STEEL BUILDINGS IN EUROPE

Single-Storey Steel Buildings
Part 2: Concept Design

2.4 Airbus Industrie hanger, Toulouse, France

The Airbus production hall in Toulouse covers 200000 m^2 of floor space and is 45 m high with a span of 117 m. It consists of 8 m deep lattice trusses composed of H sections. Compound column sections provide stability to the roof structure. The building is shown in Figure 2.5 during construction. Sliding doors create a 117 m × 32 m opening in the end of the building. Two parallel rolling cranes are installed each of 50 m span and 20 tonnes lifting capacity.

Figure 2.5 View of Airbus Industrie hanger during construction

2.5 Industrial hall, Krimpen aan den Ijssel, Netherlands

This production hall is 85 m in length, 40 m wide and 24 m high with full height doors at the end of the building, as shown in Figure 2.6. The roof structure consists of an inclined truss. Because of the lack of bracing in the end walls, the structure was designed to be stabilised through the columns assisted by in-plane bracing in the roof and side walls.

Figure 2.6 View of doors being lifted into place in Hollandia's building in Krimpen aan den Ijssel

2.6 Distribution Centre and office, Barendrecht, Netherlands

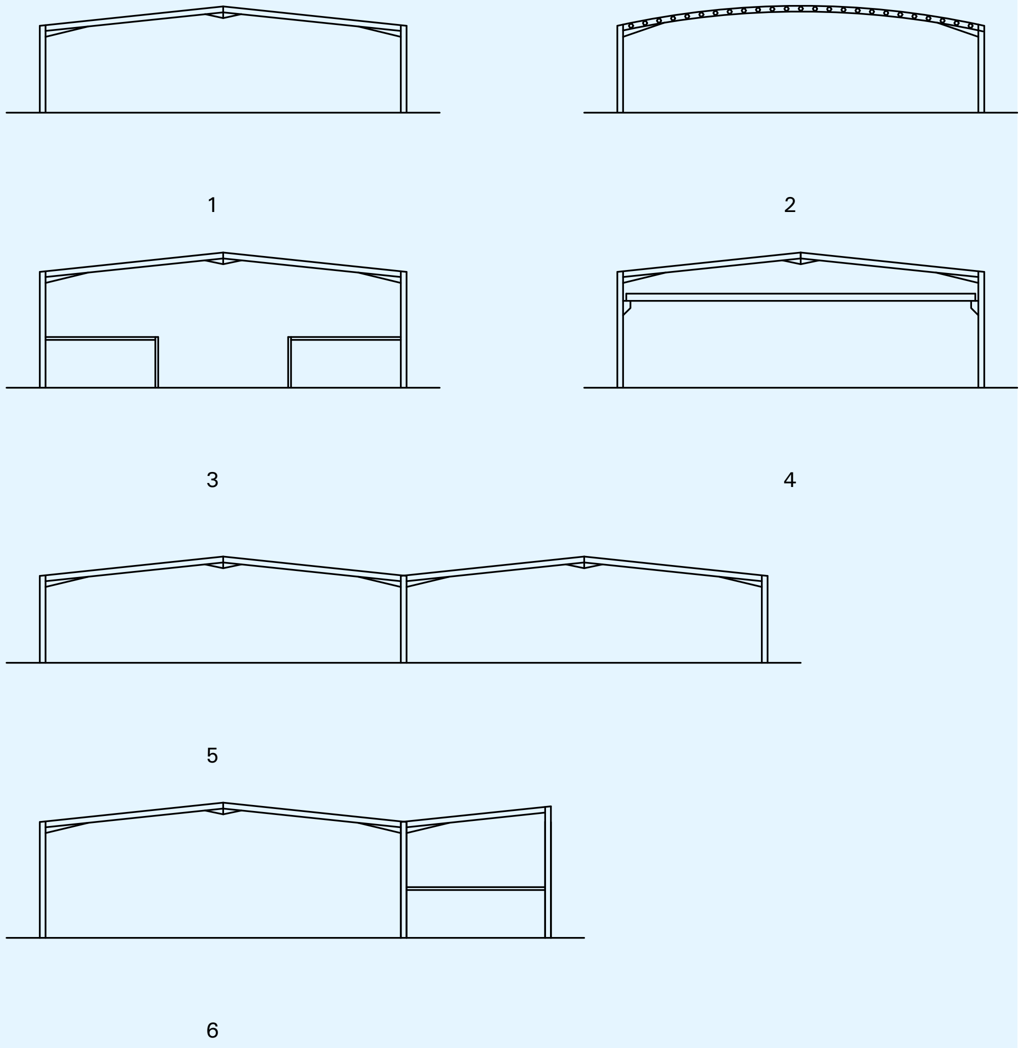
This 26000 m^2 distribution centre for a major supermarket in the Netherlands comprises a conventional steel structure for the distribution area and a two storey high office area that is suspended above an access road, as shown in Figure 2.7. This 42 m long office building comprises a 12 m cantilever supported by a two storey high internal steel structure with diagonal bracing. The structure uses H section beams and columns with tubular bracing.

Both the warehouse and office buildings are provided with sprinklers to reduce the risk of fire, and the steelwork has intumescent coating so that it can be exposed internally. The warehouse internal temperature is 2°C and the steelwork of the office is thermally isolated from the warehouse part.

Figure 2.7 Distribution centre, Barendrecht, NL showing the braced cantilever office structure

3 CONCEPT DESIGN OF PORTAL FRAMES

Steel portal frames are widely used because they combine structural efficiency with functional form. Various configurations of portal frame can be designed using the same structural concept as shown in Figure 3.1.



1 Duo-pitch portal frame
2 Curved portal frame (cellular beam)
3 Portal with internal offices
4 Portal with crane
5 Two-span portal frame
6 Portal with external offices

Figure 3.1 Various types of portal frame

3.1 Pitched roof portal frame

A single-span symmetrical portal frame (as illustrated in Figure 3.2) is typically of the following proportions:

- A span between 15 m and 50 m (25 m to 35 m is the most efficient)
- An eaves height (base to rafter centreline) of between 5 and 10 m (7,5 m is commonly adopted). The eaves height is determined by the specified clear height between the top of the floor and the underside of the haunch.
- A roof pitch between 5° and 10° (6° is commonly adopted)
- A frame spacing between 5 m and 8 m (the greater frame spacings being used in longer span portal frames)
- Members are I sections rather than H sections, because they must carry significant bending moments and provide in-plane stiffness.
- Sections are generally S235 or S275. Because deflections may be critical, the use of higher strength steel is rarely justified.
- Haunches are provided in the rafters at the eaves to enhance the bending resistance of the rafter and to facilitate a bolted connection to the column.
- Small haunches are provided at the apex, to facilitate the bolted connection

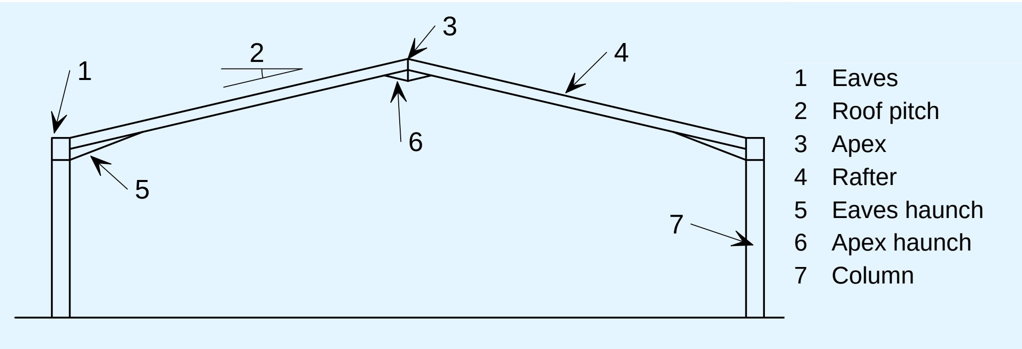


Figure 3.2 Single-span symmetric portal frame

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 span. The length of the haunch means that 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 3.3.

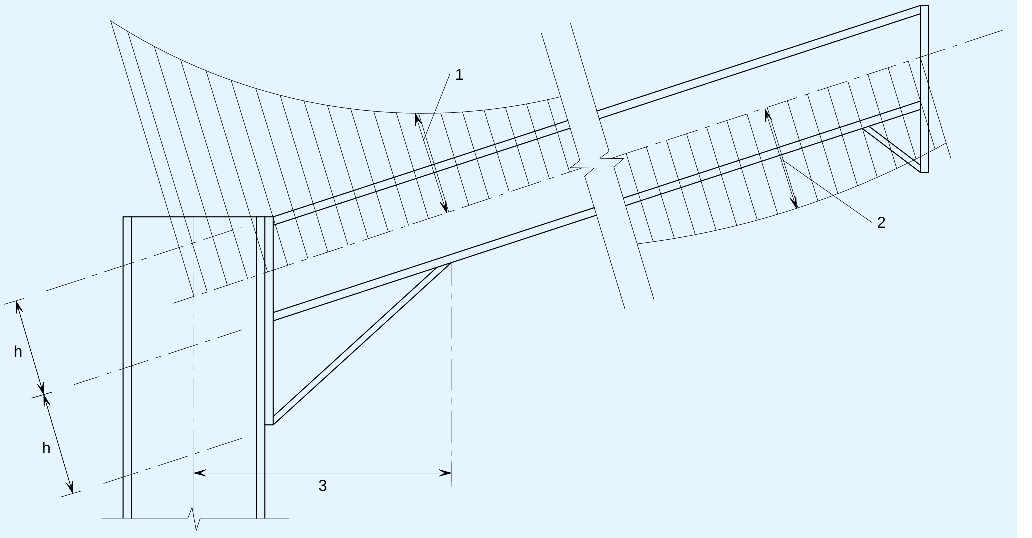


Figure 3.3 Rafter bending moment and haunch length

The final frames of a portal frame are generally called gable frames. Gable frames may be identical to the internal 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. A typical gable frame is shown in Figure 3.4.

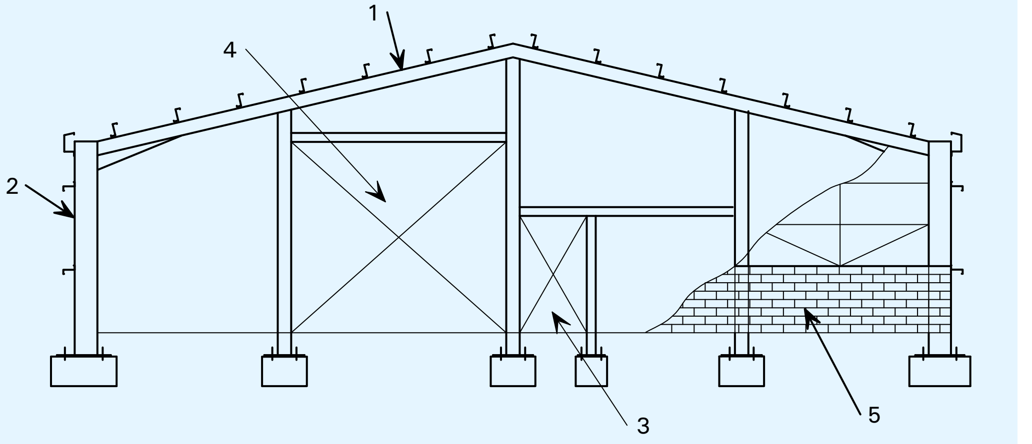


Figure 3.4 Typical details of an end gable of a portal frame building

Alternatively, gable frames can be constructed from columns and short rafters, simply supported between the columns as shown in Figure 3.5. In this case, gable bracing is required, as shown in the figure.

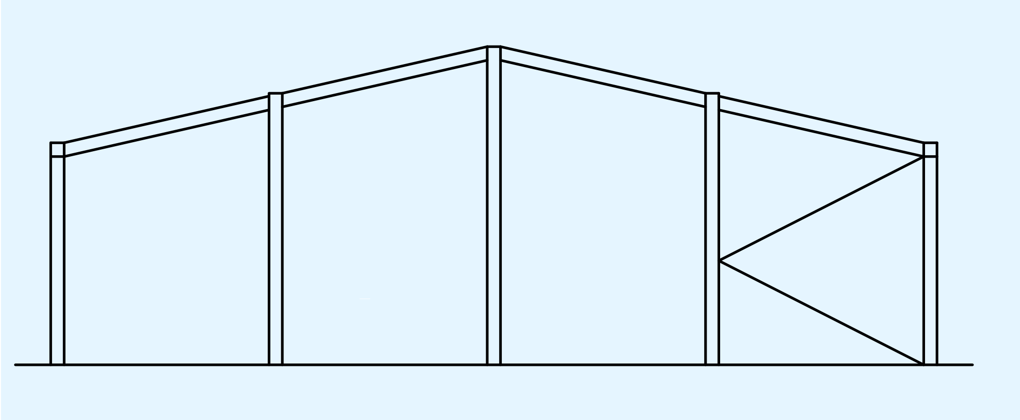
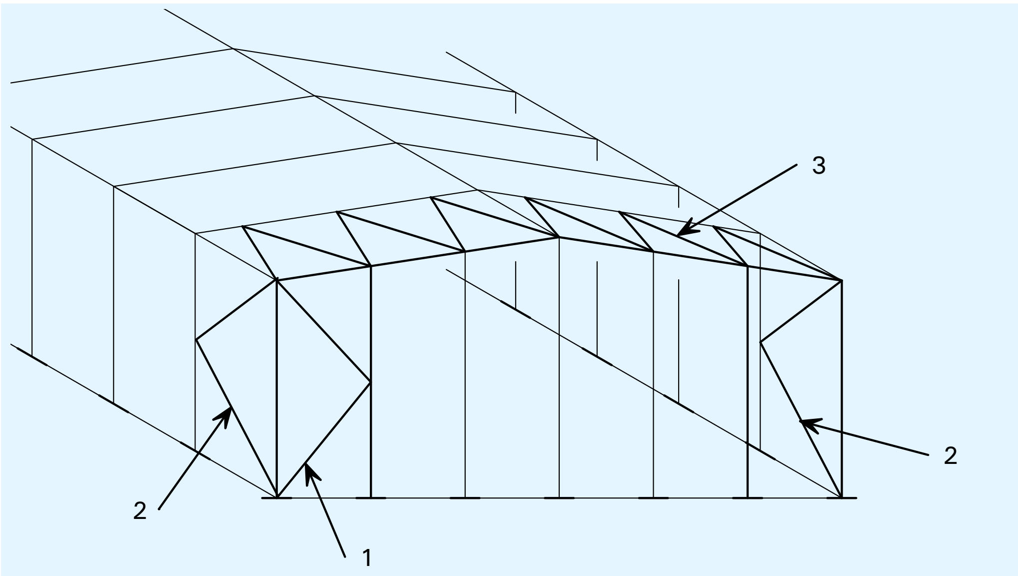


Figure 3.5 Gable frame (not a portal frame)

3.2 Frame stability

In-plane stability is provided by frame continuity. In the longitudinal direction, stability is provided by vertical bracing in the elevations. The vertical bracing may be at both ends of the building, or in one bay only. Each frame is connected to the vertical bracing by a hot-rolled member at eaves level. A typical bracing arrangement is shown in Figure 3.6.

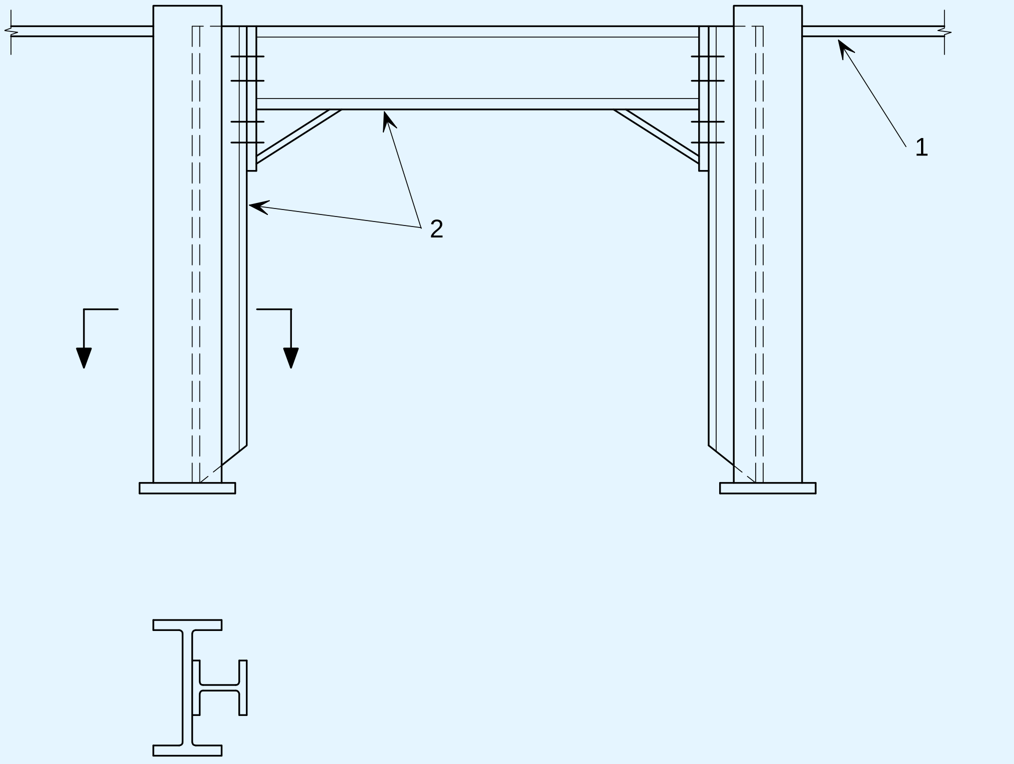


1 Vertical bracing in the gable
2 Vertical bracing in the walls
3 Roof bracing

Figure 3.6 Typical bracing in a portal frame

The gable columns span between the base and the rafter, where the reaction is carried by bracing in the plane of the roof, back to the eaves level, and to the foundations by the vertical bracing.

If diagonal bracing in the elevations cannot be accommodated, longitudinal stability can be provided by a rigid frame on the elevation, as shown in Figure 3.7.



1 Eaves strut
2 Rigid frame

Figure 3.7 Rigid frame alternative to vertical bracing

3.3 Member stability

Member stability should be checked using expressions 6.61 and 6.62 of EN 1993-1-1. For economic design, restraints to the rafter and column must be considered. The purlins and side rails are considered adequate to restrain the flange that they are attached to, but unless special measures are taken, the purlins and side rails do not restrain the inside flange. Restraint to the inside flange is commonly provided by bracing from the purlins and side rails, as shown in Figure 3.8. The bracing is usually formed of thin metal straps, designed to act in tension, or from angles designed in compression if bracing is only possible from one side.

If the bracing shown in Figure 3.8 is not permitted by national regulations, restraint may be provided by a system of hot-rolled members.

This form of bracing will be required whenever the inside flange is in compression. This situation arises:

- On the inside of the column and the inside of the rafter in the haunch region, in the gravity load combination
- Towards the apex of the rafter, in the uplift combination.

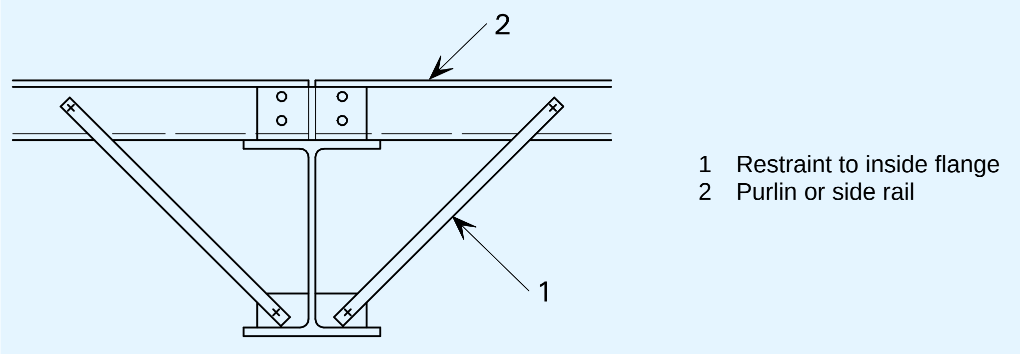


Figure 3.8 Typical bracing to the inside flange

The arrangement of restraints to the inside flange is generally similar to that shown in Figure 3.9. In some instances, it may not be possible to restrain the inside of the column flange. In these circumstances, a larger column section may have to be chosen, which is stable between the underside of the haunch and the base.

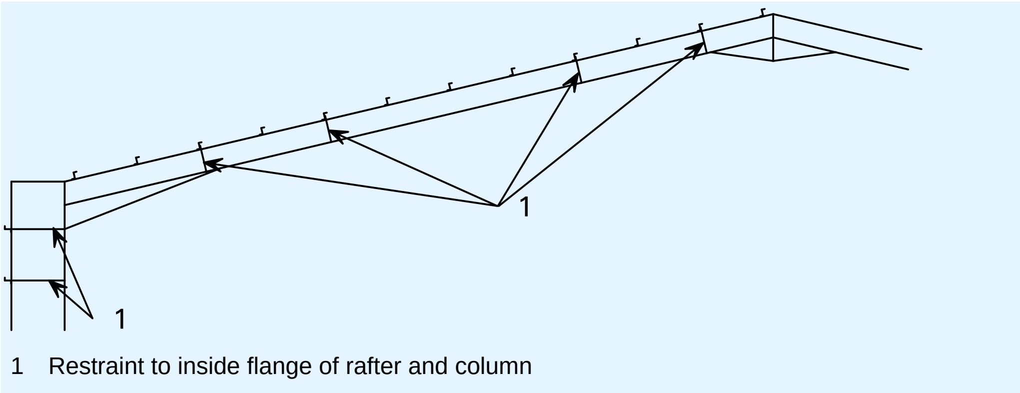


Figure 3.9 General arrangement of restraints to the inside flange

In all cases, the junction of the inside face of the column and the underside of the haunch, as shown in Figure 3.10, must be restrained. The restraint may be of the form shown in Figure 3.8, or may be by a hot-rolled member provided for that purpose.

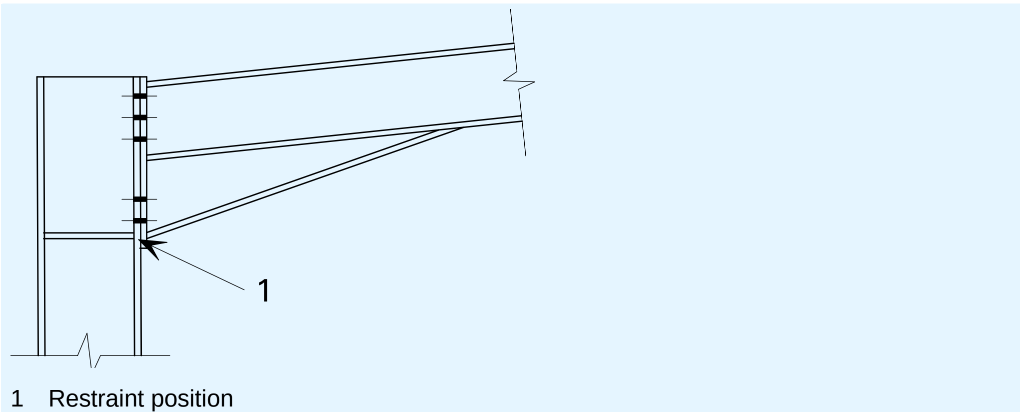


Figure 3.10 Restraint at the haunch / column junction

3.4 Preliminary Design

3.4.1 Main frames

Although efficient portal frame analysis and design will use bespoke software, preliminary design is simple. In most circumstances, a reasonable estimate of the maximum bending moments will be obtained by considering only the vertical loads. Combinations of actions including wind actions must be validated in the final design, and may be important for preliminary design if the wind actions are onerous (e.g. near the sea, or if the portal frame is tall).

Based on the vertical load alone, charts that provide initial sizes are given in Section 8.

As an alternative to the sizes given in Section 8, the bending moment at the eaves and apex can be calculated based on an elastic analysis.

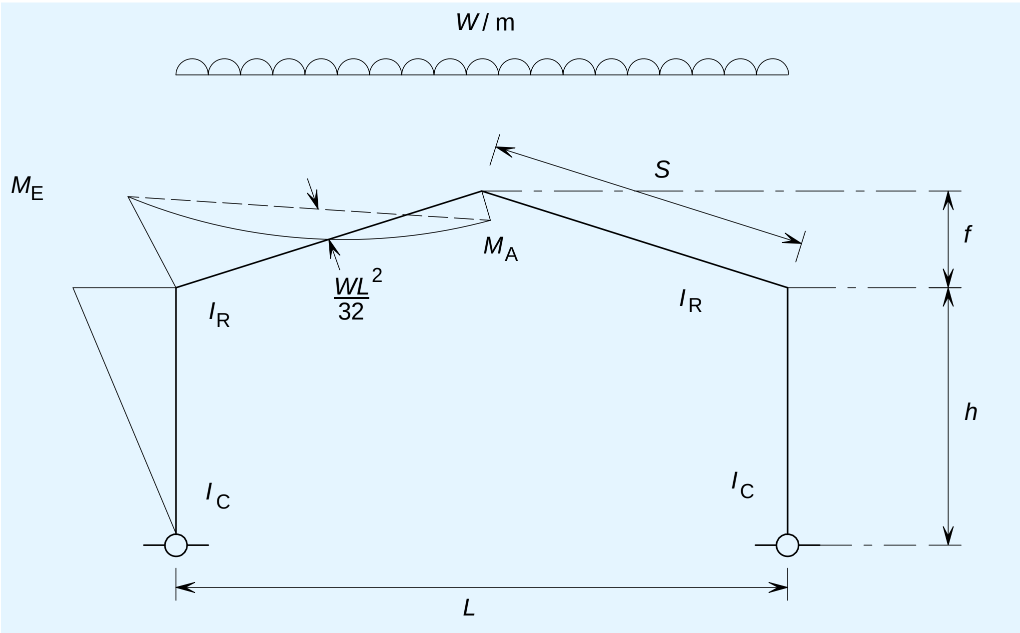


Figure 3.11 Details of a pinned base portal frame

For the pinned base frame shown in Figure 3.11, the bending moment at the eaves, M_E and at the apex M_A can be calculated as follows:

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

where:

$N = B + mC$

$C = 1 + 2m$

$B = 2(k + 1) + m$

$m = 1 + \phi$

$\phi = f/h$

$$k = \frac{I_R}{I_C}\frac{h}{s}$$

It may be assumed for preliminary design that $I_C = 1,5 \times I_R$

Given the bending moments around the frame, the rafter should be chosen so that the moment resistance exceeds both the moment at the “sharp” end of the haunch and the maximum sagging moment (a little larger than the moment at the apex).

3.4.2 Gable columns

Gable columns are generally designed as simply supported from base to rafter. The primary loads are the wind actions. The internal pressure or suction will contribute to the loading on the gable column. Often, the critical design case will be pressure inside the building and suction on the outside, when the inside flange of the gable post is unrestrained. If national regulations allow, a restraint to the inside flange may be provided from a sheeting rail to increase the buckling resistance.

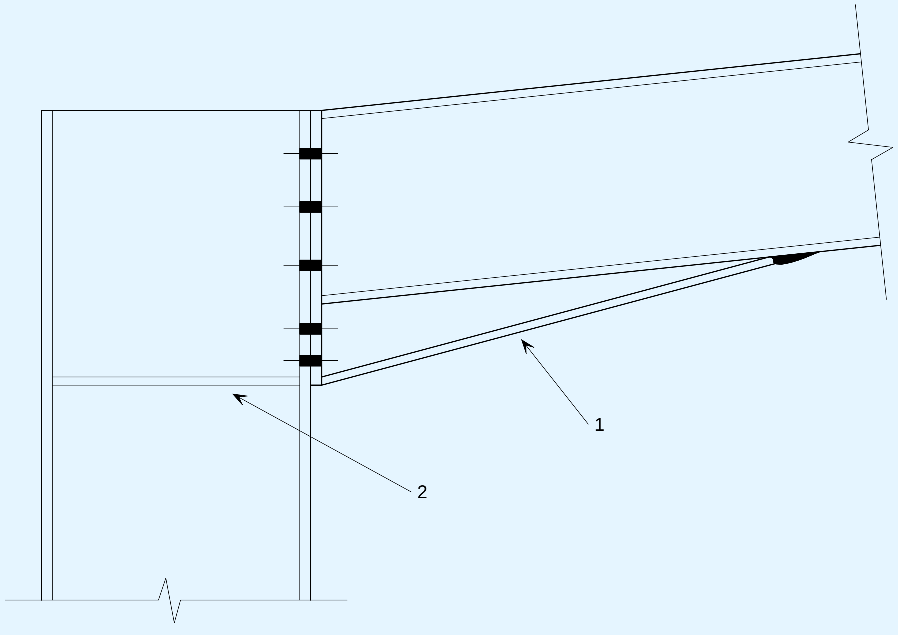
3.4.3 Bracing

At the preliminary design stage, it is convenient to calculate the overall longitudinal load on the structure. This shear must be the horizontal component of the load carried by the vertical bracing. The most heavily loaded roof bracing will be the member nearest the eaves. The longitudinal eaves member carries the load from the roof bracing to the vertical bracing. Bracing members may be hollow sections, angle sections or flat steel. Flat steel is assumed to resist tension forces only.

3.5 Connections

3.5.1 Eaves connection

A typical eaves connection is shown in Figure 3.12. In almost all cases a compression stiffener in the column (as shown, at the bottom of the haunch) will be required. Other stiffeners may be required to increase the bending resistance of the column flange, adjacent to the tension bolts, and to increase the shear resistance of the column web panel. The haunch is generally fabricated from a similar size beam to the rafter (or larger), or fabricated from equivalent plate. Typically, the bolts may be M24 8.8 and the end plate 25 mm thick S275.



1 Haunch
2 Compression stiffener

Figure 3.12 Typical eaves connection

3.5.2 Apex connection

A typical apex connection is shown in Figure 3.13. The apex connection primarily serves to increase the depth of the member to make a satisfactory bolted connection. The apex haunch is usually fabricated from the same member as the rafter, or from equivalent plate. Typically, the bolts may be M24 8.8 and the end plate 25 mm thick S275.

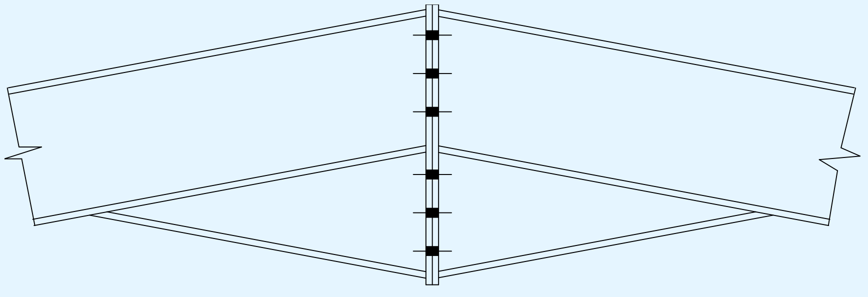


Figure 3.13 Typical apex connection

3.5.3 Bases

A typical pinned base is shown in Figure 3.14. The base plate is generally at least as thick as the flange of the column. Most authorities accept that even with four holding down bolts as shown in Figure 3.14, the base is still pinned. Alternatively, the base may have only two holding down bolts, on the axis of the column, but this may make the erection of the steelwork more difficult.

Columns are normally located on a number of steel packs, to ensure the steelwork is at the correct level, and the gap between the foundation and the steelwork filled with cementicious grout. Large bases should be provided with an air hole to facilitate complete grouting.

Holding down bolts are generally embedded in the foundation, with some freedom of lateral movement (tubes or cones) so that the steelwork can be aligned precisely. The holes in the base plate are usually 6 mm larger than the bolt diameter, to facilitate some lateral alignment.

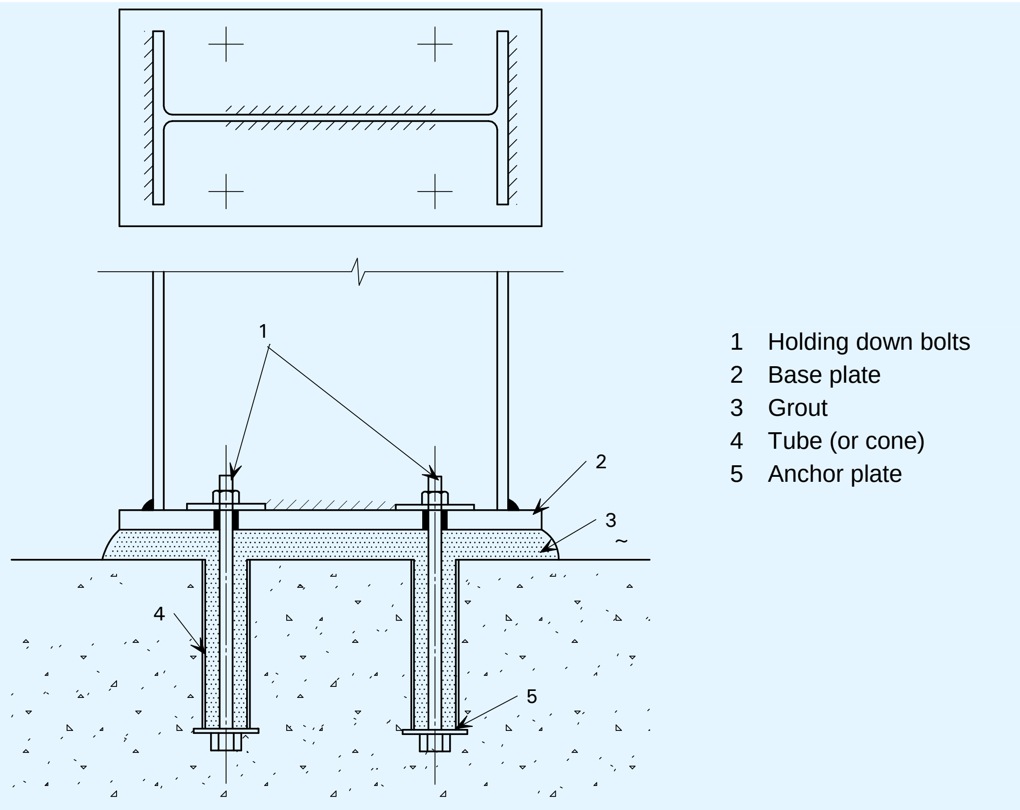


Figure 3.14 Typical portal base detail

3.5.4 Bracing Connections

Forces in portal frame bracing are generally modest. Typical connections are shown in Figure 3.15. Gusset plates should be supported on two edges, if possible.

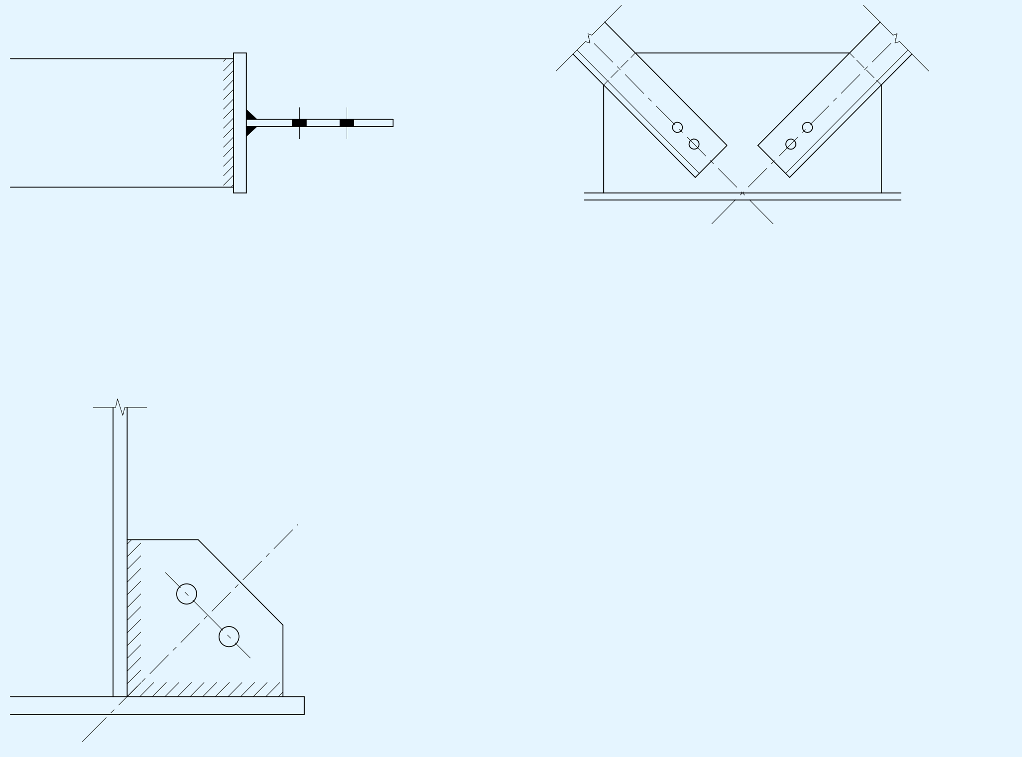
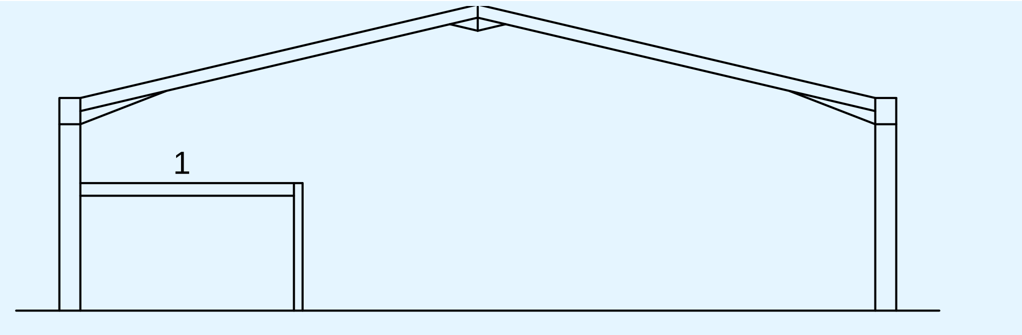


Figure 3.15 Typical bracing connections

3.6 Other types of portal frame

The features of an orthodox portal frame were described in Sections 3.1 to 3.5. The basic structural concept can be modified in a number of ways to produce a cost effective solution, as illustrated below.

3.6.1 Portal frame with a mezzanine floor



1 Mezzanine

Figure 3.16 Portal frame with internal mezzanine floor

Office accommodation is often provided within a portal frame structure using a mezzanine floor (as illustrated in Figure 3.17). The mezzanine floor may be partial or full width. It can be designed to stabilise the frame. Often, the internal floor of the office space requires fire protection.

Figure 3.17 Portal frame with intermediate floor

3.6.2 Portal frame with external mezzanine

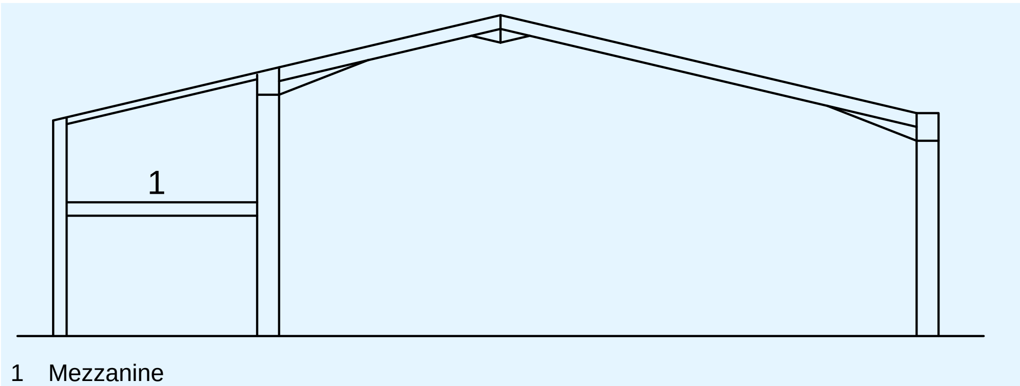


Figure 3.18 Portal frame with external mezzanine

Offices may be located externally to the portal frame which creates an asymmetric portal structure (as illustrated in Figure 3.18). The main advantage of this framework is that large columns and haunches do not obstruct the office space. Generally, this additional structure depends on the portal frame for its stability (the members often have nominally pinned connections to the main frame) and the members can be relatively lightweight.

3.6.3 Portal frame with overhead crane

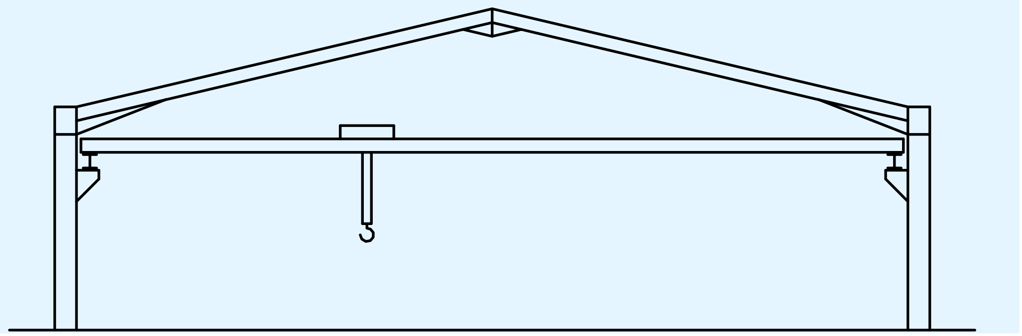


Figure 3.19 Crane portal frame with column brackets

For cranes of relatively low capacity (up to say 20 tonnes), portal frames can be used to support the crane beam and rail, as illustrated in Figure 3.19. The outward movement (spread) of the frame at the level of the crane rail is likely to be of critical importance. Use of a horizontal tie member or fixed column bases may be necessary to reduce this spread.

For larger cranes, a structure with a roof truss will be appropriate (see Section 4) as the column spread is minimised. For very heavy loads, built-up columns are appropriate, as introduced in Section 6. Detail design guides cover both the design of trusses[3] and the design of built-up columns[4].

3.6.4 Tied portal frame

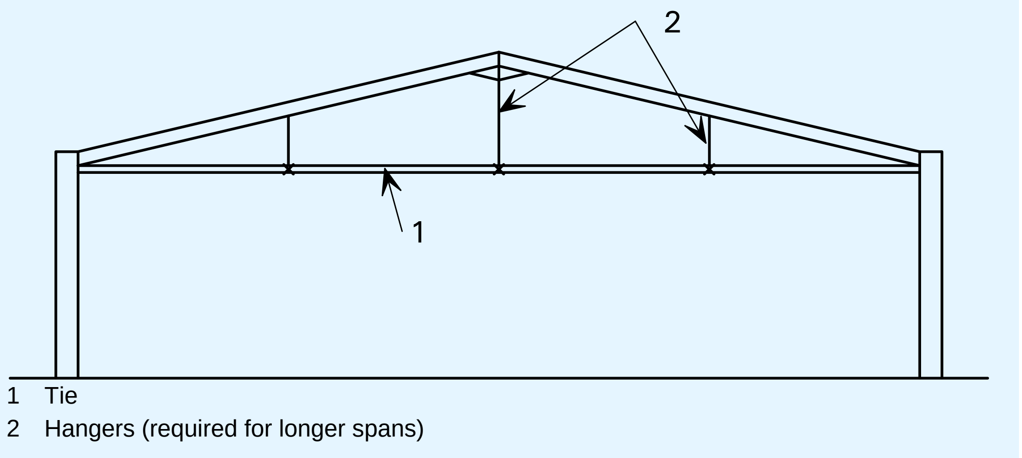


Figure 3.20 Tied portal frame

In a tied portal frame, as illustrated in Figure 3.20, the spread of the eaves and the bending moments in the frame are greatly reduced. Large compression forces will develop in the rafters, which reduce the stability of the members. Second-order software must be used for the design of tied portals.

3.6.5 Mansard or curved portal frames

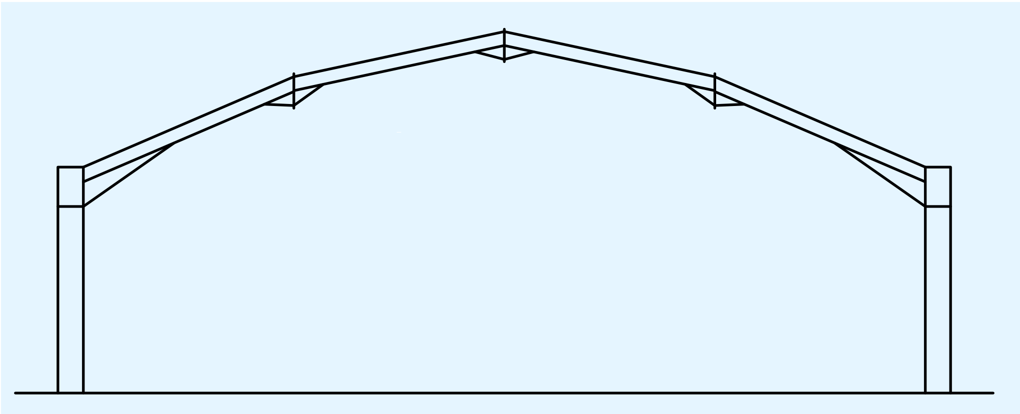


Figure 3.21 Mansard portal frame

A mansard portal frame consists of a series of rafters and haunches, as illustrated in Figure 3.21, which creates a pseudo-curved frame. The connections between the members may also have small haunches to facilitate the bolted connections.

Curved rafter portals as illustrated in Figure 3.22 are often used for architectural applications. The rafter can be curved to a radius by cold bending. For spans greater than approximately 18 m, splices may be required in the rafter because of limitations of transport.

Alternatively, a curved external roof must be produced by varying the lengths of purlin brackets supported on a rafter fabricated as a series of straight elements, as shown in Figure 3.23.

Figure 3.22 Curved beams used in a portal frame

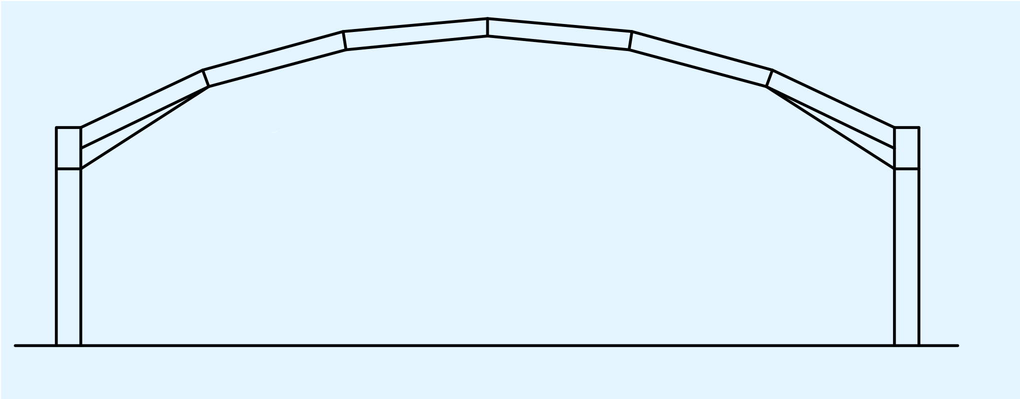
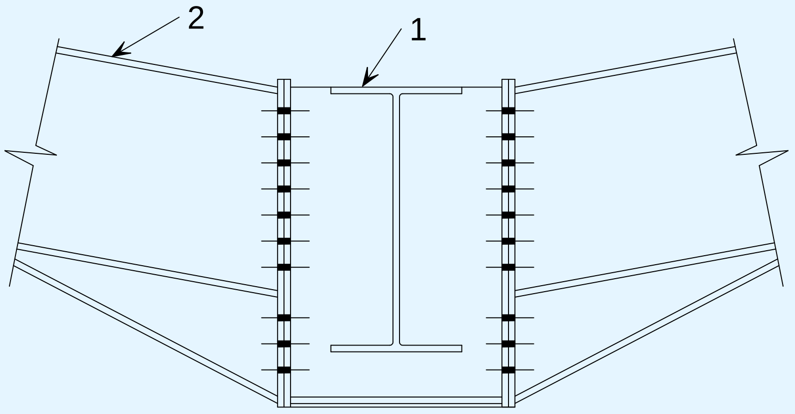


Figure 3.23 Quasi- curved portal frame

3.6.6 Multi bay portal frame

Multi-bay portal frames may be designed by using intermediate columns, as shown in Figure 3.24. If the number of internal columns must be minimised it is possible to remove every second internal column, or to only leave one internal column every third frame. Where the internal column is removed, a deep beam (often known as a “valley” beam) is designed to span between the remaining columns. Continuity of the rafters is achieved by using a haunch connection to the valley beam, as shown in Figure 3.25.

Figure 3.24 Multi-bay portal frame



1 Valley beam
2 Rafter

Figure 3.25 Connection to valley beam

4 CONCEPT DESIGN OF TRUSS BUILDINGS

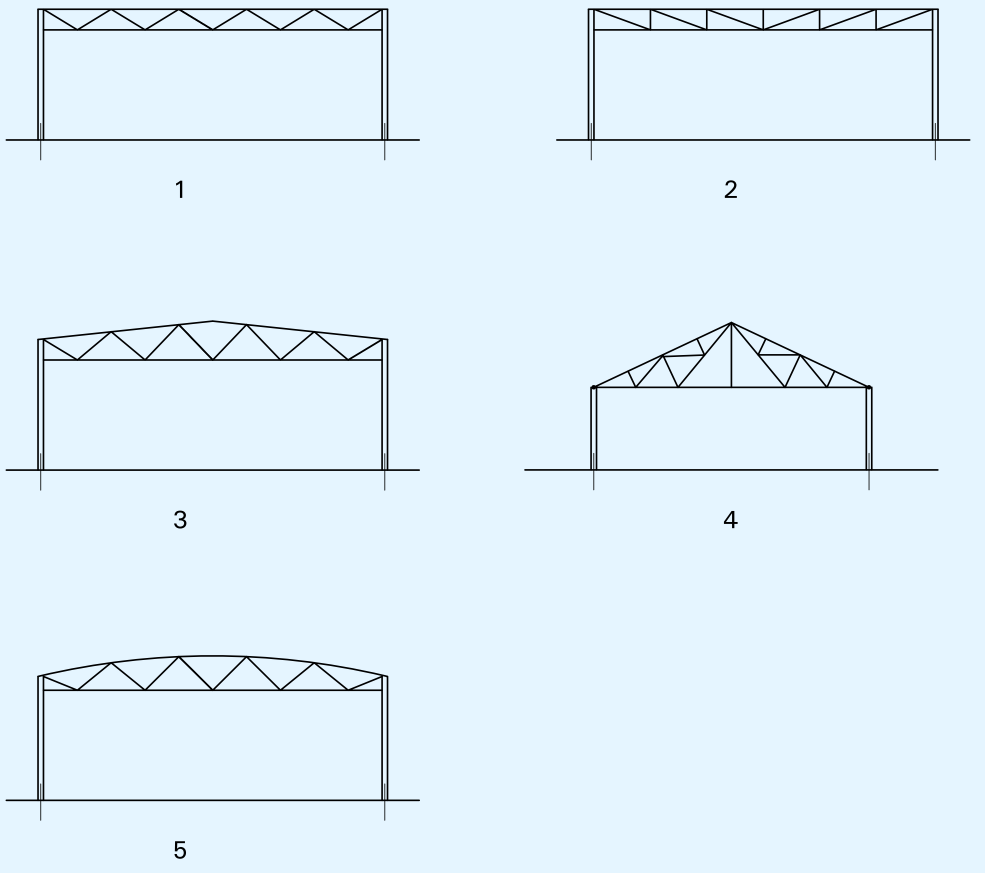
4.1 Introduction

Many forms of truss are possible. Some of the common types of truss for single storey buildings are shown in Figure 4.1.

Trusses are used for long spans, and particularly when significant loads must be carried by the roof structure, as the vertical deflection can be controlled by varying the member sizes.

For industrial buildings, the W-truss N-truss and duo-pitch truss are common. The Fink truss is generally used for smaller spans. Comparing the W-truss and N-truss:

- The W-truss has more open space between the internal members
- The internal members of the W-truss may be larger, because a long diagonal member must carry compression – the compression members in the N-truss are short.



1 W-truss
2 N-truss
3 Duo-pitch truss
4 Fink truss
5 Curved truss

Figure 4.1 Various forms of lattice truss used in industrial buildings

4.2 Truss members

Unless there are special architectural requirements, truss members are chosen to produce a simple connection between the chords and the internal members. Common combinations as shown in Figure 4.2 are:

- Tees used as chords, with angles used as web members. The angles may be welded or bolted to the stem of the Tee.
- Double angle members as chords, and single (or double) angles as internal members. The connections are made with a gusset plate welded between the angles forming the chords.
- Rolled sections as chords, with the web in the plane of the truss. The internal members are usually angle members, connected via a gusset plate welded to the chord.
- Rolled sections as chords, but with the web perpendicular to the plane of the truss. The connections to the chord members may be via gusset plates welded to the web, although the connections will need careful detailing. A simple, effective alternative is to choose chords that have the same overall depth, and connect the internal members to the outside of both flanges, generally by welding.
- For heavily loaded trusses, rolled I or H sections, or channel sections may be used as the internal members. In such a large truss, developing economic connections will be important and both the members and internal members should be chosen with this in mind.

The detailed design of trusses is covered in *Single-storey steel buildings. Part5: Detailed design of trusses*[3].

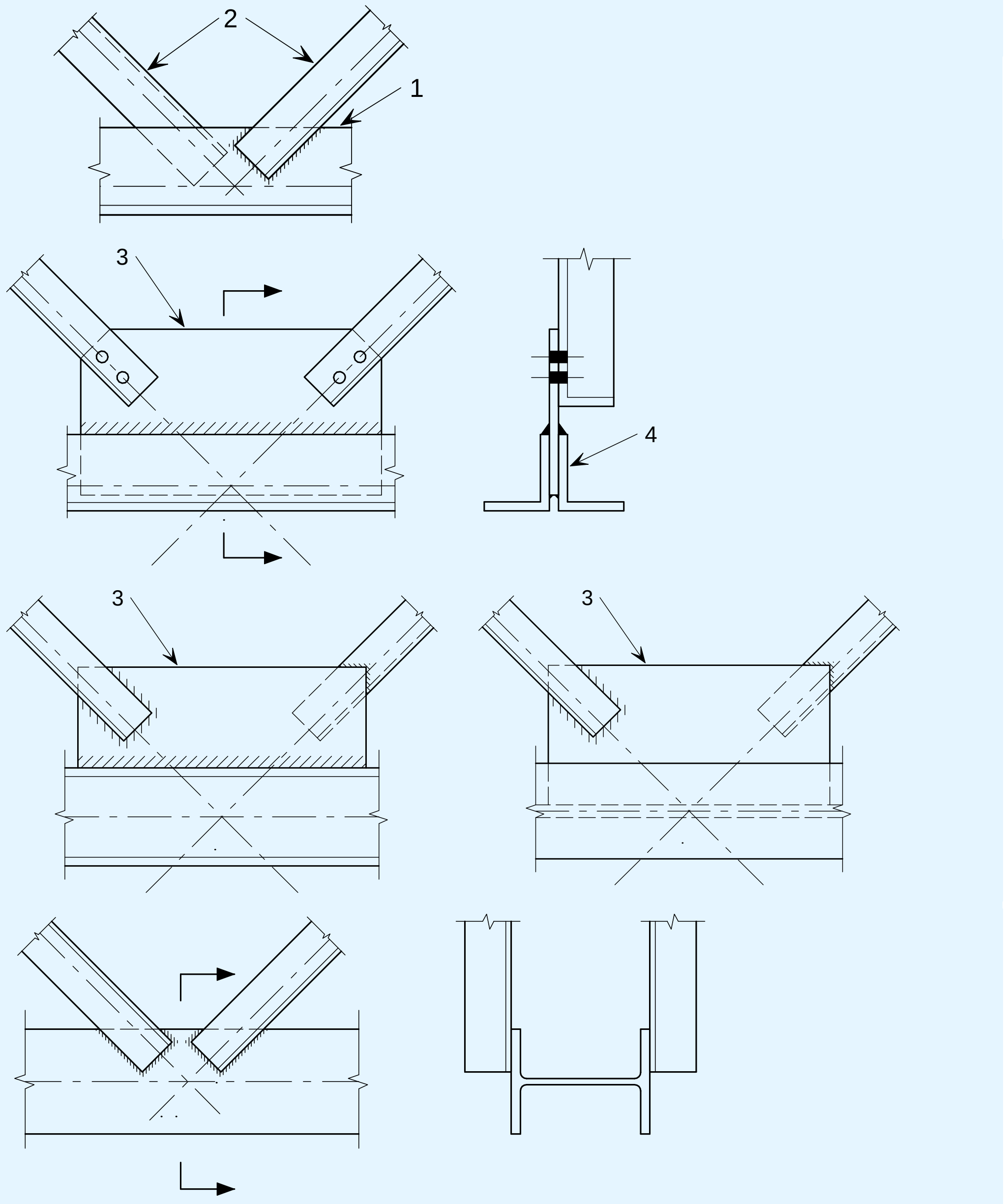


Figure 4.2 Typical truss members

A truss fabricated from rolled sections is illustrated in Figure 4.3.

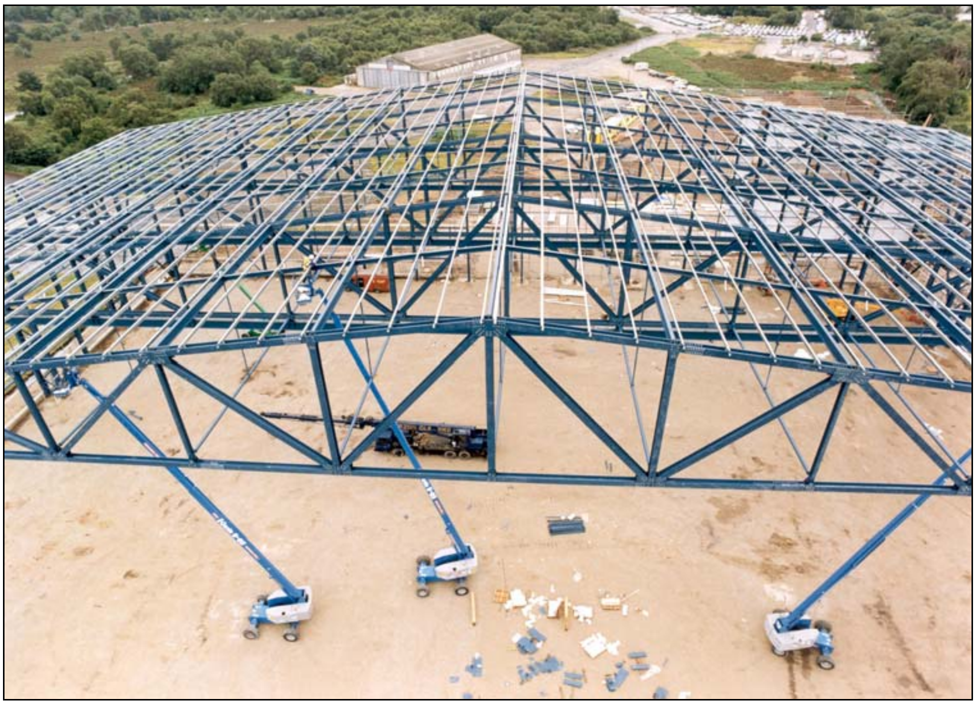
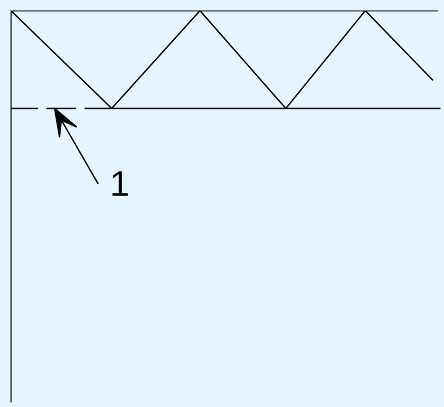


Figure 4.3 Truss fabricated from rolled sections

4.3 Frame stability

In most cases, frame stability is provided by bracing in both orthogonal directions, and the truss is simply pinned to the supporting columns. To realise a pinned connection, one of the chord members is redundant, as shown in Figure 4.4, and the connection of that redundant member to the column is usually allowed to slip in the direction of the axis of the chord.



1 Redundant member

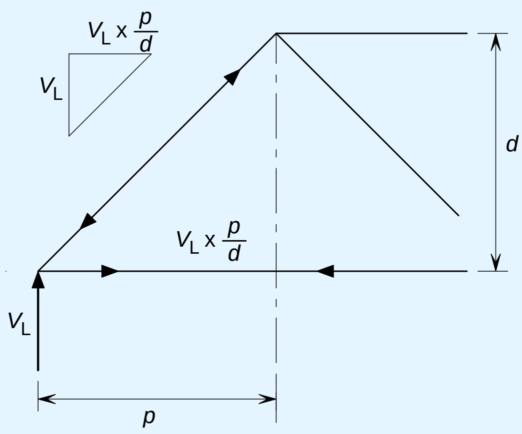
Figure 4.4 Redundant member in a simply supported truss

In the longitudinal direction, stability is usually provided by vertical bracing.

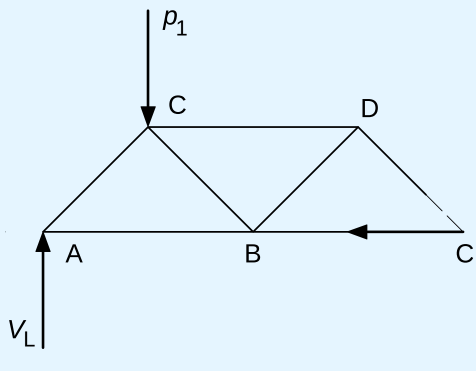
4.4 Preliminary design

At the preliminary design stage, the following process is recommended:

1. Determine the loading on the truss. See Section 1.4.1. At the preliminary design stage it is sufficient to convert all loads, including self weight, to point loads applied at the nodes and assume that the entire truss is pin-jointed. This assumption is also generally adequate for final design. As an alternative, the roof loads may be applied at the purlin positions and the chords assumed to be continuous over pinned internal members, but the precision is rarely justified.
2. Determine a truss depth and layout of internal members. A typical span : depth ratio is approximately 20 for both W- and N-trusses. Internal members are most efficient between 40° and 50°.
3. Determine the forces in the chords and internal members, assuming the truss is pin-jointed throughout. This can be done using software, or by simple manual methods of resolving forces at joints or by taking moments about a pin, as shown in Figure 4.5.



Resolving forces at joints



Taking moments around node D determines the force CB

Figure 4.5 Calculation of forces in a pin-jointed truss

A very simple approach is to calculate the maximum bending moment in the truss assuming that it behaves as a beam, and divide this moment by the distance between chords to determine the axial force in the chord.

4. Select the compression chord member. The buckling resistance is based on the length between node points for in-plane buckling. The out-of-plane

buckling is based on the length between out-of-plane restraints – usually the roof purlins or other members.

5. Select the tension chord member. The critical design case is likely to be an uplift case, when the lower chord is in compression. The out-of-plane buckling is likely to be critical. It is common to provide a dedicated system of bracing at the level of the bottom chord, to provide restraint in the reversal load combination. This additional bracing is not provided at every node of the truss, but as required to balance the tension resistance with the compression resistance.
6. Choose internal members, whilst ensuring the connections are not complicated.
7. Check truss deflections.

4.5 Rigid frame trusses

The structures described in Sections 4.1 and 4.4 are stabilised by bracing in each orthogonal direction. It is possible to stabilise the frames in-plane, by making the truss continuous with the columns. Both chords are fixed to the columns (i.e. no slip connection). The connections within the truss and to the columns may be pinned. The frame becomes similar to a portal frame. For this form of frame, the analysis is generally completed using software. Particular attention must be paid to column design, because the in-plane buckling length is usually much larger than the physical length of the member.

4.6 Connections

Truss connections are either bolted or welded to the chord members, either directly to the chord, or via gusset plates, as shown in Figure 4.6.

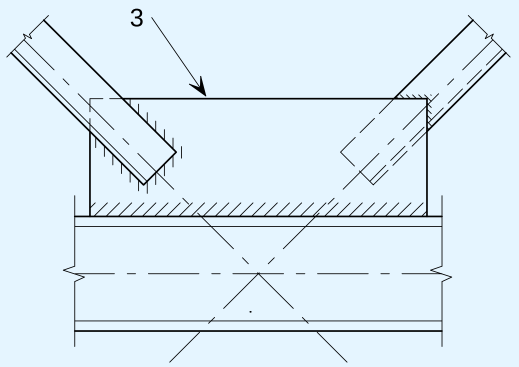


Figure 4.6 Truss connections

Trusses will generally be prefabricated in the workshop, and splices maybe required on site. In addition to splices in the chords, the internal member at the splice position will also require a site connection. Splices may be detailed with cover plates, or as “end plate” type connections, as shown in Figure 4.7.

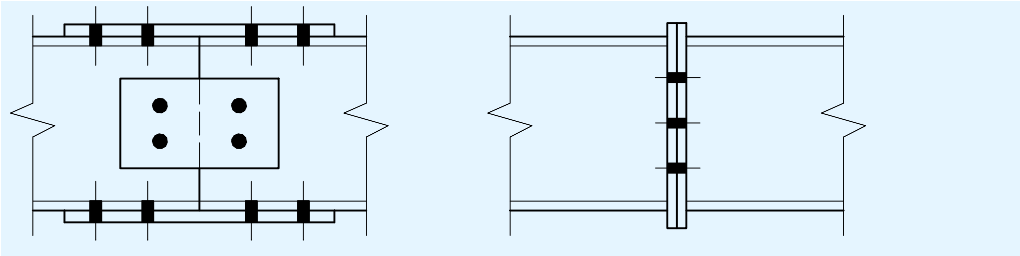


Figure 4.7 Splice details

Ordinary bolts (non-preloaded) in clearance holes may give rise to some slip in the connection. If this slip is accumulated over a large number of connections, the defection of the truss may be larger than calculated. If deflection is a critical consideration, then friction grip assemblies or welded details should be used.

5 SIMPLE BEAM STRUCTURES

For modest spans, (up to approximately 20 m) a simple beam and column structure can be provided, as illustrated in Figure 5.1. The roof beam is a single rolled section, with nominally pinned connections to the columns. The roof beam may be straight, precambered, perforated or curved. The roof may be horizontal, or more commonly with a modest slope to assist drainage. Ponding of water on the roof should be avoided with a slope, or precambered beam.

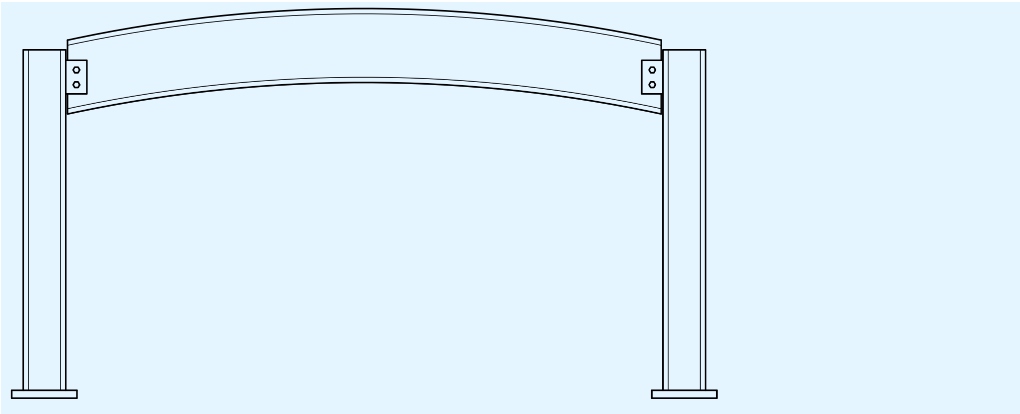


Figure 5.1 Simple beam and column frame

Frame stability for this form of structure is provided by bracing in each orthogonal direction. The beam is designed as simply supported, and the columns as simple struts, with a nominal moment applied by the beam connection. It is common to assume that the shear force from the beam is applied 100 mm from the face of the column.

6 BUILT-UP COLUMNS

Heavily loaded columns, or columns in tall industrial buildings may be in the form of built-up sections. Built-up columns often comprise HE or UPE sections in which battens (flat plate) or lacing (usually angles) are welded across the flanges, as shown in Figure 6.1.

Built-up columns are not used in portal frames, but are often used in buildings supporting heavy cranes. The roof of the structure may be duo-pitch rafters, but is more commonly a truss, as illustrated in Figure 1.4.

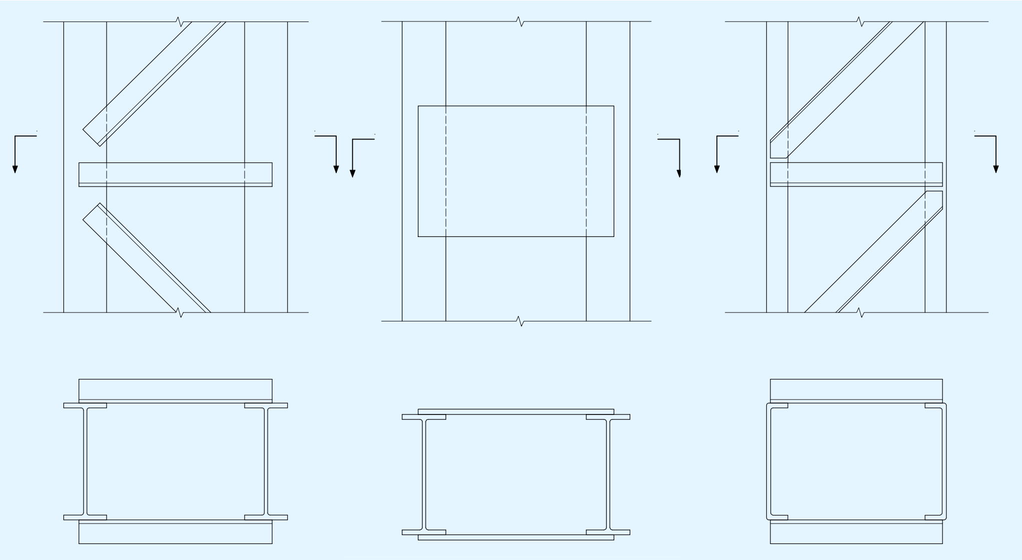


Figure 6.1 Cross-sections of built-up columns

To support the roof above the level of the crane, a single member may project for several meters. This is often known as a “bayonet” column. The projecting member may be a continuation of one of the two primary sections in the built-up section, or may be a separate section located centrally to the built-up section. Examples of built-up columns are shown in Figure 6.2. Buildings that use built-up columns are invariably heavily loaded, and commonly subjected to moving loads from cranes. Such buildings are heavily braced in two orthogonal directions.

The detailed design of built-up columns is covered in *Single-storey steel buildings. Part 6: Detailed design of built-up columns*[4] of this guide.

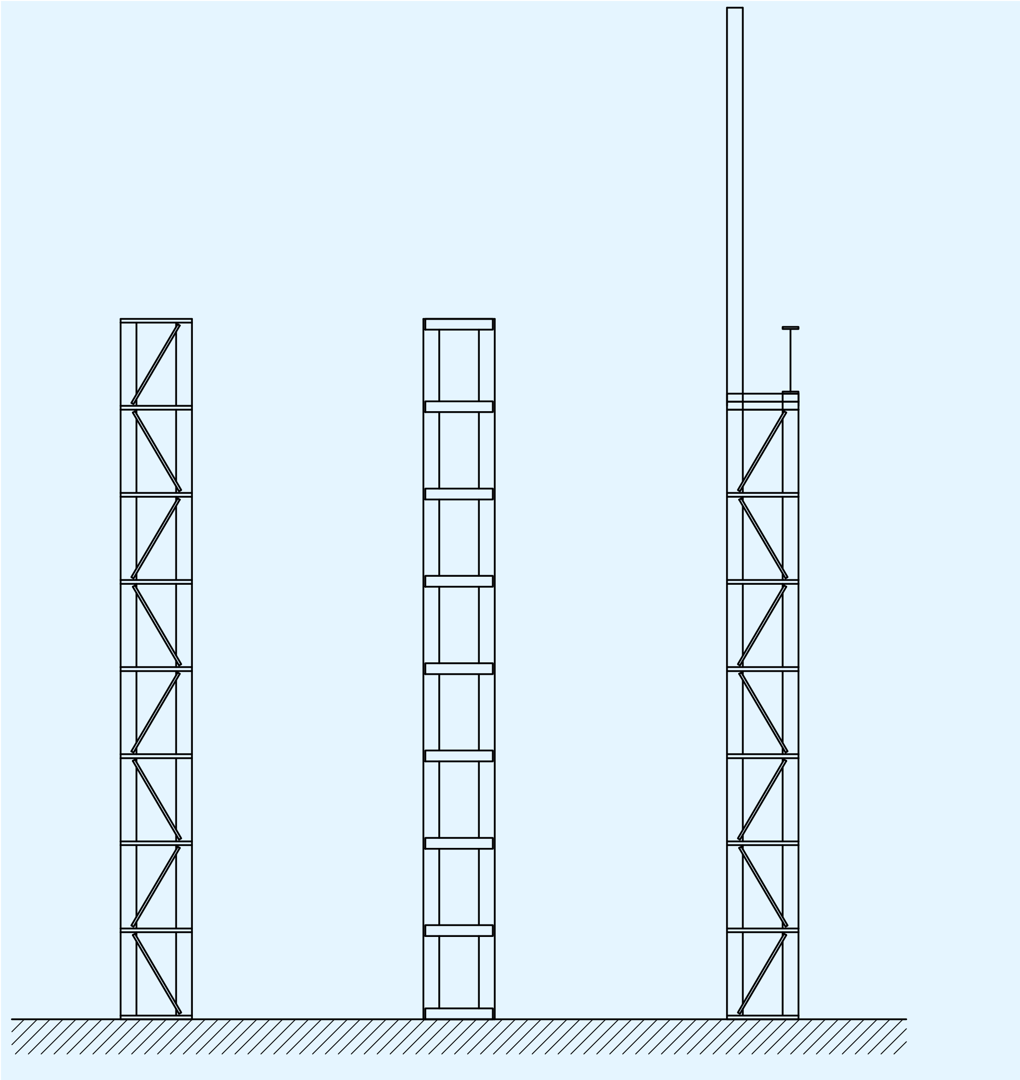


Figure 6.2 Examples of built-up columns in single storey buildings

7 CLADDING

There are a number of generic types of cladding that may be used in single storey buildings, depending on the building use. These fall into four broad categories, which are described in the following sections.

7.1 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 as low as 4° providing the laps and sealants are as recommended by the manufacturers for shallow slopes. The sheeting is fixed directly to the purlins and side rails, as illustrated in Figure 7.1 and provides positive restraint. In some cases, insulation is suspended directly beneath the sheeting.

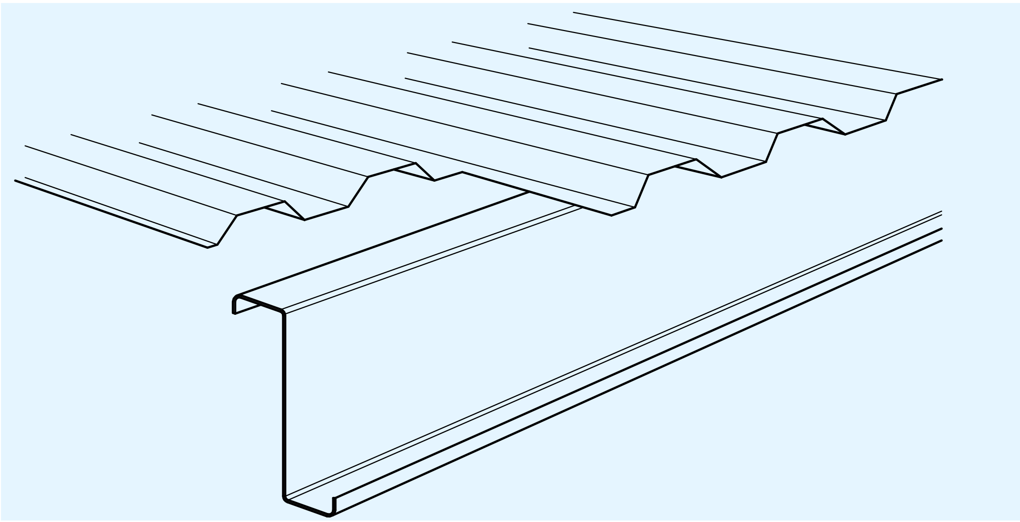


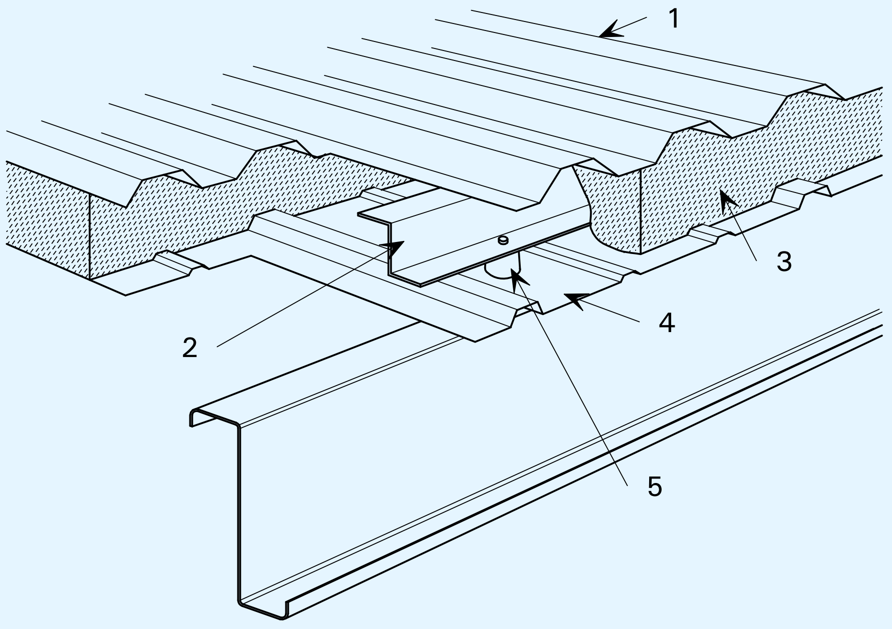
Figure 7.1 Single-skin trapezoidal sheeting

7.2 Double-skin system

Double skin or built-up roof systems usually use a steel liner tray that is fastened to the purlins, followed by a spacing system (plastic ferrule and spacer or rail and bracket spacer), insulation and the outer profiled sheeting. 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. This form of construction using plastic ferrules is shown in Figure 7.2.

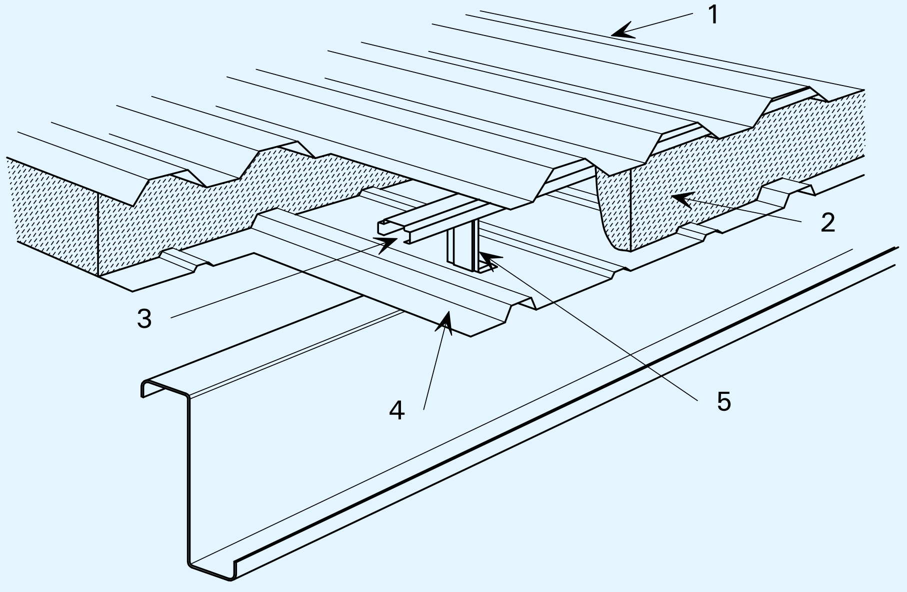
As insulation depths have increased, there has been a move towards “rail and bracket” solutions as they provide greater lateral restraint to the purlins. This system is illustrated in Figure 7.3.

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



1 Outer sheeting
2 Z spacer
3 Insulation
4 Liner tray (inner sheet)
5 Plastic ferrule

Figure 7.2 Double-skin construction using plastic ferrule and Z spacers

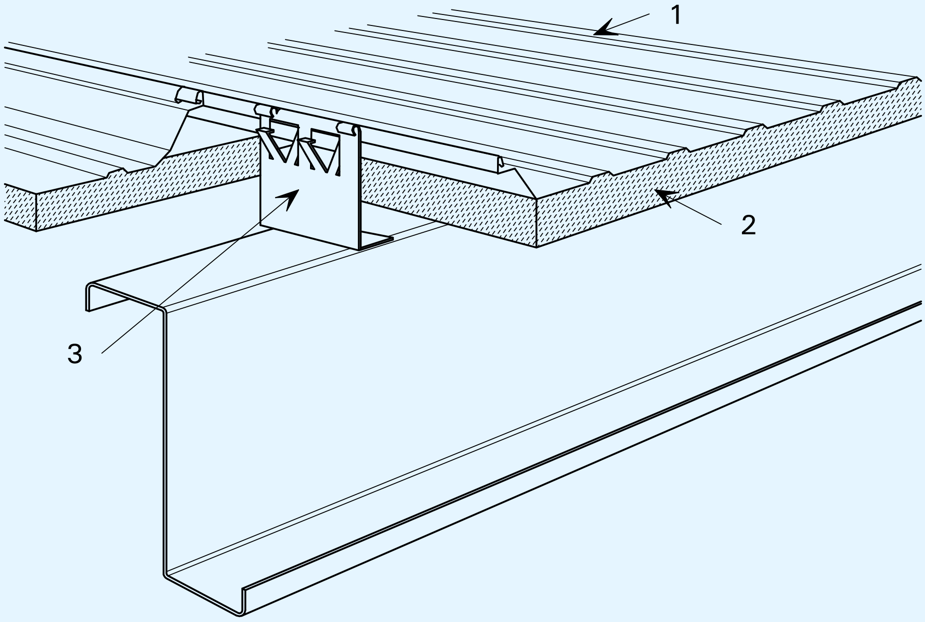


1 Outer sheet
2 Insulation
3 Rail
4 Liner tray (inner sheet)
5 Bracket

Figure 7.3 Double-skin construction using 'rail and bracket' spacers

7.3 Standing seam sheeting

Standing seam sheeting has concealed fixings and can be fixed in lengths of up to 30 m. The advantages are that there are no penetrations directly through the sheeting that could lead to water leakage and fixing of the roof sheeting is rapid. The fastenings are in the form of clips that hold the sheeting down but allow it to move longitudinally (see Figure 7.4). The disadvantage of this system is that less restraint is provided to the purlins than with a conventionally fixed system. Nevertheless, a correctly fixed liner tray should provide adequate restraint.



1 Outer sheet
2 Insulation
3 Standing seam clip

Figure 7.4 Standing seam panels with liner trays

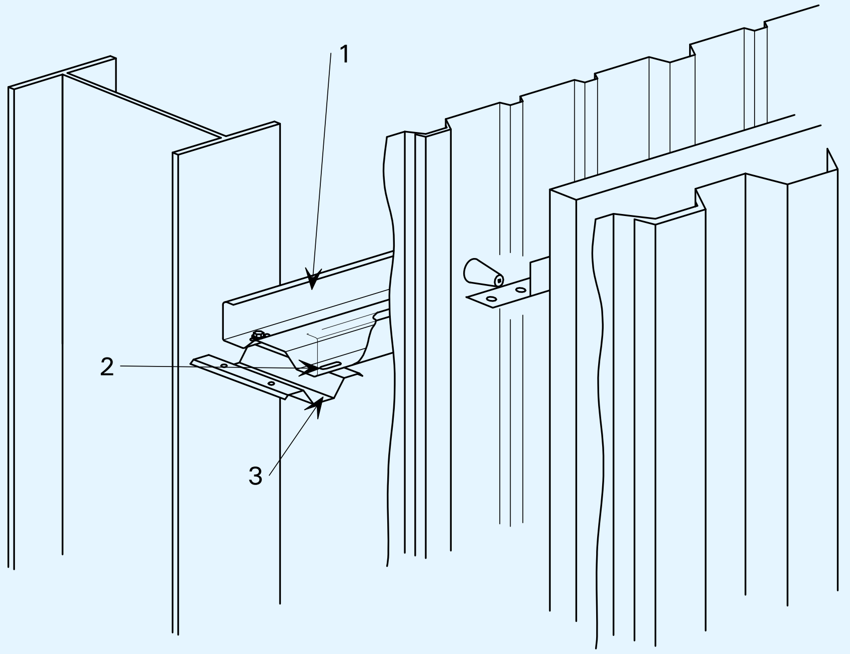
7.4 Composite or sandwich panels

Composite or sandwich panels are formed by creating a foam insulation layer between the outer and inner layer of sheeting. Composite panels have good spanning capabilities due to composite action of the core with the steel sheets. Both standing seam (see Figure 7.4) and direct fixing systems are available. These will clearly provide widely differing levels of restraint to the purlins. The manufacturers should be consulted for more information.

7.5 Fire design of walls

Where buildings are close to a site boundary, most national Building Regulations require that the wall is designed to prevent spread of fire to 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.

Some manufacturers provide slotted holes in the side rail connections to allow for thermal expansion. In order to ensure that this does not compromise the stability of the column by removing the restraint under normal conditions, the slotted holes are 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. Details of this type of system are illustrated in Figure 7.5.



1 Side rail
2 Slotted hole for expansion
3 Cleat

Figure 7.5 Typical fire wall details showing slotted holes for expansion in fire

8 PRELIMINARY DESIGN OF PORTAL FRAMES

8.1 Introduction

The following methods of determining the size of columns and rafters of single-span portal frames may be used at the preliminary design stage. Further detailed calculations will be required at the final design stage. It should be noted that the method does not take account of:

- Requirements for overall stability
- Deflections at the Serviceability Limit State.

8.2 Estimation of member sizes

The guidance for portal frames is valid in the span range between 15 to 40 m. and is presented in Table 8.1. The assumptions made in creating this table are as follows:

- The roof pitch is 6°.
- The steel grade is S235. If design is controlled by serviceability conditions, the use of smaller sections in higher grades may not be an advantage. When deflections are not a concern, for example when the structure is completely clad in metal cladding, the use of higher grades may be appropriate.
- The rafter load is the total factored permanent actions (including self weight) and factored variable actions and is in the range of 8 to 16 kN/m.
- Frames are spaced at 5 to 7,5 m.
- The haunch length is 10% of the span of the frame.
- A column is treated as restrained when torsional restraints can be provided along its length (these columns are therefore lighter than the equivalent unrestrained columns).
- A column should be considered as unrestrained when it is not possible to restrain the inside flange.

The member sizes given by the tables are suitable for rapid preliminary design. However, where strict deflection limits are specified, it may be necessary to increase the member sizes.

In all cases, a full design must be undertaken and members verified in accordance with EN 1993-1-1.

Table 8.1 Member sizes for single-span portal frame with 6° roof pitch

	Rafter load (kN/m)	Eaves height (m)	Span of frame (m)					
			15	20	25	30	35	40
Rafter	8 8 8	6 8 10	IPE 240 IPE 240 IPE 240	IPE 330 IPE 330 IPE 330	IPE 360 IPE 360 IPE 360	IPE 400 IPE 400 IPE 400	IPE 450 IPE 450 IPE 450	IPE 450 IPE 450 IPE 450
Restrained column	8 8 8	6 8 10	IPE 300 IPE 300 IPE 300	IPE 360 IPE 360 IPE 400	IPE 450 IPE 450 IPE 450	IPE 550 IPE 550 IPE 550	IPE 550 IPE 600 IPE 600	IPE 600 IPE 600 IPE 750 × 137
Unrestrained column	8 8 8	6 8 10	IPE 360 IPE 450 IPE 450	IPE 450 IPE 550 IPE 550	IPE 550 IPE 600 IPE 600	IPE 550 IPE 600 IPE 750 × 137	IPE 0 600 IPE 750 × 137 IPE 750 × 173	IPE 750 × 137 IPE 750 × 173 HE 800
Rafter	10 10 10	6 8 10	IPE 270 IPE 270 IPE 270	IPE 330 IPE 330 IPE 360	IPE 400 IPE 400 IPE 400	IPE 450 IPE 450 IPE 450	IPE 0 450 IPE 0 450 IPE 0 450	IPE 550 IPE 550 IPE 550
Restrained column	10 10 10	6 8 10	IPE 360 IPE 360 IPE 360	IPE 450 IPE 450 IPE 450	IPE 450 IPE 550 IPE 550	IPE 550 IPE 550 IPE 600	IPE 600 IPE 600 IPE 600	IPE 750 × 137 IPE 750 × 137 IPE 750 × 173
Unrestrained column	10 10 10	6 8 10	IPE 400 IPE 450 IPE 450	IPE 450 IPE 550 IPE 600	IPE 550 IPE 600 IPE 750 × 137	IPE 600 IPE 750 × 137 IPE 750 × 173	IPE 750 × 137 IPE 750 × 173 HE 800	IPE 750 × 137 HE 800 HE 800
Rafter	12 12 12	6 8 10	IPE 270 IPE 270 IPE 270	IPE 360 IPE 360 IPE 60	IPE 400 IPE 400 IPE 400	IPE 450 IPE 450 IPE 450	IPE 550 IPE 550 IPE 550	IPE 550 IPE 550 IPE 600
Restrained column	12 12 12	6 8 10	IPE 360 IPE 360 IPE 360	IPE 450 IPE 450 IPE 450	IPE 550 IPE 550 IPE 550	IPE 600 IPE 600 IPE 600	IPE 750 × 137 IPE 750 × 137 IPE 750 × 137	IPE 750 × 173 IPE 750 × 173 IPE 750 × 173
Unrestrained column	12 12 12	6 8 10	IPE 450 IPE 450 IPE 550	IPE 550 IPE 600 IPE 600	IPE 600 IPE 600 IPE 750 × 173	IPE 600 IPE 750 × 173 HE 800	IPE 750 × 137 HE 800 HE 800	IPE 750 × 173 HE 800 HE 900

Table 8.1 (Continued) Single-span portal frame with 6° roof pitch

	Rafter load (kN/m)	Eaves height (m)	Span of frame (m)					
			15	20	25	30	35	40
Rafter	14 14 14	6 8 10	IPE 330 IPE 330 IPE 330	IPE 400 IPE 400 IPE 400	IPE 450 IPE 450 IPE 450	IPE 450 IPE 450 IPE 450	IPE 550 IPE 550 IPE 550	IPE 600 IPE 600 IPE 600
Restrained column	14 14 14	6 8 10	IPE 360 IPE 400 IPE 400	IPE 450 IPE 450 IPE 450	IPE 550 IPE 550 IPE 600	IPE 600 IPE 600 IPE 750 × 137	IPE 750 × 173 IPE 750 × 173 IPE 750 × 173	IPE 750 × 173 HE 800 HE 800
Unrestrained column	14 14 14	6 8 10	IPE 450 IPE 550 IPE 550	IPE 550 IPE 600 IPE 750 × 137	IPE 600 IPE 750 × 137 IPE 750 × 173	IPE 750 × 137 IPE 750 × 173 HE 800	IPE 750 × 173 HE 800 HE 800	HE 800 HE 800 HE 900
Rafter	16 16 16	6 8 10	IPE 330 IPE 330 IPE 330	IPE 400 IPE 400 IPE 400	IPE 450 IPE 450 IPE 450	IPE 550 IPE 550 IPE 50	IPE 550 IPE 600 IPE 600	IPE 600 IPE 600 IPE 600
Restrained column	16 16 16	6 8 10	IPE 400 IPE 400 IPE 450	IPE 550 IPE 550 IPE 550	IPE 600 IPE 600 IPE 600	IPE 750 × 137 IPE 750 × 137 IPE 750 × 137	IPE 750 × 173 IPE 750 × 173 HE 800	HE 800 HE 800 HE 800
Unrestrained column	16 16 16	6 8 10	IPE 450 IPE 550 IPE 600	IPE 550 IPE 600 IPE 750 × 137	IPE 600 IPE 750 × 173 HE 800	IPE 750 × 137 HE 800 HE 800	IPE 750 × 173 HE 800 HE 900	HE 800 HE 900 HE 900

REFERENCES

1 SANSOM, M. and MEIJER, J.
Life-cycle assessment (LCA) for steel construction
European commission, 2002

2 Several assessement methods are used. For example:
- BREEAM in the UK
- HQE in France
- DNGB in Germany
- BREEAM-NL, Greencalc+ and BPR Gebouw in the Netherlands
- Valideo in Belgium
- Casa Clima in Trento Alto Adige, Italy (each region has its own approach)
- LEED, used in various countries

3 Steel Buildings in Europe
Single-storey steel buildings. Part 5: Design of trusses

4 Steel Buildings in Europe
Single-storey steel buildings. Part 6: Design of built-up columns