sr. No.	Material	UltimateLimitState	Serviceability LimitState
1	Concrete	1.50	$E_c=5000\sqrt{f_{ck}}$ MPa
2	Steel	1.15	$E_s=2\times10^5$ MPa

When assessing the strength of a structureor structuralmemberfor the limit stateof collapse, the values ofpartialsafetyfactor, should betakenas1.5 forconcrete and 1.15 forsteel.

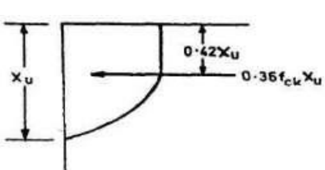
LIMITSTATEOFCOLLAPSE:FLEXURE

AssumptionsforLimitState ofCollapse (Flexure):

1) Plane section normal to the axis remains plane even after bending. i.e. strain at any point on the crosssectionis directlyproportionalto the distance fromthe N.A.



STRESS-STRAIN CURVE FOR CONCRETE



STRESS BLOCK PARAMETERS

2) Maximumstr ... outermostcompression fibre istaken as0.0035 inbending.

3) Therelations ... iv ... oncreteandthest raininconcre ... e ... parabola or any other shape ... or ... antial agreement

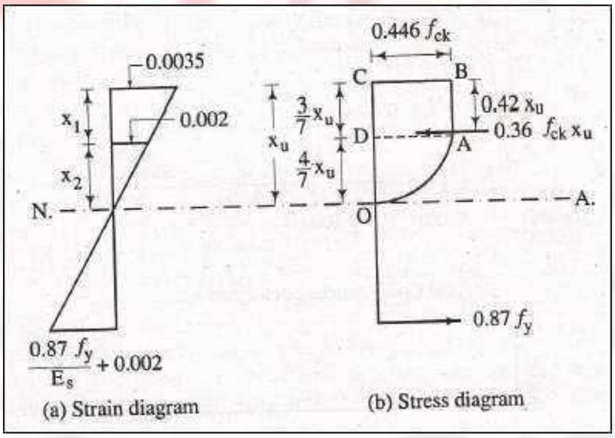
with the results of test. An acceptable stress strain curve is as shownbelow.

For design purposes, the compressive strength of concrete in the structure shall be assumed to be 0.67timesthecharacteristicstrength.Thepartialsafetyfactorγ_m =1.5shallbeappliedinadditiontothis.

NOTE - For the above stress-strain curve the design stress block parameters are as follows:Area ofstress block=0.36.f_{ck}.x_u

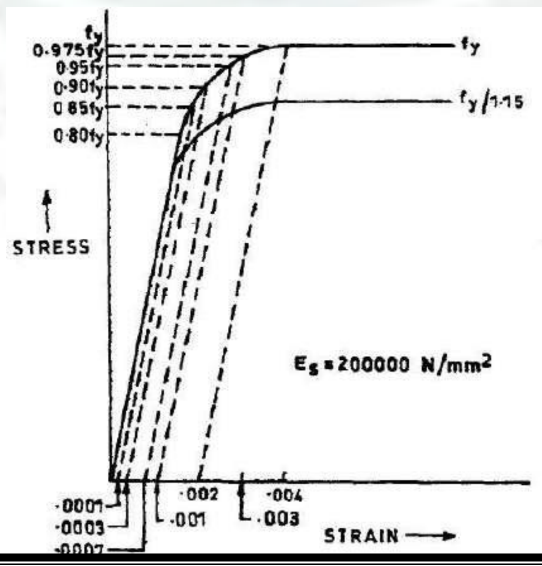
Depth of centre of compressive force = 0.42x_u from the extreme fibre in compressionWhere

f_{ck}=characteristiccompressivestrengthofconcrete,andx_u= depth ofneutralaxis.

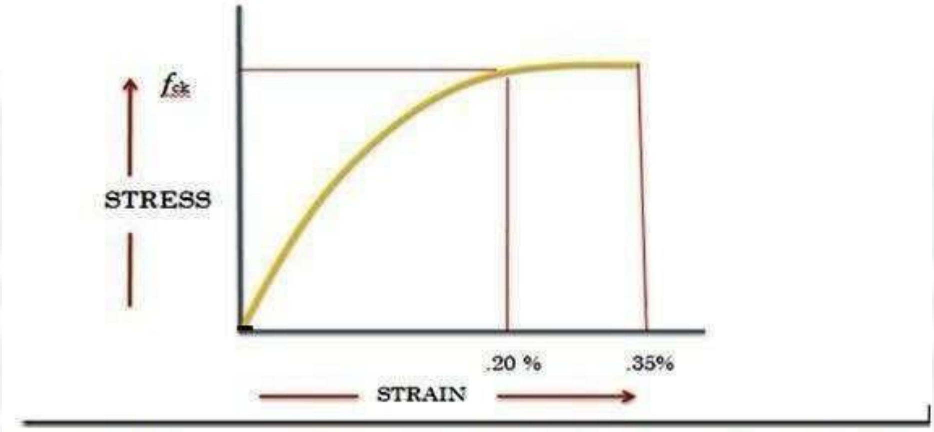


4) thetensilestrengthoftheconcreteisignored.

5) the stresses in the reinforcement are derived from representativestress –



strain curve for the type ofsteelused.



STRESS — STARIN CURVE FOR STEEL

6) themaximumstrainintension reinforcementinthesectionatfailureshallnotbelessthan

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

CHAPTER4

LIMITSTATESOFCOLLAPSEOFSINGLEREINFORCEDMEMBERSINBENDING

Limitstatemethodofdesign

- The object of the design based on the limit state concept is to achieve an acceptable probability, that astructure will not become unsuitable in it's lifetime for the use for which it is intended, i.e. It will not reach alimit state
- Astructurewithappropriatedegreeofreliabilityshould beabletowithstandsafely.
- All loads, that are reliable to act on it throughout it's life and it should also satisfy the subs s abilityrequirements,such aslimitation on deflection andcracking.
- It should also be able to maintain the required structural integrity, during and after accident, such asfires, explosion & local failure.i.e. limit sate must be consider in design to ensure an adequate degree of safetyandserviceability
- Themostimportantoftheselimitstates,whichmustbeexamineindesignareas follows

Limitstateofcollapse

- Flexure
- Compression
- Shear
- Torsi

onThisstatecorrespondstothemaximumloadcarr yingcapacity.

Typesof reinforcedconcretebeams

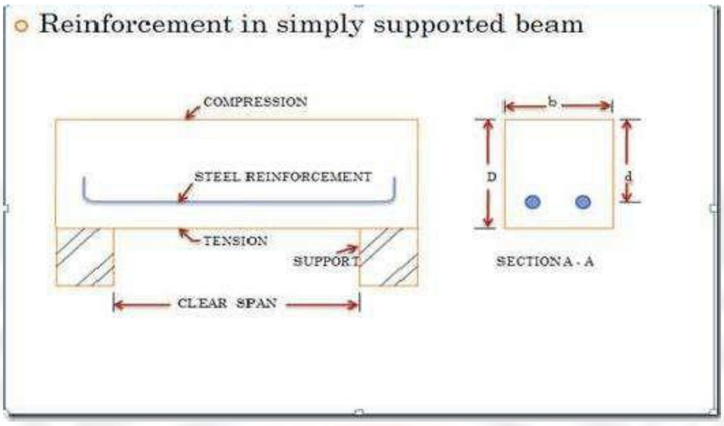
a) Singlyreinforcedbeam

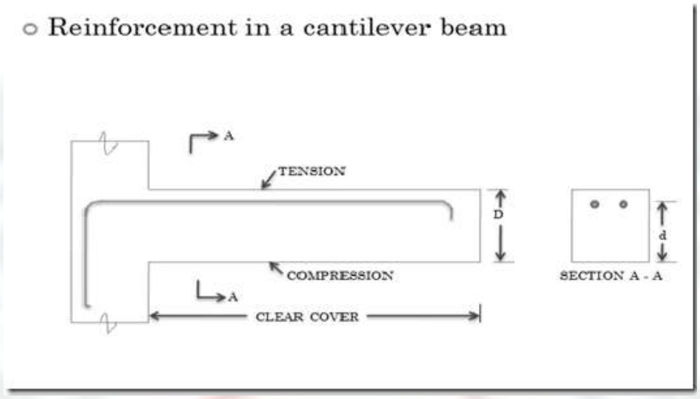
b) Doublyreinforcedbeam

c) Singlyor Doublyreinforcedflangedbeams

Singlyreinforced beam

In singly reinforced simply supported beams or slabs reinforcing steel bars are placed near the bottom of thebeamorslabswhere theyaremosteffectivein resisting the tensile stresses.





TYPESOFBEAMSECTIONS

Section in which, tension steel also reaches yield strain simultaneously as the concrete reaches thefailurestrain in bending arecalled, **'BalancedSection'.**

Section in which, tension steel also reaches yield strain at loads lower than the load at which concretereachesthe failure strainin bending are called,**'UnderReinforcedSection'.**

Section in which, tension steel also reaches yield strain at loads higher than the load at which concretereachesthe failure strain in bendingare called,**'OverReinforcedSection'.**

Sr. No.	Types ofProblems	DataGiven	DataDetermine
			If $\frac{X_u}{d} = \frac{X_u max}{d}$ ⇒ Balanced If $\frac{X_u}{d} < \frac{X_u max}{d}$ ⇒ UnderReinforced If $\frac{X_u}{d} > \frac{X_u max}{d}$ ⇒ OverReinforced
	Identifythe typeof	Grade ofConcrete	$\frac{X_u}{d} = \frac{0.87 f_y . A_{st}}{0.36 b . d f_{ck}}$

1.	section,balance,under reinforced or over reinforced	&Steel, Size ofbeam &Reinforcementprovided	$\frac{X_u max}{d}$ fy: 250, 415, 500 0.53, 0.48, 0.46

2	Calculat eMome nt ofResist ance	Grade ofConcr ete& Steel, Size ofbeam& Reinforce mentProvi ded	1)If $\frac{x_u}{d} = \frac{x_{u,max}}{d}$,*balanced* $M.R = M_u = 0.36.\frac{x_{u,max}}{d}(1 - 0.42\frac{x_{u,max}}{d})b.d^2.f_{ck}$
			2)If $\frac{x_u}{d} < \frac{x_{umax}}{d}$ UnderReinforced M.R=Mu= $0.87f_y.A_{st}.d(1 - \frac{A_{st}f_y}{b.d.f_{ck}})$ or $M.R = 0.87f_y.A_{st}.d(1 - 0.42\frac{x_u}{d})$
			3)If $\frac{X_u}{d} > \frac{X_{umax}}{d}$ → overreinforced, Revisethedepth
3	Design thebeam. Find outthe depth ofBeam D &Reinforce mentrequi redAst.	Gradeof Concrete & Steel, width ofbeam & BendingMo ment orloading on thebeam with thespan ofthebeam Reinforcement Provided	We have to design the beam as a 'Balanced Design'.Forfinding 'd'effective depthusethe equation; $M.R = M_u = 0.36.\frac{x_{u,max}}{d}(1 - 0.42\frac{x_{u,max}}{d})b.d^2.f_{ck}$ ForfindingAstusetheequation $0.87f_y.A_{st}.d(1 - \frac{A_{st}.f_y}{b.d.f_{ck}})$ or $M.R = 0.87f_y.A_{st}.d(1 - 0.42\frac{x_u}{d})$

Where d =effectivedepthofbeaminmm.b =widthofbeaminmm

x_u =depthofactualneutralaxisimmmfromextremecompressionfibre.$x_{u,max}$ =depthofcriticalneutralaxisinmmfromextremecompression fibre.

A_{st} =areaoftensilereinforcement

f_{ck} = characteristicstrengthofconcreteinMPa.

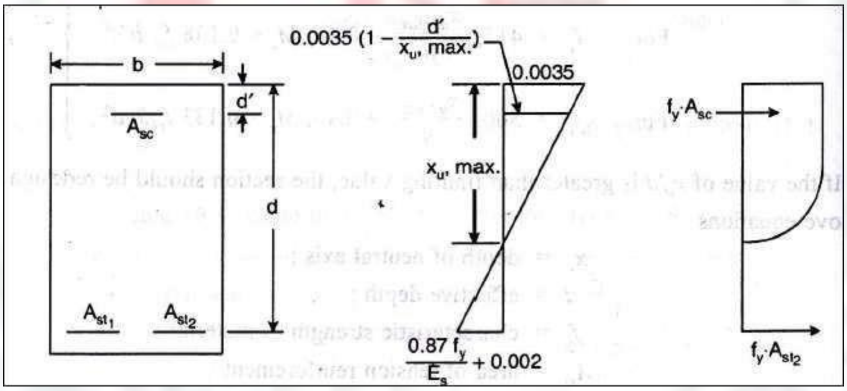
f_y =characteristicstrengthofsteelinMPa.

$M_{u,lim}$=LimitingMomentofResistanceofasectionwithoutcompressionreinforcement

DoublyReinforcedSectionor sectionswithCompressionReinforcement

DoublyReinforcedSectionsectionsareadoptedwhenthedimensionsofthebeamhavebeenpredetermined from other considerations and the design moment exceeds the moment of resistance of a singlyreinforced section. The additional moment of resistance is carried by providing compression reinforcement andadditionalreinforcementintensionzone.Themomentofresistanceofadoublyreinforcedsectionisthesumofthelimitingmomentofresistance$M_{u,lim}$ofasinglereinforcedsectionandtheadditionalmomentofresistance M_{u2}.

$M_{u2} = M_u - M_{u,lim}$



The lever arm for the additional moment of resistance is equal to the distance between the centroids of tensionandcompression reinforcement, (d – d').

$M_{u2}=0.87f_y.A_{st2}(d-d')=A_{sc}.(f_{sc}-f_{cc})(d-d')$

Where:

A_{st2}=Areaofadditionaltensilereinforcement A_{sc}=Areaofcompressionreinforcement

f_{sc}=Stressincompressionreinforcement

f_{cc}=Compressivestressinconcreteatthelevelofcompression reinforcementSincethe additionareinforcementisbalanced bythe additionalcompressiveforce.

$A_{sc}.(f_{sc}-f_{cc})=0.87f_y.A_{st2}$

Thestrainatlevelofcompressionreinforcementis

$0.0035\left(1-\frac{d'}{x_{u,max}}\right)$ Totalarea ofreinforcementshallbe obtainedby

$$A_{st} = A_{st1} + A_{st2}$$

A_{st1}=AreaofreinforcementforasinglyreinforcedsectionforM$_{u,lim}$

$$A_{st2} = \frac{A_{sc}\,(f_{sc} - f_{cc})}{0.87 f_y}$$

EXAMPLE4.1

Calculate the area of steel of grade Fe 415 required for section of 250mm wide and overall depth 500mm witheffectivecover40mmin M20,ifthe limitstateofmomentbe carriedbythe section is

a) 100KN b)146 KN c) 200KN

SOLUTION:

For f_y=415N/mm^2, $\frac{x_{u,max}}{d}=0.48$

$$M_{u,lim} = 0.36\frac{x_{u,max}}{d}\left(1-0.42\frac{x_{u,max}}{d}\right)b.d^2.f_{ck}$$

$$= 0.36X.48(1-0.42X0.48)X250X460^2X20$$

$$=146X10^6 N.mm$$

a) ForM_u= 100 KN.m< 146 KN.m

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

$$100X10^6=0.87X415XA_{st}X460\left(1-\frac{A_{st}X415}{250X460X20}\right)$$

A_{st}=686or4850mm^2,takingminimum steel686mm^2

b) M_u=146KN.m=$M_{u,lim}$=146 KN.mx_u =$x_{u,max}$

Areaoftensionreinforcementrequired

$\frac{X_u,max}{d} = \frac{0.87 f_y . A_{st}}{0.36 b.d f_{ck}}$

$A_{st} = \frac{0.48X\,0.36X20X250X460}{0.87X415} = 1100mm^2$

c) Mu=200KN.m>Mu,lim=146KN.m

Reinforcement is to be provided in the compression zone also along with the reinforcement in tension zone.Mu=Mu,lim=fsc.Asc(d–d')

f_{sc} is stress corresponding to strain of $\frac{0.0035(x_{u,max} - d')}{x_{u,lim}} = \frac{0.0035(0.48X\,460 - 40)}{0.48X460} = 0.002866$

$f_{sc}=360.8N/mm^2$

$(200-146)X10^6=360.8.A_{sc}(460-40)$

$A_{sc}=356mm^2$

Ast1=Area of tension reinforcement corresponding to $M_{u,lim}$

$146X\,10^6=0.87X\,415A_{st1}\,460X(1-\frac{A_{st}X415}{250\,X460X\,20})$

$A_{st1}=1094mm^2$

$A_{st2}=A_{sc}.f_{sc}/0.87X415=356mm^2$

$A_{st}=A_{st1}+A_{st2}=1094+356=1450mm^2$

EXAMPLE:4.2

Design a rectangular beam which carries a maximum limiting bending moment of 65KN. m. Use M20 and Fe415 as reinforcement.

At balanced failure condition $M_u = M_{u,lim}$

$$M_{u,lim} = 0.36.\frac{x_{u,max}}{d}(1 - 0.42\frac{x_{u,max}}{d})b.d^2.f_{ck}$$

$M_{u,lim} = 0.36X0.48X20(1-0.42X0.48)bd^2$

$=2.759bd^2$

Assumingwidth ofbeamas250 mm

d= =307mm

Areaofreinforcement

$$\frac{X_{u,max}}{d} = \frac{0.87 f_y . A_{st}}{0.36 b . d f_{ck}}$$

$$0.48 = \frac{0.87 X 415 X A_{st}}{0.36 X 20 X 250 X 307}$$

$A_{st} = 734.66 mm^2$

EXAMPLE:4.3

Find out the factored moment of resistance of a beam section 300mm wide X 450mm effective depthreinforced with 2 X 20mm diameter bars as compression reinforcement at an effective cover of 50mm and 4 X25mmdiameterbarsas tensionreinforcement. Thematerials areM20gradeconcreteandFe 415HYSD bars.

Solution:

Given;

Width=b=300mm

Effectivedepth=d=450mm

Covertocompressionreinforcement=d'= 50mm

$\frac{d'}{d} = \frac{50}{450} \approx 0.11$, nexthighervalue 0.15maybe adopted.

A_{sc}=area compression reinforcement = 2 π

16^2 =

628mm^2 A_{st}=areaofreinforcementintension=4

xπ25^2=1964mm^2 f_{sc}=stressincompressionsteel

=342N/mm^2

Equatingtotalforce

$0.36f_{ck}.b.x_u+f_{sc}.A_{sc}=0.87f_y.A_{st}$

$0.36X20X\ 300x_u+628X342=0.87X\ 415X1964$

x_u=228.85mm

But $x_{u,max}$=0.48d for Fe415

$x_{u,max}$= 0.48 X 450 =

216mm So $x_u > x_{u,max}$,

 `overreinforced

The moment of resistance can be found out by takin moments of compressive forces about centroid of tensile reinforcement.

M_u=2160x_u(450-0.42x_u)+214776(450-50)X10^{-6}

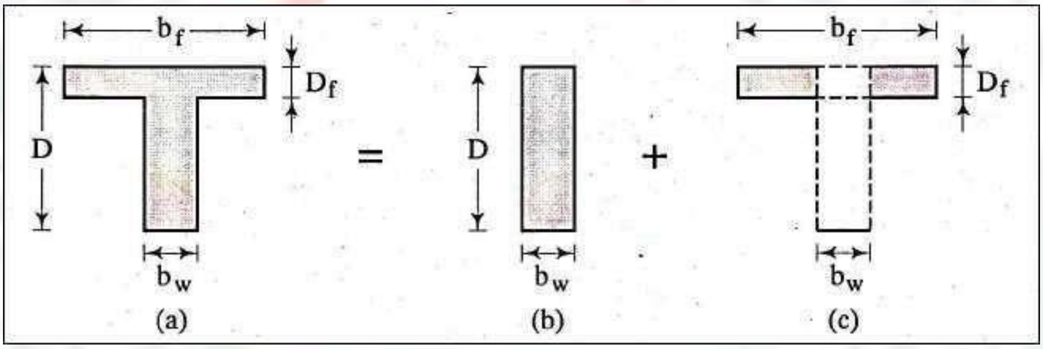
Putting x_u =

216mm M_u=25

3.54KN.m

BEHAVIORSOF‘T’AND‘L’BEAMS(FLANGEDBEAM)

A‘T’beamor‘L’beamcanbeconsideredasarectangularbeamwithdimensionsb_w. Dplusaflangeofsize $(b_f-b_w)XD_f$.Itis shown in the figure beam(a)is equivalentto beam(b)+beam (c).



The flanged beam analysis and design are analogous to doubly reinforced rectangular beam. In doublyreinforced beams additional compressive is provided by adding reinforcement in compression zone, whereas inflanged beams, this is provided by the slab concrete, where the spanning of the slab is perpendicular to that ofbeamand slab is in compression zone.

Ifthespanningoftheslabisparalleltothatofthebeam,someportionofslabcanbem adetospaninthedirection perpendicularto thatofthe beambyadding some reinforcementin theslab.

Aflangedbeamcanbealsodoublyreinforced.

The moment of resistance of a T beam is sum of themoment of resistance of beam (a) is the summomentofresistance ofbeam(b)and momentofresistance ofbeam(c)

CHAPTER-5

LIMITSTATEOFCOLLAPSEINSHEAR(DesignofShearbyLSM)

5.1. SHEARSTRESSINREINFORCEDCONCRETEBEAMS:-

Whenabeamisloadedwithtransverseloadsthe BendingMoment(BM)variesfromsectio ntosection.Shearing stresses in beams are caused by this variation of BM in the beam span. Due to the variation of BM attwo sections distance dx apart, there are unequal bending stresses at the same fibre. Thisinequality ofbending stresses produces a tendency in each horizontal fibre to slide over adjacent horizontal fibre causinghorizontalshearstress, whichisaccompaniedbycomplimentaryshearstressinverticaldirection.

SHEARCRACKSINBEAMS:-

Under the transverse loading , at any section of the beam, there exists both Bending Moment(BM) and ShearForce(V).Depending upon theratio of Bending Moment(BM) toShearForce(V) at different sections,theremaybe three regionsof shear cracksinthe beamasfollows.

(a) RegionI :Region offlexureCracks.
(b) RegionII:RegionofflexureshearCracks.
(c) Region II: Regionof web shearCracksordiagonaltensioncracks.

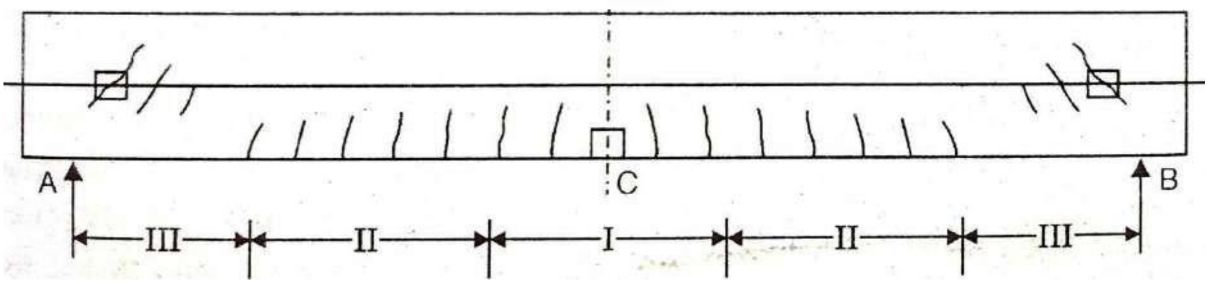


Fig-5.2.1DIFFERENTREGIONOFCRACKSINBEAMS

(a) RegionI :RegionofflexureCracks.

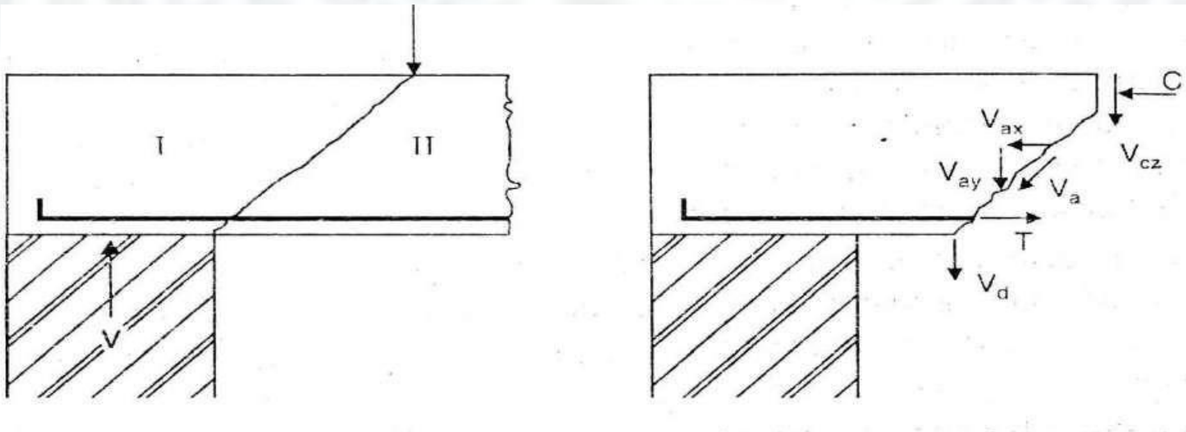
This region normally occurs adjacent to mid-span where BM is large and shear force is either zero orvery small.The principal planes are perpendicular to beam axis. When the principal tensile stressreaches the tensile strength of the concrete (which is quite low) tensile cracks develop vertically. Thecracksare known asflexuralcracksresultingprimarilydueto flexture.

(b) **Region II:Region of flexure shearCracks.**This regions are near the quarter span, to boththe sides, where BM is considerable and at the same time Shear force is significant. The cracksin this region are initiated at the tension face, travel vertically (due to flexture) and graduallytend to develop in the inclined direction towards the Nutral Axis(N.A.), as the shear stress goeson increasing towards the N.A. Since the cracks develop under the combined action of BM andShear,thesecracksareknownasflexure-shearcracks.

(c) **RegionII:RegionofwebshearCracks ordiagonaltensioncracks.**

ThisregionsareadjacenttoeachsupportofthebeamwhereS.Fispredominant.SinceShearstressis maximumat theN.A.,inclined cracks starts developing attheN.A.along thediagonalofanelement subject to the action of pure shear.Hence these cracks known as diagonal tension cracks orweb-shearcracks.

MECHANISM OF SHEAR TRANSFER IN REINFORCE CONCRETE BEAMWITHOUTSHEAR.



(a) Diagonal tension crack (b) Flexural shear crack

Fig-5.3.1

ShearistransferredbetweentwoadjacentplanesinaRCbeambythefollowingmechanism.

(a) ShearresistanceV$_{cz}$oftheuncrackedportionofconcrete.

(b) VerticalComponentV$_{ay}$oftheinterface shearoraggregateinterlockforceV$_a$.and

(c) DowelforceV$_d$inthetensionreinforcement,duetodowelaction.

Thus $V = V_{cz} + V_{ay} + V_d$.

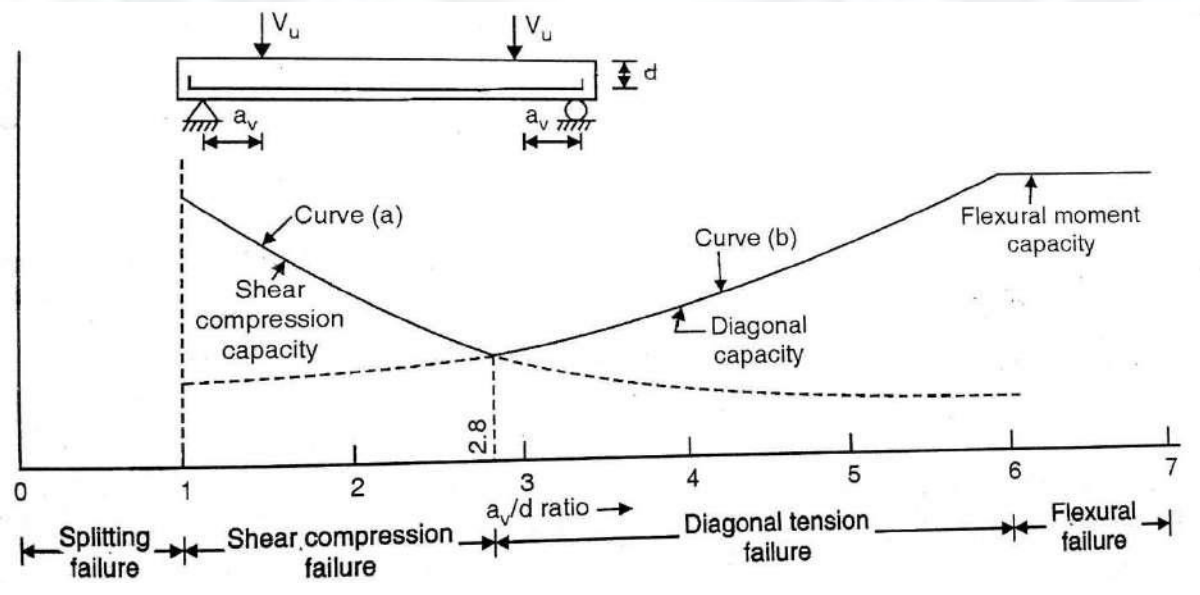
The relative contribution of each of the above three mechanism depend upon the stage of loading andextent of cracking. In the initial stage before the flexural cracking starts, the entire shear is resisted bythe shear resistance oftheconcrete(i.eV= V_{cz}).

As the flexural cracking starts, the interface shear comes into action resulting in the redistribution ofstresses. Further extension of flexural cracks results in sharing the shear by the dowel force $V_d$of thetension reinforcement. Thus at the final stage of collapse , the shear is transferred by the shear is bornbyall thethree mechanism expressed bythe equation above.

MODESOFSHEARFAILURE

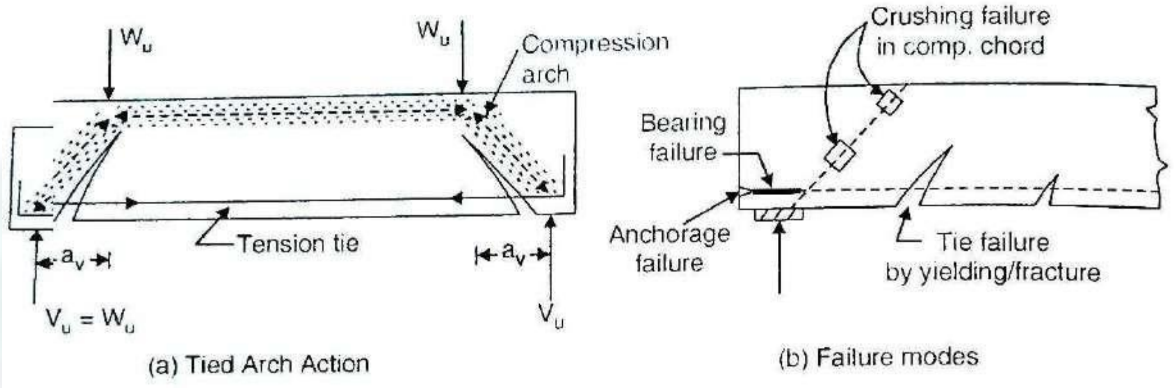
The shear Failure of a R C beam, without shear reinforcement is governed by a_v / d, ratio. A beammayexperience followingtypesofshear failure.

1. CaseI:$a_v/d<1$:Splittingorcompressionfailure.
2. CaseII:$1<a_v/d< 2.8$:Shearcompressionorsheartensionfailure.
3. CaseIII:$2.8 <a_v/d< 6$:Diagonaltensionfailure.
4. Case-IV:$a_v/d>6$:Flexurefailure



Fig– 5.4.1. EFFECT OFa_v/ d ONSHEARSTRENGTHOFR C BEAM

CASEI:av/d< 1(DeepBeams):Splitting orcompression failure:



Fig– 5.4.2. CASE I: av/ d< 1 (DEEPBEAMS)

Thiscasecorrespondtoadeepbeamwithoutshearreinforcementswheretheinclinedcrackingtransformsthebeamintoatiedarch(Fig-a).Theloadiscarriedby(i)directcompressionintheconcretebetweentheloadandreactionpointbycrossingofconcreteandby (ii)tensioninthelongitudinalsteel byyieldingorfracture oranchorage failureor bearingfailure .

CASEII: 1<av/d<2.8 :Shearcompressionorshear tensionfailure.

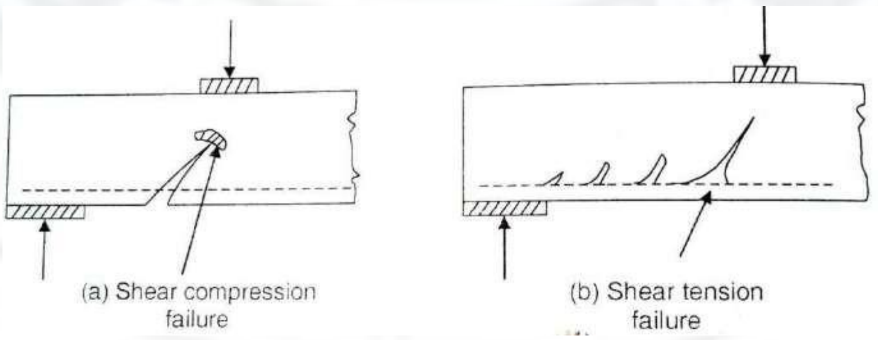


Fig-5.4.3 CASE II: 1< av/ d <2.8

This case is common in short beams with av / d ratio between 1 to 2.8, where failure is initiated by aninclined crack– more commonly a flexural shear crack. Fig-a shows the shear compression failureduetoverticalcompressivestressesdevelopedinthevicinity oftheload.Similarly theverticalcompressivestressoverthereactionlimitsthebondsplittinganddiagonalcrackingalongthesteel.The crack extends towards the tension reinforcement and then propagates along the reinforcements(Fig-b)resultingin the failure ofthe beambyanchoragefailure.

CASEIII: 2.8 < av/ d<6 :Diagonaltensionfailure.

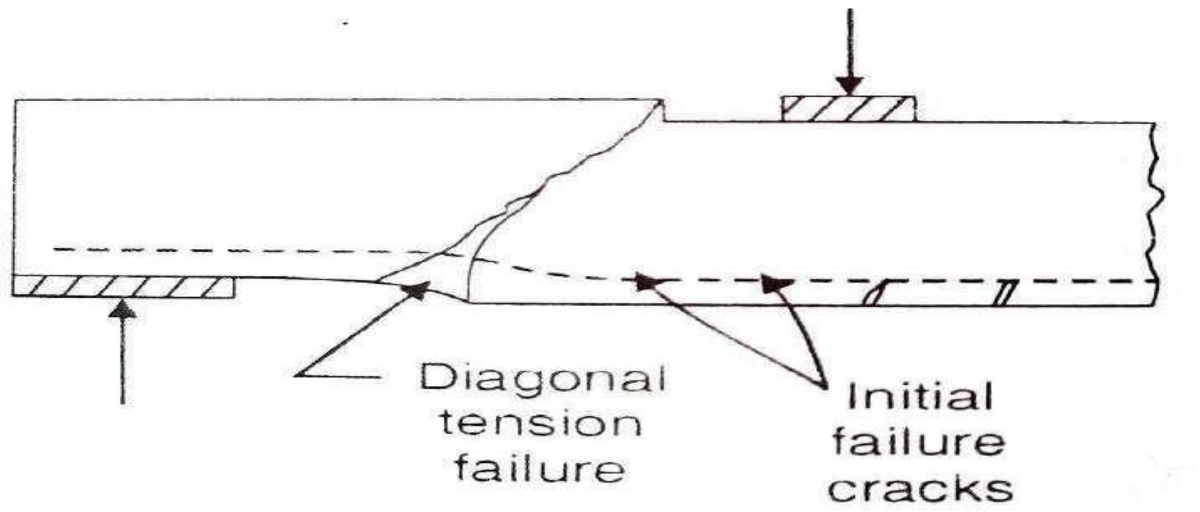


Fig-5.4.4 CASEIII:2.8< av/ d <6

Diagonal tension failure occurs when the shear span to the effective depth ratio is in the range of 2.8to 6 . Normal beams have av / d ratio in excess of 2.8. Such beams may fail either in shear or inflexure.

CASE- IV: av/ d>6 :Flexurefailure

Flexural failure is encountered when av/ d ratio > 6. Two cases may be encountered; (i) underreinforcedbeamand(ii)overreinforcedbeam.Inthecaseofunderreinforcedbeam, tensionreinforcement is less than the limiting one, due to which failure is initiated by yielding of tensionreinforcement, leading to the ultimate failure due to crushing of concrete in compression zone. Such aductile failure is known as flexural tension failure, which is quite slow giving enough warning. In theoverreinforcedsectionsfailureoccursduetocrushingofconcreteincompressionzon ebeforeyielding of tension reinforcement. Such afailure, known as flexural compression failure is quitesudden.

FACTORSAFFECTING THESHEARRESISTANCEOFARCMEMBER.

The shear resistance of rectangular beams, without shear reinforcements depends on the followingfactors.

1.**Gradeofconcrete:**Highergradeofconcretehashighercharacteristicstrengthwhichinturn results in (i) higher tensile strength (ii0 greater dowel shear resistance (iii) greater aggregateinterlock capacity, and (iv) greater concrete strength in compression zone. Hence shear resistanceincreaseswiththeincrease in the gradeofconcrete.

2. Percentage and grade of longitudinal tensile reinforcement : The increase in percentage (p_t) of longitudinal tensile reinforcement results in the increase in dowel shear (V_d). Due to this reason, the design Codes make the shear strength (τ_c) of concrete a function of p_t and grade of concrete (see Table 5.1). However, higher grade of steel results in lesser shear resistance of R.C. beam because the percentage of steel (p_t) corresponding to a higher grade steel is less than that required for a low grade steel, say mild steel.

3. Ratio of shear span to effective depth (i.e. a_v/d ratio) : As discussed in the previous article, for a_v/d ratio between 6 and 2.8, the shear capacity, being governed by inclined crack resistance, decrease with decrease in a_v/d ratio (curve b of Fig.5.4.1). However, for a value of a_v/d less than 2.8, the shear capacity, being dependent on *shear-compression* or *shear-bond capacity*, increases rapidly. The minimum shear capacity is at a_v/d ratio around 2.8.

4. Compressive force : Presence of axial compressive force result in increase of shear capacity. The effect of axial compression on the design shear strength has been taken into account by I.S. Code by increasing the design shear strength by a modification factor δ.

5. Compressive reinforcement : The shear resistance is found to increase with the increase in the percentage of compressive steel (p_c).

6. **Axialtensileforce:**Axialtensileforcereducesmarginallytheshearresistanceofconcreteas per theequation$\delta = 1 - \frac{P_w}{3.45 A_g}$.

7. **Shear reinforcement:** The shear resistance of a R C Beam increases with the increase inshear reinforcement ratio. This is due to two reasons (i) concrete gets conformed between stirrupspacingand (ii)the shear/webreinforcement itselfprovidesshearresistance.

5.6 . DESIGN SHEAR STRENGTH OF CONCRETE WITHOUT SHEARRENFORCEMENT(IS456: 2000)

The magnitude of design shear strength (v_c) depends basically on the grade of concrete (f_{ck}) and thepercentage of tension steel(P_t). As per IS 456 : 2000 the design shear strength of concrete in beamswithoutshear reinforcement shallbegiven in table5.1.

TABLE 5.1 DESIGN SHEAR STRENGTH (τ_c) OF CONCRETE, (N/mm^2)

$100 \frac{A_{st}}{bd}$	Grade of concrete					
	M 15	*M 20*	*M 25*	*M 30*	*M 35*	*M 40 and above*
≤ 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
3.0 and above	0.71	0.82	0.92	0.96	0.99	1.01

Analyticalexpressionfordesignshearstrength:

The Values of vc given in the above table by the code are based on the following semi empiricalexpression (SP24, 1983).

$$\tau_c = \frac{0.85\sqrt{0.8 f_{ck}}\,(\sqrt{1+5\beta}-1)}{6\beta} \qquad \text{...... 5.6.1}$$

where $\beta = \frac{0.8 f_{ck}}{6.89 p_t}$, but not less than 1

$p_t = \frac{100 A_{st}}{bd}$ (percentage steel in rib width only)

$0.8 f_{ck}$ = cylinder strength in terms of cube strength

0.85 = reduction factor similar to $1/\gamma_m$

The formula in BS 8110 for design shear strength of concrete is slightly different, and is given by the expression

$$\tau_c = 0.79\,(p_t)^{1/3}\left(\frac{400}{d}\right)^{1/4}\left(\frac{1}{\gamma_m}\right)\left(\frac{f_{ck}}{25}\right)^{1/3} \quad \ldots\ 5.6.2$$

where $\left(\frac{400}{d}\right)$ = the correction factor for depth and should not be less than 1

$\left(\frac{f_{ck}}{25}\right)$ = the correction factor for the strength of concrete and should not be greater than 40

$\gamma_m = 1.25$

p_t = percentage steel, the value of which should not exceed 3

Design shear strength for solid slabs

For solid slabs, the design shear strength for concrete shall be $\tau_c \cdot k$, where k has the values given in Table 5.2

TABLE 5.2 VALUES OF k (IS 456 : 2000)

Overall depth of slab (mm)	300 or more	275	250	225	200	175	150 or less
k	1.00	1.05	1.10	1.15	1.20	1.25	1.30

Note : The above provision shall not apply to flat slabs.

Shear strength of members under axial compression (IS 456 : 2000)

For members subjected to axial compression P_{uc}, the design shear strength of concrete, given in Table 7.1, shall be multiplied by the following factor :

$$\delta = 1 + \frac{3\,P_{uc}}{A_g \cdot f_{ck}}, \text{ but not exceeding } 1.5 \quad \ldots.\ 5.6.3$$

where P_{uc} = factored axial compressive force in Newtons

A_g = gross area of concrete section in mm^2, and

f_{ck} = characteristic compressive strength of concrete, in N/mm^2

Shear strength of members under axial tension (ACI Code, 1989):

Though it is evident that there is some reduction in design shear strength of a member under axial tension, IS Code (IS 456 : 2000) does not explicitly mention this case. However, the following simplified expression for δ, based on ACI Code (1989) may be used :

$$\delta = 1 - \frac{P_{ut}}{3.45\,A_g} \quad \ldots\ldots\ 5.6.4$$

where P_{ut} = factored axial tensile force in Newtons.

Maximum shear stress in concrete with shear reinforcement (IS 456 : 2000)

(*a*) ***Maximum shear stress in beams*** :

Under no circumstances, even with shear reinforcement, shall the nominal shear stress (τ_v) in beams exceed $\tau_{c,max}$ given is Table 5.3

TABLE5.3.MAXIMUM SHEARSTRESSv_C,max (N/mm^2)

Gradeofconcrete	M15	M20	M25	M30	M35	M40&above
v_c,max (N/mm^2)	2.5	2.8	3.1	3.5	3.7	4.0

(b) Maximumshearstressinsolidslabs

Forsolidslabsthenominalshearstressshallnotexceedhalftheappropriatevaluesgivenintable5.3.

WEBREINFORCEMENTFORDIAGONALTENSION:

As stated earlier, proper reinforcement must be provided to resist the diagonal tension. The shear resisted by shear reinforcement can be worked out by considering the equilibrium of forces across a potential diagonal crack, which is assumed to be inclined at an angle of 45° with axis of the beam. Fig. 7.11 shows a diagonal crack AB. Let the web reinforcement be inclined at angle α with the axis of the beam, and be spaced at distance s_v apart. Let the diagonal crack AB intersect n number of *web reinforcing bars*.

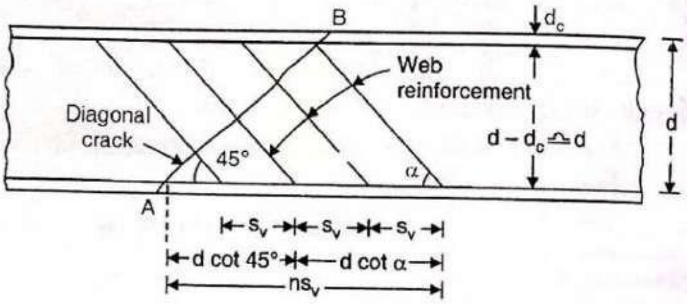


FIG. 5.7.1 SHEAR RESISTED BY WEB STEEL

Let V_{us} = Ultimate shear carried by shear (or web) reinforcement

f_{vd} = design yield stress in web steel $= 0.87\,f_y$

n = number of bars/links crossing the crack

α = inclination of web steel

A_{sv} = total cross-section area of each set of bar or link.

The web reinforcement is anchored to the main tensile steel at the bottom, and to the holding bars (at a cover d_c) at the top. Hence the vertical component of length of inclined bar $= (d - d_c)$. Since d_c in normally quite small . comparison to d, we can take $(d - d_c \simeq d)$, as marked in Fig. 5.7.1 Now, for equilibrium

Shear carried by shear reinforcement = *Sum of vertical components of tensile forces developed in shear reinforcement*

$$\therefore \quad V_{us} = n\,A_{sv}\,f_{vd}\sin\alpha \qquad \text{..... 5.7.1 (a)}$$

In order to get the value of n, we have from geometry,

$$n\,s_v = d\cot 45° + d\cot\alpha \quad \text{or} \quad n = \frac{d\cot 45° + d\cot\alpha}{s_v} = \frac{d\,(1+\cot\alpha)}{s_v} \qquad \text{... 5.7.1 (b)}$$

Substituting the value of n and f_{vd} ($= 0.87\,f_y$) in Eq. 7.13 (a) we get

$$V_{us} = \frac{d\,(1+\cot\alpha)}{s_v}\,.\,A_{sv}\,(0.87\,f_y)\sin\alpha = \frac{0.87\,f_y\,A_{sv}\,d}{s_v}(\sin\alpha + \cos\alpha) \qquad \text{.... 5.7.2}$$

Rearranging the above, we get

$$s_v = \frac{0.87\,f_y\,A_{sv}\,d}{V_{us}}(\sin\alpha + \cos\alpha) \qquad \text{..... 5.7.2 (a)}$$

Theaboveequationgivesthespacingofthebarsinclinedatαwithhorizontal.

HereA_{sv}=Area ofC/S ofbarsXNo oflegs=AØXNo oflegs.

SpecialCases:

(i) **Barsinclinedat 45⁰.**(i.e.α=45⁰)

If $\alpha = 45°$, $$V_{us} = \frac{0.87\, f_y\, A_{sv}\, d}{s_v}(\sqrt{2}) \quad \text{..... 5.7.3}$$

or $$s_v = \frac{0.87\, f_y\, A_{sv}\, d}{V_{us}}\sqrt{2} \quad \text{..... 5.7.3 (a)}$$

(*ii*) *Bars inclined at 90° (i.e. vertical stirrups)*

$$V_{us} = \frac{0.87\, f_y\, A_{sv}\, d}{s_v} \quad \text{.... 5.7.4}$$

or $$s_v = \frac{0.87\, f_y\, A_{sv}\, d}{V_{us}} \quad \text{...... 5.7.4 (a)}$$

(*iii*) *Single bar or single group of bars*

For a single bar, or single group of bars, all bent up at the same cross-section, we get from Eq. 5.7.1 (a) taking $n = 1$

$$V_{us} = 0.87\, f_y \,.\, A_{sv} \sin\alpha \quad \text{.... 5.7.5}$$

TYPESOFSHEARREINFORCEMENT.

Shear reinforcement is necessary if the nominal shear stress (v_v) exceeds the design shear stress vc .Ingeneral shearreinforcementisprovided inanyone ofthefollowingthreeforms.

(a) Verticalstirrups
(b) Bendupbarsalongwiththestirrups.
(c) Inclinedstirrups.

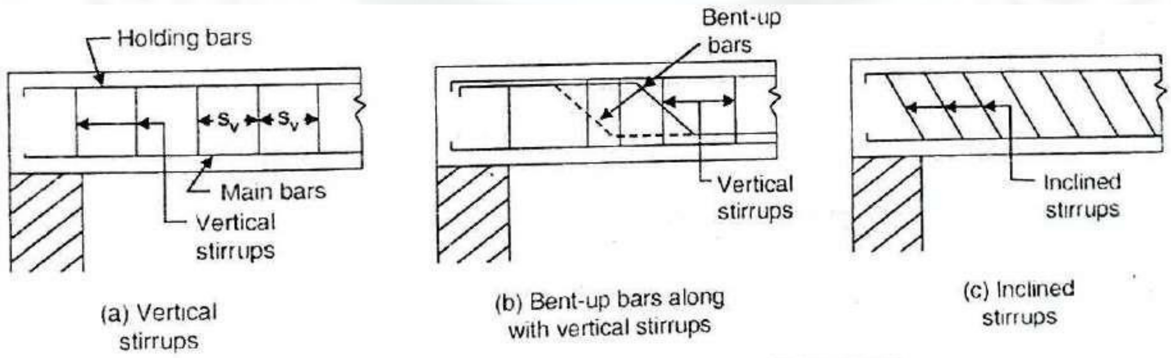


FIG. 5.8 TYPES OF SHEAR REINFORCEMENT

Wherebent-

upbarsareprovided,theircontributiontowardsshearresistanceshallnotbemorethan halfthat oftotal shear reinforcement.

The total external shear V_u is jointly resisted by concrete as well as shear reinforcement and isrepresentedbythe expression

$V_u = V_{uc} + V_{us}$

WhereV_{uc}=Shearstrengthofconcre

teandV_{us}=Shear reinforcement.

VERTICAL STIRRUPS:

Shear reinforcement in the form of vertical stirrups consists of 5 mm to 15 mm dia steel bars bendroundthetensilereinforcementwhereit isanchoredto6to 12mmdia.Anchorbarsorholdingbars.Depending upon the magnitude of the shear stress to be resisted , a stirrup may be one legged, twolegged,four legged or multilegged, asshown in Figure.

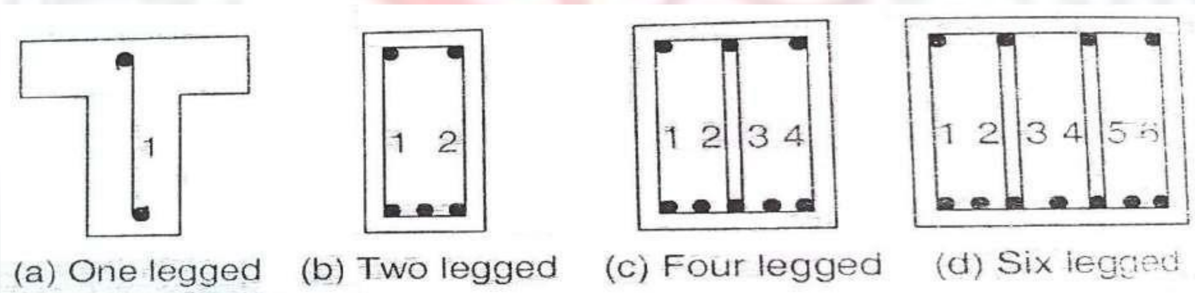


FIG-5.9. FORMSOFVERTICALSTIRRUPS

Thestrengthofshearreinforcementintheformofverticalstirrupsisgivenby

$$V_{uc} = \frac{0.87 f_y A_{sv} d}{S_v} \quad \text{.........5.9.1}$$

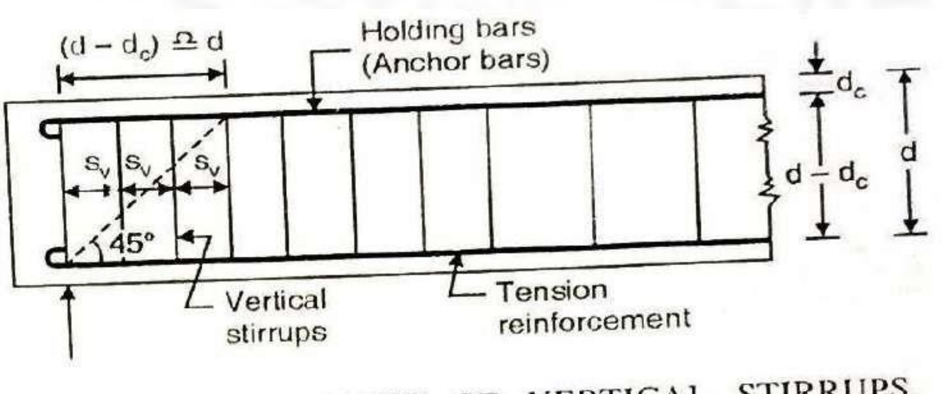


FIG. 5.9.1 SPACING OF VERTICAL STIRRUPS

Here $A_{sv} = mA_{\emptyset}$, where m = no of legs in the stirrups and $A_{\emptyset}$ = Area of c/s of the bar stirrups.

Let us assume that in absense of shear reinforcement, the beam fails in diagonal tension, theinclination of the tenson crack being at 45^0 to the axis of the beam and extended up to a horizontaldistanceequalto (d- dc) =d

HenceNo ofstirrups resistingshearforce =d/Sv, Or,

MINIMUMSHEARREINFORCEMENT (IS456 :2000)

The shear reinforcement in the form of stirrups remain unstressed till the diagonal crack occurs at thecritical location. However, the instant a diagonal crack occurs. The web reinforcement receivessudden increase in stress. If web reinforcement is not provided. Shear failure may occur withoutgiving any warning. The code therefore, specifies that all the beams should be provided with atleastsomeminimumreinforcementcallednominalshearreinforcementevenifnominalshearstressislessthanthe design shear stressof concrete.

Reasonsforprovidingminimumshearreinforcement:

1. It prevents *sudden* shear failure with the formation of diagonal tension crack, and imparts *ductility* to provide sufficient warning of impending failure. Thus *brittle shear failure* is prevented.

2. It guards against any sudden failure of a beam if concrete cover bursts and bond to tension steel is lost.

3. It holds the main reinforcements in place while pouring the concrete. Thus minimum requirement of cover and clear distance between longitudinal bars are maintained.

4. It acts as necessary ties for the compression steel (if any) and makes it effective.

5. It prevents pressing down of the longitudinal reinforcement, thereby maintaining the *dowel capacity*.

6. It confines the concrete, thereby increasing its strength and rotation capacity.

7. It prevents failure that can be caused by tension due to shrinkage and thermal stresses and internal cracking in the beam.

As per IS 456 : 2000, minimum shear reinforcement in the form of stirrups shall be provided such that 5.10.1

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

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 beam or breadth of the web of flanged beam.

f_y = characteristic strength of stirrup reinforcement in N/mm^2, which *shall not be taken greater than 415 N/mm^2*.

Hence spacing based on minimum shear reinforcement is given by 5.10.2

$$s_v \leq \frac{0.87\,f_y\,.\,A_{sv}}{0.4\,b} \leq \frac{2.175\,f_y\,A_{sv}}{b}$$

However, 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.

Shear resistance of minimum shear reinforcement

The shear resistance of minimum reinforcement envisaged in Eq. 5.10.1 is found by substituting the value of $\frac{0.87\,f_y\,.\,A_{sv}}{s_v} = 0.4\,b$ 5.10.3

Thus, $$V_{us,\,min} = \left(0.87\frac{f_y\,A_{sv}}{s_v}\right)d = (0.4\,b)\,d = 0.4\,bd$$

Thus, shear carried by concrete and that carried by minimum stirrups is given by 5.10.4

$$V_{u,\,min} = \tau_c\,.\,bd + 0.4\,bd$$

MAXIMUMSPACING OFSHEARREINFORCEMENT:-

The maximum spacing of shear reinforcement measured along the axis of the member shall notexceed 0.75d for vertical stirrups and d for inclined stirrups at 45^0, where d is the effective depth ofthesectionunderconsideration. In nocase shallthespacingexceed300 mm.

Example-5.1.

A reinforced concrete beam 250 mm wide and 400 mm effective depth is subjected to ultimatedesignshearforceof150KNatthecriticalsectionnearsupports.Thetensilereinforcementatthesection near supports is 0.5 percent. Design the shear stirrups near the supports also design theminimumshearreinforcementatthemid span.Assume M20concreteand Fe250mildsteel.

Solution : Given : $b = 250$ mm ; $d = 400$ mm; $A_{st}/bd = 0.5\% = 0.005$

$$\tau_v = \frac{V_u}{bd} = \frac{150 \times 10^3}{250 \times 400} = 1.5 \text{ N/mm}^2$$

From Table 5.1 $\tau_c = 0.48$ N/mm² for M 20 concrete and $100\, A_{st}/bd = 0.5$

Also, from Table 5.3 $\tau_{c,\,max} = 2.8$ N/mm² for M 20 concrete.

Thus, τ_v is less than $\tau_{c,\,max}$, but greater than τ_c. Hence shear reinforcement is necessary.

$$V_{uc} = \tau_c\, bd = 0.48 \times 250 \times 400 = 48000 \text{ N}$$

Hence $V_{us} = V_u - V_{uc} = 150000 - 48000 = 102000$ N

The shear resistance of nominal stirrups is given by

$$V_{us,\,min} = 0.4\, bd = 0.4 \times 250 \times 400 = 40000 \text{ N} < V_{us}$$

Hence nominal stirrups are *not* sufficient t the section near supports.

We Know that $s_v = \dfrac{0.87 f_y A_{sv}}{V_{us}} \cdot d$

Using two legged stirrups of 10 mm dia. bars, $A_{sv} = 2\,\frac{\pi}{4}(10)^2 = 157.08$ mm²

$$\therefore \quad s_v = \frac{0.87 \times 250 \times 157.08}{102000} \times 400 \approx 134 \text{ mm}$$

Again we know $s_v = \dfrac{0.87 f_y A_{sv}}{(\tau_v - \tau_c)\, b} = \dfrac{0.87 \times 250 \times 157.08}{(1.5 - 0.48)\, 250} \approx 134$ mm

Maximum spacing = $0.75\, d$ or 300 mm, which ever is less.

Hence provide 10 mm dia. two legged stirrups @ 130 mm c/c at the section near supports.

At mid-span, the spacing of minimum shear reinforcement for 10 mm ϕ – 2 lgd stirrups is given by Eqn 5.10.2

$$s_v = 0.87\,\frac{f_y A_{sv}}{0.4\, b} = \frac{0.87 \times 250 \times 157.08}{0.4 \times 250} = 341.6 \text{ mm}$$

However, maximum spacing is limited to $0.75\, d$ or 300 mm which ever is less.

Hence s_v = 300 mm

Hence provide 10 mm dia. two legged stirrups @ 300 mm c/c at the mid-span.

Example- 5.2 -

A simply supported beam, 300 mm wide and 500 mm effective depth carries a uniformly distributedload of 50 KN/m, including its own weight over an effective span of 6 m. Design the shearreinforcementintheformofverticalstirrups.Assumethatthebeamcontains0.75 %ofreinforcementthroughout the length. The concrete is of M 20 grade and steel for stirrups is of Fe 250 grade. Takewidth ofsupport as400 mm.

Solution:- W_u= 1.5 X 50 = 75

KN/m.V_{umax}=W_uL/2 =(75

X6)/2 =225 KN

Thecriticalsectionliesatadistanceofd= 500mmfrom thefaceofsupport oratadistanceof500 +400/2= 700 mm from thecentre ofthe support.

V_{uD}= 225-75 X0.7 = 172.5 kN.

And v_v=(172.5 X10^3)/ (300 X 500) = 1.15 N/ mm^2.

From Table-5.1 for 100 A_s/bd = 0.75%, we get v_c= 0.56 N/ mm^2 for

M20 Concrete.V_{uc}+ 0.56 X300 X500 =84000 N = 84 KN.

Also, $\tau_{v,max} = 2.8$ N/mm^2 for M 20 concrete. Since $\tau_v < \tau_{v,max}$ it is OK.

However, $\tau_v > \tau_c$; hence shear reinforcement is necessary.

$$V_{us} = V_{uD} - V_{uc} = 172500 - 84000 = 88500 \text{ N}$$

Using 10 mm φ 2-lgd vertical stirrups, $A_{sv} = 2\frac{\pi}{4}(10)^2 = 157.1$ mm^2

$$\therefore \text{ Spacing } s_v = \frac{0.87 f_y \cdot A_{sv} \cdot d}{V_{us}} = \frac{0.87 \times 250 \times 157.1 \times 500}{88500} = 193 \text{ mm} \simeq 190 \text{ mm (say)}$$

Spacing corresponding to minimum shear reinforcements is

$$s_v = \frac{0.87 f_y A_{sv}}{0.4\, b} = \frac{0.87 \times 250 \times 157.1}{0.4 \times 300} = 284.7 \text{ m} \simeq 280 \text{ mm (say)}$$

However in no case should the spacing exceed $0.75\, d = 0.75 \times 500 = 375$ mm, or 300 m whichever is less. Hence the spacing is to vary from 190 mm at the end section @ 280 mm at a section distant x m (say) from the mid-span. Let us locate this section where the S.F. is V_{ux}.

$$\therefore \quad V_{ux} = \frac{V_{u,max}}{3} x = \frac{225000}{3} x = 75000\, x$$

$$\therefore \quad V_{us} = V_{ux} - V_{uc} = 75000\, x - 84000$$

$$s_v = 280 = \frac{0.87 \times 250 \times 157.1 \times 500}{75000\, x - 84000}$$

from which, we get $x = 1.93$ m from mid-span or 1.07 m from supports. Hence provide 8 mm φ 2 lgd stirrups at a spacing of 190 mm c/c from supports to a section distant 1.07 m from the centre of either supports. For the remaining length, provide the stirrups @ 280 mm c/c.

CHAPTER-6

BOND,ANCHORAGE,DEVELOPMENTLENGTHS,ANDSPLICING

5. **BOND :**

One of the most important assumption in the behavior of reinforced concrete structure is that there isproper ‘bond’ between concrete and reinforcing bars. The force which prevents the slippage betweenthe two constituent materials is known as bond. In fact , bond is responsible for providing ‘ straincompatibility ’ and composite action of concrete and steel. It is through the action of bond resistancethat the axial stress (tensile or compressive) in a reinforcing bar can undergo variation from point topoint along its length. This is required to accommodate the variation in bending moment along thelengthofthe flexural member.

When steel bars are embedded in concrete, the concrete, after setting, adheres to the surface ofthebarandthusresistsanyforcethattendstopullorpushthisrod.Theintensityofthisadhesiveforce bond stress. The bond stresses are the longitudinal shearing stress acting on the surface betweenthe steel and concrete, along its length. Hence bond stress is also known as interfacial shear. Hencebondstressistheshearstressactingparalleltothereinforcingbarontheinterfacebetweenthebarandtheconcrete.

TYPESOFBOND:-

Bond stress along the length of a reinforcing bar may be induced under two loading situations, andaccordinglybond stressesare twotypes:

1. FlexuralbondorLocalbond

2. Anchoragebond ordevelopmentbond

Flexural bond (τbf) is one which arises from the change in tensile force carried by the bar, along itslength, due to change in bending moment along the length of the member. Evidently, flexural bond iscritical at points where the shear (V=dM/dx) is significant. Since this occurs at a particular section,flexuralbond stressisknown aslocal bond stress[Fig-5.1(b)].

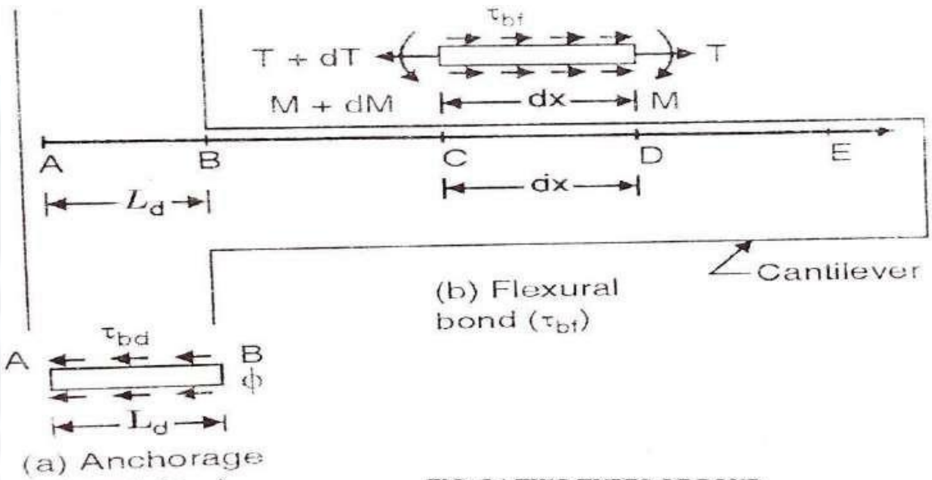


FIG- 5.1 TWO TYPES OF BOND

Anchoragebond(τ_{bd})isthatwhicharisesoverthelengthofanchorageprovidedforabar.It alsoarisesneartheendorcutoffpointofreinforcingbar.Theanchoragebondresiststhe'p ullingout'ofthe bar if it is in tension or 'pushing in' of the bar if it is in compression. Fig.[8.1 (a)] shows thesituation of anchorage bond over a length AB(=L_d). Since bond stresses are developed over specifiedlengthL_d,anchoragebondstressisalsoknownasdevelopedoveraspecifiedleng thL_d,anchoragebondstressisalso knownasdevelopment bondstress.

Anchoring of reinforcing bars is necessary when the development length of the reinforcement islarger than the structure. Anchorageis used so that thesteel's intended tension load can bereachedandpop-outswill notoccur. Anchorageshapescantake theform of180 or90degreehooks.

5.2. ANCHORAGEBONDSTRESS:

Fig- 5.2 shows a steel bar embedded in concrete And subjected to a tensile force T. Due to this forceThere will be a tendency of bar to slip out and this tendency is resisted by the bond stress developedoverthe perimeter ofthe bar,alongitslength ofembedment .

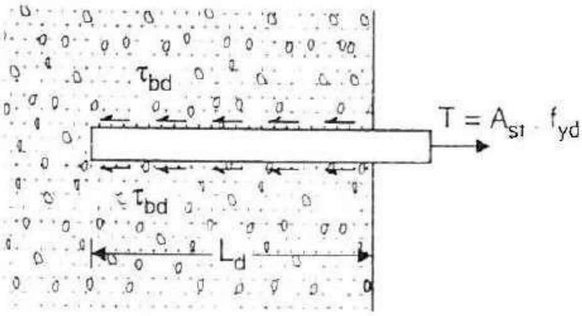


FIG- 5.2

Let us assume that average uniform bond stress is developed along the length. The required lengthnecessary todevelopfullresisting forceiscalled**Anchoragelength**in caseof axialtensionorcompressionand**developmentlength**incaseofflexuraltensionand isdenoted byL_d.

DESIGNBONDSTRESS:-

Thedesignbondstressinlimitstatemethodforplainbarsintensionshallbeasgivenbelow(Table 6.1)

Table- 6.1

Gradeof concrete	M 20	M 25	M 30	M 35	M40 and above
Designbondstressτ_{bd}(N/ mm^2)	1.2	1.4	1.5	1.7	1.9

Design bond stresses for deformed bars in tension : For deformed bars conforming to IS 1786.Thesevaluesshallbe increasedby 60%.

Designbondstressforbarsincompression: For barsincompression,thevaluesofbondstressforintension shall beincreased by25%.

DEVELOPMENTLENGTH OF BARS(IS456 :2000)

Thedevelopmentlengthisdefinedasthelengthofthebarrequiredoneithersideofthesection under consideration, to develop the required stress in steel at that section through bond. Thedevelopmentlength $L_d$givenby

$$L_d=\varphi \sigma_s/4\ \tau_{bd}=k_d\varphi \text{ ....5.4.1}$$

Where φ=nominaldiameterofthebar

σ_s=stressinbaratthesectionconsideredatdesig

nloadk_d=development length factor= $\sigma_s/4\tau_{bd}$

Note: Thedevelopmentlengthincludes theanchoragevalues ofhooks intensionreinforcementTaking

σ_s=0.87fyatthecollapsestage,kd= 0.87fy/4τ_{bd}

5.4.2

For bars in compression, the value of τ_{bd}given in table 1.1 are to be increased by 25%. Hencedevelopedlength (L_{dc}) for barsin copressio isgivenby

L_{ds}=φ σ_{sc}/5 τ_{bd} 5.4.3

Hencethevaluesof$k_d$forbarsIn compressionwillbe $=0.87f_y/5\tau_{bd}$

Table 6.2 gives thevalues of development length factor forvariousgradesofconcrete and thevarious grades of steel,both in tension as wellas compression. Thevalues havebeen rounded-off tothehigher side.

TABLE6.2-VALUESOFDEVELOPMENT LENGTHFACTOR

Gradeof concrete	M 20			M 25		
Gradeof steel	Fe250	Fe415	Fe500	Fe250	Fe415	Fe500
Barsin tension	46	47	57	39	41	49
Barsin comp.	37	38	46	31	33	39

Grade ofcon crete	M30			M35			M40		
Gradeof steel	Fe250	Fe415	Fe500	Fe250	Fe415	Fe500	Fe250	Fe415	Fe500
Bars in tension	37	38	46	32	34	40	29	30	36
Bars incomp.	29	31	37	26	27	32	23	24	29

Note : When the actual reinforcement provided is more than that theoretically required, so that theactual stress (σ_s) in steel is less than thefull deign stress (0.87 fy), thedevelopment length requiredmaybe reduced bythe followingrelation:

Reduceddevelopmentlength $L_{dr}=L_d(A_{st}\text{required}\div A_{st}\text{provided})$

This principle is used in the design of footing and other short bending members where bond iscritical.Byprovidingmoresteel, the bondrequirementsare satisfied.

Bars bundled in contact : The development length of each bar bundled bars shall be that forthe individual by 10% for two bars in contact,20% for three bars in contact and 33% for four bars incontact.

STANDARDHOOKS&BENDSFORENDANCHORAGEANCHORAGELENGTH

The development length required at the end of a bar is known as *anchorage length.* However, in the case of development length, the force in the bar is developped by transfer. of force from concrete to steel, while in the case of anchorage length, there is dissipation of force from steel to concrete.

Quite often, space available at the end of beam is limited to accommodate the full development length L_d. In that case, hooks or bends are provided. The anchorage value (L_e) of hooks or bend is accounted as contribution to the development length L_d.

Fig. 5.5 (*ai*) shows a semi-circular hook, fully dimensioned, with respect to a factor *K*. The value of *K* is taken as 2 in the case of mild steel conforming to IS : 432–1966, (specifications for Mild-Steel and Medium Tensile Steel bars and Hard-Drawn steel wires for concrete reinforcement) or IS : 1139–1959. (specifications for 'Hot rolled mild steel and medium tensile steel deformed bars for concrete reinforcement'). The hook with $K = 2$ is shown in Fig. 5.5 (*aii*) with equivalent horizontal length of the hook. For the case of Medium Tensile Steel conforming to IS : 432–1966 or IS : 1139–1959, *K* is taken as 3. In the case of cold worked steel conforming to IS : 1986–1961, (specifications for cold twisted steel bars for concrete reinforcement), *K* is taken as 4. In the case of bars above 25 mm, however, it is desirable to increase the value of *K* to 3, 4 and 6 respectively.

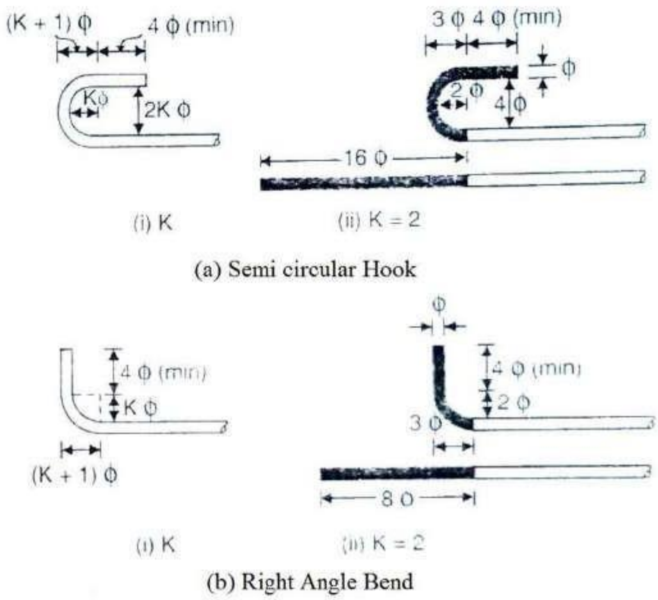


FIG- 5.5

Fig-5.5showsarightangledbend,withdimensionsintermsofK,thevalueofwhichmaybetakenas2 forordinarymildsteel for diametersbelow25mm and 3for diametersabove 25mm.

In the case of deformed bars , the value of bond stress for various grades of concreteisgreater by60% than the plane bars. Hence deformed bars may be used without hooks, provided anchoragerequirementsare adequatelymetwith.

CODEREQUIREMENTSFORANCHORINGREINFORCINGBARS (IS456:2000)

(i) **Anchoring Bars in Tension :-** Deformed bars may be used without end anchoragesprovided development length required is satisfied. Hooks should normally be provided forplain bars in tension. The anchorage value of bend shall be taken as 4 timesthe diameterof the barfor each 45° bend subject to amaximum of 16 times the diameter of the bar.The anchorage value of a standard U-type hook shall be equal to 16 times the diameter ofthebar.

(ii) **Anchoring Bars in Compression :-** The anchorage length of straight bar in compressionshallbeequaltothedevelopmentlengthofbarsincompression. Theprojectedlengthof hooks, bends and straight lengths beyond bends if provided for a bar in compression, shallbeconsidered for developmentlength.

(iii) **AnchoringShearReinforcement:-**

Inclined bars :- The developments length shall be as for bars in tension ; this length shallbe measured as under : (1) in tension zone from the end of the sloping or inclined portionofthebar and (2)inthecompressionzone, frommiddepthofthebeam.

Stirrups :- Notwithstanding any of the provisions of this standard, in case of secondaryreinforcement , such as stirrups and traverse ties,complete development lengths andanchorageshallbe deemedtohave beenprovidedwhenthe barisbentthroughanangleofatleast 90° round a bar of atleast its own diameter and is continued beyond the end of thecurve for a length of atleast eight diameters, or when the bar is bent through an angle of135° and is continued beyond the end of curve for a length of atleast six bar diameters orwhen the bar is bent through an angle of 180° and iscontinued beyond the end of thecurveforalengthatleastfourbardiameters.

CHECKINGDEVELOPMENTSLENGTHOF TENSIONBARS:-

As statedearlier,thecomputedstress (σ_s)in a reinforcing bar,at every section mustbedevelopments on both the sides of section. This is done by providing development length $L_d$to bothsides of the section. Such a developments length isusually available at mid-span location wherepositive (or sagging) B.M. is maximum for simply supported beams. Similarly, such a developmentslengthisusually availableattheintermediatesupportofacontinuousbeamwherenegative(orhogging) B.M. is maximum. Hence no special checking may be necessary in such locations. Howeverspecialcheckingfor developmentslength isessential atthe followinglocations:

1. At simplesupports
2. Atcantileversupports
3. Inflexuralmembersthathaverelatively shortspans
4. Atpointsofcontraflexure
5. Atlapsplices
6. Atpointsofbarcutoff
7. Forstirrupsandtransverselies.

DEVELOPMENTS LENGTH REQUIREMENTS AT SIMPLE SUPPORTS

:DIAMETEROF REINFORCING BARS:-

The code stipulates that at the simple supports (and at the point of inflection), the positive momenttensionreinforcement shall belimitedto adiameter such that

$$L_d \leq M_1/V + L_o \text{...} 5.8.1$$

Where L_d=developmentslengthcomputedfordesignstress f_{yd}(=0.87fy) fromEq^n

M_1= Moments resistance of the section assuming allreinforcement at the sectionto bestressedto f_{yd}(= 0.87 fy)

V=Shearforceatthesectionduetodesignloads

Lo=sumofanchoragebeyondthecentreofsupportsandtheequivalentanchoragevalueofanyhookormechanicalanchorageatthesimplesupport(Atthepointofinflexion,Lo islimitedto*d*or12φwhich ever isgreater).

The code further recommends that the value of *M1/V* in eq^n - 5.8.1 may be increased by

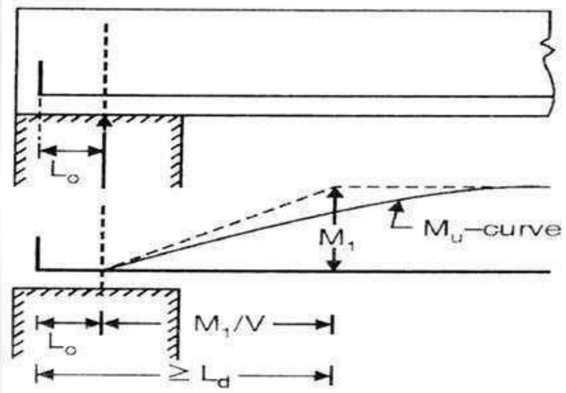


FIG-5.8.1

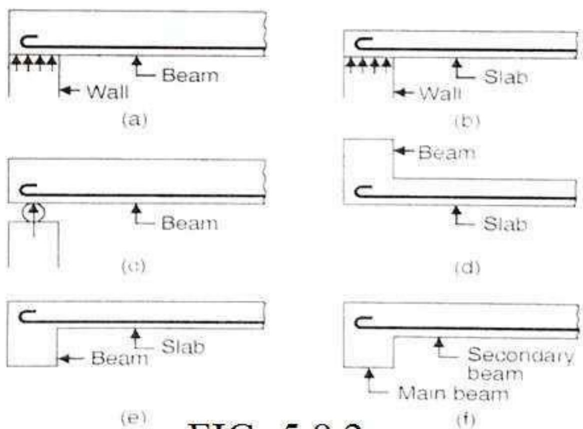


FIG- 5.8.2

30%whentheendsofthereinforcementareconfinedbyacompressivereaction.Thisconditionofconfinementofreinforcingbarsmaynot beavailable atallthe typesofsimplesupports.

Four type of simple supports are shown in fig-5.8.2. In fig- 5.8.2 (a) , the beam is simplysupported on a wall which offers a compressive reaction which confines the ends of reinforcement.Hence a factor 1.3 will be applicable. However in fig-5.8.1 (c) and (d) though a simple support isavailable , the reaction does not confine the ends of the reinforcement, hence the factor 1.3 will not beapplicable with M1/V term.Simillarly for the case of a slab connected to a beam Fig- 5.8.2€ or for thecaseofsecondarybeam connected to amainbeam [Fig-5.8.2(f)]

Tensilereactionisinducedandhenceafactor1.3willnotbeavailable.

Thusatsimplesupportswherethecompressivereactionconfinestheendsofreinforcing barswehave$L_d \leq 1.3 M_1/V + L_0$................................ 5.8.2

ComputationoftheMomentof Resistance M_1ofbarsavailableatsupports:

In eqn 5.8.1 , M_1 = Moment of Resistance of the section corresponding to the area of steel (A_{st})continuedintothe supportandstressedtodesignstressequaltodesign stressequalto 0.87fy.

$M_1 = 0.87 f_y . A_{st} (d - 0.416 X_u)$ 5.8.3

Where $X_u = 0.87 f_y . A_{st} / 0.36 f_{ck} b$ ....5.83(a)

ComputationofLength(L_0) :

For the computation of L_0 , the supportwidth should be known.Fig- 5.8.3 (a) and (b) show abeamwithend supportwitha standard hook and90^0bendrespectively.

Let X be the sidecover to the hook (Or bend) and X_0 be the distance of the beginning of the hook (OrBend) from thecenterline ofthesupport.

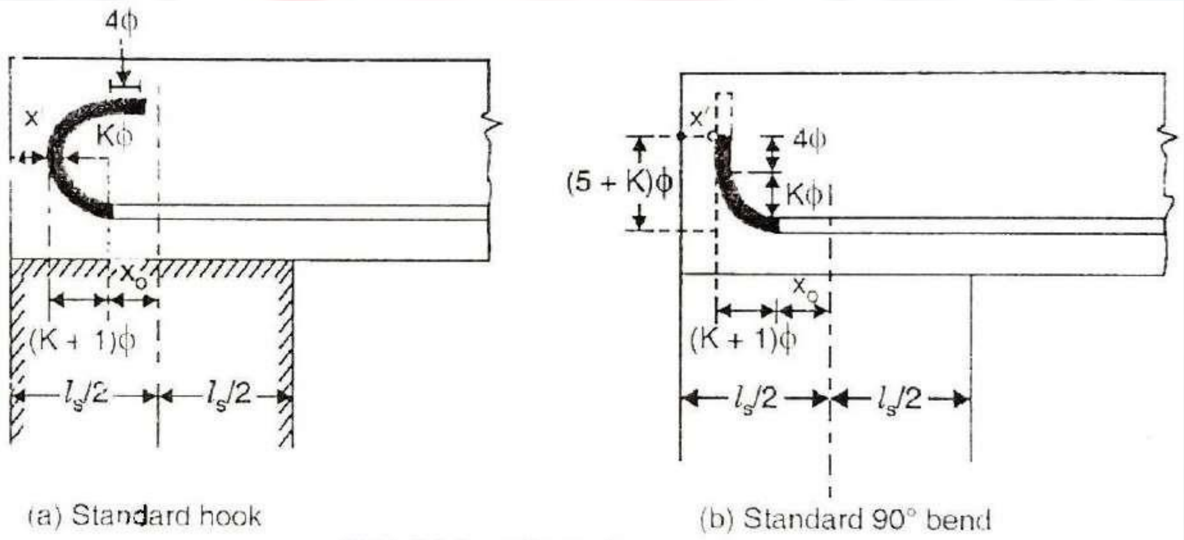


FIG 5.8.3 COMPUTATION OF L_0

(a) **Case-I : Standard Hook at the end [Fig-5.8.3(a)]:-** The dark portion showsthehookwhich has an anchorage value of 163 (IS 456: 2000) for all types of steel. The distance ofthe beginning of the hook from its apex of the semi circle is equal to (K+1)3 . For mild steelbars K=2 and for HYSD bars, K=4, Hence the distance 3 for mild steel and 53 for HYSDbars.Let/ bethewidthofthesupport.

CONDITIONS FOR CURTAILMENT OF REINFORCEMENT

In most of the cases, the B.M. varies appreciably along the span of the beam. From the point of view of economy, the moment of resistance of the beam should be reduced along the span according to the variation of B.M. This is effectively achieved by reducing the area of reinforcement, i.e. by curtailing the reinforcement provided for maximum B.M. In general, all steel, whether in tension or in compression, should extend d or $12\,\phi$ (which ever is greater) beyond the theoretical point of cut off (TPC).

Conditionsforterminationoftensionreinforcementinflexuralmembers:

Curtailment of Flexural tension reinforcement results in the loss of shear strength in the region ofcutoffandhenceitisnecessarytomakeprovisiontoguardagainstsuchloss.Flexuralreinforcementshallnotbeterminatedina tensionzoneunlessanyone ofthefollowingconditionissatisfied.

(*a*) The shear at the cutoff point does not exceed two thirds that permitted, including the shear strength of web reinforcement. In other words, the total *shear capacity* shall be atleast 1.5 times the applied shear at the point of curtailment, thus

$$V_u \not> \frac{2}{3}(V_{uc} + V_{us}) \quad \text{or} \quad V_{uc} + V_{us} \geq 1.5\,V_u$$

Where V_{uc} = shear capacity of concrete, based on continuing reinforcement only.

V_{us} = shear capacity of shear reinforcement

V_u = applied shear at the point of curtailment.

(*b*) Stirrup area in excess of that required for shear and torsion is provided along each terminated bar over a distance from cutoff point equal to three fourth the effective depth of the member. Excess area of shear reinforcement is given by :

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

where $$s_v \not< \frac{d}{8\,\beta_b} \not> \frac{0.87\,f_y\,A_{sv}}{0.4\,b}$$

$$\beta_b = \frac{\text{area of bars cutoff at the section}}{\text{total area of bars at the section}}$$

(*c*) For 36 mm or smaller bars, the continuing bars provide double the area required for flexure at the cutoff point and the shear does not exceed three fourth that permitted.

Thus, $$M_{ur} \geq 2\,M_u$$

and $$V_{uc} + V_{us} \geq 1.33\,V_u$$

where M_{ur} = moment of resistance of remaining (or continued) bars

M_u = B.M. at cutoff point ; V_u = S.F. at cutoff point

5.9 DEVELOPMENT LENGTH AT POINT OF INFLEXION

Fig. 8.8 shows the conditions at a point of inflection (P.I.) As already indicated in § 8.11, the Code states that the following condition be satisfied

$$\left(\frac{M_1}{V} + L_0\right) \geq L_d \quad \text{...... 5.9.1}$$

where L_0 should not be greater than d or 12 φ whichever is greater, and V is the shear force at the point of inflexion.

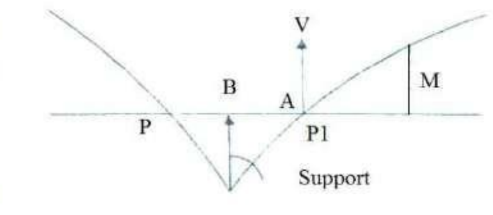


FIG. 5.9 DEVELOPMENT LENGTH AT A POINT OF INFLEXION

SPLICING:

(a) Thepurposeofsplicingistotransfereffectivelytheaxialforcefromtheterminatingbartotheconnectingbarwiththesamelineofactionat thejunction.[Fig-5.10(a)].

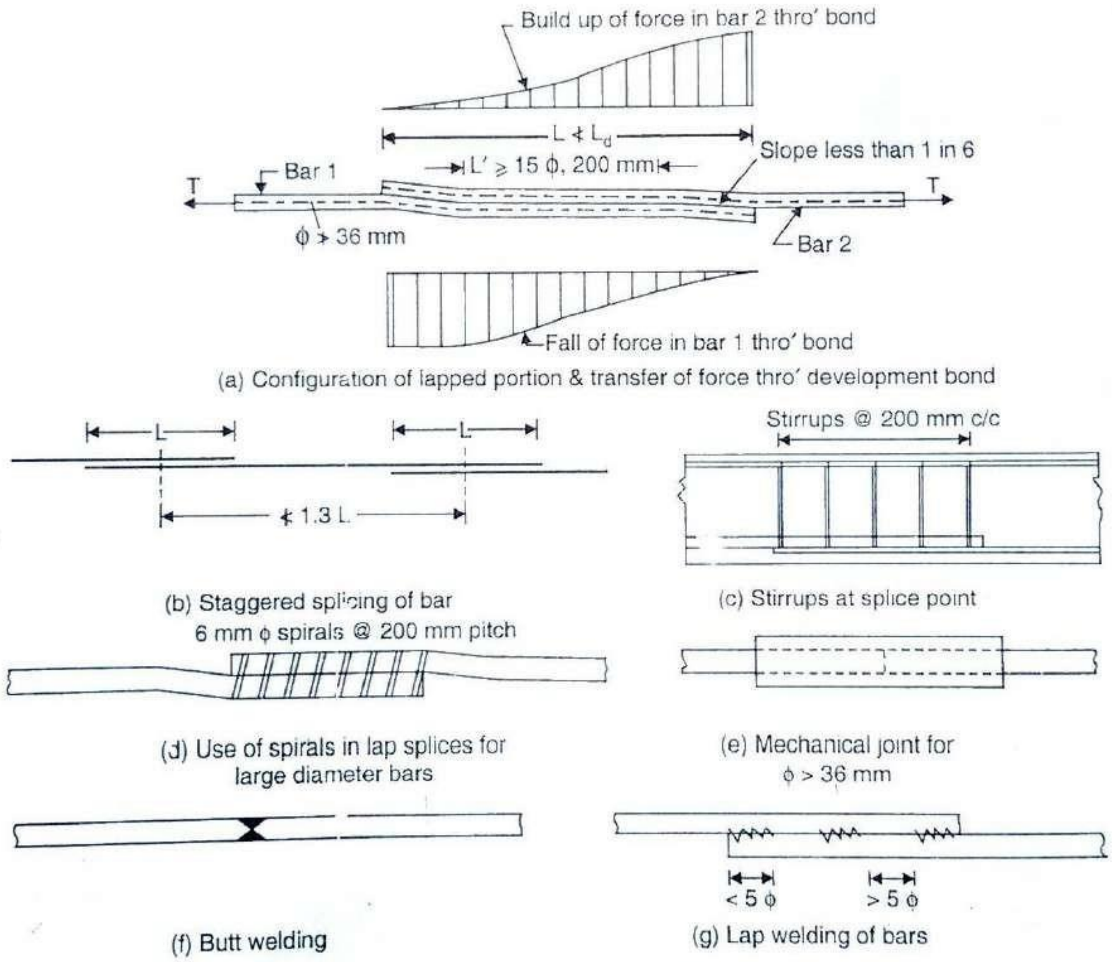


FIG- 5.10 REINFORCEMENT SPLICING

Slicingofabarisessentialinthefieldduetoeitherthe requirementsofconstructionornona vailability of bars of desired length. The Figures given are as per the recommendation of the IS 456 :2000.

(a) Lap slices shall not be used for bars larger than 36 mm. For larger diameters bars may be weld. Incase where welding is not practicable , lapping of bars larger than 36mm may be permitted, in whichcaseadditionalspiralshouldbeprovidedaroundthelappedbars[Fig-5.10(d)].

(*b*) Lap splices shall be considered as staggered if the centre to centre distance of the splices is not less than 1.3 times the lap length calculated as described in (*c*).

(*c*) The lap length including anchorage value of hooks for bars in *flexural tension shall be* L_d *or* 30 φ whichever is greater and for *direct tension* shall be 2 L_d or 30 φ whichever is greater. The *straight length* (*L'*) of the lap shall not be less than 15 φ or 200 mm (Fig. 5.10 [a]) The following provisions shall also apply :

(1) Top of a section as cast and the minimum cover is less than twice the diameter of the lapper bar, the lapped length shall be increased by a factor of 1.4.

(2) Corner of a section and minimum cover to either face is less than twice the diameter of the lapped bar or where the clear distance between adjacent laps is less than 75 mm or 6 times the diameter of lapper bar, whichever is greater, the lap length should be increased by a factor of 1.4.

Where both conditions (1) and (2) apply , the lap length should be increased by a factor of 2.0.

Note : Splices in tension members shall be enclosed in spirals made of bars not less than 6 mm diameter with pitch not more than 100 mm.

(*d*) The lap length in compression shall be equal to the development length in compression, but not less than 24 φ.

(*e*) When bars of two different diameter are to be spliced, the lap length shall be calculated on the basis of diameter of the smaller bar.

(*f*) When splicing of welded wire fabric is to carried out, lap splices of wires shall be made so that overlap measured between the extreme cross wires shall be not less than spacing of cross wires plus 100 mm.

(*g*) In case of bundled bars, lapped splices of bundled bars shall be made by splicing one bar at a time : such individual splices within a bundle shall be staggered.

Strengthof Welds:

The following values may be used where the strength of weld has been proved by tests to be at least asgreatasthat ofthe parentbars.

(a) Splicesincompression:

Forweldedsplicesandmechanicalconnection,100percentofthedesignstrengthofjoinedbars.

(b) Splicesintension:

(1) 80%ofthedesignstrengthofweldedbars(100%ifweldingisstrictlysupervised andifatanyc/softhemembernot morethan20%ofthetensilereinforcementiswelded)

(2) 100%ofthedesignstrengthofmechanicalconnection.

End Bearing Splices: End bearing splices should be used only for bars in compression. These are ofsquarecut andconcentricbearingensured bysuitable devices.

EXAMPLE-6.1

A SIMPLY SUPPORTED IS 25 cm X50cm and has 2 – 20 mm TOR bars going into the support. Ifthe shear force at the center of the support is 110 KN at working loads, determine the anchoragelength.assumeM20mixand Fe 415 grade TORsteel.

Solution:-

Foraloadfactorequalto1.5,thefactoredSF=1.5x110=16

5kN.Assuming25mm clearcover tothe

longitudinalbars

Effectivedepth =5000- 25– 20/2= 465mm.

Characteristic strength of TOR steel σ_y = 415 N/mm^2

Moment of resistance $M_1 = 0.87\,\sigma_y A_t (d - 0.42\,x)$

$$x = \frac{0.87\sigma_y A_t}{0.36\sigma_{ck}\, b} = \frac{0.87 \times 415 \times 628}{0.36 \times 20 \times 250} = 126 \text{ mm} < x_m \qquad \text{OK}$$

or $M_1 = 0.87 \times 415 \times 2 \times \pi/4 \times 20^2 (465 - 0.42 \times 126) = 93.45 \times 10^6$ Nmm

Bond stress τ_{bd} = 1.2 N/mm^2 for M20 mix. It can be increased by 60% in case of TOR bars.

$$\text{Development length } L_d = \frac{\phi\sigma_s}{4\tau_{bd}} = \frac{0.87 \times 415\phi}{4 \times (1.6 \times 1.2)} = 47\,\phi$$

If the bar is given a 90° bend at the centre of support, its anchorage value

$$L_o = 8\,\phi = 8 \times 20 = 160 \text{ mm}$$

$$L_d \le 1.3\, M_1/V + L_o$$

$$47\,\phi \le \left[\frac{1.3 \times 93.45 \times 10^6}{165 \times 1000}\right] + 160$$

or, $\phi \le 19$ mm

Since actual bar diameter of 20 mm is greater than 19 mm, there is a need to increase the anchorage length. Let us increase the anchorage length L_o to 240 mm. It gives

$$\phi \le 20.8 \text{ mm} \qquad \text{OK}$$

The arrangement of 90° bend is shown in Fig. 8.19a.

Alternatively

Provide a U bend at the centre of support, its anchorage value,

$$L_o = 16\phi = 320 \text{ mm}$$

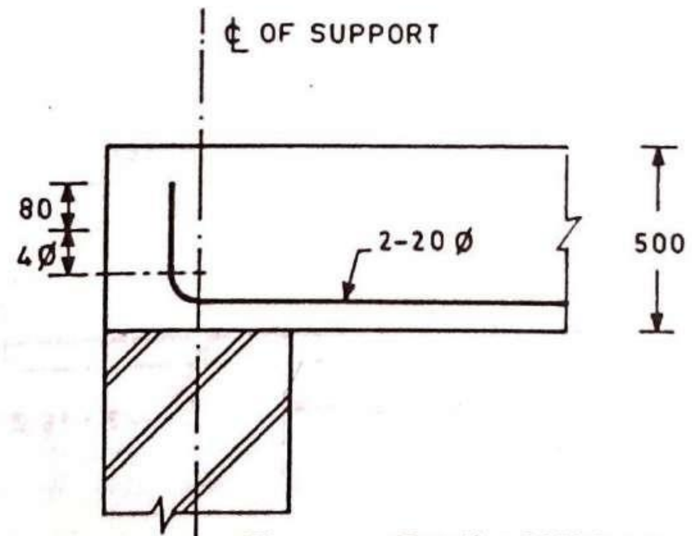


Fig. Ex 1.1 **Details of 90° hook**

$L_d \le 1.3 M_1/V + L_0$.

$47\text{Ø} \le [\frac{1.3 \text{X} 93.45 \text{X} 10^6}{165 \text{X} 1000}] + 320$

Or. $\text{Ø} \le 22.47$mm

Actual bar diameter provided is 20 mm < 22.47 mm.The arrangementofU-Bendisshownin Fig-Ex1.2.

InHighstrength reinforcedbarsU-Bend shouldbeavoidedasfaraspossiblesincetheymaybebrittleand mayfracture with bending.

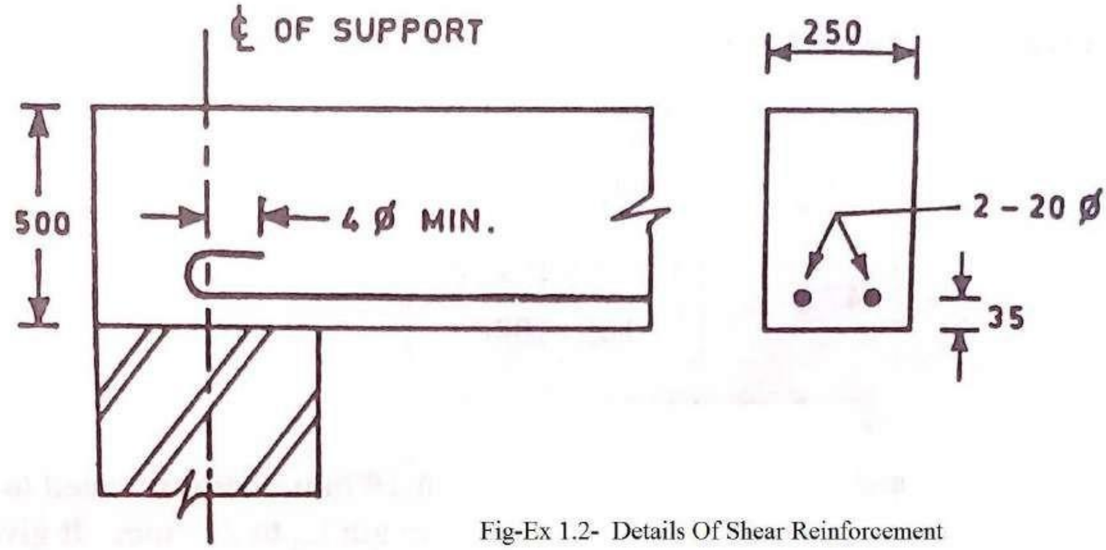


Fig-Ex 1.2- Details Of Shear Reinforcement

Example 5.2:

Acontinuousbeam25cmX40cmcarries3-16mmlongitudinalbarsbeyondthepointofinflectionin the sagging moment region as shown in Fig.Ex 1.3,.If the factored SF at the point of inflection is150KN,σ_{ck}= 20N/mm^{2}and σ_y=415 N/mm^2,checkifthe beam issafe in bond?

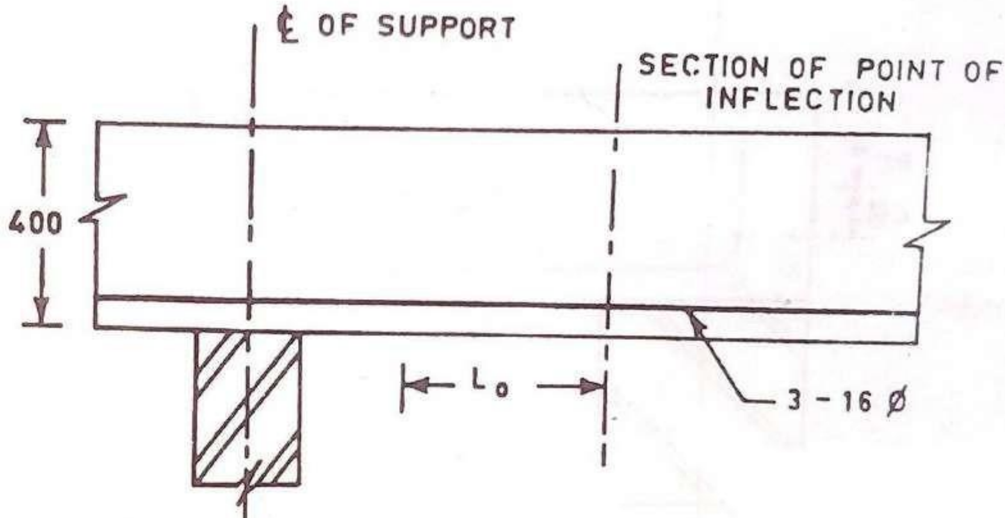


Fig- Ex 1.3 - Section of Continious Beam

Depth of neutral axis $x = \dfrac{0.87\sigma_y A_t}{0.36\sigma_{ck} b} = \dfrac{0.87 \times 415 \times 3 \times \pi / 4 \times 16^2}{0.36 \times 20 \times 250}$

$= 120 \text{ mm} < x_m (= 0.48 \text{ d})$ OK

Moment of resistance $M_1 = 0.87\, \sigma_y A_t (d - 0.42\, x)$

$= 0.87 \times 415 \times 603\,(367 - 0.42 \times 120) = 68.90 \times 10^6$ Nmm

Development length $L_d = \dfrac{\sigma_s \phi}{4\tau_{bd}}$

Bond stress $\tau_{bd} = 1.6 \times 1.2$ N/mm^2 for M20 mix and HSD steel

or $L_d = \dfrac{0.87 \times 415\phi}{4 \times 1.6 \times 1.2} = 47\,\phi$

Anchorage length $L_o =$ greater of d or 12 ϕ

= greater of 367 mm, or $12 \times 16 = 192$ mm

= 367 mm

$$L_d \le \frac{M_1}{V} + L_o$$

or $47\,\phi \le \dfrac{68.9 \times 10^6}{150 \times 1000} + 367$ or, $\phi \le 17.6$ mm

Thus, 16 mm bars are safe in bond at the point of inflection.