

ELASTIC DESIGN OF SINGLE-SPAN STEEL PORTAL FRAME BUILDINGS TO EUROCODE 3



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FOREWORD

Steel portal frames are the most common form of single-storey construction in the UK. The design of portal frames is well covered in BS 5950-1, supported by extensive guidance in other publications and commonly undertaken using bespoke software. The new national Standard BS EN 1993-1-1 does not contain such extensive (and UK-centric) guidance as BS 5950, giving rise to some lack of clarity in how the Eurocode rules should be applied to produce safe, structurally efficient and cost-effective frames.

This publication has been developed in response to the need for new guidance. Here, the guidance is limited to symmetric single span, elastically designed frames. Further work is needed before definitive guidance can be provided for the range of frames commonly designed and constructed in the UK.

This guide was prepared by Dorota Koschmidder and David Brown of the SCI, with technical support from David Iles and Abdul Malik, also of the SCI.

A number of industry experts contributed to the development of this publication and their support is acknowledged with gratitude. The expert group comprised:

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Dave Chapman	Tata Steel
John Clayton	RPS Burks Green
Gerrard Cox	Atlas Ward Structures
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SUMMARY

This publication provides guidance for the design of single-span symmetric portal frames in the UK in accordance with the Eurocodes and the UK National Annexes.

The publication gives an overview of the main portal frame elements, loading and initial design before providing more detail on elastic frame analysis and the design of the major components.

The key technical issues that differ from previous practice are:

- The assessment of frame stability (the significance of second-order effects)
- The allowance for second-order effects, if these are significant
- The inclusion of allowances for imperfections in the analysis
- The in-plane buckling checks of members
- The resistance of members with restraint to the tension flange.

In addition, the Eurocode presentation (for example the presentation of actions or of design calculation) differs from BS 5950, which, whilst not being a technical challenge, adds unfamiliarity that may initially contribute to a feeling of uncertainty.

A comprehensive worked example is provided to illustrate the analysis and design process, although it is acknowledged that economic portal frame design is best achieved by the use of bespoke software.

The scope of this publication is limited to symmetric, single span, elastically designed frames. Further work is underway to extend the guidance to other frames. The key uncertainty is the application of Expression 6.61 of BS EN 1993-1-1 to members of portal frames. Work is in progress to clarify the underlying requirements of Expression 6.61 in relation to the design of portal frames. If, as is hoped, it can be shown that these requirements can be accounted for within the global analysis of the portal frame, in-plane buckling of individual members need not be verified using expression 6.61. This would modify the guidance given in Sections 7.3.3 and 7.3.4, and the numerical example sections 10.1.7 and 10.2.3. If expression 6.61 is applied to members in a portal frame, designers should make their own assessment of the flexural buckling length in the major axis.

INTRODUCTION

Approximately 50% of constructional steelwork in the UK is used in the primary framework of single-storey buildings. In this market sector portal frames are the most common structural form in pitched roof buildings. Portal frames are lightweight, efficient and familiar to UK designers in both design and detailing. This form of construction was comprehensively covered in BS 5950-1^[1], which devoted a whole section to advice on portal frame design. BS EN 1993-1-1^[2] does not cover portal frames in such depth, but generally provides design principles and general application rules.

This publication guides the designer through the detailed steps involved in the elastic design of single span portal frames to BS EN 1993-1-1, although it is readily acknowledged that design using software is a more realistic solution for efficient design. The publication provides guidance on the manual methods used for initial sizing, determination of actions, assessment of frame stability and verification of members in accordance with BS EN 1993-1-1. The importance of appropriate design details is emphasised, with good practice illustrated.

Whilst manual design may be useful for initial sizing of members, and a thorough understanding of the design process is necessary, the use of software offers the means to achieve the greatest structural efficiency. Widely available bespoke software for portal frame design will:

- undertake elastic-plastic analysis
- allow for second-order effects
- verify members
- verify connections.

Generally, a number of different load combinations will have to be considered during the design of a portal frame. Software that verifies the members for all load combinations will shorten the design process considerably.

1.1 Scope

The guidance in this publication is limited to the elastic design of single-span symmetric portal frames using hot rolled steel I sections. Plastic design is not addressed in this publication. The publication refers principally to BS EN 1993-1-1 and its UK National Annex^[3]. Where appropriate, non-contradictory complementary information (NCCI) is referenced.

This publication does not address portal frames with ties between eaves. This form of portal frame is relatively rare. The ties modify the distribution of bending moments substantially and increase the axial force in the rafters dramatically. Second-order analysis software must be used for the design of portal frames with ties at eaves level because first order analysis does not allow for the significant second-order effects in such frames.

1.2 Why choose a portal frame?

Steel portal frames are appreciated as a highly efficient and cost-effective way to support an envelope, enclosing a usable volume. Steel portal frames are highly suited to carrying relatively modest loads. By their very nature they are relatively flexible; less onerous deflection limits are generally applied to portal frames than for other forms of construction. The careful detailing of cladding, flashings etc. is sufficient to ensure that the flexible behaviour of a steel portal frame is not detrimental to the performance of the envelope.

Although the deflection of steel portal frames can be reduced, for example by the use of ever larger steel sections, the cost-effectiveness of the solution will be adversely affected. If deflections are critical, or the frame is carrying high loads (from suspended machinery, for example), it may be more appropriate to select an alternative structural form, such as a truss.

ANATOMY OF A PORTAL FRAME BUILDING

A portal frame building comprises a series of unbraced transverse frames, braced longitudinally. The primary steelwork consists of columns and rafters, which form the portal frames, and longitudinal bracing, as shown in Figure 2.1. The end frame (gable frame) can be either a portal frame or a braced arrangement of columns and rafters.

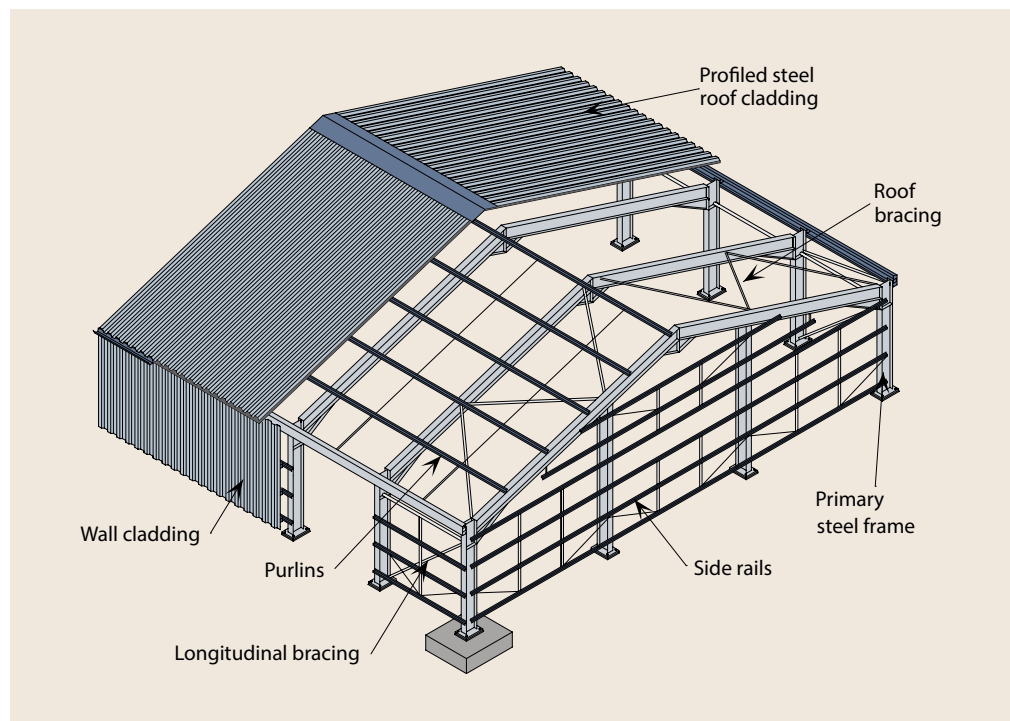


Figure 2.1
Principal building
components

The secondary steelwork supporting the cladding consists of side rails for walls and purlins for the roof. The secondary steelwork also plays an important role in restraining the primary steelwork members against buckling out-of-plane.

The roof and wall cladding separate the enclosed space from the external environment and provide thermal and acoustic insulation. The cladding transfers loads to secondary steelwork and restrains the flange of the purlin or rail to which it is attached.

2.1 Main frame

A portal frame is a continuous frame with moment-resisting connections. The continuous nature of the frame provides stability in-plane, and the resistance to lateral loads. Frame stability and frame deflections are therefore dependant on the

stiffness of the members. The members are normally hot-rolled steel sections, with the resistance of the rafters enhanced locally by a haunch at the eaves, where the bending moments are greatest. In most cases, the frame will be assumed to have nominally pinned bases, even if the actual base details possess appreciable stiffness.

The main frames are generally fabricated from UKB sections. The eaves haunch is created by adding a tapered length cut from a rolled section, or fabricated from plate. A typical frame is shown diagrammatically in Figure 2.2 and is characterised by:

- A span between 15 m and 50 m
- A clear height (from the top of the floor to the underside of the haunch) between 5 and 15 m
- A roof pitch between 5° and 10° (6° is commonly adopted)
- A frame spacing between 6 m and 8 m
- Haunches in the rafters at the eaves
- The second moment of area (I_{yy}) of the columns is typically 50% larger than that of the rafters.
- Thin gauge (light steel) cold rolled steel purlins and side rails
- Light steel diagonal restraints from some purlins and side rails, to restrain the inside flange of the frame at certain locations.

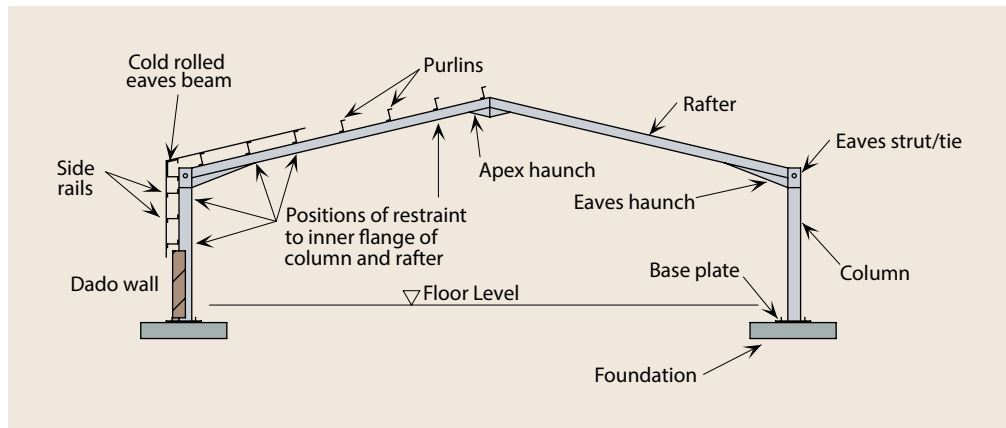


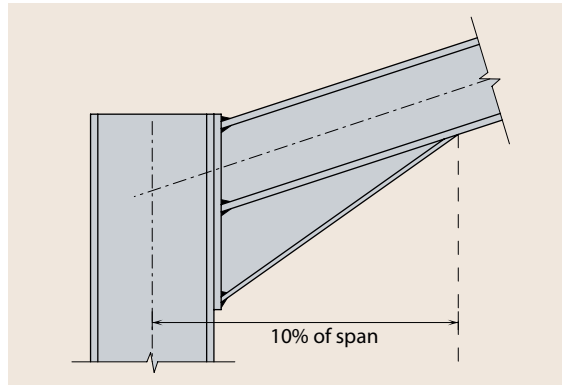
Figure 2.2
Cross section
showing a portal
frame and
its restraints

2.2 Haunches

The use of a haunch at the eaves reduces the required depth of rafter by increasing the bending resistance of the member where the design moments are highest. The haunch also adds stiffness to the frame, reducing deflections, and facilitates an efficient bolted moment-resisting connection.

The eaves haunch is typically cut from the same size rolled section as the rafter, or one slightly larger, and is welded to the underside of the rafter. The length of the eaves haunch is generally 10% of the frame span, as shown in Figure 2.3. A haunch of this length generally means that the hogging moment at the end of the haunch is approximately equal to the largest sagging moment close to the apex. The depth of haunch below the rafter is approximately equal to the depth of the rafter section.

Figure 2.3
Approximate
haunch size



The apex haunch may be cut from a rolled section – often from the same size as the rafter, or fabricated from plate. The apex haunch is not usually modelled in the frame analysis and is not needed to enhance the bending resistance; it is only used to facilitate a bolted connection.

2.3 Bracing

Bracing is required both in the plane of the rafters and vertically in the plane of the side walls. The vertical bracing in the walls is often provided at both ends of the building, or in one bay only. Each frame is connected to the vertical bracing by a hot-rolled member (the eaves strut/tie) at eaves level. A typical bracing arrangement is shown in Figure 2.4. A single diagonal member is often used instead of the “k” vertical bracing illustrated.

At both ends of the building the roof bracing intersects with the top of gable columns and transfers loads in the plane of the roof to the eaves.

Commonly used bracing section types include circular hollow sections (CHS), angles and crossed flats. Crossed flats are generally only considered suitable for vertical bracing.

If diagonal bracing in the side walls cannot be accommodated, longitudinal stability can be provided by a rigid frame, as shown in Figure 2.5.

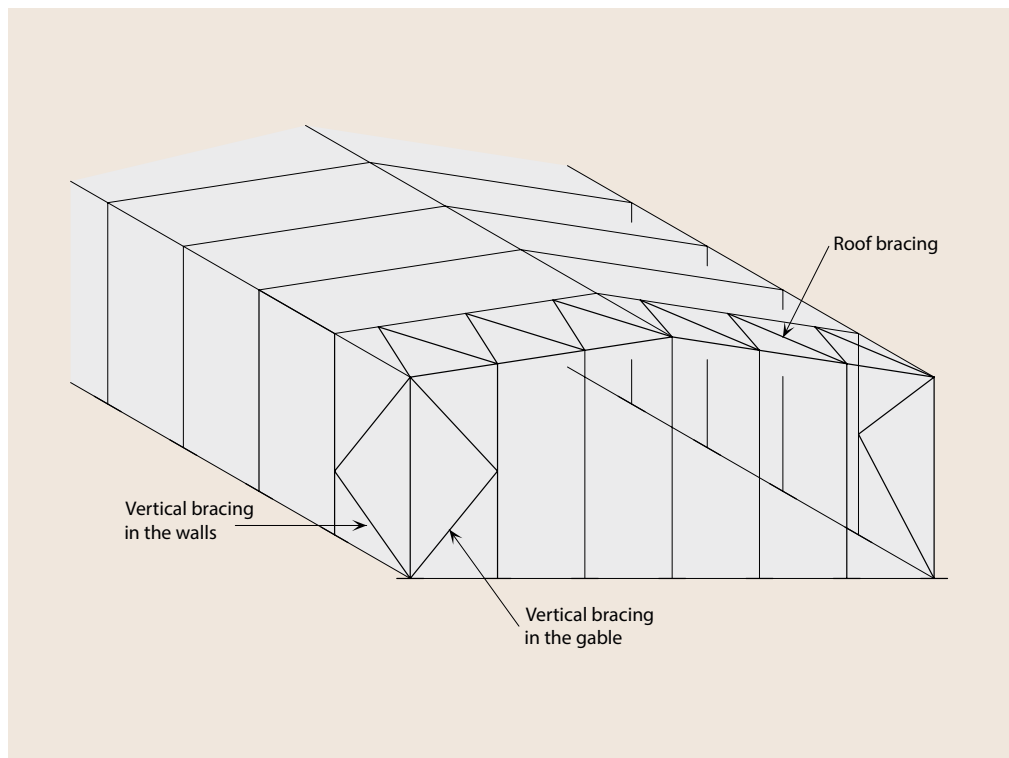


Figure 2.4
Typical bracing in a
portal frame

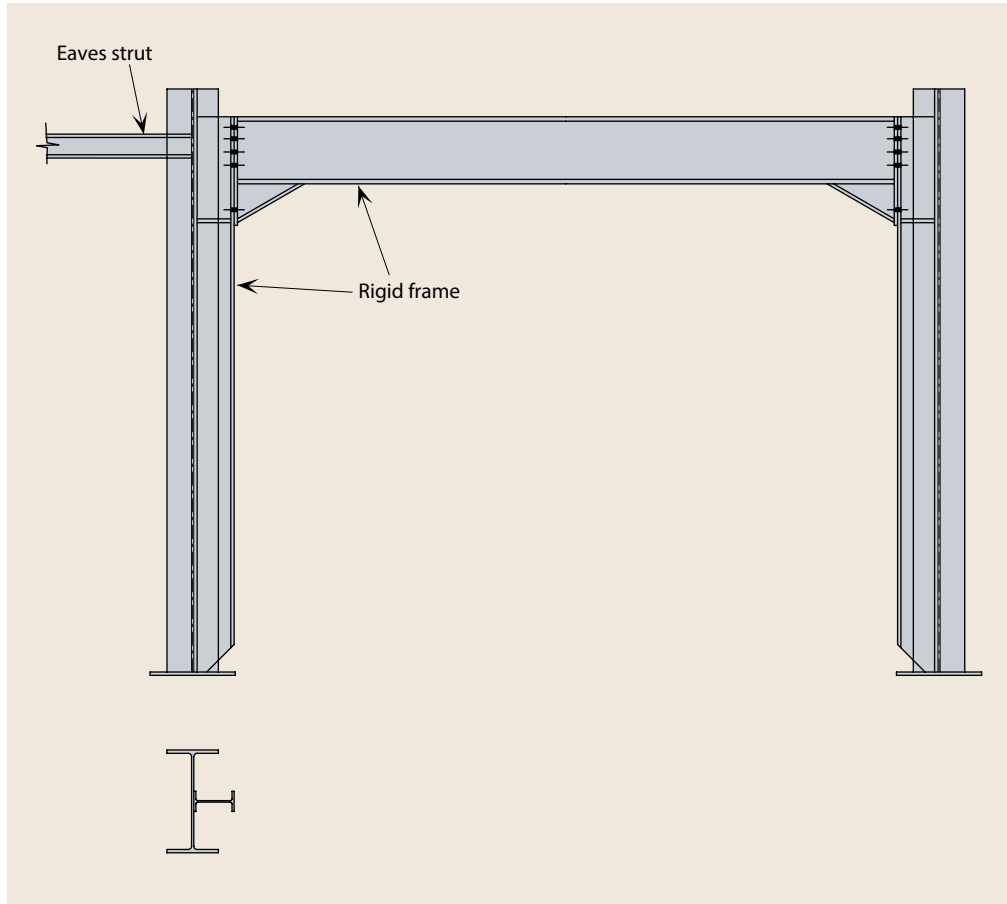


Figure 2.5
Rigid frame
alternative to
vertical bracing in
the side walls

2.4 Secondary steelwork

Secondary steelwork members, such as purlins, side rails and eaves beams are generally cold rolled thin gauge galvanized steel sections available from a wide range of manufacturers in a variety of shapes and sizes. Common section types used for purlins and side rails are illustrated in Figure 2.6.

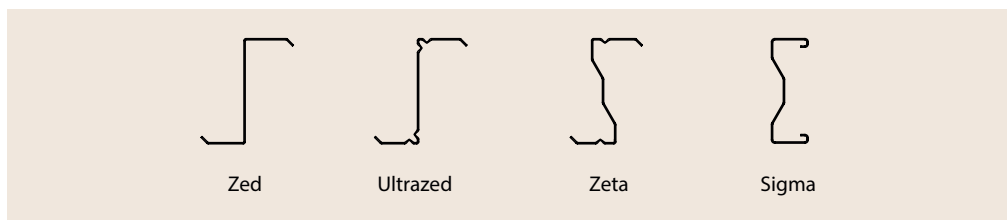


Figure 2.6
Common shapes of
secondary steelwork

The primary function of purlins and side rails is to transfer forces from the cladding to the primary steel frame. The forces arise from wind pressures, and for roofs, snow loads and imposed loads due to access. Purlins and side rails may also be used to provide restraint to the rafters and columns at certain positions, as illustrated in Figure 2.7 and Figure 2.8.

Purlins can be arranged in a variety of layouts, from single spans between individual frames to continuous members over the length of the building (see Section 11 for

typical layouts). The greater bending moments that arise in the end span and at the support positions of a continuous member can be addressed by providing increased resistance at the critical sections, by using thicker material, by overlapping purlins or by providing purlin sleeves.

Figure 2.7
General arrangement
of restraints to the
inside flange

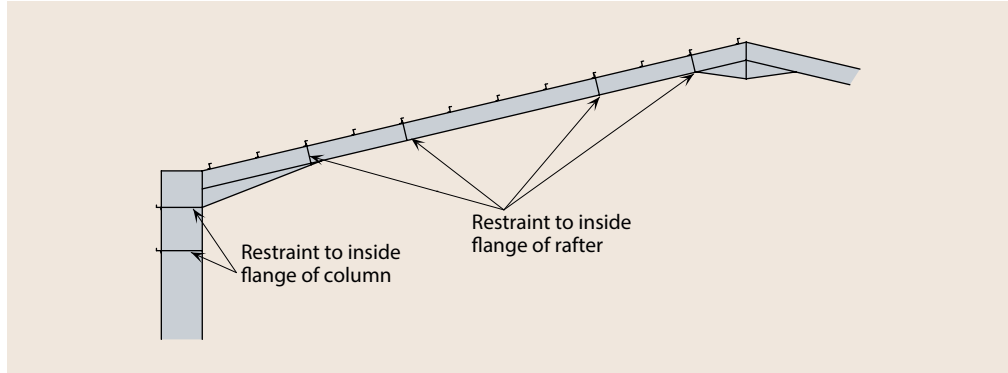
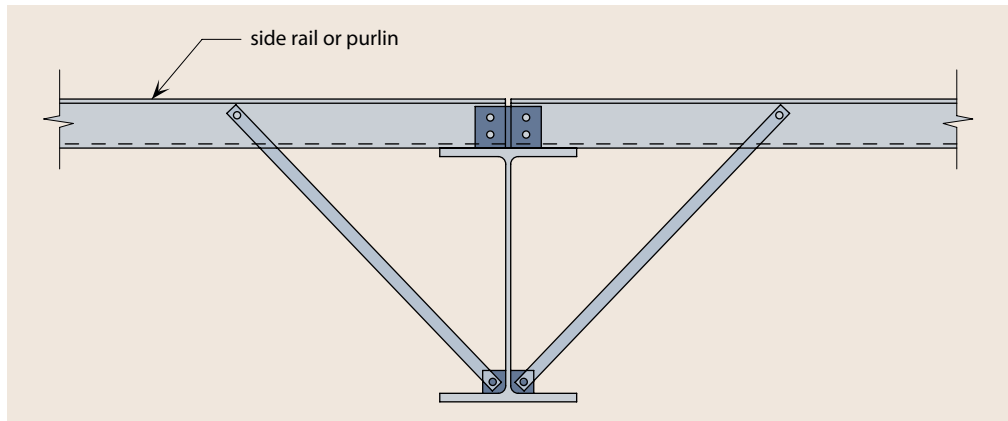


Figure 2.8
Typical restraint to
the inside flange

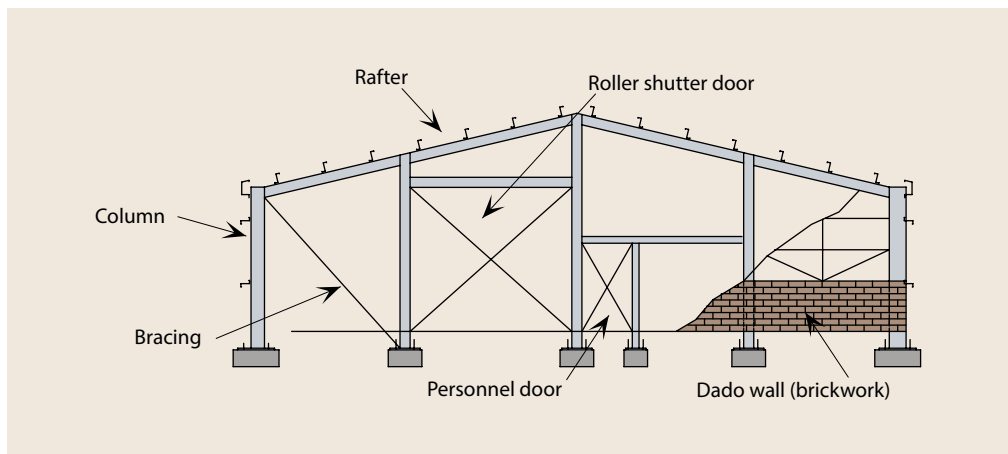


2.5 Gables

2.5.1 Plane gable frames

The end frames of a portal frame building are generally called gable frames. A typical gable frame, in which the gable columns support the gable rafters, is shown in Figure 2.9.

Figure 2.9
Typical details of an
end gable of a portal
frame building



Alternatively, gable frames may be identical to the internal portal frames, even though they experience lighter loads. If future extension to the building is envisaged, portal frames are commonly used as the gable frames, to reduce the impact of the structural works during the extension. In this type of gable frame, the gable columns span as vertical beams from the ground to the portal frame; they may be detailed with connections to accommodate the anticipated deflection of the rafter.

2.5.2 Hipped roof frames

A hipped roof is shown in Figure 2.10. The line of the apex is stopped short of the end of the building, and pitched roof provided in the longitudinal direction. Hipped roofs are often used to enhance the appearance of an industrial structure or where a more traditional roof shape is required, for example in a supermarket, sports hall, or car park.

Hipped roofs can be constructed in a number of ways:

The end frame can be placed at the same spacing as the main frames and the rafters angled to meet at the apex of the penultimate frame. This usually leads to an end roof pitch that is steeper than the general roof pitch.

A gablette feature can be introduced, where the inclined rafters meet below the apex of the penultimate frame (Figure 2.11). This reduces the pitch of the hip.

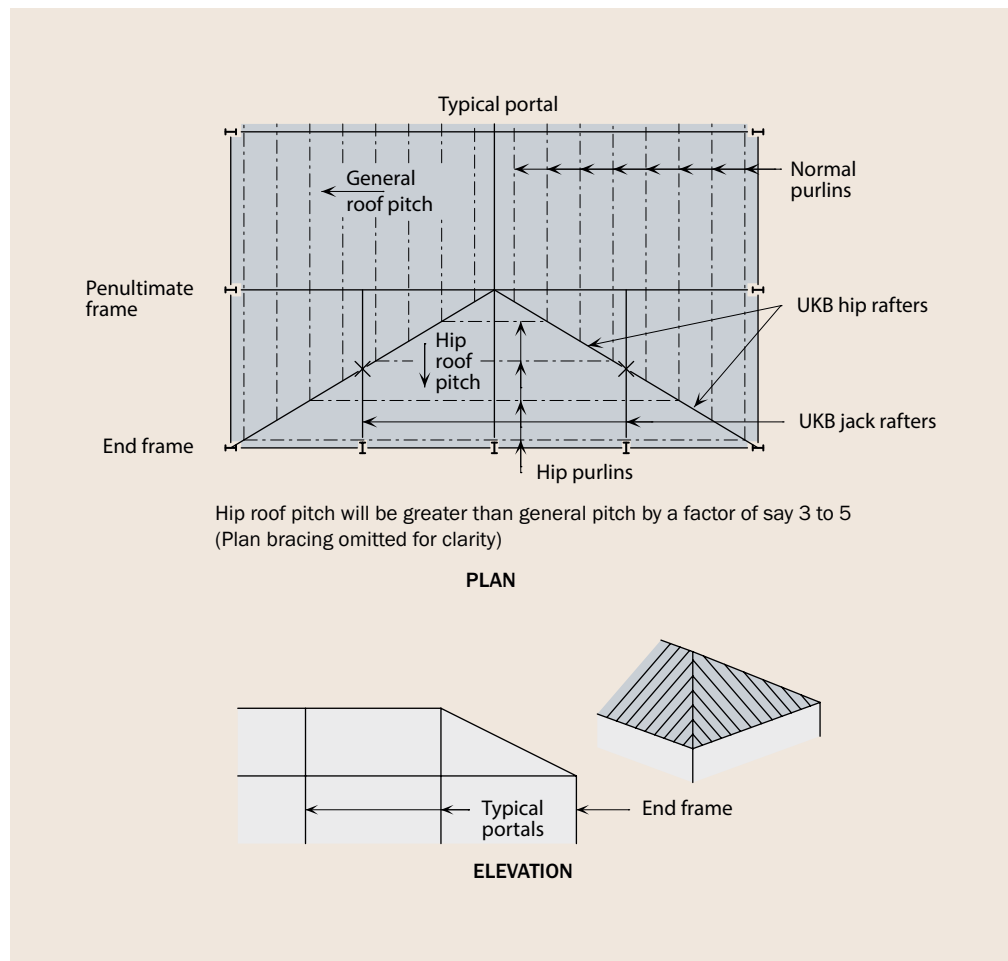


Figure 2.10 Framing for hipped roof with a hip roof pitch greater than that of the main roof

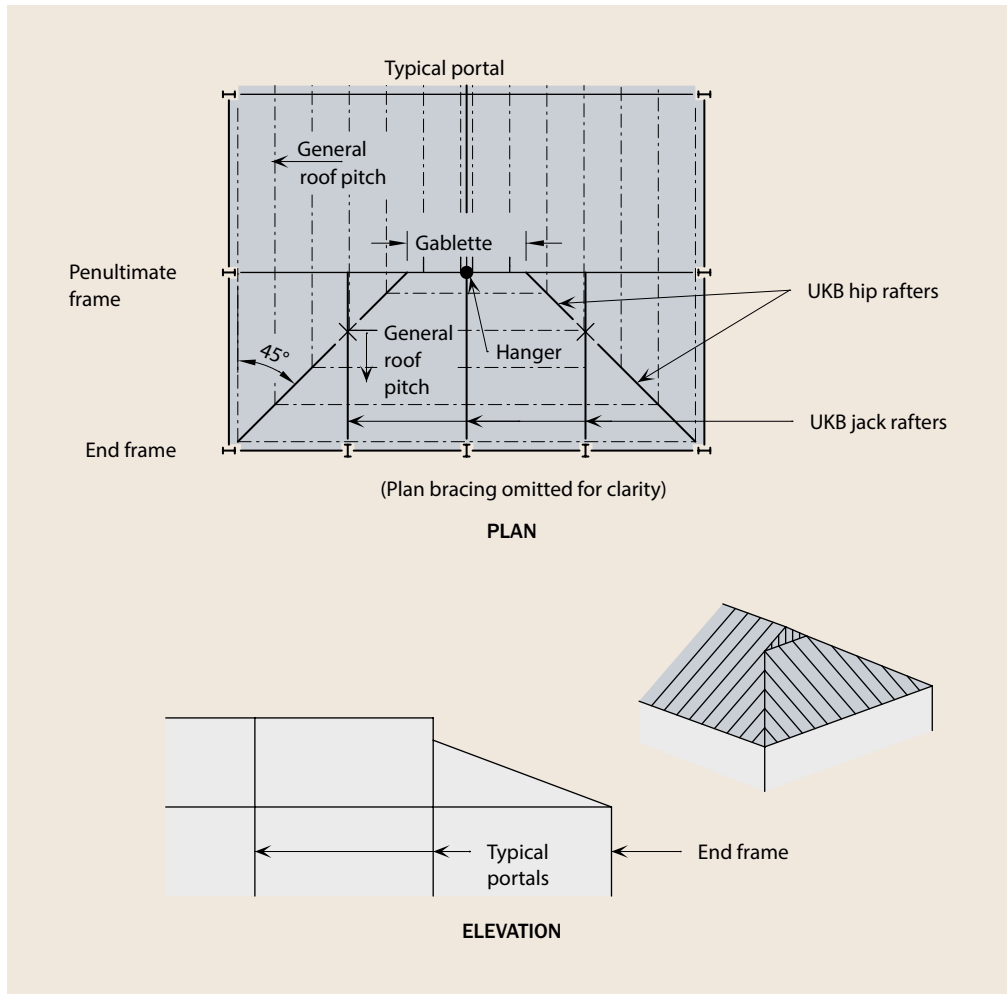


Figure 2.11 Framing for hipped roof with a hip roof pitch equal to that of the main roof, with a gablette feature

The rafters can be arranged at an angle of 45° on plan to create a roof pitch identical to the main roof pitch. Intermediate frames may be required which can take the form of a flat-topped mansard portal frame, as shown in Figure 2.12. A gablette may be used, as shown in Figure 2.11, or the hip rafters may be continued to intersect with the apex, as shown in Figure 2.12. The intersection of the hip rafters and the apex will often not fall conveniently at a portal frame; cranked steelwork may be required to support the tops of the hip rafters.

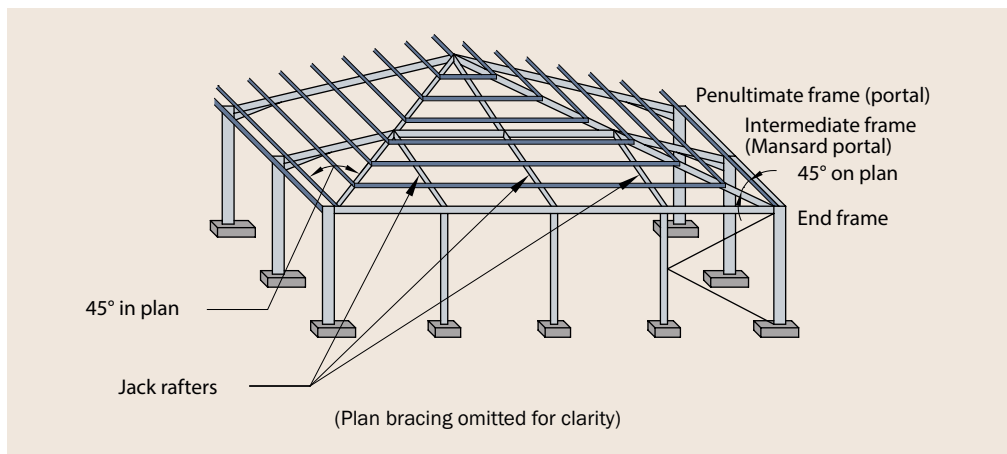


Figure 2.12 Framing for hipped roof using a flat-topped mansard as the penultimate frame

The rafters supporting the hip slope are sometimes called jack rafters; they may be simply supported or have a moment connection to the gable posts. Vertical bracing is required in the end elevation, as shown in Figure 2.12.

As an alternative to the layout shown in Figure 2.11, all the jack rafters may be arranged to span to the penultimate frame. This has the advantage that the hip rafters are smaller, since they have an intermediate support at the jack rafters.

2.6 Building envelope

The basic function of the building envelope is the provision of a controlled internal environment that is protected from the variable and uncontrollable external climate. A well designed building envelope is essential to regulate the heat gain and loss, and to enable the maintenance of the required internal environment.

In addition to forming the building envelope, the roof and wall cladding will have an important role to play in the structural performance of the building, by providing restraint to the secondary steelwork. Purlin and side rail manufacturers generally assume that the cladding restrains the flange of the secondary steelwork to which it is connected; they provide tables of resistances (or software) based on that assumption. If the cladding does not restrain the connected flange of the purlin (or side rail), the system manufacturer should be consulted for appropriate resistance values.

If restraint is required to the inside flange of the secondary member (typically in the uplift condition), so-called anti-sag systems are specified, as illustrated in Figure 2.13. The anti-sag system restrains the inner flange for the secondary member at discrete points; the anti-sag system also assists in the aligning of the secondary steelwork until the cladding is fixed.

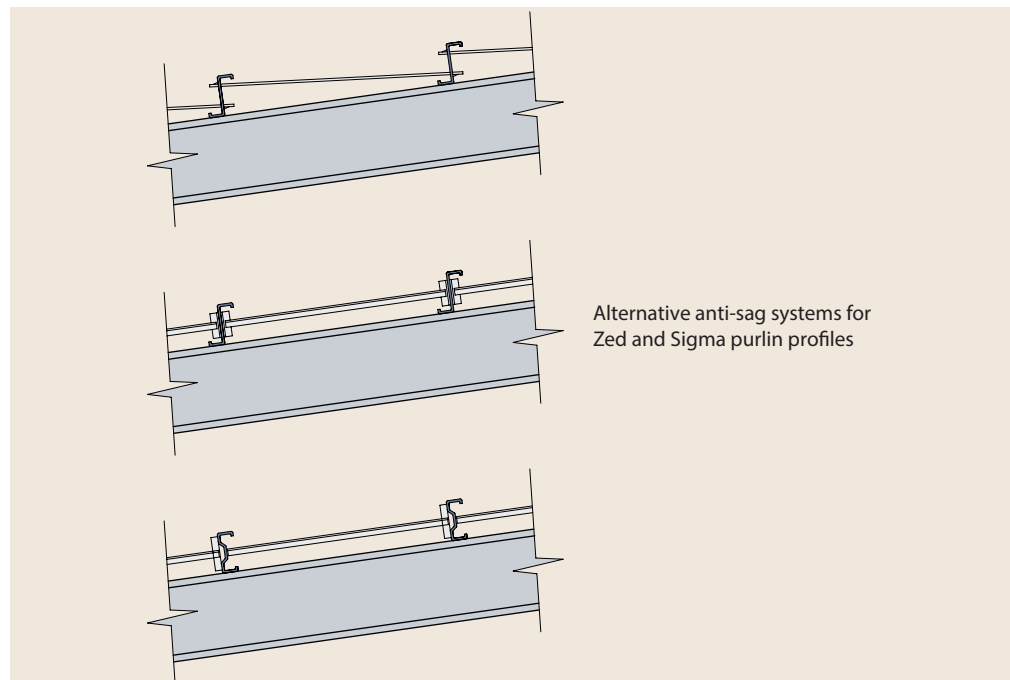


Figure 2.13
Anti-sag systems

Additional guidance covering secondary steelwork and cladding is given in *Best practice for the specification and installation of metal cladding and secondary steelwork* (P346)^[4].

2.6.1 Metal cladding systems

Metal cladding systems provide an efficient solution to the building envelope needs of single storey buildings. Cladding systems range from single skin metal cladding, often associated with agricultural buildings, to highly developed systems used in commercial and industrial applications.

The main types of cladding systems are summarised below:

Roofing

- Single skin sheeting, used when no insulation is required.
- 'Built up' or double layer systems, comprising a liner tray fixed to the purlins, a spacing system, insulation and an outer sheet.
- Standing seam systems, which fasten via clips and have no fixing penetrations through the sheeting.
- Composite panels (also known as sandwich panels), which have a foam or mineral wool insulation layer bonded between the inner and outer layers of sheeting. Composite panels may have direct or standing seam fixings.
- Deep decking, spanning between main frames, supporting insulation, with an external metal sheet or waterproof membrane.

Wall cladding

- Sheetting, orientated vertically and supported on side rails.
- Sheetting or structural liner trays, spanning horizontally between columns.
- Composite or sandwich panels spanning horizontally between columns (thus eliminating side rails).
- Metallic cassette panels, supported by side rails.

Different forms of cladding may be used together for visual effect in the same façade. Examples are illustrated in Figure 2.14, Figure 2.15 and Figure 2.16.



Figure 2.14
Horizontal spanning
sheeting

Figure 2.15 (Left)
Large windows and use of composite panels with dado brick wall



Figure 2.16 (Right)
Horizontal composite panels and 'ribbon' windows



2.6.2 Dado brickwork wall

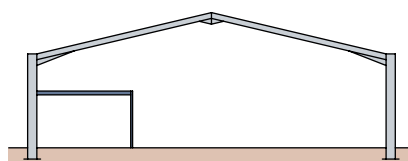
Brickwork is often used at low level along the perimeter of the structure to provide resistance to minor impact, as shown in Figure 2.15. The dado wall may be up to the level of the windows, or in an industrial frame, up to approximately 2.5 m. The dado wall can have a significant effect on the design of the structure, because the deflection limits for the frames will be more onerous than those for completely clad structures. The location of the wall may also affect the location and type of vertical bracing.

For taller walls, lateral restraint may be required at the top and/or mid height of the wall. This can be achieved by providing a small hot rolled section with its web placed horizontally or, alternatively, a cold rolled C or Z section. Guidance on appropriate detailing between the steelwork and masonry is given in *Brick cladding to steel framed buildings*^[5].

2.7 Other types of portal frames

Although this publication concentrates on the design of single span, symmetric portal frames, many different forms of portal frames can be constructed. Frame types described below give an overview of other types of portal construction, with typical features illustrated. This information only provides typical details and is not meant to dictate any limits on the use of any particular structural form.

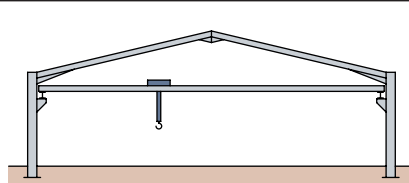
Table 2.1
Other types of portal frame



Portal frame with internal mezzanine floor

Office accommodation is often provided within a portal frame structure using a partial width mezzanine floor.

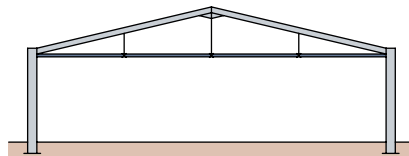
The assessment of frame stability must include the effect of the mezzanine^[6].



Portal frame with overhead travelling crane

Where a travelling crane of relatively low capacity (up to say 20 tonnes) is required, brackets can be fixed to the columns to support the crane rails. Use of rigid column bases may be necessary to reduce the eaves deflection.

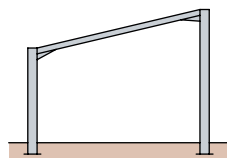
The spread of the frame at crane rail level may be of critical importance to the functioning of the crane; requirements should be agreed with the client and with the crane manufacturer.



Tied portal frame

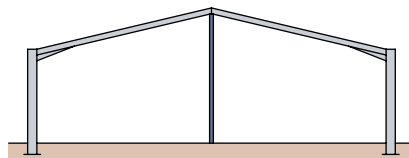
In a tied portal frame the horizontal movement of the eaves and the bending moments in the columns and rafters are reduced. A tie may be useful to limit spread in a crane-supporting structure.

The high axial forces introduced in the frame when a tie is used necessitate the use of second-order frame analysis.



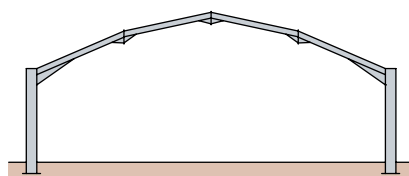
Mono-pitch portal frame

A mono-pitch portal frame is usually chosen for small spans (up to 15 m) or because of its proximity to other buildings. It is a simple variation of the pitched roof portal frame, and tends to be used for smaller buildings. Mono-pitch frames can be very sensitive to wind loading.



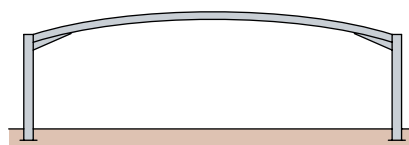
Propped portal frame

Where the span of a portal frame is large and there is no requirement to provide a clear span, a propped portal frame can be used to reduce the rafter size and the horizontal shear forces at the column bases.



Mansard portal frame

The steeper initial pitch is used for aesthetic reasons, with the shallower upper pitches used to reduce overall height and internal volume.



Curved rafter portal frame

Portal frames may be constructed using curved rafters, mainly for architectural reasons. The curved member is often modelled for analysis as a series of straight elements. Guidance on the stability of curved rafters in portal frames is given in reference 7.

Alternatively, the rafter can be fabricated as a series of straight elements with purlin cleats of varying height to achieve the curved external profile.



Cellular beam portal frame

Rafters may be fabricated from cellular beams for aesthetic reasons or when providing long spans.

The sections used cannot develop plastic hinges, so only elastic design is used.

Table 2.1
Other types of
portal frame
(continued)

DESIGN PROCESS

The process of designing the main frame involves several stages, each of which is described in detail in the following Sections of this guide. For clarity, the outline of the whole design process is introduced in Table 3.1, with reference to the relevant Section of this publication.

DESIGN STEP	SECTION	COMMENTS
Geometry	5.1	Establish the clear span and height based on the client's requirements. The geometry used in the analysis should be a little conservative to allow for subsequent changes in member size.
Actions	4	Establish actions depending on location, site altitude and local topography. Permanent actions may be estimated, based on selected cladding type.
Preliminary design	5.2	Use tables from Appendix A or the preliminary design method described in Section 5.2.
Initial member selection	5.2	Select member sizes based on their cross section resistance and buckling resistance. It may be assumed that the influence of shear or axial load on the bending resistance can be neglected for initial design. Rafter At this stage it may be assumed that sufficient restraints can be introduced to limit member buckling, Column The lateral-torsional buckling resistance of the column is likely to be the critical check, so preliminary checks over the restrained lengths will be necessary. If intermediate restraints cannot be introduced to the column (because the side rails are not continuous), a larger column section will be required.
Frame stability	6.3.3	Sensitivity to second-order effects must be assessed. It is likely that second-order effects must be allowed for, either by amplifying the results of a first-order analysis, or by completing a second-order analysis. When assessing frame stability, it is recommended that bases are modelled in accordance with the guidance given in Section 6.4.
Member verification	7.1.1	Classification of member cross sections.
	7.1.1, 7.1.2, 7.1.3	Verification of cross section resistance to bending, shear and compression. Bending interaction with shear or compression is generally not critical.
	7.2	Buckling resistance is checked, establishing the position of the restraints to both flanges and thus the buckling lengths. Out of plane buckling resistance is verified against flexural and lateral torsional buckling. Interaction of bending moment and axial force is checked with the use of Expression 6.62 of BS EN 1993-1-1. In plane buckling resistance is checked with the use of expression 6.61 of BS EN 1993-1-1. It is not possible to provide intermediate restraints that resist in-plane flexural buckling.
SLS	13	Frame deflections are checked against client requirements.

Table 3.1
Design sequence

ACTIONS

This Section covers the actions that should be considered in the design of a steel portal frame, and the combination of those actions at the ultimate limit state and the serviceability limit state.

Rules for actions can be found in BS EN 1991^[8], and on the design combinations of actions in BS EN 1990^[9]. It is important to refer to the National Annex for the relevant Eurocode part and the country the structure is to be constructed in - this Section reflects the recommendations of the UK National Annexes.

4.1 Permanent actions

Permanent actions are the self weight of the structure, secondary steelwork and cladding. Where possible, unit weights of materials should be obtained from manufacturers' data. Where information is not available, these may be determined from data in BS EN 1991-1-1^[10].

Typical weights of materials used in roofing are given in Table 4.1. For a roof that only carries normal imposed roof loads (i.e. no suspended machinery or similar), the self weight of the cladding plus secondary steelwork is typically 0.2 to 0.4 kN/m² when expressed over the plan area of the roof.

MATERIAL	WEIGHT (kN/m²)
Steel roof sheeting (single skin)	0.07 - 0.12
Aluminium roof sheeting (single skin)	0.04
Insulation (boards, per 25 mm thickness)	0.07
Insulation (glass fibre, per 100 mm thickness)	0.01
Liner trays (0.4 mm – 0.7 mm thickness)	0.04 - 0.07
Composite panels (40 mm – 100 mm thickness)	0.1 - 0.15
Steel purlins (distributed over the roof area)	0.03
Steel decking	0.2

Table 4.1
Typical weights of
roofing materials

Another component of loading to be considered as a permanent action is the self weight of any services (commonly called service loading). Depending on the use of the building, the weight of the services varies significantly.

At the preliminary design stage, the service loading is usually assumed to be between 0.15 and 0.4 kN/m² on plan over the whole roof area. High service loads are likely to result in substantial point loads applied to the purlin; the attachment details and member resistance should be verified in accordance with the manufacturer's recommendations. It is also important to recognise that the services may be removed during the life of the structure and, where service loads have a beneficial effect in opposing wind uplift, they should be neglected.

At the final design stage, the structure should be checked for the actual service loads.

4.2 Variable actions

4.2.1 Imposed roof loads

Imposed loads on roofs that are not accessible, except for normal maintenance and repair, are classed under category H in BS EN 1991-1-1^[10]. For that category of roof, The UK NA to BS EN 1991-1-1^[14] gives imposed loads on roofs that depend on the roof slope. A point load, Q_k is given, which is used for local verification of roof materials and fixings, and a uniformly distributed load, q_k , applied vertically and used for the design of the structure. The loading for roofs not accessible except for normal maintenance and repair is given in Table 4.2.

Table 4.2
Imposed loads
on roofs, taken
from the UK NA to
BS EN 1991-1-1

ROOF SLOPE, α	q_k (kN/m ²)
$\alpha < 30^\circ$	0.6
$30^\circ \leq \alpha < 60^\circ$	$0.6[60 - \alpha]/30]$
$\alpha \geq 60^\circ$	0

It should be noted that, following Clause 3.3.2(1) of BS EN 1991-1-1^[10], imposed loads on roofs should not be combined with either snow loads or wind actions.

4.2.2 Snow loads

Snow loads in the UK should be determined from BS EN 1991-1-3^[12] and its National Annex^[13].

The characteristic snow load on the ground, s_k , depends on the site location and altitude, so it will vary for different sites. The characteristic snow load on a roof, s , is taken as s_k multiplied by the snow load shape coefficient μ (which allows for the roof shape), the exposure coefficient C_e and the thermal coefficient C_t . BS EN 1991-1-3 Clause 5.2(7) recommends that both C_e and C_t be taken as 1.0.

According to Clause NA.2.2 of the UK National Annex to BS EN 1991-1-3, the following design situations should be considered:

- undrifted snow
- drifted snow (partial removal of snow from one slope)
- exceptional snow drifts, which should be treated as accidental actions.

Both undrifted and drifted snow should be considered in persistent design situations at ULS, and should be combined with other actions using expressions 6.10 or 6.10a and 6.10b of BS EN 1990 (See Section 4.6 of this publication). The exceptional snow drift case is an accidental action, and should be considered in combination with other actions using Expression 6.11b.

Exceptional snow drifts should be considered:

- behind parapets
- in valleys of multi-span frames
- behind obstructions on the roof
- from snow blown off adjacent buildings.

Because the exceptional snow drifts are considered as accidental actions, it is likely that they have little or no influence on the design of the main frame members. However, exceptional snow drifts are likely to be an important design consideration for the secondary steelwork, which may mean selecting larger sections, or a thicker gauge, or specifying reduced centres.

4.2.3 Wind actions

Wind actions in the UK should be determined from BS EN 1991-1-4^[14] and its National Annex^[15]. The UK NA is a substantial document, with many provisions to be observed when determining wind actions in the UK. Guidance on the determination of wind actions is given in SCI publication P394^[16].

For single span portal frame buildings in the UK designed to previous National Standards, it was uncommon for load combinations including wind to determine sizes of members. With design to the Eurocodes, wind actions are more significant than previous practice because:

- Wind actions can appear as the leading variable action in the ULS combination of actions, with a partial factor of 1.5 applied to characteristic values.
- Although wind actions do not occur simultaneously with imposed roof loads, wind actions are to be combined with snow loads. The combination of actions including both wind and snow may lead to greater bending moments than the combination including only imposed roof loads with permanent actions.

Although wind actions may not be significant at the preliminary design stage, combinations including wind must always be checked as part of the final design process, both at ULS and SLS. Combinations including wind actions may govern in some situations, such as:

- Where deflections at SLS are critical, i.e.:
 - if the portal frame supports an overhead travelling crane, or
 - if masonry or some other relatively brittle wall construction is used.
- In uplift conditions (a reversed bending moment diagram), as this will determine restraint positions for the rafters (see Section 7.4.2).

Several alternative routes can be chosen to calculate wind actions. More design effort will generally lead to reduced loads. The procedure of calculating wind actions includes five stages:

1. Calculation of the peak velocity pressure
2. Determination of external pressure coefficients
3. Determination of internal pressure coefficients
4. Calculation of the structural factor
5. Calculation of wind forces.

Wind pressures are calculated as the product of the peak velocity pressure, the structural factor and pressure coefficients. External and internal pressure coefficients are given in the Eurocode – but note that external pressure coefficients for the roof should be taken from the UK National Annex. In the Eurocode, coefficients are given for elements with loaded areas of up to 1 m² and loaded areas of over 10 m², with logarithmic interpolation for areas between the two. The UK NA simplifies this, allowing the use of the coefficients for 10 m², known as c_{pe} , for any loaded area larger than 1 m².

For the purpose of calculating overall loads on the structure, for example to determine design effects for bracing systems, the Eurocode provides force coefficients, which include friction effects. When designing individual portal frames, loads on elements are required. These loads depend on the internal and external pressures.

Peak velocity pressure

Calculating the peak velocity pressure can be carried out using one of the four alternative approaches summarised in Table 4.3. Each approach demands different levels of information about the site, and involves different levels of calculation effort.

	APPROACH 1	APPROACH 2	APPROACH 3	APPROACH 4
Peak velocity pressure	Calculated in 12 directions at 30° intervals	Most onerous factors from 360° around the site	Calculated in 4 quadrants	Calculated in 4 quadrants at ±45° to the normal to the building face
Building orientation	Not required	Not required	Not required	Required
Outcome	Generally the least onerous result	The most onerous result	Generally a less onerous pressure than Approach 2	Generally a less onerous pressure than Approach 2, but specific to each building face
Calculation effort	Significant	Least effort	Modest effort	Modest effort
Application	Both orthogonal directions use the same peak velocity pressure	Both orthogonal directions use the same peak velocity pressure	Both orthogonal directions use the same peak velocity pressure	Actions on each face may use a different peak velocity pressure
Comments	Least onerous peak velocity pressure, but significant calculation effort	Simple, conservative	Recommended approach	Useful for asymmetric structures or where dominant openings are significant

*Table 4.3
Comparison of approaches to determine the peak velocity pressure*

Although Table 4.3 indicates that building orientation is “not required” for Approach 1, it can be beneficial to account for building orientation, especially if there is an asymmetry, or particular features of the building, (for example, a dominant opening) that mean actions determined with respect to a building face are useful. If the building orientation is not accounted for, the same peak velocity pressure must be used in both orthogonal directions.

It should be noted that software is widely available to calculate wind actions in accordance with Approach 1, relieving the designer of the calculation effort. Usually, software will account for the building orientation, leading to a value of peak velocity pressure normal to each building face.

For regular structures, it is recommended that Approach 3 is adopted if undertaking calculations without software, balancing the need for site-specific information, calculation effort and resulting actions.

External pressure coefficients

External pressure coefficients for walls and roofs are given in the Eurocode and the UK NA. The pressure coefficients on a portal frame building fall into a number of zones, with higher suctions next to corners, such as the vertical corner of a wall, or adjacent to the eaves and ridge of a duopitch roof.

The demarcation between roof zones does not correspond to the zones on the walls, which complicates the assessment of actions on individual frames in typical structures. Some engineering judgement is required to identify the most onerous combinations of actions on the most heavily loaded frame. In many low rise industrial structures, the penultimate frame is likely to be the most critical frame.

Internal pressure coefficients

Internal pressure coefficients are given by Clause 7.2.9 of BS EN 1991-1-4. Where there are no dominant openings, the value of the internal pressure coefficient can be calculated based on the opening ratio in the face under consideration and taken from Figure 7.13 of the Eurocode.

The UK NA gives the permeability of a limited selection of forms of construction. If estimating wall permeability is not possible, or not considered justified for a particular case, the Eurocode recommends that c_{pi} should be taken as the more onerous of +0.2 and –0.3.

If all four walls of a rectangular building are equally permeable, the relationship between the building geometry and the internal pressure coefficient is shown in Figure 4.1.

For orthodox completely clad portal frame buildings, Figure 4.1 shows that the highest internal suction coefficient approaches –0.3. which led to the recommendation in BRE Digest 436^[47] that for this type of structure, an internal pressure coefficient of –0.3 is appropriate.

Unless the building is square ($d/b = 1$), Figure 4.1 can be used to determine different internal pressure coefficients for the two orthogonal wind directions. The internal pressure coefficients

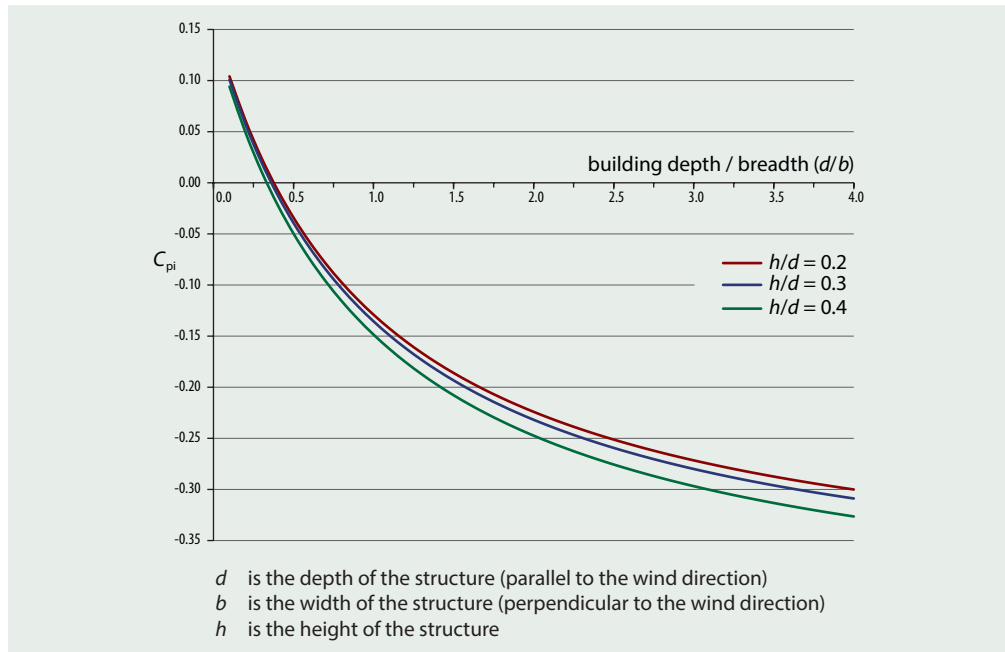


Figure 4.1
Internal pressure coefficient for rectangular buildings with walls of equal permeability

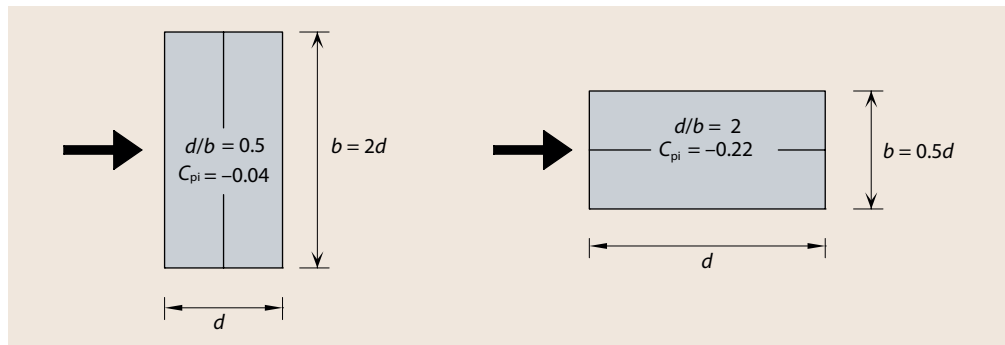


Figure 4.2
Internal pressure coefficients for a rectangular building with walls of equal permeability

appropriate for each orthogonal wind direction for a building with one plan dimension twice that of the other are shown in Figure 4.2, taking h/d as 0.2, which is a typical value.

Dominant openings

Where there is a dominant opening, the internal pressure or suction can be as high as 75% or 90% of the C_{pe} value at the opening, depending on the size of the opening compared to the openings in the other faces. The designer must decide if the openings might be open during a severe storm, or if it is reasonable to assume that the openings will be shut. If the openings are assumed to be shut, an accidental design situation must be considered, with the dominant opening being open. Common practice in the UK is to carry out this second verification with a probability factor c_{prob} applied to the basic wind velocity. Practice is to use a c_{prob} factor of 0.8, which leads to a reduced peak velocity pressure of 0.64 of the original pressure. The use of a c_{prob} factor of 0.8 presumes that procedures will be in place to ensure the openings are closed in a severe storm.

Structural factor

When calculating wind forces, a structural factor $c_s c_d$ may be applied. For low-rise steel buildings, this may be conservatively taken as 1.0. According to the UK NA, a factor of

less than unity may be determined by considering c_s and c_d separately. The $c_s c_d$ factor can only be applied when determining overall force coefficients and external pressure coefficients; it is not to be applied when determining internal pressures.

Calculation of wind forces

For the calculation of overall loads using pressure coefficients, the National Annex allows a factor of 0.85 (from Clause 7.2.2(3) of BS EN 1991-1-4) to be applied to all the horizontal force components for both walls and roof, due to the lack of correlation between the maximum forces calculated for the windward and leeward faces. When the wind is considered blowing parallel to the apex, no reduction should be applied to the frame under consideration.

4.3 Crane loads

Cranes impose both vertical and horizontal loads on the structure. Consideration should be given to the following loads:

- vertical actions, comprising the self weight of the crane bridge, crab, hook plus the lifted load;
- horizontal actions due to crane surge and crabbing.

More information on crane loading is given in *Eurocode load combinations for steel structures*^[18].

4.4 Equivalent horizontal forces

Equivalent horizontal forces (EHF) are not strictly actions, but are forces applied to the frame, in combination with other actions, to model the effect of frame imperfections. The simple approach of applying the EHF is recommended in this publication – the alternative is to model the frame out-of-plumb.

The Eurocode distinguishes the following types of imperfections to be taken into account:

- global imperfections for frames and bracing systems
- local imperfections for individual members.

Most member imperfections are incorporated within the formulae given for member buckling resistance in Section 6.3 of BS EN 1993-1-1. However, according to Clause 5.3.2(6) of BS EN 1993-1-1, if frames are sensitive to second-order effects (see Section 6.3.3 of this publication), member imperfections must be modelled in the analysis model if the member has a moment-resisting joint, at least at one end of the member, and the non-dimensional slenderness $\bar{\lambda}$ exceeds a limiting value. The limiting slenderness is given by:

$$\bar{\lambda} > 0.5 \sqrt{\frac{Af_y}{N_{Ed}}}$$

where:

- A is the area of the section
- N_{Ed} is the design value of the compression force
- $\bar{\lambda}$ is the in-plane non-dimensional slenderness calculated for the member considered as hinged at its ends.

More conveniently the limit may be expressed as $N_{Ed} > 0.25N_{cr}$

where:

- $N_{cr} = \frac{\pi^2 EI_y}{L^2}$
- I_y is the second moment of area of the section, about the major axis
- L is the length of the member.

Determination of the EHF

The magnitudes of the equivalent horizontal forces (EHF) are based on the initial sway imperfection ϕ , given by Equation 5.5 of BS EN 1993-1-1 as:

$$\phi = \phi_0 \alpha_h \alpha_m$$

where:

- $\phi_0 = 1/200$
- α_h is the reduction factor for height h applicable to columns: $= \frac{2}{\sqrt{h}}$
but $\frac{2}{3} \leq \alpha_h \leq 1.0$
- h is the height of the structure in meters. For a single-span portal frame, h should be taken as the height of the columns
- $\alpha_m = \sqrt{0.5(1 + 1/m)}$
- m is the number of vertical members contributing to the horizontal force on the bracing system. For single span portal frames $m = 2$.

The equivalent horizontal force at the top of each column can be determined as ϕN_{Ed} , in which:

- ϕ is the initial sway imperfection
- N_{Ed} is the design value of the compression force in the column; for a single-span portal frame it is equal to the design value of the vertical reaction at the column base.

For a single span portal frame, equivalent horizontal forces should be applied in the same direction at the top of each column. The most onerous direction should be considered, normally in the same direction as the wind actions.

The initial sway imperfections may be disregarded if:

$$H_{Ed} \geq 0.15V_{Ed}$$

where:

H_{Ed} is the design value of the horizontal loads
 V_{Ed} is the design value of the vertical loads.

It is possible that frame imperfections need not be included in combinations including wind actions. This can be assessed by comparing the net vertical loads with the net horizontal loads (wind actions generally lead to asymmetric horizontal and vertical reactions at the bases). It is conservative to simply include the EHF in all combinations – their impact is modest in portal frames, because the vertical loads are generally relatively small. For portal frames supporting mezzanine floors or cranes, the impact of the EHF will be more significant.

4.5 Accidental actions

Rules for accidental actions and accidental design situations are given in BS EN 1991-1-7^[19]. The three accidental actions that may need to be considered are:

- Fire (see Section 4.5.1)
- Drifted snow (see Section 4.2.2)
- The opening of a dominant opening that was assumed to be shut at ULS (see Section 4.2.3).

Each project should be individually assessed to determine whether any other accidental actions need to be considered.

4.5.1 Actions due to fire

Fire is considered as an accidental design situation, resulting in indirect actions (such as forces and moments caused by expansion) and modification of material properties. The UK Building Regulations state that a member supporting only a roof is excluded from requirements for fire resistances, so actions due to fire do not normally need to be evaluated. However, special provisions are generally required when structures are close to a boundary, to prevent fire spread. These requirements are given in Building Regulations, rather than the Eurocodes.

When the building is close to the boundary, there are several requirements aimed at stopping fire spread by keeping the boundary intact:

- the use of fire resistant cladding
- application of fire protection to the steel up to the underside of the haunch
- the provision of moment-resisting bases (as it is assumed that in the fire condition rafters go into catenary).

Comprehensive advice is available in SCI publication P313^[20].

4.5.2 Impact

The structure should be protected from impact loads, so consideration of these loads will not normally form part of the design of the portal frame.

4.5.3 Robustness

Robustness is the ability of a structure to withstand events like fire, explosion, impact or the consequence of human error, without being damaged to an extent disproportionate to the original cause.

BS EN 1990 sets the requirement to design and construct robust buildings in order to avoid disproportionate collapse under accidental design situations. BS EN 1991-1-7^[19] gives details of how this requirement should be met.

The design calculations for robustness are carried out separately from those for the normal verification, and substantial permanent deformation is acceptable for the accidental design situation. Generally, the resistances of elements and connection components are based on ultimate strength, rather than yield strength.

In the UK, official guidance documents are published to explain how to achieve the requirement for robustness in a structure. These documents apply as follows:

- In England and Wales – Building Regulations 2000: Approved Document A^[21].
- In Scotland - The Scottish Building Standards Agency (SBSA) Technical Handbooks^[22].
- In Northern Ireland - The Building Regulations (Northern Ireland), Technical Booklet D^[23].

The strategy to be adopted depends on the classification of the structure, as defined in the Building Regulations and Annex A of BS EN 1991-1-7. Portal frames are usually structures of Consequence Class 1 (agricultural buildings) or 2a (most non-agricultural portal frame structures). If a portal frame structure is open to significant numbers of the public and greater than 5000 m², it should be considered a Class 3 structure.

CONSEQUENCE CLASS	BUILDING TYPE & OCCUPANCY	REQUIREMENT	COMMENT
1	Agricultural buildings	No specified requirements, but minimum 75 kN tying force is recommended for all connections	Achieved by all orthodox connections
2a	Industrial buildings, public access but < 2000 m ²	Horizontal ties	Tying force depending on the vertical loading on the member, but not less than 75 kN
2b	Public access, < 5000 m ²	Horizontal ties and vertical ties	Tying force depending on the vertical loading on the member, but not less than 75 kN (the additional rules for vertical tying are not applicable for single storey portal frames)
3	Public access, > 5000 m ²	Risk Assessment	As class 2b, but with a risk assessment. No changes to connections, but possible outcomes might include impact protection, or additional redundancy

Table 4.4
Summary of
robustness
requirements

Design guidelines and recommended practice on the issues connected with robustness are given in SCI Publication P391^[24]. The practical application of the rules to portal frames is summarised in Table 4.4. The calculated tying forces in the direction of the portal frames will be easily accommodated by the normal connections in a portal frame. In the longitudinal direction, there is no vertical load on the eaves strut, so the design tying force will be 75 kN – easily accommodated by orthodox members and connections. Class 3 structures demand a risk assessment, which may result in changes to the structure to introduce more redundancy, or to protect vulnerable members, for example. Class 2b and Class 3 structures should be provided with more than one set of vertical bracing, to provide a degree of redundancy.

4.6 Combinations of actions

BS EN 1990 gives rules for establishing combinations of actions. Values of partial factors and combination factors to be used in the UK are given in the UK National Annex. BS EN 1990 gives expressions for the effects of combined actions for both the ultimate limit state (ULS) and the serviceability limit state (SLS). For the SLS, onward reference is made to the material parts of the Eurocodes (for example BS EN 1993-1-1 for steelwork), to identify which expression should be used and the SLS limits that should be observed.

All combinations of actions that can occur together should be considered. If certain actions cannot be applied simultaneously, they should not be combined.

Ultimate limit state combinations

For persistent (referring to conditions of normal use) or transient (e.g. during execution or repair) design situations, the UK NA allows the designer to use the STR set of expressions to establish the ULS forces and moments for member verification. Expression 6.10 or the less favourable of 6.10a and 6.10b may be used, as shown in Table 4.5. Taking the less favourable of expressions 6.10a and 6.10b will generally result in the most economic solution, compared to the use of 6.10, although for a roof, the advantage may not be significant.

EXPRESSION	PERMANENT ACTIONS		VARIABLE ACTIONS	
	UNFAVOURABLE	FAVOURABLE	LEADING	ACCOMPANYING
6.10	$\gamma_{Gj,sup} G_{kj,sup}$	$\gamma_{Gj,inf} G_{kj,inf}$	$\gamma_{Q,1} Q_{k,1}$	$\gamma_{Q,i} \psi_{0,i} Q_{k,i}$
6.10a	$\gamma_{Gj,sup} G_{kj,sup}$	$\gamma_{Gj,inf} G_{kj,inf}$	$\gamma_{Q,i} \psi_{0,i} Q_{k,i}$	$\gamma_{Q,i} \psi_{0,i} Q_{k,i}$
6.10b	$\xi \gamma_{Gj,sup} G_{kj,sup}$	$\gamma_{Gj,inf} G_{kj,inf}$	$\gamma_{Q,1} Q_{k,1}$	$\gamma_{Q,i} \psi_{0,i} Q_{k,i}$

Table 4.5
Design values of
actions for persistent
or transient design
situations (taken from
BS EN 1990)

For structures in the UK, the following values of partial factors and combination factors, taken from the UK NA, should be used:

$\gamma_{Gj,sup}$	= 1.35	partial factor for unfavourable permanent actions
$\gamma_{Gj,inf}$	= 1.0	partial factor for favourable permanent actions
$\gamma_{Qj,sup}$	= 1.5	partial factor for unfavourable variable actions
$\gamma_{Qj,inf}$	= 0	partial factor for favourable variable actions
ψ_0	= 0.5	combination factor for wind actions
ψ_0	= 0.5	combination factor for snow loads for site altitude below 1000 m above sea level
ψ_0	= 0.7	combination factor for imposed roof loads
ξ	= 0.925	

The factors applied to the characteristic values of actions for combinations at the Ultimate Limit State, based on equation 6.10 and using the values given in the UK NA, are shown in Table 4.6.

ACTIONS	PERMANENT	IMPOSED	SNOW	WIND	WIND UPLIFT	EHF
factors for combinations of actions	1.35	1.5				To be included
	1.35		1.5			To be included
	1.35		1.5	0.5×1.5 ($\gamma_Q \times \psi_0$)		*
	1.35		0.5×1.5 ($\gamma_Q \times \psi_0$)	1.5		*
	1.0				1.5	*

Table 4.6
Factors for design combinations at ULS

Note: * indicates that EHF may not need to be included if $H_{Ed} \geq 0.15 V_{Ed}$. Since the EHF are a proportion of the ultimate loads, no additional factor is required.

Note that, in accordance with BS EN 1991-1-1, imposed roof loads are not considered in combination with either wind actions or snow loads. Snow loads and wind actions are considered in combination, with each action in turn as the leading variable action.

Guidance on combinations of actions including cranes is given in *Eurocode load combinations for steel structures*^[18].

Serviceability limit state combinations

BS EN 1990 defines three combinations of actions for serviceability limit states; the characteristic combination, the frequent combination and the quasi-permanent combination. The UK NA to BS EN 1993-1-1 recommends the use of the characteristic combination of actions when checking SLS, and that permanent actions need not be

included. Therefore, in the UK the following expression can be used to determine the effects of SLS combinations of actions:

$$Q_{k,1} "+" \sum_{i>1} \psi_{0,i} Q_{k,i}$$

(where "+" means 'combined with')

Detailed consideration of serviceability aspects of frame design is given in Section 13.

INITIAL DESIGN

The following overall design requirements should be considered at the initial design stage of the structure, depending on the building form and use:

- Space use, for example, specific requirements for handling of materials or components in a production facility
- Accommodation of loading doors and loading docks
- Flexibility of space in current and future use
- Future expansion
- Speed of construction
- Environmental performance, including services requirements, air tightness and thermal performance
- Aesthetics and visual impact
- Acoustic insulation, particularly in production facilities
- Access and security
- Sustainability considerations
- Design life and maintenance requirements, including end of life issues.

5.1 Building layout

The determination of the overall height and width of the frame is critical to give adequate clear internal dimensions and adequate clearance for the internal functions of the building. Precise dimensions can only be determined by carrying out a preliminary design to determine member sizes. Guidance on preliminary sizing of members is given in Appendix A. It is unlikely that modest conservatism in the analysis model will be detrimental, so precise dimensions are generally not critical.

Although frames can be spaced to suit particular building features, regular spacing is generally considered to have advantages:

- The loading on each frame is identical, and only one frame design is required
- The stiffnesses (and therefore deflections) of the frames are the same
- The foundation setting out is regular
- Longitudinal members, such as the purlins, eaves strut, etc are all regular.

5.1.1 Clear span and height

The clear span and height required by the client are key to determining the dimensions to be used in design, and should be established clearly at the outset. The critical client

requirement is likely to be the clear distance between the flanges of the opposing columns – the structural span will therefore be larger, by the column section depth. Any requirements for brickwork or blockwork around the columns should be established, as this may also affect the design span.

Where a clear internal height is specified, this will usually be measured from the finished floor level to the underside of the haunch or suspended ceiling.

The calculation of the height to eaves (the intersection of the rafter and column centre lines) for analysis should allow for:

- The distance from the top of the foundation to the finished floor level.
- The specified clear internal height.
- The requirements for any ceiling below the lowest point of the haunch.
- The requirements for any services below the lowest point of the haunch.
- The depth of the haunch.
- Half the depth of the rafter (calculated vertically).

5.1.2 Haunch dimensions

The depth of the haunch is often defined differently, depending on the context:

- For some software, the haunch depth is defined as the vertical distance from the intersection point of the centre-line of the rafter and the column to the bottom of the haunch at the end plate (Figure 5.1).
- Steelwork contractors generally specify the cutting depth of the haunch as the depth from the underside of the rafter to the bottom of the haunch (Figure 5.1)
- In some instances, the haunch depth is referred to as the depth from the top of the rafter to the bottom of the haunch.

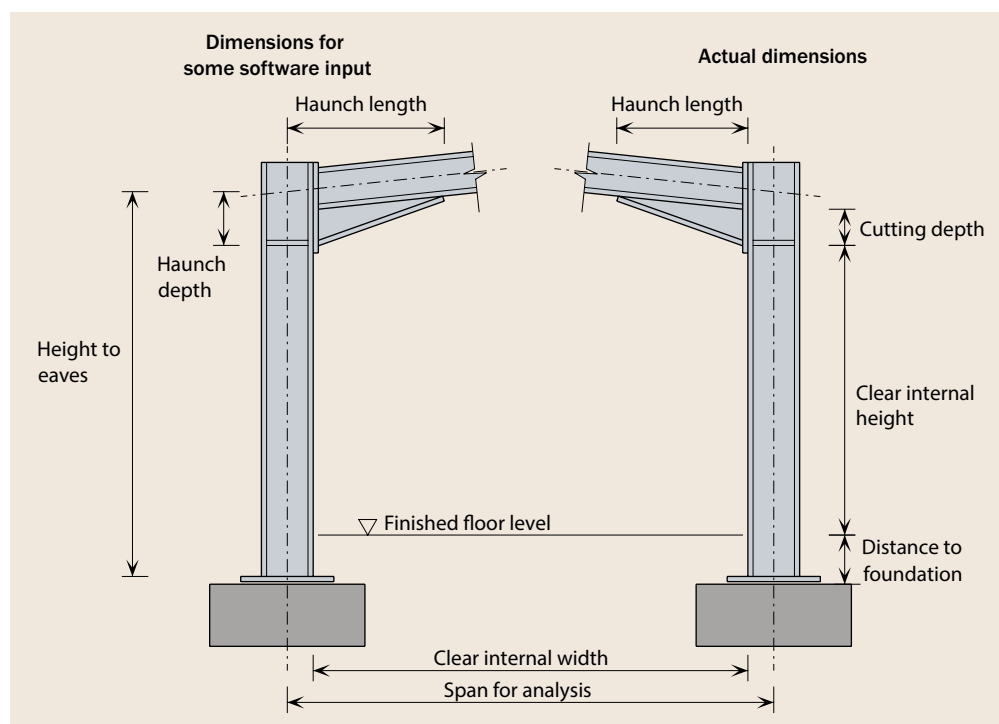


Figure 5.1
Dimensions used for
analysis and clear
internal dimensions

Similarly, the length of the haunch may be defined either from the centre-line or from the face of the column. The haunch length measured horizontally from the column centre-line to the end of the tapered section is usually chosen to be 10% of the portal span. This length means that in elastic design the hogging bending moment at the “sharp” end of the haunch is approximately the same as the maximum sagging bending moment towards the apex, as shown in Figure 5.2.

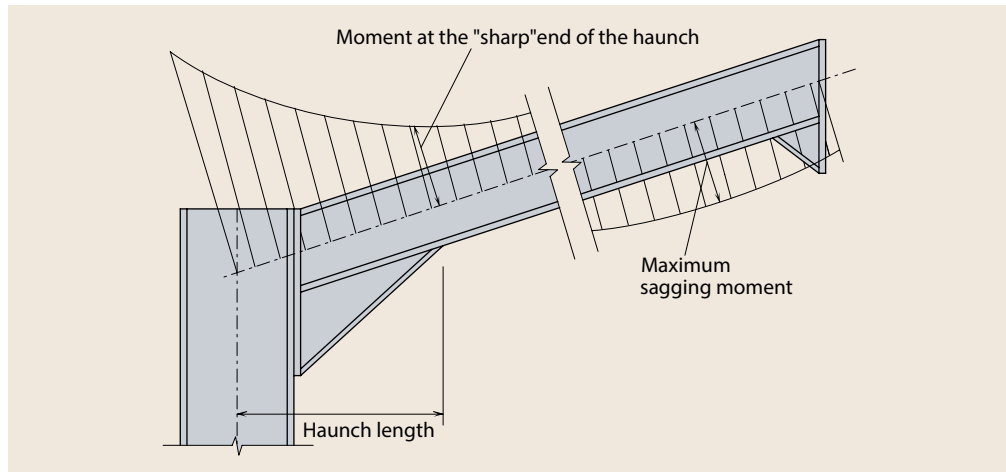


Figure 5.2
Rafter bending
moment and
haunch length

Positions of restraints

During initial design, the rafter members are normally selected according to their cross sectional resistance to bending and axial force. In later design stages, stability against buckling needs to be verified and restraints positioned judiciously (Figure 5.3).

The initial selection of a column section is likely to be based on its buckling resistance, rather than its cross sectional resistance. Compared to a rafter, there is usually less freedom to position rails to restrain buckling, as rail positions may be dictated by doors or windows in the elevation.

If the provision of sufficient intermediate restraints to the column is not possible, the buckling resistance will determine the initial section size selection. It is therefore essential to determine at this early stage whether the side rails can be used to provide restraint to the columns. Only continuous side rails are effective in providing restraint. Side rails interrupted by (for example) roller shutter doors cannot be relied on to provide adequate restraint, unless additional bracing is provided.

Where the compression flange of the rafter or column is not restrained by purlins or side rails, restraint can be provided at specific locations by column and rafter stays, as shown in Figure 5.3.

Further advice on the positioning of restraints is given in Section 7.2 of this publication.

5.1.3 Steel grade and sub-grade

S355 material is usually selected, as the higher strength material offers strength:cost benefits compared to S275 steel.

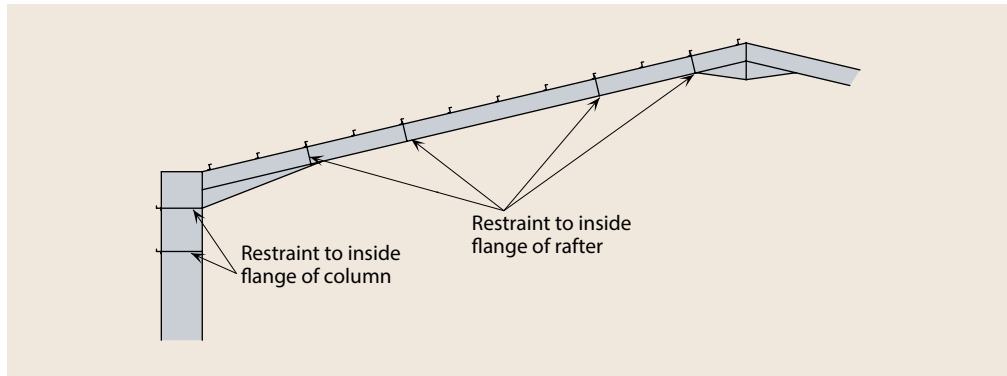


Figure 5.3
General arrangement
of restraints to the
inside flange

Steel sub-grade would normally be selected in accordance with BS EN 1993-1-10^[25], although the use of PD 6695-1-10^[26] is strongly recommended as a simpler route. Within a portal frame, details will generally fall under the category of “welded generally”. Care should be taken if the steel is exposed to low temperatures (either externally, or in a refrigerated warehouse, for example), when a tougher sub-grade will probably be required.

Selection of a steel sub-grade in accordance with the Eurocodes is covered in SCI Document ED007^[27].

5.2 Preliminary analysis

Although efficient portal frame analysis and design will use bespoke software, which is likely to be using elastic-plastic analysis, preliminary manual elastic analysis is simple. In most circumstances, a reasonable estimate of the maximum bending moments will be obtained by considering only the vertical loads. Appropriate sections can then be chosen on the basis of this analysis.

For the preliminary analysis, it is common to assume that the second moment of area of the column is 1.5 times that of the rafter section.

For the pinned base frame shown in Figure 5.4, the bending moment at the eaves, M_E , and at the apex, M_A , can be calculated as follows:

$$M_E = \frac{wL^2(3+5m)}{16N} \quad \text{and} \quad M_A = \frac{wL^2}{8} + m \times M_E$$

where:

$$\begin{aligned} N &= B + mC \\ C &= 1 + 2m \\ B &= 2(k + 1) + m \\ m &= 1 + \phi \\ \phi &= f/h \\ k &= \frac{I_R}{I_C} \frac{h}{s} \end{aligned}$$

As noted above, it may be assumed for preliminary analysis that $I_C = 1.5 \times I_R$

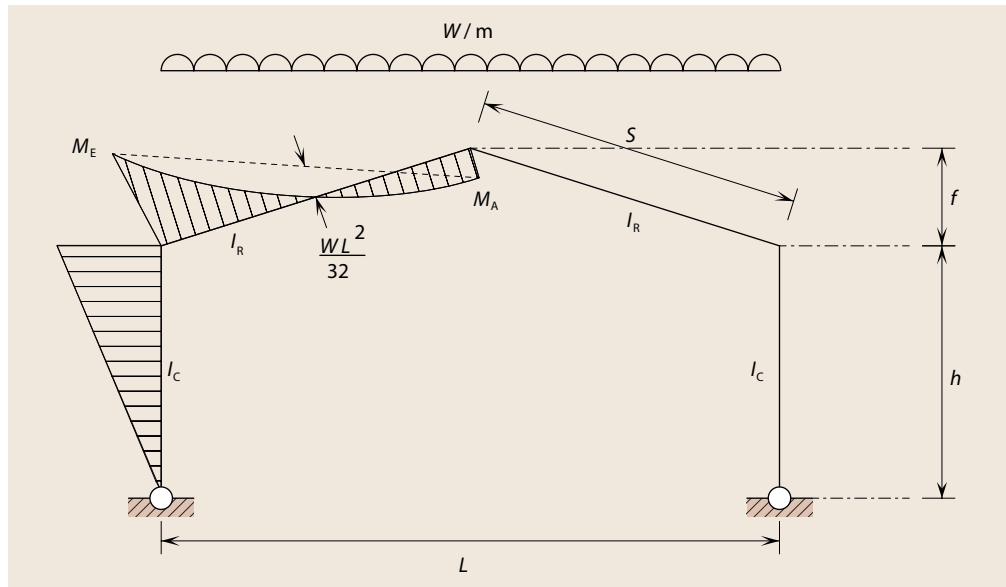


Figure 5.4
Details of a pinned
base portal frame

Second-order effects

It is likely that many economic frames will be sensitive to second-order effects (see Section 6.3), which are likely to increase the design moments by up to 15%. If undertaking a preliminary analysis, bending moments from a first-order analysis should be amplified to allow for these second-order effects.

5.2.2 Selection of members

Because the primary effect in a portal frame is bending, UKB sections are invariably selected for rafters and columns. The larger second moment of area of a UKB (compared to a UKC) also helps to control deflections.

The rafter should be selected such that its cross sectional resistance $M_{c,y,Rd}$ exceeds the maximum design moment, which will be at the sharp end of the haunch, or near the apex. Generally, it will be possible to introduce sufficient restraints that the lateral torsional buckling resistance of the rafter is not critical. If a more detailed check is warranted at the initial design stage, the lateral torsional buckling resistance may be taken as the value of $M_{b,Rd}$ over a length equal to the purlin spacing (say 2000 mm maximum). When calculating $M_{b,Rd}$ it may be conservatively assumed that the bending moment is uniform (i.e. $C_1 = 1.0$). The lateral torsional buckling resistance may be obtained from SCI publication P363^[28].

The column should be selected such that its cross sectional resistance, $M_{c,y,Rd}$ is at least equal to the moment at the underside of the haunch. In addition, the lateral torsional buckling resistance between restraints must exceed the applied moment. The lateral torsional buckling resistance is almost certainly the critical check. If no restraints are assumed (or none can be utilised) between the underside of the haunch and the base, a value of $C_1 = 1.77$ (for a triangular bending moment diagram) may be assumed when calculating $M_{b,Rd}$ over the column height. If intermediate restraints are utilised, a value of $C_1 = 1.1$ for the length between restraints is a reasonable initial assumption.

FRAME ANALYSIS AT THE ULTIMATE LIMIT STATE

Methods of frame analysis at the ultimate limit state fall broadly into two types – elastic analysis and plastic analysis. The latter term covers both rigid-plastic and elastic-plastic analysis.

Elastic analysis is the most common method of analysis for general structures, but will usually give less economical designs for portal structures than plastic analysis. It is not uncommon to perform an initial analysis using elastic analysis.

Most bespoke software for portal frame analysis will carry out an elastic-plastic analysis.

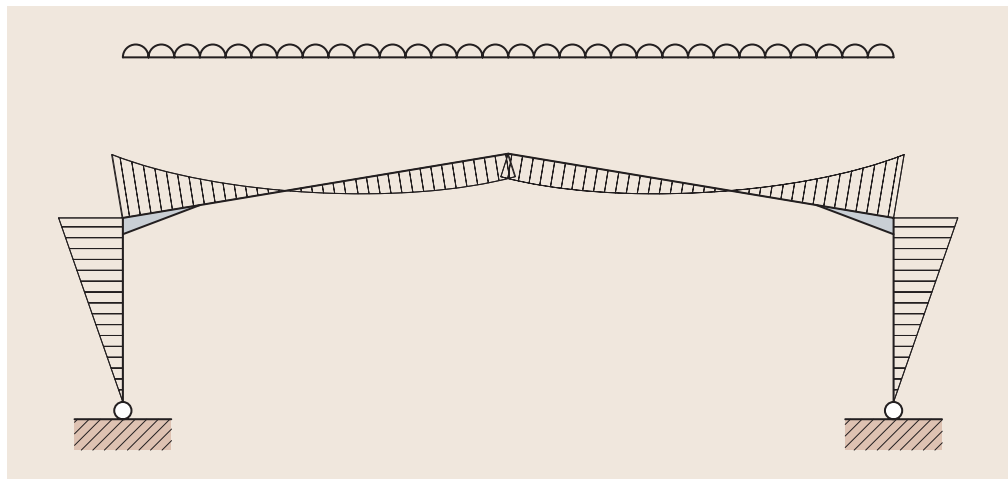
For completeness, both types of analysis are described below but, as noted in Section 1.1, plastic analysis is outside the scope of this publication.

6.1 Elastic frame analysis

A typical bending moment diagram resulting from an elastic analysis of a frame with pinned bases is shown in Figure 6.1. The haunch length is chosen so that under predominantly gravity combinations of actions the hogging bending moment at the end of the haunch is approximately equal to the sagging bending moment adjacent to the apex.

BS EN 1993-1-1 allows the plastic cross sectional resistance to be used with the results of elastic frame analysis, provided the section is Class 1 or Class 2. In addition, for Class 1 and 2 sections, the Standard allows 15% of the maximum moment to be redistributed as defined in BS EN 1993-1-1 Clause 5.4.1.4(B). In practice, the redistribution from the initial elastic analysis is rarely used in steel design.

*Figure 6.1
Bending moment
diagram resulting
from the elastic
analysis of a
symmetrical portal
frame under
symmetrical loading*



6.2 Plastic frame analysis

Plastic analysis generally results in a more economical frame than an elastic analysis because it allows relatively large redistribution of bending moments throughout the frame, due to plastic hinge rotations. This redistribution 'relieves' the highly stressed regions and allow the capacity of under-utilised parts of the frame to be mobilised.

Plastic hinge rotations occur at sections where the bending moment reaches the plastic moment of resistance of the cross section at load levels below the full ULS loading.

An idealised 'plastic' bending moment diagram for a symmetrical portal under symmetrical vertical loads is shown in Figure 6.2. The potential positions of the plastic hinges for the plastic collapse mechanism are shown. The first hinge to form is normally adjacent to the haunch (shown in the column in this case). At higher load levels, depending on the proportions of the portal frame, hinges form just below the apex, at the point of maximum sagging moment.

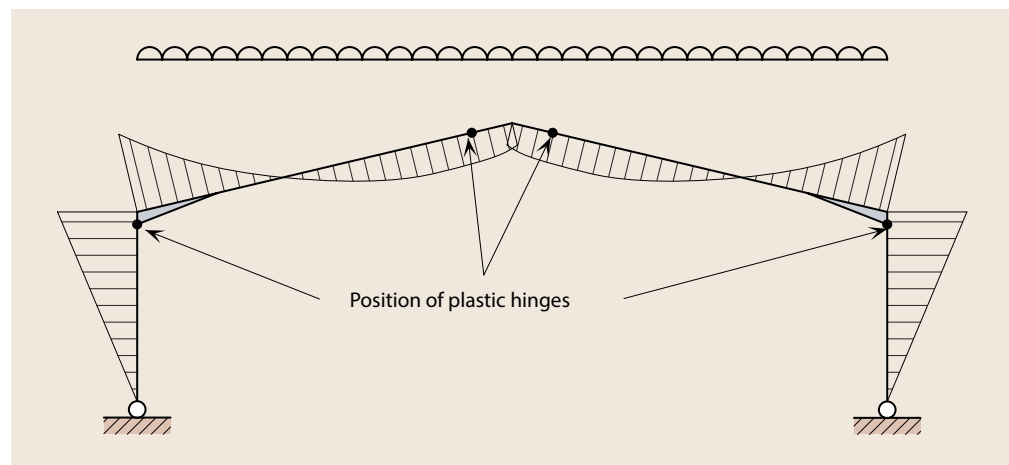


Figure 6.2
Bending moment
diagram resulting from
the plastic analysis
of a symmetrical
portal frame under
symmetrical loading

A portal frame with pinned bases has a single degree of indeterminacy. Therefore, two hinges are required to create a mechanism. The four hinges shown in Figure 6.2 only arise because of symmetry. In practice, due to variations in material strength and section size, only one apex hinge and one eaves hinge will form to create the mechanism.

As there is uncertainty as to which hinges will form in the real structure, a symmetrical arrangement is assumed, and hinge positions on each side of the frame restrained.

Where deflections (at SLS) govern design, there may be no advantage in using plastic analysis for the ULS. If stiffer sections are selected in order to control deflections, it is quite possible that no plastic hinges form and the frame remains elastic at ULS.

BS EN 1993-1-1 contains guidance on the verification of segments containing plastic hinges, stable lengths adjacent to plastic hinges, the necessary restraint at plastic hinges and limitations on member classification when carrying out plastic analysis and design.

At the time of writing (2012), the detailed application of the in-plane buckling checks to members containing plastic hinges is unclear. Detailed guidance on plastically designed portal frames is therefore outside the scope of this present publication.

6.3 First and second-order elastic analysis

The significance of second-order effects must always be considered, and such effects allowed for if necessary. Although the term second-order effects can cover a range of effects, such as those due to residual stresses, material non-linearity etc, the main concern here is the deformation of the structure. Other second-order effects are allowed for in the member resistance calculations. If second-order effects are small, a first-order analysis is sufficiently accurate. If second-order effects are significant, they need to be taken into account, either by second-order analysis or by modifying (amplifying) first-order effects.

6.3.1 Frame behaviour

When a portal frame is loaded, it deflects: its shape under load is different from the undeformed shape. The deflection has a number of effects:

- The vertical forces at the tops of the columns are eccentric to the bases, which leads to further deflection
- The apex drops, reducing the arching action
- The vertical actions cause bending in the rafters; axial compression in curved members causes increased curvature. Increased curvature can be considered as a symptom of reduced stiffness.

Taken together, these effects mean that a frame is less stable (nearer collapse) than a first-order analysis suggests. The first step in assessing frame stability is to determine whether the difference between the results of a first and second-order analysis are significant.

The deflected shape of a portal frame under combined vertical and horizontal loads is shown diagrammatically in Figure 6.3.

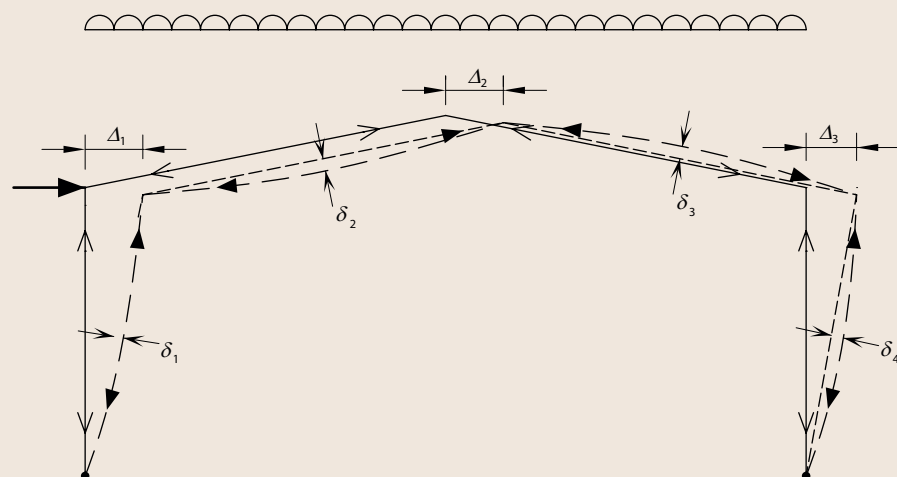


Figure 6.3
Asymmetric or sway
mode deflection

As shown in Figure 6.3, when considering the effects of deformed geometry, there are two categories of second-order effects:

- Effects of displacements of the intersections of members, usually called $P-\Delta$ effects.
- Effects of deflections within the length of members, usually called $P-\delta$ effects.

$P-\delta$ effects arise from two different causes:

- Bending due to external actions, which curve the member
- Curvature due to initial member imperfections.

6.3.2 Analysis and design approaches in BS EN 1993-1-1

Clause 5.2.2 of BS EN 1993-1-1 recognises a number of alternative ways of allowing for second-order effects and imperfections. One approach is to account for all effects within the frame analysis. This would mean carrying out a second-order analysis that would include $P-\Delta$ and $P-\delta$ effects and account for the initial imperfections (both in members and the frame itself). If all these effects are accounted for in the analysis, no member buckling checks are necessary, only verification of cross sectional resistances. This approach is discussed further in Section 7.

A second approach is to allow for second-order effects and the initial imperfection of the frame in the frame analysis, and then allow for the effects of initial imperfections in the members by verifying the buckling resistance of members.

Initial member imperfections are automatically allowed for (together with residual stresses and other second-order effects) if the member design is carried out in accordance with Section 6.3 of BS EN 1993-1-1.

Allowing for second-order effects does not mean completing a second-order analysis, although this will often be the preferred option if using software for analysis and design. Second-order effects can also be allowed for by amplifying the results of a first order analysis, as described in Section 6.7 of this publication.

6.3.3 Assessing the significance of second-order effects

The second-order effects due to the deformed geometry are assessed in BS EN 1993-1-1 by calculating the factor α_{cr} , defined as:

$$\alpha_{cr} = \frac{F_{cr}}{F_{Ed}}$$

where:

F_{cr} is the elastic critical buckling load for global instability mode, based on initial elastic stiffnesses

F_{Ed} is the design load on the structure.

The value of α_{cr} may be found using software or, as long as the frame meets certain geometric limits and the axial force in the rafter is not “significant”, by using the

approximation given by Expression 5.2 from BS EN 1993-1-1. Rules are given in Clause 5.2 to identify when the axial force is significant.

When the frame falls outside the specified limits, as is the case for very many orthodox portal frames, the simplified expression cannot be used. In these circumstances, an alternative expression has been developed^[29] to calculate an approximate value of α_{cr} , referred to as $\alpha_{cr,est}$. Further details are given in Section 6.6.

According to Clause 5.2.1(3) of BS EN 1993-1-1, the effects of deformed geometry can be neglected (and a first-order analysis used without modification) if α_{cr} is above certain limits.

For elastic analysis, the effects of deformed geometry can be neglected if $\alpha_{cr} \geq 10$.

6.4 Base stiffness

Unless bases are detailed as truly pinned joints, the bases will possess some degree of rotational stiffness. In many cases, the stiffness is small and the bases can be classed as nominally pinned. UK practice is to consider typical portal frame base details with four bolts as nominally pinned.

Benefit may be taken of the stiffness of nominally pinned bases to reduce frame deflections at SLS and to reduce the effects of deformed geometry (improve frame stability), manifest in a higher value of α_{cr} . When nominal base stiffness is assumed at SLS, and when calculating α_{cr} , the resulting moments at the base are ignored.

If any base stiffness is assumed at ULS, the base details and foundation must be designed to have sufficient resistance to sustain the calculated moments and forces. Moment-resisting foundations are generally unwelcome in the UK, due to high cost induced by more complicated design, larger size and increase in reinforcement compared to a simple foundation.

The recommended approach for orthodox portal frame construction with nominally pinned bases is:

- Carry out an analysis at ULS assuming perfect pins at the bases, to avoid the requirement to transfer bending moment through the base into the foundation.
- Assess SLS deflections utilising the benefit of the nominal base stiffness (see Section 6.4.2 below), ignoring moments that arise at the base.
- Assess frame stability utilising the benefit of nominal base stiffness (see Section 6.4.2 below), ignoring moments that arise at the base.

Pinned bases

Truly pinned bases are not used in orthodox portal frame construction. If such bases are used for architectural reasons, the rotational stiffness is effectively zero. Where they are adopted, careful consideration needs to be given to the temporary stability of the column during erection.

Rigid column bases

Because rigid column bases are extremely uncommon in the UK, no recommendations for design stiffness for rigid bases are given in this publication. Details can be obtained from non-contradictory complementary information (NCCI)^[30].

Nominally pinned bases

If a column base is nominally pinned, it is recommended that the base be modelled as a simple joint (perfectly pinned) when using elastic global analysis to calculate the moments and forces in the frame under ULS loading. A nominally pinned base is illustrated in Figure 6.4. Although the bolts are outside the profile of the column section, and the column is welded to the baseplate, UK practice is to consider this form of base as nominally pinned.

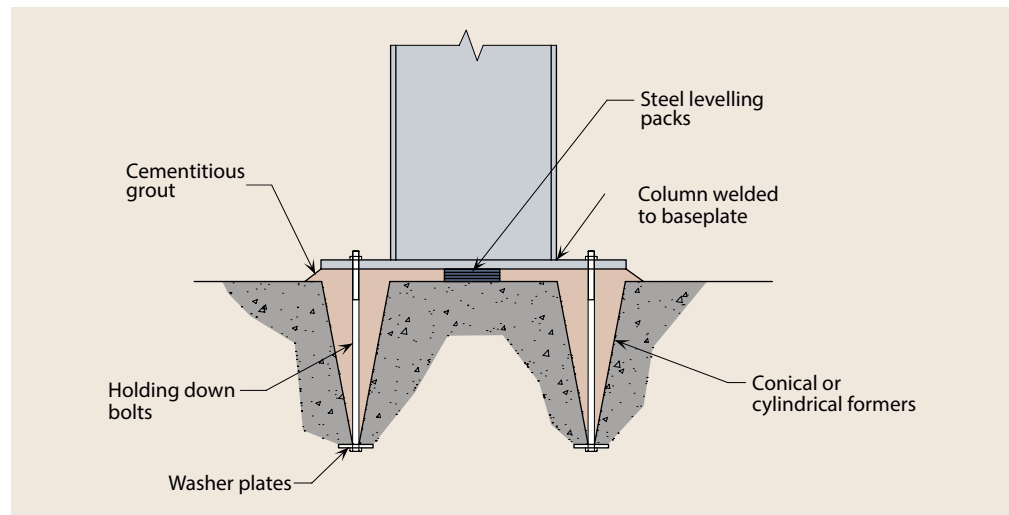


Figure 6.4
Example of a
nominally pinned base

6.4.2 Modelling of nominally pinned bases

For the assessment of frame stability and for the assessment of deflections at SLS, the base may be modelled with a stiffness assumed to be a proportion of the column stiffness, as follows:

- 10% when assessing frame stability (see Section 6.3.3 of this publication)
- 20% when calculating deflections at SLS.

It is conservative to neglect any nominal base stiffness and model the base as perfectly pinned.

Bespoke software normally has the facility to select values of base stiffness. Alternatively, the base stiffness may be modelled by the use of a spring stiffness or dummy members at the column base.

Spring stiffness

10% of the column stiffness may be modelled by using a spring stiffness equal to $0.4EI_{\text{column}}/L_{\text{column}}$.

20% of the column stiffness may be modelled by using a spring stiffness equal to $0.8EI_{\text{column}}/L_{\text{column}}$.

Modelling with dummy base members

If the software cannot accommodate a rotational spring, the base fixity may be modelled by a dummy member of equivalent stiffness, as shown in Figure 6.5.

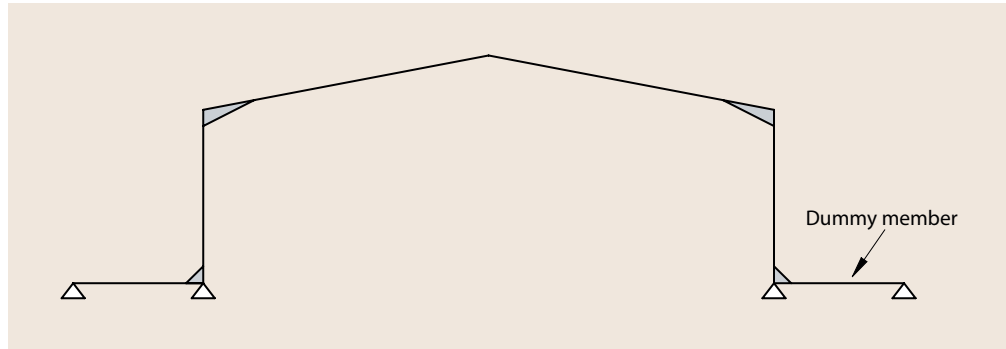


Figure 6.5
Modelling base fixity
by a dummy member

When modelling a nominally pinned base, the second moment of area (I_y) of the *dummy* member should be taken as:

When assessing frame stability: $I_y = 0.1I_{y,\text{column}}$.

When calculating deflections at SLS: $I_y = 0.2I_{y,\text{column}}$.

In both cases, the length of the dummy member is $L = 0.75L_{\text{column}}$, and it is modelled with a pinned support at the extreme end.

Reactions from analysis with the use of dummy members should not be used explicitly, as the provision of an additional support will affect the base reactions. The vertical base reaction should be taken as the axial force at the base of the column.

6.4.3 Fire boundary considerations

When the proximity to a site boundary means that the portal base must be designed to resist an overturning moment in the fire condition, it is normal for the base detail to be considerably strengthened, with a thicker baseplate or in some cases, a small haunch. Although it would appear clear that such a base will exhibit at least semi-continuous behaviour, usual practice in the UK is still to consider the base only as nominally pinned for normal temperature design.

6.5 Evaluation of in-plane frame stability

The stability of frames is expressed by the parameter α_{cr} . According to Clause 5.2.1(4)B, the value of α_{cr} is given by:

$$\alpha_{\text{cr}} = \left(\frac{H_{\text{Ed}}}{V_{\text{Ed}}} \right) \left(\frac{h}{\delta_{\text{H,Ed}}} \right) \quad (5.2)$$

where, for an individual portal frame:

- H_{Ed} is the algebraic sum of the base shear on the two columns – due to the horizontal loads and the EHF
- V_{Ed} is the total design vertical load on the frame – the algebraic sum of the two base reactions
- $\delta_{H,Ed}$ is the maximum horizontal deflection at the top of either column, relative to the base, when the frame is loaded with horizontal loads (e.g. wind) and the EHF
- h is the column height.

Because loads on a pitched roof contribute to the lateral deflection at the top of the portal column, it is recommended that α_{cr} be calculated based on H_{Ed} and $\delta_{H,Ed}$ due only to notional horizontal forces (NHF). The NHF should be taken as 1/200 of the design vertical base reaction, and they should be applied to each column, in the same direction, at eaves level. When following this recommendation, note that the horizontal forces used when determining α_{cr} are NHF, not EHF (the latter may include the effect of α_h and α_m).

The expression for α_{cr} then becomes:

$$\alpha_{cr} = \frac{h}{200\delta_{NHF}}$$

where:

- h is the height to eaves
- δ_{NHF} is the lateral deflection at the top of the column due to the NHF.

Notes 1B and 2B of Clause 5.2.1 limit the application of the expression to roof slopes no steeper than 26° and where the axial force in the rafter is not significant. Axial force in the rafter may be assumed to be significant if

$$\bar{\lambda} \geq 0.3 \sqrt{\frac{Af_y}{N_{Ed}}}$$

where:

- $\bar{\lambda}$ is the non-dimensional slenderness of the rafter pair, for flexural buckling about the major axis. When calculating $\bar{\lambda}$, the buckling length is taken as the developed length of the rafter pair from column to column, taken as $\text{span}/\cos \theta$, where θ is the roof slope.
- A is the cross sectional area of the rafter
- f_y is the yield strength of the rafter
- N_{Ed} is the axial compression in the rafter.

A convenient way to express the limitation on the axial force is that the axial force is not significant if:

$$N_{Ed} \leq 0.09N_{cr}$$

where:

N_{cr} is the elastic critical buckling load for buckling about the major axis for the complete span of the rafter pair, i.e. $N_{cr} = \frac{\pi^2 EI}{L^2}$

L is the developed length of the rafter pair from column to column, taken as span/Cos θ , where θ is the roof slope

I is the second moment of area of the rafter about the major axis (I_{yy})

E is the modulus of elasticity (210000 N/mm²)

If the limits are satisfied, then Expression (5.2), given above, may be used to calculate α_{cr} . In most practical portal frames, the axial load in the rafter will be significant and Expression (5.2) cannot be used.

The benefit of accounting for base stiffness when performing the stability analysis should not be underestimated, as, for a portal frame with nominally pinned bases, nominal base stiffness can increase the value of α_{cr} significantly.

6.6 Evaluation of in-plane stability when the axial force in the rafter is significant

When the axial force in the rafter is significant, a conservative measure of frame stability, defined as $\alpha_{cr,est}$ may be calculated^[29]. More accurate (higher) values of α_{cr} will be obtained from software.

For frames with pitched rafters:

$$\alpha_{cr,est} = \min(\alpha_{cr,s,est}; \alpha_{cr,r,est})$$

where:

$\alpha_{cr,s,est}$ is the estimate of α_{cr} for the sway buckling mode

$\alpha_{cr,r,est}$ is the estimate of α_{cr} for the rafter snap-through buckling mode. This mode only needs to be checked when there are three or more spans, or if the rafter is horizontal, or when the columns are not vertical.

6.6.1 Factor $\alpha_{cr,s,est}$

The value of $\alpha_{cr,s,est}$ is given by:

$$\alpha_{cr,s,est} = 0.8 \left\{ 1 - \left(\frac{N_{Ed}}{N_{cr,R}} \right)_{\max} \right\} \alpha_{cr}$$

where:

$\left(\frac{N_{Ed}}{N_{cr}}\right)_{\max}$ is the maximum ratio in any of the rafters

N_{Ed} , N_{cr} , α_{cr} are as previously defined.

The lowest value of α_{cr} for any column is used for the frame as a whole.

The calculation process is:

1. Complete a frame analysis with pinned bases under the design value of combination of actions to determine the vertical base reactions and the axial compression in the rafter, N_{Ed} (see Figure 6.6).
2. Calculate the values of NHF as 1/200 of the base reactions given by the analysis.
3. Complete a second analysis, with only the NHF on the otherwise unloaded frame, and determine the horizontal deflections δ_{NHF} at the tops of the columns (see Figure 6.7). Utilising the beneficial stiffness of nominally pinned bases given in Section 6.4 for this analysis is recommended.

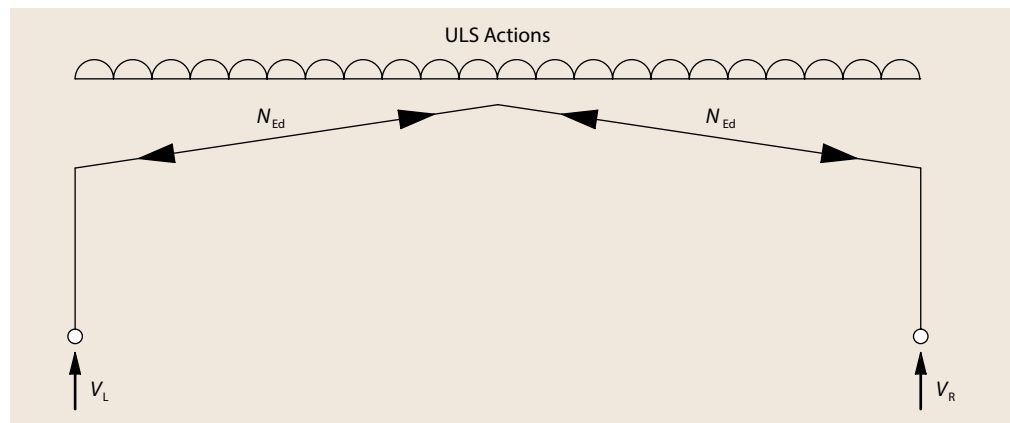


Figure 6.6
Analysis to establish rafter force and base reactions

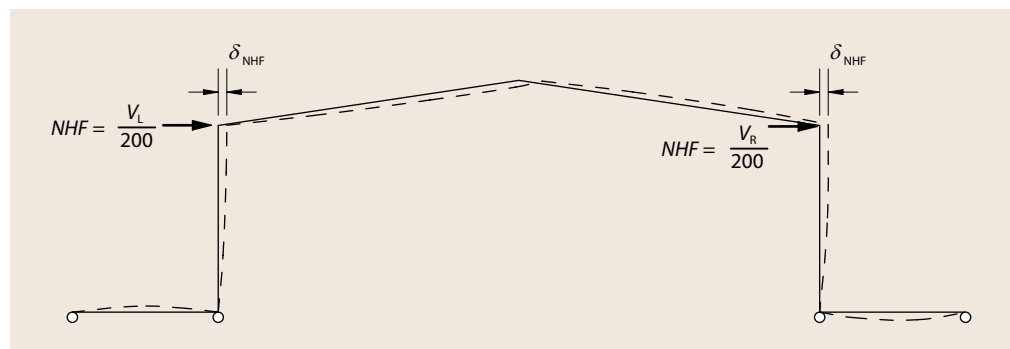


Figure 6.7
Analysis to establish horizontal deflection under NHF

6.6.2 Factor $\alpha_{cr,est}$

For single span portal frames, this calculation should only be carried out if the rafter is horizontal or when the columns are not vertical.

For frames with rafter slopes not steeper than 1:2 (26°), $\alpha_{cr,est}$ may be taken as:

$$\alpha_{cr,est} = \left(\frac{D}{L}\right) \left(\frac{55.7(4+L/h)}{\Omega-1}\right) \left(\frac{I_c + I_r}{I_r}\right) \left(\frac{275}{f_{yr}}\right) (\tan 2\theta_r)$$

But where $\Omega \leq 1$, $\alpha_{cr,est} = \infty$

where:

D	is the cross sectional depth of the rafter, h
L	is the span of the frame
h	is the mean height of the column from base to eaves or valley
I_c	is the in-plane second moment of area of the column (taken as zero if the column is not rigidly connected to the rafter)
I_r	is the in-plane second moment of area of the rafter
f_{yr}	is the nominal yield strength of the rafters in N/mm ²
θ_r	is the roof slope
h_r	is the height of the apex of the roof above the straight line between the tops of the columns
Ω	is the arching ratio, given by $\Omega = W_r / W_0$
W_0	is the value of W_r for plastic failure of the rafters as a fixed ended beam of the span L
W_r	is the total design vertical load on the rafters of a frame.

6.7 Modified first-order analysis

The ‘amplified moment method’ is the simplest method of allowing for second-order effects in a first-order elastic frame analysis; the principle is given in BS EN 1993-1-1, Clause 5.2.2(5B).

A first-order linear elastic analysis is carried out and α_{cr} (or $\alpha_{cr,est}$) determined (see Section 6.3.3). If second-order effects are significant, all horizontal actions are increased by an amplification factor to allow for the second-order effects. The horizontal actions comprise the externally applied actions, such as the wind load, and the equivalent horizontal forces (EHF) used to allow for frame imperfections; both are amplified. Note that the EHF are amplified, not the NHF used to calculate α_{cr} .

Provided $\alpha_{cr} \geq 3.0$ the amplification factor is given by:

$$\left(\frac{1}{1 - 1/\alpha_{cr}} \right)$$

If the axial load in the rafter is significant, and $\alpha_{cr,est}$ has been calculated in accordance with Section 6.6, the amplifier is given by:

$$\left(\frac{1}{1 - 1/\alpha_{cr,est}} \right)$$

If α_{cr} or $\alpha_{cr,est}$ is less than 3.0, second-order analysis must be used; the simple amplification is not sufficiently accurate.

MEMBER VERIFICATION

Once the analysis has been completed, allowing for second-order effects if necessary, the frame members must be verified. In general, both the cross-sectional resistance and member buckling resistance must be verified. Member buckling resistance is often referred to as member stability; the terms are equivalent.

If *all* second-order effects have been allowed for in the frame analysis, only the cross-sectional resistance needs to be verified. As discussed in Section 6.3.1, common UK practice is not to allow for the effects of initial imperfections in the frame analysis. Member verification must therefore include buckling resistance checks in accordance with Section 6.3 of BS EN 1993-1-1.

7.1 Cross-sectional resistance

Member bending, axial and shear resistances must be verified. If the shear or axial force is high, the bending resistance is reduced, so resistance to coexisting shear force and bending moment, and coexisting axial force and bending moment needs to be verified. In typical portal frames, neither the shear force nor the axial load is sufficiently high to reduce the bending resistance. When the portal frame forms the chord of a bracing system, the axial force in the rafter may be significant, and the combined effects should be verified.

7.1.1 Classification of cross section

In BS EN 1993-1-1, cross sections are classified according to the width to thickness ratio of the flanges and web, dependant on the magnitude of the bending moment and axial compression on the section. The Class of a section is the highest Class of either the flanges or the web.

The Classes indicate the following structural behaviour:

- | | |
|---------|--|
| Class 1 | can support a rotating plastic hinge without any loss of resistance from local buckling. |
| Class 2 | can develop full plastic moment but with limited rotation capacity before local buckling reduces resistance. |
| Class 3 | can develop yield in extreme fibres but local buckling prevents development of the plastic resistance. |
| Class 4 | has proportions such that local buckling will occur at stresses below first yield. |

The classification of the haunch section is more complex than that of a uniform section. It is convenient to check if the various elements of the cross section are at least Class 2 under the most onerous loading conditions (i.e. to assume that the cross section is in pure compression). If this criterion is satisfied, plastic properties may be used.

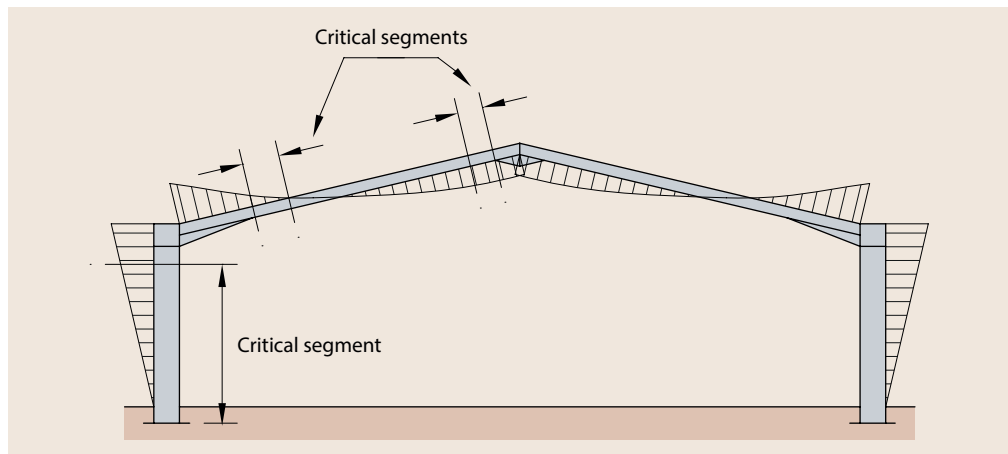
If elements of the haunch are not at least Class 2, yet the flanges are at least Class 2, it is convenient to calculate an effective plastic modulus, only utilising the stable lengths of web. This approach is demonstrated in the worked example in Appendix D.

7.1.2 Column and rafter cross section verification

Although all cross sections need to be verified, the likely key points are at the positions of maximum bending moment as shown in Figure 7.1:

- in the column at the underside of the haunch
- in the rafter at the sharp end of the haunch
- in the rafter at the maximum sagging location adjacent to the apex.

Figure 7.1
Typical bending
moment diagram for
a frame with pinned
bases subject to
gravity loading



At the position of the maximum bending moment for each member a number of effects must be considered, as follows:

Bending

The bending resistance of the cross section should be verified. This check is usually not critical, as the highest utilisation ratio is found when combined compression and bending are considered.

Shear

The shear resistance of the cross section should be verified. Although high shears can serve to reduce moment resistance, shears in portal frame members are generally not high enough to reduce the bending resistance of the cross section.

Compression

The compression resistance of the cross section should be verified. It is very unlikely that this check will be critical.

Combined bending and compression

The presence of compression in the section reduces the moment resistance, though the Eurocode provides limits in Clause 6.2.9.1(4) for Class 1 and Class 2 sections, specifying when the compression is low enough to be ignored. For Class 1 or 2 cross sections, it is likely that the axial force is low enough to have no effect on the bending resistance. For Class 3 cross sections, the axial force will reduce the bending resistance and Clause 6.2.9.2 needs to be followed.

The most straightforward way of determining the reduced moment resistance is to use the 'Blue Book'^[28], which provides values of the design moment resistance reduced due to axial force, $M_{n,y,Rd}$ for various levels of axial force.

7.1.3 Haunch cross section verification

Elements of the haunch (the webs and the flanges) should be classified in accordance with Table 5.2 of BS EN 1993-1-1. A simple approach is recommended, conservatively assuming that the web elements for the rafter and the haunch cutting are in uniform compression, depending on the load combination being considered. If every element is at least Class 2, the cross sectional resistance may be calculated based on plastic properties.

In the common cases where the web is not at least Class 2, an appropriate approach is to calculate an effective plastic cross section, assuming that the web is only effective for a distance of $20t_w\varepsilon$ from the flanges, where:

$$\begin{aligned} t_w & \text{ is the thickness of the web} \\ \varepsilon & = \sqrt{\frac{235}{f_y}} \end{aligned}$$

The classification may vary at different points along the haunch, so should be carried out at intermediate positions – to divide the haunch into five segments is appropriate.

Bending resistance

In most cases, the axial force will be very small compared to the cross sectional resistance and can therefore be ignored when calculating the bending resistance.

If plastic or effective plastic cross sectional properties have been calculated, the bending resistance may be calculated neglecting the (small) part of the cross section allocated to the compression force. This is demonstrated in Section 11.1.2 of the worked example.

It is very unlikely in portal frame construction that the applied shear will be sufficient to reduce the bending resistance of the haunch section.

Shear resistance

The shear area of the cross section, A_v can be taken as the depth of the compound section multiplied by the minimum web thickness.

Compression

The compression resistance may be calculated using the gross area, or, if effective properties have been calculated, the effective area. It is most unlikely that compression will be critical.

7.2 Member buckling resistance

In portal frames, the columns and rafters are subject to combined axial force and bending. Member verification involves the determination of the flexural buckling resistance, the lateral-torsional buckling resistance and the resistance under combined axial force and bending. Both in-plane and out-of-plane buckling must be considered.

Verification of the in-plane buckling resistance considers the interaction of flexural buckling about the major axis and lateral-torsional buckling. In order to calculate the in-plane flexural resistance, the in-plane slenderness, (and thus the buckling length in-plane) must be determined: there are no intermediate restraints when considering the in-plane flexural buckling of a member in a portal frame. Secondary steelwork can provide restraint against lateral-torsional buckling.

Out-of-plane buckling resistance depends on the position and type of restraint provided by the secondary steelwork. Lateral-torsional buckling is governed by the distance between restraints to the compression flange; flexural buckling is governed by the distance between lateral restraints to the member; both resistances may be increased with restraints to the tension flange.

For elastically designed portal frames, these two verifications are represented in BS EN 1993-1-1 expressions 6.61 (in-plane buckling) and 6.62 (out-of-plane buckling), as shown below.

$$\frac{N_{Ed}}{\chi_y N_{Rk}} + k_{yy} \frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT} \frac{M_{y,Rk}}{\gamma_{M1}}} + k_{yz} \frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1 \quad (6.61)$$

$$\frac{N_{Ed}}{\chi_z N_{Rk}} + k_{zy} \frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT} \frac{M_{y,Rk}}{\gamma_{M1}}} + k_{zz} \frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1 \quad (6.62)$$

Where all the symbols are as defined in BS EN 1993-1-1, Clause 6.3.3.

For Class 1, 2, 3 and bi-symmetric Class 4 sections, $\Delta M_{y,Ed} = \Delta M_{z,Ed} = 0$

It is helpful to define $\chi_y \frac{N_{y,Rk}}{\gamma_{M1}}$ as $N_{b,y,Rd}$ and $\chi_{LT} \frac{M_{y,Rk}}{\gamma_{M1}}$ as $M_{b,Rd}$.

$M_{z,Ed} = 0$ because the frame is only loaded in its plane.

The expressions therefore reduce to:

$$\frac{N_{Ed}}{N_{b,y,Rd}} + k_{yy} \frac{M_{y,Ed}}{M_{b,Rd}} \leq 1.0 \quad (\text{from Expression 6.61})$$

$$\text{and } \frac{N_{\text{Ed}}}{N_{\text{b,z,Rd}}} + k_{zy} \frac{M_{y,\text{Ed}}}{M_{\text{b,Rd}}} \leq 1.0 \text{ (from Expression 6.62).}$$

Further simplification for I sections is offered in a paper by Banfil^[31]. This simplified approach is conservative, but easy to apply because the k_{yy} and k_{zy} factors have been simplified considerably.

For major axis effects on I sections (from Expression 6.61), the simplified interaction is:

$$\frac{N_{\text{Ed}}}{N_{\text{b,y,Rd}}} + C_{\text{my}} \frac{M_{y,\text{Ed}}}{M_{\text{b,Rd}}} \leq 0.85$$

For minor axis effects on I sections (from Expression 6.62), the simplified expression is:

$$\frac{N_{\text{Ed}}}{N_{\text{b,z,Rd}}} + 0.78 \frac{M_{y,\text{Ed}}}{M_{\text{b,Rd}}} \leq 0.78 \text{ for Class 1 and 2 sections}$$

$$\text{and } \frac{N_{\text{Ed}}}{N_{\text{b,z,Rd}}} + 0.85 \frac{M_{y,\text{Ed}}}{M_{\text{b,Rd}}} \leq 0.85 \text{ for Class 3 and 4 sections}$$

where:

C_{my} is the equivalent uniform moment factor given in Table B.3 of BS EN 1993-1-1 and is less than or equal to 1.

7.2.1 Influence of moment gradient

The distribution of the bending moments along an unrestrained length of beam has an important influence on the lateral-torsional buckling resistance. A uniform bending moment is the most onerous loading.

The moment gradient is also important in the interaction expressions 6.61 and 6.62, and is accounted for by various C factors (see below). Although it is conservative to take these C factors as 1.0, which reflects the onerous case of a uniform bending moment, this is not recommended. At many points in a portal frame, a segment between restraints has a significantly varying bending moment, which may be beneficially allowed for in design.

Lateral-torsional buckling resistance

When calculating the lateral-torsional buckling resistance, the influence of the moment gradient is accounted for in the C_1 factor when calculating M_{cr} (see Appendix B.2).

Interaction expressions 6.61 and 6.62

In the calculation of k_{yy} and k_{zy} the influence of the moment gradient is accounted for by C_{my} , C_{mx} and $C_{\text{m,LT}}$. The C and k factors used in expressions 6.61 and 6.62 are determined from Annex A or B of BS EN 1993-1-1. The use of Annex B is recommended when undertaking manual design, and is the option demonstrated in the worked example.

7.2.2 Restraint and member buckling

Figure 7.2 shows an idealised representation of the deflected profile of a portal frame rafter, taken in isolation from the rest of the frame, illustrating the vertical deflection in-plane, and the out-of-plane buckling of the upper and lower flanges.

Figure 7.2 shows a typical arrangement of purlins providing restraint to the outside flange of the rafter. Stays to the inside flange are provided at positions 5 (adjacent to the sharp end of the haunch) and 11 (near the apex). The bending moment produces compression in the inside flange of the rafter in the eaves region, and compression in the outside flange for the remainder of the rafter. The following points should be noted:

- There are no intermediate points of restraint for in-plane flexural buckling.
- Purlins provide intermediate lateral restraint to one flange. Depending on the bending moment diagram this may be either the tension or compression flange.
- Restraints to the inside flange can be provided at purlin positions, producing a torsional restraint at that location.

In-plane, the member can buckle over its entire length. The buckling length of the rafter pair may be longer or shorter than the actual length of one rafter, depending on the geometry of the frame and in-plane fixity at the ends of the rafters. Out of plane,

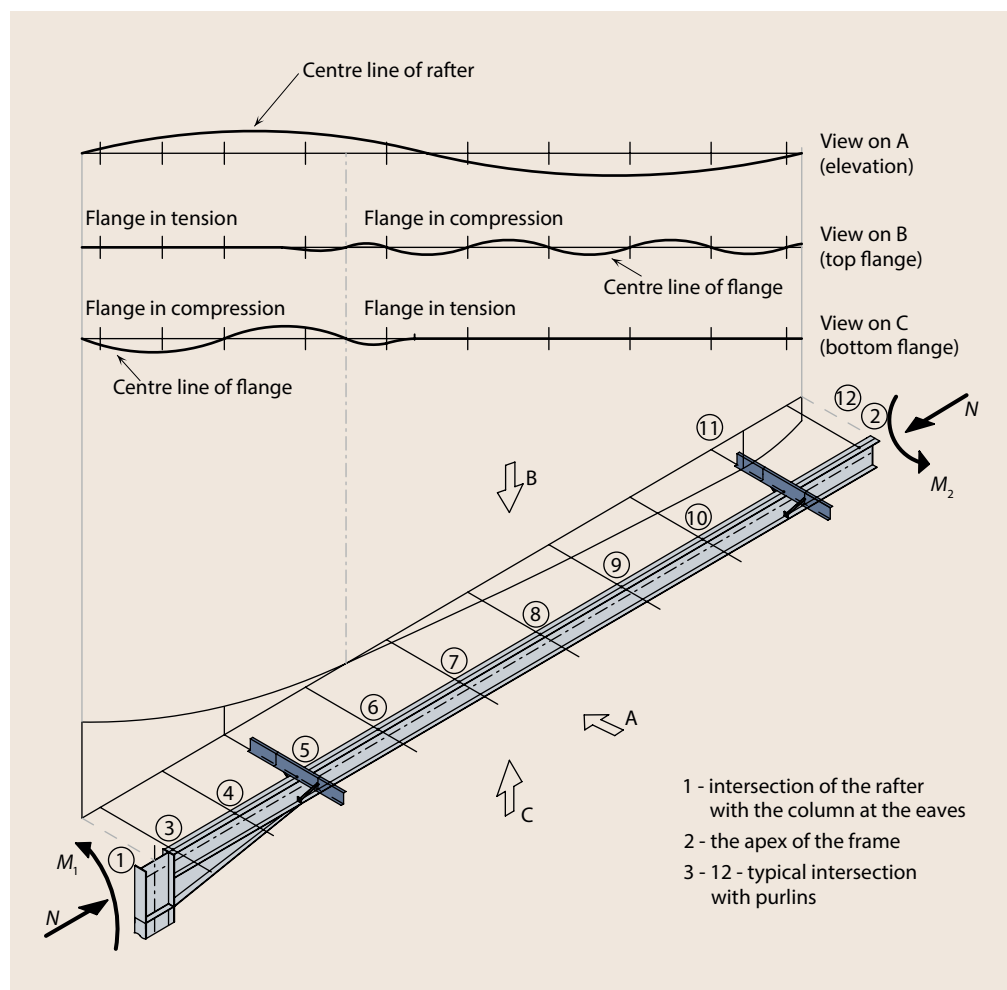


Figure 7.2
Rafter deflection

the member buckles between points of restraint, which are for the designer of the structure to specify, by judicious positioning of the purlins and stays to the inner flange. Typical details to restrain the inner flange are shown in Figure 7.3.

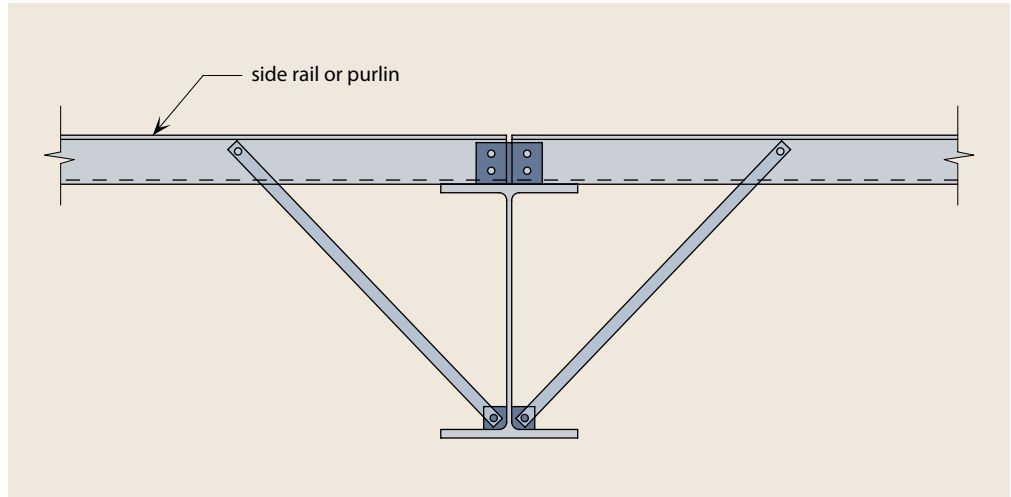


Figure 7.3
Restraint to
inner flange

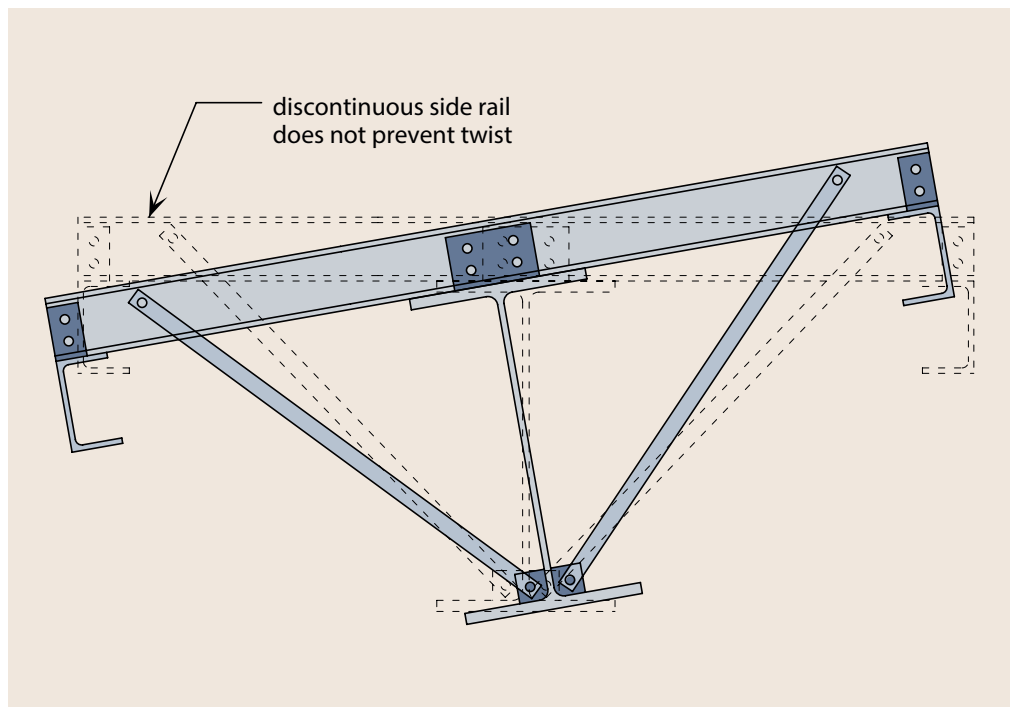


Figure 7.4
Ineffective restraint
to inner flange

Purlins and side rails must be continuous between one frame and the next in order to offer adequate restraint. A side rail that is not continuous (for example, interrupted by industrial doors) cannot be relied upon to provide adequate restraint as the entire arrangement may rotate, as illustrated in Figure 7.4. If the rail is not continuous, additional bracing must be provided.

7.2.3 Tension flange restraint

Even if no restraints are provided to the compression flange, restraints to the tension flange can be effective in increasing the buckling resistance of a member.

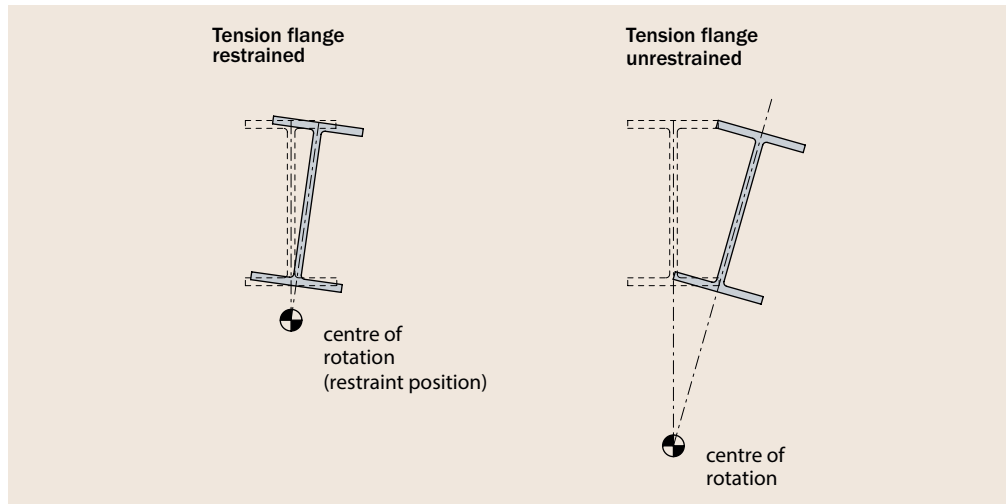


Figure 7.5
Effect of restraint to a
tension flange

The restraints to the tension flange must be provided at sufficiently close centres to be considered a continuous line of support. The restraints to the tension flange reduce the lateral buckling of the compression flange, as shown in Figure 7.5. The design approach to utilise the benefits of tension flange restraint is described in more detail in Appendix C.

Tension flange restraint improves flexural buckling resistance about the minor axis much more significantly than lateral-torsional buckling resistance. The increase in flexural buckling resistance is generally of the order of 15%. For lateral-torsional buckling resistance, the increase is only up to approximately 5%. Further guidance can be found in references 32 and 33.

7.3 Column stability

7.3.1 Column stability under gravity loading

A torsional restraint to the column should always be provided at the underside of the haunch. This may be from a side rail positioned at that level, with stays to the inside flange, or by some other means. It is generally not sufficient to assume that a combination of a side rail and web stiffeners at this position is sufficient to provide a torsional restraint.

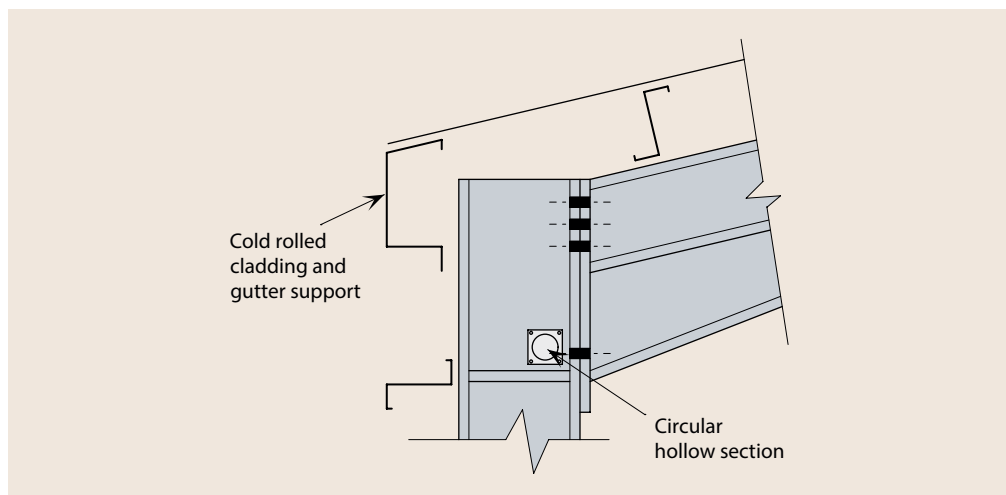


Figure 7.6
Typical eaves detail
using a circular
hollow section
as a longitudinal
bracing member

As an alternative to a side rail and stays to the inside flange, it may be convenient to provide a hot-rolled member (see Figure 7.6), typically a hollow section, to provide lateral restraint to the inner flange. It is essential to connect this line of bracing on the inner flange to the outer flange (or to the foundation) at some point in the line of the bracing, as the objective is to restrain the inner flange with respect to the outer flange (*i.e.* provide a torsional restraint), not merely to connect all the inner flanges.

Intermediate torsional restraints may be required between the underside of the haunch and the column base because the side rails are attached to the (outer) tension flange; unless restraints are provided, the inner compression flange is unrestrained. As noted in Section 7.2.2, a side rail that is not continuous (for example, interrupted by industrial doors) cannot be relied upon to provide torsional restraint. The column section may need to be increased if intermediate restraints to the compression flange cannot be provided.

Figure 7.7 shows a typical moment distribution in the column in the gravity combination of actions and indicates the positions of restraints on a typical column. Verification using Expression 6.62 of BS EN 1993-1-1 is required between torsional restraints. If between torsional restraints there are intermediate restraints to the tension flange and these are spaced at sufficiently close centres, these may be used to calculate an increased buckling resistance, as described in Appendix C.

Although the frame is analysed and designed elastically, it may be convenient to verify member stability using rules primarily intended for use for segments containing plastic hinges. If a member can be verified using a plastic criterion, then the less onerous elastic situation must also be satisfactory. This approach may be valuable in locations where the bending moment approaches the plastic resistance moment of the section (typically

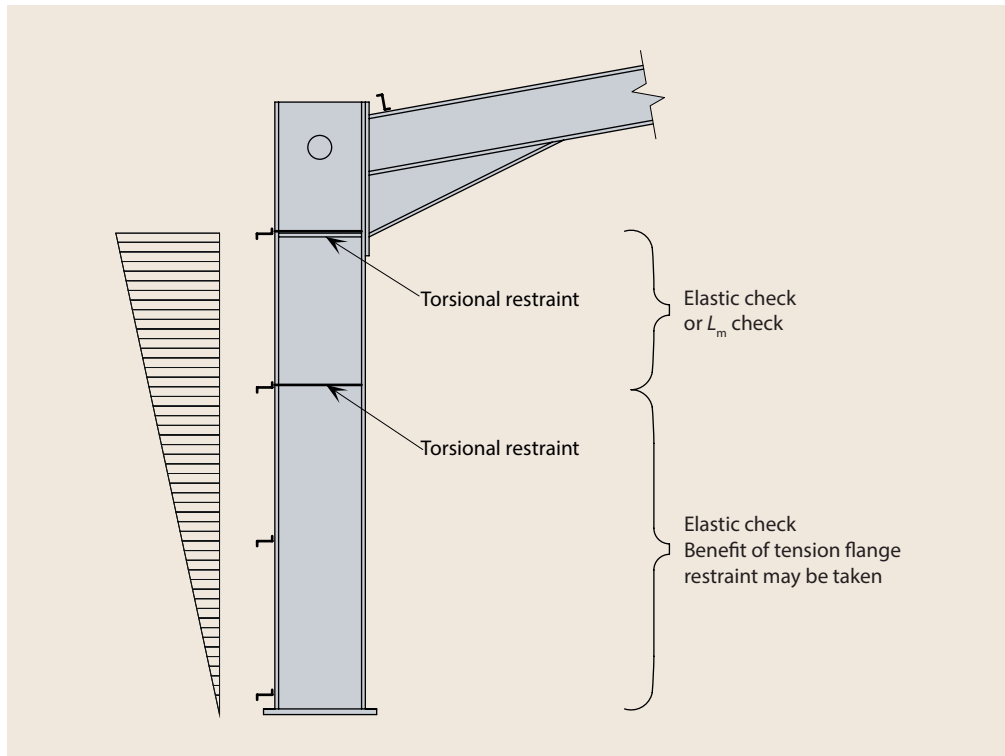


Figure 7.7
Typical portal frame
column with plastic
hinge at underside
of haunch

immediately under the haunch). As indicated in Figure 7.7, a typical application of this philosophy is to ensure that a torsional restraint is located within a distance L_m (see Clause BB.3) from the torsional restraint at the underside of the haunch.

If the stability between torsional restraints cannot be verified, it may be necessary to introduce additional torsional restraints. Figure 7.7 illustrates the addition of an intermediate torsional restraint in the length between the underside of the haunch and the base.

7.3.2 Out-of-plane stability under uplift combinations

When the frame is subject to uplift, the column moment will reverse. The bending moments will generally be significantly smaller than those under gravity loading combinations, and the side rails restrain the compression flange.

The out-of-plane stability of the column should be verified in accordance with Expression 6.62 (see Section 7.2). The side rails connected to the compression flange provide restraint to lateral torsional buckling, and in these circumstances (low axial compression, high bending moment) are considered to provide lateral restraint to flexural buckling.

7.3.3 Column in-plane stability

In addition to verification of out-of-plane stability, in-plane stability must be verified using Expression 6.61.

For this verification the buckling length required to calculate the compression resistance $\chi_y N_{Ed} / \gamma_{M1}$ should be taken as the system length of the column. The ratio $M_{y,Ed} / M_{b,Rd}$ should be taken as the maximum in any segment between restraints to the column (each ratio will have been calculated for each segment considered in the verification of out-of-plane stability). When calculating k_{yy} , the bending moment diagram should be taken as that over the system length of the column from base to eaves. When calculating the C factors from Table B.3, M_h is the maximum value of the bending moment at the end of the member. M_s is the value of the bending moment at mid-height.

7.4 Rafter stability

7.4.1 Out-of-plane stability under gravity loading

Under gravity loading, rafters are subject to high bending moments, which vary from a hogging moment at the junction with the column to a sagging moment close to the apex, as shown in Figure 7.8. Compression is introduced in the rafters due to the shear forces in the columns. The rafters are not subject to any minor axis moments.

Figure 7.9 shows a typical moment distribution for the gravity combination of actions, typical purlin positions and typical restraint positions.

Purlins are generally placed at up to 1.8 m spacing but this spacing may need to be reduced in the high moment regions near the eaves. Figure 7.9 identifies three stability zones (A, B, and C), which are discussed separately below.

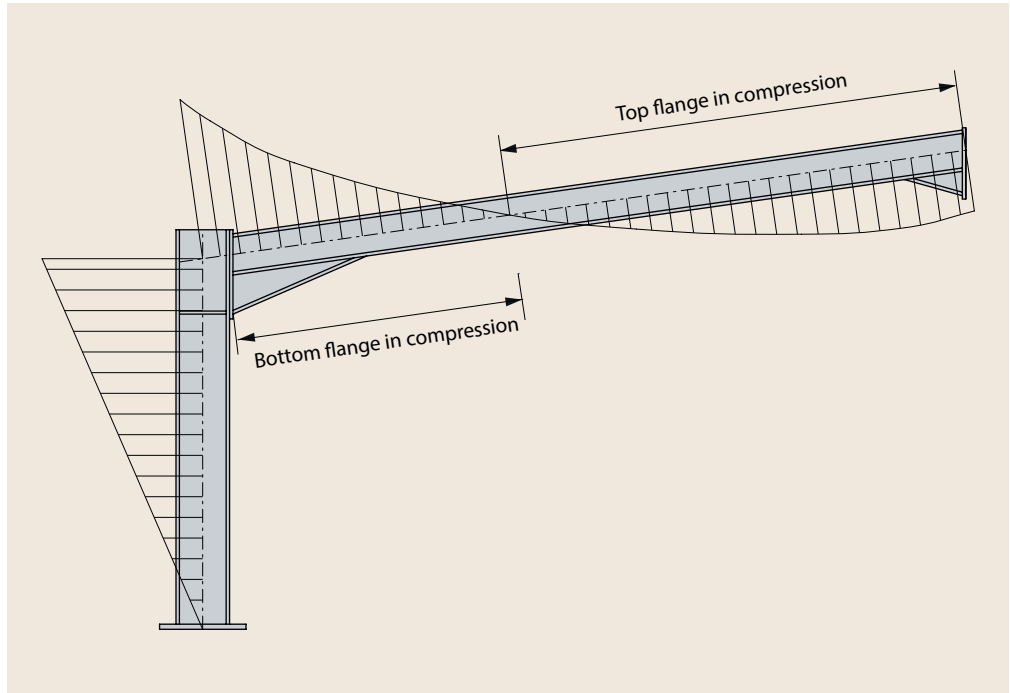


Figure 7.8
Portal frame
bending moments,
gravity actions

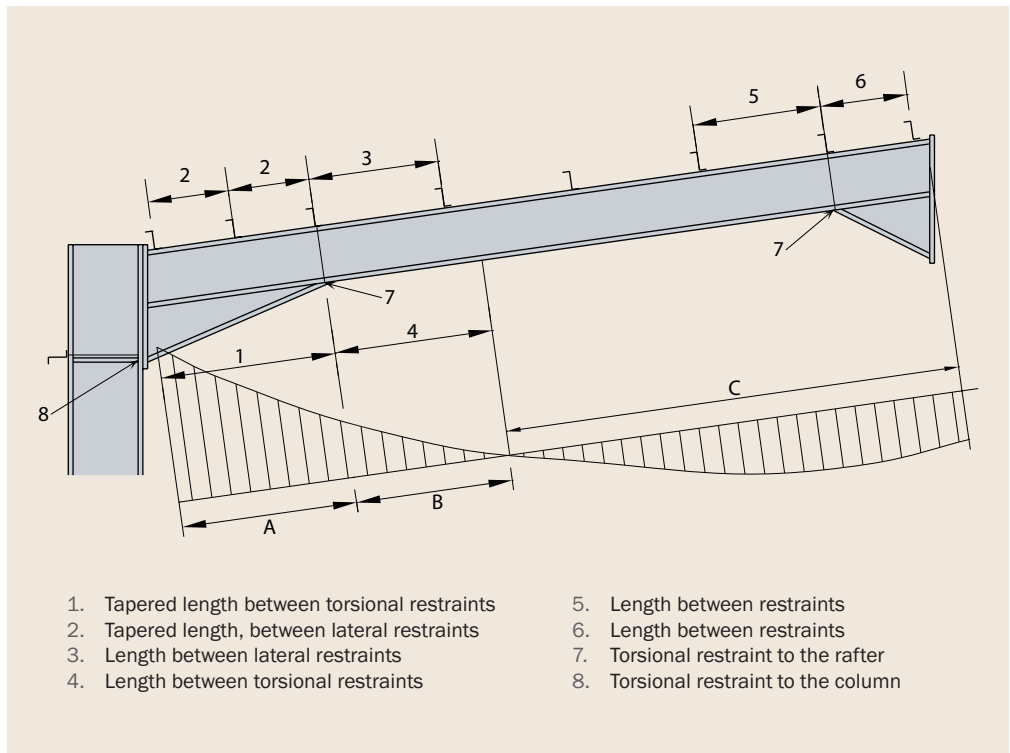


Figure 7.9
Typical bending
moment diagram
under the gravity
combination of
actions

- | | |
|--|--------------------------------------|
| 1. Tapered length between torsional restraints | 5. Length between restraints |
| 2. Tapered length, between lateral restraints | 6. Length between restraints |
| 3. Length between lateral restraints | 7. Torsional restraint to the rafter |
| 4. Length between torsional restraints | 8. Torsional restraint to the column |

The selection of the appropriate criterion for each zone depends on the shape of the bending moment diagram and the geometry of the section (three flanges or two flanges). The objective is to provide sufficient restraints to ensure the rafter is stable out-of-plane.

Haunch stability in Zone A

In Zone A, the bottom flange of the haunch is in compression. The stability verification is complicated by the variation in geometry along the haunch.

The junction of the inside column flange and the underside of the haunch (point 8 in Figure 7.9) should always be restrained. The ‘sharp’ end of the haunch (point 7 in Figure 7.9) usually has restraint to the bottom flange, from a purlin located at this position, forming a torsional restraint at this point.

BS EN 1993-1-1 currently appears to contain no clear guidance on how haunched members are to be verified, other than the so-called ‘general method’ of Clause 6.3.4. A conservative approach to verify the stability of an elastic haunch section is to consider the compression zone as a Tee section, or use a ‘plastic’ verification, as described in Section 7.4.3.

Rafter stability in Zone B

Zone B generally extends from the ‘sharp’ end of the haunch to the point of contraflexure, or to the first purlin beyond the point of contraflexure (see Figure 7.9). The bottom flange is partially or wholly in compression over this length.

In this zone, torsional and lateral restraint will be provided at the ‘sharp’ end of the haunch. At the upper end, restraint will be provided by a purlin beyond the point of contraflexure.

In the UK, practice is to consider the point of contraflexure as a torsional restraint, provided the following conditions are satisfied:

- The rafter is a universal beam
- At least two bolts are provided in the purlin-to-rafter connections
- The depth of the purlin is not less than 0.25 times the depth of the rafter.

The purlins connected to the compression flange provide restraint to lateral torsional buckling, and in these circumstances (low axial compression, high bending moment) are considered to provide lateral restraint to flexural buckling.

Rafter stability in Zone C

In Zone C, the purlins provide lateral restraint to the top (compression) flange. It is assumed that the diaphragm action of the roof sheeting is sufficient to carry the restraint forces to the bracing system.

Out-of-plane stability is verified in accordance with Expression 6.62 (see Section 7.2 of this publication). Normally, if the purlins are regularly spaced, it is sufficient to verify the rafter for the maximum bending moment with maximum compression force.

It is normal practice to provide a torsional restraint at the penultimate purlin to the apex, as this will be necessary when considering the uplift combinations of actions – the bottom flange will be in compression. This restraint is indicated at point 7 in Figure 7.9.

7.4.2 Rafter and haunch stability under uplift conditions

Under uplift, most of the bottom flange of the rafter is in compression. A typical bending moment diagram is shown in Figure 7.10, which indicates two zones, (E and F) for which stability is verified.

This type of bending moment diagram will generally occur under internal pressure and wind uplift. Normally, the magnitude of the design bending moments are smaller than under gravity loading.

Haunch stability in Zone E

In Zone E (see Figure 7.10), the top flange of the haunch will be in compression and will be restrained by the purlins.

Although the moments and axial forces are smaller than those under gravity loads, the problem remains that there is no adequate guidance in BS EN 1993-1-1 that applies to the elastic verification of tapered members. However, if the haunch is stable in the gravity combination of actions, it will certainly be so in the uplift condition, being restrained at least as well, and under reduced loads. By inspection, it should be clear that the haunch section in this zone is satisfactory.

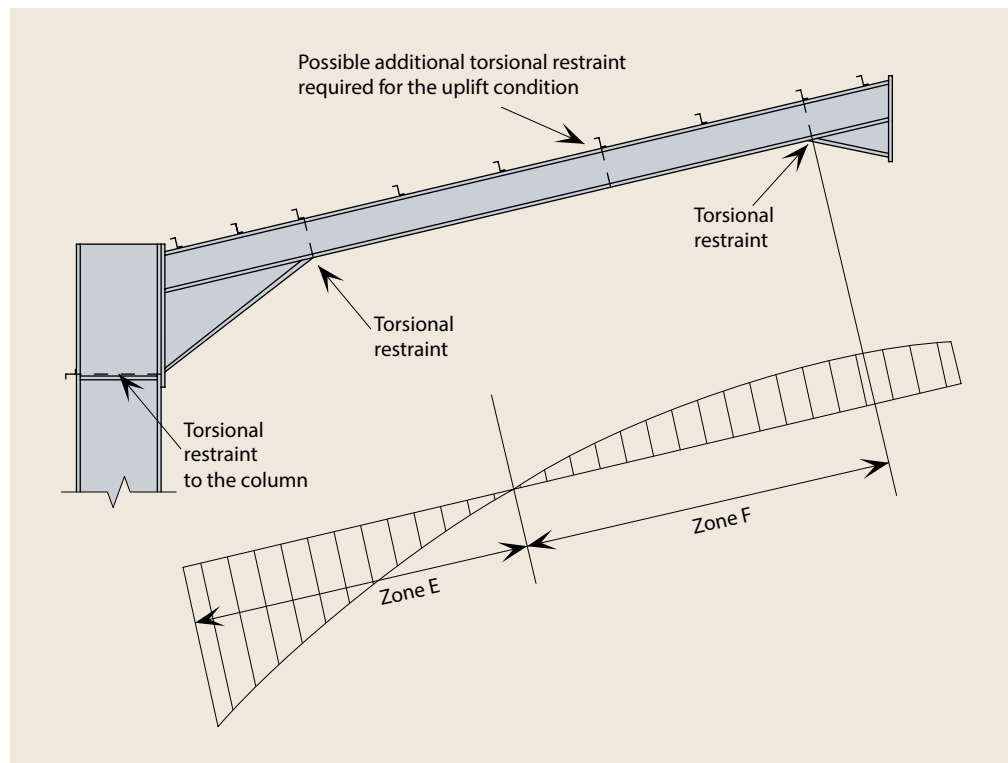


Figure 7.10
Typical purlin and rafter
stay arrangement for
wind uplift

Stability in Zone F

In Zone F, the purlins will not restrain the bottom flange, which is in compression.

The rafter must be verified between torsional restraints. A torsional restraint will generally be provided adjacent to the apex, as shown in Figure 7.10. The rafter may be stable between this point and the virtual restraint at the point of contraflexure, as the moments are generally modest in the uplift combination. If the rafter is not stable over this length, additional torsional restraints should be introduced, and each length between restraints verified.

This verification should be carried out using Expression 6.62.

The beneficial effects of the restraints to the tension flange (the top flange, in this combination) may be accounted for using a modification factor C_m , taken from Clause BB.3.3.1(1)B for linear moment gradients and from Clause BB.3.3.2(1)B for non-linear moment gradients. If this benefit is utilised, the spacing of the intermediate restraints should also satisfy the requirements for L_m in Clause BB.3.1.1.

7.4.3 Haunch buckling resistance

The elastic verifications provided in BS EN 1993-1-1 are only appropriate for members with a uniform cross section. In the absence of any specific checks, two alternatives are proposed; to calculate the resistance of the compression flange between restraints, assuming it to be a Tee section, or to use a verification intended for a haunched length containing a plastic hinge. If the haunch is verified using a ‘plastic’ check, the reasonable assumption is that an elastic member will certainly be adequate.

Equivalent compression flange

Following the principles contained in BS EN 1993-1-1 Clause 6.3.2.4, a Tee-shaped equivalent compression flange can be determined, composed of the flange itself plus 1/3 of the compressed part of the web area. The section properties for this Tee may be calculated at a point 1/3 of the length of the haunch. The principle of the equivalent compression flange is shown in Figure 7.11. The equivalent compression flange is simply treated as a strut, and its resistance compared to the design force.

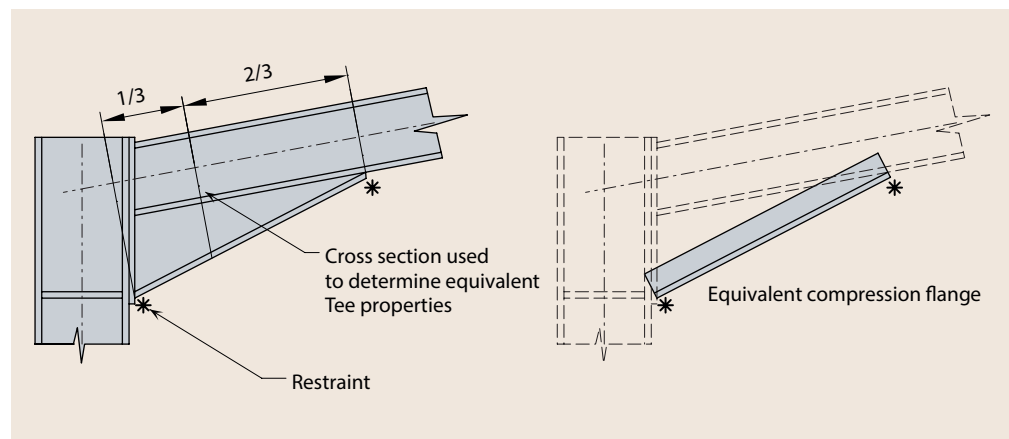


Figure 7.11
Equivalent
compression flange

‘Plastic’ verification of a haunched member

Appendix BB.3 of BS EN 1993-1-1 contains clauses which may be used to verify the stable length of segments containing plastic hinges. Additional clauses cover tapered members, the contribution of intermediate restraints to the tension flange and the influence of the shape of the bending moment diagram. If a haunch can be verified using these ‘plastic’ criteria, it will be stable when designed elastically.

7.4.4 In-plane stability of rafters

In addition to verification of out-of-plane stability, in-plane stability must be verified using Expression 6.61.

For this verification, the compression resistance $\frac{\chi_y N_{Ed}}{\gamma_{M1}}$ is based on the buckling length of the rafter. For single span symmetric portals of orthodox construction, it is currently recommended that the buckling length is taken as the developed length from eaves to apex. Although M_{cr} (and therefore $M_{b,Rd}$) can be calculated for the entire length of the rafter using software such as LTBeam^[41], it is currently recommended that the ratio $\frac{M_{y,Ed}}{M_{b,Rd}}$ should be taken as the maximum in any segment between restraints to the rafter excluding the haunch (each ratio will have been calculated for each segment considered in the verification of out-of-plane stability). When calculating k_{yy} , the bending moment diagram should be taken as that over the entire length of the rafter from eaves to apex. When calculating the C factors from Table B.3, M_h is the maximum value of the bending moment at eaves or apex and M_s is the value of the bending moment halfway between eaves and apex.

BRACING

Vertical bracing in the side walls is required to resist longitudinal actions due to wind and cranes. Roof bracing is required to support the tops of the gable columns, to resist any restraint forces or wind forces carried by the purlins, and to carry these forces to the vertical bracing.

8.1 Vertical bracing

The primary functions of vertical bracing in the side walls of the frame are:

- To transmit horizontal forces to the foundations. The horizontal forces include forces due to wind and cranes.
- To provide stability during erection.

8.1.1 Bracing location

The bracing may be located:

- At one or both ends of the building.
- Within the length of the building.
- In each portion between expansion joints (where these occur).

Where the side wall bracing is not in the same bay as the plan bracing in the roof, an eaves strut is essential to transmit the forces from the roof bracing into the wall bracing and to ensure the tops of the columns are adequately restrained in position. An eaves strut is also required:

- To assist during the construction of the structure.
- To stabilise the tops of the columns if a fire boundary condition exists.

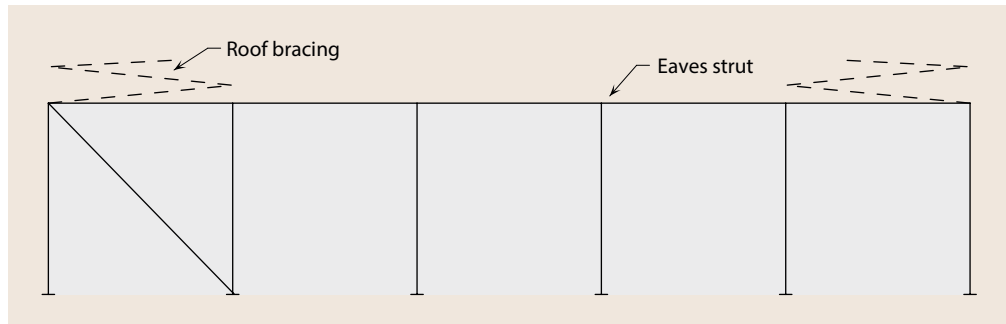
Bracing in a single bay

For vertical bracing provided in a single bay, an eaves strut is required along the full length of the elevation, to transmit wind forces from the roof bracing into the vertical bracing (Figure 8.1). When vertical bracing is provided in only one end bay, any thermal expansion produces movement at the far end of the structure.

Single central braced bay

The concept of providing a single braced bay near the centre of a structure (Figure 8.2) is unpopular because of the preference to start erection from a braced bay and to erect

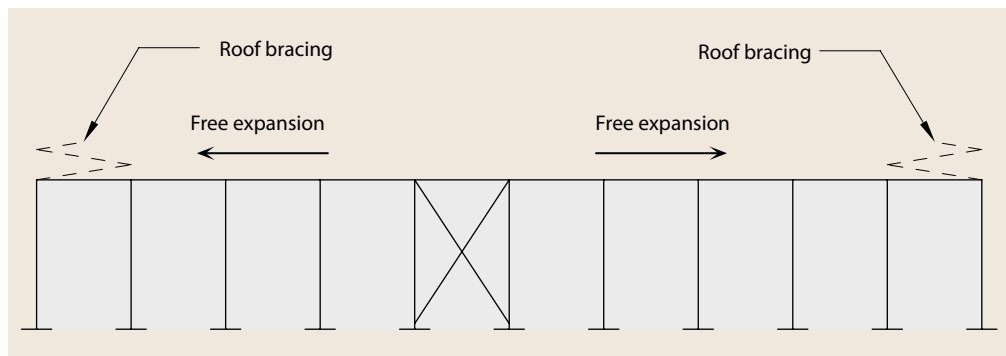
Figure 8.1
Bracing in a single
end bay with an
eaves strut



steelwork along the full length of a building from that point. If a central braced bay is used, it may be necessary to provide additional temporary bracing in the end bays to assist in erection.

Bracing in the middle of the building, as shown in Figure 8.2, or at one end only, has the advantage that it allows free thermal expansion of the structure.

Figure 8.2
Typical cross bracing
at centre of the
structure to allow free
thermal expansion



Multiple braced bays

Vertical bracing is commonly provided in more than one bay, frequently at both ends of the structure, as shown in Figure 8.3. Although this arrangement may be considered to constrain thermal expansion, it has been commonly used with no evidence of detrimental effects. In the UK, the expected temperature range is modest, typically -10°C to $+30^{\circ}\text{C}$, and overall expansion is not generally considered to be a problem in orthodox portal frame construction.

8.1.2 Bracing using hollow sections, flats and angles

The bracing system in the walls will usually take the form of:

- A single diagonal hollow section
- Hollow sections in a K pattern
- Crossed flats, considered to act in tension only
- Crossed hot rolled angles.

Hollow sections are very efficient in compression, which eliminates the need for cross bracing. Where the height to eaves is approximately equal to the spacing of the frames, a single bracing member at each location is economic (Figure 8.3a). Where the eaves height is large in relation to the frame spacing, a K brace is often used (Figure 8.3b). Crossed flats are often used within a masonry cavity wall.

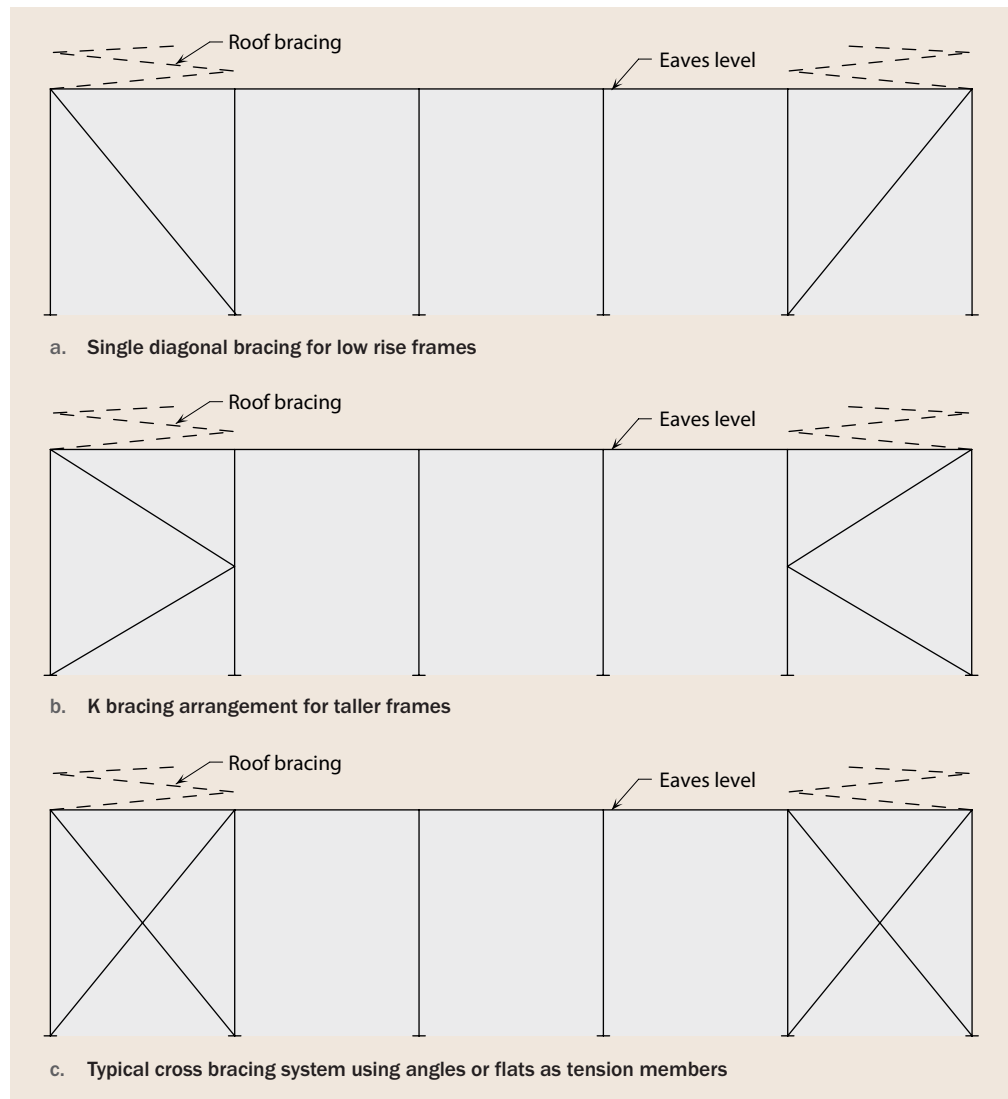


Figure 8.3
Common bracing
systems

8.1.3 Bracing using portalised bays

Where it is difficult or impossible to brace the frame vertically by conventional bracing, it is necessary to introduce moment-resisting frames in the elevations. There are two basic possibilities:

- A moment-resisting frame in one or more bays, as shown in Figure 8.4a.
- Separated moment-resisting connections, as shown in Figure 8.4b.
- The moment-resisting connections are often located in the end bays, where the end column is turned through 90° to provide increased stiffness in the longitudinal direction. This arrangement is only possible if the end frame (the gable) is constructed from a beam and column arrangement, rather than a portal frame.

In the design of both systems, it is recommended that two deflection criteria are verified. Firstly, the longitudinal displacement should be verified against the specified SLS limit for the structure (commonly $h/150$ for metal clad structures, where h is the height of the portalised bay). Secondly, longitudinal deflection under the EHF (see Section 4.4) should be restricted to $h/1000$, where the equivalent horizontal forces are calculated

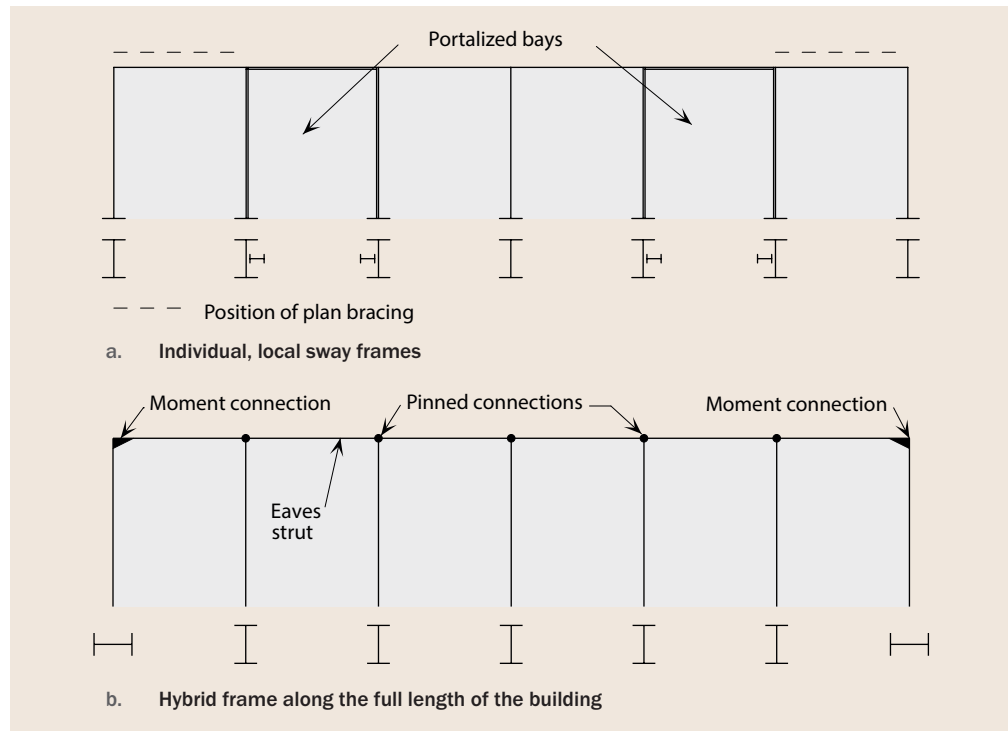


Figure 8.4
Longitudinal stability
using portalised bays

based on the whole of the roof area, not just the axial force in the columns forming the portalised bay. This second limit has two purposes;

- to ensure that second-order effects in the longitudinal direction are small enough to be ignored.
- to ensure that the portal frames can be considered to be adequately restrained in position in the longitudinal direction.

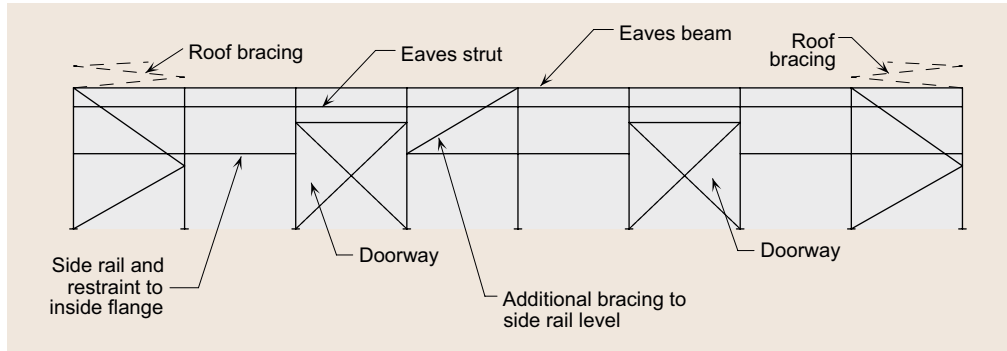
In some cases, it is possible to provide conventional bracing on one elevation, and provide moment-resisting frames on the other. Loads should be distributed to the bracing systems in proportion to their stiffness. The differential movement due to the difference in stiffness of the two sides is generally neglected. If diaphragm action is assumed in larger structures, it is recommended that the cladding and fixings be verified for the anticipated forces. Guidance on stressed skin action is given in an SBI publication^[34].

8.1.4 Bracing to restrain columns

If side rails and column stays provide lateral or torsional restraint to the column, it is important to identify the route of the restraint force to the vertical bracing system. If the continuity of side rails is interrupted by openings in the side of the building, additional intermediate bracing may be required such as shown diagrammatically in Figure 8.5. This bracing should be provided as close to the plane of the side rail as possible.

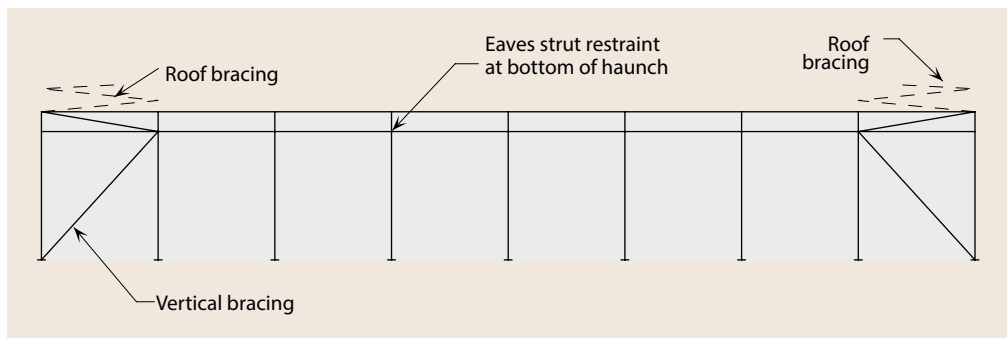
It is not normally necessary for side rails that provide restraint at column stay positions to be aligned with a node of the vertical bracing system. It can be assumed that diaphragm action in the vertical sheeting and the transverse stiffness of the column can transmit the load into the vertical bracing system.

Figure 8.5
Typical bracing
pattern in the side of a
building with openings



If, instead of a side rail and restraints to the inner flange (see Figure 7.3), a separate member is used to provide restraint to the inside flange, it is essential that it is tied properly into the bracing system. The objective is to provide a torsional restraint with respect to the outer flange, not merely to connect all the inner flanges. This can result in the configuration shown in Figure 8.6. Where there is an opening in the side of the building that interrupts the restraining member, additional intermediate bracing will be required in a similar way to that described above.

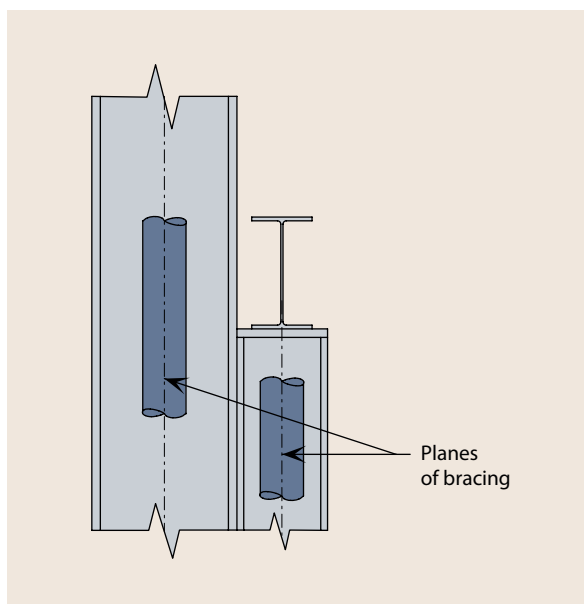
Figure 8.6
Typical bracing
pattern in a building
using a member to
restrain the underside
of the haunch



8.1.5 Bracing to restrain longitudinal loads from cranes

If a crane is directly supported by the frame, the longitudinal surge force will be eccentric to the column and will tend to cause the column to twist, unless additional restraint is

Figure 8.7
Detail showing
additional bracing
in the plane of the
crane girder



provided. A horizontal truss at the level of the crane girder top flange or, for lighter cranes, a horizontal member on the inside face of the column flange tied into the vertical bracing, may be adequate to provide the necessary restraint.

For large horizontal forces, additional bracing should be provided in the plane of the crane girder as indicated in Figure 8.7. Bracing requirements taken from Fisher^[35] are summarised in Table 8.1.

DESIGN VALUE OF LONGITUDINAL FORCE	BRACING REQUIREMENT
Small (<15 kN)	Use wind bracing
Medium (15 – 30 kN)	Use horizontal bracing to transfer force from the crane to plane of bracing
Large (> 30 kN)	Provide additional bracing in the plane of the longitudinal crane forces

Table 8.1
Bracing requirements
for crane girders

8.2 Roof bracing

Roof bracing is located in the plane of the roof. The primary functions of the roof bracing are:

- To transmit wind forces from the gable posts to the vertical bracing in the walls.
- To transmit any frictional forces from wind on the roof to the vertical bracing.
- To provide a stiff anchorage for the purlins that are used to restrain the rafters.
- To provide stability during erection.

The roof bracing should be arranged to provide support at the top of the gable posts.

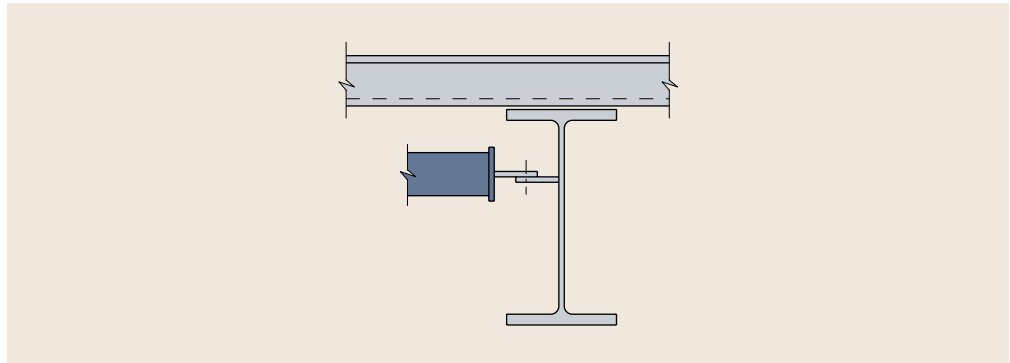
In modern construction, circular hollow section bracing members are generally used in the roof, as shown in Figure 8.8, and are designed to resist both tension and compression. Many arrangements are possible, depending on the spacing of the frames and the positions of the gable posts.

The bracing is usually attached to gusset plates welded to the web of the rafter, as shown in Figure 8.9. The attachment points are usually positioned close to the top flange, allowing for the size of the member and the connection.



Figure 8.8
Plan view showing
both end bays braced

Figure 8.9
Typical connection
detail for hollow
section roof bracing



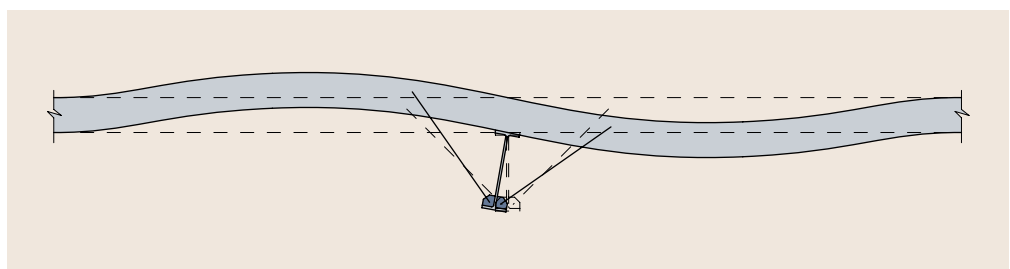
8.3 Restraint to inner flanges

Restraint to the inner flanges of rafters or columns is often most conveniently formed by diagonal struts from the purlins or sheeting rails to small plates welded to the inner flange and web. Pressed steel flat ties are commonly used. Where restraint is only possible from one side, the restraint must be able to carry compression; typically angle sections of minimum size 40 × 40 mm are used. The stay and its connections should be designed to resist a force equal to 2.5% of the maximum force in the column or rafter compression flange between adjacent restraints.

The effectiveness of such restraint depends on the stiffness of the system, especially the stiffness of the purlins. The effect of purlin flexibility on the bracing is shown in Figure 8.10. As a rule of thumb, it will be adequate to provide a purlin or side rail of at least 25% of the depth of the member being restrained. Where the proportions of the members, purlins and spacings differ from proven previous practice, the effectiveness should be verified.

In the absence guidance in the Eurocodes, the stiffness may be verified as suggested by Horne and Ajmani^[36].

Figure 8.10
Effect of purlin
flexibility on bracing



GABLES

9.1 Types of gable frame

Gable frames are typically one of two basic forms:

- A portal frame identical to the remainder of the structure. The gable columns are located in the plane of the end frame but do not support the rafters. This form of gable is used for simplicity, or because there is the possibility of extending the structure in the future.
- A gable frame comprising gable posts with simply supported rafters between the posts. Gable frames of this form require bracing in the plane of the gable, as shown in Figure 9.1. The advantage of this form of gable is that the rafters and external columns are smaller than those in a portal frame.

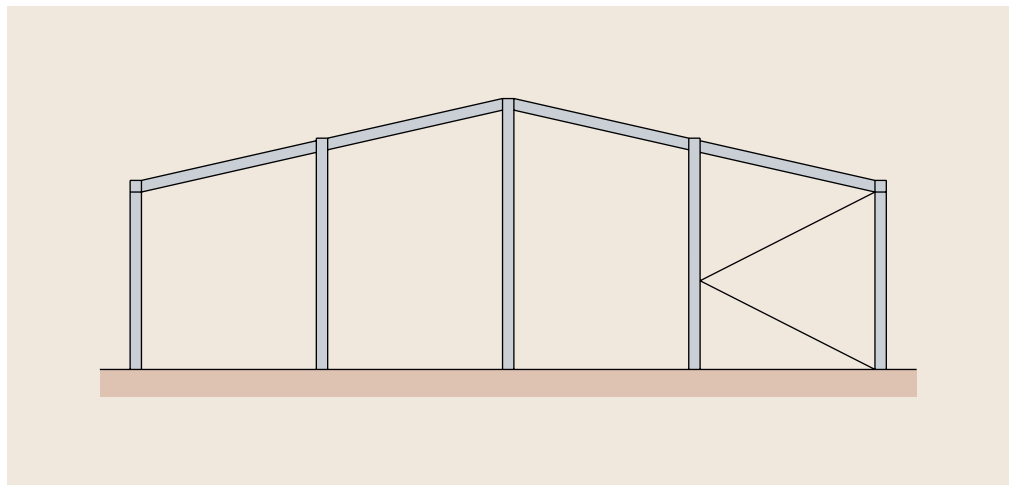


Figure 9.1
Gable frame
from columns,
beams and bracing

9.2 Gable columns

In both types of gable frame, gable columns are designed as vertical beams, spanning between the base and the rafter. At rafter level, the horizontal reaction from the gable column is transferred into the roof bracing, to the eaves, and then to the ground via the bracing in the elevations.

The gable columns are designed for wind pressure and suction. The maximum suction may occur when the gable is on the downwind elevation, as shown in Figure 9.2(a), or, more likely (due to the higher suctions), when the gable is parallel to the wind direction, as shown in Figure 9.2(b).

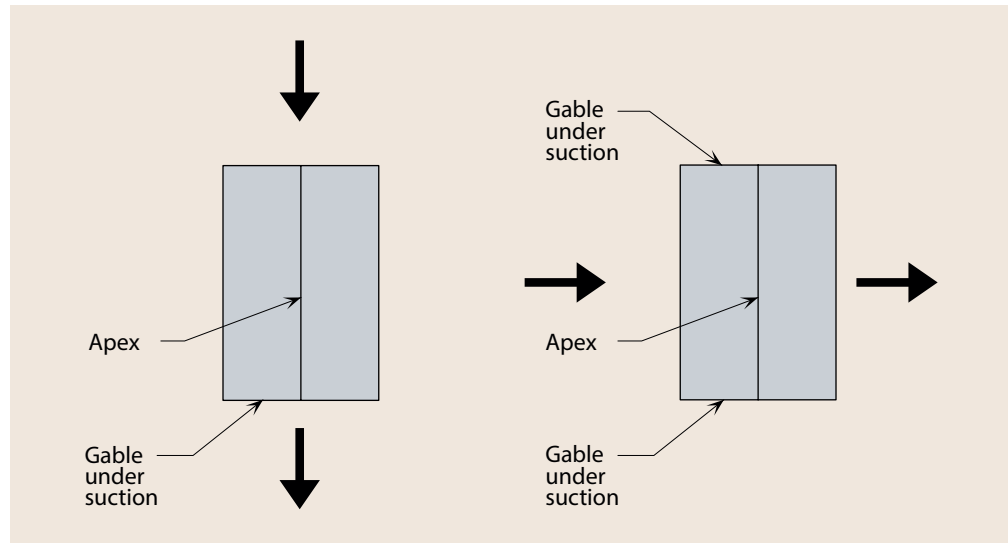


Figure 9.2
Wind loads on gables

The internal pressure or suction contributes to the net loads on the gable. When the net loads are equivalent to an external pressure, the outside flanges of the gable columns are in compression, but are restrained out-of-plane by the side rails. When the net loads are equivalent to an external suction, the inside flanges of the gable columns are in compression. This design case may be the more onerous of the two conditions. It may be possible to reduce the length of the unrestrained inside flange of the gable columns by introducing column stays from the side rails, as illustrated in Figure 7.3.

9.3 Gable rafters

If the gable is of the form shown in Figure 9.1, the gable rafters are usually simply supported UKB section members. In addition to carrying the vertical loads, the gable rafters often act as chord members in the roof bracing system and this design situation must be verified.

If a portal frame is adopted as a gable frame, it is common to adopt an identical frame size, even though the vertical loads on the end frame are rather less than on intermediate frames.

CONNECTIONS

The major connections in a portal frame are those at the eaves and the apex, which are both required to be moment-resisting. The eaves connection in particular is generally subject to a very large design bending moment. Both the eaves and apex connections are likely to experience loading reversal in certain design situations and this can be an important design consideration for the connection. For economy, connections should be arranged to minimise any requirement for additional reinforcement (commonly called stiffeners). This is generally achieved by:

- Making the haunch deeper (increasing the lever arm to the bolt rows)
- Extending the eaves connection above the top flange of the rafter (providing an additional bolt row)
- Adding bolt rows within the depth of the connection
- Selecting a stronger column section.

Guidance on the design of moment-resisting connections is given in SCI Publication P398^[37].

10.1 Eaves connections

A typical eaves connection is shown in Figure 10.1. In addition to increasing the bending resistance of the rafter, the haunch increases the lever arms of the bolts in the tension zone. Generally the bolts in the tension zone (the upper bolts under gravity loading) are nominally allocated to carry tension, whilst the lower bolts (adjacent to the compression stiffener) are nominally allocated to carry the vertical shear, which is generally modest. The compression force is transferred at the level of the bottom flange.

Because the portal frame members are chosen for bending resistance, deep members with relatively thin webs are common in portal frames. A compression stiffener in the column is usually required. The web panel of the column may also need to be reinforced, either with a diagonal stiffener, or an additional web plate (referred to as a supplementary web plate).

The end plate on the rafter is unlikely to require stiffening as it can simply be made thicker, but it is common to find that the column flange requires strengthening locally in the tension zone. Stiffeners are expensive, so good connection design would minimise the need for stiffeners by judicious choice of connection geometry.

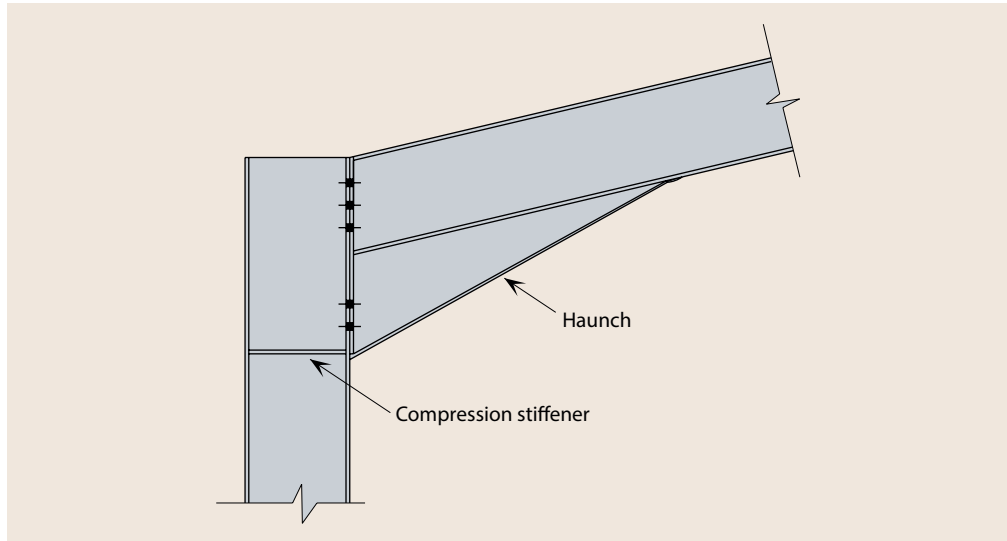


Figure 10.1
Typical eaves
connection

If the reversed moment under uplift is significant, it may be necessary to provide a stiffener to the column web at the top of the column, aligned with the top flange of the rafter, to resist the compressive force. A stiffener at the top of the column is often referred to as a cap plate. Under gravity loading, a cap plate increases the resistance of the column web in tension and the column flange in bending.

Property class 8.8 bolts are invariably used in the UK for moment-resisting connections; M24 are common in larger connections and M20 in more modest connections. Plate components are commonly S275 or S355. End plates are generally at least as thick as the bolt diameter.

10.2 Apex connections

A typical apex connection is shown in Figure 10.2. Under gravity loading, the bottom of the connection is in tension. The haunch below the rafter serves to increase the lever arms to the tension bolts, thus increasing the moment resistance. The haunch is usually small and short, and is not considered in the global analysis of the frame. In lightly loaded frames, a simple extended end plate may suffice. Plate grades and bolt details are the same as for eaves connections.

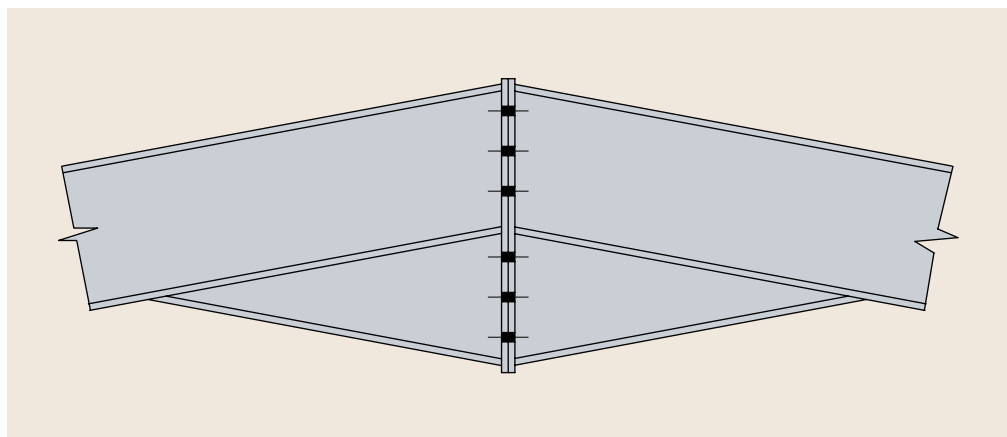


Figure 10.2
Typical apex
connection

10.3 Bases, base plates and foundations

In the majority of cases, a nominally pinned base is provided, because of the difficulty and expense of providing a rigid base. A rigid base would involve a more expensive steelwork detail and, more significantly, the foundation would also have to resist the moment, which increases foundation costs significantly compared to a nominally pinned base.

Where crane girders are supported by the column, moment resisting bases may be required, to reduce deflections to acceptable limits.

10.3.1 Nominally pinned bases

A typical nominally pinned base detail is shown in Figure 10.3.

For larger columns, the bolts may be located entirely within the column profile, as shown. For smaller columns (less than approximately 350 mm), the base plate would be made larger so that the bolts can be moved outside the flanges. Even with larger columns, it is common practice to detail the base with bolts outside the section, as this provides some stability during erection. Because steelwork is erected on levelling packs, typically

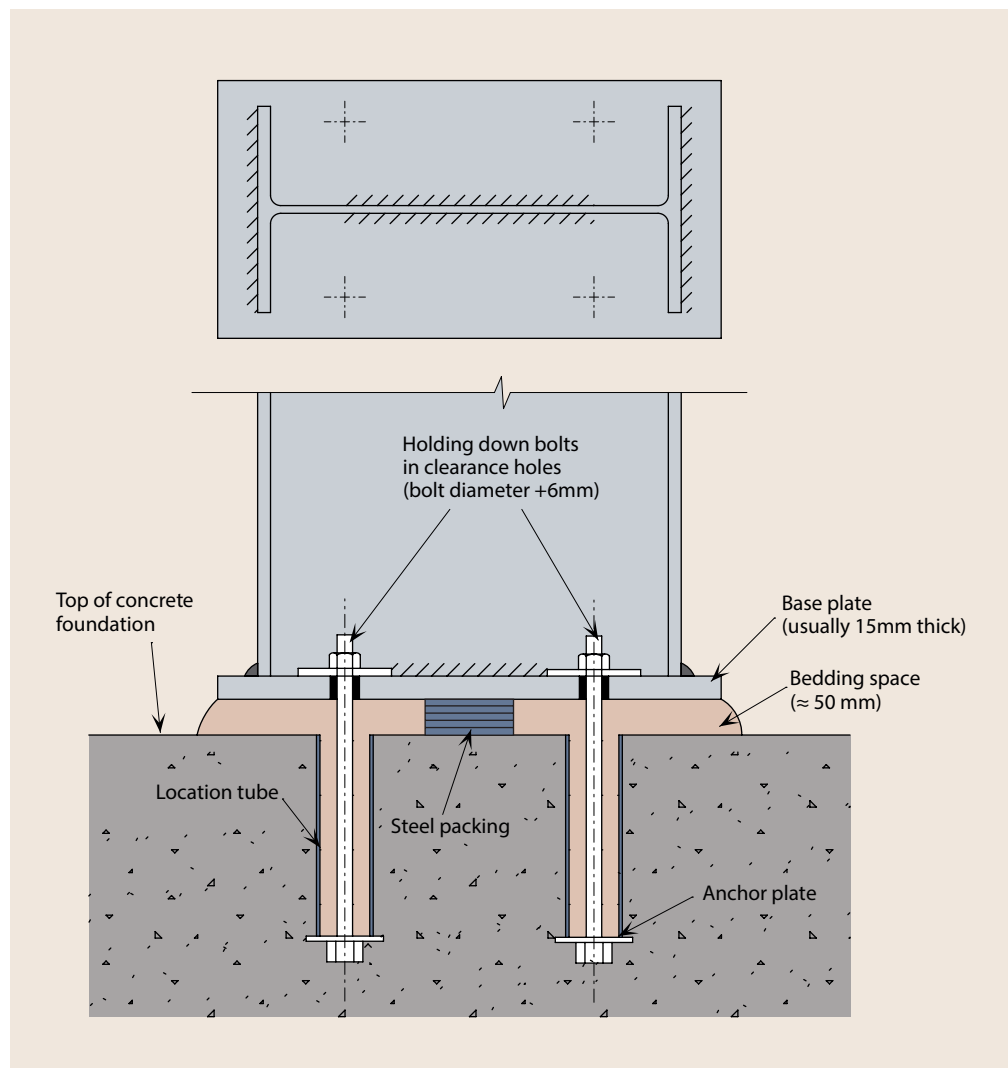


Figure 10.3
Typical nominally
pinned base for a
large column

100 mm square placed directly under the column, if holding down bolts are detailed at very close centres, there may be very little concrete for the packs to bear on.

10.3.2 Rigid bases

A rigid, moment-resisting base is typically achieved by providing a bigger lever arm for the bolts and by increasing the plate thickness. Additional gusset plates may be required for bases subject to large bending moments.

10.3.3 Resistance to horizontal forces

Horizontal reactions can be resisted in a number of ways, including:

- Passive earth pressure on the side of the foundation, as shown in Figure 10.4(a).
- A tie cast into the floor slab, connected to the base of the column, as shown in Figure 10.4(b).
- A tie across the full width of the frame connecting both columns beneath or within the floor slab, as shown in Figure 10.4(c) and (d).

The most popular method of resisting horizontal forces is to use passive earth pressure. This has economic advantages in that the foundation size required to resist uplift is usually sufficient to provide adequate bearing resistance against the ground. However, the passive resistance of the surrounding ground can be less than anticipated if the ground is not compacted correctly, and drainage and service trenches alongside the frame can reduce the passive resistance considerably.

As an alternative, a bar connected to the column and cast into the floor slab, and wrapped at the end to allow differential vertical movement, can be a relatively cheap solution. This detail may lead to some local cracking of the floor slab and, where a high specification floor slab is used, the warranty on the slab may be invalidated. The length of the bar should be determined by the ultimate pull-out resistance required to resist the horizontal force.

A tie across the full width of the frame connected to the column at each side is the most certain way of resisting horizontal forces. It is more expensive in terms of materials and labour and can be damaged by site activities. A full width tie may impede the erection of the structure, which generally will be undertaken from within the footprint of the building.

The transfer of shear between the steelwork and foundation must be considered carefully. Common and successful practice is to consider that modest shear forces can be transferred in friction. Transfer by shear in the holding down bolts is less certain, as shear is unlikely to be shared equally between bolts.

10.3.4 Base plates and holding down bolts

The steelwork contractor will usually be responsible for detailing the base plate and holding down bolts; commonly, another designer is responsible for the foundations. It should be made clear in the contract documentation where the responsibility lies

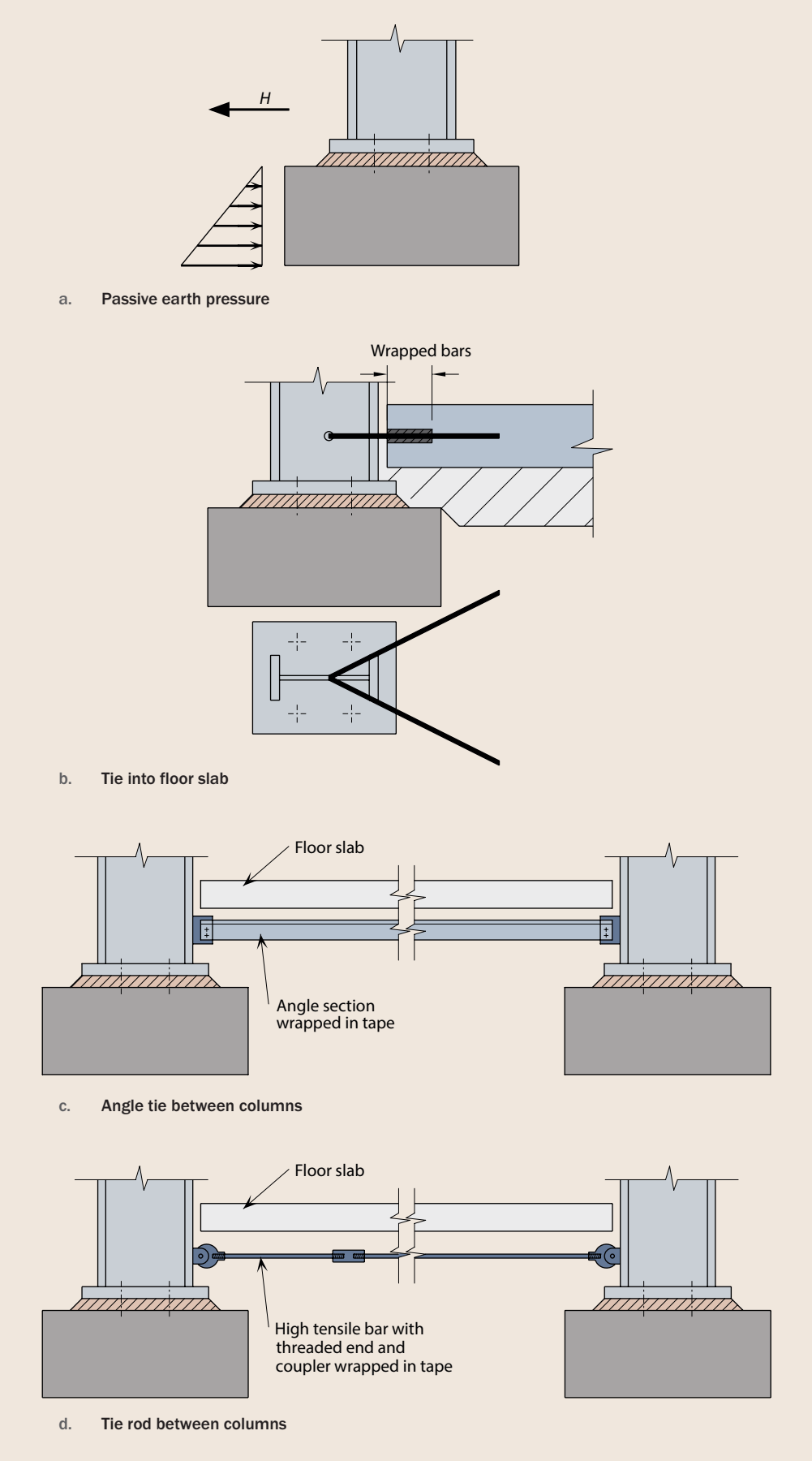


Figure 10.4
Alternative
approaches to resist
horizontal forces at
column bases

for the interface between the steelwork and the foundation, as special reinforcement spacing or details may be required. Best practice is to ensure that the holding down details are integrated with the foundation details.

Base plates will usually be grade S275 steel. Holding down bolts are usually property class 8.8.

The diameter of the bolts will generally be determined by consideration of the uplift and shear forces applied to them, but will not normally be less than 20 mm. There is often generous over-provision, to allow for the incalculable effects of incorrect location of bolts and combined shear force and bending on the bolt if the grout does not completely fill the void under the baseplate.

The length of the bolt should be determined by considering the properties of the concrete, the spacing of the bolts, and the design tensile force. A simple method of determining the embedment length is to assume that the bolt force is resisted by a conical surface of concrete. Where greater uplift resistance is required, angles or plates may be used to anchor the bolts together as an alternative to individual anchor plates.

Advice on the design of holding down systems is given in SCI Publication P398^[37].

10.3.5 Base design at the fire limit state

If the foundation is designed to resist a moment due to rafter collapse in the accidental fire situation, both the base plate and the foundation itself should be designed to resist that moment. As noted in Section 6.4, it is usual to consider the base as nominally pinned for the frame analysis.

To resist the base moment at the fire limit state, the following options may be considered:

- increasing the thickness of the base plate
- adding more holding down bolts on the tension side of the column
- adding a haunch on the compression side.

SECONDARY STRUCTURAL COMPONENTS

11.1 Eaves beam

There is often confusion in terminology when discussing the longitudinal member(s) at eaves level between portal frames. In this publication, 'eaves beam' refers to the member (usually a cold rolled section) that supports the roof cladding, the wall cladding and any gutter. The eaves beam is commonly located outside the outer flange of the column – in some cases at a significant distance. Although the eaves beam can be designed to resist axial force, in this publication it is assumed that the longitudinal bracing force and positional restraint at the eaves level is provided by a hot-rolled member, referred to as an 'eaves strut'.

Both the eaves beam and eaves strut are shown in Figure 11.1.

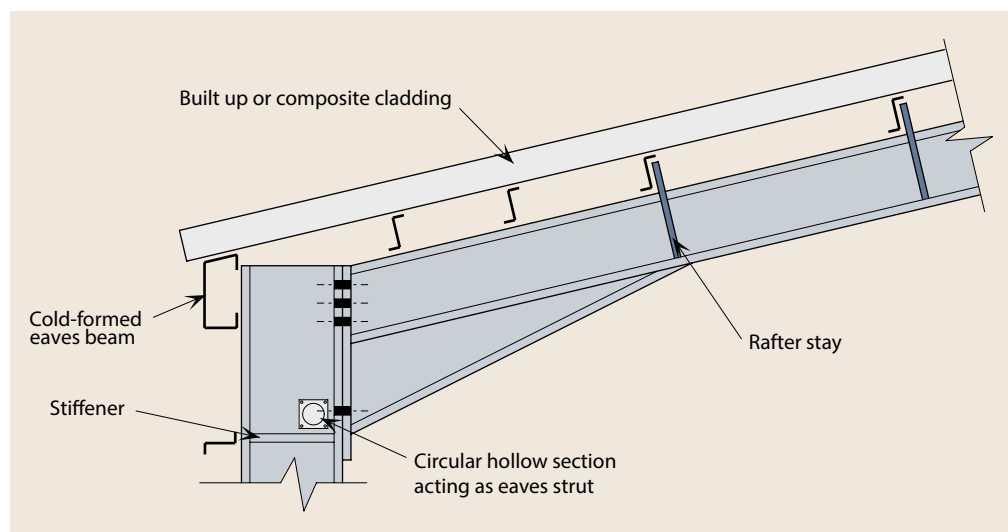


Figure 11.1
Haunch detail with
eaves beam and
eaves strut

11.2 Eaves strut

An eaves strut carries longitudinal forces, provides positional restraint to the top of the portal columns and is generally located within the depth of the column section, as shown in Figure 11.1.

Although there is no theoretical need for an eaves strut if there is vertical bracing at both ends of the structure (see Section 8.1), a strut is normally provided, to give positional restraint, meet robustness requirements (see Section 4.5.3) and stabilise frames during erection.

11.3 Purlins

Purlins are usually proprietary cold rolled thin gauge galvanized sections. Suitably shaped sections have been developed and tested by manufacturers and the resulting design data is presented in terms of load/span tables or software. The designer should calculate the loading on the purlins, noting that the loads may act 'up' or 'down', and should use the manufacturer's data to determine the size, type, and spacing of purlins. Particular care should be taken to understand the level of restraint to the purlins assumed in the tables (see Section 11.3.4). Manufacturers' tables normally assume that the purlins are restrained by the cladding and designers must ensure that this is so, or determine (usually with support from the manufacturer) the resistance of an unrestrained section.

Purlin systems may be specified with increased resistance (by overlapping, sleeves or thicker material), where required, notably in the end bay and at the supports of continuous members. If the section size is minimised by making the purlin continuous, any necessary single spans should be identified and carefully considered, as these could determine the section depth required, if a thicker section of the same size is not adequate.

Because purlins are required to provide restraint to the rafters, their location and spacing should be considered at the same time as verifying the stability of the rafters. A regular spacing of purlins along the length of the rafters is preferred, although closer spacing may be necessary around the eaves.

11.3.1 Localised loading

Local increases in loading arise due to wind loads on the end bays or due to snow drifting against an adjacent structure, parapet or gable. The usual way of dealing with these higher local loads is to provide an increased purlin thickness in these locations, without changing purlin spacing or depth. Alternatively, purlin spacing may be reduced.

11.3.2 Types of purlin

A range of purlin cross section shapes is available; typical shapes are shown in Figure 2.6. Each manufacturer produces its own specific shapes to maximise efficiency. Depths typically range from 120 to 340 mm, and thicknesses range from 1.2 to 3.2 mm. These purlins are generally suitable for frame spacings between 5 and 8 m and purlin spacings between 1.5 and 2 m, although spans and spacings can exceed these values in some cases, depending on the loading.

Within each manufacturer's range, there are specific shapes (and associated components such as anti-sag systems) that are used for longer spans, flatter roof pitches, complex roof details, and variations in types of roofing (for metal sheeting, tiled roofs, etc.). Reference to the manufacturer's catalogue or software and early discussions with the manufacturer will ensure the correct selection of purlin type.

11.3.3 Purlin cleats

Purlins are attached to rafters using cleats that are usually welded or bolted to the rafter in the shop before delivery to site. The cleats are often provided by the purlin manufacturer, in which case it is likely that they will have been designed for the specific shape of purlin. However, generic bolted cleats made from an angle section or simple flat plates welded to the rafter may also be used in many cases, either unstiffened or stiffened.

If the main member is drilled to accommodate bolted cleats;

- Ordinary fastener holes in a compression zone of the cross section need not be allowed for, provided that they are filled by fasteners
- Fastener holes in the tension flange may be ignored, provided that for the tension flange:

$$\frac{A_{f,\text{net}} 0.9 f_u}{\gamma_{M2}} \geq \frac{A_f f_y}{\gamma_{M0}}$$

where;

$A_{f,\text{net}}$	is the net area of the tension flange
A_f	is the gross area of the tension flange
f_u	is the ultimate tensile strength of the member
f_y	is the yield strength of the member
γ_{M2}	is the partial factor for resistance of cross sections in tension to fracture (given as 1.1 in the UK NA)
γ_{M0}	is the partial factor for resistance of cross sections (given as 1.0 in the UK NA).

11.3.4 Purlin restraint

Restraint by cladding

Tabulated values of purlin resistances are generally based on the assumption that the flange attached to the cladding is continuously restrained; designers should satisfy themselves that the cladding will provide this restraint.

Use of anti-sag systems

Under uplift (wind) loading, the inside flange of the purlin is in compression and unrestrained by the sheeting. Cladding fixed to the outer flange increases the resistance to lateral-torsional buckling (by acting as a tension flange restraint) – it may be that no additional restraint is needed to the inner flange.

Resistance may also be increased by specifying an anti-sag system, consisting of ties between the purlins, which provide restraint to the inner flange at discreet points within the purlin span.

Anti-sag ties are usually small rods, tubes or angles bolted or clipped between purlins at mid-span or third points. A typical arrangement of anti-sag ties is shown in Figure 11.2. Anti-sag ties are used for the following purposes:

- to restrain the purlins against lateral-torsional buckling under wind uplift conditions
- to locate and align the purlins in the construction condition (before the installation of the cladding)
- to provide additional support to the down-slope component of the load on the cladding.

Detailed information for specific requirements should be obtained from the manufacturer’s catalogue and software, which will give information on the slopes, spans, and purlin spacing at which ties are required.

As an alternative to the use of anti-sag ties, heavier purlin sections can be provided – this option has the benefit of reducing work at height, but may make installation of cladding more difficult, without the anti-sag ties to help align the purlins.

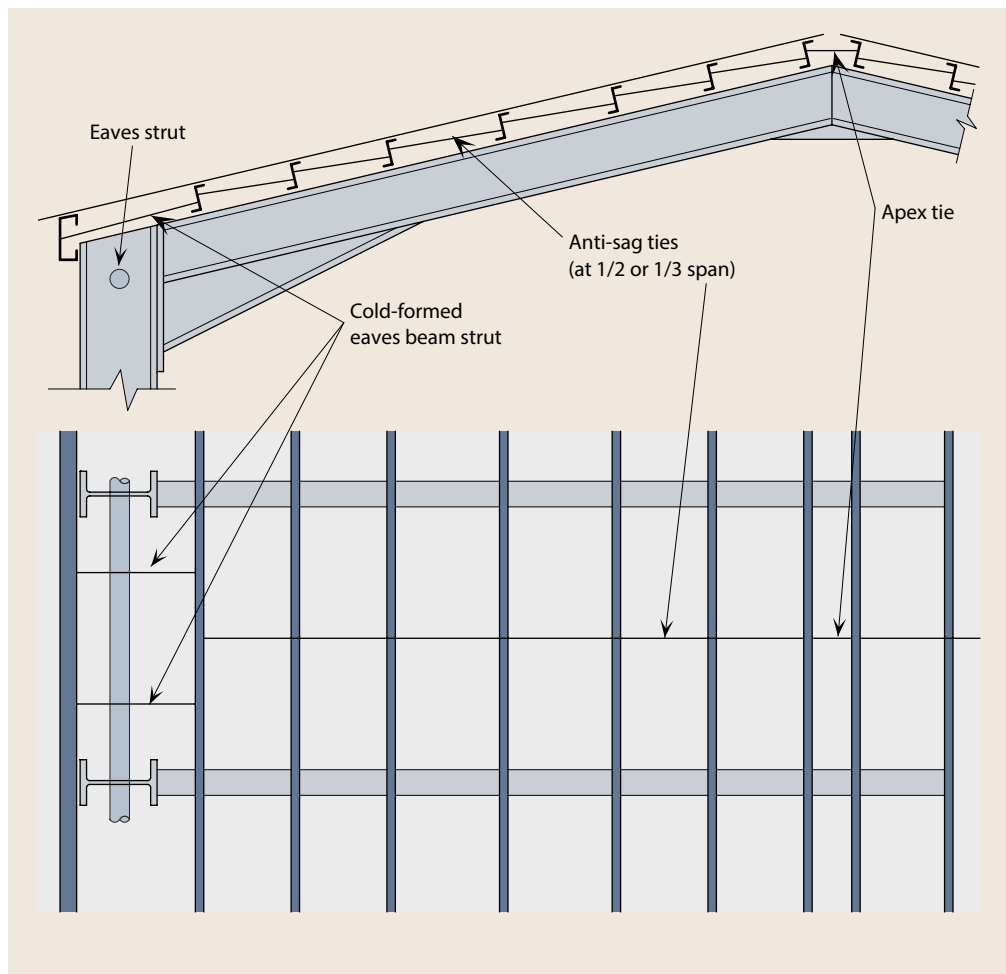
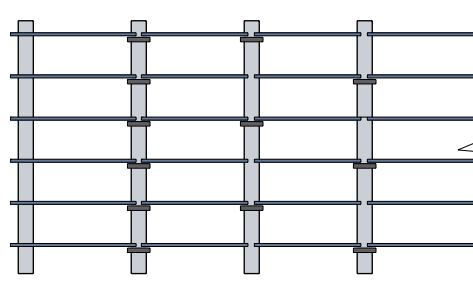


Figure 11.2
Typical anti-sag
system

11.3.5 Purlin layout

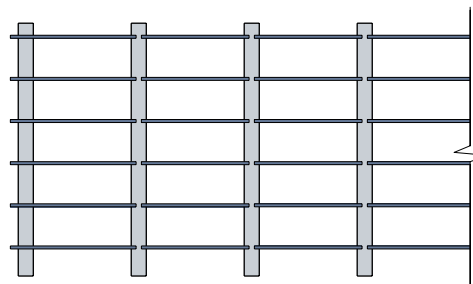
Most manufacturers produce guidance on typical purlin layouts that are efficient for various situations. These layouts are governed by such aspects as maximum purlin length (generally not more than 16 m for transport and site access reasons) and the opportunity to provide continuity by the use of sleeves or overlaps for maximum efficiency.

The examples of purlin arrangement shown in Table 11.1 are provided to illustrate the most commonly used systems.



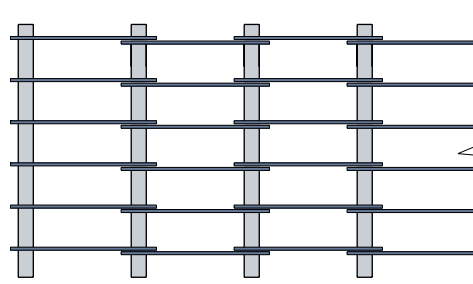
Single-span lengths - sleeved system

In sleeved systems, each purlin is the length of a single span but sleeves are provided at alternate supports so that each purlin is effectively continuous across two spans. At the penultimate support, sleeves are provided at each purlin, to provide semi-continuity and additional resistance in the end bay. This system is considered to be the most efficient for buildings with frame centres between 5 m and 7 m. Heavier sections can be provided in the end bay, if necessary.



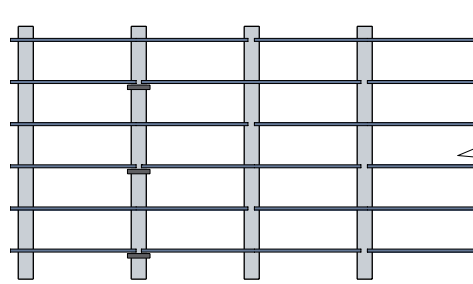
Single-span lengths - butted system

Single-span systems have a lower capacity than the other systems, but are simpler to fix, either over the rafters or between rafter webs. This layout may be used for small buildings with close frame centres, such as agricultural buildings.



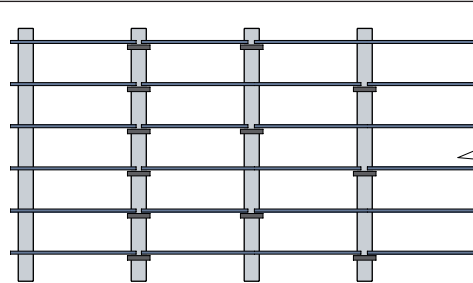
Single-span lengths - overlapping system

An overlapping system provides greater continuity and can be used for heavy loads and long spans. It is best suited to buildings with a large number of bays.



Double-span lengths - non-sleeved system

In this system, the double-span lengths are staggered. Sleeves are provided at the penultimate supports to cater for the hogging moment at this location. The resistance will generally be less than for the equivalent double-span sleeved system, but double-span purlins use fewer components and lead to faster erection.



Double-span lengths - sleeved system

In double-span sleeved systems, the double-span lengths are staggered and sleeves are provided at alternate supports to ensure semi-continuity. Sleeves are provided to every purlin at the penultimate support. A double-span sleeved system has a slightly higher resistance than the double-span non-sleeved system and has the advantages of semi-continuity at all sleeve positions. This system is generally limited to bay sizes less than 8 m, for reasons of transport and erection. Heavier purlins can be provided in the end bays, if necessary.

Table 11.1
Typical purlin
arrangements

11.3.6 Purlins providing rafter restraint

Purlins are usually required to provide restraint to the rafters. It is common practice not to check purlins for forces arising from the restraint of portal frame rafters, provided that all the following conditions are met:

- The purlins are restrained by roof sheeting.
- The bracing to the rafters is of adequate stiffness in the plane of the roof, or alternatively the roof sheeting is capable of acting as a diaphragm.
- The loading arises predominantly from roof loads.

The three conditions are usually satisfied, because roof sheeting will be adequately fixed, with sufficient robustness to act as a diaphragm.

Where it is necessary to design the purlins for axial force, the designer should refer to Section 10 of BS EN 1993-1-3^[38].

11.4 Side rails

Essentially, side rail design and detailing is very similar to that for purlins, and often the sections used are the same. In the case of side rails, the major loading to be resisted is that due to wind on the sides of the building. The self weight deflection of the side rails due to bending about the weak axis is counteracted by the provision of anti-sag bars and tension wires at mid-span or third points (Figure 11.3).

Special details are required for side rails in fire boundary conditions (see Section 12.1.1).

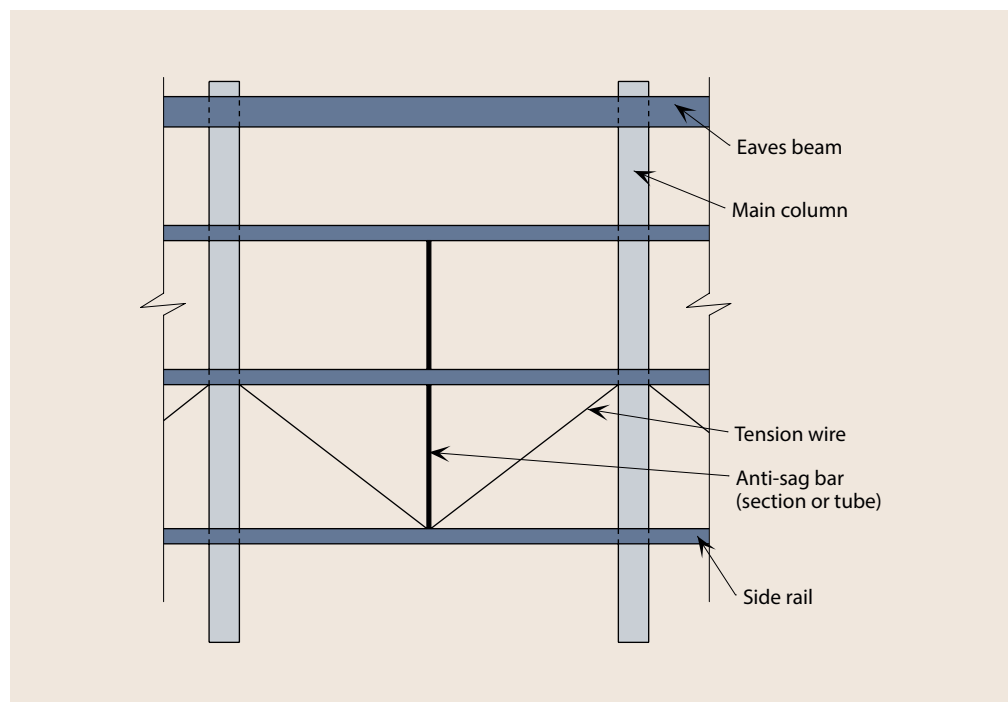
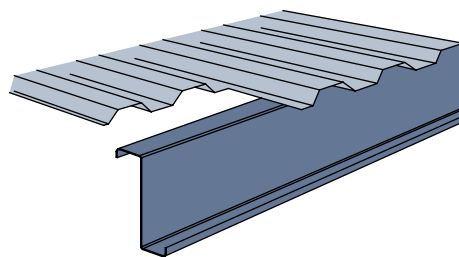


Figure 11.3
Typical side rail layout

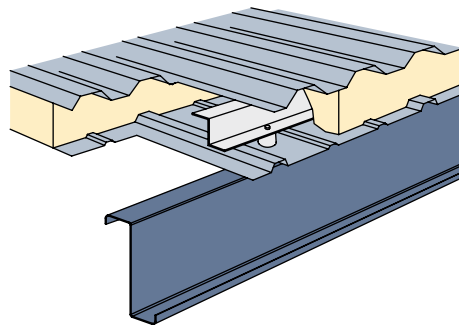
CLADDING

There are a number of proprietary types of cladding on the market. These tend to fall into broad categories, which are described in Table 12.1.

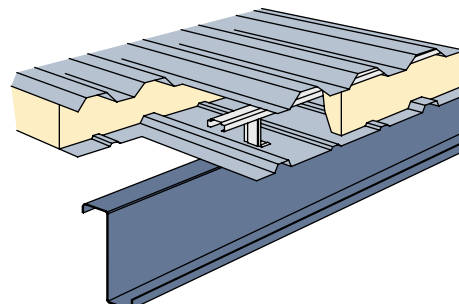


Single-skin trapezoidal sheeting

Single-skin sheeting is widely used in agricultural and industrial structures where no insulation is required. It can generally be used on roof slopes down to 4°, provided that the laps and sealants are as recommended by the manufacturers for shallow slopes. The sheeting is fixed directly to the purlins and side rails, and provides positive restraint. In some cases, single-skin sheeting is used with insulation suspended directly beneath the sheeting.



a. System with plastic ferrule and Z spacers



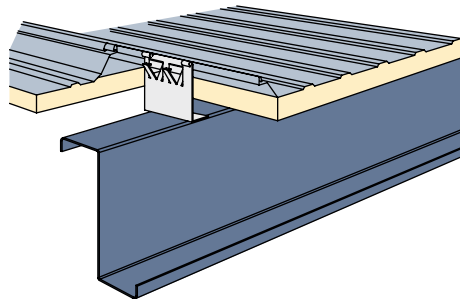
b. System with rail-and-bracket spacers

Double-skin system

Double skin or built-up roof systems usually use a steel liner fastened to the purlins, followed by a spacer system, insulation, and outer sheet. Because the connection between the outer and inner sheets may not be sufficiently stiff, the liner tray and fixings must be chosen so that they alone will provide the required level of restraint to the purlins. Alternative forms of spacer systems using a plastic ferrule and z or rail-and-bracket combination are available.

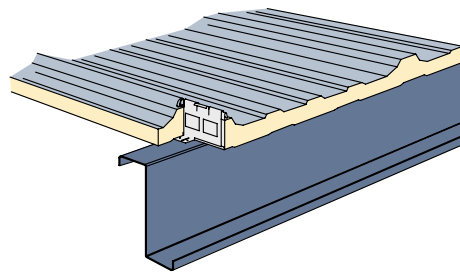
With adequate sealing of joints, the liner trays may be used to form an airtight boundary. Alternatively, an impermeable membrane can be provided on top of the liner tray.

Table 12.1
Cladding systems



Standing seam sheeting

Standing seam sheeting has concealed fixings and can be fixed in lengths of up to 30 m. The advantage is that there are no penetrations directly through the sheeting that could lead to water leakage. The fastenings are in the form of clips that connect internally to the profiled joints between adjacent sheets that hold the sheeting down but allow it to move longitudinally. Although the standing seam system provides less restraint to the purlins than a conventional system, a correctly fixed liner tray will provide adequate restraint.



Composite or sandwich panels

Composite or sandwich panels are formed by creating a foam or mineral wool insulation layer between the outer and inner layers of sheeting. Composite panels have good spanning capabilities, due to composite action in bending. Both standing seam and direct fixing systems are available. The manufacturers should be consulted for more information on the level of restraint provided to the purlins.

Table 12.1
Cladding systems
(continued)

12.1.1 Walls in fire boundary conditions

Where buildings are close to a site boundary, the UK Building Regulations require that the wall is designed to prevent spread of fire to the adjacent property. Fire tests have shown that a number of types of panel can perform adequately, provided that they remain fixed to the structure. Further guidance should be sought from the manufacturers.

Due to the relatively small scale used for the fire test specimens, slotted holes in the rails were needed to accommodate thermal expansion. For this reason, some manufacturers consider slotted holes are necessary in full size construction. In order to ensure that this does not compromise the stability of the column by removing the restraint under normal conditions, the slotted holes should be fitted with washers made from a material that will melt at high temperatures and allow the side rail to move relative to the column under fire conditions only (Figure 12.1).

Although slotted holes and plastic washers may be required so as to not invalidate the manufacturer's warranty, others consider that there is sufficient out-of-plane movement in full size panels to accommodate the thermal expansion and that slotted holes are unnecessary^[20].

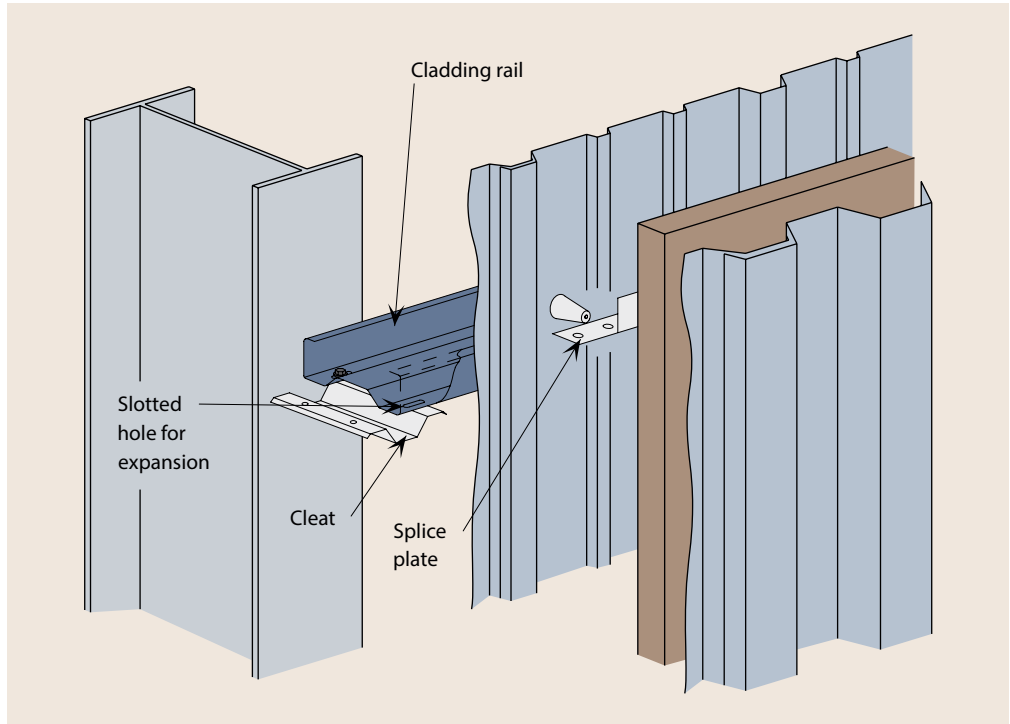


Figure 12.1
Typical fire wall
details showing
slotted holes for
expansion

SERVICEABILITY ASPECTS OF FRAME DESIGN

13.1 Deflections

Elastic analysis is used to determine the deflections of the frame at the serviceability limit state. Only deflections due to variable actions are considered.

In some cases, the frame is preset such that the deflections under the permanent actions leave the frame with (for example) vertical columns. The degree of pre-set is partly a matter of calculation and partly a matter of experience – the steelwork contractor should be consulted if pre-setting the frame is being considered.

The maximum acceptable deflections in portal frames will depend on many factors, such as appearance, the building use and cladding type, and should be agreed with the client.

13.1.1 Deflection sensitive details

Advice is given in the following paragraphs on the influence of construction details on appropriate deflection limits.

Sheeting

Limits on differential deflection between adjacent portal frames are necessary to prevent the fixings between the sheets and the frame from becoming overstressed, resulting in tearing of the sheeting, and leakage.

Portal frames clad in steel sheeting deflect significantly less than the deflection calculated for the bare frame. This is due to the sheeting acting as a stressed skin diaphragm, which provides a considerable stiffening effect to the structure. The actual deflection depends on the building proportions and cladding type, but reductions in horizontal deflections of over 50% (from those calculated for the bare frame) are typical of real structures.

Gables

A sheeted and/or braced gable end is very stiff in its own plane and the deflections can be ignored. The calculated differential deflections between the end frame and the adjacent frame (at the ridge and at the eaves) can be very high. This differential deflection will always be modified by the presence of the roof sheeting and roof bracing, particularly if the roof bracing is located in the end bays.

Masonry

When brick or blockwork side walls are constructed such that they receive support from the steel frame, they should be detailed to allow them to deflect with the frame by using a compressible damp proof course at the base of the wall. Suitable restraint should be provided at the top of the brickwork panel and at intermediate points, if necessary. If brickwork is continued around the steel columns, forming stiff piers, it is unreasonable to expect the panels to deflect with the frame. In this case, more onerous deflection limits should be applied to the frame.

Base fixity

In order to provide stability during erection, it has become common to use four holding down bolts, even with nominally pinned bases. In this situation, it is reasonable to use a base stiffness of 20% of the column stiffness when calculating SLS deflections, as noted in Section 6.4.

Cranes

Where crane girders are supported directly from portal frames, the need to control deflections at the crane level is likely to result in stiffer sections for the frames. The limit on spread should be determined in agreement with the client and the crane manufacturer.

Ponding

To ensure proper discharge of rainwater from a nominally flat roof, or from a very low-pitched roof (slope less than 1:20), deflections under permanent and variable actions should be checked to ensure that water does not pond.

13.1.2 Existing guidance on deflection limits

The recommendations presented in Table 13.1 are taken from SCI Publication P070^[39]. That publication noted (in 1991) that “Early feedback on this table has suggested that some of the values may be more stringent than is necessary. Pending outcome of a wider consultation on this subject the indicative numerical values given in this table should be regarded as provisional.” Despite this note, the deflection limits have been accepted by many designers as the *de facto* standard and are unchanged in Table 13.1.

When considering horizontal deflections, the more onerous of the requirements for the side cladding and the roof cladding should be adopted. For the vertical deflection at the ridge, both the criteria should be observed.

Differential deflection

The criteria for differential deflection between frames will be most critical for the frame nearest the gable end or next to any internal or division walls that are connected to the steel frame.

It is recognised that the in-plane stiffness of the roofing will reduce the differential deflection between adjacent frames to varying degrees, depending on the form of the roofing and geometrical factors such as the slope of the roof and the spacing of the frames. This is particularly important for the penultimate frame adjacent to a stiffer end gable.

Table 13.1 gives recommended limits for both the absolute and the differential deflections of portal frames. The absolute deflection of portal frame buildings depends on the plan proportions of the building as well as on the type of roof system. The absolute deflection limits in Table 13.1 should therefore be compared with the calculated deflection of a bare steel frame, taking account of base fixity, unless the designer is able to justify the stiffening effect of the roof and cladding system. Note that standing seam roof systems possess little stiffness, whereas double skin roof systems are quite stiff.

The differential deflection limits in Table 13.1 may be compared with the calculated deflection of a frame that has restraint from the roof system.

a. Horizontal deflection at eaves:

TYPE OF CLADDING	ABSOLUTE DEFLECTION	DIFFERENTIAL DEFLECTION RELATIVE TO ADJACENT FRAME
<i>Side cladding:</i>		
Profiled metal sheeting	$\leq h/100$	--
Fibre reinforced sheeting	$\leq h/150$	--
Brickwork	$\leq h/300$	$\leq (h^2 + b^2)^{0.5}/660$
Hollow concrete blockwork	$\leq h/200$	$\leq (h^2 + b^2)^{0.5}/500$
Precast concrete units	$\leq h/200$	$\leq (h^2 + b^2)^{0.5}/330$
<i>Roof cladding:</i>		
Profiled metal sheeting	--	$\leq b/200$
Fibre reinforced sheeting	--	$\leq b/250$

b. Vertical deflection at ridge (for rafter slopes $\geq 3^\circ$):

TYPE OF ROOF CLADDING	DIFFERENTIAL DEFLECTION RELATIVE TO ADJACENT FRAME
Profiled metal sheeting	$\leq b/100$ and $\leq (b^2 + s^2)^{0.5}/125$
Fibre reinforced sheeting	$\leq b/100$ and $\leq (b^2 + s^2)^{0.5}/165$

Table 13.1
Recommended
deflection limits

Notes:

- The calculated deflections are those due to:
 - wind actions
 - imposed roof loads
 - snow loads
 - 80% of (wind actions and snow loads).
- The above values are recommendations from reference 39. Some of the values may be more stringent than necessary.
- The values of h , b , and s are defined in Figure 13.1.
- The height h should always be taken as the height to eaves, not the height of the masonry panel.

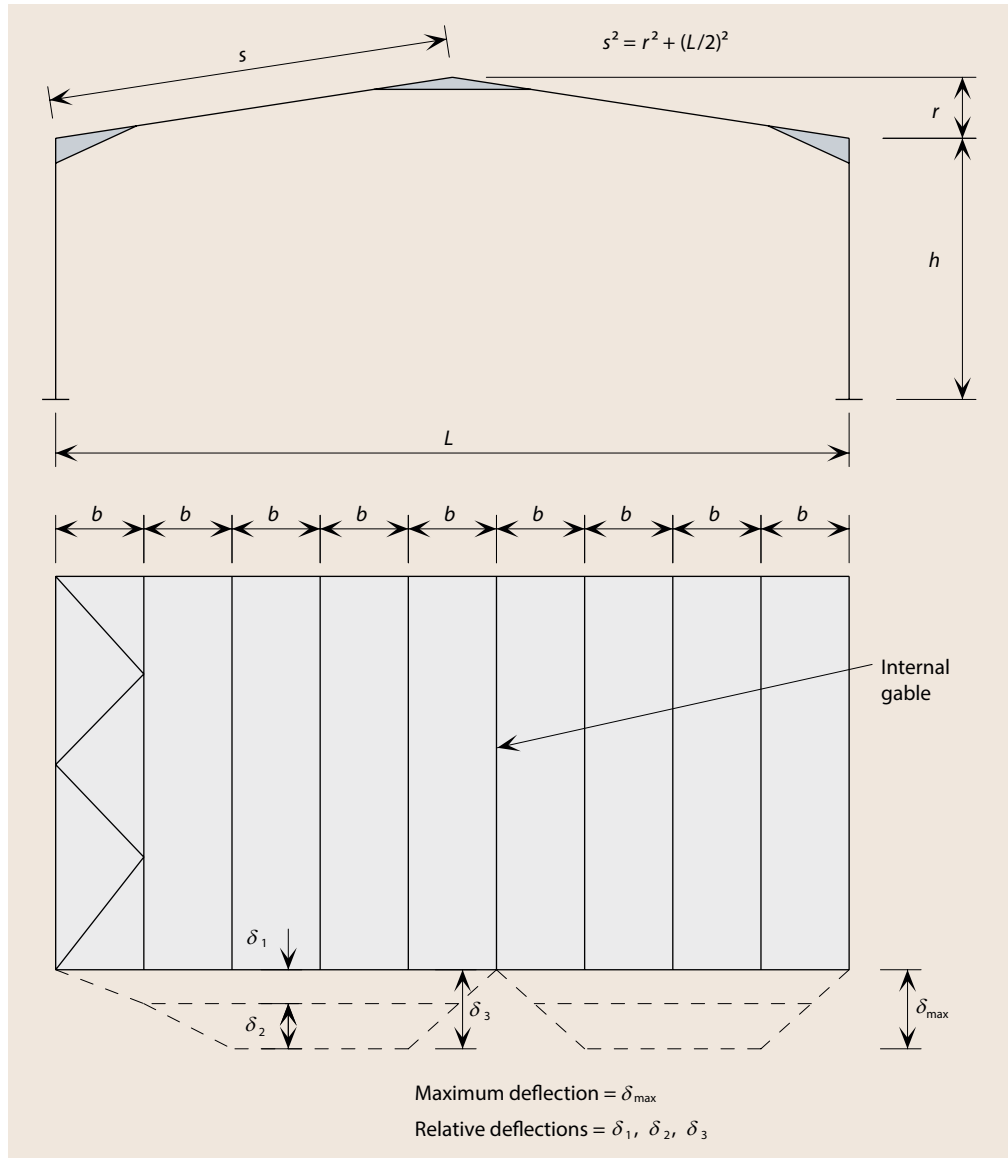


Figure 13.1
Dimensions to be used in determining deflection limits in Table 13.1

13.2 Thermal expansion

In the UK, temperature movements are generally small and no additional calculations are required where the spacing of expansion joints is within the limits in Table 13.2.

COMPONENT	SITUATION	SPACING (m)
Steel framed industrial buildings	Generally	150
	With high internal temperatures	125
Roof sheeting ¹	Simple construction	100
	Continuous construction	50
Brick or block walls ²	Clay bricks	12
	Calcium silicate bricks	8
	Concrete blocks	6

Table 13.2
Maximum spacing of expansion joints in buildings

- 1 A portal frame building that is braced in the longitudinal direction to resist wind loads would be considered as being of simple construction in this direction.
- 2 This is a guide only and refers to the expansion joints in the brickwork to structure connection; reference should be made to UK NA to BS EN 1996-2^[40].

13.2.1 Expansion joints

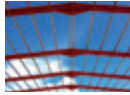
The provision of satisfactory expansion joints is neither easy nor cheap, and it is usually better to detail the structure such that joints can be avoided. Expansion joints should be provided only when they are absolutely necessary and the alternative of resisting expansion by the use of braced bays should always be considered. Where expansion joints are provided, care should be taken to ensure that they are properly detailed to ensure that they cannot cause leaks in the cladding due to differential movement.

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CREDITS



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13 Example of horizontal spanning sheeting



14 Example of large windows and use of composite panels with dado brick wall



14 Example of horizontal composite panels and 'ribbon' windows

APPENDIX A - PRELIMINARY SIZING

A.1 Introduction

This Appendix contains tables of member sizes for columns and rafters of single span portal frames at the preliminary design stage. Further detailed calculations will be required at the final design stage. The tabulated sizes take no account of:

- Stability at the ultimate limit state.
- Deflections at the serviceability limit state.

Further checks will therefore be required, which may necessitate increasing the size of the members in some cases.

A.2 Column and rafter sizes

Table A.1 presents data that will enable a rapid determination of member size to be made for estimating purposes. The span range is 15 to 40 m and the steel grade is S355. The information is based on earlier tables for S275, designed to BS 5950, given in *Design of single-span portal frames to BS 5950-1:2000* (SCI Publication P252). The S355 sections have been selected to provide a similar bending resistance to the S275 sections presented in P252. The assumptions made in creating this table are as follows:

- The roof pitch is 6°.
- The steel grade is S355.
- The rafter load is the design value of the permanent actions (including self weight) plus variable action (the imposed roof load). Wind loading has not been included - the presumption is that “gravity” loading will dominate the choice of member sizes.
- The haunch length is 10% of the span of the frame.
- A column is treated as restrained when torsional restraints are provided along its length (these columns are therefore lighter than the equivalent unrestrained columns).
- A column is treated as unrestrained if no torsional restraint can be provided along its length.

The member sizes given in the tables are suitable for preliminary design only. Where an asterisk (*) is shown in the table, a suitable section size has not been calculated, as the wind effects and second order effects are likely to be significant.

Table A.1 Preliminary sizes of columns and rafters for symmetrical single-span portal frame with 6° roof pitch (S355 steel)

DESIGN LOAD ON RAFTER (kN/m)	EAVES HEIGHT (m)	SPAN OF FRAME (m)						
		15	20	25	30	35	40	
Rafter	8	254x102x22 UKB	254x146x31 UKB	356x127x39 UKB	356x171x45 UKB	356x171x57 UKB	356x171x67 UKB	356x171x67 UKB
	8	254x102x22 UKB	254x146x31 UKB	356x127x39 UKB	356x171x51 UKB	356x171x67 UKB	356x171x82 UKB	457x191x82 UKB
	8	254x102x22 UKB	254x146x31 UKB	356x127x39 UKB	356x171x51 UKB	356x171x67 UKB	356x171x82 UKB	457x191x82 UKB
	8	*	254x146x31 UKB	356x127x39 UKB	356x171x51 UKB	356x171x67 UKB	356x171x82 UKB	457x191x82 UKB
Restrained column	8	305x165x40 UKB	305x165x46 UKB	406x178x67 UKB	457x191x82 UKB	533x210x92 UKB	610x229x113 UKB	610x229x113 UKB
	8	305x165x40 UKB	305x165x46 UKB	406x178x67 UKB	457x191x82 UKB	533x210x101 UKB	610x229x113 UKB	610x229x113 UKB
	8	305x165x40 UKB	305x165x54 UKB	406x178x67 UKB	457x191x82 UKB	533x210x101 UKB	610x229x125 UKB	610x229x125 UKB
	8	*	305x165x54 UKB	406x178x67 UKB	457x191x82 UKB	533x210x101 UKB	610x229x125 UKB	610x229x125 UKB
Unrestrained column	8	305x165x46 UKB	457x191x67 UKB	533x210x82 UKB	533x210x92 UKB	610x229x113 UKB	610x229x125 UKB	610x229x125 UKB
	8	406x178x60 UKB	457x191x82 UKB	610x229x101 UKB	610x229x113 UKB	686x254x125 UKB	762x267x147 UKB	762x267x147 UKB
	8	406x178x67 UKB	533x210x92 UKB	610x229x113 UKB	610x229x125 UKB	762x267x147 UKB	838x292x194 UKB	838x292x194 UKB
	8	*	610x229x101 UKB	610x229x125 UKB	762x267x147 UKB	838x292x194 UKB	914x305x224 UKB	914x305x224 UKB
Rafter	10	254x102x25 UKB	254x146x31 UKB	356x171x45 UKB	356x171x57 UKB	356x171x67 UKB	356x171x82 UKB	457x191x82 UKB
	10	254x102x25 UKB	254x146x31 UKB	356x171x45 UKB	356x171x57 UKB	356x171x67 UKB	356x171x82 UKB	457x191x82 UKB
	10	254x102x25 UKB	356x127x39 UKB	356x171x45 UKB	356x171x57 UKB	356x171x67 UKB	356x171x82 UKB	457x191x82 UKB
	10	*	356x127x39 UKB	356x171x45 UKB	356x171x57 UKB	356x171x67 UKB	356x171x82 UKB	457x191x82 UKB
Restrained column	10	305x165x40 UKB	406x178x60 UKB	406x178x74 UKB	533x210x92 UKB	610x229x113 UKB	610x229x125 UKB	610x229x125 UKB
	10	305x165x40 UKB	406x178x60 UKB	457x191x82 UKB	533x210x92 UKB	610x229x113 UKB	610x229x125 UKB	610x229x125 UKB
	10	305x165x40 UKB	406x178x60 UKB	457x191x82 UKB	533x210x101 UKB	610x229x113 UKB	610x229x140 UKB	610x229x140 UKB
	10	*	406x178x60 UKB	457x191x82 UKB	533x210x101 UKB	610x229x125 UKB	610x229x140 UKB	610x229x140 UKB
Unrestrained column	10	406x178x54 UKB	457x191x74 UKB	533x210x92 UKB	610x229x101 UKB	610x229x125 UKB	610x229x125 UKB	610x229x125 UKB
	10	457x191x67 UKB	533x210x92 UKB	610x229x113 UKB	686x254x125 UKB	686x254x140 UKB	610x305x149 UKB	610x305x149 UKB
	10	457x191x74 UKB	610x229x101 UKB	610x229x125 UKB	762x267x147 UKB	838x292x194 UKB	838x292x194 UKB	838x292x194 UKB
	10	*	610x229x113 UKB	686x254x140 UKB	838x292x194 UKB	914x305x224 UKB	914x305x224 UKB	914x305x224 UKB
Rafter	12	254x102x28 UKB	356x127x39 UKB	356x171x51 UKB	356x171x67 UKB	457x191x82 UKB	533x210x92 UKB	533x210x92 UKB
	12	254x102x28 UKB	356x127x39 UKB	356x171x51 UKB	356x171x67 UKB	457x191x82 UKB	457x191x82 UKB	457x191x82 UKB
	12	254x102x28 UKB	356x127x39 UKB	356x171x51 UKB	356x171x67 UKB	457x191x82 UKB	457x191x82 UKB	457x191x82 UKB
	12	*	356x127x39 UKB	356x171x51 UKB	356x171x67 UKB	457x191x82 UKB	457x191x82 UKB	457x191x82 UKB
Restrained column	12	305x165x40 UKB	406x178x67 UKB	457x191x82 UKB	533x210x101 UKB	610x229x125 UKB	610x229x140 UKB	610x229x140 UKB
	12	305x165x40 UKB	406x178x67 UKB	457x191x82 UKB	533x210x101 UKB	610x229x125 UKB	686x254x140 UKB	686x254x140 UKB
	12	305x165x46 UKB	406x178x67 UKB	533x210x92 UKB	610x229x113 UKB	610x229x125 UKB	686x254x140 UKB	686x254x140 UKB
	12	*	406x178x67 UKB	533x210x92 UKB	610x229x113 UKB	610x229x125 UKB	686x254x140 UKB	686x254x140 UKB
Unrestrained column	12	406x178x60 UKB	533x210x82 UKB	610x229x101 UKB	610x229x113 UKB	610x229x125 UKB	610x229x125 UKB	610x229x125 UKB
	12	457x191x74 UKB	610x229x101 UKB	610x229x113 UKB	686x254x140 UKB	610x305x149 UKB	610x305x149 UKB	610x305x149 UKB
	12	457x191x82 UKB	610x229x113 UKB	686x254x140 UKB	838x292x194 UKB	838x292x194 UKB	838x292x194 UKB	838x292x194 UKB
	12	*	610x229x125 UKB	610x229x149 UKB	836x292x176 UKB	914x305x224 UKB	914x305x224 UKB	914x305x224 UKB

Table A.1 Preliminary sizes of columns and rafters for symmetrical single-span portal frame with 6° roof pitch (S355 steel) (continued)

DESIGN LOAD ON RAFTER (kN/m)	EAVES HEIGHT (m)	SPAN OF FRAME (m)							
		15	20	25	30	35	40		
Rafter	6	254x146x31 UKB	356x171x45 UKB	356x171x57 UKB	457x191x67 UKB	457x191x82 UKB	457x191x92 UKB	533x210x92 UKB	533x210x92 UKB
	8	254x146x31 UKB	356x171x45 UKB	356x171x57 UKB	457x191x67 UKB	457x191x82 UKB	457x191x92 UKB	533x210x92 UKB	533x210x92 UKB
	10	254x146x31 UKB	356x171x45 UKB	356x171x57 UKB	457x191x67 UKB	457x191x82 UKB	457x191x92 UKB	533x210x92 UKB	533x210x92 UKB
	12	*	356x171x45 UKB	356x171x57 UKB	457x191x67 UKB	457x191x82 UKB	457x191x92 UKB	533x210x92 UKB	610x229x101 UKB
Restrained column	6	305x165x46 UKB	406x178x74 UKB	533x210x92 UKB	610x229x113 UKB	610x229x140 UKB	610x229x140 UKB	686x254x140 UKB	686x254x140 UKB
	8	305x165x54 UKB	406x178x74 UKB	533x210x92 UKB	610x229x113 UKB	610x229x140 UKB	610x229x140 UKB	686x254x170 UKB	686x254x170 UKB
	10	305x165x54 UKB	406x178x74 UKB	533x210x101 UKB	610x229x125 UKB	610x229x140 UKB	610x229x140 UKB	686x254x170 UKB	686x254x170 UKB
	12	*	406x178x74 UKB	533x210x101 UKB	610x229x125 UKB	610x229x140 UKB	610x229x140 UKB	686x254x170 UKB	686x254x170 UKB
Unrestrained column	6	457x191x67 UKB	533x210x82 UKB	610x229x101 UKB	610x229x125 UKB	610x229x140 UKB	610x229x140 UKB	610x305x149 UKB	610x305x149 UKB
	8	457x191x82 UKB	610x229x101 UKB	686x254x125 UKB	762x267x147 UKB	762x267x147 UKB	762x267x147 UKB	838x292x176 UKB	838x292x176 UKB
	10	533x210x92 UKB	610x229x125 UKB	762x267x147 UKB	610x305x149 UKB	610x305x149 UKB	610x305x149 UKB	914x305x224 UKB	914x305x224 UKB
	12	*	686x254x140 UKB	610x305x149 UKB	610x305x149 UKB	610x305x224 UKB	610x305x224 UKB	914x305x289 UKB	914x305x289 UKB
Rafter	6	254x146x31 UKB	356x171x45 UKB	356x171x67 UKB	457x191x82 UKB	457x191x82 UKB	457x191x82 UKB	533x210x92 UKB	533x210x92 UKB
	8	254x146x31 UKB	356x171x45 UKB	356x171x67 UKB	457x191x82 UKB	457x191x82 UKB	457x191x82 UKB	533x210x92 UKB	533x210x92 UKB
	10	254x146x31 UKB	356x171x45 UKB	356x171x67 UKB	457x191x82 UKB	457x191x82 UKB	457x191x82 UKB	533x210x92 UKB	533x210x92 UKB
	12	*	356x171x45 UKB	356x171x67 UKB	457x191x82 UKB	457x191x82 UKB	457x191x82 UKB	533x210x92 UKB	533x210x92 UKB
Restrained column	6	305x165x54 UKB	457x191x82 UKB	533x210x101 UKB	610x229x125 UKB	610x229x140 UKB	610x229x140 UKB	686x254x170 UKB	686x254x170 UKB
	8	305x165x54 UKB	457x191x82 UKB	533x210x101 UKB	610x229x125 UKB	610x229x140 UKB	610x229x140 UKB	762x267x173 UKB	762x267x173 UKB
	10	406x178x60 UKB	457x191x82 UKB	533x210x101 UKB	610x229x125 UKB	610x229x140 UKB	610x229x140 UKB	762x267x173 UKB	762x267x173 UKB
	12	*	457x191x82 UKB	533x210x101 UKB	610x229x125 UKB	610x229x140 UKB	610x229x140 UKB	838x292x176 UKB	838x292x176 UKB
Unrestrained column	6	457x191x67 UKB	533x210x92 UKB	610x229x113 UKB	610x229x125 UKB	610x229x140 UKB	610x229x140 UKB	610x305x149 UKB	610x305x149 UKB
	8	457x191x82 UKB	610x229x113 UKB	686x254x140 UKB	610x305x149 UKB	610x305x149 UKB	610x305x149 UKB	838x292x194 UKB	838x292x194 UKB
	10	533x210x101 UKB	610x229x125 UKB	610x305x149 UKB	610x305x149 UKB	610x305x224 UKB	610x305x224 UKB	914x305x253 UKB	914x305x253 UKB
	12	*	686x254x140 UKB	610x305x149 UKB	610x305x149 UKB	610x305x224 UKB	610x305x224 UKB	914x305x289 UKB	914x305x289 UKB

APPENDIX B - DETERMINATION OF THE ELASTIC CRITICAL FORCE AND MOMENT

Values of elastic critical force N_{cr} and elastic critical moment M_{cr} for an unrestrained length of member are given in sections B.1 and B.2 respectively.

The value of N_{cr} is required when calculating α_{cr} (see Section 6.5) and when calculating the flexural buckling resistance of members in accordance with Clause 6.3.1.2 of BS EN 1993-1-1. Note that an alternative approach to calculate the flexural buckling resistance, not requiring N_{cr} , is given in Clause 6.3.1.3; the two approaches give identical resistances.

The value of M_{cr} is required when calculating the lateral-torsional buckling resistance of members in accordance with Clause 6.3.2.3 of BS EN 1993-1-1. For rolled sections, the use of Clause 6.3.2.3 in preference to 6.3.2.2 (the general case) is recommended, as a higher lateral-torsional buckling resistance results.

B.1 N_{cr} for uniform members

$$N_{cr} = \frac{\pi^2 EI}{L^2}$$

Where:

- E is the modulus of elasticity (210000 N/mm²)
- I is the second moment of area about the relevant axis (the axis of buckling being considered)
- L is the buckling length.

B.2 M_{cr} for uniform members

B.2.1 General expression

This expression only applies to uniform straight members and when the cross section is bi-symmetric. Assuming the ends are not restrained against warping and assuming that the load is not destabilising, then:

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{L^2} \sqrt{\frac{I_w}{I_z} + \frac{L^2 GI_T}{\pi^2 EI_z}}$$

where:

- E is the modulus of elasticity (210000 N/mm²)
- G is the shear modulus (81000 N/mm²)
- I_z is the second moment of area about the minor axis
- I_T is the torsional constant of the member
- I_w is the warping constant of the member
- L is the beam length between points of lateral restraint
- C_1 is a factor that accounts for the shape of the bending moment diagram.

B.2.2 C_1 factor

According to the UK National Annex to BS EN 1993-1-1,

$$C_1 = \frac{M_{cr} \text{ for the actual bending moment diagram}}{M_{cr} \text{ for a uniform bending moment diagram}}$$

The factor C_1 may be determined from Table B.1 for a member with end moments or with intermediate transverse loading.

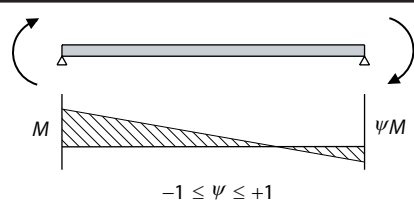


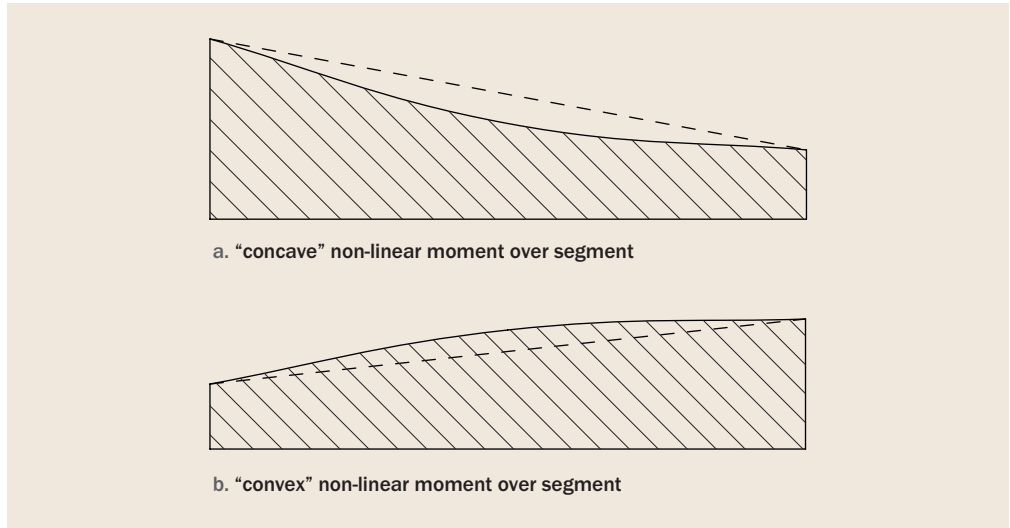
END MOMENT LOADING	ψ	C_1
	+1.00	1.00
	+0.75	1.17
	+0.50	1.36
	+0.25	1.56
	0.00	1.77
	-0.25	2.00
	-0.50	2.24
	-0.75	2.49
	-1.00	2.76
INTERMEDIATE TRANSVERSE LOADING		
		1.13
		2.60

Table B.1
 C_1 factor

If a non-linear bending moment diagram is concave, compared to a straight line between end moments, as shown in Figure B.1a, the bending moment shape may be conservatively taken as linear and the value of C_1 can be determined depending on ψ . For a non-linear diagram shape, as shown in Figure B.1b, which is convex compared to a straight line between the end moments, it is not conservative to assume a linear moment. It will always be conservative to assume a uniform moment ($C_1 = 1.0$). Alternatively, C_1 (and indeed M_{cr}) can be determined by software.

Figure B.1
Typical bending
moment diagrams
found in
portal frames



APPENDIX C - DESIGN OF MEMBERS WITH DISCRETE RESTRAINTS TO THE TENSION FLANGE

It is possible to take account of restraints to the tension flange. This may lead to a greater design buckling resistance of the member.

In order to take advantage of the stabilising effect of tension flange restraints the spacing between them must be less than the value of L_m as given in Clause BB.3.1.1 of BS EN 1993-1-1. Although this expression primarily covers segments between torsional restraints containing a plastic hinge, it can be used for the less onerous case of a segment without a plastic hinge.

Assuming the intermediate restraints to the tension flange are more closely spaced than L_m , values of the elastic critical buckling force and moment can be determined, that account for the benefits of tension flange restraint.

Limiting length, L_m

$$L_m = \frac{38i_z}{\sqrt{\frac{1}{57.4} \left(\frac{N_{Ed}}{A} \right) + \frac{1}{756C_1^2} \left(\frac{W_{ply}^2}{AI_T} \right) \left(\frac{f_y}{235} \right)^2}}$$

where

- N_{Ed} is the design value of the compression force in the member
- A is the cross sectional area of the member
- W_{ply} is the plastic section modulus of the member
- I_T is the torsional constant of the member
- f_y is the yield strength
- C_1 is a factor that accounts for the shape of the bending moment diagram over the length of the segment and is given in Section B.2.2 of this document. In calculating L_m , the most onerous value of C_1 anywhere in the segment should be taken.

C.1 N_{crT} for uniform members with discrete restraints to the tension flange

The elastic critical torsional buckling force for a length of I section between torsional restraints and with intermediate restraints to the tension flange is given in Clause BB.3.3.1 as:

$$N_{crT} = \frac{1}{i_s^2} \left(\frac{\pi^2 EI_z a^2}{L_t^2} + \frac{\pi^2 EI_w}{L_t^2} + GI_T \right)$$

where:

$$i_s^2 = i_y^2 + i_z^2 + a^2$$

L_t is the length of the segment along the member between torsional restraints to both flanges

a is the distance between the centroid of the member and the centroid of the restraining members, such as purlins restraining rafters (see Figure C.3)

I_T is the torsional constant of the member

I_w is the warping constant of the section

I_z is the second moment of area about the weak axis

E is the modulus of elasticity (210000 N/mm²)

G is the shear modulus (81000 N/mm²).

For tapered or haunched members, N_{crT} is calculated using the section properties of the shallow end.

C.2 M_{cr} for uniform members with discrete restraints to the tension flange

C.2.1 General expression

For the general case of a beam of varying depth but symmetrical about the minor axis, subject to a non-uniform moment:

$$M_{cr} = c^2 C_m M_{cr0} \quad \text{for beams with a linear bending moment diagram}$$

or

$$M_{cr} = c^2 C_n M_{cr0} \quad \text{for beams with a non-linear bending moment diagram}$$

where:

M_{cr0} is the elastic critical buckling moment for a beam with intermediate restraints to the tension flange, subject to uniform moment, calculated in accordance with Section C.2.2

c accounts for the taper ($c = 1$ for a uniform member)

The value of c is given by BS EN 1993-1-1 Clause BB.3.3.3, based on the depth at the shallower end of the member and limited to members where $1 \leq h_{\max}/h_{\min} \leq 3$. For non-uniform members with constant flanges, for which $h \geq 1.2b$ and $h/t_f \geq 20$, the following equations may be used: for tapered members or segments, see Figure C.1(a)

$$c = 1 + \frac{3}{\left(\frac{h}{t_f} - 9\right)} \left(\frac{h_{\max}}{h_{\min}} - 1\right)^{2/3} \quad (\text{BB.16})$$

for haunched members or segments, see Figure C.1(b) and Figure C.1(c)

$$c = 1 + \frac{3}{\left(\frac{h}{t_f} - 9\right)} \left(\frac{h_h}{h_s} - 1\right)^{2/3} \sqrt{\frac{L_h}{L_y}} \quad (\text{BB.17})$$

where:

- h_h is the additional depth of the haunch or taper, as shown in Figure C.1
- h_{\max} is the maximum depth of cross section within the length L_y
- h_{\min} is the minimum depth of cross section within the length L_y
- h_s is the vertical depth of the un-haunched section
- L_h is the length of haunch within the length L_y
- L_y is the length between points at which the compression flange is laterally restrained
- (h/t_f) is to be derived from the shallowest section.

The relevant dimensions when calculating the taper factor c are illustrated in Figure C.1.

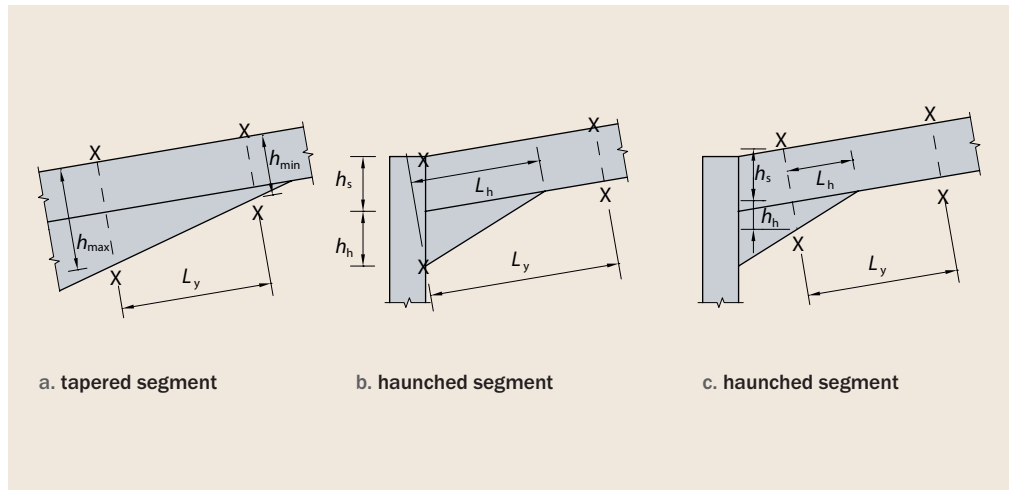


Figure C.1
Dimensions defining
taper factor

The factor C_m accounts for linear moment gradients. The value is given by the Expression (BB.13) of BS EN 1993-1-1 Annex BB as:

$$C_m = \frac{1}{B_0 + B_1\beta_t + B_2\beta_t^2} \quad (\text{BB.13})$$

where:

$$B_0 = \frac{1+10\eta}{1+20\eta}$$

$$B_1 = \frac{5\sqrt{\eta}}{\pi+10\sqrt{\eta}}$$

$$B_2 = \frac{0.5}{1+\pi\sqrt{\eta}} - \frac{0.5}{1+20\eta}$$

$$\eta = \frac{N_{crE}}{N_{crT}}$$

$$N_{crE} = \frac{\pi EI_z}{L_t^2}$$

L_y is the distance between the torsional restraints.

N_{crT} is the elastic critical torsional force for members with restraints to the tension flange, as defined in Section C.1 of this document.

β_t is the ratio of the algebraically smaller end moment to the larger end moment.

The factor C_n accounts for non-linear moment gradients. The value is given by expression (BB.14) of BS EN 1993-1-1, Clause BB.3.3.2. The expression given in BS EN 1993-1-1 has been corrected below.

$$C_n = \frac{12R_{max}}{\left[R_1 + 3R_2 + 4R_3 + 3R_4 + R_5 + 2(R_s - R_E) \right]}$$

where:

R_1 to R_5 are the values of R according to the following equation

$$R = \frac{M_{y,Ed} + aN_{Ed}}{f_y W_{pl,y}}$$

at the ends, quarter points and mid-length (see Figure C.2)

and only positive values of R should be included. In addition, only positive values of $(R_s - R_E)$ should be included, where:

R_E is the greater of R_1 or R_5

R_s is the maximum value of R anywhere in the length L_y

R_{max} is the maximum value of R anywhere in the length L_y , taking only positive values of R .

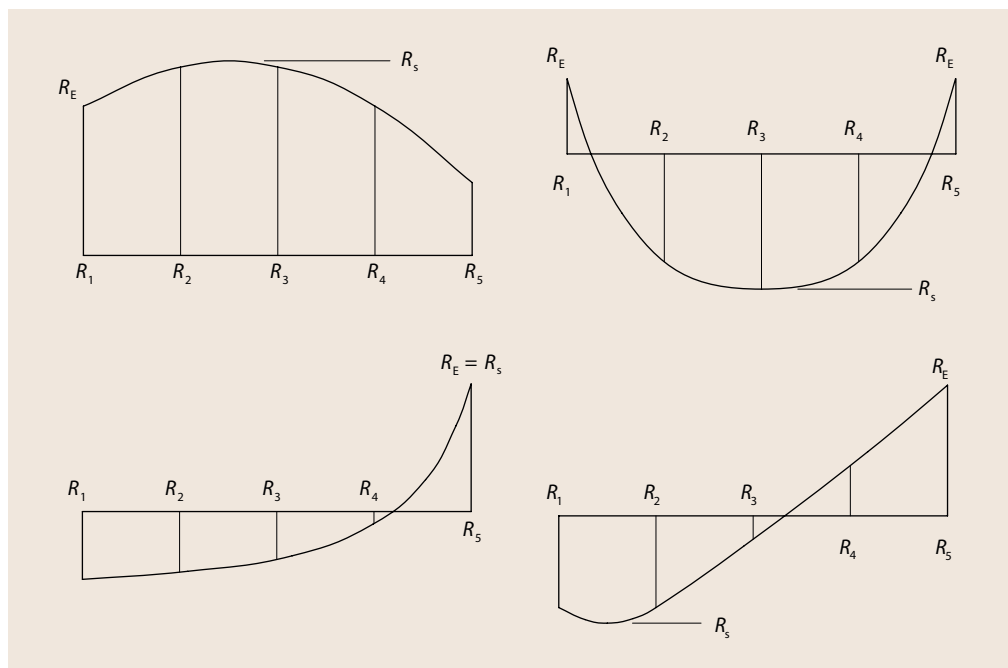


Figure C.2
Moment values

When calculating C_m (in accordance with Clause BB.3.3.1) or C_n (in accordance with Clause BB.3.3.2), bending moments that produce compression in the non-restrained flange should be taken as positive. Only positive values of R should be taken.

When calculating C_n (in accordance with Clause BB.3.3.2) it is assumed that the loads are applied at the shear centre.

C.2.2 Calculation of M_{cr0}

For uniform sections, symmetric about the minor axis, restrained along the tension flange at intervals:

$$M_{cr0} = \frac{1}{2a} \left(\frac{\pi^2 EI_z a^2}{L_t^2} + \frac{\pi^2 EI_w}{L_t^2} + GI_T \right),$$

therefore if the value of N_{crT} is calculated beforehand,

$$M_{cr0} = \frac{i_s^2}{2a} N_{crT}$$

$$\text{but } M_{cr0} \leq \frac{\pi^2 EI_z}{s^2} \sqrt{\frac{I_w}{I_z} + \frac{s^2 GI_T}{\pi^2 EI_z}}$$

where:

- N_{crT} is the elastic critical torsional buckling force as defined in Section C.1 of this document
- s is the distance between the restraints along the restrained longitudinal axis (e.g. the spacing of the purlins)
- I_T is the torsional constant of the member
- I_w is the warping constant of the member
- I_z is the second moment of area about the weak axis
- E is the modulus of elasticity (210000 N/mm²)
- G is the shear modulus (81000 N/mm²).

For tapered or haunched members, M_{cr0} is calculated using the section properties at the shallow end.

The parameters a , L_t and s are shown in Figure C.3.

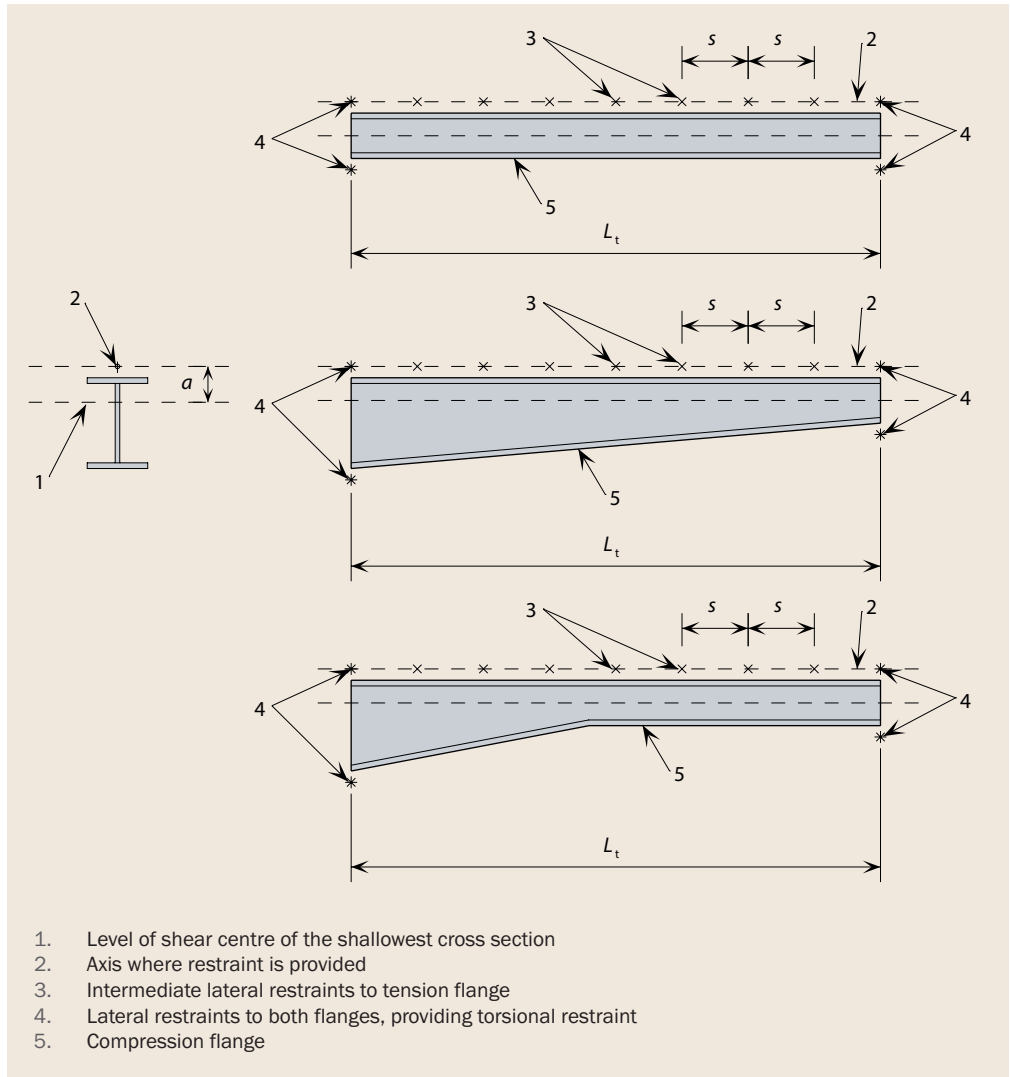


Figure C.3
 Arrangement of
 tension flange
 restraints

APPENDIX D - WORKED EXAMPLE

This example covers the elastic analysis and design of a single span symmetrical portal frame. It is intended to illustrate the steps involved in design; it is recommended that for efficient design, bespoke software is used.

Advance Universal Beam sections are used for the rafters and columns. All section dimensions and properties have been taken from the publication *Steel Building Design: Design Data in Accordance with Eurocodes and the UK National Annexes* (SCI/BCSA publication P363).

The following steps are demonstrated in this example:

1. Determination of frame geometry
2. Determination of Actions
3. Combinations of Actions
4. Preliminary sizing
5. Initial analysis
6. Assessment of sensitivity to second order effects
7. Calculation of frame imperfections
8. Frame analysis, including allowance for frame imperfections
9. Cross section verification
10. Buckling verification of the column and rafter
11. Haunch verification
12. Serviceability limit state verification.

Cross sectional and buckling verifications are presented for one combination of actions only; all combinations must be verified for completeness.

1 Frame geometry

The proposed frame is defined as shown below.

The nodal height of the columns is established from a requirement for 12 m clear height between the finished floor level (ffl) and the underside of the haunch. The base is assumed to be 300 mm below ffl. The rafter is assumed to be 500 mm deep, and the distance from the underside of the haunch to the node is assumed to be $1.5 \times$ rafter depth.

Nodal height = $300 + 12000 + 1.5 \times 500 = 13050$, say 13100 mm

The haunch length is assumed to be 10% of the span.

The portal frames are spaced at 8 m centres.

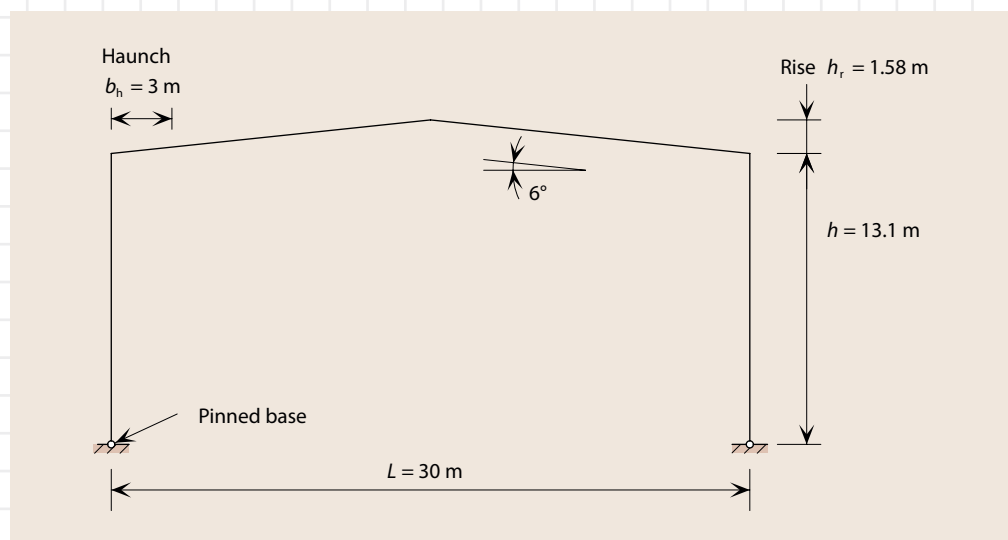


Figure D.1
Portal frame
dimensions

The cladding to the roof and walls is supported by purlins and side rails. The spacing of the rails and purlins, and the locations of restraints to the inside flanges, will be defined at the design stage.

2 Actions

2.1 Permanent actions

The characteristic value of load due to self weight is given by:

$$g_k = g_{\text{self weight}} + g_{\text{roof}}$$

$g_{\text{self weight}}$ is the self weight of the rafters

g_{roof} is the self weight of the roofing with purlins and services, taken as 0.55 kN/m^2

Therefore for an internal frame:

$$g_{\text{roof}} = 0.55 \times 8.0 = 4.4 \text{ kN/m}$$

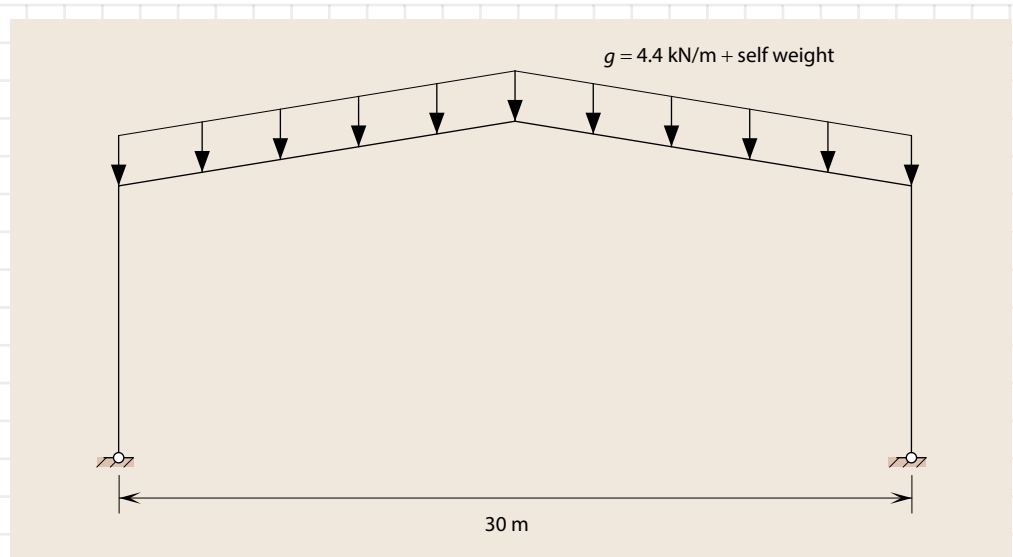


Figure D.2
Permanent actions

2.2 Snow loads

NA to
BS EN 1991-1-3

The snow load on the roof s has been determined from BS EN 1991-1-3 and the UK NA.

$$s = 0.4 \text{ kN/m}^2$$

Therefore for an internal frame:

$$q_s = 0.4 \times 8.0 = 3.2 \text{ kN/m}$$

This example only considers uniform snow loading. The asymmetric load cases described in the NA to BS EN 1991-1-3 NA.2.17 and Figure NA.2 should also be considered.

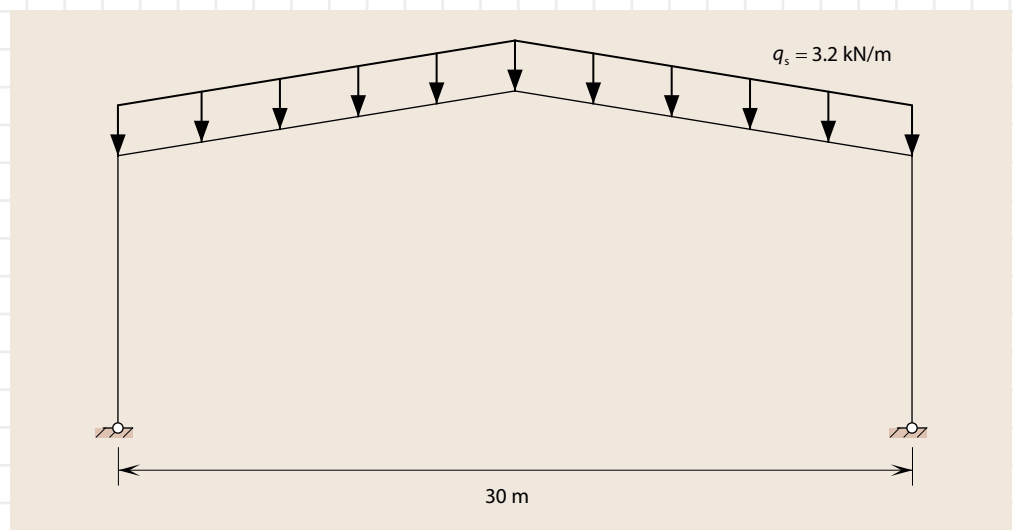


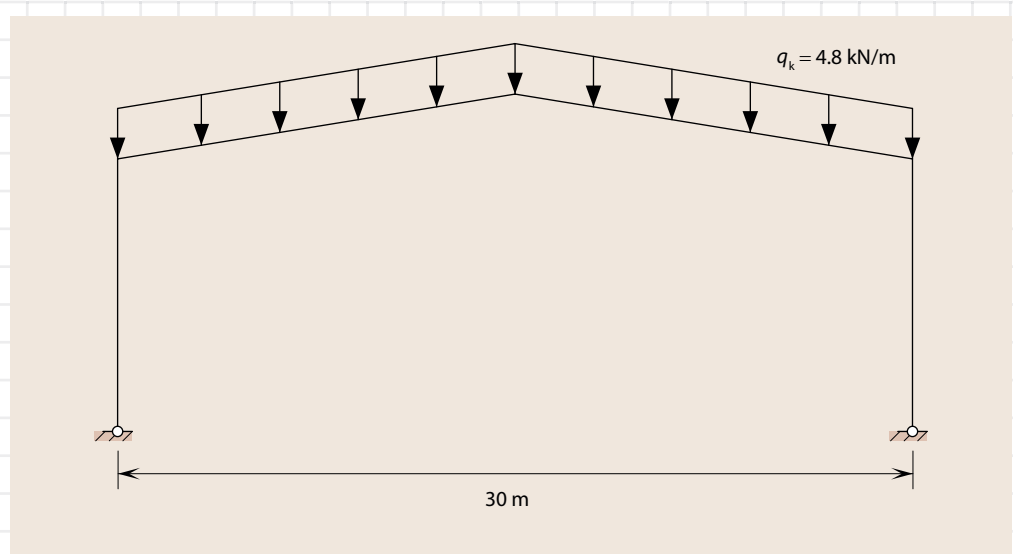
Figure D.3
Snow load on roof

2.3 Imposed load on roof

NA to
BS EN 1991-1-1
Table NA.7

The characteristic value for imposed loading on the roof (type H: not accessible) is determined from BS EN 1991-1-1 and the UK NA.

Figure D.4
Imposed load
on roof



$$q_k = 0.6 \text{ kN/m}^2$$

Therefore for an internal frame:

$$q_k = 0.6 \times 8.0 = 4.8 \text{ kN/m}$$

2.4 Wind actions

The peak velocity pressure q_p has been calculated for the chosen location, and internal and external pressure coefficients, all determined from BS EN 1991-1-4 and the UK National Annex.

Two load cases have been adopted as shown in Figure D.5 and Figure D.6 to determine (in Figure D.5) the most onerous uplift condition and (in Figure D.6) the most onerous wind loading to be used in combination with gravity loads. Figure D.5 and Figure D.6 show the characteristic values of loading on each frame due to wind pressures and represent the situation mid-way along the building.

Figure D.5
Wind loading to
be used for the
most onerous uplift
combination

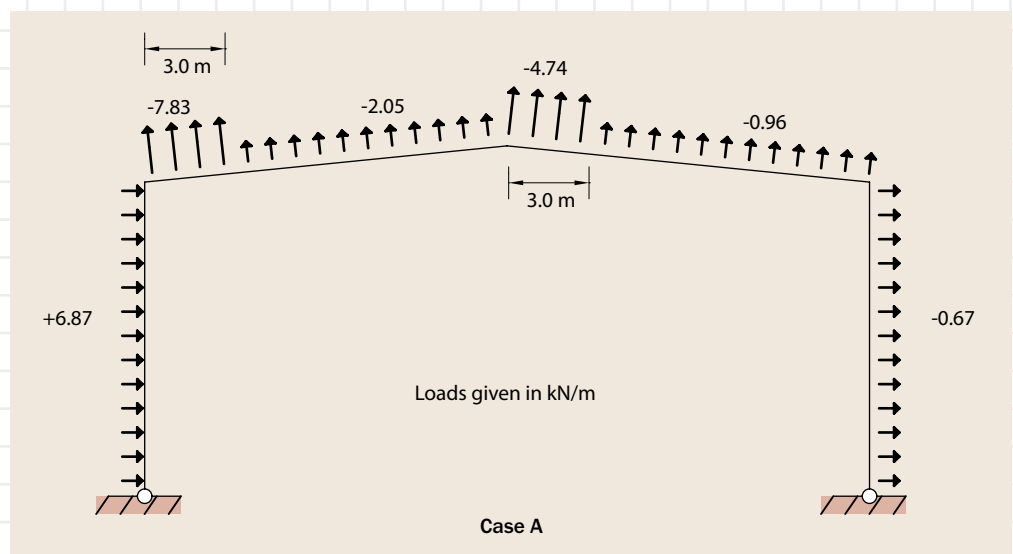
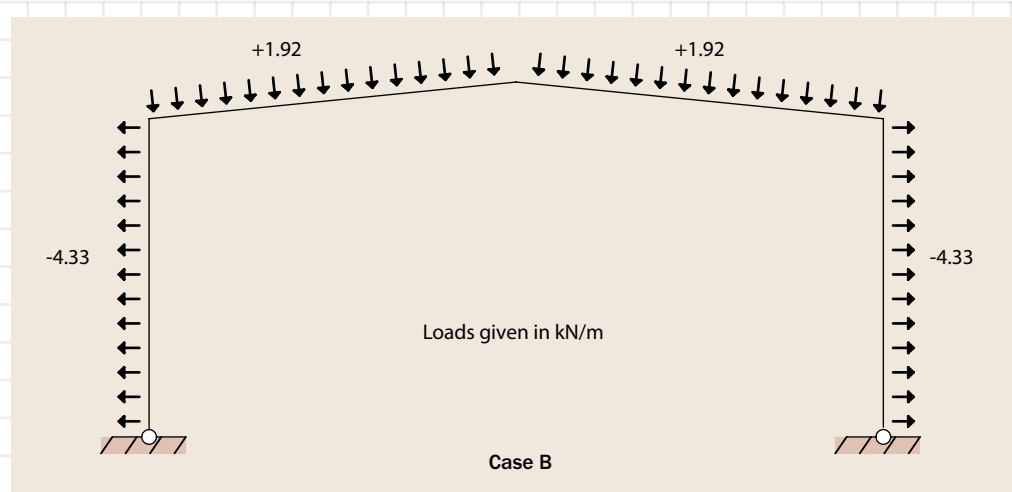


Figure D.6
Wind loading
to be used in
combination with
permanent actions
and snow loading



3 Combinations of actions

BS EN 1991-1
3.3.2(1)

According to BS EN 1991-1, imposed actions on a roof are not considered with either the wind actions or the snow load.

The critical combinations of actions for preliminary sizing are likely to be from:

1. Permanent actions with imposed roof loads, or
2. Permanent actions with snow loads, or
3. Permanent actions with snow loads and wind actions.

As the imposed roof loads are greater than the snow loads, combination 2 can be ignored. Combination 3 must be considered with both the wind actions and the snow load as the leading variable action, in turn.

For preliminary sizing, it is assumed that the combination considering wind uplift will not be critical.

4 Preliminary sizing

The section sizes given in Appendix A of this publication provide appropriate preliminary steelwork sizes for columns and rafters, in S355 steel. The tabulated sizes should only be used for preliminary sizing, prior to analysis.

After selecting preliminary sizes, an initial analysis is recommended. The design resistance of the selected members can then be reviewed against the design effects.

Once preliminary sections have been chosen, it is wise to check the sensitivity of the frame to second order effects (see Section 6 of these calculations), as this could increase the design moments.

After undertaking a preliminary analysis and some initial checks of member resistance, the following sections were selected:

Rafter: 533 × 210 × 101 UKB, Grade S355
Column: 686 × 254 × 125 UKB, Grade S355

4.1 Section properties

Column section: 686 × 254 × 125 UKB, Grade S355

Section properties:

h	= 677.9 mm	A	= 15900 mm ²		
b	= 253.0 mm	$W_{el,y}$	= 3480 × 10 ³ mm ³		
t_w	= 11.7 mm	$W_{pl,y}$	= 3990 × 10 ³ mm ³	i_y	= 272 mm
t_f	= 16.2 mm	I_y	= 118000 × 10 ⁴ mm ⁴	i_z	= 52.4 mm
r	= 15.2 mm	I_z	= 4380 × 10 ⁴ mm ⁴		
h_w	= 645.5 mm	I_T	= 116 × 10 ⁴ mm ⁴		
d	= 615.1 mm	I_w	= 4800 × 10 ⁹ mm ⁶		

BS EN 10025-2 Because $t_f > 16$ mm, $f_y = 345$ N/mm²

Rafter section: 533 × 210 × 101 UKB, Grade S355

Section properties:

h	= 536.7 mm	A	= 12900 mm ²		
b	= 210 mm	$W_{el,y}$	= 2290 mm ³		
t_w	= 10.8 mm	$W_{pl,y}$	= 2610 mm ³	i_y	= 219 mm
t_f	= 17.4 mm	I_y	= 61500 mm ⁴	i_z	= 45.7 mm
r	= 12.7 mm	I_z	= 2690 mm ⁴		
h_w	= 501.9 mm	I_T	= 101 mm ⁴		
d	= 476.5 mm	I_w	= 1810 mm ⁶		

BS EN 10025-2 Because $t_f > 16$ mm, $f_y = 345$ N/mm²

5 Initial analysis

The results from an initial analysis using the selected sections are used to determine if the frame is sensitive to second-order effects, and if an allowance for frame imperfections needs to be made. The combinations of actions considered, with partial factors and combination factors, are shown in Table D.1.

	COMBINATION	PERMANENT ACTIONS	VARIABLE ACTIONS				
			PARTIAL FACTOR, γ_G	LEADING ACTION		ACCOMPANYING ACTION	
				ACTION	PARTIAL FACTOR, γ_Q	ACTION	PARTIAL FACTOR, γ_Q
1	Permanent & imposed	1.35	Imposed	1.5			
2	Permanent, snow & wind	1.35	Snow	1.5	Wind	1.5	0.5
3	Permanent, wind & snow	1.35	Wind	1.5	Snow	1.5	0.5
4	Permanent & wind	1.0	Wind	1.5			

Table D.1
Partial factors
on actions

Table D.2
Initial analysis
results for
combinations 1 - 4

COMBINATION	BASE REACTIONS (kN)				AXIAL FORCE IN RAFTER (kN)
	$F_{V,L}$	$F_{V,R}$	$F_{H,L}$	$F_{H,R}$	N_{Ed}
1	241	241	74	-74	97
2	227	227	91	-91	70
3	213	213	108	-108	43
4	-5	91	-93	52	46

The results of the initial analysis are shown in Table D.2. The base reactions at the left and right column in Combination 4 (subscript L and R) differ due to the asymmetric wind actions included in that combination.

6 Sensitivity to second order effects

BS EN 1993-1-1
§5.2.1

In order to evaluate the sensitivity of the frame to effects of deformed geometry, the factor α_{cr} has to be calculated. If α_{cr} is greater than 10, second-order effects are small enough to be ignored. Hand calculation methods to determine α_{cr} are generally more conservative than software solutions.

BS EN 1993-1-1
§5.2.1(4)

BS EN 1993-1-1 offers a simple approximation to calculate α_{cr} , but this can only be used when the roof slope is less than 26° and the axial force in the rafter is not significant.

If the axial compression in the rafter is significant, Section 6.6 of this publication describes an alternative approach to estimate α_{cr} .

6.1 Axial compression in the rafter

BS EN 1993-1-1
§5.2.1(4)
Note 2B

The axial compression is significant if $\bar{\lambda} \geq 0.3 \sqrt{\frac{Af_y}{N_{Ed}}}$, which may be rearranged to show that the compression is significant if $N_{Ed} \geq 0.09N_{cr}$.

N_{Ed} is the design value of the compression force in the rafter at ULS.

Appendix B.1

$$N_{cr} = \frac{\pi^2 EI_y}{L_{cr}^2}$$

L_{cr} is taken as the developed length of the rafter pair between columns.

$$\text{Hence, } L_{cr} = \frac{30}{\cos 6^\circ} = 30.165 \text{ m}$$

$$N_{cr} = \frac{\pi^2 EI_y}{L_{cr}^2} = \frac{\pi^2 \times 210000 \times 61500 \times 10^4}{(30.165 \times 10^3)^2} \times 10^{-3} = 1401 \text{ kN}$$

$$0.09N_{cr} = 0.09 \times 1401 = 126 \text{ kN}$$

$$N_{Ed} = 97 \text{ kN}, < 126 \text{ kN}$$

Therefore the axial compression in the rafter is not significant and Expression 5.2 of BS EN 1993-1-1 may be used to calculate α_{cr} .

6.2 Calculation of α_{cr}

The expression for α_{cr} offered in Section 5.2 of BS EN 1993-1-1 is not immediately applicable to portal frames, as the vertical loads dominate the calculated horizontal deflections at the eaves. In Section 6.5 of this publication it is recommended that for portal frames, α_{cr} should be calculated from the expression:

Section 6.5 of this document

$$\alpha_{cr} = \frac{h}{200 \times \delta_{NHF}}$$

where:

h is the height to the eaves

δ_{NHF} is the horizontal deflection at the eaves, under a notional horizontal force applied at each eaves node, equal to 1/200 of the factored vertical base reaction.

Combination 1

In combination 1, the notional horizontal force at the top of each column is:

$$H_{NHF} = \frac{1}{200} F_V = \frac{1}{200} \times 241 = 1.205 \text{ kN}$$

Following the guidance given in Section 6.4.2 for nominally pinned bases, a base stiffness equal to 10% of the column stiffness has been assumed in the analysis.

In this example, the nominally pinned base has been modelled using an additional member, with an inertia set as 10% of the column member, and a length equal to 75% of the column length. The end of the dummy member is pinned.

The horizontal deflection of the top of the column under this force, allowing for nominal base stiffness, is obtained from an elastic analysis as 7.2 mm.

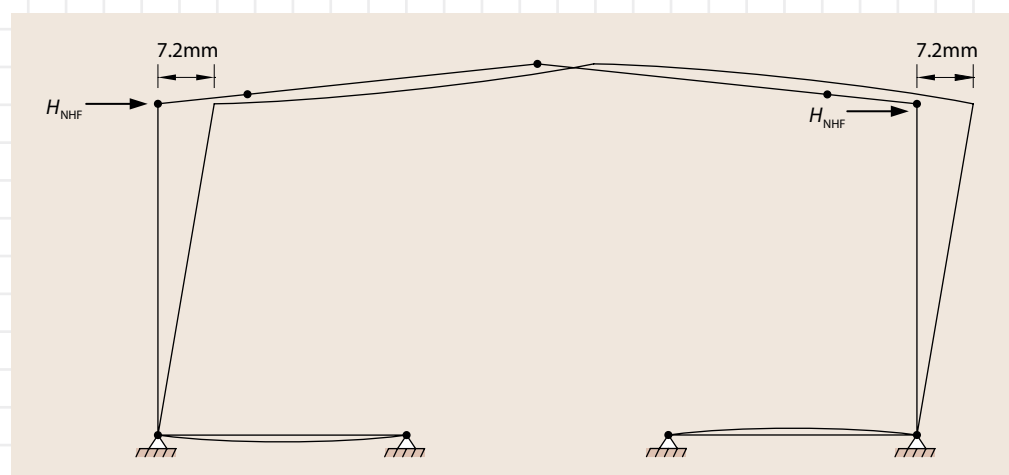


Figure D.7
Deflected shape
of the frame with
dummy members
modelling the nominal
stiffness of bases

$$\alpha_{cr} = \frac{h}{200 \times \delta_{NHF}}$$

$$= \frac{13100}{200 \times 7.2} = 9.1$$

Because $\alpha_{cr} = 9.1 < 10$, second order effects cannot be ignored.

Second order effects may be allowed for by amplifying the results of the first order analysis by the factor:

$$\frac{1}{1 - \frac{1}{\alpha_{cr}}} = \frac{1}{1 - \frac{1}{9.1}} = 1.12$$

Combination 2

For Combination 2, $N_{Ed} = 70$ kN, < 126 kN, therefore axial compression in the rafter is not significant.

The NHF at each column are:

$$H_{NHF} = \frac{1}{200} F_v = \frac{1}{200} \times 227 = 1.14 \text{ kN}$$

From the elastic analysis, including the modelling of the nominally pinned bases,
 $\delta_{NHF} = 6.7$ mm

$$\alpha_{cr} = \frac{13100}{200 \times 6.7} = 9.8$$

Because $\alpha_{cr} < 10$, second order effects cannot be ignored.

First order analysis results need to be amplified by the factor:

$$\frac{1}{1 - \frac{1}{\alpha_{cr}}} = \frac{1}{1 - \frac{1}{9.8}} = 1.11$$

Combination 3

For Combination 3, $N_{Ed} = 43$ kN, < 126 kN, therefore the axial compression in the rafter is not significant.

$\alpha_{cr} = 10.4$, > 10 , so second order effects may be ignored.

Combination 4

For Combination 4, $N_{Ed} = 46 \text{ kN}$, $< 126 \text{ kN}$, therefore the axial compression in the rafter is not significant.

$\alpha_{cr} = 46.8$, > 10 , so second order effects may be ignored.

6.3 α_{cr} calculated by bespoke software

As a comparison with the hand calculations, α_{cr} was checked using bespoke software, with the results presented in Table D.3.

Table D.3
Comparison of α_{cr}
from software and
hand calculation

CALCULATION METHOD	COMBINATION 1	COMBINATION 2	COMBINATION 3	COMBINATION 4
Software solution	$\alpha_{cr} = 9.6$	$\alpha_{cr} = 11.2$	$\alpha_{cr} = 13.6$	$\alpha_{cr} = 26.6$
Hand calculation	$\alpha_{cr} = 9.1$	$\alpha_{cr} = 9.8$	$\alpha_{cr} = 10.4$	$\alpha_{cr} = 46.8$

The above comparison demonstrates that the hand calculations are reasonable for initial design for the more critical combinations.

7 Frame imperfections

BS EN 1993-1-1
§5.3.2

The global initial sway imperfection may be determined from:

$$\begin{aligned} \phi &= \phi_0 \alpha_h \alpha_m \\ \phi_0 &= 1/200 \\ h &= 13.1 \text{ m (height to eaves)} \\ m &= 2 \text{ (number of columns)} \\ \alpha_h &= \frac{2}{\sqrt{h}} = \frac{2}{\sqrt{13.1}} = 0.55 \text{ but } \alpha_h > 2/3, \text{ hence } \alpha_h = 0.67 \\ \alpha_m &= \sqrt{0.5(1 + \frac{1}{m})} = \sqrt{0.5(1 + \frac{1}{2})} = 0.87 \\ \phi &= \frac{1}{200} \times 0.67 \times 0.87 = 2.9 \times 10^{-3} \end{aligned}$$

Initial sway imperfections may be conveniently included in the analysis by applying equivalent horizontal forces (EHF).

BS EN 1993-1-1
§5.3.2(4)

Sway imperfections may be disregarded where $H_{Ed} \geq 0.15V_{Ed}$.

Table D.4 shows the total reactions for the structure to determine H_{Ed} and V_{Ed} for Combination 1.

$$H_{Ed} = 0 \text{ kN and } 0.15V_{Ed} = 72.3 \text{ kN}$$

Table D.4
Vertical and
horizontal reactions
– Combination 1

	LEFT COLUMN (kN)		RIGHT COLUMN (kN)		TOTAL REACTION (kN)		0.15V _{Ed} (kN)
	F _H	F _V	F _H	F _V	H _{Ed}	V _{Ed}	
Reactions	74	241	-74	241	0	482	72.3

Because $H_{Ed} < 0.15H_{Ed}$ the initial sway imperfections need to be taken into account in Combination 1.

The equivalent horizontal forces are taken as a proportion of the design base vertical reactions:

$$H_{EHF} = \phi V_{Ed} = 2.9 \times 10^{-3} \times 241 = 0.7 \text{ kN}$$

This force is applied horizontally at the top of each column, in the same direction, in combination with the permanent and variable actions.

It can be demonstrated that equivalent horizontal forces must also be included in Combinations 2 and 3. In Combination 4, initial sway imperfections may be disregarded.

8 Analysis results

For the ULS analysis, the bases are modelled as truly pinned. Otherwise, the bases and foundations would need to be designed for the resulting moment.

8.1 Final analysis results

For the load combinations in which α_{cr} is less than 10, second-order effects need to be allowed for. In this example the amplified moment method has been used; the amplification factors are shown in Table D.5.

Table D.5
Amplification
factors

	COMBINATION 1	COMBINATION 2	COMBINATION 3	COMBINATION 4
Calculated α_{cr}	$\alpha_{cr} = 9.1$	$\alpha_{cr} = 9.8$	$\alpha_{cr} = 10.4$	$\alpha_{cr} = 46.8$
Amplification factor	1.12	1.11	does not apply	does not apply

The following diagrams show the results of the frame analysis for Combinations 1 and 2 including amplification and equivalent horizontal forces.

Figure D.10 shows the bending moment diagram for Combination 3, including the EHF.

Figure D.11 shows the bending moment diagram for Combination 4. Note that there is no need to include the EHF in Combination 4, and second order effects are small enough to be ignored.

Figure D.8
Design bending moment diagram for Combination 1 including EHF

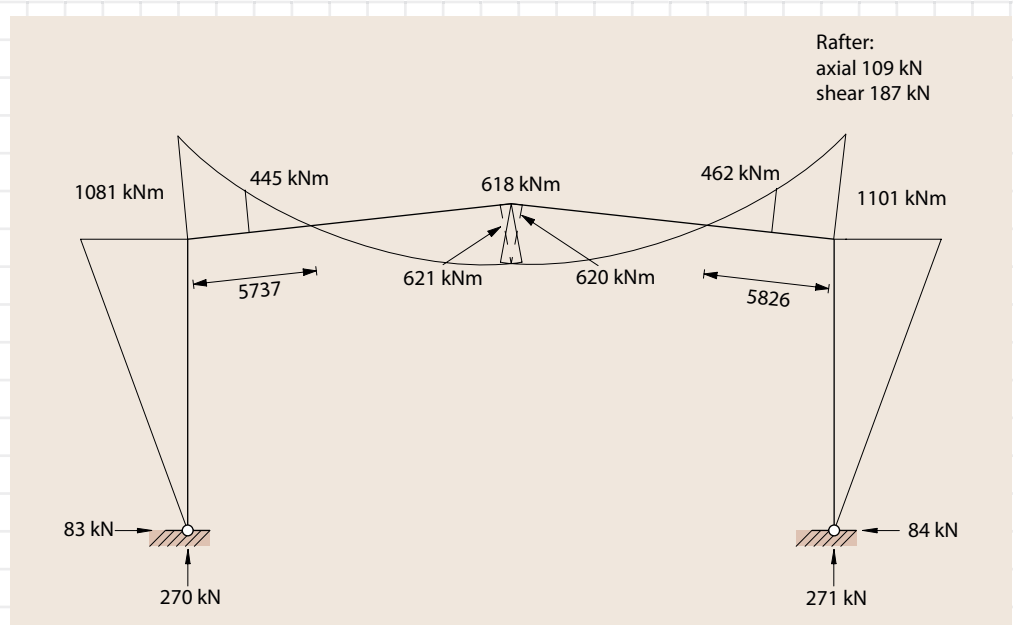


Figure D.9
Design bending moment diagram for Combination 2 including EHF

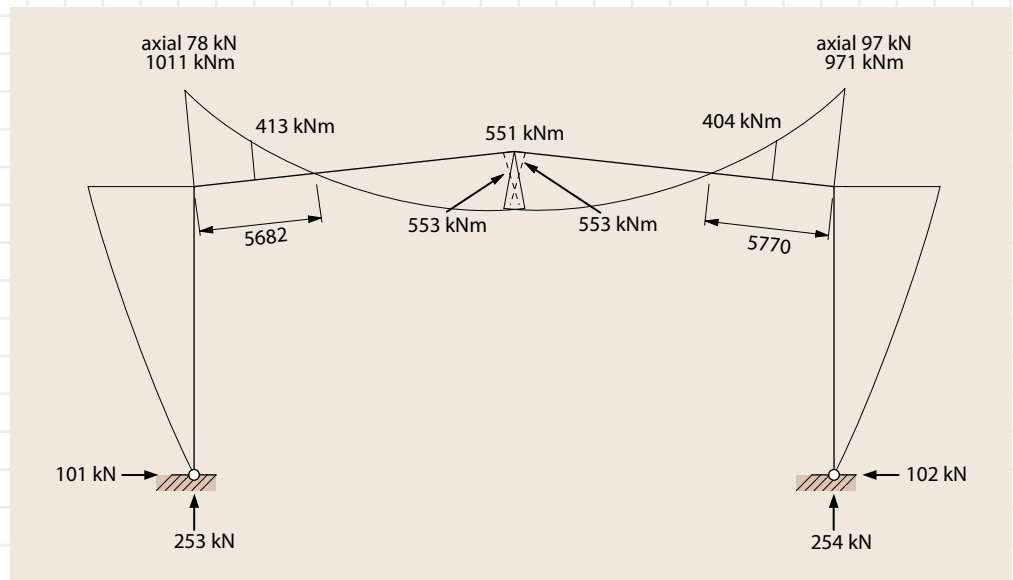
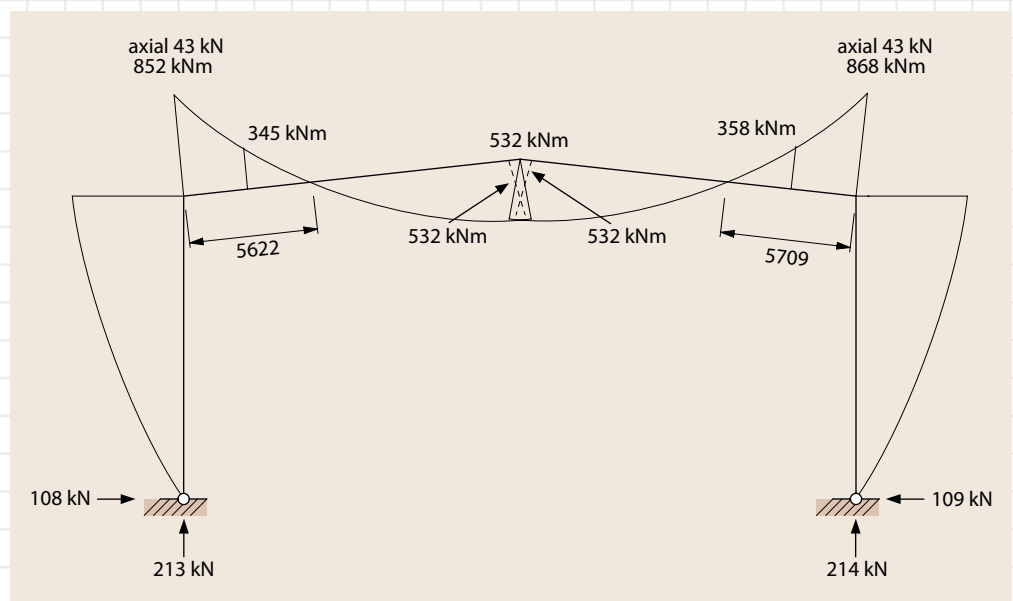


Figure D.10
Design bending moment diagram for Combination 3 including EHF



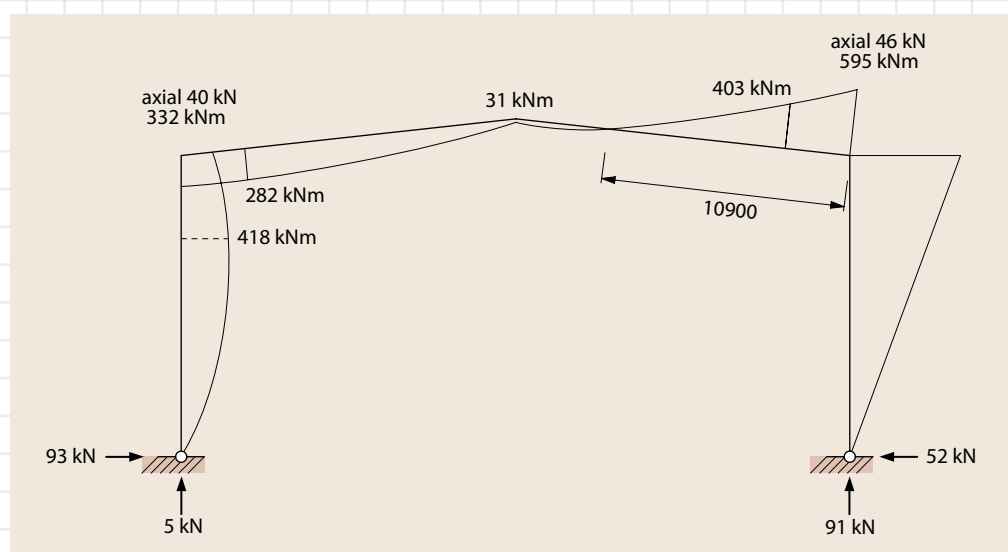


Figure D.11
Design bending
moment diagram
for Combination 4

9 Cross section verification

9.1 Section classification

Before determining the cross sectional resistance, the sections must be classified in accordance with Section 5.5 of BS EN 1993-1-1. The following calculations demonstrate section classification for Combination 1 under the forces and moments given in Figure D.8.

UK NA to
BS EN 1993-1-1

$\gamma_{M0} = 1.0$, and is taken from the UK NA.

9.1.1 Column classification

Web classification

BS EN 1993-1-1
Table 5.2

$$\frac{c}{t_w} = \frac{615.1}{11.7} = 52.6$$

$$\alpha = \frac{1}{2} \left(1 + \frac{N_{Ed}}{f_y t_w c} \right) = \frac{1}{2} \left(1 + \frac{271 \times 10^3}{345 \times 11.7 \times 615.1} \right) = 0.55 > 0.50$$

$$\varepsilon = \sqrt{\frac{235}{345}} = 0.83$$

$$\text{The limit for Class 1 is: } \frac{396\varepsilon}{13\alpha - 1} = \frac{396 \times 0.83}{13 \times 0.55 - 1} = 53.4$$

$$52.6 < 53.4$$

Therefore the web is Class 1.

BS EN 1993-1-1
Table 5.2

Flange classification

$$c = 0.5 \times (253 - 11.7 - 2 \times 15.2) = 105.5 \text{ mm}$$

$$\frac{c}{t_f} = \frac{105.5}{16.2} = 6.51$$

The limit for Class 1 is: $9\varepsilon = 9 \times 0.83 = 7.47$

$$6.51 < 7.47$$

Therefore the flange is Class 1.

Section classification

Because both the web and flanges are Class 1, the column section is Class 1.

9.1.2 Rafter classification

Web classification

BS EN 1993-1-1
Table 5.2

$$\frac{c}{t_w} = \frac{476.5}{10.8} = 44.1$$

$$\alpha = \frac{1}{2} \left(1 + \frac{N_{Ed}}{f_y t_w c} \right) = \frac{1}{2} \left(1 + \frac{109 \times 10^3}{345 \times 10.8 \times 476.5} \right) = 0.53 > 0.50$$

The limit for Class 1 is: $\frac{396\varepsilon}{13\alpha - 1} = \frac{396 \times 0.83}{13 \times 0.53 - 1} = 55.8$

$$44.1 < 55.8$$

Therefore the web is Class 1.

Flange classification

BS EN 1993-1-1
Table 5.2

$$c = 0.5 \times (210 - 10.8 - 2 \times 12.7) = 86.9 \text{ mm}$$

$$\frac{c}{t_f} = \frac{86.9}{17.4} = 4.99$$

The limit for Class 1 is: $9\varepsilon = 9 \times 0.83 = 7.47$

$$4.99 < 7.47$$

Therefore the flange is Class 1.

Section classification

Because both the web and flanges are Class 1, the rafter section is Class 1.

9.2 Resistance of the cross sections**9.2.1 Column****Shear resistance**

BS EN 1993-1-1
§6.2.6

Shear area: $A_v = A - 2bt_f + (t_w + 2r)t_f$ but not less than $\eta h_w t_w$

$$A_v = 15900 - 2 \times 253.0 \times 16.2 + (11.7 + 2 \times 15.2) \times 16.2 = 8385 \text{ mm}^2$$

BS EN 1993-1-1
§6.2.6(3)

Conservatively $\eta = 1.0$. Therefore:

$$A_v \not\leq \eta h_w t_w = 1.0 \times 645.5 \times 11.7 = 7552 \text{ mm}^2$$

$$\therefore A_v = 8385 \text{ mm}^2$$

$$V_{pl,Rd} = \frac{A_v (f_y / \sqrt{3})}{\gamma_{M0}} = \frac{8385 \times (345 / \sqrt{3})}{1.0} \times 10^{-3} = 1670 \text{ kN}$$

From Figure D.8, $V_{Ed} = F_H = 84 \text{ kN}$

84 kN < 1670 kN, OK

BS EN 1993-1-1
§6.2.8

Bending and shear interaction

When shear force and bending moment act simultaneously on a cross section, the effect of the shear force can be ignored if it is smaller than 50% of the plastic shear resistance.

$$0.5V_{pl,Rd} = 0.5 \times 1670 \text{ kN} = 835 \text{ kN}$$

84 kN < 835 kN, therefore the effect of the shear force on the moment resistance may be neglected.

Compression resistance

BS EN 1993-1-1
§6.2.4

$$N_{c,Rd} = \frac{A f_y}{\gamma_{M0}} = \frac{15900 \times 345}{1.0} \times 10^{-3} = 5486 \text{ kN}$$

From Figure D.8, $N_{Ed} = F_v = 271 \text{ kN}$

271 kN < 5486 kN, OK

Combined bending and axial force

BS EN 1993-1-1
§6.2.9

When axial force and bending moment act simultaneously on a cross section, the effect of the axial force can be ignored provided the following two conditions are satisfied:

$$N_{Ed} \leq 0.25N_{pl,Rd} \quad \text{and} \quad N_{Ed} \leq \frac{0.5h_w t_w f_y}{\gamma_{M0}}$$

$$0.25N_{pl,Rd} = 0.25 \times 5486 = 1371 \text{ kN}$$

$$\frac{0.5h_w t_w f_y}{\gamma_{M0}} = \frac{0.5 \times 645.5 \times 11.7 \times 345}{1.0} \times 10^{-3} = 1303 \text{ kN}$$

$$271 \text{ kN} < 1371 \text{ kN} \quad \text{and} \quad 271 \text{ kN} < 1303 \text{ kN}$$

Therefore the effect of the axial force on the moment resistance may be neglected.

Bending resistance

BS EN 1993-1-1
§6.2.5

$$M_{pl,y,Rd} = \frac{W_{pl,y} f_y}{\gamma_{M0}} = \frac{3990 \times 10^3 \times 345}{1.0} \times 10^{-6} = 1377 \text{ kNm}$$

From Figure D.8, taking the haunch as 800 mm deep, (from the centreline intersections of rafter and column) the bending moment at the underside of the

$$\text{haunch, } M_{y,Ed} = 1101 \times \frac{13.1 - 0.8}{13.1} = 1034 \text{ kNm.}$$

$$M_{y,Ed} = 1034 \text{ kNm}, < 1377 \text{ kNm, OK}$$

9.2.2 Rafter

Shear resistance

BS EN 1993-1-1
§6.2.6

Shear area: $A_v = A - 2bt_f + (t_w + 2r)t_f$ but not less than $\eta h_w t_w$

$$A_v = 12900 - 2 \times 210 \times 17.4 + (10.8 + 2 \times 12.7) \times 17.4 = 6222 \text{ mm}^2$$

BS EN 1993-1-1
§6.2.6(3)

Conservatively $\eta = 1.0$. Therefore:

$$A_v \not\leq \eta h_w t_w = 1.0 \times 501.9 \times 10.8 = 5421 \text{ mm}^2$$

$$\therefore A_v = 6222 \text{ mm}^2$$

$$V_{pl,Rd} = \frac{A_v (f_y / \sqrt{3})}{\gamma_{M0}} = \frac{6222 \times (345 / \sqrt{3})}{1.0} \times 10^{-3} = 1239 \text{ kN}$$

From Figure D.8, $V_{Ed} = 187$ kN

187 kN < 1239 kN, OK

Bending and shear interaction

BS EN 1993-1-1
§6.2.8

$$0.5V_{pl,Rd} = 0.5 \times 1239 = 620 \text{ kN}$$

187 kN < 620 kN, therefore the effect of the shear force on the moment resistance may be neglected.

Compression resistance

BS EN 1993-1-1
§6.2.4

$$N_{c,Rd} = \frac{Af_y}{\gamma_{M0}} = \frac{12900 \times 345}{1.0} \times 10^{-3} = 4451 \text{ kN}$$

From Figure D.8, $N_{Ed} = 109$ kN.

109 kN < 4451 kN, OK

Combined bending and axial force

BS EN 1993-1-1
§6.2.9

$$0.25N_{pl,Rd} = 0.25 \times 4451 = 1113 \text{ kN}$$

$$\frac{0.5h_w t_w f_y}{\gamma_{M0}} = \frac{0.5 \times 501.9 \times 10.8 \times 345}{1.0} \times 10^{-3} = 935 \text{ kN}$$

109 kN < 1113 kN and 109 kN < 935 kN

Therefore the effect of the axial force on the moment resistance may be neglected.

Bending resistance

BS EN 1993-1-1
§6.2.5

$$M_{pl,y,Rd} = \frac{W_{pl,y} f_y}{\gamma_{M0}} = \frac{2610 \times 10^3 \times 345}{1.0} \times 10^{-6} = 900 \text{ kNm}$$

From Figure D.8, the maximum bending moment in the rafter is 620 kNm, adjacent to the apex.

$M_{y,Ed} = 620$ kNm, < 900 kNm, OK

10 Buckling verification

The rafters and columns must be verified for buckling between restraints.

As noted in Section 7.2, because $M_{z,Ed} = 0$, as there is no minor axis bending, expressions 6.61 and 6.62 reduce to:

$$\frac{N_{Ed}}{N_{b,y,Rd}} + k_{yy} \frac{M_{y,Ed}}{M_{b,Rd}} \leq 1.0$$

and

$$\frac{N_{Ed}}{N_{b,z,Rd}} + k_{zy} \frac{M_{y,Ed}}{M_{b,Rd}} \leq 1.0$$

where:

$N_{b,y,Rd}$ is the flexural buckling resistance in the major axis

$N_{b,z,Rd}$ is the flexural buckling resistance in the minor axis

$M_{b,Rd}$ is the lateral torsional buckling resistance.

The initial setting out of the purlins and side rails is shown in Figure D.12. At some purlin and side rail positions, stays to the inner flange will be used to provide a torsional restraint at that location.

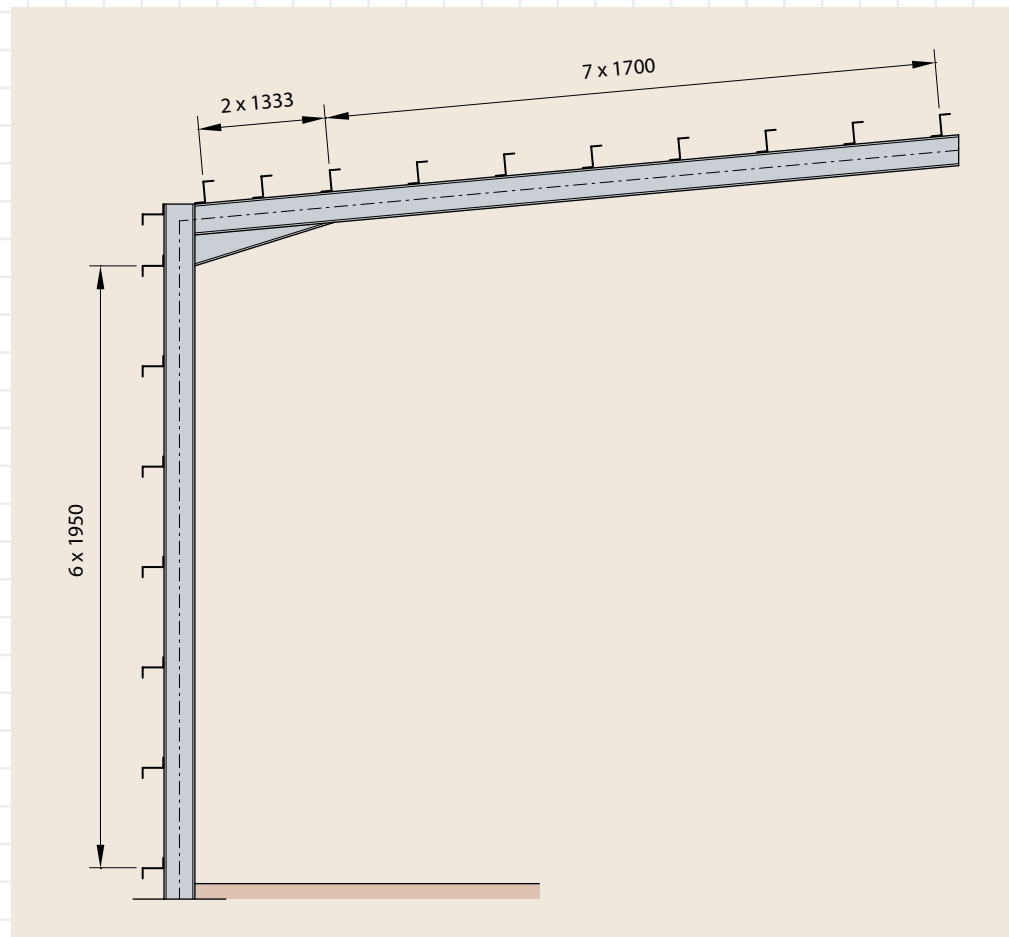


Figure D.12
Initial setting
out of the purlins
and rails

BS EN 1993-1-1 allows the benefit of intermediate restraints to the tension flange to be utilised to increase the buckling resistance, provided the spacing of such tension flange restraints is within a limiting distance, to ensure they are effective.

10.1 Column verification

Firstly, the column will be assessed using Expression 6.62, which considers minor axis flexural buckling and lateral torsional buckling between restraints. The possibility of

utilising the benefits of tension flange restraint will be investigated. Once the column has been verified according to Expression 6.62, column stability will be assessed using Expression 6.61, which considers major axis flexural buckling.

10.1.1 Spacing of restraints to the tension flange

Section 7.2.3

Intermediate restraints to the tension flange are provided by side rails spaced at 1950 mm, as shown in Figure D.12. As discussed in Section 7.2.3, it may be assumed that the restraints to the tension flange are effective in increasing the resistance to lateral-torsional buckling if their spacing does not exceed L_m , where L_m is given by:

BS EN 1993-1-1
BB.3.1.1

$$L_m = \frac{38i_z}{\sqrt{\frac{1}{57.4} \left(\frac{N_{Ed}}{A} \right) + \frac{1}{756C_1^2} \frac{W_{pl,y}^2}{AI_T} \left(\frac{f_y}{235} \right)^2}}$$

C_1 is a factor that accounts for the shape of the bending moment diagram.

C_1 values for different shapes of bending moment diagrams can be found in Appendix B of this publication.

For a linear bending moment diagram, C_1 depends on the ratio of the minimum and the maximum bending moments in the segment being considered. Values of the bending moment at restraint positions are shown in Figure D.13.

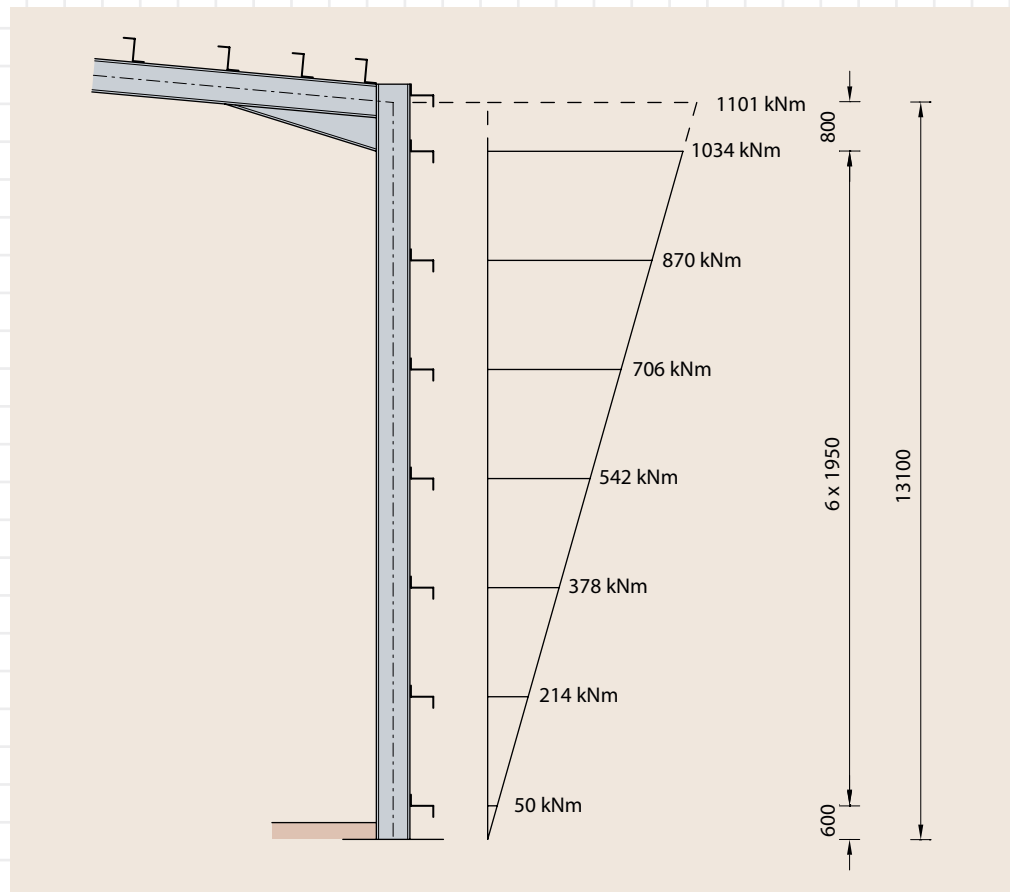


Figure D.13
Side rail layout
with corresponding
bending moment
values

The ratios of bending moments for the column segments, from the top of the column, are as follows:

$$\psi = \frac{870}{1034} = 0.84 \quad \therefore C_1 = 1.11$$

$$\psi = \frac{706}{870} = 0.81 \quad \therefore C_1 = 1.13$$

$$\psi = \frac{542}{706} = 0.77 \quad \therefore C_1 = 1.16$$

$$\psi = \frac{378}{542} = 0.70 \quad \therefore C_1 = 1.21$$

$$\psi = \frac{214}{378} = 0.57 \quad \therefore C_1 = 1.31$$

$$\psi = \frac{50}{214} = 0.23 \quad \therefore C_1 = 1.57$$

$$\psi = \frac{0}{50} = 0.00 \quad \therefore C_1 = 1.77$$

The most onerous value of C_1 is 1.11; therefore this case will be assessed.

Substituting the necessary section properties and design strength:

$$L_m = \frac{38 \times 52.4}{\sqrt{\frac{1}{57.4} \left(\frac{271 \times 10^3}{15900} \right)^2 + \frac{1}{756 \times 1.11^2} \times \frac{(3990 \times 10^3)^2}{15900 \times 116 \times 10^4} \left(\frac{345}{235} \right)^2}}$$

$$L_m = 1315 \text{ mm}$$

The side rail spacing is 1950 mm, which exceeds this limiting value.

Therefore the restraints to the tension flange are not close enough to be used to enhance the resistance to lateral-torsional buckling.

10.1.2 Verification with no intermediate restraints

Firstly, an attempt is made to verify the column between the restraint at the underside of the haunch and the base, assuming no intermediate restraints. If the flexural buckling, lateral torsional buckling and interaction checks are satisfied for this length, no intermediate restraints are required. Otherwise, intermediate torsional restraints need to be introduced to the column or the column size increased. For a column of this height, it is very likely that intermediate restraints will be needed.

The flexural and lateral torsional buckling checks are first conducted independently before proceeding to verify the interaction between the two.

Flexural buckling resistance about the minor axis, $N_{b,z,Rd}$

$$\frac{h}{b} = \frac{677.9}{253.0} = 2.68$$

$$t_f = 16.2 \text{ mm} \therefore f_y = 345 \text{ N/mm}^2$$

BS EN 1993-1-1 §6.3.1.3

$$\lambda_1 = \pi \sqrt{\frac{E}{f_y}} = \pi \sqrt{\frac{210000}{345}} = 77.5$$

$$\bar{\lambda}_z = \frac{L_{cr}}{i_z} \frac{1}{\lambda_1} = \frac{12300}{52.4} \times \frac{1}{77.5} = 3.029$$

BS EN 1993-1-1 Table 6.2, Table 6.1

For buckling about z-z axis, use curve **b** for hot rolled I sections with $\alpha_z = 0.34$.

BS EN 1993-1-1 §6.3.1.2

$$\phi_z = 0.5 [1 + \alpha_z (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2]$$

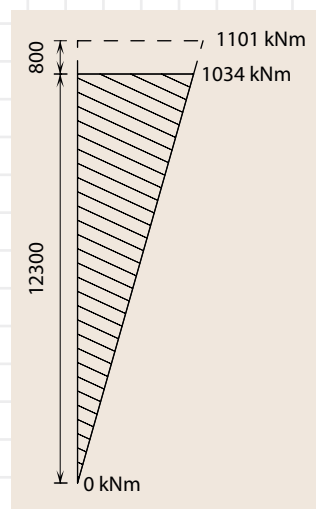
$$= 0.5 \times [1 + 0.34 \times (3.029 - 0.2) + 3.029^2] = 5.567$$

$$\chi_z = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}_z^2}} = \frac{1}{5.567 + \sqrt{5.567^2 - 3.029^2}} = 0.098$$

$$N_{b,z,Rd} = \frac{\chi_z A f_y}{\gamma_{M1}} = \frac{0.098 \times 15900 \times 345}{1.0} \times 10^{-3} = 536 \text{ kN}$$

$$N_{Ed} = 271 \text{ kN}, < 536 \text{ kN}, \text{ OK}$$

Lateral-torsional buckling resistance, $M_{b,Rd}$



C_1 is calculated based on the bending moment diagram over the column length between the base and the underside of the haunch.

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$$\psi = \frac{0}{1034} = 0 \quad \therefore C_1 = 1.77$$

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{L^2} \sqrt{\frac{I_w}{I_z} + \frac{L^2 GI_T}{\pi^2 EI_z}}$$

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$$= 1.77 \times \frac{\pi^2 \times 210000 \times 4380 \times 10^4}{12300^2} \times \sqrt{\frac{4800 \times 10^9}{4380 \times 10^4} + \frac{12300^2 \times 81000 \times 116 \times 10^4}{\pi^2 \times 210000 \times 4380 \times 10^4}}$$

$$M_{cr} = 548 \times 10^6 \text{ Nmm}$$

The non-dimensional slenderness, $\bar{\lambda}_{LT}$, is given by:

BS EN 1993-1-1
§6.3.2.2

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}} = \sqrt{\frac{3990 \times 10^3 \times 345}{548 \times 10^6}} = 1.585$$

BS EN 1993-1-1 §6.3.2.3 will be used to calculate the reduction factor, χ_{LT} .

NA to
BS EN 1993-1-1
NA.2.17

$$\phi_{LT} = 0.5 [1 + \alpha_{LT} (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \bar{\lambda}_{LT}^2]$$

The UK NA to BS EN 1993-1-1 recommends the following values for rolled sections:

NA to
BS EN 1993-1-1
NA.2.17

$$\bar{\lambda}_{LT,0} = 0.4$$

$$\beta = 0.75$$

$$\frac{h}{b} = 2.68, \text{ therefore curve } \mathbf{c} \text{ should be used for hot rolled I sections for which } \alpha_{LT} = 0.49$$

$$\phi_{LT} = 0.5 \times [1 + 0.49 \times (1.585 - 0.4) + 0.75 \times 1.585^2] = 1.732$$

BS EN 1993-1-1
§6.3.2.3

$$\chi_{LT} = \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \beta \bar{\lambda}_{LT}^2}}$$

$$\chi_{LT} = \frac{1}{1.732 + \sqrt{1.732^2 - 0.75 \times 1.585^2}} = 0.359$$

$$\text{But } \chi_{LT} < \frac{1}{\bar{\lambda}_{LT}^2} = \frac{1}{1.585^2} = 0.398$$

$$\therefore \chi_{LT} = 0.359$$

$$M_{b,Rd} = \frac{\chi_{LT} W_{pl,y} f_y}{\gamma_{M1}} = \frac{0.359 \times 3990 \times 10^3 \times 345}{1.0} \times 10^{-6} = 494 \text{ kNm}$$

$$M_{Ed} = 1034 \text{ kNm}, > 494 \text{ kNm}, \text{ Unsatisfactory}$$

It is clear that intermediate restraints will be required.

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In practice, rather than exhaustive hand calculations demonstrated above, it may be more convenient to obtain the member resistances from the Blue Book.

From the Blue Book, interpolating between values of $C_1 = 1.5$ and $C_1 = 2$ for a value of $C_1 = 1.77$ over a length of 12 m, $M_{b,Rd} = 509$ kNm.

10.1.3 Revised restraint arrangement

Intermediate restraints must be at a side rail position, since bracing from the side rail to the inner flange is used to provide the torsional restraint. The chosen arrangement of torsional restraints is shown in Figure D.14.

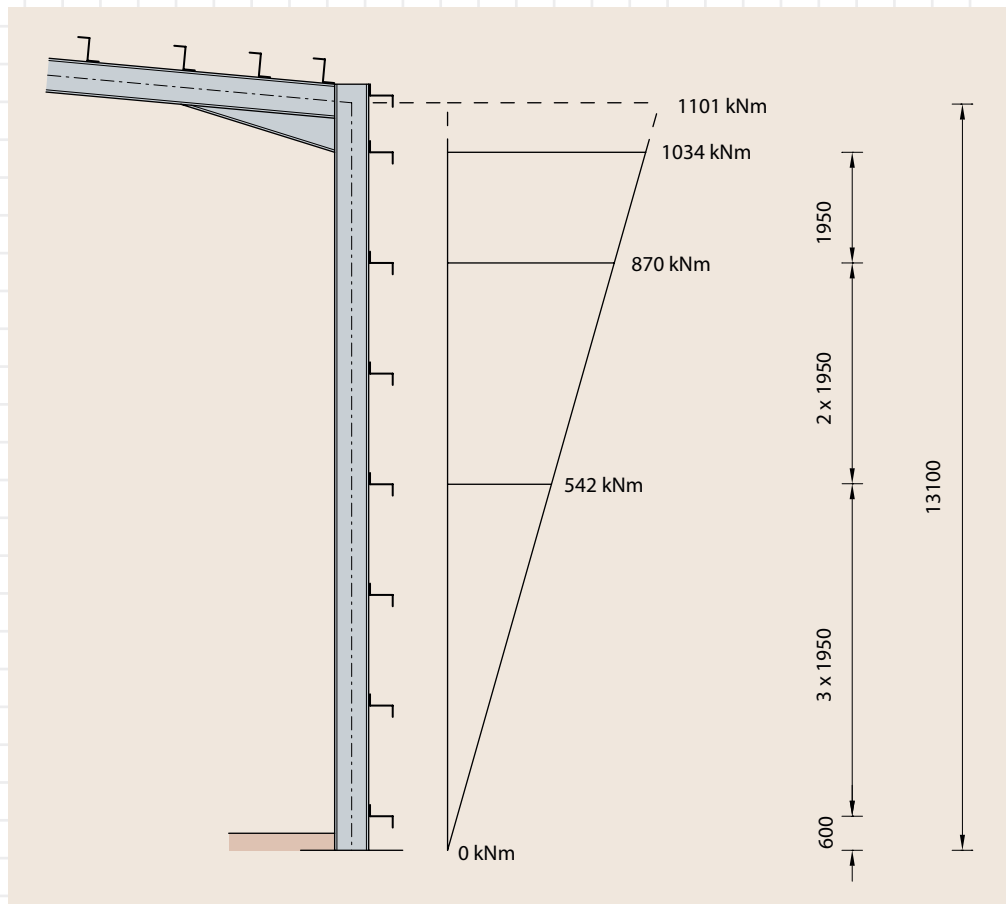


Figure D.14
Torsional restraints
with corresponding
bending moments

10.1.4 Verification of revised restraint arrangement - upper segment

Firstly the upper segment is checked. The flexural buckling and the lateral torsional buckling verifications are carried out independently before proceeding to verify the interaction between the two.

Flexural buckling resistance about the minor axis, $N_{b,z,Rd}$

$\frac{h}{b} = 2.68$ and $\lambda_1 = 77.5$, as before

$$\bar{\lambda}_z = \frac{L_{cr}}{i_z} \frac{1}{\lambda_1} = \frac{1950}{52.4} \times \frac{1}{77.5} = 0.480$$

BS EN 1993-1-1
Table 6.2, Table 6.1

For buckling about the z-z axis, curve **b** should be used for hot rolled I sections, with $\alpha_z = 0.34$.

BS EN 1993-1-1
§6.3.1.2

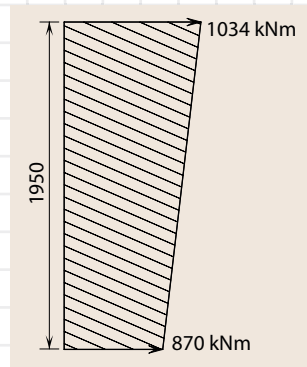
$$\begin{aligned}\phi_z &= 0.5[1 + \alpha_z(\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2] \\ &= 0.5 \times [1 + 0.34 \times (0.480 - 0.2) + 0.480^2] = 0.663\end{aligned}$$

$$\chi_z = \frac{1}{\phi_z + \sqrt{\phi_z^2 - \bar{\lambda}_z^2}} = \frac{1}{0.663 + \sqrt{0.663^2 - 0.480^2}} = 0.893$$

$$N_{b,z,Rd} = \frac{\chi_z A f_y}{\gamma_{M1}} = \frac{0.893 \times 15900 \times 345}{1.0} \times 10^{-3} = 4898 \text{ kN}$$

$$N_{Ed} = 271 \text{ kN}, < 4898 \text{ kN, OK}$$

Lateral-torsional buckling resistance, $M_{b,Rd}$



C_1 is calculated based on the bending moment diagram over the segment.

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$$\psi = \frac{870}{1034} = 0.84 \quad \therefore C_1 = 1.11$$

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{L^2} \sqrt{\frac{I_w}{I_z} + \frac{L^2 GI_T}{\pi^2 EI_z}}$$

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this publication

$$= 1.11 \times \frac{\pi^2 \times 210000 \times 4380 \times 10^4}{1950^2} \times \sqrt{\frac{4800 \times 10^9}{4380 \times 10^4} + \frac{1950^2 \times 81000 \times 116 \times 10^4}{\pi^2 \times 210000 \times 4380 \times 10^4}}$$

$$M_{cr} = 8908 \times 10^6 \text{ Nmm}$$

BS EN 1993-1-1
§6.3.2.2

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}} = \sqrt{\frac{3990 \times 10^3 \times 345}{8907 \times 10^6}} = 0.393$$

NA to
BS EN 1993-1-1
NA.2.17

As before, curve **c** should be used, with $\alpha_{LT} = 0.49$

$$\phi_{LT} = 0.5 [1 + \alpha_{LT}(\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \bar{\lambda}_{LT}^2]$$

$$\phi_{LT} = 0.5 \times [1 + 0.49 \times (0.393 - 0.4) + 0.75 \times 0.393^2] = 0.556$$

BS EN 1993-1-1
§6.3.2.3

$$\chi_{LT} = \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \beta \bar{\lambda}_{LT}^2}}$$

$$\chi_{LT} = \frac{1}{0.556 + \sqrt{0.556^2 - 0.75 \times 0.393^2}} = 1.004$$

χ_{LT} cannot be greater than 1.0, therefore $\chi_{LT} = 1.0$

There is no advantage in calculating $\chi_{LT,mod}$

$$M_{b,Rd} = \frac{\chi_{LT} W_{pl,y} f_y}{\gamma_{M1}} = \frac{1.0 \times 3990 \times 10^3 \times 345}{1.0} \times 10^{-6} = 1377 \text{ kNm}$$

$$M_{Ed} = 1034 \text{ kNm}, < 1377 \text{ kNm}, \text{ OK}$$

Interaction of axial force and bending moment according to Expression 6.62

BS EN 1993-1-1
§6.3.3(4)

As noted earlier, in this situation expression 6.62 reduces to:

$$\frac{N_{Ed}}{N_{b,z,Rd}} + k_{zy} \frac{M_{y,Ed}}{M_{b,Rd}} \leq 1.0$$

BS EN 1993-1-1
Table B.2

For $\bar{\lambda}_z \geq 0.4$, the interaction factor k_{zy} is given by:

$$k_{zy} = \max \left[\left(1 - \frac{0.1 \bar{\lambda}_z}{(C_{mLT} - 0.25)} \frac{N_{Ed}}{N_{b,Rd,z}} \right); \left(1 - \frac{0.1}{(C_{mLT} - 0.25)} \frac{N_{Ed}}{N_{b,Rd,z}} \right) \right]$$

BS EN 1993-1-1
Table B.3

$$C_{mLT} = 0.6 + 0.4 \psi$$

$$\psi = \frac{870}{1034} = 0.84$$

$$C_{mLT} = 0.6 + 0.4 \times 0.84 = 0.937 > 0.4$$

$$\therefore C_{mLT} = 0.937$$

$$k_{zy} = \max \left[\left(1 - \frac{0.1 \times 0.480}{(0.937 - 0.25)} \frac{271}{4898} \right); \left(1 - \frac{0.1}{(0.937 - 0.25)} \frac{271}{4898} \right) \right]$$

$$k_{zy} = \max (0.996; 0.992) = 0.996$$

$$\frac{N_{Ed}}{N_{b,z,Rd}} + k_{zy} \frac{M_{y,Ed}}{M_{b,Rd}} = \frac{271}{4898} + 0.996 \times \frac{1034}{1377} = 0.80 < 1.0, \text{ OK}$$

10.1.5 Verification of revised restraint arrangement - intermediate segment

Flexural buckling resistance about the minor axis, $N_{b,z,Rd}$

As demonstrated for the upper section, $\lambda_1 = 77.5$

$$\bar{\lambda}_z = \frac{L_{cr}}{i_z} \frac{1}{\lambda_1} = \frac{3900}{52.4} \times \frac{1}{77.5} = 0.960$$

BS EN 1993-1-1
§6.3.1.2

As before, curve **b** is to be used, with $\alpha_z = 0.34$

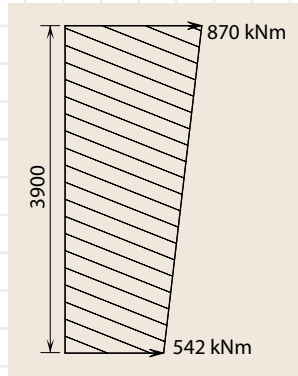
$$\begin{aligned} \phi_z &= 0.5 [1 + \alpha_z (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2] \\ &= 0.5 \times [1 + 0.34 \times (0.960 - 0.2) + 0.960^2] = 1.090 \end{aligned}$$

$$\chi_z = \frac{1}{\phi_z + \sqrt{\phi_z^2 - \bar{\lambda}_z^2}} = \frac{1}{1.090 + \sqrt{1.090^2 - 0.960^2}} = 0.622$$

$$N_{b,z,Rd} = \frac{\chi_z A f_y}{\gamma_{M1}} = \frac{0.622 \times 15900 \times 345}{1.0} \times 10^{-3} = 3414 \text{ kN}$$

$N_{Ed} = 271 \text{ kN}, < 3414 \text{ kN}, \text{ OK}$

Lateral-torsional buckling resistance, $M_{b,Rd}$



C_1 is calculated based on the bending moment diagram over the segment.

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$$\psi = \frac{542}{870} = 0.623 \quad \therefore C_1 = 1.26$$

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{L^2} \sqrt{\frac{I_w}{I_z} + \frac{L^2 GI_T}{\pi^2 EI_z}}$$

$$= 1.26 \times \frac{\pi^2 \times 210000 \times 4380 \times 10^4}{3900^2} \times \sqrt{\frac{4800 \times 10^9}{4380 \times 10^4} + \frac{3900^2 \times 81000 \times 116 \times 10^4}{\pi^2 \times 210000 \times 4380 \times 10^4}}$$

$$M_{cr} = 2671 \times 10^6 \text{ Nmm}$$

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}} = \sqrt{\frac{3990 \times 10^3 \times 345}{2671 \times 10^6}} = 0.718$$

As before, curve **c** is to be used, with $\alpha_{LT} = 0.49$

NA to
BS EN 1993-1-1
NA.2.17

$$\phi_{LT} = 0.5 [1 + \alpha_{LT} (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \bar{\lambda}_{LT}^2]$$

$$\phi_{LT} = 0.5 \times [1 + 0.49 \times (0.718 - 0.4) + 0.75 \times 0.718^2] = 0.771$$

BS EN 1993-1-1
§6.3.2.3

$$\chi_{LT} = \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \beta \bar{\lambda}_{LT}^2}}$$

$$\chi_{LT} = \frac{1}{0.771 + \sqrt{0.771^2 - 0.75 \times 0.718^2}} = 0.815$$

To calculate the modification factor, f

NA to
BS EN 1993-1-1
NA.2.18

$$k_c = \frac{1}{\sqrt{C_1}} = \frac{1}{\sqrt{1.26}} = 0.89$$

BS EN 1993-1-1
§6.3.2.3

$$f = 1 - 0.5(1 - k_c)[1 - 2(\bar{\lambda}_{LT} - 0.8)^2]$$

$$f = 1 - 0.5(1 - 0.89)[1 - 2(0.718 - 0.8)^2] = 0.945$$

$$\chi_{LT,mod} = \frac{\chi_{LT}}{f} = \frac{0.815}{0.945} = 0.862$$

$$M_{b,Rd} = \frac{\chi_{LT,mod} W_{pl,y} f_y}{\gamma_{M1}} = \frac{0.862 \times 3990 \times 10^3 \times 345}{1.0} \times 10^{-6} = 1187 \text{ kNm}$$

$$M_{Ed} = 870 \text{ kNm}, < 1187 \text{ kNm}, \text{ OK}$$

Interaction of axial force and bending moment

BS EN 1993-1-1
§6.3.3(4)

As noted earlier, in this situation, Expression 6.62 reduces to:

$$\frac{N_{Ed}}{N_{b,z,Rd}} + k_{zy} \frac{M_{y,Ed}}{M_{b,Rd}} \leq 1.0$$

For $\bar{\lambda}_z \geq 0.4$, the interaction factor k_{zy} is given by:

BS EN 1993-1-1
Table B.2

$$k_{zy} = \max \left[\left(1 - \frac{0.1 \bar{\lambda}_z}{(C_{mLT} - 0.25)} \frac{N_{Ed}}{N_{b,Rd,z}} \right); \left(1 - \frac{0.1}{(C_{mLT} - 0.25)} \frac{N_{Ed}}{N_{b,Rd,z}} \right) \right]$$

BS EN 1993-1-1
Table B.3

$$C_{mLT} = 0.6 + 0.4 \psi$$

$$\psi = \frac{542}{870} = 0.623$$

$$C_{mLT} = 0.6 + 0.4 \times 0.623 = 0.849 > 0.4$$

$$\therefore C_{mLT} = 0.849$$

$$k_{zy} = \max \left[\left(1 - \frac{0.1 \times 0.480}{(0.849 - 0.25)} \frac{271}{3414} \right); \left(1 - \frac{0.1}{(0.849 - 0.25)} \frac{271}{3414} \right) \right]$$

$$k_{zy} = \max (0.987; 0.987) = 0.987$$

$$\frac{N_{Ed}}{N_{b,z,Rd}} + k_{zy} \frac{M_{y,Ed}}{M_{b,Rd}} = \frac{271}{3414} + 0.987 \times \frac{870}{1187} = 0.80 < 1.0, \text{OK}$$

10.1.6 Verification of revised restraint arrangement - lower segment

Flexural buckling resistance about the minor axis, $N_{b,z,Rd}$

As demonstrated for the upper section $\lambda_1 = 77.5$

$$\bar{\lambda}_z = \frac{L_{cr}}{i_z} \frac{1}{\lambda_1} = \frac{6450}{52.4} \times \frac{1}{77.5} = 1.588$$

BS EN 1993-1-1
§6.3.1.2

As before, curve **b** is to be used, with $\alpha_z = 0.34$

$$\phi_z = 0.5 [1 + \alpha_z (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2]$$

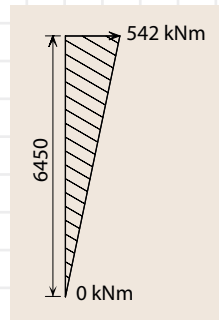
$$= 0.5 \times [1 + 0.34 \times (1.588 - 0.2) + 1.588^2] = 1.997$$

$$\chi_z = \frac{1}{\phi_z + \sqrt{\phi_z^2 - \bar{\lambda}_z^2}} = \frac{1}{1.997 + \sqrt{1.997^2 - 1.588^2}} = 0.312$$

$$N_{b,z,Rd} = \frac{\chi_z A f_y}{\gamma_{M1}} = \frac{0.312 \times 15900 \times 345}{1.0} \times 10^{-3} = 1710 \text{ kN}$$

$$N_{Ed} = 271 \text{ kN}, < 1710 \text{ kN}, \text{ OK}$$

Lateral-torsional buckling resistance, $M_{b,Rd}$



C_1 is calculated based on the bending moment diagram over the segment.

$$\psi = \frac{0}{542} = 0 \quad \therefore C_1 = 1.77$$

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{L^2} \sqrt{\frac{I_w}{I_z} + \frac{L^2 GI_T}{\pi^2 EI_z}}$$

$$= 1.77 \times \frac{\pi^2 \times 210000 \times 4380 \times 10^4}{6450^2} \times \sqrt{\frac{4800 \times 10^9}{4380 \times 10^4} + \frac{6450^2 \times 81000 \times 116 \times 10^4}{\pi^2 \times 210000 \times 4380 \times 10^4}}$$

$$M_{cr} = 1507 \times 10^6 \text{ Nmm}$$

BS EN 1993-1-1 §6.3.2.2

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}} = \sqrt{\frac{3990 \times 10^3 \times 345}{1507 \times 10^6}} = 0.956$$

NA to BS EN 1993-1-1 NA.2.17
As before, curve **c** is to be used, with $\alpha_{LT} = 0.49$

$$\phi_{LT} = 0.5 [1 + \alpha_{LT} (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \bar{\lambda}_{LT}^2]$$

$$\phi_{LT} = 0.5 \times [1 + 0.49 \times (0.956 - 0.4) + 0.75 \times 0.956^2] = 0.979$$

BS EN 1993-1-1 §6.3.2.3

$$\chi_{LT} = \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \beta \bar{\lambda}_{LT}^2}}$$

$$\chi_{LT} = \frac{1}{0.979 + \sqrt{0.979^2 - 0.75 \times 0.956^2}} = 0.666$$

To calculate the modification factor, f

NA to BS EN 1993-1-1 NA.2.18

$$k_c = \frac{1}{\sqrt{C_1}} = \frac{1}{\sqrt{1.77}} = 0.75$$

BS EN 1993-1-1
§6.3.2.3

$$f = 1 - 0.5(1 - k_\varphi)[1 - 2(\bar{\lambda}_{LT} - 0.8)^2]$$

$$f = 1 - 0.5(1 - 0.75)[1 - 2(0.956 - 0.8)^2] = 0.88$$

$$\chi_{LT,mod} = \frac{\chi_{LT}}{f} = \frac{0.666}{0.88} = 0.757$$

$$M_{b,Rd} = \frac{\chi_{LT,mod} W_{pl,y} f_y}{\gamma_{M1}} = \frac{0.757 \times 3990 \times 10^3 \times 345}{1.0} \times 10^{-6} = 1042 \text{ kNm}$$

$$M_{Ed} = 542 \text{ kNm}, < 1042 \text{ kNm}, \text{ OK}$$

Interaction of axial force and bending moment in accordance with Expression 6.62

BS EN 1993-1-1
§6.3.3(4)

As before, Expression 6.62 reduces to:

$$\frac{N_{Ed}}{N_{b,z,Rd}} + k_{zy} \frac{M_{y,Ed}}{M_{b,Rd}} \leq 1.0$$

BS EN 1993-1-1
Table B.2

For $\bar{\lambda}_z \geq 0.4$, the interaction factor k_{zy} is calculated as:

$$k_{zy} = \max \left[\left(1 - \frac{0.1 \bar{\lambda}_z}{(C_{mLT} - 0.25)} \frac{N_{Ed}}{N_{b,Rd,z}} \right); \left(1 - \frac{0.1}{(C_{mLT} - 0.25)} \frac{N_{Ed}}{N_{b,Rd,z}} \right) \right]$$

BS EN 1993-1-1
Table B.3

$$C_{mLT} = 0.6 + 0.4\psi$$

$$\psi = \frac{0}{542} = 0$$

$$C_{mLT} = 0.6 + 0.4 \times 0 = 0.6 > 0.4$$

$$\therefore C_{mLT} = 0.6$$

$$k_{zy} = \max \left[\left(1 - \frac{0.1 \times 1.588}{(0.6 - 0.25)} \frac{271}{1710} \right); \left(1 - \frac{0.1}{(0.6 - 0.25)} \frac{271}{1710} \right) \right]$$

$$k_{zy} = \max (0.928; 0.955) = 0.955$$

$$\frac{N_{Ed}}{N_{b,z,Rd}} + k_{zy} \frac{M_{y,Ed}}{M_{b,Rd}} = \frac{271}{1710} + 0.955 \times \frac{542}{1042} = 0.66 < 1.0, \text{ OK}$$

10.1.7 Verification of revised restraint arrangement – major axis

Flexural buckling resistance about the major axis, $N_{b,y,Rd}$

As before, $\frac{h}{b} = 2.68$ and $\lambda_1 = 77.5$

BS EN 1993-1-1
Table 6.2
Table 6.1

For buckling about the y-y axis, curve **a** should be used for hot rolled I sections with $\alpha_y = 0.21$.

In this example, the buckling length is taken as the system length, which is the distance between nodes (i.e. the length of the column), $L = 13100$ mm.

As demonstrated previously, $\lambda_1 = 77.5$

$$\bar{\lambda}_y = \frac{L_{cr}}{i_y} \frac{1}{\lambda_1} = \frac{13100}{246} \times \frac{1}{77.5} = 0.621$$

BS EN 1993-1-1
§6.3.1.2

$$\phi_y = 0.5 [1 + \alpha_y (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2]$$

$$= 0.5 \times [1 + 0.21 \times (0.621 - 0.2) + 0.621^2] = 0.737$$

BS EN 1993-1-1
§6.3.1.2

$$\chi_y = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}^2}} = \frac{1}{0.737 + \sqrt{0.737^2 - 0.621^2}} = 0.882$$

$$N_{b,y,Rd} = \frac{\chi_y A f_y}{\gamma_{M1}} = \frac{0.882 \times 15900 \times 345}{1.0} \times 10^{-3} = 4837 \text{ kN}$$

$$N_{Ed} = 271 \text{ kN}, < 4837 \text{ kN, OK}$$

Interaction of axial force and bending moment in accordance with Expression 6.61

As noted earlier, in this situation, Expression 6.61 reduces to:

$$\frac{N_{Ed}}{N_{b,y,Rd}} + k_{yy} \frac{M_{y,Ed}}{M_{b,Rd}} \leq 1.0$$

The most onerous ratio of $\frac{M_{y,Ed}}{M_{b,Rd}}$ from the three segments will be considered in combination with the major axis flexural buckling.

$$\text{Upper} \quad \frac{M_{y,Ed}}{M_{b,Rd}} = \frac{1034}{1377} = 0.75$$

$$\text{Intermediate} \quad \frac{M_{y,Ed}}{M_{b,Rd}} = \frac{870}{1187} = 0.73$$

$$\text{Lower} \quad \frac{M_{y,Ed}}{M_{b,Rd}} = \frac{542}{1042} = 0.52$$

The interaction factor k_{yy} is given by:

$$k_{yy} = \min \left[C_{my} \left(1 + (\bar{\lambda}_y - 0.2) \frac{N_{Ed}}{N_{b,y,Rd}} \right); C_{my} \left(1 + 0.8 \frac{N_{Ed}}{N_{b,y,Rd}} \right) \right]$$

For C_{my} , the relevant braced points are the torsional restraints at the ends of the member.

From Table B.3, C_{my} is:

$$C_{my} = 0.6 + 0.4\psi \geq 0.4$$

$$\psi = \frac{0}{1034} = 0$$

$$C_{my} = 0.6 + 0.4 \times 0 = 0.6$$

$$k_{yy} = \min \left[0.6 \left(1 + (0.621 - 0.2) \frac{271}{4837} \right); 0.6 \left(1 + 0.8 \frac{271}{4837} \right) \right]$$

$$= \min(0.614; 0.627) = 0.614$$

$$\frac{N_{Ed}}{N_{b,y,Rd}} + k_{yy} \frac{M_{y,Ed}}{M_{b,Rd}} = \frac{271}{4837} + 0.614 \times 0.75 = 0.52 < 1.0, \text{ OK}$$

10.1.8 Summary: Adequacy of the column section

The cross sectional resistance, flexural buckling resistance and lateral torsional buckling resistance have been demonstrated to be adequate. The interaction of flexural and lateral torsional buckling has been verified using expressions 6.61 and 6.62.

Therefore it is concluded that a $686 \times 254 \times 125$ UKB, Grade S355 is adequate for use as the column in this portal frame, considering load Combination 1.

Similar verifications will be required for the other load combinations.

10.2 Rafter Verification

Figure D.15 shows the bending moment diagram over one rafter for Combination 1, together with the purlin positions.

From the analysis, the forces and moments in the rafter are:

$$V_{Ed} = 187 \text{ kN (maximum value)}$$

$$N_{Ed} = 109 \text{ kN (maximum value)}$$

$$M_{Ed} = 462 \text{ kNm (at the end of the haunch)}$$

$$M_{Ed} = 620 \text{ kNm (adjacent to the apex)}$$

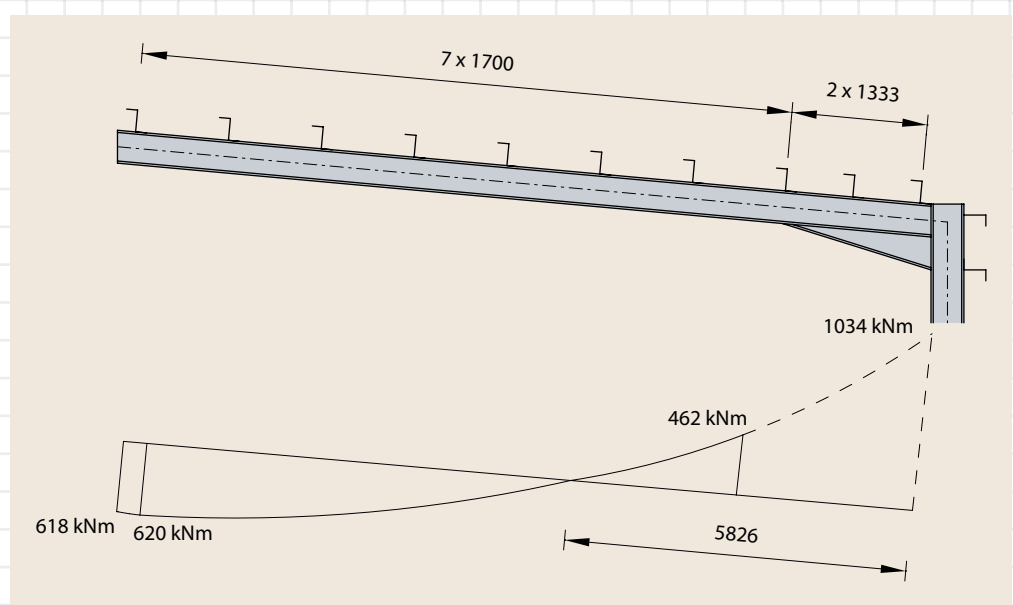


Figure D.15
Bending moment diagram over the length of the rafter

Each segment between torsional restraints must be checked under each loading combination for lateral-torsional buckling and flexural buckling about the minor axis. In this example, typical checks are demonstrated for the most onerous segment in Zone C (see Figure D.16) and the most onerous segment in Zone B. Buckling about the major axis is then checked.

The haunch region is checked separately in Section 11 of this example.

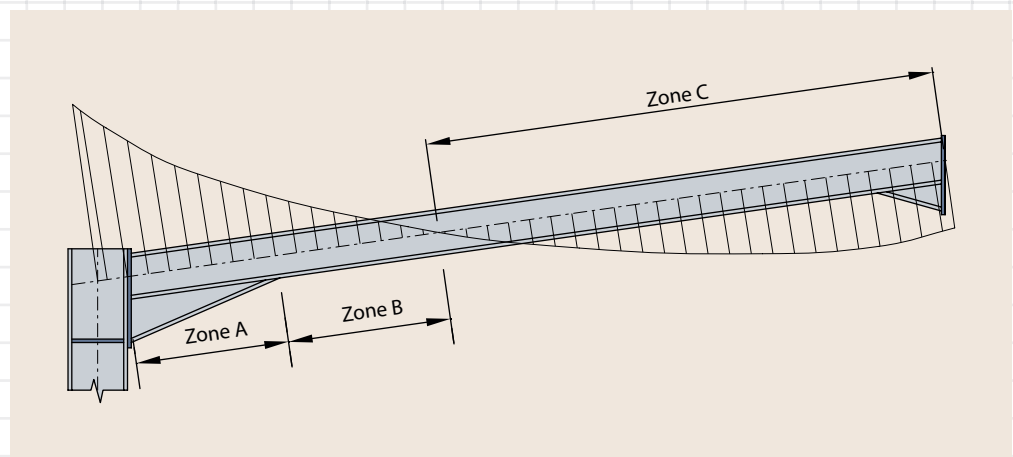


Figure D.16
Definition of zones

10.2.1 Zone C – sagging region

For hand calculations, it is conservative to verify the rafter assuming that the maximum moment is uniform between purlins, as shown in Figure D.17.

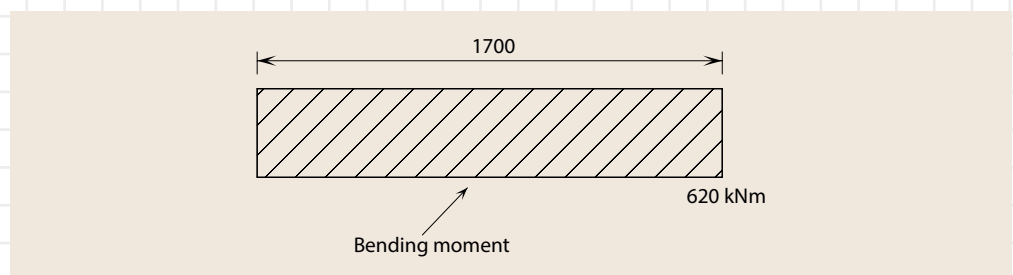


Figure D.17
Assumed uniform moment distribution between purlins

Flexural buckling resistance about the minor axis, $N_{b,z,Rd}$

$$\frac{h}{b} = \frac{536.7}{210} = 2.56$$

$$t_f = 17.4 \text{ mm}, \therefore f_y = 345 \text{ N/mm}^2. \text{ As before, } \lambda_1 = 77.5$$

$$\bar{\lambda}_z = \frac{L_{cr}}{i_z} \frac{1}{\lambda_1} = \frac{1700}{42} \times \frac{1}{77.5} = 0.480$$

BS EN 1993-1-1
Table 6.1, Table 6.2

For buckling about z-z axis, curve **b** is to be used, with $\alpha_z = 0.34$

BS EN 1993-1-1
§6.3.1.2

$$\phi_z = 0.5 [1 + \alpha_z (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2]$$

$$\phi_z = 0.5 \times [1 + 0.34 \times (0.480 - 0.2) + 0.480^2] = 0.663$$

$$\chi_z = \frac{1}{\phi_z + \sqrt{\phi_z^2 - \bar{\lambda}_z^2}} = \frac{1}{0.663 + \sqrt{0.663^2 - 0.480^2}} = 0.893$$

$$N_{b,z,Rd} = \frac{\chi_z A f_y}{\gamma_{M1}} = \frac{0.893 \times 12900 \times 345}{1.0} \times 10^{-3} = 3974 \text{ kN}$$

$$N_{Ed} = 109 \text{ kN}, < 3974 \text{ kN, OK}$$

Lateral-torsional buckling resistance, $M_{b,Rd}$

Appendix B of
this document

In this case, the bending moment diagram is assumed to be constant along the segment in consideration, so $\psi = 1.0$. Therefore $C_1 = 1.0$.

Appendix B of
this document

$$M_{cr} = C_1 \frac{\pi^2 E I_z}{L^2} \sqrt{\frac{I_w}{I_z} + \frac{L^2 G I_T}{\pi^2 E I_z}}$$

$$= 1.0 \times \frac{\pi^2 \times 210000 \times 2690 \times 10^4}{1700^2} \times \sqrt{\frac{1810 \times 10^9}{2690 \times 10^4} + \frac{1700^2 \times 81000 \times 101 \times 10^4}{\pi^2 \times 210000 \times 2690 \times 10^4}}$$

$$M_{cr} = 5159 \times 10^6 \text{ Nmm}$$

BS EN 1993-1-1
§6.3.2.2

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_{pl,y} f_y}{M_{cr}}} = \sqrt{\frac{2610 \times 10^3 \times 345}{5159 \times 10^6}} = 0.418$$

BS EN 1993-1-1
Table 6.3
Table 6.5

$$\frac{h}{b} = 2.56, \text{ so curve } \mathbf{c} \text{ is to be used, with } \alpha_{LT} = 0.49$$

BS EN 1993-1-1
§6.3.2.3

$$\phi_{LT} = 0.5 [1 + \alpha_{LT} (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \bar{\lambda}_{LT}^2]$$

$$\phi_{LT} = 0.5 \times [1 + 0.49 \times (0.418 - 0.4) + 0.75 \times 0.418^2] = 0.570$$

BS EN 1993-1-1
§6.3.2.3

$$\chi_{LT} = \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \beta \bar{\lambda}_{LT}^2}}$$

$$\chi_{LT} = \frac{1}{0.570 + \sqrt{0.570^2 - 0.75 \times 0.418^2}} = 0.99$$

$$\frac{1}{\bar{\lambda}_{LT}^2} = \frac{1}{0.407^2} = 6.05$$

$$\therefore \chi_{LT} = 0.99$$

Because χ_{LT} is so close to 1.0, there is little point in calculating $\chi_{LT,mod}$, since this cannot exceed 1.0.

$$M_{b,Rd} = \frac{\chi_{LT} W_{pl,y} f_y}{\gamma_{M1}} = \frac{0.990 \times 2610 \times 10^3 \times 345}{1.0} \times 10^{-6} = 892 \text{ kNm}$$

$$M_{y,Ed} = 620 \text{ kNm}, < 892 \text{ kNm}, \text{ OK}$$

BS EN 1993-1-1
§6.3.3(4)

Interaction of axial force and bending moment in accordance with Expression 6.62

As noted earlier, in this situation, Expression 6.62 reduces to:

$$\frac{N_{Ed}}{N_{b,z,Rd}} + k_{zy} \frac{M_{y,Ed}}{M_{b,Rd}} \leq 1.0$$

For $\bar{\lambda}_z \geq 0.4$, the interaction factor k_{zy} is calculated as:

$$k_{zy} = \max \left[\left(1 - \frac{0.1 \bar{\lambda}_z}{(C_{mLT} - 0.25)} \frac{N_{Ed}}{N_{b,z,Rd}} \right); \left(1 - \frac{0.1}{(C_{mLT} - 0.25)} \frac{N_{Ed}}{N_{b,z,Rd}} \right) \right]$$

BS EN 1993-1-1
Table B.3

The bending moment is assumed to be uniform. Therefore C_{mLT} is taken as 1.0.

BS EN 1993-1-1
Table B.2

$$k_{zy} = \max \left[\left(1 - \frac{0.1 \times 0.480}{(1 - 0.25)} \frac{84}{3974} \right); \left(1 - \frac{0.1}{(1 - 0.25)} \frac{84}{3974} \right) \right]$$

$$= \max (0.999; 0.997) = 0.999$$

$$\frac{N_{Ed}}{N_{b,z,Rd}} + k_{zy} \frac{M_{y,Ed}}{M_{b,Rd}} = \frac{109}{3974} + 0.999 \times \frac{620}{892} = 0.722 < 1.0, \text{ OK}$$

10.2.2 Zone B – hogging region

In this region, in Combination 1, the bottom flange is in compression. Torsional restraints are provided at certain locations by stays from the purlins to the inside flange.

As discussed in Section 7.4.1, in this zone the buckling length is taken from the torsional restraint at the sharp end of the haunch to the ‘virtual’ restraint at the point of contraflexure of the bending moment diagram. If the rafter cannot be verified over this length, additional restraints to the inside flange will be required.

Section 7.4.1 of this publication

A virtual restraint may be assumed at the point of contraflexure, as the rafter is a UKB section, the depth of the purlins is not less than 0.25 times the depth of the rafter and the purlin-to-rafter connection comprises at least two bolts.

For the cases when the above conditions are not satisfied, the buckling length should be taken to the next purlin past the point of contraflexure (i.e. the first restraint to the compression flange).

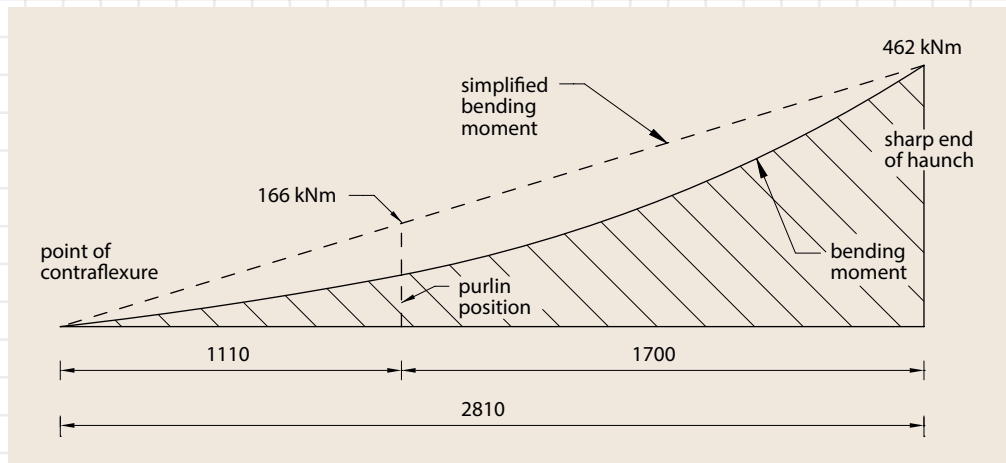
From the analysis, distance to the point of contraflexure is 5826 mm (See Figure D.15).

The buckling length from the end of the haunch to the point of contraflexure is therefore $5826 - \frac{3000}{\cos 6^\circ} = 2810$ mm.

Section 7.2.3 of this publication

If the spacing of the intermediate restraints to the tension flange is small enough, the design buckling resistance may be increased. The restraints are effective if their spacing does not exceed L_m .

Figure D.18
Bending moment diagram between the end of the haunch and the point of contraflexure (Combination 1)



Spacing of restraints to the tension flange

The limiting spacing is given by:

BS EN 1993-1-1
BB.3.1.1

$$L_m = \frac{38i_z}{\sqrt{\frac{1}{57.4} \left(\frac{N_{Ed}}{A} \right) + \frac{1}{756C_1^2} \frac{W_{ply}^2}{AI_T} \left(\frac{f_y}{235} \right)^2}}$$

Appendix B of this publication

$$\psi = \frac{166}{464} = 0.358 \quad \therefore C_1 = 1.469$$

$$L_m = \frac{38 \times 45.7}{\sqrt{\frac{1}{57.4} \left(\frac{109 \times 10^3}{12900} \right) + \frac{1}{756 \times 1.469^2} \frac{(2610 \times 10^3)^2}{12900 \times 101 \times 10^4} \left(\frac{345}{235} \right)^2}}$$

$$L_m = 1897 \text{ mm}$$

Restraint to the tension flange is provided by purlins spaced at 1700 mm, which is less than the limiting spacing of 1897 mm.

Therefore advantage may be taken of the restraints to the tension flange.

Initially, the hogging region is verified assuming no intermediate torsional restraints.

If the flexural buckling, lateral torsional buckling and interaction checks are satisfied for the length of the whole hogging region, no further torsional restraints are required.

Otherwise, intermediate torsional restraints will need to be introduced to the rafter in the hogging zone or the rafter size increased.

Flexural buckling resistance about the minor axis, $N_{b,z,Rd}$

The advantage of tension flange restraint will be utilised.

The distance a between the restrained longitudinal axis and the shear centre of the rafter assuming 200 mm deep purlins is given by:

$$a = 0.5 \times 536.7 + 0.5 \times 200 = 368.4 \text{ mm}$$

BS EN 1993-1-1
BB.3.3.1

The elastic critical torsional buckling force between torsional restraints is given by:

$$N_{crT} = \frac{1}{i_s^2} \left(\frac{\pi^2 EI_z a^2}{L_t^2} + \frac{\pi^2 EI_w}{L_t^2} + GI_T \right)$$

In which

$$i_s^2 = i_y^2 + i_z^2 + a^2 = 45.72 + 2192 + 368.42 = 185768 \text{ mm}^2$$

$$N_{crT} = \frac{1}{185768} \left(\frac{\pi^2 \times 2.1 \times 2690 \times 10^9 \times 368.4^2}{2810^2} + \frac{\pi^2 \times 2.1 \times 1810 \times 10^{14}}{2810^2} + 8.1 \times 101 \times 10^9 \right)$$

$$N_{crT} = 8155 \text{ kN}$$

BS EN 1993-1-1
§6.3.1.3

$$\bar{\lambda}_z = \sqrt{\frac{A \times f_y}{N_{cr}}} = \sqrt{\frac{12900 \times 345}{8155}} = 0.739$$

BS EN 1993-1-1
Table 6.2, Table 6.1

As before, $\frac{h}{b} = 2.56$, and curve **b** should be used, with $\alpha_z = 0.34$

BS EN 1993 1 1
§6.3.1.2

$$\phi_z = 0.5 [1 + \alpha_z (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2]$$

$$\phi_z = 0.5 \times [1 + 0.34 \times (0.739 - 0.2) + 0.739^2] = 0.864$$

$$\chi_z = \frac{1}{\phi_z + \sqrt{\phi_z^2 - \lambda_z^2}} = \frac{1}{0.864 + \sqrt{0.864^2 - 0.739^2}} = 0.761$$

$$N_{b,z,Rd} = \frac{\chi_z A f_y}{\gamma_{M1}} = \frac{0.761 \times 12900 \times 345}{1.0} \times 10^{-3} = 3389 \text{ kN}$$

$$N_{Ed} = 109 \text{ kN}, < 3389 \text{ kN, OK}$$

Lateral-torsional buckling resistance, $M_{b,Rd}$

To determine the non-dimensional slenderness for lateral-torsional buckling, the value of M_{cr0} must first be calculated for a member with intermediate restraints subject to a uniform moment.

Section C.2.2 of
this publication

$$M_{cr0} = \frac{i_s^2}{2a} N_{cr} = \frac{185768}{2 \times 368.4} \times 8155 \times 10^{-3} = 2056 \text{ kNm}$$

Then, for a linear bending moment diagram, M_{cr} is given by $M_{cr} = c^2 C_m M_{cr0}$.

Section C.2.1 of
this publication

To determine C_m , $N_{cr,E}$ must be calculated

$$N_{cr,E} = \frac{\pi^2 EI_z}{L_t^2} = \frac{\pi^2 \times 210 \times 10^3 \times 2690 \times 10^4}{2810^2} \times 10^{-3} = 7061 \text{ kN}$$

$$\eta = \frac{N_{cr,E}}{N_{cr,T}} = \frac{7061}{8155} = 0.866$$

$$B_0 = \frac{1+10\eta}{1+20\eta} = \frac{1+10 \times 0.866}{1+20 \times 0.866} = 0.527$$

$$B_1 = \frac{5\sqrt{\eta}}{\pi + 10\sqrt{\eta}} = \frac{5\sqrt{0.866}}{\pi + 10\sqrt{0.866}} = 0.374$$

$$B_2 = \frac{0.5}{1 + \pi\sqrt{\eta}} - \frac{0.5}{1 + 20\eta} = \frac{0.5}{1 + \pi\sqrt{0.866}} - \frac{0.5}{1 + 20 \times 0.866} = 0.100$$

$$\beta_t = \frac{M_{\min}}{M_{\max}} = \frac{0}{464} = 0$$

$$C_m = \frac{1}{B_0 + B_1\beta_t + B_2\beta_t^2} = \frac{1}{0.527 + 0.374 \times 0 + 0.100 \times 0^2} = 1.898$$

Because the member is uniform, the taper factor, c , is taken as 1.0

$$M_{cr} = c^2 C_m M_{cr0} = 1^2 \times 1.898 \times 2056 = 3902 \text{ kNm}$$

The non-dimensional slenderness, $\bar{\lambda}_{LT}$, is given by:

BS EN 1993-1-1
§6.3.2.2

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}} = \sqrt{\frac{2610 \times 10^3 \times 345}{3902 \times 10^6}} = 0.480$$

The reduction factor χ_{LT} is given by Clause 6.3.2.3 for rolled sections:

BS EN 1993-1-1
Table 6.3
Table 6.5

As before, $h/b = 2.56$ therefore curve **c** is used, with $\alpha_{LT} = 0.49$

$$\phi_{LT} = 0.5[1 + \alpha_{LT}(\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \bar{\lambda}_{LT}^2]$$

$$\phi_{LT} = 0.5 \times [1 + 0.49 \times (0.480 - 0.4) + 0.75 \times 0.480^2] = 0.606$$

BS EN 1993-1-1
§6.3.2.3

$$\chi_{LT} = \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \beta \bar{\lambda}_{LT}^2}}$$

$$\chi_{LT} = \frac{1}{0.606 + \sqrt{0.606^2 - 0.75 \times 0.480^2}} = 0.955$$

$$\frac{1}{\bar{\lambda}_{LT}^2} = \frac{1}{0.480^2} = 4.330$$

$$\therefore \chi_{LT} = 0.955$$

NA to
BS EN 1993-1-1
NA.2.18

To calculate the modification factor, f

$$k_c = \frac{1}{\sqrt{C_1}} = \frac{1}{\sqrt{1.77}} = 0.75$$

BS EN 1993-1-1
§6.3.2.3

$$f = 1 - 0.5(1 - k_c)[1 - 2(\bar{\lambda}_{LT} - 0.8)^2]$$

$$f = 1 - 0.5(1 - 0.75)[1 - 2(0.481 - 0.8)^2] = 0.90$$

The modified reduction factor is given by

$$\chi_{LT,mod} = \frac{\chi_{LT}}{f} = \frac{0.955}{0.90} = 1.06, \text{ but limited to } 1.0$$

$$\chi_{LT,mod} = 1.0$$

The bucking resistance moment is given by:

$$M_{b,Rd} = \frac{\chi_{LT,mod} W_{pl,y} f_y}{\gamma_{M1}} = \frac{1.0 \times 2610 \times 10^3 \times 345}{1.0} \times 10^{-6} = 901 \text{ kNm}$$

$$M_{Ed} = 462 \text{ kNm}, < 901 \text{ kNm}, \text{ OK}$$

BS EN 1993-1-1
§6.3.3(4)

Interaction of axial force and bending moment in accordance with Expression 6.62

As noted earlier, in this situation, Expression 6.62 reduces to:

$$\frac{N_{Ed}}{N_{b,z,Rd}} + k_{zy} \frac{M_{y,Ed}}{M_{b,Rd}} \leq 1.0$$

BS EN 1993-1-1
Table B.2

For $\bar{\lambda}_z \geq 0.4$, the interaction factor k_{zy} is given by:

$$k_{zy} = \max \left[\left(1 - \frac{0.1 \bar{\lambda}_z}{(C_{mLT} - 0.25)} \frac{N_{Ed}}{N_{b,Rd,z}} \right); \left(1 - \frac{0.1}{(C_{mLT} - 0.25)} \frac{N_{Ed}}{N_{b,Rd,z}} \right) \right]$$

BS EN 1993-1-1
Table B.3

$$C_{mLT} = 0.6 + 0.4 \psi$$

$$\psi = \frac{0}{464} = 0$$

$$C_{mLT} = 0.6 + 0.4 \times 0 = 0.6 > 0.4$$

$$\therefore C_{mLT} = 0.6$$

$$k_{zy} = \max \left[\left(1 - \frac{0.1 \times 0.739}{(0.6 - 0.25)} \frac{109}{3389} \right); \left(1 - \frac{0.1}{(0.6 - 0.25)} \frac{109}{3389} \right) \right]$$

$$k_{zy} = \max (0.993; 0.991) = 0.993$$

$$\frac{N_{Ed}}{N_{b,z,Rd}} + k_{zy} \frac{M_{y,Ed}}{M_{b,Rd}} = \frac{109}{3389} + 0.993 \times \frac{464}{901} = 0.54 < 1.0, \text{ OK}$$

10.2.3 Resistance to in-plane buckling and bending

Flexural buckling resistance about the major axis, $N_{b,y,Rd}$

As before, $\frac{h}{b} = 2.56$ and $\lambda_1 = 77.5$

BS EN 1993-1-1
Table 6.2, Table 6.1
Section 7.4.4

For buckling about the y-y axis, curve **a** is used, with $\alpha_y = 0.21$

The buckling length must be determined. Software may calculate this length precisely.

For a symmetric, single span, elastically designed portal frame of orthodox geometry,

a reasonable approximation is to assume that the buckling length is the developed length from eaves to apex. Hence,

Section 7.4.4
$$L_{cr} = \frac{15000}{\cos 6^\circ} = 15083 \text{ mm}$$

$$\bar{\lambda}_y = \frac{L_{cr}}{i_y} \frac{1}{\lambda_1} = \frac{15083}{188} \times \frac{1}{77.5} = 0.889$$

BS EN 1993-1-1 §6.3.1.2
$$\phi_y = 0.5 [1 + \alpha_y (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2]$$

$$= 0.5 \times [1 + 0.21 \times (0.889 - 0.2) + 0.889^2] = 0.967$$

$$\chi_y = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}^2}} = \frac{1}{0.967 + \sqrt{0.967^2 - 0.889^2}} = 0.741$$

$$N_{b,y,Rd} = \frac{\chi_y A f_y}{\gamma_{M1}} = \frac{0.741 \times 12900 \times 345}{1.0} \times 10^{-3} = 3300 \text{ kN}$$

$$N_{Ed} = 109 \text{ kN}, < 3300 \text{ kN, OK}$$

Interaction of axial force and bending moment in accordance with Expression 6.61

As noted earlier, in this situation, Expression 6.61 reduces to:

Section 7.2 of this publication
$$\frac{N_{Ed}}{N_{b,y,Rd}} + k_{yy} \frac{M_{y,Ed}}{M_{b,Rd}} \leq 1.0$$

The most onerous ratio of $\frac{M_{y,Ed}}{M_{b,Rd}}$ from Zones B and C will be considered in combination with the major axis flexural buckling.

Adjacent to the haunch
$$\frac{M_{y,Ed}}{M_{b,Rd}} = \frac{464}{901} = 0.52$$

Adjacent to the apex
$$\frac{M_{y,Ed}}{M_{b,Rd}} = \frac{620}{829} = 0.75$$

The bending moment diagram along the entire length of the rafter (as shown in Figure D.19) is considered when determining C_{my} .

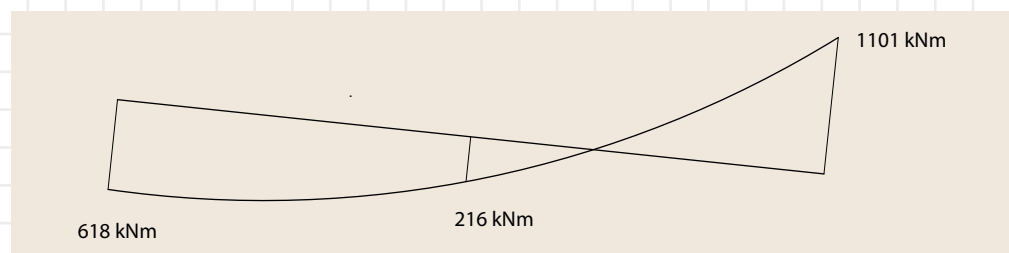


Figure D.19
Rafter bending moment diagram

BS EN 1993-1-1
Table B.2

The interaction factor, k_{yy} , is calculated as follows:

$$k_{yy} = \min \left[C_{my} \left(1 + (\bar{\lambda}_y - 0.2) \frac{N_{Ed}}{N_{b,y,Rd}} \right); C_{my} \left(1 + 0.8 \frac{N_{Ed}}{N_{b,y,Rd}} \right) \right]$$

BS EN 1993-1-1
Table B.3

The expression for C_{my} depends on the values of α_s (the ratio of the midspan moment to the larger end moment) and ψ (the ratio of the end moments).

$$\psi = \frac{-618}{1101} = -0.56$$

The midspan moment (determined from the analysis) is 216 kNm

$$\alpha_s = \frac{M_s}{M_h} = \frac{-216}{1101} = -0.20$$

Because $-1 \leq \alpha_s < 0$ and $-1 \leq \psi < 0$, C_{my} is calculated as:

$$C_{my} = 0.1(1 - \psi) - 0.8\alpha_s \text{ but } \geq 0.4$$

$$C_{my} = 0.1(1 - (-0.56)) - 0.8 \times (-0.20) = 0.32, \text{ but } C_{my} \geq 0.4, \text{ so } C_{my} = 0.4$$

BS EN 1993-1-1
Table B.2
Table B.1

$$k_{yy} = \min \left[0.4 \left(1 + (0.889 - 0.2) \frac{109}{3300} \right); 0.4 \left(1.0 + 0.8 \frac{109}{3300} \right) \right]$$

$$= \min [0.41; 0.41] = 0.41$$

Using the most onerous $\frac{M_{y,Ed}}{M_{b,Rd}}$ ratio:

$$\frac{N_{Ed}}{N_{b,y,Rd}} + k_{yy} \frac{M_{y,Ed}}{M_{b,Rd}} = \frac{109}{3300} + 0.41 \times 0.76 = 0.34 < 1.0, \text{ OK}$$

10.2.4 Summary: Adequacy of the rafter section

The cross sectional resistance, flexural buckling resistance and lateral torsional buckling resistance have been demonstrated to be adequate. The interaction of lateral torsional buckling with both in-plane and out-of-plane flexural buckling has been verified using Expressions 6.61 and 6.62.

Therefore it is concluded that a 533 × 210 × 101 UKB, S355 is adequate for use as the rafter in this portal frame, for load Combination 1.

11 Verification of haunched length

The haunch is fabricated from a cutting of 610 × 229 × 125 UKB, S355 section.

Checks are be carried out at the end and quarter points, as indicated in Figure D.20.

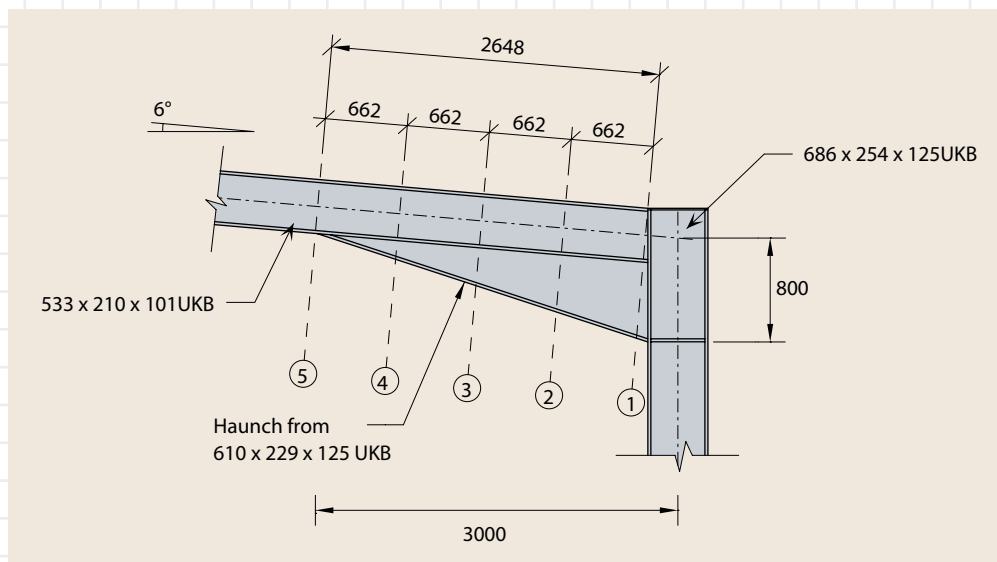


Figure D.20
Division of the haunch into relevant sections

Table D.6 shows the cross sectional properties of the compound section at each of the cross sections indicated in Figure D.20. Table D.6 also gives the calculated design compression and bending moment at the respective cross section.

CROSS SECTION NO.	CUTTING DEPTH (mm)	OVERALL DEPTH (mm)	GROSS AREA, A (mm ²)	I_y (cm ⁴)	$W_{el,min}$ (cm ³)	N_{Ed} (kN)	M_{Ed} (kNm)
1	545.1	1081.8	23500	332000	5960	108	1016
2	408.8	945.5	21900	242000	4850	107	867
3	272.6	809.3	20200	171000	3870	106	724
4	136.3	673.0	18600	118000	3060	105	589
5	0.0	536.7	12900	61500	2290	103	462

Table D.6
Gross section properties and design forces and moments

The section properties are calculated normal to the longitudinal axis of the rafter section.

The middle flange has been included in the section properties presented in Table D.6.

For cross section 1 the values of N_{Ed} and M_{Ed} are taken at the face of the column.

The design bending moments are illustrated in Figure D.21.

11.1 Cross section classification

A simple approach is adopted to classify the compound section. The elements are classified assuming a conservative stress distribution, simply to identify whether they are Class 2 or better. If the cross section is at least Class 2, the bending resistance will be calculated based on the plastic properties.

In the gravity load combination, the web of the haunch is likely to be the critical element, especially at deeper cross sections.

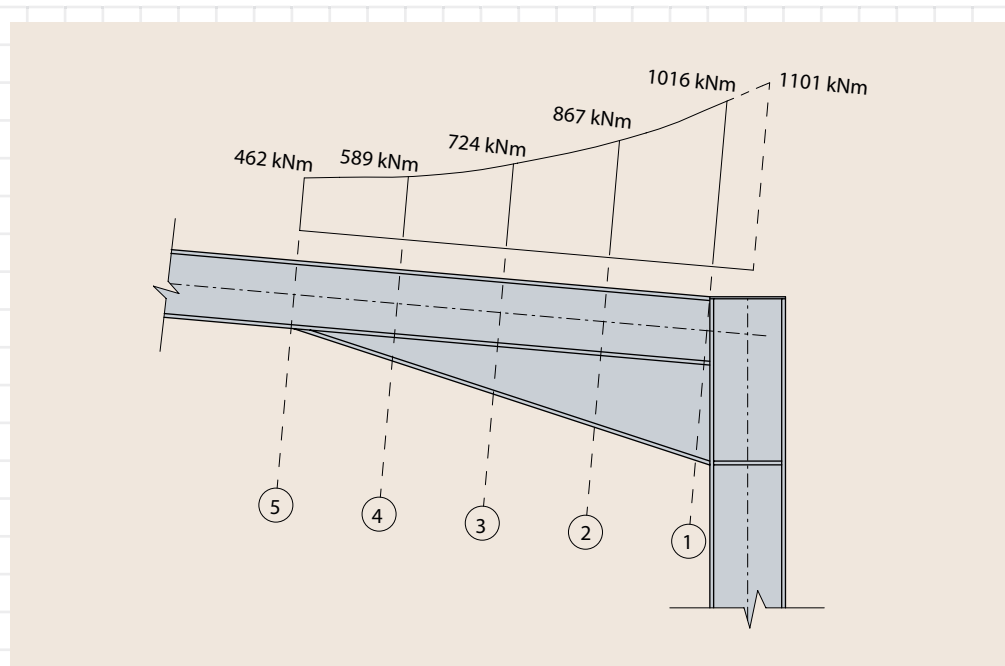


Figure D.21
Design bending
moment at each
cross section

Section 7.1.3 of
this publication

If the flanges are Class 1 or 2, but the web is not, effective plastic properties are calculated.

This classification must be completed for each cross section along the haunch.

Full calculations are presented for cross section 1, being the deepest section. Classes for the remaining cross sections are summarised in Table D.8.

As the axial compression in the rafter is small, most elements of the rafter section will be in tension under the gravity load combination (due to the large bending moment).

It is therefore not necessary to classify the elements of the rafter section.

Haunch web

Assume the haunch web is subject only to compression (the most onerous condition).

Because $t_f = 19.6 \text{ mm}$, $f_y = 345 \text{ N/mm}^2$.

$$\text{Therefore } \varepsilon = \sqrt{\frac{235}{345}} = 0.825$$

$$\text{Class 2 } c/t \text{ limit} = 38\varepsilon = 38 \times 0.825 = 31.4$$

$$\text{Actual } c/t = \frac{545.1 - 19.6}{11.9} = 44.1$$

According to this simple check, the haunch web is not Class 2.

Haunch flange

$$\text{Class 2 } c/t \text{ limit} = 10\varepsilon = 10 \times 0.825 = 8.25$$

$$\text{Actual } c/t = \frac{95.9}{19.6} = 4.9$$

$4.9 < 8.25$, therefore the haunch flange is at least Class 2.

BS EN 1993-1-1
Table 5.2

11.1.2 Effective plastic modulus

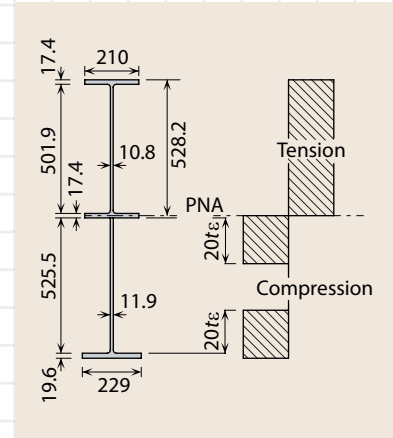
Section 7.1.3 of this publication

Because the haunch web is not Class 2, effective properties will be calculated, assuming the haunch web is only effective over a distance of $20t\epsilon$ from the flanges. Conservatively, the fillets have been ignored, and the $20t\epsilon$ has been taken from the face of the flange, not from the end of the fillet.

$$20t\epsilon = 20 \times 11.9 \times 0.825 = 196 \text{ mm}$$

Areas of the haunch components are:

Top flange	210×17.4	$= 3654 \text{ mm}^2$
Rafter web	501.9×10.8	$= 5420 \text{ mm}^2$
Middle flange		$= 3654 \text{ mm}^2$
$20t\epsilon$ (upper) $\times t$	$= 196 \times 11.9$	$= 2338 \text{ mm}^2$
$20t\epsilon$ (lower)		$= 2338 \text{ mm}^2$
Haunch flange	$= 229 \times 19.6$	$= 4488 \text{ mm}^2$
		$\Sigma = 21892 \text{ mm}^2$



By inspection of the areas calculated, the plastic neutral axis lies within the middle flange.

If the position of the plastic neutral axis is x mm down from the top of the middle flange, then

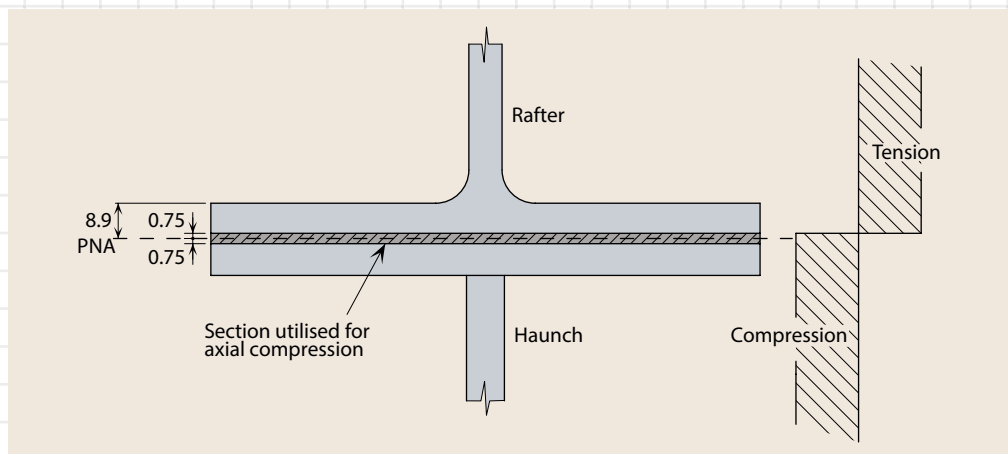
$$3654 + 5420 + 210x = 4488 + 2 \times 2338 + (17.4 - x) \times 210$$

then $x = 8.9$ mm

The plastic neutral axis is 528.2 mm from the top of the compound section.

Part of the cross section, distributed equally about the plastic neutral axis, is utilised in carrying the axial compression (108 kN at this section).

$$\text{Total depth of section carrying compression} = \frac{108 \times 10^3}{345 \times 210} = 1.5 \text{ mm or } 0.75 \text{ mm each side of the PNA.}$$



Bending resistance

The moment resistance of the effective section is the summation of the various areas multiplied by the lever arm from the plastic neutral axis, multiplied by the design strength. The areas allocated to carrying the axial compression are excluded from the calculation of moment resistance. This is illustrated in Table D.7.

	AREA (mm ²)	LEVER ARM (mm)	RESISTANCE (kNm)
Top flange	3654	520	655
Rafter web	5420	260	486
Top of middle flange	1871	4.5	2.8
Bottom of middle flange	1782	4.2	2.6
Upper 20t _ε	2338	107	86
Lower 20t _ε	2338	436	351
Bottom flange	4488	544	842
	Σ = 21892		
Area for axial compression (above PNA)	0.75 × 210	0.5 × 0.75	-0.02
Area for axial compression (below PNA)	0.75 × 210	0.5 × 0.75	-0.02
			Σ = 2425 kNm

Table D.7
Calculation of
moment resistance

Thus $M_{c,Rd} = 2425$ kNm

$M_{Ed} = 1016$ kNm

1016 kNm < 2425 kNm, OK

Table D.8 summarises the classification and bending resistance at each cross section.

CROSS SECTION NO.	CLASSIFICATION	MODULUS	BENDING RESISTANCE (kNm)
1	Not Class 2	Effective plastic	2426
2	Not Class 2	Effective plastic	2104
3	At least Class 2	Plastic	1706
4	At least Class 2	Plastic	1383
5*	Class 1	Rafter alone	900

Table D.8
Cross section
classification and
bending resistance

* See Sections 9.1.2 and 9.2.2 for classification and bending resistance of rafter.

Shear resistance

The shear area of cross section 1 can be conservatively taken as:

$$A_v = h_w t_{w,\min}$$

where h_w is taken as the depth between the top and bottom flanges of the compound section.

$$A_v = (501.9 + 17.4 + 525.5) \times 10.8 = 11284 \text{ mm}^2$$

BS EN 1993-1-1
§6.2.6

$$V_{pl,Rd} = \frac{A_v (f_y / \sqrt{3})}{\gamma_{M0}} = \frac{11284 \times (345 / \sqrt{3})}{1.0} \times 10^{-3} = 2248 \text{ kN}$$

$$V_{Ed} = 231 \text{ kN}, < 2248 \text{ kN, OK}$$

Bending and shear interaction

BS EN 1993-1-1
§6.2.8

When shear force and bending moment act simultaneously on a cross section, the effect of the shear force can be ignored if it is less than 50% of the plastic shear resistance.

$$0.5V_{pl,Rd} = 0.5 \times 2248 = 1124 \text{ kN}$$

$$V_{Ed} = 231 \text{ kN}, < 1124 \text{ kN, OK}$$

Therefore the effect of the shear force on the moment resistance may be neglected.

The same calculation must be carried out for the remaining cross sections. Table D.9 summarizes the shear resistance verification for the haunched member:

Table D.9
Shear verification
for cross sections
1 to 5

CROSS SECTION NO.	V_{Ed} (kN)	A_v (mm ²)	$V_{pl,Rd}$ (kN)	$V_{Ed} \leq V_{Rd}$	$0.5V_{Rd}$ (kN)	BENDING AND SHEAR INTERACTION
1	231	11284	2248	Yes	1124	No
2	220	9812	1954	Yes	977	No
3	209	8340	1661	Yes	831	No
4	198	6869	1368	Yes	684	No
5	187	5397	1075	Yes	537	No

Compression resistance

The compression resistance of cross section 1, using the effective area calculated above, (see Table D.7) is given by:

BS EN 1993-1-1
§6.2.4

$$N_{c,Rd} = \frac{A f_y}{\gamma_{M0}} = \frac{21892 \times 345}{1.0} \times 10^{-3} = 7553 \text{ kN}$$

$$N_{Ed} = 108 \text{ kN}, < 7553 \text{ kN, OK}$$

Bending and axial force interaction

The following conservative criterion may be used

BS EN 1993-1-1
§6.2.1(7)

$$\frac{N_{Ed}}{N_{Rd}} + \frac{M_{Ed}}{M_{Rd}} \leq 1$$

For cross section 1:

$$\frac{N_{Ed}}{N_{Rd}} + \frac{M_{Ed}}{M_{Rd}} = \frac{108}{7553} + \frac{1016}{2425} = 0.43 < 1 \quad \text{OK}$$

A similar calculation must be carried out for the remaining cross sections. Table D.10 summarizes the verification for the haunched member at each cross section.

CROSS SECTION NO.	N_{Ed} (kN)	$N_{c,Rd}$ (kN)	M_{Ed} (kN)	$M_{c,Rd}$ (kN)	$\frac{N_{Ed}}{N_{Rd}} + \frac{M_{Ed}}{M_{Rd}}$	
1	108	7553	1016	2425	0.43	OK
2	107	7552	867	2104	0.43	OK
3	106	6979	724	1706	0.44	OK
4	105	6419	589	1383	0.44	OK
5	103	4451	462	900	0.54	OK

Table D.10
Utilization
verification for cross
sections 1 to 5

Because neither shear nor compression reduces the bending resistance of the cross section, bending resistances remain as shown in Table D.8.

11.2 Buckling resistance

Section 7.4 of
this publication

The elastic verification of a haunched member is not covered explicitly in the Eurocode. Two alternative approaches are illustrated in this example. Firstly, in Section 11.2.1, the stability of the haunch is verified by considering an equivalent compression flange, following the principle illustrated in Clause 6.3.2.4 of BS EN 1993-1-1. Secondly, in Section 11.2.2, the haunch stability is verified using a check appropriate for a haunched length containing a plastic hinge, recognising that an elastic situation must therefore be adequate.

11.2.1 Simplified assessment method

In this approach, an equivalent compression flange is verified, composed of the compression flange plus 1/3 the compressed part of the web. The resistance of this Tee-shaped equivalent flange is compared to the force in the Tee arising from the axial compression and the bending moment.

There are restraints to the compression flange adjacent to the column, and at the sharp end of the haunch, as shown in Figure D.22.

The dimensions of the equivalent compression flange are determined at a point 1/3 of the haunch length from the deepest section (adjacent to the column), as shown in Figure D.23.

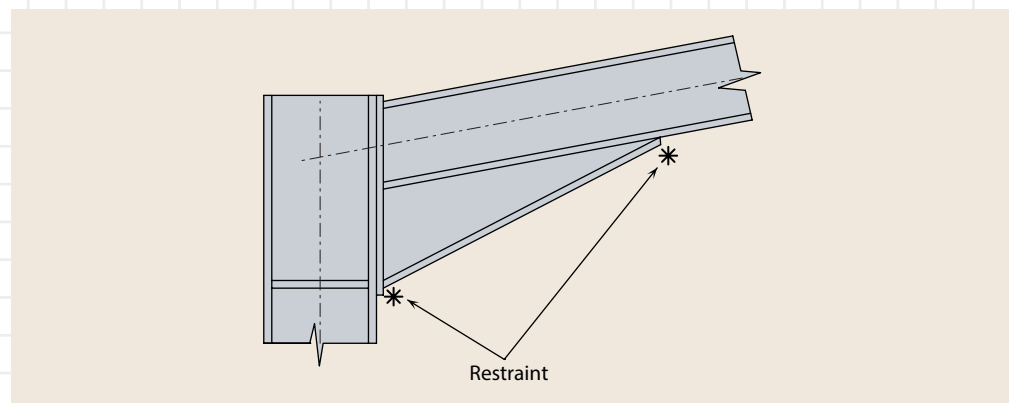
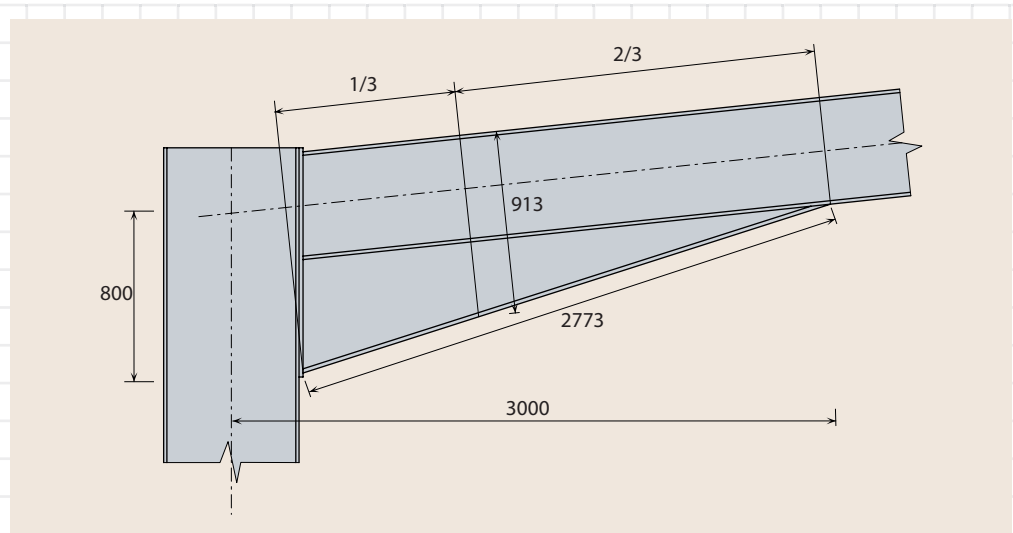


Figure D.22
Restraints to the
compression flange
of the haunch

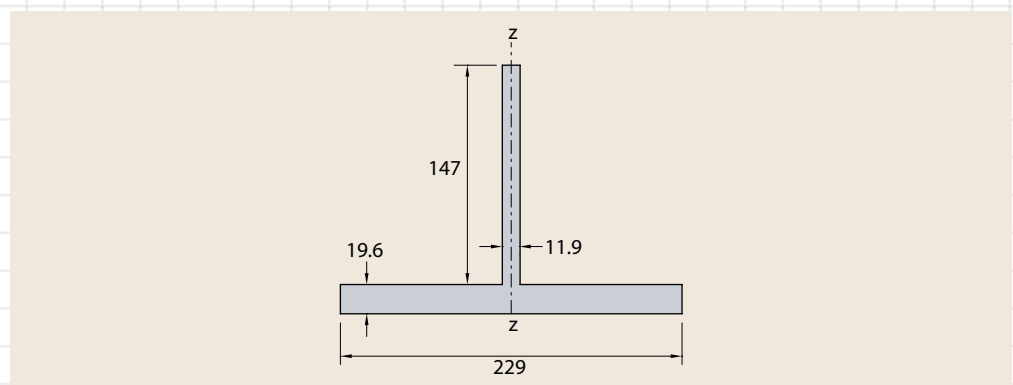
Figure D.23
Cross section
used to determine
equivalent
compression flange
dimensions



At the cross section indicated in Figure D.23, the overall depth is 913 mm. The depth of the web between flanges is therefore $913 - 17.4 - 16.2 = 879$ mm.

Assuming that half the web is in compression, $1/3$ of the compressed part of the web $= 879/6 = 147$ mm. The dimensions of the equivalent compression flange, ignoring the root radius, are shown in Figure D.24.

Figure D.24
Dimensions
of equivalent
compression flange



$$A = 11.9 \times 147 + 229 \times 19.6 = 6238 \text{ mm}^2$$

$$I_z = \frac{147 \times 11.9^3}{12} + \frac{19.6 \times 229^3}{12} = 19.6 \times 10^6 \text{ mm}^4$$

$$i_z = \sqrt{\frac{19.6 \times 10^6}{6238}} = 56 \text{ mm}$$

To calculate the flexural buckling resistance,

BS EN 1993-1-1 §6.3.1.3 $\bar{\lambda} = \frac{L_{cr}}{i} \frac{1}{\lambda_1}$, where, as before, $\lambda_1 = 77.5$

$$\text{Then } \bar{\lambda} = \frac{2773}{56} \frac{1}{77.5} = 0.64$$

BS EN 1993-1-1
§6.3.1.2

Curve **c** is to be used, with $\alpha = 0.49$

BS EN 1993-1-1
§6.3.1.2

$$\phi_y = 0.5 [1 + \alpha (\bar{\lambda} - 0.2) + \bar{\lambda}^2]$$

$$= 0.5 \times [1 + 0.49 \times (0.64 - 0.2) + 0.64^2] = 0.81$$

$$\chi_y = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}^2}} = \frac{1}{0.81 + \sqrt{0.81^2 - 0.64^2}} = 0.77$$

$$N_{b,Rd} = \chi \frac{Af_y}{\gamma_{M1}} = 0.77 \times \frac{6238 \times 345}{1.0} \times 10^{-3} = 1657 \text{ kN}$$

The maximum compressive stress and bending stress on the gross cross section can be determined from Table D.6.

$$\text{The maximum compressive stress} = \frac{103 \times 10^3}{12900} = 8 \text{ N/mm}^2$$

$$\text{The maximum bending stress is at cross section 5} = \frac{462 \times 10^6}{2290 \times 10^3} = 202 \text{ N/mm}^2$$

$$\text{The force in the equivalent compression flange (assuming uniform, maximum stress)} \\ = 6238 \times (8 + 202) \times 10^{-3} = 1310 \text{ kN}$$

1310 kN < 1657 kN, so the haunch is stable between restraints to the compression flange.

11.2.2 Verification using a plastic check

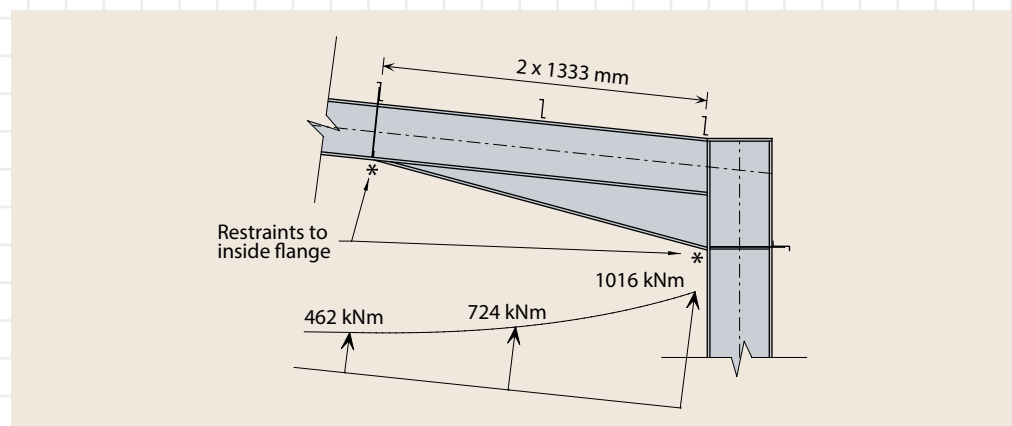
There is a torsional restraint at each end of the haunched length, as shown in Figure D.25. There is one intermediate purlin.

BS EN 1993-1-1
§BB.3.2.2

Lateral torsional buckling effects may be ignored when the distance between torsional restraints is not greater than L_s if there is at least one intermediate lateral restraint between the torsional restraints, at a spacing not greater than L_m .

Although this check is primarily intended for lengths adjacent to a plastic hinge, it is used for convenience in this example. Because there is no plastic hinge, if the haunch satisfies this check, it will be adequate.

Figure D.25
Initial purlin positions and design bending moments along the haunch



BS EN 1993-1-1
§BB.3.2.1

$$L_m = \frac{38i_z}{\sqrt{\frac{1}{57.4} \left(\frac{N_{Ed}}{A} \right) + \frac{1}{756C_1^2} \frac{W_{ply}^2}{AI_T} \left(\frac{f_y}{235} \right)^2}}$$

For simplicity, the purlin at mid point of the haunched member is assumed to be aligned with cross section 3 (see Figure D.20).

Similarly, the purlin at the end of the haunched member is assumed to be aligned with cross section 1.

Between cross sections 1 and 3:

Appendix B of
this publication

$$\psi = \frac{724}{1016} = 0.71 \quad \therefore C_1 = 1.20$$

Between cross sections 3 and 5:

Appendix B of
this publication

$$\psi = \frac{462}{724} = 0.64 \quad \therefore C_1 = 1.29$$

The value leading to the most onerous length L_m will be used, i.e. $C_1 = 1.20$

BS EN 1993-1-1
§BB.3.2.1

According to the Eurocode, the ratio $\frac{W_{pl}^2}{AI_T}$ should be taken as the maximum value in the segment.

In this case, cross sections 1 and 3 have been considered, as shown in Table D.11.

$\frac{W_{pl}^2}{AI_T}$ ratio for cross
sections 1,
3 and 5

CROSS SECTION NO.	A (mm ²)	I _T (mm ⁴) × 10 ⁴	W _{pl} (mm ³) × 10 ³	$\frac{W_{pl}^2}{AI_T}$
1	23500	188	7544	1289
3	20200	173	4994	713
5	12900	101	2610	506

The section properties of cross section 1 give the maximum ratio $\frac{W_{pl}^2}{AI_T}$. Therefore L_m is calculated using the section properties of cross section 1.

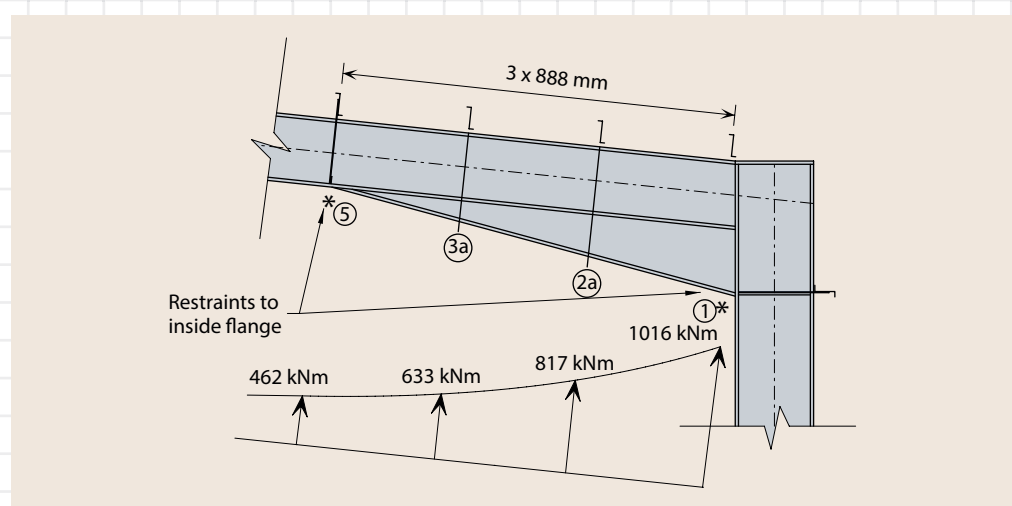
$$I_z = 4660 \times 10^4 \text{ mm}^4$$

$$i_z = \sqrt{\frac{I_z}{A}} = \sqrt{\frac{4660 \times 10^4}{23500}} = 44.5 \text{ mm}$$

$$L_m = \frac{38 \times 44.5}{\sqrt{\frac{1}{57.4} \left(\frac{108 \times 10^3}{23500} \right) + \frac{1}{756 \times 1.2^2} \times \frac{(7544 \times 10^3)^2}{23500 \times 188 \times 10^4} \left(\frac{345}{235} \right)^2}}$$

$$L_m = 1042 \text{ mm}$$

Figure D.26
Revised purlin
positions and
design bending
moments along
the haunch



Purlin spacing is 1333 mm, > 1042 mm, OK

Therefore, the lateral restraints need to be at smaller spacing in order to follow the Annex BB method.

An additional lateral restraint is introduced, so that the layout is as shown in Figure D.26.

Geometric data for the compound sections aligning with the revised purlin location required for further calculation is shown in Table D.12.

Table D.12
Geometric data for
sections at revised
purlin locations

CROSS SECTION NO.	CUTTING DEPTH (mm)	GROSS AREA, A (mm ²)	I_z (cm ⁴) × 10 ⁴	i_z (cm)	PLASTIC* NEUTRAL AXIS (mm)	N_{Ed} (kN)	M_{Ed} (kNm)
1	545	23500	4660	44.5	549.9	108	1016
2a	363	21300	4660	46.7	373.3	106	817
3a	182	19200	4660	49.3	196.8	105	633
5	0	12900	2690	45.7	268.4	103	462

* From underside of section.

The limiting spacing between intermediate restraints is now given by:

BS EN 1993-1-1
§BB.3.2.1

$$L_m = \frac{38i_z}{\sqrt{\frac{1}{57.4} \left(\frac{N_{Ed}}{A} \right) + \frac{1}{756C_1^2} \frac{W_{ply}^2}{AI_T} \left(\frac{f_y}{235} \right)^2}}$$

Appendix B of
this publication

To determine the lowest value of C_1 , each segment is considered:

$$\psi = \frac{817}{1016} = 0.80 \quad \therefore C_1 = 1.13$$

$$\psi = \frac{633}{817} = 0.78 \quad \therefore C_1 = 1.15$$

$$\psi = \frac{462}{633} = 0.73 \quad \therefore C_1 = 1.19$$

The value leading to the most onerous length L_m will be used, i.e. $C_1 = 1.13$

BS EN 1993-1-1
§BB.3.2.1

According to the Eurocode, the ratio $\frac{W_{pl}^2}{AI_T}$ should be taken as the maximum value in the segment.

In this case cross sections 1, 2a, 3a and 5 have been considered, as shown in Table D.13.

$\frac{W_{pl}^2}{AI_T}$ Table D.13
ratio for cross
sections 1,
2a, 3a and 5

CROSS SECTION NO.	A (mm ²)	I _t (mm ⁴) × 10 ⁴	W _{pl} (mm ³) × 10 ³	$\frac{W_{pl}^2}{AI_T}$
1	23500	188	7544	1289
2a	21329	178	5751	872
3a	19166	168	4329	583
5	12900	101	2610	506

The section properties of cross section 1 give the maximum ratio $\frac{W_{pl}^2}{AI_T}$. Therefore L_m is calculated using the section properties of cross section 1.

$$I_z = 4660 \times 10^4 \text{ mm}^4$$

$$i_z = \sqrt{\frac{I_z}{A}} = \sqrt{\frac{4660 \times 10^4}{23500}} = 44.5 \text{ mm}$$

$$L_m = \frac{38 \times 44.5}{\sqrt{\frac{1}{57.4} \left(\frac{108 \times 10^3}{23500} \right)^2 + \frac{1}{756 \times 1.13^2} \times \frac{(7544 \times 10^3)^2}{23500 \times 188 \times 10^4} \left(\frac{345}{235} \right)^2}}$$

$$L_m = 984 \text{ mm}$$

Purlin spacing is 888 mm, < 984 mm, OK

BS EN 1993-1-1
§BB.3.2.2

Therefore L_s may be calculated, having satisfied the requirement for L_m

$$\text{Then } L_s = \frac{\sqrt{C_n} L_k}{c}$$

BS EN 1993-1-1
(BB.14)

$$\text{where } C_n = \frac{12R_{\max}}{[R_1 + 3R_2 + 4R_3 + 3R_4 + R_5 + 2(R_s - R_E)]}$$

where

$$(BB.15) \quad R = \frac{M_{y,Ed} + aN_{Ed}}{f_y W_{pl,y}}$$

BS EN 1993-1-1
§BB.3.1.2

$$L_k = \frac{\left(5.4 + \frac{600f_y}{E} \right) \left(\frac{h}{t_f} \right) i_z}{\sqrt{5.4 \left(\frac{f_y}{E} \right) \left(\frac{h}{t_f} \right)^2 - 1}}$$

BS EN 1993-1-1
(BB.17)
See Section C.2.1

$$c = 1 + \frac{3}{\left(\frac{h}{t_f} - 9\right)} \left(\frac{h_h}{h_s}\right)^{2/3} \sqrt{\frac{L_h}{L_y}}$$

The following table gives the values of R at the ends, quarter points and mid-length, (which correspond to cross sections 1, 2, 3, 4 and 5 shown in Figure D.21). Dimension a is the distance between the centroid of the purlin and the centroid of the compound section.

CROSS SECTION NO. (SEE FIGURE D.21)	DIMENSION a (mm)	N_{Ed} (kN)	M_{Ed} (kNm)	$M_y + aN_{Ed}$ (kNm)	$f_y W_{ply}$ (kNm)	R VALUE
5	632	108	1016	1084	2603	0.417
4	628	107	867	934	2127	0.439
3	624	106	724	790	1723	0.459
2	620	105	589	654	1391	0.470
1	368	103	462	500	886	0.564

Table D.14
 R values for
haunch segment

$$R_E = \max(R_1, R_5) = 0.564$$

$$R_S = \max(R_1, R_2, R_3, R_4, R_5) = 0.564$$

$$R_{max} = \max(R_1, R_2, R_3, R_4, R_5) = 0.564$$

$$C_n = \frac{12 \times 0.564}{\left[0.417 + 3 \times 0.439 + 4 \times 0.459 + 3 \times 0.470 + 0.564 + 2(0.564 - 0.564)\right]}$$

$$C_n = 1.221 \text{ (Conservatively, } C_n \text{ may be taken as 1.0)}$$

$$c = 1 + \frac{3}{\left(\frac{536.7}{17.4} - 9\right)} \left(\frac{545.1}{536.7}\right)^{2/3} \sqrt{\frac{2647}{2647}}$$

$$c = 1.14$$

$$L_k = \frac{\left(5.4 + \frac{600 \times 345}{210000}\right) \left(\frac{536.7}{17.4}\right) 45.7}{\sqrt{5.4 \left(\frac{345}{210000}\right) \left(\frac{536.7}{17.4}\right)^2 - 1}}$$

$$L_k = 3300 \text{ mm}$$

Therefore

$$L_s = \frac{\sqrt{1.221} \times 3300}{1.14}$$

$$L_s = 3199 \text{ mm}$$

Spacing between torsional restraints at each end of the haunch is 2648 mm (see Figure D.20).

$$2648 \text{ mm} < 3199 \text{ mm, OK}$$

The haunch has been demonstrated to be adequate in terms of strength and buckling stability.

12 Deflections

The horizontal deflections of the portal frame subject to the characteristic load values of actions are shown in Table D.15. In calculating these deflections, the analysis has allowed for the nominally pinned bases having a stiffness equal to 20% of that of the column.

Table D.15
Horizontal displacement of column top under characteristic values of actions

ACTION	δ (mm)
snow	6.7
imposed	10.1
wind – case A	14.6
wind – case B	0.8

The deflection requirements for a given portal frame design must be agreed with the client.

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ELASTIC DESIGN OF SINGLE-SPAN STEEL PORTAL FRAME BUILDINGS TO EUROCODE 3

Steel portal frames are firmly established in the UK as a lightweight, highly efficient and cost-effective way of enclosing usable volumes, accounting for the majority of the single storey market. This publication provides guidance on the design of portal frames in accordance with the Eurocodes, making recommendations where the design Standard is unclear. The publication covers single span, symmetric frames designed elastically and provides a comprehensive introduction to the design and detailing of portals. A fully worked example demonstrates the application of the Eurocodes covering actions, frame stability and member verification.

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