12 Reinforced and Prestressed Concrete Design

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12.1 Introduction

The design of reinforced and prestressed concrete has been increasingly codified during the past 40 years. Before the Second World War, recommendations for design had been published in the UK in a Code of Practice prepared by the Department of Scientific and Industrial Research, which was issued in 1934, and in the Building By-laws of the London County Council of 1938. After the war, the DSIR Code was revised and became the British Standard Code of Practice, CP 114, in 1948.

The Institution of Structural Engineers published its First Report on Prestressed Concrete in 1951, which gave design procedures for prestressed construction. This report was subsequently revised and issued as BS Code of Practice, CP 115, in 1959. The BS Code of Practice for the Design of Precast Concrete, CP 116, appeared in 1965 and supplemented the two earlier BS codes. By this time, a number of codes dealing with specialized forms of concrete construction were being prepared. Codes of Practice 114, 115 and 116 have been updated from time to time and are currently adopted as deemed-to-satisfy documents in the Building Regulations.

An important innovation took place in 1972 when a unified BS code, CP 110, for the structural use of concrete was published. This code, which it was intended should supersede Codes 114, 115 and 116, introduced a new feature in design, namely limit state design in which account was taken directly of the possibility of failure or unserviceability occurring during the life of the structure being designed. The particular factors considered included the risks resulting from variability of the materials, inaccuracy in design assumptions and construction, variability of loading and the incidence of accidental damage. Whilst the approach to design was modified, many existing methods of analysis and calculation were retained. Provision was made for the incorporation of new data on loading and materials, and on structural performance and methods of construction as they became available. The basis for this approach had been developed by the European Committee for Concrete assisted by the International Federation for Prestressing who published a jointly prepared code in 1978 having previously issued separate codes. This code was used in the production of a code for the European Economic Community.

Code of Practice 110 did not replace the earlier Codes 114, 115 and 116, which still remain in force, but it has now been revised as BS 8110. The approach adopted in CP 110 has been retained and the content has been brought up to date. In addition, a manual has been prepared by the Institution of Structural Engineers conforming with its recommendations but presented in simpler form and dealing with a more limited range of construction. The guidance given in this chapter is related directly to the contents of BS 8110.

Whilst these developments were taking place in the UK, somewhat similar changes were occurring elsewhere. Some idea of the differences between the recommendations adopted in the UK and elsewhere are given in Table 12.1, which makes some comparisons between BS 8110, the American Building Code, ACI 318-83 and the EEC Code.

12.2 Behaviour of structural concrete

The characteristics of concrete that have conditioned its development as a structural material are its high compressive strength and relatively low tensile strength. In consequence its use for flexible members did not become practicable until it was discovered that steel reinforcement could be cast in the concrete to carry the bending tensile stresses whilst relying on the concrete to carry the bending compressive stresses. Experiment showed that mild steel, when present in the tension zone in relatively small amounts, provided a material with characteristics for deformation and strength which complemented those for concrete and provided a practical form of construction. Early research workers concluded that the presence of the steel increased the extensibility of the concrete. Later experiments showed, however, that this was not so. It then became clear that as the tensile stress in the steel of a beam increased beyond a small amount, which is appreciably less than that developed under service loading, cracks developed in the concrete. These cracks were controlled in width and numbers by the position of the reinforcement relative to the concrete surface and by the size of internal forces in reinforced concrete, generally consists of one of three types of material: plain round mild-steel bar produced by hot-rolling; plain square or plain chamfered square twisted mild-steel bar which has had its yield stress raised by cold-working; ribbed bars, which may be hot-rolled from steel with high yield stress or cold-worked by twisting from hot-rolled mild-steel.

Since steel reinforcement can only develop an effective tensile force by extension of the concrete by cracking, there is a limit on the maximum strength of steel that can be used. In general the yield stress should not exceed 500 N/mm² although higher strength steels may be used if particular care is taken to avoid excessive cracking or deflection.

(2) Tendons are used to impart a prestress to concrete before service loads are applied which offsets the tensile stresses which will later result from the application of these loads. Tendons are usually comprised of plain, indented or deformed cold-drawn carbon steel wire, of seven-wire or nineteen-wire strand spun from one or two layers respectively of cold-drawn carbon steel wire around a core wire, or of high-tensile alloy steel bar. The strength of steel used must be high enough for it to be extended sufficiently to avoid excessive loss of tension due to elastic contraction, creep and shrinkage of the concrete. In general it is not of lower tensile strength than about 1000 N/mm².

(3) In prestressed concrete, prestressing may be effected by pretensioning or post-tensioning the tendons. Pretensioned tendons are stressed before the concrete is cast. They are stretched either between temporary anchorages placed sufficiently far apart for a number of moulds to be assembled in line around the tendons, i.e. the 'long-line' method, or between the ends of specially strong moulds, i.e. the 'individual' mould method; in each case, concrete is then cast and allowed to harden before the tendons are released from their temporary anchorages. The methods are best-suited to mass production in the factory and usually use wire or the smaller sizes of strand as tendons.

With post-tensioning, however, the tendons are stressed after the concrete has hardened and are usually accommodated in ducts within the concrete being held at their ends by anchorages, of which there are various proprietary types. Subsequently the ducts are grouted with cement grout to protect the tendons from corrosion. This method is mostly applied to site construction and tends to use tendons of relatively large size.

12.1.1 Definitions

This chapter is concerned with the basic approach to design of reinforced and prestressed concrete. It deals with both cast-in-place and precast concrete whether reinforced or prestressed. It includes information on the use of plain or deformed steel reinforcing bars and with tendons which may be either pretensioned or post-tensioned. In this context some definitions and an indication of limitations may be useful.

(1) Reinforcement which is used to provide the tensile compo-
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<table>
<thead>
<tr>
<th>Table 12.1 Notes on different Codes of Practice (British, American and EEC)</th>
<th>(A) BS 8110 – Structural use of concrete&lt;sup&gt;18&lt;/sup&gt;</th>
<th>(B) ACI 318 – Building code requirements for reinforced concrete&lt;sup&gt;12&lt;/sup&gt;</th>
<th>(C) Eurocode No. 2 – ‘Common unified rules for concrete structures’&lt;sup&gt;9&lt;/sup&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>Status</td>
<td>(A) This national code was prepared by the British Standards Institution, an organization with some direct support from Government, and accepted as providing conformity with British Building Regulations, but not in itself mandatory, other authenticated design procedures may be acceptable.</td>
<td>(B) This national code was prepared by the American Concrete Institute; it is used extensively in State regulations for building control. It is widely recognized internationally and is adopted in part or wholly in the codes of a number of other countries.</td>
<td>(C) The code has been prepared by the Commission of the European Community for use in member countries, and is one of a number now being produced to deal with all common materials and forms of construction. It is likely to be adopted for building control in those countries and will be recognized as satisfying the requirements of national regulations. The code has drawn on the work of international organizations, which are supported worldwide, and hence it is likely to have an important influence on the formulation and revision of codes in other countries outside as well as inside the Community.</td>
</tr>
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</table>

Design procedure

(A) Limit state procedures (described in this chapter) are adopted following closely the 1964 Recommendations of the European Committee for Concrete, which were used subsequently in developing the EEC Code; the basic approach in the two codes is therefore very similar.

The ultimate limit states include strength and stability under dead and imposed loads, wind loads, and earth and water pressure for which partial safety factors are defined depending on load groupings, and the effects of accidental loading and damage. Durability and fire resistance are not treated as limit states but are included in the design process, the former being given more emphasis than in previous codes. As far as possible the analysis of structures is based on ultimate behaviour but, where methods have not been developed, elastic analysis is accepted. The strength of sections is based on the strength of the materials, as reduced by partial safety factors, and compatibility between stress and strain using idealized stress–strain relationships. Simplifying assumptions relating to these stress–strain relationships are allowable for many types of construction.

The serviceability limit states include deflection and cracking under dead, imposed and wind loads, appropriate partial safety factors for combinations of loading being given with limitations on deflection and crack width. Where necessary, allowance is required for the effects of shrinkage and creep and of temperature change. For common types of construction, limits on deflections are imposed by placing limits on span:depth ratio or by checking that the long-term deflections do not exceed defined limiting values. Cracking is controlled by limitation of calculated crack width and provision of reinforcement for both reinforced and prestressed concrete. In (A), on the other hand, cracking of prestressed concrete may be controlled by limitation of tensile stress, the nominal tensile stress being related to amount and distribution of secondary reinforcement.

The ACI Code has an appendix which gives an alternative method of design for reinforced concrete which is based on permissible stresses in the materials.
ratios, and cracking may be controlled for reinforced concrete by defining the form that the reinforcement should take.

Since many practical engineers have pressed for the retention of permissible stress methods of design, the previous code applicable for reinforced concrete has been retained in use. It seems likely, however, that it will be withdrawn in the longer term.

of bars used. Thus with closely spaced bars near the surface, a large number of small cracks would develop, but with large widely spaced bars, the cracks would be fewer in number and much larger for the same stress in the steel. If the stress in the steel were increased the size of the cracks increased and their size was little influenced by the surface roughness of the steel, although at one time it was thought that roughening of the surface resulted in appreciably smaller cracks of larger numbers. It was eventually established that the main benefit of using bars with a roughened surface was in developing good end-anchorage.

Because steel needs to extend to develop stress and hence causes cracking and deformation of the concrete, there is a limit to the strength of steel that can be used efficiently for reinforcement, since unsightly cracking, which could lead to severe corrosion in adverse conditions and unacceptable deflections, must be avoided. The use of steel in prestressed concrete, where the stress in the steel is imposed before the concrete member is subjected to external load, avoids this problem, since the initial tensile force is developed without extending the concrete, and so no upper limit is imposed on the strength of steel that can be employed. This was not, however, appreciated in the early development of prestressed concrete. Then, steel of relatively low strength was used with a small initial tension. The experimenters found that, although this was effective at the start, the initial prestress disappeared with time. Eventually, however, it was established that this nonelastic behaviour was limited in extent and that if a sufficiently large elastic extension was imparted to the steel, the nonelastic effects of creep and shrinkage of the concrete did no more than reduce the prestress by an acceptable amount. Although for a time there was a tendency to underestimate the losses of prestress due to contraction of the concrete and to ignore creep in the steel tendons, research has now, however, clearly set the limits on what needs to be considered in design.

The performance of reinforced concrete and prestressed concrete beams under increasing load is characteristically different since cracking develops in different ways in each form of construction. This is illustrated by the results of tests on beams in each form of construction as illustrated in Figures 12.1 and 12.2.

Examined in more detail the deformation of the reinforced concrete beam under load is linear until cracking occurs; thereafter it approximates to a linear relationship until the steel yields as cracking becomes more extensive for beams of normal design. Subsequent deformation leads to the development of a hinge with continued yielding of the steel accompanied by damage to the concrete. This deformation continues at approximately constant moment until a stage is reached where the resistance reduces. The occurrence of this stage is influenced by the amount of transverse shear reinforcement in the section.

| Table 12.1 Notes on different Codes of Practice (British, American and EEC)—continued |
|---|---|---|
| (A) BS 8110 – Structural use of concrete | (B) ACI 318 – Building code requirements for reinforced concrete | (C) Eurocode No. 2 – “Common unified rules for concrete structures” |

Concluding comment: Of necessity, these comparisons are very limited and superficial in character but should serve to show that developments in codes proceeding currently in different countries have much in common. This trend is likely to increase through the medium of the extensive international collaboration that now takes place.

The prestressed concrete beam, however, remains uncracked usually until the service load is exceeded, and in this range its deformation is elastic. Once cracking has occurred deformation increases disproportionately rapidly with increasing load as cracks widen until the maximum load is reached. Subsequently there is a rapid reduction in resistance. Since the prestressed concrete beam is usually uncracked under service conditions its stiffness is greater than that of reinforced concrete beams of the same overall depth.

In continuous construction subjected to applied loads of short duration, deformation of both reinforced concrete and prestressed concrete members is elastic or effectively elastic until service loads are exceeded. With further loading, as the applied moment at any section approaches the resistance moment at that section, there is a tendency for the moment to be relaxed and redistributed to sections that are less seriously stressed. Thus a loaded beam, built in at each end, may reach its...
maximum resistance moment at mid-span before the maximum resistance moments at the supports are attained; a hinge then forms at midspan with the applied moment there remaining sensibly constant whilst the applied moments at the supports increase until hinges form at the supports. The beam has then reached its maximum carrying capacity. The capability of reinforced and prestressed concrete beams for rotation at hinges is limited, however, and restrictions therefore need to be placed on allowances in design for redistribution of moment. These allowances are smaller for prestressed concrete sections than for reinforced concrete sections since their rotational capacities are smaller.

Under long-term loading, the deflection of reinforced concrete beams increases usually to about 2 or 3 times the initial deflection. Although the initial deflection is primarily influenced by the amount of steel in the section and its stress, the subsequent deflection is largely the result of creep of the concrete, breakdown of bond between the steel and the concrete in the tension zone between cracks which initially stiffens the beam, and the effect of the reinforcement in restraining the shrinkage of the concrete.

Since prestressed concrete is usually uncracked under long-term load the initial deflection is mainly due to the deformation of the concrete. The subsequent deflection results mainly from creep of the concrete and depends on the combined effects of the prestress and the stresses due to applied load. The former tends to deform the member in the opposite direction to the latter. In consequence, a loaded prestressed concrete member may initially have an upward deflection which can continue to develop upwards or downwards depending on how heavily it is loaded.

Under cyclic loading, reinforced concrete members usually fail in fatigue by fracture or yield of the reinforcement. The properties of most reinforcing steels, provided that they are free from welded connections, are, however, such that the ranges of stress experienced under service loading determined for static conditions are usually within the fatigue range. Cyclic loading leads to some increase in deflection of reinforced concrete members partly due to deformation of the concrete and partly due to breakdown of bond between cracks. Since prestressed concrete is uncracked under normal static service load conditions, the fluctuations of stress in the steel under cyclic loading are small. Fatigue failure of the steel only occurs when substantial cracks have developed and deflections are generally unacceptable. The effect of cyclic loading on prestressed concrete is to increase deflection by a small amount, i.e. 20 to 30% largely as a result of creep of the concrete. Large numbers of repetitions within the normal range of service loading do not reduce the ultimate strength of prestressed or reinforced concrete. Because of its freedom from cracking, prestressed concrete behaves better than reinforced concrete under severe cyclic loading and has therefore been used extensively for railway sleepers.

Resistance of beams to impact is indicated by the energy absorbed in deforming which is given by the area of the load deflection curves. Referring again to Figures 12.1 and 12.2, the deformation of prestressed and reinforced concrete beams has been defined in three stages. In stage I, deformation is elastic and largely recoverable; in stage II, deformation is in part elastic but accompanied by cracking and is partly recoverable; whilst in stage III, deformation is mainly due to permanent damage to the materials. Since stages I and II represent the largest amounts of absorbed energy for prestressed concrete, this material has a considerable capacity for recovery after impact. For reinforced concrete, the energy absorbed in stage III is substantially greater than in the other two stages. Thus, reinforced concrete does not show much recovery after impact but has a high ultimate impact resistance which is appreciably higher than that for prestressed beams designed for the same static loads. Prestressed concrete beams are, however, better in resisting repetitions of relatively light impacts with little residual damage.

So far, performance has been considered mainly in terms of bending conditions, but conditions of direct stress in compression exist in columns and walls. In such construction, unless high bending moments are also likely to occur, prestressed concrete would be unsuitable and reinforced concrete should be used with the steel acting in compression. For columns, transverse steel in the form of links is essential to contain the longitudinal steel and ensure ultimate resistance to strains in excess of those causing failure of plain concrete. Evidence from long-term tests also shows that the effect of creep of the concrete in a column under load is to raise the stress in the longitudinal steel to its yield stress and hence there is a need to retain it in its correct alignment. Walls when lightly reinforced are slightly weaker than walls without reinforcement and they can therefore only be treated as reinforced when the longitudinal reinforcement exceeds a specific minimum.

Other aspects of behaviour which are of importance are shear and torsion. In each case if these cause failure, the mode of failure tends to be brittle and less ductile than bending failures. Hence in design, the procedure is to avoid such failure by the inclusion of sufficient transverse reinforcement to ensure bending or compression failure in the event of severe overloading.

Members subjected solely to tension are relatively rare. If they are of reinforced concrete, then the role of the concrete is to protect the reinforcement which is designed to take the whole tensile force. In prestressed members, however, the precompressed concrete can sustain the tension until the load exceeds the cracking loading when the behaviour reverts to that of reinforced concrete with the steel carrying the whole of the tension, stiffened to some extent between cracks by the concrete.

For most building structures, the Building Regulations define fire resistance requirements, which are expressed in terms of a required endurance under service load when components are subjected to a standard heating regime. Both reinforced concrete and prestressed concrete are primarily influenced in their behaviour in fire by the behaviour of the steel at high temperature; as its temperature is raised its strength and yield characteristics are reduced. For reinforcing steels the rate of reduction in strength is lower than for steels used in tendons and hence greater amounts of protection are needed for prestressed concrete. This may take the form of concrete cover and the optional addition of insulating material. It is often easier, however, to provide the greater thicknesses of cover needed for tendons without loss of efficiency than that needed for reinforcement, since the positioning of tendons is governed by different requirements.

The need to provide adequate durability also affects the amount of cover required to the reinforcement or tendons. As concrete ages, carbon dioxide in the air causes carbonation of the concrete which, as it progresses, reduces its capacity for inhibiting rusting of the steel. For dense concrete the rate of progress is very low but, since defects exist, experience has shown that a greater thickness of concrete is required to prevent spalling of the concrete caused by expansion of the corrosion products on rusting. Cover requirements also affect the width of cracks that are likely to occur and hence need attention in dealing with serviceability.

These characteristics of the behaviour of both reinforced and prestressed concrete are considered in more detail in presenting design procedures.

12.3 Philosophy of design

The early developments of the design of reinforced concrete were crystallized in this country by the issue in 1934 of Recom-
mendations for a Code of Practice prepared by a committee set up by the Department of Scientific and Industrial Research. It was based on the premise that the stresses in the steel and concrete should not exceed certain permissible values, related to the strengths of the materials by safety factors, when the structure was subjected to the maximum loads that it would need to carry in service. The materials were assumed to behave elastically and compatibility of strains between steel and concrete was ensured by assigning a value for the ratio of their moduli of elasticity. Some account was taken of the inelastic effects of creep of concrete by adopting a low value for the modulus of elasticity of concrete in determining the modular ratio for use in the design calculations. No account was taken of the effects of shrinkage and no estimate was made of the ultimate strength of the structure. When the British Standards Institution issued its first Code for Reinforced Concrete, CP 114, in 1948, it followed the same general approach. In the revision in 1957, however, there was an alternative method for design in flexure which limited the stresses to the same permissible values as for elastic design but assumed that they were distributed as at failure and avoided the use of the modular ratio; this was therefore a form of ultimate strength design.

Limitations on the permissible stresses in the steel and on span:depth ratios were imposed to guard against excessive deflection or cracking. Thus it could be argued that CP 114 provided for safety against failure and for the avoidance of unserviceability.

The earliest formal presentation of a design procedure for prestressed concrete was contained in the First Report on Prestressed Concrete published by the Institution of Structural Engineers in 1951. Many of the recommendations in that report found their way into the British Standard Code of Practice for Prestressed Concrete, CP 115, issued in 1959. It conformed with CP 114 in the sense that it was based primarily on the limitation of stresses to permissible values related to the strengths of the materials with the object of preventing cracking and avoiding excessive deflection. It also provided for the calculation of ultimate strength and introduced separate requirements for minimum load factors for the dead and imposed loads.

Thus, when the drafting of CP 110 commenced in 1964 it had already been demonstrated that there were a number of limiting conditions or limit states which had to be considered by the designer in the overall conception of structural safety and adequacy. These were primarily limits of collapse, deformation and cracking, but other matters such as the effects of vibration, of fatigue, of deterioration with time or as a result of fire, needed attention in the design process.

A further major change in the content of structural codes first introduced in CP 110 in 1972 was the move towards considering the coordinated design of the structure as a whole for safety and serviceability rather than the separate design of its component parts with only limited appreciation of their interaction. This development has become necessary partly as a result of the evolution of design philosophy and partly because the utilization of the materials has become more onerous following the general increase in the levels of stress in both concrete and steel under service conditions.

12.3.1 Criteria for limit state design

The aim in limit state design is to codify the procedures normally adopted by engineers in the design of structures to provide safe, serviceable and economic construction with a reasonable degree of certainty, and to do this with a better appreciation of the margins of safety and of ignorance involved. As far as possible, it takes into account the variations likely to occur in the loads on the structure and in the strength of the materials of which it is comprised; it can allow for inadequacies of construction and methods of analysis, and should lead to design being more closely related to the risk of occurrence of specific conditions of failure and unserviceability.

For the purposes of design, both loads and strengths are expressed in terms of characteristic values. For loads, these are defined loads with a small but acceptable risk that they will be exceeded in service; they are given in the British Standard loadings for buildings, in BS 5400 for highway bridges and in other standards for other construction. To meet the needs of limit state design, there has been a move in recent years away from specifying loads as maximum values and towards expressing them in terms of their likelihood of occurrence where possible determined from observations of their imposition on structures (see Chapter 19).

The characteristic values of loads allow for normally expected variations in loading but not for: (1) unforeseen loading effects; (2) lack of precision in design calculations; (3) inadequacies in the methods of analysis; and (4) dimensional errors in construction which alter the assumed positions or directions of loads and their effects, e.g. incorrect positioning of reinforcement and inaccurate alignment of columns in successive storeys. The values for loads used in design are therefore increased by partial safety factors to cater for these effects and to provide the margin of safety appropriate to the need for ensuring that a particular limit state is not reached. Thus, for conditions of failure, higher values are used than for those of serviceability. Where a combination of loads is assumed to be acting, the partial safety factors for each source of loading are smaller since the simultaneous occurrence of high values for each load is less likely. The loads for use in the design are therefore the sums of the products of the appropriate characteristic loads and their partial safety factors for the limit states and combination of loads being considered. For simplicity, the structural code for concrete, BS 8110, reduces the number of situations needing consideration to a minimum, as will be seen later.

Characteristic values for the strengths of materials are usually given in the relevant standard or code. Research on materials shows that their strengths conform reasonably closely to a normal distribution, and their characteristic strengths can therefore be stated as follows:

Characteristic strength = mean strength - \( k_1 \times \text{standard deviation} \) or:

\[
\bar{f} = \bar{f}_c - k_1 \times \sigma_f
\]  

(12.1)

\( k_1 \) is usually given a value of 1.64, which ensures for a normal distribution that not more than 5% of strengths are less than the characteristic strength. This definition of strength has been adopted in British Standards for both steel and concrete.

The magnitude of the loads used in design is therefore increased by factors, partial safety factors for loads, to cater for these effects and to provide a margin of safety appropriate to the need for ensuring that any particular limit state is not reached. Thus, when envisaging conditions of failure, higher values for the factors are adopted than when considering serviceability.

The strengths of the materials used in the design calculations are those defined in the specification for the structure, which are checked by physical tests. The strengths of the materials as they exist in the structure, however, are likely to differ from those determined from test specimens and some allowance is also required for changes or deterioration with time. Partial safety factors for the materials are therefore introduced and the strengths taken for design are the characteristic strengths divided by a partial safety factor, \( \gamma_m \), which has a value depending on the limit state being considered and the nature of the material, being less for steel than for concrete.
An idealized and simplified situation for a homogeneous material is illustrated in Figure 12.3. The provisions for safety outlined so far then require:

\[ F_k \cdot \gamma_e \leq f_s / \gamma_m \]  

(12.2)

where \( F_k \) = the characteristic load

This conforms reasonably closely with what has now become accepted practice in the recent revisions of British Standards codes, and was first adopted in CP 110 in 1972. Current thought, however, accepts the view that a further partial safety factor should be introduced to take account of the nature of the construction and its behaviour under overload conditions, e.g. whether it is capable of sustaining large deformations and so giving warning of the imminence of collapse, and of the seriousness of failure in terms of the risks to health, life and property. This factor, \( \gamma_e \), might have a value of less than 1 for temporary construction not normally occupied by human beings but of more than 1 for buildings with large spans used for public assemblies. Thus design would then require:

\[ F_k \cdot \gamma_e \cdot \gamma_m \leq f_s / \gamma_m \]  

(12.3)

For an idealized situation, the global factor of safety relating characteristic loads to characteristic strength is then \( \gamma_f \cdot \gamma_m \).

If the concept of relating the factors of safety to the nature of the construction is not followed, then the global factor is \( \gamma_e \cdot \gamma_m \). Since reinforced concrete and prestressed concrete are composite materials, the value of the global factor for each limit state cannot be expressed as simply as this; it is dependent on the interaction between steel and concrete, each of which has a different value for \( \gamma_m \). Also, \( \gamma_e \) cannot be given a single value for each limit state since the partial safety factors for dead, imposed, wind and other loads may differ and change with different combinations of loads. Hence, only upper and lower values for the global factor can be defined which makes comparison of the new Code with earlier or other codes imprecise. Nevertheless, in preparing the new Code, the aim has been to avoid substantial changes in the dimensions of the resulting structures whilst at the same time obtaining more consistent levels of safety and leaving room for development on more rational lines in the future.

It is convenient to divide the limit states to be considered in design into two kinds, namely those concerned with collapse and those concerned with serviceability. Limit states of collapse deal with overturning of the complete structure, failure of the whole or a large part of the structure as a result of overstressing of a number of sections or buckling of a number of compression members or as a result of a serious accident; the effects of fire and fatigue may also be included. Deflection, cracking, deterioration, corrosion and vibration are all aspects of serviceability and require limits of acceptability to be set for consideration. In the Code for Structural Concrete the limit states specifically dealt with are ultimate conditions in general, and deflection and cracking under the heading of serviceability. The criteria defining the serviceability limits are set out in Table 12.2.

The partial safety factors \( \gamma_e \) to be used with the characteristic loads for dead, imposed and wind loads obtained from CP 3, Chapter V, or other appropriate specification, are set out in Table 12.3 with notes on interpretation for ultimate and serviceability limit states. The combinations of loading to be taken are those which create the most severe conditions within the limits specified.

The partial safety factors for materials, \( \gamma_m \), for the limit states considered are given in Table 12.4 also with notes on their interpretation.

The Code for Structural Concrete has special provisions to satisfy the requirement that, when a building suffers accidental damage, the amount of damage caused shall not be inconsistent with the original cause. It would seem reasonable to apply this same approach to other structures where safety and avoidance of excessive damage are necessary considerations in the event of accidents. To achieve this in buildings, attention should be given to the choice of an appropriate plan form since this may have a large influence on the mode of collapse as a result of an accident. When it is necessary to consider the effects of excessive loads outside those normally likely to be experienced or the residual strength of a structure after accidental damage the value of \( \gamma_e \).
12.3.2 Characteristics of materials

The grades of concrete used for reinforced and prestressed concrete construction in the Structural Concrete Code are expressed as the characteristic strengths determined from 28-day tests on cubes; they are given in Table 12.5 with their properties relevant to design, including the increase in cube strength with age. No data are given for lightweight aggregate concrete since its properties are dependent on density in addition to strength as well as on the type of aggregate. The figures for flexural and indirect tensile strength refer to concretes made with smooth gravel aggregates; for crushed rock aggregates of tough texture, tensile strengths for the same grades of concrete would be somewhat higher. Generally, the minimum grade of concrete for reinforced concrete will be grade 25; there are, however, areas in Britain where the natural aggregates are not of high enough quality for concrete to meet this grade even though its cement content is sufficient to conform with requirements for durability. Unless there are special needs, grades stronger than grade 40 are unlikely to be used for reinforced concrete. When lightweight aggregate is used, a lower grade, grade 15, is acceptable for reinforced concrete but it is preferable to use a higher grade for the lightweight aggregates of higher strength. No upper limit needs to be set on the strength for prestressed concrete and higher grades than grade 60 may therefore be used, but only special circumstances would justify the much greater cost and need for control and supervision.

Calculations for conformity with ultimate and serviceability limit states require the strength and deformation characteristics for concrete to be defined in numerical terms. In particular, data are required on the relationships between stress and strain in compression under short-term loading and on creep and shrinkage when serviceability in the longer term is being considered. These aspects of behaviour are dealt with in section 12.4 and are simplified for design later in this section, but it must be recognized that there are substantial variations in the behaviour of concrete, depending on its constituent materials and environment, and that the values given for calculation should only be adopted if more reliable data are not available.

The strength properties of steel reinforcement and steel tendons are defined in British Standards which are summarized in Tables 12.6 and 12.7. For reinforcing bars of hot-rolled steel the characteristic strength is derived from the yield stress, but for cold-worked bars or wire reinforcement, it is derived from the 0.2% proof stress. The characteristic strength of tendons for prestressed concrete, however, is derived from their ultimate tensile strengths. In each case these are the relevant strengths for calculating ultimate strength for structural concrete members. Also in each case, the conformity with the specified characteristic strength is determined by ensuring that not more than two in forty consecutive results of tests made during the production of the steel falls below the specified value.

The design calculations for serviceability of structural concrete require information on the modulus of elasticity of steel.
The values adopted in the new code are: for reinforcement for all types of loading 200 kN/mm\(^2\), and for short-term loading for wire and strand of small diameter 200 kN/mm\(^2\) and for alloy bars and strand of large diameter 175 kN/mm\(^2\).

In prestressed concrete, considerations of serviceability require allowance not only for the effects of creep and shrinkage of the concrete but also relaxation of the tendons which may modify the prestress conditions substantially. Appropriate requirements are incorporated in the standards which therefore provide guidance on values for relaxation to be used in design.

The stress–strain characteristics for concrete and steel may be needed for calculations of the deformation of structural members under short-term loading or for assessing ultimate strength.

These are given in Figure 12.4 for concrete, in Figure 12.5 for reinforcement and in Figure 12.6 for tendons.

In interpreting these curves, the value of \( \gamma_m \) appropriate to the limit state being considered should be obtained from Table 12.4.

The creep and shrinkage characteristics of concrete are considered in section 12.4. Where it is necessary to calculate long-term deformation, the effects of creep can be conveniently allowed for by adopting an effective modulus:

Table 12.5 Grades and properties of structural concrete

| Grade* (characteristic strength 28 days) (N/mm\(^2\)) | Cube strength* (N/mm\(^2\)) at the age of: 7 days | 28 days | 2 mths | 3 mths | 6 mths | 1 year | Flexural strength at 28 days (N/mm\(^2\)) | Indirect tensile strength at 28 days (N/mm\(^2\)) | Modulus* of elasticity at 28 days (kN/mm\(^2\)) | Use* |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 15 | — | 15 | — | — | — | — | — | — | — | Reinforced concrete with lightweight aggregate |
| 20 | 13.5 | 20 | 22 | 23 | 24 | 25 | 3.3 | 1.5 | 24 | Reinforced concrete with natural dense aggregates |
| 25 | 16.5 | 25 | 27.5 | 29 | 30 | 31 | 3.7 | 2.1 | 26 | Prestressed concrete for post-tensioning, \( < 15 \) N/mm\(^2\) at transfer |
| 30 | 20 | 30 | 33 | 35 | 36 | 37 | 3.1 | 2.1 | 26 | Prestressed concrete for pretensioning, \( < 15 \) N/mm\(^2\) at transfer |
| 35 | 28 | 40 | 44 | 45.5 | 47.5 | 50 | 3.7 | 2.5 | 30 | |
| 40 | 36 | 50 | 54 | 55.5 | 57.5 | 60 | 4.2 | 2.8 | 32 | |
| 45 | 48 | — | — | — | — | — | — | — | — | |

*Recommendations in the Code of Practice for Structural Concrete.

Table 12.6 British Standards for reinforcing bars for concrete

<table>
<thead>
<tr>
<th>Type of steel*</th>
<th>Specified characteristic strength* (N/mm(^2))</th>
<th>Elongation at fracture (%)</th>
<th>Diam. for 180° bend test (no. of bar diam.)</th>
<th>Upper limit for:</th>
<th>Use*</th>
</tr>
</thead>
<tbody>
<tr>
<td>BS 4449: 1978</td>
<td>250</td>
<td>22</td>
<td>2</td>
<td>carbon content (%)</td>
<td>reinforced concrete with lightweight aggregate</td>
</tr>
<tr>
<td></td>
<td>460</td>
<td>12</td>
<td>3</td>
<td>sulphur content (%)</td>
<td></td>
</tr>
<tr>
<td>BS 4461: 1978</td>
<td>250</td>
<td>22</td>
<td>2</td>
<td>phosphorus content (%)</td>
<td></td>
</tr>
<tr>
<td>BS 4461: 1978</td>
<td>460</td>
<td>12</td>
<td>3</td>
<td>carbon content (%)</td>
<td></td>
</tr>
<tr>
<td>BS 4482: 1969</td>
<td>460</td>
<td>12</td>
<td>3</td>
<td>sulphur content (%)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>BS 4482: 1969</td>
<td>485</td>
<td>—</td>
<td>phosphorus content (%)</td>
<td></td>
</tr>
</tbody>
</table>

*Preferred sizes: BS 4449 | 8, 10, 12, 16, 20, 25, 32 and 40 mm diameter, if a smaller size is required then 6 mm, if larger 50 mm diameter |
| BS 4461 | BS 4482 – 5, 6, 8, 10 and 12 mm diameter. |

The characteristic strength is the yield stress below which not more than 5% of results should fall.

Notes: *Preferred sizes: BS 4449 | 8, 10, 12, 16, 20, 25, 32 and 40 mm diameter, if a smaller size is required then 6 mm, if larger 50 mm diameter |
| BS 4461 | BS 4482 – 5, 6, 8, 10 and 12 mm diameter. |

The characteristic strength is the yield stress below which not more than 5% of results should fall.
where \( E_{c,eff} \) is the short-term modulus of elasticity of concrete and \( \phi \) is the creep of concrete under a unit stress of 1 N/mm\(^2\).

The effects of shrinkage may be treated by assuming that the concrete contracts without a change in stress except for that caused by the effect of the change in strain on the stress in the steel. Some readjustment of strains then becomes necessary to balance the forces in the cross-section by assuming that the concrete is stressed under this strain in proportion to the effective modulus.

\[
E_{c,eff} = E_c(1 + \phi, E_c) \tag{12.4}
\]

### Table 12.7 British Standards for prestressing tendons for concrete

<table>
<thead>
<tr>
<th>Type of steel</th>
<th>Range of sizes available (dia. in mm)</th>
<th>Range of specified breaking load (kN)</th>
<th>Other information</th>
</tr>
</thead>
<tbody>
<tr>
<td>BS 5896:1980 High-tensile steel wire and strand for the prestressing of concrete</td>
<td>3-5</td>
<td>12.2-30.8</td>
<td>Relaxation: 1000 h</td>
</tr>
<tr>
<td>Cold-drawn steel wire in mill coils</td>
<td></td>
<td></td>
<td>60% breaking load</td>
</tr>
<tr>
<td>Stress relieved and may be crimped or indented and treated to reduce relaxation</td>
<td>4-7</td>
<td>21.0-64.3</td>
<td>Relaxation: 1000 h</td>
</tr>
<tr>
<td>Strand seven wire stress-relieved</td>
<td></td>
<td></td>
<td>Class 1</td>
</tr>
<tr>
<td>standard</td>
<td>9.3-15.2</td>
<td>92-232</td>
<td>60% b.l.</td>
</tr>
<tr>
<td>super</td>
<td>8.0-15.7</td>
<td>70-265</td>
<td>70% b.l.</td>
</tr>
<tr>
<td>drawn</td>
<td>12.7-18.0</td>
<td>209-380</td>
<td>80% b.l.</td>
</tr>
<tr>
<td>BS 4757:1971 Nineteen wire strand for the prestressing of concrete</td>
<td>as spun strand 25.4-31.8</td>
<td>659-979</td>
<td>Relaxation: 1000 h</td>
</tr>
<tr>
<td>normal relaxation strand</td>
<td>18.0</td>
<td>370</td>
<td>60% b.l.</td>
</tr>
<tr>
<td>low-relaxation strand</td>
<td>18.0</td>
<td>370</td>
<td>60% b.l.</td>
</tr>
<tr>
<td>BS 4486:1980 Hot-rolled and processed high-tensile alloy steel bars for the prestressing of concrete</td>
<td>hot-rolled 20-40</td>
<td>325-1300</td>
<td>Relaxation: 1000 h</td>
</tr>
<tr>
<td>processed</td>
<td>20-32</td>
<td>385-990</td>
<td>60% b.l.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>70% b.l.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>80% b.l.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1.5%</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>3.5%</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>6.0%</td>
</tr>
</tbody>
</table>

*Note: These values for relaxation at 1000 h apply in temperate climates and are those obtained at 20°C. When the prestressing steel is used at higher temperatures to prestress concrete, these percentage values should be increased. The increase may be as much as 2% for each 10°C increase in temperature.*

\[
E_{c,eff} = E_c(1 + \phi, E_c) \tag{12.4}
\]

**12.4 Analytical and design procedures**

**12.4.1 Objectives**

A recent trend in the approach to the initial design is to place much greater emphasis on the requirements for the durability of construction, since experience has shown that deterioration is a more serious cause of failure and of high maintenance costs than shortcomings in the structural calculations. It has therefore become more necessary to treat compliance with requirements for the quality of concrete, as placed in the construction and for...
the protection of embedded steel by adequate concrete cover or other means, as being at least as important as compliance with requirements derived from design calculations. Whilst it is not practicable to define requirements for durability in terms of a limit state, it is nevertheless an aspect of the overall design process requiring primary attention.

For somewhat similar reasons, more care is now given to the requirements for fire resistance and information is presented in the Code (Part 2) which can be used for an analytical approach as an alternative to satisfying somewhat arbitrary requirements for concrete cover in order to obtain the necessary fire grading.

Inevitably, structural calculations continue to be a major part of design. Whilst the principles of limit state design require all possible limit states to be examined in the design of a particular structure, part of the purpose of the Code is to provide guidance on containing the effort required in design within reasonable limits without overlooking significant features, i.e. limit states. In doing this, the Code relies on the experience of the designer to ensure that the interpretation is sensible in each instance.

12.4.2 General assumptions

For most forms of concrete construction, with the possible exception of slabs, it is most convenient at the present time to base all design on elastic analysis of the structural system. The analysis would then apply directly to the serviceability limit states of deflection and cracking and, with some limited redistribution of moments and shear forces to the ultimate state. For slabs, other than one-way spanning slabs, it will usually be more satisfactory to use yield line methods or the strip method for ultimate design. For most construction it will usually be preferable to determine conformity with the serviceability limit states by using the arbitrary rules given in the Code for span:depth ratios and reinforcement detailing instead of calculating deflections and widths of cracks.

The Code recommends procedures in Part 1 for the detailed design of beams, solid slabs supported by beams or walls, flat slabs, columns, walls, staircases and bases, which are given at some length, and generally apply to both reinforced concrete and prestressed concrete. More information dealing with ultimate strength, serviceability and deformation due to creep, shrinkage and temperature effects is contained in Part 2. In this relatively brief summary, it is only possible to cover the more basic recommendations, and detailed design therefore requires reference to the main documents.

Other methods of analysis and design, where experimental procedures are used to develop the theoretical approach or to determine performance, are acceptable but will normally only be employed for specially complex structures or where repetition justifies more refinement than is obtained by established methods of calculation. The assessment of stresses in the region of load concentrations or of holes in continuous construction may be determined by photoelastic procedures. Model testing using special materials or scaled concrete has found applications in developing design methods, e.g. in the design of concrete box-girder bridges and pressure vessels for nuclear power stations. In precast concrete construction particularly, the behaviour of joints can only be established by tests on full-scale assemblies. It may also be economic to derive the dimensions of precast components for mass production by testing successively refined prototypes to obtain the final form; this approach applies particularly in dealing with the requirements for fire resistance. The interpretation of test data for design requires the special care of experienced engineers since tests cannot embrace all the loads and load effects that may need to be sustained and the circumstances that exist in actual structures cannot necessarily be fully reproduced experimentally. When test results are applied, therefore, there is a need to show convincingly the
justification for departures from established practice, especially so, if these lead to less conservative design. If test data are applied in contexts for which they were not originally sought even more caution is necessary.

For the purpose of analysis, the Code offers three alternative methods for estimating beam and column stiffness: (1) the concrete section; (2) the gross section; and (3) the transformed section. The concrete section is the whole concrete section excluding the reinforcement, the gross section is the whole concrete section including the reinforcement allowing for the modular ratio, usually taken as 15, and the transformed section is the section of concrete in compression together with the reinforcement again allowing for the modular ratio. Generally, the concrete section is most convenient for use in design. For checking existing structures or for design in special circumstances, it would be more appropriate to use the transformed section for reinforced concrete; in construction where flexural cracking has occurred, however, the actual stiffnesses obtained by this assumption will be greater since the concrete exerts some tensile stiffening in the regions between cracks through bond with the reinforcement. The appropriate section for checking the design of existing or special prestressed concrete structures is the gross section since cracking does not usually occur with elastic deformation under service loads even for class-3 prestressed concrete.

12.4.3 Robustness

The Building Regulations require that all buildings of more than four storeys in height should be designed to resist accidental damage. The Regulations require that these buildings should be capable of sustaining removal of a structural member without excessive collapse resulting or should be able to withstand an internal pressure of 34 kN/m² without collapse.

The layout of the structure and its general form should not be sensitive to accidental damage whatever the cause. It is more realistic to interpret this as meaning that, in the event of an accident, the resulting damage should not be disproportionate to the magnitude of the cause. Where impact from vehicles is a possibility, buildings should be protected by barriers, such as bollards or earth banks. Greater margins should be allowed in design when the occupancy of a building may result in a greater than normal risk of accident, e.g. in flour mills and bonded stores.

Provisions envisaged in the Code go further in some respects in dealing with the effects of accidents than the Regulations require. The recommendations for robustness deal with both expected and accidental forms of loading, and include the following:

(1) All buildings should be so designed that all dead, imposed and wind loads are safely transmitted to the foundations.

(2) All buildings should be capable of withstanding a horizontal design ultimate load applied at roof and each floor level simultaneously corresponding to 1.5% of the dead-weight of the structure between the mid height of the storey below and mid height of the storey above for floors, and the surface for the roof. This, in effect, sets a lower limit for wind loading for the first two combinations of loading in Table 12.3.

(3) All buildings should be tied with effectively anchored and continuous reinforcement which is capable of withstanding the notional forces outlined in the following paragraphs. This reinforcement may consist of bars provided to resist stresses due to normal loads, which may be ignored for this purpose, and it may be assumed to be stressed up to its characteristic strength.

Buildings of four storeys or less require tying horizontally in two directions approximately at right angles with internal ties and peripheral ties.

Internal ties, which should be anchored to the peripheral ties and should be accommodated in the beams or slabs, should be capable of resisting a notional force of: \( \frac{(g_u + q_u) \times 7.5(\rho / 5)F_c}{1F_cF_cF_cF_c} \) kN/m or 1F_c kN/m width, whichever is the greater, where \((g_u + q_u)\) is the sum of the average characteristic dead and imposed load in kilonewtons per square metre, and \(\rho\) is the greater of the distances in metres between the centres of supporting columns, frames or walls of any adjacent floor spans parallel to the tie. \( F_c \) is the lesser of \( (20 + 4n) \) or 60, \( n \) being the number of storeys. The spacing of the ties should not be more than 1.5\( F_c \).

Peripheral ties should be provided at each floor and roof level and be capable of withstanding a notional force of not less than 1F_c kN. They should be located within 1.2 m of the edge of the building.

Horizontal ties to external columns and walls should be provided for each external column, in two directions for corner columns, and for each metre length of external wall at each floor and at roof level. The notional force considered should be the greater of the following: 2F_c (or \((F_c / 2.5) \times \) ceiling height \( m \)) kN, or 3% of the total design ultimate load carried by the column or wall at that level.

Buildings of five storeys or more require additional provision for robustness, which usually will be met by the inclusion of vertical ties in all walls and columns. These should be designed for a notional force corresponding to the maximum design ultimate dead and imposed load received by the column or wall from any one storey or roof.

(4) Where there are key elements in a building design, the failure of which might cause extensive collapse, their design should take their importance into account if their use cannot be avoided. Where vertical ties cannot be provided (see (3) above), provision should be made for bridging by the structure above in the event of their removal.

The purpose of these recommendations is to ensure that all structures are insensitive to damage from localized disturbances. It is therefore important in providing ties, for bridging or any other action, that the arrangements are sound engineering.

12.4.4 Beams and slabs

The effective span (l) of beams or slabs, which are simply supported, is taken as either the distance between the centres of bearings or the clear distance between supports plus the effective depth, whichever is the smaller. For continuous members, however, the effective span is the distance between the centres of the supports. Whilst for a cantilever which forms part of a continuous beam or slab, it is to the centre of the support, but for an isolated cantilever the effective span is to the centre of the support plus half the effective depth.

The effective width of a flange to a T-beam may be taken as the smaller of the width of the web plus one-fifth of the distance between points of zero moment or the actual width. Similarly, the effective width of flange for an L-beam is taken as the smaller of the width of the web plus one-tenth of the distance between points of zero moment; for continuous beams the distance between points of zero moment may be assumed to be 0.7L.

The lateral stability of beams may need attention, usually by providing for adequate restraints and stiffness. The limits between lateral restraints for simply supported beams or continuous beams should not exceed 60b, or 250b/d, where d is the effective depth and b, the breadth of the compression face midway between supports. For cantilevers restrained only at the support, its length should not exceed 25b or 100b^2/d.
The following loading conditions should usually be considered in the design of continuous beams and slabs: (1) the design ultimate load of \(1.4G_1 + 1.6G_2\) on all spans; and (2) the design ultimate load as (1) on alternate spans with \(1G_2\) on intermediate spans. When moments at sections are determined by elastic analysis, the maximum moment may be reduced by redistribution provided that the calculated depth of the neutral axis is not greater than \(\beta_d = 0.4d\) where \(d\) is the effective depth and \(\beta\) is:

\[
\text{moment at the section after redistribution} \over \text{moment at the section before redistribution} > 1
\]

and that the resistance moment at any section is not less than 70% of the moment at that section from elastic analysis.

12.4.5 Continuous and two-way solid slabs

Slabs which are continuous in extent in one or two directions may be designed as simply supported, provided that continuous ties that may be required for overall stability of the structure are incorporated in the construction. In such cases, cracking will develop in the top surface of the floors at their supports and some provision will be needed for dealing with this in applying floor finishes.

Where slabs are required to span in one direction over a number of supports, they should be designed for moments and shears, calculated in similar manner to those for continuous beams.

If solid slabs are required to span in two directions, yield line analysis or the strip method of design may be used. British Standard 8110, however, gives simple methods for the design of rectangular slabs for simply supported two-way panels and two-way continuous or restrained slabs.

12.4.6 Flat slab construction

Flat slab construction usually consists of a slab which spans between columns in two directions without supporting beams. Drops may be provided over the columns by increasing the depths of the slab and sometimes the column heads may be flared to reduce shear stresses. The slabs may be solid or ribbed in two directions.

British Standard 8110 offers a method of design but does not exclude the use of other methods such as finite element analysis or other procedures. In the BS method, it is assumed that the slab is supported by a rectangular grid of columns in which the ratio of the longer spans to the shorter spans is not greater than 2. The slabs are divided longitudinally and transversely into column strips and middle strips; the columns and column trips are designed as frames spanning in each direction. Each frame is then analysed elastically; a simplified method is given for the situation where the structure is braced against lateral loading and the column grid has a regular layout. Procedures are given for determining the widths of column strips and for the treatment of drops.

12.4.7 Frames

The loads to be adopted in the design of frames with their factors have already been given in Table 12.3. When considering the ultimate limit state, the forces, shears and moments calculated for design should be the worst combinations of loading regarded as feasible. British Standard 8110 gives some simplified procedures, which may be used for a number of common forms of construction. These analyse frameworks by breaking them down into subframes and make some provision for redistribution of moments. Two types of frame are dealt with – the no-sway frame, in which bracing, such as shear walls and lift or stair wells, are used to restrain sidesway, and sway frames, in which the frame itself provides the lateral restraint. For the latter, the amount of moment redistribution allowed is restricted to 70% of the moment at that section from elastic analysis.

12.4.8 Columns and walls

The determination of the loads and moments on columns is given in BS 8110 to which reference should be made for details. A column is described as slender when the ratio of the effective length to the corresponding breadth with respect to either axis is greater than 12 (10 for lightweight aggregate concrete); if the ratio is less than 12, the column is said to be short. The effective length is dependent on the length of the column and on the degree of restraint at the top and bottom connections with the structure. Generally, the slenderness ratio for a column should not be greater than 60. A distinction is made between braced and unbraced columns; a column being described as braced when the lateral stability of the whole structure is ensured by providing walls or bracing to resist all horizontal forces.

The procedures for dealing with walls in BS 8110 have much in common with those for columns. A concrete component is defined as a wall when the greater of the lateral dimensions is at least 4 times the smaller. For plain walls, however, the ratio may be less (since columns without reinforcement are not recognized) and reduction factors are then applied. To be described as a reinforced wall, the area of vertical reinforcement should not be less than 0.4% of the cross-sectional area of concrete; if the amount of reinforcement is less, the wall should be designed as a plain wall. Some reinforcement may be required in plain walls to control cracking. A stocky wall is one in which the ratio of effective length to thickness does not exceed 12 (10 for lightweight concrete), otherwise the wall should be treated as being slender. As for columns, the effective length is dependent on the height and conditions of end-restraint. Methods for calculating the loads and moments on walls are also given in some detail in BS 8110 to which reference should be made.

Provided the recommendations in the British Standards are followed, the deflections of columns and walls should not be excessive.

12.5 Reinforced concrete

12.5.1 General

In the design of reinforced concrete to meet the requirements of the Code, BS 8110, it will usually be most appropriate to consider the ultimate limit state first and then check the design against the requirements for cracking and deflection. This might be inappropriate in exceptional circumstances, e.g. where steels of characteristic strengths in excess of 500 N/mm² are being used or where spans were exceptionally long: in these cases cracking or deflection might govern design. In the sections that follow, design will be treated on the assumptions that normal conditions obtain. For these the Code gives simplified treatments for dealing with both cracking and deflection. It also gives methods more suited to the exceptional cases for which reference to the Code should be made.

12.5.2 Beams

12.5.2.1 Bending

Ultimate resistance in bending is calculated by assuming that: