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Design of Single-Span Steel Portal Frames to BS 5950-1:2000

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FOREWORD

Single-span steel portal frames are a common form of construction for single-storey buildings in the UK, but there is relatively little published guidance on the design of such structures.

This publication concentrates on the design of portal frames using hot rolled steel sections rather than fabricated members. It has been written in response to questions raised by designers and steelwork contractors. It is the result of extensive consultation with structural engineers closely involved in the design and construction of steel portal frames.

The initial draft of this publication was prepared by the late Paul Salter. Paul was a well respected colleague who made invaluable contribution to the development of the publication and SCI wishes to express its gratitude for his input. Subsequent development of the publication, including revision to bring it fully into line with BS 5950-1:2000, was carried out by Abdul Malik and Charles King, both of The Steel Construction Institute.

Alan Rathbone of CSC (UK) Ltd. carried out the computer analysis and provided the output summary for Appendix D.

Further assistance was provided by Dr R M Lawson, Mr D G Brown, and Dr J W Rackham of the SCI, and Mr P J Grubb, a consultant to the SCI. The following contributed to the preparation of the publication and made extensive comments on the various drafts:

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A J Todd	Corus Construction & Industrial

In addition, the authors are grateful to the various SCI and BCSA member organisations who responded to a questionnaire on aspects of portal frame design that would benefit from better explanation and more information.

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Contents

SUMMARY

This publication provides an introduction to the design of single-span steel portal frames and brings together existing design guidance on this common form of construction and those aspects of design that are not properly covered by existing guidance.

The publication reviews briefly the range of different types of steel portal frame, before expanding on the design of single-span portal frames in detail. The design considerations for all the major components (columns, rafters, haunches, purlins, etc.) are explained. The use of elastic and plastic frame analysis for portal frames is examined, and all aspects of frame stability are considered. Serviceability and fire limit state design are addressed, as regards their effect on the overall design of the building.

Preliminary design methods are summarised in an Appendix. A worked example based on BS 5950-1:2000 is presented in two further Appendices, one showing manual calculation, the other showing output from a standard computer program.

Dimensionnement de portiques en acier à simple portée selon la BS 5950-1:2000

Résumé

Cette publication a pour objectif de constituer une introduction au dimensionnement des portiques en acier à simple portée et de rassembler les guidances techniques existantes pour ce type de structure très répandu. Elle aborde également des aspects du dimensionnement qui ne sont pas couverts par les guides existants.

La publication passe brièvement en revue les domaines d'application de différents types de portique en acier avant d'exposer en détail le calcul de ces portiques. Les problèmes de dimensionnement sont expliqués pour tous les composants principaux du portique (poteaux, poutres, jarrets, pannes, …). L'utilisation de l'analyse élastique et de l'analyse plastique est discutée ainsi que les problèmes d'instabilité du portique. Les conditions de service et la résistance à l'incendie sont pris en compte dans le dimensionnement de la structure.

Des méthodes de dimensionnement préliminaire sont données en annexe. Un exemple complet basé sur la BS 5950-1:2000 est également donné dans deux autres annexes; la première montrant un calcul manuel et la seconde présentant les résultats d'un programme de calcul classique.

Berechnung von einschiffigen Stahlrahmen nach BS 5950-1:2000

Zusammenfassung

Diese Publikation ist eine Einführung in die Berechnung von einschiffigen Stahlrahmen und bündelt bestehende Anleitungen zur Berechnung dieser gebräuchlichen Bauform. Sie konzentriert sich auf jene Aspekte der Berechnung, die in bestehenden Leitfäden nicht richtig behandelt werden.

Die Publikation gibt einen kurzen Überblick über die verschiedenen Typen von Stahlrahmen, bevor die Berechnung von einschiffigen Rahmen im Detail besprochen wird. Die

Überlegungen zur Berechnung der wichtigsten Bauteile (Stützen, Riegel, Vouten, Pfetten, etc.) werden erläutert. Die Anwendung elastischer und plastischer Rahmenberechnung wird untersucht und alle Aspekte der Rahmenstabilität werden berücksichtigt. Gebrauchstauglichkeit und die Berechnung im Brandfall werden angesprochen, soweit sie die Gesamtberechnung des Gebäudes beeinflussen.

In einem Anhang werden überschlägige Berechnungsmethoden zusammengefasst. Ein Berechnungsbeispiel wird in zwei Anhängen vorgestellt, einmal als Handrechnung und einmal als Ergebnis eines Computer-Programms.

Progettazione di portali in acciaio a campata singola in accordo alla normativa BS 5950-1:2000

Sommario

Questa pubblicazione fornisce una introduzione alla progettazione di portali in acciaio a campata singola e riporta le principali regole progettuali esistenti per questa comune tipologia strutturale. In particolare, vengono affrontate quelle tematiche che non sono affrontate in dettaglio nella guida progettuale esistente.

Prima di analizzare in dettaglio le strutture a campata singola, nella pubblicazione sono criticamente esaminate, in modo sintetico, le possibili soluzioni per portali in acciaio. Per i portali a campata singola, vengono introdotte e discusse le procedure di progetto delle componenti principali (colonne, capriate, elementi di attacco, arcarecci, ecc.). L'uso dell'analisi sia elastica sia plastica è considerata e vengono trattati i problemi associati alla stabilità del telaio. Gli stati limite di servizio e lo stato limite ultimo di resistenza al fuoco sono considerati con riferimento alla loro influenza sulla progettazione globale dell'edificio.

Una appendice alla pubblicazione è dedicata ai principali criteri di predimensionamento, o progettazione preliminare. Due ulteriori appendici sono invece relative a un esempio applicativo: la prima appendice riporta i calcoli manuali mentre la seconda contiene il tabulato finale di un programma di calcolo automatico.

Proyecto en estructuras aporticadas de un solo vano de acuerdo con la BS 5950-1:2000

Resumen

Esta publicación suministra una introducción al proyecto de estructuras aporticadas de un solo vano y reúne la información existente para el proyecto de este tipo tan común de estructuras. Se concentra en los aspectos de proyecto que no están cubiertos por otras recomendaciones existentes.

Antes de comentar en detalle el proyecto de los entramados de un solo vano, la publicación repasa brevemente las diferentes tipologías de estructuras aporticadas de acero. Se explican los detalles del proyecto para entramados de un solo vano y para todos los componentes principales (columnas, capiteles, vigas, correas, etc). Se explora el uso de los métodos de cálculo elástico y plástico así como la estabilidad de la estructura. Se trata en detalle el proyecto de columnas, vigas y contravientos y se comentan diversos aspectos de las situaciones de servicio y del estado límite del fuego.

En un Apéndice se resumen métodos de anteproyecto. En dos apéndices más, se desarrolla un ejemplo basado en la norma BS 5950-1:2000; en uno de ellos con cálculo manual y en el otro con una salida de ordenador.

Dimensionering av enspanns hallramar enligt BS 5950-1:2000

Sammanfattning

Denna publikation ger en introduktion till dimensionering av enspanns hallramar i stål, och sammanfattar befintliga handledningar för denna vanliga byggnadsdel. Tyngdpunkten ligger på de konstruktionsområden som inte helt täcks av befintlig handledning.

Publikationen ger en kort översikt av olika typer av hallramar i stål, för att därefter fördjupas kring dimensionering av hallramar i detalj. Konstruktionsförutsättningarna för alla viktigare ingående komponenter (pelare, takstolar, voter, takåsar etc) förklaras. Användningen av elastisk och plastisk ramanalys för hallramar studeras, och ramstabilitet beaktas ur alla synvinklar. Bruksgränstillstånd och branddimensionering berörs avseende dessas övergripande inverkan på byggnadens utformning.

Överslagsmässiga dimensioneringsmetoder sammanfattas i Appendix. Ett beräkningsexempel som baseras på BS 5950-1:2000 presenteras i ytterligare två bilagor, varav det ena visar en manuell beräkning och det andra resultat från ett vanligt datorprogram.

1 INTRODUCTION

It is estimated that 50% of all constructional steelwork used in the UK is in the primary framework of single-storey buildings. Within this major market sector, the steel portal frame has become the most common structural form in pitched roof buildings, because of its economy and versatility for a wide range of spans. Although the use of steel portal frames is well established in the UK, there is no publication which defines best practice in this form of construction.

The guidance in this publication concentrates on the design of single-span portal frames using hot rolled steel I sections, but the general principles also apply to multi-span portals and to the use of fabricated sections.

Where possible, the guidance given has been agreed with designers, steelwork contractors and those concerned with checking for building control purposes. It deals with the issues that occur reasonably often in design practice and which are amenable to general guidance. Aspects required for concept or preliminary design are covered first, followed by more details for final design. Secondary elements, such as purlins, end gables and cladding are also reviewed.

The use of computer-aided design has made manual calculations almost redundant for regular portal frames, and therefore detailed guidance on manual methods of analysis is not included. However, tables and charts for preliminary design are presented, and reference is made to other publications for manual analysis techniques. Output from the CSC Fastrak program is included in Appendix D, as this program is widely used by steel fabricators in the UK.

Where guidance is given in detail elsewhere, for example on the design of portal frames in fire boundary conditions, established publications are referred to, with a brief explanation and review of their contents. Cross-reference is made to the relevant clauses of BS 5950-1:2000[1].

The amendment of BS 5950-1 from the 1990 version to the 2000 version gave rise to some technical changes that affect the design of portal frames. Also, clauses were renumbered in the 2000 version. The main changes that affect detailed design of portal frames are as follows:

- Section classification
- Lateral-torsional buckling
- Equivalent uniform moment factors for buckling checks
- Sway stability.

2 TYPES OF STEEL PORTAL FRAME

Portal frames are generally low-rise structures, comprising *columns* and horizontal or pitched *rafters*, connected by moment-resisting connections (Figure 2.1). The frame relies on the bending resistance of the connections, which are stiffened by a suitable *haunch* or deepening of the rafter sections. This form of *rigid* frame structure is stable in its plane and provides a clear span that is unobstructed by bracing.

A number of types of structure can be classified broadly as portal frames. These are described briefly in Sections 2.1 to 2.12, but the subsequent Sections of this publication concentrate on the design of single-span symmetric portal frames. All the frame types described can be designed for a range of base fixity; selection of appropriate fixity is an important design decision (see Sections 5 and 12). Nominally pinned base is the most common for convenience of foundation design and construction. It may not give the most economic total solution to foundation and structure because even modest base stiffness often gives major improvements in frame stability.

The information given with regard to spans, roof pitch, etc. is typical of the frames that are illustrated. It is not intended that the information should dictate limits on the use of these forms of construction, although each has its optimum application and span.

2.1 Pitched roof portal (fabricated from UBs)

A single-span symmetrical pitched roof portal frame (Figure 2.1) will typically have:

- A span between 15 m and 50 m
- An eaves height between 5 and 10 m
- A roof pitch between 5° and 10° (6° is commonly adopted)
- A frame spacing between 5 m and 8 m (the greater spacings being associated with the longer span portal frames)
- Haunches in the rafters at the eaves and apex.

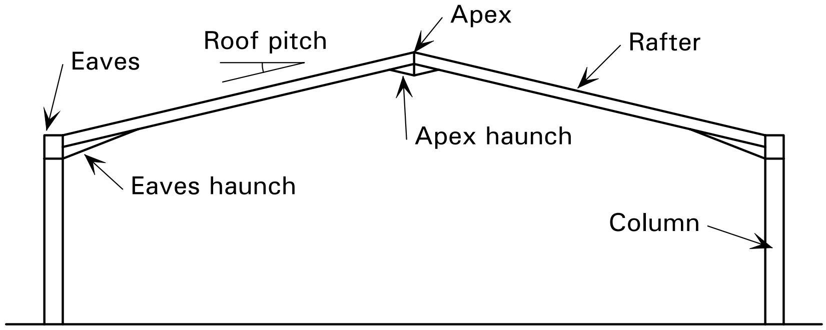


Figure 2.1 *Single-span symmetric portal frame*

Most of these characteristics are dictated by the economics of portal frames relative to other forms of construction. The use of haunches at the eaves and apex both reduces the required depth of rafter and achieves an efficient moment connection at these points.

2.2 Portal frame with a mezzanine floor

Office accommodation is often provided within a portal frame structure using a partial width mezzanine floor (Figure 2.2).

The portal frame must be designed to stabilise the mezzanine as shown in SCI publication *In-plane stability of portal frames to BS 5950-1:2000* (P292) [2].

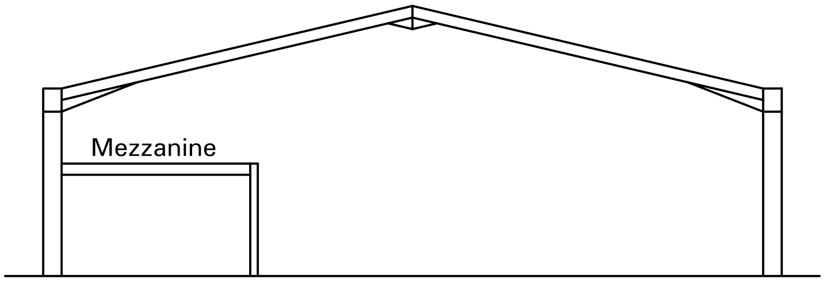


Figure 2.2 *Portal frame with internal mezzanine floor*

2.3 Portal frame with 'lean-to'

Placing the offices externally to the portal frame may create an asymmetric portal structure as shown in Figure 2.3. The main advantage of this configuration is that large columns and haunches do not obstruct the office space.

Where the office structure is truly a 'lean-to', it depends on the portal for its stability. Therefore the portal frame must be designed to stabilise the office structure, in the same way as the portal frame stabilises a mezzanine as shown in P292.

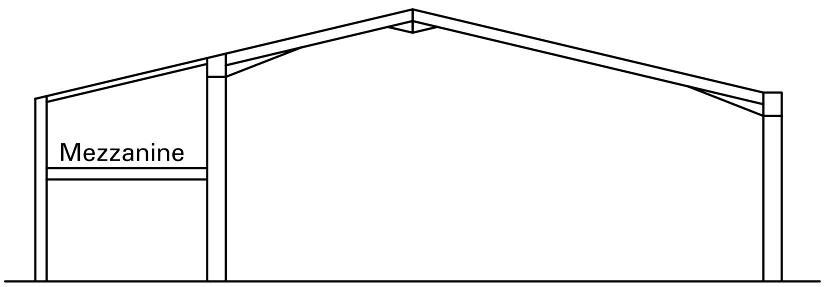


Figure 2.3 *Portal frame with external mezzanine*

2.4 Crane portal frame with column brackets

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 (Figure 2.4). Use of a tie member or nominally 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. It is advisable to check the requirements with the client and with manufacturer of the crane.

The spread can be reduced by a number of approaches, including:

- Selecting stiffer members for column and rafters.
- Reducing the pitch of the roof.
- Introducing ties at eaves level (if these do not clash with the crane).
- Using nominally rigid column bases.

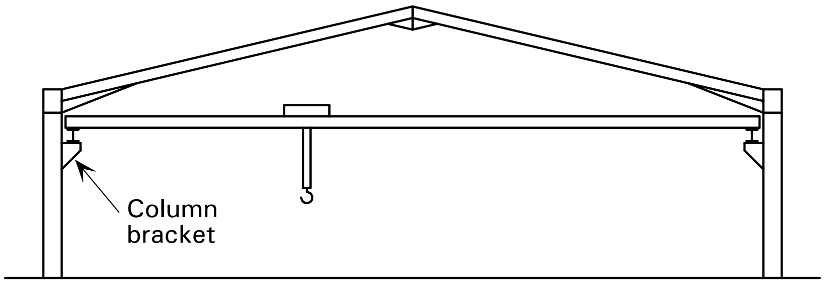


Figure 2.4 *Crane portal frame with column brackets*

2.5 Mono-pitch portal frame

A mono-pitch portal frame. as shown in Figure 2.5, is usually chosen for small spans 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 (up to 15 m span).

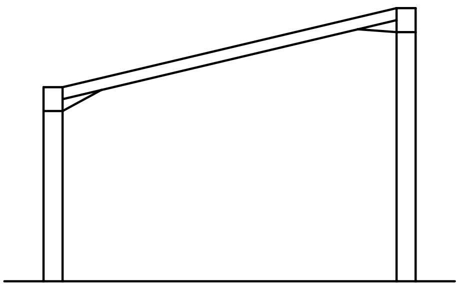


Figure 2.5 *Mono-pitch portal frame*

2.6 Propped portal frame

Where the span of a portal frame is large (greater than say 30 m), and there is no need to provide a clear span, a propped portal frame (Figure 2.6) can reduce the rafter size and also the horizontal thrust at the base, giving economies in both steelwork and foundation costs.

This type of frame is sometimes referred to as a “single span propped portal”, but it is a two-span portal frame in terms of structural behaviour.

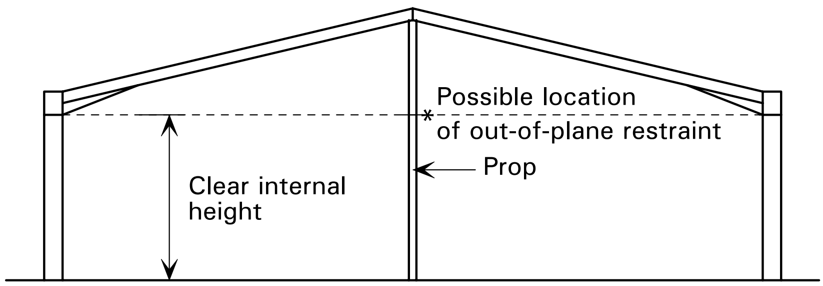


Figure 2.6 *Propped portal frame*

In designing these structures, the following points should be noted:

- Where the prop is designed as an axially-loaded pin-ended member, the connection at both ends should be detailed to realise this assumption. However, pin-ended props reduce frame stability and can cause difficulties during erection, so such props are not recommended.
- To reduce the deflections due to horizontal loads and improve the frame stability, the prop can be designed as a column with a rigid connection at the base and/or the top. The column size may need to be increased relative to that of pin ended prop to resist moments induced by the rigidity of the connections.
- The use of haunches either side of the central prop will allow the rafter section to be reduced, because the haunch is better able to resist the large hogging moments over the prop. However, the reduction in the rafter size will tend to decrease the in-plane stability of the frame.
- Depending on the use of the building, it may be possible to provide an out-of-plane restraint to the prop at the level of the clear internal height. This restraint should be tied back into vertical bracing at some point within the building.
- Where the prop is designed to resist moments in conjunction with an apex haunch, the eaves haunches can be made smaller, thus increasing the useable internal height of the building.
- Where a propped frame is used in a boundary condition in terms of fire (see Section 15), the prop should be fire protected to avoid having to consider the frame as a clear single span for design at the fire limit state.

2.7 Tied portal frame

In a tied portal frame (Figure 2.7), the horizontal movement of the eaves and the moments in the columns are reduced. This can be useful when a crane is designed to span across the structure below the tie level. The disadvantage is that the available headroom is also reduced. For roof slopes of less than 15°, very large forces will develop in the rafters and the tie. These large forces reduce the stability of the frame, so the design and analysis should be undertaken with special caution.

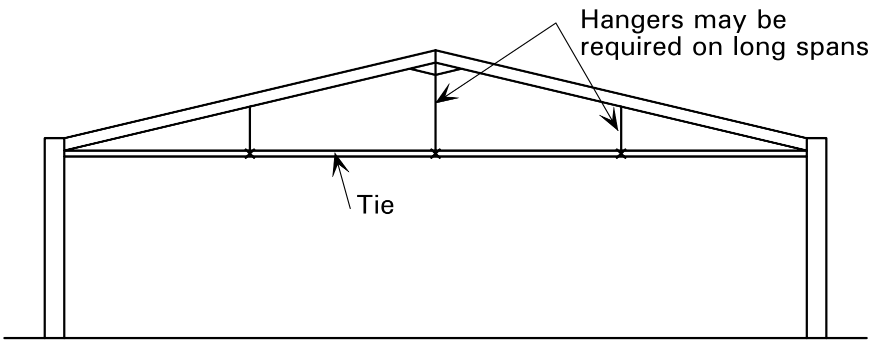


Figure 2.7 *Tied portal frame*

The tie connection at the eaves is relatively expensive. Such connections must be sufficiently stiff and free from movement to prevent deflections not included in the design calculations.

For conventional tied portal frames, it is recommended that the ties should be sized to remain elastic (i.e. tensile force less than $p_y A_{net}$) at the ultimate limit state to ensure that the analysis reflects the behaviour of the structure. It is also recommended that the end connections should include adjustment to allow for fabrication and erection tolerances.

For tied portal frames, BS 5950-1 Clause 5.5.4.6 requires that the in-plane stability of the frame should be checked using second-order analysis. The in-plane buckling of the rafters must be considered, because of the high axial compression in the rafters, which are small relative to the span of the portal. The analysis should allow for the increase in the tie force due to the reduction in the lever arm from the apex to the tie, caused by the extension of the tie and deformation of the rafter, unless the tie is supported by a hanger designed to avoid reducing this lever arm.

2.8 Mansard portal frame

A mansard portal frame may be used where a large clear span is required (Figure 2.8) but the eaves height of the building has to be minimised. A tied mansard portal frame may be economic solution where there is a need to restrict eaves spread.

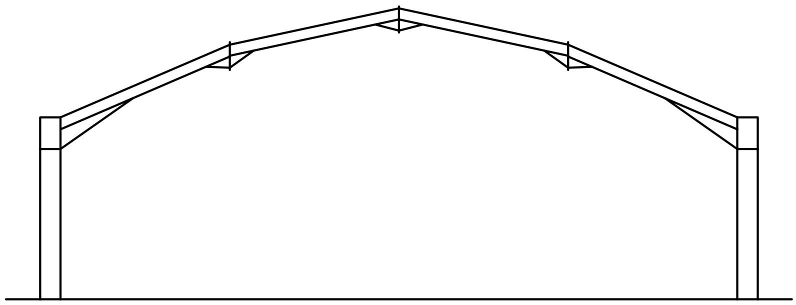


Figure 2.8 *Mansard portal frame*

2.9 Curved rafter portal frame

A number of curved rafter portal frame buildings (Figure 2.9(a)) have been constructed in recent years, mainly for architectural applications. The rafter can be curved to a radius by cold bending. For spans greater than 16 m, splices may be required in the rafter because of limitations of transport. These splices should be carefully detailed, for architectural reasons.

These portal frames are often analysed using a model in which the curve is represented by a series of four straight elements. For guidance on the member stability of curved rafters in portal frames, see SCI publication *Design of curved steel* (P281)[3].

Alternatively, where the roof must be curved the rafter need not be curved. The rafter can be fabricated as a series of straight elements (Figure 2.9(b)). For this case, it may be necessary to "stool" the purlin cleats to achieve the curved roof.

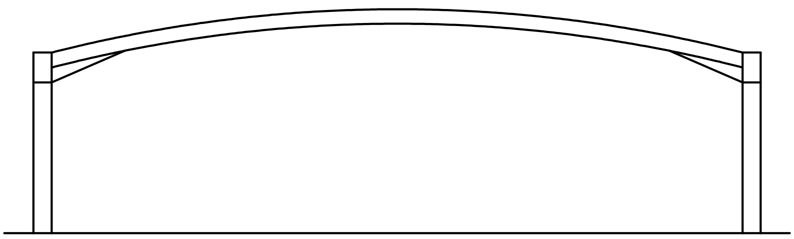


Figure 2.9 **(a)** *Curved rafter portal frame*

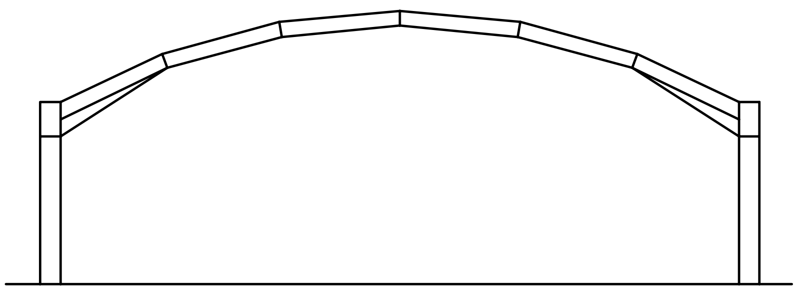


Figure 2.9 **(b)** *Quasi-curved portal frame*

2.10 Cellular beam portal frame

In recent years a series of portal frames have been constructed using cellular beams. Such frames commonly have curved rafters (Figure 2.10), which are easily achieved using cellular beams. Where splices are required in the rafter (for transport), they should be carefully detailed, to preserve the architectural features for this form of construction.

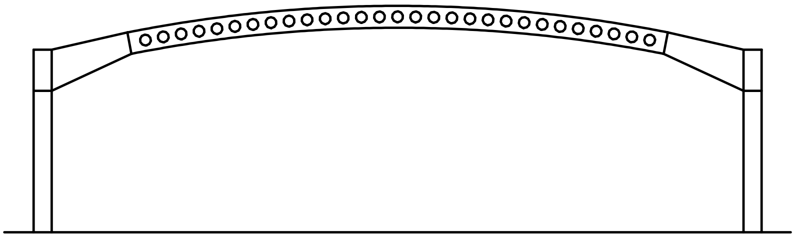


Figure 2.10 *Cellular beam portal frame*

Many cellular beam portal frames in the span range of 40 m to 55 m have been constructed; greater spans are possible.

Elastic design is used because the sections used cannot develop plastic hinges at a cross-section, which is an essential criterion for plastic design.

2.11 Gable wall frames

Gable wall frames are located at the ends of the building and may comprise posts and simply-supported rafters rather than a portal frame (Figure 2.11). If the building is to be extended later, a portal frame of the same size as the internal frames is preferred.

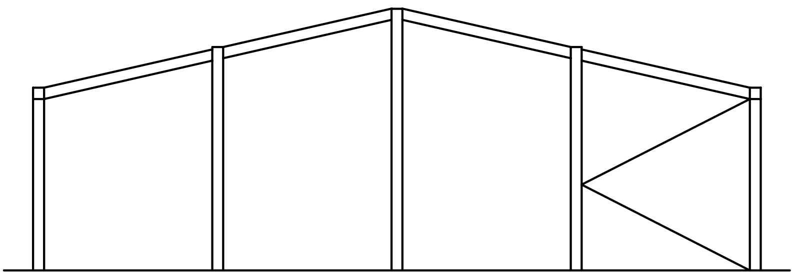


Figure 2.11 *Gable frame to a portal frame structure*

The in-plane stability of the lightweight gable frame is usually provided by bracing between the gable posts. The out-of-plane stability is provided by bracing in the roof and in the walls, which is normally designed to resist the wind loads on the end of the building.

The gable posts are commonly designed as simply-supported members spanning between the foundation and the horizontal plan bracing in the roof. The rafters are usually designed as simply-supported beams spanning between the tops of the posts. They also support vertical loads from the purlins in addition to compression from their function as a chord of the plan bracing system. More detailed consideration of gable frames is given in Section 10.

2.12 Hipped roof frames

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 penultimate frame. This usually leads to an end roof pitch that is steeper than the general roof pitch (Figure 2.12).
- A *gablette* feature can be introduced, where the inclined rafters meet below the apex of the penultimate frame (Figure 2.13). This reduces the pitch of the hip.
- The rafters can be arranged at an angle of 45° to create a roof pitch equal to the main roof pitch. In this case, the spacing between the penultimate frame and the end frame is equal to half the span and intermediate frames may be required; these can take the form of a flat-topped mansard portal (Figure 2.14).

In all these cases, the end frame will usually be a simple braced rafter and post frame.

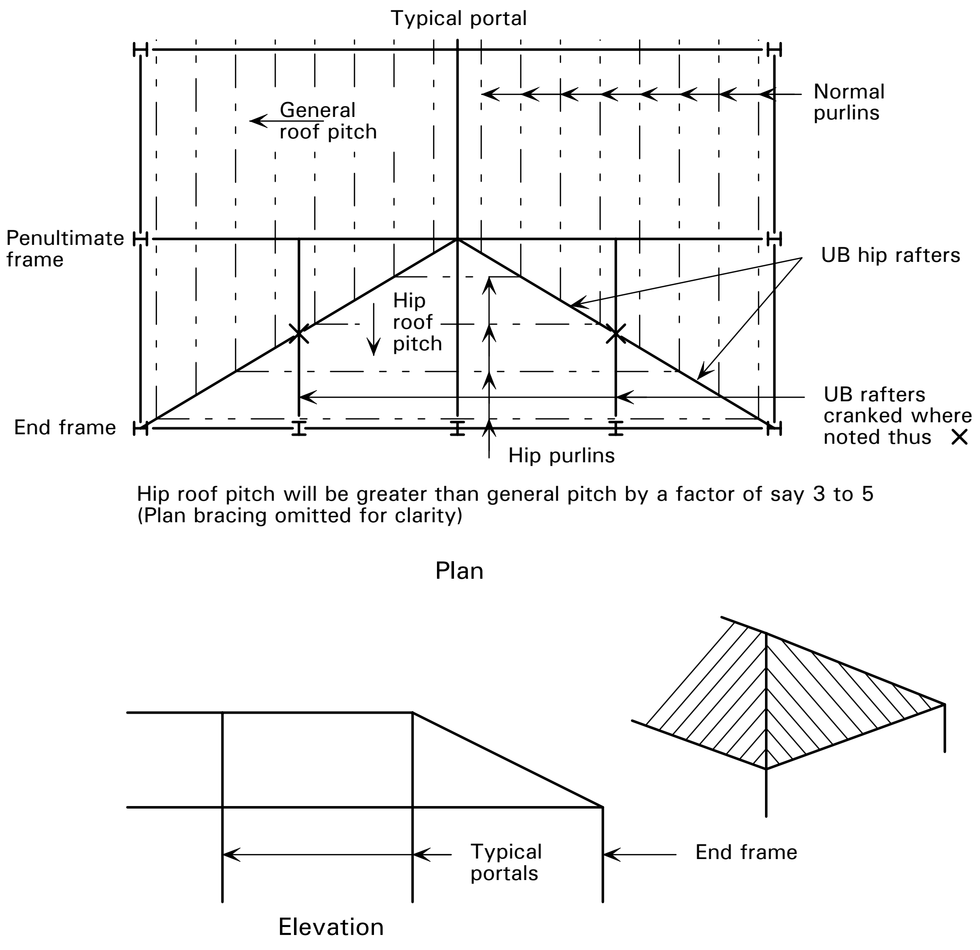


Figure 2.12 *Framing for hipped roof with a hip roof pitch greater than that of the main roof*

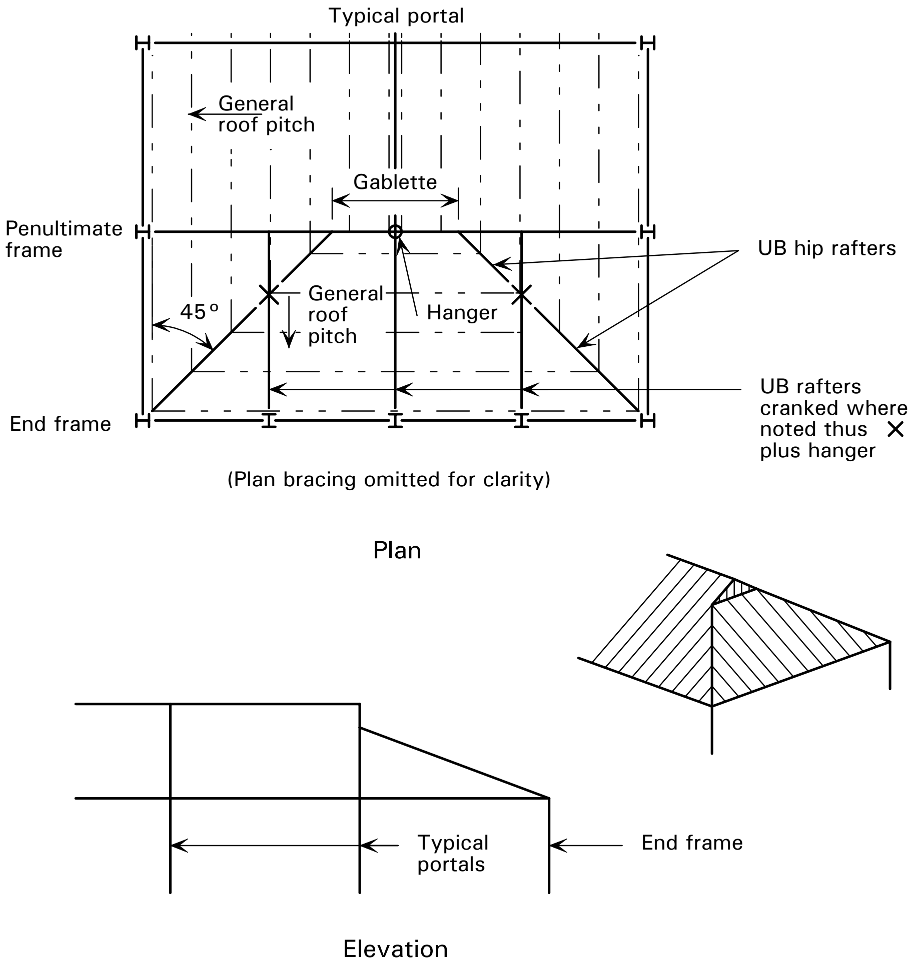


Figure 2.13 *Framing for hipped roof with a hip roof pitch equal to that of the main roof, with a gablette feature*

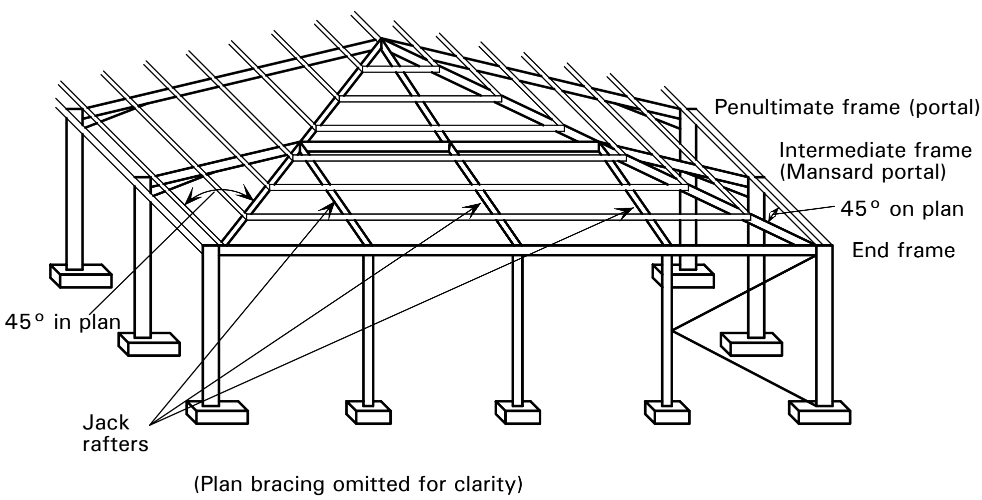


Figure 2.14 *Framing for hipped roof using a flat-topped mansard as the penultimate frame*

3 DESIGN CONSIDERATIONS

3.1 Introduction

3.1.1 General

In the design and construction of any structure, a large number of inter-related design requirements should be considered at each stage in the design process. There are three stages of design:

- Conceptual design, which refers to the stage at which decisions are made about the overall dimensions and form of the structure.
- Preliminary design, during which members are sized approximately for estimating purposes.
- Final design, during which all relevant load cases are considered, detailed checks are carried out on the members, the positions of the restraints are finalised, and the connections are designed.

In practice, there is seldom a clear distinction between each stage of the design. The conceptual and preliminary designs are often developed together as approximate member sizes are determined and the concept design is improved.

Typical details of a steel portal frame structure are shown in Figure 3.1. The following discussion of its constituent parts is intended to give the designer an understanding of the inter-relationship of the various elements with the final construction, so that the decisions required at each stage in the design process can be made with an understanding of their implications.

3.1.2 Steel grades

Steel sections used in portal frame structures are usually specified in grade S275 or S355 steel. Use of S355 steel is rarely economic in structures where serviceability (i.e. deflection) criteria control the design.

The steel toughness quality, e.g. JR, J0, J2, is determined in accordance with BS 5950-1 Clause 2.4.4.

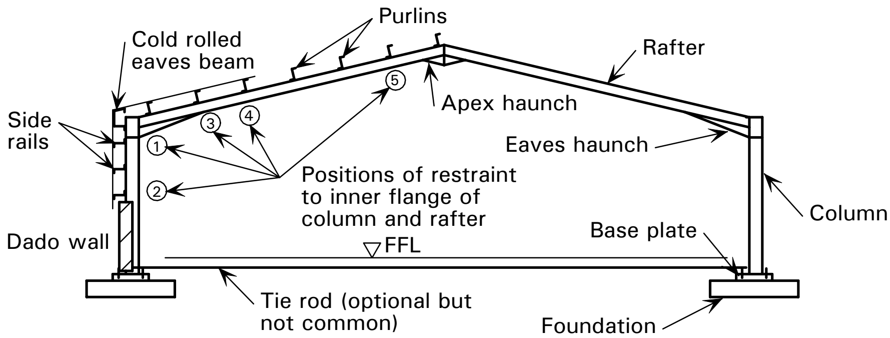
3.1.3 Cross-section restrictions

In plastically designed portal frames, *Class 1 plastic* sections must be used at hinge positions that rotate, *Class 2 compact* sections can be used elsewhere.

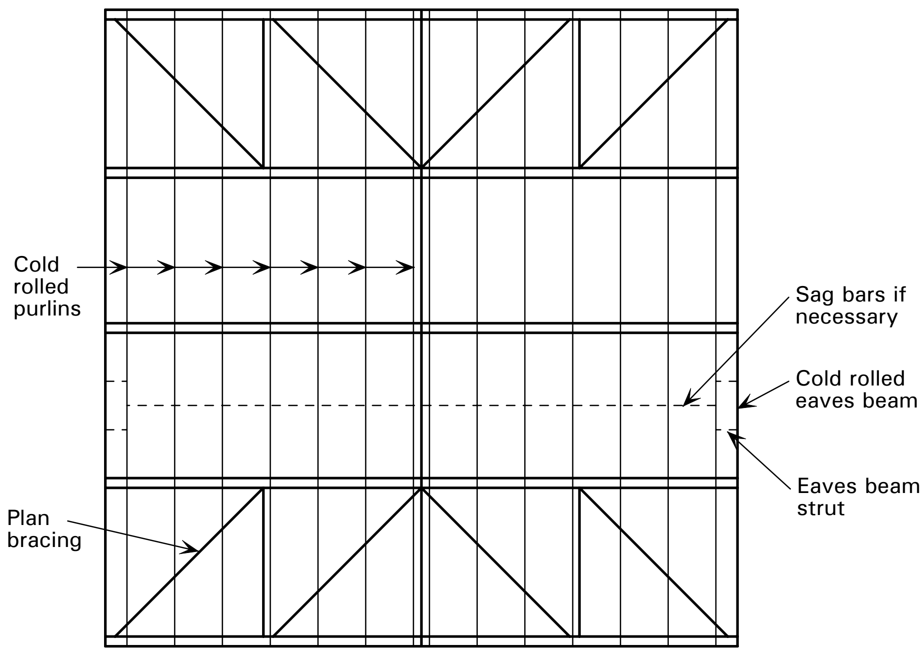
Not all Universal Beam (UB) sections in grade S355 under pure bending are *Class 1 plastic*.

All UB sections in grade S275 steel under pure bending are *Class 1 plastic*.

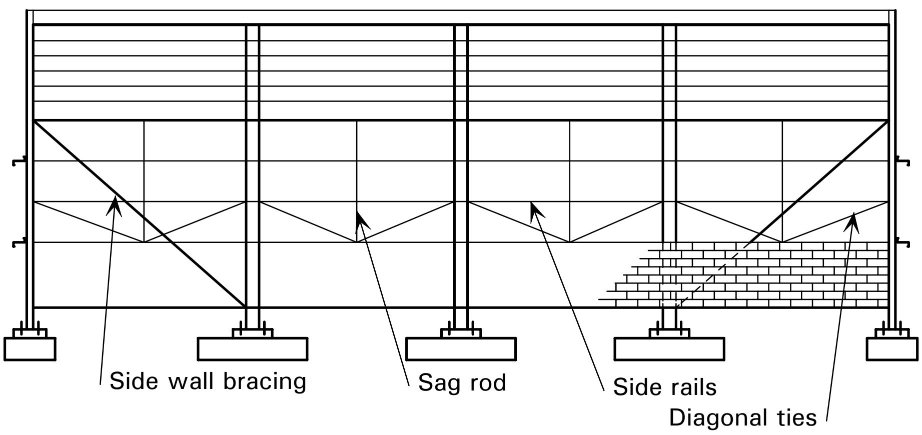
The effect of axial load on the classification of members should be considered. However, in many members, the axial force is so small compared with the bending moment that the classification is not affected.



(a) Cross-section showing the portal frame and its restraints

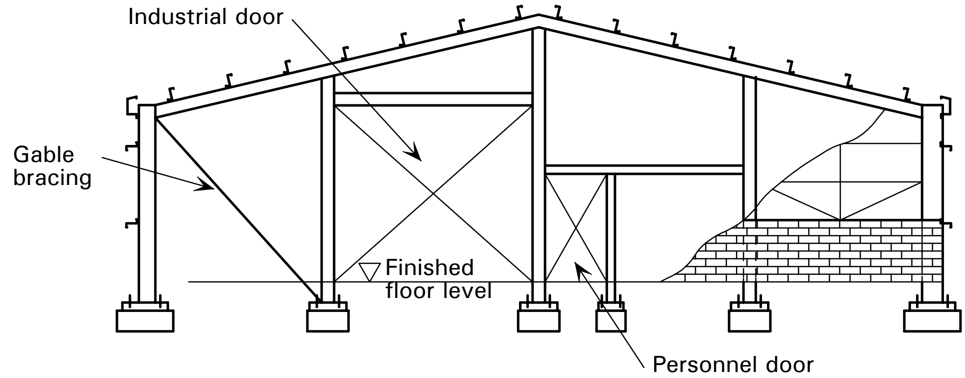


(b) Roof steelwork plan

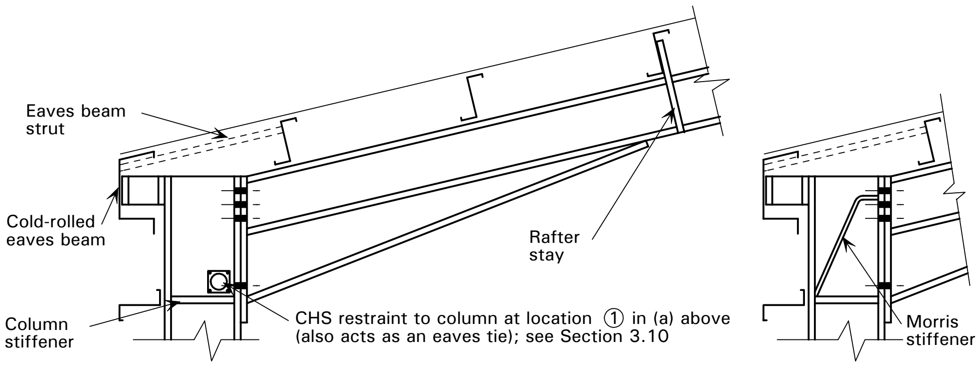


(c) Side elevation

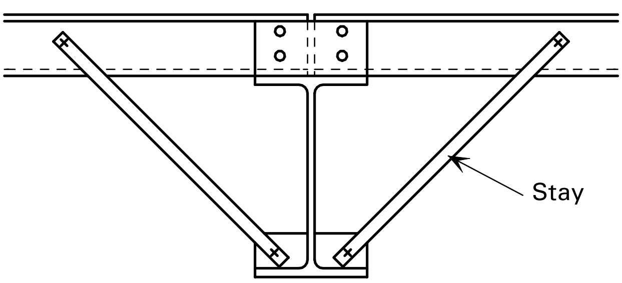
Figure 3.1 *Typical details of a building using a portal frame structure (continued)*



(d) Gable frame



(e) Eaves detail



(f) Detail of column and rafter stays, at locations ② ③ ④ and ⑤ in (a) above

Figure 3.1 *Typical details of a building using a portal frame structure (concluded)*

3.2 Frame dimensions

A critical decision at the conceptual design stage is the overall height and width of the frame, to give adequate clear internal dimensions and adequate clearance for the internal functions of the building. Accurate 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.

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

The calculation of the height to eaves for analysis should allow for:

- The distance from the top of the foundation to the finished floor level.
- The clear internal height.
- The depth of the haunch.
- Half the depth of the rafter (if the analysis is based on centre-line dimensions).

Great care is needed to define the depth of the haunch because there are at least three different uses of the terms "depth of the haunch":

- The depth from the intersection point of the centre-line of the rafter and the column to the bottom of the haunch (used for some software input) (Figure 3.2).
- The depth from the underside of the rafter to the bottom of the haunch (Figure 3.2). This is usually used by steelwork contractors to specify the *cutting depth* of the haunch and is defined as D_h in BS 5950-1:-2000 Figure 17.
- The depth from the top of the rafter to the bottom of the haunch (Figure 3.3).

Similarly, the length of the haunch may be defined either from the centre-line or from the face of the column. The depth of the haunch below the rafter (i.e. the cutting depth) would then, in most cases, be taken as being equal to the rafter depth less the thickness of one flange and the root radius (but see Section 3.4).

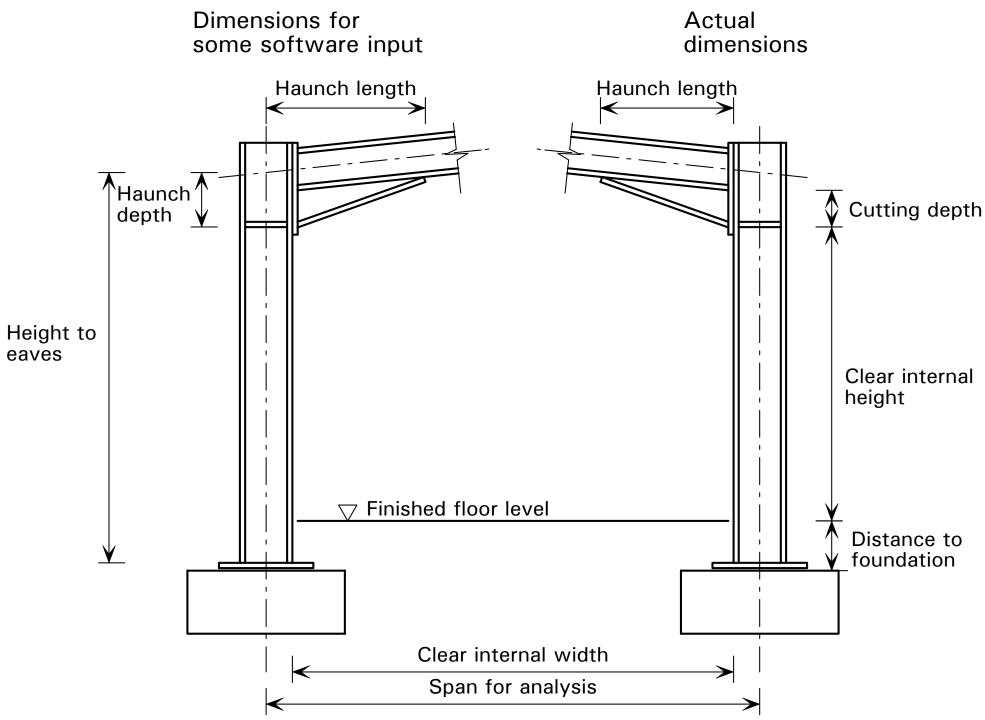


Figure 3.2 *Dimensions used for analysis and clear internal dimensions*

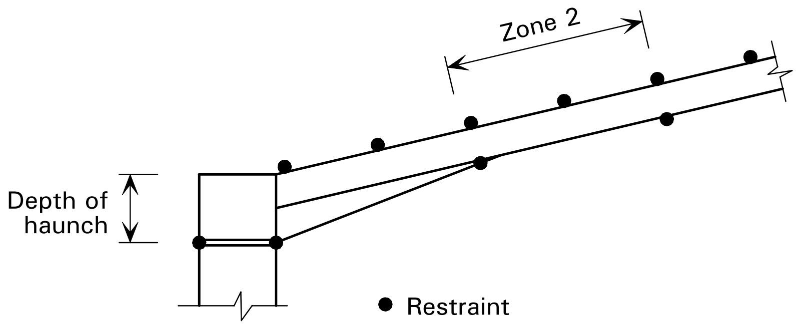


Figure 3.3 *Typical restraint to rafter adjacent to haunched region of a portal frame*

3.3 Rafters and columns

Rafters and external columns are usually chosen from the range of Universal Beam sections because the dominant load effects are those due to bending rather than axial load. Sizes can be estimated using the methods in Appendix A. Fabricated sections can be used for long spans or unusual loading conditions, and are used in a number of proprietary portal frame systems.

Rafter and column sections are normally selected according to their cross-sectional resistance to bending moment plus axial force. The necessary restraint positions are then calculated (Figure 3.3). However, where lateral restraint is not possible, for example where there are doors between all the columns, member stability will determine the section size.

3.4 Eaves haunch

The eaves haunch is required to:

- Supplement the bending resistance of the rafter in the area of highest moment, permitting a smaller rafter to be used.
- Provide adequate depth at the rafter/column interface to achieve an efficient connection. The haunch depth is often determined by the lever arm to the bolts required to achieve the necessary moment capacity.

The eaves haunch can be cut from a hot rolled section or fabricated from plate. Cuttings from rolled sections are generally preferred, and it is convenient to use a similar section to the column or rafter, although the actual size could be dictated by stability and connection considerations. If the chosen rolled section is not deep enough to provide sufficient haunch depth, an infill plate can be used.

It should be recognised that if the haunch is made from the same section size and weight as the rafter, it may not always be possible for the connection to achieve its required bending resistance. In terms of connection design, a deeper and heavier haunch may be more suitable. This will reduce the tensile force in the bolts and the force in the compression zone at the bottom of the haunch, and will therefore reduce the bolt sizes and stiffening requirements. It will also reduce the shear force (caused by the tension in the bolts) in the top of the column. Increasing the weight of the section from which the haunch is cut also increases the stability of the haunch.

The haunch may affect the overall height of the structure because clients may require a clear depth to the underside of the haunch. From this point of view it is important to minimise the haunch depth. It is generally agreed that the haunch will usually be most efficient in terms of the overall frame design if:

- The depth of haunch below the rafter is approximately equal to that of the rafter.
- The length of the haunch from the centre-line of the column is approximately 10% of the span of the portal frame.

There are at least two different definitions of the term “length of the haunch” in common use. One is the length from the point of intersection of the centre-lines of the column and rafter to the end of the haunch. Another is from the face of the column to the end of the haunch (see Figure 3.2).

3.5 Apex haunch

The purpose of the apex haunch is to achieve an efficient connection between the rafter members. It will usually be fabricated from plates and its detailed design will be part of the connection design. The size and details need not usually be considered at the preliminary design stages.

3.6 Base plates and foundation

At the conceptual design stage, it is necessary to decide the in-plane rotational stiffness of the column bases. Unless there are good reasons, such as the need to restrict deflections, a nominally pinned base is usually provided. This leads to a smaller base plate connection and concrete foundation than would be the case for a nominally rigid base.

The base plate and the foundation will generally be analysed as a pinned connection at the base of the column even when four bolts are provided for stability during erection. BS 5950-1 does allow some degree of base fixity to be taken into account, which can provide a significant benefit in terms of stability and deflection criteria. This aspect is dealt with in more detail in Section 12.

If the external wall is close to a boundary, the size of the base plate, bolts, and concrete foundation may have to be increased to resist the moment due to collapse of the rafters in a fire (Sections 12 and 15).

3.7 Positions of restraint to the column and rafter

Under vertical loading, the internal flanges of the column and lower part of the rafter near the eaves will be in compression, and restraint will be required at intervals to both flanges of the rafter and both flanges of the column. Strategically-located purlins and sheeting rails are used as part of the restraint system. Purlins and side rails will normally be able to provide the required restraint but, where they are not able to do so, the rafter or column size may have to be increased (based on stability checks at the detailed design stage). At the conceptual and preliminary design stages, the following issues should therefore be considered:

- Are the purlins and side rails large enough in proportion to the rafter and column to provide restraint?
- Are the purlins and side rails adequately restrained by the sheeting or an independent system of bracing?
- Are the purlins and side rails tied into the roof bracing and vertical bracing respectively?
- Are there side rails present in a position to provide restraint to the columns, or are there large openings in the side of the structure?
- How is the bottom flange of the eaves haunch restrained?

Where a plastic hinge occurs in the frame it is necessary to provide lateral restraint to both flanges. Restraint options are discussed in Section 8.2.1.

Restraints are also required at other locations where the compression flange is not attached to purlins or sheeting rails, and restraints can be provided by column and rafter stays. For example, restraints will usually be required to the bottom flange of the rafter near the apex (position ⑤ in Figure 3.1(a)) in order to restrain the flange in the wind uplift case. Further details are given in Section 13.

It is important that the bottom flange of the eaves haunch is restrained at the intersection with the column. Where a restraint is not provided, it must be demonstrated that the column is not destabilised in the lateral torsional buckling mode by the compression force in the bottom flange of the haunch

3.8 Dado masonry wall

A dado masonry wall up to a height of 2.5 m is often provided along the perimeter of the structure. This wall can have a significant effect on the design of the structure, owing to:

- The need to comply with deflection criteria compatible with masonry construction.
- The influence of the wall on the location of vertical bracing and on the detailing of restraints.

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 formed C or Z section. Details of methods of interfacing the masonry and the steel structure are given in *Brick cladding to steel framed buildings*[4].

The location of vertical bracing relative to the masonry wall should be established at the conceptual design stage in order to ensure that adequate space is available for the bracing.

3.9 Bracing

Bracing is required both in the plane of the rafters, and vertically in the side walls (see Figures 3.1(b) and (c)) in order to provide:

- Stability, both during erection and in the completed building.

- Resistance against wind loading in the longitudinal direction.
- An adequate anchorage for the purlins and sheeting rails in their function of restraining the rafters and columns.

It is common practice to provide bracing in the vertical plane at each end of the building. The location of the bracing will not generally affect the preliminary design of the frame. Where there are large openings in the side of the building, it may be necessary to provide flat-topped portal frames (wind portal frames) instead of diagonal bracing in the vertical plane. The stiffness and stability of these wind portal frames should be carefully considered as indicated in Section 9.2.6. The inclusion of such wind portal frames may restrict the width of the door opening, and this should be considered at the conceptual design stage. Further details of various forms of bracing are given in Section 9.

3.10 Eaves strut/tie

An eaves strut/tie, is normally located between the flanges of the column at eaves level, and is required to transmit longitudinal wind forces along the structure from the plan bracing in the roof to the vertical bracing in the walls. It may also be required to provide out-of-plane stability to the frames which are not directly attached to the bracing. A circular hollow section is most efficient in this application (see Figure 3.1 (e)).

Where vertical bracing is provided at both ends of the structure in the same bay as the plan bracing, an eaves strut is not required to transmit wind loads as the bracing members are normally capable of resisting tension and compression. It is normal practice to provide a hot rolled steel section between the columns to act as a tie.

Normally it is necessary to have an eaves strut/tie throughout the structure to provide stability during erection. It is generally preferable that this is a hot-rolled member so that it can be supplied by the fabricator at the same time as the columns.

3.11 Eaves beam

Eaves beams are usually made from cold formed sections (see Figure 3.1(e)). Their primary function is to support the roof sheeting, side walls, and guttering along the eaves. Further details are given in Section 13.

3.12 Eaves beam strut

The eaves beam strut (see Figures 3.1(b) and (e)) has a moment resisting connection into the eaves beam and a shear connection to the adjacent purlin. It prevents the eaves beam from twisting under the eccentric load of the guttering etc.

3.13 Connections

The connections at the eaves and apex are required to be moment-resisting connections and to provide both adequate stiffness and bending resistance. The design of the connections need not be carried out in detail at the preliminary design stages, although it will be necessary to provide a connection of adequate depth to resist the applied moment. Also, the column must resist the shear induced by tensile loads in the bolts at the top of the haunch. In some cases, strengthening of the web may be needed. The design of these connections is considered in Section 11.

3.14 Column stiffener

A stiffener if required should be provided on each side of the column web at the base of the haunch to prevent web bearing and/or buckling failure of the column under the compression force from the bottom flange of the haunch.

3.15 Purlins and side rails

Generally the purlins and side rails are cold formed sections that are available from a wide range of manufacturers. They are required to:

- Support the roof and side wall sheeting.
- Provide restraint to the rafters or columns.

Purlins will usually be continuous over the rafters (see Section 13 for typical layouts). However, the purlins may be discontinuous simple spans in some places, for example, at a hipped roof (Figure 3.4). The frame layout should be chosen to ensure that the purlin size is not dictated by a few long or single spans of purlins.

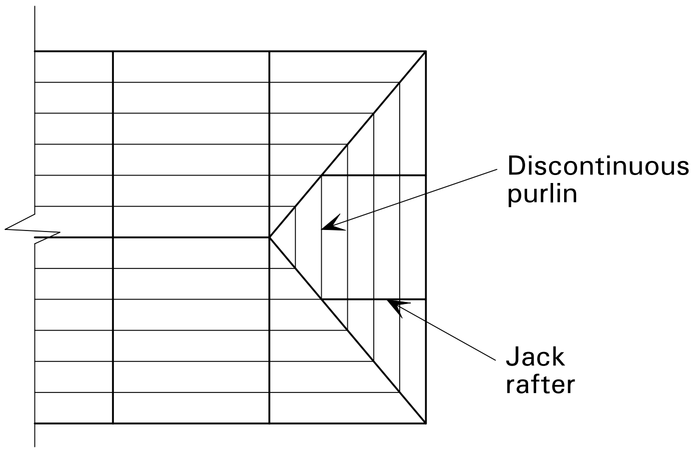


Figure 3.4 *Single-span purlins at hipped end*

3.16 Horizontal forces at the column bases

Portal frames develop large horizontal forces at the bottom of the columns. These forces can be resisted in a number of ways:

- By passive pressure of the soil against the foundation.
- By tying the column into the floor slab.
- By providing a tie across the full width of the building.

A more detailed consideration of these options is given in Section 12.3. The provision of the horizontal restraint may affect the preliminary design if there are other non-structural factors to be taken into account, such as drainage runs (which may affect the development of passive pressure in the soil).

3.17 Vehicular impact

Portal frames are not particularly good at resisting local impact loads (unless specifically designed to do so), owing to the modest size of the members. However, the structure should be protected against forces that may forseeably occur, for example, impact from vehicles. Such protection could be by the means of bollards standing on independent foundations, or by reinforced concrete casing around the columns to at least 1.5 m height.

Other measures to provide structural integrity are discussed in Section 4.8.2.

3.18 Design summary

Decisions about the conceptual and preliminary design of steel portal frames will generally be taken in the following order:

- Generate a conceptual design of the frame, including the critical dimensions (clear spans and heights), and define the constituent parts of the frame (see Sections 3.1 and 3.2).
- Carry out preliminary sizing of the members using Appendix A.
- Select the depth of the eaves haunch below the rafter, based on the size of the rafter (generally between 1 and 1.5 times the rafter depth) (see Section 3.4).
- Check the clear spans and heights based on these preliminary sizes.
- Select the length of the eaves haunch (generally 10% of the span from the centre-line of the column).
- Determine the base fixity - the base will generally be assumed to be nominally pinned (see Section 3.6).
- Determine the approximate position of purlins and side rails to support the cladding and to provide member stability (see Section 3.7).
- Determine the location of walls and the position of vertical bracing (see Sections 3.8 and 3.9).
- Choose other secondary components, such as eaves strut/tie (see Sections 3.10 to 3.14).
- Determine the means of resisting horizontal base forces (see Section 3.15).
- Consider the requirements for any accidental loads (see Section 3.16).

4 LOADING

The loads and load combinations described in this Section should be considered in the design of a steel portal frame. Imposed, wind, and snow loads are given in BS 6399-1 to BS 6399-3[5].

4.1 Dead loads

Where possible, unit weights of materials should be obtained from manufacturers' data. Where information is not available, basic unit weights of materials can be obtained from BS 648[6]. Alternatively, the figures given in Table 4.1 may be taken as typical of roofing materials used in portal frame construction. The self weight of the steel frame is typically 0.2 to 0.4 kN/m^2, expressed over the plan area.

Table 4.1 *Typical weights of roofing materials*

Material	Weight (kN/m^2)
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
Purlins (distributed over the roof area)	0.03
Steel decking	0.2
Three layers of felt with chippings	0.29
Slates	0.4/0.5
Tiling (clay or plain concrete)	0.6 - 0.8
Tiling (concrete interlocking)	0.5 - 0.8
Timber battens (including timber rafters)	0.1

4.2 Service loads

Loading due to services will vary greatly, depending on the use of the building. In a portal frame structure, heavy point loads may occur from such items as suspended walkways, air handling units, and runway and lifting beams. In certain situations, it may be more appropriate to use a truss or lattice girder, rather than a portal frame, to support heavy local loads.

At the preliminary design stage:

- Service loading is to be taken as a dead load according to BS 6399-1.
- Assume a service loading over the whole of the roof area of between 0.1 and 0.25 kN/m^2 on plan, depending on the use of the building and whether or not a sprinkler system is provided.

- Recognise that some dead load may be removed in the life of the structure and, where service loads have a beneficial effect in opposing wind uplift, use no more than 0.15 kN/m^2.
- Identify other sources of loading.

At the final design stage, the structure should be checked for the precise service loads, if known. Where the specified service loads are of the order of 0.5 kN/m^2 or more, it is probable that the attachment loads will exceed the capacity of some proprietary attachment systems.

4.3 Imposed roof loads

BS 6399-3 defines six types of imposed roof load:

- A minimum load of 0.6 kN/m^2 (on plan) for roof slopes less than 30°, where no access other than for cleaning and maintenance is provided.
- A concentrated load of 0.9 kN - this will only affect sheeting design.
- A uniformly distributed load due to snow over the complete roof area. The value of the load depends on the building's location and height above sea level.
- Asymmetric snow load due to *major* redistribution of snow. This occurs when the wind transports snow from the windward to the leeward roof area. This load condition only applies to roof pitches greater than 15° (see Clause 7.2.3.3 of BS 6399-3).
- Asymmetric snow load due to *artificial* redistribution of snow. This occurs as a result of excessive heat loss through a small section of the roof or manual clearance of snow to maintain access to a service door (see Clause 4.5 of BS 6399-3).

  Generally, industry practice is *not* to consider this type of loading except where specifically requested.
- Non-uniform loads caused by snow drifting in areas of obstruction or abrupt changes in height, such as parapet walls or walls of adjacent buildings. The magnitude of this loading can be much larger than the uniform snow loading, but is localised and is more likely to affect the purlins and sheeting than the overall design of the structure. Snow drifting is regarded by BS 6399-3 as an exceptional load and it should be assumed that there is no snow on the rest of the roof. Reduced load factor of 1.05 may therefore be used (see Section 4.9). Frames with abrupt changes in roof height may be affected by this condition.

A detailed explanation of the background and use of BS 6399-3 is given in the BRE *Handbook of imposed roof loads*[7].

4.4 Wind loads

4.4.1 General

It is uncommon for load combinations including wind to determine the size of members in low-rise single-span portal frames. Therefore, wind loading can usually be ignored for preliminary design, unless the height/span ratio is large (greater than say 0.5) or if the dynamic pressure is high (in the North of the UK and at higher altitudes, or where dynamic pressure is increased due to the local topography).

However, in two-span and other multi-span portal frames, load combinations including wind may often determine the sizes of the members when using the pressures from BS 6399-2.

The following points should be noted:

- Deflection due to wind loading should be considered for the preliminary design at the serviceability limit state if:
  - the portal frame supports an overhead travelling crane, or
  - masonry or other relatively brittle wall construction is used.
- Wind uplift may be important in terms of rafter stability but, provided that adequate restraint can be provided to stabilise the bottom flange of the rafter near the apex (see Section 7.3), it need not be considered at the preliminary design stage.
- For wind blowing across the portal frame, the external pressure on the windward rafter can be positive according to Figure 20 and Table 10 in BS 6399-2 and this might become a critical design case. However, with the limitations given in Advisory Desk Note AD 273[9] it is not necessary to check this design case at SLS (see also Section 14.2).

Wind loads should be determined from BS 6399-2.

BS 6399-2 provides three methods for calculating the wind loading: the Standard method, the Directional method and the Hybrid method (combining the Standard and the Directional method). The Directional method is suited to computer calculation rather than manual methods.

The Standard and Hybrid method are suitable for hand calculations. The SCI publication P286 *Guide to evaluating design wind loads to BS 6399-2:1997*[8] describes two further methods - the Simplified Standard Method and the Simplified Hybrid Method. Both of these methods reduce the required calculation effort but can result in higher estimates of wind speeds, compared to the respective Standard and Hybrid methods.

The four hand methods are summarised below, in order of increasing calculation effort and decreasing potential conservatism:

- Simplified Standard Method – where the most onerous values for all the factors that affect the effective wind speed are assumed, irrespective of direction. One value of effective wind speed is calculated and is applied to all orthogonal directions of the building. This method will generally produce higher estimates of wind pressure compared to other design methods, but requires the least amount of calculation effort.

- Standard Method – Where the effective wind speed in each orthogonal direction is calculated by considering the most onerous value for all the factors that affect the wind speed over a range ±45° either side of the orthogonal directions. If the orientation of the building is unknown, the most onerous value for the direction factor is assumed.
- Simplified Hybrid Method – Where the most onerous values for the factors that affect the wind speed are used to determine the directional effective wind speed (using the approach in the Directional Method) over four 90° directional ranges (quadrants). The method increases the calculation effort slightly compared to the Standard Method, but can significantly reduce the conservative estimates of wind speed obtained when using the Standard Method.
- Hybrid Method – Where the directional effective wind speed is calculated (using the approach in the Directional Method) in twelve directions at 30° intervals. The highest effective wind speed in the range ±45° either side of the orthogonal directions is used with the corresponding standard pressure coefficients.

Using one method for all design situations is not practicable. To obtain the optimum method for individual designs, consideration should be given to:

- Time-scale for completion of calculations.
- Capital cost of the project.
- Influence of the terrain, topography, obstructions and distance from the sea and how these vary with wind directional factors (S_d) in different directions around the site.
- Accuracy of the available information that defines the terrain, topography, obstructions and distance from the sea in each direction around the site.
- For 'back-of-the-envelope' calculations, the Simplified Standard Method can be used. it may also be argued that this level of calculation is all that is required for buildings with low capital costs.

If the availability or accuracy of information about terrain, topography, obstructions and distance from the sea in each direction is poor, and conservative estimates have to be taken, then the use of detailed calculations may not be justified.

Although the choice of the optimum method is site specific, a suitable balance between calculation effort and determining economical estimates of the wind pressures can be obtained by using the Simplified Hybrid Method.

Further guidance on the use of BS 6399-2:1997 is given in BRE Digest 436[10], *Recommended application of BS 6399-2*[11] and, *A practical guide to BS 6399-2, Wind loads on buildings*[12].

Three aspects of BS 6399-2 have particular relevance for portal frames.

Dominant openings

Clause 2.6.1.3 of BS 6399-2 specifies that if dominant openings are considered shut for ULS design, a serviceability load case should be considered with the dominant opening open. SCI publication P286[8] clarifies that this should be an accidental loadcase, the dynamic pressure should be recalculated, using $S_p = 0.8$. In this accidental loadcase, all load factors should be taken as 1.0.

Internal pressure coefficients

BRE Digest 436 advises that the positive value of $C_{pi} = 0.2$ is less likely to be a critical design case, as the circumstances giving rise to this possibility involve a permeable front face and impermeable back and side faces. Further advice in BRE Digest 436 is that the internal pressure coefficient for completely clad enclosed warehouse-type buildings without opening windows may be taken as $C_{pi} = -0.3$.

Diagonal dimension a

The size effect factor, C_a, serves to reduce surface loads if load-sharing takes place over a large area. Diagonal dimension, a, is the length of the diagonal of this area. Load sharing between portal frames takes place if these are braced together or, perhaps, by stressed skin action of the roof cladding, incorporating appropriate components and fixings. If loadsharing is assumed to take place over more than one frame, the designer should make sure that means are in place to ensure the assumed loadingsharing is realised.

4.4.2 Minimum wind load

BS 5950-1 Clause 2.4.2.3 required that the factored wind load should not be taken as less than 1% of the factored dead load applied horizontally. This is to provide a practical level or robustness against the effects of incidental loading even in the foundations. As the specified loads from overhead travelling cranes already include significant horizontal loads, it is not necessary to include vertical crane loads when calculating the minimum wind load.

4.5 Crane loads

Cranes impose both vertical and horizontal loads on the structure; part of the loading is due to dynamic effects. Consideration should be given to the following loads:

- The vertical load will be composed of a load due to the weight of the crane bridge, crab, hook and the weight of the lifted load. All these loads should be treated as imposed loads (see Advisory Desk Note AD 121)[13].
- The horizontal loads due to crane surge and crabbing will not necessarily be coincident with the maximum reaction from a wheel, as surge and crabbing could occur when the crab is at the centre or the opposite side of the structure. The distribution of horizontal forces between the rails at each side of the frame depends on whether or not the wheels are double flanged. With single-flanged wheels, the horizontal reaction is supported by one rail alone.

4.6 Fire loading

The fire limit state should be regarded as an exceptional situation, so reduced load factors can be used; see BS 5950-8[1] and the advice given in *Single storey steel framed buildings in fire boundary conditions (*P313)[14].

For design at the fire limit state (see Section 15), it can be assumed that the imposed load will not be present on the roof and that the dead load is reduced considerably by the action of the fire. The SCI publication P313 provides a table of materials with the percentage that can be assumed in this condition.

4.7 Notional horizontal forces

Notional horizontal forces are defined in BS 5950-1, Clause 2.4.2.4, and should be applied to all structures for Ultimate Limit State checks. They allow for the effects of practical imperfections such as lack of verticality of the columns, and to provide a certain degree of robustness in structures with low wind loading. They should **not** be confused with the notional forces used to check the stability of rigid frames, which are discussed in Section 6.

In a portal frame, the notional horizontal forces are to be taken as a horizontal force applied at the eaves in any one direction at a time* equal to 0.5% of the factored vertical dead and imposed loads. They should be included in load combination 1, as required by Clause 2.4.1.2 (Dead and Imposed (gravity loads)).

* For asymmetric frames, the notional force should be considered in both directions (one at a time) because, in each direction, it will have a relieving effect on the maximum moment in certain columns.

These notional forces **should not** be:

- Applied when considering overturning.
- Applied when considering pattern loads (unlikely to be applicable in a portal frame).
- Combined with actual applied horizontal loads.
- Combined with temperature effects.
- Taken to contribute to the net reaction at the foundations. However, where base fixity is assumed in checking adequacy at ULS under loading that includes the notional horizontal forces, the moments on the individual bases should not be ignored.

As the specified loads from overhead travelling cranes already include significant horizontal loads, any crane loads should not be included when calculating notional horizontal forces.

In most cases, notional forces will have only a very small effect on the design of a single span portal frame, so can be neglected for preliminary design.

When resistance to horizontal loading is assumed to be provided by a structural element other than the steel frame, the steelwork design should clearly indicate the need for such construction and state the forces acting on it.

4.8 Accidental loads

Two kinds of accidental loads require consideration:

- Loads that may actually occur in practice and are specified in the client's brief; these will generally be some form of impact or unusual loading.
- Loads that are specified in Regulations and standards in terms of structural integrity.

4.8.1 Impact loading

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.

4.8.2 Disproportionate collapse

Approved Document A[16] deals with the requirements which are contained in the Building Regulations[15].

Requirement A3 of the Regulations states that "The building shall be constructed so that in the event of an accident the building will not suffer collapse to an extent disproportionate to the cause."

Until the 2004 Edition of Approved Document A, requirement A3 only applied to buildings having five or more storeys, therefore it did not apply to single storey structures covered by this publication. However, under the 2004 Edition of Approved Document A, the application limit to five storeys has been removed so as to bring ALL buildings (including portal frames) under the control of A3 Requirement. The new guidance includes a system of classifications of buildings to determine requirements for different types of buildings and different types of use. Therefore, it is important for designers to check that they comply with the latest relevant requirements when designing a new building or when considering changes of use of a building.

It is expected that BS 5950 1:2000 will be amended in accordance with the detailed guidance in the 2004 Edition of Approved Document A.

(*For latest revisions and amendments to the Building Regulations, the reader is advised to check the information on ODPM website: www.odpm.gov.uk and The Stationery Office website: www.tso.co.uk*)

4.9 Load factors and load combinations

BS 5950-1: 2000 gives the load factors and the load combinations for ULS in Clause 2.4.1 and for SLS in Clause 2.5.

Portal designers have for many years used their engineering judgement to interpret the concept in the second sentence of BS 5950-1:2000 Clause 2.4.1.1:

"The factored loads should be applied in the most unfavourable *realistic combination* for the part or effect under consideration." (italics and underlining do not appear in the code).

This sentence is used as the most important statement on loading and is used to interpret how the Load Combinations 1, 2 & 3 of BS 5950-1:2000 Clause 2.4.1.2 "should be taken into account" for different buildings. It is Load Combination 3 that appears to have a wide range of interpretations. These arise from the improbability of having much or any imposed load on a roof when the wind is at its maximum.

- Firstly, it is difficult to imagine a uniformly distributed load (including maintenance load as specified in BS 6399-3, clause 4.3) of 0.6kN/m^2 (for roof with 'no access') over more than a small proportion of a large portal roof at any one time. It is even more difficult to imagine it being over an entire roof with the wind at maximum speed. It is improbable that the roof will be maintained or cleaned during a windstorm.

- Secondly, it is difficult to imagine that snow will remain on a roof when the wind is at maximum speed; bearing in mind that snow drifting is an alternative design case.

There are a variety of opinions about what should be the appropriate loads, load combinations and load factors for both SLS and ULS for low-rise frames.

It is understood that there already are numerous low-rise frames that are in service that have been designed for less onerous loadings in that the combination including both Imposed and Wind was judged not to be a "*realistic combination* for the part or effect under consideration".

As an initial step at providing uniform guidance, it is suggested that the load factors and combinations given in Table 4.2 and Table 4.3 below be adopted.

In most situations, it is unlikely that all the combinations need be checked. Those load cases that are most likely to govern are shown in bold, and preliminary design should be carried out for these cases. Notional horizontal forces can usually be neglected for preliminary design, as they have such a small effect on single-storey/single-span structures.

Table 4.2 *ULS Load factors and combinations for frames without cranes*

Ultimate limit state load	Load factors for different load combinations								
	BS 5950 Clause 2.4.1.2 Load Combination								
	(1)				(2)		(3)		
Dead	**1.4**	1.4	1.4	1.4	1.4	**1.0**	1.2	1.2	1.2
Imposed									
Uniform snow	**1.6**						1.2		
Asymmetric snow[1]		1.6							1.2
Drifted snow			1.05[4]					1.05[4]	
Minimum Imposed[2] (Including maintenance)				1.6					
Real & definable[3]	**1.6**	1.6	1.6	1.6			1.2	1.2	1.2
Wind					1.4	**1.4**	1.2	1.2	1.2
Notional horizontal	**1.0**	1.0	1.0	1.0					

1 Only applies for roof pitches greater than 15° (see BS 6399-3:1988 Clause 7.2.3.3)

2 For roofs with 'No access' (i.e. access for cleaning and repair only) UDL of 0.6 kN/m^2 or point load of 0.9 kN.

For further details see BS 6399-3:1988 Clause 4.3

3 Any additional potential imposed roof loads not specifically included in the above, e.g. suspended platform or walkway etc.

4 Consider this as exceptional snow load (see BS 6399-3:1988 Clause 7.4.1)

Table 4.3 *SLS Load factors and combinations for frames without cranes*

Serviceability limit state load	BS 5950-1 Clause 2.5.1					
	Imposed			Wind	Imposed + wind	
Dead						
Imposed						
Uniform snow	**1.0**				**0.8**	
Asymmetric snow[1]		1.0				0.8
Minimum Imposed[2] (Including maintenance)			1.0			
Real & definable[3]	**1.0**	1.0	1.0		**0.8**	0.8
Wind				1.0	**0.8**	0.8

1 Only applies for roof pitches greater than 15° (see BS 6399-3:1988 Clause 7.2.3.3)

2 For roofs with "No access" (i.e. access for cleaning and repair only) UDL of 0.6 kN/m^2 or point load of 0.9 kN.

For further details see BS 6399-3:1988 Clause 4.3

3 Any additional potential imposed roof loads not specifically included in the above, e.g. suspended platform or walkway etc.

5 FRAME ANALYSIS AT ULTIMATE LIMIT STATE

5.1 General

At the ultimate limit state, the methods of frame analysis fall broadly into two types: elastic analysis and plastic analysis. The term *plastic analysis* is used to cover both rigid-plastic and elastic-plastic analysis. Plastic analysis commonly results in a more economical frame because it allows relatively large redistribution of bending moments throughout the frame, due to plastic hinge rotations. These plastic hinge rotations occur at sections where the bending moment reaches the plastic moment at loads below the full ULS loading. The rotations are normally considered to be localised at "plastic hinges".

A typical "plastic" bending moment diagram for a symmetrical portal under symmetrical vertical loads is shown in Figure 5.1. This shows the position of the plastic hinges for the plastic collapse mechanism. The first hinge to form is normally adjacent to the haunch (shown in the column in this case). Later, depending on the proportions of the portal frame, hinges form just below the apex at the point of maximum sagging moment.

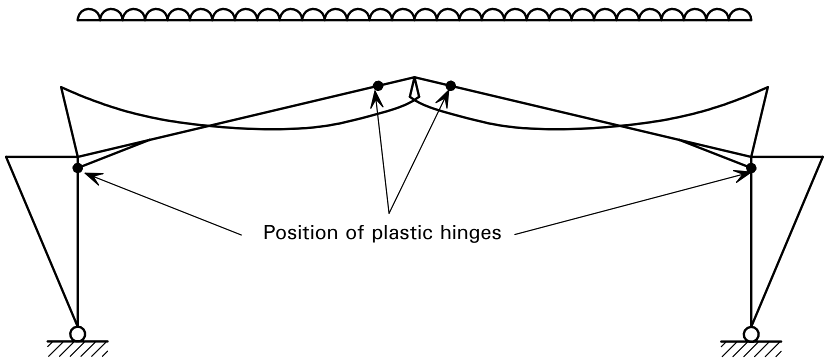


Figure 5.1 *Bending moment diagram resulting from the plastic analysis of a symmetrical portal frame under symmetrical loading*

Under BS 5950-1:2000, most load combinations will be asymmetric because they include either notional horizontal loads or wind loads. A typical loading diagram and bending moment diagram is shown in Figure 5.2

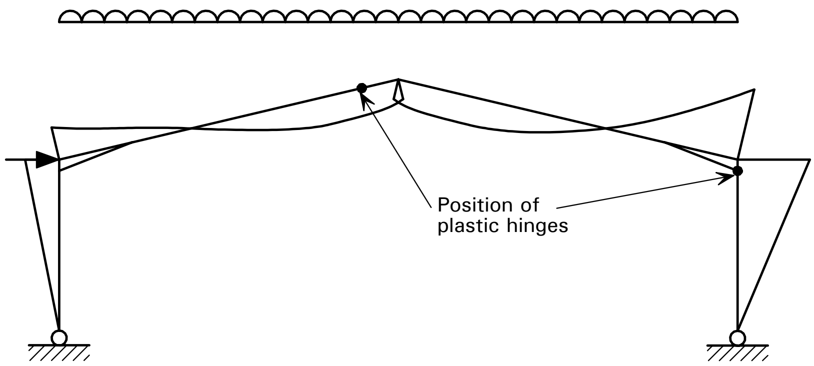


Figure 5.2 *Bending moment diagram resulting from plastic analysis of a symmetrical portal frame under asymmetric loading*

A typical bending moment diagram resulting from an elastic analysis of a frame with pinned bases is shown in Figure 5.3. In this case, the maximum moment is higher, and the structure has to be designed for this higher moment regime.

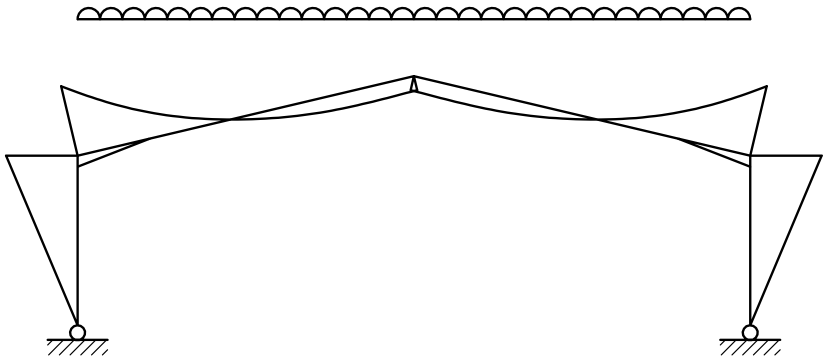


Figure 5.3 *Bending moment diagram resulting from the elastic analysis of a symmetrical portal frame under symmetrical loading*

In order to carry out a plastic analysis, the cross-sections of the members containing plastic hinges (see Section 3.1.3) must be classified as *plastic,* as determined from Tables 11 or 12 of BS 5950-1. This is to prevent local buckling occurring due to the high strains involved in the formation of plastic hinges. An elastic frame analysis may be used for all classes of cross-section, as no plastic deformations occur in the members.

5.2 First-order and second-order analysis

For either plastic analysis of frames, or elastic analysis of frames, the choice of first-order or second-order analysis depends on the in-plane flexibility of the frame and the method to be used to check the in-plane stability of the frame. BS 5950-1:2000 gives three different methods for checking the in-plane stability (see Section 6). Two of these methods use first-order analysis. Almost all single-span ordinary portals (not tied portals, as shown in Section 2.7) will have sufficient stiffness to be analysed by first-order analysis. BS 5950-1:2000 requires that tied portals are always analysed using second-order analysis.

5.3 Plastic analysis

Various methods of plastic analysis are explained in detail in the publication *Plastic design to BS 5950*[17]. A brief summary of possible methods is presented, below:

5.3.1 The graphical method

In the graphical method, the free bending moments are drawn superimposed on the fixed bending moment diagram, with the maximum values of the bending moments limited by the plastic moment capacity of the member. The method is generally suitable for the analysis of simple structures or the preliminary design of structures where the critical load cases are clear. It can also be a useful and rapid method of checking the results from a computer analysis. If applied correctly, the method will determine the upper bound of the bending moments or lower bound of load factor, and so it will always be safe.

5.3.2 Virtual work - rigid plastic mechanism method

The virtual work method calculates the load factor at collapse assuming a rigid plastic collapse mechanism. Relatively complex structures can be analysed without the use of a computer, but this is time consuming. The method assumes that the frame remains rigid until the formation of the final collapse mechanism, at which point failure occurs. The method has a number of disadvantages, as follows:

- It can only determine the load factor for the assumed mechanism for which the analysis is carried out. If the true collapse mechanism is not identified, the load factor will be over-estimated. To be safe, the complete bending moment diagram for the assumed mechanism should be established to ensure that the plastic moment capacity of the member is not exceeded at any point.
- The directions of the rotations should be checked carefully, to ensure that they are consistent with the assumed mechanism.
- The method will not identify any plastic hinges that form, then reverse and unload during the loading history of the frame.
- An accurate BMD at any load factor other than collapse cannot be determined.

5.3.3 Elastic-plastic analysis

This method is employed by the more sophisticated portal frame analysis programs, and depends on applying the load incrementally. Plastic hinges are formed in the members within the structure as their plastic moment capacity is reached. It assumes that the members behave elastically up to the full value of the plastic moment capacity, then plastically (without strain hardening) to allow redistribution of moments around the frame until a collapse mechanism is reached. This may entail the formation and reversal of hinges at particular locations.

This method has several advantages including:

(i) It will not only predict the final collapse mechanism with accuracy, but will also predict the formation and possible reversal of hinges taking place at loads less than the collapse load. This will identify hinges that form but are not part of the collapse mechanism.

(ii) It gives the bending moment diagram at ULS loading, compared with other methods that give the bending moment diagram at collapse.

(iii) It automatically performs an elastic analysis if no plastic hinges form at ULS.

5.4 Elastic analysis

Although the use of plastic hinge analysis of portal frames at the ultimate limit state is well established in the UK, it is not widely used internationally. Furthermore, there are situations where elastic analysis is more appropriate e.g. where:

- Tapered members are used.
- Instability of the frame is a controlling factor.
- Deflections are critical to the design of the structure.

5.5 Base fixity

Column bases are usually considered as being nominally pinned at the ultimate limit state. This simplifies the design considerably, which is an important factor when the analysis is carried out by hand.

When a computer program is used to analyse the structure, a degree of base stiffness may be considered. Stiffness at the base can reduce the deflections and increase the stability of the frame considerably. However, foundations that are designed to resist moments are considerably larger than those designed for axial load and shear forces only and consequently, much more costly.

BS 5950-1 Clause 5.1.3 gives recommendations on the base stiffness that may be assumed in the analysis as follows.

For a nominally rigid base

Where a column is rigidly connected to a suitable foundation, the following may be assumed:

- In elastic global analysis, the stiffness of the base should be taken as equal to the stiffness of the column for all ULS calculations. However, in determining deflections under serviceability loads, the base may be treated as rigid.
- In plastic global analysis, any base moment capacity between zero and the plastic moment capacity of the column may be assumed, provided that the foundation is designed to resist a moment equal to this assumed moment capacity, together with the forces derived from the consequent analysis. In elastic-plastic global analysis, the assumed base stiffness should be consistent with the assumed base moment capacity, but should not exceed the stiffness of the column.

For a nominally pinned base:

Where a column is nominally pin-connected to a foundation that is designed assuming that the base moment is zero, the base should be assumed to be pinned when using elastic global analysis to calculate the other moments and forces in the frame under ULS loading.

The stiffness of the base may be assumed to be equal to the following proportion of the column stiffness:

- 10% when checking frame stability or determining in-plane effective lengths
- 20% when calculating deflections under serviceability loads

For a nominally semi-rigid base:

A nominal base stiffness of up to 20% of the stiffness of the column may be assumed in elastic global analysis, provided that the foundation is designed for the moments and forces obtained from this analysis.

5.5.1 Modelling

In practice, allowance for base fixity is usually by the use of spring stiffness or dummy members at the column base.

Spring stiffness

At the ultimate limit state:

- A nominally rigid base can be modelled with a spring stiffness equal to $4EI_{column}/L_{column}$.
- A nominally pinned base can be modelled with a spring stiffness equal to $0.4EI_{column}/L_{column}$ for frame stability checks.

At the serviceability limit state:

- A nominally rigid base can be modelled with full fixity.
- A nominally pinned base can be modelled with a spring stiffness equal to $0.8EI_{column}/L_{column}$.

For dummy members:

If the computer program cannot provide a rotational spring, then the base fixity may be modelled by a *dummy* member of equivalent stiffness[18] (Figure 5.4).

The second moment of area (I) and the length (L) of the *dummy* member should be taken as follows:

- For a nominally rigid base: $I = I_{column}$ at ULS.
- For a nominally pinned base: $I = 0.1\ I_{column}$ at ULS and $0.2\ I_{column}$ at SLS.

In both cases, the length of the *dummy* member is $L = 0.75\ L_{column}$.

The provision of an additional support will affect the base reactions and therefore the reaction from the computer output should be corrected to the value of the axial force in the column.

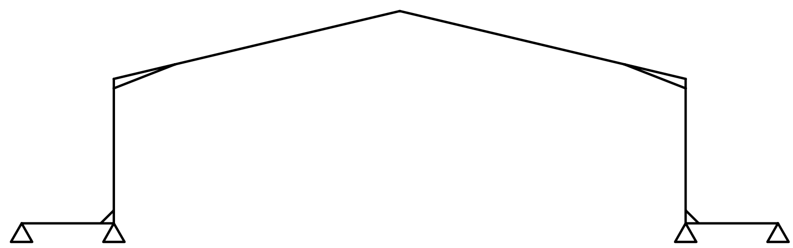


Figure 5.4 *Modelling base fixity by a dummy member*

5.5.2 Base moments for foundation design

It should be noted that, as far as the base moment (and associated forces) for foundation design is concerned, the following applies:

- Where partial base fixity is used to reduce the moments for which frame members have to be designed (compared to those obtained assuming pinned bases) the base moments should be taken into account in designing the foundations. This applies for both elastic analysis and plastic analysis of the frame.
- Where a nominal 10% base stiffness is used only in assessing effective lengths (or elastic critical load factors) or in determining whether an unbraced frame is 'sway-sensitive' or 'non-sway' it is not necessary to take account of the base fixity moment in foundation design.

- Where a nominal 20% base stiffness is used only in deflection calculations it is again not necessary to take account of the base fixity moment in foundation design.

5.6 Fire considerations

When it is necessary to design the bases to resist a rafter collapse moment at the fire limit state (see Section 12.6.3), it is not necessary to re-analyse the frame at the ultimate and serviceability limit states to take account of the additional fixity.

Where special foundations are required to resist the moments in fire conditions, it may be more economic to re-design the frame with a larger base size and stiffness.

5.7 Design summary

For single bay portal frames, the following points may be noted:

- Greater economy is generally achieved by the use of plastic analysis at the ultimate limit state and elastic analysis at the serviceability limit state.
- Sections used for plastic hinge analysis must be classed as *Class 1 plastic* according to Tables 11 and 12 of BS 5950-1 at those hinge positions that have formed and rotated prior to ULS and may be compact elsewhere.
- Frames are usually designed with nominally pinned bases but increased fixity of the bases may be considered at the SLS.
- For portal frames in boundary conditions at the fire limit state, a larger base may be required.

6 IN-PLANE FRAME STABILITY

6.1 Introduction

6.1.1 Methods in BS 5950-1:2000

Single storey portal frames need to be checked to ensure that they have adequate in-plane stability, whether designed by elastic or plastic methods. The in-plane stability of a portal frame depends on the stiffness of the frame as a whole, which is provided by the combined stiffness of members and their connections.

This type of frame cannot be checked by the simple method for multi-storey frames in BS 5950-1 Clause 2.4.2.6 and 2.4.2.7, because axial compression in the rafter is not considered in that method.

Axial loads in portal rafters have a much greater effect on the stability of the frame than the axial loads that might occur in the beams of common beam and column buildings.

BS 5950-1 gives three methods for checking the in-plane stability of single storey frames:

- The sway-check method.
- The amplified moment method.
- Second-order analysis.

Full details for in-plane stability of portal frames is given in SCI publication P292[2] and summarised below.

6.1.2 Use of required load factor, λ_r

The required load factor λ_r is used to account for the effects of frame deflection under load when first-order analysis is used. For elastic design of portal frames, the output from a first-order elastic analysis with ULS loads must be multiplied by λ_r before the member resistances are checked. For plastic design, the plastic collapse factor, λ_p, calculated by first-order global analysis with ULS loads must not be less than λ_r. Member strength and stability calculations should be made at $\lambda_r \times$ ULS rather than $1.0 \times$ ULS.

6.2 Sway check method

The sway check method for checking the in-plane stability of a portal frame requires only simple analysis techniques. The check identifies frames in which the second-order effects in the gravity load case ($1.4 \times$ Dead load and $1.6 \times$ Imposed load) are sufficiently small that they may be ignored. The check also identifies the required load factor for lateral load cases for use as described in Section 6.1.2.

The sway check method may be used for portals that are not tied portals and which satisfy the following geometrical limitations:

- Span/height to eaves is not more than 5.

- Rise of apex above column tops is not more than span/4 for symmetrical spans.
- Asymmetric rafter satisfies the criterion of Clause 5.5.4.2.1(c) of BS 5950-1.

The sway check method is the simplest method and gives economical designs if the frame is sufficiently stiff to satisfy either the h/1000 check or the formula check (Clause 5.5.4.2.2 of BS 5950-1). The sway-check method will often give the most economical designs for single span portals that tend to be relatively stiff. Economy is achieved because there is no reduction in frame strength for the gravity load cases, i.e. load combination 1 of Clause 2.4.1.2 of BS 5950-1 that are generally the critical design load cases.

For multi-bay frames "snap-through" stability should also be checked for the internal bays as given in Clause 5.5.4.3 of BS 5950-1.

6.2.1 Design steps – sway check method – gravity load case for elastic or plastic design

Steps required to satisfy Clause 5.5.2 or 5.5.3 of BS 5950-1 using the sway check method for **gravity loads** as given in Clause 5.5.4.2.2 of BS 5950-1.

The loads considered are those in load combination 1 Dead and Imposed Loads (gravity) (Clause 2.4.1.2 of BS 5950-1).

Step 1 Check that the geometry of the frame is within the geometrical limits (Clause 5.5.4.2.1 of BS 5950-1).

Step 2 Check the sway stiffness of the frame using h/1000 check or formula method

***h*/1000 check method**

- Calculate the notional horizontal forces (NHF), i.e. 0.5% of the factored vertical dead and imposed loads.
- Apply NHF at the top of each column and calculate the column top deflection δ.
- Sway stiffness ok if $\delta \leq \dfrac{column\ height}{1000}$

Formula Method (Clause 5.5.4.2.2 of BS 5950-1)

- Check effective span to depth ratio of the rafter satisfies:

$$\frac{L_b}{D} \leq \frac{44L}{\Omega h}\left[\frac{\rho}{4 + \rho L_r / L}\right]\left[\frac{275}{p_{yr}}\right]$$

Step 3 If the *h*/1000 check or 'Formula Method' is satisfied, then the required load factor for frame stability λ_r = 1.0 for the gravity load case. Otherwise use amplified moment method (see Section 6.3) or second order analysis (see 6.4). Other alternatives would be to include some base stiffness when calculating the deflection under NHF or if the *h*/1000 check is almost satisfied then increase the column size.

For plastic design:

Steps 1 to 3 from above, plus:

Step 4P Carry out plastic analysis (first order) of the frame. Apply gravity loads plus NHF to the frame. Calculate plastic collapse load factor λ_p.

Step 5P Check the strength of the frame. Ensure $\lambda_p \geq \lambda_r$

Step 6P Check the member strength and out-of-plane stability.

Conservatively, the member strength and out-of-plane stability can be checked for the forces and bending moments at λ_p, i.e. formation of plastic hinges. Alternatively, the forces and bending moments from the elastic plastic analysis at a load factor of λ_r may be used.

For elastic design:

Steps 1 to 3 from above, plus:

Step 4E Carry out an elastic analysis (first order) of the frame. Apply gravity loads plus NHF to the frame and calculate the forces and moments around the frame.

Step 5E Multiply forces and moments from Step 4E by λ_r.

Step 6E Check the member strength and out-of-plane stability at λ_r, using the amplified forces and moments from Step 5E.

6.2.2 Design steps – sway check method – lateral load case for elastic or plastic design

Steps required to satisfy Clause 5.2.2 or 5.5.3 of BS 5950-1, using sway-check method for horizontal loads, as given in Clause 5.5.4.2.3.

The loads considered are those in load combination 2 (dead + wind loads) and combination 3 (dead + imposed + wind loads) (see Clause 2.4.1.2 of BS 5950-1).

It is applicable to frames that satisfy requirements of Steps 1 and 2 given in 6.2.1 of "Sway Check Method – Gravity Load Case".

Step 1 Calculate the lowest elastic critical load factors for the frame for the "Sway check method" λ_{sc}.

- Calculate the NHF from load combinations 2 or 3.
- Apply NHF at the top of each column and calculate the column top deflection δ.

- $\lambda_{sc} = \dfrac{h}{200\delta}$ or,

  λ_{sc} may be approximated using:

- $\lambda_{sc} = \dfrac{220\ DL}{\Omega\, h\, L_b}\left(\dfrac{\rho}{4 + \rho L_r / L}\right)\left(\dfrac{275}{p_{yr}}\right)$

  (see Clause 5.5.4.2.3 in BS 5950-1).

  If $\lambda_{sc} < 5.0$, then second order analysis should be used.

Step 2 Calculate the required load factor, λ_r, for frame stability:

$$\lambda_r = \frac{\lambda_{sc}}{\lambda_{sc} - 1}$$

For plastic design:

Steps 1 and 2 from 6.2.2 above, plus:

Step 3P Carry out plastic analysis (first order) of the frame. Apply gravity and horizontal loads (load combination 2 or 3) to the frame, without any NHF. Calculate plastic collapse load factor λ_p.

Step 4P Check the strength of the frame. Ensure $\lambda_p \geq \lambda_r$.

Step 5P Check the member strength and out-of-plane stability.

Conservatively, the member strength and out-of-plane stability can be checked for the forces and bending moments at λ_p, i.e. formation of plastic hinges. Alternatively, the forces and bending moments from the elastic plastic analysis at a load factor of λ_r may be used.

For elastic design:

Steps 1 and 2 from 6.2.2 above, plus:

Step 3E Carry out a linear elastic analysis (first order) of the frame. Apply gravity and horizontal loads (load combination 2 or 3) to the frame without NHF and calculate the forces and moments around the frame.

Step 4E Multiply forces and moments from Step 3E by λ_r.

Step 5E Check the member strength and out-of-plane stability λ_r, using the amplified forces and moments from Step 4E.

Note: Amplified forces and moments may be obtained directly by carrying out a linear elastic analysis (first order) of the frame by increasing the ULS load factors to $\lambda_r \times$ partial factors in BS 5950-1, Clause 2.4.1.

6.3 The amplified moment method

The amplified moment method is an alternative method that may be used where the frame does not meet the limitations of the sway check method, or it may be used for portals that are not tied portals which have an elastic critical buckling ratio, λ_{cr}, not less than 4.6.

The method requires the determination of the lowest elastic critical load factor, λ_{cr}, for the particular loadcase on the frame.

The amplified moment method is a simple method to apply when the value of λ_{cr} is known.

The method gives reasonably economical designs if the frame is relatively stiff.

6.3.1 Design steps – amplified moment method

For elastic or plastic design:

Steps required to satisfy Clause 5.5.2 or 5.5.3 of BS 5950-1 using the amplified moment method of Clause 5.5.4.4.

Step 1 Calculate λ_{cr} (see SCI P292[2], Section 4.3 and 4.4). No method of determining λ_{cr} is given in BS 5950-1.

Step 2 Calculate the required load factor for frame stability, λ_r

If $\lambda_{cr} \geq 10$, $\lambda_r = 1.0$

If $10 > \lambda_{cr} \geq 4.6$, $\lambda_r = \dfrac{0.9\,\lambda_{cr}}{\lambda_{cr} - 1}$

Note: If $\lambda_{cr} < 4.6$, the amplified moment method is NOT applicable.

For plastic design:

Steps 1 to 2 from Section 6.3.1 above, plus:

Step 3P Carry out plastic analysis (first order) of the frame and determine plastic collapse load factor, λ_p.

Step 4P Check the strength of the frame. Ensure $\lambda_p \geq \lambda_r$.

Step 5P Check the member strength and out-of-plane stability.

Conservatively, the member strength and out-of-plane stability can be checked for the forces and bending moments at λ_p, i.e. formation of plastic hinges. Alternatively, the forces and bending moments from the elastic plastic analysis at a load factor of λ_r may be used.

For elastic design:

Steps 1 to 2 from Section 6.3.1 above, plus:

Step 3E Carry out linear elastic analysis (first order analysis) of the frame and determine the forces and moments around the frame.

Step 4E Multiply forces and moments from Step 3E by λ_r.

Step 5E Check the member strength and out-of-plane stability λ_r, using the amplified forces and moments from Step 4E.

Note: Amplified forces and moments may be obtained directly by carrying out a linear elastic analysis (first order) of the frame by increasing the ULS load factors to $\lambda_r \times$ partial factors in BS 5950-1, Clause 2.4.1.

6.4 Second order analysis

Second order analysis is another alternative method that may be used where the frame does not meet the limitations of the sway check method (see Section 6.2.1), or it may be used for all portals including tied portals.

Second order analysis is the term used to describe analysis methods in which the effects of increasing deflection under increasing load are considered explicitly.

Second order analysis is simple to apply if there is easy-to-use software available. It will give the most economical designs for more flexible frames, such as multi-span frames. It may give less economical designs than the other methods for stiffer frames because it will always calculate a reduction of frame strength from second-order (P-delta) effects. The other methods have threshold stiffness values above which the strength is not reduced.

6.4.1 Design steps – second order analysis

For elastic or plastic design:

Steps required to satisfy Clause 5.5.2 or 5.5.3 of BS 5950-1 using second order analysis as Clause 5.5.4.5. For second order analysis, $\lambda_r = 1.0$.

For plastic design:

Step 1P Carry out plastic analysis of the frame, and determine second order plastic collapse load factor λ_p

Note: The value of this λ_p is different from the λ_p calculated in Step 4P of Section 6.2.1, Step 3P of Section 6.2.2 and Step 3P of Section 6.3.1.

Step 2P Check the strength of the frame. Ensure $\lambda_p \geq 1.0$.

Step 3P Check the member strength and out-of-plane stability at $1.0 \times$ ULS forces and moments calculated by second order analysis

For elastic design:

Step 1E Carry out a second order elastic analysis of the frame and determine the forces and moments around the frame.

Step 2E Check the member strength and out-of-plane stability at $1.0 \times$ ULS forces and moments calculated by second order elastic analysis.

7 RAFTER DESIGN AND STABILITY

7.1 General

The size of the rafter will usually be determined at the preliminary design stage from the required cross-sectional resistance for combined bending moment and axial compression. However, in the final design, rafters will have to be checked for member stability. In-plane buckling will have been satisfied by consideration of the overall frame stability, presented in Section 6 for frames that are not "tied portals".

Three basic types of restraint can be provided to assist in preventing out-of-plane buckling:

- Lateral restraint, which prevents lateral movement of the compression flange.
- Torsional restraint, which prevents rotation of a member about its longitudinal axis.
- Intermediate restraint on the tension flange, which allows the distance between torsional restraints to be increased.

BS 5950-1 generally requires that members in bending and/or compression are checked for stability between restraints to the compression flange. In the case of the rafter of a portal frame, for gravity loads, the compression flange changes from the top to bottom flange of the rafter, as illustrated in Figure 7.1.

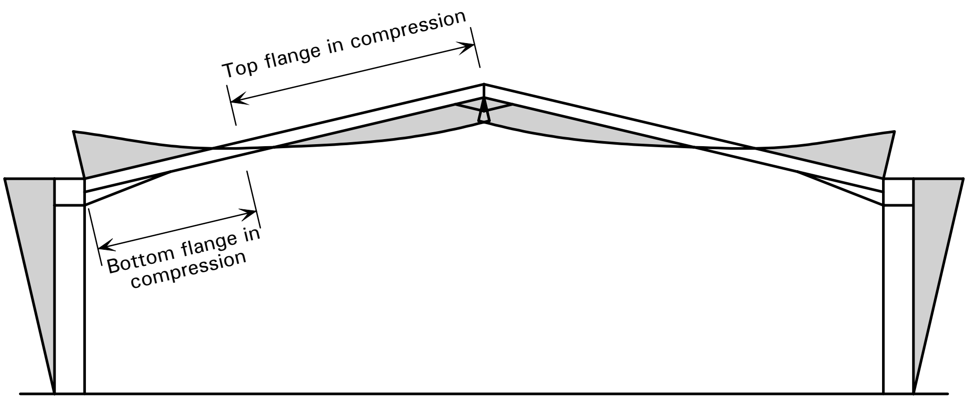


Figure 7.1 *Typical bending moment diagram under gravity loading*

7.1.1 Rafter restraint by purlins

Purlins attached to the top flange of the rafter provide stability to the rafter in a number of ways:

- Direct lateral restraint, when connected to the compression flange of the rafter.
- Intermediate lateral restraint between torsional restraints, when connected to the tension flange.
- Torsional restraint to the rafter when the purlin is attached to the tension flange and used in conjunction with a rafter stay to the compression flange (see Section 13.3).

In all the cases, the purlins should be tied back into a system of bracing in the plane of the rafters.

The position of the purlins should be determined at the final design stage before the stability of the rafter is checked. The purlin spacing will usually be determined from manufacturers' load tables. Spacing may have to be adjusted slightly to provide stays at strategic points along the rafter.

7.1.2 Point of contraflexure (POC) as a position of restraint

"POC" should not automatically be assumed to be a position of lateral restraint. However "POC" can be assumed to be a position of virtual restraint provided the following conditions are satisfied:

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

7.1.3 Position of plastic hinges

Single-bay portal frames are usually designed on the basis that a hinge will form in the columns at the underside of the haunch and in the rafter adjacent to the apex, with the area at the shallow end of the haunch remaining elastic. This is the old approach, based on the concern that plastic hinges at the top of the eaves haunch could cause early instability (see work by Morris and Nakane[19]).

In practice, plastic hinges may occur at the shallow end of the haunch and are acceptable provided that:

- the plastic hinge in the rafter is restrained in the conventional way by a rafter stay to a purlin
- lateral restraint is provided to the column at the bottom of the haunch.

7.2 Rafter and haunch stability for dead plus imposed load

Figure 7.2 shows a typical moment distribution and restraints for dead plus imposed load acting on a 20 to 30 m span portal frame. Purlins are placed at about 1.8 m spacing, but this spacing may need to be reduced in the high moment regions near the eaves and apex. Four stability zones are noted on the figure and the stability of the rafter in each of the zones is discussed in the following sections. The guidance is based on the assumption that plastic hinges form in the columns at the underside of the haunch and in the rafter near the apex.

BS 5950-1 provides various methods for checking the stability of segments adjacent to and away from plastic hinge position. The rules for checking the segment adjacent to a plastic hinge are onerous, but can conservatively be applied to segments away from hinge position.

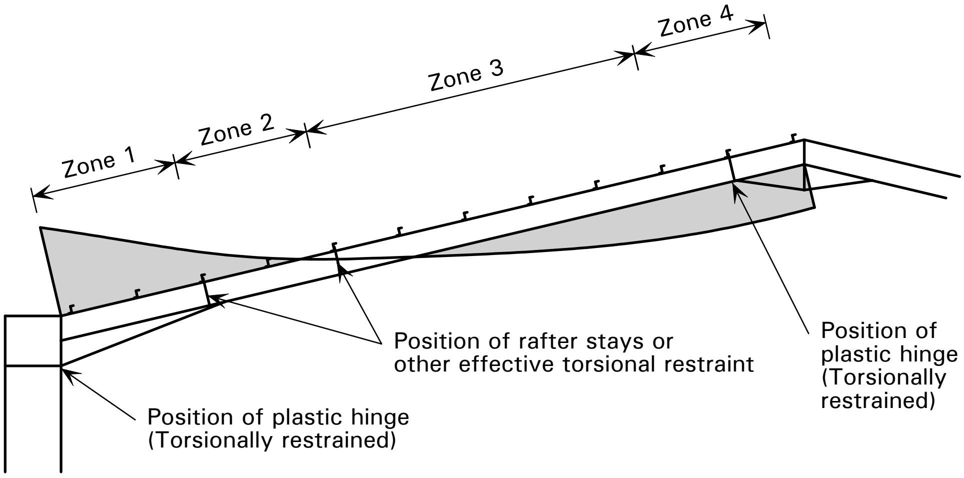


Figure 7.2 *Typical purlin and rafter stay arrangement for gravity loading*

7.2.1 Haunch stability in Zone 1

In Zone 1, the bottom flange of the haunch is in compression. The lower end of the zone is restrained by the restraints to the column and the upper end is restrained by rafter stays (Figures 7.2 and 7.3). It is usual, but not essential, for the upper end of the zone to coincide with the shallow end of the haunch.

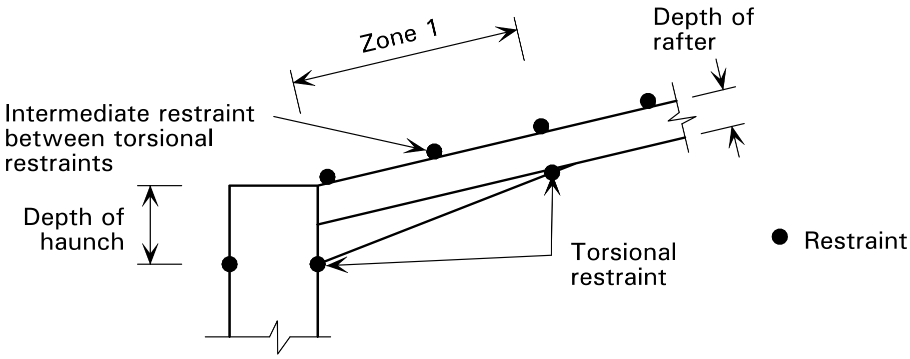


Figure 7.3 *Restraints to haunched region of a portal frame*

BS 5950-1 provides four methods for checking the stability of such a segment. It is assumed that torsional restraints are provided at both ends of the zone.

The third and fourth methods take advantage of the intermediate restraint (to the tension flange) between the torsional restraints (Figure 7.3).

First method (Clause 4.8.3.3.1(b))

$$\frac{F_c}{P_{cy}} + \frac{m_{LT}M_{LT}}{M_b} + \frac{m_y M_y}{p_y Z_y} \quad \le 1$$

for tapered or haunched I-sections M_b should be determined using the properties as given in Clause B.2.5 of BS 5950-1

Second method (Clause 5.3.3 (a))

The second and simplest method (Clause 5.3.3(a)) is to ensure that the distance between torsional restraints does not exceed length L_m given by:

$$L_m = L_u = \frac{38\, r_y}{\left[\frac{f_c}{130} + \left(\frac{x}{36}\right)^2 \left(\frac{p_y}{275}^{\;2}\right)\right]^{0.5}}$$

where:

f_c	is the compressive stress in the rafter due to axial force (N/mm^2)
p_y	is the rafter design strength (N/mm^2)
r_y	is the radius of gyration of the rafter about the minor axis
x	is the torsional index of the rafter.

If the member has unequal flanges, r_y should be taken as the lesser of the value for the compression flange and for the whole section.

Where the cross-section of the member varies within the length between the torsional restraints, the minimum value of r_y and the maximum value of x should be used.

Third method (Clause 5.3.4)

This is a simplified form of the check given in BS 5950-1: Appendix G, which takes account of the beneficial effects of intermediate tension flange restraints between torsional restraints and of the shape of the moment diagram.

For this method to be applicable, the following conditions should be satisfied:

- The member is an I section with $D/B \geq 1.2$.
- For haunched segments, $D_h \leq 2D_s$.
- For haunches, the haunch flange is not smaller than the member flange.
- The steel grade is S275 or S355.
- Member buckling resistance check (Clause 4.8.3.3 or Annex I.1) should be satisfied for out-of-plane buckling when checked using an effective length equal to the spacing of intermediate lateral restrains (spacing need not be less than L_m).

For this zone, the distance, between restraints to the compression flange should not exceed the value of L_s given by:

$$L_s = \frac{620\, r_y}{K_1 \left(72 - (100/x)^2\right)^{0.5}} \quad \text{for S275 steel}$$

$$L_s = \frac{645\, r_y}{K_1 \left(94 - (100/x)^2\right)^{0.5}} \quad \text{for S355 steel}$$

where:

K_1 = 1.0 for an unhaunched segment

K_1 = 1.25 for a haunch with $D_h/D_s = 1$

K_1 = 1.4 for a haunch with D_h/D_s = 2

K_1 = $1 + 0.25\,(D_h/D_s)^{2/3}$ for a haunch generally

r_y is the minor axis radius of gyration of the unhaunched rafter

x is the torsional index of the unhaunched rafter.

Fourth method (Appendix G)

The BS 5950-1 Appendix G method should be used where the conditions applied to the third method (Clause 5.3.4) cannot be satisfied, or where a greater allowable distance between restraints is required. The Appendix G method takes account of the restraints to the tension flange and the moment gradient between the restraints. However, this method is not recommended for manual calculations because of the number and complexity of the calculations.

7.2.2 Rafter stability in Zone 2

Zone 2 is generally in the area near the point of contraflexure (Figures 7.2 and 7.4).

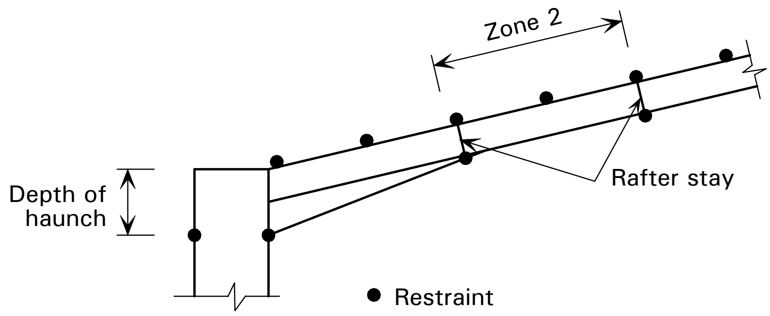


Figure 7.4 *Restraint to rafter adjacent to haunched region of a portal frame*

In this zone, torsional restraint will be provided at the lower end by a rafter stay. At the upper end, a torsional restraint may be provided by a rafter stay or a virtual restraint assumed at the point of contraflexure provided the conditions given in Section 7.1.2 are satisfied.

Any of the four methods described above for Zone 1 (see Section 7.2.1) may be used to check the stability of the rafter in Zone 2.

It is also possible to check the stability in Zone 2 using out-of-plane buckling of Clause 5.3.3(b), as given below.

Approximate method allowing for moment gradient (Clause 5.3.3(b))

For I section members with uniform cross-sections, equal flanges and $D/B \geq 1.2$ where f_c does not exceed 80 N/mm^2, the limiting length L_m is given by:

$$L_m = \phi L_u$$

L_u is as given above in second method of Section 7.2.1 and ϕ is given as follows:

For $1 \geq \beta \geq \beta_u$ $\qquad \phi = 1$

For $\beta_u > \beta \geq 0$ $\qquad \phi = 1 - (1 - KK_0)(\beta_u - \beta)/\beta_u$

For $0 \geq \beta > -0.75$ $\phi = K(K_0 - 4(1 - K_0)\beta/3)$

For $\beta \leq -0.75$ $\phi = K$

Where β is the ratio of end moments of the segment under consideration and β_u is given by:

$$\beta_u = 0.44 + \frac{x}{270} - \frac{f_c}{200} \quad \text{for S275 steel}$$

$$\beta_u = 0.47 + \frac{x}{270} - \frac{f_c}{250} \quad \text{for S355 steel}$$

$$K_o = (180 + x)/300$$

$$K = 2.3 + 0.03x - x\ f_c/3000 \quad \text{for } 20 \leq x \leq 30$$

$$K = 0.8 + 0.08x - (x - 10)\ f_c/2000 \quad \text{for } 30 \leq x \leq 50$$

7.2.3 Rafter stability in Zone 3

In Zone 3, purlins can generally be assumed to provide restraint to the compression flange and the length between the purlins should be checked according to the rules given in BS 5950-1, Clause 4.8.3.3.

7.2.4 Rafter stability in Zone 4

In zone 4, where a plastic hinge occurs just below the apex (see Figures 7.2 and 7.5). A torsional restraint is essential if a hinge forms at this position prior to ULS. However, it is not essential but considered good practice to provide a torsional restraint if it can be demonstrated that a plastic hinge does not form below ULS.

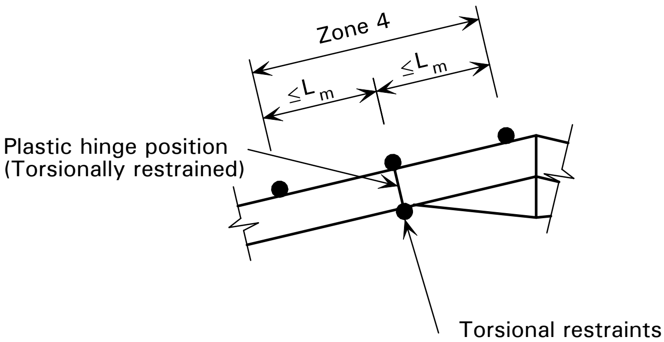


Figure 7.5 *Restraints adjacent to plastic hinge position*

If a torsional restraint is provided, it will usually take the form of a rafter stay to a purlin at or near the position at which the hinge has been assumed to form in the analysis.

The limiting distance to an adjacent lateral restraint is then given by the distance L_m (see Second method in Section 7.2.1 and Figure 7.5).

Where no plastic hinge occurs in Zone 4, torsional restraints are not required in this zone and the member stability should be checked between the lateral restraints provided by the purlins using the normal methods in BS 5950-1: Section 4.

7.3 Rafter and haunch stability under uplift conditions

Under uplift, most of the bottom flange is in compression. A typical reverse moment case and restraints due to wind uplift is shown in Figure 7.6.

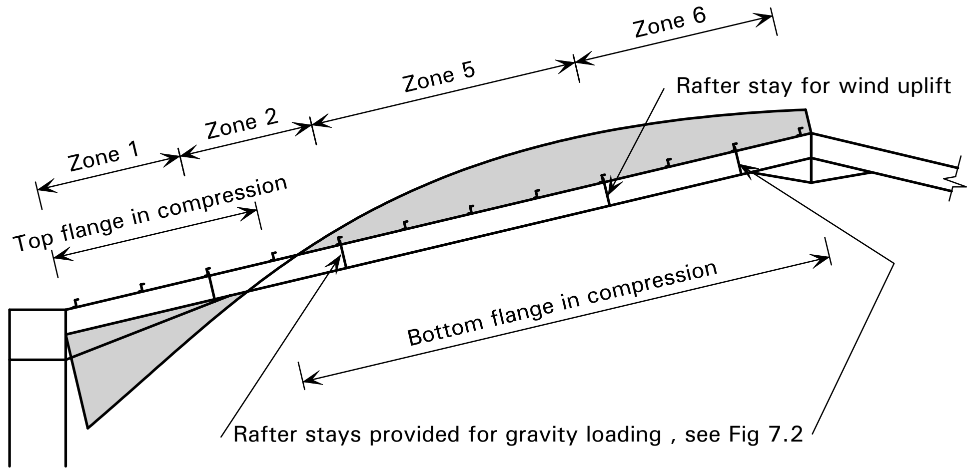


Figure 7.6 *Typical purlin and rafter stay arrangement for wind uplift*

This type of bending moment diagram will generally occur under internal pressure/wind uplift and the members will remain elastic. The stability checks recommended below assume that plastic hinges will not occur in this condition.

7.3.1 Haunch stability in Zones 1 and 2

In zones 1 and 2, the top flange of the haunch will be in compression and will be restrained by the purlins. No further restraint will be required, although the stability of the length between the purlins should be checked by the methods given in BS 5950-1, Clause 4.8.3.3.

7.3.2 Stability in Zones 5 and 6

In zones 5 and 6, the purlins will not restrain the bottom flange, which is in compression in these zones. The length considered for buckling checks should be between a torsional restraint provided at the upper end of Zone 2 (or at a point of contraflexure (see 7.1.2)) and the next torsional restraint. An additional rafter stay may be required in Zone 5. For greatest economy, the length should be checked using the method given in BS 5950-1: Appendix G which takes account of the restraints to the tension flange provided by the purlins. Alternatively, the method given in BS 5950-1, Clause 4.8.3.3. could conservatively be used. This method would then be repeated along the rafter by checking lengths between torsional restraints in Zones 5 and 6.

7.4 Design summary

- Determine the size of the rafter for the required cross-sectional resistance to bending moment and axial compression.
- Determine the spacing of purlins, based on the load capacity of the purlins and sheeting as given by manufacturers' load tables.
- Assume suitable positions of rafter stays (see Section 7.2).

- Check the stability of individual zones of the rafter under gravity loads and then under wind uplift. These cases are summarised in Figure 7.7 and Table 7.1.
- If any checks fail, adjust the purlin spacing and /or provide extra restraints.

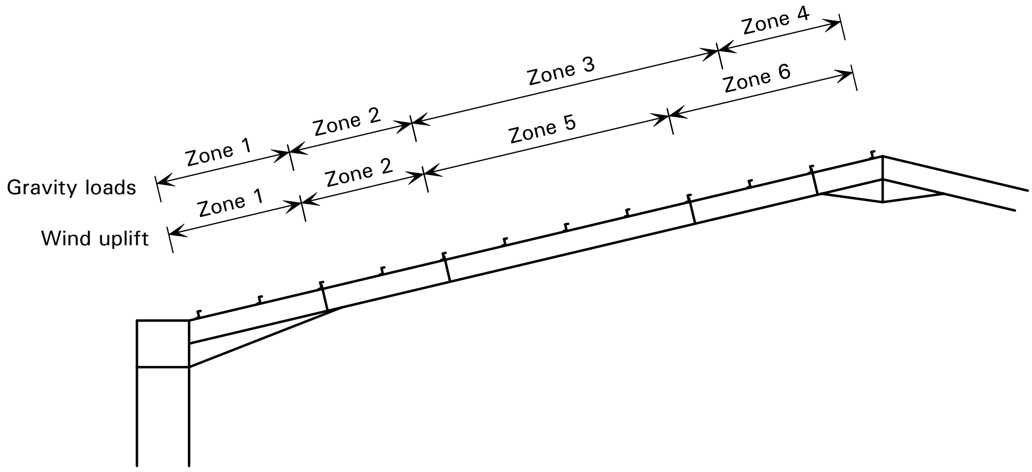


Figure 7.7 *Typical purlin and rafter stays and zones for checking stability*

Table 7.1 *Rafter stability checks*

Zone	Loading	Length	Criteria	BS 5950-1 Clause	Section in this publication
1	Gravity	Deep end of haunch to rafter stay	- $\leq L_m$ $\leq L_s$ -	4.8.3.3.1(b) 5.3.3 (a) 5.3.4 G.2.2	7.2.1
1	Wind uplift	Purlin to purlin	Axial and bending	4.8.3.3	7.3.1
2	Gravity	Rafter stay to rafter stay or point of contraflexure[1]	As for Zone 1 (Gravity loading)		7.2.2
2	Wind uplift	Purlin to purlin	Axial and bending	4.8.3.3	7.3.1
3	Gravity	Purlin to purlin	Axial and bending	4.8.3.3	7.2.3
5	Wind uplift	Rafter stay to rafter stay	$\leq L_s$ -	5.3.4 G.2.1	7.3.2
4	Gravity	Purlin to purlin	Axial and bending	4.8.3.3	7.2.4
		Plastic hinge[2] rafter stay/apex[3]	$\leq L_m$	5.3.3	7.2.4
6	Wind uplift	Rafter stay to rafter stay/apex[3]	$\leq L_s$ -	5.3.4 G.2.1	7.3.2

1 The point of contraflexure can provide torsional restraint if certain conditions are met (see Section 7.1.2).

2 Torsional restraints need not be provided if the hinge can be shown to be the last to form.

3 The apex can be taken to provide a torsional restraint.

8 COLUMN DESIGN AND STABILITY

8.1 General

The column size will generally be determined at the preliminary design stage on the basis of the required bending and compression resistances. Columns will normally be chosen as Universal Beam sections, which recognise the fact that the predominant loading is bending rather than axial.

At the position of maximum moment (generally assumed to be at the top of the column/base of the haunch, see Figure 8.1), a number of other effects should be considered:

Combined bending and axial load. Compression in the column reduces the moment capacity of the column slightly. The most straightforward way of calculating the reduced moment capacity is to use the SCI publication *Steelwork design guide to BS 5950-1:2000 - Volume 1: Section properties and member capacities*[20] .

Shear. The shear resistance of the column should be checked for the local shear stresses generated by the moment at the rafter/column intersection. These stresses are usually ignored at the preliminary design stage as they will depend on the connection details adopted. Universal Beam sections with thin webs may require stiffeners. It is not necessary to check combined bending and shear in the column web panel.

Web bearing and buckling. The local resistance of the column web should be checked for the compressive force in the bottom flange of the haunch. As for shear, no combined actions are considered but Universal Beam sections with thin webs combined with use of shallow haunches may require stiffening. A full depth web stiffener will be required in these cases.

Plastic hinge formation. Where a plastic hinge occurs in the column at the base of the haunch (see Section 7.1.2), it is necessary to provide a full depth stiffener at this location (see BS 5950-1, Clause 5.2.3.7).

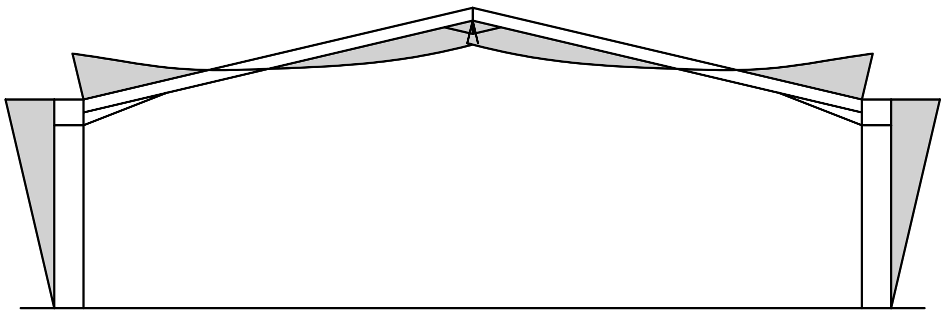


Figure 8.1 *Typical bending moment diagram for column with pinned base subject to gravity loading*

8.2 Column stability

Whether the frame is designed plastically or elastically, a torsional restraint should always be provided at the top of the column i.e. at the bottom of the eaves haunch. Additionally, a further torsional restraint may be required within the length of the column because the side rails are attached to the (outer) tension flange rather than to the compression flange.

8.2.1 Torsional restraints at the top of the column

At plastic hinge positions

BS 5950-1 requires that torsional restraints are provided at the plastic hinge positions (see Clause 5.3.2) which is at the bottom of haunch. A number of methods are available to provide the torsional restraint required, as follows:

- For columns of depths less than 610 mm, a column stay as shown in Figure 8.2 may be used. In order to ensure adequate stiffness, it is recommended that the depth of the side rail is at least 25% of the depth of the column.
- For all spans, a possible method is to provide a longitudinal member close to the compression flange at the bottom of the haunch, which is tied into the vertical bracing (Figure 8.3). The torsional resistance can then be provided by the circular hollow section acting with the side rails on the outer face of the column. It is important that plan bracing is provided between these components at some point in the length of the structure.

For elastically designed frames

If a frame is designed elastically, the torsional restraint at the top of the column may be designed in the same way as given above.

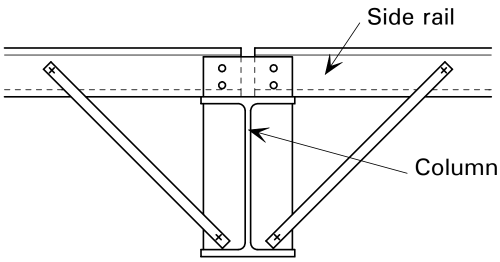


Figure 8.2 *Typical eaves detail using a column stay*

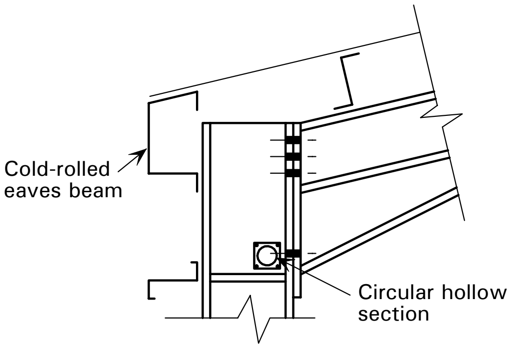


Figure 8.3 *Typical eaves detail using a circular hollow section as a longitudinal bracing member*

8.2.2 Stability of the column adjacent to the plastic hinge

BS 5950-1, Clauses 5.3.3, 5.3.4 and Appendix G give a number of possible methods of checking the stability of the column adjacent to a plastic hinge position, as described below.

The first method assumes that a restraint to the compression flange is also provided within a limiting distance L_m, as given in BS 5950-1, Clause 5.3.3(a). This approach does not make allowance for either the shape of the moment diagram or restraint to the tension flange and is therefore conservative.

The second method (Clause 5.3.3(b)) allows for the shape of the moment diagram but not the restraint to the tension flange.

A third possible approach for columns comprising Universal Beam sections is to use the same approach as for rafters (BS 5950-1, Clause 5.3.4) using a depth of haunch to depth of column ratio of 1.0. This is a simplified form of the check to BS 5950-1: Appendix G, which takes account of the beneficial effects of tension flange restraints between torsional restraints, and is conservative as it makes no use of the shape of the moment diagram. It is important to check that the spacing of the restraints to the tension flange between torsional restraints is adequate. The requirements in BS 5950-1, Clause 4.8.3.3.1 or 4.8.3.3.2 (or Clause 5.3.3 adjacent to a plastic hinge location) should be satisfied.

The fourth method is to use the method given in BS 5950-1: Appendix G, which takes account of the restraints to the tension flange and the moment gradient between the restraints. This method is not recommended for manual calculations.

A column stay generally provides a torsional restraint. Where masonry construction is used (which is not generally considered to provide restraint), or there is a large opening in the side of the building, the limiting distance should exceed the distance to the base of the column so that no intermediate restraints are required. This can increase the column size.

8.2.3 Stability of sections remote from plastic hinge positions or in an elastically analysed frame

In areas remote from a plastic hinge location or in elastically analysed frame, the member stability may be checked using the normal method for members between restraints to the compression flange (i.e. BS 5950-1, Clauses 5.3.4 or 4.8.3.3); otherwise, the more complex approach of BS 5950-1, Appendix G may be used.

8.3 Design summary

- Determine the size of the column for the required bending moment and compression resistance.
- Determine the spacing of the side rails, based on the load capacity of the side rail and the sheeting, from manufacturers' load tables.
- Provide restraints at the bottom of the haunch.
- Provide a restraint at the required distance from the hinge position for stability of the column (see Section 8.2.2).
- Check the remaining length of the column between the *adjacent* restraint and the base, by providing further restraints if necessary (see Section 8.2.3).

9 BRACING

9.1 Plan bracing

9.1.1 General

Plan bracing is placed in the horizontal plane, or in the plane of the roof. The primary functions of the plan bracing are:

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

In order to transmit the wind forces efficiently, the plan bracing should connect to the top of the gable posts wherever possible.

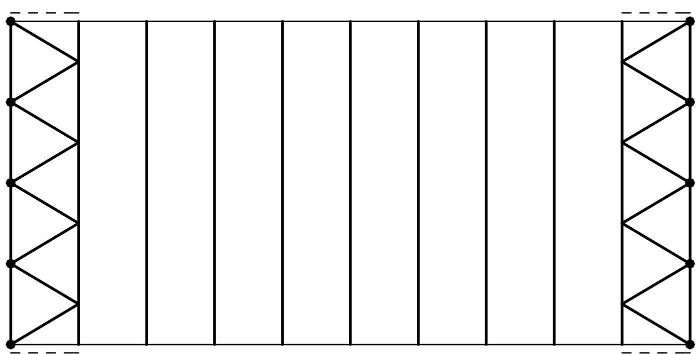
The plan bracing should be designed for the forces obtained from the rules given in BS 5950-1, Clause 4.3.2.2.3.

The purlins are not usually designed to resist axial forces due to wind loading.

For plan bracing, the following design cases may be considered:

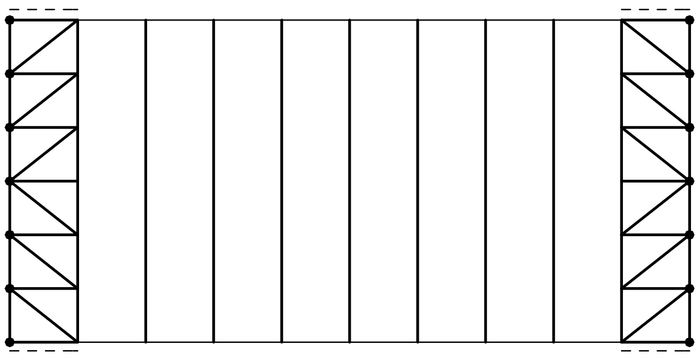
9.1.2 Bracing using circular hollow sections

In modern construction, circular hollow section bracing members are generally used in the roof 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. Two typical arrangements are shown in Figures 9.1 and 9.2. The bracing is usually attached to cleats on the web of the rafter, as shown in Figure 9.3. The attachment points should be as close to the top flange as possible, allowing for the size of the member and the connection.



• Position of gable posts
– – – – Location of vertical bracing

Figure 9.1 *Plan view showing both end bays braced (using circular hollow section members)*



• Position of gable posts
– – – – Location of vertical bracing

Figure 9.2 *Plan view showing end bay braced where the gable posts are closely spaced*

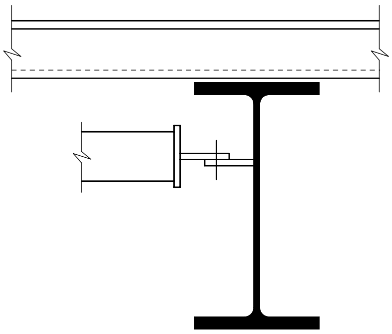
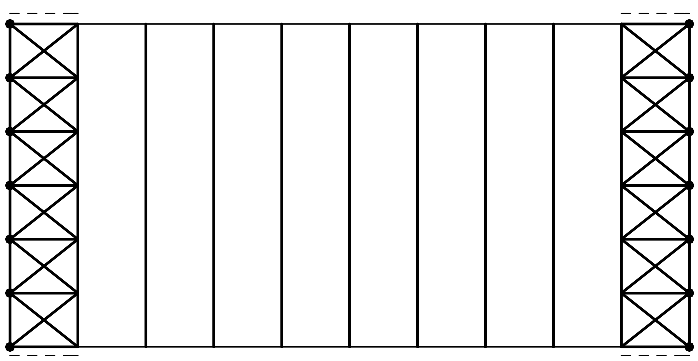


Figure 9.3 *Typical connection detail for circular hollow section bracing*

The alternative bracing pattern shown in Figure 9.2 reduce the effective lengths of the compression members, but uses more members and more complex connections. It is suitable for closely spaced gable posts.

9.1.3 Bracing using angle sections

The use of angles is not common in modern structures, but cross-braced angles have an advantage in that the diagonal members are relatively small because they are designed to resist tension only (Figure 9.4).



• Position of gable posts
– – – – Location of vertical bracing

Figure 9.4 *Plan view showing both end bays using crossed angle*

9.2 Side wall bracing

9.2.1 General

The primary function of vertical bracing in the side walls of buildings are:

- To transmit the horizontal loads, acting on the end of the building, to the ground.
- To provide a rigid framework to which side rails may be attached so that they can in turn provide stability to the columns.
- To provide temporary stability during erection.

The bracing system will usually take the form of:

- Circular hollow sections in a V pattern.
- Circular hollow sections in a K pattern.
- Crossed flats (within a cavity wall).
- Crossed hot rolled angles.

The bracing may be located at:

- One or both ends of the building, depending on the length of the structure.
- At the centre of the building (but this is rarely done due to the need to begin erection from one braced bay at, or close to, the end 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 required to transmit the forces from the plan bracing into the wall bracing.

9.2.2 Bracing using circular hollow sections

Circular 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 9.5). Where the eaves height is large in relation to the frame spacing, a K brace is often used (Figure 9.6).

An eaves strut may be required in the end bays, depending on the configuration of the plan bracing. The plan bracing shown in Figure 9.1 does not require an eaves strut, whereas that shown in Figure 9.2 does. In all cases, it is good practice to provide an eaves tie along the length of the building.

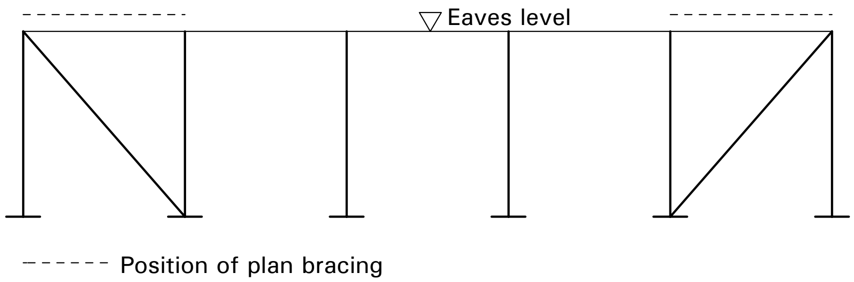


Figure 9.5 *Single diagonal bracing for low frames using circular hollow sections*

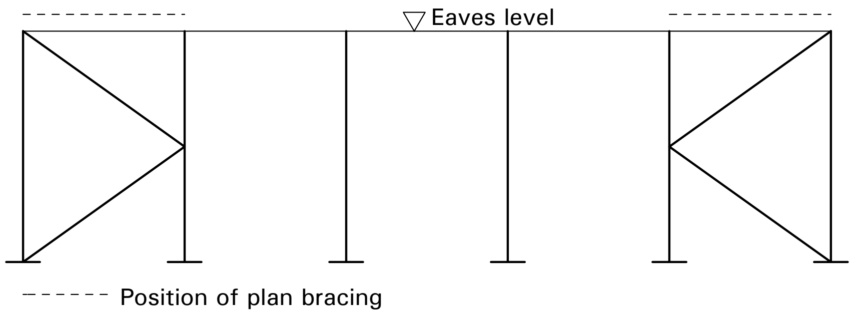


Figure 9.6 *K bracing arrangement for taller frames using circular hollow sections*

9.2.3 Bracing using angle sections or flats

Cross braced angles or flats (within a masonry cavity wall) may be used as bracing (as shown in Figure 9.7). In this case, it is assumed that only one of the diagonal members acts in tension under wind load.

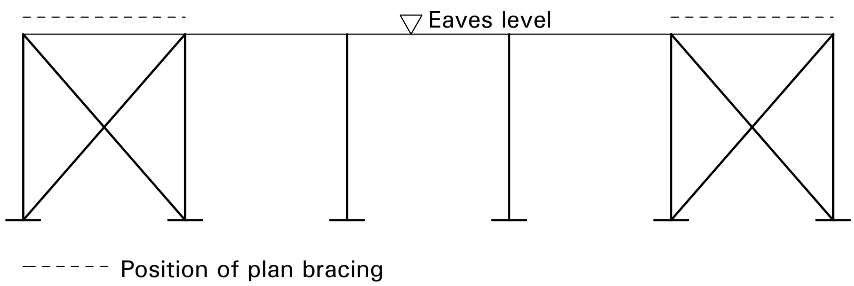


Figure 9.7 *Typical cross bracing system using angles or flats as tension members*

9.2.4 Bracing in a single bay

For vertical bracing in a single bay, an eaves strut/tie is required to transmit wind forces from the far end plan bracing into the vertical bracing (Figure 9.8). It may be possible to use the hollow section member at the bottom of the eaves haunch to also act as an eaves strut (Figure 9.9). Further details of eaves struts are given in Section 13.2.

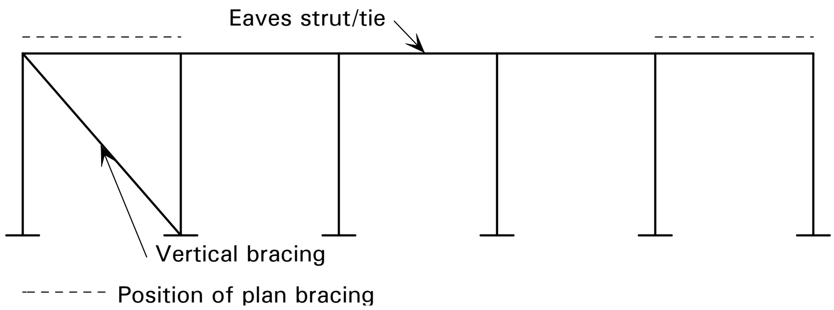


Figure 9.8 *Bracing in a single end bay with an eaves strut*

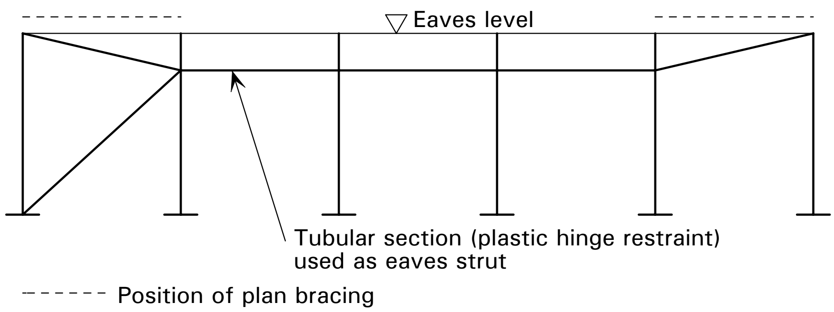


Figure 9.9 *Bracing arrangement using the hollow section member as a restraint and as an eaves strut*

9.2.5 Single central braced bay

The concept of providing a single braced bay near the centre of a structure (Figure 9.10) is unpopular because of the need to start erection from a braced bay and to work down the full length of a building from that point. However, bracing in the middle of the building has the advantage that it allows free expansion of the structure, which is particularly valuable in locations such as the Middle East where the diurnal temperature range is very large. In the UK, the expected temperature range is taken as –5°C to +35°C, and overall expansion is not generally considered to be a problem. If a central braced bay is used, it may be necessary to provide additional temporary bracing in the end bays to assist in erection. For the case of a central braced bay and plan bracing at the ends of the building, an eaves strut will be required to transmit wind forces.

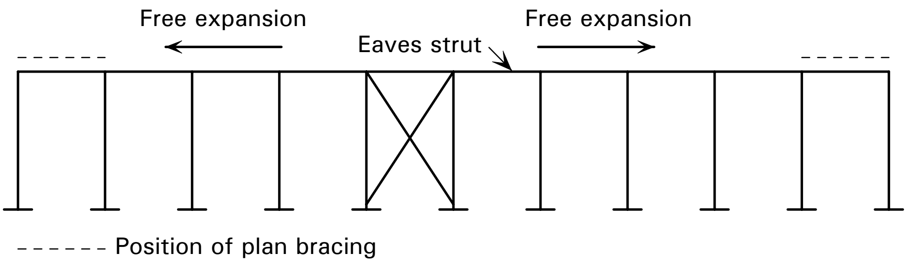


Figure 9.10 *Typical cross bracing at centre of the structure to allow free thermal expansion*

9.2.6 Portalised braced bays

Where it is difficult or impossible to brace the frame vertically by conventional bracing, a portalised structure can be provided. There are two basic possibilities:

- A portal structure in one or more bays (Figure 9.11).
- A hybrid portal/pinned structure down the full length of the side (Figure 9.12).

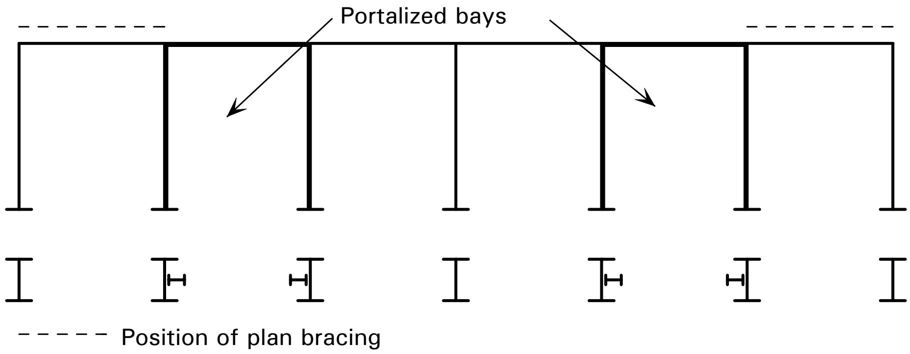


Figure 9.11 *Portalising individual bays*

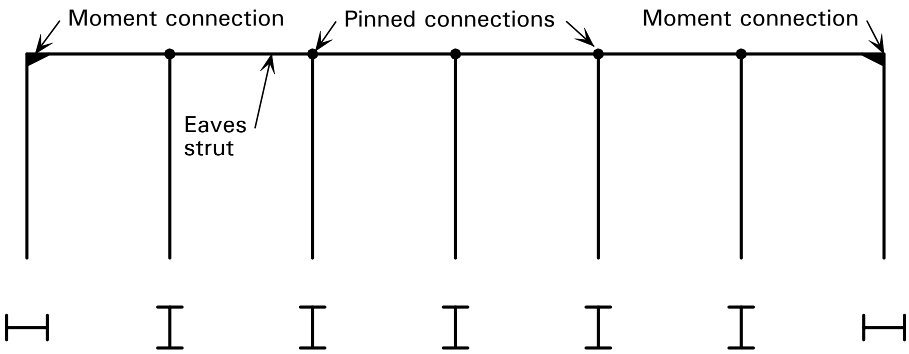


Figure 9.12 *Hybrid portal along the full length of the building*

The advantage of the first approach is that the conventional portal structure can be determined relatively early. It has the disadvantage that additional members are required and that openings in the side of the building may be restricted.

The second approach provides a lighter and much more open structure. Although in practice its actual stiffness is perhaps less than calculated (due to the flexibility of the internal struts), it is a method that has been used successfully.

In design of both systems, it is suggested that:

- The bending resistance of the portalised bay (not the main portal frame) is checked using an elastic frame analysis
- Stability is checked by using the sway-check method (see Section 6.2) and restricting the deflection under the notional horizontal forces to $h/1000$.
- The stiffness is assured by restricting serviceability deflections to a maximum of $h/360$, where h is the height of the portalised bay.

Where the opposite face is conventionally clad, vertical bracing may be provided on that face. The effects of racking action due to the difference in stiffness of the sides is generally negligible, provided there is adequate bracing in the end bays (Figure 9.13).

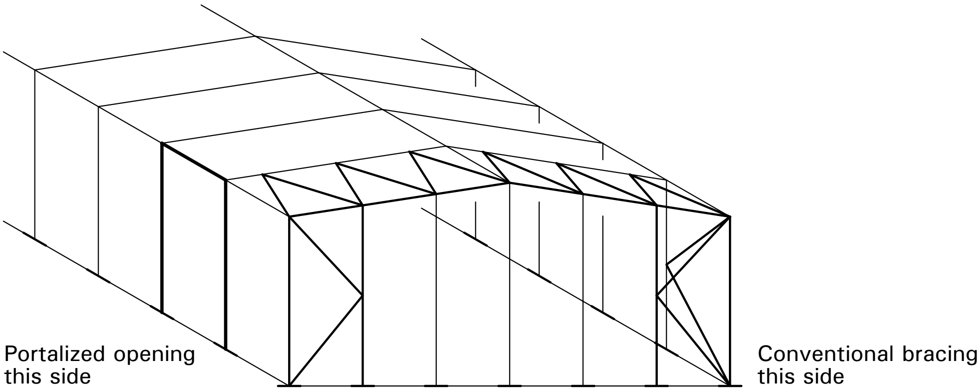


Figure 9.13 *Portalising an opening on one side with conventional bracing on the other side of the structure*

9.2.7 Bracing to restrain columns

If side rails and column stays (fly braces) 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 there is more than one opening in the side of the building, additional intermediate bracing may be required. This bracing should be provided as close to the plane of the side rail as possible, preferably on the inside face of the outer flange (Figure 9.14).

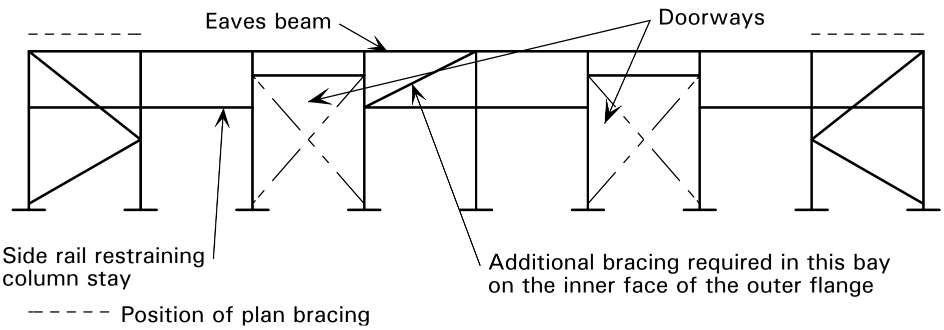


Figure 9.14 *Typical bracing pattern in side of building with openings*

It is not necessary to *node* the restraining rail exactly at the bracing position, as it can be assumed that diaphragm action in the sheeting and the transverse stiffness of the column can transmit the load into the vertical bracing system.

Where a member is used to restrain the position of a plastic hinge in the column (see Section 8.2.1), it is essential that it is tied properly into the bracing system. This can result in the configuration shown in Figure 9.15. Where there is more than one opening in the side of the building, additional intermediate bracing will be required above the haunch level in a similar way to that described above. In this case, the additional bracing should be located on the inner face of the inner flange.

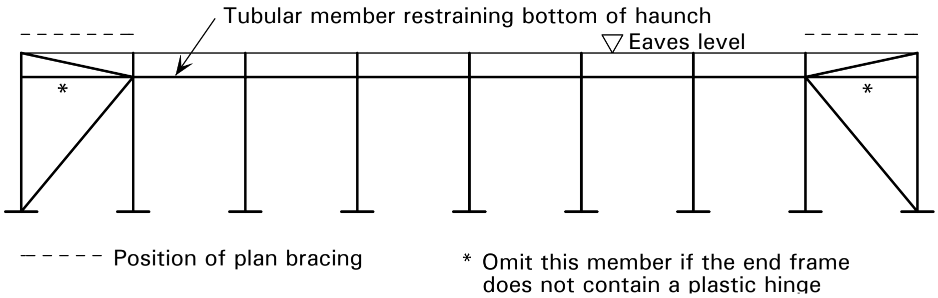


Figure 9.15 *Typical bracing pattern in building using a hollow section member to restrain a plastic hinge at the base of the haunch*

9.2.8 Bracing to restrain longitudinal crane surge

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 provided. A horizontal truss at the level of the girder top flange or, for lighter cranes, a horizontal member on the inside face of the column flange tied into the wall 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 (Figures 9.16 and 9.17). The criteria given in Table 9.1 were given by Fisher[21] to define the bracing requirements.

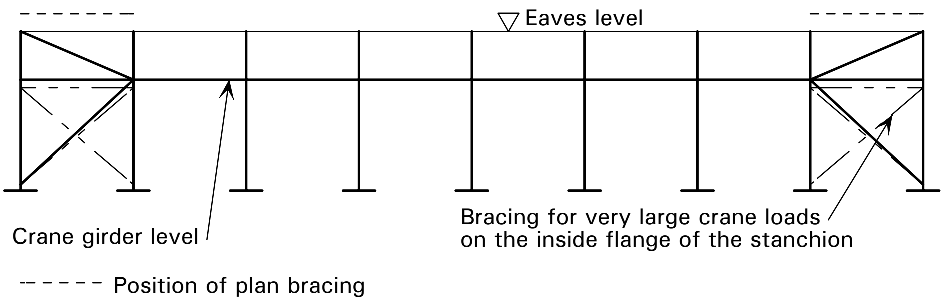


Figure 9.16 *Elevation showing position of additional bracing in the plane of the crane girder*

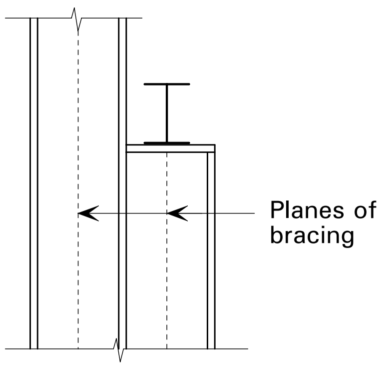


Figure 9.17 *Detail showing position of additional bracing in the plane of the crane girder*

Table 9.1 *Bracing requirements for crane girders*

Factored longitudinal force	Bracing requirement
Small (<15 kN)	Use wind bracing
Medium (15 - 30 kN)	Use horizontal bracing to transfer force to plane of bracing
Large (> 30 kN)	Provide additional bracing in the plane of the longitudinal force

If the bracing is attached directly to the column, it will tend to attract vertical load and, for heavily loaded crane girders, it may be necessary to provide an additional horizontal member to prevent fatigue failure of the connection (Figure 9.18).

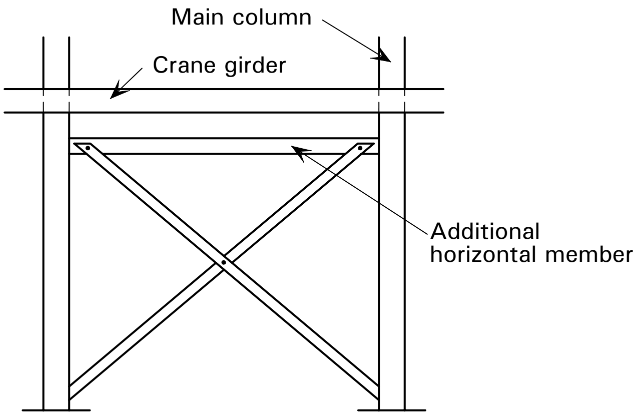


Figure 9.18 *Alternative configuration of additional bracing to prevent fatigue failure*

9.3 Stressed skin design

All structures fully clad with profiled steel sheeting will be stiffened to some extent by the stressed skin action of the cladding. Where the sheeting is attached with adequate fixings, it can be assumed that stressed skin action will:

- Reduce sway/lateral deflections.
- Reduce secondary moments due to sway.
- Act as restraints to the compression flanges of members (if the sheeting is connected directly to the flange).
- Act as plan or side wall bracing to the structure.

The full benefits of using stressed skin action may be determined using BS 5950-9[1], which sets out the principles of stressed skin design and provides a number of worked examples to illustrate the methods.

A number of major difficulties exist in utilising this design method:

- The method can be complex. Designers working within limited design budgets are reluctant to use a time-consuming method of design to save what appears to be a small amount of bracing (which may be required anyway for erection purposes).

- The UK construction industry is organised in a way that does not often allow the designer to decide the manufacturer of the cladding, and therefore the type of cladding and method of fixing is not generally known at the design stage.
- The future use and modification of the building, including the cladding, roof openings etc. may affect the original design assumptions.
- The cladding fixings play a major part in the design and are subject to reversal of load. This load reversal could lead to leaks due to elongation of the holes around the fixings.

Given time and careful thought, there is no real reason why any of these objections should stand in the way of appropriate use of stressed skin design, especially where large numbers of similar buildings are to be constructed. However, the consequences of possible future changes to the structure must be considered. The CDM regulations require that such structural issues are set down in documents retained by the client, so this should be less of a problem than it has been in the past. See guidance in SCI publication P162[22].

Although formal stressed skin design is not widely used, it is recognised that most steel roof and cladding systems demonstrate high in-plane stiffness and shear resistance, and reduce differential sway deflections between adjacent frames. This beneficial effect is dependent on the type of cladding system that is used:

Modern *double skin* roofs possess good in-plane stiffness largely as a result of the stiffness of the liner tray. Tests also show that the Z spacers in double skin roofs are capable of transferring shear to the external sheeting.

Standing seam roofs possess much less in-plane stiffness, and any in-plane contributions to stiffness should be neglected, unless justified by tests.

Composite or sandwich panels possess intermediate stiffness, depending on the form of attachment to the purlins and the thickness of the steel skins and insulation layer.

A paper by Davies and Lawson[23] gives sensible in-plan resistance and stiffnesses for modern roofing systems. The stiffness of the roof and wall cladding also influences the distribution of dynamic wind loading over the building, and particularly the wind loading that can be assumed to be transferred between adjacent frames.

9.4 Design summary

9.4.1 Bracing to resist wind loads

The following procedure may be used to design the bracing members subject to wind and other horizontal loads:

- Establish the wind loads on the structure.
- Determine the layout of bracing on plan and elevation required to resist wind loads.

- Check the load paths through the structure. Identify how wind loads are transferred from the gable frame into the plan bracing in the roof then into the vertical bracing in the walls and down into the foundations.
- Identify large openings in the walls that may affect the bracing system.
- Where the plan and the vertical bracing are not provided in the same bay, provide an eaves strut to transfer the loads.
- Check continuity of roof and wall bracing and other members forming part of the system.
- Reduce the eccentricities between the line of action of the force and the plane of the bracing members to a minimum.
- Determine which foundations act as restraints against overall shear, uplift, and overturning effects.
- Determine any load sharing between bracings, ensuring that intermediate members are capable of carrying the load.
- Check that tie members are of a practical size. In BS 5950-1:2000 there are no restrictions on the maximum slenderness for tie members subject to stress reversal. However, it is recommended that the old maximum slenderness (L/r) limit of 350 is observed, unless special provision is made to counteract the self-weight deflection of the tie.
- Design the members for the factored loading (depending on the load factors for loads in combination).

9.4.2 Bracing to restrain members

Identify members or frames that should be restrained to prevent instability, and identify points of restraints, i.e. purlins and side rail positions, column and rafter stays to purlins and rails, longitudinal hollow section restraints at the bottom of a haunch to restrain the plastic hinge.

- Identify members that can provide restraint including those that provide restraint to more than one member.
- Check that the restraints are tied back into a system of bracing.
- Reduce the eccentricities between the line of action of the force and the plane of the bracing member to a minimum.
- Design the restraining members as follows:
    - Purlins and sheeting rails need not generally be designed for specific forces (see Section 13.4.6.).
    - Rafter and column stays should be checked in accordance with Section 13.3 of this guide.
    - Vertical and plan bracing should be designed in accordance with the combination rules given in BS 5950-1, Clause 4.3.2.2.3. The forces from the restraints need not be taken as acting coincidently with those from wind loading.

10 GABLE WALL FRAMES

10.1 General

Gable wall frames may be classified as one of two basic types:

- Rafter and post frame.
- Portal frames.

The rafter and post frame will usually be fabricated from hot rolled sections, although cold formed sections may be used. Different types of gable wall frame are discussed in Section 2.11.

The following sections describe details of gable wall frames using a nominally pinned and braced rafter and post system.

10.2 Gable rafters

Gable rafters will usually be designed as simply supported between the gable posts, even though the ratters may in practice be continuous across the top or the external face of the posts (Figure 10.1).

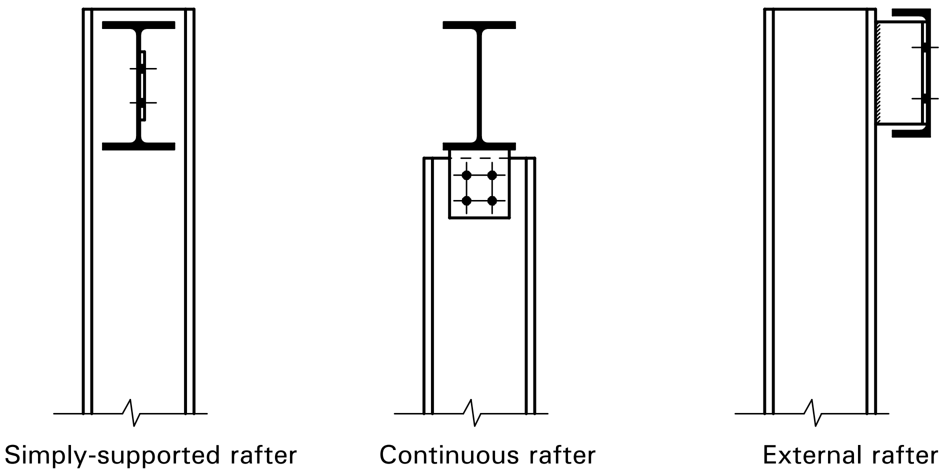


Figure 10.1 *Details of gable rafter support*

The configuration chosen will depend on the details of the gable wall (requirements of doors, positions of posts, etc.). Where the post can be located at the apex (Figure 10.2a), the rafter is normally simply supported at apex post. Where there is no apex post, a cranked rafter is required (see Figure 10.2b); the cranked potion of the rafter is normally simply supported.

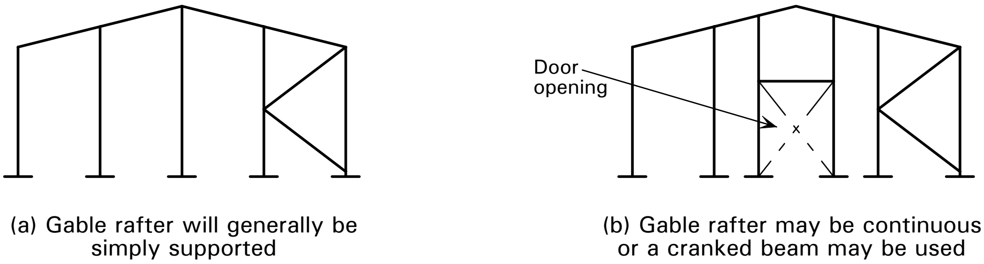


Figure 10.2 *Choice of simply supported or continuous gable rafter*

The gable rafters are required to support:

- Vertical loads from the roof
- Forces from the horizontal plan bracing
- Lateral forces from the ends of the purlins which are transferred into the plan bracing system
- Loads from the top of the gable posts which are transferred into the plan bracing (if the plan bracing does not node with the top of the gable posts).

Stability of the gable rafters is provided by the purlins when the top flange is in compression and by rafter stays (if necessary) in the uplift or reverse moment condition.

Where the rafter is continuous, there is often an eccentricity of the wind load reaction at the top of the post from the plane of the plan bracing. In Figure 10.3a, a stay from the top of the post into the bracing system can be provided to reduce the eccentric force on the rafter. The stay should not usually be connected directly to a purlin, unless the purlin is designed to resist this axial force and moment.

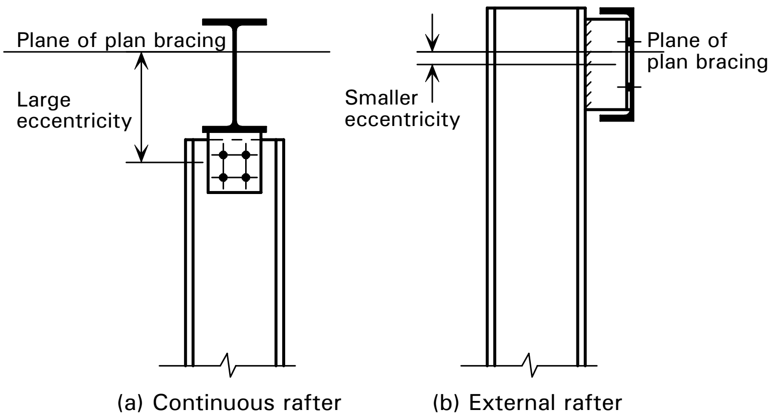


Figure 10.3 *Configuration of continuous gable rafters*

A rafter detail shown in Figure 10.3b can partially overcome the problem of eccentricity with the bracing but, clearly, an eccentricity of the vertical load is created in the other plane, which should be allowed for in design of the gable post.

10.3 Corner posts

The corner posts in a rafter and post type gable frame will usually be nominally pinned at the base and connected to a base that is smaller than those provided for the main frames. In small structures, the post may be connected to the floor slab. The post will usually be orientated the same way as the columns of the portal frames.

The corner post is required to resist compression from vertical load on the roof, uplift forces from wind reversal, and tension/compression from the action with the side wall bracing. Unless there are large openings in the sides of the building at the corners, bracing by side rails can be provided, supplemented by column stays.

Where the side wall bracing connects between the penultimate portal frame and a smaller corner post, their relative sizes and positions require careful consideration to prevent large eccentricities. Extended cleats can be provided on the corner post for side rails where the outer flanges do not align. To avoid large eccentricities, the corner post should generally be not less than 50% of the depth of the portal frame column plus 50 mm.

10.4 Intermediate posts

The primary function of the intermediate posts is to resist bending from the wind on the end wall and to carry axial load from the roof. They should be designed as simply-supported members spanning between the base and the bracing level. For positive external forces, the outer (compression) flange will be restrained by the side rails (where present), and for internal positive pressures, the inner (compression) flange will be restrained by column stays from the side rails. Where such restraint cannot be provided, the post should be designed as unrestrained over its full length for this load case.

10.5 Buildings with internal gables

Where it is known that a building will contain permanent internal walls, a light internal rafter and post gable wall may be provided instead of a full frame at those locations. This is more economic, but it restricts future modification of the building. Consideration should be given to the effects of the relative vertical deflection of the apex between the internal gable frame and the adjacent full portal frames.

10.6 Fire considerations

If a portal frame is used in the gable wall, the frame can be designed following the guidance given in *Single storey steel framed buildings in fire boundary conditions* (P313)[14].

Where a rafter and post frame is used, the method for portal frames given in the above SCI publication (P313) is not valid, and instead special recommendations are given for the design of gable posts.

Alternatively, the rafter in both types of frame can be fire protected, although this is an expensive solution and is rarely adopted.

10.7 Differential deflections

Generally, where a rafter and post frame has been used, it will be braced and will therefore be much stiffer than the adjacent portal frames. In practice this is also true with a portal frame gable wall because it will be stiffened by the cladding. Differential deflection between the gable frame and penultimate frame can therefore be relatively large, and may be of particular concern if there are cranes, masonry construction, or sensitive cladding attached to the frame.

Ways of reducing differential deflections include:

- Bracing in the roof between the gable frame and the adjacent frame will reduce the deflection of the adjacent portal frame to some extent, but this is normally not quantifiable without a 3-D analysis of the whole structure.
- A penultimate frame can be provided of greater stiffness than the other frames to reduce the differential deflection due to eaves spread and wind loading. This is not usually a sensible option in terms of fabrication efficiency.
- The portal frames should be pre-set carefully to ensure that all dead load deflections result in frames that line up with the gable frame under dead load only, thus reducing to some extent the differential deflection due to eaves spread.

Further consideration of deflections is given in Section 14.1.

11 CONNECTIONS

11.1 General

The two major connections in a single bay portal frame are those at the eaves and the apex. Typical connections are shown in Figure 11.1.

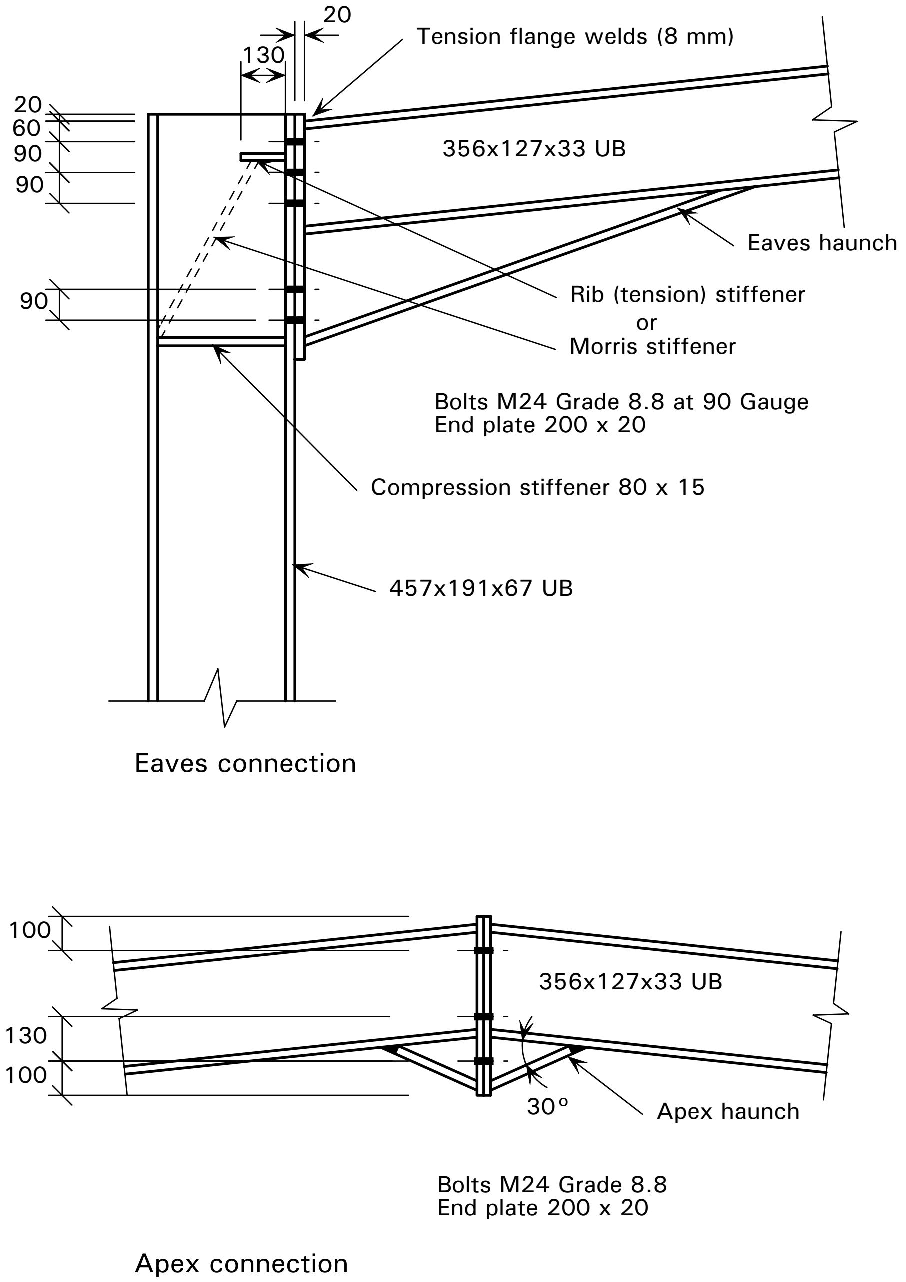


Figure 11.1 *Typical eaves and apex connections in a portal frame*

The detailed design of the connections is generally carried out by the steelwork contractor, but the consultant should be aware of the design process, as sensibly proportioned haunches and columns can reduce the need for stiffening (see Sections 3.4 and 8.1).

Preliminary estimates of connection sizes and details can be obtained from *Joints in steel construction: Moment connections* (P207/95)[24], which presents a number of standard connections in a tabular and easily used form.

In general, three methods, given in the following publications are used in practice for the detailed design of connections:

- *Joints in steel construction: Moment connections*[24]
- Owens and Cheal[25]
- Horne and Morris[26]

All these methods are variations on a similar theme, and within limits they will provide very similar connections. The method given in P207/95 is now widely adopted in the UK as an industry standard, and it is recommended that designs are based on this method. It is a common, agreed approach for use by consulting engineers, steelwork contractors, and checking authorities, which reduces checking time and avoids later disputes.

A series of typical design tables for haunched connections are given in P207/95. These tables do not cover all design cases, but are a useful start point. Final design should be carried out using readily available appropriate software package.

11.2 Types of bolt

Normally, for the eaves and apex connections, non-preloaded bolts (24 mm diameter grade 8.8 bolts) in normal clearance holes are used. For secondary steelwork, 12 mm diameter grade 4.6 bolts are commonly used.

Preloaded high strength friction grip bolts are not normally used, except in cases of heavy dynamic loading, such as where the frame is designed to support crane girders. Such bolts may be used in those connections directly supporting the crane to prevent fretting of the connections and loosening of the bolts. Other connections that may be affected by vibration can be fitted with lock nuts or spring washers to prevent loosening of the bolts.

12 BASES, BASE PLATES AND FOUNDATIONS

12.1 General

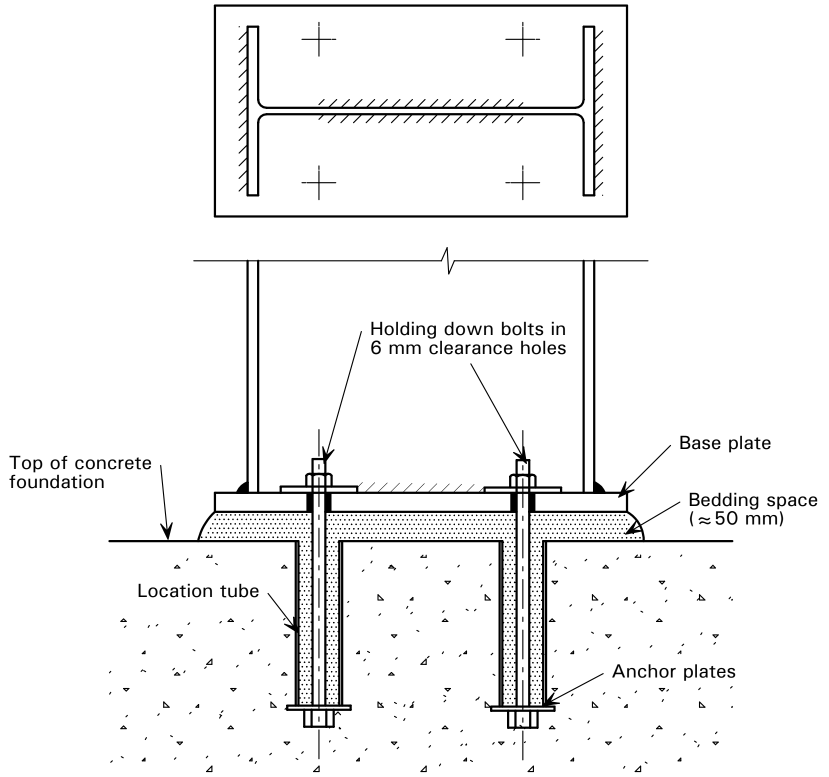
The following terminology for the components at or in the foundations is used in this publication:

- Base - the combined arrangement of base plate, holding down bolts, and concrete foundation. The terms *nominally pinned* and *nominally rigid* are usually applied to the performance of the base, in relation to its restraint of the column.
- Base plate - the steel plate at the base of the column, connected to the column by fillet welds.
- Holding down bolts - bolts through the base plate that are anchored into the concrete foundation.
- Foundation - the concrete footing required to resist compression, uplift, and, where necessary, over-turning moments.
- Anchor plates - plates or angles used to anchor the holding down bolts into the foundation. They should be of such a size as to provide an adequate factor of safety against bearing failure of the concrete.

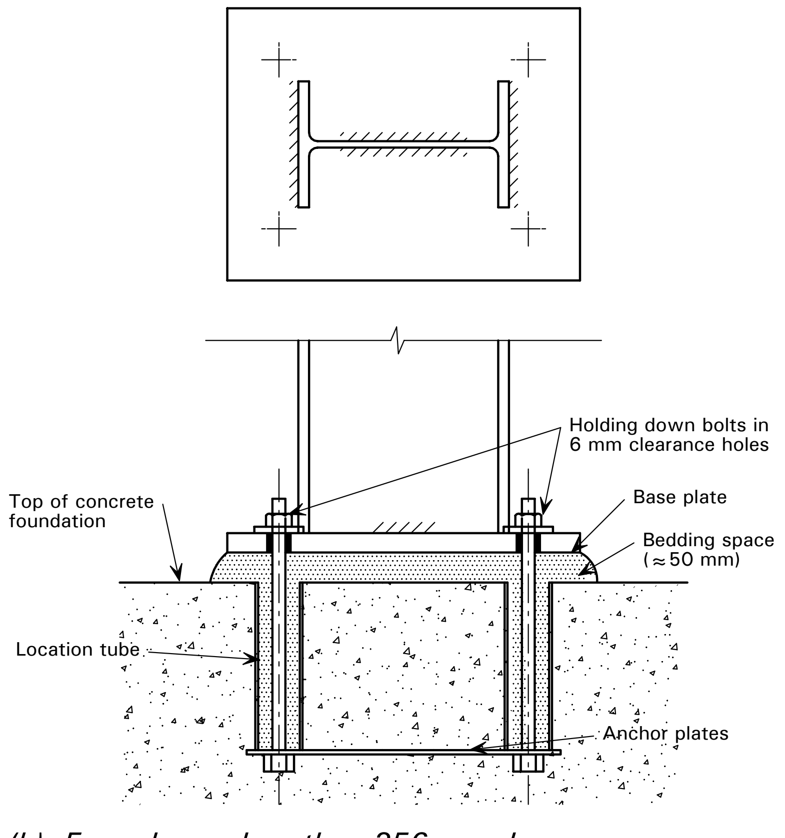
The type of base that is selected will clearly be dictated by a number of issues. In the majority of cases, a nominally pinned base is provided, because of the difficulty and expense of providing a nominally rigid base that is moment resisting. Where crane girders are supported by the column, moment resisting bases may be required to reduce deflections to acceptable limits. Typical base plate/foundation details are shown in Figures 12.1 to 12.4.

In a nominally pinned base for larger columns, the bolts can be located entirely inside a line across the tips of the flanges (Figure 12.1a). For smaller columns (less than say 356 mm), the base plate is made wider so that the bolts can be moved outside the flanges (Figure 12.1b).

A nominally rigid, moment resisting base is achieved by providing a bigger lever arm for the bolts and a stiffer base plate (Figure 12.2). Additional gusset plates may be required for heavy moment connections (Figure 12.3).



(a) For columns greater than or equal to 356 mm deep



(b) For columns less than 356 mm deep

Figure 12.1 *Typical nominally pinned bases*

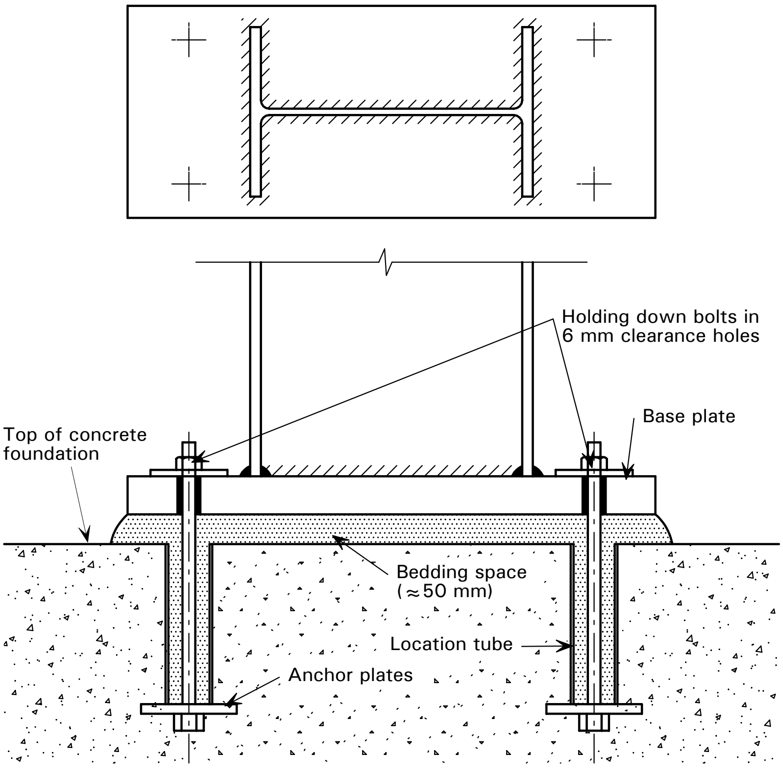


Figure 12.2 *Typical nominally rigid moment resisting base*

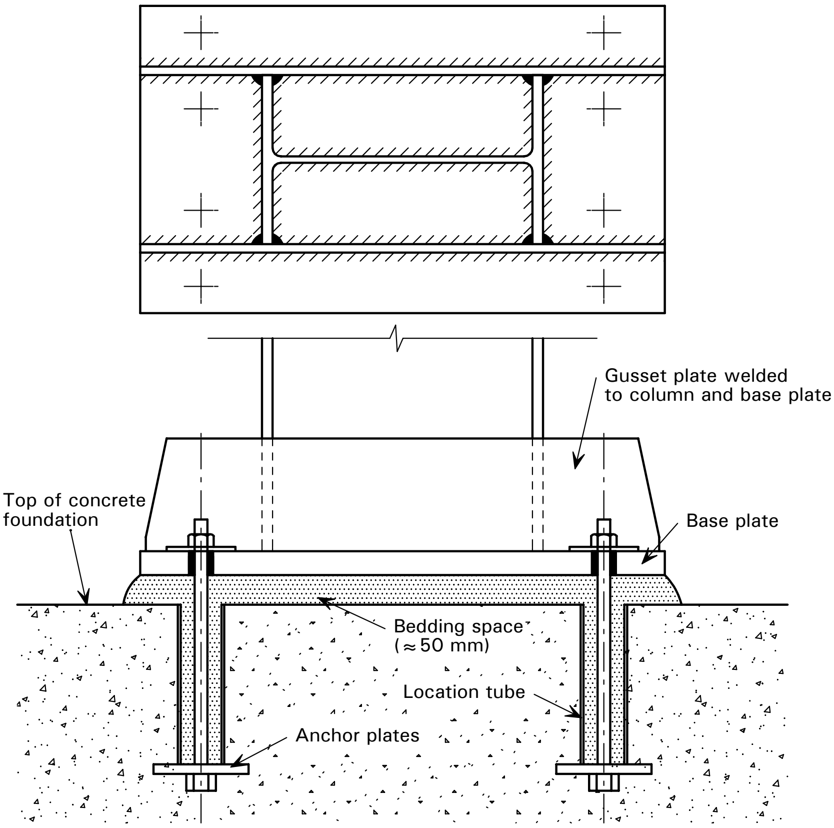


Figure 12.3 *Nominally rigid, moment resisting base with gusset plates for high moments*

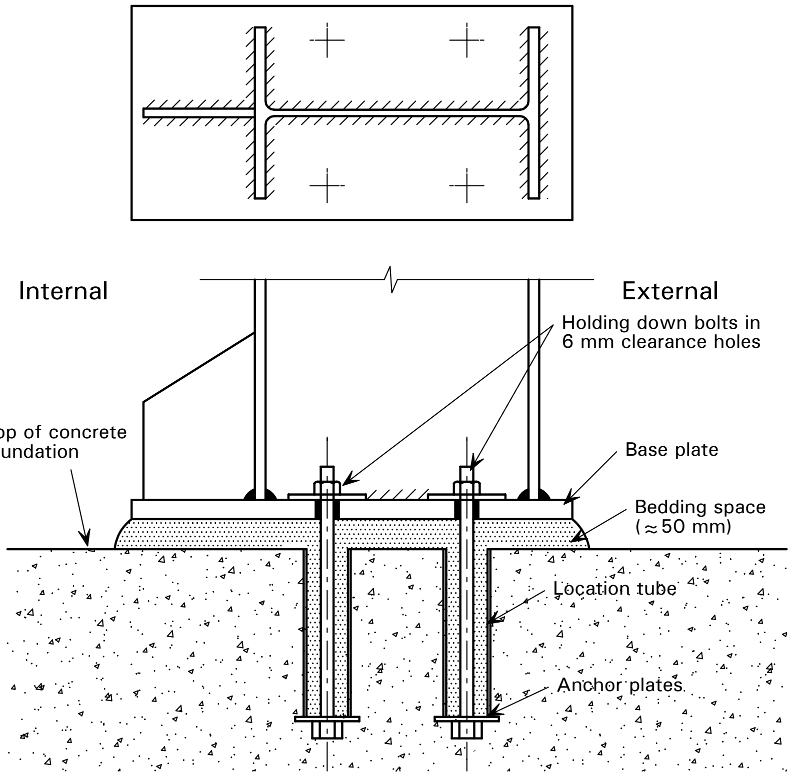


Figure 12.4 *Typical detail with an offset base providing a nominal pin at ultimate limit state and a moment connection at fire limit state*

Where a nominally pinned base is assumed for design at the ultimate limit state, but a moment resisting base is required at the fire limit state, the detail shown in Figure 12.4 may be used.

Detailed design methods are presented in the SCI/BCSA publication *Joints in Steel Construction – Simple Connections* (P212)[27] and *Moment connections* (P207)[24].

12.2 Safety in erection

It is usual to provide at least four bolts in the base plate for stability during erection. The alternative would be to provide temporary guys immediately after the erection of the column, which on most sites would be impractical and is likely to create a hazard.

12.3 Resistance to horizontal forces

The highest horizontal forces acting at the base of the column are generally those that act outwards as a result of bending in the column caused by vertical loading on the roof.

Horizontal reactions acting outwards can be resisted in a number of ways, by:

- Passive earth pressure on the side of the foundation (Figure 12.5a)
- A tie cast into the floor slab connected to the base of the column (Figure 12.5b)

- A tie across the full width of the frame connecting both columns beneath or within the floor slab (Figure 12.5c,d).

By far the most popular method of resisting horizontal forces is to use passive earth pressure. This has economic advantages in that the base size required to resist uplift is usually adequate to provide adequate passive bearing against the ground. However, the passive resistance of the surrounding ground can be suspect 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 short bar connected to the column and cast into the floor slab, and wrapped at the end to allow vertical movement, can be relatively cheap. 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.

In summary, there can be no single recommendation for resisting horizontal forces in all design cases. Each case should be judged on its merits. Recognition of the actual site conditions, and presence of pipes and drains etc., is vital. Adequate liaison between the steel designer, the foundation designer, and the contractor is important at the early stage of the design process to ensure that the correct base detail and tying system is selected.

12.4 Uplift conditions

A key factor in the determination of the size of pad footings for light industrial buildings is the uplift under wind loading.

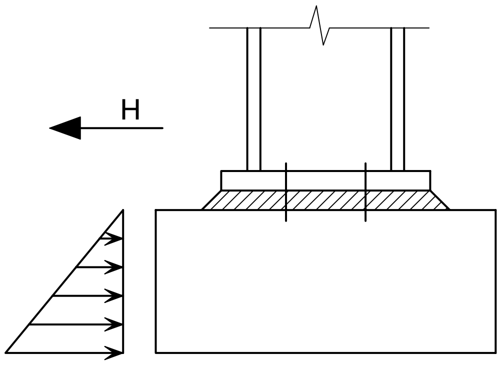
Forces contributing to uplift consist of:

- Direct tension in the column leg, due to internal wind pressure.
- Uplift due to overturning from wind forces on the side of the structure.
- Uplift due to tensile forces in vertical side bracing.

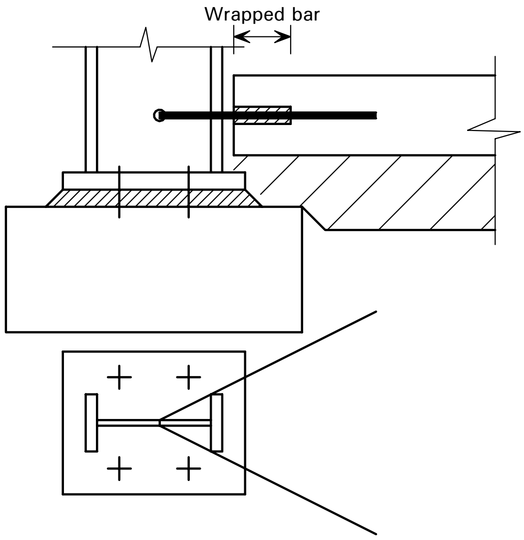
Methods of resisting uplift can be one or more of the following forms:

- Dead weight of the frame.
- Dead weight of the foundation.
- Friction on the side of the foundation in cohesive materials.
- Dead weight of part of the floor slab.

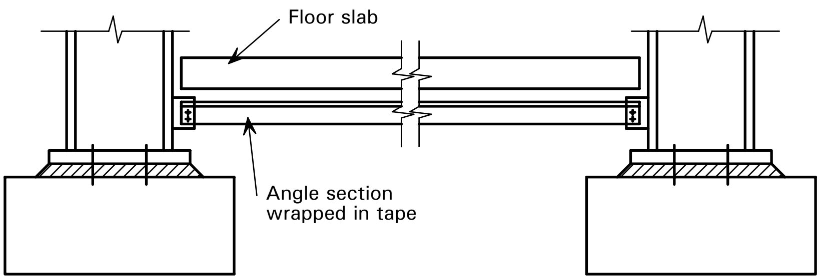
Inclusion of part of the weight of the floor slab requires some consideration. Apart from the weight of the slab and fill directly above the footing, the slab extending beyond the edge of the footing can also contribute. Detailed calculations are not generally appropriate, as they would involve a yield line analysis of the slab, including reinforcement and joints. It is reasonable to take account of a strip of slab of 1 m beyond the edge of the footing as contributing to the restraining force resisting uplift.



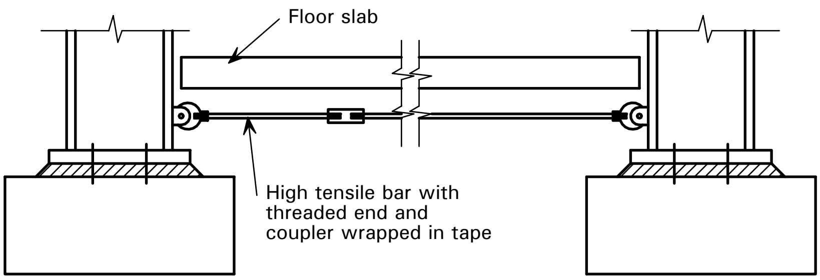
(a) Passive earth pressure



(b) Tie into floor slab



(c) Angle tie between columns



(d) Tie rod between columns

Figure 12.5 *Methods of providing resistance to horizontal forces at the foundations*

12.5 Base plates and holding down bolts

Usually the steelwork contractor will be responsible for detailing the base plate and holding down bolts. However, it should be made clear in the contract documentation where the responsibility for the design of the holding down bolts lies.

Base plates will usually be in grade S275 steel.

The diameter of the bolt will generally be determined by consideration of the uplift and shear forces applied to the bolts, 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 forces and bending on the bolt where grouting is incomplete. Oversize holes are often used to assist with location tolerances in configuration with Form G washers.

The length of the bolt should be determined by the properties of the concrete, the spacing of the bolts, and the 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 may be used to join the bolts together in pairs as an alternative to individual anchor plates. Calculations should be carried out by the designer at the final design stage to check the viability of the proposed bolt spacing.

Detailed design considerations and worked examples are given in the SCI/BCSA publications already mentioned[27][24].

12.6 Foundations

12.6.1 Design at the ultimate and serviceability limit states

If a nominally rigid base is assumed in the frame design, the foundations should be designed for the required moment. If a nominally pinned base is assumed, the foundations are designed only for axial load. In most cases, neither the 'pinned' nor 'rigid' assumptions will be achieved in practice, because:

- where a nominally rigid base is assumed, some rotation will occur, and the moments in the frame will be affected marginally, or
- unless an actual rocker base is provided, the base plate will not be truly pinned and some moment will act at the base of the column.

Generally, this inconsistency is not of great significance and is ignored in designs of hot rolled steel frames, which permit considerable redistribution of internal forces.

Partial fixity at the base is recognised in Clause 5.1.3.4 of BS 5950-1, which gives recommendations for taking account of base stiffness in frame and foundation design. Further guidance on base stiffness is given in Advisory Desk Note AD 194[28]. That advice may be summarised as follows:

1. *The foundation need not be designed for the resulting moment if a nominally pinned base has been assumed to have a partial base fixity equivalent to 10% of the column stiffness for the purpose of:*
    - *evaluating the effective length of the columns*

- *evaluating the elastic critical load factor*
- *classifying the frame as "sway-sensitive" or "non sway".*

2. *The foundation need not be designed for the resulting moment if a nominally pinned base is assumed to have a partial base fixity equivalent to 20% of the column stiffness for the purpose of calculating deflections.*

3. *When advantage is taken of any value of base stiffness (including the nominal 10% column stiffness) at the ultimate limit state, the foundation should be designed for the resulting moment.*

12.6.2 Ground conditions

Generally, portal frames exert a very low ground bearing pressure. The size of base required to resist uplift under wind loading will usually ensure that the bearing pressure under vertical loads is below 100 kN/m^2.

Where portal frames are founded on expansive clays or uncompacted fill, the following design options should be considered:

- Locate the frame on a structural raft, or on a wide reinforced concrete strip footing.
- Use precast ground beams and mini-piles.
- Use bored or driven piles under each footing with linking in-situ or precast ground beams.
- Consider ground improvement techniques, i.e. vibro-replacement under each base.
- If adequate information is available, design the frame for some base movement. A method of dealing with settlement of supports is given in the publication by Davies and Brown[17].

Normal practice is to construct the foundations first, then the steel frame, the cladding, and finally the ground floor slab (which is often power floated). This tends to make the construction of a large structural raft prior to the erection of the frame not only expensive, but also disruptive in terms of the construction programme.

12.6.3 Foundation design at the fire limit state

If the foundation is designed to resist a moment due to rafter collapse in a fire, both the base plate and the foundation itself should be designed to resist the moment (Figure 12.6a). The detail shown in Figure 12.4 may be adopted.

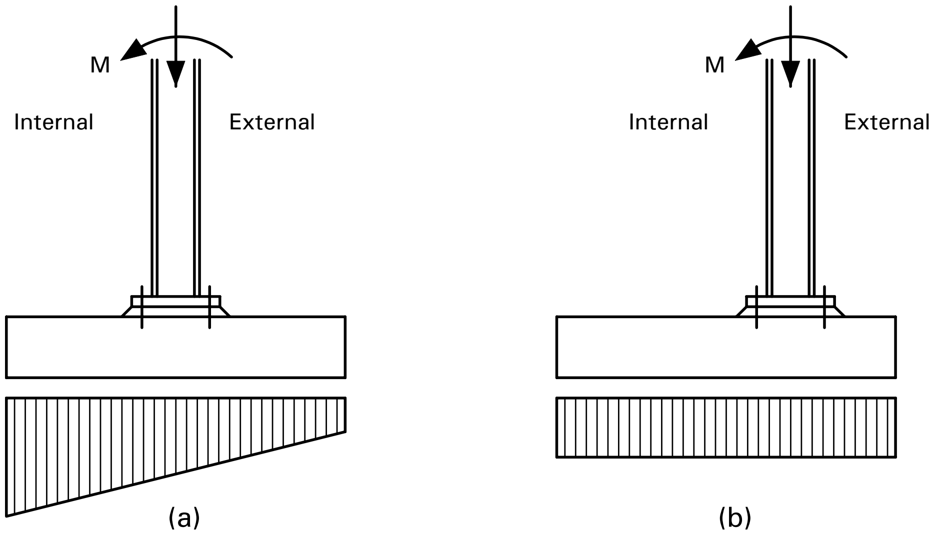


Figure 12.6 *Foundation for portal frame in a fire boundary condition*

It may be possible to offset the base to reduce or eliminate the eccentricity generated by the moment to give a uniform pressure distribution under the base (Figure 12.6b).

A number of factors can be considered in order to reduce the size of the base for design at the fire limit state:

- The ultimate bearing pressure of the ground may be used in this extreme load case. This will usually be in the order of three times the allowable pressure at working load. The ground investigation company may be able to justify a higher bearing strength for the foundation.
- In resisting moment, it has been argued that where the column abuts a floor slab, rotation can take place about the point of contact and the moment can be resisted by passive pressure of the ground on the side of the foundation as well as friction on the base of the concrete (Figure 12.7). In some circumstances this may be valid, but it depends on the condition of the ground and also on the possible presence of services and drainage gullies running parallel to the side of the building.
- Where a tie has been provided below the slab level to resist horizontal forces, this may be utilised to resist moments by tying action in fire conditions.

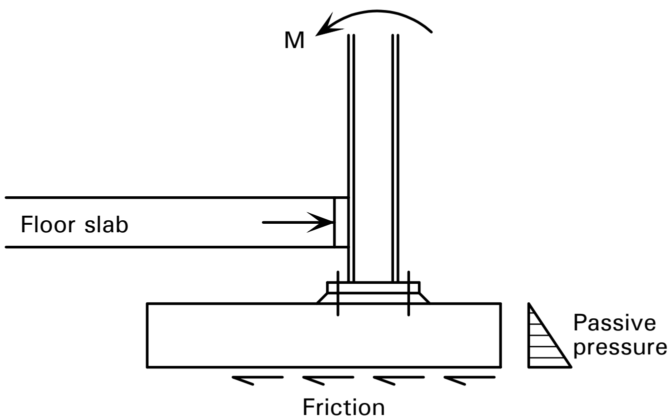


Figure 12.7 *Design of base utilising rotation about the floor slab*

13 SECONDARY STRUCTURAL COMPONENTS

13.1 Eaves beam

The eaves beam connects the individual frames at eaves level (Figure 13.1)

Its primary function is to support the roof cladding, side walls, and guttering along the eaves, but it may also be used to provide lateral restraint at the top of the outer flange of the column.

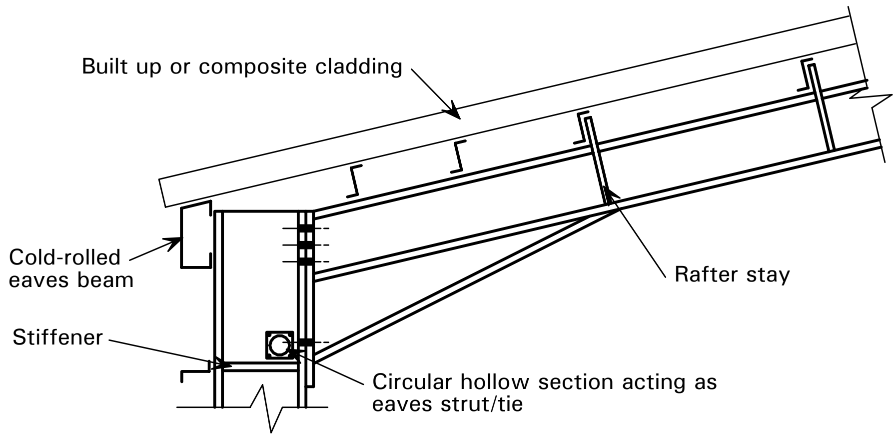


Figure 13.1 *Haunch detail with eaves beam*

13.2 Eaves strut/tie

If vertical side wall bracing capable of resisting tension and compression is provided at both ends of the structure (see Section 9.2), an eaves strut is not required other than in the end bays. However, it is good practice to provide a member between the columns to act as a tie during erection and provide additional robustness to the structure.

If a circular hollow section is used to restrain the plastic hinge at the bottom of the eaves (Figure 13.1), this can fulfil the role of a longitudinal strut/tie as well as restraining the plastic hinge. Further details are given in Section 8.2.1. If a member is provided as an eaves strut/tie above this level (Figure 13.2), it is ineffective in restraining the plastic hinge at the bottom of the haunch.

13.3 Column and rafter stay

A column or a rafter stay is a convenient method of providing bracing to a compression flange that is remote from the flange to which the purlins or side rails are connected. It provides torsional restraint at the location when connected to a suitable purlin or side rail (Figure 13.3).

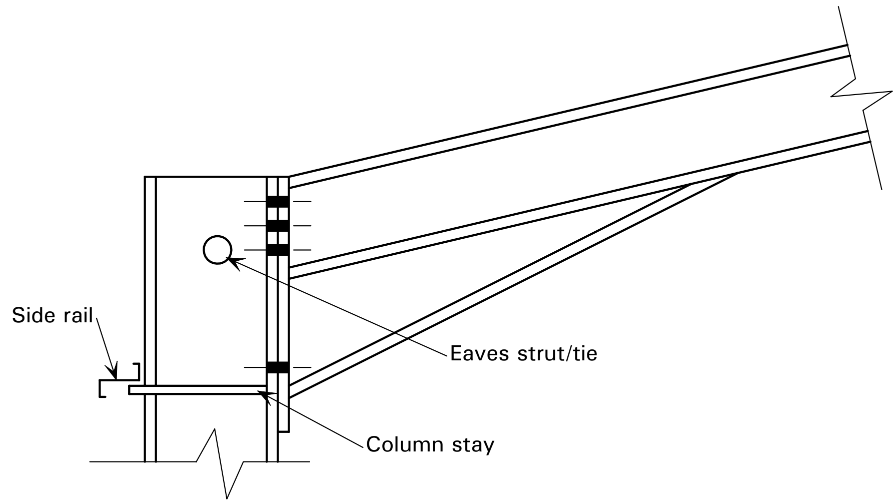


Figure 13.2 *Eaves detail where the eaves strut/tie does not provide restraint at the bottom of the haunch*

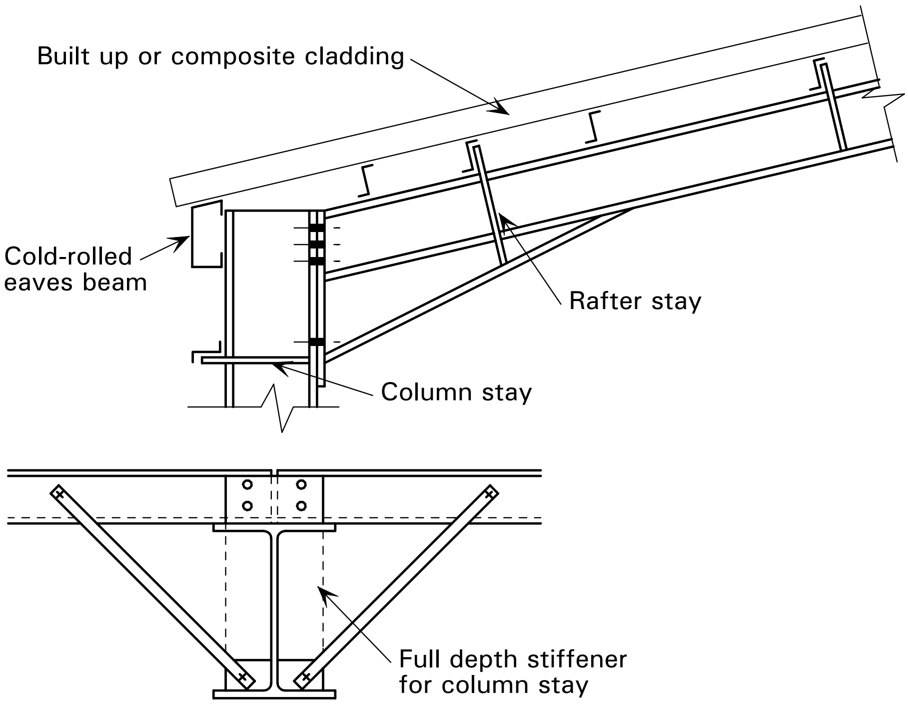


Figure 13.3 *Details of column and rafter stay and connection*

Flats or angles may be used as stays. If flats are used as stays, it should be assumed that they will act in tension only. They are therefore required at each side of the rafter or column (Figure 13.3). If, for detailing reasons, only one stay can be provided, an angle section of minimum size 40 × 40 mm should be 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 the adjacent restraints.

It is important that the purlins or side rails are large enough to provide the required stiffness to act as restraints to the rafter/column. 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 differ from this rule of thumb, the following formula has been suggested by Horne and Ajmani[29] to arrive at a satisfactory size of purlin or side rail:

$$\frac{I_{\mathrm{s}}}{I_{\mathrm{f}}} \geq \frac{f_{\mathrm{y}}}{190 \times 10^{3}} \left(\frac{B(L_1 + L_2)}{L_1 L_2} \right)$$

where:

I_{s} is the second moment of area of purlin or side rail about its major axis

I_{f} is the second moment of area of the rafter or column about its major axis

f_{y} is the yield stress of the rafter or column (in N/mm^2)

B is the span of the purlin or side rail

L_1 and L_2 are the distances either side of the plastic hinge to the eaves, point of contraflexure, or column base, as appropriate.

13.4 Purlin design

13.4.1 General

Purlins are usually proprietary cold formed galvanized sections. The sections have been developed and tested by the manufacturers, and resulting design data is presented in terms of load/span tables or software. The designer should calculate construction and final state loading on the pulins, 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 assumed in the tables and at what point during the construction process this restraint will be provided (see 13.4.4). Manufacturers' tables normally assume that the purlins are restrained by the cladding and designers must ensure that this is so.

Modern continuous purlins are designed using overlaps or sleeves at their supports to achieve greater economy. For hipped roofs (see Figure 3.4), single spans across the hip should be considered carefully, as their depth could determine the depth for the whole roof. Thicker purlins are often used in these cases.

Where purlins are required to provide restraint to the rafter, their location and spacing should be considered with reference to the stability of the rafter near the eaves and apex (see Section 7.1.1).

13.4.2 Localised loading

Local increases in loading exist due to wind loads on the end bays or drifting snow against an adjacent structure, parapet or gable. The best way of dealing with these higher loads will generally be to maintain the same purlin spacing and depth, by providing an increased purlin thickness in these locations. Alternatively, purlin spacing may be reduced.

13.4.3 Types of purlin

A number of types of purlins are produced in the UK and typical shapes are shown in Figure 13.4. Each manufacturer produces its own specific shapes to from 1.3 to 2.8 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 correct selection of purlin type.

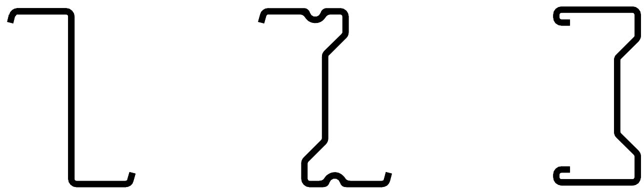


Figure 13.4 *Common types of purlin*

13.4.4 Purlin restraint

To allow an economical choice of purlin, restraint should be provided by the cladding through the fixings. How and when the restraint will be provided on site should be carefully considered by the designer (the specifier of the purlins).

If "built-up" cladding is used, the liner tray must be sufficiently robust and have sufficient fixings to laterally restrain the top flange of the purlin and to prevent LTB under gravity (downward) construction loads. Because of the insulation thickness that is now required (following the 2002 amendments of Building Regulations Parts L2 and J)[16], composite interaction between the inner and outer sheets of "built-up" systems may be limited. It is, therefore, recommended that, in the final state, the liner tray must be chosen so that it alone is still adequate to restrain the purlin under the now more significant downward loading. Under uplift (wind) loading, it is the lower flange of the purlin that is in compression. The size of the purlin can be reduced if the liner trays (and fixings) provide sufficient torsional restraint to the top flange, in conjunction with the torsional stiffness of the purlin itself, to limit movement of the lower flange. When "composite panels" are used, they will provide significant lateral and torsional restraint to the purlin, provided that the number and strength of the fixings is adequate.

Manufacturers give guidance on the types of liner tray that they feel are adequate to provide restraint. The most important thing for the purlin specifically is NOT simply to assume that adequate restraint will be provided. Components should be chosen to ensure restraint is provided, or bigger purlins (which will work even if unrestrained) chosen.

Anti-sag ties (usually small rods or angles bolted or clipped between purlins) at mid-span or third points may also be used to restrain the purlins and these have the added benefit of reducing misalignment of the purlins during fixing of 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. A typical arrangement of anti-sag ties is shown in Figure 13.5. As an alternative to the use of anti-sag ties, the use of bigger purlins could have the benefit of reduced impact on the construction process.

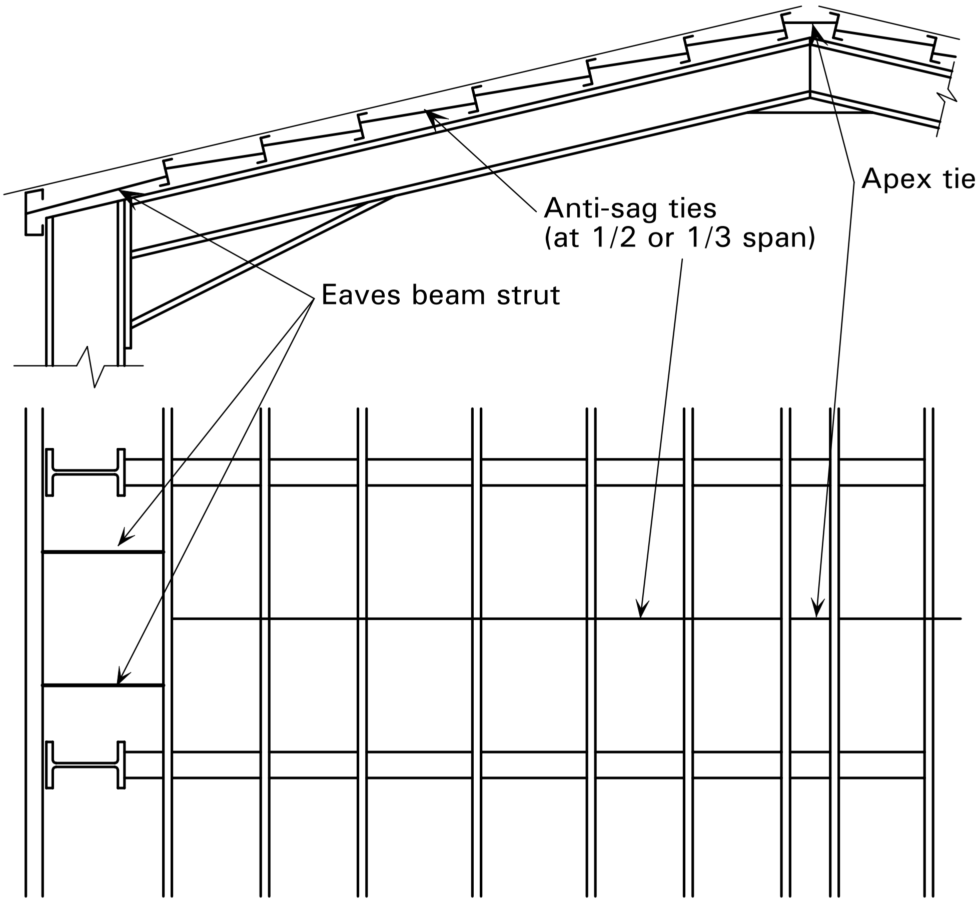


Figure 13.5 *Typical anti-sag ties and eaves beam strut layout*

13.4.5 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 ability to provide semi continuity by the use of sleeves or overlaps for maximum efficiency.

The purlin manufacturer should be consulted before the layout is finalised. The following examples are given to illustrate the issues involved in the choice of layout.

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 (Figure 13.6). At the penultimate support, sleeves are provided at each purlin, to provide semi-continuity and additional strength in the end bay. This system is considered to be the most efficient for buildings with bay centres between 5 m and 7 m. Heavier sections can be provided in the end bay, if necessary.

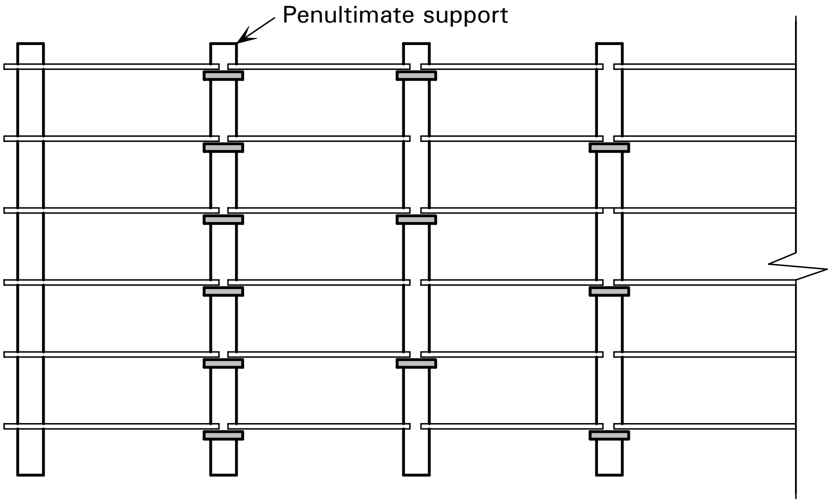


Figure 13.6 *Single-span lengths - sleeved system*

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 (Figure 13.7). Frames are often spaced closer to reduce the purlin size.

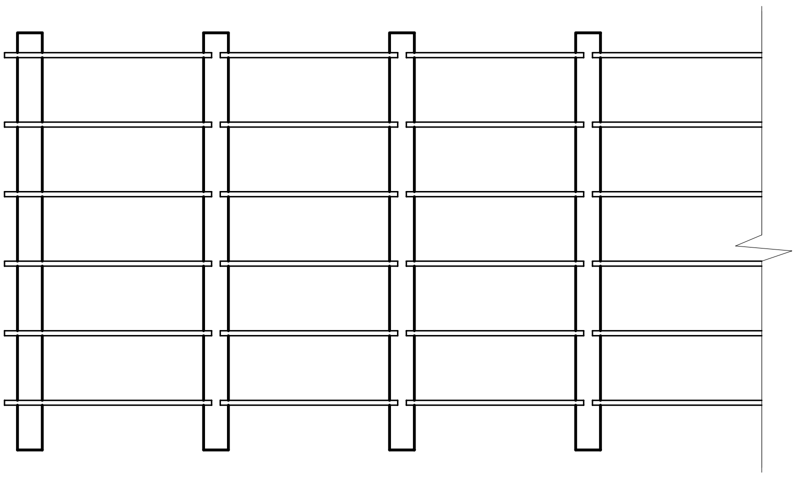


Figure 13.7 *Single-span lengths - butted system*

Single-span lengths - overlapping system

An overlapping system provides greater continuity and can be used for heavy loads and long spans (Figure 13.8). It is best suited to buildings with a large number of bays.

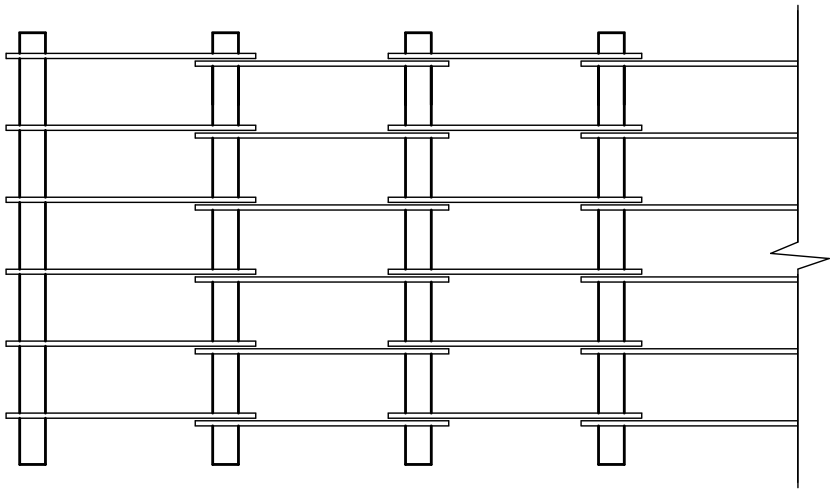


Figure 13.8 *Single-span lengths - overlapping system*

Double-span lengths – non sleeved system

In this system, the double-span lengths are staggered (Figure 13.9). Sleeves are provided at the penultimate supports to ensure semi-continuity. The capacity will generally be less than for the equivalent double-span sleeved system, but double-span purlins use fewer components and lead to faster erection. This system is limited to bay sizes less than 8 m, for reasons of transport and erection of the purlins.

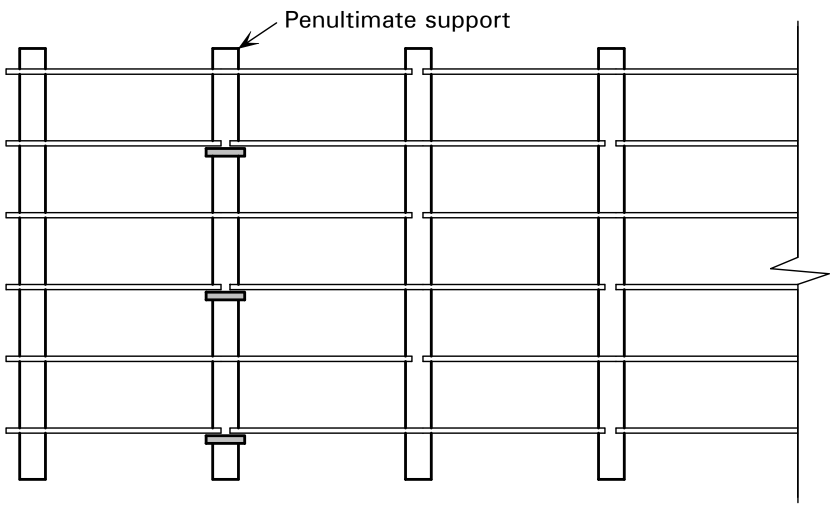


Figure 13.9 *Double-span lengths – non sleeved system*

Double-span lengths - sleeved system

In double-span sleeved systems, the double-span lengths are staggered and sleeves are provided at alternate supports (Figure 13.10). Sleeves are provided to every purlin at the penultimate support to ensure semi-continuity. A double-span sleeved system has a slightly higher capacity than the double-span non-sleeved system and has the advantages of semi-continuity at all "sleeve" positions. This system is 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.

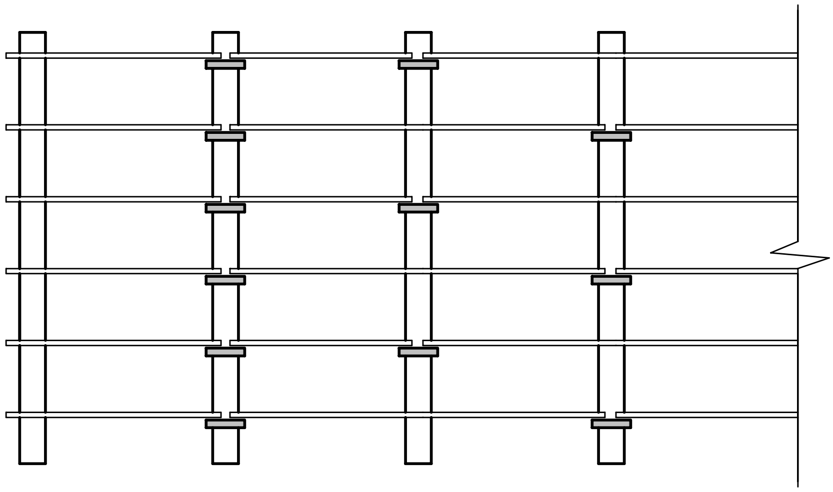


Figure 13.10 *Double span lengths - sleeved system*

13.4.6 Purlins providing rafter restraint

Purlins are usually required to provide restraint to the rafters. BS 5950-1, Clause 4.3.2.2.4 states that purlins need not be checked for forces arising from the restraint of rafters of portal frames, 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 their plane, or alternatively the roof sheeting is capable of acting as a diaphragm.
- The loading arises predominantly from roof loads (see below).

The three conditions are usually satisfied, because roof sheeting will be adequately fixed, sufficiently robust liner trays or composite panels (but see the discussion in Section 13.4.4). Furthermore, bracing is generally located in the plane of the roof at the end of the building.

Disagreement may arise over the expression *roof loads*. Within the context of Clause 4.3.2.2.4, the term can be understood to mean uniformly distributed load that is transmitted to the rafter via the purlins and would include most normal service loads. This would ensure that, within reason, the purlin size is commensurate with that of the rafter it is restraining, and that there are no high local point loads that would require additional restraint. It is therefore reasonable to assume that in most cases the purlins should not be designed for axial loads arising from their function as a rafter restraint.

Where it is necessary to design the purlins for axial load, they should be capable of resisting forces determined in accordance with BS 5950-1, Clause 4.3.2.2.

13.4.7 Purlin cleats

Purlins are attached to rafters using cleats that are usually welded to the rafter in the shop before delivery to site. However, the practice of bolting cleats is becoming more common because:

- Economy is provided by modern punching and drilling lines
- Transportation is easier and cheaper, as the rafters stack more compactly (this is a particularly important consideration where the steelwork is for export)
- There is greater flexibility for alignment of the purlins, which will facilitate fixing of some cladding systems.

If the purlin cleats are bolted, two important considerations should be taken into account:

- The moment capacity of the section will be reduced if the holes reduce the area of the flange by more than 20% for S275 steel and more than 10% for S355 steel (see BS 5950-1, Clause 4.2.5.5).
- Holes should either be drilled full size or punched at least 2 mm undersize and then reamed in areas adjacent to plastic hinge positions (see BS 5950-1, Clause 5.2.3.4).

13.5 Side rails

Essentially, side rail design and detailing are 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 side 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 13.11).

Special details are required for side rails in fire boundary conditions (see Section 13.6.5).

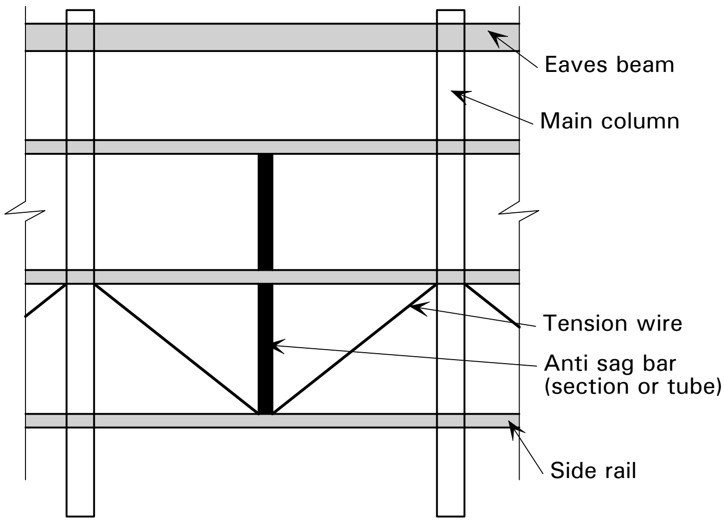


Figure 13.11 *Side rail restraint*

13.6 Cladding

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

13.6.1 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 (Figure 13.12). In some cases, single-skin sheeting is used with insulation suspended directly beneath the sheeting.

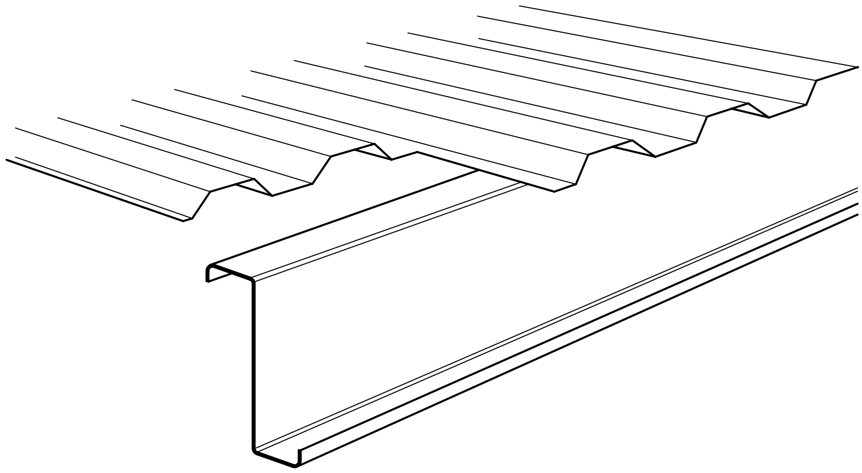


Figure 13.12 *Single-skin trapezoidal sheeting*

13.6.2 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 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 level of restraint to the purlins. Alternative forms of construction using plastic ferrule and Z or rail-and-bracket spacers are shown in Figures 13.13 and 13.14.

As insulation depths have increased, there has been a move towards rail-and-bracket solutions as they provide greater stability.

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.

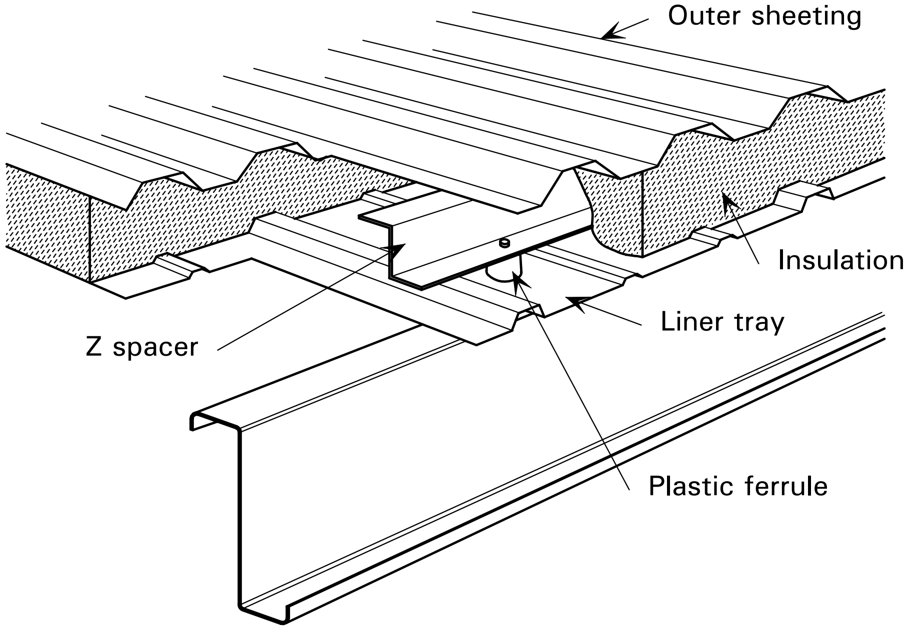


Figure 13.13 *Double-skin construction using plastic ferrule and Z spacers*

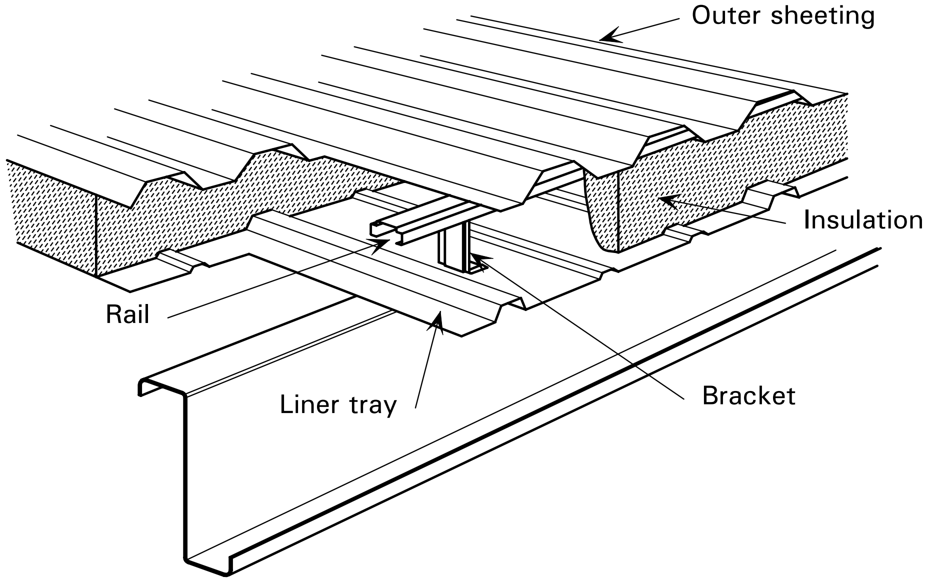


Figure 13.14 *Double-skin construction using rail-and-bracket spacers*

13.6.3 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 is rapid. The fastenings are in the form of clips that hold the sheeting down but allow it to move longitudinally (Figure 13.15). The disadvantage is that significantly less restraint is provided to the purlins than with a conventionally fixed system. Nevertheless, a correctly fixed liner tray will provide adequate restraint.

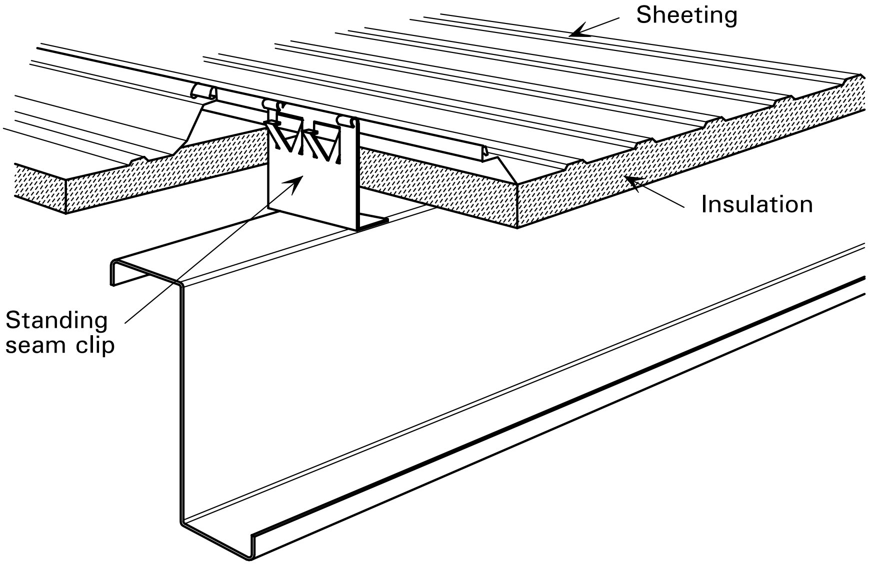


Figure 13.15 *Standing seam panels*

13.6.4 Composite or sandwich panels

Composite or sandwich panels are formed by creating a foam 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 (Figure 3.16) 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.

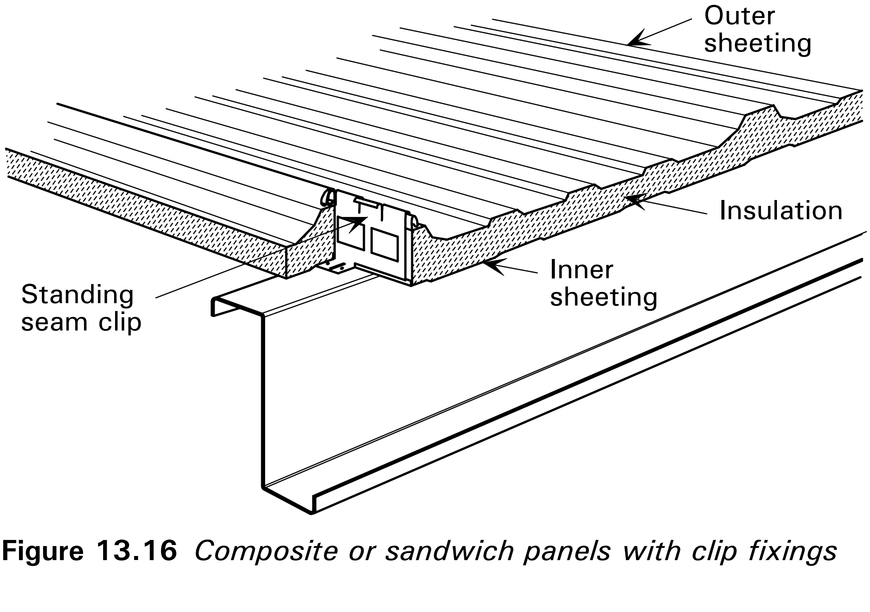


Figure 13.16 *Composite or sandwich panels with clip fixings*

13.6.5 Walls in fire boundary conditions

Where buildings are close to a site boundary (see Section 15), the 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. Due to the construction used for the fire test specimens, it is considered necessary by some manufacturers and local authorities to 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 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 13.17).

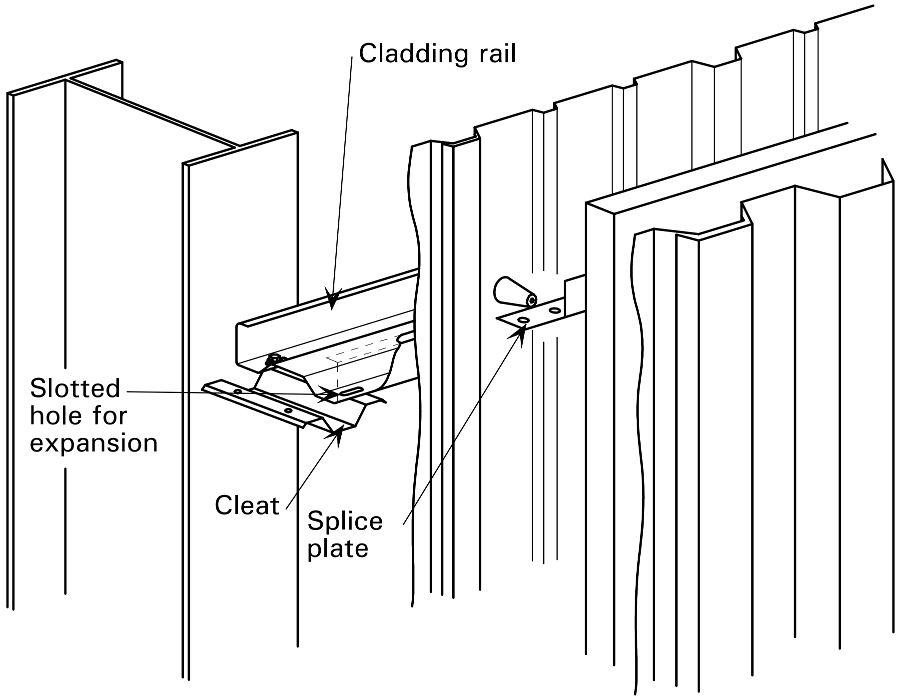


Figure 13.17 *Typical fire wall details showing slotted holes for expansion*

14 SERVICEABILITY ASPECTS OF FRAME DESIGN

14.1 Deflections

Elastic analysis is used to determine the deflections of the frame

Deflections due to dead load and self weight are often allowed for by pre-setting the frame. As a consequence of this, the columns will exhibit a certain amount of *lean in* when initially erected. These deflections and presets are not considered at the Serviceability Limit State.

At the Serviceability Limit State, deflections due to imposed load and wind load only are considered.

Portal frames clad in steel sheeting deflect significantly less than 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 (see Section 9.3). The actual deflection depends on the building proportions and cladding type, but reductions in horizontal deflections (from those calculated for the bare frame) of over 50% are typical of real structures.

The maximum acceptable deflections in portal frames depend on many factors, such as the building use and cladding type, which is perhaps why BS 5950-1 does not give a limitation on deflections at SLS. Since its first publication in 1985, a number of attempts have been made to give sensible limitations for acceptable deflections.

The SCI publication (P070) *Steelwork design guide to BS 5950 - Volume 4: Essential data for designers*[30] presents a number of indicative deflection limits for portal frames, taking into consideration the following:

- **Sheeting**: Limits on differential deflection between frames are necessary to prevent the fixings between the sheets and the frame from becoming overstrained, resulting in tearing of the sheeting, and leakage.
- **Gables**: Sheeted and/or braced gable ends are very stiff in their own plane and their deflections can be ignored. This often makes differential deflections between the end frame and the adjacent frame appear very high both horizontally and vertically.
- **Masonry**: When brick or blockwork side walls are constructed so that they receive support from the steel frame when resisting wind loads, they should be detailed such that they can deflect with the frame by using a compressible damp proof course at the base of the wall.
- **Base fixity**: In order to provide stability during erection, it has become common to use four holding down bolts in nominally pinned bases. In this situation, it would be reasonable to use a base stiffness of up to 20% of the column stiffness at the serviceability limit state for nominally pinned bases and to adopt full fixity for nominally rigid bases.
- **Cranes**: Where crane girders are supported directly from portal frames, the critical deflection is that which causes a difference in the spacing of the rails from one side of the building to the other. Possible ways of reducing this difference are to provide nominally rigid bases, tie the frame at the

eaves (but note headroom restrictions), and/or provide stepped crane columns.

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

The recommendations given in *Steelwork design guide to BS 5950 - Volume 4: Essential data for designers* are presented in Table 14.1. A special note was issued with the guide stating 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." No further progress has yet been made with regard to a definitive set of values.

Masonry cladding, comprising brickwork, concrete blockwork, or precast concrete units, is assumed to be seated on a damp proof layer and supported against wind by the steel frame. The height h should always be taken as the height to eaves, **not** the height of the masonry panel.

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 of the given criteria should be observed.

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 (see Section 10.7).

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 14.1 gives provisional 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 14.1 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. In particular, standing seam roof systems possess little stiffness, whereas *double skin* roof systems are quite stiff (see Section 9.3).

The **differential** deflection limits in Table 14.1 may be compared with the calculated deflection of a frame that has restraint from the roof system. Designers often take notional account of the stiffness of the roof system by reducing the calculated **differential** deflections between adjacent bare frames by up to 50%. Again, the actual reduction taken will depend on the stiffness of the cladding.

Note: BS 5950-1 includes 'deflection limits' related to checks on frame stability: these should not be confused with limitations on deflection at SLS.

Table 14.1 *Recommended deflection limits (taken from Steelwork design guide to BS 5950 - Volume 4: Essential data for designers[30])*

a) Horizontal deflection at eaves level - due to unfactored wind load or unfactored imposed load or 80% of unfactored (wind and imposed) loads

Type of cladding	Absolute deflection	Differential deflection relative to adjacent frame
Side cladding:		
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	$\leq h/200$	$\leq (h^2+b^2)^{0.5}/330$
Roof cladding:		
Profiled metal sheeting	--	$\leq b/200$
Fibre reinforced sheeting	--	$\leq b/250$
Felted metal decking	--	$\leq b/400$

b) Vertical deflection at ridge (for rafter slopes $\geq 3^{\circ}$) - due to unfactored wind load or unfactored imposed load or 80% of unfactored (wind and imposed) loads

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 $\leq (b^2+s^2)^{0.5}/165$
Felted metal decking	
- supported on purlins	$\leq b/100$ and $\leq (b^2+s^2)^{0.5}/125$
- supported on rafter	$\leq b/200$ and $\leq (b^2+s^2)^{0.5}/250$

General: The above values are provisional recommendations from *Steelwork design guide to BS 5950 - Volume 4: Essential data for designers*; feedback has suggested that some of the values may be more stringent than necessary.

The values of *h, b,* and *s* are defined in Figure 14.1.

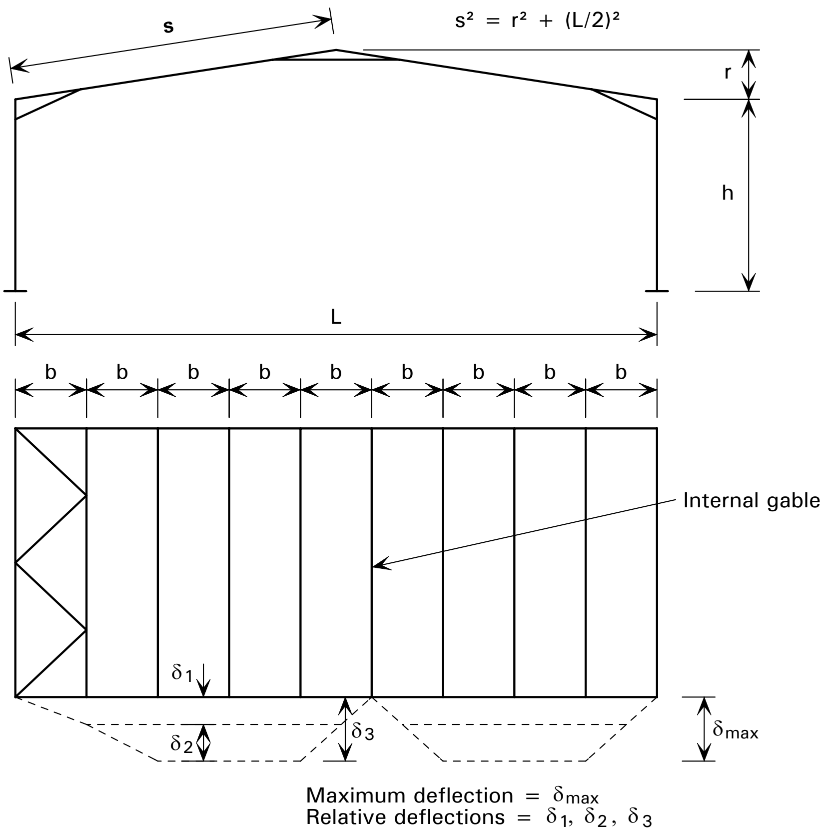


Figure 14.1 *Portal dimensions used in determining deflection limitations in Table 14.1*

14.2 Thermal expansion

14.2.1 Introduction

In the UK, temperature movements are generally small and no additional calculations are required where the spacing of expansion joints given in Table 14.2 is observed.

Table 14.2 *Maximum spacing of expansion joints in buildings*

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	15
	Calcium silicate bricks	9
	Concrete blocks	6

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 BS 5628 [31].

The publication P070[30] addresses the subject of thermal expansion in detail. It notes that concern about provision for expansion joints is based on two main issues:

- Longitudinal thermal expansion of long buildings and of frames relative to the foundations.
- The movement of the sheeting or brickwork cladding relative to the protected steel frame (the movement could be greater or less). Allowance for differential movement between the frame and the cladding should be made.

The guide explains why, based on the BS 5950-1 temperature range of –5°C to +35°C, various limiting lengths between expansion joints have been proposed in the past for particular situations, and warns against the use of a general limit for all structures. Buildings designed in hotter or more variable climates may require stricter limits.

14.2.2 Thermal performance of structural elements

Consideration should be given to the performance of individual components within buildings when subject to change of temperature. The following points should be noted:

Pitched rafters

The effect of thermal expansion will have an insignificant effect on the stresses in the frame. For tied frames, the stress in the tie will increase when the rafters become hotter than the tie.

Clearance holes

Theoretically, clearance holes allow a 2 mm movement, but due to different fit-up on a number of bolts, it is likely that the actual movement of a single bolt group will be in the order of only 1 mm.

Purlins and sheeting rails, which are at most continuous over two bays, have a possible expansion of 2.4 mm over a length of 10 m, compared to a possible slip in the connections of 1 mm each end. There is therefore no significant problem with the expansion of individual purlins and rails.

Braced bays

It is usual to provide a braced bay at each end of the structure, thereby restraining thermal expansion. In long buildings, it may be advantageous to provide braced bays at intervals along the building in order to restrain longitudinal movement of the whole structure.

Sheeting

For profiled steel sheeting, movements along the building can be accommodated by the *concertina* action of the sheeting perpendicular to the span. Parallel to the span, movement occurs by some slippage in the fixings. There is generally no problem with sheet lengths up to 20 m, but manufacturers' literature should be consulted for detailed information.

Brickwork and blockwork

Differential expansion between the steel frame and the masonry is covered by the provisions for expansion joints in BS 5628[31], and there is no need to provide additional expansion joints in the steel frame to accommodate this movement (see Table 14.2). Where expansion causes eaves spread, the movement can usually be accommodated by the rotation of the base of the brickwork about the damp proof layer.

14.2.3 Expansion joints

The provision of satisfactory expansion joints is neither easy nor cheap, and it is usually better to detail the structure such that such 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.

The following points should be considered when detailing these expansion joints:

Joints in steelwork

In heavier steel members, simple slotted holes are unlikely to be effective, and sliding bearings are unlikely to be economic. The most practical solution is to provide a complete break in the steel structure. In many cases, joints can be provided in the purlins, side rails, and sheeting between the frames resulting in two separate buildings joined only by purlins and side rails. Each individual portion of the building should be braced and vertical braced bays should be positioned midway between expansion joints.

Joints in sheeting rails

Slotted holes may be used in the purlins and sheeting rails. Special bolts, nuts, and washers, such as shouldered bolts, spring washers, and/or lock nuts, should be considered to allow expansion and prevent the bolts coming loose. Where the purlins or rails provide restraint to the members, an alternative restraint system must be provided at the expansion joints.

Joints in crane girders and runway beams

Where longitudinal expansion joints are required, the two adjacent crane girders should be supported separately, although a halving joint with a sliding bearing is also possible. The rail should be connected to the girder by the use of clips, in such a way that differential movement is possible. Scarf joints in the crane rail should be provided where necessary, and provision made for easy replacement of the expansion joint section as this usually receives the most wear. Where expansion of long-span frames causes eaves spread, consideration should be given to the effect that this may have on the crane use, particularly if the building may be subject to extreme variations of internal temperature. In all cases, the crane manufacturer should be consulted to ensure that specific requirements regarding tolerances, etc. can be accommodated.

Runway beams should preferably not have expansion joints, although consideration should be given to providing some flexibility in the connection to the main frame to allow for differential movement. This can usually be achieved by the flexibility of the supporting members.

15 PORTAL FRAMES IN FIRE BOUNDARY CONDITIONS

15.1 Introduction

Structural elements of multi-storey buildings are required, by building regulations, to have fire resistance to prevent, amongst other things, structural collapse in the event of a fire. However, single storey buildings are only required to have fire resistance when fire spread between buildings is of concern. Fire resistance is normally only specified for the external wall and the proportion of the wall that is required to be fire resistant reduces as the distance from the site boundary increases. Any structure that provides support to such walls also has to have fire resistance. In common practice, if any part of an external wall requires fire resistance, the building is said to have a *fire boundary condition*.

SCI publication P313 *Single storey steel framed buildings in fire boundary conditions*[14] provides design recommendations and guidance for single storey buildings for design in fire boundary situations. It shows that fire protection to the roof structure, which would be expensive to provide, is not necessary, provided that recommendations on column base design are followed. The advice and recommendations cover single and multi-bay portal frames, monopitch portal frames, gable frames and frames with trussed roofs.

The recommendations are generally applicable throughout the UK, although the separate regulations and other documents for England and Wales, for Scotland and Northern Ireland should be consulted. In each of the sets of regulations, the treatment of external walls is similar but there is a difference in the way the benefits of sprinklers are dealt with.

15.2 Building regulations

15.2.1 England and Wales

In England and Wales, the *Building Regulations*[15] contain simple functional requirements. These requirements use words such as *reasonable* and *adequate* but impose no specific limits, for example, in terms of the period of fire resistance. Periods of fire resistance and other quantified requirements are given in *Approved Document B* [16]. Compliance with Approved Document B is considered as evidence that the requirements of the Building Regulation have been met.

The Regulations state:

"The building shall be so constructed that, in the event of fire, its stability will be maintained for a reasonable period".

and

"The external walls of the building shall offer adequate resistance to the spread of fire over the walls and from one building to another, having regard to the height, use and position of the building".

The first of these points is important, as it implies that the design solution adopted need only be "reasonable". The second point leads to the general requirements for "adequate" space separation.

15.2.2 Scotland

In Scotland, requirements are set out in the *Building Standards (Scotland) Regulations*[32]. A set of *Technical Standards*[33] is provided as a guide to compliance with the regulations. Compliance with these Technical Standards constitutes compliance with the Regulations, although alternative solutions can be accepted by local authorities.

Unlike the Regulations for England and Wales, the Scottish Regulations limit the size of compartments in single storey buildings. For industrial and low risk storage buildings, these limits are not onerous.

15.2.3 Northern Ireland

The Building Regulations (Northern Ireland)[34] express their requirements in terms of performance rather than prescribed methods and standards. *Technical Booklet E*[35] provides advice on methods and standards that will satisfy the requirements of the Building Regulations. However, this is not a prescriptive document and there is no obligation to comply with the guidance given.

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APPENDIX A Preliminary design methods

A.1 Introduction

This Appendix contains two alternative methods for determining the size of columns and rafters of single span portal frames at the preliminary design stage. Further detailed calculations will be required at the final design stage. Both methods are conservative relative to detailed design at the ultimate limit state, but it should be noted that neither method takes 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 Estimation of member sizes

A.2.1 Method 1 - tabulated member sizes

The publication *Portal frames*[36] presents tables that permit a rapid determination of member size to be made for estimating purposes. The span range is 15 to 40 m. A reformatted version of the tables from the publication is presented in Table A.1 here. The assumptions made in creating this table are as follows:

- The roof pitch is 6°.
- The steel grade is S275.
- The rafter load is the total factored dead load (including self weight) and factored imposed load.
- 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 in its length.

The member sizes given by the tables are suitable for rapid preliminary design, or at the estimating stage. However, where strict deflections limits are specified, it may be necessary to increase the member sizes. Where an asterisk (*) is shown in the table, a suitable section size has not been calculated.

Table A.1 *Symmetrical 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	6	254 × 102 × 22 UB	356 × 127 × 33 UB	406 × 140 × 39 UB	406 × 140 × 46 UB	406 × 178 × 60 UB	457 × 191 × 67 UB
	8	8	254 × 102 × 22 UB	356 × 127 × 33 UB	406 × 140 × 39 UB	406 × 178 × 54 UB	457 × 191 × 67 UB	457 × 191 × 74 UB
	8	10	254 × 102 × 22 UB	356 × 127 × 33 UB	406 × 140 × 39 UB	406 × 178 × 54 UB	457 × 191 × 67 UB	457 × 191 × 74 UB
	8	12	*	356 × 127 × 33 UB	406 × 140 × 39 UB	406 × 178 × 54 UB	457 × 191 × 67 UB	457 × 191 × 74 UB
Restrained column	8	6	305 × 165 × 40 UB	356 × 171 × 51 UB	457 × 191 × 67 UB	533 × 210 × 82 UB	533 × 210 × 92 UB	610 × 229 × 113 UB
	8	8	305 × 165 × 40 UB	356 × 171 × 51 UB	457 × 191 × 67 UB	533 × 210 × 82 UB	610 × 229 × 101 UB	610 × 229 × 113 UB
	8	10	305 × 165 × 40 UB	406 × 178 × 54 UB	457 × 191 × 67 UB	533 × 210 × 82 UB	610 × 229 × 101 UB	686 × 254 × 125 UB
	8	12	*	406 × 178 × 54 UB	457 × 191 × 67 UB	533 × 210 × 82 UB	610 × 229 × 101 UB	686 × 254 × 125 UB
Unrestrained column	8	6	356 × 171 × 51 UB	457 × 191 × 67 UB	533 × 210 × 82 UB	533 × 210 × 92 UB	610 × 229 × 113 UB	686 × 254 × 125 UB
	8	8	406 × 178 × 60 UB	533 × 210 × 82 UB	610 × 229 × 101 UB	610 × 229 × 113 UB	686 × 254 × 125 UB	762 × 267 × 147 UB
	8	10	457 × 191 × 67 UB	533 × 210 × 92 UB	610 × 229 × 113 UB	686 × 254 × 125 UB	762 × 267 × 147 UB	762 × 267 × 173 UB
	8	12	*	610 × 229 × 101 UB	686 × 254 × 125 UB	762 × 267 × 147 UB	762 × 267 × 173 UB	838 × 292 × 194 UB
Rafter	10	6	305 × 102 × 25 UB	356 × 127 × 33 UB	406 × 140 × 46 UB	406 × 178 × 60 UB	457 × 191 × 67 UB	533 × 210 × 82 UB
	10	8	305 × 102 × 25 UB	356 × 127 × 33 UB	406 × 140 × 46 UB	406 × 178 × 60 UB	457 × 191 × 74 UB	533 × 210 × 82 UB
	10	10	305 × 102 × 25 UB	406 × 140 × 39 UB	406 × 140 × 46 UB	406 × 178 × 60 UB	457 × 191 × 74 UB	533 × 210 × 82 UB
	10	12	*	406 × 140 × 39 UB	406 × 140 × 46 UB	457 × 191 × 67 UB	457 × 191 × 74 UB	533 × 210 × 92 UB
Restrained column	10	6	356 × 171 × 45 UB	406 × 178 × 60 UB	457 × 191 × 74 UB	533 × 210 × 92 UB	610 × 229 × 113 UB	686 × 254 × 125 UB
	10	8	356 × 171 × 45 UB	406 × 178 × 60 UB	533 × 210 × 82 UB	533 × 210 × 92 UB	610 × 229 × 113 UB	686 × 254 × 125 UB
	10	10	356 × 171 × 45 UB	406 × 178 × 60 UB	533 × 210 × 82 UB	610 × 229 × 101 UB	610 × 229 × 113 UB	686 × 254 × 140 UB
	10	12	*	406 × 178 × 60 UB	533 × 210 × 82 UB	610 × 229 × 101 UB	686 × 254 × 125 UB	686 × 254 × 140 UB
Unrestrained column	10	6	406 × 178 × 54 UB	457 × 191 × 74 UB	533 × 210 × 92 UB	610 × 229 × 101 UB	686 × 254 × 125 UB	686 × 254 × 125 UB
	10	8	457 × 191 × 67 UB	533 × 210 × 92 UB	610 × 229 × 113 UB	686 × 254 × 125 UB	686 × 254 × 140 UB	762 × 267 × 173 UB
	10	10	457 × 191 × 74 UB	610 × 229 × 101 UB	686 × 254 × 125 UB	762 × 267 × 147 UB	762 × 267 × 173 UB	838 × 292 × 194 UB
	10	12	*	610 × 229 × 113 UB	686 × 254 × 140 UB	762 × 267 × 173 UB	838 × 292 × 194 UB	914 × 305 × 224 UB
Rafter	12	6	305 × 102 × 28 UB	406 × 140 × 39 UB	406 × 178 × 54 UB	457 × 191 × 67 UB	533 × 210 × 82 UB	533 × 210 × 92 UB
	12	8	305 × 102 × 28 UB	406 × 140 × 39 UB	406 × 178 × 54 UB	457 × 191 × 67 UB	533 × 210 × 82 UB	533 × 210 × 92 UB
	12	10	305 × 102 × 28 UB	406 × 140 × 39 UB	406 × 178 × 54 UB	457 × 191 × 67 UB	533 × 210 × 82 UB	610 × 229 × 101 UB
	12	12	*	406 × 140 × 39 UB	406 × 178 × 54 UB	457 × 191 × 67 UB	533 × 210 × 82 UB	610 × 229 × 101 UB
Restrained column	12	6	356 × 141 × 45 UB	457 × 191 × 67 UB	533 × 210 × 82 UB	610 × 229 × 101 UB	686 × 254 × 125 UB	686 × 254 × 140 UB
	12	8	356 × 171 × 45 UB	457 × 191 × 67 UB	533 × 210 × 82 UB	610 × 229 × 101 UB	686 × 254 × 125 UB	762 × 267 × 147 UB
	12	10	356 × 171 × 51 UB	457 × 191 × 67 UB	533 × 210 × 92 UB	610 × 229 × 113 UB	686 × 254 × 125 UB	762 × 267 × 147 UB
	12	12	*	457 × 191 × 67 UB	533 × 210 × 92 UB	610 × 229 × 113 UB	686 × 254 × 125 UB	762 × 267 × 147 UB
Unrestrained column	12	6	406 × 178 × 60 UB	533 × 210 × 82 UB	610 × 229 × 101 UB	610 × 229 × 113 UB	686 × 254 × 125 UB	762 × 267 × 147 UB
	12	8	457 × 191 × 74 UB	610 × 229 × 101 UB	610 × 229 × 113 UB	686 × 254 × 140 UB	762 × 267 × 173 UB	838 × 292 × 176 UB
	12	10	533 × 210 × 82 UB	610 × 229 × 113 UB	686 × 254 × 140 UB	762 × 267 × 173 UB	838 × 292 × 194 UB	914 × 305 × 224 UB
	12	12	*	686 × 254 × 125 UB	762 × 267 × 173 UB	836 × 292 × 176 UB	914 × 305 × 224 UB	914 × 305 × 253 UB

Table A.1 (Continued) *Symmetrical 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	6	356 × 127 × 33 UB	406 × 140 × 46 UB	406 × 178 × 60 UB	457 × 191 × 74 UB	533 × 210 × 82 UB	610 × 229 × 101 UB
	14	8	356 × 127 × 33 UB	406 × 140 × 46 UB	406 × 178 × 60 UB	457 × 191 × 74 UB	533 × 210 × 92 UB	610 × 229 × 101 UB
	14	10	356 × 127 × 33 UB	406 × 140 × 46 UB	406 × 178 × 60 UB	457 × 191 × 74 UB	533 × 210 × 92 UB	610 × 229 × 101 UB
	14	12	*	406 × 140 × 46 UB	406 × 178 × 60 UB	533 × 210 × 82 UB	533 × 210 × 92 UB	610 × 229 × 113 UB
Restrained column	14	6	356 × 171 × 51 UB	457 × 191 × 74 UB	533 × 210 × 92 UB	610 × 229 × 113 UB	686 × 254 × 140 UB	762 × 267 × 147 UB
	14	8	406 × 178 × 54 UB	457 × 191 × 74 UB	533 × 210 × 92 UB	610 × 229 × 113 UB	686 × 254 × 140 UB	762 × 267 × 173 UB
	14	10	406 × 178 × 54 UB	457 × 191 × 74 UB	610 × 229 × 101 UB	686 × 254 × 125 UB	686 × 254 × 140 UB	762 × 267 × 173 UB
	14	12	*	457 × 191 × 74 UB	610 × 229 × 101 UB	686 × 254 × 125 UB	762 × 267 × 147 UB	762 × 267 × 173 UB
Unrestrained column	14	6	457 × 191 × 67 UB	533 × 210 × 82 UB	610 × 229 × 101 UB	686 × 254 × 125 UB	686 × 254 × 140 UB	762 × 267 × 173 UB
	14	8	533 × 210 × 82 UB	610 × 229 × 101 UB	686 × 254 × 125 UB	762 × 267 × 147 UB	762 × 267 × 173 UB	838 × 292 × 176 UB
	14	10	533 × 210 × 92 UB	686 × 254 × 125 UB	762 × 267 × 147 UB	762 × 267 × 173 UB	838 × 292 × 194 UB	914 × 305 × 224 UB
	14	12	*	686 × 254 × 140 UB	762 × 267 × 173 UB	838 × 292 × 194 UB	914 × 305 × 224 UB	914 × 305 × 289 UB
Rafter	16	6	356 × 127 × 33 UB	406 × 140 × 46 UB	457 × 191 × 67 UB	533 × 210 × 82 UB	533 × 210 × 92 UB	610 × 229 × 113 UB
	16	8	356 × 127 × 33 UB	406 × 140 × 46 UB	457 × 191 × 67 UB	533 × 210 × 82 UB	610 × 229 × 101 UB	610 × 229 × 113 UB
	16	10	356 × 127 × 33 UB	406 × 140 × 46 UB	457 × 191 × 67 UB	533 × 210 × 82 UB	610 × 229 × 101 UB	610 × 229 × 113 UB
	16	12	*	406 × 140 × 46 UB	457 × 191 × 67 UB	533 × 210 × 82 UB	610 × 229 × 101 UB	686 × 254 × 125 UB
Restrained column	16	6	406 × 178 × 54 UB	533 × 210 × 82 UB	610 × 229 × 101 UB	686 × 254 × 125 UB	762 × 267 × 147 UB	762 × 267 × 173 UB
	16	8	406 × 178 × 54 UB	533 × 210 × 82 UB	610 × 229 × 101 UB	686 × 254 × 125 UB	762 × 267 × 147 UB	838 × 292 × 176 UB
	16	10	406 × 178 × 60 UB	533 × 210 × 82 UB	610 × 229 × 101 UB	686 × 254 × 125 UB	762 × 267 × 173 UB	838 × 292 × 176 UB
	16	12	*	533 × 210 × 82 UB	610 × 229 × 101 UB	686 × 254 × 125 UB	762 × 267 × 173 UB	838 × 292 × 194 UB
Unrestrained column	16	6	457 × 191 × 67 UB	533 × 210 × 92 UB	610 × 229 × 113 UB	686 × 254 × 125 UB	762 × 267 × 147 UB	762 × 267 × 173 UB
	16	8	533 × 210 × 82 UB	610 × 229 × 113 UB	686 × 254 × 140 UB	762 × 267 × 173 UB	838 × 292 × 176 UB	914 × 305 × 201 UB
	16	10	610 × 229 × 101 UB	686 × 254 × 125 UB	762 × 267 × 173 UB	838 × 292 × 194 UB	914 × 305 × 224 UB	914 × 305 × 253 UB
	16	12	*	686 × 254 × 140 UB	838 × 292 × 176 UB	914 × 305 × 224 UB	914 × 305 × 253 UB	914 × 305 × 289 UB

Use the frame details and factored loading given in the worked example in Appendix C.

Eaves height	= 7 m
Span	= 30 m
Rafter loading	= 11.3 kN/m

Using method 1

From Table A.1, assuming eaves height = 8 m, span = 30 m, rafter loading = 12 kN/m

Select
Rafter 457 × 191 × 67UB Grade S275

Restrained column 610 × 229 × 101UB Grade S275

A.2.2 Method 2 - design charts/graphs

Design charts/graphs available in a number of publications, assist in the rapid estimation of horizontal base force, moments in the rafters and the columns, and the position of the rafter hinge. These charts require slightly more work than the tables mentioned in A.2.1, but are much more flexible and accurate for the particular design case.

Charts/graphs for portal frames with pinned bases devised by A D Weller are given in *Introduction to steelwork design to BS 5950-1:2000*[37]. Charts for the design of portal frames with bases having various degrees of restraint, devised by Surtees and Yeap, have been published in *The Structural Engineer*[38].

The graphs for portal frames with pinned bases are reproduced in Figure A.1 to Figure A.4. They are based on the following assumptions:

- Plastic hinges form in the column at the bottom of the haunch and near the apex in the rafter.
- The rafter depth is approximately frame span/55.
- The haunch depth below the rafter is approximately the same as the rafter depth.
- The haunch length is approximately 10% of the frame span.
- The moment in the rafter at the top of the eaves haunch $\leq$ 0.87 M_p, i.e. the haunch region remains elastic.
- Wind loading does not control design.
- The chosen sections must be checked separately for stability.

The notation for the graphs is as follows:

H is the horizontal base reaction

w is the factored load (dead + imposed) per unit length on the rafter

L is the span of the frame

M_{pr} is the required plastic moment resistance of the rafter

M_{pl} is the required plastic moment resistance of the column

ℓ' is the distance of the point of maximum moment in the rafter from the column.

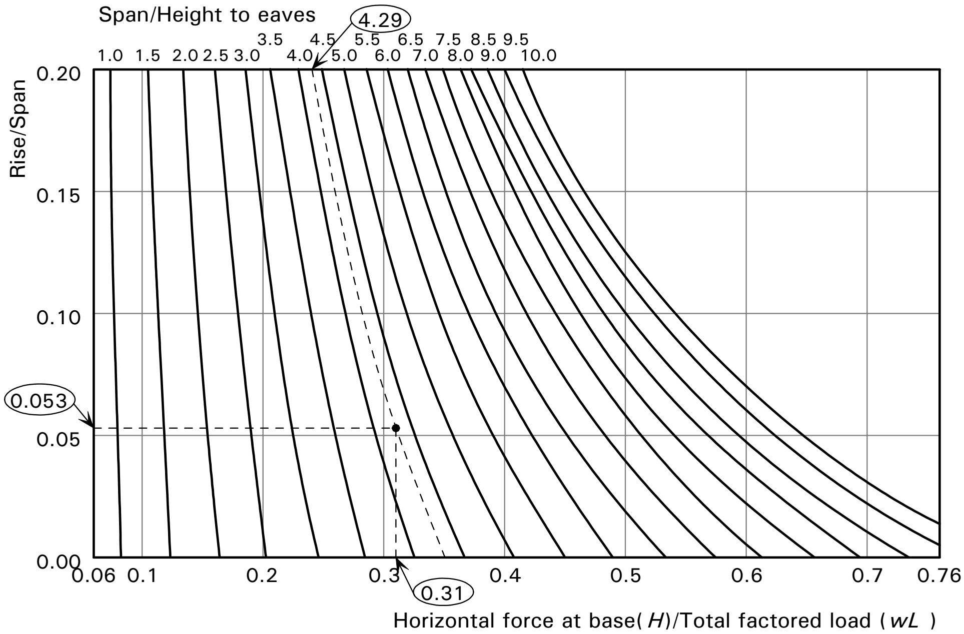

Figure A.1 *Graph 1. Horizontal force ratio at base*

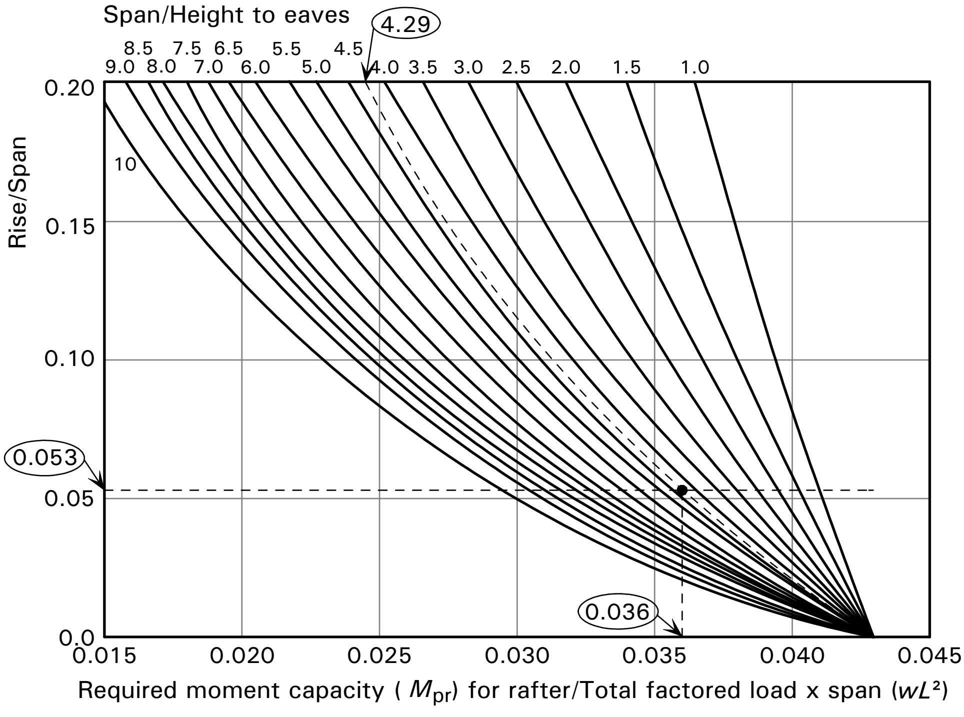

Figure A.2 *Graph 2. Moment capacity ratio required in rafter*

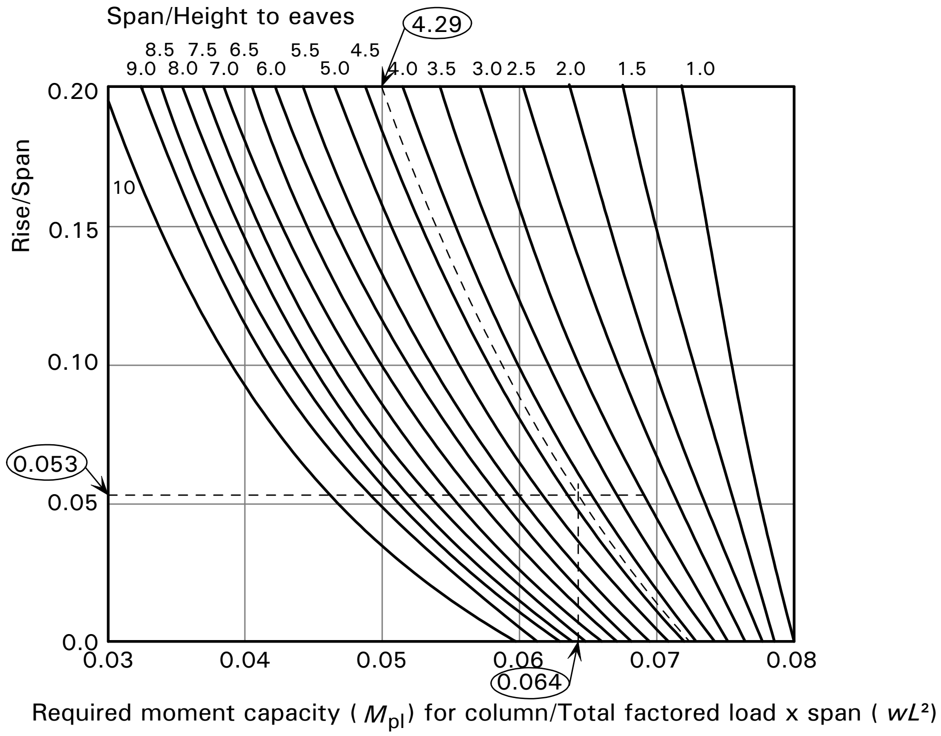

Figure A.3 *Graph 3. Moment capacity ratio required in column*

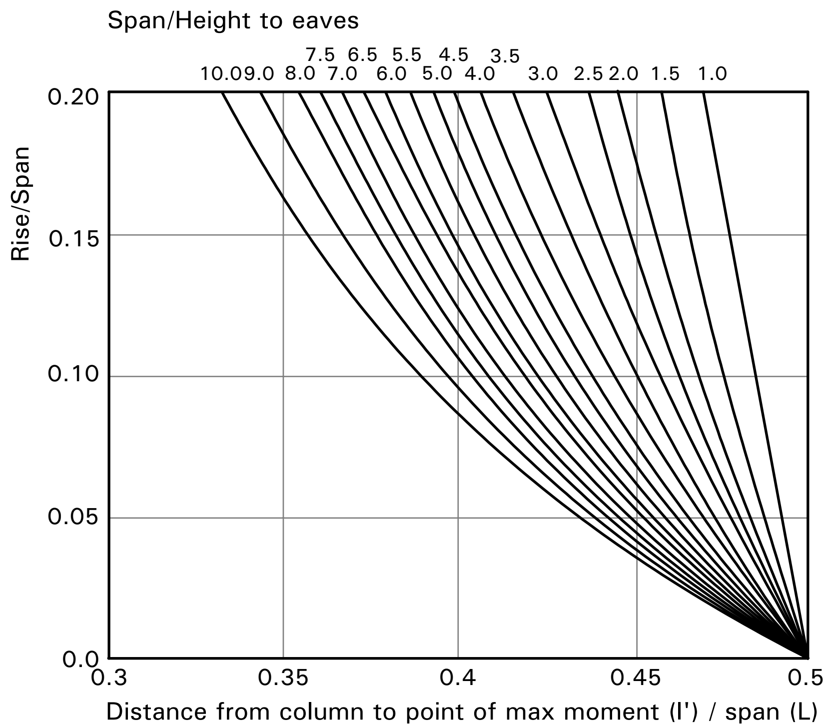

Figure A.4 *Graph 4. Distance to position of maximum moment in rafter*

Rise is the difference between the apex and eaves height. The graphs cover the range of span/height to eaves between 1 and 10, and a rise/span ratio of 0 to 0.2 (i.e. flat to 22°). Interpolation is permissible but extrapolation is not.

In Figure A.1, Graph 1 gives the horizontal force at the foot of the frame as a proportion of the total factored load wL.

In Figure A.2, Graph 2 gives the value of the required moment resistance of the rafters as a proportion of wL^2.

In Figure A.3, Graph 3 gives the value of the required moment resistance of the columns as a proportion of wL^2.

In Figure A.4, Graph 4 gives the position of the rafter hinge as a proportion of the span L.

Method of use

- Determine the ratio span/height to eaves (based on the intersection of the centre-lines of the members).
- Determine the ratio rise/span.
- Calculate wL and wL^2.
- Look up the values from the graphs, as follows:
- Horizontal reaction H = value from Graph 1 × wL.
- Rafter: M_{pr} = value from Graph 2 × wL^2.
- Column: M_{pl} = value from Graph 3 × wL^2.
- Distance to the position of maximum moment in the rafter l' = value from Graph 4 × L.

A worked example using the graphs is given in Appendix C.

In this example, Method 2 gives the following size rafter and column.

For rafter, from Figure A.2 (Graph 2)

$$\frac{M_{pr}}{wL^2} = 0.036$$

$\therefore M_{pr} = 0.036 \times 10170 = 366$ kNm
(457 × 191 × 67 UB Grade S275 $M_{cx} = 405$ kNm)

For column, from Figure A.3 (Graph 3)

$$\frac{M_{pl}}{wL^2} = 0.064$$

$\therefore M_{pl} = 0.064 \times 10170 = 651$ kNm
(533 × 210 × 101 UB Grade S275 $M_{cx} = 692$ kNm)

APPENDIX B Instability considerations

B.1 Second-order effects

Second-order effects occur due to sway of the frame. The sway causes eccentricity of vertical loading that generates second-order moments in the columns. These second-order moments are commonly referred to as being due to P-Δ effects, i.e. an axial load P applied at an eccentricity Δ (Figure B.1).

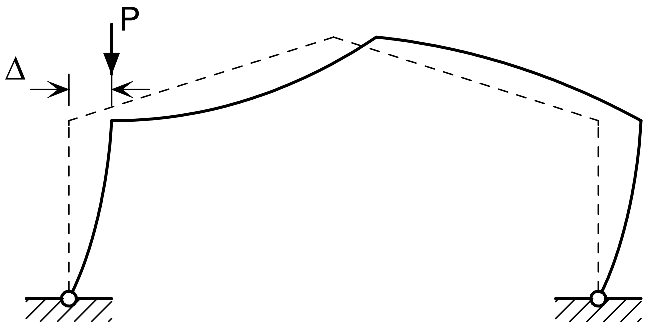

Figure B.1 *P-Δ effects in a portal frame*

It should be recognised that:

- P-Δ effects are due not only to horizontal loading but also to effects such as:
  - Eaves spread.
  - Asymmetry of structure.
  - Asymmetry of loading.
  - Lack of verticality or out-of-straightness of the columns.
- P-Δ effects always reduce the stability but may or may not be significant in the overall frame design.

B.2 Instability

The vertical load at which instability of a member or frame occurs is known as the elastic critical load and the ratio between this load and the actual applied ultimate factored load on the member is known as the elastic critical load factor, λ_{cr}.

$$\lambda_{cr} = \frac{\textit{Elastic critical load}}{\textit{Ultimate factored load}}$$

Two forms of in-plane instability are checked by the rules in BS 5950-1. These are termed sway and *snap-through* instability. Sway instability can occur in a single-bay portal frame when the columns sway in the same direction.

According to BS 5950-1, snap-through instability can only occur in the internal bays of multi-bay frames and will not therefore be considered further here.

Sway instability can be guarded against in a number of ways. BS 5950-1 gives three possible methods that are described in Section 6 of this publication. A further method, based on the Merchant-Rankine-Wood equations, can be described as the *elastic critical load factor method.*

B.3 The elastic critical load factor method

It is first necessary to calculate a value of the elastic critical load factor, λ_{cr}. This value is then used to calculate either a value of the required plastic collapse load factor λ_p, where plastic analysis is used, or an amplification factor, where elastic frame analysis is used.

If $\lambda_{cr} \geq 10$, in-plane stability effects may be ignored.

If $4.6 \leq \lambda_{cr} \geq 10$ then:

Where plastic hinge analysis is used, the required plastic collapse load factor to prevent instability is given by:

$$\lambda_p = \frac{0.9\lambda_{cr}}{\lambda_{cr} - 1}$$

Where elastic frame analysis is used, the moments from the analysis should be multiplied by the amplification factor.

$$\frac{\lambda_{cr}}{\lambda_{cr} - 1}$$

If $\lambda_{cr} \leq 4.6$, the in-plane stability effects should be taken into account by means of a second-order analysis of the complete frame.

Portal frames designed using plastic hinge analysis will generally fall into the intermediate category, which requires that the load factor for plastic design should be increased. In practice, this will usually mean ensuring that the plastic moment capacity of the section that is provided is λ_p times greater than that required by the first-order analyses.

The value of the elastic critical load or the elastic critical load factor can be derived from some specialist computer programs. Where such a program is not available, a suitable manual method has been derived for single-span pitched roof portal frames with nominally pinned and nominally rigid bases. This method, though derived for symmetric frames, will provide conservative results for asymmetric frames, provided that both sides of the frame are checked independently and the lowest value of λ_{cr} is used.

For a nominally pinned base frame with a base stiffness of 10% of the column stiffness, the elastic critical load factor is given by:

$$\lambda_{cr} = \frac{(1 + 0.1R)}{\left(\frac{P_r}{P_{r.cr}}\right) + (2.9 + 2.7R)\left(\frac{P_c}{P_{c.cr}}\right)}$$

For a nominally rigid base frame, with a base stiffness taken as equal to the column stiffness, the elastic critical load factor is given by:

$$\lambda_{cr} = \frac{(1 + 0.08R)}{\left(\frac{P_r}{P_{r.cr}}\right) + (0.8 + 0{,}52R)\left(\frac{P_c}{P_{c.cr}}\right)}$$

where:

E is the modulus of elasticity of steel, taken as 205 kN/mm^2

I_c is the second moment of area of the column section in the plane of the portal

I_r is the second moment of area of the rafter section in the plane of the portal

s is the rafter length along the slope (eaves to apex)

h is the column height

$$R = \frac{\text{column stiffness}}{\text{rafter stiffness}} = \left(\frac{I_c / h}{I_r / s}\right) = \frac{I_c s}{I_r h}$$

P_c is the axial compression in column from elastic analysis

Note: This differs from BS 5950-1 notation, which defines P_c as the compression resistance.

P_r is the axial compression in rafter from elastic analysis

$P_{c.cr}$ = elastic critical load for the column $= \frac{\pi^2 E I_c}{h^2}$

$P_{r.cr}$ = elastic critical load for the rafter $= \frac{\pi^2 E I_r}{s^2}$

APPENDIX C Worked example for the design of a single-span portal frame

INDEX TO APPENDIX C

ref to BS 5950-1: 2000 or otherwise

Introduction

The following calculations demonstrate the use of those clauses in BS 5950-1: 2000[1] that are suitable for manual design of portal frames.

Where more accurate but more complex methods are available that lead to greater economy, these methods and their possible conclusions are noted without detailed calculations being presented.

In the worked example, only one load combination has been checked (i.e. dead + imposed load). This will usually be the only load case considered at the preliminary design stage. At the detailed design stage, all relevant load combinations should be considered.

The following design sequence has been adopted in this example.

1. Frame dimensions
2. Loading
3. Initial sizing of members
4. Section properties
5. Reduced moment capacity due to axial load*
6. Classification of sections with axial load*
7. In-plane frame stability
8. Layout of purlins and side rails*
9. Determination of the collapse load*
10. Accurate moment capacity check*
11. Column stability
12. Rafter stability below the apex
13. Eaves haunch stability
14. Rafter stability above the haunch.

* These design checks would not normally be carried out when a rapid preliminary sizing of the sections for estimating purposes is required. Further details appropriate to each check are given within the example.

All values from section tables have been taken from *Steelwork design guide to BS 5950-1:2000 – Volume 1: Section properties & member capacities* [20]. All code references are to BS 5950-1:2000.

The preliminary sizing of members has been carried out by the method given by A D Weller in the SCI publication *Introduction to steelwork design to BS 5950-1:2000*[37]. The graphs are also given in Appendix A of this publication (P252). The method is based on a portal frame carrying vertical roof load only at the ultimate limit state and is suitable for the preliminary design stage.

C.1 Frame dimensions

The proposed frame is defined at the concept design stage by the line diagram as in Figure C.1, where the lines represent the centrelines of the members

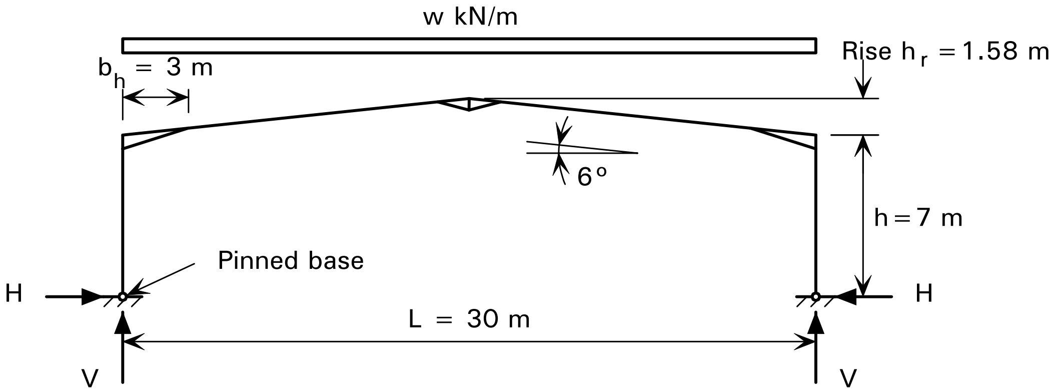

Figure C.1 *Portal frame dimensions*

Frame centres = L_s = 6 m

The haunch dimensions are identified as follows, where the length of the haunch is taken as 10% of the span of the frame.

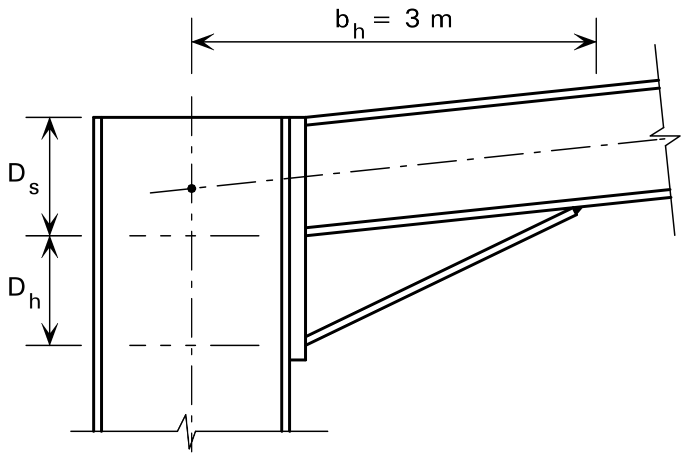

Figure C.2 *Haunch dimensions*

C.2 Loading

Approximate loads are given in Section 4. The following loads have been selected by reference to manufacturers' information and BS 6399-3[5]

C.2.1 Unfactored loads

Dead loads:	Sheeting	= 0.20 kN/m^2
	Purlins	= 0.07 kN/m^2
	Frame	= 0.11 kN/m^2
	Services	= 0.28 kN/m^2
Total dead load		= 0.66 kN/m^2
Imposed load		= 0.60 kN/m^2

C.2.2 Load combinations

The vertical load (Dead and Imposed) at the ultimate limit state is usually used to determine the size of the members for preliminary design purposes. At the detailed design stage, other load combinations should also be checked at the ultimate and the serviceability limit states (see Section 4 of this publication).

Total factored load w $= L_s(\gamma_{fd} \times 0.66 + \gamma_{fi} \times 0.60)$ 2.4.1.2

$= 6\,(1.4 \times 0.66 + 1.6 \times 0.60)$

$= 11.30$ kN/m

Silwood Park, Ascot, Berks SL5 7QN
Telephone: (01344) 623345
Fax: (01344) 622944

C.3 Initial sizing of members

In order to use the graphical method given in Appendix A, four parameters are required:

Span/height to eaves L/h	$= 30/7$	$= 4.29$
Rise/span h_r/L	$= 1.58/30$	$= 0.053$
Vertical load wL	$= 11.3 \times 30$	$= 339$ kN
wL^2	$= 11.3 \times 30^2$	$= 10170$ kNm

From the graphs, the following were obtained: — This publication (AppendixA)

Horizontal thrust at feet $H = 0.31 \times 339 = 105$ kN — (Fig A.1)

Required moment capacity of rafter $M_{pr} = 0.036 \times 10170 = 366$ kNm — (Fig A.2)

Required moment capacity of column $M_{pl} = 0.064 \times 10170 = 651$ kNm — (Fig A.3)

Based on the required moment capacities, the Universal Beam sections that are adequate are:

Rafter 457 × 191 × 67 UB in S275 steel $M_{cx} = 405$ kNm — Vol 1 (C-59)

Column 533 × 210 × 101 UB in S275 steel $M_{cx} = 692$ kNm — (C-59)

Both sections are classified as plastic under bending only. — (C-59)

The axial load in both the column and the rafter is low, so:

- the moment capacity in the presence of axial loads (M_{rx}) is unlikely to be much less than M_{cx} — Vol 1 (C-110) (C-108)
- the section classification is unlikely to change from plastic when axial load and bending are combined. — Table 11

The above sections can thus be assumed to be adequate for preliminary design

C.4 Section properties

C.4.1 Column

533 × 210 × 101 UB in S275 steel — Vol 1

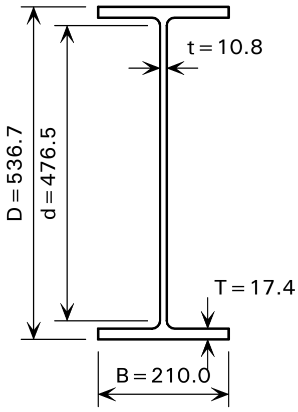

Figure C.3 *Column*

M_{cx} = 692 kNm — (C-59)
b/T = 6.03 — (B-2/B-3)
d/t = 44.1
I_x = 61500 cm^4
I_y = 2690 cm^4
r_x = 21.9 cm
r_y = 4.57 cm
Z_x = 2290 cm^3
S_x = 2610 cm^3
u = 0.873
x = 33.2
A = 129 cm^2

As $T > 16$ mm, $p_y = 265$ N/mm^2 — Table 9

C.4.2 Rafter

457 × 191 × 67 UB S275 steel — Vol 1

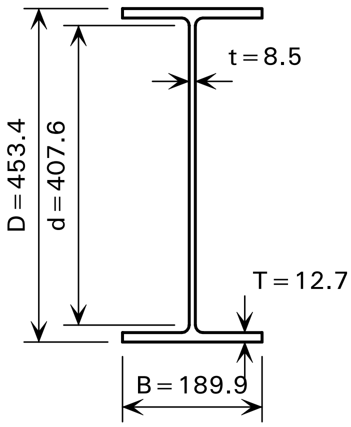

Figure C.4 *Rafter*

M_{cx} = 405 kNm — (C-59)
b/T = 7.48 — (B-4/B-5)
d/t = 48.0
I_x = 29400 cm^4
I_y = 1450 cm^4
r_x = 18.5 cm
r_y = 4.12 cm
Z_x = 1300 cm^3
S_x = 1470 cm^3
u = 0.872
x = 37.9
A = 85.5 cm^2

As $T < 16$ mm, $p_y = 275$ N/mm^2

C.5 Reduced moment capacity due to axial load

The moment capacity of both the column and the rafter will be reduced slightly because of the axial loads. The reduction in moment capacity is so small that it is usually ignored at the preliminary design stage.

C.5.1 Column (533 × 210 × 101 UB Grade S275)

Axial Force F_c^* $= V = wL/2 = 11.3 \times 30/2 = 170\text{kN}$

$n = F_c/Ap_y = 170 \times 10^3 / (129 \times 10^2 \times 265) = 0.05$

Reduced moment capacity of the column $M_{l..rx} = 687$ kNm > 651 kNm required — Vol 1 (C-108)

C.5.2 Rafter (457 × 191 × 67 UB Grade S275)

Axial force $F_c^* = H\cos\theta + V\sin\theta = 105 \cos 6^o + 170 \sin 6^o$
$= 122$ kN

$n = F_c/Ap_y = 122 \times 10^3 / (85.5 \times 10^2 \times 275)$
$= 0.05$

Reduced moment capacity of the rafter $M_{r.rx} = 402$ kNm > 366 kNm required — Vol 1 (C-110)

* The axial load should be that which is relevant to the load case being checked. In practice, however, the axial load is so low that the conservatism in using the largest axial load is negligible for low pitch roofs.

C.6 Classification of sections with axial load

At the detailed design stage, it is necessary to ensure that the sections can be classified as plastic cross-sections. The axial load is usually so low that, providing the section can be classified as plastic when subject to bending only, it will remain plastic when the axial load is taken into account, and this check is usually not carried out at the preliminary design stage.

C.6.1 For column (533 × 210 × 101 UB Grade S275)

$\varepsilon = (275/p_y)^{½} = (275/265)^{½} = 1.02$

Flange b/T = 6.03

Limiting b/T value for Class 1 plastic flange = 9ε = 9.18 — Table 11

6.03 < 9.18 ∴ flange is classified as plastic

Web d/t = 44.1

Limiting d/t value for Class 1 plastic "Web Generally" = $\dfrac{80\varepsilon}{1+r_1}$ but $\geq 40\varepsilon$ — Table 11

80ε = 81.6

$r_1 = \dfrac{F_c}{dtp_{yw}}$ but $-1 < r_1 \leq 1$ — 3.5.5(a)

$= \dfrac{170 \times 10^3}{476.5 \times 10.8 \times 265} = 0.125$

$1 + r_1 = 1 + 0.125 = 1.125$

Limiting d/t value $= \dfrac{80\varepsilon}{1+r_1} = \dfrac{81.6}{1.125}$

$= 72.5$

44.1 < 72.5 ∴ web is classified as plastic

Both the flange and the web are classified as plastic, so the section can be classified as plastic.

C.6.2 For rafter (475 × 191 × 67 UB Grade S275)

$\varepsilon = (275/p_y)^{½} = (275/275)^{½} = 1.0$

Flange b/T = 7.48

Limiting b/T value for Class 1 plastic flange $= 9\varepsilon = 9.0$ — Table 11

$7.48 < 9.0$ ∴ flange is classified as plastic

Web $d/t = 48.0$

The axial load in the rafter is generally so small that it can be assumed that the neutral axis is at mid-depth and the d/t limit can be taken as 80ε, but for completeness the axial load will be taken into account.

Limiting d/t value for Class 1 plastic "Web Generally" $= \dfrac{80\varepsilon}{1 + r_1}$ but $\geq 40\varepsilon$ — Table 11

$$r_1 = \frac{F_c}{dtp_{yw}} = \frac{122 \times 10^3}{407.6 \times 8.5 \times 275} = 0.128$$ — 3.5.5(a)

$$\text{Limiting } d/t \text{ value} = \frac{80\varepsilon}{1 + r_1} = \frac{80}{1.128} = 70.9$$

$48.0 < 70.9$ ∴ web is classified as plastic

Both the flange and the web are classified as plastic, so the section can be classified as plastic.

C.7 In-plane frame stability

Section 6

This check should be carried out at the preliminary design stage, after the size of the sections is determined. It should be carried out again at the detailed design stage if the section sizes are reduced.

In this example, in-plane stability of the frame is checked using the sway check method given in Section 6.2 of this publication.

Check the geometry of the Frame 5.5.4.2.1

(a) $L \leq 5h$

$L = 30$ m, $5h = 5 \times 7 = 35$ m

30 m < 35 m $\therefore$ OK

(b) $h_r \leq 0.25L$

$h_r = 1.58$ m $0.25L = 0.25 \times 30 = 7.5$ m

$\therefore$ 1.58 m < 7.5 m $\therefore$ OK

$\therefore$ geometry of the frame is within the limits

Formula Method- Gravity Loads 5.5.4.2.2

Check effective span to depth ratio of the rafter satisfies the condition:

$$\frac{L_b}{D} \leq \frac{44L}{\Omega h}\left[\frac{\rho}{4 + \rho L_r / L}\right]\left[\frac{275}{p_{yr}}\right]$$

where $L_b = L - \left(\dfrac{2D_h}{D_s + D_h}\right)L_h$

assuming $D_h \approx D_s$

L_h is the length of a <u>single</u> haunch (= 3 m)

$\therefore$ $L_b = 30 - 3 = 27$ m

$$\rho = \left[\frac{2I_c}{I_r}\right]\left[\frac{L}{h}\right] = \frac{2 \times 61500}{29400} \times \frac{30}{7} = 17.9$$

$L_r = L/\cos\theta = 30/\cos 6^{\circ} = 30.2$ m

$\Omega = W_r/W_o$

$W_r = wL = 11.3 \times 30 = 339$ kN

W_o is the maximum value of W_r that causes failure of the rafter treated as a fixed ended beam of span L

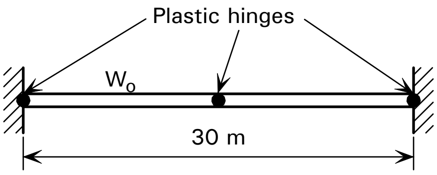

Figure C.5 *Calculation of W_o*

$M_p = M_{cx} = W_oL/16$

$W_o = 16M_{cx}/L = 16 \times 405/30 = 216$ kN

$\Omega = W_r/W_o = 339/216 = 1.57$

$$\frac{44L}{\Omega h}\left[\frac{\rho}{4+\rho L_r/L}\right]\left[\frac{275}{p_y}\right]$$

$$= \frac{44\times 30}{1.57\times 7}\left[\frac{17.9}{4+17.9\times 30.2/30}\right]\frac{275}{275} = 97.6$$

$L_b/D = 27 \times 10^3/453.4 = 59.6$

As $L_b/D < 97.6$, the frame is stable under gravity loads

$\therefore$ The required load factor for frame stability
$\lambda_r = 1.0$ for gravity load case

<u>For horizontal loads</u> 5.5.4.2.3

Required load factor λ_r for frame stability

$$\lambda_r = \frac{\lambda_{sc}}{\lambda_{sc}-1}$$

$$\lambda_{sc} = \frac{220\,DL}{\Omega h L_b}\left(\frac{\rho}{4+\rho L_r/L}\right)\left(\frac{275}{p_{yr}}\right)$$

$$= \frac{220\times 0.4534\times 30}{1.57\times 7\times 27}\left(\frac{17.9}{4+17.9\times 30.2/30}\right)\left(\frac{275}{275}\right)$$

$= 8.20$

$$\therefore \lambda_r = \frac{8.20}{8.20-1} = 1.14$$

Therefore, for this load case, λ_p must not be less than 1.14. The actual value of λ_p would depend on the magnitude of the applied horizontal loads, but generally λ_p would be greater than λ_r. In this example it is assumed that gravity load case is critical.

C.8 Layout of purlins and side rails

At this stage, a more detailed assessment of the frame geometry can be made. It is also useful to determine a layout of purlins and side rails that can provide restraint to plastic hinges and adjacent lengths. This layout will be determined from manufacturers' tables based on the load capacity of the purlins and the cladding system.

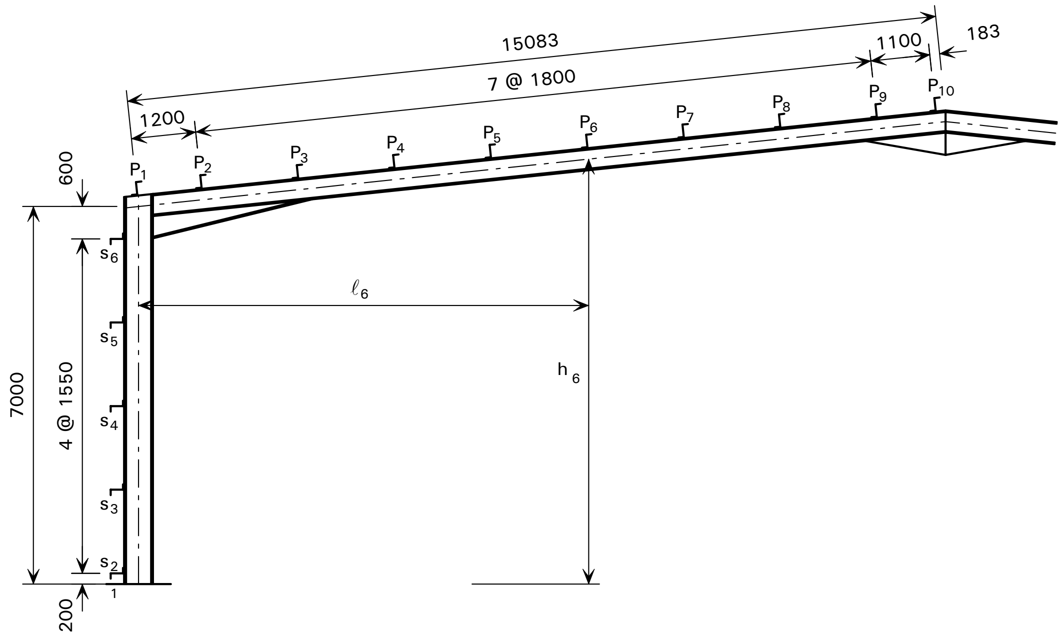

Figure C.6 *Purlin and side rail spacing*

The value of the bending moment can be found at any point in the rafter from the formula:

$M_x \quad = V\ell_x - H\,h_x - w\,\ell_x^2/2$

where ℓ_x is the horizontal distance to the point considered

h_x is the height of the point considered $= h + \ell_x \tan\theta$

V is the vertical reaction at the base due to w

H is the horizontal reaction at the base due to w

w is the load per unit length of the frame (factored loading)

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C.9 Determination of the collapse load

At the preliminary design stage, it is not necessary to calculate the loading that causes the collapse mechanism. At the detailed design stage, however, the loading causing the collapse mechanism (w') will be used instead of the applied factored loading (w). This is because, as a result of plastic redistribution, most of the members will be subject to moments and forces corresponding to the collapse mechanism even before it has formed.

Assume that the plastic hinges are located in the column at the bottom of the eaves haunch and in the rafters at the second purlin from the ridge (i.e. P_9 in Figure C.6).

The moment in the rafter at P_9 is then given by:

$$M@P_9 \ = V'\ell_9 \ - H'h_9 - \frac{w'}{2} \ (\ell_9)^2$$

and the moment in the column @ the bottom of the eaves haunch is given by:

$$M@S_6 \ \approx H'(h - D_h - D_s/2)$$

w' is the collapse load

V' and H' are the reactions at the base due to w'

At the point of collapse, the moment $M@P_9$ and $M@S_6$ must be equal to the reduced moment capacities of the rafter and column sections provided (see Section 5.1 and 5.2 of these calculations).

Thus $M@P_9 = M_{r.rx} = 402$ kNm
$M@S_6 = M_{l.rx} = 687$ kNm

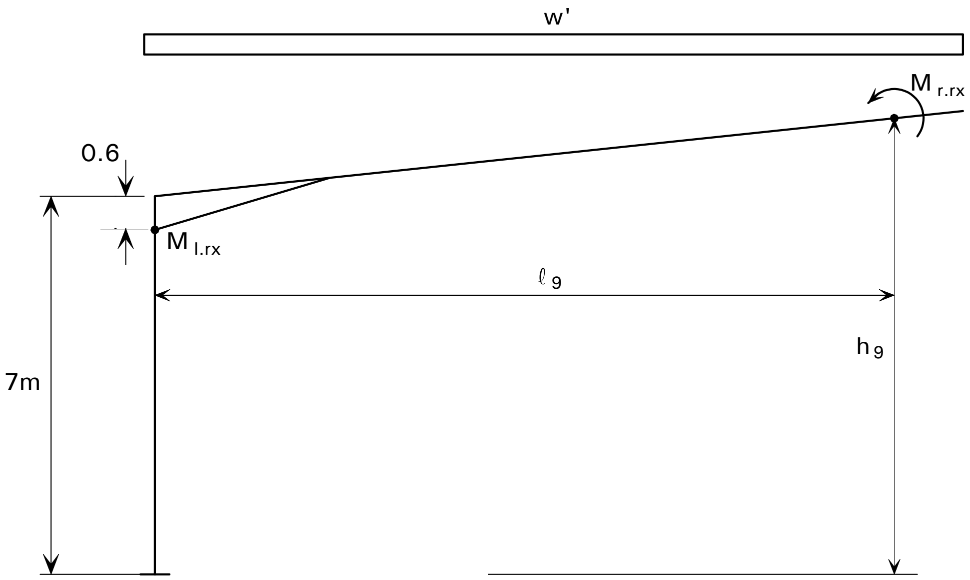


Figure C.7 *Determination of collapse load w′*

$\ell_9 = 13.8 \cos \theta = 13.8 \cos 6^{\circ} = 13.72$ m

$h_9 = h + \ell_9 \tan \theta = 7 + 13.72 \tan 6^{\circ} = 8.44$ m

$H' = (M@S_6)/(h@S_6) = 687/6.4 = 107.3$ kN

$V' = w'L/2 = 15w'$

Substituting:

$M@P_9 = 15w' \times 13.72 - 107.3 \times 8.44 - w' \times 13.72^2/2$

Equating $M@P_9$ to $M_{r.rx}$

$402 = 111.7w' - 905.6$

$w' = 11.71$ kN/m

Collapse load w' = 11.71 kN/m (compare with the applied factored load w of 11.30 kN/m calculated originally)

The corresponding base reactions are:

$H' = 107.3$ kN

$V' = 175.7$ kN

$$\lambda_p = \frac{w'}{w} = \frac{11.71}{11.30} = 1.04$$

C.10 Accurate moment capacity check

At the detailed design stage, a check is required to ensure that the moment at any other point on the rafter is not greater than the reduced moment capacity of the rafter ($M_{r.rx}$).

The moment M_x at any point x along the rafter is given by:

$M_x \quad = V' \ell_x - H' h_x - w' \ell_x^2/2$

Where: ℓ_x is the horizontal distance to that point

h_x is the vertical height of that point

(see Figure C.6)

The axial load F_x at any point x along the rafter is given by:

$F_x \quad = H' \cos \theta + V' \sin \theta - \ell_x w' \sin \theta$

Table C.1 *Moment and axial load in rafter*

Position	L_{slope} m	ℓ_x m	h_x m	M_x kNm	F_x kN
P_1	0	0.00	7.00	-751	125
Face	0.27	0.27	7.03	-707	125
P_2	1.2	1.19	7.13	-563	124
P_3	3	2.98	7.31	-313	121
P_4	4.8	4.77	7.50	-100	119
P_5	6.6	6.56	7.69	76	117
P_6	8.4	8.35	7.88	214	115
P_7	10.2	10.14	8.07	314	113
P_8	12	11.93	8.25	377	110
P_9	13.8	13.72	8.44	402	108
P_{10}	14.9	14.82	8.56	400	107

$M_{r.rx}$ (reduced due to axial load) $= 402$ kNm

It can be seen from the above table that the moment between P_3 and P_9 does not exceed $M_{r.rx}$ and that the correct position of hinges has been chosen.

C.11 Column stability

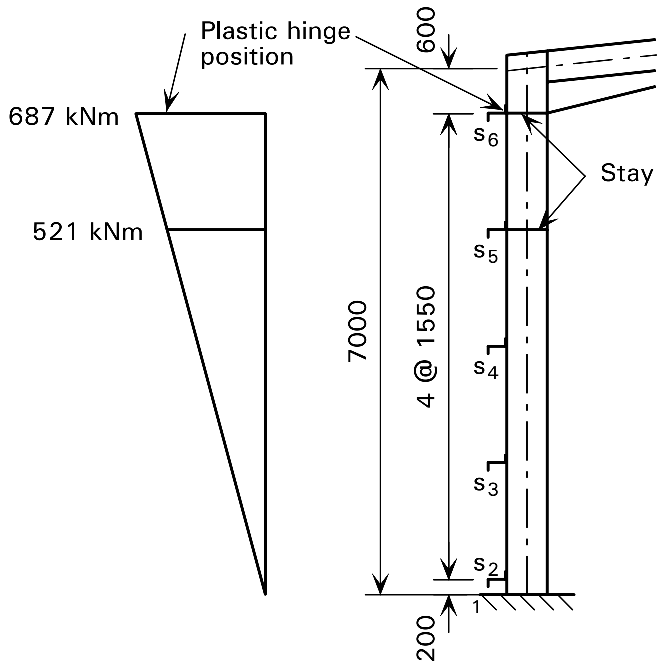


Figure C.8 *Bending moment diagram and restraints to column*

The hinge position will be torsionally restrained by the provision of a rafter stay at the base of the haunch to side rail S_6. 5.3.2

The distance to the next lateral restraint to the compression flange from the plastic hinge position can be determined by one of four approaches: 5.3.3 and 5.3.4

1. Calculate the limiting distance L_m. 5.3.3(a)
2. For UB section, calculate the modified distance L_m to account for moment gradient. 5.3.3(b)
3. For UB section, calculate limiting distance L_s. 5.3.4
4. Use Appendix G of BS5950-1:2000

The first approach assumes no restraint to the tension flange and is conservative.

The second approach also assumes no restraint to tension flange, but involves more work because it takes account of the shape of bending moment and hence permits use of a greater length between restraints.

The third approach is a simplification of Appendix G for UB sections.

The fourth approach is generally too complex for manual design and shows little advantage for the column as usually only one restraint is required between the plastic hinge position and the base.

In many cases, it will be adequate to provide an adjacent lateral restraint to the compression flange at a distance L_m from the plastic hinge position. The section between this restraint and the base should then prove adequate when checked to BS 5950-1, Clause 4.8.3.3.

C.11.1 Check length between side rails S_6 and S_5

BS 5950-1:2000 Clause 5.3.3(a) is used to check the length between restraints at side rails S_6 and S_5. Assume restraint is provided at S_6 and S_5 by means of column stays.

Limiting length L_m is given by:

$$L_m = \frac{38 r_y}{\left[\frac{f_c}{130} + \left(\frac{x}{36}\right)^2 \left(\frac{p_y}{275}\right)^2\right]^{1/2}}$$ 5.3.3(a)

f_c = $V'/A = 175.7 \times 10^3/129 \times 10^2$ = 13.6 N/mm²

$$L_m = \frac{38 \times 45.7}{\left[\frac{13.6}{130} + \left(\frac{33.2}{36}\right)^2 \left(\frac{265}{275}\right)^2\right]^{1/2}} = 1836 \text{ mm}$$

Thus, the length of 1550 mm from the plastic hinge position at side rail S_6 to the column stay at side rail S_5 is stable.

C.11.2 Check the length between side rail S_5 and the base (1)

There is no plastic hinge in the length between S_5 and the base and a restraint to the compression flange has been provided at side rail S_5 by means of a column stay.

For external columns, the relevant check is for out-of-plane buckling only, because in-plane member stability is assured by the in-plane frame stability checks given in Section 7 of this example. It is therefore required that:

$$\frac{F_c}{P_{cy}} + \frac{m_{LT} M_{LT}}{M_b} \le 1$$ 4.8.3.3.2(a) out-of-plane

M_{LT} = $M@S_5$ = 521 kNm

F_c = V' = 175.7 kN

m_{LT} = 0.6 for $\beta = 0.0$ Table 18

To calculate P_{cy} and M_b, using Vol 1, Page C-109 — Vol 1 (C-109)

$$\frac{F_c}{P_z} = \frac{175.7}{3420} = 0.05 < 0.348 \quad \therefore \text{ Section at least Class 2, compact.}$$

For $L_{EY} = 4.85$ m, $P_{cy} = 1660$ kN

$L_E = 4.85$ m, $M_b = 402$ kNm

$$\therefore \quad \frac{F_c}{P_{cy}} + \frac{m_{LT} M_{LT}}{M_b} = \frac{175.7}{1660} + \frac{0.6 \times 521}{402} = 0.106 + 0.778 = 0.88 < 1$$

$\therefore$ No further column restraints are required between side rail S_5 and the base.

C.12 Rafter stability below the apex

The plastic hinge (at purlin P_9) will be restrained by the purlin P_9 and a rafter stay to give torsional restraint.

The rafter near the apex is subject to a sagging moment in this laod case. The compression flange is stabilised by the purlin.

C.12.1 Check the length between purlins P_9 and P_8

The length, adjacent to the plastic hinge, between P_9 and P_8, will be checked by the formula in BS 5950-1:2000 Clause 5.3.3(a).

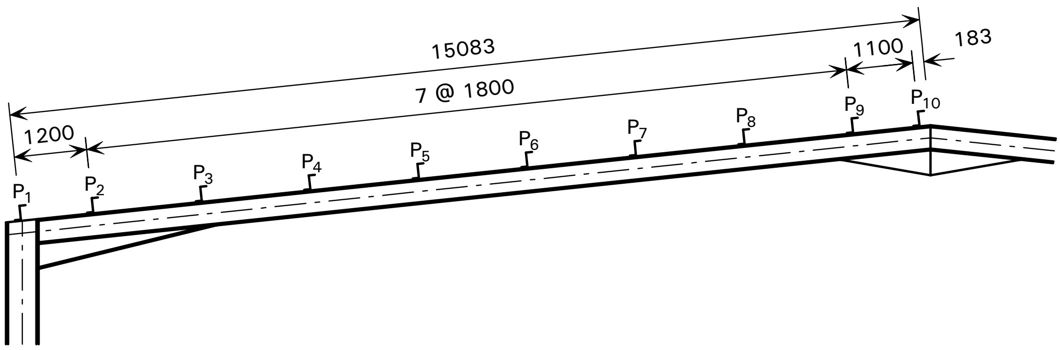


Figure C.9 *Purlin spacing*

$$L_m = \frac{38 r_y}{\left[\frac{f_c}{130} + \left(\frac{x}{36}\right)^2 \left(\frac{p_y}{275}\right)^2\right]^{1/2}}$$ 5.3.3(a)

Where: $f_c = F/A$ $\quad F@P_8 = 110$ kN — Table C.1 in this example

$\therefore f_c = 110 \times 10^3/(85.5 \times 10^2) = 12.9$ N/mm^2

$$L_m = \frac{38 \times 41.2}{\left[\frac{12.9}{130} + \left(\frac{37.9}{36}\right)^2 \left(\frac{275}{275}\right)^2\right]^{1/2}} = 1425 \text{ mm}$$

The distance between purlins P_9 and P_8 = 1800 mm. This is greater than L_m, therefore an additional purlin is required between P_9 and P_8.

Alternatively, the computer analysis shows that at purlin P_9, the plastic hinge is not yet fully formed or it would be the last plastic hinge to be formed. In this case there is no need to comply with Clause 5.3.3(a) and provide a restraint at a distance less than L_m. Instead the length between purlins P_9 and P_8 could be checked in accordance with Clause 4.8.3.3.2(a) "out-of-plane buckling" and shown to be adequate (see Sheet 19 of Appendix D). 5.3.1

C.13 Eaves haunch stability

This Section includes the length up to purlin P_3.

Four design options exist for the situation where the tension flange is restrained between restraints to the compression flange. 5.3.1

1. Ignore the tension flange restraints and design to Clause 4.8.3.3.1 providing restraints to the compression flange as necessary.
2. Limit the length between compression flange restraint to L_m given by Clause 5.3.3.
3. Limit the length between compression flange restraints to L_s as given by Clause 5.3.4.
4. Check the length according to Appendix G of BS 5950-1:2000.

Method 2 will be conservative as it ignores the restraint to the tension flange between torsional restraints.

Method 3 is relatively straightforward but using the limiting length L_s requires the distance between tension flange restraints to be adequate when checked to Clause 4.8.3.3 (or Clause I.1)

Method 4 would not normally be carried out manually although it can be shown that rafter stays to the compression flange at purlins P_3 and P_5 would be adequate.

Method 3 will be demonstrated here.

Method 3, Clause 5.3.4 approach (Simple Method)

Provided the geometrical limitations are complied with, the spacing L_y between restraints to the compression flange should not exceed the limiting spacing L_s.

For S275

5.3.4

$$L_s = \frac{620\,r_y}{K_1\left[72 - \left(\frac{100}{x}\right)^2\right]^{0.5}}$$

r_y and x for the un-haunched section (i.e. rafter)

$$\frac{D_h}{D_s} \approx 1, \qquad \therefore K_1 = 1.25$$

$$\therefore\ L_s = \frac{620 \times 41.2}{1.25\left[72 - \left(\frac{100}{37.9}\right)^2\right]^{0.5}}$$

$$\therefore\ L_s = 2534 \text{ mm}$$

The length of the haunch (between the column face and purlin P_3) is 2710 mm. This is greater than L_s, therefore an additional stay at purlin P_2 would be required.
Alternatively the length between the face of the column and purlin P_3 could be checked according to Appendix G.2.2 to show that no additional stay is required at P_2.

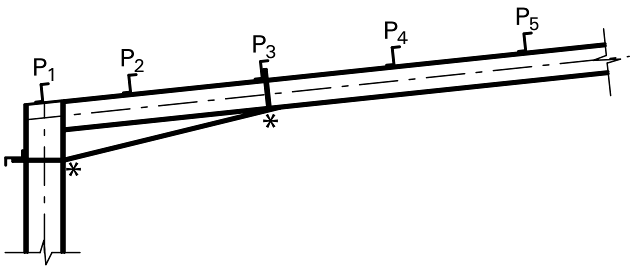


Figure C.10 *Haunch restraints*

C.14 Rafter stability above the haunch (between purlins P_3 and P_5)

The length between purlin P_3 and P_5 is mostly in the hogging region and does not contain a plastic hinge. Restraints (stays) are provided to the compression flange at purlins P_3 and P_5

Using 4.8.3.3.2(a) out-of-plane buckling because the in-plane stability of the rafter is assured by the in-plane frame stability. — 4.8.3.3.2(a) out-of-plane

$$\frac{F_c}{P_{cy}} + \frac{m_{LT} M_{LT}}{M_b} \leq 1$$

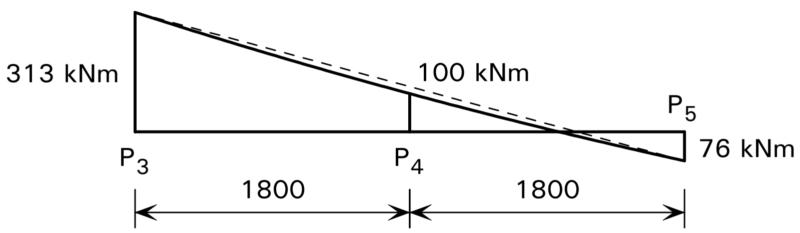


Figure C.11 *Moment diagram between P_3 and P_5*

$F_c = 121$ kN (i.e. F@P_3) — Table C.1

$P_z = 2350$ kN — Vol 1 (C-110/111)

$$\therefore \frac{F_c}{P_z} = \frac{121}{2350} = 0.05$$

For effective length $L_E = 3.6$ m

$P_{cy} = 1420$ kN

$M_b = 272$ kNm

$$\beta \approx -\frac{76}{313} = -0.24$$ — Table 18

$\therefore m_{LT} = 0.50$

$M_{LT} = M@P_3 = 313$ kNm

$$\frac{F_c}{P_{cy}} + \frac{m_{LT} M_{LT}}{M_b} \leq 1$$

$$\frac{121}{1420} + \frac{0.5 \times 313}{272} = 0.085 + 0.575$$

$= 0.66 < 1$ $\therefore$ OK

This section is therefore stable between P_3 and P_5

C.15 Summary

As a result of the calculations carried out so far, the frame is shown to be adequate when subject to dead plus imposed loading, with member sizes and restraints as shown in Figure C.12

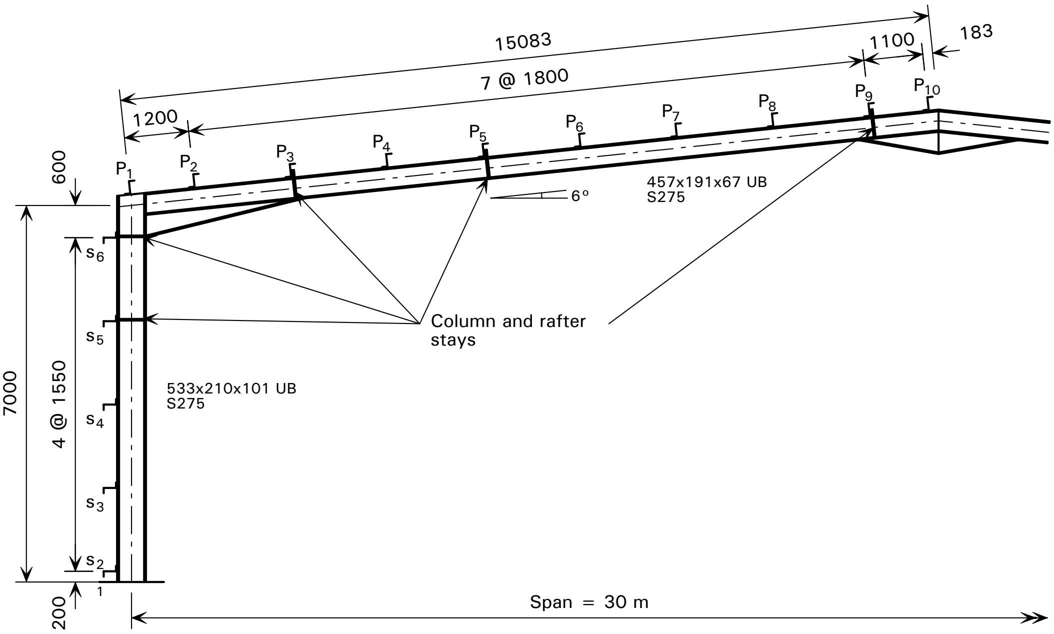


Figure C.12 *Summary – member sizes and restraints based on gravity loads*

Similar checks to those shown so far should be carried out using load combinations involving wind load (see Section 4 of this publication). Consideration of the wind uplift combination may necessitate the use of an additional rafter stay at about 3 m from the apex. This is because the uplift case will apply hogging moments to the section of rafter which are in sagging under gravity loads.

APPENDIX D Output from CSC Fastrak program

The following output has been produced by the Fastrak Portal Frame Design Program (version 4.1, January 2004, CSC (UK) Ltd, Yeadon House, New Street, Pudsey, Leeds LS28 8AQ. Tel: 0113 239 3000; Fax: 0113 236 0546). The output has been produced to mirror the worked example in Appendix C. In order to reduce the text in this publication, the output has been cut and modified slightly in presentation so that only detailed design calculations relevant to the left hand side of the frame are shown.

No comment is made on the output. Differences between the computer analysis and hand calculations are small.

NOTES*:*

a) The design is in accordance with BS 5950-1:2000.

b) The output is for load combination 1 only, i.e. dead load + imposed load + Notional Horizontal Forces (NHF). The inclusion of the NHF, as required by BS 5950-1, allows for frame imperfections.

c) In the final design, load combination 2 (i.e. dead load + wind load) and load combination 3 (i.e. dead load + imposed load + wind load) should also be considered. This may result in modification of the purlin and side rail positions and the addition of torsional restraints.

INDEX TO APPENDIX D **Page**

D.1 Project details

Job	**BCB 766**
Project	SCI Publication P252 Design of portal frames
Structure	Computer output
Calcs. by	AJR

D.2 Building loading

Dead Load	**0.270**	**kN/m^2**
Service Load	0.280	kN/m^2
Imposed Load	0.600	kN/m^2

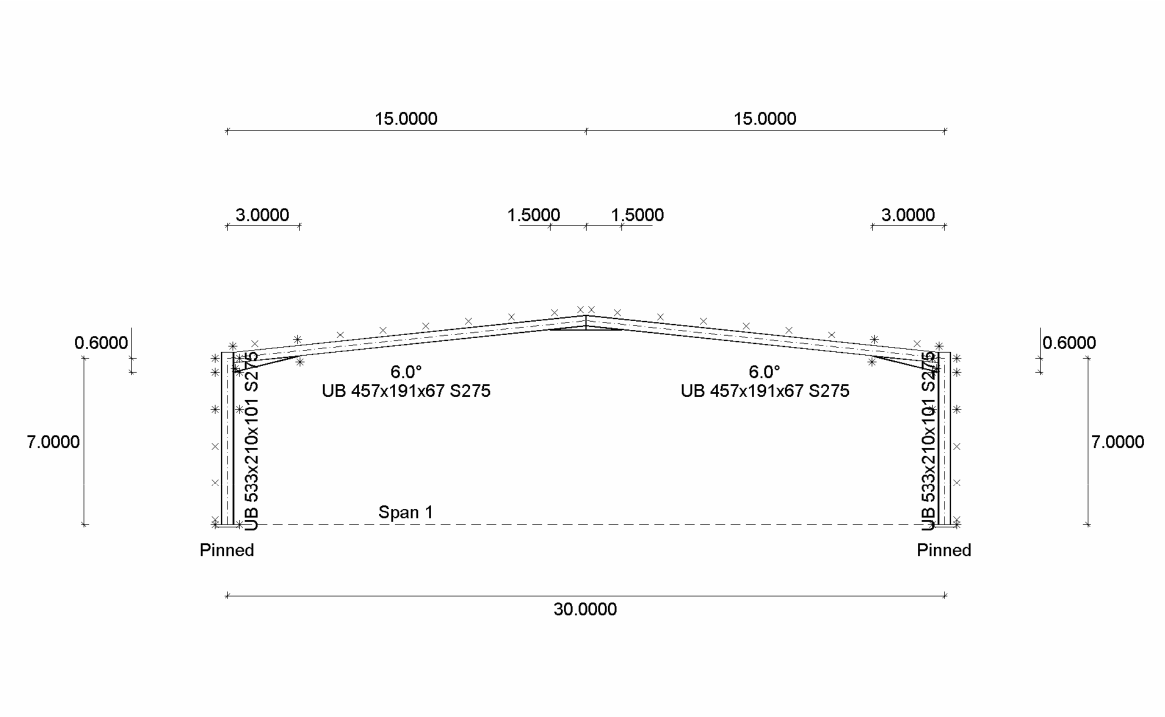
D.3 Frame summary table

Frame Reference	Status	Design Status	Weight (kg)
Frame Type 1	Design Complete	Pass	3685.77

D.4 Reference: frame type 1

D.4.1 Frame Details

No. Spans	**1**	
Frame Centres Near Face	6.0000	m
Frame Centres Far Face	6.0000	m
Effective Frame Centres	6.0000	m



D.4.2 Frame Span Geometry Table

Span	Type	Axis	Lh Eaves	Lh Apex	Apex	Rh Apex	Rh Eaves
		m	m	m	m	m	m
1	Standard	Y	0.0000		15.0000		30.0000
		Z	7.0000		8.5766		7.0000

D.4.3 Frame Span Haunch Table

Haunch	Length	Depth	Beta	Gamma	Offset X1	Offset X2	Filler
	m	m	°	°	mm	mm	Plate
Span 1 Lh Haunch	3.0000	0.6000	13.9	7.9	92.1	2700.9	
Span 1 Apex Lh Haunch	1.5000	0.4000	0.1	6.1	0.2	1479.8	
Span 1 Apex Rh Haunch	1.5000	0.4000	0.1	6.1	0.2	1479.8	
Span 1 Rh Haunch	3.0000	0.6000	13.9	7.9	92.1	2700.9	

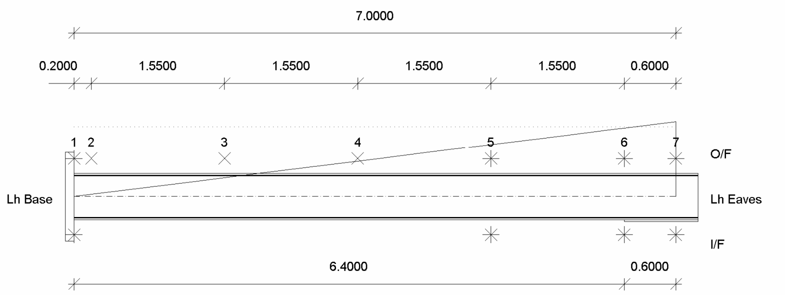
D.4.4 Frame Member Table

Member	Section	Grade	Strength
			N/mm^2
Span 1 Lh Column	UB 533x210x101	S275	265.0
Span 1 Lh Haunch	UB 457x191x67	S275	275.0
Span 1 Lh Rafter	UB 457x191x67	S275	275.0
Span 1 Apex Lh Haunch	UB 457x191x67	S275	275.0
Span 1 Apex Rh Haunch	UB 457x191x67	S275	275.0
Span 1 Rh Rafter	UB 457x191x67	S275	275.0
Span 1 Rh Haunch	UB 457x191x67	S275	275.0
Span 1 Rh Column	UB 533x210x101	S275	265.0

D.4.5 Frame Member Restraints

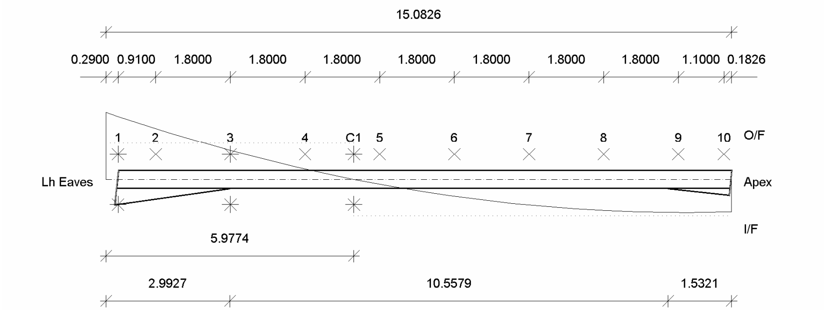
D.4.5.1 Span 1 Lh Column

No.	Position	Type
	m	
Base	0.0000	Torsional
S2	0.2000	O/S Flange
S3	1.7500	O/S Flange
S4	3.3000	O/S Flange
S5	4.8500	Torsional
S6	6.4000	Torsional
S7	7.0000	Torsional



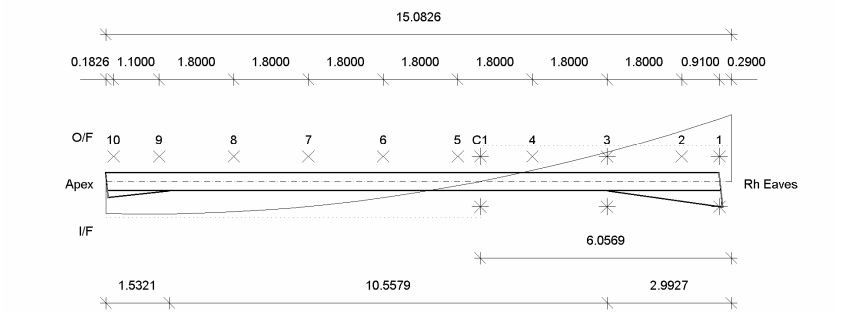
D.4.5.2 Span 1 Lh Rafter

No.	Position	Type
	m	
P1	0.2900	Torsional
P2	1.2000	O/S Flange
P3	3.0000	Torsional
P4	4.8000	O/S Flange
P5	6.6000	O/S Flange
P6	8.4000	O/S Flange
P7	10.2000	O/S Flange
P8	12.0000	O/S Flange
P9	13.8000	O/S Flange
P10	14.9000	O/S Flange



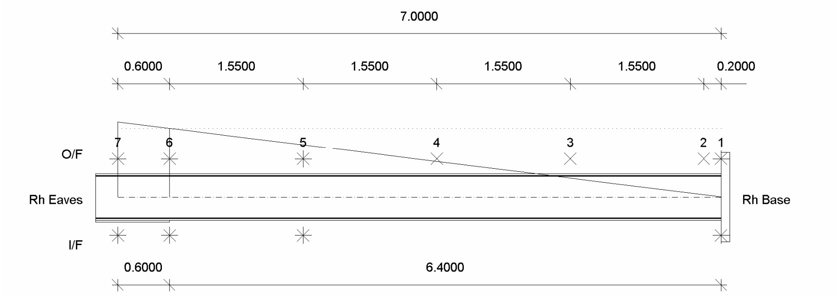
D.4.5.3 Span 1 Rh Rafter

No.	Position	Type
	m	
P1	0.2900	Torsional
P2	1.2000	O/S Flange
P3	3.0000	Torsional
P4	4.8000	O/S Flange
P5	6.6000	O/S Flange
P6	8.4000	O/S Flange
P7	10.2000	O/S Flange
P8	12.0000	O/S Flange
P9	13.8000	O/S Flange
P10	14.9000	O/S Flange



D.4.5.4 Span 1 Rh Column

No.	Position	Type
	m	
Base	0.0000	Torsional
S2	0.2000	O/S Flange
S3	1.7500	O/S Flange
S4	3.3000	O/S Flange
S5	4.8500	Torsional
S6	6.4000	Torsional
S7	7.0000	Torsional



D.5 Frame loadcases

D.5.1 Frame Self Weight Dead

D.5.2 Frame Dead Load Dead

Area Loads

Span\Member	Direction	Value End(1)	Offset End(1)	Value End(2)	Offset End(2)	Centres
		kN/m^2	m	kN/m^2	m	m
Span 1	Vertical	0.270				6.0000

D.5.3 Frame Service Load Dead

Area Loads

Span\Member	Direction	Value End(1)	Offset End(1)	Value End(2)	Offset End(2)	Centres
		kN/m^2	m	kN/m^2	m	m
Span 1	Vertical	0.280				6.0000

D.5.4 Frame Imposed Load Imposed

Area Loads

Span\Member	Direction	Value End(1)	Offset End(1)	Value End(2)	Offset End(2)	Centres
		kN/m^2	m	kN/m^2	m	m
Span 1	Vertical	0.600				6.0000

D.5.5 Frame Design Combinations

D.5.5.1 Sw + Dead + Serv. + Imp. + NHF

Frame Imperfection on, left to right

Loadcase	Type	Ultimate	Service
		P.S.F.	P.S.F.
Frame Self Weight	Dead	1.40	1.00
Frame Dead Load	Dead	1.40	1.00
Frame Service Load	Dead	1.40	1.00
Frame Imposed Load	Imposed	1.60	1.00

D.6 Frame Design

D.6.1 Summary

Design Combination	Lambda p	Lambda r	Status
Sw+Dead+Serv.+Imp.+NHF	1.0301	1.0000	Pass

Sw+Dead+Serv.+Imp.+NHF

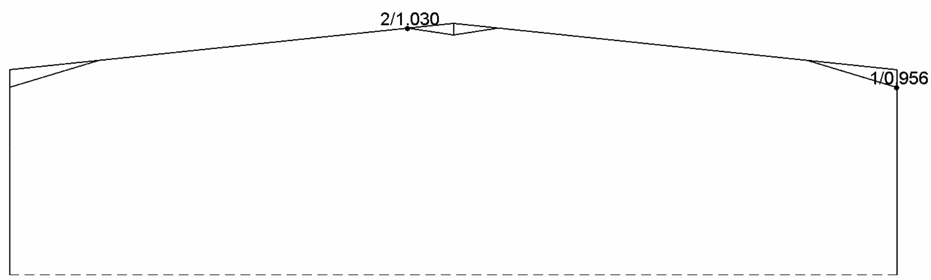
Design Combination	Status
Hinge History	Pass
Strength	Pass
Serviceability	Not Checked
Frame Stability - Notional Load Check	Pass
Member Stability	Pass

D.6.2 Sections

D.6.2.1 Sw + Dead + Serv. + Imp. + NHF

Member	Section	Grade	Weight	Mpr	UR
Span 1 Lh Column	UB 533x210x101	S275	707.0401	687.7861	0.9842
Span 1 Lh Haunch	UB 457x191x67	S275	87.6483	n/a	n/a
Span 1 Lh Rafter	UB 457x191x67	S275	1012.4242	401.7641	0.9048
Span 1 Apex Lh Haunch	UB 457x191x67	S275	35.7828	n/a	n/a
Span 1 Apex Rh Haunch	UB 457x191x67	S275	35.7828	n/a	n/a
Span 1 Rh Rafter	UB 457x191x67	S275	1012.3990	401.7641	0.9018
Span 1 Rh Haunch	UB 457x191x67	S275	87.6483	n/a	n/a
Span 1 Rh Column	UB 533x210x101	S275	707.0401	687.7861	1.0000
Total Frame Weight			3685.7655		

D.6.3 Hinge History Diagram



D.6.4 Hinges

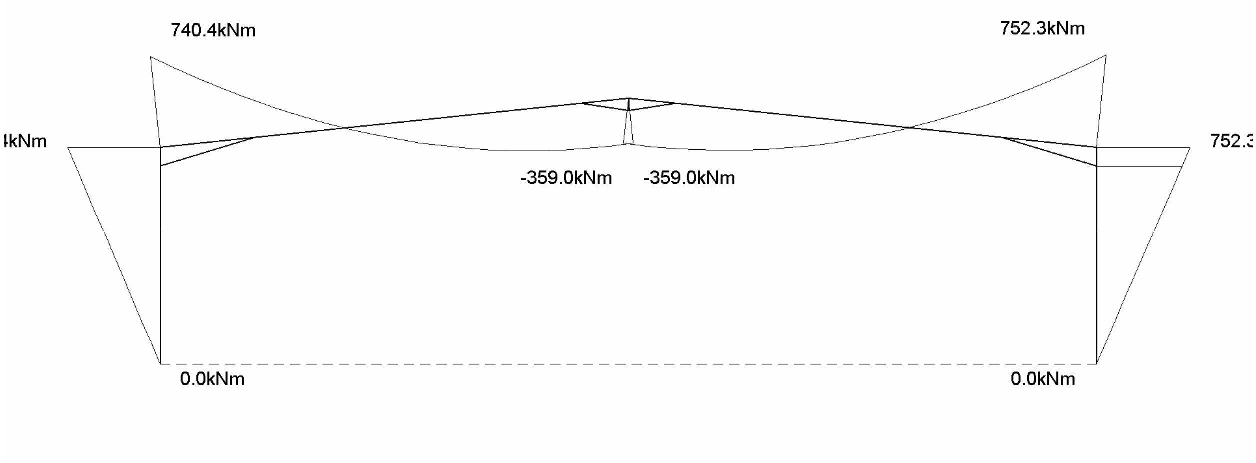
D.6.4.1 Sw + Dead + Serv. + Imp. + NHF

D.6.4.2 Hinge History

Event	Lambda	Status	Distance	Member
1	0.9560	Formed	6.4000	Span 1 Rh Column
2	1.0301	Formed	13.5510	Span 1 Lh Rafter

D.6.5 Bending Moment Diagram

D.6.5.1 Sw + Dead + Serv. + Imp. + NHF



D.6.6 Strength

D.6.6.1 Sw + Dead + Serv. + Imp. + NHF

D.6.6.2 ULS Member Checks

Member	Status	Class	Shear	Moment	Axial	Axial\Moment
Span 1 Lh Column	Pass	Class 1	0.1157	0.9779	0.0525	0.9815
Span 1 Lh Haunch	Pass	Class 2	0.1696	0.7909	0.0380	0.8228
Span 1 Lh Rafter	Pass	Class 1	0.1947	0.8986	0.0511	0.9014
Span 1 Apex Lh Haunch	Pass	Class 2	0.0131	0.7347	0.0351	0.7698
Span 1 Apex Rh Haunch	Pass	Class 2	0.0122	0.7327	0.0351	0.7679
Span 1 Rh Rafter	Pass	Class 1	0.1959	0.8956	0.0512	0.8984
Span 1 Rh Haunch	Pass	Class 2	0.1706	0.8042	0.0380	0.8361
Span 1 Rh Column	Pass	Class 1	0.1166	0.9936	0.0528	0.9973

Span 1 Lh Column

Item	**Value**	**Units**	**Clause of BS 5950**
Section	UB 533x210x101		
Grade	S275		
Design strength, py	265.0	N/mm^2	Cl. 3.1.1 Table 9
Provided capacity, Mp	692.2	kNm	
Reduced capacity, Mpr	687.8	kNm	
Pass			
Section Class	Class 1		Table 11
Pass			
Web Shear Buckling Check:			
Pass			
Shear Capacity Check :	0.0000	m	
Shear, F_v	-106.618	kN	
Shear capacity, P_v	921.621	kN	Cl 4.2.3
Ratio	0.1157		
Pass			
Moment Capacity Check :	6.4000	m	
Moment, M_{xx}	676.9	kNm	
Moment capacity, M_{cx}	692.2	kNm	Cl 4.2.5
Ratio	0.9779		
Pass			
Axial Capacity Check :	0.0000	m	
Axial force, F_c	179.105	kN	
Axial capacity, $A_g p_y$	3409.747	kN	
Ratio	0.0525		Cl 4.8.3.2
Pass			
Axial Moment Capacity Check :	6.4000	m	
Applied moment, M_{xx}	676.9	kNm	
Reduced capacity, M_{rx}	689.7	kNm	
Ratio	0.9815		Cl 4.8.2.3
Pass			

Span 1 Lh Haunch

Check	Class	Shear	Moment	Axial	Axial+Moment
Distance along CL 0.290	Class 2	0.1309	0.7909	0.0319	0.8228
Distance along CL 0.598	Class 2	0.1348	0.7809	0.0327	0.8135
Distance along CL 1.197	Class 2	0.1437	0.7802	0.0343	0.8144
Distance along CL 1.795	Class 2	0.1549	0.7661	0.0360	0.8021
Distance along CL 2.394	Class 2	0.1696	0.7326	0.0380	0.7706

Distance along CL 0.290

Item	Value	Units	Clause of BS 5950
Section	UB 457x191x67		
Grade	S275		
Design strength, py	275.0	N/mm^2	
Pass			
Section Class	Class 2		Table 11
Pass			
Web Shear Buckling Check:			
Pass			
Shear Capacity Check :	0.2899	m	
Shear, F_v	154.074	kN	
Shear capacity, P_v	1177.447	kN	Cl 4.2.3
Ratio	0.1309		
Pass			
Moment Capacity Check :	0.2899	m	
Moment, M_{xx}	695.2	kNm	
Moment capacity, M_{cx}	879.0	kNm	Cl 4.2.5
Ratio	0.7909		
Pass			
Cross- section < p_y Z			
Axial Capacity Check :	0.2899	m	
Axial force, F_c	123.402	kN	
Axial capacity, $A_g p_y$	3863.030	kN	
Ratio	0.0319		Cl 4.8.3.2
Pass			
Axial Moment Capacity Check :	0.2899	m	
Axial Force, F_c	123.402	kN	
$A_g p_y$	3863.030	kN	
Moment, M_{xx}	695.2	kNm	
Moment capacity, M_{cx}	879.0	kNm	
Ratio	0.8228		Cl 4.8.3.2
Pass			

Ref.	Calculations	Output

Distance along CL 0.598

Item	Value	Units	Clause of BS 5950
Section	UB 457x191x67		
Grade	S275		
Design strength, py	275.0	N/mm^2	
Pass			
Section Class	Class 2		Table 11
Pass			
Web Shear Buckling Check:			
Pass			
Shear Capacity Check :	0.5980	m	
Shear, F_v	150.625	kN	
Shear capacity, P_v	1117.757	kN	Cl 4.2.3
Ratio	0.1348		
Pass			
Moment Capacity Check :	0.5980	m	
Moment, M_{xx}	648.3	kNm	
Moment capacity, M_{cx}	830.2	kNm	Cl 4.2.5
Ratio	0.7809		
Pass			
Cross-section < p_y Z			
Axial Capacity Check :	0.5980	m	
Axial force, F_c	123.039	kN	
Axial capacity, $A_g p_y$	3763.547	kN	
Ratio	0.0327		Cl 4.8.3.2
Pass			
Axial Moment Capacity Check :	0.5980	m	
Axial Force, F_c	123.039	kN	
$A_g p_y$	3763.547	kN	
Moment, M_{xx}	648.3	kNm	
Moment capacity, M_{cx}	830.2	kNm	
Ratio	0.8135		Cl 4.8.3.2
Pass			

Ref.	Calculations	Output

Distance along CL 1.197

Item	Value	Units	Clause of BS 5950
Section	UB 457x191x67		
Grade	S275		
Design strength, py	275.0	N/mm^2	
Pass			
Section Class	Class 2		Table 11
Pass			
Web Shear Buckling Check:			
Pass			
Shear Capacity Check :	1.1970	m	
Shear, F_v	143.919	kN	
Shear capacity, P_v	1001.696	kN	Cl 4.2.3
Ratio	0.1437		
Pass			
Moment Capacity Check :	1.1970	m	
Moment, M_{xx}	560.1	kNm	
Moment capacity, M_{cx}	717.9	kNm	Cl 4.2.5
Ratio	0.7802		
Pass			
Cross-section < p_y Z			
Axial Capacity Check :	1.1970	m	
Axial force, F_c	122.335	kN	
Axial capacity, $A_g p_y$	3570.112	kN	
Ratio	0.0343		Cl 4.8.3.2
Pass			
Axial Moment Capacity Check :	1.1970	m	
Axial Force, F_c	122.335	kN	
$A_g p_y$	3570.112	kN	
Moment, M_{xx}	560.1	kNm	
Moment capacity, M_{cx}	717.9	kNm	
Ratio	0.8144		Cl 4.8.3.2
Pass			

Distance along CL 1.795

Item	Value	Units	Clause of BS 5950
Section	UB 457x191x67		
Grade	S275		
Design strength, py	275.0	N/mm^2	
Pass			
Section Class	Class 2		Table 11
Pass			
Web Shear Buckling Check:			
Pass			
Shear Capacity Check :	1.7950	m	
Shear, F_v	137.224	kN	
Shear capacity, P_v	885.828	kN	Cl 4.2.3
Ratio	0.1549		
Pass			
Moment Capacity Check :	1.7950	m	
Moment, M_{xx}	476.0	kNm	
Moment capacity, M_{cx}	621.4	kNm	Cl 4.2.5
Ratio	0.7661		
Pass			
Cross- section < p_y Z			
Axial Capacity Check :	1.7950	m	
Axial force, F_c	121.632	kN	
Axial capacity, $A_g p_y$	3376.999	kN	
Ratio	0.0360		Cl 4.8.3.2
Pass			
Axial Moment Capacity Check :	1.7950	m	
Axial Force, F_c	121.632	kN	
$A_g p_y$	3376.999	kN	
Moment, M_{xx}	476.0	kNm	
Moment capacity, M_{cx}	621.4	kNm	
Ratio	0.8021		Cl 4.8.3.2
Pass			

Distance along CL 2.394

Item	Value	Units	Clause of BS 5950
Section	UB 457x191x67		
Grade	S275		
Design strength, py	275.0	N/mm^2	
Pass			
Section Class	Class 2		Table 11
Pass			
Web Shear Buckling Check:			
Pass			
Shear Capacity Check :	2.3940	m	
Shear, F_v	130.518	kN	
Shear capacity, P_v	769.767	kN	Cl 4.2.3
Ratio	0.1696		
Pass			
Moment Capacity Check :	2.3940	m	
Moment, M_{xx}	395.8	kNm	
Moment capacity, M_{cx}	540.3	kNm	Cl 4.2.5
Ratio	0.7326		
Pass			
Cross-section < p_y Z			
Axial Capacity Check :	2.3940	m	
Axial force, F_c	120.928	kN	
Axial capacity, $A_g p_y$	3183.563	kN	
Ratio	0.0380		Cl 4.8.3.2
Pass			
Axial Moment Capacity Check :	2.3940	m	
Axial Force, F_c	120.928	kN	
$A_g p_y$	3183.563	kN	
Moment, M_{xx}	395.8	kNm	
Moment capacity, M_{cx}	540.3	kNm	
Ratio	0.7706		Cl 4.8.3.2
Pass			

Ref.	Calculations	Output

Span 1 Lh Rafter

Item	Value	Units	Clause of BS 5950
Section	UB 457x191x67		
Grade	S275		
Design strength, py	275.0	N/mm^2	Cl. 3.1.1 Table 9
Provided capacity, Mp	404.5	kNm	
Reduced capacity, Mpr	401.8	kNm	
Pass			
Section Class	Class 1		Table 11
Pass			
Web Shear Buckling Check:			
Pass			
Shear Capacity Check :	2.9930	m	
Shear, F_v	123.811	kN	
Shear capacity, P_v	635.893	kN	Cl 4.2.3
Ratio	0.1947		
Pass			
Moment Capacity Check :	13.5509	m	
Moment, M_{xx}	-363.5	kNm	
Moment capacity, M_{cx}	404.5	kNm	Cl 4.2.5
Ratio	0.8986		
Pass			
Axial Capacity Check :	2.9930	m	
Axial force, F_c	120.223	kN	
Axial capacity, $A_g p_y$	2351.461	kN	
Ratio	0.0511		Cl 4.8.3.2
Pass			
Axial Moment Capacity Check :	13.5509	m	
Applied moment, M_{xx}	-363.5	kNm	
Reduced capacity, M_{rx}	403.3	kNm	
Ratio	0.9014		Cl 4.8.2.3
Pass			

Ref.	Calculations	Output

Span 1 Apex Lh Haunch

Check	Class	Shear	Moment	Axial	Axial+Moment
Distance along CL 13.857	Class 2	0.0031	0.7347	0.0351	0.7698
Distance along CL 14.052	Class 2	0.0000	0.7112	0.0345	0.7457
Distance along CL 14.163	Class 2	0.0017	0.6978	0.0341	0.7320
Distance along CL 14.470	Class 2	0.0059	0.6600	0.0332	0.6933
Distance along CL 14.776	Class 2	0.0097	0.6217	0.0323	0.6540
Distance along CL 15.082	Class 2	0.0131	0.5831	0.0315	0.6146

Yeadon House, New Street
Pudsey, W Yorks, LS28 8AQ
Tel: 0113 2393000 Fax: 0113 239 0546

Ref.	Calculations	Output

D.6.7 Frame Stability

D.6.7.1 Sw + Dead + Serv. + Imp. + NHF

D.6.7.2 Notional Sway

Span	Status
Span 1 Lh Eaves	Pass
Span 1 Rh Eaves	Pass

Span 1 Lh Eaves

Item	Value	Units	Clause of BS 5950
Total horizontal load	0.849	kN	
Horizontal deflection, δ	3.139	mm	
Cladding stiffness	0.0	%	
Horizontal deflection, δ_{clad}	3.139	mm	
Deflection limit	7.000	mm	Cl. 5.5.4.2.3
Pass			

Total horizontal load

Item	Value	Units	Clause of BS 5950
Member	Span 1 Lh Column		
Height, h_i	7.0000	m	
Horizontal Load	0.849	kN	
Total horizontal load	0.849	kN	

Span 1 Rh Eaves

Item	Value	Units	Clause of BS 5950
Total horizontal load	0.849	kN	
Horizontal deflection, δ	3.139	mm	
Cladding stiffness	0.0	%	
Horizontal deflection, δ_{clad}	3.139	mm	
Deflection limit	7.000	mm	Cl. 5.5.4.2.3
Pass			

Total horizontal load

Item	Value	Units	Clause of BS 5950
Member	Span 1 Rh Column		
Height, h_i	7.0000	m	
Horizontal Load	0.849	kN	
Total horizontal load	0.849	kN	

D.6.8 Member Stability

D.6.8.1 Sw + Dead + Serv. + Imp. + NHF

Member	Status	
Span 1 Lh Column	Pass	
Span 1 Lh Rafter	Pass	
Span 1 Rh Rafter	Pass	
Span 1 Rh Column	Pass	

Span 1 Lh Column

Check	Restraint End 1	Restraint End 2	Distance End 1	Distance End 2	Status
Clause 4.8.3.3.2	Base	S5	0.0000	4.8500	Pass
Clause 5.3.3	S5	S6	4.8500	6.4000	Pass

Clause 4.8.3.3.2

Item	Value	Units	Clause of BS 5950
Restraint distance (Base)	0.0000	m	
Restraint distance (S5)	4.8500	m	
Length	4.8500	m	
Length	Uniform		
Load type	Normal		
Axial Compression, F_c	179.105	kN	
Axial Capacity, P_{cy}	1645.834	kN	Cl 4.7.4
Equivalent uniform moment factor, m_{LT}	0.6000		Cl 4.8.3.3.4
Maximum moment, M_{LT}	513.0	kNm	
Buckling Resistance Moment, M_b	401.0	kNm	Cl 4.3.6.4
Cl. 4.8.3.3.2	0.8763		
Pass			

Clause 5.3.3

Item	Value	Units	Clause of BS 5950
Restraint distance (S5)	4.8500	m	
Restraint distance (S6)	6.4000	m	
Length	1.5500	m	
Length	Uniform		
Load type	Normal		
Radius gyration, r_y	45.7	mm	
Compressive stress, f_c	13.4	N/mm^2	
Torsional index, x	33.1828		B.2.3
Design strength, p_y	265.0	N/mm^2	Table 9
Length, L_u	1.8403	m	5.3.3.a
ϕ	1.0000		5.3.3.b
Length L_m	1.8403	m	5.3.3.b
Pass			

Span 1 Lh Rafter

Check	Restraint End 1	Restraint End 2	Distance End 1	Distance End 2	Status
Clause 4.8.3.3.1	P1	P3	0.2900	3.0000	Pass
Clause 4.8.3.3.2	P3	P5	3.0000	6.6000	Pass
Clause 4.8.3.3.2	P5	P6	6.6000	8.4000	Pass
Clause 4.8.3.3.2	P6	P7	8.4000	10.2000	Pass
Clause 4.8.3.3.2	P7	P8	10.2000	12.0000	Pass
Clause 4.8.3.3.2	P8	P9	12.0000	13.8000	Pass
Clause 4.8.3.3.2	P9	P10	13.8000	14.9000	Pass

Clause 4.8.3.3.1

Item	Value	Units	Clause of BS 5950
Restraint distance (P1)	0.2900	m	
Restraint distance (P3)	3.0000	m	
Length	2.7100	m	
Length	Tapered		
Load type	Normal		
Cl. 4.8.3.3.1 (2)	0.9653		
Pass			

Cl. 4.8.3.3.1 (2)

Item	Value	Units	Clause of BS 5950
Axial Compression, F_c	123.402	kN	
Axial Capacity, P_c	1796.648	kN	Cl 4.7.4
Equivalent uniform moment factor, m_{LT}	1.0000		Cl 4.8.3.3.4
Maximum moment, M_{LT}	695.2	kNm	
Buckling resistance moment M_b	775.4	kNm	Cl 4.3.6.4
Cl. 4.8.3.3.1 (2)	0.9653		

Ref.	Calculations	Output

Clause 4.8.3.3.2

Item	Value	Units	Clause of BS 5950
Restraint distance (P3)	3.0000	m	
Restraint distance (P5)	6.6000	m	
Length	3.6000	m	
Length	Uniform		
Load type	Normal		
Axial Compression, F_c	120.215	kN	
Axial Capacity, P_{cy}	1419.138	kN	Cl 4.7.4
Equivalent uniform moment factor, m_{LT}	0.4909		Cl 4.8.3.3.4
Maximum moment, M_{LT}	318.8	kNm	
Buckling Resistance Moment, M_b	270.6	kNm	Cl 4.3.6.4
Cl. 4.8.3.3.2	0.6631		
Pass			

Clause 4.8.3.3.2

Item	Value	Units	Clause of BS 5950
Restraint distance (P5)	6.6000	m	
Restraint distance (P6)	8.4000	m	
Length	1.8000	m	
Length	Uniform		
Load type	Normal		
Axial Compression, F_c	115.982	kN	
Axial Capacity, P_{cy}	2098.062	kN	Cl 4.7.4
Equivalent uniform moment factor, m_{LT}	0.7339		Cl 4.8.3.3.4
Maximum moment, M_{LT}	-186.1	kNm	
Buckling Resistance Moment, M_b	393.7	kNm	Cl 4.3.6.4
Cl. 4.8.3.3.2	0.4022		
Pass			

Clause 4.8.3.3.2

Item	Value	Units	Clause of BS 5950
Restraint distance (P6)	8.4000	m	
Restraint distance (P7)	10.2000	m	
Length	1.8000	m	
Length	Uniform		
Load type	Normal		
Axial Compression, F_c	113.865	kN	
Axial Capacity, P_{cy}	2098.062	kN	Cl 4.7.4
Equivalent uniform moment factor, m_{LT}	0.8758		Cl 4.8.3.3.4
Maximum moment, M_{LT}	-281.9	kNm	
Buckling Resistance Moment, M_b	393.7	kNm	Cl 4.3.6.4
Cl. 4.8.3.3.2	0.6813		
Pass			

Clause 4.8.3.3.2

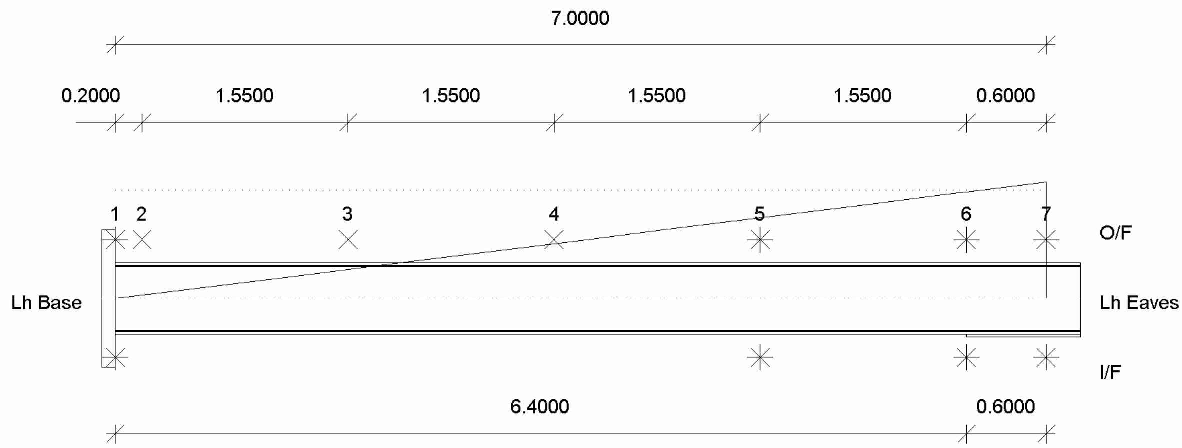
Item	Value	Units	Clause of BS 5950
Restraint distance (P7)	10.2000	m	
Restraint distance (P8)	12.0000	m	
Length	1.8000	m	
Length	Uniform		
Load type	Normal		
Axial Compression, F_c	111.749	kN	
Axial Capacity, P_{cy}	2098.062	kN	Cl 4.7.4
Equivalent uniform moment factor, m_{LT}	0.9399		Cl 4.8.3.3.4
Maximum moment, M_{LT}	-341.4	kNm	
Buckling Resistance Moment, M_b	393.7	kNm	Cl 4.3.6.4
Cl. 4.8.3.3.2	0.8682		
Pass			

Clause 4.8.3.3.2

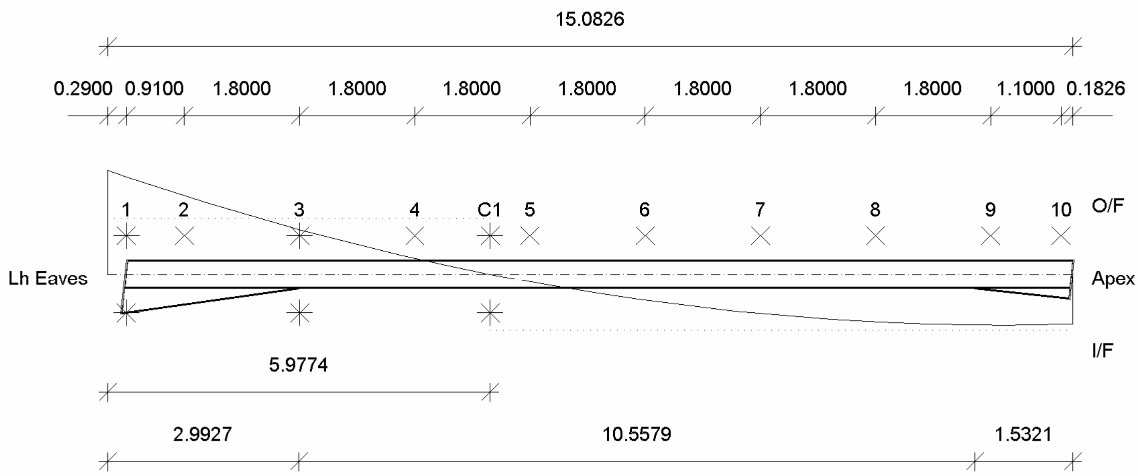
Item	Value	Units	Clause of BS 5950
Restraint distance (P8)	12.0000	m	
Restraint distance (P9)	13.8000	m	
Length	1.8000	m	
Length	Uniform		
Load type	Normal		
Axial Compression, F_c	109.632	kN	
Axial Capacity, P_{cy}	2098.062	kN	Cl 4.7.4
Equivalent uniform moment factor, m_{LT}	0.9836		Cl 4.8.3.3.4
Maximum moment, M_{LT}	-364.6	kNm	
Buckling Resistance Moment, M_b	393.7	kNm	Cl 4.3.6.4
Cl. 4.8.3.3.2	0.9630		
Pass			

Clause 4.8.3.3.2

Item	Value	Units	Clause of BS 5950
Restraint distance (P9)	13.8000	m	
Restraint distance (P10)	14.9000	m	
Length	1.1000	m	
Length	Uniform		
Load type	Normal		
Axial Compression, F_c	107.516	kN	
Axial Capacity, P_{cy}	2267.953	kN	Cl 4.7.4
Equivalent uniform moment factor, m_{LT}	0.9986		Cl 4.8.3.3.4
Maximum moment, M_{LT}	-364.9	kNm	
Buckling Resistance Moment, M_b	404.5	kNm	Cl 4.3.6.4
Cl. 4.8.3.3.2	0.9482		
Pass			



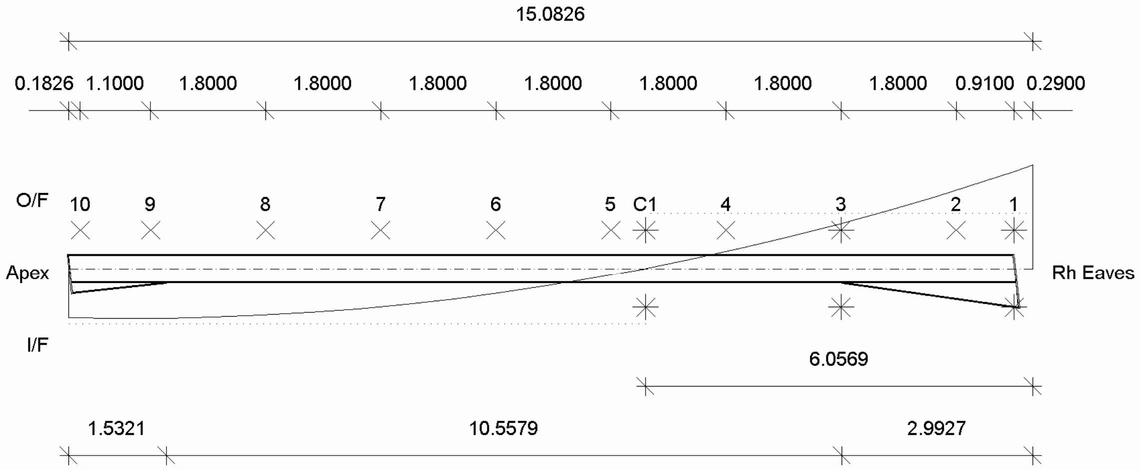
Member Stability Diagram Span 1 Lh Column



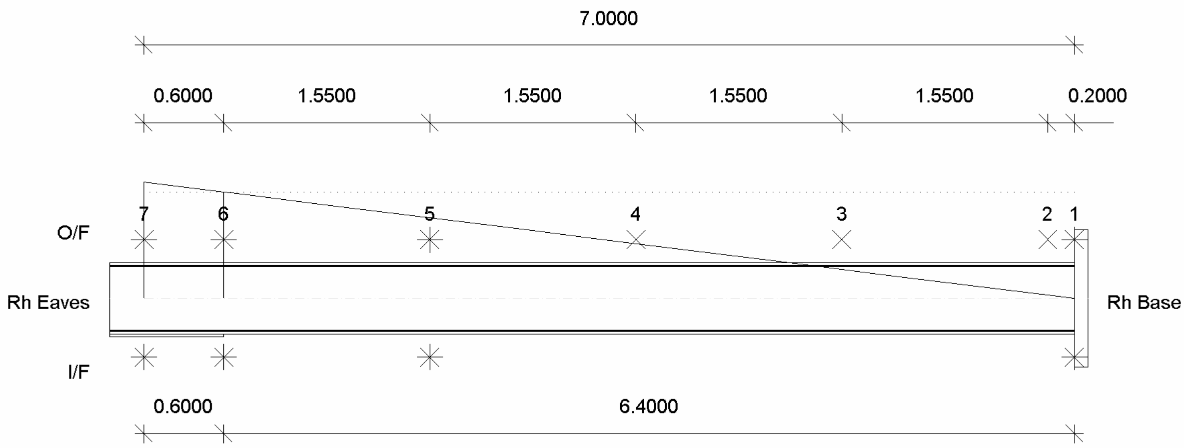
Member Stability Diagram Span 1 Lh Rafter (C1 = point of contraflexure)

CSC (UK) Ltd Yeadon House, New Street Pudsey, W Yorks, LS28 8AQ Tel: 0113 2393000 Fax: 0113 239 0546	**Project** Design of single-span steel portal frames				**Job Ref.** BCB 766	
	Part of Structure Computer output				**Sheet no./rev.** 21 of 22	
	Calc. by AJR	**Date** Feb. 2004	**Chck'd by** ASM	**Date** Feb. 2004	**App'd by** DGB	**Date** March 2004

Ref.	Calculations	Output



Member Stability Diagram Span 1 Rh Rafter (C1 = point of contraflexure)



Member Stability Diagram Span 1 Rh Column

D.6.9 Base Loads

D.6.9.1 Factored plastic distribution

D.6.9.2 Sw + Dead + Serv. + Imp. + NHF

Base	Mx (kNm)	Fv (kN)	F (kN)
Span 1 Lh Base	0.0000	106.6176	179.1046
Span 1 Rh Base	0.0000	-106.6176	179.8970

D.6.10 Connection Forces

D.6.10.1 Sw + Dead + Serv. + Imp. + NHF

Connection	Face	Mx (kNm)	Fv (kN)	F (kN)	Lh Mse (kNm)	Rh Mse (kNm)
Span 1 Lh Eaves	Rh	695.2324	166.1294	106.6204	319.6613	
Span 1 Apex	n/a	-358.9797	0.3967	106.6333	-363.5420	-362.3347
Span 1 Rh Eaves	Lh	706.8897	166.9218	106.6205	329.1885	

D.6.11 Foundation Loads

Unfactored elastic distribution

D.6.11.1 Frame Self Weight

Base	Mx (kNm)	Fv (kN)	F (kN)
Span 1 Lh Base	0.0000	6.5257	16.8676
Span 1 Rh Base	0.0000	-6.5257	16.8676

D.6.11.2 Frame Dead Load

Base	Mx (kNm)	Fv (kN)	F (kN)
Span 1 Lh Base	0.0000	16.0546	24.4336
Span 1 Rh Base	0.0000	-16.0546	24.4336

D.6.11.3 Frame Service Load

Base	Mx (kNm)	Fv (kN)	F (kN)
Span 1 Lh Base	0.0000	16.5580	25.1998
Span 1 Rh Base	0.0000	-16.5580	25.1998

D.6.11.4 Frame Imposed Load

Base	Mx (kNm)	Fv (kN)	F (kN)
Span 1 Lh Base	0.0000	35.4815	53.9996
Span 1 Rh Base	0.0000	-35.4815	53.9996