# DESIGN OF REINFORCED CONCRETE STRUCTURES AND THE USE OF VARIOUS GRADES OF REINFORCING STEEL

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## 1.1 Introduction

Reinforced concrete is used as construction material in every country. In many, including Bangladesh, reinforced concrete is a dominant structural material in engineered construction. The universal nature of reinforced concrete construction stems from the wide availability of reinforcing bars and the constituents of concrete, stones, sand, and cement, the relatively simple skills required in concrete construction, and the economy of the reinforced concrete compared to other forms of construction. Concrete is a relatively brittle material whose tensile strength is small compared with its compressive strength. This prevents its economical use in structural members that are subject to tension either entirely or over part of their cross section as in beams or in other flexural members.

To offset this limitation, it was found possible, in the second half of the nineteenth century, to use steel with its high tensile strength to reinforce concrete, chiefly in those places where its low tensile strength would limit the carrying capacity of the member. The reinforcement, usually round steel rods with appropriate surface deformations to provide interlocking, is placed in the forms in advance of the concrete. When completely surrounded by the hardened concrete mass, it forms an integral part of the member. The resulting combination of two materials, known as reinforced concrete, combines many of the advantages of each: the relatively low cost, good weather and fire resistance, good compressive strength, and excellent formability of concrete and the high tensile strength and much greater ductility and toughness of steel. It is this combination that allows the almost unlimited range of uses and possibilities of reinforced concrete in the construction of buildings, bridges, dams, tanks, reservoirs, and a host of other structures.

In more recent times, it has been found possible to produce steels, at relatively low cost, whose yield strength is 3 to 4 times and more that of ordinary reinforcing steels. Likewise, it is possible to produce concrete 4 to 5 times as strong in compression as the more ordinary concretes. These high-strength materials offer many advantages, including smaller member cross sections, reduced dead load, and longer spans. However, there are limits to the strengths of the constituent materials beyond which certain problems arise. To be sure, the strength of such a member would increase roughly in proportion to those of the materials. However, the high strains that result from the high stresses that would otherwise be permissible would lead to large deformations and consequently large deflections of such members under ordinary loading conditions. Equally important, the large strains in such high-strength reinforcing steel would induce large cracks in the surrounding low tensile strength concrete, cracks that would not only be unsightly but that could significantly reduce the durability of the structure. This limits the useful yield strength of high-strength reinforcing steel to 80 ksi according to many codes and specifications; 40 ksi, 50 ksi and 60 ksi steel are most commonly used.

## 1.2 Historical Development of Reinforced Concrete as Structural Material

W. B. Wilkinson of Newcastle-upon-Tyne obtained a patent in 1854 for a reinforced concrete floor system that used hollow plaster domes as forms. The ribs between the forms were filled with concrete and were reinforced with discarded steel mine-hoist ropes in the center of the ribs. In France, Lambot built a rowboat of concrete reinforced with wire in 1848 and patented it in 1855. His patent included drawings of a reinforced concrete beam and a column reinforced with four round iron bars. In 1861, another Frenchman, Coignet, published a book illustrating uses of reinforced concrete.

The American lawyer and engineer Thaddeus Hayatt experimented with reinforced concrete beams in the 1850s. His beams had longitudinal bars in the tension zone and vertical stirrups for shear. Unfortunately, Hyatt's work was not known until he privately published a book describing his tests and building system in 1877.

Perhaps the greatest incentive to the early development of the scientific knowledge of reinforced concrete came from the work of Joseph Monier, owner of a French nursery garden. Monier began experimenting about 1850 with concrete tubs reinforced with iron for planting trees. He patented this idea in 1867. This patent was rapidly followed by patents for reinforced pipes and tanks (1868), flat plates (1869), bridges (1873), and stairs (1875). In these products and structures, the reinforcement was arbitrarily placed to conform with the shape of the element, without a thorough understanding of the mechanics involved. In 1880-1881, Monier received German patents for many of the same applications. These were licensed to the construction firm Wayss and Freitag, which commissioned Professors Morsch and Bach of the University of Stuttgart to test the strength of reinforced concrete and commissioned Mr. Koenen, chief building inspector for Prussia, to develop a method of computing the strength of reinforced concrete. Koenen's book, published in 1886, presented an analysis which assumed that the neutral axis was at the midheight of the member.

The first reinforced concrete building in the United States was a house built on Long Island in 1875 by W. E. Ward, a mechanical engineer. E. L. Ransome of California experimented with reinforced concrete in the 1870s and patented a twisted steel reinforcing bar in 1884. In the same year, Ransome independently developed his own set of design procedures. In 1888 he constructed a building having cast-iron columns and a reinforced concrete floor system consisting of beams and a slab made from flat metal arches covered with concrete. In 1890, Ransome built the Leiand Stanford, Jr. Museum in San Francisco. This two-story building used discarded cable car rope as beam reinforcement. In 1903 in Pennsylvania he built the first building in the United States completely framed with reinforced concrete.

In the period from 1875 to 1900, the science of reinforced concrete developed through a series of patents. An English textbook published in 1904 listed 43 patented systems, 15 in France, 14 in Germany or Austria-Hungary, 8 in the United States, 3 in the United Kingdom, and 3 elsewhere. Most of these differed in the shape of the bars and the manner in which the bars were bent.

From 1890 to 1920, practicing engineers gradually gained a knowledge of the mechanics of reinforced concrete, as books, technical articles, and codes presented the theories. In an 1894 paper to the French Society of Civil Engineers, Coignet (son of the earlier Coignet) and de Tedeskko extended Koenen's theories to develop the working stress design method for flexure, which was used universally from 1900 to 1950.

During the past several decades extensive research has been carried out on various aspects of reinforced concrete behavior, resulting in design procedures in which the designer considers a series of limit states (failure states) involving various modes of collapse or unsuitable service load behavior. During this period the size of a reinforced concrete member required to carry a given load has decreased dramatically, although, unfortunately, the complexity of the design calculations has increased in many areas. At the same time as these improvements in knowledge and design practice occurred, equally important improvements in construction methods were occurring which reduced the cost of concrete construction.

### 1.2.1 Design Specifications for Reinforced Concrete

The first sets of building regulations for reinforced concrete were drafted under the leadership of Professor Morsch of the University of Stuttgart and were issued in Prussia in 1904. Design regulations were issued in Britain, France, Austria, and Switzerland between 1907 and 1909.

The American Railway Engineering Association appointed a Committee on Masonry in 1890. In 1903 this committee presented specifications for portland cement concrete. Between 1908 and 1910 (Ref. 1) a series of committee reports led to the Standard Building Regulations for the Use of Reinforced Concrete published in 1910 by the National Association of Cement Users which subsequently became the American Concrete Institute.

A Joint Committee on Concrete and Reinforced Concrete was established in 1904 by the American Society of Civil Engineers, American Society for Testing and Materials, the American Railway Engineering Association, and the Association of American Portland Cement Manufacturers. This group was later joined by the American Concrete Institute. Between 1904 and 1910 the Joint Committee carried out research. A preliminary report issued in 1913 (Ref. 2) lists the more important papers and books on reinforced concrete published between 1898 and 1911. The final report of this committee was published in 1916 (Ref. 3). The history of reinforced concrete building codes in the United States was reviewed in 1954 by Kerekes and Reid (Ref. 4).

## 2.0 Objectives of Design

The structural engineer is a member of a team whose members work together to design a building, bridge, or other structure that will fulfill the specific needs of a client. In the case of a building, an architect generally provides the overall layout, and mechanical, electrical, and structural engineers design individual systems within the building. A geotechnical or foundation engineer provides information necessary for the design of foundations, basement walls, and so on.

The structure should satisfy four major criteria:

1. **Appropriateness.** The arrangement of spaces, spans, ceiling heights, access, and traffic flow must complement the intended use. The structure should fit its environment and be aesthetically pleasing wherever possible. Although such decisions are frequently in the architect's domain, the structural engineer should bear them in mind in choosing framing systems for a building or in laying out structures such as bridges.
2. **Economy.** The overall cost of the structure should not exceed the client's budget. Frequently, teamwork in design will lead to overall economies.
3. **Structural adequacy.** Structural adequacy involves two major aspects:
   a. A structure must be sufficiently strong to support safely, without collapse, all anticipated loadings.

b. A structure must not deflect, tilt, vibrate, or crack in a manner that impairs its usefulness.

4. **Maintainability.** A structure should be designed to require a minimum of maintenance and/or to be able to be maintained in a simple fashion.

## 2.1 Design Process

The design process is a sequential and iterative decision-making process. The three major phases are:

1. **Definition of the client's needs and priorities.** All buildings or other structures are built to fulfil some need. These may include such things as functional requirements, aesthetic requirements, and budgetary requirements. The latter include minimum first cost, rapid construction to allow early occupancy, minimum upkeep, and other factors. Since, for example, it may not be possible to have large column-free areas and high-quality finishes in a building having a minimum first cost the client must set priorities on his or her requirements.
2. **Development of concept of project.** Based on the client's needs and priorities, a number of possible layouts of the architectural, structural, mechanical and other systems are developed. Preliminary cost estimates are made and the final choice of the system to be used is based on how well the overall design satisfies the prioritized needs within the budget available. This stage is an interactive process where a structural or mechanical design decision may require, for example, modification of a prior architectural decision.
   During this stage the overall structural concept is selected. Several potential systems may be considered. Based on approximate analyses of the moments, shears, and axial forces, preliminary member sizes are selected for each potential scheme. Once this is done. it is possible to estimate costs and select the most desirable structural system. The overall thrust in this stage of the structural design is to satisfy the design criteria dealing with appropriateness, economy, and to some extent, maintainability.
3. **Design of individual systems.** Once the overall layout and general structural concept have been selected, the structural system can be designed to ensure structural adequacy. Structural design involves three main steps. Based on the preliminary design selected in step 2, a structural analysis is carried out to determine the moments, shears, and axial forces in the structure. The individual members are then proportioned to resist these forces. The proportioning, sometimes referred to as member design, must also consider overall aesthetics, the constructability of the design, and the maintainability of the final structure.
   The final stage in the design process is to prepare construction drawings and specifications.

## 2.2 Limit States and the Design of Reinforced Concrete

### 2.2.1 Limit States

When a structure or structural element becomes unfit for its intended use, it is said to have reached a limit state. The limit states for reinforced concrete structures can be divided into three basic groups:

1. Ultimate limit states. These involve a structural collapse of part or all of the structure. Such a limit state should have a very low probability of occurrence since it may lead to loss of life and major financial losses. The major ultimate states are:

   a. Loss of equilibrium of a part or all of the structure when considered as a rigid body. Such a failure would generally involve tipping or sliding of the entire structure and would occur if the reactions necessary for equilibrium could not be developed.
   b. Rupture of critical parts of the structure, leading to partial or complete collapse.
   c. Progressive collapse. In some cases a minor localized failure may cause adjacent members to be overloaded and fail, until the entire structure has collapsed. Progressive collapse is prevented or slowed by correct structural detailing to tie the structure together and to provide alternative load paths in case of a localized failure. Since such failures frequently occur during construction, the designer should be aware of construction loads and procedures.
   d. Formation of a plastic mechanism. A mechanism is formed when the reinforcement yields to form plastic hinges at enough sections to make the structure unstable.
   e. Instability due to deformations of the structure. This type of failure involves buckling due to gravity load effects.
   f. Fatigue. Fracture of members due to repeated stress cycles may cause collapse of part or all of a structure. This has been listed as an ultimate limit siate since it leads to a structural collapse. Fatigue failures result from repeated application of service loads.
2. Serviceability limit states. These involve disruption of the functional use of the structure but not collapse. Since there is less danger of loss of life, a higher probability of occurrence can generally be tolerated than in the case of an ultimate limit state. The major serviceability limit states include:
   a. Excessive deflections for normal service. Excessive deflections may cause machinery to malfunction, may be visually unacceptable, and may lead to damage to nonstructural elements or to changes in the distribution of forces. In the case of very flexible roofs, the deflections due to the weight of water on the roof may lead to increased deflections, increased depth of water, and so on, until the capacity of the roof is exceeded. This is a ponding failure and in essence is a collapse brought about by a lack of serviceability.
   b. Excessive crack width. Although reinforced concrete must crack before the reinforcement can act, it is possible to detail the reinforcement to minimize the crack widths. Excessive crack widths lead to leakage through the cracks, corrosion of the reinforcement, and gradual deterioration of the concrete. In addition, wide cracks may be unsightly. Special detailing is required in structures that must be watertight.
   c. Undesirable vibrations. Vertical vibrations of floors or bridges and lateral and torsional vibrations of tall buildings may disturb the users. Vibration has rarely been a problem in reinforced concrete buildings.
3. Special limit states. This class of limit states involves damage to the structure or failure of the structure due to abnormal conditions or abnormal loadings and includes:
   a. Damage or collapse in extreme earthquakes
   b. Structural effects of fire, explosions, or vehicular collisions
   c. Structural effects of corrosion or deterioration, particularly when exposed to chloride environments as in coastal areas.

### 2.2.2 Limit States Design

Limit states design is a design process that involves:

1. Identification of all potential modes of failure (i.e.. identification of the significant limit states)
2. Determination of acceptable levels of safety against occurrence of each limit state

For normal structures this is carried out by the building code authorities, who specify the load combinations and safety factors to be used in checking the limit states. For unusual or very large structures the engineer may need to check whether the normal levels of safety are adequate.

3. Consideration by the designer of the significant limit states
   Frequently, for building structures, a limit states design is carried out starting by proportioning for the ultimate limit states followed by a check of whether the structure will exceed any of the serviceability limit states. This sequence is followed since the major function of structural members in buildings is to resist loads without endangering the occupants. In the case of a water tank, however, the limit state of excessive crack width is of equal importance to any of the ultimate limit states if the structure is to remain watertight. In such a structure the design might start with a consideration of the limit state of crack width, followed by a check of the ultimate limit states. In the design of support beams for an elevated monorail or other rapid-transit system, the smoothness of the ride is extremely important, and as a result, the limit state of permanent deflection may govern the design.

## 2.3 Structural Safety

There are three main reasons why some sort of safety factors, such as load and resistance factors, are necessary in structural design:

1. Variability in resistance. The actual strengths (resistances) of beams, columns, or other structural members will almost always differ from the values calculated by the designer. The main reasons for this are (Ref. 5):

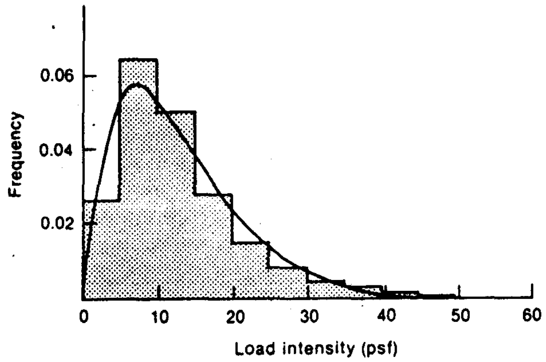
   (a) Variability of the strengths of concrete and reinforcement
   (b) Differences between the as-built dimensions and those shown on the structural drawings
   (c) Effects of simplifying assumptions made in deriving the equations for member resistance

2. Variability in loadings. All loadings are variable, especially live loads and environmental loads due to wind or earthquake. Figure 1a (Ref. 6) compares the sustained component of live loads measured in a family of 151-$ft^2$ areas in offices. Although the average sustained live load was 13 psf in this sample, 1% of the measured loads exceeded 44 psf. For this type of occupancy and area, building codes specify live loads of 50 psf. For larger areas the mean sustained live load remains close to 13 psf but the variability decreases, as shown in Fig. 1b (Ref. 6). An extraordinary live load representing unusual loadings due to parties, temporary storage, and so on, must be added to get the total live load. As a result, the maximum live load on a given office will generally exceed the 13 to 44 psf quoted above.

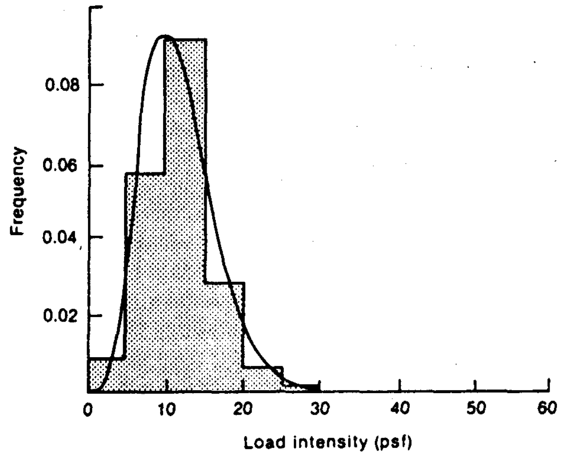
In addition to actual variations in the loads themselves, the assumptions and approximations made in carrying out structural analyses lead to differences between the actual forces and moments and those computed by the designer (Ref. 5).

Due to the variabilities of resistances and load effects, there is a definite chance that a weaker-than-average structure may be subjected to a higher-than-average load. In extreme cases failure may occur. The load factors and resistance factors are selected to reduce the probability of failure to a very small level.

A third factor that must be considered in establishing the level of safety required in a particular structure is:




(a) Area = 151 ft²




(b) Area = 2069 ft²

Fig. 1 Frequency distribution of sustained component of live loads in offices.

3. Consequences of failure. A number of subjective factors must be considered in determining an acceptable level of safety for a particular class of structure. These include such things as:
   (a) Cost of clearing the debris and replacing the structure and its contents.
   (b) Potential loss of life. It may be desirable to have a higher factor of safety for an auditorium than for a storage building.
   (c) Cost to society in lost time, lost revenue or indirect loss of life or property due to a failure. For example, the failure of a bridge may result in intangible costs due to traffic jams, and so on, which could approach the cost of the damage.
   (d) Type of failure, warning of failure, existence of alternative load paths. If the failure of a member is preceded by excessive deflections, as in the case of a flexural failure of a reinforced concrete beam, the persons endangered by the impending collapse will be warned and will have a chance to leave the building prior to failure. This may not be possible if a member fails suddenly without warning, as may be the case with a tied column. Thus the required level of safety may not need to be as high for a beam as for a column. In some structures, the yielding or failure of one member causes a redistribution of load to adjacent members. In other structures, the failure of one member causes complete collapse. If no redistribution is possible, a higher level of safety is required.

## 2.4 Reinforcing Steels for Concrete

The useful strength of ordinary reinforcing steels in tension as well as in compression, i.e., the yield strength, is about 15 times the comprcssive strength of common structural concrete, and well over 100 times its tensile strength. On the other hand, steel is a high-cost material compared with concrete. It follows that the two materials are best used in combination if the concrete is made to resist the compressive stresses and the steel the tensile stresses. Thus, in reinforced concrete beams, the concrete resists the compressive force, longitudinal steel reinforcing bars are located close to the tension face to resist the tension force, and usually additional steel bars are so disposed that they resist the inclined tension stresses that are caused by the shear force in the beams. However, reinforcement is also used for resisting compressive forces primarily where it is desired to reduce the cross-sectional dimensions of compression members, as in the lower-floor columns of multistory buildings. Even if no such necessity exists, a minimum amount of reinforcement is placed in all compression members to safeguard them against the effects of small accidental bending moments that might crack and even fail an unreinforced member.

For most effective reinforcing action, it is essential that steel and concrete deform together, i.e., that there be a sufficiently strong bond between the two materials to ensure that no relative movements of the steel bars and the surrounding concrete occur. This bond is provided by the relatively large chemical adhesion that develops at the steel-concrete interface, by the natural roughness of the mill scale of hot-rolled reinforcing bars, and by the closely spaced rib-shaped surface deformations with which reinforcing bars are furnished in order to provide a high degree of interlocking of the two materials.

Additional features that make for the satisfactory joint performance of steel and concrete are the following:

1. The thermal expansion coefficients of the two materials, about $6.5 \times 10^{-6}$ for steel vs. an average of $5.5 \times 10^{-6}$ for concrete, are sufficiently close to forestall cracking and other undesirable effects of differential thermal deformations.
2. While the corrosion resistance of bare steel is poor, the concrete that surrounds the steel reinforcement provides excellent corrosion protection, minimizing corrosion problems and corresponding maintenance costs.
3. The fire resistance of unprotected steel is impaired by its high thermal conductivity and by the fact that its strength decreases sizably at high temperatures. Conversely, the thermal conductivity of concrete is relatively low. Thus, damage caused by even prolonged fire exposure, if any, is generally limited to the outer layer of concrete, and a moderate amount of concrete cover provides sufficient thermal insulation for the embedded reinforcement.

Steel is used in two different ways in concrete structures: as reinforcing steel and as prestressing steel. Reinforcing steel is placed in the forms prior to casting of the concrete. Stresses in the steel, as in the hardened concrete, are caused only by the loads on the structure, except for possible parasitic stresses from shrinkage or similar causes. In contrast, in prestressed concrete structures large tension forces are applied to the reinforcement prior to letting it act jointly with the concrete in resisting external loads. The steels for these two uses are very different and will be discussed separately.

### 2.4.1 Reinforcing Bars

The most common type of reinforcing steel is in the form of round bars, often called rebars, available in a large range of diameters from about $\frac{3}{8}$ to $1\frac{3}{8}$ inch for ordinary applications and in two heavy bar sizes of about $1\frac{3}{4}$ and $2\frac{3}{4}$ inch. These bars are furnished with surface deformations for the purpose of increasing resistance to slip between steel and concrete. Minimum requirements for these deformations (spacing, projection, etc.) have been developed in experimental research. Different bar producers use different patterns, all of which satisfy these requirements.

#### 1. Grades and Strengths

In reinforced concrete, a long-term trend is evident toward the use of higher-strength materials, both steel and concrete. Reinforcing bars with 40 ksi yield stress, almost standard 30 years ago, have largely been replaced by bars with 60 ksi yield stress, both because they are more economical and because their use tends to reduce steel congestion in the forms. Bars with yield stress of 75 ksi are used increasingly in columns. Table 1 lists all presently available reinforcing steels, their grade designations, the ASTM specifications that define their properties (including deformations) in detail, and their two main minimum specified strength values.

Table 1: Summary of Minimum ASTM Strength Requirements

| Product | ASTM Specification | Designation | Minimum Yield Strength, psi (MPa) | Minimum Tensile Strength, psi (MPa) |
|---|---|---|---|---|
| Reinforcing bars | A615 | Grade 40<br>Grade 60<br>Grade 75 | 40,000 (280)<br>60,000 (420)<br>75,000 (520) | 60,000 (420)<br>90,000 (620)<br>100,000 (690) |
| | A706 | Grade 60 | 60,000 (420)<br>[78,000 (540) maximum] | 80,000 (550)[a] |
| | A996 | Grade 40<br>Grade 50<br>Grade 60 | 40,000 (280)<br>50,000 (350)<br>60,000 (420) | 60,000 (420)<br>80,000 (550)<br>90,000 (620) |

a. But not less then 1.25 times the actual yield strength.

The conversion to SI units described above also applies to the strength grades. Thus. Grade 40 is also designated as Grade 280 (for a yield strength of 280 MPa). Grade 60 is designated Grade 420 and Grade 75 is designated Grade 520. The values, 280, 420 and 520 result in minimum yield strengths of 40.6, 60.9 and 75.4 ksi: i.e., reinforcing steel is slightly stronger than implied by the grade in ksi.

Welding of reinforcing bars in making splices, or for convenience in fabricating reinforcing cages for placement in the forms, may result in metallurgical changes that reduce both strength and ductility, and special restrictions must he placed both on the type of steel used and the welding procedures. The provisions of ASTM A706 relate specifically to welding.

The ACI Code permits reinforcing steels up to yield strength of 80 ksi. Such high-strength steels usually yield gradually but have no yield plateau (See Fig. 2). In this situation it is required that at the specified minimum yield strength the total strain shall not exceed 0.0035. This is necessary to make current design methods, which were developed for sharp-yielding steels with a yield plateau applicable to such higher-strength steels. Under special circumstances steel in this higher-strength range has its place. e.g. in lower-story columns of high-rise buildings.

## 2. Stress-Strain Curves

The two chief numerical characteristics that determine the character of bar reinforcement are its yield point (generally identical in tension and compression) and its modulus of elasticity, $E_s$. The latter is practically the same for all reinforcing steels and is taken as $E_s = 29{,}000{,}000$ psi.

In addition, however, the shape of the stress-strain curve, and particularly of its initial portion, has significant influence on the performance of reinforced concrete members. Typical stress-strain curves for reinforcing steels are shown in Fig. 2. The complete stress-strain curves are shown in the left part of the figure: the right part gives the initial portions of the curves magnified 10 times.

Low-carbon steels, typified by the Grade 40 curve, show an elastic portion followed by a yield plateau, i.e. a horizontal portion of the curve where strain continues to increase at constant stress. For such steels, the yield point is that stress at which the yield plateau establishes itself. With further strains, the stress begins to increase again, though at a slower rate, a process that is known as strain-hardening. The curve flattens out when the tensile strength is reached; it then turns down until fracture occurs. Higher-strength carbon steels, e.g. those with 60 ksi yield stress or higher, say 75 ksi or 90 ksi either have a yield plateau of much shorter length or enter strain-hardening immediately without any continued yielding at constant stress. In the latter case, the ACI Code specifies that the yield stress $f_y$ be the stress corresponding to a strain of 0.0035, as shown in Fig. 2. Low alloy, high-strength steels rarely show any yield plateau and usually enter strain-hardening immediately upon beginning to yield.

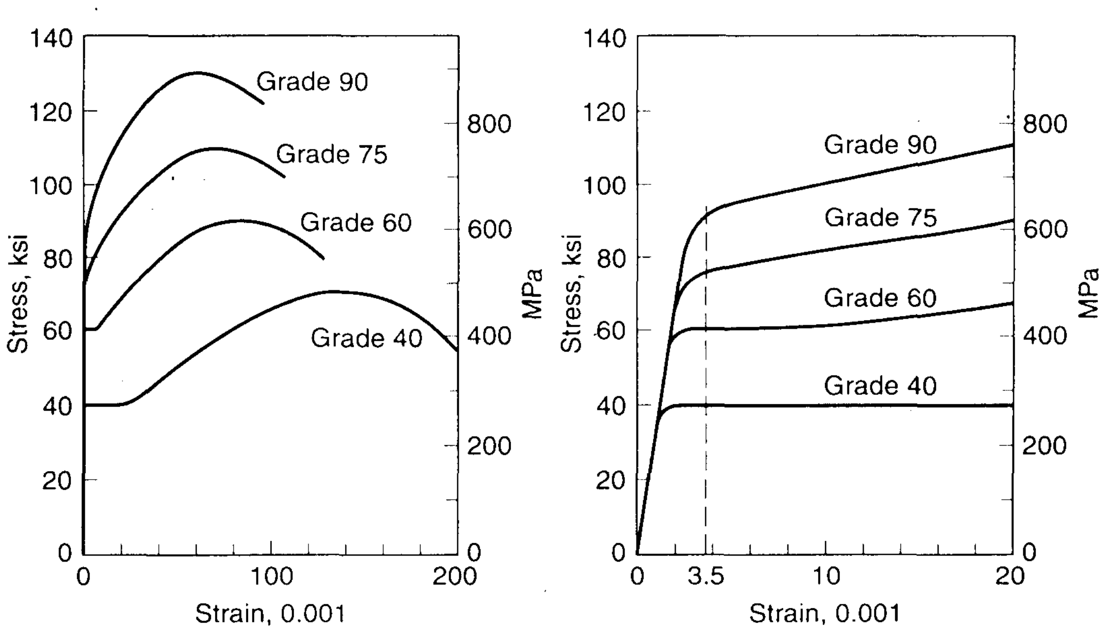



Fig. 2: Typical Stress-Strain Curves for Reinforcing Bars

## 3. Strength at High Temperatures

Deformed reinforcement subjected to high temperatures in fires tends to lose some of its strength as shown in Fig. 3. When the temperature of the reinforcement exceeds about 850°F, the yield and ultimate strengths both drop significantly. One of the functions of concrete cover on reinforcement is to prevent the reinforcement from getting hot enough to lose strength.

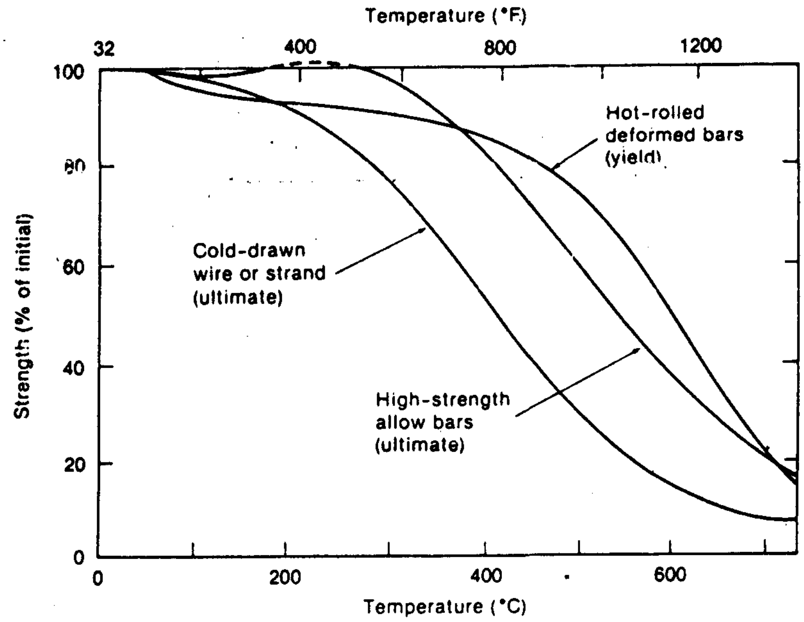



Fig. 3 Strength of Reinforcing Steels at High Temperatures

## 2.5 General Concerns for Concrete

In reinforced concrete the concrete itself is relied on primarily only for resistance of compressive stress. Its limitation in this regard is defined by the assumed design strength ($f'_c$), which is established essentially by a compression test on the material. Almost all other structural properties are based on this defined strength limit. The major resistance to tension is assigned to the steel reinforcement, so that investigations of the concrete are limited essentially to concerns for maximum compressive stress conditions.

Because of the nature of interaction of the two materials (concrete and steel) in the composite reinforced concrete structure, stress distributions between the materials are affected by their relative stiffness, as indicated by the modulus of elasticity of the materials. The modulus of the steel remains constant through all grades of the reinforcement. However, the modulus of the concrete changes. For this purpose, as well as any investigations of structural deformations, the concrete modulus of elasticity must also be established.

## 2.6 General Concerns for Reinforcement

Most steel used for reinforcement is highly ductile in nature. Its usable strength is its yield strength, as this stress condition initiates such a magnitude of deformation (into the plastic yielding range of the steel), that major cracking will occur in the concrete. Since the yield strength of the steel is quite clearly defined and controlled, this establishes a very precise reference in structural investigations. An early design decision is that for the yield strength (specified by the grade of steel used) that is to be used in the design work.

Several different grades of steel may be used for large projects, with a minimum grade for ordinary tasks and higher grades for more demanding ones. Cost increases generally for higher grades, so some feasibility studies must be made to see if the better steel in smaller quantities is really cheaper than a larger quantity of a lower grade. Actually, higher grades are often used to permit smaller concrete members, relating to the space problems for placement of the reinforcement.

Even though the steel ordinarily constitutes only a few percent of the total volume of reinforced concrete, it is a major cost factor. This includes the cost of the steel, the forming of the deformed bars, the cutting and bending required, and the installation in the forms. A cost-saving factor is usually represented by the general attempt to use the minimum reinforcement and the most concrete, reflecting typical unit costs for the two materials.

## 2.7 Investigation Methods

There are two fundamentally different methods for investigation and design of reinforced concrete: the working stress method and the strength method. The strength method is now used almost exclusively for professional design work and is the basis for development of the various computer-assisted processes now in wide use.

### 2.7.1 The Working Stress Method

As applied to the investigation of behaviors and the design of members of reinforced concrete, the working stress method consists of the determination of stresses in members that are induced by the actual loading under working conditions – called service load conditions. The stresses thus determined are then compared to the limits established for the situation under investigation. These limiting stresses – called allowable stresses – are established by code requirements as are the methods by which the actual stresses are determined. If the actual stresses do not exceed the allowable stresses, the member is considered to be adequate.

For concrete, allowable stresses are essentially based on the established design strength of the material. This strength is the so-called ultimate compressive strength, designated as $f'_c$, which is determined from the testing of standard samples.

Allowable stresses for steel are based on the yield strength of the steel, designated as $f_y$. Formulas used for determination of actual stresses in both the concrete and the steel are essentially derived from consideration of elastic behavior of members. In many cases, however, adjustments are made on the purely elastic formulas to account for the nonlinear stress-strain behavior of the concrete.

The working stress method is no longer favored by the codes and has largely been replaced in professional design practice by strength design methods. In general the working stress method is simpler to explain and its methods are easier and less complex to use. In some situations it is still used by designers and is allowed by most building codes. The latest edition of the ACI Code (Ref. 7) still provides for its use in a limited number of situations.

### 2.7.2 The Strength Method

Application of the working stress method consists of designing members to work in an adequate manner (without exceeding established stress limits) under actual service load conditions. The basic procedure in strength design is to design members to fail; thus the ultimate strength of the member at failure (called its design strength) is the only type of resistance considered. Safety in strength design is not provided by limiting stresses, as in the working stress method, but by using a factored design load (called the required strength) that is greater than the service load. The code establishes the value of the required strength, called U, as not less than

$$U = 1.2\,D + 1.6\,L$$

where D = effect of dead load
L = effect of live load

Other adjustment factors are provided when design conditions involve consideration of the effects of wind, earth pressure, differential settlement, creep, shrinkage, or temperature change.

The design strength of structural members (i.e., their usable ultimate strength) is determined by the application of assumptions and requirements given in the code and is further modified by the use of a strength reduction factor $\phi$ as follows:

$\phi$ = 0.90 for flexure, axial tension, and combinations of flexure and tension
$\phi$ = 0.70 for columns with spirals
$\phi$ = 0.65 for columns with ties
$\phi$ = 0.75 for shear and torsion
$\phi$ = 0.65 for compressive bearing

Thus while formula U = 1.2 D + 1.6 L may imply a relatively low safety factor, an additional margin of safety is provided by the stress reduction factors.

## 2.8 Safety Provisions

Structural members must always be proportioned to resist loads greater than the service or actual load in order to provide proper safety against failure. In the strength design method, the member is designed to resist factored loads, which are obtained by multiplying the service loads by load factors. Different factors are used for different loadings. Because dead loads can be estimated quite accurately, their load factors are smaller than those of live loads, which have a high degree of uncertainty. Several load combinations must be considered in the design to compute the maximum and minimum design forces. Reduction factors are used for some combinations of loads to reflect the low probability of their simultaneous occurrences. The ACI Code presents specific values of load factors to be used in the design of concrete structures.

In addition to load factors, the ACI Code specifies another factor to allow an additional reserve in the capacity of the structural member. The nominal strength is generally calculated using accepted analytical procedure based on statistics and equilibrium; however, in order to account for the degree of accuracy within which the nominal strength can be calculated and for adverse variations in materials and dimensions, a strength-reduction factor $\phi$ should be used in the strength design method.

To summarize this discussion, the ACI Code (2002 and 2005 versions) has separated the safety provision into an overload or load factor and to an under capacity (or strength-reduction) factor

$\phi$. A safe design is achieved when the structure's strength, obtained by multiplying the nominal strength by the reduction factor $\phi$, exceeds or equals the strength needed to withstand the factored loadings (service loads times their load factors). For example,

$$M_u \leq \phi M_n \qquad \text{and} \qquad V_u \leq \phi V_n$$

where $M_u$ and $V_u$ = external factored moment and shear forces
$M_n$ and $V_n$ =nominal ultimate moment and shear capacity of the member, respectively

Given a load factor of 1.2 for dead load and a load factor of 1.6 for live load, the overall safety factor for a structure loaded by a dead load D and a live load L is

$$\textit{Factor}\text{ of safety} = \frac{1.2\text{D} + 1.6\text{ L}}{D + L}\left(\frac{1}{\phi}\right) = \frac{1.2 + 1.6(L/D)}{1 + (L/D)}\left(\frac{1}{\phi}\right)$$

The factors of safety for the various values of $\phi$ and L/D ratios are shown in the following table.

| $\phi$ | 0.9 | | | | 0.75 | | | | 0.7 | | | | 0.7 | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| L/D | 0 | 1 | 2 | 3 | 0 | 1 | 2 | 3 | 0 | 1 | 2 | 3 | 0 | 1 | 2 | 3 |
| Safety factor | 1.33 | 1.56 | 1.63 | 1.67 | 1.60 | 1.87 | 1.96 | 2.00 | 1.71 | 2.00 | 2.10 | 2.14 | 1.85 | 2.15 | 2.26 | 2.31 |

For members subjected to flexure (beams), $\phi$ = 0.9, and the factor of safety ranges between 1.33 for L/D = 0 and 1.67 for L/D = 3.

For members subjected to axial forces (columns), $\phi$ = 0.65, and the factor of safety ranges between 1.85 for L/D = 0 and 2.30 for L/D = 3. The increase in the factor of safety in columns reflects the greater overall safety requirements of these critical building elements.

According to earlier provisions of American Concrete Institute (ACI) Building Codes (1995) the load factors for dead load and live load were 1.4 and 1.7 respectively. The strength reduction factors $\phi$ were as follows:

- $\phi$ = 0.90 for flexure, axial tension, and combinations of flexure and tension
- $\phi$ = 0.75 for columns with spirals
- $\phi$ = 0.70 for columns with ties
- $\phi$ = 0.85 for shear and torsion
- $\phi$ = 0.70 for compressive bearing

The overall safety factor for a structure loaded by a dead load D and a live load L is

$$\text{Factor of safety} = \frac{1.4\text{D} + 1.7\text{ L}}{\text{D} + \text{L}}\left(\frac{1}{\phi}\right) = \frac{1.4 + 1.7(\text{L}/\text{D})}{1 + (\text{L}/\text{D})}\left(\frac{1}{\phi}\right)$$

The factors of safety for the various values of $\phi$ and L/D ratios are shown in the following table.

| $\phi$ | 0.9 | | | | 0.8 | | | | 0.7 | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| L/D | 0 | 1 | 2 | 3 | 0 | 1 | 2 | 3 | 0 | 1 | 2 | 3 |
| Safety factor | 1.56 | 1.72 | 1.78 | 1.81 | 1.75 | 1.94 | 2.00 | 2.03 | 2.00 | 2.21 | 2.29 | 2.32 |

For members subjected to flexure (beams), $\phi = 0.9$, and the factor of safety ranges between 1.56 for L/D = 0 and 1.81 for L/D = 3.

For members subjected to axial forces (columns), $\phi = 0.7$, and the factor of safety ranges between 2.00 for L/D = 0 and 2.32 for L/D = 3.

## 3.0 Composition of Reinforced Concrete and Design of Structural Elements

### 3.1 Reinforced Concrete

In most structural applications, tension stresses of considerable magnitude have to be accommodated. For this purpose steel reinforcements (rods, bars or wires) are embedded in the concrete at the time of casting, so forming the composite material known as reinforced concrete. The reinforcements being steel have a high tensile strength and by judicious design they can be so disposed in the concrete as to be available to take all tensile stresses wherever these occur, whether as a result of direct tension forces or bending, shear or torsion. In this way full advantage is taken of the strength of the concrete in the compression zones of the structure, and the reinforcements provide the tensile strength which unreinforced concrete lacks. Reinforcements suitably disposed; can also serve to increase the strength of concrete members in compression, as when as control the effects of shrinkage and temperature changes.

Thus concrete, intelligently reinforced, is transformed from being brittle and unreliable into a composite material having compressive, tensile, bending, shear and tensional strength which the designer can adjust with economy to suit his requirements by varying the amount and disposition of the reinforcements. In this way works can be constructed in reinforced concrete using members of considerably reduced dimensions with consequent savings in weight, space and cost. Indeed the field of application for reinforced concrete extends far beyond the limit where the bulky nature of unreinforced concrete would render its use impracticable.

#### 3.1.1 Concrete

Concrete is a general term for conglomerates made artificially from cement, with sand and broken stone or similar materials described generally as fine and coarse aggregates. These ingredients, mixed together with water, form a plastic mass which sets on standing into the hard solid material known as concrete. Concrete resembles stone in weight, hardness, brittleness and strength. Depending on the quality and the proportions of ingredients used in the mix, the properties of concrete can vary almost as widely as the different kinds of stone that occur in nature.

Concrete when not reinforced has considerable strength in compression but very little strength in tension; the ratios varying between about 10: 1 and 15: 1. Having a low tensile strength, it follows that concrete is weak in bending, shear and torsion. The tensile strength besides being low is also unreliable: it may be entirely destroyed by shock or sudden jar, or as a result of shrinkage arising from setting or drying, or due to thermal contraction. Hence the use of unreinforced concrete is normally limited to applications where great compressive strength and weight are the principal requirements, and where tension and bending stresses are either totally absent or, if they occur at all are extremely small. Examples of the use of unreinforced concrete include thick spread foundations, dock walls, dams and gravity retaining walls.

### 3.1.2 Reinforcements

Steel reinforcement for concrete consists of bars, wires and welded wire fabric, all of which are manufactured in accordance with ASTM Standards. To increase the bond between concrete and steel, projections called *deformations* are rolled on the bar surface in accordance with ASTM Specification. The most common types of reinforcement for reinforced concrete member are hot-rolled deformed bars. The following ASTM Specifications specify certain dimensions, certain chemical and mechanical properties.

**ASTM A615:** Specification for Deformed and Plain Billet-Steel Bars for Concrete Reinforcement. This specification covers the manufacture of deformed bars from steel billets. These are available in Grade 40, Grade 60 and Grade 75 (yield strengths 40, 60 and 75 ksi) of which Grade 40 and Grade 60 are the most common types of reinforcing bars.

**ASTM A706**: Specification for Low-Alloy Steel Deformed Bars for Concrete Reinforcement. This specification covers bars intended for special applications where weldability, bendability, or ductility are important. ACI Appendix A or, seismic design requires the use of A706 bars or A615 bars meeting special requirements.

Reinforcing bars are usually available in three grades, with yield strengths of 40, 50 and 60 ksi, referred to as Grades 40, 50 and 60 steels, respectively. Of these, the third is most commonly used in buildings, while the first is common in other types of structures. Generally speaking, the 40-ksi reinforcement is the most ductile of all the Grades. At various times high-strength deformed bars with yield strengths of 75 or 80 ksi have been used as reinforcement in columns. The use of such high strength deformed bars are increasing these days. In recent years Grade 50 steel has become less and less available.

Idealized stress-strain relationships are given in Fig. 2.0 for Grade 40, Grade 60, Grade 75 and Grade 90 reinforcing bars. The initial tangent modulus of elasticity for all reinforcing bars can be taken as $29 \times 10^6$ psi. High-strength bars, Grade 75 and Grade 90 generally do not have a well-defined yield point.

The ASTM Specifications for reinforcing bars define the yield strength as the stress at a strain of 0.005. It has been already reported in 2.4.1 that ACI Code specifies the yield strength as the stress at a strain of 0.0035. This is done because concrete crushes at a strain of roughly this value, and when bars are used in compression, as in columns, a strain of 0.005 may never be reached. The 0.005 strain yield strength often exceeds the 0.0035 yield strength for high-strength bars. Thus yield strengths determined in accordance with the ASTM Specifications may be higher than those allowed by the ACI Code.

### 3.1.3 Steel Stresses

The availability of reinforcing steel of various strengths leads the designer to choose freely for reinforced concrete designs. During sixties of the last century 40 grade steel was mainly used in reinforced concrete designs in all over the world. Use of steel of this grade produced congestion in many cases. Later on grade 50 and grade 60 bars were available for the use and the designers immediately switched to high grade and preferred grade 60 in almost all cases. Because of relatively little extra cost, the grade 60 steel has rapidly replaced the formerly predominantly used grade 40 steel.

It has long been established that in reinforced concrete construction the protection afforded to the reinforcement by the covering concrete is not impaired by the formation of very fine cracks in the concrete. When the high strength steel such as grade 75 and higher is used in reinforced concrete there is a possibility that the normal hair cracks open up to excessive widths. The possibility led to the use of grade 75 steels in the columns for a reasonable time. With the use of such high grade steel the total reinforcement in columns is reduced. These days the 75 grade steel is being increasingly used in many designs. For the use of 75 grade steel higher concrete compressive strength is preferred. **However, from ultimate strength consideration the compressive strength of 3000 psi is also compatible to be used with 75 grade steel.**

### 3.2 Design of Reinforced Concrete Elements

#### 3.2.1 Flexure in Beams

Loads acting on a structure, be they live gravity loads or other types such as horizontal wind loads or those due to shrinkage and temperature, result in bending and deformation of the constituent structural elements. The bending of the beam element is the result of the deformational strain caused by the flexural stresses due to the external load.

As the load is increased, the beam sustains additional strain and deflection, leading to development of flexural cracks along the span of the beam. Continuous increases in the level of the load lead to failure of the structural element when the external load reaches the capacity of the element. Such a load level is termed the limit state of failure in flexure. Consequently, the designer has to design the cross section of the element or beam such that it would not develop excessive cracking at service load, levels and have adequate safety and reserve strength to withstand the applied loads or stresses without failure.

Flexural stresses are a result of the external bending moments. They control in most cases the selection of the geometrical dimensions of a reinforced concrete section. The design process through the selection and analysis of a section is usually started by satisfying the flexural (bending) requirements, except for special components such as footings. Thereafter, other factors, such as shear capacity, deflection, cracking, and bond development of the reinforcement, are analyzed and satisfied.

While the input data for the analysis of sections differ from the data needed for design, every design is essentially an analysis. One assumes the geometrical properties of a section in a design and proceeds to analyze such a section to determine if it can safely carry the required external loads. Hence a good understanding of the fundamental principles in the analysis procedure significantly simplifies the task of designing sections. The basic mechanics of materials principles of equilibrium of internal couples have to be adhered to at all stages of loading.

The following assumptions are made in defining the behavior of the section:

1. Strain distribution is assumed to be linear. This assumption is based on Bernoulli's

hypothesis that plane sections before bending remain plane and perpendicular to the neutral axis after bending.

2. Strain in the steel and the surrounding concrete is the same prior to cracking of the concrete or yielding of the steel
3. Concrete is weak in tension. It cracks at an early stage of loading at about 10% of its limit compressive strength. Consequently, concrete in the tension zone of the section is neglected in the flexural analysis and design computations, and the tension reinforcement is assumed to take the total tensile force.

### 3.2.2 Combined Compression and Bending: Columns

Columns are vertical compression members of a structural frame intended to support the load-carrying beams. They transmit loads from the upper floors to the lower levels and then to the soil through the foundations. Since columns are compression elements failure of one column in a critical location can cause the progressive collapse of the adjoining floors and the ultimate total collapse of the entire structure.

Structural column failure is of major significance in terms of economic as well as human loss. Thus extreme care needs to be taken in column design with a higher reserve strength than in the case of beams and other horizontal structural elements, particularly since compression failure provides little visual warning. The ACI Code requires a considerably lower strength reduction factor $\phi$ in the design of compression members than the $\phi$ factors in flexure, shear, or torsion.

The principles of stress and strain compatibility used in the analysis (design) of beams discussed earlier are equally applicable to columns. A new factor is introduced, however: the addition of an external axial force to the bending moments acting on the critical section. Consequently an adjustment becomes necessary to the force and moment equilibrium equations developed for beams to account for combined compression and bending.

The amount of reinforcement in the case of beams was controlled so as to have ductile failure behavior. In the case of columns, the axial load will occasionally dominate; hence compression failure behavior in cases of a large axial load/bending moment ratio cannot be avoided.

As the load on a column continues to increase, cracking becomes more intense along the height of the column at the transverse tie locations. At the limit state of failure the concrete cover in tied columns or the shell of concrete outside the spirals of spirally confined columns spalls and the longitudinal bars become exposed, Additional load leads to failure and local buckling of the individual longitudinal bars at the unsupported length between the tics. It is noted that at the limit state of failure the concrete cover to the reinforcement spalls first after the bond is destroyed.

As in the case of beams the strength of columns is evaluated on the basis of the following principles:

1. A linear strain distribution exists across the thickness of the column.
2. There is no slippage between the concrete and the steel (i.e. the strain in steel and in the adjoining concrete is the same).
3. The maximum allowable concrete strain at failure for the purpose of strength calculations = 0.003 in./in
4. The tensile resistance of the concrete is negligible and is disregarded in computations.

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