Section 2. Limit states design

2.1 General principles and design methods

2.1.1 General principles

2.1.1.1 ***Aims of structural design***

The aim of structural design should be to provide, with due regard to economy, a structure capable of fulfilling its intended function and sustaining the specified loads for its intended life. The design should facilitate safe fabrication, transport, handling and erection. It should also take account of the needs of future maintenance, final demolition, recycling and reuse of materials.

The structure should be designed to behave as a one three-dimensional entity. The layout of its constituent parts, such as foundations, steelwork, joints and other structural components should constitute a robust and stable structure under normal loading to ensure that, in the event of misuse or accident, damage will not be disproportionate to the cause.

To achieve these aims the basic anatomy of the structure by which the loads are transmitted to the foundations should be clearly defined. Any features of the structure that have a critical influence on its overall stability should be identified and taken account of in the design.

Each part of the structure should be sufficiently robust and insensitive to the effects of minor incidental loads applied during service that the safety of other parts is not prejudiced. Reference should be made to **2.4.5**.

Whilst the ultimate limit state capacities and resistances given in this standard are to be regarded as limiting values, the purpose in design should be to reach these limits in as many parts of the structure as possible, to adopt a layout such that maximum structural efficiency is attained and to rationalize the steel member sizes and details in order to obtain the optimum combination of materials and workmanship, consistent with the overall requirements of the structure.

2.1.1.2 ***Overall stability***

The designer who is responsible for the overall stability of the structure should be clearly identified. This designer should ensure the compatibility of the structural design and detailing between all those structural parts and components that are required for overall stability, even if some or all of the structural design and detailing of those structural parts and components is carried out by another designer.

2.1.1.3 ***Accuracy of calculation***

For the purpose of deciding whether a particular recommendation is satisfied, the final value, observed or calculated, expressing the result of a test or analysis should be rounded off. The number of significant places retained in the rounded off value should be the same as in the relevant value recommended in this standard.

2.1.2 Methods of design

2.1.2.1 ***General***

Structures should be designed using the methods given in **2.1.2.2**, **2.1.2.3**, **2.1.2.4** and **2.1.2.5**.

In each case the details of the joints should be such as to fulfil the assumptions made in the relevant design method, without adversely affecting any other part of the structure.

2.1.2.2 ***Simple design***

The joints should be assumed not to develop moments adversely affecting either the members or the structure as a whole.

The distribution of forces may be determined assuming that members intersecting at a joint are pin connected. The necessary flexibility in the connections may result in some non-elastic deformation of the materials, other than the bolts.

The structure should be laterally restrained, both in-plane and out-of-plane, to provide sway stability, see **2.4.2.5**, and resist horizontal forces, see **2.4.2.3**.

2.1.2.3 ***Continuous design***

Either elastic or plastic analysis may be used.

For elastic analysis the joints should have sufficient rotational stiffness to justify analysis based on full continuity. The joints should also be capable of resisting the moments and forces resulting from the analysis.

For plastic analysis the joints should have sufficient moment capacity to justify analysis assuming plastic hinges in the members. The joints should also have sufficient rotational stiffness for in-plane stability.

2.1.2.4 ***Semi-continuous design***

This method may be used where the joints have some degree of strength and stiffness, but insufficient to develop full continuity. Either elastic or plastic analysis may be used.

The moment capacity, rotational stiffness and rotation capacity of the joints should be based on experimental evidence. This may permit some limited plasticity, provided that the capacity of the bolts or welds is not the failure criterion. On this basis, the design should satisfy the strength, stiffness and in-plane stability requirements of all parts of the structure when partial continuity at the joints is taken into account in determining the moments and forces in the members.

NOTE Details of design procedures of this type are given in references [1] and [2], see Bibliography.

2.1.2.5 ***Experimental verification***

Where design of a structure or element by calculation in accordance with any of the preceding methods is not practicable, or is inappropriate, the strength, stability, stiffness and deformation capacity may be confirmed by appropriate loading tests in accordance with Section 7.

2.1.3 Limit states concept

Structures should be designed by considering the limit states beyond which they would become unfit for their intended use. Appropriate partial factors should be applied to provide adequate degrees of reliability for ultimate limit states and serviceability limit states. Ultimate limit states concern the safety of the whole or part of the structure. Serviceability limit states correspond to limits beyond which specified service criteria are no longer met.

Examples of limit states relevant to steel structures are given in Table 1. In design, the limit states relevant to that structure or part should be considered.

The overall factor in any design has to cover variability of:

— material strength: γ_m

— loading: γ_ℓ

— structural performance: γ_p

In this code the material factor γ_m is incorporated in the recommended design strengths. For structural steel the material factor is taken as 1.0 applied to the yield strength Y_s or 1.2 applied to the tensile strength U_s. Different values are used for bolts and welds.

The values assigned for γ_ℓ and γ_p depend on the type of load and the load combination. Their product is the factor γ_f by which the specified loads are to be multiplied in checking the strength and stability of a structure, see **2.4**. A detailed breakdown of γ factors is given in Annex A.

Table 1 — Limit states

Ultimate limit states (ULS)	Serviceability limit states (SLS)
Strength (including general yielding, rupture, buckling and forming a mechanism), see **2.4.1**.	Deflection, see **2.5.2**.
Stability against overturning and sway stability, see **2.4.2**.	Vibration, see **2.5.3**.
Fracture due to fatigue, see **2.4.3**.	Wind induced oscillation, see **2.5.3**.
Brittle fracture, see **2.4.4**.	Durability, see **2.5.4**.

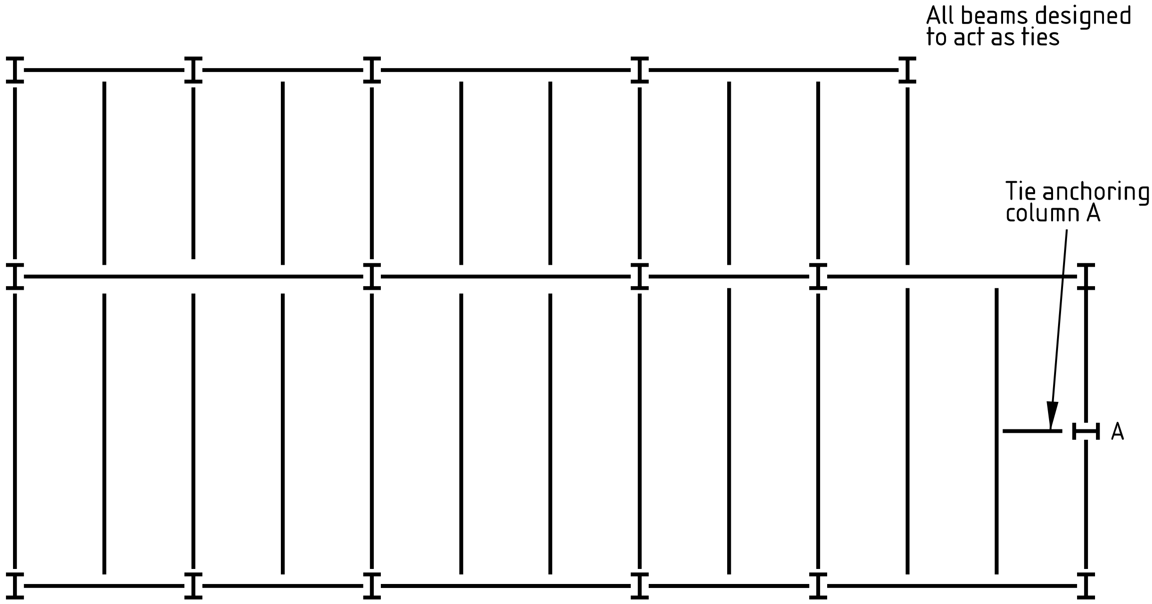

Figure 2 — Example of general tying of a building

2.4.5.4 ***Key elements***

In a multi-storey building that is required by regulations to be designed to avoid disproportionate collapse, a member that is recommended in **2.4.5.3** to be designed as a key element should be designed for the accidental loading specified in BS 6399-1.

Any other steel member or other structural component that provides lateral restraint vital to the stability of a key element should itself also be designed as a key element for the same accidental loading.

The accidental loading should be applied to the member from all horizontal and vertical directions, in one direction at a time, together with the reactions from other building components attached to the member that are subject to the same accidental loading, but limited to the maximum reactions that could reasonably be transmitted, considering the breaking resistances of such components and their connections.

In this check the effects of ordinary loads should also be considered, to the same extent and with the same partial factor γ_f as recommended in **2.4.5.3**.

2.5 Serviceability limit states

2.5.1 Serviceability loads

Generally the serviceability loads should be taken as the unfactored specified values. However, exceptional snow load (due to local drifting on roofs, see **7.4** in BS 6399-3:1988) should not be included in the imposed load when checking serviceability.

In the case of combined imposed load and wind load, only 80 % of the full specified values need be considered when checking serviceability. In the case of combined horizontal crane loads and wind load, only the greater effect need be considered when checking serviceability.

2.5.2 Deflection

The deflections of a building or part under serviceability loads should not impair the strength or efficiency of the structure or its components, nor cause damage to the finishings.

When checking for deflections the most adverse realistic combination and arrangement of serviceability loads should be assumed, and the structure may be assumed to behave elastically.

4.2.3 Shear capacity

The shear force F_v should not be greater than the shear capacity P_v given by:

$P_v = 0.6p_yA_v$

in which A_v is the shear area, taken as follows:

a) rolled I, H and channel sections, load parallel to web:	tD
b) welded I-sections, load parallel to web:	td
c) rectangular hollow sections, load parallel to webs:	$AD/(D + B)$
d) welded box sections, load parallel to webs:	$2td$
e) rolled T-sections, load parallel to web:	tD
f) welded T-sections, load parallel to web:	$t(D - T)$
g) circular hollow sections:	$0.6A$
h) solid bars and plates:	$0.9A$
i) any other case:	$0.9A_o$

where

A is the area of the cross-section;

A_o is the area of that rectilinear element of the cross-section which has the largest dimension in the direction parallel to the shear force;

B is the overall breadth;

D is the overall depth;

d is the depth of the web;

t is the web thickness.

In CHS and RHS sections the shear area should be assumed to be located adjacent to the neutral axis.

For the effect of bolt holes on shear capacity, reference should be made to **6.2.3**.

If the ratio d/t exceeds 70ε for a rolled section, or 62ε for a welded section, the web should be checked for shear buckling in accordance with **4.4.5**.

4.2.4 Elastic shear stress

In cross-sections with webs that vary in thickness the distribution of shear stresses should be calculated from first principles assuming linear elastic behaviour. In this case the peak value of the shear stress distribution should not exceed $0.7p_y$. For cross-sections with openings significantly larger than those normally required for bolts, reference should be made to **4.15**.

4.2.5 Moment capacity

4.2.5.1 *General*

The moment capacity M_c should be determined from **4.2.5.2**, **4.2.5.3** and **4.2.5.4** allowing for the effects of co-existing shear. The effects of bolt holes should be allowed for as detailed in **4.2.5.5**.

To avoid irreversible deformation under serviceability loads, the value of M_c should be limited to $1.5p_yZ$ generally and to $1.2p_yZ$ in the case of a simply supported beam or a cantilever.

4.2.5.2 ***Low shear***

Provided that the shear force F_v does not exceed 60 % of the shear capacity P_v:

— for class 1 plastic or class 2 compact cross-sections:

$M_c = p_y S$

— for class 3 semi-compact sections:

$M_c = p_y Z$ or alternatively $M_c = p_y S_{eff}$

— for class 4 slender cross-sections:

$M_c = p_y Z_{eff}$

where

S is the plastic modulus;

S_{eff} is the effective plastic modulus, see **3.5.6**;

Z is the section modulus;

Z_{eff} is the effective section modulus, see **3.6.2**.

4.2.5.3 ***High shear***

Where $F_v > 0.6P_v$:

— for class 1 plastic or class 2 compact cross-sections:

$M_c = p_y(S - \rho S_v)$

— for class 3 semi-compact cross-sections:

$M_c = p_y(Z - \rho S_v/1.5)$ or alternatively $M_c = p_y(S_{eff} - \rho S_v)$

— for class 4 slender cross-sections:

$M_c = p_y(Z_{eff} - \rho S_v/1.5)$

in which S_v is obtained from the following:

— for sections with unequal flanges:

$S_v = S - S_f$

in which S_f is the plastic modulus of the effective section excluding the shear area A_v defined in **4.2.3**;

— otherwise:

S_v is the plastic modulus of the shear area A_v defined in **4.2.3**;

and ρ is given by:

$\rho = [2(F_v/P_v) - 1]^2$

NOTE The reduction factor ρ starts when F_v exceeds $0.5P_v$ but the resulting reduction in moment capacity is negligible unless F_v exceeds $0.6P_v$.

Alternatively, for class 3 semi-compact cross-sections reference may be made to **H.3**, or for class 4 slender cross-sections reference may be made to **3.6** and **H.3**.

If the ratio d/t exceeds 70ε for a rolled section, or 62ε for a welded section, the moment capacity should be determined allowing for shear buckling in accordance with **4.4.4**.

4.2.5.4 ***Notched ends***

For notched ends of I, H or channel section members the moment capacity M_c should be taken as follows.

a) *Low shear:* where $F_v \leq 0.75P_v$:

— for singly notched ends:

$$M_c = p_y Z$$

— for doubly notched ends:

$$M_c = p_y td^2/6$$

b) *High shear:* where $F_v > 0.75P_v$:

— for singly notched ends:

$$M_c = 1.5 p_y Z \sqrt{1 - (F_v / P_v)^2}$$

— for doubly notched ends:

$$M_c = (p_y td^2 / 4) \sqrt{1 - (F_v / P_v)^2}$$

where

d is the residual depth of a doubly notched end;

Z is the relevant section modulus of the residual tee at a singly notched end.

4.2.5.5 ***Bolt holes***

No allowance need be made for bolt holes in a compression flange (or leg). No allowance need be made for bolt holes in a tension flange (or leg) if, for the tension element:

$$a_{t.net} \geq a_t / K_e$$

where

a_t is the area of the tension element;

$a_{t.net}$ is the net area of the tension element after deducting bolt holes;

K_e is the factor for effective net area given in **3.4.3**.

No allowance need be made for bolt holes in the tension zone of a web unless there are also bolt holes in the tension flange at the same location. Furthermore, no allowance need be made for bolt holes in a web if the condition given above is satisfied when both a_t and $a_{t.net}$ are based upon the complete tension zone, comprising the tension flange plus the tension zone of the web.

If $a_{t.net}$ is less than a_t/K_e then an effective net area of $K_e a_{t.net}$ may be used.

4.3 Lateral-torsional buckling

4.3.1 General

Unless a beam or cantilever has full lateral restraint to its compression flange as described in **4.2.2**, then in addition to satisfying **4.2** its resistance to lateral-torsional buckling should also be checked.

Generally the resistance of a member to lateral-torsional buckling should be checked as detailed in **4.3.2**, **4.3.3**, **4.3.4**, **4.3.5**, **4.3.6**, **4.3.7** and **4.3.8**. However, for members that satisfy the conditions given in **G.1**, advantage may be taken of the methods for members with one flange restrained given in **G.2**.

4.3.2 Intermediate lateral restraints

4.3.2.1 ***General***

If a member that is subject to bending needs intermediate lateral restraints within its length in order to develop the required buckling resistance moment, these restraints should have sufficient stiffness and strength to inhibit lateral movement of the compression flange relative to the supports.

