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1

Introduction to design and properties of reinforced concrete

Chapter introduction

Structural design may be considered as a series of interrelated and overlapping stages. In their simplest forms these consist of:

- *Conceptual design* in which a range of potential structural forms and materials will be considered.
- *Preliminary design* which will typically involve simple and approximate hand calculations to assess the viability of a range of alternative conceptual solutions.
- *Detailed design* to include full analysis and calculations for the selected scheme(s).

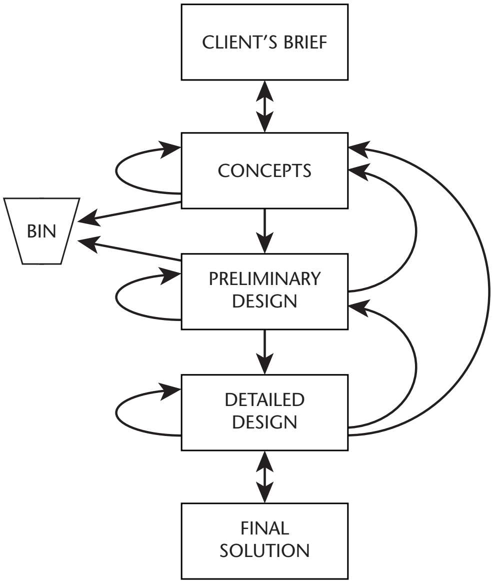
Reinforced concrete is a strong durable building material which can be formed into many varied shapes and sizes, ranging from a simple rectangular beam or column to a slender curved dome or shell. Its utility and versatility are achieved by combining the best features of concrete and steel.

This chapter can present only a brief introduction to the major issues to be considered in design, and the basic properties of concrete and its steel reinforcement. For a more comprehensive study it is recommended that reference should be made to the specialised texts and websites listed in Further Reading at the end of the book.

1.1 Design processes

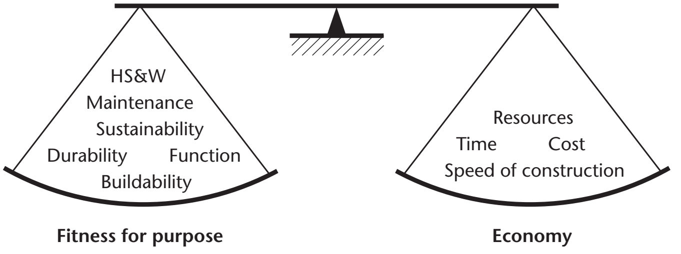
The three basic stages identified above are not linear in nature, as illustrated in figure 1.1, but involve a series of iterations in which alternatives are compared, modified, and refined to produce a workable solution taking account of requirements and constraints.

Figure 1.1
The iterative nature of design



This will usually require assumptions, judgements and decisions to be made on the basis of available, often incomplete, information. The objective is to achieve a balance between fitness for purpose, including function and durability, and economy, encompassing finance, time and resources with due consideration of environmental and sustainability together with Health and Safety considerations. This is illustrated in figure 1.2.

Figure 1.2
Balancing design criteria



The ease of construction (buildability) with available manpower, skills and equipment will be important, together with provision for future maintenance.

1.1.1 Stages of design

Three basic design stages have been identified above: conceptual, preliminary and detailed.

Conceptual design

This is the first stage of the design process and requires many considerations beyond the calculations associated with the later stages using the principles and procedures which form the basis of this book.

In particular, it is first necessary to fully understand the requirements of the client and the design brief. In addition to the aspects indicated above, any special requirements associated with the site (including ground conditions and access), usage (including acoustic, thermal, or radiation insulation and dynamic performance requirements), and relevant Codes and Regulations must be identified. Stability both during construction and in service must be considered, including the concept of braced or unbraced structural form, as well the need for robustness under accidental loads including explosion or vehicle impact. Aesthetic, sustainability and environmental issues must also be taken into account and the latter two aspects are considered more fully below.

The process may involve input from a range of professionals including architects, geotechnical engineers, services engineers, and quantity surveyors whilst it is often very important that potential contractors are involved at an early stage for major projects. Good communications between team members is a key feature of a successful project.

The first step may involve brainstorming sessions to identify alternatives of layout, structural form and materials on the basis of the requirements and constraints. These will be supported by preliminary calculations based on initial estimates of dimensions and loads to establish the structural feasibility of particular concepts. Approximate methods are used and understanding will be aided by the use of sketches which are approximately to scale.

Preliminary design

This initial calculation stage, as outlined above, will help establish the viability of potential conceptual solutions, and enable their development and refinement. Initial procedures will usually be based on hand calculations which are considered below, and may lead into subsequent computer analysis for complex structures. Bespoke software such as the *Concrete Centre's* CONCEPT spreadsheet suite may be used to quickly evaluate options and to select the final structure for detailed design.

Detailed design

It is at this stage that a preferred potential solution will be fully analysed using computer packages as appropriate and refined to produce detailed calculations, drawings and other documentation necessary for costing and construction.

1.1.2 Hand calculations

These will typically involve simplified analysis, with elements assumed to be simply supported or fully fixed, and use design aids such as tables and charts that are provided in this book. Loading will generally be considered as uniformly distributed or point, and approximate member sizing will be based on span–depth ratios or load–span tables. These will be used to assess and compare the viability of different proposed schemes including the feasibility of differing foundation solutions.

Hand calculations will also be important as a check on computer generated design solutions when they may also include checks that, for example, the sum of reactions is equal to the applied load both at element and overall structure levels. Other useful checks include confirming that the increase in column loads at a floor level matches the load on the floor area supported and that the sum of (support + span) moments in a beam total $wl^2/8$ when carrying a uniformly distributed load (UDL) over the whole span.

Throughout this book are worked examples that emphasise the use of hand calculations such that the reader can gain a sound understanding of the fundamental principles of reinforced and prestressed concrete design and the ability to carry out those routine and non-routine design calculations that would be expected of a competent engineer.

1.1.3 Role of computers

In modern design offices the use of computer software is an accepted and intrinsic part of the design process and there are, indeed, many sophisticated packages and spreadsheet systems that are used to rapidly explore conceptual design options and to finalise the detailed design. Such software is often linked to 2- and 3-dimensional modelling packages that aid visualisation and allow the electronic production of accurate construction and reinforcement detailing drawings. Through common database systems they can also facilitate the sharing of information between all parties involved in the design and this may include engineers, architects and planners working in different offices in different parts of the world who can access, check and share each others' work.

The ability to effectively use such software is a high-level skill in itself but it is generally recognised that it is unsafe practice for engineers to use sophisticated software if they themselves do not understand the fundamental engineering principles on which the software is based and cannot recognise, check and challenge computer output that may not be correct. For this reason this text focuses on the understanding and application of the engineering principles that underpin the design of reinforced and prestressed concrete such that, through gaining this understanding, the reader can develop as a competent engineer who may or may not subsequently use sophisticated commercial software in their design work.

When using any form of computerised design system a competent designer will therefore look for and check the following to ensure that reliable results are achieved:

- *Suitability of software* to provide the required results for the particular problem must be established, including the effects of torsion and structural form. There must be compliance with appropriate Codes of Practice and any assumptions relating to the detailed design procedures must be identified and approved. It may also be necessary to check the relevance of stiffness values to ultimate or serviceability limit states.
- *Input data* must be accurate including dimensions, materials properties, applied loadings and member restraints.
- *Verification of output* involving qualitative judgements such as:
  - are load paths sensible?
  - is the deflected shape sensible?
  - is there symmetry of output (e.g. reactions and moments) where there should be?
  - based on experience, does the quantity and distribution of reinforcement, the size of members and so on seem sensible?

  This will be supported by simple quantitative hand calculation checks as discussed in section 1.1.2.

- *Quality Systems* that ensure calculations and designs are third-party checked and that all parties who have access to the design data-base are aware of the consequences of any design changes that have been made by any party.

1.1.4 Sustainability and environmental considerations.

Sustainable development is widely quoted as that which 'meets the needs of the present without compromising the ability of future generations to meet their own needs'. Engineers have a responsibility to contribute to the sustainability agenda by promoting sustainable methods of construction and, indeed, the Institution of Civil Engineers (ICE) promotes the concept that engineers are 'at the heart of society, delivering sustainable development through knowledge, skills and professional expertise.' It is, therefore, a responsibility of the designer to embrace design methodologies that promote sustainable methods of construction and the use of sustainable materials.

The choice of structural form and appropriate construction materials will be a decision that will balance many factors. However, choosing reinforced concrete construction will contribute significantly to the sustainability agenda. Concrete is generally locally sourced, requiring short travel distances to site, and can utilise waste by-product materials of other industries such as fly ash. As a material it has a high thermal mass and reinforced concrete walls and floors can absorb, retain and release heat in domestic and commercial buildings, leading to a reduced requirement for heating and ventilating systems with reduced energy demands and associated CO_2 emissions.

Concrete is also a robust material, requiring little maintenance and providing good fire, flood and sound resistance. The mass of reinforced concrete can ensure that vibration effects are minimised; essential in buildings where there may be equipment that is sensitive to small movements. Concrete buildings can also be readily adapted for alternative use other than that for which originally intended and at the end of the structure's life the concrete can be crushed and recycled for other usages. Further information on sustainable concrete construction can be found in references 27 and 36.

1.1.5 Health, Safety and Welfare

The construction industry is, by its very nature, a dangerous place to work and it is generally recognised that each year there are many fatalities and major injuries that could be reduced in number and impact if appropriate action were taken by all parties to the construction process.

As a result, in the UK and in many other parts of the world, issues of Health, Safety and Welfare (HS&W) are given a high priority both at the Design and Construction stages and throughout the in-service life of the structure. Indeed, the legal requirement to address such issues is defined in legislation which in the UK is partially driven by a European Directive which specifies minimum health and safety requirements for temporary or mobile construction sites.

The consequence of this Directive has been the introduction in 2007 of the Construction Design and Management (CDM) regulations. The regulations are supported by an *Approved Code of Practice* (*ACoP*) which has a special legal status and gives advice to those involved in construction work such that, by following the advice, compliance with the law may be assured.

The aim of CDM (2007) is to improve standards of HS&W through simplification and clarification of regulations; improved co-operation and co-ordination of all parties to the construction process and the planning and managing of work. The regulations define and articulate the responsibility of all parties including the Client, the Designer, the Principal Contractor, other contractors, workers and the CDM coordinators.

The regulations also clearly define the responsibilities of the designer and, although this text focuses on the principles of design of reinforced and prestressed concrete elements, the reader should be aware that when engaged in a design role they should be not only applying sound engineering judgements based on a knowledge of the principles of design but should be designing in a way that is compliant with the CDM regulations.

A clear requirement of the regulations is that the designer should eliminate hazards and reduce risks during design. The regulations are explicit that foreseeable risks should be avoided although the designer cannot be held responsible for unforeseeable risks. Hence the designer must carry out a risk assessment through, for example, inspecting the site to identify hazards, seeking relevant H&S information from the client, such as the presence of buried contaminated materials, and co-ordinating their work with others involved in the project to consider the H&S implications of the chosen design and the consequential buildability issues.

Such buildability issues may include consideration of temporary works and detailed construction procedures such as sequence, pouring rates and prop removal, taking into account access and equipment requirements. They must also take into account the safety of the construction workforce which may influence, for example, the choice of foundation types where a deep foundation may necessitate work in potentially hazardous deep excavations whereas a shallow or piled foundation would be less hazardous to the workforce. The final choice of foundation will, however, be a judgement taking into account possible engineering solutions and cost together with buildability issues linked to H&S considerations. Consideration must be given to H&S issues arising from construction, maintenance and demolition and from the intended usage of the building, excepting future usage that cannot reasonably be anticipated.

It is also a designer's responsibility to provide information about any identified but remaining risks and communicate with all parties who may be affected by such risks. This must be communicated in an appropriate way, often in the form of annotation on the designer's drawings or in a written Designer's Risk Assessment. While the designer is not expected to specify construction methods, when a design is based on the assumption of, for example, a particular sequence of construction then this information must be advised to the Contractor or a serious H&S issue may arise.

When a project is deemed to be *notifiable* the designer is also required to provide information for the project's *Health & Safety File*, a statutory document held and maintained by the Client in which health and safety information is recorded and kept for future reference when a project is completed.

1.2 Composite action

Concrete and steel have widely differing properties, some of which are listed below:

	Concrete	**Steel**
Strength in tension	poor	good
Strength in compression	good	good, but slender bars will buckle
Strength in shear	fair	good
Durability	good	corrodes if unprotected
Fire resistance	good	poor – suffers rapid loss of strength at high temperatures

It can be seen from this list that the materials are more or less complementary. Thus, when they are combined, the steel is able to provide the tensile strength and probably some of the shear strength while the concrete, strong in compression, protects the steel to give it durability and fire resistance.

The tensile strength of concrete is only about 10 per cent of the compressive strength. Because of this, nearly all reinforced concrete structures are designed on the assumption that the concrete does not resist any tensile forces. Reinforcement is designed to carry these tensile forces, which are transferred by bond between the interface of the two materials. If this bond is not adequate, the reinforcing bars will just slip within the concrete and there will not be a composite action. Thus members should be detailed so that the concrete can be well compacted around the reinforcement during construction. In addition, bars are normally ribbed so that there is an extra mechanical grip.

In the analysis and design of the composite reinforced concrete section, it is assumed that there is a perfect bond, so that the strain in the reinforcement is identical to the strain in the adjacent concrete. This ensures that there is what is known as 'compatibility of strains' across the cross-section of the member.

The coefficients of thermal expansion for steel and for concrete are of the order of 10×10^{-6} per °C and $7\text{–}12 \times 10^{-6}$ per °C respectively. These values are sufficiently close that problems with bond seldom arise from differential expansion between the two materials over normal temperature ranges.

Figure 1.3 illustrates the behaviour of a simply supported beam subjected to bending and shows the position of steel reinforcement to resist the tensile forces, while the compression forces in the top of the beam are carried by the concrete.

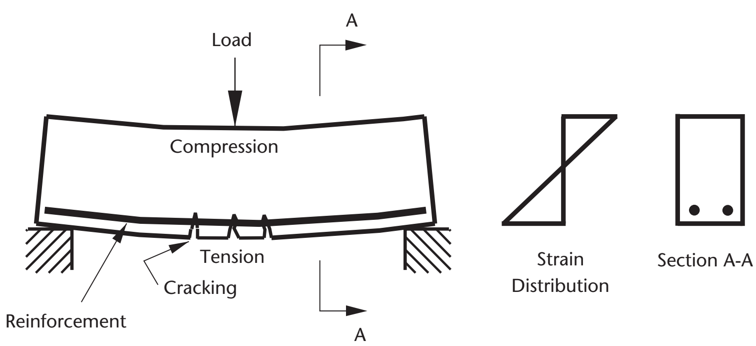


Figure 1.3
Composite action

Wherever tension occurs it is likely that cracking of the concrete will take place. This cracking, however, does not detract from the safety of the structure provided there is good reinforcement bonding to ensure that the cracks are restrained from opening so that the embedded steel continues to be protected from corrosion.

When the compressive or shearing forces exceed the strength of the concrete, then steel reinforcement must again be provided, but in these cases it is only required to supplement the load-carrying capacity of the concrete. For example, compression reinforcement is generally required in a column, where it takes the form of vertical bars spaced near the perimeter. To prevent these bars buckling, steel binders are used to assist the restraint provided by the surrounding concrete.

1.3 Stress–strain relations

The loads on a structure cause distortion of its members with resulting stresses and strains in the concrete and the steel reinforcement. To carry out the analysis and design of a member it is necessary to have a knowledge of the relationship between these stresses and strains. This knowledge is particularly important when dealing with reinforced concrete which is a composite material; for in this case the analysis of the stresses on a cross-section of a member must consider the equilibrium of the forces in the concrete and steel, and also the compatibility of the strains across the cross-section.

1.3.1 Concrete

Concrete is a very variable material, having a wide range of strengths and stress–strain curves. A typical curve for concrete in compression is shown in figure 1.4. As the load is applied, the ratio between the stresses and strains is approximately linear at first and the concrete behaves almost as an elastic material with virtually full recovery of displacement if the load is removed. Eventually, the curve is no longer linear and the concrete behaves more and more as a plastic material. If the load were removed during the plastic range the recovery would no longer be complete and a permanent deformation would remain. The ultimate strain for most structural concretes tends to be a constant value of approximately 0.0035, although this is likely to reduce for concretes with cube strengths above about 60 N/mm^2. BS EN1992 'Design of Concrete Structures' – commonly known as Eurocode 2 (or EC2) recommends values for use in such cases. The precise shape of the stress–strain curve is very dependent on the length of time the load is applied, a factor which will be further discussed in section 1.5 on creep. Figure 1.4 is typical for a short-term loading.

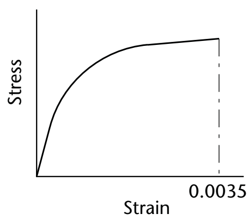


Figure 1.4
Stress–strain curve for concrete in compression

Concrete generally increases its strength with age. This characteristic is illustrated by the graph in figure 1.5 which shows how the increase is rapid at first, becoming more gradual later. The precise relationship will depend upon the type of cement used. That shown is for the typical variation of an adequately cured concrete made with commonly used class 42.5 N Portland Cement. Some codes of practice allow the concrete strength used in design to be varied according to the age of the concrete when it supports the

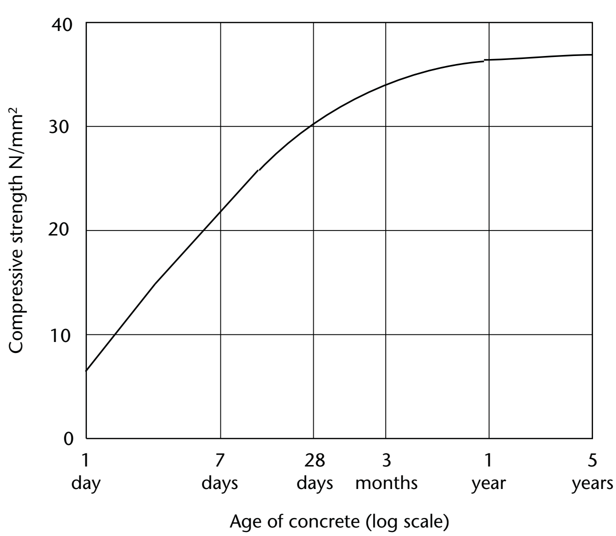


Figure 1.5
Increase of concrete strength with age. Typical curve for a concrete made with a class 42.5 N Portland cement with a 28 day cube compressive strength of 30 N/mm^2

design load. European Codes, however, do not permit the use of strengths greater than the 28-day value in calculations, but the modulus of elasticity may be modified to account for age as shown later.

In the United Kingdom, compressive stress has traditionally been measured and expressed in terms of 150 mm cube crushing strength at an age of 28 days. Most other countries use 150 mm diameter cylinders which are 300 mm long. For normal strength concretes, the cylinder strength is, on average, about 0.8 × the cube strength. All design calculations to EC2 are based on the characteristic cylinder strength f_{ck} as defined in section 2.2.1. Cube strengths may however be used for compliance purposes, with the characteristic strength identified as $f_{ck,\,cube}$.

Concretes will normally be specified in terms of these 28-day characteristic strengths, for example strength class C35/45 concrete has a characteristic cylinder strength of 35 N/mm^2 and a characteristic cube strength of 45 N/mm^2. It will be noted that there is some 'rounding off' in these values, which are usually quoted in multiples of 5 N/mm^2 for cube strength. Concretes made with lightweight aggregates are identified by the prefix LC.

Modulus of elasticity of concrete

It is seen from the stress–strain curve for concrete that although elastic behaviour may be assumed for stresses below about one-third of the ultimate compressive strength, this relationship is not truly linear. Consequently it is necessary to define precisely what value is to be taken as the modulus of elasticity.

$$E = \frac{\text{stress}}{\text{strain}}$$

A number of alternative definitions exist, but the most commonly adopted is $E = E_{cm}$ where E_{cm} is known as the *secant* or *static modulus*. This is measured for a particular concrete by means of a static test in which a cylinder is loaded to just above one-third of the corresponding mean control cube stress $f_{cm,\,cube}$, or 0.4 mean cylinder strength, and then cycled back to zero stress. This removes the effect of initial 'bedding-in' and minor stress redistributions in the concrete under load. The load is reapplied and the behaviour will then be almost linear; the average slope of the line up to the specified stress is taken as the value for E_{cm}. The test is described in detail in BS 1881 and the result is generally known as the *secant modulus of elasticity*.

The *dynamic modulus of elasticity*, E_d, is sometimes referred to since this is much easier to measure in the laboratory and there is a fairly well-defined relationship between E_{cm} and E_d. The standard test is based on determining the resonant frequency of a prism specimen and is also described in BS 1881. It is also possible to obtain a good estimate of E_d from ultrasonic measuring techniques, which may sometimes be used on site to assess the concrete in an actual structure. The standard test for E_d is on an unstressed specimen. It can be seen from figure 1.6 that the value obtained represents the slope of the tangent at zero stress and E_d is therefore higher than E_{cm}. The relationship between the two moduli is often taken as

$$\text{Secant modulus } E_{cm} = (1.25E_d - 19) \text{ kN/mm}^2$$

This equation is sufficiently accurate for normal design purposes.

The actual value of E for a concrete depends on many factors related to the mix, but a general relationship is considered to exist between the modulus of elasticity and the compressive strength.

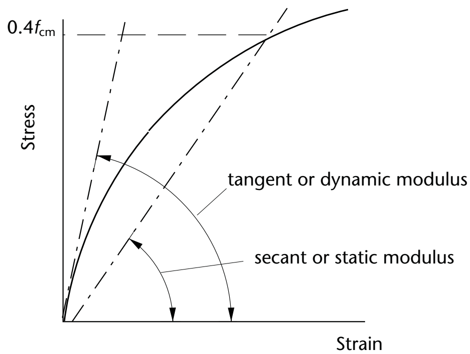


Figure 1.6
Moduli of elasticity of concrete

Table 1.1 Short-term modulus of elasticity of normal-weight gravel concrete

28 day characteristic strength (N/mm^2) $f_{ck}/f_{ck,\,cube}$ *(cylinder/cube)*	*Static (secant) modulus* E_{cm} *(kN/mm^2) Mean*
20/25	30
25/30	31
30/37	33
35/45	34
40/50	35
45/55	36
50/60	37
60/75	39
70/85	41
80/95	42
90/105	44

Typical values of E_{cm} for various concrete classes using gravel aggregates which are suitable for design are shown in table 1.1. For limestone aggregates these values should be reduced by a factor of 0.9, or for basalt increased by a factor of 1.2. The magnitude of the modulus of elasticity is required when investigating the deflection and cracking of a structure. When considering short-term effects, member stiffness will be based on the static modulus E_{cm} defined above. If long-term effects are being considered, it can be shown that the effect of creep can be represented by modifying the value of E_{cm} to an effective value $E_{c,\,eff}$, and this is discussed in section 6.3.2.

The elastic modulus at an age other than 28 days may be estimated from this table by using the anticipated strength value at that age. If a typical value of Poisson's ratio is needed, this should be taken as 0.2 for regions which are not subject to tension cracking.

1.3.2 Steel

Figure 1.7 shows typical stress–strain curves for (a) hot rolled high yield steel, and (b) cold-worked high yield steel. Mild steel behaves as an elastic material, with the strain proportional to the stress up to the yield, at which point there is a sudden increase in strain with no change in stress. After the yield point, this becomes a plastic material and the strain increases rapidly up to the ultimate value. High yield steel, which is most commonly used for reinforcement, may behave in a similar manner or may, on the other

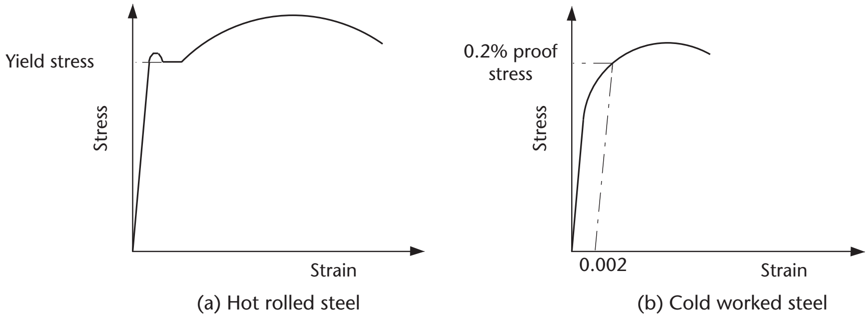


Figure 1.7
Stress–strain curves for high yield reinforcing steel

hand, not have such a definite yield point but may show a more gradual change from elastic to plastic behaviour and reduced ductility depending on the manufacturing process. All materials have a similar slope of the elastic region with elastic modulus $E_s = 200$ kN/mm^2 approximately.

The specified strength used in design is based on either the yield stress or a specified proof stress. A 0.2 per cent proof stress is defined in figure 1.7 by the broken line drawn parallel to the linear part of the stress–strain curve.

Removal of the load within the plastic range would result in the stress–strain diagram following a line approximately parallel to the loading portion – see line BC in figure 1.8. The steel will be left with a permanent strain AC, which is known as 'slip'. If the steel is again loaded, the stress–strain diagram will follow the unloading curve until it almost reaches the original stress at B and then it will curve in the direction of the first loading. Thus, the proportional limit for the second loading is higher than for the initial loading. This action is referred to as 'strain hardening' or 'work hardening'.

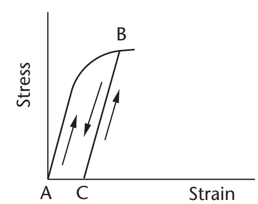


Figure 1.8
Strain hardening

The load deformation of the steel is also dependent on the length of time the load is applied. Under a constant stress the strains will gradually increase – this phenomenon is known as 'creep' or 'relaxation'. The amount of creep that takes place over a period of time depends on the grade of steel and the magnitude of the stress. Creep of the steel is of little significance in normal reinforced concrete work, but it is an important factor in prestressed concrete where the prestressing steel is very highly stressed.

1.4 Shrinkage and thermal movement

As concrete hardens there is a reduction in volume. This shrinkage is liable to cause cracking of the concrete, but it also has the beneficial effect of strengthening the bond between the concrete and the steel reinforcement. Shrinkage begins to take place as soon as the concrete is mixed, and is caused initially by the absorption of the water by the concrete and the aggregate. Further shrinkage is caused by evaporation of the water which rises to the concrete surface. During the setting process the hydration of the cement causes a great deal of heat to be generated, and as the concrete cools, further shrinkage takes place as a result of thermal contraction. Even after the concrete has hardened, shrinkage continues as drying out persists over many months, and any subsequent wetting and drying can also cause swelling and shrinkage. Thermal shrinkage may be reduced by restricting the temperature rise during hydration, which may be achieved by the following procedures:

1. Use a mix design with a low cement content or suitable cement replacement e.g. fly ash (pulverised fuel ash) or ground granulated blast furnace slag.

2. Avoid rapid hardening and finely ground cement if possible.
3. Keep aggregates and mixing water cool.
4. Use steel shuttering and cool with a water spray.
5. Strike the shuttering early to allow the heat of hydration to dissipate.

A low water–cement ratio will help to reduce drying shrinkage by keeping to a minimum the volume of moisture that can be lost.

If the change in volume of the concrete is allowed to take place freely and without restraint, there will be no stress change within the concrete. Restraint of the shrinkage, on the other hand, will cause tensile strains and stresses. The restraint may be caused externally by fixity with adjoining members or friction against an earth surface, and internally by the action of the steel reinforcement. For a long wall or floor slab, the restraint from adjoining concrete may be reduced by constructing successive bays instead of alternate bays. This allows the free end of every bay to contract before the next bay is cast.

Day-to-day thermal expansion of the concrete can be greater than the movements caused by shrinkage. Thermal stresses and strains may be controlled by the correct positioning of movement or expansion joints in a structure. For example, the joints should be placed at an abrupt change in cross-section and they should, in general, pass completely through the structure in one plane.

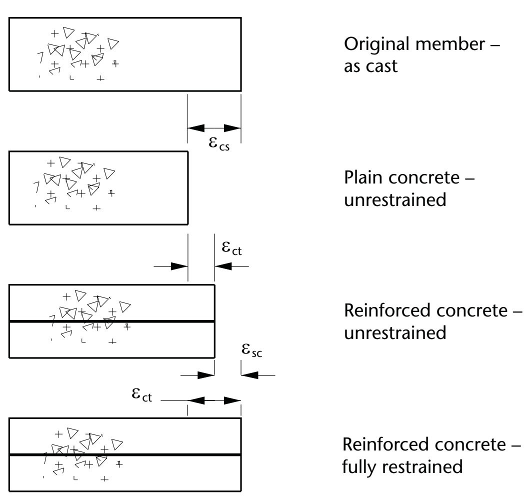
When the tensile stresses caused by shrinkage or thermal movement exceed the strength of the concrete, cracking will occur. To control the crack widths, steel reinforcement must be provided close to the concrete surface; the codes of practice specify minimum quantities of reinforcement in a member for this purpose.

Calculation of stresses induced by shrinkage

(a) Shrinkage restrained by the reinforcement

The shrinkage stresses caused by reinforcement in an otherwise unrestrained member may be calculated quite simply. The member shown in figure 1.9 has a free shrinkage strain of ε_{cs} if made of plain concrete, but this overall movement is reduced by the inclusion of reinforcement, giving a compressive strain ε_{sc} in the steel and causing an effective tensile strain ε_{ct} the concrete.

Figure 1.9
Shrinkage strains



Thus

$$\varepsilon_{cs} = \varepsilon_{ct} + \varepsilon_{sc} = \frac{f_{ct}}{E_{cm}} + \frac{f_{sc}}{E_s} \tag{1.1}$$

where f_{ct} is the tensile stress in concrete area A_c and f_{sc} is the compressive stress in steel area A_s

Equating forces in the concrete and steel for equilibrium gives

$$A_c f_{ct} = A_s f_{sc} \tag{1.2}$$

therefore

$$f_{ct} = \frac{A_s}{A_c} f_{sc}$$

Substituting for f_{ct} in equation 1.1

$$\varepsilon_{cs} = f_{sc}\left(\frac{A_s}{A_c E_{cm}} + \frac{1}{E_s}\right)$$

Thus if $\alpha_e = \dfrac{E_s}{E_{cm}}$

$$\begin{aligned}\varepsilon_{cs} &= f_{sc}\left(\frac{\alpha_e A_s}{A_c E_s} + \frac{1}{E_s}\right)\\ &= \frac{f_{sc}}{E_s}\left(\frac{\alpha_e A_s}{A_c} + 1\right)\end{aligned}$$

Therefore steel stress

$$f_{sc} = \frac{\varepsilon_{cs} E_s}{1 + \dfrac{\alpha_e A_s}{A_c}} \tag{1.3}$$

EXAMPLE 1.1

Calculation of shrinkage stresses in concrete that is restrained by reinforcement only

A member contains 1.0 per cent reinforcement, and the free shrinkage strain ε_{cs} of the concrete is 200×10^{-6}. For steel, $E_s = 200$ kN/mm^2 and for concrete $E_{cm} = 15$ kN/mm^2. Hence from equation 1.3:

$$\begin{aligned}\text{stress in reinforcement } f_{sc} &= \frac{\varepsilon_{cs} E_s}{1 + \alpha_e \dfrac{A_s}{A_c}}\\ &= \frac{200 \times 10^{-6} \times 200 \times 10^3}{1 + \dfrac{200}{15} \times 0.01}\\ &= 35.3 \text{ N/mm}^2 \text{ compression}\end{aligned}$$

$$\begin{aligned}\text{stress in concrete } f_{ct} &= \frac{A_s}{A_c} f_{sc}\\ &= 0.01 \times 35.3\\ &= 0.35 \text{ N/mm}^2 \text{ tension}\end{aligned}$$

The stresses produced in members free from external restraint are generally small as in example 1.1, and can be easily withstood both by the steel and the concrete.

(b) Shrinkage fully restrained

If the member is fully restrained, then the steel cannot be in compression since $\varepsilon_{sc} = 0$ and hence $f_{sc} = 0$ (figure 1.9). In this case the tensile strain induced in the concrete ε_{ct} must be equal to the free shrinkage strain ε_{cs}, and the corresponding stress will probably be high enough to cause cracking in immature concrete.

EXAMPLE 1.2

Calculation of fully restrained shrinkage stresses

If the member in example 1.1 were fully restrained, the stress in the concrete would be given by

$$f_{ct} = \varepsilon_{ct} E_{cm}$$

where

$$\varepsilon_{ct} = \varepsilon_{cs} = 200 \times 10^{-6}$$

then

$$\begin{aligned} f_{ct} &= 200 \times 10^{-6} \times 15 \times 10^3 \\ &= 3.0\text{N/mm}^2 \end{aligned}$$

When cracking occurs, the uncracked lengths of concrete try to contract so that the embedded steel between cracks is in compression while the steel across the cracks is in tension. This feature is accompanied by localised bond breakdown, adjacent to each crack. The equilibrium of the concrete and reinforcement is shown in figure 1.10 and calculations may be developed to relate crack widths and spacings to properties of the cross-section; this is examined in more detail in chapters 6 and 12, which deal with serviceability requirements and water retaining structures, respectively. Examples of crack width calculations are given in those chapters.

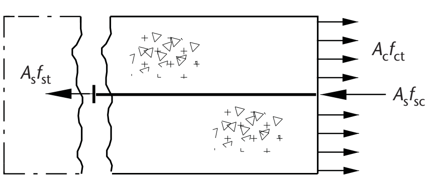


Figure 1.10
Shrinkage forces adjacent to a crack

Thermal movement

As the coefficients of thermal expansion of steel and concrete ($\alpha_{T,s}$ and $\alpha_{T,c}$) are similar, differential movement between the steel and concrete will only be very small and is unlikely to cause cracking.

The differential thermal strain due to a temperature change T may be calculated as

$$T(\alpha_{T,c} - \alpha_{T,s})$$

and should be added to the shrinkage strain ε_{cs} if significant.

The overall thermal contraction of concrete is, however, frequently effective in producing the first crack in a restrained member, since the required temperature changes could easily occur overnight in a newly cast member, even with good control of the heat generated during the hydration processes.

EXAMPLE 1.3

Thermal shrinkage

Find the fall in temperature required to cause cracking in a restrained member if ultimate tensile strength of the concrete $f_{ct,\,eff} = 2$ N/mm^2, $E_{cm} = 16$ kN/mm^2 and $\alpha_{T,\,c} = \alpha_{T,\,s} = 10 \times 10^{-6}$ per °C.

Ultimate tensile strain of concrete

$$\varepsilon_{ult} = \frac{f_{ct,\,eff}}{E_{cm}}$$

$$= \frac{2}{16 \times 10^3} = 125 \times 10^{-6}$$

Minimum temperature drop to cause cracking

$$= \frac{\varepsilon_{ult}}{\alpha_{T,\,c}} = \frac{125}{10} = 12.5^\circ\text{C}$$

It should be noted that full restraint, as assumed in this example, is unlikely to occur in practice; thus the temperature change required to cause cracking is increased. A maximum 'restraint factor' of 0.5 is often used, with lower values where external restraint is likely to be small. The temperature drop required would then be given by the theoretical minimum divided by the 'restraint factor'. i.e. $12.5/0.5 = 25^\circ$C in this example. Restraint factors and their use are considered more fully in chapters 6 and 12.

1.5 Creep

Creep is the continuous deformation of a member under sustained load. It is a phenomenon associated with many materials, but it is particularly evident with concrete. The precise behaviour of a particular concrete depends on the aggregates and the mix design as well as the ambient humidity, member cross-section, and age at first loading, but the general pattern is illustrated by considering a member subjected to axial compression. For such a member, a typical variation of deformation with time is shown by the curve in figure 1.11.

The characteristics of creep are

1. The final deformation of the member can be three to four times the short-term elastic deformation.
2. The deformation is roughly proportional to the intensity of loading and to the inverse of the concrete strength.
3. If the load is removed, only the instantaneous elastic deformation will recover – the plastic deformation will not.
4. There is a redistribution of load between the concrete and any steel present.

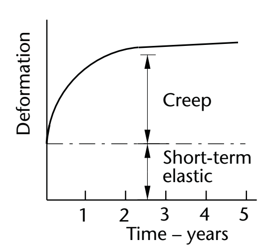


Figure 1.11
Typical increase of deformation with time for concrete

The redistribution of load is caused by the changes in compressive strains being transferred to the reinforcing steel. Thus the compressive stresses in the steel are increased so that the steel takes a larger proportion of the load.

The effects of creep are particularly important in beams, where the increased deflections may cause the opening of cracks, damage to finishes, and the non-alignment of mechanical equipment. Redistribution of stress between concrete and steel occurs primarily in the uncracked compressive areas and has little effect on the tension reinforcement other than reducing shrinkage stresses in some instances. The provision of reinforcement in the compressive zone of a flexural member, however, often helps to restrain the deflections due to creep.

1.6 Durability

Concrete structures, properly designed and constructed, are long lasting and should require little maintenance. The durability of the concrete is influenced by

1. the exposure conditions;
2. the cement type;
3. the concrete quality;
4. the cover to the reinforcement;
5. the width of any cracks.

Concrete can be exposed to a wide range of conditions such as the soil, sea water, de-icing salts, stored chemicals or the atmosphere. The severity of the exposure governs the type of concrete mix required and the minimum cover to the reinforcing steel. Whatever the exposure, the concrete mix should be made from impervious and chemically inert aggregates. A dense, well-compacted concrete with a low water–cement ratio is all important and for some soil conditions it is advisable to use a sulfate-resisting cement. Air entrainment is usually specified where it is necessary to cater for repeated freezing and thawing.

Adequate cover is essential to prevent corrosive agents reaching the reinforcement through cracks and pervious concrete. The thickness of cover required depends on the severity of the exposure and the quality of the concrete (as shown in table 6.2). The cover is also necessary to protect the reinforcement against a rapid rise in temperature and subsequent loss of strength during a fire. Part 1.2 of EC2 provides guidance on this and other aspects of fire design. Durability requirements with related design calculations to check and control crack widths and depths are described in more detail in chapter 6.

1.7 Specification of materials

1.7.1 Concrete

The selection of the type of concrete is frequently governed by the strength required, which in turn depends on the intensity of loading and the form and size of the structural members. For example, in the lower columns of a multi-storey building a higher-strength concrete may be chosen in preference to greatly increasing the size of the column section with a resultant loss in clear floor space.

As indicated in section 1.3.1, the concrete strength is assessed by measuring the crushing strength of cubes or cylinders of concrete made from the mix. These are usually cured, and tested after 28 days according to standard procedures. Concrete of a given strength is identified by its 'class' – a Class 25/30 concrete has a characteristic cylinder crushing strength (f_{ck}) of 25 N/mm^2 and cube strength of 30 N/mm^2. Table 1.2 shows a list of commonly used classes and also the lowest class normally appropriate for various types of construction.

Exposure conditions and durability can also affect the choice of the mix design and the class of concrete. A structure subject to corrosive conditions in a chemical plant, for example, would require a denser and higher class of concrete than, say, the interior members of a school or office block. Although Class 42.5 N (CEM I) Portland cement would be used in most structures, other cement types can also be used to advantage. Blast-furnace or sulfate-resisting cement may be used to resist chemical attack, low-heat cements in massive sections to reduce the heat of hydration, or rapid-hardening cement when a high early strength is required. In some circumstances it may be useful to replace some of the cement by materials such as fly ash (pulverised fuel ash) or ground granulated blast furnace slag which have slowly developing cementitious properties. These will reduce the heat of hydration and may also lead to a smaller pore structure and increased durability. Generally, natural aggregates found locally are preferred; however, manufactured lightweight material may be used when self-weight is important, or a special dense aggregate when radiation shielding is required.

The concrete mix may either be classified as 'designed' or 'designated'. A 'designed concrete' is one where the strength class, cement type, and limits to composition, including water–cement ratio and cement content, are specified. With a 'designated concrete' the producer must provide a material to satisfy the designated strength class and consistence (workability) using a particular aggregate size. 'Designated concretes' are identified as RC30 (for example) based on cube strength up to RC50 according to the application involved. 'Designed concretes' are needed in situations where 'designated concretes' cannot be used on the basis of durability requirements

Table 1.2 Strength classes of concrete

Class	*f_{ck} (N/mm^2)*	*Normal lowest class for use as specified*
C16/20	16	Plain concrete
C20/25	20	Reinforced concrete
C25/30	25	
C28/35	28	Prestressed concrete/Reinforced concrete subject to chlorides
C30/37	30	Reinforced concrete in foundations
C32/40	32	
C35/45	35	
C40/50	40	
C45/55	45	
C50/60	50	
C55/67	55	
C60/75	60	
C70/85	70	
C80/95	80	
C90/105	90	

(e.g. chloride-induced corrosion). Detailed requirements for mix specification and compliance are given by BS EN206 ‘Concrete – Performance, Production, Placing and Compliance Criteria’ and BS8500 ‘Concrete – Complementary British Standard to BS EN206’.

1.7.2 Reinforcing steel

Table 1.3 lists the characteristic design strengths of some of the more common types of reinforcement currently used in the UK. Grade 500 (500N/mm^2 characteristic strength) has replaced Grade 250 and Grade 460 reinforcing steel. EC2 permits the use of reinforcement of up to Grade 600. The nominal size of a bar is the diameter of an equivalent circular area.

Grade 250 bars are hot-rolled mild-steel bars which usually have a smooth surface so that the bond with the concrete is by adhesion only. This type of bar can be more readily bent, so they have in the past been used where small radius bends are necessary, such as links in narrow beams or columns, but plain bars are not now recognised in the European Union and they are no longer available for general use in the UK.

High-yield bars are manufactured with a ribbed surface or in the form of a twisted square. Square twisted bars have inferior bond characteristics and have been used in the past, although they are now obsolete. Deformed bars have a mechanical bond with the concrete, thus enhancing ultimate bond stresses as described in section 5.2. The bending of high-yield bars through a small radius is liable to cause tension cracking of the steel, and to avoid this the radius of the bend should not be less than two times the nominal bar size for small bars ($\leq$16 mm) or 3½ times for larger bars (see figure 5.11). The ductility of reinforcing steel is also classified for design purposes. Ribbed high yield bars may be classified as:

Class A – which is normally associated with small diameter ($\leq$12 mm) cold-worked bars used in mesh and fabric. This is the lowest ductility category and will include limits on moment redistribution which can be applied (see section 4.7) and higher quantities for fire resistance.
Class B – which is most commonly used for reinforcing bars.
Class C – high ductility which may be used in earthquake design or similar situations.

Floor slabs, walls, shells and roads may be reinforced with a welded fabric of reinforcement, supplied in rolls and having a square or rectangular mesh. This can give large economies in the detailing of the reinforcement and also in site labour costs of handling and fixing. Prefabricated reinforcement bar assemblies are also becoming

Table 1.3 Strength of reinforcement

Designation	*Normal sizes (mm)*	*Specified characteristic strength f_{yk} (N/mm^2)*
Hot-rolled high yield (BS4449)	All sizes	500
Cold-worked high yield (BS4449)	Up to and including 12	500

BS4449 should be used in conjunction with BS EN10080.

increasingly popular for similar reasons. Welded fabric mesh made of ribbed wire greater than 6 mm diameter may be of any of the ductility classes listed above.

The cross-sectional areas and perimeters of various sizes of bars, and the cross-sectional area per unit width of slabs are listed in the Appendix. Reinforcing bars in a member should either be straight or bent to standard shapes. These shapes must be fully dimensioned and listed in a schedule of the reinforcement which is used on site for the bending and fixing of the bars. Standard bar shapes and a method of scheduling are specified in BS8666. The bar types as previously described are commonly identified by the following codes: H for high yield steel, irrespective of ductility class or HA, HB, HC where a specific ductility class is required; this notation is generally used throughout this book.

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