The Institution of Structural Engineers
The Institution of Civil Engineers

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# Manual for the design of steelwork building structures to EC3

Published for the Institution of Structural Engineers

# Constitution



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# Contents

# Foreword

The prestandard (ENV) version of the Eurocode for the Design of Steel Structures, EC3 has been available since 1992 and the Euronorm version (EN) is expected to be published shortly. The Institution of Structural Engineers and the Institution of Civil Engineers decided to update the 1989 'grey' book *Manual for the design of steelwork building structures* as a guidance for using EC3 and a Task Group was constituted for this purpose.

This *Manual* is intended to provide guidance on the design of many common steel building frames and to show how the provisions of EC3 can be selected for this purpose. Certain limitations have been introduced both to simplify the design process and to select from the comprehensive provisions of EC3 in order to aid in the process of familiarisation. It is the Committee's view that it will provide satisfactory design procedures for the majority of steel framed buildings that will lie within its scope and defined limitations.

The Committee has drawn heavily on the 1989 grey book for the scope and contents of this update. The advice contained in the earlier publication remains a useful aid for design to BS 5950. However, in the course of its work, the opportunity has been taken to take account of more recent research and of comments received particularly relating to the effective length of columns. Attention is drawn to an important amendment to the 1989 grey book published in *The Structural Engineer* Vol. 76, No 12, dated 16 June 1998.

The mandate for the Task Group was to produce a *Manual*, the guidance of which would comply with ENV EC3 and the UK National Application Document (NAD). This imposed certain constraints. In particular we have adopted the UK boxed values, refer to British Standard loading codes (used for calibrating EC3) and to BS 8110 for the fire design. During the drafting process, further ENV standards have been published, in particular for loading (EC1) and for fire design (EC3, Pt 1.2). The designer can use this *Manual* with these ENV pre-standards in engineering with their own NADs.

Thanks are due to all members of the Task Group (and their organisations) who have given their valuable time voluntarily during a period of economic pressure in the construction industry. I am grateful for the assistance given by Institution staff and particularly to Bob Milne and latterly Terence Gray, who have acted as secretaries to the Task Group. Members of the Institutions have provided useful comments as the *Manual* proceeded throughout the drafting process and have contributed to its present format. Users of the *Manual* are encouraged to send further comments to the Institution so that these can be taken into account in the next revision.

Brian Simpson
*Chairman*

# 1 Introduction

## 1.1 Aims of the *Manual*

This *Manual* provides guidance on the design of single and multistorey building structures using structural steelwork. Structures designed in accordance with this *Manual* will normally comply with *Eurocode 3; Design of steel structures,* Part 1.1 *General rules and rules for buildings*[1] (together with United Kingdom National Application Document, see 1.6* for explanation of National Application Document) published as a draft for development with the reference DD ENV 1993–1–1: 1992[1] (and hereinafter referred to as EC3).

## 1.2 Scope of the *Manual*

The range of structures covered by the *Manual* are:

- braced multistorey structures that do not rely on bending resistance of columns for their overall stability
- single-storey structures using portal frames, posts and latticed trusses or posts and pitched roof trusses

EC3 also covers structures which are outside the scope of this *Manual*.

## 1.3 Limitations and assumptions

To simplify the application of the Eurocode which is written for a wide range of materials, it has been assumed that:

- steel will be of two basic grades; S 275 and S 355 material less than 40mm thickness
- steel bolts for connections will be generally grade 8.8.

As a further simplification, the values of $f_y$ and $f_u$ have been taken for material that is less than 40mm thickness. If material thicker than 40mm is to be used, the values given in EC3 must be used.

Note that the thickness of universal sections, both beams and columns is taken as the thickness of the flanges in the section tables.

Elastic analysis is assumed except for the design of portal frames where the traditional methods of plastic design are covered.

The design of Class 4 cross-sections (see 1.9) is not covered by this *Manual*, and these have been excluded from the tables.

## 1.4 Contents of the *Manual*

The *Manual* covers the following:

- guidance on structural form, framing and bracing, including advice on the selection of floors, roofing and cladding systems, and advice on deflection, thermal expansion, fire and corrosion protection
- step-by-step procedures for designing the different types of structure and structural elements including verification of robustness and design of connections.

* Other clauses in this document are referenced by numbers i.e. 1.6. Where clauses in EC3 are referenced then they are noted as EC3 5.5.

## 1.5 General format of the *Manual*

In the design of structural steelwork it is not practical to include all the information necessary for section design within the covers of one book. Section properties and capacities have been included in the *Manual* where appropriate, but nevertheless reference will need to be made to *Design Guide to Eurocode 3 Section Properties and Member Resistances*[2] SCI Publication No. 1582.

## 1.6 Eurocode system

The structural Eurocodes were initiated by the European Commission but are now produced by the European Committee for Standardisation (CEN) which is the European standards organisation, its members being the national standards bodies of the EU and EFTA countries, e.g. BSI.

CEN will eventually publish these design standards as full European Standards EN, but initially they are being issued as Prestandards ENV. Normally an ENV has a life of about 3 years to permit familiarisation and trial use of the standard by member states. After formal voting by the member bodies, ENVs are converted into ENs taking into account the national comments on the ENV document. At present the following Eurocode parts have been published as ENVs but as yet none has been converted to an EN:

DD ENV 1991-1-1: Basis of design and actions on structures (EC 1)
DD ENV 1992-1-1: Design of concrete structures (EC2)
DD ENV 1993-1-1: Design of steel structures (EC3)
DD ENV 1994-1-1: Design of composite steel and concrete structures (EC4)
DD ENV 1995-1-1: Design of timber structures (EC5)
DD ENV 1996-1-1: Design of masonry structures (EC6)
DD ENV 1997-1-1: Geotechnical design (EC7)
DD ENV 1998-1-1: Earthquake resistant design of structures (EC8)
DD ENV 1999-1-1: Design of aluminium alloy structures (EC9)

Each Eurocode is published in a number of parts usually with ‘General rules and rules for buildings’ in Part 1-1. The various parts of EC3 are:

| | |
|---|---|
| Part 1-1 | General rules and rules for buildings; |
| Part 1-2 | Supplementary rules for structural fire design; |
| Part 1-3 | Supplementary rules for the design of cold formed thin gauge members and sheeting |
| Part 1-4 | Supplementary rules for the use stainless steel |

Further parts of Eurocode 3 cover the following fields of design:

| | |
|---|---|
| DD ENV 1993-2 | Bridges and plated structures |
| DD ENV 1993-3 | Towers, masts and chimneys |
| DD ENV 1993-4 | Silos, tanks and pipelines |
| DD ENV 1993-5 | Piling |
| DD ENV 1993-6 | Crane supporting structures |
| DD ENV 1993-7 | Marine and maritime structures |
| DD ENV 1993-8 | Agricultural structures. |

All Eurocodes follow a common editorial style. The codes contain ‘Principles’ and ‘Application rules’. Principles are general statements, definitions, requirements and sometimes analytical models. All designs must comply with the Principles, and no alternative is permitted.

Application rules are rules commonly adopted in design. They follow the Principles and satisfy their requirements. Alternative rules may be used provided that compliance with the Principles can be demonstrated.

Some parameters in Eurocodes are designated by $\boxed{\phantom{00}}$, commonly referred to as boxed values. The boxed values in the Codes are indicative guidance values. Each member state is required to fix the boxed value applicable within its jurisdiction. Such information would be found in the National Application Document (NAD) which is published as part of each DD ENV.

There are also other purposes for NADs. The NAD is meant to provide operational information to enable the ENV to be used. For certain aspects of the design, the ENV may refer to national standards or to CEN standard in preparation or ISO standards. The NAD is meant to provide appropriate guidance including modifications required to maintain compatibility between the documents. Very occasionally the NAD might contain particular clauses of the code rewritten in the interest of safety or economy.

## 1.7 Meaning of Eurocode terms

In order to rationalise the meaning of various technical terms for easy translation some of the terms used in the past have been modified and given precise meanings. The following, which are adopted in this *Manual*, are of particular importance in the understanding of Eurocodes:

| | |
|---|---|
| **Accidental action** | Action, usually of short duration, which is unlikely to occur with significant magnitude over the period of time under consideration during the design working life. This will generally be impact, fire or explosion. |
| **Action** | A force (load) applied to a structure (direct action), or an imposed deformation (indirect action) such as temperature effects or settlement. |
| **Buckling length** | The distance between the points of contraflexure in the fully buckled mode of a compression member or flange. This will normally be the system length multiplied by an appropriate factor. In BS 5950 terms this is the effective length. |
| **Execution** | The act of constructing the works. For steel structures this includes both fabrication and erection. |
| **Frame** | An assembly of members capable of carrying actions. |
| **Global analysis** | Any analysis of all or part of a structure, this includes beams in simple frames as well as the complete analysis of rigid jointed structures. This analysis may be either elastic or plastic. |
| **Limit states** | States beyond which the structure no longer satisfies the design performance requirements. |
| **Permanent actions** | Dead loads, such as self-weight of the structure or fittings, ancillaries and fixed equipment. |
| **Resistance** | The strength of a member in a particular mode of failure. |
| **Serviceability limit states** | Correspond to limit states beyond which specified service criteria are no longer met, with no increase in action (load). |
| **Subframe** | Any part of a frame taken to make analysis simpler. |
| **System length** | Distance between the intersection of the centre lines of members at each end, or at intermediate points where the member is supported or restrained. |
| **Variable action** | Imposed loads, wind loads or snow loads. *Note*: these are all given the same treatment regarding partial factors, unlike |

| | |
|---|---|
| | British Standards. |
| **Ultimate limit states** | Those associated with collapse, or with other forms of structural failure that may endanger the safety of people. |

## 1.8 Changes of axes nomenclature

The use of traditional axis terminology in the UK has been altered, to match the computer approach being used in design. Fig. 1.1 gives the notation used throughout the Eurocodes for member axes. The *x-x* axis lies along the length of the member. Details of the Eurocode system of symbols are given in the Appendix.

## 1.9 Section classification

Sections are classed in the Eurocode as:

| | |
|---|---|
| **Class 1.cross-sections** | are those that can form a plastic hinge with the rotation capacity for required plastic analysis. |
| **Class 2.cross-sections** | are those that can develop their plastic moment resistance, but have limited rotation capacity. |
| **Class 3.cross-sections** | are those in which the calculated stress in the extreme compression fibre of the steel member can reach its yield strength, but local buckling is liable to prevent development of the plastic moment of resistance. |
| **Class 4.cross-sections** | are those in which it is necessary to make explicit allowances for the effects of local buckling when determining their moment resistance or compression. |

The limits for the various conditions are given in EC3 in Table 5.3.1 and the effective width of slender cross-sections is given in Tables 5.3.2 and 5.3.3.

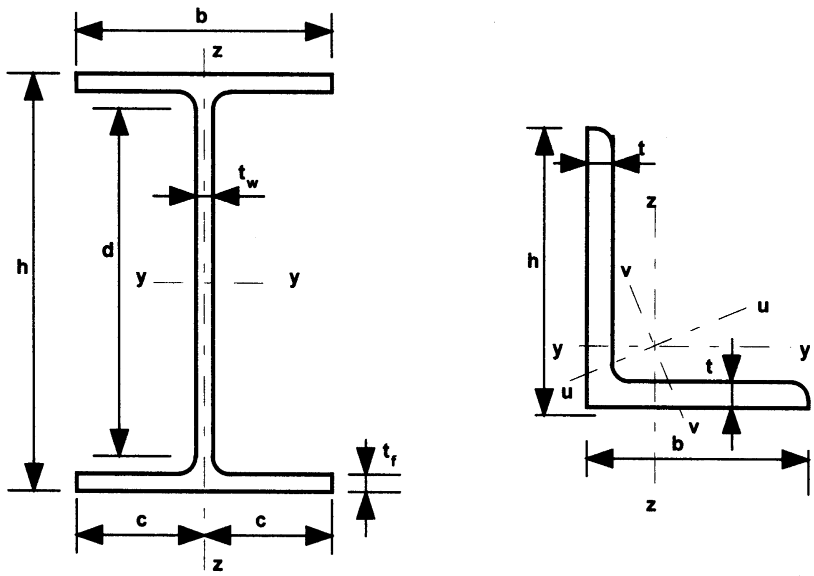



*Fig. 1.1 Member axes and dimensions used in EC3*

# 2 General principles

This Chapter outlines the general principles that apply to the design of structural steel buildings.

## 2.1 General

One engineer should be responsible for the overall design, including stability, so that the design of all structural parts and components is compatible even where some or all of the design and details of the parts and components are not made by the same engineer. The engineer should also be responsible for seeing that the connection details reflect the design assumptions, including situations where more than one structural material is employed.

The structure should be so arranged that it transmits the variable, wind and permanent actions in a direct manner to the foundations. The general arrangement should lead to a robust and stable structure that will not overturn or collapse progressively under the effects of misuse or accidental damage to any one element. Consideration should also be given to the erection procedures and stability during construction.

## 2.2 Stability

### 2.2.1 Multistorey braced structures

Lateral stability in two directions approximately at right-angles to each other should be provided by a system of vertical and horizontal bracing within the structure so that the columns will not be subject to sway moments. Bracing can generally be provided in the walls enclosing the stairs, lifts, service ducts, etc. Additional stiffness may also be provided by bracing within other internal or external walls. The bracing should, preferably, be distributed throughout the structure so that the combined shear centre is located approximately on the line of the resultant on plan of the applied overturning forces. Where this is not possible, torsional moments may result which must be considered when calculating the load carried by each bracing system.

Braced bays should be effective throughout the full height of the building. If it is essential for the bracing to be discontinuous at one level, provision must be made to transfer the forces to other braced bays. Bracing should be arranged so that the angle with the horizontal is not greater than 60°.

### 2.2.2 Single-storey structures

Lateral stability to these structures should be provided in two directions approximately at right-angles to each other. This may be achieved by:

- rigid framing, or
- vertical braced bays in conjunction with plan bracing.

### 2.2.3 Forms of bracing

Bracing may take of the following forms:

- horizontal bracing
  - triangulated steel framing
  - concrete floors or roofs
  - adequately designed and fixed profiled steel decking
- vertical bracing
  - triangulated steel framing
  - reinforced concrete walls preferably not less than 180mm in thickness masonry walls preferably not less than 150mm in thickness adequately connected to the steel frames.

    Walls should not be used as a principal means of vertical bracing if they can be removed at a later stage.

    *Note*: temporary bracing should be provided during erection before concrete or masonry walls are constructed.

## 2.3 Robustness

All members of a structure should be effectively tied together in the longitudinal, transverse and vertical directions as set out in Chapters 7 and 9. In framing the structure care should be taken to avoid members whose failure would cause disproportionate collapse. Where this is not possible, alternative load paths should be identified or the member in question designed as a key element EC3 NAD A4.

## 2.4 Movement joints

Joints should be provided to minimise the effects of movements arising from temperature variations and settlement. The effectiveness of movement joints depends on their location, which should divide the whole structure into a number of individual sections. The joints should pass through the whole structure above ground level in one plane. The structure should be framed on each side of the joint, and each section should be structurally independent and be designed to be stable and robust without relying on the stability of adjacent sections.

Joints may also be required where there is a significant change in the type of foundation, plan configuration or height of the structure. Where detailed calculations are not made in the design, joints to permit horizontal movement of 15 to 25mm should normally be provided in the UK at approximately 50m centres both longitudinally and transversely. For single-storey sheeted buildings it may be appropriate to increase these spacings. It is necessary to incorporate joints in finishes and in the cladding at the movement joint locations, in addition to joints required by the type of cladding.

A gap should generally be allowed between steelwork and masonry cladding to allow for the movement of columns under loading (except where the panel acts as an infill shear wall).

## 2.5 Loading

This *Manual* uses the limit state principle and load factor format of EC3. Characteristic loads (actions) are those obtained from the appropriate loading codes, which for structures erected within the UK, the NAD requires that the following are used:

(a) BS 648: 1964[3], for the weight of the materials in the structure, $G_k$,

(b) BS 6399: Part 1: 1984[4], for vertical loading, $Q_k$,
(c) BS 6399: Part 3: 1988[5], for snow and roof loading, $Q_k$,
(d) CP3 Chapter V: Part 2: 1972[6], for wind loading, $Q_k$. It should be noted that the NAD states that the wind loading should be taken as 90% of these values.

These traditional values of loading are to be used as characteristic actions, according to the EC3 NAD. It should be noted that the documents referred to above have been updated but the strict wording of the EC3 NAD prohibits the use of the latest versions. The reason for this is that the safety levels were calibrated against the versions of the documents available at the time of publication.

## 2.6 Limit states

### 2.6.1 Limit state of strength

In order to determine the design loading from the characteristic actions they must be multiplied by appropriate safety and combination factors. The equation for obtaining the design value is given in EC3 clause 2.3.2.2. This could lead to a large volume of complex calculations, if followed exactly. To simplify every-day design for buildings an alternative equation has been given in EC3 clause 2.3.3.1 (5). This gives fewer cases to consider; if there is one clear unfavourable action then there would be only two load cases to consider. The expression is:

$$\sum_{j} \gamma_{g,j} = \gamma_{Q,1} Q_{k,1}$$

when there is more than one variable action acting on a member, requiring the actions to be combined, the expression is:

$$\sum_{j} \gamma_{G,j} G_{k,j} + 0.9 \sum_{i \geq 1} \gamma_{Q,i} Q_{k,i}$$

These are expressed as in Table 2.1.

**Table 2.1 Load factors $\gamma$ for combined cases for ultimate strength**

| Load combination | $\gamma_{Gk}$ (Dead, $G_k$) | | $\gamma_{Qk}$ (Imposed, $Q_k$) | | |
|---|---|---|---|---|---|
| (Actions) | Adverse | Beneficial | Adverse | Beneficial | Wind |
| Dead + imposed | 1.35 | 1.0 | 1.5 | 0 | |
| Dead + wind† | 1.35 | 1.0 | | | 1.5 |
| Dead + imposed + wind† | 1.35 | 1.0 | 1.35 | 0 | 1.35 |

The adverse and beneficial factors should be used to produce the most onerous realistic condition. When appropriate, temperature effects should be considered with the load combinations above. Further details of the $\gamma$-factors may be obtained from the EC3 NAD.
†The EC3 NAD states that the wind loading should be taken as 90% of the value obtained from CP3: Ch V: Part 2: 1972[6]

## 2.6.2 Serviceability limit states

### 2.6.2.1 Deflection

The deflections and vibration of a structure should be limited so that the following conditions are satisfied:

(a) deflections do not adversely affect the appearance, or adversely affect the use of the structure
(b) there should not be vibration, oscillation or sway that cause discomfort or damage
(c) there should not be deformations or vibration that cause damage to the finishes or non-structural element.

The structure and its members should be checked for deflection under the characteristic actions, i.e. those not multiplied by γ-factors. EC3 Chapter 4 requires that the deflections are calculated using the 'rare combination' of loading given in EC3 clause 2.3.4. These take the form:

$$\sum_{j} G_{kj} + Q_{k,i}$$

for a single action on the structure, or:

$$\sum_{j} G_{k,j} + 0.9 \sum_{i \geq 1} \gamma_{Q,i} Q_{k,i}$$

for the case where there are several actions at the same time.

These give values, which are be expressed as in Table 2.2.

EC3 (Fig. 2.1 and Table 2.3) considers the maximum deflection of a member in relation to:

$\delta_1$, the deflection of the member arising from permanent loads
$\delta_2$, the deflection of the member arising from variable loads
$\delta_0$, the precamber of the member during fabrication.

The final deflection $d_{max}$ is given as $\delta_1 + \delta_2 - \delta_0$.

**Table 2.2 Load combinations for deflections**

| Load combination | Dead, $G_k$ | Imposed, $Q_k$ | Wind, $Q_k$ |
|---|---|---|---|
| Dead + imposed | 1.0 | 1.0 | |
| Dead + wind | 1.0 | | 1.0 |
| Dead + imposed + wind | 1.0 | 0.9 | 0.9* |

* This 0.9 is on the actual wind load assumed in design, taken as 0.9 of the value derived from CP3: Ch. 5: Part 2 : 1972[6].

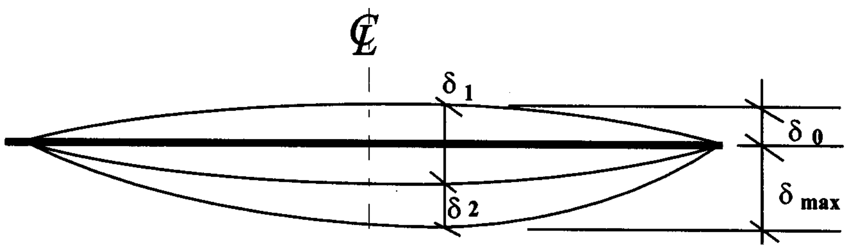



*Fig. 2.1 Deflection limits*

**Table 2.3 Deflection limits**

| Conditions | Limits (see Fig. 2.1) | |
|---|---|---|
| | $\delta_{max}$ | $\delta_2$ |
| Roofs generally | L/200 | L/250 |
| Roofs frequently carrying personnel other than for maintenance | L/250 | L/300 |
| Floors generally | L/250 | L/300 |
| Floors supporting plaster or other brittle finish or non-flexible partitions | L/250 | L/350 |
| Floors supporting columns (unless the deflection has been included in global analysis for the ultimate limit state) | L/400 | L/500 |
| Where $\delta$ can impair the appearance of the building | L/250 | |

The limiting values for beams are given in Table 2.3. Horizontal deflections are limited to height/300, except that columns in portal frames are limited to height/150 and the total deflection of a multistorey frame is limited to height/500.

It should be pointed out that the deflection criteria provided in any code can be used only as a guide to the serviceability of the structure and may not be taken as an absolute guide to satisfactory performance in all cases. It is the responsibility of the engineer to see that the limits used in the design are appropriate for the structure under consideration (see also 11.6).

#### 2.6.2.2 Vibration

Vibration can occur in buildings causing discomfort or structural distress. Problems can also arise in cases where there is moving plant and machinery.

In cases where there is an induced oscillation arising from vibrating equipment the control should be obtained by ensuring that the frequency of the disturbing motion is not close to that of the structure, or one of its harmonics. This may involve some complex analysis to derive the values of the frequencies; it should be noted that altering the safety factors will not guarantee a solution to the problem.

In cases where the possibility of vibration will cause discomfort to the users some guidance has been given in EC3 clause 4.3 for public buildings. Two cases have been highlighted; these are:

(a) the fundamental frequency of floors in dwellings and offices should not be less than 3 cycles/second. This may be deemed to be satisfied when ($\delta_1 + \delta_2$ (see Fig. 2.1) is less than 28mm.

(b) the fundamental frequency of floors used for dancing and gymnasia should not be less than 5 cycles/second. This may be deemed to be satisfied when ($\delta_1 + \delta_2$ (see Fig. 2.1) is less than 10mm.

## 2.7 Corrosion protection

In order to corrode, steel must be in the presence of both air and water, as most structures are in air then some form of protection will normally be required. In a dry building shell this will normally be light protection, often more of a cosmetic nature, but in any situation where there is any form of moisture then a properly design protective system will be required. EC3 gives the points that should be considered, but no guidance as to how the level of protection should be determined. The points given are:

- the use of the structure
- the required performance criteria
- the expected environmental conditions
- the composition of the steel
- the shape of the members and structural detailing
- the protective measures
- the likely maintenance during the intended life.

Guidance on the level of protection suitable for various exposures is given in the guides to protection published by British Steel. These look at the exposure risk and give suggested levels of protection for periods to first maintenance.

Bolts and other fittings should be protected to the same level as the main structural members. It may often be necessary to apply the protection after construction so that there is no damage to the protective layer. Any damage to the protective layer during handling and erection should be made good to the manufacturer's specification.

## 2.8 Fire resistance

Structural steel members may require protection by insulating materials to enable them to sustain their actions during a fire. The type and thickness of insulation to be applied depends on the period of fire resistance required and the section factor of the member (heated perimeter/cross sectional area).

Fire resistance is given in Building Regulations in terms of a standard fire resistance period, usually ½, 1, 2 or 4 hours, depending on the nature of the building and the consequences of the failure of the structure.

The fire section of EC3 has been published as DD ENV 1993–1–2[7] and may now be used to determine the fire resistance of steel structures. Other useful information may be found in BS 5950: Part 8[8] and in Guidelines for the construction of fire-resisting elements[9]. Details of various proprietary systems may be found in the Association of Specialist Fire Protection Contractors and Manufacturers manual on fire protection[10].

## 2.9 Material properties

### 2.9.1 Partial factors for materials*

The EC3 NAD gives $\gamma_M$ the value of 1.05 for structural sections and plates for structures erected in the UK.

### 2.9.2 Design strength

The design strength in EC3 is taken as the yield strength of the material, except that for thickness up to 40mm the strength is based on the stress of the thinnest material (usually up to 16mm thick). This *Manual* covers material only up to 40mm in thickness which means that the design strengths given in Table 2.4 may be used.

### 2.9.3 Coefficient of thermal expansion

This is given in EC3 as $12 \times 10^{-6}$ per °C at room temperature. It should be noted that the

* It should be noted that there are several partial material factors ($\gamma_M$) used in the Eurocode, these have been designated as ($\gamma_{M0}$, $\gamma_{M1}$, $\gamma_{M2}$, etc. $\gamma_{M0}$ covers the design of members in bending, tension and compression, while ($\gamma_{M1}$ covers the design of members where buckling takes place. The other $\gamma$ values are used for the various facets of connection design.

**Table 2.4 Yield strengths of steels**

| Nominal steel grade | Yield strength, N/mm$^2$ |
|---|---|
| BS EN 10025[11] | |
| S 275<br>S 355 | 275<br>355 |
| EN 10113[12] | |
| S 275<br>S 355 | 275<br>355 |

value changes with temperature and that for fire engineering a value of $14 \times 10^{-6}$ is used as a good average over the temperature range normally encountered in fire.

### 2.9.4 Modulus of elasticity

For normal design at room temperatures the value of the modulus of elasticity should be taken as 210 000N/mm$^2$. This is higher than the value given in BS 5950: Part 1[13]. It should also be noted that the value changes with temperature.

### 2.9.5 Brittle fracture

The risk of brittle fracture arises:

1. in parts of members where there is tension,
2. in thick material,
3. where the temperatures are low.

For the common cases in normal building construction limiting conditions are given in Table 3.2 of EC3. A shortened version of this Table is given as Table 2.5 for the type of structure covered by this *Manual*. For other cases, reference should be made to EC3.

## 2.10 Methods of analysis

### 2.10.1 General approach

EC3 uses two approaches to the analysis of a structure:

1. Elastic global analysis, where the member moments and forces are derived from the actions by means of an elastic analysis of the frame for both ultimate and service ability limit states,
2. Plastic analysis for the ultimate limit state, where the plastic behaviour is taken into account. This may be further divided into two classes:
   (a) Rigid plastic methods, usually considered as simple plastic design in the UK.
   (b) Elastic plastic methods, where a more detailed study of the plasticity of the frame is considered.

With the exception of the analysis of portal frames, plastic analysis is not covered in this *Manual*.

**Table 2.5 Guide to maximum thickness for statically loaded structural elements.**

| Steel grade and quality | Maximum thickness (mm) for lowest service temperature of: | | | |
|---|---|---|---|---|
| | Lowest temperature for internal service conditions –5°C from UK NAD | | Lowest temperature for internal service conditions –15°C from UK NAD | |
| EN 10025 : 1993[11] | Non-welded or welded, but in compression or welded and in tension but stress $\leq 0.2 f_y$ | Welded and in tension, with stress $> 0.2 f_y$ | Non-welded or welded, but in compression or welded and in tension but stress $\leq 0.2 f_y$ | Welded and in tension, with stress $> 0.2 f_y$ |
| S 275 JR | 120 | 32 | 81 | 23 |
| S 275 JO | 250 | 82 | 235 | 57 |
| S 275 J2 | 250 | 250 | 250 | 150 |
| S 355 JR | 60 | 20 | 43 | 13 |
| S 355 JO | 150 | 43 | 115 | 31 |
| S 355 J2 | 250 | 117 | 250 | 79 |
| S 355 K2 | 250 | 168 | 250 | 142 |

*Notes*:
1. The thicknesses in this table exceed the actual maximum supply thickness given in BS EN 10025[11] for certain cases in which case the latter obviously governs.
2. The maximum thicknesses apply to baseplates transmitting moments to the foundation, but not to baseplates in S 275 and S 355 subject to compression only.
3. For steel grade S 355 K2, the specified minimum Charpy V-notch energy value is 40J at –20°C. The entries in the Table for this grade assume an equivalent value of 27J at –30°C.

In addition to the two main classes of analysis there are also two classes of frame. The selection of the frame class is dependent on its susceptibility to sway deformations. Frames that are inherently stiff and do not lose much strength because of sway deformations are known as non-sway frames; those where the possibility of sway reducing the resistance of the frame are known as sway frames. As this *Manual* is intended to cover only braced and simple frames the requirements for sway stability will not be covered in detail.

## 2.10.2 Types of elastic framing

Three classes of framing are permitted in EC3:

1. Simple framing, assuming that all joints are pinned. This type of framing will always require either bracing or shear walls of some form to resist horizontal loading.
2. Semi-continuous framing, assuming that there is some stiffness in the joints and that the moment rotation behaviour can be reliably predicted.

3. Fully continuous framing, assuming that the joints are fully rigid and that continuity is provided.

These methods are similar to those in common use in the UK in the past.

Guidance on the extent of the global analysis required for each type of framing employed in the structure follows.

For simple framing the structure may be taken as statically determinate, which means that beams are taken as being pinned at the supports and columns and ties are designed as simple members, although attention is drawn to the requirements in the UK NAD for column eccentricities. Trusses may be taken as pinned or rigid, although no specific criteria are included, except in Annex K which covers the design of connections in tubular structures.

Continuous beams, including those that have partial fixity at the supports, and frames where sway effects are small are analysed under appropriate loading arrangements to determine the critical forces for member design. It should be noted that all frames must be capable of resisting an equivalent lateral load (see 2.10.3 below).

Semi-rigid structures are analysed in a similar manner to those with fully rigid connections, except that account must also be taken of the effects of rotation at the joints, based on the moment rotation curves for the connections. This will generally lead to frames with higher deflections and loss of stiffness. Braced frames with semi-rigid connections are probably the closest approach to the actual behaviour of the structure as designed and detailed. For designers wishing to use this approach reference should be made to Annex J of EC3.

## 2.10.3 Resistance to horizontal loading

All structures must resist lateral loading, which may arise from wind, applied forces or the effects of structural or load asymmetry or imperfections in construction. EC3 requires that the structure is checked to see that it can resist the effect of frame imperfections and other actions. These are treated as rotations, but may be converted into nominal forces in addition to applied lateral and other forces. It is easier to follow the treatment if the change is made.

The details of these imperfection/forces are given in EC3 clause 5.2.4.3, where they are derived from a sway imperfection; given as:

$$\phi = k_c \, k_s \, \phi_o$$

where: $\phi$ is the sway imperfection
$\phi_o$ is a constant = $1/200$
$k_c$ is a column factor = $[0.5 + 1/n_c]$ 0.5 but $\leq 1.0$
$n_c$ is the number of columns in each plane
$k_s$ is a storey factor = $[0.2 + 1/n_s]$ 0.5 but $\leq 1.0$
$n_s$ is the number of storeys.

This imperfection is converted into an equivalent horizontal force applied at each floor by multiplying the design loads by $\phi$, see Fig. 2.2 overleaf.

It should be noted that the horizontal forces are used in the frame analysis only, the resulting forces must be added to the horizontal loads when designing the frame for lateral loading – this is unlike BS 5950 where the equivalent, or notional, forces are treated separately. They should not be applied in the design of the lateral shears or vertical loads on the foundations.

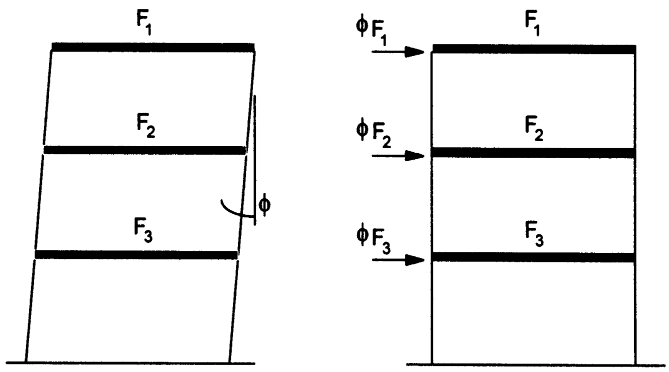



*Fig. 2.2 Application of equivalent horizontal forces*

# 3 Braced multistorey buildings – general

## 3.1 Introduction

This Chapter offers advice on the general principles to be applied when preparing a scheme for a braced multistorey structure. The aim should be to establish a simple structural scheme that is practicable, sensibly economic, and not unduly sensitive to the various changes that are likely to be imposed as the overall design develops.

Loads should be carried to the foundation by the shortest and most direct routes. In constructional terms, simplicity implies (among other matters) repetition, avoidance of congested, awkward or structurally sensitive details, with straightforward temporary works and minimal requirements for unorthodox sequencing to achieve the intended behaviour of the completed structure. Sizing of structural members should be based on the longest relevant spans (slabs and beams) and largest areas of roof and/or floors carried (beams, columns, walls and foundations). The same sections should be assumed for similar but less onerous cases – this saves design time and is of actual advantage in producing visual and constructional repetition with consequent cost benefits. Simple structural schemes are quick to design and easy to build. They may be complicated later by other members of the design team trying to achieve their optimum conditions, but a simple scheme provides a good 'benchmark'. Scheme drawings should be prepared for discussion and budgeting purposes incorporating such items as general arrangement of the structure including bracing, type of floor construction, critical and typical beam and column sizes, and typical edge details, critical and unusual connection details, and proposals for fire and corrosion protection. When the comments of the other members of the design team have been received and assimilated, the scheme should be revised and the structural members redesigned as necessary.

## 3.2 Actions (loads)

In preliminary design, to save time, load reductions need not be taken into consideration. The load factors, $\gamma_Q$ and $\gamma_G$, for use in design should be obtained from Table 2.1.

Temperature effects should also be considered where appropriate, especially when the distance between movement joints is greater than recommended in 2.4.

The effect of using beneficial load factors should be considered, and adverse load factors used if these will result in the use of a larger section.

Care should be taken not to underestimate the dead loads, and the following figures should be used to provide adequate loads in the absence of firm details:

| | |
|---|---|
| floor finish (screed) | 1.8kN/m$^2$ on plan |
| ceiling and service load | 0.5kN/m$^2$ on plan |
| demountable lightweight partitions | 1.0kN/m$^2$ on plan |
| blockwork partitions | 2.5kN/m$^2$ on plan |
| external walling – curtain walling and glazing | 0.5kN/m$^2$ on elevation |
| cavity walls (lightweight block/brick) | 3.5kN/m$^2$ on elevation |

Density of normal weight aggregate concrete should be taken as 24kN/m$^3$.

Density of lightweight aggregate concrete should be taken as 19kN/m$^3$.

## 3.3 Material selection

For multistorey construction in the UK, S 355 steel may be used for beams acting compositely with the floors or where deflection does not govern the design; otherwise S 275 steel should be used for beams. Similar sections of differing grades of steel should not be employed in the same project, unless there is exceptional quality control to prevent the members being exchanged. For columns, S 355 steel should be considered where it is important to reduce overall sizes to a minimum. Grade 8.8 bolts should normally be used throughout.

## 3.4 Structural form and framing

The method for 'simple construction' as given in Appendix B of the NAD should be used and the following measures adopted:

(a) provide braced construction by arranging suitable braced bays or cores deployed symmetrically wherever possible to provide stability against lateral forces in two directions approximately at right-angles to each other
(b) adopt a simple arrangement of slabs, beams and columns so that loads are carried to the foundations by the shortest and most direct routes using UC sections for the columns
(c) tie all columns effectively in two directions approximately at right-angles to each other at each floor and roof level. This may be achieved by the provision of beams or effective ties in continuous lines placed as close as practicable to the columns and to the edges of the floors and roofs
(d) select a floor construction that provides adequate lateral restraint to the beams (see 3.8)
(e) allow for movement joints (see 2.4)
(f) if large uninterrupted floor space is required and/or height is at a premium, choose a profiled-steel-decking composite floor construction that does not require propping. As a guide, limit the span of the decking to 2.5–3.6m; the span of the secondary beams to 7–10m; and the span of the primary beams to 5–7m
(g) in other cases, choose a precast or an *in situ* reinforced concrete floor, as a guide limiting their span to 5–6m, and the span of the beams to 6–10m.

The arrangement should take account of possible large openings for services and problems with foundations, e.g. columns immediately adjacent to site boundaries may require balanced or other special foundations.

An alternative to the above methods is the use of slim floor construction, where the beams are incorporated in the depth of the slab, which is usually increased in depth to give a uniform soffit.

## 3.5 Fire resistance

In the absence of specific information, choose a fire resistance period of 1h for the superstructure and 2h for ground floor construction over a basement and the basement structure. This may be achieved by choosing one of the alternatives in Table 3.1.

## 3.6 Corrosion protection

For multistorey buildings on non-polluted inland sites general guidance on systems for protection of steelwork in certain locations follows. For other environments and for more

**Table 3.1 Fire protection type of protection**

| Type of protection | Thickness in mm for period of fire resistance | |
|---|---|---|
| | 60 min | 120 min |
| spray | 20 | 35 |
| boarding | 15 | 30 |
| intumescent coating (normally up to 1h) | 1–5 | – |
| reinforced concrete casing – loadbearing | 50 | 50 |
| reinforced concrete casing (Min. Grade 20) – non-loadbearing | 25 | 25 |

More detailed guidance is given in:
Guidelines for the construction of fire resisting structural elements[9]
Fire protection for structural steel in building[10]
BS 5950: Part 8[8].

detailed advice, reference should be made to BS 5493[15] and ECCS technical note 90[16] and to publications from British Steel, British Constructional Steelwork Association, Zinc Development Association, and the Paint Research Association. The general guidance is given below:

(a) *For steelwork integral with external cladding, particularly where not readily accessible for inspection and maintenance:* provide an adequate gap between the outside skin and the structural steel and:
 (i) concrete encasement, or
 (ii) an applied coating system to give very long life such as: hot-dip galvanise to BS 729[17] (85μm), or blast clean SA2½, isocyanate pitch epoxy (450μm) (BS 5493[15] system reference SK7)

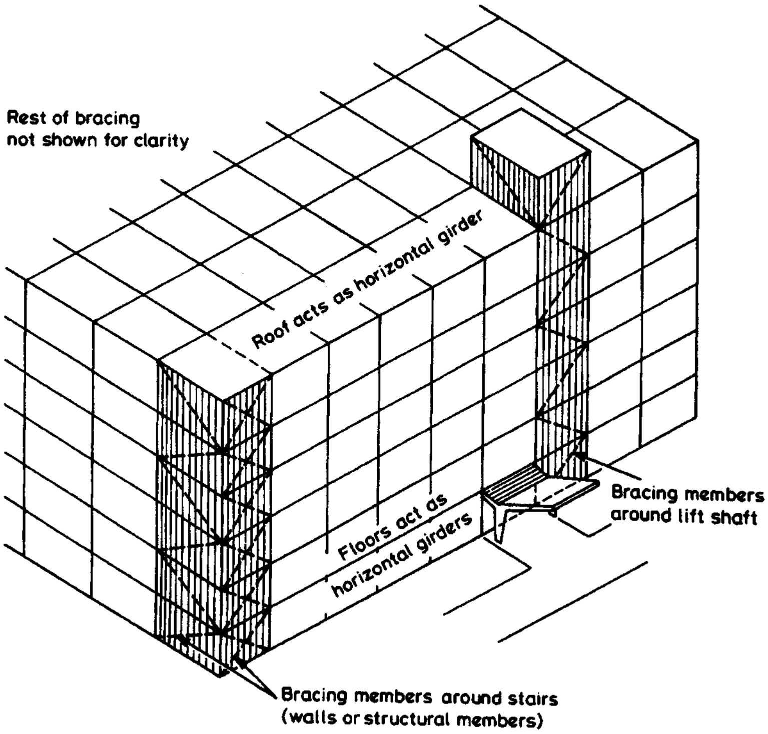
(b) *For internal steelwork not readily accessible, subject to condensation and/or significant corrosion risk:* A system to give long to very long life depending on corrosion risk such as:
 (i) blast clean SA2½, coal-tar epoxy (360μm), (SK7) or
 (ii) blast clean SA2½, 2 pack zinc-rich epoxy (80μm), epoxy MI0 (120μm), (SK1)

c) *For external exposed steelwork, accessible:* A system to give medium life (or longer with appropriate maintenance cycles) such as blast clean SA2½, HB zinc phosphate (70μm), modified alkyd (70μm), alkyd finish (35μm), (SF7)

(d*) Internal steelwork, heated building with negligible corrosion risk*:* It is feasible to avoid treatment altogether in the right environment. Exposed steelwork not requiring fire protection will need a ‘low life’ coating system or better for decorative purposes. Otherwise, steelwork may require ‘low life’ protection to cover the period of delay before the cladding is erected.
 For sprayed fire protection systems the coating must be compatible. Suitable systems include:
 (i) shop applied: blast clean to SA2½, HB zinc phosphate (80μm)
 (ii) site applied: manual clean C St 2, wetting zinc phosphate (80μm) or manual clean C St 2, HB bitumen (150μm).

 *Unless a high-quality finish is required for appearance, only nominal protection is required.

Note that in all cases blast cleaning should be in accordance with BS 7079: Part 1[18].




Note that the roof and floors will act as horizontal girders provided that they are designed to do so.

*Fig. 3.1 Braced frame rectangular or square on plan*

## 3.7 Bracing

Choose the location and form of bracing in accordance with the recommendations in 2.2.3 and 3.4(a). Typical locations are shown on Figs 3.1 and 3.2 for different shaped buildings.

The combination of the wind and notional horizontal forces on the structure, to give the critical load cases, should be assessed and divided into the number of bracing bays resisting the horizontal forces in each direction.

Provide adequate bracing, using horizontal and vertical temporary bracing systems if necessary during the construction period.

## 3.8 Flooring

It is essential at the start of the design of structural steelwork, to consider the details of the flooring system to be used, since these have a significant effect on the design of the structure.

Table 3.2 (p28) summarises the salient features of the various types of flooring commonly used in the UK.

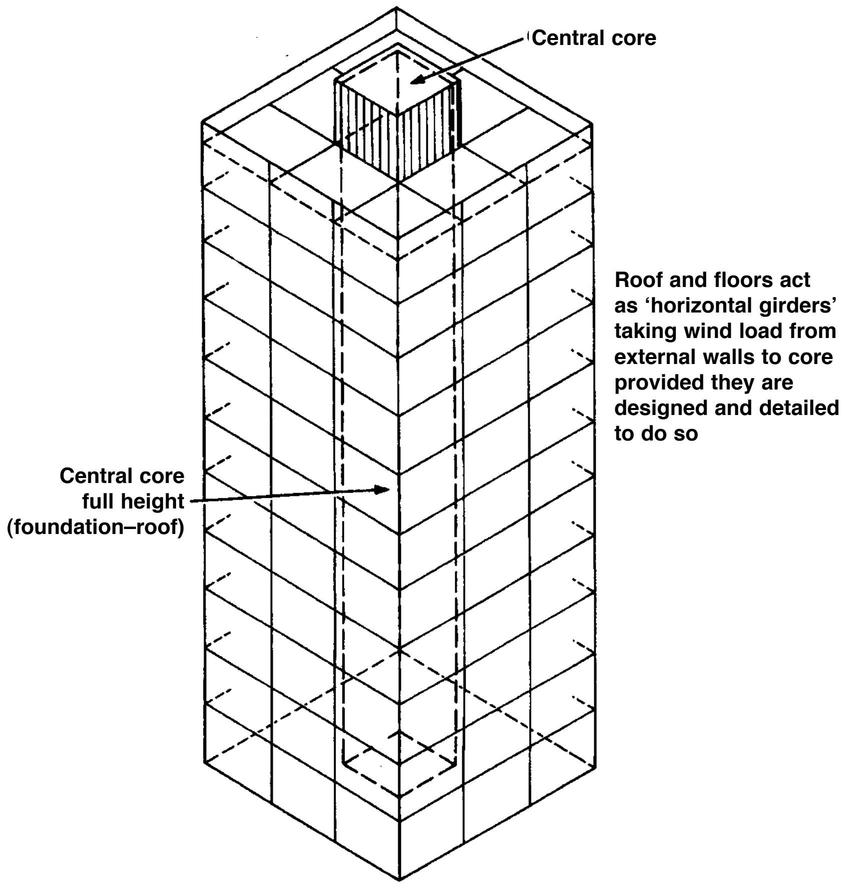



*Fig. 3.2 Braced frame square on plan – central core*

**Table 3.2 Details of typical flooring systems and their relative merits**

| Floor type | Typical span range (m) | Typical depth (mm) | Construction time | Degree of lateral restraint to beams | Degree of diaphragm action | Main areas of usage and remarks |
|---|---|---|---|---|---|---|
| Timber | 2.5–4 | 150–300 | medium | poor | poor | Domestic |
| *in situ* concrete | 3–6 | 150–250 | medium | very good | very good | All categories but not often used for multistorey steel construction, as formwork and propping are required |
| Precast concrete | 3–6 | 110–200 | fast | fair–good | fair–good | All categories especially multistorey commercial. Ensure that the joints between units are designed and executed to prevent differential movement. |
| Profiled steel decking composite with concrete topping 4–6.5m unpropped 5–8.5m propped | Shallow decks 2–5–3.6 unpropped. Deep decks | 110–150<br><br>200+ | fast | very good | very good | All categories. A check on the effects of vibration will be required |

Notes to Table 3.2

1. Timber floors should be designed to BS 5268[19].
2. *In situ* concrete floors should be designed to BS 8110[20] or to the IStructE/ICE *Manual for the design of reinforced concrete building structures*[21].
3. Precast concrete floors should be designed to BS 8110[20] and to the guides provided by the manufacturer of proprietary flooring systems.
4. Profiled-steel-decking/composite floors should be designed to BS 5950: Part 4[22] and to the literature provided by the manufacturers of the proprietary metal-decking systems.
5. Profiled steel decking 210mm or 225mm deep may be used for spans up to 6.5m unpropped or 9.0m propped. This is generally used in shallow floor construction as in the "Slimflor" system developed by British Steel.

# 4 Beams – bending only

## 4.1 Uncased non-composite beams in buildings

The first step in the design of these beams is to identify the restraint condition and the location of the loads applied to the beams in relation to the location of the restraints.

In this *Manual* the following conditions are identified:

**Condition I:** Full lateral restraint provided (e.g. beams supporting concrete floors) This condition will be satisfied if the frictional force or positive connection between the compression flange of the member and the floor it supports is capable of resisting a lateral force of at least 2½% of the force in the compression flange arising from the design loads.

**Condition II:** Full lateral restraint not provided and load applied directly to the member at or between restraint points.

The design procedures are described separately for each condition.

### 4.1.1 Condition I: Full lateral restraint provided

**Design procedure Class 1 cross-sections**

(a) Calculate the design load = 1.5 × imposed load + 1.35 × dead load, and then calculate the maximum design bending moment ($M_{Sd}$), and the design shear forces

(b) Choose a section such that its moment of resistance $M_{c,Rd}$ about its major axis ≥ $M_{Sd}$. In order to choose a trial section that will not be critical in local buckling, it is necessary to note that elements and cross-sections have been classified according to their behaviour with regard to local buckling. Descriptions of the section classes are given in 1.9.

The limiting width/thickness ratios of the elements of the sections stated in Table 5.3.1 of EC3 should be consulted for the determination of the class of section. In order to assist the selection of suitable sections for use as beams in bending the classifications are given in Table 4.1.

The value of the moment capacity $M_{Rd}$ of a beam about its major axis may be determined from:

$M_{Rd} = W_{pl} f_y / \gamma_{M0}$ for class 1 and 2 cross-sections, and
$M_{Rd} = W_{el} f_y / \gamma_{M0}$ for class 3 cross-sections.

where:
$W_{pl}$ is the plastic modulus of the section about the major axis
$W_{el}$ is the elastic modulus of the section about the major axis
$f_y$ is the design strength of the steel obtained from Table 2 according to the steel grade and flange thickness.
$\gamma_{M0}$ is the partial material factor taken as 1.05 from the UK NAD.

This *Manual* does not cover the design of class 4 cross-sections.

Where the shear on the section is more than 0.5 of the shear resistance (see (d)), then the moment resistance must be reduced. It will be found that this will only be critical if there are heavy point loads near a support, or continuous members or cantilevers. Details of the calculations will be found in EC3 clause 5.4.7.

**Table 4.1 Section classification for bending only**

All equal flanged rolled sections are plastic or compact for bending about the major axis **except** as follows:

| Grade S 275 steel | Grade S 355 steel |
|---|---|
| 152 × 152 × 23 UC<br>356 × 368 × 129 UC | 152 × 152 × 23 UC<br>305 × 305 × 97 UC<br>356 × 368 × 129 UC |

If there are bolt holes in the flanges at a point of high moment then the moment resistance of the section must be reduced, see EC3 clause 5.4.2.2.

(c) Calculate the deflection required to satisfy the several deflection limitations described in 2.6.2.1 from the second moment of area (*I*). It should be noted that, unlike the British Standard requirement for a single check the deflection must comply with several limits.

For the simply supported beams covered in this manual, the value of δ for a uniform load may be taken as:

$$\delta = \frac{6.2WL^3}{I}$$

When the beam carries a point load, the following equation may be employed:

$$\delta = \frac{CWL^3}{I}$$

where:
- $\delta$ is the maximum deflection for limit being considered (mm)
- $W$ is the characteristic point or total load (action) (kN)
- $L$ is the span (m)
- $C$ is the coefficient from Fig. 4.1 for the deflection at the centre of the span
- $I$ is the second moment of area of the section ($cm^4$)

When more than one load is imposed on the beam the principle of superposition may be used for each point load in turn. The determination and use of the central deflection is normally sufficiently accurate for practical design.

For cantilevers and continuous beams the deflections should be calculated from first principles taking into account the slopes at the supports and the ratio of the length of the cantilever to the span of its adjoining member.

(d) Check the shear resistance of the selected beam.

The shear resistance $V_{pl,Rd}$ is given by the following equation:

$$V_{pl,Rd} = A_v(f_y/\sqrt{3})/\gamma_{M0}$$

where:
- $A_v$ may be taken as 1.04 $h\,t_w$.
- $f_y$ is the design strength of the material

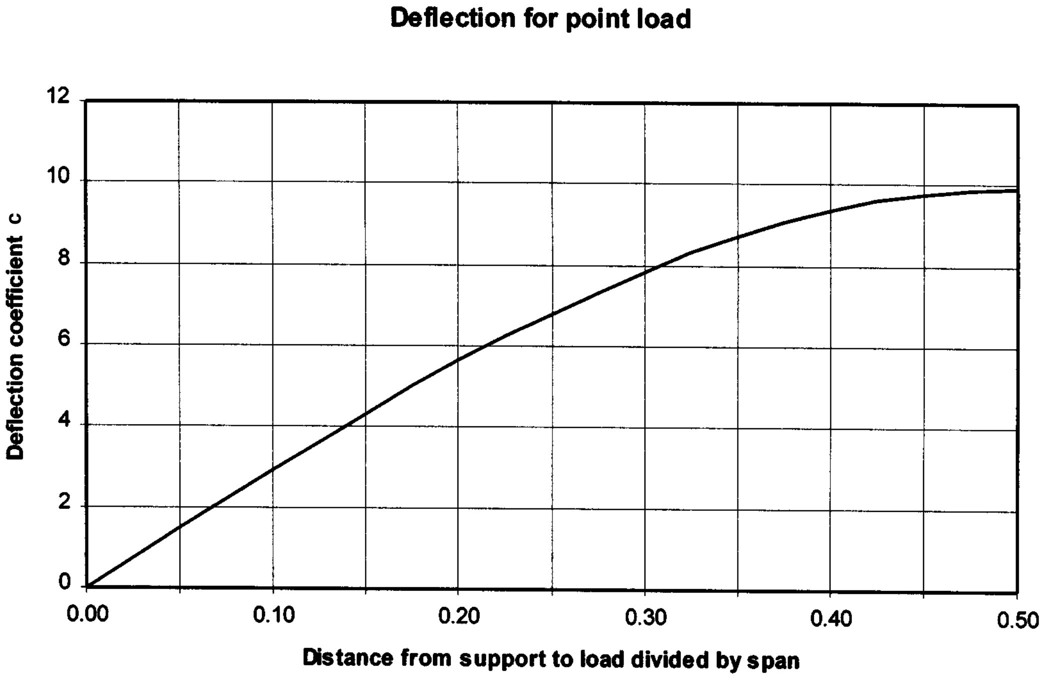



*Fig. 4.1 Deflection coefficient* , c, *at centre of span for point load at any point*

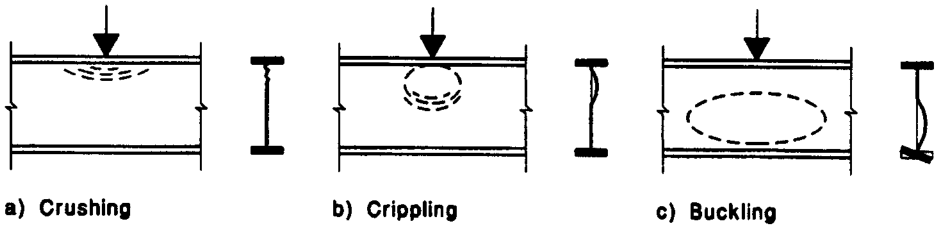



*Fig. 4.2 Types of web failure at supports*

(e) For beams supported on bearings the following additional checks are also required on the resistance of the webs to transverse forces, see also Fig. 4.2:

| | |
|---|---|
| **Crushing resistance** | of the web close to the flange, accompanied by plastic deformation of the flange. This has been known as web bearing in the British Standards. |
| **Crippling resistance** | where there is localised buckling and crushing of the web close to the flange, accompanied by plastic deformation of the flange. There has been no British Standard requirement for this check, which is new. |
| **Buckling resistance** | the buckling of the web over most of the depth of the member. This check is similar to that in the British Standards. |

These checks on the section require, as with the British Standards, the length of the stiff bearing to be determined. This may be determined from EC3 clause 5.7.2 as shown in Fig. 4.3. It must be noted that the dispersion of the stiff bearing length should be taken through solid packs and that loose packs should not be included.

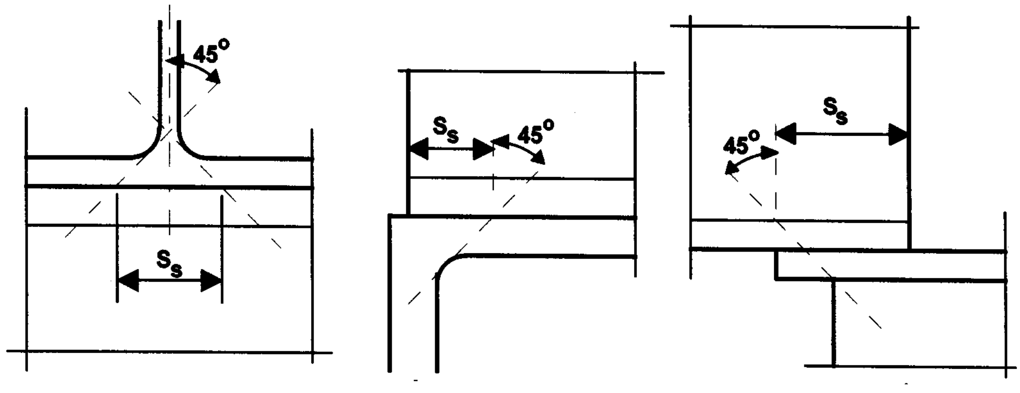



*Fig. 4.3 Determination of length of stiff bearing,* $s_s$

**Determination of crushing resistance**

The crushing resistance, ($R_{y,Rd}$), which must be greater than the reaction, is given by the formula:

$$R_{y,Rd} = (s_s + s_y)t_w f_{yw}/\lambda_{M1}$$

where: $s_s$ is the stiff bearing length
$s_y$ is a length determined as shown below
$t_w$ is the thickness of the web
$f_{yf}$ is the design strength of the flanges
$f_{yw}$ is the yield strength of the web,
$\gamma_{M1}$ is the partial factor for buckling, given a value of 1.05 in the UK NAD.

The value of $s_y$ is determined from one of the following equations:

for end bearing:

$$s_y = t_f(b_f/t_w)^{0.5}\left[f_{yf}/f_{yw}\right]^{0.5}\left[1-(\lambda_{M0}\sigma f_{,Ed}/f_{yf})^2\right]^{0.5}$$

for an intermediate load:

$$s_y = 2t_f(b_f/t_w)^{0.5}\left[f_{yf}/f_{yw}\right]^{0.5}\left[1-(\lambda_{M0}\sigma_{f,Ed}/f_{yf})^2\right]^{0.5}$$

where $\sigma_{f,Ed}$ is the longitudinal stress in the flange.

The following may be assumed in the design of rolled beams in buildings:

$f_{yf} = f_{yw}$ for rolled sections so the first term in [ ] may be omitted.
$\sigma_{Ed} = 0$ for end supports.

**Determination of the crippling resistance.**

The crippling resistance ($R_{a,Rd}$) is given by the following equation:

$$R_{a,Rd} = 0.5t_w^2(Ef_{yw})^{0.5}\left[(t_f/t_w)^{0.5} + (t_w/t_f)\,(s_s/d)\right]/\lambda_{M1})$$

where $t_f$ is the thickness of the flange.

The other symbols are the same as for the crushing resistance.

**Determination of the web buckling resistance**
This is a similar process to that in the British Standards, except that the value of the slenderness rotation λ, is based on the restraint provided to the flanges, see Fig. 4.2, and the effective length of the web, when there is a stiff bearing, is determined from a combination equation, see Fig. 4.3. The strength of the web is determined by assuming that it acts as a strut, with the effective length determined as in Table 4.2 and a breadth determined as in Table 4.3.

**Table 4.2 Effective lengths of webs**

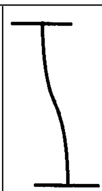
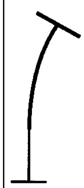
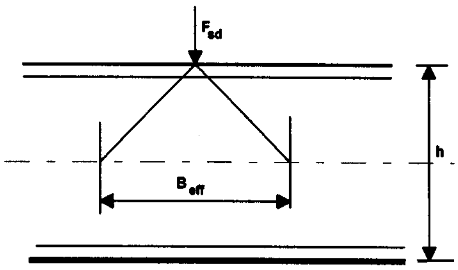
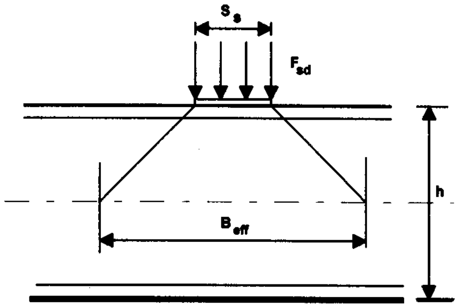
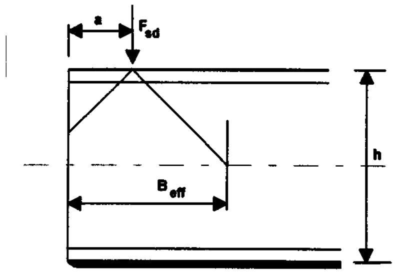
| | | | |
|---|---|---|---|
|  | Both ends restrained against relative lateral movement and rotation<br>$\lambda = 2.5\ d/t$ |  | Ends restrained against rotation but not relative movement<br>$\lambda = 3.5\ h/t$ |
|  | Ends restrained against relative lateral movement but not rotation<br>$\lambda = 4.2\ h/t$ |  | Ends not restrained against relative movement and only one end restrained against rotation<br>$\lambda = 7.0\ h/t$ |

**Table 4.3 Effective breadth for web buckling resistance**

| | |
|---|---|
|  | $B_{\text{eff}} = h$ |
|  | $B_{\text{eff}} = \left[h^2 + s_s^2\right]^{0.5}$ |
|  | $B_{\text{eff}} = \frac{h}{2} + a$<br>but $B_{\text{eff}} \leq h$ |

The method of determining the strength is given in Chapter 5, using the value of $\lambda$ as determined previously and strut curve (c).

If the beam is supported on a bearing and any of the above checks are not satisfied then there are three options:

1. To add to the length of stiff bearing
2. To add web stiffeners, see EC3 for details
3. To modify the section so that it satisfies all the conditions, including bending and deflection.

The solution adopted is dependent on the additional strength required and the limitations imposed by the project requirements.

## 4.1.2 Condition II: Beams without full lateral restraint

These sections may fail by means of lateral torsional buckling, see Fig. 4.4.

This clause applies to universal sections, with the bending taking place about the strong (y–y) axis:

(a) as beams with a load at or between points of restraint, and
(b) as columns subject to bending moments.

It has been assumed that the forces and actions are applied through the shear centre of the section and that the sections are of classes 1, 2 or 3. For all other cases see EC3.

All unrestrained beams must satisfy all the requirements for restrained beams as well as complying with the additional rules given for the unrestrained condition.

**Design procedure**

(a) Determine the maximum design moments and shears on the section and the second moment of area as for a restrained section.
(b) Select a trial section
(c) For the selected section determine the value of $\lambda_{LT}$ using $a_{LT}$ and $i_{LT}$ from section property tables and using the following equation:

$$\lambda_{LT} = \frac{L/i_{LT}}{(C_1)^{0.5}\left[1 + \frac{(L/a_{LT})^2}{25.66}\right]^{0.25}}$$

where: $L$ is the effective unrestrained length. This will normally be the distance between restraint points, if there is significant torsional or warping restraint some economy may be obtained by using the full design sequence in EC3. Guidance is given in Tables 4.5 & 4.6 on the determination of the effective length of members from the actual length.

$C_1$ is a factor based on the shape of the moment diagram between the points of lateral restraint, given in Table 4.4.

(d) look up the value of $\chi_{LT}$ from Fig. 4.6.
(e) Determine the resistance moment from:

$$M_{b,Rd} = \chi_{LT}\beta_w W_{pl,y} f_y / \gamma_{M1}$$

where: $\lambda_{LT} = \lambda_{LT}/93.9\varepsilon$ (Note for grade S 275 steels $93.9\varepsilon = 86.8$)

$\beta_w$ is 1.0 for class 1 and 2 cross-sections and $W_{el}/W_{pl}$ for class 3 cross-sections

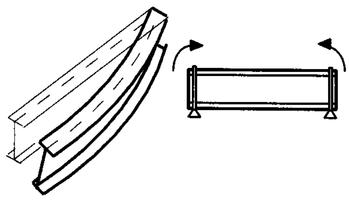


*Fig 4.4 Behaviour of unrestrained beam*

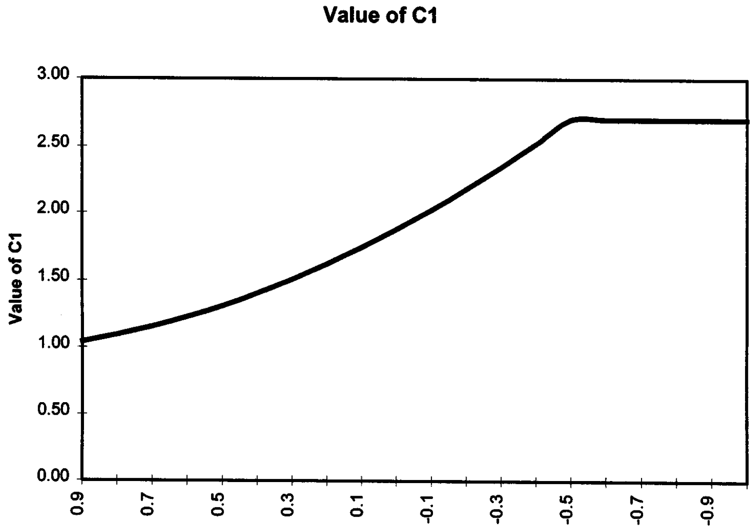



*Fig. 4.5 Graph of C1 from $\psi$*

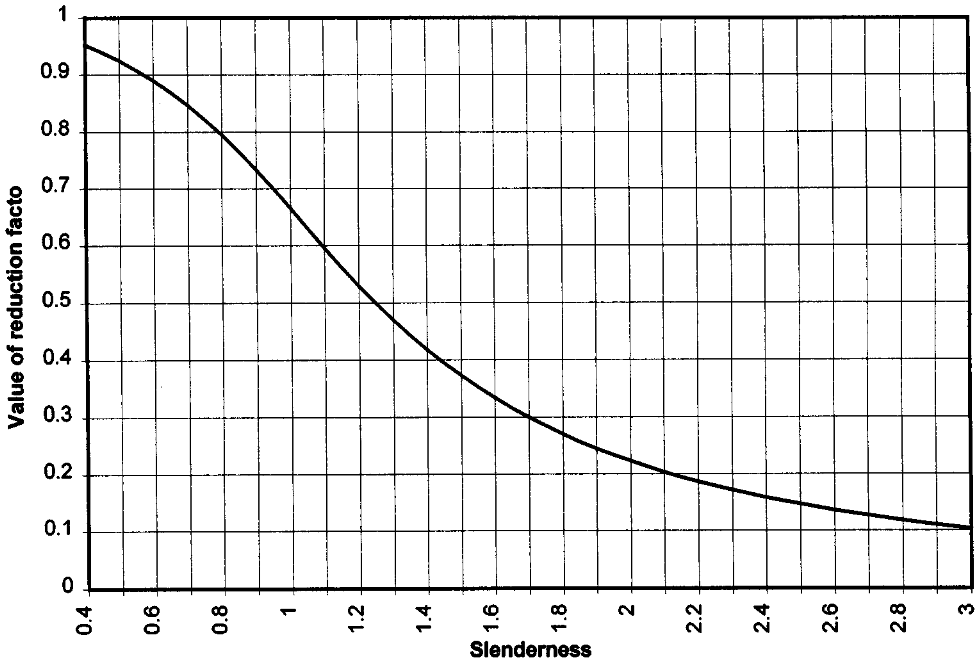
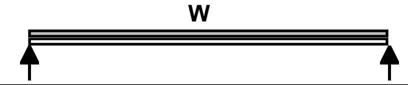
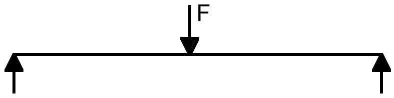
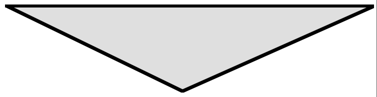
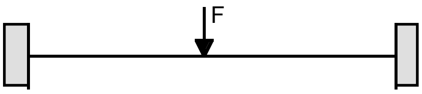
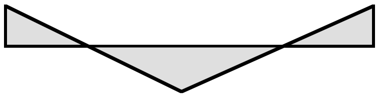
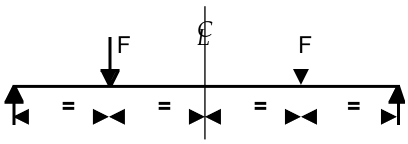
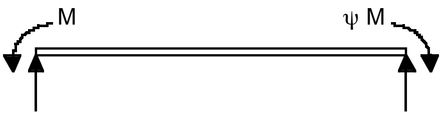
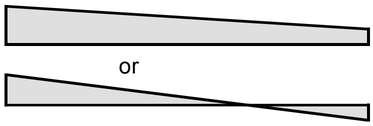



*Fig. 4.6 Graph of reduction factor $\chi_{LT}$ from slenderness $\lambda$*

**Table 4.4 Determination of $C_1$**

| Loading and support conditions | Bending moment diagram | $C_1$ |
|---|---|---|
|   |  | 1.132 |
|   |  | 1.285 |
|   |  | 1.365 |
|   |  | 1.565 |
|   |  | 1.406 |
|   |   | See Fig. 4.5 |

If $M_{b,Rd}$ is greater than or equal to the maximum moment on the part of the member being considered then, providing the restrained beam checks are satisfied and there are no other unrestrained lengths the section may be taken as adequate. If there are other unrestrained lengths then these must be checked in the same way as before.

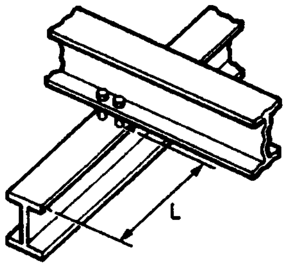
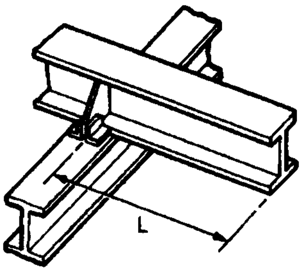
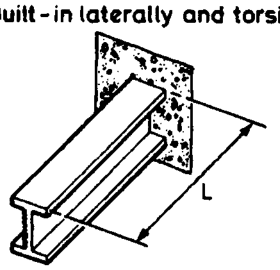
–

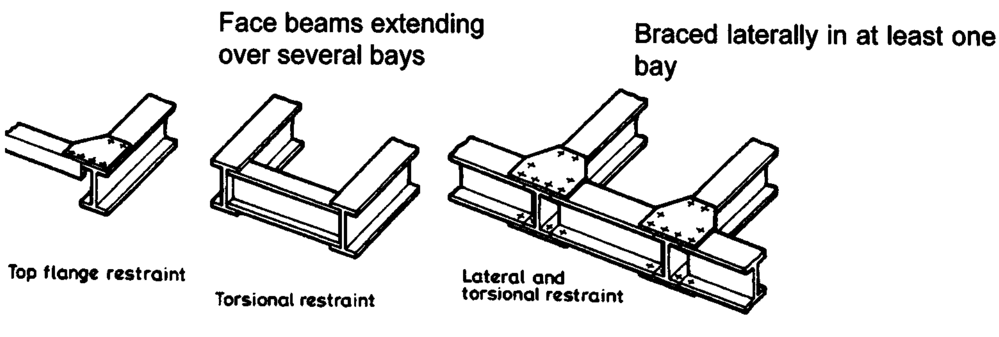
**Table 4.5 Effective length of beams**

| Conditions of restraint at the ends of beams | | Loading conditions | |
|---|---|---|---|
| | | Normal | Destabilising |
| Compression flange laterally restrained: beam fully restrained against torsion | both flanges fully restrained against rotation on plan | 0.7*L* | 0.85*L* |
| | both flanges partially restrained against rotation on plan | 0.85*L* | 1.0*L* |
| | both flanges free to rotate on plan | 1.0*L* | 1.2*L* |
| Compression flange laterally unrestrained: both flanges free to rotate on plan | restraint against torsion provided only by positive connection of bottom flange to supports | 1.0*L* + 2*D* | 1.2*L* + 2*D* |
| | restraint against torsion provided only by dead bearing of bottom flange on supports | 1.2*L*+ 2*D* | 1.4*L* + 2*D* |

*Notes:*
1. *L* is the system length between supports.
2. *D* is the overall depth of the section.
3. It should be noted that a destabilising load exists when a load is applied to the top flange and both the load and flange are free to move laterally.

## Table 4.6 Effective length of cantilevers

| At support | At tip | Normal | Destablising |
|---|---|---|---|
| Continuous with lateral restraint only   | Free<br>Laterally restrained on top flange only<br>torsionally restrained only<br>laterally and torsionally restrained | 3.0$L$<br>2.7$L$<br>2.4$L$<br>2.1$L$ | 7.5$L$<br>7.5$L$<br>4.5$L$<br>3.6$L$ |
| Continuous with lateral and torsional restraint   | Free<br>Laterally restrained on top flange only<br>torsionally restrained only<br>laterally and torsionally restrained | 1.0$L$<br>0.9$L$<br>0.8$L$<br>0.7$L$ | 2.5$L$<br>2.5$L$<br>1.5$L$<br>1.2$L$ |
| Built-in laterally and torsionally   | Free<br>Laterally restrained on top flange only<br>torsionally restrained only<br>laterally and torsionally restrained | 0.8$L$<br>0.7$L$<br>0.6$L$<br>0.5$L$ | 1.4$L$<br>1.4$L$<br>0.6$L$<br>0.5$L$ |



# 5 Braced multistorey buildings – columns in compression and bending

## 5.1 Uncased columns

This Chapter describes the design of uncased columns for braced multistorey construction that are subject to compression and bending in simple construction.

In designing these columns it is assumed that the moments are due only to nominal eccentricities and small moments due to the presence of cantilevers. If a full frame analysis is carried out then the requirements for effective lengths and other features of the design should be determined using EC3.

The resistance of a column is based on the cross-sectional area of the member and its effective length. The resistance to axial loads is further reduced by the simultaneous application of bending moments, giving additional stresses in the extreme fibres of the column cross-section. The resistance of columns with axial compression combined with bending moments may be checked by adding the component resistances in axial loading and bending moments in several locations along the column. Particular points occur where the column is likely to buckle and its axial resistance will be limited by its slenderness ratio, and at positions of connections where bending moments are likely to be of maximum values.

The first step is to determine the slenderness ratio $\lambda$ from the effective lengths $L_{eff}$ of the column about its major and minor axis:

$$\lambda = L_{eff}/i$$

where: $L_{eff}$ is the effective length of the member, see 5.2.
$i$ is the radius of gyration of the section about the appropriate axis

## 5.2 Determination of effective length of columns

For braced multistorey buildings using simple framing the effective length $L_{eff}$ depends on the degree of restraint in direction (i.e. rotational restraint) afforded by the beams attached to the columns at each floor level or the foundations. Fig. 5.1 illustrates typical joint and foundation restraint conditions.

It should be noted that unless more detailed information is available for a beam to column connection to be effective the following approximate guidelines may be adopted:

(a) end plates should be at least 0.6 times the beam depth
(b) web cleats should be at least 0.6 times the beam depth
(c) flange cleats should be provided.

From the degree of restraint assessed at each end, the effective length $L_{eff}$ should be determined from Table 5.1, where $L$ should be taken as the distance between the points of effective restraints on each axis. Further guidance on the effective length of columns can be found in Moore & Nethercot[23].

## 5.3 Column selection

Before selecting a trial section it is necessary to note that elements and cross-sections have been classified as class 1, class 2, class 3 or class 4 in combined compression and bending according to the limiting width/thickness ratios stated in EC3 clause 5.3.2 and Table 5.3.1. In this *Manual* class 4 sections are not considered. It should be noted that all UCs, RSCs

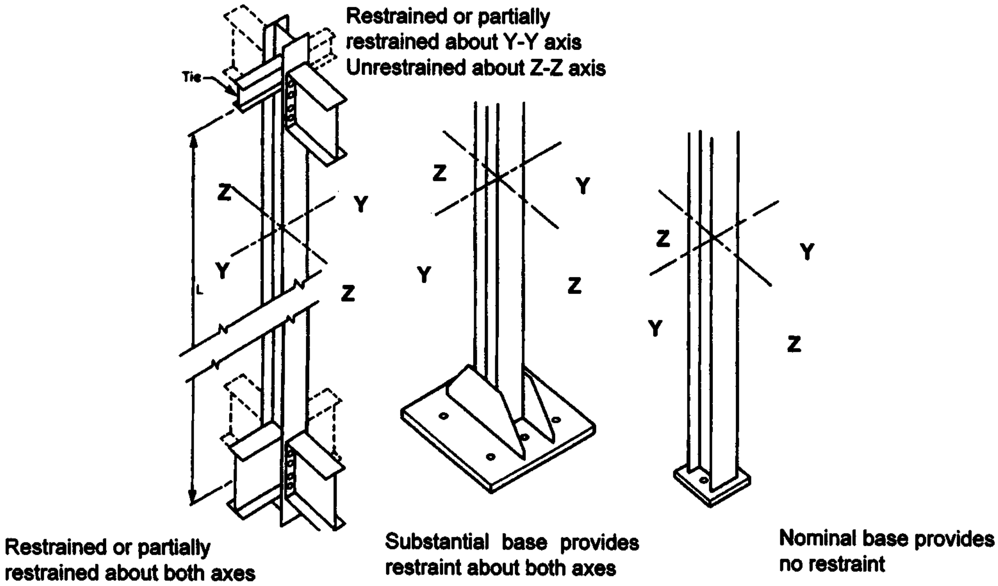



*Fig. 5.1 Typical joint and foundation restraint conditions*

together with the universal beam sections shown in Table 5.2, which are not class 4, may be chosen.

If it becomes essential to use other sections detailed design must be undertaken to EC3.

## 5.4 Columns in simple construction

For simple multistorey construction braced in both directions the columns should be designed by applying nominal moments only at the beam-to-column connections. The following should be met:

(a) columns should be effectively continuous at their splices
(b) pattern loading may be ignored
(c) all beams framing into the columns are assumed to be fully loaded
(d) nominal moments are applied to the columns about the two axes
(e) nominal moments are proportioned between the length above and below the beam connection according to the stiffnesses *i*/*L* of each length
(f) nominal moments may be assumed to have no effects at the levels above and below the level at which they are applied
(g) the slenderness ratio $\lambda$ of the columns, see 5.1, should not exceed 180.

Note that the nominal moments as calculated in 5.5(d) are the minimum moments to be used for column design.

## 5.5 Design procedure

(a) Calculate the factored beam reactions = 1.5 × imposed load + 1.35 × dead load from the beams bearing onto the column from each axis at the level considered. It may be necessary to calculate the reactions for different load factors for different load combinations.
(b) Calculate the factored axial force $N_{Sd}$ on the column at level being considered.

**Table 5.1 Effective length $L_{eff}$ – braced frames**

| Condition of restraint effective length (in plane under consideration) | $L_{eff}$ |
|---|---|
| Both ends unrestrained in direction, or one end partially restrained in direction and the other end unrestrained in direction | 1.0$L$ |
| Both ends partially restrained in direction, or one end restrained in direction and the other partially restrained in direction | 0.85$L$ |
| Both ends restrained in direction | 0.70$L$ |

**Table 5.2 UB Sections not class 4 in direct compression**

| Grade S 275 | | Grade S 355 |
|---|---|---|
| 914 × 419 × 388 | 305 × 127 × 37 | 610 × 305 × 238 |
| 610 × 235 × 238 | 254 × 146 × 43 | 356 × 171 × 67 |
| 610 × 235 × 179 | 254 × 146 × 37 | 305 × 165 × 54 |
| 533 × 210 × 122 | 254 × 146 × 31 | 305 × 127 × 48 |
| 457 × 191 × 98 | 254 × 102 × 28 | 305 × 127 × 42 |
| 457 × 191 × 89 | 254 × 102 × 25 | 254 × 146 × 43 |
| 457 × 152 × 82 | 203 × 133 × 30 | 203 × 133 × 30 |
| 406 × 178 × 74 | 203 × 133 × 25 | 203 × 133 × 25 |
| 356 × 171 × 67 | 203 × 102 × 23 | 203 × 102 × 23 |
| 356 × 171 × 57 | 178 × 102 × 19 | 178 × 102 × 19 |
| 305 × 165 × 54 | 152 × 89 × 16 | 152 × 89 × 16 |
| 305 × 127 × 48 | 127 × 76 × 13 | 127 × 76 × 13 |
| 305 × 127 × 42 | | |

(c) Choose a section for the lowest column length, the following may be used as a guide to the size required:

203 UC for buildings up to 3 storeys high
254 UC for buildings up to 5 storeys high
305 UC for buildings up to 8 storeys high
356 UC for buildings from 8 to 12 storeys high

If UC sections are not acceptable choose a UB section from Table 5.2, noting that as the slenderness ratio is less for a UB than a UC a larger area of section will be required.

(d) Calculate the nominal moments applied to the column about the two axes by multiplying the factored beam reactions by eccentricities based on the assumption that the loads act 100mm from the face of the column, or at the centre of a stiff bearing, whichever is greater.

If the beam is supported on a cap plate the load should be taken as acting at the edge of the column or edge of any packing (EC3, NAD B4)

(e) Obtain the nominal moments $M_{y.Sd}$ and $M_{z.Sd}$ applied to each length of the column above and below the beam connections by proportioning the total applied nominal moments, from (d) according to the rule stated in 5.4 (e).

(f) Members with class 1 and class 2 cross-sections subject to combined bending and axial compression shall satisfy the following two requirements to ensure that buckling will not take place:

$$\frac{N_{sd}}{\chi_{min}Af_y/\gamma_{M1}} + \frac{k_y M_{y,sd}}{W_{pl,y}f_y/\gamma_{M1}} + \frac{k_z M_{z,sd}}{W_{pl,z}f_y/\lambda_{M1}} \leq 1$$

and

$$\frac{N_{sd}}{\chi_{min}Af_y/\gamma_{M1}} + \frac{k_{LT} M_{y,sd}}{\gamma_{LT}W_{p,}f_y/\gamma_{M1}} + \frac{k_z M_{z,sd}}{W_{p,}f_y/\lambda_{M1}} \leq 1$$

In addition the members should also satisfy the following expression to ensure that there will not be failure at the ends:

$$\left[\frac{M_{y,sd}}{M_{N,Rd}}\right]^{\alpha} + \left[\frac{M_{z,sd}}{M_{Nz,Rd}}\right]^{\beta}$$

where: $N_{sd}$ is the design axial load on the member
$M_{y,sd}$ is the design moment about the major axis
$M_{z,sd}$ is the design moment about the minor axis
$A$ is the cross-section area of the member
$W_{pl}$ is the relevant plastic modulus of the section
$\gamma_{M1}$ is the partial material factor – taken as 1.05 in the NAD
$f_y$ is the yield strength of the material
$\chi_{min}$ the reduction factor for 'flexural buckling' about the weakest axis determined as follows:

Determine the effective length of the member about both axes
Calculate the slenderness ratio ($\lambda$) = $L_{eff}/i$ about the appropriate axis, take the largest value

Determine the non-dimensional slenderness $\bar{\lambda} = \lambda/(93.9\varepsilon)$,

$\varepsilon = \sqrt{235/f_y}$ (Note: For grade S275 steel $93.9\varepsilon = 86.8$)

Select the column curve from Table 5.3 for the appropriate section, if the selected sec tion is not in this table refer to EC3.

Determine $\chi_{min}$ from Fig. 5. 2 for the appropriate column curve.

$k_y$, $k_z$ and $k_{LT}$ are correction factors to allow for the combined effects of axial load and moment. They may be taken, very conservatively, as having the following values:

$k_y$ = 1.5
$k_z$ = 1.5
$k_{LT}$ = 1.0

Calculated values may be obtained from EC3.

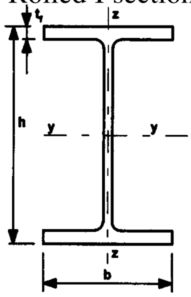
$M_{Ny,RD}$ is the moment resistance of the section in the presence of the axial load, this may be obtained from the tables used with BS 5950[13], or by using the following equations:

$M_{Ny,RD} = 1{,}11 M_{pl,y,Rd}\,(1-n)$ but $\leq M_{pl,y,Rd}$

$n = N_{sd}/N_{pl,Rd}$

for $n > 0.2$ $M_{Nz,Rd} = 1.56 M_{pl,z,Rd}\,(1-n)\,(n + 0.6)$ $\alpha = 2$ $\beta = 5n$ but $\beta \geq 1.0$

**Table 5.3    Selection of buckling curve**

| Cross section | Limits | Buckling about axis | Buckling curve |
|---|---|---|---|
| Rolled I sections  | ***h/b* > 1.2:** | | |
| | $t_f \leq$ 40mm | y-y | a |
| | | z-z | b |
| | 40mm< $t_f \leq$ 100mm | y-y | b |
| | | z-z | c |
| | ***h/b* ≤ 1.2:** | | |
| | $t_f \leq$ 100mm | y-y | b |
| | | z-z | c |
| | $t_f$ > 100mm | y-y | d |
| | | z-z | d |

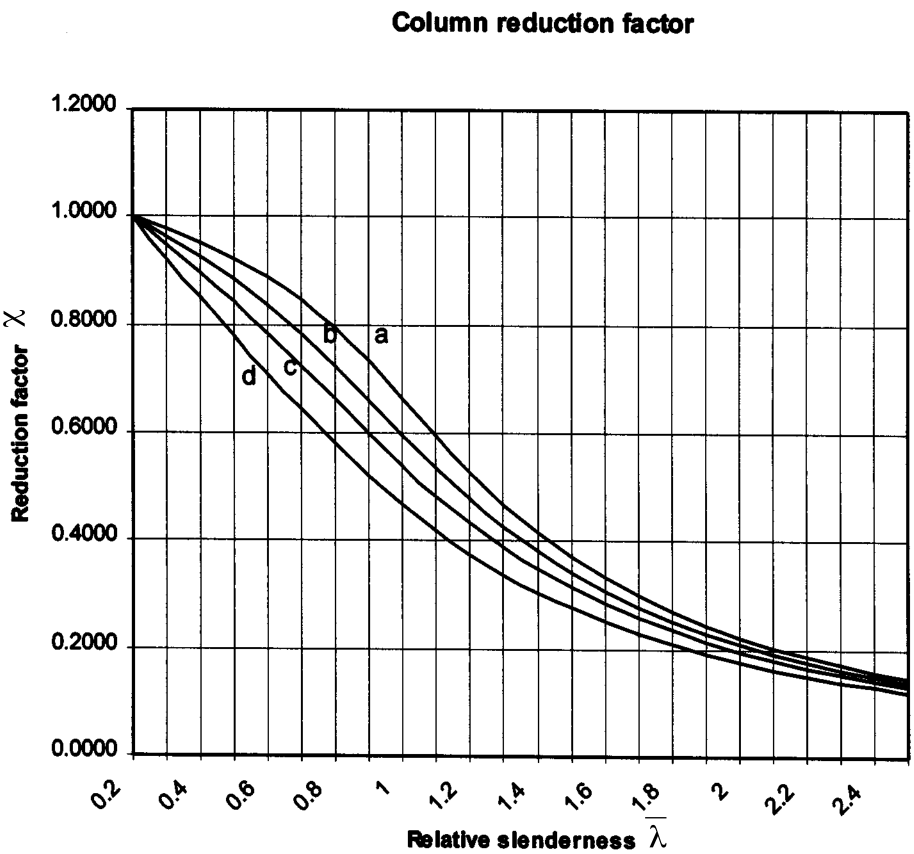



*Fig. 5.2  Reduction factor $\chi$ from relative slenderness ratio $\bar{\lambda}$*

(g) Members with class 3 cross-sections subject to combined bending and axial compression shall satisfy the following two requirements to ensure that buckling will not take place:

$$\frac{N_{sd}}{\chi_{min} A f_y/\gamma_{M1}} + \frac{k_y M_{y,sd}}{W_{el,y} f_y/\gamma_{M1}} + \frac{k_z M_{z,sd}}{W_{el,z} f_y/\lambda_{M1}} \leq 1 \qquad \text{and}$$

$$\frac{N_{sd}}{\chi A f_y} + \frac{K_{LT} M_{y,sd}}{\chi_{LT} W_{el,y} f_y} + \frac{k_z M_{z,sd}}{W_{el,z} f_y/\lambda_{M1}} \geq 1.0$$

In addition the members should also satisfy the following expression to ensure that there will not be failure at the ends:

$$\frac{N_{Sd}}{\chi A f_y} + \frac{M_{y,Sd}}{W_{el,y} f_y} + \frac{M_{z,sd}}{W_{el,z} f} \geq 1.0$$

where: $W_{el,y}$ and $W_{elz}$ are the elastic modulii of the section about the major and minor axes
$\chi_z$ is the reduction coefficient for axial load about the $z$ axis
$\chi_{LT}$ is the reduction factor for lateral torsional buckling.

The other symbols repeat those given previously.

# 6 Braced multistorey buildings – bracing and other members

## 6.1 Introduction

This Chapter describes the design of bracing and other members that are subject to compression or tension only.

In all cases the value of the slenderness ratio λ obtained by dividing the effective length $L_{\text{eff}}$ by the radius of gyration; about the relevant axis should not exceed the following:

| | |
|---|---|
| (a) for members resisting loads other than wind loads | 180 |
| (b) for members resisting self-weight and wind loads only | 250 |
| (c) for members normally acting as a tie but subject to reversal of stress resulting from wind action | 350 |

(see National Application Document)

## 6.2 Cross-sectional areas

For the design of compression members the area taken should be the gross area and no allowance made for holes for connections, although larger holes should be taken into account.

In the design of tension members the net area and gross area should always be considered.

The gross area is the total area of the section reduced to allow for any openings greater than those required for fastenings.

The net area is the gross area of the section reduced to allow for any holes for fasteners.

For single T-sections connected through the flange only and RSC sections connected through the web the net area should be calculated as the effective net area of the connected part (i.e. flange or web area with area of holes deducted) plus half the area of the outstanding elements (i.e. web of T-section and flanges of RSC).

In all other cases the net area should be calculated as the gross area with the deduction of any holes within the cross-section considered.

Where angles are connected by one leg the following rules should apply:

(a) for an equal leg angle or an unequal leg angle connected by its large leg the effective area should be taken as the gross area (EC3 clause 6.6.10).
(b) for an unequal leg angle connected by its shorter leg the effective area should be taken as equal to the gross cross-sectional area of an equivalent equal leg angle of leg size equal to the connected leg.

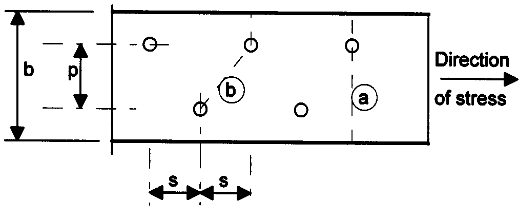
When the holes for the bolts are staggered the area to be deducted for the bolts is taken as:

(a) The area of the holes at the cross-section considered.

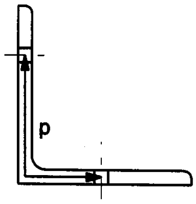
(b) The area of holes in any diagonal less $\frac{s^2 t}{4p}$ (EC3 clause 5.4.2.2)

$t$ is the material thickness

i.e. area $= b \times t - \left(2dt - \frac{s^2}{4p}\right)$




In the case of angles with holes staggered in both legs, $p$ should be measured along the thickness of material, i.e.




## 6.3 Buckling lengths and slenderness ratio

Member buckling resistance is governed by the slenderness ratio $\lambda$ which is the relevant effective length $L_E$ divided by the radius of gyration $i$ about the relevant axis.

For web members in lattice construction the system length should be taken as the distance between the points of intersection of incoming members on any axis.

For chord members the system length should be taken as the distance between the intersection of web members in the plane of the truss and between the external restraints out of the plane of the truss.

The effective length $L_E$ should be taken as the system length unless a detailed analysis is carried out by the engineer giving a smaller value.

## 6.4 Angles in compression

For angles used in compression, provided that the connection is made by at least two bolts the eccentricities may be ignored and effective slenderness ratio $\bar{\lambda}_{eff}$ obtained as follows:

for buckling about vv-axis $\bar{\lambda}_{effv} = 0.35+0.7\bar{\lambda}_v$
for buckling about yy-axis $\bar{\lambda}_{effy} = 0.5+0.7\bar{\lambda}_y$ (EC3 5.8.3)
for buckling about zz-axis $\bar{\lambda}_{effz} = 0.35+0.7\bar{\lambda}_z$

where: $\bar{\lambda} =$ $\lambda / (93.9\varepsilon)$ about the relevant axis
$\lambda$ $L_E/i$ about the relevant axis
$i$ radius of gyration about the relevant axis,
$L_E$ effective length about the relevant axis
$\varepsilon = \sqrt{235/f_y}$
*Note*: For grade 275 steel $\varepsilon = \sqrt{235/275} = 0.924 \therefore \bar{\lambda} = \lambda/(86.8)$

Where the connection is made by one bolt the effective lengths must be taken as the system length.

## 6.5 Resistance of section in axial compression

Members subject to axial compression must be checked for:

(a) The design compression resistance (EC3 clause 5.4.4)
(b) Local buckling resistance (EC3 clause 5.4.4)
(c) The design buckling resistance (EC3 clause 5.5.1)

**Design compression resistance**
Design compressive force $N_{sd}$ should, at each cross-section, meet the following requirements:

$$N_{sd} < N_{c.Rd}$$

where $N_{c.Rd}$ is the design compression resistance.

The value of $N_{c.Rd}$ should be taken as the smaller of the two values below:

1. Design plastic resistance of the gross section:

$$N_{p1Rd} = Af_y/\gamma_{M0}$$

where: $A$ is the area of cross-section
$f_y$ is the yield stress
$\gamma_{mo}$ is a partial factor for the material, given a value of 1,05 in the UK NAD.

2. The design local buckling resistance

$$N_{c,Rd} = A_{eff}f_y/\gamma_{MI} \text{ (EC3 5.4.4)}$$

where: $A_{eff}$ is the effective area of section. For class 1, 2 or 3 sections this is the gross area. For class 4 sections reference should be made to EC3 5.3.5.
$f_y$ is the yield stress
$\gamma_{M1}$ is the partial factor for the material in buckling situations given a value of 1.05 in the UK NAD

**Design buckling resistance of section in compression**

$$N_{b,Rd} = \chi\beta_A A f_y/\gamma_{MI} \text{ (clause 5.5.1)}$$

where: $\beta_A$ is 1 for class 1, 2 or 3 sections
$\gamma_{m1}$ is the partial factor for the material in buckling situations given a value of 1.05 in the UK NAD
$\chi$ is a reduction factor for buckling mode taken from curve (c) in Fig. 5.2 for the maximum $\bar{\lambda}_{eff}$.

## 6.6 Resistance of sections in axial tension (EC3 5.4.3)

For members subject to axial tension the design load $N_{sd}$ should meet the requirements of the following:

$N_{sd} < N_{t.Rd}$ where $N_{t.Rd}$ is the tension resistance of the cross-section.

$N_{t.Rd}$ should be taken as the smaller of the following two expressions.

(a) The design plastic resistance of the cross-section

$$N_{pl,Rd} = \frac{Af_y}{\gamma_{M0}}$$

where: $A$ is the gross area
$f_y$ is the yield stress
$\gamma_{M0}$ is a partial factor for the material, given a value of 1.05 in the UK NAD.

(b) The design ultimate resistance of the section at the location of the fixing bolts

$$N_{u,Rd} = 0.9 \frac{A_{net} f_u}{\gamma_{M2}}$$

where: $A_{net}$ is the net area
$f_u$ is the ultimate strength
$\gamma_{m2}$ is a partial safety factor given as 1.2 in the UK NAD

For connections using the pre-loaded (HSFG) bolts designed to be slip resistant at the ultimate limit state, the plastic resistance of the net section at the holes for fasteners $N_{net.Rd}$ should be taken as not more than:

$$N_{net.Rd} = A_{net} f_y/\gamma_{M0}$$

To ensure that the full ductile capacity is achieved before failure the plastic resistance $N_{p1,Rd}$ should be less than the ultimate resistance at the bolt holes $N_{u,Rd}$, i.e.:

$$N_{u,Rd} > N_{p1,Rd}$$

This condition will be satisfied if:

$$0.9\left\{\frac{A_{net}}{A}\right\} > \left\{\frac{f_y \gamma_{M2}}{f_u \gamma_{M1}}\right\}$$

## 6.7 Angles connected by one leg (EC3 6.5.2.3)

Angles connected by a single row of bolts in one leg may be treated as concentrically loaded and eccentricity ignored provided that the ultimate resistance of the net section is determined as follows:

1 bolt $N_{u,Rd} = \dfrac{2.0(e_2 - 0.5d_0)tf_u}{\gamma_{M2}}$

2 bolts $N_{u,Rd} = \dfrac{\beta^2 A_{net} f_u}{\gamma_{M2}}$

3 bolts $N_{u,Rd} = \dfrac{\beta^3 A_{net} f_u}{\gamma_{M2}}$

where: $e_1$, $e_2$, and $p_1$ are shown in Fig. 6.1
$A_{net}$ is the net area of angle
$d_o$ is the hole diameter
$f_u$ is the ultimate tensile strength of the material
$\beta_2$ and $\beta_3$ are reduction factors dependent on pitch of bolts as below

| Pitch $p_1$ | $< 2.5d_o$ | $< 5.0d_o$ |
|---|---|---|
| 2 bolt $\beta_2$ | 0.4 | 0.7 |
| 3 bolts $\beta_3$ | 0.5 | 0.7 |

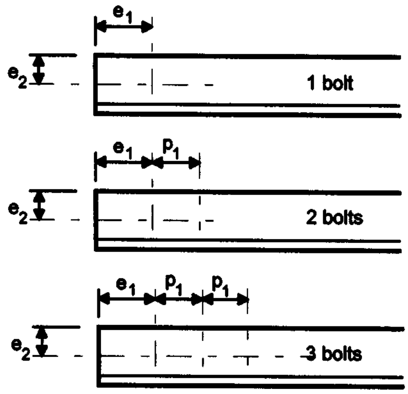



*Fig. 6.1 Edge and end distances*

For intermediate values of $p_1$, β can be determined by interpolation.

When checking values for the buckling resistance in compression of single angles the values should be based on gross area, but in no circumstances should this exceed the values of $A$ given by the above expression.

T-sections connected by their stalks or RSCs connected by their flanges should have the net area calculated by the above expressions.

## 6.8 Closely spaced built up-members (EC3 5.9.4)

For the design procedures for compound members see EC3.

## 6.9 Bending and axial tension (EC3 5.5.3)

Where members are subject to combined bending and axial tension they should be checked as in EC3.

# 7 Braced multistorey buildings – robustness

Multistorey construction that has been framed in accordance with the recommendations given in 3.4, and designed in accordance with the rest of the *Manual*, should produce a robust construction subject to the connections also being designed in accordance with the *Manual*. However in order to demonstrate that the requirements for robustness are met, the following checks should be carried out:

## 7.1 For all structures

All columns should be tied in two directions, approximately at right angles, at each principal floor and roof level which they support.

Where possible the ties should be provided in continuous lines, throughout the level under consideration. The ties should be distributed over the whole area.

The connections for the ties should be of a standard of robustness commensurate with the structure and must be designed to resist a minimum factored load of 75kN at each floor level and 40kN at the roof level. In addition those ties connected directly to a column should also be capable of carrying at least 1% of the axial load in the column at that level.

## 7.2 For structures 5 storeys or more in height

In addition to the above general requirements buildings in the UK greater than 4 storeys should be designed for the following:

1. Ties at each floor and roof should be checked for the following factored tensile forces which need not be considered as additive to other loads:

   internal ties $0.5\ W_f\ S_t\ L_a$
   edge ties $0.25\ W_f\ S_t\ L_a$

   where: $W_f$ is the total factored dead and imposed load per unit area
   $S_t$ is the mean transverse spacing of the ties, and
   $L_a$ is the greatest distance in the direction of the tie or beam, between adja cent lines of columns or vertical supports.

   For composite and *in situ* floors the reinforcement in the floor construction may be used to resist the tying forces.

   For the purposes of this provision it may be assumed that substantial deformation of members and their connections is acceptable.

2. Column splices should be checked for a tensile force of not less than two-thirds of the factored vertical load applied to the column from the next floor level below the splice.

## 7.3 Localisation of damage

In an accidental situation the removal of a column or beam carrying a column should be considered using the following alternative procedures:

(a) Area of damage, following the notional removal at each storey in turn of a single column or beam supporting a column, should be assessed. Failure should be contained within the adjacent storeys and restricted to an area within those storeys of approximately either $70m^2$ or 15% of the area of the storey, whichever is less.

When carrying out the check the following should be noted:

(i) It may be assumed that substantial permanent deformation of members and their connections is acceptable, e.g. the loads can be carried by catenary action.
(ii) Except in buildings used predominately for storage or where the imposed loads are of permanent nature, one-third of the normal imposed loads should be used. Also, only one-third of the normal wind loads need be considered.
(iii) Partial safety factor for loads, $\gamma_f$, should be taken as 1.05 except that $\gamma_f$ of 0.9 should be used for dead loads restoring overturning action.

(b) Where damage cannot be localised, using the method in (a) above, the relevant members should be designed as follows:

(i) Stipulated accidental load (normally taken as $34kN/m^2$) should be applied to the member in question, from the appropriate direction. The reaction on the member from other components attached to the member should be calculated on the basis of the same accidental load but should be limited to the ultimate strength of these components or their connections
(ii) In conjunction with the above the effects of ordinary loads should be considered using partial safety factors noted in (a) (iii) above
(iii) Any structural component vital to the stability of the relevant members as defined in (b) should itself be designed in a similar way to that described in clause (b)
(iv) Subclauses 7.3(a)(i), (ii) and (iii) apply.

# 8 Braced multistorey buildings – the next step

## 8.1 Introduction

Preliminary general-arrangement drawings should be prepared when the design of the structural steel members has been completed, including sections through the entire structure. If necessary, typical connection details should be prepared and sent to other members of the design team for comment, together with a brief statement of the principal design assumptions, i.e. imposed loadings, weights of finishes, fire ratings and durability.

## 8.2 Connections

It is important to establish the general form and type of connections assumed in the design of the members and to check that they are practicable. It is also important to consider the location of edge beams and splices, the method and sequence of erection of steelwork, access and identification of any special problems and their effects on connections, such as splicing/connection of steelwork erected against existing walls. The extent of welding should also be decided, as it may have an effect on cost. Preliminary design of typical connections is necessary when:

- appearance of exposed steelwork is critical
- primary and secondary stresses occur that may have a direct influence on the sizing of the members
- connections are likely to affect finishes such as splices affecting column casing sizes and ceiling voids
- steelwork is connected to reinforced concrete or masonry, when greater constructional tolerances may be required, which can affect the size and appearance of the connections
- unusual geometry or arrangement of members occurs
- holding down bolts and foundation details are required
- a detail is highly repetitive and can thus critically affect the cost.

When necessary the design of connections should be carried out in accordance with Chapters 14 and 15.

## 8.3 Finalisation of design

Before the design of the structure can be finalised, it is necessary to obtain approval of the preliminary drawings from the other members of the design team. The drawings may require further amendment, and it may be necessary to repeat this process until approval is given by all parties. When all the comments have been received, it is then important to marshal all the information received into a logical format ready for use in the final design. This may be carried out in the following sequence:

- checking all information
- preparation of a list of design data
- amendments of drawings as a basis for final calculations.

## 8.4 Checking all information

The comments and any other information received from the client and the members of the design team and the results of the ground investigation should be checked to verify that the design assumptions are still valid.

## 8.5 Stability

Check that no amendments have been made to the sizes and to the disposition of the bracing to the structure. Check that any openings required by the client can be accommodated in the final design.

## 8.6 Movement joints

Check that no amendments have been made to the disposition of the movement joints.

## 8.7 Loading

Check that the loading assumptions are still correct. This applies to dead and imposed loading such as floor finishes, ceilings, services, partitions, and external wall thicknesses, materials and finishes thereto. Verify the design wind loading, and consider whether or not loadings such as earthquake, accidental, constructional or other temporary loadings should be taken into account.

## 8.8 Fire resistance, durability and sound insulation

Confirm with other members of the design team the fire resistance required for each part of the structure, the corrosion protection that applies to each exposure condition and the mass of floors and walls (including finishes) required for sound insulation.

## 8.9 Foundations

Examine the information from the ground investigation and decide on the type of foundation to be used in the final design. Consider especially any existing or future structure adjacent to the perimeter of the structure that may influence not only the location of the foundations but also any possible effect on the superstructure and on adjacent buildings.

## 8.10 Performance criteria

Confirm which codes of practice and other design criteria are to be used in the design.

## 8.11 Materials

Confirm the grade of steel and type of bolts or welds to be used in the final design for each or all parts of the structure. The use of different grades of bolts and different types of welds on the same structure should be avoided.

## 8.12 Erection

Review the effect on the design of the method and sequence of erection and the use of any temporary erection bracing.

## 8.13 Preparation of design data list

The information obtained from the above check and that resulting from any discussions with the client, design team members, building control authorities and material suppliers should be entered into a design information data list. This list should be sent to the design team leader for approval before the final design is commenced.

## 8.14 Amendment of drawings as a basis for final calculations

The preliminary drawings should be brought up to date, incorporating any amendment arising out of the final check of the information previously accumulated and finally approved. In addition, the details listed in 8.15–8.17 should be added to all the preliminary drawings as an aid to the final calculations.

## 8.15 Grid lines

Establish grid lines in two directions, which should if possible be at right-angles to each other. Identify these on the plans.

## 8.16 Members

Give all slabs, beams and columns unique reference numbers or a combination of letters and numbers related if possible to the grid, so that they can be readily identified on the drawings and in the calculations.

## 8.17 Loading

Mark on the preliminary drawings the loads that are to be carried by each floor or roof slab. It is also desirable to mark on the plans the width and location of any walls or other special loads to be carried by the slabs or beams.

## 8.18 Sequence for finalising design

When all the above checks, design information, data lists and preparation of the preliminary drawings have been carried out, the design of the structure should be finally checked. This should be carried out in the same logical sequence, as in the preceding sections, e.g.:

- floors
- beams
- columns
- bracing and other members
- robustness
- connections.

The redesign of any steel members that may be necessary should be carried out as described for each member in the preceding sections.

# 9 Single-storey buildings – general

## 9.1 Introduction

This Chapter offers advice on the general principles to be applied when preparing a scheme for a single-storey building. The aim should be to establish a structural scheme that is practical, sensibly economic and not unduly sensitive to the various changes that are likely to be imposed as the overall design develops.

Loads should be carried to the foundations by the shortest and most direct routes. Try to achieve economy through simplicity with maximum repetition of fabricated members and details, avoiding congested, awkward or structurally sensitive details. Aim at using straightforward temporary works and avoid designing structures which require unusual erection sequences to achieve the intended behaviour.

When sizing members, start with those of longest span and those carrying largest loads. The same sizes of members can then be used for those of slightly shorter span and those carrying somewhat less load. Consider changing to smaller or lighter sections only after considering the economy of such changes. Adopting a uniform size for members of slightly variable span is usually less costly in design, detailing, procurement, fabrication and erection than using a variety of sizes to produce the lowest possible weight of steel.

Simple structural schemes are quick to design and easy to build. They may be complicated later by other members of the design team trying to achieve their optimum conditions, but a simple scheme provides a good start to any design work and is more easily amended to accommodate other designers' requirements than a complex one.

Scheme drawings should be prepared for discussion and budgeting purposes, incorporating a general arrangement of the structure showing bracing, types of roof and wall cladding, beam and column sizes, typical edge details, any critical and unusual details and proposals for fire and corrosion protection.

When the comments of other members of the design team have been received and assimilated, the structural scheme should be revised and the structural members checked and redesigned as necessary.

## 9.2 Actions (loads)

Loads should be based on BS 648: 1964[3], BS 6399: Parts 1: 1984[4] and 3: 1988[5] and CP3: Chapter V: Part 2: 1972[6]. Note that these are not necessarily the latest revisions but are included in the NAD as a result of calibration.

## 9.3 Permanent actions *G*

Permanent actions are dead and service loading and include the weight of the roof sheeting and equipment fixed to the roof, the structural steelwork, the ceiling and any services. The following approximate loads may be used for preliminary designs in the absence of actual loads:

| | |
|---|---|
| Roof sheeting and side cladding | 0.1 to 0.2 kN/m$^2$ |
| Steelwork | 0.1 to 0.3 kN/m$^2$ |
| Ceiling and services | 0.1 to 0.3 kN/m$^2$ |

Although services loading is treated as a dead load it should not be included when determining the resistance to overturning and uplift.

### 9.3.1 Variable actions *Q*

Variable actions include live loads such as imposed loads, wind load and snow load. Imposed loading is specified in BS 6399: Part 3: 1988[5] as a minimum of 0.60kN/m$^2$ for pitched roofs of 30° to the horizontal or flatter. For roofs where access, other than for normal maintenance, is required, the minimum imposed loading should be increased to 1.5kN/m$^2$. BS 6399: Part 3: 1988[5] also gives the loads arising from the effects of uniformly distributed and drifting snow.

The UK NAD specifies wind loading as 90% of the value obtained from CP3: Chapter V; Part 2: 1972[6]. Wind loading varies with roof pitch and with the location of the building and presence of dominant openings.

### 9.3.2 Equivalent horizontal loading arising from frame imperfections

The derivation and application of this loading is given in 2.10.3 of this *Manual*.

### 9.3.3 Strength and stability limit states

The partial factors ($\gamma$) are given in Tables 1 and 2 and the combinations factors ($\psi$) in Tables 3 and 4 of the NAD. The factored loads to be used for each load combination are obtained by multiplying the unfactored loads by the appropriate partial factors and combination factors. The usual factors needed for design are summarised in Table 2.1 of this *Manual*. Equivalent horizontal loads arising from frame imperfections are to be included in the dead and imposed loads in this Table.

## 9.4 Material selection

In the UK, material for rafters and columns should generally be of grade S 275, although it may be cost-effective to use grade S 355 unless deflection is likely to be critical. For example, S 355 may be used for latticed and trussed members. Grade 8.8 bolts should normally be used throughout and, in the interests of simplicity, a uniform diameter should be adopted for all bolted connections. M20 bolts are suitable for general use except for heavily loaded connections where M24 or larger bolts can be used. For light latticed frames, M16 bolts may be appropriate. In order to save confusion and mistakes on site it is recommended that a single size and type of bolt be used on a project, as far as is practical.

## 9.5 Structural form and framing

The most common forms of single-storey frames, illustrated in Figs. 9.1 and 9.2, are:

- portal frames with pinned bases
- posts with pinned bases and nominally parallel lattice girders.
- posts with castellated (or 'cellform') beams

Fixed bases can be used instead of pinned bases, but these generally require larger and more costly foundations which may not justify any saving in the weight of steel frame resulting from fixing column bases. The choice of structural form will depend on such factors as the appearance of the building, the extent that supports are required in the roof for ceiling and services and on overall economy.

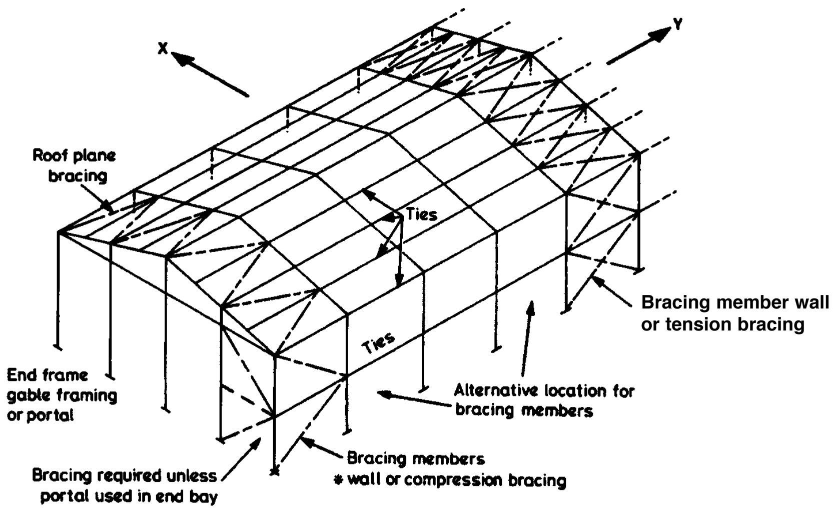



*Fig. 9.1 Typical bracing system*

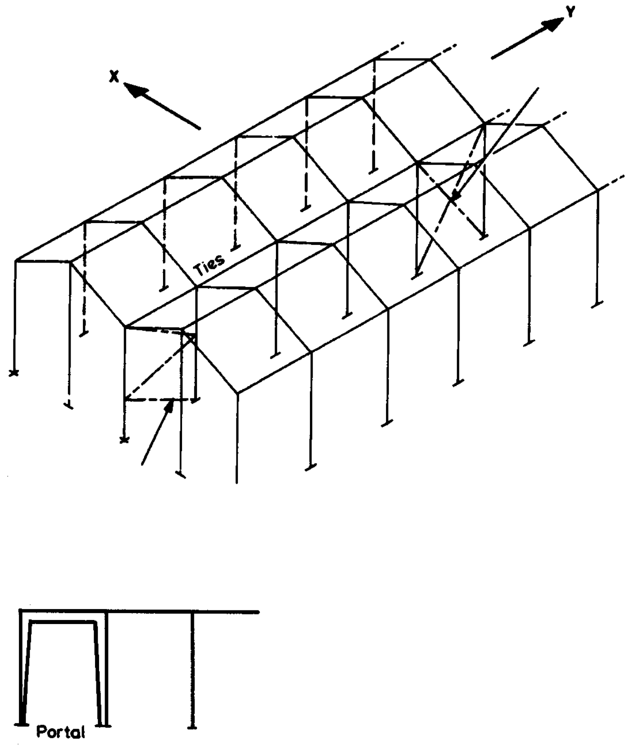



*Fig. 9.2 Framing for multi-bay structures*

The following information relating to single-storey buildings is based on previous experience in determining sound and economical arrangements of structural frames.

Main frames of up to 30m spans and spaced from 4.5m to 7m apart have proved to be economical.

Portal framed buildings have resistance to horizontal loads in the plane of the frames. At right angles to the plane of the frames, vertical bracing in the side walls is required, arranged symmetrically wherever possible.

Buildings which do not have portal frame action to resist horizontal loads in bending require vertical bracing arranged in planes at right angles to each other.

For all types of building, bracing is required in the roof slope to transfer horizontal loads to the vertical bracing system. Such roof bracing can be avoided if the roofing material can be designed to act as a membrane to transmit horizontal loading, but bracing is a useful aid to steelwork erection and without it temporary means of ensuring stability of the steel frame during construction will be essential. In general, it is better to provide permanent bracing.

It may be necessary also to provide horizontal bracing at the level of the bottom of trusses or latticed girders to stabilise the bottom members if they are subject to reversals of loading from the action of wind. Such horizontal bracing may also be required to transmit horizontal loads from gable ends of buildings to the vertical bracing in the side walls.

Consider the provision of movement joints or the effects of thermal expansion for buildings in the UK where plan dimensions exceed 50m. Bear in mind that thermal movements will occur from the location of braced bays and that where movement joints are used it is necessary to provide bracing in each section of a building separated by such joints.

If purlins are located at the node points of trusses or latticed girders, the rafters are not subject to local bending. If purlins are not located at nodes, then local bending must be taken into account when sizing the rafters.

The arrangement of the framing must take account of openings for doors and windows and any support required for services. Framing may be required to transfer horizontal loads at openings to the bracing system.

## 9.6 Fire resistance

Fire protection should be considered for those frames that provide lateral stability to perimeter or party walls and which are required to have a fire rating. Protection can be achieved by casing either the whole frame or only the columns in fire-resistant material. If only the columns are so treated, the stability of the columns acting as cantilevers without the lateral support provided by the rafters or trusses must be checked to ensure the stability of the whole structure is not in danger if the effectiveness of these members is removed by the action of a fire. Fire protection should also be considered for structural members supporting mezzanine floors enclosed by single-storey buildings. Further guidance on fire protection can be found in references 24 & 25.

## 9.7 Corrosion protection

Structural steelwork should be protected from corrosion. See 2.7 of this *Manual*.

## 9.8 Bracing

Choose the location and form of bracing in accordance with the recommendations in 9.4 and 2.2.3. Typical alternative locations are shown on Figs 9.1 and 9.2 for single-storey buildings.

Wind loads on the structure should be assessed for the appropriate load combinations and divided into the number of bracing bays resisting the horizontal forces in each direction.

## 9.9 Roof and wall cladding

Although this *Manual* is concerned with the design of structural steelwork, it is essential at the start of the design to consider the details of the roof and cladding systems to be used, since these have a significant effect on the design of steelwork.

The choice of cladding material largely depends on whether the roof is flat or pitched. For the purposes of this *Manual*, a roof will be considered flat if the roof pitch is less than 6°. It should be noted, however, that roofs with pitches between 6° and 10° will often require special laps and seals to avoid problems with wind-driven rain etc.

The variety of materials available for pitched roofs is vast and cannot be dealt with in detail in a *Manual* such as this. However, a brief description of the most common forms is included in Table 9.1 which summarises the salient features of the various types of lightweight roofing systems commonly used in the UK.

**Table 9.1 Lightweight roofing systems and their relative merits**

| Description | Minimum pitch | Typical depth mm | Typical span mm | Degree of lateral restraint to supports | Comments |
|---|---|---|---|---|---|
| Galvanized corrugated steel sheets | 10° | 75 sinusoidal profile | 1800 to 2500 | Good if fixed direct to purlins | Low-budget industrial and agricultural buildings limited design life, not normally used with insulated liner system |
| Fibre-cement sheeting | 10° | 25 to 88 | 925 to 1800 | Fair | Low-budget industrial and agricultural buildings brittle construction usually fixed to purlins with hook bolts |
| Profiled aluminium (insulated or uninsulated) | 6° | 30 to 65 | 1200 to 3500 | Good if fixed direct to purlins | Good corrosion resistance but check fire requirements & bimetallic corrosion with mild steel supporting members |
| Profiled coated steel sheeting (insulated or uninsulated) | 6° | 25 to 65 | 1500 to 4500 | Good if fixed direct to purlins | The most popular form of lightweight roof cladding used for industrial-type buildings; wide range of manufacturers, profile types & finishes |
| 'Standing seam' roof sheeting (steel or aluminium) | 2° | 45 | 1100 to 2200 | No restraint afforded by cladding, clip fixings | Used for low-pitch roof and has few or no laps in direction of fall; usually requires secondary supports or decking, which may restrain main purlins |
| Galvanised steel or aluminium decking systems | Nominally flat | 32 to 100 | 1700 to 6000 | Very good | Used for flat roof with insulation and vapour barrier and waterproof membrane over fire and bimetallic corrosion to be checked if aluminium deck used |
| Timber | Nominally flat | General guidance as for timber floors | | | |
| Reinforced woodwool slabs | Nominally flat | 50 to 150 | 2200 to 5800 | Good if positively fixed to beam flanges | Pre-screeded type can prevent woodwool becoming saturated during construction |

In general, most of the profiled decking systems described in Table 9.1 for pitched roofs are available for use as wall cladding. Where insulation is required this can be provided either as bonded to the sheeting or in a ‘dry-lining’ form with the internal lining fixed to the inside face of the sheeting rails. Similarly, fire protection of the walls of industrial buildings can be achieved by using boarding with fire-resistant properties on the inside face of the sheeting rails.

It is not uncommon to provide brickwork as cladding for the lower 2.0–2.5m of industrial buildings, because of the vulnerability to mechanical damage. Where this detail is required it is usually necessary to provide a horizontal steel member at the top of the wall spanning between stanchions to support such brick panel walls against lateral loading.

# 10 Single-storey buildings – purlins and side rails

## 10.1 Purlins

As the design rules for purlins are not specifically covered in EC3 the NAD gives, as an alternative, the use of the traditional empirical rules in BS 5950[13]. Purlins may consist of cold-formed, rolled or hollow sections.

### 10.1.1 Cold-formed sections

For cold-formed sections empirical rules and design formulae are given in BS 5950: Part 5[26]. However, the section sizes for cold-formed purlins can be determined from the safe load tables in the technical literature provided by the manufacturers of cold-formed members for use as purlins. Anti-sag bars tied across the apex should be provided as recommended by the manufacturers.

### 10.1.2 Angles and hollow sections

Angles and hollow section purlins may be designed in accordance with the empirical method, provided that they comply with the following rules:

- claddings and fixing thereof to be capable of providing lateral restraints to the purlins
- grade of steel to be S 275
- unfactored loads to be considered, and loading to be basically uniformly distributed
- imposed load used in design to be not less than $0.75 kN/m^2$
- span not to exceed 6.5m, and roof pitch not to exceed 30°
- purlins to be connected at each end by at least two fixings.

If these rules cannot be complied with then the purlins should be designed as beams. If the empirical rules are complied with then the elastic modulus, $Z$, and dimensions $D$ and $B$, depth and width, respectively, should not be less than the following:

| **Section** | **$Z$, cm$^3$** | **$D$, mm** | **$B$, mm** |
|---|---|---|---|
| angles | $\frac{WL}{1800}$ | $L/45$ | $L/60$ |
| CHS | $\frac{WL}{2000}$ | $L/65$ | |
| RHS | $\frac{WL}{1800}$ | $L/70$ | $L/150$ |

where: $W$ is the total unfactored load in kN due to either (dead plus imposed) or (wind less dead), and

$L$ is the span in mm.

## 10.2 Side rails

Side rails may consist of angles, hollow sections or cold-formed sections.

### 10.2.1 Cold-formed sections

The section sizes for cold-formed side rails can be determined from the safe load tables in the technical literature provided by the manufacturers of cold-formed members for use as side rails. Anti-sag rods tied to an eaves beam should be provided as recommended by the manufacturers.

### 10.2.2 Angles and hollow sections

Angle and hollow-section side rails may be designed by the empirical method provided that they comply with the following rules:

- claddings and fixing thereto to be capable of providing lateral restraint to the side rails
- grade of steel to be S 275
- characteristic loads to be considered and the loading to consist of wind and self-weight of cladding only
- span not to exceed 6.5m
- side rails to be connected at each end by at least two fixings.

If the empirical rules are complied with then the elastic modulus $Z_1$ and $Z_2$ of the axes parallel and perpendicular to the plane of the cladding, respectively, and the dimensions $D$ and $B$ perpendicular and parallel to the plane of the cladding should not be less than the following:

| Section | $Z_1$, cm$^3$ | $Z_2$, cm$^3$ | $D$, mm | $B$, mm |
|---|---|---|---|---|
| angles | $\frac{W_1 L}{1800}$ | $\frac{W_2,L}{1200}$ | $L/45$ | $L/60$ |
| CHS | $\frac{W_1 L}{2000}$ | $\frac{W_2 L}{1350}$ | $L/65$ | – |
| RHS | $\frac{W_1 L}{1800}$ | $\frac{W_2 L}{1200}$ | $L/70$ | $L/70$ |

where: $W_1$ and $W_2$ are the unfactored loads in kN acting perpendicular and parallel, respectively, to the plane of the cladding, and
$L$ is the span in mm.

Note: For $Z_2$ and $B$, $L$ may be taken as the span between anti-sag rods provided that these are properly supported.)

If these rules cannot be complied with then the side rails should be designed as beams.

# 11 Portal frames with pinned bases

## 11.1 Elastic design

Elastic frame analysis may be used to obtain the forces and moments on the frame (actions). The members should then be designed using the procedures given in Chapters 4 and 5 of this guide for the design of beams and columns. In addition, sway stability checks should be carried out as for plastic design.

## 11.2 Plastic analysis

Plastic methods of analysis are commonly used in the design of portal frames. Guidance is given on the design of single-span portals with pinned bases and where wind loading does not control the design.

For multibay frames of equal spans where the same rafter section is used throughout, the design is almost invariably governed by that for external bays. The internal stanchions are subjected to very little bending unless the loading is asymmetrical, and may therefore conservatively be made the same size as the stanchions for the single-span case.

It should however be noted that the eaves deflections of pitched multibay frames should be carefully checked, as the horizontal deflections will be cumulative. This applies particularly to frames which have a steep pitch.

If it is necessary to refine the design for multibay frames and for frames where wind is likely to govern the design, then specialist literature should be consulted and/or computer programs used.

The procedure for the plastic method of design of portal frames with pinned bases is given in this *Manual* in the following sequence:

1. sizing of rafters and stanchions
2. check on sway and snap-through stability
3. check on serviceability – deflection
4. check on position of plastic hinge and calculation of load capacity of frame
5. check on stability of plastic hinges, rafter, haunch and stanchions.

## 11.3 Single-storey portals – sizing of rafters and stanchions

The initial trial sizes for the plastic analysis of a portal frame with pinned bases is carried out in the *Manual* by the selection of members from graphs. This method is based on the following assumptions:

(a) plastic hinges are formed at the bottom of the haunch in the stanchion and near the apex in the rafter, the exact position being determined by the frame geometry
(b) the depth of the haunch below the rafter is approximately the same as the depth of the rafter
(c) the haunch length is 10% of the span of the frame, an amount generally regarded as providing a balance between economy and stability
(d) the moment in the rafter at the top of the haunch is $0.87M_p$, and it is assumed that the haunch region remains elastic
(e) the calculated values of $M_p$ are provided exactly by the sections and that there are no

stability problems. Clearly these conditions will not always be met, and the chosen sections should be fully checked for all aspects
(f) wind loading does not control design.

## 11.4 Design procedure

The procedure to be adopted is set out below, and the various dimensions are shown on Fig. 11.1

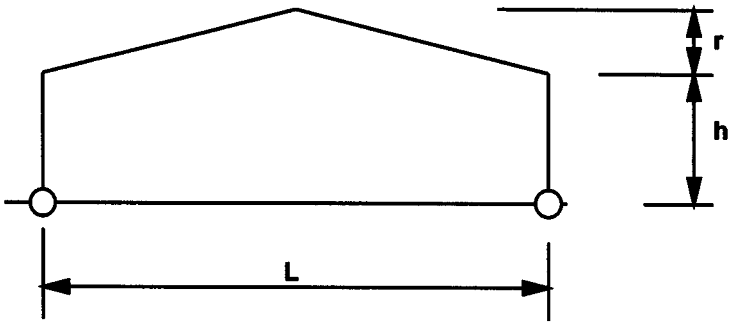



*Fig. 11.1 Dimensions of portal*

(a) calculate the span/height to eaves ratio = $L/h$
(b) calculate the rise/span ratio = $r/L$
(c) calculate the total design load $WL$ on the frame from 9.2, and then calculate $WL^2$, where $W$ is the load per unit length of span $L$ (e.g. $W = ws$, where $w$ is the total factored load per $m^2$ and $s$ is the bay spacing)
(d) from Fig. 11.2 obtain the horizontal force ratio $H_{FR}$ at the base from $r/L$ and $L/h$
(e) calculate the horizontal force at base of span $H = H_{FR}\ WL$
(f) from Fig. 11.3 obtain the rafter $M_p$ ratio $M_{PR}$ from $r/L$ and $L/h$
(g) calculate the $M_p$ required in the rafter from $M_p$ (rafter) = $M_{pr}WL^2$
(h) from Fig. 11.4 obtain the stanchion $M_p$ ratio $M_{pl}$ from $r/L$ and $r/h$
(i) calculate the $M_p$ required in the stanchion from $M_p$ (stanchion) = $M_{pl}WL^2$
(j) determine the plastic moduli for the rafter $S_{XR}$ and stanchion $S_{XL}$ from:

$$W_{pfy} = M_p\ (\text{rafter})/f_y$$
$$W_{pfy} = M_p\ (\text{stanchion})/f_y$$

where $f_y$ is the yield strength obtained from Table 2.2.

Using these plastic moduli, the rafter and stanchion sections may be chosen from the range of plastic sections as so defined in the section books.

Surtees and Yeap[27] have provided graphs which give additional design aids to the cases illustrated in Figs. 11.2, 11.3 and 11.4.

## 11.5 Sway and rafter stability

All structures move when loaded, regardless of the size of the frame and magnitude of the actions, this will have an effect on the stability of the frame. In general the movement coupled with any axial compressive force ($P\Delta$ effect) will reduce the capacity of the frame. As no methods are given in EC3 it is suggested that the procedure given in papers by Davies[28,29] be adopted.

Two modes of failure have been identified for portal frames, the first may occur in any frame and is called 'sway stability'. The mode of failure is caused by the change in frame geometry arising from applied loading which gives rise to the $P\Delta$ effect, when axial loads

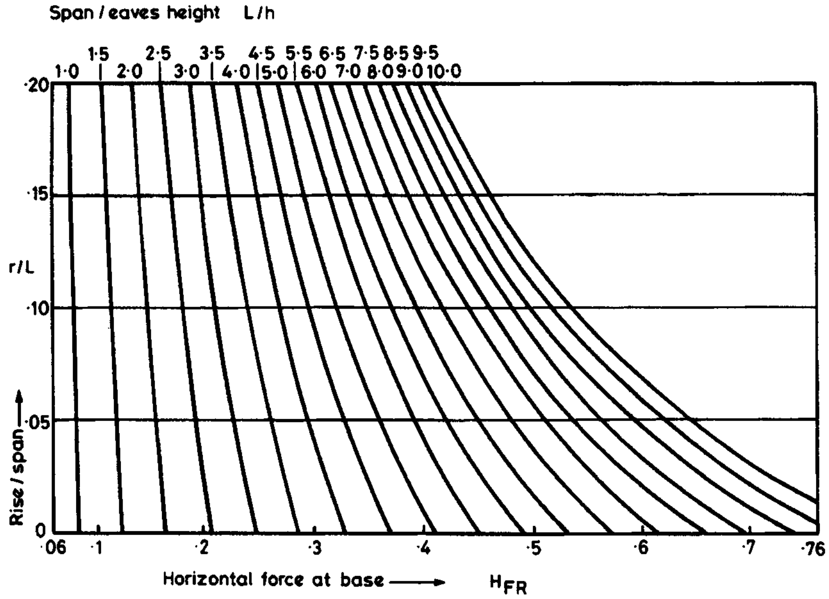



*Fig. 11.2 Horizontal force at base*

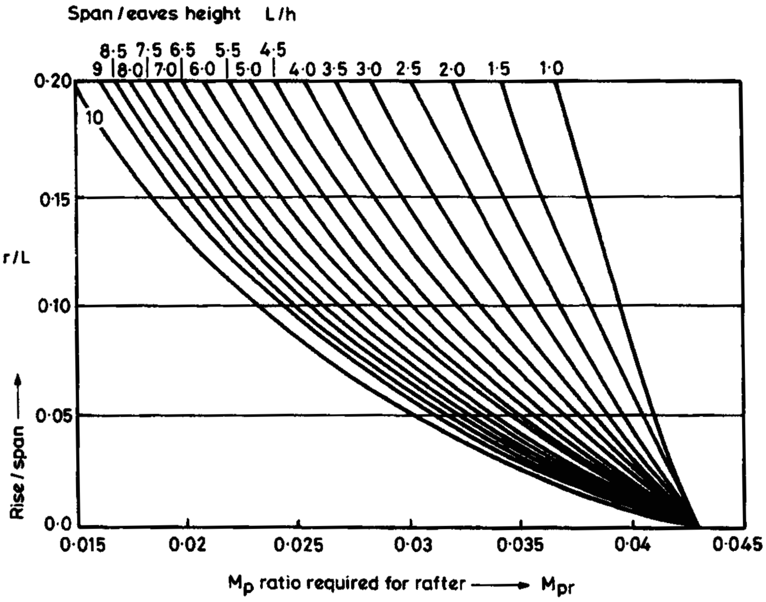



*Fig. 11.3 $M_p$ ratio required for rafter $M_{pr}$*

on compression members displaced from their normal positions give moments which reduce the frame's resistance.

The second mode can take place when the rafter frames of three or more bays have their sections reduced because full advantage has been taken of the fixity provided in the valleys. In this case the risk of snap-through should be considered.

This *Manual* gives equations so that both of these cases can be checked. If either check

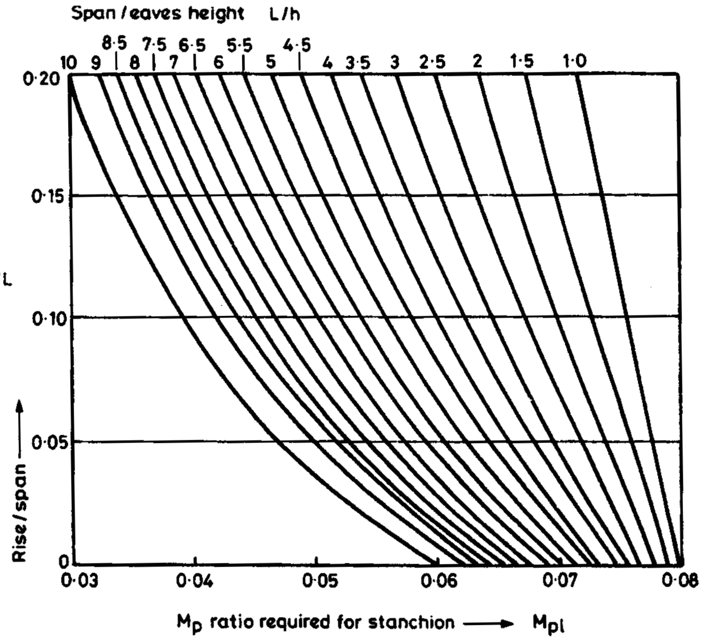



*Fig. 11.4 $M_p$ ratio required for stanchion $M_{pl}$*

is not satisfied then it is essential that the member sizes are adjusted so that the frame passes these checks. If access to more elaborate methods of calculation is available then the deflection of the frame against certain criteria may be used as a less conservative approach.

*Sway stability check*
The sensitivity of the frame to sway stability failure may be determined from the elastic critical load. EC3 uses the elastic critical load on frames to determine the possibility of sway failure, a similar procedure may be used for portal frames. This may be determined from the following equations:

$$\lambda_{cr} = \frac{3EI_r}{s[(1+\frac{1.2}{R})P_c h + 0.3P_r s]} \text{ for a pinned base frame, and}$$

$$\lambda_{cr} = \frac{5E(10+R)}{\frac{5P_r s^2}{I_r} + \frac{2RP_c h^2}{I_c}} \quad \text{for fixed base frames.}$$

where: $P_c$ is the axial force in the outer column
$P_r$ is the axial force in the outer rafter
$I_c$ is the second moment of area of the outer column
$I_r$ is the second moment of area of the outer rafter
$h$ is the height to eaves
$s$ is the length of the rafter
$R$ is the ratio of the stiffness of the column to that of the rafter

If $\lambda_{cr} \geq 10$, the inplane stability of the frame can be considered as satisfactory. The consequences of this are:

(a) For plastically analysed frames the plastic collapse capacity may be taken as the failure strength of the frame.
(b) For elastically analysed frames the members may be taken as being adequate if the checks are made assuming the results of elastic analysis in the direction of the frame.
(c) The effective length of the members may be taken as zero in the plane of the frame, as there is no requirement to enhance the members for stability.

If $4.6 \leq \lambda_{cr} \leq 10$ then the following should be applied:

(a) For plastically analysed frames the plastic collapse strength should be greater than the total factored actions multiplied by:

$$\frac{0.9\lambda_{cr}}{(\lambda_{cr} + 1)}$$

(b) For elastically analysed frames the moments derived from elastic analysis should be enhanced by:

$$\frac{\lambda_{cr}}{(\lambda_{cr} + 1)}$$

(c) The effective length of the members in the plane of the frame is taken as zero, since the inplane stability has been taken into account by adopting the above procedures.

If $\lambda_{cr}$ is $\leq 4.6$ it will probably be found that the frame will be rejected for other reasons, such as deflection at working load.

The whole frame of multiple bays may be checked by taking the lower value of $\lambda_{cr}$ of the two outer bays and applying this value to the whole frame.

For frames with three or more bays the stability of the internal rafters should also be checked in order to do this the value of $\lambda_{cr}$ for the rafter must be determined from:

$$\lambda_{cr} = \frac{E(20 + 3R)}{\frac{2P_r L_r^2}{I_r} + \frac{0.3P_c h^2}{I_c}}$$

If this gives a value of:

$$\lambda_{cr} > \frac{17EI}{P_r L_r^2}$$

then the alternative expression:

$$\lambda_{cr} = \frac{E(180 + 3R)}{\frac{11P_r L_r^2}{I_r} + \frac{0.3P_c h^2}{I_c}}$$

should be used.

The average stiffness may be used for members which are tapered.

Special consideration should be given to portal frames which have no internal legs, but are supported on valley beams. The original papers[28,29] published in *The Structural Engineer* should be studied before determining $\lambda_{cr}$.

## 11.6 Serviceability check – deflection

The horizontal deflection at the eaves may be estimated for unfactored loads by obtaining the deflection factor $D$ from Fig. 11.5 using $L/h$ and the angle of the roof slope $q$.

The estimated horizontal deflection of one side stanchion at the eaves, $d_E$, is then obtained from:

$$d_E = D\left[\frac{hL}{d_f} \cdot \frac{f_y}{\lambda_p}\right] \times 10^{-5}$$

where: $h$ = height to eaves in mm
$L$ = span in mm
$d_r$ = depth of rafter in mm
$f_y$ = yield strength in N/mm$^2$, and
$\gamma_p$ = the load factor on the frame, which may be taken as 1.5 for this check.

This horizontal deflection $d_E$ should not normally exceed $h/300$ as indicated in 2.6.2.1 unless claddings are used which can accommodate larger deflections.

The deflection of the ridge should be obtained from

$$d_{RE} = d_E \times \cot q$$

and should be small enough to have no adverse effect on the cladding, finishes and appearance.

Member sizes should be adjusted if either $d_E$ or $d_{RE}$ exceeds the chosen permissible values.

It should be noted that more accurate estimates of the deflections may be obtained by the use of suitable computer programs.

## 11.7 Check on position of plastic hinge in rafter and calculation of load capacity

In order to check that the correct mode of failure has been assumed a reactant diagram should be drawn. This is obtained by plotting the moments due to the applied forces and known moments at hinge locations, including feet. If the moments at all points in the frame are less than the values of $M_p$ and only equal to $M_p$ at the hinge locations then the assumptions may be considered as satisfactory. If $M_p$ of the frame is exceeded at any point in the frame then the diagram must be adjusted to take this into account.

In order to check the position of the plastic hinge and the load capacity of the frame previously designed the following simple procedure may be carried out:

(a) consider a pinned based portal frame subject to vertical loading as shown in Fig. 11.6
(b) calculate $H = M_p$ (stanchion)$/h_1$
(c) calculate $r/L$ and $L/h$ and then determine $x$ from Fig. 11.7
(d) take moments about the rafter hinge position giving
(e) $M$ (rafter) $= w'L/2 + H\,h_2 - w'x^2/2$
(f) as the moment of resistance of the rafter is known ($M_p$) then the value of $w'$ may be determined.
(g) redesign the frame if the load capacity $w'$ is less than the design load on the frame.

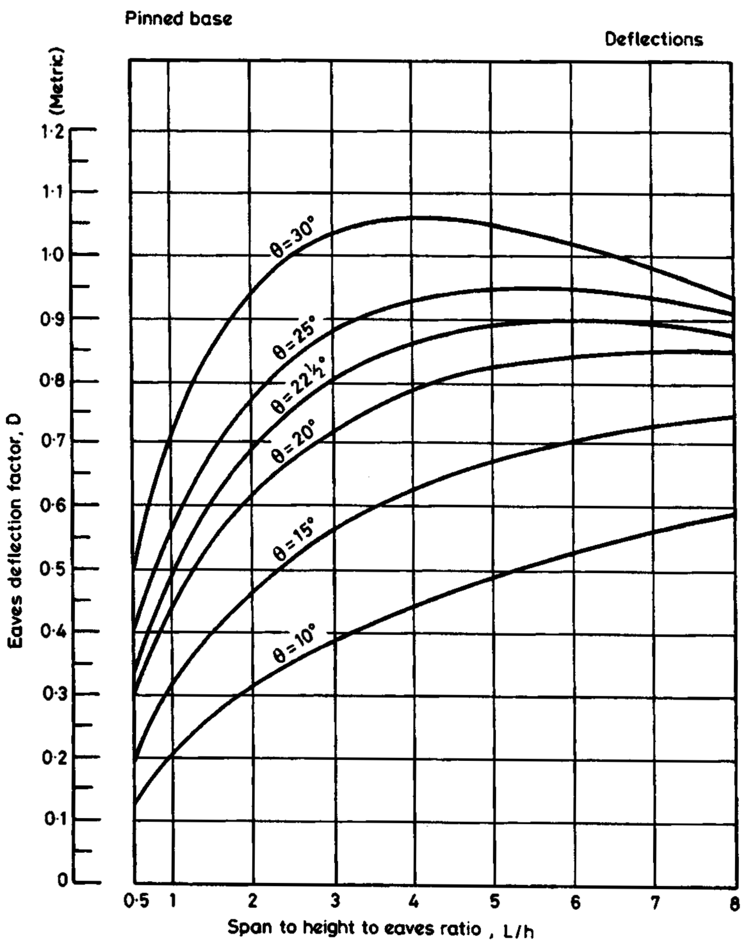
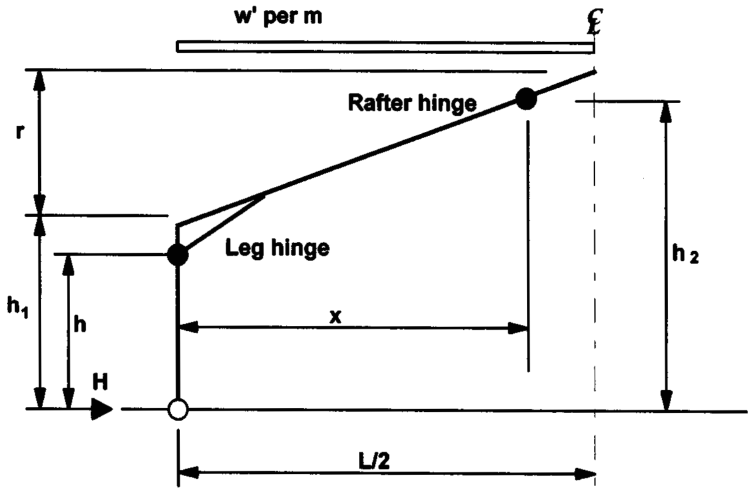



*Fig. 11.5 Deflections at unfactored loads – pinned-base frames*




**Note:**
**w' is the total load on the frame at failure.**

*Fig. 11.6 Vertically loaded pinned-base portal frame*

## 11.8 Stability checks

The following stability checks should be carried out:

- restraint of plastic hinges
- stability of rafter
- stability of haunch
- stability of stanchion.

### 11.8.1 Restraint of plastic hinges

(a) A restraint should be provided to both flanges at each plastic hinge location. If this is not practicable, the restraint should be provided within a distance of half the depth of the member along the flanges of the member from the location of the plastic hinges.

(b) The maximum distance from the hinge restraint to the next adjacent restraint should not give a value of $\bar{\lambda}_{LT}$ greater than 0.4, as this is the limit of the plateau for lateral torsional buckling.

(c) If the member is restrained on the tension flange then the maximum distance to the nearest restraint on the compression flange may be taken as $L_t$ calculated as for the stability of haunch, using the rules given in BS 5950[13] (see 11.7.3).

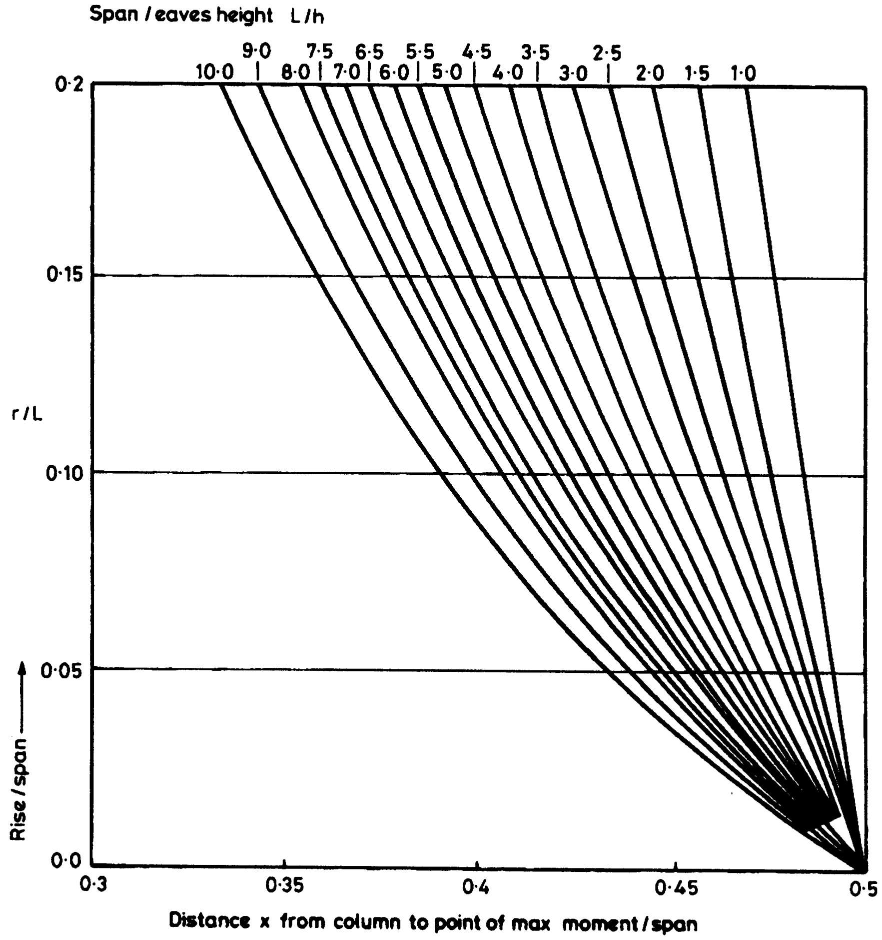



*Fig. 11.7 Distance x from column to point of maximum moment/span*

## 11.8.2 Rafter stability

The rafter should be checked to see that stability is maintained in all load cases. Unless there is wind uplift, the following checks should be made:

(a) the plastic hinge location as obtained in 11.6 near the ridge should be restrained for the case of a uniform load
(b) a purlin or other restraint is needed on the compression flange at a distance determined in the same way as mentioned above for plastic hinge restraint
(c) further restraints to the top flange are required so that the rafter satisfies the requirements of Condition II (see Chapter 4) for beams without full lateral restraint. These conditions will automatically be satisfied if the purlin restraints are at spacings less than those obtained from the slenderness $\lambda_{LT}$ as for plastic hinge restraint
(d) in areas where there is compression on the bottom flange the procedure given for haunches in 11.7.3 should be applied using constants applicable to haunch/depth of rafter = 1
(e) in cases where there is wind reversal the rafters should be checked as in 11.7.2(d).

## 11.8.3 Stability of haunch

Provided that the tension flange of the haunch is restrained, then the maximum length between restraint to the compression flange of the haunch should be limited to the $L_t$ obtained as shown below. This has, in the absence of guidance in EC3 been taken from BS 5950[13], and is subject to the following:

(a) the rafter is a UB section
(b) the haunch flange is not smaller than the rafter flange
(c) the depth of the haunch is not greater than 3 times the depth of the rafter
(d) the buckling resistance is satisfactory if it is checked as though it were a compression flange in accordance with 4.4 or 4.5 using an effective length $L_E$ equal to the spacing of the tension flange restraints.

$L_t$ may conservatively be taken as:

$$\frac{K_1 r_z x}{\sqrt{(72x^2 10^4)}} \quad \text{for grade S 275 steel,}$$

and $$\frac{K_2 r_z x}{\sqrt{(94x^2 10^4)}} \quad \text{for grade S 360 steel,}$$

where: $r_z$ is the minimum radius of gyration of the rafter section
$x$ is the torsional index of the rafter section
$K_1$ and $K_2$ have the following values:

| Depth of haunch / Depth of rafter | $K_1$ | $K_2$ |
|---|---|---|
| 1 | 620 | 645 |
| 2 | 495 | 515 |
| 3 | 445 | 465 |

If no restraint is provided to the tension flange then the limiting length to the nearest restraint on the compression flange should be calculated as for restraint of plastic hinges.

## 11.8.4 Stability of stanchion

Near the top of the stanchion a restraint should be provided at the location of the plastic hinge, together with a further restraint at a distance below the position of the hinge restraint, based on that given for the spacing of restraints at plastic hinges.

If the stanchion is restrained on the tension flange as described in 11.7.3(d) then the distance to the nearest restraint on the compression flange may be taken as calculated for the stability of the haunch.

The stanchion should then be checked in accordance with the overall buckling check in 5.5 to see if a further compression flange restraint is required.

This restraint should be provided if found necessary using the side rails. Side rails may be positioned to suit the cladding if no further compressive restraint is required.

# 12 Lattice girders and trusses with pin-base columns

## 12.1 Lattice girders and trusses

Building with trussed roof members has now largely been superseded by the use of portal frames. However there are still instances where trusses should be used. These include buildings with spans in excess of 40–50m where portal frames become uneconomical or where there are special runways, etc. that require connections at the eaves level throughout the building and sometimes purely for aesthetic reasons or because of the requirement of the surrounding buildings.

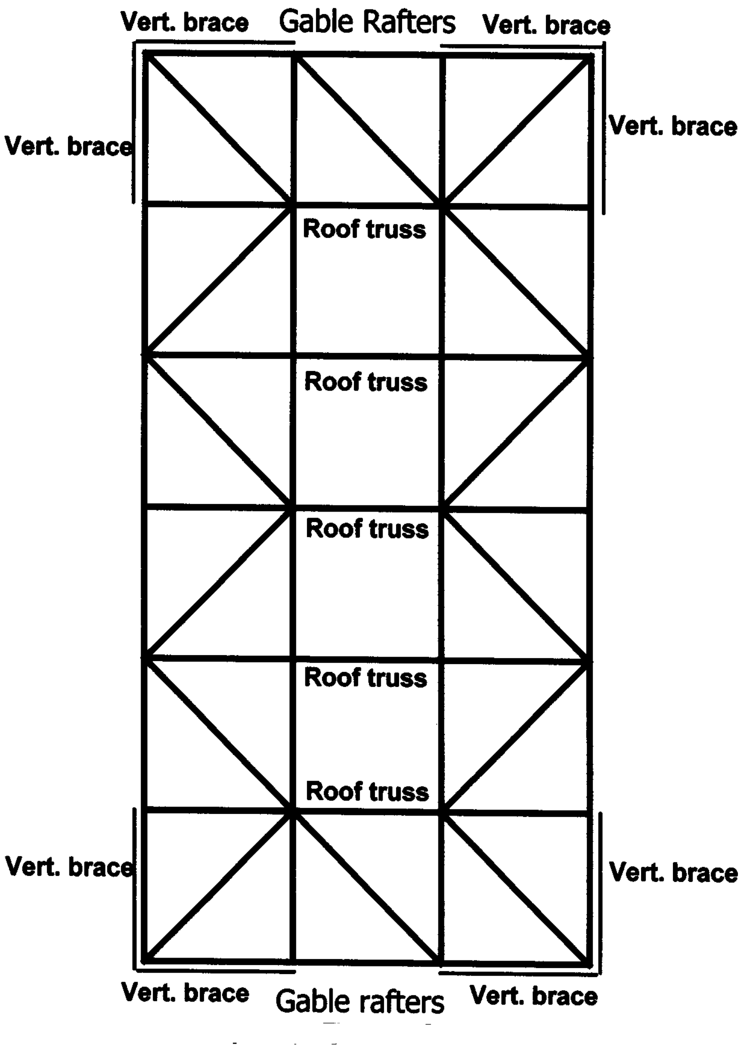



*Fig. 12.1 Typical bracing plan*

Trusses are generally split into two types.

The first type is the steep-pitched truss where the pitch is dependent on the type of roof covering. This pitch was usually leaning at 17½° for fibre-cement sheeting, 22½° for tiled roofs and 30° for traditional slate roofs. This type of truss is now however very rarely used.

The second is the type of truss that is basically a lattice girder with the top boom sloped at a pitch of 6° minimum and is used with the type of metal cladding most usually found on a structure today. It is quite common for this type of truss to be fabricated out of hollow section, but as hollow section is generally more expensive, fabrication and maintenance costs should be taken into account. Hollow sections are usually used where aesthetic considerations are paramount.

The overall depth of this type of truss should be approximately 1/12 to 1/15th of the span. If the depth is less than this, deflection can be critical.

When using trusses with pin bases the horizontal forces can be taken care of in one of two ways.

The first way is by making the building a braced box and transferring all wind forces by the use of horizontal bracing to vertical bracing in the side and end gable (see Fig. 12.1).

Note in the steep pitch trusses this bracing should be at bottom tie level, but in the lattice girders it can be at top boom level. Additional bracing may be required in the plane of the top chords of the trusses to restrain the roof trusses from lateral movement.

Alternatively, the horizontal forces can be taken as a moment at the eaves. In the cases of a steep pitch truss this involves making the eaves into a knee-braced truss (see Fig. 12.2).

There is an alternative to both these methods which imposes moments on the foundation. This Section deals with pin bases only.

Lattice girders or trusses should be designed using the following criteria:

(a) Connections between internal and chord members may be assumed pinned for calculation of axial forces in the members.
(b) Members meeting at a node should be arranged so that their centroidal axis/or line of bolt groups coincide. When this is not possible the members should be designed to resist the resulting bending moments caused by the eccentricities of connections in addition to the axial forces, similarly bending moments between node points (other than self-weight) should be taken into account.
(c) Fixity of connections and rigidity of members may be taken into account for calculating the effective lengths of members.

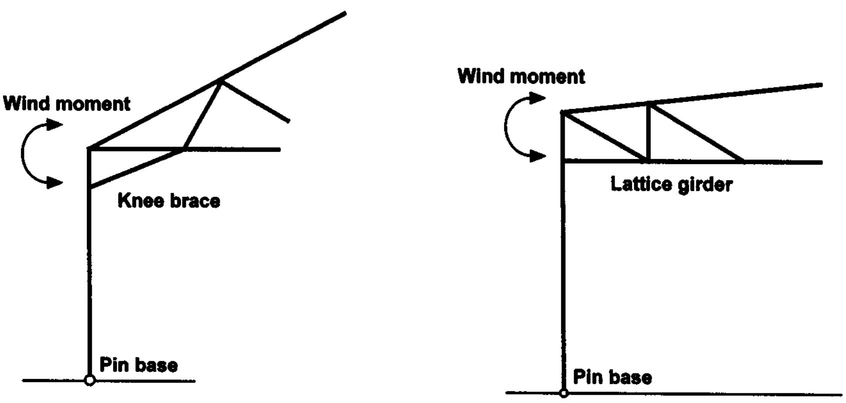



*Fig. 12.2 Resistance to wind forces*

(d) Secondary stresses in the chord members may be ignored provided that the:

- loads are applied at node points
- length/depth ratio of the chord members in the plane of the girder or truss is greater than 12
- length/depth ratio of the web members in the plane of the girder or truss is greater than 24.

(e) The length of a chord member should be taken as the distance between connections of the web members in the plane of the girder or truss and the distance between the longitudinal ties or purlins in the plane of the roof cladding.
(f) Ties to the chords should be properly connected to an adequate restraint system.
(g) Bottom members should be checked for load reversal arising from uplift.
(h) Where secondary stresses arising from local bending exist, as a rule of thumb, a bending moment can be taken as 70% of the bending moment caused by a point load acting between points of support.

The procedures to be adopted to size the members of lattice girders or trusses is:

1. Calculate the total design load on the roof from 9.2.
2. Determine the forces in the members for all relevant load combinations by graphical methods, force diagrams, method of sections, resolution of joints or a computer analysis.

## 12.2 Determination of section sizes

Section sizes can then be determined as follows.

**Compression members**
Using the methods outlined in Chapter 6 of this *Manual* or to published tables giving the compression resistance.

**Tension members**
Using the methods outlined in Chapter 6 of this *Manual* or to published tables.

**Deflection**
The deflection of the truss or lattice girder should be checked to see that serviceability with particular reference to roof drainage is not impaired. However it will usually be found that providing the aspect ratios of span/truss depth is approximate between 12 and 15 deflection will not be a problem. Preset cambers can be built in to the girders during fabrication to offset the effects of dead load deflections.

Where splices occur in chord and internal members, careful consideration should be given to the effects of possible bolt slip in the splice connections.

## 12.3 Columns for single-storey buildings braced in both directions– design procedure

(a) Calculate the factored load $N$ on the column from the roof and from the side cladding.
(b) Calculate the factored wind loads on the roof and sides.
(c) Calculate the vertical and horizontal components of the roof wind for the bracing design.
(d) Calculate the factored side wall wind loads $W_{w1}$ and $W_{w2}$ on external columns.
(e) Calculate the maximum factored moments arising from wind on the columns from the

greater of $W_{w1}$ or $W_{w2}$ × H/8 where $H$ = height of column from base to eaves.

(f) Calculate the factors nominal moments at the cap arising from any eccentricity of the truss (Note: If the truss is supported on a cap plate the eccentricity may be ignored.) In all other cases it should be taken as half the column depth + 100mm or to the centre of bearing whichever is greater.

(g) Select a section and check column design as shown in Chapter 5 of this *Manual*.

## 12.4 Columns for single-storey buildings braced in one direction only in the side walls and/or the valleys – design procedure

(a) Calculate the factored load $N$ on the column from the roof and side cladding.

(b) Calculate the factored wind loading on the roof and sides.

(c) Calculate the horizontal component of the roof wind for the design of the bracing.

(d) Calculate the total factored side wall wind loading $W_{w1}$ or $W_{w2}$ on the external columns.

(e) Calculate the maximum factored moments on the columns arising from wind and dead imposed loading by elastic analysis assuming the columns and trusses act as frames in the unbraced direction.

(f) Select a section and choose the design of the column as shown in Chapter 5 of this *Manual*.

# 13 Single-storey buildings – other members, etc.

## 13.1 Gable posts

Calculate the factored axial loads and factored wind moments on these posts. Select a section and check design as in 5.5.

## 13.2 Bracing and tie members

Assess the appropriate factored wind load on the bracing and tying members in each braced bay, and then design the members in accordance with the methods described in Chapter 6.

## 13.3 Other members

It may be necessary to provide framing for door, window and services opening in the side-walls of the single-storey building. These members should be sized in accordance with the methods recommended above for gable posts or bracing members, depending on the loading in or location of the member.

## 13.4 The next step

Preliminary general arrangement drawings should be prepared when the design of the structural members has been completed, and sent to other members of the design team for comments. It is important to establish the general form and type of connections assumed in the design of the members and to check that they are practicable. Reference should be made to 8.2 and Chapter 15 as the items described therein also apply to single-storey buildings The details to be shown, checking of information, preparation of a list of design data, the finalisation of design, etc. should be carried as described in Chapter 8 for multistorey buildings.

# 14 Connections

## 14.1 Bolted connections

The connections for frames analysed as pin jointed and single-storey portal frames are covered in this *Manual*.

EC3 defines seven categories of bolted connections:

| | |
|---|---|
| Simple framing | – Pin joint analysis |
| Continuous framing | – Elastic analysis |
| | – Rigid-plastic analysis |
| | – Elastic-plastic analysis |
| Semi-continuous framing | – Elastic analysis |
| | – Rigid-plastic analysis |
| | – Elastic-plastic analysis |

## 14.2 Welded connections

It is common practice in Europe, reflected in EC3, to define weld sizes by the throat thickness and to use the total length of the weld. This is contrary to practice in the UK where the weld size is taken as the leg length and an allowance is made for the starting and stopping of the weld of 4 times the weld size. Designers and fabricators must ensure that they take cognisance of these differences and that the documentation for a project makes the assumptions by the designer clear.

In the case of fillet welds made by submerged arc automatic process, the design throat thickness may be increased by the smaller of 20% or 2mm without preliminary trials.

## 14.3 Connection design

This *Manual* covers connections in pin-jointed and continuous frames. Connections for frames where the plastic hinge can form in the joint and semi-continuous frames are not covered.

The connection should be designed on a realistic, and consistent, assumption of the distribution of internal forces in the connection, which are in equilibrium with the externally applied loads. Each element in the connection must have sufficient resistance and deformation capacity.

The following points should be noted:

(a) The centroidal axes of the connected members should meet at a point; otherwise the effect of eccentricity of the connection should be taken into account in the design of the member.
(b) Bolts and welds in splice connections should be designed to carry all forces, except where provision is made for direct bearing where compressive forces are to be transferred by direct contact.
(c) EC3 provides for the use of ‘ordinary bolts’ or ‘high strength bolts’ in grades from 4.6 up to 10.9. Generally grade 8.8 M20 bolts should be used for connections in members which will accommodate this size, and M16 used for smaller members. Heavily loaded connections may require M24 or M30 bolts, or the use of 10.9 grade bolts. As far as possible only one size and grade should be used on a project (see also section 9.4).
(d) The local ability of the connected members to transfer the applied forces should be checked and stiffeners provided where necessary. Where the connection is not at the

end of a member the design may have to take account of the combination of local and primary stresses in the member occuring at the connection.

(e) Bolted shear connections can be one of the following:

**Category A:** Bearing type with appropriate shear and bearing resistance.
**Category B:** Pre-loaded bolts for slip-resistance at serviceability limit state only. Can be used for steelwork in building structures subjected to vibration or impact, and when slip is unacceptable.
**Category C:** Pre-loaded bolts for slip-resistance at ultimate limit state. In connections where there may be some vibration, but slip is permitted, an ordinary bolt assembly complete with a locking device may be adopted.

(f) Bolted tension connections can be one of the following:

**Category D:** Ordinary bolts, without preloading, may be used to resist wind forces and where there is little variation in other forces.
**Category E:** Preloaded high strength bolts should be used where there is vibration, or variable loading which would require fatigue consideration.

(g) The corrosive protection for the bolts should be compatible with the system used for the main frame. When applicable fastener threads should be checked after treatment to so that they allow free running and satisfactory tightening performance.
(h) The bolt assembly should include a washer under the part rotated during tightening to avoid damage to the painted surface.
(i) Where dissimilar metals are likely to be in contact in a moist environment, suitable isolators such as neoprene washers and sleeves should be incorporated to prevent bimetallic corrosion.
(j) Bolts should always be checked on site.

## 14.4 Bolts

### 14.4.1 Spacing and edge distances

A summary of the requirements is given in Table 14.1.

**Table 14.1 Bolt spacing and end distance**

| Requirement | Distance |
|---|---|
| **Minimum distances based on hole diameter** | |
| End and edge distance | $2.0d_0$ ($d_0$ = hole diameter) |
| Minimum end distance ($e_1$) (with reduced bearing capacity) | $1.2d_0$ ($d_0$ = hole diameter) |
| Minimum edge distance – right angles to load ($e_2$) | $1.5d_0$ |
| **Maximum end and edge distances** | |
| – in any environment | > $12t$ or 150mm |
| – when exposed to weather or corrosion | 40mm + $4t$ |
| **Recommended spacing – direction of load ($p_1$)** | $3.5d_0$ |
| Minimum spacing – direction of load ($p_1$) | $2.2d_0$ |
| Minimum spacing – rows of fasteners ($p_2$) | $3.0d_0$ |
| **Maximum spacing** | |
| – in compression elements and outer row tension elements | $14t$ or 200mm |
| – inner row tension elements | $28t$ or 400mm |

Note: $t$ is the minimum thickness of the connected parts.

## 14.4.2 Strength checks

The strengths of ordinary bolts should be checked using the formulae and other criteria in Table 14.2.

**Table 14.2 Strength check for bolts**

| Design resistance for bolts | | Formula |
|---|---|---|
| Shear resistance per shear plane | grades 4.6 & 8.8 | $F_{v.Rd} = \dfrac{0.6F_{ub}A_s}{\gamma_{Mb}}$ |
| | grade 10.9 | $F_{v.Rd} = \dfrac{0.5f_{ub}A_s}{\gamma_{Mb}}$ |
| N.B. when a shear connection is longer than 15$d$ the shear capacity is multiplied by the factor $\beta_{LF}$ | | $\beta_{LF} = 1 - \dfrac{L_1 - 15d}{200d}$ |
| Bearing resistance at recommended spacing: ($e_1 = 2.0d$ and $p_1 = 3.5d$ – see Table 14.1) | | $F_{b.Rd} = 1.0f_u dt$ |
| Tension resistance | | $F_{t.Rd} = \dfrac{0.9f_{ub}A_s}{\gamma_{Mb}}$ |
| Combined shear and tension resistance | | $\dfrac{F_{v,Sd}}{F} + \dfrac{F_{t,Sd}}{1.4F_{t.Rd}} \leq 1.0$ |
| Reduction factor $\beta_p$ when using bolts through packings whose thickness is greater than $d$/3 | | $\beta_p = \dfrac{9d}{8d + 3t_p}$ but $\beta \leq 1.0$ |

$f_{ub}$ is the strength of the bolt
$f_u$ is the strength of the element
$F_{v,Sd}$ is the actual shear force on the bolt
$F_{t,Sd}$ is the actual tensile force on the bolt
$A_s$ is the tensile stress area of the bolt
$d$ is the diameter of the bolt
$\gamma_{Mb}$ is the partial factor = 1.35

**Table 14.3 Nominal yield strength and ultimate strengths for bolts**
(used as characteristic values in calculations)

| Bolt grade | 4.6 | 8.8 | 10.9 |
|---|---|---|---|
| Yield $f_y$, N/mm$^2$ | 240 | 640 | 900 |
| Ultimate $f_y$ N/mm$^2$ | 400 | 800 | 1000 |

**Table 14.4 Nominal yield strength and ultimate strengths for steel**

| Steel grade | $t \leq$ 40mm | | 40mm < $t \leq$ 100mm | |
|---|---|---|---|---|
| | yield $f_y$ | ultimate $f_u$ | yield $f_y$ | ultimate $f_u$ |
| | N/mm$^2$ | N/mm$^2$ | N/mm$^2$ | N/mm$^2$ |
| S 275 | 275 | 430 | 255 | 410 |
| S 355 | 355 | 510 | 335 | 490 |

$t$ is the nominal thickness of the element

**Table 14.5 Capacities of grade 8.8 ordinary bolts: design grade S 275 material**

| Bolt size | Tensile stress area | Tensile capacity at 533N/mm$^2$ | Shear capacity at 355 N/mm$^2$ threads in shear plane | | Bearing capacity at 430N/mm$^2$ foredge distance $e$ = 2.0 and spacing $p$ = 3.5$d$ | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Thickness – mm – plate passed through | | | | | | | |
| | mm$^2$ | kN | Single | Double | 6 | 8 | 10 | 12 | 15 | 20 | 25 | 30 |
| M16 | 157 | 83.7 | 55.7 | 111 | 41.3 | 55.1 | 68.8 | 82.6 | 103 | 138 | | |
| M20 | 245 | 131 | 87.0 | 174 | 51.6 | 68.8 | 86.0 | 103 | 129 | 172 | | |
| M24 | 353 | 188 | 125 | 251 | 61.9 | 82.5 | 103 | 124 | 155 | 206 | 258 | |
| M30 | 561 | 299 | 199 | 398 | 77.4 | 103 | 129 | 155 | 194 | 258 | 323 | 387 |

**Table 14.6 Capacities of grade 8.8 ordinary bolts: design grade S 355 material**

| Bolt size | Tensile stress area | Tensile capacity at 533N/mm$^2$ | Shear capacity at 355 N/mm$^2$ threads in shear plane | | Bearing capacity at 510 N/mm$^2$ for edge distance $e$ = 2.0 and spacing $p$ = 3.5$d$ | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Thickness – mm – plate passed through | | | | | | |
| | mm$^2$ | kN | Single | Double | 6 | 8 | 10 | 12 | 15 | 20 | 25 |
| M16 | 157 | 83.7 | 55.7 | 111 | 49.0 | 65.3 | 61.6 | 97.9 | 122 | | |
| M20 | 245 | 131 | 87.0 | 174 | 61.2 | 81.6 | 102 | 122 | 153 | 204 | |
| M24 | 353 | 188 | 125 | 251 | 73.4 | 97.9 | 122 | 147 | 184 | 245 | |
| M30 | 561 | 299 | 199 | 398 | 91.8 | 122 | 153 | 184 | 230 | 306 | 383 |

## 14.4.3 High strength bolts in slip-resistance connections

The formula for slip-resistance (HSFG) connections are given in Table 14.8. Bolts in accordance with BS 4395[30] (ISO 7411[31] and 7412[32]) may be adopted. Note that bolts used as shear/bearing connectors may provide higher load carrying capacity.

**Table 14.7 Capacities of grade 10.9 ordinary bolts: design grade S 355 material with increased spacing of bolts**

| Bolt size | Tensile stress area | Tensile capacity at 667N/mm² | Shear capacity at at 370N/mm² threads in shear plane | | Bearing capacity at 1020N/mm² for edge distance $e$ = 2.0 and spacing $p$ = 3.5$d$ | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | Thickness – mm – plate passed through | | | | |
| | mm² | kN | Single | Double | 6 | 8 | 10 | 12 | 15 |
| M16 | 157 | 105 | 58.1 | 1161 | 98.0 | 131 | | | |
| M20 | 245 | 163 | 90.7 | 181 | 122 | 163 | 204 | | |
| M24 | 353 | 235 | 131 | 161 | 147 | 196 | 245 | 294 | |
| M30 | 561 | 374 | 208 | 416 | 184 | 245 | 306 | 367 | 459 |

**Table 14.8 Strength check for preloaded bolts**

| Design resistance | Formula |
|---|---|
| Slip resistance | $F_{s,Rd} = \dfrac{k_2 \eta \mu}{\gamma_{Mb}} F_{p,Cd}$ |

$F_{p,Cd}$ is the design preloading force
$f_{ub}$ is the strength of the bolt
(for BS 4395 Gen. grade bolts $f_{ub}$ = 827N/mm² ≤ M24, 727N/mm² > M24
$A_s$ is the tensile stress area of bolt
$k_2$ is a factor = 1.0 for nominal clearance holes:

= 0.85 for oversize holes (4mm for M20, 6mm for M24 6r with a hardened washer
= 0.7 for long slot holes (4mm for M20, 6mm for M24 6r with a cover plate)

$\eta$ is the number of friction interfaces
$\mu$ is the slip factor:

= 0.5 for blast cleaned surfaces and some metal sprayed surfaces (class A)
= 0.4 for blast cleaned and painted with an alkali zinc silicate coating of 50–80μm (class B)
= 0.3 for surfaces wire brushed or flame cleaned (class C)
= 0.2 for surfaces not treated (class D)

The recommended preparation for surfaces is blast cleaning.

$\gamma_{Mb}$ is a partial safety factor with the following values:

1.20 for the serviceability limit state, with nominal clearance holes or a slot perpendicular to load transfer
1.35 for the ultimate limit state, with nominal clearance holes or a slot perpendicular to load transfer
1.35 for the ultimate limit state, with slot perpendicular to load transfer and Category C slip resistant

**Table 14.9 Capacities for high strength slip resistant bolts**

Capacities in kN for category B Preloaded Bolts in nominal clearance holes
General grade bolts to BS 4395, Partial safety factor, $\gamma_{Ms} = 1.20$

| Bolt size | Tensile stress area mm$^2$ | Slip resistance | | | |
|---|---|---|---|---|---|
| | | Class A | | Class B | |
| | | Single shear | Double shear | Single shear | Double shear |
| M16 | 157 | 37.9 | 75.8 | 30.3 | 60.6 |
| M20 | 245 | 59.1 | 118 | 47.3 | 94.6 |
| M24 | 353 | 85.1 | 170 | 68.1 | 136 |
| M30 | 561 | 119 | 238 | 95.2 | 190 |

## 14.5 Welds

Welds should be tested using non-destructive methods. The scope of inspection and weld quality should not be less than that shown in Tables 1 and 2 of the *National Structural Steelwork Specification for Building Construction*.

### 14.5.1 Fillet welds

Fillet welds up to corners should not terminate at the corner but should be returned round the corner for twice the leg length.

Fillet welds may be designed as continuous or intermittent, but intermittent welds should not be used in corrosive conditions or when subject to fatigue.

EC3 permits the design to be made as fillet welds, when the fusion faces form an angle between 60° and 12°.

A single fillet weld is not permitted in conditions which will produce tension at the root of the weld.

The effective length of a fillet weld is the overall length including end returns, providing the weld is full size throughout this length.

The throat thickness should not be less than 4mm.

In the case of fillet welds made by submerged arc automatic process, the design throat thickness may be increased by the smaller of 20% or 2mm without preliminary trials.

The resultant (vector sum) of all the connection forces on a weld, acting on a unit length of weld, must be less than the shear strength per unit weld given in Table 14.10.

An alternative method which may give a higher value is given in Annex 'M'of EC3.

### 14.5.2 Butt welds

The weld may be a full penetration or a partial penetration butt weld.

A full penetration weld has a design strength equal to that of the weaker part joined, provided it is made with a suitable electrode.

A partial penetration butt weld has a design strength calculated for a deep penetration fillet weld. (See 14.5.1 above and Table 14.10).

Where the weld penetration is of the U-, V-, J- or bevel type, the throat thickness should be taken as the nominal depth of preparation minus 2mm, unless a larger value is proved by preliminary trials.

**Table 14.10 Strength check for fillet welds**

| Design resistance | Formula |
|---|---|
| Shear strength per unit length | $F_{w.Rd} = \dfrac{f_u a / \sqrt{3}}{\beta_w \gamma_{Mw}}$ |

$f_u$ is the nominal ultimate tensile strength of the weaker part joined.
$\beta_w$ is a correlation factor as follows:

| Steel grade | S 275 | S 355 |
|---|---|---|
| $f_u$ N/mm$^2$ | 490 | 510 |
| $\beta_w$ | 0.85 | 0.9 |

$a$ is the throat thickness of the weld
$\gamma_{Mw}$ is the partial safety factor 1.35

N.B. There is a reduction factor to take into account in the design resistance of a fillet weld, except where the stress distribution along the weld corresponds with that in the parent metal. (e.g. the flange to web weld in a plate girder).

In lap joints longer than 150$a$ the reduction factor is:

$$\beta_{Lw,1} = 1.2 - 0.2L_1/(150a)$$

$L_1$ is the overall length of the lap in the direction of the force transfer. Fillet welds connecting transverse stiffeners in plated members which are longer than 1.7m, the reduction factor is $\beta_{Lw,2}$

$$\beta_{Lw,2} = 1.1 - L_w/17$$

$L_w$ is the length of the weld in metres.

**Table 14.11 Fillet weld capacities**

| Weld strengths for S 275 | | Weld strengths for S 355 | |
|---|---|---|---|
| Throat thickness mm | Capacity at 215N/mm$^2$ kN/mm | Throat thickness mm | Capacity at 242N/mm$^2$ kN/mm |
| 4 | 0.86 | 4 | 0.97 |
| 6 | 1.29 | 6 | 1.46 |
| 8 | 1.73 | 8 | 1.94 |
| 10 | 2.15 | 10 | 2.43 |

# 15 Typical connections

## 15.1 Introduction

This Chapter describes typical connections for braced multistorey buildings of simple construction, and for single-storey buildings, including portal frames. A procedure is given for design of the connection.

Most of the connections shown here have limited rotational stiffness. With such connections, it may be necessary to arrange construction so that floor restraints are in position in one area before proceeding further, or to provide temporary bracing, to ensure stability of the frame throughout its erection.

For more comprehensive methods of design, worked examples and standardised details, reference should be made to *Joints in simple construction* (SCI/BCSA[33,34]) and *Moment connections* (SCI/BCSA[35]).

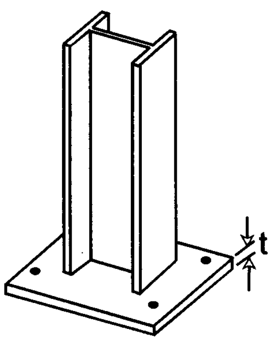
The design methods used in these publication differ in some respects from the Application rules given in the Eurocode, however it is permissible to use them since they give equivalent resistance, serviceability and durability in accordance with EC3 clause 1.2 (5).

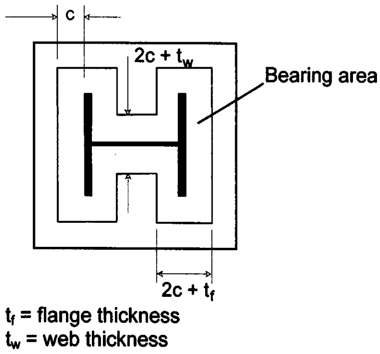
## 15.2 Column bases

### 15.2.1 General

Column bases should be of sufficient size, stiffness and strength to transmit safely the forces in the columns to the foundations. Linear pressure distribution may be assumed in the calculation of contact pressures.

### 15.2.2 Design of base plates




**Procedure**

(i) Choose base plate size, length, breadth, thickness

(ii) Choose bearing strength $f_j$ – see Table 15.1

(iii) Calculate $c$ from:

$$c = t\left[\frac{f_y}{3 f_j \gamma_{M0}}\right]$$

where: $f_y$ is the yield strength of the base plate
$f_j$ is the bearing strength
$\gamma_{M0}$ is the partial factor for steel (1.05)

**Table 15.1 Concrete design values and bearing strengths**

| | Concrete grades | | | | |
|---|---|---|---|---|---|
| $f_{ck}/f_{ck,cube}$ | C20/25 | C25/30 | C30/37 | C35/45 | C40/50 |
| $f_{cd}$ (N/mm$^2$) | 13 | 16.7 | 20 | 23 | 33 |
| $f_j$ (N/mm$^2$) | 8.7 | 11.2 | 13.4 | 15.4 | 22.1 |

where:
- $f_{ck}$ is the concrete characteristic cylinder strength
- $f_{ck,cube}$ is the concrete characteristic cube strength
- $f_{cd}$ is the design value of the concrete compressive strength $= f_{ck} / \gamma_c$ ($\gamma_c = 15$)
- $f_j$ is the maximum bearing strength of the joint (grout and concrete) $= \beta_j k_j f_{sd}$
- $\beta_j$ is the joint coefficient taken as 0.67 for condition when characteristic strength of grout $\geq$ 0.2 of characteristic strength of concrete and grout thickness $\leq$ 0.2 of smallest width of base plate
- $k_j$ is the concentration factor taken very conservatively as 1.0. A significant gain may be obtained by using the equation in EC3.

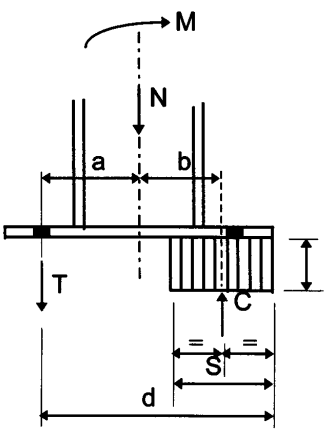
(iv) Check that $c$ is within chosen base size
(v) Calculate bearing area $A_b$ and adjust for $c$ as necessary.
(vi) Calculate capacity of base = $A_b f_j$
(vii) Make provision for shear if adequate friction is not available based on coefficient of friction steel/concrete of 0.2.

Design welds to accommodate any shear or tension.

## 15.2.3 Design of bases (moment + axial load)

Using a plastic design method, a rectangular stress block in compression may assumed. A worked example of the method can be found in *Joints in steel construction – moment connections*, by SCI/BCSA[35].

**Procedure**
(i) Choose base plate size and HD bolt size and bolt positions
(ii) Choose concrete/grout bearing stress $f_j$ – see Table 15.1



(iii) Determine bolt tension and extent of stress block using equilibrium equations:
(iv) $N = C - T$
(v) $M = Ta + Cb$
(vi) Determine plate thickness, taking account of bending of the base in both compressive and tensile loads
(vii) Check bolt size and determine anchorage requirements
(viii) Check shear transfer to concrete
(ix) Design flange welds for tension and compression and web weld for shear, use the same size weld for the whole connection based on the largest required.

## 15.3 Beam-to-column and beam-to-beam connections in simple construction

Procedures are given below for four recognised types of connection used in simple construction. These are:

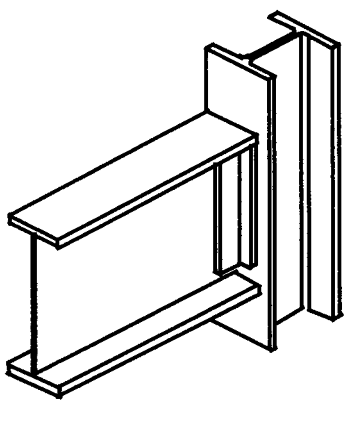
- Web cleats
- Flexible end plates
- Fin plates

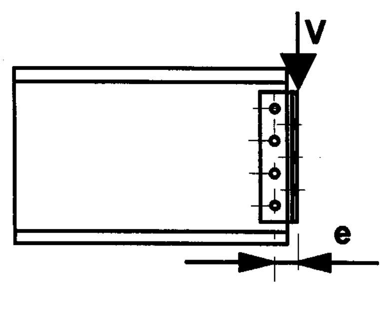
Fin plates should not be used to connect universal beams which are greater than 610mm deep, until research currently taking place, has proved that the connection allows sufficient rotational movement to justify the simple method of design.

### 15.3.1 Beam-to-column web cleats

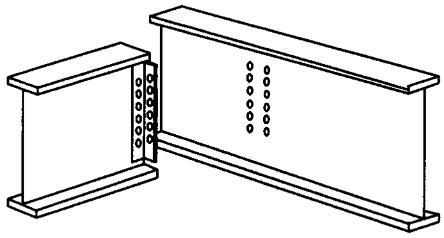
**Procedure:**
(i) Choose size of pair of cleats not less than 8mm thick and with sufficient length to take the required number of bolts. If the beam is to provide moment restraint to the column then the angles should be at least 60% of the depth of the beam in length. Large sections, over 500mm deep may require thicker angle cleats.
(ii) Calculate the size/number of bolts required in beam web to resist both shear $V$ and moment $Ve$. See Table 14.2 for resistances.
(iii) Check shear and bearing value of cleats (both legs).
(iv) Check shear and bearing of beam web.

Check for block shear failure as given in EC3 clause 6.5.2.2. A simpler procedure giving identical results will be found in *Joints in simple construction* published by BCSA and SCI[33,34]. It will be found in many cases, as determined form experience, that with the bolt arrangements adopted in the UK this mode of failure will rarely govern the design.





## 15.3.2 Beam-to-beam web cleats



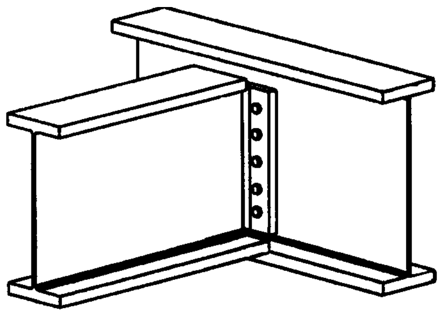
**Procedure:**
All as beam-to-column connection above, plus:

(v) Check local shear and bearing of supporting beam web.
(vi) Check web bending at the junction of a double notched beam.

## 15.3.3 Beam-to-column flexible end plates

**Procedure:**
(i) Choose plate not less than 8mm thick using the same guidelines as given for the selection of angles in 15.3.1.
(ii) Calculate the size/number of bolts required in plate to resist shear $V$
(iii) See Table 14.2 for resistances
(iv) Check bearing value of plate and bolts.
(v) Check shear and bearing of beam web
(vi) Check shear capacity of beam web at the end plate
(vii) Choose fillet weld throat size to suit double length of weld

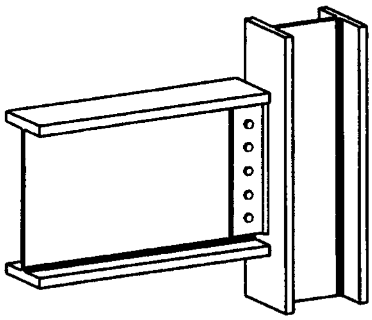
## 15.3.4 Beam-to-beam flexible end plates

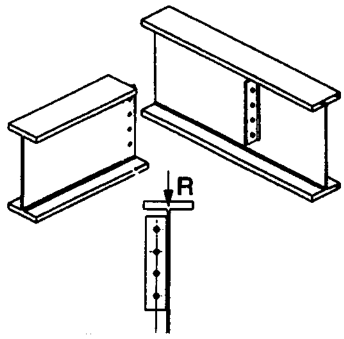


**Procedure:**
(i) all as beam-to-column connection in 15.3.1 and 15.3.3.
(ii) plus checks similar to those listed in 15.3.2.

## 15.3.5 Fin plates

**Procedure:**
(i) Choose fin plate size so that thickness is ≥ 0.5 bolt diameter and length ≥ 0.6 web depth.
(ii) Calculate the size/number of bolts required in fin plate to resist both shear $V$ and moment $Ve$.






(iii) Use bolts of grades 8.8 or 10.9.
(iv) See Table 14.2 for resistances.
(v) Check shear and bearing value of plate and bolts.
(vi) Check shear capacity of beam web taking account of plain shear and block shear.
(vii) If beam is notched – check local stability of notch.
(viii) Make the fillet weld throat size each side of plate 0.55 × plate thickness.
(ix) Check local shear capacity of supporting beam or column web for beams supported on one or both sides.
(x) Check block shear failure as described in 15.3.1.

Note that a fin plate connection should only be used to connect beams of depth greater than 610mm when:

(i) the beam span/depth ratio is less than 20
(ii) the gap between the beam end and the face of the joint is less than 20mm.

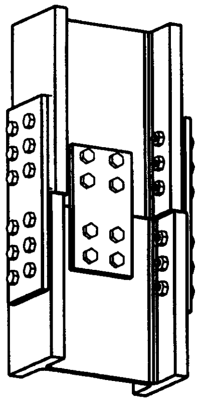
## 15.4 Column-to-column bearing splices

Column splices should be located adjacent to and above floor levels and designed to meet the following requirements:

(i) They should be designed to hold the connected members in place.
(ii) Wherever practicable the members should be arranged so that the centroidal axis of any splice material coincides with the centroidal axis of the member. If eccentricity is present, then the resulting forces must be taken into account.
(iii) They should provide continuity of stiffness about both axes, and should resist any tension.
(iv) They should provide the resistance to tensile forces to comply with the accidental action requirements of Chapter 7.

### 15.4.1 Column bearing splices

**Procedure**
(i) Flange cover plates should be of a length at least twice the upper column flange width.
(ii) The thickness of flange cover plates should be the greater of half the thickness of the upper column flange or 10mm.
(iii) When the upper and lower lengths are the same column serial size, nominal web cover plates may be used.
(iv) When the upper and lower lengths are of different sizes, then web cleats and a division plate should be used to give a load dispersal of 45°.



(v) Use M24 grade 8.8 bolts for splices in columns 305 series and above.

(vi) Column ends to be prepared so that any gaps between the bearing surfaces are not greater than:

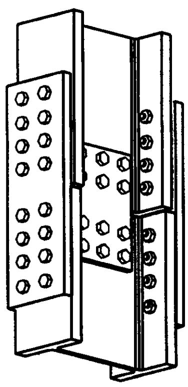
$$\frac{h}{1000} + 1\text{mm}$$

where $h$ is the depth of the column section in mm.

(vii) If the recommended sizes are adopted, no design calculations are necessary, except when moments are present, or in the case of tensile forces described in 15.4 (iv) previously.

## 15.4.2 Column-to-column non-bearing splices

Axial loads are shared between the web and flanges, in proportion to their areas, and bending moments are deemed to be carried by the flanges.



**Procedure:**

(i) Choose the splice plate sizes, number and type of bolts. (Use slip resistance bolts in braced bays).

(ii) Calculate forces in bolts arising from axial loads and moments.

(iii) Check bolt strengths in single shear.

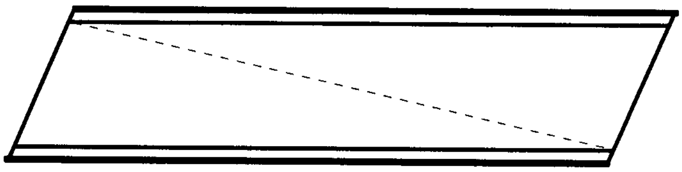
(iv) Check the bearing stress in the flanges and splice covers.

(v) Check the tensile capacity and shear stress across in the plate across the net area after deducting the hole areas.

# 15.5 Portal frame connections

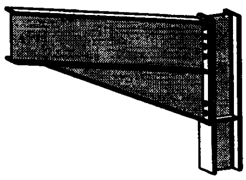
Where portal frames are analysed by the plastic method the following should be adopted:

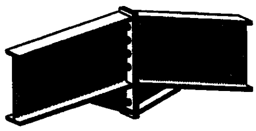
(i) a plastic hinge should not be allowed to form in the joint
(ii) a long haunch should be treated as a frame member
(iii) the design moment of resistance of the connection should be at least 1.2 × the plastic moment of the smaller member at the connection, which in a haunched portal frame will normally be the leg.
(iv) The depth of haunch chosen may conveniently be arranged so that two haunches may cut from a single length of the same section as that of the rafter thus:

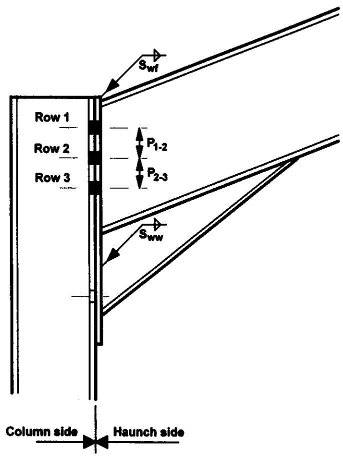


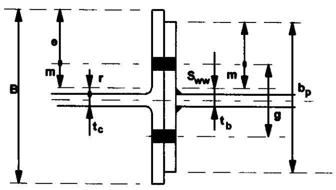
(v) The design method shown here is for bolt rows each being composed of two of bolts only.
(vi) Only the bolts in the top half of the connection should be considered as tension fasteners.

## 15.5.1 Geometry – eaves haunch








**For column flange**

$$m = \frac{g}{2} - \frac{t_c}{2} - 0.8r$$

$$e = \frac{B}{2} - \frac{g}{2}$$

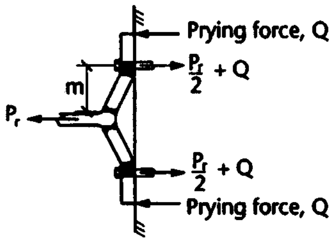
**For end plate**

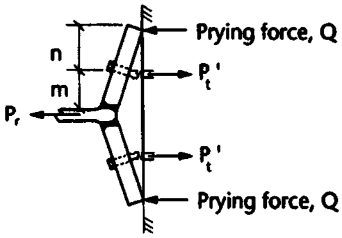
$$m = \frac{g}{2} - \frac{t_b}{2} - 0.8S_w$$

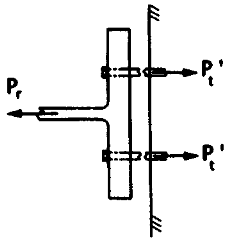
$$e = \frac{b_p}{2} - \frac{g}{2}$$

## 15.5.2 Resistance of column flange in bending

Three modes of failure have to be considered as follows:




**Mode 1**
**Column flange yielding**

**Mode 2**
**Bolt failure with flange yielding**

**Mode3**
**Bolt failure**

$$p_r = \frac{4M_p}{m}$$

$$p_r = \frac{4M_p + n\left(\sum p'_t\right)}{m + n}$$

$$p_r = \sum p'_t$$

where: $M_p$ = plastic moment capacity of an equivalent T-stub

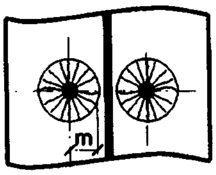
$= \frac{L_{eff} t^2 p_y}{4}$

$L_{eff}$ = effective length of yield line in equivalent T-stub (15.4.3)
$t$ = column flange thickness
$p_y$ = design strength of column flange
$p_f$ = potential resistance of the bolt row or bolt group
$p_r$ = bolt tension resistance

## 15.5.3 Effective length of equivalent T-stub

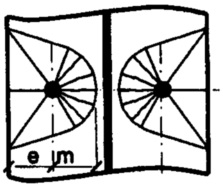
The following patterns should be considered for cases where there are a pair of bolts with no stiffeners, assuming the holes are remote from the end of the member.

**Circular yielding pattern**



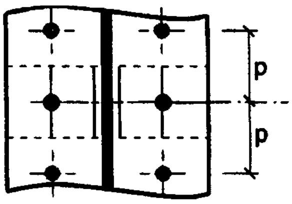

$L_{eff} = 2\pi m$

**Side yielding pattern**




$L_{eff} = 4m + 1.25e$

**Intermediate row of a group**

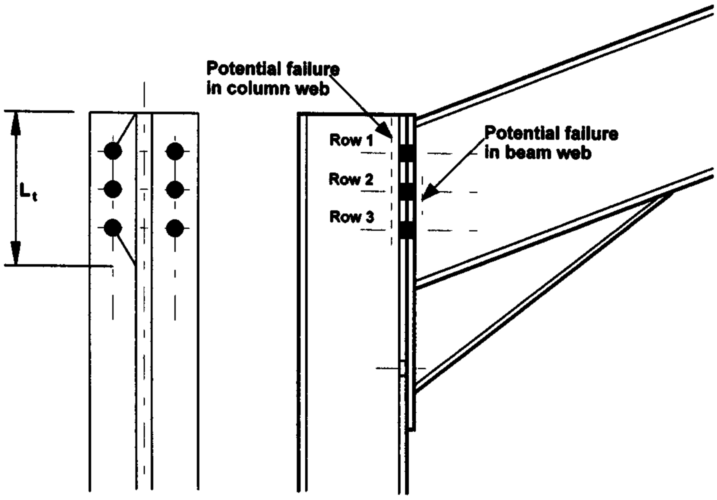



$L_{eff} = p$

## 15.5.4 Web tension in beam or column

Consider the web tension conditions on the columns and the welds between the beam and end plate as follows:

for the column the tensile strength $P_t = L_t\, t_w\, p_y$
for the beam the force on the weld $P_t = L_t\, R_w$ on each side of the beam




where: $P_t$ is the potential force for the group of bolts being considered

$L_t$ is the effective length of the web assuming a maximum spread of 60° from the centre of the bolts to the centre of the web. For a weld the spread is taken to the weld line

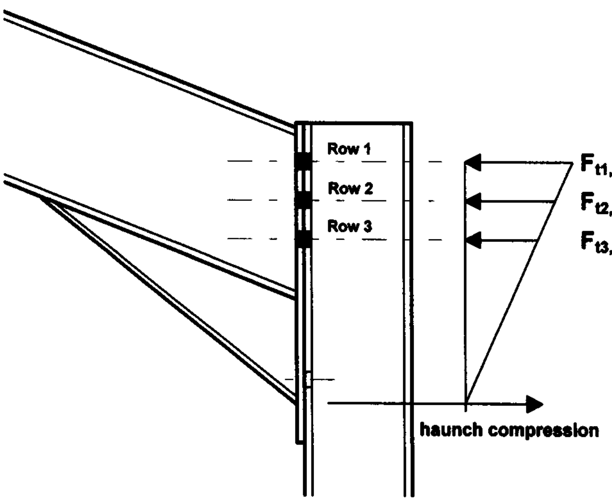
$t_w$ is the thickness of the column

$p_y$ is the design strength of the column

$R_w$ is the resistance of the weld

### 15.5.5 Simplified distribution of bolt row forces

Consider the minimum bolt row forces (flange yielding, bolt failure or web tension) on the column side using the failure mechanisms given in the earlier text. This gives the tensile resistance $F_{t1,Sd}$ of the top bolt in the group, the forces on the lower bolts may then be determined from linear distribution shown in the figure below. If the distance between the top two rows of bolts is less than 10% of the total depth of the connection then it may be assumed that both of these rows of bolts are capable of carrying the full tensile resistance of the bolts.



## 15.5.6 Final resistance moment and end plate thickness

Calculate the moment resistance of the bolts from considering rotation about centre of haunch compression. The minimum resistance of the top row, as given in 15.5.2 may be used to determine the maximum force in the bolts. The resistance of the other rows should be determined as given in 15.5.5 & 15.5.4 but should not extend beyond the triangular line indicated in the diagram with 15.5.5. If a moment value results which is less than that required, set new configuration for connection. If value is greater than that required, by a significant amount, then consideration may be given to reducing the number of bolts provided in the connection.

Since bolt row forces have been established from considering the column side of the connection and bending of the column flange (15.5.2), the beam side will be satisfactory if the end plate size and thickness is made equal or greater than the column flange thickness.

A more thorough design and analysis adopting other configurations for the end plate, and methods of stiffening the column flange and the end plate may be found in *Joints in steel construction – moment connections* by SCI/BCSA[35].

## 15.5.7 Compression checks

The compression force, from the bottom of the haunch, may be taken as the sum of the bolt row forces calculated as above. The resulting reactions on the column should be greater than the resistance of the column checked, as described in Chapter 4 of this *Manual*, for the following conditions:

- Crushing resistance of the web,
- Crippling resistance of the web,
- Buckling resistance of the web.

## 15.5.8 Column web panel shear

The column panel must be capable of resisting the shear forces arising from the tensile forces in the bolts. In EC3 form this is $V_{sd}$ and should be less than the plastic resistance of the web to shear ($V_{pl,Rd}$). In the event of the shear being greater than the web resistance stiffening should be provided. This may be obtained by the use of diagonal stiffeners, or the use of a reinforcing web plate welded to the sides of the web.

The plastic resistance of the web may be derived as in (d) of Condition I in Chapter 4. In the event of the shear being greater than half the plastic resistance of the web then the plastic moment resistance of the column above the bottom of the haunch must be adjusted.

## 15.5.9 Vertical shear

The welds connecting the haunch to the end plate should be checked for the shear forces in addition to the tensile forces due to the moments.

The bolts connecting the end plate to the face of the column should be checked for shear as well as tension. In most practical portal frames it will be found that the bolts needed to hold the bottom of the haunch in place will be capable of carrying the shear without the necessity of adding shear to the bolts at the top carrying tensile loads.

## 15.5.10 Summary of procedure

(i) Choose the number of bolts and the geometry for a trial connection making end plate thickness equal or greater than the column flange thickness.

(ii) Calculate the potential resistances of bolt rows in tension taking account of:

• Column flange bending
• Column web tension

(and if necessary beam web tension for rows not adjacent to beam flange).

(iii) Calculate the total resistance of the bolt rows in tension, which equates to the total compressive force in the connection after allowance made for axial forces.
(iv) Calculate the moment of resistance of the connection.
(v) Re-configure connection if the moment of resistance is less than that required and repeat steps (i) & (ii).
(vi) Check column web crushing resistance and web buckling resistance, providing compression stiffeners if found necessary.
(vii) Check column web panel shear resistance, providing compression stiffeners if found necessary.
(viii) Check beam compression flange bearing.
(ix) Check shear capacity of connection.
(x) Design the welds.

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# Appendix
# Symbols used in Eurocode 3

## Introduction

A standard system of symbols has been introduced in the Eurocodes to enable the various documents to be followed easily. The system consists of a leading letter main symbol followed by various appropriate subscript letters defining the details of the main symbol. Where there is a need for further expansion of the detail then a full stop is used followed by further subscript letters.

For example the design axial force on a member is given as $N$. The compressive force, resulting from the applied actions, is then given as $N_{Sd}$. This may be interpreted as :

| | |
|---|---|
| *N* | defines the fact that we have an axial force |
| *S* | defines it as an internal action |
| *d* | defines it as a design value |

In EC3 clause 5.4.4 we are then required to ensure that this is less than $N_{c.Rd}$, where the following applies:

| | |
|---|---|
| *N* | is the axial force |
| *c* | is the cross-section property |
| *R* | is the resistance |
| *d* | indicates that it is derived from the design |

The complete list of symbols is as follows:

**Latin upper case letters**

| | |
|---|---|
| *A* | Accidental action |
| *A* | Area |
| *B* | Bolt force |
| *C* | Capacity; Fixed value; Factor |
| *D* | Damage (fatigue assessment) |
| *E* | Modulus of elasticity |
| *E* | Effect of actions |
| *F* | Action |
| *F* | Force |
| *G* | Permanent action |
| *G* | Shear modulus |
| *H* | Total horizontal load or reaction |
| *I* | Second moment of area |
| *K* | Stiffness factor (*I*/*L*) |
| *L* | Length; Span; System length |
| *M* | Moment in general |
| *M* | Bending moment |
| *N* | Axial force |
| *Q* | Variable action |
| *R* | Resistance; Reaction |
| *S* | Internal forces and moments (with subscripts d or k) |
| *S* | Stiffness (shear, rotational ... stiffness with subscripts v, j…) |
| *T* | Torsional moment; Temperature |

| | |
|---|---|
| $V$ | Shear force; Total vertical load or reaction |
| $W$ | Section modulus |
| $X$ | Value of a property of a material |

**Greek upper case letters**

| | |
|---|---|
| Δ | Difference in... (precedes main symbol) |

**Latin lower case letters**

| | |
|---|---|
| a | Distance; Geometrical data |
| a | Throat thickness of a weld |
| a | Area ratio |
| b | Width; Breadth |
| c | Distance; Outstand |
| d | Diameter; Depth; Length of diagonal |
| e | Eccentricity; Shift of centroidal axis |
| e | Edge distance; End distance |
| f | Strength (of a material) |
| g | Gap; Width of a tension field |
| h | Height |
| i | Radius of gyration; Integer |
| k | Coefficient; Factor |
| I (or $l$, or L) | Length; Span; Buckling length |
| n | Ratio of normal forces or normal stresses |
| n | Number of ….. |
| p | Pitch; Spacing |
| q | Uniformly distributed force |
| r | Radius; Root radius |
| s | Staggered pitch; Distance |
| t | Thickness |
| uu | Major axis |
| vv | Minor axis |
| xx, yy, zz | Rectangular axes |

**Greek lower case letters**

| | | |
|---|---|---|
| α | (alpha) | Angle; Ratio; Factor |
| α | | Coefficient of linear thermal expansion |
| β | (beta) | Angle; Ratio; Factor |
| γ | (gamma) | Partial safety factor; Ratio |
| δ | (delta) | Deflection; Deformation |
| ε | (epsilon) | Strain; Coefficient = $[235/f_y]^{0.5}$ ($f_y$ in N/mm$^2$) |
| η | (eta) | Coefficient (in Annex E) |
| θ | (theta) | Angle; Slope |
| λ | (lambda) | Slenderness ratio; Ratio |
| μ | (mu) | Slip factor; Factor |
| ν | (nu) | Poisson’s ratio |
| ρ | (rho) | Reduction factor; Unit mass |
| σ | (sigma) | Normal stress |
| τ | (tau) | Shear stress |
| φ | (phi) | Rotation; Slope; Ratio |
| χ | (chi) | Reduction factor (for buckling) |

| | | |
|---|---|---|
| ψ | (psi) | Stress ratio; Reduction factor |
| ψ | | Factors defining representative values of variable actions |

**Subscripts**

| | |
|---|---|
| A | Accidental; Area |
| a | Average (yield strength) |
| a,b.... | First, second ... alternative |
| b | Basic (yield strength) |
| b | Bearing; Buckling |
| b | Bolt; Beam; Batten |
| C | Capacity; Consequences |
| c | Cross-section |
| c | Concrete; column |
| com | Compression |
| cr | Critical |
| d | Design; Diagonal |
| dst | Destablizing |
| E | Effect of actions (with d or k) |
| E | Euler |
| eff | Effective |
| e | Effective (with further subscript) |
| e, | Elastic |
| ext | External |
| f | Flange; Fastener |
| g | Gross |
| G | Permanent action |
| h | Height; Higher |
| h | Horizontal |
| 1 | Inner |
| inf | Inferior; Lower |
| i, j, k | Indices (replace by numeral) |
| j | Joint |
| k | Characteristic |
| l | Lower |
| L | Long |
| LT | Lateral torsional |
| M | Material |
| M | (Allowing for) bending moment |
| m | Bending; Mean |
| max | Maximum |
| min | Minimum |
| N | (Allowing for) axial force |
| n | Normal |
| net | Net |
| nom | Nominal |
| o | Hole; Initial; Outer |
| o | Local buckling |
| o | Point of zero moment |
| ov | Overlap |
| p | Plate; Pin; Packing |
| p | Preloading (force) |
| p | Partial; Punching shear |
| *pl* | Plastic |

| | |
|---|---|
| Q | Variable action |
| R | Resistance |
| r | Rivet; Restraint |
| rep | Representative |
| S | Internal force; Internal moment |
| s | Tensile stress (area) |
| s | Slip; Storey |
| s | Stiff; Stiffener |
| ser | Serviceability |
| stb | Stabilizing |
| sup | Superior; Upper |
| t (or ten) | Tension; Tensile |
| t (or tor) | Torsion |
| u | Major axis of cross-section |
| u | Ultimate (tensile strength) |
| ult | Ultimate (limit state) |
| V | (Allowing for) shear force |
| v | Shear; Vertical |
| v | Minor axis of cross-section |
| vec | Vectorial effects |
| w | Web; Weld; Warping |
| x | Axis along member; Extension |
| y | Yield |
| V | Axis of cross-section |
| z | Axis of cross-section |
| s | Normal stress |
| τ | Shear stress |
| ⊥ | Perpendicular |
| // | Parallel |