CHAPTER 02

Basic Design Concepts

2.1 Introduction

Concrete is such a wonderful construction material that an *Italian architect* once said:

"Concrete has liberated us from the rectangle".

It is because of the flexibility and mouldability property of concrete (when wet) that we are now having structure of any shape and size. It has replaced the conventional building materials like stone, timber and steel. Any reinforced concrete structure (be it simple or complex), is an assembly of basic structural elements like beams, columns, footings, slabs and walls. Thus, design of reinforced concrete structures lies in the design of these basic structural elements i.e. the design of beams, columns, slabs, walls, footings etc. Numerous theories were proposed in the last century for the design of concrete structures. In this chapter, we will look into the various theories for design of concrete structure.

2.2 Necessity of Designing Reinforced Concrete Structures

The principal aim of structural design is that the structures should perform its intended function *safely* at ultimate loads within their life time and also *serviceable* at service or working loads.

The term '**safety**' includes the following parameters:

1. Reserve strength of the material of the structure.
2. Limited or permissible deformation(s) in the structure.
3. Durability of the structure in the long run.

Thus, **safety** implies that the possibility of the failure of the structure (partial or complete failure) is very low even at **ultimate loads** considering appropriate factor of safety.

Serviceability implies that the structure should perform its intended function very well at **working loads**. It includes deflection, vibration, crack widths, durability, permeability, acoustics, thermal insulation etc.

Thus, the basic objectives to fulfil serviceability criteria can be summarized as:

1. Properly designed structures should perform its intended function at service loads quite satisfactorily.
2. Structure should bear all the loads and should deform within the permissible limits.
3. The structures should be durable enough against the adverse environmental conditions.

4. The designed structure must adequately resist the probable hazardous effects of structural misuse and fire.

Remember

Increasing the **factor of safety** in the design of structures increases the safety and serviceability of the structure but at the same time it increases the cost of structure also. Here comes the role of **economy** in the design of structures. While considering the overall economy of a structure, the increased cost associated with the increased factor of safety must be properly weighed against the possibility of structural failure.

2.2.1 Objectives of Structural Design

The rational design of a structure must satisfy the following requirements:

1. **Stability:** The structure must be stable enough to resist the failure of structure in terms of overturning, sliding, buckling of the structure or parts thereof under the severe action of loads viz. both permanent (dead load) and temporary (superimposed live load etc.).
2. **Strength:** The structure must be able to carry safely the stresses induced by the severe most possible combination of loads acting on the structure.
3. **Serviceability:** The structure must perform its intended function i.e. it must be serviceable. This implies that the deflections, vibrations, crack-widths, permeability to water etc. are within the permissible limits.
4. **Aesthetics:** The structure must be in harmony with the surroundings and should look pleasing. It is purely an architectural consideration.
5. **Economy:** At last, economy plays the most important role in structural design. The cost of the structure and its associated facilities must not be so gargantuan that it may dictate the overall functional requirement of the structure.

Do You Know

It is a very common misconception that use of more cement in concrete is better provided the cost of cement is not a constraint. But this is NOT TRUE. Excessive use of cement in concrete leads to cracking of concrete because large amount of heat of hydration is generated from the hydration of cement in concrete and also due to plastic shrinkage of cement paste. It also leads to long term effects of creep and drying shrinkage resulting in large deflections and cracking in the structure.

2.3 Hydraulic and Non-Hydraulic Cements

Cements obtained from the calcination of gypsum and lime stones are **non-hydraulic** because the products of hydration of this type of cement are not resistant to water however the addition of pozzolanic materials in these cements make them **hydraulic**.

2.4 Tests on Cement

Production of quality concrete primarily depends on the quality of cement used for making the concrete. Tests on cement are conducted keeping in view the recommendations of IS 269: 1976 and IS 4031: 1988 in order to assess the following:

1. **Chemical composition:** It is done to determine the oxides of calcium, silica, aluminium, iron, sulphur and magnesium and to check whether these impurities are within the permissible limits.
2. **Fineness of cement:** It measures the size of cement particles expressed in terms of specific surface (defined as surface area per unit mass). Higher the fineness of cement, higher is the hydration rate of cement leading to early development of strength.
3. **Consistency:** It is done to determine the quantity of water to be mixed in order to produce a standard cement paste.
4. **Initial and final setting time:** It measures the rate at which cement paste solidifies. Initial setting time indicates the time after which cement paste becomes unworkable while the final setting time indicates the time to reach the state of complete solidification of cement paste. It is measured using Vicat's apparatus.
5. **Soundness :** It indicates the quality of cement paste by virtue of which it does not undergo large change in volumes.
6. **Strength :** It is measured by performing tests on hardened cement concrete/cement paste and is expressed in terms of stress at failure of the specimen subjected to compression and tension tests.

Do You Know

Earlier the strength of cement was specified as 7 days strength but later on it was found that many other types of cement like the Rapid Hardening Portland Cement gains strength much earlier than the strength corresponding to 7 days strength of other types of cement however, the ultimate strength of both the types of cement (viz. rapid hardening cement and ordinary Portland cement) at the end of 28 days is more or less same. Most of the types of cement gain a significant amount of long term strength at about 28 days only.

2.5 Methods to Increase the Durability of Concrete against Chemical Attack

By reducing the permeability of concrete in following ways:

1. Use of high strength concrete
2. By proper curing of concrete
3. By having a low water cement ratio
4. Using improved quality of admixtures
5. By having the maximum possible compaction of concrete
6. By the use of well graded, dense aggregates
7. By minimizing the possibility of cracks at the design phase itself

By providing protection to reinforcing steel in following ways:

1. By providing adequate clear cover to reinforcing steel as per relevant stipulations of IS codes.
2. Using coated steel or suitable corrosion resistant steel.
3. By preventing the corrosion of steel by the methods like sacrificial protection etc.
4. By the use of suitable corrosion resistant cement.
5. Avoid use of alkali reactive aggregates.
6. By preventing ponding on roof slabs and on other concrete surfaces by proper drainage of water.
7. By having a proper control on chloride and sulphate content in the concrete mix.

2.6 Design Philosophies for the Design of Reinforced Concrete Structures

Numerous design philosophies have been proposed in the last century for the design of reinforced concrete structures. Every **design philosophy** is based on certain assumptions.

Cl. 18.2 of IS 456: 2000 accepts three methods of design of reinforced concrete structures viz. *Limit State Method, Working Stress Method* and *Method based on experimental approach.* **Working Stress Method** is the earliest codified design philosophy which is based on linear elastic theory. This method is now out dated and in the latest version of IS 456: 2000, the provisions of *working stress method* of design has been shifted from the main text of the code to **Annexure 'B'** so as to have larger emphasis on *Limit State Method.*

Ultimate Load Method of design is based on the strength of concrete at ultimate loads. This method was in fact a replacement of traditional WSM of design which was incorporated in the **IS: 456** in the year 1964.

Later on it was realized that there involves a lot of uncertainty in the precise estimation of loads and material properties. Every time designing a structure for the most possible severe combination of loads make the structure highly safe which in fact may not be subjected to that much loads in actual life and at the same time it increases the cost and works out to be uneconomical. However, these uncertainties can be handled by the **theory of probability.** The risk associated with the design of structure was quantified in terms of **probability of failure** of the structure. This type of probability based method is called as **reliability based method.** The major drawback of these methods is that they are highly complicated in terms of mathematical calculations.

In order to overcome the above stated problem of reliability based methods, instead of determining the actual **probability of failure** of the structure, we introduce **many (partial) safety factors**. The European Committee for Concrete **(CEB)** and the International Federation of Prestressing **(FIP)** were among the first to introduce the concept of **Limit State Method of Design,** which is a **reliability based method.** Later on, based on the recommendations of **CEB-FIP,** the limit state method of design got introduced in the British Code (BS-8110-1997) and in the **Indian Code (IS 456:1978).**

2.6.1 The Working Stress Method (WSM) of Design

It is the most traditional method for the design of **reinforced concrete, structural steel and timber.** The basic design philosophy of this method is:

1. The material behaves in a linear elastic manner.
2. Adequate safety can be ensured by restricting the stresses in the materials that are induced due to the application of working loads/service loads.

Now the question arises:

"How to justify the assumption that material behaves in a linear elastic manner?"

The answer to the above question lies in the second assumption, where the allowable stresses are kept well below the strength of material i.e. the allowable stresses lie in the linear phase of stress-strain curve so the assumption of linear elastic behavior of material is justified.

The second assumption as stated above also introduces the concept of **factor of safety** which is expressed as:

$$\text{Factor of safety} = \frac{\text{Strength of the material}}{\text{Allowable stress in the material}}$$

The stresses induced due to the application of service or working loads are determined using the principles of **Strength of Materials.** But reinforced concrete is a **composite material** having two altogether different materials viz. concrete and steel. To apply the principles of Strength of Materials to a composite structure like reinforced concrete, the concept of **strain compatibility** is employed wherein it is assumed that there exists a perfect bond between the two materials (steel and concrete) and that the strain in steel is equal to the strain in the surrounding concrete due to that perfect bond.

It is quite evident that the stress is related to the strain and thus it follows that stress in steel is related to the strain in steel. But strain in steel is equal to the strain in surrounding concrete. Therefore, the stress in steel is indirectly related to the strain in the surrounding concrete. This indirect relation (between the steel and concrete) is expressed in terms of **modular ratio (m)** as:

$$\text{Modular Ratio} = \frac{\text{Stress in steel } (f_s)}{\text{Stress in concrete } (f_c)}$$

$$= \frac{\text{Strain in steel } (\varepsilon_s) \times \text{Modulus of elasticity of steel } (E_s)}{\text{Strain in concrete } (\varepsilon_c) \times \text{Modulus of elasticity of concrete } (E_c)}$$

$$= \frac{E_s}{E_c} \qquad (\text{Since } \varepsilon_c = \varepsilon_s)$$

Limitations of WSM of design

1. It fails to give relative importance to the different types of loads that act on a structure i.e. dead loads, live loads, snow load, seismic loads, temperature load etc. All of these act on the structure with different uncertainties. This often leads to over conservative designs and sometimes under conservative designs (when two loads are acting oppositely i.e. dead and wind loads, live and seismic loads etc.).
2. WSM of design gives large sections (compared to LSM and ULM) of the designed reinforced concrete members.

Remember Although WSM of design has been put at annexure in IS 456: 2000 and superseded by LSM, it still remains the accepted method of design for certain type of structures like reinforced concrete water tanks **(IS 3370)**, chimneys **(IS 4998)** and bridges **(IRC-21)**.

2.6.2 The Ultimate Load Method (ULM) of Design

This method is an improvement over the traditional WSM of design and takes into account the shortcomings of the earlier method. The ULM is also called as the **load factor method** or the **ultimate strength method.** In this method, the non-linear stress strain behavior of steel and concrete are accounted for and stresses induced in the structure at the verge of failure at ultimate loads are considered. The problems associated with the modular ratio (m) are entirely avoided in this method. The safety in the design of structure is taken care by the concept of **load factor** which is expressed as:

$$\text{Load Factor} = \frac{\text{Ultimate or design load}}{\text{Working or service load}}$$

This concept of load factor makes it possible to assign different factors of safety (in terms of load factors) to different types of loads (like dead loads, live loads, seismic loads, wind loads, snow loads etc.) and can be suitably combined; which was a major drawback in the WSM of design.

Limitations of ULM of design

1. The major drawback of this method is that, one cannot say with 100% assurance that if a structure performs well at ultimate loads (**strength**), the same structure will perform its function satisfactorily at service loads also (**serviceability**).
2. Another drawback of this method is that, the assumed nonlinear stress strain behavior of concrete and steel is relevant only if nonlinear analysis is performed on the structure. But nonlinear analysis of structures is too cumbersome to be done for routine type of structures.

For assessing the distribution of stresses at ultimate loads, we use the stress distribution at service loads magnified by the load factor, but this approach is NOT correct owing to the fact that as the loading is increased from the service load level to the ultimate load level, significant inelastic and nonlinear behavior of materials comes in with considerable stress redistribution.

2.6.3 The Limit State Method (LSM) of Design

The earlier design methods includes the working stress method (WSM) and the ultimate load method (ULM) of design. **WSM is based on service loads conditions** alone whereas the **ULM is based on ultimate load conditions** alone. However, LSM takes into account the safety at ultimate load and serviceability at service loads. LSM employs different safety factors at ultimate loads and service loads. These multiple safety factors are based on probabilistic approach with separate approaches for each type of failure, type of materials and types of loads.

Limit State: Limit state is the state of 'about to collapse' or 'impending failure', beyond which, the structure is not of any practical use i.e. either the structure collapses or becomes unserviceable. In LSM, two types of limit states are defined which are:

1. **Limit state of collapse:** This limit state deals with the strength of the structure in terms of collapse, overturning, sliding, buckling etc.

   Various limit states of collapse are:

   - Flexure
   - Compression
   - Shear
   - Torsion

2. **Limit state of serviceability:** This limit state deals with the deformation of the structure to such an extent that the structure becomes unserviceable due to excessive deflection, cracks, vibration, leakage etc.

   Various limit states of serviceability are:

   - Deflection
   - Excessive vibrations
   - Corrosion
   - Cracking (Do not consider the tensile strength of concrete)

NOTE If a structure has attained the limit state of serviceability and then the loads are removed, then the structure will return to its original state. However, if the structure has attained the limit state of collapse, then the structure will not return to its original shape.

2.7 Load and Resistance Factor Design

Load and resistance factor design (LRFD) is the simplest of all the design methods that are currently in use. According to this method, the following condition must be satisfied:

Design resistance ≥ Design load

$$\phi R_n \geq \gamma S_n \qquad \text{...(i)}$$

Where, R_n and S_n are nominal/characteristic values of resistance and load effects respectively and ϕ, γ represent the **resistance factor** and **load factor** respectively.

Resistance factor (ϕ) takes into account the possible uncertainties in the determination of material characteristics like Poisson's ratio, modulus of elasticity etc. and is less than unity whereas the load factor (γ) takes into account the possible overloading of the structure and is greater than unity.

From eq.(i), it can be inferred that,

$$S_n \leq \frac{R_n}{(\gamma/\phi)}$$

The above safety concept is being adopted by **Working Stress Method (WSM)** of design. Here (γ/ϕ) represents the **factor of safety** which is applied to material strength to arrive at the permissible stresses.

From eq.(i), it can also be inferred that,

$$R_n \geq \frac{S_n}{(\phi/\gamma)}$$

The above safety concept is adopted in **Ultimate Load Method (ULM)** of design. Here, (ϕ/γ) represents the **load factor** which is applied to loads in order to arrive at ultimate load.

2.7.1 'Multiple Partial Safety Factor' Method of Design

This method is adopted by the IS 456: 2000 and is expressed as:

$$R_d \geq S_d$$

where, R_d is the design resistance of the material which is computed at little bit lower material strength ($0.67 f_{ck}/\gamma_c$ for concrete and f_y/γ_s for steel). Thus, it involves two partial safety factors viz. for concrete and for steel. S_d is the design load which is calculated at the enhanced load effect involving separate safety factors for dead load, live load, earthquake load, wind load etc.

2.8 IS 456: 2000 Recommendations for LSM of Design

The salient features of the limit state method (LSM) of design as described in the code (**IS 456: 2000**) are as follows:

1. **Characteristic Strength and Characteristic Load:**

   Characteristic strength of a material is that strength below which not more than 5% of the test results are expected to fall i.e. not more than 1 in 20 test results should fall below this characteristic value. For concrete, this value is called as *characteristic strength of concrete* and for steel, this value is called as *yield strength of steel.*

   For Example: For M30 concrete, characteristic strength (f_{ck}) is 30 N/mm^2 and for Fe415, characteristic strength or yield strength (f_y) is 415 N/mm^2.

   NOTE: M5 and M7.5 grades of concrete are lean concrete mixes used for simple bases and foundation of masonry walls.

   IS 456: 2000 does not allow concrete grade lower than M20 in reinforced concrete and this grade of concrete should not be lower than M30 in coastal areas.

   Characteristic load (Cl. 36.2 of IS 456: 2000) is that load which has the 95% probability of not being exceeded during the life time of the structure.

2. **Partial safety factor for materials:** The design strength of concrete and steel are obtained by dividing the respective characteristic strengths by appropriate partial safety factors for concrete

and steel. For concrete, f_{ck} is the characteristic cube strength and in actual structures, the characteristic strength of concrete is taken as $0.67f_{ck}$. Thus, **design strength of concrete is $0.67f_{ck}/\gamma_c$**. Similarly, **design strength of steel is f_y/γ_s**.

At ultimate loads i.e. **at ultimate limit state**, partial safety factor for concrete γ_c = 1.5 and partial safety factor for steel γ_s = 1.15. A higher partial safety factor is assigned for concrete because there is greater possibility of variation in quality of concrete than steel, particularly in fields.

$$\text{Thus, permissible stress in concrete} = \frac{0.67\, f_{ck}}{1.5} = 0.446\, f_{ck}$$

$$\text{and permissible stress in steel} = \frac{f_y}{1.15} = 0.87f_y$$

At serviceability limit states, $\gamma_c = \gamma_s$ = 1. It is taken as unity because we are interested in actual deflections and crack widths at service loads rather than ultimate deflections and crack widths at ultimate loads.

3. **Partial safety factor for loads:** IS 456: 2000 recommends the following load combinations for Dead Loads (DL), Live Loads (LL) and Earthquake Load (EQ) for assessing the ultimate load:
   (a) 1.5(DL + LL)
   (b) 1.5(DL + EQ)
   (c) 0.9 DL + 1.5 EQ
   (d) 1.2(DL + LL + EQ)

   In the last load combination i.e. 1.2 (DL + LL + EQ), a reduced load factor of 1.2 instead of 1.5 is used since it is quite rational to assume that all the three loads will not act at their peak values simultaneously during the event of an earthquake. For the purpose of structural design, the design load resistance with partial safety factors for materials should be greater than the maximum load effect that arises from the above load combinations.

   For serviceability conditions, IS:456 recommends the following load combinations:
   (a) 1.0 (DL + LL)
   (b) 1.0 (DL + EQ)
   (c) DL + 0.8 LL + 0.8 EQ

Table 2.1 : *Values of partial safety factors for loads (Table 18 of **IS 456: 2000**)*

Load Combination	Limit State of Collapse			Limit State of Serviceability		
	DL	LL/IL	WL or EQ	DL	LL/IL	WL or EQ
1. DL + LL	1.5	1.5	–	1.0	1.0	–
2. DL + WL or EQ	1.5 or 0.9*	–	1.5	1.0	–	1.0
3. DL + LL + WL or EQ	1.2	1.2	1.2	1.0	0.8	0.8

(*) This value is considered for stability against overturning or where there is a possibility of stress reversal.

Design Load = Partial safety factor for load × Characteristic Load

Remember: EQ and WL are not considered simultaneously in design.

4. **Design stress-strain curve for concrete:** **IS 456: 2000** recommends the following characteristic and design stress strain curve in flexural compression for concrete as shown in Fig.2.1.

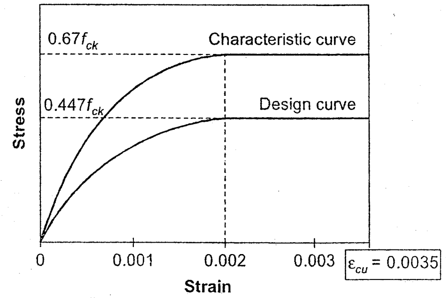

Fig.2.1 *Characteristic and design stress strain curve of concrete in flexural/bending compression*

Maximum strength in characteristic curve is $0.67f_{ck}$ for concrete structures. (Characteristic strength for concrete cube is f_{ck}). For limit state design, a partial safety factor for concrete γ_c = 1.5 is applied to characteristic strength of concrete (= $0.67f_{ck}$). Thus **design stress strain curve of concrete is obtained by dividing the ordinates of characteristic stress strain curve of concrete by partial safety factor of concrete γ_c (=1.5)**. Thus maximum stress in design stress strain curve of concrete is $0.67f_{ck}/1.5 = 0.446f_{ck}$.

For uniformly compressed concrete (as in axially loaded columns), the maximum strain in concrete is taken as 0.002 or 0.2% and the corresponding maximum design stress of concrete is $0.446f_{ck}$.

NOTE The stress strain curve of concrete under pure compression is NOT given by the IS 456: 2000, because limit state of collapse under pure compression of concrete is not relevant in design considerations of structures.

When concrete is subjected to axial compression and flexure, then the maximum strain in concrete is limited in between 0.002 and 0.0035 depending upon the neutral axis location with respect to section of concrete under consideration. However, the maximum design stress level for this case remains equal to $0.446f_{ck}$.

5. **Design stress strain curve for reinforcing steel:** The design yield strength of steel f_{yd} is obtained by dividing the yield strength or characteristic strength of steel (f_y) by partial safety factor of steel (γ_s) which is equal to 1.15.

$$\text{Thus,} \quad f_{yd} = \frac{f_y}{\gamma_s} = \frac{f_y}{1.15} = 0.87f_y$$

The corresponding design yield strain is $\dfrac{0.87f_y}{E_s}$

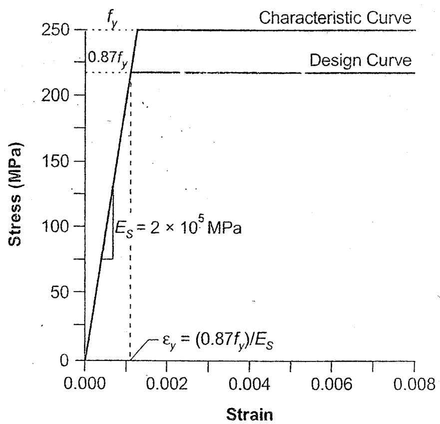

Fig.2.2 *Characteristic and design stress strain curve for mild steel (Fe 250)*

Cold worked bars (Fe 415 and Fe 500) does not have a definite yield point and thus the transition from linear elastic behavior to nonlinear behavior is assumed to occur at a stress level of $0.8f_y$ in characteristic curve and $0.8f_{yd}$ in design stress strain curve of steel. Full design yield strength occurs at a proof strain of 0.2% or 0.002. Thus, design stress of $0.8f_{yd}$ corresponds to zero inelastic strain and the design stress of $1.0f_{yd}$ corresponds to inelastic strain of 0.002. Design yield strain of HYSD steel is,

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

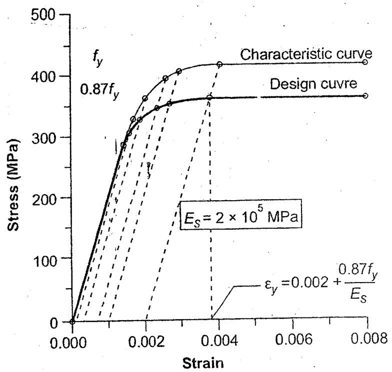

Fig. 2.3 *Characteristic and design stress strain curve for cold worked Fe 415 grade steel*

2.9 Various types of Young's Modulus of Elasticity of Concrete (E_c)

1. **Initial tangent modulus of elasticity:** It is the slope of stress – strain curve of concrete at origin. It is the maximum value of modulus of elasticity of concrete (E_c). IS 456: 2000 defines this value as $5000\sqrt{f_{ck}}$.
2. **Tangent modulus of elasticity:** The slope of tangent at any point on the stress – strain curve is referred to as tangent modulus of elasticity.
3. **Secant Modulus of elasticity:** It is the slope of the line joining any point on stress – strain curve to the origin.
4. **Long term modulus of elasticity:** It considers the effect of creep and is defined as:

$$E_{long} = \frac{E_c}{(1+\theta)} = \frac{5000\sqrt{f_{ck}}}{(1+\theta)}$$

where, θ = Creep coefficient = $\dfrac{\text{Ultimate creep strain}}{\text{Elastic strain}}$

Table 2.2: *Values of creep coefficient as per IS 456: 2000*

Age of Loading	Creep Coefficient (θ)
7 Days	2.2
28 Days	1.6
1 Year	1.1

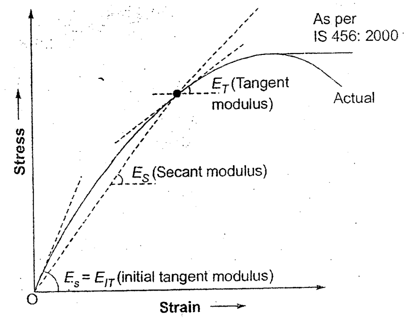

Fig. 2.4 *Various Types of Modulus of Elasticity of Steel*

Remember: For design purposes, we should take secant modulus but actually we are using initial tangent modulus i.e. $5000\sqrt{f_{ck}}$.

2.9.1 Modulus of Rupture of Concrete

As per IS 456: 2000, it is defined as:

$$f_r\,(\text{N/mm}^2) = 0.7\sqrt{f_{ck}}$$

2.10 Properties of Reinforcement and its use in Reinforced Concrete Structures

The reinforcing steel is classified according to the yield strength of the steel that is to be used in construction for structural purposes. This specified yield strength is called as **grade of steel**. The grade of steel (e.g. Fe 250, Fe 415 etc.) represents yield strength of steel (expressed as N/mm² or MPa).

As stated earlier, for certain types of steel, yield point cannot be located distinctly on the stress strain curve. For such types of steel, the stress corresponding to 0.2% (or 0.002) offset strain is defined as yield point.

The slope of the elastic portion of stress strain curve is called as **modulus of elasticity** of steel (E_s) and for all types of steel it is taken as **2×10^5 N/mm²**.

2.10.1 Types of Reinforcing Steel

1. **Mild Steel (Fe 250):** It can either be ORDINARY or HOT ROLLED with characteristic strength of 250 N/mm².
2. **Medium Tensile Steel:** It can also be either ORDINARY or HOT ROLLED.
3. **Cold Twisted Bar:** These are HYSD (HIGH YIELD STRENGTH DEFORMED) bars. e.g. Fe 415 and Fe 500 HYSD bars.
4. **TMT Bars:** These are THERMO MECHANICALLY TREATED (TMT) bars. e.g. Fe 415, Fe 500, Fe 600 TMT bars. Their outer shell has very high tensile strength but inner core is soft and ductile. These are usually coated with anticorrosive coating.

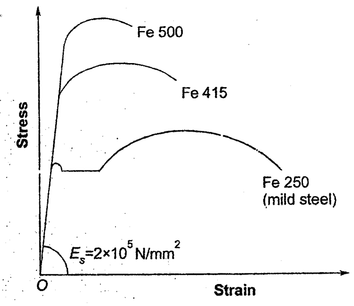

Fig. 2.5 *Stress strain baheviour of mild and HYSD steel*

About Hot Rolled Mild Steel Reinforcement: In case of Hot Rolled mild steel reinforcement, an yield plateau is observed at upper yield point resulting in large deformation in steel and further resulting in large cracking of structure.

Yield Plateau: It can be eliminated by cold working.

Cold Working: Steel reinforcement is stressed beyond yield plateau either by stretching or by twisting and then is unloaded. This process is called as cold working. Cold twisted bars are made by this process.

2.10.2 Yield Strength for HYSD Bars

Table 2.3: *Permissible tensile stresses of steel bars*

Types of bar	Permissible Tensile Stress (N/mm²)			
	Mild Steel (Fe 250)	Medium Tensile Steel	CTP/HYSD Steel	Fe 500
Tensile Strength				
Dia upto 20 mm	140	190	230	275
Dia > 20 mm	130	190	230	275
Compression bar in column	130	130	190	- -

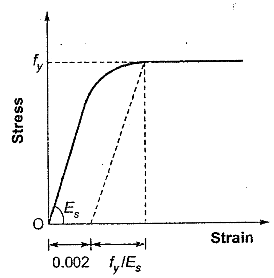

Fig. 2.6 *Stress strain behaviour of HYSD/ CTP steel bars*

For HYSD bars, yield stress is read at 0.2% proof strain i.e. at strain of 0.002 by drawing a line parallel to stress-strain curve.

5 6 8 10 12 16 18 20

Total strain at f_y, $\varepsilon_y = 0.002 + \frac{f_y}{E_s}$

2.10.3 Reinforcement Bar Sizes available in India

The following bar sizes i.e. nominal bar diameters (in mm) are available in the Indian market viz.:

5, 6, 8, 10, 12, 16, 18, 20, 25, 28, 32, 36, 40, 45, 50. Among these, most commonly available bar sizes are 6, 8, 10, 12, 16, 20, 25, 32, 40.

2.10.4 Hot Rolled Versus Cold Worked Steel Bars

Hot rolled mild steel: The stress-strain curve of hot rolled mild steel is featured by an initially straight linear elastic part of the curve which is followed by the occurrence of an **yield plateau** (strain increases at constant stress) and followed by **strain hardening zone** wherein the stress increases with increasing strain till peak tensile strength is reached. Then this is followed by a decreasing limb of the stress strain curve till fracture occurs. For **Fe 250, ultimate tensile strength is 412 MPa and minimum percentage elongation is 20 to 22%.**

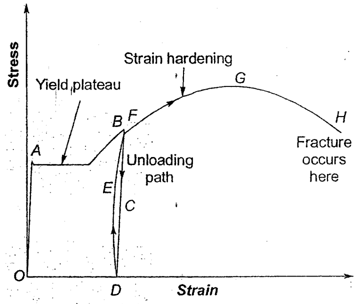

Fig. 2.7 *Effect of cold-working of mild steel bars*

Cold worked bars involve the process of stretching and twisting of mild steel beyond the yield plateau which is followed by the release of load. This specimen on reloading will follow the path *DEF* as shown in the figure below. The process of unloading after the yield stress and then again reloading gives rise to a **hysteresis loop**. The unloading and reloading curves are nearly straight and parallel to *OA* as shown below. Thus the reloading process follows a linear elastic path (with the same modulus of elasticity E_s as that of mild steel) till the point where the unloading process started. Then the material (steel) enters into the strain hardening stage. The process of cold working increases the yield strength of steel but at the same time, it reduces the ductility of steel. **Cold working also decreases the margin between the yield strength and the ultimate strength of steel.**

Do You Know?

Some Interesting Facts about Mild Steel (Fe250):

1. In LSM, mild steel (Fe 250) which has a clearly defined yield point, the steel yields i.e. reaches the stress 'f_y' even at strains less than $\left(\frac{0.87 f_y}{E_s} + 0.002\right)$ and thus at values of x_u which are slightly greater than $x_{u\,lim}$, the stress in mild steel remains 0.87f_y.
2. Ductility requirement may be partially satisfied with the use of Fe 250 steel even if x_u is slightly greater than $x_{u\,lim}$.
3. Due to the ability of mild steel to yield even at strains less than $\left(\frac{0.87 f_y}{E_s} + 0.002\right)$, the actual under-reinforced behavior gives values of percentage of tensile reinforcement (p_t) greater than $p_{t\,lim}$. But **IS 456: 2000** limit the use of p_t values greater than $p_{t\,lim}$ for design purposes.

2.11 Cover Requirements as per IS 456: 2000

In order to prevent the reinforcement from corrosion, a layer of concrete should be there in reinforced concrete structures. In actual construction practice, the **specified minimum cover is very difficult to maintain.** Thus, **IS 456: 2000** specifies tolerance levels ranging from 0 mm to +10 mm i.e. no reduction in clear cover is permitted but the nominal cover can be increased up to 10 mm.

IS 456: 2000 has eliminated the restriction on clear cover up to 75 mm but this does not convey the message to use large covers as the use of very large covers (like 100 mm or more) lead to increased crack widths (size greater than 0.3 mm) and allows the ingress of moisture and chemical attack to concrete.

Table 2.4: *Min. conctete grade and nominal cover requirements*

Exposure Conditions	Minimum Concrete Grade	Nominal Cover (mm)
Mild	M20	20
Moderate	M25	30
Severe	M30	45
Very Severe	M35	50
Extreme	M40	75

Note :
- Nominal cover can be decreased by 5 mm for main reinforcing bars of less than 12 mm diameter.
- Nominal cover can be decreased by 5 mm if concrete grade is M35 or higher.

Table 2.5: *Nominal Cover to Meet the Fire Resistance Requirement as per Table 16A of IS 456: 2000*

Fire Resistance (hours)	Nominal Cover (mm)						
	Beams		Slabs		Ribs		Columns
	Simply Supported	Continuous	Simply Supported	Continuous	Simply Supported	Continuous	
≤0.5	20	20	20	20	20	20	40
1	20	20	20	20	20	20	40
1.5	20	20	25	20	35*	20	40
2	40*	30	35*	25	45	35*	40
3	60	40*	45	35*	55	45	40
4	70	50	55	40	65	55	40

(*) Requires attention to additional measures necessary to reduce the risks of spalling.

2.12 Spacing of Reinforcement

Proper placement of reinforcement is an important component in the reinforced concrete design. The required area of steel reinforcement is provided in the form of bars adequately placed in the concrete. Closely placed bars i.e. densely reinforced concrete members create problem in pouring of concrete and its compaction. On the other hand, too widely spaced bars cannot control the cracks in concrete due to shrinkage.

2.13 Other Important Considerations in Reinforced Concrete

Given below are some other important considerations for the design and preparation of concrete mixes for reinforced concrete..

1. **Mixing:** Cl. 10.3 of **IS 456: 2000** states that concrete should be mixed in a mechanical mixer for at least **two minutes** in order to have uniform distribution of materials and to have a uniform consistency.
2. **Transporting, placing, compaction and curing:** Cl. 13 of **IS 456: 2000** states that concrete must be transferred to the formwork immediately after mixing to avoid segregation of concrete constituents, loss of workability and possibility of mixing of foreign impurity in the concrete. Moreover, adequate precautions must be taken to prevent loss of water from concrete due to evaporation.

The **compaction of concrete** should start immediately after placing and before the initial setting time so that it is not disturbed after the start of initial setting time. While placing concrete, it must be ensured that reinforcing bars should not get displaced from their positions and formwork should not get displaced.

Compaction of concrete around the reinforcement should be done with mechanical vibrators but excessive vibration should be avoided as it leads to segregation of concrete.

Curing prevents the loss of moisture from concrete thereby making available sufficient water for proper hydration of cement in concrete mix.

3. **Formwork: Cl. 11 of IS 456: 2000** states that properly designed formwork should be used in order to maintain its rigidity during placing and compaction of concrete. The stripping time of formwork should be such that the concrete attains strength of at least twice the stress that the concrete may be subjected to when formwork is removed from the concrete.
4. **Assembly of reinforcement: Cl. 12 of IS 456: 2000** states that reinforcement bars designed for flexural moments, shear forces and axial forces must be properly accommodated and a proper bar bending schedule should be prepared. Reinforcement bars should be placed over blocks, spacers, supporting bars etc. to maintain their position and the required concrete cover. **High strength bars must NOT be re-bent.** Reinforcement bars must be properly assembled to have a good flow of concrete through it without segregation.
5. **Sampling and strength of concrete mix: Cl. 15 of IS 456: 2000** recommends that random samples of concrete cubes shall be made from the fresh concrete which is then properly cured and tested at 28 days as per **IS 516: 1959**. The number of samples will depend on the total quantity of concrete as per **Cl. 15.2.2 of IS 456: 2000.**

   At least one sample shall be taken from each shift. Where concrete is produced continuously (e.g. in RMC Plants), there, frequency of sampling may be agreed upon mutually by the supplier and the buyer.

***Table 2.6:** Min. frequency of sampling of concrete*

Quantity of concrete in work (m^3)	Number of samples
0 - 5	1
6 - 15	2
16 - 30	3
31 - 50	4
51 and above	4 plus one additional sample for each 50 m^3 or part thereof.

6. **Acceptance Criteria: Cl. 16 of IS 456: 2000** states the limit on acceptance criteria i.e. concrete should be considered satisfactory when both the mean strength of any group of four consecutive test results and any individual test result on compressive strength and flexural strength comply with the limits specified in **Cl. 16.1** and **Cl. 16.2 of IS 456: 2000.**
7. **Inspection and testing of structures: Cl. 17 of IS 456: 2000** lays down the systematic procedure for inspection which covers materials, records, workmanship and construction.
8. **It is always desirable to limit the number of different types of beams to a few standard sized beams** which makes the reinforcement detailing process simpler and also greatly helps in the construction of formwork at site.
9. **Unit weight of reinforced concrete: Cl. 19.2.1 of IS 456: 2000** specifies the unit weight of reinforced concrete to be taken as 25 kN/m^3.

2.14 Major Reasons of Structure Failure

There are so many reasons for a building failure. By the term **'failure'**, it implies failure in terms of either collapse or serviceability or both. Some of the major reasons of structure failure are as follows:

Failure during construction or soon after	Failure after a long time of construction
Design fault/significant shift from actual design	Collapse/failure of primary load carrying member by accident or otherwise
Poor detailing	Change in use of the structure leading to over loading
Inferior quality of materials	Factors which are beyond human control like fire, earthquake, blast etc.
inferior construction quality	Lack of proper repair and maintenance
Substandard formwork and/or scaffolding	Exposure to adverse environment which was not envisaged in design

2.15 List of Major Indian Standard (IS) Codes Relating to Reinforced Concrete

2.15.1 Codes for Cement

IS 269: 1989	:	Specification for 33 Grade Ordinary Portland cement
IS 8112: 1989	:	Specification for 43 Grade Ordinary Portland cement
IS12269: 1987	:	Specification for 53 Grade Ordinary Portland cement
IS 8041: 1990	:	Specification for rapid hardening Portland cement
IS 4031: 1988	:	Methods of physical tests for hydraulic cement
IS 6909: 1990	:	Specification for supersulphated cement
IS 8042: 1978	:	Specification for Portland white cement
IS 12600: 1989	:	Specification for low heat Portland cement
IS 1489: 1991	:	Specification for Portland pozzolana cement Part – I Flyash based Part - II Calcined clay based
IS 8043: 1991	:	Specification for hydrophobic Portland cement
IS 12330: 1988	:	Specification for sulphate resisting cement
IS 6452: 1989	:	Specification for high alumina cement for structural use

2.15.2 Codes for steel used as reinforcement in RCC structures

IS 432 (Part-I): 1982	:	Specification for mild steel and medium tensile steel bars for concrete reinforcement
IS 1786: 1985	:	Specification for high strength deformed steel bars for concrete reinforcement
IS 1608: 1995	:	Mechanical testing of metals-Tensile testing
IS 2062: 1999	:	Steel for general structural purposes-Specification
IS 1566: 1982	:	Specification for hard drawn steel wire fabric for concrete reinforcement

2.15.3 Codes for concrete

IS 10262: 1982	:	Recommended guidelines for concrete mix design
IS 7861 (Part-I): 1975	:	Code of Practice for extreme weather concreting (Part-I recommended for hot weather concreting)
IS 3370 (Part –I): 1965	:	Code of Practice for storage of liquids (Part-I : General)

IS 516: 1959 : Methods of tests for strength of concrete
IS 1199: 1959 : Methods of sampling and analysis of concrete
IS 1343: 1980 : Code of Practice for Prestressed Concrete

2.15.4 Codes for aggregates, admixtures and water to be used in RCC

IS 383: 1970 : Specification for coarse and fine aggregates from natural sources for concrete
IS 2386 (Parts 1 to 8) : Methods of tests for aggregate for concrete
IS 1344: 1981 : Specification for calcined clay pozzolana
IS 3812: 1981 : Specification for flyash for use as pozzolana and admixture
IS 9103: 1999 : Specification for admixtures for concrete
IS 3025 (Parts 17 to 32) : Methods of sampling and test (physical and chemical) for water and waste water.

2.16 Major Challenges for a Structural Designer

In order to design a structure economically , a structural designer faces the following challenges:

1. Analysing a structure on the basis of **highly simplified structural analysis theories** which are far from actual material (steel, concrete) behavior.
2. Construction of structure by the unorganized sector of construction workers and there always exists a possibility of human error.
3. Structure subjected to a completely unpredictable natural environment.

Objective Brain Teasers

Q.1 By the term '*safety*' in reinforced concrete, we actually mean:
(a) Material reserve strength
(b) Limited deformation in structure
(c) Structural durability
(d) All of the above

Q.2. If ϕ is the resistance factor which takes into account the material uncertainties and γ is the load factor, then as per ULM of design, the ratio ϕ/γ should be:
(a) Less than unity (b) Greater than unity
(c) Equal to unity (d) Can't say

Q.3 A structure will return to its original state when it has reached:
(a) The limit state of collapse
(b) The limit state of serviceability
(c) Both (a) and (b)
(d) Structure will never return to its original shape.

Q.4 Which of the following load combination is NOT recommended by IS 456: 2000 for limit state of serviceability?
(a) 1.0(DL + LL)
(b) 1.5(DL + LL)
(c) DL + 0.8LL + 0.8EQ
(d) 1.0(DL + EQ)

Q.5 The design stress-strain curve of mild steel (Fe 250) in limit state method can be obtained from the corresponding characteristic stress-strain curve by:
(a) Multiplying the ordinates of characteristic curve by 1.15
(b) Multiplying the ordinates of characteristic curve by 1.5
(c) Dividing the ordinates of characteristic curve by 1.15
(d) Dividing the ordinates of characteristic curve by 1.5

Q.6 IS 456: 2000 defines *Modulus of Elasticity* of concrete as 5000 $\sqrt{f_{ck}}$. This in the stress strain curve of concrete refers to:
(a) Secant modulus of elasticity
(b) Initial tangent modulus of elasticity
(c) Tangent modulus of elasticity
(d) Long term modulus of elasticity

Q.7 Which of the following curve is NOT covered by IS 456: 2000?
(a) Stress strain curve of concrete in tension.
(b) Stress strain curve for mild steel.
(c) Stress strain curve of concrete in compression.
(d) Stress strain curve for cold worked steel.

Q.8 The number of concrete cube samples required to be taken for 35 m^3 of concreting work as per IS 456: 2000 is:
(a) 1 (b) 2
(c) 3 (d) 4

Q.9 As per IS 456: 2000 recommendations, which of the following is true?
(a) High strength bars can be re-bent.
(b) High strength bars must not be re-bent.
(c) There is no need to prepare bar bending schedule.
(d) Reinforcement bars should not be placed over supporting bars.

Q.10 Which of the following reinforced concrete design philosophy do not distinguish between the different load cases?
(a) Limit State Method
(b) Working Stress Method
(c) Ultimate Load Method
(d) All of the above

Q.11 If 'f_{ck}' is the characteristic strength of concrete cube then design strength of concrete is
(a) 0.67 f_{ck} (b) 0.447 f_{ck}
(c) f_{ck} (d) 0.5 f_{ck}

Q.12 Minimum grade of concrete that can be used in structures as per IS 456: 2000 is
(a) M15 (b) M20
(c) M25 (d) M30

Q.13 At what flexural stress, the first crack in RCC member made of M30 concrete will occur?
(a) 4.5 MPa (b) 5 MPa
(c) 3.83 MPa (d) 30 MPa

Q.14 The cover to reinforcement in a RC beam shall not be less than
(i) 25 mm
(ii) diameter of the bar (ϕ)
(iii) spacing between the bars
(iv) 5 mm
Which of the above statement(s) is/are true?
(a) (i) and (ii) (b) (i) only
(c) (ii) only (d) (i) and (iii)

Q.15 Shear span is defined as the region where
(a) shear force is constant
(b) shear force is zero
(c) flexural moment is constant
(d) flexural moment is zero

Answers

1. (d)	2. (a)	3. (b)	4. (b)	5. (c)
6. (b)	7. (a)	8. (d)	9. (b)	10. (b)
11. (b)	12. (b)	13. (c)	14. (a)	15. (a)

Hints:

4. (b)
1.5 (DL+LL) is recommended for limit state of collapse and not for limit state of serviceability.

11. (b)
Strength of concrete taken for actual structure $= 0.67\, f_{ck}$
Design strength of concrete

$$= \frac{0.67 f_{ck}}{FOS} = \frac{0.67 f_{ck}}{1.5} = 0.4467 f_{ck}$$

13. (c)

$$f_{cr} = 0.7\sqrt{f_{ck}} = 0.7\sqrt{30} = 3.83 \text{ MPa}$$