LAWRENCE MARTIN
JOHN PURKISS

SECOND EDITION

CONCRETE DESIGN

TO EN 1992

ROUTLEDGE

Concrete Design to EN 1992

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Concrete Design to EN 1992

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Preface

This book conforms with the latest recommendations for the design of reinforced and prestressed concrete structures as described in Eurocode 2: Design of Concrete Structures – Part 1-1: General rules and rules for buildings. References to relevant clauses of the Code are given where appropriate.

Where necessary the process of design has been aided by the production of design charts.

Whilst it has not been assumed that the reader has a knowledge of structural design, a knowledge of structural mechanics and stress analysis is a prerequisite. The book contains detailed explanations of the principles underlying concrete design and provides references to research where appropriate.

The text should prove useful to students reading for engineering degrees at University especially for design projects. It will also aid designers who require an introduction to the new EuroCode.

For those familiar with current practice, the major changes are:

(1) Calculations may be more extensive and complex.
(2) Design values of steel stresses are increased.
(3) High strength concrete is encompassed by the Eurocode by modifications to the flexural stress block.
(4) There is no component from the concrete when designing shear links for beams. This is not the case for slabs where there is a concrete component.
(5) Bond resistance is more complex.
(6) Calculations for column design are more complicated.
(7) For fire performance, the distance from the exposed face to the centroid of the bar (axis distance) is specified rather than the cover.

NOTE: As this text has been produced before the availability of the National Annex which will amend, if felt appropriate, any Nationally Determined Parameters, the recommended values of such parameters have been used. The one exception is the value of the coefficient α_{cc} allowing for long term effects has been taken as 0,85 (the traditional UK value) rather than the recommended value of 1,0.

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Principal Symbols

Listed below are the symbols and suffixes common to Eurocodes

LATIN UPPER AND LOWER CASE

A	accidental action; area
a	distance; shear span
B	width
b	width
D	depth; diameter of mandrel
d	effective depth
E	modulus of elasticity
e	eccentricity
F	action; force
f	strength of a material; stress
G	permanent action; shear modulus
H	total horizontal load or reaction
h	height
I	second moment of area
i	radius of gyration
k	depth factor for shear resistance
L	length; span
l	buckling length; bond length
M	bending moment
N	axial force
n	number
P	prestressing force
Q	variable action
q	uniformly distributed force
R	resistance; reaction; low strength steel
r	radius
s	spacing of links
T	torsional moment; high strength steel
t	thickness
u	perimeter
V	shear force

v	shear stress
x	neutral axis depth
W	load
w	load per unit length
Z	section modulus
z	lever arm

GREEK LOWER CASE

α	coefficient of linear thermal expansion; angle of link; ratio; bond factor
β	angle; ratio; factor
γ	partial safety factor
∂	deflection; deformation
ε	strain
η	strength factor
θ	angle of compression strut; slope
λ	slenderness ratio; ratio
μ	coefficient of friction
ν	strength reduction factor for concrete
ρ	unit mass; reinforcement ratio
σ	normal stress; standard deviation
τ	shear stress
ϕ	rotation; slope; ratio; diameter of a reinforcing bar; creep coefficient
φ	factors defining representative values of variable actions

SUFFIXES

b	bond
c	concrete
d	design value
ef	effective
f	flange
i	initial
k	characteristic
l	longitudinal
lim	limit
m	mean
max	maximum
min	minimum
o	original
p	prestress

R	resistance
req	required
s	steel
sw	self weight
t	time; tensile; transfer
u	ultimate
v	shear
w	web; wires; shear reinforcement
y	yield
x, y, z	axes

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Chapter 1 / General

1.1 Description of Concrete Structures

1.1.1 Types of Load Bearing Concrete Structures

The development of reinforced concrete, circa 1900, provided an additional building material to stone, brick, timber, wrought iron and cast iron. The advantages of reinforced concrete are cheapness of aggregates, flexibility of form, durability and low maintenance. A disadvantage is greater self weight as compared with steel, timber or aluminium.

Reinforced concrete structures include low rise and high rise buildings, bridges, towers, floors, foundations etc. The structures are essentially composed of load bearing frames and members which resist the actions imposed on the structure, e.g. self weight, dead loads and external imposed loads (wind, snow, traffic etc.).

Structures with load bearing frames may be classified as:

(a) Miscellaneous isolated simple structural elements, e.g. beams and columns or simple groups of elements, e.g. floors.
(b) Bridgeworks.
(c) Single storey factory units, e.g. portal frames.
(d) Multi-storey units, e.g. tower blocks.
(e) Shear walls, foundations and retaining walls.

Two typical load bearing frames are shown in Fig. 1.1. When subject to lateral loading, some frames (Fig. 1.1(a)) deflect and are called sway, or unbraced, frames. Others (Fig. 1.1(b)) however are stiffened, e.g. by a lift shaft and are called non-sway, or braced, frames. This distinction is important when analysing frames as shown in Chapters 4 and 9. For analysis purposes, frames are idealized and shown as a series of centre lines (Fig. 1.1(c)). Elements of a structure are defined in cl. 5.3.1, EN.

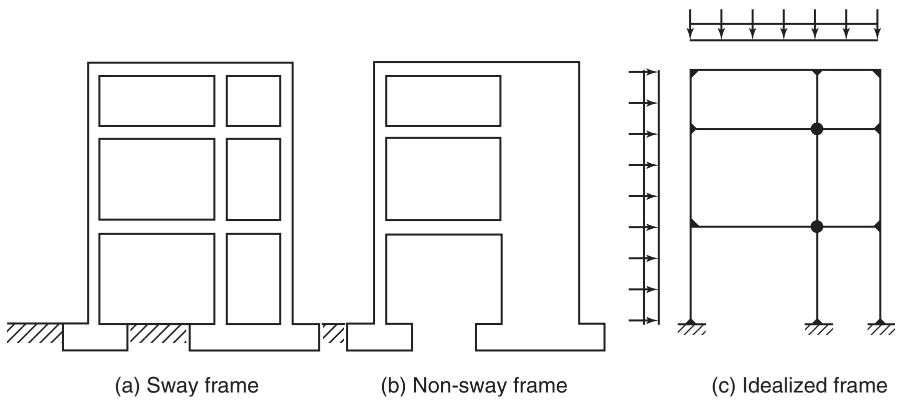

FIGURE 1.1 Typical load bearing frames

1.1.2 Load Bearing Members

A load bearing frame is composed of load bearing members (or elements) e.g. beams and columns. Structural elements are required to resist forces and displacements in a variety of ways, and may act in tension, compression, flexure, shear, torsion, or in any combination of these forces. The structural behaviour of a reinforced concrete element depends on the nature of the forces, the length and shape of the cross section of the member, elastic and plastic properties of the materials, yield strength of the steel, crushing strength of the concrete and crack widths. Modes of behaviour of structural elements are considered in detail in the following chapters.

A particular advantage of reinforced concrete is that a variety of reinforced concrete sections (Fig. 1.2) can be manufactured to resist combinations of forces (actions).

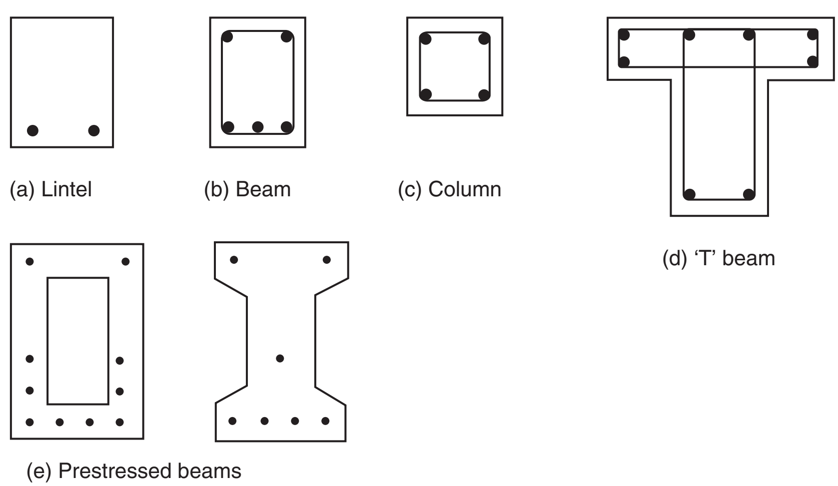

FIGURE 1.2 Typical reinforced concrete sections

The optimization of costs in reinforced construction favours the repeated use of moulds to produce reinforced concrete elements with the same cross sections and lengths. These members are common for use in floors either reinforced or prestressed. The particular advantage of precast concrete units is that the concrete is matured and will take the full loading at erection. The disadvantage, in some situations, is the difficulty of making connections.

1.1.3 Connections

The structural elements are made to act as a frame by connections. For reinforced concrete, these are composed of reinforcing bars bent, lapped or welded which are arranged to resist the forces involved. A connection may be subject to any combination of axial force, shear force and bending moment in relation to three perpendicular axes, but for simplicity, where appropriate, the situation is reduced to forces in one plane. The transfer of forces through the components of a connection is often complex and Chapter 8 contains explanations, research references and typical design examples.

There are other types of joints in structures which are not structural connections. For example, a movement joint is introduced into a structure to take up the free expansion and contraction that may occur on either side of the joint due to temperature, shrinkage, expansion, creep, settlement, etc. These joints may be detailed to be watertight but do not generally transmit forces. Detailed recommendations are given by Alexander and Lawson (1981). A construction joint is introduced because components are manufactured to a convenient size for transportation and need to be connected together on site. In some cases these joints transmit forces but in other situations may only need to be waterproof.

1.2 Development and Manufacture of Reinforced Concrete

1.2.1 Outline of the Development of Reinforced Concrete

Concrete is a mixture of cement, fine aggregate, coarse aggregate and water. It is thus a relative cheap structural material but it took years to develop the binding material which is the cement. Modern cement is a mixture of calcareous (limestone or chalk) and argillaceous (clay) material burnt at a clinkering temperature and ground to a fine powder.

The Romans used lime mortar and added crushed stone or tiles to form a weak concrete. Lime mortar does not harden under water and the roman solution was to grind together lime and volcanic ash. This produced pozzolanic cement named after a village Pozzuoli near Mount Vesuvius.

The impetus to improve cement was the industrial revolution of the eighteenth century. John Smeaton, when building Eddystone lighthouse, made experiments with various mixes of lime, clay and pozzolana. In 1824, a patent was granted to Joseph Aspin for a cement but he did not specify the proportions of limestone and clay, nor did he fire it to such a high temperature as modern cement. However it was used extensively by Brunel in 1828 for the Thames tunnel. Modern cement is based on the work of Johnson who was granted a patent in 1872. He experimented with different mixes of clay and chalk, fired at different temperatures and ground the resulting clinker.

The mass production of cement was made possible by the invention of the rotary kiln by Crampton in 1877. The control, quality and reliability of the cement improved over the years and the proportions of fine and coarse aggregate required for strong durable concrete have been optimized. This lead to the development of prestressed concrete using high strength concrete. Most reinforced concrete construction uses timber as shuttering but where members are repeated, e.g. floor beams, steel is preferred because it is more robust.

Wilkinson in 1854 took out a patent for concrete reinforced with wire rope. Early investigators were concerned with end anchorage but eventually continuous bond between the steel and the concrete was considered adequate. Later, round mild steel bars with end hooks were standardized with a yield strength of approximately 200 MPa which gradually increased until today 500 MPa is common. From 1855 reinforced concrete research, construction and theory was developed, notably in Germany, and results were published by Morsch in 1902.

To overcome the problem of cracking at service load, prestressed concrete was developed by Freyssinet in 1928. Mild steel was not suitable for prestressed concrete because all the prestress was lost but high strength piano wires (700 MPa) were found to be suitable. Initially the post-tensioned system was adopted where the concrete was cast, allowed to mature and the wires, free to move in ducts, were stressed and anchored at the ends of the member. Later the pretensioned system was developed where spaced single wires were prestressed between end frames and the concrete cast round them. When the concrete had matured, the stress was released and the force transferred to the concrete by bond between the steel and concrete.

1.2.2 Modern Method of Concrete Production

The ingredients of concrete, i.e. water, cement, fine aggregate and coarse aggregate are mixed to satisfy the requirements of strength, workability, durability and economy. The water content is the most important factor which influences the workability of the mix, while the water/cement ratio influences the strength and durability. The cement paste (water and cement) should fill the voids in the fine aggregate and the mortar (water, cement and sand) should fill the voids in the coarse aggregate. To optimize this process, the aggregates are graded. Further understanding of the process can be obtained from methods of mix design (Teychenne *et al.*, 1975).

The cement production is carefully controlled and tested to produce a consistent product. The ingredients (lime, silica, alumina and iron) are mixed (puddled) and fed into a rotary kiln in a continuous process to produce a finely ground clinker. The chemistry of cement is complicated and not fully understood. For practical purposes the end product is recognized as either ordinary Portland cement or rapid-hardening Portland cement.

Fine aggregates (sand) are less than 5 mm, while coarse aggregates (crushed stone) range from 5 to 40 mm depending on the dimensions of the member. The aggregates are generally excavated from quarries, washed, mixed to form concrete and, for large projects, transported to the construction site. For small projects, the concrete may be mixed on site from stockpiled materials. Some manufactured aggregates are available, including light-weight aggregates, but they are not often used.

1.3 Development and Manufacture of Steel

Steel was first produced in 1740 but was not available in large quantities until Bessemer invented the converter in 1856. By 1840, standard shapes in wrought iron were in regular production. Gradually wrought iron was refined to control its composition and remove impurities to produce steel. Further information on the history of steel making can be found in Buchanan (1972); Cossons (1975); Derry (1960); Pannel (1964) and Rolt (1970).

Currently there are two methods of steel making:

(a) Basic Oxygen Steelmaking (BOS) used for large scale production. Iron ore is smelted in a blast furnace to produce pig iron. The pig iron is transferred to a converter where it is blasted with oxygen and impurities are removed to produce steel. Some scrap metal may be used.
(b) Electric Arc Furnace (EAF) used for small scale production. The method, uses scrap metal almost entirely which is fed into the furnace and heated by means of an electrical discharge from carbon electrodes. Little refining is required.

In both processes carbon, manganese and silicon in particular, are controlled. Other elements which may affect coldworking, weldability and bendability may also be limited. BOS steel has lower levels of sulphur, phosphorous and nitrogen while EAF steel has higher levels of copper, nickel and tin.

Traditionally steel was cast into ingots but impurities segregated to the top of the ingot which had to be removed. Ingots have been replaced by a continuous casting process to produce a water cooled billet. The billet is reheated to a temperature of approximately 1150°C and progressively rolled through a mill to reduce its section. This process also removes defects, homogenizes the steel and forms ribs to improve the bond. With the addition of tempering by controlled quenching it is possible to produce a relatively soft ductile core with a hard surface layer (Cares, 2004).

1.4 STRUCTURAL DESIGN

1.4.1 Initiation of a Design

The demand for a structure originates with the client. The client may be a private person, private or public firm, local or national government or a nationalized industry.

In the first stage, preliminary drawings and estimates of costs are produced, followed by consideration of which structural materials to use, i.e. reinforced concrete, steel, timber, brickwork, etc. If the structure is a building, an architect only may be involved at this stage, but if the structure is a bridge or industrial building then a civil or structural engineer prepares the documents.

If the client is satisfied with the layout and estimated costs then detailed design calculations, drawings and costs are prepared and incorporated in a legal contract document. The design documents should be adequate to detail, fabricate and erect the structure.

The contract document is usually prepared by the consultant engineer and work is carried out by a contractor who is supervised by the consultant engineer. However, larger firms, local and national government and nationalized industries, generally employ their own consultant engineer.

The work is generally carried out by a contractor, but alternatively direct labour may be used. A further alternative is for the contractor to produce a design and construct package, where the contractor is responsible for all parts and stages of the work.

1.4.2 The Object of Structural Design (cl. 2.1 EN, Section 2 EN 1990)

The object of structural design is to produce a structure that will not become unserviceable or collapse in its lifetime, and which fulfils the requirements of the client and user at reasonable cost.

The requirements of the client and user may include the following:

(a) The structure should not collapse locally or overall.
(b) It should not be so flexible that deformations under load are unsightly or alarming, or cause damage to the internal partitions and fixtures; neither should any movement due to live loads, such as wind, cause discomfort or alarm to the occupants/users.
(c) It should not require excessive repair or maintenance due to accidental overload or because of the action of weather.
(d) In the case of a building, the structure should be sufficiently fire resistant to, give the occupants time to escape, enable the fire brigade to fight the fire in safety, and to restrict the spread of fire to adjacent structures.
(e) The working life of the structure should be acceptable (generally varies between 10 and 120 years).

The designer should be conscious of the costs involved which include:

(a) Initial cost which includes fees, site preparation, cost of materials and construction.
(b) Maintenance costs, e.g. decoration and structural repair.
(c) Insurance chiefly against fire damage.
(d) Eventual demolition.

It is the responsibility of the structural engineer to design a structure that is safe and which conforms to the requirements of the local bye-laws and building regulations. Information and methods of design are obtained from Standards and Codes of Practice and these are "deemed to satisfy" the local bye-laws and building regulations. In exceptional circumstances, e.g. the use of methods validated by research or testing, an alternative design may be accepted.

A structural engineer is expected to keep up to date with the latest research information. In the event of a collapse or malfunction where it can be shown that the engineer has failed to reasonably anticipate the cause or action leading to collapse or has failed to apply properly the information at his disposal, i.e. Codes of Practice, British Standards, Building Regulations, research or information supplied by the manufactures, then he may be sued for professional negligence. Consultants and contractors carry liability insurance to mitigate the effects of such legal action.

1.4.3 Statistical Basis for Design

When a material such as concrete is manufactured to a specified mean strength, tests on samples show that the actual strength deviates from the mean strength to a varying degree depending on how closely the process is controlled and on the

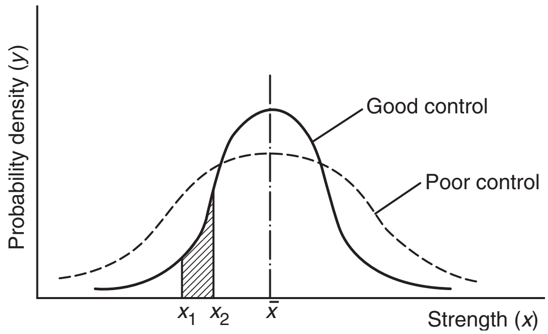

FIGURE 1.3 Normal distribution curve

variation in strength of the component materials. It is found that the spread of results approximates to a normal distribution curve, as shown in Fig. 1.3.

The probability of a test result falling between two values of strength such as x_1 and x_2 is given by the area under the curve between the two values, i.e. the shaded area in the figure. The area under the whole curve is thus equal to unity.

The effect of quality control is as shown, poor control producing a flatter curve. The probability of a test falling below a specified value is clearly greater when the quality of control is reduced.

The equation of the normal distribution curve is

$$y = 1/[\sigma(2\pi)^{0,5}]\exp[-(x-\bar{x})^2/(2\sigma^2)] \tag{1.1}$$

which shows that the curve is fully defined by the mean $\bar{x}$ and the standard deviation σ of the variable x.

For a set of n values of x, the mean is given by

$$\bar{x} = \Sigma x/n \tag{1.2}$$

The standard deviation which is a measure of the dispersion, is the root mean square of the deviations of x from the mean, given by

$$\sigma = [\Sigma(x-\bar{x})^2/n]^{0,5} \tag{1.3}$$

In practice, it is not usually possible to obtain all of the values of x that would theoretically be available. For example, it would not be possible to test all the concrete in a structure and it is therefore necessary to obtain estimates of x and σ by sampling. In this case, the best estimate of the mean is still that given by Eq. (1.2), but the best estimation for the standard deviation is given by

$$\sigma = [\Sigma(x-\bar{x})^2/(n-1)]^{0,5} \tag{1.4}$$

where n is the number of test results in the sample.

For hand calculations, a more convenient form of Eq. (1.4) is

$$\sigma = [(\Sigma x^2 - n\bar{x}^2)/(n-1)]^{0,5} \tag{1.5}$$

Statistical distributions can also be obtained to show the variation in strength of other structural materials such as steel reinforcement and prestressing tendons. It is also reasonable to presume that if sufficient statistical data were available, distributions could be defined for the loads carried by a structure. It follows that it is impossible to predict with certainty that the strength of a structural member will always be greater than the load applied to it or that failure will not occur in some other way during the life of the structure. The philosophy of limit state design is to establish limits, based on statistical data, experimental results and engineering experience and judgement, that will ensure an acceptably low probability of failure. At present there is insufficient information to enable distributions of all the structural variables to be defined and it is unlikely that is will ever be possible to formulate general rules for the construction of a statistical model of anything so complicated as an actual structure.

1.4.4 Limit State Design (cl. 2.2 EN)

It is self evident that a structure should be 'safe' during its lifetime, i.e. free from the risk of collapse. There are, however, other risks associated with a structure and the term safe is now replaced by the term 'serviceable'. A structure should not during its lifetime become 'unserviceable', i.e. it should be free from risk of collapse, rapid deterioration, fire, cracking, excessive deflection etc.

Ideally it should be possible to calculate mathematically the risk involved in structural safety based on the variation in strengths of the material and variation in the loads. Reports, such as the CIRIA Report 63, have introduced the designer to elegant and powerful concept of 'structural reliability'. Methods have been devised whereby engineering judgement and experience can be combined with statistical analysis for the rational computation of partial safety factors in codes of practice. However, in the absence of complete understanding and data concerning aspects of structural behaviour, absolute values of reliability cannot be determined.

It is not practical, nor is it economically possible, to design a structure that will never fail. It is always possible that the structure will contain material that is less than the required strength or that it will be subject to loads greater than the design loads.

It is therefore accepted that 5 per cent of the material in a structure is below the design strength, and that 5 per cent of the applied loads are greater than the design loads. This does not mean therefore that collapse is inevitable, because it is extremely unlikely that the weak material and overloading will combine simultaneously to produce collapse.

The philosophy and objectives must be translated into a tangible form using calculations. A structure should be designed to be safe under all conditions of its useful life and to ensure that this is accomplished certain distinct performance requirements, called 'limit states', have been identified. The method of limit state design recognizes the variability of loads, materials construction methods and approximations in the theory and calculations (BS EN 1990).

Limit states may be at any stage of the life of a structure, or at any stage of loading. The limit states which are important for the design of reinforced concrete are at ultimate and serviceability (cl 2.2 EN). Calculations for limit states involve loads and load factors (Chapter 3), and material factors and strengths (Chapter 2).

Stability, an ultimate limit state, is the ability of a structure or part of a structure, to resist overturning, overall failure and sway. Calculations should consider the worst realistic combination of loads at all stages of construction.

All structures, and parts of structures, should be capable of resisting sway forces, e.g. by the use of bracing, 'rigid' joints or shear walls. Sway forces arise from horizontal loads, e.g. winds, and also from practical imperfections, e.g. lack of verticality.

Also involved in limit state design is the concept of structural integrity. Essentially this means that the structure should be tied together as a whole, but if damage occurs, it should be localized. This was illustrated in a tower block (Canning Town Report, 1968) when a gas explosion in one flat caused the progressive collapse of other flats on one corner of the building.

Deflection is a serviceability limit state. Deflections should not impair the efficiency of a structure or its components, nor cause damage to the finishes. Generally the worst realistic combination of unfactored imposed loads is used to calculate elastic deflections. These values are compared with limit states of deformation (cl. 7.4, EN).

Dynamic effects to be considered at the serviceability limit state are vibrations caused by machines, and oscillations caused by harmonic resonance, e.g. wind gusts on buildings. The natural frequency of the building should be different from the exciting source to avoid resonance.

Fortunately there are few structural failures and when they do occur they are often associated with human error involved in design calculations, or construction, or in the use of the structure.

1.4.5 Errors

The consequences of an error in structural design can lead to loss of life and damage to property and it is necessary to appreciate where errors can occur. Small errors in design calculations can occur in the rounding off of figures but these generally do not lead to failures. The common sense advice is that the accuracy of the calculation should match the accuracy of the values given in the European Code.

Errors that can occur in structural design calculations are:

(1) Ignorance of the physical behaviour of the structure under load which introduces errors in the basic theoretical assumptions.
(2) Errors in estimating the loads, especially the erection forces.
(3) Numerical errors in the calculations. These should be eliminated by checking, but when speed is paramount, checks are often ignored.
(4) Ignorance of the significance of certain effects, e.g. creep, fatigue, etc.
(5) Introduction of new materials or methods, which have not been tested.
(6) Insufficient allowance for tolerances or temperature strains.
(7) Insufficient information, e.g. in erection procedures.

Errors that can occur on construction sites are:

(1) Using the wrong number, diameter, cover and spacing of bars.
(2) Using the wrong or poor mix of concrete.
(3) Errors in manufacture, e.g. holes in the wrong position.

Errors that occur in the life of a structure and also affect safety are:

(1) Overloading.
(2) Removal of structural material, e.g. to insert service ducts.
(3) Poor maintenance.

1.5 Production of Reinforced Concrete Structures

1.5.1 Drawings

Detailed design calculations are essential for any reinforced concrete design but the sizes of the members, dimensions and geometrical arrangement are usually presented as drawings. The drawings are used by the contractor on site, but if there are precast units, the drawing for these may be required for a subcontractor. General arrangement drawings are often drawn to scale of 1:100, while details are drawn to a scale of 1:20 or 1:10. Special details are drawn to larger scales where necessary.

Drawing should be easy to read and should not include superfluous detail. Some important notes are:

(a) Members and components should be identified by logically related mark numbers, e.g. related to the grid system used in the drawings.
(b) The main members should be presented by a bold outline (0,4 mm wide) and dimension lines should be unobtrusive (0,1 mm wide).

(c) Dimensions should be related to centre lines, or from one end; strings of dimensions should be avoided. Dimensions should appear once only so that ambiguity cannot arise when revisions occur. Fabricators should not be put in the position of having to do arithmetic in order to obtain an essential dimension.
(d) Tolerances for erection purposes should be clearly shown.
(e) The grade of steel and concrete to be used should be clearly indicated.
(f) Detailing should take account of possible variations due to fabrication.
(g) Keep the design and construction as simple as possible and avoid changes in section along the length of a member.
(h) Site access, transport and available cranage should be considered.

Reinforcement drawings are prepared primarily for the steel fixer and should conform to the standard method of detailing reinforced concrete (Conc. Soc. and I.S.E report, 1973). The main points to be noted are:

(1) The outline of the concrete should be slightly finer than the reinforcement.
(2) In walls, slabs and columns a series of bars of a particular mark is indicated by the end bars and only one bar in the series is shown in full; the other bar is shown as a short line. In Fig. 1.4(a), for example, the reinforcement consists of two series of bars forming a rectangular grid in plan.
(3) Each series is identified by a code which has the following form: Number, type, diameter-bar mark-spacing, comment. For example in Fig. 1.4(a) 25T20-1-250 indicates that the series contains 25 bars in high yields steel (T), 20 mm diameter, with bar mark 1, spaced at 250 mm between centres. The complete code is used only once for a particular series and wherever possible should appear in the plan or elevation of the member. In sections the bars are identified simply by their bar mark; and again only the end bars need to be shown.
(4) Since the bars are delivered on site bent to the correct radii, bends do not need to be detailed and may be drawn as sharp angles.
(5) Dimensions should be in mm rounded to a multiple of 5 mm. There is no need to write mm on the drawing.
(6) Except with the case of very simple structures the dimensions should refer only to the reinforcement and should be given when the steel fixer could not reasonably be expected to locate the reinforcement properly without them. In many cases, since the bars are supplied to the correct length and shape, no dimensions are necessary, e.g. Fig. 1.4(a). Dimensions should be given from some existing reference point, preferably the face of concrete already been cast. For example, in Fig. 1.4(b) the dimension to the first link is given from the surface of the kicker, which would have been cast with the floor slab and is used to locate the column to be constructed. The formwork for the column is attached to the kicker and the vertical reinforcement starts from the kicker.

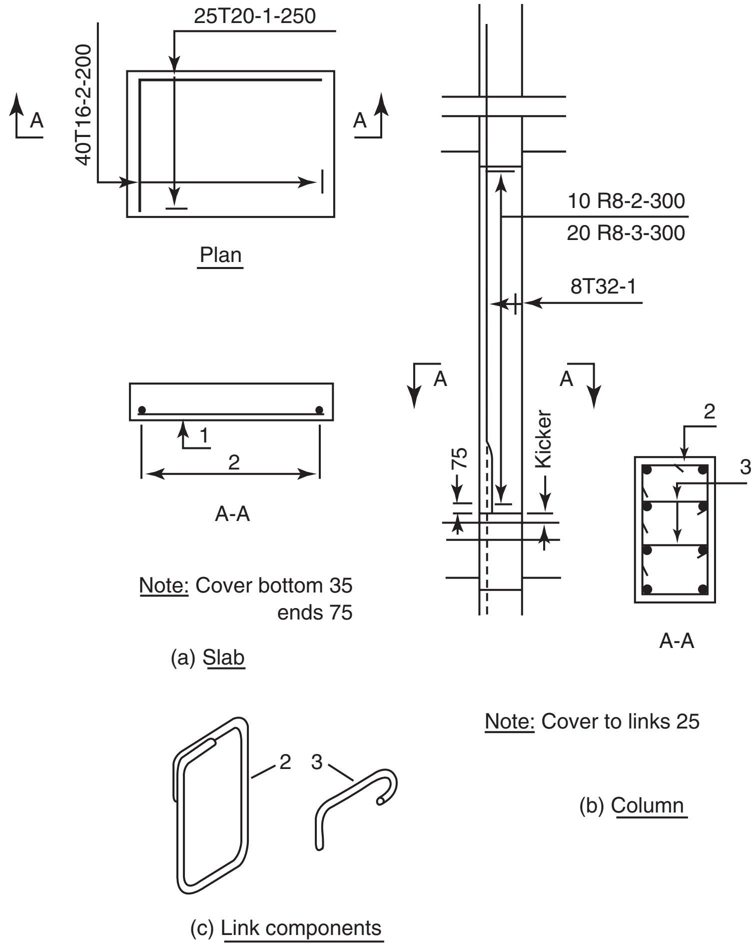

FIGURE 1.4 Reinforcement of slab and column

(7) Cover to the ends and sides of the bars is usually given in the form of notes on the drawing.
(8) Only part of the adjoining members is shown and in sections, only the outlines of the concrete which has been cut through are shown; any concrete beyond the section is generally omitted. The reinforcement in adjoining members is also omitted except where it is necessary to show the relative positions of intersecting or lapping bars. In such cases it is shown as broken lines. In Fig. 1.4(b), for example, it can be seen that the reinforcement of the lower column is to be left protruding from the kicker to form the lap with the reinforcement of the column to be constructed. Bars with diameters less than or equal to 12 mm need not be cranked at the laps.
(9) Sections are drawn to show the relative positions of bars and the shape of links. In the case of beams and columns they are usually drawn to a larger scale than the elevation.

(10) The ends of overlapping bars in the same plane are shown as ticks. For example the conventional representation of the links is shown in the section of the column in Fig. 1.4(b). The actual shapes of the link components are shown in Fig. 1.4(c). There is no really any need to indicate overlapping in links except when the separate components are to be fixed on site, as in this case. Where links are supplied in one piece, as for the beam in Fig. 1.5, the ticks are frequently omitted.

(11) Section arrows for beams and columns should always be in the same direction, i.e. left facing for beams, downwards for columns. The section letters (numbers are not recommended) should always be between the arrows and should be written in the upright position.

(12) When detailing beams all the bars are shown in full. In elevations, the start and finish of the bars in the same place are indicated by ticks. The start is indicated by the full bar code, the end simply by the bar mark (Fig. 1.5) which shows the elevation and sections of part of a beam. Note that the reinforcement in the adjoining column and integral floor slab is not shown; separate drawings would be provided for these. Note also that the dimension

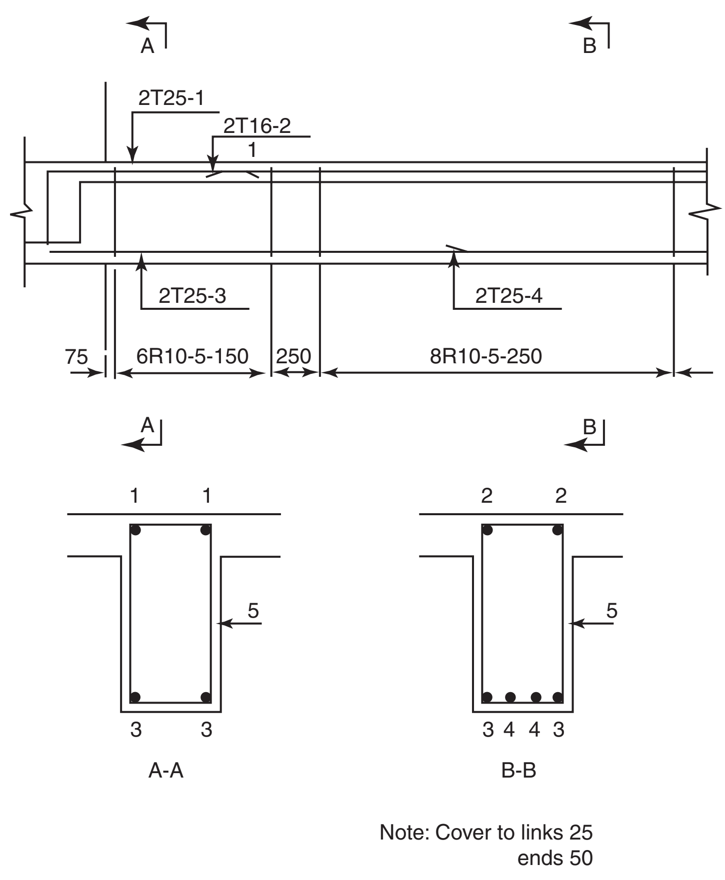

FIGURE 1.5 Reinforcement of a beam

of the start of the first series of links is given from the face of the column already constructed and that a dimension is given between each series of links.

(13) Although the reinforcement in adjoining members is not shown, it is important for the designer to ensure that it will not obstruct the reinforcement of the detailed member. Thus it is especially important in beam–column construction to ensure that the reinforcements of a beam and a column do not intersect in the same plane.

1.5.2 Bar Bending Schedules

Scheduling, dimensioning, bending and cutting of steel reinforcement for concrete is given in BS 8666 (2000). Bars should be designed to have as few bends as possible and should conform to the preferred shapes.

Work on the reinforcement, i.e. cutting, bending and fixing in place before the concrete is placed, is usually dealt with by specialist sub-contractors. Information on reinforcement is conveyed from the engineer to the sub-contractor by use of a bar bending schedule, which is a table which provides the following information about each bar.

Member. A reference identifying a particular structural member, or group of identical members.
Bar mark. An identifying number which is unique to each bar in the schedule.
Type and size. A code letter: 'T' for high yield steel, 'R' for mild steel following by size of bar in mm, e.g. R16 denotes a mild steel bar 16 mm in diameter.
Number of members. The number of identical members in a group.
Number of bars in each.
Total number.
Length of each bar (mm).
Shape code.
Dimensions required for bending. Five columns specifying the standard dimensions corresponding to the particular shape code. These dimensions contain allowances for tolerances. If a bar does not conform to one of the preferred shapes then a dimensioned drawing is supplied.

Bars are delivered to the site in bundles, each of which is labelled with the reference number of the bar schedule and the bar mark. These two numbers uniquely identify every bar in the structure.

1.5.3 Tolerances (cl. 4.4.1.3, EN)

Tolerance are limits placed on unintentional inaccuracies that occur in dimension which must be allowed for in design if structural elements and components are to resist forces and remain durable.

The formwork and falsework should be sufficiently stiff to ensure that the tolerance for the structure, as stipulated by the designer, are satisfied. Tolerance for members are related to controlling the error in the size of section, and vary from ± 5 mm for a 150 mm section to ± 30 mm for a 2500 mm section. Tolerances for position of prestressing tendons are related to the width of depth of a section.

The tolerances associated with concrete cover to reinforcement are important to maintain structural integrity and to resistance corrosion. The concrete cover is the distance between the surface of the reinforcement closest to the nearest concrete surface. The nominal cover is specified in drawings, which is the minimum cover c_{min} (cl 4.4.1.2, EN) plus an allowance for design deviation Δc_{dev} (cl 4.4.1.3, EN). The usual value of $\Delta c_{\mathrm{dev}} = 10$ mm.

1.6 Site Conditions

1.6.1 General

The drawings produced by the structural designer are used by the contractor on site. Most of the reinforced concrete is cast in situ, on site, and transport and access is generally no problem, except for the basic materials. Most concrete is ready mix obtained from specialist suppliers. This avoids having to provide space for storing cement and aggregates and ensures high quality concrete. Prefabricated products, e.g. floor beams are delivered by road and erected by crane. Most large sites have a crane available. On site, the general contractor is responsible for the assembly, erection, connections, alignment and leveling of the complete structure. During assembly on site it is inevitable that some components will not fit, despite the tolerances that have been allowed. The correction of some faults and the consequent litigation can be expensive.

1.6.2 Construction Rules

To ensure that the structure is constructed as specified by the designers, construction rules are introduced. For concrete, these cover quality of concrete, formwork, surface finish, temporary works, removal of formwork and falsework. For steelwork, the rules cover transport, storage, fabrication, welding, joints and placing. For prestressed concrete there are additional rules for tensioning, grouting and sealing.

1.6.3 Quality Control

Quality control is necessary to ensure that the construction work is carried out to the required standards and rules. Control covers the quality of materials, standard of workmanship and the quality of the components. For materials, it includes the mix of concrete, storage, handling, cutting, welding, prestressing forces and grouting (BS EN 206-1, 2000).

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Chapter 2 / Mechanical Properties of Reinforced Concrete

2.1 Variation of Material Properties

The properties of manufactured materials vary because the particles of the material are not uniform and because of inconsistencies in the manufacturing process which are dependent on the degree of control. These variations must be recognized and incorporated into the design process.

For reinforced concrete, the material property that is of most importance is strength. If a number of samples are tested for strength, and the number of specimens with the same strength (frequency) plotted against the strength, then the results approximately fit a normal distribution curve (Fig. 2.1). The equation shown in Fig. 2.1 defines the curve mathematically and can be used to define 'safe' values for design purposes.

2.2 Characteristic Strength

A strength to be used as a basis for design must be selected from the variation in values (Fig. 2.1). This strength, when defined, is called the characteristic strength. If the characteristic strength is defined as the mean strength, then 50 per cent of the material is below this value and this is unsafe. Ideally the characteristic strength should include 100 per cent of the samples, but this is impractical because it is a low value and results in heavy and costly structures. A risk is therefore accepted and it is therefore recognized that 5 per cent of the samples for strength fall below the characteristic strength. The characteristic strength is calculated from the equation

$$f_{ck} = f_{mean} - 1{,}64\ \sigma \tag{2.1a}$$

where for n samples the standard deviation

$$\sigma = [\Sigma(f_{mean} - f)^2/(n-1)]^{0,5} \tag{2.1b}$$

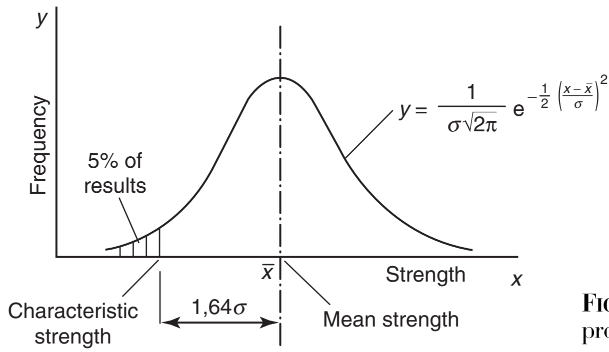

FIGURE 2.1 Variation in material properties

2.2.1 Characteristic Strength of Concrete (cl 3.1.2, EN)

Concrete is a composite material which consists of coarse aggregate, fine aggregate and a binding paste mixture of cement and water. Mix design selects the optimum proportions of these materials, but a simple way to assess the quality of concrete as it matures is to crush a standard cube or cylinder.

The mean strength which must be achieved in the mixing process can be determined if the standard deviation is known from previous experience. For example, to produce a concrete with a characteristic strength of 30 MPa in a plant for which a standard deviation of 5 MPa is expected, required mean strength $= 30 + 5 \times 1{,}64 = 38{,}2$ MPa.

The strength of concrete increases with age and it is necessary to adopt a time after casting as a standard. The characteristic compressive strength for concrete is the 28-day cylinder or cube strength, i.e. the crushing strength of a standard cylinder, or cube, cured under standard conditions for 28 days as described in BS EN 12390-1 (2000). The 28-day strength is approximately 80 per cent of the strength at one year, after which there is very little increase in strength. Strengths higher than the 28-day strength are not used for design unless there is evidence to justify the higher strength for a particular concrete.

The characteristic tensile strength of concrete is the uniaxial tensile strength ($f_{ct,ax}$) but in practice this value is difficult to obtain and the split cylinder strength ($f_{ct,sp}$) is more often used. The relation between the two values is ($f_{ct,ax}$) $= 0{,}9$ ($f_{ct,sp}$). In the absence of test values of the tensile strength it may be assumed that the mean value of the tensile strength (f_{ctm}) $= 0{,}3(f_{ck})^{2/3}$, where f_{ck} is the characteristic cylinder compressive strength. In situations where the tensile strength of concrete is critical, e.g. shear resistance, the mean value may be unsafe and a 5 per cent fractile ($f_{ctk0,05} = 0{,}7 f_{ctm}$) is used. In other situations where the tensile strength is not critical, a 95 per cent fractile ($f_{ctk0,95} = 1{,}3 f_{ctm}$) is used. These values are given in Table 3.1, EN (Annex A2).

BS EN 206-1 (2000) specifies the mix, transportation, sampling and testing of concrete. Concrete mixes are either prescribed, i.e. specified by mix proportions, or designed, i.e. specified by characteristic strength. For example, Grade C30P denotes a prescribed mix which would normally give a strength of 30 MPa; Grade C25/30 denotes a designed mix for which a cylinder strength of 25 MPa or cube strength of 30 MPa is guaranteed. The grades recommended by EN are from 12/15 to 50/60 in steps of approximately 5 MPa for normal weight aggregates. The lowest Grades recommended for prestressed concrete are C30 for post-tensioning and C40 for pretensioning.

Although the definition of characteristic strength theoretically allows that a random test result could have any value, however low, a practical specification for concrete in accordance with BS EN 206-1 (2000) would require both of the following conditions for compliance with the characteristic strength, thus excluding very low values:

(a) the average strength determined from any group of four consecutive test results exceeds the specified characteristic strength by 3 MPa for concretes of Grade C16/20 and higher, or 2 MPa for concretes of Grade C7,5–C15;
(b) The strength determined from any test result is not less than the specified characteristic strength minus 3 MPa for concretes of Grade C20 and above, or 2 MPa for concretes of Grade C7,5–C15.

Concrete not complying with the above conditions should be rejected.

2.2.2 Characteristic Strength of Steel (cls 3.2.3 and 3.3.3, EN)

The characteristic axial tensile yield strength of steel reinforcement (f_{yk}) is the yield stress for hot rolled steel and 0,2 per cent proof stress for steel with no pronounced yield stress. The value recommended in the European Code is: High yield steel (hot rolled or cold worked) 500 MPa. For prestressing tendons, the characteristic strength (f_{pk}), at 0.1 per cent proof stress, varies from 1000 to 2000 MPa depending on the size of tendon and type of steel.

2.3 Design Strength

The design strength allows for the reduction in strength between the laboratory and site. Laboratory samples of the material are processed under strictly controlled standard conditions. Conditions on site vary and in the case of concrete: segregation can occur while it is being transported; conditions for casting and compaction differ; there may be contamination by rain; and curing conditions vary especially in hot or cold weather.

The design strength of a material is therefore lower than the characteristic strength, and is obtained by dividing the characteristic strength by a partial safety factor (γ_m). The value chosen for a partial safety factor depends on the susceptibility of the material to variation in strength, e.g. steel reinforcement is less affected by site conditions than concrete.

2.3.1 Design Strength of Steel

The fundamental partial safety factor for steel $\gamma_s = 1{,}15$. For accidental loading, e.g. exceptional loads, fire or local damage, the value reduces to $\gamma_s = 1{,}0$. For earthquakes and fatigue conditions values are increased.

2.3.2 Design Strength of Concrete

The fundamental partial safety factor for concrete $\gamma_c = 1{,}5$. For accidental loading, the value reduces to $\gamma_c = 1{,}3$. For earthquakes and fatigue conditions, values are increased.

2.4 Stress–Strain Relationship for Steel (cl 3.2.7, EN)

A shape of the stress–strain curve for steel depends upon the type of steel and the treatment given to it during manufacture. For reinforcement steel, actual curves for short term tensile loading have the typical shapes shown in Fig. 2.2. Curves for compression are similar. See also BS 4449 (1997).

It can be seen (Fig. 2.2) that hot rolled reinforcing steel yields, or becomes significantly plastic, at stresses well below the failure stress and at strains well below the limiting strain for concrete (0,0035). In a reinforced concrete member, therefore, the steel reinforcement may undergo considerable plastic deformation before the Ultimate Limit State (ULS) is reached, but will not fracture. However large steel strains are accompanied by the formation of cracks in the concrete in the tensile zone, and these may become excessive and result in serviceability failure at loads below the (ULS).

For normal design purposes, the European Code recommends a single, idealized stress–strain curve (Fig. 2.3) for both hot rolled and cold worked reinforcement.

2.4.1 Modulus of Elasticity for Steel (cl 3.2.7(4), EN)

The modulus of elasticity for steel (E_s) is obtained from the linear part of the relationship between stress and strain (Fig. 2.2). This is a material property and values from a set of samples vary between 195 and 205 GPa. For design purposes, this variation is small and the European Code adopts a mean value of $E_s = 200$ GPa.

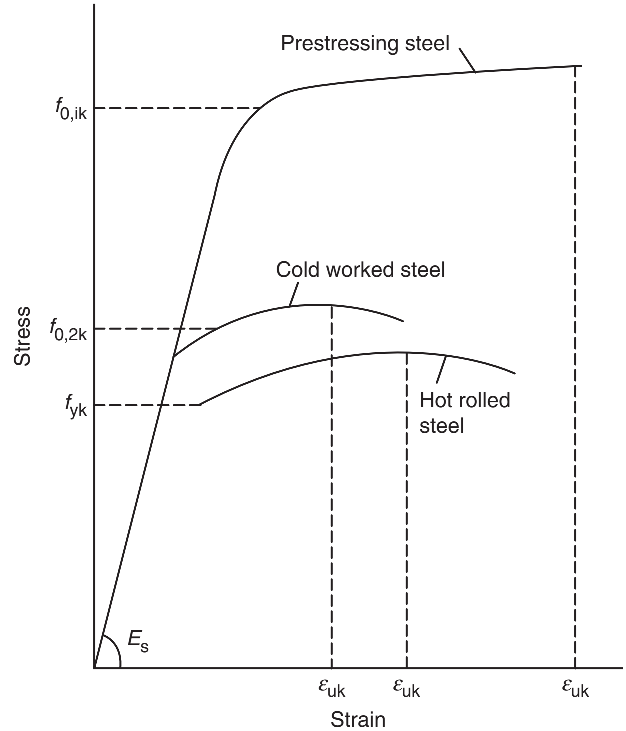

FIGURE 2.2 Typical stress–strain relationships for steel reinforcement

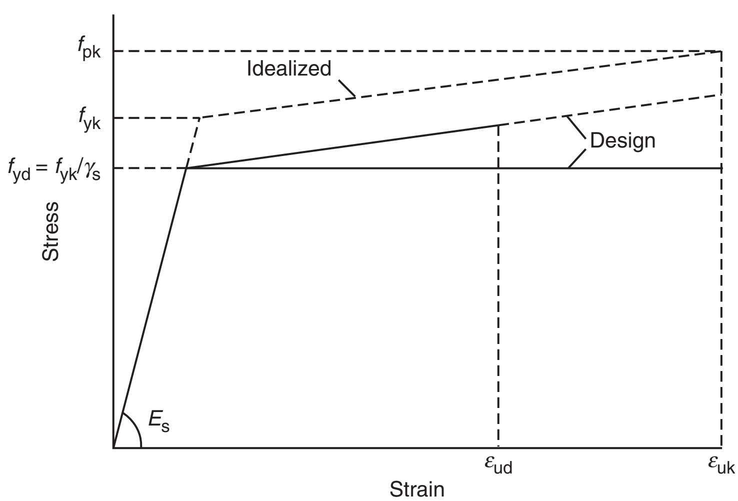

FIGURE 2.3 Design stress–strain relationship for steel reinforcement

The modulus of elasticity for prestressing steel varies with the type of steel from 175 to 195 GPa. Values of the moduli of elasticity are required in calculations involving deflections, loss of prestress and for the analysis of statically indeterminate structures.

2.5 Stress–Strain Relationship for Concrete (cl 3.1.5, EN)

A curve representing the stress–strain relationship for concrete is shown in Fig. 2.4. The important values of peak stress, peak strain and ultimate strain vary with the strength of concrete, rate of loading, age of the concrete, temperature and shrinkage. It is not easy to decide on design values that are safe and realistic.

The maximum stress (Fig. 2.4) is reached at a strain of approximately 0,002, after which the stress starts to fall. Disintegration of the concrete does not commence, however, until the strain reaches 0,0035, which is therefore taken as the limiting strain for concrete at the ultimate limit state. The maximum stress is the characteristic cylinder strength (f_{ck}).

For cross section design, the preferred idealization is shown in Fig. 2.5 and stress and strain values are given in Table 3.1, EN (Annex A2).

2.5.1 Modulus of Elasticity for Concrete (cl 3.1.3, EN)

The value of the modulus of elasticity is related to the type of aggregate and the strength of the concrete. Since the stress–strain curve for concrete is non-linear, a secant or static modulus is used. For normal weight concrete the modulus of elasticity

$$E_{cm} = 22\,(f_{cm}/10)^{0,3}\ \text{GPa} \tag{2.2}$$

where the mean cylinder compressive strength $f_{cm} = f_{ck} + 8\,\text{MPa}$.

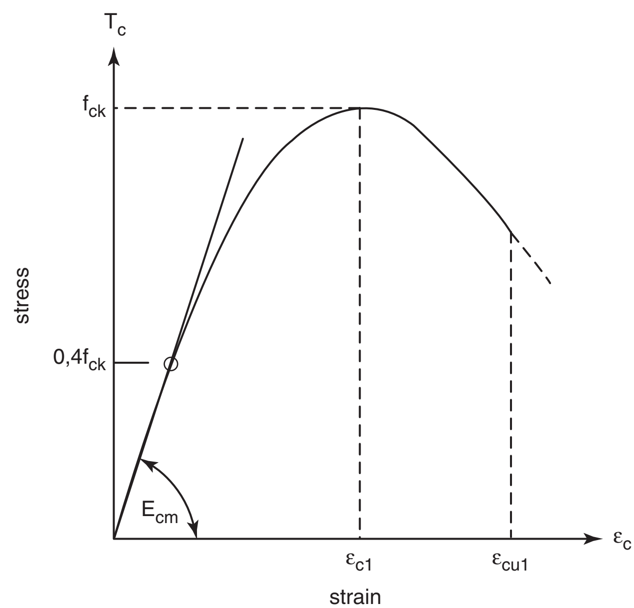

Figure 2.4 Stress–strain relationship for uniaxial compression of concrete