The Behaviour and Design of Steel Structures to EC3

Fourth Edition

N S Trahair, M A Bradford, D A Nethercot and L Gardner
The Behaviour and Design of Steel Structures to EC3
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This fourth British edition has been directed specifically to the design of steel structures in accordance with *Eurocode 3 Design of Steel Structures*. The principal part of this is *Part 1-1: General Rules and Rules for Buildings* and this is referred to generally in the text as EC3. Also referred to in the text are *Part 1-5: Plated Structural Elements*, and *Part 1-8: Design Of Joints*, which are referred to as EC3-1-5 and EC3-1-8. EC3 will be accompanied by National Annexes which will contain any National Determined Parameters for the United Kingdom which differ from the recommendations given in EC3.

Designers who have previously used BS5950 (which is discussed in the third British edition of this book) will see a number of significant differences in EC3. One of the more obvious is the notation. The notation in this book has been changed generally so that it is consistent with EC3.

Another significant difference is the general absence of tables of values computed from the basic design equations which might be used to facilitate manual design. Some designers will want to prepare their own tables, but in some cases, the complexities of the basic equations are such that computer programs are required for efficient design. This is especially the case for members under combined compression and bending, which are discussed in Chapter 7. However, the examples in this book are worked in full and do not rely on such design aids.

EC3 does not provide approximations for calculating the lateral buckling resistances of beams, but instead expects the designer to be able to determine the elastic buckling moment to be used in the design equations. Additional information to assist designers in this determination has been given in Chapter 6 of this book. EC3 also expects the designer to be able to determine the elastic buckling loads of compression members. The additional information given in Chapter 3 has been retained to assist designers in the calculation of the elastic buckling loads.

EC3 provides elementary rules for the design of members in torsion. These are generalised and extended in Chapter 10, which contains a general treatment of torsion together with a number of design aids.

The preparation of this fourth British edition has provided an opportunity to revise the text generally to incorporate the results of recent findings and research. This is in accordance with the principal objective of the book, to provide students and practising engineers with an understanding of the relationships between structural behaviour and the design criteria implied by the rules of design codes such as EC3.

N.S. Trahair, M.A. Bradford, D.A. Nethercot, and L. Gardner

April 2007
Units and conversion factors

Units

While most expressions and equations used in this book are arranged so that they are non-dimensional, there are a number of exceptions. In all of these, SI units are used which are derived from the basic units of kilogram (kg) for mass, metre (m) for length, and second (s) for time.

The SI unit of force is the newton (N), which is the force which causes a mass of 1 kg to have an acceleration of 1 m/s$^2$. The acceleration due to gravity is 9.807 m/s$^2$ approximately, and so the weight of a mass of 1 kg is 9.807 N.

The SI unit of stress is the pascal (Pa), which is the average stress exerted by a force of 1 N on an area of 1 m$^2$. The pascal is too small to be convenient in structural engineering, and it is common practice to use either the megapascal (1 MPa = 10$^6$ Pa) or the identical newton per square millimetre (1 N/mm$^2$ = 10$^6$ Pa). The newton per square millimetre (N/mm$^2$) is used generally in this book.

Table of conversion factors

<table>
<thead>
<tr>
<th>To Imperial (British) units</th>
<th>To SI units</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 kg</td>
<td>0.06853 slug</td>
</tr>
<tr>
<td>1 m</td>
<td>3.281 ft</td>
</tr>
<tr>
<td></td>
<td>39.37 in.</td>
</tr>
<tr>
<td>1 mm</td>
<td>0.003281 ft</td>
</tr>
<tr>
<td></td>
<td>0.03937 in.</td>
</tr>
<tr>
<td>1 N</td>
<td>0.2248 lb</td>
</tr>
<tr>
<td>1 kN</td>
<td>0.2248 kip</td>
</tr>
<tr>
<td></td>
<td>0.10036 ton</td>
</tr>
<tr>
<td>1 N/mm$^2$†</td>
<td>0.145 0 kip/in.$^2$ (ksi)</td>
</tr>
<tr>
<td></td>
<td>0.06475 ton/in.$^2$</td>
</tr>
<tr>
<td>1 kNm</td>
<td>0.7376 kip ft</td>
</tr>
<tr>
<td></td>
<td>0.3293 ton ft</td>
</tr>
</tbody>
</table>

Notes
- *1 N/mm$^2$ = 1 MPa.
- † There are some dimensionally inconsistent equations used in this book which arise because a numerical value (in N/mm$^2$) is substituted for the Young’s modulus of elasticity $E$ while the yield stress $f_y$ remains algebraic. The value of the yield stress $f_y$ used in these equations should therefore be expressed in N/mm$^2$. Care should be used in converting these equations from SI to Imperial units.
Glossary of terms

**Actions**  The loads to which a structure is subjected.

**Advanced analysis**  An analysis which takes account of second-order effects, inelastic behaviour, residual stresses, and geometrical imperfections.

**Beam**  A member which supports transverse loads or moments only.

**Beam-column**  A member which supports transverse loads or moments which cause bending and axial loads which cause compression.

**Biaxial bending**  The general state of a member which is subjected to bending actions in both principal planes together with axial compression and torsion actions.

**Brittle fracture**  A mode of failure under a tension action in which fracture occurs without yielding.

**Buckling**  A mode of failure in which there is a sudden deformation in a direction or plane normal to that of the loads or moments acting.

**Buckling length**  The length of an equivalent simply supported member which has the same elastic buckling load as the actual member.

**Cleat**  A short-length component (often of angle cross-section) used in a connection.

**Column**  A member which supports axial compression loads.

**Compact section**  A section capable of reaching the full plastic moment. Referred to in EC3 as a Class 2 section.

**Component method of design**  A method of joint design in which the behaviour of the joint is synthesised from the characteristics of its components.

**Connection**  A joint.

**Dead load**  The weight of all permanent construction. Referred to in EC3 as permanent load.

**Deformation capacity**  A measure of the ability of a structure to deform as a plastic collapse mechanism develops without otherwise failing.

**Design load**  A combination of factored nominal loads which the structure is required to resist.

**Design resistance**  The capacity of the structure or element to resist the design load.
Distortion  A mode of deformation in which the cross-section of a member changes shape.

Effective length  The length of an equivalent simply supported member which has the same elastic buckling load as the actual member. Referred to in EC3 as the buckling length.

Effective width  That portion of the width of a flat plate which has a non-uniform stress distribution (caused by local buckling or shear lag) which may be considered as fully effective when the non-uniformity of the stress distribution is ignored.

Elastic buckling analysis  An analysis of the elastic buckling of the member or frame out of the plane of loading.

Elastic buckling load  The load at elastic buckling. Referred to in EC3 as the elastic critical buckling load.

Elastic buckling stress  The maximum stress at elastic buckling. Referred to in EC3 as the elastic critical buckling stress.

Factor of safety  The factor by which the strength is divided to obtain the working load capacity and the maximum permissible stress.

Fastener  A bolt, pin, rivet, or weld used in a connection.

Fatigue  A mode of failure in which a member fractures after many applications of load.

First-order analysis  An analysis in which equilibrium is formulated for the undeformed position of the structure, so that the moments caused by products of the loads and deflections are ignored.

Flexural buckling  A mode of buckling in which a member deflects.

Flexural–torsional buckling  A mode of buckling in which a member deflects and twists. Referred to in EC3 as torsional–flexural buckling or lateral–torsional buckling.

Friction-grip joint  A joint in which forces are transferred by friction forces generated between plates by clamping them together with preloaded high-strength bolts.

Geometrical imperfection  Initial crookedness or twist of a member.

Girt  A horizontal member between columns which supports wall sheeting.

Gusset  A short-plate element used in a connection.

Imposed load  The load assumed to act as a result of the use of the structure, but excluding wind load.

Inelastic behaviour  Deformations accompanied by yielding.

In-plane behaviour  The behaviour of a member which deforms only in the plane of the applied loads.

Joint  The means by which members are connected together and through which forces and moments are transmitted.

Lateral buckling  Flexural–torsional buckling of a beam. Referred to in EC3 as lateral–torsional buckling.

Limit states design  A method of design in which the performance of the structure is assessed by comparison with a number of limiting conditions of usefulness.
The most common conditions are the strength limit state and the serviceability limit state.

**Load effects** Internal forces and moments induced by the loads.

**Load factor** A factor used to multiply a nominal load to obtain part of the design load.

**Loads** Forces acting on a structure.

**Local buckling** A mode of buckling which occurs locally (rather than generally) in a thin-plate element of a member.

**Mechanism** A structural system with a sufficient number of frictionless and plastic hinges to allow it to deform indefinitely under constant load.

**Member** A one-dimensional structural element which supports transverse or longitudinal loads or moments.

**Nominal load** The load magnitude determined from a loading code or specification.

**Non-uniform torsion** The general state of torsion in which the twist of the member varies non-uniformly.

**Plastic analysis** A method of analysis in which the ultimate strength of a structure is computed by considering the conditions for which there are sufficient plastic hinges to transform the structure into a mechanism.

**Plastic hinge** A fully yielded cross-section of a member which allows the member portions on either side to rotate under constant moment (the plastic moment).

**Plastic section** A section capable of reaching and maintaining the full plastic moment until a plastic collapse mechanism is formed. Referred to in EC3 as a Class 1 section.

**Post-buckling strength** A reserve of strength after buckling which is possessed by some thin-plate elements.

**Preloaded bolts** High-strength bolts used in friction-grip joints.

**Purlin** A horizontal member between main beams which supports roof sheeting.

**Reduced modulus** The modulus of elasticity used to predict the buckling of inelastic members under the so-called constant applied load, because it is reduced below the elastic modulus.

**Residual stresses** The stresses in an unloaded member caused by non-uniform plastic deformation or by uneven cooling after rolling, flame cutting, or welding.

**Rigid frame** A frame with rigid connections between members. Referred to in EC3 as a continuous frame.

**Second-order analysis** An analysis in which equilibrium is formulated for the deformed position of the structure, so that the moments caused by the products of the loads and deflections are included.

**Semi-compact section** A section which can reach the yield stress, but which does not have sufficient resistance to inelastic local buckling to allow it to reach or to maintain the full plastic moment while a plastic mechanism is forming. Referred to in EC3 as a Class 3 section.

**Semi-rigid frame** A frame with semi-rigid connections between members. Referred to in EC3 as a semi-continuous frame.
**Service loads** The design loads appropriate for the serviceability limit state.

**Shear centre** The point in the cross-section of a beam through which the resultant transverse force must act if the beam is not to twist.

**Shear lag** A phenomenon which occurs in thin wide flanges of beams in which shear straining causes the distribution of bending normal stresses to become sensibly non-uniform.

**Simple frame** A frame for which the joints may be assumed not to transmit moments.

**Slender section** A section which does not have sufficient resistance to local buckling to allow it to reach the yield stress. Referred to in EC3 as a Class 4 section.

**Splice** A connection between two similar collinear members.

**Squash load** The value of the compressive axial load which will cause yielding throughout a short member.

**Stiffener** A plate or section attached to a web to strengthen a member.

**Strain-hardening** A stress–strain state which occurs at stresses which are greater than the yield stress.

**Strength limit state** The state of collapse or loss of structural integrity.

**System length** Length between adjacent lateral brace points, or between brace point and an adjacent end of the member.

**Tangent modulus** The slope of the inelastic stress–strain curve which is used to predict buckling of inelastic members under increasing load.

**Tensile strength** The maximum nominal stress which can be reached in tension.

**Tension field** A mode of shear transfer in the thin web of a stiffened plate girder which occurs after elastic local buckling takes place. In this mode, the tension diagonal of each stiffened panel behaves in the same way as does the diagonal tension member of a parallel chord truss.

**Tension member** A member which supports axial tension loads.

**Torsional buckling** A mode of buckling in which a member twists.

**Ultimate load design** A method of design in which the ultimate load capacity of the structure is compared with factored loads.

**Uniform torque** That part of the total torque which is associated with the rate of change of the angle of twist of the member. Referred to in EC3 as St Venant torque.

**Uniform torsion** The special state of torsion in which the angle of twist of the member varies linearly. Referred to in EC3 as St Venant torsion.

**Warping** A mode of deformation in which plane cross-sections do not remain in plane.

**Warping torque** The other part of the total torque (than the uniform torque). This only occurs during non-uniform torsion, and is associated with changes in the warping of the cross-sections.

**Working load design** A method of design in which the stresses caused by the service loads are compared with maximum permissible stresses.

**Yield strength** The average stress during yielding when significant straining takes place. Usually, the minimum yield strength in tension specified for the particular steel.
The following notation is used in this book. Usually, only one meaning is assigned to each symbol, but in those cases where more meanings than one are possible, then the correct one will be evident from the context in which it is used.

**Main symbols**

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<thead>
<tr>
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<th>Meaning</th>
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<tr>
<td>A</td>
<td>Area</td>
</tr>
<tr>
<td>B</td>
<td>Bimoment</td>
</tr>
<tr>
<td>b</td>
<td>Width</td>
</tr>
<tr>
<td>C</td>
<td>Coefficient</td>
</tr>
<tr>
<td>c</td>
<td>Width of part of section</td>
</tr>
<tr>
<td>d</td>
<td>Depth, or Diameter</td>
</tr>
<tr>
<td>E</td>
<td>Young’s modulus of elasticity</td>
</tr>
<tr>
<td>e</td>
<td>Eccentricity, or Extension</td>
</tr>
<tr>
<td>F</td>
<td>Force, or Force per unit length</td>
</tr>
<tr>
<td>f</td>
<td>Stress property of steel</td>
</tr>
<tr>
<td>G</td>
<td>Dead load, or Shear modulus of elasticity</td>
</tr>
<tr>
<td>H</td>
<td>Horizontal force</td>
</tr>
<tr>
<td>h</td>
<td>Height, or Overall depth of section</td>
</tr>
<tr>
<td>I</td>
<td>Second moment of area</td>
</tr>
<tr>
<td>i</td>
<td>Integer, or Radius of gyration</td>
</tr>
<tr>
<td>k</td>
<td>Buckling coefficient, or Factor, or Relative stiffness ratio</td>
</tr>
<tr>
<td>L</td>
<td>Length</td>
</tr>
<tr>
<td>M</td>
<td>Moment</td>
</tr>
<tr>
<td>m</td>
<td>Integer</td>
</tr>
<tr>
<td>N</td>
<td>Axial force, or Number of load cycles</td>
</tr>
<tr>
<td>n</td>
<td>Integer</td>
</tr>
<tr>
<td>p</td>
<td>Distance between holes or rows of holes</td>
</tr>
<tr>
<td>Q</td>
<td>Load</td>
</tr>
<tr>
<td>q</td>
<td>Intensity of distributed load</td>
</tr>
<tr>
<td>R</td>
<td>Radius, or Reaction, or Resistance</td>
</tr>
<tr>
<td>r</td>
<td>Radius</td>
</tr>
</tbody>
</table>
### Notations

- $s$: Spacing
- $T$: Torque
- $t$: Thickness
- $U$: Strain energy
- $u$: Deflection in $x$ direction
- $V$: Shear, or Vertical load
- $v$: Deflection in $y$ direction
- $W$: Section modulus, or Work done
- $w$: Deflection in $z$ direction
- $x$: Longitudinal axis
- $y$: Principal axis of cross-section
- $z$: Principal axis of cross-section
- $\alpha$: Angle, or Factor, or Load factor at failure, or Stiffness
- $\chi$: Reduction factor
- $\Delta$: Deflection
- $\Delta \sigma$: Stress range
- $\delta$: Amplification factor, or Deflection
- $\varepsilon$: Normal strain, or Yield stress coefficient = $\sqrt{(235/f_y)}$
- $\phi$: Angle of twist rotation, or Global sway imperfection
- $\gamma$: Partial factor, or shear strain
- $\kappa$: Curvature
- $\lambda$: Plate slenderness = $(c/t)/\varepsilon$
- $\lambda\overline{\lambda}$: Generalised slenderness
- $\mu$: Slip factor
- $\nu$: Poisson’s ratio
- $\theta$: Angle
- $\sigma$: Normal stress
- $\tau$: Shear stress

### Subscripts

- $as$: Antisymmetric
- $B$: Bottom
- $b$: Beam, or Bearing, or Bending, or Bolt, or Braced
- $c$: Centroid, or Column, or Compression
- $cr$: Elastic (critical) buckling
- $d$: Design
- $Ed$: Design load effect
- $eff$: Effective
- $el$: Elastic
- $F$: Force
- $f$: Flange
- $G$: Dead load
- $I$: Imposed load
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>i</td>
<td>Initial, or Integer</td>
</tr>
<tr>
<td>j</td>
<td>Joint</td>
</tr>
<tr>
<td>k</td>
<td>Characteristic value</td>
</tr>
<tr>
<td>L</td>
<td>Left</td>
</tr>
<tr>
<td>LT</td>
<td>Lateral (or lateral–torsional) buckling</td>
</tr>
<tr>
<td>M</td>
<td>Material</td>
</tr>
<tr>
<td>m</td>
<td>Moment</td>
</tr>
<tr>
<td>max</td>
<td>Maximum</td>
</tr>
<tr>
<td>min</td>
<td>Minimum</td>
</tr>
<tr>
<td>N</td>
<td>Axial force</td>
</tr>
<tr>
<td>n</td>
<td>Integer, or Nominal value</td>
</tr>
<tr>
<td>net</td>
<td>Net</td>
</tr>
<tr>
<td>op</td>
<td>Out-of-plane</td>
</tr>
<tr>
<td>p</td>
<td>Bearing, or Plate</td>
</tr>
<tr>
<td>p, pl</td>
<td>Plastic</td>
</tr>
<tr>
<td>Q</td>
<td>Variable load</td>
</tr>
<tr>
<td>R</td>
<td>Resistance, or Right</td>
</tr>
<tr>
<td>r</td>
<td>Rafter, or Reduced</td>
</tr>
<tr>
<td>Rd</td>
<td>Design resistance</td>
</tr>
<tr>
<td>Rk</td>
<td>Characteristic resistance</td>
</tr>
<tr>
<td>s</td>
<td>Slip, or Storey, or Sway, or Symmetric</td>
</tr>
<tr>
<td>ser</td>
<td>Service</td>
</tr>
<tr>
<td>st</td>
<td>Stiffener, or Strain hardening</td>
</tr>
<tr>
<td>T</td>
<td>Top, or Torsional buckling</td>
</tr>
<tr>
<td>t</td>
<td>St Venant or uniform torsion, or Tension</td>
</tr>
<tr>
<td>TF</td>
<td>Flexural–torsional (or torsional–flexural) buckling</td>
</tr>
<tr>
<td>tf</td>
<td>Tension field</td>
</tr>
<tr>
<td>ult</td>
<td>Ultimate</td>
</tr>
<tr>
<td>V, v</td>
<td>Shear</td>
</tr>
<tr>
<td>W</td>
<td>Wind load</td>
</tr>
<tr>
<td>w</td>
<td>Warping, or Web, or Weld</td>
</tr>
<tr>
<td>x</td>
<td>x axis</td>
</tr>
<tr>
<td>y</td>
<td>y axis, or Yield</td>
</tr>
<tr>
<td>z</td>
<td>z axis</td>
</tr>
<tr>
<td>σ</td>
<td>Normal stress</td>
</tr>
<tr>
<td>τ</td>
<td>Shear stress</td>
</tr>
<tr>
<td>0</td>
<td>Initial value</td>
</tr>
<tr>
<td>1–4</td>
<td>Cross-section class</td>
</tr>
</tbody>
</table>

**Additional notations**

- $A_e$: Area enclosed by hollow section
- $A_{f,\text{max}}$: Flange area at maximum section
- $A_{f,\text{min}}$: Flange area at minimum section
<table>
<thead>
<tr>
<th>Notations</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_h$</td>
<td>Area of hole reduced for stagger</td>
</tr>
<tr>
<td>$A_{nt}, A_{nv}$</td>
<td>Net areas subjected to tension or shear</td>
</tr>
<tr>
<td>$A_s$</td>
<td>Tensile stress area of a bolt</td>
</tr>
<tr>
<td>$A_y$</td>
<td>Shear area of section</td>
</tr>
<tr>
<td>$C$</td>
<td>Index for portal frame buckling</td>
</tr>
<tr>
<td>$C_m$</td>
<td>Equivalent uniform moment factor</td>
</tr>
<tr>
<td>$D$</td>
<td>Plate rigidity $Et^3/12(1 - \nu^2)$</td>
</tr>
<tr>
<td>${D}$</td>
<td>Vector of nodal deformations</td>
</tr>
<tr>
<td>$E_r$</td>
<td>Reduced modulus</td>
</tr>
<tr>
<td>$E_t$</td>
<td>Tangent modulus</td>
</tr>
<tr>
<td>$F$</td>
<td>Buckling factor for beam-columns with unequal end moments</td>
</tr>
<tr>
<td>$F_{p,C}$</td>
<td>Bolt preload</td>
</tr>
<tr>
<td>$F_L, F_T$</td>
<td>Weld longitudinal and transverse forces per unit length</td>
</tr>
<tr>
<td>$F_{T,Rd}$</td>
<td>Design resistance of a T-stub flange</td>
</tr>
<tr>
<td>$[G]$</td>
<td>Stability matrix</td>
</tr>
<tr>
<td>$I_{cz}$</td>
<td>Second moment of area of compression flange</td>
</tr>
<tr>
<td>$I_m$</td>
<td>Second moment of area of member</td>
</tr>
<tr>
<td>$I_n$</td>
<td>$= b_n^3/12$</td>
</tr>
<tr>
<td>$I_t$</td>
<td>Second moment of area of restraining member or rafter</td>
</tr>
<tr>
<td>$I_{yz}$</td>
<td>Product second moment of area</td>
</tr>
<tr>
<td>$I_w$</td>
<td>Warping torsion section constant</td>
</tr>
<tr>
<td>$I_{zm}$</td>
<td>Value of $I_z$ for critical segment</td>
</tr>
<tr>
<td>$I_{zr}$</td>
<td>Value of $I_z$ for restraining segment</td>
</tr>
<tr>
<td>$K$</td>
<td>Beam or torsion constant $= \sqrt{(\pi^2EI_w/GJL^2)}$, or Fatigue life constant</td>
</tr>
<tr>
<td>$[K]$</td>
<td>Elastic stiffness matrix</td>
</tr>
<tr>
<td>$K_m$</td>
<td>$= \sqrt{(\pi^2EI_yd_f^2/4GIL^2)}$</td>
</tr>
<tr>
<td>$L_c$</td>
<td>Distance between restraints, or Length of column which fails under $N$ alone</td>
</tr>
<tr>
<td>$L_j$</td>
<td>Length between end bolts in a long joint</td>
</tr>
<tr>
<td>$L_m$</td>
<td>Length of critical segment, or Member length</td>
</tr>
<tr>
<td>$L_r$</td>
<td>Length of restraining segment or rafter</td>
</tr>
<tr>
<td>$L_{stable}$</td>
<td>Stable length for member with plastic hinges</td>
</tr>
<tr>
<td>$LF$</td>
<td>Load factor</td>
</tr>
<tr>
<td>$M_A, M_B$</td>
<td>End moments</td>
</tr>
<tr>
<td>$M_{b0,y,Rd}$</td>
<td>Design member moment resistance when $N = 0$ and $M_z = 0$</td>
</tr>
<tr>
<td>$M_{c0,z,Rd}$</td>
<td>Design member moment resistance when $N = 0$ and $M_y = 0$</td>
</tr>
<tr>
<td>$M_{cr,MN}$</td>
<td>Elastic buckling moment reduced for axial compression</td>
</tr>
<tr>
<td>$M_{N,y,Rd}, M_{N,z,Rd}$</td>
<td>Major and minor axis beam section moment resistances</td>
</tr>
<tr>
<td>$M_E$</td>
<td>$= (\pi/L)\sqrt{(EI_yGL)}$</td>
</tr>
<tr>
<td>$M_f$</td>
<td>First-order end moment of frame member</td>
</tr>
<tr>
<td>$M_{fb}$</td>
<td>Braced component of $M_f$</td>
</tr>
</tbody>
</table>
$M_{fp}$  Major axis moment resisted by plastic flanges
$M_{fs}$  Sway component of $M_f$
$M_I$  Inelastic beam buckling moment
$M_{Iu}$  Value of $M_I$ for uniform bending
$M_L$  Limiting end moment on a crooked and twisted beam at first yield
$M_{max,0}$  Value of $M_{max}$ when $N = 0$
$M_N$  Plastic moment reduced for axial force
$M_p$  Out-of-plane member moment resistance for bending alone
$M_{bt}$  Out-of-plane member moment resistance for bending and tension
$M_{ry}, M_{rz}$  Section moment capacities reduced for axial load
$M_S$  Simple beam moment
$M_{ty}$  Lesser of $M_{ry}$ and $M_{bt}$
$M_{xx}$  Value of $M_{cr}$ for simply supported beam in uniform bending
$M_{xxr}$  Value of $M_{xx}$ reduced for incomplete torsional end restraint
$\{N_i\}$  Vector of initial axial forces
$N_{b,Rd}$  Design member axial force resistance when $M_y = 0$ and $M_z = 0$
$N_{cr,MN}$  Elastic buckling load reduced for bending moment
$N_{cr,L}$  Reduced modulus buckling load
$N_{cr,r}$  Constant amplitude fatigue life for $i$th stress range
$N_{cr,t}$  Tangent modulus buckling load
$Q_D$  Concentrated dead load
$Q_I$  Concentrated imposed load
$Q_m$  Upper-bound mechanism estimate of $Q_{ult}$
$Q_{ms}$  Value of $Q_s$ for the critical segment
$Q_{rs}$  Value of $Q_s$ for an adjacent restraining segment
$Q_s$  Buckling load for an unrestrained segment, or Lower bound static estimate of $Q_{ult}$
$R$  Radius of circular cross-section, or Ratio of column and rafter stiffnesses, or Ratio of minimum to maximum stress
$R_H$  Ratio of rafter rise to column height
$R, R_{1-4}$  Restraint parameters
$SF$  Factor of safety
$S_j$  Joint stiffness
$T_M$  Torque exerted by bending moment
$T_P$  Torque exerted by axial load
$V_R$  Resultant shear force
$V_{Ty}, V_{Tz}$  Transverse shear forces in a fillet weld
$V_{vi}$  Shear force in $i$th fastener
$a = \sqrt{(EI_w/GJ)}$, or Distance along member, or Distance from web to shear centre, or Effective throat size of a weld, or Ratio of web to total section area, or Spacing of transverse stiffeners
<table>
<thead>
<tr>
<th>Notations</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>$a_0$</td>
<td>Distance from shear centre</td>
</tr>
<tr>
<td>$b$</td>
<td>$c_f$ or $c_w$</td>
</tr>
<tr>
<td>$c$</td>
<td>Factor for flange contribution to shear resistance</td>
</tr>
<tr>
<td>$c_m$</td>
<td>Bending coefficient for beam-columns with unequal end moments</td>
</tr>
<tr>
<td>$d_c$</td>
<td>Depth of elastic core</td>
</tr>
<tr>
<td>$d_f$</td>
<td>Distance between flange centroids</td>
</tr>
<tr>
<td>$d_0$</td>
<td>Hole diameter</td>
</tr>
<tr>
<td>$e_1$</td>
<td>End distance in a plate</td>
</tr>
<tr>
<td>$e_2$</td>
<td>Edge distance in a plate</td>
</tr>
<tr>
<td>$e_{Ny}$</td>
<td>Shift of effective compression force from centroid</td>
</tr>
<tr>
<td>$f$</td>
<td>Factor used to modify $\chi_{LT}$</td>
</tr>
<tr>
<td>$h_w$</td>
<td>Clear distance between flanges</td>
</tr>
<tr>
<td>$i_{f,z}$</td>
<td>Radius of gyration of equivalent compression flange</td>
</tr>
<tr>
<td>$i_p$</td>
<td>Polar radius of gyration</td>
</tr>
<tr>
<td>$i_0$</td>
<td>$\sqrt{i_p^2 + y_0^2 + z_0^2}$</td>
</tr>
<tr>
<td>$k$</td>
<td>Deflection coefficient, or Modulus of foundation reaction</td>
</tr>
<tr>
<td>$k_c$</td>
<td>Slenderness correction factor, or Correction factor for moment distribution</td>
</tr>
<tr>
<td>$k_{ij}$</td>
<td>Interaction factors for bending and compression</td>
</tr>
<tr>
<td>$k_s$</td>
<td>Factor for hole shape and size</td>
</tr>
<tr>
<td>$k_t$</td>
<td>Axial stiffness of connector</td>
</tr>
<tr>
<td>$k_v$</td>
<td>Shear stiffness of connector</td>
</tr>
<tr>
<td>$k_{\sigma}$</td>
<td>Plate buckling coefficient</td>
</tr>
<tr>
<td>$k_1$</td>
<td>Factor for plate tension fracture</td>
</tr>
<tr>
<td>$k_{1,2}$</td>
<td>Stiffness factors</td>
</tr>
<tr>
<td>$\ell_{eff}$</td>
<td>Effective length of a fillet weld, or Effective length of an unstiffened column flange</td>
</tr>
<tr>
<td>$\ell_y$</td>
<td>Effective loaded length</td>
</tr>
<tr>
<td>$m$</td>
<td>Fatigue life index, or Torque per unit length</td>
</tr>
<tr>
<td>$n$</td>
<td>Axial compression ratio, or Number of shear planes</td>
</tr>
<tr>
<td>$p_F$</td>
<td>Probability of failure</td>
</tr>
<tr>
<td>$p(x)$</td>
<td>Particular integral</td>
</tr>
<tr>
<td>$p_1$</td>
<td>Pitch of bolt holes</td>
</tr>
<tr>
<td>$p_2$</td>
<td>Spacing of bolt hole lines</td>
</tr>
<tr>
<td>$s$</td>
<td>Distance around thin-walled section</td>
</tr>
<tr>
<td>$s_s$</td>
<td>Stiff bearing length</td>
</tr>
<tr>
<td>$s$</td>
<td>Staggered pitch of holes</td>
</tr>
<tr>
<td>$s_m$</td>
<td>Minimum staggered pitch for no reduction in effective area</td>
</tr>
<tr>
<td>$s_{1,2}$</td>
<td>Side widths of a fillet weld</td>
</tr>
<tr>
<td>$w$</td>
<td>$W_{pl}/W_{el}$</td>
</tr>
<tr>
<td>$w_{AB}$</td>
<td>Settlement of B relative to A</td>
</tr>
<tr>
<td>$w_c$</td>
<td>Mid-span deflection</td>
</tr>
<tr>
<td>$\zeta$</td>
<td>Distance to centroid</td>
</tr>
</tbody>
</table>
\( y_p, z_p \)  
\( y_r, z_r \)  
\( y_0, z_0 \)  
\( z_c \)  
\( z_n \)  
\( z_Q \)  
\( z_t \)  
\( \alpha \)  
\( \alpha_{bc} \)  
\( \alpha_{bcI} \)  
\( \alpha_{bcu} \)  
\( \alpha_d \)  
\( \alpha_i \)  
\( \alpha_L \)  
\( \alpha_{LT} \)  
\( \alpha_L, \alpha_0 \)  
\( \alpha_m \)  
\( \alpha_n \)  
\( \alpha_r, \alpha_t \)  
\( \alpha_{x, y} \)  
\( \alpha_{st} \)  
\( \beta \)  
\( \beta_c \)  
\( \beta_{lf} \)  
\( \beta_m \)  
\( \beta_w \)  
\( \beta_y \)  
\( \beta_{2,3} \)  
\( \Delta \sigma_C \)  
\( \Delta \sigma_L \)  
\( \varepsilon \)  
\( \Phi \)  
\( \Phi_{cd} \)  
\( \phi_j \)  
\( \gamma_F, \gamma_G, \gamma_Q \)  
\( \gamma_m, \gamma_n, \gamma_s \)
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\gamma_{1,2}$</td>
<td>Relative stiffnesses</td>
</tr>
<tr>
<td>$\eta$</td>
<td>Crookedness or imperfection parameter, or Web shear resistance factor (= 1.2 for steels up to S460)</td>
</tr>
<tr>
<td>$\lambda_{c,0}$</td>
<td>Slenderness limit of equivalent compression flange</td>
</tr>
<tr>
<td>$\lambda_p$</td>
<td>Generalised plate slenderness = $\sqrt{f_y/\sigma_{cr}}$</td>
</tr>
<tr>
<td>$\lambda_1$</td>
<td>$= \pi \sqrt{E/f_y}$</td>
</tr>
<tr>
<td>$\mu$</td>
<td>$= \sqrt{N/EI}$</td>
</tr>
<tr>
<td>$\theta$</td>
<td>Central twist, or Slope change at plastic hinge, or Torsion stress function</td>
</tr>
<tr>
<td>$\rho$</td>
<td>Perpendicular distance from centroid, or Reduction factor</td>
</tr>
<tr>
<td>$\rho_m$</td>
<td>Monosymmetric section parameter = $I_{zc}/I_z$</td>
</tr>
<tr>
<td>$\rho_c, \rho_r$</td>
<td>Column and rafter factors for portal frame buckling</td>
</tr>
<tr>
<td>$\rho_0$</td>
<td>Perpendicular distance from shear centre</td>
</tr>
<tr>
<td>$\sigma_{ac}, \sigma_{at}$</td>
<td>Stresses due to axial compression and tension</td>
</tr>
<tr>
<td>$\sigma_{bcy}$</td>
<td>Compression stress due to bending about y axis</td>
</tr>
<tr>
<td>$\sigma_{bty}, \sigma_{btz}$</td>
<td>Bending stress due to bending about y, z axes</td>
</tr>
<tr>
<td>$\sigma_{cr,l}$</td>
<td>Bearing stress at local buckling</td>
</tr>
<tr>
<td>$\sigma_{cr,p}$</td>
<td>Limiting major axis stress in a crooked and twisted beam at first yield</td>
</tr>
<tr>
<td>$\tau_{h}, \tau_v$</td>
<td>Shear stresses due to $V_y, V_z$</td>
</tr>
<tr>
<td>$\tau_{hc}, \tau_{vc}$</td>
<td>Shear stresses due to a circulating shear flow</td>
</tr>
<tr>
<td>$\tau_{ho}, \tau_{vo}$</td>
<td>Shear stresses in an open section</td>
</tr>
<tr>
<td>$\psi$</td>
<td>End moment ratio, or Stress ratio</td>
</tr>
<tr>
<td>$\psi_0$</td>
<td>Load combination factor</td>
</tr>
</tbody>
</table>
1.1 Steel structures

Engineering structures are required to support loads and resist forces, and to transfer these loads and forces to the foundations of the structures. The loads and forces may arise from the masses of the structure, or from man’s use of the structures, or from the forces of nature. The uses of structures include the enclosure of space (buildings), the provision of access (bridges), the storage of materials (tanks and silos), transportation (vehicles), or the processing of materials (machines). Structures may be made from a number of different materials, including steel, concrete, wood, aluminium, stone, plastic, etc., or from combinations of these.

Structures are usually three-dimensional in their extent, but sometimes they are essentially two-dimensional (plates and shells), or even one-dimensional (lines and cables). Solid steel structures invariably include comparatively high volumes of high-cost structural steel which are understressed and uneconomic, except in very small-scale components. Because of this, steel structures are usually formed from one-dimensional members (as in rectangular and triangulated frames), or from two-dimensional members (as in box girders), or from both (as in stressed skin industrial buildings). Three-dimensional steel structures are often arranged so that they act as if composed of a number of independent two-dimensional frames or one-dimensional members (Figure 1.1).

Structural steel members may be one-dimensional as for beams and columns (whose lengths are much greater than their transverse dimensions), or two-dimensional as for plates (whose lengths and widths are much greater than their thicknesses), as shown in Figure 1.2c. While one-dimensional steel members may be solid, they are usually thin-walled, in that their thicknesses are much less than their other transverse dimensions. Thin-walled steel members are rolled in a number of cross-sectional shapes [1] or are built up by connecting together a number of rolled sections or plates, as shown in Figure 1.2b. Structural members can be classified as tension or compression members, beams, beam-columns, torsion members, or plates (Figure 1.3), according to the method by which they transmit the forces in the structure. The behaviour and design of these structural members are discussed in this book.
Structural steel members may be connected together at joints in a number of ways, and by using a variety of connectors. These include pins, rivets, bolts, and welds of various types. Steel plate gussets, or angle cleats, or other elements may also be used in the connections. The behaviour and design of these connectors and joints are also discussed in this book.
This book deals chiefly with steel frame structures composed of one-dimensional members, but much of the information given is also relevant to plate structures. The members are generally assumed to be hot-rolled or fabricated from hot-rolled elements, while the frames considered are those used in buildings. However, much of the material presented is also relevant to bridge structures [2, 3], and to structural members cold-formed from light-gauge steel plates [4–7].

The purposes of this chapter are first, to consider the complete design process and the relationships between the behaviour and analysis of steel structures and their structural design, and second, to present information of a general nature (including information on material properties and structural loads) which is required for use in the later chapters. The nature of the design process is discussed first, and then brief summaries are made of the relevant material properties of structural steel and of the structural behaviour of members and frames. The loads acting on the structures are considered, and the choice of appropriate methods of analysing the steel structures is discussed. Finally, the considerations governing the synthesis of an understanding of the structural behaviour with the results of analysis to form the design processes of EC3 [8] are treated.

1.2 Design

1.2.1 Design requirements

The principal design requirement of a structure is that it should be effective; that is, it should fulfil the objectives and satisfy the needs for which it was created. The
structure may provide shelter and protection against the environment by enclosing space, as in buildings; or it may provide access for people and materials, as in bridges; or it may store materials, as in tanks and silos; or it may form part of a machine for transporting people or materials, as in vehicles, or for operating on materials. The design requirement of effectiveness is paramount, as there is little point in considering a structure which will not fulfil its purpose.

The satisfaction of the effectiveness requirement depends on whether the structure satisfies the structural and other requirements. The structural requirements relate to the way in which the structure resists and transfers the forces and loads acting on it. The primary structural requirement is that of safety, and the first consideration of the structural engineer is to produce a structure which will not fail in its design lifetime, or which has an acceptably low risk of failure. The other important structural requirement is usually concerned with the stiffness of the structure, which must be sufficient to ensure that the serviceability of the structure is not impaired by excessive deflections, vibrations, and the like.

The other design requirements include those of economy and of harmony. The cost of the structure, which includes both the initial cost and the cost of maintenance, is usually of great importance to the owner, and the requirement of economy usually has a significant influence on the design of the structure. The cost of the structure is affected not only by the type and quantity of the materials used, but also by the methods of fabricating and erecting it. The designer must therefore give careful consideration to the methods of construction as well as to the sizes of the members of the structure.

The requirements of harmony within the structure are affected by the relationships between the different systems of the structure, including the load resistance and transfer system (the structural system), the architectural system, the mechanical and electrical systems, and the functional systems required by the use of the structure. The serviceability of the structure is usually directly affected by the harmony, or lack of it, between the systems. The structure should also be in harmony with its environment, and should not react unfavourably with either the community or its physical surroundings.

1.2.2 The design process

The overall purpose of design is to invent a structure which will satisfy the design requirements outlined in Section 1.2.1. Thus the structural engineer seeks to invent a structural system which will resist and transfer the forces and loads acting on it with adequate safety, while making due allowance for the requirements of serviceability, economy, and harmony. The process by which this may be achieved is summarised in Figure 1.4.

The first step is to define the overall problem by determining the effectiveness requirements and the constraints imposed by the social and physical environments and by the owner’s time and money. The structural engineer will need to consult the owner; the architect, the site, construction, mechanical, and electrical engineers;
and any authorities from whom permissions and approvals must be obtained. A set of objectives can then be specified, which if met, will ensure the successful solution of the overall design problem.

The second step is to invent a number of alternative overall systems and their associated structural systems which appear to meet the objectives. In doing so, the designer may use personal knowledge and experience or that which can be
Invention or modification of structural system

Preliminary analysis

Proportioning members and joints

Analysis

Evaluation

← Knowledge
← Experience
← Imagination
← Intuition
← Creativity

← Approximations
← Loads
← Behaviour

← Design criteria
← Design codes

← Design criteria
← Design codes

Figure 1.5 The structural design process.

gathered from others [9–12]; or the designer may use his or her own imagination, intuition, and creativity [13], or a combination of all of these.

Following these first two steps of definition and invention come a series of steps which include the structural design, evaluation, selection, and modification of the structural system. These may be repeated a number of times before the structural requirements are met and the structural design is finalised. A typical structural design process is summarised in Figure 1.5.

After the structural system has been invented, it must be analysed to obtain the information required for determining the member sizes. First, the loads supported by and the forces acting on the structure must be determined. For this purpose, loading codes [14, 15] are usually consulted, but sometimes the designer determines the loading conditions or commissions experts to do this. A number of approximate assumptions are made about the behaviour of the structure, which is then analysed and the forces and moments acting on the members and joints of the structure are determined. These are used to proportion the structure so that it satisfies the structural requirements, usually by referring to a design code, such as EC3 [8].

At this stage a preliminary design of the structure will have been completed, however, because of the approximate assumptions made about the structural behaviour, it is necessary to check the design. The first steps are to recalculate the loads and to
reanalyse the structure designed, and these are carried out with more precision than was either possible or appropriate for the preliminary analysis. The performance of the structure is then evaluated in relation to the structural requirements, and any changes in the member and joint sizes are decided on. These changes may require a further reanalysis and re-proportioning of the structure, and this cycle may be repeated until no further change is required. Alternatively, it may be necessary to modify the original structural system and repeat the structural design process until a satisfactory structure is achieved.

The alternative overall systems are then evaluated in terms of their serviceability, economy, and harmony, and a final system is selected, as indicated in Figure 1.4. This final overall system may be modified before the design is finalised. The detailed drawings and specifications can then be prepared, and tenders for the construction can be called for and let, and the structure can be constructed. Further modifications may have to be made as a consequence of the tenders submitted or due to unforeseen circumstances discovered during construction.

This book is concerned with the structural behaviour of steel structures, and the relationships between their behaviour and the methods of proportioning them, particularly in relation to the structural requirements of the European steel structures code EC3 and the modifications of these are given in the National Annexes. This code consists of six parts, with basic design using the conventional members being treated in Part 1. This part is divided into 12 sub-parts, with those likely to be required most frequently being:

- Part 1.1 General Rules and Rules for Buildings [8],
- Part 1.5 Plated Structural Elements [16],
- Part 1.8 Design of Joints [17], and
- Part 1.10 Selection of Steel for Fracture Toughness and Through-Thickness Properties [18].

Other parts that may be required from time to time include: Part 1.2 that covers resistance to fire, Part 1.3 dealing with cold-formed steel, Part 1.9 dealing with fatigue and Part 2 [19] that covers bridges. Composite construction is covered by EC4 [20]. Since Part 1.1 of EC3 is the document most relevant to much of the content of this text (with the exception of Chapter 9 on joints), all references to EC3 made herein should be taken to mean Part 1.1 [8], including any modifications given in the National Annex, unless otherwise indicated.

Detailed discussions of the overall design process are beyond the scope of this book, but further information is given in [13] on the definition of the design problem, the invention of solutions and their evaluation, and in [21–24] on the execution of design. Further, the conventional methods of structural analysis are adequately treated in many textbooks [25–27] and are discussed in only a few isolated cases in this book.
1.3 Material behaviour

1.3.1 Mechanical properties under static load

The important mechanical properties of most structural steels under static load are indicated in the idealised tensile stress–strain diagram shown in Figure 1.6. Initially the steel has a linear stress–strain curve whose slope is the Young’s modulus of elasticity \( E \). The values of \( E \) vary in the range 200 000–210 000 N/mm\(^2\), and the approximate value of 205 000 N/mm\(^2\) is often assumed (EC3 uses 210 000 N/mm\(^2\)). The steel remains elastic while in this linear range, and recovers perfectly on unloading. The limit of the linear elastic behaviour is often closely approximated by the yield stress \( f_y \) and the corresponding yield strain \( \varepsilon_y = f_y / E \).

Beyond this limit the steel flows plastically without any increase in stress until the strain-hardening strain \( \varepsilon_{st} \) is reached. This plastic range is usually considerable, and accounts for the ductility of the steel. The stress increases above the yield stress \( f_y \) when the strain-hardening strain \( \varepsilon_{st} \) is exceeded, and this continues until the ultimate tensile stress \( f_u \) is reached. After this, large local reductions in the cross-section occur, and the load capacity decreases until tensile fracture takes place.

The yield stress \( f_y \) is perhaps the most important strength characteristic of a structural steel. This varies significantly with the chemical constituents of the steel, the most important of which are carbon and manganese, both of which increase the yield stress. The yield stress also varies with the heat treatment used and with the amount of working which occurs during the rolling process. Thus thinner plates which are more worked have higher yield stresses than thicker plates of the same constituency. The yield stress is also increased by cold working. The rate of straining affects the yield stress, and high rates of strain increase the upper or first yield stress (see the broken line in Figure 1.6), as well as the lower yield stress \( f_y \). The strain rates used in tests to determine the yield stress of a particular

![Figure 1.6 Idealised stress-strain relationship for structural steel.](image-url)
steel type are significantly higher than the nearly static rates often encountered in actual structures.

For design purposes, a ‘minimum’ yield stress is identified for each different steel classification. For EC3, these classifications are made on the basis of the chemical composition and the heat treatment, and so the yield stresses in each classification decrease as the greatest thickness of the rolled section or plate increases. The minimum yield stress of a particular steel is determined from the results of a number of standard tension tests. There is a significant scatter in these results because of small variations in the local composition, heat treatment, amount of working, thickness, and the rate of testing, and this scatter closely follows a normal distribution curve. Because of this, the minimum yield stress \( f_y \) quoted for a particular steel and used in design is usually a characteristic value which has a particular chance (often 95%) of being exceeded in any standard tension test. Consequently, it is likely that an isolated test result will be significantly higher than the quoted yield stress. This difference will, of course, be accentuated if the test is made for any but the thickest portion of the cross-section. In EC3 [8], the yield stress to be used in design is listed in Table 3.1 for hot-rolled structural steel and for structural hollow sections for each of the structural grades.

The yield stress \( f_y \) determined for uniaxial tension is usually accepted as being valid for uniaxial compression. However, the general state of stress at a point in a thin-walled member is one of biaxial tension and/or compression, and yielding under these conditions is not so simply determined. Perhaps the most generally accepted theory of two-dimensional yielding under biaxial stresses acting in the \( 1'2' \) plane is the maximum distortion-energy theory (often associated with names of Huber, von Mises, or Hencky), and the stresses at yield according to this theory satisfy the condition

\[
\sigma_1^2 - \sigma_1 \sigma_2 + \sigma_2^2 + 3\sigma_{1'2'}^2 = f_y^2, \tag{1.1}
\]

in which \( \sigma_1, \sigma_2 \) are the normal stresses and \( \sigma_{1'2'} \) is the shear stress at the point. For the case where \( 1' \) and \( 2' \) are the principal stress directions 1 and 2, equation 1.1 takes the form of the ellipse shown in Figure 1.7, while for the case of pure shear \((\sigma_1 = \sigma_2' = 0, \text{ so that } \sigma_1 = -\sigma_2 = \sigma_{1'2'} )\), equation 1.1 reduces to

\[
\sigma_{1'2'} = f_y/\sqrt{3} = \tau_y, \tag{1.2}
\]

which defines the shear yield stress \( \tau_y \).

### 1.3.2 Fatigue failure under repeated loads

Structural steel may fracture at low average tensile stresses after a large number of cycles of fluctuating load. This high-cycle fatigue failure is initiated by local damage caused by the repeated loads, which leads to the formation of a small local crack. The extent of the fatigue crack is gradually increased by the subsequent load repetitions, until finally the effective cross-section is so reduced that catastrophic
failure may occur. High-cycle fatigue is only a design consideration when a large number of loading cycles involving tensile stresses is likely to occur during the design life of the structure (compressive stresses do not cause fatigue). This is often the case for bridges, cranes, and structures which support machinery; wind and wave loading may also lead to fatigue problems.

Factors which significantly influence the resistance to fatigue failure include the number of load cycles $N$, the range of stress

$$\Delta \sigma = \sigma_{\text{max}} - \sigma_{\text{min}}$$

(1.3)
during a load cycle, and the magnitudes of local stress concentrations. An indication of the effect of the number of load cycles is given in Figure 1.8, which shows that the maximum tensile stress decreases from its ultimate static value $f_u$ in an approximately linear fashion as the logarithm of the number of cycles, $N$, increases. As the number of cycles increases further the curve may flatten out and the maximum tensile stress may approach the endurance limit $\Delta \sigma_L$.

The effects of the stress magnitude and stress ratio on the fatigue life are demonstrated in Figure 1.9. It can be seen that the fatigue life $N$ decreases with increasing stress magnitude $\sigma_{\text{max}}$ and with decreasing stress ratio $R = \sigma_{\text{min}} / \sigma_{\text{max}}$.

The effect of stress concentration is to increase the stress locally, leading to local damage and crack initiation. Stress concentrations arise from sudden changes in the general geometry and loading of a member, and from local changes due to bolt and rivet holes and welds. Stress concentrations also occur at defects in the member, or its connectors and welds. These may be due to the original rolling of the steel, or due to subsequent fabrication processes, including punching, shearing,
and welding, or due to damage such as that caused by stray arc fusions during welding.

It is generally accepted for design purposes that the fatigue life $N$ varies with the stress range $\Delta \sigma$ according to equations of the type

$$
\left( \frac{\Delta \sigma}{\Delta \sigma_C} \right)^m \left( \frac{N}{2 \times 10^6} \right) = K
$$

in which the reference value $\Delta \sigma_C$ depends on the details of the fatigue site, and the constants $m$ and $K$ may change with the number of cycles $N$. This assumed dependence of the fatigue life on the stress range produces the approximating straight lines shown in Figure 1.9.
EC3-1-1 [8] does not provide a treatment of fatigue, since it is usually the case that either the stress range $\Delta \sigma$ or the number of high amplitude stress cycles $N$ is comparatively small. However, for structures supporting vibrating machinery and plant, reference should be made to EC3-1-9 [28]. The general relationships between the fatigue life $N$ and the service stress range $\Delta \sigma$ for constant amplitude stress cycles are shown in Figure 1.10 for reference values $\Delta \sigma_C$ which correspond to different detail categories. For $N \leq 5 \times 10^6$, $m = 3$ and $K = 1$, so that the reference value $\Delta \sigma_C$ corresponds to the value of $\Delta \sigma$ at $N = 2 \times 10^6$. For $5 \times 10^6 \leq N \leq 10^8$, $m = 5$ and $K = 0.42^{2/3} \approx 0.543$.

Fatigue failure under variable amplitude stress cycles is normally assessed using Miner’s rule [29]

$$\sum \frac{N_i}{N_{im}} \leq 1 \quad (1.5)$$

in which $N_i$ is the number of cycles of a particular stress range $\Delta \sigma_i$ and $N_{im}$ the constant amplitude fatigue life for that stress range. If any of the stress ranges exceeds the constant amplitude fatigue limit (at $N = 5 \times 10^6$), then the effects of stress ranges below this limit are included in equation 1.5.

Designing against fatigue involves a consideration of joint arrangement as well as of permissible stress. Joints should generally be so arranged as to minimise stress concentrations and produce as smooth a ‘stress flow’ through the joint as is practicable. This may be done by giving proper consideration to the layout of a joint, by making gradual changes in section, and by increasing the amount of material used at points of concentrated load. Weld details should also be determined
with this in mind, and unnecessary ‘stress-raisers’ should be avoided. It will also be advantageous to restrict, where practicable, the locations of joints to low stress regions such as at points of contraflexure or near the neutral axis. Further information and guidance on fatigue design are given in [30–33].

### 1.3.3 Brittle fracture under impact load

Structural steel does not always exhibit a ductile behaviour, and under some circumstances a sudden and catastrophic fracture may occur, even though the nominal tensile stresses are low. Brittle fracture is initiated by the existence or formation of a small crack in a region of high local stress. Once initiated, the crack may propagate in a ductile (or stable) fashion for which the external forces must supply the energy required to tear the steel. More serious are cracks which propagate at high speed in a brittle (or unstable) fashion, for which some of the internal elastic strain energy stored in steel is released and used to fracture the steel. Such a crack is self-propagating while there is sufficient internal strain energy, and will continue until arrested by ductile elements in its path which have sufficient deformation capacity to absorb the internal energy released.

The resistance of a structure to brittle fracture depends on the magnitude of local stress concentrations, on the ductility of the steel, and on the three-dimensional geometrical constraints. High local stresses facilitate crack initiation, and so stress concentrations due to poor geometry and loading arrangements (including impact loading) are dangerous. Also of great importance are flaws and defects in the material, which not only increase the local stresses, but also provide potential sites for crack initiation.

The ductility of a structural steel depends on its composition, heat treatment, and thickness, and varies with temperature and strain rate. Figure 1.11 shows the increase with temperature of the capacity of the steel to absorb energy during

![Effect of temperature on resistance to brittle fracture](image)

*Figure 1.11* Effect of temperature on resistance to brittle fracture.
impact. At low temperatures the energy absorption is low and initiation and propagation of brittle fractures are comparatively easy, while at high temperatures the energy absorption is high because of ductile yielding, and the propagation of cracks can be arrested. Between these two extremes is a transitional range in which crack initiation becomes increasingly difficult. The likelihood of brittle fracture is also increased by high strain rates due to dynamic loading, since the consequent increase in the yield stress reduces the possibility of energy absorption by ductile yielding. The chemical composition of steel has a marked influence on its ductility: brittleness is increased by the presence of excessive amounts of most non-metallic elements, while ductility is increased by the presence of some metallic elements. Steel with large grain size tends to be more brittle, and this is significantly influenced by heat treatment of the steel, and by its thickness (the grain size tends to be larger in thicker sections). EC3-1-10 [18] provides values of the maximum thickness $t_1$ for different steel grades and minimum service temperatures, as well as advice on using a more advanced fracture mechanics [34] based approach and guidance on safeguarding against lamellar tearing.

Three-dimensional geometrical constraints, such as those occurring in thicker or more massive elements, also encourage brittleness, because of the higher local stresses, and because of the greater release of energy during cracking and the consequent increase in the ease of propagation of the crack.

The risk of brittle fracture can be reduced by selecting steel types which have ductilities appropriate to the service temperatures, and by designing joints with a view to minimising stress concentrations and geometrical constraints. Fabrication techniques should be such that they will avoid introducing potentially dangerous flaws or defects. Critical details in important structures may be subjected to inspection procedures aimed at detecting significant flaws. Of course the designer must give proper consideration to the extra cost of special steels, fabrication techniques, and inspection and correction procedures. Further information on brittle fracture is given in [31, 32, 34].

1.4 Member and structure behaviour

1.4.1 Member behaviour

Structural steel members are required to transmit axial and transverse forces and moments and torques as shown in Figure 1.3. The response of a member to these actions can be described by the load-deformation characteristics shown in Figure 1.12.

A member may have the linear response shown by curve 1 in Figure 1.12, at least until the material reaches the yield stress. The magnitudes of the deformations depend on the elastic moduli $E$ and $G$. Theoretically, a member can only behave linearly while the maximum stress does not exceed the yield stress $f_y$, and so the presence of residual stresses or stress concentrations will cause early non-linearity. However, the high ductility of steel causes a local redistribution after this premature yielding, and it can often be assumed without serious error.
that the member response remains linear until more general yielding occurs. The member behaviour then becomes non-linear (curve 2) and approaches the condition associated with full plasticity (curve 6). This condition depends on the yield stress $f_y$.

The member may also exhibit geometric non-linearity, in that the bending moments and torques acting at any section may be influenced by the deformations as well as by the applied forces. This non-linearity, which depends on the elastic moduli $E$ and $G$, may cause the deformations to become very large (curve 3) as the condition of elastic buckling is approached (curve 4). This behaviour is modified when the material becomes non-linear after first yield, and the load may approach a maximum value and then decrease (curve 5).

The member may also behave in a brittle fashion because of local buckling in a thin plate element of the member (curve 7), or because of material fracture (curve 8).

The actual behaviour of an individual member will depend on the forces acting on it. Thus tension members, laterally supported beams, and torsion members remain linear until their material non-linearity becomes important, and then they approach the fully plastic condition. However, compression members and laterally unsupported beams show geometric non-linearity as they approach their buckling loads. Beam-columns are members which transmit both transverse and axial loads, and so they display both material and geometric non-linearities.

1.4.2 Structure behaviour

The behaviour of a structure depends on the load-transferring action of its members and joints. This may be almost entirely by axial tension or compression, as in the triangulated structures with joint loading as shown in Figure 1.13a.
Introduction

Alternatively, the members may support transverse loads which are transferred by bending and shear actions. Usually the bending action dominates in structures composed of one-dimensional members, such as beams and many single-storey rigid frames (Figure 1.13b), while shear becomes more important in two-dimensional plate structures (Figure 1.13c). The members of many structures are subjected to both axial forces and transverse loads, such as those in multistorey buildings (Figure 1.13d). The load-transferring action of the members of a structure depends on the arrangement of the structure, including the geometrical layout and the joint details, and on the loading arrangement.

In some structures, the loading and joints are such that the members are effectively independent. For example, in triangulated structures with joint loads, any flexural effects are secondary, and the members can be assumed to act as if pin-jointed, while in rectangular frames with simple flexible joints the moment transfers between beams and columns may be ignored. In such cases, the response of the structure is obtained directly from the individual member responses.

More generally, however, there will be interactions between the members, and the structure behaviour is not unlike the general behaviour of a member, as can be seen by comparing Figures 1.14 and 1.12. Thus, it has been traditional to assume that a steel structure behaves elastically under the service loads. This assumption ignores local premature yielding due to residual stresses and stress concentrations, but these are not usually serious. Purely flexural structures, and purely axial structures with lightly loaded compression members, behave as if linear (curve 1 in Figure 1.14). However, structures with both flexural and axial actions behave non-linearly, even near the service loads (curve 3 in Figure 1.14). This is a result of the geometrically non-linear behaviour of its members (see Figure 1.12).

Most steel structures behave non-linearly near their ultimate loads, unless they fail prematurely due to brittle fracture, fatigue, or local buckling. This non-linear behaviour is due either to material yielding (curve 2 in Figure 1.14), or member or frame buckling (curve 4), or both (curve 5). In axial structures, failure may
involve yielding of some tension members, or buckling either of some compression members or of the frame, or both. In flexural structures, failure is associated with full plasticity occurring at a sufficient number of locations that the structure can form a collapse mechanism. In structures with both axial and flexural actions, there is an interaction between yielding and buckling (curve 5 in Figure 1.14), and the failure load is often difficult to determine. The transitions shown in Figure 1.14 between the elastic and ultimate behaviour often take place in a series of non-linear steps as individual elements become fully plastic or buckle.

1.5 Loads

1.5.1 General

The loads acting on steel structures may be classified as dead loads, as imposed loads, including both gradually applied and dynamic loads, as wind loads, as earth or ground-water loads, or as indirect forces, including those due to temperature changes, foundation settlement, and the like. The more general collective term Actions is used throughout the Eurocodes. The structural engineer must evaluate the magnitudes of any of these loads which will act, and must determine those which are the most severe combinations of loads for which the structure must be designed. These loads are discussed in the following subsections, both individually and in combinations.

1.5.2 Dead loads

The dead loads acting on a structure arise from the weight of the structure including the finishes, and from any other permanent construction or equipment. The dead
loads will vary during construction, but thereafter will remain constant, unless significant modifications are made to the structure or its permanent equipment.

The dead load may be assessed from the knowledge of the dimensions and specific weights or from the total weights of all the permanent items which contribute to the total dead load. Guidance on specific weights is given in [14], the values in which are average values representative of the particular materials. The dimensions used to estimate dead loads should also be average and representative, in order that consistent estimates of the dead loads can be made. By making these assumptions, the statistical distribution of dead loads is often taken as being of a Weibull type [35]. The practice sometimes used of consistently overestimating dimensions and specific weights is often wasteful, and may also be dangerous in cases where the dead load component acts in the opposite sense to the resultant load.

### 1.5.3 Imposed loads

The imposed loads acting on a structure are gravity loads other than the dead loads, and arise from the weights of materials added to the structure as a result of its use, such as materials stored, people, and snow. Imposed loads usually vary both in space and time. Imposed loads may be sub-divided into two groups, depending on whether they are gradually applied, in which case static load equivalents can be used, or whether they are dynamic, including repeated loads and impact or impulsive loads.

Gradually applied imposed loads may be sustained over long periods of time, or may vary slowly with time [36]. The past practice, however, was to consider only the total imposed load, and so only extreme values (which occur rarely and may be regarded as lifetime maximum loads) were specified. The present imposed loads specified in loading codes [14] often represent peak loads which have 95% probability of not being exceeded over a 50-year period based on a Weibull type distribution [35].

It is usual to consider the most severe spatial distribution of the imposed loads, and this can only be determined by using both the maximum and minimum values of the imposed loads. In the absence of definite knowledge, it is often assumed that the minimum values are zero. When the distribution of imposed load over large areas is being considered, the maximum imposed loads specified, which represent rare events, are often reduced in order to make some allowance for the decreased probability that the maximum imposed loads will act on all areas at the same time.

Dynamic loads which act on structures include both repeated loads and impact and blast loads. Repeated loads are of significance in fatigue problems (see Section 1.3.2), in which case the designer is concerned with both the magnitudes, ranges, and the number of repetitions of loads which are very frequently applied. At the other extreme, impact loads (which are particularly important in the brittle fracture problems discussed in Section 1.3.3) are usually specified by values of extreme magnitude which represent rare events. In structures for which the static loads
dominate, it is common to replace the dynamic loads by static force equivalents [14]. However, such a procedure is likely to be inappropriate when the dynamic loads form a significant proportion of the total load, in which case a proper dynamic analysis [37, 38] of the structure and its response should be made.

1.5.4 Wind loads

The wind loads which act on structures have traditionally been allowed for by using static force equivalents. The first step is usually to determine a basic wind speed for the general region in which the structure is to be built by using information derived from meteorological studies. This basic wind speed may represent an extreme velocity measured at a height of 10 m and averaged over a period of 3 seconds which has a return period of 50 years (i.e. a velocity which will, on average, be reached or exceeded once in 50 years, or have a probability of being exceeded of 1/50). The basic wind speed may be adjusted to account for the topography of the site, for the ground roughness, structure size, and height above ground, and for the degree of safety required and the period of exposure. The resulting design wind speed may then be converted into the static pressure which will be exerted by the wind on a plane surface area (this is often referred to as the dynamic wind pressure because it is produced by decelerating the approaching wind velocity to zero at the surface area). The wind force acting on the structure may then be calculated by using pressure coefficients appropriate to each individual surface of the structure, or by using force coefficients appropriate to the total structure. Many values of these coefficients are tabulated in [15], but in special cases where these are inappropriate, the results of wind tunnel tests on model structures may be used.

In some cases it is not sufficient to treat wind loads as static forces. For example, when fatigue is a problem, both the magnitudes and the number of wind fluctuations must be estimated. In other cases, the dynamic response of a structure to wind loads may have to be evaluated (this is often the case with very flexible structures whose long natural periods of vibration are close to those of some of the wind gusts), and this may be done analytically [37, 38], or by specialists using wind tunnel tests. In these cases, special care must be taken to model correctly those properties of the structure which affect its response, including its mass, stiffness, and damping, as well as the wind characteristics and any interactions between wind and structure.

1.5.5 Earth or ground-water loads

Earth or ground-water loads act as pressure loads normal to the contact surface of the structure. Such loads are usually considered to be essentially static.

However, earthquake loads are dynamic in nature, and their effects on the structure must be allowed for. Very flexible structures with long natural periods of vibration respond in an equivalent static manner to the high frequencies of earthquake movements, and so can be designed as if loaded by static force equivalents.
On the other hand, stiff structures with short natural periods of vibration respond significantly, and so in such a case a proper dynamic analysis [37, 38] should be made. The intensities of earthquake loads vary with the region in which the structure is to be built, but they are not considered to be significant in the UK.

1.5.6 Indirect forces

Indirect forces may be described as those forces which result from the straining of a structure or its components, and may be distinguished from the direct forces caused by the dead and applied loads and pressures. The straining may arise from temperature changes, from foundation settlement, from shrinkage, creep, or cracking of structural or other materials, and from the manufacturing process as in the case of residual stresses. The values of indirect forces are not usually specified, and so it is common for the designer to determine which of these forces should be allowed for, and what force magnitudes should be adopted.

1.5.7 Combinations of loads

The different loads discussed in the preceding subsections do not occur alone, but in combinations, and so the designer must determine which combination is the most critical for the structure. However, if the individual loads, which have different probabilities of occurrence and degrees of variability, were combined directly, the resulting load combination would have a greatly reduced probability. Thus, it is logical to reduce the magnitudes of the various components of a combination according to their probabilities of occurrence. This is similar to the procedure used in reducing the imposed load intensities used over large areas.

The past design practice was to use the worst combination of dead load with imposed load and/or wind load, and to allow increased stresses whenever the wind load was included (which is equivalent to reducing the load magnitudes). These increases seem to be logical when imposed, and wind loads act together because the probability that both of these loads will attain their maximum values simultaneously is greatly reduced. However, they are unjustified when applied in the case of dead and wind load, for which the probability of occurrence is virtually unchanged from that of the wind load.

A different and more logical method of combining loads is used in the EC3 limit states design method [8], which is based on statistical analyses of the loads and the structure capacities (see Section 1.7.3.4). Strength design is usually carried out for the most severe combination of actions for normal (termed persistent) or temporary (termed transient) conditions using

$$\sum_{j \geq 1} \gamma_{G,j} G_{k,j} + \gamma_{Q,1} Q_{k,1} + \sum_{i > 1} \gamma_{Q,i} \psi_0, i Q_{k,i}$$

(1.6)

where $\Sigma$ implies “the combined effect of”, $\gamma_G$ and $\gamma_Q$ are partial factors for the persistent $G$ and variable $Q$ actions, and $\psi_0$ is a combination factor. The concept is
### Table 1.1 Partial load factors for common situations

<table>
<thead>
<tr>
<th>Ultimate limit state</th>
<th>Permanent actions $\gamma_G$</th>
<th>Variable actions $\gamma_Q$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Unfavourable</td>
<td>Favourable</td>
</tr>
<tr>
<td>EQU</td>
<td>1.1</td>
<td>0.9</td>
</tr>
<tr>
<td>STR/GEO</td>
<td>1.35</td>
<td>1.0</td>
</tr>
</tbody>
</table>

thus to use all the persistent actions $G_{k,j}$ such as self-weight and fixed equipment with a leading variable action $Q_{k,1}$ such as imposed, snow, or wind load, and reduced values of the other variable actions $Q_{k,i}$. More information on the Eurocode approach to loading for steel structures is given in [39–41].

This approach is applied to the following forms of ultimate limit state:

- **EQU** = loss of static equilibrium of the structure on any part of it
- **STR** = failure by excessive deformation, transformation of the structure or any part of it into a mechanism, rupture or loss of stability of the structure or of any part of it
- **GEO** = failure or excessive deformation of the ground
- **FAT** = fatigue failure

For the most common set of design situations, use of the appropriate values from [41] gives the load factors of Table 1.1.

Using the combination factors of $\psi_0 = 0.7$ and 0.5 of [41] for most variable actions and wind actions, respectively leads to the following common STR load combinations for buildings:

- $1.35\ G + 1.5\ Q_I + 0.75\ Q_W$
- $1.35\ G + 1.05\ Q_I + 1.5\ Q_W$
- $-1.0\ G + 1.5\ Q_I$, and
- $-1.0\ G + 1.5\ Q_W$.

in which the minus signs indicate that the permanent action is favourable.

### 1.6 Analysis of steel structures

#### 1.6.1 General

In the design process, the assessment of whether the structural design requirements will be met or not requires the knowledge of the stiffness and strength of the structure under load, and of its local stresses and deformations. The term structural analysis is used to denote the analytical process by which this knowledge
of the response of the structure can be obtained. The basis for this process is the knowledge of the material behaviour, and this is used first to analyse the behaviour of the individual members and joints of the structure. The behaviour of the complete structure is then synthesised from these individual behaviours.

The methods of structural analysis are fully treated in many textbooks [e.g. 25–27], and so details of these are not within the scope of this book. However, some discussion of the concepts and assumptions of structural analysis is necessary so that the designer can make appropriate assumptions about the structure and make a suitable choice of the method of analysis.

In most methods of structural analysis, the distribution of forces and moments throughout the structure is determined by using the conditions of static equilibrium and of geometric compatibility between the members at the joints. The way in which this is done depends on whether a structure is statically determinate (in which case the complete distribution of forces and moments can be determined by statics alone), or is statically indeterminate (in which case the compatibility conditions for the deformed structure must also be used before the analysis can be completed).

An important feature of the methods of structural analysis is the constitutive relationships between the forces and moments acting on a member or connection and its deformations. These play the same role for the structural element as do the stress–strain relationships for an infinitesimal element of a structural material. The constitutive relationship may be linear (force proportional to deflection) and elastic (perfect recovery on unloading), or they may be non-linear because of material non-linearities such as yielding (inelastic), or because of geometrical non-linearities (elastic) such as when the deformations themselves induce additional moments, as in stability problems.

It is common for the designer to idealise the structure and its behaviour so as to simplify the analysis. A three-dimensional frame structure may be analysed as the group of a number of independent two-dimensional frames, while individual members are usually considered as one-dimensional and the joints as points. The joints may be assumed to be frictionless hinges, or to be semi-rigid or rigid. In some cases, the analysis may be replaced or supplemented by tests made on an idealised model which approximates part or all of the structure.

### 1.6.2 Analysis of statically determinate members and structures

For an isolated statically determinate member, the forces and moments acting on the member are already known, and the structural analysis is only used to determine the stiffness and strength of the member. A linear elastic (or first-order elastic) analysis is usually made of the stiffness of the member when the material non-linearities are generally unimportant and the geometrical non-linearities are often small. The strength of the member, however, is not so easily determined, as one or both of the material and geometric non-linearities are most important. Instead,
the designer usually relies on a design code or specification for this information. The strength of the isolated statically determinate members is fully discussed in Chapters 2–7 and 10.

For a statically determinate structure, the principles of static equilibrium are used in the structural analysis to determine the member forces and moments, and the stiffness and strength of each member are then determined in the same way as for statically determinate members.

1.6.3 Analysis of statically indeterminate structures

A statically indeterminate structure can be approximately analysed if a sufficient number of assumptions are made about its behaviour to allow it to be treated as if determinate. One method of doing this is to guess the locations of points of zero bending moment and to assume there are frictionless hinges at a sufficient number of these locations that the member forces and moments can be determined by statics alone. Such a procedure is commonly used in the preliminary analysis of a structure, and followed at a later stage by a more precise analysis. However, a structure designed only on the basis of an approximate analysis can still be safe, provided the structure has sufficient ductility to redistribute any excess forces and moments. Indeed, the method is often conservative, and its economy increases with the accuracy of the estimated locations of the points of zero bending moment. More commonly, a preliminary analysis is made of the structure based on the linear elastic computer methods of analysis [42, 43], using approximate member stiffnesses.

The accurate analysis of statically indeterminate structures is complicated by the interaction between members: the equilibrium and compatibility conditions and the constitutive relationships must all be used in determining the member forces and moments. There are a number of different types of analysis which might be made, and some indication of the relevance of these is given in Figure 1.15 and in the following discussion. Many of these can only be used for two-dimensional frames.

For many structures, it is common to use a first-order elastic analysis which is based on linear elastic constitutive relationships and which ignores any geometrical non-linearities and associated instability problems. The deformations determined by such an analysis are proportional to the applied loads, and so the principle of superposition can be used to simplify the analysis. It is often assumed that axial and shear deformations can be ignored in structures whose action is predominantly flexural, and that flexural and shear deformations can be ignored in structures whose member forces are predominantly axial. These assumptions further simplify the analysis, which can then be carried out by any of the well-known methods [25–27], for which many computer programs are available [44, 45]. Some of these programs can be used for three-dimensional frames.

However, a first-order elastic analysis will underestimate the forces and moments in and the deformations of a structure when instability effects are present.
Some estimate of the importance of these in the absence of flexural effects can be obtained by making an elastic stability analysis. A second-order elastic analysis accounts for both flexure and instability, but this is difficult to carry out, although computer programs are now generally available [44, 45]. EC3 permits the use of the results of an elastic stability analysis in the amplification of the first-order moments as an alternative to second-order analysis.

The analysis of statically indeterminate structures near the ultimate load is further complicated by the decisive influence of the material and geometrical non-linearities. In structures without material non-linearities, an elastic stability analysis is appropriate when there are no flexural effects, but this is a rare occurrence. On the other hand, many flexural structures have very small axial forces and instability effects, in which case it is comparatively easy to use a first-order plastic analysis, according to which a sufficient number of plastic hinges must form to transform the structure into a collapse mechanism.

More generally, the effects of instability must be allowed for, and as a first approximation the nominal first yield load determined from a second-order elastic analysis may be used as a conservative estimate of the ultimate load. A much more accurate estimate may be obtained for structures where local and lateral buckling is prevented by using an advanced analysis [46] in which the actual behaviour is closely analysed by allowing for instability, yielding, residual stresses, and initial crookedness. However, this method is not yet in general use.

1.7 Design of steel structures

1.7.1 Structural requirements and design criteria

The designer’s task of assessing whether or not a structure will satisfy the structural requirements of serviceability and strength is complicated by the existence of errors
and uncertainties in his or her analysis of the structural behaviour and estimation of
the loads acting, and even in the structural requirements themselves. The designer
usually simplifies this task by using a number of design criteria which allow him or
her to relate the structural behaviour predicted by his or her analysis to the structural
requirements. Thus the designer equates the satisfaction of these criteria by the
predicted structural behaviour with satisfaction of the structural requirements by
the actual structure.

In general, the various structural design requirements relate to corresponding
limit states, and so the design of a structure to satisfy all the appropriate require-
ments is often referred to as a limit states design. The requirements are commonly
presented in a deterministic fashion, by requiring that the structure shall not fail,
or that its deflections shall not exceed prescribed limits. However, it is not possible
to be completely certain about the structure and its loading, and so the structural
requirements may also be presented in probabilistic forms, or in deterministic
forms derived from probabilistic considerations. This may be done by defining an
acceptably low risk of failure within the design life of the structure, after reaching
some sort of balance between the initial cost of the structure and the economic and
human losses resulting from failure. In many cases there will be a number of struc-
tural requirements which operate at different load levels, and it is not unusual to
require a structure to suffer no damage at one load level, but to permit some minor
damage to occur at a higher load level, provided there is no catastrophic failure.

The structural design criteria may be determined by the designer, or he or she
may use those stated or implied in design codes. The stiffness design criteria
adopted are usually related to the serviceability limit state of the structure under
the service loads, and are concerned with ensuring that the structure has sufficient
stiffness to prevent, excessive deflections such as sagging, distortion, and settle-
ment, and excessive motions under dynamic load, including sway, bounce, and
vibration.

The strength limit state design criteria are related to the possible methods of
failure of the structure under overload and understrength conditions, and so these
design criteria are concerned with yielding, buckling, brittle fracture, and fatigue.
Also of importance is the ductility of the structure at and near failure: ductile
structures give a warning of the impending failure and often redistribute the load
effects away from the critical regions, while ductility provides a method of energy
dissipation which will reduce the damage due to earthquake and blast loading. On
the other hand, a brittle failure is more serious, as it occurs with no warning of
failure, and in a catastrophic fashion with a consequent release of stored energy
and increase in damage. Other design criteria may also be adopted, such as those
related to corrosion and fire.

1.7.2 Errors and uncertainties

In determining the limitations prescribed by design criteria, account must be taken
of the deliberate and accidental errors made by the designer, and of the uncertainties
in his or her knowledge of the structure and its loads. Deliberate errors include those resulting from the assumptions made to simplify the analysis of the loading and of the structural behaviour. These assumptions are often made so that any errors involved are on the safe side, but in many cases the nature of the errors involved is not precisely known, and some possibility of danger exists.

Accidental errors include those due to a general lack of precision, either in the estimation of the loads and the analysis of the structural behaviour, or in the manufacture and erection of the structure. The designer usually attempts to control the magnitudes of these, by limiting them to what he or she judges to be suitably small values. Other accidental errors include what are usually termed blunders. These may be of a gross magnitude leading to failure or to uneconomic structures, or they may be less important. Attempts are usually made to eliminate blunders by using checking procedures, but often these are unreliable, and the logic of such a process is open to debate.

As well as the errors described above, there exist a number of uncertainties about the structure itself and its loads. The material properties of steel fluctuate, especially the yield stress and the residual stresses. The practice of using a minimum or characteristic yield stress for design purposes usually leads to oversafe designs, especially for redundant structures of reasonable size, for which an average yield stress would be more appropriate because of the redistribution of load which takes place after early yielding. Variations in the residual stress levels are not often accounted for in design codes, but there is a growing tendency to adjust design criteria in accordance with the method of manufacture so as to make some allowance for gross variations in the residual stresses. This is undertaken to some extent in EC3.

The cross-sectional dimensions of rolled-steel sections vary, and the values given in section handbooks are only nominal, especially for the thicknesses of universal sections. The fabricated lengths of a structural member will vary slightly from the nominal length, but this is usually of little importance, except where the variation induces additional stresses because of lack-of-fit problems, or where there is a cumulative geometrical effect. Of some significance to members subject to instability problems are the variations in their straightness which arise during manufacture, fabrication, and erection. Some allowances for these are usually made in design codes, while fabrication and erection tolerances are specified in EN1090 [47] to prevent excessive crookedness.

The loads acting on a structure vary significantly. Uncertainty exists in the designer’s estimate of the magnitude of the dead load because of the variations in the densities of materials, and because of the minor modifications to the structure during or subsequent to its erection. Usually these variations are not very significant and a common practice is to err on the safe side by making conservative assumptions. Imposed loadings fluctuate significantly during the design usage of the structure, and may change dramatically with changes in usage. These fluctuations are usually accounted for by specifying what appear to be extreme values in loading codes, but there is often a finite chance that these values will be exceeded.
Wind loads vary greatly and the magnitudes specified in loading codes are usually obtained by probabilistic methods.

1.7.3 **Strength design**

1.7.3.1 *Load and capacity factors, and factors of safety*

The errors and uncertainties involved in the estimation of the loads on and the behaviour of a structure may be allowed for in strength design by using load factors to increase the nominal loads and capacity factors to decrease the structural strength. In the previous codes that employed the traditional working stress design, this was achieved by using factors of safety to reduce the failure stresses to permissible working stress values. The purpose of using various factors is to ensure that the probability of failure under the most adverse conditions of structural overload and understrength remains very small. The use of these factors is discussed in the following subsections.

1.7.3.2 *Working stress design*

The working stress methods of design given in previous codes and specifications required that the stresses calculated from the most adverse combination of loads must not exceed the specified permissible stresses. These specified stresses were obtained after making some allowances for the non-linear stability and material effects on the strength of isolated members, and in effect, were expressions of their ultimate strengths divided by the factors of safety SF. Thus

\[
\text{Working stress} \leq \text{Permissible stress} \approx \frac{\text{Ultimate stress}}{\text{SF}} \quad (1.6)
\]

It was traditional to use factors of safety of 1.7 approximately.

The working stress method of a previous steel design code [48] has been replaced by the limit states design method of EC3. Detailed discussions of the working stress method are available in the first edition of this book [49].

1.7.3.3 *Ultimate load design*

The ultimate load methods of designing steel structures required that the calculated ultimate load-carrying capacity of the complete structure must not exceed the most adverse combination of the loads obtained by multiplying the working loads by the appropriate load factors LF. Thus

\[
\sum (\text{Working load} \times \text{LF}) \leq \text{Ultimate load} \quad (1.7)
\]

These load factors allowed some margins for any deliberate and accidental errors, and for the uncertainties in the structure and its loads, and also provided the
structure with a reserve of strength. The values of the factors should depend on the load type and combination, and also on the risk of failure that could be expected and the consequences of failure. A simplified approach often employed (perhaps illogically) was to use a single load factor on the most adverse combination of the working loads.

A previous code [48] allowed the use of the plastic method of ultimate load design when stability effects were unimportant. These have used load factors of 1.70 approximately. However, this ultimate load method has also been replaced by the limit states design method in EC3, and will not be discussed further.

1.7.3.4 Limit states design

It was pointed out in Section 1.5.6 that different types of load have different probabilities of occurrence and different degrees of variability, and that the probabilities associated with these loads change in different ways as the degree of overload considered increases. Because of this, different load factors should be used for the different load types.

Thus for limit states design, the structure is deemed to be satisfactory if its design load effect does not exceed its design resistance. The design load effect is an appropriate bending moment, torque, axial force, or shear force, and is calculated from the sum of the effects of the specified (or characteristic) loads $F_k$ multiplied by partial factors $\gamma_G, \gamma_Q$ which allow for the variabilities of the loads and the structural behaviour. The design resistance $R_k/\gamma_M$ is calculated from the specified (or characteristic) resistance $R_k$ divided by the partial factor $\gamma_M$ which allows for the variability of the resistance. Thus

\[
\text{Design load effect} \leq \text{Design resistance} \quad (1.8a)
\]

or

\[
\sum \gamma_{g,Q} \times (\text{effect of specified loads}) \leq (\text{specified resistance}/\gamma_M) \quad (1.8b)
\]

Although the limit states design method is presented in deterministic form in equations 1.8, the partial factors involved are usually obtained by using probabilistic models based on statistical distributions of the loads and the capacities. Typical statistical distributions of the total load and the structural capacity are shown in Figure 1.16. The probability of failure $p_F$ is indicated by the region for which the load distribution exceeds that for the structural capacity.

In the development of limit state codes, the probability of failure $p_F$ is usually related to a parameter $\beta$, called the safety index, by the transformation

\[
\Phi(-\beta) = p_F, \quad (1.9)
\]

where the function $\Phi$ is the cumulative frequency distribution of a standard normal variate [35]. The relationship between $\beta$ and $p_F$ shown in Figure 1.17 indicates
that an increase in $\beta$ of 0.5 implies a decrease in the probability of failure by approximately an order of magnitude.

The concept of the safety index was used to derive the partial factors for EC3. This was done with reference to previous national codes such as BS 5950 [50] to obtain comparable values of the probability of failure $p_F$, although much of the detailed calibration treated the load and resistance sides of equations 1.8 separately.
1.7.4 Stiffness design

In the stiffness design of steel structures, the designer seeks to make the structure sufficiently stiff so that its deflections under the most adverse working load conditions will not impair its strength or serviceability. These deflections are usually calculated by a first-order linear elastic analysis, although the effects of geometrical non-linearities should be included when these are significant, as in structures which are susceptible to instability problems. The design criteria used in the stiffness design relate principally to the serviceability of the structure, in that the flexibility of the structure should not lead to damage of any non-structural components, while the deflections should not be unsightly, and the structure should not vibrate excessively. It is usually left to the designer to choose limiting values for use in these criteria which are appropriate to the structure, although a few values are suggested in some design codes. The stiffness design criteria which relate to the strength of the structure itself are automatically satisfied when the appropriate strength design criteria are satisfied.

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Tension members


Compression members

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In-plane bending of beams


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