Chapter 1

# Introduction

## 1.1 Composite beams and slabs

The design of structures for buildings and bridges is mainly concerned with the provision and support of load-bearing horizontal surfaces. Except in some long-span structures, these floors or decks are usually made of reinforced concrete, for no other material provides a better combination of low cost, high strength, and resistance to corrosion, abrasion and fire.

The economical span for a uniform reinforced concrete slab is little more than that at which its thickness becomes sufficient to resist the point loads to which it may be subjected or, in buildings, to provide the sound insulation required. For spans of more than a few metres, it is cheaper to support the slab on beams, ribs or walls than to thicken it. Where the beams or ribs are also of concrete, the monolithic nature of the construction makes it possible for a substantial breadth of slab to act as the top flange of the beam that supports it.

At spans of more than about 10 m, and especially where the susceptibility of steel to loss of strength from fire is not a problem, as in most bridges, steel beams often become cheaper than concrete beams. It was at first customary to design the steelwork to carry the whole weight of the concrete slab and its loading; but by about 1950 the development of shear connectors had made it practicable to connect the slab to the beam, and so to obtain the T-beam action that had long been used in concrete construction. The term 'composite beam' as used in this book refers to this type of structure.

The same term is in use for beams in which prestressed and *in situ* concrete act together; and there are many other examples of composite action in structures, such as between brick walls and beams supporting them, or between a steel-framed shed and its cladding; but these are outside the scope of this book.

No income is received from money invested in construction of a multi-storey building such as a large office block until the building is occupied.

The construction time is strongly influenced by the time taken to construct a typical floor of the building, and here structural steel has an advantage over *in situ* concrete.

Even more time can be saved if the floor slabs are cast on permanent steel formwork, which acts first as a working platform and then as bottom reinforcement for the slab. The use of this formwork, known as *profiled steel sheeting*, commenced in North America [1], and is now standard practice in Europe and elsewhere. These floors span in one direction only, and are known as *composite slabs*. Where the steel sheet is flat, so that two-way spanning occurs, the structure is known as a *composite plate*. These occur in box-girder bridges.

Steel profiled sheeting and partial-thickness precast concrete slabs are known as *structurally participating* formwork. Cement or plastic profiled sheeting reinforced by fibres is sometimes used. Its contribution to the strength of the finished slab is normally ignored in design.

The degree of fire protection that must be provided is another factor that influences the choice between concrete, composite and steel structures, and here concrete has an advantage. Little or no fire protection is required for open multi-storey car parks, a moderate amount for office blocks, and most of all for public buildings and warehouses. Many methods have been developed for providing steelwork with fire protection.

Design against fire and the prediction of fire resistance is known as fire engineering [2]. Several of the Eurocodes have a Part 1.2 devoted to it. Full or partial encasement in concrete is an economical method for steel columns, since the casing makes the columns much stronger. Full encasement of steel beams, once common, is now more expensive than the use of lightweight non-structural materials. Concrete encasement of the web only, done before the beam is erected, is more common in continental Europe than in the UK, and is covered in EN 1994-1-1 [3]. It enhances the buckling resistance of the member (Section 4.2.4) as well as providing fire protection.

The choice between steel, concrete and composite construction for a particular structure thus depends on many factors that are outside the scope of this book. Composite construction is particularly competitive for medium- or long-span structures where a concrete slab or deck is needed for other reasons, where there is a premium for rapid construction, and where a low or medium level of fire protection to steelwork is sufficient.

## 1.2 Composite columns and frames

When the columns in steel frames were first encased in concrete to protect them from fire, they were designed for the applied load as if uncased. It was then found that encasement reduced the effective slenderness of the column, and so increased its buckling load. Empirical methods for calculating the reduced slenderness still survive in some design codes for steelwork.

This simple approach is not rational, for the encasement also carries its share of both the axial load and the bending moments. More rational methods, validated by tests, are given in EN 1994 (Section 5.6).

A composite column can also be constructed without the use of formwork, by filling a steel tube with concrete. A notable early use of filled tubes (1966) was in a four-level motorway interchange [4]. Their design is covered in Section 5.6.7.

In framed structures, there may be steel members, composite beams, composite columns, or all of these, and there are many types of beam-to-column connection. Their behaviour can range from 'nominally pinned' to 'rigid', and influences bending moments throughout the frame. Two buildings with rigid-jointed composite frames were built in England in the early 1960s, one in Cambridge [5] and one in London [6]. Current practice is mainly to use nominally pinned joints. In buildings it is expensive to make joints so stiff that they act as 'rigid'. Joints are usually treated as pins, even though many have sufficient stiffness to reduce deflections of beams to a useful extent. Intensive research in recent years [7, 8, 9] has enabled comprehensive design rules for joints in steel and composite frames to be given in Eurocodes 3 [10] and 4. Some of them lead to extensive calculation, but they provide the basis for design aids that, when available, may bring semi-rigid connections into general use.

## 1.3 Design philosophy and the Eurocodes

### 1.3.1 *Background*

In design, account must be taken of the random nature of loading, the variability of materials, and the defects that occur in construction, to reduce the probability of unserviceability or failure of the structure during its design life to an acceptably low level. Extensive study of this subject since about 1950 has led to the incorporation of the older 'safety factor' and 'load factor' design methods into a comprehensive 'limit state' design philosophy. Its first important application in British standards was in 1972, in CP 110, *The structural use of concrete*. It is used in all current British codes for the design of structures.

Work on international codes began after the Second World War, first on concrete structures and then on steel structures. A committee for composite structures, set up in 1971, prepared the Model Code of 1981 [11]. The Commission of the European Communities has supported work on Eurocodes since 1982, and has delegated its management to the Comité Européen Normalisation (CEN), based in Brussels. This is an association of the national standards institutions (NSIs) of the countries of the European Union, the European Free Trade Area, and a growing number of other countries from central and eastern Europe.

The Eurocodes EN 1990 to 1999, with over 50 Parts, each with a national annex, are being published by the NSIs, from 2002 until about 2007, as explained in the Preface. Those most relevant to this book are listed as References 3, 10 and 12–16, with the expected or actual date of publication in English by the British Standards Institution. They provide a coherent system, in which duplication of information has been minimised. For example, EN 1994 refers to EN 1990, *Basis of structural design* [12], for design philosophy, most definitions, limit state requirements, and values of partial factors for loads and other actions.

Values for loads and other actions that do not depend on the material used for the structure (the great majority) are given in EN 1991, *Actions on structures* [13]. All provisions for structural steel that apply to both steel and composite structures are in EN 1993, *Design of steel structures* [15]. Similarly, for concrete, EN 1994 refers to but does not repeat material from EN 1992, *Design of concrete structures* [14].

Even within Eurocode 4, material is divided between that which applies to both buildings and bridges, to buildings only, and to bridges only. The first is found in the 'General' clauses of EN 1994-1-1, the second in clauses in EN 1994-1-1 marked 'for buildings', and the third in EN 1994-2, 'Rules for bridges'. Structural fire design is found in EN 1994-1-2 [16], which cross-refers for the high-temperature properties of materials to the 'Fire' parts of EN 1992 and EN 1993, as appropriate.

Design of foundations is covered in EN 1997, *Geotechnical design*, and seismic design in EN 1998, *Design of structures for earthquake resistance*.

This book presents the theories, methods, and models of the 'General' and 'for buildings' rules of Eurocode 4, including relevant material from Eurocodes 1, 2 and 3, but does not refer to or comment on specific clauses. Commentary on EN 1994-1-1 will be found in Reference [17], and on the other codes in the relevant 'Designers' Guides', such as Reference [18].

The British codes current in 2004 that are most relevant to this book are Part 3: Section 3.1 and Part 4 of BS 5950 [19]. They have much in common with EN 1994 as they were developed in parallel with it, but their scope is narrower. For example, columns, web-encased beams, and box girders are not covered.

### 1.3.2 *Limit state design philosophy*

#### 1.3.2.1 *Basis of design, and actions*

Parts 1.1 of ENs 1992, 1993 and 1994 each have a Section 2, 'Basis of design', that refers to EN 1990 for the presentation of limit state design as used in the Eurocodes. Its Section 4, 'Basic variables', classifies these as actions, environmental influences, properties of materials and products, and geometrical data (e.g., initial out-of-plumb of a column). Actions are either:

- direct actions (forces or loads applied to the structure) or
- indirect actions, such as deformations imposed on the structure, for example by settlement of foundations, change of temperature, or shrinkage of concrete.

'Actions' thus has a wider meaning than 'loads'. Similarly, the Eurocode term 'effect of actions' has a wider meaning than 'stress resultant', because it includes stresses, strains, deformations, crack widths, etc., as well as bending moments, shear forces, etc. The Eurocode term for 'stress resultant' is 'internal force or moment'.

The scope of the following introduction to limit state design is limited to that of the design examples in this book. There are two classes of *limit states*:

- ultimate (denoted ULS), which are associated with structural failure, whether by rupture, crushing, buckling, fatigue or overturning, and
- serviceability (SLS), such as excessive deformation, vibration, or width of cracks in concrete.

Either type of limit state may be reached as a consequence of poor design, construction, or maintenance, or from overloading, insufficient durability, fire, etc.

There are three types of *design situation*:

- persistent, corresponding to normal use;
- transient, for example during construction, refurbishment or repair;
- accidental, such as fire, explosion or earthquake.

There are three main types of action:

- permanent ($G$ or $g$), such as self-weight of a structure (formerly 'dead load') and including shrinkage of concrete;

- variable ($Q$ or $q$), such as imposed, wind or snow load (formerly 'live load') and including expected changes of temperature;
- accidental ($A$), such as impact from a vehicle and high temperature from a fire.

The spatial variation of an action is either:

- fixed (typical of permanent actions) or
- free (typical of other actions), and meaning that the action may occur over only a part of the area or length concerned.

Permanent actions are represented (and specified) by a *characteristic value*, $G_k$. 'Characteristic' implies a defined fractile of an assumed statistical distribution of the action, modelled as a random variable. For permanent loads, it is usually the mean value (50% fractile).

Variable actions have four *representative values*:

- characteristic ($Q_k$), normally the upper 5% fractile;
- combination ($\psi_0 Q_k$), for use where the action is assumed to accompany the design ultimate value of another variable action, which is the 'leading action';
- frequent ($\psi_1 Q_k$), for example, occurring at least once a week, and
- quasi-permanent ($\psi_2 Q_k$).

Recommended values for the combination factors $\psi_0$, $\psi_1$ and $\psi_2$ (all less than 1.0) are given in EN 1990. Definitive values, usually those recommended, are given in national annexes. For example, for imposed loads on the floors of offices, the recommended values are $\psi_0 = 0.7$, $\psi_1 = 0.5$, and $\psi_2 = 0.3$.

*Design values of actions* are, in general, $F_d = \gamma_F F_k$, and in particular,

$$G_d = \gamma_G G_k \tag{1.1}$$

$$Q_d = \gamma_Q Q_k \quad \text{or} \quad Q_d = \gamma_Q \psi_i Q_k \tag{1.2}$$

where $\gamma_G$ and $\gamma_Q$ are partial factors for actions, recommended in EN 1990 and given in national annexes. They depend on the limit state considered, and on whether the action is unfavourable or favourable for (i.e., tends to increase or decrease) the action effect considered. The values used in this book are given in Table 1.1.

The *effects of actions* are the responses of the structure to the actions:

$$E_d = E(F_d) \tag{1.3}$$

**Table 1.1** Values of $\gamma_G$ and $\gamma_Q$ for persistent design situations

| Type of action | Permanent unfavourable | Permanent favourable | Variable unfavourable | Variable favourable |
|---|---|---|---|---|
| Ultimate limit states | 1.35* | 1.35* | 1.5 | 0 |
| Serviceability limit states | 1.0 | 1.0 | 1.0 | 0 |

*Except for checking loss of equilibrium, or where the coefficient of variation is large

where the function $E$ represents the process of structural analysis. Where the effect is an internal force or moment, *verification for an ultimate limit state* consists of checking that

$$E_d \leq R_d \tag{1.4}$$

where $R_d$ is the relevant design resistance of the system or member or cross-section considered.

#### 1.3.2.2 *Resistances*

Resistances, $R_d$, are calculated using design values of properties of materials, $X_d$, given by

$$X_d = X_k/\gamma_M \tag{1.5}$$

where $X_k$ is a characteristic value of the property and $\gamma_M$ is the partial factor for that property.

The characteristic value is typically a 5% lower fractile (e.g., for compressive strength of concrete). Where the statistical distribution is not well established, it is replaced by a *nominal* value (e.g., the yield strength of structural steel), so chosen that it can be used in design in place of $X_k$.

The subscript M in $\gamma_M$ is often replaced by a letter that indicates the material concerned, as shown in Table 1.2, which gives the values of $\gamma_M$

**Table 1.2** Recommended values for $\gamma_M$ for strengths of materials and for resistances

| Material | Structural steel | Profiled sheeting | Reinforcing steel | Concrete | Shear connection |
|---|---|---|---|---|---|
| Property | $f_y$ | $f_y$ | $f_{sk}$ | $f_{ck}$ or $f_{cu}$ | $P_{Rk}$ |
| Symbol for $\gamma_M$ | $\gamma_A$ | $\gamma_A$ | $\gamma_S$ | $\gamma_C$ | $\gamma_V$ or $\gamma_{Vs}$ |
| Ultimate limit states | 1.0 | 1.0 | 1.15 | 1.5 | 1.25 |
| Serviceability limit states | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |

Notation: for concrete, $f_{ck}$ and $f_{cu}$ are respectively characteristic cylinder and cube strengths; symbol $\gamma_{Vs}$ is for shear resistance of a composite slab.

used in this book. A welded stud shear connector is treated like a single material, even though its design resistance to shear, $P_{Rk}/\gamma_V$, is influenced by the properties of both steel and concrete. For resistance to fracture of a steel cross-section in tension, $\gamma_A = 1.25$.

### 1.3.2.3 *Combinations of actions*

The Eurocodes treat systematically a subject for which many empirical procedures have been used in the past. For ultimate limit states, the principles are:

- permanent actions are present in all combinations;
- each variable action is chosen in turn to be the 'leading' action (i.e., to have its full design value) and is combined with lower 'combination' values of other variable actions that may co-exist with it;
- the design action effect is the most unfavourable of those found by this process.

The use of combination values allows for the limited correlation between time-dependent variable actions.

As an example, it is assumed that a bending moment $M_{Ed}$ in a member is influenced by its own weight, $G$, by an imposed vertical load, $Q_1$, and by wind loading, $Q_2$. The fundamental combinations for verification for persistent design situations are:

$$\gamma_G G_k + \gamma_{Q1} Q_{k,1} + \gamma_{Q2} \psi_{0,2} Q_{k,2} \tag{1.6}$$

and

$$\gamma_G G_k + \gamma_{Q1} \psi_{0,1} Q_{k,1} + \gamma_{Q2} Q_{k,2} \tag{1.7}$$

Each term in these expressions gives the value of the action for which a bending moment is calculated, and the + symbols apply to the bending moments, not to the values of the actions. This is sometimes indicated by placing each term between quotation marks.

In practice, it is usually obvious which combination will govern. For low-rise buildings, wind is rarely critical for floors, so Expression 1.6 with imposed load leading would be used; but for a long-span lightweight roof, Expression 1.7 would govern, and both positive and negative wind pressure would be considered, with the negative pressure combined with $Q_{k,1} = 0$.

The combination for the accidental design situation of 'fire' is given in Section 3.3.7.

**Table 1.3** Combination factors

| Factor | $\psi_0$ | $\psi_1$ | $\psi_2$ |
|---|---|---|---|
| Imposed floor loading in an office area of a building | 0.7 | 0.5 | 0.3 |
| Wind loading on a building | 0.6 | 0.2 | 0 |

For serviceability limit states, three combinations are defined. The most onerous, the 'characteristic' combination, corresponds to the fundamental combination (above) with the $\gamma$ factors reduced to 1.0. For the example in Expressions 1.6 and 1.7, it is

$$G_k + Q_{k,1} + \psi_{0,2}Q_{k,2} \tag{1.8}$$

and

$$G_k + \psi_{0,1}Q_{k,1} + Q_{k,2} \tag{1.9}$$

It is normally used for verifying irreversible limit states, for example, deformations that result from the yielding of steel.

Assuming that $Q_1$ is the leading variable action, the others are:

- frequent combination,

$$G_k + \psi_{1,1}Q_{k,1} + \psi_{2,2}Q_{k,2} \tag{1.10}$$

- quasi-permanent combination,

$$G_k + \psi_{2,1}Q_{k,1} + \psi_{2,2}Q_{k,2} \tag{1.11}$$

The frequent combination is used for reversible limit states, for example, the elastic deflection of a floor under imposed loading. However, if that deformation causes cracking of a brittle floor finish or damage to fragile partitions, then the limit state is not reversible, and the check should be done for the higher (less probable) loading of the characteristic combination.

The quasi-permanent combination is used for long-term effects (e.g., deformations from creep of concrete) and for the appearance of the structure.

Some combination factors used in this book are given in Table 1.3.

#### 1.3.2.4 *Comments on limit state design philosophy*

The use of limit states has superseded earlier methods, partly because limit states provide identifiable criteria for satisfactory performance.

Stresses cannot be calculated with the same confidence as resistances of members, and high values may or may not be significant.

An apparent disadvantage of limit states design is that several sets of design calculations may be needed whereas, with some older methods, one was sufficient. This is only partly true, for it has been found possible to identify many situations in which design for, say, an ultimate limit state will ensure that certain types of unserviceability will not occur, and vice versa. In the rules of EN 1994 for buildings it has generally been possible to avoid specifying limiting stresses for serviceability limit states by using other methods to check deflections and crack widths.

## 1.4 Properties of materials

Information on the properties of structural steel, profiled sheeting, concrete and reinforcement is readily available. Only that which has particular relevance to composite structures is given here.

In the determination of the bending moments and shear forces in a beam or framed structure (known as 'global analysis'), all the materials can be assumed to behave in a linear-elastic manner, though an effective modulus is used for the concrete to allow for its creep under sustained compressive stress. Its tensile strength need not be taken as zero, provided account is taken of reductions of stiffness caused by cracking. The effects of its shrinkage are rarely significant in buildings.

Rigid-plastic global analysis can sometimes be used (Section 4.3.3) despite the profound difference between a typical stress–strain curve for concrete in compression and those for structural steel or reinforcement, in tension or compression, that is illustrated in Fig. 1.1.

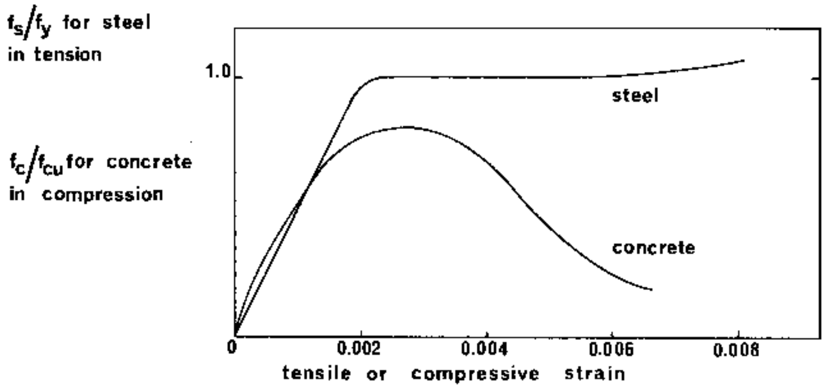


**Figure 1.1** Stress–strain curves for concrete and structural steel

Concrete reaches its maximum compressive stress at a strain of between 0.002 and 0.003, and at higher strains it crushes, losing almost all of its compressive strength. It is very brittle in tension, having a strain capacity of only about 0.0001 (i.e., 0.1 mm per metre) before it cracks. Figure 1.1 also shows that the maximum stress reached by concrete in a beam or column is well below its cube strength.

Steel yields at a strain similar to that given for the maximum stress in concrete, but on further straining the stress continues to increase slowly, until (for a typical structural steel) the total strain is at least thirty times the yield strain. Its subsequent necking and fracture is of little significance for composite members because the useful resistance of a cross-section is reached when all of the steel has yielded, when steel in compression buckles, or when concrete crushes.

Resistances of cross-sections are determined using plastic analysis wherever possible because results of elastic analyses are unreliable, unless careful account is taken of the cracking, shrinkage, and creep of concrete (which is difficult), and also because plastic analysis is simpler and leads to more economical design.

The use of a higher value of $\gamma_M$ for concrete than for steel (Table 1.2) includes allowance for the higher variability of the strength of test specimens, and the variation in the strength of concrete over the depth of a member, due to migration of water before setting. It also allows for the larger errors in the dimensions of cross-sections, particularly in the positions of reinforcing bars.

Brief comments are now given on individual materials.

### *Concrete*

In EN 1992, a typical strength class for normal-density concrete is denoted C25/30, where the specified characteristic compressive strengths at age 28 days are $f_{ck} = 25$ N/mm$^2$ (cylinder test) and $f_{cu} = 30$ N/mm$^2$ (cube test). The design formulae in EN 1994 use $f_{cd}$, which is $f_{ck}/\gamma_C$. The normal-density concrete used in worked examples here is 'Grade 30' (in British terminology), with $f_{ck}$ taken as 25 N/mm$^2$. It is used for the columns and for encasement of beam webs. The floor slabs are constructed with lightweight-aggregate concrete of grade LC25/28, with oven-dry density 1800 kg/m$^3$. Other properties for these two concretes are given in Table 1.4.

For densities, EN 1991-1-1 uses kN/m$^3$ units, and so gives densities about 2% higher than those of EN 1992-1-1, since 1800 kg/m$^3$ is 17.65 kN/m$^3$. The higher values are used here.

### *Reinforcing steel*

Strength grades for reinforcing steel are given in EN 10080 [20] in terms of a characteristic yield strength $f_{sk}$. The value used here is 500 N/mm$^2$,

**Table 1.4** Properties of concretes used in the examples, at age 28 days

| **Concrete grade** | | | **C25/30** | **LC25/28** |
|---|---|---|---|---|
| Characteristic cylinder strength, | $N/mm^2$ | $f_{ck}$ | 25 | 25 |
| Mean cylinder strength, | $N/mm^2$ | $f_{cm}$ | 33 | 33 |
| Lower tensile strength, | $N/mm^2$ | $f_{ct,0.05}$ | 1.80 | 1.60 |
| Mean tensile strength, | $N/mm^2$ | $f_{ctm}$ | 2.60 | 2.32 |
| Upper tensile strength, | $N/mm^2$ | $f_{ct,0.95}$ | 3.30 | 2.94 |
| Mean elastic modulus, | $kN/mm^2$ | $E_{cm}$ | 31.0 | 20.7 |
| Weight density, reinforced concrete, | $kN/m^3$ | | 25.0 | 19.5 |

for both ribbed bars and welded steel fabric. The modulus of elasticity for reinforcement, $E_s$, is normally taken as 200 kN/mm$^2$; but in a composite section it may be assumed to have the value for structural steel, $E_a$ = 210 kN/mm$^2$, as the error is negligible.

### *Structural steel*

Strength grades for structural steel are given in EN 10025 [21] in terms of a nominal yield strength, $f_y$, and ultimate tensile strength, $f_u$. The grade used in worked examples here is S355, for which $f_y$ = 355 N/mm$^2$, $f_u$ = 510 N/mm$^2$ for elements of all thicknesses up to 40 mm.

The density of structural steel is assumed to be 7850 kg/m$^3$. Its coefficient of linear thermal expansion is $12 \times 10^{-6}$ per °C. The difference between this value and that for normal-density concrete, $10 \times 10^{-6}$ per °C, can usually be ignored.

### *Profiled steel sheeting*

This product is available with yield strengths ranging from 235 N/mm$^2$ to at least 460 N/mm$^2$, in profiles with depths ranging from 45 mm to over 200 mm, and with a wide range of shapes. These include both re-entrant ribs, and trapezoidal troughs as in Fig. 3.9. There are various methods of achieving composite action with a concrete slab, discussed in Section 2.4.3.

Sheets are normally between 0.8 mm and 1.5 mm thick, and are protected from corrosion by a zinc coating about 0.02 mm thick on each face. Elastic properties of the material may be assumed to be as for structural steel.

### *Shear connectors*

In the early years of composite construction, many types of connector were in use. This market is now dominated by automatically-welded headed studs. Details of these and the measurement of their resistance to shear are given in Chapter 2.

## 1.5 Direct actions (loading)

The characteristic loadings to be used in worked examples are now given. They are taken from EN 1991.

The *permanent loads* (dead load) are the weight of the structure and its finishes. The structural steel or profiled sheeting component of a composite member is invariably built first, so a distinction must be made between load resisted by the steel component only, and load applied to the member after the concrete has developed sufficient strength for composite action to become effective. The division of the permanent load between these categories depends on the method of construction.

Composite beams and slabs are classified as *propped* ('shored' in North America) or *unpropped*. In propped construction, the steel member is supported at intervals along its length until the concrete has reached a certain proportion, usually three-quarters, of its design strength. When the props are removed, the whole of the dead load is assumed to be carried by the composite member. Where no props are used, it is assumed in elastic analysis that the steel member alone carries its own weight and that of the formwork and the concrete slab. Other dead loads such as floor finishes and internal walls are added later, and so are assumed to be carried by the composite member. In ultimate-strength methods of analysis (Section 3.5.3), inelastic behaviour causes extensive redistribution of stress before failure, and it can be assumed that the whole load is applied to the composite member, whatever the method of construction.

The principal vertical *variable action* in a building is a uniformly-distributed load on each floor. EN 1991-1-1 gives ranges of loads, depending on the use to be made of the area, with a recommended value. For 'office areas', this value is

$$q_k = 3.0\ \text{kN/m}^2 \tag{1.12}$$

Account is taken of point loads (e.g., a safe being moved on a trolley with small wheels) by defining an alternative point load, to be applied anywhere on the floor, on an area about 50 mm square. For the type of area defined above, this is

$$Q_k = 4.5\ \text{kN} \tag{1.13}$$

Where a member such as a column is carrying loads $q_k$ from $n$ storeys ($n > 2$), the total of these loads may be multiplied by a reduction factor $\alpha_n$. The recommended value is

$$\alpha_n = [2 + (n - 2)\psi_0]/n \tag{1.14}$$

where $\psi_0$ is the combination factor (e.g., as in Table 1.3). This allows for the low probability that all $n$ floors will be fully loaded at once.

In an office block, the location of partitions is unknown at the design stage. Their weight is usually allowed for by increasing the imposed loading, $q_k$, by an amount that depends on the expected weight per unit length of the partitions. The increases given in EN 1991-1-1 range from 0.5 to 1.2 kN/m$^2$.

The principal horizontal variable load for a building is wind. These loads are given in EN 1991-1-4. They usually consist of pressure or suction on each external surface. On large flat areas, frictional drag may also be significant. Wind loads rarely influence the design of composite beams, but can be important in framed structures not braced against side-sway and in tall buildings.

Methods of calculation for distributed and point loads are sufficient for all types of direct action. Indirect actions such as subsidence or differential changes of temperature, which occasionally influence the design of structures for buildings, are not considered in this book.

## 1.6 Methods of analysis and design

The purpose of this section is to provide a preview of the principal methods of analysis used for composite members and frames, and to show that most of them are straightforward applications of methods in common use for steel or reinforced concrete structures.

The steel designer will be familiar with the elementary elastic theory of bending, and the simple plastic theory in which the whole cross-section of a member is assumed to be at yield, in either tension or compression. Both theories are used for composite members, the differences being as follows:

- concrete in tension is usually neglected in elastic theory, and always neglected in plastic theory;
- in the elastic theory, concrete in compression is 'transformed' into an equivalent area of steel by dividing its breadth by the modular ratio $E_a/E_c$;
- in the plastic theory, the design 'yield stress' of concrete in compression is taken as $0.85f_{cd}$, where $f_{cd} = f_{ck}/\gamma_C$. Transformed sections are not used. Examples of this method are given in Sections 3.4.2 and 3.11.1.

In the strength classes for concrete in EN 1992, the ratios $f_{ck}/f_{cu}$ range from 0.78 to 0.83, so for $\gamma_C = 1.5$, the stress $0.85f_{cd}$ corresponds to a value between $0.44f_{cu}$ and $0.47f_{cu}$. This agrees closely with BS 5950 [19], which uses $0.45f_{cu}$ for the plastic resistance of cross-sections.

The factor 0.85 takes account of several differences between a standard cylinder test and what concrete experiences in a structural member. These include the longer duration of loading in the structure, the presence of a stress gradient across the section considered, and the differences in the boundary conditions for the concrete.

The concrete designer will be familiar with the method of transformed sections, and with the rectangular-stress-block theory outlined above. The basic difference from the elastic behaviour of a reinforced concrete beam is that the steel section in a composite beam is more than tension reinforcement. It has a significant bending stiffness of its own, and resists most of the vertical shear.

The formulae for the elastic properties of composite sections are more complex than those for steel or reinforced concrete sections. The chief reason is that the neutral axis for bending may lie in the web, the steel flange, or the concrete flange of the member. The theory is not in principle any more complex than that for a steel I-beam.

### *Longitudinal shear*

Students usually find this subject troublesome, even though the formula

$$\tau = VA\bar{y}/Ib \tag{1.15}$$

is familiar from their study of vertical shear in elastic beams, so a note on its use may be helpful. Its proof can be found in any undergraduate-level textbook on strength of materials.

First, let us consider the shear stresses $\tau$ in the elastic I-beam shown in Fig. 1.2, due to a vertical shear force $V$. For a cross-section 1–2 through the web, the 'excluded area', $A$ in the formula, is the top flange, of area $A_f$. The distance $\bar{y}$ of its centroid from the neutral axis (line X–X in Fig. 1.2) is $(h - t_f)/2$. The shear stress $\tau_{12}$ on plane 1–2, of breadth $t_w$, is therefore

$$\tau_{12} = VA_f(h - t_f)/(2It_w) \tag{1.16}$$

where $I$ is the second moment of area of the whole cross-section about the axis X–X through its centre of area.

This result may be recognised as the *vertical* shear stress at this cross-section. However, a shear stress is always associated with a complementary shear stress at right angles to it and of equal value; in this case, the *longitudinal* shear stress. This will now be denoted by $v$, rather than $\tau$, as $v$ is used in EN 1994.

If the cross-section in Fig. 1.2 is a composite beam, with the cross-hatched area representing the transformed area of a concrete flange, shear connection is required on plane 1–2. It has to resist this longitudinal shear

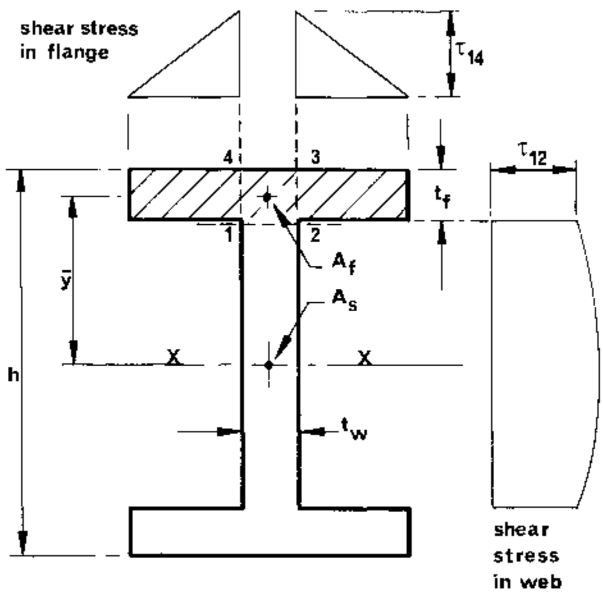


**Figure 1.2** Shear stresses in an elastic I-section

stress over a width $t_w$, so the shear force per unit length is $vt_w$. This is named the *shear flow*, and has the symbol $v_L$. From these results,

$$v_{L,12} = VA_f(h - t_f)/(2I) \tag{1.17}$$

Consideration of the longitudinal equilibrium of the small element 1234 in Fig. 1.2 shows that if its area $t_w t_f$ is much less than $A_f$, the shear flows on planes 1–4 and 2–3 are each approximately $v_{L,12}/2$, and the mean shear stress on these planes is given approximately by

$$\tau_{14} t_f = \tau_{12} t_w / 2$$

This stress is needed for checking the resistance of the concrete slab to longitudinal shear.

Repeated use of Equation 1.15 for various cross-sections shows that the variation of longitudinal shear stress is parabolic in the web and linear in the flanges, as shown in Fig. 1.2.

The second example is the elastic beam shown in section in Fig. 1.3. This represents a composite beam in sagging bending, with the neutral axis at depth $x$, a concrete slab of thickness $h_c$, and the interface between the slab and the structural steel (which is assumed to have no top flange) at level 6–5. The concrete has been transformed to steel, so the cross-hatched area is the equivalent steel section. The concrete in area ABCD is

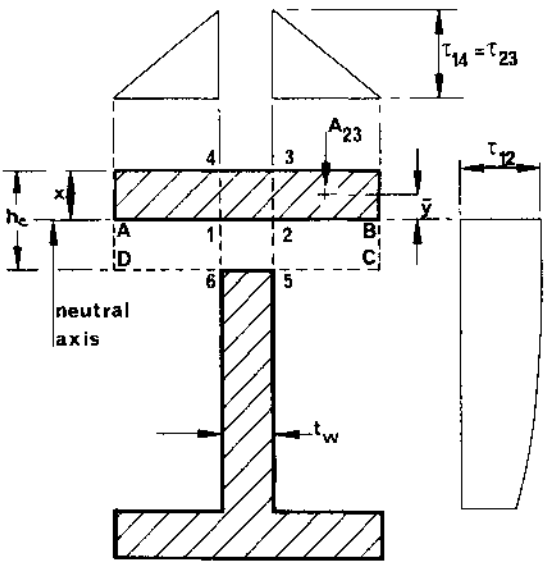


**Figure 1.3** Shear stresses in a composite section with the neutral axis in the concrete slab

assumed to be cracked. As in the theory for reinforced concrete beams, it resists no longitudinal stress but is capable of transferring shear stress.

Equation 1.15 is based on rate of change along the beam of bending stress, so in applying it here, area ABCD is omitted when the 'excluded area' is calculated. Let the cross-hatched area of flange be $A_f$, as before. The longitudinal shear stress on plane 6–5 is given by

$$v_{65} = VA_f\bar{y}/It_w \tag{1.18}$$

where $\bar{y}$ is the distance from the centroid of the excluded area to the neutral axis, *not to plane 6–5*. If $A$ and $\bar{y}$ are calculated for the cross-hatched area below plane 6–5, the same value $v_{65}$ is obtained, because it is the equality of the two products '$A\bar{y}$' that determines the value $x$.

The preceding theory relies on the assumption that the flexibility of shear connectors is negligible, and is used in bridge design and for fatigue generally. Ultimate-strength theory (Sections 3.3.2 and 3.6.2) provides an alternative that takes advantage of the plastic behaviour of stud connectors and is widely used in design for buildings.

For a plane such as 2–3 in Fig. 1.3, the shear flow is

$$v_{L,23} = VA_{23}\bar{y}/I \tag{1.19}$$

The design shear stress for the concrete on this plane is $v_{L,23}/h_c$. It is not equal to $v_{L,23}/x$ because the cracked concrete can resist shear. The depth $h_c$

does not have to be divided by the modular ratio, even though the transformed section is of steel, because the transformation is of widths, not depths. An alternative explanation is that shear flows from equations such as Equation 1.19 are independent of the material considered, because transformation does not alter the ratio $A_{23}/I$.

### *Longitudinal slip*

Shear connectors are not rigid, so that a small longitudinal slip occurs between the steel and concrete components of a composite beam. The problem does not arise in other types of structure, and relevant analyses are quite complex (Section 2.6 and Appendix A). They are not needed in design, for which simplified methods have been developed.

### *Deflections*

The effects of creep and shrinkage make calculation of deflections in reinforced concrete beams more complex than for steel beams, but the limiting span/depth ratios given in codes such as BS 8110 [22] provide a simple means of checking for excessive deflection. These ratios are unreliable for composite beams, especially where unpropped construction is used. Examples of checks on deflections are given in Sections 3.4.5 and 3.11.3.

### *Vertical shear*

The stiffness in vertical shear of the concrete slab of a composite beam is usually much less than that of the steel component, and is neglected in design. For vertical shear, the methods used for steel beams are applicable also to composite beams.

### *Buckling of flanges and webs of beams*

This will be a new problem to many designers of reinforced concrete. It leads to restrictions on the breadth/thickness ratios of unstiffened steel webs and flanges in compression (Section 3.5.2). These do not apply to the steel part of the top flange of a composite T-beam at mid-span, because local buckling is prevented by its attachment to the concrete slab.

### *Crack-width control*

The maximum spacings for reinforcing bars given in codes for reinforced concrete are intended to limit the widths of cracks in the concrete, for reasons of appearance and to avoid corrosion of reinforcement. In composite structures for buildings, cracking is likely to be a problem only where the top surfaces of continuous beams support brittle finishes or are exposed to corrosion. The principles of crack-width control are the same as for reinforced concrete. The calculations are different, but can normally be avoided by using the simplified methods given in EN 1994.

### Continuous beams

The Eurocode design methods for continuous beams (Chapter 4) make use of both simple plastic theory (as for steel beams) and redistribution of moments (as for concrete beams).

### Columns

The only British code that gives a design method for composite columns is BS 5400:Part 5, *Composite bridges*. EN 1994 gives a newer and simpler method, which is described in Section 5.6.

### Framed structures for buildings

Composite members normally form part of a frame that is essentially steel, rather than concrete, so the design methods of EN 1994 are based on those of EN 1993 for steel structures. Beam-to-column joints are classified in the same way; the same assumptions are made about geometrical imperfections, such as out-of-plumb columns; and similar allowance is made for second-order effects (increases in bending moments and reduction in stability, caused by interaction between vertical loads and lateral deflections). Frame analysis is outlined in Section 5.4.5. It may be more complex than in current practice, but includes methods for unbraced frames. Eurocodes EN 1993 and 1994 contain much new material on the design of joints.

### Structural fire design

The high thermal conductivity and slenderness of structural steel members and profiled sheeting cause them to lose strength in fire more quickly than concrete members do. Structures for buildings are required to have fire resistance of minimum duration (typically, 30 minutes to 2 hours) to enable occupants to escape and to protect fire fighters. This has led to the provision of minimum thicknesses of concrete and areas of reinforcement and of thermal insulation of steelwork.

Extensive research and the recent subject of fire engineering [2] have enabled the Eurocode rules for resistance to fire to be less onerous than older rules. Advantage is taken in design of membrane effects associated with large deformations, and of the provisions for accidental design situations. These allow for over-strength of members and the use of 'frequent' rather than 'characteristic' load levels. Explanations and worked examples are given in Sections 3.3.7, 3.4.6, 3.10, 3.11.4 and 5.6.2.